EUROPEAN STANDARD

NORME EUROPÉENNE

EUROPÄISCHE NORM

FINAL DRAFT
prEN 1998-1

December 2003

ICS 91.120.20

Will supersede ENV 1998-1-1:1994; ENV 1998-1-2:1994 and ENV 1998-1-3:1995

English version

Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings

Eurocode 8: Calcul des structures pour leur résistance aux séismes - Partie 1: Règles générales, actions sismiques et règles pour les bâtiments

Eurocode 8: Auslegung von Bauwerken gegen Erdbeben - Teil 1: Grundlagen, Erdbebeneinwirkungen und Regeln für Hochbauten

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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Ref. No. prEN 1998-1:2003 E

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Foreword

This document (EN 1990:2002) 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of an identical text or by endorsement, at the latest by MM-200Y, and conflicting national standards shall be withdrawn at the latest by MM-20YY.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 1998-1-1:1994, ENV 1998-1-2:1994 and ENV 1998-1-3:1995.

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 a 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

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

specifically covered and additional expert consideration will be required by the designer in such cases.

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 (informative).

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 1998-1

The scope of EN 1998 is defined in **1.1.1** and the scope of this Part of EN 1998 is defined in **1.1.2**. Additional Parts of EN 1998 are listed in **1.1.3**.

EN 1998-1 was developed from the merger of ENV 1998-1-1:1994, ENV 1998-1-2:1994 and ENV 1998-1-3:1995. As mentioned in **1.1.1**, attention must be paid to the fact that for the design of structures in seismic regions the provisions of EN 1998 are to be applied in addition to the provisions of the other relevant EN 1990 to EN 1997 and EN 1999.

One fundamental issue in EN 1998-1 is the definition of the seismic action. Given the wide difference of seismic hazard and seismo-genetic characteristics in the various

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

member countries, the seismic action is herein defined in general terms. The definition allows various Nationally Determined Parameters (NDP) which should be confirmed or modified in the National Annexes.

It is however considered that, by the use of a common basic model for the representation of the seismic action, an important step is taken in EN 1998-1 in terms of Code harmonisation.

EN 1998-1 contains in its section related to masonry buildings specific provisions which simplify the design of "simple masonry buildings".

National annex for EN 1998-1

This standard gives alternative procedures, values and recommendations for classes with notes indicating where national choices may be made. Therefore the National Standard implementing EN 1998-1 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 1998-1:2004 through clauses:

Reference	Item
1.1.2(7)	Informative Annexes A and B.
2.1(1)P	Reference return period T_{NCR} of seismic action for the no-collapse requirement (or, equivalently, reference probability of exceedance in 50 years, P_{NCR}).
2.1(1)P	Reference return period T_{DLR} of seismic action for the damage limitation requirement. (or, equivalently, reference probability of exceedance in 10 years, P_{DLR}).
3.1.1(4)	Conditions under which ground investigations additional to those necessary for design for non-seismic actions may be omitted and default ground classification may be used.
3.1.2(1)	Ground classification scheme accounting for deep geology, including values of parameters S, T_B, T_C and T_D defining horizontal and vertical elastic response spectra in accordance with **3.2.2.2** and **3.2.2.3**.
3.2.1(1), (2),(3)	Seismic zone maps and reference ground accelerations therein.
3.2.1(4)	Governing parameter (identification and value) for threshold of low seismicity .
3.2.1(5)	Governing parameter (identification and value) for threshold of very low seismicity .
3.2.2.1(4), 3.2.2.2(1)P	Parameters S, T_B, T_C, T_D defining shape of horizontal elastic response spectra.
3.2.2.3(1)P	Parameters a_{vg} T_B, T_C, T_D defining shape of vertical elastic response spectra.

3.2.2.5(4)P	Lower bound factor β on design spectral values.
4.2.3.2(8)	Reference to definitions of centre of stiffness and of torsional radius in multi-storey buildings meeting or not conditions (a) and (b) of **4.2.3.2(8)**
4.2.4(2)P	Values of φ for buildings.
4.2.5(5)P	Importance factor γ_I for buildings.
4.3.3.1 (4)	Decision on whether nonlinear methods of analysis may be applied for the design of non-base-isolated buildings. Reference to information on member deformation capacities and the associated partial factors for the Ultimate Limit State for design or evaluation on the basis of nonlinear analysis methods.
4.3.3.1 (8)	Threshold value of importance factor, γ_I, relating to the permitted use of analysis with two planar models.
4.4.2.5 (2).	Overstrength factor γ_{Rd} for diaphragms.
4.4.3.2 (2)	Reduction factor ν for displacements at damage limitation limit state
5.2.1(5)	Geographical limitations on use of ductility classes for concrete buildings.
5.2.2.2(10)	q_o-value for concrete buildings subjected to special Quality System Plan.
5.2.4(1), (3)	Material partial factors for concrete buildings in the seismic design situation.
5.4.3.5.2(1)	Minimum web reinforcement of large lightly reinforced concrete walls
5.8.2(3)	Minimum cross-sectional dimensions of concrete foundation beams.
5.8.2(4)	Minimum thickness and reinforcement ratio of concrete foundation slabs.
5.8.2(5)	Minimum reinforcement ratio of concrete foundation beams.
5.11.1.3.2(3)	Ductility class of precast wall panel systems.
5.11.1.4	q-factors of precast systems.
5.11.1.5(2)	Seismic action during erection of precast structures.
5.11.3.4(7)e	Minimum longitudinal steel in grouted connections of large panel walls.
6.1.2(1)	Upper limit of q for low-dissipative structural behaviour concept; limitations on structural behaviour concept; geographical limitations on use of ductility classes for steel buildings.
6.1.3(1)	Material partial factors for steel buildings in the seismic design situation.

6.2(3)	Overstrength factor for capacity design of steel buildings.
6.2 (7)	Information as to how EN 1993-1-10:2004 may be used in the seismic design situation
6.5.5(7)	Reference to complementary rules on acceptable connection design
6.7.4(2)	Residual post-buckling resistance of compression diagonals in steel frames with *V*-bracings.
7.1.2(1)	Upper limit of q for low-dissipative structural behaviour concept; limitations on structural behaviour concept; geographical limitations on use of ductility classes for composite steel-concrete buildings.
7.1.3(1), (3)	Material partial factors for composite steel-concrete buildings in the seismic design situation.
7.1.3(4)	Overstrength factor for capacity design of composite steel-concrete buildings
7.7.2(4)	Stiffness reduction factor for concrete part of a composite steel-concrete column section
8.3(1)	Ductility class for timber buildings.
9.2.1(1)	Type of masonry units with sufficient robustness.
9.2.2(1)	Minimum strength of masonry units.
9.2.3(1)	Minimum strength of mortar in masonry buildings.
9.2.4(1)	Alternative classes for perpend joints in masonry
9.3(2)	Conditions for use of unreinforced masonry satisfying provisions of EN 1996 alone.
9.3(2)	Minimum effective thickness of unreinforced masonry walls satisfying provisions of EN 1996 alone.
9.3(3)	Maximum value of ground acceleration for the use of unreinforced masonry satisfying provisions of EN. 1998-1
9.3(4), Table 9.1	q-factor values in masonry buildings.
9.3(4), Table 9.1	q-factors for buildings with masonry systems which provide enhanced ductility.
9.5.1(5)	Geometric requirements for masonry shear walls.
9.6(3)	Material partial factors in masonry buildings in the seismic design situation.
9.7.2(1)	Maximum number of storeys and minimum area of shear walls of “simple masonry building”.
9.7.2(2)b	Minimum aspect ratio in plan of “simple masonry buildings”.
9.7.2(2)c	Maximum floor area of recesses in plan for “simple masonry buildings”.
9.7.2(5)	Maximum difference in mass and wall area between adjacent

	storeys of “simple masonry buildings”.
10.3(2)P	Magnification factor on seismic displacements for isolation devices.

1 GENERAL

1.1 Scope

1.1.1 Scope of EN 1998

(1)P EN 1998 applies to the design and construction of buildings and civil engineering works in seismic regions. Its purpose is to ensure that in the event of earthquakes:

- human lives are protected;
- damage is limited; and
- structures important for civil protection remain operational.

NOTE The random nature of the seismic events and the limited resources available to counter their effects are such as to make the attainment of these goals only partially possible and only measurable in probabilistic terms. The extent of the protection that can be provided to different categories of buildings, which is only measurable in probabilistic terms, is a matter of optimal allocation of resources and is therefore expected to vary from country to country, depending on the relative importance of the seismic risk with respect to risks of other origin and on the global economic resources.

(2)P Special structures, such as nuclear power plants, offshore structures and large dams, are beyond the scope of EN 1998.

(3)P EN 1998 contains only those provisions that, in addition to the provisions of the other relevant Eurocodes, must be observed for the design of structures in seismic regions. It complements in this respect the other Eurocodes.

(4) EN 1998 is subdivided into various separate Parts (see **1.1.2** and **1.1.3**).

1.1.2 Scope of EN 1998-1

(1) EN 1998-1 applies to the design of buildings and civil engineering works in seismic regions. It is subdivided in 10 Sections, some of which are specifically devoted to the design of buildings.

(2) Section **2** of EN 1998-1 contains the basic performance requirements and compliance criteria applicable to buildings and civil engineering works in seismic regions.

(3) Section **3** of EN 1998-1 gives the rules for the representation of seismic actions and for their combination with other actions. Certain types of structures, dealt with in EN 1998-2 to EN 1998-6, need complementing rules which are given in those Parts.

(4) Section **4** of EN 1998-1 contains general design rules relevant specifically to buildings.

(5) Sections **5** to **9** of EN 1998-1 contain specific rules for various structural materials and elements, relevant specifically to buildings as follows:

– Section **5**: Specific rules for concrete buildings;

– Section **6**: Specific rules for steel buildings;

– Section **7**: Specific rules for composite steel-concrete buildings;

– Section **8**: Specific rules for timber buildings;

– Section **9**: Specific rules for masonry buildings.

(6) Section **10** contains the fundamental requirements and other relevant aspects of design and safety related to base isolation of structures and specifically to base isolation of buildings.

NOTE Specific rules for isolation of bridges are developed in EN 1998-2.

(7) Annex C contains additional elements related to the design of slab reinforcement in steel-concrete composite beams at beam-column joints of moment frames.

NOTE Informative Annex A and informative Annex B contain additional elements related to the elastic displacement response spectrum and to target displacement for pushover analysis.

1.1.3 Further Parts of EN 1998

(1)P Further Parts of EN 1998 include, in addition to EN 1998-1, the following:

– EN 1998-2 contains specific provisions relevant to bridges;

– EN 1998-3 contains provisions for the seismic assessment and retrofitting of existing buildings;

– EN 1998-4 contains specific provisions relevant to silos, tanks and pipelines;

– EN 1998-5 contains specific provisions relevant to foundations, retaining structures and geotechnical aspects;

– EN 1998-6 contains specific provisions relevant to towers, masts and chimneys.

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 1992-1-1 Eurocode 2 – Design of concrete structures – Part 1-1: General – Common rules for building and civil engineering structures

EN 1993-1-1 Eurocode 3 – Design of steel structures – Part 1-1: General – General rules

EN 1994-1-1 Eurocode 4 – Design of composite steel and concrete structures – Part 1-1: General – Common rules and rules for buildings

EN 1995-1-1 Eurocode 5 – Design of timber structures – Part 1-1: General – Common rules and rules for buildings

EN 1996-1-1 Eurocode 6 – Design of masonry structures – Part 1-1: General –Rules for reinforced and unreinforced masonry

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

1.2.2 Reference Codes and Standards

(1)P For the application of EN 1998, reference shall be made to EN 1990, to EN 1997 and to EN 1999.

(2) EN 1998 incorporates other normative references cited at the appropriate places in the text. They are listed below:

ISO 1000 The international system of units (SI) and its application;

EN 1090-1 Execution of steel structures – Part 1: General rules and rules for buildings;

prEN 12512 Timber structures – Test methods – Cyclic testing of joints made with mechanical fasteners.

1.3 Assumptions

(1) In addition to the general assumptions of EN 1990:2002, **1.3**, the following assumption applies.

(2)P It is assumed that no change in the structure will take place during the construction phase or during the subsequent life of the structure, unless proper justification and verification is provided. Due to the specific nature of the seismic response this applies even in the case of changes that lead to an increase of the structural resistance.

1.4 Distinction between principles and application rules

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

1.5 Terms and definitions

1.5.1 Terms common to all Eurocodes

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

1.5.2 Further terms used in EN 1998

(1) The following terms are used in EN 1998 with the following meanings:

behaviour factor
factor used for design purposes to reduce the forces obtained from a linear analysis, in order to account for the non-linear response of a structure, associated with the material, the structural system and the design procedures

capacity design method
design method in which elements of the structural system are chosen and suitably designed and detailed for energy dissipation under severe deformations while all other structural elements are provided with sufficient strength so that the chosen means of energy dissipation can be maintained

dissipative structure
structure which is able to dissipate energy by means of ductile hysteretic behaviour and/or by other mechanisms

dissipative zones
predetermined parts of a dissipative structure where the dissipative capabilities are mainly located

NOTE 1 These are also called critical regions.

dynamically independent unit
structure or part of a structure which is directly subjected to the ground motion and whose response is not affected by the response of adjacent units or structures

importance factor
factor which relates to the consequences of a structural failure

non-dissipative structure
structure designed for a particular seismic design situation without taking into account the non-linear material behaviour

non-structural element
architectural, mechanical or electrical element, system and component which, whether due to lack of strength or to the way it is connected to the structure, is not considered in the seismic design as load carrying element

primary seismic members
members considered as part of the structural system that resists the seismic action, modelled in the analysis for the seismic design situation and fully designed and detailed for earthquake resistance in accordance with the rules of EN 1998

secondary seismic members
members which are not considered as part of the seismic action resisting system and whose strength and stiffness against seismic actions is neglected

NOTE 2 They are not required to comply with all the rules of EN 1998, but are designed and detailed to maintain support of gravity loads when subjected to the displacements caused by the seismic design situation.

1.6 Symbols

1.6.1 General

(1) The symbols indicated in EN 1990:2002, **1.6** apply. For the material-dependent symbols, as well as for symbols not specifically related to earthquakes, the provisions of the relevant Eurocodes apply.

(2) Further symbols, used in connection with seismic actions, are defined in the text where they occur, for ease of use. However, in addition, the most frequently occurring symbols used in EN 1998-1 are listed and defined in **1.6.2** and **1.6.3**.

1.6.2 Further symbols used in Sections 2 and 3 of EN 1998-1

A_{Ed} design value of seismic action ($= \gamma_{\mathrm{I}}.A_{\mathrm{Ek}}$)

A_{Ek} characteristic value of the seismic action for the reference return period

E_{d} design value of action effects

N_{SPT} Standard Penetration Test blow-count

P_{NCR} reference probability of exceedance in 50 years of the reference seismic action for the no-collapse requirement

Q variable action

$S_{\mathrm{e}}(T)$ elastic horizontal ground acceleration response spectrum also called "elastic response spectrum”. At T=0, the spectral acceleration given by this spectrum equals the design ground acceleration on type A ground multiplied by the soil factor S.

$S_{\mathrm{ve}}(T)$ elastic vertical ground acceleration response spectrum

$S_{\mathrm{De}}(T)$ elastic displacement response spectrum

$S_{\mathrm{d}}(T)$ design spectrum (for elastic analysis). At T=0, the spectral acceleration given by this spectrum equals the design ground acceleration on type A ground multiplied by the soil factor S

S soil factor

T vibration period of a linear single degree of freedom system

T_{s} duration of the stationary part of the seismic motion

T_{NCR} reference return period of the reference seismic action for the no-collapse requirement

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

a_{g} design ground acceleration on type A ground

a_{vg} design ground acceleration in the vertical direction

c_{u} undrained shear strength of soil

d_{g} design ground displacement

g acceleration of gravity

q behaviour factor

$v_{s,30}$ average value of propagation velocity of S waves in the upper 30 m of the soil profile at shear strain of 10^{-5} or less

γ_I importance factor

η damping correction factor

ξ viscous damping ratio (in percent)

$\psi_{2,i}$ combination coefficient for the quasi-permanent value of a variable action i

$\psi_{E,i}$ combination coefficient for a variable action i, to be used when determining the effects of the design seismic action

1.6.3 Further symbols used in Section 4 of EN 1998-1

E_E effect of the seismic action

E_{Edx}, E_{Edy} design values of the action effects due to the horizontal components (x and y) of the seismic action

E_{Edz} design value of the action effects due to the vertical component of the seismic action

F_i horizontal seismic force at storey i

F_a horizontal seismic force acting on a non-structural element (appendage)

F_b base shear force

H building height from the foundation or from the top of a rigid basement

L_{max}, L_{min} larger and smaller in plan dimension of the building measured in orthogonal directions

R_d design value of resistance

S_a seismic coefficient for non-structural elements

T_1 fundamental period of vibration of a building

T_a fundamental period of vibration of a non-structural element (appendage)

W_a weight of a non-structural element (appendage)

d displacement

d_r design interstorey drift

e_a accidental eccentricity of the mass of one storey from its nominal location

h interstorey height

m_i mass of storey i

n number of storeys above the foundation or the top of a rigid basement

q_a behaviour factor of a non-structural element (appendage)

q_d displacement behaviour factor

s_i displacement of mass m_i in the fundamental mode shape of a building

z_i height of mass m_i above the level of application of the seismic action

α ratio of the design ground acceleration to the acceleration of gravity

γ_a importance factor of a non-structural element (appendage)

γ_d overstrength factor for diaphragms

θ interstorey drift sensitivity coefficient

1.6.4 Further symbols used in Section 5 of EN 1998-1

A_c Area of section of concrete member

A_{sh} total area of horizontal hoops in a beam-column joint

A_{si} total area of steel bars in each diagonal direction of a coupling beam

A_{st} area of one leg of the transverse reinforcement

$A_{sv,i}$ total area of bars between corner bars in one direction at the cross-section of a column

A_w total horizontal cross-sectional area of a wall

ΣA_{si} sum of areas of all inclined bars in both directions, in wall reinforced with inclined bars against sliding shear

ΣA_{sj} sum of areas of vertical bars of web in a wall, or of additional bars arranged in the wall boundary elements specifically for resistance against sliding shear

ΣM_{Rb} sum of design values of moments of resistance of the beams framing into a joint in the direction of interest

ΣM_{Rc} sum of design values of the moments of resistance of the columns framing into a joint in the direction of interest

D_o diameter of confined core in a circular column

$M_{i,d}$ end moment of a beam or column for the calculation of its capacity design shear

$M_{Rb,i}$ design value of beam moment of resistance at end *i*

$M_{Rc,i}$ design value of column moment of resistance at end *i*

N_{Ed} axial force from the analysis for the seismic design situation

T_1 fundamental period of the building in the horizontal direction of interest

T_C corner period at the upper limit of the constant acceleration region of the elastic spectrum

V'_{Ed} shear force in a wall from the analysis for the seismic design situation

V_{dd} dowel resistance of vertical bars in a wall

V_{Ed} design shear force in a wall

$V_{Ed,max}$ maximum acting shear force at end section of a beam from capacity design calculation

$V_{Ed,min}$ minimum acting shear force at end section of a beam from capacity design calculation

V_{fd} contribution of friction to resistance of a wall against sliding shear

V_{id} contribution of inclined bars to resistance of a wall against sliding shear

$V_{Rd,c}$ design value of shear resistance for members without shear reinforcement in accordance with EN1992-1-1:2004

$V_{Rd,S}$ design value of shear resistance against sliding

b width of bottom flange of beam

b_c cross-sectional dimension of column

b_{eff} effective flange width of beam in tension at the face of a supporting column

b_i distance between consecutive bars engaged by a corner of a tie or by a cross-tie in a column

b_o width of confined core in a column or in the boundary element of a wall (to centreline of hoops)

b_w thickness of confined parts of a wall section, or width of the web of a beam

b_{wo} thickness of web of a wall

d effective depth of section

d_{bL} longitudinal bar diameter

d_{bw} diameter of hoop

f_{cd} design value of concrete compressive strength

f_{ctm} mean value of tensile strength of concrete

f_{yd} design value of yield strength of steel

$f_{yd,\ h}$ design value of yield strength of the horizontal web reinforcement

$f_{yd,\ v}$ design value of yield strength of the vertical web reinforcement

f_{yld} design value of yield strength of the longitudinal reinforcement

f_{ywd} design value of yield strength of transverse reinforcement

h cross-sectional depth

h_c cross-sectional depth of column in the direction of interest

h_f flange depth

h_{jc} distance between extreme layers of column reinforcement in a beam-column joint

h_{jw} distance between beam top and bottom reinforcement

h_o depth of confined core in a column (to centreline of hoops)

h_s clear storey height

h_w height of wall or cross-sectional depth of beam

k_D factor reflecting the ductility class in the calculation of the required column depth for anchorage of beam bars in a joint, equal to 1 for DCH and to 2/3 for DCM

k_w factor reflecting the prevailing failure mode in structural systems with walls

l_{cl} clear length of a beam or a column

l_{cr} length of critical region

l_i distance between centrelines of the two sets of inclined bars at the base section of walls with inclined bars against sliding shear

l_w length of cross-section of wall

n total number of longitudinal bars laterally engaged by hoops or cross ties on perimeter of column section

q_o basic value of the behaviour factor

s spacing of transverse reinforcement

x_u neutral axis depth

z internal lever arm

α confinement effectiveness factor, angle between diagonal bars and axis of a coupling beam

α_o prevailing aspect ratio of walls of the structural system

α_1 multiplier of horizontal design seismic action at formation of first plastic hinge in the system

α_u multiplier of horizontal seismic design action at formation of global plastic mechanism

γ_c partial factor for concrete

γ_{Rd} model uncertainty factor on design value of resistances in the estimation of capacity design action effects, accounting for various sources of overstrength

γ_s partial factor for steel

ε_{cu2} ultimate strain of unconfined concrete

$\varepsilon_{cu2,c}$ ultimate strain of confined concrete

$\varepsilon_{su,k}$ characteristic value of ultimate elongation of reinforcing steel

$\varepsilon_{sy,d}$ design value of steel strain at yield

η reduction factor on concrete compressive strength due to tensile strains in transverse direction

ζ ratio, $V_{Ed,min}/V_{Ed,max}$, between the minimum and maximum acting shear forces at the end section of a beam

μ_f concrete-to-concrete friction coefficient under cyclic actions

μ_ϕ curvature ductility factor

μ_δ displacement ductility factor

ν axial force due in the seismic design situation, normalised to $A_c f_{cd}$

ξ normalised neutral axis depth

ρ tension reinforcement ratio

ρ' compression steel ratio in beams

σ_{cm} mean value of concrete normal stress

ρ_h reinforcement ratio of horizontal web bars in a wall

ρ_l total longitudinal reinforcement ratio

ρ_{max} maximum allowed tension steel ratio in the critical region of primary seismic beams

ρ_v reinforcement ratio of vertical web bars in a wall

ρ_w shear reinforcement ratio

ω_v mechanical ratio of vertical web reinforcement

ω_{wd} mechanical volumetric ratio of confining reinforcement

1.6.5 Further symbols used in Section 6 of EN 1998-1

L beam span

M_{Ed} design bending moment from the analysis for the seismic design situation

$M_{pl,RdA}$ design value of plastic moment resistance at end A of a member

$M_{pl,RdB}$ design value of plastic moment resistance at end B of a member

N_{Ed} design axial force from the analysis for the seismic design situation

V_{Ed} design shear force from the analysis for the seismic design situation

$N_{Ed,E}$ axial force from the analysis due to the design seismic action alone

$N_{Ed,G}$ axial force due to the non-seismic actions included in the combination of actions for the seismic design situation

$N_{pl,Rd}$ design value of yield resistance in tension of the gross cross-section of a member in accordance with EN 1993-1-1:2004

$V_{pl,Rd}$ design value of shear resistance of a member in accordance with EN 1993-1-1:2004

$N_{Rd}(M_{Ed},V_{Ed})$ design value of axial resistance of column or diagonal in accordance with EN 1993-1-1:2004, taking into account the interaction with the bending moment M_{Ed} and the shear V_{Ed} in the seismic situation

R_d resistance of connection in accordance with EN 1993-1-1:2004

R_{fy} plastic resistance of connected dissipative member based on the design yield stress of material as defined in EN 1993-1-1:2004.

V_{Ed} design shear force from the analysis for the seismic design situation

$V_{Ed,G}$ shear force due to the non seismic actions included in the combination of actions for the seismic design situation

$V_{Ed,M}$ shear force due to the application of the plastic moments of resistance at the two ends of a beam

$V_{wp,Ed}$ design shear force in web panel due to the design seismic action effects

$V_{wp,Rd}$ design shear resistance of the web panel in accordance with EN 1993- 1-1:2004

e length of seismic link

f_y nominal yield strength of steel

$f_{y,max}$ maximum permissible yield stress of steel

q behaviour factor

t_w web thickness of a seismic link

t_f flange thickness of a seismic link

Ω multiplicative factor on axial force $N_{Ed,E}$ from the analysis due to the design seismic action, for the design of the non-dissipative members in concentric or eccentric braced frames per Cl. **6.7.4** and **6.8.3** respectively

α ratio of the smaller design bending moment $M_{Ed,A}$ at one end of a seismic link to the greater bending moments $M_{Ed,B}$ at the end where plastic hinge forms, both moments taken in absolute value

α_1 multiplier of horizontal design seismic action at formation of first plastic hinge in the system

α_u multiplier of horizontal seismic design action at formation of global plastic mechanism

γ_M partial factor for material property

γ_{ov} material overstrength factor

δ beam deflection at midspan relative to tangent to beam axis at beam end (see Figure 6.11)

γ_{pb} multiplicative factor on design value $N_{pl,Rd}$ of yield resistance in tension of compression brace in a V bracing, for the estimation of the unbalanced seismic action effect on the beam to which the bracing is connected

γ_s partial factor for steel

θ_p rotation capacity of the plastic hinge region

$\overline{\lambda}$ non-dimensional slenderness of a member as defined in EN 1993-1-1:2004

1.6.6 Further symbols used in Section 7 of EN 1998-1

A_{pl} horizontal area of the plate

E_a Modulus of Elasticity of steel

E_{cm} mean value of Modulus of Elasticity of concrete in accordance with EN 1992-1-1:2004

I_a second moment of area of the steel section part of a composite section, with respect to the centroid of the composite section

I_c second moment of area of the concrete part of a composite section, with respect to the centroid of the composite section

I_{eq} equivalent second moment of area of the composite section

I_s second moment of area of the rebars in a composite section, with respect to the centroid of the composite section

$M_{pl,Rd,c}$ design value of plastic moment resistance of column, taken as lower bound and computed taking into account the concrete component of the section and only the steel components of the section classified as ductile

$M_{U,Rd,b}$ upper bound plastic resistance of beam, computed taking into account the concrete component of the section and all the steel components in the section, including those not classified as ductile

$V_{wp,Ed}$ design shear force in web panel, computed on the basis of the plastic resistance of the adjacent dissipative zones in beams or connections

$V_{wp,Rd}$ design shear resistance of the composite steel-concrete web panel in accordance with EN 1994-1-1:2004

b width of the flange

b_b width of composite beam (see Figure 7.3a) or bearing width of the concrete of the slab on the column (see Figure 7.7).

b_e partial effective width of flange on each side of the steel web

b_{eff} total effective width of concrete flange

b_o width (minimum dimension) of confined concrete core (to centreline of hoops)

d_{bL} diameter of longitudinal rebars

d_{bw} diameter of hoops

f_{yd} design yield strength of steel

f_{ydf} design yield strength of steel in the flange

f_{ydw} design strength of web reinforcement

h_b depth of composite beam

h_c depth of composite column section

k_r rib shape efficiency factor of profiled steel sheeting

k_t reduction factor of design shear resistance of connectors in accordance with EN 1994-1-1:2004

l_{cl} clear length of column

l_{cr} length of critical region

n steel-to-concrete modular ratio for short term actions

q behaviour factor

r reduction factor on concrete rigidity for the calculation of the stiffness of composite columns

t_f thickness of flange

γ_c partial factor for concrete

γ_M partial factor for material property

γ_{ov} material overstrength factor

γ_s partial factor for steel

ε_a total strain of steel at Ultimate Limit State

ε_{cu2} ultimate compressive strain of unconfined concrete

η minimum degree of connection as defined in **6.6.1.2** of EN 1994-1-1:2004

1.6.7 Further symbols used in Section 8 of EN 1998-1

E_o Modulus of Elasticity of timber for instantaneous loading

b width of timber section

d fastener-diameter

h depth of timber beams

k_{mod} modification factor for instantaneous loading on strength of timber in accordance with EN 1995-1-1:2004

q behaviour factor

γ_M partial factor for material properties

1.6.8 Further symbols used in Section 9 of EN 1998-1

$a_{g,urm}$ upper value of the design ground acceleration at the site for use of unreinforced masonry satisfying the provisions of Eurocode 8

A_{min} total cross-section area of masonry walls required in each horizontal direction for the rules for "simple masonry buildings" to apply

$f_{b,min}$ normalised compressive strength of masonry normal to the bed face

$f_{bh,min}$ normalised compressive strength of masonry parallel to the bed face in the plane of the wall

$f_{m,min}$ minimum strength for mortar

h greater clear height of the openings adjacent to the wall

h_{ef} effective height of the wall

l length of the wall

n number of storeys above ground

$p_{A,min}$ Minimum sum of horizontal cross-sectional areas of shear walls in each direction, as percentage of the total floor area per storey

p_{max} percentage of the total floor area above the level

q behaviour factor

t_{ef} effective thickness of the wall

$\Delta_{A,max}$ maximum difference in horizontal shear wall cross-sectional area between adjacent storeys of "simple masonry buildings"

$\Delta_{m,max}$ maximum difference in mass between adjacent storeys of "simple masonry buildings"

γ_m partial factors for masonry properties

γ_s partial factor for reinforcing steel

λ_{min} ratio between the length of the small and the length of the long side in plan

1.6.9 Further symbols used in Section 10 of EN 1998-1

K_{eff} effective stiffness of the isolation system in the principal horizontal direction under consideration, at a displacement equal to the design displacement d_{dc}

K_V total stiffness of the isolation system in the vertical direction

K_{xi} effective stiffness of a given unit i in the x direction

K_{yi} effective stiffness of a given unit i in the y direction

T_{eff} effective fundamental period of the superstructure corresponding to horizontal translation, the superstructure assumed as a rigid body

T_f fundamental period of the superstructure assumed fixed at the base

T_V fundamental period of the superstructure in the vertical direction, the superstructure assumed as a rigid body

M mass of the superstructure

M_s magnitude

d_{dc} design displacement of the effective stiffness centre in the direction considered

d_{db} total design displacement of an isolator unit

$e_{tot,y}$ total eccentricity in the y direction

f_j horizontal forces at each level j

r_y torsional radius of the isolation system

(x_i,y_i) co-ordinates of the isolator unit i relative to the effective stiffness centre

δ_i amplification factor

ξ_{eff} "effective damping"

1.7 S.I. Units

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

(2) For calculations, the following units are recommended:

- forces and loads: kN, kN/m, kN/m^2
- unit mass: kg/m^3, t/m^3
- mass: kg, t
- unit weight: kN/m^3
- stresses and strengths: N/mm^2 (= MN/m^2 or MPa), kN/m^2 (=kPa)
- moments (bending, etc): kNm
- acceleration: m/s^2, g (=9,81 m/s^2)

2 PERFORMANCE REQUIREMENTS AND COMPLIANCE CRITERIA

2.1 Fundamental requirements

(1)P Structures in seismic regions shall be designed and constructed in such a way that the following requirements are met, each with an adequate degree of reliability.

– No-collapse requirement.

The structure shall be designed and constructed to withstand the design seismic action defined in Section **3** without local or global collapse, thus retaining its structural integrity and a residual load bearing capacity after the seismic events. The design seismic action is expressed in terms of: a) the reference seismic action associated with a reference probability of exceedance, P_{NCR}, in 50 years or a reference return period, T_{NCR}, and b) the importance factor γ_I (see EN 1990:2002 and **(2)**P and **(3)**P of this clause) to take into account reliability differentiation.

NOTE 1 The values to be ascribed to P_{NCR} or to T_{NCR} for use in a country may be found in its National Annex of this document. The recommended values are P_{NCR} =10% and T_{NCR} = 475 years.

NOTE 2 The value of the probability of exceedance, P_R, in T_L years of a specific level of the seismic action is related to the mean return period, T_R, of this level of the seismic action in accordance with the expression $T_R = -T_L / \ln(1- P_R)$. So for a given T_L, the seismic action may equivalently be specified either via its mean return period, T_R, or its probability of exceedance, P_R in T_L years.

– Damage limitation requirement.

The structure shall be designed and constructed to withstand a seismic action having a larger probability of occurrence than the design seismic action, without the occurrence of damage and the associated limitations of use, the costs of which would be disproportionately high in comparison with the costs of the structure itself. The seismic action to be taken into account for the "damage limitation requirement" has a probability of exceedance, P_{DLR}, in 10 years and a return period, T_{DLR}. In the absence of more precise information, the reduction factor applied on the design seismic action in accordance with **4.4.3.2(2)** may be used to obtain the seismic action for the verification of the damage limitation requirement.

NOTE 3 The values to be ascribed to P_{DLR} or to T_{DLR} for use in a country may be found in its National Annex of this document. The recommended values are P_{DLR} =10% and T_{DLR} = 95 years.

(2)P Target reliabilities for the no-collapse requirement and for the damage limitation requirement are established by the National Authorities for different types of buildings or civil engineering works on the basis of the consequences of failure.

(3)P Reliability differentiation is implemented by classifying structures into different importance classes. An importance factor γ_I is assigned to each importance class. Wherever feasible this factor should be derived so as to correspond to a higher or lower value of the return period of the seismic event (with regard to the reference return period) as appropriate for the design of the specific category of structures (see **3.2.1(3)**).

(4) The different levels of reliability are obtained by multiplying the reference seismic action or, when using linear analysis, the corresponding action effects by this importance factor. Detailed guidance on the importance classes and the corresponding importance factors is given in the relevant Parts of EN 1998.

NOTE At most sites the annual rate of exceedance, $H(a_{gR})$, of the reference peak ground acceleration a_{gR} may be taken to vary with a_{gR} as: $H(a_{gR}) \sim k_0\, a_{gR}^{-k}$, with the value of the exponent k depending on seismicity, but being generally of the order of 3. Then, if the seismic action is defined in terms of the reference peak ground acceleration a_{gR}, the value of the importance factor γ_I multiplying the reference seismic action to achieve the same probability of exceedance in T_L years as in the T_{LR} years for which the reference seismic action is defined, may be computed as $\gamma_I \sim (T_{LR}/T_L)^{-1/k}$. Alternatively, the value of the importance factor γ_I that needs to multiply the reference seismic action to achieve a value of the probability of exceeding the seismic action, P_L, in T_L years other than the reference probability of exceedance P_{LR}, over the same T_L years, may be estimated as $\gamma_I \sim (P_L/P_{LR})^{-1/k}$.

2.2 Compliance Criteria

2.2.1 General

(1)P In order to satisfy the fundamental requirements in **2.1** the following limit states shall be checked (see **2.2.2** and **2.2.3**):

- ultimate limit states;
- damage limitation states.

Ultimate limit states are those associated with collapse or with other forms of structural failure which might endanger the safety of people.

Damage limitation states are those associated with damage beyond which specified service requirements are no longer met.

(2)P In order to limit the uncertainties and to promote a good behaviour of structures under seismic actions more severe than the design seismic action, a number of pertinent specific measures shall also be taken (see **2.2.4**).

(3) For well defined categories of structures in cases of low seismicity (see **3.2.1(4)**), the fundamental requirements may be satisfied through the application of rules simpler than those given in the relevant Parts of EN 1998.

(4) In cases of very low seismicity, the provisions of EN 1998 need not be observed (see **3.2.1(5)** and the notes therein for the definition of cases of very low seismicity).

(5) Specific rules for "simple masonry buildings" are given in Section **9**. By conforming to these rules, such "simple masonry buildings" are deemed to satisfy the fundamental requirements of EN 1998-1 without analytical safety verifications.

2.2.2 Ultimate limit state

(1)P It shall be verified that the structural system has the resistance and energy-dissipation capacity specified in the relevant Parts of EN 1998.

(2) The resistance and energy-dissipation capacity to be assigned to the structure are related to the extent to which its non-linear response is to be exploited. In operational terms such balance between resistance and energy-dissipation capacity is characterised by the values of the behaviour factor q and the associated ductility classification, which are given in the relevant Parts of EN 1998. As a limiting case, for the design of structures classified as non-dissipative, no account is taken of any hysteretic energy dissipation and the behaviour factor may not be taken, in general, as being greater than the value of 1,5 considered to account for overstrengths. For steel or composite steel concrete buildings, this limiting value of the q factor may be taken as being between 1,5 and 2 (see Note 1 of Table 6.1 or Note 1 of Table 7.1, respectively). For dissipative structures the behaviour factor is taken as being greater than these limiting values accounting for the hysteretic energy dissipation that mainly occurs in specifically designed zones, called dissipative zones or critical regions.

NOTE The value of the behaviour factor q should be limited by the limit state of dynamic stability of the structure and by the damage due to low-cycle fatigue of structural details (especially connections). The most unfavourable limiting condition shall be applied when the values of the q factor are determined. The values of the q factor given in the various Parts of EN 1998 are deemed to conform to this requirement.

(3)P The structure as a whole shall be checked to ensure that it is stable under the design seismic action. Both overturning and sliding stability shall be taken into account. Specific rules for checking the overturning of structures are given in the relevant Parts of EN 1998.

(4)P It shall be verified that both the foundation elements and the foundation soil are able to resist the action effects resulting from the response of the superstructure without substantial permanent deformations. In determining the reactions, due consideration shall be given to the actual resistance that can be developed by the structural element transmitting the actions.

(5)P In the analysis the possible influence of second order effects on the values of the action effects shall be taken into account.

(6)P It shall be verified that under the design seismic action the behaviour of non-structural elements does not present risks to persons and does not have a detrimental effect on the response of the structural elements. For buildings, specific rules are given in **4.3.5** and **4.3.6**.

2.2.3 Damage limitation state

(1)P An adequate degree of reliability against unacceptable damage shall be ensured by satisfying the deformation limits or other relevant limits defined in the relevant Parts of EN 1998.

(2)P In structures important for civil protection the structural system shall be verified to ensure that it has sufficient resistance and stiffness to maintain the function of the vital services in the facilities for a seismic event associated with an appropriate return period.

2.2.4 Specific measures

2.2.4.1 Design

(1) To the extent possible, structures should have simple and regular forms both in plan and elevation, (see **4.2.3**). If necessary this may be realised by subdividing the structure by joints into dynamically independent units.

(2)P In order to ensure an overall dissipative and ductile behaviour, brittle failure or the premature formation of unstable mechanisms shall be avoided. To this end, where required in the relevant Parts of EN 1998, resort shall be made to the capacity design procedure, which is used to obtain the hierarchy of resistance of the various structural components and failure modes necessary for ensuring a suitable plastic mechanism and for avoiding brittle failure modes.

(3)P Since the seismic performance of a structure is largely dependent on the behaviour of its critical regions or elements, the detailing of the structure in general and of these regions or elements in particular, shall be such as to maintain the capacity to transmit the necessary forces and to dissipate energy under cyclic conditions. To this end, the detailing of connections between structural elements and of regions where non-linear behaviour is foreseeable should receive special care in design.

(4)P The analysis shall be based on an adequate structural model, which, when necessary, shall take into account the influence of soil deformability and of non-structural elements and other aspects, such as the presence of adjacent structures.

2.2.4.2 Foundations

(1)P The stiffness of the foundations shall be adequate for transmitting the actions received from the superstructure to the ground as uniformly as possible.

(2) With the exception of bridges, only one foundation type should in general be used for the same structure, unless the latter consists of dynamically independent units.

2.2.4.3 Quality system plan

(1)P The design documents shall indicate the sizes, the details and the characteristics of the materials of the structural elements. If appropriate, the design documents shall also include the characteristics of special devices to be used and the distances between structural and non-structural elements. The necessary quality control provisions shall also be given.

(2)P Elements of special structural importance requiring special checking during construction shall be identified on the design drawings. In this case the checking methods to be used shall also be specified.

(3) In regions of high seismicity and in structures of special importance, formal quality system plans, covering design, construction, and use, additional to the control procedures prescribed in the other relevant Eurocodes, should be used.

3 GROUND CONDITIONS AND SEISMIC ACTION

3.1 Ground conditions

3.1.1 General

(1)P Appropriate investigations shall be carried out in order to identify the ground conditions in accordance with the types given in **3.1.2**.

(2) Further guidance concerning ground investigation and classification is given in EN 1998-5:2004, **4.2**.

(3) The construction site and the nature of the supporting ground should normally be free from risks of ground rupture, slope instability and permanent settlements caused by liquefaction or densification in the event of an earthquake. The possibility of occurrence of such phenomena shall be investigated in accordance with EN 1998-5:2004, Section **4**.

(4) Depending on the importance class of the structure and the particular conditions of the project, ground investigations and/or geological studies should be performed to determine the seismic action.

NOTE The conditions under which ground investigations additional to those necessary for design for non-seismic actions may be omitted and default ground classification may be used may be specified in the National Annex.

3.1.2 Identification of ground types

(1) Ground types A, B, C, D, and E, described by the stratigraphic profiles and parameters given in Table 3.1 and described hereafter, may be used to account for the influence of local ground conditions on the seismic action. This may also be done by additionally taking into account the influence of deep geology on the seismic action.

NOTE The ground classification scheme accounting for deep geology for use in a country may be specified in its National Annex, including the values of the parameters S, T_B, T_C and T_D defining the horizontal and vertical elastic response spectra in accordance with **3.2.2.2** and **3.2.2.3**.

Table 3.1: Ground types

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	_	_
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	_	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

(2) The site should be classified according to the value of the average shear wave velocity, $v_{s,30}$, if this is available. Otherwise the value of N_{SPT} should be used.

(3) The average shear wave velocity $v_{s,30}$ should be computed in accordance with the following expression:

$$v_{s,30} = \frac{30}{\sum_{i=1,N} \frac{h_i}{v_i}} \tag{3.1}$$

where h_i and v_i denote the thickness (in metres) and shear-wave velocity (at a shear strain level of 10^{-5} or less) of the *i*-th formation or layer, in a total of *N*, existing in the top 30 m.

(4)P For sites with ground conditions matching either one of the two special ground types S_1 or S_2, special studies for the definition of the seismic action are required. For these types, and particularly for S_2, the possibility of soil failure under the seismic action shall be taken into account.

NOTE Special attention should be paid if the deposit is of ground type S_1. Such soils typically have very low values of v_s, low internal damping and an abnormally extended range of linear behaviour and can therefore produce anomalous seismic site amplification and soil-structure interaction effects (see EN 1998-5:2004, Section **6**). In this case, a special study to define the seismic action should be carried out, in order to establish the dependence of the response spectrum on the thickness and v_s value of the soft clay/silt layer and on the stiffness contrast between this layer and the underlying materials.

3.2 Seismic action

3.2.1 Seismic zones

(1)P For the purpose of EN 1998, national territories shall be subdivided by the National Authorities into seismic zones, depending on the local hazard. By definition, the hazard within each zone is assumed to be constant.

(2) For most of the applications of EN 1998, the hazard is described in terms of a single parameter, i.e. the value of the reference peak ground acceleration on type A ground, a_{gR}. Additional parameters required for specific types of structures are given in the relevant Parts of EN 1998.

NOTE The reference peak ground acceleration on type A ground, a_{gR}, for use in a country or parts of the country, may be derived from zonation maps found in its National Annex.

(3) The reference peak ground acceleration, chosen by the National Authorities for each seismic zone, corresponds to the reference return period T_{NCR} of the seismic action for the no-collapse requirement (or equivalently the reference probability of exceedance in 50 years, P_{NCR}) chosen by the National Authorities (see **2.1(1)**P). An importance factor γ_I equal to 1,0 is assigned to this reference return period. For return periods other than the reference (see importance classes in **2.1(3)**P and **(4)**), the design ground acceleration on type A ground a_g is equal to a_{gR} times the importance factor γ_I ($a_g = \gamma_I.a_{gR}$). (See Note to **2.1(4)**).

(4) In cases of low seismicity, reduced or simplified seismic design procedures for certain types or categories of structures may be used.

NOTE The selection of the categories of structures, ground types and seismic zones in a country for which the provisions of low seismicity apply may be found in its National Annex. It is recommended to consider as low seismicity cases either those in which the design ground acceleration on type A ground, a_g, is not greater than 0,08 g (0,78 m/s^2), or those where the product $a_g.S$ is not greater than 0,1 g (0,98 m/s^2). The selection of whether the value of a_g, or that of the product $a_g.S$ will be used in a country to define the threshold for low seismicity cases, may be found in its National Annex.

(5)P In cases of very low seismicity, the provisions of EN 1998 need not be observed.

NOTE The selection of the categories of structures, ground types and seismic zones in a country for which the EN 1998 provisions need not be observed (cases of very low seismicity) may be found in its National Annex. It is recommended to consider as very low seismicity cases either those in which the design ground acceleration on type A ground, a_g, is not greater than 0,04 g (0,39 m/s^2), or those where the product $a_g.S$ is not greater than 0,05 g (0,49 m/s^2). The selection of whether the value of a_g, or that of the product $a_g.S$ will be used in a country to define the threshold for very low seismicity cases, can be found in its National Annex.

3.2.2 Basic representation of the seismic action

3.2.2.1 General

(1)P Within the scope of EN 1998 the earthquake motion at a given point on the surface is represented by an elastic ground acceleration response spectrum, henceforth called an “elastic response spectrum”.

(2) The shape of the elastic response spectrum is taken as being the same for the two levels of seismic action introduced in **2.1(1)**P and **2.2.1(1)**P for the no-collapse requirement (ultimate limit state – design seismic action) and for the damage limitation requirement.

(3)P The horizontal seismic action is described by two orthogonal components assumed as being independent and represented by the same response spectrum.

(4) For the three components of the seismic action, one or more alternative shapes of response spectra may be adopted, depending on the seismic sources and the earthquake magnitudes generated from them.

NOTE 1 The selection of the shape of the elastic response spectrum to be used in a country or part of the country may be found in its National Annex.

NOTE 2 In selecting the appropriate shape of the spectrum, consideration should be given to the magnitude of earthquakes that contribute most to the seismic hazard defined for the purpose of probabilistic hazard assessment, rather than on conservative upper limits (e.g. the Maximum Credible Earthquake) defined for that purpose.

(5) When the earthquakes affecting a site are generated by widely differing sources, the possibility of using more than one shape of spectra should be considered to enable the design seismic action to be adequately represented. In such circumstances, different values of a_g will normally be required for each type of spectrum and earthquake.

(6) For important structures (γ_I >1,0) topographic amplification effects should be taken into account.

NOTE Informative Annex A of EN 1998-5:2004 provides information for topographic amplification effects.

(7) Time-history representations of the earthquake motion may be used (see **3.2.3**).

(8) Allowance for the variation of ground motion in space as well as time may be required for specific types of structures (see EN 1998-2, EN 1998-4 and EN 1998-6).

3.2.2.2 Horizontal elastic response spectrum

(1)P For the horizontal components of the seismic action, the elastic response spectrum $S_e(T)$ is defined by the following expressions (see Figure. 3.1):

$$0 \le T \le T_B : \; S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2{,}5 - 1)\right] \qquad (3.2)$$

$$T_B \le T \le T_C : \; S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \qquad (3.3)$$

$$T_C \le T \le T_D : \; S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \left[\frac{T_C}{T}\right] \qquad (3.4)$$

$$T_D \le T \le 4\text{s} : \; S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \left[\frac{T_C T_D}{T^2}\right] \qquad (3.5)$$

where

$S_e(T)$ is the elastic response spectrum;

T is the vibration period of a linear single-degree-of-freedom system;

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

T_B is the lower limit of the period of the constant spectral acceleration branch;

T_C is the upper limit of the period of the constant spectral acceleration branch;

T_D is the value defining the beginning of the constant displacement response range of the spectrum;

S is the soil factor;

η is the damping correction factor with a reference value of $\eta = 1$ for 5% viscous damping, see **(3)** of this subclause.

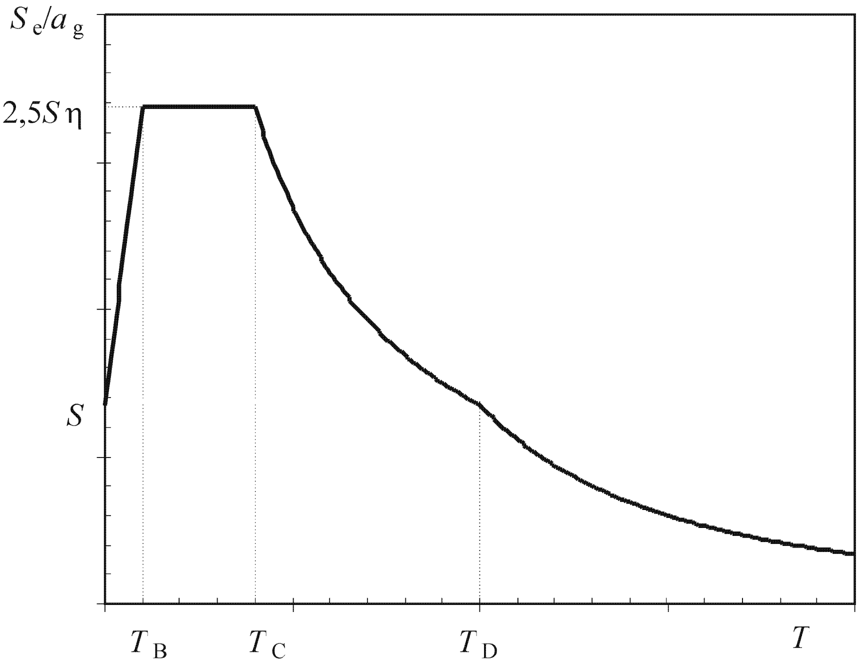


Figure 3.1: Shape of the elastic response spectrum

(2)P The values of the periods T_B, T_C and T_D and of the soil factor S describing the shape of the elastic response spectrum depend upon the ground type.

NOTE 1 The values to be ascribed to T_B, T_C, T_D and S for each ground type and type (shape) of spectrum to be used in a country may be found in its National Annex. If deep geology is not accounted for (see **3.1.2(1)**), the recommended choice is the use of two types of spectra: Type 1 and Type 2. If the earthquakes that contribute most to the seismic hazard defined for the site for the purpose of probabilistic hazard assessment have a surface-wave magnitude, M_s, not greater than 5,5, it is recommended that the Type 2 spectrum is adopted. For the five ground types A, B, C, D and E the recommended values of the parameters S, T_B, T_C and T_D are given in Table 3.2 for the Type 1 Spectrum and in Table 3.3 for the Type 2 Spectrum. Figure 3.2 and Figure 3.3 show the shapes of the recommended Type 1 and Type 2 spectra, respectively, normalised by a_g, for 5% damping. Different spectra may be defined in the National Annex, if deep geology is accounted for.

Table 3.2: Values of the parameters describing the recommended Type 1 elastic response spectra

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

Table 3.3: Values of the parameters describing the recommended Type 2 elastic response spectra

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,05	0,25	1,2
B	1,35	0,05	0,25	1,2
C	1,5	0,10	0,25	1,2
D	1,8	0,10	0,30	1,2
E	1,6	0,05	0,25	1,2

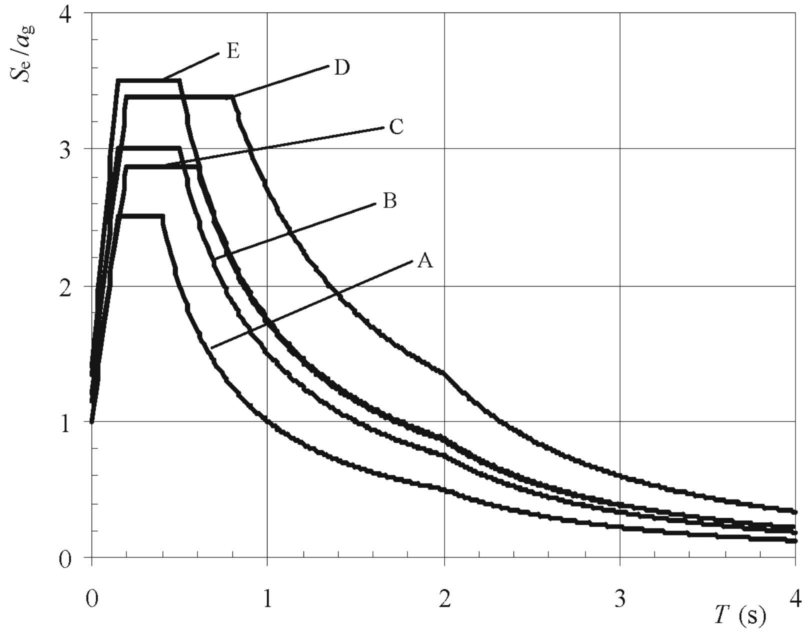


Figure 3.2: Recommended Type 1 elastic response spectra for ground types A to E (5% damping)

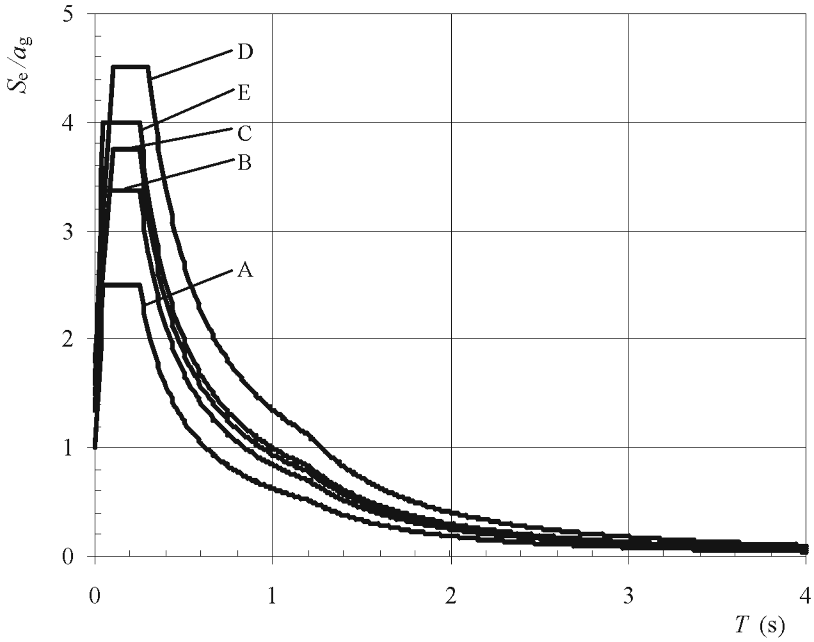


Figure 3.3: Recommended Type 2 elastic response spectra for ground types A to E (5% damping)

Note 2 For ground types S_1 and S_2, special studies should provide the corresponding values of S, T_B, T_C and T_D.

(3) The value of the damping correction factor η may be determined by the expression:

$$\eta = \sqrt{10/(5+\xi)} \geq 0{,}55 \tag{3.6}$$

where ξ is the viscous damping ratio of the structure, expressed as a percentage.

(4) If for special cases a viscous damping ratio different from 5% is to be used, this value is given in the relevant Part of EN 1998.

(5)P The elastic displacement response spectrum, $S_{De}(T)$, shall be obtained by direct transformation of the elastic acceleration response spectrum, $S_e(T)$, using the following expression:

$$S_{De}(T) = S_e(T)\left[\frac{T}{2\pi}\right]^2 \tag{3.7}$$

(6) Expression (3.7) should normally be applied for vibration periods not exceeding 4,0 s. For structures with vibration periods longer than 4,0 s, a more complete definition of the elastic displacement spectrum is possible.

NOTE For the Type 1 elastic response spectrum referred to in Note 1 to **3.2.2.2(2)**P, such a definition is presented in Informative Annex A in terms of the displacement response spectrum. For periods longer than 4,0 s, the elastic acceleration response spectrum may be derived from the elastic displacement response spectrum by inverting expression (3.7).

3.2.2.3 Vertical elastic response spectrum

(1)P The vertical component of the seismic action shall be represented by an elastic response spectrum, $S_{ve}(T)$, derived using expressions (3.8)-(3.11).

$$0 \leq T \leq T_B : S_{ve}(T) = a_{vg} \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 3{,}0 - 1)\right] \tag{3.8}$$

$$T_B \leq T \leq T_C : S_{ve}(T) = a_{vg} \cdot \eta \cdot 3{,}0 \tag{3.9}$$

$$T_C \leq T \leq T_D : S_{ve}(T) = a_{vg} \cdot \eta \cdot 3{,}0\left[\frac{T_C}{T}\right] \tag{3.10}$$

$$T_D \leq T \leq 4\text{s} : S_{ve}(T) = a_{vg} \cdot \eta \cdot 3{,}0\left[\frac{T_C . T_D}{T^2}\right] \tag{3.11}$$

NOTE The values to be ascribed to T_B, T_C, T_D and a_{vg} for each type (shape) of vertical spectrum to be used in a country may be found in its National Annex. The recommended choice is the use of two types of vertical spectra: Type 1 and Type 2. As for the spectra defining the horizontal components of the seismic action, if the earthquakes that contribute most to the seismic hazard defined for the site for the purpose of probabilistic hazard assessment have a surface-wave magnitude, M_s, not greater than 5,5, it is recommended that the Type 2 spectrum is adopted. For the five ground types A, B, C, D and E the recommended values of the parameters describing the vertical spectra are given in Table 3.4. These recommended values do not apply for special ground types S_1 and S_2.

Table 3.4: Recommended values of parameters describing the vertical elastic response spectra

Spectrum	a_{vg}/a_g	T_B (s)	T_C (s)	T_D (s)
Type 1	0,90	0,05	0,15	1,0
Type 2	0,45	0,05	0,15	1,0

3.2.2.4 Design ground displacement

(1) Unless special studies based on the available information indicate otherwise, the design ground displacement d_g, corresponding to the design ground acceleration, may be estimated by means of the following expression:

$$d_g = 0{,}025 \cdot a_g \cdot S \cdot T_C \cdot T_D \tag{3.12}$$

with a_g, S, T_C and T_D as defined in **3.2.2.2**.

3.2.2.5 Design spectrum for elastic analysis

(1) The capacity of structural systems to resist seismic actions in the non-linear range generally permits their design for resistance to seismic forces smaller than those corresponding to a linear elastic response.

(2) To avoid explicit inelastic structural analysis in design, the capacity of the structure to dissipate energy, through mainly ductile behaviour of its elements and/or other mechanisms, is taken into account by performing an elastic analysis based on a response spectrum reduced with respect to the elastic one, henceforth called a "design spectrum". This reduction is accomplished by introducing the behaviour factor q.

(3)P The behaviour factor q is an approximation of the ratio of the seismic forces that the structure would experience if its response was completely elastic with 5% viscous damping, to the seismic forces that may be used in the design, with a conventional elastic analysis model, still ensuring a satisfactory response of the structure. The values of the behaviour factor q, which also account for the influence of the viscous damping being different from 5%, are given for various materials and structural systems according to the relevant ductility classes in the various Parts of EN 1998. The value of the behaviour factor q may be different in different horizontal directions of the structure, although the ductility classification shall be the same in all directions.

(4)P For the horizontal components of the seismic action the design spectrum, $S_d(T)$, shall be defined by the following expressions:

$$0 \le T \le T_B : \; S_d(T) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2{,}5}{q} - \frac{2}{3}\right)\right] \tag{3.13}$$

$$T_{\mathrm{B}} \leq T \leq T_{\mathrm{C}}: \; S_{\mathrm{d}}(T) = a_{\mathrm{g}} \cdot S \cdot \frac{2,5}{q} \tag{3.14}$$

$$T_{\mathrm{C}} \leq T \leq T_{\mathrm{D}}: \; S_{\mathrm{d}}(T) \begin{cases} = a_{\mathrm{g}} \cdot S \cdot \dfrac{2,5}{q} \cdot \left[\dfrac{T_{\mathrm{C}}}{T}\right] \\ \geq \beta \cdot a_{\mathrm{g}} \end{cases} \tag{3.15}$$

$$T_{\mathrm{D}} \leq T: \quad S_{\mathrm{d}}(T) \begin{cases} = a_{\mathrm{g}} \cdot S \cdot \dfrac{2,5}{q} \cdot \left[\dfrac{T_{\mathrm{C}} T_{\mathrm{D}}}{T^2}\right] \\ \geq \beta \cdot a_{\mathrm{g}} \end{cases} \tag{3.16}$$

where

a_{g}, S, T_{C} and T_{D}	are as defined in **3.2.2.2**;
$S_{\mathrm{d}}(T)$	is the design spectrum;
q	is the behaviour factor;
β	is the lower bound factor for the horizontal design spectrum.

NOTE The value to be ascribed to β for use in a country can be found in its National Annex. The recommended value for β is 0,2.

(5) For the vertical component of the seismic action the design spectrum is given by expressions (3.13) to (3.16), with the design ground acceleration in the vertical direction, a_{vg} replacing a_{g}, S taken as being equal to 1,0 and the other parameters as defined in **3.2.2.3**.

(6) For the vertical component of the seismic action a behaviour factor q up to to 1,5 should generally be adopted for all materials and structural systems.

(7) The adoption of values for q greater than 1,5 in the vertical direction should be justified through an appropriate analysis.

(8)P The design spectrum as defined above is not sufficient for the design of structures with base-isolation or energy-dissipation systems.

3.2.3 Alternative representations of the seismic action

3.2.3.1 Time - history representation

3.2.3.1.1 General

(1)P The seismic motion may also be represented in terms of ground acceleration time-histories and related quantities (velocity and displacement).

(2)P When a spatial model is required, the seismic motion shall consist of three simultaneously acting accelerograms. The same accelerogram may not be used simultaneously along both horizontal directions. Simplifications are possible in accordance with the relevant Parts of EN 1998.

(3) Depending on the nature of the application and on the information actually available, the description of the seismic motion may be made by using artificial accelerograms (see **3.2.3.1.2**) and recorded or simulated accelerograms (see **3.2.3.1.3**).

3.2.3.1.2 Artificial accelerograms

(1)P Artificial accelerograms shall be generated so as to match the elastic response spectra given in **3.2.2.2** and **3.2.2.3** for 5% viscous damping (ξ = 5%).

(2)P The duration of the accelerograms shall be consistent with the magnitude and the other relevant features of the seismic event underlying the establishment of a_g.

(3) When site-specific data are not available, the minimum duration T_s of the stationary part of the accelerograms should be equal to 10 s.

(4) The suite of artificial accelerograms should observe the following rules:

a) a minimum of 3 accelerograms should be used;

b) the mean of the zero period spectral response acceleration values (calculated from the individual time histories) should not be smaller than the value of $a_g.S$ for the site in question.

c) in the range of periods between $0,2T_1$ and $2T_1$, where T_1 is the fundamental period of the structure in the direction where the accelerogram will be applied; no value of the mean 5% damping elastic spectrum, calculated from all time histories, should be less than 90% of the corresponding value of the 5% damping elastic response spectrum.

3.2.3.1.3 Recorded or simulated accelerograms

(1)P Recorded accelerograms, or accelerograms generated through a physical simulation of source and travel path mechanisms, may be used, provided that the samples used are adequately qualified with regard to the seismogenetic features of the sources and to the soil conditions appropriate to the site, and their values are scaled to the value of $a_g.S$ for the zone under consideration.

(2)P For soil amplification analyses and for dynamic slope stability verifications see EN 1998-5:2004, **2.2**.

(3) The suite of recorded or simulated accelerograms to be used should satisfy **3.2.3.1.2(4)**.

3.2.3.2 Spatial model of the seismic action

(1)P For structures with special characteristics such that the assumption of the same excitation at all support points cannot reasonably be made, spatial models of the seismic action shall be used (see **3.2.2.1(8)**).

(2)P Such spatial models shall be consistent with the elastic response spectra used for the basic definition of the seismic action in accordance with **3.2.2.2** and **3.2.2.3**.

3.2.4 Combinations of the seismic action with other actions

(1)P The design value E_d of the effects of actions in the seismic design situation shall be determined in accordance with EN 1990:2002, **6.4.3.4**.

(2)P The inertial effects of the design seismic action shall be evaluated by taking into account the presence of the masses associated with all gravity loads appearing in the following combination of actions:

$$\Sigma G_{k,j} \text{"+"} \Sigma \psi_{E,i} \cdot Q_{k,i} \tag{3.17}$$

where

$\psi_{E,i}$ is the combination coefficient for variable action i (see **4.2.4**).

(3) The combination coefficients $\psi_{E,i}$ take into account the likelihood of the loads $Q_{k,i}$ not being present over the entire structure during the earthquake. These coefficients may also account for a reduced participation of masses in the motion of the structure due to the non-rigid connection between them.

(4) Values of $\psi_{2,i}$ are given in EN 1990:2002 and values of $\psi_{E,i}$ for buildings or other types of structures are given in the relevant parts of EN 1998.

4 DESIGN OF BUILDINGS

4.1 General

4.1.1 Scope

(1)P Section **4** contains general rules for the earthquake-resistant design of buildings and shall be used in conjunction with Sections **2**, **3** and **5** to **9**.

(2) Sections **5** to **9** are concerned with specific rules for various materials and elements used in buildings.

(3) Guidance on base-isolated buildings is given in Section **10**.

4.2 Characteristics of earthquake resistant buildings

4.2.1 Basic principles of conceptual design

(1)P In seismic regions the aspect of seismic hazard shall be taken into account in the early stages of the conceptual design of a building, thus enabling the achievement of a structural system which, within acceptable costs, satisfies the fundamental requirements specified in **2.1**.

(2) The guiding principles governing this conceptual design are:

- structural simplicity;
- uniformity, symmetry and redundancy;
- bi-directional resistance and stiffness;
- torsional resistance and stiffness;
- diaphragmatic behaviour at storey level;
- adequate foundation.

These principles are further elaborated in the following subclauses.

4.2.1.1 Structural simplicity

(1) Structural simplicity, characterised by the existence of clear and direct paths for the transmission of the seismic forces, is an important objective to be pursued, since the modelling, analysis, dimensioning, detailing and construction of simple structures are subject to much less uncertainty and thus the prediction of its seismic behaviour is much more reliable.

4.2.1.2 Uniformity, symmetry and redundancy

(1) Uniformity in plan is characterised by an even distribution of the structural elements which allows short and direct transmission of the inertia forces created in the distributed masses of the building. If necessary, uniformity may be realised by subdividing the entire building by seismic joints into dynamically independent units,

provided that these joints are designed against pounding of the individual units in accordance with **4.4.2.7**.

(2) Uniformity in the development of the structure along the height of the building is also important, since it tends to eliminate the occurrence of sensitive zones where concentrations of stress or large ductility demands might prematurely cause collapse.

(3) A close relationship between the distribution of masses and the distribution of resistance and stiffness eliminates large eccentricities between mass and stiffness.

(4) If the building configuration is symmetrical or quasi-symmetrical, a symmetrical layout of structural elements, which should be well-distributed in-plan, is appropriate for the achievement of uniformity.

(5) The use of evenly distributed structural elements increases redundancy and allows a more favourable redistribution of action effects and widespread energy dissipation across the entire structure.

4.2.1.3 Bi-directional resistance and stiffness

(1)P Horizontal seismic motion is a bi-directional phenomenon and thus the building structure shall be able to resist horizontal actions in any direction.

(2) To satisfy **(1)**P, the structural elements should be arranged in an orthogonal in-plan structural pattern, ensuring similar resistance and stiffness characteristics in both main directions.

(3) The choice of the stiffness characteristics of the structure, while attempting to minimise the effects of the seismic action (taking into account its specific features at the site) should also limit the development of excessive displacements that might lead to either instabilities due to second order effects or excessive damages.

4.2.1.4 Torsional resistance and stiffness

(1) Besides lateral resistance and stiffness, building structures should possess adequate torsional resistance and stiffness in order to limit the development of torsional motions which tend to stress the different structural elements in a non-uniform way . In this respect, arrangements in which the main elements resisting the seismic action are distributed close to the periphery of the building present clear advantages.

4.2.1.5 Diaphragmatic behaviour at storey level

(1) In buildings, floors (including the roof) play a very important role in the overall seismic behaviour of the structure. They act as horizontal diaphragms that collect and transmit the inertia forces to the vertical structural systems and ensure that those systems act together in resisting the horizontal seismic action. The action of floors as diaphragms is especially relevant in cases of complex and non-uniform layouts of the vertical structural systems, or where systems with different horizontal deformability characteristics are used together (e.g. in dual or mixed systems).

(2) Floor systems and the roof should be provided with in-plane stiffness and resistance and with effective connection to the vertical structural systems. Particular

care should be taken in cases of non-compact or very elongated in-plan shapes and in cases of large floor openings, especially if the latter are located in the vicinity of the main vertical structural elements, thus hindering such effective connection between the vertical and horizontal structure.

(3) Diaphragms should have sufficient in-plane stiffness for the distribution of horizontal inertia forces to the vertical structural systems in accordance with the assumptions of the analysis (e.g. rigidity of the diaphragm, see **4.3.1(4)**), particularly when there are significant changes in stiffness or offsets of vertical elements above and below the diaphragm.

4.2.1.6 Adequate foundation

(1)P With regard to the seismic action, the design and construction of the foundations and of the connection to the superstructure shall ensure that the whole building is subjected to a uniform seismic excitation.

(2) For structures composed of a discrete number of structural walls, likely to differ in width and stiffness, a rigid, box-type or cellular foundation, containing a foundation slab and a cover slab should generally be chosen.

(3) For buildings with individual foundation elements (footings or piles), the use of a foundation slab or tie-beams between these elements in both main directions is recommended, subject to the criteria and rules of EN 1998-5:2004, **5.4.1.2**.

4.2.2 Primary and secondary seismic members

(1)P A certain number of structural members (e.g. beams and/or columns) may be designated as “secondary” seismic members (or elements), not forming part of the seismic action resisting system of the building. The strength and stiffness of these elements against seismic actions shall be neglected. They do not need to conform to the requirements of Sections **5** to **9**. Nonetheless these members and their connections shall be designed and detailed to maintain support of gravity loading when subjected to the displacements caused by the most unfavourable seismic design condition. Due allowance of 2^{nd} order effects (P-Δ effects) should be made in the design of these members.

(2) Sections **5** to **9** give rules, in addition to those of EN 1992, EN 1993, EN 1994, EN 1995 and EN 1996, for the design and detailing of secondary seismic elements.

(3) All structural members not designated as being secondary seismic members are taken as being primary seismic members. They are taken as being part of the lateral force resisting system, should be modelled in the structural analysis in accordance with **4.3.1** and designed and detailed for earthquake resistance in accordance with the rules of Sections **5** to **9**.

(4) The total contribution to lateral stiffness of all secondary seismic members should not exceed 15% of that of all primary seismic members.

(5) The designation of some structural elements as secondary seismic members is not allowed to change the classification of the structure from non-regular to regular as described in **4.2.3**.

4.2.3 Criteria for structural regularity

4.2.3.1 General

(1)P For the purpose of seismic design, building structures are categorised into being regular or non-regular.

NOTE In building structures consisting of more than one dynamically independent units, the categorisation and the relevant criteria in **4.2.3** refer to the individual dynamically independent units. In such structures, "individual dynamically independent unit" is meant for "building" in **4.2.3.**

(2) This distinction has implications for the following aspects of the seismic design:

- the structural model, which can be either a simplified planar model or a spatial model ;
- the method of analysis, which can be either a simplified response spectrum analysis (lateral force procedure) or a modal one;
- the value of the behaviour factor q, which shall be decreased for buildings non-regular in elevation (see **4.2.3.3**).

(3)P With regard to the implications of structural regularity on analysis and design, separate consideration is given to the regularity characteristics of the building in plan and in elevation (Table 4.1).

Table 4.1: Consequences of structural regularity on seismic analysis and design

Regularity		Allowed Simplification		Behaviour factor
Plan	Elevation	Model	Linear-elastic Analysis	(for linear analysis)
Yes	Yes	Planar	Lateral force[a]	Reference value
Yes	No	Planar	Modal	Decreased value
No	Yes	Spatial[b]	Lateral force[a]	Reference value
No	No	Spatial	Modal	Decreased value

[a] If the condition of **4.3.3.2.1(2)**a) is also met.
[b] Under the specific conditions given in **4.3.3.1(8)** a separate planar model may be used in each horizontal direction, in accordance with **4.3.3.1(8)**.

(4) Criteria describing regularity in plan and in elevation are given in **4.2.3.2** and **4.2.3.3.** Rules concerning modelling and analysis are given in **4.3**.

(5)P The regularity criteria given in **4.2.3.2** and **4.2.3.3** should be taken as necessary conditions. It shall be verified that the assumed regularity of the building structure is not impaired by other characteristics, not included in these criteria.

(6) The reference values of the behaviour factors are given in Sections **5** to **9**.

(7) For non-regular in elevation buildings the decreased values of the behaviour factor are given by the reference values multiplied by 0,8.

4.2.3.2 Criteria for regularity in plan

(1)P For a building to be categorised as being regular in plan, it shall satisfy all the conditions listed in the following paragraphs.

(2) With respect to the lateral stiffness and mass distribution, the building structure shall be approximately symmetrical in plan with respect to two orthogonal axes.

(3) The plan configuration shall be compact, i.e., each floor shall be delimited by a polygonal convex line. If in plan set-backs (re-entrant corners or edge recesses) exist, regularity in plan may still be considered as being satisfied, provided that these set-backs do not affect the floor in-plan stiffness and that, for each set-back, the area between the outline of the floor and a convex polygonal line enveloping the floor does not exceed 5 % of the floor area.

(4) The in-plan stiffness of the floors shall be sufficiently large in comparison with the lateral stiffness of the vertical structural elements, so that the deformation of the floor shall have a small effect on the distribution of the forces among the vertical structural elements. In this respect, the L, C, H, I, and X plan shapes should be carefully examined, notably as concerns the stiffness of the lateral branches, which should be comparable to that of the central part, in order to satisfy the rigid diaphragm condition. The application of this paragraph should be considered for the global behaviour of the building.

(5) The slenderness $\lambda = L_{max}/L_{min}$ of the building in plan shall be not higher than 4, where L_{max} and L_{min} are respectively the larger and smaller in plan dimension of the building, measured in orthogonal directions.

(6) At each level and for each direction of analysis x and y, the structural eccentricity e_o and the torsional radius r shall be in accordance with the two conditions below, which are expressed for the direction of analysis y:

$$e_{ox} \leq 0{,}30 \cdot r_x \tag{4.1a}$$

$$r_x \geq l_s \tag{4.1b}$$

where

e_{ox} is the distance between the centre of stiffness and the centre of mass, measured along the x direction, which is normal to the direction of analysis considered;

r_x is the square root of the ratio of the torsional stiffness to the lateral stiffness in the y direction ("torsional radius"); and

l_s is the radius of gyration of the floor mass in plan (square root of the ratio of (a) the polar moment of inertia of the floor mass in plan with respect to the centre of mass of the floor to (b) the floor mass).

The definitions of centre of stiffness and torsional radius *r* are provided in **(7)** to **(9)** of this subclause .

(7) In single storey buildings the centre of stiffness is defined as the centre of the lateral stiffness of all primary seismic members. The torsional radius *r* is defined as the square root of the ratio of the global torsional stiffness with respect to the centre of lateral stiffness, and the global lateral stiffness, in one direction, taking into account all of the primary seismic members in this direction.

(8) In multi-storey buildings only approximate definitions of the centre of stiffness and of the torsional radius are possible. A simplified definition, for the classification of structural regularity in plan and for the approximate analysis of torsional effects, is possible if the following two conditions are satisfied:

a) all lateral load resisting systems, such as cores, structural walls, or frames, run without interruption from the foundations to the top of the building;

b) the deflected shapes of the individual systems under horizontal loads are not very different. This condition may be considered satisfied in the case of frame systems and wall systems. In general, this condition is not satisfied in dual systems.

NOTE The National Annex can include reference to documents that might provide definitions of the centre of stiffness and of the torsional radius in multi-storey buildings, both for those that meet the conditions (a) and (b) of paragraph **(8)**, and for those that do not.

(9) In frames and in systems of slender walls with prevailing flexural deformations, the position of the centres of stiffness and the torsional radius of all storeys may be calculated as those of the moments of inertia of the cross-sections of the vertical elements. If, in addition to flexural deformations, shear deformations are also significant, they may be accounted for by using an equivalent moment of inertia of the cross-section.

4.2.3.3 Criteria for regularity in elevation

(1)P For a building to be categorised as being regular in elevation, it shall satisfy all the conditions listed in the following paragraphs.

(2) All lateral load resisting systems, such as cores, structural walls, or frames, shall run without interruption from their foundations to the top of the building or, if setbacks at different heights are present, to the top of the relevant zone of the building.

(3) Both the lateral stiffness and the mass of the individual storeys shall remain constant or reduce gradually, without abrupt changes, from the base to the top of a particular building.

(4) In framed buildings the ratio of the actual storey resistance to the resistance required by the analysis should not vary disproportionately between adjacent storeys. Within this context the special aspects of masonry infilled frames are treated in **4.3.6.3.2**.

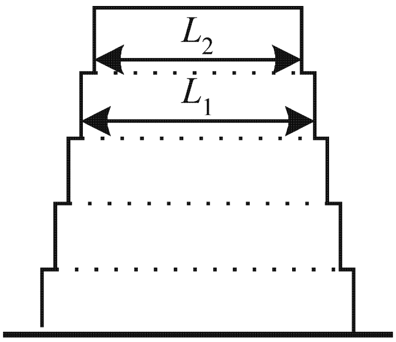
(5) When setbacks are present, the following additional conditions apply:

a) for gradual setbacks preserving axial symmetry, the setback at any floor shall be not greater than 20 % of the previous plan dimension in the direction of the setback (see Figure 4.1.a and Figure 4.1.b);

b) for a single setback within the lower 15 % of the total height of the main structural system, the setback shall be not greater than 50 % of the previous plan dimension (see Figure 4.1.c). In this case the structure of the base zone within the vertically projected perimeter of the upper storeys should be designed to resist at least 75% of the horizontal shear forces that would develop in that zone in a similar building without the base enlargement;

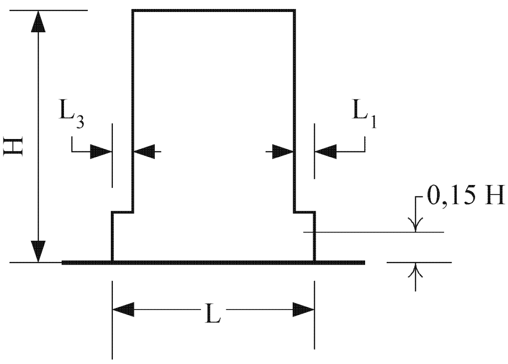
c) if the setbacks do not preserve symmetry, in each face the sum of the setbacks at all storeys shall be not greater than 30 % of the plan dimension at the ground floor above the foundation or above the top of a rigid basement, and the individual setbacks shall be not greater than 10 % of the previous plan dimension (see Figure 4.1.d).

(a)



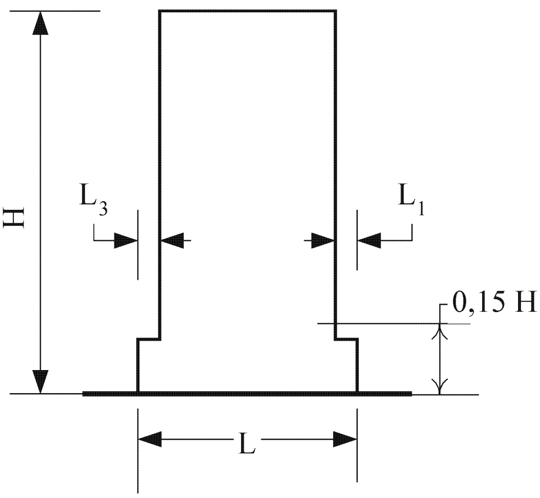
Criterion for (a): $\frac{L_1 - L_2}{L_1} \leq 0{,}20$

(b) (setback occurs above 0,15H)



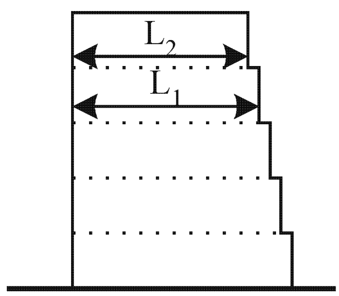
Criterion for (b): $\frac{L_3 + L_1}{L} \leq 0{,}20$

(c) (setback occurs below 0,15H)



Criterion for (c): $\frac{L_3 + L_1}{L} \leq 0{,}50$

d)



Criteria for (d): $\frac{L - L_2}{L} \leq 0{,}30$

$$\frac{L_1 - L_2}{L_1} \leq 0{,}10$$

Figure 4.1: Criteria for regularity of buildings with setbacks

4.2.4 Combination coefficients for variable actions

(1)P The combination coefficients ψ_{2i} (for the quasi-permanent value of variable action q_i) for the design of buildings (see **3.2.4**) shall be those given in EN 1990:2002, Annex A1.

(2)P The combination coefficients ψ_{Ei} introduced in **3.2.4(2)**P for the calculation of the effects of the seismic actions shall be computed from the following expression:

$$\psi_{Ei} = \varphi \cdot \psi_{2i} \tag{4.2}$$

NOTE The values to be ascribed to φ for use in a country may be found in its National Annex. The recommended values for φ are listed in Table 4.2.

Table 4.2: Values of φ for calculating ψ_{Ei}

Type of variable action	Storey	φ
Categories A-C*	Roof Storeys with correlated occupancies Independently occupied storeys	1,0 0,8 0,5
Categories D-F* and Archives		1,0

* Categories as defined in EN 1991-1-1:2002.

4.2.5 Importance classes and importance factors

(1)P Buildings are classified in 4 importance classes, depending on the consequences of collapse for human life, on their importance for public safety and civil protection in the immediate post-earthquake period, and on the social and economic consequences of collapse.

(2)P The importance classes are characterised by different importance factors γ_I as described in **2.1(3)**.

(3) The importance factor γ_I = 1,0 is associated with a seismic event having the reference return period indicated in **3.2.1(3)**.

(4) The definitions of the importance classes are given in Table 4.3.

Table 4.3 Importance classes for buildings

Importance class	Buildings
I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.
II	Ordinary buildings, not belonging in the other categories.
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.

NOTE Importance classes I, II and III or IV correspond roughly to consequences classes CC1, CC2 and CC3, respectively, defined in EN 1990:2002, Annex B.

(5)P The value of γ_I for importance class II shall be, by definition, equal to 1,0.

NOTE The values to be ascribed to γ_I for use in a country may be found in its National Annex. The values of γ_I may be different for the various seismic zones of the country, depending on the seismic hazard conditions and on public safety considerations (see Note to **2.1(4)**). The recommended values of γ_I for importance classes I, III and IV are equal to 0,8, 1,2 and 1,4, respectively.

(6) For buildings which house dangerous installations or materials the importance factor should be established in accordance with the criteria set forth in EN 1998-4.

4.3 Structural analysis

4.3.1 Modelling

(1)P The model of the building shall adequately represent the distribution of stiffness and mass in it so that all significant deformation shapes and inertia forces are properly accounted for under the seismic action considered. In the case of non-linear analysis, the model shall also adequately represent the distribution of strength.

(2) The model should also account for the contribution of joint regions to the deformability of the building, e.g. the end zones in beams or columns of frame type structures. Non-structural elements, which may influence the response of the primary seismic structure, should also be accounted for.

(3) In general the structure may be considered to consist of a number of vertical and lateral load resisting systems, connected by horizontal diaphragms.

(4) When the floor diaphragms of the building may be taken as being rigid in their planes, the masses and the moments of inertia of each floor may be lumped at the centre of gravity.

NOTE The diaphragm is taken as being rigid, if, when it is modelled with its actual in-plane flexibility, its horizontal displacements nowhere exceed those resulting from the rigid diaphragm assumption by more than 10% of the corresponding absolute horizontal displacements in the seismic design situation.

(5) For buildings conforming to the criteria for regularity in plan (see **4.2.3.2**) or with the conditions presented in **4.3.3.1(8)**, the analysis may be performed using two planar models, one for each main direction.

(6) In concrete buildings, in composite steel-concrete buildings and in masonry buildings the stiffness of the load bearing elements should, in general, be evaluated taking into account the effect of cracking. Such stiffness should correspond to the initiation of yielding of the reinforcement.

(7) Unless a more accurate analysis of the cracked elements is performed, the elastic flexural and shear stiffness properties of concrete and masonry elements may be taken to be equal to one-half of the corresponding stiffness of the uncracked elements.

(8) Infill walls which contribute significantly to the lateral stiffness and resistance of the building should be taken into account. See **4.3.6** for masonry infills of concrete, steel or composite frames.

(9)P The deformability of the foundation shall be taken into account in the model, whenever it may have an adverse overall influence on the structural response.

NOTE Foundation deformability (including the soil-structure interaction) may always be taken into account, including the cases in which it has beneficial effects.

(10)P The masses shall be calculated from the gravity loads appearing in the combination of actions indicated in **3.2.4**. The combination coefficients ψ_{Ei} are given in **4.2.4(2)**P.

4.3.2 Accidental torsional effects

(1)P In order to account for uncertainties in the location of masses and in the spatial variation of the seismic motion, the calculated centre of mass at each floor *i* shall be considered as being displaced from its nominal location in each direction by an accidental eccentricity:

$$e_{\mathrm{ai}} = \pm 0{,}05 \cdot L_{\mathrm{i}} \tag{4.3}$$

where

e_{ai} is the accidental eccentricity of storey mass *i* from its nominal location, applied in the same direction at all floors;

L_{i} is the floor-dimension perpendicular to the direction of the seismic action.

4.3.3 Methods of analysis

4.3.3.1 General

(1) Within the scope of Section **4**, the seismic effects and the effects of the other actions included in the seismic design situation may be determined on the basis of the linear-elastic behaviour of the structure.

(2)P The reference method for determining the seismic effects shall be the modal response spectrum analysis, using a linear-elastic model of the structure and the design spectrum given in **3.2.2.5**.

(3) Depending on the structural characteristics of the building one of the following two types of linear-elastic analysis may be used:

a) the “lateral force method of analysis” for buildings meeting the conditions given in **4.3.3.2**;

b) the “modal response spectrum analysis", which is applicable to all types of buildings (see **4.3.3.3**).

(4) As an alternative to a linear method, a non-linear method may also be used, such as:

c) non-linear static (pushover) analysis;

d) non-linear time history (dynamic) analysis,

provided that the conditions specified in **(5)** and **(6)** of this subclause and in **4.3.3.4** are satisfied.

NOTE For base isolated buildings the conditions under which the linear methods a) and b) or the nonlinear ones c) and d), may be used are given in Section **10**. For non-base-isolated buildings, the linear methods of **4.3.3.1(3)** may always be used, as specified in **4.3.3.2.1**. The choice of whether the nonlinear methods of **4.3.3.1(4)** may also be applied to non-base-isolated buildings in a particular country , will be found in its National Annex. The National Annex may also include reference to complementary information about member deformation capacities and the associated partial factors to be used in the Ultimate Limit State verifications in accordance with **4.4.2.2(5)**.

(5) Non-linear analyses should be properly substantiated with respect to the seismic input, the constitutive model used, the method of interpreting the results of the analysis and the requirements to be met.

(6) Non-base-isolated structures designed on the basis of non-linear pushover analysis without using the behaviour factor q (see **4.3.3.4.2.1(1)**d), should satisfy **4.4.2.2(5)**, as well as the rules of Sections **5** to **9** for dissipative structures.

(7) Linear-elastic analysis may be performed using two planar models, one for each main horizontal direction, if the criteria for regularity in plan are satisfied (see **4.2.3.2**).

(8) Depending on the importance class of the building, linear-elastic analysis may be performed using two planar models, one for each main horizontal direction, even if the criteria for regularity in plan in **4.2.3.2** are not satisfied, provided that all of the following special regularity conditions are met:

a) the building shall have well-distributed and relatively rigid cladding and partitions;

b) the building height shall not exceed 10 m;

c) the in-plane stiffness of the floors shall be large enough in comparison with the lateral stiffness of the vertical structural elements, so that a rigid diaphragm behaviour may be assumed.

d) the centres of lateral stiffness and mass shall be each approximately on a vertical line and, in the two horizontal directions of analysis, satisfy the conditions: $r_x^2 > l_s^2 + e_{ox}^2$, $r_y^2 > l_s^2 + e_{oy}^2$, where the radius of gyration l_s, the torsional radii r_x and r_y and the natural eccentricities e_{ox} and e_{oy} are defined as in **4.2.3.2(6)**.

NOTE The value of the importance factor, γ_I, below which the simplification of the analysis in accordance with **4.3.3.1(8)** is allowed in a country, may be found in its National Annex.

(9) In buildings satisfying all the conditions of **(8)** of this subclause with the exception of d), linear-elastic analysis using two planar models, one for each main horizontal direction, may also be performed, but in such cases all seismic action effects resulting from the analysis should be multiplied by 1,25.

(10)P Buildings not conforming to the criteria in **(7)** to **(9)** of this clause shall be analysed using a spatial model.

(11)P Whenever a spatial model is used, the design seismic action shall be applied along all relevant horizontal directions (with regard to the structural layout of the building) and their orthogonal horizontal directions. For buildings with resisting elements in two perpendicular directions these two directions shall be considered as the relevant directions.

4.3.3.2 Lateral force method of analysis

4.3.3.2.1 General

(1)P This type of analysis may be applied to buildings whose response is not significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction.

(2) The requirement in **(1)**P of this subclause is deemed to be satisfied in buildings which fulfil both of the two following conditions.

a) they have fundamental periods of vibration T_1 in the two main directions which are smaller than the following values

$$T_1 \leq \begin{cases} 4 \cdot T_C \\ 2{,}0 \text{ s} \end{cases} \tag{4.4}$$

where T_C is given in Table 3.2 or Table 3.3;

b) they meet the criteria for regularity in elevation given in **4.2.3.3**.

4.3.3.2.2 Base shear force

(1)P The seismic base shear force F_b, for each horizontal direction in which the building is analysed, shall be determined using the following expression:

$$F_b = S_d(T_1) \cdot m \cdot \lambda \tag{4.5}$$

where

$S_d(T_1)$ is the ordinate of the design spectrum (see **3.2.2.5**) at period T_1;

T_1 is the fundamental period of vibration of the building for lateral motion in the direction considered;

m is the total mass of the building, above the foundation or above the top of a rigid basement, computed in accordance with **3.2.4(2)**;

λ is the correction factor, the value of which is equal to: λ = 0,85 if $T_1 \leq 2\ T_C$ and the building has more than two storeys, or λ = 1,0 otherwise.

NOTE The factor λ accounts for the fact that in buildings with at least three storeys and translational degrees of freedom in each horizontal direction, the effective modal mass of the 1st (fundamental) mode is smaller, on average by 15%, than the total building mass.

(2) For the determination of the fundamental period of vibration period T_1 of the building, expressions based on methods of structural dynamics (for example the Rayleigh method) may be used.

(3) For buildings with heights of up to 40 m the value of T_1 (in s) may be approximated by the following expression:

$$T_1 = C_t \cdot H^{3/4} \tag{4.6}$$

where

C_t is 0,085 for moment resistant space steel frames, 0,075 for moment resistant space concrete frames and for eccentrically braced steel frames and 0,050 for all other structures;

H is the height of the building, in m, from the foundation or from the top of a rigid basement.

(4) Alternatively, for structures with concrete or masonry shear walls the value C_t in expression (4.6) may be taken as being

$$C_t = 0{,}075 / \sqrt{A_c} \tag{4.7}$$

where

$$A_c = \Sigma \left[A_i \cdot \left(0{,}2 + \left(l_{wi} / H\right)\right)^2\right] \tag{4.8}$$

and

A_c is the total effective area of the shear walls in the first storey of the building, in m^2;

A_i is the effective cross-sectional area of the shear wall i in the first storey of the building, in m^2;

H is as in **(3)** of this subclause;

l_{wi} is the length of the shear wall i in the first storey in the direction parallel to the applied forces, in m, with the restriction that l_{wi}/H should not exceed 0,9.

(5) Alternatively, the estimation of T_1 (in s) may be made by using the following expression:

$$T_1 = 2 \cdot \sqrt{d} \tag{4.9}$$

where

d is the lateral elastic displacement of the top of the building, in m, due to the gravity loads applied in the horizontal direction.

4.3.3.2.3 Distribution of the horizontal seismic forces

(1) The fundamental mode shapes in the horizontal directions of analysis of the building may be calculated using methods of structural dynamics or may be approximated by horizontal displacements increasing linearly along the height of the building.

(2)P The seismic action effects shall be determined by applying, to the two planar models, horizontal forces F_i to all storeys.

$$F_i = F_b \cdot \frac{s_i \cdot m_i}{\Sigma\, s_j \cdot m_j} \tag{4.10}$$

where

F_i is the horizontal force acting on storey i;

F_b is the seismic base shear in accordance with expression (4.5);

s_i, s_j are the displacements of masses m_i, m_j in the fundamental mode shape;

m_i, m_j are the storey masses computed in accordance with **3.2.4(2)**.

(3) When the fundamental mode shape is approximated by horizontal displacements increasing linearly along the height, the horizontal forces F_i should be taken as being given by:

$$F_i = F_b \cdot \frac{z_i \cdot m_i}{\Sigma\, z_j \cdot m_j} \tag{4.11}$$

where

z_i, z_j are the heights of the masses m_i m_j above the level of application of the seismic action (foundation or top of a rigid basement).

(4)P The horizontal forces F_i determined in accordance with this clause shall be distributed to the lateral load resisting system assuming the floors are rigid in their plane.

4.3.3.2.4 Torsional effects

(1) If the lateral stiffness and mass are symmetrically distributed in plan and unless the accidental eccentricity of **4.3.2(1)**P is taken into account by a more exact method (e.g. that of **4.3.3.3.3(1)**), the accidental torsional effects may be accounted for by multiplying the action effects in the individual load resisting elements resulting from the application of **4.3.3.2.3(4)** by a factor δ given by

$$\delta = 1 + 0{,}6 \cdot \frac{x}{L_e} \qquad (4.12)$$

where

x is the distance of the element under consideration from the centre of mass of the building in plan, measured perpendicularly to the direction of the seismic action considered;

L_e is the distance between the two outermost lateral load resisting elements, measured perpendicularly to the direction of the seismic action considered.

(2) If the analysis is performed using two planar models, one for each main horizontal direction, torsional effects may be determined by doubling the accidental eccentricity e_{ai} of expression (4.3) and applying **(1)** of this subclause with factor 0,6 in expression (4.12) increased to 1,2.

4.3.3.3 Modal response spectrum analysis

4.3.3.3.1 General

(1)P This type of analysis shall be applied to buildings which do not satisfy the conditions given in **4.3.3.2.1(2)** for applying the lateral force method of analysis.

(2)P The response of all modes of vibration contributing significantly to the global response shall be taken into account.

(3) The requirements specified in paragraph **(2)**P may be deemed to be satisfied if either of the following can be demonstrated:

- the sum of the effective modal masses for the modes taken into account amounts to at least 90% of the total mass of the structure;
- all modes with effective modal masses greater than 5% of the total mass are taken into account.

NOTE The effective modal mass m_k, corresponding to a mode k, is determined so that the base shear force F_{bk}, acting in the direction of application of the seismic action, may be expressed as $F_{bk} = S_d(T_k)\, m_k$. It can be shown that the sum of the effective modal masses (for all modes and a given direction) is equal to the mass of the structure.

(4) When using a spatial model, the above conditions should be verified for each relevant direction.

(5) If the requirements specified in **(3)** cannot be satisfied (e.g. in buildings with a significant contribution from torsional modes), the minimum number k of modes to be taken into account in a spatial analysis should satisfy both the two following conditions:

$$k \geq 3 \cdot \sqrt{n} \tag{4.14a}$$

and

$$T_{\mathrm{k}} \leq 0{,}20\ \mathrm{s} \tag{4.14b}$$

where

k is the number of modes taken into account;

n is the number of storeys above the foundation or the top of a rigid basement;

T_{k} is the period of vibration of mode k.

4.3.3.3.2 Combination of modal responses

(1) The response in two vibration modes i and j (including both translational and torsional modes) may be taken as independent of each other, if their periods T_{i} and T_{j} satisfy (with $T_{\mathrm{j}} \leq T_{\mathrm{i}}$) the following condition:

$$T_{\mathrm{j}} \leq 0{,}9 \cdot T_{\mathrm{i}} \tag{4.15}$$

(2) Whenever all relevant modal responses (see **4.3.3.3.1(3)-(5)**) may be regarded as independent of each other, the maximum value E_{E} of a seismic action effect may be taken as:

$$E_{\mathrm{E}} = \sqrt{\Sigma\ E_{\mathrm{Ei}}{}^{2}} \tag{4.16}$$

where

E_{E} is the seismic action effect under consideration (force, displacement, etc.);

E_{Ei} is the value of this seismic action effect due to the vibration mode i.

(3)P If **(1)** is not satisfied, more accurate procedures for the combination of the modal maxima, such as the "Complete Quadratic Combination" shall be adopted.

4.3.3.3.3 Torsional effects

(1) Whenever a spatial model is used for the analysis, the accidental torsional effects referred to in **4.3.2(1)**P may be determined as the envelope of the effects resulting from the application of static loadings, consisting of sets of torsional moments M_{ai} about the vertical axis of each storey i:

$$M_{\mathrm{ai}} = e_{\mathrm{ai}} \cdot F_{\mathrm{i}} \tag{4.17}$$

where

M_{ai} is the torsional moment applied at storey i about its vertical axis;

e_{ai} is the accidental eccentricity of storey mass i in accordance with expression (4.3) for all relevant directions;

F_i is the horizontal force acting on storey i, as derived in **4.3.3.2.3** for all relevant directions.

(2) The effects of the loadings in accordance with **(1)** should be taken into account with positive and negative signs (the same sign for all storeys).

(3) Whenever two separate planar models are used for the analysis, the torsional effects may be accounted for by applying the rules of **4.3.3.2.4(2)** to the action effects computed in accordance with **4.3.3.3.2**.

4.3.3.4 Non-linear methods

4.3.3.4.1 General

(1)P The mathematical model used for elastic analysis shall be extended to include the strength of structural elements and their post-elastic behaviour.

(2) As a minimum, a bilinear force–deformation relationship should be used at the element level. In reinforced concrete and masonry buildings, the elastic stiffness of a bilinear force-deformation relation should correspond to that of cracked sections (see **4.3.1(7)**). In ductile elements, expected to exhibit post-yield excursions during the response, the elastic stiffness of a bilinear relation should be the secant stiffness to the yield-point. Trilinear force–deformation relationships, which take into account pre-crack and post-crack stiffnesses, are allowed.

(3) Zero post-yield stiffness may be assumed. If strength degradation is expected, e.g. for masonry walls or other brittle elements, it has to be included in the force–deformation relationships of those elements.

(4) Unless otherwise specified, element properties should be based on mean values of the properties of the materials. For new structures, mean values of material properties may be estimated from the corresponding characteristic values on the basis of information provided in EN 1992 to EN 1996 or in material ENs.

(5)P Gravity loads in accordance with **3.2.4** shall be applied to appropriate elements of the mathematical model.

(6) Axial forces due to gravity loads should be taken into account when determining force – deformation relations for structural elements. Bending moments in vertical structural elements due to gravity loads may be neglected, unless they substantially influence the global structural behaviour.

(7)P The seismic action shall be applied in both positive and negative directions and the maximum seismic effects as a result of this shall be used.

4.3.3.4.2 Non-linear static (pushover) analysis

4.3.3.4.2.1 General

(1) Pushover analysis is a non-linear static analysis carried out under conditions of constant gravity loads and monotonically increasing horizontal loads. It may be applied to verify the structural performance of newly designed and of existing buildings for the following purposes:

a) to verify or revise the overstrength ratio values α_u/α_1 (see **5.2.2.2**, **6.3.2**, **7.3.2**);

b) to estimate the expected plastic mechanisms and the distribution of damage;

c) to assess the structural performance of existing or retrofitted buildings for the purposes of EN 1998-3;

d) as an alternative to the design based on linear-elastic analysis which uses the behaviour factor *q*. In that case, the target displacement indicated in **4.3.3.4.2.6(1)**P should be used as the basis of the design.

(2)P Buildings not conforming to the regularity criteria of **4.2.3.2** or the criteria of **4.3.3.1(8)**a)-e) shall be analysed using a spatial model. Two independent analyses with lateral loads applied in one direction only may be performed.

(3) For buildings conforming to the regularity criteria of **4.2.3.2** or the criteria of **4.3.3.1(8)**a)-d) the analysis may be performed using two planar models, one for each main horizontal direction.

(4) For low-rise masonry buildings, in which structural wall behaviour is dominated by shear, each storey may be analysed independently.

(5) The requirements in **(4)** are deemed to be satisfied if the number of storeys is 3 or less and if the average aspect (height to width) ratio of structural walls is less than 1,0.

4.3.3.4.2.2 Lateral loads

(1) At least two vertical distributions of the lateral loads should be applied:

- a "uniform" pattern, based on lateral forces that are proportional to mass regardless of elevation (uniform response acceleration);
- a "modal" pattern, proportional to lateral forces consistent with the lateral force distribution in the direction under consideration determined in elastic analysis (in accordance with **4.3.3.2** or **4.3.3.3**).

(2)P Lateral loads shall be applied at the location of the masses in the model. Accidental eccentricity in accordance with **4.3.2(1)**P shall be taken into account.

4.3.3.4.2.3 Capacity curve

(1) The relation between base shear force and the control displacement (the "capacity curve") should be determined by pushover analysis for values of the control

displacement ranging between zero and the value corresponding to 150% of the target displacement, defined in **4.3.3.4.2.6**.

(2) The control displacement may be taken at the centre of mass of the roof of the building. The top of a penthouse should not be considered as the roof.

4.3.3.4.2.4 Overstrength factor

(1) When the overstrength ratio (α_u/α_1) is determined by pushover analysis, the lower value of the overstrength factor obtained for the two lateral load distributions should be used.

4.3.3.4.2.5 Plastic mechanism

(1)P The plastic mechanism shall be determined for the two lateral load distributions applied. The plastic mechanisms shall conform to the mechanisms on which the behaviour factor q used in the design is based.

4.3.3.4.2.6 Target displacement

(1)P The target displacement shall be defined as the seismic demand derived from the elastic response spectrum of **3.2.2.2** in terms of the displacement of an equivalent single-degree-of-freedom system.

NOTE Informative Annex B gives a procedure for the determination of the target displacement from the elastic response spectrum.

4.3.3.4.2.7 Procedure for the estimation of the torsional effects

(1)P Pushover analysis performed with the force patterns specified in **4.3.3.4.2.2** may significantly underestimate deformations at the stiff/strong side of a torsionally flexible structure, i.e. a structure with a predominantly torsional first mode of vibration. The same applies for the stiff/strong side deformations in one direction of a structure with a predominately torsional second mode of vibration. For such structures, displacements at the stiff/strong side shall be increased, compared to those in the corresponding torsionally balanced structure.

NOTE The stiff/strong side in plan is the one that develops smaller horizontal displacements than the opposite side, under static lateral forces parallel to it. For torsionally flexible structures, the dynamic displacements at the stiff/strong side may considerably increase due to the influence of the predominantly torsional mode.

(2) The requirement specified in **(1)** of this subclause is deemed to be satisfied if the amplification factor to be applied to the displacements of the stiff/strong side is based on the results of an elastic modal analysis of the spatial model.

(3) If two planar models are used for analysis of structures which are regular in plan, the torsional effects may be estimated in accordance with **4.3.3.2.4** or **4.3.3.3.3**.

4.3.3.4.3 Non-linear time-history analysis

(1) The time-dependent response of the structure may be obtained through direct numerical integration of its differential equations of motion, using the accelerograms defined in **3.2.3.1** to represent the ground motions.

(2) The structural element models should conform to **4.3.3.4.1(2)**-**(4)** and be supplemented with rules describing the element behaviour under post-elastic unloading-reloading cycles. These rules should realistically reflect the energy dissipation in the element over the range of displacement amplitudes expected in the seismic design situation.

(3) If the response is obtained from at least 7 nonlinear time-history analyses with ground motions in accordance with **3.2.3.1**, the average of the response quantities from all of these analyses should be used as the design value of the action effect E_d in the relevant verifications of **4.4.2.2**. Otherwise, the most unfavourable value of the response quantity among the analyses should be used as E_d.

4.3.3.5 Combination of the effects of the components of the seismic action

4.3.3.5.1 Horizontal components of the seismic action

(1)P In general the horizontal components of the seismic action (see **3.2.2.1(3)**) shall be taken as acting simultaneously.

(2) The combination of the horizontal components of the seismic action may be accounted for as follows.

a) The structural response to each component shall be evaluated separately, using the combination rules for modal responses given in **4.3.3.3.2**.

b) The maximum value of each action effect on the structure due to the two horizontal components of the seismic action may then be estimated by the square root of the sum of the squared values of the action effect due to each horizontal component.

c) The rule b) generally gives a safe side estimate of the probable values of other action effects simultaneous with the maximum value obtained as in b). More accurate models may be used for the estimation of the probable simultaneous values of more than one action effect due to the two horizontal components of the seismic action.

(3) As an alternative to b) and c) of **(2)** of this subclause, the action effects due to the combination of the horizontal components of the seismic action may be computed using both of the two following combinations:

a) E_{Edx} "+" $0{,}30E_{Edy}$ (4.18)

b) $0{,}30E_{Edx}$ "+" E_{Edy} (4.19)

where

"+" implies "to be combined with";

E_{Edx} represents the action effects due to the application of the seismic action along the chosen horizontal axis x of the structure;

E_{Edy} represents the action effects due to the application of the same seismic action along the orthogonal horizontal axis y of the structure.

(4) If the structural system or the regularity classification of the building in elevation is different in different horizontal directions, the value of the behaviour factor q may also be different.

(5)P The sign of each component in the above combinations shall be taken as being the most unfavourable for the particular action effect under consideration.

(6) When using non-linear static (pushover) analysis and applying a spatial model, the combination rules of **(2)** and **(3)** in this subclause should be applied, considering the forces and deformations due to the application of the target displacement in the x direction as E_{Edx} and the forces and deformations due to the application of the target displacement in the y direction as E_{Edy}. The internal forces resulting from the combination should not exceed the corresponding capacities.

(7)P When using non-linear time-history analysis and employing a spatial model of the structure, simultaneously acting accelerograms shall be taken as acting in both horizontal directions.

(8) For buildings satisfying the regularity criteria in plan and in which walls or independent bracing systems in the two main horizontal directions are the only primary seismic elements (see **4.2.2**), the seismic action may be assumed to act separately and without combinations **(2)** and **(3)** of this subclause, along the two main orthogonal horizontal axes of the structure.

4.3.3.5.2 Vertical component of the seismic action

(1) If a_{vg} is greater than 0,25 g (2,5 m/s^2) the vertical component of the seismic action, as defined in **3.2.2.3**, should be taken into account in the cases listed below:

- for horizontal or nearly horizontal structural members spanning 20 m or more;
- for horizontal or nearly horizontal cantilever components longer than 5 m;
- for horizontal or nearly horizontal pre-stressed components;
- for beams supporting columns;
- in base-isolated structures.

(2) The analysis for determining the effects of the vertical component of the seismic action may be based on a partial model of the structure, which includes the elements on which the vertical component is considered to act (e.g. those listed in the previous paragraph) and takes into account the stiffness of the adjacent elements.

(3) The effects of the vertical component need be taken into account only for the elements under consideration (e.g. those listed in **(1)** of this subclause) and their directly associated supporting elements or substructures.

(4) If the horizontal components of the seismic action are also relevant for these elements, the rules in **4.3.3.5.1(2)** may be applied, extended to three components of the seismic action. Alternatively, all three of the following combinations may be used for the computation of the action effects:

a) E_{Edx} "+" 0,30 E_{Edy} "+" 0,30 E_{Edz} (4.20)

b) 0,30 E_{Edx} "+" E_{Edy} "+" 0,30 E_{Edz} (4.21)

c) 0,30 E_{Edx} "+" 0,30 E_{Edy} "+" E_{Edz} (4.22)

where

"+" implies "to be combined with";

E_{Edx} and E_{Edy} are as in **4.3.3.5.1(3)**;

E_{Edz} represents the action effects due to the application of the vertical component of the design seismic action as defined in **3.2.2.5(5)** and **(6)**.

(5) If non-linear static (pushover) analysis is performed, the vertical component of the seismic action may be neglected.

4.3.4 Displacement analysis

(1)P If linear analysis is performed the displacements induced by the design seismic action shall be calculated on the basis of the elastic deformations of the structural system by means of the following simplified expression:

$$d_s = q_d \, d_e \tag{4.23}$$

where

d_s is the displacement of a point of the structural system induced by the design seismic action;

q_d is the displacement behaviour factor, assumed equal to q unless otherwise specified;

d_e is the displacement of the same point of the structural system, as determined by a linear analysis based on the design response spectrum in accordance with **3.2.2.5.**

The value of d_s does not need to be larger than the value derived from the elastic spectrum.

NOTE In general q_d is larger than q if the fundamental period of the structure is less than T_C (see Figure B.2).

(2)P When determining the displacements d_e, the torsional effects of the seismic action shall be taken into account.

(3) For both static and dynamic non-linear analysis, the displacements determined are those obtained directly from the analysis without further modification.

4.3.5 Non-structural elements

4.3.5.1 General

(1)P Non-structural elements (appendages) of buildings (e.g. parapets, gables, antennae, mechanical appendages and equipment, curtain walls, partitions, railings) that might, in case of failure, cause risks to persons or affect the main structure of the building or services of critical facilities, shall, together with their supports, be verified to resist the design seismic action.

(2)P For non-structural elements of great importance or of a particularly dangerous nature, the seismic analysis shall be based on a realistic model of the relevant structures and on the use of appropriate response spectra derived from the response of the supporting structural elements of the main seismic resisting system.

(3) In all other cases properly justified simplifications of this procedure (e.g. as given in **4.3.5.2(2)**) are allowed.

4.3.5.2 Verification

(1)P The non-structural elements, as well as their connections and attachments or anchorages, shall be verified for the seismic design situation (see **3.2.4**).

NOTE The local transmission of actions to the structure by the fastening of non-structural elements and their influence on the structural behaviour should be taken into account. The requirements for fastenings to concrete are given in EN1992-1-1:2004, **2.7**.

(2) The effects of the seismic action may be determined by applying to the non-structural element a horizontal force F_a which is defined as follows:

$$F_a = \left(S_a \cdot W_a \cdot \gamma_a\right) / q_a \tag{4.24}$$

where

F_a is the horizontal seismic force, acting at the centre of mass of the non-structural element in the most unfavourable direction;

W_a is the weight of the element;

S_a is the seismic coefficient applicable to non-structural elements, (see **(3)** of this subclause);

γ_a is the importance factor of the element, see **4.3.5.3**;

q_a is the behaviour factor of the element, see Table 4.4.

(3) The seismic coefficient S_a may be calculated using the following expression:

$$S_a = \alpha \cdot S \cdot [3(1 + z/H) / (1 + (1 - T_a/T_1)^2)\text{-}0{,}5] \tag{4.25}$$

where

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

S is the soil factor;

T_a is the fundamental vibration period of the non-structural element;

T_1 is the fundamental vibration period of the building in the relevant direction;

z is the height of the non-structural element above the level of application of the seismic action; and

H is the building height measured from the foundation or from the top of a rigid basement.

The value of the seismic coefficient S_a may not be taken less than $\alpha \cdot S$.

4.3.5.3 Importance factors

(1)P For the following non-structural elements the importance factor γ_a shall not be less than 1,5:

- anchorage elements of machinery and equipment required for life safety systems;
- tanks and vessels containing toxic or explosive substances considered to be hazardous to the safety of the general public.

(2) In all other cases the importance factor γ_a of non-structural elements may be assumed to be $\gamma_a = 1,0$.

4.3.5.4 Behaviour factors

(1) Upper limit values of the behaviour factor q_a for non-structural elements are given in Table 4.4.

Table 4.4: Values of q_a for non-structural elements

Type of non-structural element	q_a
Cantilevering parapets or ornamentations Signs and billboards Chimneys, masts and tanks on legs acting as unbraced cantilevers along more than one half of their total height	1,0
Exterior and interior walls Partitions and facades Chimneys, masts and tanks on legs acting as unbraced cantilevers along less than one half of their total height, or braced or guyed to the structure at or above their centre of mass Anchorage elements for permanent cabinets and book stacks supported by the floor Anchorage elements for false (suspended) ceilings and light fixtures	2,0

4.3.6 Additional measures for masonry infilled frames

4.3.6.1 General

(1)P **4.3.6.1** to **4.3.6.3** apply to frame or frame equivalent dual concrete systems of DCH (see Section **5**) and to steel or steel-concrete composite moment resisting frames of DCH (see Sections **6** and **7**) with interacting non-engineered masonry infills that fulfil all of the following conditions:

a) they are constructed after the hardening of the concrete frames or the assembly of the steel frame;

b) they are in contact with the frame (i.e. without special separation joints), but without structural connection to it (through ties, belts, posts or shear connectors);

c) they are considered in principle as non-structural elements.

(2) Although the scope of **4.3.6.1** to **4.3.6.3** is limited in accordance with **(1)**P of this subclause, these subclauses provide criteria for good practice, which it may be advantageous to adopt for DCM or DCL concrete, steel or composite structures with masonry infills. In particular for panels that might be vulnerable to out-of-plane failure, the provision of ties can reduce the hazard of falling masonry.

(3)P The provisions in **1.3(2)** regarding possible future modification of the structure shall apply also to the infills.

(4) For wall or wall-equivalent dual concrete systems, as well as for braced steel or steel-concrete composite systems, the interaction with the masonry infills may be neglected.

(5) If engineered masonry infills constitute part of the seismic resistant structural system, analysis and design should be carried out in accordance with the criteria and rules given in Clause **9** for confined masonry.

(6) The requirements and criteria given in **4.3.6.2** are deemed to be satisfied if the rules given in **4.3.6.3** and **4.3.6.4** and the special rules in Sections **5** to **7** are followed.

4.3.6.2 Requirements and criteria

(1)P The consequences of irregularity in plan produced by the infills shall be taken into account.

(2)P The consequences of irregularity in elevation produced by the infills shall be taken into account.

(3)P Account shall be taken of the high uncertainties related to the behaviour of the infills (namely, the variability of their mechanical properties and of their attachment to the surrounding frame, their possible modification during the use of the building, as well as their non-uniform degree of damage suffered during the earthquake itself).

(4)P The possibly adverse local effects due to the frame-infill-interaction (e.g. shear failure of columns under shear forces induced by the diagonal strut action of infills) shall be taken into account (see Sections **5** to **7**).

4.3.6.3 Irregularities due to masonry infills

4.3.6.3.1 Irregularities in plan

(1) Strongly irregular, unsymmetrical or non-uniform arrangements of infills in plan should be avoided (taking into account the extent of openings and perforations in infill panels).

(2) In the case of severe irregularities in plan due to the unsymmetrical arrangement of the infills (e.g. existence of infills mainly along two consecutive faces of the building), spatial models should be used for the analysis of the structure. Infills should be included in the model and a sensitivity analysis regarding the position and the properties of the infills should be performed (e.g. by disregarding one out of three or four infill panels in a planar frame, especially on the more flexible sides). Special attention should be paid to the verification of structural elements on the flexible sides of the plan (i.e. furthest away from the side where the infills are concentrated) against the effects of any torsional response caused by the infills.

(3) Infill panels with more than one significant opening or perforation (e.g. a door and a window, etc.) should be disregarded in models for analyses in accordance with **(2)** of this subclause.

(4) When the masonry infills are not regularly distributed, but not in such a way as to constitute a severe irregularity in plan, these irregularities may be taken into account by increasing by a factor of 2,0 the effects of the accidental eccentricity calculated in accordance with **4.3.3.2.4** and **4.3.3.3.3**.

4.3.6.3.2 Irregularities in elevation

(1)P If there are considerable irregularities in elevation (e.g. drastic reduction of infills in one or more storeys compared to the others), the seismic action effects in the vertical elements of the respective storeys shall be increased.

(2) If a more precise model is not used, **(1)**P is deemed to be satisfied if the calculated seismic action effects are amplified by a magnification factor η defined as follows:

$$\eta = \left(1 + \Delta V_{\mathrm{Rw}} / \Sigma V_{\mathrm{Ed}}\right) \leq q \qquad (4.26)$$

where

ΔV_{Rw} is the total reduction of the resistance of masonry walls in the storey concerned, compared to the more infilled storey above it; and

ΣV_{Ed} is the sum of the seismic shear forces acting on all vertical primary seismic members of the storey concerned.

(3) If expression (4.26) leads to a magnification factor η lower than 1,1, there is no need for modification of action effects.

4.3.6.4 Damage limitation of infills

(1) For the structural systems quoted in **4.3.6.1(1)**P belonging to all ductility classes, DCL, M or H, except in cases of low seismicity (see **3.2.1(4)**), appropriate measures should be taken to avoid brittle failure and premature disintegration of the infill walls (in particular of masonry panels with openings or of friable materials), as well as the partial or total out-of-plane collapse of slender masonry panels. Particular attention should be paid to masonry panels with a slenderness ratio (ratio of the smaller of length or height to thickness) of greater than 15.

(2) Examples of measures in accordance with **(1)** of this subclause, to improve both in-plane and out-of-plane integrity and behaviour, include light wire meshes well anchored on one face of the wall, wall ties fixed to the columns and cast into the bedding planes of the masonry, and concrete posts and belts across the panels and through the full thickness of the wall.

(3) If there are large openings or perforations in any of the infill panels, their edges should be trimmed with belts and posts.

4.4 Safety verifications

4.4.1 General

(1)P For the safety verifications the relevant limit states (see **4.4.2** and **4.4.3** below) and specific measures (see **2.2.4**) shall be considered.

(2) For buildings of importance classes other than IV (see Table 4.3) the verifications prescribed in **4.4.2** and **4.4.3** may be considered satisfied if both of the following two conditions are met.

a) The total base shear due to the seismic design situation calculated with a behaviour factor equal to the value applicable to low-dissipative structures (see **2.2.2(2)**)is less than that due to the other relevant action combinations for which the building is designed on the basis of a linear elastic analysis. This requirement relates to the shear force over the entire structure at the base level of the building (foundation or top of a rigid basement).

b) The specific measures described in **2.2.4** are taken into account, with the exception of the provisions in **2.2.4.1(2)-(3)**.

4.4.2 Ultimate limit state

4.4.2.1 General

(1)P The no-collapse requirement (ultimate limit state) under the seismic design situation is considered to have been met if the following conditions regarding resistance, ductility, equilibrium, foundation stability and seismic joints are met.

4.4.2.2 Resistance condition

(1)P The following relation shall be satisfied for all structural elements including connections and the relevant non-structural elements:

$$E_d \leq R_d \tag{4.27}$$

where

E_d is the design value of the action effect, due to the seismic design situation (see EN 1990:2002 **6.4.3.4**), including, if necessary, second order effects (see **(2)** of this subclause). Redistribution of bending moments in accordance with EN 1992-1-1:2004, EN 1993-1:2004 and EN 1994-1-1:2004 is permitted;

R_d is the corresponding design resistance of the element, calculated in accordance with the rules specific to the material used (in terms of the characteristic values of material properties f_k and partial factor γ_M) and in accordance with the mechanical models which relate to the specific type of structural system, as given in Sections **5** to **9** of this document and in other relevant Eurocode documents.

(2) Second-order effects (P-Δ effects) need not be taken into account if the following condition is fulfilled in all storeys:

$$\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h} \leq 0{,}10 \qquad (4.28)$$

where

θ is the interstorey drift sensitivity coefficient;

P_{tot} is the total gravity load at and above the storey considered in the seismic design situation;

d_r is the design interstorey drift, evaluated as the difference of the average lateral displacements d_s at the top and bottom of the storey under consideration and calculated in accordance with **4.3.4**;

V_{tot} is the total seismic storey shear; and

h is the interstorey height.

(3) If $0{,}1 < \theta \leq 0{,}2$, the second-order effects may approximately be taken into account by multiplying the relevant seismic action effects by a factor equal to $1/(1 - \theta)$.

(4)P The value of the coefficient θ shall not exceed 0,3.

(5) If design action effects E_d are obtained through a nonlinear method of analysis (see **4.3.3.4**), **(1)**P of this subclause should be applied in terms of forces only for brittle elements. For dissipative zones, which are designed and detailed for ductility, the resistance condition, expression (4.27), should be satisfied in terms of member deformations (e.g. plastic hinge or chord rotations), with appropriate material partial factors applied on member deformation capacities (see also EN 1992-1-1:2004, **5.7(2)**; **5.7(4)**P).

(6) Fatigue resistance does not need to be verified under the seismic design situation.

4.4.2.3 Global and local ductility condition

(1)P It shall be verified that both the structural elements and the structure as a whole possess adequate ductility, taking into account the expected exploitation of ductility, which depends on the selected system and the behaviour factor.

(2)P Specific material related requirements, as defined in Sections **5** to **9**, shall be satisfied, including, when indicated, capacity design provisions in order to obtain the hierarchy of resistance of the various structural components necessary for ensuring the intended configuration of plastic hinges and for avoiding brittle failure modes.

(3)P In multi-storey buildings formation of a soft storey plastic mechanism shall be prevented, as such a mechanism might entail excessive local ductility demands in the columns of the soft storey.

(4) Unless otherwise specified in Sections **5** to **8**, to satisfy the requirement of **(3)**P, in frame buildings, including frame-equivalent ones as defined in **5.1.2(1)**, with two or more storeys, the following condition should be satisfied at all joints of primary or secondary seismic beams with primary seismic columns:

$$\sum M_{\mathrm{Rc}} \geq 1{,}3 \sum M_{\mathrm{Rb}} \tag{4.29}$$

where

$\sum M_{\mathrm{Rc}}$ is the sum of the design values of the moments of resistance of the columns framing the joint. The minimum value of column moments of resistance within the range of column axial forces produced by the seismic design situation should be used in expression (4.29); and

$\sum M_{\mathrm{Rb}}$ is the sum of the design values of the moments of resistance of the beams framing the joint. When partial strength connections are used, the moments of resistance of these connections are taken into account in the calculation of $\sum M_{\mathrm{Rb}}$.

NOTE A rigorous interpretation of expression (4.29) requires calculation of the moments at the centre of the joint. These moments correspond to development of the design values of the moments of resistance of the columns or beams at the outside faces of the joint, plus a suitable allowance for moments due to shears at the joint faces. However, the loss in accuracy is minor and the simplification achieved is considerable if the shear allowance is neglected. This approximation is then deemed to be acceptable.

(5) Expression (4.29) should be satisfied in two orthogonal vertical planes of bending, which, in buildings with frames arranged in two orthogonal directions, are defined by these two directions. It should be satisfied for both directions (positive and negative) of action of the beam moments around the joint, with the column moments always opposing the beam moments. If the structural system is a frame or equivalent to a frame in only one of the two main horizontal directions of the structural system, then expression (4.29) should be satisfied just within the vertical plane through that direction.

(6) The rules of **(4)** and **(5)** of this subclause are waived at the top level of multi-storey buildings.

(7) Capacity design rules to avoid brittle failure modes are given in Sections **5** to **7**.

(8) The requirements of **(1)**P and **(2)**P of this subclause are deemed to be satisfied if all of the following conditions are satisfied:

a) plastic mechanisms obtained by pushover analysis are satisfactory;

b) global, interstorey and local ductility and deformation demands from pushover analyses (with different lateral load patterns) do not exceed the corresponding capacities;

c) brittle elements remain in the elastic region.

4.4.2.4 Equilibrium condition

(1)P The building structure shall be stable - including overturning or sliding - in the seismic design situation specified in EN 1990:2002 **6.4.3.4**.

(2) In special cases the equilibrium may be verified by means of energy balance methods, or by geometrically non-linear methods with the seismic action defined as described in **3.2.3.1**.

4.4.2.5 Resistance of horizontal diaphragms

(1)P Diaphragms and bracings in horizontal planes shall be able to transmit, with sufficient overstrength, the effects of the design seismic action to the lateral load-resisting systems to which they are connected.

(2) The requirement in **(1)**P of this subclause is considered to be satisfied if for the relevant resistance verifications the seismic action effects in the diaphragm obtained from the analysis are multiplied by an overstrength factor γ_d greater than 1,0.

NOTE The values to be ascribed to γ_d for use in a country may be found in its National Annex. The recommended value for brittle failure modes, such as in shear in concrete diaphragms is 1.3, and for ductile failure modes is 1,1.

(3) Design provisions for concrete diaphragms are given in **5.10**.

4.4.2.6 Resistance of foundations

(1)P The foundation system shall conform to EN 1998-5:2004, Section **5** and to EN 1997-1:2004.

(2)P The action effects for the foundation elements shall be derived on the basis of capacity design considerations accounting for the development of possible overstrength, but they need not exceed the action effects corresponding to the response of the structure under the seismic design situation inherent to the assumption of an elastic behaviour (q = 1,0).

(3) If the action effects for the foundation have been determined using the value of the behaviour factor q applicable to low-dissipative structures (see **2.2.2(2)**), no capacity design considerations in accordance with **(2)**P are required.

(4) For foundations of individual vertical elements (walls or columns), **(2)**P of this subclause is considered to be satisfied if the design values of the action effects E_{Fd} on the foundations are derived as follows:

$$E_{Fd} = E_{F,G} + \gamma_{Rd} \Omega E_{F,E} \tag{4.30}$$

where

γ_{Rd} is the overstrength factor, taken as being equal to 1,0 for $q \leq 3$, or as being equal to 1,2 otherwise;

$E_{F,G}$ is the action effect due to the non-seismic actions included in the combination of actions for the seismic design situation (see EN 1990:2002, **6.4.3.4**);

$E_{F,E}$ is the action effect from the analysis of the design seismic action; and

Ω is the value of $(R_{di}/E_{di}) \leq q$ of the dissipative zone or element i of the structure which has the highest influence on the effect E_F under consideration; where

R_{di} is the design resistance of the zone or element i; and

E_{di} is the design value of the action effect on the zone or element i in the seismic design situation.

(5) For foundations of structural walls or of columns of moment-resisting frames, Ω is the minimum value of the ratio M_{Rd}/M_{Ed} in the two orthogonal principal directions at the lowest cross-section where a plastic hinge can form in the vertical element, in the seismic design situation.

(6) For the foundations of columns of concentric braced frames, Ω is the minimum value of the ratio $N_{pl,Rd}/N_{Ed}$ over all tensile diagonals of the braced frame.

(7) For the foundations of columns of eccentric braced frames, Ω is the minimum value of the ratio $V_{pl,Rd}/V_{Ed}$ over all beam plastic shear zones, or $M_{pl,Rd}/M_{Ed}$ over all beam plastic hinge zones in the braced frame.

(8) For common foundations of more than one vertical element (foundation beams, strip footings, rafts, etc.) **(2)**P is deemed to be satisfied if the value of Ω used in expression (4.30) is derived from the vertical element with the largest horizontal shear force in the design seismic situation, or, alternatively, if a value $\Omega = 1$ is used in expression (4.30) with the value of the overstrength factor γ_{Rd} increased to 1,4.

4.4.2.7 Seismic joint condition

(1)P Buildings shall be protected from earthquake-induced pounding from adjacent structures or between structurally independent units of the same building.

(2) **(1)**P is deemed to be satisfied:

(a) for buildings, or structurally independent units, that do not belong to the same property, if the distance from the property line to the potential points of impact is not less than the maximum horizontal displacement of the building at the corresponding level, calculated in accordance with expression (4.23);

(b) for buildings, or structurally independent units, belonging to the same property, if the distance between them is not less than the square root of the sum- of the squares (SRSS) of the maximum horizontal displacements of the two buildings or units at the corresponding level, calculated in accordance with expression (4.23).

(3) If the floor elevations of the building or independent unit under design are the same as those of the adjacent building or unit, the above referred minimum distance may be reduced by a factor of 0,7.

4.4.3 Damage limitation

4.4.3.1 General

(1) The "damage limitation requirement" is considered to have been satisfied, if, under a seismic action having a larger probability of occurrence than the design seismic action corresponding to the "no-collapse requirement" in accordance with **2.1(1)**P and **3.2.1(3)**, the interstorey drifts are limited in accordance with **4.4.3.2**.

(2) Additional damage limitation verifications might be required in the case of buildings important for civil protection or containing sensitive equipment.

4.4.3.2 Limitation of interstorey drift

(1) Unless otherwise specified in Sections **5** to **9**, the following limits shall be observed:

a) for buildings having non-structural elements of brittle materials attached to the structure:

$$d_r \nu \leq 0{,}005 h ; \tag{4.31}$$

b) for buildings having ductile non-structural elements:

$$d_r \nu \leq 0{,}0075 h ; \tag{4.32}$$

c) for buildings having non-structural elements fixed in a way so as not to interfere with structural deformations, or without non-structural elements:

$$d_r \nu \leq 0{,}010 h \tag{4.33}$$

where

d_r is the design interstorey drift as defined in **4.4.2.2(2)**;

h is the storey height;

ν is the reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement.

(2) The value of the reduction factor ν may also depend on the importance class of the building. Implicit in its use is the assumption that the elastic response spectrum of the seismic action under which the "damage limitation requirement" should be met (see **3.2.2.1(1)**P). has the same shape as the elastic response spectrum of the design seismic action corresponding to the "ultimate limit state requirement" in accordance with **2.1(1)**P and **3.2.1(3)**

NOTE The values to be ascribed to ν for use in a country may be found in its National Annex. Different values of ν may be defined for the various seismic zones of a country, depending on the seismic hazard conditions and on the protection of property objective. The recommended values of ν are 0,4 for importance classes III and IV and $\nu = 0{,}5$ for importance classes I and II.

5 SPECIFIC RULES FOR CONCRETE BUILDINGS

5.1 General

5.1.1 Scope

(1)P Section **5** applies to the design of reinforced concrete buildings in seismic regions, henceforth called concrete buildings. Both monolithically cast-in-situ and precast buildings are addressed.

(2)P Concrete buildings with flat slab frames used as primary seismic elements in accordance with **4.2.2** are not fully covered by this section

(3)P For the design of concrete buildings EN 1992-1-1:2004 applies. The following rules are additional to those given in EN 1992-1-1:2004.

5.1.2 Terms and definitions

(1) The following terms are used in section **5** with the following meanings:

critical region
region of a primary seismic element, where the most adverse combination of action effects (M, N, V, T) occurs and where plastic hinges may form

NOTE In concrete buildings critical regions are dissipative zones. The length of the critical region is defined for each type of primary seismic element in the relevant clause of this section.

beam
structural element subjected mainly to transverse loads and to a normalised design axial force $\nu_d = N_{Ed}/A_c f_{cd}$ of not greater than 0,1 (compression positive)

NOTE In general, beams are horizontal.

column
structural element , supporting gravity loads by axial compression or subjected to a normalised design axial force $\nu_d = N_{Ed}/A_c f_{cd}$ of greater than 0,1

NOTE In general, columns are vertical.

wall
structural element supporting other elements and having an elongated cross-section with a length to thickness ratio l_w/b_w of greater than 4

NOTE In general, the plane of a wall is vertical.

ductile wall
wall fixed at the base so that the relative rotation of the base with respect to the rest of the structural system is prevented, and that is designed and detailed to dissipate energy in a flexural plastic hinge zone free of openings or large perforations, just above its base

large lightly reinforced wall
wall with large cross-sectional dimensions, that is, a horizontal dimension l_w at least equal to 4,0 m or two-thirds of the height h_w of the wall, whichever is less, which is expected to develop limited cracking and inelastic behaviour under the seismic design situation

NOTE Such a wall is expected to transform seismic energy to potential energy (through temporary uplift of structural masses) and to energy dissipated in the soil through rigid-body rocking, etc. Due to its dimensions, or to lack-of-fixity at the base, or to connectivity with large transverse walls preventing plastic hinge rotation at the base, it cannot be designed effectively for energy dissipation through plastic hinging at the base.

coupled wall
structural element composed of two or more single walls, connected in a regular pattern by adequately ductile beams ("coupling beams"), able to reduce by at least 25% the sum of the base bending moments of the individual walls if working separately

wall system
structural system in which both vertical and lateral loads are mainly resisted by vertical structural walls, either coupled or uncoupled, whose shear resistance at the building base exceeds 65% of the total shear resistance of the whole structural system

NOTE 1 In this definition and in the ones to follow, the fraction of shear resistance may be substituted by the fraction of shear forces in the seismic design situation.

NOTE 2 If most of the total shear resistance of the walls included in the system is provided by coupled walls, the system may be considered as a coupled wall system.

frame system
structural system in which both the vertical and lateral loads are mainly resisted by spatial frames whose shear resistance at the building base exceeds 65% of the total shear resistance of the whole structural system

dual system
structural system in which support for the vertical loads is mainly provided by a spatial frame and resistance to lateral loads is contributed to in part by the frame system and in part by structural walls, coupled or uncoupled

frame-equivalent dual system
dual system in which the shear resistance of the frame system at the building base is greater than 50% of the total shear resistance of the whole structural system

wall-equivalent dual system
dual system in which the shear resistance of the walls at the building base is higher than 50% of the total seismic resistance of the whole structural system

torsionally flexible system
dual or wall system not having a minimum torsional rigidity (see **5.2.2.1(4)**P and **(6)**)

NOTE 1 An example of this is a structural system consisting of flexible frames combined with walls concentrated near the centre of the building in plan.

NOTE 2 This definition does not cover systems containing several extensively perforated walls around vertical services and facilities. For such systems the most appropriate definition of the respective overall structural configuration should be chosen on a case-by-case basis.

inverted pendulum system
system in which 50% or more of the mass is in the upper third of the height of the structure, or in which the dissipation of energy takes place mainly at the base of a single building element

NOTE One-storey frames with column tops connected along both main directions of the building and with the value of the column normalized axial load ν_d exceeding 0,3 nowhere, do not belong in this category.

5.2 Design concepts

5.2.1 Energy dissipation capacity and ductility classes

(1)P The design of earthquake resistant concrete buildings shall provide the structure with an adequate capacity to dissipate energy without substantial reduction of its overall resistance against horizontal and vertical loading. To this end, the requirements and criteria of Section **2** apply. In the seismic design situation adequate resistance of all structural elements shall be provided, and non-linear deformation demands in critical regions should be commensurate with the overall ductility assumed in calculations.

(2)P Concrete buildings may alternatively be designed for low dissipation capacity and low ductility, by applying only the rules of EN 1992-1-1:2004 for the seismic design situation, and neglecting the specific provisions given in this section, provided the requirements set forth in **5.3** are met. For buildings which are not base-isolated (see Section **10**), design with this alternative, termed ductility class L (low), is recommended only in low seismicity cases (see **3.2.1(4)**).

(3)P Earthquake resistant concrete buildings other than those to which **(2)**P of this subclause applies, shall be designed to provide energy dissipation capacity and an overall ductile behaviour. Overall ductile behaviour is ensured if the ductility demand involves globally a large volume of the structure spread to different elements and locations of all its storeys. To this end ductile modes of failure (e.g. flexure) should precede brittle failure modes (e.g. shear) with sufficient reliability.

(4)P Concrete buildings designed in accordance with **(3)**P of this subclause, are classified in two ductility classes DCM (medium ductility) and DCH (high ductility), depending on their hysteretic dissipation capacity. Both classes correspond to buildings designed, dimensioned and detailed in accordance with specific earthquake resistant provisions, enabling the structure to develop stable mechanisms associated with large dissipation of hysteretic energy under repeated reversed loading, without suffering brittle failures.

(5)P To provide the appropriate amount of ductility in ductility classes M and H , specific provisions for all structural elements shall be satisfied in each class (see **5.4** - **5.6**). In correspondence with the different available ductility in the two ductility classes, different values of the behaviour factor q are used for each class (see **5.2.2.2**).

NOTE Geographical limitations on the use of ductility classes M and H may be found in the relevant National Annex.

5.2.2 Structural types and behaviour factors

5.2.2.1 Structural types

(1)P Concrete buildings shall be classified into one of the following structural types (see **5.1.2**) according to their behaviour under horizontal seismic actions:

a) frame system;

b) dual system (frame or wall equivalent);

c) ductile wall system (coupled or uncoupled);

d) system of large lightly reinforced walls;

e) inverted pendulum system;

f) torsionally flexible system.

(2) Except for those classified as torsionally flexible systems, concrete buildings may be classified to one type of structural system in one horizontal direction and to another in the other.

(3)P A wall system shall be classified as a system of large lightly reinforced walls if, in the horizontal direction of interest, it comprises at least two walls with a horizontal dimension of not less than 4,0 m or $2h_w/3$, whichever is less, which collectively support at least 20% of the total gravity load from above in the seismic design situation, and has a fundamental period T_1, for assumed fixity at the base against rotation, less than or equal to 0,5 s. It is sufficient to have only one wall meeting the above conditions in one of the two directions, provided that: (a) the basic value of the behaviour factor, q_o, in that direction is divided by a factor of 1,5 over the value given in Table 5.1 and (b) that there are at least two walls meeting the above conditions in the orthogonal direction.

(4)P The first four types of systems (i.e. frame, dual and wall systems of both types) shall possess a minimum torsional rigidity that satisfies expression (4.1b) in both horizontal directions.

(5) For frame or wall systems with vertical elements that are well distributed in plan, the requirement specified in **(4)**P of this subclause may be considered as being satisfied without analytical verification.

(6) Frame, dual or wall systems without a minimum torsional rigidity in accordance with **(4)**P of this subclause should be classified as torsionally flexible systems.

(7) If a structural system does not qualify as a system of large lightly reinforced walls according to **(3)**P above, then all of its walls should be designed and detailed as ductile walls.

5.2.2.2 Behaviour factors for horizontal seismic actions

(1)P The upper limit value of the behaviour factor q, introduced in **3.2.2.5(3)** to account for energy dissipation capacity, shall be derived for each design direction as follows:

$$q = q_{\mathrm{o}} k_{\mathrm{w}} \geq 1{,}5 \tag{5.1}$$

where

q_{o} is the basic value of the behaviour factor, dependent on the type of the structural system and on its regularity in elevation (see **(2)** of this subclause);

k_{w} is the factor reflecting the prevailing failure mode in structural systems with walls (see **(11)**P of this subclause).

(2) For buildings that are regular in elevation in accordance with **4.2.3.3**, the basic values of q_{o} for the various structural types are given in Table 5.1.

Table 5.1: Basic value of the behaviour factor, q_{o}, for systems regular in elevation

STRUCTURAL TYPE	DCM	DCH
Frame system, dual system, coupled wall system	$3{,}0\alpha_{u}/\alpha_{1}$	$4{,}5\alpha_{u}/\alpha_{1}$
Uncoupled wall system	3,0	$4{,}0\alpha_{u}/\alpha_{1}$
Torsionally flexible system	2,0	3,0
Inverted pendulum system	1,5	2,0

(3) For buildings which are not regular in elevation, the value of q_o should be reduced by 20% (see **4.2.3.1(7)** and Table 4.1).

(4) α_1 and α_u are defined as follows:

α_1 is the value by which the horizontal seismic design action is multiplied in order to first reach the flexural resistance in any member in the structure, while all other design actions remain constant;

α_u is the value by which the horizontal seismic design action is multiplied, in order to form plastic hinges in a number of sections sufficient for the development of overall structural instability, while all other design actions remain constant. The factor α_u may be obtained from a nonlinear static (pushover) global analysis.

(5) When the multiplication factor α_u/α_1 has not been evaluated through an explicit calculation, for buildings which are regular in plan the following approximate values of α_u/α_1 may be used.

a) Frames or frame-equivalent dual systems.

- One-storey buildings: α_u/α_1=1,1;
- multistorey, one-bay frames: α_u/α_1=1,2;
- multistorey, multi-bay frames or frame-equivalent dual structures: α_u/α_1=1,3.

b) Wall- or wall-equivalent dual systems.

– wall systems with only two uncoupled walls per horizontal direction: α_u/α_1=1,0;

– other uncoupled wall systems: α_u/α_1=1,1;

– wall-equivalent dual, or coupled wall systems: α_u/α_1=1,2.

(6) For buildings which are not regular in plan (see **4.2.3.2**), the approximate value of α_u/α_1 that may be used when calculations are not performed for its evaluation are equal to the average of (a) 1,0 and of (b) the value given in **(5)** of this subclause.

(7) Values of α_u/α_1 higher than those given in **(5)** and **(6)** of this subclause may be used, provided that they are confirmed through a nonlinear static (pushover) global analysis.

(8) The maximum value of α_u/α_1 that may be used in the design is equal to 1,5, even when the analysis mentioned in **(7)** of this subclause results in higher values.

(9) The value of q_o given for inverted pendulum systems may be increased, if it can be shown that a correspondingly higher energy dissipation is ensured in the critical region of the structure.

(10) If a special and formal Quality System Plan is applied to the design, procurement and construction in addition to normal quality control schemes, increased values of q_o may be allowed. The increased values are not allowed to exceed the values given in Table 5.1 by more than 20%.

NOTE The values to be ascribed to q_o for use in a country and possibly in particular projects in the country depending on the special Quality System Plan, may be found in its National Annex.

(11)P The factor k_w reflecting the prevailing failure mode in structural systems with walls shall be taken as follows:

$$k_w = \left\{ \begin{array}{l} 1{,}00, \text{ for frame and frame} - \text{equivalent dual systems} \\ (1+\alpha_o)/3 \leq 1, \text{but not less than 0,5, for wall, wall - equivalent and torsionally} \\ \text{flexible systems} \end{array} \right\} \quad (5.2)$$

where α_o is the prevailing aspect ratio of the walls of the structural system.

(12) If the aspect ratios h_{wi}/l_{wi} of all walls i of a structural system do not significantly differ, the prevailing aspect ratio α_o may be determined from the following expression:

$$\alpha_o = \sum h_{wi} / \sum l_{wi} \quad (5.3)$$

where

h_{wi} is the height of wall i; and

l_{wi} is the length of the section of wall i.

(13) Systems of large lightly reinforced walls cannot rely on energy dissipation in plastic hinges and so should be designed as DCM structures.

5.2.3 Design criteria

5.2.3.1 General

(1) The design concepts in **5.2.1** and in Section **2** shall be implemented into the earthquake resistant structural elements of concrete buildings as specified in **5.2.3.2** - **5.2.3.7**.

(2) The design criteria in **5.2.3.2** - **5.2.3.7** are deemed to be satisfied, if the rules in **5.4** - **5.7** are observed.

5.2.3.2 Local resistance condition

(1)P All critical regions of the structure shall meet the requirements of **4.4.2.2(1)**.

5.2.3.3 Capacity design rule

(1)P Brittle failure or other undesirable failure mechanisms (e.g. concentration of plastic hinges in columns of a single storey of a multistorey building, shear failure of structural elements, failure of beam-column joints, yielding of foundations or of any element intended to remain elastic) shall be prevented, by deriving the design action effects of selected regions from equilibrium conditions, assuming that plastic hinges with their possible overstrengths have been formed in their adjacent areas.

(2) The primary seismic columns of frame or frame-equivalent concrete structures should satisfy the capacity design requirements of **4.4.2.3(4)** with the following exemptions.

a) In plane frames with at least four columns of about the same cross-sectional size, it is not necessary to satisfy expression (4.29) in all columns, but just in three out of every four columns.

b) At the bottom storey of two-storey buildings if the value of the normalised axial load ν_d does not exceed 0,3 in any column.

(3) Slab reinforcement parallel to the beam and within the effective flange width specified in **5.4.3.1.1(3)**, should be assumed to contribute to the beam flexural capacities taken into account for the calculation of $\sum M_{Rb}$ in expression (4.29), if it is anchored beyond the beam section at the face of the joint.

5.2.3.4 Local ductility condition

(1)P For the required overall ductility of the structure to be achieved, the potential regions for plastic hinge formation, to be defined later for each type of building element, shall possess high plastic rotational capacities.

(2) Paragraph **(1)**P is deemed to be satisfied if the following conditions are met:

a) a sufficient curvature ductility is provided in all critical regions of primary seismic elements, including column ends (depending on the potential for plastic hinge formation in columns) (see **(3)** of this subclause);

b) local buckling of compressed steel within potential plastic hinge regions of primary seismic elements is prevented. Relevant application rules are given in **5.4.3** and **5.5.3**;

c) appropriate concrete and steel qualities are adopted to ensure local ductility as follows:

- the steel used in critical regions of primary seismic elements should have high uniform plastic elongation (see **5.3.2(1)**P, **5.4.1.1(3)**P, **5.5.1.1(3)**P);
- the tensile strength to yield strength ratio of the steel used in critical regions of primary seismic elements should be significantly higher than unity. Reinforcing steel conforming to the requirements of **5.3.2(1)**P, **5.4.1.1(3)**P or **5.5.1.1(3)**P, as appropriate, may be deemed to satisfy this requirement;
- the concrete used in primary seismic elements should possess adequate compressive strength and a fracture strain which exceeds the strain at the maximum compressive strength by an adequate margin. Concrete conforming to the requirements of **5.4.1.1(1)**P or **5.5.1.1(1)**P, as appropriate, may be deemed to satisfy these requirements.

(3) Unless more precise data are available and except when **(4)** of this subclause applies, **(2)**a) of this subclause is deemed to be satisfied if the curvature ductility factor μ_ϕ of these regions (defined as the ratio of the post-ultimate strength curvature at 85% of the moment of resistance, to the curvature at yield, provided that the limiting strains of concrete and steel ε_{cu} and $\varepsilon_{su,k}$ are not exceeded) is at least equal to the following values:

$$\mu_\phi = 2q_o - 1 \qquad \text{if } T_1 \geq T_C \tag{5.4}$$

$$\mu_\phi = 1+2(q_o - 1)T_C/T_1 \qquad \text{if } T_1 < T_C \tag{5.5}$$

where q_o is the corresponding basic value of the behaviour factor from Table 5.1 and T_1 is the fundamental period of the building, both taken within the vertical plane in which bending takes place, and T_C is the period at the upper limit of the constant acceleration region of the spectrum, according to **3.2.2.2(2)**P.

NOTE Expressions (5.4) and (5.5) are based on the relationship between μ_ϕ and the displacement ductility factor, μ_δ: $\mu_\phi = 2\mu_\delta - 1$, which is normally a conservative approximation for concrete members, and on the following relationship between μ_δ and q: $\mu_\delta = q$ if $T_1 \geq T_C$, $\mu_\delta = 1+(q-1)T_C/T_1$ if $T_1 < T_C$ (see also B5 in Informative Annex B). The value of q_o is used instead of that of q, because q will be lower than q_o in irregular buildings, recognising that a higher lateral resistance is needed to protect them. However, the local ductility demands may actually be higher than those corresponding to the value of q, so a reduction in the curvature ductility capacity is not warranted.

(4) In critical regions of primary seismic elements with longitudinal reinforcement of steel class B in EN 1992-1-1:2004, Table C.1, the curvature ductility factor μ_ϕ should be at least equal to 1,5 times the value given by expression (5.4) or (5.5), whichever applies.

5.2.3.5 Structural redundancy

(1)P A high degree of redundancy accompanied by redistribution capacity shall be sought, enabling a more widely spread energy dissipation and an increased total dissipated energy. Consequently structural systems of lower static indeterminacy shall be assigned lower behaviour factors (see Table 5.1). The necessary redistribution capacity shall be achieved through the local ductility rules given in **5.4** to **5.6**.

5.2.3.6 Secondary seismic members and resistances

(1)P A limited number of structural members may be designated as secondary seismic members in accordance with **4.2.2**.

(2) Rules for the design and detailing of secondary seismic elements are given in **5.7**.

(3) Resistances or stabilising effects not explicitly taken into account in calculations may enhance both strength and energy dissipation (e.g. membrane reactions of slabs mobilised by upward deflections of structural walls).

(4) Non-structural elements may also contribute to energy dissipation, if they are uniformly distributed throughout the structure. Measures should be taken against possible local adverse effects due to the interaction between structural and nonstructural elements (see **5.9**).

(5) For masonry infilled frames (which are a common case of non-structural elements) special rules are given in **4.3.6** and **5.9**.

5.2.3.7 Specific additional measures

(1)P Due to the random nature of the seismic action and the uncertainties of the post-elastic cyclic behaviour of concrete structures, the overall uncertainty is substantially higher than with non-seismic actions. Therefore, measures shall be taken to reduce uncertainties related to the structural configuration, to the analysis, to the resistance and to the ductility.

(2)P Important resistance uncertainties may be produced by geometric errors. To minimize this type of uncertainty, the following rules shall be applied.

a) Certain minimum dimensions of the structural elements shall be respected (see **5.4.1.2** and **5.5.1.2**) to decrease the sensitivity to geometric errors.

b) The ratio of the minimum to the maximum dimension of linear elements shall be limited, to minimize the risk of lateral instability of these elements (see **5.4.1.2** and **5.5.1.2.1(2)**P).

c) Storey drifts shall be limited, to limit P-Δ effects in the columns (see **4.4.2.2(2)-(4)**).

d) A substantial percentage of the top reinforcement of beams at their end cross-sections shall continue along the entire length of the beam (see **5.4.3.1.2(5)**P, **5.5.3.1.3(5)**P) to account for the uncertainty in the location of the inflection point.

e) Account shall be taken of reversals of moments not predicted by the analysis by providing minimum reinforcement at the relevant side of beams (see **5.5.3.1.3**).

(3)P To minimize ductility uncertainties, the following rules shall be observed.

a) A minimum of local ductility shall be provided in all primary seismic elements, independently of the ductility class adopted in the design (see **5.4** and **5.5**).

b) A minimum amount of tension reinforcement shall be provided, to avoid brittle failure upon cracking (see **5.4.3** and **5.5.5**).

c) An appropriate limit of the normalised design axial force shall be respected (see **5.4.3.2.1(3)**P, **5.4.3.4.1(2)**, **5.5.3.2.1(3)**P and **5.5.3.4.1(2)**) to reduce the consequences of cover spalling and to avoid the large uncertainties in the available ductility at high levels of applied axial force.

5.2.4 Safety verifications

(1)P For ultimate limit state verifications the partial factors for material properties γ_c and γ_s shall take into account the possible strength degradation of the materials due to cyclic deformations.

(2) If more specific data are not available, the values of the partial factors γ_c and γ_s adopted for the persistent and transient design situations should be applied, assuming that due to the local ductility provisions the ratio between the residual strength after degradation and the initial one is roughly equal to the ratio between the γ_M values for accidental and fundamental load combinations.

(3) If the strength degradation is appropriately accounted for in the evaluation of the material properties, the γ_M values adopted for the accidental design situation may be used.

NOTE 1 The values ascribed to the material partial factors γ_c and γ_s for the persistent and transient design situations and the accidental design situations for use in a country may be found in its National Annex to EN 1992-1-1:2004.

NOTE 2 The National Annex may specify whether the γ_M values to be used for earthquake resistant design are those for the persistent and transient or for the accidental design situations. Intermediate values may even be chosen in the National Annex, depending on how the material properties under earthquake loading are evaluated. The recommended choice is that of **(2)** in this subclause, which allows the same value of the design resistance to be used for the persistent and transient design situations (e.g. gravity loads with wind) and for the seismic design situation.

5.3 Design to EN 1992-1-1

5.3.1 General

(1) Seismic design for low ductility (ductility class L), following EN 1992-1-1:2004 without any additional requirements other than those of **5.3.2**, is recommended only for low seismicity cases (see **3.2.1(4)**).

5.3.2 Materials

(1)P In primary seismic elements (see **4.2.2**), reinforcing steel of class B or C in EN 1992-1-1:2004, Table C.1 shall be used.

5.3.3 Behaviour factor

(1) A behaviour factor q of up to 1,5 may be used in deriving the seismic actions, regardless of the structural system and the regularity in elevation.

5.4 Design for DCM

5.4.1 Geometrical constraints and materials

5.4.1.1 Material requirements

(1)P Concrete of a class lower than C 16/20 shall not be used in primary seismic elements.

(2)P With the exceptions of closed stirrups and cross-ties, only ribbed bars shall be used as reinforcing steel in critical regions of primary seismic elements.

(3)P In critical regions of primary seismic elements reinforcing steel of class B or C in EN 1992-1-1:2004, Table C.1 shall be used.

(4)P Welded wire meshes may be used, if they meet the requirements in **(2)**P and **(3)**P of this subclause.

5.4.1.2 Geometrical constraints

5.4.1.2.1 Beams

(1)P The eccentricity of the beam axis shall be limited relative to that of the column into which it frames to enable efficient transfer of cyclic moments from a primary seismic beam to a column to be achieved.

(2) To enable the requirement specified in **(1)**P to be met the distance between the centroidal axes of the two members should be limited to less than $b_c/4$, where b_c is the largest cross-sectional dimension of the column normal to the longitudinal axis of the beam.

(3)P To take advantage of the favourable effect of column compression on the bond of horizontal bars passing through the joint, the width b_w of a primary seismic beam shall satisfy the following expression:

$$b_w \leq \min\{b_c + h_w ; 2b_c\} \tag{5.6}$$

where h_w is the depth of the beam and b_c is as defined in **(2)** of this subclause.

5.4.1.2.2 Columns

(1) Unless $\theta \leq 0{,}1$ (see **4.4.2.2(2)**), the cross-sectional dimensions of primary seismic columns should not be smaller than one tenth of the larger distance between the point of contraflexure and the ends of the column, for bending within a plane parallel to the column dimension considered.

5.4.1.2.3 Ductile Walls

(1) The thickness of the web, $b_{wo,}$ (in metres) should satisfy the following expression:

$$b_{wo} \geq \max\{0{,}15,\ h_s/20\} \tag{5.7}$$

where h_s is the clear storey height in metres.

(2) Additional requirements apply with respect to the thickness of the confined boundary elements of walls, as specified in **5.4.3.4.2(10)**

5.4.1.2.4 Large lightly reinforced walls

(1) The provision in **5.4.1.2.3(1)** applies also to large lightly reinforced walls.

5.4.1.2.5 Specific rules for beams supporting discontinued vertical elements

(1)P Structural walls shall not rely for their support on beams or slabs.

(2)P For a primary seismic beam supporting columns discontinued below the beam, the following rules apply:

a) there shall be no eccentricity of the column axis relative to that of the beam;

b) the beam shall be supported by at least two direct supports, such as walls or columns.

5.4.2 Design action effects

5.4.2.1 General

(1)P With the exception of ductile primary seismic walls, for which the special provisions of **5.4.2.4** apply, the design values of bending moments and axial forces shall be obtained from the analysis of the structure for the seismic design situation in accordance with EN 1990:2001 **6.4.3.4**, taking into account second order effects in accordance with **4.4.2.2** and the capacity design requirements of **5.2.3.3(2)**. Redistribution of bending moments in accordance with EN 1992-1-1 is permitted. The design values of shear forces of primary seismic beams, columns, ductile walls and lightly reinforced walls, are determined in accordance with **5.4.2.2**, **5.4.2.3**, **5.4.2.4** and **5.4.2.5**, respectively.

5.4.2.2 Beams

(1)P In primary seismic beams the design shear forces shall be determined in accordance with the capacity design rule, on the basis of the equilibrium of the beam under: a) the transverse load acting on it in the seismic design situation and b) end moments $M_{i,d}$ (with i=1,2 denoting the end sections of the beam), corresponding to plastic hinge formation for positive and negative directions of seismic loading. The plastic hinges should be taken to form at the ends of the beams or (if they form there first) in the vertical elements connected to the joints into which the beam ends frame (see Figure 5.1).

(2) Paragraph **(1)**P of this subclause should be implemented as follows.

a) At end section i, two values of the acting shear force should be calculated, i.e. the maximum $V_{Ed,max,i}$ and the minimum $V_{Ed,min,i}$ corresponding to the maximum positive and the maximum negative end moments $M_{i,d}$ that can develop at ends 1 and 2 of the beam.

b) End moments $M_{i,d}$ in **(1)**P and in **(2)** a) of this subclause may be determined as follows:

$$M_{i,d} = \gamma_{Rd} M_{Rb,i} \min(1, \frac{\sum M_{Rc}}{\sum M_{Rb}}) \tag{5.8}$$

where

γ_{Rd} is the factor accounting for possible overstrength due to steel strain hardening, which in the case of DCM beams may be taken as being equal to 1,0;

$M_{Rb,i}$ is the design value of the beam moment of resistance at end i in the sense of the seismic bending moment under the considered sense of the seismic action;

ΣM_{Rc} and ΣM_{Rb} are the sum of the design values of the moments of resistance of the columns and the sum of the design values of the moments of resistance of the beams framing into the joint, respectively (see **4.4.2.3(4)**). The value of ΣM_{Rc} should correspond to the column axial force(s) in the seismic design situation for the considered sense of the seismic action.

c) At a beam end where the beam is supported indirectly by another beam, instead of framing into a vertical member, the beam end moment $M_{i,d}$ there may be taken as being equal to the acting moment at the beam end section in the seismic design situation.

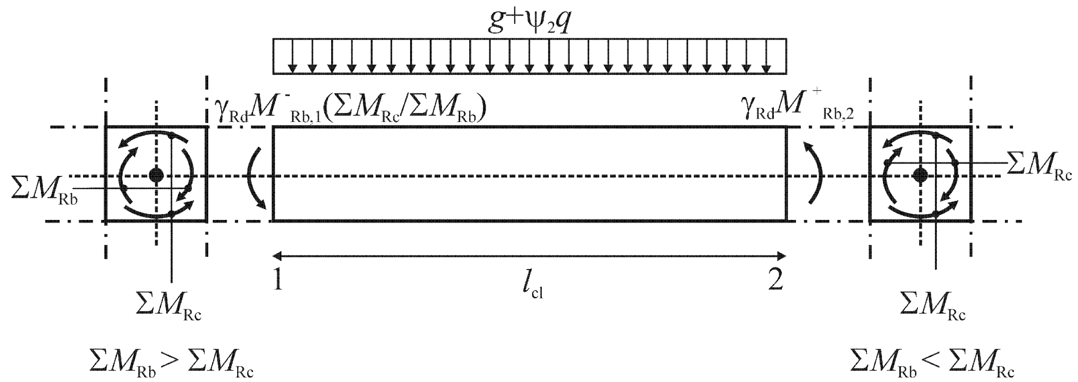


Figure 5.1: Capacity design values of shear forces on beams

5.4.2.3 Columns

(1)P In primary seismic columns the design values of shear forces shall be determined in accordance with the capacity design rule, on the basis of the equilibrium of the column under end moments $M_{i,d}$ (with i=1,2 denoting the end sections of the column), corresponding to plastic hinge formation for positive and negative directions of seismic loading. The plastic hinges should be taken to form at the ends of the beams connected to the joints into which the column end frames, or (if they form there first) in the columns (see Figure 5.2).

(2) End moments $M_{i,d}$ in **(1)**P of this subclause may be determined from the following expression:

$$M_{\mathrm{i,d}} = \gamma_{\mathrm{Rd}} M_{\mathrm{Rc,i}} \min\left(1, \frac{\sum M_{\mathrm{Rb}}}{\sum M_{\mathrm{Rc}}}\right) \tag{5.9}$$

where

γ_{Rd} is the factor accounting for overstrength due to steel strain hardening and confinement of the concrete of the compression zone of the section, taken as being equal to 1,1;

$M_{Rc,i}$ is the design value of the column moment of resistance at end i in the sense of the seismic bending moment under the considered sense of the seismic action;

ΣM_{Rc} and ΣM_{Rb} are as defined in **5.4.2.2(2)**.

(3) The values of $M_{Rc,i}$ and ΣM_{Rc} should correspond to the column axial force(s) in the seismic design situation for the considered sense of the seismic action.

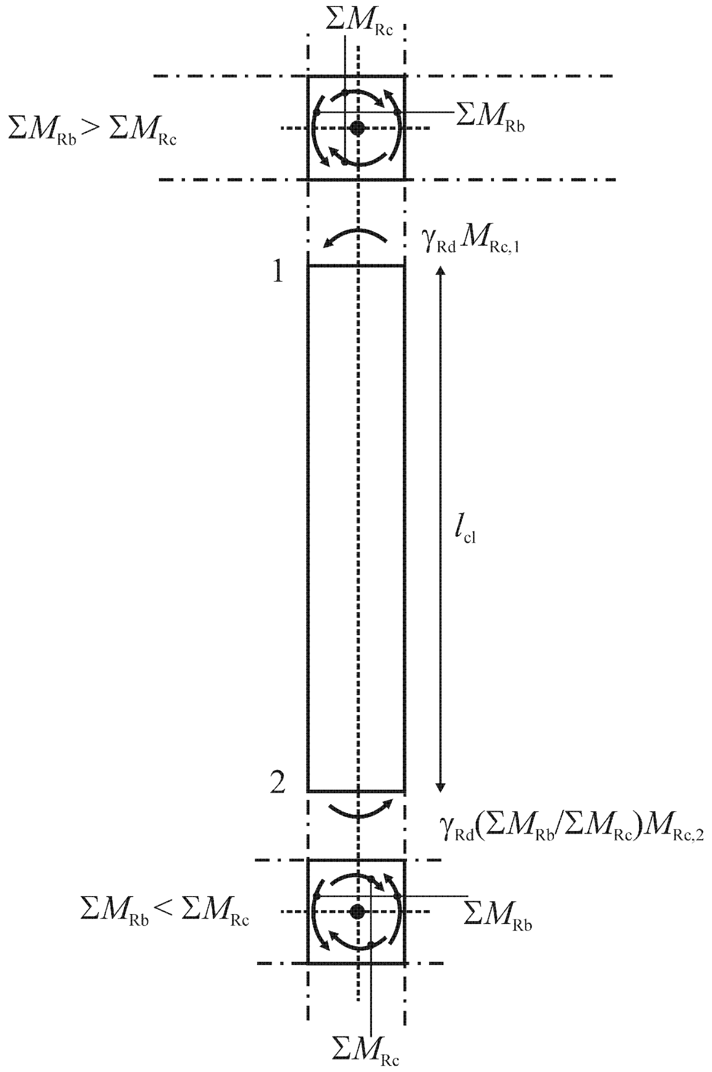


Figure 5.2: Capacity design shear force in columns

5.4.2.4 Special provisions for ductile walls

(1)P Uncertainties in the analysis and post-elastic dynamic effects shall be taken into account, at least through an appropriate simplified method. If a more precise method is not available, the rules in the following clauses for the design envelopes for bending moments, as well as the magnification factors for shear forces, may be used.

(2) Redistribution of seismic action effects between primary seismic walls of up to 30% is allowed, provided that the total resistance demand is not reduced. Shear forces should be redistributed along with the bending moments, so that the in the individual walls the ratio of bending moments to shear forces is not appreciably affected. In walls subjected to large fluctuations of axial force, as e.g. in coupled walls, moments and shears should be redistributed from the wall(s) which are under low compression or under net tension, to those which are under high axial compression.

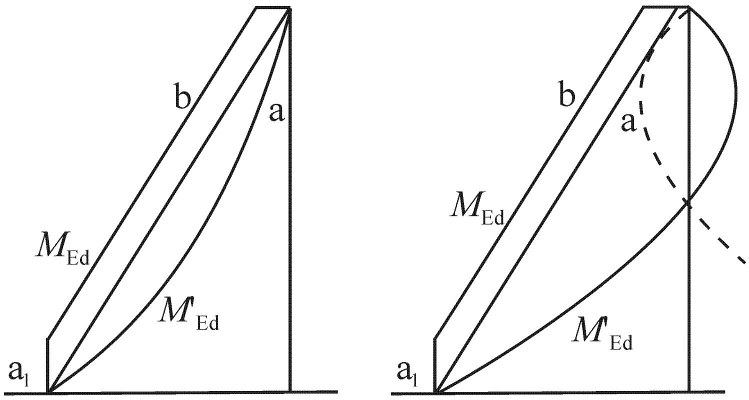
(3) In coupled walls redistribution of seismic action effects between coupling beams of different storeys of up to 20% is allowed, provided that the seismic axial force at the

base of each individual wall (the resultant of the shear forces in the coupling beams) is not affected.

(4)P Uncertainties regarding the moment distribution along the height of slender primary seismic walls (with height to length ratio h_w/l_w greater than 2,0) shall be covered.

(5) The requirement specified in **(4)**P of this subclause may be satisfied by applying, irrespective of the type of analysis used, the following simplified procedure.

The design bending moment diagram along the height of the wall should be given by an envelope of the bending moment diagram from the analysis, vertically displaced (tension shift). The envelope may be assumed linear, if the structure does not exhibit significant discontinuities of mass, stiffness or resistance over its height (see Figure 5.3). The tension shift should be consistent with the strut inclination taken in the ULS verification for shear, with a possible fan-type pattern of struts near the base, and with the floors acting as ties.



Key

a moment diagram from analysis

b design envelope

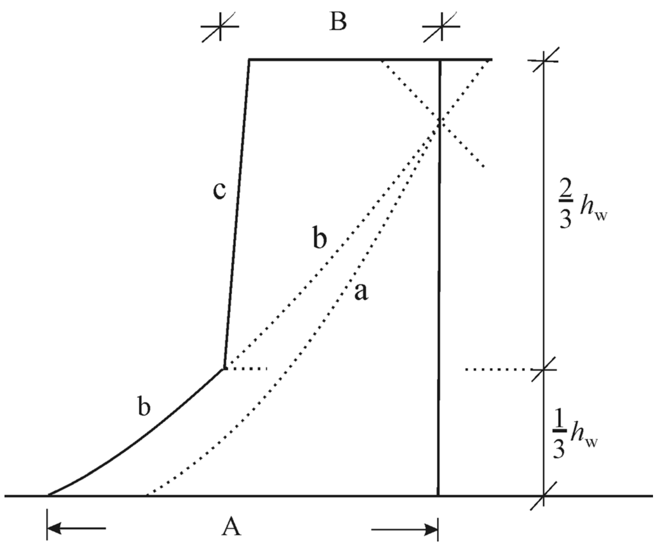
a_l tension shift

Figure 5.3: Design envelope for bending moments in slender walls (left: wall systems; right: dual systems).

(6)P The possible increase in shear forces after yielding at the base of a primary seismic wall, shall be taken into account.

(7) The requirement specified in **(6)**P of this subclause may be satisfied if the design shear forces are taken as being 50% higher than the shear forces obtained from the analysis.

(8) In dual systems containing slender walls the design envelope of shear forces in accordance with Figure 5.4 should be used, to account for uncertainties in higher mode effects.



Key

a shear diagram from analysis

b magnified shear diagram

c design envelope

A $V_{\text{wall,base}}$

B $V_{\text{wall,top}} \geq V_{\text{wall,base}}/2$

Figure 5.4: Design envelope of the shear forces in the walls of a dual system.

5.4.2.5 Special provisions for large lightly reinforced walls

(1)P To ensure that flexural yielding precedes attainment of the ULS in shear, the shear force V'_{Ed} from the analysis shall be increased.

(2) The requirement in **(1)**P of this subclause is considered to be satisfied if at every storey of the wall the design shear force V_{Ed} is obtained from the shear force calculated from the analysis, V'_{Ed}, in accordance with the following expression:

$$V_{\text{Ed}} = V'_{\text{Ed}} \frac{q+1}{2} \tag{5.10}$$

(3)P The additional dynamic axial forces developed in large walls due to uplifting from the soil, or due to the opening and closing of horizontal cracks, shall be taken into account in the ULS verification of the wall for flexure with axial force.

(4) Unless the results of a more precise calculation are available, the dynamic component of the wall axial force in **(3)**P of this subclause may be taken as being 50% of the axial force in the wall due to the gravity loads present in the seismic design situation. This force should be taken to have a plus or a minus sign, whichever is most unfavourable.

(5) If the value of the behaviour factor q does not exceed 2,0, the effect of the dynamic axial force in **(3)** and **(4)** of this subclause may be neglected.

5.4.3 ULS verifications and detailing

5.4.3.1 Beams

5.4.3.1.1 Resistance in bending and shear

(1) The bending and shear resistances should be computed in accordance with EN 1992-1-1:2004.

(2) The top-reinforcement of the end cross-sections of primary seismic beams with a T- or L-shaped section should be placed mainly within the width of the web. Only part of this reinforcement may be placed outside the width of the web, but within the effective flange width b_{eff}.

(3) The effective flange width b_{eff} may be assumed to be as follows:

a) for primary seismic beams framing into exterior columns, the effective flange width b_{eff} is taken, in the absence of a transverse beam, as being equal to the width b_c of the column (Figure 5.5b), or, if there is a transverse beam of similar depth, equal to this width increased by $2h_f$ on each side of the beam (Figure 5.5a);

b) for primary seismic beams framing into interior columns the above widths may be increased by $2h_f$ on each side of the beam (Figure 5.5c and d).

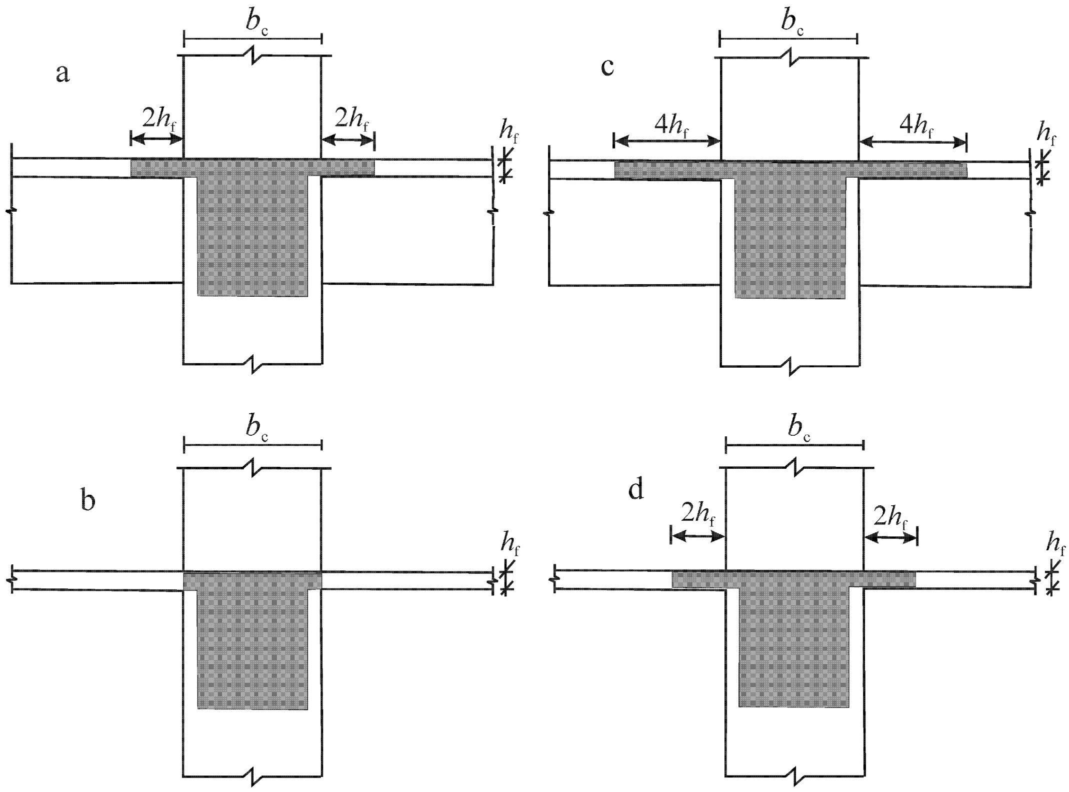


Figure 5.5: Effective flange width b_{eff} for beams framing into columns

5.4.3.1.2 Detailing for local ductility

(1)P The regions of a primary seismic beam up to a distance $l_{cr} = h_w$ (where h_w denotes the depth of the beam) from an end cross-section where the beam frames into a beam-column joint, as well as from both sides of any other cross-section liable to yield in the seismic design situation, shall be considered as being critical regions.

(2) In primary seismic beams supporting discontinued (cut-off) vertical elements, the regions up to a distance of $2h_w$ on each side of the supported vertical element should be considered as being critical regions.

(3)P To satisfy the local ductility requirement in the critical regions of primary seismic beams, the value of the curvature ductility factor μ_ϕ shall be at least equal to the value given in **5.2.3.4(3)**.

(4) The requirement specified in **(3)**P of this subclause is deemed to be satisfied, if the following conditions are met at both flanges of the beam.

a) at the compression zone reinforcement of not less than half of the reinforcement provided at the tension zone is placed, in addition to any compression reinforcement needed for the ULS verification of the beam in the seismic design situation.

b) The reinforcement ratio of the tension zone ρ does not exceed a value ρ_{max} equal to:

$$\rho_{max} = \rho' + \frac{0{,}0018}{\mu_\varphi \varepsilon_{sy,d}} \cdot \frac{f_{cd}}{f_{yd}} \tag{5.11}$$

with the reinforcement ratios of the tension zone and compression zone, ρ and ρ', both normalised to bd, where b is the width of the compression flange of the beam. If the tension zone includes a slab, the amount of slab reinforcement parallel to the beam within the effective flange width defined in **5.4.3.1.1(3)** is included in ρ.

(5)P Along the entire length of a primary seismic beam, the reinforcement ratio of the tension zone, ρ, shall be not less than the following minimum value ρ_{min}:

$$\rho_{min} = 0{,}5\left(\frac{f_{ctm}}{f_{yk}}\right) \tag{5.12}$$

(6)P Within the critical regions of primary seismic beams, hoops satisfying the following conditions shall be provided:

a) The diameter d_{bw} of the hoops (in millimetres) shall be not less than 6.

b) The spacing, s, of hoops (in millimetres) shall not exceed:

$$s = \min\{h_w/4;\ 24d_{bw};\ 225;\ 8d_{bL}\} \tag{5.13}$$

where

d_{bL} is the minimum longitudinal bar diameter (in millimetres); and

h_w the beam depth (in millimetres).

c) The first hoop shall be placed not more than 50 mm from the beam end section (see Figure 5.6).

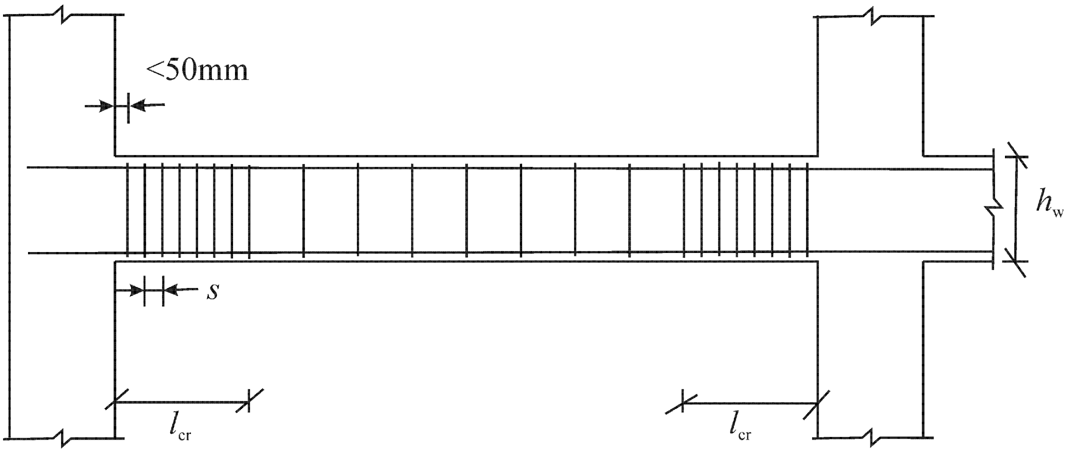


Figure 5.6: Transverse reinforcement in critical regions of beams

5.4.3.2 Columns

5.4.3.2.1 Resistances

(1)P Flexural and shear resistance shall be computed in accordance with EN 1992-1-1:2004, using the value of the axial force from the analysis in the seismic design situation.

(2) Biaxial bending may be taken into account in a simplified way by carrying out the verification separately in each direction, with the uniaxial moment of resistance reduced by 30%.

(3)P In primary seismic columns the value of the normalised axial force ν_d shall not exceed 0,65.

5.4.3.2.2 Detailing of primary seismic columns for local ductility

(1)P The total longitudinal reinforcement ratio ρ_l shall be not less than 0,01 and not more than 0,04. In symmetrical cross-sections symmetrical reinforcement should be provided ($\rho = \rho'$).

(2)P At least one intermediate bar shall be provided between corner bars along each column side, to ensure the integrity of the beam-column joints.

(3)P The regions up to a distance l_{cr} from both end sections of a primary seismic column shall be considered as being critical regions.

(4) In the absence of more precise information, the length of the critical region l_{cr} (in metres) may be computed from the following expression:

$$l_{cr} = \max\{h_c;\ l_{cl}/6;\ 0{,}45\} \qquad (5.14)$$

where

h_c is the largest cross-sectional dimension of the column (in metres); and

l_{cl} is the clear length of the column (in metres).

(5)P If l_c/h_c<3, the entire height of the primary seismic column shall be considered as being a critical region and shall be reinforced accordingly.

(6)P In the critical region at the base of primary seismic columns a value of the curvature ductility factor, μ_ϕ, should be provided, at least equal to that given in **5.2.3.4(3)**.

(7)P If for the specified value of μ_ϕ a concrete strain larger than ε_{cu2}=0,0035 is needed anywhere in the cross-section, compensation for the loss of resistance due to spalling of the concrete shall be achieved by means of adequate confinement of the concrete core, on the basis of the properties of confined concrete in EN 1992-1-1:2004, **3.1.9**.

(8) The requirements specified in **(6)**P and **(7)**P of this subclause are deemed to be satisfied if:

$$\alpha\omega_{\mathrm{wd}} \geq 30\mu_\varphi \nu_\mathrm{d} \cdot \varepsilon_{\mathrm{sy,d}} \cdot \frac{b_\mathrm{c}}{b_\mathrm{o}} - 0{,}035 \tag{5.15}$$

where

ω_{wd} is the mechanical volumetric ratio of confining hoops within the critical regions

$$\left[\omega_{\mathrm{wd}} = \frac{\text{volume of confining hoops}}{\text{volume of concrete core}} \cdot \frac{f_{\mathrm{yd}}}{f_{\mathrm{cd}}}\right];$$

μ_ϕ is the required value of the curvature ductility factor;

ν_d is the normalised design axial force ($\nu_d = N_{Ed}/A_c \cdot f_{cd}$);

$\varepsilon_{sy,d}$ is the design value of tension steel strain at yield;

h_c is the gross cross-sectional depth (parallel to the horizontal direction in which the value of μ_ϕ used in **(6)**P of this subclause applies);

h_o is the depth of confined core (to the centreline of the hoops);

b_c is the gross cross-sectional width;

b_o is the width of confined core (to the centreline of the hoops);

α is the confinement effectiveness factor, equal to $\alpha=\alpha_n \cdot \alpha_s$, with:

a) For rectangular cross-sections:

$$\alpha_\mathrm{n} = 1 - \sum_\mathrm{n} b_\mathrm{i}^2 / 6 b_\mathrm{o} h_\mathrm{o} \tag{5.16a}$$

$$\alpha_\mathrm{s} = (1 - s/2b_\mathrm{o})(1 - s/2h_\mathrm{o}) \tag{5.17a}$$

where

n is the total number of longitudinal bars laterally engaged by hoops or cross ties; and

b_i is the distance between consecutive engaged bars (see Figure 5.7; also for b_o, h_o, s).

b) For circular cross-sections with hoops and diameter of confined core D_o (to the centreline of hoops):

$$\alpha_n = 1 \tag{5.16b}$$

$$\alpha_s = (1 - s/2D_o)^2 \tag{5.17b}$$

c) For circular cross-sections with spiral reinforcement:

$$\alpha_n = 1 \tag{5.16c}$$

$$\alpha_s = (1 - s/2D_o) \tag{5.17c}$$

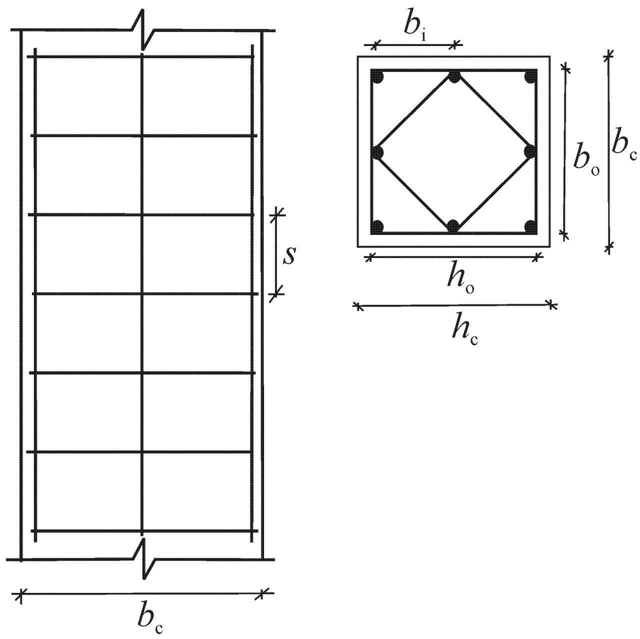


Figure 5.7: Confinement of concrete core

(9) A minimum value of ω_{wd} equal to 0,08 should be provided within the critical region at the base of the primary seismic columns.

(10)P Within the critical regions of the primary seismic columns, hoops and cross-ties, of at least 6 mm in diameter, shall be provided at a spacing such that a minimum ductility is ensured and local buckling of longitudinal bars is prevented. The hoop pattern shall be such that the cross-section benefits from the triaxial stress conditions produced by the hoops.

(11) The minimum conditions of **(10)**P of this subclause are deemed to be satisfied if the following conditions are met.

a) The spacing, s, of the hoops (in millimetres) does not exceed:

$$s = \min\{b_o/2;\ 175;\ 8d_{bL}\} \tag{5.18}$$

where

b_o (in millimetres) is the minimum dimension of the concrete core (to the centreline of the hoops); and

d_{bL} is the minimum diameter of the longitudinal bars (in millimetres).

b) The distance between consecutive longitudinal bars engaged by hoops or cross-ties does not exceed 200 mm, taking into account EN 1992-1-1:2004, **9.5.3(6)**.

(12)P The transverse reinforcement within the critical region at the base of the primary seismic columns may be determined as specified in EN 1992-1-1:2004, provided that the value of the normalised axial load in the seismic design situation is less than 0,2 and the value of the behaviour factor q used in the design does not exceed 2,0.

5.4.3.3 Beam-column joints

(1) The horizontal confinement reinforcement in joints of primary seismic beams with columns should be not less than that specified in **5.4.3.2.2(8)-(11)** for the critical regions of columns, with the exception of the case listed in the following paragraph.

(2) If beams frame into all four sides of the joint and their width is at least three-quarters of the parallel cross-sectional dimension of the column, the spacing of the horizontal confinement reinforcement in the joint may be increased to twice that specified in **(1)** of this subclause, but may not exceed 150 mm.

(3)P At least one intermediate (between column corner bars) vertical bar shall be provided at each side of a joint of primary seismic beams and columns.

5.4.3.4 Ductile Walls

5.4.3.4.1 Bending and shear resistance

(1)P Flexural and shear resistances shall be computed in accordance with EN 1992-1-1:2004, unless specified otherwise in the following paragraphs, using the value of the axial force resulting from the analysis in the seismic design situation.

(2) In primary seismic walls the value of the normalised axial load ν_d should not exceed 0,4.

(3)P Vertical web reinforcement shall be taken into account in the calculation of the flexural resistance of wall sections.

(4) Composite wall sections consisting of connected or intersecting rectangular segments (L-, T-, U-, I- or similar sections) should be taken as integral units, consisting of a web or webs parallel or approximately parallel to the direction of the acting seismic shear force and a flange or flanges normal or approximately normal to it. For the calculation of flexural resistance, the effective flange width on each side of a web should be taken to extend from the face of the web by the minimum of:

a) the actual flange width;

b) one-half of the distance to an adjacent web of the wall; and

c) 25% of the total height of the wall above the level considered.

5.4.3.4.2 Detailing for local ductility

(1) The height of the critical region h_{cr} above the base of the wall may be estimated as:

$$h_{cr} = \max\left[l_{w,}\ h_w / 6\right] \tag{5.19a}$$

but

$$h_{cr} \le \begin{cases} 2 \cdot l_w \\ \begin{cases} h_s & \text{for n} \le 6 \text{ storeys} \\ 2 \cdot h_s & \text{for n} \ge 7 \text{ storeys} \end{cases} \end{cases} \tag{5.19b}$$

where h_s is the clear storey height and where the base is defined as the level of the foundation or of the embedment in basement storeys with rigid diaphragms and perimeter walls.

(2) At the critical regions of walls a value μ_ϕ of the curvature ductility factor should be provided, that is at least equal to that calculated from expressions (5.4), (5.5) in **5.2.3.4(3)** with the basic value of the behaviour factor q_o in these expressions replaced by the product of q_o times the maximum value of the ratio M_{Ed}/M_{Rd} at the base of the wall in the seismic design situation, where M_{Ed} is the design bending moment from the analysis; and M_{Rd} is the design flexural resistance.

(3) Unless a more precise method is used, the value of μ_ϕ specified in **(2)** of this subclause may be supplied by means of confining reinforcement within edge regions of the cross-section, termed boundary elements, the extent of which should be determined in accordance with **(6)** of this subclause. The amount of confining reinforcement should be determined in accordance with **(4)** and **(5)** of this subclause:

(4) For walls of rectangular cross-section, the mechanical volumetric ratio of the required confining reinforcement ω_{wd} in boundary elements should satisfy the following expression, with the -values of μ_ϕ as specified in **(2)** of this subclause:

$$\alpha\omega_{wd} \ge 30\mu_\varphi\left(\nu_d + \omega_v\right)\varepsilon_{sy,d}\frac{b_c}{b_o} - 0{,}035 \tag{5.20}$$

where the parameters are defined in **5.4.3.2.2(8)**, except ω_v, which is the mechanical ratio of vertical web reinforcement ($\omega_v = \rho_v f_{yd,v}/f_{cd}$).

(5) For walls with barbells or flanges, or with a section consisting of several rectangular parts (T-, L-, I-, U-shaped sections, etc.) the mechanical volumetric ratio of the confining reinforcement in the boundary elements may be determined as follows:

a) The axial force and the web vertical reinforcement ratio shall be normalised to $h_c b_c f_{cd}$, with the width of the barbell or flange in compression taken as the cross-sectional width b_c ($\nu_d = N_{Ed} / h_c b_c f_{cd}$, $\omega_v = (A_{sv}/h_c b_c) f_{yd} / f_{cd}$). The neutral axis depth x_u at ultimate curvature after spalling of the concrete outside the confined core of the boundary elements may be estimated as:

$$x_u = (\nu_d + \omega_v)\frac{h_c b_c}{b_o} \tag{5.21}$$

where b_o is the width of the confined core in the barbell or flange. If the value of x_u from expression (5.21) does not exceed the depth of the barbell or flange after spalling of the cover concrete, then the mechanical volumetric ratio of the confining reinforcement in the barbell or flange is determined as in a) of this subclause (i.e. from expression (5.20), **5.4.3.4.2(4)**), with ν_d, ω_v, b_c and b_o referring to the width of the barbell or flange.

b) If the value of x_u exceeds the depth of the barbell or flange after spalling of the cover concrete, the general method based on: 1) the definition of the curvature ductility factor as $\mu_\phi = \phi_u / \phi_y$, 2) the calculation of ϕ_u as $\varepsilon_{cu2,c} / x_u$ and of ϕ_y as $\varepsilon_{sy} / (d - x_y)$, 3) section equilibrium for the estimation of neutral axis depths x_u and x_y, and 4) the strength and ultimate strain of confined concrete, $f_{ck,c}$ and $\varepsilon_{cu2,c}$ as a function of the effective lateral confining stress (see EN 1992-1-1:2004, **3.1.9**) may be followed. The required confining reinforcement, if needed, and the confined wall lengths should be calculated accordingly.

(6) The confinement of **(3)-(5)** of this subclause should extend vertically over the height h_{cr} of the critical region as defined in **5.4.3.4.2(1)** and horizontally along a length l_c measured from the extreme compression fibre of the wall up to the point where unconfined concrete may spall due to large compressive strains. If more precise data is not available, the compressive strain at which spalling is expected may be taken as being equal to ε_{cu2}=0,0035. The confined boundary element may be limited extend up to a distance of $x_u(1- \varepsilon_{cu2}/\varepsilon_{cu2,c})$ from the hoop centreline near the extreme compression fibre, with the depth of the confined compression zone x_u at ultimate curvature estimated from equilibrium (cf. expression (5.21) for a constant width b_o of the confined compression zone) and the ultimate strain $\varepsilon_{cu2,c}$ of confined concrete estimated on the basis of EN 1992-1-1:2004, **3.1.9** as $\varepsilon_{cu2,c}=0{,}0035+0{,}1\alpha\omega_{wd}$ (Figure 5.8). As a minimum, the length l_c of the confined boundary element should not be taken as being smaller than $0{,}15 \cdot l_w$ or $1{,}50 . b_w$.

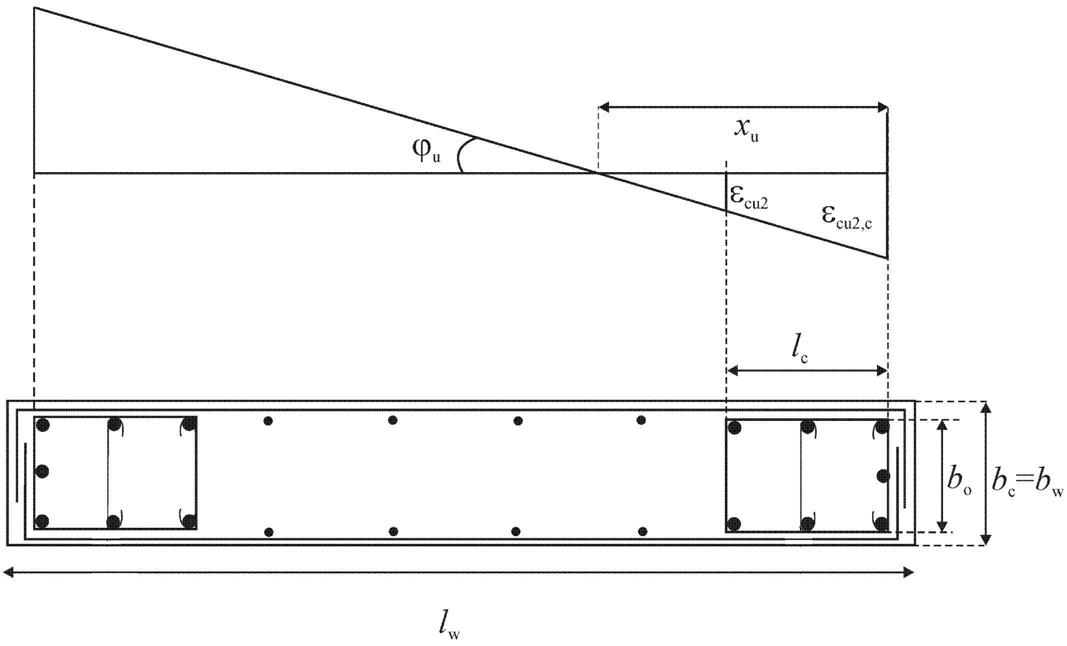


Figure 5.8: Confined boundary element of free-edge wall end

(top: strains at ultimate curvature; bottom: wall cross-section)

(7) No confined boundary element is required over wall flanges with thickness $b_f \geq h_s/15$ and width $l_f \geq h_s/5$, where h_s denotes the clear storey height (Figure 5.9). Nonetheless, confined boundary elements may be required at the ends of such flanges due to out-of-plane bending of the wall .

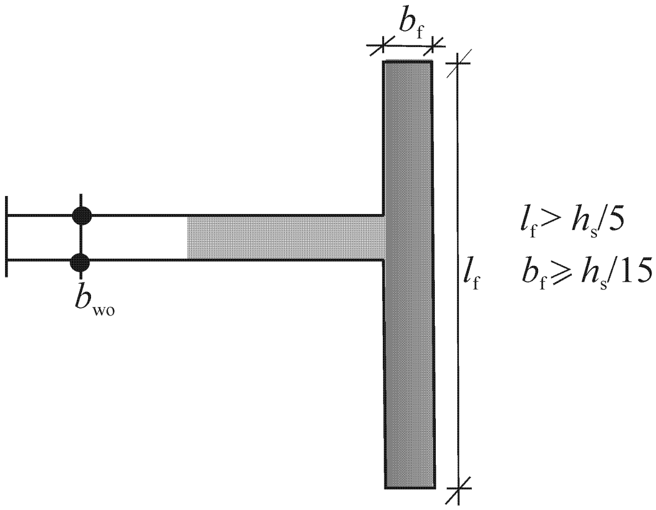


Figure 5.9: Confined boundary elements not needed at wall ends with a large transverse flange

(8) The longitudinal reinforcement ratio in the boundary elements should be not less than 0,005.

(9) The provisions of **5.4.3.2.2(9)** and **(11)** apply within the boundary elements of walls. Overlapping hoops should be used, so that every other longitudinal bar is engaged by a hoop or cross-tie.

(10) The thickness b_w of the confined parts of the wall section (boundary elements) should not be less than 200 mm. Moreover, if the length of the confined part does not exceed the maximum of $2b_w$ and $0,2l_w$, b_w should not be less than $h_s/15$, with h_s denoting the storey height. If the length of the confined part exceeds the maximum of $2b_w$ and $0,2l_w$ b_w should not be less than $h_s/10$ (See Figure 5.10).

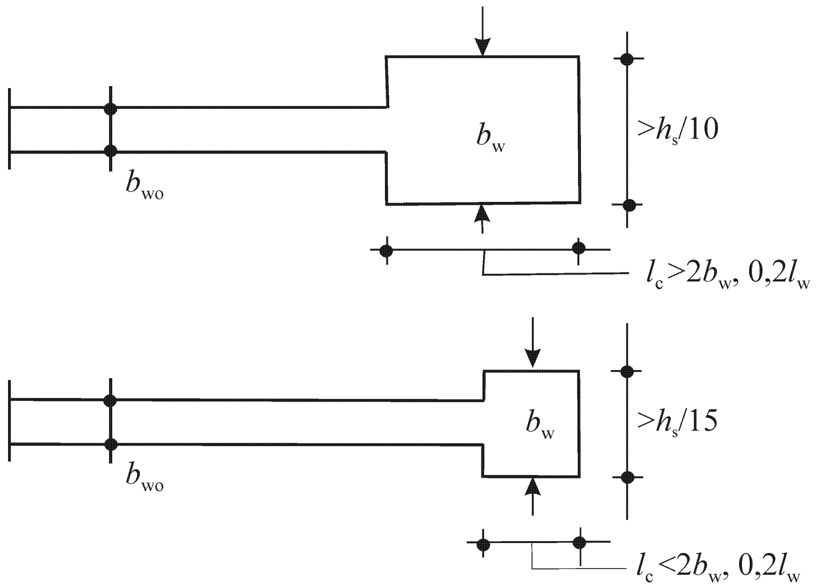


Figure 5.10: Minimum thickness of confined boundary elements

(11) In the height of the wall above the critical region only the relevant rules of EN 1992-1-1:2004 regarding vertical, horizontal and transverse reinforcement apply. However, in those parts of the section where under the seismic design situation the compressive strain ε_c exceeds 0,002, a minimum vertical reinforcement ratio of 0,005 should be provided.

(12) The transverse reinforcement of the boundary elements of **(4)-(10)** of this subclause may be determined in accordance with EN 1992-1-1:2004 alone, if one of the following conditions is fulfilled:

a) The value of the normalised design axial force ν_d is not greater than 0,15; or,

b) the value of ν_d is not greater than 0,20 and the *q*-factor used in the analysis is reduced by 15%.

5.4.3.5 Large lightly reinforced walls

5.4.3.5.1 Bending resistance

(1)P The ULS in bending with axial force shall be verified assuming horizontal cracking, in accordance with the relevant provisions of EN 1992-1-1:2004, including the plane sections assumption.

(2)P Normal stresses in the concrete shall be limited, to prevent out-of-plane instability of the wall.

(3) The requirement of **(2)**P of this subclause may be satisfied on the basis of the rules of EN 1992-1-1:2004 for second-order effects, supplemented with other rules for the normal stresses in the concrete if necessary.

(4) When the dynamic axial force of **5.4.2.5(3)**P and **(4)** is taken into account in the ULS verification for bending with axial force, the limiting strain $\varepsilon_{cu2,c}$ for unconfined concrete may be increased to 0,005. A higher value may be taken into account for confined concrete, in accordance with EN 1992-1-1:2004, **3.1.9**, provided that spalling of the unconfined concrete cover is accounted for in the verification.

5.4.3.5.2 Shear resistance

(1) Due to the safety margin provided by the magnification of design shear forces in **5.4.2.5(1)**P and **(2)** and because the response (including possible inclined cracking) is deformation-controlled, wherever the value of V_{Ed} from **5.4.2.5(2)** is less than the design value of the shear resistance $V_{Rd,c}$ in EN 1992-1-1:2004, **6.2.2**, the minimum shear reinforcement ratio $\rho_{w,min}$ in the web is not required.

NOTE The value ascribed to $\rho_{w,min}$ for use in a country may be found in its National Annex to this document. The recommended value is the minimum value for walls in EN 1992-1-1:2004 and in its National Annex.

(2) Wherever the condition $V_{Ed} \leq V_{Rd,c}$ is not fulfilled, web shear reinforcement should be calculated in accordance with EN 1992-1-1:2004, on the basis of a variable inclination truss model, or a strut-and-tie model, whichever is most appropriate for the particular geometry of the wall.

(3) If a strut-and-tie model is used, the width of the strut should take into account the presence of openings and should not exceed $0{,}25l_w$ or $4b_{wo}$, whichever is smaller.

(4) The ULS against sliding shear at horizontal construction joints should be verified in accordance with EN 1992-1-1:2004, **6.2.5**, with the anchorage length of clamping bars crossing the interface increased by 50% over that required by EN 1992-1-1:2004.

5.4.3.5.3 Detailing for local ductility

(1) Vertical bars necessary for the verification of the ULS in bending with axial force, or for the satisfaction of any minimum reinforcement provisions, should be engaged by a hoop or a cross-tie with a diameter of not less than 6 mm or one third of the vertical bar diameter, d_{bL}. Hoops and cross-ties should be at a vertical spacing of not more than 100 mm or $8d_{bL}$, whichever is less.

(2) Vertical bars necessary for the verification of the ULS in bending with axial force and laterally restrained by hoops and cross-ties in accordance with **(1)** of this subclause should be concentrated in boundary elements at the ends of the cross-section. These elements should extend in the direction of the length l_w of the wall over a length not less than b_w or $3b_w\sigma_{cm}/f_{cd}$, whichever is less, where σ_{cm} is the mean value of the concrete stress in the compression zone in the ULS of bending with axial force. The diameter of the vertical bars should not be less than 12 mm in the lower storey of the building, or in any storey where the length l_w of the wall is reduced over that of the

storey below by more than one-third of the storey height h_s. In all other storeys the diameter of vertical bars should not be less than 10 mm.

(3) To avoid a change in the mode of behaviour from one controlled by flexure to another controlled by shear, the amount of vertical reinforcement placed in the wall section should not unnecessarily exceed the amount required for the verification of the ULS in flexure with axial load and for the integrity of concrete.

(4) Continuous steel ties, horizontal or vertical, should be provided: (a) along all intersections of walls or connections with flanges; (b) at all floor levels; and (c) around openings in the wall. As a minimum, these ties should satisfy EN 1992-1-1:2004, **9.10**.

5.5 Design for DCH

5.5.1 Geometrical constraints and materials

5.5.1.1 Material requirements

(1)P A concrete class lower than C 20/25 shall not be used in primary seismic elements.

(2)P The requirement specified in paragraph **5.4.1.1(2)**P applies to this subclause.

(3)P In critical regions of primary seismic elements, reinforcing steel of class C in Table C.1 of EN 1992-1-1:2004 shall be used. Moreover, the upper characteristic (95%-fractile) value of the actual yield strength, $f_{yk,0,95}$, shall not exceed the nominal value by more than 25%.

5.5.1.2 Geometrical constraints

5.5.1.2.1 Beams

(1)P The width of primary seismic beams shall be not less than 200 mm.

(2)P The width to height ratio of the web of primary seismic beams shall satisfy expression (5.40b) of EN 1992-1-1:2004.

(3)P Paragraph **5.4.1.2.1(1)**P applies.

(4) Paragraph **5.4.1.2.1(2)** applies.

(5)P Paragraph **5.4.1.2.1(3)**P applies.

5.5.1.2.2 Columns

(1)P The minimum cross-sectional dimension of primary seismic columns shall be not less than 250 mm.

(2) Paragraph **5.4.1.2.2(1)** applies.

5.5.1.2.3 Ductile Walls

(1)P The provisions cover single primary seismic walls, as well as individual components of coupled primary seismic walls, under in-plane action effects, with full embedment and anchorage at their base in adequate basements and foundations, so that the wall is not allowed to rock. In this respect, walls supported by slabs or beams are not permitted (see also **5.4.1.2.5**).

(2) Paragraph **5.4.1.2.3(1)** applies.

(3) Additional requirements apply with respect to the thickness of the confined boundary elements of primary seismic walls, as specified in **5.5.3.4.5(8)** and **(9)**.

(4) Random openings, not regularly arranged to form coupled walls, should be avoided in primary seismic walls, unless their influence is either insignificant or accounted for in analysis, dimensioning and detailing.

5.5.1.2.4 Specific rules for beams supporting discontinued vertical elements

(1)P Paragraph **5.4.1.2.5(1)**P applies.

(2)P Paragraph **5.4.1.2.5(2)**P applies.

5.5.2 Design action effects

5.5.2.1 Beams

(1)P Paragraph **5.4.2.1(1)**P applies for the design values of bending moments and axial forces.

(2)P Paragraph **5.4.2.2(1)**P applies.

(3) Paragraph **5.4.2.2(2)** applies with a value $\gamma_{Rd} = 1,2$ in expression (5.8).

5.5.2.2 Columns

(1) Paragraph **5.4.2.1(1)**P (which refers also to the capacity design requirements in **5.2.3.3(2)**) applies for the design values of bending moments and axial forces.

(2)P Paragraph **5.4.2.3(1)**P applies.

(3) Paragraph **5.4.2.3(2)** applies with a value $\gamma_{Rd} = 1,3$ in expression (5.9).

(4) Paragraph **5.4.2.3(3)** applies.

5.5.2.3 Beam-column joints

(1)P The horizontal shear acting around the core of a joint between primary seismic beams and columns shall be determined taking into account the most adverse conditions under seismic loading, i.e. capacity design conditions for the beams framing into the joint and the lowest compatible values of shear forces in the framing elements.

(2) Simplified expressions for the horizontal shear force acting on the concrete core of the joints may be used as follows:

a) for interior beam-column joints:

$$V_{\mathrm{jhd}} = \gamma_{\mathrm{Rd}} (A_{\mathrm{s1}} + A_{\mathrm{s2}}) f_{\mathrm{yd}} - V_{\mathrm{C}} \tag{5.22}$$

b) for exterior beam-column joints:

$$V_{\mathrm{jhd}} = \gamma_{\mathrm{Rd}} \cdot A_{\mathrm{s1}} \cdot f_{\mathrm{yd}} - V_{\mathrm{C}} \tag{5.23}$$

where

A_{s1} is the area of the beam top reinforcement;

A_{s2} is the area of the beam bottom reinforcement;

V_C is the column shear force, from the analysis in the seismic design situation;

γ_{Rd} is a factor to account for overstrength due to steel strain-hardening and should be not less than 1,2.

(3) The shear forces acting on the joints shall correspond to the most adverse direction of the seismic action influencing the values A_{s1}, A_{s2} and V_C to be used in expressions (5.22) and (5.23).

5.5.2.4 Ductile Walls

5.5.2.4.1 Special provisions for in-plane slender walls

(1)P Paragraph **5.4.2.4(1)**P applies.

(2) Paragraph **5.4.2.4(2)** applies.

(3) Paragraph **5.4.2.4(3)** applies.

(4)P Paragraph **5.4.2.4(4)**P applies.

(5) Paragraph **5.4.2.4(5)** applies.

(6)P Paragraph **5.4.2.4(6)**P applies.

(7) The requirement of **(6)**P is deemed to be satisfied if the following simplified procedure is applied, incorporating the capacity design rule:

The design shear forces V_{Ed} should be derived in accordance with the expression:

$$V_{\mathrm{Ed}} = \varepsilon \cdot V_{\mathrm{Ed}}' \tag{5.24}$$

where

V'_{Ed} is the shear force from the analysis;

ε is the magnification factor, calculated from expression (5.25), but not less than 1,5:

$$\varepsilon = q \cdot \sqrt{\left(\frac{\gamma_{\mathrm{Rd}}}{q} \cdot \frac{M_{\mathrm{Rd}}}{M_{\mathrm{Ed}}}\right)^2 + 0{,}1\left(\frac{S_{\mathrm{e}}(T_{\mathrm{C}})}{S_{\mathrm{e}}(T_1)}\right)^2} \leq q \tag{5.25}$$

where

q is the behaviour factor used in the design;

M_{Ed} is the design bending moment at the base of the wall;

M_{Rd} is the design flexural resistance at the base of the wall;

γ_{Rd} is the factor to account for overstrength due to steel strain-hardening; in the absence of more precise data, γ_{Rd} may be taken equal to 1,2;

T_1 is the fundamental period of vibration of the building in the direction of shear forces V_{Ed};

T_{C} is the upper limit period of the constant spectral acceleration region of the spectrum (see **3.2.2**);

$S_{\mathrm{e}}(T)$ is the ordinate of the elastic response spectrum (see **3.2.2**).

(8) The provisions of **5.4.2.4(8)** apply to slender walls of DCH.

5.5.2.4.2 Special provisions for squat walls

(1)P In primary seismic walls with a height to length ratio, $h_{\mathrm{w}}/l_{\mathrm{w}}$, not greater than 2,0, there is no need to modify the bending moments from the analysis. Shear magnification due to dynamic effects may also be neglected.

(2) The shear force V'_{Ed} from the analysis should be increased as follows:

$$V_{\mathrm{Ed}} = \gamma_{\mathrm{Rd}} \cdot \left(\frac{M_{\mathrm{Rd}}}{M_{\mathrm{Ed}}}\right) \cdot V'_{\mathrm{Ed}} \leq q \cdot V'_{\mathrm{Ed}} \tag{5.26}$$

(see **5.5.2.4.1(7)** for definitions and values of the variables).

5.5.3 ULS verifications and detailing

5.5.3.1 Beams

5.5.3.1.1 Resistance in bending

(1)P The bending resistance shall be computed in accordance with EN 1992-1-1:2004.

(2) Paragraph **5.4.3.1.1(2)** applies.

(3) Paragraph **5.4.3.1.1(3)** applies.

5.5.3.1.2 Shear resistance

(1)P The shear resistance computations and verifications shall be carried out in accordance with EN 1992-1-1:2004, unless specified otherwise in the following paragraphs.

(2)P In the critical regions of primary seismic beams, the strut inclination θ in the truss model shall be 45^{o}.

(3) With regard to the arrangement of shear reinforcement within the critical region at an end of a primary seismic beam where the beam frames into a column, the following cases should be distinguished, depending on the algebraic value of the ratio $\zeta = V_{Ed,min}/V_{Ed,max}$ between the minimum and maximum acting shear forces, as derived in accordance with **5.5.2.1(3).**

a) If $\zeta \geq -0,5$, the shear resistance provided by the reinforcement should be computed in accordance with EN 1992-1-1:2004.

b) If $\zeta < -0,5$, i.e. when an almost full reversal of shear forces is expected, then:

i) if $\left|V_{E}\right|_{max} \leq (2+\zeta)\cdot f_{ctd}\cdot b_{w}\cdot d$ (5.27)

where f_{ctd} is the design value of the concrete tensile strength from EN 1992-1-1:2004, the same rule as in a) of this paragraph applies.

ii) if $|V_{E}|_{max}$ exceeds the limit value in expression (5.27), inclined reinforcement should be provided in two directions, either at $\pm45^{o}$ to the beam axis or along the two diagonals of the beam in elevation, and half of $|V_{E}|_{max}$ should be resisted by stirrups and half by inclined reinforcement;

– In such a case, the verification is carried out by means of the condition:

$$0,5\,V_{Emax} \leq 2A_{s}\cdot f_{yd}\cdot \cos\alpha \tag{5.28}$$

where

A_{s} is the area of the inclined reinforcement in one direction, crossing the potential sliding plane (i.e. the beam end section);

α is the angle between the inclined reinforcement and the beam axis (normally $\alpha = 45^{o}$, or $\tan\alpha \approx (d\text{-}d')/l_{b}$).

5.5.3.1.3 Detailing for local ductility

(1)P The regions of a primary seismic beam up to a distance l_{cr}=1.5h_{w} (where h_{w} denotes the height of the beam) from an end cross-section where the beam frames into a beam-column joint, as well as from both sides of any other cross-section likely to yield in the seismic design situation, shall be considered critical regions.

(2) Paragraph **5.4.3.1.2(2)** applies.

(3)P Paragraph **5.4.3.1.2(3)**P applies.

(4) Paragraph **5.4.3.1.2(4)** applies.

(5)P To satisfy the necessary ductility conditions, along the entire length of a primary seismic beam the following conditions shall be satisfied:

a) paragraph **5.4.3.1.2(5)**P shall be satisfied

b) at least two high bond bars with d_b = 14 mm shall be provided both at the top and the bottom of the beam that run along the entire length of the beam;

c) one quarter of the maximum top reinforcement at the supports shall run along the entire beam length.

(6)P **5.4.3.1.2(6)**P applies with expression (5.13) replaced by the following:

$$s=\min\{h_w/4;\ 24d_{bw};\ 175;\ 6d_{bL}\}. \tag{5.29}$$

5.5.3.2 Columns

5.5.3.2.1 Resistances

(1)P Paragraph **5.4.3.2.1(1)**P applies.

(2) Paragraph **5.4.3.2.1(2)** applies.

(3)P In primary seismic columns the value of the normalised axial force ν_d shall not exceed 0,55.

5.5.3.2.2 Detailing for local ductility

(1)P Paragraph **5.4.3.2.2(1)**P applies.

(2)P Paragraph **5.4.3.2.2(2)**P applies.

(3)P Paragraph **5.4.3.2.2(3)**P applies.

(4) In the absence of more precise information, the length of the critical region l_{cr} may be computed as follows (in metres):

$$l_{cr} = \max\{1{,}5h_c;\ l_{cl}/6;\ 0{,}6\} \tag{5.30}$$

where

h_c is the largest cross-sectional dimension of the column (in metres); and

l_{cl} is its clear length (in metres).

(5)P Paragraph **5.4.3.2.2(5)**P applies.

(6)P Paragraph **5.4.3.2.2(6)**P applies.

(7) The detailing of critical regions above the base of the column should be based on a minimum value of the curvature ductility factor μ_ϕ (see **5.2.3.4**) obtained from

5.2.3.4(3). Wherever a column is protected against plastic hinging by the capacity design procedure of **4.4.2.3(4)** (i.e. where expression (4.29) is satisfied), the value q_o in expressions (5.4) and (5.5) may be substituted by 2/3 of the value of q_o applying in a direction parallel to the cross-sectional depth h_c of the column.

(8)P Paragraph **5.4.3.2.2(7)**P applies.

(9) The requirements of **(6)**P, **(7)** and **(8)**P of this subclause are deemed to be satisfied, if **5.4.3.2.2(8)** is satisfied with the values of μ_ϕ specified in **(6)**P and **(7)** of this subclause.

(10) The minimum value of ω_{wd} to be provided is 0,12 within the critical region at the base of the column, or 0,08 in all column critical regions above the base.

(11)P Paragraph **5.4.3.2.2(10)**P applies.

(12) The minimal conditions of **(11)**P of this subclause are deemed to be satisfied if all of the following requirements are met.

a) The diameter d_{bw} of the hoops is at least equal to

$$d_{bw} \geq 0{,}4 \cdot d_{bL,\ max} \cdot \sqrt{f_{ydL} / f_{ydw}} \tag{5.31}$$

b) The spacing s of hoops (in millimetres) does not exceed:

$$s = \min\{b_o / 3; 125; 6d_{bL}\} \tag{5.32}$$

where

b_o (in millimetres) is the minimum dimension of the concrete core (to the inside of the hoops); and

d_{bL} is the the minimum diameter of the longitudinal bars (in millimetres).

c) The distance between consecutive longitudinal bars restrained by hoops or cross-ties does not exceed 150 mm.

(13)P In the lower two storeys of buildings, hoops in accordance with **(11)**P and **(12)** of this subclause shall be provided beyond the critical regions for an additional length equal to half the length of these regions.

(14) The amount of longitudinal reinforcement provided at the base of the bottom storey column (i.e. where the column is connected to the foundation) should be not less than that provided at the top.

5.5.3.3 Beam-column joints

(1)P The diagonal compression induced in the joint by the diagonal strut mechanism shall not exceed the compressive strength of concrete in the presence of transverse tensile strains.

(2) In the absence of a more precise model, the requirement of **(1)**P of this subclause may be satisfied by means of the subsequent rules.

a) At interior beam-column joints the following expression should be satisfied:

$$V_{jhd} \leq \eta f_{cd} \sqrt{1-\frac{\nu_d}{\eta}} b_j \, h_c \qquad (5.33)$$

where

$\eta = 0,6(1-f_{ck}/250)$;

ν_d is the normalised axial force in the column above the joint; and

f_{ck} is given in MPa.

b) At exterior beam-column joints:

V_{jhd} should be less than 80% of the value given by the right-hand-side of expression (5.33) where:

V_{jhd} is given by expressions (5.22) and (5.23) respectively;

and the effective joint width b_j is:

a) if $b_c > b_w$: $b_j = \min\{b_c; (b_w + 0,5 \cdot h_c)\}$; (5.34a)

b) if $b_c < b_w$: $b_j = \min\{b_w; (b_c + 0,5 \cdot h_c)\}$ (5.34b)

(3) Adequate confinement (both horizontal and vertical) of the joint should be provided, to limit the maximum diagonal tensile stress of concrete max σ_{ct} to f_{ctd}. In the absence of a more precise model, this requirement may be satisfied by providing horizontal hoops with a diameter of not less than 6 mm within the joint, such that:

$$\frac{A_{sh} \cdot f_{ywd}}{b_j \cdot h_{jw}} \geq \frac{\left(\dfrac{V_{jhd}}{b_j \cdot h_{jc}}\right)^2}{f_{ctd} + \nu_d f_{cd}} - f_{ctd} \qquad (5.35)$$

where

A_{sh} is the total area of the horizontal hoops;

V_{jhd} is as defined in expressions (5.23) and (5.24);

h_{jw} is the distance between top of the beam and the reinforcement at the bottom of the beam;

h_{jc} is the distance between extreme layers of column reinforcement;

b_j is as defined in expression (5.34);

ν_d is the normalised design axial force of the column above ($\nu_d = N_{Ed}/A_c \cdot f_{cd}$);

f_{ctd} is the design value of the tensile strength of concrete, in accordance with EN 1992-1-1:2004.

(4) As an alternative to the rule specified in **(3)** of this subclause, integrity of the joint after diagonal cracking may be ensured by horizontal hoop reinforcement. To this end the following total area of horizontal hoops should be provided in the joint.

a) In interior joints:

$$A_{sh} f_{ywd} \geq \gamma_{Rd}(A_{s1}+A_{s2}) f_{yd}(1-0{,}8\nu_d) \tag{5.36a}$$

b) In exterior joints:

$$A_{sh} f_{ywd} \geq \gamma_{Rd} A_{s2} f_{yd}(1-0{,}8\nu_d) \tag{5.36b}$$

where γ_{Rd} is equal to 1,2 (cf **5.5.2.3(2)**) and the normalised axial force ν_d refers to the column above the joint in expression (5.36a), or to the column below the joint in expression (5.36b).

(5) The horizontal hoops calculated as in **(3)** and **(4)** of this subclause should be uniformly distributed within the depth h_{jw} between the top and bottom bars of the beam. In exterior joints they should enclose the ends of beam bars bent toward the joint.

(6) Adequate vertical reinforcement of the column passing through the joint should be provided, so that:

$$A_{sv,i} \geq (2/3) \cdot A_{sh} \cdot (h_{jc} / h_{jw}) \tag{5.37}$$

where A_{sh} is the required total area of the horizontal hoops in accordance with **(3)** and **(4)** of this subclause and $A_{sv,i}$ denotes the total area of the intermediate bars placed in the relevant column faces between corner bars of the column (including bars contributing to the longitudinal reinforcement of columns).

(7) **5.4.3.3(1)** applies.

(8) **5.4.3.3(2)** applies.

(9)P **5.4.3.3(3)**P applies.

5.5.3.4 Ductile Walls

5.5.3.4.1 Bending resistance

(1)P The bending resistance shall be evaluated and verified as for columns, under the most unfavourable axial force for the seismic design situation.

(2) In primary seismic walls the value of the normalised axial force ν_d should not exceed 0,35.

5.5.3.4.2 Diagonal compression failure of the web due to shear

(1) The value of $V_{Rd,max}$ may be calculated as follows:

a) outside the critical region:

as in EN 1992-1-1:2004, with the length of the internal lever arm, z, equal to $0{,}8l_w$ and the inclination of the compression strut to the vertical, $\tan\theta$, equal to 1,0.

b) in the critical region:

40% of the value outside the critical region.

5.5.3.4.3 Diagonal tension failure of the web due to shear

(1)P The calculation of web reinforcement for the ULS verification in shear shall take into account the value of the shear ratio $\alpha_s = M_{Ed}/(V_{Ed}\, l_w)$. The maximum value of α_s in a storey should be used for the ULS verification of the storey in shear.

(2) If the ratio $\alpha_s \geq 2{,}0$, the provisions of in EN 1992-1-1:2004 **6.2.3(1)-(7)** apply, with the values of z and $\tan\theta$ taken as in **5.5.3.4.2(1)** a).

(3) If $\alpha_s < 2{,}0$ the following provisions apply:

a) the horizontal web bars should satisfy the following expression (see EN 1992-1-1:2004, **6.2.3(8)**):

$$V_{Ed} \leq V_{Rd,c} + 0{,}75\rho_h f_{yd,h} b_{wo} \alpha_s l_w \tag{5.38}$$

where

ρ_h is the reinforcement ratio of horizontal web bars ($\rho_h = A_h/(b_{wo} \cdot s_h)$);

$f_{yd,h}$ is the design value of the yield strength of the horizontal web reinforcement;

$V_{Rd,c}$ is the design value of the shear resistance for members without shear reinforcement, in accordance to EN 1992-1-1:2004,

In the critical region of the wall $V_{Rd,c}$ should be equal to 0 if the axial force N_{Ed} is tensile.

b) Vertical web bars, anchored and spliced along the height of the wall in accordance with EN 1992-1-1:2004, should be provided to satisfy the condition:

$$\rho_h f_{yd,h} b_{wo} z \leq \rho_v f_{yd,v}\ b_{wo}\ z + \min N_{Ed} \tag{5.39}$$

where

ρ_v is the reinforcement ratio of vertical web bars ($\rho_v = A_v/b_{wo} \cdot s_v$);

$f_{yd,v}$ is the design value of the yield strength of the vertical web reinforcement;

and where the axial force N_{Ed} is positive when compressive.

(4) Horizontal web bars should be fully anchored at the ends of the wall section, e.g. through 90° or 135° hooks.

(5) Horizontal web bars in the form of elongated closed or fully anchored stirrups may also be assumed to fully contribute to the confinement of the boundary elements of the wall.

5.5.3.4.4 Sliding shear failure

(1)P At potential sliding shear planes (for example, at construction joints) within critical regions the following condition shall be satisfied:

$V_{\mathrm{Ed}} \leq V_{\mathrm{Rd,\,S}}$

where $V_{\mathrm{Rd,S}}$ is the design value of the shear resistance against sliding.

(2) The value of $V_{\mathrm{Rd,\,S}}$ may be as follows:

$$V_{\mathrm{Rd,S}} = V_{\mathrm{dd}} + V_{\mathrm{id}} + V_{\mathrm{fd}} \tag{5.40}$$

with:

$$V_{\mathrm{dd}} = \min \begin{cases} 1{,}3 \cdot \Sigma A_{\mathrm{sj}} \cdot \sqrt{f_{\mathrm{cd}} \cdot f_{\mathrm{yd}}} \\ 0{,}25 \cdot f_{\mathrm{yd}} \cdot \Sigma A_{\mathrm{sj}} \end{cases} \tag{5.41}$$

$$V_{\mathrm{id}} = \Sigma A_{\mathrm{si}} \cdot f_{\mathrm{yd}} \cdot \cos\varphi \tag{5.42}$$

$$V_{\mathrm{fd}} = \min \begin{cases} \mu_{\mathrm{f}} \cdot \left[\left(\Sigma A_{\mathrm{sj}} \cdot f_{\mathrm{yd}} + N_{\mathrm{Ed}} \right) \cdot \xi + M_{\mathrm{Ed}} / z \right] \\ 0{,}5\eta \cdot f_{\mathrm{cd}} \cdot \xi \cdot l_{\mathrm{w}} \cdot b_{\mathrm{wo}} \end{cases} \tag{5.43}$$

where

V_{dd} is the dowel resistance of the vertical bars;

V_{id} is the shear resistance of inclined bars (at an angle φ to the potential sliding plane, e.g. construction joint);

V_{fd} is the friction resistance;

μ_{f} is the concrete-to-concrete friction coefficient under cyclic actions, which may be assumed equal to 0,6 for smooth interfaces and to 0,7 for rough ones, as defined in EN 1992-1-1:2004, **6.2.5**;

z is the length of the internal lever arm;

ξ is the normalised neutral axis depth;

ΣA_{sj} is the sum of the areas of the vertical bars of the web or of additional bars arranged in the boundary elements specifically for resistance against sliding;

ΣA_{si} is the sum of the areas of all inclined bars in both directions; large diameter bars are recommended for this purpose;

$$\eta = 0{,}6\ (1 - f_{\mathrm{ck}}(\mathrm{MPa})/250) \tag{5.44}$$

N_{Ed} is assumed to be positive when compressive.

(3) For squat walls the following should be satisfied :

a) at the base of the wall V_{id} should be greater than $V_{Ed}/2$;

b) at higher levels V_{id} should be greater than $V_{Ed}/4$.

(4) Inclined bars should be fully anchored on both sides of potential sliding interfaces and should cross all sections of the wall within a distance of $0{,}5\cdot l_w$ or $0{,}5\cdot h_w$, whichever is smaller, above the critical base section.

(5) Inclined bars lead to an increase of the bending resistance at the base of the wall, which should be taken into account whenever the acting shear V_{Ed} is computed in accordance with the capacity design rule (see **5.5.2.4.1(6)**P and (7) and **5.5.2.4.2(2)**). Two alternative methods may be used.

a) The increase of bending resistance ΔM_{Rd}, to be used in the calculation of V_{Ed}, may be estimated as:

$$\Delta M_{Rd} = \frac{1}{2} \cdot \Sigma A_{si} \cdot f_{yd} \cdot \sin\varphi \cdot l_i \tag{5.45}$$

where

l_i is the distance between centrelines of the two sets of inclined bars, placed at an angle of $\pm\phi$ to the potential sliding plane, measured at the base section;

and the other symbols are as in expression (5.42).

b) An acting shear V_{Ed} may be computed disregarding the effect of the inclined bars. In expression (5.42) V_{id} is the net shear resistance of the inclined bars (i.e. the actual shear resistance reduced by the increase of the acting shear). Such net shear resistance of the inclined bars against sliding may be estimated as:

$$V_{id} = \Sigma A_{si} \cdot f_{yd} \cdot \left[\cos\varphi - 0{,}5 \cdot l_i \cdot \sin\varphi / \left(\alpha_s \cdot l_w\right)\right] \tag{5.46}$$

5.5.3.4.5 Detailing for local ductility

(1) Paragraph **5.4.3.4.2(1)** applies.

(2) Paragraph **5.4.3.4.2(2)** applies.

(3) Paragraph **5.4.3.4.2(3)** applies.

(4) Paragraph **5.4.3.4.2(4)** applies.

(5) Paragraph **5.4.3.4.2(5)** applies.

(6) Paragraph **5.4.3.4.2(6)** applies.

(7) Paragraph **5.4.3.4.2(8)** applies.

(8) Paragraph **5.4.3.4.2(10)** applies.

(9) If the wall is connected to a flange with thickness $b_f \geq h_s/15$ and width $l_f \geq h_s/5$ (where h_s denotes the clear storey height), and the confined boundary element needs to extend beyond the flange into the web for an additional length of up to $3b_{wo}$, then the thickness b_w of the boundary element in the web should only follow the provisions in **5.4.1.2.3(1)** for b_{wo} (Figure 5.11).

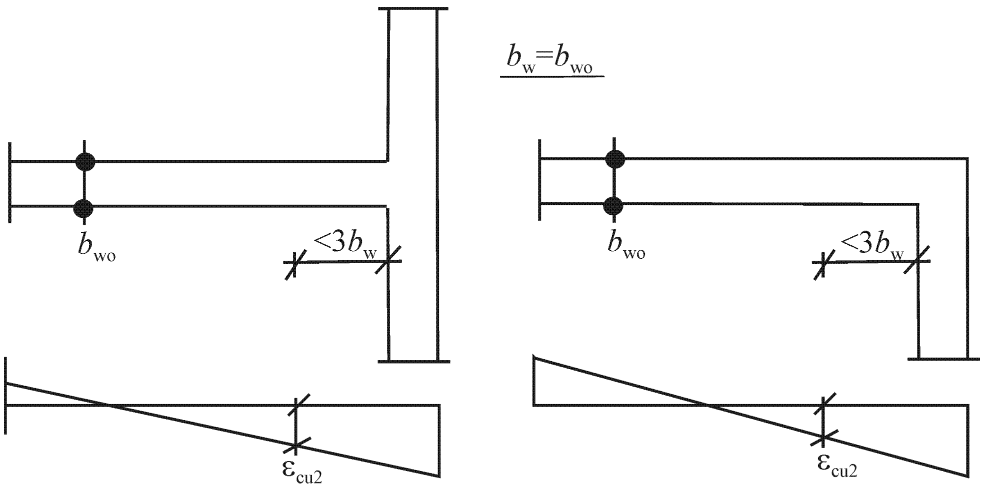


Figure 5.11: Minimum thickness of confined boundary elements in DCH walls with large flanges

(10) Within the boundary elements of walls the requirements specified in **5.5.3.2.2(12)** apply and there should be a minimum value of ω_{wd} of 0,12. Overlapping hoops should be used, so that every other longitudinal bar is engaged by a hoop or cross-tie.

(11) Above the critical region boundary elements should be provided for one more storey, with at least half the confining reinforcement required in the critical region.

(12) **5.4.3.4.2(11)** applies.

(13)P Premature web shear cracking of walls shall be prevented, by providing a minimum amount of web reinforcement: $\rho_{h,min} = \rho_{v,min} = 0,002$.

(14) The web reinforcement should be provided in the form of two grids (curtains) of bars with the same bond characteristics, one at each face of the wall. The grids should be connected through cross-ties spaced at about 500 mm.

(15) Web reinforcement should have a diameter of not less than 8 mm, but not greater than one-eighth of the width b_{wo} of the web. It should be spaced at not more than 250 mm or 25 times the bar diameter, whichever is smaller.

(16) To counterbalance the unfavourable effects of cracking along cold joints and the associated uncertainties, a minimum amount of fully anchored reinforcement should be provided across such joints. The minimum ratio of this reinforcement, ρ_{min}, necessary to re-establish the resistance of uncracked concrete against shear, is:

$$\rho_{\min} \geq \begin{cases} \left(1{,}3 \cdot f_{\text{ctd}} - \dfrac{N_{\text{Ed}}}{A_{\text{w}}}\right) / \left(f_{\text{yd}} \cdot \left(1 + 1{,}5\sqrt{f_{\text{cd}} / f_{\text{yd}}}\right)\right) \\ 0{,}0025 \end{cases} \tag{5.47}$$

where A_w is the total horizontal cross-sectional area of the wall and N_{Ed} shall be positive when compressive.

5.5.3.5 Coupling elements of coupled walls

(1)P Coupling of walls by means of slabs shall not be taken into account, as it is not effective.

(2) The provisions of **5.5.3.1** may only be applied to coupling beams, if either one of the following conditions is fulfilled:

a) Cracking in both diagonal directions is unlikely. An acceptable application rule is:

$$V_{\text{Ed}} \leq f_{\text{ctd}} b_{\text{w}} d \tag{5.48}$$

b) A prevailing flexural mode of failure is ensured. An acceptable application rule is: $l/h \geq 3$.

(3) If neither of the conditions in **(2)** is met, the resistance to seismic actions should be provided by reinforcement arranged along both diagonals of the beam, in accordance with the following (see Figure 5.12):

a) It should be ensured that the following expression is satisfied:

$$V_{\text{Ed}} \leq 2 \cdot A_{\text{si}} \cdot f_{\text{yd}} \cdot \sin\alpha \tag{5.49}$$

where

V_{Ed} is the design shear force in the coupling element ($V_{Ed} = 2 \cdot M_{Ed}/l$);

A_{si} is the total area of steel bars in each diagonal direction;

α is the angle between the diagonal bars and the axis of the beam.

b) The diagonal reinforcement should be arranged in column-like elements with side lengths at least equal to $0{,}5b_w$; its anchorage length should be 50% greater than that required by EN 1992-1-1:2004.

c) Hoops should be provided around these column-like elements to prevent buckling of the longitudinal bars. The provisions of **5.5.3.2.2(12)** apply for the hoops..

d) Longitudinal and transverse reinforcement should be provided on both lateral faces of the beam, meeting the minimum requirements specified in EN 1992-1-1:2004 for deep beams. The longitudinal reinforcement should not be anchored in the coupled walls and should only extend into them by 150 mm.

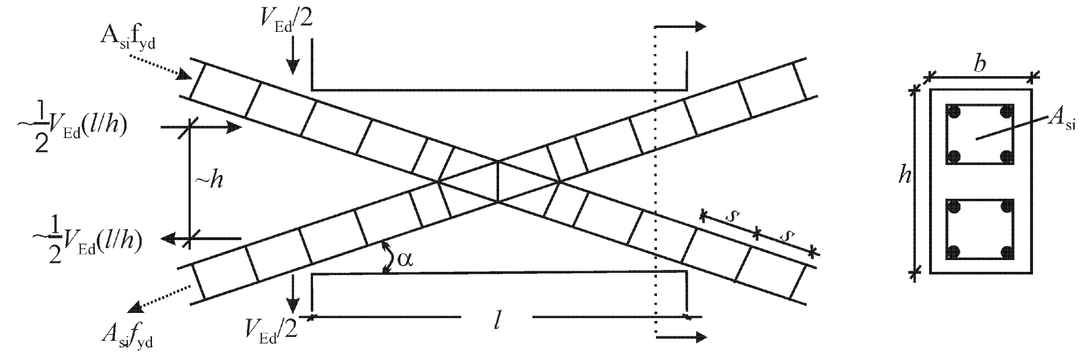


Figure 5.12: Coupling beams with diagonal reinforcement

5.6 Provisions for anchorages and splices

5.6.1 General

(1)P EN 1992-1-1:2004, Section **8** for the detailing of reinforcement applies, with the additional rules of the following sub-clauses.

(2)P For hoops used as transverse reinforcement in beams, columns or walls, closed stirrups with 135° hooks and extensions of length $10d_{bw}$ shall be used.

(3)P In DCH structures the anchorage length of beam or column bars anchored within beam-column joints shall be measured from a point on the bar at a distance $5d_{bL}$ inside the face of the joint, to take into account the yield penetration due to cyclic post-elastic deformations (for a beam example, see Figure 5.13a).

5.6.2 Anchorage of reinforcement

5.6.2.1 Columns

(1)P When calculating the anchorage or lap length of column bars which contribute to the flexural strength of elements in critical regions, the ratio of the required area of reinforcement over the actual area of reinforcement $A_{s,req}/A_{s,prov}$ shall be assumed to be 1.

(2)P If, under the seismic design situation, the axial force in a column is tensile, the anchorage lengths shall be increased to 50% longer than those specified in EN 1992-1-1:2004.

5.6.2.2 Beams

(1)P The part of beam longitudinal reinforcement bent in joints for anchorage shall always be placed inside the corresponding column hoops.

(2)P To prevent bond failure the diameter of beam longitudinal bars passing through beam-column joints, d_{bL}, shall be limited in accordance with the following expressions:

a) for interior beam-column joints:

$$\frac{d_{bL}}{h_c} \leq \frac{7{,}5 \cdot f_{ctm}}{\gamma_{Rd} \cdot f_{yd}} \cdot \frac{1+0{,}8 \cdot \nu_d}{1+0.75 k_D \cdot \rho' / \rho_{max}} \tag{5.50a}$$

b) for exterior beam-column joints:

$$\frac{d_{bL}}{h_c} \leq \frac{7{,}5 \cdot f_{ctm}}{\gamma_{Rd} \cdot f_{yd}} \cdot \left(1+0{,}8 \cdot \nu_d\right) \tag{5.50b}$$

where

h_c is the width of the column parallel to the bars;

f_{ctm} is the mean value of the tensile strength of concrete;

f_{yd} is the design value of the yield strength of steel;

ν_d is the normalised design axial force in the column, taken with its minimum value for the seismic design situation ($\nu_d = N_{Ed}/f_{cd} \cdot A_c$);

k_D is the factor reflecting the ductility class equal to 1 for DCH and to 2/3 for DCM;

ρ' is the compression steel ratio of the beam bars passing through the joint;

ρ_{max} is the maximum allowed tension steel ratio (see **5.4.3.1.2(4)** and **5.5.3.1.3(4)**);

γ_{Rd} is the model uncertainty factor on the design value of resistances, taken as being equal to1,2 or 1,0 respectively for DCH or DCM (due to overstrength owing to strain-hardening of the longitudinal steel in the beam).

The limitations above (expressions (5.50)) do not apply to diagonal bars crossing joints.

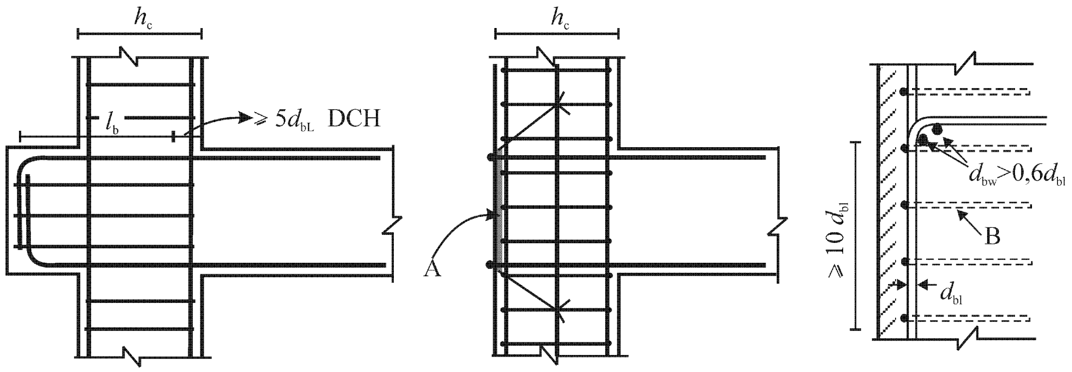
(3) If the requirement specified in **(2)**P of this clause cannot be satisfied in exterior beam-column joints because the depth, h_c, of the column parallel to the bars is too shallow, the following additional measures may be taken, to ensure anchorage of the longitudinal reinforcement of beams.

a) The beam or slab may be extended horizontally in the form of exterior stubs (see Figure 5.13a).

b) Headed bars or anchorage plates welded to the end of the bars may be used (see Figure 5.13b).

c) Bends with a minimum length of $10d_{bL}$ and transverse reinforcement placed tightly inside the bend of a group of bars may be added(see Figure 5.13c).

(4)P Top or bottom bars passing through interior joints, shall terminate in the members framing into the joint at a distance not less than l_{cr} (length of the member critical region, see **5.4.3.1.2(1)**P and **5.5.3.1.3(1)**P) from the face of the joint.



a) b) c)

Key

A anchor plate;

B hoops around column bars

Figure 5.13: Additional measures for anchorage in exterior beam-column joints

5.6.3 Splicing of bars

(1)P There shall be no lap-splicing by welding within the critical regions of structural elements.

(2)P There may be splicing by mechanical couplers in columns and walls, if these devices are covered by appropriate testing under conditions compatible with the selected ductility class.

(3)P The transverse reinforcement to be provided within the lap length shall be calculated in accordance with EN 1992-1-1:2004. In addition, the following requirements shall also be met.

a) If the anchored and the continuing bar are arranged in a plane parallel to the transverse reinforcement, the sum of the areas of all spliced bars, ΣA_{sL}, shall be used in the calculation of the transverse reinforcement.

b) If the anchored and the continuing bar are arranged within a plane normal to the transverse reinforcement, the area of transverse reinforcement shall be calculated on the basis of the area of the larger lapped longitudinal bar, A_{sL};

c) The spacing, s, of the transverse reinforcement in the lap zone (in millimetres) shall not exceed

$$s = \min\left\{h/4;100\right\} \tag{5.51}$$

where h is the minimum cross-sectional dimension (in millimetres).

(4) The required area of transverse reinforcement A_{st} within the lap zone of the longitudinal reinforcement of columns spliced at the same location (as defined in EN

1992-1-1:2004), or of the longitudinal reinforcement of boundary elements in walls, may be calculated from the following expression:

$$A_{st} = s\,(d_{bl}/50)(f_{yld}/f_{ywd}) \tag{5.52}$$

where

A_{st} is the area of one leg of the transverse reinforcement;

d_{bL} is the diameter of the spliced bar;

s is the spacing of the transverse reinforcement;

f_{yld} is the design value of the yield strength of the longitudinal reinforcement;

f_{ywd} is the design value of the yield strength of the transverse reinforcement.

5.7 Design and detailing of secondary seismic elements

(1)P Clause **5.7** applies to elements designated as secondary seismic elements, which are subjected to significant deformations in the seismic design situation (e.g. slab ribs are not subject to the requirements of **5.7**). Such elements shall be designed and detailed to maintain their capacity to support the gravity loads present in the seismic design situation, when subjected to the maximum deformations under the seismic design situation.

(2)P Maximum deformations due to the seismic design situation shall be calculated in accordance with **4.3.4** and shall account for P-Δ effects in accordance with **4.4.2.2(2)** and **(3)**. They shall be calculated from an analysis of the structure in the seismic design situation, in which the contribution of secondary seismic elements to lateral stiffness is neglected and primary seismic elements are modelled with their cracked flexural and shear stiffness.

(3) Secondary seismic elements are deemed to satisfy the requirements of **(1)**P of this subclause if bending moments and shear forces calculated for them on the basis of: a) the deformations of **(2)**P of this subclause; and b) their cracked flexural and shear stiffness, do not exceed their design flexural and shear resistance M_{Rd} and V_{Rd}, respectively, as these are determined on the basis of EN 1992-1-1:2004.

5.8 Concrete foundation elements

5.8.1 Scope

(1)P The following paragraphs apply for the design of concrete foundation elements, such as footings, tie-beams, foundation beams, foundation slabs, foundation walls, pile caps and piles, as well as for connections between such elements, or between them and vertical concrete elements. The design of these elements shall follow the rules of EN 1998-5:2004, **5.4**.

(2)P If design action effects for the design of foundation elements of dissipative structures are derived on the basis of capacity design considerations in accordance with **4.4.2.6(2)**P, no energy dissipation is expected in these elements in the seismic design situation. The design of these elements may follow the rules of **5.3.2(1)**P.

(3)P If design action effects for foundation elements of dissipative structures are derived on the basis of the analysis for the seismic design situation without the capacity design considerations of **4.4.2.6(2)**P, the design of these elements shall follow the corresponding rules for elements of the superstructure for the selected ductility class. For tie-beams and foundation beams the design shear forces need to be derived on the basis of capacity design considerations, in accordance with **5.4.2.2** in DCM buildings, or to **5.5.2.1(2)**P, **5.5.2.1(3)** in DCH buildings.

(4) If design action effects for foundation elements have been derived using a value of the behaviour factor q that is less than or equal to the upper limit of q for low dissipative behaviour (1,5 in concrete buildings, or between 1,5 and 2,0 in steel or composite steel-concrete buildings, in accordance with Note 1 of Table 6.1 or Note 1 of Table 7.1, respectively), the design of these elements may follow the rules of **5.3.2(1)**P (see also **4.4.2.6(3)**).

(5) In box-type basements of dissipative structures, comprising: a) a concrete slab acting as a rigid diaphragm at basement roof level; b) a foundation slab or a grillage of tie-beams or foundation beams at foundation level, and c) peripheral and/or interior foundation walls, designed in accordance with **(2)**P of this subclause, the columns and beams (including those at the basement roof) are expected to remain elastic under the seismic design situation and may be designed in accordance with **5.3.2(1)**P. Shear walls should be designed for plastic hinge development at the level of the basement roof slab. To this end, in walls which continue with the same cross-section above the basement roof, the critical region should be taken to extend below the basement roof level up to a depth of h_{cr} (see **5.4.3.4.2(1)** and **5.5.3.4.5(1)**). Moreover, the full free height of such walls within the basement should be dimensioned in shear assuming that the wall develops its flexural overstrength $\gamma_{Rd}.M_{Rd}$ (with γ_{Rd}=1,1 for DCM and γ_{Rd}=1,2 for DCH) at the basement roof level and zero moment at the foundation level.

5.8.2 Tie-beams and foundation beams

(1)P Stub columns between the top of a footing or pile cap and the soffit of tie-beams or foundation slabs shall be avoided. To this end, the soffit of tie-beams or foundation slabs shall be below the top of the footing or the pile cap.

(2) Axial forces in tie-beams or tie-zones of foundation slabs in accordance with **5.4.1.2(6)** and **(7)** of EN 1998-5, should be taken in the verification to act together with the action effects derived in accordance with **4.4.2.6(2)**P or **4.4.2.6(3)** for the seismic design situation, taking into account second-order effects.

(3) Tie-beams and foundation beams should have a cross-sectional width of at least $b_{w,min}$ and a cross-sectional depth of at least $h_{w,min}$.

NOTE The values ascribed to $b_{w,min}$ and $h_{w,min}$ for use in a country may be found in its National Annex to this document. The recommended values are: $b_{w,min}$ = 0,25 m and $h_{w,min}$ = 0,4 m for buildings with up to three storeys, or $h_{w,min}$ = 0,5 m for those with four storeys or more above the basement.

(4) Foundation slabs arranged in accordance with EN 1998-5:2004, **5.4.1.2(2)** for the horizontal connection of individual footings or pile caps, should have a thickness of at least t_{min} and a reinforcement ratio of at least $\rho_{s,min}$ at the top and bottom.

NOTE The values ascribed to t_{min} and $\rho_{s,min}$ for use in a country may be found in its National Annex to this document. The recommended values are: t_{min} = 0,2 m and $\rho_{s,min}$ = 0.2%.

(5) Tie-beams and foundation beams should have along their full length a longitudinal reinforcement ratio of at least $\rho_{b,min}$ at both the top and the bottom.

NOTE The value ascribed to $\rho_{b,min}$ for use in a country may be found in its National Annex to this document. The recommended value of $\rho_{b,min}$ is 0.4%.

5.8.3 Connections of vertical elements with foundation beams or walls

(1)P The common (joint) region of a foundation beam or foundation wall and a vertical element shall follow the rules of **5.4.3.3** or **5.5.3.3** as a beam-column joint region.

(2) If a foundation beam or foundation wall of a DCH structure is designed for action effects derived on the basis of capacity design considerations in accordance with **4.4.2.6(2)**P, the horizontal shear force V_{jhd} in the joint region is derived on the basis of analysis results in accordance with **4.4.2.6(2)**P, **(4)**, **(5)**, and **(6)**.

(3) If the foundation beam or foundation wall of a DCH structure is not designed in accordance with the capacity design approach of **4.4.2.6(4)**, **(5)**, **(6)** (see **5.8.1(3)**P), the horizontal shear force V_{jhd} in the joint region is determined in accordance with **5.5.2.3(2)**, expressions (5.22), (5.23), for beam-column joints.

(4) In DCM structures the connection of foundation beams or foundation walls with vertical elements may follow the rules of **5.4.3.3**.

(5) Bents or hooks at the bottom of longitudinal bars of vertical elements should be oriented so that they induce compression into the connection area.

5.8.4 Cast-in-place concrete piles and pile caps

(1)P The top of the pile up to a distance to the underside of the pile cap of twice the pile cross-sectional dimension, *d*, as well as the regions up to a distance of 2*d* on each side of an interface between two soil layers with markedly different shear stiffness (ratio of shear moduli greater than 6), shall be detailed as potential plastic hinge regions. To this end, they shall be provided with transverse and confinement reinforcement following the rules for column critical regions of the corresponding ductility class or of at least DCM.

(2)P When the requirement specified in **5.8.1(3)**P is applied for the design of piles of dissipative structures, piles shall be designed and detailed for potential plastic hinging at the head. To this end, the length over which increased transverse and confinement reinforcement is required at the top of the pile in accordance with **(1)**P of this subclause is increased by 50%. Moreover, the ULS verification of the pile in shear shall use a design shear force at least equal to that computed on the basis of **4.4.2.6(4)** to **(8)**.

(3) Piles required to resist tensile forces or assumed as rotationally fixed at the top, should be provided with anchorage in the pile cap to enable the development of the pile design uplift resistance in the soil, or of the design tensile strength of the pile

reinforcement, whichever is lower. If the part of such piles embedded in the pile cap is cast before the pile cap, dowels should be provided at the interface where the connection occurs.

5.9 Local effects due to masonry or concrete infills

(1) Because of the particular vulnerability of the infill walls of ground floors, a seismically induced irregularity is to be expected there and appropriate measures should be taken. If a more precise method is not used, the entire length of the columns of the ground floor should be considered as the critical length and confined accordingly.

(2) If the height of the infills is smaller than the clear length of the adjacent columns, the following measures should be taken:

a) the entire length of the columns is considered as critical region and should be reinforced with the amount and pattern of stirrups required for critical regions;

b) The consequences of the decrease of the shear span ratio of those columns should be appropriately covered. To this end, **5.4.2.3** and **5.5.2.2** should be applied for the calculation of the acting shear force, depending on the ductility class. In this calculation the clear length of the column, $l_{cl,}$ should be taken equal to the length of the column not in contact with the infills and themoment $M_{i,d}$ at the column section at the top of the infill wall should be taken as being equal to $\gamma_{Rd}.M_{Rc,i}$ with γ_{Rd} =1,1 for DCM and 1,3 for DCH and $M_{Rc,i}$ the design value of the moment of resistance of the column;

c) the transverse reinforcement to resist this shear force should be placed along the length of the column not in contact with the infills and extend along a length h_c (dimension of the column cross-section in the plane of the infill) into the column part in contact with the infills;

d) if the length of the column not in contact with the infills is less than 1,5h_c, the shear force should be resisted by diagonal reinforcement.

(3) Where the infills extend to the entire clear length of the adjacent columns, and there are masonry walls on only one side of the column (e.g. corner columns), the entire length of the column should be considered as a critical region and be reinforced with the amount and pattern of stirrups required for critical regions.

(4) The length, $l_{c,}$ of columns over which the diagonal strut force of the infill is applied, should be verified in shear for the smaller of the following two shear forces: a) the horizontal component of the strut force of the infill, assumed to be equal to the horizontal shear strength of the panel, as estimated on the basis of the shear strength of bed joints; or b) the shear force computed in accordance with **5.4.2.3** or **5.5.2.2**, depending on the ductility class, assuming that the overstrength flexural capacity of the column, $\gamma_{Rd}.M_{Rc,i}$, develops at the two ends of the contact length, l_c. The contact length should be assumed to be equal to the full vertical width of the diagonal strut of the infill. Unless a more accurate estimation of this width is made, taking into account the elastic properties and the geometry of the infill and the column, the strut width may be assumed to be a fixed fraction of the length of the panel diagonal.

5.10 Provisions for concrete diaphragms

(1) A solid reinforced concrete slab may be considered to serve as a diaphragm, if it has a thickness of not less than 70 mm and is reinforced in both horizontal directions with at least the minimum reinforcement specified in EN 1992-1-1:2004.

(2) A cast-in-place topping on a precast floor or roof system may be considered as a diaphragm, if: a) it meets the requirements of **(1)** of this subclause; b) it is designed to provide alone the required diaphragm stiffness and resistance; and c) it is cast over a clean, rough substrate, or connected to it through shear connectors.

(3)P The seismic design shall include the ULS verification of reinforced concrete diaphragms in DCH structures with the following properties:

– irregular geometries or divided shapes in plan, diaphragms with recesses and re-entrances;

– irregular and large openings in the diaphragm;

– irregular distribution of masses and/or stiffnesses (as e.g. in the case of set-backs or off-sets);

– basements with walls located only in part of the perimeter or only in part of the ground floor area;

(4) Action-effects in reinforced concrete diaphragms may be estimated by modelling the diaphragm as a deep beam or a plane truss or strut-and-tie model, on elastic supports.

(5) The design values of the action effects should be derived taking into account **4.4.2.5**.

(6) The design resistances should be derived in accordance with EN 1992-1-1:2004.

(7) In cases of core or wall structural systems of DCH, it should be verified that the transfer of the horizontal forces from the diaphragms to the cores or walls has occurred. In this respect the following provisions apply:

a) the design shear stress at the interface of the diaphragm and a core or wall should be limited to $1{,}5f_{\mathrm{ctd}}$, to control cracking;

b) an adequate strength to guard against shear sliding failure should be ensured, assuming that the strut inclination is 45^{o}. Additional bars should be provided, contributing to the shear strength of the interface between diaphragms and cores or walls; anchorage of these bars should follow the provisions of **5.6**.

5.11 Precast concrete structures

5.11.1 General

5.11.1.1 Scope and structural types

(1)P Clause **5.11** applies to the seismic design of concrete structures constructed partly or entirely of precast elements.

(2)P Unless otherwise specified (see **5.11.1.3.2(4)**), all provisions of Section **5** of this Eurocode and of EN 1992-1-1:2004, Section **10**, apply.

(3) The following structural types, as defined in **5.1.2** and **5.2.2.1**, are covered by **5.11**:

– frame systems;

– wall systems;

– dual systems (mixed precast frames and precast or monolithic walls).

(4) In addition the following systems are also covered:

– wall panel structures (cross wall structures);

– cell structures (precast monolithic room cell systems).

5.11.1.2 Evaluation of precast structures

(1) In modelling of precast structures, the following evaluations should be made.

a) Identification of the different roles of the structural elements as one of the following:

– those resisting only gravity loads, e.g. hinged columns around a reinforced concrete core;

– those resisting both gravity and seismic loads, e.g. frames or walls;

– those providing adequate connection between structural elements, e.g. floor or roof diaphragms.

b) Ability to fulfil the seismic resistance provisions of **5.1** to **5.10** as follows:

– precast system able to satisfy all those provisions;

– precast systems which are combined with cast-in-situ columns or walls in order to satisfy all those provisions;

– precast systems which deviate from those provisions and, by way of consequence, need additional design criteria and should be assigned lower behaviour factors.

c) Identification of non-structural elements, which may be:

– completely uncoupled from the structure; or

– partially resisting the deformation of structural elements.

d) Identification of the effect of the connections on the energy dissipation capacity of the structure:

– connections located well outside critical regions (as defined in **5.1.2(1)**), not affecting the energy dissipation capacity of the structure (see **5.11.2.1.1** and e.g. Figure 5.14.a);

– connections located within critical regions but adequately over-designed with respect to the rest of the structure, so that in the seismic design situation they remain elastic while inelastic response occurs in other critical regions (see **5.11.2.1.2** and e.g. Figure 5.14b);

– connections located within critical regions with substantial ductility (see **5.11.2.1.3** and e.g. Figure 5.14.c).

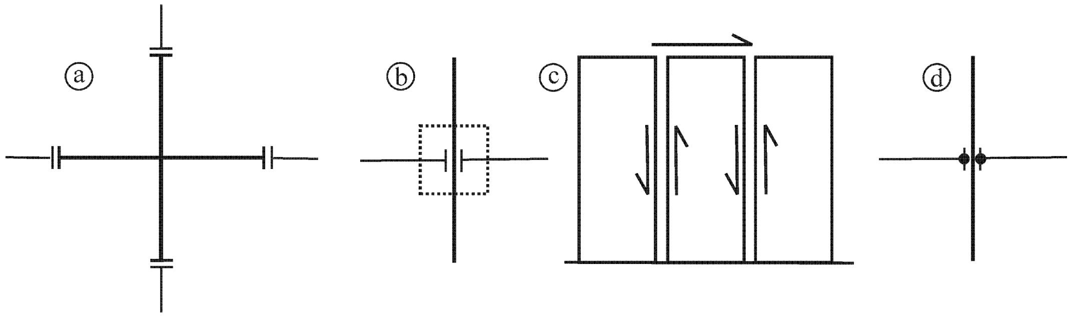


Figure 5.14: a) connection located outside critical regions; b) overdesigned connection with plastic hinges shifted outside the connection; c) ductile shear connections of large panels located within critical regions (e.g. at ground floor); and d) ductile continuity connections located within critical regions of frames

5.11.1.3 Design criteria

5.11.1.3.1 Local resistance

(1) In precast elements and their connections, the possibility of response degradation due to cyclic post-yield deformations should be taken into account. Normally such response degradation is covered by the material partial factors on steel and concrete (see **5.2.4(1)**P and **5.2.4(2)**). If it is not, the design resistance of precast connections under monotonic loading should be appropriately reduced for the verifications in the seismic design situation.

5.11.1.3.2 Energy dissipation

(1) In precast concrete structures the prevailing energy dissipation mechanism should be through plastic rotations within critical regions.

(2) Besides energy dissipation through plastic rotations in critical regions, precast structures can also dissipate energy through plastic shear mechanisms along joints, provided that both of the following conditions are satisfied:

a) the restoring force should not degrade substantially during the seismic action; and

b) the possible instabilities should be appropriately avoided.

(3) The three ductility classes provided in Section **5** for cast-in-place structures apply for precast systems as well. Only **5.2.1(2)** and **5.3** apply from Section **5**, for the design of precast buildings of Ductility Class L.

NOTE The selection of the ductility class for use in the various types of precast concrete systems in a country or the parts of the country may be found in its National Annex of this document. Ductility class L is recommended only for the low-seismicity case. For wall panel systems the recommended ductility class is M.

(4) The capacity of energy dissipation in shear may be taken into account, especially in precast wall systems, by taking into account the values of the local slip-ductility factors, μ_s, in the choice of the overall behaviour factor q.

5.11.1.3.3 Specific additional measures

(1) Only regular precast structures are covered by **5.11** (see **4.2.3**). Nonetheless, the verification of precast elements of irregular structures may be based on the provisions of this subsection.

(2) All vertical structural elements should be extended to the foundation level without a break.

(3) Uncertainties related to resistances are as in **5.2.3.7(2)**P.

(4) Uncertainties related to ductility are as in **5.2.3.7(3)**P.

5.11.1.4 Behaviour factors

(1) For precast-structures observing the provisions of **5.11**, the value of the behaviour factor q_p may be calculated from the following expression, unless special studies allow for deviations:

$$q_p = k_p \cdot q \tag{5.53}$$

where

q is the behaviour factor in accordance with expression (5.1);

k_p is the reduction factor depending on the energy dissipation capacity of the precast structure (see **(2)** of this subclause).

NOTE The values ascribed to k_p for use in a country may be found in its National Annex of this document. The recommended values are:

$$k_p \begin{cases} 1{,}00 & \text{for structures with connection according to 5.11.2.1.1, 5.11.2.1.2, or 5.11.2.1.3} \\ 0{,}5 & \text{for structures with other types of connections} \end{cases}$$

(2) For precast structures not observing the design provisions in **5.11**, the behaviour factor q_p should be assumed to be up to 1,5.

5.11.1.5 Analysis of transient situation

(1) During the erection of the structure, during which temporary bracing should be provided, seismic actions do not have to be taken into account as a design situation. However, whenever the occurrence of an earthquake might produce collapse of parts of the structure with serious risk to human life, temporary bracings should be explicitly designed for an appropriately reduced seismic action.

(2) If not otherwise specified by special studies, this action may be assumed to be equal to a fraction A_p of the design action as defined in Section **3**.

NOTE The value ascribed to A_p for use in a country may be found in its National Annex of this document. The recommended value of A_p is 30%.

5.11.2 Connections of precast elements

5.11.2.1 General provisions

5.11.2.1.1 Connections located away from critical regions

(1) Connections of precast elements considered to be away from critical regions should be located at a distance from the end face of the closest critical region, at least equal to the largest of the cross-section dimensions of the element where this critical region lies.

(2) Connections of this type should be dimensioned for: a) a shear force determined from the capacity design rule of **5.4.2.2** and **5.4.2.3** with a factor to account for overstrength due to strain-hardening of steel, γ_{Rd}, equal to 1,1 for DCM or to 1,2 for DCH; and b) a bending moment at least equal to the acting moment from the analysis and to 50% of the moment of resistance, M_{Rd}, at the end face of the nearest critical region, multiplied by the factor γ_{Rd}.

5.11.2.1.2 Overdesigned connections

(1) The design action-effects of overdesigned connections should be derived on the basis of the capacity design rules of **5.4.2.2** and **5.4.2.3**, on the basis of overstrength flexural resistances at the end sections of critical regions equal to $\gamma_{Rd}.M_{Rd}$, with the factor γ_{Rd} taken as being equal to 1,20 for DCM and to 1,35 for DCH.

(2) Terminating reinforcing bars of the overdesigned connection should be fully anchored before the end section(s) of the critical region.

(3) The reinforcement of the critical region should be fully anchored outside the overdesigned connection.

5.11.2.1.3 Energy dissipating connections

(1) Such connections should conform to the local ductility criteria in **5.2.3.4** and in the relevant paragraphs of **5.4.3** and **5.5.3**.

(2) Alternatively it should be demonstrated by cyclic inelastic tests of an appropriate number of specimens representative of the connection, that the connection possesses stable cyclic deformation and energy dissipation capacity at least equal to that of a monolithic connection which has the same resistance and conforms to the local ductility provisions of **5.4.3** or **5.5.3**.

(3) Tests on representative specimens should be performed following an appropriate cyclic history of displacements, including at least three full cycles at an amplitude corresponding to q_p in accordance with **5.2.3.4(3)**.

5.11.2.2 Evaluation of the resistance of connections

(1) The design resistance of the connections between precast concrete elements should be calculated in accordance with the provisions of EN 1992-1-1:2004, **6.2.5** and of EN 1992-1-1:2004, Section **10**, using the material partial factors of **5.2.4(2)** and **(3)**. If those provisions do not adequately cover the connection under consideration, its resistance should be evaluated by means of appropriate experimental studies.

(2) In evaluating the resistance of a connection against sliding shear, friction resistance due to external compressive stresses (as opposed to the internal stresses due to the clamping effect of bars crossing the connection) should be neglected.

(3) Welding of steel bars in energy dissipating connections may be structurally taken into account when all of the following conditions are met:

a) only weldable steels are used;

b) welding materials, techniques and personnel ensure a loss of local ductility less than 10% of the ductility factor achieved if the connection were implemented without welding.

(4) Steel elements (sections or bars) fastened on concrete members and intended to contribute to the seismic resistance should be analytically and experimentally demonstrated to resist a cyclic loading history of imposed deformation at the target ductility level, as specified in **5.11.2.1.3(2)**.

5.11.3 Elements

5.11.3.1 Beams

(1)P The relevant provisions of EN 1992-1-1:2004, Section **10** and of **5.4.2.1**, **5.4.3.1**, **5.5.2.1**, **5.5.3.1** of this Eurocode apply, in addition to the rules set forth in **5.11**.

(2)P Simply supported precast beams shall be structurally connected to columns or walls. The connection shall ensure the transmission of horizontal forces in the design seismic situation without reliance on friction.

(3) In addition to the relevant provisions of EN 1992-1-1:2004, Section **10**, the tolerance and spalling allowances of the bearings should also be sufficient for the expected displacement of the supporting member (see **4.3.4**).

5.11.3.2 Columns

(1) The relevant provisions of **5.4.3.2** and **5.5.3.2** apply, in addition to the rules set forth in **5.11**.

(2) Column-to-column connections within critical regions are allowed only in DCM.

(3) For precast frame systems with hinged column-to-beam connections, the columns should be fixed at the base with full supports in pocket foundations designed in accordance with **5.11.2.1.2**.

5.11.3.3 Beam-column joints

(1) Monolithic beam-column joints (see figure 5.14a) should follow the relevant provisions of **5.4.3.3** and **5.5.3.3**.

(2) Connections of beam-ends to columns (see figure 5.14b) and c) should be specifically checked for their resistance and ductility, as specified in **5.11.2.2.1**.

5.11.3.4 Precast large-panel walls

(1) EN 1992-1-1, Section **10** applies with the following modifications:

a) The total minimum vertical reinforcement ratio refers to the actual cross-sectional area of concrete and should include the vertical bars of the web and the boundary elements;

b) Mesh reinforcement in a single curtain is not allowed;

c) A minimum confinement should be provided to the concrete near the edge of all precast panels, as specified in **5.4.3.4.2** or **5.5.3.4.5** for columns, over a square section of side length b_w, where b_w denotes the thickness of the panel.

(2) The part of the wall panel between a vertical joint and an opening arranged closer than $2{,}5b_w$ to the joint, should be dimensioned and detailed in accordance with **5.4.3.4.2** or **5.5.3.4.5**, depending on the ductility class.

(3) Force-response degradation of the resistance of the connections should be avoided.

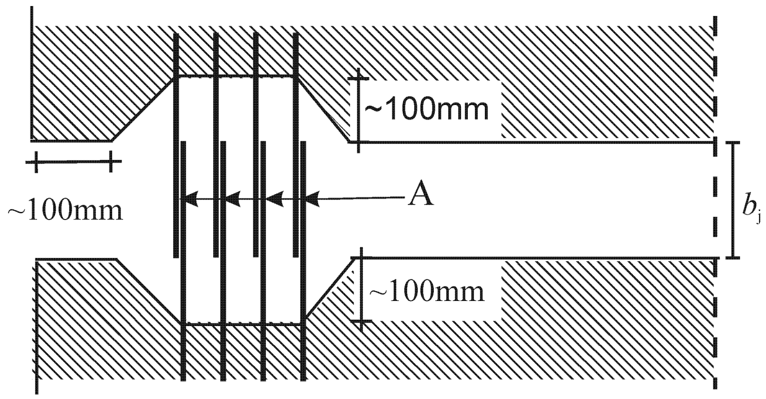
(4) To this end, all vertical joints should be rough or provided with shear keys and verified in shear.

(5) Horizontal joints under compression over their entire length may be formed without shear keys. If they are partly in compression and partly in tension, they should be provided with shear keys along the full length.

(6) The following additional rules apply for the verification of horizontal connections of walls consisting of precast large panels:

a) the total tensile force produced by axial (with respect to the wall) action-effects should be taken by vertical reinforcement arranged along the tensile area of the panel and fully anchored in the body of the upper and lower panels. The continuity of this reinforcement should be secured by ductile welding within the horizontal joint or, preferably, within special keys provided for this purpose (Figure 5.15).

b) in horizontal connections which are partly in compression and partly in tension (under the seismic design situation) the shear resistance verification (see **5.11.2.2**) should be made only along the part under compression. In such a case, the value of the axial force N_{Ed} should be replaced by the value of the total compressive force F_c acting on the compression area.



Key

A lap-welding of bars

Figure 5.15: Tensile reinforcement possibly needed at the edge of walls

(7) The following additional design rules should be observed, to enhance local ductility along the vertical connections of large panels:

a) minimum reinforcement should be provided across the connections equal to 0,10% in connections which are fully compressed, and equal to 0,25% in connections which are partly in compression and partly in tension;

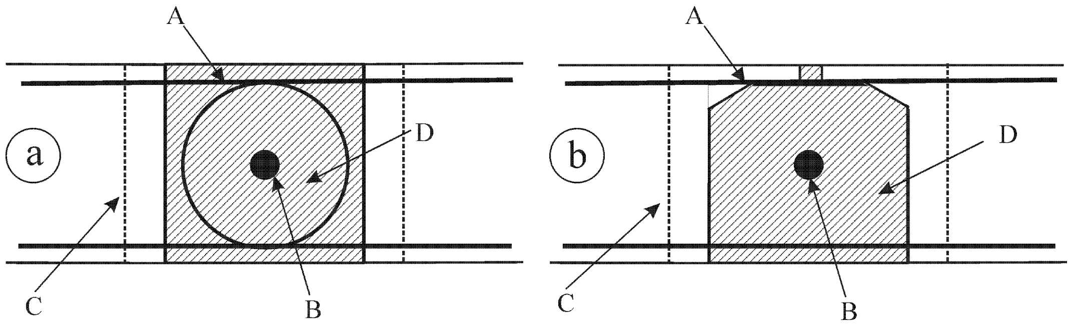
b) the amount of reinforcement across the connections should be limited, to avoid abrupt post-peak force response softening. In the absence of more specific evidence, the reinforcement ratio should not exceed 2%;

c) such reinforcement should be distributed across the entire length of the connection. In DCM this reinforcement may be concentrated in three bands (top, middle and bottom);

d) provision should be made to ensure continuity of reinforcement across panel-to-panel connections. To this end, in vertical connections steel bars should be anchored either in the form of loops or (in the case of joints with at least one face free) by welding across the connection (see Figure 5.16);

e) to secure continuity along the connection after cracking, longitudinal reinforcement at a minimum ratio of $\rho_{c,min}$ should be provided within the grout filling the space of the connection (see Figure 5.16).

NOTE The value ascribed to $\rho_{c,min}$ for use in a country may be found in its National Annex to this document. The recommended value is: $\rho_{c,min}$ = 1%.



Key

A reinforcement protruding across connection;

B reinforcement along connection;

C shear keys;

D grout filling space between panels.

Figure 5.16: Cross-section of vertical connections between precast large-panels, a) joint with two free faces; b) joint with one free face

(8) As a result of the energy dissipation capacity along the vertical (and in part along the horizontal) connections of large-panels, walls made of such precast panels are exempt from the requirements in **5.4.3.4.2** and **5.5.3.4.5** regarding the confinement of boundary elements.

5.11.3.5 Diaphragms

(1) In addition to the provisions of EN 1992-1-1:2004, Section **10** relevant to slabs and to the provisions of **5.10**, the following design rules also apply in the case of floor diaphragms made of precast elements.

(2) When the rigid diaphragm condition in accordance with **4.3.1(4)** is not satisfied, the in-plane flexibility of the floor as well as of the connections to the vertical elements should be taken into account in the model.

(3) The rigid diaphragm behaviour is enhanced if the joints in the diaphragm are located only over its supports. An appropriate topping of in-situ reinforced concrete can drastically improve the rigidity of the diaphragm. The thickness of this topping layer should be not less than 40 mm if the span between supports is less than 8 m, or not less than 50 mm for longer spans; its mesh reinforcement should be connected to the vertical resisting elements above and below.

(4) Tensile forces should be resisted by steel ties accommodated at least along the perimeter of the diaphragm, as well as along some joints of the precast slab elements. If a cast in-situ topping is used, this additional reinforcement should be located in this topping.

(5) In all cases, these ties should form a continuous system of reinforcement along and across the entire diaphragm and should be appropriately connected to each lateral force resisting element.

(6) In-plane acting shear forces along slab-to-slab or slab-to-beam connections should be computed with an overdesign factor equal to 1,30. The design resistance should be computed as in **5.11.2.2**.

(7) Primary seismic elements, both above and below the diaphragm, should be adequately connected to the diaphragm. To this end, any horizontal joints should always be properly reinforced. Friction forces due to external compressive forces should not be relied upon.

6 SPECIFIC RULES FOR STEEL BUILDINGS

6.1 General

6.1.1 Scope

(1)P For the design of steel buildings, EN 1993 applies. The following rules are additional to those given in EN 1993.

(2)P For buildings with composite steel-concrete structures, Section **7** applies.

6.1.2 Design concepts

(1)P Earthquake resistant steel buildings shall be designed in accordance with one of the following concepts (see Table 6.1):

- Concept a) Low-dissipative structural behaviour;
- Concept b) Dissipative structural behaviour.

Table 6.1: Design concepts, structural ductility classes and upper limit reference values of the behaviour factors

Design concept	Structural ductility class	Range of the reference values of the behaviour factor q
Concept a) Low dissipative structural behaviour	DCL (Low)	$\leq$ 1,5 - 2
Concept b) Dissipative structural behaviour	DCM (Medium)	$\leq$ 4 also limited by the values of Table 6.2
	DCH (High)	only limited by the values of Table 6.2

NOTE 1 The value ascribed to the upper limit of q for low dissipative behaviour, within the range of Table 6.1, for use in a country may be found in its National Annex. The recommended value of the upper limit of q for low-dissipative behaviour is 1,5.

NOTE 2 The National Annex of a particular country may give limitations on the choice of the design concept and of the ductility class which are permissible within that country.

(2)P In concept a) the action effects may be calculated on the basis of an elastic global analysis without taking into account a significant non-linear material behaviour. When using the design spectrum defined in **3.2.2.5**, the upper limit of the reference value of the behaviour factor q may be taken between 1,5 and 2 (see Note 1 to **(1)** of this subclause). In the case of irregularity in elevation the behaviour factor q should be corrected as indicated in **4.2.3.1(7)** but it need not be taken as being smaller than 1,5.

(3) In concept a), if the upper limit of the reference value of q is taken as being larger than 1,5, the primary seismic members of the structure should be of cross-sectional classes 1, 2 or 3.

(4) In concept a), the resistance of the members and of the connections should be evaluated in accordance with EN 1993 without any additional requirements. For buildings which are not seismically isolated (see Section **10**), design in accordance with concept a) is recommended only for low seismicity cases (see **3.2.1(4)**).

(5)P In concept b) the capability of parts of the structure (dissipative zones) to resist earthquake actions through inelastic behaviour is taken into account. When using the design spectrum defined in **3.2.2.5**, the reference value of behaviour factor q may be taken as being greater than the upper limit value established in Table 6.1 and in Note 1 to **(1)** of this subclause for low dissipative structural behaviour. The upper limit value of q depends on the Ductility Class and the structural type (see **6.3**). When adopting this concept b), the requirements given in **6.2** to **6.11** shall be fulfilled.

(6)P Structures designed in accordance with concept b) shall belong to structural ductility classes DCM or DCH. These classes correspond to increased ability of the structure to dissipate energy in plastic mechanisms. Depending on the ductility class, specific requirements in one or more of the following aspects shall be met: class of steel sections and rotational capacity of connections.

6.1.3 Safety verifications

(1)P For ultimate limit state verifications the partial factor for steel $\gamma_s = \gamma_M$ shall take into account the possible strength degradation due to cyclic deformations.

NOTE 1 The National Annex may give a choice of γ_s.

NOTE 2 Assuming that, due to the local ductility provisions, the ratio between the residual strength after degradation and the initial one is roughly equal to the ratio between the γ_M values for accidental and for fundamental load combinations, it is recommended that the partial factor γ_s adopted for the persistent and transient design situations be applied.

(2) In the capacity design checks specified in **6.5** to **6.8**, the possibility that the actual yield strength of steel is higher than the nominal yield strength should be taken into account by a material overstrength factor γ_{ov} (see **6.2(3)**).

6.2 Materials

(1)P Structural steel shall conform to standards referred to in EN 1993.

(2)P The distribution of material properties, such as yield strength and toughness, in the structure shall be such that dissipative zones form where they are intended to in the design.

NOTE Dissipative zones are expected to yield before other zones leave the elastic range during the earthquake.

(3) The requirement **(2)**P may be satisfied if the yield strength of the steel of dissipative zones and the design of the structure conform to one of the following conditions a), b) or c):

a) the actual maximum yield strength $f_{y,max}$ of the steel of dissipative zones satisfies the following expression $f_{y,max} \leq 1,1\gamma_{ov} f_y$

where

γ_{ov} is the overstrength factor used in design; and

f_y is the nominal yield strength specified for the steel grade.

NOTE 1 For steels of grade S235 and with γ_{ov} = 1,25 this method gives a maximum of $f_{y,max}$ = 323 N/mm^2.

NOTE 2 The value ascribed to γ_{ov} for use in a Country to check condition a) may be found in its National Annex. The recommended value is γ_{ov} = 1,25

b) the design of the structure is made on the basis of a single grade and nominal yield strength f_y for the steels both in dissipative and non dissipative zones; an upper value $f_{y,max}$ is specified for the steel of dissipative zones; the nominal value f_y of the steels specified for non dissipative zones and connections exceeds the upper value of the yield strength $f_{y,max}$ of dissipative zones.

NOTE This condition normally leads to the use of steels of grade S355 for non-dissipative members and non dissipative connections (designed on the basis of the f_y of S235 steels) and to the use of steels of grade S235 for dissipative members or connections where the upper yield strengths of steels of grade S235 is limited to $f_{y,max}$ = 355 N/mm^2.

c) the actual yield strength $f_{y,act}$ of the steel of each dissipative zone is determined from measurements and the overstrength factor is computed for each dissipative zone as $\gamma_{ov,act} = f_{y,act} / f_y$, f_y being the nominal yield strength of the steel of dissipative zones.

NOTE This condition is applicable when known steels are taken from stock or to the assessment of existing buildings or where safe side assumptions of yield strength made in design are confirmed by measurements before fabrication.

(4) If the conditions in **(3)**b of this subclause are satisfied, the overstrength factor, $\gamma_{ov,}$ may be taken as being 1,00 in the design checks for structural elements defined in **6.5** to **6.8**. In the verification of expression (6.1) for connections, the value to be used for the overstrength factor γ_{ov} is the same as in **(3)**a).

(5) If the conditions in **(3)**c) of this subclause are satisfied, the overstrength factor γ_{ov} should be taken as the maximum among the $\gamma_{ov,act}$ values computed in the verifications specified in **6.5** to **6.8**.

(6)P For dissipative zones, the value of the yield strength $f_{y,max}$ taken into account in observing the conditions in **(3)** of this subclause should be specified and noted on the drawings.

(7) The toughness of the steels and the welds should satisfy the requirements for the seismic action at the quasi-permanent value of the service temperature (see EN 1993-1-10:2004).

NOTE The National Annex may give information as to how EN 1993-1-10:2004 may be used in the seismic design situation.

(8) The required toughness of steel and welds and the lowest service temperature adopted in combination with the seismic action should be defined in the project specification.

(9) In bolted connections of primary seismic members of a building, high strength bolts of bolt grade 8.8 or 10.9 should be used.

(10)P The control of material properties shall be made in accordance with **6.11**.

6.3 Structural types and behaviour factors

6.3.1 Structural types

(1)P Steel buildings shall be assigned to one of the following structural types according to the behaviour of their primary resisting structure under seismic actions (see Figures 6.1 to 6.8).

a) Moment resisting frames, are those in which the horizontal forces are mainly resisted by members acting in an essentially flexural manner.

b) Frames with concentric bracings, are those in which the horizontal forces are mainly resisted by members subjected to axial forces.

c) Frames with eccentric bracings, are those in which the horizontal forces are mainly resisted by axially loaded members, but where the eccentricity of the layout is such that energy can be dissipated in seismic links by means of either cyclic bending or cyclic shear.

d) Inverted pendulum structures, are defined in **5.1.2**, and are structures in which dissipative zones are located at the bases of columns.

e) Structures with concrete cores or concrete walls, are those in which horizontal forces are mainly resisted by these cores or walls.

f) Moment resisting frames combined with concentric bracings.

g) Moment resisting frames combined with infills.

(2) In moment resisting frames, the dissipative zones should be mainly located in plastic hinges in the beams or the beam-column joints so that energy is dissipated by means of cyclic bending. The dissipative zones may also be located in columns:

– at the base of the frame;

– at the top of the columns in the upper storey of multi-storey buildings;

– at the top and bottom of columns in single storey buildings in which N_{Ed} in columns conform to the inequality: $N_{Ed} / N_{pl,Rd} < 0,3$.

(3) In frames with concentric bracings, the dissipative zones should be mainly located in the tensile diagonals.

The bracings may belong to one of the following categories:

- active tension diagonal bracings, in which the horizontal forces can be resisted by the tension diagonals only, neglecting the compression diagonals;
- V bracings, in which the horizontal forces can be resisted by taking into account both tension and compression diagonals. The intersection point of these diagonals lies on a horizontal member which shall be continuous.

K bracings, in which the intersection of the diagonals lies on a column (see Figure 6.9) may not be used.

(4) For frames with eccentric bracings configurations should be used that ensure that all links will be active, as shown in Figure 6.4.

(5) Inverted pendulum structures may be considered as moment resisting frames provided that the earthquake resistant structures possess more than one column in each resisting plane and that the following inequality of the limitation of axial force: $N_{Ed}< 0{,}3\ N_{pl,\,Rd}$ is satisfied in each column.

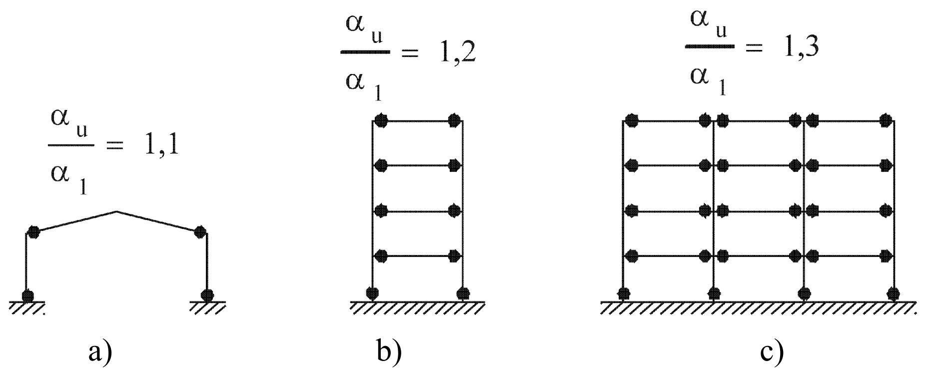


Figure 6.1: Moment resisting frames (dissipative zones in beams and at bottom of columns). Default values for α_u/α_1 (see 6.3.2(3) and Table 6.2).

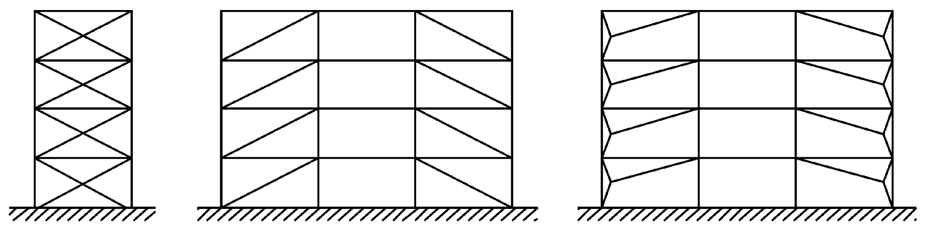


Figure 6.2: Frames with concentric diagonal bracings (dissipative zones in tension diagonals only).

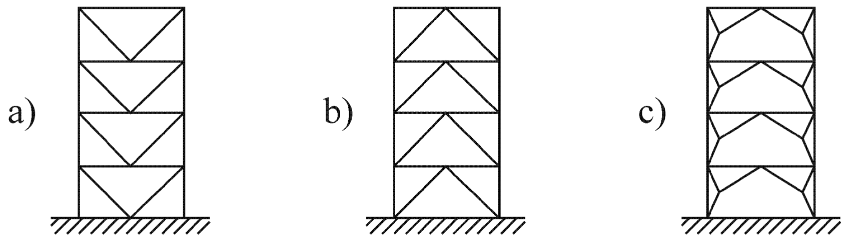


Figure 6.3: Frames with concentric V-bracings (dissipative zones in tension and compression diagonals).

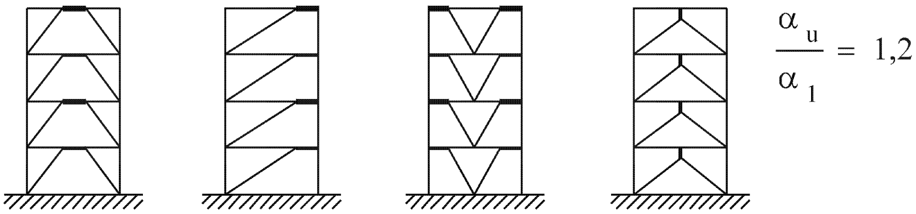


Figure 6.4: Frames with eccentric bracings (dissipative zones in bending or shear links). Default values for α_u/α_1 (see 6.3.2(3) and Table 6.2).

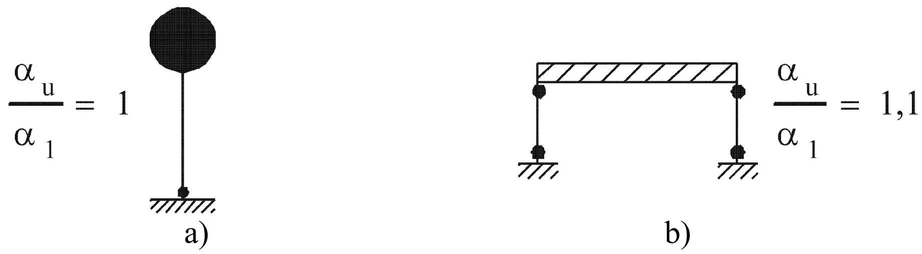


Figure 6.5: Inverted pendulum: a) dissipative zones at the column base; b) dissipative zones in columns ($N_{Ed}/N_{pl,Rd}$ < 0,3). Default values for α_u/α_1 (see 6.3.2(3) and Table 6.2).

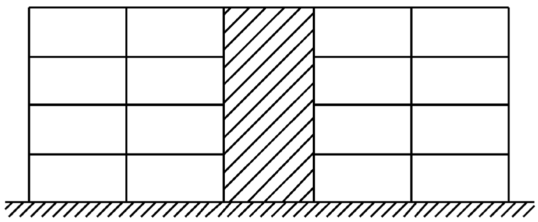


Figure 6.6: Structures with concrete cores or concrete walls.

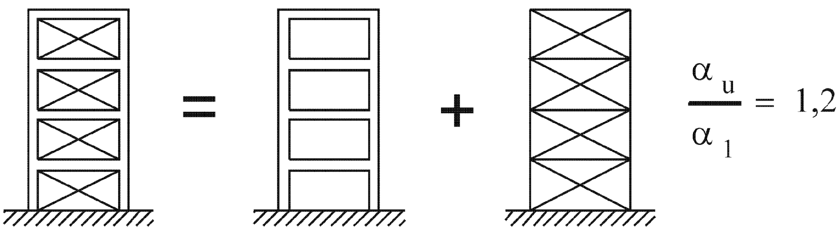


Figure 6.7: Moment resisting frame combined with concentric bracing (dissipative zones in moment frame and in tension diagonals). Default value for α_u/α_1 (see 6.3.2(3) and Table 6.2).

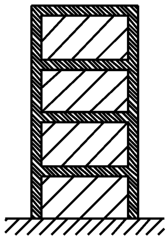


Figure 6.8: Moment resisting frame combined with infills.

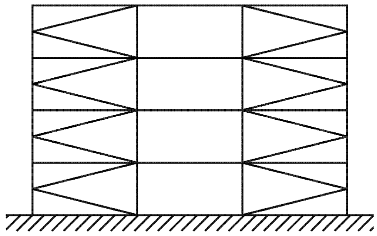


Figure 6.9: Frame with K bracings (not allowed).

6.3.2 Behaviour factors

(1) The behaviour factor q, introduced in **3.2.2.5**, accounts for the energy dissipation capacity of the structure. For regular structural systems, the behaviour factor q should be taken with upper limits to the reference values which are given in Table 6.2, provided that the rules in **6.5** to **6.11** are met.

Table 6.2: Upper limit of reference values of behaviour factors for systems regular in elevation

STRUCTURAL TYPE	Ductility Class	
	DCM	DCH
a) Moment resisting frames	4	$5\alpha_u/\alpha_1$
b) Frame with concentric bracings		
Diagonal bracings	4	4
V-bracings	2	2,5
c) Frame with eccentric bracings	4	$5\alpha_u/\alpha_1$
d) Inverted pendulum	2	$2\alpha_u/\alpha_1$
e) Structures with concrete cores or concrete walls	See section **5**	
f) Moment resisting frame with concentric bracing	4	$4\alpha_u/\alpha_1$
g) Moment resisting frames with infills		
Unconnected concrete or masonry infills, in contact with the frame	2	2
Connected reinforced concrete infills	See section **7**	
Infills isolated from moment frame (see moment frames)	4	$5\alpha_u/\alpha_1$

(2) If the building is non-regular in elevation (see **4.2.3.3**) the upper limit values of q listed in Table 6.2 should be reduced by 20 % (see **4.2.3.1(7)** and Table 4.1).

(3) For buildings that are regular in plan, if calculations to evaluate α_u/α_1, are not performed, the approximate default values of the ratio α_u/α_1 presented in Figures 6.1 to 6.8 may be used. The parameters α_1 and α_u are defined as follows:

α_1 is the value by which the horizontal seismic design action is multiplied in order to first reach the plastic resistance in any member in the structure, while all other design actions remain constant;

α_u is the value by which the horizontal seismic design action is multiplied, in order to form plastic hinges in a number of sections sufficient for the development of overall structural instability, while all other design actions remain constant. The factor α_u may be obtained from a nonlinear static (pushover) global analysis.

(4) For buildings which are not regular in plan (see **4.2.3.2**), the approximate value of α_u/α_1 that may be used when calculations are not performed for its evaluation are equal to the average of (a) 1,0 and of (b) the value given in Figures 6.1 to 6.8.

(5) Values of α_u/α_1 higher than those specified in **(3)** and **(4)** of this subclause are allowed, provided that they are confirmed by calculation of α_u/α_1 with a nonlinear static (pushover) global analysis.

(6) The maximum value of α_u/α_1 that may be used in a design is equal to 1,6, even if the analysis mentioned in **(5)** of this subclause indicates higher potential values.

6.4 Structural analysis

(1) The design of floor diaphragms should conform to **4.4.2.5**.

(2) Except where otherwise stated in this section (e.g. frames with concentric bracings, see **6.7.2(1)** and **(2)**), the analysis of the structure may be made assuming that all members of the seismic resisting structure are active.

6.5 Design criteria and detailing rules for dissipative structural behaviour common to all structural types

6.5.1 General

(1) The design criteria given in **6.5.2** should be applied to the earthquake-resistant parts of structures designed in accordance with the concept of dissipative structural behaviour.

(2) The design criteria given in **6.5.2** are deemed to be satisfied if the detailing rules given in **6.5.3** to **6.5.5** are followed.

6.5.2 Design criteria for dissipative structures

(1)P Structures with dissipative zones shall be designed such that yielding or local buckling or other phenomena due to hysteretic behaviour do not affect the overall stability of the structure.

NOTE The q factors given in Table 6.2 are deemed to conform to this requirement (see **2.2.2(2)**).

(2)P Dissipative zones shall have adequate ductility and resistance. The resistance shall be verified in accordance with EN 1993.

(3) Dissipative zones may be located in the structural members or in the connections.

(4)P If dissipative zones are located in the structural members, the non-dissipative parts and the connections of the dissipative parts to the rest of the structure shall have sufficient overstrength to allow the development of cyclic yielding in the dissipative parts.

(5)P When dissipative zones are located in the connections, the connected members shall have sufficient overstrength to allow the development of cyclic yielding in the connections.

6.5.3 Design rules for dissipative elements in compression or bending

(1)P Sufficient local ductility of members which dissipate energy in compression or bending shall be ensured by restricting the width-thickness ratio b/t according to the cross-sectional classes specified in EN 1993-1-1:2004, **5.5**.

(2) Depending on the ductility class and the behaviour factor q used in the design, the requirements regarding the cross-sectional classes of the steel elements which dissipate energy are indicated in Table 6.3

Table 6.3: Requirements on cross-sectional class of dissipative elements depending on Ductility Class and reference behaviour factor

Ductility class	Reference value of behaviour factor q	Required cross-sectional class
DCM	$1,5 < q \leq 2$	class 1, 2 or 3
	$2 < q \leq 4$	class 1 or 2
DCH	$q > 4$	class 1

6.5.4 Design rules for parts or elements in tension

(1) For tension members or parts of members in tension, the ductility requirement of EN 1993-1-1:2004, **6.2.3(3)** should be met.

6.5.5 Design rules for connections in dissipative zones

(1)P The design of connections shall be such as to limit localization of plastic strains, high residual stresses and prevent fabrication defects.

(2) Non dissipative connections of dissipative members made by means of full penetration butt welds may be deemed to satisfy the overstrength criterion.

(3) For fillet weld or bolted non dissipative connections, the following expression should be satisfied:

$$R_{\mathrm{d}} \geq 1,1\ \gamma_{\mathrm{ov}}\ R_{\mathrm{fy}} \tag{6.1}$$

where

R_{d} is the resistance of the connection in accordance with EN 1993;

R_{fy} is the plastic resistance of the connected dissipative member based on the design yield stress of the material as defined in EN 1993.

γ_{ov} is the overstrength factor (see **6.1.3(2)** and **6.2**).

(4) Categories *B* and *C* of bolted joints in shear in accordance with EN 1993-1-8:2004, **3.4.1** and category *E* of bolted joints in tension in accordance with EN 1993-1-8:2004, **3.4.2** should be used. Shear joints with fitted bolts are also allowed. Friction surfaces should belong to class A or B as defined in ENV 1090-1.

(5) For bolted shear connections, the design shear resistance of the bolts should be higher than 1,2 times the design bearing resistance.

(6) The adequacy of design should be supported by experimental evidence whereby strength and ductility of members and their connections under cyclic loading should be supported by experimental evidence, in order to conform to the specific requirements defined in **6.6** to **6.9** for each structural type and structural ductility class. This applies to partial and full strength connections in or adjacent to dissipative zones.

(7) Experimental evidence may be based on existing data. Otherwise, tests should be performed.

NOTE The National Annex may provide reference to complementary rules on acceptable connection design.

6.6 Design and detailing rules for moment resisting frames

6.6.1 Design criteria

(1)P Moment resisting frames shall be designed so that plastic hinges form in the beams or in the connections of the beams to the columns, but not in the columns, in accordance with **4.4.2.3**. This requirement is waived at the base of the frame, at the top level of multi-storey buildings and for single storey buildings.

(2)P Depending on the location of the dissipative zones, either **6.5.2(4)**P or **6.5.2(5)**P applies.

(3) The required hinge formation pattern should be achieved by conforming to **4.4.2.3**, **6.6.2**, **6.6.3** and **6.6.4**.

6.6.2 Beams

(1) Beams should be verified as having sufficient resistance against lateral and lateral torsional buckling in accordance with EN 1993, assuming the formation of a

plastic hinge at one end of the beam. The beam end that should be considered is the most stressed end in the seismic design situation.

(2) For plastic hinges in the beams it should be verified that the full plastic moment of resistance and rotation capacity are not decreased by compression and shear forces. To this end, for sections belonging to cross-sectional classes 1 and 2, the following inequalities should be verified at the location where the formation of hinges is expected:

$$\frac{M_{\mathrm{Ed}}}{M_{\mathrm{pl,Rd}}} \leq 1{,}0 \tag{6.2}$$

$$\frac{N_{\mathrm{Ed}}}{N_{\mathrm{pl,Rd}}} \leq 0{,}15 \tag{6.3}$$

$$\frac{V_{\mathrm{Ed}}}{V_{\mathrm{pl,Rd}}} \leq 0{,}5 \tag{6.4}$$

where

$$V_{\mathrm{Ed}} = V_{\mathrm{Ed,G}} + V_{\mathrm{Ed,M}}\,; \tag{6.5}$$

N_{Ed} is the design axial force;

M_{Ed} is the design bending moment;

V_{Ed} is the design shear;

$N_{\mathrm{pl,\,Rd}}$, $M_{\mathrm{pl,\,Rd}}$, $V_{\mathrm{pl,\,Rd}}$ are design resistances in accordance with EN 1993;

$V_{\mathrm{Ed,G}}$ is the design value of the shear force due to the non seismic actions;

$V_{\mathrm{Ed,M}}$ is the design value of the shear force due to the application of the plastic moments $M_{\mathrm{pl,Rd,A}}$ and $M_{\mathrm{pl,Rd,B}}$ with opposite signs at the end sections A and B of the beam.

NOTE $V_{\mathrm{Ed,M}} = (M_{\mathrm{pl,Rd,A}}+M_{\mathrm{pl,Rd,B}})/L$ is the most unfavourable condition, corresponding to a beam with span L and dissipative zones at both ends.

(3) For sections belonging to cross-sectional class 3, expressions (6.2) to (6.5) should be checked replacing $N_{\mathrm{pl,\,Rd}}$, $M_{\mathrm{pl,\,Rd}}$, $V_{\mathrm{pl,\,Rd}}$ with $N_{\mathrm{el,\,Rd}}$, $M_{\mathrm{el,\,Rd}}$, $V_{\mathrm{el,\,Rd}}$.

(4) If the condition in expression (6.3) is not verified, the requirement specified in **(2)** of this subclause is deemed to be satisfied if the provisions of EN 1993-1-1:2004, **6.2.9.1** are satisfied.

6.6.3 Columns

(1)P The columns shall be verified in compression considering the most unfavourable combination of the axial force and bending moments. In the checks, N_{Ed}, M_{Ed}, V_{Ed} should be computed as:

$$N_{Ed} = N_{Ed,G} + 1{,}1\gamma_{ov}\,\Omega N_{Ed,E}$$
$$M_{Ed} = M_{Ed,G} + 1{,}1\gamma_{ov}\,\Omega M_{Ed,E} \tag{6.6}$$
$$V_{Ed} = V_{Ed,G} + 1{,}1\gamma_{ov}\,\Omega V_{Ed,E}$$

where

$N_{Ed,G}$ ($M_{Ed,G}$, $V_{Ed,G}$) are the compression force (respectively the bending moment and shear force) in the column due to the non-seismic actions included in the combination of actions for the seismic design situation;

$N_{Ed,E}$ ($M_{Ed,E}$, $V_{Ed,E}$) are the compression force (respectively the bending moment and shear force) in the column due to the design seismic action;

γ_{ov} is the overstrength factor (see **6.1.3(2)** and **6.2(3)**)

Ω is the minimum value of $\Omega_i = M_{pl,Rd,i}/M_{Ed,i}$ of all beams in which dissipative zones are located; $M_{Ed,i}$ is the design value of the bending moment in beam *i* in the seismic design situation and $M_{pl,Rd,i}$.is the corresponding plastic moment.

(2) In columns where plastic hinges form as stated in **6.6.1(1)**P, the verification should take into account that in these plastic hinges the acting moment is equal to $M_{pl,Rd}$.

(3) The resistance verification of the columns should be made in accordance with EN 1993-1-1:2004, Section **6**.

(4) The column shear force V_{Ed} resulting from the structural analysis should satisfy the following expression :

$$\frac{V_{Ed}}{V_{pl,Rd}} \leq 0{,}5 \tag{6.7}$$

(5) The transfer of the forces from the beams to the columns should conform to the design rules given in EN 1993-1-1:2004, Section **6**.

(6) The shear resistance of framed web panels of beam/column connections (see Figure 6.10) should satisfy the following expression:

$$\frac{V_{wp,Ed}}{V_{wp,Rd}} \leq 1{,}0 \tag{6.8}$$

where

$V_{wp,Ed}$ is the design shear force in the web panel due to the action effects, taking into account the plastic resistance of the adjacent dissipative zones in beams or connections;

$V_{wp,Rd}$ is the shear resistance of the web panel in accordance with EN 1993- 1-8:2004, **6.2.4.1**. It is not required to take into account the effect of the stresses of the axial force and bending moment on the plastic resistance in shear.

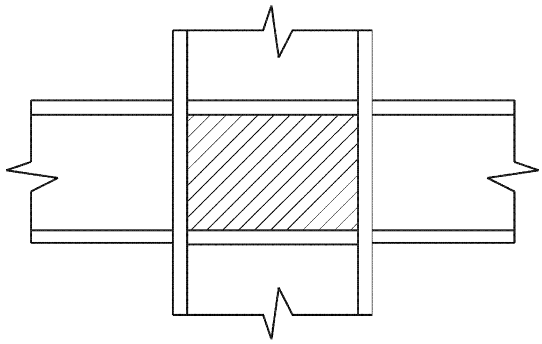


Figure 6.10: Web panel framed by flanges and stiffener

(7) The shear buckling resistance of the web panels should also be checked to ensure that it conforms to EN 1993-1-5:2004, Section **5**:

$$V_{wp,Ed} < V_{wb,Rd} \tag{6.9}$$

where

$V_{wb,Rd}$ is the shear buckling resistance of the web panel.

6.6.4 Beam to column connections

(1) If the structure is designed to dissipate energy in the beams, the connections of the beams to the columns should be designed for the required degree of overstrength (see **6.5.5**) taking into account the moment of resistance $M_{pl,Rd}$ and the shear force ($V_{Ed,G} + V_{Ed,M}$) evaluated in **6.6.2**.

(2) Dissipative semi-rigid and/or partial strength connections are permitted, provided that all of the following requirements are verified:

a) the connections have a rotation capacity consistent with the global deformations;

b) members framing into the connections are demonstrated to be stable at the ultimate limit state (ULS);

c) the effect of connection deformation on global drift is taken into account using non-linear static (pushover) global analysis or non-linear time history analysis.

(3) The connection design should be such that the rotation capacity of the plastic hinge region θ_p is not less than 35 mrad for structures of ductility class DCH and 25 mrad for structures of ductility class DCM with $q > 2$. The rotation θ_p is defined as

$$\theta_p = \delta / 0{,}5L \tag{6.10}$$

where (see Figure 6.11):

δ is the beam deflection at midspan ;

L is the beam span

The rotation capacity of the plastic hinge region θ_p should be ensured under cyclic loading without degradation of strength and stiffness greater than 20%. This requirement is valid independently of the intended location of the dissipative zones.

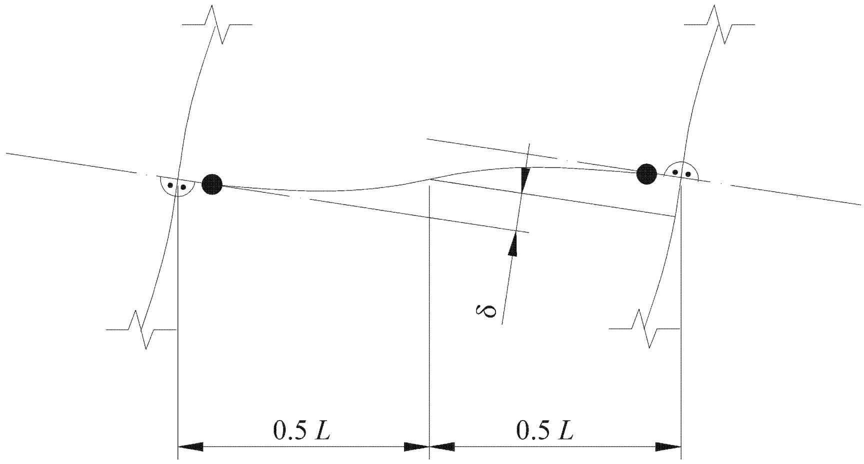


Figure 6.11: Beam deflection for the calculation of θ_p.

(4) In experiments made to assess θ_p the column web panel shear resistance should conform to expression (6.8) and the column web panel shear deformation should not contribute for more than 30% of the plastic rotation capability θ_p.

(5) The column elastic deformation should not be included in the evaluation of θ_p.

(6) When partial strength connections are used, the column capacity design should be derived from the plastic capacity of the connections.

6.7 Design and detailing rules for frames with concentric bracings

6.7.1 Design criteria

(1)P Concentric braced frames shall be designed so that yielding of the diagonals in tension will take place before failure of the connections and before yielding or buckling of the beams or columns.

(2)P The diagonal elements of bracings shall be placed in such a way that the structure exhibits similar load deflection characteristics at each storey in opposite senses of the same braced direction under load reversals.

(3) To this end, the following rule should be met at every storey:

$$\frac{\left|A^+ - A^-\right|}{A^+ + A^-} \leq 0,05 \tag{6.11}$$

where A^+ and A^- are the areas of the horizontal projections of the cross-sections of the tension diagonals, when the horizontal seismic actions have a positive or negative direction respectively (see Figure 6.12).

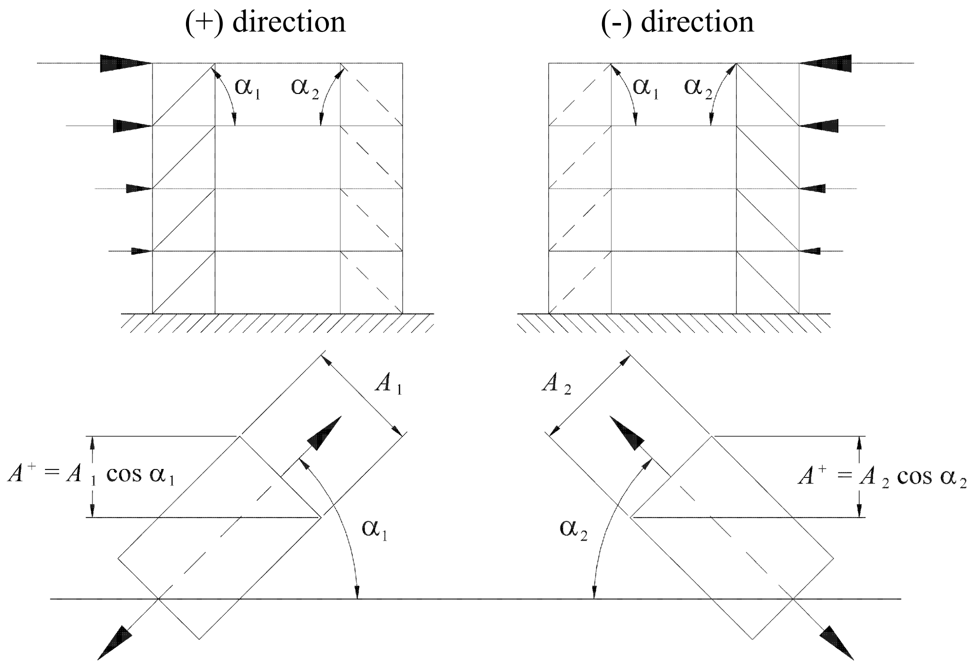


Figure 6.12: Example of application of 6.7.1(3)

6.7.2 Analysis

(1)P Under gravity load conditions, only beams and columns shall be considered to resist such loads, without taking into account the bracing members.

(2)P The diagonals shall be taken into account as follows in an elastic analysis of the structure for the seismic action:

- in frames with diagonal bracings, only the tension diagonals shall be taken into account;
- in frames with V bracings, both the tension and compression diagonals shall be taken into account.

(3) Taking into account of both tension and compression diagonals in the analysis of any type of concentric bracing is allowed provided that all of the following conditions are satisfied:

a) a non-linear static (pushover) global analysis or non-linear time history analysis is used;

b) both pre-buckling and post-buckling situations are taken into account in the modelling of the behaviour of diagonals and;

c) background information justifying the model used to represent the behaviour of diagonals is provided.

6.7.3 Diagonal members

(1) In frames with X diagonal bracings, the non-dimensional slenderness $\overline{\lambda}$ as defined in EN 1993-1-1:2004 should be limited to: $1{,}3 < \overline{\lambda} \leq 2{,}0$.

NOTE The 1,3 limit is defined to avoid overloading columns in the prebuckling stage (when both compression and tension diagonals are active) beyond the action effects obtained from an analysis at the ultimate stage where only the tension diagonal is taken as active.

(2) In frames with diagonal bracings in which the diagonals are not positioned as X diagonal bracings (see for instance Figure 6.12), the non-dimensional slenderness $\overline{\lambda}$ should be less than or equal to 2,0.

(3) In frames with V bracings, the non-dimensional slenderness $\overline{\lambda}$ should be less than or equal to 2,0.

(4) In structures up to two storeys, no limitation applies to $\overline{\lambda}$.

(5) The yield resistance $N_{\text{pl,Rd}}$ of the gross cross-section of the diagonals should be such that $N_{\text{pl,Rd}} \geq N_{\text{Ed}}$.

(6) In frames with V bracings, the compression diagonals should be designed for the compression resistance in accordance with EN 1993.

(7) The connections of the diagonals to any member should satisfy the design rules of **6.5.5**.

(8) In order to satisfy a homogeneous dissipative behaviour of the diagonals, it should be checked that the maximum overstrength Ω_i defined in **6.7.4(1)** does not differ from the minimum value Ω by more than 25%.

(9) Dissipative semi-rigid and/or partial strength connections are permitted, provided that all of the following conditions are satisfied:

a) the connections have an elongation capacity consistent with global deformations;

b) the effect of connections deformation on global drift is taken into account using non-linear static (pushover) global analysis or non-linear time history analysis.

6.7.4 Beams and columns

(1) Beams and columns with axial forces should meet the following minimum resistance requirement:

$$N_{\text{pl,Rd}}(M_{\text{Ed}}) \geq N_{\text{Ed,G}} + 1{,}1\gamma_{\text{ov}}\,\Omega . N_{\text{Ed,E}} \tag{6.12}$$

where

$N_{\text{pl,Rd}}(M_{\text{Ed}})$ is the design buckling resistance of the beam or the column in accordance with EN 1993, taking into account the interaction of the buckling resistance with the bending moment M_{Ed}, defined as its design value in the seismic design situation;

$N_{Ed,G}$ is the axial force in the beam or in the column due to the non-seismic actions included in the combination of actions for the seismic design situation;

$N_{Ed,E}$ is the axial force in the beam or in the column due to the design seismic action;

γ_{ov} is the overstrength factor (see **6.1.3(2)** and **6.2(3)**)

Ω is the minimum value of $\Omega_i = N_{pl,Rd,i}/N_{Ed,i}$ over all the diagonals of the braced frame system; where

$N_{pl,Rd,i}$ is the design resistance of diagonal i;

$N_{Ed,i}$ is the design value of the axial force in the same diagonal i in the seismic design situation.

(2) In frames with V bracings, the beams should be designed to resist:

- all non-seismic actions without considering the intermediate support given by the diagonals;
- the unbalanced vertical seismic action effect applied to the beam by the braces after buckling of the compression diagonal. This action effect is calculated using $N_{pl,Rd}$ for the brace in tension and γ_{pb} $N_{pl,Rd}$ for the brace in compression.

NOTE 1 The factor γ_{pb} is used for the estimation of the post buckling resistance of diagonals in compression.

NOTE 2 The value ascribed to γ_{pb} for use in a country may be found in its National Annex to this document. The recommended value is 0,3.

(3)P In frames with diagonal bracings in which the tension and compression diagonals are not intersecting (e.g. diagonals of Figure 6.12), the design should take into account the tensile and compression forces which develop in the columns adjacent to the diagonals in compression and correspond to compression forces in these diagonals equal to their design buckling resistance.

6.8 Design and detailing rules for frames with eccentric bracings

6.8.1 Design criteria

(1)P Frames with eccentric bracings shall be designed so that specific elements or parts of elements called seismic links are able to dissipate energy by the formation of plastic bending and/or plastic shear mechanisms.

(2)P The structural system shall be designed so that a homogeneous dissipative behaviour of the whole set of seismic links is realised.

NOTE The rules given hereafter are intended to ensure that yielding, including strain hardening effects in the plastic hinges or shear panels, will take place in the links prior to any yielding or failure elsewhere.

(3) Seismic links may be horizontal or vertical components (see Figure 6.4).

6.8.2 Seismic links

(1) The web of a link should be of single thickness without doubler plate reinforcement and without a hole or penetration.

(2) Seismic links are classified into 3 categories according to the type of plastic mechanism developed:

- short links, which dissipate energy by yielding essentially in shear;
- long links, which dissipate energy by yielding essentially in bending;
- intermediate links, in which the plastic mechanism involves bending and shear.

(3) For I sections, the following parameters are used to define the design resistances and limits of categories:

$$M_{p,link} = f_y\, b\, t_f\, (d - t_f) \tag{6.13}$$

$$V_{p,link} = (f_y/\sqrt{3})\, t_w\, (d - t_f) \tag{6.14}$$

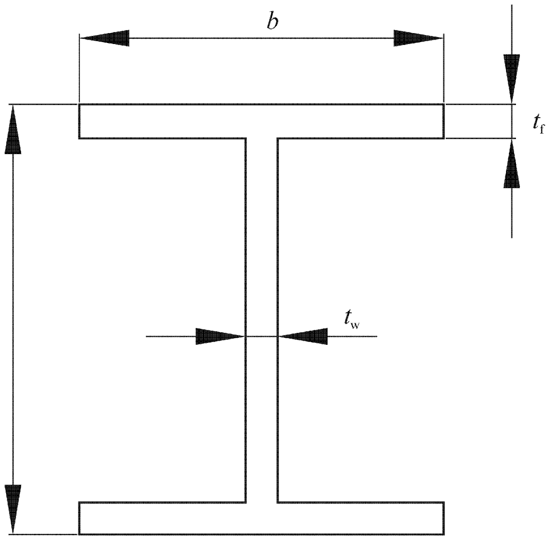


Figure 6.13: Definition of symbols for I link sections

(4) If $N_{Ed}/N_{pl,Rd} \leq 0,15$, the design resistance of the link should satisfy both of the following relationships at both ends of the link:

$$V_{Ed} \leq V_{p,link} \tag{6.15}$$

$$M_{Ed} \leq M_{p,link} \tag{6.16}$$

where

N_{Ed}, M_{Ed}, V_{Ed} are the design action effects, respectively the design axial force, design bending moment and design shear, at both ends of the link.

(5) If $N_{Ed}/N_{Rd} > 0,15$, expressions (6.15), (6.16) should be satisfied with the following reduced values $V_{p,link,r}$ and $M_{p,link,r}$ used instead of $V_{p,link}$ and $M_{p,link}$

$$V_{\mathrm{p,link,r}} = V_{\mathrm{p,link}} \left[1 - \left(N_{\mathrm{Ed}} / N_{\mathrm{pl,Rd}} \right)^2 \right]^{0,5} \quad (6.17)$$

$$M_{\mathrm{p,link,r}} = M_{\mathrm{p,link}} \left[1 - \left(N_{\mathrm{Ed}} / N_{\mathrm{pl,Rd}} \right) \right] \quad (6.18)$$

(6) If $N_{\mathrm{Ed}}/N_{\mathrm{Rd}} \geq 0,15$, the link length e should not exceed:

$e \leq 1,6\ M_{\mathrm{p,link}}/V_{\mathrm{p,link}}$ when $R < 0,3$, (6.19)

or

$e \leq (1,15 - 0,5\ R)\ 1,6\ M_{\mathrm{p,link}}/V_{\mathrm{p,link}}$ when $R \geq 0,3$ (6.20)

where $R = N_{\mathrm{Ed}}.t_{\mathrm{w}}.(d - 2t_{\mathrm{f}}) / (V_{\mathrm{Ed}}.A)$, in which A is the gross area of the link.

(7) To achieve a global dissipative behaviour of the structure, it should be checked that the individual values of the ratios Ω_{i} defined in **6.8.3(1)** do not exceed the minimum value Ω resulting from **6.8.3(1)** by more than 25% of this minimum value.

(8) In designs where equal moments would form simultaneously at both ends of the link (see Figure 6.14.a), links may be classified according to the length e. For I sections, the categories are:

- short links $e < e_{\mathrm{s}} = 1,6\ M_{\mathrm{p,link}}/V_{\mathrm{p,link}}$ (6.21)
- long links $e > e_{\mathrm{L}} = 3,0\ M_{\mathrm{p,link}}/V_{\mathrm{p,link}}$ (6.22)
- intermediate links $e_{\mathrm{s}} < e < e_{\mathrm{L}}$ (6.23)

(9) In designs where only one plastic hinge would form at one end of the link (see Figure 6.14.b), the value of the length e defines the categories of the links.For I sections the categories are:

- short links $e < e_{\mathrm{s}} = 0,8\ (1+\alpha)\ M_{\mathrm{p,link}}/V_{\mathrm{p,link}}$ (6.24)
- long links $e > e_{\mathrm{L}} = 1,5\ (1+\alpha)\ M_{\mathrm{p,link}}/V_{\mathrm{p,link}}$ (6.25)
- intermediate links $e_{\mathrm{s}} < e < e_{\mathrm{L}}$. (6.26)

where α is the ratio of the smaller bending moments $M_{\mathrm{Ed,A}}$ at one end of the link in the seismic design situation, to the greater bending moments $M_{\mathrm{Ed,B}}$ at the end where the plastic hinge would form, both moments being taken as absolute values.

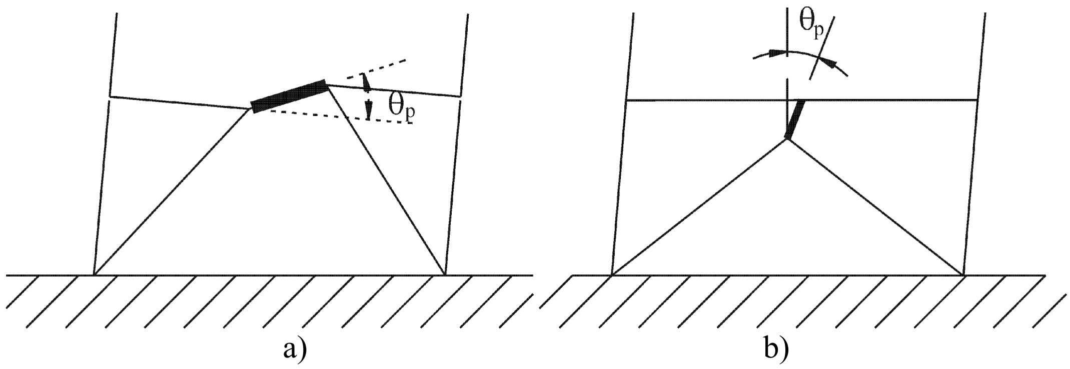


Figure 6.14: a) equal moments at link ends; b) unequal moments at link ends

(10) The link rotation angle θ_p between the link and the element outside of the link as defined in **6.6.4(3)** should be consistent with global deformations. It should not exceed the following values:

– short links $\theta_p \leq \theta_{pR} = 0{,}08$ radians (6.27)

– long links $\theta_p \leq \theta_{pR} = 0{,}02$ radians (6.28)

– intermediate links $\theta_p \leq \theta_{pR}$ = the value determined by linear interpolation between the above values. (6.29)

(11) Full-depth web stiffeners should be provided on both sides of the link web at the diagonal brace ends of the link. These stiffeners should have a combined width of not less than $(b_f - 2t_w)$ and a thickness not less than $0{,}75t_w$ nor 10 mm, whichever is larger.

(12) Links should be provided with intermediate web stiffeners as follows:

a) short links should be provided with intermediate web stiffeners spaced at intervals not exceeding $(30t_w - d/5)$ for a link rotation angle θ_p of 0,08 radians or $(52t_w - d/5)$ for link rotation angles θ_p of 0,02 radians or less. Linear interpolation should be used for values of θ_p between 0,08 and 0,02 radians;

b) long links should be provided with one intermediate web stiffener placed at a distance of 1,5 times b from each end of the link where a plastic hinge would form;

c) intermediate links should be provided with intermediate web stiffeners meeting the requirements of a) and b) above;

d) intermediate web stiffeners are not required in links of length e greater than 5 M_p/V_p;

e) intermediate web stiffeners should be full depth. For links that are less than 600 mm in depth d, stiffeners are required on only one side of the link web. The thickness of one-sided stiffeners should be not less than t_w or 10 mm, whichever is larger, and the width should be not less than $(b/2) - t_w$. For links that are 600 mm in depth or greater, similar intermediate stiffeners should be provided on both sides of the web.

(13) Fillet welds connecting a link stiffener to the link web should have a design strength adequate to resist a force of $\gamma_{ov} f_y A_{st}$, where A_{st} is the area of the stiffener. The

design strength of fillet welds fastening the stiffener to the flanges should be adequate to resist a force of $\gamma_{ov} A_{st} f_y/4$.

(14) Lateral supports should be provided at both the top and bottom link flanges at the ends of the link. End lateral supports of links should have a design axial resistance sufficient to provide lateral support for forces of 6% of the expected nominal axial strength of the link flange computed as $f_y\, b\, t_f$.

(15) In beams where a seismic link is present, the shear buckling resistance of the web panels outside of the link should be checked to conform to EN 1993-1-5:2004, Section **5**.

6.8.3 Members not containing seismic links

(1) The members not containing seismic links, like the columns and diagonal members, if horizontal links in beams are used, and also the beam members, if vertical links are used, should be verified in compression considering the most unfavourable combination of the axial force and bending moments:

$$N_{Rd}(M_{Ed}, V_{Ed}) \geq N_{Ed,G} + 1{,}1\gamma_{ov}\,\Omega N_{Ed,E} \tag{6.30}$$

where

$N_{Rd}(M_{Ed},V_{Ed})$ is the axial design resistance of the column or diagonal member in accordance with EN 1993, taking into account the interaction with the bending moment M_{Ed} and the shear V_{Ed} taken at their design value in the seismic situation;

$N_{Ed,G}$ is the compression force in the column or diagonal member due to the non-seismic actions included in the combination of actions for the seismic design situation;

$N_{Ed,E}$ is the compression force in the column or diagonal member due to the design seismic action;

γ_{ov} is the overstrength factor (see **6.1.3(2)** and **6.2(3)**)

Ω is a multiplicative factor which is the minimum of the following values:

the minimum value of $\Omega_i = 1{,}5\ V_{p,link,i}/V_{Ed,i}$ among all short links;

the minimum value of $\Omega_i = 1{,}5\ M_{p,link,i}/M_{Ed,i}$ among all intermediate and long links;

where

$V_{Ed,i}$, $M_{Ed,i}$ are the design values of the shear force and of the bending moment in link i in the seismic design situation;

$V_{p,link,i}$, $M_{p,link,i}$ are the shear and bending plastic design resistances of link i as in **6.8.2(3)**.

6.8.4 Connections of the seismic links

(1) If the structure is designed to dissipate energy in the seismic links, the connections of the links or of the element containing the links should be designed for action effects E_d computed as follows:

$$E_d \geq E_{d,G} + 1{,}1\gamma_{ov}\,\Omega_l E_{d,E} \tag{6.31}$$

where

$E_{d,G}$ is the action effect in the connection due to the non-seismic actions included in the combination of actions for the seismic design situation;

$E_{d,E}$ is the action effect in the connection due to the design seismic action;

γ_{ov} is the overstrength factor (see **6.1.3(2)** and **6.2(3)**)

Ω_i is the overstrength factor computed in accordance with **6.8.3(1)** for the link.

(2) In the case of semi-rigid and/or partial strength connections, the energy dissipation may be assumed to originate from the connections only. This is allowable, provided that all of the following conditions are satisfied:

a) the connections have rotation capacity sufficient for the corresponding deformation demands;

b) members framing into the connections are demonstrated to be stable at the ULS;

c) the effect of connection deformations on global drift is taken into account.

(3) When partial strength connections are used for the seismic links, the capacity design of the other elements in the structure should be derived from the plastic capacity of the links connections.

6.9 Design rules for inverted pendulum structures

(1) In inverted pendulum structures (defined in **6.3.1**(d)), the columns should be verified in compression considering the most unfavourable combination of the axial force and bending moments.

(2) In the checks, N_{Ed}, M_{Ed}, V_{Ed} should be computed as in **6.6.3**.

(3) The non-dimensional slenderness of the columns should be limited to $\overline{\lambda} \leq 1{,}5$.

(4) The interstorey drift sensitivity coefficient θ as defined in **4.4.2.2** should be limited to $\theta \leq 0{,}20$.

6.10 Design rules for steel structures with concrete cores or concrete walls and for moment resisting frames combined with concentric bracings or infills

6.10.1 Structures with concrete cores or concrete walls

(1)P The steel elements shall be verified in accordance with this Section and EN 1993, while the concrete elements shall be designed in accordance with Section **5**.

(2)P The elements in which an interaction between steel and concrete exists shall be verified in accordance with Section **7**.

6.10.2 Moment resisting frames combined with concentric bracings

(1) Dual structures with both moment resisting frames and braced frames acting in the same direction should be designed using a single q factor. The horizontal forces should be distributed between the different frames according to their elastic stiffness.

(2) The moment resisting frames and the braced frames should conform to **6.6**, **6.7** and **6.8**.

6.10.3 Moment resisting frames combined with infills

(1)P Moment resisting frames in which reinforced concrete infills are positively connected to the steel structure shall be designed in accordance with Section **7**.

(2)P The moment resisting frames in which the infills are structurally disconnected from the steel frame on the lateral and top sides shall be designed as steel structures.

(3) The moment resisting frames in which the infills are in contact with the steel frame, but are not positively connected to that frame, should satisfy the following rules:

a) the infills should be uniformly distributed in elevation in order not to increase locally the ductility demand on the frame elements. If this is not verified, the building should be considered as non-regular in elevation;

b) the frame-infill interaction should be taken into account. The internal forces in the beams and columns due to the diagonal strut action in the infills should be taken into account. The rules in **5.9** may be used to this end;

c) the steel frames should be verified in accordance with the rules in this clause, while the reinforced concrete or masonry infills should be designed in accordance with EN 1992-1-1:2004 and in accordance with Sections **5** or **9**.

6.11 Control of design and construction

(1)P The control of design and construction shall ensure that the real structure corresponds to the designed structure.

(2) To this end, in addition to the provisions of EN 1993, the following requirements should be met:

a) the drawings made for fabrication and erection should indicate the details of connections, sizes and qualities of bolts and welds as well as the steel grades of the members, noting the maximum permissible yield stress $f_{y,max}$ of the steel to be used by the fabricator in the dissipative zones;

b) the compliance of the materials with **6.2** should be checked;

c) the control of the tightening of the bolts and of the quality of the welds should follow the rules in EN 1090;

d) during construction it should be ensured that the yield stress of the actual steel used does not exceed f_{ymax} noted on the drawings for dissipative zones by more than 10%.

(2)P Whenever one of the above conditions is not satisfied, corrections or justifications shall be provided in order to meet the requirements of EN 1998-1 and assure the safety of the structure.

7 SPECIFIC RULES FOR COMPOSITE STEEL – CONCRETE BUILDINGS

7.1 General

7.1.1 Scope

(1)P For the design of composite steel - concrete buildings, EN 1994-1-1:2004 applies. The following rules are additional to those given in EN 1994-1-1:2004.

(2) Except where modified by the provisions of this Section, the provisions of Sections **5** and **6** apply.

7.1.2 Design concepts

(1)P Earthquake resistant composite buildings shall be designed in accordance with one of the following design concepts (see Table 7.1):

– Concept a) Low-dissipative structural behaviour.

– Concept b) Dissipative structural behaviour with composite dissipative zones;

– Concept c) Dissipative structural behaviour with steel dissipative zones.

Table 7.1: Design concepts, structural ductility classes and upper limit of reference values of the behaviour factors

Design concept	Structural ductility class	Range of the reference values of the behaviour factor q
Concept a) Low-dissipative structural behaviour	DCL (Low)	≤ 1,5 - 2
Concepts b) or c) Dissipative structural behaviour	DCM (Medium)	≤ 4 also limited by the values of Table 7.2
	DCH (High)	only limited by the values of Table 7.2

NOTE 1 The value ascribed to the upper limit of q for low dissipative behaviour, within the range of Table 7.1, for use in a country may be found in its National Annex to this document. The recommended value of the upper limit of q for low-dissipative behaviour is 1,5.

NOTE 2 The National Annex of a particular country may give limitations on the choice of the design concept and of the ductility class which are permissible within that country.

(2)P In concept a), the action effects may be calculated on the basis of an elastic analysis without taking into account non-linear material behaviour but considering the reduction in the moment of inertia due to the cracking of concrete in part of the beam spans, in accordance with the general structural analysis rules defined in **7.4** and to the specific rules defined in **7.7** to **7.11** related to each structural type. When using the design spectrum defined in **3.2.2.5**, the upper limit to the reference value of the behaviour factor q is taken between 1,5 and 2 (see Note 1 to **(1)** of this subclause). In case of irregularity in elevation the upper limit value of the behaviour factor q should be corrected as indicated in **4.2.3.1(7)** but it need not be taken as being smaller than 1,5.

(3) In concept a) the resistance of the members and of the connections should be evaluated in accordance with EN 1993 and EN 1994 without any additional requirements. For buildings which are not base-isolated (see Section **10**), design to concept a) is recommended only for low seismicity cases (see **3.2.1(4)**).

(4) In concepts b) and c), the capability of parts of the structure (dissipative zones) to resist earthquake actions through inelastic behaviour is taken into account. When using the design response spectrum defined in **3.2.2.5**, the upper limit to the reference value of the behaviour factor q is taken as being greater than the upper value established in Table 7.1 and in Note 1 to **(1)** of this subclause for low dissipative structural behaviour. The upper limit value of q depends on the ductility class and the structural type (see **7.3**). When adopting concepts b) or c) the requirements given in **7.2** to **7.12** should be fulfilled.

(5)P In concept c), structures are not meant to take advantage of composite behaviour in dissipative zones; the application of concept c) is conditioned by a strict compliance to measures that prevent involvement of the concrete in the resistance of dissipative zones. In concept c) the composite structure is designed in accordance with EN 1994-1-1:2004 under non seismic loads and in accordance with Section **6** to resist earthquake action. The measures preventing involvement of the concrete are given in **7.7.5**.

(6)P The design rules for dissipative composite structures (concept b), aim at the development of reliable local plastic mechanisms (dissipative zones) in the structure and of a reliable global plastic mechanism dissipating as much energy as possible under the design earthquake action. For each structural element or each structural type considered in this Section, rules allowing this general design objective to be achieved are given in **7.5** to **7.11** with reference to what are called the specific criteria. These criteria aim at the development of a global mechanical behaviour for which design provisions can be given.

(7)P Structures designed in accordance with concept b) shall belong to structural ductility classes DCM or DCH. These classes correspond to increased ability of the structure to dissipate energy in plastic mechanisms. A structure belonging to a given ductility class shall meet specific requirements in one or more of the following aspects: class of steel sections, rotational capacity of connections and detailing.

7.1.3 Safety verifications

(1)P **5.2.4(1)**P and **6.1.3(1)**P and its Notes apply.

(2) **5.2.4(2)** applies.

(3) **5.2.4(3)** applies.

(4) In the capacity design checks relevant for structural steel parts, **6.2(3)** and its Notes apply.

7.2 Materials

7.2.1 Concrete

(1) In dissipative zones, the prescribed concrete class should not be lower than C20/25. If the concrete class is higher than C40/50, the design is not within the scope of EN 1998-1.

7.2.2 Reinforcing steel

(1)P For ductility class DCM the reinforcing steel taken into account in the plastic resistance of dissipative zones shall be of class B or C in accordance with EN 1992-1-1:2004 Table C.1. For ductility class DCH the reinforcing steel taken into account in the plastic resistance of dissipative zones shalld be of class C according to the same Table.

(2)P Steel of class B or C (EN 1992-1-1:2004, Table C.1) shall be used in highly stressed regions of non dissipative structures. This requirement applies to both bars and welded meshes.

(3)P Except for closed stirrups or cross ties, only ribbed bars are allowed as reinforcing steel in regions with high stresses.

(4) Welded meshes not conforming to the ductility requirements of **(1)**P of this subclause should not be used in dissipative zones. If such meshes are used, ductile reinforcement duplicating the mesh should be placed and their resistance capacity accounted for in the capacity analysis.

7.2.3 Structural steel

(1)P The requirements are those specified in **6.2**.

7.3 Structural types and behaviour factors

7.3.1 Structural types

(1)P Composite steel-concrete structures shall be assigned to one of the following structural types according to the behaviour of their primary resisting structure under seismic actions:

a) <u>Composite moment resisting frames</u> are those with the same definition and limitations as in **6.3.1(1)**a, but in which beams and columns may be either structural steel or composite steel-concrete (see Figure 6.1);

b) <u>Composite concentrically braced frames</u> are those with the same definition and limitations as in **6.3.1(1)**b and Figures 6.2 and 6.3. Columns and beams may be either structural steel or composite steel-concrete. Braces shall be structural steel;

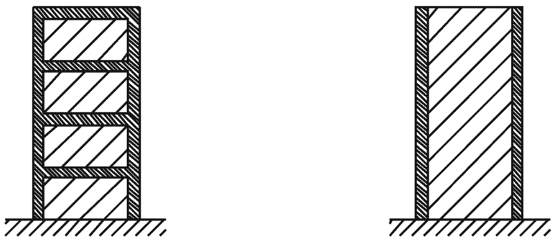
c) <u>Composite eccentrically braced frames</u> are those with the same definition and configurations as in **6.3.1(1)**c and Figure 6.4. The members which do not contain the links may be either structural steel or composite steel-concrete. Other than for the slab, the links shall be structural steel. Energy dissipation shall occur only through yielding in bending or shear of these links;

d) Inverted pendulum structures, have the same definition and limitations as in **6.3.1(1)**d (see Figure 6.5);

e) Composite structural systems are those which behave essentially as reinforced concrete walls. The composite systems may belong to one of the following types:

- Type 1 corresponds to a steel or composite frame working together with concrete infill panels connected to the steel structure (see Figure 7.1a);
- Type 2 is a reinforced concrete wall in which encased steel sections connected to the concrete structure are used as vertical edge reinforcement (see Figure 7.1b);
- Type 3, steel or composite beams are used to couple two or more reinforced concrete or composite walls (see Figure 7.2);

f) Composite steel plate shear walls are those consisting of a vertical steel plate continuous over the height of the building with reinforced concrete encasement on one or both faces of the plate and of the structural steel or composite boundary members.



a) b)

Figure 7.1: Composite structural systems. Composite walls: a) Type 1 – steel or composite moment frame with connected concrete infill panels; b) Type 2 – composite walls reinforced by connected encased vertical steel sections.

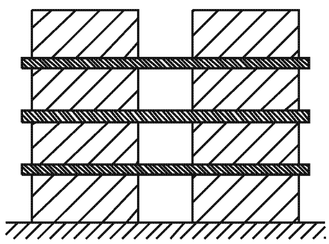


Figure 7.2: Composite structural systems. Type 3 - composite or concrete walls coupled by steel or composite beams.

(2) In all types of composite structural systems the energy dissipation takes place in the vertical steel sections and in the vertical reinforcements of the walls. In type 3 composite structural systems, energy dissipation may also take place in the coupling beams;

(3) If, in composite structural systems the wall elements are not connected to the steel structure, Sections **5** and **6** apply.

7.3.2 Behaviour factors

(1) The behaviour factor q, introduced in **3.2.2.5**, accounts for the energy dissipation capacity of the structure. For regular structural systems, the behaviour factor q should be taken with upper limits to the reference value which are given in Table 6.2 or in Table 7.2, provided that the rules in **7.5** to **7.11** are met.

Table 7.2: Upper limits to reference values of behaviour factors for systems regular elevation

STRUCTURAL TYPE	Ductility Class	
	DCM	DCH
a), b), c) and d)	See Table 6.2	
e) Composite structural systems		
Composite walls (Type 1 and Type 2)	$3\alpha_u/\alpha_1$	$4\alpha_u/\alpha_1$
Composite or concrete walls coupled by steel or composite beams (Type 3)	$3\alpha_u/\alpha_1$	$4{,}5\alpha_u/\alpha_1$
f) Composite steel plate shear walls	$3\alpha_u/\alpha_1$	$4\alpha_u/\alpha_1$

(2) If the building is non-regular in elevation (see **4.2.3.3**) the values of q listed in Table 6.2 and Table 7.2 should be reduced by 20 % (see **4.2.3.1(7)** and Table 4.1).

(3) For buildings that are regular in plan, if calculations to evaluate α_u/α_1 (see **6.3.2(3)**), are not performed, the approximate default values of the ratio α_u/α_1 presented in Figures 6.1 to 6.8 may be used. For composite structural systems the default value may be taken as being $\alpha_u/\alpha_1 = 1{,}1$. For composite steel plate shear walls the default value may be taken as being $\alpha_u/\alpha_1 = 1{,}2$.

(4) For buildings which are not regular in plan (see **4.2.3.2**), the approximate value of α_u/α_1 that may be used when calculations are not performed for its evaluation are equal to the average of (a) 1,0 and of (b) the value given in **(3)** of this subclause.

(5) Values of α_u/α_1 higher than those given in **(3)** and **(4)** of this subclause are allowed, provided that they are confirmed by calculating α_u/α_1 with a nonlinear static (pushover) global analysis.

(6) The maximum value of α_u/α_1 that may be used in the design is equal to 1,6, even if the analysis mentioned in **(5)** of this subclause indicates higher potential values.

7.4 Structural analysis

7.4.1 Scope

(1) The following rules apply to the analysis of the structure under earthquake action with the lateral force analysis method and with the modal response spectrum analysis method.

7.4.2 Stiffness of sections

(1) The stiffness of composite sections in which the concrete is in compression should be computed using a modular ratio n

$$n = E_a / E_{cm} = 7 \qquad (7.1)$$

(2) For composite beams with slab in compression, the second moment of area of the section, referred to as I_1, should be computed taking into account the effective width of slab defined in **7.6.3**.

(3) The stiffness of composite sections in which the concrete is in tension should be computed assuming that the concrete is cracked and that only the steel parts of the section are active.

(4) For composite beams with slab in tension, the second moment of area of the section, referred to as I_2, should be computed taking into account the effective width of slab defined in **7.6.3**.

(5) The structure should be analysed taking into account the presence of concrete in compression in some zones and concrete in tension in other zones; the distribution of the zones is given in **7.7** to **7.11** for the various structural types.

7.5 Design criteria and detailing rules for dissipative structural behaviour common to all structural types

7.5.1 General

(1) The design criteria given in **7.5.2** should be applied to the earthquake-resistant parts of structures designed in accordance with the concept of dissipative structural behaviour.

(2) The design criteria given in **7.5.2** are deemed to be satisfied, if the rules given in **7.5.3** and **7.5.4** and in **7.6** to **7.11** are observed.

7.5.2 Design criteria for dissipative structures

(1)P Structures with dissipative zones shall be designed such that yielding or local buckling or other phenomena due to hysteretic behaviour in those zones do not affect the overall stability of the structure.

NOTE The q factors given in Table 7.2 are deemed to conform to this requirement (see **2.2.2(2)**).

(2)P Dissipative zones shall have adequate ductility and resistance. The resistance shall be determined in accordance with EN 1993 and Section **6** for concept c) (see **7.1.2**) and to EN 1994-1-1:2004 and Section **7** for concept b) (see **7.1.2**). Ductility is achieved by compliance to detailing rules.

(3) Dissipative zones may be located in the structural members or in the connections.

(4)P If dissipative zones are located in the structural members, the non-dissipative parts and the connections of the dissipative parts to the rest of the structure shall have sufficient overstrength to allow the development of cyclic yielding in the dissipative parts.

(5)P When dissipative zones are located in the connections, the connected members shall have sufficient overstrength to allow the development of cyclic yielding in the connections.

7.5.3 Plastic resistance of dissipative zones

(1)P Two plastic resistances of dissipative zones are used in the design of composite steel - concrete structures: a lower bound plastic resistance (index: *pl*, *Rd*) and an upper bound plastic resistance (index: *U*, *Rd*).

(2)P The lower bound plastic resistance of dissipative zones is the one taken into account in design checks concerning sections of dissipative elements; e.g. $M_{\mathrm{Ed}} < M_{\mathrm{pl,Rd}}$. The lower bound plastic resistance of dissipative zones is computed taking into account the concrete component of the section and only the steel components of the section which are classified as ductile.

(3)P The upper bound plastic resistance of dissipative zones is the one used in the capacity design of elements adjacent to the dissipative zone: for instance in the capacity design verification of **4.4.2.3(4)**, the design values of the moments of resistance of beams are the upper bound plastic resistances, $M_{\mathrm{U,Rd,b}}$, whereas those of the columns are the lower bound ones, $M_{\mathrm{pl,Rd,c}}$.

(4)P The upper bound plastic resistance is computed taking into account the concrete component of the section and all the steel components present in the section, including those that are not classified as ductile.

(5)P Action effects, which are directly related to the resistance of dissipative zones, shall be determined on the basis of the upper bound resistance of composite dissipative sections; e.g. the design shear force at the end of a dissipative composite beam shall be determined on the basis of the upper bound plastic moment of the composite section.

7.5.4 Detailing rules for composite connections in dissipative zones

(1)P The design shall limit localization of plastic strains and high residual stresses and prevent fabrication defects.

(2)P The integrity of the concrete in compression shall be maintained during the seismic event and yielding shall be limited to the steel sections.

(3) Yielding of the reinforcing bars in a slab should be allowed only if beams are designed to conform to **7.6.2(8)**.

(4) For the design of welds and bolts, **6.5** applies.

(5) The local design of the reinforcing bars needed in the concrete of the joint region should be justified by models that satisfy equilibrium (e.g. Annex C for slabs).

(6) **6.5.5(6)**, **6.5.5(7)** and Note 1 to **6.5.5** apply.

(7) In fully encased framed web panels of beam/column connections, the panel zone resistance may be computed as the sum of contributions from the concrete and steel shear panel, if all the following conditions are satisfied:

a) the aspect ratio h_b/h_c of the panel zone is:

$$0,6 < h_b/h_c < 1,4 \qquad (7.2)$$

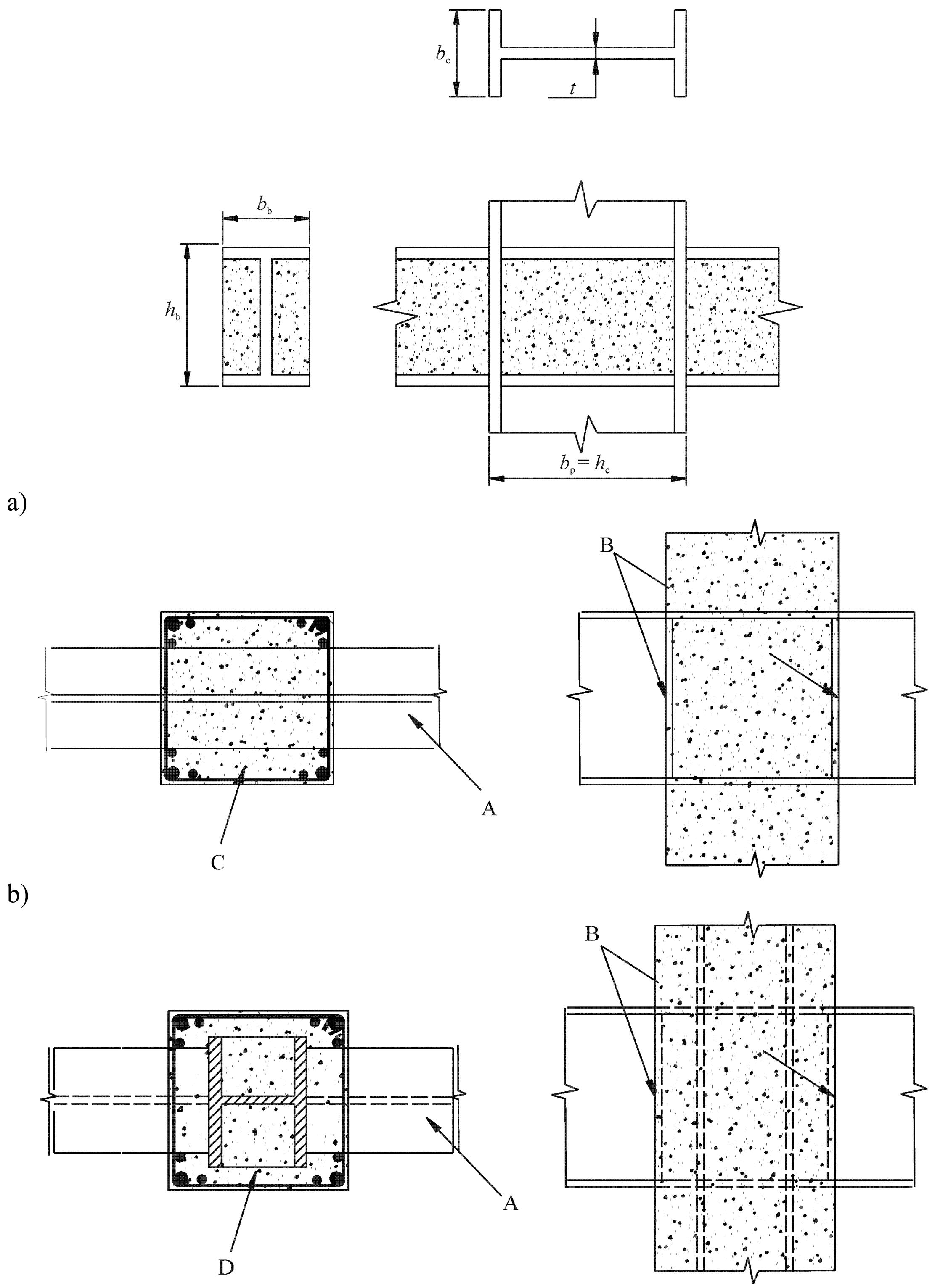
b) $V_{wp,Ed} < 0,8\ V_{wp,Rd}$ (7.3)

where

$V_{wp,Ed}$ is the design shear force in the web panel due to the action effects, taking into account the plastic resistance of the adjacent composite dissipative zones in beams or connections;

$V_{wp,Rd}$ is the shear resistance of the composite steel - concrete web panel in accordance with EN 1994-1-1:2004;

h_b, h_c are as defined in Figure 7.3a).



Key

A steel beam;

B face bearing plates;

C reinforced concrete column;

D composite encased column

Figure 7.3: Beam column connections.

(8) In partially encased stiffened web panels, an assessment similar to that in **(7)** of this subclause is permitted if, in addition to the requirements of **(9)**, one of the following conditions is fulfilled:

a) straight links of the type defined in **7.6.5(4)** and complying with **7.6.5(5)** and **(6)** are provided at a maximum spacing $s_1 = c$ in the partially encased stiffened web panel; these links are oriented perpendicularly to the longest side of the column web panel and no other reinforcement of the web panel is required; or

b) no reinforcement is present, provided that $h_b/b_b < 1,2$ and $h_c/b_c < 1,2$

where h_b, b_b, b_c and h_c are as defined in Figure 7.3a).

(9) When a dissipative steel or composite beam is framing into a reinforced concrete column as shown in Figure 7.3b), vertical column reinforcement with design axial strength at least equal to the shear strength of the coupling beam should be placed close to the stiffener or face bearing plate adjacent to the dissipative zone. It is permitted to use vertical reinforcement placed for other purposes as part of the required vertical reinforcement. The presence of face bearing plates is required; they should be full depth stiffeners of a combined width not less than (b_b – 2 t); their thickness should be not less than 0,75 t or 8 mm; b_b and t are respectively the beam flange width and the panel web thickness (see Figure 7.3).

(10) When a dissipative steel or composite beam is framing into a fully encased composite column as shown at Figure 7.3c), the beam column connection may be designed either as a beam/steel column connection or a beam/composite column connection. In the latter case, vertical column reinforcements may be calculated either as in **(9)** of this subclause or by distributing the shear strength of the beam between the column steel section and the column reinforcement. In both instances, the presence of face bearing plates as described in **(9)** is required.

(11) The vertical column reinforcement specified in **(9)** and **(10)** of this subclause should be confined by transverse reinforcement that meets the requirements for members defined in **7.6**.

7.6 Rules for members

7.6.1 General

(1)P Composite members, which are primary seismic members, shall conform to EN 1994-1-1:2004 and to additional rules defined in this Section.

(2)P The earthquake resistant structure is designed with reference to a global plastic mechanism involving local dissipative zones; this global mechanism identifies the members in which dissipative zones are located and indirectly the members without dissipative zones.

(3) For tension members or parts of members in tension, the ductility requirement of EN 1993-1-1:2004, **6.2.3(3)** should be met.

(4) Sufficient local ductility of members which dissipate energy under compression and/or bending should be ensured by restricting the width-to-thickness ratios of their walls. Steel dissipative zones and the not encased steel parts of composite members should meet the requirements of **6.5.3(1)** and Table 6.3. Dissipative zones of encased composite members should meet the requirements of Table 7.3. The limits given for flange outstands of partially or fully encased members may be relaxed if special details are provided as described in **7.6.4(9)** and **7.6.5(4)** to **(6)**.

Table 7.3: Relation between behaviour factor and limits of wall slenderness.

Ductility Class of Structure	DCM		DCH
Reference value of behaviour factor (q)	$q \leq 1{,}5 - 2$	$1{,}5 - 2 < q < 4$	$q > 4$
Partially Encased H or I Section Fully Encased H or I Section flange outstand limits c/t_f:	20 ε	14 ε	9 ε
Filled Rectangular Section h/t limits:	52 ε	38 ε	24 ε
Filled Circular Section d/t limits:	90 ε^2	85 ε^2	80 ε^2

where

$\varepsilon = (f_y/235)^{0,5}$

c/t_f is as defined in Figure 7.8

d/t and h/t are the ratio between the maximum external dimension and the wall thickness

(5) More specific detailing rules for composite members are given in **7.6.2**, **7.6.4**, **7.6.5** and **7.6.6**.

(6) In the design of all types of composite columns, the resistance of the steel section alone or the combined resistances of the steel section and the concrete encasement or infill may be taken into account.

(7) The design of columns in which the member resistance is taken to be provided only by the steel section may be carried out in accordance with the provisions of Section **6**. In the case of dissipative columns, the capacity design rules in **7.5.2(4)** and **(5)** and **7.5.3(3)** should be satisfied.

(8) For fully encased columns with composite behaviour, the minimum cross-sectional dimensions b, h or d should be not less than 250 mm.

(9) The resistance, including shear resistance, of non-dissipative composite columns should be determined in accordance with the rules of EN 1994-1-1:2004.

(10) In columns, when the concrete encasement or infill are assumed to contribute to the axial and/or flexural resistance of the member, the design rules in **7.6.4** to **7.6.6** apply. These rules ensure full shear transfer between the concrete and the steel parts in a section and protect the dissipative zones against premature inelastic failure.

(11) For earthquake-resistant design, the design shear strength given in EN 1994-1-1:2004, Table 6.6, should be multiplied by a reduction factor of 0,5.

(12) When, for capacity design purposes, the full composite resistance of a column is employed, complete shear transfer between the steel and reinforced concrete parts should be ensured. If insufficient shear transfer is achieved through bond and friction, shear connectors should be provided to ensure full composite action.

(13) Wherever a composite column is subjected to predominately axial forces, sufficient shear transfer should be provided to ensure that the steel and concrete parts share the loads applied to the column at connections to beams and bracing members.

(14) Except at their base in some structural types, columns are generally not designed to be dissipative. However, because of uncertainties in the behaviour, confining reinforcement is required in regions called "critical regions" as specified in **7.6.4**.

(15) Subclauses **5.6.2.1** and **5.6.3** concerning anchorage and splices in the design of reinforced concrete columns apply also to the reinforcements of composite columns.

7.6.2 Steel beams composite with slab

(1)P The design objective of this subclause is to maintain the integrity of the concrete slab during the seismic event, while yielding takes place in the bottom part of the steel section and/or in the rebars of the slab.

(2)P If it is not intended to take advantage of the composite character of the beam section for energy dissipation, **7.7.5** shall be applied.

(3) Beams intended to behave as composite elements in dissipative zones of the earthquake resistant structure may be designed for full or partial shear connection in accordance with EN 1994-1-1:2004. The minimum degree of connection η as defined in EN 1994-1-1:2004 **6.6.1.2** should be not less than 0,8 and the total resistance of the shear connectors within any hogging moment region not less than the plastic resistance of the reinforcement.

(4) The design resistance of connectors in dissipative zones is obtained from the design resistance provided in EN 1994-1-1:2004 multiplied by a reduction factor of 0,75.

(5) Full shear connection is required when non-ductile connectors are used.

(6) When a profiled steel sheeting with ribs transverse to the supporting beams is used, the reduction factor k_t of the design shear resistance of connectors given by EN 1994-1-1 should be further reduced by multiplying it by the rib shape efficiency factor k_r given in Figure 7.4.

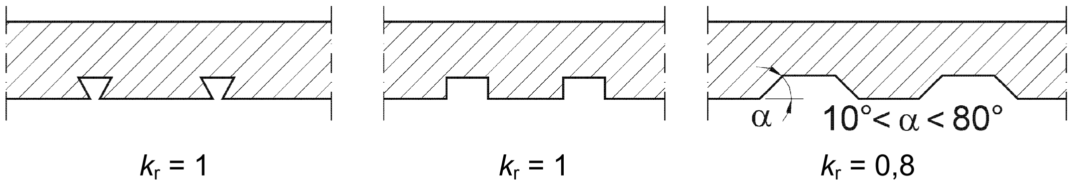


Figure 7.4: Values of the rib shape efficiency factor.

(7) To achieve ductility in plastic hinges, the ratio x/d of the distance x between the top concrete compression fibre and the plastic neutral axis, to the depth d of the composite section, should conform to the following expression:

$$x/d < \varepsilon_{cu2} / (\varepsilon_{cu2+} \varepsilon_a) \tag{7.4}$$

where

ε_{cu2} is the ultimate compressive strain of concrete (see EN 1992-1-1:2004);

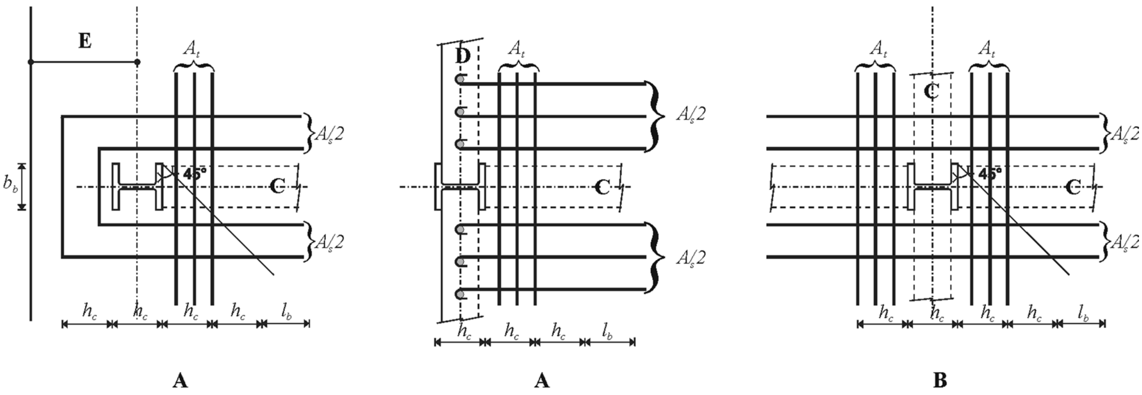
ε_a is the total strain in steel at Ultimate Limit State.

(8) The rule in **(7)** of this subclause is deemed to be satisfied when x/d of a section is less than the limits given in Table 7.4.

Table 7.4: Limit values of *x/d* for ductility of beams with slab

Ductility class	q	f_y (N/mm^2)	x/d upper limit
DCM	$1,5 < q \leq 4$	355	0,27
	$1,5 < q \leq 4$	235	0,36
DCH	$q > 4$	355	0,20
	$q > 4$	235	0,27

(9) In dissipative zones of beams, specific ductile steel reinforcement of the slab called "seismic rebars" (see Figure 7.5), should be present in the connection zone of the beam and the column. Its design and the symbols used in Figure 7.5 are specifed in Annex C.



Key

A	Exterior Node

B	Interior Node

C	Steel beam

D	Façade steel beam

E	Reinforced concrete cantilever edge strip

Figure 7.5: Layout of "seismic rebars"

7.6.3 Effective width of slab

(1) The total effective width b_{eff} of concrete flange associated with each steel web should be taken as the sum of the partial effective widths b_{e1} and b_{e2} of the portion of the flange on each side of the centreline of the steel web (Figure 7.6). The partial effective width on each side should be taken as b_e given in Table 7.5, but not greater than the actual available widths b_1 and b_2 defined in **(2)** of this subclause.

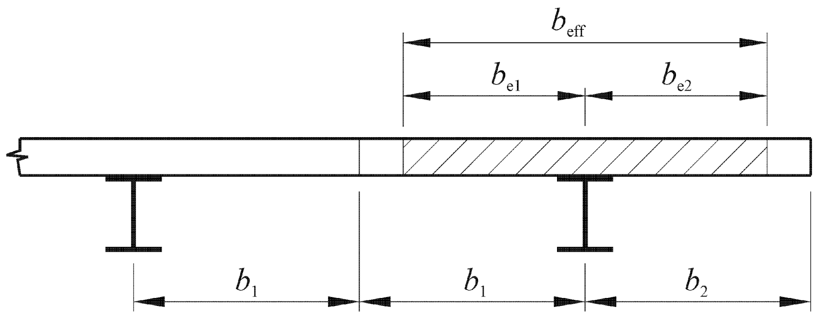
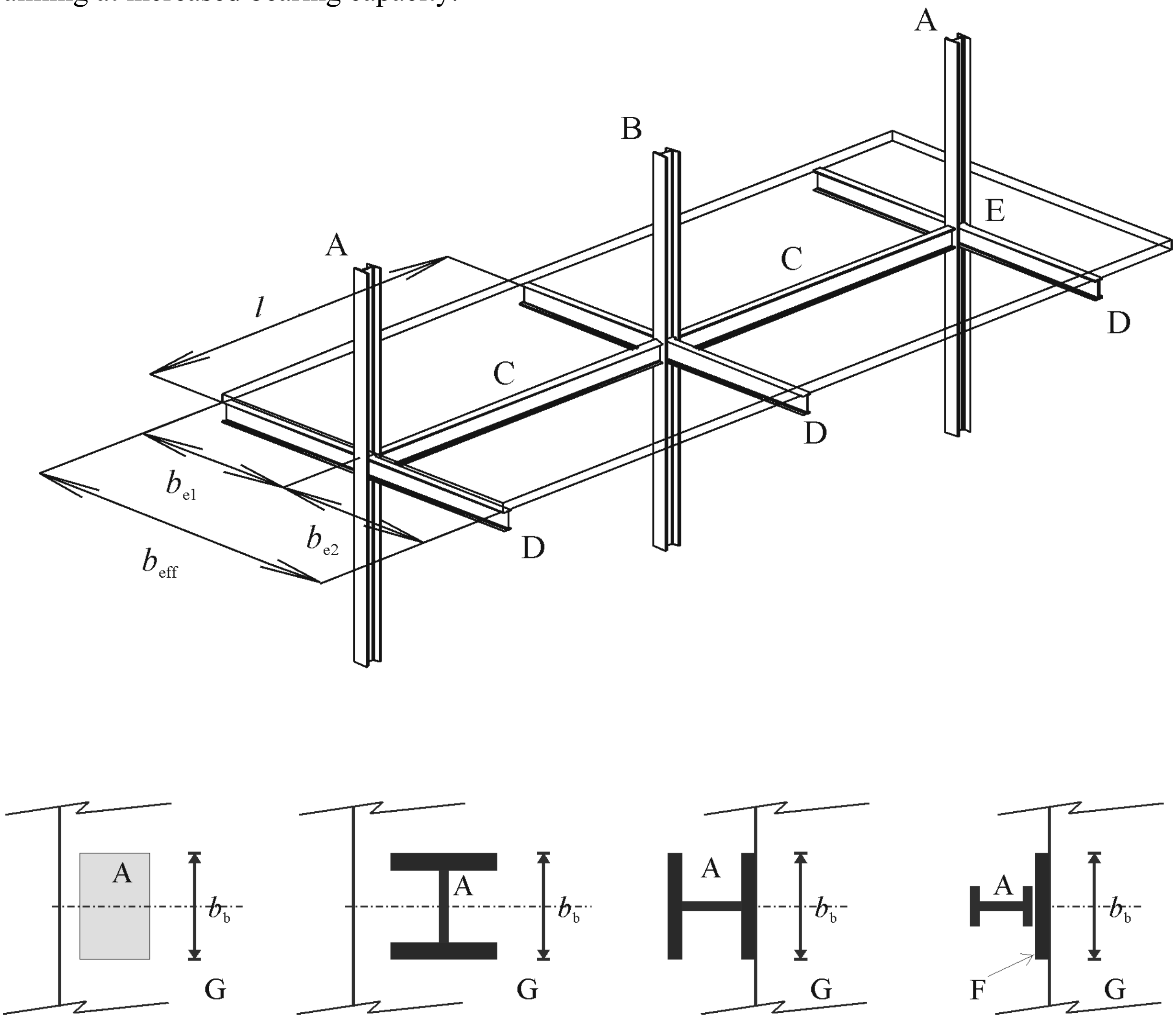


Figure 7.6: Definition of effective width b_e and b_{eff}

(2) The actual width b of each portion should be taken as half the distance from the web to the adjacent web, except that at a free edge the actual width is the distance from the web to the free edge.

(3) The partial effective width b_e of the slab to be used in the determination of the elastic and plastic properties of the composite T sections made of a steel section connected to a slab are defined in Table 7.5 and Figure 7.7. These values are valid for beams positioned as beams C in Figure 7.5 and if the design of the slab reinforcement and of the connection of the slab to the steel beams and columns are in accordance with

Annex C. In Table 7.5 those moments which induce compression in the slab are considered as positive and those which induce tension in the slab are considered as negative. Symbols b_b , h_c , b_e , b_{eff} and l used in Tables 7.5 I and 7.5 II are defined in Figures 7.5, 7.6 and 7.7. b_b is the bearing width of the concrete of the slab on the column in the horizontal direction perpendicular to the beam for which the effective width is computed; this bearing width possibly includes additional plates or devices aiming at increased bearing capacity.



Key

A Exterior column;

B Interior column;

C Longitudinal beam;

D Transverse beam or steel façade beam;

E Cantilever concrete edge strip;

F Extended bearing;

G Concrete slab

Figure 7.7: Definition of elements in moment frame structures.

Table 7.5 I: Partial effective width b_e of slab for elastic analysis of the structure

b_e	Transverse element	b_e for I (ELASTIC)
At interior column	Present or not present	For negative M : 0,05 l
At exterior column	Present	For positive M : 0,0375 l
At exterior column	Not present, or re-bars not anchored	For negative M : 0 For positive M : 0,025 l

Table 7.5 II: Partial effective width b_e of slab for evaluation of plastic moment resistance

Sign of bending moment M	Location	Transverse element	b_e for M_{Rd} (PLASTIC)
Negative M	Interior column	Seismic re-bars	0,1 l
Negative M	Exterior column	All layouts with re-bars anchored to façade beam or to concrete cantilever edge strip	0,1 l
Negative M	Exterior column	All layouts with re-bars not anchored to façade beam or to concrete cantilever edge strip	0,0
Positive M	Interior column	Seismic re-bars	0,075 l
Positive M	Exterior column	Steel transverse beam with connectors. Concrete slab up to exterior face of column of H section with strong axis oriented as in Fig. 7.5 or beyond (concrete edge strip). Seismic re-bars	0,075 l
Positive M	Exterior column	No steel transverse beam or steel transverse beam without connectors. Concrete slab up to exterior face of column of H section with strong axis oriented as in Fig. 7.5, or beyond (edge strip). Seismic re-bars	$b_b/2 + 0,7\ h_c/2$
Positive M	Exterior column	All other layouts. Seismic re-bars	$b_b/2 \leq b_{e,max}$ $b_{e,max} = 0,05l$

7.6.4 Fully encased composite columns

(1) In dissipative structures, critical regions are present at both ends of all column clear lengths in moment frames and in the portion of columns adjacent to links in eccentrically braced frames. The lengths l_{cr} of these critical regions (in metres) are specified by expression (5.14) for ductility class M, or by expression (5.30) for ductility class H, with h_c in these expressions denoting the depth of the composite section (in metres).

(2) To satisfy plastic rotation demands and to compensate for loss of resistance due to spalling of cover concrete, the following expression should be satisfied within the critical regions defined above:

$$\alpha \cdot \omega_{wd} \geq 30 \cdot \mu_{\phi} \cdot \nu_d \cdot \varepsilon_{sy,d} \cdot \frac{b_c}{b_o} - 0{,}035 \qquad (7.5)$$

in which the variables are as defined in **5.4.3.2.2(8)** and the normalised design axial force ν_d is defined as:

$$\nu_d = N_{Ed}/N_{pl,Rd} = N_{Ed}/(A_a f_{yd} + A_c f_{cd} + A_s f_{sd}) \qquad (7.6)$$

(3) The spacing, *s,* (in millimetres) of confining hoops in critical regions should not exceed

$s = \min(b_o/2, 260, 9\ d_{bL})$ in ductility class DCM; (7.7)

$s = \min(b_o/2, 175, 8\ d_{bL})$ in ductility class DCH (7.8)

or at the lower part of the lower storey, in ductility class DCH

$s = \min(b_o/2, 150, 6 d_{bL})$ (7.9)

where

b_o is the minimum dimension of the concrete core (to the centreline of the hoops, in millimetres);

d_{bL} is the minimum diameter of the longitudinal rebars (in millimetres).

(4) The diameter of the hoops, d_{bw}, (in millimetres) should be at least

$d_{bw} = 6$ in ductility class DCM (7.10)

$d_{bw} = \max(0{,}35\ d_{bL,max}[f_{ydL}/f_{ydw}]^{0,5}, 6)$ in ductility class DCH (7.11)

where

$d_{bL,max}$ is the maximum diameter of the longitudinal rebars (in millimetres).

(5) In critical regions, the distance between consecutive longitudinal bars restrained by hoop bends or cross-ties should not exceed 250 mm in ductility class DCM or 200 mm in ductility class DCH.

(6) In the lower two storeys of a building, hoops in accordance with **(3)**, **(4)** and **(5)** should be provided beyond the critical regions for an additional length equal to half the length of the critical regions.

(7) In dissipative composite columns, the shear resistance should be determined on the basis of the structural steel section alone.

(8) The relationship between the ductility class of the structure and the allowable slenderness (c/t_f) of the flange outstand in dissipative zones is given in Table 7.3.

(9) Confining hoops can delay local buckling in the dissipative zones. The limits given in Table 7.3 for flange slenderness may be increased if the hoops are provided at a longitudinal spacing, *s,* which is less than the flange outstand: $s/c < 1{,}0$. For $s/c < 0{,}5$

the limits given in Table 7.3 may be increased by up to 50%. For values of $0,5 < s/c < 1,0$ linear interpolation may be used.

(10) The diameter d_{bw} of confining hoops used to prevent flange buckling should be not less than

$$d_{bw} = \left[(b \cdot t_f / 8)(f_{ydf} / f_{ydw})\right]^{0,5} \tag{7.12}$$

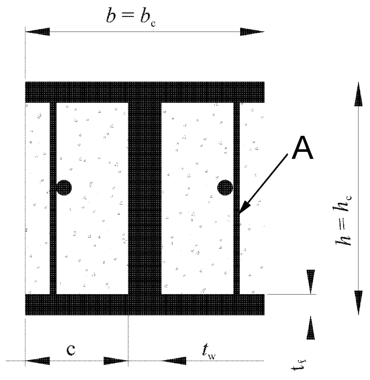
in which b and t_f are the width and thickness of the flange respectively and f_{ydf} and f_{ydw} are the design yield strengths of the flange and reinforcement respectively.

7.6.5 Partially-encased members

(1) In dissipative zones where energy is dissipated by plastic bending of a composite section, the longitudinal spacing of the transverse reinforcement, s, should satisfy the requirements of **7.6.4(3)** over a length greater or equal to l_{cr} for dissipative zones at the end of a member and $2l_{cr}$ for dissipative zones in the member.

(2) In dissipative members, the shear resistance should be determined on the basis of the structural steel section alone, unless special details are provided to mobilise the shear resistance of the concrete encasement.

(3) The relationship between the ductility class of the structure and the allowable slenderness (c/t) of the flange outstand in dissipative zones is given in Table 7.3.



Key

A Additional straight bars (links)

Figure 7.8: Detail of transverse reinforcement, with the additional straight bars (links) welded to the flanges.

(4) Straight links welded to the inside of the flanges, as shown in Figure 7.8, additional to the reinforcements required by EN 1994-1-1, can delay local buckling in the dissipative zones. In this case, the limits given in Table 7.3 for flange slenderness may be increased if these bars are provided at a longitudinal spacing, s_1. which is less than the flange outstand: $s_1/c < 1,0$. For $s_1/c < 0,5$ the limits given in Table 7.3 may be increased by up to 50%. For values of $0,5 < s_1/c < 1,0$ linear interpolation may be used.

The additional straight links should also conform to the rules in **(5)** and **(6)** of this subclause.

(5) The diameter, $d_{bw,}$ of the additional straight links referred to in **(4)** of this subclause should be at least 6 mm. When transverse links are employed to delay local flange buckling as described in **(4)**, d_{bw} should be not less than the value given by expression (7.12).

(6) The additional straight links referred to in **(4)** should be welded to the flanges at both ends and the capacity of the welds should be not less than the tensile yield strength of the straight links. A clear concrete cover of at least 20 mm, but not exceeding 40 mm, should be provided to these links.

(7) The design of partially-encased composite members may take into account the resistance of the steel section alone, or the composite resistance of the steel section and of concrete encasement.

(8) The design of partially-encased members in which only the steel section is assumed to contribute to member resistance may be carried out in accordance with the provisions of Section **6**, but the capacity design provisions of **7.5.2(4)** and **(5)** and **7.5.3(3)** should be applied.

7.6.6 Filled Composite Columns

(1) The relationship between the ductility class of the structure and the allowable slenderness d/t or h/t is given in Table 7.3.

(2) The shear resistance of dissipative columns should be determined on the basis of the structural steel section or on the basis of the reinforced concrete section with the steel hollow section taken only as shear reinforcement.

(3) In non-dissipative members, the shear resistance of the column should be determined in accordance with EN 1994-1-1.

7.7 Design and detailing rules for moment frames

7.7.1 Specific criteria

(1)P **6.6.1(1)**P applies.

(2)P The composite beams shall be designed for ductility and so that the integrity of the concrete is maintained.

(3) Depending on the location of the dissipative zones, either **7.5.2(4)** or **7.5.2(5)** applies.

(4) The required hinge formation pattern should be achieved by observing the rules given in **4.4.2.3**, **7.7.3**, **7.7.4** and **7.7.5**.

7.7.2 Analysis

(1)P The analysis of the structure shall be performed on the basis of the section properties defined in **7.4**.

(2) In beams, two different flexural stiffnesses should be taken into account: EI_1 for the part of the spans submitted to positive (sagging) bending (uncracked section) and EI_2 for the part of the span submitted to negative (hogging) bending (cracked section).

(3) The analysis may alternatively be performed taking into account for the entire beam an equivalent second moment of area I_{eq} constant for the entire span:

$$I_{eq} = 0,6\, I_1 + 0,4\, I_2 \tag{7.13}$$

(4) For composite columns, the flexural stiffness is given by:

$$(EI)_c = 0,9(\, EI_a + r\, E_{cm}\, I_c + E\, I_s\,) \tag{7.14}$$

where

E and E_{cm} are the modulus of elasticity for steel and concrete respectively;

r is the reduction factor depending on the type of column cross-section;

I_a, I_c and I_s denote the second moment of area of the steel section, of the concrete and of the rebars respectively.

NOTE The value ascribed to r for use in a country may be found in its National Annex of this document. The recommended value is $r = 0,5$.

7.7.3 Rules for beams and columns

(1)P Composite T beam design shall conform to **7.6.2**. Partially encased beams shall conform to **7.6.5**.

(2)P Beams shall be verified for lateral and lateral torsional buckling in accordance with EN 1994-1-1, assuming the formation of a negative plastic moment at one end of the beam.

(3) **6.6.2(2)** applies.

(4) Composite trusses should not be used as dissipative beams.

(5)P **6.6.3(1)**P applies.

(6) In columns where plastic hinges form as stated in **7.7.1(1)**, the verification should assume that $M_{pl,Rd}$ is realised in these plastic hinges.

(7) The following expression should apply for all composite columns:

$$N_{Ed}/N_{pl,Rd} < 0,30 \tag{7.15}$$

(8) The resistance verifications of the columns should be made in accordance with EN 1994-1-1:2004, **4.8**.

(9) The column shear force V_{Ed} (from the analysis) should be limited in accordance with expression (6.4).

7.7.4 Beam to column connections

(1) The provisions given in **6.6.4** apply.

7.7.5 Condition for disregarding the composite character of beams with slab.

(1)P The plastic resistance of a beam section composite with slab (lower or upper bound plastic resistance of dissipative zones) may be computed taking into account only the steel section (design in accordance with concept c) as defined in **7.1.2**) if the slab is totally disconnected from the steel frame in a circular zone around a column of diameter $2b_{eff}$, with b_{eff} being the larger of the effective widths of the beams connected to that column.

(2) For the purposes of **(1)**P, "totally disconnected" means that there is no contact between slab and any vertical side of any steel element (e.g. columns, shear connectors, connecting plates, corrugated flange, steel deck nailed to flange of steel section).

(3) In partially encased beams, the contribution of concrete between the flanges of the steel section should be taken into account.

7.8 Design and detailing rules for composite concentrically braced frames

7.8.1 Specific criteria

(1)P **6.7.1(1)**P applies.

(2)P Columns and beams shall be either structural steel or composite.

(3)P Braces shall be structural steel.

(4) **6.7.1(2)**P applies

7.8.2 Analysis

(1) The provisions given in **6.7.2** apply.

7.8.3 Diagonal members

(1) The provisions given in **6.7.3** apply.

7.8.4 Beams and columns

(1) The provisions given in **6.7.4** apply.

7.9 Design and detailing rules for composite eccentrically braced frames

7.9.1 Specific criteria

(1)P Composite frames with eccentric bracings shall be designed so that the dissipative action will occur essentially through yielding in shear of the links. All other members shall remain elastic and failure of connections shall be prevented.

(2)P Columns, beams and braces shall be either structural steel or composite.

(3)P The braces, columns and beam segments outside the link segments shall be designed to remain elastic under the maximum forces that can be generated by the fully yielded and cyclically strain-hardened beam link.

(4)P **6.8.1(2)**P applies.

7.9.2 Analysis

(1)P The analysis of the structure is based on the section properties defined in **7.4.2**.

(2) In beams, two different flexural stiffnesses are taken into account: EI_1 for the part of the spans submitted to positive (sagging) bending (uncracked section) and EI_2 for the part of the span submitted to negative (hogging) bending (cracked section).

7.9.3 Links

(1)P Links shall be made of steel sections, possibly composite with slabs. They may not be encased.

(2) The rules on seismic links and their stiffeners given in **6.8.2** apply. Links should be of short or intermediate length with a maximum length e:

– In structures where two plastic hinges would form at link ends

$$e = 2M_{\mathrm{p,\,link}} / V_{\mathrm{p,\,link}}. \tag{7.16}$$

– In structures where one plastic hinge would form at one end of a link

$$e < M_{\mathrm{p,\,link}} / V_{\mathrm{p,\,link}} \tag{7.17}$$

The definitions of $M_{\mathrm{p,link}}$ and $V_{\mathrm{p,link}}$ are given in **6.8.2(3)**. For $M_{\mathrm{p,link}}$, only the steel components of the link section, disregarding the concrete slab, are taken into account in the evaluation.

(3) When the seismic link frames into a reinforced concrete column or an encased column, face bearing plates should be provided on both sides of the link at the face of the column and in the end section of the link. These bearing plates should conform to **7.5.4**.

(4) The design of beam/column connections adjacent to dissipative links should conform to **7.5.4**.

(5) Connections should meet the requirements of the connections of eccentrically braced steel frames as in **6.8.4**.

7.9.4 Members not containing seismic links

(1) The members not containing seismic links should conform to the rules in **6.8.3**, taking into account the combined resistance of steel and concrete in the case of composite elements and the relevant rules for members in **7.6** and in EN 1994-1-1:2004.

(2) Where a link is adjacent to a fully encased composite column, transverse reinforcement meeting the requirements of **7.6.5** should be provided above and below the link connection.

(3) In case of a composite brace under tension, only the cross-section of the structural steel section should be taken into account in the evaluation of the resistance of the brace.

7.10 Design and detailing rules for structural systems made of reinforced concrete shear walls composite with structural steel elements

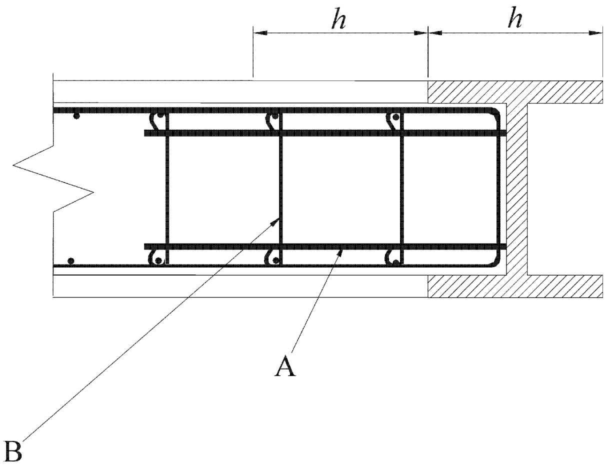
7.10.1 Specific criteria

(1)P The provisions in this subclause apply to composite structural systems belonging in one of the three types defined in **7.3.1e**.

(2)P Structural system types 1 and 2 shall be designed to behave as shear walls and dissipate energy in the vertical steel sections and in the vertical reinforcement. The infills shall be tied to the boundary elements to prevent separation.

(3)P In structural system type 1, the storey shear forces shall be carried by horizontal shear in the wall and in the interface between the wall and beams.

(4)P Structural system type 3 shall be designed to dissipate energy in the shear walls and in the coupling beams.

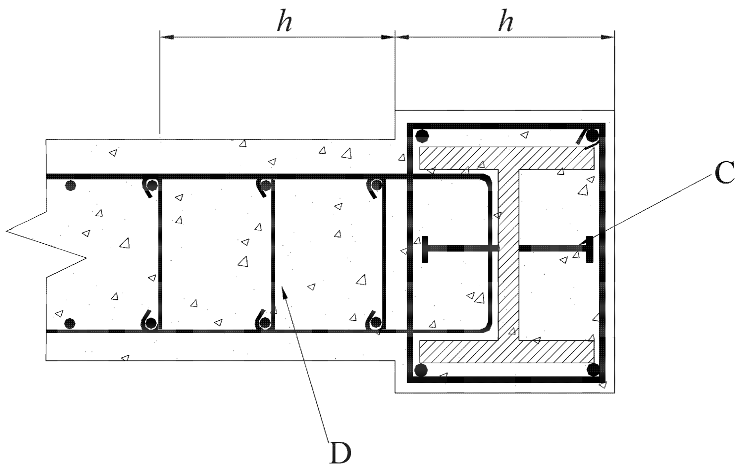


Key

A bars welded to column;

B transverse reinforcement

Figure 7.9a: Details of partially encased composite boundary elements (details of transverse reinforcements are for ductility class DCH).

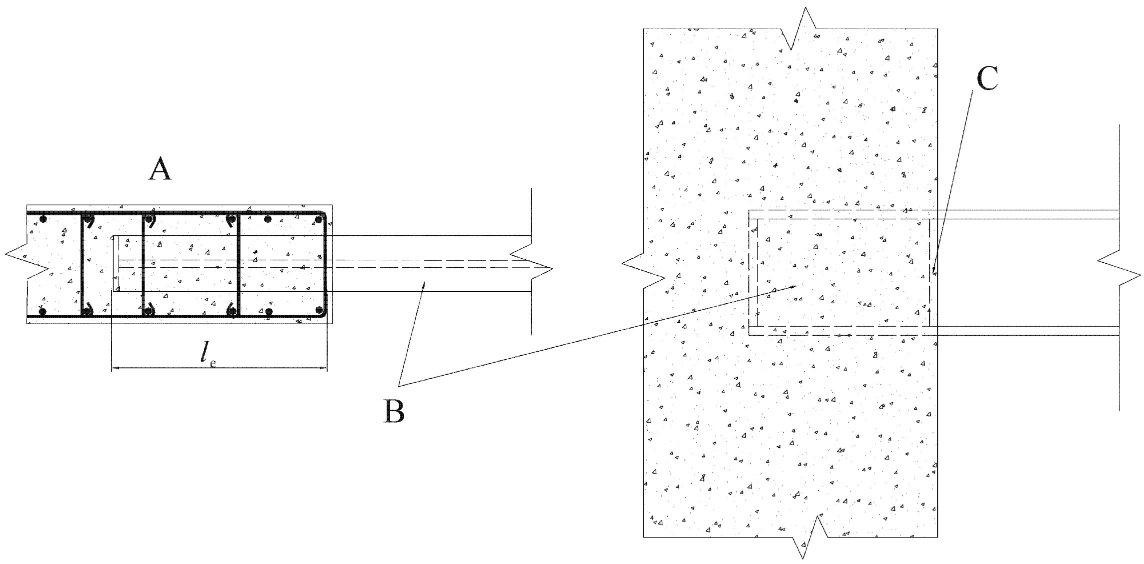


Key

C = shear connectors;

D = cross tie

Figure 7.9b: Details of fully encased composite boundary elements (details of transverse reinforcements are for ductility class DCH).



Key

A Additional wall reinforcement at embedment of steel beam;
B Steel coupling beam;
C Force bearing plate

Figure 7.10: Details of coupling beam framing into a wall (details are for ductility class DCH

7.10.2 Analysis

(1)P The analysis of the structure shall be based on the section properties defined in Section **5** for concrete walls and in **7.4.2** for composite beams.

(2)P In structural systems of type 1 or type 2, when vertical fully encased or partially encased structural steel sections act as boundary members of reinforced concrete infill panels, the analysis shall be made assuming that the seismic action effects in these vertical boundary elements are axial forces only.

(3) These axial forces should be determined assuming that the shear forces are carried by the reinforced concrete wall and that the entire gravity and overturning forces are carried by the shear wall acting composedly with the vertical boundary members.

(4) In structural system of type 3, if composite coupling beams are used, **7.7.2(2)** and **(3)** apply.

7.10.3 Detailing rules for composite walls of ductility class DCM

(1)P The reinforced concrete infill panels in Type 1 and the reinforced concrete walls in Types 2 and 3 shall meet the requirements of Section **5** for ductile walls of DCM.

(2)P Partially encased steel sections used as boundary members of reinforced concrete panels shall belong to a class of cross-section related to the behaviour factor of the structure as indicated in Table 7.3.

(3)P Fully encased structural steel sections used as boundary members in reinforced concrete panels shall be designed in accordance with **7.6.4**.

(4)P Partially encased structural steel sections used as boundary members of reinforced concrete panels shall be designed in accordance with **7.6.5**.

(5) Headed shear studs or tie reinforcement (welded to, anchored through holes in the steel members or anchored around the steel member) should be provided to transfer vertical and horizontal shear forces between the structural steel of the boundary elements and the reinforced concrete.

7.10.4 Detailing rules for coupling beams of ductility class DCM

(1)P Coupling beams shall have an embedment length into the reinforced concrete wall sufficient to resist the most adverse combination of moment and shear generated by the bending and shear strength of the coupling beam. The embedment length l_e shall be taken to begin inside the first layer of the confining reinforcement in the wall boundary member (see Figure 7.10). The embedment length l_e shall be not less than 1,5 times the height of the coupling beam

(2)P The design of beam/wall connections shall conform to **7.5.4**.

(3) The vertical wall reinforcements, defined in **7.5.4(9)** and **(10)** with design axial strength equal to the shear strength of the coupling beam, should be placed over the embedment length of the beam with two-thirds of the steel located over the first half of the embedment length. This wall reinforcement should extend a distance of at least one anchorage length above and below the flanges of the coupling beam. It is permitted to use vertical reinforcement placed for other purposes, such as for vertical boundary members, as part of the required vertical reinforcement. Transverse reinforcement should conform to **7.6**.

7.10.5 Additional detailing rules for ductility class DCH.

(1)P Transverse reinforcement for confinement of the composite boundary members, either partially or fully encased, shall be used. Reinforcement shall extend to a distance of $2h$ into the concrete walls where h is the depth of the boundary element in the plane of the wall (see Figure 7.9a) and b)).

(2)P The requirements for the links in frames with eccentric bracings apply to the coupling beams.

7.11 Design and detailing rules for composite steel plate shear walls

7.11.1 Specific criteria

(1)P Composite steel plate shear walls shall be designed to yield through shear of the steel plate.

(2) The steel plate should be stiffened by one or two sided concrete encasement and attachment to the reinforced concrete encasement in order to prevent buckling of steel.

7.11.2 Analysis

(1) The analysis of the structure should be based on the materials and section properties defined in **7.4.2** and **7.6**.

7.11.3 Detailing rules

(1)P It shall be checked that

$$V_{Ed} < V_{Rd} \tag{7.18}$$

with the shear resistance given by:

$$V_{Rd} = A_{pl} \times f_{yd} / \sqrt{3} \tag{7.19}$$

where

f_{yd} is the design yield strength of the plate; and

A_{pl} is the horizontal area of the plate.

(2)P The connections between the plate and the boundary members (columns and beams), as well as the connections between the plate and the concrete encasement, shall be designed such that full yield strength of the plate can be developed.

(3)P The steel plate shall be continuously connected on all edges to structural steel framing and boundary members with welds and/or bolts to develop the yield strength of the plate in shear.

(4)P The boundary members shall be designed to meet the requirements of **7.10**.

(5) The concrete thickness should be not less than 200 mm when it is provided on one side and 100 mm on each side when provided on both sides.

(6) The minimum reinforcement ratio in both directions shall be not less than 0,25%.

(7) Openings in the steel plate shall be stiffened as required by analysis.

7.12 Control of design and construction

(1) For the control of design and construction, **6.11** applies.

8 SPECIFIC RULES FOR TIMBER BUILDINGS

8.1 General

8.1.1 Scope

(1)P For the design of timber buildings EN 1995 applies. The following rules are additional to those given in EN 1995.

8.1.2 Definitions

(1)P The following terms are used in this section with the following meanings:

static ductility
ratio between the ultimate deformation and the deformation at the end of elastic behaviour evaluated in quasi-static cyclic tests (see **8.3(3)**P);

semi-rigid joints
joints with significant flexibility, the influence of which has to be taken into account in structural analysis in accordance with EN 1995 (e.g. dowel-type joints);

rigid joints
joints with negligible flexibility in accordance with EN 1995 (e.g. glued solid timber joints);

Dowel-type joints
joints with dowel-type mechanical fasteners (nails, staples, screws, dowels, bolts etc.) loaded perpendicular to their axis;

Carpenter joints
joints, where loads are transferred by means of pressure areas and without mechanical fasteners (e.g. skew notch, tenon, half joint).

8.1.3 Design concepts

(1)P Earthquake-resistant timber buildings shall be designed in accordance with one of the following concepts:

a) dissipative structural behaviour;

b) low-dissipative structural behaviour.

(2) In concept a) the capability of parts of the structure (dissipative zones) to resist earthquake actions out of their elastic range is taken into account. When using the design spectrum defined in **3.2.2.5**, the behaviour factor q may be taken as being greater than 1,5. The value of q depends on the ductility class (see **8.3**).

(3)P Structures designed in accordance with concept a) shall belong to structural ductility classes M or H. A structure belonging to a given ductility class shall meet specific requirements in one or more of the following aspects: structural type, type and rotational ductility capacity of connections.

(4)P Dissipative zones shall be located in joints and connections, whereas the timber members themselves shall be regarded as behaving elastically.

(5) The properties of dissipative zones should be determined by tests either on single joints, on whole structures or on parts thereof in accordance with prEN 12512.

(6) In concept b) the action effects are calculated on the basis of an elastic global analysis without taking into account non-linear material behaviour. When using the design spectrum defined in **3.2.2.5**, the behaviour factor q should not be taken greater than 1,5. The resistance of the members and connections should be calculated in accordance with EN 1995-1:2004 without any additional requirements. This concept is termed ductility class L (low) and is appropriate only for certain structural types (see Table 8.1).

8.2 Materials and properties of dissipative zones

(1)P The relevant provisions of EN 1995 apply. With respect to the properties of steel elements, EN 1993 applies.

(2)P When using the concept of dissipative structural behaviour, the following provisions apply:

a) only materials and mechanical fasteners providing appropriate low cycle fatigue behaviour may be used in joints regarded as dissipative zones;

b) glued joints shall be considered as non-dissipative zones;

c) carpenter joints may only be used when they can provide sufficient energy dissipation capacity, without presenting risks of brittle failure in shear or tension perpendicular to the grain. The decision on their use shall be based on appropriate test results.

(3) **(2)**P a) of this subclause is deemed to be satisfied if **8.3(3)**P is fulfilled.

(4) For sheathing-material in shear walls and diaphragms, **(2)**P a) is deemed to be satisfied, if the following conditions are met:

a) particleboard-panels have a density of at least 650 kg/m^3;

b) plywood-sheathing is at least 9 mm thick;

c) particleboard - and fibreboard-sheathing are at least 13 mm thick.

(5)P Steel material for connections shall conform to the following conditions:

a) all connection elements made of cast steel shall fulfil the relevant requirements in EN 1993;

b) The ductility properties of the connections in trusses and between the sheathing material and the timber framing in Ductility Class M or H structures (see (**8.3**)) shall be tested for compliance with **8.3(3)**P by cyclic tests on the relevant combination of the connected parts and fastener.

8.3 Ductility classes and behaviour factors

(1)P Depending on their ductile behaviour and energy dissipation capacity under seismic actions, timber buildings shall be assigned to one of the three ductility classes L, M or H as given in Table 8.1, where the corresponding upper limit values of the behaviour factors are also given.

NOTE Geographical limitations on the use of ductility classes M and H may be found in the relevant National Annex.

Table 8.1: Design concept, Structural types and upper limit values of the behaviour factors for the three ductility classes.

Design concept and ductility class	q	Examples of structures
Low capacity to dissipate energy - DCL	1,5	Cantilevers; Beams; Arches with two or three pinned joints; Trusses joined with connectors.
Medium capacity to dissipate energy - DCM	2	Glued wall panels with glued diaphragms, connected with nails and bolts; Trusses with doweled and bolted joints; Mixed structures consisting of timber framing (resisting the horizontal forces) and non-load bearing infill.
	2,5	Hyperstatic portal frames with doweled and bolted joints (see **8.1.3(3)**P).
High capacity to dissipate energy - DCH	3	Nailed wall panels with glued diaphragms, connected with nails and bolts; Trusses with nailed joints.
	4	Hyperstatic portal frames with doweled and bolted joints (see **8.1.3(3)**P).
	5	Nailed wall panels with nailed diaphragms, connected with nails and bolts.

(2) If the building is non-regular in elevation (see **4.2.3.3**) the q-values listed in Table 8.1 should be reduced by 20%, but need not be taken less than q = 1,5 (see **4.2.3.1(7)** and Table 4.1).

(3)P In order to ensure that the given values of the behaviour factor may be used, the dissipative zones shall be able to deform plastically for at least three fully reversed cycles at a static ductility ratio of 4 for ductility class M structures and at a static ductility ratio of 6 for ductility class H structures, without more than a 20% reduction of their resistance.

(4) The provisions of **(3)**P of this subclause and of **8.2(2)** a) and **8.2(5)** b) may be regarded as satisfied in the dissipative zones of all structural types if the following provisions are met:

a) in doweled, bolted and nailed timber-to-timber and steel-to-timber joints, the minimum thickness of the connected members is $10 \cdot d$ and the fastener-diameter d does not exceed 12 mm;

b) In shear walls and diaphragms, the sheathing material is wood-based with a minimum thickness of $4d$, where the nail diameter d does not exceed 3,1 mm.

If the above requirements are not met, but the minimum member thickness of $8d$ and $3d$ for case a) and case b), respectively, is assured, reduced upper limit values for the behaviour factor q, as given in Table 8.2, should be used.

Table 8.2: Structural types and reduced upper limits of behaviour factors

Structural types	Behaviour factor q
Hyperstatic portal frames with doweled and bolted joints	2,5
Nailed wall panels with nailed diaphragms	4,0

(5) For structures having different and independent properties in the two horizontal directions, the q factors to be used for the calculation of the seismic action effects in each main direction should correspond to the properties of the structural system in that direction and can be different.

8.4 Structural analysis

(1)P In the analysis the slip in the joints of the structure shall be taken into account.

(2)P An E_0-modulus-value for instantaneous loading (10% higher than the short term one) shall be used.

(3) Floor diaphragms may be considered as rigid in the structural model without further verification, if both of the following conditions are met:

a) the detailing rules for horizontal diaphragms given in **8.5.3** are applied;

and

b) their openings do not significantly affect the overall in-plane rigidity of the floors.

8.5 Detailing rules

8.5.1 General

(1)P The detailing rules given in **8.5.2** and **8.5.3** apply for earthquake-resistant parts of structures designed in accordance with the concept of dissipative structural behaviour (Ductility classes M and H).

(2)P Structures with dissipative zones shall be designed so that these zones are located mainly in those parts of the structure where yielding or local buckling or other phenomena due to hysteretic behaviour do not affect the overall stability of the structure.

8.5.2 Detailing rules for connections

(1)P Compression members and their connections (e.g. carpenter joints), which may fail due to deformations caused by load reversals, shall be designed in such a way that they are prevented from separating and remain in their original position.

(2)P Bolts and dowels shall be tightened and tight fitted in the holes. Large bolts and dowels (d > 16 mm) shall not be used in timber-to-timber and steel-to-timber connections, except in combination with timber connectors.

(3) Dowels, smooth nails and staples should not be used without additional provision against withdrawal.

(4) In the case of tension perpendicular to the grain, additional provisions should be met to avoid splitting (e.g. nailed metal or plywood plates).

8.5.3 Detailing rules for horizontal diaphragms

(1)P For horizontal diaphragms under seismic actions EN 1995-1-1:2004 applies with the following modifications:

a) the increasing factor 1,2 for resistance of fasteners at sheet edges shall not be used;

b) when the sheets are staggered, the increasing factor of 1,5 for the nail spacing along the discontinuous panel edges shall not be used;

c) the distribution of the shear forces in the diaphragms shall be evaluated by taking into account the in-plan position of the lateral load resisting vertical elements.

(2)P All sheathing edges not meeting on framing members shall be supported on and connected to transverse blocking placed between the wooden beams. Blocking shall also be provided in the horizontal diaphragms above the lateral load resisting vertical elements (e.g. walls).

(3)P The continuity of beams shall be ensured, including the trimmer joists in areas where the diaphragm is disturbed by holes.

(4)P Without intermediate transverse blocking over the full height of the beams, the height-to-width ratio (h/b) of the timber beams should be less than 4.

(5)P If $a_g.S \geq 0{,}2 \cdot g$ the spacing of fasteners in areas of discontinuity shall be reduced by 25%, but not to less than the minimum spacing given in EN 1995-1:2004.

(6)P When floors are considered as rigid in plan for structural analysis, there shall be no change of span-direction of the beams over supports, where horizontal forces are transferred to vertical elements (e.g. shear-walls).

8.6 Safety verifications

(1)P The strength values of the timber material shall be determined taking into account the k_{mod}-values for instantaneous loading in accordance with EN 1995-1-1:2004.

(2)P For ultimate limit state verifications of structures designed in accordance with the concept of non-dissipative structural behaviour (Ductility class L), the partial factors for material properties γ_M for fundamental load combinations from EN 1995 apply.

(3)P For ultimate limit state verifications of structures designed in accordance with the concept of dissipative structural behaviour (Ductility classes M or H), the partial factors for material properties γ_M for accidental load combinations from EN 1995 apply.

(4)P In order to ensure the development of cyclic yielding in the dissipative zones, all other structural members and connections shall be designed with sufficient overstrength. This overstrength requirement applies especially to:

- anchor-ties and any connections to massive sub-elements;
- connections between horizontal diaphragms and lateral load resisting vertical elements.

(5) Carpenter joints do not present risks of brittle failure if the verification of the shear stress in accordance with EN 1995 is made with an additional partial factor of 1,3.

8.7 Control of design and construction

(1)P The provisions given in EN 1995 apply.

(2)P The following structural elements shall be identified on the design drawings and specifications for their special control during construction shall be provided:

- anchor-ties and any connections to foundation elements;
- diagonal tension steel trusses used for bracing;
- connections between horizontal diaphragms and lateral load resisting vertical elements;
- connections between sheathing panels and timber framing in horizontal and vertical diaphragms.

(3)P The special construction control shall refer to the material properties and the accuracy of execution.

9 SPECIFIC RULES FOR MASONRY BUILDINGS

9.1 Scope

(1)P This section applies to the design of buildings of unreinforced, confined and reinforced masonry in seismic regions.

(2)P For the design of masonry buildings EN 1996 applies. The following rules are additional to those given in EN 1996.

9.2 Materials and bonding patterns

9.2.1 Types of masonry units

(1) Masonry units should have sufficient robustness in order to avoid local brittle failure.

NOTE The National Annex may select the type of masonry units from EN 1996-1:2004, Table 3.1 that satisfy **(1)**.

9.2.2 Minimum strength of masonry units

(1) Except in cases of low seismicity, the normalised compressive strength of masonry units, derived in accordance with EN 772-1, should be not less than the minimum values as follows:

- normal to the bed face: $f_{b,min}$;
- parallel to the bed face in the plane of the wall: $f_{bh,min}$.

NOTE The values ascribed to $f_{b,min}$ and $f_{b,min}$ for use in a country may be found in its National Annex of this document. The recommended values are $f_{b,min}$ = 5 N/mm^2 $f_{bh,min}$ = 2 N/mm^2.

9.2.3 Mortar

(1) A minimum strength is required for mortar, $f_{m,min}$, which generally exceeds the minimum specified in EN 1996.

NOTE The value ascribed to $f_{m,min}$ for use in a country may be found in its National Annex of this document. The recommended value is $f_{m,min}$ = 5 N/mm^2 for unreinforced or confined masonry and $f_{m,min}$ = 10 N/mm^2 for reinforced masonry.

9.2.4 Masonry bond

(1) There are three alternative classes of perpend joints:

a) joints fully grouted with mortar;

b) ungrouted joints;

c) ungrouted joints with mechanical interlocking between masonry units.

NOTE The National Annex may specify which ones among the three classes above will be allowed to be used in a country or parts of the country.

9.3 Types of construction and behaviour factors

(1) Depending on the masonry type used for the seismic resistant elements, masonry buildings should be assigned to one of the following types of construction:

a) unreinforced masonry construction;

b) confined masonry construction;

c) reinforced masonry construction;

NOTE 1 Construction with masonry systems which provide an enhanced ductility of the structure is also included (see Note 2 to Table 9.1).

NOTE 2 Frames with infill masonry are not covered in this section.

(2) Due to its low tensile strength and low ductility, unreinforced masonry that follows the provisions of EN 1996 alone is considered to offer low-dissipation capacity (DCL) and its use should be limited, provided that the effective thickness of walls, t_{ef}, is not less than a minimum value, $t_{ef,min}$.

NOTE 1 The conditions under which unreinforced masonry that follows the provisions of EN 1996 alone may be used in a country, may be found in its National Annex to this document. Such use is recommended only in low seismicity cases (see **3.2.1(4)**)

NOTE 2 The value ascribed to $t_{ef,min}$ for use in a country of unreinforced masonry that follows the provisions of EN 1996 alone, may be found in its National Annex of this document. The recommended values of $t_{ef,min}$ are those in the 2nd column, 2nd and 3rd rows of Table 9.2.

(3) For the reasons noted in **(2)** of this subclause, unreinforced masonry satisfying the provisions of the present Eurocode may not be used if the value of $a_g.S$, exceeds a certain limit, $a_{g,urm}$.

NOTE The value ascribed to $a_{g,urm}$ for use in a country may be found in its National Annex of this document. This value should not be less than that corresponding to the threshold for the low seismicity cases. The value ascribed to $a_{g,urm}$ should be consistent with the values adopted for the minimum strength of masonry units, $f_{b,min}$, $f_{bh,min}$ and of mortar, $f_{m,min}$. For the values recommended in the Notes to **9.2.2** and **9.2.3**, the recommended value of $a_{g,urm}$ is 0,20 g.

(4) For types a) to c) the ranges of permissible values of the upper limit value of the behaviour factor q are given in Table 9.1.

Table 9.1: Types of construction and upper limit of the behaviour factor

Type of construction	Behaviour factor q
Unreinforced masonry in accordance with EN 1996 alone (recommended only for low seismicity cases).	1,5
Unreinforced masonry in accordance with EN 1998-1	1,5 - 2,5
Confined masonry	2,0 – 3,0
Reinforced masonry	2,5 - 3,0

NOTE 1 The upper limit values ascribed to q for use in a country (within the ranges of Table 9.1) may be found in its National Annex. The recommended values are the lower limits of the ranges in Table 9.1.

NOTE 2 For buildings constructed with masonry systems which provide an enhanced ductility of the structure, specific values of the behaviour factor q may be used, provided that the system and the related values for q are verified experimentally. The values ascribed to q for use in a country for such buildings may be found in its National Annex of these document.

(5) If the building is non-regular in elevation (see **4.2.3.3**) the q-values listed in Table 9.1 should be reduced by 20%, but need not be taken less than q = 1,5 (see **4.2.3.1(7)** and Table 4.1)

9.4 Structural analysis

(1)P The structural model for the analysis of the building shall represent the stiffness properties of the entire system.

(2)P The stiffness of the structural elements shall be evaluated taking into account both their flexural and shear flexibility and, if relevant, their axial flexibility. Uncracked elastic stiffness may be used for analysis or, preferably and more realistically, cracked stiffness in order to account for the influence of cracking on deformations and to better approximate the slope of the first branch of a bilinear force-deformation model for the structural element.

(3) In the absence of an accurate evaluation of the stiffness properties, substantiated by rational analysis, the cracked bending and shear stiffness may be taken as one half of the gross section uncracked elastic stiffness.

(4) In the structural model masonry spandrels may be taken into account as coupling beams between two wall elements if they are regularly bonded to the adjoining walls and connected both to the floor tie beam and to the lintel below.

(5) If the structural model takes into account the coupling beams, a frame analysis may be used for the determination of the action effects in the vertical and horizontal structural elements.

(6) The base shear in the various walls, as obtained by the linear analysis described in Section **4**, may be redistributed among the walls, provided that:

a) the global equilibrium is satisfied (i.e. the same total base shear and position of the force resultant is achieved);

b) the shear in any wall is neither reduced more than 25 %, nor increased by more than 33%; and

c) the consequences of the redistribution for the diaphragm(s) are taken into account.

9.5 Design criteria and construction rules

9.5.1 General

(1)P Masonry buildings shall be composed of floors and walls, which are connected in two orthogonal horizontal directions and in the vertical direction.

(2)P The connection between the floors and walls shall be provided by steel ties or reinforced concrete ring beams.

(3) Any type of floors may be used, provided that the general requirements of continuity and effective diaphragm action are satisfied.

(4)P Shear walls shall be provided in at least two orthogonal directions.

(5) Shear walls should conform to certain geometric requirements, namely:

a) the effective thickness of shear walls, t_{ef}, may not be less than a minimum value, $t_{ef,min}$;

b) the ratio h_{ef}/t_{ef} of the effective wall height (see EN 1996-1-1:2004) to its effective thickess may not exceed a maximum value, $(h_{ef}/t_{ef})_{max}$; and

c) the ratio of the length of the wall, l, to the greater clear height, h, of the openings adjacent to the wall, may not be less than a minimum value, $(l/h)_{min}$.

NOTE The values ascribed to $t_{ef,min}$, $(h_{ef}/t_{ef})_{max}$ and $(l/h)_{min}$, for use in a country may be found in its National Annex of this document. The recommended values of $t_{ef,min}$, $(h_{ef}/t_{ef})_{max}$ and $(l/h)_{min}$ are listed in Table 9.2.

Table 9.2: Recommended geometric requirements for shear walls

Masonry type	$t_{ef,min}$ (mm)	$(h_{ef}/t_{ef})_{max}$	$(l/h)_{min}$
Unreinforced, with natural stone units	350	9	0,5
Unreinforced, with any other type of units	240	12	0,4
Unreinforced, with any other type of units, in cases of low seismicity	170	15	0,35
Confined masonry	240	15	0,3
Reinforced masonry	240	15	No restriction
Symbols used have the following meaning: t_{ef} thickness of the wall (see EN 1996-1-1:2004); h_{ef} effective height of the wall (see EN 1996-1-1:2004); h greater clear height of the openings adjacent to the wall; l length of the wall.			

(6) Shear walls not conforming to the minimum geometric requirements of **(5)** of this subclause may be considered as secondary seismic elements. They should conform to **9.5.2(1)** and **(2)**.

9.5.2 Additional requirements for unreinforced masonry satisfying EN 1998-1

(1) Horizontal concrete beams or, alternatively, steel ties should be placed in the plane of the wall at every floor level and in any case with a vertical spacing not more than 4 m. These beams or ties should form continuous bounding elements physically connected to each other .

NOTE Beams or ties continuous over the entire periphery are essential.

(2) The horizontal concrete beams should have longitudinal reinforcement with a cross-sectional area of not less than 200 mm^2.

9.5.3 Additional requirements for confined masonry

(1)P The horizontal and vertical confining elements shall be bonded together and anchored to the elements of the main structural system.

(2)P In order to obtain an effective bond between the confining elements and the masonry, the concrete of the confining elements shall be cast after the masonry has been built.

(3) The cross-sectional dimensions of both horizontal and vertical confining elements may not be less than 150 mm. In double-leaf walls the thickness of confining elements should assure the connection of the two leaves and their effective confinement.

(4) Vertical confining elements should be placed:

– at the free edges of each structural wall element;

– at both sides of any wall opening with an area of more than 1,5 m^2;

- within the wall if necessary in order not to exceed a spacing of 5 m between the confining elements;
- at the intersections of structural walls, wherever the confining elements imposed by the above rules are at a distance larger than 1,5 m.

(5) Horizontal confining elements shall be placed in the plane of the wall at every floor level and in any case with a vertical spacing of not more than 4 m.

(6) The longitudinal reinforcement of confining elements may not have a cross-sectional area less than 300 mm^2, nor than 1% of the cross-sectional area of the confining element.

(7) Stirrups not less than 5 mm in diameter and spaced not more than 150 mm should be provided around the longitudinal reinforcement.

(8) Reinforcing steel should be of Class B or C in accordance with EN 1992-1-1:2004, Table C.1.

(9) Lap splices may not be less than 60 bar diameters in length.

9.5.4 Additional requirements for reinforced masonry

(1) Horizontal reinforcement should be placed in the bed joints or in suitable grooves in the units, with a vertical spacing not exceeding 600 mm.

(2) Masonry units with recesses should accommodate the reinforcement needed in lintels and parapets.

(3) Reinforcing steel bars of not less than 4 mm diameter, bent around the vertical bars at the edges of the wall, should be used.

(4) The minimum percentage of horizontal reinforcement in the wall, normalised with respect to the gross area of the section, should not be less than 0,05 %.

(5)P High percentages of horizontal reinforcement leading to compressive failure of the units prior to the yielding of the steel, shall be avoided.

(6) The vertical reinforcement spread in the wall, as a percentage of the gross area of the horizontal section of the wall, should not be less than 0,08%.

(7) Vertical reinforcement should be located in pockets, cavities or holes in the units.

(8) Vertical reinforcements with a cross-sectional area of not less than 200 mm^2 should be arranged:

- at both free edges of every wall element;
- at every wall intersection;
- within the wall, in order not to exceed a spacing of 5 m between such reinforcements.

(9) **9.5.3(7)**, **(8)** and **(9)** apply.

(10)P The parapets and lintels shall be regularly bonded to the masonry of the adjoining walls and linked to them by horizontal reinforcement.

9.6 Safety verification

(1)P The verification of the building's safety against collapse shall be explicitly provided, except for buildings satisfying the rules for "simple masonry buildings" given in **9.7.2**.

(2)P For the verification of safety against collapse, the design resistance of each structural element shall be evaluated in accordance with EN 1996-1-1:2004.

(3) In ultimate limit state verifications for the seismic design situation, partial factors γ_m for masonry properties and γ_s for reinforcing steel should be used.

NOTE The values ascribed to the material partial factors γ_m and γ_s for use in a country in the seismic design situation may be found in its National Annex of this document. The recommended value for γ_m is 2/3 of the value specified in the National Annex to EN 1996-1-1:2004, but not less than 1,5. The recommended value for γ_s is 1,0.

9.7 Rules for "simple masonry buildings"

9.7.1 General

(1) Buildings belonging to importance classes I or II and conforming to **9.2, 9.5** and **9.7.2** may be classified as "simple masonry buildings".

(2) For such buildings an explicit safety verification in accordance with **9.6** is not mandatory.

9.7.2 Rules

(1) Depending on the product $a_g \cdot S$ at the site and the type of construction, the allowable number of storeys above ground, n, should be limited and walls in two orthogonal directions with a minimum total cross-sectional area A_{min}, in each direction, should be provided. The minimum cross-sectional area is expressed as a minimum percentage, $p_{A,min}$, of the total floor area per storey.

NOTE The values ascribed to n and $p_{A,min}$ for use in a country may by found in its National Annex of this document. Recommended values are given in Table 9.3. These values, which depend also on a corrective factor k, are based on a minimum unit strength of 12 N/mm² for unreinforced masonry and 5 N/mm² for confined and reinforced masonry, respectively.
For buildings where at least 70% of the shear walls under consideration are longer than 2m, the factor k is given by $k = 1 + (l_{av} - 2)/4 \leq 2$ where l_{av} is the average length, expressed in m, of the shear walls considered. For other cases $k = 1$.
Independently of the value of k, the limitation of use of unreinforced masonry presented in **9.3(3)** should be respected.
A further distinction for different unit strengths, types of construction and use of k may be found in the National Annex.

Table 9.3: Recommended allowable number of storeys above ground and minimum area of shear walls for "simple masonry buildings".

Acceleration at site $a_g.S$		$\leq 0{,}07\ k \cdot g$	$\leq 0{,}10\ k \cdot g$	$\leq 0{,}15\ k \cdot g$	$\leq 0{,}20\ k \cdot g$
Type of construction	Number of storeys (n)**	Minimum sum of cross-sections areas of horizontal shear walls in each direction, as percentage of the total floor area per storey ($p_{A,min}$)			
Unreinforced masonry	1	2,0%	2,0%	3,5%	n/a
	2	2,0%	2,5%	5,0%	n/a
	3	3,0%	5,0%	n/a	n/a
	4	5,0 %	n/a*	n/a	n/a
Confined masonry	2	2,0%	2,5%	3,0%	3,5%
	3	2,0%	3,0%	4,0%	n/a
	4	4,0%	5,0%	n/a	n/a
	5	6,0%	n/a	n/a	n/a
Reinforced masonry	2	2,0%	2,0%	2,0%	3,5%
	3	2,0%	2,0%	3,0%	5,0%
	4	3,0%	4,0%	5,0%	n/a
	5	4,0%	5,0%	n/a	n/a

* n/a means “not acceptable”.

** Roof space above full storeys is not included in the number of storeys.

(2) The plan configuration of the building should fulfil all the following conditions:

a) The plan should be approximately rectangular;

b) The ratio between the length of the small side and the length of the long side in plan should be not less than a minimum value, λ_{min};

NOTE The value to be ascribed to λ_{min} for use in a country may be found in its National Annex of this document. The recommended value of λ_{min} is 0,25.

c) The area of projections of recesses from the rectangular shape should be not greater than a percentage p_{max} of the total floor area above the level considered.

NOTE The value to be ascribed to p_{max} for use in a country may be found in its National Annex of this documentThe recommended value is 15%.

(3) The shear walls of the building should fulfil all of the following conditions:

a) the building should be stiffened by shear walls, arranged almost symmetrically in plan in two orthogonal directions;

b) a minimum of two parallel walls should be placed in two orthogonal directions, the length of each wall being greater than 30 % of the length of the building in the direction of the wall under consideration;

c) at least for the walls in one direction, the distance between these walls should be greater than 75 % of the length of the building in the other direction;

d) at least 75 % of the vertical loads should be supported by the shear walls;

e) shear walls should be continuous from the top to the bottom of the building.

(4) In cases of low seismicity (see **3.2.1(4)**) the wall length required in **(3)**b of this subclause may be provided by the cumulative length of the shear walls (see **9.5.1(5)**) in one axis, separated by openings. In this case, at least one shear wall in each direction should have a length, *l,* not less than that corresponding to twice the minimum value of *l/h* defined in **9.5.1(5)**c.

(5) In both orthogonal horizontal directions the difference in mass and in the horizontal shear wall cross-sectional area between adjacent storeys should be limited to a maximum value of $\Delta_{m,max}$ and $\Delta_{A,max}$.

NOTE The values to be ascribed to $\Delta_{m,max}$ and to $\Delta_{A,max}$ for use in a country may be found in its National Annex to this document. The recommended values are $\Delta_{m,max}$ = 20%, $\Delta_{A,max}$ = 20%.

(6) For unreinforced masonry buildings, walls in one direction should be connected with walls in the orthogonal direction at a maximum spacing of 7 m.

10 BASE ISOLATION

10.1 Scope

(1)P This section covers the design of seismically isolated structures in which the isolation system, located below the main mass of the structure, aims at reducing the seismic response of the lateral-force resisting system.

(2) The reduction of the seismic response of the lateral-force resisting system may be obtained by increasing the fundamental period of the seismically isolated structure, by modifying the shape of the fundamental mode and by increasing the damping, or by a combination of these effects. The isolation system may consist of linear or non-linear springs and/or dampers.

(3) Specific rules concerning base isolation of buildings are given in this section.

(4) This section does not cover passive energy dissipation systems that are not arranged on a single interface, but are distributed over several storeys or levels of the structure.

10.2 Definitions

(1)P The following terms are used in this section with the following meanings:

isolation system
collection of components used for providing seismic isolation, which are arranged over the isolation interface

NOTE These are usually located below the main mass of the structure.

isolation interface
surface which separates the substructure and the superstructure and where the isolation system is located.

NOTE Arrangement of the isolation interface at the base of the structure is usual in buildings, tanks and silos. In bridges the isolation system is usually combined with the bearings and the isolation interface lies between the deck and the piers or abutments.

isolator units
elements constituting the isolation system.
The devices considered in this section consist of laminated elastomeric bearings, elasto-plastic devices, viscous or friction dampers, pendulums, and other devices the behaviour of which conforms to **10.1(2)**. Each unit provides a single or a combination of the following functions:

- vertical–load carrying capability combined with increased lateral flexibility and high vertical rigidity;
- energy dissipation, either hysteretic or viscous;
- recentering capability;
- lateral restraint (sufficient elastic rigidity) under non-seismic service lateral loads.

Substructure
part of the structure which is located under the isolation interface, including the foundation

NOTE The lateral flexibility of the substructure(s) is generally negligible in comparison to that of the isolation system, but this is not always the case (for instance in bridges).

Superstructure
part of the structure which is isolated and is located above the isolation interface

Full isolation
the superstructure is fully isolated if, in the design seismic situation, it remains within the elastic range. Otherwise, the superstructure is partially isolated.

Effective stiffness centre
stiffness centre above the isolation interface i.e. including the flexibility of the isolator units and of the substructure(s).

NOTE In buildings, tanks and similar structures, the flexibility of the superstructure may be neglected in the determination of this point, which then coincides with the stiffness centre of the isolator units.

Design displacement (of the isolation system in a principal direction)
maximum horizontal displacement at the effective stiffness centre between the top of the substructure and the bottom of the superstructure, occurring under the design seismic action

Total design displacement (of an isolator unit in a principal direction)
maximum horizontal displacement at the location of the unit, including that due to the design displacement and to the global rotation due to torsion about the vertical axis

Effective stiffness (of the isolation system in a principal direction)
ratio of the value of the total horizontal force transferred through the isolation interface when the design displacement takes place in the same direction, divided by the absolute value of that design displacement (secant stiffness).

NOTE The effective stiffness is generally obtained by iterative dynamic analysis.

Effective Period
fundamental period, in the direction considered, of a single degree of freedom system having the mass of the superstructure and the stiffness equal to the effective stiffness of the isolation system;

Effective damping (of the isolation system in a principal direction)
value of the effective viscous damping that corresponds to the energy dissipated by the isolation system during cyclic response at the design displacement.

10.3 Fundamental requirements

(1)P The fundamental requirements in **2.1** and in the corresponding Parts of this Eurocode, according to the type of structure considered, shall be satisfied.

(2)P Increased reliability is required for the isolating devices. This shall be effected by applying a magnification factor γ_x on seismic displacements of each unit.

NOTE The value to be ascribed to γ_x for use in a country may be found in its National Annex of this document, depending on the type of isolating device used. For buildings the recommended value is γ_x =1,2.

10.4 Compliance criteria

(1)P In order to conform to the fundamental requirements, the limit states defined in **2.2.1(1)** shall be checked.

(2)P At the damage limitation state, all lifelines crossing the joints around the isolated structure shall remain within the elastic range.

(3) In buildings, at the damage limitation state, the interstorey drift should be limited in the substructure and the superstructure in accordance with **4.4.3.2**.

(4)P At the ultimate limit state, the ultimate capacity of the isolating devices in terms of strength and deformability shall not be exceeded, with the relevant safety factors (see **10.10(6)**P).

(5) Only full isolation is considered in the present section.

(6) Although it may be acceptable that, in certain cases, the substructure has inelastic behaviour, it is considered in the present section that it remains in the elastic range.

(7) At the Ultimate limit state, the isolating devices may attain their ultimate capacity, while the superstructure and the substructure remain in the elastic range. Then there is no need for capacity design and ductile detailing in either the superstructure or the substructure.

(8)P At the Ultimate limit state, gas lines and other hazardous lifelines crossing the joints separating the superstructure from the surrounding ground or constructions shall be designed to accommodate safely the relative displacement between the isolated superstructure and the surrounding ground or constructions, taking into account the γ_x factor defined in **10.3(2)**P.

10.5 General design provisions

10.5.1 General provisions concerning the devices

(1)P Sufficient space between the superstructure and substructure shall be provided, together with other necessary arrangements, to allow inspection, maintenance and replacement of the devices during the lifetime of the structure.

(2) If necessary, the devices should be protected from potential hazardous effects, such as fire, and chemical or biological attack.

(3) Materials used in the design and construction of the devices should conform to the relevant existing norms.

10.5.2 Control of undesirable movements

(1) To minimise torsional effects, the effective stiffness centre and the centre of damping of the isolation system should be as close as possible to the projection of the centre of mass on the isolation interface.

(2) To minimise different behaviour of isolating devices, the compressive stress induced in them by the permanent actions should be as uniform as possible.

(3)P Devices shall be fixed to the superstructure and the substructure.

(4)P The isolation system shall be designed so that shocks and potential torsional movements are controlled by appropriate measures.

(5) Requirement **(4)**P concerning shocks is deemed to be satisfied if potential shock effects are avoided through appropriate devices (e.g. dampers, shock-absorbers, etc.).

10.5.3 Control of differential seismic ground motions

(1) The structural elements located above and below the isolation interface should be sufficiently rigid in both horizontal and vertical directions, so that the effects of differential seismic ground displacements are minimised. This does not apply to bridges or elevated structures, where the piles and piers located under the isolation interface may be deformable.

(2) In buildings, **(1)** is considered satisfied if all the conditions stated below are satisfied:

a) A rigid diaphragm is provided above and under the isolation system, consisting of a reinforced concrete slab or a grid of tie-beams, designed taking into account all relevant local and global modes of buckling. This rigid diaphragm is not necessary if the structures consist of rigid boxed structures;

b) The devices constituting the isolation system are fixed at both ends to the rigid diaphragms defined above, either directly or, if not practicable, by means of vertical elements, the relative horizontal displacement of which in the seismic design situation should be lower than 1/20 of the relative displacement of the isolation system.

10.5.4 Control of displacements relative to surrounding ground and constructions

(1)P Sufficient space shall be provided between the isolated superstructure and the surrounding ground or constructions, to allow its displacement in all directions in the seismic design situation.

10.5.5 Conceptual design of base isolated buildings

(1) The principles of conceptual design for base isolated buildings should be based on those in Section **2** and in **4.2**, with additional provisions given in this section.

10.6 Seismic action

(1)P The two horizontal and the vertical components of the seismic action shall be assumed to act simultaneously.

(2) Each component of the seismic action is defined in **3.2**, in terms of the elastic spectrum for the applicable local ground conditions and design ground acceleration a_g.

(3) In buildings of importance class IV, site-specific spectra including near source effects should also be taken into account, if the building is located at a distance less than 15 km from the nearest potentially active fault with a magnitude $M_s \geq 6{,}5$. Such spectra should not be taken as being less than the standard spectra defined in **(2)** of this subclause.

(4) In buildings, combinations of the components of the seismic action are given in **4.3.3.5**.

(5) If time-history analyses are required, a set of at least three ground motion records should be used and should conform to the requirements of **3.2.3.1** and **3.2.3.2**.

10.7 Behaviour factor

(1)P Except as provided in **10.10(5)**, the value of the behaviour factor shall be taken as being equal to $q = 1$.

10.8 Properties of the isolation system

(1)P Values of physical and mechanical properties of the isolation system to be used in the analysis shall be the most unfavourable ones to be attained during the lifetime of the structure. They shall reflect, where relevant, the influence of:

– rate of loading;

– magnitude of the simultaneous vertical load;

– magnitude of simultaneous horizontal load in the transverse direction;

– temperature;

– change of properties over projected service life.

(2) Accelerations and inertia forces induced by the earthquake should be evaluated taking into account the maximum value of the stiffness and the minimum value of the damping and friction coefficients.

(3) Displacements should be evaluated taking into account the minimum value of stiffness and damping and friction coefficients.

(4) In buildings of importance classes I or II, mean values of physical and mechanical properties may be used, provided that extreme (maximum or minimum) values do not differ by more than 15% from the mean values.

10.9 Structural analysis

10.9.1 General

(1)P The dynamic response of the structural system shall be analysed in terms of accelerations, inertia forces and displacements.

(2)P In buildings, torsional effects, including the effects of the accidental eccentricity defined in **4.3.2**, shall be taken into account.

(3) Modelling of the isolation system should reflect with a sufficient accuracy the spatial distribution of the isolator units, so that the translation in both horizontal directions, the corresponding overturning effects and the rotation about the vertical axis are adequately accounted for. It should reflect adequately the characteristics of the different types of units used in the isolation system.

10.9.2 Equivalent linear analysis

(1) Subject to the conditions in **(5)** of this subclause, the isolation system may be modelled with equivalent linear visco-elastic behaviour, if it consists of devices such as laminated elastomeric bearings, or with bilinear hysteretic behaviour if the system consists of elasto-plastic types of devices.

(2) If an equivalent linear model is used, the effective stiffness of each isolator unit (i.e. the secant value of the stiffness at the total design displacement d_{db}) should be used, while respecting **10.8(1)**P. The effective stiffness K_{eff} of the isolation system is the sum of the effective stiffnesses of the isolator units.

(3) If an equivalent linear model is used, the energy dissipation of the isolation system should be expressed in terms of an equivalent viscous damping, as the "effective damping" (ξ_{eff}). The energy dissipation in bearings should be expressed from the measured energy dissipated in cycles with frequency in the range of the natural frequencies of the modes considered. For higher modes outside this range, the modal damping ratio of the complete structure should be that of a fixed base superstructure.

(4) When the effective stiffness or the effective damping of certain isolator units depend on the design displacement d_{dc}, an iterative procedure should be applied, until the difference between assumed and calculated values of d_{dc} does not exceed 5% of the assumed value.

(5) The behaviour of the isolation system may be considered as being equivalent to linear if all the following conditions are met:

a) the effective stiffness of the isolation system, as defined in **(2)** of this subclause, is at least 50% of the effective stiffness at a displacement of $0{,}2d_{\mathrm{dc}}$;

b) the effective damping ratio of the isolation system, as defined in **(3)** of this subclause, does not exceed 30%;

c) the force-displacement characteristics of the isolation system does not vary by more than 10% due to the rate of loading or due to the vertical loads;

d) the increase of the restoring force in the isolation system for displacements between $0{,}5d_{dc}$ and d_{dc} is at least 2,5% of the total gravity load above the isolation system.

(6) If the behaviour of the isolation system is considered as equivalent linear and the seismic action is defined through the elastic spectrum as per **10.6(2)**, a damping correction should be performed in accordance with **3.2.2.2(3)**.

10.9.3 Simplified linear analysis

(1) The simplified linear analysis method considers two horizontal dynamic translations and superimposes static torsional effects. It assumes that the superstructure is a rigid solid translating above the isolation system, subject to the conditions of **(2)** and **(3)** of this subclause. Then the effective period of translation is:

$$T_{\mathrm{eff}} = 2\pi \sqrt{\frac{M}{K_{\mathrm{eff}}}} \tag{10.1}$$

where

M is the mass of the superstructure;

K_{eff} is the effective horizontal stiffness of the isolation system as defined in **10.9.2(2)**.

(2) The torsional movement about the vertical axis may be neglected in the evaluation of the effective horizontal stiffness and in the simplified linear analysis if, in each of the two principal horizontal directions, the total eccentricity (including the accidental eccentricity) between the stiffness centre of the isolation system and the vertical projection of the centre of mass of the superstructure does not exceed 7,5% of the length of the superstructure transverse to the horizontal direction considered. This is a condition for the application of the simplified linear analysis method.

(3) The simplified method may be applied to isolation systems with equivalent linear damped behaviour, if they also conform to all of the following conditions:

a) the distance from the site to the nearest potentially active fault with a magnitude $M_s \geq 6{,}5$ is greater than 15 km;

b) the largest dimension of the superstructure in plan is not greater than 50 m;

c) the substructure is sufficiently rigid to minimise the effects of differential displacements of the ground;

d) all devices are located above elements of the substructure which support the vertical loads;

e) the effective period T_{eff} satisfies the following condition:

$$3T_{\mathrm{f}} \leq T_{\mathrm{eff}} \leq 3\ s \tag{10.2}$$

where T_{f} is the fundamental period of the superstructure with a fixed base (estimated through a simplified expression).

(4) In buildings, in addition to **(3)** of this subclause, all of the following conditions should be satisfied for the simplified method to be applied to isolation systems with equivalent linear damped behaviour:

a) the lateral-load resisting system of the superstructure should be regularly and symmetrically arranged along the two main axes of the structure in plan;

b) the rocking rotation at the base of the substructure should be negligible;

c) the ratio between the vertical and the horizontal stiffness of the isolation system should satisfy the following expression:

$$\frac{K_{\mathrm{v}}}{K_{\mathrm{eff}}} \geq 150 \tag{10.3}$$

d) the fundamental period in the vertical direction, T_{V}, should be not longer than 0,1 s, where:

$$T_{\mathrm{V}} = 2\pi \sqrt{\frac{M}{K_{\mathrm{V}}}} \tag{10.4}$$

(5) The displacement of the stiffness centre due to the seismic action should be calculated in each horizontal direction, from the following expression:

$$d_{\mathrm{dc}} = \frac{M\ S_{\mathrm{e}}(T_{\mathrm{eff}}, \xi_{\mathrm{eff}})}{K_{\mathrm{eff,min}}} \tag{10.5}$$

where $S_{\mathrm{e}}(T_{\mathrm{eff}}, \xi_{\mathrm{eff}})$ is the spectral acceleration defined in **3.2.2.2**, taking into account the appropriate value of effective damping ξ_{eff} in accordance with **10.9.2(3)**.

(6) The horizontal forces applied at each level of the superstructure should be calculated, in each horizontal direction through the following expression:

$$f_{\mathrm{j}} = m_{\mathrm{j}}\ S_{\mathrm{e}}(T_{\mathrm{eff}}, \xi_{\mathrm{eff}}) \tag{10.6}$$

where m_{j} is the mass at level j

(7) The system of forces considered in **(6)** induces torsional effects due to the combined natural and accidental eccentricities.

(8) If the condition in **(2)** of this subclause for neglecting torsional movement about the vertical axis is satisfied, the torsional effects in the individual isolator units may be accounted for by amplifying in each direction the action effects defined in **(5)** and **(6)** with a factor δ_{i} given (for the action in the x direction) by:

$$\delta_{\mathrm{xi}} = 1 + \frac{e_{\mathrm{tot,y}}}{r_{\mathrm{y}}^{2}} y_{\mathrm{i}} \tag{10.7}$$

where

y is the horizontal direction transverse to the direction x under consideration;

(x_i, y_i) are the co-ordinates of the isolator unit i relative to the effective stiffness centre;

$e_{tot,y}$ is the total eccentricity in the y direction;

r_y is the torsional radius of the isolation system, as given by the following expression:

$$r_y^{\ 2} = \sum \left(x_i^{\ 2} K_{yi} + y_i^{\ 2} K_{xi} \right) / \sum K_{xi} \tag{10.8}$$

K_{xi} and K_{yi} being the effective stiffness of a given unit i in the x and y directions, respectively.

(9) Torsional effects in the superstructure should be estimated in accordance with **4.3.3.2.4**.

10.9.4 Modal simplified linear analysis

(1) If the behaviour of the devices may be considered as equivalent linear but all the conditions of **10.9.3(2)**, **(3)** and – if applicable - **(4)** are not met, a modal analysis may be performed in accordance with **4.3.3.3**.

(2) If conditions **10.9.3(3)** and - if applicable - **(4)** are met, a simplified analysis may be used considering the horizontal displacements and the torsional movement about the vertical axis and assuming that the substructures and the superstructures behave rigidly. In that case, the total eccentricity (including the accidental eccentricity as per **4.3.2(1)**P) of the mass of the superstructure should be taken into account in the analysis. Displacements at every point of the structure should then be calculated combining the translational and rotational displacements. This applies notably for the evaluation of the effective stiffness of each isolator unit. The inertial forces and moments should be taken into account for the verification of the isolator units and of the substructures and the superstructures.

10.9.5 Time-history analysis

(1)P If an isolation system may not be represented by an equivalent linear model (i.e. if the conditions in **10.9.2(5)** are not met), the seismic response shall be evaluated by means of a time-history analysis, using a constitutive law of the devices which can adequately reproduce the behaviour of the system in the range of deformations and velocities anticipated in the seismic design situation.

10.9.6 Non structural elements

(1)P In buildings, non-structural elements shall be analysed in accordance with **4.3.5**, with due consideration of the dynamic effects of the isolation (see **4.3.5.1(2)** and **(3)**).

10.10 Safety verifications at Ultimate Limit State

(1)P The substructure shall be verified under the inertia forces directly applied to it and the forces and moments transmitted to it by the isolation system.

(2)P The Ultimate Limit State of the substructure and the superstructure shall be checked using the values of γ_M defined in the relevant sections of this Eurocode.

(3)P In buildings, safety verifications regarding equilibrium and resistance in the substructure and in the superstructure shall be performed in accordance with **4.4**. Capacity design and global or local ductility conditions do not need to be satisfied.

(4) In buildings, the structural elements of the substructure and the superstructure may be designed as non-dissipative. For concrete, steel or steel-concrete composite buildings Ductility Class L may be adopted and **5.3**, **6.1.2(2)P**, **(3)** and **(4)** or **7.1.2(2)P** and **(3)**, respectively, applied.

(5) In buildings, the resistance condition of the structural elements of the superstructure may be satisfied taking into account seismic action effects divided by a behaviour factor not greater than 1,5.

(6)P Taking into account possible buckling failure of the devices and using nationally determined γ_M values, the resistance of the isolation system shall be evaluated taking into account the γ_x factor defined in **10.3(2)**P.

(7) According to the type of device considered, the resistance of the isolator units should be evaluated at the Ultimate Limit State in terms of either of the following:

a) forces, taking into account the maximum possible vertical and horizontal forces in the seismic design situation, including overturning effects;

b) total relative horizontal displacement between lower and upper faces of the unit. The total horizontal displacement should include the distortion due to the design seismic action and the effects of shrinkage, creep, temperature and post tensioning (if the superstructure is prestressed).

ANNEX A (Informative)
ELASTIC DISPLACEMENT RESPONSE SPECTRUM

A.1 For structures of long vibration period, the seismic action may be represented in the form of a displacement response spectrum, $S_{De}(T)$, as shown in Figure A.1.

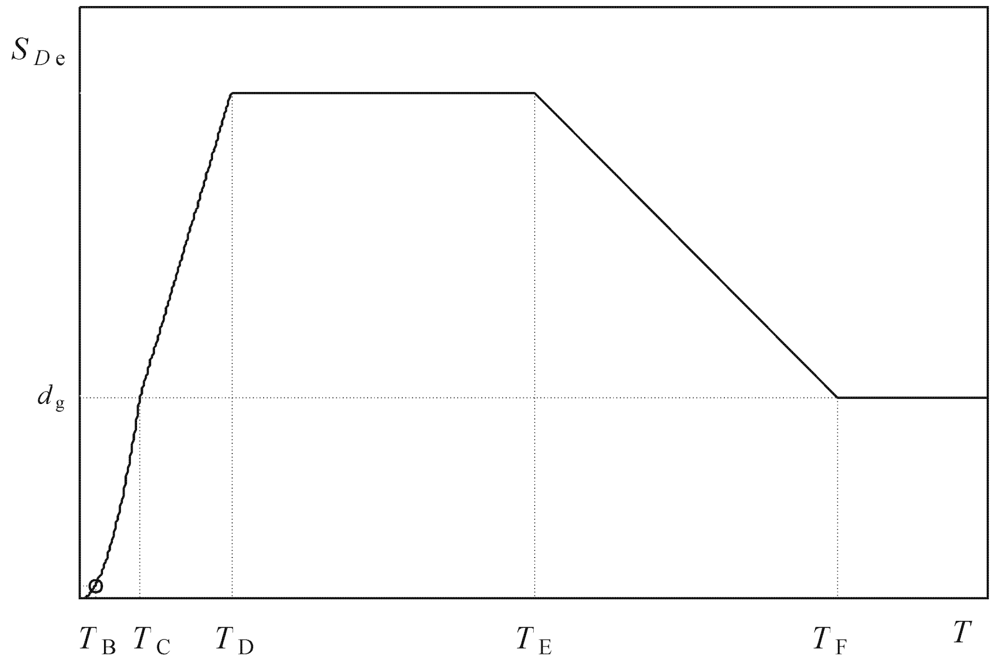


Figure A.1: Elastic displacement response spectrum.

A.2 Up to the control period T_E, the spectral ordinates are obtained from expressions (3.1)-(3.4) converting $S_e(T)$ to $S_{De}(T)$ through expression (3.7). For vibration periods beyond T_E, the ordinates of the elastic displacement response spectrum are obtained from expressions (A.1) and (A.2).

$$T_E \leq T \leq T_F : \; S_{De}(T) = 0{,}025 a_g \cdot S \cdot T_C \cdot T_D \left[2{,}5\eta + \left(\frac{T - T_E}{T_F - T_E} \right)(1 - 2{,}5\eta) \right] \tag{A.1}$$

$$T \geq T_F : \; S_{De}(T) = d_g \tag{A.2}$$

where S, T_C, T_D are given in Tables 3.2 and 3.3, η is given by expression (3.6) and d_g is given by expression (3.12). The control periods T_E and T_F are presented in Table A.1.

Table A.1: Additional control periods for Type 1 displacement spectrum.

Ground type	T_E (s)	T_F (s)
A	4,5	10,0
B	5,0	10,0
C	6,0	10,0
D	6,0	10,0
E	6,0	10,0

ANNEX B (Informative)
DETERMINATION OF THE TARGET DISPLACEMENT FOR NONLINEAR STATIC (PUSHOVER) ANALYSIS

B.1 General

The target displacement is determined from the elastic response spectrum (see **3.2.2.2**). The capacity curve, which represents the relation between base shear force and control node displacement, is determined in accordance with **4.3.3.4.2.3**.

The following relation between normalized lateral forces $\overline{F}_i$ and normalized displacements Φ_i is assumed:

$$\overline{F}_i = m_i \Phi_i \tag{B.1}$$

where m_i is the mass in the i-th storey.

Displacements are normalized in such a way that $\Phi_n = 1$, where n is the control node (usually, n denotes the roof level). Consequently, $\overline{F}_n = m_n$.

B.2 Transformation to an equivalent Single Degree of Freedom (SDOF) system

The mass of an equivalent SDOF system m^* is determined as:

$$m^* = \sum m_i \Phi_i = \sum \overline{F}_i \tag{B.2}$$

and the transformation factor is given by:

$$\Gamma = \frac{m^*}{\sum m_i \Phi_i^2} = \frac{\sum \overline{F}_i}{\sum \left(\frac{\overline{F}_i^2}{m_i} \right)} \tag{B.3}$$

The force F^* and displacement d^* of the equivalent SDOF system are computed as:

$$F^* = \frac{F_b}{\Gamma} \tag{B.4}$$

$$d^* = \frac{d_n}{\Gamma} \tag{B.5}$$

where F_b and d_n are, respectively, the base shear force and the control node displacement of the Multi Degree of Freedom (MDOF) system.

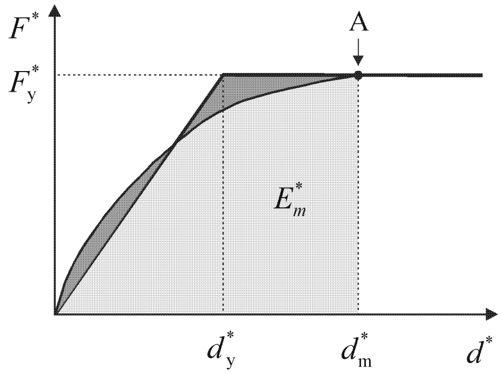
B.3 Determination of the idealized elasto-perfectly plastic force – displacement relationship

The yield force F_y^*, which represents also the ultimate strength of the idealized system, is equal to the base shear force at the formation of the plastic mechanism. The initial stiffness of the idealized system is determined in such a way that the areas under the actual and the idealized force – deformation curves are equal (see Figure B.1).

Based on this assumption, the yield displacement of the idealised SDOF system d_y^* is given by:

$$d_{\mathrm{y}}^{*} = 2\left(d_{\mathrm{m}}^{*} - \frac{E_{\mathrm{m}}^{*}}{F_{\mathrm{y}}^{*}}\right) \tag{B.6}$$

where E_m^* is the actual deformation energy up to the formation of the plastic mechanism.



Key

A plastic mechanism

Figure B.1: Determination of the idealized elasto - perfectly plastic force – displacement relationship.

B.4 Determination of the period of the idealized equivalent SDOF system

The period T^* of the idealized equivalent SDOF system is determined by:

$$T^{*} = 2\pi\sqrt{\frac{m^{*}d_{\mathrm{y}}^{*}}{F_{\mathrm{y}}^{*}}} \tag{B.7}$$

B.5 Determination of the target displacement for the equivalent SDOF system

The target displacement of the structure with period T^* and unlimited elastic behaviour is given by:

$$d_{\mathrm{et}}^{*} = S_{\mathrm{e}}(T^{*})\left[\frac{T^{*}}{2\pi}\right]^{2} \tag{B.8}$$

where $S_e(T^*)$ is the elastic acceleration response spectrum at the period T^*.

For the determination of the target displacement d_t^* for structures in the short-period range and for structures in the medium and long-period ranges different expressions should be used as indicated below. The corner period between the short- and medium-period range is T_C (see Figure 3.1 and Tables 3.2 and 3.3).

a) $T^* < T_C$ (short period range)

If $F_y^* / m^* \geq S_e(T^*)$, the response is elastic and thus

$$d_t^* = d_{et}^* \tag{B.9}$$

If $F_y^* / m^* < S_e(T^*)$, the response is nonlinear and

$$d_t^* = \frac{d_{et}^*}{q_u}\left(1 + (q_u - 1)\frac{T_C}{T^*}\right) \geq d_{et}^* \tag{B.10}$$

where q_u is the ratio between the acceleration in the structure with unlimited elastic behaviour $S_e(T^*)$ and in the structure with limited strength F_y^* / m^*.

$$q_u = \frac{S_e(T^*)m^*}{F_y^*} \tag{B.11}$$

b) $T^* \geq T_C$ (medium and long period range)

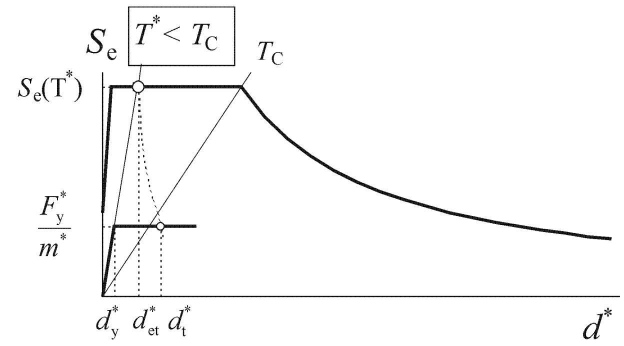
$$d_t^* = d_{et}^* \tag{B.12}$$

d_t* need not exceed 3 d_{et}*.

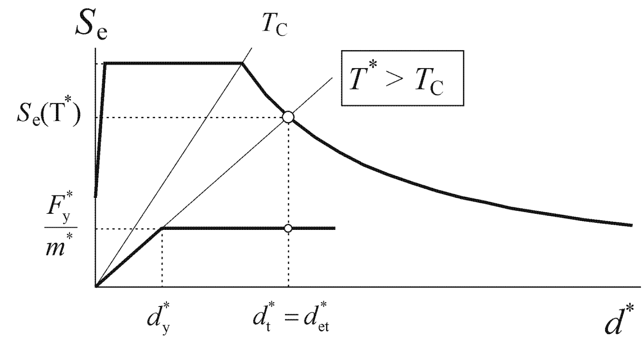
The relation between different quantities can be visualized in Figures B.2 a) and b). The figures are plotted in acceleration - displacement format. Period T^* is represented by the radial line from the origin of the coordinate system to the point at the elastic response spectrum defined by coordinates $d^* = S_e(T^*)(T^*/2\pi)^2$ and $S_e(T^*)$.

Iterative procedure (optional)

If the target displacement d_t^* determined in the 4th step is much different from the displacement d_m^* (Figure B.1) used for the determination of the idealized elasto-perfectly plastic force – displacement relationship in the 2nd step, an iterative procedure may be applied, in which steps 2 to 4 are repeated by using in the 2nd step d_t^* (and the corresponding F_y^*) instead of d_m^*.



a) Short period range



b) Medium and long period range

Figure B.2: Determination of the target displacement for the equivalent SDOF system

B.6 Determination of the target displacement for the MDOF system

The target displacement of the MDOF system is given by:

$$d_t = \Gamma d_t^* \tag{B.13}$$

The target displacement corresponds to the control node.

ANNEX C (Normative)
DESIGN OF THE SLAB OF STEEL-CONCRETE COMPOSITE BEAMS AT BEAM-COLUMN JOINTS IN MOMENT RESISTING FRAMES

C.1 General

(1) This annex refers to the design of the slab and of its connection to the steel frame in moment resisting frames in which beams are composite T-beams comprising a steel section with a slab.

(2) The annex has been developed and validated experimentally in the context of composite moment frames with rigid connections and plastic hinges forming in the beams. The expressions in this annex have not been validated for cases with partial strength connections in which deformations are more localised in the joints.

(3) Plastic hinges at beam ends in a composite moment frame shall be ductile. According to this annex two requirements shall be fulfilled to ensure that a high ductility in bending is obtained:

- early buckling of the steel part shall be avoided;
- early crushing of the concrete of the slab shall be avoided.

(4) The first condition imposes an upper limit on the cross-sectional area A_s of the longitudinal reinforcement in the effective width of the slab. The second condition imposes a lower limit on the cross-sectional area A_T of the transverse reinforcement in front of the column.

C.2 Rules for prevention of premature buckling of the steel section

(1) Paragraph **7.6.1(4)** applies.

C.3 Rules for prevention of premature crushing of concrete

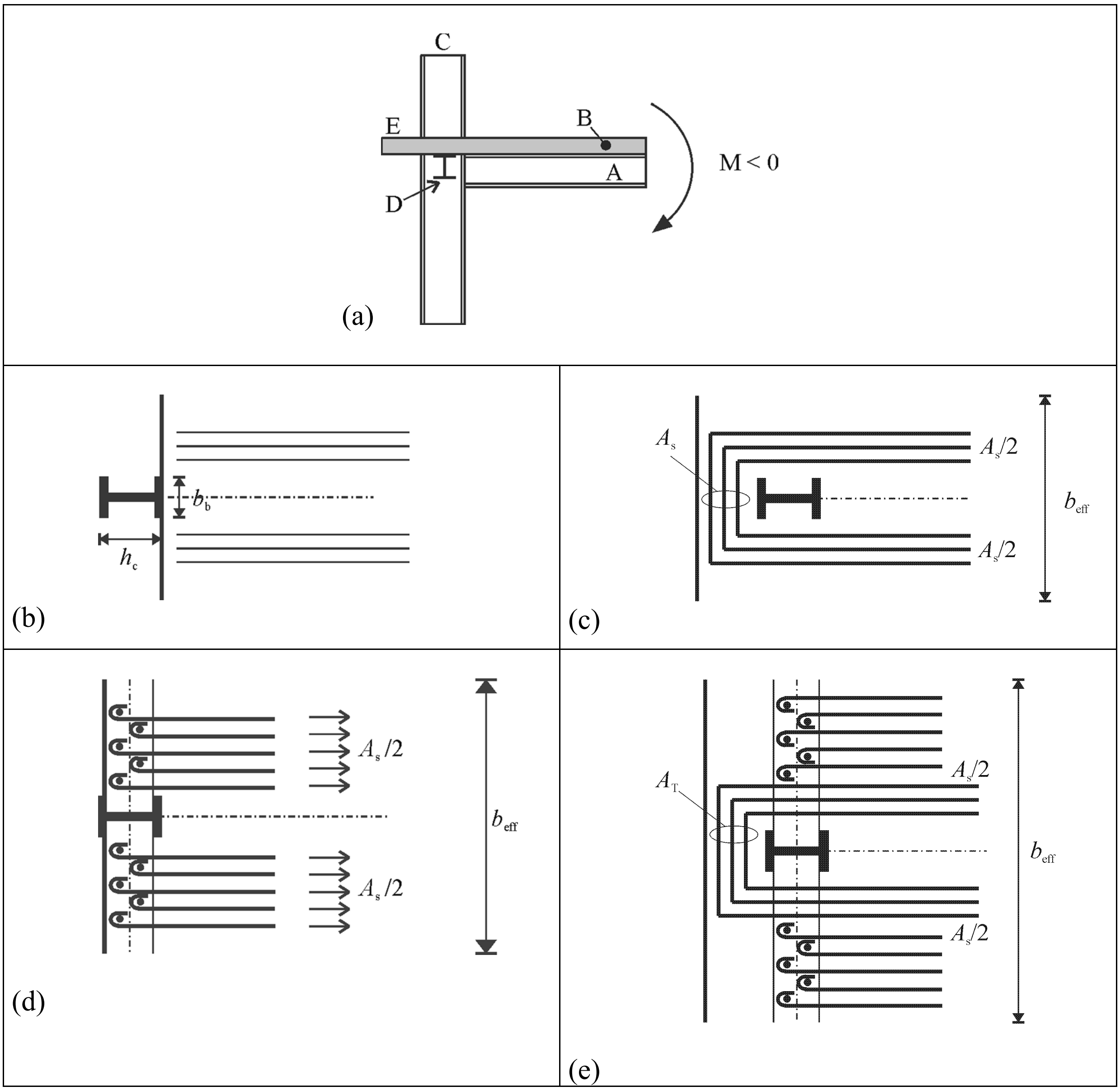
C.3.1 Exterior column - Bending of the column in direction perpendicular to façade; applied beam bending moment negative: $M < 0$

C.3.1.1 No façade steel beam; no concrete cantilever edge strip (Figure C.1(b)).

(1) When there is no façade steel beam and no concrete cantilever edge strip, the moment capacity of the joint should be taken as the plastic moment resistance of the steel beam alone.

C.3.1.2 No façade steel beam; concrete cantilever edge strip present (Figure C.1(c)).

(1) When there is a concrete cantilever edge strip but no façade steel beam, EN 1994-1-1:2004 applies for the calculation of the moment capacity of the joint.



Key:

(a) elevation

(b) no concrete cantilever edge strip – no façade steel beam – see **C.3.1.1**.

(c) concrete cantilever edge strip – no façade steel beam – see **C.3.1.2**.

(d) no concrete cantilever edge strip – façade steel beam – see **C.3.1.3**.

(e) concrete cantilever edge strip – façade steel beam – see **C.3.1.4**.

A main beam;

B slab;

C exterior column;

D façade steel beam;

E concrete cantilever edge strip

Figure C.1: Configurations of exterior composite beam-to-column joints under negative bending moment in a direction perpendicular to façade

C.3.1.3 Façade steel beam present; slab extending up to column outside face; no concrete cantilever edge strip (Figure C.1(d)).

(1) When there is a façade steel beam but no concrete cantilever edge strip, the moment capacity of the joint may include the contribution of the slab reinforcements provided that the requirements in **(2)** to **(7)** of this subclause are satisfied.

(2) Reinforcing bars of the slab should be effectively anchored to the shear connectors of the façade steel beam.

(3) The façade steel beam should be fixed to the column.

(4)P The cross-sectional area of reinforcing steel A_s shall be such that yielding of the reinforcing steel takes place before failure of the connectors and of the façade beams.

(5)P The cross-sectional area of reinforcing steel A_s and the connectors shall be placed over a width equal to the effective width defined in **7.6.3** and Table 7.5 II.

(6) The connectors should be such that:

$$n \cdot P_{Rd} \geq 1{,}1\ F_{Rds} \tag{C.1}$$

where

n is the number of connectors in the effective width;

P_{Rd} is the design resistance of one connector;

F_{Rds} is the design resistance of the re-bars present in the effective width: $F_{Rds} = A_s \cdot f_{yd}$

f_{yd} is the design yield strength of the slab reinforcement.

(7) The façade steel beam should be verified in bending, shear and torsion under the horizontal force F_{Rds} applied at the connectors.

C.3.1.4 Façade steel beam and concrete cantilever edge strip present (Figure C.1(e)).

(1) When there is both a façade steel beam and a concrete cantilever edge strip, the moment capacity of the joint may include the contribution of: (a) the force transferred through the façade steel beam as described in **C.3.1.3** (see **(2)** of this subclause) and (b) the force transferred through the mechanism described in EN 1994-1-1:2004 (see **(3)** of this subclause).

(2) The part of the capacity which is due to the cross-sectional area of reinforcing bars anchored to the transverse façade steel beam, may be calculated in accordance with **C.3.1.3**, provided that the requirements in **(2)** to **(7)** of **C.3.1.3** are satisfied.

(3) The part of the capacity which is due to the cross-sectional area of reinforcing bars anchored within the concrete cantilever edge strip may be calculated in accordance with **C.3.1.2**.

C.3.2 Exterior column - Bending of the column in direction perpendicular to façade; applied beam bending moment positive: $M > 0$

C.3.2.1 No façade steel beam; slab extending up to the column inside face (Figure C.2(b-c)).

(1) When the concrete slab is limited to the interior face of the column, the moment capacity of the joint may be calculated on the basis of the transfer of forces by direct compression (bearing) of the concrete on the column flange. This capacity may be calculated from the compressive force computed in accordance with **(2)** of this subclause, provided that the confining reinforcement in the slab satisfies **(4)** of this subclause.

(2) The maximum value of the force transmitted to the slab may be taken as:

$$F_{\mathrm{Rd1}} = b_{\mathrm{b}}\, d_{\mathrm{eff}} f_{\mathrm{cd}} \tag{C.2}$$

where

d_{eff} is the overall depth of the slab in case of solid slabs or the thickness of the slab above the ribs of the profiled sheeting for composite slabs;

b_{b} is the bearing width of the concrete of the slab on the column (see Figure 7.7).

(3) Confinement of the concrete next to the column flange is necessary. The cross-sectional area of confining reinforcement should satisfy the following expression:

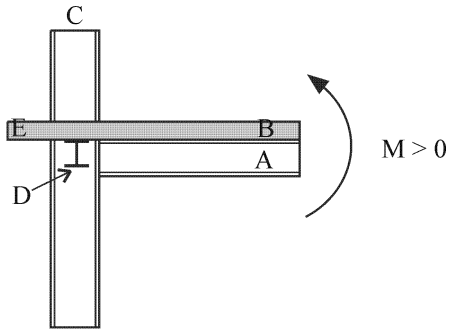
$$A_{\mathrm{T}} \geq 0{,}25 d_{\mathrm{eff}} b_{\mathrm{b}} \frac{0{,}15 l - b_{\mathrm{b}}}{0{,}15 l} \frac{f_{\mathrm{cd}}}{f_{\mathrm{yd,T}}} \tag{C.3}$$

where

$f_{\mathrm{yd,T}}$ is the design yield strength of the transverse reinforcement in the slab.

The cross-sectional area A_{T} of this reinforcement should be uniformly distributed over a length of the beam equal to b_b. The distance of the first reinforcing bar to the column flange should not exceed 30 mm.

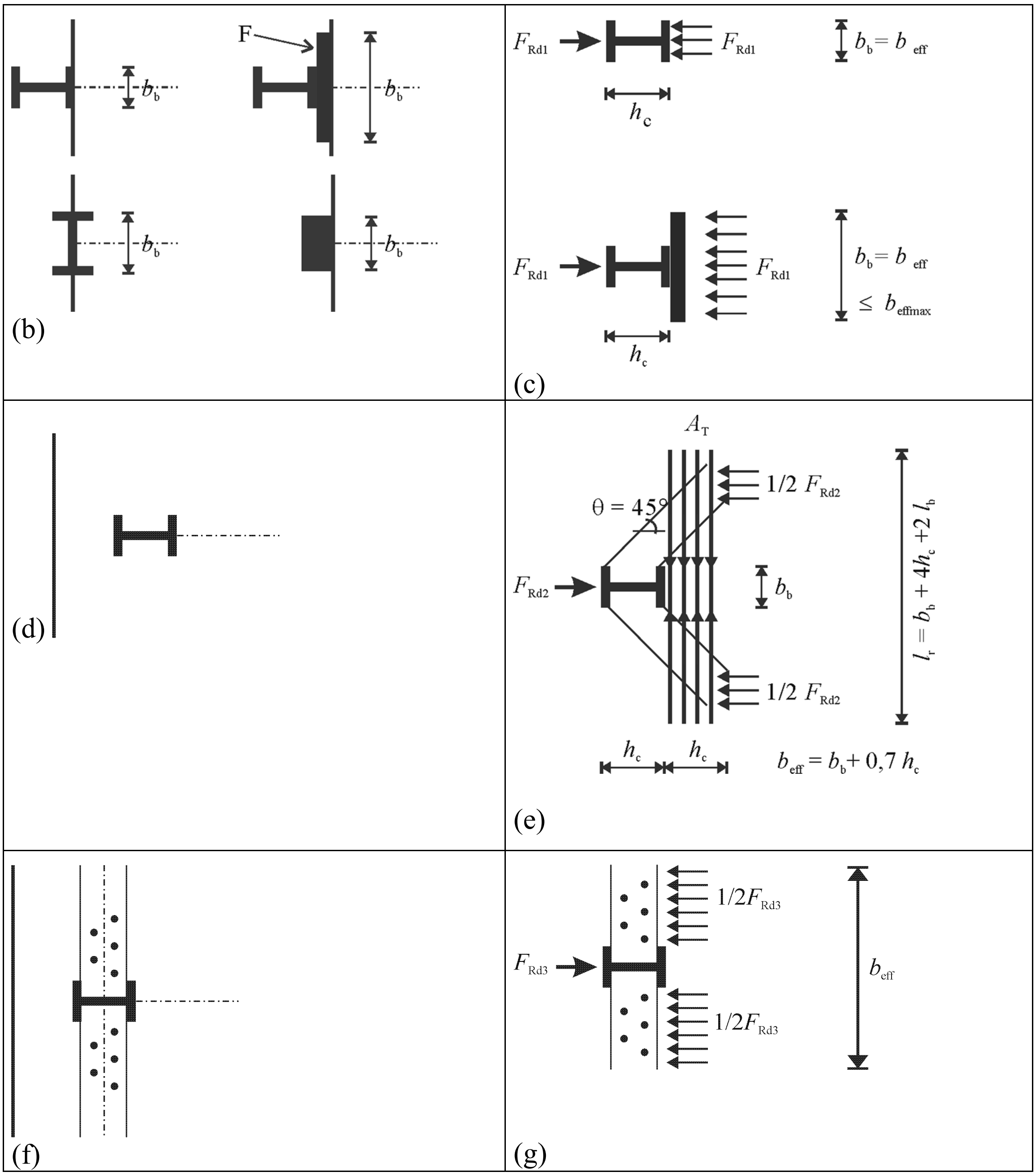
(4) The cross-sectional area A_{T} of steel defined in **(3)** may be partly or totally provided by reinforcing bars placed for other purposes, for instance for the bending resistance of the slab.



(a)

Key:
(a) elevation;
A main beam;
B slab;
C exterior column;
D façade steel beam;
E concrete cantilever edge strip

Figure C.2: Configurations of exterior composite beam-to-column joints under positive bending moments in a direction perpendicular to façade and possible transfer of slab forces



Key:

(b) no concrete cantilever edge strip – no façade steel beam – see **C.3.2.1**;

(c) mechanism 1;

(d) slab extending up to the column outside face or beyond as a concrete cantilever edge strip – no façade steel beam – see **C.3.2.2**;

(e) mechanism 2;

(f) slab extending up to the column outside face or beyond as a concrete cantilever edge strip – façade steel beam present – see **C.3.2.3**;

(g) mechanism 3.

F additional device fixed to the column for bearing.

Figure C.2 (continuation): Configurations of exterior composite beam-to-column joints under positive bending moment in direction perpendicular to façade and possible transfer of slab forces.

C.3.2.2 No façade steel beam; slab extending up to column outside face or beyond as a concrete cantilever edge strip (Figure C.2(c-d-e))

(1) When no façade steel beam is present, the moment capacity of the joint may be calculated from the compressive force developed by the combination of the following two mechanisms:

mechanism 1: direct compression on the column. The design value of the force that is transferred by means of this mechanism should not exceed the value given by the following expression

$$F_{Rd1} = b_b\, d_{eff} f_{cd} \tag{C.4}$$

mechanism 2: compressed concrete struts inclined to the column sides. If the angle of inclination is equal to 45°, the design value of the force that is transferred by means of this mechanism should not exceed the value given by the following expression:

$$F_{Rd2} = 0{,}7 h_c\, d_{eff} f_{cd} \tag{C.5}$$

where

h_c is the depth of the column steel section.

(2) The tension-tie total steel cross-sectional area A_T should satisfy the following expression (see Figure C.2.(e)):

$$A_T \geq \frac{F_{Rd2}}{f_{yd,T}} \tag{C.6}$$

(3) The steel area A_T should be distributed over a length of beam equal to h_c and be fully anchored. The required length of reinforcing bars is $L = b_b + 4\, h_c + 2\, l_b$, where l_b is the anchorage length of these bars in accordance with EN 1992-1-1:2004.

(4) The moment capacity of the joint may be calculated from the design value of the maximum compression force that can be transmitted:

$$F_{Rd1} + F_{Rd2} = b_{eff}\, d_{eff}\, f_{cd} \tag{C.7}$$

b_{eff} is the effective width of the slab at the joint as deduced from **7.6.3** and in Table 7.5II. In this case $b_{eff} = 0{,}7\, h_c + b_b$.

C.3.2.3 Façade steel beam present; slab extending up to column outside face or beyond as a concrete cantilever edge strip (Figure C.2(c-e-f-g)).

(1) When a façade steel beam is present, a third mechanism of force transfer F_{Rd3} is activated in compression involving the façade steel beam.

$$F_{Rd3} = n \cdot P_{Rd} \tag{C.8}$$

where

n is the number of connectors within the effective width computed from **7.6.3** and Table 7.5II;

P_{Rd} is the design resistance of one connector.

(2) **C.3.2.2** applies

(3) The design value of the maximum compression force that can be transmitted is $b_{eff}\ d_{eff}\ f_{cd}$. It is transmitted if the following expression is satisfied:

$$F_{Rd1} + F_{Rd2} + F_{Rd3} > b_{eff}\, d_{eff} f_{cd}. \tag{C.9}$$

The "full" composite plastic moment resistance is achieved by choosing the number n of connectors so as to achieve an adequate force F_{Rd3}. The maximum effective width corresponds to b_{eff} defined in **7.6.3** and Table 7.5 II. In this case, $b_{eff} = 0{,}15\ l$.

C.3.3 Interior column

C.3.3.1 No transverse beam present (Figure C.3(b-c)).

(1) When no transverse beam is present, the moment capacity of the joint may be calculated from the compressive force developed by the combination of the following two mechanisms:

mechanism 1: direct compression on the column. The design value of the force that is transferred by means of this mechanism should not exceed the value given by the following expression:

$$F_{Rd1} = b_b\, d_{eff} f_{cd}. \tag{C.10}$$

mechanism 2: compressed concrete struts inclined at 45° to the column sides. The design value of the force that is transferred by means of this mechanism should not exceed the value given by the following expression:

$$F_{Rd2} = 0{,}7\ h_c\, d_{eff} f_{cd}. \tag{C.11}$$

(2) The tension-tie cross-sectional area A_T required for the development of mechanism 2 should satisfy the following expression:

$$A_T \geq \frac{F_{Rd2}}{f_{yd,T}} \tag{C.12}$$

(3) The same cross-sectional area A_T should be placed on each side of the column to provide for the reversal of bending moments.

(4) The design value of the compressive force developed by the combination of the two mechanisms is

$$F_{Rd1} + F_{Rd2} = (0{,}7\ h_c + b_b)\ d_{eff} f_{cd} \tag{C.13}$$

(5) The total action effect which is developed in the slab due to the bending moments on opposite sides of the column and needs to be transferred to the column

through the combination of mechanisms 1 and 2 is the sum of the tension force F_{st} in the reinforcing bars parallel to the beam at the side of the column where the moment is negative and of the compression force F_{sc} in the concrete at the side of the column where the moment is positive:

$$F_{st} + F_{sc} = A_s f_{yd} + b_{eff} d_{eff} f_{cd} \tag{C.14}$$

where

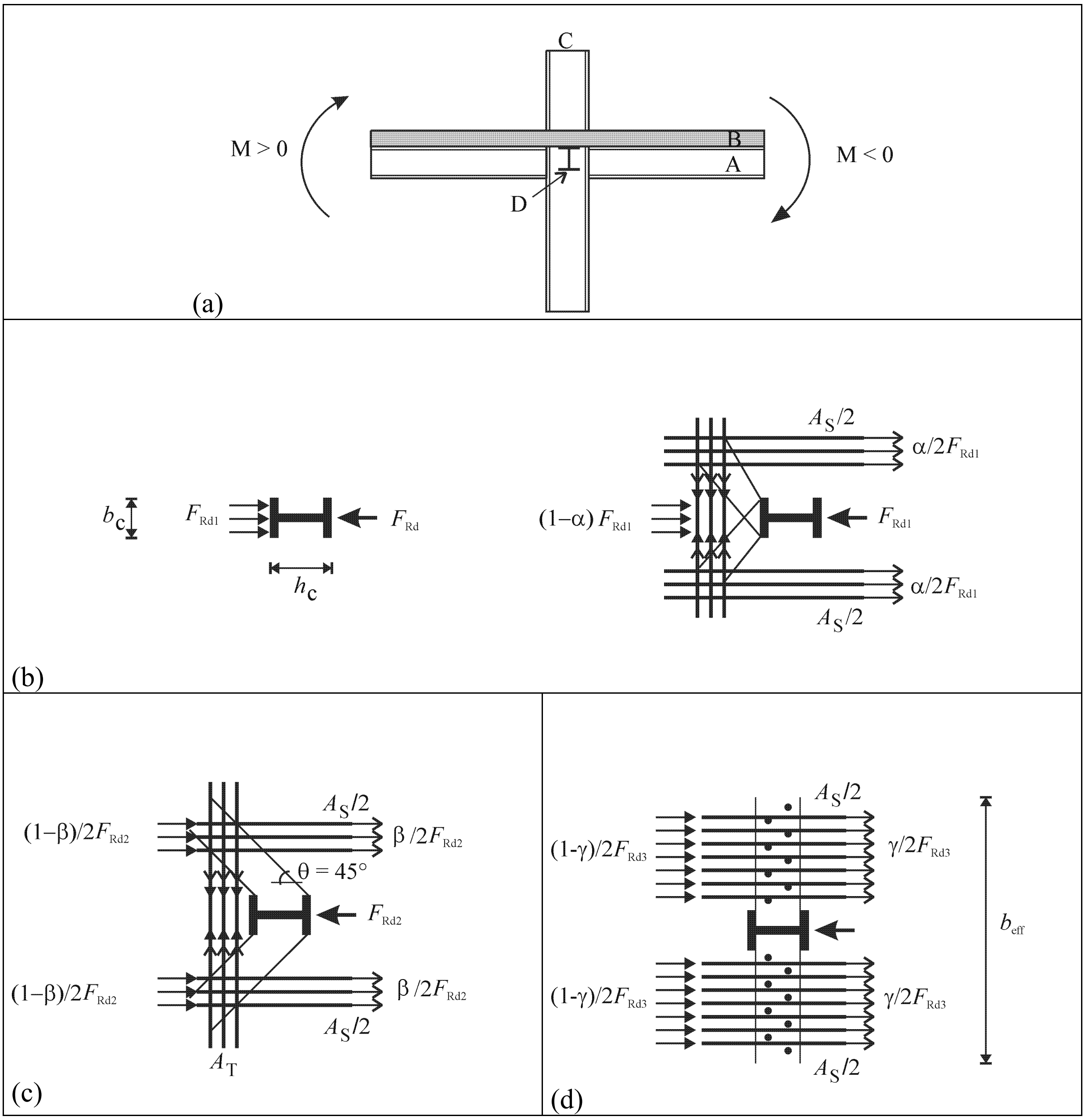
A_s is the cross-sectional area of bars within the effective width in negative bending b_{eff} specified in **7.6.3** and Table 7.5 II; and

b_{eff} is the effective width in positive bending as specified in **7.6.3** and Table 7.5 II. In this case, $b_{eff} = 0,15\ l$.

(6) For the design to achieve yielding in the bottom flange of the steel section without crushing of the slab concrete, the following condition should be fulfilled

$$1,2\ (F_{sc} + F_{st}) \leq F_{Rd1} + F_{Rd2} \tag{C.15}$$

If the above condition is not fulfilled, the capability of the joint to transfer forces from the slab to the column should be increased, either by the presence of a transverse beam (see **C.3.3.2**), or by increasing the direct compression of the concrete on the column by additional devices (see **C.3.2.1**).



Key:
(a) elevation;
(b) mechanism 1;
(c) mechanism 2;
(d) mechanism 3
A main beam;
B slab;
C interior column;
D transverse beam

Figure C.3. Possible transfer of slab forces in an interior composite beam-to-column joint with and without a transverse beam, under a positive bending moment on one side and a negative bending moment on the other side.

C.3.3.2 Transverse beam present (Figure C.3(d)).

(1) When a transverse beam is present, a third mechanism of force transfer F_{Rd3} is activated involving the transverse steel beam.

$$F_{Rd3} = n \cdot P_{Rd} \tag{C.16}$$

where

n is the number of connectors in the effective width computed using **7.6.3** and Table 7.5 II.

P_{Rd} is the design resistance of one connector

(2) **C.3.3.1(2)** applies for the tension-tie.

(3) The design value of the compressive force developed by the combination of the three mechanisms is:

$$F_{Rd1} + F_{Rd2} + F_{Rd3} = (0{,}7\ h_c + b_b)\ d_{eff} f_{cd} + n \cdot P_{Rd} \tag{C.17}$$

where n is the number of connectors in b_{eff} for negative moment or for positive moment as defined in **7.6.3** and Table 7.5 II, whichever is greater out of the two beams framing into the column.

(4) **C.3.3.1(5)** applies for the calculation of the total action effect, $F_{st} + F_{sc}$, developed in the slab due to the bending moments on opposite sides of the column.

(5) For the design to achieve yielding in the bottom flange of the steel section without crushing of the concrete in the slab, the following condition should be fulfilled

$$1{,}2\ (F_{sc} + F_{st}) \leq F_{Rd1} + F_{Rd2} + F_{Rd3} \tag{C.18}$$

EUROPEAN STANDARD

NORME EUROPÉENNE

EUROPÄISCHE NORM

FINAL DRAFT
prEN 1998-2

March 2005

ICS 91.120.25; 93.040 | Will supersede ENV 1998-2:1994

English version

Eurocode 8 - Design of structures for earthquake resistance - Part 2: Bridges

Eurocode 8 - Calcul des structures pour leur résistance aux séismes - Partie 2: Ponts

Eurocode 8 - Auslegung von Bauwerken gegen Erdbeben - Teil 2: Brücken

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

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

Ref. No. prEN 1998-2:2005: E

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Foreword

This European Standard EN 1998-2, Eurocode 8: Design of structures for earthquake resistance: Bridges, has been prepared by Technical Committee CEN/TC250 «Structural Eurocodes», the Secretariat of which is held by BSI. CEN/TC250 is responsible for all Structural Eurocodes.

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 2005, and conflicting national standards shall be withdrawn at latest by MM 2010.

This document supersedes ENV 1998-2:1994.

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

Background of the Eurocode programme

In 1975, the Commission of the European Community decided on an action programme in the field of construction, based on article 95 of the Treaty. The objective of the programme was the elimination of technical obstacles to trade and the 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 1980s.

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

[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).

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
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

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[2] In accordance with 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] In accordance with 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.

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.

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 use of informative annexes, and

– 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 1998-2

The scope of this Part of EN 1998 is defined in **1.1**.

Except where otherwise specified in this Part, the seismic actions are as defined in EN 1998-1:2004, Section **3**.

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

Due to the peculiarities of the bridge seismic resisting systems, in comparison to those of buildings and other structures, all other sections of this Part are in general not directly related to those of EN 1998-1:2004. However several provisions of EN 1998-1:2004 are used by direct reference.

Since the seismic action is mainly resisted by the piers and the latter are usually constructed of reinforced concrete, a greater emphasis has been given to such piers.

Bearings are in many cases important parts of the seismic resisting system of a bridge and are therefore treated accordingly. The same holds for seismic isolation devices.

National annex for EN 1998-2

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 1998-2 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 1998-2:200X through clauses:

Reference	Item
1.1.1(8)	Informative Annexes A, B, C, D, E, F, H and JJ
2.1(3)P	Reference return period T_{NCR} of seismic action for the no-collapse requirement of the bridge (or, equivalently, reference probability of exceedance in 50 years, P_{NCR}).
2.1(4)P	Importance classes for bridges
2.1(6)	Importance factors for bridges
2.2.2(5)	Conditions under which the seismic action may be considered as accidental action, and the requirements of **2.2.2(3)** and **2.2.2 (4)** may be relaxed.
2.3.5.3(1)	Expression for the length of plastic hinges
2.3.6.3(5)	Fractions of design displacements for non-critical structural elements
2.3.7(1)	Cases of low seismicity
2.3.7(1)	Simplified criteria for the design of bridges in cases of low seismicity
3.2.2.3	Definition of active fault
3.3(1)P	Length of continuous deck beyond which the spatial variability of seismic action may have to be taken into account
3.3(6)	Distance beyond which the seismic ground motions may be considered as completely uncorrelated
3.3(6)	factor accounting for the magnitude of ground displacements occurring in opposite direction at adjacent supports
4.1.2(4)P	ψ_{21} values for traffic loads assumed concurrent with the design seismic action

4.1.8(2)	Upper limit for the value in the left-hand-side of expression (4.4) for the seismic behaviour of a bridge to be considered irregular
5.3(4)	Value of ovestrength factor γ_o
5.4(1)	Simplified methods for second order effects in linear analysis
5.6.2(2)P b	Value of additional safety factor γ_{Bd1} on shear resistance
6.2.1.4(1)P	Type of confinement reinforcement
6.5.1(1)P	Simplified verification rules for bridges of limited ductile behaviour in low seismicity cases
6.6.2.3(3)	Allowable extent of damage of elastomeric bearings in bridges where the seismic action is considered as accidental action, but is not resisted entirely by elastomeric bearings
6.6.3.2(1)P	Percentage of the compressive (downward) reaction due to the permanent load that is exceeded by the total vertical reaction on a support due to the design seismic action, for holding-down devices to be required.
6.7.3(7)	Upper value of design seismic displacement to limit damage of the soil or embankment behind abutments rigidly connected to the deck.
7.4.1(1)P	Value of control period T_D for the design spectrum of bridges with seismic isolation
7.6.2(1)P	Value of amplication factor γ_{IS} on design displacement of isolator units
7.6.2(5)	Value of γ_m for elastomeric bearings
7.7.1(2)	Values of factors δ_w and δ_b for the lateral restoring capability of the isolation system
J.1(2)	Values of minimum isolator temperature in the seismic design situation
J.2(1)	Values of λ-factors for commonly used isolators

1 INTRODUCTION

1.1 Scope

1.1.1 Scope of EN 1998-2

(1)P The scope of Eurocode 8 is defined in EN 1998-1:2004, **1.1.1** and the scope of this Standard is defined in **1.1.1**. Additional parts of Eurocode 8 are indicated in EN 1998-1:2004, **1.1.3**.

(2) Within the framework of the scope set forth in EN 1998-1:2004, this part of the Standard contains the particular Performance Requirements, Compliance Criteria and Application Rules applicable to the design of earthquake resistant bridges.

(3) This Part primarily covers the seismic design of bridges in which the horizontal seismic actions are mainly resisted through bending of the piers or at the abutments; i.e. of bridges composed of vertical or nearly vertical pier systems supporting the traffic deck superstructure. It is also applicable to the seismic design of cable-stayed and arched bridges, although its provisions should not be considered as fully covering these cases.

(4) Suspension bridges, timber and masonry bridges, moveable bridges and floating bridges are not included in the scope of this Part.

(5) This Part contains only those provisions that, in addition to other relevant Eurocodes or relevant Parts of EN 1998, should be observed for the design of bridges in seismic regions.

(6) The following topics are dealt with in the text of this Part:

- Basic requirements and Compliance Criteria,
- Seismic Action,
- Analysis,
- Strength Verification,
- Detailing.

This Part also includes a special section on seismic isolation with provisions covering the application of this method of seismic protection to bridges.

(7) Annex G contains rules of the calculation of capacity design effects.

(8) Annex J contains rules regarding the variation of design properties of seismic isolator units and how it may be taken into account in design.

NOTE 1 Informative Annex A provides information for the probabilities of the reference seismic event and recommendations for the selection of the design seismic action during the construction phase.

NOTE 2 Informative Annex B provides information on the relationship between the displacement ductility and the curvature ductility of plastic hinges in concrete piers.

NOTE 3 Informative Annex C provides information for the estimation of the effective stiffness of reinforced concrete ductile members.

NOTE 4 Informative Annex D provides information for modelling and analysis for the spatial variability of earthquake ground motion.

NOTE 5 Informative Annex E gives information on probable material properties and plastic hinge deformation capacities for non-linear analyses.

NOTE 6 Informative Annex F gives information and guidance for the added mass of entrained water in immersed piers.

NOTE 7 Informative Annex H provides guidance and information for static non-linear analysis (pushover).

NOTE 8 Informative Annex JJ provides information on λ-factors for common isolator types.

NOTE 9 Informative Annex K contains tests requirements for validation of design properties of seismic isolator units.

1.1.2 Further parts of EN 1998

See EN 1998-1:2004.

1.2 Normative References

1.2.1 Use

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

1.2.2 General reference standards

EN 1998-1:2004, **1.2.1** applies.

1.2.3 Reference Codes and Standards

EN 1998-1:2004, **1.2.2** applies.

1.2.4 Additional general and other reference standards for bridges

EN 1990: Annex A2	Basis of structural design: Application for bridges
EN 1991-2:2003	Actions on structures: Traffic loads on bridges

EN 1992-2:2005 Design of concrete structures. Part 2 – Bridges

EN 1993-2:2005 Design of steel structures. Part 2 – Bridges

EN 1994-2:2005 Design of composite (steel-concrete) structures. Part 2 – Bridges

EN 1998-1:2004 Design of structures for earthquake resistance. General rules, seismic actions and rules for buildings

EN 1998-5:2004 Design of structures for earthquake resistance. Foundations, retaining structures and geotechnical aspects.

EN 1337-2:2000 Structural bearings – Part 2: Sliding elements

EN 1337-3:1996 Structural bearings – Part 3: Elastomeric bearings

prEN 15129:200X Antiseismic Devices

1.3 Assumptions

(1) In addition to the general assumptions of EN 1990:2002, **1.3** the following assumption applies.

(2)P It is assumed that no change of the structure will take place during the construction phase or during the subsequent life of the structure, unless proper justification and verification is provided. Due to the specific nature of the seismic response this applies even in the case of changes that lead to an increase of the structural resistance of members.

1.4 Distinction between principles and application rules

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

1.5 Definitions

1.5.1 General

(1) For the purposes of this standard the following definitions are applicable.

1.5.2 Terms common to all Eurocodes

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

1.5.3 Further terms used in EN 1998-2

capacity design
design procedure used when designing structures of ductile behaviour to ensure the hierarchy of strengths of the various structural components necessary for leading to the intended configuration of plastic hinges and for avoiding brittle failure modes

ductile members
members able to dissipate energy through the formation of plastic hinges

ductile structure
structure that under strong seismic motions can dissipate significant amounts of input energy through the formation of an intended configuration of plastic hinges or by other mechanisms

limited ductile behaviour
seismic behaviour of bridges, without significant dissipation of energy in plastic hinges under the design seismic action

positive linkage
connection implemented by seismic links

seismic isolation
provision of bridge structures with special isolating devices for the purpose of reducing the seismic response (forces and/or displacements)

spatial variability (of seismic action)
situation in which the ground motion at different supports of the bridge differs and, hence, the seismic action cannot be based on the characterisation of the motion at a single point

seismic behaviour
behaviour of the bridge under the design seismic event which, depending on the characteristics of the global force-displacement relationship of the structure, can be ductile or limited ductile/essentially elastic

seismic links
restrainers through which part or all of the seismic action may be transmitted. Used in combination with bearings, they may be provided with appropriate slack, so as to be activated only in the case when the design seismic displacement is exceeded

minimum overlap length
safety measure in the form of a minimum distance between the inner edge of the supported and the outer edge of the supporting member. The minimum overlap is intended to ensure that the function of the support is maintained under extreme seismic displacements

design seismic displacement
displacement induced by the design seismic actions.

total design displacement in the seismic design situation
displacement used to determine adequate clearances for the protection of critical or major structural members. It includes the design seismic displacement, the displacement due to the long term effect of the permanent and quasi-permanent actions and an appropriate fraction of the displacement due to thermal movements.

1.6 Symbols

1.6.1 General

(1) The symbols indicated in EN 1990:2002, **1.6** apply. For the material-dependent symbols, as well as for symbols not specifically related to earthquakes, the provisions of the relevant Eurocodes apply.

(2) Further symbols, used in connection with the seismic actions, are defined in the text where they occur, for ease of use. However, in addition, the most frequently occurring symbols in EN 1998-2 are listed and defined in the following subsections.

1.6.2 Further symbols used in Sections 2 and 3 of EN 1998-2

d_E design seismic displacement (due only to the design seismic action)

d_{Ee} seismic displacement determined from linear analysis

d_G long term displacement due to the permanent and quasi-permanent actions

d_g design ground displacement in accordance with EN 1998-1:2004, **3.2.2.4**

d_i ground displacement of set B at support *i*

d_{ri} ground displacement at support *i* relative to reference support 0

d_T displacement due to thermal movements

d_u ultimate displacement

d_y yield displacement

A_{ed} design seismic action

F_{Rd} design value of resisting force to the earthquake action

L_g distance beyond which the ground motion may be considered completely uncorrelated

L_i distance of support *i* from reference support 0

$L_{i-1,i}$ distance between consecutive supports *i-1* and *i*

R_i reaction force at the base of pier *i*

S_a site-averaged response spectrum

S_i site-dependent response spectrum

T_{eff} effective period of the isolation system

γ_I importance factor

Δd_i ground displacement of intermediate support *i* relative to adjacent supports

i-1 and *i+1*

μ_d displacement ductility factor

ψ_2 combination factor for the quasi-permanent value of thermal action

1.6.3 Further symbols used in Section 4 of EN 1998-2

d_a average of the displacements in the transverse direction of all pier tops under the transverse seismic action, or under the action of a transverse load of similar distribution

d_i displacement of the *i*-th nodal point

d_m asymptotic value of the spectrum for the *m*-th motion for long periods, expressed in terms of displacements

e $e_a + e_d$

e_a accidental mass eccentricity (= 0,03L, or 0,03B)

e_d additional eccentricity reflecting the dynamic effect of simultaneous translational and torsional vibration (= 0,05L or 0,05B)

e_o theoretical eccentricity

g acceleration of gravity

h depth of the cross-section in the direction of flexure of the plastic hinge

k_m effect of the *m*-th independent motion

r_i required local force reduction factor at ductile member *i*

r_{min} minimum value of r_i

r_{max} maximum value of r_i

A_{Ed} design seismic action

A_{Ex} seismic action in direction x

A_{Ey} seismic action in direction y

A_{Ez} seismic action in direction y

B width of the deck

E probable maximum value of an action effect

E_i response in mode *i*

F horizontal force determined in accordance with the fundamental mode method

G total effective weight of the structure, equal to the weight of the deck plus the weight of the top half of the piers

G_i weight concentrated at the *i*-th nodal point

K stiffness of the system

L total length of the continuous deck

L_s distance from the plastic hinge to the point of zero moment

M total mass

$M_{Ed,i}$ maximum value of design moment in the seismic design situation at the intended location of plastic hinge of ductile member *i*

$M_{Rd,i}$ design flexural resistance of the plastic hinge section of ductile member *i*

M_t equivalent static moment about the vertical axis through the centre of mass of the deck

$Q_{k,1}$ characteristic value of traffic load

R_d design value of resistance

$S_d(T)$ spectral acceleration of the design spectrum

T period of the fundamental mode of vibration for the direction under consideration

X horizontal longitudinal axis of the bridge

Y horizontal transverse axis of the bridge

Z vertical axis

α_s shear span ratio of the pier

Δ_d maximum difference of the displacements in the transverse direction of all pier tops under the transverse seismic action, or under the action of a transverse load of similar distribution

η_k normalized axial force (= $N_{Ed}/(A_c f_{ck})$)

$\theta_{p,d}$ design value of plastic rotation capacity

$\theta_{p,E}$ plastic hinge rotation demand

ξ viscous damping ratio

$\psi_{2,i}$ factor for quasi-permanent value of variable action i

1.6.4 Further symbols used in Section 5 of EN 1998-2

d_{Ed} relative transverse displacement of the ends of the ductile member under consideration

f_{ck} characteristic value of concrete strength

f_{ctd} design value of tensile strength of concrete

f_{sd} reduced stress of reinforcement, for limitation of cracking

f_{sy} design value of yield strength of the joint reinforcement

z_b internal lever arm of the beam end sections

z_c internal lever arm of the plastic hinge section of the column

A_C (V_C, M_C, N_C) capacity design effects

A_c area of the concrete section

A_{Ed} design seismic action (seismic action alone)

A_{Sd} action in the seismic design situation

A_{sx} area of horizontal joint reinforcement

E_d design value of action effect of in the seismic design situation

G_k characteristic value of permanent load

M_o overstrength moment

M_{Ed} design moment in the seismic design situation

M_{Rd} design value of flexural strength of the section

N_{Ed} axial force in the seismic design situation

N_{cG} axial force in the column under the permanent and the quasi-permanent actions in the seismic design situation

N_{jz} vertical axial force in a joint

Q_{1k} characteristic value of the traffic load

Q_2 quasi-permanent value of actions of long duration

P_k characteristic value of prestressing after all losses

R_d design value of the resistance of the section

R_{df} design value of the maximum friction force of sliding bearing

T_{Rc} resultant force of the tensile reinforcement of the column

$V_{E,d}$ design value of shear force

V_{jx} design value of horizontal shear of the joint

V_{jz} design value of vertical shear of the joint

V_{1bC} shear force of the beam adjacent to the tensile face of the column

γ_M material partial factor

γ_o overstrength factor

γ_{of} magnification factor for friction due to ageing effects

γ_{Bd}, γ_{Bd1} additional safety factor against brittle failure modes

ρ_x ratio of horizontal reinforcement

ρ_y reinforcement ratio of closed stirrups in the transverse direction of the joint panel (orthogonal to the plane of action)

ρ_z ratio of vertical reinforcement

ψ_{21} combination factor

1.6.5 Further symbols used in Section 6 of EN 1998-2

a_g design ground acceleration on type A ground (see EN 1998-1:2004, **3.2.2.2**).

b cross-sectional dimension of the concrete core perpendicular to the direction of the confinement under consideration, measured to the centre line of the perimeter hoop

b_{min} smallest dimension of the concrete core

d_{bL} diameter of longitudinal bar

d_{eg} effective displacement due to the spatial variation of the seismic ground displacement

d_{es} effective seismic displacement of the support due to the deformation of the structure

d_g design peak ground displacement as specified by EN 1998-1:2004, **3.2.2.4**

f_t	tensile strength
f_y	yield strength
f_{ys}	yield strength of the longitudinal reinforcement
f_{yt}	yield strength of the tie
l_m	minimum support length securing the safe transmission of the vertical reaction
l_{ov}	minimum overlap length
s	spacing of tie legs on centres
s_L	maximum (longitudinal) spacing
s_T	spacing of between hoop legs or supplementary cross ties on centres
s_t	transverse spacing
v_g	design ground velocity
v_s	shear wave velocity in the soil at small shear strains
A_c	area of the gross concrete section
A_{cc}	cross-sectional area of the confined concrete core of the section
A_{sp}	cross-sectional area of the spiral or hoop bar
A_{sw}	total cross-sectional area of hoops or ties in the one transverse direction of confinement
A_t	cross-sectional area of one tie leg
D_i	inside diameter
D_{sp}	diameter of the spiral or hoop bar
E_d	total earth pressure acting on the abutment under seismic conditions as per EN 1998-5: 2004
F_{Rd}	design resistance
L_h	design length of plastic hinges
L_{eff}	effective length of deck
Q_d	weight of the section of the deck linked to a pier or abutment, or the least of the weights of the two deck sections on either side of an intermediate separation joint
S	soil factor specified in EN 1998-1:2004, **3.2.2.2**
T_C	corner period of elastic spectrum as specified in EN 1998-1:2004, **3.2.2.2**
α_g	design ground acceleration on type A ground
γ_I	importance factor
γ_s	free-field seismic shear deformation of the soil
δ	parameter depending on the ratio f_t/f_y
μ_Φ	required curvature ductility factor
ΣA_s	sum of the cross-sectional areas of the longitudinal bars restrained by the tie

ρ_L ratio of the longitudinal reinforcement

ρ_w transverse reinforcement ratio

ω_{wd} mechanical ratio of confinement reinforcement

1.6.6 Further symbols used in Section 7 and Annexes J, JJ and K of EN 1998-2

a_g design ground acceleration on type A ground

$a_{g,R}$ reference peak ground acceleration on type A ground reference

d design displacement

d_b displacement of isolator

d_{bd} design displacement of isolator corresponding to the design displacement of the isolating system d_{cd}

d_{bi} displacement of isolator i

$d_{bi,a}$ increased design displacement of isolator i

$d_{bi,d}$ design displacement of isolator i

d_{cd} design displacement of the isolating system

d_{cf} design displacement of the isolating system resulting from the fundamental mode method

$d_{d,m}$ displacement of the stiffness centre derived from the analysis

d_{id} displacement of the superstructure at the location of substructure and isolator i

d_m displacement capacity of the isolating system

d_{max} maximum displacement

d_n, d_p minimum negative and positive displacement in test respectively

d_{rm} residual displacement of the isolating system

d_y yield displacement

e_x eccentricity in the longitudinal bridge direction

r radius of gyration of the deck mass about vertical axis through its centre of mass

$sign(\dot{d}_b)$ sign of the velocity vector $\dot{d}_b$

t_e total elastomer thickness

v velocity of motion of a viscous isolator

v_{max} maximum velocity of motion of a viscous isolator

x_i, y_i co-ordinates of pier i in plan

A_b effective cross-sectional area of elastomeric bearing

E_D dissipated energy per cycle at the design displacement of isolating system d_{cd}

E_{Di} dissipated energy per cycle of isolator unit i, at the design displacement of isolating system d_{cd}

E_E design seismic forces

E_{EA} seismic internal forces derived from the analysis

F_{max} max force corresponding to the design displacement

F_n, F_p minimum negative and maximum positive forces of test, respectively, for units with hysteretic or frictional behaviour, or negative and positive forces of test respectively corresponding to d_n and d_p, respectively, for units with viscoelastic behaviour

F_y yield force under monotonic loading

F_0 force at zero displacement under cyclic loading

G_b shear modulus of elastomeric bearing

G_g apparent conventional shear modulus of elastomeric bearing in accordance with EN 1337-3: 1996, **4.3.1.1**

H_i height of pier *i*

K_{bi} effective stiffness of isolator unit *i*

K_e elastic stiffness of bilinear hysteretic isolator under monotonic loading

K_L stiffness of lead core of lead-rubber bearing

K_p post elastic stiffness of bilinear hysteretic isolator

K_{eff} effective stiffness of the isolation system in the principal horizontal direction under consideration, at a displacement equal to the design displacement d_{cd}

$K_{eff,i}$ composite stiffness of isolator units and the corresponding pier *i*

K_{fi} rotation stiffness of foundation of pier *i*

K_R stiffness of rubber of lead-rubber bearing

K_{ri} rotation stiffness of foundation of pier *i*

K_{si} displacement stiffness of shaft of pier *i*

K_{ti} translation stiffness of foundation of pier *i*

K_{xi}, K_{yi} effective composite stiffness of isolator unit and pier *i*

M_d mass of the superstructure

N_{Sd} axial force through the isolator

Q_G permanent axial load of isolator

R_b radius of spherical sliding surface

S soil factor of elastic spectrum in accordance with EN 1998-1:2004, **3.2.2.2**

T_C, T_D corner periods of the elastic spectrum in accordance with **7.4.1(1)**P and EN 1998-1:2004, **3.2.2.2**

T_{eff} effective period of the isolating system

$T_{min,b}$ minimum bearing temperature for seismic design

V_d maximum shear force transferred through the isolation interface

V_f maximum shear force estimated through the fundamental mode method

UBDP Upper bound design properties of isolators

LBDP Lower bound design properties of isolators

α_b exponent of velocity of viscous damper

γ_I importance factor of the bridge

ΔF_{Ed} additional vertical load due to seismic overturning effects

ΔF_m force increase between displacements $d_m/2$ and d_m

μ_d dynamic friction coefficient

ξ equivalent viscous damping ratio

ξ_b contribution of isolators to effective damping

ξ_{eff} effective damping of the isolation system

ψ_{fi} combination factor

2 BASIC REQUIREMENTS AND COMPLIANCE CRITERIA

2.1 Design seismic action

(1)P The design philosophy of this Standard is to achieve with appropriate reliability the non-collapse requirement of **2.2.2** and of EN 1998-1:2004, **2.1(1)**P, for the design seismic action (A_{Ed}).

(2)P Unless otherwise specified in this part, the elastic spectrum of the design seismic action in accordance with EN 1998-1:2004, **3.2.2.2**, **3.2.2.3** and **3.2.2.4** applies. For application of the equivalent linear method of **4.1.6** (using the behaviour factor q) the spectrum shall be the design spectrum in accordance with EN 1998-1:2004, **3.2.2.5**.

(3)P The design seismic action, A_{Ed}, is expressed in terms of: (a) the reference seismic action, A_{Ek}, associated with a reference probability of exceedance, P_{NCR}, in 50 years or a reference return period, T_{NCR}, (see EN 1998-1:2004, **2.1(1)**P and **3.2.1(3)**) and (b) the importance factor γ_I (see EN 1990: 2002 and EN 1998-1:2004, **2.1(2)**P, **2.1(3)**P and **(4)**) to take into account reliability differentiation:

$$A_{Ed} = \gamma_I A_{Ek} \tag{2.1}$$

NOTE 1 The value to be ascribed to the reference return period, T_{NCR}, associated with the reference seismic action for use in a country, may be found in its National Annex. The recommended value is: T_{NCR} = 475 years.

NOTE 2 Informative Annex A gives information on the reference seismic action and on the selection of the design seismic action during the construction phase.

(4)P Bridges shall be classified in importance classes, depending on the consequences of their failure for human life, on their importance for maintaining communications, especially in the immediate post-earthquake period, and on the economic consequences of collapse.

NOTE The definitions of the importance classes for bridges in a country may be found in its National Annex. The recommended classification is in three importance classes, as follows:

In general bridges on motorways and national roads, as well as railroad bridges, are considered to belong to importance class II (average importance).

Importance class III comprises bridges of critical importance for maintaining communications, especially in the immediate post-earthquake period, bridges where failure is associated with a large number of probable fatalities and major bridges where a design life greater than normal is required.

A bridge may be classified to importance class I (less than average importance) when both following conditions are met.

- the bridge is not critical for communications, and
- the adoption of either the reference probability of exceedance, P_{NCR}, in 50 years for the design seismic action, or of the standard bridge design life of 50 years is not economically justified.

Importance classes I, II and III correspond roughly to consequences classes CC1, CC2 and CC3, respectively, defined in EN 1990:2002, **B3.1**.

(5)P The importance classes are characterised by different importance factors γ_I as described in **2.1(3)**P and in EN 1998-1:2004, **2.1(3)**P.

(6) The importance factor γ_I = 1,0 is associated with a seismic action having the reference return period indicated in **2.1(3)**P and in EN 1998-1:2004, **3.2.1(3)**.

NOTE The values to be ascribed to γ_I for use in a country may be found in its National Annex. The values of γ_I may be different for the various seismic zones of the country, depending on the seismic hazard conditions and on public safety considerations (see NOTE to EN 1998-1:2004, **2.1(4)**). The recommended values of γ_I for importance classes I, and III are equal to 0,85, and 1,3, respectively.

2.2 Basic requirements

2.2.1 General

(1)P The design shall aim at fulfilling the following two basic requirements.

2.2.2 No-collapse (ultimate limit state)

(1)P After occurrence of the design seismic action, the bridge shall retain its structural integrity and adequate residual resistance, although at some parts of the bridge considerable damage may occur.

(2) Flexural yielding of specific sections (i.e. the formation of plastic hinges) is allowed to occur in the piers. When no seismic isolation is provided, such flexural yielding is in general necessary in regions of high seismicity, in order to reduce the design seismic action to a level corresponding to a reasonable increase of the additional construction cost, compared to a bridge not designed for earthquake resistance.

(3) The bridge deck should in general be designed to avoid damage, other than locally to secondary components such as expansion joints, continuity slabs (see **2.3.2.2(4)**) or parapets.

(4) When the design seismic action has a substantial probability of exceedance within the design life of the bridge, the design should aim at a damage tolerant structure. Parts of the bridge susceptible to damage by their contribution to energy dissipation under the design seismic action should be designed to enable the bridge to be used by emergency traffic, following the design seismic action, and to be easily repairable.

(5) When the design seismic action has low probability of being exceeded within the design life of the bridge, the seismic action may be considered as accidental action, in accordance with EN 1990:2002, **1.5.3.5** and **4.1.1(2)**. In such a case the requirements of **(3)** and **(4)** may be relaxed.

NOTE The National Annex may specify the conditions under which **(5)** will be applied, as well as the extent of the relevant relaxations of **(3)** and **(4)**. It is recommended that **(3)** and **(4)** are applicable when the reference return period T_{NCR} is approximately equal to 475 years.

2.2.3 Minimisation of damage (serviceability limit state)

(1)P A seismic action with a high probability of occurrence may cause only minor damage to secondary components and to those parts of the bridge intended to contribute to energy dissipation. All other parts of the bridge should remain undamaged.

2.3 Compliance criteria

2.3.1 General

(1)P To conform to the basic requirements set forth in **2.2**, the design shall comply with the criteria outlined in the following Clauses. In general the criteria, while aiming explicitly at satisfying the no-collapse requirement (**2.2.2**), implicitly cover the damage minimisation requirement (**2.2.3**) as well.

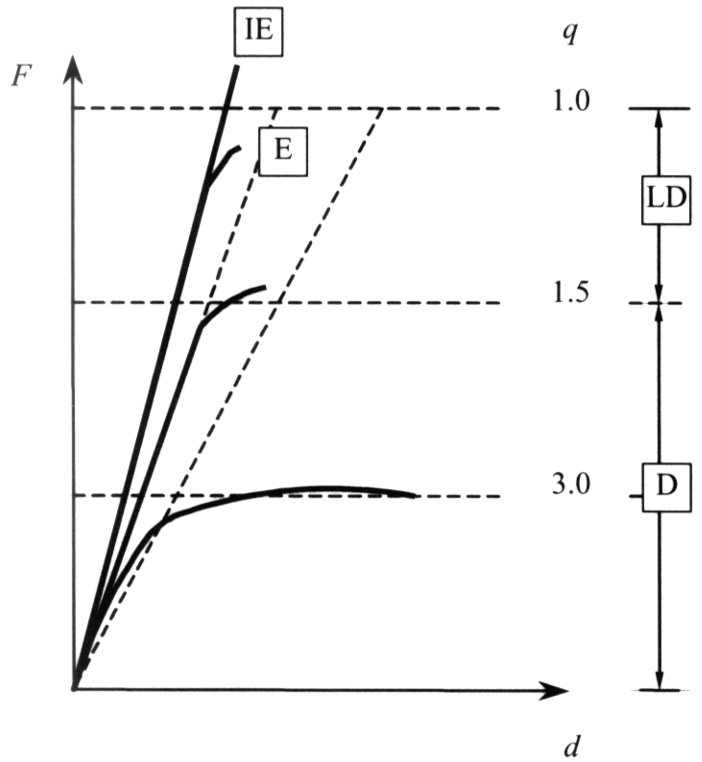
(2) Compliance with the criteria set forth in this standard is deemed to satisfy all basic requirements of **2.2**.

(3)P The compliance criteria depend on the behaviour which is intended for the bridge under the design seismic action. This behaviour may be selected in accordance with **2.3.2**.

2.3.2 Intended seismic behaviour

2.3.2.1 General

(1)P The bridge shall be designed so that its behaviour under the design seismic action is either ductile, or limited ductile/essentially elastic, depending on the seismicity of the site, on whether seismic isolation is adopted for its design, or any other constraints which may prevail. This behaviour (ductile or limited ductile) is characterised by the global force-displacement relationship of the structure, shown schematically in Figure 2.1 (see also Table 4.1).



Key
q – Behaviour factor
IE - Ideal elastic
E - Essentially elastic
LD - Limited ductile
D - Ductile

Figure 2.1: Seismic behaviour

2.3.2.2 Ductile behaviour

(1) In regions of moderate to high seismicity it is usually preferable, both for economic and safety reasons, to design a bridge for ductile behaviour, i.e. to provide it with reliable means to dissipate a significant amount of the input energy under severe earthquakes. This is accomplished by providing for the formation of an intended configuration of flexural plastic hinges or by using isolating devices in accordance with Section **7**. The part of this sub-clause that follows refers to ductile behaviour achieved by flexural plastic hinges.

(2)P Bridges of ductile behaviour shall be designed so that a dependably stable partial or full mechanism can develop in the structure through the formation of flexural plastic hinges. These hinges normally form in the piers and act as the primary energy dissipating components.

(3) As far as is feasible, the location of plastic hinges should be selected at points accessible for inspection and repair.

(4)P The bridge deck shall remain within the elastic range. However, formation of plastic hinges (in bending about the transverse axis) is allowed in flexible ductile concrete slabs providing top slab continuity between adjacent simply-supported precast concrete girder spans.

(5)P Plastic hinges shall not be formed in reinforced concrete sections where the normalised axial force η_k defined in **5.3(4)** exceeds 0,6.

(6)P This standard does not contain rules for provision of ductility in prestressed or post-tensioned members. Consequently such members should be protected from formation of plastic hinges under the design seismic action.

(7) Flexural plastic hinges need not necessarily form in all piers. However the optimum post-elastic seismic behaviour of a bridge is achieved if plastic hinges develop approximately simultaneously in as many piers as possible.

(8) The capability of the structure to form flexural hinges is necessary, in order to ensure energy dissipation and consequently ductile behaviour (see **4.1.6(2)**).

NOTE The deformation of bridges supported exclusively by normal elastomeric bearings is predominantly elastic and does not lead in general to ductile behaviour (see **4.1.6(11)**P).

(9) The global force-displacement relationship should exhibit a significant force plateau at yield and should ensure hysteretic energy dissipation over at least five inelastic deformation cycles (see Figures 2.1, 2.2 and 2.3).

NOTE Elastomeric bearings used over some supports in combination with monolithic support on other piers, may cause the resisting force to increase with increasing displacements, after plastic hinges have formed in the other supporting members. However, the rate of increase of the resisting force should be appreciably reduced after the formation of plastic hinges.

(10) Supporting members (piers or abutments) connected to the deck through sliding or flexible mountings (sliding bearings or flexible elastomeric bearings) should, in general, remain within the elastic range.

2.3.2.3 Limited ductile behaviour

(1) In structures with limited ductile behaviour, a yielding region with significant reduction in secant stiffness need not appear under the design seismic action. In terms of force-displacement characteristics, the formation of a force plateau is not required, while deviation from the ideal elastic behaviour provides some hysteretic energy dissipation. Such behaviour corresponds to a value of the behaviour factor $q \leq 1,5$ and shall be referred to, in this Standard, as "limited ductile".

NOTE Values of q in the range $1 \leq q \leq 1,5$ are mainly attributed to the inherent margin between design and probable strength in the seismic design situation.

(2) For bridges where the seismic response may be dominated by higher mode effects (e.g cable-stayed bridges), or where the detailing of plastic hinges for ductility may not be reliable (e.g. due to a high axial force or a low shear-span ratio), a behaviour factor of $q = 1$ is recommended, corresponding to elastic behaviour.

2.3.3 Resistance verifications

(1)P In bridges designed for ductile behaviour the regions of plastic hinges shall be verified to have adequate flexural strength to resist the design seismic action effects as specified in **5.5**. The shear resistance of the plastic hinges, as well as both the shear and flexural resistances of all other regions, shall be designed to resist the "capacity design effects" specified in **2.3.4** (see also **5.3**).

(2) In bridges designed for limited ductile behaviour, all sections should be verified to have adequate strength to resist the design action seismic effects of **5.5** (see **5.6.2**).

2.3.4 Capacity design

(1)P For bridges of ductile behaviour, capacity design shall be used to ensure that an appropriate hierarchy of resistance exists within the various structural components. This is to ensure that the intended configuration of plastic hinges will form and that brittle failure modes are avoided.

(2)P Fulfilment of **(1)**P shall be achieved by designing all members intended to remain elastic against all brittle modes of failure, using "capacity design effects". Such effects result from equilibrium conditions at the intended plastic mechanism, when all flexural hinges have developed an upper fractile of their flexural resistance (overstrength), as specified in **5.3**.

(3) For bridges of limited ductile behaviour the application of the capacity design procedure is not required.

2.3.5 Provisions for ductility

2.3.5.1 General requirement

(1)P The intended plastic hinges shall be provided with adequate ductility, to ensure the required overall global ductility of the structure.

NOTE The definitions of global and local ductilities, given in **2.3.5.2** and **2.3.5.3**, are intended to provide the theoretical basis of ductile behaviour. In general they are not required for practical verification of ductility, which is effected in accordance with **2.3.5.4**.

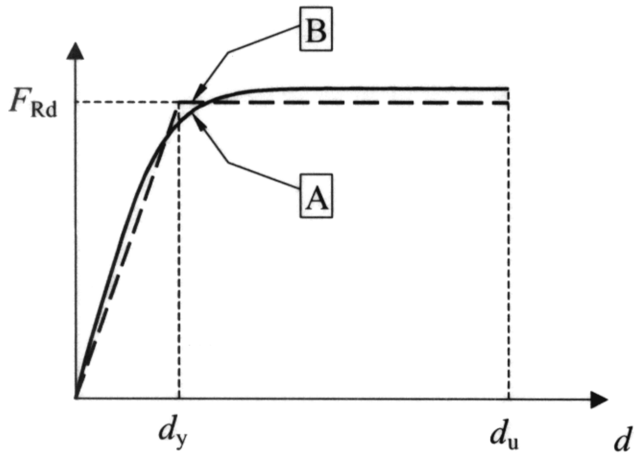
2.3.5.2 Global ductility

(1) Referring to an equivalent one-degree-of-freedom system with an idealised elastic-perfectly plastic force-displacement relationship, as shown in Figure 2.2, the design value of the ductility factor of the structure (available displacement ductility factor) is defined as the ratio of the ultimate limit state displacement (d_u) to the yield displacement (d_y), both measured at the centre of mass: i.e. $\mu_d = d_u / d_y$.

(2) When an equivalent linear analysis is performed, the yield force of the global elastic-perfectly plastic force-displacement is assumed equal to the design value of the resisting force, F_{Rd}. The yield displacement defining the elastic branch is selected so as to best approximate the design force-displacement curve (for monotonic loading).

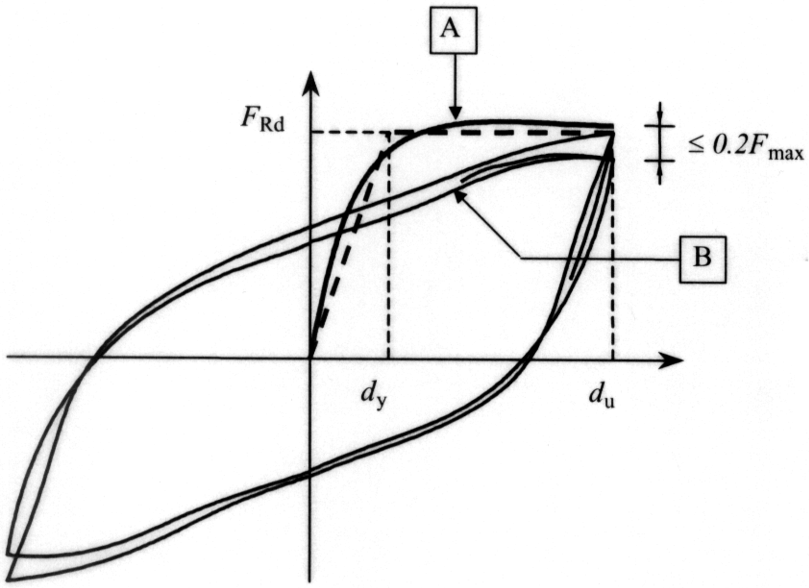
(3) The ultimate displacement d_u is defined as the maximum displacement satisfying the following condition. The structure should be capable of sustaining at least 5 full cycles of deformation to the ultimate displacement:

- without initiation of failure of the confining reinforcement for reinforced concrete sections, or local buckling effects for steel sections; and
- without a drop of the resisting force for steel ductile members or without a drop exceeding 20% of the ultimate resisting force for reinforced concrete ductile members (see Figure 2.3).



Key
A - Design
B - Elastoplastic

Figure 2.2: Global force-displacement diagram (Monotonic loading)



Key
A - Monotonic loading
B -5th cycle

Figure 2.3: Force-displacement cycles (Reinforced concrete)

2.3.5.3 Local ductility at the plastic hinges

(1) The global ductility of the structure depends on the available local ductility at the plastic hinges (see Figure 2.4). This can be expressed in terms of the curvature ductility factor of the cross-section:

$$\mu_{\Phi} = \Phi_{\mathrm{u}}/\Phi_{\mathrm{y}} \tag{2.2}$$

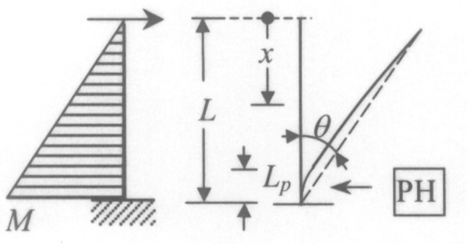
or, in terms of the chord rotation ductility factor at the end where the plastic hinge forms, that depends on the plastic rotation capacity, $\theta_{\mathrm{p,u}} = \theta_{\mathrm{u}}-\theta_{\mathrm{y}}$, of the plastic hinge:

$$\mu_{\theta} = \theta_{\mathrm{u}}/\theta_{\mathrm{y}} = 1 + (\theta_{\mathrm{u}} - \theta_{\mathrm{y}})/\theta_{\mathrm{y}} = 1 + \theta_{\mathrm{p,u}}/\theta_{\mathrm{y}} \tag{2.3}$$

The chord rotation is measured over the length L, between the end section of the plastic hinge and the section of zero moment, as shown in Figure 2.4.

NOTE 1 For concrete members the relationship between θ_{p}, Φ_{u}, Φ_{y}, L and L_{p} is given by equation (E16b) in **E.3.2** of Informative Annex **E**.

NOTE 2 The length of plastic hinges L_{p} for concrete members may be specified in the National Annex, as a function of the geometry and other characteristics of the member. The recommended expression is that given in Annex **E**.



Key
PH - Plastic hinge

Figure 2.4: Chord rotation $\theta = \frac{1}{L}\int_0^L \phi\, x\, dx$

(2) In the above expressions the ultimate deformations should conform to the definitions in **2.3.5.2(3)**.

NOTE The relationship between curvature ductility of a plastic hinge and the global displacement ductility factor for a simple case is given in **Annex B**. That relationship is not intended for ductility verification.

2.3.5.4 Ductility verification

(1)P Conformance to the Specific Rules specified in **Section 6** is deemed to ensure the availability of adequate local and global ductility.

(2)P When non-linear static or dynamic analysis is performed, chord rotation demands shall be checked against available rotation capacities of the plastic hinges (see **4.2.4.4**).

(3) For bridges of limited ductile behaviour the provisions of **6.5** should be applied.

2.3.6 Connections - Control of displacements - Detailing

2.3.6.1 Effective stiffness - Design seismic displacement

(1)P When equivalent linear analysis methods are used, the stiffness of each member shall be chosen corresponding to its secant stiffness under the maximum calculated stresses under the design seismic action. For members containing plastic hinges this corresponds to the secant stiffness at the theoretical yield point (See Figure 2.5).

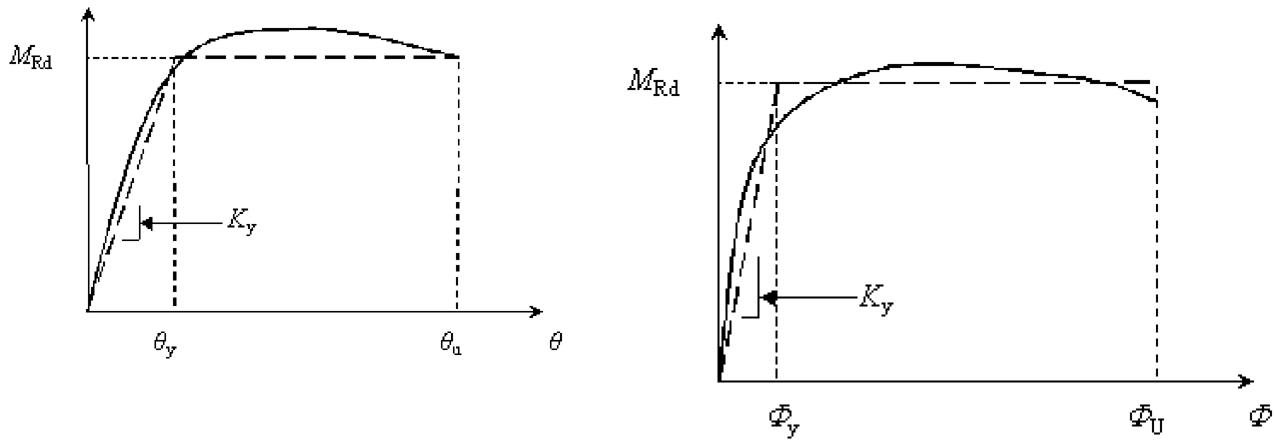


Figure 2.5: Moment - deformation diagrams at plastic hinges

Left: Moment-rotation relationship of plastic hinge for structural steel;
Right: Moment-curvature relationship of cross-section for reinforced concrete.

(2) For reinforced concrete members in bridges designed for ductile behaviour, and unless a more accurate method is used for its estimation, the effective flexural stiffness to be used in linear analysis (static or dynamic) for the design seismic action may be estimated as follows.

– For reinforced concrete piers, a value calculated on the basis of the secant stiffness at the theoretical yield point.

– For prestressed or reinforced concrete decks, the stiffness of the uncracked gross concrete sections.

NOTE **Annex C** gives guidance for the estimation of the effective stiffness of reinforced concrete members.

(3) In bridges designed for limited ductile behaviour, either the rules of **(2)** may be applied or the flexural stiffness of the uncracked gross concrete sections may be used for the entire structure.

(4) For both ductile and limited ductile bridges, the significant reduction of the torsional stiffness of concrete decks, in relation to the torsional stiffness of the uncracked deck, should be accounted for. Unless a more accurate calculation is made, the following fractions of the torsional stiffness of the uncracked gross section may be used:

– for open sections or slabs, the torsional stiffness may be ignored;

– for prestressed box sections, 50% of the uncracked gross section stiffness;

– for reinforced concrete box sections, 30% of the uncracked gross section stiffness.

(5) For both ductile and limited ductile bridges, displacements obtained from an analysis in accordance with **(2)** and **(3)** should be multiplied by the ratio of (a) the flexural stiffness of the member used in the analysis to (b) the value of flexural stiffness that corresponds to the level of stresses resulting from the analysis.

NOTE It is noted that in the case of equivalent linear analysis (see **4.1.6(1)**P) an overestimation of the effective stiffness leads to results which are on the safe side regarding the seismic action effects. In such a case, only the displacements need be corrected after the analysis, on the basis of the flexural stiffness that corresponds to the resulting level of moments. On the other hand, if the effective stiffness initially assumed is significantly lower than that corresponding to the stresses from the analysis, the analysis should be repeated using a better approximation of the effective stiffness.

(6)P If linear seismic analysis based on the design spectrum in accordance with EN 1998-1:2004, **3.2.2.5** is used, the design seismic displacements, d_E, shall be derived from the displacements, d_{Ee}, determined from such an analysis as follows:

$$d_E = \pm\ \eta \mu_d d_{Ee} \tag{2.4}$$

where

η is the damping correction factor specified in EN 1998-1:2004, **3.2.2.2(3)** determined with the ξ values specified for damping in **4.1.3(1)**.

(7) When the displacements d_{Ee} are derived from a linear elastic analysis based on the elastic spectrum in accordance with EN 1998-1:2004, **3.2.2.2** (q = 1.0), the design displacement, d_E, shall be taken as equal to d_{Ee}.

(8)P The displacement ductility factor shall be assumed as follows:

- when the fundamental period T in the considered horizontal direction is $T \geq T_o = 1{,}25T_C$, where T_C is the corner period defined in accordance with EN 1998-1:2004, **3.2.2.2,** then

$$\mu_d = q \tag{2.5}$$

- if $T < T_o$, then

$$\mu_d = (q-1)\frac{T_o}{T} + 1 \leq 5q - 4 \tag{2.6}$$

where q is the value of the behaviour factor assumed in the analysis that results in the value of d_{Ee}.

NOTE Expression (2.6) provides a smooth transition between the “equal displacement” rule that is applicable for $T \geq T_o$, and the short period range (not typical to bridges) where the assumption of a low q-value is expedient. For very small periods (T < 0,033 sec), q = 1 should be assumed (see also **4.1.6(9)**), giving: μ_d = 1.

(9)P When non-linear time-history analysis is used, the deformation characteristics of the yielding members shall approximate their actual post-elastic behaviour, both as far as the loading and unloading branches of the hysteresis loops are concerned, as well as potential degradation effects (see **4.2.4.4**).

2.3.6.2 Connections

(1)P Connections between supporting and supported members shall be designed in order to ensure structural integrity and avoid unseating under extreme seismic displacements.

(2) Unless otherwise specified in this Part, bearings, links and holding-down devices used for securing structural integrity, should be designed using capacity design effects (see **5.3**, **6.6.2.1**, **6.6.3.1** and **6.6.3.2**).

(3) In new bridges appropriate overlap lengths should be provided between supporting and supported members at moveable connections, in order to avoid unseating (see **6.6.4**).

(4) In retrofitting existing bridges as an alternative to the provision of overlap length, positive linkage between supporting and supported members may be used (see **6.6.1(3)**P and **6.6.3.1(1)**).

2.3.6.3 Control of displacements - Detailing

(1)P In addition to ensuring the required overall ductility, structural and non-structural detailing of the bridge and its components shall be provided to accommodate the displacements in the seismic design situation.

(2)P Clearances shall be provided for protection of critical or major structural members. Such clearances shall accommodate the total design value of the displacement in the seismic design situation, d_{Ed}, determined as follows:

$$d_{Ed} = d_E + d_G + \psi_2 d_T \tag{2.7}$$

where the following displacements shall be combined with the most unfavourable sign:

d_E is the design seismic displacement in accordance with **2.3.6.1**;

d_G is the long term displacement due to the permanent and quasi-permanent actions (e.g. post-tensioning, shrinkage and creep for concrete decks);

d_T is the displacement due to thermal movements;

ψ_2 is the combination factor for the quasi-permanent value of thermal action, in accordance with EN 1990:2002, Tables A2.1, A2.2 or A2.3.

Second order effects shall be taken into account in the calculation of the total design value of the displacement in the seismic design situation, when such effects are significant.

(3) The relative design seismic displacement, d_E, between two independent sections of a bridge may be estimated as the square root of the sum of squares of the values of the design seismic displacement calculated for each section in accordance with **2.3.6.1**.

(4)P Large shock forces, caused by unpredictable impact between major structural members, shall be prevented by means of ductile/resilient members or special energy absorbing devices (buffers). Such members shall possess a slack at least equal to the total design value of the displacement in the seismic design situation, d_{Ed}.

(5) The detailing of non-critical structural components (e.g. deck movement joints and abutment back-walls), expected to be damaged due to the design seismic action, should cater for a predictable mode of damage, and provide for the possibility of permanent repair. Clearances should accommodate appropriate fractions of the design seismic displacement and of the thermal movement, p_E and p_T, respectively, after allowing for any long term creep and shrinkage effects, so that damage under frequent earthquakes is avoided. The appropriate values of such fractions may be chosen, based on a judgement of the cost-effectiveness of the measures taken to prevent damage.

NOTE 1 The value ascribed to p_E and p_T for use in a country in the absence of an explicit optimisation may be found in its National Annex. The recommended values are as follows: p_E = 0,4 (for the design seismic displacement); p_T = 0,5 (for the thermal movement).

NOTE 2 At joints of railway bridges, transverse differential displacement may have to be either avoided or limited to values appropriate for preventing derailment.

2.3.7 Simplified criteria

(1) In cases of low seismicity, simplified design criteria may be established.

NOTE 1: The selection of the categories of bridge, ground type and seismic zone in a country for which the provisions of low seismicity apply may be found in its National Annex. It is recommended that cases of low seismicity (and by consequence those of moderate to high seismicity) should be defined as recommended in the Note in EN 1998-1:2004, **3.2.1(4)**.

NOTE 2: Classification of bridges and simplified criteria for the seismic design pertaining to individual bridge classes in cases of low seismicity may be established by the National Annex. It is recommended that these simplified criteria are based on a limited ductile/essentially elastic seismic behaviour of the bridge, for which no special ductility requirements are necessary.

2.4 Conceptual design

(1) Consideration of the implications of the seismic action at the conceptual stage of the design of bridges is important, even in cases of low to moderate seismicity.

(2) In cases of low seismicity the type of intended seismic behaviour of the bridge (see **2.3.2**) should be decided. If a limited ductile (or essentially elastic) behaviour is selected, simplified criteria, in accordance with **2.3.7** may be applied.

(3) In cases of moderate or high seismicity, the selection of ductile behaviour is generally expedient. Its implementation, either by providing for the formation of a dependable plastic mechanism or by using seismic isolation and energy dissipation devices, should be decided. When a ductile behaviour is selected, **(4)** to **(8)** should be observed.

(4) The number of supporting members (piers and abutments) that will be used to resist the seismic forces in the longitudinal and transverse directions should be decided. In general bridges with continuous deck behave better under seismic conditions than those with many movement joints. The optimum post-elastic seismic behaviour is achieved if plastic hinges develop approximately simultaneously in as many piers as possible. However, the number of the piers that resist the seismic action may have to be less than the total number of piers, by using sliding or flexible mountings between the

deck and some piers in the longitudinal direction, to reduce the stresses arising from imposed deck deformations due to thermal actions, shrinkage and other non-seismic actions.

(5) A balance should be maintained between the strength and the flexibility requirements of the horizontal supports. High flexibility reduces the magnitude of lateral forces induced by the design seismic action but increases the movement at the joints and moveable bearings and may lead to high second order effects.

(6) In the case of bridges with a continuous deck and with transverse stiffness of the abutments and of the adjacent piers which is very high compared to that of the other piers (as may occur in steep-sided valleys), it may be preferable to use transversally sliding or elastomeric bearings over the short piers or the abutments to avoid unfavourable distribution of the transverse seismic action among the piers and the abutments such as that exemplified in Figure 2.6.

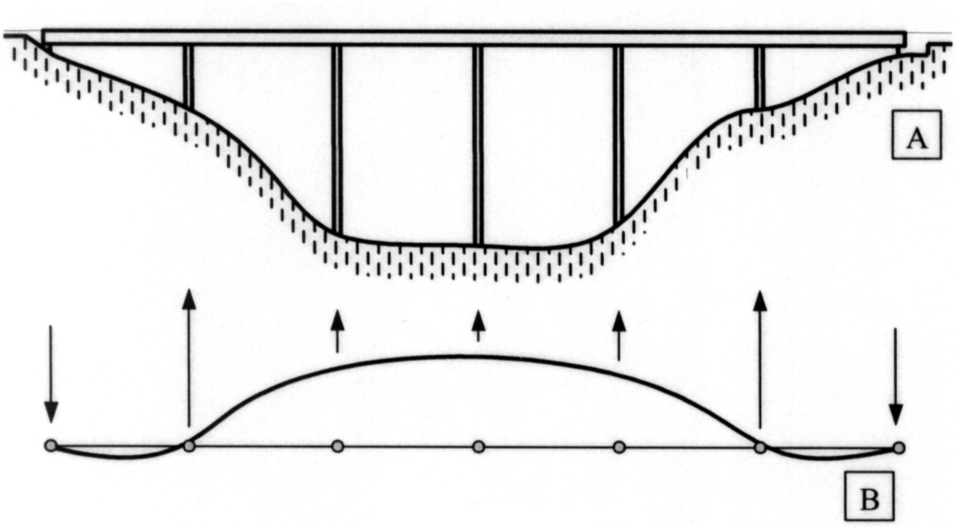
(7) The locations selected for energy dissipation should be chosen so as to ensure accessibility for inspection and repair. Such locations should be clearly indicated in the appropriate design documents.

(8) The location of areas of potential or expected seismic damage other than those in **(7)** should be identified and the difficulty of repairs should be minimised.

(9) In exceptionally long bridges, or in bridges crossing non-homogeneous soil formations, the number and location of intermediate movement joints should be decided.

(10) In bridges crossing potentially active tectonic faults, the probable discontinuity of the ground displacement should be estimated and accommodated either by adequate flexibility of the structure or by provision of suitable movement joints.

(11) The liquefaction potential of the foundation soil should be investigated in accordance with the relevant provisions of EN 1998-5:2004.



Key
A – Elevation
B - Plan

Figure 2.6: Unfavourable distribution of transverse seismic action

3 SEISMIC ACTION

3.1 Definition of the seismic action

3.1.1 General

(1)P The complexity of the model selected to describe the seismic action shall be appropriate to the relevant earthquake motion to be described and the importance of the structure and commensurate with the sophistication of the model used in the analysis of the bridge.

(2)P In this Section only the shaking transmitted by the ground to the structure is considered in the quantification of the seismic action. However, earthquakes can induce permanent displacements in soils arising from ground failure or fault rupture. These displacements may result in imposed deformations with severe consequences for bridges. This type of hazard shall be evaluated through specific studies. Its consequences shall be minimised by appropriate measures, such as selecting a suitable structural system. Tsunami effects are not treated in this Standard.

3.1.2 Application of the components of the motion

1(P) In general only the three translational components of the seismic action need to be taken into account for the design of bridges. When the response spectrum method is applied, the bridge may be analysed separately for the translational components of the seismic action in the longitudinal, transverse and vertical directions. In this case the seismic action is represented by three one-component actions, one for each direction, quantified in accordance with **3.2**. The action effects shall be combined in accordance with **4.2.1.4**.

2(P) When non-linear time-history analysis is performed, the bridge shall be analysed under the simultaneous action of the different components.

(3) The seismic action is applied at the interface between the structure and the ground. If springs are used to represent the soil stiffness either in connection with spread footings or with deep foundations, such as piles, shafts (caissons), etc. (see EN 1998-5:2004), the motion is applied at the soil end of the springs.

3.2 Quantification of the components

3.2.1 General

(1)P Each component of the earthquake motion shall be quantified in terms of a response spectrum, or a time-history representation (mutually consistent) as set out in EN 1998-1:2004, Section **3**, which also provides the basic definitions.

3.2.2 Site dependent elastic response spectrum

3.2.2.1 Horizontal component

(1)P The horizontal component shall be in accordance with EN 1998-1:2004, **3.2.2.2**, depending on the ground type at the foundation of the supports of the bridge. When more than one ground types correspond to these supports, then **3.3** applies.

3.2.2.2 Vertical component

(1)P When the vertical component of the seismic motion needs to be taken into account (see **4.1.7**), the site-dependent response spectrum of this component shall be taken in accordance with EN 1998-1:2004, **3.2.2.3**.

3.2.2.3 Near source effects

(1)P Site-specific spectra considering near source effects shall be used, when the site is located within 10 km horizontally of a known active seismotectonic fault that may produce an event of Moment Magnitude higher than 6,5.

NOTE Unless the National Annex defines otherwise, it is recommended that a seismotectonic fault be considered to be active for the purposes of this requirement when there is an average historic slip rate of at least 1 mm/year and topographic evidence of seismic activity within the Holocene times (past 11000 years).

3.2.3 Time-history representation

(1)P When a non-linear time-history analysis is carried-out, at least three pairs of horizontal ground motion time-history components shall be used. The pairs should be selected from recorded events with magnitudes, source distances, and mechanisms consistent with those that define the design seismic action.

(2) When the required number of pairs of appropriate recorded ground motions is not available, appropriate modified recordings or simulated accelerograms may replace the missing recorded motions.

(3)P Consistency to the relevant 5% damped elastic response spectrum of the design seismic action shall be established by scaling the amplitude of motions as follows.

a. For each earthquake consisting of a pair of horizontal motions, the SRSS spectrum shall be established by taking the square root of the sum of squares of the 5%-damped spectra of each component.
b. The spectrum of the ensemble of earthquakes shall be formed by taking the average value of the SRSS spectra of the individual earthquakes of the previous step.
c. The ensemble spectrum shall be scaled so that it is not lower than 1,3 times the 5%-damped elastic response spectrum of the design seismic action, in the period range between $0{,}2T_1$ and $1{,}5T_1$, where T_1 is the natural period of the fundamental mode of the structure in the case of a ductile bridge, or the effective period (T_{eff}) of the isolation system in the case of a bridge with seismic isolation (see **7.2**).

d. The scaling factor derived from the previous step shall be applied to all individual seismic motion components.

(4) When the SRSS spectrum of the components of a recorded accelerogram gives accelerations the ratio of which to the corresponding values of the elastic response spectrum of the design seismic action shows large variation in the period range in **(3)**Pc, modification of the recorded accelerogram may be carried out, so that the SRSS spectrum of the modified components is in closer agreement with the elastic response spectrum of the design seismic action.

(5)P The components of each pair of time-histories shall be applied simultaneously.

(6) When three component ground motion time-history recordings are used for non-linear time-history analysis, scaling of the horizontal pairs of components may be carried out in accordance with **(3)**P, independently from the scaling of the vertical components. The latter shall be effected so that the average of the relevant spectra of the ensemble is not lower by more than 10% of the 5% damped elastic response spectrum of the vertical design seismic action in the period range between $0,2T_v$ and $1,5T_v$, where T_v is the period of the lowest mode where the response to the vertical component prevails over the response to the horizontal components (e.g, in terms of participating mass).

(7) The use of pairs of horizontal ground motion recordings in combination with vertical recordings of different seismic motions, consistent with the requirements of **(1)**P above, is also allowed. The independent scaling of the pairs of horizontal recordings and of the vertical recordings shall be carried out as in **(6)**.

(8) Modification of the recorded vertical component in **(6)** and **(7)** is permitted using the method specified in **(4)**.

3.2.4 Site dependent design spectrum for linear analysis

(1)P Both ductile and limited ductile structures shall be designed by performing linear analysis using a reduced response spectrum, called design spectrum, as specified by EN 1998-1:2004, **3.2.2.5**.

3.3 Spatial variability of the seismic action

(1)P For bridge sections with a continuous deck the spatial variability shall be considered when one or both of the following two conditions hold.

- Soil properties along the bridge vary to the extent that more than one ground types (as specified in EN 1998-1:2004, **3.1.1**) correspond to the supports of the bridge deck.
- Soil properties along the bridge are approximately uniform, but the length of the continuous deck exceeds an appropriate limiting length, L_{lim}.

NOTE The value ascribed to L_{lim} for use in a country may be found in its National Annex. The recommended value is: $L_{lim} = L_g/1.5$ where the length L_g is defined in **(6)** below.

(2)P The model describing spatial variability should account, even if only in a simplified way, for the propagative character of the seismic waves, as well as for the progressive loss of correlation between motions at different locations due to the random non homogeneity of the soil, involving complex reflections and refractions of the waves. The model should also account, even if only in a simplified way, for the further increase in loss of correlation due to differences in the mechanical properties of the soil along the bridge, which also modify the frequency content from one support to the other.

NOTE Models of the spatial variability of the earthquake motions and appropriate methods of analysis are presented in informative Annex **D**.

(3) Unless a more accurate evaluation is made, the simplified method specified in the paragraphs **(4)** to **(7)** may be used.

(4) The inertia response should be accounted for by one of the methods specified in Section **4** (see **4.2.1**, **4.2.3** and **4.2.4**) using a single input seismic action for the entire structure (e.g. a single response spectrum or corresponding accelerogram sets), corresponding to the most severe ground type underneath the bridge supports.

(5) The spatial variation of the seismic action may be estimated by pseudo-static effects of appropriate displacement sets, imposed at the foundation of the supports of the bridge deck. These sets should reflect probable configurations of the spatial variability of the seismic motion at free field and should be selected so as to induce maximum values of the seismic action effect under investigation.

(6) The requirements in **(5)** are deemed to be satisfied, by imposing each of the following two sets of horizontal displacements, applied separately, in each horizontal direction of the analysis, on the relevant support foundations or on the soil end of the relevant spring representing the soil stiffness. The effects of the two sets need not be combined.

a. Set A

Set A consists of relative displacements:

$$d_{\mathrm{ri}} = \varepsilon_{\mathrm{r}} L_{\mathrm{i}} \le d_{\mathrm{g}}\sqrt{2}$$

with $\varepsilon_{\mathrm{r}} = \dfrac{d_{\mathrm{g}}\sqrt{2}}{L_{\mathrm{g}}}$

applied simultaneously with the same sign (+ or –) to all supports of the bridge (*1* to *n*) in the horizontal direction considered (see Figure 3.1).

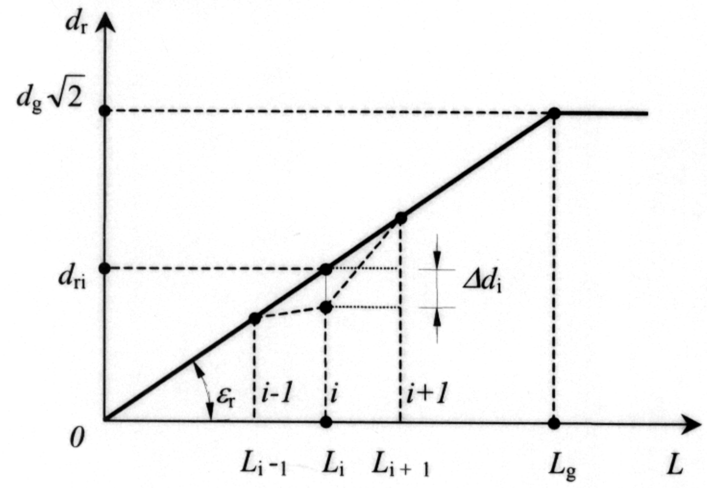


Figure 3.1 : Displacement Set A

where:

d_g is the design ground displacement corresponding to the ground type of support *i*, in accordance with EN 1998-1:2004, **3.2.2.4**;

L_i is the distance (projection on the horizontal plane) of support *i* from a reference support *i* = 0, that may be conveniently selected at one of the end supports;

L_g is the distance beyond which the ground motions may be considered as completely uncorrelated.

NOTE 1: The value ascribed to L_g for use in a country may be found in its National Annex. The recommended value is given in Table 3.1N, depending on the ground type:

Table 3.1N: distance beyond which ground motions may be considered uncorrelated

Ground Type	A	B	C	D	E
L_g (m)	600	500	400	300	500

b. Set B

Set B covers the influence of ground displacements occurring in opposite directions at adjacent piers. This is accounted for by assuming displacements Δd_i of any intermediate support i (>1) relative to its adjacent supports i-1 and i+1 considered undisplaced (see Figure 3.1).

$$\Delta d_i = \pm\beta_r\varepsilon_r L_{\alpha v,i}$$

where:

$L_{\alpha v,i}$ is the average of the distances $L_{i-1,i}$ and $L_{i,i+1}$ of intermediate support i to its adjacent supports i-1 and i+1 respectively. For the end supports (*0* and *n*) $L_{\alpha v,0} = L_{01}$ and $L_{\alpha v,n} = L_{n-1,n}$;

β_r is a factor accounting for the magnitude of ground displacements occurring in opposite direction at adjacent supports.

NOTE 2: The value ascribed to β_r for use in a country may be found in its National Annex. The recommended value is:

β_r = 0.5 when all three supports have the same ground type

β_r = 1.0 when the ground type at one of the supports is different than at the other two.

ε_r is as defined for set A above. If a change of ground type appears between two supports, the maximum value of ε_r should be used.

Set B consists of the following configuration of imposed absolute displacements with opposed sign at adjacent supports *i* and *i+1*, for *i* = 0 to *n-1* (see Figure 3.2).

$d_i = \pm \Delta d_i/2$

$d_{i+1} = \pm \Delta d_{i+1}/2$

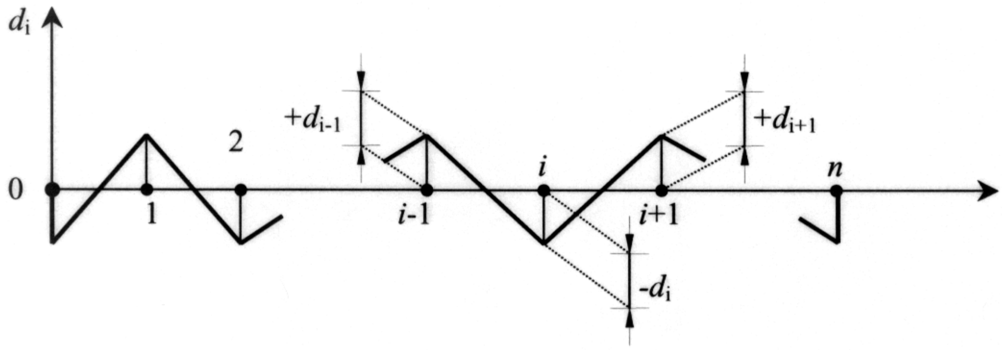


Figure 3.2 : Displacement Set B

(7)P In each horizontal direction the most severe effects resulting from the pseudo static analyses of **(5)** and **(6)** shall be combined with the relevant effects of the inertia response of **(4)**, by using the SSRS rule (square root of the sum of squares). The result of this combination constitutes the effects of the analysis in the direction considered. For the combination of the effects of the different components of seismic action, the rules of **4.2.1.4** are applicable.

(8) When time-history analysis is performed the seismic motions applied at each support should reflect with sufficient reliability the probable spatial variability of the seismic action.

NOTE Guidance for generating samples of seismic motion reflecting the probable spatial variability is given in **D.2** of Informative **Annex D**.

4 ANALYSIS

4.1 Modelling

4.1.1 Dynamic degrees of freedom

(1)P The model of the bridge and the selection of the dynamic degrees of freedom shall represent the distribution of stiffness and mass so that all significant deformation modes and inertia forces are activated under the design seismic excitation.

(2) It is sufficient, in certain cases, to use two separate models in the analysis, one for modelling the response in the longitudinal direction of the bridge, and the other for the transverse direction. The cases when it is necessary to consider the vertical component of the seismic action are defined in **4.1.7**.

4.1.2 Masses

(1)P The mean values of the permanent masses and the quasi-permanent values of the masses corresponding to the variable actions shall be considered.

(2) Distributed masses may be lumped at nodes in accordance with the selected degrees of freedom.

(3)P For design purposes the mean values of the permanent actions should be taken equal to their characteristic values. The quasi-permanent values of variable actions should be taken as equal to $\psi_{2,1}Q_{k,1}$ where $Q_{k,1}$ is the characteristic value of traffic load. In general and in accordance with EN 1990:2002, **Annex A2,** the value of $\psi_{2,1}$=0 shall be used for bridges with normal traffic and for footbridges.

(4)P For bridges with intense traffic $\psi_{2,1}$-values shall be applied to the uniform load of Model 1 (LM 1) in accordance with EN 1991-2:2003.

NOTE The value ascribed to: $\psi_{2,1}$ for use in a country in bridges with intense traffic may be found in its National Annex. The recommended values are:

For road bridges $\psi_{2,1} = 0,2$

For railway bridges $\psi_{2,1} = 0,3$

(5) When the piers are immersed in water, and unless a more accurate assessment of the hydrodynamic interaction is made, this effect may be estimated by taking into account an added mass of entrained water acting in the horizontal directions per unit length of the immersed pier. The hydrodynamic influence on the vertical seismic action may be omitted.

NOTE Informative **Annex F** gives a procedure for the calculation of the added mass of entrained water in the horizontal directions, for immersed piers.

4.1.3 Damping of the structure and stiffness of members

(1) When response spectrum analysis is used, the following values of equivalent viscous damping ratio ξ may be assumed, on the basis of the material of the members

where the larger part of the deformation energy is dissipated during the seismic response. In general this will occur in the piers.

Welded steel	0.02
Bolted steel	0.04
Reinforced concrete	0.05
Prestressed concrete	0.02

NOTE When the structure comprises several components *i* with different viscous damping ratios, ξ_i, the effective viscous damping of the structure ξ_{eff} may be estimated as:

$$\xi_{eff} = \frac{\Sigma \xi_i E_{di}}{\Sigma E_{di}}$$

where E_{di} is the deformation energy induced in component *i* by the seismic action. Effective damping ratios may be conveniently estimated separately for each eigenmode, on the basis of the relevant value of E_{di}.

(2) Member stiffness may be estimated in accordance with **2.3.6.1**.

(3) In concrete decks consisting of precast concrete beams and cast in-situ slabs, continuity slabs (see **2.3.2.2(4)**) should be included in the model of seismic analysis, taking into account their eccentricity relative to the deck axis and a reduced value of their flexural stiffness. Unless this stiffness is estimated on the basis of the rotation of the relevant plastic hinges, a value of 25% of the flexural stiffness of the uncracked gross concrete section may be used.

(4) For second order effects **2.4 (5)** and **5.4 (1)** apply. Significant second order effects may occur in bridges with slender piers and in special bridges, like arch and cable-stayed bridges.

4.1.4 Modelling of the soil

(1)P For the seismic analysis of the global system, the supporting members which transmit the seismic action from the soil to the deck shall, in general, be assumed as fixed relative to the foundation soil (see **3.1.2(3)**). Soil-structure interaction effects may be considered in accordance with EN 1998-5:2004, using appropriate impedances or appropriately defined soil springs.

(2) Soil-structure interaction effects should always be accounted for in piers where, under the action of a unit horizontal load in a given direction at the top of the pier, the soil flexibility contributes more than 20% of the total displacement at the top of the pier.

(3) Effects of soil-structure interaction on piles or shafts (caissons) shall be determined in accordance with EN 1998-5:2004, **5.4.2,** taking into account the provisions of **6.4.2**.

(4) In cases in which it is difficult to estimate reliably the mechanical properties of the soil, the analysis should be carried out using the estimated probable highest and

lowest values. High estimates of soil stiffness should be used for calculating the internal forces and low estimates for calculating the displacements of the bridge.

4.1.5 Torsional effects

(1)P Torsional motions of the bridge about a vertical axis shall be considered only in skewed bridges (skew angle $\varphi > 20^{o}$) and bridges with a ratio $B/L > 2,0$.

NOTE Such bridges tend to rotate about the vertical axis, even when the centre of mass theoretically coincides with the centre of stiffness. (L is the total length of the continuous deck and B is the width of the deck).

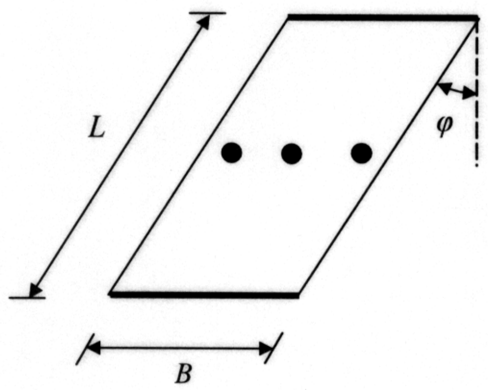


Figure 4.1: Skewed bridge

(2) Highly skewed bridges ($\varphi > 45^{o}$) should in general be avoided in high seismicity regions. If this is not possible, and the bridge is supported on the abutments through bearings, the actual horizontal stiffness of the bearings should be accurately modelled, taking into account the concentration of vertical reactions near the obtuse angles. Alternatively, an increased accidental eccentricity may be used.

(3)P When using the Fundamental Mode Method (see **4.2.2**) for the design of skewed bridges, the following equivalent static moment shall be considered to act about the vertical axis at the centre of gravity of the deck:

$$M_{\mathrm{t}} = \pm F\, e \tag{4.1}$$

where:

F is the horizontal force determined in accordance with expression (4.12);

$e = e_{\mathrm{a}} + e_{\mathrm{d}}$

$e_{\mathrm{a}} = 0{,}03L$ or $0{,}03B$ is the accidental eccentricity of the mass; and

$e_{\mathrm{d}} = 0{,}05L$ or $0{,}05B$ is an additional eccentricity reflecting the dynamic effect of simultaneous translational and torsional vibration.

For the calculation of e_{a} and e_{d} the dimension L or B transverse to the direction of excitation shall be used.

(4) When using a Full Dynamic Model (space model), the dynamic part of the torsional excitation is taken into account if the centre of mass is displaced by the accidental eccentricity e_a in the most unfavourable direction and sense. However, the torsional effects may also be estimated using the static torsional moment of expression (4.1).

(5)P The torsional resistance of a bridge structure shall not rely on the torsional rigidity of a single pier. In single span bridges the bearings shall be designed to resist the torsional effects.

4.1.6 Behaviour factors for linear analysis

(1)P The reference procedure of the present standard is a response spectrum analysis for the design spectrum defined in EN 1998-1:2004, **3.2.2.5** (see **3.2.4(1)**). The behaviour factor is defined globally for the entire structure and reflects its ductility capacity, i.e. the capability of the ductile members to withstand, with acceptable damage but without failure, seismic actions in the post-elastic range. The available levels of ductility are specified in **2.3.2**. The capability of ductile members to develop flexural plastic hinges is an essential requirement for the application of the values of the behaviour factor q specified in Table 4.1 for ductile behaviour.

NOTE The linear analysis method, using sufficiently conservative global force reduction factors (behaviour factors as defined by Table 4.1), is generally considered to be a reasonable compromise between the uncertainties intrinsic to the seismic problem and the relevant admissible errors on the one hand and the required effort for the analysis and design on the other.

(2) This required capability of ductile members to develop flexural plastic hinges is deemed to be ensured when the detailing rules of Section **6** are followed and capacity design in accordance with **5.3** is performed.

(3)P The maximum values of the behaviour factor q which may be used for the two horizontal seismic components are specified in Table 4.1, depending on the post-elastic behaviour of the ductile members where the main energy dissipation takes place. If a bridge has various types of ductile members, the behaviour factor q corresponding to the type-group with the major contribution to the seismic resistance shall be used. Different values of the behaviour factor q may be used in each of the two horizontal directions.

NOTE Use of behaviour factor values less than the maximum allowable specified in Table 4.1 will normally lead to reduced ductility demands, implying in general a reduction of potential damage. Such a use is therefore at the discretion of the designer and the owner.

Table 4.1: Maximum values of the behaviour factor *q*

Type of Ductile Members	Seismic Behaviour	
	Limited Ductile	Ductile
Reinforced concrete piers:		
Vertical piers in bending	1,5	3,5 $\lambda(\alpha_s)$
Inclined struts in bending	1,2	2,1 $\lambda(\alpha_s)$
Steel Piers:		
Vertical piers in bending	1,5	3,5
Inclined struts in bending	1,2	2,0
Piers with normal bracing	1,5	2,5
Piers with eccentric bracing	-	3,5
Abutments rigidly connected to the deck:		
In general	1,5	1,5
Locked-in structures (see. **4.1.6(9)**, **(10)**)	1,0	1,0
Arches	1,2	2,0
* $\alpha_s = L_s/h$ is the shear span ratio of the pier, where L_s is the distance from the plastic hinge to the point of zero moment and h is the depth of the cross-section in the direction of flexure of the plastic hinge. For $\alpha_s \geq 3$ $\lambda(\alpha_s) = 1{,}0$ $3 > \alpha_s \geq 1{,}0$ $\lambda(\alpha_s) = \sqrt{\frac{\alpha_s}{3}}$		

NOTE In piers of rectangular shape, when under the seismic action in the global direction under consideration, the compression zone has triangular shape, the minimum of the values of α_s, corresponding to the two sides of the section, should be used.

(4) For all bridges with regular seismic behaviour as specified in **4.1.8**, the values of the q-factor specified in Table 4.1 for Ductile Behaviour may be used without any special verification of the available ductility, provided that the detailing requirements specified in Section **6** are met. When only the requirements specified in **6.5** are met, the values of the q-factor specified in Table 4.1 for Limited Ductile Behaviour may be used without any special verification of the available ductility, regardless of the regularity or irregularity of the bridge.

(5)P For reinforced concrete ductile members the values of q-factors specified in Table 4.1 are valid when the normalised axial force η_k defined in **5.3(4)** does not exceed 0,30. If $0{,}30 < \eta_k \leq 0{,}60$ even in a single ductile member, the value of the behaviour factor shall be reduced to:

$$q_r = q - \frac{\eta_k - 0{,}3}{0{,}3}(q-1) \geq 1{,}0 \tag{4.2}$$

A value for q_r = 1,0 (elastic behaviour) should be used for bridges in which the seismic force resisting system contains members with $\eta_k \geq 0,6$.

(6) The values of the q-factor for Ductile Behaviour specified in Table 4.1 may be used only if the locations of all the relevant plastic hinges are accessible for inspection and repair. Otherwise, the values of Table 4.1 shall be multiplied by 0,6; however, final q-values less than 1,0 need not be used.

NOTE The term "accessible", as used in the paragraph above, has the meaning of "accessible even with reasonable difficulty". The foot of a pier shaft located in backfill, even at substantial depth, is considered to be "accessible". On the contrary, the foot of a pier shaft immersed in deep water, or the heads of piles beneath a large pile cap, should not be considered as "accessible".

(7) When energy dissipation is intended to occur at plastic hinges located in piles designed for ductile behaviour, and at points which are not accessible, the final q-value to be used need not be less than 2,1 for vertical piles and 1,5 for inclined piles (see also EN 1998-5:2004, **5.4.2(5)**).

(8) Subclause **2.3.2.2(4)**P applies for plastic hinge formation in the deck.

NOTE The potential formation of plastic hinges in secondary deck members (continuity slabs) is allowed in this case, but should not be relied upon to increase the value of q.

(9) Bridge structures the mass of which essentially follows the horizontal seismic motion of the ground ("locked-in" structures) do not experience significant amplification of the horizontal ground acceleration. Such structures are characterised by a very low value of the natural period in the horizontal directions ($T \leq 0,03$ s). The inertial response of these structures in the horizontal directions may be assessed by calculating the horizontal inertia forces directly from the design seismic ground acceleration and $q = 1$. Abutments flexibly connected to the deck belong to this category.

(10) Bridge structures consisting of an essentially horizontal deck rigidly connected to both abutments (either monolithically or through fixed bearings or links), may be considered to belong to the category of **(9)** irrespective of the value of the natural period, if the abutments are embedded in stiff natural soil formations over at least 80 % of their lateral area. If these conditions are not met, then the interaction with the soil at the abutments should be included in the model, using realistic soil stiffness parameters. If $T > 0,03$ s, then the design spectrum defined in EN 1998-1:2004, **3.2.2.5** should be used with $q = 1,50$.

(11)P When the main part of the design seismic action is resisted by elastomeric bearings, the flexibility of the bearings leads to a practically elastic behaviour of the system. Such bridges shall be designed in accordance with Section **7**.

NOTE: In general no plastic hinges will develop in piers which are flexibly connected to the deck in the direction considered. A similar situation will occur in individual piers with very low stiffness in comparison to the other piers (see **2.3.2.2(7)** and Note under **(9)**). Such members have negligible contribution in resisting the seismic actions and therefore do not affect the value of the q-factor (see **4.1.6(3)**P).

(12)P The behaviour factor for the analysis in the vertical direction shall always be taken as equal to 1,0.

4.1.7 Vertical component of the seismic action

(1) The effects of the vertical seismic component on the piers may be omitted in cases of low and moderate seismicity. In zones of high seismicity these effects need only be taken into account if the piers are subjected to high bending stresses due to vertical permanent actions of the deck, or when the bridge is located within 5 km of an active seismotectonic fault, with the vertical seismic action determined in accordance with **3.2.2.3**

(2)P The effects of the vertical seismic component acting in the upward direction on prestressed concrete decks, shall be always taken into account.

(3)P The effects of the vertical seismic component on bearings and links shall always be taken into account.

(4) The estimation of the effects of the vertical component may be carried out using the Fundamental Mode Method and the Flexible Deck Model (see **4.2.2.4**).

4.1.8 Regular and irregular seismic behaviour of ductile bridges

(1) Designating by $M_{\mathrm{Ed,i}}$ the maximum value of design moment at the intended plastic hinge location of ductile member i as derived from the analysis for the seismic design situation and by $M_{\mathrm{Rd,i}}$ the design flexural resistance of the same section with its actual reinforcement under the concurrent action of the non-seismic action effects in the seismic design situation, then the local force reduction factor r_{i} associated with member i, under the specific seismic action is defined as:

$$r_{\mathrm{i}} = q\frac{M_{\mathrm{Ed,i}}}{M_{\mathrm{Rd,i}}} \tag{4.3}$$

Note 1 Since $M_{\mathrm{Edi}} \leq M_{\mathrm{Rdi}}$, it follows that $r_{\mathrm{i}} \leq q$

Note 2 When in a regular bridge the maximum value of r_{i} among all ductile members, r_{max}, is substantially lower than q, the design cannot fully exploit the allowable maximum q-values. When $r_{\mathrm{max}} = 1{,}0$ the bridge responds elastically to the design earthquake considered.

(2)P A bridge shall be considered to have regular seismic behaviour in the considered horizontal direction, when the following condition is satisfied

$$\rho = \frac{r_{\mathrm{max}}}{r_{\mathrm{min}}} \leq \rho_{\mathrm{o}} \tag{4.4}$$

where:

r_{min} is the minimum value of r_{i} and

r_{max} is the maximum value of r_{i} among all ductile members i, and;

ρ_{o} is a limit value selected so as to ensure that sequential yielding of the ductile members will not cause unacceptably high ductility demands on one member.

NOTE The value ascribed to ρ_o for use in a country may be in found in its National Annex. The recommended value is $\rho_o = 2,0$.

(3) One or more ductile members (piers) may be exempted from the above calculation of r_{min} and r_{max}, if their shear contribution does not exceed 20% of the total seismic shear in the considered horizontal direction divided by the number of the piers resisting the seismic action.

(4)P Bridges that do not conform to expression (4.4), shall be considered to have irregular seismic behaviour, in the considered horizontal direction. Such bridges shall either be designed using a reduced q-value:

$$q_r = q\frac{\rho_o}{\rho} \geq 1,0 \tag{4.5}$$

or shall be designed based on results of non-linear analysis in accordance with **4.1.9**.

4.1.9 Non-linear analysis of irregular bridges

(1) In bridges of irregular seismic behaviour, the sequential yielding of the ductile members (piers) may cause substantial deviations of the results of the equivalent linear analysis performed with the assumption of a global force reduction factor q (behaviour factor) from those of the non-linear response of the bridge structure. The deviations are due mainly to the following effects.

- The plastic hinges which appear first usually develop the maximum post-elastic strains, which may lead to concentration of unacceptably high ductility demands in these hinges;
- Following the formation of the first plastic hinges (normally in the stiffer members), the distribution of stiffnesses and hence of forces may change from that predicted by the equivalent linear analysis. This may lead to a substantial change in the assumed pattern of plastic hinges.

(2) In general the realistic response of irregular bridges under the design seismic action may be estimated by means of a dynamic non-linear time-history analysis, performed in accordance with **4.2.4**.

(3) An approximation of the non-linear response may also be obtained by a combination of an equivalent linear analysis with a non-linear static analysis (pushover analysis) in accordance with **4.2.5**.

4.2 Methods of analysis

4.2.1 Linear dynamic analysis - Response spectrum method

4.2.1.1 Definition and field of application

(1) The Response Spectrum Analysis is an elastic calculation of the peak dynamic responses of all significant modes of the structure, using the ordinates of the site-

dependent design spectrum (see EN 1998-1:2004, **3.2.2.5**). The overall response is obtained by statistical combination of the maximum modal contributions. Such an analysis may be applied in all cases in which a linear analysis is allowed.

(2)P The earthquake action effects shall be determined from an appropriate discrete linear model (Full Dynamic Model), idealised in accordance with the laws of mechanics and the principles of structural analysis, and compatible with an associated idealisation of the seismic action. In general this model is a space model.

4.2.1.2 Significant modes

(1)P All modes making significant contribution to the total structural response shall be taken into account.

(2) For bridges in which the total mass M can be considered as a sum of "effective modal masses" M_i, the criterion **(1)** is deemed to be satisfied if the sum of the effective modal masses for the modes considered, $(\Sigma M_i)_c$, amounts to at least 90% of the total mass of the bridge.

(3) If the condition **(2)** is not satisfied after consideration of all modes with $T \geq$ 0,033 sec, the number of modes considered may be deemed acceptable provided that both of the following conditions are satisfied:

- $(\Sigma M_i)_c/M \geq 0{,}70$
- The final values of the seismic action effects are multiplied by $M/(\Sigma M_i)_c$

4.2.1.3 Combination of modal responses

(1)P In general the probable maximum value E of a seismic action effect (force, displacement etc.), shall be taken as equal to the square root of the sum of squares of the modal responses, E_i (SRSS-rule)

$$E = \sqrt{\sum E_i^2} \tag{4.6}$$

This action effect shall be assumed to act with plus and minus signs.

(2)P When two modes have closely spaced natural periods the SRSS rule (expression (4.6)) is unconservative and more accurate rules shall be applied. Two natural periods, T_i, T_j, may be considered as closely spaced natural periods if they satisfy the condition:

$$\frac{0{,}1}{0{,}1+\sqrt{\xi_i \xi_j}} \leq \rho_{ij} = T_i/T_j \leq 1+10\sqrt{\xi_i \xi_j} \tag{4.7}$$

where ξ_i and ξ_j are the viscous damping ratios of modes i and j respectively (see **(3)**),.

(3) For any two modes satisfying expression (4.7), the method of the Complete Quadratic Combination (CQC) may be used instead of the SRSS rule:

$$E = \sqrt{\Sigma_i \Sigma_j E_i r_{ij} E_j} \tag{4.8}$$

with: $i = 1 \ldots n$, $j = 1 \ldots n$

In expression (4.8) r_{ij} is the correlation factor:

$$r_{ij} = \frac{8\sqrt{\xi_i \xi_j}(\xi_i + \rho_{ij}\xi_j)\rho_{ij}^{3/2}}{(1-\rho_{ij}^2)^2 + 4\xi_i\xi_j\rho_{ij}(1+\rho_{ij}^2) + 4(\xi_i^2+\xi_j^2)\rho_{ij}^2} \tag{4.9}$$

where:

ξ_i, ξ_j are the viscous damping ratios i corresponding to modes i and j respectively.

NOTE Expression (4.9) gives $r_{ij} = r_{ji}$. When $T_i = T_j$, then $\xi_i = \xi_j$ and $r_{ij} = 1$.

4.2.1.4 Combination of the components of the seismic action

(1) The probable maximum action effect E, due to the simultaneous occurrence of the components of the seismic action along the horizontal axes X and Y and the vertical axis Z, may be estimated in accordance with EN 1998-1: 2004, **4.3.3.5.2(4)**, i.e. through application of the SRSS rule to the maximum action effects E_x, E_y and E_z due to independent seismic action along each axis:

$$E = \sqrt{E_x^2 + E_y^2 + E_z^2} \tag{4.10}$$

(2) Again in accordance with EN 1998-1: 2004, **4.3.3.5.2(4)**, the probable maximum action effect E may be taken as the most adverse of the effects calculated from EN 1998-1: 2004, expressions (4.18), (4.19), (4.20).

4.2.2 Fundamental mode method

4.2.2.1 Definition

(1) In the Fundamental mode method, equivalent static seismic forces are derived from the inertia forces corresponding to the fundamental mode and natural period of the structure in the direction under consideration, using the relevant ordinate of the site dependent design spectrum. The method also includes simplifications regarding the shape of the first mode and the estimation of the fundamental period.

(2) Depending on the particular characteristics of the bridge, this method may be applied using three different approaches for the model, namely:

– the Rigid Deck Model

– the Flexible Deck Model

– the Individual Pier Model

(3)P The rules of **4.2.1.4** for the combination of the components of seismic action shall be applied.

4.2.2.2 Field of application

(1) The method may be applied in all cases in which the dynamic behaviour of the structure can be sufficiently approximated by a single dynamic degree of freedom model. This condition is considered to be satisfied in the following cases.

(a) In the longitudinal direction of approximately straight bridges with continuous deck, when the seismic forces are carried by piers the total mass of which is less than 20% of the mass of the deck.

(b) In the transverse direction of case (a), if the structural system is approximately symmetric about the centre of the deck, i.e. when the theoretical eccentricity e_o between the centre of stiffness of the supporting members and the centre of mass of the deck does not exceed 5% of the length of the deck (L).

(c) In the case of piers carrying simply-supported spans, if no significant interaction between piers is expected and the total mass of each pier is less than 20% of the tributary mass of the deck.

4.2.2.3 Rigid deck model

(1) This model may only be applied, when, under the seismic action, the deformation of the deck within a horizontal plane is negligible compared to the horizontal displacements of the pier tops. This condition is always met in the longitudinal direction of approximately straight bridges with continuous deck. In the transverse direction the deck may be assumed rigid either if $L/B \leq 4,0$, or if the following condition is satisfied:

$$\frac{\Delta_d}{d_a} \leq 0,20 \qquad (4.11)$$

where:

L is the total length of the continuous deck;

B is the width of the deck; and

Δ_d and d_a are respectively the maximum difference and the average of the displacements in the transverse direction of all pier tops under the transverse seismic action, or under the action of a transverse load of similar distribution.

(2)P The earthquake effects shall be determined by applying a horizontal equivalent static force F at the deck given by the expression:

$$F = M\,S_d(T) \qquad (4.12)$$

where:

M is the total effective mass of the structure, equal to the mass of the deck plus the mass of the upper half of the piers;

$S_d(T)$ is the spectral acceleration of the design spectrum (EN 1998-1:2004, **3.2.2.5**) corresponding to the fundamental period T of the bridge, estimated as:

$$T = 2\pi\sqrt{\frac{M}{K}} \tag{4.13}$$

where $K=\Sigma K_i$ is the stiffness of the system, equal to the sum of the stiffnesses of the resisting members.

(3) In the transverse direction the force F may be distributed along the deck proportionally to the distribution of the effective masses.

4.2.2.4 Flexible deck model

(1)P The Flexible Deck Model shall be used when expression (4.11) is not satisfied.

(2) Unless a more accurate calculation is made, the fundamental period of the structure in the horizontal direction considered, may be estimated via the Rayleigh quotient, using a generalised single-degree-of-freedom system, as follows:

$$T = 2\pi\sqrt{\frac{\sum M_i d_i^2}{g\sum M_i d_i}} \tag{4.14}$$

where:

M_i is the mass at the *i*-th nodal point

d_i is the displacement in the direction under examination when the structure is acted upon by forces gM_i acting at all nodal points in the horizontal direction considered.

(3)P The earthquake effects shall be determined by applying horizontal forces F_i at all nodal points given by:

$$F_i = \frac{4\pi^2}{gT^2} S_d(T) d_i M_i \tag{4.15}$$

where:

T is the period of the fundamental mode of vibration for the horizontal direction considered,

M_i is the mass concentrated at the *i*-th point,

d_i is the displacement of the *i*-th nodal point in an approximation of the shape of the first mode (may be taken as equal to the values determined in **(2)** above),

$S_d(T)$ is the spectral acceleration of the design spectrum (EN 1998-1:2004, **3.2.2.5**), and

g is the acceleration of gravity.

4.2.2.5 Torsional effects in the transverse direction (rotation about the vertical axis)

(1) When the Rigid or the Flexible Deck Model is used in the transverse direction of a bridge, torsional effects may be estimated by applying a static torsional moment M_t in accordance with expression (4.1) of **4.1.5(3)**P. The relevant eccentricity shall be estimated as follows:

$$e = e_o + e_a \tag{4.16}$$

where:

e_o is the theoretical eccentricity (see case (b) of **4.2.2.2(1)**)

e_a = $0{,}05L$ is an additional eccentricity accounting for accidental and dynamic amplification effects

(2) The force F may be determined either from expression (4.12), or as ΣF_i from expression (4.15). The moment M_t may be distributed to the supporting members using the Rigid Deck Model.

4.2.2.6 Individual pier model

(1) In some cases the seismic action in the transverse direction of the bridge is resisted mainly by the piers, without significant interaction between adjacent piers. In such cases the seismic action effects acting in the i-th pier may be approximated by applying on it an equivalent static force:

$$F_i = M_i \, S_d(T_i) \tag{4.17}$$

where

M_i is the effective mass attributed to pier i and

$$T_i = 2\pi \sqrt{\frac{M_i}{K_i}} \tag{4.18}$$

is the fundamental period of the same pier, considered independently of the rest of the bridge.

(2) This simplification may be applied as a first approximation for preliminary analyses, when the following condition is met by the results of expression (4.18) for all adjacent piers i and i+1:

$$0{,}90 \leq T_i / T_{i+1} \leq 1{,}10 \tag{4.19}$$

Otherwise a redistribution of the effective masses attributed to each pier is required, leading to the satisfaction of the above condition.

4.2.3 Alternative linear methods

4.2.3.1 Time series analysis

(1)P In a time series analysis, the design seismic action shall be taken as the average of the extreme response computed for each accelerogram in a set of time-histories considered. Subclause **3.2.3** applies for the choice of time-histories.

4.2.4 Non-linear dynamic time-history analysis

4.2.4.1 General

(1)P The time dependent response of the structure shall be obtained through direct numerical integration of its non-linear differential equations of motion. The seismic input shall consist of ground motion time-histories (accelerograms, see **3.2.3**). The effects of gravity loads and of the other quasi-permanent actions in the seismic design situation, as well as second order effects, shall be taken into account.

(2)P Unless otherwise specified in this Part, this method can be used only in combination with a standard response spectrum analysis to provide insight into the post -elastic response and comparison between required and available local ductility demands. Generally, the results of the non-linear analysis shall not be used to relax requirements resulting from the response spectrum analysis. However, in the cases of bridges with isolating devices (see Section **7**) or irregular bridges (see **4.1.8**) lower values estimated from a rigorous time-history analysis may be substituted for the results of the response spectrum analysis.

4.2.4.2 Ground motions and design combination

(1)P The provisions of **3.2.3** apply.

(2)P The provisions of **5.5(1)** and **4.1.2** apply.

4.2.4.3 Design response effects

(1)P When non-linear dynamic analysis is performed for at least seven independent pairs of horizontal ground motions, the average of the individual responses may be used as the design value of the action effects, except if otherwise required in this part. When less than seven non-linear dynamic analyses are performed for the corresponding independent pairs of input motions, the maximum responses of the ensemble should be used as design action effects.

4.2.4.4 Ductile structures

(1) **Objectives**

The main objectives of a non-linear time-history analysis of a ductile bridge are the following.

- The identification of the actual pattern of plastic hinge formation
- The estimation and verification of the probable post-yield deformation demands in plastic hinges and the estimation of the displacement demands
- The determination of the strength requirements for the prevention of non-ductile failure modes in the superstructure and for the verification of the soil.

(2) **Requirements**

For a ductile structure subjected to high local ductility demands, achievement of the above objectives requires the following.

(a) A realistic identification of the extent of the structure that remains elastic. Such an identification should be based on probable values of the yield stresses and strains of the materials.

(b) In the regions of plastic hinges, the stress-strain diagrams for both concrete and reinforcement or structural steel, should reflect the probable post-yield behaviour, taking into account confinement of concrete, when relevant, and strain hardening and/or local buckling effects for steel. The shape of hysteresis loops should be properly modelled, taking into account strength and stiffness degradation and hysteretic pinching, if indicated by appropriate laboratory tests.

(c) The verification that deformation demands are safely lower than the capacities of the plastic hinges, should be performed by comparing plastic hinge rotation demands, $\theta_{p,E}$, to the relevant design rotation capacities, $\theta_{p,d}$, as follows:

$$\theta_{p,E} \leq \theta_{p,d} \tag{4.20}$$

The design values of the plastic rotation capacities, $\theta_{p,d}$, should be derived from relevant test results or calculated from ultimate curvatures, by dividing the probable value $\theta_{p,u}$ by a factor, $\gamma_{R,p}$, that reflects local defects of the structure, uncertainties of the model and/or the dispersion of relevant test results, as follows:

$$\theta_{p,d} = \frac{\theta_{p,u}}{\gamma_{R,p}} \tag{4.21}$$

The same condition should be checked for other deformation demands and capacities of dissipative zones of steel structures (e.g. elongation of tensile members in diagonals and shear deformation of shear panels in eccentric bracings).

NOTE Informative Annex E gives information for the estimation of θ_{pd} and for $\gamma_{R,p}$

(d) Member strength verification against bending with axial force is not needed, as such a verification is inherent in the non-linear analysis procedure according to (a) above. However it should be verified that no significant yield occurs in the deck (**5.6.3.6(1)**P and **(2)**).

(e) Verification of members against non-ductile failure modes (shear of members and shear in joints adjacent to plastic hinges), as well as of foundation failure, should be performed in accordance with the relevant rules of Section **5**. The capacity design action effects should be taken as the action effects resulting from the non-linear analysis multiplied by γ_{Bd1}, in accordance with **5.6.2(2)**Pb. These values should not exceed the design resistances R_d $(= R_k/\gamma_M)$ of the corresponding sections, i.e.:

$$\max E_d \leq R_d \tag{4.22}$$

4.2.4.5 Bridges with seismic isolation

(1) The objective of the analysis in this case is the realistic assessment of the displacement and force demands:

- properly taking into account the effect of the variability of the properties of the isolators, and
- ensuring that the isolated structure remains essentially elastic

(2) The provisions of Section **7** apply.

4.2.5 Static non-linear analysis (pushover analysis)

(1)P Pushover analysis is a static non-linear analysis of the structure under constant vertical (gravity) loads and monotonically increased horizontal loads, representing the effect of a horizontal seismic component. Second order effects shall be accounted for. The horizontal loads are increased until a target displacement is reached at a reference point.

(2) The main objectives of the analysis are the following.

- The estimation of the sequence and the final pattern of plastic hinge formation;
- The estimation of the redistribution of forces following the formation of plastic hinges;
- The assessment of the force-displacement curve of the structure ("capacity curve") and of the deformation demands of the plastic hinges up to the target displacement.

(3) The method may be applied to the entire bridge structure or to individual components.

(4) The requirements of **4.2.4.4(2)** apply, with the exception of the requirement for modelling of the hysteresis loop shape in **4.2.4.4(2)**b.

NOTE 1 A recommended procedure for the application of this method is given in Informative Annex **H**.

NOTE 2 It is noted that a static non-linear (pushover) analysis, such as the one given in Annex **H**, leads to realistic results in structures, the response of which to the horizontal seismic action in the direction considered can be reasonably approximated by a generalized one degree of freedom system. Assuming the influence of the pier masses to be minor, the above condition is always met in the longitudinal direction of approximately straight bridges. The condition is also met in the transverse direction, when the distribution of the stiffness of piers along the bridge provides a more or less uniform lateral support to a relatively stiff deck. This is the most common case for bridges where the height of the piers decreases towards the abutments or does not present intense variations. When, however, the bridge has one exceptionally stiff and unyielding pier, located between groups of regular piers, the system cannot be approximated in the transverse direction by a single-degree-of-freedom and the pushover analysis may not lead to realistic results. A similar exception holds for long bridges, when very stiff piers are located between groups of regular ones, or in bridges in which the mass of some piers has a significant effect on the dynamic behaviour, in either of the two directions. Such irregular arrangements may be avoided, e.g. by providing sliding connection between the deck and the pier(s) causing the irregularity. If this is not possible or expedient, then non-linear time history analysis should be used.

5 STRENGTH VERIFICATION

5.1 General

(1)P The provisions of this Section apply to the earthquake resisting system of bridges designed by an equivalent linear method taking into account a ductile or limited ductile behaviour of the structure (see **4.1.6**). For bridges provided with isolating devices, Section **7** shall be applied. For verifications on the basis of results of non-linear analysis, **4.2.4** applies. In both latter cases **5.2.1** applies.

5.2 Materials and design strength

5.2.1 Materials

(1)P In bridges designed for ductile behaviour with $q > 1,5$, concrete members where plastic hinges may form, shall be reinforced with steel of Class C in accordance with EN 1992-1-1:2004, Table C.1.

(2) Concrete members of bridges designed for ductile behaviour, where no plastic hinges may form (as a consequence of capacity design), as well as all concrete members of bridges designed for limited ductile behaviour ($q \leq 1,5$) or all concrete members of bridges with seismic isolation in accordance with Section **7**, may be reinforced using steel of Class B in accordance with EN 1992-1-1:2004, Table C.4.

(3)P Structural steel members of all bridges shall conform to EN 1998-1: 2004, **6.2**.

5.2.2 Design strength

(1)P The design value of member resistance shall be determined in accordance with EN 1998-1:2004, **5.2.4**, **6.1.3** or **7.1.3**, as appropriate.

5.3 Capacity design

(1)P For structures designed for ductile behaviour, capacity design effects F_C (V_C, M_C, N_C) shall be calculated by analysing the intended plastic mechanism under:

a) the non-seismic actions in the design seismic situation and

b) the level of seismic action in the direction under consideration (see **(6)**) at which all intended flexural hinges have developed bending moments equal to an upper fractile of their flexural resistance, called the overstrength moment, M_o.

(2) The capacity design effects need not be taken as greater than those resulting at the seismic design situation (see **5.5**) in the direction under consideration, with the seismic action effects multiplied by the behaviour factor q used in the analysis for the design seismic action.

(3)P The overstrength moment of a section shall be calculated as:

$$M_o = \gamma_o M_{Rd} \tag{5.1}$$

where:

γ_o is the overstrength factor;

M_{Rd} is the design flexural strength of the section, in the selected direction and sign, based on the actual section geometry, including reinforcement where relevant, and material properties (with $\gamma_M$values for fundamental design situations). In determining M_{Rd}, biaxial bending shall be taken into account under: (a) the action effects of the non-seismic actions in the seismic design situation and (b) the other seismic action effects corresponding to the design seismic action with the selected direction and sign.

(4) The value of the overstrength factor should reflect the variability of material strength properties, and the ratio of the ultimate strength to the yield strength.

NOTE The value ascribed to γ_o for use in a country may be found in its National Annex. The recommended values are:
For concrete members: $\gamma_o = 1{,}35$;
For steel members: $\gamma_o = 1{,}25$.

In the case of reinforced concrete sections with special confining reinforcement in accordance with **6.2.1**, and with the value of the normalized axial force

$$\eta_k = N_{Ed}/(A_c f_{ck}) \tag{5.2}$$

exceeding 0,1, the value of the overstrength factor shall be multiplied by $1+2(\eta_k-0{,}1)^2$

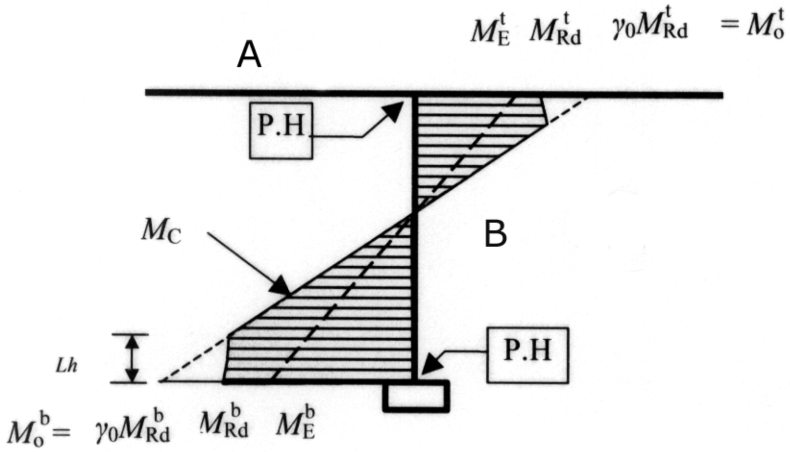
where:

N_{Ed} is the value of the axial force at the plastic hinge seismic design situation, positive if compressive;

A_c is the cross-sectional area of the section; and

f_{ck} is the characteristic concrete strength.

(5)P Within the length of members that develop plastic hinge(s), the capacity design bending moment M_c at the vicinity of the hinge (see Figure 5.1) shall not be assumed to be greater than the relevant design flexural resistance M_{Rd} of the nearest hinge calculated in accordance with **5.6.3.1**.



Key
A – Deck
B – Pier
PH – Plastic Hinge

Figure 5.1: Capacity design moments M_C within the length of member containing plastic hinges

NOTE 1: The M_{Rd}-diagrams shown in Figure 5.1 correspond to a pier with variable cross-section (increasing downwards). In the case of a constant cross-section with constant reinforcement, M_{Rd} is also constant.

NOTE 2: For L_h see **6.2.1.5**.

(6) In general capacity design effects should be calculated separately for seismic action acting (with + and – sign) in each of the longitudinal and the transverse directions. A relevant procedure and simplifications are given in Annex **G**.

(7)P When sliding bearings participate in the plastic mechanism, their capacity shall be assumed as equal to $\gamma_{of} R_{df}$, where:

$\gamma_{of} = 1,30$ is a magnification factor for friction due to ageing effects and

R_{df} is the maximum design friction force of the bearing.

(8)P In bridges with elastomeric bearings and intended to have ductile behaviour, members where no plastic hinges are intended to form and which resist shear forces from the bearings shall be designed as follows. The capacity design effects shall be calculated on the basis of the maximum deformation of the bearings corresponding to the design displacement of the deck and a bearing stiffness increased by 30%.

5.4 Second order effects

(1) For linear analysis, approximate methods may be used for estimating the influence of second order effects on the critical sections (plastic hinges), also taking into account the cyclic character of the seismic action wherever it has a significant unfavourable effect.

NOTE: Approximate methods for use in a country to estimate second order effects under seismic actions may be found in its National Annex. The recommended procedure is to assume that the increase of bending moments of the plastic hinge section due to second order effects, is:

$$\Delta M = \frac{1+q}{2} d_{\mathrm{Ed}} N_{\mathrm{Ed}} \tag{5.3}$$

where N_{Ed} is the axial force and d_{Ed} is the relative transverse displacement of the ends of the considered ductile member, both in the design seismic situation.

5.5 Combination of the seismic action with other actions

(1)P The design value E_{d} of the effects of actions in the seismic design situation shall be determined in accordance with EN 1990:2002, **6.4.3.4** and EN 1998-1:2004, **3.2.4(1)** as:

$$E_{\mathrm{d}} = G_{\mathrm{k}} \text{"+"} P_{\mathrm{k}} \text{"+"} A_{\mathrm{Ed}} \text{"+"} \psi_{21} Q_{1\mathrm{k}} \text{"+"} Q_2 \tag{5.4}$$

where:

“+” implies “to be combined with”;

G_{k} are the permanent actions with their characteristic values;

P_{k} is the characteristic value of prestressing after all losses;

A_{Ed} is the design seismic action;

$Q_{1\mathrm{k}}$ is the characteristic value of the traffic load;

ψ_{21} is the combination factor for traffic loads in accordance with **4.1.2 (3)**P; and

Q_2 is the quasi-permanent value of actions of long duration (e.g. earth pressure, buoyancy, currents etc.)

NOTE Actions of long duration are considered to be concurrent with the design seismic action.

(2)P Seismic action effects need not be combined with action effects due to imposed deformations (caused by temperature, shrinkage, settlements of supports, residual ground movements due to seismic faulting).

(3)P An exception to the rule in **(2)**P is the case of bridges in which the seismic action is resisted by elastomeric laminated bearings (see also **6.6.2.3(4)**). In such a case elastic behaviour of the system shall be assumed and the action effects due to imposed deformations shall be accounted for.

NOTE In the case of **(3)**P the displacement due to creep does not normally induce additional stresses to the system and can therefore be neglected. Creep also reduces the effective stresses induced in the structure by long-term imposed deformations (e.g. by shrinkage).

(4)P Wind and snow actions shall be neglected in the design value E_{d} of the effects of actions in the seismic design situation (expression (5.4)).

5.6 Resistance verification of concrete sections

5.6.1 Design resistance

(1) When the resistance of a section depends on multi-component action effects (e.g. bending moment, uniaxial or biaxial and axial force), the Ultimate Limit State conditions specified in **5.6.2** and **5.6.3** may be satisfied by considering separately the extreme (maximum or minimum) value of each component of the action effect with the concurrent values of all other components of the action effect.

5.6.2 Structures of limited ductile behaviour

(1)P For flexural resistance of sections the following condition shall be satisfied:

$$E_{\mathrm{d}} \leq R_{\mathrm{d}} \tag{5.5}$$

where:

E_{d} is the design action effect in the seismic design situation including second order effects; and

R_{d} is the design flexural resistance of the section in accordance with EN 1992-1-1:2004, **6.1** and with **5.6.1(1)**.

(2)P Verifications of shear resistance of concrete members shall be carried out in accordance with EN 1992-1-1:2004, **6.2,** with the following additional rules.

a) The design action effects shall be calculated in accordance with **5.5(1)**P, where the seismic action effect A_{Ed} shall be multiplied by the behaviour factor q used in the linear analysis.

b) The resistance values, $V_{\mathrm{Rd,c}}$, $V_{\mathrm{Rd,s}}$ and $V_{\mathrm{Rd,max}}$ derived in accordance with EN 1992-1-1:2004, **6.2** shall be divided by an additional safety factor γ_{Bd1} against brittle failure.

NOTE The value ascribed to γ_{Bd1} fro use in a country may be found in its National Annex. The recommended value is γ_{Bd1} = 1,25.

5.6.3 Structures of ductile behaviour

5.6.3.1 Flexural resistance of sections of plastic hinges

(1)P The following condition shall be satisfied.

$$M_{\mathrm{Ed}} \leq M_{\mathrm{Rd}} \tag{5.6}$$

where:

M_{Ed} is the design value of the moment as derived from the analysis for the seismic design situation, including second order effects; and

M_{Rd} is the design flexural resistance of the section, in accordance with **5.6.1(1)**.

(2)P The longitudinal reinforcement of the member containing the hinge shall remain constant and fully effective over the length L_h shown in Figure 5.1 and specified in **6.2.1.5**.

5.6.3.2 Flexural resistance of sections outside the region of plastic hinges

(1)P The following condition shall be satisfied.

$$M_C \leq M_{Rd} \tag{5.7}$$

where :

M_C is the capacity design moment as specified in **5.3**; and

M_{Rd} is the design resistance of the section in accordance with EN 1992-1-1:2004, **6.1** taking into account the interaction of the other components of the design action effect (axial force and, when applicable, bending moment in the orthogonal direction).

NOTE As a consequence of **5.3(5)**P, the cross-section and the longitudinal reinforcement of the plastic hinge section shall not be affected by the capacity design verification.

5.6.3.3 Shear resistance of members outside the region of plastic hinges

(1)P Verifications of shear resistance shall be carried out in accordance with EN 1992-1-1:2004, **6.2,** with the following additional rules:

a) The design action effects shall be assumed equal to the capacity design effects in accordance with **5.3**;

b) The resistance values, $V_{Rd,c}$, $V_{Rd,s}$ and $V_{Rd,max}$ derived in accordance with EN 1992-1-1:2004, **6.2** shall be divided by an additional safety factor γ_{Bd} against brittle failure. The following value of γ_{Bd} shall be used.

$$1 \leq \gamma_{Bd} = \gamma_{Bd1} + 1 - \frac{qV_{Ed}}{V_{C,o}} \leq \gamma_{Bd1} \tag{5.8}$$

where:

γ_{Bd1} is in accordance with **5.6.2(2)**P;

V_{Ed} is the maximum value of the shear in seismic design situation of **5.5(1)**P; and

$V_{C,o}$ is the capacity design shear determined in accordance with **5.3**, without considering the limitation of **5.3(2)**.

(2) Unless a more accurate calculation is made, for circular concrete sections of radius r where the longitudinal reinforcement is distributed over a circle with radius r_s, the effective depth:

$$d_e = r + \frac{2r_s}{\pi} \tag{5.9}$$

may be used instead of d in the relevant expressions for the shear resistance. The value of the internal lever arm z may be assumed to be equal to: $z = 0{,}9d_e$.

5.6.3.4 Shear resistance of plastic hinges

(1)P Subclause **5.6.3.3(1)**P applies.

(2)P The angle θ between the concrete compression strut and the main tension chord shall be assumed to be equal to 45°.

(3)P The dimensions of the confined concrete core to the centre line of the perimeter hoop shall be used in lieu of the section dimensions b_w and d.

(4) Subclause **5.6.3.3(2)** may be applied using the dimensions of the confined concrete core.

(5) For members with shear span ratio $\alpha_s < 2{,}0$ (see Table 4.1 for the definition of α_s), verification of the pier against diagonal tension and sliding failure should be carried out in accordance with EN 1998-1:2004, **5.5.3.4.3** and **5.5.3.4.4,** respectively. In these verifications, the capacity design effects should be used as design action effects.

5.6.3.5 Verification of joints adjacent to plastic hinges

5.6.3.5.1 General

(1)P Any joint between a vertical ductile pier and the deck or a foundation element adjacent to a plastic hinge in the pier, shall be designed in shear to resist the capacity design effects of the plastic hinge in the relevant direction. The pier is indexed in the following paragraphs with "c" (for "column"), while any other member framing into the same joint is referred to as "beam" and indexed with "b".

(2)P For a vertical solid pier of depth h_c and of width b_c transverse to the direction of flexure of the plastic hinge, the effective width of the joint shall be assumed as follows:

– when the pier frames into a slab or a transverse rib of a hollow slab:

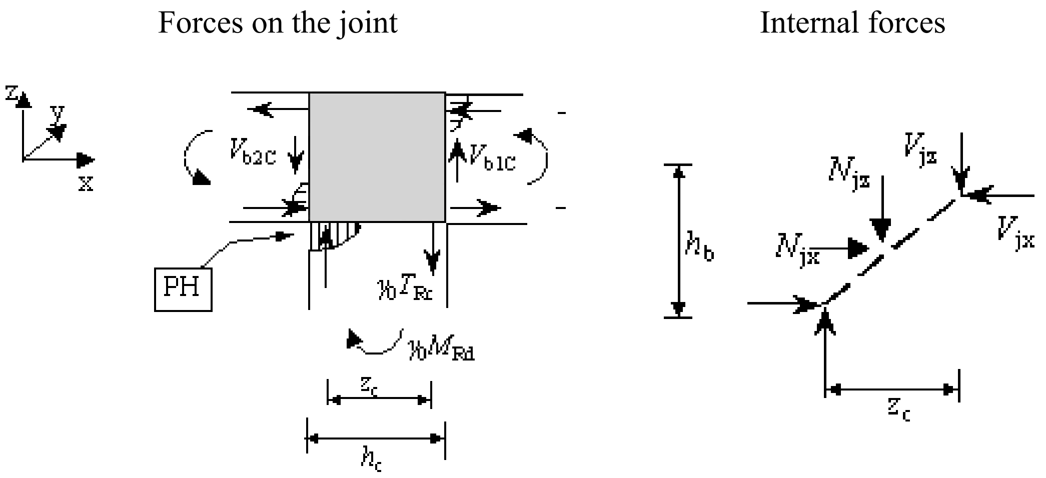
$$b_j = b_c + 0{,}5h_c \tag{5.10}$$

– when the pier frames directly into a longitudinal web of width b_w (b_w is parallel to b_c):

$$b_j = \min(b_w;\ b_c + 0{,}5h_c) \tag{5.11}$$

– for circular piers of diameter d_c, the above definitions are applied assuming $b_c = h_c = 0{,}9d_c$

5.6.3.5.2 Joint forces and stresses



Key
PH – Plastic Hinge

Figure 5.2: Joint forces

(1)P The design vertical shear of the joint, V_{jz}, shall be assumed as:

$$V_{jz} = \gamma_0 T_{Rc} - V_{b1C} \tag{5.12}$$

where:

T_{Rc} is the resultant force of the tensile reinforcement of the pier corresponding to the design flexural resistance, M_{Rd}, of the plastic hinge in accordance with **5.3(3)**P, and γ_0 is the overstrength factor in accordance with **5.3(3)**P and **5.3(4)** (capacity design); and

V_{1bC} is the shear force of the "beam" adjacent to the tensile face of the column, corresponding to the capacity design effects of the plastic hinge.

(2) The design horizontal shear of the joint V_{jx} may be calculated as (see Figure 5.2):

$$V_{jx} = V_{jz} \frac{z_c}{z_b} \tag{5.13}$$

where z_c and z_b are the internal lever arms of the plastic hinge and the "beam" end sections, respectively, and z_c and z_b may be assumed to be equal to 0,9 times the relevant effective section depths (see **5.6.3.3** and **5.6.3.4**).

(3) The shear verification should be carried out at the centre of the joint, where, in addition to V_{jz} and V_{jx}, the influence of following axial forces may be taken into account:

– vertical axial joint force N_{jz} equal to:

$$N_{jz} = \frac{b_c}{2b_j} N_{cG} \tag{5.14}$$

where:

N_{cG} is the axial force of the column under the non-seismic actions in the design seismic situation;

horizontal force N_{jx} equal to the capacity design axial force effects in the "beam", including the effects of longitudinal prestressing after all losses, if such axial forces are actually effective throughout the width b_j of the joint;

horizontal force N_{jy} in the transverse direction equal to the effect of transverse prestressing after all losses, effective within the depth h_c, if such prestressing is provided.

(4) For the joint verification the following average nominal stresses are used.

Shear stresses:

$$v_j = v_x = v_z = \frac{V_{jx}}{b_j z_c} = \frac{V_{jz}}{b_j z_b} \tag{5.15}$$

Axial stresses:

$$n_z = \frac{N_{jz}}{b_j h_c} \tag{5.16}$$

$$n_x = \frac{N_{jx}}{b_j h_b} \tag{5.17}$$

$$n_y = \frac{N_{jy}}{h_b h_c} \tag{5.18}$$

NOTE: As pointed out in **5.3(6)**, the capacity design, and therefore the relevant joint verification, should be carried out with both signs of the seismic action, + and -. It is also noted that at knee-joints (e.g. over the end column of a multi-column bent in the transverse bridge direction), the sign of M_{Rd} and V_{b1C} may be opposite to that shown in Figure 5.2 and N_{jx} may be tensile.

5.6.3.5.3 Verifications

(1) If the average shear stress in the joint, v_j, does not exceed the cracking shear capacity of the joint, $v_{j,cr}$, as given by expression (5.19), then minimum reinforcement should be provided, in accordance with **(6)**P.

$$v_j \leq v_{j,cr} = f_{ctd}\sqrt{\left(1+\frac{n_x}{f_{ctd}}\right)\left(1+\frac{n_z}{f_{ctd}}\right)} \leq 1{,}50 f_{ctd} \tag{5.19}$$

where: $f_{ctd} = f_{ctk0,05}/\gamma_c$ is the design value of the tensile strength of concrete.

(2)P The diagonal compression induced in the joint by the diagonal strut mechanism shall not exceed the compressive strength of concrete in the presence of transverse tensile strains, taking into account also confining pressures and reinforcement.

(3) Unless a more accurate model, the requirement of **(2)**P above is deemed to be satisfied, if the following condition is met.

$$v_{j} \leq v_{j,Rd} = 0{,}5\alpha_{c}\nu f_{cd} \tag{5.20}$$

where,

$$\nu = 0{,}6\ (1-(f_{ck}/250)) \quad \text{(with } f_{ck} \text{ in MPa)} \tag{5.21}$$

The factor α_c in expression (5.20) accounts for the effects of any confining pressure (n_{jy}) and/or reinforcement (ρ_y) in the transverse direction y, on the compressive strength of the diagonal strut:

$$\alpha_{c} = 1 + 2(n_{jy} + \rho_{y}f_{sd})/f_{cd} \leq 1{,}5 \tag{5.22}$$

where:

$\rho_y = A_{sy}/(h_c h_b)$ is the reinforcement ratio of any closed stirrups in the transverse direction of the joint panel (orthogonal to the plane of action), and

$f_{sd} = 300$ MPa is a reduced stress of this transverse reinforcement, for reasons of limitation of cracking.

(4) Reinforcement, both horizontal and vertical, should be provided in the joint, at amounts adequate to carry the design shear force. This requirement may be satisfied by providing horizontal and vertical reinforcement ratios, ρ_x and ρ_z, respectively, such that:

$$\rho_{x} = \frac{v_{j} - n_{x}}{f_{sy}} \tag{5.23}$$

$$\rho_{z} = \frac{v_{j} - n_{z}}{f_{sy}} \tag{5.24}$$

where:

$\rho_{x} = \dfrac{A_{sx}}{b_{j}h_{b}}$ is the reinforcement ratio in the joint panel in the horizontal direction,

$\rho_{z} = \dfrac{A_{sz}}{b_{j}h_{c}}$ is the reinforcement ratio in the joint panel in the vertical direction, and

f_{sy}, is the design yield strength of the joint reinforcement.

(5)P The joint reinforcement ratios ρ_x and ρ_y shall not exceed the maximum value:

$$\rho_{max} = \frac{\nu f_{cd}}{2f_{sy}} \tag{5.25}$$

where ν is given by expression (5.21)

(6)P A minimum amount of shear reinforcement shall be provided in the joint panel in both horizontal directions, in the form of closed links. The required minimum amount is,

$$\rho_{\min} = \frac{f_{\mathrm{ctd}}}{f_{\mathrm{sy}}} \tag{5.26}$$

5.6.3.5.4 Reinforcement arrangement

(1) Vertical stirrups should enclose the longitudinal “beam” reinforcement at the face opposite to the pier. Horizontal stirrups should enclose the pier vertical reinforcement, as well as “beam” horizontal bars anchored into the joint. Continuation of pier stirrups/hoops into the joint is recommended.

(2) Up to 50% of the total amount of vertical stirrups required in the joint may be U-bars, enclosing the longitudinal “beam” reinforcement at the face opposite to the column (see Figure 5.3).

(3) 50% of the bars of the top and bottom longitudinal reinforcement of the “beams”, when continuous through the joint body and adequately anchored beyond it, may be taken into account for covering the required horizontal joint reinforcement area A_{sx}.

(4) The longitudinal (vertical) pier reinforcement should reach as far as possible into the “beam”, ending just before the reinforcement layers of the “beam” at the face opposite to the pier-“beam” interface. In the direction of flexure of the plastic hinge, the bars of both tensile regions of the pier should be anchored by a rectangular hook directed towards the centre of the pier.

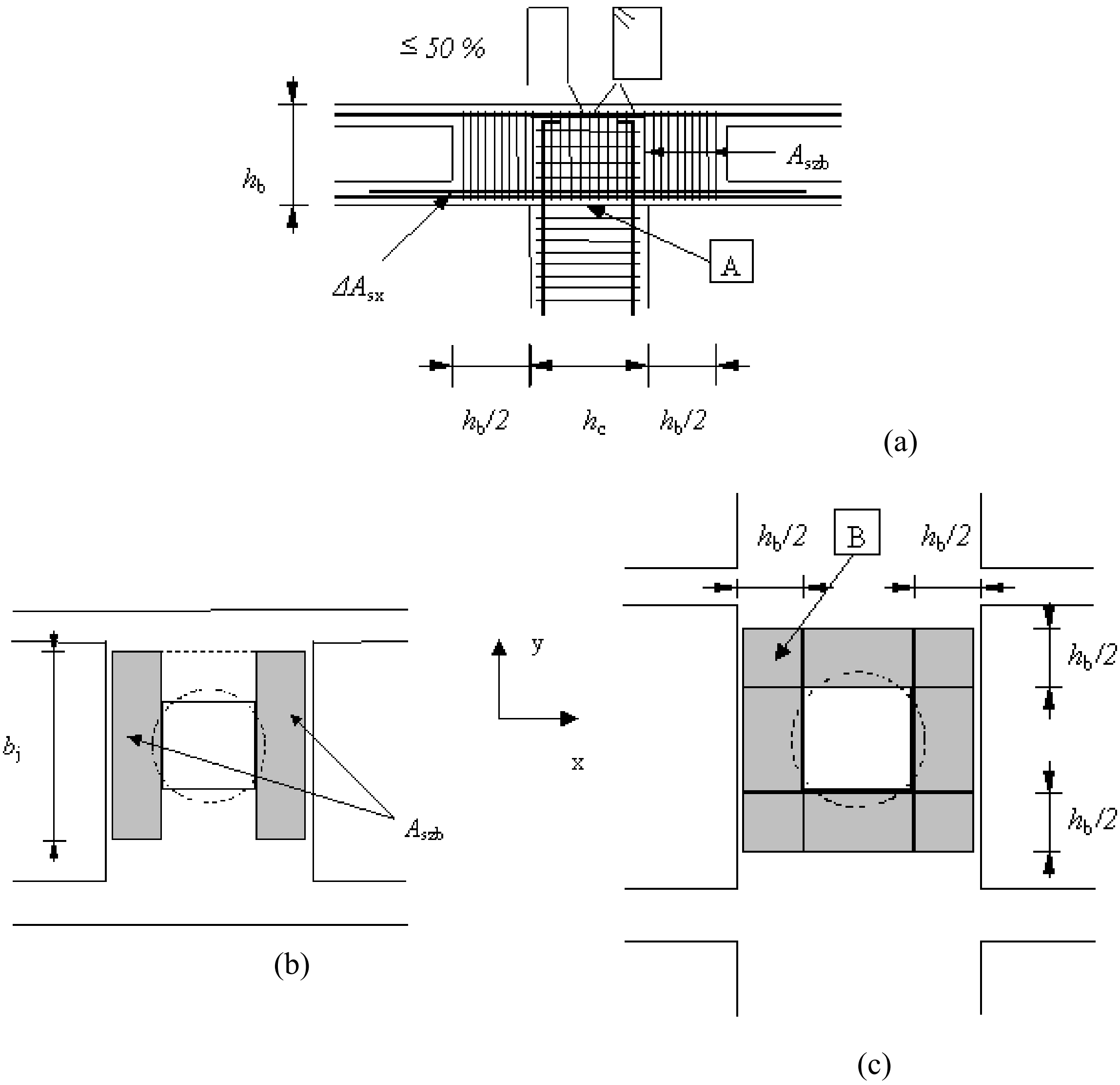
(5) When the amount of required reinforcement A_{sz} and/or A_{sx}, in accordance with expressions (5.24) and (5.23) is so high as to impair constructability of the joint, then the alternative arrangement, described in **(6)** and **(7)**, may be applied (see Figure 5.3).

(6) Vertical stirrups of amount $\rho_{1\mathrm{z}} \geq \rho_{\min}$, acceptable from the constructability point of view, may be placed within the joint body. The remaining area $A_{\mathrm{szb}} = (\rho_{\mathrm{z}} - \rho_{1\mathrm{z}})b_{\mathrm{j}}h_{\mathrm{b}}$, should be placed on each side of the “beam”, within the joint width b_{j} and not further than $0{,}5h_{\mathrm{b}}$ from the corresponding pier face.

(7) The horizontal stirrups, placed within the joint body, may be reduced by ΔA_{sx}, and the tensile reinforcement requirements of the “beam” fibres at the extension of the “beam”-pier interface, should be increased by:

$$\Delta A_{\mathrm{sx}} = 0{,}5\, \rho_{\mathrm{jz}}\, b_{\mathrm{j}}\, h_{\mathrm{b}} \tag{5.27}$$

in addition to the reinforcement required in the relevant sections for the verification in flexure under capacity design effects. Additional bars to cover this requirement should be placed within the joint width b_{j}; these bars should and be adequately anchored, so as to be fully effective at a distance h_{b} from the pier face.



Key
A : “Beam”-pier interface
B : Stirrups in common areas count in both directions

Figure 5.3: Alternative arrangement of joint reinforcement; (a) vertical section within plane xz; (b) plan view for plastic hinges forming in the x-direction; (c) plan view for plastic hinges in the x- and the y- directions.

5.6.3.6 Deck verification

(1)P It shall be verified that no significant yielding occurs in the deck. This verification shall be carried out:

– for bridges of limited ductile behaviour, under the most adverse design action effect in accordance with **5.5**;

– for bridges of ductile behaviour, under the capacity design effects determined in accordance with **5.3**.

(2) When the horizontal component of the seismic action in the transverse direction of the bridge is considered, yielding of the deck for flexure within a horizontal plane is considered to be significant if the reinforcement of the top slab of the deck yields up to a distance from its edge equal to 10% of the top slab width, or up to the junction of the top slab with a web, whichever is closer to the edge of the top slab.

(3) When verifying the deck on the basis of capacity design effects for the seismic action acting in the transverse direction of the bridge, the significant reduction of the torsional stiffness of the deck with increasing torsional moments should be accounted for. Unless a more accurate calculation is made, the values specified in **2.3.6.1(4)** may be assumed for bridges of limited ductile behaviour, or 70% of these values for bridges of ductile behaviour.

5.7 Resistance verification for steel and composite members

5.7.1 Steel piers

5.7.1.1 General

(1) For the verification of the pier under multi-component action effects, **5.6.1(1)** applies.

(2)P Energy dissipation is allowed to take place only in the piers and not in the deck.

(3)P For bridges designed for ductile behaviour, the provisions of EN 1998-1:2004, **6.5.2**, **6.5.4** and **6.5.5** for dissipative structures apply.

(4) The provisions of EN 1998-1:2004, **6.5.3** apply. However cross-sectional class 3 is allowed only when $q \leq 1{,}5$.

(5) The provisions of EN 1998-1:2004, **6.9** apply for all bridge piers.

5.7.1.2 Piers as moment resisting frames

(1)P In bridges designed for ductile behaviour, the design values of the axial force, N_{Ed}, and shear forces, $V_{E,d}$, in piers consisting of moment resisting frames shall be assumed to be equal to the capacity design action effects N_C and V_C, respectively, as the latter are specified in **5.3**.

(2)P The design of the sections of plastic hinges both in beams and columns of the pier shall satisfy the provisions of EN 1998-1:2004, **6.6.2**, **6.6.3** and **6.6.4**, using the values of N_{Ed} and V_{Ed} as specified in **(1)**P.

5.7.1.3 Piers as frames with concentric bracings

(1)P The provisions of EN 1998-1: 2004 apply with the following modifications for bridges designed for ductile behaviour.

- The design values for the axial shear force shall be in accordance with **5.3,** taking the force in all diagonals as corresponding to the overstrength $\gamma_o N_{pl,d}$ of the weakest diagonal (see **5.3** for γ_o).
- The second part of expression (6.12) in EN 1998-1:2004, **6.7.4** shall be replaced by the capacity design action $N_{Ed} = N_C$

5.7.1.4 Piers as frames with eccentric bracings

(1)P The provisions of EN 1998-1:2004, **6.8** apply.

5.7.2 Steel or composite deck

(1)P In bridges designed for ductile behaviour ($q > 1,5$) the deck shall be verified for the capacity design effects in accordance with **5.3**. In bridges designed for limited ductile behaviour ($q \leq 1,5$) the verification of the deck shall be carried out using the design action effects from the analysis in accordance with expression (5.4). The verifications may be carried out in accordance with the relevant rules of EN 1993-2:2005 or EN 1994-2:2005 for steel or composite decks, respectively.

5.8 Foundations

5.8.1 General

(1)P Bridge foundation systems shall be designed to conform to the general requirements set forth in EN 1998-5:2004, **5.1**. Bridge foundations shall not be intentionally used as sources of hysteretic energy dissipation and therefore shall, as far as practicable, be designed to remain elastic under the design seismic action.

(2)P Soil structure interaction shall be assessed where necessary on the basis of the relevant provisions of EN 1998-5: 2004, Section **6**.

5.8.2 Design action effects

(1)P For the purpose of resistance verifications, the design action effects on the foundations shall be determined in accordance with **(2)**P to **(4)**.

(2)P Bridges of limited ductile behaviour ($q \leq 1,5$) and bridges with seismic isolation

The design action effects shall be those resulting from expression (5.4) with seismic effects obtained from the linear analysis of the structure for the seismic design situation in accordance with **5.5**, with the analysis results for the design seismic action multiplied by the q-factor used (i.e. effectively using $q = 1$).

(3)P Bridges of ductile behaviour ($q > 1,5$).

The design action effects shall be obtained by applying the capacity design procedure to the piers in accordance with **5.3**.

(4) For bridges designed on the basis of non-linear analysis, the provisions of **4.2.4.4(2)e** apply.

5.8.3 Resistance verification

(1)P The resistance verification of the foundations shall be carried out in accordance with EN 1998-5:2004, **5.4.1** (Direct foundations) and **5.4.2** (Piles and piers).

6 DETAILING

6.1 General

(1)P The rules of this Section apply only to bridges designed for ductile behaviour and aim to ensure a minimum level of curvature/rotation ductility at the plastic hinges.

(2)P For bridges of limited ductile behaviour, rules for the detailing of critical sections and specific non-ductile components are specified in **6.5**.

(3)P In general, plastic hinge formation is not allowed in the deck. Therefore there is no need for the application of special detailing rules other than those applying for the design of bridges for the non-seismic actions.

6.2 Concrete piers

6.2.1 Confinement

6.2.1.1 General requirements

(1)P Ductile behaviour of the compression concrete zone shall be ensured within the potential plastic hinge regions.

(2)P In potential hinge regions where the normalised axial force (see **5.3(3)**) exceeds the limit:

$$\eta_k = N_{Ed}/A_c f_{ck} > 0{,}08 \tag{6.1}$$

confinement of the compression zone in accordance with **6.2.1.4** should be provided, except as specified in **(3)**.

(3)P No confinement is required in piers if, under ultimate limit state conditions, a curvature ductility μ_Φ = 13 for bridges of ductile behaviour, or μ_Φ = 7 for bridges of limited ductile behaviour, is attainable, with the maximum compressive strain in the concrete not exceeding the value of:

$$\varepsilon_{cu2} = 0{,}35\% \tag{6.2}$$

NOTE: The condition of **(3)**P may be attainable in piers with flanged section, when sufficient flange area is available in the compressive zone.

(4) In cases of deep compression zones, the confinement should extend at least up to the depth where the value of the compressive strain exceeds $0{,}5\varepsilon_{cu2}$

(5)P The quantity of confining reinforcement is defined through the mechanical reinforcement ratio:

$$\omega_{wd} = \rho_w f_{yd}/f_{cd} \tag{6.3}$$

where:

(a) In rectangular sections:

ρ_w is the transverse reinforcement ratio defined as:

$$\rho_w = \frac{A_{sw}}{s_L b} \tag{6.4}$$

where:

A_{sw} is the total area of hoops or ties in the one direction of confinement;

s_L is the spacing of hoops or ties in the longitudinal direction;

b is the dimension of the concrete core perpendicular to the direction of the confinement under consideration, measured to the outside of the perimeter hoop.

(b) In circular sections:

The volumetric ratio ρ_w of the spiral reinforcement relative to the concrete core is used:

$$\rho_w = \frac{4A_{sp}}{D_{sp} \cdot s_L} \tag{6.5}$$

where:

A_{sp} is the area of the spiral or hoop bar

D_{sp} is the diameter of the spiral or hoop bar

s_L is the spacing of these bars.

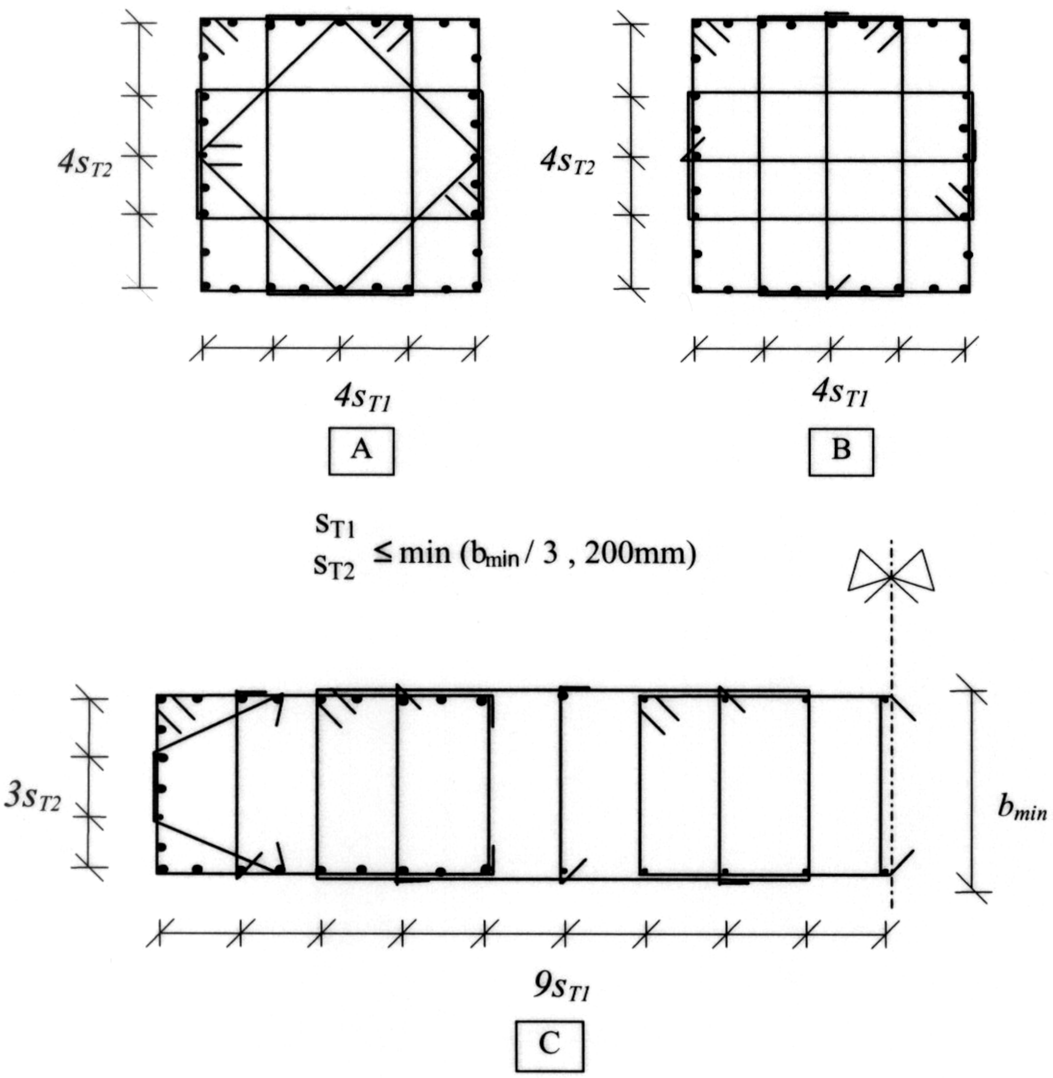
6.2.1.2 Rectangular sections

(1)P The spacing of hoops or ties in the longitudinal direction, s_L, shall satisfy both of the following conditions:

- $s_L \leq 6$ times the longitudinal bar diameter, d_{bL}
- $s_L \leq 1/5$ of the smallest dimension of the confined concrete core, to the hoop centre line.

(2)P The transverse distance s_T between hoop legs or supplementary cross-ties shall not exceed 1/3 of the smallest dimension b_{min} of the concrete core to the hoop centre line, nor 200mm (see Figure 6.1a).

(3)P Bars inclined at an angle $\alpha > 0$ to the transverse direction in which ρ_w refers to shall be assumed to contribute to the total area A_{sw} of expression (6.4) by their area multiplied by *cos α*.



Key
A : 4 closed overlapping hoops
B : 3 closed overlapping hoops plus cross-ties
C : closed overlapping hoops plus cross-ties

Figure 6.1a: Typical confinement details in concrete piers with rectangular section using overlapping rectangular hoops and cross-ties

6.2.1.3 Circular sections

(1)P The spacing of spiral or hoop bars, s_L, shall satisfy both of the following conditions:

$s_L \leq 6$ times the longitudinal bar diameter, d_{bL}

$s_L \leq 1/5$ of the diameter of the confined concrete core to the hoop centre line.

6.2.1.4 Required confining reinforcement

(1)P Confinement is implemented through rectangular hoops and/or cross-ties or through circular hoops or spirals.

NOTE The National Annex may prohibit the use of a certain type of confinement reinforcement. It is recommended that all types of confinement are allowed.

(2)P The minimum amount of confining reinforcement shall be determined as follows:

– for rectangular hoops and cross-ties

$$\omega_{\mathrm{wd,r}} \geq \max\left(\omega_{\mathrm{w,req}};\ \frac{2}{3}\omega_{\mathrm{w,min}}\right) \tag{6.6}$$

where:

$$\omega_{\mathrm{w,req}} = \frac{A_{\mathrm{c}}}{A_{\mathrm{cc}}}\lambda\eta_{\mathrm{k}} + 0{,}13\frac{f_{\mathrm{yd}}}{f_{\mathrm{cd}}}(\rho_{\mathrm{L}}\text{-}0{,}01) \tag{6.7}$$

where:

A_{c} is the area of the gross concrete section;

A_{cc} is the confined (core) concrete area of the section to the hoop centerline;

$\omega_{\mathrm{w,min}}$, λ are factors specified in Table 6.1; and

ρ_{L} is the reinforcement ratio of the longitudinal reinforcement.

Depending on the intended seismic behaviour of the bridge, the minimum values specified in Table 6.1 apply.

Table 6.1: Minimum values of λ and $\omega_{\mathrm{w,min}}$

Seismic Behaviour	λ	$\omega_{\mathrm{w,min}}$
Ductile	0,37	0,18
Limited ductile	0,28	0,12

– for circular hoops or spirals

$$\omega_{\mathrm{wd.c}} \geq \max\left(1{,}4\omega_{\mathrm{w,req}};\ \omega_{\mathrm{w,min}}\right) \tag{6.8}$$

(3)P When rectangular hoops and cross-ties are used, the minimum reinforcement condition shall be satisfied in both transverse directions.

(4)P Interlocking spirals/hoops are quite efficient for confining approximately rectangular sections. The distance between the centres of interlocking spirals/hoops shall not exceed $0{,}6D_{\mathrm{sp}}$, where D_{sp} is the diameter of the spiral/hoop (see Figure 6.1b).

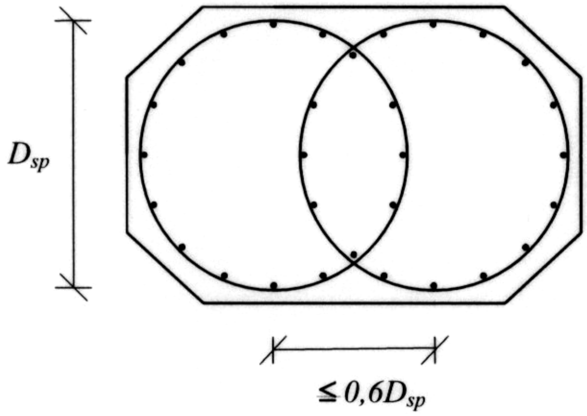


Figure 6.1b: Typical confinement detail in concrete piers using interlocking spirals/hoops

6.2.1.5 Extent of confinement - Length of potential plastic hinges

(1)P When $\eta_k = N_{Ed}/A_c f_{ck} \leq 0,3$ the design length L_h of potential plastic hinges shall be estimated as the largest of the following values:

- the depth of the pier section within the plane of bending (perpendicular to the axis of rotation of the hinge);
- the distance from the point of maximum moment to the point where the design moment is less than 80% of the value of the maximum moment.

(2)P When $0,6 \geq \eta_k > 0,3$ the design length of the potential plastic hinges as determined in **(1)**P shall be increased by 50%.

(3) The design length of plastic hinges (L_h) defined above should be used exclusively for detailing the reinforcement of the plastic hinge. It should not be used for estimating the plastic hinge rotation.

(4)P When confining reinforcement is required, the amount specified in **6.2.1.4** shall be provided over the entire length of the plastic hinge. Outside the length of the hinge the transverse reinforcement may be gradually reduced to the amount required by other criteria. The amount of transverse reinforcement provided over an additional length L_h adjacent to the theoretical end of the plastic hinge shall not be less than 50% of the amount of the confining reinforcement required in the plastic hinge.

6.2.2 Buckling of longitudinal compression reinforcement

(1)P Buckling of longitudinal reinforcement shall be avoided along potential hinge areas, even after several cycles into the post-yield region.

(2) To meet the requirement in **(1)**P, all main longitudinal bars should be restrained against outward buckling by transverse reinforcement (hoops or cross-ties)

perpendicular to the longitudinal bars at a (longitudinal) spacing s_L not exceeding δd_{bL}, where d_{bL} is the diameter of the longitudinal bars. Coefficient δ depends on the ratio f_t/f_y of the tensile strength f_{tk} to the yield strength f_{yk} of the transverse reinforcement, in terms of characteristic values, in accordance with the following relation:

$$5 \leq \delta = 2{,}5\,(f_{tk}/f_{yk}) + 2{,}25 \ \leq \ 6 \tag{6.9}$$

(3) Along straight section boundaries, restraining of longitudinal bars should be acheived in either one of the following ways:

a) through a perimeter tie engaged by intermediate cross-ties at alternate locations of longitudinal bars, at transverse (horizontal) spacing s_t not exceeding 200 mm. The cross-ties shall have 135°-hooks at one end and 135°-hooks or 90°-hook at the other. Cross-ties with 135°-hooks at both ends may consist of two lapped spliced pieces. If η_k > 0,30, 90°-hooks are not allowed for the cross-ties. If the cross-ties have dissimilar hooks at the two ends, these hooks should be alternated in adjacent cross-ties, both horizontally and vertically. In sections of large dimensions the perimeter tie may be spliced using appropriate lapping length combined with hooks;

b) through overlapping closed ties arranged so that every corner bar and at least every alternate internal longitudinal bar is engaged by a tie leg. The transverse (horizontal) spacing s_T of the tie legs should not exceed 200 mm.

(4)P The minimum amount of transverse ties shall be determined as follows:

$$\min\left(\frac{A_t}{s_T}\right) = \frac{\Sigma A_s f_{ys}}{1{,}6 f_{yt}} (mm^2/m) \tag{6.10}$$

where:

A_t is the area of one tie leg, in mm^2;

s_T is the transverse distance between tie legs, in m;

ΣA_s is the sum of the areas of the longitudinal bars restrained by the tie, in mm^2;

f_{yt} is the yield strength of the tie; and

f_{ys} is the yield strength of the longitudinal reinforcement.

6.2.3 Other rules

(1)P Due to the potential loss of concrete cover in the plastic hinge region, the confining reinforcement shall be anchored by 135°-hooks (unless a 90°-hook is used in accordance with **6.2.2(3)**a) surrounding a longitudinal bar plus adequate extension (min. 10 diameters) into the core concrete.

(2)P Similar anchoring or a full strength weld is required for the lapping of spirals or hoops within potential plastic hinge regions. In this case laps of successive spirals or hoops, when located along the perimeter of the member, should be staggered in accordance with EN 1992-1-1:2004, **8.7.2**.

(3)P No splicing by lapping or welding of longitudinal reinforcement is allowed within the plastic hinge region. For mechanical couplers see EN 1998-1:2004, **5.6.3(2)**.

6.2.4 Hollow piers

(1) The rules of **(2)** to **(4)** are not required in cases of low seismicity.

NOTE: For cases of low seismicity the Notes in **2.3.7(1)** apply.

(2) Unless appropriate justification is provided, the ratio b/h of the clear width b to the thickness h of the walls, in the plastic hinge region (length L_h in accordance with **6.2.1.5**) of hollow piers with a single or multiple box cross-section, should not exceed 8.

(3) For hollow cylindrical piers the limitation **(2)** applies to the ratio D_i/h, where D_i is the inside diameter.

(4) In piers with simple or multiple box section and when the value of the ratio η_k defined in expression (6.1) does not exceed 0,20, there is no need for verification of the confining reinforcement in accordance with **6.2.1**, provided that the requirements of **6.2.2** are met.

6.3 Steel piers

(1)P For bridges designed for ductile behaviour, the detailing rules of EN 1998-1:2004, **6.5**, **6.6**, **6.7** and **6.8**, as modified by **5.7** of the present Part, shall be applied.

6.4 Foundations

6.4.1 Spread foundation

(1)P Spread foundations such as footings, rafts, box-type caissons, piers etc., shall not enter the plastic range under the design seismic action, and hence do not require special detailing reinforcement.

6.4.2 Pile foundations

(1)P When it is not feasible to avoid localised hinging in the piles, using the capacity design procedure (see **5.3**), pile integrity and ductile behaviour shall be ensured. For this case following rules apply.

(2) The following locations along the pile should be detailed as potential plastic hinges.

(a) At the pile heads adjacent to the pile cap, when the rotation of the pile cap about a horizontal axis transverse to the seismic action is restrained by the large stiffness of the pile group in this degree-of-freedom.

(b) At the depth where the maximum bending moment develops in the pile. This depth should be estimated by an analysis that takes into account the effective pile flexural stiffness (see **2.3.6.1**), the lateral soil stiffness and the rotational stiffness of the pile group at the pile cap.

(c) At the interfaces of soil layers with markedly different shear deformability, due to kinematic pile-soil interaction (see EN 1998-5:2004, **5.4.2(1)**P).

(3) At locations of type (a) in **(2),** confining reinforcement of the amount specified in **6.2.1.4** along a vertical length equal to 3 times the pile diameter, should be provided.

(4) Unless a more accurate analysis is made, , , longitudinal as well as confining reinforcement of the same amount as that required at the pile head shall be provided over a length of two pile diameters on each side of the point of maximum moment at locations of type (b) in **(2)**, and of each side of the interface at locations of type (c) in **(2)**.

6.5 Structures of limited ductile behaviour

6.5.1 Verification of ductility of critical sections

(1)P The following rules apply at the critical sections of structures designed for limited ductile behaviour (with $q \leq 1{,}5$) in cases other than those of low seismicity, to ensure a minimum of limited ductility.

NOTE 1: For the definition of cases of low seismicity see Note 1 in **2.3.7(1).**

NOTE 2: The National Annex may define simplified verification rules for bridges designed for limited ductile behaviour in low seismicity cases. It is recommended to apply the same rules as in cases other than those of low seismicity.

(2)P A section is considered to be critical, i.e. location of a potential plastic hinge, when:

$$M_{\mathrm{Rd}} / M_{\mathrm{Ed}} < 1{,}30 \tag{6.11}$$

where:

M_{Ed} is the maximum design moment at the section in the seismic design situation, and

M_{Rd} is the minimum flexural resistance of the section in the seismic design situation.

(3) As far as possible, the location of potential plastic hinges should be accessible for inspection.

(4)P Unless confinement is not necessary according to **6.2.1.1(3)**P, confining reinforcement as required by **6.2.1.4** for limited ductility (see Table 6.1), shall be provided in concrete members. In such cases it is also required to secure the longitudinal reinforcement against buckling in accordance with **6.2.2**.

6.5.2 Avoidance of brittle failure of specific non-ductile components

(1)P Non-ductile structural components, such as fixed bearings, sockets and anchorages for cables and stays and other non-ductile connections shall be designed using either seismic action effects multiplied by the q-factor used in the analysis, or capacity design effects. The latter shall be determined from the strength of the relevant ductile members (e.g. the cables) and an overstrength factor of at least 1,3.

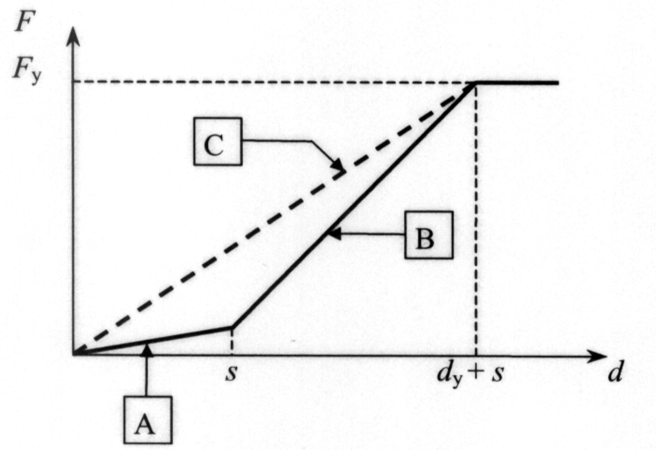
(2)P This verification may be omitted if it can be demonstrated that the integrity of the structure is not affected by failure of such connections. This demonstration shall also address the possibility of sequential failure, such as may occur in stays of cable-stayed bridges.

6.6 Bearings and seismic links

6.6.1 General requirements

(1)P Non-seismic horizontal actions on the deck shall be transmitted to the supporting members (abutments or piers) through the structural connections, which may be monolithic, or through bearings. For non-seismic actions the bearings shall be verified in accordance with the relevant standards (Parts 2 of the relevant Eurocodes and EN 1337).

(2)P In general the design seismic action shall be transmitted through the bearings. However, seismic links (as specified in **6.6.3**) may be used to transmit the entire design seismic action, provided that dynamic shock effects are mitigated and taken into account in the design. Seismic links should generally allow the non-seismic displacements of the bridge to develop, without transmitting significant loads. When seismic links are used, the connection between the deck and the substructure should be properly modelled. As a minimum, a linear approximation of the force-displacement relationship of the linked structure shall be used (see Figure 6.2).



Key

s Slack of the link
d_y Yield deflection of supporting element
A : Stiffness of bearing
B : Stiffness of supporting element
C : Linear approximation of the curve

Figure 6.2: Force-displacement relationship for linked structure

NOTE: Certain types of seismic links may not be applicable to bridges subject to large horizontal non-seismic actions, or to bridges with special displacement limitations, as for instance in railway bridges.

(3)P The structural integrity of the bridge shall be ensured under extreme seismic displacements. At fixed supports this requirement shall be implemented either through capacity design of the normal bearings (see **6.6.2.1**), or through provision of additional links as a second line of defence (see **6.6.2.1(2)** and **6.6.3.1(2)**(b). At moveable connections adequate overlap (seat) lengths in accordance with **6.6.4** shall be provided. In cases of retrofitting of existing bridge seismic links may be used as an alternative.

(4)P All types of bearings and seismic links shall be accessible for inspection and maintenance and shall be replaceable without major difficulty.

6.6.2 Bearings

6.6.2.1 Fixed bearings

(1)P Except under the conditions of **(2)**, the design seismic action effects on fixed bearings shall be determined through capacity design.

(2) Fixed bearings may be designed solely for the effects of the seismic design situation from the analysis, provided that they can be replaced without difficulties and that seismic links are provided as a second line of defence.

6.6.2.2 Moveable bearings

(1)P Moveable bearings shall accommodate without damage the total design value of the displacement in the seismic design situation determined in accordance with **2.3.6.3(2)**.

6.6.2.3 Elastomeric bearings

(1) Elastomeric bearings may be used in the following arrangements:

a. on individual supports, to accommodate imposed deformations and resist only non-seismic horizontal actions, while the resistance to the design seismic action is provided by structural connections (monolithic or through fixed bearings) of the deck to other supporting members (piers or abutments);

b. on all or on individual supports, with the same function as in (a) above, combined with seismic links which are designed to resist the seismic action;

c. on all supports, to resist both the non-seismic and the seismic actions.

(2) Elastomeric bearings used in arrangements (a) and (b) of **(1)** shall be designed to resist the maximum shear deformation due to the design seismic action in accordance with **7.6.2(5)**.

(3) Under the conditions specified in **2.2.2(5)**, significant damage of elastomeric bearings of **(2)** is acceptable.

NOTE: The National Annex may define the extent of damage and the relevant verifications.

(4) The seismic behaviour of bridges, in which the design seismic action is resisted entirely by elastomeric bearings on all supports (arrangement **(1)c** above), is governed by the large flexibility of the bearings. Such bridges and the bearings shall be designed in accordance with Section **7**.

6.6.3 Seismic links, holding-down devices, shock transmission units

6.6.3.1 Seismic links

(1) Seismic links may consist of shear key arrangements, buffers, and/or linkage bolts or cables. Friction connections are not considered as positive linkage.

(2) Seismic links are required in the following cases.

(a) In combination with elastomeric bearings, where the links are designed to carry the design seismic action.

(b) In combination with fixed bearings not designed for capacity design effects.

(c) In the longitudinal direction at moveable end-supports between the deck and the abutment or pier of existing bridges being retrofitted, if the requirements for minimum overlap length in **6.6.4** are not met.

(d) Between adjacent sections of the deck at intermediate separation joints (located within the span).

(3)P The design actions for the seismic links of the previous paragraph shall be determined as follows.

– In cases (a), (b) and (c) of **(2)** as capacity design effects (the horizontal resistance of the bearings shall be assumed to be equal to zero).

– In the case of (d) of **(2)**, and unless a more accurate analysis is made taking into account the dynamic interaction of adjacent sections of the deck, the linkage elements may be designed for an action equal to $1{,}5\alpha_g SM_d$ where α_g is the design ground acceleration on type A ground, S is the soil factor from EN 1998-1: 2004, **3.2.2.2** and M_d is the mass of the section of the deck linked to a pier or abutment, or the least of the masses of the two deck sections on either side of the intermediate separation joint.

(4)P The links shall be provided with adequate slack or margins, so as to remain inactive:

– under the design seismic action in cases (c) and (d) of **(2)**

– under any non-seismic actions in case (a) of **(2)**.

(5) When using seismic links, means for reducing shock effects should be provided.

6.6.3.2 Holding-down devices

(1)P Holding down devices shall be provided at all supports where the total vertical reaction due to the design seismic action opposes and exceeds a percentage, p_H, of the compressive (downward) reaction due to the permanent load.

NOTE The value ascribed to p_H for use in a country may be found in its National Annex. The recommended value are as follows:

- p_H = 80% in bridges of ductile behaviour, where the vertical reaction due to the design seismic action is determined as a capacity design effect.
- p_H = 50% in bridges of limited ductile behaviour, where the vertical reaction due to the design seismic action is determined from the analysis under the design seismic action alone (including the contribution of the vertical seismic component).

(2) The requirement **(1)** refers to the total vertical reaction of the deck on a support and does not apply to individual bearings of the same support. However, no up-lift of individual bearings may take place in the seismic design situation in accordance with **5.5**.

6.6.3.3 Shock transmission units (STUs)

(1) Shock transmission units (STUs) are devices which provide velocity-dependent restraint of the relative displacement between the deck and the supporting element (pier or abutment), as follows.

- For low velocity movements ($v < v_1$), such as those due to temperature effects or creep and shrinkage of the deck, the movement is practically free (with very low reaction).
- For high velocity movements ($v > v_2$), such as those due to seismic or braking actions, the movement is blocked and the device acts practically as rigid connection.
- The units can also have a force limiting function, that limits the force transmitted through it (for $v > v_2$) to a defined upper bound, F_{max}, beyond which movement takes place.

NOTE The properties and the design of STUs will be covered by prEN 15129:200X (Antiseismic Devices). The order of magnitude of the velocities mentioned above is $v_1 \cong 0,1$ mm/s, $v_2 \cong 1,0$ mm/s.

(2)P Full description of the laws defining the behaviour of the units used (force-displacement and force-velocity relationships) shall be available at the design stage (from the manufacturer of the units), including any influence of environmental factors (mainly temperature, ageing, cumulative travel) on this behaviour. All values of parameters necessary for the definition of the behaviour of the units (including the values of v_1, v_2, F_{max}, for the cases mentioned in **(1)**), as well as the geometric data and design resistance F_{Rd} of the units and their connections, shall also be available. Such information shall be based on appropriate official test results, or an ETA.

(3)P When STUs without force limiting function are used to resist seismic forces, they shall have a design resistance, F_{Rd}, as follows.

- For ductile bridges: F_{Rd} should be not less than the reaction corresponding to the capacity design effects,
- For limited ductile bridges: F_{Rd} should be not less than the reaction due to the design seismic action from the analysis, multiplied by the q-factor used.

The devices shall provide sufficient displacement capability for all slow velocity actions and shall retain their force capacity at their displaced state.

(4)P When STUs with force limiting function are used to resist seismic forces, the devices shall provide sufficient displacement capability to accommodate the total design value of the relative displacement, d_{Ed}, in the seismic design situation determined in accordance with **2.3.6.3(2)**P**,** or in accordance with **7.6.2(2)** for bridges with seismic isolation.

(5)P All STUs shall be accessible for inspection and maintenance/replacement.

6.6.4 Minimum overlap lengths

(1)P At supports where relative displacement between supported and supporting members is intended under seismic conditions, a minimum overlap length shall be provided.

(2)P The overlap length shall be such as to ensure that the function of the support is maintained under extreme seismic displacements.

(3) At an end support of an abutment the minimum overlap length l_{ov} may be estimated as follows:

$$l_{ov} = l_m + d_{eg} + d_{es} \tag{6.12}$$

$$d_{eg} = \varepsilon_e L_{eff} \leq 2d_g \tag{6.13}$$

$$\varepsilon_e = \frac{2d_g}{L_g} \tag{6.14}$$

where:

l_m is the minimum support length ensuring the safe transmission of the vertical reaction, but no less than 400 mm,

d_{eg} is the effective displacement of the two parts due to the spatial variation of the seismic ground displacement. When the bridge site is at a distance less than 5km of a known seismically active fault, capable of producing a seismic event of magnitude $M \geq 6.5$, and unless a specific seismological investigation is available, the value of d_{eg} to be used should be taken as double that obtained from expression (6.13).

d_g is the design ground displacement in accordance with EN 1998-1:2004, **3.2.2.4**,

L_g is the distance parameter specified in **3.3(6)**.

L_{eff} is the effective length of the deck, taken as the distance from the deck joint in question to the nearest full connection of the deck to the substructure. If the deck is fully connected to a group of more than one piers, then L_{eff} shall be taken as the distance between the support and the centre of the group of piers. In this context “full connection” means a connection of the deck or deck section to a substructure member, either monolithically or through fixed bearings, seismic links, or STUs, without force limiting function.

d_{es} is the effective seismic displacement of the support due to the deformation of the structure, estimated as follows.

– For decks connected to piers either monolithically or through fixed bearings acting as full seismic links:

$$d_{\mathrm{es}} = d_{\mathrm{Ed}}, \tag{6.15a}$$

where d_{Ed} is the total design value of the longitudinal displacement in the seismic design situation determined in accordance with expression (2.7) in **2.3.6.3**.

– For decks connected to piers or to an abutment through seismic links with slack equal to s:

$$d_{\mathrm{es}} = d_{\mathrm{Ed}} + s \tag{6.15b}$$

(4) In the case of an intermediate separation joint between two sections of the deck, l_{ov} should be estimated by taking the square root of the sum of the squares of the values calculated for each of the two sections of the deck in accordance with **(3)**. At an end support of a deck section on an intermediate pier, l_{ov} should be taken as the value estimated in accordance with **(3)** plus the maximum displacement of the top of the pier in the seismic design situation, d_{E}.

6.7 Concrete abutments and retaining walls

6.7.1 General requirements

(1)P All critical structural components of the abutments shall be designed to remain essentially elastic under the design seismic action. The design of the foundation shall be in accordance with **5.8**. Depending on the structural function of the horizontal connection between the abutment and the deck the provisions of **6.7.2** and **6.7.3** apply.

NOTE: Regarding controlled damage in abutment back-walls see **2.3.6.3(5)**.

6.7.2 Abutments flexibly connected to the deck

(1) In abutments flexibly connected to the deck, the deck is supported through sliding or elastomeric bearings. The elastomeric bearings (or the seismic links, if provided) may be designed to contribute to the seismic resistance of the deck, but not to that of the abutments.

(2) The following actions, assumed to act in phase, should be taken into account for the seismic design of these abutments.

a. Earth pressures including seismic effects determined in accordance with EN 1998-5:2004, Section **7**.

b. Inertia forces acting on the mass of the abutment and on the mass of earthfill lying over its foundation. In general these effects may be determined on the basis of the design ground acceleration at the top of the ground of the site, $a_g S$.

c. Actions from the bearings determined as capacity design effects in accordance with **5.3(7)**P and **5.3(8)**P if a ductile behaviour has been assumed for the bridge. If the bridge is designed for q = 1,0, then the reactions on the bearings resulting from the seismic analysis shall be used.

(3) When the earth pressures assumed in **(2)**a are determined in accordance with EN 1998-5:2004, on the basis of an acceptable displacement of the abutment, provision for this displacement should be made in determining the gap between the deck and the abutment back-wall. In this case it should also be ensured that the displacement assumed in determining the actions in **(2)**a, can actually take place before a potential failure of the abutment itself occurs. This requirement is deemed to be satisfied if the design of the body of the abutment is effected using the seismic part of the actions in **(2)**a increased by 30%.

6.7.3 Abutments rigidly connected to the deck

(1) The connection of the abutment to the deck is considered as rigid, if it is either monolithic, or through fixed bearings, or through links designed to carry the seismic action. Such abutments have a major contribution to the seismic resistance, both in the longitudinal and in the transverse direction.

(2) The analysis model should incorporate the effect of interaction of the soil and the abutments, using either best-estimate values of the relevant soil stiffness parameters or values corresponding to upper and lower bound stiffness.

(3) When the seismic resistance of the bridge is provided by both piers and abutments, the use of upper and lower bound estimates of the soil stiffness is recommended, in order to arrive at results which are on the safe side both for the abutments and for the piers.

(4)P A behaviour factor q = 1,5 shall be used, in the analysis of the bridge.

(5) The following actions should be taken into account in the longitudinal direction.

a. Inertia forces acting on the mass of the structure, which may be estimated using the Fundamental Mode Method (see **4.2.2**).

b. Static earth pressures acting on both abutments (E_o).

c. The additional seismic earth pressures

$$\Delta E_d = E_d - E_o \tag{6.16}$$

where:

E_d is the total earth pressure acting on the abutment under the design seismic action in accordance with EN 1998-5:2004. The pressures ΔE_d are assumed to act in the same direction on both abutments.

(6) The connection of the deck to the abutment (including fixed bearings or links, if provided) should be designed for the action effects resulting from the above paragraphs. Reactions on the passive side may be taken into account in accordance with **(8)**.

(7) In order that damage of the soil or the embankment behind an abutment rigidly connected to the deck is kept within acceptable limits, the design seismic displacement should not exceed a limit value, d_{lim}, depending on the importance class of the bridge.

NOTE: The value ascribed to d_{lim} for use in a country may be found in its National Annex. The recommended values of d_{lim} are as follows:

Table 6.2N. Recommended limit value of design seismic displacement at abutments rigidly connected to the deck

Bridge Importance Class	Displacement Limit d_{lim} (mm)
III	30
II	60
I	No limitation

(8) The soil reaction activated by the movement of the abutment, and of any wing-walls monolithically connected to it, towards the fill is assumed to act on the following surfaces.
- In the longitudinal direction, on the external face of the back-wall of that abutment which moves against the soil or fill.
- In the transverse direction, on the internal face of those wing-walls which move against the fill.

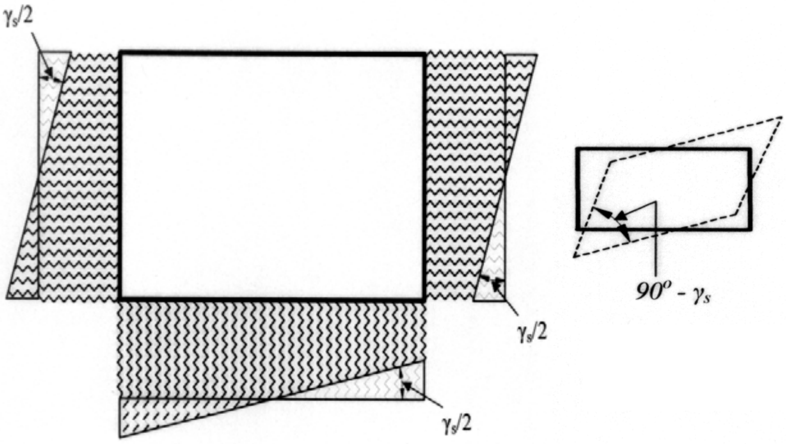
These reactions may be estimated on the basis of horizontal soil moduli corresponding to the specific geotechnical conditions.

The relevant abutment should be designed to resist this soil reaction, in addition to the static earth pressures.

(9) When an abutment is embedded in stiff natural soil formations over more than 80% of its height, it can be considered as fully locked-in. In that case $q = 1$ should be used and the inertia forces should be determined on the basis of the design ground acceleration at the top of the ground of the site, $a_g S$ (that is without spectral amplification).

6.7.4 Culverts with large overburden

(1) In culverts with a large depth of fill over the top slab (exceeding 50% of its span), the assumptions of inertial seismic response used in **6.7.3** may not be applied, as they lead to unrealistic results. In such a case the inertial response should be neglected and the response should be calculated on the basis of kinematic compatibility between the culvert structure and free-field seismic deformation of the surrounding soil corresponding to the design seismic action.



Key

γ_S: Free-field soil deformation

Figure 6.3: Kinematic response of culvert

(2) To this end the free-field seismic soil deformation may be assumed as a uniform shear-strain field (see Figure 6.3) with shear strain:

$$\gamma_s = \frac{v_g}{v_s} \tag{6.17}$$

where

v_g is the peak ground velocity (see **(3)** below)

v_s is the shear wave velocity in the soil under the shear strain corresponding to the ground acceleration. This value may be estimated from the value $v_{s,max}$ for small strains, from EN 1998-5:2004, Table 4.1.

(3) In the absence of specific data, the peak ground velocity should be estimated from the design ground acceleration a_g on type A ground, using the relation

$$v_g = \frac{ST_C a_g}{2\pi} \tag{6.18}$$

where S and T_C are in accordance with EN 1998-1:2004, **3.2.2.2**.

6.7.5 Retaining walls

(1)P Free standing retaining walls shall be designed in accordance with **6.7.2(2)** and **(3)**, without any action from bearings.

7 BRIDGES WITH SEISMIC ISOLATION

7.1 General

(1)P This Section covers the design of bridges that are provided with a special isolating system, aiming to reduce their response due to horizontal seismic action. The isolating units are arranged over the isolation interface, usually located under the deck and over the top of the piers/abutments.

(2) The reduction of the response may be achieved:

- by lengthening of the fundamental period of the structure (effect of period shift in the response spectrum), which reduces forces but increases displacements;
- by increasing the damping, which reduces displacements and may reduce forces;
- (preferably) by a combination of the two effects.

7.2 Definitions

isolating system
collection of components used for providing seismic isolation, located at the isolation interface

isolator units or isolators
the individual components, constituting the isolation system. Each unit provides a single or a combination of the following functions:

- vertical-load carrying capability, combined with high lateral flexibility and high vertical rigidity;
- energy dissipation (hysteretic, viscous, frictional);
- lateral restoring capability;
- horizontal restraint (sufficient elastic stiffness) under non-seismic service horizontal loads

substructure(s)
part(s) of the structure located under the isolation interface, usually consisting of the piers and abutments. The horizontal flexibility of the substructures should in general be accounted for.

superstructure
part of the structure located above the isolation interface. In bridges this part is usually the deck

effective stiffness centre
stiffness centre C at the top of the isolation interface, considering the superstructure as rigid, but accounting for the flexibilities of the isolator units and of the substructure(s)

design displacement (d_{cd}) of the isolating system in a principal direction

maximum horizontal displacement (relative to the ground) of the superstructure at the stiffness centre, occurring under the design seismic action

design displacement (d_{bi}) of an isolator *i*
displacement of the superstructure relative to the substructure at the location of the isolator, corresponding to the design displacement of the isolating system

increased design displacement ($d_{bi,a}$) of isolator *i*
design displacement of the isolator, multiplied by the amplification factor γ_{IS} in accordance with **7.6.2**

maximum total displacement of isolator unit *i*
sum of the increased design displacement of the isolator and the offset displacements induced by permanent actions

effective stiffness of the isolating system in a principal direction
ratio of the value of the total horizontal force transferred through the isolation interface, concurrent to the design displacement in the same direction, divided by the absolute value of the design displacement (secant stiffness).

effective period
fundamental period in the direction considered, of a single-degree-of-freedom system having the mass of the superstructure and stiffness equal to the effective stiffness of the isolating system, as specified in **7.5.4**

effective damping of the isolating system
value of viscous damping ratio, corresponding to the energy dissipated by the isolation system during cyclic response at the design displacement

simple low-damping elastomeric bearings
laminated low-damping elastomeric bearings in accordance with EN 1337-3:1996, not subject to prEN 15129:200X (Antiseismic Devices) (see **7.5.2.3.3(5)**)

special elastomeric bearings
laminated high damping elastomeric bearings successfully tested in accordance with the requirements of prEN 15129:200X (Antiseismic Devices) (see **7.5.2.3.3(7)**).

7.3 Basic requirements and compliance criteria

(1)P The basic requirements set forth in **2.2** shall be satisfied.

(2)P The seismic response of the superstructure and substructures under the design seismic design situations shall be assumed as limited ductile ($q \leq 1,5$).

(3) The bridge is deemed to satisfy the basic requirements, if it is designed in accordance with **7.4** and **7.5** and conforms to **7.6** and **7.7**.

(4)P Increased reliability is required for the strength and integrity of the isolating system, due to the critical role of its displacement capability for the safety of the bridge. This reliability is deemed to be achieved if the isolating system is designed in accordance with the requirements of **7.6.2**.

(5)P For all types of isolator units, with the exception of simple elastomeric bearings in accordance with **7.5.2.3.3(5)** and **(6)** and the flat sliding bearings in accordance with **7.5.2.3.5(5)**, the design properties shall be validated on the basis of Qualification and Prototype tests.

NOTE Informative annex **K** is intended to provide guidance on prototype testing in cases where prEN 15129:200X ("Anti-seismic devices") does not include detailed requirements for type testing

7.4 Seismic action

7.4.1 Design spectra

(1)P The spectrum used shall be not lower than the design response spectrum specified in EN 1998-1:2004, **3.2.2.5** for non-isolated structures (see EN 1998-1:2004, **3.2.2.5(8)**P).

NOTE Particular attention should be given to the fact that the safety of structures with seismic isolation depends mainly on the displacement demands for the isolating system that are directly proportional to the value of period T_D. Therefore, and in accordance with EN 1998-1:2004, **3.2.2.5(8)**P, the National Annex to this Part of Eurocode 8 may specify a value of T_D specifically for the design of bridges with seismic isolation that is more conservative (longer) than the value ascribed to T_D in the National Annex to EN 1998-1 :2004 (see also **3.2.2.3**).

7.4.2 Time-history representation

(1)P The provisions of **3.2.3** apply.

7.5 Analysis procedures and modelling

7.5.1 General

(1) The following analysis procedures, with conditions for application specified in **7.5.3**, are provided for bridges with seismic isolation.

a) Fundamental mode spectrum analysis

b) Multi-mode spectrum analysis

c) Time-history non-linear analysis

(2)P In addition to the conditions specified in **7.5.3**, the following are prerequisites for the application of methods (a) and (b) in **(1)**

- The usually non-linear force–displacement relationship of the isolating system shall be approximated with sufficient accuracy by the effective stiffness (K_{eff}), i.e. the secant value of the stiffness at the design displacement (see Figure 7.1). This representation shall be based on successive approximations of the design displacement (d_{cd}).

– The energy dissipation of the isolating system shall be expressed in terms of an equivalent viscous damping as the “effective damping” (ξ_{eff}).

(3) If the isolating system consists exclusively of simple low damping elastomeric bearings (equivalent viscous damping ratio approximately 0,05), the normal linear dynamic analysis methods specified in **4.2** may be applied. The elastomeric bearings may be considered as linear elastic members, deforming in shear (and possibly in compression). Their damping may be assumed equal to the global viscous damping of the structure (see also **7.5.2.3.3(2)**). The entire structure should remain essentially elastic.

7.5.2 Design properties of the isolating system

7.5.2.1 General

(1)P All isolators shall conform to EN pr15129:200X (Antiseismic Devices) or be covered by an ETA (European Technical Approval).

NOTE 1: prEN 15129:200X: Antiseismic Devices is being prepared by CEN/TC340. Until this EN is published by CEN, as well as for the case of isolators whose Prototype tests are not fully covered by this latter EN, the requirements given in Informative Annex **K** of the present standard may be used.

NOTE 2: Regarding simple elastomeric bearings in accordance with **7.5.2.3.3(4)**, **(5)** and **(6)** and lubricated PTFE (polytetrafluorethylene) flat sliding bearings used in accordance with **7.5.2.3.5(5)** see references above as well as **7.5.2.4 (5)**, **(6)** and **(7)**.

7.5.2.2 Stiffness in vertical direction

(1)P The isolator units that carry vertical loads shall be sufficiently stiff in the vertical direction.

(2) The requirement in **(1)**P is deemed to be satisfied if the horizontal displacement at the centre of mass of the superstructure, due to the vertical flexibility of the isolator units, is less than 5% of the design displacement d_{cd}. This condition need not be checked if sliding or normal laminated elastomeric bearings are used as vertical load carrying elements at the isolation interface.

7.5.2.3 Design properties in horizontal directions

7.5.2.3.1 General

(1) The design properties of the isolators depend on their behaviour, which may be one or a combination of those described in subclauses **7.5.2.3.2** to **7.5.2.3.5**.

7.5.2.3.2 Hysteretic behaviour

(1) The force-displacement relationship of the isolator unit in the horizontal direction may be approximated by a bi-linear relationship, as shown in Figure 7.1, for an isolator unit *i* (index *i* is omitted).

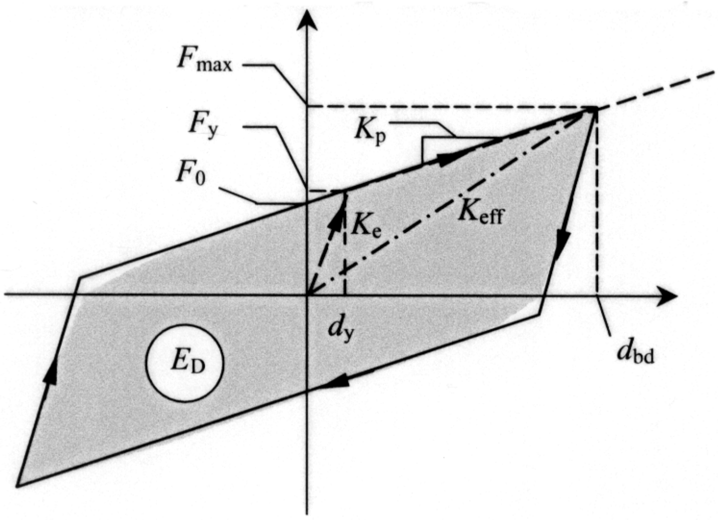


Figure 7.1: Bilinear approximation of hysteretic force-displacement behaviour

(2) The parameters of the bi-linear approximation are the following:

d_y = yield displacement;

d_{bd} = design displacement of the isolator corresponding to the design displacement d_{cd} of the isolating system;

E_D = dissipated energy per cycle at the design displacement d_{bd}, equal to the area enclosed by the actual hysteresis loop = $4(F_y d_{bd} - F_{max} d_y)$;

F_y = yield force under monotonic loading;

F_0 = force at zero displacement under cyclic loading = $F_y - K_p\, d_y$;

F_{max} = maximum force, corresponding to the design displacement d_{bd};

K_e = elastic stiffness at monotonic loading = F_y/d_y , equal also to the unloading stiffness in cyclic loading;

K_p = post-elastic (tangent) stiffness = $(F_{max} - F_y)/(d_{bd} - d_y)$.

7.5.2.3.3 Behaviour of elastomeric bearings

(1) Elastomeric bearings considered in this Part are laminated rubber bearings consisting of rubber layers reinforced by integrally bonded steel plates. With regard to damping, elastomeric bearings are distinguished in low damping and high damping bearings.

(2) Low damping elastomeric bearings are those with an equivalent viscous damping ratio ξ less than 0,06. Such bearings have a cyclic behaviour similar to hysteretic behaviour with very slender hysteresis loops. Their behaviour should be approximated by that of a linear elastic member with equivalent elastic stiffness in the horizontal direction equal to $G_b A_b / t_e$ where G_b is the shear modulus of the elastomer (see **7.5.2.4(5)**), A_b its effective horizontal area and t_e is the total thickness of the elastomer.

(3) High damping elastomeric bearings exhibit substantial hysteresis loops, corresponding to an equivalent viscous damping ratio ξ usually between 0,10 and 0,20. Their behaviour should be considered as linear hysteretic.

(4) From the point of view of required special tests for seismic performance, elastomeric bearings are distinguished in this part as simple low-damping and special elastomeric bearings.

(5) Low damping bearings conforming to EN 1337-3:1996 are defined as simple low-damping elastomeric bearings.

(6) Simple low-damping elastomeric bearings may be used as isolators, without being subjected to special tests for seismic performance.

(7) Special elastomeric bearings are high damping elastomeric bearings specially tested in accordance with the requirements of EN pr15129:200X (Antiseismic Devices).

(8) The design properties of elastomeric bearings used in this Section should cover both the unscragged and the scragged conditions of the bearings.

NOTE Scragging is exhibited by elastomeric bearings if they have been previously (i.e. before testing) subjected to one or more cycles of high shear deformation. Scragged bearings show a significant drop of the shear stiffness in subsequent cycles. It appears however that the original (virgin) shear stiffness of the bearings is practically recovered after a certain time (a few months). This effect is prominent mainly in high damping and in low shear modulus bearings and should be accounted for by using an appropriate range of design parameters (see **K.2.1** and **K.2.3.3** R4).

(9) Lead Rubber Bearings (LRB) consist of low damping elastomeric bearings with a cylindrical lead core. Yielding of the lead core provides such devices with substantial hysteretic behaviour. This hysteretic behaviour may be represented by the bilinear approximation shown in Figure 7.1 with the following parameters:

– Elastic stiffness: $K_e = K_L + K_R$

  where K_R and K_L are the shear stiffnesses of the elastomeric and lead parts of the device, respectively;

– Post-elastic stiffness: $K_p = K_R$;

– Yield force: $F_y = F_{Ly}\ (1+K_R/K_L)$

where F_{Ly} is the yield force of the lead core.

NOTE 1: When $K_R << K_L$, then $K_e \cong K_L$ and $F_y \cong F_{Ly}$

NOTE 2: LRBs should be in accordance with EN pr15129:200X: Antiseismic Devices.

7.5.2.3.4 Fluid viscous dampers

(1) The reaction of fluid viscous dampers is proportional to v^{α_b}, where

$v = \dot{d}_b = \frac{d}{dt}(d_b)$ is the velocity of motion. This reaction is zero at the maximum displacement $d_{max} = d_{bd}$ and therefore does not contribute to the effective stiffness of the isolating system. The force-displacement relationship of a fluid viscous damper is shown in Figure7.2 (for sinusoidal motion), depending on the value of the exponent α_b.

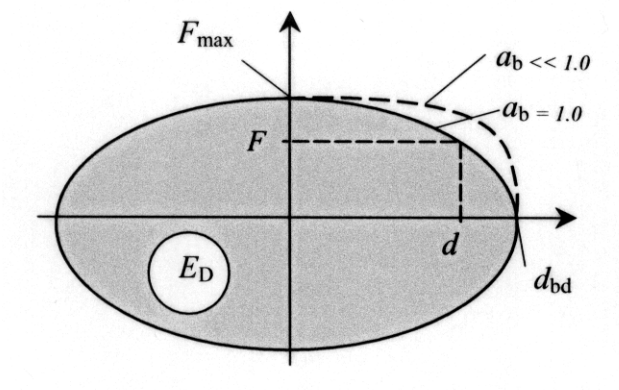


Figure 7.2: Viscous force-displacement behaviour

$d_b = d_{bd}\, sin(\omega t)$, with $\omega = 2\pi/T_{eff}$

$F = Cv^{\alpha_b} = F_{max}(cos(\omega t))^{\alpha_b}$

$F_{max} = C(d_{bd}\omega)^{\alpha_b}$

$E_D = \lambda(\alpha_b)\, F_{max}\, d_{bd}$

$$\lambda(\alpha_b) = 2^{2+\alpha_b} \frac{\Gamma^2(1+0{,}5\alpha_b)}{\Gamma(2+\alpha_b)}$$

$\Gamma(\ )$ = is the gamma function

NOTE: In certain cases of viscous devices (fluid dampers) with low α_b-values, combination of the viscous element with a linear spring in series (reflecting the fluid compressibility) is necessary to give satisfactory agreement of the force-velocity relationship with test results for E_D. However this has only minor influence on the energy (E_D) dissipated by the device.

7.5.2.3.5 Friction behaviour

(1) Sliding devices with a flat sliding surface limit the force transmitted to the superstructure to:

$$F_{max} = \mu_d N_{Sd} sign(\dot{d}_b) \quad (7.1)$$

where:

μ_d is the dynamic friction coefficient

N_{Sd} is the normal force through the device, and

$sign(\dot{d}_b)$ is the sign of the velocity vector $\dot{d}_b$

d_b is the relative displacement of the two sliding surfaces

Such devices however can result in substantial permanent displacements. Therefore they should be used in combination with devices providing adequate restoring capability (see **7.7.1**).

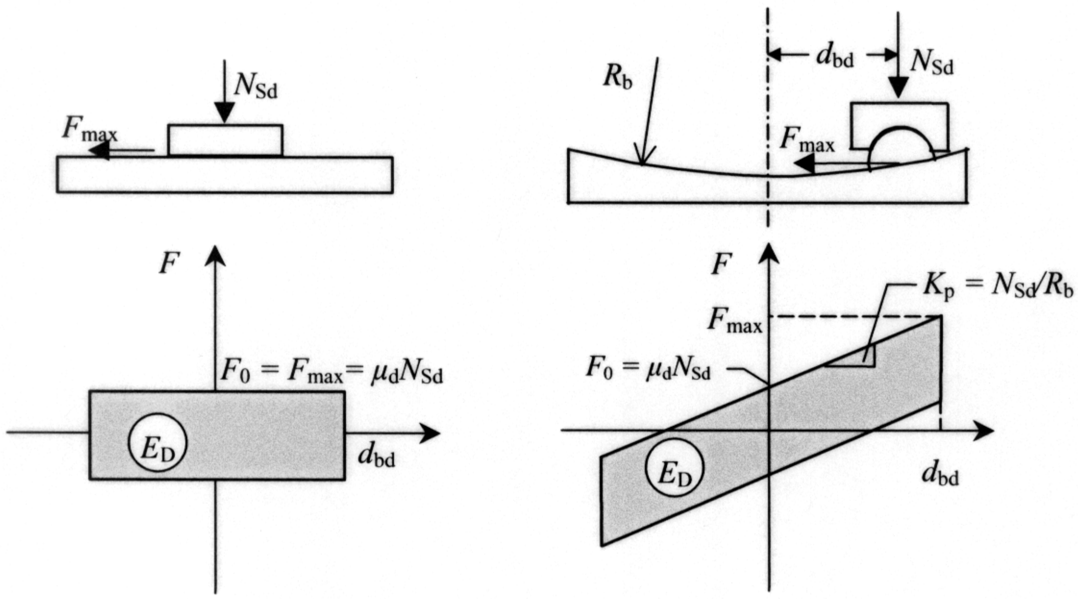


Figure 7.3: Friction force-displacement behaviour

(2) Sliding devices with a spherical sliding surface of radius R_b provide a restoring force at displacement d_b equal to $N_{Sd}d_b/R_b$. For such a device the force displacement relationship is:

$$F_{max} = \frac{N_{Sd}}{R_b} d_{bd} + \mu_d N_{Sd} sign(\dot{d}_{bd}) \qquad (7.2)$$

NOTE: Expression (7.2) offers sufficient approximation when $d_b/R_b \leq 0,25$

(3) In both the above cases the energy dissipated per cycle E_D (see Figure 7.3), at the design displacement d_{bd} amounts to:

$$E_D = 4\mu_d N_{Sd} d_{bd} \qquad (7.3)$$

(4) The dynamic friction coefficient μ_d depends mainly on:

- the composition of the sliding surfaces;
- the use or not of lubrication;
- the bearing pressure on the sliding surface in the seismic design situation;
- the velocity of sliding

and should be determined by appropriate tests.

NOTE: Information on tests that may be used for the determination of the dynamic friction coefficient is given in Informative Annex **K.** It should be noted that for lubricated pure virgin PTFE that slides on polished stainless steel surface, the dynamic friction coefficient may be quite low (≤ 0,01) at the range of velocities corresponding to seismic motions and under the usual range of bearing pressures on the sliding surface in the seismic design situation.

(5) Provided that the equivalent damping of the isolating system is assessed ignoring any contribution from these elements, sliding bearings with a lubricated PTFE flat sliding surface allowing sliding in both horizontal directions in accordance with EN 1337-2:2000 and elastomeric bearings with sliding lubricated PTFE elements allowing sliding in one horizontal direction, while in the other direction they behave as normal elastomeric bearings, in accordance with EN 1337-2:2000 and EN 1337-3:1996, are not subject to special tests for seismic performance.

7.5.2.4 Variability of properties of the isolator units

(1)P The nominal design properties (DP) of isolator units shall be validated in general in accordance with prEN15129:200X: Antiseismic Devices or be included in a ETA, with the exception of the special cases of normal elastomeric bearings in accordance with **7.5.2.3.3(5)** and **7.5.2.3.3(6)**, and of sliding bearings in accordance with **7.5.2.3.5(5)**, for which **(4)**, **(5)** and **(6)** below apply.

NOTE See also Note under **7.5.2.1(1)**P.

(2)P The nominal properties of the isolator units, and hence those of the isolating system, may be affected by ageing, temperature, loading history (scragging), contamination, and cumulative travel (wear). This variability shall be accounted for in accordance with Annex **J,** by using the following two sets of design properties of the isolating system, properly established,:

- Upper bound design properties (UBDP), and
- Lower bound design properties (LBDP).

(3)P In general and independently of the method of analysis, two analyses shall be performed: one using the UBDPs and leading to the maximum forces in the substructure and the deck, and another using the LBDPs and leading to the maximum displacements of the isolating system and the deck.

(4) Multi-mode spectrum analysis or Time-history analysis may be performed on the basis of the set of the nominal design properties, only if the design displacements d_{cd}, resulting from a Fundamental mode analysis, in accordance with **7.5.4**, based on UBDPs and LBDPs, do not differ from that corresponding to the design properties by more than ±15%.

(5) The nominal design properties of simple elastomeric bearings in accordance with **7.5.2.3.3(5)** and **(6)**, may be assumed as follows:

- Shear modulus $G_b = 1{,}1\ G_g$

– where G_g is the value of the "apparent conventional shear modulus" in accordance with EN 1337-3:1996, **4.3.1.1;**

– Equivalent viscous damping $\xi_{eff} = 0,05$

(6) The variability of the design properties of normal elastomeric bearings, due to ageing and temperature, may be limited to the value of G_b and assumed as follows:

– LBDPs $G_{b,min} = G_b$

– UBDPs depend on the "minimum bearing temperature for seismic design" $T_{min,b}$ (see **J.1(2)**) as follows:

- when $T_{min,b} \geq 0^oC$

  $G_{b,max} = 1,5\ G_b$

- when $T_{min,b} < 0^oC$

  the value of G_{max} should correspond to $T_{min,b}$.

NOTE: In the absence of relevant test results the $G_{b,max}$ value given as UBDPs may be adjusted regarding temperature in accordance with the λ_{max} values corresponding to K_p specified in Table JJ.2.

(7) Values of friction parameters of the sliding elements whose contribution in the energy dissipation is ignored in accordance with **7.5.2.3.5(5)**, should be taken in accordance with EN 1337-2:2000.

7.5.3 Conditions for application of analysis methods

(1)P The Fundamental mode spectrum analysis may be applied if all of the following conditions are met:

a. The distance of the bridge site to the nearest known seismically active fault exceeds 10 km.

b. The ground conditions of the site correspond to one of the ground types A, B, C or E of EN 1998-1:2004, **3.1.1**.

c. The effective damping ratio does not exceed 0,30.

(2)P Multi-mode Spectrum Analysis may be applied if both conditions b and c of **(1)**P are met.

(3) Time-history non-linear analysis may be applied for the design of any isolated bridge.

7.5.4 Fundamental mode spectrum analysis

(1) The rigid deck model (see **4.2.2.3**) should be used in all cases.

(2)P The shear force transferred through the isolating interface in each principal direction shall be determined considering the superstructure as a single-degree-of-freedom system and using:

- the effective stiffness of the isolation system, K_{eff}
- the effective damping of the isolation system, ξ_{eff}
- the mass of the superstructure, M_{d}
- the spectral acceleration $S_{\mathrm{e}}(T_{\mathrm{eff}}, \eta_{\mathrm{eff}})$ (see EN 1998-1:2004, **3.2.2.2**) corresponding to the effective period, T_{eff}, with $\eta_{\mathrm{eff}} = \eta(\xi_{\mathrm{eff}})$

The values of these parameters should be determined as follows:

- Effective stiffness

$$K_{\mathrm{eff}} = \Sigma\, \mathrm{K}_{\mathrm{eff,i}} \tag{7.4}$$

where $\mathrm{K}_{eff,i}$ is the composite stiffness of the isolator unit and the corresponding substructure (pier) *i*.

- Effective damping

$$\xi_{\mathrm{eff}} = \frac{1}{2\pi}\left[\frac{\Sigma E_{\mathrm{D,i}}}{K_{\mathrm{eff}} d_{\mathrm{cd}}^2}\right] \tag{7.5}$$

where:

$\Sigma E_{\mathrm{D,i}}$ is the sum of dissipated energies of all isolators *i* in a full deformation cycle at the design displacement d_{cd}.

- Effective Period

$$T_{\mathrm{eff}} = 2\pi\sqrt{\frac{M_{\mathrm{d}}}{K_{\mathrm{eff}}}} \tag{7.6}$$

(3) This leads to the results shown in Table 7.1 and Figure 7.4.

Table 7.1: Spectral acceleration S_{e} and design displacement d_{cd}

T_{eff}	S_{e}	d_{cd}
$T_{\mathrm{C}} \leq T_{\mathrm{eff}} < T_{\mathrm{D}}$	$2{,}5\,\dfrac{T_{\mathrm{C}}}{T_{\mathrm{eff}}}\eta_{\mathrm{eff}} a_{\mathrm{g}} S$	$\dfrac{T_{\mathrm{eff}}}{T_{\mathrm{C}}} d_{\mathrm{C}}$
$T_{\mathrm{D}} \leq T_{\mathrm{eff}}$	$2{,}5\,\dfrac{T_{\mathrm{C}} T_{\mathrm{D}}}{T_{\mathrm{eff}}^2} S \eta_{\mathrm{eff}} a_{\mathrm{g}}$	$\dfrac{T_{\mathrm{D}}}{T_{\mathrm{C}}} d_{\mathrm{C}}$

where:

$$a_{\mathrm{g}} = \gamma_{\mathrm{I}} a_{\mathrm{g,R}} \tag{7.7}$$

and

$$d_{\mathrm{C}} = \frac{0{,}625}{\pi^2} a_g S \eta_{\mathrm{eff}} T_{\mathrm{C}}^2 \tag{7.8}$$

The value of η_{eff} should be taken from the expression

$$\eta_{\mathrm{eff}} = \sqrt{\frac{0{,}10}{0{,}05+\xi_{\mathrm{eff}}}} \tag{7.9}$$

Maximum shear force

$$V_{\mathrm{d}} = M_{\mathrm{d}} S_{\mathrm{e}} = K_{\mathrm{eff}} d_{\mathrm{ed}} \tag{7.10}$$

where:

S, T_C and T_D are parameters of the design spectrum depending on the ground type, in accordance with **7.4.1(1)**P and EN 1998-1:2004, **3.2.2.2**;

a_g is the design ground acceleration on type A ground corresponding to the importance category of the bridge;

γ_I is the importance factor of the bridge; and

$a_{g,R}$ is the reference design ground acceleration (corresponding to the reference return period).

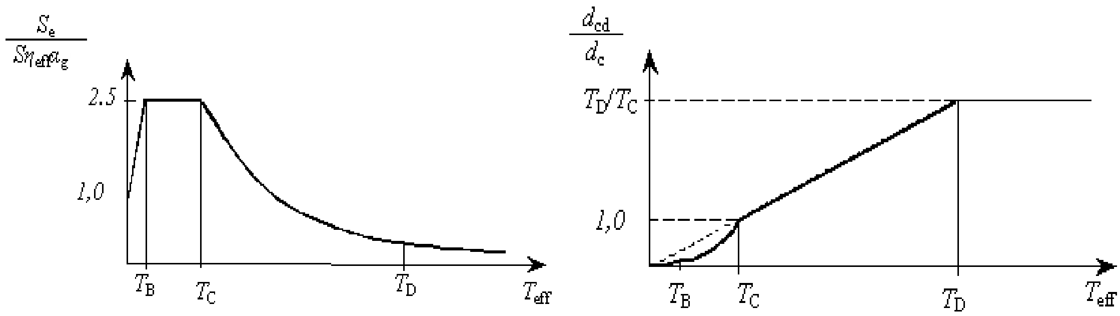
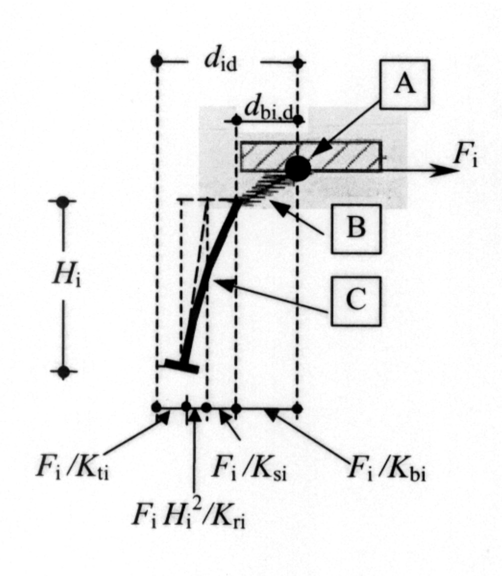


Figure 7.4: Acceleration and displacement spectra

NOTE: For a pier of height H_i with a displacement stiffness K_{si} (kN/m), supported by a foundation with translation stiffness K_{ti} (kN/m), rotation stiffness K_{fi} (kNm/rad), and carrying isolator unit i with effective stiffness K_{bi} (kN/m), the composite stiffness $K_{eff,i}$ is (see Figure 7.5N):

$$\frac{1}{K_{\mathrm{eff,i}}} = \frac{1}{K_{\mathrm{bi}}} + \frac{1}{K_{\mathrm{ti}}} + \frac{1}{K_{\mathrm{si}}} + \frac{H_{\mathrm{i}}^2}{K_{\mathrm{fi}}} \tag{7.11N}$$

The flexibility of the isolator and its relative displacement $d_{bi} = \frac{F_i}{K_{bi}}$ typically is much larger than the other components of the superstructure displacement. For this reason the effective damping of the system depends only on the sum of dissipated energies of the isolators, ΣE_{Di}, and the relative displacement of the isolator is practically equal to the displacement of the superstructure at this point ($d_{bi}/d_{id} = K_{eff,i}/K_{bi} \cong 1$).



Key
A – Superstructure
B – Isolator *i*
C – Pier *i*

Figure 7.5N: Composite stiffness of pier and isolator *i*

(4) In essentially non-linear systems, K_{eff} and ξ_{eff} depend on the design displacement d_{cd} (see d_{bd} in Figure 7.1). Successive approximations of d_{cd} should be performed to limit deviations between the assumed and calculated values within ±5%.

(5) For the determination of the seismic action effects on the isolating system and the substructures in the principal transverse direction (let's say direction *y*), the influence of plan eccentricity in the longitudinal direction e_x (between the effective stiffness centre and the centre of mass of the deck) on the superstructure displacement d_{id} over pier *i*, should be evaluated as follows:

$$d_{id} = \delta_i d_{cd} \tag{7.12}$$

$$\delta_i = 1 + \frac{e_x}{r\, r_x} x_i \tag{7.13}$$

with:

$$r_x^2 = \frac{\Sigma\left(x_i^2 K_{yi} + y_i^2 K_{xi}\right)}{\Sigma K_{yi}} \tag{7.14}$$

where:

e_x is the eccentricity in the longitudinal direction;

r is the radius of gyration of the deck mass about the vertical axis through its centre of mass;

x_i and y_i are the coordinates of pier *i* relative to the effective stiffness center;

K_{yi} and K_{xi} are the effective composite stiffnesses of isolator unit and pier i, in the y and x directions, respectively.

NOTE: In straight bridges usually $y_i << x_i$. In such cases the term $y_i^2 K_{xi}$ in expression (7.14) may be omitted.

(6)P Subclause **4.2.1.4(2)** shall be applied for the combination of components of the seismic action.

7.5.5 Multi-mode Spectrum Analysis

(1)P The modelling of the isolating system shall reflect with sufficient accuracy:

- the spatial distribution of the isolator units and the relevant overturning effects, and
- the translation in both horizontal directions and the rotation about the vertical axis of the superstructure.

(2)P The modelling of the superstructure shall reflect with sufficient accuracy its deformation in plan. Accidental mass eccentricity need not be considered.

(3) The modelling of the substructures should reflect with sufficient accuracy the distribution of their stiffness properties and at least the rotational stiffness of the foundation. When the pier has significant mass and height, or if it is immersed in water, its mass distribution should also be properly modelled.

(4) The effective damping given by expression (7.5) may be applied only to modes having periods higher than $0{,}8T_{\text{eff}}$. For all other modes, unless a more accurate estimation of the relevant damping ratio is made, the damping ratio corresponding to the structure without seismic isolation should be used.

(5)P Subclause **4.2.1.4(2)** shall be applied for the combination of the horizontal components of the seismic action.

(6) The resulting displacement of the stiffness centre of the isolating system (d_{cd}) and the resulting total shear force transferred through the isolation interface (V_d) in each of the two-horizontal directions, are subject to lower bounds as follows:

$$\rho_d = \frac{d_{cd}}{d_{cf}} \geq 0{,}80 \tag{7.15}$$

$$\rho_v = \frac{V_d}{V_f} \geq 0{,}80 \tag{7.16}$$

where:

d_{cf}, V_f are respectively the design displacement and the shear force transferred through the isolation interface, calculated in accordance with the Fundamental mode spectrum analysis of **7.5.4**. For the needs of the verification of expressions (7.15) and (7.16), the limitations of **7.5.3(1)**P do not apply.

(7) In case the conditions in **(6)** are not met, the relevant effects on the isolation system, the deck and the substructures should be multiplied times:

$$\frac{0,80}{\rho_{\mathrm{d}}} \text{ for the seismic displacements, or} \tag{7.17}$$

$$\frac{0,80}{\rho_{\mathrm{v}}} \text{ for the seismic forces and moments} \tag{7.18}$$

(8) The limitations of **(6)** and the relevant corrections in **(7)**, need not be applied if the bridge cannot be approximated (even crudely) as a single-degree-of-freedom model. Such cases may appear in:

- bridges with high piers, the mass of which has a significant influence on the displacement of the deck
- bridges with a substantial eccentricity e_{x} in the longitudinal direction between the centre of mass of the deck and the effective stiffness centre ($e_{\mathrm{x}} > 0,10L$)

In such cases it is recommended that the limitations and corrections of **(6)** and **(7)** are applied in each direction to displacements and forces derived from the fundamental mode of the actual bridge model in the same direction.

7.5.6 Time history analysis

(1)P Subclauses **7.5.5(1)**P, **(2)**P, **(3)**, **(6)**, **(7)**P and **(8)**P apply.

7.5.7 Vertical component of seismic action

(1) The effects of the vertical component of the seismic action may be determined by linear response spectrum analysis, regardless of the method used for the determination of the response to the horizontal seismic action. For the combination of the action effects **4.2.1.4** applies.

7.6 Verifications

7.6.1 Seismic design situation

(1)P The seismic design situation is described by expression (5.4) in **5.5(1)**P.

(2)P The design seismic action effects for the isolating system shall be taken in accordance with **7.6.2** and those for the superstructure and substructure in accordance with **7.6.3**.

7.6.2 Isolating system

(1)P The required increased reliability of the isolating system (see **7.3(4)**P) shall be implemented by designing each isolator i for increased design displacements $d_{\mathrm{bi,a}}$:

$$d_{\mathrm{bi,a}} = \gamma_{\mathrm{IS}} d_{\mathrm{bi,d}} \tag{7.19}$$

where γ_{IS} is an amplification factor that is applied only on the design displacement $d_{\mathrm{bi,d}}$ of each isolator i resulting from one of the procedures specified in **7.5**.

If the spatial variability of the seismic action is accounted for through the simplified method of **3.3(4)**, **(5)**, **(6)** and **(7)**P, the increased design displacements shall be estimated by application of the rule of **3.3(7)**P, where the displacements $d_{\mathrm{bi,d}}$ due the inertia response determined in accordance with one of the methods in **7.5** shall be amplified in accordance with expression (7.19) above, while those corresponding to the spatial variability determined in accordance with **3.3.(5)** and **(6),** need not be amplified.

NOTE The value ascribed to γ_{IS} for use in a country may be defined in its National Annex. The recommended value is γ_{IS} = 1,50.

(2)P The maximum total displacement of each isolator unit in each direction shall be obtained by adding to the above increased design seismic displacement, the offset displacement potentially induced by:

a) the permanent actions;

b) the long-term deformations (concrete shrinkage and creep) of the superstructure; and

c) 50% of the thermal action.

(3)P All components of the isolating system shall be capable of functioning at the total maximum displacements.

(4)P The design resistance of each load-carrying member of the isolation system, including its anchorage, shall exceed the force acting on the member at the total maximum displacement. It shall also exceed the design force caused by wind loading of the structure in the relevant direction.

NOTE The maximum reaction of hydraulic viscous dampers (see **7.5.2.3.4**) corresponding to the increased displacement $d_{\mathrm{bi,a}}$ may be estimated by multiplying the reaction resulting from the analysis times $\gamma_{\mathrm{IS}}^{\alpha_{\mathrm{b}}/2}$, with α_{b} as defined in **7.5.2.3.4**

(5) Isolator units consisting of normal elastomeric bearings should be verified for the action effects in **(1)**P to **(4)**P, in accordance with the rules of **5.3.3** of EN 1337-3:1996 and using the value of K_{L} = 1 in expression (5.1) of EN 1337-3:1996.

NOTE The value ascribed to γ_{m} in expression (5.2) of EN 1337-3:1996 for use in a country may be specified in the National Annex. For the needs of this Standard the recommended value is γ_{m} = 1,15.

(6) For normal elastomeric bearings, in addition to the verification of **(5)**, the following condition should be verified:

$$\varepsilon_{\mathrm{q,d}} \leq 2{,}0 \tag{7.20}$$

where $\varepsilon_{\mathrm{q,d}}$ is the shear strain calculated in accordance with expression (5.9) of **5.3.3.3** of EN 1337-3: 1996. In this context $v_{\mathrm{x,d}}$ and $v_{\mathrm{y,d}}$ should be taken equal to the maximum total relative displacements in the corresponding directions, as specified in **(2)** above.

(7) No uplift of isolators carrying vertical force is allowed in the seismic design situation with the seismic action as specified by **7.4**.

(8) The sliding elements mentioned in **7.5.2.3.5(5)** should be designed in accordance with EN 1337-2:2000, for seismic design displacement in accordance with **(1)**P above.

7.6.3 Substructures and superstructure

(1)P The seismic internal forces E_{EA} in the substructures and superstructure due to the design seismic action alone, shall be derived from the results of an analysis in accordance with **7.5**.

(2) The design seismic forces E_E due to the design seismic action alone, may be derived from the forces E_{EA} of **(1)**P, after division by the q-factor corresponding to limited ductile/essentially elastic behaviour, i.e. $F_E = F_{E,A}/q$ with $q \leq 1,50$.

(3) All members of the structure should be verified to have an essentially elastic behaviour in accordance with the rules of **5.6.2** and **6.5**.

(4)P Design action effects for the foundation shall be in accordance with **5.8.2(2)**P.

(5) The design horizontal forces of supporting members (piers or abutments) carrying sliding bearings described in **7.5.2.3.5(5)**, should be derived from the maximum friction values in accordance with the relevant provision of EN 1337-2:2000.

(6) In the case of **(5)** above and when the same supporting member also carries viscous fluid dampers, then:

(a) the design horizontal seismic force of the supporting member in the direction of the action of the damper should be increased by the maximum seismic force of the damper (see expression (7.21)).

(b) the design horizontal force of non-seismic design situations under imposed deformation actions (temperature variation) should be increased by the damper reaction, estimated as 10% of the maximum seismic force of the damper, used in (a) above.

(7) When single or multiple mode spectral analysis is carried out for isolating systems consisting of combination of elastomeric bearings and fluid viscous dampers supported on the same supporting element(s), the phase difference between the maxima of the elastic and the viscous elements may be taken into account, by the following approximation. The seismic forces should be determined as the most adverse of those corresponding to the following characteristic states:

a. At the state of maximum displacement, as given by expression (7.10). The damper forces are then equal to zero.

b. At the state of maximum velocity and zero displacement, when the maximum damper forces should be determined by assuming the maximum velocity to be:

$$v_{max} = 2\pi d_{bd}/T_{eff} \tag{7.21}$$

where d_{bd} is the maximum damper displacement corresponding to the design displacement d_{cd} of the isolating system.

c. At the state of the maximum inertial force on the superstructure, that should be estimated as follows:

$$F_{max} = (f_1 + 2\xi_b f_2) S_e M_d \tag{7.22}$$

where S_e is determined from Table 7.1 with K_{eff} in accordance with expression (7.4), without any stiffness contribution from the dampers, and

$$f_1 = cos[arctan(2\xi_b)] \tag{7.23a}$$

$$f_2 = sin[arctan(2\xi_b)] \tag{7.23b}$$

where ξ_b is the contribution of the dampers to the effective damping ξ_{eff} of expression (7.5).

At this state the displacement amounts to $f_1 d_{cd}$ and the velocity of the dampers to $v = f_2 v_{max}$

(8) In isolating systems consisting of a combination of fluid viscous dampers and elastomeric bearings, as in the case of **(7)**, without sliding elements, the design horizontal force acting on supporting element(s) that carry both bearings and dampers, for non-seismic situations of imposed deformation actions (temperature variation, etc.) should be determined by assuming that the damper reactions are zero.

7.7 Special requirements for the isolating system

7.7.1 Lateral restoring capability

(1)P The isolating system shall present self-restoring capability in both principal directions, to avoid cumulative build-up of displacements. This capability is available when the system has either one of the following two properties (see also Figure 7.6):

- small residual displacement d_{rm} in relation to its displacement capacity d_m
- starting from the position of residual displacement, the system presents substantially smaller stiffness to movement in the direction towards the centre than in the opposite direction. In the latter direction an adequate displacement margin should be available.

(2) The requirements in **(1)**P are considered to be satisfied when both following conditions are met:

$$\Delta F_m \geq \delta_w W_d \, d_{rm} / d_m \tag{7.24a}$$

$$d_{rm} \leq d_m - \delta_d \, d_{a,max} \tag{7.24b}$$

where:

ΔF_m is the force increase between displacements $d_m/2$ and d_m,

W_d is the weight of the superstructure mass,

d_m is the displacement capacity of the isolating system in the considered direction, i.e. the maximum displacement that the system can sustain in this direction,

d_{rm} is the residual displacement of the isolating system, corresponding to d_m, i.e. the residual displacement when the force F_m, required to induce displacement d_m, is removed, under quasi-static conditions

$d_{a,max}$ is the maximum value of the design displacement of the isolating system, increased according to expression (7.19), and

δ_w and δ_d are numerical coefficients expressing appropriate fractions of W_d and $d_{a,max}$ respectively.

NOTE: The values ascribed to δ_w and δ_d for use in a country may be found in its National Annex. The recommended values are: $\delta_w = 0{,}015$, $\delta_d = 0{,}5$

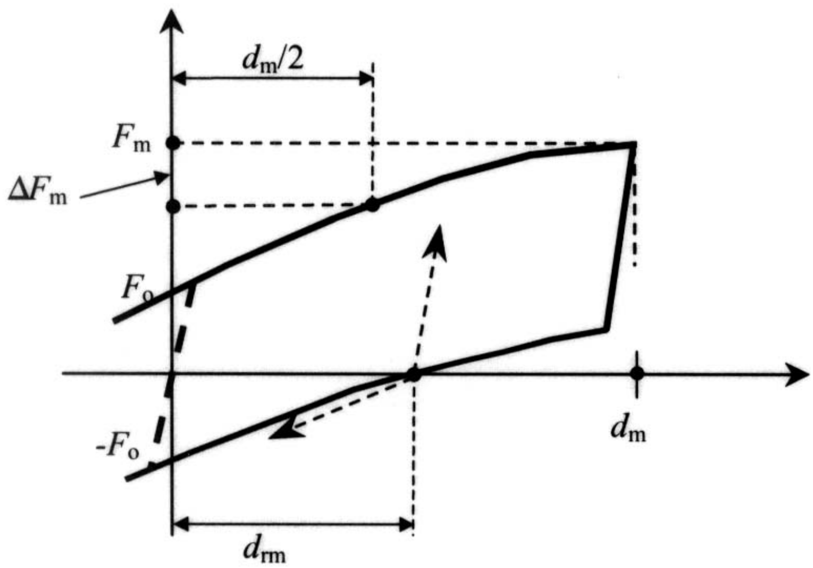


Figure 7.6: Lateral restoring capability of isolating system

(3) The nominal design properties of the isolators under dynamic conditions may be used for the conservative determination of d_{rm} and ΔF_m

NOTE 1 Isolating systems satisfying the expressions (7.24) conform to the second bullet point of **7.7.1(1)P**. Consequently, such systems tend to recentre, when the force equilibrium is disturbed. For the same reason, for such systems the residual displacement need not be considered for the determination of the displacement capacity, following a seismic event

NOTE 2 For systems with bilinear hysteretic behaviour according to **7.5.2.3.2**, the residual displacement d_{rm} should be determined from $d_r = F_0/K_p = F_y/K_p - d_y$ as a function of d_m, as shown in Table 7.2N:

Table 7.2N. Determination of system residual displacement from $d_r = F_0/K_p$ and the displacement capacity of isolating system with bilinear hysteretic behaviour

Range of d_m	d_{rm}

$d_r + 2d_y \le d_m$	d_r
$d_y < d_m < d_r + 2d_y$	$d_r(d_m - d_y) / (d_r + d_y)$
$d_m \le d_y$	0

NOTE 3: For systems of sliding devices with spherical sliding surface (see **7.5.2.3.5(2)**) the residual displacement is $d_{rm} = \mu_d R_b$

7.7.2 Lateral restraint at the isolation interface

(1)P The isolating system shall provide sufficient lateral restraint at the isolation interface to satisfy any relevant requirements of other Eurocodes or Standards regarding limitation of displacements/deformations under serviceability criteria.

NOTE This requirement is usually critical for braking action in railway bridges.

(2) When sacrificial bracings (a fuse system) are used at certain support(s) in the final bridge system for implementing serviceability displacement restraints between the deck and substructures, their yield capacity should not exceed 40% of the design seismic force transferred through the isolation interface of the isolated structure, at the same support and direction. If this requirement is not met, the serviceability state requirements (except fatigue) of the relevant material Eurocodes (EN 1992-2:2005, EN 1993-2:2005 or EN 1994-2:2005) should be satisfied for the members of the bridge structure, under the loading for which the restraining bracing is designed, when this loading is increased so that the relevant reaction reaches the yield capacity of the bracing.

(3) When shock transmission units with force limiting function (see **6.6.3.3**) are used for implementing serviceability displacement restraints, the shock transmission units should be included in the model, in the verifications and in the testing procedure of the isolating system.

7.7.3 Inspection and Maintenance

(1)P All isolator units shall be accessible for inspection and maintenance.

(2)P An inspection and maintenance programme for the isolating system and all components crossing the isolation interface shall be prepared.

(3)P Repair, replacement or retrofitting of any isolator unit or component crossing the isolation interface shall be performed under the direction of the entity responsible for the maintenance of the bridge, and shall be recorded in detail in a relevant report.

ANNEX A (Informative)
PROBABILITIES RELATED TO THE REFERENCE SEISMIC ACTION. GUIDANCE FOR THE SELECTION OF DESIGN SEISMIC ACTION DURING THE CONSTRUCTION PHASE

A.1 Reference seismic action

(1) The reference seismic action can be defined by selecting an acceptably low probability (p) of it being exceeded within the design life (t_L) of the structure. Then the return period of the event (T_R) is given by the expression:

$$T_R = 1/(1-(1-p)^{1/t_L} \tag{A.1}$$

(2) The reference seismic action (corresponding to γ_I = 1,0) usually reflects a seismic event with a reference return period, T_{NCR}, of 475 years. Such an event has a probability of exceedance between 0,10 and 0,19 for a design life ranging between 50 and 100 years respectively. This level of design action is applicable to the majority of the bridges considered to be of average importance.

A.2 Design seismic action for the construction phase

(1) Assuming that t_c is the duration of the construction phase of a bridge and p is the acceptable probability of exceedance of the design seismic event during this phase, the return period T_{Rc} is given by expression (A.1), using t_c instead of t_L. For the relatively small values usually associated with t_c (t_c ≤ 5 years), expression (A.1) may be approximated by the following simpler relationship:

$$T_{Rc} \cong \frac{t_c}{p} \tag{A.2}$$

It is recommended that the value of p does not exceed 0,05.

(2) The value of the design ground acceleration a_{gc} corresponding to a return period T_{Rc}, depends on the seismicity of the region. In many cases the following relationship offers an acceptable approximation

$$\frac{a_{gc}}{a_{g,R}} = (\frac{T_{Rc}}{T_{NCR}})^k \tag{A.3}$$

where:

$a_{g,R}$ is the reference peak ground acceleration corresponding to the reference return period T_{NCR}.

The value of the exponent k depends on the seismicity of the region. Normally, values in the range of 0,30 – 0,40 may be used.

(3) The robustness of all partial bridge structures should be ensured during the construction phases independently of the design seismic actions.

ANNEX B(INFORMATIVE) RELATIONSHIP BETWEEN DISPLACEMENT DUCTILITY AND CURVATURE DUCTILITY FACTORS OF PLASTIC HINGES IN CONCRETE PIERS

(1) Assuming that:

- the horizontal displacement at the centre of mass of the deck is due only to the deformation of a fully fixed cantilever pier of length L, that
- the mass of the pier is negligible compared to that of the deck, and that
- L_p is the length of the plastic hinge developing at the base of the pier,

the required curvature ductility factor μ_Φ of the hinge corresponding to a structure displacement ductility factor μ_d, as defined in **2.3.5.2**, is:

$$\mu_\Phi = \frac{\Phi_u}{\Phi_y} = 1 + \frac{\mu_d - 1}{3\lambda(1 - 0,5\lambda)} \qquad \text{(B.1)}$$

where: $\lambda = L_p/L$

(2) In reinforced concrete sections (where the curvature ductility factor is used as a measure of the ductility of the plastic hinge), the value of the ratio λ is influenced by such effects as the reinforcement tensile strain penetration in the adjoining member, the inclined cracking due to shear-flexure interaction etc. The value of L_p in accordance with **E.3.2(5)** may be used.

(3) When a considerable part of the deck displacement is due to the deformation of other components which remain elastic after the formation of the plastic hinge, the required curvature ductility factor $\mu_{\Phi d}$ is given by the expression

$$\mu_{\Phi d} = 1 + f\,(\mu_\Phi - 1) \qquad \text{(B.2)}$$

where:

$f = d_{tot}/d_p$ is the ratio of the total deck displacement d_{tot} to the displacement d_p, due to the deformation of the pier only, and

μ_Φ is calculated from expression (B.1).

NOTE: If the seismic action is transferred between deck and pier through flexible elastomeric bearings inducing for example a value of $f = 5$ and assuming that for example $\mu_\Phi = 15$, would be required in the case of rigid connection between the deck and the pier, the required value of $\mu_{\Phi d}$ in accordance with equation (B.2) amounts to 71, which is certainly not available. It is therefore evident that the high flexibility of the elastomeric bearings, used in the same force path with the stiff pier, imposes a practically elastic overall behaviour of the system.

ANNEX C (INFORMATIVE) ESTIMATION OF THE EFFECTIVE STIFFNESS OF REINFORCED CONCRETE DUCTILE MEMBERS

C.1 General

(1) The effective stiffness of ductile concrete components used in linear seismic analysis should be equal to the secant stiffness at the theoretical yield point. Unless otherwise substantiated by calculation, one of the following approximate methods may be used to determine the secant stiffness at the theoretical yield point:

C.2 Method 1

(1) The effective moment of inertia J_{eff} of a pier of constant cross section may be estimated as follows:

$$J_{eff} = 0,08\, J_{un} + J_{cr} \tag{C.1}$$

where:

J_{un} is the moment of inertia of the gross section of the uncracked pier;

J_{cr} is the moment of inertia of the cracked section at the yield point of the tensile reinforcement. This may be estimated from the expression:

$$J_{cr} = M_y/(E_c.\Phi_y) \tag{C.2}$$

in which M_y and Φ_y are the yield moment and curvature of the section respectively and E_c is the elastic modulus of concrete.

(2) These expressions have been derived from a parametric analysis of a simplified non-linear model of a cantilever pier with hollow rectangular and hollow and solid circular cross-sections.

C.3 Method 2

(1) The effective stiffness may be estimated from the design ultimate moment M_{Rd} and the yield curvature Φ_y of the plastic hinge section as follows:

$$E_c J_{eff} = \nu M_{Rd}/\Phi_y \tag{C.3}$$

where:

$\nu = 1,20$ is a correction coefficient reflecting the stiffening effect of the uncracked part of the pier.

The curvature at yield Φ_y may be determined as follows:

$$\Phi_y = (\varepsilon_{sy} - \varepsilon_{cy})/d_s \tag{C.4}$$

and

d_s is the depth of the section to the centre of the tension reinforcement

ε_{sy} is the yield strain of the reinforcement,

ε_{cy} is the compressive strain of concrete at yielding of the tension reinforcement.

The value of ε_{cy} may be estimated by a section analysis on the basis of ε_{sy} and the actual force in the seismic design situation, N_{Ed}.

(2) The assumptions of the following value for the yield curvature:

for rectangular sections: $\Phi_y = 2{,}1\ \varepsilon_{sy}/d$ (C.5)

and for circular sections: $\Phi_y = 2{,}4\ \varepsilon_{sy}/d$ (C.6)

where d is the effective depth of the section, give in general satisfactory approximation.

(3) The analysis performed on the basis of a value of E_cJ_{eff} based on an assumed value of M_{Rd} needs to be corrected only if the finally required value of flexural capacity, $M_{Rd,req}$ is significantly higher than the assumed value M_{Rd}. If $M_{Rd,req} < M_{Rd}$, the correction may just entail multiplication of the displacements resulting from the first analysis times the ratio $M_{Rd}/M_{Rd,req}$.

ANNEX D (INFORMATIVE)
SPATIAL VARIABILITY OF EARTHQUAKE GROUND MOTION: MODEL AND METHODS OF ANALYSIS

D.1 Description of the model

(1) Spatial variability can be described by means of a vector of zero-mean random processes. Under the assumption of stationarity, this vector is fully defined by means of its symmetric n x n matrix of auto- and cross-power spectral density functions:

$$G(\omega)=\begin{bmatrix} G_{11}(\omega) & G_{12}(\omega) & \dots & G_{1n}(\omega) \\ & G_{22}(\omega) & \dots & G_{2n}(\omega) \\ & & \dots & \dots \\ & & & G_{nn}(\omega) \end{bmatrix} \tag{D.1}$$

where n is the number of supports.

It is useful to introduce the following non-dimensional complex-valued function, called *coherency function*:

$$\gamma_{ij}(\omega)=\frac{G_{ij}(\omega)}{\sqrt{G_{ii}(\omega)G_{jj}(\omega)}} \tag{D.2}$$

Its modulus is bounded by 0 and 1,0 and provides a measure of the linear statistical dependence of the two processes at the supports i and j, whose distance is d_{ij}.

(2) The following form of the coherency function is frequently referred to [1][2]:

$$\gamma_{ij}(\omega)=\gamma_{ij,1}(\omega)\cdot\gamma_{ij,2}(\omega)\cdot\gamma_{ij,3}(\omega)=\exp\left[-\left(\frac{\alpha\omega d_{ij}}{v_s}\right)^2\right]\cdot\exp\left[i\frac{\omega d_{ij}^L}{v_{app}}\right]\cdot\exp\left[i\theta_{ij}(\omega)\right] \tag{D.3}$$

where:

v_{s} is the shear-wave velocity,

a is a constant,

v_{app} is the so-called apparent velocity of waves,

$d_{\mathrm{ij}}^{\mathrm{L}}$ is the distance between supports i and j projected along the direction of propagation of the waves, and

$\theta_{\mathrm{ij}}(\omega)$ is a frequency-dependent phase angle.

(3) The factors $\gamma_{\mathrm{ij},1}(\omega)$, $\gamma_{\mathrm{ij},2}(\omega)$ and $\gamma_{\mathrm{ij},3}(\omega)$ account for the loss of correlation due to reflections/refractions in the propagation medium, for the finiteness of the propagation velocity of the waves and their angle of incidence at the surface and for the different soil conditions at the two supports, respectively. The difference of the soil properties at

two supports is taken into account in the model by considering two soil columns representing the two soil profiles acted upon at their base by a stationary white noise of intensity G_0. The soil columns are characterised by transfer functions $H_i(\omega)$ and $H_j(\omega)$, respectively, which are such as to provide the desired spectral content and intensity of the motion at the upper surface in locations *i* and *j*

$$G_{ii}(\omega) = G_0 \left| H_i(\omega) \right|^2 \tag{D.4}$$

(4)P The power density spectrum at the site shall be consistent with the elastic response spectrum as given in EN 1998-1: 2004, **3.2.2.2**.

It can also be shown that:

$$\theta_{ij}(\omega) = \tan^{-1} \left\{ \frac{\mathrm{Im}\left[H_i(\omega) H_j(-\omega) \right]}{\mathrm{Re}\left[H_i(\omega) H_j(-\omega) \right]} \right\} \tag{D.5}$$

D.2 Generation of samples

(1) For the purposes of structural analysis samples of the vector of random processes described in **D.1** may need to be derived. To this end the matrix $\mathbf{G}(\omega)$ is first decomposed into the product:

$$\mathbf{G}(\omega) = \mathbf{L}(\omega) \mathbf{L}^{*T}(\omega) \tag{D.6}$$

between matrix $\mathbf{L}(\omega)$ and the transpose of its complex conjugate. If Cholesky decomposition is employed $\mathbf{L}(\omega)$ is a lower triangular matrix.

According to [3] a sample of the acceleration motion at the generic support *i* is obtained from the series:

$$a_i(t) = 2 \sum_{j=1}^{i} \sum_{k=1}^{N} \left| L_{ij}(\omega_k) \right| \sqrt{\Delta\omega} \cos\left[\omega_k t - \theta_{ij}(\omega_k) + \phi_{jk} \right] \tag{D.7}$$

where:

N is the total number of frequencies ω_k into which the significant bandwidth of $L_{ij}(\omega)$ is discretised;

$\Delta\omega = \omega_{max}/N$, and the angles ϕ_{jk} are, for any *j*, a set of *N* independent random variables uniformly distributed between zero and 2π.

Samples generated according to Expression (D.7) are characterised by the desired local frequency content as well as the assigned degree of correlation.

D.3 Methods of analysis

D.3.1 General

(1) Based on **D.1** and **D.2**, the options described in **D.3.2** to **D.3.4** are available for determining the structural response to spatially varying ground motions.

D.3.2 Linear random vibration analysis

(1) A linear random vibration analysis is performed, using either modal analysis of frequency-dependent transfer matrices and input given by the matrix $\mathbf{G}(\omega)$.

(2) The elastic action effects are assumed as the mean values from the probability distribution of the largest extreme value of the response for the duration consistent with the seismic event underlying the establishment of a_g.

(3) The design values are determined by dividing the elastic effects by the appropriate behaviour factor q and ductile response is assured by conformity to the relevant rules of the normative part of this Standard.

D.3.3 Time history analysis with samples of correlated motions

(1) Linear time-history analysis can be performed using sample motions generated as indicated in **D.2,** starting from power spectra consistent with the elastic response spectra at the supports.

(2) The number of samples used should be such as to yield stable estimates of the mean of the maximum responses of interest. The elastic action effects are assumed as the mean values of the above maxima. The design values are determined by dividing the elastic action effects by the appropriate behaviour factor q and ductile response is assured by conformity to the relevant rules of the normative part of this Standard.

(3) Non-linear time-history analysis may be performed using sample motions generated as indicated in **D.2** starting from power spectra consistent with the elastic response spectra at the supports. The number of samples used should be such as to yield stable estimates of the mean of the maximum responses of interest.

(4) The design values of the action effects E_d are assumed as the mean values of the above maxima. The comparison between action effect E_d and design resistance R_d is to be performed in accordance with EN 1998-1:2004.

D.3.4 Response spectrum for multiple-support input

D.3.4.1 General

(1) A solution for the elastic response of a structure subjected to multiple support input in terms of response spectra has been derived in [4]. An outline is given here. For complete information refer to [4].

D.3.4.2 Linear response to multiple-support input

(1) The equations of motion for a discretised, n-degrees of freedom linear system subjected to m support motions can be written as:

$$\begin{bmatrix} M & M_C \\ M_C^T & M_g \end{bmatrix} \begin{bmatrix} \ddot{x} \\ \ddot{u} \end{bmatrix} + \begin{bmatrix} C & C_C \\ C_C^T & C_g \end{bmatrix} \begin{bmatrix} \dot{x} \\ \dot{u} \end{bmatrix} + \begin{bmatrix} K & K_C \\ K_C^T & K_g \end{bmatrix} \begin{bmatrix} x \\ u \end{bmatrix} = \begin{bmatrix} 0 \\ F \end{bmatrix} \quad \text{(D.8)}$$

where:

x is the n-vector of the total displacements at the unconstrained degrees of freedom;

u is the m-vector of prescribed support displacements;

M, **C** and **K** are the n x n mass, damping and stiffness matrices associated with the unconstrained degrees of freedom, respectively;

$\mathbf{M}_g$, $\mathbf{C}_g$ and $\mathbf{K}_g$ are the m x m mass, damping and stiffness matrices associated with the support degrees of freedom, respectively;

$\mathbf{M}_c$, $\mathbf{C}_c$ and $\mathbf{K}_c$ are the n x m coupling matrices; and

F is the m-vector of the reacting forces at the support degrees of freedom.

(2) The total response is decomposed as:

$$\mathbf{x} = \mathbf{x}^s + \mathbf{x}^d \quad \text{(D.9)}$$

where $\mathbf{x}^s$, called pseudo-static component, is the solution of expression (D.8) without the inertia and damping terms, i.e.:

$$\mathbf{x}^s = -\mathbf{K}^{-1}\mathbf{K}_c\mathbf{u} = \mathbf{R}\mathbf{u} \quad \text{(D.10)}$$

Substituting expression (D.9) and (D.10) into expression (D.8), the differential equation for the dynamic component is obtained in the form:

$$\mathbf{M}\ddot{\mathbf{x}}^d + \mathbf{C}\dot{\mathbf{x}}^d + \mathbf{K}\mathbf{x}^d \cong -(\mathbf{M}\mathbf{R} + \mathbf{M}_c)\ddot{\mathbf{u}} \quad \text{(D.11)}$$

after eliminating the comparatively negligible term $-(\mathbf{C}\mathbf{R} + \mathbf{C}_c)\dot{\mathbf{u}}$.

(3) Let $\mathbf{\Phi}$, ω_i and ξ_i be the matrix of modal shapes, the modal frequencies and corresponding damping ratios of the fixed base structure. Setting $\mathbf{x}^d = \mathbf{\Phi}\mathbf{y}$ in Expression (D.11), the uncoupled modal equations are obtained as:

$$\ddot{y}_i + 2\xi_i\omega_i\dot{y}_i + \omega_i^2 y_i = \sum_{k=1}^{m} \beta_{ki}\ddot{u}_k(t) \qquad i = 1,\ldots,n \quad \text{(D.12)}$$

where the modal participation factor has the form:

$$\beta_{ki} = \frac{\boldsymbol{\varphi}_i^T (\mathbf{M}\mathbf{r}_k + \mathbf{M}_c \mathbf{i}_k)}{\boldsymbol{\varphi}_i^T \mathbf{M} \boldsymbol{\varphi}_i} \tag{D.13}$$

in which $\mathbf{r}_k$ is the *k*-th column of **R** and i_k is the *k*-th column of a *n* x *n* identity matrix.

(4) It is convenient to define a normalised modal response $s_{ki}(t)$, representing the response of a single-degree-of-freedom oscillator with frequency and damping ratio of the *i*-th mode, and subjected to the base acceleration $\ddot{u}_k(t)$:

$$\ddot{s}_{ki} + 2\xi_i \omega_i \dot{s}_{ki} + \omega_i^2 s_{ki} = \ddot{u}_k(t) \tag{D.14}$$

Clearly one has:

$$y_i(t) = \sum_{k=1}^{m} \beta_{ki} s_{ki}(t) \tag{D.15}$$

(5) A generic response quantity of interest $z(t)$ (nodal displacement, internal force, etc) can be expressed as a linear function of the nodal displacement $\mathbf{x}(t)$:

$$z(t) = \mathbf{q}^T \mathbf{x}(t) = \mathbf{q}^T \left[\mathbf{x}^s(t) + \mathbf{x}^d(t)\right] \tag{D.16}$$

Substituting for the expressions obtained for $\mathbf{x}^s$ and $\mathbf{x}^d$ one arrives at:

$$z(t) = \sum_{k=1}^{m} a_k u_k(t) + \sum_{k=1}^{m} \sum_{i=1}^{n} b_{ki} s_{ki}(t) \tag{D.17}$$

in which:

$$a_k(t) = \mathbf{q}^T \mathbf{r}_k \qquad b_{ki} = \mathbf{q}^T \boldsymbol{\varphi}_i \beta_{ki} \tag{D.18}$$

D.3.4.3 Response spectrum solution

(1) Using basic random vibration theory in conjunction with a model such as that described in **D.1** for the support motions $\mathbf{u}(t)$, the standard deviation of the response quantity of interest $z(t)$ can be directly determined in terms of the standard deviations of the input processes $\mathbf{u}(t)$ and of the normalised modal responses $\mathbf{s}(t)$, as well as of the correlation between input and output quantities.

(2) Further, by taking into account the relationship between the power spectral densities of the input processes, $\mathbf{G}_{\ddot{\mathbf{u}}\ddot{\mathbf{u}}}(\omega)$[5], and the above standard deviations and correlations, as well as the relationships between power spectral density of the response

[5] $\mathbf{G}_{\ddot{u}\ddot{u}}(\omega)$ denotes the power spectral densities matrix of the ground acceleration processes which, for simplicity of notation, is denoted in D.1 simply by $\mathbf{G}(\omega)$.

process and response spectrum, the following expression is derived for the mean value of the maximum response (i.e. the elastic action effect)[6]:

$$\mu_{z_{\max}} = \sqrt{\sum_{k=1}^{m}\sum_{l=1}^{m} a_k a_l \rho_{u_k u_l} u_{k,\max} u_{l,\max} + \sum_{k=1}^{m}\sum_{l=1}^{m}\sum_{i=1}^{n}\sum_{j=1}^{n} b_{ki} b_{lj} \rho_{s_{ki} s_{lj}} D_k(\omega_i, \xi_i) D_l(\omega_j, \xi_j)} \qquad \text{(D.19)}$$

where $u_{k,\max}$ and $u_{l,\max}$ are the peak ground displacements at supports k and l consistent with the respective local elastic response spectrum as given in prEN 1998-1:2004, **3.2.2.4**; $D_k(\omega_i, \xi_i)$ and $D_l(\omega_j, \xi_j)$ are the elastic displacement response spectra values at supports k and l for frequencies and damping ratios of the considered modes, consistent with the respective local elastic response spectrum as given in prEN 1998-1:2004, **3.2.2.2**.

(3) The correlation coefficients $\rho_{u_k u_l}$, between peak ground displacements, and $\rho_{s_{ki} s_{lj}}$, between normalised modal responses, are given by:

$$\begin{aligned}
\rho_{u_k u_l} &= \frac{1}{\sigma_{u_k}\sigma_{u_l}} \int_{-\infty}^{\infty} G_{u_k u_l}(\omega) d\omega \\
\rho_{s_{ki} s_{lj}} &= \frac{1}{\sigma_{s_{ki}}\sigma_{s_{lj}}} \int_{-\infty}^{\infty} H_i(\omega) H_j(-\omega) G_{\ddot{u}_k \ddot{u}_l}(\omega) d\omega \\
\sigma_{u_k}^2 &= \int_{-\infty}^{\infty} G_{u_k u_k}(\omega) d\omega \\
\sigma_{s_{ki}}^2 &= \int_{-\infty}^{\infty} \left|H_i(\omega)\right|^2 G_{\ddot{u}_k \ddot{u}_k}(\omega) d\omega
\end{aligned} \qquad \text{(D.20)}$$

where $G_{u_k u_l}(\omega)$ is the kl-term of the power spectral densities matrix of the ground displacement processes, related to the corresponding one for the acceleration processes by: $\mathbf{G}_{\mathbf{uu}}(\omega) = \frac{1}{\omega^4}\mathbf{G}_{\ddot{\mathbf{u}}\ddot{\mathbf{u}}}(\omega)$; $H_i(\omega)$ is the frequency transfer function of the normalised modal displacement, given by:

$$H_i(\omega) = \frac{1}{\omega_i^2 - \omega^2 + i2\xi_i\omega_i\omega} \qquad \text{(D.21)}$$

(4) In order to evaluate the integrals in Expression (D.20) the power spectral densities should be related to the response spectra that represent the information supposed to be available to the user of the present approach. The following approximate expression, slightly adjusted from that proposed in [4], can be used to relate response and power spectrum at any station:

[6] In Expression (D.19) one contribution has been omitted, which accounts for the correlation between the u -terms and the s -terms, i.e. $\rho_{u_k s_{lj}}$. Numerical analyses show that this contribution is insignificant and can be disregarded.

$$G_{iiii}(\omega)=\omega^2\left(\frac{2\xi\omega}{\pi}+\frac{4}{\pi\tau}\right)\left[\frac{D(\omega,\xi)}{2.5}\right]^2 \qquad \omega\geq 0 \tag{D.22}$$

where τ is the duration of the stationary part of the ground motion to be taken consistently with the seismic event underlying the establishment of a_g.

(5) In practical cases, when local soil conditions differ from one support to another, the effect of this difference tends to dominate over the other two phenomena generating loss of correlation. Numerical analyses show in addition that the consideration of the third term $\gamma_{ij,3}(\omega)$ in the coherency function has small influence on the results so that it can be, in approximation, set to zero. Based on these considerations and taking into account the approximate character of the described response spectrum procedure, a significant simplification is to consider a diagonal matrix $\mathbf{G}(\omega)$, i.e. to consider the structure as subjected to a vector of independent ground motion processes, each one characterised by its own power spectral density function. Correspondingly, Expression (D.19) simplifies to:

$$\mu_{z_{\max}}=\sqrt{\sum_{k=1}^{m}a_k^2u_{k,\max}^2+\sum_{k=1}^{m}\sum_{i=1}^{n}\sum_{j=1}^{n}b_{ki}b_{kj}\rho_{s_{ki}s_{kj}}D_k(\omega_i,\xi_i)D_k(\omega_j,\xi_j)} \tag{D.23}$$

References

[1] Luco, J. and Wong, H., 1986, "Response of a rigid foundation to a spatially random ground motion" Earth. Eng. Struct. Dyn., 14: 891-908

[2] Der Kiureghian, A., 1996, "A coherency model for spatially varying ground motions" Earth. Eng. Struct. Dyn., 25: 99-111

[3] Shinozuka, M., 1972, "Monte Carlo solution of structural dynamics" Comp. Struct., 2: 855-874

[4] Der Kiureghian, A. and Neuenhofer, A., 1992, "Response spectrum method for multi-support seismic excitations" Earth Eng. Struct. Dyn., 21: 713-740

ANNEX E (INFORMATIVE)
PROBABLE MATERIAL PROPERTIES AND PLASTIC HINGE DEFORMATION CAPACITIES FOR NON-LINEAR ANALYSES

E.1 General

(1) This Annex provides guidance for the selection of the probable material properties and for the estimation of the deformation capacities of the plastic hinges. Both are intended for use exclusively for non-linear analyses in accordance with **4.2.4** and **4.2.5**.

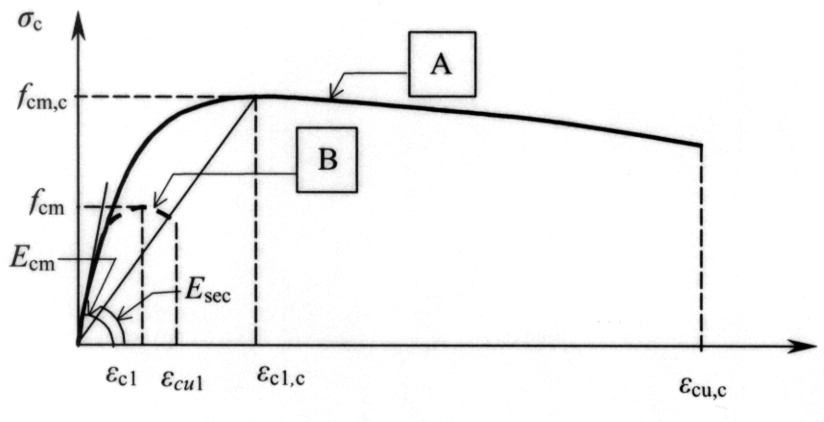
E.2 Probable material properties

E.2.1 Concrete

(1) Mean values f_{cm}, E_{cm} in accordance with EN 1992-1-1: 2004, Table 3.1 should be used.

(2) For unconfined concrete the stress-strain relationship for non-linear analysis specified in EN 1992-1-1:2004, **3.1.5(1)**, should be used, with the values of strains ε_{c1} and ε_{cu1} as specified in Table 3.1 of the same standard.

(3) For confined concrete the following procedure may be used, as an alternative to EN 1992-1-1:2004, **3.1.9** (see Figure E.1):



Key
A – Confined concrete
B – Unconfined concrete

Figure E.1: Stress-strain relationship for confined concrete

NOTE This model of confined concrete properties is compatible with the values for Φ_u and L_p given in expressions (E.18) and (E.19) respectively.

(a) Concrete stress σ_c:

$$\frac{\sigma_c}{f_{cm,c}} = \frac{xr}{r-1+x^r} \tag{E.1}$$

where:

$$x = \frac{\varepsilon_c}{\varepsilon_{c1,c}} \tag{E.2}$$

$$r = \frac{E_{cm}}{E_{cm} - E_{sec}} \tag{E.3}$$

secant modulus to ultimate strength:

$$E_{sec} = \frac{f_{cm,c}}{\varepsilon_{c1,c}} \tag{E.4}$$

ultimate strength:

$$f_{cm,c} = f_{cm}\lambda_c \tag{E.5}$$

$$\lambda_c = 2{,}254\sqrt{1+7{,}94\frac{\sigma_e}{f_{cm}}} - \frac{2\sigma_e}{f_{cm}} - 1{,}254 \tag{E.6}$$

strain at ultimate strength:

$$\varepsilon_{c1,c} = 0{,}002\left[1+5\left(\frac{f_{cm,c}}{f_{cm}}-1\right)\right] \tag{E.7}$$

(b) Effective confining stress σ_e:

σ_e is the effective confining stress acting in both transverse directions 2 and 3 ($\sigma_e = \sigma_{e2} = \sigma_{e3}$). This stress may be estimated on the basis of the ratio of confining reinforcement ρ_w, as defined in **6.2.1.2** or **6.2.1.3**, and its probable yield stress f_{ym} as follows:

– For circular hoops or spirals:

$$\sigma_e = \frac{1}{2}\alpha\rho_w f_{ym} \tag{E.8}$$

– For rectangular hoops or ties:

$$\sigma_e = \alpha\rho_w f_{ym} \tag{E.9}$$

where α is the confinement effectiveness factor (see EN 1998-1: 2004, **5.4.3.2.2**)

For bridge piers confined in accordance with the detailing rules of **6.2.1** and with a minimum dimension $b_{min} \cong 1{,}0$ m, the value $\alpha \cong 1{,}0$ may be assumed.

NOTE If, in the case of orthogonal hoops, the values of ρ_w in the two transverse directions are not equal ($\rho_{w2} \neq \rho_{w3}$), the effective confining stress may be estimated as $\sigma_e = \sqrt{\sigma_{e2}\sigma_{e3}}$.

(c) Ultimate concrete strain $\varepsilon_{cu,c}$

This strain should correspond to the first fracture of confining hoop reinforcement. Unless otherwise substantiated, it may be assumed as follows:

$$\varepsilon_{cu,c} = 0{,}004 + \frac{1{,}4\rho_s f_{ym}\varepsilon_{su}}{f_{cm,c}} \tag{E.10}$$

where:

$\rho_s = \rho_w$ for circular spirals or hoops

$\rho_s = 2\rho_w$ for orthogonal hoops, and

$\varepsilon_{su} = \varepsilon_{um}$ is the mean value of the reinforcement steel elongation at maximum force (see EN 1992-1-1:2004, **3.2.2.2**)

E.2.2 Reinforcement steel

(1) In the absence of relevant information on the specific steel for the project, the following values may be used:

$$\frac{f_{ym}}{f_{yk}} = 1{,}15 \tag{E.11}$$

$$\frac{f_{tm}}{f_{tk}} = 1{,}20 \tag{E.12}$$

$$\varepsilon_{su} = \varepsilon_{uk} \tag{E.13}$$

E.2.3 Structural steel

(1) In the absence of relevant information on the specific steel for the project, the following values may be used:

$$\frac{f_{ym}}{f_{yn}} = 1{,}25 \tag{E.14}$$

$$\frac{f_{um}}{f_{un}} = 1{,}30 \tag{E.15}$$

where f_{yn} and f_{un} are the nominal values of the yield and ultimate tensile strength respectively.

E.3 Rotation capacity of plastic hinges

E.3.1 General

(1) In general the rotation capacity of plastic hinges, $\theta_{p,u}$ (see **4.2.4.4(2)c**) should be evaluated on the basis of laboratory tests, satisfying the conditions of **2.3.5.2(3)**, that have been carried out on similar components. This applies for the deformation capacities of tensile members or of plastic shear mechanisms used in eccentric structural steel bracings.

(2) The similarity mentioned above refers to the following aspects of the components where relevant:

- geometry of the component
- loading rate
- ratios between action effects (bending moment, axial force, shear)
- reinforcement configuration (longitudinal and transverse reinforcement, including confinement), for reinforced concrete components
- local and/or shear buckling conditions for steel components

(3) In the absence of specific justification based on actual data, the reduction factor $\gamma_{R,p}$ of expression (4.21) may be assumed as $\gamma_{R,p}$ = 1,40.

E.3.2 Reinforced concrete

(1) In the absence of appropriate laboratory test results, as mentioned in **E.3.1**, the plastic rotation capacity $\theta_{p,u}$, and the total chord rotation θ_u of plastic hinges (see Figure 2.4) may be estimated on the basis of the ultimate curvature Φ_u and the plastic hinge length L_p (see Figure E.2), as follows:

$$\theta_u = \theta_y + \theta_{p,u} \tag{E.16a}$$

$$\theta_{p,u} = (\Phi_u - \Phi_y)L_p(1 - \frac{L_p}{2L}) \tag{E.16b}$$

where:

L is the distance from the end section of the plastic hinge to the point of zero moment in the pier

Φ_y is the yield curvature

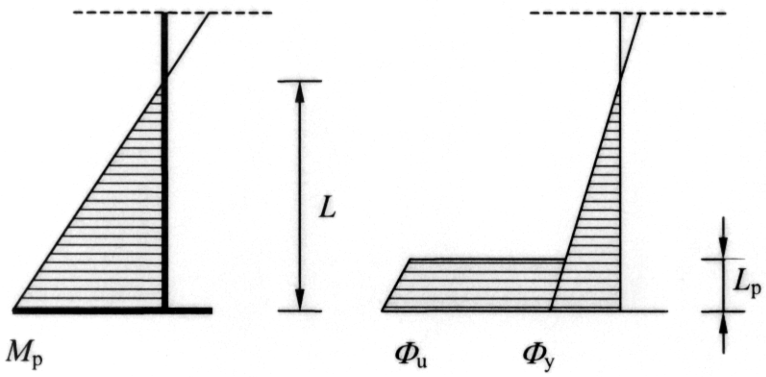


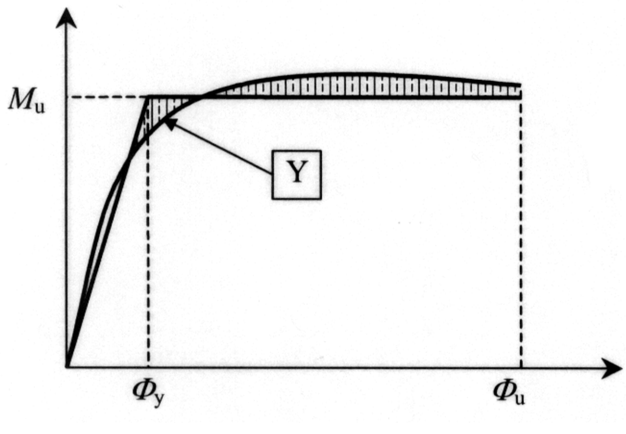
Figure E.2: Φ_y and Φ_u

For linear variation of the bending moment, the yield rotation θ_y may be assumed as:

$$\theta_y = \frac{\Phi_y L}{3} \tag{E.17}$$

(2) Both Φ_y and Φ_u should be determined by means of a moment curvature analysis of the section under the axial load corresponding to the design seismic combination (see also **(4)**). When $\varepsilon_c \geq \varepsilon_{cu1}$, only the confined concrete core section should be taken into an account.

(3) Φ_y should be evaluated by idealising the actual M-Φ diagram by a bilinear diagram of equal area beyond the first yield of reinforcement, as shown in Figure E.3.



Key

Y – Yield of first bar

Figure E.3: Definition of Φ_y

(4) The ultimate curvature Φ_u at the plastic hinge of the member should be taken as:

$$\Phi_u = \frac{\varepsilon_s - \varepsilon_c}{d} \tag{E.18}$$

where

d is the effective section depth

ε_s and ε_c are the reinforcement and concrete strains respectively (compressive strains negative), derived from the condition that either of the two or both have reached the following ultimate values:

- ε_{cu1} for the compression strain of unconfined concrete (see EN 1992-1-1:2004, Table 3.1)
- $\varepsilon_{cu,c}$ for the compression strain of confined concrete (see **E.2.1(3)(c)** or EN 1992-1-1: 2004, **3.1.9(2)**)
- ε_{su} for the tensile strain of reinforcement (see **E.2.1(3)(c)**)

(5) For a plastic hinge occurring at the top or the bottom junction of a pier with the deck or the foundation body (footing or pile cap), with longitudinal reinforcement of characteristic yield stress f_{yk} (in MPa) and bar diameter d_{bL}, the plastic hinge length L_p may be assumed as follows:

$$L_p = 0{,}10L + 0{,}015 f_{yk} d_{bL} \tag{E.19}$$

where L is the distance from the plastic hinge section to the section of zero moment, under the seismic action.

(6) The above estimation of the plastic rotation capacity is valid for piers with shear span ratio

$$\alpha_s = \frac{L}{d} \geq 3{,}0 \tag{E.20}$$

For $1{,}0 \leq \alpha_s < 3{,}0$ the plastic rotation capacity should be multiplied by the reduction factor

$$\lambda(\alpha_s) = \sqrt{\frac{\alpha_s}{3}} \tag{E.21}$$

ANNEX F (INFORMATIVE)
ADDED MASS OF ENTRAINED WATER FOR IMMERSED PIERS

(1) Unless otherwise substantiated by calculation, the total effective mass in a horizontal direction of an immersed pier should be assumed equal to the sum of:

- the actual mass of the pier (without allowance for buoyancy);
- the mass of water possibly enclosed within the pier (for hollow piers);
- the added mass m_a of externally entrained water per unit length of immersed pier.

(2) For piers of circular cross-section of radius R, m_a may be estimated as:

$$m_a = \rho \pi R^2 \tag{F.1}$$

where ρ is the water density.

(3) For piers of elliptical section (see Figure F1) with axes $2a_x$ and $2a_y$ and horizontal seismic action at an angle θ to the x-axis of the section, m_a may be estimated as:

$$m_a = \rho \pi \left(a_y^2 \cos^2\theta + a_x^2 \sin^2\theta\right) \tag{F.2}$$

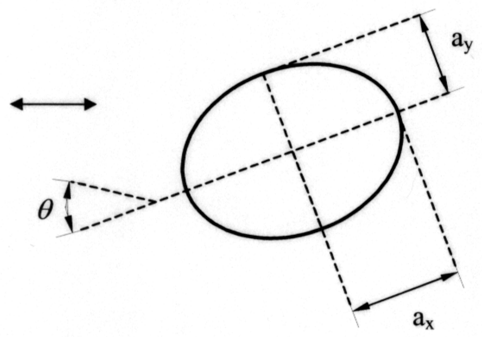


Figure F.1: Definition of dimensions of elliptical pier section

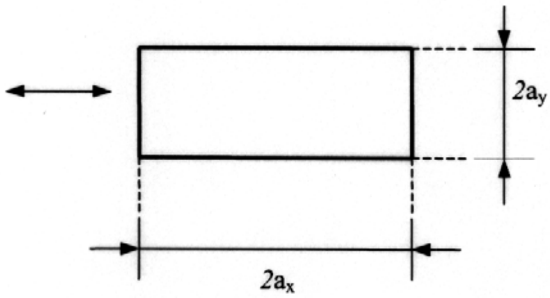


Figure F.2: Definition of dimensions of rectangular pier section

(4) For piers of rectangular section with dimensions $2a_x$ by $2a_y$ and for earthquake action in the x-direction (see Figure F.2), m_a may be estimated as:

$$m_a = k\rho\pi a_y^2 \quad \text{(F.3)}$$

where the value of k is taken from Table F.1(linear interpolation is permitted).

Table F.1 Dependence of added mass coefficient of rectangular piers on cross-sectional aspect ratio

a_y/a_x	k
0,1	2,23
0,2	1,98
0,5	1,70
1,0	1,51
2,0	1,36
5,0	1,21
10,0	1,14
∞	1,00

ANNEX G (NORMATIVE) CALCULATION OF CAPACITY DESIGN EFFECTS

G.1 General procedure

(1)P The following procedure shall be applied in general, separately for each of the two horizontal components of the design seismic action with signs + or –:

(2)P Step 1:

Calculation of the design flexural strengths $M_{Rd,h}$ of the sections of the intended plastic hinges, corresponding to the selected horizontal direction of the seismic action (A_E) with the sign considered (+ or –). The strengths shall be based on the actual dimensions of the cross-sections and the final amount of longitudinal reinforcement. The calculation shall consider the interaction with the axial force and possibly with the bending moment in the orthogonal direction, both resulting from the analysis in the design seismic situation of expression (5.4) of **5.5**.

(3)P Step 2:

Calculation of the change of action effects ΔA_C of the plastic mechanism, corresponding to the increase of the moments of the plastic hinges (ΔM_h), from (a) the values due to the permanent actions ($M_{G,h}$) to (b) the overstrength moments of the sections.

$$\Delta M_h = \gamma_o M_{Rd,h} - M_{G,h} \tag{G.1}$$

where γ_o is the overstrength factor specified in **5.3**.

(4) The effects ΔA_C may in general be estimated from equilibrium conditions, while reasonable approximations regarding the compatibility of deformations are acceptable.

(5)P Step 3:

The final capacity design effects A_C shall be obtained by superimposing the change ΔA_C to the permanent action effects A_G

$$A_C = A_G + \Delta A_C \tag{G.2}$$

G.2 Simplifications

(1) Simplifications of the general procedure specified in **G.1** are allowed, as long as **G.1(4)** is satisfied.

(2) When the bending moment due to the permanent actions at the plastic hinge is negligible compared to the moment overstrength of the section ($M_{G,h} << \gamma_0 M_{Rd,h}$), Step 2 in **G.1(3)**P may be replaced by a direct estimation of the effects ΔA_C from the effects A_E of the design seismic action. This is usually the case in the transverse direction of the piers, or in both directions when the piers are hinged to the deck. In such cases the capacity design shear of pier "*i*" may be estimated as follows:

$$V_{\mathrm{C,i}} = \Delta V_{\mathrm{i}} = \frac{\gamma_{\mathrm{o}} M_{\mathrm{Rd,h,I}}}{M_{\mathrm{E,i}}} V_{\mathrm{E,i}} \tag{G.3}$$

and the capacity design effects on the deck and on the abutments may be estimated from the relationship:

$$\Delta A_{\mathrm{C}} \cong \frac{\Sigma V_{\mathrm{C,i}}}{\Sigma V_{\mathrm{E,i}}} A_{\mathrm{E}} \tag{G.4}$$

ANNEX H (INFORMATIVE)
STATIC NON-LINEAR ANALYSIS (PUSHOVER)

H.1 Analysis directions, reference point and target displacements

(1) The non-linear static analysis specified in **4.2.5** should be carried out in the following two horizontal directions:

- the longitudinal direction *x*, as defined by the centres of the two end-sections of the deck.
- the transverse direction *y*, that should be assumed to be orthogonal to the longitudinal direction.

(2) The reference point should be the centre of mass of the deformed deck.

(3) In each of the two horizontal directions *x* and *y*, defined in **(1)** above, a static non-linear analysis in accordance with **4.2.5** should be carried out, until the following target displacements of the reference point are reached:

- in x-direction (longitudinal):

$$d_{\mathrm{T,x}} = d_{\mathrm{Ex}} \tag{H.1}$$

- in y-direction: (transverse) :

$$d_{\mathrm{T,y}} = d_{\mathrm{Ey}} \tag{H.2}$$

where:

$d_{\mathrm{E,x}}$ is the displacement in the *x*-direction, at the centre of mass of the deformed deck, resulting from equivalent linear multi-mode spectrum analysis (in accordance with **4.2.1.3**) assuming q = 1,0 due to E_{x} "+" 0,3E_{y}. The spectrum analysis should be carried out using effective stiffness of ductile members as specified in **2.3.6.1**.

$d_{\mathrm{E,y}}$ is the displacement in *y*-direction at the same point calculated similarly to $d_{\mathrm{E,x}}$ above.

H.2 Load distribution

(1) The horizontal load increments $\Delta F_{\mathrm{i,j}}$ assumed acting on lumped mass M_{i}, in the direction investigated, at each loading step *j*, should be taken as equal to:

$$\Delta F_{\mathrm{i,j}} = \Delta \alpha_{\mathrm{j}}\, g\, M_{\mathrm{i}}\, \zeta_{\mathrm{i}} \tag{H.3}$$

where:

$\Delta \alpha_{\mathrm{j}}$ is the horizontal force increment, normalized to the weight gM_{i}, applied in step *j*, and

ζ_{i} is a shape factor defining the load distribution along the structure.

(2) Unless a better approximation is used, both of the following distributions should be investigated:

a) *constant along the deck*, where

for the deck

$$\zeta_i = 1 \tag{H.4}$$

and for the piers connected to the deck

$$\zeta_i = \frac{z_i}{z_P} \tag{H.5}$$

where

z_i is the height of point *i* above the foundation of the individual pier and

z_P is the total height of pier *P* (distance from the ground to the centre line of the deck).

b) *proportional to the first mode shape*, where

ζ_i is proportional to the component, in the considered horizontal direction, of the modal displacement at point *i*, of the first mode, in the same direction. The mode with the largest participation factor in the considered direction, should be taken as first mode in this direction. Especially for the piers, the following approximation may be used alternatively

$$\zeta_i = \zeta_{T,P} \frac{z_i}{z_P} \tag{H.6}$$

where $\zeta_{T,P}$ is the value of ζ corresponding to the joint connecting the deck and pier *P*.

H.3 Deformation demands

(1) Deformation demands at each plastic hinge should be verified using expression (4.20) where θ_{Ed} denotes the maximum chord rotation demands, when the target displacement is reached (see **4.2.4.4(2)**c).

(2) In each direction, the total deformation at the first loading step when the two sides of expression (4.20) become equal at any plastic hinge, defines the design ultimate deformation state of the bridge. If, at this state, the displacement of the reference point is less than the target displacement in the relevant direction, the design should be considered unsatisfactory and should be modified.

NOTE 1: Increasing the longitudinal reinforcement of the critical plastic hinge sections, within the limits of constructability, leads primarily to a corresponding increase of the effective stiffness of the ductile members (in accordance with **2.3.6.1**) and consequently to a reduction of the target displacement in accordance with **H.1(3)**, and of the deformation demands θ_{Ed} of **H.3(1)**. In general increasing the dimensions of the sections of the ductile members leads to a reduction of the deformation demands, as well as to an increase in the deformation capacities of the members.

NOTE 2: A design procedure of the ductile members along these lines involves only deformation/displacement verifications (no strength verifications). However, non-ductile failure verifications (shear) of both the ductile and non-ductile members are carried out through strength verifications, in accordance with **4.2.4.4(2)(e)**.

(3) In the longitudinal direction of an essentially straight bridge, the displacements of all pier heads connected to the deck are practically equal to the displacement of the reference point. In this case the deformation demands of the plastic hinges can be assessed directly from the target displacement.

H.4 Deck verification

(1) It should be verified that no significant yielding, in accordance with **5.6.3.6(2)** and **5.6.3.6(3)**, occurs in the deck before the target displacement is reached (see **4.2.4.4(2)d**).

(2) Up-lift of all bearings at the same support, before the target displacement is reached, should be avoided. Up-lift of individual bearings of the same support, before the target displacement is reached, is acceptable, if it has no detrimental effect on the bearings.

H.5 Verification of non-ductile failure modes and of the foundation soil

(1) All members should be verified against non-ductile failure modes (shear), in accordance with **4.2.4.4(2)e**, using the force distribution corresponding to the target displacement as design actions. The same applies for the verification of the foundation soil.

ANNEX J (NORMATIVE) VARIATION OF DESIGN PROPERTIES OF SEISMIC ISOLATOR UNITS

J.1 Factors causing variation of design properties

(1) The assessment of Upper Bound Design Properties and Lower Bound Design Properties (UBDPs and LBDPs) required for the design of the isolating system in accordance with **7.5.2.4,** should be established by evaluating the influence of the following factors on each property:

- f_1: ageing (including corrosion);
- f_2: temperature (minimum isolator design temperature $T_{min,b}$);
- f_3: contamination;
- f_4: cumulative travel (wear).

In general the design properties of cyclic response influenced by the above factors are the following (see Figure 7.1 and Figure 7.3).

- The post elastic stiffness K_p.
- The force at zero displacement F_o.

(2) The minimum isolator temperature for the seismic design situation, $T_{min,b}$, should correspond to the climatic conditions of the bridge location.

NOTE: The value of the minimum isolator temperature for use in a country in the seismic design situation may be found in its the National Annex. The recommended value is as follows:

$T_{min,b} = \psi_2 T_{min} + \Delta T_1$

where

T_{min} is the value of the minimum shade air temperature at the bridge location having an annual probability of (negative) exceedance of 0.02, in accordance with EN 1990-1-5:2004, **6.1.3.2**.

ψ_2 = 0.50 is the combination factor for thermal actions for seismic design situation, in accordance with EN 1990:2002 – Annex **A2** and

ΔT_1 takes the following values depending on the material of the bridge deck, in accordance with Figure 6.1 of EN 1991-1-5: 2003..

Table J.1N: Value of ΔT_1 for the determination of the minimum isolator temperature

Deck	Concrete	Composite	Steel
ΔT_1 (°C)	7.5	5.0	-2.5

J.2 Evaluation of the variation

(1) In general the effect of each of the factors f_i $(i$ = 1 to 4) listed in **J.1** on each design property, should be evaluated by comparing: (a) the maximum and minimum values ($maxDP_{fi}$ and $minDP_{fi}$) of the design property, resulting from the influence of factor f_i,, to (b) the maximum and minimum nominal values ($maxDP_{nom}$ and $minDP_{nom}$) respectively, of the same property, as measured by Prototype tests. The following ratios should be the established for the influence of each factor f_i on the investigated design property.

$$\lambda_{\max,\mathrm{f}i} = \frac{maxDP_{\mathrm{f}i}}{maxDP_{\mathrm{nom}}} \tag{J.2}$$

$$\lambda_{\min,\mathrm{f}i} = \frac{minDP_{\mathrm{f}i}}{minDP_{\mathrm{nom}}} \tag{J.3}$$

NOTE 1: Informative Annex K provides guidance on prototype (or type) tests in cases where prEN 15129:200X ("Anti-seismic devices" does not include detailed requirements for such tests

NOTE 2: The values to be ascribed to the λ-factors for use in a country may be found in its National Annex. Recommended values/guidance for commonly used isolators, i.e. special elastomeric bearings, lead-rubber bearings, sliding isolating units and hydraulic viscous dampers, is given in Informative Annex **JJ**.

(2) The effective UBDP used in the design should be estimated as follows:

$$UBDP = maxDP_{\mathrm{nom}}.\lambda_{\mathrm{U,f1}}.\lambda_{\mathrm{U,f2}} \ldots \lambda_{\mathrm{U,f5}} \tag{J.4}$$

with modification factors

$$\lambda_{\mathrm{U,f1}} = 1 + (\lambda_{\max,\mathrm{fi}} - 1)\,\psi_{\mathrm{fi}} \tag{J.5}$$

where, the combination factors ψ_{fi} account for the reduced probability of simultaneous occurrence of the maximum adverse effects of all factors and should be assumed in accordance with Table J.2:

Table J.2: Combination factors ψ_{fi}

Importance Class	ψ_{fi}
III	0,90
II	0,70
I	0,60

(3) In general, for the effective LBDP (and relevant modification factors $\lambda_{\mathrm{L,fi}}$) a similar format as that of expressions (J.4) and (J.5) should be used, in conjunction with $\lambda_{\min,\mathrm{fi}}$. However for the commonly used elastomeric and friction bearings, it may be assumed in general that:

$$\lambda_{\min,\,\mathrm{fi}} = 1 \tag{J.6}$$

and therefore

$$LBDP = minDP_{\mathrm{nom}} \tag{J.7}$$

(4) For hydraulic dampers and in the absence of specific tests, it may be assumed that:

UBDP = $maxDP_{\mathrm{nom}}$

LBDP = $minDP_{\mathrm{nom}}$

ANNEX JJ (INFORMATIVE)
λ-FACTORS FOR COMMON ISOLATOR TYPES

JJ.1 λ_{max}-*values* for elastomeric bearings

(1) Unless different values are substantiated by appropriate tests, the λ_{max}-*values* specified in following Tables JJ.1 to JJ.4 may be used for estimation of the UBDP.

Table JJ.1: f_1 - Ageing

Component	$\lambda_{max,\ f1}$ for	
	K_p	F_o
LDRB	1,1	1,1
HDRB1	1,2	1,2
HDRB2	1,3	1,3
Lead core	-	1,0

with the following designation for the rubber components:

LDRB: Low damping rubber bearing with shear modulus larger than 0,5 MPa

HDRB1: High damping rubber bearing with $\xi_{eff} \leq 0,15$ and shear modulus larger than 0,5 MPa

HDRB2: High damping rubber bearing with $\xi_{eff} > 0,15$ or shear modulus larger than 0,5 MPa

Lead core: Lead core for Lead rubber bearings (LRB)

Table JJ.2: f_2 - Temperature

Design Temperature $T_{min,b}$ (°C)	$\lambda_{max,\ f2}$ for					
	K_p			F_o		
	LDRB	HDRB1	HDRB2	LDRB	HDRB1	HDRB2
20	1,0	1,0	1,0	1,0	1,0	1,0
0	1,3	1,3	1,3	1,1	1,1	1,2
-10	1,4	1,4	1,4	1,1	1,2	1,4
-30	1,5	2,0	2,5	1,3	1,4	2,0

$T_{min,b}$ is the minimum isolator temperature for the seismic design situation, corresponding to the bridge location (see **(2)** of **J.1** of **Annex J**).

Table JJ.3: f_3 - Contamination

$\lambda_{max,f3} = 1,0$

Table JJ.4: f_4 – Cumulative travel

Rubber	$\lambda_{max,f4} = 1,0$
Lead core	To be established by test

JJ.2 λ_{max}-*values* for sliding isolator units

(1) Unless different values are substantiated by appropriate test results, the λ_{max}-*values* specified in the following Tables JJ.5 to JJ.8 may be used for the estimation of

the maximum force at zero displacement F_o corresponding to the UBDP. The values given for unlubricated PTFE may be taken to apply also for Friction Pendulum bearings.

Table JJ.5: f_1 - Ageing

<table>
<tr><td></td><td colspan="6">$\lambda_{max,f1}$</td></tr>
<tr><td>Component</td><td colspan="2">Unlubricated PTFE</td><td colspan="2">Lubricated PTFE</td><td colspan="2">Bimetallic Interfaces</td></tr>
<tr><td>Environment</td><td>Sealed</td><td>Unsealed</td><td>Sealed</td><td>Unsealed</td><td>Sealed</td><td>Unsealed</td></tr>
<tr><td>Normal</td><td>1,1</td><td>1,2</td><td>1,3</td><td>1,4</td><td>2,0</td><td>2,2</td></tr>
<tr><td>Severe</td><td>1,2</td><td>1,5</td><td>1,4</td><td>1,8</td><td>2,2</td><td>2,5</td></tr>
</table>

The values in Table JJ.5 refer to the following conditions:

– Stainless steel sliding plates are assumed

– Unsealed conditions are assumed, to allow exposure of the sliding surfaces to water and salt

– Severe environment includes marine and industrial conditions

Values for bimetallic interfaces apply to stainless steel and bronze interface.

Table JJ.6: f_2 - Temperature

<table>
<tr><td>Design Temperature</td><td colspan="3">$\lambda_{max,f2}$</td></tr>
<tr><td>$T_{min,b}$ (° C)</td><td>Unlubricated PTFE</td><td>Lubricated PTFE</td><td>Bimetallic Interfaces</td></tr>
<tr><td>20</td><td>1,0</td><td>1,0</td><td rowspan="4">To be established by test</td></tr>
<tr><td>0</td><td>1,1</td><td>1,3</td></tr>
<tr><td>-10</td><td>1,2</td><td>1,5</td></tr>
<tr><td>-30</td><td>1,5</td><td>3,0</td></tr>
</table>

Table JJ.7: f_3 - Contamination

<table>
<tr><td></td><td colspan="3">$\lambda_{max,f3}$</td></tr>
<tr><td>Installation</td><td>Unlubricated PTFE</td><td>Lubricated PTFE</td><td>Bimetallic Interfaces</td></tr>
<tr><td>Sealed, with stainless steel surface facing down</td><td>1,0</td><td>1,0</td><td>1,0</td></tr>
<tr><td>Sealed, with stainless steel surface facing up</td><td>1,1</td><td>1,1</td><td>1,1</td></tr>
<tr><td>Unsealed, with stainless steel surface facing down</td><td>1,2</td><td>3,0</td><td>1,1</td></tr>
</table>

The values in Table JJ.7 refer to the following conditions:

– Sealing of bearings is assumed to offer contamination protection under all serviceability conditions

Table JJ.8: f_4 – Cumulative travel

	$\lambda_{max, f4}$		
Cumulative Travel (km)	Unlubricated PTFE	Lubricated PTFE	Bimetallic Interfaces
≤ 1,0	1,0	1,0	To be established by test
1,0 < and ≤ 2	1,2	1,0	To be established by test

.

ANNEX K (INFORMATIVE) TESTS FOR VALIDATION OF DESIGN PROPERTIES OF SEISMIC ISOLATOR UNITS

K.1 Scope

(1) This Informative Annex is intended to provide guidance on prototype (or type) testing in cases where prEN 15129:200X ("Anti-seismic devices" does not include detailed requirements for such testing.

(2) The range of values of the deformation characteristics and damping values of the isolator units used in the design and analysis of seismic-isolated bridges may be validated by the tests described in this Annex. These tests are not intended for use as quality control tests.

(3) The prototype tests specified in **K.2** aim to establish or validate the range of nominal design properties of the isolator units assumed in the design. These tests in general may be project specific. However, available results of tests performed on specimens of similar type and size and with similar values of design parameters are acceptable.

(4) The purpose of the tests of **K.3** is to substantiate properties of the isolators, which are usually not project specific.

K.2 Prototype tests

K.2.1 General

(1) The tests should be performed on a minimum of two specimens. Specimens should not be subjected to any lateral or vertical loading prior to prototype testing.

(2) In general, full size specimens should be used. The competent authority may allow performance of certain tests on reduced scale specimens, only when existing testing facilities do not have the capacity required for testing full-size specimens.

(3) When reduced scale specimens are used, they should be of the same material and type, geometrically similar to the full-size specimens, and should be manufactured with the same process and quality control.

K.2.2 Sequence of tests

(1) The following sequence of tests should be performed for the prescribed number of cycles, at a vertical load equal to the average permanent load, on all isolator units of a common type and size.

T_1 Three fully reversed cycles at plus and minus the maximum thermal displacement at a test velocity not less than 0,1 mm/min.

T_2 Twenty fully reversed cycles of loading at plus and minus the maximum non-seismic design reaction, at an average test frequency of 0,5 Hz. Following the cyclic testing, the load should be held on the specimen for 1 minute.

T_3 Five fully reversed cycles at the total design seismic displacement.

T_4 Fifteen fully reversed cycles at the total design displacement. The cycles may be applied in three groups of five cycles each, with each group separated by idle time to allow for specimen cooling down.

T_5 Repetition of test T_2 but with the number of cycles reduced to three.

T_6 If an isolator unit is also a vertical load-carrying element, then it should also be tested for one fully reversed cycle at the total design seismic displacement under the following vertical loads:

$$1{,}2\ Q_G + |\Delta F_{Ed}|$$

$$0{,}8\ Q_G - |\Delta F_{Ed}|$$

where

Q_G is the permanent load and

ΔF_{Ed} is the additional vertical load due to seismic overturning effects, based on peak response under the design seismic action.

(2) Tests T_3, T_4 and T_6 should be performed at a frequency equal to the inverse of the effective period of the isolating system. Exception from this rule is permitted for isolator units that are not dependent on the rate of loading (the rate of loading has as primary effect the viscous or frictional heating of the specimen). The force displacement characteristics of an isolator unit are considered to be independent of the rate of loading, when there is less than 15% difference on either of the values of F_o and K_p defining the hysteresis loop (see Figure 7.1), when tested for three fully reversed cycles at the design displacement and frequencies in the range of 0,2 to 2 times the inverse of the effective period of the isolating system.

K.2.3 Determination of isolators characteristics

K.2.3.1 Force-displacement characteristics

(1) The effective stiffness of an isolator unit should be calculated for each cycle of loading as follows:

$$K_{eff} = \frac{F_p - F_n}{d_p - d_n} \tag{K.1}$$

where:

d_p and d_n are the maximum positive and maximum negative test displacement, respectively, and

F_p and F_n are the maximum positive and negative forces, respectively, for units with hysteretic and frictional behaviour, or the positive and negative forces corresponding to d_p and d_n, respectively, for units with viscoelastic behaviour.

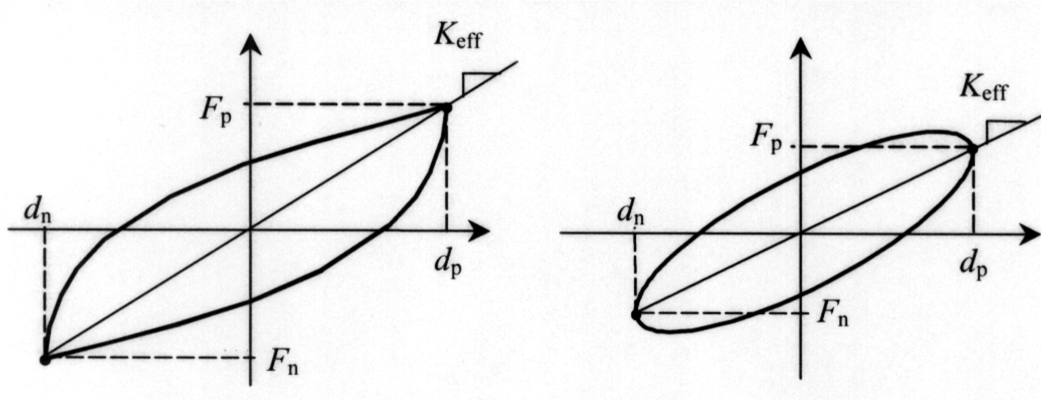


Figure K1: Force-displacement diagrams of tests (Left: hysteretic or friction behaviour; right: viscous behaviour)

K.2.3.2 Damping characteristics

(1) The energy dissipated per cycle E_{Di} of an isolator unit *i,* should be determined for each cycle of loading as the area of the relevant hysteresis loop of the five fully reversed cycles at the total design displacement of test T_3 of **K.2.2**.

K.2.3.3 System adequacy

(1) The performance of the test specimens should be considered as adequate if the following requirements are satisfied:

R_1 except for fluid viscous dampers, the force-displacement plots of all tests specified in **K.2.2** should have a positive incremental force-carrying capacity.

R_2 in test T_1 of **K.2.2** the maximum measured force should not exceed the design value by more than 5%.

R_3 in tests T_2 and T_5 of **K.2.2** the maximum measured displacement should be not less than 90% of the design value.

R_4 in test T_3 of **K.2.2**, the maximum and minimum values of the effective stiffness K_{effi} of isolator unit *i* (and the corresponding force-displacement diagrams), as well as of the energy dissipated per cycle, E_{Di}, should be determined as the maximum and minimum, respectively, of the average of each of the four pairs of consecutive cycles of the test. These nominal properties should be within the range of nominal properties, assumed by the design.

R_5 In test T_4 of **K.2.2**, the ratio of the minimum to the maximum effective stiffness measured in each of the 15 cycles should be not less than 0,7.

R_6 In test T_4 of **K.2.2**, the ratio $\min E_D/\max E_D$ for each of the 15 cycles should not be less than 0,7.

R_7 All vertical load-carrying units should remain stable (i.e. with positive incremental stiffness) during the test $T6$ of **K.2.2**.

R_8 Following the conclusion of the tests, all test specimens should be inspected for evidence of significant deterioration, which may constitute cause for rejection, such as (where relevant):

– Lack of rubber to steel bond
– Laminate placement fault
– Surface rubber cracks wider or deeper than 70% of rubber cover thickness
– Material peeling over more than 5% of the bonded area
– Lack of PTFE to metal bond over more than 5% of the bonded area
– Scoring of stainless steel plate by marks deeper or wider than 0,5 mm and over a length exceeding 20 mm
– Permanent deformation
– Leakage

K.3 Other tests

K.3.1 Wear and fatigue tests

(1) These tests should account for the influence of cumulative travel due to displacements caused by thermal and traffic loadings, over a service life to at least 30 years.

(2) For bridges of normal length (up to about 200 m) and unless a different value is substantiated by calculation, the minimum cumulative travel may be taken as 2000 m.

K.3.2 Low temperature tests

(1) If the isolator units are intended to be used in low temperature areas, with minimum isolator temperature for seismic design $T_{\mathrm{min,b}} < 0^{\mathrm{o}}C$ (see **J.1(2)**), then a test should be performed at this temperature, consisting of five fully reversed cycles at the design displacement, with the remaining conditions as specified in test T_3 of **K.2.2**. The specimen should be kept below freezing for at least two days before the test. The results should be evaluated as specified in R4 of **K.2.3.3(1)**.

(2) In the tests of **K.3.1**, 10% of the travel should be performed under temperature $T_{\mathrm{min,b}}$.

EUROPEAN STANDARD
NORME EUROPÉENNE
EUROPÄISCHE NORM

prEN 1998-3

Doc CEN/TC250/SC8/N388B

English version

Eurocode 8: Design of structures for earthquake resistance

Part 3: Assessment and retrofitting of buildings

DRAFT No 7

Version for translation (Stage 49)

June 2004

CEN

European Committee for Standardization

Comité Européen de Normalisation

Europäisches Komitee für Normung

Central Secretariat: rue de Stassart 36, B1050 Brussels

Contents Page

Foreword

This document (EN 1998-3:200X) 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 1998-1-4:1996.

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

EN 1994 Eurocode 4: Design of composite steel and concrete 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 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 a 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

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

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.

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 (informative).

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 1998-3

Although assessment and retrofitting of existing structures for non-seismic actions is not yet covered by the relevant material-dependent Eurocodes, this Part of Eurocode 8 was specifically developed because:

– For most of the old structures seismic design was not taken into account originally, whereas the non-seismic actions were taken into account, at least by means of traditional construction rules
– Seismic hazard evaluations in accordance with present knowledge may indicate the need for retrofitting campaigns.

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

– The occurrence of earthquakes may create the need for important repairs.

Furthermore, since within the philosophy of Eurocode 8 the seismic design of new structures is based on a certain acceptable degree of structural damage in the event of the design earthquake, criteria for seismic assessment (of structures designed in accordance with Eurocode 8 and subsequently damaged) constitute an integral part of the entire process for seismic structural safety.

In seismic retrofitting situations, qualitative verifications for the identification and elimination of major structural defects are very important and should not be discouraged by the quantitative analytical approach proper to this Part of Eurocode 8. Preparation of documents of more qualitative nature is left to the initiative of the National Authorities.

This Standard addresses the structural aspects of seismic assessment and retrofitting, which is only one component of a broader strategy for seismic risk mitigation that includes pre and/or post-earthquake steps to be taken by competent authorities.

In cases of low seismicity (see EN1998-1, **3.2.1(4)**), this Standard may be adapted to local conditions by appropriate National Annexes.

National annex for EN 1998-3

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 1998-3:200X 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 1998-3:200X through clauses:

Reference	Item
1.1(3)	Informative Annexes A, B and C.
2.1(2)P	Number of Limit States to be considered
2.1(3)P	Return period of seismic actions under which the Limit States should not be exceeded.
2.1(4)P	Simplified provisions
2.2.1(7)P	Partial factors for strengthening materials
3.3.1(4)	Confidence factors
3.4.4(1)	Levels of inspection and testing
4.4.2(1)P	Maximum value of the ratio ρ_{max}/ρ_{min}

1 GENERAL

1.1 Scope

(1)P The scope of Eurocode 8 is defined in EN 1998-1:2004, **1.1.1** and the scope of this Standard is defined in **1.1**. Additional parts of Eurocode 8 are indicated in EN 1998-1:2004, **1.1.3**.

(2) The scope of EN 1998-3 is as follows:

- To provide criteria for the evaluation of the seismic performance of existing individual building structures.
- To describe the approach in selecting necessary corrective measures
- To set forth criteria for the design of retrofitting measures (i.e. conception, structural analysis including intervention measures, final dimensioning of structural parts and their connections to existing structural elements).

NOTE For the purposes of this standard, retrofitting covers both the strengthening of undamaged structures and the repair of earthquake damaged structures.

(3) When designing a structural intervention to provide adequate resistance against seismic actions, structural verifications should also be made with respect to non-seismic load combinations.

(4) Reflecting the basic requirements of EN 1998-1:2004, this Standard covers the seismic assessment and retrofitting of buildings made of the more commonly used structural materials: concrete, steel, and masonry.

NOTE Informative Annexes A, B and C contain additional information related to the assessment of reinforced concrete, steel and masonry buildings, respectively, and to their upgrading when necessary.

(5) Although the provisions of this Standard are applicable to all categories of buildings, the seismic assessment and retrofitting of monuments and historical buildings often requires different types of provisions and approaches, depending on the nature of the monuments.

(6) Since existing structures:

(i) reflect the state of knowledge at the time of their construction,

(ii) possibly contain hidden gross errors,

(iii) may have been submitted to previous earthquakes or other accidental actions with unknown effects,

structural evaluation and possible structural intervention are typically subjected to a different degree of uncertainty (level of knowledge) than the design of new structures. Different sets of material and structural safety factors are therefore required, as well as different analysis procedures, depending on the completeness and reliability of the information available.

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 1998-1 Eurocode 8 - Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings

1.3 Assumptions

(1) Reference is made to EN 1998-1:2004, **1.3**.

(2) The provisions of this Standard assume that the data collection and tests is performed by experienced personnel and that the engineer responsible for the assessment, the possible design of the retrofitting and the execution of work has appropriate experience of the type of structures being strengthened or repaired.

(3) Inspection procedures, check-lists and other data-collection procedures should be documented and filed, and should be referred to in the design documents.

1.4 Distinction between principles and application rules

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

1.5 Definitions

(1) Reference is made to EN 1998-1:2004, **1.5**.

1.6 Symbols

1.6.1 General

(1) Reference is made to EN 1998-1:2004, **1.6**.

(2) Further symbols used in this Standard are defined in the text where they occur.

1.6.2 Symbols used in Annex A

b width of steel straps in steel jacket

b_o and h_o dimension of confined concrete core to the centreline of the hoop

b_i centreline spacing of longitudinal bars

c concrete cover to reinforcement

d effective depth of section (depth to the tension reinforcement)

d' depth to the compression reinforcement

d_{bL} diameter of tension reinforcement

f_c concrete compressive strength (MPa)

f_{cc} confined concrete strength

f_{cd} design value of concrete strength

f_{ctm} concrete mean tensile strength

$f_{fdd,e}$ design value of FRP (fibre-reinforced polymer) effective debonding strength

$f_{fu,W}(R)$ ultimate strength of FRP sheet wrapped around corner with radius R, expression (A.25)

f_y estimated mean value of steel yield strength

f_{yd} design value of yield strength of (longitudinal) reinforcement

$f_{yj,d}$ design value of yield strength jacket steel

f_{yw} yield stress of transverse or confinement reinforcement

h depth of cross-section

$k_b = \sqrt{1{,}5 \cdot (2 - w_f / s_f)/(1 + w_f / 100 \text{ mm})}$ covering coefficient of FRP (fibre-reinforced polymer) strips/sheet

n number of spliced bars along perimeter p

p length of perimeter line in column section along the inside of longitudinal steel

s centreline spacing of stirrups

s_f centreline spacing of FRP (fibre-reinforced polymer) strips (=w_f for FRP sheets)

t_f thickness of FRP (fibre-reinforced polymer) sheet

t_j thickness of steel jacket

x compression zone depth

w_f width of FRP (fibre-reinforced polymer) strip/sheet

z length of section internal lever arm

A_c column cross-section area

A_f = $t_f \cdot w_f \cdot \sin\beta$: horizontally projected cross-section area of FRP (fibre-reinforced polymer) strip/sheet with thickness t_f, width w_f and angle β

A_s cross-sectional area of longitudinal steel reinforcement

A_{sw} cross-sectional area of stirrup

E_f FRP (fibre-reinforced polymer) modulus

L_V=M/V shear span at member end

N axial force (positive for compression)

$V_{R,c}$ shear resistance of member without web reinforcement

$V_{R,max}$ shear resistance as determined by crushing in the diagonal compression strut

V_w contribution of transverse reinforcement to shear resistance

α confinement effectiveness factor

ε_{cu} concrete ultimate strain

ε_{ju} FRP (fibre-reinforced polymer) ultimate strain

$\varepsilon_{su,w}$ ultimate strain of confinement reinforcement

φ_u ultimate curvature at end section

φ_y yield curvature at end section

ν $= N / bhf_c$ (b width of compression zone)

γ_{el} safety factor, greater than 1,0 for primary seismic and 1,0 secondary seismic elements

γ_{fd} partial factor for FRP (fibre-reinforced polymer) debonding

θ strut inclination angle in shear design

θ_y chord rotation at yielding of concrete member

θ_u ultimate chord rotation of concrete member

ρ_d steel ratio of diagonal reinforcement

ρ_f volumetric ratio of FRP (fibre-reinforced polymer)

ρ_s geometric steel ratio

ρ_{sx} $= A_{sx} / b_w s_h$ = ratio of transverse steel parallel to direction x of loading (s_h = stirrup spacing)

ρ_{tot} total longitudinal reinforcement ratio

ρ_{sw} volumetric ratio of confinement reinforcement

ρ_w transverse reinforcement ratio

ω, ω' mechanical reinforcement ratio of tension and compression reinforcement

1.6.3 Symbols used in Annex B

b_{cp} width of the cover plate

b_f flange width

d_c column depth

d_z panel-zone depth between continuity plates

e distance between the plastic hinge and the column face

f_c concrete compressive strength

f_{ct} tensile strength of the concrete

f_{uw} tensile strength of the welds

f_{ywh} yield strength of transverse reinforcement

$f_{y,pl}$ nominal yield strength of each flange

l_{cp} length of the cover plate

t_{cp} is the thickness of the cover plate

t_f thickness

t_{hw} web thickness

w_z panel-zone width between column flanges

A_g gross area of the section

A_{hf} area of the haunch flange

A_{pl} area of each flange

B_S width of the steel flat-bar brace

B width of the composite section

E Young's modulus of the beam

E_B is the elastic modulus of the RC (reinforced concrete) panel;

F_t seismic base shear

H frame height

H_c storey height of the frame

K_φ connection rotation stiffness

I moment of inertia

L beam span

$M_{pb,Rd}$ beam plastic moment

N_d design axial

N_y yield strength of the steel brace

S_x beam elastic (major) modulus;

T_C thickness of the panel

$V_{pl,Rd,b}$ shear force at a beam plastic hinge

Z_b plastic modulus of the beam

Z_e effective plastic modulus of the section at the plastic hinge location

ρ_w ratio of transverse reinforcement

1.7 S.I. Units

(1) Reference is made to EN 1998-1:2004, **1.7**.

2 PERFORMANCE REQUIREMENTS AND COMPLIANCE CRITERIA

2.1 Fundamental requirements

(1)P The fundamental requirements refer to the state of damage in the structure, herein defined through three Limit States (LS), namely Near Collapse (NC), Significant Damage (SD), and Damage Limitation (DL). These Limit States shall be characterised as follows:

LS of Near Collapse (NC). The structure is heavily damaged, with low residual lateral strength and stiffness, although vertical elements are still capable of sustaining vertical loads. Most non-structural components have collapsed. Large permanent drifts are present. The structure is near collapse and would probably not survive another earthquake, even of moderate intensity.

LS of Significant Damage (SD). The structure is significantly damaged, with some residual lateral strength and stiffness, and vertical elements are capable of sustaining vertical loads. Non-structural components are damaged, although partitions and infills have not failed out-of-plane. Moderate permanent drifts are present. The structure can sustain after-shocks of moderate intensity. The structure is likely to be uneconomic to repair.

LS of Damage Limitation (DL). The structure is only lightly damaged, with structural elements prevented from significant yielding and retaining their strength and stiffness properties. Non-structural components, such as partitions and infills, may show distributed cracking, but the damage could be economically repaired. Permanent drifts are negligible. The structure does not need any repair measures.

NOTE The definition of the Limit State of collapse given in this Part 3 of Eurocode 8 is closer to the actual collapse of the building than the one given in EN1998-1:2004 and corresponds to the fullest exploitation of the deformation capacity of the structural elements. The Limit State associated with the 'no collapse' requirement in EN1998-1:2004 is roughly equivalent to the one that is here defined as Limit State of Significant Damage.

(2)P The National Authorities decide whether all three Limit States shall be checked, or two of them, or just one of them.

NOTE The choice of the Limit States will be checked in a country, among the three Limit States defined in **2.1(1)**P, may be found in the National Annex.

(3)P The appropriate levels of protection are achieved by selecting, for each of the Limit States, a return period for the seismic action.

NOTE The return periods ascribed to the various Limit States to be checked in a country may be found in its National Annex. The protection normally considered appropriate for ordinary new buildings is considered to be achieved by selecting the following values for the return periods:
– LS of Near Collapse (NC): 2.475 years, corresponding to a probability of exceedance of 2% in 50 years
– LS of Significant Damage (SD): 475 years, corresponding to a probability of exceedance of 10% in 50 years
– LS of Damage Limitation (DL): 225 years, corresponding to a probability of exceedance of 20% in 50 years.

2.2 Compliance criteria

2.2.1 General

(1)P Compliance with the requirements in **2.1** is achieved by adoption of the seismic action, method of analysis, verification and detailing procedures contained in this part of EN 1998, as appropriate for the different structural materials within its scope (i.e. concrete, steel, masonry).

(2)P Compliance is checked by making use of the full (unreduced, elastic) seismic action as defined in **2.1** for the appropriate return period.

(3)P For the verification of the structural elements a distinction is made between ‘ductile’ and ‘brittle’ ones. The former shall be verified by checking that demands do not exceed the corresponding capacities in terms of deformations. The latter shall be verified by checking that demands do not exceed the corresponding capacities in terms of strengths.

NOTE Information for classifying components/mechanisms as “ductile” or “brittle” may be found in the relevant material-related Annexes.

(4)P Alternatively, a *q*-factor approach may be used, where use is made of a seismic action reduced by a *q*-factor, as indicated in **4.2(3)**P. All structural elements shall be verified by checking that demands due to the reduced seismic action do not exceed the corresponding capacities in terms of strengths evaluated in accordance with **(5)**P.

(5)P For the calculation of the capacities of ductile or brittle elements, where these will be compared with demands for safety verifications in accordance with **(3)**P, mean value properties of the existing materials shall be used as directly obtained from *in-situ* tests and from the additional sources of information, appropriately divided by the confidence factors defined in **3.5**, accounting for the level of knowledge attained. Nominal properties shall be used for new or added materials.

(6)P Some of the existing structural elements may be designated as “secondary seismic”, in accordance with the definitions in EN 1998-1:2004, **4.2.2** **(1)**P, **(2)** and **(3)**. “Secondary seismic” elements shall be verified with the same compliance criteria as primary seismic ones, but using less conservative estimates of their capacity than for the elements considered as “primary seismic”.

(7)P In the calculation of strength capacities of brittle “primary seismic”elements, material strengths shall be divided by the partial factor of the material.

NOTE: The values ascribed to the partial factors for steel, concrete, structural steel, masonry and other materials for use in a country can be found in the National Annex to this standard. Notes to clauses **5.2.4(3), 6.1.3(1), 7.1.3(1)** and **9.6(3)** in EN1998-1: 2004 refer to the values of partial factors for steel, concrete, structural steel and masonry to be used for the design of new buildings in different countries.

2.2.2 Limit State of Near Collapse (NC)

(1)P Demands shall be based on the design seismic action relevant to this Limit State. For ductile and brittle elements demands shall be evaluated based on the results of the analysis. If a linear method of analysis is used, demands on brittle elements shall be modified in accordance to **4.5.1(1)**P.

(2)P Capacities shall be based on appropriately defined ultimate deformations for ductile elements and on ultimate strengths for brittle ones.

(3) The q-factor approach (see **2.2.1(4)**P, **4.2(3)**P) is generally not suitable for checking this Limit State.

NOTE The values of q = 1,5 and 2,0 quoted in **4.2(3)**P for reinforced concrete and steel structures, respectively, as well as the higher values of q possibly justified with reference to the local and global available ductility in accordance with the relevant provisions of EN 1998-1:2004, correspond to fulfilment of the Significant Damage Limit State. If it is chosen to use this approach to check the Near Collapse Limit State, then **2.2.3(3)**P may be applied, with a value of the q-factor exceeding those in **4.2(3)**P by about one-third.

2.2.3 Limit State of Significant Damage (SD)

(1)P Demands shall be based on the design seismic action relevant to this Limit State. For ductile and brittle elements demands shall be evaluated based on the results of the analysis. In case a linear method of analysis is used, demands on brittle elements shall be modified in accordance to **4.5.1(1)**P.

(2)P Capacities shall be based on damage-related deformations for ductile elements and on conservatively estimated strengths for brittle ones.

(3)P In the q-factor approach (see **2.2.1(4)**P, **4.2(3)**P), demands shall be based on the reduced seismic action and capacities shall be evaluated as for non-seismic design situations.

2.2.4 Limit State of Damage Limitation

(1)P Demands shall be based on the design seismic action relevant to this Limit State.

(2)P Capacities shall be based on yield strengths for all structural elements, both ductile and brittle. Capacities of infills shall be based on mean interstorey drift capacity for the infills.

(3)P In the q-factor approach (see **2.2.1(4)**P, **4.2(3)**P), demands shall be based on the reduced seismic action and capacities shall be based on mean interstorey drift capacity for the infills.

3 INFORMATION FOR STRUCTURAL ASSESSMENT

3.1 General information and history

(1)P In assessing the earthquake resistance of existing structures, the input data shall be collected from a variety of sources, including:

– available documentation specific to the building in question,

– relevant generic data sources (e.g. contemporary codes and standards),

– field investigations and,

– in most cases, in-situ and/or laboratory measurements and tests, as described in more detail in **3.2** and **3.4**.

(2) Cross-checks should be made between the data collected from different sources to minimise uncertainties.

3.2 Required input data

(1) In general, the information for structural evaluation should cover the following points from a) to i).

a) Identification of the structural system and of its compliance with the regularity criteria in EN 1998-1:2004, **4.2.3**. The information should be collected either from on site investigation or from original design drawings, if available. In this latter case, information on possible structural changes since construction should also be collected.

b) Identification of the type of building foundations.

c) Identification of the ground conditions as categorised in EN 1998-1:2004, **3.1**.

d) Information about the overall dimensions and cross-sectional properties of the building elements and the mechanical properties and condition of constituent materials.

e) Information about identifiable material defects and inadequate detailing.

f) Information on the seismic design criteria used for the initial design, including the value of the force reduction factor (q-factor), if applicable.

g) Description of the present and/or the planned use of the building (with identification of its importance category, as described in EN 1998-1:2004, **4.2.5**).

h) Re-assessment of imposed actions taking into account the use of the building.

i) Information about the type and extent of previous and present structural damage, if any, including earlier repair measures.

(2)P Depending on the amount and quality of the information collected on the points above, different types of analysis and different values of the confidence factors shall be adopted, as indicated in **3.3**.

3.3 Knowledge levels

3.3.1 Definition of knowledge levels

(1) For the purpose of choosing the admissible type of analysis and the appropriate confidence factor values, the following three knowledge levels are defined:

KL1 : Limited knowledge

KL2 : Normal knowledge

KL3 : Full knowledge

(2) The factors determining the appropriate knowledge level (i.e. KL1, KL2 or KL3) are:

i) *geometry*: the geometrical properties of the structural system, and of such non-structural elements (e.g. masonry infill panels) as may affect structural response.

ii) *details*: these include the amount and detailing of reinforcement in reinforced concrete, connections between steel members, the connection of floor diaphragms to lateral resisting structure, the bond and mortar jointing of masonry and the nature of any reinforcing elements in masonry,

iii) *materials*: the mechanical properties of the constituent materials.

(3) The knowledge level achieved determines the allowable method of analysis (see **4.4**), as well as the values to be adopted for the confidence factors (CF). The procedures for obtaining the required data are given in **3.4**.

(4) The relationship between knowledge levels and applicable methods of analysis and confidence factors is illustrated in Table 3.1. The definitions of the terms ‘visual’, ‘full’, ‘limited’, ‘extended’ and ‘comprehensive’ in the Table are given in **3.4**.

Table 3.1: Knowledge levels and corresponding methods of analysis (LF: Lateral Force procedure, MRS: Modal Response Spectrum analysis) and confidence factors (CF).

Knowledge Level	Geometry	Details	Materials	Analysis	CF
KL1	From original outline construction drawings with sample **visual** survey ***or*** from **full** survey	Simulated design in accordance with relevant practice ***and*** from **limited** *in-situ* inspection	Default values in accordance with standards of the time of construction ***and*** from **limited** *in-situ* testing	LF-MRS	CF_{KL1}
KL2		From incomplete original detailed construction drawings with **limited** *in-situ* inspection ***or*** from **extended** *in-situ* inspection	From original design specifications with **limited** *in-situ* testing ***or*** from **extended** *in-situ* testing	All	CF_{KL2}
KL3		From original detailed construction drawings with **limited** *in-situ* inspection ***or*** from **comprehensive** *in-situ* inspection	From original test reports with **limited** *in-situ* testing ***or*** from **comprehensive** *in-situ* testing	All	CF_{KL3}

NOTE The values ascribed to the confidence factors to be used in a country may be found in its National Annex. The recommended values are CF_{KL1} = 1,35, CF_{KL2} = 1,20 and CF_{KL3} = 1,00.

3.3.2 KL1: Limited knowledge

(1) KL1 corresponds to the following state of knowledge:

i) *geometry*: the overall structural geometry and member sizes are known either (a) from survey; or (b) from original outline construction drawings used for both the original construction and any subsequent modifications. In case (b), a sufficient sample of dimensions of both overall geometry and member sizes should be checked on site; if there are significant discrepancies from the outline construction drawings, a fuller dimensional survey should be performed.

ii) *details*: the structural details are not known from detailed construction drawings and may be assumed based on simulated design in accordance with usual practice at the time of construction; in this case, limited inspections in the most critical elements should be performed to check that the assumptions correspond to the actual situation. Otherwise, more extensive *in*-situ inspection is required.

iii) *materials*: no direct information on the mechanical properties of the construction materials is available, either from original design specifications or from original test reports. Default values should be assumed in accordance with standards at the time of construction, accompanied by limited *in-situ* testing in the most critical elements.

(2) The information collected should be sufficient for performing local verifications of element capacity and for setting up a linear structural analysis model.

(3) Structural evaluation based on a state of limited knowledge should be performed through linear analysis methods, either static or dynamic (see **4.4**).

3.3.3 KL2: Normal knowledge

(1) KL2 corresponds to the following state of knowledge:

i) *geometry*: the overall structural geometry and member sizes are known either (a) from an extended survey or (b) from outline construction drawings used for both the original construction and any subsequent modifications. In case (b), a sufficient sample of dimensions of both overall geometry and member sizes should be checked on site; if there are significant discrepancies from the outline construction drawings, a fuller dimensional survey is required.

ii) *details*: the structural details are known either from extended *in-situ* inspection or from incomplete detailed construction drawings. In the latter case, limited *in-situ* inspections in the most critical elements should be performed to check that the available information corresponds to the actual situation.

iii) *materials*: information on the mechanical properties of the construction materials is available either from extended *in-situ* testing or from original design specifications. In this latter case, limited *in-situ* testing should be performed.

(2) The information collected should be sufficient for performing local verifications of element capacity and for setting up a linear or nonlinear structural model.

(3) Structural evaluation based on this state of knowledge may be performed through either linear or nonlinear analysis methods, either static or dynamic (see **4.4**).

3.3.4 KL3: Full knowledge

(1) KL3 corresponds to the following state of knowledge:

i) *geometry*: the overall structural geometry and member sizes are known either (a) from a comprehensive survey or (b) from the complete set of outline construction drawings used for both the original construction and any subsequent modifications. In case (b), a sufficient sample of both overall geometry and member sizes should be checked on site; if there are significant discrepancies from the outline construction drawings, a fuller dimensional survey is required.

ii) *details*: the structural details are known either from comprehensive *in-situ* inspection or from a complete set of detailed construction drawings. In the latter case, limited *in-situ* inspections in the most critical elements should be performed to check that the available information corresponds to the actual situation.

iii) *materials*: information on the mechanical properties of the construction materials is available either from comprehensive *in-situ* testing or from original test reports. In this latter case, limited *in-situ* testing should be performed.

(2) **3.3.3(2)** applies.

(3) **3.3.3(3)** applies.

3.4 Identification of the Knowledge Level

3.4.1 Geometry

3.4.1.1 Outline construction drawings

(1) The outline construction drawings are those documents that describe the geometry of the structure, allowing for identification of structural components and their dimensions, as well as the structural system to resist both vertical and lateral actions.

3.4.1.2 Detailed construction drawings

(1) The detailed drawings are those documents that describe the geometry of the structure, allowing for identification of structural components and their dimensions, as well as the structural system to resist both vertical and lateral actions. In addition, it contains information about details (as specified in **3.3**).

3.4.1.3 Visual survey

(1) A visual survey is a procedure for checking correspondence between the actual geometry of the structure with the available outline construction drawings. Sample geometry measurements on selected elements should be carried out. Possible structural changes which may have occurred during or after construction should be subjected to a survey as in **3.4.1.4**.

3.4.1.4 Full survey

(1) A full survey is a procedure resulting in the production of structural drawings that describe the geometry of the structure, allowing for identification of structural components and their dimensions, as well as the structural system to resist both vertical and lateral actions.

3.4.2 Details

(1) Reliable non-destructive methods may be adopted in the inspections specified as follows:

3.4.2.1 Simulated design

(1) A simulated design is a procedure resulting in the definition of the amount and layout of reinforcement, both longitudinal and transverse, in all elements participating in the vertical and lateral resistance of the building. The design should be carried out based on regulatory documents and state of the practice used at the time of construction.

3.4.2.2 Limited in-situ inspection

(1) A limited *in-situ* inspection is a procedure for checking correspondence between the actual details of the structure with either the available detailed construction drawings or the results of the simulated design in **3.4.2.1**. This entails performing inspections as indicated in **3.4.4(1)**P.

3.4.2.3 Extended in-situ inspection

(1) An extended *in-situ* inspection is a procedure used when the original detailed construction drawings are not available. This entails performing inspections as indicated in **3.4.4(1)**P.

3.4.2.4 Comprehensive in-situ inspection

(1) A comprehensive *in-situ* inspection is a procedure used when the original detailed construction drawings are not available and when a higher knowledge level is pursued. This entails performing inspections as indicated in **3.4.4(1)**P

3.4.3 Materials

3.4.3.1 Destructive and non-destructive testing

(1) Use of non-destructive test methods (e.g., Schmidt hammer test, etc.) should be considered; however such tests should not be used in isolation, but only in conjunction with destructive tests.

3.4.3.2 Limited in-situ testing

(1) A limited programme of *in-situ* testing is a procedure for complementing the information on material properties derived either from standards at the time of construction, or from original design specifications, or from original test reports. This entails performing tests as indicated in **3.4.4(1)**P. However, if values from tests are lower than default values in accordance with standards of the time of construction, an extended *in-situ* testing is required.

3.4.3.3 Extended in-situ testing

(1) An extended programme of *in-situ* testing is a procedure for obtaining information when neither the original design specification nor the test reports are available. This entails performing tests as indicated in **3.4.4(1)**P.

3.4.3.4 Comprehensive in-situ testing

(1) A comprehensive programme of *in-situ* testing is a procedure for obtaining information when neither the original design specification nor the test reports are available and when a higher knowledge level is pursued. This entails performing tests as indicated in **3.4.4(1)**P.

3.4.4 Definition of the levels of inspection and testing

(1)P The classification of the levels of inspection and testing depend on the percentage of structural elements that have to be checked for details, as well as on the number of material samples per floor that have to taken for testing.

NOTE The amount of inspection and testing to be used in a country may be found in its National Annex. For ordinary situations the recommended minimum values are given in Table 3.2. There might be cases requiring modifications to increase some of them. These cases will be indicated in the National Annex.

Table 3.2: Recommended minimum requirements for different levels of inspection and testing.

	Inspection (of details)	Testing (of materials)
	For each type of primary element (beam, column, wall):	
Level of inspection and testing	Percentage of elements that are checked for details	Material samples per floor
Limited	20	1
Extended	50	2
Comprehensive	80	3

3.5 Confidence factors

(1)P To determine the properties of existing materials to be used in the calculation of the capacity, when capacity is to be compared with demand for safety verification, the mean values obtained from *in-situ* tests and from the additional sources of information, shall be divided by the confidence factor, CF, given in Table 3.1 for the appropriate knowledge level (see **2.2.1(5)**P).

(2)P To determine the properties to be used in the calculation of the force capacity (strength) of ductile components delivering action effects to brittle components/ mechanisms, for use in **4.5.1(1)**P(b), the mean value properties of existing materials obtained from *in-situ* tests and from the additional sources of information, shall be multiplied by the confidence factor, CF, given in Table 3.1 for the appropriate knowledge level.

4 ASSESSMENT

4.1 General

(1) Assessment is a quantitative procedure for checking whether an existing undamaged or damaged building will satisfy the required limit state appropriate to the seismic action under consideration, as specified in **2.1**.

(2)P This Standard is intended for the assessment of individual buildings, to decide on the need for structural intervention and to design the retrofitting measures that may be necessary. It is not intended for the vulnerability assessment of populations or groups of buildings for seismic risk evaluation for various purposes (e.g. for determining insurance risk, for setting risk mitigation priorities, etc.).

(3)P The assessment procedure shall be carried out by means of the general analysis methods specified in EN 1998-1:2004, **4.3**, as modified in this Standard to suit the specific problems encountered in the assessment.

(4) Whenever possible, the method used should incorporate information of the observed behaviour of the same type of building or similar buildings during previous earthquakes.

4.2 Seismic action and seismic load combination

(1)P The basic models for the definition of the seismic motion are those presented in EN 1998-1:2004, **3.2.2** and **3.2.3**.

(2)P Reference is made in particular to the elastic response spectrum specified in EN 1998-1:2004, **3.2.2.2**, scaled to the values of the design ground acceleration established for the verification of the different Limit States. The alternative representations allowed in EN 1998-1:2004, **3.2.3** in terms of either artificial or recorded accelerograms are also applicable.

(3)P In the q-factor approach (see **2.2.1**), the design spectrum for elastic analysis is obtained from the elastic response spectrum specified in EN 1998-1:2004, **3.2.2.2**, as indicated in EN 1998-1:2004, **3.2.2.5**. A value of q = 1,5 and 2,0 for reinforced concrete and steel structures, respectively, may be adopted regardless of the structural type. Higher values of q may be adopted if suitably justified with reference to the local and global available ductility, evaluated in accordance with the relevant provisions of EN 1998-1:2004.

(4)P The design seismic action shall be combined with the other appropriate permanent and variable actions in accordance with EN 1998-1:2004, **3.2.4**.

4.3 Structural modelling

(1)P Based on information collected as indicated in **3.2**, a model of the structure shall be set up. The model shall be such that the action effects in all structural elements can be determined under the seismic load combination given in **4.2**.

(2)P All provisions of EN 1998-1:2004 regarding modelling (EN 1998-1: 2004, **4.3.1**) and accidental torsional effects (EN 1998-1: 2004, **4.3.2**) shall be applied without modifications.

(3) The strength and the stiffness of secondary seismic elements**,** (see **2.2.1(6)**P) against lateral actions may in general be neglected in the analysis.

(4) Taking into account secondary seismic elements in the overall structural model, however, is advisable if nonlinear analysis is applied. The choice of the elements to be considered as secondary seismic may be varied after the results of a preliminary analysis. In no case the selection of these elements should be such as to change the classification of the structure from non regular to regular, in accordance with the definitions in EN 1998-1:2004, **4.2.3**.

(5)P Mean values of material properties shall be used in the structural model.

4.4 Methods of analysis

4.4.1 General

(1) The seismic action effects, to be combined with the effects of the other permanent and variable loads in accordance with the seismic load combination in **4.2(4)**P, may be evaluated using one of the following methods:

- lateral force analysis (linear),
- modal response spectrum analysis (linear),
- non-linear static (pushover) analysis,
- non-linear time history dynamic analysis.

(2)P Except in the *q*-factor approach of **2.2.1(4)**P and **4.2(3)**P, the seismic action to be used shall be the one corresponding to the elastic (*i.e.*, un-reduced by the behaviour factor *q*) response spectrum in EN 1998-1:2004, **3.2.2.2**, or its equivalent alternative representations in EN 1998-1:2004, **3.2.3**, factored by the appropriate importance factor γ_I (see EN 1998-1:2004, **4.2.5**).

(3)P The *q*-factor approach of **2.2.1(4)**P is applicable only to the linear types of analyses, with the seismic action as defined in **4.2(3)**P.

(4) Clause **4.3.3.1(5)** of EN1998-1: 2004 applies.

(5) The above-listed methods of analysis are applicable subject to the conditions specified in **4.4.2** to **4.4.5**, with the exception of masonry structures for which procedures accounting for the peculiarities of this construction typology need to be used.

NOTE Complementary information on these procedures may be found in the relevant material-related Informative Annex.

4.4.2 Lateral force analysis

(1)P The conditions for this method to be applicable are given in EN 1998-1:2004, **4.3.3.2.1**, with the addition of the following:

Denoting by $\rho_i = D_i/C_i$ the ratio between the bending moment demand D_i obtained from the analysis under the seismic load combination, and the corresponding capacity C_i for the i-th primary element of the structure (only considering $\rho_i \geq 1$), and by ρ_{max} and ρ_{min} the maximum and minimum values of ρ_i, respectively, over all primary elements of the structure, the ratio ρ_{max}/ρ_{min} does not exceed a maximum acceptable value in the range of 2 to 3. The ratio ρ_I needs to be evaluated only at the sections around beam-column joints where plastic hinges are expected to form on the basis of the comparison of the sum of beam flexural capacities to that of columns. **4.3(5)**P applies for the calculation of the capacities C_i. For the determination of the bending moment capacities C_i of vertical elements, the value of the axial force may be taken equal to that due to the vertical loads only.

NOTE 1 The value ascribed to this limit of ρ_{max}/ρ_{min} for use in a country (within the range indicated above) may be found in its National Annex. The recommended value is 2,5.

NOTE 2 As an additional condition, the capacity C_i of the "brittle" components should be larger than the corresponding demand D_i, evaluated in accordance with **4.5.1(1)**P, **(2)** and **(3)**. Nonetheless, enforcing it as a criterion for the applicability of linear analysis is redundant, because, in accordance with **2.2.2(2)**P, **2.2.3(2)**P and **2.2.4(2)**P, this condition will ultimately be fulfilled in all elements of the assessed or retrofitted structure, irrespective of the mehod of analysis.

(2)P The method shall be applied as described in EN 1998-1:2004, **4.3.3.2.2**, **4.3.3.2.3** and **4.3.3.2.4**, except that the response spectrum in expression (4.3) shall be the elastic spectrum $S_e(T_1)$ instead of the design spectrum $S_d(T_1)$.

(3)P In the q-factor approach, the method shall be applied as described in EN 1998-1:2004, **4.3.3.2.2**, **4.3.3.2.3** and **4.3.3.2.4**. In this case, the additional conditions listed in the above clause **(1)**P need not be met.

4.4.3 Multi-modal response spectrum analysis

(1)P The conditions of applicability for this method are given in EN 1998-1:2004, **4.3.3.3.1** with the addition of the conditions specified in **4.4.2**.

(2)P The method shall be applied as described in EN 1998-1:2004, **4.3.3.3.2/3**, using the elastic response spectrum $S_e(T_1)$.

(3)P In the q-factor approach, the method shall be applied as described in EN 1998-1:2004, **4.3.3.2.2**, **4.3.3.2.3** and **4.3.3.2.4**. In this case, the additional conditions listed in the above clause **(1)**P need not be met.

4.4.4 Nonlinear static analysis

(1)P Nonlinear static (pushover) analysis is a non-linear static analysis under constant gravity loads and monotonically increasing horizontal loads.

(2)P Buildings not conforming with the criteria of EN 1998-1:2004, **4.3.3.4.2.1(2)**, **(3)** for regularity in plan shall be analysed using a spatial model.

(3)P For buildings conforming with the regularity criteria of EN 1998-1:2004, **4.2.3.2** the analysis may be performed using two planar models, one for each main horizontal direction of the building.

4.4.4.1 Lateral loads

(1) At least two vertical distributions of lateral loads should be applied:

- a “uniform” pattern, based on lateral forces that are proportional to mass regardless of elevation (uniform response acceleration)
- a “modal” pattern, proportional to lateral forces consistent with the lateral force distribution determined in elastic analysis

(2) Lateral loads should be applied at the location of the masses in the model. Accidental eccentricity should be taken into account.

4.4.4.2 Capacity curve

(1) The relation between base-shear force and the control displacement (the “capacity curve”) should be determined in accordance with EN 1998-1:2004, **4.3.3.4.2.3(1)**, **(2)**.

4.4.4.3 Target displacement

(1)P Target displacement is defined as seismic demand in terms of the displacement defined in EN 1998-1:2004, **4.3.3.4.2.6(1)**.

NOTE Target displacement may be determined in accordance with EN 1998-1: 2004, Informative Annex B.

4.4.4.4 Procedure for estimation of the torsional effects

(1)P The procedure given in EN 1998-1:2004, **4.3.3.4.2.7(1)** to **(3)** applies.

4.4.5 Non-linear time-history analysis

(1)P The procedure given in EN 1998-1:2004, **4.3.3.4.3(1)** to **(3)** applies.

4.4.6 Combination of the components of the seismic action

(1)P The two horizontal components of the seismic action shall be combined in accordance with EN 1998-1:2004, **4.3.3.5.1**.

(2)P The vertical component of the seismic action shall be taken into account in the cases specified in EN 1998-1:2004, **4.3.3.5.2** and, when appropriate, combined with the horizontal components as indicated in the same clause.

4.4.7 Additional measures for masonry infilled structures

(1) The provisions of EN 1998-1:2004, **4.3.6** apply, wherever relevant.

4.4.8 Combination coefficients for variable actions

(1) The provisions of EN 1998-1:2004, **4.2.4** apply

4.4.9 Importance categories and importance factors

(1) The provisions of EN 1998-1:2004, **4.2.5** apply.

4.5 Safety verifications

4.5.1 Linear methods of analysis (static or dynamic)

(1)P "Brittle" components/mechanisms shall be verified with demands calculated by means of equilibrium conditions, on the basis of the action effects delivered to the brittle component/mechanism by the ductile components. In this calculation, each action effect in a ductile component delivered to the brittle component/mechanism under consideration should be taken equal to:

(a) the value D obtained from the analysis, if the capacity C of the ductile component, evaluated using mean values of material properties, satisfies $\rho = D/C \leq 1$,

(b) the capacity of the ductile component, evaluated using mean values of material properties multiplied by the confidence factors, as defined in **3.5**, accounting for the level of knowledge attained, if $\rho = D/C > 1$, with D and C as defined in (a) above.

(2) In **(1)**b above the capacities of the beam sections around beam-column joints should be computed from expression (5.8) in EN 1998-1: 2004 and those of the column sections around such joints from expression (5.9), using in the right-hand-side of these expressions the value $\gamma_{Rd} = 1$ and mean values of material properties multiplied by the confidence factors, as defined in **3.5.**

(3) For the calculation of force demands on the "brittle" shear mechanism of walls through **(1)**b above, expression (5.26) in EN1998-1: 2004 may be applied with $\gamma_{Rd} = 1$ and using as M_{Rd} the bending moment capacity at the base, evaluated using mean values of material properties multiplied by the confidence factors, as defined in **3.5.**

(4) In **(1)**P to **(3)** above the bending moment capacities C_i of vertical elements may be based on the value of the axial force due to the vertical loads only.

(5)P The value of the capacity of both ductile and brittle components and mechanisms to be compared to demand in safety verifications, shall be in accordance with **2.2.1(5)**P.

NOTE Information for the evaluation of the capacity of components and mechanisms may be found in the relevant material related Informative Annexes A, B and C.

4.5.2 Nonlinear methods of analysis (static or dynamic)

(1)P The demands on both "ductile" and "brittle" components shall be those obtained from the analysis performed in accordance with **4.4.4** or **4.4.5**, using mean value properties of the materials.

(2)P **4.5.1(5)**P applies.

4.6 Summary of criteria for analysis and safety verifications

(1)P The following Table 4.3 summarises:

- The values of the material properties to be adopted in evaluating both the demand and capacities of the elements for the case of linear and nonlinear analysis,
- The criteria that shall be followed for the safety verification of both ductile and brittle elements for the case of linear and nonlinear analysis.

Table 4.3: Values of material properties and criteria for analysis and safety verifications.

<table>
<tr><td colspan="2"></td><td colspan="2">Linear Model (LM)</td><td colspan="2">Nonlinear Model</td></tr>
<tr><td colspan="2"></td><td>Demand</td><td>Capacity</td><td>Demand</td><td>Capacity</td></tr>
<tr><td rowspan="7">Type of element or mechanism (e/m)</td><td rowspan="4">Ductile</td><td colspan="2">Acceptability of Linear Model (for checking of $\rho_i = D_i/C_i$ values):</td><td rowspan="7">From analysis. Use mean values of properties in model.</td><td rowspan="4">In terms of deformation. Use mean values of properties <u>divided</u> by CF.</td></tr>
<tr><td>From analysis. Use mean values of properties in model.</td><td>In terms of strength. Use mean values of properties.</td></tr>
<tr><td colspan="2">Verifications (if LM accepted):</td></tr>
<tr><td>From analysis.</td><td>In terms of deformation. Use mean values of properties <u>divided</u> by CF.</td></tr>
<tr><td rowspan="3">Brittle</td><td colspan="2">Verifications (if LM accepted):</td><td rowspan="3">In terms of strength. Use mean values of properties <u>divided</u> by CF and by partial factor.</td></tr>
<tr><td>If $\rho_i \leq 1$: from analysis.</td><td rowspan="2">In terms of strength. Use mean values of properties <u>divided</u> by CF and by partial factor.</td></tr>
<tr><td>If $\rho_i > 1$: from equilibrium with strength of ductile e/m. Use mean values of properties <u>multiplied</u> by CF.</td></tr>
</table>

5 DECISIONS FOR STRUCTURAL INTERVENTION

5.1 Criteria for a structural intervention

(1) On the basis of the conclusions of the assessment of the structure and/or the nature and extent of the damage, decisions should be taken for the intervention.

NOTE: As in the design of new structures, optimal decisions are pursued, taking into account social aspects, such as the disruption of use or occupancy during the intervention.

(2) This Standard describes the technical aspects of the relevant criteria.

5.1.1 Technical criteria

(1)P The selection of the type, technique, extent and urgency of the intervention shall be based on the structural information collected during the assessment of the building.

(2) The following aspects should be taken into account:

a) All identified local gross errors should be appropriately remedied.

b) In case of highly irregular buildings (both in terms of stiffness and overstrength distributions), structural regularity should be improved as much as possible, both in elevation and in plan.

c) The required characteristics of regularity and resistance can be achieved by either modification of the strength and/or stiffness of an appropriate number of existing components, or by the introduction of new structuralelements.

d) Increase in the local ductility supply should be effected where required.

e) The increase in strength after the intervention should not reduce the available global ductility.

f) Specifically for masonry structures: non-ductile lintels should be replaced, inadequate connections between floor and walls should be improved, out-of-plane horizontal thrusts against walls should be eliminated.

5.1.2 Type of intervention

(1) An intervention may be selected from the following indicative types:

a) Local or overall modification of damaged or undamaged elements (repair, strengthening or full replacement), considering the stiffness, strength and/or ductility of these elements.

b) Addition of new structural elements (e.g. bracings or infill walls; steel, timber or reinforced concrete belts in masonry construction; etc).

c) Modification of the structural system (elimination of some structural joints; widening of joints; elimination of vulnerable elements; modification into more regular and/or more ductile arrangements)[1].

d) Addition of a new structural system to sustain some or all of the entire seismic action.

e) Possible transformation of existing non-structural elements into structural elements.

f) Introduction of passive protection devices through either dissipative bracing or base isolation.

g) Mass reduction.

h) Restriction or change of use of the building.

i) Partial demolition.

(2) One or more types in combination may be selected. In all cases, the effect of structural modifications on the foundation should be taken into account.

(3)P If base isolation is adopted, the provisions contained in EN 1998-1:2004, **10** shall be followed.

5.1.3 Non-structural elements

1(P) Decisions regarding repair or strengthening of non-structural elements shall also be taken whenever, in addition to functional requirements, the seismic behaviour of these elements may endanger the life of inhabitants or affect the value of goods stored in the building.

(2) In such cases, full or partial collapse of these elements should be avoided by means of:

a) Appropriate connections to structural elements (see EN1998-1:2004, **4.3.5**).

b) Increasing the resistance of non-structural elements (see EN 1998-1:2004, **4.3.5**).

c) Taking measures of anchorage to prevent possible falling out of parts of these elements.

(3) The possible consequences of these provisions on the behaviour of structural elements should be taken into account.

[1] This is for instance the case when vulnerable low shear-ratio columns or entire soft storeys are transformed into more ductile arrangements; similarly, when overstrength irregularities in elevation, or in-plan eccentricities are reduced by modifying the structural system.

5.1.4 Justification of the selected intervention type

(1)P In all cases, the documents relating to shall include the justification of the type of intervention selected and the description of its expected effect on the structural response.

(2) This justification should be made available to the owner.

6 DESIGN OF STRUCTURAL INTERVENTION

6.1 Retrofit design procedure

(1)P The retrofit design procedure shall include the following steps:

a) Conceptual design,

b) Analysis,

c) Verifications.

(2)P The conceptual design shall cover the following:

(i) Selection of techniques and/or materials, as well as of the type and configuration of the intervention.

(ii) Preliminary estimation of dimensions of additional structural parts.

(iii) Preliminary estimation of the modified stiffness of the retrofitted elements.

(3)P The methods of analysis of the structure specified in **4.4** shall be used, taking into account the modified characteristics of the building.

(4)P Safety verifications shall be carried out in general in accordance with **4.5**, for both existing, modified and new structural elements. For existing materials, mean values from in-situ tests and any additional sources of information shall be used in the safety verification, modified by the confidence factor CF, as specified in **4.5**. However, for new or added materials nominal properties shall be used, without modification by the confidence factor CF.

NOTE Information on the capacities of existing and new structural elements may be found in the relevant material-related Informative Annex A, B or C.

(5)P In case the structural system, comprising both existing and new structural elements, can be made to fulfill the requirements of EN1998-1:2004, the verifications may be carried out in accordance with the provisions therein.

ANNEX A (Informative)

REINFORCED CONCRETE STRUCTURES

A.1 Scope

(1) This Annex contains specific information for the assessment of reinforced concrete buildings in their present state, and for their upgrading, when necessary.

A.2 Identification of geometry, details and materials

A.2.1 General

(1) The following aspects should be carefully examined:

i. Physical condition of reinforced concrete elements and presence of any degradation, due to carbonation, steel corrosion, etc.

ii. Continuity of load paths between lateral resisting elements.

A.2.2 Geometry

(1) The collected data should include the following items:

i. Identification of the lateral resisting systems in both directions.

ii. Orientation of one-way floor slabs.

iii. Depth and width of beams, columns and walls.

iv. Width of flanges in T-beams.

v. Possible eccentricities between beams and columns axes at joints.

A.2.3 Details

(1) The collected data should include the following items:

i. Amount of longitudinal steel in beams, columns and walls.

ii. Amount and detailing of confining steel in critical regions and in beam-column joints.

iii. Amount of steel reinforcement in floor slabs contributing to the negative resisting bending moment of T-beams.

iv. Seating lengths and support conditions of horizontal elements.

v. Depth of concrete cover.

vi. Lap-splices for longitudinal reinforcement.

A.2.4 Materials

(1) The collected data should include the following items:

i. Concrete strength.

ii. Steel yield strength, ultimate strength and ultimate strain.

A.3 Capacity models for assessment

(1) The provisions given in this clause apply to both primary and secondary seismic elements.

(2) Classification of components/mechanisms:

i. "ductile": beam, columns and walls under flexure with and without axial force,

ii. "brittle": shear mechanism of beams, columns, walls and joints.

A.3.1 Beam, columns and walls under flexure with and without axial force

(1) The deformation capacity of beams, columns and walls, to be verified in accordance with **2.2.2(2)**P, **2.2.3(2)**P, **2.2.4(2)**P, is defined in terms of the chord rotation θ, *i.e.*, of the angle between the tangent to the axis at the yielding end and the chord connecting that end with the end of the shear span ($L_V = M/V$ = moment/shear at the end section), *i.e.*, the point of contraflexure. The chord rotation is also equal to the element drift ratio, *i.e.*, the deflection at the end of the shear span with respect to the tangent to the axis at the yielding end, divided by the shear span.

A.3.1.1 Limit State of near collapse (NC)

(1) The value of the total chord rotation capacity (elastic plus inelastic part) at ultimate, θ_u, of concrete members under cyclic loading may be calculated from the following expression:

$$\theta_{um} = \frac{1}{\gamma_{el}} 0{,}016 \cdot (0{,}3^{\nu}) \left[\frac{\max(0{,}01;\omega')}{\max(0{,}01;\omega)} f_c \right]^{0.225} \left(\frac{L_V}{h} \right)^{0{,}35} 25^{\left(\alpha \rho_{sx} \frac{f_{yw}}{f_c} \right)} (1{,}25^{100 \rho_d}) \quad \text{(A.1)}$$

where:

γ_{el} equal to 1,5 for primary seismic elements and 1,0 for secondary seismic elements (as defined in **2.2.1(6)**P),

h is the depth of cross-section,

$L_V = M/V$ ratio moment/shear at the end section

ν = N / bhf_c (b width of compression zone, N axial force positive for compression),

ω, ω' is the mechanical reinforcement ratio of the tension (including the web reinforcement) and compression, respectively, longitudinal reinforcement,

f_c and f_{yw} are the concrete compressive strength (MPa) and the steel yield strength (MPa), respectively, directly obtained as mean values from *in-situ* tests, and from the additional sources of information, appropriately divided by the confidence factors, as defined in **3.5(1)**P and Table 3.1, accounting for the level of knowledge attained,

ρ_{sx} = $A_{sx}/b_w s_h$= ratio of transverse steel parallel to the direction x of loading (s_h = stirrup spacing),

ρ_d is the steel ratio of diagonal reinforcement (if any), in each diagonal direction,

α is the confinement effectiveness factor, that may be taken equal to:

$$\alpha = \left(1-\frac{s_h}{2b_o}\right)\left(1-\frac{s_h}{2h_o}\right)\left(1-\frac{\sum b_i^2}{6h_o b_o}\right) \tag{A.2}$$

where:

b_o and h_o is the dimension of confined core to the centreline of the hoop,

b_i is the centerline spacing of longitudinal bars (indexed by i) laterally restrained by a stirrup corner or a cross-tie along the perimeter of the cross-section.

In walls the value given by expression (A.1) is divided by 1,6.

If cold-worked brittle steel is used the total chord rotation capacity above is divided by 1,6.

(2) The value of the plastic part of the chord rotation capacity of concrete members under cyclic loading may be calculated from the following expression:

$$\theta_{um}^{pl} = \theta_{um} - \theta_y = \frac{1}{\gamma_{el}} 0{,}0145 \cdot (0{,}25^{\nu}) \left[\frac{\max(0{,}01;\omega')}{\max(0{,}01;\omega)}\right]^{0{,}3} \cdot f_c^{\,0{,}2} \cdot \left(\frac{L_V}{h}\right)^{0{,}35} 25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{f_c}\right)} (1{,}275^{100\rho_d}) \tag{A.3}$$

where the chord rotation at yielding, θ_y, should be calculated in accordance with **A.3.1.3** and all other variables are defined as for expression (A.1).

In walls the value given by expression (A.3) is multiplied by 0,6.

If cold-worked brittle steel is used, the plastic part of the chord rotation capacity is divided by 2.

(3) In members without detailing for earthquake resistance the values given by expressions (A.1) and (A.3) are multiplied by 0,85.

(4) **(1)** and **(2)** apply to members with deformed (high bond) longitudinal bars without lapping in the vicinity of the end region where yielding is expected. If deformed longitudinal bars have straight ends lapped starting at the end section of the member - as is often the case in columns and walls with lap-splicing starting at floor level - expressions (A.1) and (A.3) should be applied with the value of the compression reinforcement ratio, ω', doubled over the value applying outside the lap splice. Moreover, if the lap length l_o is less than $40d_{bL}$, where d_{bL} is the diameter of the lapped bars, the plastic part of the chord rotation capacity given in **(2)** should be multiplied by $(l_o - 10d_{bL})/30d_{bL}$, while the value of the chord rotation at yielding, θ_y, added to it to obtain the total chord rotation capacity, should account for the effect of the lapping in accordance with **A.3.1.3(3).**

(5) In members with smooth (plain) longitudinal bars without lapping in the vicinity of the end region where yielding is expected, the total chord rotation capacity may be taken equal to the value calculated in accordance with **(1)** multiplied by 0,575, while the plastic part of the chord rotation capacity may be taken to be equal to that calculated in accordance with **(2)** multiplied by 0,375 (with these factors including the reduction factor 0,85 of **(3)** accounting for the lack of detailing for earthquake resistance). If the longitudinal bars are lapped starting at the end section of the member and their ends are provided with standard hooks and a lap length l_o of at least $15d_{bL}$, the chord rotation capacity of the member may be calculated as follows:

- In expressions (A.1), (A.3) the shear span L_V (ratio M/V - moment/shear - at the end section) is reduced by the lap length l_o, as the ultimate condition is controlled by the region right after the end of the lap.
- The total chord rotation capacity may be taken equal to the value calculated in accordance with (1) multiplied by $0{,}0025\ (180 + \min(50, l_o / d_{bL}))(1 - l_o / L_V)$, while the plastic part of the chord rotation capacity may be taken equal to that calculated in accordance with (2) multiplied by $0{,}0035\ (60 + \min(50, l_o / d_{bL}))(1 - l_o / L_V)$.

(6) For the evaluation of the ultimate chord rotation capacity an alternative expression may be used:

$$\theta_{um} = \frac{1}{\gamma_{el}}\left(\theta_y + (\varphi_u - \varphi_y)L_{pl}\left(1 - \frac{0{,}5L_{pl}}{L_V}\right)\right) \tag{A.4}$$

where

θ_y is the chord rotation at yield as defined by expression (A.11),

ϕ_u is the ultimate curvature at the end section,

ϕ_y is the yield curvature at the end section.

The value of the length L_{pl} of the plastic hinge depends on how the enhancement of strength and deformation capacity of concrete due to confinement is taken into account in the calculation of the ultimate curvature of the end section, ϕ_u.

(7) If the ultimate curvature of the end section ϕ_u, under cyclic loading is calculated with:

(a) the ultimate strain of the longitudinal reinforcement, ε_{su}, taken equal to:

- the minimum values given in EN 1992-1-1, Table C.1 for the characteristic strain at maximum force, ε_{uk}, for steel Classes A or B,
- 6% for steel Class C, and

(b) the confinement model in EN 1992-1-1:2004, **3.1.9**, with effective lateral confining stress σ_2 taken equal to $\alpha\rho_{sx}f_{yw}$, where ρ_{sx}, f_{yw} and α have been defined in **(1)**,

then, for members with detailing for earthquake resistance and without lapping of longitudinal bars in the vicinity of the section where yielding is expected, L_{pl} may be calculated from the following expression:

$$L_{pl} = 0{,}1L_V + 0{,}17h + 0{,}24\frac{d_{bL}f_y(MPa)}{\sqrt{f_c(MPa)}} \tag{A.5}$$

where h is the depth of the member and d_{bL} is the (mean) diameter of the tension reinforcement.

(8) If the ultimate curvature of the end section, ϕ_u, under cyclic loading is calculated with:

(a) the ultimate strain of the longitudinal reinforcement, ε_{su}, taken as in (7)a, and

(b) a confinement model which represents better than the model in EN 1992-1-1:2004, **3.1.9** the improvement of ϕ_u with confinement under cyclic loading; namely a model where:

- the strength of confined concrete is evaluated from:

$$f_{cc} = f_c\left[1 + 3{,}7\left(\frac{\alpha\,\rho_{sx}\,f_{yw}}{f_c}\right)^{0{,}86}\right] \tag{A.6}$$

- the strain at which the strength f_{cc} takes place is taken to increase over the value ε_{c2} of unconfined concrete as:

$$\varepsilon_{cc} = \varepsilon_{c2}\left[1 + 5\left(\frac{f_{cc}}{f_c} - 1\right)\right] \tag{A.7}$$

- and the ultimate strain of the extreme fibre of the compression zone is taken as:

$$\varepsilon_{cu} = 0{,}004 + 0{,}5\frac{a\rho_{sx}f_{yw}}{f_{cc}} \tag{A.8}$$

where:

α, f_{yw} and ρ_{sx} are as defined in **(1)** and **(7)** and f_{cc} is the concrete strength, as enhanced by confinement, then, for members with detailing for earthquake resistance and no lapping of longitudinal bars near the section where yielding is expected, L_{pl} may be calculated from the following expression:

$$L_{\text{pl}} = \frac{L_V}{30} + 0{,}2h + 0{,}11\frac{d_{bL} f_y(MPa)}{\sqrt{f_c(MPa)}} \qquad \text{(A.9)}$$

(9) If the confinement model in EN1992-1-1:2004 **3.1.9** is adopted in the calculation of the ultimate curvature of the end section, ϕ_u, and the value of L_{pl} from expression (A.5) is used in expression (A.4), then the factor γ_e therein may be taken equal to 2 for primary seismic and to 1,0 for secondary seismic elements. If the confinement model given by expressions (A.6) to (A.8) is used instead, together with expression (A.9), then the value of the factor γ_{el} may be taken equal to 1,7 for primary seismic elements and to 1,0 for secondary seismic ones.

NOTE: The values of the total chord rotation capacity calculated in accordance with **(1)** and **(2)** above (taking into account **(3)** to **(5)**) are normally very similar. Expression (A.1) is more convenient when calculations and demands are based on total chord rotations, whilst expression (A.3) is better suited for those cases when calculations and demands are based on the plastic part of chord rotations; moreover, **(4)** gives the chord rotation capacity of members with deformed longitudinal bars and straight ends lapped starting at the end section only in terms of expression (A.3). Expression (A.4) with γ_{el}=1 yields fairly similar results when used with either **(7)** or **(8)**, but differences with the predictions of **(1)** or **(2)** are larger. The scatter of test results with respect to those of expression (A.4) for γ_{el}=1 used with **(8)** is less than when it is used with **(7)**. This is reflected in the different values of γ_{el} specified in **(1)**, **(2)** and **(9),** for primary seismic elements, as γ_{el} is meant to convert mean values to mean-minus-one-standard-deviation ones. Finally, the effects of lack of detailing for earthquake resistance and of lap splicing in the plasti hinge zone are specified in **(3)** to **(5)** only in connection with expressions (A.1) and (A.3).

(10) Existing walls conforming to the definition of “large lightly reinforced walls” of EN1998-1:2004, can be verified in accordance with EN1992-1-1:2004.

A.3.1.2 Limit State of Significant Damage (SD)

(1) The chord rotation capacity corresponding to significant damage θ_{SD} may be assumed to be 3/4 of the ultimate chord rotation θ_u given in **A.3.1.1**.

A.3.1.3 Limit State of Damage Limitation (DL)

(1) The capacity for this limit state used in the verifications is the yielding bending moment under the design value of the axial load.

(2) In case the verification is carried out in terms of deformations the corresponding capacity is given by the chord rotation at yielding θ_y, evaluated as:

For beams and columns:

$$\theta_y = \phi_y \frac{L_V + a_V z}{3} + 0{,}00135\left(1 + 1{,}5\frac{h}{L_V}\right) + \frac{\varepsilon_y}{d - d_1}\frac{d_b f_y}{6\sqrt{f_c}} \qquad \text{(A.10a)}$$

For walls of rectangular, T- or barbelled section:

$$\theta_{y}=\phi_{y}\frac{L_{V}+a_{V}z}{3}+0{,}002\left(1-0{,}135\frac{L_{V}}{h}\right)+\frac{\varepsilon_{y}}{d-d_{1}}\frac{d_{b}f_{y}}{6\sqrt{f_{c}}} \tag{A.11a}$$

or from the alternative (and equivalent) expressions for beams and columns:

$$\theta_{y}=\phi_{y}\frac{L_{V}+a_{V}z}{3}+0{,}0013\left(1+1{,}5\frac{h}{L_{V}}\right)+0{,}13\phi_{y}\frac{d_{b}f_{y}}{\sqrt{f_{c}}} \tag{A.10b}$$

and for walls of rectangular, T- or barbelled section:

$$\theta_{y}=\phi_{y}\frac{L_{V}+a_{V}z}{3}+0{,}002\left(1-0{,}125\frac{L_{V}}{h}\right)+0{,}13\phi_{y}\frac{d_{b}f_{y}}{\sqrt{f_{c}}} \tag{A.11b}$$

where:

ϕ_y is the yield curvature of the end section;

$\alpha_V z$ is the tension shift of the bending moment diagram (see EN 1992-1-1: 2004, **9.2.1.3(2)**), included with α_V=1 only if shear cracking is expected to precede flexural yielding at the end section (i.e. when the yield moment at the end section, M_y, exceeds the product of L_V times the shear resistance of the member considered without shear reinforcement, $V_{R,c}$, taken in accordance with EN 1992-1-1: 2004, **6.2.2(1)**); otherwise, (i.e. if $M_y<L_V V_{R,c}$) it is taken α_V=0;

f_y and f_c are the steel yield stress and the concrete, respectively, as defined for expression (A.1), both in MPa;

ε_y is equal to f_y/E_s;

d and d' are the depths to the tension and compression reinforcement, respectively; and

d_{bL} is the (mean) diameter of the tension reinforcement.

The first term in expressions (A.10), (A.11) accounts for the flexural contribution; The second term represents the contribution of shear deformation and the third anchorage slip of bars.

NOTE: The two alternative sets of expressions: (A.10a), (A.11a) on one hand and (A.10b), (A.11b) on the other are practically equivalent. Expressions: (A.10a), (A.11a) are more rational but Expressions: (A.10ba), (A.11b) are more convenient and their use may be overall more convenient, as the calculation of ϕ_y may be difficult and more prone to errors.

(3) **(1)** and **(2)** apply to members with longitudinal bars without lapping in the vicinity of the end region where yielding is expected. If longitudinal bars are deformed with straight ends lapped starting at the end section of the member (as in columns and walls with lap-splicing starting at floor level), the yield moment M_y and the yield curvature ϕ_y in expressions (A.10), (A.11) should be computed with a compression reinforcement ratio doubled over the value applying outside the lap splice. If the straight lap length l_o is less than $25d_{bL}$, where d_{bL} is the diameter of the lapped bars, then:

– My and ϕ_y should be calculated with the yield stress, f_y, multiplied by lo/$25d_{bL}$;

- the yield strain, ε_y, in the last term of expressions (A.10a), (A.11a) should be multiplied by lo/25d_{bL};
- the second term in expressions (A.10), (A.11) should be multiplied by the ratio of the value of yield moment M_y as modified to account for the lap splicing, to the yield moment outside the lap splice;
- to determine whether term $\alpha_V z$ contributes to the first term in expressions (A.10), (A.11) with α_V=1, the product $L_V V_{R,c}$ is compared to the yield moment $M_{y.c}$ as modified for the effect of the lapping.

(4) **(1)** and **(2)** may be considered to apply also to members with smooth longitudinal bars, even when their ends, supplied with standard hooks, are lapped starting at the end section of the member (as in columns and walls with lap-splicing starting at floor level), provided that the lap length l_o is at least equal to 15d_{bL}.

(5) If the verification is carried out in terms of deformations, deformation demands should be obtained from an analysis of a structural model in which the stiffness of each element is taken to be equal to the mean value of $M_y L_V$ /3θ_y, at the two ends of the element. In this calculation the shear span at the end section, L_V, may be taken to be equal to half the element length.

A.3.2 Beam-columns and walls: shear

A.3.2.1 Limit State of Near Collapse (NC)

(1) The cyclic shear resistance, V_R, decreases with the plastic part of ductility demand, expressed in terms of ductility ratio of the transverse deflection of the shear span or of the chord rotation at member end: μ_Δ^{pl}= μ_Δ-1. For this purpose μ_Δ^{pl} may be calculated as the ratio the plastic part of the chord rotation, θ, normalized to the chord rotation at yielding, θ_y, calculated in accordance with **A.3.1.2**.

The following expression may be used for the shear strength, as controlled by the stirrups, accounting for the above reduction :

$$V_R = \frac{1}{\gamma_{el}}\left[\frac{h-x}{2L_s}\min(N; 0{,}55A_c f_c) + \left(1-0{,}05\min(5;\mu_\Delta^{pl})\right)\cdot\right.$$
$$\left.\cdot\left[0{,}16\max(0{,}5;100\rho_{tot})\left(1-0{,}16\min\left(5;\frac{L_V}{h}\right)\right)\sqrt{f_c}A_c + V_w\right]\right] \quad (A.12)$$

where:

γ_{el} equal to 1,15 for primary seismic elements and 1,0 for secondary seismic elements (as defined in **2.2.1(6)**P);

h is the depth of cross-section (equal to the diameter D for circular sections);

x is the compression zone depth;

N is the compressive axial force (positive, taken as being zero for tension);

L_V = M/V ratio moment/shear at the end section;

A_c is the cross-section area, taken as being equal to $b_w d$ for a cross-section with a rectangular web of width (thickness) b_w and structural depth d, or to $\pi D_c^2/4$ (where D_c is the diameter of the concrete core to the inside of the hoops) for circular sections;

f_c is the concrete compressive strength , as defined for expression (A.1) and for primary seismic elements also divided by the partial factor for concrete in accordance with EN1998-1:2004, **5.2.4**;

ρ_{tot} is the total longitudinal reinforcement ratio;

V_w is the contribution of transverse reinforcement to shear resistance, taken as being equal to:

a) for cross-sections with rectangular web of width (thickness) b_w:

$$V_w = \rho_w b_w z f_{yw} \tag{A.13}$$

where:

ρ_w is the transverse reinforcement ratio;

z is the length of the internal lever arm (taken as being equal to d-d' in beams, columns, or walls with barbelled or T-section, or to 0,8h in walls with rectangular section); and

f_{yw} is the yield stress of the transverse reinforcement, as defined for expression (A.1) and for primary seismic elements also divided by the partial factor for steel in accordance with EN 1998-1:2004, **5.2.4**;

b) for circular cross-sections:

$$V_w = \frac{\pi}{2}\frac{A_{sw}}{s} f_{yw}(D-2c) \tag{A.14}$$

where:

D is the diameter of the section;

A_{sw} is the cross-sectional area of a circular stirrup;

s is the centerline spacing of stirrups;

f_{yw} is as defined in (a) above; and

c is the concrete cover.

(2) The shear strength of a concrete wall, V_R, may not be taken greater than the value corresponding to failure by web crushing, $V_{R,max}$, which under cyclic loading may be calculated from the following expression (with units: MN and meters):

$$V_{R,max} = \frac{0{,}85\left(1-0{,}06\min\left(5;\,\mu_\Delta^{pl}\right)\right)}{\gamma_{el}}\left(1+1{,}8\min\left(0{,}15;\frac{N}{A_c f_c}\right)\right)\left(1+0{,}25\max(1{,}75;\,100\rho_{tot})\right)\left(1-0{,}2\min\left(2;\frac{L_V}{h}\right)\right)\sqrt{\min(100;\,f_c)}\,b_w z \tag{A.15}$$

where γ_{el} = 1,15 for primary seismic elements and 1,0 for secondary seismic ones, f_c is in MPa, b_w and z are in meters and $V_{R,max}$ in MN, and all other variables are as defined in **(1)**.

The shear strength under cyclic loading as controlled by web crushing prior to flexural yielding is obtained from expression (A.15) for μ_Δ^{pl}=0.

(3) In concrete columns with shear span ratio, L_V/h, at the end section with the maximum of the two end moments less or equal to 2, the shear strength, V_R, may not be taken greater than the value corresponding to failure by web crushing along the diagonal of the column after flexural yielding, $V_{R,max}$, which under cyclic loading may be calculated from the expression (with units: MN and meters):

$$V_{R,max} = \frac{4/7\left(1-0{,}02\min\left(5;\,\mu_\Delta^{pl}\right)\right)}{\gamma_{el}}\left(1+1{,}35\frac{N}{A_c f_c}\right)\left(1+0{,}45(100\rho_{tot})\right)\sqrt{\min(40;f_c)}\,b_w z\sin 2\theta \quad \text{(A.16)}$$

where:

θ is the angle between the diagonal and the axis of the column ($\tan\theta = h/2L_V$),

and all other variables are as defined in **(3)**.

(4) The minimum of the shear resistance calculated in accordance with EN1992-1-1: 2004 or by means of expressions (A.12)-(A.16) should be used in the assessment.

(5) Mean material properties from *in-situ* tests and from additional sources of information, should be used in the calculations.

(6) For primary seismic elements, mean material strengths in addition to being divided by the appropriate confidence factors based on the Knowledge Level, they should be divided by.the partial factors for materials in accordance with EN1998-1:2004, **5.2.4**.

A.3.2.2 Limit State of Significant Damage (SD) and of Damage Limitation (DL)

(1) The verification against the exceedance of these two LS is not required, unless these two LS are the only ones to be checked. In that case, **A.3.2.1(1)** to **(4)** apply.

A.3.3 Beam-column joints

A.3.3.1 LS of Near Collapse (NC)

(1) The shear demand on the joints is evaluated in accordance with EN 1998-1:2004, **5.5.2.3**.

(2) The shear capacity of the joints is evaluated in accordance with EN 1998-1:2004, **5.5.3.3.**

(3) **A.3.2.1(6)** applies to joints of primary seismic elements with other elements,

A.3.3.2 Limit State of Significant Damage (SD) and of Damage Limitation (DL)

(1) The verification against the exceedance of these two LS is not required, unless these two LS are the only ones to be checked. In that case, **A.3.3.1(1)** and **(2)** apply.

A.4 Capacity models for strengthening

A.4.1 General

(1) The rules for member strength and deformation capacities given in the following clauses for strengthened members refer to the capacities at the LS of NC in **A.3.1.1** and **A.3.2.1** prior to the application of the overall factor γ_{el}. The γ_{el} factors specified in **A.3.1.1** and **A.3.2.1** should be applied on the strength and deformation capacities of the retrofitted member, as determined in accordance with the following clauses.

(2) The partial factors to be applied to the new steel and concrete used for the retrofitting are those of EN1998-1: 2004, **5.2.4**, and to new structural steel used for the retrofitting are those of EN1998-1: 2004, **6.1.3(1)**P.

A.4.2 Concrete jacketing

(1) Concrete jackets are applied to columns and walls for all or some of the following purposes:

- increasing the bearing capacity,
- increasing the flexural and/or shear strength,
- increasing the deformation capacity,
- improving the strength of deficient lap-splices.

(2) The thickness of the jackets should allow for placement of both longitudinal and transverse reinforcement with an adequate cover.

(3) When jackets aim at increasing flexural strength, longitudinal bars should be continued to the adjacent storey through holes piercing the slab, while horizontal ties should be placed in the joint region through horizontal holes drilled in the beams. Ties may be omitted in the case of fully confined interior joints.

(4) When only shear strength and deformation capacity increases are of concern, jointly with a possible improvement of lap-splicing, then jackets should be terminated (both concreting and reinforcement) leaving a gap with a slab of the order of 10 mm.

A.4.2.1 Enhancement of strength, stiffness and deformation capacity

(1) For the purpose of evaluating strength and deformation capacities of jacketed elements, the following approximate simplifying assumptions may be made:

- the jacketed element behaves monolithically, with full composite action between old and new concrete;

- the fact that axial load is originally applied to the old column alone is disregarded, and the full axial load is assumed to act on the jacketed element;
- the concrete properties of the jacket are assumed to apply over the full section of the element.

(2) The following relations may be assumed to hold between the values of V_R, M_y, θ_y, and θ_u, calculated under the assumptions above and the values V_R*, M_y*, θ_y*, and θ_u* to be adopted in the capacity verifications:

- For V_R* :

$$V_R{}^* = 0{,}9 V_R \tag{A.17}$$

- For M_y*:

If dowels are provided at the interface of the jacket and the old concrete:

$$M_y{}^* = 0{,}9 M_y \tag{A.18a}$$

In all other cases:

$$M_y{}^* = M_y \tag{A.18b}$$

- For θ_y*:

If no special measures are taken to connect the jacket to the old concrete, or if dowels are provided at their interface:

$$\theta_y{}^* = 1{,}25\theta_y \tag{A.19a}$$

For all other types of measures to connect the jacket to the old concrete (chiseling of interface, connecting the old and the new longitudinal bars through welded steel inserts, etc.):

$$\theta_y{}^* = 1{,}05\theta_y \tag{A.19b}$$

- For θ_u*:

$$\theta_u{}^* = \theta_u \tag{A.20}$$

(3) The values of θ_u*, θ_y*, M_y* of the jacketed member, to be used in comparisons to demands in safety verifications, should be computed on the basis of: (a) the mean value strength of the existing steel as directly obtained from *in-situ* tests and from additional sources of information, appropriately divided by the confidence factor in **3.5**, accounting for the level of knowledge attained; and (b) the nominal strength of the added concrete and reinforcement.

(4) The value of V_R* of the jacketed member, to be compared to the demand in safety verifications, should be computed on the basis of: (a) the mean value strength of the existing steel as directly obtained from *in-situ* tests and from the additional sources of information, divided by the appropriate confidence factor in **3.5**, accounting for the level of knowledge attained; and (b) the nominal strength of the added concrete and reinforcement. In primary seismic elements the mean value strength of the existing steel and the nominal strength of the added materials should be divided by the partial factors for steel and concrete in accordance with EN 1998-1:2004, **5.2.4**.

(5) The value of M_y* of jacketed members that deliver action effects to brittle components/mechanisms, for use in **4.5.1(1)**P(b), should be computed on the basis of: (a) the mean value strength of the existing steel as directly obtained from *in-situ* tests, and from additional sources of information, appropriately multiplied by the confidence factor in **3.5**, accounting for the level of knowledge attained; and (b) the nominal strength of the added concrete and reinforcement.

A.4.3 Steel jacketing

(1) Steel jackets are mainly applied to columns for the purpose of: increasing shear strength and improving the strength of deficient lap-splices. They may also be considered to increase ductility through confinement.

(2) Steel jackets around rectangular columns are usually built up of four corner angles to which either continuous steel plates, or thicker discrete horizontal steel straps, are welded. Corner angles may be epoxy-bonded to the concrete, or just made to adhere to it without gaps along the entire height. Straps may be pre-heated just prior to welding, in order to provide afterwards some positive confinement on the column.

A.4.3.1 Shear strength

(1) The contribution of the jacket to shear strength may be assumed as additive to existing strength, provided the jacket remains well within the elastic range. This condition is necessary for the jacket to be able to control the width of internal cracks and to ensure the integrity of the concrete, thus allowing the original shear resisting mechanism to continue to operate.

(2) If only 50% of the steel yield strength of the jacket is used, the expression for the additional shear V_j carried by the jacket is:

$$V_j = 0{,}5 \frac{2 t_j b}{s} f_{yj,d} \frac{1}{\cos\alpha} \tag{A.21}$$

where:

t_j is the thickness of the steel straps;

b is the width of the steel straps; and

s is the spacing of the steel straps (b/s = 1, in case of continuous steel plates), where f_{yw} is the mean value of the yield strength.

$f_{yj,d}$ is the design yield strength of the steel of the jacket, equal to its nominal strength divided by the partial factor for structural steel in accordance with EN1998-1: 2004, **6.1.3(1)**P.

A.4.3.2 Clamping of lap-splices

(1) Steel jackets can provide effective clamping in the regions of lap-splices, to improve cyclic deformation capacity. For this result to be obtained the following is necessary:

- the length of the jacket exceeds by no less than 50% the length of the splice region,
- the jacket is pressured against the faces of the column by at least two rows of bolts on each side normal to the direction of loading,
- when splicing occurs at the base of the column, the rows of bolts should be located one at the top of the splice region and another at 1/3 of that region, starting from the base.

A.4.4 FRP plating and wrapping

(1) The main uses of externally bonded FRP (fibre-reinforced polymers) in seismic retrofitting of existing reinforced concrete elements are as follows:

- Enhancement of the shear capacity of columns and walls, by applying externally bonded FRP with the fibers in the hoop direction;
- Enhancement of the available ductility at member ends, through added confinement in the form of FRP jackets, with the fibres oriented along the perimeter;
- Prevention of lap splice failure, through increased lap confinement again with the fibers along the perimeter.

(2) The effect of FRP plating and wrapping of members on the flexural resistance of the end section and on the value of the chord rotation at yielding, θ_y, may be neglected (θ_y may be computed in accordance with **A.3.1.3(2)**).

A.4.4.1 Shear strength

(1) Shear capacity of brittle components can be enhanced in beams, columns or shear walls through the application of FRP strips or sheets. These may be applied either by fully wrapping the element, or by bonding them to the sides and the soffit of the beam (U-shaped strip or sheet), or by bonding them to the sides only.

(2) The total shear capacity, as controlled by the stirrups and the FRP, is evaluated as the sum of one contribution from the existing concrete member, evaluated in accordance with EN 1998-1: 2004 and another contribution, V_f, from the FRP.

(3) The total shear capacity may not be taken greater than the maximum shear resistance of the concrete member, $V_{R,max}$, as controlled by diagonal compression in the web. The value of $V_{R,max}$ may be calculated in accordance with EN1 992-1-1:2004. For concrete walls and for columns with shear span ratio, L_V/h, less or equal to 2, the value of $V_{R,max}$ is the minimum of the value in accordance with EN 1992-1-1:2004 and of the

value calculated from **A.3.2.1(2)** and **A.3.2.1(3)**, respectively, under inelastic cyclic loading.

(4) For members with rectangular section, the FRP contribution to shear capacity may be evaluated as:

– for full wrapping with FRP, or for U-shaped FRP strips or sheets,

$$V_{Rd,f} = 0{,}9d \cdot f_{fdd,e} \cdot 2 \cdot t_f \cdot \left(\frac{w_f}{s_f}\right)^2 \cdot (\cot\theta + \cot\beta) \cdot \sin\beta \quad \text{(A.22)}$$

– for side bonded FRP strips or sheets as:

$$V_{Rd,f} = 0{,}9d \cdot f_{fdd,e} \cdot 2 \cdot t_f \cdot \frac{\sin\beta}{\sin\theta} \cdot \frac{w_f}{s_f} \quad \text{(A.23)}$$

where:

d is the effective depth;

θ is the strut inclination angle;

$f_{\mathrm{fdd,e}}$ is the design FRP effective debonding strength, which depends on the strengthening configuration in accordance with **(5)** for fully wrapped FRP, or **(6)** for U-shaped FRP, or **(7)** for side bonded FRP;

t_{f} is the thickness of the FRP strip, sheet or fabric (on single side);

β is the angle between the (strong) fibre direction in the FRP strip, sheet or fabric and the axis of the member;

w_{f} is the width of the FRP strip or sheet, measured orthogonally to the (strong) direction of the fibres (for sheets: $w_f = \min(0{,}9d, h_w) \cdot \sin(\theta + \beta)/\sin\theta$) and;

s_{f} is the spacing of FRP strips (= w_{f} for sheets), measured orthogonally to the (strong) fibre direction.

(5) For fully wrapped (i.e., closed) or properly anchored (in the compression zone) jackets, the design FRP effective debonding strength may be taken in expressions (A.22), (A.23) as:

$$f_{\mathrm{fdd,e,W}} = f_{\mathrm{fdd}} \cdot \left[1 - k\frac{L_{\mathrm{e}}\sin\beta}{2z}\right] + \frac{1}{2}\left(f_{\mathrm{fu,W}}(R) - f_{\mathrm{fdd}}\right) \cdot \left[1 - \frac{L_{\mathrm{e}}\sin\beta}{z}\right] \quad \text{(A.24)}$$

where:

$z = 0{,}9d$ is the internal lever arm,

$k = \left(1 - \frac{2}{\pi}\right)$, and:

$$f_{\mathrm{fdd}} = \frac{1}{\gamma_{\mathrm{fd}}}\sqrt{0{,}6\frac{E_{\mathrm{f}}\, f_{\mathrm{ctm}}\, k_{\mathrm{b}}}{t_{\mathrm{f}}}} \qquad \text{(units: N, mm)} \quad \text{(A.25)}$$

is the design debonding strength, with:

γ_{fd} the partial factor for FRP debonding;

NOTE The value ascribed to γ_{fd} for use in a country can be found in its National Annex. The recommended value is γ_{fd}=1,5.

E_f the FRP sheets/plates modulus;

f_{ctm} the concrete mean tensile strength;

$k_b = \sqrt{1{,}5 \cdot (2 - w_f / s_f)/(1 + w_f / 100 \text{ mm})}$ the covering coefficient,

in which:

w_f, s_f, t_f are as defined in **(4)** and

$f_{fu,W}(R)$ is the ultimate strength of the FRP strip or sheet wrapped around the corner with a radius R, given by:

$$f_{fu,W}(R) = f_{fdd} + \langle \eta_R \cdot f_{fu} - f_{fdd} \rangle \tag{A.26}$$

where the term in $\langle\cdot\rangle$ should be taken only if positive and where the coefficient η_R depends on the rounding radius R and the beam width b_w as:

$$\eta_R = 0{,}2 + 1{,}6 \frac{R}{b_w} \qquad 0 \le \frac{R}{b_w} \le 0{,}5 \tag{A.27}$$

L_e is the effective bond length:

$$L_e = \sqrt{\frac{E_f \cdot t_f}{\sqrt{4 \cdot \tau_{max}}}} \qquad \text{(units: N, mm)} \tag{A.28}$$

with:

$\tau_{max} = 1{,}8 f_{ctm} k_b$ = maximum bond strength.

(6) For U-shaped (i.e., open) jackets, the design FRP effective debonding strength may be taken in expressions (A.22) and (A.23) as:

$$f_{fdd,e,U} = f_{fdd} \cdot \left[1 - k \frac{L_e \sin\beta}{z} \right] \tag{A.29}$$

where all variables are as defined in **(5)**.

(7) For side-bonded sheets/strips, the design FRP effective debonding strength may be taken in expressions (A.22), (A.23) as:

$$f_{fdd,e,S} = f_{fdd} \cdot \frac{z_{rid,eq}}{z} \cdot \left(1 - \sqrt{k \frac{L_{eq}}{z_{rid,eq}}} \right)^2 \tag{A.30}$$

where:

$$z_{\mathrm{rid,eq}} = z_{\mathrm{rid}} + L_{\mathrm{eq}}, \quad z_{\mathrm{rid}} = z - L_{\mathrm{e}} \cdot \sin\beta, \quad L_{\mathrm{eq}} = \frac{u_1}{\varepsilon_{\mathrm{fdd}}} \cdot \sin\beta \tag{A.31}$$

with:

$\varepsilon_{\mathrm{fdd}} = f_{\mathrm{fdd}}/E_{\mathrm{f}}$, and

$u_1 = k_{\mathrm{b}}/3$.

(8) For members with circular section having diameter D, the FRP contribution is evaluated as:

$$V_{\mathrm{f}} = 0{,}5 \ A_{\mathrm{c}} \cdot \rho_{\mathrm{f}} \cdot E_{\mathrm{f}} \cdot \varepsilon_{\mathrm{f,ed}} \tag{A.32}$$

where:

A_{c} is the column cross-section area;

ρ_{f} equal to 4 t_{f}/D is the volumetric ratio of the FRP, and

$\varepsilon_{\mathrm{f,ed}} = 0{,}004$.

(10) In members with their plastic hinge region fully wrapped in an FRP jacket over a length at least equal to the member depth h, the cyclic shear resistance, V_{R}, may be taken to decrease with the plastic part of the chord rotation ductility demand at the member end: $\mu_{\Delta}^{\mathrm{pl}} = \mu_{\Delta}-1$, in accordance with expression (A.12), adding to V_{w} (i.e. to the contribution of transverse reinforcement to shear resistance) that of the FRP jacket. The contribution of the FRP jacket to V_{w} may be computed assuming that the FRP stress reaches the design value of the FRP ultimate strength, $f_{\mathrm{u,fd}}$, at the extreme tension fibres and reduces linearly to zero over the effective depth d:

$$V_{\mathrm{w,f}} = 0{,}5 \rho_{\mathrm{f}} b_{\mathrm{w}} z f_{\mathrm{u,fd}} \tag{A.33}$$

where:

ρ_{f} equal to $2t_{\mathrm{f}}/b_{\mathrm{w}}$ is the geometric ratio of the FRP;

z is the length of the internal lever arm, taken equal to d; and

$f_{\mathrm{u,fd}}$ is the design value of the FRP ultimate strength, equal to the FRP ultimate strength, $f_{\mathrm{u,f}}$ divided by the partial factor γ_{fd} of the FRP;

NOTE The value ascribed to γ_{fd} for use in a country can be found in its National Annex. The recommended value is $\gamma_{\mathrm{fd}}=1{,}5$.

A.4.4.2 Confinement action

(1) The enhancement of deformation capacity is achieved through concrete confinement by means of FRP jackets. These are applied around the element to be strengthened in the potential plastic hinge region.

(2) The necessary amount of confinement pressure to be applied depends on the ratio $I_{\chi} = \mu_{\phi,\mathrm{tar}}/\mu_{\phi,\mathrm{ava}}$, between the target curvature ductility $\mu_{\phi,\mathrm{tar}}$ and the available curvature ductility $\mu_{\phi,\mathrm{ava}}$, and may be evaluated as:

$$f_{\mathrm{l}} = 0{,}4\ I_{\chi}^{2} \frac{f_{\mathrm{c}} \cdot \varepsilon_{\mathrm{cu}}^{2}}{\varepsilon_{\mathrm{ju}}^{1,5}} \tag{A.34}$$

where:

f_c is the concrete strength, defined as for expression (A.1);

ε_{cu} is the concrete ultimate strain; and

ε_{ju} is the adopted FRP jacket ultimate strain, which is lower than the ultimate strain of FRP, ε_{fu}.

(3) For the case of circular cross-sections wrapped with continuous sheets (not in strips), the confinement pressure applied by the FRP sheet is equal to $f_l = 1/2\rho_f\ E_f\ \varepsilon_{ju}$, with E_f being the FRP elastic modulus and ρ_f the geometric ratio of the FRP jacket related to its thickness as: $t_f = \rho_f D/4$, where D is the diameter of the jacket around the circular cross-section.

(4) For the case of rectangular cross-sections in which the corners have been rounded to allow wrapping the FRP around them (see Figure A.1), the confinement pressure applied by the FRP sheet is evaluated as: $f'_l = k_s f_l$, with $k_s = 2R_c/D$ and $f_l = 2\ E_f\varepsilon_{ju}\ t_f\ /D$, where D is the larger section width.

(5) For the case of wrapping applied through strips with spacing s_f, the confinement pressure applied by the FRP sheet is evaluated as: $f'_l = k_g f_l$, with $k_g = (1 - s_f\ /2D)^2$.

(6) For members of rectangular section with corners rounded as in Figure A.1, an alternative to **(2)** and **(4)** is to calculate the total chord rotation capacity or its plastic part through expressions (A.1) or (A.3), respectively, with the term due to confinement (i.e. the power of 25 before the last term in expressions (A.1) and (A.3)) computed with:

(a) ρ_{sx} (the ratio of transverse steel) replaced by $\rho_f = 2t_f/b_w$, i.e. the FRP ratio parallel to the loading direction;

(b) f_{yw} replaced by an effective stress, $f_{f,e}$, given by the following expression:

$$f_{\mathrm{f,e}} = \min\left(f_{\mathrm{u,f}}, \varepsilon_{\mathrm{u,f}} E_{\mathrm{f}}\right)\left(1 - 0{,}7 \min\left(f_{\mathrm{u,f}}, \varepsilon_{\mathrm{u,f}} E_{\mathrm{f}}\right)\frac{\rho_{\mathrm{f}}}{f_{\mathrm{c}}}\right) \tag{A.35}$$

where $f_{u,f}$ and E_f are the strength and Elastic modulus of the FRP and $\varepsilon_{u,f}$ a limit strain, equal to 0,011 for CFRP and to 0,027 for GFRP; and

(c) the confinement effectiveness factor, α, given by:

$$\alpha = 1 - \frac{(b-2R)^2 + (h-2R)^2}{3bh} \tag{A.36}$$

where R is the radius of the rounded corner of the section and b, h the full cross-sectional dimensions (see Figure A.1).

(7) Paragraph **(6)** applies to members with continuous deformed (high bond) or smooth (plain) longitudinal bars, with or without detailing for earthquake resistance, provided that the end region is wrapped with FRP up to a distance from the end section which is enough to ensure that the yield moment M_y in the unwrapped part will not be exceeded before the flexural overstrength $\gamma_{Rd}M_y$ is reached at the end section. To account for the increase of the flexural strength of the end section due to confinement by the FRP, γ_{Rd} should be at least equal to 1,3.

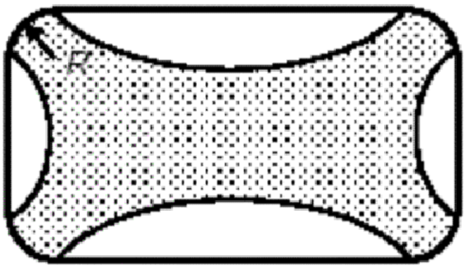


Figure A.1. Effectively confined area in an FRP-wrapped section.

A.4.4.3 Clamping of lap-splices

(1) Slippage of lap-splices can be prevented by applying a lateral pressure σ_l through FRP jackets. For circular columns, having diameter D, the necessary thickness may be estimated as:

$$t_f = \frac{D(\sigma_l - \sigma_{sw})}{2E_f \cdot 0{,}001} \qquad \text{(A.37)}$$

where σ_{sw} is the hoop stress in the stirrups at a strain of 0,001, or the active pressure from the grouting between the FRP and the column, if provided, while σ_{sw} represents the clamping stress over the lap-splice length L_s, as given by:

$$\sigma_l = \frac{A_s f_y}{\left[\frac{p}{2n} + 2(d_{bL} + c)\right]L_s} \qquad \text{(A.38)}$$

where:

A_s is the area of longitudinal steel reinforcement;

f_y is the yield strength of longitudinal steel reinforcement, taken equal to the mean value obtained from *in-situ* tests and from the additional sources of information, appropriately multiplied by the confidence factor, CF, given in Table 3.1 for the appropriate knowledge level (see **2.2.1(5)**P);

p is the perimeter line in the column cross-section along the inside of longitudinal steel;

n is the number of spliced bars along p;

d_{bL} is the (largest) diameter of longitudinal steel bars; and

c is the concrete cover thickness.

(2) For rectangular columns, the expressions above may be used by replacing D by b_w, the section width, and by reducing the effectiveness of FRP jacketing by means of the coefficient in **A.4.4.2** (**4**).

(3) For members of rectangular section with longitudinal bars lapped over a length l_o starting from the end section of the member, an alternative to **(1)** and **(2)** for the calculation of the effect of FRP wrapping over a length exceeding by no less than 25% the length of the lapping, is to calculate the total chord rotation capacity or its plastic part as the product of the value given by expressions (A.1) or (A.3), respectively, multiplied by $0{,}5+\min(0{,}5,\ l_o/100d_{bL})$.

ANNEX B (Informative)

STEEL AND COMPOSITE STRUCTURES

B.1 Scope

This section contains information for the assessment of steel and composite framed buildings in their present state and for their retrofitting, when necessary.

Seismic retrofitting may be either local or global.

B.2 Identification of geometry, details and materials

B.2.1 General

(1) The following aspects should be carefully examined:

i. Current physical conditions of base metal and connector materials including the presence of distortions.

ii. Current physical condition of primary and secondary seismic elements including the presence of any degradation.

B.2.2 Geometry

(1) The collected data should include the following items:

i. Identification of the lateral-force resisting systems.

ii. Identification of horizontal diaphragms.

iii. Original cross-sectional shape and physical dimensions.

iv. Existing cross-sectional area, section moduli, moment of inertia, and torsional properties at critical sections.

B.2.3 Details

(1) The collected data should include the following items:

(i) Size and thickness of additional connected materials, including cover plates, bracing and stiffeners.

(ii) Amount of longitudinal and transverse reinforcement steel in composite beams, columns and walls.

(iii) Amount and proper detailing of confining steel in critical regions.

(iv) As built configuration of intermediate, splice and end connections.

B.2.4 Materials

(1) The collected data should include the following items:

i. Concrete strength.

ii. Steel yield strength, strain hardening, ultimate strength and elongation.

(2) Areas of reduced stress, such as flange tips at beam-column ends and external plate edges, should be selected for inspection as far as possible.

(3) To evaluate material properties, samples should be removed from web plates of hot rolled profiles for components designed as dissipative.

(4) Flange plate specimens should be used to characterise the material properties of non dissipative members and/or joints.

(5) Gamma radiography, ultrasonic testing through the architectural fabric or boroscopic review through drilled access holes are viable testing methods when accessibility is limited or for composite components.

(6) Soundness of base and filler materials should be proved on the basis of chemical and metallurgical data.

(7) Charpy V-Notch toughness tests should be used to prove that heat affected zones, if any, and surrounding material have adequate resistance for brittle fracture.

(8) Destructive and/or non destructive tests (liquid penetrant, magnetic particle, acoustic emission) and ultrasonic or tomographic methods may be used.

B.3 Requirements on geometry and materials of new or modified parts

B.3.1 Geometry

(1) Steel sections of new elements should satisfy width-to-thickness slenderness limitations based on class section classification as in EN 1998-1:2004, Sections **6** and **7**.

(2) The transverse links enhance the rotation capacities of existing or new beam-columns even with slender flanges and webs. Such transverse bars should be welded between the flanges in compliance with EN 1998-1:2004, **7.6.5**.

(3) The transverse links of **(2)** should be spaced as transverse stirrups used for encased members.

B.3.2 Materials

B.3.2.1 Structural steel

(1) Steel satisfying EN 1998-1:2004, **6.2** should be used for new parts or for replacement of existing structural components.

(2) When the strength and stiffness of the structural components are evaluated at each LS, the effects of composite action should be taken into account.

(3) The through-thickness resistance in column flanges should be based upon the reduced strength as follows:

$$f_u = 0{,}90 \cdot f_y \tag{B.1}$$

(4) Element thickness should comply with the requirements of EN 1993-1-10:2004, Table 2.1, depending on the Charpy V-Notch (CVN) energy and other relevant parameters.

(5) Welding consumables should meet the requirements of EN 1993-1-8:2004, **4.2**.

(6) In wide flange sections coupons should be cut from intersection zones between flange and web. This is an area (*k-area*) of potentially reduced notch toughness because of the slow cooling process during fabrication.

B.3.2.2 Reinforcing steel

(1) New reinforcing steel in both dissipative and non dissipative zones of new or modified elements should be of class C in EN 1992-1-1:2004.

B.3.2.3 Concrete

(1) New concrete of new or modified components should conform with EN 1998-1:2004, **7.2.1(1)**.

B.4 System retrofitting

B.4.1 General

(1) Global retrofitting strategies should be able to increase the capacity of lateral-force resisting systems and horizontal diaphragms and/or decrease the demand imposed by seismic actions.

(2) The retrofitted structural system should satisfy the following requirements:

i. Regularity of mass, stiffness and strength distribution, to avoid detrimental torsional effects and/or soft-storey mechanisms.

ii. Masses and stiffness sufficient to avoid highly flexible structures, which may give rise to extensive non-structural damage and significant P-Δ effects.

iii. Continuity and redundancy between members, so as to ensure a clear and uniform load path and prevent brittle failures.

(3) Global interventions should include one or more of the following strategies:

i. Stiffening and strengthening of the structure and its foundation system.

ii. Enhancement of ductility of the structure.

iii. Mass reduction.

iv. Seismic isolation.

v. Supplemental damping.

(4) For all structural systems, stiffening, strengthening and enhancement of ductility may be achieved by using the strategies provided in Sections **B.5** and **B.6**.

(5) Mass reduction may be achieved through one of the following measures:

i. Replacement of heavy cladding systems with lighter systems.

ii. Removal of unused equipment and storage loads.

iii. Replacement of masonry partition walls with lighter systems.

iv. Removal of one or more storeys.

(6) Base isolation should not be used for structures with fundamental periods greater than 1,0 sec. Such periods should be computed through eingenvalue analysis.

(7) Base isolation should be designed in compliance with EN 1998-1:2004 for new buildings.

(8) Re-assessment of the foundation system (after the retrofitting) should be performed in accordance with EN 1998-1:2004, **4.4.2.6.** If linear analysis is used, the values of Ω in **4.4.2.6(4)** will normally be less than 1,0.

B.4.2 Moment resisting frames

(1) The enhancement of the composite action between steel beams and concrete slabs through shear studs, encasement of beams and columns in RC should be used to increase the global stiffness at all limit states.

(2) The length of the dissipative zones should be consistent with the hinge location given at the first row of Table B.6.

(3) Moment resisting frames may be retrofitted through semi-rigid and/or partial strength joints, either steel or composite.

(4) The fundamental period of frames with semi-rigid connections may be computed as follows:

$$T = 0{,}085 \cdot H^{\left(0{,}85 - m/180\right)} \text{ if } 5 < m < 18 \text{ (semi-rigid)} \tag{B.2}$$

$$T = 0{,}085 \cdot H^{3/4} \text{ if } m \geq 18 \text{ (rigid)} \tag{B.3}$$

where H is the frame height in metres and the parameter m is defined as follows:

$$m = \frac{\left(K_{\varphi}\right)_{\mathrm{con}}}{\left(EI/L\right)_{\mathrm{b}}} \tag{B.4}$$

where:

K_{φ} is the connection rotation stiffness;

I is the moment of inertia of the beam

L is the beam span;

E is Young's modulus of the beam.

(5) In addition to the pattern of horizontal forces given in EN 1998-1:2004, **4.3.3.2.3** and in **4.4.4.1(1)** of this standard, the following pattern of forces ($F_{\mathrm{x,i}}$) should be used in the (linear) lateral force analysis and in the nonlinear static (pushover) analysis to detect the onset of all limit states:

$$F_{\mathrm{x,i}} = \frac{W_{\mathrm{x,i}} \cdot h_{\mathrm{x,i}}^{\delta}}{\sum W_{\mathrm{x,i}} \cdot h_{\mathrm{x,i}}^{\delta}} \cdot F_{\mathrm{b}} \tag{B.5}$$

where F_{b} is the seismic base shear and δ is given by:

$$\delta = \begin{cases} 1{,}0 & \text{if} \quad T \leq 0{,}50\,\mathrm{s} \\ 0{,}50 \cdot T + 0{,}75 & \text{if} \quad 0{,}50 < T < 2{,}50\,\mathrm{s} \\ 2{,}0 & \text{if} \quad T > 2{,}50\,\mathrm{s} \end{cases} \tag{B.6}$$

B.4.3 Braced frames

(1) Eccentric and knee-braced frames should be preferred for the retrofitting to concentric braced frames.

(2) Knee-braced frames are systems in which the braces are connected to a dissipative zone (knee element), instead of the beam-to-column connection.

(3) Aluminium or stainless steel may be used for dissipative zones in concentric, eccentric or knee-braced frames, only if their use is validated by testing.

(4) Steel, concrete and/or composite walls may be used in the retrofitting to enhance ductile response and prevent beam-column instability. Their design and that of their connection with steel members should comply with EN 1998-1:2004.

(5) Steel panels may employ low-yield steel and should be shop-welded and field bolted.

(6) Bracing may be introduced in moment resisting frames to increase their lateral stiffness..

B.5 Member assessment and retrofitting

B.5.1 General requirements

(1) Beams should develop full their plastic moments without local buckling in the flange or in the web at the SD LS. Local buckling should be limited at the NC LS.

(2) At the LS of DL and of SD, axial and flexural yielding or buckling should not occur in columns.

(3) Diagonal braces should sustain plastic deformations and dissipate energy through successive cycles of yielding and buckling. At the LS of DL buckling should be avoided.

(4) Steel plates should be welded to flanges and/or webs to reduce the slenderness ratios.

(5) The moment capacity $M_{pb,Rd}$ of the beam at the location of the plastic hinge should be computed as:

$$M_{\mathrm{pb,Rd,b}} = Z_{\mathrm{e}} \cdot f_{yb} \tag{B.7}$$

where:

Z_e is the effective plastic modulus of the section at the plastic hinge location, computed with reference to the actual measured size of the section; and

f_{yb} is the yield strength of the steel in the beam; for existing steel, f_{yb} may be taken equal to the mean value obtained from *in-situ* tests and from the additional sources of information, appropriately multiplied by the confidence factor, CF, given in Table 3.1 for the appropriate knowledge level; for new steel, f_{yb} may be taken equal to the nominal value multiplied by the overstrength factor γ_{ov} for the steel of the beam, determined in accordance with EN 1998-1: 2004: **6.2(3)**, **(4)** and **(5)**.

(6) The moment demand $M_{cf,Ed}$ in the critical section at the column face is evaluated as follows:

$$M_{\mathrm{cf,Ed}} = M_{\mathrm{pl,Rd,b}} + V_{\mathrm{pl,Rd,b}} \cdot e \tag{B.8}$$

where

$M_{pl,Rd,b}$ is the beam plastic moment at the beam plastic hinge;

$V_{pl,Rd,b}$ is the shear at the beam plastic hinge;

e is the distance between the beam plastic hinge and the column face.

(7) The moment demand $M_{cc,Ed}$ in the critical section at column centreline may be calculated as follows:

$$M_{\mathrm{cc,\,Ed}} = M_{\mathrm{pl,\,Rd,\,b}} + V_{\mathrm{pl,\,Rd,\,b}} \cdot \left(e + \frac{d_{\mathrm{c}}}{2} \right) \tag{B.9}$$

where d_c is the column depth.

B.5.2 Member deformation capacities

(1) The inelastic deformation capacities of structural members at the three LSs may be taken as given in the following paragraphs.

(2) The inelastic deformation capacities of beam-to-column joints may be taken equal to those given in a Table B.6 (clause **B.6.1**), provided that connected members fulfil the requirements given in the first five rows of Table B.6.

(3) For beams and columns in flexure, the inelastic deformation capacity should be expressed in terms of the plastic rotation at the end of the member, as a multiple of the chord rotation at yielding, θ_y, at the end in question. For beams and columns with dimensionless axial load v not greater than 0,30, the inelastic deformation capacities at the three LSs may be taken in accordance with Table B.1

Table B.1: Plastic rotation capacity at the end of beams or columns with dimensionless axial load *v* not greater than 0,30.

	Limit State		
Class of cross section	DL	SD	NC
1	1,0 θ_y	6,0 θ_y	8,0 θ_y
2	0,25 θ_y	2,0 θ_y	3,0 θ_y

(4) For braces in compression the inelastic deformation capacity should be expressed in terms of the axial deformation of the brace, as a multiple of the axial deformation of the brace at buckling load, Δ_c. For braces in compression (except for braces of eccentric braced frames) the inelastic deformation capacities at the three LSs may be taken in accordance with Table B.2:

Table B.2: Axial deformation capacity of braces in compression (except braces of eccentric braced frames).

	Limit State		
Class of cross section	DL	SD	NC
1	0,25 Δ_c	4,0 Δ_c	6,0 Δ_c
2	0,25 Δ_c	1,0 Δ_c	2,0 Δ_c

(5) For braces in tension the inelastic deformation capacity should be expressed in terms of the axial deformation of the brace, as a multiple of the axial deformation of the brace at tensile yielding load, Δ_t. For braces in tension (except for braces of eccentric braced frames) with cross section class 1 or 2, the inelastic deformation capacities at the three LSs may be taken in accordance with Table B.3:

Table B.3: Axial deformation capacity of braces in tension (except braces of eccentric braced frames).

Limit State		
DL	SD	NC
0,25 Δ_t	7,0 Δ_t	9,0 Δ_t

(6) For beams or columns in tension the inelastic deformation capacity should be expressed in terms of the axial deformation of the member, as a multiple of its axial deformation at tensile yielding load, Δ_t. For beams or columns in tension (except for those in eccentric braced frames) with cross section class 1 or 2, the inelastic deformation capacities at the three LSs may be taken in accordance with Table B.4.

Table B.4: Axial deformation capacity of beams or columns in tension (except beams or columns of eccentric braced frames).

Limit State		
DL	SD	NC
0,25 Δ_t	3,0 Δ_t	5,0 Δ_t

B.5.3 Beams

B.5.3.1 Stability deficiencies

(1) Beam with span-to-depth ratios between 15 and 18 should be preferred to enhance energy absorption. Therefore, intermediate supports should be used in the retrofitting to shorten long spans.

(2) Lateral restraint should be provided to flanges with a stability deficiency. Lateral restraint of the top flange is not required, if the composite action with the slab is reliable. Otherwise, the composite action should be enhanced by fulfilling the requirements in **B.5.3.5**.

B.5.3.2 Resistance deficiencies

(1) Steel plates should be added to flanges of beams to increase deficient flexural capacity. Addition of steel to the top flange is not required, if the composite action with the slab is reliable. Alternatively, structural steel beams with deficient flexural capacity should be encased in RC.

(2) Longitudinal reinforcing bars that may be added to increase a deficient flexural capacity should be of class C in accordance with EN 1992-1-1:2004, Table C.1. for ductility class H should also be used to perform satisfactory at SD and NC.

(3) Beams retrofitted due to resistance deficiencies, should fulfil the requirements of EN 1998-1:2004 for ductility class M.

(4) Steel plates should be added to the beam web for H-section, or to the wall for hollow sections, to enhance a deficient shear capacity.

B.5.3.3 Repair of buckled and fractured flanges

(1) Buckled and/or fractured flanges should be either strengthened or replaced with new plates.

(2) Buckled bottom and/or top flanges should be repaired by adding full height web stiffeners on both sides of the beam webs in accordance with **(3)** as follows, and by heat straightening of the buckled flange or its removal and replacement with a similar plate in accordance with **(4)** and **(5)** as follows.

(3) Web stiffeners should be located at the edge and centre of the buckled flange, respectively; the stiffener thickness should be equal to the beam web.

(4) New plates should be either welded in the same location as the original flange, (i.e., directly to the beam web), or welded onto the existing flange. In both cases the added plates should be oriented with the rolling direction in the longitudinal direction.

(5) Special shoring of the flange plates should be provided during cutting and replacement.

(6) Instead of welding a thick plate onto the flange, the steel beam should be preferrably encased in RC.

B.5.3.4 Weakening of beams

(1) The ductility of steel beams may be improved by weakening of the beam flange at desired locations, to shift the dissipative zones away from the connections.

(2) Reduced beam sections (RBSs) or dog-bones behave like a fuse that protects beam-to-column connections against early fracture. The reduced beam sections should be such that they can develop at each LS the minimum rotations specified in Table B.5.

Table B.5. - Required rotation capacity of reduced beam sections, RBSs (in *radians*).

DL	SD	NC
0,010	0,025	0,040

(3) The rotations in Table B.5 may be considered to be achieved, if the design of RBS in the beam is carried out through the procedure outlined hereafter:

i. Compute the distance of the beginning of the RBS from the column face, a, and the length over which the flange will be reduced, b, as follows:

$$a = 0{,}60 b_{\mathrm{f}} \tag{B.10}$$

$$b = 0{,}75 d_{\mathrm{b}} \tag{B.11}$$

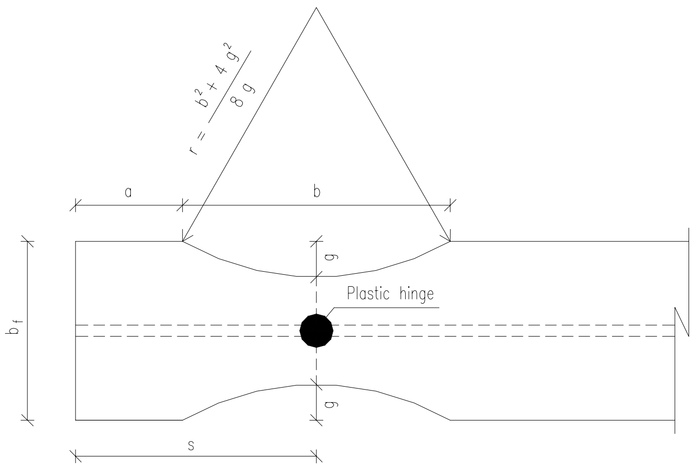
where:

b_f is the flange width

d_b is the beam depth.

ii. Compute the distance of the intended plastic hinge section at the centre of the RBS, s, from the column face as:

$$s = a + \frac{b}{2} \tag{B.12}$$



Key:

A = Plastic hinge

Figure B.1. - Geometry of flange reduction for reduced beam section (RBS).

iii. Determine the depth of the flange cut (g) on each side; this depth should be not greater than $0{,}25 \cdot b_f$. As a first trial it may be taken as:

$$g = 0{,}20 b_f \tag{B.13}$$

iv. Compute the plastic modulus (Z_{RBS}) and the plastic moment ($M_{pl,Rd,RBS}$) of the plastic hinge section at the centre of the RBS:

$$Z_{RBS} = Z_b - 2 \cdot g \cdot t_f \cdot (d_b - t_f) \tag{B.14}$$

$$M_{pl,Rd,RBS} = Z_{RBS} \cdot f_{yb} \tag{B.15}$$

where Z_b is the plastic modulus of the beam and f_{yb} is as defined in **B.5.1(5)**.

v. Compute the shear force ($V_{pl,\ RBS}$) in the section of plastic hinge formation from equilibrium of the beam part (L') between the two intended plastic hinges (Figure B.2). For a uniform gravity load w the acting on the beam in the seismic design situation:

$$V_{pl,RBS} = \frac{2 M_{pl,Rd,RBS}}{L'} + \frac{wL'}{2} \tag{B.16}$$

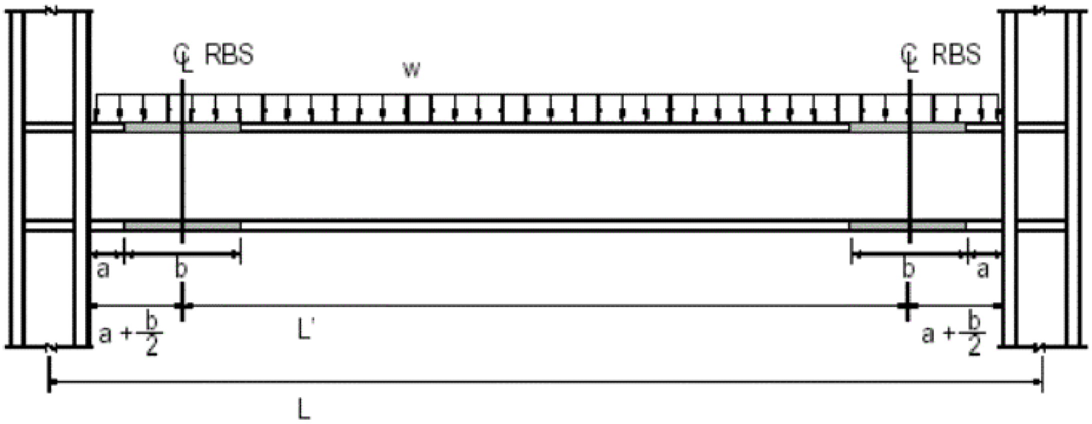
Different distributions of the gravity loads along the beam span should be properly accounted for in (the last term of) Expression (B.16).

vi. Compute the beam plastic moment away from the RBS, $M_{pl,Rd,b}$, as follows:

$$M_{pl,Rd,b} = Z_b \cdot f_{yb} \tag{B.17}$$

where Z_b and f_{yb} are as defined in step (iv) above.

vii. Verify that $M_{pl,Rd,b}$ is greater than the bending moment that develops at the column face when a plastic hinge forms at the centre of the RBS: $M_{cf,Ed} = M_{pb,Rd,RBS} + V_{pl,RBS} \cdot e$. If it is not, increase the cut-depth c and repeat steps (iv) to (vi). The length g should be chosen such that $M_{cf,Ed}$ is about 85% to 100% of $M_{pl,Rd,b}$.



Key:

w = uniform gravity load in the seismic design situation

L' = Distance between the centres of RBS cuts

L = Distance between column centerlines

Figure B.2. -Typical sub-frame assembly with reduced beam sections (RBS).

viii. Check the width-to-thickness ratios at the RBS to prevent local buckling. The flange width should be measured at the ends of the central two-thirds of the reduced section of the beam.

ix. Compute the radius (r) of the cuts in both top and bottom flanges over the length b of the RBS of the beam:

$$r = \frac{b^2 + 4g^2}{8g} \tag{B.18}$$

x. Check that the fabrication process ensures the adequate surface roughness (i.e. between 10 and 15 μm) for the finished cuts and that grind marks are not present.

B.5.3.5 Composite elements

(1) The calculation of the capacity of composite beams should take into account the degree of shear connection between the steel member and the slab.

(2) Shear connectors between steel beams and composite slabs should not be used within dissipative zones. They should be removed from existing composite beams.

(3) Studs should be attached to flanges through arc-spot welds, but without full penetration of the flange. Shot or screwed attachments should be avoided.

(4) The maximum tensile strains due to the presence of composite slabs should be checked that they do not provoke flange tearing.

(5) Encased beams should be provided with stiffeners and stirrups.

B.5.4 Columns

B.5.4.1 Stability deficiencies

(1) The width-to-thickness ratio may be reduced by welding steel plates to the flange and/or the webs.

(2) The width-to-thickness ratio of hollow sections may be reduced by welding external steel plates.

(3) Lateral restraint should be provided to both flanges, through stiffeners with strength not less than:

$$0{,}06 f_{\mathrm{yc}} \cdot b_{\mathrm{f}} \cdot t_{\mathrm{f}} \qquad \text{(B.19)}$$

where:

b_{f} is the flange width;

t_{f} is the flange thickness; and

f_{yc} is the yield strength of the steel in the column; for existing steel, f_{yc} may be taken equal to the mean value obtained from *in-situ* tests and from the additional sources of information, multiplied by the confidence factor, CF, given in Table 3.1 for the appropriate knowledge level; for new steel, f_{yc} may be taken equal to the nominal value multiplied by the overstrength factor γ_{ov} for the steel of the column, determined in accordance with EN 1998-1:2004, **6.2(3)**, **(4)** and **(5)**.

B.5.4.2 Resistance deficiencies

(1) To increase the flexural capacity of the section, steel plates may be welded to the flanges and/or webs for H-sections and to the walls for hollow sections.

(2) Structural steel columns may be encased in RC, to increase their flexural capacity.

(3) Retrofitting should ensure that in all primary seismic columns the axial compression in the design seismic situation is not greater than 1/3 of the design value of the plastic resistance to normal forces of the gross cross-section of the column $N_{pl,Rd} = (A_a f_{yd} + A_c f_{cd} + A_s f_{sd})$ at the DL LS and 1/2 of $N_{pl,Rd}$ at the SD or NC LSs.

B.5.4.3 Repair of buckled and fractured flanges and of fractures of splices

(1) Buckled and/or fractured flanges and fractured splices should be either strengthened or replaced with new plates.

(2) Buckled and fractured flanges should be repaired either through removal of the buckled plate flange and replacement with a similar plate, or through flame straightening.

(3) Splice fractures should be repaired by adding external plates on the column flanges via complete penetration groove welds. The damaged part should be removed and replaced with sound material. The thickness of the added plates should be equal to that of the existing ones. The replacement material should be aligned so that the rolling direction matches that of the column.

(4) Small holes should be drilled at the edge of cracks in columns to prevent propagation.

(5) Magnetic particle, or liquid dye penetrant tests should be used to ensure that there are no further defects and/or discontinuities up to a distance of 150mm from a cracks.

B.5.4.4 Requirements for column splices

(1) New splices should be located in the middle third of the column clear height. They should be designed to develop a design shear strength not less than the smaller of the expected shear strengths of the two connected members and a design flexural strength not less than 50% of the smaller of the expected flexural strengths of the two connected sections. Thus, each flange of welded column splices should satisfy the following:

$$A_{spl} \cdot f_{yd} \geq 0,50 \cdot f_{yc} \cdot A_{fl} \tag{B.20}$$

where:

A_{spl} is the area of each flange of the splice;

f_{yd} is the design yield strength of the flange of the splice;

A_{fl} is the flange area of the smaller of the two columns connected; and

f_{yc} is the yield strength of the column material, defined as in **B.5.4.1(3)**.

B.5.4.5 Column panel zone

(1) In the retrofitted column the panel zone at beam-column connections should remain elastic at the DL LS.

(2) The thickness, t_w, of the column panel zone (including the doubler plate, if any, see **(3)**) should satisfy the following expression, to prevent premature local buckling under large inelastic shear deformations:

$$t_w \leq \frac{d_z + w_z}{90} \tag{B.21}$$

where:

d_z is the panel-zone depth between continuity plates;

w_z is the panel-zone width between column flanges.

Plug welds should be used between the web and the added plate.

(3) Steel plates parallel to the web and welded to the tip of flanges (doubler plates) may be used to stiffen and strengthen the column web.

(4) Transverse stiffeners should be welded onto the column web, at the level of the beam flanges.

(5) To ensure satisfactory performance at all limit states, continuity plates with thickness not less than that of beam flanges should be placed symmetrically on both sides of the column web.

B.5.4.6 Composite elements

(1) Encasement in RC may be used to enhance the stiffness, strength and ductility of steel columns.

(2) To achieve effective composite action, shear stresses should be transferred between the structural steel and reinforced concrete through shear connectors placed along the column.

(3) To prevent shear bond failure, the ratio of the steel flange width to that of the composite column, b_f/B, should not be greater than the critical value of this ratio defined as follows:

$$\left(\frac{b_f}{B}\right)_{cr} = 1 - 0{,}35 \cdot \left[0{,}17 \cdot \left(1 + 0{,}073 \cdot \frac{N_{Ed}}{A_g}\right) \cdot \sqrt{f_{cd}} + 0{,}20 \cdot \rho_w \cdot f_{yw,d}\right] \tag{B.22}$$

where:

N_{Ed} is the axial force in the seismic design situation;

A_g is the gross area of the composite section;

f_{cd} is the design value of compressive strength of the concrete;

ρ_w is the ratio of transverse reinforcement;

$f_{yw,d}$ is the design value of the yield strength of transverse reinforcement;

B is the width of the composite section;

b_f is the steel flange width.

B.5.5 Bracings

B.5.5.1 Stability deficiencies

(1) **5.3.1(1)** applies for bracings consisting of hollow sections.

(2) **5.3.2(1)** applies.

(3) Any encasement of steel bracings for retrofitting should comply with EN 1998-1:2004.

(4) Lateral stiffness of diagonal braces may be improved by increasing the stiffness of the end connections.

(5) X bracings should be preferred for the retrofitting over V or inverted V bracings. K bracings may not be used.

(6) Closely spaced batten plates are effective in improving the post-buckling response of braces, particularly in double-angle or double-channel ones. If batten plates are already in place in the existing columns, new plates may be welded and/or existing batten connections should be strengthened.

B.5.5.2 Resistance deficiencies

(1) At the DL LS the axial compression in the design seismic situation should be not greater than 80% of the design value of the plastic resistance to normal forces of the cross-section of the bracing: $N_{pl,Rd.}$

(2) Unless only the NC LS is verified, the capacity in compression of the braces of concentrically braced frames should be not less than 50% of the plastic resistance to normal forces of the cross-section, $N_{pl,Rd}$.

B.5.5.3 Composite elements

(1) Encasement of steel bracings in RC increases their stiffness, strength and ductility. For steel braces with H-section, partial or full encasement may be used.

(2) Fully encased bracings should be provided with stiffeners and stirrups, and partially encased ones with straight links in accordance with EN 1998-1:2004, **7.6.5**. Stirrups should have uniform spacing along the brace and should comply with the requirements specified for ductility class M in EN 1998-1:2004, **7.6.4(3)**, **(4)**.

(3) Only the structural steel section should be taken into account in the calculation of the capacity of composite braces in tension.

B.5.5.4 Unbonded bracings

(1) Braces may be stiffened by being incorporated unbonded either in RC walls or in concrete-filled tubes.

(2) The brace should be coated with debonding material, to reduce bond between the steel component and the RC panel or the concrete infilling the tube.

(3) Low yield strength steels is appropriate for the steel brace; steel-fibre reinforced concrete may be used as unbonding material.

(4) Braces stiffened by being incorporated unbonded in RC walls should conform with the following:

$$\left(1-\frac{1}{n_{\mathrm{E}}^{\mathrm{B}}}\right)\cdot m_{\mathrm{y}}^{\mathrm{B}} > 1{,}30\cdot\frac{a}{l} \tag{B.23}$$

where:

a is the initial imperfection of the steel brace;

l is the length of the steel brace;

$m_{\mathrm{y}}^{\mathrm{B}}$ is the non-dimensional strength parameter of the RC panel:

$$m_{\mathrm{y}}^{\mathrm{B}} = \frac{M_{\mathrm{y}}^{\mathrm{B}}}{N_{\mathrm{pl,R}}\cdot l} \tag{B.24}$$

$n_{\mathrm{E}}^{\mathrm{B}}$ is the non-dimensional stiffness parameter of the RC panel:

$$n_{\mathrm{E}}^{\mathrm{B}} = \frac{N_{\mathrm{E}}^{\mathrm{B}}}{N_{\mathrm{pl,R}}} \tag{B.25}$$

where:

$$M_{\mathrm{y}}^{\mathrm{B}} = \frac{5\cdot B_{\mathrm{S}}\cdot t_{\mathrm{c}}^{2}\cdot f_{\mathrm{ct}}}{6} \tag{B.26}$$

$$N_{\mathrm{E}}^{\mathrm{B}} = \frac{5\cdot \pi^{2}\cdot B_{\mathrm{S}}\cdot E_{\mathrm{c}}\cdot t_{\mathrm{c}}^{3}}{12\cdot l^{2}} \tag{B.27}$$

where:

E_{c} is the elastic modulus of concrete;

B_{S} is the width of the steel brace in the form of a flat bar;

t_{c} is the thickness of the RC panel;

f_{ct} is the tensile strength of concrete;

$N_{\mathrm{pl,R}}$ is the plastic capacity of the steel brace in tension, computed on the basis of the mean value of steel yield stress obtained from *in-situ* tests and from the additional sources of information, divided by the confidence factor, CF, given in Table 3.1 for the appropriate knowledge level.

(6) Edge reinforcement of the RC panel should be adequately anchored to prevent failure by punching shear.

(7) The infilled concrete tubes with debonding material should be adequate to prevent buckling of the steel brace.

B.6 Connection retrofitting

(1) Connections of retrofitted members should be checked taking into account the resistance of the retrofitted members, which may be higher than that of the original ones (before retrofitting).

(2) The retrofitting strategies provided may be applied to steel or composite moment and braced frames.

B.6.1 Beam-to-column connections

(1) The retrofitting should aim at shifting the beam plastic hinge away from the column face (see first row in Table B.6).

(2) Beam-to-column connections may be retrofitted through either weld replacement, or a weakening strategy, or a strengthening strategy.

(3) To ensure development of plastic hinges in beams, rather than in columns, the column-to-beam moment ratio (*CBMR*) should satisfy the following condition:

$$CBMR = \frac{\sum M_{\mathrm{Rd,c}}}{\sum M_{\mathrm{pl,R,b}}} \geq 1{,}30 \qquad \text{(B.28)}$$

where:

(a) for the steel columns: $$\sum M_{\mathrm{Rd,c}} = \sum \left[Z_{\mathrm{c}} \cdot \left(f_{\mathrm{yd,c}} - \frac{N_{\mathrm{Ed}}}{A_{\mathrm{c}}} \right) \right]_{\mathrm{i}} \qquad \text{(B.29)}$$

where the summation extends over the column sections around the joint, and:

Z_{c} is the plastic modulus of the column section, evaluated on the basis of actual geometrical properties, if available, and taking into account haunches, if any;

N_{Ed} is the axial load of the column in the seismic design situation;

A_{c} is the area of the column section;

$f_{\mathrm{yd,c}}$ is the design yield strength of steel in the column, computed on the basis of the mean value of steel yield stress obtained from *in-situ* tests and from the additional sources of information, divided by the confidence factor, CF, given in Table 3.1 for the appropriate knowledge level..

(b) $\sum M_{\mathrm{pl,R,b}}$ is the sum of flexural strengths at plastic hinge locations in beams framing into the joint in the horizontal direction considered, taking into account the eccentricity to the column centreline:

$$\sum M_{\mathrm{pl,R,b}} = \sum \left(Z_{\mathrm{b}} \cdot f_{\mathrm{yb}} + M_{\mathrm{cc,Ed}} \right)_{\mathrm{j}} \qquad \text{(B.30)}$$

where:

Z_b is the plastic modulus of the beam section at the potential plastic hinge location, computed on the basis of the actual geometry;

f_{yb} is the yield strength of steel in the beam; defined as in **B.5.1(5).**

$M_{cc,Ed}$ is the additional moment at the column centreline due to the eccentricity of the shear force at the plastic hinge in the beam.

(4) The requirements for beams and columns in retrofitted connections are given in Table B.6. The same Table gives the rotation capacity at the three LSs that is provided by the connection if the requirements are fulfilled.

B.6.1.1 Weld replacement

(1) The existing filler material should be removed and replaced with sound material.

(2) Backing bars should be removed after welding, because they may cause initiation of cracks.

(3) Transverse stiffeners at the top and bottom of the panel zone should be used to strengthen and stiffen the column panel (see **B.5.4.5(4)**). Their thickness should be not less than that of beam flanges.

(4) Transverse and web stiffeners should be welded to column flanges and to the web via complete joint penetration welds.

Table B.6. – Requirements on retrofitted connections and resulting rotation capacities.

	IWUFCs	WBHCs	WTBHCs	WCPFCs	RBSCs
Hinge location (from column centreline)	$(d_c/2) + (d_b/2)$	$(d_c/2) + l_h$	$(d_c/2) + l_h$	$(d_c/2) + l_{cp}$	$(d_c/2)+(b/2)+a$
Beam depth (mm)	≤1000	≤1000	≤1000	≤1000	≤1000
Beam span-to-depth ratio	≥7	≥7	≥7	≥7	≥7
Beam flange thickness (mm)	≤25	≤25	≤25	≤25	≤44
Column depth (mm)	No restriction	≤570	≤570	≤570	≤570
Rotation at DL LS (rads)	0,013	0,018	0,018	0,018	0,020
Rotation at SD LS (rads)	0,030	0,038	0,038	0,040	0,030
Rotation at NC LS (rads)	0,050	0,054	0,052	0,060	0,045

Keys:

IWUFCs = Improved welded unreinforced flange connections.

WBHCs = Welded bottom haunch connections.

WTBHCs = Welded top and bottom haunch connections.

WCPFCs = Welded cover plate flange connections.

RBSCs = Reduced beam section connections.

d_c = Column depth.

d_b = Beam depth.

l_h = Haunch length.

l_{cp} = Cover plate length.

a = Distance of the radius cut from the beam edge.

b = Length of the radius-cut.

B.6.1.2 Weakening strategies

B.6.1.2.1 Connections with RBS beams

(1) Reduced Beam Sections (RBS), designed in accordance with **(5)**, can force plastic hinges to occur within the reduced section, thus decreasing the likelihood of fracture at the beam flange welds and in the surrounding heat affected zones.

(2) The beam should be connected to the column flange either through welded webs, or through shear tabs welded to the column flange face and to the beam web. The tab length should be equal to the distance between the weld access holes, with an offset of 5 mm. A minimum tab thickness of 10 mm is required. Shear tabs should be either cut square or with tapered edges (tapering corner about 15°) and should be placed on both sides of the beam web.

(3) Welding should employ groove welds or fillet welds for the column flange and fillet welds for the beam web. Bolting of the shear tab to the beam web is allowed as an alternative.

(4) Shear studs should not be placed within the RBS zones.

(5) The design procedure for RBS connections is outlined below:

i. Use RBS beams designed in accordance with the procedure in **B.5.3.4**, but computing the beam plastic moment, $M_{pl,Rd,b}$, as:

$$M_{pl,Rd,b} = Z_{RBS} \cdot f_{yb} \cdot \left(\frac{L - d_c}{L - d_c - 2 \cdot b} \right) \tag{B.31}$$

where:

f_{yb} is the yield strength of steel in the beam; defined as in **B.5.1(5).**

L is the distance between column centerlines;

d_c is the column depth; and

b is the length of RBS.

ii. Compute the beam shear, $V_{pl,Rd,b}$, in accordance with **B.5.3.4(3)**v for a span length between plastic hinges, L':

$$L' = L - d_c - 2 \cdot b \tag{B.32}$$

iii. Verify the web connection, e.g. the welded shear tab, for the shear force $V_{pl,Rd,b}$ from ii above.

iv. Check that the column-to-beam flexural capacity ratio, *CBMR*, satisfies the condition:

$$CBMR = \frac{\sum Z_c \left(f_{yd,c} - \frac{N_{Ed}}{A_c} \right)}{\sum Z_b \cdot f_{yb} \cdot \left(\frac{L - d_c}{L - d_c - 2 \cdot b} \right)} \geq 1{,}20 \tag{B.33}$$

where:

Z_b and Z_c plastic moduli of the beams and the columns, respectively;

N_{Ed} is the axial load of the column in the seismic design situation;

A_c is the area of the column section.

f_{yb} is the yield strength of steel in the beam; defined as in **B.5.1(5).**

$f_{yd,c}$ is the design yield strength of steel in the column, defined as in **B.6.1(3)**.

v. Determine the thickness of the continuity plates to stiffen the column web at the level of the top and bottom beam flange. This thickness should be at least equal to that of the beam flange.

vi. Check that the strength and stiffness of the panel zone are sufficient for the panel to remain elastic:

$$d_{\mathrm{c}} \cdot t_{\mathrm{wc}} \cdot \frac{f_{\mathrm{yw,d}}}{\sqrt{3}} \geq \frac{\sum Z_{\mathrm{b}} \cdot f_{\mathrm{yb}}}{d_{\mathrm{b}}} \cdot \left(\frac{L - d_{\mathrm{c}}}{L - d_{\mathrm{c}} - 2 \cdot b}\right) \cdot \left(\frac{H - d_{\mathrm{b}}}{H}\right) \tag{B.34}$$

where:

d_{c} is thedepth of the column web;

t_{wc} is the thickness of the column web, including the doubler plates, if any;

$f_{\mathrm{yw,d}}$ is the design yield strength of the panel zone;

Z_{b} is the plastic modulus of the beams;

N_{Ed} is the axial load of the column in the seismic design situation;

A_{c} is the area of the column section.

f_{yb} is the yield strength of steel in the beam; defined as in **B.5.1(5)**; and

H is the frame storey height.

vii. Compute and detail the welds between the joined parts.

B.6.1.2.2 Semi-rigid connections

(1) Semi-rigid and/or partial strength connections, either steel or composite, may be used to achieve large plastic deformations without risk of fracture.

(2) Full interaction shear studs should be welded onto the beam top flange.

(3) Semi-rigid connections may be designed by assuming that the shear resistance is provided by the components on the web and the flexural resistance by the beam flanges and the slab reinforcement, if any.

B.6.1.3 Strengthening strategies

B.6.1.3.1 Haunched connections

(1) Beam-to-column connections may be strengthened by adding haunches either only to the bottom, or to the top and the bottom of the beam flanges, forcing the dissipative zone to the end of the haunch. Adding haunches only to the bottom flange is more convenient, because bottom flanges are generally far more accessible than top ones; moreover, the composite slab, if any, does not have to be removed.

(2) Triangular T-shaped haunches are the most effective among the different types of haunch details. If only bottom haunches are added, their depth should be about one-quarter of the beam depth. In connections with top and bottom haunches, haunch depth should be about one-third of the beam height.

(3) Transverse stiffeners at the level of the top and bottom beam flanges should be used to strengthen the column panel zone.

(4) Transverse stiffeners should also be used at the haunch edges, to stiffen the column web and the beam web.

(5) The vertical stiffeners for the beam web should be full depth and welded on both sides of the web. Their thickness should be sufficient to resist the vertical component of the haunch flange force at that location, and should be not less than the thickness of the beam flange. The local verifications in EN 1993-1-8:2004, **6.2.6** should be satisfied.

(6) Haunches should be welded with complete joint penetration welds to both the column and the beam flanges.

(7) Bolted shear tabs may be left in place, if they exist. Shear tabs may be used in the retrofitted member, if required either for resistance or for execution purposes.

(8) A step-by-step design procedure may be applied for haunched connections, as follows.

i. Select preliminary haunch dimensions on the basis of the slenderness limitation for the haunch web. The following relationships may be used as a first trial for the haunch length, a, and for the angle of the haunch flange to the haunch of the member, θ:

$$a = 0{,}55 \cdot d_{\mathrm{b}} \qquad \text{(B.35)}$$

$$\theta = 30° \qquad \text{(B.36)}$$

where d_{b} is the beam depth. The resulting haunch depth *b,* given by:

$$b = a \cdot \tan\theta. \qquad \text{(B.37)}$$

should respect architectural constraints, e.g. ceilings and non structural elements.

ii. Compute the beam plastic moment at the haunch tip, $M_{\mathrm{pl,Rd,b,}}$ from expression (B.17).

iii. Compute the beam plastic shear ($V_{\mathrm{pl,Rd,b}}$) in accordance with **B.5.3.4(3)**v for the span length *L*' between the plastic hinges at the ends of the haunches.

iv. Verify that the column-to-beam flexural capacity ratio, *CBMR*, satisfies the condition:

$$CBMR = \frac{\sum Z_{\mathrm{c}} \cdot \left(f_{\mathrm{yd,c}} - \dfrac{N_{\mathrm{Ed}}}{A_{\mathrm{c}}} \right)}{\sum M_{\mathrm{c}}} \geq 1{,}20 \qquad \text{(B.38)}$$

where:

Z_c is the plastic section modulus of the columns,

$f_{yd,c}$ is the design yield strength of steel in the column, defined as in B.6.1(3);

N_{Ed} is the axial load of the column in the seismic design situation;

A_c is the area of the column section.

M_c is the sum of column moments at the top and bottom ends of the enlarged panel zone resulting from the development of the beam moment $M_{pl,R,b}$ within each beam of the connection:

$$\sum M_c = \left[2M_{pl,R,b} + V_{pl,Rd,b} \cdot (L - L')\right] \cdot \left(\frac{H_c - \overline{d_b}}{H_c}\right) \tag{B.39}$$

where:

L is the distance between the column centrelines;

$\overline{d_b}$ is the depth of the beam including the haunch; and

H_c is the storey height of the frame.

v. Compute the value of the non-dimensional parameter β given by:

$$\beta = \frac{b}{a} \cdot \left(\frac{3 \cdot L' \cdot d + 3 \cdot a \cdot d + 3 \cdot b \cdot L' + 4 \cdot a \cdot b}{3 \cdot d^2 + 6 \cdot b \cdot d + 4 \cdot b^2 + \dfrac{12 \cdot I_b}{A_b} + \dfrac{12 \cdot I_b}{A_{hf} \cos^3 \theta}} \right) \tag{B.40}$$

where A_{hf} is the area of the haunch flange.

vi. Compute the value of the non-dimensional parameter β_{min} as:

$$\beta_{min} = \frac{\dfrac{\left(M_{pl,Rd,b} + V_{pl,Rd,b} \cdot a\right)}{S_x} - 0{,}80 \cdot f_{uw,d}}{\dfrac{V_{pl,Rd,b} \cdot a}{S_x} + \dfrac{V_{pl,Rd,b}}{I_b \cdot \tan\theta} \cdot \left(\dfrac{d^2}{4} - \dfrac{I_b}{A_b}\right)} \tag{B.41}$$

where:

$f_{uw,d}$ is the design tensile strength of the welds;

S_x is the beam (major) elastic modulus;

d is the beam depth;

A_b and I_b are respectively the area and moment of inertia of the beam.

vii. Compare the non-dimensional β-values, as calculated above. If $\beta \geq \beta_{min}$ the haunch dimensions are sufficient and further local checks should be performed in accordance with viii below. If $\beta < \beta_{min}$ the haunch flange stiffness should be increased, by either increasing the haunch flange area A_{hf} or by modifying the haunch geometry.

viii. Perform strength and stability checks for the haunch flange:

(strength) $$A_{\mathrm{hf}} \geq \frac{\beta \cdot V_{\mathrm{pl,Rd,b}}}{f_{\mathrm{yhf,d}} \cdot \sin\theta} \tag{B.42}$$

(stability) $$\frac{b_{\mathrm{hf}}}{t_{\mathrm{hf}}} \leq 10 \cdot \sqrt{\frac{235}{f_{\mathrm{yhf,d}}}} \tag{B.43}$$

where:

$f_{\mathrm{yhf,d}}$ is the design value of the yield strength of the haunch flange;

b_{hf} and t_{hw} are the flange outstand and the flange thickness of the haunch, respectively.

ix. Perform strength and stability checks for the haunch web:

(strength) $$\tau_{\mathrm{hw}} = \frac{a \cdot V_{\mathrm{pl,Rd,b}}}{2 \cdot (1+\upsilon) \cdot I_{\mathrm{b}}} \left[\frac{L`}{2} - \frac{\beta}{\tan\theta} \left(\frac{d}{2} \right) + \frac{(1-\beta) \cdot a}{3} \right] \leq \frac{f_{\mathrm{yhwd}}}{\sqrt{3}} \tag{B.44}$$

(stability) $$\frac{2 \cdot \mathrm{a} \cdot \sin\theta}{t_{\mathrm{hw}}} \leq 33 \cdot \sqrt{\frac{235}{f_{\mathrm{yhwd}}}} \tag{B.45}$$

where:

$f_{\mathrm{yhw,d}}$ is the design value of the yield strength of the haunch web;

t_{hw} is the web thickness;

ν is the Poisson ratio of steel.

x. Check the shear capacity of the beam web in accordance with EN 1993-1-8: 2004, **6.2.6,** for a shear force to be resisted by the beam web given by:

$$V_{\mathrm{pl,Rd,bw}} = (1-\beta) \cdot V_{\mathrm{pl,Rd,b}} \tag{B.46}$$

where β is given by expression (B.40).

xi. Design transverse and beam web stiffeners to resist the concentrated force $\beta V_{\mathrm{pl,Rd,b}}/\tan\theta$. Web stiffeners should possess sufficient strength to resist, together with the beam web, the concentrated load $(1-\beta) V_{\mathrm{pl,Rd,b}}$. Width-to-thickness ratios for stiffeners should be limited to 15, to prevent local buckling.

xii. Detail welds with complete joint penetration welding to connect stiffeners to the beam flange. Two-sided 8 mm fillet welds are sufficient to connect the stiffeners to the beam web.

B.6.1.3.2 Cover plate connections

(1) Cover plate connections can reduce the stress at the welds of the beam flange and force yielding in the beam to occur at the end of the cover plates.

(2) Cover plates may be used either only at the bottom beam flange, or at the top and bottom beam flanges.

(3) Steel cover plates should have rectangular shape and should be placed with the rolling direction parallel to the beam.

(4) Connections with welded beam webs and relatively thin and short cover plates should be preferred over bolted web and heavy and long plates.

(5) Long plates should not be used for beams with short spans and high shear forces.

(6) A step-by-step design procedure may be applied for cover plate connections as follows.

i. Select the cover plate dimensions on the basis of the beam size:

$$b_{\mathrm{cp}} = b_{\mathrm{bf}} \tag{B.47}$$

$$t_{\mathrm{cp}} = 1{,}20 \cdot t_{\mathrm{bf}} \tag{B.48}$$

$$l_{\mathrm{cp}} = \frac{d_{\mathrm{b}}}{2} \tag{B.49}$$

where:

b_{cp} is the width of the cover plate;

t_{cp} is the thickness of the cover plate;

b_{cf} is the width of the beam flange;

t_{cf} is the thickness of the beam flange;

l_{cp} is the length of the cover plate; and

d_{b} is the beam depth.

ii. Compute the beam plastic moment ($M_{\mathrm{pl,Rd,b}}$) at the end of the cover plates as in expression (B.7).

iii. Compute the beam plastic shear, $V_{\mathrm{pl,Rd,b}}$, in accordance with **B.5.3.4(3)**v for the distance, L', between the plastic hinges in the beam:

$$L` = L - d_{\mathrm{c}} - 2 \cdot l_{\mathrm{cp}} \tag{B.50}$$

iv. Compute the moment at the column flange, $M_{\mathrm{cf,Ed}}$):

$$M_{\mathrm{cf,Ed}} = M_{\mathrm{pl,Rd,b}} + V_{\mathrm{pl,Rd,b}} \cdot l_{\mathrm{cp}} \tag{B.51}$$

v. Verify that the area of cover plates, A_{cp}, satisfies the requirement:

$$\left\lfloor Z_b + A_{cp} \cdot \left(d_b + t_{cp}\right)\right\rfloor \cdot f_{yd} \geq M_{cf,Sd} \tag{B.52}$$

where f_{yd} is the design yield strength of the cover plates

vi. Verify that, the column-to-beam flexural capacity ratio, *CBMR*, satisfies the condition:

$$CBMR = \frac{\sum Z_c \left(f_{yd,c} - f_a\right)}{\sum Z_b \cdot f_{yb} \cdot \left(\frac{L - d_c}{L - d_c - 2 \cdot L_{cp}}\right)} \geq 1{,}20 \tag{B.53}$$

where:

Z_b and Z_c are the plastic moduli of the beams and the columns, respectively;

f_{yb} is the yield strength of steel in the beam; defined as in **B.5.1(5);** and

$f_{yd,c}$ is the design yield strength of steel in the column, defined as in B.6.1(3).

vii. Determine the thickness of the continuity plates placed at the level of the top and bottom beam flanges to stiffen the column web. This thickness should be not less than that of the beam flange.

viii. Check that the strength and the stiffness of the panel zone are sufficient for the panel to remain elastic:

$$d_c \cdot t_{wc} \cdot \frac{f_{yw,d}}{\sqrt{3}} \geq \frac{\sum M_f}{d_b} \cdot \left(\frac{L}{L - d_c}\right) \cdot \left(\frac{H - d_b}{H}\right) \tag{B.54}$$

where:

d_c is the depth of the column web;

t_{wc} is the thickness of the column web, including the doubler plates, if any;

$f_{yw,d}$ is the design value of the yield strength of the panel zone; and

H is the frame storey height.

ix. Dimension and detail the welds between joined parts, i.e. between the beam and the cover plates, between the column and the cover plates and between the beam and the column. Weld overlays should employ the same electrodes as used in the original welds, or at least electrodes with similar mechanical properties.

B.6.2 Connections of braces and of seismic links

(1) The connections of braces and of seismic link should be designed taking into account the effects of cyclic post-buckling behaviour.

(2) Rigid connections should be preferred to nominally pinned ones (see EN 1998-1-8: 2004, **5.2.2**).

(3) To improve out-of-plane stability of the bracing connection, the continuity of beams and columns should not be interrupted.

(4) The brace and the beam centrelines should not intersect outside the seismic link.

(5) In connections of diagonal braces and beams, the centrelines of these members should intersect either within the length of the link or at its end.

(6) For connection of a seismic link to a column at column flange face, bearing end plates should be used between the beam flange plates.

(7) Retrofitting of beam-to-column connections may change the length of the seismic link. Therefore, the link should be checked after the repair strategy is adopted.

(8) Seismic links connected to the column should be short.

(9) Welded connections of a seismic link to the column weak-axis should be avoided.

ANNEX C (Informative)

MASONRY BUILDINGS

C.1 Scope

(1) This annex contains recommendations for the assessment and the design of the retrofitting of masonry buildings in seismic regions.

(2) The recommendations of this section are applicable to concrete or brick masonry lateral force resisting elements, within a building system in un-reinforced, confined or reinforced masonry.

C.2 Identification of geometry, details and materials

C.2.1 General

(1) The following aspects should be carefully examined:

i. Type of masonry unit (e.g., clay, concrete, hollow, solid, etc.);

ii. Physical condition of masonry elements and presence of any degradation;

iii. Configuration of masonry elements and their connections, as well as the continuity of load paths between lateral resisting elements;

iv. Properties of constituent materials of masonry elements and quality of connections;

v. The presence and attachment of veneers, the presence of nonstructural components, the distance between partition walls;

vi. Information on adjacent buildings potentially interacting with the building under consideration.

C.2.2 Geometry

(1) The collected data should include the following items:

i. Size and location of all shear walls, including height, length and thickness;

ii. Dimensions of masonry units;

iii. Location and size of wall openings (doors, windows);

iv. Distribution of gravity loads on bearing walls.

C.2.3 Details

(1) The collected data should include the following items:

i. Classification of the walls as un-reinforced, confined, or reinforced;

ii. Presence and quality of mortar;

iii. For reinforced masonry walls, amount of horizontal and vertical reinforcement;

iv. For multi-leaf masonry (rubble core masonry walls), identification of the number of leaves, respective distances, and location of ties, when existing;

v. For grouted masonry, evaluation of the type, quality and location of grout placements;

vi. Determination of the type and condition of the mortar and mortar joints; Examination of the resistance, erosion and hardness of the mortar; Identification of defects such as cracks, internal voids, weak components and deterioration of mortar;

vii. Identification of the type and condition of connections between orthogonal walls;

viii. Identification of the type and condition of connections between walls and floors or roofs.

ix. Identification and location of horizontal cracks in bed joints, vertical cracks in head joints and masonry units, and diagonal cracks near openings;

x. Examination of deviations in verticality of walls and separation of exterior leaves or other elements as parapets and chimneys;

xi. Identification of local condition of connections between walls and floors or roofs.

C.2.4 Materials

(1) Non-destructive testing may be used to quantify and confirm the uniformity of construction quality and the presence and degree of deterioration. The following types of tests may be used:

i. Ultrasonic or mechanical pulse velocity to detect variations in the density and modulus of masonry materials and to detect the presence of cracks and discontinuities.

ii. Impact echo test to confirm whether reinforced walls are grouted.

iii. Radiography and cover meters, where appropriate, to confirm location of reinforcing steel.

(2) Supplementary tests may be performed to enhance the level of confidence in masonry material properties, or to assess masonry condition. Possible tests are:

i. Schmidt rebound hammer test to evaluate surface hardness of exterior masonry walls.

ii. Hydraulic flat jack test to measure the in-situ shear strength of masonry. This test may be in conjunction with flat jacks applying a measured vertical load to the masonry units under test

iii. Hydraulic flat jack test to measure the *in-situ* vertical compressive stress resisted by masonry. This test provides information such as the gravity load distribution, flexural stresses in out-of-plane walls, and stresses in masonry veneer walls compressed by surrounding concrete frame.

iv. Diagonal compression test to estimate shear strength and shear modulus of masonry.

v. Large-scale destructive tests on particular regions or elements, to increase the confidence level on overall structural properties or to provide particular information such as out-of-plane strength, behaviour of connections and openings, in-plane strength and deformation capacity.

C.3 Methods of analysis

(1) In setting up the model for the analysis, the stiffness of the walls should be evaluated taking into account both flexural and shear flexibility, using cracked stiffness. In the absence of more accurate evaluations, both contributions to stiffness may be taken as one-half of their respective uncracked values.

(2) Masonry spandrels may be introduced in the model as coupling beams between two wall elements.

C.3.1 Linear methods: Static and Multi-modal

(1) These methods are applicable under the following conditions, which are additional to the general conditions of **4.4.2(1)**P:

i. The lateral load resisting walls are regularly arranged in both horizontal directions;

ii. Walls are continuous along their height;

iii. The floors possess enough in-plane stiffness and are sufficiently connected to the perimeter walls to assume that they can distribute the inertia forces among the vertical elements as a rigid diaphragm;

iv. Floors on opposite sides of a common wall are at the same height;

v. At each floor, the ratio between the lateral in-plane stiffnesses of the stiffest wall and the weakest primary seismic wall, evaluated accounting for the presence of openings, does not exceed 2,5;

vi. Spandrel elements included in the model are either made of blocks adequately interlocked to those of the adjacent walls, or have connecting ties.

C.3.2 Nonlinear methods: Static and dynamic

(1) These methods should be applied when the conditions in **C.3.1** are not met.

(2) Capacity is defined in terms of roof displacement. The ultimate displacement capacity is taken as the roof displacement at which total lateral resistance (base shear) has dropped below 80% of the peak resistance of the structure, due to progressive damage and failure of lateral load resisting elements.

(3) The demand, to be compared to the capacity, is the roof displacement corresponding to the target displacement of **4.4.4.3** and EN 1998-1:2004, **4.3.3.4.2.6(1)** for the seismic action considered.

NOTE Informative Annex B of EN 1998: 2004 gives a procedure for the determination of the target displacement from the elastic response spectrum.

C.4 Capacity models for assessment

C.4.1 Models for global assessment

C.4.1.1 LS of Near Collapse (NC)

(1) Criteria given for assessment in terms of global response measures can be applied only with results of nonlinear analysis.

(2) Global capacity at the LS of Near Collapse (NC) may be taken equal to the ultimate displacement capacity defined in **C.3.2(2)**.

C.4.1.2 LS of Significant Damage (SD)

(1) **C.4.1.2(1)** applies.

(2) Global capacity at the LS of Significant Damage (SD) may be taken equal to 3/4 of the ultimate displacement capacity defined in **C3.2(2)**.

C.4.1.3 LS of Damage Limitation (DL)

(1) If a linear analysis is performed, the criterion for global assessment is defined in terms of the base shear in the horizontal direction of the seismic action. The capacity may be taken equal to the sum of shear force capacities of the individual walls, as this is controlled by flexure (see **C.4.2.1(1)**) or by shear (see **C.4.3.1(1)**) in the horizontal direction of the seismic action. The demand is the estimate of the maximum base shear in that direction from the linear analysis.

(2) If nonlinear analysis is performed, the capacity for global assessment is defined as the yield point (yield force and yield displacement) of the idealized elasto-perfectly plastic force – displacement relationship of the equivalent Single-Degree-of-Freedom system.

NOTE Informative Annex B of EN 1998: 2004 gives a procedure for the determination of the yield force and the yield displacement of the idealized elasto-perfectly plastic force – displacement relationship of the equivalent Single-Degree-of-Freedom system.

C.4.2 Elements under normal force and bending

C.4.2.1 LS of Significant Damage (SD)

(1) The capacity of an unreinforced masonry wall is controlled by flexure, if the value of its shear force capacity given in **C.4.2.1(3)** is less than the value given in **C.4.3.1(3).**

(2) The capacity of an unreinforced masonry wall controlled by flexure may be expressed in terms of drift and taken equal to 0,008·H_0/D for primary seismic walls and to 0,012·H_0/D for secondary ones, where:

D is the in-plane horizontal dimension of the wall (depth);

H_0 is the distance between the section where the flexural capacity is attained and the contraflexure point..

(3) The shear force capacity of an unreinforced masonry wall as controlled by flexure under an axial load N, may be taken equal to:

$$V_{\mathrm{f}} = \frac{D\,N}{2\,H_0}\left(1 - 1{,}15\ \nu_{\mathrm{d}}\right) \tag{C.1}$$

where

D and H_0 are as defined in **(2)**;

ν_{d} = $N/(Dtf_{\mathrm{d}})$ is the normalized axial load (with $f_{\mathrm{d}} = f_{\mathrm{m}}/CF_{\mathrm{m}}$, where f_{m} is the mean compressive strength as obtained from in-situ tests and from the additional sources of information, and CF_{m} is the confidence factor for masonry given in Table 3.1 for the appropriate knowledge level), t is the wall thickness.

C.4.2.2 LS of Near Collapse (NC)

(1) **C.4.2.1(1)** and **C.4.2.1(3)** apply.

(2) The capacity of a masonry wall controlled by flexure may be expressed in terms of drift and taken equal to 4/3 of the values in **C.4.2.1(2)**.

C.4.2.3 LS of Damage Limitation (DL)

(1) **C.4.2.1(1)** applies.

(2) The capacity of an unreinforced masonry wall controlled by flexure may be taken as the shear force capacity given in **C.4.2.1(2)**.

C.4.3 Elements under shear force

C.4.3.1 LS of Significant Damage (SD)

(1) The capacity of an unreinforced masonry wall is controlled by shear, if the value of its shear force capacity given in **C.4.3.1(3)** is less than or equal to the value given in **C.4.2.1(3).**

(2) The capacity of an unreinforced masonry wall controlled by shear may be expressed in terms of drift and taken equal to 0,004 for primary seismic walls and to 0,006 for secondary ones.

(3) The shear force capacity of an unreinforced masonry wall controlled by shear under an axial load N, may be taken equal to:

$$V_{\mathrm{f}} = f_{\mathrm{vd}}\, D' t \tag{C.2}$$

where:

D' is the depth of the compressed area of the wall;

t is the wall thickness; and

f_{vd} is the masonry shear strength accounting for the presence of vertical load: $= f_{\mathrm{vm0}} + 0{,}4\ N/D't \leq 0{,}065 f_{\mathrm{m}}$, where f_{vm0} is the mean shear strength in the absence of vertical load and f_{m} the mean compressive strength, both as obtained from in-situ tests and from the additional sources of information, and divided by the confidence factors, as defined in the **3.5(1)**P and Table 3.1, accounting for the level of knowledge attained. In primary seismic walls, both these material strengths are futher divided by the partial factor for masonry in accordance with EN1998-1:2004, **9.6.**

C.4.3.2 LS of Near Collapse (NC)

(1) **C.4.3.1(1)** and **C.4.3.1(3)** apply.

(2) The capacity of an unreinforced masonry wall controlled by shear may be expressed in terms of drift and taken as 4/3 of the values in **C.4.2.1(2)**.

C.4.3.3 LS of Damage Limitation (DL)

(1) **C.4.3.1(1)** applies.

(2) The capacity of an unreinforced masonry wall controlled by shear may be taken as the shear force capacity given in **C.4.3.1(2)**.

C.5 Structural interventions

C.5.1 Repair and strengthening techniques

C.5.1.1 Repair of cracks

(1) If the crack width is relatively small (e.g., less than 10 mm) and the thickness of the wall is relatively small, cracks may be sealed with mortar.

(2) If the width of cracks is small but the thickness of the masonry is not, cement grout injections should be used. Where possible, no-shrinkage grout should be used. Epoxy grouting may be used instead, for fine cracks.

(3) If the crack are relatively wide (e.g., more than 10 mm), the damaged area should be reconstructed using elongated (stitching) bricks or stones. Otherwise, dove-tailed clamps, metal plates or polymer grids should be used to tie together the two faces of the crack. Voids should be filled with cement mortar of appropriate fluidity.

(4) Where bed-joints are reasonably level, the resistance of walls against vertical cracking can be considerably improved by embedding in bed-joints either small diameter stranded wire ropes or polymeric grid strips.

(5) For repair of large diagonal cracks, vertical concrete ribs may be cast into irregular chases made in the masonry wall, normally on both sides. Such ribs should be reinforced with closed stirrups and longitudinal bars. Stranded wire rope as in **(4)** should run across the concrete ribs. Alternatively, polymeric grids may be used to envelop one or both faces of the masonry walls, combined with appropriate mortar and plaster.

C.5.1.2 Repair and strengthening of wall intersections

(1) To improve connection between intersecting walls, use should be made of cross-bonded bricks or stones. The connection may be made more effective in different ways:

i. Through construction of a reinforced concrete belt,

ii. By addition of steel plates or meshes in the bed-joints,

iii. Through insertion of inclined steel bars in holes drilled in the masonry and grouting thereafter.

C.5.1.3 Strengthening and stiffening of horizontal diaphragms

(1) Timber floors may be strengthened and stiffened against in-plane distortion by:

i. nailing an additional (orthogonal or oblique) layer of timber boards onto the existing ones,

ii. casting an overlay of concrete reinforced with welded wire mesh. The concrete overlay should have shear connection with the timber floor and should be anchored to the walls,

iii. placing a doubly-diagonal mesh of flat steel ties anchored to the beams and to the perimeter walls.

(2) Roof trusses should be braced and anchored to the supporting walls. A horizontal diaphragm should be created (e.g. by adding bracing) at the level of the bottom chords of the trusses.

C.5.1.4 Tie beams

(1) If existing tie-beams between walls and floors are damaged, they should be repaired or rebuilt. If there are no tie-beams in the original building structure, such beams should be added.

C.5.1.5 Strengthening of buildings by means of steel ties

(1) The addition of steel ties, along or transversely to the walls, external or within holes drilled in the walls, is an efficient means of connecting walls and improving the overall behaviour of masonry buildings.

(2) Posttensioned ties may be used to improve the resistance of the walls against tensile stresses.

C.5.1.6 Strengthening of rubble core masonry walls (multi-leaf walls)

(1) The rubble core may be strengthened by cement grouting, if the penetration of the grout is satisfactory. If adhesion of the grout to the rubble is likely to be poor, grouting should be supplemented by steel bars inserted across the core and anchored to the outer leafs of the wall.

C.5.1.7 Strengthening of walls by means of reinforced concrete jackets or steel profiles

(1) The concrete should be applied by the shotcrete method and the jackets should be reinforced by welded wire mesh or steel bars.

(2) The jackets may be applied on only one face of the wall, or preferably on both. The two layers of the jacket applied to opposite faces of the wall, should be connected by means of transverse ties through the masonry. Jackets applied on only one face, should be connected to the masonry by chases.

(3) Steel profiles may be used in a similar way, provided they are appropriately connected to both faces of the wall or on one face only.

C.5.1.8 Strengthening of walls by means of polymer grids jackets

(1) Polymer grids may be used to strengthen existing and new masonry elements. In case of existing elements, the grids should be connected to masonry walls from one

sides or both sides and should be anchored to the perpendicular walls. In case of new elements, the intervention may involve the additional insertion of grids in the horizontal layers of mortar between bricks. Plaster covering polymeric grids should be ductile, preferably lime-cement with fibre reinforcement.

EUROPEAN STANDARD
NORME EUROPÉENNE
EUROPÄISCHE NORM

prEN 1998-4 : 2003

December 2003

UDC
Descriptors:

Doc CEN/TC250/SC8/N387

English version

Eurocode 8 : Design of structures for earthquake resistance

Part 4: **Silos, tanks and pipelines**

Calcul des structures pour leur résistance aux séismes

Partie 4 : Silos, réservoirs et réseaux de tuyauteries

Auslegung von Bauwerken gegen Erdbeben

Teil 4 : Silos, Tankbauwerke und Rohrleitungen

Draft No 2

CEN

European Committee for Standardisation
Comité Européen de Normalisation
Europäisches Komitee für Normung

Central Secretariat: rue de Stassart 36, B-1050 Brussels

~~PRÉNORME EUROPÉENNE~~
~~EUROPÄISCHE VORNORM~~

~~Doc CEN/TC250/SC8/N322~~

~~English version~~

~~Eurocode 8: Design of structures for earthquake resistance~~

~~Part 4: Silos, tanks and pipelines~~

~~DRAFT No 1~~

~~(Stage 32)~~

~~June 2002~~

~~CEN~~

~~European Committee for Standardization~~
~~Comité Européen de Normalisation~~
~~Europäisches Komitee für Normung~~

~~Central Secretariat: rue de Stassart 36, B1050 Brussels~~

~~Ref.No ENV 1998-4~~

Contents

Foreword

This document (EN 1998-4:200X) 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 1998-4:1997.

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

[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 1993 Eurocode 3: Design of steel structures

EN 1994 Eurocode 4: Design of composite steel and concrete structures

EN 1995 Eurocode 5: Design of timber structures

EN 1996 Eurocode 6: Design of masonry structures

EN 1997 Eurocode 7: Geotechnical design

EN 1998 Eurocode 8: Design of structures for earthquake resistance

EN 1999 Eurocode 9: Design of aluminium structures

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

Status and field of application of Eurocodes

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

- as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement N°1 - Mechanical resistance and stability - and Essential Requirement N°2 - Safety in case of fire;
- as a basis for specifying contracts for construction works and related engineering services;
- as a framework for drawing up harmonised technical specifications for construction products (ENs and ETAs)

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

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

standards with a view to achieving a 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.

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 (informative).

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 1998-4

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

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

EN 1998-4:200X 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.

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 1991-4, EN 1992-3, EN 1993-4-1, EN 1993-4-2 and EN 1993-4-3, which do not cover the special requirements of seismic design.

National annex for EN 1998-4

This standard gives alternative procedures, values and recommendations for classes with notes indicating where national choices may be made. Therefore the National Standard implementing EN 1998-4 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 1998-4:200X through clauses:

Reference	**Item**

1 GENERAL

1.1 Scope

(1)P This standard aims at providing principles and application rules for the seismic design of the structural aspects of facilities composed of above-ground and buried pipeline systems and of storage tanks of different types and uses, as well as for independent items, such as for example single water towers serving a specific purpose or groups of silos enclosing granular materials, etc. This standard may also be used as a basis for evaluating the resistance of existing facilities and to assess any required strengthening.

(2) P This standard includes the additional criteria and rules required for the seismic design of these structures without restrictions on their size, structural types and other functional characteristics. For some types of tanks and silos, ~~however,~~ it also provides detailed methods of assessment and verification rules.

~~-~~(3) P This standard may not be complete for those facilities associated with large risks to the population or the environment, for which additional requirements shall be established by the competent authorities. This standard is also not complete for those construction works which have uncommon structural elements and which require special measures to be taken and special studies to be performed to ensure earthquake protection. In those two cases the present standard gives general principles but not detailed application rules.

(4) The nature of lifeline systems which often characterises the facilities covered by this standard requires concepts, models and methods that may differ substantially from those in current use for more common structural types. Furthermore, the response and the stability of silos and tanks subjected to strong seismic actions may involve rather complex interaction phenomena between ~~of~~ soil-structure and stored material (either ~~-~~fluid or granular)~~interaction~~, not easily amenable to simplified design procedures. Equally challenging may prove to be the design of a pipeline system through areas with poor and possibly unstable soils. For the reasons given above, the organisation of this standard is to some extent different from that of companion Parts of EN 1998. This standard is, in general, restricted to basic principles and methodological approaches.

NOTE Detailed analysis procedures going beyond basic principles and methodological approaches are given in Annexes A, B and C for a number of typical situations.

(5) P For the formulation of the general requirements as well as for ~~their~~ its implementation, a distinction ~~can~~ shall be made between independent structures and redundant systems, via the choice of importance factors and/or through the definition of ~~adapted~~ specific verification criteria.

(6) ~~P~~ A structure may~~can~~ be considered as independent when its structural and functional behaviour during and after a seismic event is not influenced by that of other structures, and if the consequences of its failure relate only to the functions demanded from it.

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 : 2002 Eurocode - Basis of structural design

EN 1998-1 : 2004 Eurocode 8 - Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings

EN 1998-5 : 2004 Eurocode 8 - Design of structures for earthquake resistance – Part 5: Foundations, retaining structures and geotechnical aspects

EN 1998-6 : 200X 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 ~~d~~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: ~~200X~~2004, **1.5.1** applies for terms common to all Eurocodes.

1.5.2 Additional terms used in the present standard

(1) 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 4 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:

NOTE: The list of symbols shall be added later on

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.

~~1.2~~2 GENERAL RULES~~SAFETY REQUIREMENTS~~

2.1 Safety requirements

~~1.2.1~~2.1.1 General

(1) P This standard deals with structures which may differ widely in such basic features as:

– the nature and amount of stored product and associated potential danger

– the functional requirements during and after the seismic event

– the environmental conditions.

(2) Depending on the specific combination of the indicated features, different formulations of the general requirements are appropriate. For the sake of consistency with the general framework of the Eurocodes, the two-limit-states format is retained, with a suitably adjusted definition.

~~1.2.2~~2.1.2 Damage limitation ~~limit~~ state

(1) P Depending on the characteristics and the purposes of the structures considered one or both of the two following damage limitation states may need to be satisfied:

– full integrity;

– minimum operating level.

(2) P In order to satisfy t~~T~~he "full integrity" requirement, ~~implies that~~ the considered system, including a specified set of accessory elements integrated with it, shall remain~~s~~ fully serviceable and leak proof under a seismic event having an annual probability of exceedance whose value is to be established based on the consequences of its loss of function and/or of the leakage of the content.

(3) P Satisfaction of the~~The~~ "minimum operating level" requirement, means that ~~implies that~~ the considered system may suffer a certain amount of damage to some of its components, to an extent, however, that after the damage control operations have been carried out, the capacity of the system can be restored up to a predefined level of operation. The seismic event for which this limit state may not be exceeded shall have an annual probability of exceedance whose value is to be established based on the losses related to the reduced capacity of the system and to the necessary repairs.

PT NOTE: A more clear definition of the seismic events for the verification of these two damage limitation states has to be provided. It may become a NDP

~~1.2.3~~2.1.3 Ultimate limit state

(1)P The ultimate limit state of a system which shall be checked is defined as that corresponding to the loss of operational capacity of the system, with the possibility of partial recovery (in the measure defined by the responsible authority) conditional to an acceptable amount of repairs. ~~the limit state that guarantees the non collapse of the facility and the avoidance of uncontrolled loss of stored products.~~

(2)P For particular elements of the network, as well as for independent structures whose complete collapse would entail high risks, the ultimate limit state is defined as that of a state of damage that, although possibly severe, would exclude brittle failures and would allow for a controlled release of the contents. When the failure of the aforementioned elements does not involve appreciable risks to life and property, the ultimate limit state can be defined as corresponding to total collapse.

(3)P The design seismic action for which the ultimate limit state must not be exceeded shall be established based on the direct and indirect costs caused by the collapse of the system

~~1.2.4~~2.1.4 Reliability differentiation

(1) P Pipeline networks and independent structures, either tanks or silos, shall be provided with a level of protection proportioned to the number of people at risk and to the economic and environmental losses associated with their performance level being not achieved.

(2) P Reliability differentiation shall be achieved by appropriately adjusting the value of the annual probability of exceedance of the design seismic action.

(3) This adjustment should be implemented by classifying structures into different importance classes and applying to the reference seismic action an importance factor γ_I, as defined in EN 1998-1:2004~~X~~, **2.1(3)**P, the value of which depends on the importance class. Specific values of the factor γ_I, necessary to modify the action so as to correspond to a seismic event of selected return period, depend on the seismicity of each region. The value of the importance factor $\gamma_I = 1{,}0$ is associated with a seismic event having the reference return period indicated in EN 1998-1:200X, **3.2.1(3).**

NOTE For the dependence of the value of γ_I see Note to EN1998-1:2004~~X~~, **2.1(4)**

(4)P For the structures within the scope of this standard it is appropriate to consider three different Importance Classes, depending on the potential ~~exposure to~~ loss of life due to the failure of the particular structure and on the environmental, economic and social consequences of failure. Further classification may be made within each Importance Class, depending on the use and contents of the facility and the ~~ramifications~~ implications for public safety.

NOTE Importance classes I, II and III correspond roughly to consequences classes CC1~~3~~, CC2 and CC3~~1~~, respectively, defined in EN 1990:2002, Annex B.

(5)P Class III refers to situations with a high risk to life and large environmental, economic and social consequences.

(6)P Situations with medium risk to life and considerable environmental, economic or social consequences belong to Class II.

(7)P Class I~~II~~ refers to situations where the risk to life is low and the environmental, economic and social consequences of failure are small or negligible.

(8) A more detailed definition of the classes, specific for pipeline systems, is given in **4.2.1**

NOTE The values to be ascribed to γ_I for use in a country may be found in its National Annex. The values of γ_I may be different for the various seismic zones of the country, depending on the seismic hazard conditions (see Note to EN 1998-1: 2004~~X~~, **2.1(4)**) and on the public safety considerations detailed in **~~1.2.~~2.1.4**. The recommended values of γ_I are given in Table 1.1N. In the column at left there is a classification of the more common uses of these structures, while the three columns at right contain the recommended levels of protection in terms of the values of the importance factor γ_I –for three Importance Classes.

Table 2~~1~~.1N Importance factors

Use of the structure/facility	Importance Class		
	I	II	III~~3~~
Potable water supply Non-toxic, non inflammable material	0,8~~1,2~~	1,0	~~0,8~~1,2
Fire fighting water Non-volatile toxic material Low flammability petrochemicals	1,0 ~~1,4~~	1,2	~~1,0~~1,4
Volatile toxic chemicals Explosive and other high flammability liquids	1,2~~1,6~~	1,4	~~1,2~~1,6

~~1.2.5~~2.1.5 System versus element reliability

(1) P The reliability requirements set forth in **1.2.2** and **1.2.3** refer to the whole system under consideration, be it constituted by a single component or by a set of components variously connected to perform the functions required from it.

(2) Although a formal approach to system reliability analysis is outside the scope of this standard, the designer shall give explicit consideration to the role played by the various elements in ensuring the continued operation of the system, especially when it is not redundant. In the case of very complex systems the design ~~shall~~ should be based on sensitivity analyses.

(3)P Elements of the network, or of a structure in the network, which are shown to be critical, with respect to the failure of the system, shall be provided with an additional margin of protection, commensurate with the consequences of the failure. When there is no previous experience, those critical elements should be experimentally investigated to verify the acceptability of the design assumptions.

(4) If more rigorous analyses are not undertaken, the additional margin of protection for critical elements can be achieved by assigning these elements to a class of reliability (expressed in terms of Importance Class) one level higher than that proper to the system as a whole.

~~1.2.6~~2.1.6 Conceptual design

(1) P Even when the overall seismic response is specified to be elastic ~~(corresponding to a value q = 1,5 for the behaviour factor)~~, structural elements shall be designed and detailed for local ductility and constructed from ductile materials.

(2) P The design of a network or of an independent structure shall take into consideration the following general aspects for mitigation of earthquake effects:

– Redundancy of the systems

– Absence of interaction of the mechanical and electrical components with the structural elements.

– Easy access for inspection, maintenance and repair of damages;

– Quality control of the components;

(3) In order to avoid spreading of damage in redundant systems due to structural interconnection of components, the ~~necessary~~ appropriate parts should be isolated.

(4) In case of important facilities vulnerable to earthquakes, of which damage recovery is difficult or time consuming, replacement parts or subassemblies should be provided.

~~1.3~~2.2 Seismic action

(1) P The seismic action to be used in the determination of the seismic action effects for the design of silos, tanks and pipelines shall be that defined in EN 1998-1:~~-~~2004~~X~~, **3.2** in the various equivalent forms of elastic, site-dependent response spectra (EN 1998-1:~~-~~2004~~X~~, **3.2.2**), and time-history representation (EN 1998-1: 200X, **3.2.3.1**). In those cases where a behaviour factor q larger than ~~the value of~~ 1,5 (considered as resulting~~derived~~ from overstrength alone) is acceptable (see **~~1.10~~2.34.2**), the design spectrum for elastic analysis shall be used (EN 1998-1:~~200X~~2004, **3.2.2.5**). Additional provisions for the spatial variation of ground motion for buried pipelines are given in Section **5**.

(2) P The two seismic actions to be used for checking the damage limitation state and the ultimate limit state, respectively, shall be established by the competent National Authority on the basis of the seismicity of the different seismic zones and of ~~the level of~~ the ~~importance~~ Importance ~~category~~ Class of the specific facility.

(3) A reduction factor ν applied to the design seismic action, to take into account the lower return period of the seismic event associated with the damage limitation state may be considered as mentioned in EN 1998-1:~~-~~2004~~X~~, **2.1(1)**P. The value of the reduction factor ν may also depend on the Importance Class of the structure. Implicit in its use is the assumption that the elastic response spectrum of the seismic action under which the "damage limitation requirement" should be met has the same shape as the elastic response spectrum of the design seismic action corresponding to the " ultimate limit state requirement" according to EN 1998-1:~~-~~2004,~~X~~ ~~(~~**2.1(1)**P and **3.2.1(3)**~~)~~ (See EN 1998-1:~~-~~2004,~~X~~ ~~(~~**3.2.2.1(2)**). In the absence of more precise information, the reduction factor ν ~~applied on the design seismic action~~with the value according to EN 1998-1:~~-~~2004,~~X~~ ~~(~~**4.4.3.2(2)**~~)~~ may be used to obtain the seismic action for the verification of the damage limitation requirement.

NOTE The values to be ascribed to ν for use in a country may be found in its National Annex. Different values of ν may be defined for the various seismic zones of a country, depending on the seismic hazard conditions and on the protection of property objective. The recommended values of ν are 0,~~5~~4 for importance class~~es~~ I and ~~II and~~ ν = 0,4~~5~~ for importance classes II and III.

~~1.4~~2.3 Analysis

~~1.4.1~~2.3.1 ~~Methods of Analysis~~Methods of analysis

(1) P For the structures within the scope of this standard the seismic actions effects shall in general be determined on the basis of linear behaviour of the structures and of the soil in their vicinity.

(2) P Nonlinear methods of ~~analyses~~ analysis may be used to obtain the seismic action effects for those special cases where consideration of nonlinear behaviour of the structure or of the surrounding soil is dictated by the nature of the problem, or where the elastic solution would be economically unfeasible. In those cases it shall be proved that the design obtained possesses at least the same amount of reliability as the structures explicitly covered by this standard.

(3)P Analysis for the evaluation of the effects of the seismic action relevant to the damage limitation state shall be linear elastic, using the elastic spectra defined in EN 1998-1:~~-~~2004~~0X~~, **3.2.2.2** and ~~EN 1998-1: 20040X,~~ **3.2.2.3**, multiplied by the reduction factor ν ~~of~~ referred to in **~~1.9~~2.2~~3~~(3)** and entered with a weighted average value of the viscous damping that takes into account the different damping values of the different materials/elements according to **~~1.10~~2.34.5** and to EN 1998-1:~~-~~2004~~0X~~, **3.2.2.2(3).**

(4)P Analysis for the evaluation of the effects of the seismic action relevant to the ultimate limit state may be elastic, using the design spectra which are specified in EN 1998-1: 2004~~0X~~, **3.2.2.5** for a damping ratio of 5% and make use of the behaviour factor q to account for the capacity of the structure to dissipate energy, through mainly ductile behaviour of its elements and/or other mechanisms, as well as the influence of viscous damping different from 5%.

(5)P Unless otherwise specified for particular types of structures in the relevant parts of this standard, the types of analysis that may be applied are those indicated in EN 1998-1:~~-~~200~~0X~~4, **4.3.3**, namely:

a) the "lateral force method" of (linear-elastic) analysis (see EN 1998-1:~~-~~2004~~0X~~ **4.3.3.2)**;

b) the "modal response spectrum" (linear-elastic) analysis (see EN 1998-1:~~-~~2004~~0X~~, **4.3.3.3);**

c) the non-linear static (pushover) analysis (see EN 1998-1:~~-~~2004~~0X~~ **4.3.3.4.2)**;

d) the non-linear time history (dynamic) analysis (see EN 1998-1:~~-~~2004~~0X~~ **4.3.3.4.3).**

~~-~~(6) Clauses **4.3.1(1)**P, **4.3.1(2)**, **4.3.1(6)**, **4.3.1(7)**~~-~~, **4.3.1(9)**P, **4.3.3.1(5)** and **4.3.3.1(6)** of EN 1998-1: 2004~~0X~~ apply for the modelling and analysis of the types of structures covered by the present standard.

PT NOTE: The conditions for use of each type of analysis (regularity criteria, etc.), the possible use of two planar models instead of a spatial model and the consideration of accidental eccentricity, etc., will be addressed in the 3rd Draft.

(7) The "lateral force method" of linear-elastic analysis should be performed according to ~~Clauses~~ clauses **4.3.3.2.1(1)**P, **4.3.3.2.2(1)** (with λ=1,0), **4.3.3.2.2(2)** and **4.3.3.2.3(2)**P of EN

1998-1:-2004~~0X~~. It is appropriate for structures that respond to each component of the seismic action approximately as a Single-Degree-of-Freedom system: rigid (i.e. concrete) elevated tanks or silos on relatively flexible and almost massless supports.

(8) The "m~~M~~odal response spectrum" linear-elastic analysis should be performed according to Clauses **4.3.3.3.1(2)**P, **4.3.3.3.1(3)**, **4.3.3.3.1(4)** and **4.3.3.3.2** of EN 1998-1: 2004~~0X~~. It is appropriate for structures whose response is significantly affected by contributions from modes other than that of a Single-Degree-of-Freedom system in each principal direction. ~~This includes tanks, silos or pipelines which are not sufficiently stiff to be considered to respond to the seismic action as a rigid body.~~

(9) Non-linear analysis, static (pushover) or dynamic (time history), should satisfy EN 1998-1:-2004~~0X~~, **4.3.3.4.1.**

(10) Non-linear static (pushover) analysis should be performed according to ~~Clauses~~ clauses **4.3.3.4.2.2(1)**, **4.3.3.4.2.3**, **4.3.3.4.2.6** of EN 1998-1: 2004~~0X~~.

(11) Non-linear dynamic (time history) analysis should satisfy EN 1998-1: 2004~~0X~~, **4.3.3.4.3.**

~~1.4.2~~2.3.2 Behaviour factors

(1)P For structures covered by this standard~~, except welded steel above groung piping systems, and for the damage limitation state,~~ significant energy dissipation is not expected for the damage limitation state~~,~~. Hence, for the damage limitation state, the behaviour ~~coefficient~~ factor q shall be taken as equal to 1.

(2) -Use of q factors greater than 1,5 ~~is only allowed~~ in ultimate limit state verifications is only allowed, provided that the sources of energy dissipation are explicitly identified and quantified and the capability of the structure to exploit them through appropriate detailing is demonstrated.

PT NOTE: The value of q was modified to align with the general rule in EC8 in which q =1,5 may always be used in ULS verifications due to the effect of overstrength. However this has to be checked by the PT.

~~1.4.3~~2.3.3 Damping

~~1.4.3.1~~2.3.3.1 Structural damping

(1) If the damping values are not obtained from specific information~~ or by direct means~~, the following values of the damping ratio should be used in linear analysis:

a) Damage limitation state: ξ = 2%

b) Ultimate limit state: ξ = 5%

~~1.4.3.2~~2.3.3.2 Contents damping

(1) The value ξ = 0,5 % may be adopted for the damping ratio of water and other liquids, unless otherwise determined.

(2) For granular materials an appropriate value for the damping ratio should be used. In the absence of more specific information a value of ξ = 10% may be used.

~~1.4.3.3~~2.3.3.3 Foundation damping

(1) Material damping varies with the nature of the soil and the intensity of shaking. When more accurate determinations are not available, the values given in Table 4.1 of EN 1998-5: 2004~~X~~ should be used.

(2) P Radiation damping depends on the direction of motion (horizontal translation, vertical translation, rocking, etc.), on the geometry of the foundation, on soil layering and soil morphology. The values adopted in the analysis shall be compatible with actual site conditions and shall be justified with reference to acknowledged theoretical and/or experimental results. The values of the radiation damping used in the analysis shall not exceed the value~~:~~ ξ = 20 %.

NOTE Guidance for the selection and use of damping values associated with different foundation motions is given in Informative Annex B~~ of EN 1998-6: 200X,~~ and in Informative Annex B~~A~~ of EN 1998-6~~4~~: 200X .

~~1.4.4~~2.3.4 Interaction with the soil

(1) P Soil-structure interaction effects shall be addressed in accordance with ~~6 of~~ EN 1998-5:2004, Section **6**.

NOTE Additional information on procedures for accounting for soil-structure interaction is given in Informative Annex B and in Informative Annex C of EN 1998-6: 200X~~, and Informative Annex A of EN 1998-4: 200X~~.

~~1.4.5~~2.3.5 Weighted damping

(1) The global average damping of the whole system should account for the contributions of the different materials/elements to damping.

NOTE A procedure for accounting for the contributions of the different materials/elements to the global average damping of the ~~whole~~ system is given in Informative Annex B of EN 1998-6.

~~1.5~~2.4 Safety verifications

~~1.5.1~~2.4.1 General

(1) P Safety verifications shall be carried out for the limit states defined in ~~**1.2**~~**2.1**, following the specific provisions in ~~**2.4**~~**3.5**, ~~**3.5**~~**4.4, 5.5** and ~~**4.5**~~**6.4**.

(2) If plate thickness is increased to account for future corrosion effects, the verifications shall be made for both the non-increased and the increased thickness.

~~1.5.2~~2.4.2 Combinations of seismic action with other actions

(1) P The design value E_d of the effects of actions in the seismic design situation shall be determined according to EN 1990:2002, **6.4.3.4**, and the inertial effects of the design seismic action shall be evaluated according to EN 1998-1: 2004~~X~~, **3.2.4(2)P.**

(2) In partially backfilled or buried tanks, permanent loads include, in addition to the weight of the structure, the weight of earth cover and any permanent external pressures due to groundwater.

(3) The combination coefficients ψ_{2i} (for the quasi-permanent value of variable action q_i) shall be those given in EN 1990:2002, Annex A4. The combination coefficients ψ_{Ei} introduced in EN 1998-1:2004 **3.2.4(2)**P for the calculation of the effects of the seismic actions shall be taken as being equal to ψ_{2i}.

NOTE : Informative Annex A of EN1991-4 provides information for the combination coefficients ψ_{2i} (for the quasi-permanent value of variable action q_i) to be used for silos and tanks in the seismic design situation.

PT NOTE: The Note and the text may have to be adjusted at a later stage, in view of the final contents of the Annexes of EN1990 and EN1991-4.

(~~2~~4) P The effects of the contents shall be considered in the variable loads for various levels of filling. In groups of silos and tanks, different likely distributions of full and empty compartments shall be considered according to the operation rules of the facility. At least, the design situations where all compartments are either empty or full shall be considered.

~~2~~3 SPECIFIC RULES FOR SILOS

~~2.1~~3.1 Properties of stored solids and d~~D~~ynamic ~~over~~pressures

(1)P Annexes C, D and E of EN1991-4~~-~~:~~-~~200X apply for the determination of the properties of the particulate solid stored in the silo. The upper characteristic value of the solid unit weight presented in EN1991-4~~-~~:~~-~~200X, Table E1, shall be used in all calculations.

(2)P Under seismic conditions, the pressure exerted by the particulate material on the walls, the hopper and the bottom, may increase over the value relative to the condition at rest. For design purposes this increased pressure is deemed to be included in the effects of the inertia forces acting on the stored material due to the seismic excitation (see 3.3(5).~~This increased pressure is deemedassumed to be covered by the the effects of the inertia forces due to the seismic excitation.~~

~~2.2~~3.2 Combination of ground motion components

(1) P Silos shall be designed for simultaneous action of the two horizontal components and of the vertical component of the seismic action. If the structure is axisymmetric, it is allowed to consider only one horizontal component.

(2) When the structural response to each component of the seismic action is evaluated separately, EN1998-1:~~-~~2004~~X~~, **4.3.3.5.2(4)** may be applied for the determination of the most unfavourable effect of the application of the simultaneous components. If expressions (4.20), (4.21), (4.22) in EN1998-1:~~-~~2004~~X~~, **4.3.3.5.2(4)** are applied for the computation of the action effects of the simultaneous components, the sign of the action effect ~~of~~ due to each individual component shall be taken as ~~being~~ the most unfavourable for the particular action effect under consideration.

(3) P If the analysis is performed simultaneously for the three components of the seismic action using a spatial model of the structure, the peak values of the total response under the combined action of the horizontal and vertical components obtained from the analysis shall be used in the structural verifications.

~~2.3~~3.3 Analysis

~~NOTE Information on seismic analysis of vertical cylindrical silos are given in Informative Annex A.~~

(1) The –following subclauses provide rules additional to those of ~~1.10~~**2.34** which are specific to silos.

NOTE Additional information on seismic analysis of vertical cylindrical silos is given in Informative Annex A.

(2) P The model to be used for the determination of the seismic action effects shall reproduce accurately the stiffness, the mass and the geometrical properties of the containment structure, shall account for the response of the contained particulate material and for the effects of any interaction with the foundation soil. The provisions of EN 1993-4-1~~-~~:~~-~~200X,

Section 4, apply ~~rules~~ for the modelling and analysis of steel silos. ~~Numerical values for characteristics of infilled materials are given in EN1991-4: Annex E.~~

(3) P Silos shall be analysed considering elastic behaviour, unless proper justification is given for performing a nonlinear analysis.

(4) Unless more accurate evaluations are undertaken, the global seismic response and the seismic action effects in the supporting structure may be calculated assuming that the particulate contents of the silo move together with the silo shell and modelling them with their effective mass at their centre of gravity and its rotational inertia with respect to it. Unless a more accurate evaluation is made, the contents of the silo may be taken to have an effective mass equal to 80% of their total mass.

(5) Unless the mechanical properties and the dynamic response of the particulate solid are explicitly and accurately accounted for in the analysis (e.g. by using Finite Elements ~~through~~ to modelling the mechanical properties and the dynamic response of the particulate solid ~~with Finite Elements~~), the effect on the shell of the~~its~~ response of the particulate solid to the horizontal component of the seismic action may be represented through an additional normal pressure on the wall, $\Delta_{ph,s}$, (positive for compression) specified in the following paragraphs.~~:~~

(6) For circular silos (or silo compartments):

$$\Delta_{ph,s} = \Delta_{ph,so} \cos\theta$$

where

~~the reference pressure~~ $\Delta_{ph,so}$ is the reference pressure given in **(8)** of this subclause

~~and~~ θ ~~($0^o \leq \theta < 360^o$)~~ is the angle ($0^o \leq \theta < 360^o$) between the radial line to the point of interest on the wall and the direction of the horizontal component of the seismic action.

(7) For rectangular silos (or silo compartments) with walls parallel or normal to the horizontal component of the seismic action:

On the “leeward” wall which is normal to the horizontal component of the seismic action:

$$\Delta_{ph,s} = \Delta_{ph,so}$$

On the “windward” wall which is normal to the horizontal component of the seismic action:

$$\Delta_{ph,s} = -\Delta_{ph,so}$$

On the wall which is are parallel to the horizontal component of the seismic action:

$$\Delta_{ph,s} = 0$$

(8) At points on the wall with a vertical distance, z, from the hopper greater or equal to one-third of R_s^* defined as:

$$R_s^* = \min(H, B_s/2)$$

where:

H~~:~~ is the silo height;

B_s~~:~~ is the horizontal dimension of the silo parallel to the horizontal component of the seismic action (Diameter, D=2R, in circular silos or silo compartments, width b parallel to the horizontal component of the seismic action in rectangular ones),

the reference pressure $\Delta_{ph,so}$ may be taken as:

$$\Delta_{ph,so} = \alpha(z)\,\gamma\,R_s^*$$

where:

$\alpha(z)$~~:~~ is the ratio of the response acceleration ~~(in g's)~~ of the silo at the level of interest, z to the acceleration of gravity

γ~~:~~ is the bulk unit weight of the particulate material (upper characteristic value, see EN1991-4~~:~~:-200X Table E1).

(9) At the top of the silo, f~~Due to the transfer of inertia forces to the bottom of the silo, rather than to its walls, within the part of the height of the silo f~~rom z = 0 to z = R_s*/3, the value of $\Delta_{ph,so}$ increases linearly from $\Delta_{ph,so}$ =0 at z = 0 to the full value of expression (2.6) at z = R_s*/3.

(10) If only the value of the response acceleration at the centre of gravity of the particulate material is available (see, e.g., ~~1.10~~2.34.1(7) and paragraph **(4)** of the present subclause) the corresponding ratio ~~at value~~ to the acceleration of gravity may be used in expression (2~~.~~,6) for $\alpha(z)$.

(11) The value of $\Delta_{ph,s}$ at any ~~certain vertical distance z from the hopper and~~ location on the silo wall is limited by the condition that the sum of the static pressure of the particulate material on the wall and ~~of~~ the additional pressure~~one~~ given by expressions (2.1) to ~~-~~(2.4) may not be taken less than zero.

~~2.4~~3.4 Behaviour factors

(1)P The supporting structure of ~~earthquake-resistant~~ silos shall be designed according to one of the following concepts (see **5.2.1**, **6.1.2**, **7.1.2** in EN 1998-1:~~-~~2004~~X~~):

a) low-dissipative structural behaviour;

b) dissipative structural behaviour.

(2) In concept a) the seismic action effects may be calculated on the basis of an elastic global analysis without taking into account significant non-linear material behaviour. When using the design spectrum defined in EN 1998-1:~~-~~2004~~X~~, **3.2.2.5**, the value of the behaviour factor q may be taken up to 1,5. Design according to concept a) is termed design for ductility class Low (DCL~~Low~~) and is recommended only for low seismicity cases (see EN 1998-1: 2004~~X~~, **3.2.1(4)**). Selection of materials, evaluation of resistance and detailing of members

and connections should be as specified in EN 1998-1: 2004~~X~~, Section **5** to **7,** for ductility class Low (DCL).

(3) In concept b) the capability of parts of the supporting structure ~~(its dissipative zones)~~ to resist earthquake actions beyond their elastic range (its dissipative zones), ~~-~~is taken into account. Supporting structures designed according to this concept should belong to ductility class Medium (DCM) or High (DCH) defined and described in EN 1998-1:~~-~~2004~~X~~, Section **5** to **7**, depending on the structural material of the ~~the~~ supporting structure. They should meet the specific requirements specified therein regarding structural type, materials and dimensioning and detailing of members or connections for ductility. When using the design spectrum for elastic analysis defined in EN 1998-1:~~-~~2004~~X~~, **3.2.2.5**, the behaviour factor *q* may be taken as being greater than 1,5. The value of *q* depends on the selected ductility class (DCM or DCH).

(4) Due to limited redundancy and absence of non-structural elements contributing to earthquake resistance and energy dissipation, the energy dissipation capacity of the structural types commonly used to support silos is, in general, less than that of a similar structural type when used in buildings. Therefore, and due to the similarity of silos to inverted pendulum structures, in concept b) the upper limit value of the *q* factors for silos are defined in terms of the *q* factors specified in EN 1998-1:2004~~X~~, Sections **5** to **7**, for inverted pendulum structures of the selected ductility class (DCM or DCH), as follows :

- For silos supported on a single pedestal or skirt, or on irregular bracings, the upper limit of the *q* factors are those defined for inverted pendulum structures.

- For silos supported on moment resisting frames or on regular bracings, the upper limit of the *q* factors are 1,25 times the values defined ~~applying~~ for inverted pendulum structures.

- For cast-in-place concrete silos supported on concrete walls which are continuous to the foundation, the upper limit of the *q* factors are 1,5 times the values ~~applying~~ defined for inverted pendulum structures.

~~2.5~~3.5 Verifications

~~2.5.1~~3.5.1 Damage limitation state

(1) P In the seismic design situation relevant to the damage limitation state the silo structure shall be checked to satisfy the serviceability limit state verifications required by EN 1992-1-1, EN 1992-3 and EN 1993-4-1.

~~-~~(2) For steel silos, adequate reliability with respect to the occurrence of elastic or inelastic buckling phenomena is assured, if the verifications regarding these phenomena are satisfied under the seismic design situation for the ultimate limit state.

~~2.5.2~~3.5.2 Ultimate limit state

~~2.5.2.1~~3.5.2.1 Global stability

(1) P Overturning, sliding or bearing capacity failure of the soil shall not occur in the seismic design situation. The resisting shear force at the interface of the base of the structure

and the~~of its~~ foundation, shall be evaluated taking into account the effects of the vertical component of the seismic action. A limited sliding may be acceptable, if the structure is monolithic and is not connected to any piping (see also EN 1998-5:~~-~~2004~~X~~, **5.4.1.1(7)**).

(2) P Uplift is acceptable if it is adequately taken into account in the analysis and in the subsequent verifications of both the structure and of the foundation.

~~2.5.2.2~~3.5.2.2 Shell

(1) P The maximum action effects (~~axial and~~ membrane forces and bending moments) induced in the seismic design situation shall be less or equal to the resistance of the shell evaluated as ~~which applies~~ in the persistent or transient design situations. This includes all types of failure modes:

~~-:~~ F~~: f~~or steel shells:~~,~~ yielding (plastic collapse), buckling in shear or by vertical compression with simultaneous transverse tension (“elephant foot” mode of failure), etc. (see EN 1993-4-1 : 200X, Sections **5** to **9**).

- F~~; f~~or concrete shells:~~,~~ the ULS in bending with axial force, the ULS in shear for in-plane or radial shear, etc.

(2)P The calculation of resistances and the verifications shall be carried out in accordance with EN 1992-1-1, EN 1992-3, EN1993-1-1, EN1993-1-5, EN1993-1-6, EN1993-1-7 and EN 1993-4-1.

~~2.5.2.3~~3.5.2.3 Anchors

(1) Anchoring systems should generally be designed to remain elastic in the seismic design situation. However, they shall also be provided with sufficient ductility, so as to avoid brittle failures. The connection of anchoring elements to the structure and to its foundation should have an overstrength factor of not less than 1,25 with respect to the resistance of the anchoring elements.

(2) If the anchoring system is part of the dissipative mechanisms, then it should be verified that it possesses the necessary ductility capacity.

~~(1) P Anchoring systems shall be designed to remain elastic in the seismic design situation. They shall also be provided with sufficient ductility, so as to avoid brittle failures. If the anchorage system is part of the dissipating mechanisms, then it shall be appropriately verified. Their connection of anchoring elements to the structure and to its foundation shall have an overstrength factor of not less than 1,.25 with respect to the anchoring elements.~~

~~2.5.2.4~~3.5.2.4 Foundations

(1) P The foundation shall be verified according to EN 1998-5:~~-~~200X, **5.4** and to EN 1997-1.

(2) P The action effects for the verification of the foundation and of the foundation elements shall be derived according to EN 1998-5: 2004~~X~~, **5.3.1,** to EN 1998-1: 2004~~X~~, **4.4.2.6** and to EN 1998-1: 2004~~X~~, **5.8**.

~~3~~4 SPECIFIC RULES FOR TANKS

~~3.1~~4.1 Compliance criteria

~~3.1.1~~4.1.1 General

(1) P The general requirements set forth in ~~1.8~~2~~2~~.1 are deemed to be satisfied if, in addition to the verifications specified in ~~3~~4.4, the complementary measures indicated in ~~3~~4.5 are also satisfied.

~~3.1.2~~4.1.2 Damage limitation state

(1) P It shall be ensured that under the ~~relevant~~ seismic ~~design situation~~actions relevant ~~and in respect~~ to the "full integrity" limit state ~~or~~ and to the "minimum operating level" limit state:

a) Full integrity

– The tank system maintains its tightness against leakage of the contents. Adequate freeboard shall be provided, in order to prevent damage to the roof due to the pressures of the sloshing liquid or, if the tank has no rigid roof, to prevent the liquid from spilling over;

– The hydraulic systems which are part of, or are connected to the tank, are capable of accommodating stresses and distortions due to relative displacements between tanks or between tanks and soil, without their functions being impaired;

b) Minimum operating level

– Local buckling, if it occurs, does not trigger collapse and is reversible; for instance, local buckling of struts due to stress concentration is acceptable.

NOTE: The final wording of this clause may have to be adjusted in view of the Note presented in 2.1.2 and a NDP may be needed here.

~~3.1.3~~4.1.3 Ultimate limit state

(1) P It shall be ensured that under the ~~relevant~~ seismic design situation:

– The overall stability of the tank is ensured according to EN 1998-1:~~-~~2004~~X~~, **4.4.2.4**. The overall stability refers to rigid body behaviour and may be impaired by sliding or overturning. A limited amount of sliding may be accepted EN according to 1998-5: 2004~~X~~, **5.4.1.1(7)** if tolerated by the pipe system and the tank is not anchored to the ground;

– Inelastic behaviour is restricted within limited portions of the tank, and the ultimate deformations of the materials are not exceeded;

– The nature and the extent of buckling phenomena in the shell are adequately controlled;

– The hydraulic systems which are part of, or connected to the tank are designed so as to prevent loss of the tank content following failure of any of its components;

~~3.2~~4.2 Combination of ground motion components

(1) P Clause ~~2~~3.2(1)P applies to tanks.

(2) Clause ~~2~~3.2(2) applies to tanks.

(3) P Clause ~~2~~3.2(3)P applies to tanks.

~~3.3~~4.3 Methods of analysis

~~3.3.1~~4.3.1 General

(1) P The model to be used for the determination of the seismic effects shall reproduce properly the stiffness, the strength, the damping, the mass and the geometrical properties of the containment structure, and shall account for the hydrodynamic response of the contained liquid and - where necessary - for the effects of~~, and~~the interaction with the foundation soil, ~~when necessary~~.

(2) P Tanks shall be generally analysed considering elastic behaviour, unless proper justification is given for the use of nonlinear analysis in particular cases.

NOTE Information on m~~M~~ethods for seismic analysis of tanks of usual shapes are given in Informative Annex B.

(3) P The ~~localized~~localised non linear phenomena, admitted in the seismic design situation for which the ultimate limit state is verified (see ~~3~~4.1.3), shall be restricted so as to not affect the global dynamic response of the tank to any significant extent.

(4) Possible interaction between different tanks due to connecting piping~~s~~ shall be considered whenever appropriate.

~~3.3.2~~4.3.2 Behaviour factors

(1) P Tanks of type other than those mentioned below shall be either designed for elastic response (q up to 1,5, accounting for overstrength), or, for properly justified cases, for inelastic response (see ~~1.1~~02.34.1(2)), provided that its~~the~~ acceptability ~~of their inelastic response~~ is~~shall be~~ adequately demonstrated.

(2)P Clause ~~2~~3.4 applies also to elevated tanks.

(3)P For non-elevated tanks~~ other than those of (2)~~, the energy dissipation corresponding to the selected value of q shall be properly substantiated and the necessary ductility provided through ductile design. However, t~~T~~he ~~full~~ elastic response spectra (see EN 1998-1:2004, 3.2.2.2 and 3.2.2.3) ~~elastic design action (i.e., q = 1), however,~~ shall, in all cases, be used for the evaluation of the convective part of the liquid response.

(5) Steel tanks with vertical axis, supported directly on the ground or on the foundation may be designed with a behaviour factor q greater than~~>~~ 1 provided that the tank is designed in such way to allow uplift. Unless~~If~~ the inelastic behaviour is ~~not justified~~ evaluated by any more refined ~~scientifically proven~~ approach, the behaviour factor q ~~may~~ should not be ~~be~~ taken larger than~~equal to~~:

– 1,5 for unanchored tanks, provided that the design rules of EN 1993-4-2 are fulfilled, especially those concerning the thickness of the bottom plate, which shall be less than the thickness of the lower shell course.

–2 for tanks with specially designed ductile anchors allowing an ~~elongation~~ increase in length without rupture~~,~~ equal to R/200, where R is the tank radius.

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~~3.3.3~~4.3.3 Hydrodynamic effects

(1) P A rational method based on the solution of the hydrodynamic equations with the appropriate boundary conditions shall be used for the evaluation of the response of the tank system to the design seismic actions defined in ~~1.9~~2.2~~3~~.

(2) P In particular, the analysis shall properly account for the following, where relevant:

– the convective and the impulsive components of the motion of the liquid;

– the deformation of the tank shell due to the hydrodynamic pressures~~,~~ and the interaction effects with the impulsive component;

– the deformability of the foundation soil and the ensuing modification of the response.

(3) For the purpose of evaluating the dynamic response under seismic actions, the liquid may be generally assumed as incompressible.

(4) Determination of the ~~critical~~ maximum hydrodynamic pressures induced by horizontal and vertical excitation requires in principle use of nonlinear dynamic (time-history) analysis. Simplified methods allowing for a direct application of the response spectrum analysis may be used, provided that suitable conservative rules for the combination of the peak modal contributions are adopted.

NOTE Informative Annex B gives information on acceptable procedures for the combination of the peak modal contributions in response spectrum analysis.~~.~~ It~~ nformative Annex B~~ gives also ~~appropriate~~ expressions for the calculation of the sloshing wave height.

~~3.4~~4.4 Verifications

~~3.4.1~~4.4.1 Damage limitation state

(1) P Under the ~~In the~~ seismic action ~~design situation~~ relevant to the damage limitation state, ~~if it is specified,~~ the tank structure shall be checked to satisfy the –serviceability limit state verifications of the relevant material Eurocodes for tanks or liquid-retaining structures.

NOTE: The issue of damage limitation states has to be re-checked, as there are no explicit compliance criteria.

~~3.4.1.1~~4.4.1.1 Shell

4~~3~~.4.1.1.1 Reinforced and prestressed concrete shells

(1) Calculated crack widths in the seismic design situation relevant to the damage limitation state, may be compared to the values specified in ~~clause 4.4.2 of~~ EN 1992-1-1:2004, **4.4.2** taking into account the appropriate environmental exposure class and the sensitivity of the steel to corrosion.

-(2) In case of lined concrete tanks, transient concrete crack widths shall not exceed a value that might induce local deformation in the liner exceeding -50% of its ultimate uniform elongation.

4~~3~~.4.1.1.2 Steel shells

(1) Clause 2~~3~~**.5.1(2)** applies to tanks.

~~3.4.1.2~~4.4.1.2 Piping

(1) Piping needs to be verified for the damage limitation state only if special requirements are imposed to active on-line components, such as valves or pumps

(2) P Relative displacements due to differential seismic movements of the ground shall be accounted for when the piping and the tank(s) are supported on different foundations.

(3) If reliable data are not available or accurate analyses are not made, a minimum value of the imposed relative displacement between the first anchoring point of the piping and the tank may be assumed as:

$$\Delta = \frac{x d_g}{500} \quad (3.1)$$

where x (in m~~m~~) is the distance between the anchoring point of the piping and the point of connection with the tank, and d_g is the design ground displacement as given in EN 1998-1: 2004~~X~~, **3.2.2.4(1)**.

(4)P The resistance of piping elements shall be- evaluated as ~~taken equal to that applying~~in the ~~in the~~ persistent or transient design situations.

(5) The region of the tank where the piping is attached to should be designed to remain elastic under the forces transmitted by the piping amplified by a factor $\gamma_p = 1{,}3$.

~~3.4.2~~4.4.2 Ultimate limit state

~~3.4.2.1~~4.4.2.1 Stability

(1) P Clause 2~~3~~**.5.2.1(1)**P applies to tanks.

(2) P Clause 2~~3~~**.5.2.1(2)**P applies to tanks.

~~3.4.2.2~~4.4.2.2 Shell

(1) P Clause 3~~2~~.5.2.2(1) applies to tanks.

NOTE : Information on ~~Appropriate~~ expressions for checking the ultimate strength capacity of the shell, as controlled by various failure modes are given in Informative Annex B~~A~~.

~~3.4.2.3~~4.4.2.3 Piping

(1) P Under the combined effects of inertia and service loads, as well as under the imposed relative displacements, yielding of the piping at the connection to the tank shall not occur. The connection of the piping to the tank shall have an overstrength factor of not less than 1,~~.~~3 with respect to the piping.

~~3.4.2.4~~4.4.2.4 Anchorages

(1) P Clause **2.5.2.3(1)** applies to tanks.

~~3.4.2.5~~4.4.2.5 Foundations

(1) P Clause ~~2~~**3.5.2.4(1)**P applies to tanks.

(2) P Clause 3~~2~~**.5.2.4(2)**P applies to tanks.

~~3.5~~4.5 Complementary measures

~~3.5.1~~4.5.1 Bunding

(1) P Tanks, single or in groups, which are designed to control or avoid leakage in order to prevent fire, explosions and release of toxic materials shall be bunded~~,~~ (i.e. shall be surrounded by a ditch and/or an embankment), if the seismic action used for the verification of the damage limitation state is smaller than the design seismic action (used for the verification of the ultimate limit state).

(2) P If tanks are built in groups, bunding ~~shall~~ may be provided either to every individual tank or to the whole group. However, if the consequences ~~, depending on the risk~~ associated with the failure of the bund are severe, individual bunding shall be used.

(3) P The bunding shall be designed to retain its full integrity (absence of leaks) under the design seismic action considered for the ultimate limit state of the enclosed system.

~~3.5.2~~4.5.2 Sloshing

(1) P In the absence of explicit justifications, a freeboard shall be provided having a height not less than the calculated height of the sloshing waves (see ~~referred to in~~ **~~3~~4.3.3(4~~5~~))**.

(2)~~P~~ Damping devices, as for example grillages or vertical partitions may be used to reduce sloshing.

~~3.5.3~~4.5.3 Piping interaction

(1)P The piping shall be designed to ~~minimize~~minimise unfavourable effects of interaction between tanks and between tanks and other structures.

~~4~~5 SPECIFIC RULES FOR ABOVE-GROUND PIPELINES

~~4.1~~5.1 General

(1) This section aims at providing principles and application rules for the seismic design of the structural aspects of above-ground pipeline systems. This ~~Section~~ section may also be used as a basis for evaluating the resistance of existing above-ground piping and to assess any required strengthening.

(2) The seismic design of an above-ground pipeline comprises the establishment ~~determination~~ of the ~~supports~~ location and characteristics of the supports in order to limit the strain in the piping components and to limit the loads applied to the equipment located on the pipeline, such as valves, tanks, pumps or instrumentation. Those limits are not defined in this standard and should be provided by the Owner of the facility or the manufacturer of the equipment.

(3) Pipeline systems usually comprise several associated facilities, such as pumping stations, operation centres, maintenance stations, etc., each of them housing different types of mechanical and electrical equipment. Since these facilities have a considerable influence on the continued operation of the system, it is necessary to give them adequate consideration in the seismic design process aimed at satisfying the overall reliability requirements.

(4)- Explicit treatment of these facilities, however, is not within the scope of this standard. ~~; i~~In fact, some of those facilities are ~~already~~ covered in EN 1998-1, while the seismic design of mechanical and electrical equipment requires additional specific criteria that are beyond the scope of Eurocode 8.

(4) P For the formulation of the general requirements to follow, as well as for their implementation, a distinction ~~needs to be~~is made among the pipeline systems covered by the present standard i.e.:

- single lines
- ~~and~~ redundant networks.

(5) P For this purpose, a pipeline is considered as a single line when its behaviour during and after a seismic event is not influenced by that of other pipelines, and if the consequences of its failure relate only to the functions demanded from it.

~~4.2~~5.2 Safety r~~R~~equirements

5.2.1 Damage limitation state

(1) P Pipeline systems shall be constructed in such a way as to be able to maintain their supplying capability as a global servicing system after the seismic event defined for the "Minimum operating level" (see **2.1.2**), even if with considerable local damage.

(2) A global deformation up to 1,5 times the yield deformation is acceptable, provided that there is no risk of buckling and the loads applied to active equipment, such as valves, pumps, etc., are within its operating range.

5.2.2 Ultimate limit state

(1) P The main safety hazard directly associated with the pipeline rupture during a seismic event is explosion and fire, particularly with regard to gas pipelines. The remoteness of the location and the size of the population that is exposed to the impact of rupture shall be considered in establishing the level of protection.

(2) P For pipeline systems in environmentally sensitive areas, the damage to the environment due to pipeline ruptures shall also be considered in the definition of the acceptable risk..

~~4.2.1~~5.2.3 Reliability differentiation

(1) P For purposes of reliability differentiation the different components in a pipeline system are classified as follows:~~.~~

Importance Class I: Buildings, facilities and equipment that may deform inelastically to a moderate extent without unacceptable loss of function (non-critical piping support structures, buildings enclosing process operations, etc). It is unlikely that failure of the component will cause extensive loss of life.~~Structures and equipment performing vital functions that shall remain nearly elastic. Items that are essential for the safe operation of the pipeline or any facility, or components that would cause extensive loss of life or a major impact on the environment in case of damage. Other items, which are required to remain functional to avoid damage that would cause a lengthy shutdown of the facility (emergency communications systems, leak detection, fire control, etc.).~~

Importance Class II: Items that ~~shall~~ must remain operational after an earthquake, but need not operate during the event; Structures that may deform slightly in the inelastic range; Facilities that are ~~vital~~important, but whose service may be interrupted until minor repairs are made. It is unlikely that failure of the component will cause extensive loss of life.

Importance Class III: Structures and equipment performing vital functions that must remain nearly elastic. Items that are essential for the safe operation of the pipeline or any facility. Components that would cause extensive loss of life or have a major impact on the environment in case of damage. Other items, which are required to remain functional to avoid damage that would cause a lengthy shutdown of the facility (emergency communications systems, leak detection, fire control, etc.).~~Buildings, facilities and equipment that may deform inelastically to a moderate extent without unacceptable loss of function (noncritical piping support structures, buildings enclosing process operations, etc). It is unlikely that failure of the component will cause extensive loss of life.~~

(2) The values of the importance factors appropriate to each class and as function of the use of the facility are given in Table 2~~1~~.1N of ~~1.8~~2.1~~2~~.4 (4).

~~4.2.2Damage limitation requirements~~

~~(1) P Pipeline systems shall be constructed in such a way as to be able to maintain their supplying capability as a global servicing system as much as possible, even under considerable local damage due to high intensity earthquakes.~~

~~For this, a global deformation up to 1.5 times the yield deformation is acceptable, provided there is no risk of buckling and the loads applied to active equipment, such as valves, pumps, etc.; are acceptable.~~

~~4.2.3Safety requirements~~

~~(1) P The principal safety hazard directly associated with the pipeline rupture under a seismic event is explosion and fire, particularly with regard to gas pipelines. The remoteness of the location and the size of the population that is exposed to the impact of rupture shall be considered in establishing the level of protection.~~

~~(2) P For pipeline systems in environmentally sensitive areas, the damage to the environment due to pipeline ruptures shall also be considered in the definition of acceptable risk.~~

~~4.3~~5.3 Seismic action

~~4.3.1~~5.3.1 General

(1)P The following direct and indirect seismic hazard types are relevant for the seismic design of above-ground pipeline systems:

~~a)~~- Shaking of the pipelines due to the seismic movement applied to their supports.

~~b)~~- Differential movement of the supports of the pipelines.

(2) For differential movement of supports two different situations may exist:

- For supports which are directly on the ground, significant differential movement is present only if there are soil failures and/or permanent deformations

- For supports which are located on different structures its seismic response may create differential movements on the pipeline;

~~4.3.2~~5.3.2 Earthquake vibrations

(1) P The quantification of the~~one~~ horizontal components of the earthquake vibrations shall be carried out in terms of the~~a~~ response spectrum, (or a compatible time history representation ~~(mutually consistent~~) as presented in ~~of~~ EN 1998-1:~~200X~~2004, **3.2.2**~~, which is referred to as containing the basic definitions~~.

(2) Only the three translational components of the seismic action should be taken into account, (i.e., the rotational components may be neglected).

~~4.3.3~~5.3.3 Differential movement

(1) When the pipeline is supported directly on the ground, the differential movement may be neglected, except when soil failures or permanent deformations occur. In that case the amplitude of the movement should be evaluated with appropriate techniques.

(2) When the pipeline is supported on different structures, their differential movement should be defined from their analysis or by simplified envelope approaches.

~~4.4~~5.4 Methods of analysis

~~4.4.1 Above ground pipelines~~

~~4.4.1.1~~5.4.1 ~~Modeling~~Modelling

(1) P The model of the pipeline shall be able to represent the stiffness, the ~~and~~ damping and the mass properties, as well as the dynamic degrees of freedom of the system, with explicit consideration of the following aspects, as appropriate:

– flexibility of the foundation soil and foundation system

– mass of the fluid inside the pipeline

– dynamic characteristics of the supporting structures

– type of connection between pipeline and supporting structure

– joints along the pipeline and between the supports

~~4.4.1.2~~5.4.2 Analysis

(1)~~P~~ Above ground pipelines may be analysed by means of the ~~multi~~modal response spectrum analysis with the associated design response spectrum as given in EN 1998-1: 2004~~X~~, **3.2.2.5**~~.~~ and combining the modal responses according to EN 1998-1:2004, **4.3.3.3.2.**

NOTE Additional information regarding the combination of modal responses, namely for the use of the Complete Quadratic Combination is given in EN 1998-2: 2004, **4.2.1.3**.

(2) Time history analysis with spectrum compatible accelerograms according to EN~~V~~ 1998-1:2004~~X~~, **3.2.3** is also allowed.

(3) Simplified ~~static~~ lateral force analyses are acceptable, provided that the value of the applied acceleration is justified. A value equal to 1~~.~~,5 times the peak of the support spectrum is acceptable.

PT NOTE: This rule is under discussion. Possible link to cl.4.3.5.2 of EN1998-1:2004.

(~~2~~4)P The seismic action shall be applied separately along two orthogonal directions (transverse and longitudinal, for straight pipelines) and the maximum combined response shall be obtained according to ~~, if the response spectrum approach is used, by using the SRSSrule~~EN 1998-1:2004, **4.3.3.5.1(2)** and **(3)**.

~~-(3) Guidance on the choice between the two methods is given in EN 1998-2: 200X, **4.2.1.3**.~~

~~-~~(4~~5~~)P Spatial variability of the motion shall be considered whenever the length of the pipeline exceeds 600 m or when geological discontinuities or marked topographical changes are present.

NOTE Appropriate models to take into account the spatial variability of the motion are given in Informative Annex D of EN 1998-2: 200X

~~4.4.1.3~~5.4.3 Behaviour factors

(1) The dissipative capacity of an above-ground pipeline, if any, is restricted to its supporting structure, since it ~~would be~~is both difficult and inconvenient to develop energy dissipation in the supported pipes, except for welded steel pipes. On the other hand, shapes and material used for the supports vary widely, which makes it unfeasible to establish values of the behaviour factors of general applicability.

(2) For the supporting structures, appropriate values of *q* may be taken from EN 1998-1 and EN 1998-2, on the basis of the specific layout, material and level of detailing.

~~(2)~~(3) Welded steel pipelines exhibit significant deformation and dissipation capacity, ~~as soon as~~provided that -their thickness is sufficient. For pipelines which have a radius over thickness ~~(R/t)~~ ratio (R/t) -less than 50, the behaviour factor, *q*, to be used for the verification of the pipes ~~shall~~ may be taken equal to 3. If this ratio is less than 100, *q* ~~shall~~ may be taken equal to 2. Otherwise, *q* ~~is~~ may be taken equal to 1.

PT NOTE: Possible use q=1,5 as the minimum on account for overstrength is under discussion .

(4) For the verification of the supports, the seismic loads derived from the analysis should be multiplied by (1+*q*)/2.

PT NOTE: It has to be clarified whether the q factor takes the value of the behaviour factor used for the verification of the pipelines or of the supporting structure.

~~(3)For other cases, appropriate values of *q* may be taken from EN 1998-1 and EN 1998-2, on the basis of the specific layout, material and level of detailing.~~

~~4.5~~5.5 Verifications

(1)-P The load effect induced in the supporting elements (piers, frames, etc) in the seismic design situation shall be less than or equal to the resistance evaluated as for the persistent or transient design situation.

(2)-P Under the most unfavourable combination of axial and rotational deformations, due to the application of the seismic action defined for the “Minimum operating level” requirement, it shall be verified that the joints do not suffer damage inducing loss of tightness.~~the joints shall not suffer damage incompatible with the specified serviceability requirements.~~

~~5~~6 SPECIFIC RULES FOR BURIED PIPELINES

~~5.1~~6.1 General

(1) P This Section aims at providing principles and application rules for the evaluation of the earthquake resistance of buried pipeline systems. ~~This wording allows for~~It applies both for the design of new and for the evaluation of existing systems.

(2) P Although large diameter pipelines are within the scope of this standard, the corresponding design criteria may not be used for apparently similar facilities, like tunnels and large underground cavities.

(3) Even though ~~various~~ distinction~~s~~ ~~can~~ could be made among different pipeline systems, like for instance single lines and redundant systems, for the sake of practicality, a pipeline is considered here as a single line if its mechanical behaviour during and after the seismic event is not influenced by that of other pipelines, and if the consequences of its possible failure relate only to the functions demanded from it.

(4) Networks are often too extensive and complex to be treated as a whole, and it is both feasible and convenient to identify separate networks within the overall network. The identification may result from the separation of the larger scale part of the system (e.g. regional distribution) from the finer one (e.g. urban distribution), or from the distinction between separate functions accomplished by the same system.

(5) As an example of the latter situation, an urban water distribution system may be separated into a network serving street fire extinguishers and a second one serving private users. The separation would facilitate providing different reliability levels to the two systems. It is to be noted that the separation is related to functions and it is therefore not necessarily physical: two distinct networks can have several elements in common.

(6) The design of pipeline~~s~~ networks involves additional reliability requirements and design approaches with respect to those provided in the present standard.

~~5.2~~6.2 Safety r~~R~~equirements

6.2.1 Damage limitation state

(1)P Buried pipeline systems shall be constructed in such a way as to maintain their integrity or some of their supplying capacity after the seismic events defined for the “Full integrity” or “Minimum operating level” (see **2.1.2**), even if with considerable local damage..

6.2.2 Ultimate limit state

(1)P Clause **5.2.2(1)**P applies to buried pipelines.

(2)P Clause **5.2.2(2)**P applies to buried pipelines.

~~5.2.1~~6.2.3 Reliability differentiation

(1) P A pipeline system traversing a large geographical region normally encounters a wide variety of seismic hazards and soil conditions. In addition, a number of subsystems may be located along a pipeline transmission system, which may be either associated facilities (tanks, storage reservoirs etc.), or pipeline facilities (valves, pumps, etc.). Under such circumstances, critical stretches of the pipeline (for instance, less redundant parts of the system) and critical components (pumps, compressors, control equipment, etc.) shall be designed to provide larger reliability with regard to seismic events. Other components, that are less essential and for which some amount of damage is acceptable, need not be designed to such stringent criteria (see **2.1.4**)~~Under such circumstances, where seismic resistance is deemed to be important, critical components (pumps, compressors, control equipment, etc.) shall be designed under criteria that provide for sufficient integrity in the event of a major severe earthquake. Other components, that are less essential and are allowed to sustain greater amounts of damage, need not be designed to such stringent criteria.~~

(2) P Clause **5.2.3(1)**P applies to buried pipelines. ~~In order to adapt the reliability to the importance of the stakes, the different elements in a pipeline system shall be classified as follows.~~

~~Class I:~~ ~~Two types of pipeline system elements are considered: those for which integrity shall be assured due to the risk they represent for their environment, and those which shall remain operational after the earthquake (significant example: water supply for fire fighting). The elements of this class may undergo limited plastic deformations, which are compatible with the above requirements.~~

~~Class II:~~ ~~The elements of pipeline systems which present a limited or negligible risk. The elements of this class may undergo moderate plastic deformations.~~

(3) Clause **5.2.3(2)** applies to buried pipelines.

~~5.2.2 Damage limitation requirements~~

~~(1)P Buried pipeline systems shall be constructed in such a way as to maintain their integrity, or in special cases, when absolutely needed, some of their supplying capacity, specifically identified for given purposes, even under considerable local damage due to high intensity earthquakes.~~

~~5.2.3 Safety requirements~~

~~(1)P The risks to which goods, people and the environment are exposed in the vicinity of a pipeline system depend on various factors, either linked to the pipeline, like the transported fluid, its pressure, the pipeline diameter, etc., or linked to the environment of the pipeline: all the human, economical and environmental factors in the considered site, which are also designated by "what is at stake".~~

~~(2) P The importance of what is at stake, together with the importance of the seismic hazard, define the risk level. It's the latter which is managed by means of the pipeline design.~~

~~5.3~~6.3 Seismic action

~~5.3.1~~6.3.1 General

(1) P The following direct and indirect seismic hazard types are relevant for the seismic design of buried pipeline systems:

a) seismic waves propagating on firm ground and producing different ground shaking intensity at distinct points on the surface and spatial soil deformation patterns within the soil medium.

b) permanent deformations induced by earthquakes such as seismic fault displacements, landslides, ground displacements induced by liquefaction.

(2) P The general requirements regarding ~~the~~ damage limitation and the ultimate limit state shall, ~~in principle, to~~ be satisfied for all of the types of hazards listed above.

(3) However, for~~-~~ the hazards of type b) listed above it can be generally assumed that satisfaction of the ultimate limit state provides the satisfaction of the damage limitation requirements, so that only one check has to be performed.

~~The general requirements regarding the damage limitation state shall only be satisfied for utilities which need to remain functional after an earthquake (fire-fighting for example).~~

(~~3~~4) The fact that pipeline systems traverse or extend over large geographical areas, and the necessity of connecting certain locations, does not always allow for the best choices regarding the nature of the supporting soil. Furthermore, it may not be feasible to avoid crossing potentially active faults, or to avoid laying the pipelines in soils susceptible to liquefaction, as well as in areas that can be affected by seismically induced landslides and large differential permanent ground deformations.

(5)~~-~~ This situation is clearly at variance with that of other structures, for which a requisite for the very possibility to build is that the probability of soil failures of any type be negligible. Accordingly, i~~(4) In~~n most cases, the occurrence of hazards of type b) in **(1)**P ~~simply~~ cannot be ruled out. Based on available data and experience, reasoned assumptions ~~may~~ should be used to define a model for th~~e~~at hazard.

~~5.3.2~~6.3.2 Earthquake vibrations

(1)P The quantification of the components of the earthquake vibrations is given in ~~1.9~~2.2~~3~~.

~~5.3.3~~6.3.3 Modelling of seismic waves

(1) P A model for the seismic waves shall be established, from which soil strains and curvatures affecting the pipeline can be derived

NOTE Informative Annex C provides methods for the calculation of strains and curvatures in the pipeline for some cases, under certain simplifying assumptions.

(2) Ground vibrations in earthquakes are caused by a mixture of shear, dilatational, Love and Rayleigh waves. Wave velocities are a function of their travel path through lower and higher velocity material. Different particle motions associated with these wave types make the

strain and curvature also dependent upon the angle of incidence of the waves. A general rule is to assume that sites located in the proximity of the epicentre of the earthquake are more affected by shear and dilatational waves (body waves), while for sites at a larger distance, Love and Rayleigh waves (surface waves) tend to be more significant.

(3) P The selection of the waves to be considered and of the corresponding wave propagation velocities shall be based on geophysical considerations.

~~5.3.4~~6.3.4 Permanent soil movements

(1) P The ground rupture patterns associated with earthquake induced ground movements, either due to surface faulting or landslides, are likely to be complex, showing substantial variations in displacements as a function of the geologic setting, soil type and the magnitude and duration of the earthquake. The possibility of such phenomena occurring at given sites shall be established, and appropriate models shall be defined (see EN 1998-5).

6.4 Methods of analysis (wave passage)

(1)P It is acceptable to take advantage of the post-elastic deformation of pipelines. The deformation capacity of a pipeline shall be adequately evaluated.

> NOTE An acceptable analysis method for buried pipelines on stable soil, based on approximate assumptions on the characteristics of ground motion, is given in Informative Annex ~~B~~C.

~~5.4~~6.5 Verifications

~~5.4.1~~6.5.1 General

(1)P Pipelines buried in stable and sufficiently homogeneous soil need only be checked for the soil deformations due to wave passage.

(2)P Buried pipelines crossing areas where soil failures or concentrated distortions can occur, like lateral spreading, liquefaction, landslides and fault movements, shall be checked to resist these phenomena.

~~5.4.1.1~~6.5.1.1 Buried pipelines on stable soil ~~(Ultimate limit state)~~

(1) The response quantities to be obtained from the analysis are the maximum values of axial strain and curvature and, for unwelded joints (reinforced concrete or prestressed pipes) the rotations and the axial deformations at the joints.

~~a) welded steel pipelines~~

(2)P In welded steel pipelines t~~T~~he combination of axial strain and curvature due to the design seismic action shall be compatible with the available ductility of the material in tension and with the local and global buckling resistance in compression:~~.~~

– allowable tensile strain: 5%

– allowable compressive strain: min~~imum~~ ~~(~~{1~~%,~~ %; 40.t / D (%)}~~)~~

where t and D are the thickness and diameter of the pipe respectively.

~~b) Concrete pipelines~~

(3)P In concrete pipelines, u~~U~~nder the most unfavourable combination of axial strain and curvature, due to the design seismic action, the section of the pipe ~~:~~shall not exceed the ~~ultimate compressive~~limiting strains of concrete and steel.

~~– shall not exceed a tensile strain of steel such as to produce residual crack widths incompatible with the specified requirements.~~

(4)P In concrete pipelines, under the most unfavourable combination of axial strain and curvature, due to the seismic action for the damage limitation state, the tensile strain of the reinforcing steel shall not exceed the limiting values as to produce residual crack widths incompatible with the tightness requirements.

(4~~5~~)P Under the most unfavourable combination of axial and rotational deformations, the joints shall not suffer damage incompatible with the specified serviceability requirements.~~.~~

~~5.4.1.2~~6.5.1.2 Buried pipelines under differential ground movements (welded steel pipes) (~~ultimate limit state)~~

(1)~~P The load effects induced in the supporting elements (piers, frames, etc) by the seismic design situation shall be less than or equal to the resistance evaluated as for the persistent or transient design situation~~The segment of the pipeline deformed by the displacement of the ground, either due to fault movement or caused by a landslide or by lateral spreading shall be checked not to exceed the available ductility of the material in tension and not to buckle locally or globally in compression. The limit strains are those indicated in **6.5.1.1**.

~~(2)P Under the most unfavourable combination of axial and rotational deformations, the joints shall not suffer damages incompatible with the specified serviceability requirements.~~

~~(3) For the pipeline itself the relevant provisions in 5.5.1.1 apply.~~

~~5.5~~6.6 Design measures for fault crossings

(1) The decision to apply special fault crossing designs for pipelines where they cross potentially active fault zones depends upon cost, fault activity, consequences of rupture, environmental impact, and possible exposure to other hazards during the life span of the pipeline.

(2) In the design of a pipeline for fault crossing, the following considerations will generally improve the capability of the pipeline to withstand differential movements along the fault:

a) Where practical, a pipeline crossing a strike-slip fault should be oriented in such a way as to place the pipeline in tension.

b) Reverse faults should be intersected at an oblique angle, which should be as small as possible, to ~~minimize~~minimise compression strains. If significant strike-slip displacements are also anticipated, the fault crossing angle of the pipeline should be chosen to promote tensile elongation of the line.

(3) The depth of pipeline burial should be minimised in fault zones in order to reduce s~~S~~oil restraint on the pipeline during fault movement.

(4) An increase in pipe wall thickness will increase the pipeline's capacity for fault displacement at a given level of maximum tensile strain. It would be appropriate to use relatively thick-walled pipe within 50 m on each side of the fault.

(5) Reduction of the angle of interface friction between the pipeline and the soil also increases the pipeline's capacity for fault displacement at a given level of maximum strain. One way to accomplish this is to use a hard, smooth coating.

(6) Close control should be exercised over the backfill surrounding the pipeline over a distance of 50 m on each side of the fault. In general, a loose to medium granular soil without cobbles or boulders will be a suitable backfill material. If the existing soil differs substantially from this, oversize trenches should be excavated for a distance of approximately 15 m on each side of the fault.

(7) For welded steel pipelines, the most common approach to accommodate fault movement is to ~~utilize~~utilise the ability of the pipeline to deform well into the inelastic range in tension, in order to conform without rupture to the ground distortions. Wherever possible, pipeline alignment at a fault crossing should be selected such that the pipeline will be subjected to tension plus a moderate amount of bending. Alignments which might place the pipeline in compression are to be avoided to the extent possible, because the ability of the pipeline to withstand compressive strain without rupture is significantly less than that for tensile strain. When compressive strains exist, they should be limited to that strain which would cause wrinkling or local buckling of the pipeline.

(8) In all areas of potential ground rupture, pipelines should be laid in relatively straight sections taking care to avoid sharp changes in direction and elevation. To the extent possible, pipelines should be constructed without field bends, elbows and flanges that tend to anchor the pipeline to the ground

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ANNEX A (INFORMATIVE) SEISMIC ANALYSIS OF SILOS

A.1 Introduction and scope

This annex provides information on seismic analysis procedures for vertical cylindrical silos subjected to horizontal seismic action.

Unlike liquid storage tanks, silos containing solid-granular material and subjected to earthquake excitation have not been studied intensively. The literature in the subject is scarce (a list of few relevant publications is given below) and in spite of the rather complex mathematics involved, the available solutions are based on a number of simplifying assumptions and idealisations, leaving thus to the designer the decision on to what extent they are relevant for the case at hand. Further, again unlike the case of liquid storage tanks, the available analytical solutions are not of the form allowing an analogy to be established with simpler mechanical problems, whose solution can be rapidly obtained with the ordinary tools of earthquake engineering. Hence, when the data, or the other characteristics of a specific problem, such as for example the geometry of the silo or the properties of the insulated material, differ from those for which solution graphs and tables are provided in the references below, recourse has presently to be made to a ad-hoc modelling of both the material and the structure containing it.

This annex presents the essential features of the results given in the references 1-4, for selected combinations of parameters, without analytical derivations and formulas, with the purpose of allowing the user to check whether they are of use for the case at hand.

A.2 System considered and materials modelling

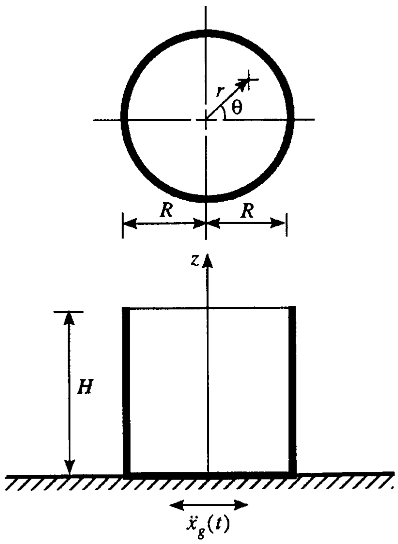

Figure A.1: System considered.

The system considered shown in ~~Fig.~~Figure A.1, is a vertical cylindrical silo assumed to be fixed to a rigid base to which the seismic motion is imposed.

The parameters defining the silo are: the height H, the radius R, the constant thickness t_w, the mass density ρ_w, the shear modulus G_w, the Poisson ratio ν_w and the damping ratio $2\xi_w$. The tank is filled with a homogeneous viscoelastic solid whose material properties are denoted by

ρ, G, ν and 2ξ in the same order as for the silo. These data completely define the elastic behaviour of the system, in particular periods and mode shapes of both the separate parts (i.e., the silo and the column of the filling material) and the combined system. In what follows the main results will be shown for the (more unfavourable) case in which the internal solid can be assumed as fully bonded to the internal face of the cylinder (rough interface).

A.3: Maximum responses of the system

The response parameters considered are: the profile along the height of the maximum pressures on the wall (these pressures vary along the circumference as cosθ), the maximum base shear, and the height (from the base) at which the resultant of the inertia forces is located. In the results shown subsequently, the mass of the silo is assumed to be negligible compared to the mass of the retained material. Corrections to account for walls inertia, when the above assumption is not satisfied, are given in ref. 4.

Following the approach used in ref. 1-4, the results are given as the product of two terms. The first one represents the response to a constant acceleration acting at the base. This part of the total response is indicated “static”. The total reponse (due to an arbitrary seismic excitation) is obtained by multiplying the static component by an appropriate amplification factor.

Static effects

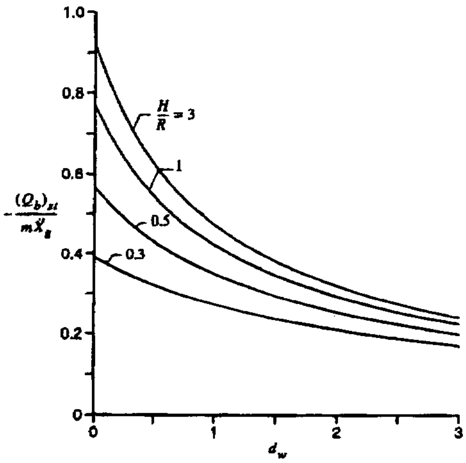

Figure A.2: Normalised values of base shear for statically excited systems with different wall flexibilities and slenderness ratios; $m_w = 0$ and $\nu = 1/3$.

The static value of the maximum base shear in the silo wall: $(Q_b)_{st}$ is plotted in ~~Fig.~~Figure A.2 as function of the relative flexibility factor:

$$d_w = \frac{G \cdot R}{G_w t_w} \tag{A.1}$$

for different slenderness ratios H/R. The values are normalised with respect to the product: $m\ddot{X}_g$, where m is the total contained mass and $\ddot{X}_g$ is the constant acceleration value. The normalising factor is thus the inertia force that would act on the mass if it were a rigid body. The results in ~~Fig.~~Figure A.2 are for $\nu = 1/3$. It is observed from ~~Fig.~~Figure A.2 that the base shear, and hence the proportion of the contained mass contributing to this shear, is highly dependent on both the slenderness ratio H/R and the relative flexibility factor d_w. For rigid (d_w

≅ 0), tall silos with values of H/R of the order 3 or more, the inertia forces for all the retained material are effectively transmitted to the wall by horizontal shearing action, and practically the entire mass of the silo content may be considered to contribute to the wall force. With decreasing H/R, a progressively larger portion of the inertia forces gets transferred by horizontal shearing action to the base, and the effective portion of the retained mass is reduced.

The effect of wall flexibility (increasing values of d_w) is to reduce the horizontal extensional stiffness of the contained material relative to its shearing stiffness, and this reduction, in turn, reduces the magnitude of the resulting pressures on and associated forces in the silo wall.

It is observed that the reduced response of the flexible silos is in sharp contrast to the well-established behaviour of liquid containing tanks, for which the effect of wall flexibility is to increase rather than decrease the impulsive components of the wall pressures and forces that dominate the response of such systems.

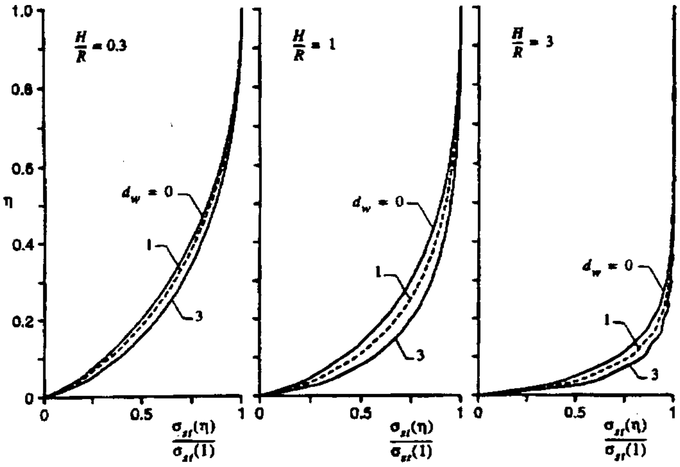

Figure A.3: Heightwise variations of static values of normal wall pressures induced in silos of different flexibilities and slenderness ratios; m_w = 0 and ν = 1/3.

The height wise variation of the maximum pressures is shown in ~~Fig.~~Figure A.3 for different values of H/R and d_w. It is observed that for broad silos these pressures increase from base to top approximately as a quarter-sine, whereas for the rather slender silos, the distribution becomes practically uniform.

Table A.1 collects the values of the maximum pressures at the top of the silo as well as the maximum base shear with the accompanying location of the centre of pressure for different combinations of the parameters H/R and d_w.

Table A.1: Static values of top radial pressure $\sigma_{st}(1)$, base shear $(Q_b)_{st}$, and effective height h; $m_w = 0$, $\nu = 1/3$ and rough interface.

H/R (1)	$d_w = 0$ (2)	$d_w = 0.5$ (3)	$d_w = 1$ (4)	$d_w = 1.5$ (5)	$d_w = 2$ (6)	$d_w = 3$ (7)
			(a) Values of $-\sigma_{st}(1)/\rho\ddot{X}_g R$			
0.30	0.365	0.293	0.245	0.210	0.183	0.146
0.50	0.540	0.398	0.315	0.260	0.222	0.170
0.80	0.671	0.466	0.356	0.288	0.241	0.182
1.00	0.709	0.483	0.365	0.294	0.246	0.185
1.25	0.731	0.492	0.370	0.297	0.247	0.186
1.50	0.740	0.495	0.372	0.298	0.248	0.186
1.75	0.744	0.496	0.372	0.298	0.248	0.186
2.00	0.746	0.497	0.373	0.298	0.248	0.186
2.50	0.745	0.497	0.372	0.298	0.248	0.186
3.00	0.744	0.497	0.372	0.298	0.248	0.186
			(b) Values of $-(Q_b)_{st}/m\ddot{X}_g$			
0.30	0.391	0.320	0.272	0.236	0.209	0.170
0.50	0.567	0.430	0.348	0.293	0.253	0.199
0.80	0.715	0.513	0.402	0.331	0.282	0.217
1.00	0.770	0.543	0.421	0.345	0.292	0.224
1.25	0.816	0.567	0.436	0.355	0.300	0.229
1.50	0.846	0.583	0.447	0.362	0.305	0.232
1.75	0.868	0.594	0.454	0.367	0.309	0.234
2.00	0.884	0.603	0.459	0.371	0.312	0.236
2.50	0.906	0.615	0.467	0.376	0.315	0.238
3.00	0.921	0.623	0.472	0.380	0.318	0.240
			(c) Values of h/H			
0.30	0.595	0.589	0.584	0.580	0.576	0.570
0.50	0.590	0.582	0.575	0.570	0.565	0.558
0.80	0.580	0.570	0.562	0.557	0.552	0.546
1.00	0.573	0.562	0.555	0.550	0.546	0.539
1.25	0.565	0.555	0.548	0.543	0.539	0.534
1.50	0.559	0.548	0.542	0.538	0.534	0.529
1.75	0.553	0.543	0.537	0.533	0.530	0.526
2.00	0.548	0.539	0.534	0.530	0.527	0.523
2.50	0.540	0.533	0.528	0.525	0.523	0.519
3.00	0.535	0.528	0.524	0.521	0.519	0.517

Total seismic response

Base shear

The maximum total dynamic base shear: $(Q_b)_{max}$, is obtained by multiplying the corresponding static value $(Q_b)_{st}$ times the so-called dynamic amplification factor *AF*. Numerical studies show that this latter factor is essentially a function of the flexibility ratio d_w, of the slenderness ratio H/R and of the fundamental period of the solid-silo system. With respect to this latter parameter, *AF* remains close to unity in the range of very short periods (i.e. for rigid tanks), then increases sharply and remains practically constant up to the periods of 0,5-0,6sec, beyond which it decreases rapidly to values lower than unity. Since the range of periods of practical importance is 0,1-0,5sec, within which *AF* does not vary significantly, the average value of *AF* in this range has been evaluated (using as input motion the El Centro N-S, 1940 record) and is reported in ~~Fig.~~Figure A.4 as a funcion of d_w and for a number of H/R values. The figure shows that for rigid silos *AF* increases fast with the increase of the slenderness ratio H/R, while the dependence tends to vanish for flexible silos.

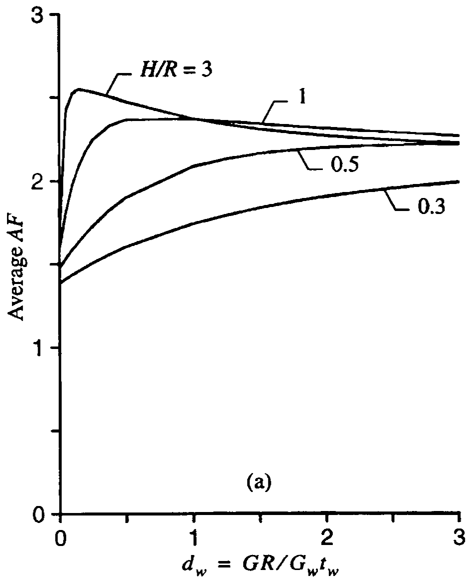

Figure A.4: Effects of silo flexibility on average amplification factor of base shear in wall of system with $0{,}1 \le T \le 0{,}5$, $\rho_w = 0$, $2\xi_w = 0{,}04$, $2\xi = 0{,}10$ subjected to ElCentro record

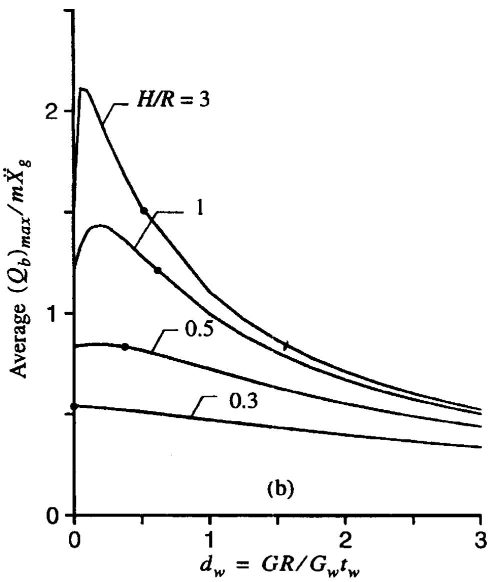

Figure A.5: Effects of silo flexibility on $(Q_b)_{\max} / m\ddot{X}_g$ average amplification factor of base shear in wall of system with $0{,}1 \le T \le 0{,}5$, $\rho_w = 0$, $2\xi_w = 0{,}04$, $2\xi = 0{,}10$ subjected to ElCentro record

The normalised dynamic base shear $(Q_b)_{max}$ corresponding to the *AF* values in ~~Fig.~~Figure A.4 is reported in ~~Fig.~~Figure A.5 as a function of d_w and for a number of *H*/*R* values. The contained material has $\nu = 1/3$ and $2\xi = 0{,}1$.

There are two main points worth commenting. The maximum response does not vary monotonically with d_w, i.e. with wall flexibility. Speifically, for systems represented by points to the right of the dots in ~~Fig.~~Figure A.5, the effect of wall flexibility is to reduce the response below the level for rigid silos (d_w = 0). Only for slender systems with moderate wall flexibility is the response likely to be higher than for the corresponding rigid silos. As already noted, this reducion of the maximum response with wall flexibility is completely at difference with what occurs with liquid-containing tanks, where wall flexibility systematically increases the response.

The second observation from the figure is that, depending on the slenderness and the flexibility of the silo, the base shear may significantly exceed the rigid-body inertia drag force: $m\ddot{X}_g$, implying that the effective mass can be considerably in excess of the total mass of the contained solid.

Overturning moment

The value of overturning base moment may be conveniently expressed as the product of the total base shear and an appropriate height *h*. The variation of the ratio *h*/*H* is not very sensitive to the wall flexibility and slenderness parameters, and does not change significantly from the static to the total response. The values are comprised between 0,5 for slender silos, for which the heightwise variation of the pressure is practically uniform, and 0,6 for squat silos whose heightwise variation is close to a quarter-sine (see ~~Fig.~~Figure A.3).

Wall pressures

References 1-4 do not contain explicitly the values of the amplification *AF* applicable to wall pressures. However, taking into account that the vertical distribution of the total wall pressures does not deviate appreciably from that of the static case, one can infer that the *AF* value appropriate for the base shear can be used in approximation also for obtaining the total wall pressures.

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ANNEX BA (IINFORMATIVE) SEISMIC ANALYSIS PROCEDURES FOR TANKS

B.1 Introduction and scope

This Annex provides information on seismic analysis procedures for tanks subjected to horizontal and vertical excitation and having the following characteristics:

a) cylindrical shape, with vertical axis and circular or rectangular cross-section;

b) rigid or flexible foundation;

c) fully or partially anchored to the foundation.

Extensions required for dealing with elevated tanks are briefly discussed, as it is the case for cylindrical tanks with horizontal axis.

A rigorous analysis of the phenomenon of dynamic interaction between the motion of the contained fluid, the deformation of the tank walls and that of the underlying foundation soil, including possible uplift, is a problem of considerable analytical complexity and requiring unusually high computational resources and efforts. Although solutions to the more simple cases of seismic response of tanks are known from the early seventies, progress in the treatment of the more complex ones is continuing up to the present, and it is still incomplete.

Numerous studies are being published, offering new, more or less approximate, procedures valid for specific design situations. Since their accuracy is problem-dependent, a proper choice requires a certain amount of specialiazed knowledge from the designer. Attention is called to the importance of a uniform level of accuracy across the design process: it would not be consistent, for ex., to select an accurate solution for the determination of the hydrodynamic pressures, and then not to use a correspondingly refined mechanical model of the tank (e.g., a F.E. model) for evaluating the stresses due to the pressures.

The necessary limitations in the scope and space of this Annex do not allow to go beyond a detailed presentation of the seismic design procedure for the simplest of all cases: rigid circular tanks anchored to a rigid base. For all the situations which make the problem more complex, as for example. the flexibility of the tank, and/or that of the foundation soil, and/or that of the anchoring system, since exact solutions are either complicated and lengthy, or non existing, a brief explanation is given of the physical phenomena distinguishing the particular situation from the reference case, and approximate solutions are either summarized or reference is made to pertinent literature.

At present, the most comprehensive documents giving guidelines for the seismic design of tanks are the ASCE volume: "Guidelines for the seismic design of oil and gas pipeline systems", 1984, ref. [5], and the Recommendations of a New Zealand Study Group: "Seismic Design of Storage Tanks", 1986, ref. [10]. Although more than ten years old they are still valuable in that they cover in detail a wide range of cases. Both documents are used as sources for the present Annex.

B.2 Vertical rigid circular tanks

B.2.1 Horizontal earthquake excitation

The complete solution of the Laplace equation for the motion of the fluid contained in a rigid cylinder can be expressed as the sum of two separate contributions, called "rigid impulsive", and "convective", respectively. The "rigid impulsive" component of the solution satisfies exactly the boundary conditions at the walls and at the bottom of the tank (compatibility between the velocities of the fluid and of the tank), but gives (incorrectly, due to the presence of the waves) zero pressure at the free surface of the fluid. A second term must therefore be added, which does not alter those boundary conditions that are already satisfied, and re-establishes the correct equilibrium condition at the top.

Use is made of a cylindrical coordinate system: r, z, θ, with origin at the cent~~e~~re of the tank bottom, and the z axis vertical. The height and the radius of the tank are denoted by H and R, respectively, ρ is the mass density of the fluid, and $\xi = r/R$, $\varsigma = z/H$, are the non~~a~~dimensional coordinates.

B.2.1.1 Rigid impulsive pressure

The spatial-temporal variation of this component is given by the expression:

$$p_{\mathrm{i}}(\xi,\varsigma,\theta,t) = C_{\mathrm{i}}(\xi,\varsigma)\rho H \cos\theta A_{\mathrm{g}}(t) \tag{B.1}$$

where:

$$C_{\mathrm{i}}(\xi,\varsigma) = \sum_{n=o}^{\infty} \frac{(-1)^{\mathrm{n}}}{I_1'(\nu_{\mathrm{n}}/\gamma)\nu_{\mathrm{n}}^2}\cos(\nu_{\mathrm{n}}\varsigma) I_1\left(\frac{\nu_{\mathrm{n}}}{\gamma}\xi\right) \tag{B.2}$$

in which:

$$\nu_{\mathrm{n}} = \frac{2n+1}{2}\pi; \gamma = H/R$$

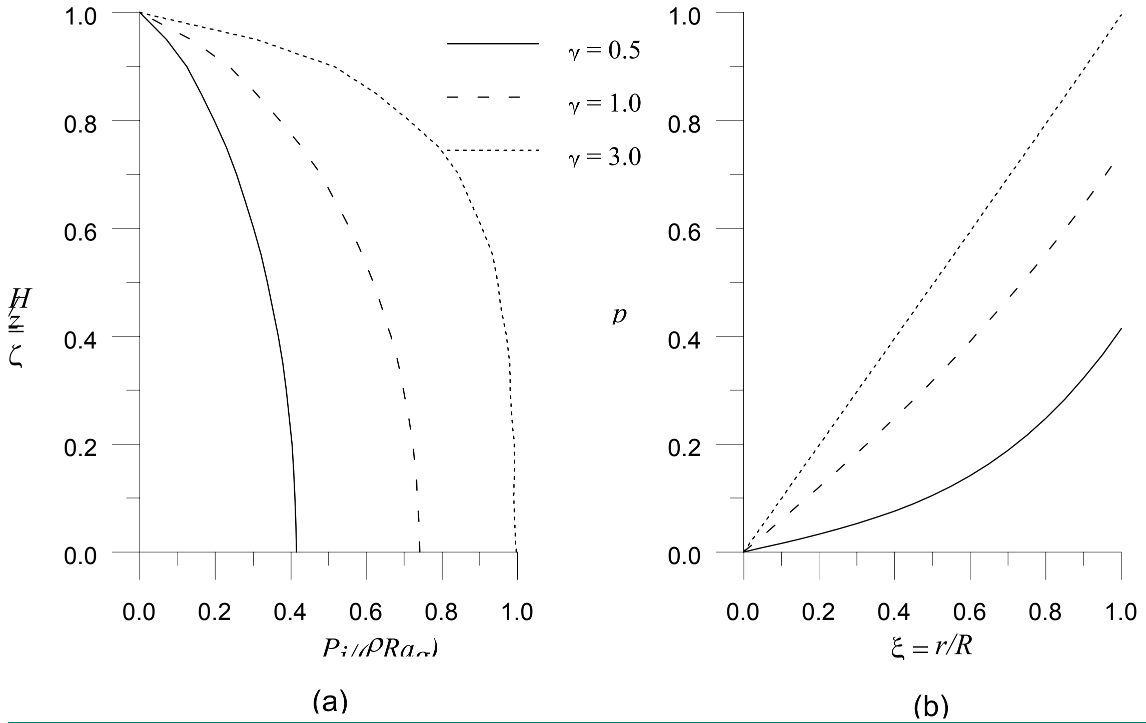
$I_1(\cdot)$ and $I_1'(\cdot)$ denote the modified Bessel function of order 1 and its derivative[5].

The time-dependence of the pressure p_{i} in eq. (B.1) is given by the function $A_{\mathrm{g}}(t)$, which represents here the free-field motion of the ground (the peak value of $A_{\mathrm{g}}(t)$ is denoted by a_{g}). The distribution along the height of p_{i} in eq. (B.1) is given by the function C_{i} and is represented in ~~Fig.~~ Figure B.1(a) for $\xi = 1$ (i.e. at the wall of the tank) and $\cos\theta = 1$ (i.e. on the plane which contains the motion), normalized to ρR a_{g} and for three values of $\gamma = H/R$.

[5] The derivative can be expressed in terms of the modified Bessel functions of order 0 and 1 as:

$$I_1'(x) = \frac{dI_1(x)}{dx} = I_0(x) + \frac{I_1(x)}{x}$$

The circumferential variation of p_i follows the function $\cos\theta$ ~~Fig.~~Figure B.1(b) shows the radial variation of p_i on the tank bottom as a function of the slenderness parameter γ. For increasing values of γ the pressure distribution on the bottom tends to become linear.



~~Fig.~~Figure B.1: Variation of the impulsive pressure for three values of $\gamma = H/R$. (a) variation along the height; (b) radial variation on the tank bottom. (Values normalized to $\rho R\, a_g$)

Pressure resultants

For a number of purposes it is useful to evaluate the horizontal resultant of the pressure at the base of the wall: Q_i, as well as the moment of the pressures with respect to an axis orthogonal to the direction of the motion: M_i. The total moment M_i immediately below the tank bottom includes the contributions of the pressures on the walls and of those on the bottom.

By making use of eq. (B.1) and (B.2) and performing the appropriate integrals one gets:

– impulsive base shear:

$$Q_i(t) = m_i A_g(t) \tag{B.3}$$

where m_i indicates the mass of the contained fluid which moves together with the walls, is called *impulsive mass,* and has the expression:

$$m_i = m 2\gamma \sum_{n=0}^{\infty} \frac{I_1(\nu_n/\gamma)}{\nu_n^3 I_1'(\nu_n/\gamma)} \tag{B.4}$$

with $m = \rho\pi R^2$ total contained mass of the fluid.

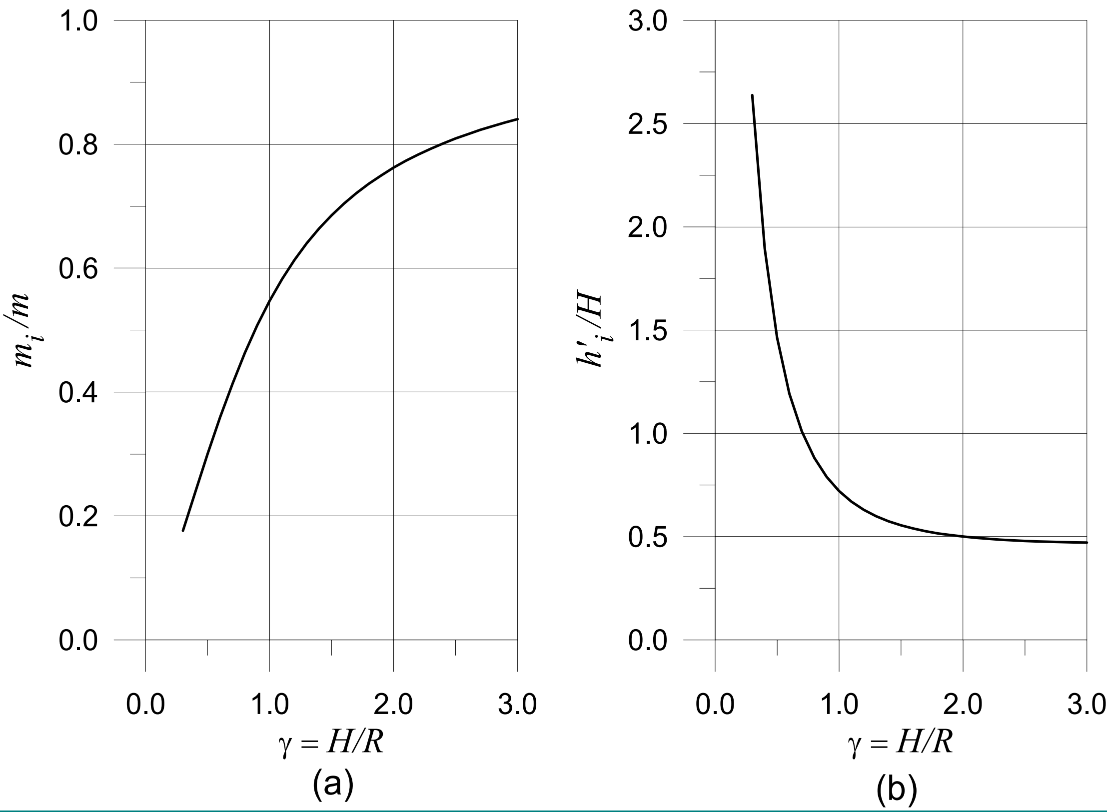
– impulsive base moment:

$$M_{\mathrm{i}}(t) = m_{\mathrm{i}} h_{\mathrm{i}}^{'} A_{\mathrm{g}}(t) \tag{B.5}$$

with

$$h_{\mathrm{i}}^{'} = H \frac{\dfrac{1}{2} + 2\gamma \sum_{\mathrm{n}=0}^{\infty} \dfrac{\nu_{\mathrm{n}} + 2(-1)^{\mathrm{n}+1}}{\nu_n^4} \dfrac{I_1(\nu_{\mathrm{n}}/\gamma)}{I_1^{'}(\nu_{\mathrm{n}}/\gamma)}}{2\gamma \sum_{n=0}^{\infty} \dfrac{I_1(\nu_{\mathrm{n}}/\gamma)}{\nu_{\mathrm{n}}^3 I_1^{'}(\nu_{\mathrm{n}}/\gamma)}} \tag{B.6}$$

The two quantities m_{i} and $h_{\mathrm{i}}^{'}$ are plotted in ~~Fig.~~ Figure B.2 as functions of the ratio $\gamma = H/R$.



~~Fig.~~ Figure B.2: Ratios m_{i} / m and h'_{i} / H as functions of the slenderness of the tank

It is noted from ~~Fig.~~ Figure B.2 that m_{i} increases with γ, to become close to the total mass for high values of this parameter, while $h_i^{'}$ tends to stabilize at about mid height. Values of $h_i^{'}$ larger than H for squat tanks are due to the predominant contribution of the pressures on the bottom.

B.2.1.2 Convective pressure component

The spatial-temporal variation of this component is given by the expression:

$$p_c(\xi,\varsigma,\theta,t)=\rho\sum_{n=1}^{\infty}\psi_n \cosh(\lambda_n\gamma\varsigma)J_1(\lambda_n\xi)\cos\theta A_n(t) \qquad \text{(B.7)}$$

with

$$\psi_n=\frac{2R}{(\lambda_n^2-1)J_1(\lambda_n)\cos h(\lambda_n\gamma)} \qquad \text{(B.8)}$$

$\lambda_1=1{,}8112 \qquad \lambda_2=5{,}3314 \qquad \lambda_3=8{,}5363$

J_1 = Bessel function of the first order

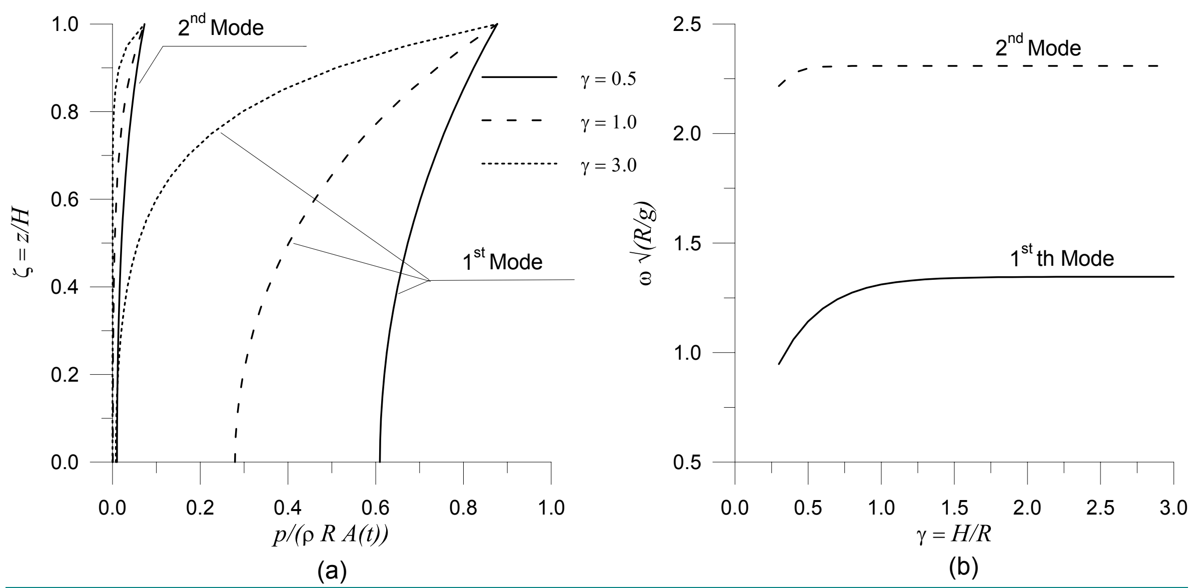
$A_n(t)$ = response acceleration of a single degree of freedom oscillator having a frequency ω_{cn}:

$$\omega_{cn}^2=g\frac{\lambda_n}{R}\tan h(\lambda_n\gamma) \qquad \text{(B.9)}$$

and a damping factor value appropriate for the fluid.

Eq. (B.7) shows that the total pressure is the combination of an infinite number of modal terms, each one corresponding to a wave form of the oscillating liquid. Only the first oscillating, or sloshing, mode and frequency, needs in most cases to be considered for design purposes.

The vertical distribution of the sloshing pressures for the first two modes are shown in ~~Fig.~~Figure B.3(a), while ~~Fig.~~Figure B.3(b) gives the values of the first two frequencies, as functions of the ratio H/R.



~~Fig.~~Figure B.3: Variation of the first two modes sloshing pressures along the height (a) and values of the first two sloshing frequencies as functions of γ (b)

One can observe from ~~Fig.~~Figure B.3 that in squat tanks the sloshing pressures maintain relatively high values down to the bottom, while in slender tanks the sloshing effect is superficial.

For the same value of the response acceleration, the contribution of the second mode is seen to be negligible. The other interesting result from ~~Fig.~~Figure B.3(b) is that the sloshing frequencies become almost independent of the parameter γ, when this is larger than about 1.

The value of ω_{c1} in this case is approximately given by the expression:

$$\omega_{c1} = 4{,}2/\sqrt{R} \qquad (R \text{ in metres}) \tag{B.10}$$

which, for the usual values of R in petrochemical plants yields periods of oscillation of the order of few seconds (for instance, T_{c1} = 4,7 sec for R = 10 m).

Pressure resultants

In a way analogous to that followed for the impulsive component one may arrive at the expressions for the base shear resultant and the total moment immediately below the bottom plate of the tank.

The base shear is given by:

$$Q_c(t) = \sum_{n=1}^{\infty} m_{cn} A_n(t) \tag{B.11}$$

with the nth modal convective mass:

$$m_{cn} = m \frac{2\tan h(\lambda_n \gamma)}{\gamma \lambda_n (\lambda_n^2 - 1)} \tag{B.12}$$

From eq. (B.11) one can note that the total shear force is given by the instantaneous sum of the forces contributed by the (infinite) oscillators having masses m_{cn}, attached to the rigid tank by means of springs having stiffnesses: $K_n = \omega_n^2 m_{cn}$. The tank is subjected to the ground acceleration $A_g(t)$ and the masses respond with accelerations $A_n(t)$.

From ~~Fig.~~Figure B.3 (and the following, ~~Fig.~~Figure B.4) one can verify that only the first of the sloshing masses needs to be considered.

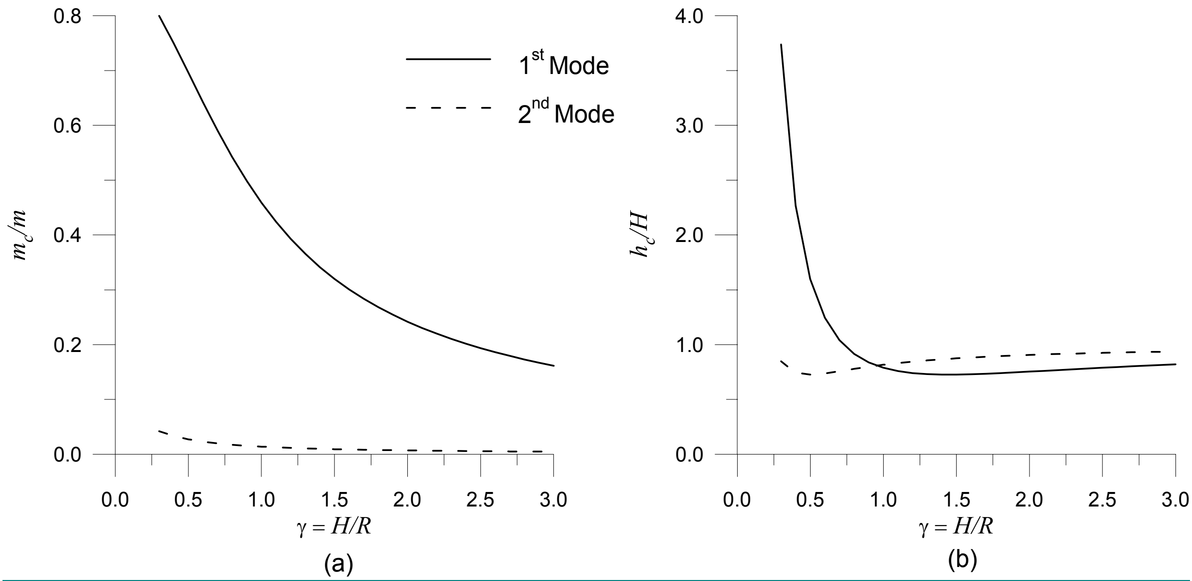
The total moment can be expressed as:

$$M_c(t) = \sum_{n=1}^{\infty} (m_{cn} A_n(t)) h_{cn} = \sum_{n=1}^{\infty} Q_{cn}(t) h_{cn} \tag{B.13}$$

where h_{cn} is the level where the equivalent oscillator has to be applied in order to give the correct value of M_{cn}:

$$h_{cn} = H\left(1 + \frac{2 - \cos h(\lambda_n \gamma)}{\lambda_n \gamma \sinh(\lambda_n \gamma)}\right) \tag{B.14}$$

The values of m_{c1} and m_{c2}, and the corresponding values of h_{c1} and h_{c2} are shown in ~~Fig.~~Figure B.4, as functions of γ.



~~Fig.~~Figure B.4: First two sloshing modal masses ~~Fig.~~(a) and corresponding heights h_{c1} and h_{c2} ~~Fig.~~(b) as functions of γ

B.2.1.3 Height of the convective wave

The predominant contribution to the sloshing wave height is provided by the first mode, and the expression of the peak at the edge is:

$$d_{\max} = 0{,}84 R S_{\mathrm{e}}\left(T_{\mathrm{c}1}\right) \qquad \text{(B.15)}$$

where $S_e(\cdot)$ is the appropriate elastic acceleration response spectrum, expressed in g (acceleration of gravity).

B.2.1.4 Combination of impulsive and convective pressures

The time-history of the total pressure is the sum of the two time-histories, the impulsive one being driven by $A_g(t)$, the convective one by $A_{c1}(t)$ (neglecting higher order components).

If, as it is customary in design practice, a response spectrum approach is preferred, the problem of suitably combining the two maxima arises. Given the generally wide separation between the central frequencies of the ground motion and the sloshing frequency, the "square root of the sum of squares" rule may become unconservative, so that the alternative, upper bound, rule of adding the absolute values of the two maxima is recommended for general use.

B.2.1.5 Effect of walls inertia

For steel tanks, the inertia forces acting on the shell due to its own mass are small in comparison with the hydrodynamic forces, and can normally be neglected. For concrete tanks however, the wall inertia forces may not be completely negligible. The inertia forces are contained in the same vertical plane of the seismic excitation; considering their component normal to the surface of the shell one has for the pressure the following expression:

$$p_{\mathrm{w}} = \rho_{\mathrm{w}} s \cos\theta A_{\mathrm{g}}(t) \tag{B.16}$$

with

ρ_{w} = mass density of the wall material

s = wall thickness

This pressure component, which is constant along the height, has to be added to the impulsive component given by eq. (B.1). The total shear at the base is obtained by simply considering the total mass of the tank multiplied by the acceleration of the ground.

B.2.2 Vertical earthquake excitation

The hydrodynamic pressure on the walls of a rigid tank due to ~~a~~ vertical ground acceleration $A_{\mathrm{v}}(t)$ is given by:

$$p_{\mathrm{vr}}(\varsigma, t) = \rho H(1-\varsigma) A_{v}(t) \tag{B.17}$$

B.2.3 Combination of pressures due to horizontal and vertical excitation

The peak combined pressure due to horizontal and vertical excitation can be obtained by applying the rule given in 3.2.

B.3 Vertical deformable circular tanks

B.3.1 Horizontal earthquake excitation

When the tank cannot be considered as rigid (this is almost always the case for steel tanks) the complete solution of the Laplace equation is ordinarily sought in the form of the sum of three contributions, referred to as: "rigid impulsive", "sloshing" and "flexible".

The third contribution is new with respect to the case of rigid tanks: it satisfies the condition that the radial velocity of the fluid along the wall equals the deformation velocity of the tank wall, plus the conditions of zero vertical velocity at the tank bottom and zero pressure at the free surface of the fluid.

Since the deformation of the wall is also due to the sloshing pressures, the sloshing and the flexible components of the solution are theoretically coupled, a fact which makes the determination of the solution quite involved. Fortunately, the dynamic coupling is very weak, due to the separation which exists between the frequencies of the two motions, and this allows to determine the third component independently of the others with almost complete accuracy. The rigid impulsive and the sloshing components examined in **B.2** remain therefore unaffected.

No closed-form expression is possible for the flexible component, since the pressure distribution depends on the modes of vibration of the tank-fluid system, and hence on the geometric and stiffness properties of the tank. These modes cannot be obtained directly from usual eigenvalue algorithms, since the participating mass of the fluid is not known a priori and also because only the modes of the type: $f(\varsigma,\theta) = f(\theta)\cos\theta$ are of interest (and these modes may be laborious to find among all other modes of a tank).

Assuming the modes as known (only the fundamental one is normally sufficient, so that in the following expressions both the mode index and the summation over all modal contributions are dropped) the flexible pressure distribution has the form:

$$p_{\mathrm{f}}(\varsigma,\theta,t)=\rho H\psi\sum_{n=0}^{\infty}d_{\mathrm{n}}\cos(\nu_{\mathrm{n}}\varsigma)\cos\theta A_{\mathrm{f}}(t) \tag{B.18}$$

with:

$$\psi=\frac{\int_0^1 f(\varsigma)\left[\frac{\rho_{\mathrm{s}}}{\rho}\frac{s}{H}+\sum_{n=0}^{\infty}b_{\mathrm{n}}^{'}\cos(\nu_{\mathrm{n}}\varsigma)\right]d\varsigma}{\int_0^1 f(\varsigma)\left[\frac{\rho_{\mathrm{s}}}{\rho}\frac{s}{H}f(\varsigma)+\sum_{n=0}^{\infty}d_{\mathrm{n}}\cos(\nu_{\mathrm{n}}\varsigma)\right]d\varsigma} \tag{B.19}$$

$$b_{\mathrm{n}}^{'}=2\frac{(-1)^{\mathrm{n}}}{\nu_{\mathrm{n}}^{2}}\frac{I_1(\nu_{\mathrm{n}}/\gamma)}{I_1^{'}(\nu_{\mathrm{n}}/\gamma)} \tag{B.20}$$

$$d_{\mathrm{n}}=2\frac{\int_0^1 f(\varsigma)\cos(\nu_{\mathrm{n}}\varsigma)d\varsigma}{\nu_{\mathrm{n}}}\frac{I_1(\nu_{\mathrm{n}}/\gamma)}{I_1^{'}(\nu_{\mathrm{n}}/\gamma)} \tag{B.21}$$

ρ_{s} is the mass density of the shell, s is its thickness and $A_{\mathrm{f}}(t)$ is the response acceleration (relative to its base) of a simple oscillator having the fundamental frequency and damping factor of the first mode.

In most cases of flexible tanks, the pressure $p_{\mathrm{f}}(\cdot)$ in eq. (B.18) provides the predominant contribution to the total pressure, due to the fact that, while the rigid impulsive term (eq. (B.1)) varies with the ground acceleration $A_{\mathrm{g}}(t)$, the flexible term (eq. (B.18)) varies with the response acceleration which, given the usual range of periods of the tank-fluid systems, is considerably amplified with respect to $A_{\mathrm{g}}(t)$.

For the determination of the first mode shape of the tank, the following iterative procedure is suggested in ref. [2]. Starting from a trial shape $f(\varsigma)$ and denoting with $f^{i}(\varsigma)$ the one corresponding to the *i*-th iteration step, an "effective" mass of the shell is evaluated as:

$$\rho^{i}(\varsigma)=\frac{p_s^{i}(\varsigma)}{2g\,s(\varsigma)\,f^{i}(\varsigma)}+\rho_s \tag{B.22}$$

where $p_s^{i}(\varsigma)$ is the amplitude of the pressure evaluated with eq. (B.18) at the *i*-th step, and $s(\varsigma)$ is the thickness of the shell, respectively.

The effective density from eq. (B.22) can then be used in a structural analysis of the tank to evaluate the (*i*+1)th mode shape, and so forth until convergence is achieved.

The fundamental frequency of the tank-fluid system can be evaluated by means of the following approximate expression:

$$f_s = \left(E\, s\, (\varsigma) / \rho\, H\right)^{1/2} / 2R\, g(\gamma) \qquad \text{(with } \varsigma = 1/3\text{)} \tag{B.23}$$

with

$$g(\gamma) = 0{,}01675\gamma^2 - 0{,}15\gamma + 0{,}46 \tag{B.24}$$

Pressure resultants

Starting from eq. (B.18), the resultant base shear and total moment at the base can be evaluated, arriving at expressions in the form:

- base shear $Q_f(t) = m_f\, A_f(t) \quad (1\text{st mode only})$ (B.25)

with

$$m_f = m\psi\gamma \sum_{n=0}^{\infty} \frac{(-1)^n}{\nu_n} d_n \tag{B.26}$$

- total moment $M_f(t) = m_f\, h_f\, A_f(t)$ (B.27)

with

$$h_f = H \frac{\left[\gamma \sum_{n=0}^{\infty} d_n \dfrac{(-1)^n \nu_n - 2}{\nu_n^2} + \sum_{n=0}^{\infty} \dfrac{d_n I_1'(\nu_n/\gamma)}{\nu_n}\right]}{\gamma \sum_{n=0}^{\infty} d_n' \dfrac{(-1)^n}{\nu_n}} \tag{B.28}$$

B.3.2 Combination of pressures terms due to horizontal excitation

The time-history of the total pressure is, in the case of flexible tanks, the sum of three time-histories: of the rigid impulsive one (eq. (B.1)), of the convective one (eq. (B.7)), and of the flexible one (eq. (B.18)) each of them differently distributed along the height and having a different variation with time.

Referring for simplicity to the base shears produced by these pressures (eqs. (B.3), (B.11) and (B.25)) one has:

$$Q(t) = m_i\, A_g(t) + \sum_{n=1}^{\infty} m_{cn}\, A_n(t) + m_f\, A_f(t) \tag{B.29}$$

where, it is recalled, $A_n(t)$ is the total or absolute response acceleration of a simple oscillator of frequency ω_n (eq. (B.9)) subjected to a base acceleration $A_g(t)$; while $A_f(t)$ is the response acceleration, relative to the base, of a simple oscillator of frequency ω_f (eq. (B.23)), and damping appropriate for the tank-fluid system, also subjected to $A_g(t)$.

If the individual maxima of the terms in eq. (B.29) are known, which can be achieved by using a response spectrum of absolute and relative accelerations, the corresponding pressures on the tank needed for a detailed stress analysis can be obtained by spreading the resultant over the tank walls and floor according to the relevant distribution.

To expedite the design process, the masses m_i, m_{cn} and m_f, the latter based on assumed first mode shapes, have been calculated as functions of the ratio γ, and are available in tabular form or in diagrams, for ex. in ref. [5] and [10].

Use of eq. (B.29) in combination with response spectra, however, poses the problem of how to superimpose the maxima~~B~~. Apart from the necessity of deriving a relative acceleration response spectrum for $A_f(t)$, there is no accurate way of combining the peak of $A_g(t)$ with that of $A_f(t)$.

In fact, since the input and its response cannot be assumed as independent in the relatively high range of frequency under consideration, the "square root of the sum of squares" rule is unconservative. On the other hand, the simple addition of the individual maxima can lead to overconservative estimates.

Given these difficulties, various approximate approaches based on the theory previously discussed have been proposed.

Two of these, presented as alternatives and illustrated in detail in ref. [5], are due to Veletsos-Yang (V.Y.) and Haroun-Housner (H.H.).

The V.Y. proposal consists essentially in replacing eq. (B.29) with the equation:

$$Q(t) = m_i A_{fa}(t) + \sum_{n=1}^{\infty} m_{cn} A_n(t) \qquad \text{(B.30)}$$

i.e., in assuming the entire impulsive mass to respond with the amplified absolute response acceleration of flexible tank system ($A_{fa}(t) = A_f(t) + A_g(t)$). The maximum of $A_{fa}(t)$ is obtained directly from the appropriate response spectrum.

The V.Y. procedure is an upper bound solution, whose approximation has been proven to be acceptable for H/R ratios not much larger than 1. Above this value, corrections to decrease the conservativeness are suggested. In view of the conservative nature of the method, the effects of tank inertia may generally be neglected. If desired, the total base shear can be evaluated approximately by the expression:

$$Q_w(t) = (\varepsilon_o \cdot m) \cdot A_{fa}(t) \qquad \text{(B.31)}$$

where $A_{fa}(t)$ is the pseudoacceleration response of the tank-fluid system, and ($\varepsilon_o \cdot m$) is the effective participating mass of the tank wall in the first mode, where m is the total mass of the tank and the factor ε_o may be determined approximately from:

H/R	0,5	1,0	3,0
ε_o	0,5	0,7	0,9

The H.H. proposal starts by writing eq. (B.29) in the form:

$$Q(t) = m_{\mathrm{i}} A_{\mathrm{g}}(t) + \sum_{n=1}^{\infty} m_{\mathrm{cn}} A_{\mathrm{n}}(t) + m_{\mathrm{f}}\left(A_{\mathrm{fa}}(t) - A_{\mathrm{g}}(t)\right) \tag{B.32}$$

which can be re-arranged as:

$$Q(t) = (m_{\mathrm{i}} - m_{\mathrm{f}}) A_{\mathrm{g}}(t) + \sum_{n=1}^{\infty} m_{\mathrm{cn}} A_{\mathrm{n}}(t) + m_{\mathrm{f}} A_{\mathrm{fa}}(t) \tag{B.33}$$

i.e., in a form suitable for the use of the response spectrum.

The masses m_{i} and m_{j} are given in graphs as functions of H/R and s/R, together with the heights at which these masses must be located to yield the correct value of the moment (see ref. [5]).

The effects of the inertia of the tank wall are incorporated in the values of the masses and of their heights.

The "square root of the sum of squares" rule is used to combine the maximum values of the three components in eq. (B.33).

In the H.H. approach, the problem of distributing heightwise the total shear force at the base is solved by assuming a uniform pressure distribution over the tank height, which leads to a value of the hoop stress σ equal to:

$$\sigma_{\max} = \frac{1}{\pi} \frac{Q_{\max}}{H\, s} \tag{B.34}$$

Along lines similar to those of Veletsos-Yang, an even more simplified approach has been elaborated by Malhotra (1997) [8], which is reported in full below.

B.3.2.1 Simplified procedure for fixed base cylindrical tanks (Malhotra, 1997)[8]

Model

The hydrodynamic effects in a tank are evaluated by the superposition of these two components: (1) The impulsive component, which represents the action of the liquid near the base of the tank that moves rigidly with the flexible wall of the tank; and (2) the convective component, which represents the action of the liquid that experiences sloshing motion near the free-surface. In this analysis, the tank-liquid system is modeled by two single-degree-of-freedom systems, one corresponding to the impulsive and the other corresponding to the convective action. The impulsive and convective responses are combined by taking their numerical-sum rather than their root-mean-square value.

Natural periods: The natural periods of the impulsive and the convective responses, in seconds, are

$$T_{\mathrm{imp}} = C_{\mathrm{i}} \frac{\sqrt{\rho}\, H}{\sqrt{\mathrm{s/R}}\,\sqrt{\mathrm{E}}} \tag{B.35}$$

$$T_{\text{con}} = C_c \sqrt{R} \qquad \text{(B.36)}$$

where H = design liquid height, R = tank's radius, s = equivalent uniform thickness of the tank wall, ρ = mass density of liquid, and E = Young's modulus of elasticity of tank material. The coefficients C_i and C_c are obtained from Table B.1. The coefficient C_i is dimensionless, while C_c is expressed in $s/m^{1/2}$; substituting R in meters in eq. (B.36), therefore, gives the correct value of the convective period. For tanks with nonuniform wall thickness, s may be computed by taking a weighted average over the wetted height of the tank wall, assigning highest weight to the thickness near the base of the tank where the strain is maximum.

Impulsive and convective masses: The impulsive and convective masses m_i and m_c are given in Table B.1 as fractions of the total liquid mass m.

Table B.1:

H/R	C_i	C_c	m_i/m	m_c/m	h_i/H	h_c/H	h'_i/H	h'_c/H
0,3	9,28	2,09	0,176	0,824	0,400	0,521	2,640	3,414
0,5	7,74	1,74	0,300	0,700	0,400	0,543	1,460	1,517
0,7	6,97	1,60	0,414	0,586	0,401	0,571	1,009	1,011
1,0	6,36	1,52	0,548	0,452	0,419	0,616	0,721	0,785
1,5	6,06	1,48	0,686	0,314	0,439	0,690	0,555	0,734
2,0	6,21	1,48	0,763	0,237	0,448	0,751	0,500	0,764
2,5	6,56	1,48	0,810	0,190	0,452	0,794	0,480	0,796
3,0	7,03	1,48	0,842	0,158	0,453	0,825	0,472	0,825

Note: C_c is expressed in $s/m^{1/2}$

Seismic response

Base shear: The total base shear is

$$Q = (m_i + m_w + m_r) S_e(T_{\text{imp}}) + m_c S_e(T_{\text{con}}) \qquad \text{(B.37)}$$

where, m_w = the mass of tank wall, m_r = the mass of tank roof; $S_e(T_{\text{imp}})$ = the impulsive spectral acceleration, obtained from a 2 percent damped elastic response spectrum for steel or prestressed concrete tanks and a 5 percent damped elastic response spectrum for concrete tanks; $S_e(T_{\text{con}})$ = the convective spectral acceleration, obtained from a 0,5 percent damped elastic response spectrum.

Overturning moment above the base plate: The overturning moment above the base plate, in combination with ordinary beam theory, gives the axial stress at the base of the tank wall.

The net overturning moment immediately above the base plate is

$$M = \left(m_{\mathrm{i}}\ h_{\mathrm{i}} + m_{\mathrm{w}}\ h_{\mathrm{w}} + m_{\mathrm{r}}\ h_{\mathrm{r}}\right) S_{\mathrm{e}}\left(T_{\mathrm{imp}}\right) + m_{\mathrm{c}}\ h_{\mathrm{c}}\ S_{\mathrm{e}}\left(T_{\mathrm{con}}\right) \tag{B.38}$$

where, h_i and h_c are the heights of the centroid of the impulsive and convective hydrodynamic wall pressure; they are obtained from Table B1; h_w and h_r are heights of the centres of gravity of the tank wall and roof, respectively.

Overturning moment below the base plate: The overturning moment immediately below the base plate is on account of the hydrodynamic pressure on the tank wall as well as that on the base plate. It is given by

$$M^{'} = \left(m_{\mathrm{i}}\ h^{'}_{\mathrm{i}} + m_{\mathrm{w}}\ h_{\mathrm{w}} + m_{\mathrm{r}}\ h_{\mathrm{r}}\right) S_{\mathrm{e}}\left(T_{\mathrm{imp}}\right) + m_{\mathrm{c}}\ h^{'}_{\mathrm{c}}\ S_{\mathrm{e}}\left(T_{\mathrm{con}}\right) \tag{B.39}$$

where heights h'_i and h'_c are obtained from Table B.1.

If the tank is supported on a ring foundation, moment M should be used to design the tank wall, base anchors and the foundation. If the tank is supported on a mat foundation, moment M should be used to design the tank wall and anchors, while M' should be used to design the foundation.

Free-surface wave-height: The vertical displacement of liquid surface due to sloshing is given by eq (B.15).

B.3.3 Vertical earthquake excitation

In addition to the pressure $p_{vr}(\varsigma,t)$ given by eq. (B.17), due to the tank moving rigidly in the vertical direction with acceleration $A_v(t)$, a pressure contribution $p_{vf}(\varsigma,t)$ resulting from the deformability (radial "breathing") of the shell must be considered. This additional term has the expression:

$$p_{\mathrm{vf}}\left(\varsigma, t\right) = 0{,}815 f\left(\gamma\right) \rho H \cos\left(\frac{\pi}{2}\varsigma\right) A_{\mathrm{vf}}\left(t\right) \tag{B.40}$$

where:

$f(\gamma)$ = 1,078 + 0,274 ln γ for $0{,}8 \leq \gamma < 4$

$f(\gamma)$ = 1,0 for $\gamma < 0{,}8$

$A_{vf}(t)$ is the acceleration response function of a simple oscillator having a frequency equal to the fundamental frequency of the axisymmetric interaction vibration of the tank and the fluid.

The fundamental frequency can be estimated by means of the expression:

$$f_{\mathrm{vd}} = \frac{1}{4R}\left[\frac{2EI_1\left(\gamma_1\right)s\left(\varsigma\right)}{\pi \rho H\left(1-\nu^2\right)I_o\left(\gamma_1\right)}\right]^{1/2} \qquad \text{(with } \varsigma = 1/3\text{)} \tag{B.41}$$

in which $\gamma_1 = \pi / (2\gamma)$ and where E and ν are Young modulus and Poisson ratio of the tank material, respectively.

The maximum value of $p_{vf}(t)$ is obtained from the vertical acceleration response spectrum for the appropriate values of the period and the damping. If soil flexibility is neglected (see **B.**7) the applicable damping values are those of the material (steel, concrete) of the shell.

The maximum value of the pressure due to the combined effect of the rigid: $p_{vr}(\cdot)$ and flexible: $p_{vf}(\cdot)$ contributions can be obtained by applying the "square root of the sum of squares" rule to the individual maxima~~B~~.

B.3.4 Combination of pressures due to horizontal and vertical excitation

The maximum value of the pressure due to the combined effect of horizontal and vertical excitation can be obtained by applying the "square root of the sum of squares" rule to the maximum pressures produced by each type of excitation.

B.4 Rectangular tanks

For tanks whose walls can be assumed as rigid, a solution of the Laplace equation for horizontal excitation can be obtained in a form analogous to that described for cylindrical tanks, so that the total pressure is again given by the sum of an impulsive and a convective contribution:

$$p(z,t) = p_{\mathrm{i}}(z,t) + p_{\mathrm{c}}(z,t) \tag{B.42}$$

The impulsive component has the expression:

$$p_{\mathrm{i}}(z,t) = q_o(z)\rho L A_{\mathrm{g}}(t) \tag{B.43}$$

where L is the half-width of the tank in the direction of the seismic action, and the function $q_o(z)$, which gives the variation of $p_i(\cdot)$ along the height ($p_i(\cdot)$ is constant in the direction orthogonal to the seismic action), is plotted in ~~Fig.~~Figure B.5.

The trend and the numerical values of the function $q_o(z)$ are quite close to those of a cylindrical tank with radius $R = L$.

The convective pressure component is given by a summation of modal terms (sloshing modes), each one having a different variation with time. As for cylindrical tanks, the dominant contribution is that of the fundamental mode, that is:

$$p_{\mathrm{c1}}(z,t) = q_{\mathrm{c1}}(z)\rho L A_1(t) \tag{B.44}$$

where the function $q_{c1}(z)$ is shown in ~~Fig.~~Figure B.6 together with the 2nd mode contribution $q_{c2}(z)$ and $A_1(t)$ is the acceleration response function of a simple oscillator having the frequency of the first mode, the appropriate value of the damping, and subjected to an input acceleration $A_g(t)$.

The period of oscillation of the first sloshing mode is:

$$T_1 = 2\pi \left(\frac{L/g}{\frac{\pi}{2}\tanh\left(\frac{\pi}{2}\frac{H}{L}\right)} \right)^{1/2} \qquad \text{(B.45)}$$

Pressure resultants

The base shear and the moment on the foundation could be evaluated on the basis of expressions (B.43) and (B.44).

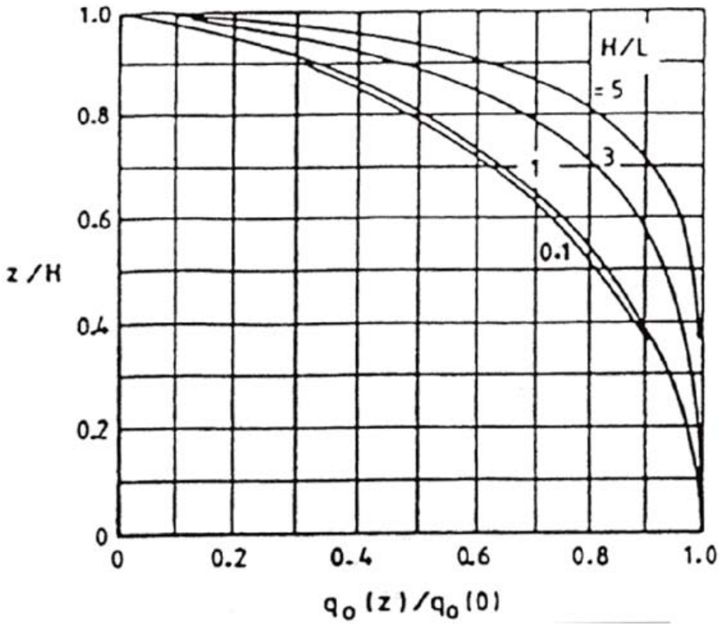
According to reference [10], for design purposes the values of the masses m_i and m_{c1}, as well as of the corresponding heights above the base: h'_i and h_{c1}, calculated for cylindrical tanks and given by the expressions (B.4), (B.12) and (B.6), (B.14), respectively, may be adopted for rectangular tanks as well (with L replacing R), with a margin of approximation not exceeding 15%.

Flexible walls

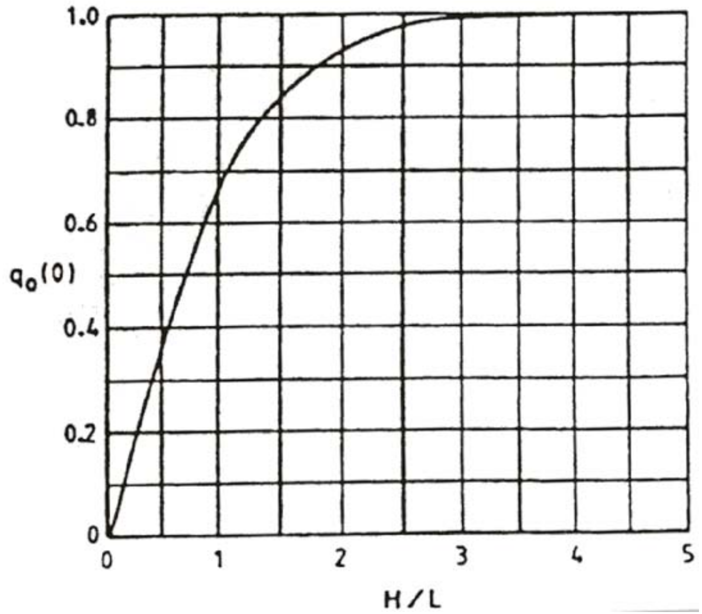
Wall flexibility produces generally a significant increase of the impulsive pressures, while leaving the convective pressures practically unchanged. The reason for this difference is the same discussed previously for the case of cylindrical tanks, i.e., the uncoupling of the sloshing oscillations from the dynamic deformations of the walls, due to the separation of their respective periods.

Studies on the behaviour of flexible rectangular tanks are not numerous, and the solutions are not amenable to a form suitable for direct use in design: for a recent treatment of the subject see for example. ref. [6].

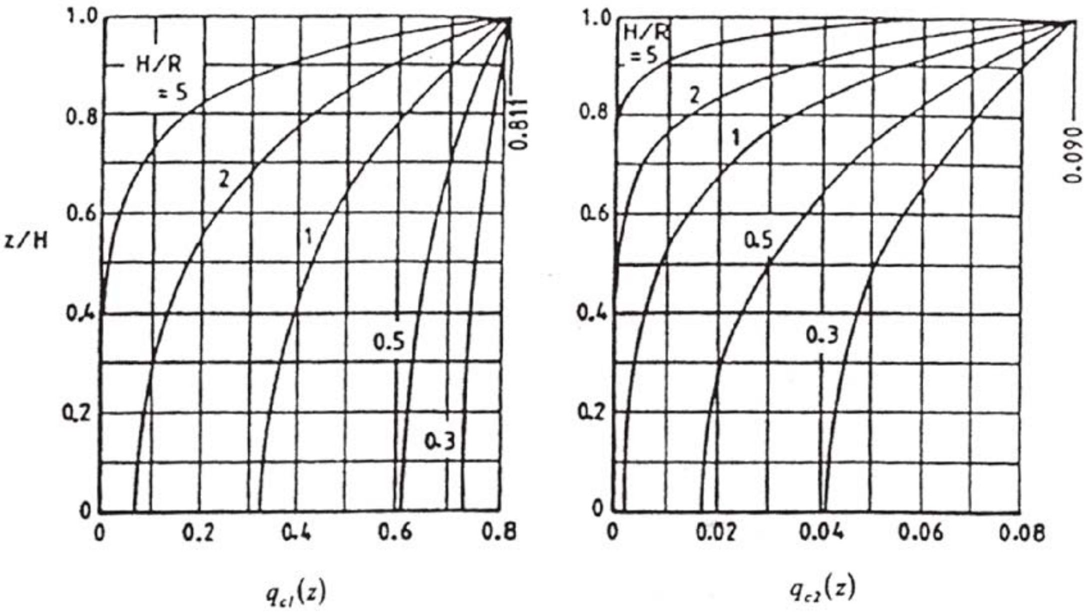
For design purposes, an approximation which is suggested in ref. [10] is to use the same vertical pressure distribution valid for rigid walls, see eq. (B.43) and ~~Fig.~~Figure B.5, but to replace the ground acceleration $A_g(t)$ in eq. (B.43) with the response acceleration of a simple oscillator having the frequency and the damping factor of the first impulsive tank-liquid mode.



~~Fig.~~Figure B.5(a): Dimensionless impulsive pressures on rectangular tank wall perpendicular to direction of earthquake (from ref. [10])



~~Fig.~~Figure B.5(b): Peak value of dimensionless impulsive pressures on rectangular wall perpendicular to direction of earthquake (from ref. [10])



~~Fig.~~Figure B.6: Dimensionless convective pressures on rectangular tank wall perpendicular to direction of earthquake (from ref. [10])

This period of vibration is given approximately by:

$$T_{\mathrm{f}} = 2\pi\left(d_{\mathrm{f}} / g\right)^{1/2} \tag{B.46}$$

where:

d_{f} is the deflection of the wall on the vertical centre-line and at the height of the impulsive mass, when the wall is loaded by a load uniform in the direction of the ground motion and of magnitude: $m_{\mathrm{i}}\, g/4BH$.

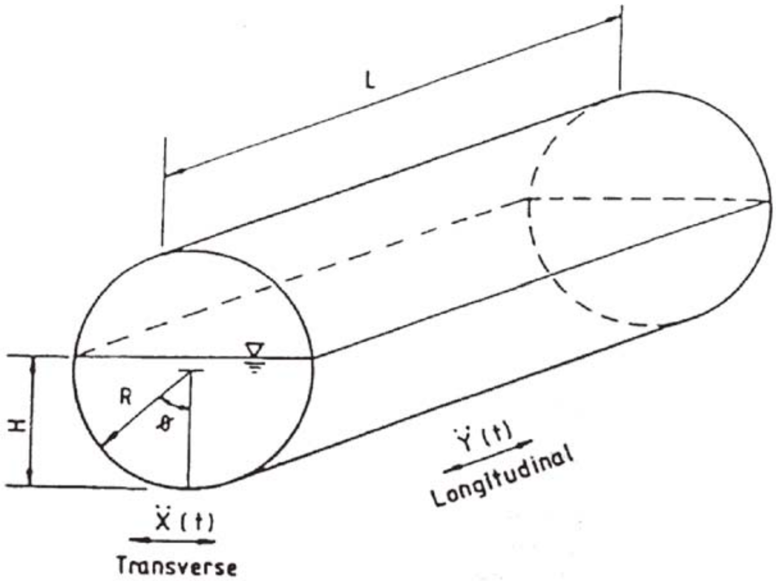
$2B$ is the tank width perpendicular to the direction of loading.

The impulsive mass m_{i} can be obtained from eq. (B.4), but should include the wall mass.

B.5 Horizontal circular cylindrical tanks

The information contained in this section **B.**5 is taken from ref. [10].

Horizontal cylindrical tanks need to be analyzed both along the longitudinal and the transverse axis: see ~~Fig.~~Figure B.7 for nomenclature.



~~Fig.~~Figure B.7: Nomenclature for horizontal axis cylindrical tank (from ref. [10])

Approximate values for hydrodynamic pressures induced by horizontal excitation in either the longitudinal or transverse direction can be obtained from solutions for the rectangular tank of equal dimension at the liquid level and in the direction of motion, and of a depth required to give equal liquid volume. This approximation is sufficiently accurate for design purposes over the range of H/R between 0,5 and 1,6. When H/R exceeds 1,6, the tank should be assumed to behave as if it were full, i.e., with the total mass of the fluid acting solidly with the tank.

For a seismic excitation perpendicular to the axis, a more accurate solution is available for partially full tanks.

The impulsive pressure distribution is given in this case by:

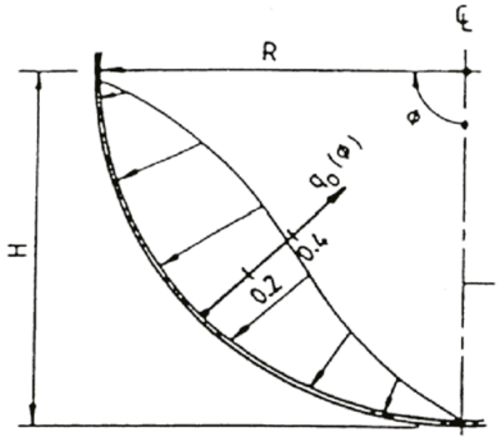
$$p_{\mathrm{i}}(\phi) = q_o(\phi)\gamma R A_{\mathrm{g}}(t) \tag{B.47}$$

For $H = R$ the pressure function $q_{\mathrm{o}}(\cdot)$ takes the form:

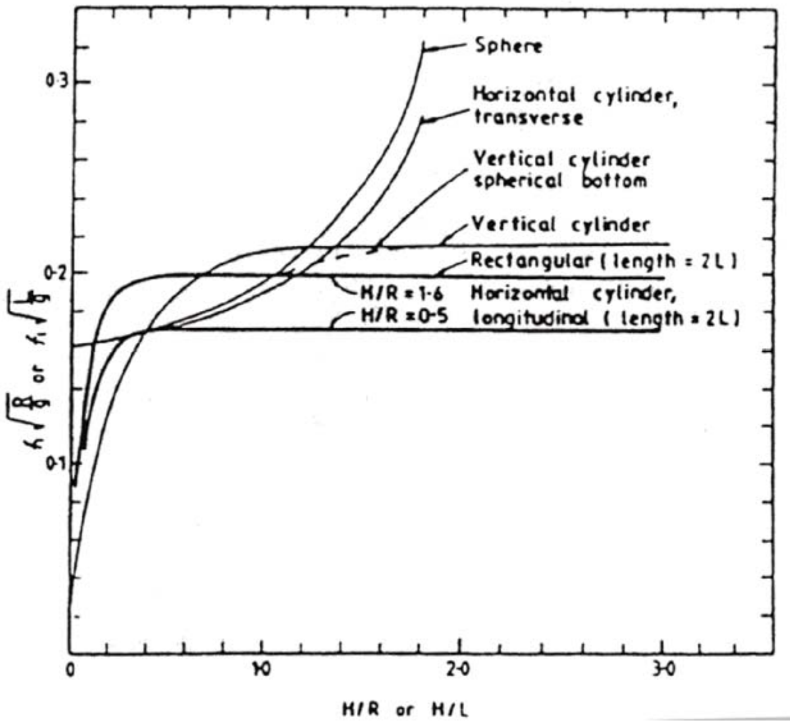
$$q_o(\phi) = \frac{H}{\pi}\sum_{1=n}^{\infty}\frac{(-1)^{n-1}}{(2n)^2-1}\sin 2n\phi \tag{B.48}$$

The function $p_{\mathrm{o}}(\cdot)$ is plotted in ~~Fig.~~Figure B.8. By integrating the pressure distribution the impulsive mass is evaluated to be:

$$m_{\mathrm{i}} = 0{,}4m \tag{B.49}$$



~~Fig.~~Figure B.8: Impulsive pressures on horizontal cylinder with $H = R$. Transverse excitation (from ref. [10])



~~Fig.~~Figure B.9 - Dimensionless first convective mode frequency for rigid tanks of various shapes (from ref. [10])

Because the pressures are in the radial direction, the forces acting on the cylinder pass through the centre of the circular section, and both the impulsive and the convective masses should be assumed to act at this point.

Solutions for the convective pressures are not available in a convenient form for design. When the tank is approximately half full ($H \cong R$), the first sloshing mode mass can be evaluated as:

$$m_{c1} = 0,6m \tag{B.50}$$

The two expressions given for the masses m_i and m_{c1} are expected to be reasonable approximations for values of H/R ranging from 0,8 to 1,2.

The first mode sloshing frequencies for tanks of various shapes, including horizontal circular cylinders, with motion along and transverse to the axis, are shown in ~~Fig.~~Figure B.9.

B.6 Elevated tanks

Elevated tanks can have supporting structures of different types, from simple cylindrical towers to frame or truss-like structures. For the purpose of the analysis, the presence of the liquid in the supported tank can be accounted for considering two masses: an impulsive mass m_i located at a height h'_i above the tank bottom (eq. (B.4) and (B.6), respectively), and a mass m_{c1} located at a height h_{c1} (eq. (B.12) and (B.14), respectively).

The mass m_i is rigidly connected to the tank walls, while the mass m_{c1} is connected to the walls through a spring of stiffness: $K_{c1} = \omega^2_{c1}\, m_{c1}$, where ω_{c1} is given by eq. (B.9).

The mass of the tank is included in the structural model which describes also the supporting structure. The response of the system can be evaluated using standard modal analysis and response spectra methods.

In the simplest possible case, the global model has only two degrees-of-freedom, corresponding to the masses m_i and m_{c1} (the mass of the tank and an appropriate portion of the mass of the support has to be added to m_i). The mass ($m_i + \Delta_m$) is connected to the ground by a spring representing the stiffness of the support.

In some cases, the rotational inertia of the mass ($m_i + \Delta_m$), and the corresponding additional degree of freedom, need also to be considered.

In the relatively common case where the shape of the elevated tank is a truncated inverted cone (or close to it), an equivalent cylinder can be considered, having the same volume of liquid as the real tank, and a diameter equal to that of the cone at the level of the liquid.

B.7 Soil-structure interaction effects

For tanks founded on relatively deformable soils, the resulting base motion can be significantly different from the free-field motion, and it includes generally a rocking component, in addition to a modified translational component.

Accurate solutions for the interaction problem between tank-fluid and soil systems have been developed only recently for the case of tanks with rigid foundation on homogeneous soil: see ref. [14], [15], [16]. The solution procedures are based on the sub-structuring approach, whereby the response of the deformable tank and of the soil beneath the foundation are first expressed separately for an excitation consisting of a horizontal and a rocking motion: equilibrium and compatibility conditions imposed at the interface yield a set of two equations on the unknown ground displacement components.

Analyses performed on tanks of various geometries confirm what was known from previous studies on building systems. Increasing the flexibility of the supporting medium lengthens the period of the tank-fluid system and reduces the peak of the response (for the same input) due to an increase of the total damping. For a given soil flexibility, the increase in the fundamental period is more pronounced for tall, slender tanks, because the contribution of the rocking component is greater for these structures than for short, broad tanks. The reduction of the peak response, however, is in general less significant for tall tanks, since the damping associated with rocking is smaller than the damping associated with horizontal translation.

Although the method in ref. [15] would be easily implemented in a computer code, simpler procedures are desirable for design purposes. One such procedure has been proposed for buildings already several years ago, see ref. [13], and consists of a modification (increase) of the fundamental period and of the damping of the structure, considered to rest on a rigid soil and subjected to the free-field motion.

This procedure has been extended to tanks, see refs. [15] and [16], and more specifically, to the impulsive (rigid and flexible) components of the response. The convective periods and pressures are assumed not to be affected by soil-structure interaction.

The recent study in ref. [15] confirms the good approximation that can be obtained through the use of an equivalent simple oscillator with parameters adjusted to match frequency and peak response of the actual system.

The properties of the replacement oscillator are given in ref. [15] in the form of graphs, as functions of the ratio H/R and for fixed values of the other parameters: wall thickness ratio s/R, initial damping, etc. These graphs can be effectively used whenever applicable.

Alternatively, the less approximate procedure of ref. [2] and [10], as summarised below, can still be adopted.

Since the hydrodynamic effects considered in **B.**2 to **B.**5 and, specifically, the impulsive rigid and impulsive flexible pressure contributions, are mathematically equivalent to a single degree-of-freedom system, and they are uncoupled from each other, the procedure operates by simply changing separately their frequency and damping factors.

In particular, for the rigid impulsive pressure components, whose variation with time is given by the free-field horizontal: $A_g(t)$, and vertical: $A_V(t)$ accelerations, inclusion of soil-structure interaction effects involves replacing the time-histories above with the response acceleration functions of a single degree of freedom oscillator having frequency and damping factors values as specified below.

Modified natural periods

– "rigid tank" impulsive effect, horizontal

$$T_i^* = 2\pi\left(\frac{m_i + m_o}{k_x \alpha_x} + \frac{m_i h_i'^2}{k_\theta \alpha_\theta}\right)^{1/2} \tag{B.51}$$

– "deformable tank" impulsive effect, horizontal

$$T_f^* = T_f\left(1 + \frac{k_f}{k_x \alpha_x} + \left[1 + \frac{k_x h_f^2}{k_\theta \alpha_\theta}\right]\right) \tag{B.52}$$

– "rigid tank", vertical

$$T_{vr}^{*} = 2\pi \left(\frac{m_{tot}}{k_v \alpha_v} \right)^{1/2} \quad (B.53)$$

– "deformable tank", vertical

$$T_{vd}^{*} = T_{vd} \left(1 + \frac{k_1}{k_v \alpha_v} \right)^{1/2} \quad (B.54)$$

where:

m_i , h'_i mass and height of the impulsive component

m_o mass of the foundation

k_f stiffness associated to the "deformable tank" = $4\pi^2 \frac{m_f}{T_f^2}$

m_{tot} total mass of the filled tank, including foundation

k_1 $= 4\pi^2 \frac{m_1}{T_{vd}^2}$, with m_l = mass of the contained liquid

where:

k_x, k_θ, k_v horizontal, rocking and vertical stiffness of the foundation

$\alpha_x, \alpha_\theta, \alpha_v$ frequency dependent factors which convert the static stiffnesses into the corresponding dynamic ones

Modified damping values

The general expression for the effective damping ratio of the tank-foundation system is:

$$\xi = \xi_s + \frac{\xi_m}{\left(T^* / T\right)^3} \quad (B.55)$$

where:

ξ_s radiation damping in the soil

ξ_m material damping in the tank

Both ξ_s and ξ_m depend on the specific oscillation mode.

In particular for ξ_s one has:

– for the horizontal impulsive "rigid tank" mode

$$\xi_s = 2\pi^2 \frac{a}{T_i^*} \left(\frac{\beta_x}{\alpha_x} + \frac{k_x h_i'^2 \beta_\theta}{k_\theta \alpha_\theta} \right) \quad (B.56)$$

– for the horizontal impulsive “deformable tank” mode

$$\xi_{\mathrm{s}} = \frac{2\pi^2 m_{\mathrm{f}}}{k_{\mathrm{x}} T_{\mathrm{f}}^{*2}} a \left(\frac{\beta_{\mathrm{x}}}{\alpha_{\mathrm{x}}} + \frac{k_{\mathrm{x}} h_{\mathrm{f}}^2 \beta_{\theta}}{k_{\theta} \alpha_{\theta}} \right) \tag{B.57}$$

– for the vertical “rigid tank” mode

$$\xi_{\mathrm{s}} = 2\pi^2 \frac{a}{T_{\mathrm{vr}}^*} \frac{\beta_v}{\alpha_v} \tag{B.58}$$

where:

a dimensionless frequency function = $\dfrac{2\pi R}{V_{\mathrm{s}} T}$ (V_{s} = shear wave velocity of the soil)

$\beta_x, \beta_v, \beta_\theta$ frequency-dependent factors providing radiation damping values for horizontal vertical and rocking motions

Expressions for the factors $\alpha_x, \alpha_\theta, \alpha_v$ and $\beta_x, \beta_\theta, \beta_v$ can be found for example in ref.[4].

B.8 Unanchored tanks

Tanks are often built with the walls not anchored to the foundation, for reasons of economy. In case of earthquake, if the overturning moment due to the hydrodynamic forces is larger than the stabilizing one some uplift occurs. It is difficult to avoid in this case plastic deformations in the tank, at least in the base plate. Leakage of the liquid, however, can be prevented by proper design.

The mechanism of tank uplift is obviously complex and substant~~e~~ially sensitive to several parameters, both from the point of view of tank response and of the subsequent stress analysis.

In most cases, the effects of the uplift, and of the accompanying rocking motion, on the magnitude and the distribution of the pressures is disregarded, and the pressures calculated for an anchored tank are used. This is believed to be in many a conservative approach, due to the fact that rocking adds flexibility to the tank-fluid system, and hence shifts the period into a range of lesser amplification. This approach is accepted in ref. [5].

The only approximate design procedure elaborated thus far which accounts for the dynamic nature of the problem is presented in ref. [3], and can be used if deemed appropriate.

For the purpose of the present Annex a conceptual outline of the procedure in ref. [3] is adequate.

– The sloshing and the rigid impulsive pressure components are assumed to remain unaffected by the rocking motion.

– The flexible impulsive component is treated using expressions analogous to eq. (B.18) to (B.28), but on the basis of a first mode shape which includes, in addition to the deformation of the shell, the uplift of the base. Modified values of the mass m_{f} and of its height h_f are obtained, as functions, as before, of the ratio H/R; of course these modified values depend on the amount of uplift, but this dependence is found numerically to be weak so that average values can be used.

- For what concerns the dynamic response, the objective is to find the fundamental period of a system made up of a deformable tank-fluid sub-system, linked to the ground by means of vertical springs characterized by a non-linear force-uplift relationship.

- The non-linearity of the base springs is treated in an "equivalent" linear way by assuming their average stiffness for a vertical deformation going from zero to the maximum value reached during the response. Based on extensive Finite Element analyses on steel tanks typical of oil industry, results have been obtained in the form of graphs, which give the fundamental period of the whole system in the form:

$$T_f = 2\pi\sqrt{\frac{R}{g}}F\left(\frac{d_{max}}{R},\frac{H}{R}\right) \tag{B.59}$$

where d_{max} is the maximum displacement at the level h_f where the mass m_f is located, and $F(\cdot)$ is an empirical function of the two non~~a~~dimensional parameters indicated.

The procedure then works iteratively as follows:

- starting with the fixed-base value of the overturning moment, a value of d_{max} is obtained using a non-dimensional graph prepared for different H/R values;

- based on this value, the period of the system is calculated from eq. (B.59), and using the appropriate response spectrum, the impulsive flexible component of the response is obtained;

- combining the latter response with the sloshing and the rigid-one, a new value of the total overturning moment is obtained, and so forth until convergence is achieved.

The limitation in the use of the procedure described is that available design charts refer to specific values of important parameters, as for ex. the thickness ratio of the wall, the soil stiffness, the wall foundation type, etc., which are known to influence the response to a significant extent.

Once the hydrodynamic pressures are known, whether determined ignoring or considering occurrence of uplift, the following step of calculating the stresses in the critical regions of the tank is a matter of structural analysis, an area in which the designer must have a certain freedom in selecting the level of sophistication of the method he uses, under the condition that the less approximate ones must be clearly on the safe side.

For an uplifting tank, an accurate model would necessarily involve a Finite Element method with non-linear capabilities, a fact which is still out of common practice. At the other extreme, rather crude methods, not requiring the use of computer, have been developed long ago, and they are still proposed in current design standards, as for ex. in ref. [10].

These methods have been proven to be unconservative by experiments and by more refined analyses and, more generally, to be inadequate for accounting of all the variables entering the problem.

Simplified but comprehensive computer methods have been proposed recently in the literature, see for ex. ref. [7] and [9], and they will gradually replace the present ones.

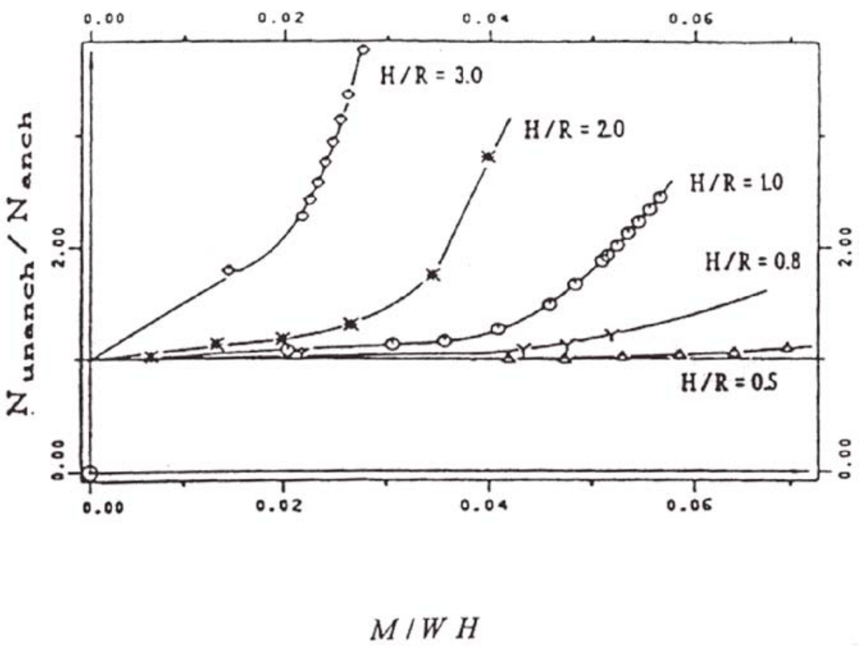
The principal effect of uplift is to increase the compressive vertical stress in the shell, which is critical with regard to buckling-related types of failure. At the opposite side of the wall where the compression is maximum, hoop compressive stresses are generated in the shell, due to the membrane action of the base plate.

These latter stresses, however, in combination with the other stress components, are not critical for the stability of the tank. Finally, flexural yielding is accepted to take place in the base plate, and a check of the maximum tensile stress is appropriate.

Compressive axial stress in the wall due to uplift

The increase of the vertical stress due to uplift (N_u) with respect to the stress in the anchored case (N_a) can be estimated from ~~Fig.~~Figure B.10, taken from ref. [12]. The ratio N_u/N_a is given in ~~Fig.~~Figure B.10 as function of the nona~~a~~dimensional overturning moment: M/WH (W = total weight of the liquid).

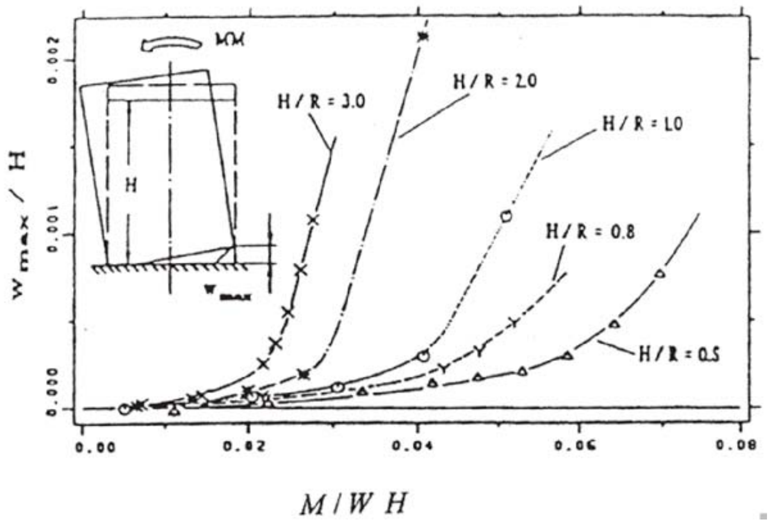
It is seen that for slender tanks the increase is very significant. The values in ~~Fig.~~Figure B.10 should be on the safe side, since they have been calculated (using static Finite Element analysis) assuming the underlying soil to be quite rigid (Winkler coefficient k=4000 N/cm^3) which is an unfavourable situation for the considered effect.



~~Fig.~~Figure B.10: Ratio of maximum compressive axial membrane force for unanchored and anchored tanks versus overturning moment (from ref. [12])

Shell uplift and uplifted length of the base plate

From a parametric study with F.E. models, performed on a number of tanks of commonly used geometry, the amount of uplift has been derived in ref. [12], and it is given in ~~Fig.~~Figure B.11 as a function of the overturning moment *M*/*WH*, for different values of the ratio *H*/*R*. For estimating the radial membrane stresses in the plate, the length *L* of the uplifted part of the tank bottom is also necessary. Results obtained from the parametric study mentioned above are shown in ~~Fig.~~Figure B.12. The dependence of *L* on the uplift *w* is almost linear, the values of *L* being larger (for a given *w*) for squat tanks than for slender ones.



~~Fig.~~Figure B.11: Maximum uplift height versus overturning moment *M*/*WH* (from ref. [12])

Radial membrane stresses in the base plate

An estimate of the membrane stress σ_{rb} in the base plate due to uplift has been derived in ref. [1]:

$$\sigma_{\mathrm{rb}} = \frac{1}{t}\left(\frac{2}{3}E\left(1-\nu^2\right)tp^2R^2\left(1-\mu\right)^2\right)^{1/3} \qquad \text{(B.60)}$$

where

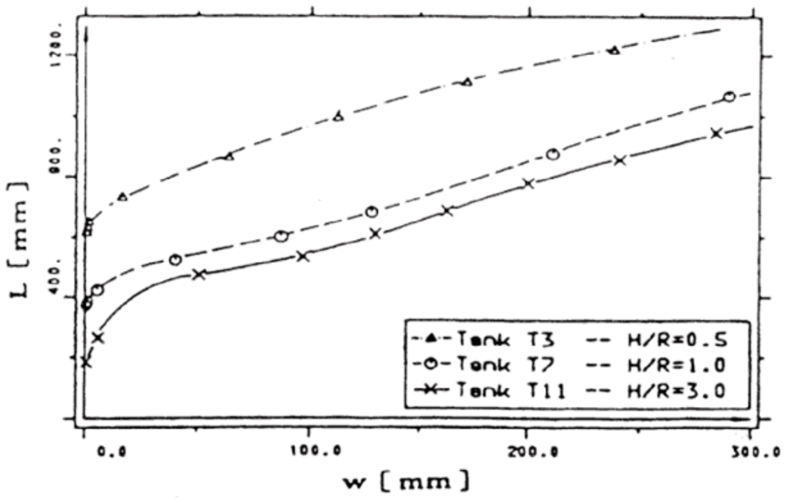
t is the thickness of the plate

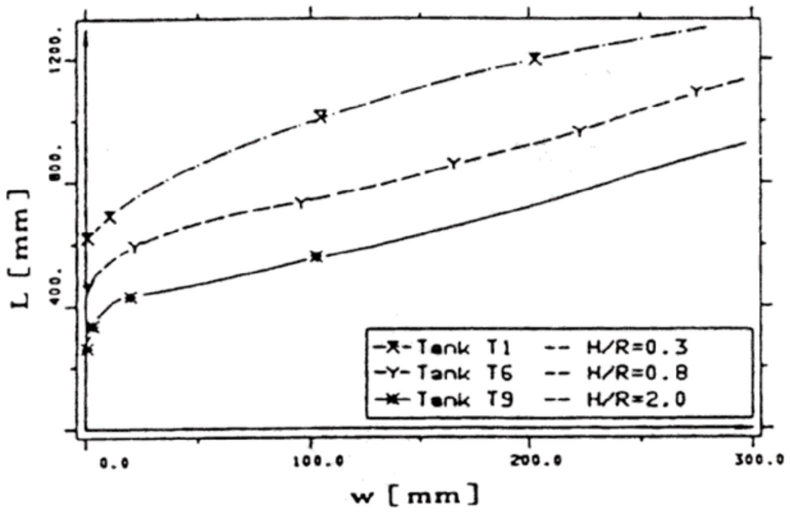
p is the hydrostatic pressure on the base

μ = $(R/L)/R$, with L = uplifted part of the base

Plastic rotation of the base plate

A recommended practice is to design the bottom annular ring with a thickness less than the wall thickness, so as to avoid flexural yielding at the base of the wall.





~~Fig.~~Figure B.12: Length of the uplifted part as a function of the uplift (from ref. [12])

The rotation of the plastic hinge in the tank base must be compatible with the available flexural ductility.

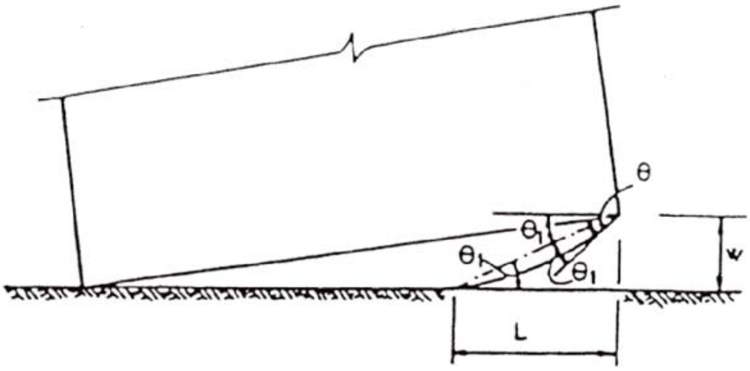
Assuming a maximum allowable steel strain of 0,05 and a length of the plastic hinge equal to 2 t, the maximum allowable rotation is.

$$\theta = \left(\frac{0{,}05}{t/2}\right) 2t = 0{,}20 \text{ radians} \tag{B.61}$$

From ~~Fig.~~Figure B.13 the rotation associated to an uplift w and a base separation of L is:

$$\theta = \left(\frac{2w}{L} - \frac{w}{2R}\right) \tag{B.62}$$

which must be less than 0,20 radians.



~~Fig.~~Figure B.13: Plastic rotation of base plate of uplifting tank (from ref. [10])

B.9 Stability verifications for steel tanks

Stability verifications have to be performed with respect to two possible failure modes.

a) Elastic buckling

This form of buckling has been observed to occur in those parts of the shell where the thickness is reduced with respect to the thickness of the base, and the internal pressure (which has a stabilising effect) is also reduced with respect to the maximum value it attains at the base. This verification should be carried out assuming the vertical component of the seismic excitation to give zero contribution to the internal pressure.

Denoting by σ_m the maximum vertical membrane stress, the following inequality shall be satisfied:

$$\frac{\sigma_m}{\sigma_{c1}} \leq 0{,}19 + 0{,}81\frac{\sigma_p}{\sigma_{c1}} \qquad \text{(B. 63)}$$

where

$$\sigma_{c1} = 0{,}6 \cdot E\frac{s}{R} \qquad \text{(B.64)}$$

(ideal critical buckling stress for cylinders loaded in axial compression)

$$\sigma_p = \sigma_{c1}\left[1 - \left(1 - \frac{\bar{p}}{5}\right)^2\left(1 - \frac{\sigma_o}{\sigma_{c1}}\right)^2\right]^{1/2} \leq \sigma_{c1} \qquad \text{(B.65)}$$

$$\bar{p} = \frac{pR}{s\sigma_{c1}} < 5 \qquad \text{(B.66)}$$

$$\sigma_o = f_y\left(1 - \frac{\lambda^2}{4}\right) \text{if} : \lambda^2 = \frac{f_y}{\bar{\sigma}\sigma_{c1}} \leq 2 \qquad \text{(B.67.a)}$$

$$\sigma_o = \bar{\sigma}\,\sigma_{c1} \qquad \text{if:} \qquad \lambda^2 \geq 2 \tag{B.67.b}$$

$$\bar{\sigma} = 1 - 1{,}24\left(\frac{\delta}{s}\right)\left[\left(1 + \frac{2}{1{,}24\left(\frac{\delta}{s}\right)}\right)^{1/2} - 1\right] \tag{B.68}$$

$\left(\frac{\delta}{s}\right)$ is the ratio of maximum imperfection amplitude to wall thickness which can be taken as (see ref. [10]):

$$\left(\frac{\delta}{s}\right) = \frac{0{,}06}{a}\sqrt{\frac{R}{s}} \tag{B.69}$$

with:

$a = 1$ for normal construction

$a = 1{,}5$ for quality construction

$a = 2{,}5$ for very high quality construction

In eq. (B.65), the second term within square brackets at the right hand side takes into account of the favourable effect of the internal pressure, while the third one (which is set as a factor of the previous one) provides the reduction of the critical stress due to the imperfections.

b) Elastic-plastic collapse

This form of buckling occurs normally close to the base of the tank, due to a combination of vertical compressive stresses, tensile hoop stresses and high shear, inducing an inelastic biaxial state of stress: the mode of collapse is referred to as ‘elephant’s foot’.

The empirical equation developed in ref. [11] to check for this form of instability is:

$$\sigma_{\mathrm{m}} = \sigma_{\mathrm{c1}}\left[1 - \left(\frac{pR}{s f_{\mathrm{y}}}\right)^2\right]\left(1 - \frac{1}{1{,}12 + r^{1{,}15}}\right)\left[\frac{r + f_{\mathrm{y}}/250}{r+1}\right] \tag{B.70}$$

where

$r = \dfrac{R/s}{400}$ and f_{y} is expressed in MPa~~B~~.

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ANNEX C (INFORMATIVE) BURIED PIPELINES

C.1 General design considerations

(1) As a rule, pipelines should be laid on soils which are checked to remain stable under the design seismic action. When the condition above cannot be satisfied, the nature and the extent of the adverse phenomena should be explicitly assessed, and appropriate design counter measures applied.

(2) Two extreme cases: Soil liquefaction and fault movements are worth being mentioned, since they require in general design solutions specific to each particular case.

(3) Soil liquefaction, whenever it did occur, has been a major contributor to pipelines distress in past earthquakes.

(4) Depending on the circumstances, the solution may consist either in increasing the burial depth, possibly also encasing the pipes in larger stiff conduits, or in placing the pipeline above-ground, supporting it at rather large distances on well founded piers. In the latter case flexible joints should also be considered to allow for relative displacements between supports.

(5) Design for fault movements requires estimating, sometimes postulating, a number of parameters including: location, size of the area affected, type and measure of the fault displacement. Given these parameters, the simplest way of modelling the phenomenon is to consider a rigid displacement between the soil masses interfacing at the fault.

(6) The general criterion for minimizing the effect of an imposed displacement is that of introducing the maximum flexibility into the system which is subjected to it.

(7) In the case under consideration this can be done:

– by decreasing the burial depth so as to reduce the soil restraint

– by providing a large ditch for the pipes, to be filled with soft material

– by putting the pipeline above ground, and introducing flexible and extensible piping elements.

C.2 Seismic actions on buried pipelines

(1) The ground motion propagating beneath the soil surface is made up of a mixture of body (compression, shear) and surface (Rayleigh, Love, etc) waves, the actual composition depending most significantly on the focal depth and on the distance between the focus and the site.

(2) The various types of waves have different propagation velocities, and different motions of the particles (i.e. parallel to the propagation of the wave, orthogonal to it, elliptical, etc.). Although geophysical-seismological studies can provide some insight, they are generally unable to predict the actual wave pattern, so that conservative assumptions have to be made.

(3) One often made assumption is to consider in turn the wave pattern to consist entirely of a single type of wave, whatever is more unfavourable for a particular effect on the pipeline.

(4) The wave trains can in this case be easily constructed on the basis of the frequency content underlying the elastic response spectrum appropriate for the site, by assigning to each frequency component an estimated value of the propagation velocity.

(5) Theoretical arguments and a number of numerical simulations indicate that the inertia forces arising from the interaction between pipe and soil are much smaller than the forces induced by the soil deformation: this fact allows to reduce the soil-pipeline interaction problem to a static one, i.e., one where the pipeline is deformed by the passage of a displacement wave, without consideration of dynamic effects.

(6) The forces on the pipeline can therefore be obtained by a time-history analysis, where time is a parameter whose function is to displace the wave along or across, the structure, which is connected to the soil through radial and longitudinal springs.

(7) A much simpler method is often used, whose accuracy has been proved to be comparable with the more rigorous approach described above, and which yields in any case an upper bound estimate of the strains in the pipeline, since it assumes it to be flexible enough to follow without slippage nor interaction the deformation of the soil.

(8) According to this method, due to Newmark,[6] the soil motion is represented by a single sinusoidal wave:

$$u(x,t) = d \sin \omega (t - \frac{x}{c}) \tag{C.1}$$

where d is the total displacement amplitude, and c is the apparent wave speed.

(9) The particle motion is assumed in turn to be along the direction of propagation (compression waves), and normal to it (shear waves) and, for simplicity and in order to take the worst case, the pipeline axis and the direction of propagation coincide.

(10) The longitudinal particle movement produces strains in the soil and in the pipeline given by the expression:

$$\varepsilon = \frac{\partial u}{\partial x} = -\frac{\omega d}{c} \cos \omega (t - \frac{x}{c}) \tag{C.2}$$

whose maximum value is:

$$\varepsilon_{\max} = \frac{v}{c} \tag{C.3}$$

with $v = \omega d$ being the peak soil velocity

[6] Newmark, N. M. 1967, Problems In Wave Propagation In Soil And Rock, Proc. Intnl. Symp. on Wave Propagation and Dynamic Properties of Earth Materials, Univ. of New Mexico, Albuquerque, New Mexico, 7-26

(11) The transverse particle movement produces a curvature χ in the soil and in the pipe given by the expression:

$$\chi = \frac{\partial^2 u}{\partial x^2} = -\frac{\omega^2 d}{c^2} \sin \omega (t - \frac{x}{c}) \tag{C.4}$$

whose maximum value is:

$$\chi_{\max} = \frac{a}{c^2} \tag{C.5}$$

with $a = \omega^2 d$ being the peak soil acceleration.

(12) If the directions of the pipeline and of the propagation do not coincide, in both cases of wave types longitudinal strains and curvatures are produced, which are functioning of the angle ~~ϑ~~θ formed by the two directions. The longitudinal strains are given in this case by

$$\varepsilon(\theta) = \frac{v}{c} \cdot f_1(\theta) + \frac{a}{c^2} f_2(\theta) \cdot R \tag{C.6}$$

where R is the diameter of the pipe. Since the second term is in general small compared with the first one, the maximum of the sum occurs when the first term is at its maximum, that is, with a value: v/c.

(13) For the condition of perfect bond between pipe and soil to be satisfied, the available friction force per unit length must equilibrate the variation of the longitudinal force leading to:

$$\tau_{av} = s\,E \frac{a}{c^2} \tag{C.7}$$

where E and s are the Modulus of Elasticity and thickness of the pipe, and τ_{av} is the average shear stress between pipe and soil which depends on the friction coefficient between soil and pipe, and on the burial depth.

EUROPEAN STANDARD

NORME EUROPÉENNE

EUROPÄISCHE NORM

FINAL DRAFT
prEN 1998-5

December 2003

ICS 91.060.00; 91.120.20

Will supersede ENV 1998-5:1994

English version

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

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

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

Ref. No. prEN 1998-5:2003 E

Contents

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

$$\tau_e = 0,65\ \alpha \cdot S \cdot \sigma_{vo} \qquad (4.4)$$

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:

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

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.

$$V_{\mathrm{Ed}} \leq F_{\mathrm{Rd}} + E_{\mathrm{pd}} \qquad (5.2)$$

(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:

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

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

$\pm 0{,}6\ \alpha \cdot S \cdot N_{\mathrm{Ed}}$ 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:

$$k_h = \alpha \frac{S}{r} \tag{7.1}$$

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

$$k_v = \pm 0{,}33 k_h \qquad \text{otherwise} \tag{7.3}$$

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:

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

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

$$R_d > E_d \qquad (7.5)$$

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**.

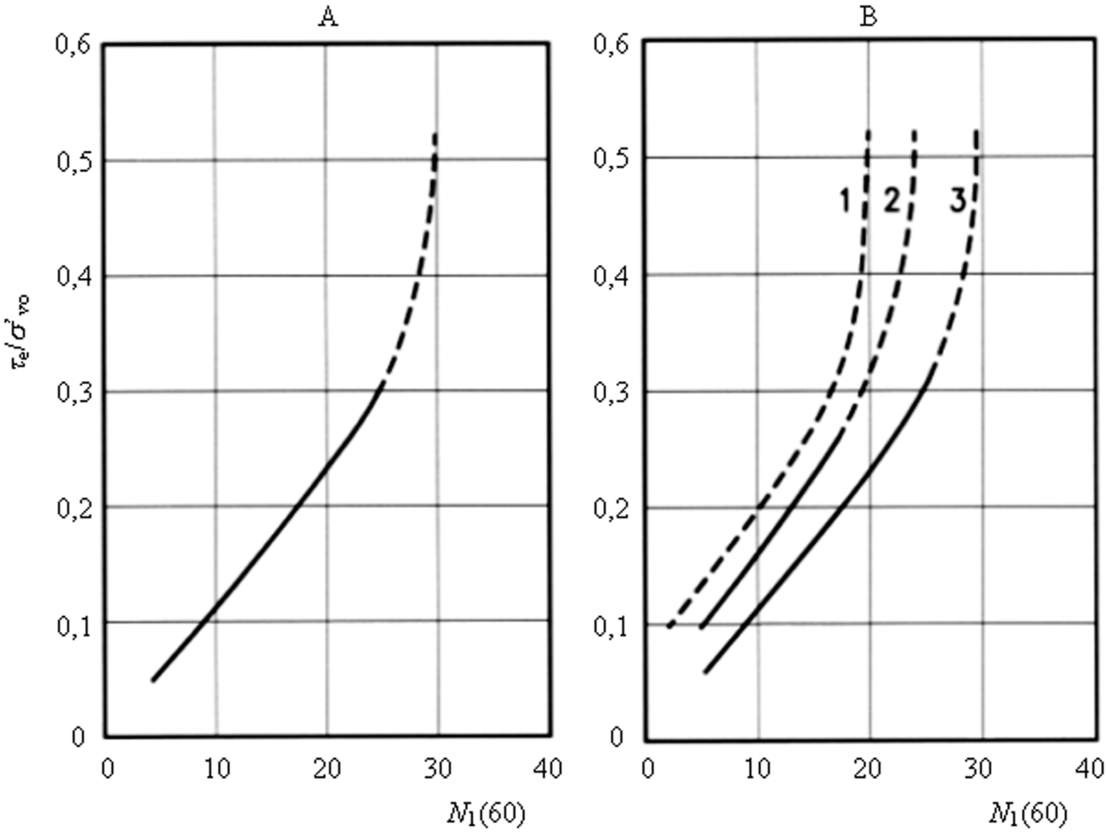
Liquefaction is not likely to occur below a certain threshold of τ_e, because the soil behaves elastically and no pore-pressure accumulation takes place. Therefore, the limiting curve is not extrapolated back to the origin. To apply the present criterion to earthquake magnitudes different from M_S = 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):

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

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)

$$\tan\theta = \frac{k_h}{1 \mp k_v} \qquad \text{(E.6)}$$

$$E_{wd} = 0 \qquad \text{(E.7)}$$

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

$$\tan\theta_A = \frac{k_h}{1 + k_v} \qquad \text{(E.8)}$$

and

$$\tan\theta_B = \frac{k_h}{1 - k_v} \qquad \text{(E.9)}$$

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.

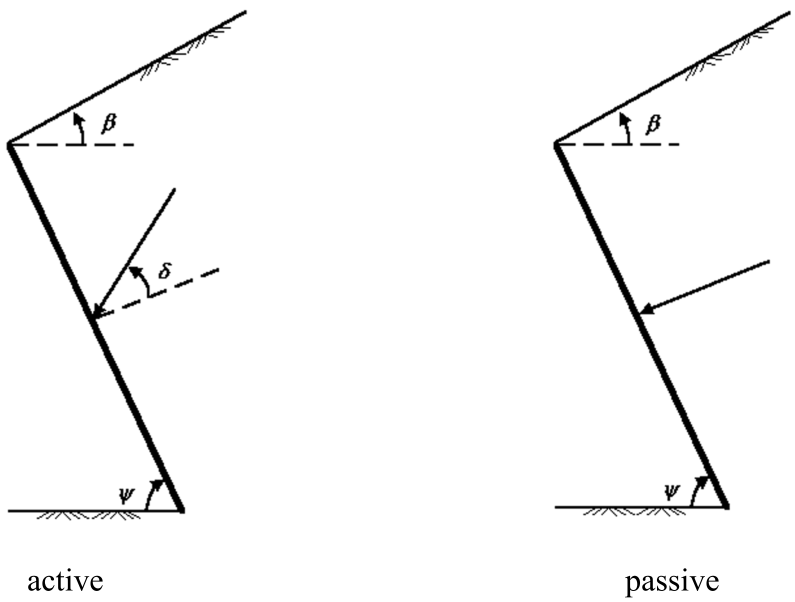


Figure E.1 — Convention for angles in formulae for calculating the earth pressure coefficient

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

$$\frac{\left(1-e\overline{F}\right)^{c_{\mathrm{T}}}\left(\beta\overline{V}\right)^{c_{\mathrm{T}}}}{\left(\overline{N}\right)^{a}\left[\left(1-m\overline{F}^{k}\right)^{k'}-\overline{N}\right]^{b}}+\frac{\left(1-f\overline{F}\right)^{c'_{\mathrm{M}}}\left(\gamma\overline{M}\right)^{c_{\mathrm{M}}}}{\left(\overline{N}\right)^{c}\left[\left(1-m\overline{F}^{k}\right)^{k'}-\overline{N}\right]^{d}}-1\leq 0 \qquad \text{(F.1)}$$

where:

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

$N_{\max}$ 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

$$N_{\max}=\left(\pi+2\right)\frac{\bar{c}}{\gamma_{\mathrm{M}}}B \qquad \text{(F.3)}$$

where

$\bar{c}$ is the undrained shear strength of soil, c_{u}, for cohesive soil, or the cyclic undrained shear strength, $\tau_{\mathrm{cy,u}}$, 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

$$\overline{F}=\frac{\rho\cdot a_{\mathrm{g}}\cdot S\cdot B}{\bar{c}} \qquad \text{(F.4)}$$

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

EUROPEAN PRESTANDARD
PRÉNORME EUROPÉENNE
EUROPÄISCHE VORNORM

prEN 1998-6

English version

Eurocode 8: Design provisions for earthquake resistance of structures

Part 6: Towers, masts and chimneys

(Revised project team draft. PreStage 49)

September 2003

CEN
European Committee for Standardisation
Comité Européen de Normalisation
Europäisches Komitee für Normung

Central Secretariat: rue de Stassart 36, B-1050 Brussels

Contents

Specific information to EN 1998-6

(1) For the design of structures in seismic regions the provisions of this Prestandard are to be applied in addition to the provisions of the other relevant Eurocodes. In particular, the provisions of the present Prestandard complement those of Eurocode 3, Part 7-1 " Towers and Masts ", and Part 7-2 " Chimneys", which do not cover the special requirements of seismic design.

National annex for EN 1998-6

Notes indicate where national choices have to be made. The National Standard implementing EN 1998-6 shall have a National annex containing values for all Nationally Determined Parameters to be used for the design in the country. National choice is required in the following sections.

Reference section	*Item*
2.1	Rules for low seismicity region. Value of the soil peak acceleration for a site being in this category.
4	Importance factors for musts, towers, and chimneys.
4.3	Proportion of ice load to be included among loads, for towers and masts in cold regions.
4.6	Height and a_g for which structures should be analysed with proper consideration to a spatial model of the seismic excitation.
4.7.2.1	Height of the structure below which a simplified analysis is allowed.
4.14	Values of the reduction factor ν that takes into account the shorter return period of the seismic action associated with the damage limitation requirement.
7.7	Behaviour factor of towers when tension occur at the base of the columns.
7.7	Behaviour factors for towers made of trussed tubes.
8.3	Drift ratio for masts.

1 GENERAL

1.1 Scope of Part 6 of Eurocode 8

(1) P EN 1998-6 establishes requirements, criteria, and rules for design of tall slender structures: towers, including bell-towers, intake towers, radio and tv-towers, masts, industrial chimneys and lighthouses. Different provisions apply to reinforced concrete and to steel structures. Requirements are set up for non-structural elements, such as the lining material of an industrial chimney, antennae and other technological equipment.

(2) P The present provisions do not apply to cooling towers and offshore structures. For towers supporting tanks, see EN 1998-4.

1.2 References

(1) P For the application of EN1998-6, reference shall be made to the entire set of Eurocodes.

(2) P Eurocode 8-6 incorporates other normative references cited at the appropriate places in the text. The most updated edition of the document shall be applied. They are listed below:

ISO 1000S I Units and recommendations for the use of their multiples and of certain other units.

ISO 8930 General principles on reliability for structures - List of equivalent terms.

ISO, Structural steel - Cold formed, welded, hollow sections -Dimensions and sectional properties. International Standard, ISI/DIS 4019, edited by ISO/TC 5/SC1.

ISO 12494 Atmospheric icing of structures.

EN 1090-1 Execution of steel structures - General rules and rules for buildings.

EN 10025 Hot rolled products of non-alloy structural steels - Technical delivery conditions.

EN 1337-1 Structural bearings - General requirements.

EN 10080-1 Steel for reinforcing of concrete - Weldable reinforcing steel- Part 1: General requirements.

EN 10080-2 Steel for reinforcing of concrete - Weldable reinforcing steel-Part 2: Technical delivery conditions for class A.

EN 10080-3 Steel for reinforcing of concrete - Weldable reinforcing steel-Part 6: Technical delivery conditions for class B.

EN 10080-4 Steel for reinforcing of concrete - Weldable reinforcing steel-Part 4: Technical delivery conditions for class C.

EN 10080-5 Steel for reinforcing of concrete - Weldable reinforcing steel-Part 5: Technical delivery conditions for welded fabric.

EN 10080-6 Steel for reinforcing of concrete - Weldable reinforcing steel-Part 6:Technical delivery conditions for lattice girders.

EN 206:2000 Concrete. - Part1: Specification, performance, production and conformity.

EN 10138 Prestressing steel. Part 1: General requirements. Part 2: Stress relieved cold drawn wire. Part 6: Strand. Part 4: Hot rolled and processed bars. Part 5: Quenched and tempered wire.

EN 10088 Stainless steels.

EN 10113 Hot rolled products in weldable fine grain structural steels.

EN 10137 Plates and wide flats made of high yield strength structural steels in the quenched and tempered or precipitation hardened conditions.

EN 10155 Structural steels with improved atmospheric corrosion resistance. Technical delivery conditions.

EN 13084-1 Free standing industrial chimneys – Part 1: General Requirements.

EN 25817 Arc-welded joints in steel: Guidance on quality levels for imperfections.

EN ISO 12944 Corrosion protection.

1.3 Assumptions

(1) P The following assumptions apply:

- Qualified and experienced personnel accomplish the design of structures.
- Adequate supervision and quality systems are provided in design offices, factories, and on site.
- Personnel having the appropriate skill and experience carry out the construction.
- The construction materials and products are used as specified in the Eurocodes or in the relevant material or product specifications.
- The structure will be adequately maintained.
- The structure will be used in accordance with the design brief.
- No change of the structure will be made during the construction phase or during the subsequent life of the structure, unless proper justification and verification is provided.

Due to the specific nature of the seismic response, this applies even in the case of changes that lead to an increase of structural resistance.

1.4 Distinction between principles and application rules

(1) The rules of clause 1.4 of EN 1990:2002 apply.

1.5 Definitions

(1) Unless otherwise stated in the following, the terminology used in International Standard ISO 8930 applies.

1.5.1 Special terms used in EN 1998-6

Stack: Stacks, flues, chimneys are construction works or building components that conduct waste gases, other flue gases, supply or exhaust air.

Supporting shaft or shell: It is the structural component, which supports the waste gas flues.

Waste gas flue: The flue that conducts waste gases is a component that carries waste gases from fireplaces through the stack outlet into atmosphere.

Internal flue: It is a waste gas conducting flue that is installed inside of the supporting shaft which protects all other stack components against thermal and chemical strains and aggressions.

Transmission tower: a tower used to support electric transmission cables, either at low or high voltage.

Tangent towers: Electric transmission towers used where the cable line is straight or has an angle not exceeding 3 degrees in plane. They support vertical loads, a transverse load from the angular pull of the wires, a longitudinal load due to unequal spans, and forces resulting from the wire-stringing operation, or a broken wire.

Angle towers: Towers used where the line changes direction by more than 3 degrees in plane. They support the same kinds of load as the tangent tower.

Dead-end towers (also called anchor towers): Towers able to support dead-end pulls from all the wires on one side, in addition to the vertical and transverse loads.

Other special, earthquake-related terms of structural significance used in Part 6 are defined in 1.4.2 of Part 1-1.

1.6 Symbols

1.6.1 General

(1) For the material-dependent symbols as well as for symbols not specifically related to earthquakes the provisions of the relevant Eurocode apply.

(2) Further symbols, used in connection with seismic actions, are defined in the text where they occur, for ease of use. However, in addition, the most frequently occurring symbols used in EN 1998-6 are listed and defined in 1.6.2.

1.6.2 Further symbols used in Part 6

E_{eq} = equivalent modulus of elasticity,
γ = specific weight of the cable per unit volume,
σ = tensile stress in the cable.

R^{θ} = (given a one degree of freedom oscillator), the ratio between the maximum moment on the oscillator spring and the rotational moment of inertia about the axis of rotation. The diagram of R^{θ} versus the natural period is the rotation response spectrum.

$R^{\theta}{}_{x}$, $R^{\theta}{}_{y}$, $R^{\theta}{}_{z}$, the rotation response spectra around the axis x, y and z, in rad/sec^2
$\bar{\xi}_j$ = equivalent modal damping ratio of the j-th mode,

M^i = effective modal mass for the i-th mode of vibration.

1.7 S.I. Units

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

(2) Forces are expressed in Newton's or kiloNewtons, masses in kg or tons, and geometric dimensions in meters or mm.

2. PERFORMANCE REQUIREMENTS AND COMPLIANCE CRITERIA

2.1 Fundamental requirements

(1) P The design philosophy of EN 1998-6 is based on the general requirement that, under earthquake conditions, 1) danger to people, nearby buildings and adjacent facilities shall be prevented, and 2), the continuity of the function of plants, industries, and communication systems has to be maintained. The first condition identifies for the present structures with the non-collapse requirement defined in section 2.1, of EN1998-1-1, and the second condition with the damage limitation requirement defined in the same 2.1 of EN1998-1-1.

(2) P The damage limitation requirement refers to a seismic action having a probability of occurrence higher than that of the design seismic action. The structure shall be designed and constructed to withstand this action without damage and limitation of use, the cost of damage being measured with regards to the cost of involved equipment, and cost of limitation of use with regards to the cost of the interruption of activity of the plant. To this requirement importance classes are defined in section 4.1.

(3) In regions of low seismicity, the rule 2.2.1 and the application of reduced or simplified earthquake forces may satisfy the fundamental requirements.

NOTE: The definition of low seismicity region for this category of structures can be found in the National Annex. In this Annex the relevant design provisions are defined.

2.2 Compliance criteria

2.2.1 General

(1) P Concrete structures shall conform to EN 1992, steel structures to EN 1993, and composite structures to EN 1994. Wind snow, and ice loads are defined in EN 1991.

(2) P With the only exceptions explicitly mentioned in 2.2.4.2, foundation design shall conform to EN 1998-5.

2.2.2 Ultimate limit state

(1) Most of the present structures are classified as non-dissipative, thus no account is taken of hysteretic energy dissipation and a behaviour factor not higher than 1,5 is adopted. For dissipative structures a behaviour factor higher than 1,5 is adopted. It accounts for hysteretic energy dissipation occurring in specifically designed zones, called dissipative zones or critical regions.

(2) P The structure shall be designed so that after the occurrence of the design seismic event, it shall retain its structural integrity, with appropriate reliability, with respect to both vertical and horizontal loads. For each structural element, the amount of inelastic deformation shall be confined within the limits of the ductile behaviour, without substantial deterioration of the ultimate resistance of the element.

2.2.3 Damage limitation state

(1) In the absence of a well precise requirement of the Owner, satisfying the deformation limits defined in Section 4.14 will ensure that damage would be prevented to the structure itself, to non-structural elements and to the installed equipment. The limits are established with reference to a seismic action having a probability of occurrence higher than that of the design seismic action.

(2) Unless special precautions are taken, provisions of the Code do not specifically provide protection against damage to equipment and non-structural elements during the design seismic event.

2.2.4.2 Foundations

(1)P The action effects for the foundation elements shall be derived on the basis of capacity design considerations accounting for the development of possible overstrength. The action effects need not exceed the action effects corresponding to the response of the structure under the seismic design situation inherent to the assumption of an elastic behaviour. In the evaluation of actions, combination of the earthquake components according to 3.5 shall be made.

(2) Requirement (1) is satisfied if the design values of the action effects on the foundations are derived as follows:

$$N_{Fd} = N_{FG} + \gamma\, N_{Ed}$$
$$M_{Fd} = \gamma\, M_{Rd}\, (N_{Fd})$$
$$V_{Fd} = \gamma\, V_{Ed}$$

where:
N_{Fd}, M_{Fd} design values of actions,

γ overstrength factor, taken equal to 1,0 for $q \leq 3$, or to 1,2 otherwise;
N_{FG} axial action due to the non-seismic actions included in the combination of actions for the seismic design situation;
N_{Ed} axial action due to the seismic actions;
M_{Rd} (N_{Fd}) bending moment resistance of the element, corresponding to N_{Fd};
V_{Ed} shear action due to the seismic actions.

Note: according to section 3.5, the axial action N_{Ed} due to the earthquake need not take into account the uplift of the foundation due to downward component of the vertical acceleration.

3 SEISMIC ACTION

3.1 Definition of the seismic input

(1)The free-field seismic excitation is specified through the definition of the translation motion at a point. For tall slender structures, the spatial variability of the translation motion at a point is important. The rotation motion at the point defines it.

(2) The translation motion is defined as in EN 1998-1-1 and the rotation motion is defined in Annex A.

3.2 Elastic response spectrum

(1)P The elastic response spectrum for acceleration is defined by EN 1998-1, section 3.2.2.2. The influence of local ground conditions on the seismic action shall generally be accounted for by considering the five subsoil classes A, B, C, D and E described in clause 3.1.1 of EN 1998-1, according to the stratigraphic profiles. The transmission level is the elevation of the lower-most level of the foundation, or the top of the piles, if present.

3.3 Design response spectrum

(1) The design response spectrum is the q-reduced response spectrum, defined in EN 1998-1, clause 3.2.2.5. The behaviour factor q incorporates the elastic dissipation in the structure, that due the soil-structure interaction, and the inelastic hysteretic behaviour of the structure.

(3) For towers and masts, depending on the member's cross section, an elastic analysis may be suited. In this case the design spectrum is applied, assigning a q factor limited to q = 1,5. Alternatively, the response spectrum can be the elastic response spectrum, characterised by a proper damping factor. In this case, if a modal analysis is developed, the damping need be assigned mode by mode. A suitable procedure is given in the Annex B.

3.4 Time-history representation

(1) If time-domain analyses are performed, both artificial accelerograms and records of historic strong motion can be used. Time-histories are generally used for non-linear step by step analyses. The relevant peak value and frequency content should be consistent with the elastic response spectrum, (not with the q- reduced design response spectrum).

(2) In case artificial accelerograms are used, independent time history can be generated for translation and rotation acceleration.

(3) The strong motion duration should be selected in a way consistent with section 3.2.3.1.2 of EN 1998-1.

3.5 Long period components of the motion at a point

(1) Towers, masts, and chimneys are sensitive to the long period components of the seismic excitation. Soft soils or peculiar topographic conditions might provide abnormal amplifications to these components.

(2) A suitable geological and geotechnical survey should be developed, to identify the soil properties. It should be extended at least until the depth at which the static effects of the structure, due to dead load, are significant.

(3) Lacking the geotechnical survey, the design spectrum corresponding to a soil profile more unfavourable for the structure should be assumed, (see 3.2.2.2 of EN 1998-1), with a soil parameter S = 1,5.

(4) Where site-specific studies of the ground motion have been carried out, with particular reference to the long period motions, the limitation of EN 1998-1, clause 3.2.2.5, $S_d \geq 0,2\ \alpha$, may be relaxed to $S_d \geq 0,1\ \alpha$.

3.6 Ground motion components

(1) The two horizontal components and the vertical component of the ground seismic acceleration occur simultaneously.

(2) In general, the rules in 4.3.3.5.1 and 4.3.3.5.2 EN1998-1 should be applied.

(3) Alternatively, if independent analyses are made for each one of them, their effects can be combined through the following combinations:

a) E_{Ex} "+" $0,30\ E_{Ey}$ "+" $0,30\ E_{Ez}$ (3.1)

b) $0,30\ E_{Ex}$ "+" E_{Ey} "+" $0,30\ E_{Ez}$ (3.2)

c) $0,30\ E_{Ex}$ "+" $0,30\ E_{Ey}$ "+" E_{Ez} (3.3)

where:
"+" implies "to be combined with";
E_{Ex} , E_{Ey} , and E_{Ex} are the effects provided by the ground acceleration in the x, y, and z direction, respectively.

Combination a), b), and c) will consider accelerations along each axis, both in the positive and in the negative direction. However, for foundation design, the effects due to the downward vertical acceleration can be omitted.

(4) The effects of the translation and the rotation components of the ground excitation can be combined each to the other assuming as global effect the square root of the sum of the squares of the single effects, (SRSS combination).

4 DESIGN OF EARTHQUAKE RESISTANT TOWERS MASTS AND CHIMNEYS

4.1 Importance factors

(1) The following factors are applicable, in the absence of a more detailed risk analysis:

γ_I = 1,4 for structures whose operation is of strategic importance, in particular if vital component of a water supply system, an electric power plant or a communication facility.

γ_I = 1,2 for structures the height of which is greater than the distance from the surrounding buildings, for structures built in an area likely to be crowded, or for structures whose collapse may cause the shutdown of industries.

γ_I = 1,1 for all structures taller than 80 m, not pertaining to the above category.

γ_I = 1,0 for the remaining cases.

NOTE: The values to be ascribed to γ_i for use in the Country may be found in the National Annex.

4.2 Number of degrees of freedom

(1) The mathematical model should consider:

- Rocking and translation stiffness of foundations;
- An adequate number of masses and degrees of freedom to determine the response of any significant structural element, equipment, and appendages;
- The mass and stiffness of cables and guys;
- The relative displacement among supports of equipment or machinery (for a chimney, the interaction between internal and external tubes);
- Significant effects such as piping interactions, externally applied structural restraints, hydrodynamics loads (both mass and stiffness effects);

(2) The torsion stiffness of the foundation shall be included if significant.

(3) For electric transmission towers, unless a complete dynamic model is made for a representative portion of the entire line, a group of at least three towers should be modelled, so that an acceptable evaluation of the cable mass and stiffness can be accounted for the central tower.

4.3 Masses

(1) P The model shall include a discretization of masses so that a suitable representation of the inertia effects is ensured. As appropriate, translation and/or rotational mass shall be considered.

(2) P The masses shall include all permanent constructions, fittings, insulation, dust loads, clinging ash, present and future coatings, liners and the effect of fluids or moisture on density of liners, if relevant, and equipment. Permanent masses of structures and quasi-permanent equipment masses shall be considered.

(3) Applicable ψ_{2i} values are given in EN1990.

(4) Unless the client or the competent authority requires other values, a characteristic imposed load on platforms equal to 2,0 kN/m is suggested, to account for maintenance and temporary equipment.

(5) For towers and masts in cold regions a proportion of ice load should be included.

Note: The National Annex will define the proportion of ice load in cold regions.

(6) P If cables are present, a correct representation of the relevant masses shall be included in the model.

(7) When the mass of the cable is significant in relation to that of the tower, the cable should be represented as chain of elements connecting lumped masses.

Note: Idealising a cable as a single spring does not allow for its inertia in the dynamic response.

(8) P The total effective mass of the immersed part of intake towers shall be assumed equal to the sum of:

- the actual mass of the tower shaft (without allowance for buoyancy),
- the mass of the water possibly enclosed within the tower (hollow towers),
- the added mass of externally entrained water.

(9) In absence of rigorous analysis, the added mass of entrained water may be estimated according to Annex F of EN 1998-2.

4.4 Stiffness

(1) In concrete structures, if the analysis is made on the basis of a suitable q factor greater than 1, the section properties should be evaluated taking into account the effect of cracking, and yielding of the reinforcement. One such procedure is given in 4.3.1 of EN 1998-1. If q =1, and the analysis is based on the elastic response spectrum or a corresponding time-history of the ground motion, the element stiffness should take into account the cracked cross-section properties in agreement with the expected level of stress.

(2) Due regards should be given to the temperature effect on the stiffness and strength of the steel in steel chimneys structures, and those of concrete in concrete R/C chimneys structures.

(3) In case cables are integral part of a structure, a careful modelling of their stiffness should be done.

(4) If the sag of the cable is significant, the spring value should account for it. An iterative solution may be generally required. It can be based on the use of the following equivalent modulus of elasticity:

$$E_{eq} = \frac{\mathrm{E_c}}{1 + \frac{(\gamma l)^2}{12\sigma^3} E_c} \qquad (4.4.1)$$

where

E_{eq} = equivalent modulus of elasticity,

γ = specific weight per unit volume of the cable,

σ = tensile stress in the cable.

l = cable length.

$\mathrm{E_c}$ = modulus of elasticity of cable material.

(5) For wrapped up ropes, E_C is generally lower than the single cord modulus of elasticity E. An applicable reduction is

$$\frac{E_c}{E} = \cos^3 \beta \qquad (4.4.2)$$

where β is the wrapping angle of the single cord.

(6) In cases where the sag of the cable is meaningful, the likelihood of impulsive loading between tower and the cable ends should be analysed.

(7) If the preload of the cable is such that the sag is meaningless, or if the tower is short, (less than 40 meters), then the presence of the cable can be represented in the dynamic model by a linear spring.

4.5 Damping

(1) If the analysis is performed without resorting to q > 1,5, it is allowed to consider damping values different from 5%. In this case, the damping ratio of each mode of vibration may be defined according to Annex B and the corresponding elastic spectral ordinates as prescribed in 3.2.2.2 (3) of EN 1998-1.

4.6 Soil-structure interaction

(1) The design earthquake motion is defined at the soil surface, in free-field conditions, i.e. where the inertial forces due to the presence of structures do not affect it. When the structure is founded on soil deposits or soft media, the resulting motion at the base of the structure will differ from that at the same elevation in the free field, due to the soil deformability. Annex C provides suitable rules to account for soil compliance during earthquakes.

(2) For tall structures, (the height being over two times the maximum base dimension), the rocking compliance of the soil is important and may significantly increase the second order effects.

(3) In general, tall structures may be sensitive to a spatially varying vertical excitation: a vertical ground motion propagating in any horizontal direction is expected to cause rocking of the structure, concurrent with the rocking caused by the horizontal excitation along that direction.

(4) Tall structures, in regions of high seismic activity, should be analysed with proper consideration to a spatial model of the seismic excitation.

Note: Suggested conditions for this analysis are 80 m for the height, and $a_g > 0{,}25$ for the seismic activity. A National annex may define these parameters.

4.7 Methods of analysis

4.7.1 Applicable methods

(1) The standard method of analysis is the linear analysis using the q-reduced design spectrum either as a simplified dynamic analysis or a multimodal analysis.

(2) Non-linear methods of analysis may be applied, provided that they are properly substantiated with respect to the seismic input, the costitutive model, the method of interpreting the results of the analysis, and the requirements to be met, see 4.3.3.1 of EN 1998-1.

(3) For regular structures, the method set forth in the literature based on the "rigid diaphragm" assumption can be applied. For steel masts and towers, a horizontal bracing system, capable of providing the required rigid diaphragm action, should be present. In the absence of it, a three dimensional dynamic analysis capable of identifying the distortion in the horizontal plane is suitable.

(4) P For steel chimneys, horizontal-stiffening rings shall be present in the design, for the "rigid diaphragm" assumption being applicable. Otherwise, a suitable dynamic analysis, capable of identifying hoop stresses, is required.

(5) For reinforced concrete towers and chimneys, hoop reinforcement should take into account ovalling of the horizontal cross section due to lateral forces. A dynamic analysis capable of identifying hoop stresses may be suitable.

4.7.2 Simplified dynamic analysis

4.7.2.1 General

(1) This type of analysis is applicable to regular structures that can be represented by two planar models and whose response is not significantly affected by contributions of higher modes of vibration. The "rigid diaphragm" assumption should be appropriate.

(2) Piping and equipment supported at different points should be analysed taking into account the relative motion between supports. This motion may be larger that that conceived by the simplified analysis.

(3) It is suggested to rely on simplified dynamic analysis only if the importance factor is $\gamma_i \leq$ 1,2, and the height is H < 80 m.

Note: The National Annex will define the height below which simplified dynamic analysis is allowed, and the relevant importance factor.

4.7.2.2 Seismic forces

(1) The effects induced by the seismic action are determined by subdividing the structure into n distinct concentrated masses, including the masses of the foundations, to which the horizontal forces F_i, i = 1, 2.....n, are applied, given by the expression:

$$F_i = \frac{h_i w_i}{\sum_1^n h_j w_j} F_t \qquad (4.7.1)$$

where

$$F_t = S_d(T) \sum_1^n w_j \qquad (4.7.2)$$

w_i weight of the i-th mass including permanent load and variable loads multiplied by the pertinent combination factor specified in clause 4.3;

h_i is the elevation of the i-th mass from the level of application of the seismic excitation;

S_d(T) is the ordinate of the design spectrum as defined in EN 1998-1, for the fundamental period of vibration T. In case the period T is not evaluated through a valuable structural model, the spectral value S_d (T_c) should be accounted for.

Note: The above method may provide a substantial overevaluation of the seismic action in the case of tapered towers where the mass distribution substantially decreases with elevation.

4.7.3 Modal analysis

4.7.3.1 General

(1) This method of analysis can be applied to any structure, with an input motion defined by a response spectrum or by the corresponding time history.

4.7.3.2 Number of modes

(1) A practical rule to establish the sufficient number of modes is the following. For each mode i, and for each direction of the excitation, the "equivalent modal mass" M_i is evaluated. Then, for each direction, the sum of M_i is performed and is compared to the total mass of the structure M. If

$$\sum_{i}^{N} M_i \geq 0{,}9 \text{ M} \qquad (4.7.3)$$

then the considered number of modes is adequate. An exception to the above rule may occur in case when light equipment or a light structural appendix is concerned. Appendix D provides hints for the practical application of Eq. (4.7.3)

Note: For a continuously distributed mass structure, cantilevering from the soil, the minimum number of modes, necessary to assure participation of all significant modes, is higher than the number suitable for a "shear type" building, with lumped masses.

The minimum number of modes which is necessary to evaluate internal actions at the top of the structure is generally higher than that which is sufficient for evaluating the overturning moment or the total shear at the base of the structure.

4.7.3.3 Combination of modes

(1) P For each quantity, (force, displacement, stress), the maximum value S of the earthquake effect in general shall be obtained as the square root of the sum of the squares of the contributions of individual modes, (SRSS combination):

$$S = \pm\sqrt{(s_1^{\,2} + s_2^{\,2} + s_3^{\,2} + \ldots)} \qquad (4.7.4)$$

where s_1, s_2, s_3 ... are the contributions to the selected quantity of modes 1, 2,3... This action effect assumes both the positive and the negative sign.

(2) P For any one direction of the seismic excitation, when two significant modes i and j indicate closely spaced periods, with the ratio T_j/T_i exceeding 0,9 with $T_j < T_i$, the above rules becomes unconservative and more accurate rules must be applied.

4.7.3.4 Combination of internal actions

(1) P When combining internal actions, each internal action is to be computed according to rule 4.7.3.4. All physically possible combinations shall be considered.

4-8 Combinations of the seismic action with other actions

(1) Values of coefficient ψ_{2i} are given in EN1990, and values of ψ_{Ei} are given in Part 1, 4.2.4. EN 1998-1. Lacking a precise information, for the present structures $\psi_{Ei} = \psi_{2i}$ should be assumed.

4.9 Displacements

(1) P The displacements induced by the design seismic action shall be calculated on the basis of the elastic deformation of the structural system by means of the following simplified expression:

$$d_s = q_d\, d_e\, \gamma_i \qquad (4.9.1)$$

where

d_s displacement of a point of the structural system induced by the design seismic action,
q_d displacement behaviour factor. Lacking a more precise calculation, q_d can be assumed equal to q,
d_e displacement of the same point of the structural system, as determined by a linear analysis based on the design response spectrum.
γ_i importance factor.

4.10 Safety verifications

4.10.1 Ultimate limit state

(1) P The safety against collapse (ultimate limit state) under the seismic design situation is considered to be ensured if the conditions regarding resistance, ductility and stability are met.

4.10.2 **Resistance capacity of the structural elements**

(1) P In general, the following relation must be satisfied for all structural elements

$$R_d \geq E_d\ (\gamma_i\, E, G, P, \ldots\ldots\ldots\ldots\ldots) \qquad (4.10.1)$$

where

R_d design resistance capacity of the element, calculated according to the mechanical models and the rules specific to the material, (characteristic value of property f_k, and partial safety factor γ_m).

E_d design value of the effect in the combination of actions including, if necessary, P-Δ effects and thermal effects.

(3)P For steel structures, the members in axial tension shall be checked for plastic resistance of gross cross-section, and ultimate resistance of net cross-section, according to 6.2.3 of EN 1993-1-1.

4.10.3 Second order effects

(1) P Second order, P - Δ, effects shall be evaluated considering the displacements computed as indicated in Section 4.7, unless the condition (2) is respected.

(2) Second order, P-Δ, effects need not be considered when the following condition is fulfilled.

$$\delta M / M_o < 0{,}10 \qquad (4.10.2)$$

where

δM overturning moment due to P-Δ effect

M_o first-order overturning moment

4.10.4 Connections

(1) P For weld or bolted non dissipative connections, resistance shall be according to EN 1993-1-1;

(2) P For weld or bolted dissipative connections, resistance shall be greater than the plastic resistance of the connected dissipative member based on the design yield stress of material as defined in EN 1993-1-1, taking into account the overstrength factor (see 6.1.3(2) and 6.2 of EN1998-1)

(3) For requirements and properties for bolts and welding consumables, see EN 1993-1-8.
(4) For requirements and properties of ropes, strands, wires and fittings see EN 1993-1-11.
(5) Non dissipative connections of dissipative members made by means of full penetration butt welds are deemed to satisfy the overstrength criterion.

4.10.5 Guys and fittings

(1) For requirements and properties of ropes, strands, wires and fittings see EN 1993-1-11.

4.11 Thermal effects

(1) If the operating temperature of structural elements is above 100 ^{o}C, then the thermal effects on the mechanical properties of the structural element such as elastic modulus, yield stress and thermal expansion coefficient should be taken into account.

4.12 Ductility condition

(1) P It shall be verified that the structural elements and the structure as a whole possess adequate ductility to its expected exploitation, which depends on the selected system and the adopted behaviour factor.

(2) P Toughness requirements for the steels shall be determined according to EN 1993-1-10.

(3) Application rules are given in the Specific rules of sections 5, 6, 7 and 8, for each different category of structures.

4.13 Stability

(1) The stability of the structure shall be verified under the set of forces induced by the combination rules including piping interaction and hydrodynamic loads, if present.

(2) Special criteria of stability verification are reported for steel chimneys and steel towers and masts in EN 1993-7.

(3)P The design buckling resistance of a compression member in a lattice tower or mast shall be taken as:

$N_R = \chi \, \eta_j \, A \, f_y / \gamma_{M1}$

where: η_j = 0,8 for single angle members connected by one bolt at each end;
= 0,9 for single angle members connected by one bolt at one end and continuous or rigidly connected at the other end;
= 1,0 for all other cases;
χ is the reduction factor for the relevant buckling mode, function of the non-dimensional slenderness parameter ë, as given in EN1993-1.1.
A is the gross cross-sectional area.
f_y is yield resistance of member:
γ_{M1} = 1,10.

(4) P For sections type 4, see 5.5 1993-7.

(5) The slenderness for leg members should generally be not more than 120.
(6) The slenderness λ for primary bracing members should generally be not more than 180 and for secondary bracing not more than 250.

4.14 Serviceability limit state

(1) The damage limitation requirement establishes limits to displacements under earthquake excitations. Sections 5, 6, 7 and 8 provide limits depending on structures.

(2) P Deflections for the serviceability limit state shall be calculated by reducing the displacements given in the expression (4.9.1), according to the factor ν, defined in (2).

(3) The reduction factor ν takes into account the shorter return period of the seismic action associated with the damage limitation requirement. Suggested values are: ν = 0,4 for structures to which $\gamma_i > 1$ is assigned, and ν = 0,5 for other structures.

Note: values of the reduction factor ν that takes into account the shorter return period of the seismic action associated with the damage limitation requirement may be defined in the National Annex. Different values for towers and masts may be prescribed, depending on the relevant scope.

(4) If certain use of the structure is significantly affected by deflections, (for example in telecommunication towers peak transient deflections might lead to permanent damage), the deflection should be limited to appropriate values.

4.15 Behaviour factor

4.15.1 General

(1) P The behaviour factor q is given by the product:

$$q = q_o \, k_r \geq 1{,}5, \qquad (4.15.1)$$

where:

q_o basic behaviour factor, reflecting the ductility of the lateral load resisting system, with values defined in sections 5, 6, 7 and 8 for each different structure.

k_r modification factor reflecting departure from a regular distribution of mass, stiffness or strength, with values defined in 4.15.2.

4.15.2 Values of factor k_r

(1) P The value of k_r shall be taken as follows, depending of the existence of the following irregularities on the structure.

a) Horizontal mass eccentricity at a section exceeding 5% of the relevant structure dimension $k_r = 0{,}8$

b) Opening in shaft causing a 30% or larger reduction of the moment of inertia of the cross section $k_r = 0{,}8$

c) Concentrated mass within the top third elevation contributing 50% or more to the base overturning moment, $k_r = 0{,}7$

(2) P When more than one irregularity is present, k_r shall be assumed equal to the product of 0,9 by the lowest values of k_r.

5 SPECIFIC RULES FOR REINFORCED CONCRETE CHIMNEYS

5.1 Basic Behaviour factor

(1) P Critical region shall be considered the entire shell up to a distance d (where d denotes the outer diameter of the shell) above the bottom cross-section, and above sections where an abrupt change of thickness is made. Critical region shall also be considered the concrete wall where more than one opening exist, and up to a distance d above and below those openings.

(2)If a local curvature ductility of the critical sections of at least $\mu_{1/r} = 9$ is secured, by providing confining reinforcement, an applicable value is: $q_o = 3$
In all other cases: $q_o = 1,5$

Note: The design of chimneys is generally governed by wind considerations, with the exception of locations with medium to high seismicity, chimneys with large elevated masses, and chimneys with unusual geometry.

5.2 Materials

(1) All materials and material tests should conform to Eurocode 2.

(2) The same brand and type of cement should be used throughout the construction of the chimney wall. The maximum size of coarse aggregate should not be larger than 1/8 of the narrower dimension between forms nor larger than 1/2 the minimum clear distance between parallel reinforcing bars.

(3) The specified concrete should be of a class not lower than C20/25 as defined in Eurocode 2.

5.3 General

5.3.1 Minimum reinforcement (vertical and horizontal)

(1) P For a chimney with an outside diameter of 4 m or more, the minimum ratio of the vertical reinforcement to the gross-sectional area shall be not less than 0,003. The reinforcement shall be distributed in layers towards the inner and the outer face, with not less than half the reinforcement in the layers towards the outer face.

(2) P Close to the chimney top, where stresses due to permanent load are rather small, a minimum vertical reinforcement equivalent to that for the horizontal direction must be provided.

(3) P A chimney with an outside diameter of 4 m or more must be provided with layers of horizontal reinforcement in the proximity of both surfaces and the ratio to the gross-sectional area shall not be less than 0,0025.

(4) P In chimneys with an outside diameter of less than 4 m, the inner layer of reinforcement may be omitted, but in that case the ratio of outer layer reinforcement to gross-sectional area shall not be less than 0,002 per direction.

(5) Circumferential bars should be placed around the exterior of, and secured to the vertical bars. All reinforcing bars should be tied at intervals of not more than 60 cm.

(6)Particular attention should be paid to placing and securing the circumferential reinforcing so that it cannot bulge or be displaced during the pouring and working of the concrete, so as to result in less than the required concrete cover of the circumferential reinforcement. Circumferential bars should be closed preferably by welding. No closure by splicing should be permitted.

5.3.2 Distance between reinforcement bars.

(1) The distance between vertical bars should be not more than 250 mm and the distance between horizontal bars should not exceed 200 mm.

5.3.3 Minimum reinforcement around openings

(1) In addition to the reinforcement determined by the stability and temperature, extra reinforcement should be provided at the sides, bottom, top and corners of openings as hereinafter specified. This extra reinforcement should be placed near the outside surface of the chimney shell as close to the opening as proper spacing of bars will permit. Unless otherwise specified, all extra reinforcement should extend past the opening a sufficient distance to develop the bars in bond.

(2) The minimum vertical reinforcement ratio should be 0,0075, in a distance of half the width of the opening. Both sides of the opening should be reinforced.

5.3.4 Minimum cover to the reinforcement

(1) The concrete cover to the circumferential reinforcement should be 30 mm minimum with a tolerance of + 20 mm and - 10 mm.

5.3.5 Reinforcement splicing

(1) Not more than 50 % of the bars should be spliced along any plane unless specifically permitted on drawings or approved by the responsible engineering.

5.3.6 Concrete placement

(1) In the concrete shell no vertical construction joint should be used. Horizontal construction joint for jump form construction should be maintained at approximately uniform spacing throughout the height of the chimney.

(2) Concrete should be deposited in approximately level layers no greater than 40 cm deep. Particular care should be exercised when placing concrete in thin wall sections where two layers of reinforcing are present.

5.3.7 Construction tolerances

(1) The vertical alignment of centerpoint should not vary along the vertical axis by more than 1/1000 of the height of the shell at the time of measurement, or 2 cm, whichever is greater.

(2) The measured outside shell diameter at any section should not vary from the specified diameter by more than 1/100 of the specified or theoretical diameter.

(3) The measured wall thickness should not vary from the specified or theoretical wall thickness by more than -1 +2 cm. A single wall thickness is defined as the average of at least four measurements taken over a 60-degree arc.

(4) Tolerances on the size and location of opening and embedments should be established depending on the nature of their use. Lacking further requirements, tolerances for opening and embedment sizes and locations are 1/100 of the shell outside diameter.

5.4 Design loads

5.4.1 Construction loading.

(1)In the design of a chimney for horizontal earthquake forces, only one horizontal direction need be considered. Unlike building structures, chimneys are generally axisymmetric, and the effects on an orthogonal plane from horizontal earthquake acting along one direction are negligible. However some attention should be given to asymmetric opening.

(2)The effect of the vertical component of the earthquake on the chimney is generally of no design significance, and can be disregarded.

(3)In cases the lining (brick, steel, or other materials) is laterally supported by the chimney shell at discrete points close one to the other (so that a meaningful relative movement is not expected during a seismic shaking), the lining can be taken into account by incorporating its mass into that of the shell.

(4)For cases in which the chimney lining is supported at the top of the chimney shell or at intermediate points distant one to the other so that a meaningful relative movement is expected, a dynamic analysis including both concrete shell and liner should be used.

(5)When using the elastic response spectrum, appropriate damping values should be used for the liner depending on its construction (e.g., 1,5 per-cent for steel liners, 4,0 percent for brick liners, and 2,0 per-cent for fiber reinforced plastic liners).

(6)Consideration should be given to the construction loading, during the construction phase. In particular, if required during construction, temporary access openings may be provided in the construction shell. However, for the design of the shell, these openings should be designed as permanent openings.

5.5 Serviceability limit states

(1) P Waste gas flues in chimneys shall be checked for imposed deformations between support points, and imposed clearances between internal elements, so that gas tightness is not lost and sufficient reserve in maintained against the flue gas tube collapse.

(2) The requirement for limiting damage is considered satisfied if the maximum lateral deflection of the top of the structure, prior to the application of load factors, does not exceed the limits set forth by the following equation:

$$d_{max}\,\nu = 0{,}005 \times H \qquad (5.1)$$

where

d_{max} is the lateral deflection at the top of the chimney,

H is the height of the structure,

ν the reduction factor to take into account the lower return period of the seismic action associated with the damage limitation requirement defined in 4.14.

(3)The relative deflection between shell and lining as well as the deflection of the supporting platform should be limited to ensure the serviceability of the lining. Unless otherwise specified by the Owner, the following limits on the relative deflection of adjacent supporting platforms shall be observed:

a) if provisions are taken to allow relative movement between separate parts of liner, such as if the liner is made by tubes independent one from the other, with suitable clearance,

$$d_r \leq 0,02\ \Delta H \qquad (5.2)$$

b) In other case,

$$d_r \leq 0,012\ \Delta H \qquad (5.3)$$

where ΔH is the distance along the vertical axis, between liner supporting platforms.

(3) The deflection limit can be compared against the deflection calculated using uncracked concrete sections and a fixed base.

Note: Limiting deflections also serves to reduce the effects of secondary bending moments.

5.6 Ultimate limit state

(1) In the calculation of limit-state bending moments, allowance needs to be made for the moment caused by the weight of the chimney in its deflected shape.

6 SPECIAL RULES FOR STEEL CHIMNEYS

6.1 Basic behaviour factor

(1) Steel frames or trusses structures lateral supporting flue gas ducts of chimneys:

a) Structures designed for dissipative behaviour according to the specific rules for steel buildings of 6 of EN 1998-1:

Moment resisting frames or with eccentric bracing's $q_o = 5$
Frames with concentric diagonal bracing's $q_o = 4$
Frames with V-bracing's, see also figure 1, $q_o = 2$

b) Structures not designed for dissipative behaviour, see also figure 1
$q_o = 1,5$

(2) Steel shell-type structures:

a) Structures with cross sections satisfying the requirements of 5.3.3 EN 1993-1-1 for plastic global analysis $q_o = 2,5$

b) all other structures $q_o = 1,5$

Note: Guyed steel stacks and chimneys are generally lightweight. As such the design loads due to natural hazards are generally governed by wind. On occasion, large flares or other elevated masses located near the top may require an in-depth seismic analysis.

6.2 Materials

6.2.1 General

(1) P The mechanical properties and the chemical composition of structural steel shall comply with the European Standards requirements, in the series EN 10000.

(2) As a result of the qualification tests for materials, a tensile strength up to 20 N/mm^2 less than the prescribed value is allowable for all steels. The upper limit of the tensile strength may be exceeded by:

20 N/mm^2 for all steels of class C, D and DD;

30 N/mm^2 for all flat products of a thickness less than 3 mm, made from steel of class A, B, C, D and DD.

(3) All qualified steels can be used, provided that the minimum notch toughness of 28 joules is respected, and the minimum elongation of 3 %, on a standard specimen, with a gauge length L = 5 D, is granted.

(4) The use of special steel which does not respect the above limit, is dissuaded, unless it can be demonstrated that the thickness required for earthquake loading is conveniently less than the provided thickness.

(5) Where stainless steel or alloy steel components are connected to carbon steel, bolted connections are preferred. In order to avoid accelerated corrosion due to galvanic action, such connections should include insulating gaskets. Welded procedure is permitted, provided specialised metallurgical control is exercised with regard to welding procedure, and electrode selection.

6.3.2 Mechanical properties for structural carbon steels

(1)P The mechanical properties of structural carbon steels S 235, S 275, S 355, S 420, S 460 according to EN 10025 or EN 10113 or EN 10137 shall be taken from EN 1993-1-1. For properties at higher temperatures see prEN 13084-7.
(2) The most frequent qualification grades are B and C. In severe environmental conditions, mainly in case of low temperature, grade D should be used.

6.2.3 Mechanical properties for weathering steels

(1) The mechanical properties for weathering steels according to EN 10155 should be determined as for structural steels S 235, S 275 or S 355 using the guaranteed values specified therein. For properties at higher temperatures see prEN 13084-7.

6.2.4 Mechanical properties of stainless steels

(1) Mechanical properties related to stainless steels should be taken from EN 1993-1-4 valid for temperature up to 400°C. For properties at higher temperatures see EN 10088 and prEN 13084-7.

6.2.5 Connections

(1) For connection material, welding consumables, etc., reference should be made to the relevant product standards specified in EN 1993-1-1.

6.3 Design loads

(1) The permanent load should include the weight of all permanent constructions, fittings, linings, flues, insulation, present and future loading, including corrosion allowances. For process plants in which a carry over of ash or dust burden is present, which can adhere to the interior surface of the structural shell or liner, an additional dead load should be added to the permanent load.

(2) The weight of the chimney and its lining should take into account long-term effects of fluids or moisture on the density of linings if relevant.

6.4 Serviceability limit state

(1) P Waste gas flues in chimneys shall be checked for imposed deformations between support points, and imposed clearances between internal elements, so that gas tightness is not lost and sufficient reserve in maintained against the flue gas tube collapse.

(2) The requirement for limiting damage is considered satisfied if the maximum lateral deflection of the top of the structure, prior to the application of load factors, does not exceed the limits set forth by the following equation:

$$d_{max}\,\nu = 0{,}005 \times H \qquad (6.1)$$

where

d_{max} is the lateral deflection at the top of the chimney,

H is the height of the structure,

ν the reduction factor to take into account the lower return period of the seismic action associated with the damage limitation requirement defined in 4.14.

6.5 Ultimate limit state

(1) The use of the present procedure, combined with the partial safety factors will ensure that low cycle fatigue will not contribute to the failure of the chimney.

(2) In the design of details such as flanges, ultimate limit state may take into account of plastic stress distribution.

(3) At the time of construction, in the stress verifications, the minimum thickness allowance for corrosion is 2 mm, unless special care is exercised to minimise corrosion. See also EN1993-3-2.

(4) Weakening of cross-section components by cut-outs and openings (manholes, flue inlet) shall be compensated for by adequately sized reinforcement, taking into account local stability of shell. Stiffeners may be required around edges, see 5.3 EN1993-7.

7 SPECIAL RULES FOR TOWERS

7.1 General and basic behaviour factor

(1) Basic behaviour factors are defined according to the most appropriate identification of the structural arrangement with respect to those represented in figure 1.
(2) For structures that cannot be identified among those in Fig. 1, guidance for the selection of the most appropriate q_o factor should be searched by examining the general concepts provided by Chapter 6 EN1998-1.

Note: The behaviour factor values shown in Figure 1 reflect the inelastic reserve strength of the structural systems during an earthquake event. The values presented for these types of structures were determined based on a review of published values established for building structures and nonbuilding structures. In general, the q_o values shown reflect the earthquake performance of these structural systems and engineering judgement. Other values may be appropriate if determined using sound engineering data.

(3) P Identification of cross sections class is given in 5.5 EN1993-1.

(4) P Depending on the chosen cross sections, the behaviour factor is limited to the values quoted in Table 1.

Behaviour factor q	Cross sectional class
1,5	4
$1,5 < q \leq 2,5$	3
$2,5 < q \leq 4$	2
$4 < q$	1

Table 1: behaviour factor q and cross sectional class.

Note: The design of electrical transmission, substation wire supports, and distribution structures are typically controlled by high wind, ice-wind combinations, and unbalance longitudinal wire loads. Seismic loads generally do not control their design. Earthquake performance of these structures has demonstrated that seismic loads can be resisted based on traditional electrical transmission, substation, and distribution wire support structure loading. However heavy equipment, such as transformer in distribution structures, may result in significant seismic load.

Besides, earthquake-related damage to electrical transmission, substation wire support, and distribution structures is typically caused by large displacements of the foundations due to landslides, ground failure, and liquefaction. These situations have resulted in structural failure or damaged structural members without complete loss of structure function.

The fundamental frequency of these structure types typically ranges from 0.5 to 6 Hz. Single pole type structures have fundamental mode frequencies in the 0.5 to 1.5 Hz range. H-frame structures have fundamental mode frequencies in the 1 to 3 Hz ranges, with the lower frequencies in the direction normal to the plane of the structure and the higher frequencies in plane. Four legged lattice structures have fundamental mode frequencies in the range of 2 to 6 Hz. Lattice tangent structures typically have lower frequencies with the higher frequencies being representative of angle and dead end structures. These frequency ranges can be used to determine if earthquake loading should be a design consideration. If it is determined that earthquake loads are significant then a more detailed evaluation of the structure vibration frequencies and mode shapes should be performed. This can be accomplished using available commercial finite element computer programs.

(5) For the use of the elastic response spectrum, the default viscous damping value to be used in such an analysis should be 2 percent. A higher damping value can be used if determined using sound engineering data.

(6) A minimum importance factor $\gamma_I = 1,0$ is required to minimise the loss of function after an earthquake event, even though these systems are normally redundant.

7.2 Materials

(1) Welding and bolts should conform to the requirements prescribed in clause 3 of ENV 1993-1-1

(2) When hot rolled angles are used for lattice towers, the mechanical properties and the composition of the steel should comply with EN 10025 or other equivalent standards.

(3) Hot rolled angles in high tensile steel should comply with Euronorm 10049. Low alloy, cold formed steel, are acceptable. When high strength, their deformability should comply to EN 10049.

(4) Thickness of cold-formed members for towers should be at least 3 mm.

(5) In bolted connections preferably high strength bolts in category 8.8, or 10.9 should be used. Bolts of category 12.9 are allowed in shear connections, but are not recommended in general.

(6) The value of the yield strength $f_{y\ act}$ which cannot be exceeded by the actual material used in the fabrication of the structure should be specified and noted on the drawings; $f_{y\ act}$ should not be more than 10% higher than the design yield stress f_{yd} used in the design of dissipative zones.

Note: Steel towers are normally designed to be in service, without any maintenance, for 30-40 years or more. Weathering steel is thus used, unless protection against corrosion is applied, like hot dip galvanising.

7.3 Design loads

(1) In relation to the regional climate, ice loads may be included among the design loads, both on the structure and on the conductors, when they are present. In this case the loading combination for earthquake includes the ice loading with a factor for ice equal to 1.

7.4 Structural types

(1) Some typical configurations are reported in figure 1. All of them pertain to the category of frames with concentric bracing, in which members subjected to axial forces mainly resist the horizontal forces.

The bracing may belong to one of the following categories:

- Active tension diagonal bracing, in which the horizontal forces can be resisted by the tension diagonals only, neglecting the compression diagonals. Dissipative zones may be located in the tensile diagonals.

- V-bracing, in which the horizontal forces can be resisted by considering both tension and compression diagonals. The intersection point of these diagonals lies on a horizontal member, which must be continuous. Mechanism of dissipation in this configuration is not dependable.

- K- bracing, in which the diagonal intersection lies on a column. This last configuration is not recommended.

(2) For structures intitled *of* $q \geq 3,5$, a horizontal bracing system should be applied according to fig. 2.

Note: In the terminology of EN1993-6-1, bracing of Fig. 2 is mentioned as *fully triangulated*. See that section for a proper identification.

7.5 Electric transmission towers

(1) In the present section a minimum requirement for accounting the effects of cables between tower and tower, is assessed.

(2) The structure should be analysed under the effect of two concurrent sets of seismic loads:

- A set of horizontal forces at the top of the tower, provided by the cables under the assumption that each tower moves statically with respect to the adjacent towers, in the most adverse direction. The assumed displacement should be equal to twice the maximum ground displacement specified in clause 3.2.2.4 of Part 1. A set of all physically possible relative displacements between towers should be analysed, under the assumption of properly fixed towers at their base.

- Inertia loads resulting from a dynamic analysis. Unless a dynamic model is made for a representative portion of the entire line, a group of at least three towers should be modelled, so that an acceptable evaluation of the cable mass and stiffness can be accounted for the central tower. In the three towers model, a limiting assumption may be made for the two adjacent towers, if tangent towers. In this case, inertia loads can be computed assuming the adjacent tower as elastically supported at the cable elevation along the direction of the cables.

7.6 Serviceability limit state

(1) Section 4,14 applies.

Note: Unlike other structures, for steel transmission towers serviceability limit state for deflection is not critical. Steel towers can tolerate relatively large elastic and residual displacements.

7.7 Rules of practice

(1)Trussed tubes, involving major diagonals, suffer from inadequate ductility, and therefore are generally not recommended under severe earthquake conditions. Behaviour factor not higher than 2 should be adopted.

Note: The National Annex may define the behaviour factor of towers made by trussed tubes.

(2) For tubular steel towers, a particular care should be devoted to joints. "Telescope joints" can be used only if experimentally qualified.

(3) When tension is likely to occur at the base of the columns, the corresponding anchorage to foundation should be able to transmit the full tension evaluated under the assumption of a behaviour factor suitably reduced. A suggested value is $q \leq 2$.

Note: The National Annex is expected to prescribe a suitable value for this circumstance.

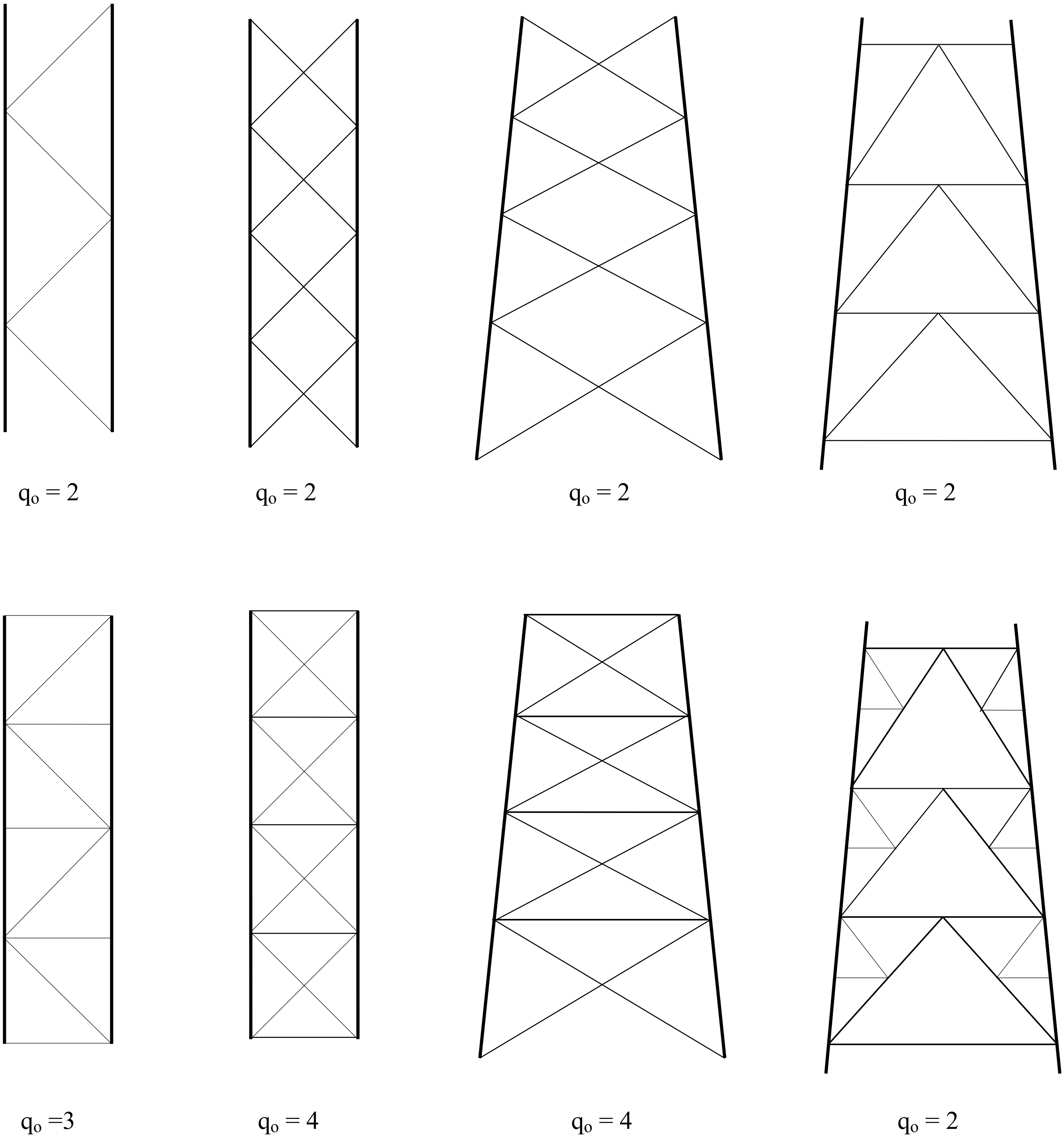
(4) Further critical items in relation to the seismic loading are:

- angles under alternate compression and tension;

- bolted connections, especially single bolt connections;

For these items, guidance should be got from EN1993-7.

(4) Connections should be qualified to withstand a suitable number of cycles of alternate actions, up to their design intensity, without deteriorating the stiffness.



1: Basic behaviour

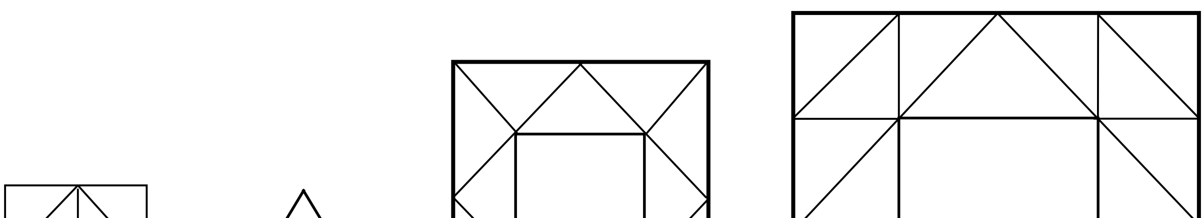


Fig. 2: Typical horizontal bracing according to 6 EN1993-7.

8 SPECIAL RULES FOR MASTS

8.1 Basic behaviour factor

(1) Basic behaviour factors are defined according to the most appropriate identification of the structural arrangement with respect to those represented in figure 1.

(2) For structures that cannot be identified among those in Fig. 1, guidance for the selection of the most appropriate q factor should be searched by examining the general concepts provided by 6 EN1998-1.

(3) P Identification of cross sections class is given in 5.5 EN1993-1.

(4) P Depending on the chosen cross sections, the behaviour factor is limited to the values quoted in Table 1.

Behaviour factor q	Cross sectional class
1,5	4
$1,5 < q \leq 2,5$	3
$2,5 < q \leq 4$	2
$4 < q$	1

Table 1: behaviour factor q and cross sectional class.

(5) For structures intitled *of* $q \geq 3,5$, a horizontal bracing system should be applied according to fig. 2.

8.2 Materials

(1) Masts are generally built from laminated open profiles or tubes. Steels normally used are S235, S275, and S355. The most frequent qualification grade is B. When welding is envisaged, grade C is generally mandatory. However, in very severe environmental conditions, mainly in case of very low temperatures, grade D should be used.

Note: Standards do not put obstacles to the evolution process of production and usage of other steel types with enhanced properties. However under severe cycles of load reversal, the use of high strength steel should be dissuaded, unless appropriate experimental evidence is provided both on members and on connections.

(2) Hot rolled sections, mainly angles are the most widely used. They can be connected with bolts or welding. Tubes are also used, because of their advantage mainly relating to triangular towers and masts. Warnings are given in 7.7.

8.3 Serviceability limit state

(1) The requirement for limiting damage is considered satisfied if the maximum lateral deflection of the top of the structure, prior to the application of load factors, does not exceed the limits set forth by the following equation:

$$d_{max}\, \nu = 0,005 \, . \, H \qquad (8.1)$$

where

d_{max} is the lateral deflection at the top of the mast,

H is the height of the mast,

ν the reduction factor to take into account the lower return period of the seismic action associated with the damage limitation requirement defined in 4.14.

(2) A limiting drift ratio between horizontal stiffening elements should be allocated, depending on the mast exercise.

8.4 Guyed masts

(1) A guyed mast (or a guyed tower) is essentially a slender column that is either fixed or hinged at the base and elastically restrained by the cables.

(2) As to the stiffness of the elastic restraint provided by the cables to the tower, they can be subdivided into two broad categories:

- Relatively short towers, (in the neighbourhood of 30÷40m), for which the cables are usually assumed as straight beams;
- Tall towers, for which the sag of the cables is large and should be accounted for.

(3) The main difference between the two cases is that the stiffness of a straight cable remains constant as the tower bends, whereas the stiffness of a sagging cable varies with tower deformations, (see clause 4.4).

(4) Cable icing is likely to induce significant sagging, even in relatively short cables (icing loads are often in major importance in region of severe winter conditions, and may be of long duration).

(5) For both sagging and straight cables, the horizontal component of the cable stiffness is

$$\cos^2(\alpha)\,\frac{A_c\,E_c}{l} \qquad (8.2)$$

in which A_C is the cross section area of the cable, E_C is the effective modulus of elasticity of the cable, (accounting for the sag, if the case), l is the length and α is the angle of the cable with respect to the horizontal axis. In cases in which the sag of the cable is large, the spring value should account for it. In this case the likelihood of impulsive loading both on the tower and on the cable end should be analysed.

ANNEX A (Informative) Linear dynamic analysis accounting for a rotational seismic spectrum

(1) The design ground motion during the earthquake is represented by three translation and three rotation response spectra.

(2) The translation ones are the elastic response spectra for the two horizontal components, (axis x and y), and the vertical component, (axis z), referred to in Part 1-1.

(3) The rotation response spectrum is defined in an analogous way as translation response spectrum, i.e. by consideration at a single degree of freedom oscillator, of rotational nature acted upon by the rotation motion. The natural period is denoted by T and damping with respect to the critical damping is denoted by ξ.

(4) Let R^{θ} be the ratio between the maximum moment on the oscillator spring and the rotational moment of inertia about its axis of rotation. The diagram of R^{θ} versus the natural period T, for given values of ξ, is the rotation response spectrum.

(5) Unless results of a specific investigation are available, the rotational response spectra are defined by:

$$R_x^{\theta}(T) = 1{,}7\ \pi\ S_e(T) / (v_S\ T) \qquad (A.1)$$

$$R_y^{\theta}(T) = 1{,}7\ \pi\ S_e(T) / (v_S\ T) \qquad (A.2)$$

$$R_z^{\theta}(T) = 2{,}0\ \pi\ S_e(T) / (v_S\ T) \qquad (A.3)$$

where:

R_x^{θ}, R_y^{θ} and R_z^{θ} are the rotation response spectra around axis x, y and z, in rad/sec^2;

$S_e(T)$ is the site dependent response spectra for the horizontal components, in m/sec^2;

T is the period in seconds.

v_S is the S-wave velocity, in m/sec, of the upper layer of the soil profile, or the average S-wave velocity of the first 30 m. The value corresponding to low amplitude vibrations, i.e., to shear deformations of the order of 10^{-6}, can be selected.

(6) The quantity v_S is directly evaluated by field measurements, or through the laboratory measurement of the shear modulus of elasticity G, at low strain, and the soil density ρ, being:

$$v_S = \sqrt{(G / \rho)}$$

(7) In cases v_S is not evaluated by an apposite experimental measurement, the following values are consistent with the subsoil classification:

Subsoil class	shear wave velocity v_S m/sec
A	800
B	580
C	270
D	150

(1) Consider ground acceleration $\ddot{u}(t)$ along the horizontal direction, and a rotation acceleration $\omega(t)$ in he plane u-z. If the inertia matrix is [M], the stiffness matrix [K], and

the damping matrix [C], the equations of motion for the resulting multi-degree-of-freedom system are given by:

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = -(\{m\}\,\ddot{x} + \{m\,h\}\,\ddot{\theta}) \qquad (A.4)$$

where

$\{\ddot{u}\}$ vector comprising the system's displacements relative to the base,

{m} vector comprising the translation masses in the direction of the u excitation. This vector coincides with the main diagonal of the mass [M] when the vector {X} includes only the translation displacements in u direction.

$\ddot{x}$ (t) translation ground acceleration, represented by S_e.

$\ddot{\theta}$ rotation ground acceleration, represented by R^{θ}.

(10) To account for the term {m} ü the participation factor in the modal analysis of mode k is:

$$a_{ku} = \frac{\{\Phi^T\}\{m\}}{\{\Phi^T\}[M]\{\Phi\}}$$

while, for the term $\{m\,h\}\,\ddot{\theta}$, the participation factor is:

$$a_{k\theta} = \frac{\{(\Phi\, h)^T\}\{m\}}{\{\Phi^T\}[M]\{\Phi\}}$$

where:

$\{\Phi\}$ is the k-th modal vector

$\{\Phi^T\}$ is the trasposed of the k-th modal vector

$\{\Phi\, h\}$ is the vector of the products of the modal amplitude Φ_i, at the i-th nodal point, and its elevation h_i.

(11) The effects of the two forcing functions are to be superimposed instant by instant. They are generally not in phase, and accordingly the effects of the rotational ground excitation can be combined with the effects of the translation excitation as a square root of the sum of the squares.

ANNEX B (Informative) Analysis procedure for damping

(1) When the design response spectrum is applied the behaviour factor q incorporates the elastic dissipation in the structure and that due the soil-to structure interaction and to the inelastic hysteretic behaviour of the structure. In those instances when the elastic spectrum is applied, the damping factor (or damping ratio relative to the critical damping) need be explicitly defined, and when the modal analysis is being performed, the damping factors need be defined for each mode of vibration. If a mode involves essentially a single structural material, than the damping ratio should conform to the material dissipation property and to the amplitude of deformation. Suggested ranges of values of damping ratios are:

	damping ratios		
steel elements	0,01	÷	0,04
Concrete elements	0,02	÷	0,07
Ceramic cladding	0,015	÷	0,05
Brickwork lining	0,03	÷	0,1

(2) In case evidence is brought that non-structural elements contribute to energy dissipation, higher values of damping can be assumed. Due to the dependency on the amplitude of deformation, in general lower bounds of the ratios are suitable for the serviceability analysis, while upper bounds of the ratios are suitable for the ultimate state analysis.

(3) As to the energy dissipation in the soil, representative numbers for the dashpot associated with stiffness is:

swaying soil compliance	0,10	÷	0,20
Rocking soil compliance	0,07	÷	0,15
Vertical soil compliance	0,15	÷	0,20

(4) For linear footings, consistent compliance coefficients should be applied.

(5) Low dashpot values are assigned to foundations on a shallow soil deposit, over stiff bedrock.

(6) In general, for the present structures any mode of vibration involves the deformation of more than one material. In this case, for each mode, an average modal damping based on the elastic energy of deformation stored in that mode of vibration is appropriate.

(7) The formulation leads to

$$\bar{\xi}_j = \frac{\{\phi\}^T [\overline{\mathbf{K}}] \{\phi\}}{\{\phi\}^T [\mathrm{K}] \{\phi\}} \qquad \text{(B.1)}$$

where

$\bar{\xi}_j$ = = equivalent modal damping ratio of the j-th mode,

[K] = stiffness matrix,

[$\overline{K}$] = modified stiffness matrix constructed by the product of the damping ratio appropriate for the element and the stiffness,

{ϕ} = j-th modal vector.

(8) Other techniques can be used when more detailed data on the damping characteristics of structural subsystems are available.

(9) For each mode of vibration, the upper bound of the equivalent modal damping ratio, ≤ 0,15 is advisable, unless a suitable set of damping data is available on an experimental basis.

ANNEX C (Informative)

Soil-structure interaction

(1) The design earthquake motion is defined at the soil surface, in free-field conditions, i.e. where the inertial forces due to the presence of structure do not affect it. When the structure is founded on soil deposits or soft media, the resulting motion at the base of the structure will differ from that at the same elevation in the free-field, due to the soil deformability. For elevated structures, the rocking compliance of the soil may be important and may significantly increase the second order effects.

(2) The modelling methods of soil-structure interaction should consider, 1) the extent of embedment, 2) the depth of the possible bedrock, 3) the layering of the soil strata, 4) the intrinsic variability of the soil moduli in any single stratum, and 5), the strain-dependence of soil properties, (shear modulus and damping).

(3) The assumption of horizontal layering is generally acceptable.

(4) Unless the soil investigation suggests a suitable range of variability for the dynamic soil moduli, the upper bound of the soil stiffness may be obtained by multiplying by 2 the entire set of the best estimate moduli, and the lower bound by multiplying the entire set by 0,5.

(5) Being strain-dependent, damping and shear moduli for each soil layer should be consistent to the effective shear strain intensity expected during the excitation. An equivalent linear method is acceptable. In this case the analysis should be performed iteratively. In each iteration the analysis is linear but the soil properties are adjusted from iteration to iteration until the computed strain are compatible with the soil properties used in the analysis. The iterative procedure can be developed on the free-field soil deposit, disregarding the presence of the structure.

(6) The effective shear strain amplitudes in any one layer, to be used to evaluate the dynamic moduli and damping in equivalent linear methods, can be taken as

$$\gamma_{eff} = 0{,}65\,\gamma_{max,t} \qquad \text{(C.1)}$$

where $\gamma_{max,t}$ is the maximum value of the shear deformation in the soil layer, during the free-field excitation.

(7) If the finite elements modelling method for soil media is used, the criteria for determining the location of the bottom boundary and the side boundary should be justified. In general, the forcing functions to simulate the earthquake motion are applied at these boundaries. In such cases, it is required to generate excitation system acting at boundaries such that the response motion of the soil media at the surface free field is identical to the design ground motion. The procedures and theories for generation of such excitation system should be discussed.

(8) If the half-space (lumped parameters) modelling method is used, the parameters used in the analysis for the soil deformability should account for the layering. Besides, it should consider the intrinsic variability of soil moduli, and strain-dependent properties.

(9) Any other modelling methods used for soil-structure interaction analysis is to be clearly explained, as is any basis for not including soil-structure interaction analysis.

ANNEX D (Informative)

Number of degrees of freedom and number of modes of vibration

(1) A dynamic analysis (e.g., response spectrum, power spectrum, or time history method) should be used when the use of the equivalent static load cannot be justified.

(2) The analysis should include:

- Consideration of the torsion, rocking and translation response of the foundations.
- An adequate number of masses and degrees of freedom to determine the response of any structural element and plant equipment.
- A sufficient number of modes to assure participation of all significant modes.
- Consideration of the maximum relative displacement among supports of equipment or machinery (for a chimney, the interaction between internal and external tubes).
- Significant effects such as piping interactions, externally applied structural restraints, hydrodynamic loads (both mass and stiffness effects), and possible non-linear behaviour.
- Development of "floor response spectra", in the case of presence of important light equipment or appendices.

(3) The effective modal mass M^i mentioned in para. 5.4, can be computed as

$$M^i = [\{\phi\}^T [M] \{i\}]^2 / \{\phi\}^T [M] \{\phi\}, \qquad \text{(D.1)}$$

where

$\{\phi\}$ = i-th modal vector.

$\{i\}$ column vector, usually with 1 or 0 nondimensional components, which represents the displacement induced in the structure when its base is subjected to a unit static displacement in the relevant direction.

(4) The criterion indicated in D.3 does not assure the adequacy of the mass discretization, in the particular case where light equipment or a structural appendix is concerned. In this case the above condition might be fulfilled but the mathematical model of the structure could be inadequate to represent the equipment or appendix motion. When the analysis of the equipment or appendix is necessary, a "floor response spectrum", applicable for the floor elevation where the equipment/appendix is located, should be developed. This approach is also advisable when a portion of the structure need to be analysed independently, for instance, an internal masonry tube of a chimney, supported at individual brackets inserted in the main shaft.

ANNEX E (Informative) **MASONRY CHIMNEYS**

E 1 General. A masonry chimney is a chimney constructed of concrete blocks, or masonry, hereinafter referred to as masonry. Masonry chimneys should be constructed, anchored, supported and reinforced as required in this chapter.

E 2 Footings and foundations. Foundations for masonry chimneys should be constructed of concrete or solid masonry at least 300 mm thick and should extend at least 150 mm beyond the face of the foundation or support wall on all sides. Footings should be founded on natural undisturbed earth or engineered fill below frost depth. In areas not subjected to freezing, footings should be at least 300 mm below finished grade.

E 3 Behaviour factor Masonry chimneys should be constructed, anchored, supported and reinforced as required in order to fulfil the requirement of the present code, by assuming a behaviour factor $q_o = 1.5$.

E 4 Minimum vertical reinforcing. For chimneys up to one meter wide, four Φ 12 continuous vertical bars anchored in the foundation should be placed in the concrete, between wythes of solid masonry or within the cells of hollow unit masonry and grouted. Grout should be prevented from bonding with the flue liner so that the flue liner is free to move with thermal expansion. For chimneys greater than one meter wide, two additional Φ 12 vertical bars should be provided for each additional meter in width or fraction thereof.

E 5 Minimum horizontal reinforcing. Vertical reinforcement should be enclosed within 6 mm ties, or other reinforcing of equivalent net cross-sectional area, spaced not to exceed 400 mm on centre, or placed in the bed joints of unit masonry, at a minimum of every 400 mm of vertical height. Two such ties should be provided at each bend in the vertical bars.

E 6 Minimum seismic anchorage. Masonry chimneys and foundations should be anchored at each floor, ceiling or roof line more than two meters above grade, except where constructed completely within the exterior walls. Two 5 mm $\times$ 25 mm straps should be embedded a minimum of 300 mm into the chimney. Straps should be hooked around the outer bars and extend 150 mm beyond the bend. Each strap should be fastened to a minimum of four floor joists with two 12-mm bolts.

E 7 Corbeling. Masonry chimneys should not be corbeled more than half of the chimney's wall thickness from a wall or foundation, nor should a chimney be corbeled from a wall or foundation that is less than 300 mm in thickness unless it projects equally on each side of the wall, except that on the second story of a two-story dwelling, corbeling of chimneys on the exterior of the enclosing walls is permitted to equal the wall thickness. The projection of a single course should not exceed one-half the unit height or one-third of the unit bed depth, whichever is less.

E 8 Changes in dimension. The chimney wall or chimney flue lining should not change in size or shape within 150 mm above or below where the chimney passes through floor components, ceiling components or roof components.

E 9 Offsets. Where a masonry chimney is constructed with a fireclay flue liner surrounded by one wythe of masonry, the maximum offset should be such that the centerline of the flue above the offset does not extend beyond the centre of the chimney wall below the offset. Where the chimney offset is supported by masonry below the offset in an approved manner, the maximum offset limitations should not apply.

E 10 Additional load. Chimneys should not support loads other than their own weight unless they are designed and constructed to support the additional load. Masonry chimneys are permitted as part of the masonry walls or concrete walls of the building.

E 11 Wall thickness. Masonry chimney walls should be constructed of concrete blocks, solid masonry units, or hollow masonry units grouted solid with not less than 100 mm nominal thickness.