FOREWORD

Objectives of the Eurocodes

(1) The "Structural Eurocodes" comprise a group of standards for the structural and geotechnical design of buildings and civil engineering works.

(2) They cover execution and control only to the extent that is necessary to indicate the quality of the construction products, and the standard of the workmanship needed to comply with the assumptions of the design rules.

(3) Until the necessary set of harmonized technical specifications for products and for the methods of testing their performance are available, some of the Structural Eurocodes cover some of these aspects in informative Annexes.

Background of the Eurocode Programme

(4) The Commission of the European Communities (CEC) initiated the work of establishing a set of harmonized technical rules for the design of building and civil engineering works which would initially serve as an alternative to the different rules in force in the various Member States and would ultimately replace them. These technical rules became known as the "Structural Eurocodes".

(5) In 1990, after consulting their respective Member States, the CEC transferred the work of further development, issue and updating of the Structural Eurocodes to CEN, and the EFTA Secretariat agreed to support the CEN work.

(6) CEN Technical Committee CEN/TC250 is responsible for all Structural Eurocodes.

Eurocode Programme

(7) Work is in hand on the following Structural Eurocodes, each generally consisting of a number of parts:

EN 1991 Eurocode 1 Basis of design and 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 structures
EN 1998 Eurocode 8 Design provisions for earthquake resistance of
EN 1999 Eurocode 9 Design of aluminium alloy structures

(8) Separate sub-committees have been formed by CEN/TC250 for the various Eurocodes listed above.

(9) This Part 1-1 of Eurocode 8 is being published as a European Prestandard (ENV) with an initial life of three years.

(10) This Prestandard is intended for experimental application and for the submission of comments.

(11) After approximately two years CEN members will be invited to submit formal comments to be taken into account in determining future actions.

(12) Meanwhile feedback and comments on this Prestandard should be sent to the Secretariat of CEN/TC250/SC8 at the following adress:

IPQ c/o LNEC
Avenida do Brasil 101
P - 1799 LISBOA Codex
PORTUGAL

or to your national standards organization.

National Application Documents (NAD's)

(13) In view of the responsibilities of authorities in member countries for safety, health and other matters covered by the essential requirements of the Construction Products Directive (CPD), certain safety elements in this ENV have been assigned indicative values which are identified by [] ("boxed values"). The authorities in each member country are expected to review the "boxed values" and may substitute alternative definitive values for these safety elements for use in national application.

(14) Some of the supporting European or International standards may not be available by the time this Prestandard is issued. It is therefore anticipated that a National Application Document (NAD) giving any substitute definitive values for safety elements, referencing compatible supporting standards and providing guidance on the national application of this Prestandard, will be issued by each member country or its Standards Organization.

(15) It is intended that this Prestandard is used in conjunction with the NAD valid in the country where the building or civil engineering works is located.

Matters specific to this Prestandard

(16) The scope of Eurocode 8 is defined in clause 1.1.1 and the scope of this Part of Eurocode 8 is defined in clause 1.1.2. Additional Parts of Eurocode 8 which are planned are indicated in clause 1.1.3.

(17) This Prestandard was developed from one of the Parts that was included in the Draft of Eurocode 8 dated May 1988 published by the CEC and subjected to public enquiry. Such Draft contained also Parts 1-2 and 1-3 which are now presented as separate Prestandards.

(18) As mentioned in clause 1.1.1, attention must be paid to the fact that for the design of structures in seismic regions the provisions of Eurocode 8 are to be applied in addition to the provisions of the other relevant Eurocodes.

(19) In using this Prestandard in practice, particular regard should be paid to the underlying assumptions given in clause 1.3.

(20) One fundamental issue in this Prestandard is the definition of the seismic action. Given the wide difference of seismic hazard and seismo-genetic characteristics in the various member countries, the seismic action is herein defined by a sufficiently large number of parameters whose numerical values are within [] so that the authorities in each member country may adjust the seismic action to their specific conditions. 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 this Prestandard in terms of Code harmonization.

1 GENERAL

1.1 Scope

1.1.1 Scope of Eurocode 8

(1)P Eurocode 8 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,
- 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 probabilistic protection that can be provided to different categories of buildings 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.
>
> To provide the necessary flexibility in this respect, Eurocode 8 contains a set of safety elements whose values are left to the National Authorities to decide so that they can adjust the level of protection to their respective optimal value.

(2)P Special structures with increased risks for the population, such as nuclear power plants and large dams, are beyond the scope of Eurocode 8.

(3)P Eurocode 8 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) Eurocode 8 is subdivided into various separate Parts, see 1.1.2 and 1.1.3.

1.1.2 Scope of Part 1-1

(1) Part 1-1 contains the basic requirements and compliance criteria applicable to buildings and civil engineering works in seismic regions.

(2) In addition, Part 1-1 gives the rules for the representation of seismic actions and for their combination with other actions. Certain types of structures, dealt with in Parts 2 - 5, need complementing rules which are given in these Parts.

1.1.3 Further Parts of Eurocode 8

(1)P Further Parts of Eurocode 8 include - in addition to Part 1-1 - the following:

- Part 1-2 contains general design rules relevant specifically to buildings,
- Part 1-3 contains specific rules for various structural materials and elements, relevant specifically to buildings,
- Part 1-4 contains provisions for the seismic strengthening and repair of existing buildings,
- Part 2 contains specific provisions relevant to bridges,
- Part 3 contains specific provisions relevant to towers, masts and chimneys,
- Part 4 contains specific provisions relevant to tanks, silos and pipelines,
- Part 5 contains specific provisions relevant to foundations, retaining structures and geotechnical aspects.

1.2 Distinction between Principles and Application Rules

(1)P Depending on the character of the individual clauses, distinction is made in this Eurocode between **Principles** and **Application Rules**.

(2)P The **Principles** comprise:

- general statements and definitions for which there is no alternative,
- requirements and analytical models for which no alternative is permitted unless specifically stated.

(3)P The **Application Rules** are generally recognised rules which follow the Principles and satisfy their requirements.

(4) The Principles are identified by the letter P, following the paragraph number. The other items (without P) are Application Rules, e.g. as this paragraph.

(5) It is permissible to use alternative design rules which differ from the Application Rules given in Eurocode 8, provided that it is shown that the alternative rules accord with the relevant Principles and are at least equivalent with regard to the safety and serviceability achieved for the structures designed according to the Application Rules of Eurocode 8.

T	vibration period of a linear single degree of freedom system;
a_g	effective peak ground acceleration (also called "design ground acceleration") in rock or firm soil for the reference return period;
d_g	peak ground displacement;
g	acceleration of gravity;
q	behaviour factor;
α	ratio of the design ground acceleration to the acceleration of gravity;
γ_I	importance factor;
ψ_{2i}	combination coefficient for the quasi-permanent value of a variable action i;
ψ_{Ei}	combination coefficient for a variable action i, to be considered when determining the effects of the design seismic action.

1.7 Reference Codes

(1)P For the application of Eurocode 8, reference shall be made to the other Eurocodes 1 to 7 and 9.

(2) Eurocode 8 incorporates other normative references cited at the appropriate places in the text. They are listed below:

ISO 1000	S.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.
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.
prEN 1337-1	Structural bearings - General requirements.

2 FUNDAMENTAL 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 Chapter 4 without local or general collapse, thus retaining its structural integrity and a residual load bearing capacity after the seismic event[1].

- **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.

(2)P Target reliabilities for the **"no collapse requirement"** and 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. The numerical values included in the safety related provisions - given only as indications in this Eurocode - shall be consistent with the chosen target reliabilities.

(3)P Reliability differentiation is implemented by classifying structures into different importance categories. To each importance category an importance factor γ_I is assigned. 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, as appropriate for the design of the specific category of structures, with respect to the reference value (see 4.1.(3)).

(4) The different levels of reliability are obtained by modifying the reference seismic action or - when using linear analysis - the corresponding action effects with this importance factor. Detailed guidance on the importance categories and the corresponding importance factors is given in the relevant Parts of Eurocode 8.

2.2 Compliance criteria

2.2.1 General

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

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[1] The design seismic action is generally selected on the basis of a chosen return period and need not coincide with the event of maximum intensity that may occur at a given site. It is assumed that through proper selection of the value of the return period and proper calibration of the design procedures and associated safety elements, the target probability of failure is satisfied.

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

- **Serviceability limit states**
  are those associated with damage occurrence, corresponding to states beyond which specified service requirements are no longer met.

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

(3) For well defined categories of structures in zones with low seismicity (see 4.1) the fundamental requirements can be satisfied through the application of rules simpler than those given in the relevant Parts of Eurocode 8.

(4) Specific rules for "simple masonry buildings" are given in Section 6 of Part 1-3. By complying with those rules the fundamental requirements for such "simple masonry buildings" are deemed to be satisfied without analytical safety verifications.

2.2.2 Ultimate limit state

(1)P The structural system shall be verified as having the resistance and ductility specified in the relevant Parts of Eurocode 8.

(2) The resistance and ductility 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 ductility is characterized by the values of the behaviour factor q, which are given in the relevant Parts of Eurocode 8. 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 is equal to 1,0. For dissipative structures the behaviour factor is taken greater than 1,0 accounting for the hysteretic energy dissipation that occurs in specifically designed zones called dissipative zones or critical regions.

(3)P The structure as a whole shall be checked to be stable under the design seismic action. Both overturning and sliding stability shall be considered. Specific rules for checking the overturning of structures are given in the relevant Parts of Eurocode 8.

(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.

2.2.3 Serviceability limit 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 Eurocode 8.

(2)P In structures important for civil protection the structural system shall be verified to possess 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) Structures should have simple and regular forms both in plan and elevation, see e.g. clause 2 of Part 1-2. If necessary this may be realized by subdividing the structure by joints into dynamically independent units.

(2)P In order to ensure an overall ductile behaviour, brittle failure or the premature formation of unstable mechanisms shall be avoided. To this end, it may be necessary, as indicated in the relevant Parts of this Eurocode, to resort to the capacity design procedure which is used to obtain the hierarchy of resistance of the various structural components necessary for ensuring the intended configuration of dissipative elements 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 under cyclic conditions the capacity to transmit the necessary forces and to dissipate energy. 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) In order to limit the consequences of the seismic event, National Authorities may specify restrictions on the height or other characteristics of a structure depending on local seismicity, importance category, ground conditions, city planning and environmental planning.

(5)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 members.

(6)P No change of the structure is allowed 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.

2.2.4.2 Foundations

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

(2) 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 cases of high seismicity and of 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

3.1 General

(1)P Appropriate investigations shall be carried out in order to classify the ground according to the classes given in 3.2.

(2) Further guidance concerning soil investigation and classification is given in clause 4.2 of Part 5.

(3)P 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 according to section 4 of Part 5.

(4) For structures of low importance ($\gamma_I \leq [1,0]$) in low seismicity zones (see 4.1) ground investigations may be omitted. In this case and in the absence of more accurate information the seismic action may be determined assuming ground conditions according to subsoil class B (see 3.2).

3.2 Classification of subsoil conditions

(1)P The influence of local ground conditions on the seismic action shall generally be accounted for by considering the three subsoil classes A, B, C, described by the following stratigraphic profiles:

- **Subsoil class A**
  - Rock or other geological formation characterized by a shear wave velocity v_s of at least 800 m/s, including at most 5 m of weaker material at the surface.
  - Stiff deposits of sand, gravel or overconsolidated clay, at least several tens of m thick, characterized by a gradual increase of the mechanical properties with depth and by v_s-values of at least 400 m/s at a depth of 10 m).

- **Subsoil class B**

  Deep deposits of medium dense sand, gravel or medium stiff clays with thickness from several tens to many hundreds of m, characterized by v_s-values of at least 200 m/s at a depth of 10 m, increasing to at least 350 m/s at a depth of 50 m.

- **Subsoil class C**
  - Loose cohesionless soil deposits with or without some soft cohesive layers, characterized by v_s-values below 200 m/s in the uppermost 20 m.
  - Deposits with predominant soft-to-medium stiff cohesive soils, characterized by v_s-values below 200 m/s in the uppermost 20 m.

(2) Additions and/or modifications to this classification may be necessary to better conform with special soil conditions.

4 SEISMIC ACTION

4.1 Seismic zones

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

(2) For most of the applications of this Eurocode, the hazard is described in terms of a single parameter, i.e. the value a_g of the effective peak ground acceleration in rock or firm soil, henceforth called "design ground acceleration". Additional parameters required for specific types of structures are given in the relevant Parts of Eurocode 8.

NOTE: The concept of the "effective peak ground acceleration" is an attempt to compensate for the inadequacy in general of the actual single peak to describe the damaging potential of the ground motion in terms of maximum acceleration and/or velocity induced to the structures.

There is not a unique established definition and corresponding techniques for deriving a_g from the ground motion characteristics, the methods actually varying as functions of these latter. In general, a_g tends to coincide with the actual peak for moderate-to-high magnitude of medium-to-long distance events, which are characterized (on firm ground) by a broad and approximately uniform frequency spectrum, while a_g will be more or less reduced relative to the actual peak for near field, low magnitude events.

(3) The design ground acceleration, chosen by the National Authorities for each seismic zone, corresponds to a reference return period of [475] years. To this reference return period an importance factor γ_I equal to 1,0 is assigned.

(4) Seismic zones with a design ground acceleration a_g not greater than $[0,10] \cdot g$ are low seismicity zones, for which reduced or simplified seismic design procedures for certain types or categories of structures may be used.

(5)P In seismic zones with a design ground acceleration a_g not greater than $[0,04] \cdot g$ the provisions of Eurocode 8 need not be observed.

4.2 Basic representation of the seismic action

4.2.1 General

(1)P Within the scope of Eurocode 8 the earthquake motion at a given point of the surface is generally represented by an elastic ground acceleration response spectrum, henceforth called "elastic response spectrum".

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

(3) Unless specific studies indicate otherwise, the vertical component of the seismic action should be represented by the response spectrum as defined for the horizontal seismic action, but with the ordinates reduced as follows:

- For vibration periods T smaller than 0,15 s the ordinates are multiplied by a factor of [0,70].

- For vibration periods T greater than 0,50 s the ordinates are multiplied by a factor of [0,50].

- For vibration periods T between 0,15 s and 0,50 s a linear interpolation shall be used.

(4) For special conditions more than one spectrum may be needed to adequately represent the seismic hazard over an area. This may be necessary when the earthquakes affecting the area are generated by sources differing widely in distance, focal mechanism or travel path geology, as in the case of shallow depth and intermediate depth earthquakes. In such circumstances, different values of a_g as well as different shapes of the response spectrum for each type of earthquake would normally be required.

(5) For important structures in high seismicity zones it is recommended to consider topographic amplification effects according to Annex B of Part 5.

(6) Alternative representations of the earthquake motion - e.g. power spectrum or time history representation - may be used (see 4.3).

(7) Allowance for the variation of ground motion in space as well as time may be required for specific types of structures (see Parts 2, 3 and 4).

4.2.2 Elastic response spectrum

(1)P The elastic response spectrum $S_e(T)$ for the reference return period is defined by the following expressions (see figure 4.1):

$$0 \le T \le T_B: \quad S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot \beta_o - 1)\right] \tag{4.1}$$

$$T_B \le T \le T_C: \quad S_e(T) = a_g \cdot S \cdot \eta \cdot \beta_o \tag{4.2}$$

$$T_C \le T \le T_D: \quad S_e(T) = a_g \cdot S \cdot \eta \cdot \beta_o \cdot \left[\frac{T_C}{T}\right]^{k_1} \tag{4.3}$$

$$T_D \le T: \quad S_e(T) = a_g \cdot S \cdot \eta \cdot \beta_o \cdot \left[\frac{T_C}{T_D}\right]^{k_1} \cdot \left[\frac{T_D}{T}\right]^{k_2} \tag{4.4}$$

where

$S_e(T)$	ordinate of the elastic response spectrum,
T	vibration period of a linear single degree of freedom system,
a_g	design ground acceleration for the reference return period,
$ß_o$	spectral acceleration amplification factor for 5 % viscous damping,
T_B, T_C	limits of the constant spectral acceleration branch,
T_D	value defining the beginning of the constant displacement range of the spectrum,
k_1, k_2	exponents which influence the shape of the spectrum for a vibration period greater than T_C, T_D respectively,
S	soil parameter
η	damping correction factor with reference value $\eta = 1$ for 5 % viscous damping, see (6).

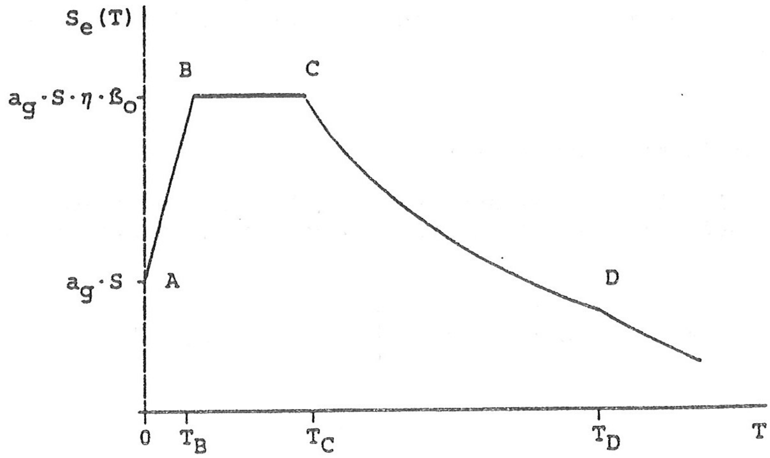

Figure 4.1: Elastic response spectrum

(2) For the three subsoil classes A, B and C the values of the parameters $ß_o$, T_B, T_C, T_D, k_1, k_2, S are given in table 4.1.

Table 4.1: Values of the parameters describing the elastic response spectrum*

sub-soil class	S	β_o	k_1	k_2	T_B [s]	T_C [s]	T_D [s]
A	[1,0]	[2,5]	[1,0]	[2,0]	[0,10]	[0,40]	[3,0]
B	[1,0]	[2,5]	[1,0]	[2,0]	[0,15]	[0,60]	[3,0]
C	[0,9]	[2,5]	[1,0]	[2,0]	[0,20]	[0,80]	[3,0]

* These values are selected so that the ordinates of the elastic response spectrum have a uniform probability of exceedance over all periods (uniform risk spectrum) equal to 50%.

(3) When the subsoil profile includes an alluvial surface layer with thickness varying between 5 and 20 m, underlain by much stiffer materials of class A, the spectrum shape for subsoil class B can be used together with an increased soil parameter S equal to [1,4], unless a special study is performed.

(4) For sites with ground conditions not matching the three subsoil classes A, B, C special studies for the definition of the seismic action may be required.

(5) Special attention should be paid in the case of a deposit of subsoil class C which consists - or contains a layer at least 10 m thick - of soft clays/silts with high plasticity index (PI > 40) and high water content. 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 Section 6 of Part 5. In this case, a special study for the definition of the seismic action should be carried out, in order to establish the dependance 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.

(6) The value of the damping correction factor η can be determined by the expression

$$\eta = \sqrt{7/(2+\xi)} \geq 0,7 \qquad (4.5)$$

where ξ is the value of the viscous damping ratio of the structure, expressed in percent. If for special studies a viscous damping ratio different from 5 % is to be used, this value will be given in the relevant Parts of Eurocode 8.

4.2.3 Peak ground displacement

(1) Unless special studies based on the available information indicate otherwise the value d_g of the peak ground displacement may be

estimated by means of the following expression

$$d_g = [0,05] \cdot a_g \cdot S \cdot T_C \cdot T_D \qquad (4.6)$$

with the values of a_g, S, T_C, T_D as defined in 4.2.2.

4.2.4 Design spectrum for linear analysis

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

(2) To avoid explicit nonlinear structural analysis in design, the energy dissipation capacity of the structure, through mainly ductile behaviour of its elements and/or other mechanisms, is taken into account by performing a linear analysis based on a response spectrum, reduced with respect to the elastic one, henceforth called "design spectrum". This reduction is accomplished by introducing the behaviour factor q. Additionally, modified exponents k_{d1} and k_{d2} are generally used.

(3) 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 minimum seismic forces that may be used in design - with a conventional linear model - still ensuring a satisfactory response of the structure. The values of the behaviour factor q, which also accounts for the influence of the viscous damping being different from 5%, are given for the various materials and structural systems and according to various ductility levels in the relevant Parts of Eurocode 8.

(4)P For the reference return period the design spectrum $S_d(T)$, normalized by the acceleration of gravity g, is defined by the following expressions:

$$0 \leq T \leq T_B: \quad S_d(T) = \alpha \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot \left(\frac{\beta_o}{q} - 1\right)\right] \qquad (4.7)$$

$$T_B \leq T \leq T_C: \quad S_d(T) = \alpha \cdot S \cdot \frac{\beta_o}{q} \qquad (4.8)$$

$$T_C \leq T \leq T_D: \quad S_d(T) \begin{cases} = \alpha \cdot S \cdot \dfrac{\beta_o}{q} \cdot \left[\dfrac{T_C}{T}\right]^{k_{d1}} \\ \geq [0,20] \cdot \alpha \end{cases} \qquad (4.9)$$

$$T_D \leq T: \quad S_d(T) \begin{cases} = \alpha \cdot S \cdot \dfrac{\beta_o}{q} \cdot \left[\dfrac{T_C}{T_D}\right]^{k_{d1}} \cdot \left[\dfrac{T_D}{T}\right]^{k_{d2}} \\ \geq [0,20] \cdot \alpha \end{cases} \qquad (4.10)$$

where

$S_d(T)$ ordinate of the design spectrum, which is normalized by g,

α ratio of the design ground acceleration a_g to the acceleration of gravity g ($\alpha = a_g/g$),

q behaviour factor,

k_{d1}, k_{d2} exponents which influence the shape of the design spectrum for a vibration period greater than T_C, T_D respectively.

(5) Values of the parameters β_o, T_B, T_C, T_D, S are given in table 4.1, values of the parameters k_{d1}, k_{d2} are given in table 4.2.

Table 4.2: Values of k_{d1} and k_{d2}

subsoil class	k_{d1}	k_{d2}
A	[2/3]	[5/3]
B	[2/3]	[5/3]
C	[2/3]	[5/3]

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

4.3 Alternative representations of the seismic action

4.3.1 Power spectrum representation

(1)P The seismic motion at a given point on the ground surface may also be represented as a random process, defined by a power spectrum - i.e. the power spectral density function of the acceleration process - associated with a certain duration, consistent with the magnitude and the other relevant features of the seismic event.

(2)P The power spectrum shall be consistent with the elastic response spectrum used for the basic definition of the seismic action according to 4.2.2.

(3) Consistency is considered to be achieved when the 50%-fractile values from the distribution of the maxima of the response of a single-degree-of-freedom-system subjected to a random process defined by the power spectrum coincide, within a tolerance of [± 10%], over the range of periods from 0,20 s to 3,5 s with the ordinates of the elastic response spectrum given in 4.2.2.

(4)P The seismic motion shall consist of three independent random processes simultaneously acting along two arbitrarily chosen horizontal orthogonal axes x and y, and the vertical axis z, this latter process being appropriately scaled according to 4.2.1 (3). Simplifications are possible according to the relevant Parts of Eurocode 8.

4.3.2 Time-history representation

4.3.2.1 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 according to the relevant Parts of Eurocode 8.

(3) Depending on the nature of the application and on the information actually available, the description of the seismic motion can be made by using artificial accelerograms (see 4.3.2.2) and recorded or simulated accelerograms (see 4.3.2.3).

4.3.2.2 Artificial accelerograms

(1)P Artificial accelerograms shall be generated so as to match the elastic response spectrum given in 4.2.2.

(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 specific data is not available, the minimum duration T_s of the stationary part of the accelerograms for epicentral areas should be correlated to the value of $\gamma_I \cdot \alpha$ $(= \gamma_I \cdot a_g/g)$ as indicated in table 4.3.

Table 4.3: Duration Ts of the stationary part of the generated accelerograms as a function of $\gamma_I \cdot \alpha$ for epicentral areas

$\gamma_I \cdot \alpha$	0,10	0,20	0,30	0,40
T_s	[10] s	[15] s	[20] s	[25] s

(4)P The number of the accelerograms to be used shall be such as to give a stable statistical measure (mean and variance) of the response quantities of interest. The amplitude and the frequency content of the accelerograms shall be chosen such that their use results in an overall level of reliability commensurate with that implied by the use of the elastic response spectrum of 4.2.2.

(5) Paragraph (4)P is deemed to be satisfied if the following rules are observed:

a) A minimum of [5] accelerograms is used.

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

c) In the period range T_B to T_C of the elastic response spectrum for the site in question, the average of the values of the mean spectrum from all time histories (calculated with an adequate number of control periods) is not smaller than the value $a_g \cdot S \cdot ß_o$ of the elastic response spectrum.

d) No value of the mean spectrum - calculated from all time histories - is more than 10% below the corresponding value of the elastic response spectrum.

4.3.2.3 Recorded or simulated accelerograms

(1)P The use of recorded accelerograms - or of accelerograms generated through a physical simulation of source and travel path mechanisms - is allowed provided the samples used (which shall not be less than [3]) 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 $a_g \cdot S$ for the zone under consideration.

(2)P For soil amplification analyses and for dynamic slope stability verifications see clause 2.2 of Part 5.

4.3.3 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 be reasonably made, spatial models of the seismic action shall be used (see 4.2.1.(7)).

(2)P Such spatial models shall be consistent with the elastic response spectra used for the basic definition of the seismic action according to 4.2.2.

1 GENERAL

1.1 Scope

(1)P Part 1-2 is concerned with buildings. It contains general rules for the earthquake-resistant design of buildings and shall be used in conjunction with Parts 1-1 and 1-3.

(2)P Whereas guidance on base-isolated buildings is not given in the code, the use of base-isolation is not precluded, provided special studies are undertaken.

1.2 Symbols

In addition to the symbols listed in Part 1-1, the following symbols are used in Part 1-2 with the following meanings:

E_E	effect of the seismic action
E_{Edx}, E_{Edy}	design values of the action effects due to the horizontal components of the seismic action;
E_{Edz}	design value of the action effects due to the vertical component of the seismic action;
F	horizontal seismic force;
F_a	horizontal seismic force acting on a non-structural element (appendage);
H	building height;
R_d	design resistance;
T_1	fundamental vibration-period of a building;
T_a	fundamental vibration-period of a non-structural element (appendage);
W	weight;
W_a	weight of a non-structural element (appendage);
d	displacement;
d_r	design interstorey drift;
e_1	accidental eccentricity of a storey mass from its nominal location;
h	interstorey height;

m mass;

q_a behaviour factor of a non-structural element;

q_d displacement behaviour factor;

s displacement of a mass m in the fundamental mode shape of a building;

z height of the mass m above the level of application of the seismic action;

γ_a importance factor of a non-structural element;

θ interstorey drift sensitivity coefficient.

2 CHARACTERISTICS OF EARTHQUAKE RESISTANT BUILDINGS

2.1 Basic principles of conceptual design

(1)P The aspect of seismic hazard shall be taken into consideration in the early stages of the conceptual design of the building.

(2) The guiding principles governing this conceptual design against seismic hazard are

- structural simplicity,
- uniformity and symmetry,
- redundancy,
- bidirectional resistance and stiffness,
- torsional resistance and stiffness,
- diaphragmatic action at storey level,
- adequate foundation.

(3) Commentaries to these principles are given in Annex B.

2.2 Structural regularity

2.2.1 General

(1)P For the purpose of seismic design, building structures are distinguished as regular and non-regular.

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

- the structural model, which can be either a simplified planar or a spatial one,

- the method of analysis, which can be either a simplified modal or a multi-modal one,

- the value of the behaviour factor q, which can be decreased depending on the type of non-regularity in elevation, i.e.

  - geometric non-regularity exceeding the limits given in 2.2.3.(4),

  - non-regular distribution of overstrength in elevation exceeding the limits given in 2.2.3.(3).

(3)P With regard to the implications of structural regularity on the design, separate consideration is given to the regularity characteristics of the building in plan and in elevation, according to table 2.1.

Table 2.1: Consequences of structural regularity on seismic design

REGULARITY		ALLOWED SIMPLIFICATION		BEHAVIOUR
PLAN	ELEVATION	MODEL	ANALYSIS	FACTOR
YES	YES	PLANAR	SIMPLIFIED*	REFERENCE
YES	NO	PLANAR	MULTIMODAL	DECREASED
NO	YES	SPATIAL**	MULTIMODAL**	REFERENCE
NO	NO	SPATIAL	MULTIMODAL	DECREASED

* If the condition of 3.3.2.1.(2)b) is also met.

** Under the specific conditions given in clause A1 of Annex A simpler models and methods of analysis, described in Annex A, may be used.

(4) Criteria describing regularity in plan and in elevation are given in 2.2.2 and 2.2.3; rules concerning modelling and analysis are given in 3; the relevant behaviour factors are given in Part 1-3.

(5)P The regularity criteria given in 2.2.2 and 2.2.3 are to be considered as necessary conditions. The designer shall verify that the assumed regularity of the building structure is not impaired by other characteristics, not included in these criteria.

c) in case the setbacks do not preserve symmetry, in each face the sum of the setbacks at all storeys is not greater than 30 % of the plan dimension at the first storey, and the individual setbacks are not greater than 10 % of the previous plan dimension (see fig. 2.1.d).

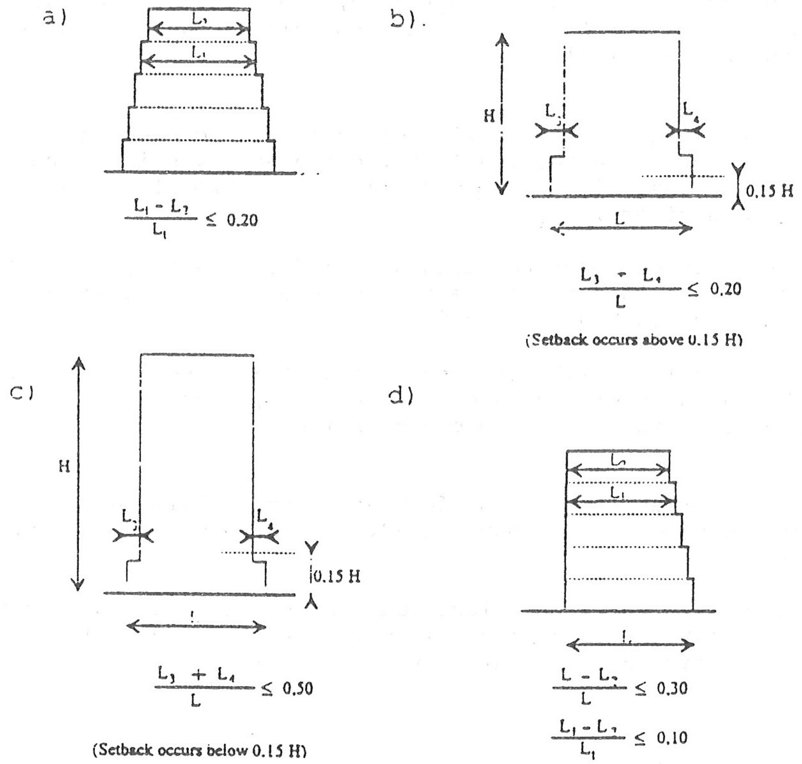

Figure 2.1: Criteria for regularity of setbacks

3 STRUCTURAL ANALYSIS

3.1 Modelling

(1)P The model of the building shall adequately represent the distribution of stiffness and mass so that all significant deformation shapes and inertia forces are properly accounted for under the seismic action considered[1].

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[1]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 main resisting structural system, should also be accounted for.

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

(3) When the floor diaphragms of the building are sufficiently rigid in their plane, the masses and the moments of inertia of each floor may be lumped at the center of gravity, thus reducing the degrees of freedom to three per floor (two horizontal displacements and a rotation about the vertical axis).

(4) For buildings complying with the criteria for regularity in plan (see 2.2.2) or with the regularity criteria given in clause A1 of Annex A, the analysis can be performed using two planar models, one for each main direction.

(5) In reinforced concrete buildings and in masonry buildings the stiffness of the load bearing elements should, in general, be evaluated assuming uncracked sections[2].

(6) Infill walls which increase significantly the lateral stiffness of the building should be taken into account, see clause 2.9 of Part 1-3 for masonry infills of concrete frames.

(7)P The deformability of the foundation soil shall be considered in the model whenever it may have an adverse influence on the structural response.

(8)P The masses shall be calculated from the gravity loads appearing in the combination of actions given in 4.4.(2) of Part 1-1. The combination coefficients ψ_{Ei} are given in 3.6.(2).

3.2 Accidental torsional effects

(1)P In addition to the actual eccentricity, in order to cover uncertainties in the location of masses and in the spatial variation of the seismic motion, the calculated center of mass at each floor i shall be considered displaced from its nominal location in each direction by an additional accidental eccentricity:

$$e_{1i} = \pm 0,05 \cdot L_i \qquad (3.1)$$

where

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[2] In reinforced concrete buildings this assumption may lead to unconservative estimates of the displacements, especially when high values of the behaviour factor q are assumed. In such cases and if displacements are critical, a more accurate estimation of the stiffness of the elements under the seismic action may be necessary with regard to the displacement analysis according to 3.4..

e_{1i} accidental eccentricity of storey mass i from its nominal location, applied in the same direction at all floors,

L_i floor-dimension perpendicular to the direction of the seismic action.

3.3 Methods of analysis

3.3.1 General

(1)P Within the scope of Part 1-2, the seismic effects and the other action effects to be considered according to the combination rules given in clause 4.4 of Part 1-1 may be determined on the basis of a linear-elastic behaviour of the structure.

(2)P The reference method for determining the seismic effects is the modal response analysis, using a linear-elastic model of the structure and the design spectrum given in clause 4.2.4 of Part 1-1.

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

- the "simplified modal response spectrum analysis" for buildings meeting the conditions given in 3.3.2,

- the "multi-modal response spectrum analysis", which is applicable to all types of buildings (see 3.3.3).

(4) As alternatives to these basic methods other methods of structural analysis, such as

- power spectrum analysis,

- (non-linear) time history analysis,

- frequency domain analysis,

are allowed under the conditions specified in paragraphs (5) and (6)P and in 3.3.4.

(5) Non-linear analyses can be used, provided they are 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)P If a non-linear analysis is used, the amplitudes of the accelerograms derived for the reference return period (see clause 4.3.2 of Part 1-1) shall be multiplied by the importance factor γ_I of the building (see 3.7).

3.3.2 Simplified modal response spectrum analysis

3.3.2.1 General

(1)P This type of analysis can be applied to buildings that can be analysed by two planar models and whose response is not significantly affected by contributions from higher modes of vibration.

(2) These requirements are deemed to be satisfied by buildings which

a1) meet the criteria for regularity in plan and in elevation given in 2.2.2 and 2.2.3

or

a2) meet the criteria for regularity in elevation given in 2.2.3 and the regularity criteria given in clause A1 of Annex A

and

b) have fundamental periods of vibration T_1 in the two main directions less than the following values

$$T_1 \leq \begin{cases} 4 \cdot T_C \\ 2{,}0 \text{ s} \end{cases} \tag{3.2}$$

where T_C is given in table 4.1 of Part 1-1.

3.3.2.2 Base shear force

(1)P The seismic base shear force F_b for each main direction is determined as follows:

$$F_b = S_d(T_1) \cdot W \tag{3.3}$$

where

$S_d(T_1)$ ordinate of the design spectrum (see clause 4.2.4 of Part 1-1) at period T_1,

T_1 fundamental period of vibration of the building for translational motion in the direction considered,

W total weight of the building computed in accordance with 3.1.(8).

(2) For the purpose of determining the fundamental vibration periods T_1 of both planar models of the building, approximate expressions based on methods of structural dynamics (e.g. by Rayleigh method) may be used[3].

[3]For preliminary design purposes the approximate expressions for T_1 given in Annex C may be used.

3.3.2.3 Distribution of the horizontal seismic forces

(1)P The fundamental mode shapes of both planar models 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 storey masses m_i.

(3)P The forces shall be determined by assuming the entire mass of the structure as a substitute mass of the fundamental mode of vibration, hence:

$$F_i = F_b \cdot \frac{s_i \cdot W_i}{\Sigma s_j \cdot W_j} \qquad (3.4)$$

where

F_i horizontal force acting on storey i,

F_b seismic base shear according to exp.(3.3);

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

W_i, W_j weights of masses m_i, m_j computed according to 3.1.(8).

(4) When the fundamental mode shape is approximated by horizontal displacements increasing linearly along the height, the horizontal forces F_i are given by:

$$F_i = F_b \cdot \frac{z_i \cdot W_i}{\Sigma z_j \cdot W_j} \qquad (3.5)$$

where

z_i, z_j heights of the masses m_i, m_j above the level of application of the seismic action (foundation).

(5)P The horizontal forces F_i determined in the above manners shall be distributed to the lateral load resisting system assuming rigid floors.

3.3.2.4 Torsional effects

(1) In case of symmetric distribution of lateral stiffness and mass and if no more exact method is applied regarding 3.2, the accidental torsional effects may be accounted for by amplifying the action

effects in the individual load resisting elements - evaluated according to 3.3.2.3.(5) - with a factor δ given by:

$$\delta = 1 + 0,6 \cdot \frac{x}{L_e} \qquad (3.6)$$

where

x distance of the element under consideration from the centre of the building measured perpendicularly to the direction of the seismic action considered,

L_e distance between the two outermost lateral load resisting elements measured as previously.

(2) Whenever the conditions given in clause A1 of Annex A are met, the approximate analysis of torsional effects as described in Annex A can be applied.

3.3.3 Multi-modal response spectrum analysis

3.3.3.1 General

(1)P This type of analysis shall be applied to buildings which do not satisfy the conditions given in 3.3.2.1.(2) for applying the simplified modal response spectrum analysis.

(2) For buildings complying with the criteria for regularity in plan (see 2.2.2) or with the regularity criteria given in Clause A1 of Annex A, the analysis can be performed using two planar models, one for each main direction.

(3)P Buildings not complying with these criteria shall be analysed using a spatial model.

(4)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 axes. For buildings with resisting elements in two perpendicular directions these two directions are considered as the relevant ones.

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

(6) Paragraph (5) may be satisfied by either of the following:

- By demonstrating that the sum of the effective modal masses for the modes considered amounts to at least 90% of the total mass of the structure.

- By demonstrating that all modes with effective modal masses greater than 5 % of the total mass are considered.

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 $F_{bk} = S_d(T_k) \cdot m_k \cdot g$. 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.

(7) When using a spatial model, the above conditions have to be verified for each relevant direction.

(8) If paragraph (6) cannot be satisfied (e.g. in buildings with a significant contribution from torsional modes), the minimum number k of modes to be considered in a spatial analysis should satisfy the following conditions:

$$k \geq 3 \cdot \sqrt{n} \tag{3.7}$$

and

$$T_k \leq 0{,}20 \text{ s} \tag{3.8}$$

where

k number of modes considered,

n number of storeys above ground,

T_k period of vibration of mode k.

3.3.3.2 Combination of modal responses

(1)P The response in two vibration modes i and j (including both translational and torsional modes) may be considered as independent of each other when their periods T_i and T_j satisfy the following condition:

$$T_j \leq 0{,}9 \cdot T_i \tag{3.9}$$

(2) Whenever all relevant modal responses (see 3.3.3.1.(5)-(8)) can be regarded as independent of each other, the maximum value E_E of a seismic action effect may be taken as

$$E_E = \sqrt{\Sigma E_{Ei}^2} \tag{3.10}$$

where

E_E seismic action effect under consideration (force, displacement, etc.),

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

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

3.3.3.3 Torsional effects

(1) Whenever a spatial model is used for the analysis, the accidental torsional effects referred in 3.2 may be determined as the envelope of the effects resulting from an analysis for static loadings, consisting of torsional moments M_{1i} about the vertical axis of each storey i:

$$M_{1i} = e_{1i} \cdot F_i \qquad (3.11)$$

where

M_{1i} torsional moment of storey i about its vertical axis,

e_{1i} accidental eccentricity of storey mass i according to eq.(3.1) for all relevant directions, see 3.3.3.1.(4),

F_i horizontal force acting on storey i, as derived in 3.3.2.3 for all relevant directions.

(2) The effects of the loading according to paragraph (1) should be considered with alternate signs (the same 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 3.3.2.4.(1) or of Annex A to the action effects computed according to 3.3.3.2.

3.3.4 Alternative methods of analysis

3.3.4.1 General

(1)P If the alternative methods of analysis described below are used, it shall be demonstrated, that the fundamental requirements according to clause 2.1 of Part 1-1 are met with a level of reliability commensurate with the use of the reference method described in 3.3.3.

(2) Paragraph (1)P may be satisfied by either of the following:

a) By demonstrating that the sum of the computed horizontal shear forces at all supports in each of two orthogonal directions is not less than 80 % of the corresponding sums obtained by multimodal analysis according to 3.3.3.

b) Where the sum in either direction is less than 80 % of the value from multi-modal analysis, the computed values of all response variables shall be scaled proportionately by the scale factor required to bring the base shear force to the value needed for satisfying the condition a).

3.3.4.2 Power spectrum analysis

(1) A linear stochastic analysis of the structure can be performed, either by using modal analysis or frequency dependent response matrices, using as input the acceleration power spectrum, defined in clause 4.3.1 of Part 1-1.

(2)P The elastic action effects shall be defined as the 50%-fractile of the probability distribution of the peak response in a time interval equal to the assumed duration of the motion.

(3)P The design values shall be determined by dividing these elastic effects by the ratio of the ordinate of the elastic response spectrum to the ordinate of the design spectrum corresponding to the fundamental period of the building, multiplied by g.

3.3.4.3 Time-history analysis

(1) The time dependent response of the structure can be obtained through direct numerical integration of its differential equations of motion, using the accelerograms, defined in clause 4.3.2 of Part 1-1 to represent the ground motions.

(2) When the structure is considered to behave non-linearly, the provisions of 3.3.1.(5)-(6)P apply.

3.3.4.4 Frequency domain analysis

(1)P The seismic action input is the same as in 3.3.4.3, but with each accelerogram cast in the form of a Fourier summation. The response is obtained by convolving over the frequency domain the harmonic components of the input with their respective frequency response matrices or functions.

(2)P The elastic action effects shall be defined as the mean values of the peak responses calculated for the various accelerograms.

(3)P The design values shall be determined by dividing the elastic effects by the ratio of the ordinate of the elastic response spectrum to the ordinate of the design spectrum corresponding to the fundamental period of the building, multiplied by g.

3.3.5 Combination of the components of the seismic action

3.3.5.1 Horizontal components of the seismic action

(1)P In general the horizontal components of the seismic action (see clause 4.2.1.(2) of Part 1-1) shall be considered as acting simultaneously.

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

- The structural response to each horizontal component shall be evaluated separately, using the combination rules for modal responses as given in 3.3.3.2.

- 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 responses to each horizontal component.

(3) As an alternative to paragraph (2) the action effects due to the combination of the horizontal components of the seismic action may be computed using the two following combinations:

a) E_{Edx} "+" $0{,}30 \cdot E_{Edy}$

b) $0{,}30 \cdot E_{Edx}$ "+" E_{Edy}

where

"+" implies "to be combined with",

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

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

(4) The sign of each component in the above combinations shall be taken as the most unfavourable for the effect under consideration.

(5)P For buildings satisfying the regularity criteria in plan and in which walls are the only horizontal load resisting components, the seismic action may be assumed to act separately along the two main orthogonal horizontal axes of the structure.

(6)P When using time-history analysis according to 3.3.4.3 and employing a spatial model of the structure, simultaneously acting accelerograms shall be considered for both horizontal components.

3.3.5.2 Vertical component of the seismic action

(1)P The vertical component of the seismic action, as defined in clause 4.2.1.(3) of Part 1-1, shall be taken into account in the following cases:

- Horizontal or nearly horizontal structural members spanning 20 meters or more;

- Horizontal or nearly horizontal cantilever components;

- Horizontal or nearly horizontal prestressed components;

- Beams supporting columns.

(2) In general, the analysis for determining the effects of the vertical component of the seismic action can be made based on a partial model of the structure which includes the elements under consideration and takes into account the stiffness of the adjacent elements.

(3) The effects of the vertical component need only be considered for the elements under consideration and their directly associated supporting elements or substructures.

(4) In case the horizontal components of the seismic action are also relevant for these elements, the following three combinations may be used for the computation of the action effects:

a) $0{,}30 \cdot E_{Edx}$ "+" $0{,}30 \cdot E_{Edy}$ "+" E_{Edz}

b) E_{Edx} "+" $0{,}30 \cdot E_{Edy}$ "+" $0{,}30 \cdot E_{Edz}$

c) $0{,}30 \cdot E_{Edx}$ "+" E_{Edy} "+" $0{,}30 \cdot E_{Edz}$

where

E_{Edx} see 3.3.5.1.(3),

E_{Edy} see 3.3.5.1.(3),

E_{Edz} action effects due to the application of the vertical component of the design seismic action as defined in clause 4.2.1.(3) of Part 1-1.

3.4 Displacement analysis

(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:

$$d_s = q_d \cdot d_e \cdot \gamma_I \qquad (3.12)$$

where

4 SAFETY VERIFICATIONS

4.1 General

(1)P For the safety verifications the relevant limit states (see 4.2 and 4.3) and specific measures (see clause 2.2.4 of Part 1-1) shall be considered.

(2) For buildings of importance categories II - IV (see table 3.3) the verifications prescribed in 4.2 and 4.3 may be considered satisfied, if the following two conditions are met:

a) The total base shear due to the seismic design combination (see clause 4.4 of Part 1-1), calculated with a behaviour factor q = [1,0], 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.

b) The specific measures described in clause 2.2.4 of Part 1-1 are taken, with the exception, that the provisions contained in clause 2.2.4.1.(2)-(3) of Part 1-1 need not be demonstrated as having been met.

4.2 Ultimate limit state

4.2.1 General

(1)P The safety against collapse (ultimate limit state) under the seismic design situation is considered to be ensured if the following conditions regarding resistance, ductility, equilibrium, foundation stability and seismic joints are met.

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 (see 3.5.1.(1)):

$$E_d \leq R_d \qquad (4.1)$$

where

E_d = $E\{\Sigma G_{kj},\ \gamma_I \cdot A_{Ed},\ P_k,\ \Sigma\psi_{2i} \cdot Q_{ki}\}$

design value of the action effect, due to the seismic design situation (see Clause 4.4 of Part 1-1), including - if necessary - second order effects (see (2)),

R_d = $R\{f_k/\gamma_M\}$

corresponding design resistance of the element, calculated according to the rules specific to the pertinent material (characteristic value of property f_k and partial safety factor γ_M) and according to the mechanical models which relate to the specific type of structural system, as given in Part 1-3 and in the relevant Eurocodes.

(2) Second order effects (P-Δ-effects) need not be considered when the following condition is fulfilled in all storeys:

$$\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h} \leq 0{,}10 \qquad (4.2)$$

where

θ interstorey drift sensitivity coefficient,

P_{tot} total gravity load at and above the storey considered, in accordance with the assumptions made for the computation of the seismic action effects,

d_r design interstorey drift, evaluated as the difference of the average lateral displacements at the top and bottom of the storey under consideration and calculated according to 3.4,

V_{tot} total seismic storey shear,

h interstorey height.

(3) In cases when $0{,}1 < \theta \leq 0{,}2$, the second order effects can approximately be taken into account by increasing 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.

4.2.3 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 Part 1-3 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) Capacity design rules are presented in detail in Part 1-3.

4.2.4 Equilibrium condition

(1)P The building structure shall be stable under the set of actions given by the combination rules of clause 4.4 of Part 1-1. Herein are included such effects as overturning and sliding.

(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 clause 4.3.2 of Part 1-1 (see also 3.3.1.(5)-(6)).

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 various lateral load resisting systems to which they are connected.

(2) Paragraph (1) is considered satisfied if for the relevant resistance verifications the forces obtained from the analysis are multiplied by a factor equal to 1,3.

4.2.6 Resistance of foundations

(1)P The foundation system shall be verified according clause 5.4 of Part 5 and to Eurocode 7.

(2)P The action effects for the foundations 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 a behaviour factor $q \leq [1,5]$, no capacity design considerations according to (2)P are required.

4.2.7 Seismic joint condition

(1)P Buildings shall be protected from collisions with adjacent structures induced by earthquakes.

(2) Paragraph (1) is deemed to be satisfied if the distance from the boundary line to the potential points of impact is not less than the maximum horizontal displacement according to eq. (3.12).

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

(4) Alternatively, this separation distance is not required, if appropriate shear walls are provided on the perimeter of the building to act as collision walls ("bumpers"). At least two such walls must be placed at each side subject to pounding and must extend over the total height of the building. They must be perpendicular to the side subject to collisions and they can end on the boundary line. Then the separation distance for the rest of the building can be reduced to [4,0] cm.

4.3 Serviceability limit state

4.3.1 General

(1)P The requirement for limiting damage (serviceability limit state) is considered satisfied, if - under a seismic action having a larger probability of occurrence than the design seismic action - the interstorey drifts are limited according to 4.3.2.

(2) Additional verifications for the serviceability limit state may be required in the case of buildings important for civil protection or containing sensitive equipment.

4.3.2 Limitation of interstorey drift

(1)P Unless otherwise specified in Part 1-3, the following limits shall be observed:

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

$$d_r/\nu \leq [0,004]\cdot h \tag{4.3}$$

b) for buildings having non-structural elements fixed in a way as not to interfere with structural deformations:

$$d_r/\nu \leq [0,006]\cdot h \tag{4.4}$$

where

d_r design interstorey drift as defined in 4.2.2.(2),

h storey height,

ν reduction factor to take into account the lower return period of the seismic event associated with the serviceability limit state.

(2) The reduction factor can also depend on the importance category of the building. Values of ν are given in table 4.1.

Table 4.1: Values of the reduction factor ν

Importance category	I	II	III	IV
Reduction Factor ν	[2,5]	[2,5]	[2,0]	[2,0]

(3) Different values of ν may be required for the various seismic zones of a country.

SECTION 2 SPECIFIC RULES FOR CONCRETE BUILDINGS

2.1 General

2.1.1 Scope

(1)P This section applies to the design of reinforced concrete buildings in seismic regions, henceforth called concrete buildings.

Note 1: In order to make the practical application of this section easier, flowcharts for its use are given in annex A.

Note 2: Additional rules for the design of precast concrete buildings in seismic regions are given in annex B.

Note 3: For buildings with a steel-concrete composite structure see annex D.

Note 4: An annex covering prestressed concrete buildings will be drafted during the ENV-phase of Part 1-3.

(2)P Concrete buildings with flat slab frames used as seismic resistant elements are not covered by this section[1].

(3)P For the design of concrete buildings Eurocode 2 applies. The following rules are additional to those given in Eurocode 2.

2.1.2 Definitions

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

- **Critical region:** Region of a structural element, where the most adverse combination of action-effects (M, N, V, T) occurs and where plastic hinges may form (dissipative zone). The length of the critical region is defined for each structural element in the relevant clause of this section.

- **Residual resistance:** Resistance of a structural element after the cyclic deformation history induced by the most adverse seismic conditions, including degradation.

- **Beam:** Structural element (in general horizontal), subjected mainly to transverse loads and to a normalized design axial force of $\nu_d = N_{Sd}/A_c \cdot f_{cd}$ not greater than 0,1.

---

[1] Because of the essentially non-dissipative behaviour of flat slab frames, additional measures are needed (e.g. the possible combination with other seismic resistant structural systems) and/or additional conditions should be prescribed (such as consideration of the low local ductility available or limitations related to the form and height of the building).

- Column: Structural element (in general vertical), supporting other elements and/or subjected to a normalized design axial force $\nu_d = N_{Sd}/A_c \cdot f_{cd}$ greater than 0,1.

- Wall: Structural element (in general vertical) without perforations in the areas where ductility is requested, supporting other elements and having an elongated cross-section with a length to thickness ratio l_w/b_w greater than 4.

- **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[2]. A minimum torsional rigidity must also be provided (see 2.3.1).

- **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[2]. A minimum torsional rigidity must also be provided (see 2.3.1).

- **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 in part by the frame system and in part by structural walls, single or coupled. A minimum torsional rigidity must also be provided (see 2.3.1).

- **Frame-equivalent dual system:** Dual system in which the shear resistance of the frame system at the building base is higher than 50% of the total shear resistance of the whole structural system[2].

- **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[2].

- **Core system:** Dual or wall system not having a minimum torsional rigidity (see 2.3.1), e.g. a structural system composed of flexible frames combined with walls concentrated near the centre of the building in plan.

  Note: This definition does not cover systems containing several heavily perforated wall arrangements around vertical services and facilities. For such systems the engineer should give the most appropriate definition of the respective overall structural configuration on a case by case basis.

---

[2] Normally the percentages of shear resistance mentioned in 2.1.2 may be substituted by percentages of shear action effects in the seismic design situation.

- **Inverted pendulum system:** System in which 50% or more of its mass is located in the upper third of the height of the structure, or in which the dissipation of energy takes place mainly at the base of one isolated building element.

  Note: One storey frames with column-tops connected along both main directions of the building do not belong to this category.

2.1.3 Design concepts

(1)P The design of earthquake resistant concrete buildings shall provide an adequate energy dissipation capacity to the structure without substantial reduction of its overall resistance against horizontal and vertical loading. To this end, the requirements and criteria of clause 2 of Part 1-1 apply. Adequate resistance of all structural elements shall be provided under the seismic combination of actions, whereas non-linear deformations in critical regions should allow for the overall ductility assumed in calculations.

(2) An overall ductile behaviour is ensured if the ductility demand is spread over a large number of elements and locations per element[3]. To this end, ductile modes of failure (e.g. flexure) should precede brittle failure modes (e.g. shear) with sufficient reliability.

(3)P With regard to the required hysteretic dissipation capacity three ductility classes DC"L" (low ductility), DC"M" (medium ductility) and DC"H"(high ductility) are distinguished for concrete structures:

- DC"L"

  Ductility Class "L" corresponds to structures designed and dimensioned according to Eurocode 2, supplemented by rules enhancing available ductility.

- DC"M"

  Ductility Class "M" corresponds to structures designed, dimensioned and detailed according to specific earthquake resistant provisions, enabling the structure to enter well within the inelastic range under repeated reversed loading, without suffering brittle failures.

---

[3] Obviously, a single structural wall concentrates its plastic region near its base only; however well designed wall systems and dual systems may still generate energy dissipation at several locations.

- DC"H"

Ductility Class "H" corresponds to structures for which the design, dimensioning and detailing provisions are such as to ensure, in response to the seismic excitation, the development of chosen stable mechanisms associated with large dissipation of hysteretic energy.

(4)P In order to provide in the three ductility classes the appropriate amount of ductility, specific provisions for all structural elements shall be satisfied in each class (see 2.6 - 2.11).

(5) In correspondence with the different available ductility in the three ductility classes, different values of the behaviour factor q are used for each class (see 2.3.2).

(6)P In low seismicity-zones (see clause 4.1 of Part 1-1) concrete buildings may be designed under the seismic load combination following only the rules of Eurocode 2 and neglecting the specific provisions given in this Section, provided a specific assessment of the behaviour factor q is made based on the principles of Eurocode 8.

2.2 Materials

2.2.1 Concrete

(1)P The use of concrete class lower than C 16/20 for DC"L" or C 20/25 for DC"M" and DC"H" is not allowed.

2.2.2 Reinforcing steel

(1)P Except for closed stirrups or cross-ties, only ribbed bars are allowed as reinforcing steel in critical regions.

(2)P In critical regions reinforcing steel shall satisfy the additional requirements given in table 2.1.

(3)P Welded meshes are allowed if they observe the conditions specified in paragraphs (1) and (2) above.

Table 2.1: Additional requirements for reinforcing steel in critical regions[4]

<table>
<tr><th colspan="3">PROPERTIES</th><th>DC"L"</th><th>DC"M"</th><th>DC"H"</th></tr>
<tr><td>i</td><td>Uniform elongation at maximum load (charact. value)</td><td>$\epsilon_{su,k}$</td><td rowspan="4">high ductility steel acc. EC 2</td><td>≥ 6%</td><td>≥ 9%</td></tr>
<tr><td>ii</td><td rowspan="2">Tensile strength to yield strength ratio (mean value of the ratio)</td><td rowspan="2">f_t/f_y</td><td>≥ 1,15</td><td>≥ 1,20</td></tr>
<tr><td>iii</td><td>≤ 1,35</td><td>≤ 1,35</td></tr>
<tr><td>iv</td><td>Actual to nominal yield strength ratio</td><td>$\frac{f_{y,act}}{f_{y,nom}}$</td><td>≤ 1,25</td><td>≤ 1,20</td></tr>
<tr><td colspan="6">Comments

(i) and (ii): These rules aim at ensuring an adequate plastic hinge length and high local ductilities (high rotational capacities). In addition, higher $\epsilon_{su,k}$ and f_t/f_y values lead to higher resistances after cover spalling.

(iii) and (iv): These rules are necessary to ensure an economical and reliable control of the desired inelastic mechanisms through the capacity design procedures. If they are not respected, then appropriately higher γ_{Rd}-factors than those given in 2.7 - 2.11 should be used, e.g. by multiplying them with $(f_t/f_y)_{act}/1,35$.</td></tr>
</table>

2.3 Structural types and behaviour factors

2.3.1 Structural types

(1)P Concrete buildings shall be considered to belong to one of the following structural types (see 2.1.2) according to their behaviour under horizontal seismic actions:

- **frame system,**
- dual system (frame or wall equivalent),
- wall system (coupled or uncoupled),

---

[4]Developments are expected in the technology of reinforcing steel production and standardization in Europe, which may result in changes of these provisions.

- core system,

- inverted pendulum system.

(2)P Frame, dual and wall systems shall possess a minimum torsional rigidity.

(3) Such requirement of a minimum torsional rigidity is considered satisfied, if the following condition is fulfilled:

$$r/l_s \geq 0{,}8 \tag{2.1}$$

where

r minimum torsional radius for all relevant horizontal directions (see annex A of Part 1-2),

l_s radius of gyration of the structure in plan (see annex A of Part 1-2).

(4) For frame systems in which all the vertical elements are well distributed in plan the condition set forth in paragraph (2) above may be considered as satisfied without analytical verification.

(5) Frame, dual or wall systems without a minimum torsional rigidity according to exp.(2.1) should be classified as core systems.

2.3.2 Behaviour factors

2.3.2.1 Horizontal seismic actions

(1)P The behaviour factor q, introduced in clause 2.2.2 of Part 1-1 to account for energy dissipation capacity, shall be derived for each design direction as follows:

$$q = q_o \cdot k_D \cdot k_R \cdot k_W \geq 1{,}5 \tag{2.2}$$

where

q_o basic value of the behaviour factor, dependent on the structural type (see paragraph (2)),

k_D factor reflecting the ductility class (see paragraph (6)),

k_R factor reflecting the structural regularity in elevation (see paragraph (8)),

k_W factor reflecting the prevailing failure mode in structural systems with walls (see paragraph (9)).

(2) The basic values q_o for the various structural types are given in table 2.2.

Table 2.2: Basic value q_o of behaviour factor

STRUCTURAL TYPE		q_o
Frame system		5,0
Dual system	frame equivalent	5,0
	wall equivalent, with coupled walls	5,0
	wall equivalent, with uncoupled walls	4,5
Wall system	with coupled walls	5,0
	with uncoupled walls	4,0
Core system		3,5
Inverted pendulum system		2,0

(3)P When both coupled and uncoupled walls are included in the same structural system, the selection of the appropriate value q_o shall be made on the basis of the type of walls resisting the larger part of shear at the base of the building.

(4) The value of q_o given for inverted pendulum systems may be increased provided that the designer proves that a correspondingly higher energy dissipation is ensured in the critical region of the structure.

(5)P In cases where a special and formal Quality System Plan is applied in addition to normal quality control schemes, increased values of q_o may be allowed after a specific decision of the National Authorities. However, the increased values shall not exceed the values given in table 2.2 by more than 20 %.

(6)P The factor k_D reflecting the ductility class shall be taken as follows:

$$k_D = \begin{cases} 1{,}00 & \text{for DC"H"} \\ 0{,}75 & \text{for DC"M"} \\ 0{,}50 & \text{for DC"L"} \end{cases} \qquad (2.3)$$

(7)P The same ductility class shall be adopted for each design direction.

(8)P The factor k_R reflecting the regularity in elevation according to clause 2.2.3 of Part 1-2 shall be taken as follows:

$$k_R = \begin{cases} 1{,}00 & \text{for regular structures} \\ 0{,}80 & \text{for non-regular structures} \end{cases} \qquad (2.4)$$

(9)P The factor k_W reflecting the prevailing failure mode in structural systems with walls shall be taken as follows:

$$k_W = \begin{cases} 1{,}00 & \text{for frame and frame equivalent dual systems} \\ 1/(2{,}5-0{,}5\cdot\alpha_o) \leq 1 & \text{for wall, wall equivalent and core systems} \end{cases} \quad (2.5)$$

where

α_o prevailing aspect ratio of the walls of the structural system ($\alpha_o \doteq$ prev.(H_w/l_w)).

(10) 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 as follows:

$$\alpha_o = \Sigma H_{wi}/\Sigma l_{wi} \quad (2.6)$$

where

H_{wi} height of wall i,

l_{wi} length of the section of wall i.

2.3.2.2 Vertical seismic actions

(1) For the vertical component of the seismic action a behaviour factor q equal to 1,0 should in general be adopted for all structural systems.

(2)P The adoption of q-values greater than 1,0 shall be justified through an appropriate analysis.

2.4 Design criteria

2.4.1 General

(1) The design concepts described in 2.1.3 and in Part 1-1 are implemented into the earthquake resistant structural elements of concrete buildings as specified in 2.4.2 - 2.4.7.

(2) The design criteria given in 2.4.2 - 2.4.7 are deemed to be satisfied, if the provisions given in 2.6 - 2.12 are observed.

2.4.2 Local resistance criterion

(1)P All critical regions of the structure shall have a resistance adequately higher than the corresponding action effects occurring in these regions under the seismic design situation.

(2)P Second order effects shall be taken into account as prescribed in Part 1-2.

SECTION 3 SPECIFIC RULES FOR STEEL BUILDINGS

3.1 General

3.1.1 Scope

(1)P For the design of steel buildings Eurocode 3 applies. The following rules are additional to those given in Eurocode 3.

(2) For buildings with steel-concrete composite structures see the informative annex D.

3.1.2 Definitions

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

- **Cantilever structure:** Structure that may be modelled essentially as a column with a free end.
- **Inverted pendulum structure:** Cantilever structure with most of its mass being located in the upper region of the height.
- **Dual structure:** Structure in which horizontal forces are resisted in part by moment resisting frames and in part by braced frames acting in the same plane.
- **Mixed structure:** Structure consisting of steel moment resisting frames and horizontal load bearing reinforced concrete or masonry infills.

3.1.3 Design concepts

(1)P Earthquake resistant steel buildings shall be designed according to one of the following concepts:

a) **Dissipative structural behaviour**

b) **Non-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 clause 4.2.4 of Part 1-1, the behaviour factor q is taken greater than 1,0. The value of q depends on the structural type (see 3.3).

(3) In concept b) the action effects - regardless of the structural type (see 3.3) - 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 clause 4.2.4 of Part 1-1, the behaviour factor q is taken equal to 1,0.

(4)P For structures designed using concept b) the resistance of the members and of the connections shall be evaluated in accordance with the rules for elastic or plastic resistances presented in Eurocode 3, not having to satisfy the ductility requirements given in 3.5 below.

3.2 Materials

(1)P Steel sections, welding and bolts shall conform to the requirements specified in clause 3 of Part 1-1 of Eurocode 3, unless specified otherwise hereafter.

(2) In dissipative zones the following additional rules apply:

a) Structural steel should conform to EN 10025.

b) In bolted connections high strength bolts in category 8.8 or 10.9 should be used in order to comply with the needs of capacity design (see 3.5.3.2). Bolts of category 12.9 are only allowed in shear connections.

c) The value of the yield strength which cannot be exceeded by the material used in the fabrication of the structure and the tensile strength of the steel used should be specified.

3.3 Structural types and behaviour factors

3.3.1 Structural types

(1)P Steel buildings shall be assigned to one of the following structural types according to their behaviour under seismic actions (see fig. 3.1):

a) **Moment resisting frames**, which resist horizontal forces acting in an essentially flexural manner. In these structures the dissipative zones are mainly located in plastic hinges near the beam-column joints and energy is dissipated by means of cyclic bending.

b) **Concentric braced frames**, in which the horizontal forces are mainly resisted by members subjected to axial forces. In these structures the dissipative zones are mainly located in the tensile diagonals. Concentric braced frames are subdivided into the following three 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 considering both tension and compression diagonals. The intersection point of these diagonals lies on a horizontal member which must be continuous.
- **K-bracings**, in which the diagonal intersection lies on a column. This category must not be considered as dissipative when the yielding mechanism involves yielding of the column.

c) **Eccentric braced frames**, in which the horizontal forces are mainly resisted by axially loaded members and the eccentricity of the layout is such that energy can be dissipated in the beams by means of either cyclic bending or cyclic shearing. Eccentric braced frames can only be classified as dissipative, when yielding due to bending or shear of the bending members precedes the attainment of the limit strengths of the tension or compression members.

d) **Cantilever structures or inverted pendulum structures**, as defined in 3.1.2 and in which dissipative zones are mainly located at the base.

e) **Structures with concrete cores or concrete walls**, in which the horizontal forces are mainly resisted by these cores or walls.

f) **Dual structures**, as defined in 3.1.2.

g) **Mixed structures**, as defined in 3.1.2

3.3.2 Behaviour factors

(1)P The behaviour factor q, introduced in clause 4.2.4 of Part 1-1 to account for energy dissipation capacity, takes the values given in fig. 3.1, provided the regularity requirements laid down in Part 1-2 and the detailing rules given in 3.5 are met.

(2) If the building is non-regular in elevation (see clause 2.2.3 of Part 1-2) the q-values listed in fig. 3.1 should be reduced by 20% (but need not be taken less than $q = 1,0$).

(3) For regular buildings in zones of low seismicity (see clause 4.1 (4) of Part 1-1) having structural systems made from rolled sections or from welded sections with similar size as rolled sections and conforming to the structural types listed in fig.3.1, a behaviour factor q equal to 1,5 may be adopted (the K-bracing excepted) without taking account of the detailing rules given in 3.5. The no-collapse requirement set forth in Part 1-1 may then be deemed to be satisfied by the resistance verifications according to Eurocode 3[1].

(4) The following parameters are used in fig. 3.1:

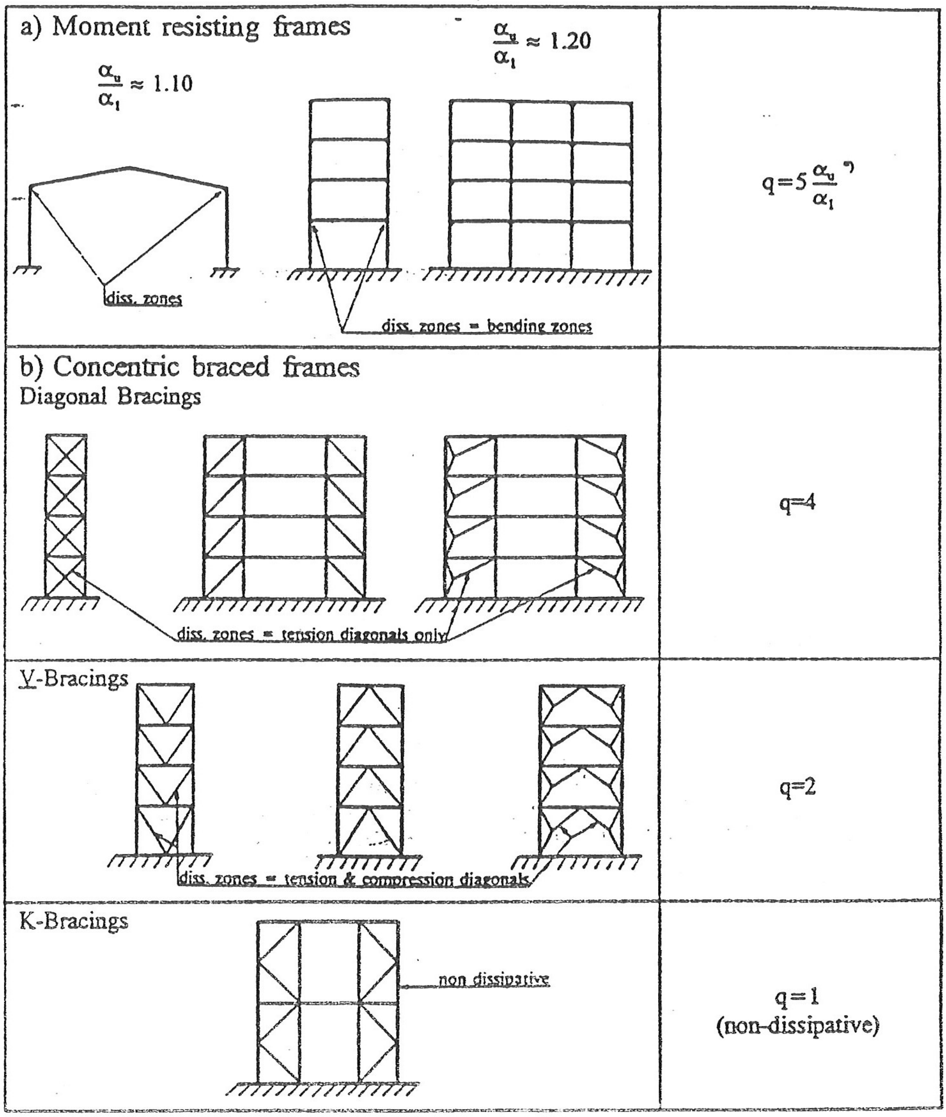
α_1: multiplier of the horizontal design seismic action, while keeping constant all other design actions, which corresponds to the point where the most strained cross-section (taking into account locations of stiffeners) reaches its plastic moment resistance.

---

[1]This allows the earthquake resistant design of steel structures in zones of low seismicity to be similar to the design for wind resistance. It avoids the use of special measures for achieving ductility, when the wind combination may be the more critical design combination. This procedure is justified by the fact that all steel structures, designed according to Eurocode 3, exhibit a certain amount of ductility beyond yielding, as they reach their limit states.

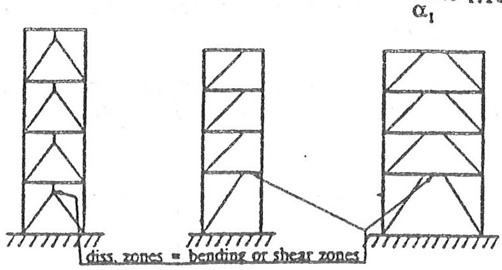
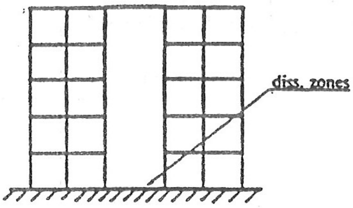
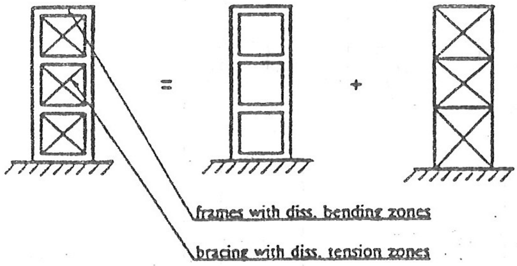
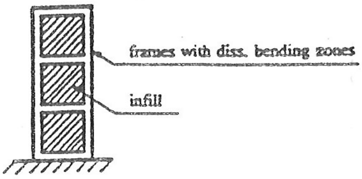
α_u multiplier of the horizontal design seismic action, while keeping constant all other design actions, which corresponds to the point where a number of sections, sufficient for the development of overall structural instability, reach their plastic moment resistance.

(5) When calculations are not performed in order to evaluate the multiplier α_u, the approximate values of the ratio α_u/α_1 presented in fig. 3.1 may be used.



*) $\frac{\alpha_u}{\alpha_1}$ should be limited to 1.6.

Figure 3.1: Structural types and behaviour factors (cont.)

<table>
<tr><td>c) Eccentric braced frames $\frac{\alpha_u}{\alpha_1} \approx 1.10$

</td><td>$q = 5\,\frac{\alpha_u}{\alpha_1}$ *)</td></tr>
<tr><td>d) Cantilever structures
Restrictions: $\bar{\lambda} \leq 1.5$; $\Theta \leq 0.2$ (see clause 3.5.7)</td><td>q=2</td></tr>
<tr><td>e) Structures with concrete cores or concrete walls

</td><td>see section 2</td></tr>
<tr><td>f) Dual structures

</td><td>$q = 5\,\frac{\alpha_u}{\alpha_1}$ *)</td></tr>
<tr><td>g) Mixed structures (Steel moment resisting frames with reinforced concrete or masonry infills)

</td><td>q=2</td></tr>
</table>

*) $\frac{\alpha_u}{\alpha_1}$ should be limited to 1.6.

Figure 3.1: Structural types and behaviour factors (concl.)

SECTION 4 SPECIFIC RULES FOR TIMBER BUILDINGS

4.1 General

4.1.1 Scope

(1)P For the design of timber buildings Eurocode 5 applies. The following rules are additional to those given in Eurocode 5.

4.1.2 Definitions

(1)P The following terms are used in this chapter 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 4.3.(4)P).

- **Semi-rigid joints:** Joints with significant flexibility, the influence of which has to be considered in structural analysis according to Eurocode 5 (e.g. dowel-type joints).

- **Rigid joints:** Joints with negligible flexibility according to Eurocode 5 (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).

4.1.3 Design concepts

(1)P Earthquake-resistant timber buildings shall be designed according to one of the following concepts:

a) **Dissipative structural behaviour**

b) **Non-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 clause 4.2.4 of Part 1-1, the behaviour factor q is taken greater than 1,0. The value of q depends on the structural type (see 4.3).

(3)P Dissipative zones shall be regarded as located in joints and connections with mechanical fasteners, whereas the timber members themselves shall be regarded as behaving elastically.

(4) The properties of dissipative zones shall be determined by tests either on single joints, on whole structures or on parts thereof according to EN XX[1]. Provisions for avoiding such tests are given in 4.3.(5).

(5) In concept b) the action effects - regardless of the structural type - 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 clause 4.2.4 of Part 1-1, the behaviour factor q is taken equal to 1,0.

4.2 Materials and properties of dissipative zones

(1)P Clauses 3, 6 and 7 of Part 1-1 of Eurocode 5 apply. With respect to the properties of steel elements, clause 3 of Part 1-1 of Eurocode 3 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. The decision on their use shall be based on appropriate test results.

(3) Paragraph (2) a) is deemed to be satisfied if the following is met:

When tested according to EN XX[1] joints shall be verified to have appropriate low cycle fatigue properties under large amplitudes to ensure a sufficient ductility in respect to their intended deformational mechanism and to justify the q-value assumed in the analysis (see 4.3.(4)).

(4) For sheathing-material in diaphragms, paragraph (2) 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.

---

[1]At the time of publication of this ENV no related EN exists. In the meantime reference should be made to agreed international recommendations (e.g. RILEM - TC 109 TSA).

(5)P Steel material for connections shall comply with the following conditions:

a) All connection elements made of cast steel shall fulfill the relevant requirements in Eurocode 3 and in section 3 of this Part.

b) The ductility properties of the connections between the sheathing material and the timber framing in type C and D structures (see 4.3) shall be tested for compliance with 4.3.(4) by cyclic tests on the relevant combination of sheathing material and fastener.

4.3 Structural types and behaviour factors

(1)P According to their ductile behaviour and energy dissipation capacity under seismic actions timber buildings shall be assigned to one of the four types A - D given in table 4.1 with the corresponding behaviour factors.

Table 4.1: Structural types and behaviour factors

Type	Description	Behaviour Factor q
A	Non-dissipative structures	1,0
B	Structures having low capacity to dissipate energy	1,5
C	Structures having medium capacity to dissipate energy	2,0
D	Structures having good capacity to dissipate energy	3,0

(2) Examples of structural systems belonging to these different types are given in fig. 4.1.

(3) If the building is non-regular in elevation (see clause 2.2.3 of Part 1-2) the q-values listed in table 4.1 should be reduced by 20% (but need not be taken less than q = 1,0).

(4)P In order to ensure that the given values of the behaviour factor can 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 type B structures and 6 for type C and D structures without more than a 20% reduction of their resistance.

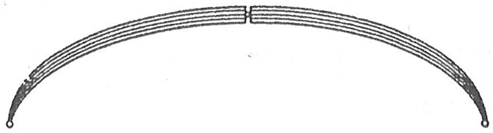
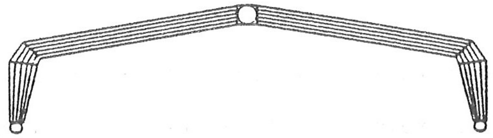
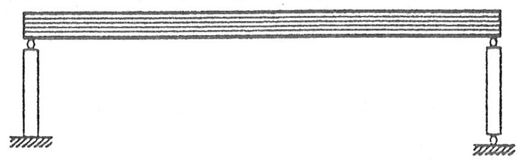
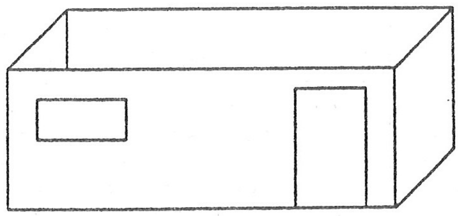
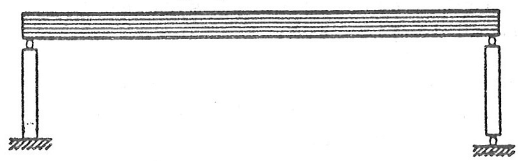
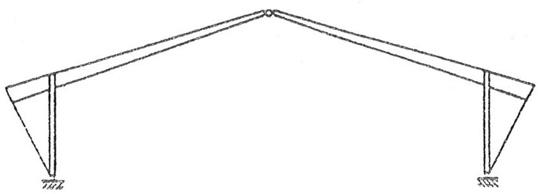
Type	Description	q	Examples
A	**Non-dissipative structures** having only few mechanical fasteners beyond the dissipative zones	1,0	Arches with hinged joints   Cantilever structures with rigid connections at the base  Buildings with vertical diaphragms resisting the horizontal forces without mechanical fasteners for both interconnection and between sheathing and timber framing. 
B	**Structures having low capacity of energy-dissipation**	1,5	Structures with semi-rigidly fixed-based columns  Structures with few but effective dissipative zones 

Figure 4.1: Examples of structural systems (continued)

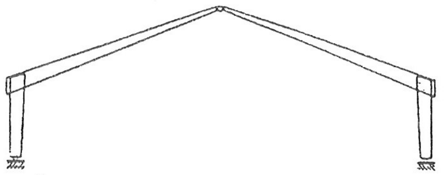
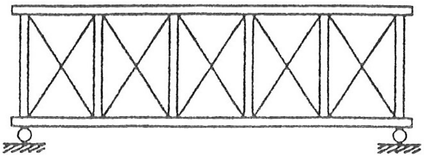
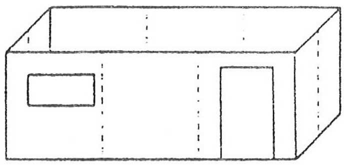
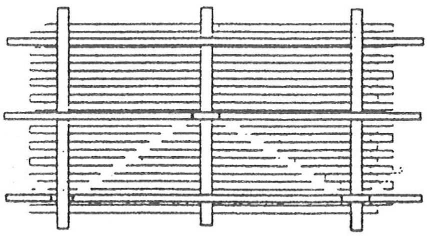
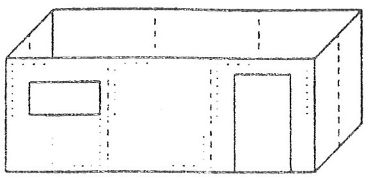
Type	Description	q	Examples
C	**Structures having medium capacity of energy-dissipation**	2,0	Frames or beam-column structures with semirigid joints between all members. Connections with foundations may be semirigid or hinged (according to the load-carrying system)  Braced frame structures with mechanical fasteners in the joints of the frame and/or the connections of the bracing elements  Buildings with vertical diaphragms resisting the horizontal forces, where sheathing is glued to the framing. Diaphragms are interconnected by mechanical fasteners (horizontal diaphragms may be glued or nailed)  Mixed structures consisting of timber framing (resisting the horizontal forces) and non-load-bearing infillment 
D	**Structures having good capacity of energy-dissipation**	3,0	Buildings with vertical diaphragms resisting the horizontal forces, where sheathing is fixed to the framing by mechanical fasteners as well as the interconnection of the wall-elements (horizontal diaphragms may be glued or nailed) 

Figure 4.1: Examples of structural systems (concluded)

(5) The provisions of paragraph (4) above and of 4.2.(2) a) and 4.2.(5) b) may be regarded as satisfied in the dissipative zones of all structural types if the following provisions are met:

a) In doweled and nailed timber-to-timber and steel-to-timber joints the minimum thickness of the connected members is $8 \cdot d$ and the dowel-diameter does not exceed 12 mm.

b) In the connection of sheathing to the timber framing of diaphragms the sheathing material is wood-based and the minimum thickness of the sheathing material is $4 \cdot d$, where d does not exceed 3,0 mm.

(6) Behaviour factors different from those presented in table 4.1 may be used for specific structures, if their validity is demonstrated on the base of analytical simulations and tests under a representative number of earthquakes (see clause 4.3.2 of Part 1-1).

(7) For structures having different and independent properties in the x- and y-directions (see fig. 4.2), different q-factors may be used for the calculation of the seismic action effects in each main direction.

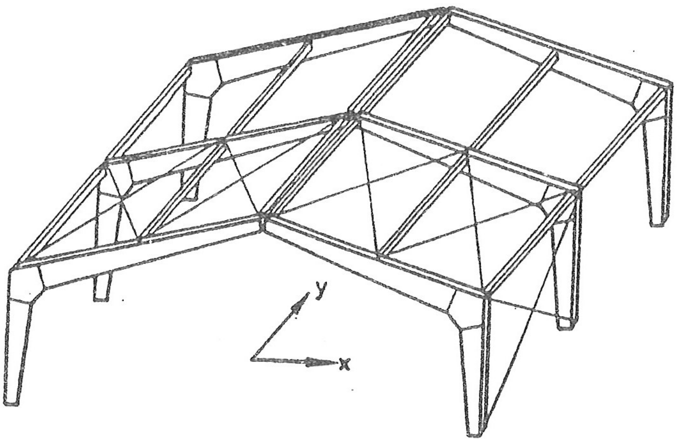


Figure 4.2: Example of a structure with different behaviour in the main directions (type A in x- and type C in y-direction)

4.4 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 rigid in the structural model without further verification, if

SECTION 5 SPECIFIC RULES FOR MASONRY BUILDINGS

5.1 Scope

(1)P This section applies to the design of buildings in unreinforced, confined and reinforced masonry in seismic regions.

(2)P For the design of masonry buildings Eurocode 6 applies. The following rules are additional to those given in Eurocode 6.

5.2 Materials and bonding patterns

5.2.1 Types of masonry units

(1)P In order to avoid local brittle failures units with formed holes shall meet the following requirements:

a) the units have not more than [50] % by volume of formed holes,

b) the minimum thickness of shells is [15] mm,

c) vertical webs in hollow and cellular units extend over the entire horizontal length of the unit.

(2) Types of units different from the above can be used in reinforced masonry systems provided the prescribed tests (see 5.5.5) prove that the ductility requirements to the walls inherent to the system are met.

(3) In zones of low seismicity (see clause 4.1.(4) of Part 1-1) National Authorities may allow the use of units different from the above.

5.2.2 Minimum strength of masonry units

(1)P The normalized compressive strength of masonry units, derived in accordance with clause 3.1.2.1 of Part 1-1 of Eurocode 6, shall not be less than the following values:

- normal to the bed face: $f_b = 2,5$ N/mm²
- parallel to the bed face in the plane of the wall: $f_{bh} = [2,0]$ N/mm²

5.2.3 Mortar

(1)P For unreinforced and confined masonry only mortars type M5 or stronger are allowed.

(2) Depending on the seismicity of the region and the importance of the building for unreinforced and confined masonry National Authorities may allow the use of mortars of lower resistance.

(3)P For reinforced masonry only mortars type M10 or stronger are allowed.

5.2.4 Masonry bond

(1)P Except in zones of low seismicity perpend joints shall be fully filled with mortar.

5.3 Types of construction and behaviour factors

(1) The low tensile strength and low ductility of unreinforced masonry impose the limitation of its use in areas of high seismicity. However, its association with reinforcing steel can provide higher ductility and limit the strength degradation under cyclic actions. Such improved properties may be taken into account in the design.

(2)P Depending on the masonry type used for the seismic resistant elements masonry buildings shall be assigned to one of the following types of construction:

a) **unreinforced masonry construction,**

b) **confined masonry construction,**

c) **reinforced masonry construction,**

d) **construction with reinforced masonry systems.**

(3) For types a) to c) the values of the behaviour factor q are given in table 5.1.

Table 5.1: Types of construction and behaviour factors

Type of construction	Behaviour factor q
Unreinforced masonry	[1,5]
Confined masonry	[2,0]
Reinforced masonry	[2,5]

(4)P For buildings constructed with reinforced masonry systems the values of the behaviour factor q shall be derived from the results of the ductility tests referred to in 5.5.5.

2. BASIC REQUIREMENTS AND COMPLIANCE CRITERIA

2.1 Design seismic event

(1)P The design philosophy of this Code, regarding the seismic resistance of bridges, is based on the general requirement that emergency communications shall be maintained, with appropriate reliability, after the design seismic event.

(2) Target reliabilites are selected on the basis of Clause 2.1 (2) of Part 1.1 (see also Annex A of this Part).

(3) Differentiation of target reliability may, in the absence of reliable statistical evaluation of seismological data, be obtained by mulitiplying the design seismic action with an importance factor γ_I having the following values:

Bridge Importance Category	Importance Factor γ_I
Greater than average	[1.30]
Average	1.00
Less than average	[0.70]

(4) To the category of "greater than average" importance belong bridges of critical importance for maintaining communications, especially after a disaster, bridges whose failure is associated with a large number of probable fatalities, and major bridges for which a design life greater than normal is required.

(5) To the category of "less than average" importance belong bridges which are not critical for communications and for which the adoption of either the standard probability of exceedence of the design seismic event or the normal bridge design life, is not economically justifiable.

(6) Recommendations for the selection of the design seismic event, appropriate for use during the construction period of bridges are given in Annex A.

2.2 Basic requirements

(1)P With reference to the probability of occurrence of an earthquake event in the design life of the bridge, the following two basic requirements are defined.

2.2.1 Non-collapse requirement (ultimate limit state)

(1)P After the occurrence of the design seismic event, the bridge shall retain its structural integrity and adequate residual resistance, although at some parts of the bridge considerable damage may occur.

(2)P The bridge shall be damage-tolerant, i.e. those parts of the bridge susceptible to damage by their contribution to energy dissipation during the design seismic event shall be designed in such a manner as to ensure that the structure can sustain the actions from emergency traffic, and inspections and repair can be performed easily.

(3)P To this end, flexural yielding of specific sections (i.e. the formation of plastic hinges) is allowed in the piers, and is in general necessary, in regions of high seismicity, in order to reduce the design seismic action to a level requiring reasonable additional construction costs.

(4)P The bridge deck however shall in general be protected from the formation of plastic hinges and from unseating under extreme seismic displacements.

2.2.2 Minimisation of damage (serviceability limit state)

(1)P After seismic actions with high probability of occurrence during the design life of the bridge, the parts of the bridge intended to contribute to energy dissipation during the design seismic event, shall only undergo minor damage without giving rise to any reduction of the traffic or the need of immediate repair.

2.3 Compliance criteria

2.3.1 General

(1)P In order to satisfy the basic requirements set forth in Clause 2.2, the design must comply with the criteria outlined in the following Clauses. In general the criteria while aiming explicitly at satisfying the non-collapse requirement (2.2.1), implicitly cover the damage minimization requirement (2.2.2) as well.

(2)P The compliance criteria depend on the behaviour which is intended for the bridge under the design seismic action. This behaviour may be selected according to the following Clause.

2.3.2 Intended seismic behaviour

(1)P The bridge shall be designed so that its behaviour under the design seismic action is ductile, or limited ductile/essentially elastic, depending on the characteristics of the global force-displacement relationship of the structure (see Fig. 2.1)

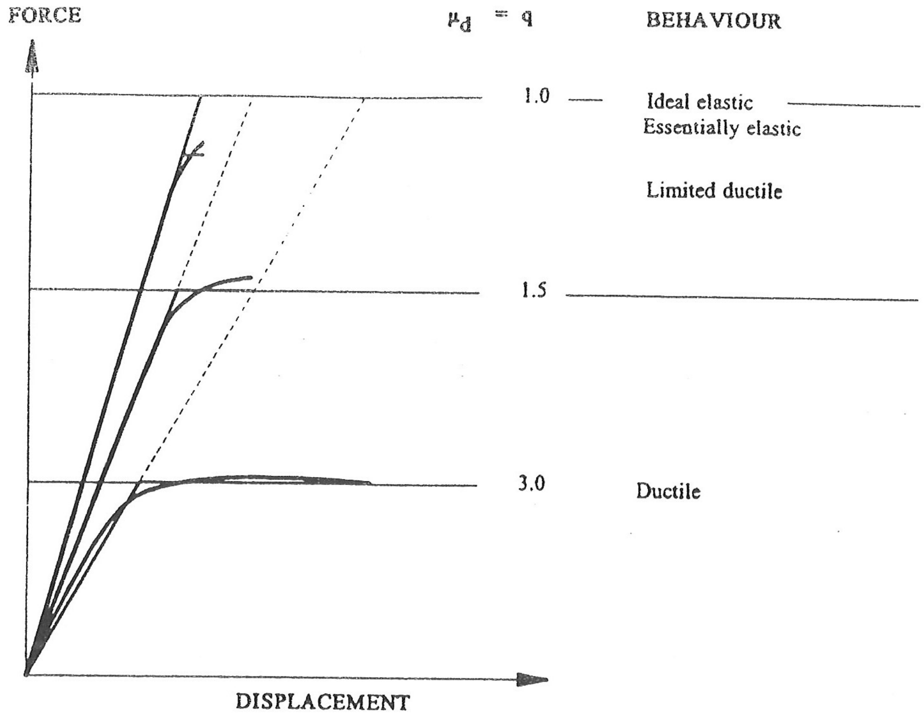


Figure 2.1
Seismic behaviour

2.3.2.1 Ductile behaviour

(1)P 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 according to Section 7. The part of this subclause that follows refers to ductile behaviour achieved by flexural plastic hinges.

(2)P The bridge shall be designed so that a dependably stable plastic mechanism can form in the structure through the formation of flexural plastic hinges, normally in the piers, which act as the primary energy dissipating components.

(3)P As far as possible the location of plastic hinges shall be selected at points accessible for inspection and repair. In general the bridge deck shall remain within the elastic range.

(4)P The formation of plastic hinges is not allowed in reinforced concrete sections where the normalized axial force η_k defined in 5.3.(3) exceeds 0.6.

(5)P The global force-displacement relationship shall show a significant force plateau at yield and shall be reversible in order to ensure hysteretic energy dissipation at least over 5 deformation cycles. (See Figures 2.1, 2.2 and 2.3).

Note
Elastomeric bearings used over some supports may cause some increase of the resisting force, with increasing displacements, after plastic hinges have formed in the other supporting elements. However the rate of increase of the resisting force should be appreciably reduced after the formation of plastic hinges.

(6) Flexural 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.

(7) Supporting elements (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.

(8) It is pointed out that the formation of flexural hinges is necessary in order to ensure energy dissipation and consequently ductile behaviour (See Clause 4.1.6 (2)). The deformation of the usual elastomeric bearings is mainly elastic and does not lead to ductile behaviour. (See Clause 4.1.6 (10)) When no plastic hinge develops, no ductile behaviour can be considered.

2.3.2.2 Limited ductile/essentially elastic behaviour

(1)P No significant yield appears under the design earthquake. 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 a behaviour corresponds to a behaviour factor $q \leq 1.5$ and shall be referred to, in this Code, as "limited ductile".

2.3.3 Resistance verifications

(1)P In bridges of ductile behaviour the regions of plastic hinges shall be verified to have adequate flexural strength to resist the design seismic effects as defined in Clause 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" defined in Clause 2.3.4 below (see also Clause 5.3).

(2)P In bridges of limited ductile behaviour all sections shall be verified to have adequate strength to resist the design seismic effects.

2.3.4 Capacity design

(1)P For bridges of ductile behaviour, capacity design shall be used to secure the hierarchy of resistances of the various structural components necessary for leading to the intended configuration of plastic hinges and for avoiding brittle failure modes.

(2)P This shall be effected by designing all elements intended to remain elastic for "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 defined in Clause 5.3.

(3)P For bridges of limited ductile behaviour the application of the capacity design procedure is not compulsory.

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 structure ductility.

Note

The definitions of structure and local ductilities, given in subclauses 2.3.5.2 and 2.3.5.3 that follow, are intended to provide the theoretical basis of ductile behaviour. In general they are not required for practical verification of ductility, which is effected according to subclause 2.3.5.4.

2.3.5.2 Structure ductility

(1)P Referring to an equivalent one degree of freedom system, having an idealized elasto-plastic force-displacement relationship, as shown in Figure 2.2., the design value of the ductility of the structure (available displacement ductility) 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)P The constant maximum force of the global elasto-plastic diagram is assumed equal to the design resisting force F_{Rd}. The yield displacement defining the elastic branch is selected so as to best approximate the design curve (for repeated loading) up to F_{Rd}.

(3)P The ultimate displacement d_u is defined as the maximum displacement satisfying the following condition. The structure is capable to sustain at least 5 full cycles of deformation to the ultimate displacement,

- without initiation of failure of the confining reinforcement for R.C. sections, or local buckling effects for steel sections, and

- without drop of the resisting force for steel ductile elements or a drop exceeding 0.20 F_{Rd} for R.C. ductile elements (see Figure 2.3).

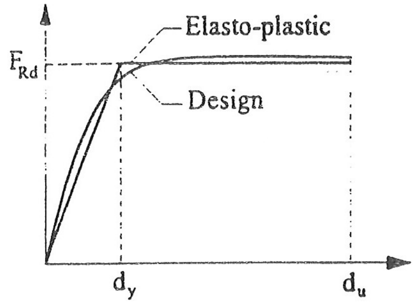


Figure 2.2
Global force-displacement diagram
(Skeleton curve)

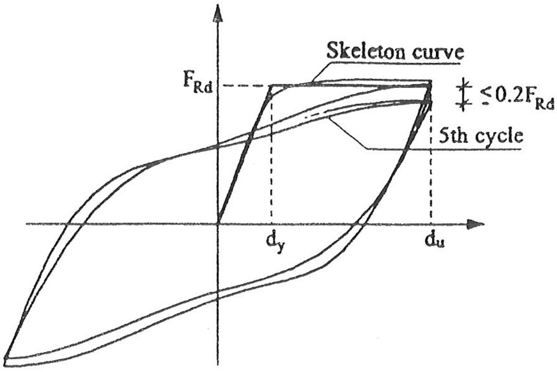


Figure 2.3
Force-displacement cycles

2.3.5.3 Local ductility at the plastic hinges

(1)P The structure ductility depends on the available local ductility at the plastic hinges (See Figure 2.4) expressed as curvature ductility of the cross-section:

$$\mu_c = C_u/C_y \tag{2.2}$$

or as rotation ductility of the hinge:

$$\mu_\varphi = \varphi_u/\varphi_y = 1+(\varphi_u-\varphi_y)/\varphi_y = 1+R \tag{2.3}$$

R is the rotation capacity of the plastic hinge

(2)P In the above expressions the ultimate deformations are defined subject to the conditions of 2.3.5.2 (3).

2.3.5.4 Ductility verification

(1)P Compliance with the Specific Rules given in Section 6 shall be deemed, in general, to ensure the availability of adequate local and overall structure ductility.

(2)P In special cases the assumed ductility may be directly verified on the basis of the available curvature ductilities and the lengths of the plastic hinges, or the rotation ductilities (See Annex B).

(3)P When non-linear dynamic analysis is performed, ductility demands shall be checked against available local ductility capacities of the plastic hinges.

(4)P For bridges of limited ductile behaviour the provisions of Clause 6.5 must be applied.

2.3.6 Connections - Control of displacements - Detailing

2.3.6.1 Design seismic displacement - Effective stiffness

(1)P Within the framework of equivalent linear analysis methods allowed by the present code, the stiffness of each element shall adequately approximate its deformation under the maximum stresses induced by the design seismic action. For elements containing plastic hinges this corresponds to the secant stiffness at the theoretical yield point. (See Figure 2.4).

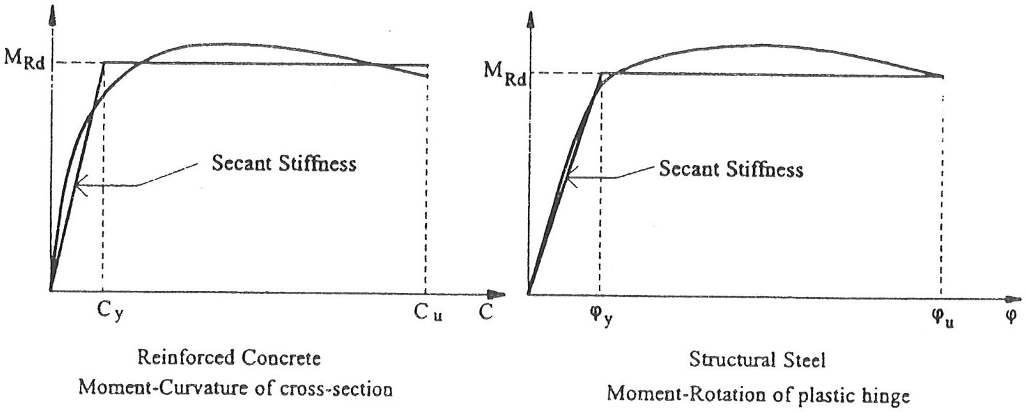


Figure 2.4
Moment - deformation diagrams at plastic hinges

(2) For reinforced concrete elements in bridges of ductile behaviour, in the absence of a more rigorous method , the effective stiffness may be estimated as follows:

- For ductile components (piers), a value calculated on the basis of the secant stiffness at the theoretical yield point of the plastic hinge (See Annex C).
- For reinforced and/or prestressed concrete components remaining within the elastic range, the stiffness of the uncracked sections.

(3)P In limited ductile bridges the stiffness of the uncracked sections shall be used globally.

Note
It is noted that an overestimation of the effective stiffness leads to results which are on the safe side regarding the seismic actions. In such a case, only the displacements need be corrected, after the analysis, on the basis of the resulting level of actual stresses (see Annex C). On the other hand, if the initial assumption of effective stiffness is significantly lower than that corresponding to the actual stresses, the analysis must be repeated using a better approximation of the effective stiffness.

(4)P The displacements d_{Ee} determined from linear seismic analysis, static or dynamic, shall be multiplied by the q-factor (behaviour factor) used in the analysis (see 4.1.6), to obtain the design seismic displacements d_E.

$$d_E = \pm\, q d_{Ee} \qquad (2.4)$$

(5)P When the fundamental period of the bridge T is less than $T_o=1.5T_C$, where T_C has the values given by Table 4.1 of EC8: Part 1.1 (see also 3.2.2.2.1 (5)), the design seismic displacement shall be estimated as follows:

$$d_E = \pm\, \mu_d d_{Ee} \qquad (2.5)$$

where the displacement ductility μ_d shall be estimated as

$$\mu_d = (q-1)\frac{T_o}{T} + 1 \qquad (2.6)$$

and T is the fundamental period of the bridge in the direction under consideration.

(6)P When non-linear time domain analysis is used, the deformation characteristics of the yielding elements shall adequately approximate their actual post-elastic behaviour, both in the loading and unloading branches of the hysteresis loops, as well as the eventual degradation effects.

2.3.6.2 Connections

(1)P Connections between supporting and supported elements shall be adequately designed in order to secure structural integrity and avoid unseating under extreme seismic displacements.

(2)P Bearings, links and holding-down devices used for securing structural integrity, shall be designed using capacity design effects, (see Clauses 5.3 and 6.6.2.1). Appropriate overlap lengths shall be provided between supporting and supported elements at moveable connections, in order to avoid unseating.

(3) Alternatively, positive linkage between connected elements may be used, (see Clause 6.6.3) as a second line of defense beyond the design seismic actions and displacements.

2.3.6.3 Control of displacements - Detailing

(1)P In addition to ensuring a satisfactory overall ductility, structural and non- structural detailing must ensure adequate behaviour of the bridge and its components under the design seismic displacements.

(2)P Adequate clearances shall be provided for protection of critical or major structural elements. Such clearances shall accommodate the total design value of the displacement under seismic conditions d_{Ed} determined as follows:

$$d_{Ed} = d_E + d_G \pm d_{Ts} \qquad (2.7)$$

where:

d_E is the design seismic displacement according to equation (2.4)

d_G is the displacement due to the permanent and quasi-permanent actions measured in long term (e.g. post-tensioning, shrinkage and creep for concrete decks).

d_{Ts} is the displacement due to thermal movements, corresponding to a representative value T_s of the temperature variation considered appropriate for combination with seismic effects, to be defined by the relevant Part of EC1. Until such a definition is available the following estimation may be used:

$$d_{Ts}=[0.4]d_T$$

where d_T is the design displacement due to thermal movements.

The total design seismic displacement shall be increased by the displacement due to second order effects when such effects have a significant contribution.

(3) The relative 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 calculated for each section.

(4)P Large shock forces, caused by unpredicted impact between major structural elements, shall be prevented by means of ductile/resilient elements or special energy absorbing devices (buffers). Such elements shall have a slack at least equal to the total design displacement d_{Ed}

(5)P At joints of railway bridges transverse differential displacement shall be either avoided or limited to values appropriate for preventing derailments.

(6) The detailing of non-critical structural elements (e.g. deck movement joints), expected to be damaged during the design seismic event, should cater, as far as possible, for a predictable mode of damage and provide for the possibility of permanent repair. Relevant clearances should accommodate appropriate fractions of the design seismic displacement and thermal movement, after allowing for any long term creep and strinkage effects, so that damage under frequent earthquakes can be avoided.

(7) The appropriate values of such fractions depend on techno-economical considerations. In the absence of an explicit optimization the following values are recommended:

[40%] of the design seismic displacement
[50%] of the thermal movement

2.3.7 Simplified criteria

(1) In regions of low or moderate seismicity ($a_g \leq [0.10g]$), the National Application Documents may establish an appropriate classification of the bridges and specify simplified compliance criteria, pertaining to the individual classes. These simplified criteria may be 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 seismic effects at the conceptual stage of the design of bridges is important even in low to moderate seismicity regions.

(2) In areas of low to moderate seismicity ($a_g \leq [0.10g]$) the type of intended seismic behaviour of the bridge (see Clause 2.3.2) must be decided. If a limited ductile (or essentially elastic) behaviour is selected, the requirements of the following Clauses should be applied:

- Clause 6.5, regarding the accessibility of potential plastic hinges.
- Clause 6.6, regarding the design of bearings and links and the required seating lengths.

(3) In areas of moderate to 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 base isolation and energy dissipating devices, must be decided. When a ductile behaviour is selected the following main points should be considered:

(4) The number of supporting elements (piers and abutments) that will be used to resist the seismic forces in the longitudinal and the transverse direction must be decided. In general continuous structures behave better under earthquake conditions than bridges having 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 earthquake resisting piers may have to be reduced, by using flexible mountings between deck and piers in one or in both directions, to avoid either high reactions due to restrained deformations or an undesirable distribution of the seismic actions and/or of the capacity design effects. (See also (6) below)

(5) A balance should be maintained between the strength and the flexibility requirements of the horizontal supports. High flexibility reduces the level of 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 continuous deck in which the transverse stiffness of the abutments and the adjacent piers is very high in relation to that of the other piers (as may occur in steep sided valleys), a very unfavourable distribution of the transverse seismic action on these elements may take place, as shown in figure 2.5. In such a case it may be preferable to use transversally moveable or flexible bearings over the abutments or the short piers.

(7) The location of the energy dissipating points must be chosen so as to ensure their accessibility for inspection and repair.

(8) The location of other potential or expected damage areas under severe motions must be identified and the difficulty of repairs must be minimised.

(9) In exceptionally long bridges, or in bridges crossing non-homogeneous soil formations, the number and location of intermediate movement joints must be decided.

(10) In the case of bridges going over 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 must be investigated according to the relevant provisions of EC8: Part 5.

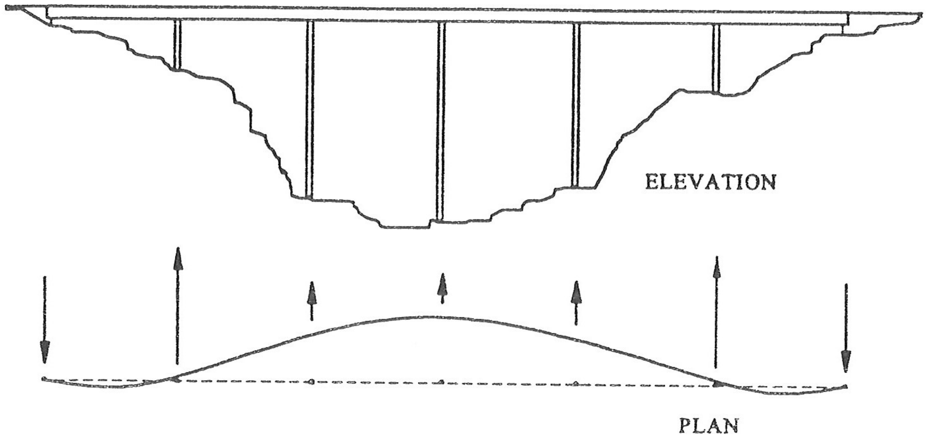


Figure 2.5
Unfavourable distribution of transverse seismic action