2 PERFORMANCE REQUIREMENTS AND COMPLIANCE CRITERIA

2.1 Fundamental requirements

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

– No-collapse requirement:

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

Note 1: The values to be ascribed to P_{NCR} or to T_{NCR} for use in a Country may be found in its National Annex The recommended values are P_{NCR} =10% and T_{NCR} = 475 years.

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

– Damage limitation requirement:

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

Note 3: The values to be ascribed to P_{DLR} or to T_{DLR} for use in a Country may be found in its National Annex. The recommended values are P_{DLR} =10% and T_{DLR} = 95 years.

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

(3)P Reliability differentiation is implemented by classifying structures into different importance classes. To each importance class 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 (with regard to the reference return period), as appropriate for the design of the specific category of structures (see 3.2.1(3)).

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

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

2.2 Compliance Criteria

2.2.1 General

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

– Ultimate limit states

  are those associated with collapse or with other forms of structural failure which may endanger the safety of people.

– Damage limitation 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 and to promote a good behaviour of structures under seismic actions more severe than the design one, a number of pertinent specific measures shall also be taken (see 2.2.4).

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

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

(5) Specific rules for "simple masonry buildings" are given in Section 9. 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 energy-dissipation capacity specified in the relevant Parts of EN 1998.

(2) The resistance and energy-dissipation capacity to be assigned to the structure are related to the extent to which its non-linear response is to be exploited. In operational terms such balance between resistance and energy-dissipation capacity is characterised by the values of the behaviour factor q and the associated ductility classification, which are given in the relevant Parts of EN 1998. As a limiting case, for the design of structures classified as non-dissipative, no account is taken of any hysteretic energy dissipation and the behaviour factor may be taken not greater than the value of 1,5, considered to account for overstrengths. For dissipative structures the behaviour factor is taken greater than 1,5 accounting for the hysteretic energy dissipation that mainly 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 EN 1998.

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

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

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

2.2.3 Damage limitation state

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

(2)P In structures important for civil protection the structural system shall be verified to 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 4.2.3). If necessary this may be realised by subdividing the structure by joints into dynamically independent units.

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

(3)P Since the seismic performance of a structure is largely dependent on the behaviour of its critical regions or elements, the detailing of the structure in general and of these regions or elements in particular, shall be such as to maintain 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)P The analysis shall be based on an adequate structural model, which, when necessary, shall take into account the influence of soil deformability and of non-structural elements and other aspects, such as the presence of adjacent structures.

2.2.4.2 Foundations

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

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

2.2.4.3 Quality system plan

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

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

(3) In regions of high seismicity and 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 AND SEISMIC ACTION

3.1 Ground conditions

(1)P Appropriate investigations shall be carried out in order to identify the ground conditions according to the types given in 3.1.1.

(2) Further guidance concerning soil investigation and classification is given in clause 4.2 of EN 1998-5:200X.

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

(4) If the value of the importance factor γ_I defined in 2.1(3)P is less than 1,0, ground investigations additional to those necessary for the design for non-seismic actions may be omitted. In this case and in the absence of more accurate information on soil conditions, the seismic action may be determined assuming ground conditions according to ground type B (see 3.1.1).

3.1.1 Identification of ground types

(1)P The influence of local ground conditions on the seismic action shall generally be accounted for by considering the five ground types A, B, C, D and E, described by the stratigraphic profiles and parameters given in Table 3.1.

Table 3.1: Ground types

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

(2) The average shear wave velocity $V_{s,30}$ is computed according to the following expression:

$$V_{s,30} = \frac{30}{\sum_{i=1,N} \frac{h_i}{V_i}} \qquad (3.1)$$

where h_i and V_i denote the thickness (in m) and shear-wave velocity (at shear strain level of 10^{-6} or less) of the i-th formation or layer, in a total of N, existing in the top 30 metres. The site will be classified according to the value of $V_{s,30}$ if this is available, otherwise the value of N_{SPT} will be used.

(3)P For sites with ground conditions matching the two special ground types S_1 and S_2, special studies for the definition of the seismic action are required. For these types, and particularly for S_2, the possibility of soil failure under the seismic action shall be considered.

Note: Special attention should be paid if the deposit is of ground type S_1. Such soils typically have very low values of V_s, low internal damping and an abnormally extended range of linear behaviour and can therefore produce anomalous seismic site amplification and soil-structure interaction effects; see EN 1998-5:200X, Section 6. In this case, a special study for the definition of the seismic action should be carried out, in order to establish the dependence of the response spectrum on the thickness and V_s value of the soft clay/silt layer and on the stiffness contrast between this layer and the underlying materials.

(4) Further sub-division of this classification is permitted to better conform with special ground conditions. The seismic actions defined for any sub-type should not be less than those corresponding to the main type as specified in Table 3.1, unless this is supported by special site-classification studies.

3.2 Seismic action

3.2.1 Seismic zones

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

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

Note 1: The value to be ascribed to k for use in a Country may be found in its National Annex. The recommended value is $k = 1$.

Note 2: The reference peak ground acceleration on type A ground, a_{gR}, or the reference ground acceleration on type A ground, $k.a_{gR}$, may be derived from zonation maps in the National Annex.

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

Note: See Note to 2.1(4)

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

Note: The selection of the categories of structures, ground types and seismic zones in a Country for which the provisions of low seismicity apply may be found in its National Annex. It is recommended to consider as low seismicity cases those in which the product $a_g.S$ is not greater than 0,1 g.

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

Note: The selection of the categories of structures, ground types and seismic zones in a Country for which the EN 1998 provisions need not be observed (cases of very low seismicity) may be found in its National Annex It is recommended to consider as very low seismicity cases those in which the product $a_g.S$ is not greater than 0,05 g.

3.2.2 Basic representation of the seismic action

3.2.2.1 General

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

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

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

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

Note 1: The selection of the shape of the elastic response spectrum to be used in a Country or part thereof may be found in the its National Annex.

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

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

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

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

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

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

3.2.2.2 Horizontal elastic response spectrum

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

$$0 \leq T \leq T_B : S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2{,}5 - 1) \right] \tag{3.1}$$

$$T_B \leq T \leq T_C : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \tag{3.2}$$

$$T_C \leq T \leq T_D : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \left[\frac{T_C}{T} \right] \tag{3.3}$$

$$T_D \leq T \leq 4s : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \left[\frac{T_C T_D}{T^2} \right] \tag{3.4}$$

where

$S_e(T)$ elastic response spectrum,

T vibration period of a linear single-degree-of-freedom system,

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

T_B, T_C limits of the constant spectral acceleration branch,

T_D value defining the beginning of the constant displacement response range of the spectrum,

S soil factor,

η damping correction factor with reference value $\eta = 1$ for 5% viscous damping, see (3).

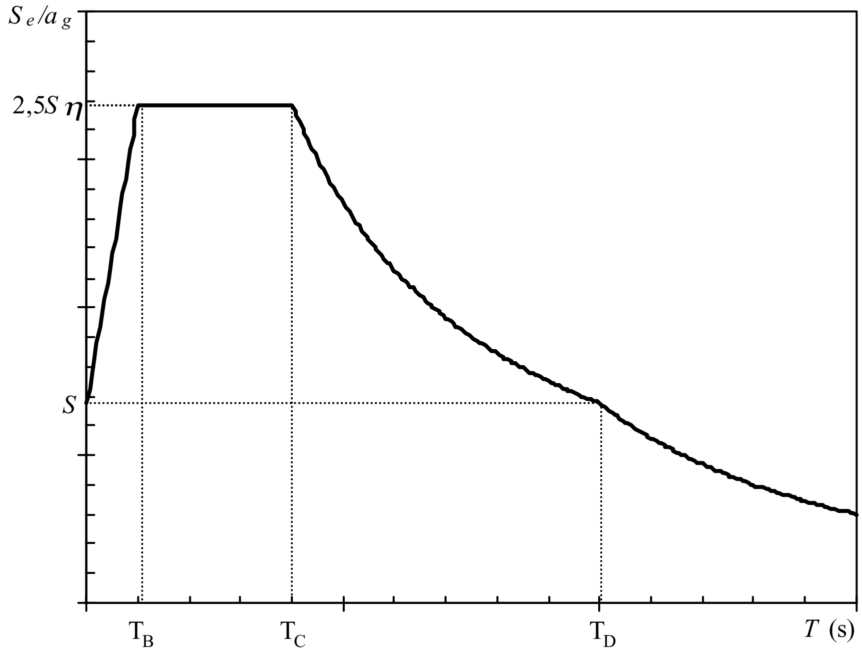

Figure 3.1: Shape of elastic response spectrum

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

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

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

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

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

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

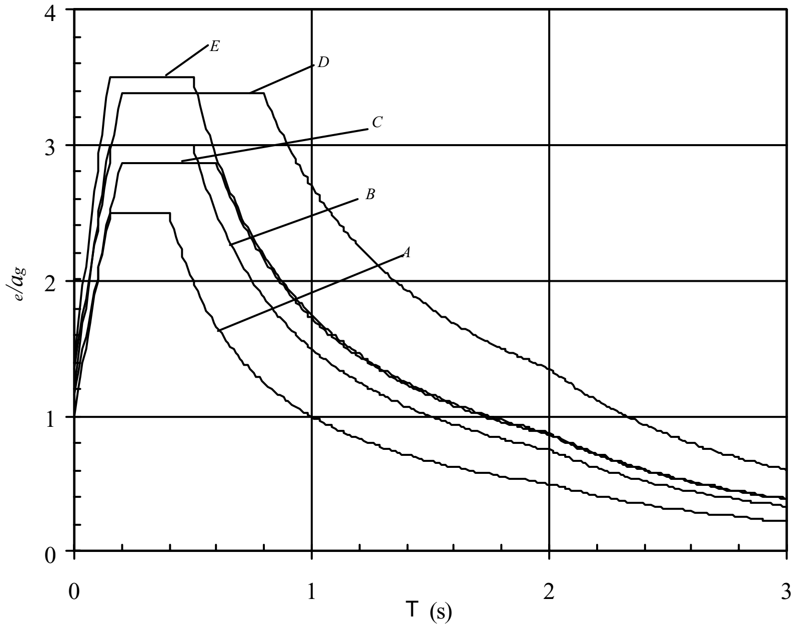

Figure 3.2: Recommended Type 1 elastic response spectrum for ground types A to E (5% damping)

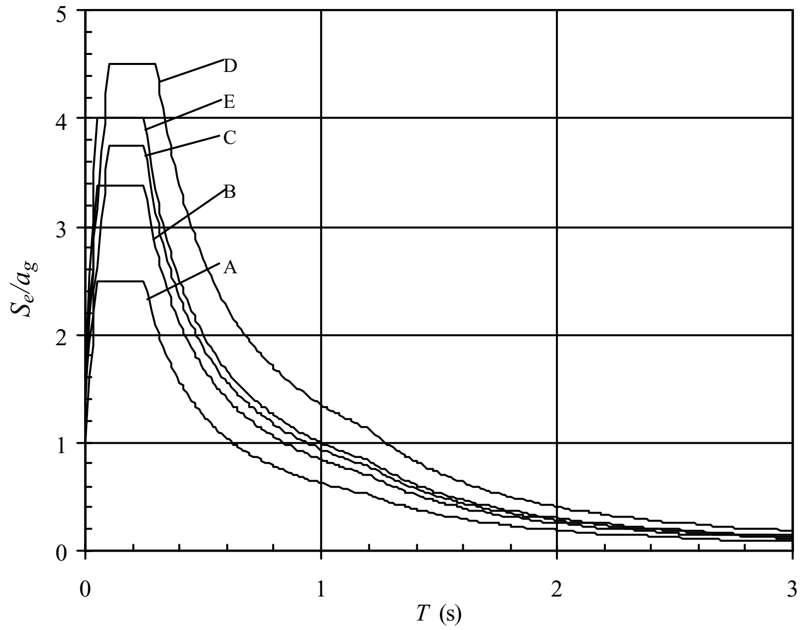

Figure 3.3: Recommended Type 2 elastic response spectrum for ground types A to E (5% damping)

Note 2: The special site-classification studies referred in 3.1.1(4) should also provide the corresponding values of S, T_B, T_C and T_D.

Note 3: For ground types S_1 and S_2, special studies should provide the corresponding values of S, T_B, T_C and T_D.

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

$$\eta = \sqrt{10/(5+\xi)} \geq 0,55 \tag{3.5}$$

where

ξ viscous damping ratio of the structure, expressed in percent.

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

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

$$S_{De}(T) = S_e(T)\left[\frac{T}{2\pi}\right]^2 \tag{3.6}$$

(6) Expression (3.6) should normally be applied for vibration periods not exceeding 4,0 s. For structures with vibration periods longer than 4,0 s, a more complete definition of the elastic displacement spectrum may be possible.

Note: For the Type 1 elastic response spectrum referred in Note 1 to 3.2.2.2(2), such a definition is presented in Informative Annex A in terms of the displacement response spectrum. For periods longer than 4,0 s the elastic acceleration spectrum may be derived from the elastic displacement spectrum by inverting expression (3.6).

3.2.2.3 Vertical elastic response spectrum

(1)P The vertical component of the seismic action shall be represented by an elastic response spectrum, $S_{ve}(T)$, derived using expressions (3.7)-(3.10).

$$0 \leq T \leq T_B : S_{ve}(T) = a_{vg} \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 3,0 - 1)\right] \tag{3.7}$$

$$T_B \leq T \leq T_C : S_{ve}(T) = a_{vg} \cdot \eta \cdot 3,0 \tag{3.8}$$

$$T_C \leq T \leq T_D : S_{ve}(T) = a_{vg} \cdot \eta \cdot 3,0\left[\frac{T_C}{T}\right] \tag{3.9}$$

$$4s \geq T \geq T_D : S_{ve}(T) = a_{vg} \cdot \eta \cdot 3,0\left[\frac{T_C . T_D}{T^2}\right] \tag{3.10}$$

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

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

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

3.2.2.4 Design ground displacement

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

$$d_g = 0{,}025 \cdot a_g \cdot S \cdot T_C \cdot T_D \qquad (3.11)$$

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

3.2.2.5 Design spectrum for elastic analysis

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

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

(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 elastic analysis model - still ensuring a satisfactory response of the structure. The value 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 the relevant ductility classes in the various Parts of EN 1998. The value of the behaviour factor q may be different in different horizontal

directions of the structure, although the ductility classification must be the same in all directions.

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

$$0 \le T \le T_B : S_d(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot \left(\frac{2,5}{q} - 1 \right) \right] \tag{3.12}$$

$$T_B \le T \le T_C : S_d(T) = a_g \cdot S \cdot \frac{2,5}{q} \tag{3.13}$$

$$T_C \le T \le T_D : S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2,5}{q} \cdot \left[\frac{T_C}{T} \right] \\ \ge \beta \cdot a_g \end{cases} \tag{3.14}$$

$$T_D \le T : \quad S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2,5}{q} \cdot \left[\frac{T_C T_D}{T^2} \right] \\ \ge \beta \cdot a_g \end{cases} \tag{3.15}$$

where:

a_g, S, T_C, T_D : as defined in 3.2.2.2.

$S_d(T)$ design spectrum,

q behaviour factor,

β lower bound factor for the horizontal design spectrum

Note: The value to be ascribed to β for use in a Country may be found in its National Annex. The recommended value is β=0,2.

(5) For the vertical component of the seismic action the design spectrum is given by expressions (3.12) to (3.15), with the design ground acceleration in the vertical direction, a_{vg} replacing a_g, S taken equal to 1,0 and parameters as defined in 3.2.2.3.

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

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

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

3.2.3 Alternative representations of the seismic action

3.2.3.1 Time - history representation

3.2.3.1.1 General

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

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

(3) Depending on the nature of the application and on the information actually available, the description of the seismic motion may be made by using artificial accelerograms (see 3.2.3.1.2) and recorded or simulated accelerograms (see 3.2.3.2).

3.2.3.1.2 Artificial accelerograms

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

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

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

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

a) A minimum of 3 accelerograms 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 $a_g.S$ for the site in question.

c) No value of the mean 5% damping elastic spectrum - calculated from all time histories - is less than 90% of the corresponding value of the 5% damping elastic response spectrum.

3.2.3.2 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 that the samples used are adequately qualified with regard to the seismogenetic features of the sources and to the soil conditions appropriate to the site, and their values are scaled to the value of $a_g.S$ for the zone under consideration.

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

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

3.2.3.3 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 3.2.2.1(8)).

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

3.2.4 Combinations of the seismic action with other actions

(1)P The design value E_d of the effects of actions in the seismic design situation shall be determined according to 6.4.3.4 of EN 1990

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

$$\Sigma G_{k,j} \text{"+"} \Sigma \psi_{E,i} \cdot Q_{k,i} \tag{3.16}$$

where

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

(3) The combination coefficients $\psi_{E,i}$ take into account the likelihood of the loads $\psi_{2,i} \cdot Q_{k,i}$ being not present over the entire structure during the occurrence of the earthquake. These coefficients may also account for a reduced participation of masses in the motion of the structure due to the non-rigid connection between them.

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

4 DESIGN OF BUILDINGS

4.1 General

4.1.1 Scope

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

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

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

4.2 Characteristics of earthquake resistant buildings

4.2.1 Basic principles of conceptual design

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

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

– structural simplicity,

– uniformity, symmetry and redundancy

– bi-directional resistance and stiffness,

– torsional resistance and stiffness,

– diaphragmatic behaviour at storey level,

– adequate foundation.

These principles are further elaborated in the following clauses.

4.2.1.1 Structural simplicity

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

4.2.1.2 Uniformity, symmetry and redundancy

(1) Uniformity is characterised by an even distribution of the structural elements which, if fulfilled in-plan, allows short and direct transmission of the inertia forces created in the distributed masses of the building. If necessary, uniformity may be realised by subdividing the entire building by seismic joints into dynamically independent units, provided that these joints are designed against pounding of the individual units according to 4.4.2.7.

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

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

(4) If the building configuration is symmetrical or quasi-symmetrical, a symmetrical structural layout, well-distributed in-plan, is an obvious solution for the achievement of uniformity.

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

4.2.1.3 Bi-directional resistance and stiffness

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

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

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

4.2.1.4 Torsional resistance and stiffness

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

4.2.1.5 Diaphragmatic behaviour at storey level

(1) In buildings, floors (including the roof) play a very important role in the overall seismic behaviour of the structure. They act as horizontal diaphragms that collect and transmit the inertia forces to the vertical structural systems and ensure that those

systems act together in resisting the horizontal seismic action. The action of floors as diaphragms is especially relevant in cases of complex and non-uniform layouts of the vertical structural systems, or where systems with different horizontal deformability characteristics are used together (e.g. in dual or mixed systems).

(2) Floor systems and the roof should be provided with in-plane stiffness and resistance and with effective connection to the vertical structural systems. Particular care should be taken in cases of non-compact or very elongated in-plan shapes and in cases of large floor openings, especially if the latter are located in the vicinity of the main vertical structural elements, thus hindering such effective connection.

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

4.2.1.6 Adequate foundation

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

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

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

4.2.2 Primary and secondary seismic members

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

(2) Sections 5 to 9 give deemed to satisfy rules – additional to those of EN 1992 to EN 1996 - for the design and detailing of secondary seismic elements.

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

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

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

4.2.3 Criteria for structural regularity

4.2.3.1 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 response spectrum analysis (lateral force procedure) or a 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 4.2.3.3(4),
- non-regular distribution of overstrength in elevation exceeding the limits given in 4.2.3.3(3).

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

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

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

* If the condition of 4.3.3.2.1(2)a) is also met.

** Under the specific conditions given in 4.3.3.1(7) a separate planar model may be used in each horizontal direction, according to 4.3.3.1(7).

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

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

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

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

4.2.3.2 Criteria for regularity in plan

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

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

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

(4) The slenderness $\lambda = L_x/L_y$ of the building in plan is not higher than 4.

(5) At each level and for each direction of analysis *x* or *y*, the structural eccentricity and the torsional radius verify the two conditions below, which are expressed for the direction of analysis y:

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

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

where:

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

r_x square root of the ratio between torsional stiffness and lateral stiffness in the *y* direction ("torsional radius").

l_s radius of gyration of the floor in plan (square root of the ratio between polar moment of inertia of the floor in plan with respect to floor mass centre and the floor plan area).

The definitions of centre of stiffness and torsional radius r_x are provided in the following clauses.

(6) In single storey buildings the centre of stiffness is defined as the centre of the lateral stiffness of all primary seismic elements. The torsional radius *r* is defined as the

square root of the ratio of the global torsional stiffness with respect to the centre of lateral stiffness, and the global lateral stiffness in one direction, taking into account all the primary seismic elements in such direction.

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

a) All lateral load resisting systems, like cores, structural walls or frames, run without interruption from the foundations to the top of the building.

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

(8) If both conditions a) and b) of (7) are met, the position of the centres of stiffness and the torsional radius of all storeys may be calculated as those of certain quantities, proportional to a system of forces, having the distribution specified in 4.3.3.2.3 and producing a unit displacement at the top of the individual lateral load resisting systems.

(9) In frames and in systems of slender walls with prevailing flexural deformations, the quantities in (8) above may be the taken as the moments of inertia of the cross sections of the vertical elements. If, in addition to flexural deformations, shear deformations are also significant, they may be accounted for by using an equivalent moment of inertia of the cross section.

4.2.3.3 Criteria for regularity in elevation

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

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

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

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

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

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

c) if the setbacks do not preserve symmetry, in each face the sum of the setbacks at all storeys 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. 4.1.d).

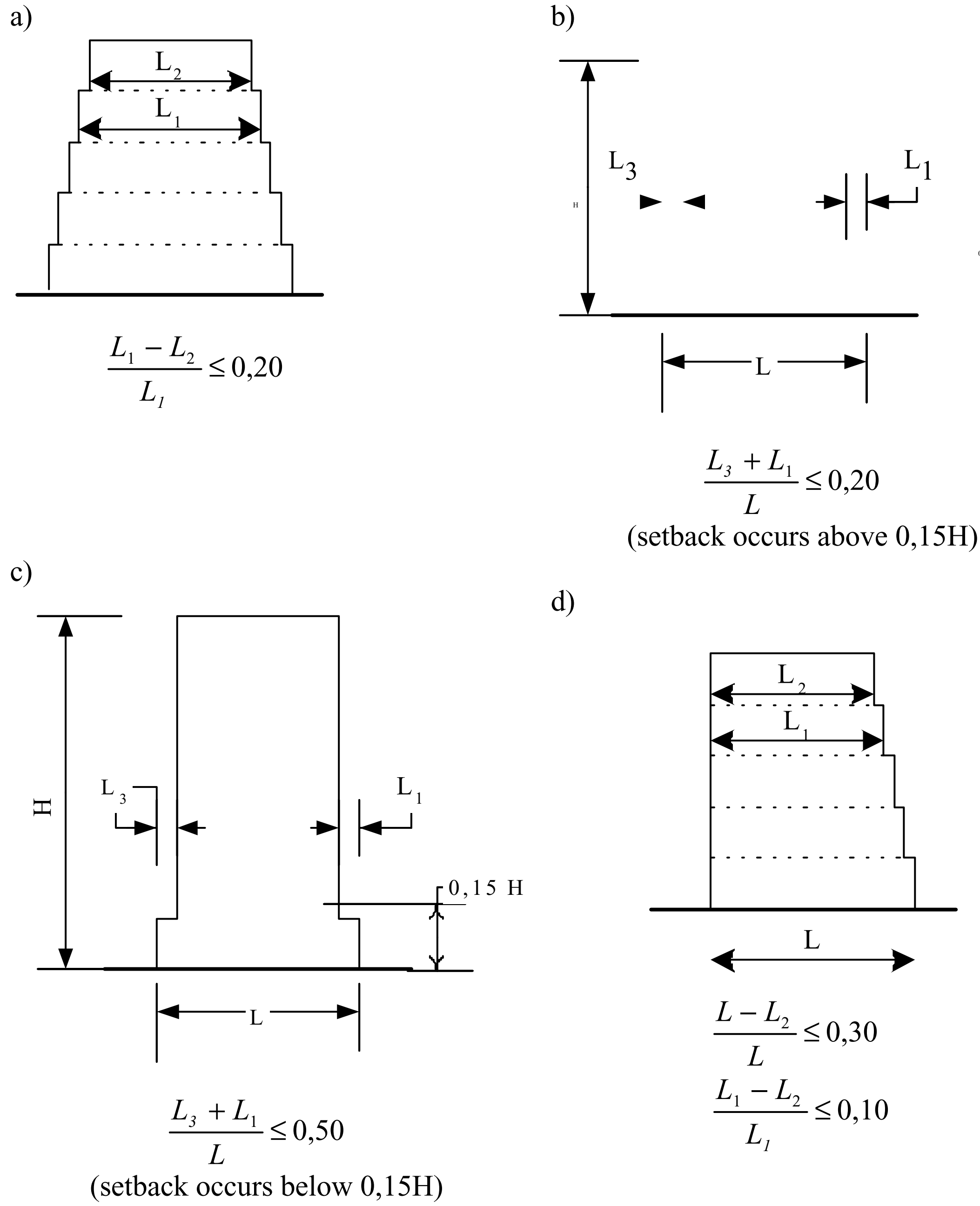

Figure 4.1: Criteria for regularity of buildings with setbacks

4.2.4 Combination coefficients for variable actions

(1)P The combination coefficients ψ_{2i} for the design of buildings (see 3.2.4(1)P) are given in Annex A1 of EN 1990:2002.

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

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

Note: The values to be ascribed to *ö* for use in a Country may be found in its National Annex. The recommended values are listed in Table 4.2.

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

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

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

4.2.5 Importance classes and importance factors

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

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

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

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

Table 4.3 Importance classes for buildings

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

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

(5)P The value of γ_I for importance class III is by definition equal to 1,0.

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

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

4.3 Structural analysis

4.3.1 Modelling

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

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

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

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

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

(5) For buildings complying with the criteria for regularity in plan (see 4.2.3.2) or with the regularity criteria in 4.3.3.3.1(2), the analysis may be performed using two planar models, one for each main direction.

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

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

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

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

Note: Foundation deformability (including the soil-structure interaction) may always be taken into account, including the case it has beneficial effects.

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

4.3.2 Accidental torsional effects

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

$$e_{1i} = \pm 0{,}05 \cdot L_i \tag{4.3}$$

where

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.

4.3.3 Methods of analysis

4.3.3.1 General

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

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

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

– the "lateral force method of analysis" for buildings meeting the conditions given in 4.3.3.2,

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

(4) Non-linear methods may also be used, such as:

– non-linear static (pushover) analysis,

– non-linear time history (dynamic) analysis,

under the conditions specified in (5) and (6)P below and in 4.3.3.4.

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

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

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

(8) In buildings with importance factor, γ_I, not greater than 1,0, linear-elastic analysis may be performed using two planar models, one for each main horizontal direction, even if the criteria for regularity in plan in 4.2.3.2 are not satisfied, if all the following special regularity criteria are met:

(a) The building has well-distributed and relatively rigid cladding and partitions.

(b) The building height does not exceed 10 m.

(c) The building aspect ratio (height/length) in both main directions does not exceed 0,4.

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

(e) The centres of lateral stiffness and mass are each approximately on a vertical line and in the two horizontal directions of analysis satisfy the conditions: $r_x^2 > l_s^2 + e_{ox}^2$, $r_y^2 > l_s^2 + e_{oy}^2$, where the radius of gyration l_s the torsional radii r_x and r_y and the natural eccentricities e_{ox} and e_{oy} are defined as in 4.2.3.2(5).

(8) Linear-elastic analysis may be performed using two planar models, one for each main horizontal direction, also in buildings satisfying all the conditions and criteria of par. (7) above except (e), provided all seismic action effects from the analysis are multiplied by 1,25.

(9)P Buildings not complying with the criteria in (6) to (8) above, shall be analysed using a spatial model.

(10)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.

4.3.3.2 Lateral force method of analysis

4.3.3.2.1 General

(1)P This type of analysis may be applied to buildings whose response is not significantly affected by contributions from higher modes of vibration.

(2) These requirements are deemed to be satisfied in buildings which fulfil both of the following two conditions:

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

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

where T_C is given in Tables 3.2 or 3.3,

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

4.3.3.2.2 Base shear force

(1)P The seismic base shear force F_b, for each horizontal direction in which the building is analysed, is determined as follows:

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

where

$S_d(T_1)$ ordinate of the design spectrum (see 3.2.2.5) at period T_1,

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

m total mass of the building computed in accordance with 3.2.4(2),

λ correction factor, the value of which is equal to:

λ= 0,85 if $T_1 \leq 2\ T_C$ and the building has more than two storeys, or λ= 1,0 otherwise.

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

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

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

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

where

$$C_t \left| \begin{array}{l} 0{,}085 \text{ for moment resistant space steel frames} \\ 0{,}075 \text{ for moment resistant space concrete frames and for eccentrically braced steel frames} \\ 0{,}050 \text{ for all other structures} \end{array} \right.$$

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

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

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

where

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

and

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

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

H as in paragraph (3) above,

l_{wi} length of the shear wall i in the first storey in the direction parallel to the applied forces, in m,

with the restriction that l_{wi}/H shall not exceed 0,9.

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

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

where

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

4.3.3.2.3 Distribution of the horizontal seismic forces

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

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

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

where

F_i horizontal force acting on storey i,

F_b seismic base shear according to expression (4.5);

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

m_i,m_j computed according to 3.2.4(2).

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

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

where

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

(4)P The horizontal forces F_i determined according to the above paragraphs shall be distributed to the lateral load resisting system assuming rigid floors.

4.3.3.2.4 Torsional effects

(1) If the lateral stiffness and mass are symmetrically distributed in plan and unless the accidental eccentricity of 4.3.2(1)P is not taken into account by a more exact method (e.g. that of 4.3.3.3.3(1)), the accidental torsional effects may be accounted for by multiplying the action effects resulting in the individual load resisting elements from the application of 4.3.3.2.3(4) with a factor δ given by:

$$\ddot{a} = 1 + 0,6 \cdot \frac{x}{L_e} \tag{4.12}$$

where

x distance of the element under consideration from the centre of the building in plan, 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) If the analysis is performed using two planar models, one for each main horizontal direction, torsional effects may be determined by doubling the accidental eccentricity e_{1i} of expression (4.3) and applying par.(1) above with factor 0,6 in expression (4.12) increased to 1,2.

4.3.3.3 Modal response spectrum analysis

4.3.3.3.1 General

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

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

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

- By demonstrating that the sum of the effective modal masses for the modes taken into account amounts to at least 90% of the total mass of the structure.
- 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 $F_{bk} = S_d(T_k)\ m_k$. It can be shown that the sum of effective modal masses (for all modes and a given direction) is equal to the mass of the structure.

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

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

$$k \geq 3 \cdot \sqrt{n} \qquad (4.14a)$$

and

$$T_k \leq 0{,}20\, s \qquad (4.14b)$$

where

k number of modes taken into account,

n number of storeys above ground,

T_k period of vibration of mode k

4.3.3.3.2 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, if their periods T_i and T_j satisfy (with $T_j \leq T_i$) the following condition:

$$T_j \leq 0{,}9 \cdot T_i \tag{4.15}$$

(2) Whenever all relevant modal responses (see 4.3.3.3.1(5)-(8)) may be regarded as independent of each other, the maximum value E_E of a seismic action effect may be taken as

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

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 (1) P is not satisfied, more accurate procedures for the combination of the modal maxima shall be adopted.

(4) Paragraph (3)P is deemed to be satisfied if procedures such as the "Complete Quadratic Combination" are used.

4.3.3.3.3 Torsional effects

(1) Whenever a spatial model is used for the analysis, the accidental torsional effects referred in 4.3.2(1)P 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:

$$M_{1i} = e_{1i} \cdot F_i \tag{4.17}$$

where

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

e_{1i} accidental eccentricity of storey mass i according to expression (4.3) for all relevant directions,

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

(2) The effects of the loading according to (1) should be taken into account with positive and negative 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 4.3.3.2.4(2) to the action effects computed according to 4.3.3.3.2.

4.3.3.4 Non-linear methods

4.3.3.4.1 General

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

(2) As a minimum, bilinear force – deformation envelopes should be used at the element level. In reinforced concrete and masonry buildings, the elastic stiffness of a bilinear force-deformation relation should correspond to cracked sections (see 4.3.1(8)). In ductile elements, expected to exhibit post-yield excursions during the response, the elastic stiffness of a bilinear relation should be the secant stiffness to the yield-point. Trilinear envelopes, which take into account pre-crack and post-crack stiffnesses, are allowed.

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

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

(5)P Gravity loads according to 3.2.4 shall be applied to appropriate elements of the mathematical model.

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

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

4.3.3.4.2 Non-linear static (pushover) analysis

4.3.3.4.2.1 General

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

a) to verify or revise the overstrength ratio values α_u/α_1 (see 5.2.2.2.1, 6.3.2, 7.3.2)

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

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

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

(2)P Buildings not complying with the regularity criteria of 4.2.3.2 or the criteria of 4.3.3.1(7)a)-e) shall be analysed using a spatial model.

(3) For buildings complying with the regularity criteria of 4.2.3.2 or the criteria of 4.3.3.1(7)a)-e) the analysis may be performed using two planar models, one for each main horizontal direction.

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

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

4.3.3.4.2.2 Lateral loads

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

- a “uniform” pattern, based on lateral forces that are proportional to mass regardless of elevation (uniform response acceleration)
- a “modal” pattern, proportional to lateral forces consistent with the lateral force distribution determined in elastic analysis (according to 4.3.3.2 or 4.3.3.3)

(2)P Lateral loads shall be applied at the location of the masses in the model. Accidental eccentricity according to 4.3.2(1)P shall be considered.

4.3.3.4.2.3 Capacity curve

(1) The relation between base shear force and the control displacement (the “capacity curve”) should be determined by pushover analysis for values of the control displacement ranging between zero and the value corresponding to 150% of the target displacement, defined in 4.3.3.4.2.6.

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

4.3.3.4.2.4 Overstrength factor

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

4.4.3.4.2.5 Plastic mechanism

(1)P The plastic mechanism shall be determined for both lateral load distributions. The plastic mechanisms should comply with the mechanisms on which the behaviour factor q used in the design is based.

4.3.3.4.2.6 Target displacement

(1)P Target displacement is defined as the seismic demand derived from the elastic response spectrum of 3.2.2.2 in terms of the displacement of an equivalent single-degree-of-freedom system.

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

4.3.3.4.2.7 Procedure for estimation of the torsional effects

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

Note: The stiff/strong side in plan is the one which develops smaller horizontal displacements than the opposite side, under the action of lateral forces parallel to it.

(2) The requirement above is deemed to be satisfied if the amplification factor to be applied to the displacements of the stiff/strong side is based on results of elastic modal analysis of the spatial model.

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

4.3.3.4.3 Non-linear time-history analysis

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

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

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

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

4.3.3.5.1 Horizontal components of the seismic action

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

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

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

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

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

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

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

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

where

"+" implies "to be combined with",

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

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

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

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

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

4.3.3.5.2 Vertical component of the seismic action

(1) The vertical component of the seismic action, as defined in clause 3.2.2.3 should be taken into account in the cases below, if a_{vg} is greater than 0,25 g:

– For horizontal or nearly horizontal structural members spanning 20 m or more;

– For horizontal or nearly horizontal cantilever components longer than 5 m;

– For horizontal or nearly horizontal prestressed components;

– For beams supporting columns;

– In base-isolated structures.

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

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

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

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

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

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

where

"+" implies "to be combined with",

E_{Edx} and E_{Edy} see 4.3.3.5.1(3),

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

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

4.3.4 Displacement analysis

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

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

where

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

q_d displacement behaviour factor, assumed equal to q unless otherwise specified.

d_e displacement of the same point of the structural system, as determined by a linear analysis based on the design response spectrum according to 3.2.2.5.

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

Note: In general q_d is larger than q if the fundamental period is less than T_C (see B3 in Informative Annex B).

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

(3) For non-linear analysis, static or dynamic, the displacements are those obtained from the analysis.

4.3.5 Non-structural elements

4.3.5.1 General

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

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

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

4.3.5.2 Analysis

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

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

$$F_a = \left(S_a \cdot W_a \cdot \tilde{a}_a\right) / q_a \qquad (4.24)$$

where

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

W_a weight of the element,

S_a seismic coefficient pertinent to non-structural elements, see (3),

γ_a importance factor of the element, see 4.3.5.3,

q_a behaviour factor of the element, see Table 4.4,

(3) The seismic coefficient S_a may be calculated as follows:

$$S_a = 2 \cdot \alpha \cdot S \cdot (1 + z/H) \,/\, (1 + (1 - T_a/T_1)^2) \qquad (4.25)$$

where

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

S soil factor,

T_a fundamental vibration period of the non-structural element,

T_1 fundamental vibration period of the building in the relevant direction,

z height of the non-structural element above the level of application of the seismic action,

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

4.3.5.3 Importance factors

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

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

(2) In all other cases the importance factor γ_a of a non-structural element may be assumed to have the same value as the importance factor γ_I of the building concerned.

4.3.5.4 Behaviour factors

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

Table 4.4: Values of q_a for non-structural elements

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

4.3.6 Additional measures for masonry infilled frames

4.3.6.1 General

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

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

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

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

(2) Although the scope of clauses 4.3.6.1 to 4.3.6.3 is limited according to paragraph (1)P above, these clauses provide for much good practice, which may to advantage be adopted for DCM or DCL concrete, steel or composite structures with masonry infills. (In particular for panels that may be vulnerable to out-of-plane failure, the provision of ties can control the hazard from falling masonry).

(3)P The provisions in 1.2(2) regarding possible future modification of the structure apply also to the infills.

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

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

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

4.3.6.2 Requirements and criteria

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

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

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

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

4.3.6.3 Irregularities due to masonry infills

4.3.6.3.1 Irregularities in plan

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

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

(3) Infill panels with more than one significant openings or perforations (e.g. a door and a window, etc.) should be disregarded in the model for an analysis according to paragraph (2) above.

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

4.3.6.3.2 Irregularities in elevation

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

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

$$\varsigma = \left(1 + \Delta V_{Rw} / \Sigma V_{Sd}\right) \leq q \tag{4.26}$$

where

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

ΣV_{Sd} sum of the seismic shear forces acting on all vertical primary seismic elements of the storey concerned.

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

4.3.6.4 Damage limitation of infills

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

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

(3) If there are large openings or perforations in an infill panel, their edges should be trimmed with belts and posts.

4.4 Safety verifications

4.4.1 General

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

(2) For buildings of importance classes II - IV (see Table 4.3) the verifications prescribed in 4.4.2 and 4.4.3 may be considered satisfied, if the following two conditions are met:

a) The total base shear due to the seismic design situation calculated with a behaviour factor q=1,5 is less than that due to the other relevant action combinations for which the building is designed on the basis of a linear elastic analysis. This requirement should be fulfilled by the total storey shear at the ground level, when such level is different from that at the building base.

b) The specific measures described in 2.2.4 are taken into account, with the exception of the provisions in 2.2.4.1(2)-(3), which need not be demonstrated as having been met.

4.4.2 Ultimate limit state

4.4.2.1 General

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

4.4.2.2 Resistance condition

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

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

E_d is the design value of the action effect, due to the seismic design situation (see clause 6.4.3.4 of EN 1990:2002), including – if necessary – second order effects (see(2)). Redistribution of bending moments according to EN 1992-1, EN 1993-1, EN 1994-1 is permitted.

R_d is the corresponding design resistance of the element, calculated according to the rules specific to the pertinent material (in terms of characteristic values of properties 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 Sections 5 to 9 and in the relevant Eurocodes.

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

$$è = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h} \le 0{,}10 \tag{4.28}$$

where

θ interstorey drift sensitivity coefficient,

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

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 4.3.4,

V_{tot} total seismic storey shear,

h interstorey height.

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

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

(5) If design action effects E_d are obtained through a nonlinear method of analysis (see 4.4.3.4), then par.(1)P above should be applied in terms of forces only for brittle elements. For dissipative zones, which are designed and detailed for ductility, the resistance condition, expression (4.27), should be satisfied in terms of member deformations (e.g. plastic hinge or chord rotations), with appropriate material safety factors applied on member deformation capacities (see also 5.7(2)P and 5.7(4)P in EN 1992-1:200X).

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

4.4.2.3 Global and local ductility condition

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

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

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

(4) Unless otherwise specified in Sections 5 to 8, to satisfy the requirement of (3), at all beam-column joints of frame buildings, including frame-equivalent ones in the meaning of 5.1.2(1), with two or more storeys, the following condition should be satisfied:

$$\sum M_{Rc} \geq 1{,}3 \sum M_{Rb} \quad (4.29)$$

where:

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

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

Note: Normally the moments at the centre of the joint corresponding to development of the design values of the moments of resistance of the columns or beams framing into the joint should be used in ΣM_{Rc} and ΣM_{Rb}. Nonetheless the loss in accuracy is minor and the simplification achieved by using instead the design values of moments of resistance is considerable.

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

(6) The requirement of (4) and (5) is waived at the top level of multi-storey buildings.

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

(8) The requirements of (1) and (2) are deemed to be satisfied if:

a) plastic mechanisms obtained by pushover analysis are satisfactory

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

c) brittle elements remain in the elastic region

4.4.2.4 Equilibrium condition

(1)P The building structure shall be stable under the set of actions of the seismic design situation of clause 6.4.3.4 of EN 1990:2001. 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 3.2.3.1.

4.4.2.5 Resistance of horizontal diaphragms

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

(2) Paragraph (1) is considered satisfied if for the relevant resistance verifications the seismic action effects obtained from the analysis for the diaphragm are multiplied by an overstrength factor γ_d greater than 1,0.

Note: The values to be ascribed to γ_d for use in a Country may be found in its National Annex The recommended value is 1,3 for brittle failure modes – such as in shear in concrete diaphragms – and 1.1 for ductile failure modes.

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

4.4.2.6 Resistance of foundations

(1)P The foundation system shall be verified according to 5.4 of EN 1998-5 and to EN 1997-1.

(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 (e.g. for concrete, steel or composite buildings of ductility class L, see Sections 5, 6, 7), no capacity design considerations according to (2) P are required.

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

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

where

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

$E_{F,G}$ action effect due to the non-seismic actions included in the combination of actions for the seismic design situation (see clause 6.4.3.4 of EN 1990:2001),

$E_{F,E}$ action effect due to the design seismic action,

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

R_{di} design resistance of the zone or element i,

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

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

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

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

(8) For common foundations of more than one vertical element (foundation beams, strip footings, rafts, etc.) (2) is deemed to be satisfied if the value of $\boldsymbol{\Omega}$ used in expression (4.30) is derived from the vertical element with the largest horizontal shear

force in the design seismic situation, or, alternatively, if the value $\Omega = 1$ is used in expression (4.30) with the value of the overstrength factor γ_{Rd} increased to 1,4.

4.4.2.7 Seismic joint condition

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

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

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

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

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

4.4.3 Damage limitation

4.4.3.1 General

(1) The "damage limitation requirement" (serviceability limit state) is considered satisfied, if - under a seismic action having a larger probability of occurrence than the seismic action used for the verification of the "no-collapse requirement" - the interstorey drifts are limited according to 4.4.3.2.

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

4.4.3.2 Limitation of interstorey drift

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

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

$$d_r \cdot \nu \leq 0{,}005\, h \qquad (4.31)$$

b) for buildings having non-structural elements fixed in a way as not to interfere with structural deformations or being composed of ductile elements.

$$d_r \cdot \nu \leq 0{,}0075\, h \qquad (4.32)$$

where

d_r design interstorey drift as defined in 4.4.2.2 (2),

h storey height,

ν reduction factor to take into account the lower return period of the seismic action associated with the damage limitation state.

(2) The value of the reduction factor ν may also depend on the importance class of the building. Implicit in its use is the assumption that the elastic response spectrum of the seismic action for the "no-collapse requirement" has the same shape as the spectrum of the seismic action for "damage limitation" (see 3.2.2.1(1)P).

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