MODEL BUILDING CODE FOR EARTHQUAKES

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ACS AEC

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FOREWORD

INTRODUCTION

Recognising the need for each country susceptible to disasters to have appropriate construction standards, the Association of Caribbean States (ACS), with financial assistance from the Government of Italy, through its Trust Fund managed by the Inter-American Development Bank (IDB), and from STIRANA (Foundation for Disaster Preparedness of the Netherlands Antilles), has embarked on a project aimed at "***Updating Building Codes of the Greater Caribbean for Winds and Earthquakes***" and thereby reducing the vulnerability to natural disasters. This initiative is consistent with the goal of the ACS Special Committee on Natural Disasters to reduce risks and losses caused by natural disasters in ACS Members Countries.

The objective of the first phase of the project was to produce and disseminate state-of-the-art model codes for wind loads and earthquakes as well as recommendations for the updating of existing codes, so that ACS Member Countries be able to endow themselves with new appropriate codes or improve the existing ones, in order to develop better construction practices and techniques for the building of safe and reliable buildings.

EVALUATION OF EXISTING BUILDING CODES IN THE GREATER CARIBBEAN

The first part of the project was devoted to a thorough analysis of the situation of present codes for earthquake resistant design in ACS Spanish- and English-speaking Member Countries. To accomplish this task, ad-hoc Evaluation Forms were prepared, the entries of which included all the main items that should be found in a state-of-the-art code. Subsequently, the existing earthquake codes of ACS Spanish- and English-speaking Member Countries were thoroughly reviewed and evaluated, and the Forms were completed. At the end of each Evaluation Form, salient recommendations for code improvement were formulated. The Forms were finally disseminated to ACS Member Countries.

An extremely diversified situation emerged from these evaluations.

PREPARATION OF A MODEL CODE

In the second part of the project a Model Code was drafted, to be used by each State in updating/preparing actual Codes of Practice, inspired by common concepts.

Given the diversity of the situations in each country, the project team decided to prepare a conceptual model code that would not only be complete in its scope, but also capable of allowing the development of actual codes of practice at different levels of complexity.

This step required a clear distinction between principles, to be adopted as the basis of design and safety rules, and recommendations to implement these principles into practical rules.

The conceptual choice of the Model Code implied that no reference to specific construction materials and structural systems should be made, since these should be treated at a national or regional level.

These decisions were implemented adopting as a basic reference document the European Standard for Earthquake Resistant Design of Structures (Eurocode 8, final version of January 2003), since similar problems had to be faced in Europe to harmonise the seismic code standards of the different countries.

Due to its conceptual basis, the Model Code is intended for use by code makers and authorities, not by single professionals.

SEISMIC ZONATION

The seismic zonation map referred to in the Model Code should be enforced at the State level, but should be possibly based on global comprehensive and consistent scientific studies for the entire Greater Caribbean Region, to avoid inconsistency at the borders between different states. **It is therefore recommended that a "model seismic zonation map" be developed for such the Greater Caribbean Region.**

Seismic zonation maps shall be developed using internationally accepted methods, up-to-date data and transparent and repeatable procedures. Periodic revisions should be foreseen.

ENFORCING AND MONITORING THE USE OF A CODE

Countries of the Greater Caribbean Region should **give priority to the strengthening of existing building codes or the development of new codes**.

However, the development of relatively advanced national codes based on the present model code will not automatically produce a reduction of seismic risk. Such reduction requires side measures to enforce the use of the code, to monitor its performance, to increase the level of understanding and the specific preparation of professionals and consultants.

Enforcing the use of a code requires making its application mandatory, implying therefore some sort of control of the application of the code in designing, assessment and strengthening, through the **creation of enforcement and inspection mechanisms**. This objective may be pursued by defining strategies and creating special offices in charge of collecting design data, responding to technical questions, and checking the actual and appropriate use of the code in given fractions of the designed and constructed cases. Such fractions of the designed building stock to be checked may be defined for different building importance categories (e.g.: 5 % for importance class IV, 10 % for importance class III, 50 % for importance class II, 100% for importance class I).

To reinforce these building regulations, governments should work with private-sector financial and insurance companies to **encourage the development of financial incentives,** such as premium reductions or reduced-rate loans, for properly constructed buildings using established standards and regulations.

EDUCATION AND DISSEMINATION

The importance of **assuring a high level of competence of the designers** cannot be overemphasized. With the adoption of state-of-the-art building codes throughout the region, building inspectors, designers, engineers, builders and construction workers have to be trained on the new codes. Control measures for the training and the qualification of those actors should also be put in place. It is therefore recommended that all means of increasing the understanding of concepts and rules defined in the codes be exploited. Appropriate measures may include organization of short courses, possibly using e–learning tools, preparation of manuals and on–line helping tools, periodical verification of the effective competence of professionals.

PERIODICAL REVISIONS

It is recommended that a procedure be established for the **periodic updating of the model and national codes**, based on scientific progress and on the results of the monitoring process. These revisions should be considered at time intervals in the range of 5 years with a maximum of 10 years.

I. SCOPE

1.1 EXPLICIT CONCEPTS

This model code is intended for the design and construction of new buildings in seismic regions, as well as for the retrofitting of existing buildings.

Non building structures such as bridges, tanks, dams, and off-shore structures are beyond the scope of this document.

The model code contains only provisions that must be observed for the design of structures in seismic regions, in addition to the provisions of the other relevant structural design and construction codes applicable in each country.

1.2 PERFORMANCE OBJECTIVES AND FUNDAMENTAL SAFETY REQUIREMENTS

The purpose of the code is to ensure that in the event of earthquakes:

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

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

1.2.1 Safety requirements

No-collapse requirement: The structure, including seismic isolation and dissipation devices if present, 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.

Damage limitation requirement: The structure, including equipment relevant to the function of the building, 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. For particular categories of buildings, that must remain fully operational even after violent earthquakes, the values of the design action can be increased, referring to occurrence probabilities similar or closer to those governing the no-collapse requirement.

In order to satisfy the no-collapse safety requirement the provisions set forth in this code shall be followed, in particular as regards:

- the selection of the seismic design action with respect to the seismic zonation and the classification of ground types described in 2.5;
- the adoption of a mechanical model of the structure capable of accurately describing its response under dynamic excitation;
- the selection of a method of analysis suitable for the characteristics of the structure, as indicated in 5.2;
- the positive verification of strength and displacement compatibility;
- the adoption of all detailing rules that ensure adequate ductility resources to structural elements and to the construction as a whole, as appropriate for each construction material.

The damage limitation requirement is considered satisfied if the rules set forth in this model code are satisfied, with reference to 5.4.

II. SEISMIC ZONING AND SITE CHARACTERIZATION

2.1 SEISMIC ZONING AND RELATED CRITERIA, BASIS FOR IMPORTANCE FACTORS

For the implementation of this code, the national territory of a Country shall be subdivided into seismic zones, depending on the local hazard. By definition, each zone is characterised by a constant hazard, quantified by a different value of the parameter a_g, reference peak horizontal ground acceleration on type A ground (defined under 2.5), as shown in 2.2.

The reference peak ground acceleration chosen by the National Authorities for each seismic zone, shall generally correspond to a reference return period of the seismic action equal to 475 years for the no-collapse requirement, or equivalently to a reference probability of exceedance of 10% in 50 years.

To the reference return period of 475 years, an importance factor γ_I equal to 1.0 is assigned. Different values of γ_I shall be introduced to classify structures into different importance classes, as shown in 4.1, associated to different reliability requirements

For return periods other than the reference, the design ground acceleration on type A ground is equal to $a_g \gamma_I$.

The reference peak ground acceleration on type A ground, a_g, for use in a Country or parts thereof, shall be derived from zonation maps representing the values of a_g on Type A ground for the reference return period of 475 years.

2.2 LEVELS OF SEISMIC INTENSITY

The values of the maximum horizontal acceleration on type A ground a_g, expressed as a fraction of the acceleration of gravity g (= 9.81 m/s^2), to be adopted in the seismic zones will indicatively be the following:

Seismic zone	**Maximum horizontal ground acceleration with 10 % exceedance probability in 50 years**	**Value of a_g**
1	> 0.30 g	0.35 g
2	0.20-0.30 g	0.25 g
3	0.10-0.20 g	0.15 g
4	<0.10 g	0.05 g

Attribution of an area to a seismic zone should be based on seismic hazard evaluation, consistent with 2.1.

2.3 NEAR FAULT CONSIDERATIONS

Buildings of importance classes I, II, III defined in 4.1 shall generally not be erected in the immediate vicinity of tectonic faults recognised as seismically active in official documents issued by competent national authorities.

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

Special geological investigations shall be carried out for urban planning purposes and for important structures to be erected near potentially active faults in areas of high and moderate seismicity (seismic zones 1 and 2), in order to determine the ensuing hazard in terms of ground rupture and severity of ground shaking.

2.4 REQUIREMENTS ON CONSTRUCTION SITE AND FOUNDATION SOILS

Sufficient in situ investigations shall be carried out to ensure that the construction site and the soils present therein be generally free from hazards of slope instability and landsliding, as well as of permanent settlements and local rupture phenomena caused by liquefaction or by excessive settlements in the event of an earthquake.

A verification of ground stability shall be carried out for structures to be erected on or near natural or artificial slopes, in order to ensure that the safety and/or serviceability of the structures is preserved under the design earthquake.
Depending on the importance class of the structure and the particular conditions of the project, ground investigations and/or geologic studies should be performed to classify the foundation soils according to the ground types described in 2.5, and for the ensuing determination of the seismic action.

2.5 IDENTIFICATION OF GROUND TYPES

Ground types A, B, C, D, and E, described by the profiles and parameters given hereafter, shall be used to account for the influence of local ground conditions on the seismic action (depths are measured from the lower level of the foundations):

A - *Rock, rocklike formations, or very stiff cemented homogeneous soils* with values of average shear wave velocity V_{S30} larger than 800 m/s, including at most 5 m of weaker material at the surface

B - *Deposits of very dense sand/gravel or of stiff clays*, at least several tens of m in thickness, characterised by a gradual increase of mechanical properties with depth, and by V_{S30} values between 360 m/s e 800 m/s (or standard penetration resistance N_{SPT} > 50, or undrained shear strength c_u>250 kPa).

C - *Deposits of dense or medium dense sand/gravel or of stiff clay*, with thickness from several tens to many hundreds of m, and V_{S30} values between 180 e 360 m/s (15 < N_{SPT} < 50, 70 <c_u<250 kPa).

D - *Deposits of loose-to-medium cohesionless soil* (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive*,* with values of V_{S30} < 180 m/s (N_{SPT} < 15, c_u<70 kPa).

E - *Shallow alluvium soil profiles,* with V_{S30} values similar to those of types C or D and thickness between about 5 m and 20 m, overlying a stiffer ground formation of type A.

The average shear wave velocity V_{S30} may computed according to the following expression:

$$V_{s30} = \frac{30}{\sum_{1,N} \frac{h_i}{v_{s,i}}}$$

where h_i and v_i denote the thickness (in m) and shear-wave velocity of the *i* -th formation or layer, in a total of *N*, existing in the top 30 m.

Seismic design actions for structures to be erected on ground types A, B, C, D, and E are specified in 3.1.

For particular ground profiles not included in the previous classification, such as those including at least 10 m of very soft, high plasticity clays, or soil deposits susceptible to liquefaction, special studies are required for the definition of the seismic action.

III. SEISMIC ACTIONS

3.1 ELASTIC RESPONSE SPECTRA (HORIZONTAL AND VERTICAL)

The reference model for the description of earthquake motion at a point on the ground surface is represented by an elastic ground acceleration response spectrum, hereinafter called "elastic response spectrum".

For certain applications, the earthquake motion may be described by acceleration time series (accelerograms), as specified in 3.3.

The horizontal earthquake motion consists of two independent perpendicular components, having the same response spectrum.

In the absence of documented specific information, the vertical component of earthquake ground motion shall be represented through an elastic response spectrum different from that of the horizontal components, as specified hereinafter.

The elastic response spectrum is composed of a spectral shape (normalised spectrum), assumed to be independent of the level of seismic intensity, multiplied by the peak horizontal ground acceleration (a_gS) applicable at the construction site.

The horizontal elastic response spectrum is defined by the following expressions:

$$
\begin{aligned}
&0 \le T < T_B && S_e(T) = a_g \cdot S \cdot \left(1 + \frac{T}{T_B} \cdot (\eta \cdot 2.5 - 1)\right) \\
&T_B \le T < T_C && S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \\
&T_C \le T < T_D && S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \left(\frac{T_C}{T} \cdot\right) \\
&T_D \le T && S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \cdot \left(\frac{T_C T_D}{T^2}\right)
\end{aligned}
\qquad (3.2)
$$

where S_e = elastic response spectrum
S = soil amplification factor (independent of the vibration period);
η damping correction factor with reference value $\eta = 1$ for 5% viscous damping, which may be determined by the expression:

$$\eta = \sqrt{10/(5+\xi)} \geq 0{,}55 \qquad (3.3)$$

where ξ = □ viscous damping ratio[1] of the structure, expressed in percent
T = natural period of vibration of the simple oscillator;
T_B, T_C = limits of the constant spectral acceleration branch (Fig. 3.1), depending on ground type
T_D = value at which the constant displacement response range of the spectrum begins (Fig. 3.1), also depending on ground type

In the absence of detailed experimental data, the values of T_B, T_C, T_D and S for the horizontal components of motion and for the ground types defined in 2.5 given in Table 3.1 may be used.

Figure 3.1 – Shape of elastic response spectrum

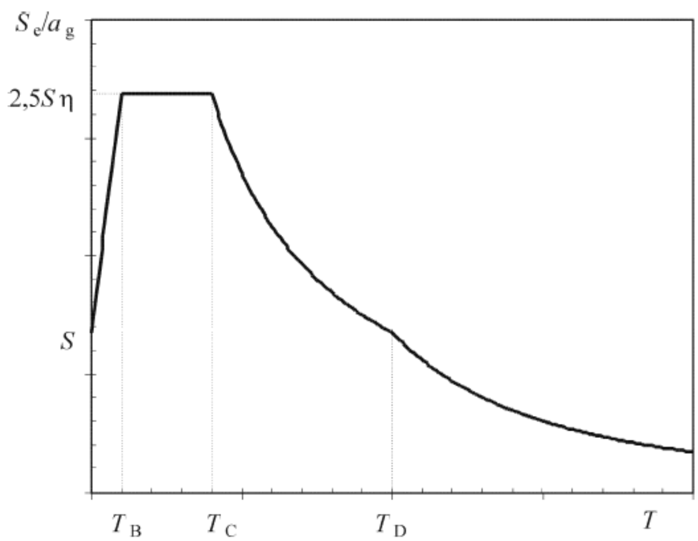

Table 3.1 - Values of the parameters describing the horizontal response spectrum in expressions (3.2)

Ground type	S	T_B	T_C	T_D
A	1.0	0.15	0.40	2.0
B, C, E	1.25	0.15	0.50	2.0
D	1.35	0.20	0.80	2.0

[1] In this document, the ratio will always be considered as the geometric ratio or for the quotient

The elastic response spectrum of the vertical component of earthquake motion is defined by the following expressions:

$$
\begin{aligned}
&0 \le T < T_B && S_{ve}(T) = 0.9 a_g \cdot S \cdot \left(1 + \frac{T}{T_B} \cdot (\eta \cdot 3.0 - 1)\right) \\
&T_B \le T < T_C && S_{ve}(T) = 0.9\, a_g \cdot S \cdot \eta \cdot 3.0 \\
&T_C \le T < T_D && S_{ve}(T) = 0.9\, a_g \cdot S \cdot \eta \cdot 3.0 \left(\frac{T_C}{T} \cdot\right) \\
&T_D \le T && S_{ve}(T) = 0.9\, a_g \cdot S \cdot \eta \cdot 3.0 \cdot \left(\frac{T_C T_D}{T^2}\right)
\end{aligned}
\tag{3.4}
$$

with the values of the parameters defining the spectral shape given in Table 3.2.

Table 3.2 - Values of the parameters describing the vertical response spectrum in expressions (3.4)

Ground type	S	T_B	T_C	T_D
A, B, C, D, E	1,0	0,05	0,15	1,0

When the ground profile at the construction site cannot be clearly assigned to one of the ground types defined in 2.5, and excluding in any case the ground types referred to in the last paragraph of 2.5, ground type D shall generally be adopted. If attribution to either one of two ground types is uncertain, the most conservative condition shall be adopted.

3.2 DESIGN SPECTRA

3.2.1 Design spectra for the no-collapse requirement

To avoid explicit inelastic structural analysis in design, the capacity of the structure to dissipate energy through mainly ductile behaviour may be taken into account by performing an elastic analysis based on a response spectrum reduced with respect to the elastic one, hereafter called “design spectrum”. This reduction is accomplished by introducing a factor reducing the elastic forces, denominated behaviour factor q. The design seismic action $S_d(T)$ is therefore given by the ordinate of the elastic response spectrum (3.2) introduced in 3.1, divided by q. The numerical values of the behaviour factor q are given for the various materials and structural systems, in accordance with 4.2.
The design spectrum for the horizontal components of the seismic action is defined by the following expressions:

$$0 \le T < T_B \qquad S_d(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot \left(\frac{2.5}{q} - 1\right)\right]$$

$$T_B \le T < T_C \qquad S_d(T) = a_g \cdot S \cdot \frac{2.5}{q}$$

$$T_C \le T < T_D \qquad S_d(T) = a_g \cdot S \cdot \frac{2.5}{q}\left(\frac{T_C}{T} \cdot\right) \tag{3.5}$$

$$T_D \le T \qquad S_d(T) = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left(\frac{T_C T_D}{T^2}\right)$$

where S, T_A, T_B, T_C, T_D have been defined in Table 3.1.
It will in any case be assumed $S_d(T) \ge 0.2a_g$.

In the absence of specific supporting analyses, for the vertical component of the seismic action a behaviour factor q = 1.5 should normally be adopted for all materials and structural systems.

3.2.2 Design spectra for the damage limitation requirement

The design spectrum to be adopted to meet the damage limitation requirement established in 1.2.1 may be obtained by dividing the ordinates of the elastic spectrum (3.2) by 2.5.

3.3 ALTERNATIVE REPRESENTATIONS OF THE SEISMIC ACTION: ACCELERATION TIME-HISTORIES

The seismic action may also be represented in terms of ground-acceleration time-histories.

When a spatial model of ground motion is required, the seismic action shall be represented by sets of three different accelerograms acting simultaneously along the three principal directions of the structure.

The response spectrum of the selected accelerograms shall be consistent with the elastic spectrum (3.2).

The duration of the accelerograms shall be consistent with the earthquake magnitude and the other relevant features of the seismic event underlying the establishment of a_g and S. When site-specific data are not available, the minimum duration of the stationary part of the accelerograms should not be less than 10 s.

The number of accelerograms or, for spatial analyses, of groups of accelerograms shall not be less than 3.

Artificial accelerograms shall be generated so as to match the elastic response spectra (3.2) and (3.3) for 5% viscous damping.

For a set of artificial accelerograms:

- the mean of the zero period response spectral acceleration values shall not be smaller than the the value of $a_g S$ for the site in question,
- no value of the mean 5% damped elastic spectrum – calculated from all time histories – shall be less than 90% of the corresponding value of the elastic spectrum (3.2) or (3.3) in the period intervals 0.15 s ÷ 2.0 s and 0.15 s ÷ 2T, where T is the fundamental elastic vibration period of the structure.

The use of recorded accelerograms – or of accelerograms generated through a physical simulation of earthquake source and travel path mechanisms – (in any case not less than 3 in number), is allowed provided that the time-histories are adequately qualified with regard to the seismogenic features of the sources and the site-specific soil conditions, and that their values are scaled to the value of $a_g S$ for the zone under consideration.

3.4 DESIGN GROUND DISPLACEMENT

Unless special studies indicate otherwise, the value d_g of the design ground displacement may be estimated by means of the following expression

$$d_g = 0{,}025 \cdot S \cdot T_C \cdot T_D \cdot a_g \qquad (3.6)$$

IV. GENERAL RULES AND PARAMETERS FOR STRUCTURAL CLASSIFICATION

4.1 IMPORTANCE CLASSES AND FACTORS

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.

The importance classes are characterised by different importance factors γ_I. 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, see 2.1), as appropriate for the design of the specific category of structures.

The definitions of the importance classes and the corresponding values of the importance factors are given in Table 4.1.

Table 4.1 - Importance classes for buildings

Importance class	Buildings	Importance factor (γ_I)
I	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.	1.4
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.	1.2
III	Ordinary buildings, not belonging to the other categories	1.0
IV	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.	0.8

4.2 STRUCTURAL TYPES AND BEHAVIOUR FACTORS

4.2.1 Structural types

Buildings shall be classified in structural types according to their behaviour under horizontal seismic actions, as follows:

a) **Very high ductility system:** structural systems in which both the vertical and lateral loads are mainly resisted by highly ductile spatial frames (concrete or steel) whose shear resistance at the building base exceeds 65% of the total shear resistance of the whole structural system.

b) **High ductility system:** structural systems in which resistance to lateral loads is contributed by ductile frames and/or by structural walls, single or coupled. Walls are fixed at the base so that the relative rotation of the base with respect to the rest of the structural system is prevented, designed and detailed to dissipate energy in a flexural plastic hinge zone free of openings or large perforations, just above its base.

c) **Moderate ductility system:** structural systems in which 50% or more of the mass is in the upper third of the height of the structure, or in which the dissipation of energy takes place mainly at the base of a single building element (Inverted pendulum system); or dual or wall system not having a minimum torsional rigidity (e.g. a structural system consisting of flexible frames combined with walls concentrated near the centre of the building in plan, Core system); or reinforced masonry systems properly detailed.

d) **Low ductility system:** structural systems in which the lateral loads are resisted by low ductility elements, such as unreinforced masonry systems.

Buildings may be classified as one type of structural system in one horizontal direction and as another in the other direction, only when in one direction an elastic response is assumed. In all other cases the lowest applicable force reduction factor shall be adopted.

High and very high ductility systems shall possess a minimum torsional rigidity in both horizontal directions, with vertical elements well distributed in plan. Frame, dual or wall systems without a minimum torsional rigidity be classified as moderate ductility systems.

4.2.2 Behaviour factors

The behaviour factor q, introduced in 3.2 to account for energy dissipation capacity, shall be derived for each design direction as follows:

$$q = q_o \cdot k_D \cdot k_R \cdot k_O \geq 1{,}5 \tag{4.1}$$

where q_o = basic value of the behaviour factor, dependent on the type of the structural system, as discussed above and specifically defined for each construction material

k_D = factor reflecting the level of detailing, influencing the local plastic deformation capacity of the structural elements

k_R = factor depending on the vertical regularity

k_O = factor reflecting the structural redundancy and the overstrength of the system

The basic values q_o for the various structural types can be assigned to the ranges given in Table 4.2.

Table 4.2 - Basic value q_0 of behaviour factor

STRUCTURAL TYPE	
Very high ductility systems	4.0 – 5.0
High ductility systems	3.0 – 4.0
Moderate ductility systems	2.0 – 3.0
Low ductility systems	1.5 – 2.0

The factor k_D can be assumed equal to 1.0 for high ductility detailing and to 0.7 for low ductility detailing.

The factor k_R can be assumed equal to 1.0 for structures regular in elevation and to 0.8 for structures irregular in elevation, as described in 4.3.3.

The factor k_O can be assumed as equal α_u / α_1 where α_1 and α_u are defined as follows:

α_1 multiplier of the horizontal seismic design action at first attainment of member flexural resistance anywhere in the structure, while all other design actions remain constant.

α_u multiplier of the horizontal seismic design action, with all other design actions constant, at formation of plastic hinges in a number of sections sufficient for the development of overall structural instability. Factor α_u may be obtained from a geometric first-order global inelastic analysis.

When k_O is not evaluated through calculations, the following approximate values may be used:

a) Frames or frame-equivalent dual systems:

- One-storey buildings: k_O =1.1
- Multi-storey, one-bay frames: k_O =1.2
- Multi-storey, multi-bay frames or frame-equivalent dual structures: k_O =1.3

b) Wall- or wall-equivalent dual systems:

- Wall systems with only two uncoupled walls per horizontal direction: k_O =1.0
- Other uncoupled wall systems: k_O =1.1
- Wall-equivalent dual, or coupled wall systems: k_O =1.2.

Values of α_u / α_1 higher than those given above are allowed, provided that they are confirmed through a nonlinear static (pushover) global analysis.

The maximum value of α_u / α_1 to be used in design is equal to 1.5, even when the analysis results in higher values.

4.3 CHARACTERISTICS OF EARTHQUAKE RESISTANT BUILDINGS AND STRUCTURAL REGULARITY

4.3.1 General

The guiding principles governing the 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;
- structural redundancy.

Structural simplicity

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.

Uniformity, symmetry and redundancy

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 6.1.

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.

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

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.

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.

Bi-directional resistance and stiffness

Horizontal seismic motion is a bi-directional phenomenon and thus the building structure shall be able to resist horizontal actions in any direction. The structural elements should therefore be arranged in an orthogonal in-plan structural pattern, ensuring similar resistance and stiffness characteristics in both main directions.

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.

Torsional resistance and stiffness

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.

Diaphragmatic behaviour at storey level

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).

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.

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 section 5, particularly when there are significant changes in stiffness or offsets of vertical elements above and beneath the diaphragm.

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 center of gravity.

Adequate foundation

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.

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.
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.

Structural redundancy

A high degree of redundancy accompanied by redistribution capacity shall be sought, enabling a more widely spread energy dissipation and an increased total dissipated energy. Consequently structural systems of lower static indeterminacy are assigned lower behaviour factors (see 4.2.2). The necessary redistribution capacity is achieved through local ductility rules given for each construction material.

4.3.2 Regularity

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

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 irregularity in elevation.

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 4.3).

Table 4.3 - 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 conditions of 5.3 are also met.
** 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

Criteria describing regularity in plan and in elevation are given in 4.3.3 and 4.3.4, rules concerning modelling and analysis are given in section 5.

The regularity criteria 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.

4.3.3 Criteria for regularity in plan

A building is *regular in plan* when:

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

b) 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.

c) 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.

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

e) All lateral load resisting systems, like cores, structural walls or frames, run without interruption from the foundations to the top of the building and the deflected shapes of the individual systems under horizontal loads are not very different.

4.3.4 Criteria for regularity in elevation

A building is regular in elevation when:

a) 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.

b) 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.

c) 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 6.6

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

- 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. 4.1a and 4.1b),
- 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. 4.1c). 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.
- 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. 4.1d).

4.3.5 Primary and secondary seismic members

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 structural members. 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-□ effects) should be made in the design of these members.

It is recommended that the contribution of all secondary seismic elements to lateral stiffness does not exceed 15% of that of all primary elements.

The designation of some structural elements as secondary seismic is not allowed to change the classification of the structure according to 4.3.3 and 4.3.4 from non-regular to regular.

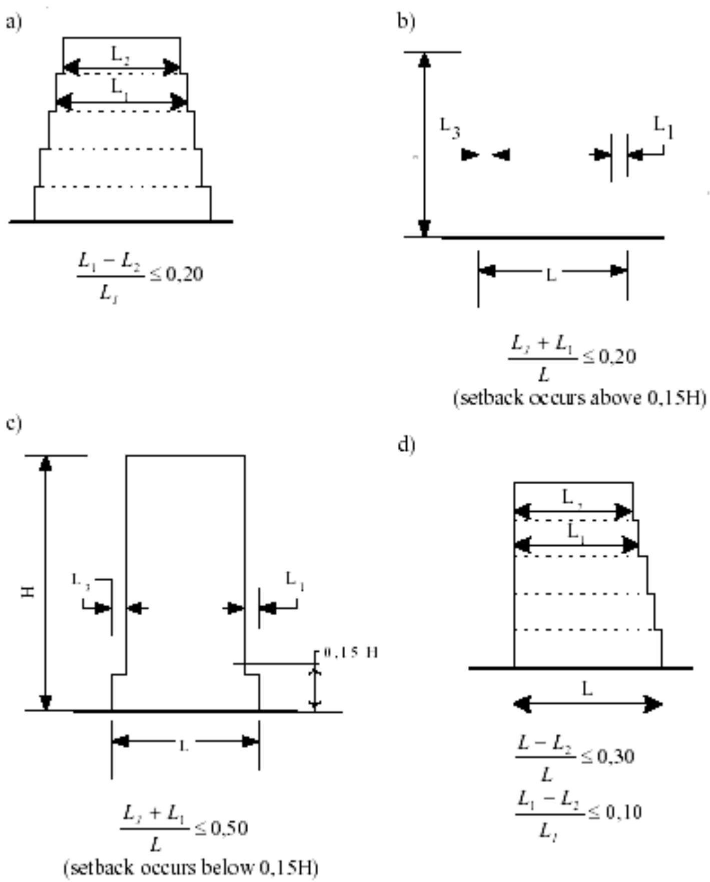

Figure 4.1 - Criteria for regularity of buildings with setbacks

4.3.6 Additional measures

Due to the random nature of the seismic action and the uncertainties of the post-elastic cyclic behaviour of concrete structures, the overall uncertainty is substantially higher than under non-seismic actions. Therefore measures shall be taken to reduce uncertainties related to the structural configuration, to the analysis, to the resistance and to the ductility.

Important resistance uncertainties may be produced by geometric errors. To minimize this type of uncertainties, the following rules shall be applied:

a) Certain minimum dimensions of the structural elements shall be respected to decrease the sensitivity to geometric errors.

b) The ratio of minimum to maximum dimension of linear elements shall be limited, to minimize the risk of lateral instability of these elements.

c) Storey drifts shall be limited, to limit P-Δ effects in the columns (see 6.4).

d) Account shall be taken of reversals of moment not predicted by the analysis.

To minimize ductility uncertainties, the following rules shall be observed:

a) A minimum of local ductility shall be provided in all primary seismic elements, independently of the ductility class adopted in design.

b) An appropriate limit of the normalised design axial force shall be respected.

4.4 DUCTILITY OF ELEMENTS AND COMPONENTS

4.4.1 Global ductility conditions

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.

Specific material related requirements shall be satisfied, including - when appropriate - 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.

These requirements are deemed to be satisfied if:

a) plastic mechanisms obtained by pushover analysis are satisfactory;

b) global, inter-storey 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.

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

To satisfy this requirement, at all beam-column joints of frame buildings, including frame-equivalent ones (i.e. dual system in which the shear resistance of the frame system at the building base is higher than 50% of the total shear resistance of the whole structural system), with two or more storeys, the following condition should be satisfied:

$$\sum M_{Rc} \geq 1{,}3 \sum M_{Rb} \tag{4.2}$$

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 (4.2).

$\sum M_{Rb}$ = 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 $\sum M_{Rb}$.

Expression (4.2) 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 (4.2) should be satisfied just within the vertical plane through that direction.

This requirement is waived at the top level of multi-storey buildings.

4.4.2 Local ductility conditions

For the overall ductility of the structure to be achieved, the potential regions for plastic hinge formation - to be defined for each type of building element – shall possess high plastic rotational capacities.

This is deemed to be satisfied if a sufficient curvature ductility is provided in all critical regions of primary seismic elements, including column ends (depending on the potential for plastic hinge formation in columns), and local buckling of compressed steel within potential plastic hinge regions of primary seismic elements is prevented.

Unless more precise data are available, the curvature ductility factor μ_ϕ of these regions (defined as the ratio of the post-ultimate strength curvature, at 85% of the moment of resistance, to the curvature at yield, provided that the limiting strains of materials are not exceeded) is higher than the following values:

$$\mu_\phi = 2q - 1 \qquad \text{if } T_1 \geq T_C \qquad (4.3)$$

$$\mu_\phi = 1 + 2(q - 1)T_C/T_1 \qquad \text{if } T_1 < T_C \qquad (4.4)$$

where q = value of the behaviour factor used in the analysis
T_1 = fundamental period of the building, both within the vertical plane in which bending takes place and
T_C = period at the upper limit of the constant acceleration region of the spectrum.

As an example, in reinforced concrete elements, appropriate concrete and steel qualities are adopted to ensure local ductility as follows:

- the steel used in critical regions of primary seismic elements should have high uniform plastic elongation.
- the tensile strength to yield strength ratio of the steel used in critical regions of primary seismic elements is adequately higher than unity.
- the concrete used in primary seismic elements possesses a minimum of strength and a minimum of post-ultimate-strength deformation capacity.

V. DESIGN FORCES, METHODS OF ANALYSIS AND DRIFT LIMITATIONS

5.1 LOAD COMBINATIONS INCLUDING ORTHOGONAL SEISMIC LOAD EFFECTS

5.1.1 Horizontal components of the seismic action

In general the horizontal components of the seismic action shall be considered as acting simultaneously.

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

a) The structural response to each component may be evaluated separately, using the combination rules for modal responses given in 5.2.3.

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.

As an alternative to the rules given 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} "+" 0.30 E_{Edy} (5.1)

b) 0.30 E_{Edx} "+" E_{Edy} (5.2)

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

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

When using non-linear static (pushover) analysis and applying a spatial model, E_{Edx} should be considered as the forces and deformations due to the target displacement in the x direction and E_{Edy} as 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.

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.

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, the seismic action may be assumed to act separately and without the combinations given above, along the two main orthogonal horizontal axes of the structure.

5.1.2 Vertical component of the seismic action

The vertical component of the seismic action, as defined in section 3, should be taken into account in the following cases, provided α_{vg} is greater than 0,25 g:

- Horizontal or nearly horizontal structural members spanning 20 m or more;
- Horizontal or nearly horizontal cantilever components longer than 5 m;
- Horizontal or nearly horizontal pre-stressed components;
- Beams supporting columns;
- Base-isolated structures.

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.

The effects of the vertical component need be taken into account only for the elements under consideration and their directly associated supporting elements or substructures.

If the horizontal components of the seismic action are also relevant for these elements, the combination rules in 5.1.1 may be applied, extended to three components of the seismic action. Equations (5.1) and (5.2) are modified as follows, including all three of the following combinations for the computation of the action effects:

a) $0.30\, E_{Edx}$ "+" $0.30\, E_{Edy}$ "+" E_{Edz} (5.3)

b) E_{Edx} "+" $0.30\, E_{Edy}$ "+" $0.30\, E_{Edz}$ (5.4)

c) $0.30\, E_{Edx}$ "+" E_{Edy} "+" $0.30\, E_{Edz}$ (5.5)

where: "+" implies "to be combined with"
E_{Edx} and E_{Edy} : see above,
E_{Edz} = action effects due to the application of the vertical component of the design seismic action as defined in section 3.

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

5.1.3 Combination of the seismic action with other actions

The seismic action shall be combined with other actions according to the following rule:

$$E + G_K + P_K + \sum_i (\psi_{ji} Q_{Ki}) \qquad (5.6)$$

where: E = seismic action as appropriate for the considered limit state, accounting for the importance factor
G_K = characteristic (95 – percentile) value of the action due to permanent gravity load;
P_K = characteristic value of the prestressing action
ψ_{ji} = ψ_{2i} combination coefficient to obtain the quasi – permanent value of the variable action Q_i, to be used at the no – collapse limit state
ψ_{0i} = combination coefficient to obtain the rare value of the variable action Q_i, to be used at the damage control limit state
Q_{Ki} = characteristic value of variable action Q_i.

The seismic action effects shall be computed considering the masses associate to the following gravity load combination:

$$G_K + \sum_i (\psi_{Ei} Q_{Ki}) \qquad (5.7)$$

where ψ_{Ei} = combination coefficient to be associated with the variable action Q_i, to consider the reduced probability that all variable actions $\psi_{0i} Q_{Ki}$ (damage) or $\psi_{2i} Q_{Ki}$ (no – collapse) are present, and is obtained multiplying ψ_{0i} or ψ_{2i} by φ.

Recommended coefficient values are given in the following tables:

Table 5.1 - Coefficients ψ_{0i}, ψ_{2i} as a function of occupancy or load type

Occupancy or load type	ψ_{0i}	ψ_{2i}
Residential, private offices	0,70	0,30
Public offices, schools, commercial	0,70	0,60
Volcanic ashes	0,70	0,35
Warehouse	1,00	0,80
Wind	0,00	0,00

Table 5.2 - Coefficient φ

		φ
Uncorrelated loads	Last floor	1,0
	Other floors	0,5
Partially correlated loads	Last floors	1,0
	Floors with correlated loads	0,8
	Other floors	0,5

5.2 METHODS OF ANALYSIS

5.2.1 General

In the design of buildings, the seismic effects and the effects of the other actions included in the seismic design situation, may be determined on the basis of four different methods:

a) Linear static procedures

b) Mode superposition procedures

c) Non linear static (pushover) procedures

d) Non linear dynamic (time history) procedures.

The reference method for determining the seismic effects is the modal response spectrum analysis (item b) above), using a linear-elastic model of the structure and the design spectrum given in section 3. It is applicable to all types of buildings, as described in 5.2.3.

Linear static procedures may be used for buildings meeting the conditions given in 5.2.2.

Non-linear methods may be used under the conditions specified in 5.2.4 (non-linear static (pushover) analysis and non-linear time history (dynamic) analysis).

5.2.2 Linear Static Procedures

General

This type of analysis may be applied to buildings the response of which is not significantly affected by contributions from higher modes of vibration.

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

$$T_1 \le \begin{cases} 4 \cdot T_C \\ 2{,}0\,\mathrm{s} \end{cases} \quad (5.6$$

where T_C is given in Table 3.1,

b) they meet the criteria for regularity in elevation given in 4.3.4.

Base shear force

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

$$F_b = S_d(T_1) \cdot m \cdot \lambda \quad (5.7)$$

where $S_d(T_1)$ = ordinate of the design spectrum (see section 3) 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
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 $\lambda = 1{,}0$ otherwise.

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.

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

$$T_1 = C_t \cdot H^{3/4} \tag{5.8}$$

where $C_t = \begin{cases} 0{,}085 \text{ for moment resistant space steel frames} \\ 0{,}075 \text{ for moment resistant space concrete frames and for eccentrically braced steel frames} \\ 0{,}050 \text{ for all other structures} \end{cases}$

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

Alternatively, the value C_t in expression (5.8) for structures with concrete or masonry shear walls may be taken as

$$C_t = 0{,}075 / \sqrt{A_c} \tag{5.9}$$

where $$A_c = \Sigma \left[A_i \cdot \left(0{,}2 + \left(l_{wi} / H\right)\right)^2 \right] \tag{5.10}$$

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 defined 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.

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

$$T_1 = 2 \cdot \sqrt{d} \tag{5.11}$$

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

Distribution of the horizontal seismic forces

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.

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

$$F_i = F_b \cdot \frac{s_i \cdot m_i}{\Sigma s_j \cdot m_j} \tag{5.12}$$

where: F_i = horizontal force acting on storey i
F_b = seismic base shear according to expression (5.7)
s_i,s_j = Displacements of masses m_i, m_j in the fundamental mode shape.

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

$$F_i = F_b \cdot \frac{z_i \cdot m_i}{\Sigma z_j \cdot m_j} \tag{5.13}$$

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

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

Simplified procedures

The analysis may be performed using two planar models, one for each main horizontal direction, if the criteria for regularity in plan are satisfied (see 4.3.3).

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 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.

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 above except (e), provided all seismic action effects from the analysis are multiplied by 1,25.

Buildings not complying with the criteria above, shall be analysed using a spatial model.

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

5.2.3 Mode superposition methods

<u>General</u>

The mode superposition type of analysis shall be applied to buildings which do not satisfy the conditions given in 5.2.2 for applying the lateral force method of analysis, and is applicable to all types of structures.

The response of all modes of vibration contributing significantly to the global response of the building shall be taken into account.

This requirement 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.

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

Alternatively, the minimum number k of modes to be taken into account in a spatial analysis should satisfy the following conditions:

$$k \geq 3 \cdot \sqrt{n} \tag{5.14a}$$

and

$$T_k \leq 0{,}20\ s \tag{5.14b}$$

where: k = number of modes taken into account
n = number of storeys above ground
T_k = period of vibration of mode k

<u>*Combination of modal responses*</u>

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 Ti and Tj satisfy (with Tj ≤□Ti) the following condition:

$$T_j \leq 0{,}9 \cdot T_i \tag{5.15}$$

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

$$E_E = \sqrt{\Sigma\, E_{Ei}{}^2} \tag{5.16}$$

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.

If eq. 5.15 is not satisfied, more accurate procedures for the combination of the modal maxima shall be adopted, e.g. using procedures such as the "Complete Quadratic Combination".

5.2.4 Non-Linear Methods.

<u>*General*</u>

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.

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

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. 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.

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.

Unless otherwise specified, element properties should be based on mean values of the properties of the materials.

Gravity loads shall be applied to appropriate elements of the mathematical model.

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.

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

Non-linear static (pushover) analysis

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 values of the overstrength ratio α_u/α_1 introduced in 4.2.2;

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

c) to assess the structural performance of existing or retrofitted buildings;

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

Buildings not complying with the regularity criteria of 4.3 shall be analysed using a spatial model.

For buildings complying with the regularity criteria of 4.3 the analysis may be performed using two planar models, one for each main horizontal direction.

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

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.

Lateral loads shall be applied at the location of the masses in the model. Accidental eccentricity shall be considered.

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 below.

The control displacement may be taken at the centre of mass at the roof of the building.

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.

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.

Target displacement is defined as the seismic demand derived from the elastic response spectrum of section 3 in terms of the displacement of an equivalent single-degree-of-freedom system.

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.

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.

If two planar models are used for analysis of structures regular in plan, the torsional effects may be estimated according to 5.3.

Non-linear time-history analysis

The time-dependent response of the structure may be obtained through direct numerical integration of its differential equations of motion, using the acceleration time series defined in 3.3 to represent the ground motions.

The element models 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.

If the response is obtained from at least 7 nonlinear time-history analyses to ground motions according to 3.3, the average of the response quantities from all these analyses should be used as action effect E_d. Otherwise, the most unfavourable value of the response quantity among the analyses should be used as E_d.

5.3 TORSIONAL CONSIDERATIONS

5.3.1 Additional accidental eccentricity

If the lateral stiffness and mass are symmetrically distributed in plan and unless accidental eccentricities is not taken into account by a more exact method, the accidental torsional effects may be accounted for by multiplying the action effects resulting in the individual load resisting elements with a factor δ given by:

$$\delta = 1 + 0.6 \cdot \frac{x}{L_e} \tag{5.17}$$

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.

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 (5.19) and applying expression (5.17) above with factor 0.6 increased to 1.2.

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

5.3.2 Additional accidental eccentricity for simplified analysis

Whenever a spatial model is used for the analysis, the accidental torsional effects referred in 5.3.3 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*:

$$M_{1i} = e_{1i} \cdot F_i \tag{5.18}$$

where: M_{1i} = torsional moment applied at storey *i* about its vertical axis,
e_{1i} = accidental eccentricity of storey mass *i* according to expression (5.19) for all relevant directions,
F_i = horizontal force acting on storey *i* for all relevant directions.

The effects of the loading should be taken into account with positive and negative signs (the same for all storeys).

Whenever two separate planar models are used for the analysis, the torsional effects may be accounted for by applying the rules of 5.3.1 to the action effects.

5.3.3 Accidental torsional effects

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:

$$e_{1i} = \pm 0{,}05 \cdot L_i \tag{5.19}$$

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.

5.4 DRIFT LIMITATIONS

Limitations are specified to limit the damage in case of an event with higher probability of exceedance, as defined in 1.2.1.

Allowed drift is between 0.5 and 0.75 % of the storey height, depending on the properties of non – structural elements, as defined below:

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

$$d_r \cdot \nu \leq 0.005 \cdot h \tag{5.20}$$

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

$$d_r \cdot \nu \leq 0.0075 \cdot h \, . \tag{5.21}$$

where: d_r = design interstorey drift
h = storey height
ν = reduction factor to take into account the lower return period of the seismic action associated with the damage limitation state (see 3.2.2).

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” (section 3).

5.5 SOIL-STRUCTURE INTERACTION CONSIDERATIONS

The effects of dynamic soil-structure interaction shall be taken into account in:

a) buildings where P-Δ (2nd order) effects play a significant role

b) buildings supported on very soft soils, with average shear wave velocity V_{s30} (as defined in 2.5) less than 100 m/s.

The effects of soil-structure interaction on foundation piles shall be assessed by taking into account both the inertia forces transmitted by the superstructure, and the kinematic forces arising from the deformation of the surrounding soil due to the passage of seismic waves.

VI. SAFETY VERIFICATIONS

6.1 BUILDING SEPARATION

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

This 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 (6.1);

(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 (6.1).

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.

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:

$$d_s = q_d \, d_e \tag{6.1}$$

where: d_s = displacement of a point of the structural system induced by the design seismic action

q_d = displacement behaviour factor, normally assumed equal to q

d_e = displacement of the same point of the structural system, as determined by a linear analysis based on the design response spectrum according to 3.2

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

6.2 RESISTANCE OF HORIZONTAL DIAPHRAGMS

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.

This requirement 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.

6.3 REQUIREMENTS FOR FOUNDATIONS

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

Only one foundation type should in general be used for the same structure, unless the latter consists of dynamically independent units.

6.4 P-Δ CONSIDERATIONS

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

$$\theta = \frac{\mathrm{P}_{\mathrm{tot}} \cdot d_r}{V_{tot} \cdot h} \leq 0.10 \qquad (6.2)$$

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
V_{tot} = total seismic storey shear
h = interstorey height.

In any case it shall be $\theta < 0.3$.

If $0.1 < \theta < 0.2$, the second-order effects may approximately be taken into account by multiplying the relevant seismic action effects by a factor equal to $1/(1 - \theta)$.

If design action effects E_d are obtained through a nonlinear method of analysis, then the previous condition should be applied in terms of forces only for brittle elements. For dissipative zones, which are designed and detailed for ductility, the resistance condition 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.

6.5 DESIGN AND DETAILING OF SECONDARY AND NON – STRUCTURAL SEISMIC ELEMENTS

6.5.1 General

The present subsection applies to elements designated as secondary seismic (see 4.3.5), which are subjected to significant deformations in the seismic design situation. Such elements shall be designed and detailed to maintain their capacity to support the gravity loads present in the seismic design situation, when subjected to the maximum deformations under the seismic design situation.

Maximum deformations due to the seismic design situation shall be calculated calculated from an analysis of the structure for the seismic design situation, in which the contribution of secondary seismic elements to lateral stiffness is neglected and primary seismic elements are modelled with their cracked flexural and shear stiffness.

6.5.2 Non-structural components

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.

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.

In all other cases properly justified simplifications of this procedure are allowed.

<u>*Analysis*</u>

The non-structural elements, as well as their connections and attachments or anchorages, shall be verified for the seismic design situation.

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:

$$F_a = (S_a \cdot W_a \cdot \gamma_a)/q_a \qquad (6.3)$$

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, as defined below,
γ_a = importance factor of the element,
q_a = behaviour factor of the element, see Table 6.1.

The seismic coefficient S_a may be calculated as follows:

$$S_a = 2 \cdot \alpha \cdot S \cdot (1 + z/H)/\left(1 + (1 - T_a/T_1)^2\right) \qquad (6.4)$$

where: α = ratio of the design ground acceleration on type A ground, to the acceleration of gravity a_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

<u>*Importance factors*</u>

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.

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.

<u>*Behaviour factors*</u>

Values of the behaviour factor q_a for non-structural elements are given in Table 6.1**.**

Table 6.1: 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

6.6 MEASURES FOR MASONRY INFILLED FRAMES

6.6.1 General

This subsection applies to frame or frame equivalent dual concrete systems of and to mixed steel or (steel-concrete) composite structures of with interacting non-engineered masonry infill walls that fulfill 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.

For wall or wall-equivalent-dual concrete systems, as well as for braced steel or steel-concrete composite systems, the interaction with the masonry infill walls may be neglected.

If engineered masonry infill walls constitute part of the seismic resistant structural system, analysis and design should be carried out according to appropriate criteria and rules.

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

6.6.2 Requirements and criteria

The consequences of irregularity in plan produced by the infill walls shall be taken into account.

The consequences of irregularity in elevation produced by the infill walls shall be taken into account.

Account shall be taken of the high uncertainties related to the behaviour of the infill walls (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).

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 infill walls) shall be taken into account.

6.6.3 Irregularities due to masonry infill walls

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

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. Infill walls should be included in the model and a sensitivity analysis regarding the position and the properties of the Infill walls 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 Infill walls are concentrated) against the effects of any torsional response caused by the Infill walls.

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.

When the masonry Infill walls 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.

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

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

$$\eta = \left(1 + \Delta V_{Rw} / \sum V_{Sd}\right) \leq q \qquad (6.5)$$

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.

If expression (6.5) leads to a magnification factor η lower than 1.1, there is no need for such a modification of action effects.

6.6.4 Damage limitation of infill walls

For the structural systems, except in cases of low and very low seismicity (section 2) 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.

Examples of measures 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, insertion of steel trusses in mortar beds.

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

VII. PROVISIONS FOR BASE ISOLATION

7.1 GENERAL

This section covers the design of seismically isolated structures in which the isolating system, located below the main mass of the structure, aims at reducing the seismic response of the lateral-force resisting system.

The reduction of the seismic response of the lateral-force resisting system may be obtained by increasing the fundamental period of the seismically isolated structure, by modifying the shape of the fundamental mode and by increasing the damping, or by a combination of these effects. The isolating system may consist of linear or non-linear springs and/or dampers.

This section does not cover passive energy dissipation systems that are not arranged on a single interface, but are distributed over several storeys or levels of the structure.

7.2 COMPLIANCE CRITERIA

The following limit states shall be checked:

- No – collapse 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.

At the Damage limitation limit state, all lifelines crossing the joints around the isolated structure shall remain within the elastic range, and the interstorey drift should be limited in the substructure and the superstructure according to 5.4.

At the No – collapse limit states:

- The ultimate capacity of the isolating devices in terms of strength and deformability shall not be exceeded, with the relevant safety factors;
- Although it may be acceptable that, in certain cases, the substructure has inelastic behaviour, it is considered in the present subsection that it remains in the elastic range;

- The isolating devices may attain their ultimate capacity, while the superstructure and the substructure remain in the elastic range. Then there is no need for capacity design and ductile detailing in either the superstructure or the substructure;
- Gas lines and other hazardous lifelines crossing the joints separating the superstructure from the surrounding ground or constructions shall be designed to accommodate safely the relative displacement between the isolated superstructure and surrounding ground or constructions, taking into account a safety factor $\gamma_x = 1.2$.

7.3 GENERAL DESIGN PROVISIONS

7.3.1 Devices and control of undesirable movements

Sufficient space between the superstructure and substructure shall be provided, together with other necessary arrangements, to allow inspection, maintenance and replacement of the devices during the lifetime of the structure.

If necessary, the devices should be protected from potential hazardous effects, such as fire, chemical or biological attack.

Materials used in the design and construction of the devices should conform to the relevant existing norms.

To minimise torsional effects, the effective stiffness centre and the centre of damping of the isolating system should be as close as possible to the projection of the centre of mass on the isolation interface.

To minimise different behaviour of isolating devices, the compressive stress induced in them by the permanent actions should be as uniform as possible.

Design provisions shall prevent uncontrolled sliding between the isolating devices and the substructure or the superstructure in the seismic and the other design situations.

Devices, the behaviour of which can induce uncontrolled shocks or torsional movements, shall not be used. This is deemed to be satisfied if potential shock effects are avoided through appropriate devices (e.g. dampers, shock-absorbers, etc.).

7.3.2 Control of differential seismic ground motions

The structural elements located above and below the isolation interface should be sufficiently rigid in both horizontal and vertical directions, so that the effects of differential seismic ground displacements are minimised.

This is considered satisfied if all the conditions stated below are satisfied:

a) A rigid diaphragm is provided above and under the isolating system, consisting of a reinforced concrete slab or a grid of tie-beams, designed taking into account all possible local and global modes of buckling. This rigid diaphragm is not necessary if the structures consist of rigid boxed structures;

b) The devices constituting the isolating system are fixed at both ends to the rigid diaphragms defined above, either directly or, if not practicable, by means of vertical elements, the relative horizontal displacement of which in the seismic design situation should be lower than 1/20 of the relative displacement of the isolating system.

Sufficient space shall be provided between the isolated superstructure and the surrounding ground or constructions, to allow its displacement in all directions in the seismic design situation.

7.4 SEISMIC ACTION

The three components of the seismic action shall be assumed to act simultaneously.

Each component of the seismic action is defined in terms of the elastic spectrum (see section 3) for the applicable local ground conditions and design ground acceleration Sa_g.

In buildings of importance class I, site-specific spectra including near source effects should also be taken into account, if the building is located at a distance less than 15 km from the nearest potentially active fault with a magnitude $M_s \geq 6,5$. Such spectra should not be taken less than the standard spectra defined in section 3.

If time-history analyses are required, a set of at least three ground motion records should be used.

7.5 BEHAVIOUR FACTOR

The value of the behaviour factor shall be taken equal to $q = 1$.

7.6 PROPERTIES OF THE ISOLATING SYSTEM

Values of physical and mechanical properties of the isolating system to be used in the analysis shall be the most unfavourable ones to be attained during the lifetime of the structure. They shall reflect, where relevant, the influence of:

- rate of loading;
- magnitude of the simultaneous vertical load;
- magnitude of simultaneous horizontal load in the transverse direction;
- temperature;
- change of properties over projected service life.

Accelerations and inertia forces induced by the earthquake should be evaluated taking into account the maximum value of the stiffness and the minimum value of the damping and friction coefficients.

Displacements should be evaluated taking into account the minimum value of stiffness and damping and friction coefficients.

In buildings of importance classes III and IV, mean values of physical and mechanical properties may be used, provided that extreme (maximum or minimum) values do not differ by more than 15% from the mean values.

7.7 STRUCTURAL ANALYSIS

7.7.1 General

The dynamic response of the structural system shall be analysed in terms of accelerations, inertia forces and displacements.

Torsional effects, including the effects of the accidental eccentricity defined in 5.3, shall be taken into account.

Modelling of the isolating system should reflect with a sufficient accuracy the spatial distribution of the isolator units, so that the translation in both horizontal directions, the corresponding overturning effects and the rotation about the vertical axis are adequately accounted for. It should reflect adequately the characteristics of the different types of units used in the isolating system.

7.7.2 Equivalent linear analysis

Subject to the conditions given below, the isolating system may be modelled with equivalent linear visco-elastic behaviour, if it consists of devices such as laminated elastomeric bearings, or with bilinear hysteretic behaviour if the system consists of elasto-plastic type of devices.

If an equivalent linear model is used, the effective stiffness of each isolating unit (i.e. the secant value of the stiffness at the total design displacement, d_{db}, of the unit) should be used. The effective stiffness K_{eff} of the isolating system is the sum of the effective stiffnesses of the isolating units.

If an equivalent linear model is used, the energy dissipation of the isolating system should be expressed in terms of an equivalent viscous damping, as the "effective damping" (ξ_{eff}). The energy dissipation in bearings should be expressed from the measured energy dissipated in cycles with frequency in the range of the natural frequencies of the modes considered. For higher modes outside this range, the modal damping ratio of the complete structure should be that of a fixed base superstructure.

When the effective stiffness or the effective damping of certain isolator units depend on the design displacement d_{dc} (displacement of the effective stiffness centre of the isolating system in the direction considered) , an iterative procedure should be applied, until the difference between assumed and calculated values of d_{dc} does not exceed 5% of the assumed value.

The behaviour of the isolating system may be considered as equivalent linear if all the following conditions are met:

a) The effective stiffness of the isolating system is at least 50% of the effective stiffness at a displacement of $0.2d_{dc}$;

b) The effective damping ratio of the isolating system does not exceed 30%;

c) The force-displacement characteristics of the isolating system do not vary by more than 10% due to the rate of loading or due to the vertical loads.

d) The increase of the restoring force in the isolating system for displacements between $0{,}5d_{dc}$ and d_{dc} is at least 2.5% of the total gravity load above the isolating system.

If the behaviour of the isolating system is considered as equivalent linear and the seismic action is defined through the elastic spectrum, a damping correction should be performed according to section 3.

7.7.3 Simplified linear analysis

The simplified linear analysis method considers two horizontal dynamic translations and superimposes static torsional effects. It assumes that the superstructure is a rigid solid translating above the isolating system, subject to the conditions given below. Then the effective period of translation is:

$$T_{eff} = 2\pi\sqrt{\frac{M}{K_{eff}}} \tag{7.1}$$

where: M = mass of the superstructure
K_{eff} = effective horizontal stiffness of the isolating system.

The torsional movement about the vertical axis may be neglected in the evaluation of the effective horizontal stiffness and in the simplified linear analysis if, in each of the two principal horizontal directions, the total eccentricity (including the accidental eccentricity) between the stiffness centre of the isolating system and the vertical projection of the centre of mass of the superstructure does not exceed 7,5% of the length of the superstructure transverse to the horizontal direction considered. This is a condition for the application of the simplified linear analysis method.

The simplified method may be applied to isolating systems with equivalent linear damped behaviour, if they comply also with all of the following conditions:

a) the distance from the site to the nearest potentially active fault with a magnitude $M_s \geq 6{,}5$ is greater than 15 km;

b) the largest dimension of the superstructure in plan is less than 50 m;

c) the substructure is sufficiently rigid to minimise the effects of differential displacements of the ground;

d) all devices are located above elements of the substructure which support the vertical loads;

e) the effective period T_{eff} satisfies the following condition:

$$3T_f \leq T_{eff} \leq 3s \tag{7.2}$$

where: T_f = fundamental period of the superstructure with a fixed base (estimated through a simplified expression).

f) The lateral-load resisting system of the superstructure is regularly and symmetrically arranged along the two main axes of the structure in plan;

g) The rocking rotation at the base of the substructure is negligible;

h) The ratio between the vertical and the horizontal stiffness of the isolating system satisfies the following condition:

$$\frac{K_v}{K_{eff}} \geq 150 \tag{7.3}$$

i) The fundamental period in the vertical direction, T_V, is not longer than 0.1 s,

where:

$$T_V = 2\pi\sqrt{\frac{M}{K_V}} \tag{7.4}$$

The displacement of the stiffness centre due to the seismic action should be calculated in each horizontal direction, through the following expression:

$$d_{dc} = \frac{M\ S_e(T_{eff}, \xi_{eff})}{K_{eff,\min}} \tag{7.5}$$

where: $S_e(T_{eff}, x_{eff})$ = pectral acceleration defined in section 3, taking into account the appropriate value of effective damping ξ_{eff}.

The horizontal forces applied at each level of the superstructure should be calculated, in each horizontal direction through the following expression:

$$f_j = m_j\ S_e(T_{eff}, \xi_{eff}) \tag{7.6}$$

where m_j is the mass at level j

The system of forces considered above induces torsional effects due to the combined natural and accidental eccentricities.

If the condition above for neglecting torsional movement about the vertical axis is satisfied, the torsional effects in the individual isolator units may be accounted for by amplifying in each direction the action effects with a factor δ_{xi} given (for the action in the x direction) by:

$$\delta_{xi} = 1 + \frac{e_{tot,y}}{r_y^2} y_i \qquad (7.7)$$

where:
- y = horizontal direction transverse to the direction x under consideration,
- (x_i, y_i) = co-ordinates of the isolator unit i relative to the effective stiffness centre,
- $e_{tot,y}$ = total eccentricity in the y direction,
- r_y = torsional radius of the isolating system, as given by the following expression:

$$r_y^{\ 2} = \sum \left(x_i^{\ 2} K_{yi} + y_i^{\ 2} K_{xi} \right) / \sum K_{xi} \qquad (7.8)$$

K_{xi} and K_{yi} being the effective stiffness of a given unit i in the x and y directions, respectively

Torsional effects in the superstructure should be estimated according to 5.3.

7.7.4 Modal simplified linear analysis

If the behaviour of the devices may be considered as equivalent linear but all the conditions given above to allow to performing a simplified linear analysis are not met, a modal analysis may be performed.

7.7.5 Time-history analysis

If an isolating system can not be represented by an equivalent linear model, the response shall be evaluated by means of a time-history analysis, using a constitutive law of the devices which can adequately reproduce the behaviour of the system in the range of deformations and velocities anticipated in the seismic design situation.

7.8 SAFETY VERIFICATIONS AT ULTIMATE LIMIT STATE

The substructure shall be verified under the inertia forces directly applied to it and the forces and moments transmitted to it by the isolating system.

Safety verifications regarding equilibrium and resistance in the substructure and in the superstructure shall be performed without satisfying capacity design and global or local ductility conditions.

The structural elements of the substructure and the superstructure may be designed as non-dissipative. For concrete, steel or steel-concrete composite buildings, low ductility classes details may be adopted.

The resistance condition of the structural elements of the superstructure may be satisfied considering seismic action effects divided by a behaviour factor of 1.5.

Considering possible buckling failure of the devices, the resistance of the isolating system shall be evaluated taking into account the γ_x factor defined in 7.2.

According to the type of device considered, the resistance of the isolating units should be evaluated at the No – collapse Limit State in terms of either of the following:

a) forces, taking into account the maximum possible vertical and horizontal forces in the seismic design situation, including overturning effects;

b) total horizontal displacement between lower and upper faces of the unit. The total horizontal displacement should include the distortion due to the design seismic action and the effects of shrinkage, creep, temperature and post tensioning, if present.

VIII. SIMPLE BUILDINGS

8.1 SCOPE

This section applies to buildings in importance classes III and IV according to 4.1, that may be designed applying simplified rules because of their dimensions, simplicity and regularity characteristics.

A building can be defined "simple" if it meets all the regularity criteria defined in 4.3.3 and 4.3.4 and in addition it meets all the following criteria:

a) The resisting system of the building is approximately symmetric in plan in two orthogonal directions.

b) All vertical elements of the resisting system are continuous from the foundation to the roof.

c) All storey heights do not exceed 3.5 m and the area of each floor does not exceed 300 m^2.

d) The resisting system is reasonably distributed in plan, with a significant portion of the vertical resisting elements positioned near the exterior parts of the buildings; specific rules shall be provided for each construction material and system.

e) The number of storeys does not exceed a maximum number that will be provided for each construction material as a function of seismicity, structural system and ratio of resisting structural area and floor area; the number of storey shall not in any case exceed 3.

8.2 DESIGN AND SAFETY VERIFICATIONS

Simple buildings can be designed without performing any specific analysis and safety verification, provided that all previous requirements are fulfilled, in addition to those specified for each construction material and structural system.

If a building in a low and very low seismicity zone does not exceeds the limits given in terms of storey height and number of storeys, it can be designed adding to the non – seismic load combinations two horizontal systems of forces, in two orthogonal directions, defined according to expression 5.13, assuming as the total base shear force a fraction of the total weight of the building. The appropriate fraction to be considered will be defined for each construction material and structural system, but in no case shall be lower than 0.05.

8.3 SPECIFIC RULES AND DETAILING

The general rules given in 4.3.1 and all the specific rules and detailing prescriptions given for each construction material and construction system shall be applied, particularly for what concerns horizontal diaphragms, adequacy of the foundation, building separation, geometry prescriptions, proper connections between different elements and element detailing related to specific construction materials.

IX. PROVISIONS FOR EXISTING BUILDINGS

9.1 GENERAL

The scope of this section is:

- To provide criteria for the assessment of the seismic performance of existing individual building structures;
- To describe the approach in selecting necessary corrective measures;
- To set forth criteria for the design of the repair/strengthening measures (i.e. conception, structural analysis including intervention measures, final dimensioning of structural parts and their connections to existing structural elements).

Since existing structures:

a) reflect the state of knowledge of the time of their construction;
b) possibly contain hidden gross errors;
c) may have been submitted to previous earthquakes with unknown effects;
d) structural evaluation and possible structural intervention may be subject to a much larger degree of uncertainty than the design of new structures.

9.2 INFORMATION FOR STRUCTURAL ASSESSMENT

9.2.1 General information and history

In assessing the earthquake resistance of existing structures, taking also into account the effects of actions in other design situations, the input data shall be collected from available public records, relevant information, field investigations and, in most cases, from in-situ and/or laboratory measurements and tests.

The data collection and tests shall be performed by qualified personnel.

Cross-examination of the results of each data-source shall be performed to minimise uncertainties.

Inspection, check-lists and other data-collection procedures, should follow the recommendations of pertinent professional organisations and should reflect the availability of local resources for inspection, investigation and repair/strengthening measures.

9.2.2 Required input data

In general, the information for structural evaluation should cover the following points, however, such comprehensive information could be very difficult to establish for traditional buildings, and may not be obtainable. In such cases, appropriate allowance must be made for the resulting uncertainty, as discussed in 9.2.3.

a) Identification of the structural system;

b) Date of construction and information on structural changes since construction, likely altering the structural behaviour;

c) Identification of the subsoil conditions as categorised in 2.5;

d) Identification of the type of building foundations;

e) Identification of the exposure class regarding environmental influence;

f) Information about the overall dimensions and cross-sectional properties of the building elements and the quality and condition of constituent materials;

g) Description of the present and/or the planned use of the building (with identification of its importance category, as described in 4.1);

h) Evaluation of the potential live loads by considering the actual use of the various spaces;

i) Information on the quality of existing materials, expressed in quantitative terms where possible;

j) Information about the type and extent of previous and present structural damages, if any, including earlier repair measures;

k) Information about identifiable gross errors in structural concept, about material defects and inadequate detailing.

9.2.3 Levels of knowledge, methods of analysis and partial safety factors

General

Assessment and redesign of existing structures may be based on appropriately modified load and resistance safety-factors (in comparison with the design of new structures) in order to account for the higher or lower uncertainty with respect to the knowledge of geometry, detailing, materials and loads. Similarly, the level of knowledge determinates the selection of the method of analysis to be adopted.

For these purposes, three levels of knowledge are defined, as follows and as described in the following table.

LC1: limited knowledge
LC2: adequate knowledge
LC3: accurate knowledge

In the table, the words *limited*, *extensive* and *accurate*, related to on – site control of details and on – site testing, have the following indicative meaning:

Limited on site control: details are verified for at least 10% of each type of primary structural elements
Extensive on site control: details are verified for at least 20% of each type of primary structural elements
Accurate on – site control: details are verified for at least 30% of each type of primary structural elements
Limited on site testing: at least one test per story for each structural material
Extensive on site testing: at least two tests per story for each structural material
Accurate on - site testing: at least three tests per story for each structural material.

Table 9.1 - Methods of analysis and modification of the partial safety factors as a function of the level of knowledge

Level of knowledge	**Geometry**	**Details**	**Materials**	**Recommended method of analysis**	**Load and resistance safety-factors**
LC1	From original drawings with limited visual inspection	From a design simulation according to the original design code plus limited on site control	Standard values according to the time of construction and limited on site testing	Linear, static or dynamic	Increased with respect to new design
LC2	From a complete and detailed on – site survey	From original construction drawings plus limited on site control, or as above with extensive on site control	From design specs with limited on site testing, or from extensive on site testing	Any	Same as for new design
LC3	From a complete and detailed on – site survey	From original construction drawings plus extensive on site control, or as for LC1 with accurate on site control	From original testing certificates, with limited on site testing, or from accurate on site testing	Any	Decreased with respect to new design

9.3 ASSESSMENT

9.3.1 General

Assessment consists of the evaluation of the seismic resistance of an existing damaged or undamaged building, taking into account both non-seismic and seismic actions, for the period of its intended lifetime.

Assessment is made for individual buildings, in order to decide about the need for structural intervention and about the strengthening or repair measures to be implemented. Depending on the importance of the building and on the extent of its possible damages, as well as on the available information, time and resources, the evaluation shall be made by means of one of the methods presented in 9.3.5

Whenever possible, the method used should incorporate information of the observed behaviour of the same type of building or similar buildings during previous earthquakes.

9.3.2 Seismic action and seismic load combination.

The basic models for the definition of the seismic input and load combinations are those presented in section 3 and 5.

9.3.3 Structural modeling

Based on the information collected as indicated in 9.2 a model of the structure shall be set up. The model shall be adequate for determining the action effects in all structural elements under the seismic load combination.

All provisions of 5.3 regarding torsional effects apply without modifications.

In accordance with 4.3.5, some of the existing structural members can be designated as “secondary”.

The strength and the stiffness of these members against lateral actions shall be neglected, but they shall be checked to maintain their integrity and capacity of supporting gravity loads when subjected to the design displacements, with due allowance for 2nd order effects. The choice of the members to be considered as secondary can be varied after the results of a preliminary analysis, but it no case the selection of these elements shall be such as to change the classification of the structure from non regular to regular, according to the definitions given in 4.4.2 – 4.3.4.

9.3.4 Methods of analysis

General

The seismic action effects, to be combined with the effects of the other permanent and variable loads according to the seismic combination in 5.1, may be evaluated using one of the following methods, as applicable:

- linear static analysis
- multi-modal response spectrum analysis
- non-linear static analysis
- non-linear time history dynamic analyses.

In all cases, the seismic action to be used is the one corresponding to the elastic (i.e., unreduced by the behaviour factor q) response spectrum, or its equivalent alternative representations given in 3.1 and 3.3, considering the appropriate importance factor γ_I.

Non-linear analyses shall be properly substantiated with respect to the definitions of the seismic input, to the structural model adopted, to the criteria for the interpretation of the results of the analysis, and to the requirements to be met.

The two horizontal components of the seismic action shall be combined according to 5.1.1.

The vertical component of the seismic action shall be considered in the cases contemplated in 5.1.2 and, when appropriate, combined with the horizontal components as indicated in the same clause.

The provisions related to additional measures for masonry infilled structures (6.5.3), combination coefficients for variable actions (5.1.3), importance categories and importance factors (4.1) apply, whenever relevant.

Linear static analysis

The conditions for this method to be applicable are given in 5.2.2, with the addition of the following one:

- the maximum value (over all primary elements of the structure) of the ratio between the action effect (bending moment, shear force, shear in the joints, etc.) obtained from the analysis under the seismic load combination, and the corresponding capacity, does not exceed the value of 2.

The method shall be applied as described in 5.2.2, except that the response spectrum in exp. (3.5) shall be the elastic spectrum $S_e(T)$ instead of the design spectrum $S_d(T)$.

Multi-modal response spectrum analysis

The conditions of applicability for this method are given in 5.2.3 with the addition of the following one:

- the maximum value (over all primary elements of the structure) of the ratio between the action effect (bending moment, shear force, shear in the joints, etc.) obtained from the analysis under the seismic load combination, and the corresponding capacity, does not exceed the value of 2.

The method shall be applied as described in 5.2.3, using the elastic response spectrum $S_e(T)$.

Nonlinear static analysis

Nonlinear static (pushover) analysis is a non-linear static analysis under constant gravity loads and monotonically increasing horizontal loads (see 5.2.4).

Buildings not complying with the criteria of 4.3.3 for regularity in plan shall be analysed using a spatial model.

For buildings complying with the regularity criteria of 4.3.3 the analysis may be performed using two planar models, one for each main direction.

Nonlinear time-history analysis

The time dependent response of the structure can be obtained through direct numerical integration of its differential equations of motion, as described in 5.2.4.

The mechanical model shall be able to describe the behaviour of the elements under post-elastic unloading and reloading cycles. The model should also reflect realistically the energy dissipation in the elements over the range of displacement amplitudes expected in the seismic design situation.

9.3.5 Safety verifications

<u>*Linear methods of analysis (static or dynamic)*</u>

A distinction shall be made between “ductile” and “brittle” components or mechanisms. The classification of components/mechanisms as “ductile” or “brittle” is given in relation to specific materials.

For “ductile” components the design value E_d of the action effect due to seismic action shall be the one obtained from the analysis of the structure under the elastic response spectrum, divided a local, component specific, behaviour factor q_c. The values of q_c. for the different components are given in relation to specific materials.

The resulting design value E_d of the action effect in the seismic design situation shall be checked to be lower than the corresponding design strength, this latter evaluated using the appropriate γ_m factors as discussed in 9.2.3.

For “brittle” components, the design value of the action effect shall be obtained by means of equilibrium conditions, considering the strength of the components delivering load to the component under consideration. The design value of the action effect shall be checked to be lower than the corresponding strength of the component, this latter evaluated using the appropriate resistance factors, as modified according to table 9.1.

<u>*Nonlinear methods of analysis (static or dynamic)*</u>

The seismic action effects on both “ductile” and “brittle” components shall be those obtained from the analysis performed according to 9.3.5.

“Ductile” components shall be checked to possess a deformation capacity not less than the maximum calculated deformations. The capacities shall be determined according to the rules or to the default values provided for specific materials.

“Brittle” components shall be checked to have strengths not less than the maximum calculated action effects. The strength of the components shall be evaluated using the appropriate resistance factors, as modified according to table 9.1.

9.4 Criteria for structural intervention

General criteria

For the definition of the intervention measures, the following shall be considered:

a) Costs, both initial and future (i.e. accounting for maintenance costs and possible future damage), versus the importance of the structure under consideration;

b) Available workmanship (it is of fundamental importance that the measures should be feasible with the available workmanship and equipment);

c) Availability of appropriate quality control;

d) Occupancy (impact on the use of the building, both during and after the works);

e) Aesthetics (the intervention policy may vary from a totally invisible solution to an intentionally identifiable new/additional structural scheme);

f) Preservation of the architectural identity of historical buildings and consideration of the level of reversibility of intervention;

g) Duration of the works.

Technical criteria

The selection of the type, technique, extent and urgency of the intervention shall be based on the structural information collected during the assessment of the building, considering the following guidelines:

a) All identified local gross errors should be appropriately remedied;

b) In case of highly irregular buildings (both in terms of stiffness and overstrength distributions), their structural regularity should be improved as much as possible.

c) If the low damageability requirement of 5.4 regarding the non-structural elements is not sufficiently fulfilled, appropriate intervention measures should be taken (e.g. stiffening, separation of vulnerable non-structural elements from load-bearing elements, etc). However, in case of disproportionately high costs or adverse structural implications, the criteria of 5.4, regarding limitations of damage, may be modified.

d) The minimum possible modification of the local stiffness should be sought, unless required otherwise by criteria a) and b) above.

e) Where possible, the increase of local ductility should be sought in critical areas. Care should be taken that, as far as possible, local repair and/or strengthening does not reduce the available ductility of critical areas.

f) The durability of both the original and new elements, as well as the possibility of accelerated deterioration when in contact with each other, should be taken into account.

Type of intervention

Observing the criteria given above, an intervention may be selected from the following indicative types; one or more types in combination may be selected. In most cases, the effect of structural modifications on the foundation has to be considered:

a) Restriction or change of use of the building;

b) Local or overall modification of damaged or undamaged elements (repair or strengthening), considering their stiffness, strength and/or ductility;

c) Possible transformation (or incorporation) of existing non-structural elements into structural elements (e.g. adding elastic and damping connectors between brittle infills and frame, when the resistance of these infills allows it);

d) Modification of the structural system (elimination of some structural joints; elimination of vulnerable elements; modification into more regular and/or more ductile arrangements). This is for instance the case when vulnerable low shear-ratio columns or entire soft storeys are transformed into more ductile arrangements; similarly, when overstrength irregularities in elevation, or in-plan eccentricities are reduced by modifying the structural system;

e) Modification of the structural system aimed at a beneficial change of the natural period of the structure, may also be envisaged;

f) Mass reduction;

g) Addition of new structural elements (e.g. bracings or infill walls; steel, timber or reinforced concrete belts in masonry construction; etc);

h) Full replacement of inadequate or heavily damaged elements;

i) Redistribution of action-effects (e.g. by means of re-levelling of supports or by adding external prestressing);

j) Addition of a new structural system to take the seismic action.

k) Addition of local friction, global damping devices or passive control at appropriate locations of the building;

l) Base isolation;

m) Partial or total demolition.

Ductility considerations

It has to be taken into account that an increase in resistance of structural elements may frequently be accompanied by a reduction in ductility, unless specific measures are taken.

Non structural elements

Decisions regarding repair or strengthening of non-structural elements shall also be taken whenever, in addition to functional requirements, the seismic behaviour of these elements may jeopardise the life of inhabitants or affect the value of goods stored in the building.

In such cases, full or partial collapse of these elements should be avoided by means of:

a) appropriate connections to structural elements;

b) increasing the resistance of non-structural elements;

c) taking measures of retainment to prevent possible falling out of parts of these elements.

The possible consequences of these provisions on the behaviour of structural elements should be taken into account.

Justification of the selected intervention type

In all cases, the redesign documents shall include the justification of the type of intervention selected and the description of its expected structural function and consequences.

This justification should be made available to the person or organisation responsible for the long-term maintenance of the structure.

9.5 REDESIGN OF REPAIR AND/OR STRENGTHENING

9.5.1 Redesign procedure

The redesign process shall include the following steps:

a) Conceptual design

(i) Selection of techniques and/or materials, as well as of the type and configuration of the intervention;
(ii) Preliminary estimation of dimensions of additional structural parts;
(iii) Preliminary estimation of the modified stiffness of the repaired/strengthened elements.

b) Analysis

The methods of analysis of the structure as redesigned shall be those indicated in 9.3.5, as appropriate considering the new characteristics of the building.

c) Verifications

Safety verifications shall be carried out in accordance with 9.3.6. Material safety factors shall in principle be selected in accordance with 9.2.3 and with what specified in the relevant material provisions.

Under well defined conditions, simplified redesign procedures may be followed for simple buildings to be repaired or strengthened against future seismic events. These procedures should be established in specific documents, where the definition of "simple" buildings is presented and the simplified rules are deemed to satisfy the requirements of this Standard.

9.5.2 Basic data for force transfer

The structural characteristics of repaired and/or strengthened building elements (in particular their critical regions) shall be conservatively estimated on the basis of the data originally collected for the assessment, taking into account the force transfer mechanisms along the interfaces between existing-to-existing or between existing-to-additional components, as described below.

In every case, the interaction (and corresponding possible resistance reduction) between these mechanisms shall be taken into account.

Compression against pre cracked interfaces

Reloading after tension cracking may cause compressive forces before full recovery of previous extensional deformations; inelastic distortions along the crack, and crack debris inside the crack prevent crack to close completely.
It is allowed to account for this phenomenon by means of an appropriate model. However, such a model shall conservatively account for cyclic reloading, which tends to minimise such an early compressive resistance of "open" cracks.

A conservative simplification consists in disregarding compressive stresses before full closure of previous extensional cracks.

Adhesion between non-metallic materials

"Shear adhesion versus slip" relationships between existing and new materials may be accounted for by means of appropriate models, taking into account the effects of curing and the characteristics of possible bonding agents.

Slip may be disregarded if shear adhesion stresses are below conservative estimates of the long term shear adhesion-resistance.

Friction between non-metallic materials

Friction resistance may be accounted for as a function of the relative displacement (slip) along a discontinuity or the interface. A constitutive law should be used for this purpose, valid for the relevant normal stress (σ) acting across the interface.

In case the slip needed to mobilise the maximum friction resistance (τ_u) is relatively low, a friction coefficient $\mu = \tau_u / \sigma$ may be used. However, for low σ values, the strong dependence of μ on σ values should be appropriately taken into account.

Whenever a cyclic reversal of the relative displacements is expected, the corresponding degradation of the friction-resistance shall be considered.

Load transfer through resin layers

The design tensile resistance of a joint (i.e. the contact interface) between a resin layer and a given material may be taken equal to the tensile resistance of the weakest of the two divided by γ_m (at least equal to [1,5]), or the tensile resistance of the interface (divided by [2,0]), whatever is lower.

The influence of the thickness of the resin layer, as well as the conditions of surface preparation shall be taken into account appropriately in evaluating the tensile resistance.

The local shear force response along such a joint is a function of the local slip and the normal stress acting on the considered area.

Clamping effect of steel across interfaces

If large relative displacements along the interface are not expected or cannot be tolerated, the friction resistance shall be evaluated considering the compatibility of displacements on both faces or the interface.

The friction mobilised across a sheared interface transversely reinforced with well anchored steel bars, may, in case of expected large relative displacements along the interface, be evaluated as follows:

$$\tau_R = \mu.(\rho\, f_{sy} + \sigma_0) < \tau_{u,m} \tag{9.1}$$

where
- μ = friction coefficient available under normal stress $\sigma_{tot} = \rho\, f_{sy} + \sigma_0$
- ρ = steel ratio across the interface
- f_{sy} = yield strength of steel
- σ_0 = external normal stress across the interface
- $\tau_{u,m}$ = the shear resistance of the material itself

In applying this model, appropriate design-values shall be used for μ, f_{sy} and $\tau_{u,m}$.

Dowel action

The design value of the maximum shear force which may be transferred by a bar crossing an interface (dowel action) shall be calculated appropriately, taking into account the strength and deformability of both the dowel and the connected material, and the spacing between dowels and their distance to the edges of the cross-section.

Anchoring of new reinforcement

Anchorage over the length of critical regions (i.e. potential plastic hinges) shall be avoided.

Anchorage lengths of steel bars in concrete or in masonry shall take into account the cyclic nature of seismic loading. Due to cyclic effects and related uncertainties, in high seismicity areas, it is not allowed to rely entirely on bond for the anchorage of new reinforcement. Appropriate mechanical means (such as end plates, dowels, etc.) are required to ensure an anchorage force resistance of F_{am} as:

$$F_{am} > \max\left(F_a - \frac{2}{3}F_b\ ,\ F_a/2\right)$$

where: F_a = required total anchorage force
F_b = anchorage force resisted by bond resistance

Anchorage of additional steel bars by welding onto existing bars (directly or via additional welded spacers) or on appropriately anchored fixing elements may be considered as rigid; weldability of existing and added steel elements shall be established. It shall be verified that the bond ensured by the existing bar is sufficient to resist the total force acting on both the existing and added bars.

The force-slip relationship for the pullout of embedded bars or fixing elements may be predicted by means of appropriate models or empirical formulae, accounting for the bar's diameter, the concrete cover, the maximum local bond strength, the embedment length and the yield strength of the steel.

If full anchorage of new reinforcement is not feasible, its strength should be appropriately reduced, and the consequences of this defect on the local ductility of the structural element should be taken into account.

Welding of steel elements

In designing steel-to-steel connections through welding, not only the weld resistance but also the stiffness of the connection should be considered:

a) Direct welding of additional longitudinal bars or shapes onto existing ones, ensures a complete force transfer with almost zero displacement;

b) In case the force transfer is achieved via intermediate elements, the stiffness of such elements may reduce the overall stiffness of the strengthened element(s) and should be considered in the overall force-transfer distributions;

c) Flexural behaviour of eccentrically welded structural steel elements or bars should be evaluated, accounting for contacts or restraining effects to existing or added elements. Simplifications should be based on adequate experience or experimental evidence;

d) Welding should be avoided on highly stressed steel bars or elements.

Connections of timber-to-timber elements

In evaluating the ultimate resistance of these connections (nailing, doweling, bolting, wedging) the force-transfer between existing and added timber elements shall be estimated conservatively, taking into account displacements (in the connections) due to the local deformability of wood. Durability conditions shall also be accounted for. Simplifications shall be based on experience or experimental evidence.

9.5.3 Local and global ductility

The repair/strengthening intervention should maximise the energy dissipation capacity of the structure, within the cost constraints of the project, considering the following aspects:

- Abrupt modifications of local strength and stiffness of building elements should be avoided, unless such a modification contributes to an improvement of the regularity of the strengthened/repaired structure;
- The overstrength and the interstorey drift of consecutive floors should be kept as constant as possible over the height of the building. To this end, the strengthening of a building element should extend beyond the level strictly needed for resistance purposes, thus avoiding creating a soft-storey effect;
- The areas of expected inelasticity should not be concentrated in only one storey; whenever possible, appropriate measures should be taken so that inelastic areas are well distributed over the entire structure.

9.5.4 Post intervention stiffnesses and resistances

General

The structural characteristics (resistance, deflections, crack widths) of repaired or strengthened building elements, used in verification of the Ultimate Limit State shall be evaluated taking into account load-transfer mechanisms, as described in 9.5.2, as well as the:

a) actual resistances of the existing materials;

b) additional resistances due to the connection of the new materials to the existing ones;

c) possible positive influence of the intervention on the residual structural characteristics of the building element in addition to the main role of intervention.

Appropriate factors for model – uncertainties shall be used (see 9.2.3).

Depending on the reliability of the available data and the importance of the structure, two approaches for the evaluation of resistances may be followed, namely:

a) analytical estimation of resistances, based on physical models;

b) simplified estimation of resistances, based on practical rules.

Analytical estimation of stiffness and resistance

Analytical models shall be based on constitutive laws describing the force/ deformation characteristics of all connected materials. For each force transfer mechanism, the value of the force mobilised is calculated for the global deformational behaviour of the critical region. The sum of all mobilised forces under compatible deformations may be accepted, under the following conditions:

a) Conservative constitutive laws are considered, accounting for the response degradation due to cyclic post-yield deformations. The softening branch of these laws shall be considered to an extent consistent with the overall ductility demand assumed in the analysis.

b) When appropriate structural measures have been taken (e.g. adequate confinement in the case of masonry or reinforced concrete elements) the constituent materials may be represented by their monotonic constitutive laws. However, this is not the case with connections between materials where, because of cyclic degradation, considerably conservative strength and deformational values need to be considered.

c) Possible interactions between individual force-transfer mechanisms (e.g. pullout bond and dowel action) are also to be considered.

Simplifications based on evidence from parametric studies or experiments may also be introduced in these calculations.

Simplified estimation of stiffness and resistance

For simple buildings (see section 8) and under limited conditions, it is allowed to use simplified behaviour models.

When estimating residual characteristics after damage, global "correction factors" r_K for stiffnesses and r_R for resistances may be applied, depending on the damage level assessed.

Thus, the residual resistance R_{res} of a damaged structural element may be estimated as

$$R_{res} = r_R.R_0 \tag{9.3}$$

where R_0 denotes its original resistance before damage.

Similarly, for residual stiffness K_{res} of a damaged element

$$K_{res} = r_K.K_0 \tag{9.4}$$

where K_0 denotes its original stiffness before damage.

The factors r_R and r_K shall not exceed 1.

When estimating added resistances due to additional material (repair or strengthening), partial correction factors may be applied accounting for the different degree of mobilisation of forces in each contributing transfer mechanism.

When estimating the resistance R and stiffness K of a repaired/strengthened region, the respective structural characteristics R_{mon}, K_{mon} of a supposedly "monolithic" region may be first calculated, disregarding existing discontinuities or interfaces. Subsequently, appropriate "model correction factors"

k_r for resistances
k_k for stiffnesses

may be used, in order to account (in an empirical way) for the effects of discontinuities or interfaces disregarded in the previous step. Since interfaces between existing and additional materials produce lower initial stiffness and resistance values than for a supposed "monolithic" region,[2]

$$k_k \leq 1 \text{ and } k_r \leq 1 \tag{9.5}$$

Thus, the final characteristics of the repaired/strengthened region may be estimated as:

$$R_{res} = k_r.R_{mon} \tag{9.6}$$

[2] Normally $k_k \leq k_r$

and $K_{res} = k_k.K_{mon}$ (9.7)

The values of aforementioned factors r, and k to be used in evaluation or in redesign should be conservatively chosen, taking into account available technical literature and local experience.

When carrying out these simplified estimates, account should be taken of the performance of similar structures in previous earthquakes.