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EARTHQUAKE RESISTANCE REGULATIONS IN LISBON PAST, PRESENT AND FUTURE

Summary

This paper presents the principal characteristics of the earthquake resistance regulations for buildings in Lisbon. Criteria for the evaluation of regulations are discussed and applied to those codes, needed developments are identified and some proposals are presented.

1 - PREAMBLE

This paper presents an evaluation of the earthquake resistance regulations for buildings in Lisbon. These are: i) the 1958 Code of Safety for Seismic Actions (past); and related codes (Annexes A and B); ii) the Code on Safety and Actions for Building and Bridge Structures & Code for Reinforced and Prestressed Concrete Structures (present - Annex C); iii) the Eurocode 8 - Structures in Seismic Regions - Design - Part 1 - General and Building - Draft, 1988 (future[1] - Annex D). As a basis for this evaluation, the past, present and future codes are briefly described[2] and related to the perception of the earthquake predicament facing the community responsible for their elaboration-enforcement-evolution. Next, the role of regulations in the mitigation of earthquake risks is discussed and criteria for their effectiveness are presented. Those criteria are used in the analysis of the Lisbon codes and deficiencies in present and future code are identified; recommendations to make good those deficiencies are proposed (Annex E).

2 - THE EVALUATION OF EARTHQUAKE RESISTANCE CODES

It may be considered that an earthquake resistant code should fulfil four purposes:

1) ensure that the constructions have a sufficient resistance to earthquakes;
2) safeguard the responsibility of the owner-designer-builder of the construction;
3) make available and useful existing knowledge;
4) promote technical progress.

Sufficient resistance to earthquakes includes both survival capacity in the case of very strong earthquakes and damage control in the case of more frequent less strong earthquakes.

The safeguard of the responsibilities is particularly important because in the case of seismic design the uncertainties are much less controllable than in the case of design for other actions. Hence, society should bear itself the risks which are unavoidable in earthquake zones. Once a given level of reliability is attained, earthquake engineering starts to be ruled by economic criteria; to have the structure that corresponds to minimum (generalised) cost, the best knowledge on structural materials and structural behaviour is oftentimes needed. Hence, codes should reflect existing knowledge.

Codes may have also an important role in the promotion of technical progress; on the other hand they may not only hinder technical progress but also steer it on wrong directions.

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[1]It is certain that this draft of the Eurocode 8 will suffer substantial transformations before its publication as a provisory european norm.

[2]Due to the very different extensions between the Portuguese codes and the 1988 Eurocode, the description of the Portuguese codes is reasonably complete while the description of the Eurocode is clearly not exhaustive.

Due to the complexity of the subject, it is almost impossible for codes to fulfil all the four purposes. This paper submits that the 1985 code and the 1988 Eurocode 8 because they tried to attain purpose 3 failed in respect to purpose 1 for a situation with the Lisbon seismicity and with the building types commonly found in Lisbon, i.e. reinforced concrete structures with a very limited amount of walls and heavily infilled with masonry shear-walls. While this indeed implies a negative evaluation of the 1985 code, it only implies that the 1988 Eurocode 8 is unsuited for the Lisbon area, not for the rest of Europe. An example of a code that would economically fulfil purpose 1) is presented as Annex E.

3 - CRITIQUE OF PRESENT AND FUTURE CODES

The 1983 Code and the 1988 Eurocode 8 are based in a code format which was a direct development of classical structural safety theory (CSST). This theory has the particular advantage of being based on very elementary result of probability theory and has not evolved to incorporate more powerful probabilistic and statistic methods as can be readily appreciated by the study of the so-called level 2 methods. The essential assumptions of CSST is that actions and structural behaviours are described by a small number of independent variables[3] and that structural behaviour is linear or linearizable. Those assumption are not appropriate for seismic design because of the markedly non-linear nature of the structural behaviour expected in a strong earthquake event.

In earthquake engineering, problems have always a large (potentially infinite) number of dimensions because actions are represented by time-histories of vibration and the resistance also must be formulated in terms of time-histories of action effects. Although several approaches for simplifications were moderately successful it have not been possible so far to identify a small set of earthquake intensity measures (peak ground acceleration, velocity and displacement, duration, Arias intensity...) and damage indexes (maximum ductility, dissipated energy, strength degradation...) that would give reliable results for a significant number of cases of practical interest. In consequence, the effective problems of earthquake engineering are extrapolation problems (the number of data points for interpolations is at least equal to the total number of variables plus one) and thus much more difficult than the interpolation problems usual in common engineering.

In linear problems, the distinction of actions from action effects and the separation of design methods from safety verifications is essentially arbitrary because it can be formulated in,a rigorous way. In non-linear problems, the overall accuracy is very dependent on the division of the global problems into separate elementary problems. In earthquake engineering, in particular, the clear distinction between design methods and safety verifications is very important, at least for a rational organisation of the different subjects. The 1985 code only considers safety verifications and ignore design completely while the 1988 Eurocode 8 in a significant number of clauses confuses design and safety verifications.

CSST lacks logical precision because of non-dichotomy division of actions and of structural materials. One consequence of this is that the seismic action is classified as an accidental action in the 1988 Eurocode 8 while it should be classified as a variable action if a more correct procedure was followed. Another consequence is that structures are classified by structural materials (e.g. reinforced concrete, steel, masonry...) when in general they are almost always composed of different materials and are thus of a mixed type. In particular, and considering the Lisbon type of seismicity, structural systems based on the complementary capacities of elements of masonry (strength) and of reinforced concrete (ductility) seems specially appropriate. This type of association is almost completely ignored in the 1985 Code and are defficiently taken into account in the 1988 Eurocode 8 (See section D.6). The blindness towards the fundamental importance in both design and analysis of the interaction between reinforced concrete structure and masonry infill panels makes irrelevant for the Lisbon situation much of the on-going research on the seismic behaviour of reinforced concrete frames.

However, the most consequential failings of the 1985 code and of the 1988 Eurocode 8 is that they do not provide guidance for the designer on the selection of the type of structure given their very different expected performance for the whole range of intensity of the possible earthquake events and their possibly important interaction with nearby structures. This last aspect is critical in southern Europe towns were the rule is that buildings are built contiguously and the result is that, at least, new reinforced concrete buildings endanger old masonry buildings.

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[3]n most case in CSST literature probabilistic variables are generally set-up as independent or in a form that may be easily transformed into a set of independent variables.

4 - A PROPOSAL FOR ACTION

Recent earthquakes have shown that a sizeable portion of engineered structures collapsed when acted by their first strong earthquake which, by definition, should be considered to have a significantly smaller intensity than the design earthquake. This shows that present engineering practices should be urgently corrected. Given the large number of uncertainties in future earthquake actions and on the behaviour of complex structures, a sensible solution would be:

- Design of common structures by simplified criteria in a code format simillar to the model code in Annex E, i.e. relying on a sufficient number of shear-walls to provide adequate stiffness and strength;
- Design of structures with functional requirements incompatible with the required distribution of shear-walls by advanced methods of non-linear dynamics. Most of the material presently existing in the 1988 Eurocode 8 would be highly suitable to this type of design-analysis.

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ANNEX A - THE 1958 CODE

A.1 - Introduction

The first modern Portuguese earthquake resistance regulation, the "Regulamento de Segurança das Construções contra os Sismos" (Code on Safety of Constructions for Earthquakes) was issued in 1958 (Decreto n° 41 658 of May 31st), following the realisation of the Symposium on the Effects of Earthquakes and its Consideration in the Design of Constructions, which was held to mark the two hundredth anniversary of the 1755 Lisbon earthquake. This Code was accompanied by a large justificative paper explaining the options selected for the different subjects. The explicit purpose of the 1958 Code was to evitate the collapse of constructions as a result of earthquakes, thus trying to ensure the safety of persons and properties.

Some prescriptions of this code were integrated in 1961 in the "Regulamento de Solicitações em Edifícios e Pontes" (Code of Actions in Buildings and Bridges) and in 1967 in the "Regulamento das Estruturas de Betão Armado" (Code on Reinforced Concrete Structures).

Three seismic zones were considered, designed by A, B and C, corresponding to high, medium and low seismic risk. The definition of those zones for mainland Portugal are presented in figure 1. All the Azores islands belong to zone A with the exception of the Flores and Corvo Islands which, together with the Madeira Islands, belong to zone C. Construction in zones A and B shall obey the prescriptions of the Code, but constructions in zone C didn't have to.

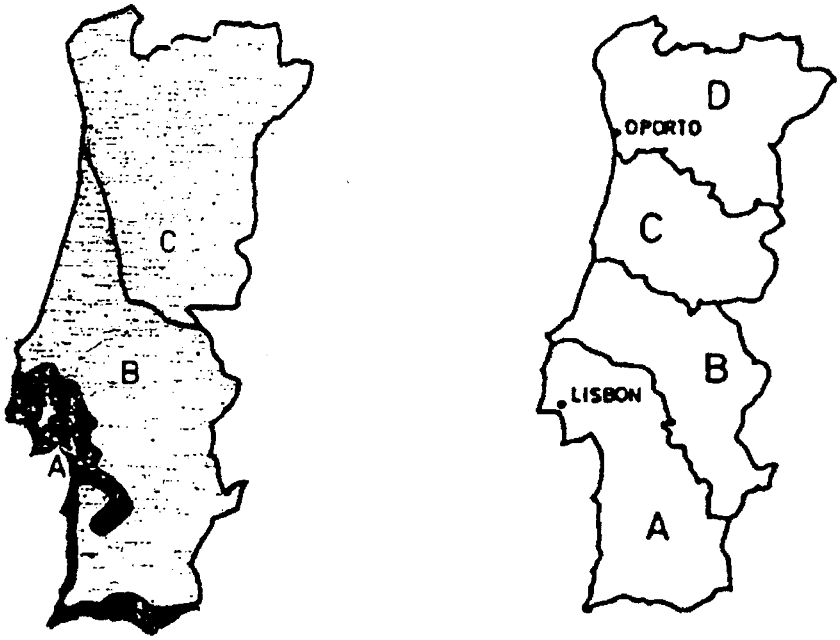

Fig. 1 - Seismic zoning for mainland Portugal (left: 1958 Code; right: 1985 Code).

A.3 - Seismic actions

The earthquake actions are represented, for design, by static horizontal forces, which can act in every direction. The horizontal force corresponding to each element of the construction shall have an intensity equal to the product of the weight of that element by the seismic coefficient *c* and shall act in its centre of mass. The minimum values of the seismic coefficient for the design of the construction and of its elements are those in table I. Although not obligatory, higher values of the seismic coefficient should be used whenever specially unfavourable situations of foundation soils, constructions types or utilisation patterns are encountered. For the evaluation of the horizontal forces the weight of all the parts of the construction (structure, infills and finishes) shall be considered, as well as other permanent loads such fixed equipment. Materials in reservoirs, silos and depots shall be considered at full level and as permanent loads.

Table I - Seismic coefficient c

	Seismic zone	A	B
Construction (global)	Common constructions (buildings)[1]	0.10	0.05
	Towerlike constructions	0.20	0.10
Construction elements	Walls and other elements	0.20	0.10
	Balconies, chimneys	0.30	0.15

Notes: 1 - industrial chimneys and elevated reservoirs

The earthquake equivalent horizontal forces are to be used in the design of buildings, bridges, towers, chimneys and similar structures. For other structures such as dams, retaining walls, embankments and similar structures, the designer shall use seismic forces and design methods appropriate for the specific kind of structure. The seismic action shall be considered to act at the same time as the permanent loads, but independent of snow, wind and temperature gradients.

A.4 - Design

All constructions and their elements shall be designed to withstand the seismic action and the permanent loads. However, and although the application of the Code is recommended, the following types of buildings do not have to be designed:

a. Small buildings with only one storey in zone A and two storeys in zone B;
b. Masonry buildings with a maximum of three storeys in zone A and four storeys in zone B if some reinforced concrete elements are present.

ANNEX B - THE 1961 CODE

B.1 - Safety checking rules

This code introduced the concept of specific rules for checking safety. Loadings were divided into permanent and accidental. Accidental loadings were subdivided into habitual and exceptional. Structures should be checked for two types of loading combinations:

Type I - Permanent loadings plus habitual accidental loadings;
Type II - Permanent loadings plus exceptional accidental loadings.

Seismic actions and strong winds were classified as exceptional accidental loadings.

B.2 - Seismic zoning

The seismic zoning was identical to the one in the 1958 code (Figure 1); similarly, seismic actions need not be considered for zone C.

B.3 - Seismic actions

The seismic actions are considered as static horizontal forces acting in any direction. Those forces, which should correspond to the mass elements of the construction and applied at their gravity centres, shall have an intensity equal to the product of the weight of each element by a seismic coefficient. The minimum values of the seismic coefficient, either for the checking of the construction or of its elements, are indicated in the Table 2, considering the types of construction, the seismic zone and the type of the soil.

Table II - Values of seismic coefficient

		Zone A		Zone B	
		(1)	(2)	(1)	(2)
Construction	C1	0.10	0.15	0.05	0.075
(global)	C2	0.15	0.20	0.075	0.10
Elements of the	E1	0.20		0.10	
construction	E2	0.30		0.15	

The values indicated in the columns (1) should be used in the cases of usual foundation soils. The values indicated in the columns (2) should be used wherever the foundation soils present unfavourable characteristics in seismic situations, which namely is the case for alluviums, soft clays and loams, if the thickness of those layers is greater than about ten meters and even if the foundations cross those layers to seek support on more resistant soils. The values indicated in line C1 are for buildings with an additional resistance to horizontal forces due to non-structural elements, such as infill walls, at every storey and appropriately distributed and connected to the structural elements; values indicated in line C2 are for constructions without such additional resistance. The values indicated in line E1 should be applied to the design of walls and similar elements; the values indicated in line E2 should be applied to balconies, chimneys and other elements protruding from the exterior walls or from the roof.

Higher values for the seismic coefficient are recommended for cases in which the earthquake damage should be minimised, namely buildings needed for rescue operations.

Evaluation of resistances is performed using the same criteria as for static loadings.

ANNEX C - THE 1985 CODE

C1 - Introduction

The rapid development of earthquake engineering and of structural safety theory have made imperative the substitution of the 1958 portuguese seismic code. The substitution was also called for by the much greater diversity of bridges and building structures which were being erected, as compared to 1958, as well as by the widespread use of non-traditional construction techniques. According to the trends in design code formats, there is not a separate seismic code, but earthquake problems are dealt with in the several codes that constitute the new portuguese structural regulations. Those codes are, up to now, the "Regulamento de Segurança e Acções para Estruturas de Edifícios e Pontes-RSA" (Code on the Safety and Actions for Building and Bridge Structures), issued in 1983 (Decreto-Lei N° 235/83 of May 31st), the "Regulamento de Estruturas de Betão Armado e Pré-esforçado - REBAP" (Code for Reinforced and Pre-stressed Concrete Structures), also issued in 1983 (Decreto-Lei N° 349-c/83, of July 30th) and the "Regulamento de Estruturas de Aço para Edifícios - REAE" (Code for Steel Structures for Buildings) to be issued this year (1986). The RSA sets the rules that shall be used in checking safety and also the intensity of the actions that shall be considered. The REBAP and REAE define the resistance characteristics of structural concrete structures and of steel structures, respectively. The application of these codes begin in 1985, two years after their publication.

C.2 - Safety checking rules

Checking of the structure's safety is carried out by considering certain limit states, and comparing those limit states with the states reached by the structures under the relevant actions, quantified and combined according to specific "combination" rules. When earthquake actions are considered, only ultimate limit states need to be checked.

The "earthquake" design values of the acting internal forces are obtained through the combination rule:

$$S_d = \sum_{i=1}^{m} S_{Gik} + \gamma_q S_{Ek} + \sum_{j=2}^{m} \psi_{2j} S_{Qjk}$$

where S_{Gik}, S_{Ek}, and S_{Qjk} are the internal forces due to a permanent action, to the earthquake action (considered to be the basic action in the combination) and to a variable action other than the earthquake action, respectively: γ_q is the partial safety factor for the variable actions (γ_q = 1.5) and ψ_{n2j} is the quasi-permanent ψ coefficient. The values of the earthquake action and of the variable action to be used in expression 2 are the characteristic values (0.95 brittle) of the probabilistic distribution of the action maximum in the reference period

(50 years); the values of the permanent actions to be used, in principle, are the quasi-permanent values, but the characteristic values of the distribution of the maximum in the reference period may be used because the variability of permanent actions is small.

C.3 - Definition of the seismic actions

C.3.1 - Seismic zoning

Four seismic zones are considered, A, B, C and D, ordered by decreasing earthquake severity which is quantified by a seismicity coefficient α that takes the values 1, 0.7, 0.5 and 0.3, respectively. The characteristic values of the seismic action for zone A, defined through the peak values of acceleration and velocity, are taken as 150 cm/s^2 and 16 cm/s. The characteristics values for the other zones are obtained multiplying the peak values for zone A by the seismicity coefficient. The seismic zones in mainland Portugal are presented in figure 1. The boundaries between zones follow the boundaries between "concelhos" (counties) in order to avoid indefiniteness. All the Azores islands belong to zone A with the exception of the Flores and Corvo islands which, together with Madeira islands, belong to zone D.

C.3.2 - Local soil conditions

Site geology is taken into account through the definition of three types of soil conditions:

- Type I: rock and stiff cohesive soils;
- Type II: intermediate soils;
- Type III: soft cohesive soils and loose cohesiveness soils.

Code users are warned that in some conditions, for instance where there are large horizontal soil layers over bedrock, special precautions should be taken and the code definition of the seismic action does not apply.

C.3.3 - Earthquake components

The 1985 code indicates that, as a rule, it is only necessary to take into account the horizontal components of the seismic action and that the consideration of the vertical component is necessary only for structures particularly sensitive to vertical vibrations. Structures with natural modes of vibration with significant displacements in the vertical direction and natural frequencies lower than 10Hz are pointed out as an example of such structures.

C.3.4 - Quantification of the seismic action

The code states that although structures shall withstand earthquake motions with different frequency content and duration for a given level of intensity, as could arise from the probable combinations of magnitude and focal distance, it is only necessary to check safety against two "earthquake loading" the seismic action type 1 and the seismic action type 2, which provide for the characteristics of small magnitude earthquakes at short focal distances (type 1) and of larger magnitude earthquake at longer focal distances (type 2).

The general definition of the seismic action is that this action is an ensemble of ground vibration time histories. The ensemble elements are spatially variable motions originating from P, S, Rayleigh and Love waves. At each point the motion is a sample of a stochastic vector gossip stationary process. Sample duration is 10s for the seismic action type 1 and 30s for the seismic action type 2. Considering a referential with co-ordinate orthogonal axis X_1, X_2 and Y_3, with X_3, in the vertical direction, the stochastic process, which underlies the ensemble of ground vibration time histories that constitute the seismic action, has to obey appropriate mathematical conditions.

C.3.5 - Simplified definition of the seismic action

In most cases there would not be enough information to work out completely an earthquake vibration model according to the general definition of the seismic action, and the sensitivity of the structure to the subtleties of the base motion contained in the general definition (with the exception of underground structures) would also in general not require such a model. Thus the following simplifications may be introduced:

1. At each point the cross-spectral densities between motions along orthogonal axis are zero;
2. The spatial variability of the earthquake action may be quantified from the spatial auto-correlation function and it is admissible to take this function as zero for distances larger than 2 to 6 wavelengths, for each frequency band to be considered;
3. When the maximum distance between any two supports of the structure is less than 100 m it is admissible to consider a rigid base and to substitute the spatial variability of the earthquake action by adequate rotational motions.

These hypotheses are sufficient, together with the value of the apparent shear wave velocity, to derive the spectral densities matrix for the six components of the motion at a point.

C.3.6 - Response spectra definition of the seismic action

For structures with well separated natural frequencies the seismic action may be modelled by response spectra. The response spectra for horizontal motion are presented in figures 2, 3 and 4 for the different soil conditions and for seismic zone A. For other seismic zones the spectra should be multiplied by the seismicity coefficient α. The response spectra for the vertical component should be obtained multiplying by 2/3 the horizontal component spectra. Spectra for the rotational components may be obtained from the translation spectra with suitable assumptions.

Natural frequencies are considered well separated if the ratio between any two natural frequencies, whose natural modes gave a significant contribution to response, is comprised between 0.67 and 1.5. The response of the structure shall be computed by the square root of the sum of the squares method. When a more involved and proven method is used to combine the modal responses, the analysis is considered to be a simplified stochastic analysis and the response spectra definition of the seismic action may be used despite the closeness between natural frequencies.

The code states that the response spectrum model is only to be used for rigid base structures unless a response spectra model of the spatial variability of the motion is considered.

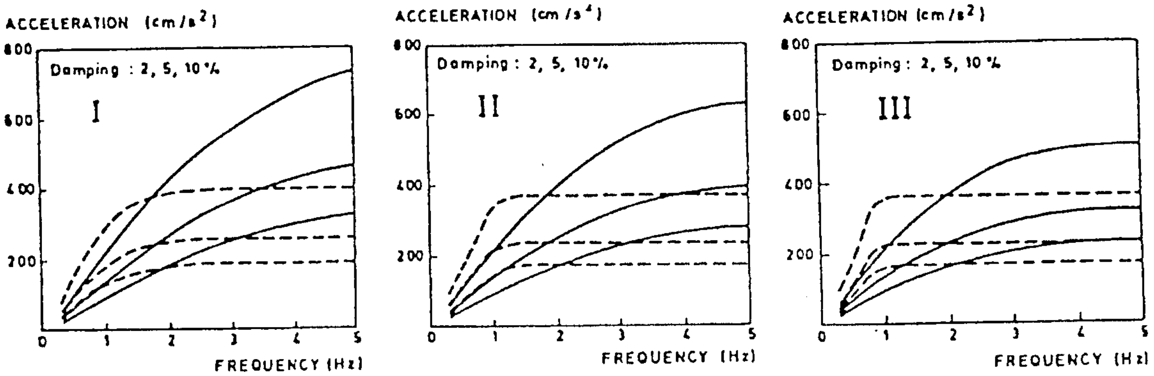

Fig. 2 - Response spectra for seismic zone A, seismic action type 1(---) and type 2 (---) and soil conditions type I, II and III.

C.4 - Determination of the earthquake action effects

C.4.1 - General

The determination of the earthquake action effects, that is, the values of S_{Ek} to be used in the earthquake combination rule (expression 2), shall be performed by an "exact" method of dynamic analysis or by simplified methods: a linear dynamic analysis, a lateral force coefficient method and a "last recourse" method. The first two simplified methods are considered to be "deducible" from the "exact" method. A bound, however, is imposed on the minimum value of the lateral resistance of the structures, whichever the method of analysis. This bound is expressed in terms of the maximum reaction force, $R(\theta)$, of the structure in direction θ. Let W be the weight of the structure and R the minimum value of $R(\theta)$. Then $\delta = R/W$ shall not be smaller than 0.04 α (α is the seismicity coefficient). Should δ be greater than 0.04 α, all the internal forces obtained in the analysis will have to be multiplied by 0.04 α/δ. There is also a facultative bound on the required maximum lateral resistance: if the coefficient δ is larger than 0.16 α and the structure has some ductility, all the internal forces may be divided by δ/0.16 α.

Soil - structure interaction should be taken into account, as well as hydrodynamic effects for totally or partially submerged structures. P - Δ effects can be disregarded if the maximum relative horizontal displacement between the base and the top of every vertical structural elements smaller than 0.015 times its height.

C.4.2 - The "exact" method of analysis

In the "exact" method of analysis, which constitutes the reference method, the structure is idealised considering the mean value for the "stiffness" and the "resistance", and its inertial properties shall correspond to the mean values of the permanent loads and to the quasi-permanent values of its inertial loads. Analysis is carried out through step-by-step integration for a sample of ground motion, large enough to be considered as representative of the ensemble of ground motions that constitutes the seismic action. The earthquake effect is taken at the average of the maximum value of the response to each motion in the sample. To keep computations at a reasonable minimum, some other statistic may be selected in place of the sample average and the sample size reduced accordingly.

C.4.3 - Linear dynamic analysis

The code states that it is possible to correct the results of a linear dynamic analysis by means of "behaviour coefficients" which depend on the type of structural system and its ductility characteristics, and also on the values of some parameters used on the linear analysis, namely the modal damping values. The number of behaviour coefficients that have to be used depends on the complexity of the non-linear dynamic behaviour characteristic of the class of structures under consideration. When this behaviour is particularly simple (and/or safety margins will not cause an excessive penalization) only two behaviour coefficients will be necessary, one for displacements and deformations, and one for internal forces. This last coefficient represents, in a sense, the ratio between the internal forces computed by linear analysis and the internal forces computed by non-linear analysis, for the highest severity of the earthquake action that causes a not yet unacceptable behaviour of the structure. Values for the behaviour coefficients are listed in the code for each structural material and are presented below.

For structures which have their elements disposed along two orthogonal directions, it is admissible to consider the seismic action acting separately along each direction, if a complementary analysis to provide for torsion effects is also performed.

C.4.4 - Lateral forces analysis

For buildings that present some regularity features, the dynamic analysis may be substituted by a lateral forces analysis in two orthogonal directions. Those building and bridges will be designated as "common", although this does not imply that they are the type most frequent to be found. Common buildings shall comply with the following conditions:

1. Horizontal distribution of mass and stiffness should not be dissimilar. As a practical rule, if it is possible to define a centre of stiffness for each story, the distances between this centre and the centre of mass, measured along the two orthogonal directions, shall not be larger than 15% of the building plan dimensions measured in the same directions;
2. Vertical distributions of mass and stiffness should not present sharp variations. This condition prescribes for instance significant variations of the "stiffness" due to "non-structural" materials like infill masonry panels;
3. The structure shall have its elements disposed along two orthogonal directions and shall not be excessively flexible. In practice this last condition is fulfilled if the fundamental natural frequency is higher than 0.5 Hz and 8 Hz divided by the number of storeys. This and condition b) is a guarantee that maximum relative horizontal displacements will be smaller than the 0.015 distortion limit; hence P - Δ effects can be disregarded;
4. Story structure should be sufficiently stiff for them to be considered as non deformed diaphragms.

The intensity of the static lateral forces is obtained from the seismic coefficient β, which is computed from:

$$\beta = \alpha\beta_0 / \eta$$

where α is the seismicity coefficient, β_0 the reference seismic coefficient and η is the behaviour coefficient. The bound on minimum lateral resistance is expressed in this level of analysis through the prescription that the minimum value to be used for the seismic coefficient β is 0.04 α. On the other hand, if the structure has some ductility, it is not obligatory to use values of β greater than 0.16 α.

The reference seismic coefficient β_0 depends on the local soil conditions and on the fundamental natural frequency of the structure for the direction under analysis. The values of β_0 correspond, within some approximations, to the values of the largest of the 5% response spectra applicable. The fundamental natural frequency, f, shall be obtained by appropriate analytical or experimental methods; the influence of "non-structural" elements, such as masonry infill panels, shall be considered. The use of "Rayleigh's method" for estimating the value of f is suggested, as well as some partially empirical formulae, which take into account the structural system (for the direction under analysis) and the height of the building (h) or the number of stories (n). Those formulae are (f in Hz):

$$f = 12/n \quad f = 16/n \quad f = 6b/h \tag{3}$$

respectively for frame structures, composite structures and shear-wall structures (b is the depth of the shear wall section).

The behaviour coefficients for common structures are prescribed in the codes defining the resistance (REBAP and REAE) and should be considered as upper bounds of the values to be used for structures that are not "common", which have to be properly justified. The values of the behaviour coefficients suppose that there are the usual amount of "non-structural" infill panels. Different behaviour coefficients shall be used for the effects due to the horizontal and vertical components. The values of the behaviour coefficients for internal forces due to vertical seismic actions are 1.0 (RC structures) and 0.8 (steel structures); the values of the behaviour coefficients for horizontal seismic actions are presented in table 3; in this table, a composite steel structure is one with booth braced and unbraced frames (in the same direction) and bridges are classified according to the energy dissipation mechanism (flexure or shear) or its absence (deck rigidly connected to the abutments) when dissipation occur through soil-structure interaction; improved ductility, RC structures have special reinforcement detailing to increase their energy dissipation capacity. For buildings and bridges which must remain operational after a strong earthquake the values of the internal forces behaviour coefficients to be used shall be 30% lower than those on table 3, but they need not be smaller than unity.

The lateral forces are distributed in the structure taking into account the distribution of mass and the deformed of the structure. For buildings the lateral forces are applied at the story levels and its intensity which follows an "inverted" triangular distribution is given by

$$F_i = \beta h_i C_i (\sum_i G_i) / (\sum_i h_i G_i) \tag{4}$$

where F_i is the force for story i, β is the seismic coefficient and h_i and G_i are the i-th story height and the weight of the masses concentrated at the same story. The lateral forces shall be applied with eccentricities e_{1i} and e_{2i}, relative to the story centre of mass, as will be most unfavourable. The eccêntricities values are obtained by

$$e_{1i} = 0{,}5b_i + 0{,}05a \qquad e_{2i} = 0{,}05a \tag{5}$$

where b_i is the eccentricity of the story stiffness centre and a is the building plan dimension measured perpendicularly to the direction of the lateral forces; eccentricity e_{2i} is to be measured towards the stiffness centre and e_{1i} in the opposed direction.

In the case of bridges, the mass shall be supposed concentrated at appropriate mass points. Then the lateral force F_i to be applied at point i is given by

$$F_i = (2\pi f)^2 \beta G_i d_i / g \tag{6}$$

where f is the natural frequency of the structure for the direction under analysis, β is the seismic coefficient, G_i, is the weight of the masses concentrated at point i, d_i is the displacement at point i supposing the weights G_i acting in the direction under analysis and g is the gravity acceleration. The distribution of the lateral forces follows roughly the distribution of the first natural mode accelerations.

Table 3 - Values of the behaviour coefficients for horizontal seismic actions

Type of structure		Internal forces ductility class normal	Internal forces ductility class improv.	displacements
steel buildings	unbraced frames	2,5	-	0,7
	composite	2,0	-	0,7
	braced frames	1,5	-	0,7
reinforced concrete buildings	frame	2,5	3,5	1,0
	composite	2,0	2,5	1,0
	shear-wall	1,5	2,0	1,0
pre-stressed and R.C. bridges	benging	2,0	3,0	1,0
	shear	1,4	1,7	1,0
	soil-struct. interaction	1,2	-	1,0

ANNEX D - THE 1988 EUROCODE

D.1 - Objectives of the Eurocodes

The structural Eurocodes comprise a group of standards for the structural and geotechnical design of buildings and civil engineering works. They are intended to serve as reference documents for the following purposes: (a) As a means to prove compliance of building and civil engineering works with the essential requirements of the Construction Products Directive; (b) As a framework for drawing up harmonised technical specifications for construction products.

The Eurocode 8 (EC8) provides the basis for the design and construction of structures in seismic regions and provides operational rules for application. Its purpose is to ensure, with adequate reliability, that:

The entire structure is planned, designed and constructed such it can withstand without local or general collapse the design seismic action, thus retaining its integrity and a residual load capacity after this seismic action has ceased;

The structure as a whole, including structural and non-structural elements shall be planned designed and constructed such that it is protected against the occurrence of damages and limitations of use as a consequence of an earthquake having a larger probability of occurrence than the design earthquake.

D.2 - Definition of the seismic action

The effects of local soil characteristics are accounted by the modification of the shape of the response spectrum. Three different soil profiles are considered.

The capability of a structural system to record seismic actions in the non-linear range is accounted for by the possibility designing it for forces smaller than those inherent to a linear elastic response. For design purposes and in order to avoid the need for a non-linear analysis, the concept of behaviour factor q is introduced. This parameter, which takes into account the energy dissipation capacity through ductile behaviour, is used to correct the results obtained from a linear analysis in order to get an estimate of the non-linear response. Accordingly, the determination of the internal forces for strength verifications may be based on the design spectra presented below.

The values of the behaviour factor q are given, for the various materials and structural systems and according to various ductility levels.

The design spectra to be used in the analysis of structures are defined by the following expressions:

$$0 < T < T_1 \quad \beta(T) = \alpha.S\left[1+\frac{T}{T_1}\left(\frac{\eta\beta_0}{q}-1\right)\right] \tag{7}$$

$$T_1 < T < T_2 \quad \beta(T) = \frac{\alpha.\eta.S.\beta_0}{q} \tag{8}$$

$$T > T_2 \quad \beta(T) = \frac{\alpha.\eta.S.\beta_0}{q}\left[\frac{T_2^k}{T}\right] \tag{9}$$

$$\beta(T) > \ 0{,}20.\alpha \tag{10}$$

where:

α is the ratio of the peak ground acceleration to the acceleration of gravity
η is a corrective factor for structures with damping different from $\xi = 5\%$.
q is the behaviour factor.

D.3 - Determination of the action effects

The determination of the seismic effects on the structure shall be based on a idealised mathematical model which is adequate for representing the actual structure. In general the model shall also account for a possible non-planar motion of the structure, and for all non-structural elements that can influence the response of the main resisting system.

If the structure can vibrate in two orthogonal directions without significant coupling between translation and torsion vibrations, it may by analysed by means of two separate planar models, one for each orthogonal direction.

The following types of analysis are considered in the EC8 although only the first two types of analysis are specified:

1. response spectrum analysis
1.1 Multi-modal response spectrum analysis
1.2 Simplified dynamic analysis
2. Static analysis
3. Power spectrum analysis
4. Time domain dynamic analysis

D.4 - Combination of seismic action with other actions

When checking for the earthquake effects, the following combination with other actions shall be considered:

$$\pm \gamma_1 E + G + P + \sum_i \psi_{2i} Q_{ik} \tag{11}$$

where the symbols have the following meaning:

γ_i important factor dependent on the importance category.
E design seismic action.
G permanent loads evaluated at their characteristic values
P pre-stressing at its final value
Q_{ik} variable loads at their characteristic values.
ψ_{2i} combination factors affecting the variable loads.

D.5 - Torsion Effects

For the application of simplified methods of analysis and for the determination of the behaviour factors q, a distinction should be drawn between regular and non-regular buildings.

In general the structure may be considered to consist of a number of vertical resisting system connected by horizontal diaphragms rigid in their plane. For buildings complying with the regularity requirements the analysis can be made by using two planar models, one for each main direction. Buildings not complying with such regularity requirements shall be analysed by means of three-dimensional models, which, if applicable and justified, may maintain the rigid floor assumption. When this procedure is not applicable but the regularity requirements are satisfied, the following approximate evaluation of torsion effects, still using a planar structural model for the determination of seismic forces, may be applied, provided the following conditions are satisfied:

a) The centres of stiffness of the individual storeys are approximately vertical above each other.

b) The centres of mass of the individual storeys which may not coincide with the centres of stiffness are also approximately vertical above each other.

If these conditions are satisfied, at each floor of the building, the application point of the horizontal seismic force is assumed to be displaced from its nominal location in relation to the mass centre, perpendicularly to the direction of the considered seismic action, by the most unfavourable of the two eccentricities:

$$\Delta e_{max} = e_1 + e_2 \quad \text{or} \quad \Delta e_{min} = -e_2 \tag{12}$$

where

e_1 is the additional eccentricity taking account of the dynamic effect of simultaneous translation and torsion vibrations

e_2 is the accidental eccentricity of storey masses from their nominal location

Denoting by B and L the building dimensions parallel and perpendicular to the considered seismic action, the following approximation may be applied to e_1 and e_2:

$$e_2 = 0{,}05L \tag{13}$$

$$e_1 = 0{,}1(L+B)\sqrt{10e_0 / L} \cong 0{,}1(L+B) \tag{14}$$

$$e_1 = \frac{1}{2.e_0} l_s^2 - e_0 - r^2 + \sqrt{(l_0^2 + e_0^2 - r^2)^2 + 4e_0^2.r^2} \tag{15}$$

$$l_s^2 = (L^2 + B^2)/12 \tag{16}$$

where

e_0 is the actual geometric eccentricity (distance between the stiffness centre and a straight line through the mass centre running parallel to the seismic action considered)

L is the total horizontal dimension of the building, perpendicular to the direction of the seismic action considered

B is the total horizontal dimension of the building, parallel to the direction of the seismic action considered

r^2 is the ratio between torsion and translation stiffness of the structure

e_1 is assigned the lower of the two values, (14) or (15).

D.6 - Case of infilled reinforced concrete frames

The consequences of possible in-plan-irregularities produced by the infills should be considered. To this end, a practical rule is given (increase of eccentricity for torsion effects).

The consequences of the possible in-elevation-irregularity produced by the infills should be considered; vertical discontinuities of over-strength (possible "soft storeys") may result in disproportionate local ductility demands. In order to account for such an event, a relevant decrease of the behaviour factor q is imposed.

The modified response of a R.C structure because of the stiffening effect of the infills shall be considered, taking however into account the random behaviour of the infills (namely the variability of their mechanical properties, the possible modifications of their integrity during the use of the building, as well as the non-uniform degree of their damage during the earthquake itself). The uncertainties of the structural model of an infilled R.C building should be adequately covered. Relevant design rules are given imposing one or two analytical models depending on the in-elevation-regularity of the building. The possibly adverse local effects because of the frame/infill interaction should be taken into account.

The damageability of the infills should be appropriately considered, both regarding the risk to persons and the possibility of repair after the design earthquake.

a) In case of low in-plan-irregularity due to non-uniform distribution of infill walls in-plan, an additional eccentricity $\Delta e = 0{,}05l$ will be taken into account for calculation of the additional torsion effects due to the infills ("l" denotes the length of the floor in the direction under consideration).

b) In case of high in-plan-irregularity, due to non-uniform distribution of infill walls in-plan, torsion effects due to infills should be taken into account by means of an appropriate analysis model.

c) In this context, the following degrees of in-plan-irregularity considered:
- Low irregularity: When infills are arranged in a practically symmetrical way in-plan.
- High irregularity: When infills are arranged in a substantially unsymmetrical way in plan, mainly along two consecutive faces of the building.

The vertical irregularity of a building taking also into account the actual infills, shall be quantified by an "over-strength index", which is calculated for each separate floor of the building and which affects the behaviour factor to be used.

The design shear forces shall be modified because of the decrease of natural period of reinforced concrete structures after the addition of infills. The calculation of the natural period of infilled structures may not be needed if the natural period of the bare structure is close to the value T_2 of the design spectrum. The natural period of the infilled structure may be estimated from the following formula:

$$T_n = \min\begin{cases} 0.065n \\ 0.080\dfrac{h}{B}\sqrt{\dfrac{h}{h+B}} \end{cases} \tag{17}$$

where

n is the number of floors
h the total height of the building
B the width of the building in the direction considered.

The vertical irregularity of the structure being small, it can be admitted that all frames are equally braced; therefore, action-effects on all building elements are uniformly reduced as long as the contribution of the infills is ensured. However, it is a conservative and simplified solution to adopt the rule that the analysis shall be made only on the bare frame.

If the natural period of the bare structure is considerably higher than T_2, the calculation of the natural period of the infilled structure may be made by means of a simplified method. The modelling of the infills may be carried-out by means of an equivalent strut method.

The premature shear failure of columns under possible additional shear forces due to the diagonal strut action, shall be avoided. To this purpose, columns should be appropriately reinforced against shear in the regions where these additional shear forces are expected to act.

ANNEX E - MODEL CODE FOR EARTHQUAKE RESISTANT RC BUILDINGS

1. All buildings shall have 3 shear-walls that will be identified by, I, II and III.
2. Shear-walls I and II are parallel and shear-wall III is orthogonal to shear-walls I and II.
3. The distance between shear-walls I and II, measured in the direction orthogonal to their plans, shall be greater than 60% of the largest plan dimension of the building.
4. The width and thickness of a shear-wall shall be greater than *1.00 m* and *0.20 m*, respectively.
5. The area of the sections of shear-walls I and II and of shear-wall III shall be greater than $0.003A$ and $0.005A$, respectively, where A is the area of all floors above the level considered.
6. Each shear-wall shall have a reinforcement 50% higher than the minimum reinforcement.
7. Other shear-wall systems may be used if their resistance in the two orthogonal directions and in torsion is greater than the resistance of shear-walls I, II and III.