EC 8: Eurocode Approach to Seismic Design

EC 8: Un Eurocode pour le projet de structures résistant aux séismes

EC 8: Der Eurocode-Ansatz für die Erdbebenbemessung

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SUMMARY

After a general presentation of the organisation of the Eurocode 8, which will cover all the common types of civil engineering works, the philosophy of seismic design adopted by EC 8 is outlined, with specific reference to reinforced concrete buildings. The essence lies in endowing the structure with a global ductile behaviour, as a result of a careful dimensioning of the single elements and of a structural concept such that an element's ductility is properly exploited.

RESUME

Après une présentation générale de l'organisation de l'Eurocode 8, qui couvre tous les types courants de constructions, l'approche adoptée par l'Eurocode 8 est décrite en particulier pour le projet de bâtiments en béton armé résistant aux séismes. Le principe essentiel consiste en la réalisation de structures possédant un comportement global ductile, ce qui ressort d'un dimensionnement approprié des éléments structuraux, ainsi que d'une conception structurale d'ensemble qui permet une exploitation diffusée des ductilités individuelles.

ZUSAMMENFASSUNG

Eingangs wird der generelle Aufbau von EC 8 vorgestellt, der nach seiner Fertigstellung alle gängigen Ingenieurbauwerke behandeln wird. Die Philosophie der erdbebensicheren Bemessungen wird am Beispiel von Stahlbetongebäuden illustriert. Der Kerngedanke ist, eine globale Tragwerkszähigkeit zu erreichen, als Resultat sowohl einer sorgfältigen Durchgestaltung der einzelnen Tragelemente als auch eines Gesamtkonzepts, das die Zähigkeit der Einzelelemente richtig ausnützt.

1. INTRODUCTION

The Eurocode N.8 will consist in its final form of a set of six normative documents, covering the field of the more common engineering works in the following order:

Part 1 - General and Buildings
Part 2 - Bridges
Part 3 - Towers, Masts and Chimneys
Part 4 - Tanks, Silos and Pipelines
Part 5 - Foundations, Retaining Structures and Geotechnical Aspects
Part 6 - (Appendix) - Prefabricated Structures

More specifically, Part 1: General, describes the principles of seismic design proper to EC8 which are meant to be applicable to all types of structures. These essentially include: a) the specification for the seismic case of the two Limit-States of Serviceability and of Collapse, b) the physical model adopted to describe the seismic motion, c) the adaptation required to convert it into a design action (most notable the notion of force-reduction factor (called behaviour factor), d) the admissible methods of analysis for determining the earthquake effects, e) the combination of the seismic action with the other permanent and variable actions to be used for the safety verification.

The remainder of Part 1 is devoted to buildings, and contains one material-independent chapter which specializes to buildings the admissible methods of analysis, the combination factors for variable loads and the format of the safety verifications, followed by seven chapters covering: Concrete Structures, Steel Structures, Composite Structures, Timber Structures, Masonry Structures, Mixed Structures, Elements and Appendages.
The last chapter on Strengthening and Repair is presently (June 1992) at the stage of a first draft and its final revision is due by the end of 1993.

Of the six documents of EC8, only Part 1 (without Strengthening and Repair) has been published (by the end of 1988) and distributed within CEE and EFTA countries. The enquiry phase, which lasted until mid 1991, brought a large amount of comments, which have been already processed by the competent panel and are in the course of being introduced in a new draft. Although the form of presentation will undergo noticeable modifications, the substance of the document will remain essentially unchanged.

Part 2 (Bridges) and Part 5 (Foundations) have also been already diffused in a draft stage among CEE and EFTA member countries, so that by this time a number of engineers of these countries should have a certain familiarity with them.
All the six Parts of EC8 will be finished by the end of 1993 and subsequently published as Pre-European Norms (ENV).

Considering the state of progress described in the above, this presentation will focus on Part 1: Buildings, only. Also, for reasons of space, Reinforced Concrete Buildings only will be treated. Even with these restrictions, the presentation does not aim at providing a too much detailed account of the various procedures and rules, but rather at ensuring a global vision of the approach adopted in EC8 to fulfill the stated objectives of protection against the earthquake hazard.

2. SEISMIC ZONATION. MODEL OF SEISMIC MOTION

In order for EC8 to be applicable, national territories must be subdivided into seismic zones, and for each zone the values of one intensity parameter characterized by chosen probabilities of exceedance must be given.

The zonation parameter in EC8 has the dimensions of an acceleration, and is meant to be used as the scaling factor of either a normalized response spectrum or of a unit peak time history representation of the motion.
Therefore this parameter has to be understood as an "effective" peak ground acceleration, not necessarily coincident with the actual peak (typical is the case of near field shocks, characterized by short duration and by few single-frequency pulses, for which the "effective" PGA is much lower than the actual one, due to the smaller damaging potential of this excitation, as compared with that of one of long duration and wide-band spectrum).

In line with the prevailing tendency within modern seismic regulations, EC8 does not present the user immediately with the design action, but derives this last from an (idealized) physical model of the phenomenon. This has the advantage of allowing to check the suitability of the underlying model to the characteristics of the various regions of applicability of the code, and gives also a better understanding of, and a greater flexibility in the modifications required for adapting the model to local situations, and for transforming it into a design action.

The reference model adopted in EC8 for the definition of one component of the earthquake motion is represented by an elastic response spectrum. This point-like definition of the motion is adequate for all but the extended-in-plan structures, such as bridges, pipelines, and unusually large buildings.

In EC8, the motion at a point is described in the most general terms by three translational and three rotational components, assumed to be independent from each other.
Rotational components are only considered for tall structures, such as towers and bridge piers: they will be briefly mentioned later on, together with the model for the spatial motion.

2.1 Normalized elastic response spectrum

The shape of the elastic response spectrum normalized to unit peak effective acceleration is shown in fig.1. The spectrum is meant for a damping factor of 5% and the ordinates are assumed to have a probability of exceedance of 0,20÷0,30. With the further indication of an "effective" duration, the definition of the motion in terms of an elastic response spectrum is entirely equivalent to one in terms of a pseudo-stationary random process characterized by a power density spectrum univocally related to the given re-

sponse spectrum and, consequently, to the samples of the random process that can be generated from it.
The use of the equivalent representation in terms of compatible power density spectrum and of artificially generated accelerograms is explicitly allowed in EC8.

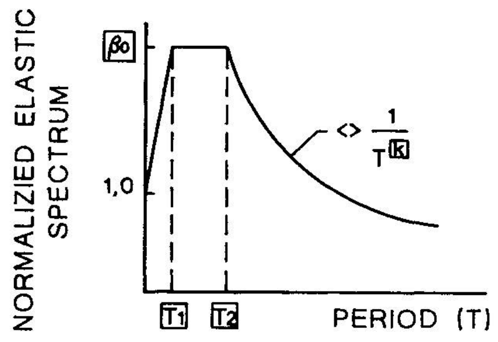

Fig.1 General shape of the normalized elastic response spectrum

The spectrum in fig. 1 is defined by the four parameters: β_0, T_1, T_2 and k, where β_0 is the spectral amplification in the constant acceleration branch, which is limited by the two corner periods T_1 and T_2. The values of T_1 and T_2, (as well as β_0 and k) are left for decision to each National Authority, since they depend on the range of magnitudes of interest and on the distance from the dominating sources to the site. β_0 depends on the expected frequency content and duration of the motion, while for not-too-distant and moderate-to-large magnitudes the value of k is normally comprised between 0,9 and 1,0.

The four parameters model of fig.1 is flexible enough to describe a variety of possible local conditions in the high-to-medium frequency range. For periods longer than about 5 secs the hyperbolic shape becomes inadequate, since it corresponds to an indefinitely constant pseudo-velocity spectrum and to a linearly increasing displacement spectrum.
It is stated in EC8 that when long periods are of interest the spectrum may be modified in that range based on documented assumptions. One simple possibility consists in making use of the peak soil displacement: d, which is also given in the code in the form:

$$d = \alpha\ d_0$$

where α is the peak soil acceleration (in percent of gravity) d_0 a factor which depends on site soil conditions (for rock or stiff soil the suggested value is d_0 = 60 cm).
Assuming a response amplification factor for d (which for stiff soil is in the order of 1,5), the spectrum could then be modified by introducing a constant displacement branch, which would start at the value of the period for which the original and the modified spectral displacements coincide.

2.2 Site - dependent elastic response spectra

Local soil conditions are known to influence the vibration characteristics of the earthquake at the surface. This effect is accounted for in EC8 by modified shapes of the response spectra as a function of the soil profile.

Although additions and/or modifications are allowed in order to meet specific situations, EC8 considers in general three soil profiles:

Soil profile A: Rock or stiff soils

Soil profile B: Medium soils

Soil profile C: Loose granular soils or clays with reduced stiffness

The corresponding response spectra are obtained from the basic definition for the rock situation with appropriately changed parameters (β_0; T_1; T_2 and k) further multiplied by an additional soil parameter S.

This soil parameter, whose values are suggested in the Commentary (along with the other shaping parameters), is intended to cover the influence that the type of soil may have on the peak ground acceleration. In fact it has been found that, keeping all other relevant variables constant, the peak acceleration in large intensity earthquakes tends to be smaller in soft soils due to their response in the nonlinear range under strong intensity vibrations. The value suggested for S in soil profile C is 0,8.

In Fig. 2 the normalized elastic site dependent response spectra for the three soil profiles are depicted in accordance with the values of the shaping parameters suggested in the Commentary to the EC8 text.

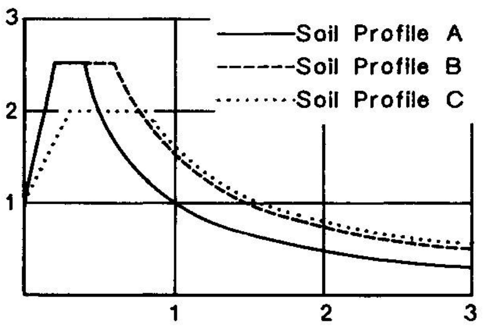

Fig.2 Normalized site dependent response spectra

2.3 Design Spectra

For most of the structures that EC8 is intended to cover the design is made by deliberately exploiting, to various extents, the capability of energy dissipation that intervenes after the elastic threshold of the materials is exceeded.
To this purpose the design forces are reduced with respect to those obtained considering the response as elastic, of an amount which depends on the ductility that each particular structure can offer and the designer is willing to use.

Inelastic response spectra for given ductility factors can be derived rigorously for single d.o.f. oscillators having specific restoring force characteristics: this approach is actually retained in some seismic codes.

Similarly to other recent codes however, EC8 prefers a simpler (and perhaps more realistic) approach: the design spectrum is obtained from the elastic one having the selected return period by dividing its ordinates by a factor accounting for the energy dissipation capacity of the whole structure. In EC8 this factor is taken as constant for all the periods, except in the range from $T=0$ to $T=T_1$. Since it is known that the ductility demand (for the same strength) increases with decreasing periods in the low periods range, the solution adopted is to make the reduction factor decreasing linearly from its actual value at $T=T_1$ to the value of unity for $T=0$.

The only further modification suggested in EC8 for deriving the design spectrum from the elastic one is the change of the exponent for the descending branch. Instead of $k=1$, it is suggested to assume $k=2/3$: it is essentially for pragmatical reasons, in order to avoid too low design forces in the long periods range. Also, a lower ceiling to the design forces is introduced, equal to $(0,20)\alpha$ (the factor 0,20 is at the discretion of the National Authorities).
As one example, in fig.3 various design spectra for different values of the behaviour factors (q) are shown, using the values of the shaping parameters suggested by the code for a soil type B.

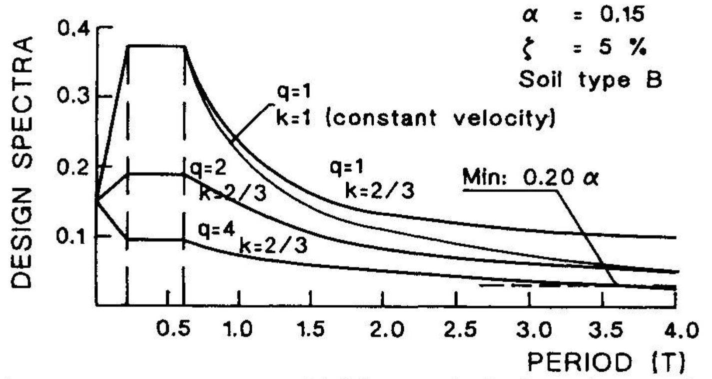

Fig.3 Design Spectra for different behaviour factors q

2.4 Wave propagation effects

When the propagating nature of the seismic disturbances is taken into account, the definition of the motion at points lying on the soil surface should consider two additional aspects:
- the presence of rotational components about the horizontal and vertical axis;
- the variability of the motion from point to point.

Quantification of both effects requires knowledge of the relative contributions of the different types of body (P,S) and surface (R,L) waves to the total motion, together with their respective frequency-dependent and dissipative characteristics.
This decomposition, which varies with the distance from the source, is hardly feasible in practical situations, so that recourse to highly simplified conservative models is unavoidable.

For the rotational effects, a simple kinematic model where the entire motion is attributed to a single type of wave, gives for the angular acceleration response and power density spectra the expressions:

$$R_i^{\vartheta}(T) = \gamma_i R_i(T)/(c\ T) \qquad i = x, y, z$$

$$S_i^{\vartheta}(\omega) = \delta_i S_i(\omega)\ \omega^2/c$$

where γ_i, δ_i are factors, c is the wave velocity, and $R_i(T)$ and S_i are the corresponding spectra for the translational components.
The previous expressions are given in EC8 Part2, with the indication that their use is only warranted for special soil conditions (low values of c), to be specified by National Authorities.

The spatial variability is more difficult to model realistically, since the main contribution to it usually not provided by the phase lag but from the complex phenomena arising from the presence of geological inhomogeneities and discontinuities.

EC8 Part 2 gives in the Appendix a simple stochastic model based on wave propagation/attenuation, according to which the power density functions at stations a and b situated at a distance d are:

$$S_{pq}^{bb}(\omega) = r_p r_q S_{pq}^{aa}(\omega) \qquad p,q = x, y, z$$

$$S_{pq}^{ba}(\omega) = r_p \exp(-i\omega d / c) S_{pq}^{aa}(\omega)$$

where r_i measures the energy dissipation between a and b due to radiation and frictional effects, for the ith direction of motion.
It is stated in EC8 that a model of this type should only be needed for cases of geological discontinuities or marked topographical features.

3. THE SAFETY FORMAT OF EC8

It is well recognized that designing for earthquake actions is more challenging than for other types of loadings, one of the major reasons being the wider amount of uncertainties presiding over almost all steps of the analysis. This fact makes the reliability conferred to a structure through the design process sensitive to possible inadequacies in the assumptions and in the procedures all along the process; it is therefore important to point out at least the more significant of the design steps to which a certain contribution to the overall reliability is allocated and to discuss their relative importance.
The ensemble of the measures taken to achieve the required performance is termed the "Safety Format" of a Code: the one proper to EC8 is now outlined in schematic sequential form.

1) *The selection of the acceptable annual probabilities of exceedance (i.e. of their average return periods) of the seismic events to be considered for the design, with respect to the two L.S.'s of damage and of collapse.*

This is obviously a major factor for the quantification of the global safety (for brevity we will concentrate in the following on the L.S. of collapse).
It might be worth recalling that the definition of the RP of the seismic event for the collapse L.S. is only one component contributing to the RP of the collapse event itself, the difference being due to the compounding of all the further uncertainties. These uncertainties are such that given the design event there is still a not negligible (and accepted) probability that a code-designed structure may actually collapse.

2) *The selection of the value of the behaviour factor q.*

The importance of this step becomes obvious if one considers that the value of q may range from 1 to 5 or more, and that it combines directly with the elastic spectrum of step 1 to yield the design action.
After the combination, an error of, say, +50% in the value of q is indistinguishable from a variation of +50% of the elastic spectrum and consequently in a substantial variation of its RP. This amounts to say that the choice of q is exactly as critical as selection of the RP of the design seismic event.

Considering for example the case of R.C. buildings, the q values are given in EC8 as functions of the following factors:
- structural typology (there are three structural types);
- degree of regularity (there are two regularity types);
- level of structural ductility(there are three ductility levels).

There are in total 3x2x3=18 possible combinations of the above factors in EC8 Part1, and a corresponding number of q values, not all of them different. The values are supposed to be calibrated to give the same amount of protection to the population of buildings falling in the various combinations.

The most difficult and still open problem here lies in a rational quantitative definition of structural regularity. The common understanding of this attribute is the ability of a building to vibrate inelastically with a ductility demand spread almost uniformly among the chosen dissipative elements, and with a vibrational shape not departing substantially from the elastic one, and also predictable by using simplified models and methods.

Until now, it has not been possible to relate with sufficient rational support the features of the response described above to the morphological and mechanical (i.e., strength, stiffness and mass distribution) characteristics of the structure.
The rules presently contained in EC8 are under revision: the perception is that if the structure is analyzed dynamically and with a full three-dimensional model (which is now affordable by professionals involved in seismic design) the essence of the response is captured, and only few additional penalties should be provided to account for post-elastic behaviour, mainly in the cases of pronounced vertical discontinuities in strength and for very abnormal distributions of the resisting elements in plan.
Research is going on to devise synthetic indicators for these two situations and to arrive at quantitative relations between them and the values of q.

Structural ductility is obviously the major factor influencing the value of q. This quality comes as the result of a combination of a number of factors: the presence in the structure of inherently ductile structural elements, the arrangement of these elements in a way that their dissipative capacity can actually be exploited during the dynamic response, the proportioning of the non-dissipative elements such that they remain essentially undamaged, to permit the dissipative ones to perform as expected.

Table I: Basic behaviour factors q for $T>T_1$

Structural system		Ductility Class	Regularity class Rh	Regularity class Rm
Frame		L	2.0	1.5
		M	3.0	2.5
		H	5.0	4.0
R.C. walls	Coupled	L	2.0	1.5
		M	3.0	2.5
		H	4.5	4.0
	Uncoupled	L	1.5	1.0
		M	2.5	2.0
		H	3.5	3.0
Core structures		L	1.5	1.0
		M	2.0	1.5
		H	2.5	2.0
Dual	Hframe >65%	as per frame systems		
	Hwalls >65%	as per wall systems		
Regular structures behaving essentially like inverted pendulum		L	1.0	
		M	1.3	
		H	1.7	

In a building of given geometrical-structural layout the global ductility can be enhanced by jointly increasing the dissipative capacity of ductile elements ("detailing for ductility") and ensuring that the ductility demand concentrates in the maximum possible number of ductile elements only ("capacity design").

EC8 offers the possibility of opting for three different levels of global ductility, called ductility classes (DC), with q factor values calibrated accordingly.

The values of q presently considered in EC8 for concrete structures are given in Table I.

3) *Structural model and methods of analysis.*

In the codes of the past, the design seismic action was introduced directly as a system of external forces to be applied to the whole building, and with the limited computing resources available until recently, engineers had frequently to resort to more or less ingenuous and accurate "dissections" of the structure into separate vertical resisting frames and/or walls; the effect of these practices on the reliability of the final design was essentially dependent on the skill and the conservatism proper to the engineer.
In EC8 the adoption of a complete model for the whole structure is taken for granted: what type of model and associated analysis depends exclusively from the characteristics of regularity of the structure.

For <u>regular</u> buildings, use can be made of planar models (that is, all the vertical resisting elements in one direction squeezed in a plane), one for each principal direction.
Torsional effects due to unintentional eccentricities between the centres of gravity and stiffness are accounted for in a simplified way by amplifying the action effects found from the planar model by the factor:

$$z = 1 + 0,6\,\frac{x}{L}$$

where L is the dimension in plan of the building, and x is the distance of the element (frame or wall) under consideration from the centre of symmetry, both measured perpendicularly to the direction of the seismic action.

For <u>regular</u> buildings a static analysis is permitted (denominated in EC8 as "simplified dynamic analysis" in that corresponds to a first mode response spectrum approach, with a linear modal shape and all the mass of the building attributed to this mode); with the limitation, however, that the fundamental period does not exceed the value of: $2T_2$, where T_2 marks the end of the horizontal plateau of the spectrum.

<u>Non regular</u> buildings have to be analyzed on the basis of a three-dimensional model, using dynamic multi-mode response spectrum analysis. This latter analysis is also required for regular buildings whose period is longer than: $2T_2$.

4) *Combination of seismic action with other actions.*

A proper accounting of the presence of the various types of actions during the design seismic event is of obvious relevance within the reliability format.
In the seismic case the problem of load combination is two-fold, since gravitational loads present in a building contribute to the horizontal inertia forces, in addition to acting vertically on the structural elements, but the probabilities of the total loads possibly present at all floors and the local loads in single elements

are clearly different. Therefore, different values of the combination factors are to be used for the two purposes.

In EC8, the combination of actions to be considered when checking the elements for the U.L.S. takes the following form:

$$+ \gamma_I E + G + P + \Sigma_i \psi_{2i} Q_{ik}$$

which is generally consistent with the fundamental expression given in EC1 - Basis of Design, in the sense that all γ factors are taken as unity, and the variable actions are assumed at their quasi-permanent value.

G and P indicate the permanent loads at their characteristic values and the prestressing forces at their long-term values respectively, while the factor γ_I, also called "importance factor" has the effect of varying the intensity and hence the return period of the seismic event according to the importance category to which the building belongs.
At this moment EC8 is proposing four categories, with suggested γ_I values ranging from 0,8 to 1,4.

The values of the ψ_{2i} have to be taken from the standardized data of EC1. For the determination of E, however, the variable loads Q_{ik} must be affected by factors accounting for the probability of their not being present over the entire structure at the occurrence of the design event, as well as of the probability of their presence with values smaller than their characteristic values. The reduced factors are given in EC8 in the following form:

$$\psi_{Ei} = \varphi \, \psi_{2i}$$

with proposed values of φ for multistorey buildings and various categories of loads ranging from to 0,5 to 1.

Having regard to the steps of the design process following the determination of the action effects as given by the formal expression above, it is appropriate to anticipate from now that these effects will be assumed directly for the design of only some, not of all, the structural elements. The design of the remaining elements will not be based on the results of the analysis, but on the capacity of the elements already dimensioned. This procedure is outlined in the following paragraph.

5) *Capacity design procedures.*

Seismic design according to EC8 being based on the possibility of dissipating energy, in different amounts, through inelastic behaviour, specific provisions are incorporated to enforce this behaviour: these go under the general name of "capacity design" (CD) procedures.
The concept has been developed in New Zealand codes since almost 20 years now, and it is increasingly adopted in the revised codes of all the major seismic countries in the world.
It consists in choosing the desired post-elastic mechanism of the structure (i.e. the most reliably dissipative one) and then in providing all elements and mechanisms for which inelasticity is not expected with greater strength than required to resist the likely maximum attainable by the yielding elements.

The desired mechanisms are: the formation of plastic hinges at all beam extremities and at the bases of the columns (beams sidesway mechanism) for framed structures, and the formation of plastic hinges at the bases of the walls and yielding in all coupling beams in wall structures.

Unwanted mechanisms are: hinging at the extremities of the columns (with few well defined exceptions), inelastic shear deformations in beams, columns and walls, inelastic behaviour due to cracking and loss of bond in beam-column joints, inelastic behaviour of foundation structures and foundation soil.

As an example, the application of the CD procedure to the design of an interior column under bending, axial force and shear for the higher ductility class will be illustrated.

- The bending moments (with the corresponding axial forces) obtained from the analysis under the seismic load combination are denoted by M_{Sc1} and M'_{Sc2}, see fig.4

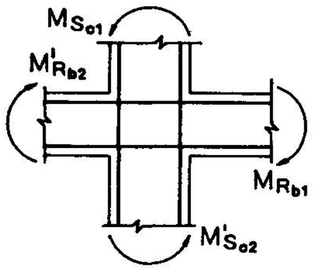

Fig.4 Application of capacity design procedure at a beam column joint

- The ultimate bending moments developed by the end sections of the beams framing into the node are indicated with M_{Rb1} and M'_{Rb2}. These moments are evaluated considering the actual amount of reinforcement present but using the usual factors for the strengths of concrete and steel .

- From the equilibrium of the node under the assumption of full reversal of the moments in the beams the following ratio is constructed (see fig.4): (the opposite signs should also be considered and the maximum value of α_{CD} in the two cases should be assumed)

$$\alpha_{CD} = \frac{\gamma_{Rd}(|M_{Rb1}| + |M'_{Rb2}|)}{(|M_{Sc1}| + |M'_{Sc2}|)}$$

The factor γ_{Rd}, equal to 1,35 for the ductility class "H", is meant to recoup for the strength factor γ_S=1,15 used in evaluating the beam strengths, and also to account for both the variability of the yield strength of steel and a certain amount of strain hardening occurring during the plastic rotations at beams ends.

- The factor α_{CD} ($\alpha_{CD} \leq$ q) is then applied to the moments M_{SC1} and M_{SC2} to obtain their respective design values.

- The CD criterion is finally used sequentially to evaluate the design shear forces in the columns, i.e.:

$$V_{dc} = \gamma_{Rd} \frac{M^u_{Rc} + M^l_{Rc}}{l_c}$$

where M^u_{Rc} and M^l_{Rc} are the resisting moments of the column upper and lower sections evaluated with the same criteria indicated for the beams.

Capacity design principles apply in an analogous fashion to the other cases mentioned before: it is seen that their implementation is relatively easy both conceptually and practically.

A note is in order, however, concerning the calibration of the CD factors. EC8, like all the other EC's, adopts a Level I, partial safety factors format and all the factors affecting the material parameters (γ_m) have been calibrated in the past under the assumption that a uniform lower-than-average distribution of strengths would represent the most unfavourable case.

On the contrary, given the objectives of ductile seismic design, the scatter towards higher-than-average values of the strength in some elements may lead to dangerous situations, because undesired mechanisms may be activated in other parts of the structure. The variability of the material properties must therefore be controlled in the ranges of both the lower and the upper fractiles.

The problem of a correct calibration of CD factors is not irrelevant, since a fully deterministic approach might lead to overconservative and uneconomical results. On the other hand, a system reliability approach to calibration has not been attempted so far in large scale because of its onerosity, and it would be scarcely compatible with a Level I safety format.
Considering the importance of the CD procedures in the overall process of seismic design, the lack of a rigorous support for the values of the factors presently adopted is a lacuna worth some efforts for its elimination.

6) *Dimensioning and detailing.*

A few highlights only on these aspects will be presented, mainly for the sake of completing the review of the safety format of EC8. Once the design action effects are obtained, the design of the different elements involves essentially the satisfaction of two requisites: strength and ductility.

<u>Strength</u>

Design for bending and bending with axial force for beams, columns and walls is made at the ULS using the same procedures and material factors given in EC2 independently of the type of action.

In principle, for combinations which include one accidental action, the optimal (from a probabilistic viewpoint) values of the γ_m factors should be close to unity, although this fact is not explicit mentioned in EC1: Basis of Design. The use of the ordinary set of γ_m values is justified, however, in EC8 with the argument that the reasons for taking the γ_m's close to unity are counterbalanced by the fact that material properties suffer a certain amount of deterioration due to repeated cyclic imposed deformations.

Design for shear in linear elements is also carried out as in EC2, except that for beams the contribution of concrete is taken as zero in the zones of potential hinge formation.
For walls, different expressions are used to evaluate the amount of horizontal and vertical reinforcement necessary to avoid web diagonal tension failure, depending on the value of the shear ratio: $M_d/V_d.l_w$, where M_d and V_d are the design values of M and V at the base of the wall, and l_w is the width of the wall.

<u>Ductility</u>

High local ductility in the critical regions of the elements is the prerequisite for achieving the stipulated amount of global dissipation from the structure.
EC8 defines curvature ductility as the ratio of the curvature at 85% of the peak moment on the descending branch over the curvature at yield of the tensile reinforcement (Conventional Curvature Ductility Factor: CCDF).

In the case of beams, adequate amount of CCDF is assumed to be achieved through proper detailing, while for columns and walls the required values of the CCDF are specified, and expressions for the amount of longitudinal and confining reinforcement needed to comply with the values are given.

The ductility provisions for beams are similar to those already well experimented in other codes, and are graded according to the chosen ductility level. For example for the higher DL, confining hoops must be provided in each portion of the beam close to a column, for a length not shorter than twice the depth, with a spacing taken as the minimum of: 1/4 of the depth, 24 times the hoop bars diameter, or 150 mm, and also not greater than 6 times the diameter of the longitudinal bars, this last provision intended to avoid the buckling of the bars.

For longitudinal reinforcement, prescriptions are given concerning the total geometrical percentage, which must not be less than: 1/2 f_{ctm}/f_{yk} (f_{ctm} = average tensile strength of concrete; f_{yk}= characteristic yield strength of steel) to avoid fragility, nor greater than 1/6 f_{ck}/f_{yk}, to avoid congestion.
Additionally, a minimum of 2 bars of 14 mm diameter must run continuously at both sides, but at the top side the running reinforcement must not be less than 1/4 of the maximum, while at the bottom side in the end sections the reinforcement cannot be less than half of the top one.
All the provisions above aim at covering possible deviations of the actual bending moment distributions with respect to those given by the analysis.

As already mentioned, columns are explicitly requested to possess specified amounts of CCDF, as a safety measure additional to the

use of CD procedures, which by themselves are expected to reduce the ductility demand almost to zero.
The required values are 15,6 and 3 for the three ductility levels.

Calculations to check the above limits are not mandatory, deemed-to-satisfy rules are given for the necessary amount of confining hoops to be provided in the potential plastic hinge lengths. Prescriptions analogous to those for beams regarding the spacing and the pattern of the hoops are also given.

For walls an adequate ductility at the base represents their only line of defense (for coupled walls is less so due to the dissipation in the coupling beams), a fact which justifies a more analytical attention to the problem: the requested CCDF for DL "H" are: 1,2 q^2 and 1,0q^2 for single and coupled walls, respectively, q being the behaviour factor used in determining the design actions.
If a direct method is not used, the requirement on ductility can be assumed to be satisfied if a confining reinforcement is provided in accordance with an analytical expression of essentially empirical origin.

4. CONCLUSIONS

An outline presentation of EC8 for buildings has been given using as leit-motiv the sequence of steps where the most significant contributions to the overall reliability are allocated.
Six steps have been identified and comments and remarks on their present state and development needs have been summarily given.

1) The safety format of EC8 starts with the selection of the return period for the design seismic event. This event is described in terms of a single intensity parameter: the effective peak ground acceleration. From this step on, the purpose of the code is to ensure that, *given the occurrence of the design event,* code-designed buildings have a negligible residual probability of exceeding the ultimate limit state of near-collapse.

2) The design seismic forces are reduced with respect to those that would be induced if the structure were to respond elastically. This crucial step is accomplished via a reduction factor (q) which is a function of three characteristics of the structure: a) structural typology, b) geometrical-mechanical arrangement of the resisting elements (regularity), c)local/global ductility. The dependence of q on b) is still unsatisfactorily understood, while for c) efficient procedures exist to enhance ductility locally and to enforce a global dissipative behaviour.

3) The use of realistic mechanical models of the whole building is indispensable for a reliable evaluation of the effects induced by the seismic motion acting at the base. For buildings not provided with structural and inertial symmetry, a dynamic analysis on a full 3D model is contemplated by EC8. This somewhat onerous requirement is deemed as necessary and sufficient to capture the bulk of the effects of the non-symmetry on the response. The effects obtained with elastic models are in any case adjusted to account for post-elastic behaviour.

4) In seismic design it is necessary to distinguish between the variable loads to be considered as present in the building at the occurrence of the earthquake (which contribute to the inertia forces activated by the seismic excitation), and those which may act locally on any single element. This fact calls for two sets of load combination factors, one for the safety verifications of the elements, the second one for the analysis of the seismic effects. For buildings of usual destination the quantification of the two sets does not pose great difficulties, while for buildings having occupancies of mixed and less common types a rigorous approach is still missing.

5) Capacity design procedures are undoubtedly the single most influential step for controlling reliability in seismic design. They are able to cover uncertainties stemming from as diverse sources as the randomness of the input, the inaccuracy of the mechanical model, the simplifications in the methods of analysis, and the variability in the mechanical properties. Their aim is to ensure that the structure cannot but behave according to a preselected post-yielding mechanism, and this is done at present by using conservatively estimated upper values of the strengths that may be developed by the yielding elements. Economy and consistency in the reliability treatment will benefit from a future extensive calibration of these procedures.

6) The behaviour of R.C. elements under large reversed deformations is today known and analytically predictable to a satisfactory extent. This knowledge is translated into rules for the dimensioning and detailing of the elements which yield with adequate accuracy the desired amount of dissipative capacity. This aspect of seismic design is therefore arrived at his stage of maturity.