A LOOK INTO EUROCODE 8: THE UNIFIED EUROPEAN CODE FOR THE DESIGN OF EARTHQUAKE RESISTANT STRUCTURES

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ABSTRACT

The European Union, enlarged to include the EFTA countries for a total of eighteen european states, is concluding the first phase of preparation of a homogeneous set of Standards for structural design, called the Eurocodes. It is intended that these Standards will ultimately acquire a supranational level and will supersede national codes. Eurocode 8, dealing with seismic design, has just recently reached the status of a Pre-Standard, which allows it to be adopted in any of the above states. By providing an outline of the content of Eurocode 8, it is hoped to raise the interest of the international community towards it, both with a view to the benefits that can be expected from their interaction and, in the longer run, to a more far reaching harmonization of technical codes.

1 PROLOGUE

Reading a new code for the purpose of putting it to professional use is certainly a dry matter but the need for practical application should partly compensate for the effort.

The reading of a presentation about a foreign code whose differences from a more familiar one may be more difficult to find than the similarities, whose comprehension is made difficult by the imperfection and oddity of the language used, and whose impact in one's own professional life is uncertain and in any case may not arise in the near future, calls for a very special kind of reader.

Writing a presentation about a code, on the other hand, is also a thankless task. From the start, one is discouraged by an awareness of the likely unenthusiastic attitude of most of the readers. Thinking about a possible style to make it more attractive, one can hardly draw any inspiration from the existing "literature". On the other hand, sticking to an impersonal summary would leave one with a sense of an unintelligent and wasteful effort.

But, one might then ask, if the objective is really such a painful one for all sides, why are code presentations written at all? Well, at least one obvious reason exists in favour of doing this. A code, as a subject of wide public interest, cannot be put into use without an adequate introduction.

Seen from this perspective, at least for this writer, the task becomes clearer and easier. Introductions, it is generally agreed, gain in being short and incisive: their aim is to draw the attention due to the importance of the subject on the basis of its essential traits. If the subject is valid, it will made itself known fully through its own merits.

It is with these consolatory thoughts in mind that the following presentation has been written.

2 INTRODUCTION

The Commission of the European Community (CEC), which is the executive arm of the Community (now Union), decided in the late seventies to produce a set of harmonized norms covering the whole field of structural design, for application in all member states. The decision was connected to the intention of removing trade barriers within the Community, barriers due to differences in design and construction rules and practice, and of enhancing the competitiveness of the Community as a whole in the global scene.

Eurocode 8 is the eighth document in a family of nine. Since it treats the aspects related to seismic resistance only, it has to be used in conjunction with the other EC's, as relevant: EC2 for reinforced concrete structures, EC3 for steel structures, EC4 for composite steel and concrete structures, EC5 for timber structures, EC6 for masonry structures and EC7 for geotechnical design.

EC8 in turn is composed of six separate documents, covering different types of structures. Part 1 of EC8, which is the subject of this presentation, is titled ***General and Buildings***, in that it contains a section describing the principles applicable to all types of structures, followed by the implementation of these principles into rules valid for buildings.

An earlier version of EC8 Part 1 was published at the end of 1988 and made public in Europe by means of ad hoc courses, seminars and presentations in conferences, some of them international. In this way the news of its existence started spreading outside Europe.

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In 1992, as a result of an enquiry among member states which brought a substantial amount of comment, the document has been put into a revision process which ended with the formal vote of approval of a new text on June 1994.

The Prestandard EC8 [1], as it is called, has an initial life of three years and can be immediately used, provided individual countries insert the appropriate values for certain key safety elements which have been given only indicatively within brackets (boxed values), but which are intended as elements of flexibility to suit local conditions of economy, hazard and technological level.

After approximately two years of experimental application, member countries will be asked to submit formal comments which may lead either to approve the document as a Standard, or to revise it into a new Prestandard to remain in use for a further period of two years. Once the Standard is approved, there is a fixed time limit within which all existing conflicting national Standards have to be withdrawn.

3 PART 1.1 SEISMIC ACTIONS AND GENERAL REQUIREMENTS FOR STRUCTURES

3.1 General Requirements

This Part of EC8 delineates with statements of qualitative nature the character and the purpose of the code. Further, it contains a description of the seismic action whose format is meant to be valid for all types of structures.

Since their birth, all the Eurocodes are commonly defined as "performance oriented" and "reliability based" types of documents, in the sense that their purpose is explicitly stated at the beginning and that the fulfilment of their objectives is only measurable in probabilistic terms.

At the outset the purpose of EC8 is stated as being that of ensuring that, in the event of earthquakes:
- human lives are protected
- damage is limited
- important public structures remain operational

In a second step, these general design objectives are translated into two requirements of structural significance:
- the no-collapse requirement
- the damage limitation requirement

To check satisfaction of these requirements, recourse is made as usual to two limit states (LS): the Ultimate and the Serviceability LS.

The ULS is defined as a state in which the structure preserves its integrity and a residual capacity of carrying vertical loads, but it is close to the exhaustion of its available ductility.

The SLS is defined as corresponding to an amount of damage and interruption of use whose cost is of the same order of the cost of the structure itself.

3.2 Model of seismic action

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

This is no small feat to accomplish, since at present countries have hazard maps derived independently and based on different macroseismic scales (IMM, MSK,MCS, magnitude,...), which obviously do not fit with each other across the borders.

The zonation parameter in EC8 has the dimension 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.

This parameter has therefore 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 can be much lower than the actual one).

In line with the prevailing tendency within modern seismic regulations, EC8 does not immediately introduce the design action, but derives this last from an (idealized) physical model. This gives the necessary 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 translation 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.

3.2.1 Normalized elastic response spectrum

The shape of the elastic response spectrum normalized to unit peak effective acceleration is shown in Figure 1. Unless otherwise specified, it is assumed that the shape is not dependent on the magnitude of the scaling factor. 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

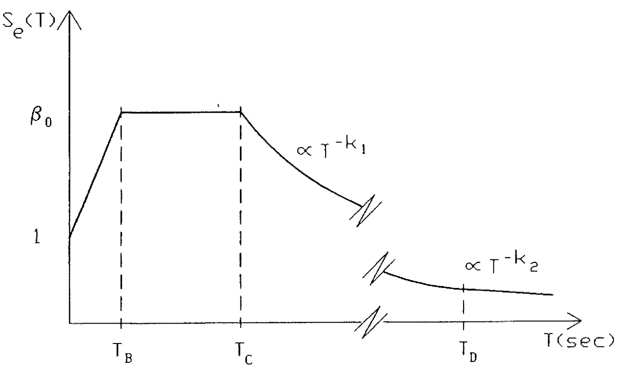


***Figure 1** Normalised elastic response spectrum*

random process characterized by a power density spectrum univocally related to the given response 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.

The spectrum in Figure 1 is defined by six parameters:
- β_0 which gives the spectral amplification in the constant acceleration branch, whose suggested (boxed) value is 2.5;
- T_B, T_C, T_D, the corner periods separating the three branches having constant spectral acceleration, velocity and displacement, respectively;
- K_1, K_2, the exponents of the descending portions (inversely proportional to T_k), whose values must be $K_1 = 1$ between T_C and T_D) and $K_2 = 2$ for $T \geq T_D$ if the spectral velocities and displacements have to be constant.

The influence of local soil conditions is accounted for by considering three subsoil classes: A, B, C, classified in decreasing order of the overall stiffness of the soil. The type of subsoil is reflected essentially in the values of the corner periods T_B and T_C, which are both shifted towards higher values in going from A to C, while only for class C (weak soils) a reduction (of 10%) is foreseen for β_0.

The standard classification is not applicable and ad hoc amplification studies must be undertaken in the two cases:
- when the profile includes an alluvial surface layer of thickness varying between 5 and 20 m underlain by much stiffer material of class A.
- when a soil deposit of class C consists of, or contains, a layer of at least 10 m thickness made of soft clays/silts with plasticity index PI > 40 and high water content. This latter case is considered to have a potential for abnormally high selective amplification.

3.2.2 Design spectrum

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 intervening once the elastic limit is exceeded.

To this end, 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 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 formal terms, the so-called behaviour factor q is defined as the ratio between the forces a structure would experience if its response were fully elastic and the minimum forces that can be used, in conjunction with a linear analysis and all the rules foreseen in EC8 for the dimensioning and detailing of the elements, to obtain a design which satisfies the ULS with an acceptable degree of reliability.

In EC8, q is taken as constant for all the periods, except in the range from $T = 0$ to $T = T_B$. Since it is known that the ductility demand (for the same strength) increases with decreasing periods in the low period range, the solution adopted is to make the reduction factor decrease linearly from its actual value at $T = T_B$ to the value of unity for $T = 0$.

The second modification, in passing from the elastic to the design response spectrum, applies to the exponents K_1 and K_2, whose suggested values are 2/3 and 5/3, respectively.

This is made on purely pragmatical basis, i.e., in order to avoid too low design forces in the long period range.

3.2.3 Alternative representations of the seismic actions

Detailed guidance is given in Part 1.1 on the adoption of power spectrum and time-history representations so that they are equivalent to the elastic response spectrum.

The former approach is often used in problems of soil-structure interaction, while the second is needed whenever a non-linear analysis is required or desired.

4 PART 1.2 GENERAL RULES FOR BUILDINGS. PART 1.3 SPECIFIC RULES FOR VARIOUS MATERIALS AND ELEMENTS.

The content of these two Parts is organized as follows.

Part 1.2 contains essentially three subjects:
- the rules defining what is a regular building, with the implications that this characteristic has on the models and methods of analysis, and on the values of the behaviour factor
- the description of the allowable methods of analysis
- the list of the verifications to be made to check the ULS and the SLS.

Part 1.3 is subdivided into separate chapters for: reinforced concrete, steel, composite, timber and masonry structures, with an Annex of non normative character for precast structures.

The one on reinforced concrete is by far the most extensive, with about 60 pages against 15 for steel and less for the other materials.

This obvious imbalance is the effect of at least two causes. One is the fact that steel construction is not widely used yet in the seismic regions of Europe, with the implication that the few structures designed are made by specialists who know where else to look for what might be missing. The second one, which interacts with the first, is that the body of regionally made experimental and theoretical research, on which to base detailed recommendations, is incomparably inferior to that on reinforced concrete. The situation is, however, bound to change, though slowly.

The sub-chapter on reinforced concrete gives in order: the requirements on materials, the structural types covered and the corresponding q factors, the provisions for anchorages and splicing, the design and detailing rules for beams, columns, masonry-infilled frames, beam-column joints, structural walls and diaphragms.

To cover one by one all the points which have been just listed to compose Parts 1.2. and 1.3, even in the way of an introductory outline and restricting the attention to reinforced concrete only, would be out of the scope of a single paper.

Alternatively, a reasonable idea of the main choices characterizing EC8 can be obtained from an overview of the fundamental steps which are taken by EC8 to fulfil the stated aims.

Six of these critical steps are identified; of course many of them interact together, and it is only through their integrated consideration that the intended performances can be statistically ensured.

4.1 The selection of the average return periods (RP) of the design seismic events, for the ULS and SLS

This is obviously a major factor for the quantification of the global safety (for brevity we will concentrate in the following on the LS of collapse).

It might be worth recalling that the definition of the RP of the seismic event for the collapse LS 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.

4.2 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 linearly with the ordinates of 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 not distinguishable 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.

The value of the behaviour factor q is given by the following expression:

$$q = q_o.K_D.K_R.K_W \geq 1.5,$$

where

- q_o = basic value of q, dependent on structural type (3 main types);
- K_D = factor reflecting the ductility class (3 classes);
- K_R = factor reflecting the structural regularity in elevation (2 regularity degrees);
- K_W = factor reflecting the predominant failure mode for walls.

There are in total 3x3x2 = 18 possible combinations of the three factors q_o, K_D and K_R, 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 a population of buildings with each one representing a given combination.

The three main structural types are: frame system (q_o = 5); dual system (q_o = 5 if frames predominate; q_o = 5 or 4.5 if walls predominate, coupled or single, respectively); wall system (q_o = 5 or 4 for coupled and single walls, respectively).

EC8 offers the possibility of opting for three different levels of global ductility, called ductility classes, with K_D values of 1.0, 0.75 and 0.5 for ductility classes H, M, and L, respectively.

Given the geometrical-structural layout (structural type and degree of regularity) 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 largest possible number of ductile elements only ("capacity design"). The severity of the detailing of reinforcement and of the measures of capacity design determines the value of K_D and hence of q.

The treatment of the regularity in EC8 is as unsatisfactory as it is in all other existing codes. The attributes of a "regular" structure can actually be clearly defined: a building which is able 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.

In EC8, a distinction is made between regularity in plan and in elevation. This distinction is relevant for the required modelling and methods of analysis, and for the value of K_R.

Regularity in plan is defined according to the usual criteria of double symmetry, compact shape, etc., plus an analytical criterion, whose check requires a prior static analysis of the building. Similarly to recent US and NZ practice, the criterion asks that, under the design distribution of horizontal forces, at any storey the maximum displacement in the direction of the seismic forces does not exceed the average storey displacement by more than 20%.

Regularity in elevation is linked mostly to geometrical criteria related to the admissible shapes and dimensions of the setbacks, and to the continuity of the lateral load-resisting systems (cores, walls, frames) from their foundations to the top of the building. Mass and lateral stiffness of the individual storeys should also not have abrupt changes along the height and, equally important, the ratio of the actual storey resistance to the resistance required by the analysis should not vary disproportionately between adjacent storeys.

When this last condition is violated due to discontinuous presence of external infills, ad hoc local provisions (not just the global decrease of q) must be followed.

The consequences of structural regularity on various design aspects are shown in the table below.

Regularity		Allowed simplification		Behaviour factor (K_D)
Plan	Elevation	Model	Analysis	
Yes	Yes	Planar	Static	1
Yes	No	Planar	Multimodal	0.8
No	Yes	Spatial	Multimodal	1
No	No	Spatial	Multimodal	0.8

It is seen that the "penalty" for irregularity in plan only is limited to the requirement of a more accurate (spatial) modelling and of a dynamic type of analysis. It is anticipated that this decision will not meet with general consensus among code writers.

4.3 Structural model and methods of analysis

EC8 takes for granted that its user has time and ability to set up adequate three dimensional models and to perform dynamic modal analyses of the structure without loosing sight of the physical reality. The code committee went over this policy time and again with a number of reversals of thought. What is sure now is that the requirements as they are reflect the last conviction of the committee members. Experience of use of EC8 will show whether this conviction was ill founded or not.

For the structural modelling, only for fully *regular* buildings can use be made of planar models (that is, all the vertical resisting elements in one direction squeezed into 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:

$$z = 1 + 0.6\, x/L$$

where L is the distance between the two outermost lateral load resisting elements, 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 *regular* buildings a static analysis is permitted (denoted in EC8 as "simplified dynamic analysis" in that it 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 $2T_C$, where T_C marks the end of the horizontal plateau of the spectrum.

Non-regular 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 $2T_C$

The analysis, either static or dynamic, needs to be made for two orthogonal directions of the seismic action. The action effects produced by each of the two components are then combined by taking the square root of the sum of the squares or, alternatively, by using the following expressions:

$$E_{EX} \text{ "+" } 0.30\, E_{EY}; \quad 0.30\, E_{EX} \text{ "+" } E_{EY}$$

where "+" implies combination with the most unfavourable sign.

Only in the case of wall systems, regular at least in plan, it is allowed to consider separately the effects produced by the two orthogonal components of the seismic motion.

4.4 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 on 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 ULS takes the following form:

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

where: 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 reduction factors are given in EC8 in the following form:

$$\Psi_{Ei} = \phi\, \Psi_{2i}$$

with proposed values of ϕ for multistorey buildings and various categories of loads ranging from to 0.5 to 1.

4.5 Capacity design procedures

Seismic design according to EC8 being based on the possibility of dissipating energy, in different amounts, through stable inelastic mechanisms, 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 for 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 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.

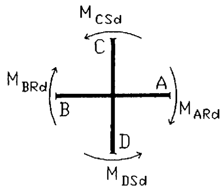
Unwanted mechanisms are: hinging at the extremities of the columns (with few well defined exceptions), inelastic shear deformations in beams, columns and walls, inelastic behaviour due to cracking and loss of bond in beam-column joints, inelastic behaviour of foundation structures and foundation soil.

Capacity design is applied to DC "H" and DC "M" structures only, since for DC "L" the inelastic behaviour is not expected to be significant.

For example, with reference to columns of framed structures the design bending moment is obtained from

$$M_{Sd,CD} = \alpha_{CD} \bullet M_{Sd}$$

where M_{Sd} is the moment given by the analysis of the structure under the seismic load combination, and α_{CD} (capacity design amplification factor) is calculated from (the signs of the moments are shown at left; the opposite signs have to be considered also, and the maximum value of α_{CD} adopted):



$$\alpha_{CD} = \gamma_{Rd} \frac{M_{ARd} + M_{BRd}}{M_{CSd} + M_{DSd}}$$

where:

- M_{ARd}, M_{BRd} are the design flexural strengths of the end sections of the beams (assuming full moment reversal) calculated with the actual reinforcement provided, but with the usual strength reduction factors for concrete (γ_c = 1.5) and steel (γ_s = 1.15)

- M_{CSd}, M_{DSd} are the moments in the columns given by the analysis of the structure

- γ_{Rd} = 1.35 for DC H
1.20 for DC M

This latter factor is meant to compensate for the strength reduction factors used for the resisting moments of the beams, and also to account for both the variability of the yield strength of steel and of a certain amount of strain hardening occurring during the plastic rotations at beams ends.

In the evaluation of α_{CD}, no difference is made depending on whether the structural analysis has been static or dynamic. This is justified by the fact that for periods below $2T_C$ the effect of the higher modes of vibration is not very significant, and is covered by the conservative assumption adopted for the static analysis (the full mass of the structure participating in the first mode).

When the gravity loads are comparatively important with respect to the seismic loads, the values of α_{CD} calculated as above may be unnecessarily severe. To cope with these situations, the following procedure is suggested. A "moment reversal factor" δ is computed (for each sign of the seismic action):

$$\delta = \frac{|M_{ASd} - M_{BSd}|}{M_{ARd} + M_{BRd}}$$

which is clearly always less than unity. A modified amplification factor α_{CD} is then assumed in the form:

$$\overline{\alpha}_{CD} = [1 + (\alpha_{CD} - 1) . \delta]$$

4.6 Dimensioning and detailing

Once the design action effects are obtained, using capacity design procedures when appropriate, the remaining steps involve the satisfaction (for the ULS) of two requisites: strength and ductility. The format of the presentation adopted in EC8 is as follows.

The different elements: beams, columns, walls, are treated separately in sequence. For each element, the procedures related to strength evaluation and to detailing which are not dependent on the ductility class are given first, followed by sub-paragraphs containing the figures and the special provisions applicable to the various DC's.

- *Evaluation of strength*

The design of beams, columns and walls for bending and bending with axial force is made at the ULS using the same formulas and material factors given in the concrete code (EC2) for non seismic types of actions.

In principle, for combinations of actions including one which is of accidental type (as it is the case for the seismic action) the most suitable (from a probabilistic viewpoint) values of the γ_m factors should be close to unity. The use of the ordinary set of γ_m values is justified in EC8 with the argument that the reasons for taking the γ_ms 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 the 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.

- *Provisions for ductility*

High local ductility in the critical regions of the elements is a prerequisite for achieving the stipulated amount of global dissipation from the structure.

EC8 defines curvature ductility as the ratio between the curvature at 85% of the peak moment on the descending branch and the curvature at 'yield of the tensile reinforcement (Conventional Curvature Ductility Factor, CCDF).

In the case of beams, an 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 DC, 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 the DC "M", the provisions above are relaxed to: the portion to be provided with hoops not shorter than 1.5 times the depth, with hoops spacing the minimum of: 1/4 of the depth, 24 times the hoop bar diameter, or 200 mm, and not larger than 7 times the diameter of the longitudinal bars.

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 CCDF values are 13,7 and 4 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 this 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 DC "H" are 1.0 q^2 and 0.8 q^2 for single and coupled walls, respectively, q being the behaviour factor used in determining the design actions.

5 CONCLUSIONS

The character and the main traits of Eurocode 8 have been briefly presented. The preparation of this document has cost a great effort to mediate between well established differences on ways of thinking, bases of knowledge, and traditional practices. From this point of view, its very existence is a success of political significance.

The technical content of EC8 may be less than perfect, but it embodies the essence of what knowledge is available today for the purposes of codified design; improvements, where necessary, will be easily introduced along the way.

Although much work has been done on the preliminary version of 1988 in the direction of simplifying, eliminating redundancies, making it more straight forward and easy to use, the result has not yet been achieved completely. Authoritative voices both inside and outside Europe [2] have expressed opinions along these lines. However, now that the framework is established, the task of revising the code will not be such a daunting one, particularly to those who have lived through the hard technical-ideological discussions on what the content should be.

In particular, if the document has to acquire acceptance outside Europe, it would be a good policy to convene international workshops to assist in the development of EC8 into its final status.

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