ASIAN JOURNAL OF CIVIL ENGINEERING (BUILDING AND HOUSING) VOL. 6, NO. 5 (2005)
PAGES 447-456

THE EUROCODE 8–PART 3: THE NEW EUROPEAN CODE FOR THE SEISMIC ASSESSMENT OF EXISTING STRUCTURES

Paolo Emilio Pinto*
University of Rome "La Sapienza"
Rome, Italy

ABSTRACT

There is a growing worldwide perception that the built environment, both historic and of recent construction, is characterised by an unacceptably high level of seismic risk. Given the technical, economical and social problems involved, mitigation of this risk cannot be but a long-term objective, whose achievement requires the availability of efficient normative documents allowing for rational and cost-effective interventions. The paper illustrates the main features of a recently released European code (Eurocode 8 Part 3) devoted to the subject. The document is aligned with the present international trend of explicitly stating and pursuing the sought performances, but contains also elements of novelty. These latter regard essentially the introduction of so-called "levels of knowledge", as a factor for deciding type of analysis and use of newly defined confidence factors in member verification, plus a set of entirely new capacity expressions for the ultimate and damage states of the members.

Keywords: performance requirements; levels of knowledge, confidence factors; capacity formulas

1. INTRODUCTION

On February 11th, 2005, Part 3 of Eurocode 8 (EC8/3,[1]) has been unanimously positively voted by the representatives of the 23 countries adhering to CEN (Comité Européen de Normalisation), which include both EU and EFTA member countries. The document is entirely new with respect to a previous draft, which was issued in the '90s, and it took only about three years to be completed and to meet with general acceptance. This is quite a remarkable fact, if one considers that for documents of much less controversial nature, as for example Part 1 of EC8, which deals with the design of new structures, it took about ten years to reach the consensus for passing from the 1994 Pre-Standard version to the present status of a European Standard. The reason for this apparent success may not be sought so much in the quality of the document, documents of truly high quality take often years of minute discussions to get approved, but rather in

* Email-address of the corresponding author: Paolo.Pinto@Uniroma1.It

the ever increasing awareness of the urgent need of doing something in the direction of alleviating the problem of existing structures.

The second half of the past century has in fact witnessed an accelerating process of growth of urban areas, which has taken place worldwide with little, if any, consideration of the existence of a serious seismic hazard and also, quite frequently, according to sub-standard design and construction practices. The full realisation of the gravity of the situation in terms of expected human and economic losses is a relatively recent fact, dating back essentially to the economically disastrous events in California at the end of the eighties (Loma Prieta, 1989) and reinforced by the following equally disastrous events in Japan (1995), Turkey (1999), etc.

There are no quick fixes to the present situation: the push towards urbanisation and industrial concentration will continue to grow and in a few cases only this process will be risk-controlled. Due to the very large economic resources required to reduce the present risk to more acceptable levels, long-term planning is the only viable approach. In this context, the availability of effective technical regulations for the seismic assessment acquires a critical role, in that it leads to a drastic reduction of the arbitrariness in the diagnosis of the capacity of the structures in their present state, and it requires an analytical demonstration of the necessity and of the effectiveness of the proposed interventions.

Unfortunately, however, the available international documents on the assessment and upgrading of existing structures cannot be said to possess a degree of maturity comparable with that of the modern seismic design codes. Work in this area has started much later, priority having been assigned to the improvement of the procedures for new designs, with the consequence that the published documents are still, with the exception of the Eurocode 8, in the form of recommendations, or at most of Prestandards, Ref.[2].

Further, it must be added that all of these documents address the assessment and retrofit of engineered structures, even if not specifically designed to resist earthquakes, and that they require modelling, analysis and verification procedures more detailed and extended than those necessary for the design of new structures, since those latter are designed so as to exclude a priori the possibility of difficult to analyse, unfavourable, local and global failure mechanisms. Hence, it is anticipated that these documents will be of use essentially in those areas of the world where, apart from economic resources, both a good number of qualified engineers and construction firms exist, and a good proportion of the building stock is made of modern materials and structural types. Knowing the real situation, this amounts to saying that, in a worldwide perspective, the impact of these new documents in mitigating the present level of risk will be percentually modest, and documents of more qualitative nature, specific for the various local construction practices, are also urgently needed.

2. THE STRUCTURE OF THE CODE

The Eurocode 8/3 adheres in full to the so-called displacement-based approach. First, three hazard levels are selected, and a performance requirement is associated to each of them. The hazard is described in the form of elastic, five percent damping response spectra having specified average return periods. The seismic action is applied to the structure without any ductility-related reduction factor, and the state of the structure (displacements, stresses) is evaluated by means of linear or non-linear types of analyses, depending on the characterisation of the structure and the choice of the engineer.

The verifications of the structural elements/mechanisms vary, depending on their nature. If they qualify as 'ductile' (bending with and without axial force) one has to check that the calculated deformation (curvature, drift) is not greater than the admissible deformation for the considered performance level. If they are of the 'brittle' type (shear, beam-column joints), one has to check that their capacity in terms of strength is not exceeded by the corresponding forces transmitted to them.

As it will be shown, a number of innovative or of uncommon features characterise the code. One of these is the introduction, in addition to the ordinary material partial factors, of new safety elements called 'confidence factors', that account for the different degree of knowledge one may achieve, or be content of, regarding geometry, amount and quality of reinforcement, etc., on the structure to be assessed. A second one is the introduction of new expressions for the ultimate flexural deformation of concrete elements, which have been obtained through statistical analysis of a large number of experimental data accumulated in the last twenty-thirty years. Finally, a third feature, which is not exclusive since it is accepted in [2] also, is the possibility of using non-linear static (push-over) analysis as a standard tool for assessment purposes.

3. PERFORMANCE REQUIREMENTS

The fundamental requirements refer to the state of damage in the structure, attention being focussed in particular on the following three Limit States (LS): Near-Collapse (NC), Significant Damage (SD) and Damage Limitation (DL).

The definition of the LS of collapse is close to the actual collapse of the building, and corresponds to the fullest exploitation of the deformation capacity of the structural elements, while the definition of Significant Damage is roughly equivalent to what is called Ultimate LS (or no-collapse) in EC8 Part 1 dealing with the design of new buildings. The return periods (T_R) of the design action indicated as appropriate for the three LS's and for buildings of ordinary importance are 2475, 475 and 225 years, respectively.

The reason for the introduction of an additional, more severe, LS to be checked is easy to justify. The values of T_R applicable for the two classical verifications at SD and DL, on which the design of new structures is based, do not possess other support than the proven fact that their use leads to structures having an acceptable value of the total risk. New structures however, are designed using capacity design criteria and detailing rules for ductility in order to ensure that, in case of the occurrence of a seismic event more intense than the design one, the probability of collapse as function of the intensity I: $P_f(I)$ does not increase disproportionately. This behaviour corresponds to the thick line in Figure 1, which shows that $P_f(I)$ remains a smoothly increasing function of the intensity I beyond its design value at DS.

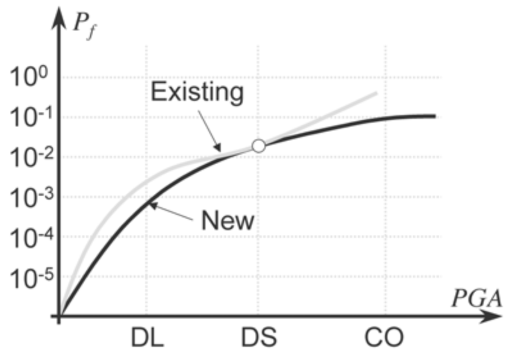

Figure 1. Probability of failure as function of the Peak Ground Acceleration (PGA) for a new and an existing building structure

The opposite may occur in case of an existing building, grey line in Figure 1. Even if such a building possesses the same value of $P_f(I)$ of the new one for the intensity at DS, as indicated in the figure, the absence of ductility provisions or other defects may well precipitate a brittle type of collapse for values of I only slightly larger. The total risk of the two buildings, as given by the integral:

$$P_f = \int_0^\infty P_f(i) \cdot f_I(i) di \tag{1}$$

where $f_I(i)di$ is the (annual) probability that the intensity falls in the generic interval {i-i+di} would then be clearly much larger for the old building. The implication is that the additional check at CO with a larger value of *I* required for existing buildings is just a means for ensuring that they possess the same degree of protection, not a larger one, as the newly designed ones.

4. KNOWLEDGE LEVELS AND CONFIDENCE FACTORS

The safety format common to all Eurocodes makes use of the well-established system of the probability-related partial factors, affecting the characteristic values of both actions and material properties. Extension of this format to cover the problem of assessing existing structures requires non-trivial adjustments: the solution provided in EC8/3 is plausible from a logical point of view but of course it cannot yet profit from the experience of its use in practice. In consideration of this lack of experience, the numerical values of many quantities entering in the proposed framework have been left to the National Authorities for final decision.

A prominent distinctive feature of the existing structures with respect to the new ones is the fact that their structural properties may be known, depending on the case, with widely different degrees of accuracy, ranging from very complete to very poor. Two sequential problems arise from this basic fact: how to define quantitatively the level of knowledge first and, second, how to account for the actual level of knowledge in the analytic assessment process.

In EC8/3, the global level of knowledge is defined by the combination of the knowledge available or achieved in the following factors: *geometry*, *details* and *materials* (see Table 1). With reference for example to reinforced concrete structures, *geometry* refers to the geometrical identification of the structural resisting system, *details* to amount and detailing of the reinforcement, and *materials* to the mechanical properties of the steel and concrete.

Knowledge on *geometry* is provided either by the original construction drawings and/or by survey, *details* and *materials* are known through inspection and testing, respectively, that can be of various degrees of exhaustiveness.

Three levels of knowledge are defined, denoted by KL1, KL2 and KL3 in increasing order of completeness and a factor, denoted 'confidence factor' (CF) is associated with each level (The recommended values are CF_{KL1}=1.35, CF_{KL2}=1.20 CF_{KL3}=1.00). The level of knowledge determines the allowable method of analysis, with KL1 permitting the use of linear methods only, while the associated values of the CF's play the role of partial factors to be used in the verification phase as explained in the following. Table 1 summarises the combinations of information, which define the knowledge levels: the terms 'visual', 'full', 'limited', 'extended' and 'comprehensive' are defined in the code together with corresponding recommended minimum amount of operations related to survey, inspection and testing.

Table 1. Knowledge levels and corresponding methods of analysis (LF: Lateral Force procedure, MRS: Modal Response Spectrum analysis) and confidence factors (CF)

Knowledge Level	Geometry	Details	Materials	Analysis	CF
KL1	From original outline construction drawings with sample visual survey *or* from full survey	Simulated design in accordance with relevant practice and from limited *in-situ* inspection	Default values in accordance with standards of the time of construction *and* from limited *in-situ* testing	LF-MRS	CF_{KL1}
KL2		From incomplete original detailed construction drawings with limited *in-situ* inspection *or* from extended in-situ inspection	From original design specifications with limited *in-situ* testing *or* from extended *in-situ* testing	All	CF_{KL2}
KL3		From original detailed construction drawings with limited *in-situ* inspection *or* from comprehensive *in-situ* inspection	From original test reports with limited *in-situ* testing *or* from comprehensive *in-situ* testing	All	CF_{KL3}

5. METHODS OF ANALYSIS

In general, the same four options as for the design of new buildings are possible, i.e., linear and non-linear methods, either static or dynamic. The use of linear methods, however, is subject to more restrictive conditions than in the case of new buildings. Actually, in this latter case modal analysis can always be applied, even if the building does not comply with the certain 'regularity' in plan and in elevation, while use of the static lateral force method is limited to buildings which are 'regular' in elevation and whose fundamental period does not exceed either 2 seconds or four times the corner period (T_C) of the appropriate design spectrum, i.e., the period that separates the constant acceleration from the constant velocity branches.

A major conceptual difference, however, exists in the way the linear approach is used in the case of existing structures. The full elastic design spectrum is applied, assuming a linear first mode, without previous factoring by a ductility-related force reduction factor (or behaviour factor, in the EC8 terminology), and the analysis is carried out assuming elastic behaviour of the structure.

The condition for the method to be applicable is that the ratio ρ_i between bending moment demand D_i and the corresponding capacity C_i is sufficiently uniform across all the primary resisting elements of the structure, i.e., the ratio ρ_{max}/ρ_{min} does not exceed a value in the range between 2 and 3. The assumption underlying the method is that if the structure moves into the inelastic range with an approximately uniform distribution of inelastic demands (expressed in terms of the D_i/C_i ratios), the result of the analysis is acceptably accurate for what concerns the displacements. It is an extension of the 'equal displacement' rule, approximately valid for a single-degree-of-freedom oscillator, to a whole building, hence the condition that geometry, stiffness and mass distribution be 'regular'. The method has been under elaboration internationally for almost a decade, has made its first appearance in a normative document in the USA [3], to be then adopted in a modified version by EC8/3, the modifications being quite substantial in the way the elements verifications are carried out, as it will be illustrated in the following.

When the linear methods of analysis are not applicable, the alternative more likely to be used in practice is the non-linear static one, given the much larger complexity of the non-linear dynamic. The pushover analysis is accepted by EC8/3 as a standard assessment method. The characterisation of the method as incorporated into EC8 is described below.

There are no conditions of applicability related to regularity, both in plan and in elevation: for the analysis of a non-regular building a spatial model is requested.

Two patterns of forces need to be applied, one 'uniform' (i.e., corresponding to a rigid translational mode), one 'modal' (i.e., corresponding to the inertia forces pattern from the first mode in the direction under consideration): element verifications are carried out for the most unfavourable result.

The so-called 'capacity curve', i.e., the curve relating the lateral resultant (base shear) with the centre point at the top of the building, must be evaluated for a maximum displacement equal to 150% of the 'target' displacement d_t.

The target displacement is obtained from the ordinate of the elastic displacement response spectrum at the effective period T^* of the building. This latter is evaluated using the stiffness of the bi-linearised capacity curve and the modal mass m^*.

The target displacement is assumed to be exactly equal to the elastic response displacement d_{te} if $T^* \geq T_C$, while for $T^* < T_C$ the following corrective expression is used:

$$d_t = d_{te} \cdot \frac{1}{q_u}\left(1 + (q_u - 1)\frac{T_C}{T^*}\right) \tag{2}$$

where q_u is the equivalent ductility, as given by the ratio of the elastic inertia force: $m^* \cdot S_a(T^*)$ and the yield capacity F_y^* from the bi-linearised curve.

The combination of the effects from the two horizontal components is made in a conventional way by first carrying out a full push-over analysis separately for each component, and then combining the respective action effects using both of the following combinations:

$$E_x + 0.30E_y; \quad E_y + 0.30E_x \tag{3}$$

where E_x and E_y represent the action effects due to the application of the seismic action along the chosen orthogonal axes *x* and *y*, respectively.

6. VERIFICATIONS

The verification phase is where the knowledge-related confidence factors enter into the reliability format of EC8/3. It is appropriate to distinguish the two cases of linear and non-linear methods of analysis. In case of a linear analysis, the action effects (demands, D_i) on 'ductile' and 'brittle' types of elements are evaluated differently, according to a 'capacity design' philosophy aiming to check that undesirable failure mechanisms are less likely to occur than those who are acceptable. In particular, the demands on ductile mechanisms consist of the chord rotations at the ends of columns and beams, as taken directly from the analysis. (The chord rotation is the angle between the tangent to the element axis at the element end and the chord connecting the end with the point of contraflexure). The demands in the 'brittle' mechanisms, on the contrary, are calculated by means of equilibrium conditions, considering the actions transmitted to them by the pertinent ductile components. These actions are those from the analysis, if the ductile element satisfies the condition $D_i/C_i \leq 1$ (i.e., if the element remains below yielding), while they equal the capacity of the element (evaluated with the mean values of the material properties), multiplied by the appropriate CF if $D_i/C_i > 1$.

From the capacity side, 'ductile' mechanisms are checked in terms of deformation, and the values of the capacities for the different LS's are obtained from the pertinent expressions using the mean values of the mechanical properties divided by the CF's. 'Brittle' mechanisms are checked in terms of strength, and the values of the capacities are obtained from the pertinent expressions using the mean values of the mechanical properties divided by both the usual partial factors and by the CF's.

If a non-linear method of analysis is used, instead of a linear one, the only difference is that the demands on both 'ductile' and 'brittle' mechanisms are directly those obtained from the analysis (to be carried out using mean values of the mechanical properties). The criteria for analysis and verification described above are summarised in Table 2.

Table 2. Values of material properties and criteria for analysis and safety verifications

		Linear Model (LM)		**Non-linear Model**	
		Demand	**Capacity**	**Demand**	**Capacity**
Type of element or mechanism (e/m)	Ductile	Acceptability of Linear Model (for checking of $\rho_i = D_i/C_i$ values)		From analysis. Use mean values of properties in model.	In terms of deformation. Use mean values of properties divided by CF.
		From analysis. Use mean values of properties in model.	In terms of strength. Use mean values of properties		
		Verifications (if LM accepted)			
		From analysis.	In terms of deformation. Use mean values of properties divided by CF.		
	Brittle	Verifications (if LM accepted)			
		If $\rho_i \leq 1$: from analysis. If $\rho_i > 1$: from equilibrium with strength of ductile e/m. Use mean values of properties multiplied by CF.	In terms of strength. Use mean values of properties divided by CF and by partial factor.		In terms of strength. Use mean values of properties divided by CF and by partial factor.

7. CAPACITY MODELS FOR ASSESSMENT AND STRENGTHENING

Knowledge of ultimate deformation and strength capacity of poorly designed reinforced concrete members is obviously an essential element of any credible procedure of seismic assessment. Detailed mechanical modelling of the behaviour of these elements, however, has proven to unfeasible due to the complex interactions taking place among the numerous relevant factors, such as: axial force ratio, shear span length, amount and spacing of transverse reinforcement, compression-to-tension reinforcement ratio, defective anchorage of transverse and longitudinal bars, cyclic loading history and, broadly speaking, poor quality of both materials and workmanship. The lack of mechanical models has forced researchers and code-committees to look for alternatives, in the form of empirical relationships based on experimental results. Large databases have been assembled independently in the USA and in Europe, and from one of these, [4], containing more than 1000 data from monotonic and cyclic tests on both old type and seismically detailed elements, the expressions in the EC8/3 have been derived. Specifically, EC8/3 provides formulas for the flexural deformation capacity and for the shear strength of beam-column elements and walls. The expressions are lengthy and reference is made to the document. Regarding flexural capacity, formulas are provided for the (average) total chord rotation capacity (the formula fits the data with a median of experimental to predicted values equal to 1.0 and a coefficient of variation of 47%), the plastic part of it alone, and the yield capacity. This latter is used for the verification of the DL state.

Regarding shear strength, the now widely diffused three-terms additive format has been adopted, i.e., one for the contribution of web reinforcement, one ductility-dependent contribution of concrete and a third one due to the beneficial effect of the axial compression. The expression for the shear strength has been derived using the same database as for the flexural capacity, completed by additional test results of specimens failing in shear after initial flexural yielding. The equation fits the data with a median of the ratio of experimental to calculated values equal to 1.0 and a coefficient of variation of 15%.

Following the part describing assessment capacity models, a section is present in EC8/3 where guidance is given on how to evaluate the capacity of strengthened elements. Three of the most common ways of strengthening are covered: concrete or steel jacketing and FRP plating and wrapping, this latter being treated in a more extensive and in some aspects original way.

Externally bonded FRP can enhance the behaviour of an existing element in one or more of the three following aspects: increase of shear strength, increase of flexural ductility at the member ends, prevention of lap-splice failure through added confinement. The effect of FRP wrapping on increasing flexural resistance and chord rotation capacity at yielding (not ultimate) is considered as negligible.

The contribution of shear strength due to FRP is assumed as additive to the strength of the existing elements, and this latter degrades with the plastic chord rotation ductility demand in the same way as for non strengthened elements.

The necessary enhancement of deformation capacity through FRP jackets is expressed by the ratio between the target curvature ductility and the available one. Depending on this ratio, a formula is given for the necessary amount of confinement pressure from the wrapping, which allows, in turn, to proportion the amount of FRP material for circular and rectangular cross sections.

To avoid slip of lap-splices, the lateral pressure through FRP jackets must equal the difference between the value required to prevent sliding of the longitudinal bars at yield, and that already provided by the hoop stress in stirrups at a strain of 1/1000.

8. CONCLUDING REMARKS

The outline description of the newly released Eurocode 8/3 on assessment and strengthening given in the paper is principally intended as information that the problem of existing sub-standard buildings in seismic areas is being recognised in its enormous importance, and that steps are being taken towards the mitigation of this risk. Of course a code is only a necessary pre-requisite for the solution of the problem, for which considerable time and resources will be required; to some extent, however, a code may also act as a catalyser for taking action, in its showing that at least from a technical point of view a rational way of facing the problem is available.

The present generation of codes for assessment and retrofit, to which EC8/3 represents the latest addition, should be regarded for what it is, i.e., a first generation, characterised by lack of fundamental knowledge in certain areas and, more importantly, by lack of feedback from widespread use in practical situations. Even if they were in a more advanced stage of maturity, however, it is acknowledged that codes of such a highly technical nature are not suited in regions where the built environment is essentially made of non-engineered construction, as it is the case in so many parts of the world. There, the solution passes through the availability of simpler, more prescriptive, codes of practice, tailored to the construction types specific of each region. Even these latter types of documents, however, will need to be structured, be it in embryonic form, according to a performance based conceptual framework, of which the higher level codes provide examples of the implementation and possible sources of inspiration.

REFERENCES

1. Cen, 2004. Eurocode 8: Design of structures for earthquake resistance Part 3: Assessment and retrofitting of buildings. June 2004, Doc. CEN/TC250/SC8/N388B. Comité Européen de Normalisation, Bruxelles.
2. ASCE, 2000. Prestandard for the Seismic Rehabilitation of Buildings. ASCE for Federal Emergency Management Agency (FEMA Report 356), Reston, VA, USA.
3. ATC, 1997. NEHRP Guidelines for the seismic rehabilitation of buildings. ATC for Federal Emergency Management Agency (FEMA Report 273), Washington, D.C., USA.
4. Panagiotakos, T. and Fardis, M.N., Deformation of RC members at yielding and ultimate, *ACI Structural J.,* No. 2, **98**(2001)135-148.