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PERFORMANCE OF R/C BUILDINGS DESIGNED FOR DIFFERENT DUCTILITY CLASSES

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ABSTRACT

The present paper reports on seismic non-linear analyses of two eight-storey R/C buildings designed according to the Eurocodes 2 and 8. Structural seismic performance and safety are evaluated. The nonlinear analyses were performed using a new beam element able to follow the stiffness drop due to cracking and yielding and the ductile cyclic behaviour. It has been validated against experimental results obtained at ELSA. A first attempt has been made to evaluate the structure safety through their failure probability and results are mainly used for comparison between cases. Results confirmed that EC8 design leads to quite comparable seismic performance of the structures, throughout different design options. Moreover, it has been recognized that seismic performance improves when ductility class passes from lower to higher levels.

KEYWORDS

Eurocode 8, R/C buildings, Flexibility based element, Hazard curves, Reliability, Ductility class, Irregularity.

INTRODUCTION

A series of numerical non-linear analyses of reinforced concrete structures subjected to earthquake loading has been programmed within the activities of the Pre-normative Research Programme in support of Eurocode 8 (PREC8) - Buildings. A general evaluation of the seismic performance of EC8 designed structures, considering different seismicities, three ductility classes and different design analysis methods, is sought. Several typologies (frame, shear wall, number of storeys, regular or irregular) were considered in order to cover a wide range of solutions. Hopefully the results could serve as background for possible improvement of code design methods and provisions.

The present study focus on the nonlinear analyses of two 8-storey buildings, performed with a new beam element specifically developed to follow the structure stiffness drop due to both cracking and yielding. Global section constitutive relations are used to control member end sections. The model has been validated by experimental results of a full scale R/C frame tested in ELSA and proved to be very efficient while reproducing quite well the results for various intensities of lateral loading.

Besides a large set of results, helpful to the qualitative assessment of the structural seismic performance, failure probabilities have been computed aiming at an explicit evaluation of the earthquake safety. For this purpose the Vulnerability Function Methodology has been adopted with the assumption of collapse mechanisms, the seismic action has been quantified through hazard curves for two seismicity levels and the ultimate capacity of sections has been probabilistically defined according to existing experimental studies.

STRUCTURES UNDER ANALYSIS

The analysed structures consist of two basic configurations (C2 and C6) of 8-storey R/C buildings shown in Fig. 1. Both configurations are symmetric in the horizontal directions XX and YY; C2 is regular in plan and in elevation but C6 exhibits irregularity in elevation due to the first storey columns higher than the remaining ones and to the absence of some columns cut-off at the first storey.

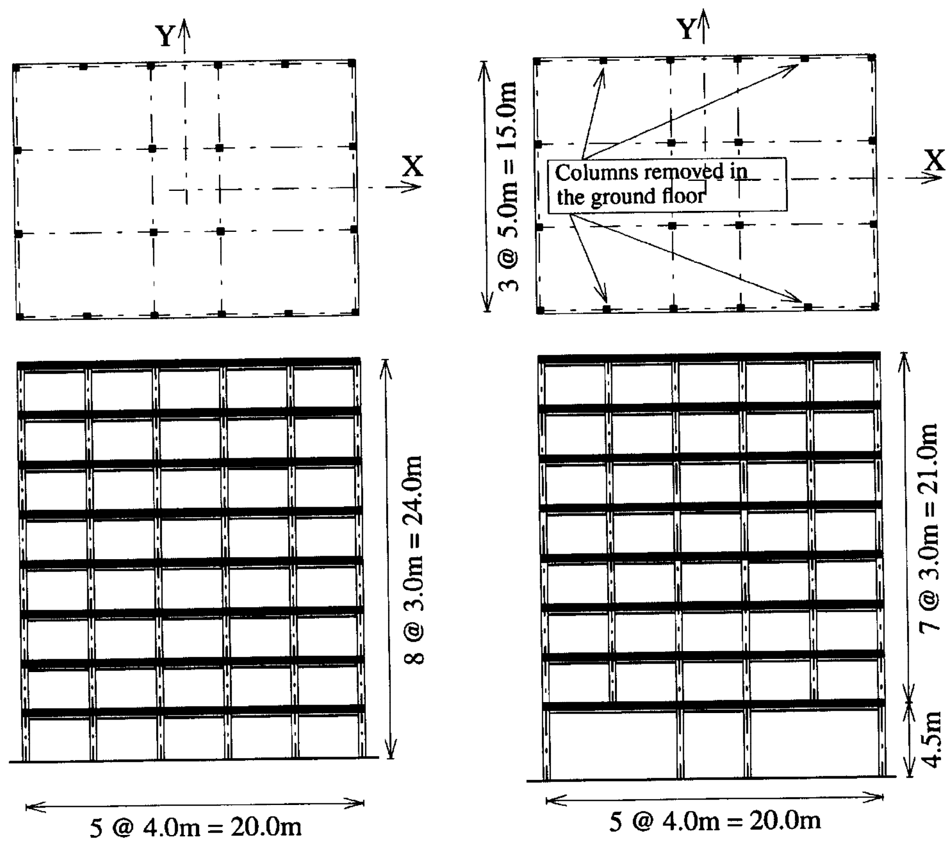

Fig. 1. Structural configurations: a) Regular structure C2 and b) Irregular structure C6

Each configuration has been designed according to the EC2 and EC8, for different ductility classes (L, M and H) and for two design accelerations (0.15g and 0.3g) (Fardis, 1994). Configuration 6 for the ductility class M and design acceleration 0.3g has been also designed using a simplified static analysis herein labelled by "Mst". The combination of these design assumptions leads to nine distinct trial cases, denoted in the following by short-names such as C2_15L, where 2 (or 6) stands for the configuration, 15 (or 30) refers to the design acceleration and L (or M, or H, or Mst) identifies the ductility class.

According to the data used in the design, vertical static loads (self-weight, finishing and live load) have been considered and combined with the seismic action using appropriate coefficients (Eurocode 8, 1994).

The seismic action has been simulated by a set of four artificial accelerograms. Such accelerograms of 10 s duration, were generated to fit the EC8 response spectrum and normalized to a unitary base acceleration. The accelerograms have been scaled for the design acceleration of each trial case. Then, each series has been multiplied by several intensities (1.0, 1.5 and 2.0), leading to more than 230 non-linear dynamic analyses that have been performed. Further details concerning structural discretization and assumptions, mass and damping characteristics can be found in (Arede *et al.*, 1995).

METHODS

The explicit evaluation of the earthquake safety for each design solution, requires the adoption of a high level measure of earthquake performance allowing the comparison between all structures and design solutions. The probability of failure, or in general, the probability of attaining a specific limit state, has been adopted in this study. It requires to go through the three key issues of the problem, namely: an adequate non-linear modelling of the structures, the probabilistic quantification of the seismic action (using the associated seismic risk curves) and the quantification of the ultimate capacity of the structural members.

Non-linear Modelling

The non-linear analyses have been performed using a global element model recently developed (Arede *et al.*, 1996) and implemented in the general purpose computer code CASTEM 2000 (CEA, 1990). The element is based on the flexibility formulation and makes no use of displacement shape functions. Instead, force shape functions are used as they have the advantage of being independent from the damage in the element.

Global section constitutive relations of moment-curvature type are specified for the extreme sections, based on material models including the effects of concrete confinement and steel hardening. Inside the element no pre-defined sections are considered, but use is made of the concept of "cracking" and "yielding" sections (where "cracking" and "yielding" moments can be found, respectively). These special internal sections, associated to a particular and simple behaviour, are continuously moving during the loading process, thus leading to an up-dated sub-division of the element in five zones for the case of linear distribution of moments. Therefore, at any load level it is possible to have a "picture" of both the cracking and the yielding progression along the element, as shown in Fig. 2-a. This allows to closely follow the structure stiffness drop and to reproduce the frequency modifications in dynamic analysis, as shown in two validation tests included in Fig. 2. Comparison with the experimental results obtained at ELSA (Negro *et al.*, 1994) confirms a quite good agreement; more details can be found in (Arede *et al.*, 1996).

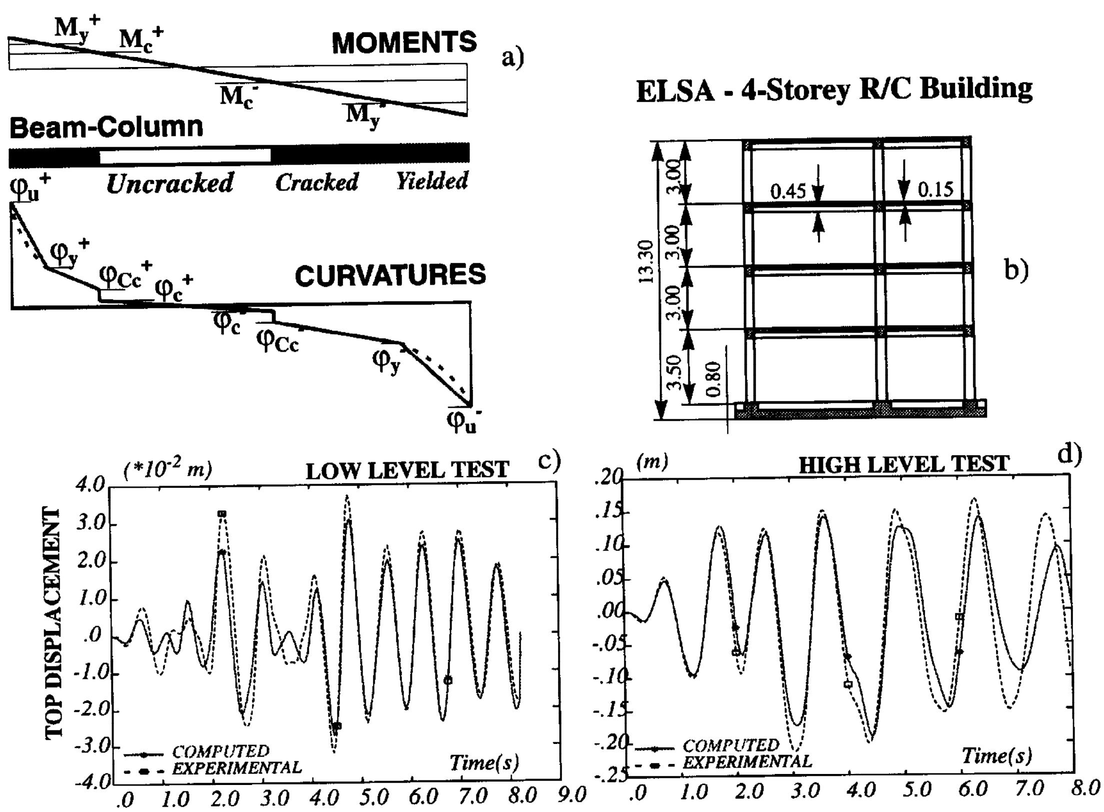

Fig. 2. a) Global beam-column element flexibility based model and application to a real structure b). Results comparison for c) low level and d) high level experimental tests.

The non-linear analyses led to the definition of vulnerability functions for several response variables. In particular, for the critical sections (plastic hinges), the vulnerability functions are built-up in terms of the chord rotation damage defined according to Park and Ang (Park and Ang, 1984). To compute the failure probability, analytical expressions for the vulnerability functions were obtained by curve fitting.

Probabilistic Quantification of the Seismic Action and the Section Ultimate Capacity

The probabilistic quantification of the seismic action is made by the hazard curves. These are obtained from seismic risk studies for a specific seismic region and so should be assumed valid only for that region. However, from this study, it is intended to draw generalized conclusions as EC8 will be the European unified common code for design in seismic regions. Therefore, three characteristic hazard curves shown in Fig. 3 were assumed, corresponding to Low, Medium and High seismicity zones, although only the two last ones have been used.

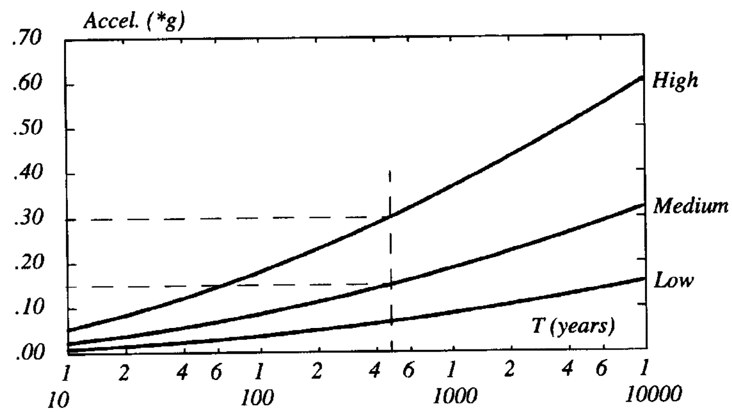

Fig. 3. Proposed hazard curves for Low, Medium and High seismicity

The probabilistic quantification of resistance in terms of damage for each plastic hinge has been assumed as log-normal distributed with mean 1.0 and COV 0.52 as suggested in (Park and Ang, 1984).

Bounds of Failure Probability

The Vulnerability Function Methodology (Duarte, 1991) has been followed here with a further improvement related to the assumption of collapse mechanisms (Costa, 1993). Basically, the local probability of failure (plastic hinge failure) is computed using the above referred damage vulnerability functions and the probabilistic seismic action to obtain the probability distribution of the action effects (damage); the convolution integral of this distribution with that of the damage capacity, yields the local probability of failure. Then, the bounds of the probability of failure associated to each mechanism are computed from the local ones and, from them, the final bounds of the structure probability of failure are obtained.

In the present study only the beam sidesway mechanism has been considered. Moreover, due to the difficulty in establishing a probabilistic dependency of the resistance between the mechanism hinges only the upper and lower bounds of the probability of failure can be computed. Assuming that the resistances are independent, then the events (failure/survival) of the various hinges are also independent and the probability of failure, corresponding to the Lower bound, is given by $P_L = 1 - \prod_i (1 - P_i)$, where P_i is the probability of failure for the plastic hinge i. On the other hand, for perfectly dependent resistances, the probability of failure of the system equals the probability that the most loaded hinge fails $P_U = max(P_i)$, which is the Upper bound of the probability of failure.

RESULTS

General Results

In addition to the dynamic analyses, each structure has been statically calculated for a monotonic increasing lateral load with inverted triangular distribution along the height. These push-over analyses led to shear-displacement diagrams used for a global overstrength estimation. In particular, from the base shear-top displacement diagrams, values of the global yielding threshold were obtained and compared with the design values. Their ratio (obtained value / design value) is one part of the involved overstrength and ranges between 1.10 and 1.35. The other contribution to the overstrength, depending on the deformation levels, arises from the global post-yielding stiffness which appears to be significant. Therefore, structures are likely to have an important overstrength due to the design procedure (capacity design rules and material safety factors) and to the high design forces evaluated on the basis of the uncracked natural period of the structure. Actually, these periods appear to increase significantly due the structure cracking, causing the structural frequency to move into a range of less powerful seismic input.

The used beam-column element, able to follow automatically the stiffness variations due to cracking and yielding, avoids the rough tuning of stiffness of the inner linear element usually assumed in the one component beam element models. In fact, as shown in Fig. 4, the development of those cracking zones over the structure is not uniform and depends on the loading intensity which would complicate the calculation of an equivalent stiffness for the inner linear element. Moreover, the extensive cracking, found in all cases, appears mainly in the beams for the design intensity and clearly extends to the columns for twice the design intensity. Also, the ratio (beam plastic hinge length / section depth) varies quite a lot with the earthquake input motion: values up to 0.30 for the design intensity and up to 0.50 for twice the design intensity can be found, thus confirming the advantage of the used model.

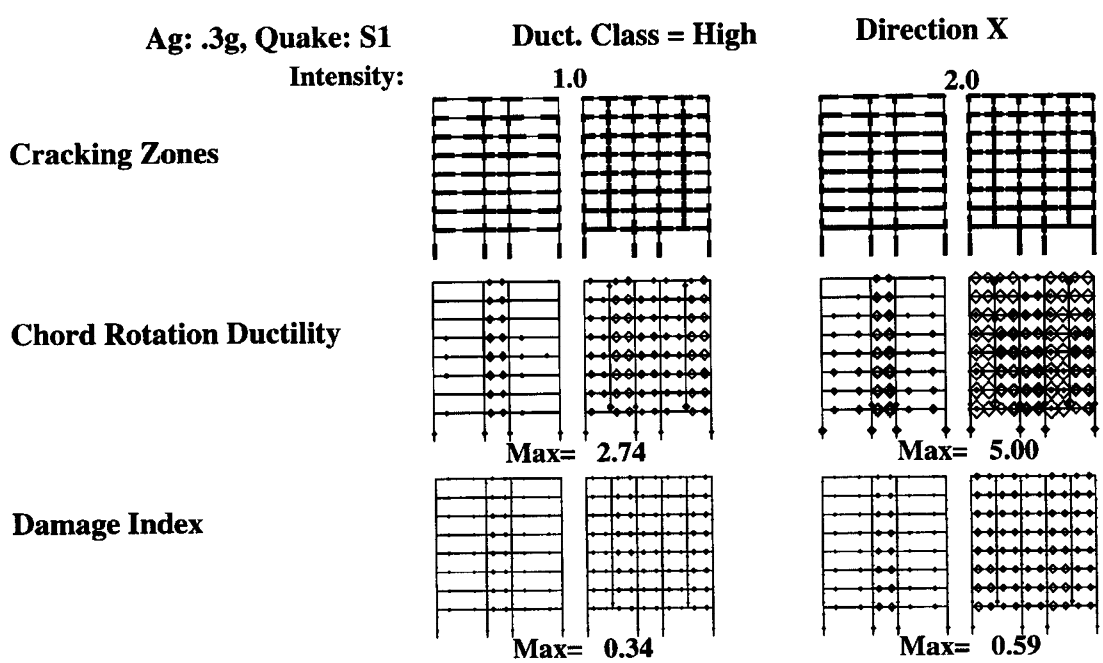

Fig. 4. Cracking, chord rotation ductility and damage index patterns (Configuration 6, Direction X, Design acceleration 0.3g, Ductility Class High, Earthquake S1)

Fig. 4 also shows the chord rotation ductility pattern, which indirectly identifies where yielding takes place because it is only plotted for ductility higher than 1. Generally, a quite uniform distribution of hinging is found for both earthquake intensities (1.0 and 2.0); moreover, for the same design acceleration (0.15g or 0.3g), the increase in Ductility Class (DC) leads to more uniform plastic hinge distribution. As expected, plastic hinging appears mainly in beams and in the base columns (ground floor); however, it also appears at the base of the cut-off columns of the irregular structure and in the columns at top floor level. Therefore, the subjacente beam sidesway dissipative mechanism effectively develops. However, the maximum ductility demands in these column plastic hinge zones are quite low for the design earthquake intensity, as only slight yielding is found: for twice the design earthquake intensity, the upper bounds of ductility demand are 2.0 and 3.0, respectively for the 0.15g and 0.3g design acceleration options.

Despite some local high ductility demands, a uniform distribution of damage is achieved (see Fig. 4), which confirms that appropriate detailing was provided to the structural elements. Furthermore, for higher DC the obtained damage values are lower and more uniform.

The obtained results also allowed for EC8 safety verifications, concerning namely the sensitivity coefficient to second order effects (Eurocode 8, 1994) and the inter-storey drift. The sensitivity coefficient limit is always largely verified; the inter-storey drift is verified for the 0.15g structures but in the case of high-seismicity ones (0.30g structures) the limit is generally overtaken by a factor of 1.5, which is due to the cracking inside the elements, taken into account in the non-linear analysis but not in the design process.

In order to compare the analysed cases, a set of global averaged quantities has been plotted. For example, the global damage index, defined as an energy weighted value of the element damage indices, is obtained for each earthquake and the resulting average is plotted in Fig. 5 for every structure and for the different seismic intensities. In spite of high local damage values found in some plastic hinges, it can be seen that the structural damage presents acceptable values ranging between 0.10 and 0.20 for the design intensity and between 0.25 and 0.50 for the double intensity. Comparison between the different cases are included in the next section.

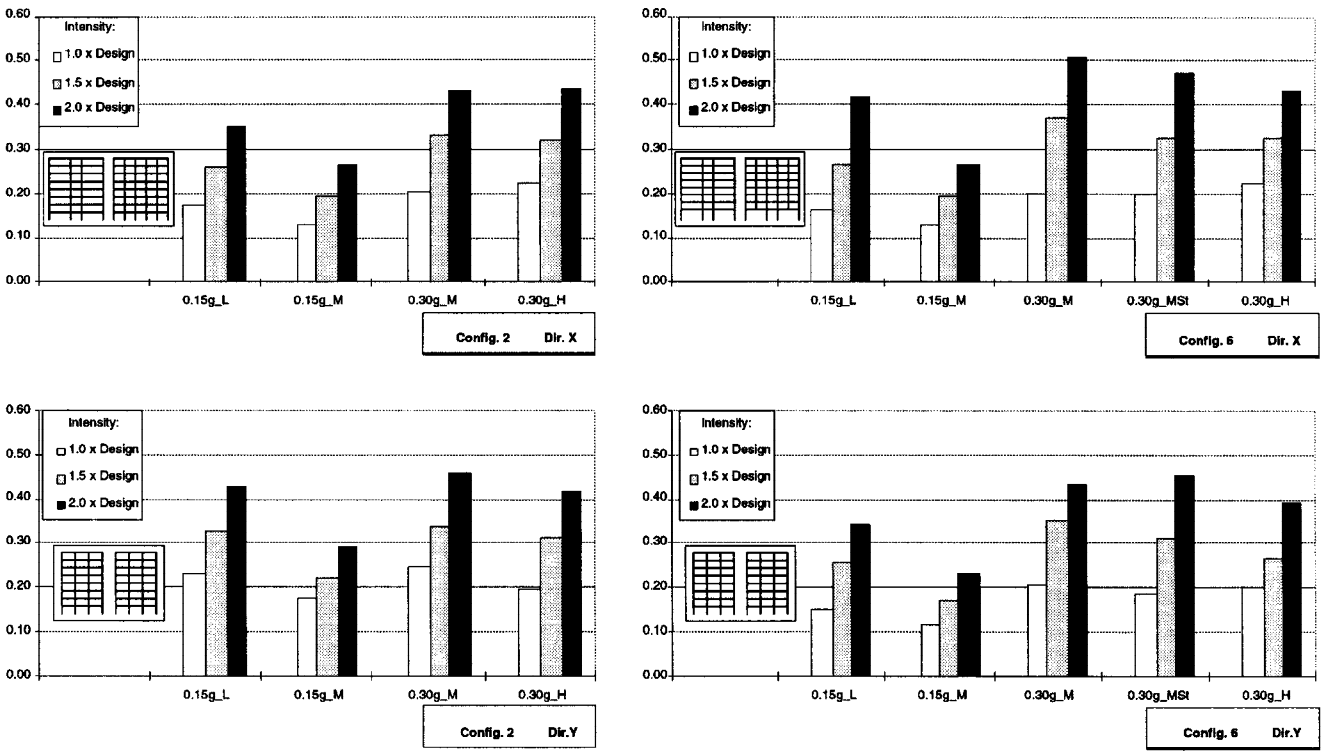

Fig. 5. Global average damage for all cases

Probabilities of Failure and Comparison of Design Cases

Following the methodology and assumptions above mentioned, the upper and the lower bounds of the structure probability of failure were computed for all cases and a comparative representation of these values is shown in Fig. 6.

Ductility Class appears to play an important role in the probability of failure, as higher DC's lead to lower probabilities of failure. This fact agrees with the results in Fig. 5, where lower damage values are found for increasing ductility class.

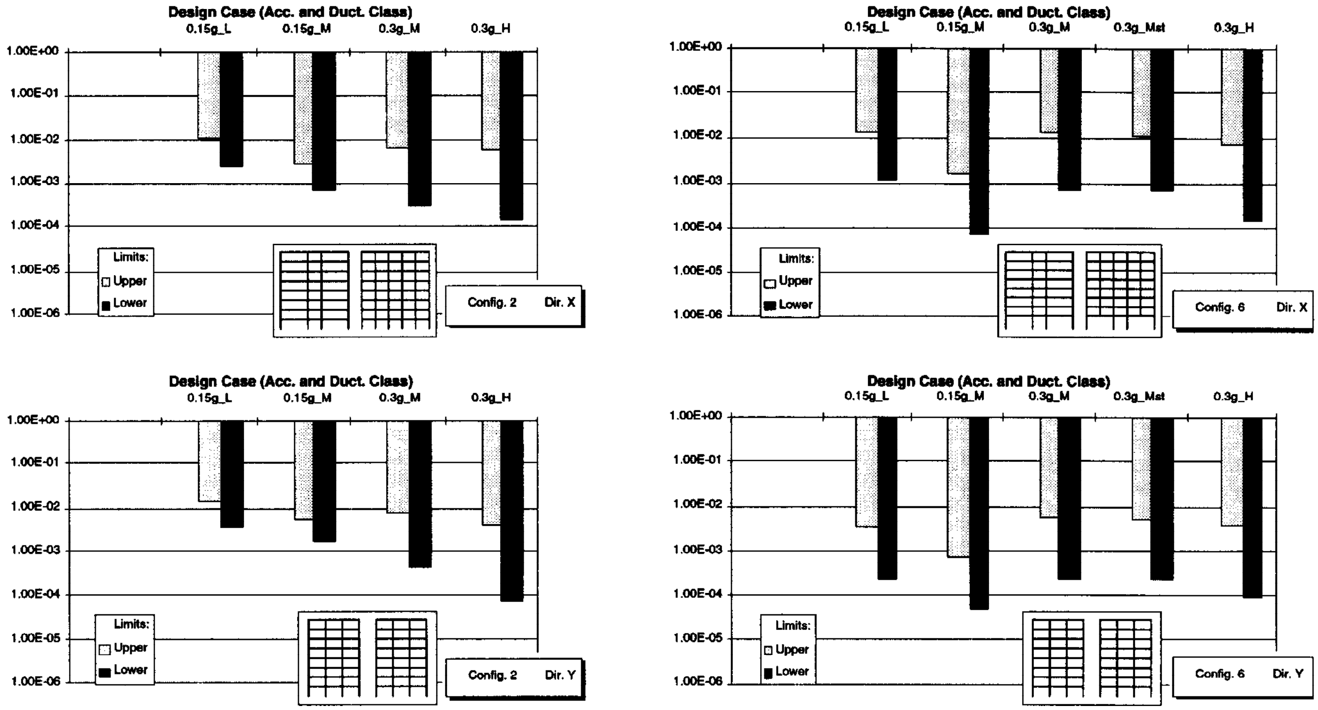

Fig. 6. Lower and upper bounds of probability of failure

Additionally, it should be underlined that, mainly for the lower DC's, those probabilities of failure will increase if other mechanisms (e.g. storey mechanism) are taken into account. Due to the capacity design rules, which are less severe for lower DC's, this storey mechanism tends to develop earlier in the lower DC structures and, consequently, their probability of failure will increase even more compared to the higher DC structures. This is, in the authors opinion, an important result because some recent publications (e.g. Faella and Realfonzo, 1994) based purely on damage analysis and basic costs (amount of steel and workmanship) tend to conclude that no clear advantage arises from designing for higher DC. Moreover, the quantification of structural members capacity appears to be quite crude and does not take into account its ductility class (detailing, control, etc.). Thus, it is thought that further improvement in the ultimate capacity quantification would lead to lower COV's for the higher DC's and, consequently, to a reduction of their probability of failure.

Concerning the "seismicity" aspect (0.15g versus 0.30g design acceleration, for the ductility class M), the results show higher damage values for increased seismicity and, consequently, higher probabilities of failure. To interpret this fact, one must be aware that the design of 0.15g structures has been more controlled by non-seismic actions, therefore having higher overstrength. This is even more clear for the DC M than for the DC L, which justifies the lower damage values obtained. Thus, the difference in terms of probability of failure between the two seismicities does not necessary implies less safety for structures in high seismic regions; instead it appears to be related to the different predominance of seismic action effects over the remaining ones.

The different design methods used in the 0.30g_M and 0.30g_Mst cases did not reveal significant differences neither in terms of the global damage nor in the probabilities of failure. Thus, design of these (analysed) structures considering a static equivalent analysis did not lead to less safe solutions.

Finally, there should be noticed the very high values (10e-2) for the upper bounds of the probability of failure and also the lower bounds (10e-3 to 10e-4), quite high compared to the values usually adopted for other loading types (~10e-5). This requires an exhaustive confirmation of both the methodology and the assumptions herein made. In particular, the criterion for the definition of the ultimate deformation capacity should be revised, because it has been found to be responsible for some high damage values obtained for Low and Medium ductility structures.

CONCLUSIONS

Two 8-storey buildings designed according to Eurocode 8 have been analysed in order to evaluate their seismic performance and safety. Aspects related to the design earthquake intensity, ductility class, irregularity and design analysis methods were to be investigated. The general approach and the type of results to be obtained have been previously defined within the PREC8-Buildings research group.

Two innovative components were included in this study: the modelling has been done with a new beam element able to trace out continuously the cracking and yielding of the structural elements, and an attempt was made to evaluate quantitatively the safety of the structures through their probability of failure.

In spite of the differences on the design options related to ductility class, irregularity and structural layout, the results confirmed that the EC8 design leads to quite comparable seismic performance of the structures which improves when passing from lower to higher ductility classes. However, the values found for the probability of failure of the structures under analysis are quite high compared to the ones adopted for other loading types. Therefore, this requires a detailed analysis of both the used methods and the considered assumptions, in order to clarify the causes of such high probabilities of failure.

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