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ON THE SEISMIC BEHAVIOR OF RC FRAMES DESIGNED ACCORDING TO EUROCODE 8

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SUMMARY

The present paper addresses the seismic non-linear analysis of two eight-storey RC buildings designed according to the Eurocodes 2 and 8. Structural seismic performance and safety are evaluated. The nonlinear analyses were performed using a flexibility based beam-column element able to follow the stiffness modifications due to cracking and yielding and the ductile cyclic behaviour. Estimates of failure probability were obtained aiming at safety assessment, mainly for comparative purposes between trial cases. Results have shown quite good performance of these EC8 designed structures, with significant overstrength and low-moderate ductility demands. Overall response indicators (drifts and average damage) showed large margins to failure even for seismic actions of twice the design intensity. Lower average damage was found as the ductility class was increased and, for a given peak ground design acceleration, structures were found identically safe regardless of their ductility class.

INTRODUCTION

The forthcoming approval of Eurocode 8 (EC8) as the European standard for seismic design of structures has motivated several research projects, focusing on the safety level and behaviour assessment of structures designed in fulfilment of EC8 rules. Among those projects, the so called "Pre-normative Research Programme in Support of Eurocode 8" (PREC8) was accomplished and, within its specific topic "Reinforced concrete frames and walls", several building structures were designed according to EC8 and numerically analysed to find out the influence of some EC8 design parameters and options on the performance of RC structures under seismic conditions. Accordingly, the present paper discusses the results of the numerical study of eight storey building structures associated with two basic configurations (regular/irregular), two design accelerations and different ductility classes. This study mostly follows the steps of a preliminary analysis presented elsewhere [Arede *et al.* 1996] but updated results and findings are presented herein according to some reviewed issues and more detailed analysis reported at length in [Arede 1997].

The structural behaviour is simulated using a flexibility based global element (beam-column) model such that the non-linearity spread inside structural elements can be adequately taken into account. Non-linear dynamic analyses of each trial case were carried out for increasing intensities of four artificial accelerograms and additional push-over analyses were performed aiming at global overstrength quantification.

The wide variety of obtained results is discussed herein, mostly for comparison between trial cases. Particularly, global overstrength factors are quantified in terms of total base-shear, the spread of seismic effects is analysed through cracking patterns and spatial distributions of ductility demands and damage, and global response parameters are obtained, namely the total drift, the inter-storey drift and the damage index. Additionally, a brief insight

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on the structural safety assessment is given through the quantification of probabilities of failure mainly for comparative purposes between trial cases rather than an absolute safety evaluation.

THE SEISMIC ANALYSIS PROGRAMME - TRIAL CASES AND METHODOLOGIES

Structure layout and actions

As stated in a preliminary study [Arede *et al.* 1996], two basic configurations (C2 and C6) of eight storey RC buildings were considered (Fig. 1). Structures are symmetric in both horizontal directions (XX and YY) and, while configuration 2 is regular in plan and in elevation, configuration 6 exhibits two sources of irregularity in elevation: *i)* the first storey is softer than the remaining ones, due to its greater height and to some columns cut off below that storey; *ii)* the existence of these cut-off columns, supported by medium-long span beams.

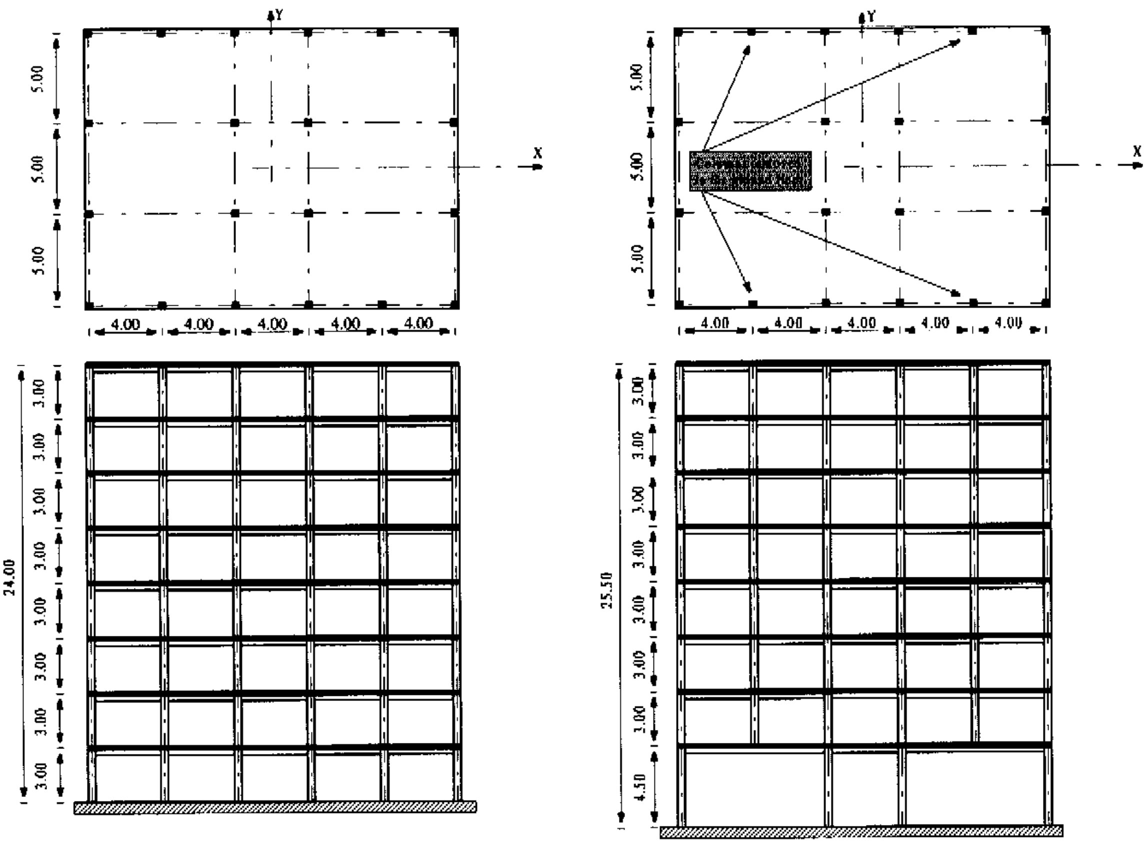


Fig. 1. Structural configurations: a) Regular structure C2 and b) Irregular structure C6

Each configuration was designed according to EC2 and EC8, for different ductility classes (L, M and H) and for two design accelerations (0.15g and 0.3g). Furthermore, the case of configuration 6 for ductility class M and design acceleration 0.3g was also designed using the simplified static analysis of paragraph 3.3.2 of EC8, Part 1.2, herein labelled as "Mst". The combination of these design assumptions (ductility class, design acceleration and analysis method) leads to the nine distinct trial cases listed in Table 1, which also includes the design behaviour factors (*q* factors) and the reference names (labels) identifying each trial case in the following paragraphs.

All the relevant data obtained from the design process is extensively described in [Fardis 1994], namely concerning section design forces, cross-section dimensions, reinforcement details (longitudinal and transversal) and adopted slab widths contributing for beam strength and stiffness. According to this data, vertical static loads (self-weight, finishing and live load) were quantified and combined using appropriate coefficients prescribed in EC8. These loads were applied prior to any seismic input in order to start the seismic analysis with the effects of dead and live loads already taken into account (namely, in what concerns stiffness).

The seismic action was simulated by a set of four artificial accelerograms that were provided to all the participant teams in the PREC8 project. The accelerograms of 10 s duration were generated to fit the EC8 response spectrum for soil type B and 5% damping. Each accelerogram was normalized to a unitary base acceleration, then scaled for the design acceleration corresponding to each of the nine trial cases listed in Table 1 and finally factored by the intensities 1.0, 1.5 and 2.0

Table 1.Trial cases, design behaviour factors and earthquake intensities

Configuration 2 (Regular)				Configuration 6 (Irregular)			
Design Acc.	**Duct. Class**	**Ref. Name**	***q* factor**	**Design Acc.**	**Duct. Class**	**Ref. Name**	***q* factor**
0.15g	L	2_15L	2.5	0.15g	L	6_15L	2.0
	M	2_15M	3.75		M	6_15M	3.0
0.30g	M	2_30M	3.75	0.30g	M	6_30M	3.0
					Mst	6_30Mst	3.0
	H	2_30H	5.0		H	6_30H	4.0

Structure modelling

Each trial case structure was discretized for independent plane frame analysis in the XX and YY directions. By recourse to symmetry properties, only the association of two distinct frames (one internal and other external) was considered in each direction of analysis with double values of stiffness, strength, vertical static load and mass.

Equal horizontal displacements were imposed to all the nodes at the same floor in order to accomplish the assumption of rigid floor diaphragm. Each structural member was discretized by only one flexibility based element using a global element model [Arede 1997] in which, rather than using displacement shape functions, only force shape functions are used as they have the advantage of being independent of the damage in the element. The non-linear behaviour is controlled by means of global section constitutive relations of moment-curvature specified for the element end-sections; for such relations, material models were considered taking into account the effects of concrete confinement and of steel strain hardening. Inside the element, "cracking" and "yielding" sections are continuously activated such that, along with the element end sections and a mid-span section, different behaviour zones (yielded, cracked and uncracked) can be sucessively updated. Therefore, both cracking and yielding spread inside the element can be taken into account at any load stage, which allows to closely follow the structure stiffness variations and the inherent frequency modifications in dynamic analysis. Calculations were performed using the general purpose computer code CASTEM 2000 [CEA 1990] where the above referred model was implemented; details on the flexibility global element model and its validation against experimental evidence can be found in [Arede 1997].

Response variables and quantification of failure probability

Results obtained from non-linear analysis of the above referred trial cases were expressed in terms of common response variables such as total base shear, top displacement (or total drift), inter-storey drift (relative to storey height), member displacements, ductility demands and damage. Chord rotations at member end sections were adopted to quantify both ductility and damage, the later being defined according to the well known Park and Ang proposal [Park and Ang 1984]. For yielding and ultimate chord-rotation computation, criteria and procedures were adopted as detailed in [Arede 1997].

The non-linear analyses carried out for several intensities of the seismic action enabled the definition of vulnerability functions for the response variables of interest. Particularly, aiming at failure probability quantification, damage vulnerability functions were defined for all critical sections (plastic hinges) by curve fitting to the numerically obtained results of damage values. The vulnerability function is indeed a key issue for failure probability calculation, through which the seismic action intensity is related to the action effects, the damage index in the present case. The seismic action is defined by hazard curves and then conveyed into the space of action effects, such that the probabilistic description of damage in each plastic hinge can be obtained. In order to perform the convolution integral leading to the local probability of failure [Campos Costa 1993], the damage capacity (defined as the damage threshold for which a given plastic hinge fails) has to be described in probabilistic terms.

The probabilistic quantification of the seismic intensity was made by recourse to broad studies by Campos Costa and Pinto [Campos Costa and Pinto 1997] attempting to characterize the European seismic hazard scenarios, based on a large seismic database catalogue. The seismic hazard is categorized into five different classes of increasing severity ranging from Very Low to High; for each seismicity class, hazard estimates were obtained relating a series of the return periods (*T*) with the expected peak ground accelerations and Weibull distributions were adopted to represent the hazard curves. Two hazard curves were adopted herein as shown in Fig. 2, such that peak ground accelerations of 0.30g and 0.15g are obtained for the return period of 475 years (the reference one in the EC8). This corresponds to the direct adoption of the High seismicity class (for approximately 0.30g) and of scaled hazard from the Moderate High class to match the 0.15g acceleration at T=475 years.

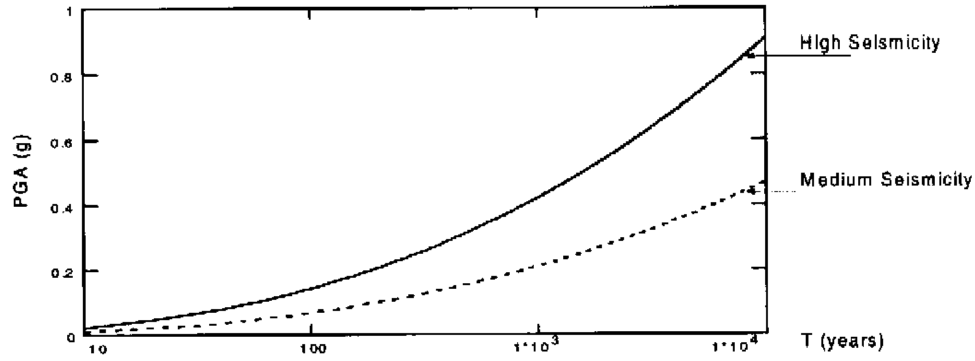


Fig. 2. Medium and High seismicity hazard curves

The damage capacity for each plastic hinge was defined in a probabilistic sense by a log-normal distribution with mean 1.0 and COV 0.5 according to the results of statistical analyses carried-out by Park *et al.* (1984) on a large set of experimental results on reinforced concrete elements tested up to failure.

Once the local failure probabilities of the relevant plastic hinges are obtained, the global probability of failure of the structure can be obtained by combination of local failure probabilities. However, in practice it appears rather difficult to establish the combination of local failure modes since it depends on several aspects such as the type of loading (directly influencing the failure mechanisms) and the correlation between action effects and between damage capacities [Pinto 1997]. Therefore, numerical approximations are often used, as for example the reliability bounds which consist of estimates of lower and upper limits for the global probability of failure, rather than its "exact" value. In the present study, the so-called Cornell bounds [Madsen *et al.* 1986] were adopted, although their application for statically indeterminate structures is not strictly valid since they were derived for series (weakest-link) systems; indeed, it can be seen [Arede 1997] that the use of such bounds for redundant structures tends to overestimate the global probability of failure. Thus, a simple methodology was adopted in which the design mechanism is assumed to control the failure mode (a beam sidesway mechanism) and, despite the above mentioned overestimation trend, the Cornell bounds were considered for the event set consisting of plastic hinge failure in all beams and in the base end-zones of ground floor columns; details on the adopted strategy can be found in [Arede 1997].

RESULTS ANALYSIS AND DISCUSSION

Structural Strength

The structural strength engaged during seismic response was estimated in global terms by recourse to the total peak base shear force ($R_{\text{seism}}^{\max}$). The ratio of such force to the corresponding design value (R_d) provides a measure of the global overstrength ($\psi_m = R_{\text{seism}}^{\max}/R_d$) involving two distinct types of contributions, namely those related to design aspects (material safety factors, minimum reinforcement requirements and bar rounding up, capacity design and gravity loads) and those related to the deformation level reached during the response (post-yielding hardening at the section level, strength mechanisms actually activated and the system effect arising from the non-simultaneous yielding of plastic hinges assumed in the design). Values of ψ_m up to about 1.6 and 2.1 were found, respectively, for the *design intensity* and for *twice the design intensity*, showing an important reserve of strength of those structures.

In order to check the influence of the above referred contribution to the global overstrength, two other factors were computed [Arede 1997]: *i)* the overstrength factor at yielding $\psi_o = R_y/R_d$, where R_y is the global yielding base shear force approximately estimated in base shear-top displacement curves obtained from push-over analysis; *ii)* the global hardening factor defined by the ratio $\psi_h = R_{seism}^{max}/R_y$. For the overstrength at yielding ψ_o values range from about 1.1 to 1.5, with a clear trend to exceed 1.3, confirming the significant strength reserve for most cases. The factor ψ_h, which is actually a measure of the structural hardening at the global level, was found to reach important values as high as 1.5 for *twice the design intensity.*

Spread of Seismic Effects

The use of the flexibility based beam-column element model with moving control sections allowed to define the cracked zones in structural members. Quite extensive cracking was found for the *design intensity,* mostly in beams and also, to some extent, in the internal columns of the irregular frame (the external one in configuration 6). For *twice the design intensity,* cracking develops further (particularly in the columns) but it is clear that the most significant stiffness drop due to cracking takes place for the design intensity.

Patterns of chord rotation ductility demands were also obtained (though not included herein), showing that for 0.15g designed structures very low demands are found for the *design intensity* as a consequence of the important strength reserve. For *twice the design intensity* (still for 0.15g designed structures), plastic hinge formation tends to spread all over the structure. Except for the cut-off columns (more slender than the remaining ones), plastic hinging is found only in beams and at the end-zones of ground floor columns, thus agreeing with the dissipation mechanism foreseen in the design. Structures designed for 0.30g exhibit larger ductility demands in accordance with the clear onset of yielding for the *design intensity,* although somewhat low in view of the adopted behaviour factor. Comparing to the 0.15g structures, the 6_30M and 6_30H cases show a more uniform spread of plastic hinging, which, for *twice the design intensity,* engages almost all the critical zones of the beam sidesway mechanism underlying the design philosophy.

As for the rotation ductility demands, damage patterns were also plotted as shown in Fig. 3 for the internal and external frames of configuration 6, in direction X, under earthquake S1 for *twice the design intensity.* It can be observed that the increase of design acceleration leads to larger damage values and, on the other hand, for higher ductility classes (with the same design seismic input) lower damage is obtained as a result of better design detailing, particularly concerning transversal reinforcement which enhances the section (and member) ultimate ductile capacity. Despite some significant values in the cut-off columns, damage in configuration 6 (Fig. 3) is better distributed in the external frame because beam spans are uniform. By contrast, the large difference of span lengths in the internal frame cause the damage to concentrate in the shorter central span; a similar result is actually obtained in the internal frame of configuration 2. From the obtained results, the damage distribution appears more affected by the non-uniformity of beam spans rather than the presence of cut-off columns.

Overall Response

Global response parameters (drifts and damage) were computed for each structure taking average estimates of peak values obtained from the response to each of the four accelerograms; maxima over the entire structure are retained for comparison between trial cases.

The maximum total drift (ratio of top displacement to the structure height) did not show significant and systematic differences between distinct ductility classes for the same design acceleration as expected according to the "equal displacement principle" (Paulay and Priestley, 1992); nevertheless, a certain trend can be observed in some 0.15g designed cases for larger drifts when the ductility class is increased. In average, configuration 2 leads to drifts higher than configuration 6, as a consequence of the slender columns of the former; however, even for *twice the design intensity,* low drift values are obtained (below 1.6%) when compared to values at near-failure stage. Indeed, as a reference, one can look back at the pseudo-dynamic and cyclic tests carried out on a full-scale four storey building (ductility class high) in the ELSA laboratory at Ispra (Italy) where a total drift of 4.8% was reached for the final stage when failure was considered imminent [Arede 1997].

The obtained results allowed also for EC8 safety verifications [Eurocode 8, 1994], concerning namely the sensitivity coefficient to second order effects and the inter-storey drift (a serviceability limit state verification for damage control). The sensitivity coefficient limit is always largely verified; the inter-storey drift is verified for the 0.15g designed structures but for the 0.30g cases it is somewhat exceed, particularly for configuration 2 [Arede

1997]. This is due to the stiffness drop caused by the generalized cracking spread over the whole structure, taken into account in the non-linear analysis but not in the design, and confirms the expectable non-conservatism of displacement estimates based on uncracked behaviour as suggested in the paragraph 3.1 of part 1-2 of EC8.

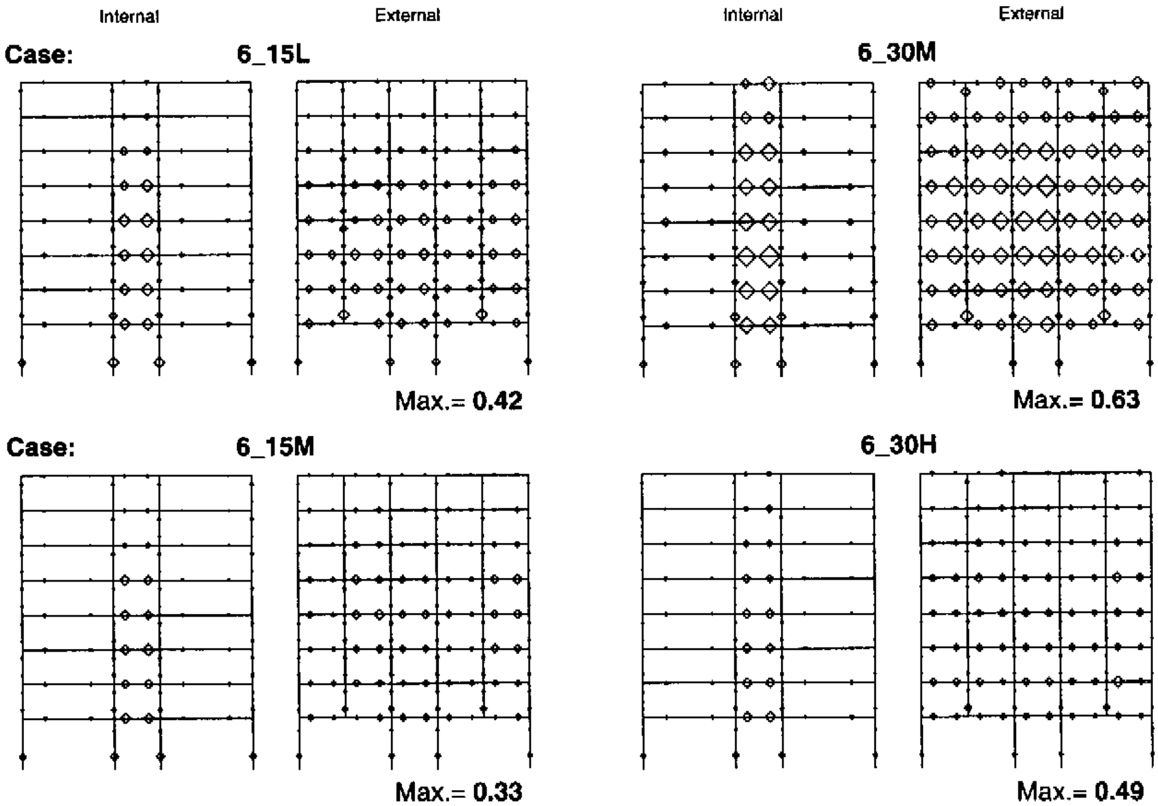
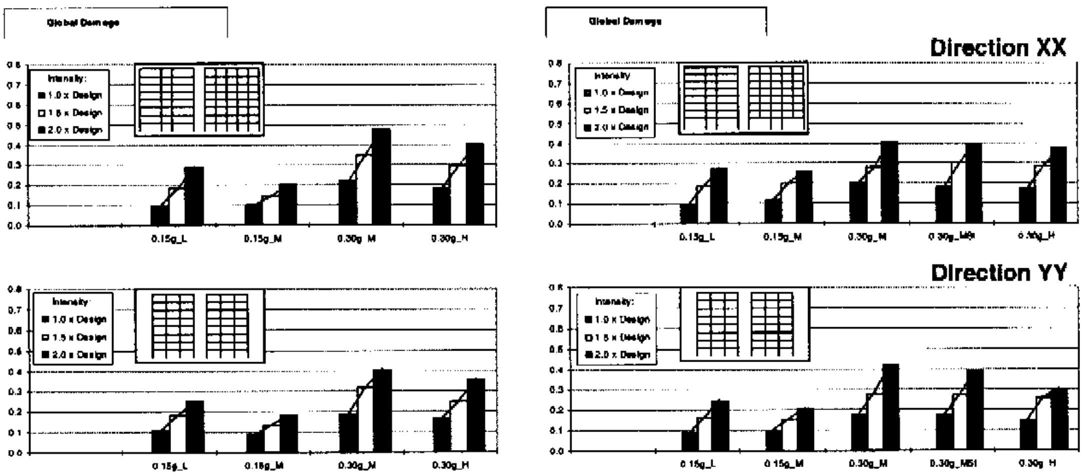


Fig. 3. Damage index patterns (Configuration 6, Direction X, Earthquake S1 for intensity 2.0)

The global damage index, defined as an energy weighted value of the element damage indices, was obtained for each accelerogram of the earthquake set and the resulting average, for every structure and for the different seismic intensities, is plotted in Fig. 4. From these results it is apparent the trend for lower damage when the ductility class is increased, particularly for the highest intensity of the design seismic action. Typically, the 0.15g structures exhibit low average damage (around 0.1 for the *design intensity* and between 0.2 and 0.3 for *twice the design intensity*) which is mainly a consequence of their significant overstrength and demand reduction due to stiffness and frequency decrease. For the 0.30g structures the average damage slightly exceeds 0.45 in the DCM case of configuration 2 (direction XX), which appears to be the most critical. Such average damage is quite acceptable, particularly because it refers to twice the design intensity. Thus, despite some locally higher damage, this result highlights the significant reserve of structural capacity to withstand earthquake loads beyond the design ones while keeping "its structural integrity and a residual load bearing capacity after the seismic event" [Eurocode 8, 1994]. Further details and comparisons between trial cases are fully addressed in [Arede 1997].

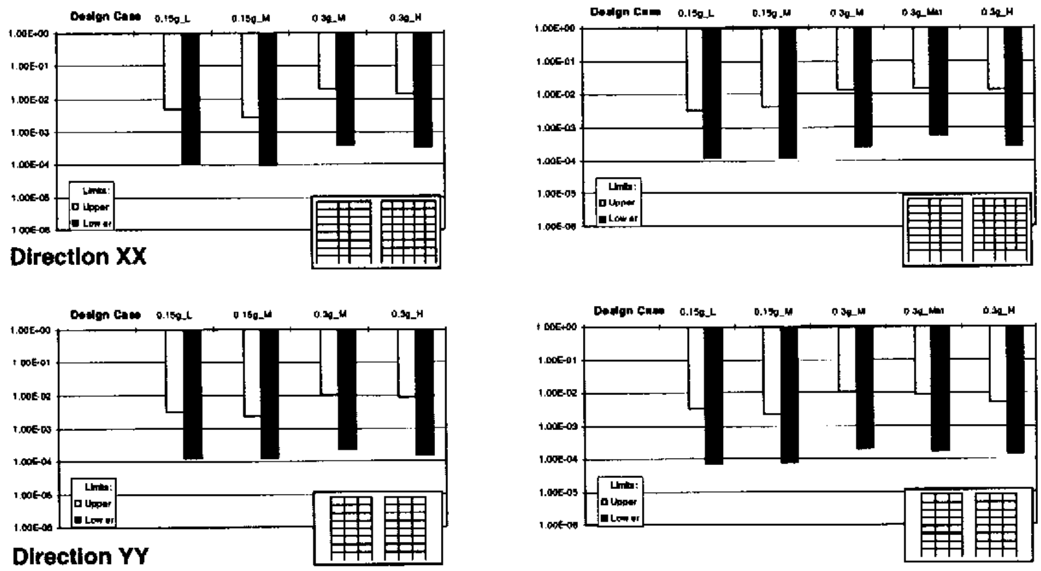
Comparative Analysis of Failure Probabilities

The above referred methodology was applied for all trial cases under analysis and the obtained upper and lower bounds of global annual probability of failure are plotted in the logarithmic scale charts of Fig. 5. The obtained values are considerably higher than what should be expected; for comparison purposes, Paulay and Priestley [Paulay and Priestley 1992] suggest annual failure probabilities around $2x10^{-4}$ as appropriate for the survival limit state of office buildings. Such high values are deemed to be related to the way the ultimate chord rotation is obtained for damage index calculation and to the assumptions concerning the combination of local failures modes; therefore, these topics shall be further investigated.



a) Configuration 2 b) Configuration 6

Fig. 4. Global average damage for all cases



a) Configuration 2 b) Configuration 6

Fig. 5. Lower and upper bounds of probability of failure

The 0.15g designed structures show a trend for probabilities of failure (10^{-4} to 10^{-3}) lower than the 0.30g cases (10^{-3} to 10^{-2}), particularly in the direction XX, which is coherent with the less damage found in 0.15g structures. For 0.30g structures in the direction XX (where irregularities do exist due to both the cut-off columns and the large span beams), the probability of failure tends to be higher than in the direction YY where lateral loads are resisted by regular frames; this means that irregularities contribute, as expected, for less safe solutions. Additionally, it is noteworthy that configuration 2, considered regular despite having adjacent beams with so different spans, shows probabilities of failure higher than the assumed irregular structure. This sustains the adequacy of the 80% reduction of the q-factor to account for irregularity in configuration 6 and suggests that, besides taking into account the cut-off column irregularity, this reduction also contributes to soften the negative effects of the significant contrast of beam spans whose influence is quite apparent in configuration 2. The 0.30g_Mst and 0.30g_M cases (configuration 6) show quite similar probabilities of failure, which confirms the adequacy of the simplified static analysis procedure allowed in EC8.

Last but not the least, an interesting and important result is that, for a given design acceleration, the ductility class does not seem to affect the structure reliability, which is a fundamental issue from the design code standpoint.

CONCLUSIONS

In the framework of the PREC8 research programme, two configurations of 8-storey buildings designed according to EC8 were numerically analysed for seismic performance and safety assessment. Several design aspects such as the earthquake intensity, ductility class, irregularity and design analysis methods were investigated.

Structures were modelled for planar analysis with a flexibility based beam-column element model suitable to trace out cracking and yielding spread in structural elements. The seismic response analysis was carried out by recourse to typical (local and global) response variables and an additional comparative study of failure probabilities elicited some comments on the safety issue.

Generally, the analysed structures showed an important strength reserve, confirmed by both the seismic analysis results and additional pushover analysis. Extensive cracking was found along structural members, leading to significant stiffness drop. These two factors were found responsible for low-moderate ductility demands. Also for the maximum seismic intensity considered in the analysis, low drift values were obtained (particularly when compared to near-failure values), indicating a large margin to failure; however, in some cases (corresponding to the highest design intensity considered), the EC8 limit for interstorey drift was somewhat exceeded. Estimation of displacement demands in this force-based design should therefore be reviewed.

Typically, lower average damage was obtained when passing from lower to higher ductility classes (while keeping the same design acceleration) as a result of the more stringent design provisions for ductility enhancement. However, structures designed for a give peak ground acceleration, were found identically safe regardless of the ductility class they belong to. Furthermore, both the behaviour factor reduction to account for irregularity and the simplified static analysis design method allowed in EC8 appeared adequate for the analysed cases.

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