INELASTIC RESPONSE OF MULTI-STOREY ASYMMETRIC BUILDINGS

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SUMMARY

In this paper the authors analyse the seismic response of multi-storey mass eccentric buildings. Structures are schematised by a spatial set of plane frames which sustain forces in their plane only; within such frames columns and beams are modelled by elements having elastic-perfectly plastic behaviour. Different distributions of stiffness and positions of mass centre are considered so as to represent both torsionally flexible and torsionally rigid systems with low or high structural eccentricity. The strength of the resisting elements is assigned in two different ways: by the standard multi-modal analysis and according to a proposed design procedure which requires a double application of the multi-modal analysis. The structural behaviour is evaluated by means of a step-by-step analysis using thirty artificially generated spectrum compatible accelerograms. The results of the parametric analysis highlight the necessity of an improvement of the design procedure based on the standard application of the multi-modal analysis and the effectiveness of the proposed formulation of the design eccentricity.

INTRODUCTION

Owing to the great number of parameters which govern the inelastic behaviour of multi-storey asymmetric-plan buildings, for many years the seismic response of such systems has been studied by means of one-storey models. The analysis of such simplified scheme has allowed to highlight the influence of the geometric and dynamic characteristics of in-plan irregular structures on their seismic response; furthermore, the application of many strength distributions and design procedures has marked the importance of the design criteria on the level of the maximum ductility demands. A thorough study of the seismic response of one-storey models has allowed the authors to point out in the past the characteristics of the structural behaviour of in-plan irregular systems and has led to the proposition of a design procedure aiming at limiting the ductility demands of such models to those of the corresponding torsionally balanced systems i.e. of schemes with coincident mass and stiffness centres [Ghersi and Rossi, 1999]. The design procedure has been tested on mass and stiffness eccentric models subjected to mono and bi-directional accelerograms [Rossi, 1998; Ghersi and Rossi, 1998] showing appreciable reductions of both displacement and hysteretic energy ductility demands [Rossi, 2000].

Many researchers are now moving their attention to the study of the response of multi-storey asymmetric-plan models in order to verify the reliability of the results obtained by the analysis of the more simple one-storey systems. Some disagreement has soon appeared among the researchers on the definition of the model and on the choice of the design criteria to be adopted [Duan and Chandler, 1993; Moghadam and Tso, 1996]. The difficulty in analysing and interpreting the response of actual multi-storey asymmetric-plan buildings has indeed forced some researchers to consider models which in different ways simplify the numerical calculus and the post-processing analysis. The authors focus their attention on the seismic response of regularly asymmetric multi-storey systems so as defined in Chopra and Hejal [1987] and propose some simplifications of the numerical model which can help in the comprehension of the results and in the comparison with those of one-storey models. When resisting elements are allowed to experience plastic deformations during earthquakes the seismic response of multi-storey asymmetric-plan schemes is influenced by two different characteristics of the systems: the dynamic properties (structural eccentricity, uncoupled lateral-torsional frequency ratio, uncoupled period of vibration etc.) and the distribution of strength between the resisting elements. A fundamental aim of the design is the attainment of a global collapse mechanism; according to the philosophy of the capacity design we can thus

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impose that inelastic deformations be experienced by the ending cross-sections of the beams and by the bottom cross-sections of the first order column only. The analysed model is therefore a simplified multi-storey asymmetric-plan scheme in which all column cross-sections apart from those at the base have elastic behaviour while the other ending cross-sections may present plastic hinges. The scheme grants the targeted global collapse mechanism and allows to separately study the problem of limiting the ductility demands of the cross-sections where plastic hinges are expected and that (not discussed in the paper) of appropriately predicting the minimum strength to assign to the other cross-sections so as they do not yield. It is furthermore to notice that in many studies multi-storey asymmetric-plan systems are schematised by means of models for which the presence of the vertical loads in the phase of design does not influence the definition of the strength of the resisting elements: this choice may remarkably affect the seismic behaviour and therefore requires great attention [Ghersi et al., 1999]. In order to consider this aspect in the numerical analyses and to make the results easily comparable with those of the corresponding one-storey models, the authors take into account the effect of the vertical loads in design but adopt a constant value of the overstrength factor for all the frames.

MULTI-STOREY MODELS

The numerical investigation analyses six-storey in-plan irregular buildings having one symmetry axis (x-axis). The deck, rectangular in shape (L=29.50 m $\times$ B=12.50 m) and having mass of 187.3 t at each floor, is rigid in its own plane. The structure is constituted by 12 frames (4 seven-bay frames along the longitudinal direction and 8 three-bay frames along the transversal one), symmetrically disposed with respect to the geometrical centre of the deck C_G and having stiffness and strength in their plane only (Figure 1). This study analyses just mass eccentric systems, because stiffness eccentric systems have been shown to have a similar behaviour [Ghersi and Rossi, 1999]. Two cross-sections have been used in each frame, one for the columns and the other one for the beams; they have been varied proportionally from one frame to the other, so as to obtain the required value of the stiffness radius of gyration. Values of the uncoupled lateral-torsional frequency ratio Ω_θ equal to 0.6, 0.9, 1.1 and 1.4 have been considered, so as to analyse schemes representative of both torsionally flexible and torsionally stiff structures. For each geometrical scheme three different distributions of mass have been considered, so as to maintain symmetry with respect to x-axis and to obtain structural eccentricity e_s=0 (torsionally balanced system), 0.05 L (small eccentricity) and 0.15 L (large eccentricity) in the orthogonal direction. In every case a transla-

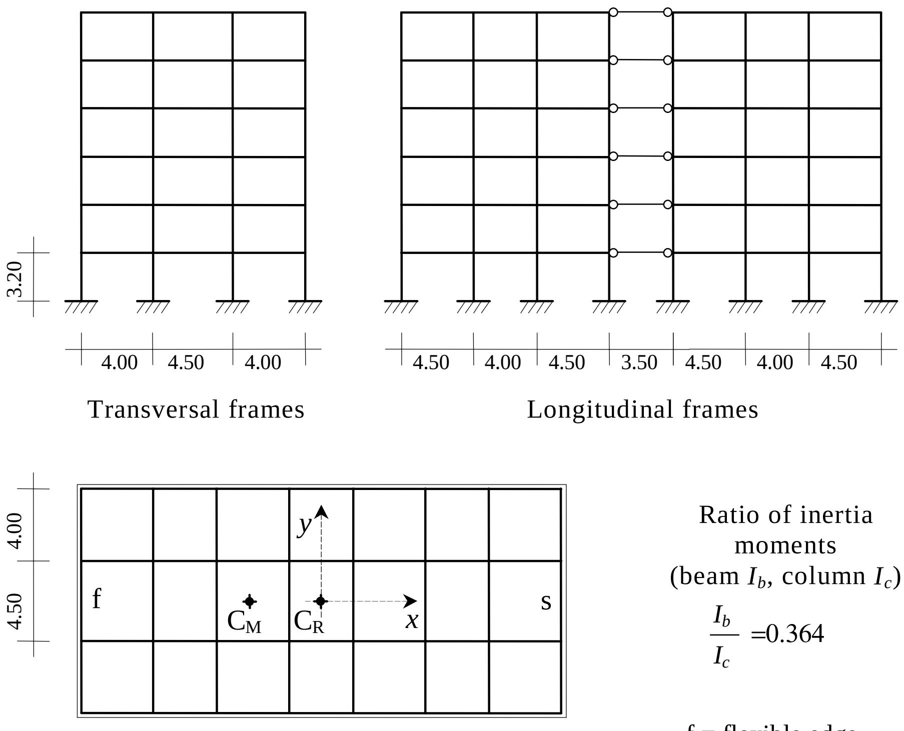

Figure 1. Model plan and scheme of the frames

tional period $T_x = T_y = 1$ s and a ratio of the torsional stiffness due to the elements along the x-axis to the total torsional stiffness γ_x equal to 0.2 have been assumed. In the analyses the columns of the frames are considered to be axially inextensible and the plastic domain of the bottom cross-section of the first storey columns not dependent on the axial force.

DESIGN CRITERIA

Each one of the schemes, obtained by varying Ω_θ and the location of C_M, has been designed twice: once by means of the standard application of the multi-modal analysis and the second time by a proposed design procedure. In both cases the strength of the ends of the beams and that of the bottom cross-section of the first order columns have been evaluated by two load conditions:

– vertical loads, increased by the coefficients γ_g and γ_q according to ultimate limit state conditions;
– vertical loads, reduced by the coefficient ψ, and seismic action evaluated according to the elastic spectrum proposed by Eurocode 8 for soil A with α=0.35, reduced by a behaviour factor q=5.

No accidental eccentricity has been taken into account neither in the phase of design nor in the numerical analyses. The use of different load conditions gives each frame an overstrength which depends on the entity of the internal actions produced by the vertical loads with respect to those caused by the seismic actions. In practical applications the values of the overstrength are generally quite variable among the frames and may be very large in torsionally flexible schemes; in such models, indeed, the design forces are very small for some frames, both because the outermost frames are extremely flexible to obtain Ω_θ<<1 and because the design displacement of some frames is very small. In order to limit the influence of the overstrength on the inelastic response of structures the vertical loads are distributed in such a way to obtain a constant value of the overstrength in torsionally balanced systems.

The proposed design procedure requires that the effect of the seismic action be evaluated by the envelope of a double application of the multi-modal analysis with the CQC rule, i.e. combining the contributions of the different modes of vibration by means of the correlation factors given by Der Kiureghian et al. [1981]. The first analysis is carried out with reference to the nominal locations of the mass and stiffness centres, while the second one is performed with reference to the location of the mass centre displaced towards the stiffness centre of a quantity e_d, named *design eccentricity*. The values of the design eccentricity have been evaluated according to the formulation proposed in the past by the authors with reference to one-storey asymmetric models:

$$e_d = \max \begin{cases} k\,(e_s - e_r) \\ 0.6\, e_s \end{cases} \qquad (1)$$

where:

$$\begin{aligned} k &= \max \begin{cases} 3.3 - 2.5\,\Omega_\theta + 0.04\,q \\ 1 \end{cases} \\ e_r &= \max \begin{cases} 0.1\left(0.5\Omega_\theta - 0.4\right)L \\ 0.01\,L \end{cases} \end{aligned} \qquad (2)$$

The overstrength ratio of the i^{th} frame O_i, defined as the ratio of the strength of the frame evaluated by means of push-over analysis to the design seismic base shear, has been fixed to 1.5 for the torsionally balanced system in order to cancel the influence of different values of the overstrength between the frames in torsionally balanced structures and to mitigate it in asymmetric-plan systems. Figure 2 shows the trend of the overstrength ratio both in torsionally flexible and in torsionally rigid structures. When standard multi-modal analysis is used, the variation of the overstrength ratio between asymmetric-plan and torsionally balanced systems is generally low for small structural eccentricity but can rise even above 30% in the case of torsionally rigid structures with high structural eccentricity. The location of the maximum values is between the flexible side and the centre in torsionally flexible systems and it moves toward the rigid side as either the eccentricity or the torsionally rigidity increases. When the proposed design procedure is applied, very slight changes are denoted in the case of torsionally flexible systems, while greater differences are evident at the stiff edge of torsionally rigid structures where the overstrength ratio induced by the proposed design approach reaches values close to those of the corresponding torsionally balanced schemes.

RESULTS

In order to statistically analyse the seismic behaviour of each asymmetric-plan system, its inelastic response has been evaluated to a set of thirty artificially generated accelerograms [Ghersi and Rossi, 1998], matching the elastic response spectrum proposed by Eurocode 8 for hard layer soil (class A) and a 5% damping coefficient. The accelerograms, characterised by a duration of the stationary part of 22.5 s, comply with the requirements of Eurocode 8.

For each accelerogram the attention has been focused on local and global ductility. Their values have been normalised to those of the corresponding torsionally balanced system in order to directly compare results of both asymmetric and symmetric structures and then statistically analysed; the mean of the thirty maximum normalised values has been finally assumed to synthesise the seismic response of the asymmetric system.

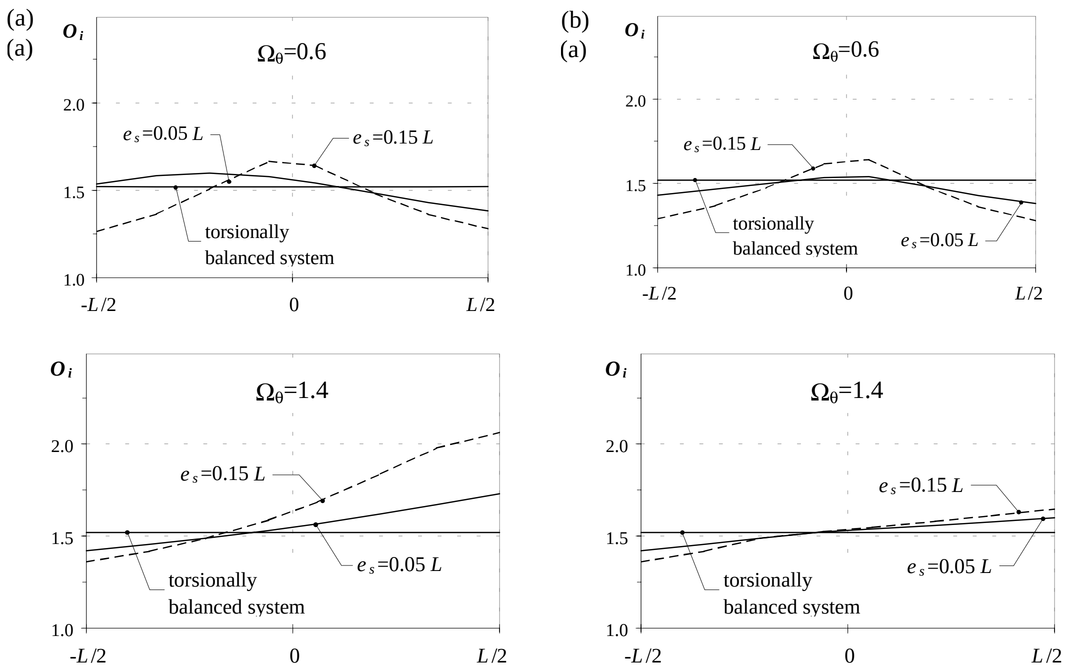

Figure 2 Overstrength ratio of mass eccentric multi-storey systems designed by means of the standard application of the multi-modal analysis (a) and by the proposed procedure (b)

In order to find similarities between the results of one-storey and multi-storey asymmetric systems a global dis experienced by the frame during the dynamic analysis to a conventional yield displacement of the frame; this one has been obtained approximating the actual base shear-top sway curve of the frame with an elastic perfectly plastic law having an horizontal segment at the level of strength of the frame and an inclined segment tangent to the actual curve of the system at the origin.

The mean value of the normalised displacement ductility demand d is shown in Figure 3 with reference to both torsionally flexible and rigid structures with low and high structural eccentricity. When multi-modal analysis is applied without design eccentricity the normalised displacement ductility demand is in any case not much higher (≤10-15%) than that of the corresponding torsionally balanced systems; the greatest values are found for asymmetric-plan structures having small structural eccentricity, while values close to unity are always displayed when $\Omega\theta$ is in the range 0.9-1.1 (not shown in figure). The normalised displacement ductility demand increases at the flexible edge of torsionally flexible systems and at the stiff edge of torsionally rigid systems, because in such parts of these structures the design displacements produced by the multi-modal analysis (lower than those of the corresponding torsionally balanced systems) do not agree with those denoted by the actual inelastic response, which is generally more translational than the elastic one. The proposed procedure modifies the values of strength, imposing higher values where the analysis without design eccentricity has shown the greatest normalised global displacement ductility demands. The resulting values of the examined parameter are remarkably influenced by the new distribution of strength and in all the cases lower than unity. The values of ductility demands evaluated for the corresponding one-storey models (Figure 4) show a similar trend but quite different values, probably caused by the conventional evaluation of the global ductility index in multi-storey systems.

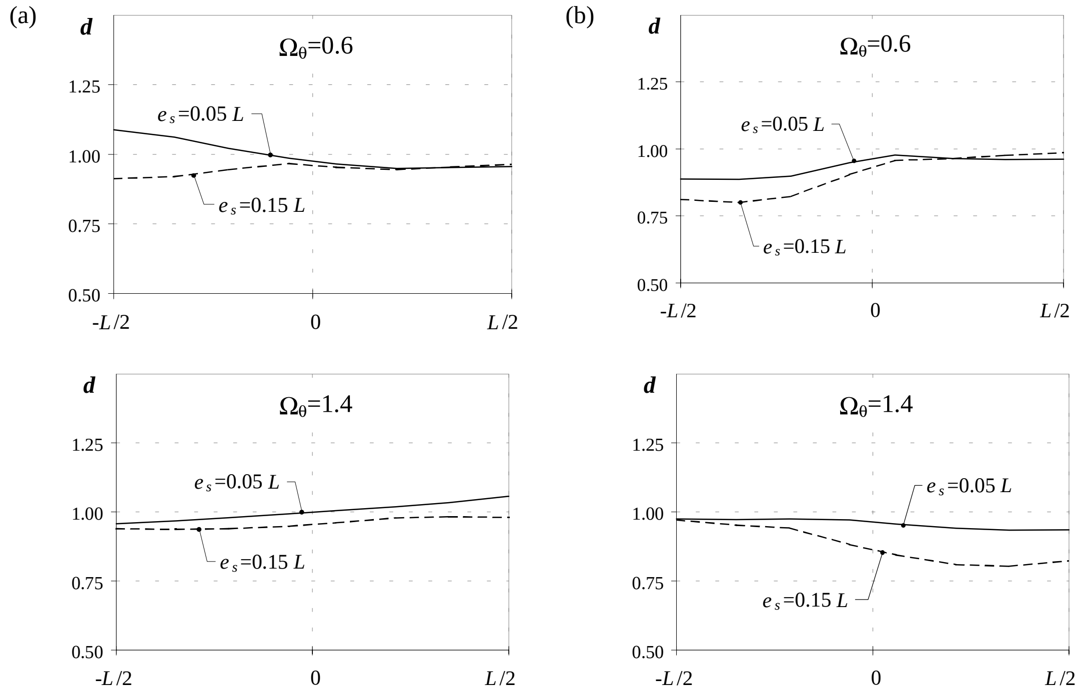

Figure 3 Normalised global ductility demands of mass eccentric multi-storey systems designed by means of the standard application of the multi-modal analysis (a) and by the proposed procedure (b)

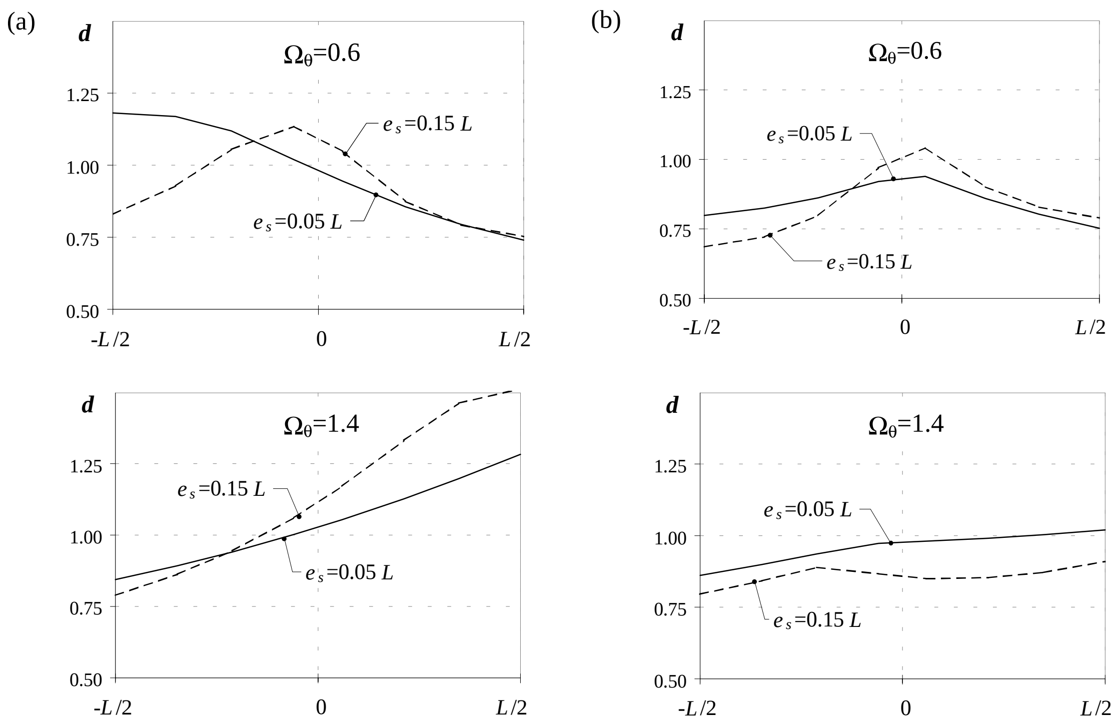

Figure 4 Normalised displacement ductility demands of mass eccentric one-storey systems designed by means of the standard application of the multi-modal analysis (a) and by the proposed procedure (b)

values of the normalised ductility demands of the single element, obtained subjecting the system to the selected set of thirty accelerograms, are in this case more scattered than in structures characterised by small eccentricity. The formulation of the design eccentricity aims, indeed, at limiting to unity the mean value of the maximum normalised ductility demands [Ghersi and Rossi, 1999; Rossi, 1998] which are experienced during the earthquakes by the whole system and not by the single resisting element; this condition is obviously more strict than that imposed to the single element so as previously defined and shown in Figure 5. The comparison with the results of one-storey models (Figure 4) highlights the remarkable analogy between the ductility demands of one-storey models and the rotational ductility demands of the bottom cross-sections of the columns of the first order of multi-storey schemes.

The rotational ductility index has been calculated also for the beams. The trend of the mean value of the normalised rotational ductility demands of all ending cross-sections of the beams of the same floor and frame is shown in Figure 6 with reference to systems characterised by different values of the uncoupled lateral-torsional frequency ratio Ωθ and structural eccentricity es. The values of the normalised rotational ductility demand are comparable with those of the columns apart from the upper storey where they are very high and at the edge opposite to that where, at the lower storey, the mean normalised rotational ductility demand reaches its maximum. Anyway, the absolute values (Figure 7) are always significantly smaller than those of the columns and, at the upper storey, not much higher than those of the beams at the lower storeys. The use of the proposed formulation produces at the lower storeys reductions similar to those highlighted for the columns, but no significant decrease is obtained at the upper storey where the strength of the beams is governed by the vertical loads only.

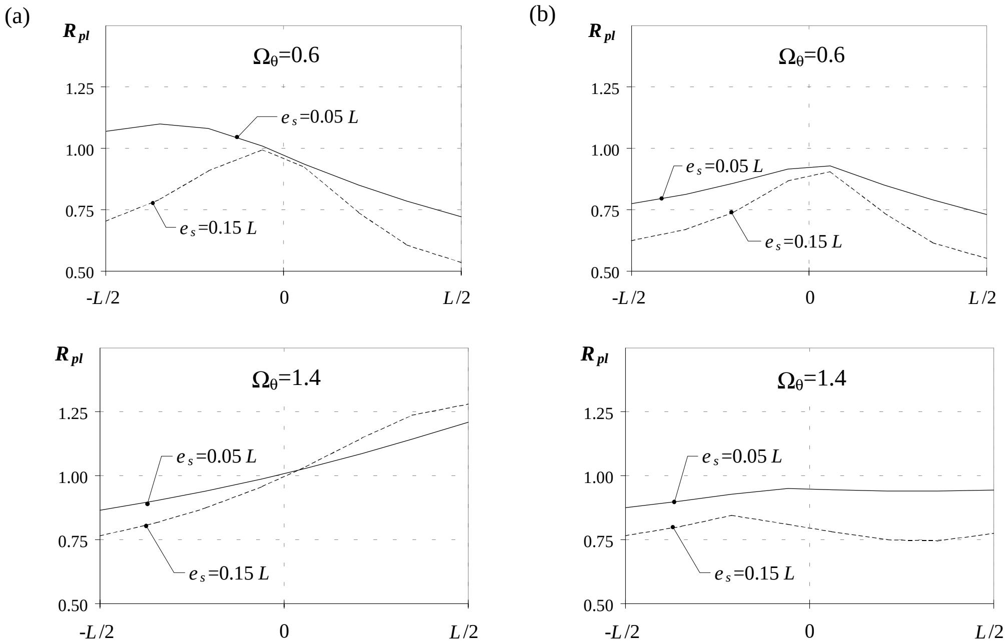

Figure 5 Normalised rotational ductility demands of the bottom cross-section of the first order columns of mass eccentric systems designed by means of the multi-modal analysis (a) and by the proposed procedure (b)

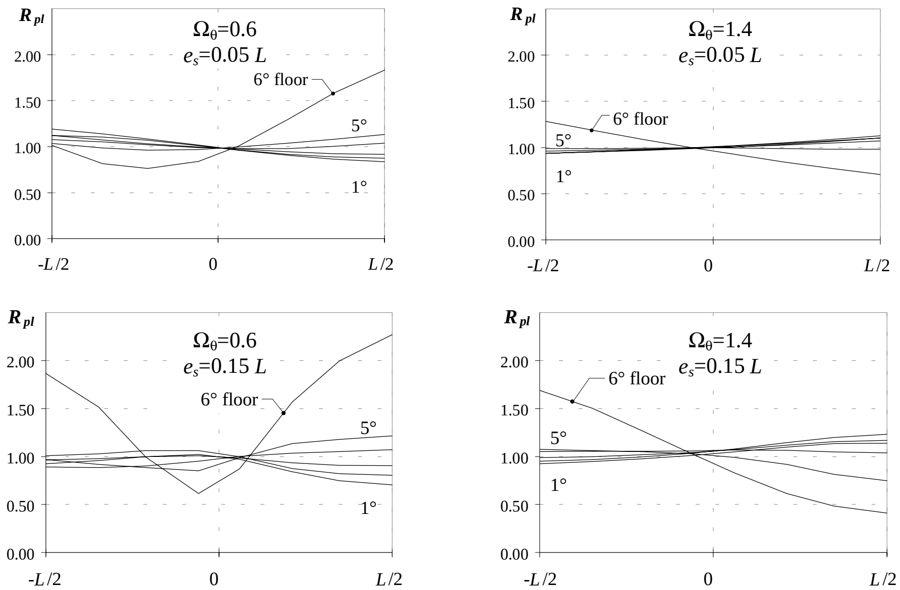

Figure 6 Normalised rotational ductility demands of the ending cross-section of the beams of mass eccentric systems designed by means of the standard application of the multi-modal analysis.

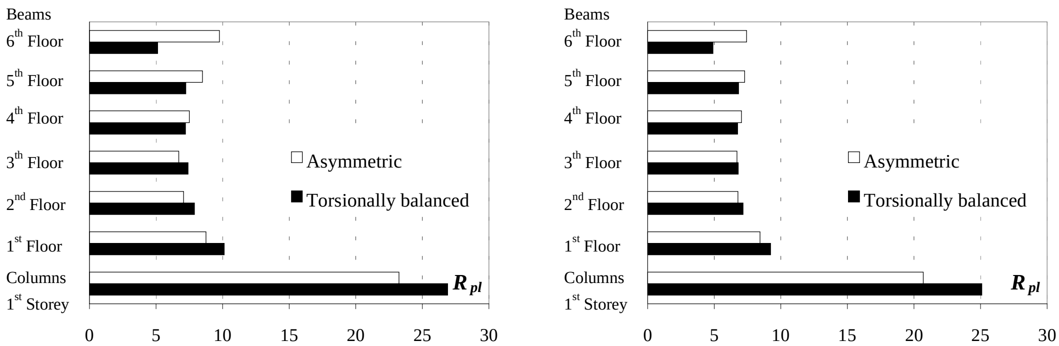

Figure 7 Rotational ductility demands of the ending cross-section of the beams of mass eccentric systems designed by means of the standard application of the multi-modal analysis.

CONCLUSIONS

The paper analyses the influence of the design criteria on the seismic response of regularly asymmetric multi-storey buildings. The authors compare the effect of the standard application of the multi-modal analysis and that of a proposed design procedure on the behaviour of multi-storey asymmetric-plan framed structures in seismic areas leading to the following conclusions:

1. Many analogies of behaviour exist between one-storey and multi-storey models so that a thorough study of the more simple and manageable one-storey model may provide important information on the behaviour of regularly asymmetric buildings.
2. The standard application of the multi-modal analysis to regularly asymmetric buildings leads to rotational ductility demands at the base of the columns often not very high, while the rotational ductility demand of the beams reaches at the upper storey values which are about the double of those of the corresponding torsionally balanced systems. The application of an appropriate design procedure seems to be opportune to reduce such values to those of the corresponding torsionally balanced systems.
3. The design procedure proposed by the authors reaches the aim of limiting the global and local ductility demands of the multi-storey asymmetric-plan structures. Only the high values of the rotational ductility de-

mand of the beams at the upper storey are not reduced by the use of the procedure, but they are anyway quite low if compared to the actual capacity of the members.

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