INFLUENCE OF DESIGN CRITERIA ON THE INELASTIC RESPONSE OF REGULARLY ASYMMETRIC MULTI-STOREY BUILDINGS

Aurelio Ghersi[1], Edoardo Marino[2], Pier Paolo Rossi[3]

ABSTRACT

The paper presents the results of a parametric analysis on multi-storey regularly asymmetric framed buildings. The numerical tests are carried out with reference to wide ranges of the uncoupled lateral-torsional fundamental frequency ratio and structural eccentricity so as to analyse the seismic behaviour of torsionally rigid or flexible systems characterised by low or quite high structural eccentricity. The buildings are designed according to different design procedures based on either multi-modal or static analysis, so as to highlight their deficiencies and good qualities. The systems are subjected to a set of thirty artificial accelerograms scaled to a peak ground acceleration equal to 0.35 g and compatible to the elastic response spectrum proposed by Eurocode 8 for hard layer soil. The structural response is analysed in terms of global and local damage parameters and compared to that of the corresponding torsionally balanced systems.

INTRODUCTION

The influence of either mass or stiffness plan-asymmetry on the seismic response of multi-storey buildings has been for a long time studied by means of one-storey models. This choice has been justified and promoted in the past by the analytical demonstration that allowed to relate the elastic response of a particular class of multi-storey asymmetric buildings, named *regularly asymmetric systems* [2], to that of the corresponding multi-storey torsionally balanced system and one-storey torsionally coupled scheme. Owing to its simplicity the one-storey model has been later on used also in the study of the inelastic behaviour of multi-storey asymmetric systems, although the aforementioned analytical demonstration was not valid in the inelastic range.

In order to confirm or deny the results of the analysis on one-storey models some studies have been carried out in the last years [1] on regularly asymmetric multi-storey models. Because highly refined plan-asymmetric multi-storey systems would have involved a cumbersome analysis of the dynamic response, simplified mechanical models have been differently proposed by some researchers according to the various behavioural aspects they desired to focus. In most cases the analysed schemes were designed

[1] Prof.; [2] Ph.D. Student; [3] Ph.D.
Dept. "Ingegneria Civile ed Ambientale", Faculty of Engineering, Catania (Italy)

according to static analysis, improved by the adoption of design eccentricities aiming at providing an elastic behaviour in occurrence of low intensity earthquakes and limiting the ductility demands of the resisting elements in occurrence of strong ground motions. The use of the multi-modal analysis has been instead often neglected in spite of the undoubted advantages which it provides in the evaluation of the elastic structural response; indeed the application of the multi-modal analysis with actual locations of both mass and stiffness centres allows to obtain a quite perfect estimate of the elastic response without any particular effort while static analysis needs the definition of a primary design eccentricity to overcome the deficiencies of an approximated design analysis. Furthermore the standard application of both static analysis and multi-modal analysis needs improvements in order to reduce the maximum ductility demands of the resisting elements [1] [5] [8]. Such result, conceptually known to the researchers, is further underlined by a recent study [6] focused on the different effectiveness of the two design procedures, studied by means of the ratio of the maximum ductility demands of in-plan asymmetric models designed by static and multi-modal analyses. A reduction of maximum ductility demands to those of the corresponding torsionally balanced systems may be achieved by means of a double application of the multi-modal analysis based on an analytical expression of the design eccentricity [5].

In order to grant a global mechanism, in this paper the strength of the columns of the analysed schemes is supposed to be indefinitely high apart from the bottom cross-section of the first storey. At such ends and at the ending cross-sections of the beams the strength is differently defined by means of some design procedures proposed in the past based on either static or multi-modal analyses. The paper discusses only the influence of the design criteria on the maximum ductility demands; the differences between the strength conferred by the design procedures and that required by the numerical analyses to grant the desired behaviour to the column cross-sections which are deemed to remain elastic are not herein commented.

NUMERICAL MODEL

The numerical investigation analyses six-storey in-plan irregular buildings having one symmetry axis (*x*-axis). The deck, rectangular in shape (29.50×12.50 m) and having mass of 187 t at each floor, is rigid in its own plane. It is supported by a set of frames arranged along two orthogonal directions and having stiffness and strength in their plane only (Fig. 1). 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. In this study only mass eccentric systems have been analysed because stiffness eccentric systems have been shown to have a similar behaviour [5]. Only 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. In order to reduce the computational burden the columns of the frames are considered to be axially inextensible and the plastic domain of the bottom cross-sections of the first storey column not dependent on the axial force. Different schemes have been used with uncoupled lateral-torsional frequency ratio Ω_θ equal to 0.6, 0.9, 1.1 and 1.4, so as to analyse schemes representative of torsionally flexible or torsionally stiff structures. For each geometrical scheme three different dis-

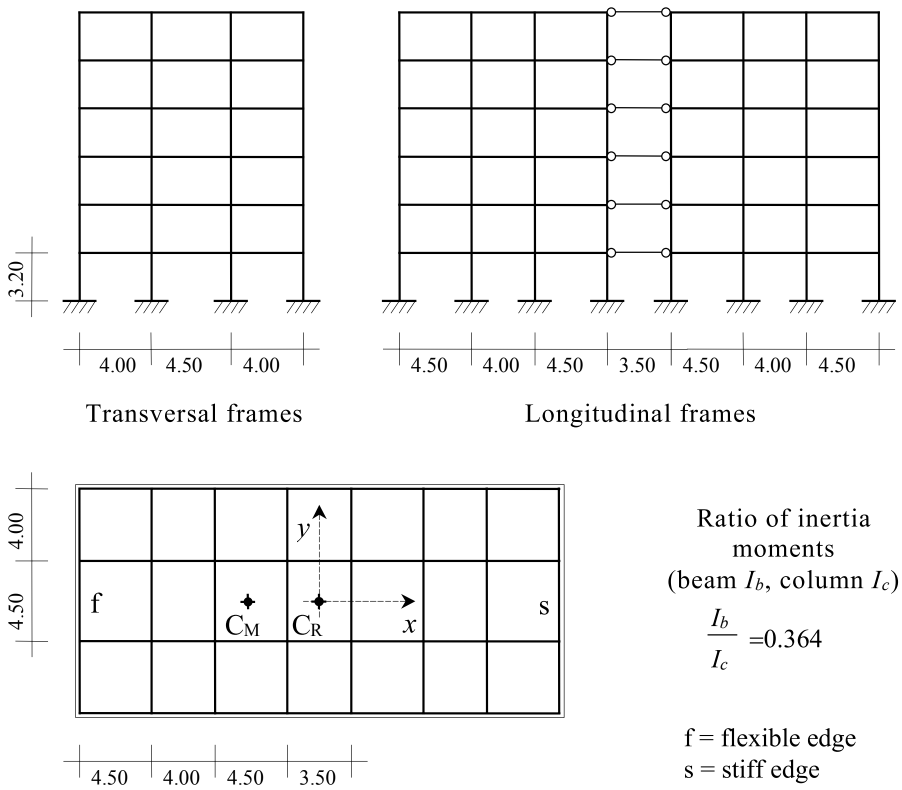

Figure 1 – Model plan and scheme of the frames

tributions 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 translational 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.

DESIGN CRITERIA

Each one of the schemes, obtained by varying Ω_θ and the location of C_M, has been designed by means of the design procedures proposed in the past by different researchers [3] [5].

The strength of the ends of the beams and that of the bottom cross-section of the first order columns has been fixed, independently of the stiffness of the resisting element, as the maximum between the values given by two load conditions:

– vertical loads, increased by the coefficients γ_g and γ_q according to the ultimate limit state approach (Eurocode 2 and 3);

– 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 and 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. In order to fulfil the capacity design criterion all the cross-sections of the columns apart from the bottom ones at the first order have been assumed to have unlimited strength so as to grant that the targeted global collapse mechanism occur during the earthquake.

The design procedure proposed by Ghersi and Rossi

The design procedure proposed by Ghersi and Rossi [5] requires to evaluate the effect of the seismic action by a double application of the multi-modal analysis with the complete quadratic combination rule: 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 must be evaluated according to the formulation proposed by the authors with reference to one-storey asymmetric models:

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

where:

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

A simplified version of the afore-mentioned design procedure may be obtained by assuming a design eccentricity e_d equal to the structural eccentricity e_s, i.e. by considering the effect of both a standard modal analysis and a translational analysis. Such value of the design eccentricity does not therefore allow any reduction of strength respect to that of the corresponding torsionally balanced system, according to a general provision of Uniform Building Code (1997).

The design procedure proposed by Chandler and Duan

The design procedure proposed by Chandler and Duan [3] involves a double application of the static analysis by means of the following design primary and secondary eccentricity, referred by the author to the stiffness centre:

$$e_{d_1} = A_1\, e_s \tag{3}$$

where:

$$A_1 = 2.6 - 3.6\left(e_s / L\right) \quad \geq 1.4 \tag{4}$$

and

$$e_{d_2} = 0.5\ e_s \tag{5}$$

In order to obtain low ductility demands of the outermost stiff-edge resisting element Chandler and Duan propose to reduce the behaviour factor in the evaluation of the strength of such element according to the expressions:

$$\begin{aligned}
&q' = q && 0 < e_s / L < 0.1 \\
&q' = q - \left(e_s / b - 0.1\right) / 0.1 && 0.1 < e_s / L < 0.2 \\
&q' = 0.8\, q && 0.2 < e_s / L
\end{aligned} \tag{6}$$

Even if not specified in [3], where the model is constituted by three resisting elements only, the behaviour factor has been linearly varied in this study from the stiff edge to the stiffness centre so as to increase the strength of all the elements located on the stiff side. No design concentrated force has been applied at the top of the building, as a modification to the linear vertical distribution of design loading: such design adjustment has been indeed considered by Chandler and Duan useless for asymmetric systems with structural eccentricity lower than 0.2 L.

It must be remarked that the above design procedure has been proposed by the authors only for torsionally stiff structures and verified in [3] with reference to asymmetric buildings having uncoupled lateral-torsional frequency ratio Ω_θ equal to 1.0. For sake of completeness the procedure has been applied in this study also to torsionally very flexible structures, but the negative performance of so designed schemes must not fall on the procedure itself; indeed the static analysis is not at all adequate to foresee the elastic response of torsionally very flexible systems and the proposed approach does not provide adjustment to overcome this problem.

Overstrength

If different load conditions are used in the phase of design the overstrength ratio of the i^{th} frame O_i, defined as the ratio of the limit base shear of the frame evaluated by means of push-over analysis to the design base shear, is always greater than unity. Its value 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 overstrength are generally quite variable among the frames and sometimes even very large [7]. In order to limit the influence of overstrength distributions variable between the frames on the inelastic response of the asymmetric schemes and to make easily interpretable the results, in this study the vertical loads are distributed in such a way to obtain in the corresponding torsionally balanced systems a constant value of the overstrength. On the base of the examination of some actual framed buildings the overstrength ratio of each frame has been fixed equal to 1.5.

Furthermore, in order to make comparable the results of the numerical analyses of structures designed by different approaches, the strength of the torsionally balanced structures calculated by procedures which use multi-modal analysis have been amplified so as to confer to such systems a base shear equal to that of the same schemes designed by means of static analysis. The same amplifying factor has been then applied to the values of strength of the asymmetric structures.

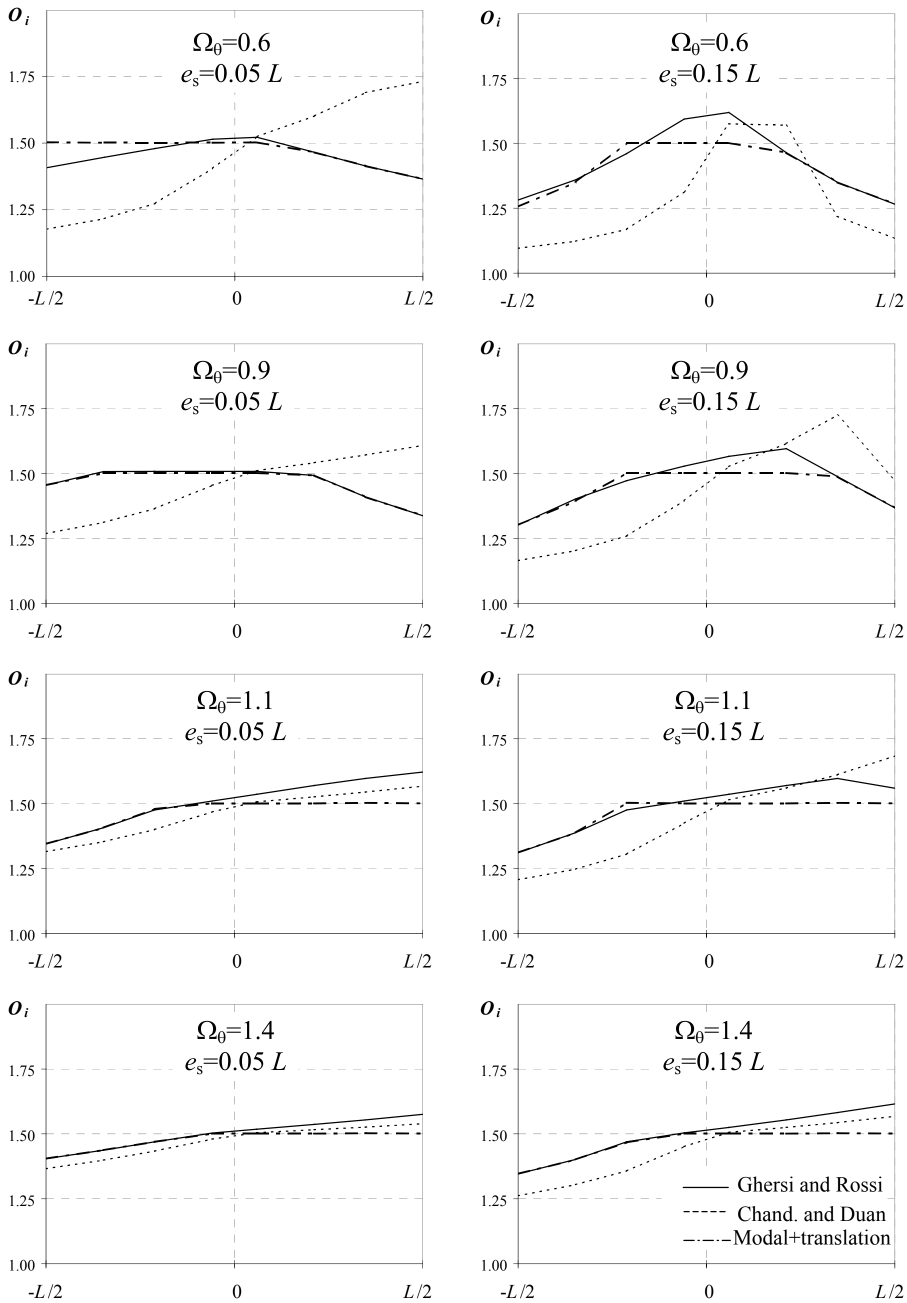

Figure 2 – Overstrength ratio of the frames arranged along y-direction

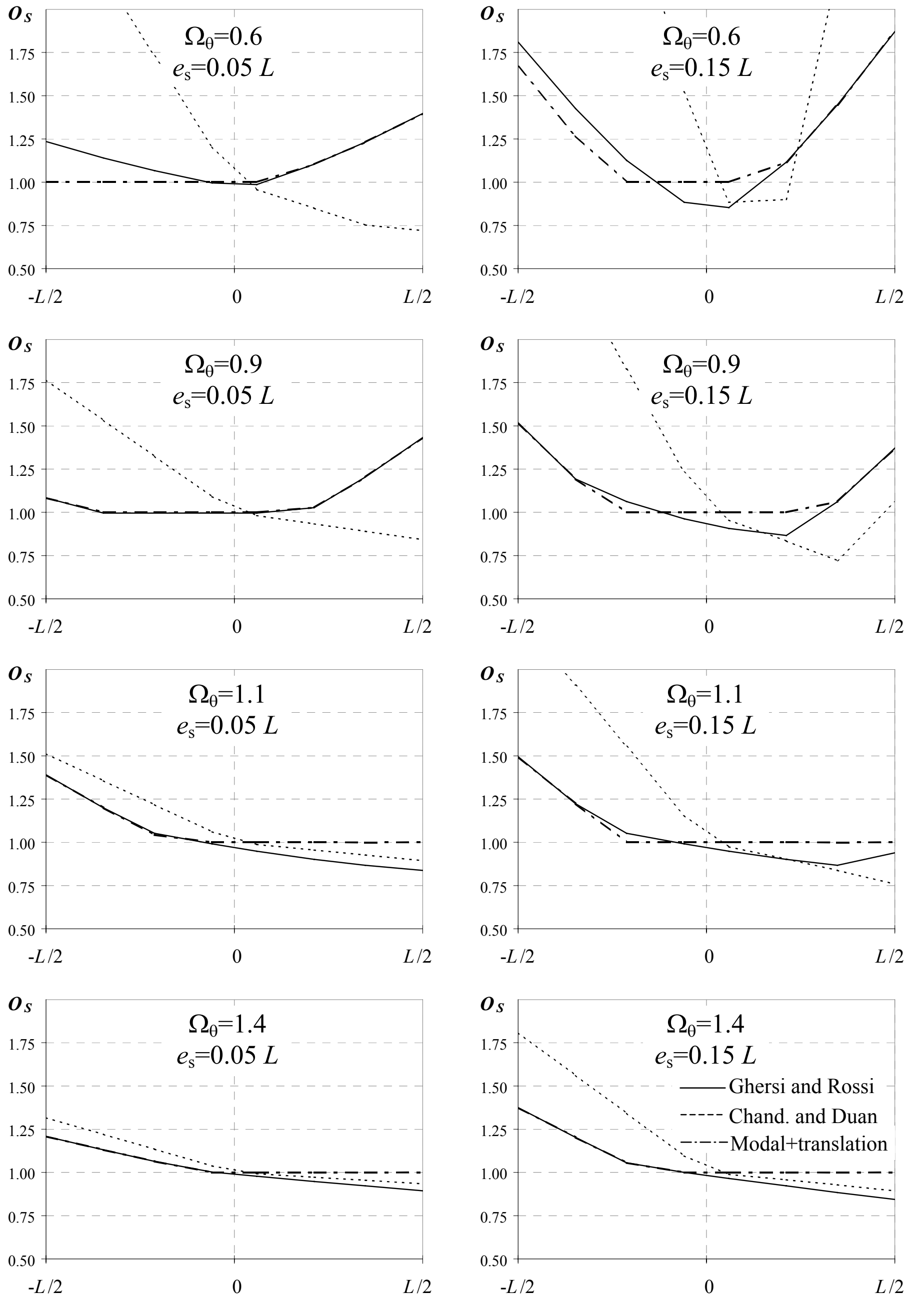

Figure 3 – Ratio of the base design shear of the i^{th} frame of the asymmetric-plan system to that of the corresponding torsionally balanced system

In spite of the adjustments proposed in the phase of design, the overstrength ratio O_i shows in the asymmetric-plan systems not negligible scattering respect to the value fixed for the torsionally balanced systems (Figure 2). The greatest modifications of the default value are generally evident in the outermost elements of torsionally flexible structures because in such elements the increase of the design shear is large in percentage respect to that of the corresponding balanced system. More close values of the overstrength ratio are instead characteristic of torsionally rigid systems, lower at the flexible edge and higher than the default value at the stiff side.

Other considerations on the global strength provided by the different design approaches may be based on the analysis of the normalised base shear O_s, i.e. the ratio of the design shear of each frame of the asymmetric-plan system to that of the corresponding torsionally balanced scheme (Figure 3); these values are proportional to those of the normalised design displacements.

In torsionally flexible systems the design eccentricity suggested by the simplified method (modal + translational analysis) is slightly lower than that proposed by Ghersi and Rossi so that some reduction of strength at the flexible side can be by it provided respect to that required by the procedure proposed by Ghersi and Rossi. The design procedure proposed by Chandler and Duan gives in most cases an inadequate strength at the rigid side of the structure, as it may be expected being such systems out of the range of application of their formulations.

In torsionally rigid structures the application of the simplified procedure produces the greatest values of strength for the rigid side-elements, being the reduction suggested by the other design procedures not at all negligible. The design procedure proposed by Chandler and Duan gives generally high values of the normalised base shear on the flexible side of buildings, particularly in systems with large structural eccentricity, owing to the high primary design eccentricity suggested by the authors.

Table 1 finally reports the normalised global overstrength factor, i.e. the ratio of the sum of the limit base shears of all the frames of an asymmetric system arranged along *y*-direction (given by push-over analysis) to the sum of the analogous values evaluated for the corresponding balanced scheme; this parameter is somehow representative of the structural cost consequent to the adoption of a design criterion. The values are generally quite high for the design method proposed by Chandler and Duan, mainly in the case of high structural eccentricity. The design approach proposed by Ghersi and Rossi gives in most cases the lowest values. It must be noted that the values here referred are slightly smaller than those evaluated for single-storey systems [5] because of the influence of vertical loads on global overstrength, neglected in those schemes.

RESULTS

The selected asymmetric systems have been subjected to a set of thirty artificial accelerograms compatible with the elastic response spectrum proposed by Eurocode 8 for hard layer soil and characterised by a damping factor of 5%. All the accelerograms, scaled so as to have a peak ground acceleration equal to 0.35 g, present a duration of the strong motion phase equal to 22.5 s and a total duration of 30 s [4].

Table 1 – Normalised global overstrength factor of the elements in y-direction

Ω_θ	e_s	Chandler and Duan	Modal + translation	Ghersi and Rossi
0.6	0.05 L	1.11	1.02	1.03
	0.15 L	1.39	1.04	1.01
0.9	0.05 L	1.07	1.03	1.03
	0.15 L	1.22	1.05	1.03
1.1	0.05 L	1.06	1.04	1.02
	0.15 L	1.17	1.05	1.03
1.4	0.05 L	1.06	1.04	1.02
	0.15 L	1.16	1.07	1.04

The values of the local and global ductility demand, evaluated for each accelerogram, have been at first 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 as parameter of comparison, chosen so as to synthetically represent the seismic response of the selected asymmetric systems.

In order to find similarities between the results of one-storey and multi-storey asymmetric systems the global displacement ductility demand of each plane frame has been examined. It has been defined as the ratio of the maximum top sway displacement 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 4 with reference to both torsionally flexible and rigid structures with low and high structural eccentricity.

In torsionally very flexible structures (Ω_θ=0.6), both the procedures based on the application of multi-modal analysis provide values of the normalised global ductility close to unity and quite uniform for all the frames. Conversely the design procedure proposed by Chandler and Duan causes non uniform values of the normalised global ductility; in particular, quite low values are shown on the flexible side of the building owing to the high primary design eccentricity used, while high values are noticed on the stiff side, confirming the incorrectness of using this design approach for torsionally flexible systems.

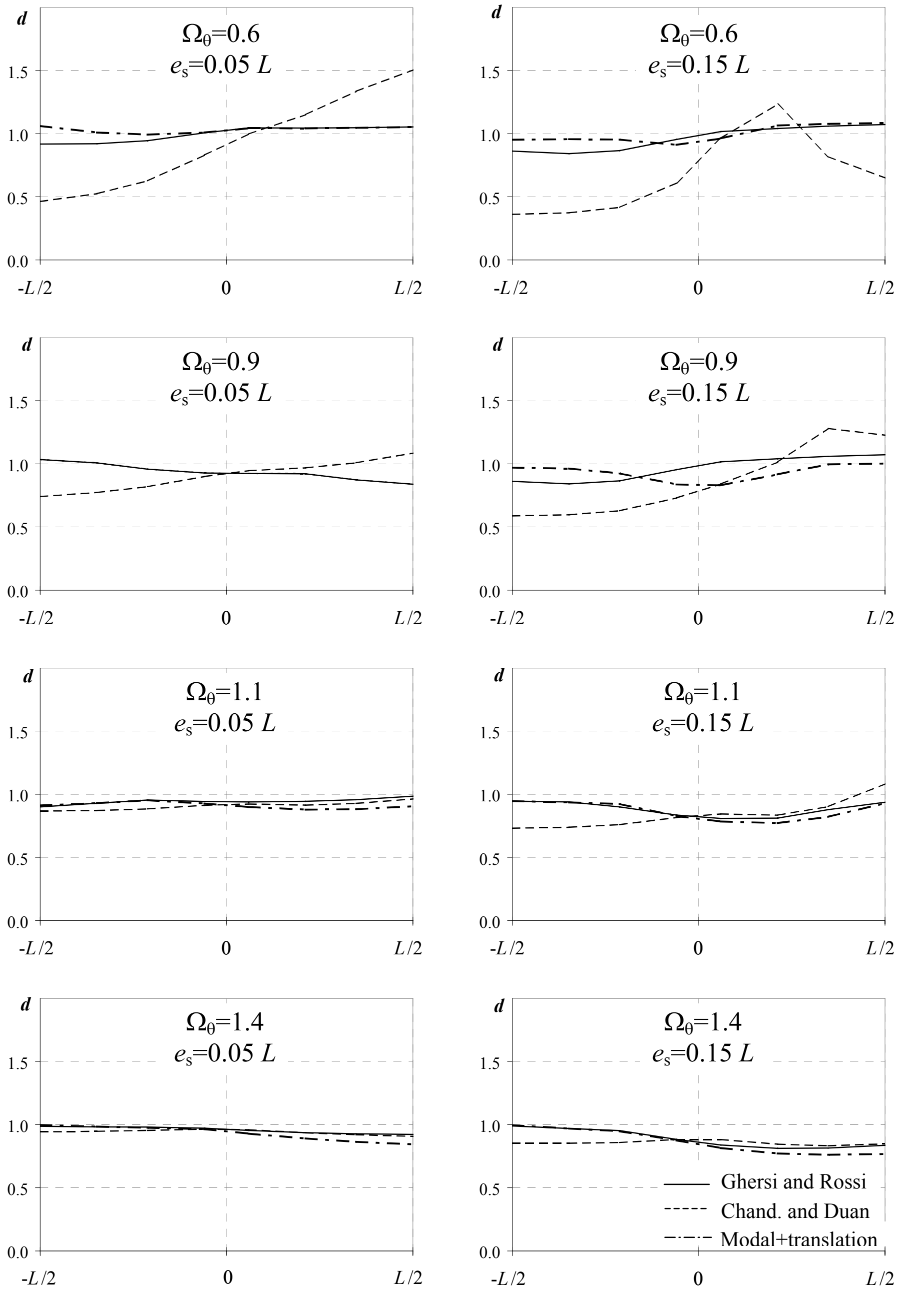

Figure 4 – Mean value of the normalised global displacement ductility demand d of torsionally flexible and rigid systems

For structures having Ω_θ equal to 0.9 and low structural eccentricity the design procedures using multi-modal analysis still provide values of the normalised global ductility approximately coincident and not far from unity; some difference may instead noticed for high values of the structural eccentricity when the diagrams of the two systems just go far away each other. The design procedure proposed by Chandler and Duan shows also in this case deficiencies deriving both from the use of the static analysis and from the adoption of design adjustments which are not able to opportunely modify the strength distribution consequent to the standard application of the static analysis.

For torsionally rigid structures (Ω_θ=1.1 and 1.4) with low structural eccentricity, the use of all the design procedures leads to similar and uniform values of the normalised global ductility. On increasing the structural eccentricity the procedure proposed by Chandler and Duan shows lower values of the ductility of the flexible side owing to the proposed high values of the primary design eccentricity.

The local behaviour of the members in which an inelastic response was expected, i.e. the bottom cross-sections of the first order columns and the ending cross-sections of all the beams has been further studied by means of the rotational ductility demand of each cross-section. This one has been defined as ratio of the total rotation to the elastic share evaluated approximately as the elastic rotation of a part 30 cm long of the member. As referred for the global displacement ductility index, the rotational ductility demands have been normalised by the values of the corresponding torsionally balanced systems.

The observations regarding the global ductility demand can be quite completely repeated for the local ductility demand of columns (Fig. 5).

For torsionally flexible systems designed according to the procedure proposed by Ghersi and Rossi, the normalised rotational ductility demand is slightly smaller than the global ductility demand at the outermost elements, mainly at the stiff edge. The use of static analysis with the eccentricity suggested by Chandler and Duan still gives unacceptable values at the stiff edge, while at the flexible edge the rotational demand is even smaller than the global ductility demand.

In torsionally rigid structures the values of rotational demand of structures designed according to the different approaches are more scattered and generally lower than the corresponding global ductility demands. When the torsional rigidity is very high (Ω_θ=1.4, not shown in the figure) the simplified procedure (modal + translational analysis) leads to damage indexes which are often very low at the stiff side, while the design eccentricities suggested by Ghersi and Rossi seem to lead to more uniform distributions of the normalised local indexes, with values always much below unity.

The normalised rotational ductility demand of the beams is shown in Figure 6. In any case the values of the rotational ductility demand are generally quite low apart from the top storey, where the vertical load condition govern the design of the elements. The absolute values of the rotational ductility are anyway smaller than those of the columns and, at the upper storey, not much higher than those of the beams at the lower storey, as it has been already observed in [8].

The application of both the design procedures based on the multi-modal analysis leads to similar results, while the approach proposed by Chandler and Duan seems to produce values of the normalised rotational ductility demand less uniform in general and quite higher at the stiff edge in comparison to the global and the column values.

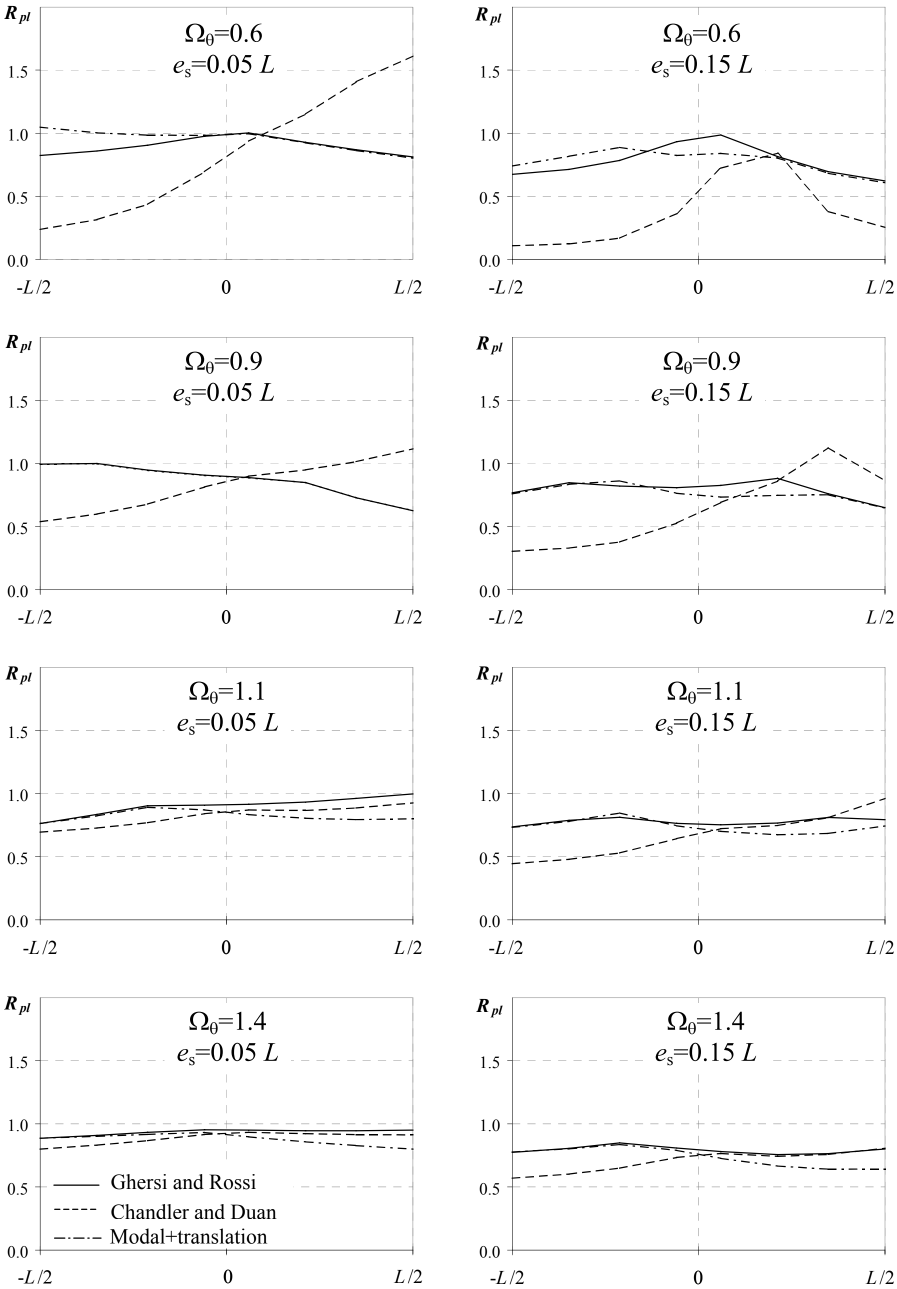

Figure 5 – Mean value of the normalised rotational ductility demand of columns of torsionally flexible and rigid systems

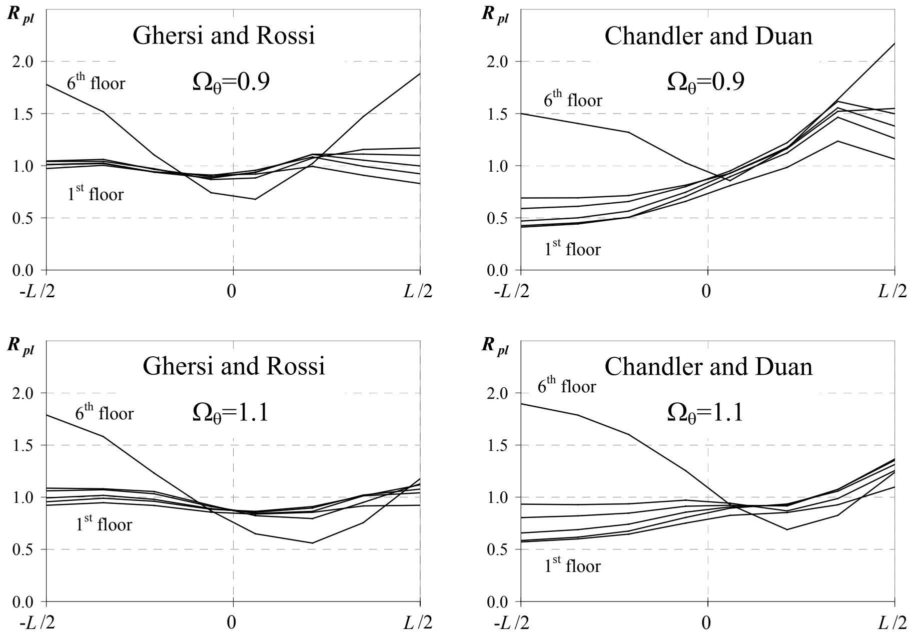

Figure 6 – Mean value of the normalised rotational ductility demand of beams (design procedures by Ghersi, Rossi and Chandler, Duan, e_s=0.15 L)

CONCLUSIONS

In this paper the influence of the design procedures proposed by Chandler and Duan and by the Authors have been studied. The wide set of numerical analyses performed on torsionally flexible and rigid systems allows to affirm that:

a) the use of static analysis with the eccentricities proposed by Chandler and Duan
   - is not at all appropriate for torsionally flexible systems (as assessed also by Chandler and Duan, who in a more drastically way state that torsionally flexible structures should not be built);
   - generally leads to high global overstrength even in torsionally rigid systems;
   - suggests primary design eccentricities which determine quite always high values of the design displacements on the flexible side of the structure;
   - suggests secondary design eccentricities which, at the stiff side of slightly torsionally-rigid systems, result in values of the local damage parameters slightly higher than unity.

b) The simplified design procedure characterised by a design eccentricity equal to the structural eccentricity (modal + translational analysis)
   - leads to values of overstrength in some cases slightly higher than those of the design procedure proposed by Ghersi and Rossi;
   - shows global and local ductility demands similar to those produced by the same design procedure;

– its application however simpler because it does not need the evaluation of the periods of vibration of the structure.

In conclusion, in the opinion of the Authors the static analysis is not adapt to the design of torsionally flexible systems. It is indeed inadequate to predict the elastic displacements; furthermore, the corrections necessary to provide a good estimate of the elastic response and those needed to opportunely limit the ductility demands to the values of the corresponding balanced systems seem not to be easy to produce. Conversely multi-modal analysis provides a correct evaluation of the elastic response of such systems and, even without design eccentricity, induces not excessive values of the ductility demands; the introduction of proper design eccentricity reduces even more such values and provides the targeted damage level.

In torsionally rigid structure both design methods, with proper corrections, manage to limit the ductility demands to the values of the corresponding balanced systems, but static analysis needs the use of primary dynamic eccentricities to well approximate the elastic response.

REFERENCES

[1] **Chandler, A.M. and Duan, X.N.** (1993). "Inelastic seismic response of code-designed multistorey frame buildings with regular asymmetry", *Earthquake Engineering and Structural Dynamics*, Vol. 22, 431-445.

[2] **Chopra, A. K. and Hejal, H.** (1987). "Earthquake response of torsionally-coupled buildings", *Earthquake Engineering Research Center*, Report n° UBC/EERC-87/20, College of Engineering, University of California, Berkeley,

[3] **Duan, X.N. and Chandler, A.M.** (1993). "A modified static procedure for the design of torsionally unbalanced multistorey frame buildings", *Earthquake Engineering and Structural Dynamics*, Vol. 22, 447-462.

[4] **Ghersi, A. and Rossi, P. P.** (1998). "Behaviour of in plan irregular buildings subjected to bi-directional ground motions", *Proc. of the Eleventh European Conference on Earthquake Engineering*, Paris.

[5] **Ghersi, A. and Rossi, P. P.** (1999). "Formulation of design eccentricity to reduce ductility demand in asymmetric buildings", *Engineering Structures* – in printing.

[6] **Ghersi, A., Marino, E., Rossi, P.P.** (1999). "Un confronto tra analisi statica e modale quali strumenti di progetto di edifici multipiano planimetricamente irregolari soggetti ad azioni sismiche", *Proc. IX National Conference "L'Ingegneria sismica in Italia"*, Torino (Italy) – in printing

[7] **Ghersi, A., Marino, E., Rossi, P.P.** (1999). "From one-storey to multi-storey asymmetric systems: conceptual differences and problems", *Proc. of European Workshop on the Seismic Behaviour of Asymmetric and Set-back Structures*, Istanbul, Turkey.

[8] **Ghersi, A., Marino, E., Rossi, P.P.** (2000). "Inelastic response of multistory symmetric buildings", *Proc. 12th World Conference on Earthquake Engineering*, Auckland (New Zealand) – in printing.

[9] **Rossi, P.P.** (1998): "Comportamento sismico di edifici planimetricamente irregolari", Ph.D. Thesis in Structural Engineering, Faculty of Engineering, Catania (Italy).