# THE KEY ROLE OF OVERSTRENGTH IN THE SEISMIC BEHAVIOUR OF MULTI-STOREY REGULARLY ASYMMETRIC BUILDINGS

MARIO DE STEFANO
Department of Constructions, University of Florence
Piazza Brunelleschi, 6 – 50121 Firenze, ITALY

EDOARDO MICHELE MARINO
Department of Civil and Environmental Engineering, University of Catania
Viale A. Doria, 6 – 95125 Catania, ITALY

PIER PAOLO ROSSI
Department of Civil and Environmental Engineering, University of Catania
Viale A. Doria, 6 – 95125 Catania, ITALY

## ABSTRACT

In past years, seismic response of asymmetric structures has been frequently analysed by means of single-storey models, because of their simplicity and low computational cost. However, it is widely believed that use of more realistic multi-storey models is needed in order to investigate effects of some system characteristics (such as overstrength, higher modes of vibration, etc.) that make behaviour of multi-storey schemes different from that of single-storey systems. This paper examines effects of the overstrength in element cross-sections on the seismic behaviour of multi-storey asymmetric buildings. It is shown that in actual buildings this characteristic, which is sometimes very variable both in plan and along the height of the building, may lead to distributions of ductility demands different from those expected according to the results from single-storey models. Consequently, torsional provisions, which aim at reducing ductility demands of single-storey asymmetric systems to those of the corresponding torsionally balanced systems, should be re-checked in light of the behaviour of realistic multi-storey buildings.

## INTRODUCTION

Past earthquakes have evidenced the great seismic vulnerability of plan-wise irregular structures with respect to torsionally balanced buildings. Because of such an evidence, large research efforts have been devoted to examining effects of the lateral-torsional coupling on building seismic behaviour [3], [4], [5], [9] and to developing and proposing design procedures [1], [3], [6], [8], [15], [16] aimed at providing both plan-regular and plan-irregular systems with a similar level of seismic protection. To this purpose, most studies on seismic response of asymmetric structures have analysed response of single-storey models [2], [3], [4], [5], [8], [9], [15], [16], representing the most extreme idealization of plan irregular buildings. Such models usually consist of a floor deck, rigid in its own plane and supported by massless, axially inextensible vertical resisting elements, characterised by a bi-linear elastic-hardening behaviour.

Use of single-storey models leads to an accurate evaluation of the seismic response of multi-storey irregular systems only when referring to the elastic range of behaviour of a special class of multi-storey irregular systems, named *regularly asymmetric systems* [10]. The geometric and mechanical conditions characterising such buildings are very restrictive and thus few actual systems fulfil all the requirements of the above-mentioned special class of buildings. Indeed, in such systems resisting elements must be arranged along an orthogonal grid and characterised, along either of the two directions, by stiffness matrices mutually proportional. Furthermore, mass centres must be aligned on a vertical line and mass radii of gyration must be equal at all floors. Nevertheless, the behaviour of actual structures is often substantially similar to that of regularly asymmetric systems.

When the structure is excited well into the inelastic range, simplified single storey models can give only qualitative information, since they cannot represent actual dissipative mechanism that develop in multi-storey frame, particularly formation of different numbers of plastic hinges at different locations. Indeed, it is well known that in a multi-storey framed system which exceeds the elastic limit, plastic hinges arise only in a few cross-sections. In asymmetric buildings this phenomenon occurs differently in frames because of the deck rotations, i.e. at a given time some frames may be widespread yielded, some others may be characterised by only a few plastic hinges, while the remaining part of the structure is in the elastic range. At that moment the stiffness matrices of the frames, initially proportional to each other, loose their proportionality making the building irregularly asymmetric.

In spite of the awareness of such aspect of the inelastic seismic behaviour of actual asymmetric structures, in the past many researchers have dealt with the study of the inelastic response of asymmetric buildings by means of single-storey schemes, due to of their simplicity and their dependence on few key mechanical parameters. In recent years, evaluation of the inelastic response of multi-storey systems has become more feasible due to development of powerful computational tools; therefore, the check and the enrichment of the previous results referring to single-storey models is believed to be auspiciable. In the wake of this kind of research Duan and Chandler [1], [6], as well as De La Lera and Chopra [2], have recently analysed multi-storey asymmetric buildings with shear-type frames. Unfortunately, such models, which present a convenient simplified behaviour, consider that plastic hinges may occur at the ends of columns only and, therefore, they allow the investigation of buildings characterised uniquely by undesidered storey collapse mechanisms. A more realistic model have been instead considered by Moghadam and Tso [12], [13], constituted of slabs supported by plane frames designed according to the capacity design criterion so as to develop plastic hinges in beams as well as in columns. Such studies have given very interesting contributions to the understanding of the inelastic response of actual irregular buildings. Nevertheless, they do not explain why and to which extent some structural mechanical and dynamic characteristics (e.g. overstrength of single cross-sections, higher modes of vibration, etc.) may influence seismic response of multi-storey schemes. It is opinion of the Authors that an in-depth investigation of the influence of some aspects of the structural design and dynamic on the seismic response of asymmetric buildings may allow a more precise evaluation of the limits within which the present knowledge based on studies of single-storey schemes may be extended to actual multi-storey systems.

To this purpose, this paper focuses on the effects of the cross-section overstrength on the plan-wise distribution of the ductility demand. In actual buildings overstrength, sometimes very variable both in plan and in elevation, may lead to distributions of ductility demands remarkably different from those expected according to the results from single-storey models. Owing to the usual disomogenous differences between the design internal actions and the real strength of cross-sections, torsional provisions considered in codes and aiming at reducing the ductility demands of single-storey asymmetric schemes to those of the corresponding balanced systems may unexpectedly fail their target in actual multi-storey buildings.

## GLOBAL AND LOCAL OVERSTRENGTH

In real frames, which are designed by taking into account technogical constraints, only a few plastic hinges arise under the design horizontal forces, being the expected collapse mechanism characterised by much larger horizontal forces. This phenomenon is justified by the overstrength of most cross-sections, i.e. by strength values larger than those strictly required by the design analysis. Design of steel frames is largely affected by overstrength due to technological and commercial constraints imposing the use of a limited set of cross section shapes, while design of reinforced concrete structures is less influenced due to use of multiple reinforcing bars. Furthermore, whatever is the material of construction, the overstrength of the structure is affected by the use in design of multiple load combinations (increased vertical loads without horizontal forces, reduced vertical loads with positive or negative horizontal forces according to EuroCode 8) [7], [11].

If frames are characterised by global collapse mechanisms, as it happens if the strong column-weak beam capacity design criterion is satisfied, the value of the collapse multiplier $O_S$ of seismic design forces may be calculated by means of the kinematic theorem of the limit analysis. In moment resisting frames fulfilling the above-mentioned design criterion, being known the collapse mechanism and the flexural strength of the sections where plastification is expected, the balance between the energy dissipated by plastic hinges and the work produced by the horizontal design forces due to rigid body displacements related to the assumed global mechanism leads to the following expression for the collapse multiplier of design seismic forces:

$$O_S = \frac{\sum_{i=1}^{n_c} M_{c,i1}^{B} + \sum_{k=1}^{n_s} \sum_{i=1}^{n_b} \left( M_{b,ik}^{L} + M_{b,ik}^{R} \right)}{\sum_{k=1}^{n_s} F_{d,k} \cdot h_k} \tag{1}$$

where $M_{b,ik}^{L}$ and $M_{b,ik}^{R}$ are the values of the flexural strength at the two ends of the $i^{th}$ beam at the $k^{th}$ storey, $M_{c,jk}^{B}$ the flexural strength at the bottom end of the $j^{th}$ column at the first storey and $h_k$ the height of the $k^{th}$ floor measured from the base of the frame.

The denominator of Eq. (1), which represents the overturning moment $M_{d,ov}$ of seismic forces, may be obtained by imposing the rotational equilibrium of the frame subjected to the seismic design forces and to the corresponding internal forces as:

$$M_{d,ov} = \sum_{k=1}^{n_s} F_{d,k} \cdot h_k = \sum_{i=1}^{n_c} M_{c,i1}^{B,0} + \sum_{k=1}^{n_s} \sum_{i=1}^{n_b} \left( M_{b,ik}^{L,0} + M_{b,ik}^{R,0} \right) \tag{2}$$

Consequently, by substituting Eq. (2) in Eq. (1) the collapse multiplier of the seismic design forces may be expressed as:

$$O_S = \frac{\sum_{i=1}^{n_c} M_{c,i1}^{B} + \sum_{k=1}^{n_s} \sum_{i=1}^{n_b} \left( M_{b,ik}^{L} + M_{b,ik}^{R} \right)}{M_{d,ov}} \tag{3}$$

It is important to point out that Eq. (3) does not require the knowledge of the seismic forces $F_{d,k}$. Therefore, it may be used also when a modal analysis is performed to evaluate the seismic response of frames.

The collapse multiplier $O_S$ evaluates to what extent the resisting base shear $V_u$, sum of the shear forces of the first order columns in the collapse condition, is percentually higher than the seismic base shear $V_{d,1}$ and, therefore, represents the *global overstrength* of the frame. Its value is always larger than unity. It would be equal to one only if the flexural strength of each cross-section were equal to the corresponding design bending moment produced by seismic actions. This is impossible because of the presence of vertical loads.

Furthermore, Eq. (3) can be re-formulated by separating the contributions of the different floor levels, i.e. defining the contribution of the first order columns (level 0), that of the first floor beams (level 1), etc. On the basis of such considerations, the resisting base shear $V_u$, which is obtained by the product of the collapse multiplier $O_S$ and the design base seismic shear force, can be written as:

$$V_u = \frac{V_{d,1}}{M_{d,ov}} \left[ \sum_{i=1}^{n_c} M_{c,i1}^{B} + \sum_{i=1}^{n_b} \left( M_{b,i1}^{L} + M_{b,i1}^{R} \right) + \cdots + \sum_{i=1}^{n_b} \left( M_{b,in_s}^{L} + M_{b,in_s}^{R} \right) \right] = \sum_{k=0}^{n_s} V_{u,k} \tag{4}$$

Eq. (4) expresses the shear force $V_u$ corresponding to the collapse as the sum of the contributions $V_{u,k}$ provided by the elements of each level. Contribution of first order columns is given by:

$$V_{u,0} = \frac{V_{d,1}}{M_{d,ov}} \sum_{i=1}^{n_c} M_{c,i1}^{B} \tag{5}$$

while that of the beams at the $k^{th}$ level is:

$$V_{u,k} = \frac{V_{d,1}}{M_{d,ov}} \sum_{i=1}^{n_b} \left( M_{b,ik}^{L} + M_{b,ik}^{R} \right) \tag{6}$$

Analogously to the above-mentioned global overstrength, we may define for the $k^{th}$ floor level the *storey overstrength* $O_{S,k}$ as the ratio of the shear force evaluated at that level in correspondence of the structural collapse to the shear force produced by the design seismic forces, evaluated at the same floor level of the frame. The storey overstrength is expressed by means of the following relationships for the first order columns:

$$O_{S,0} = \frac{V_{u,0}}{V_{u,0}^{0}} = \frac{\sum_{i=1}^{n_c} M_{c,i1}^{B}}{\sum_{i=1}^{n_c} M_{c,i1}^{B,0}} \tag{7}$$

and for the beams at $k^{th}$ level:

$$O_{S,k} = \frac{V_{u,k}}{V_{u,k}^{0}} = \frac{\sum_{i=1}^{n_b} \left( M_{b,ik}^{L} + M_{b,ik}^{R} \right)}{\sum_{i=1}^{n_b} \left( M_{b,ik}^{L,0} + M_{b,ik}^{R,0} \right)} \tag{8}$$

If the same procedure is repeated with reference to the generic single cross-section we obtain the *local overstrength* $O_{S,ik}^{E}$, defined as the ratio of the flexural strength to the bending moment required by the design seismic actions. In particular, the overstrength of the cross-section of $i^{th}$ element (beam or column) at $k^{th}$ level is:

$$O_{S,ik}^{E} = \frac{M_{ik}^{E}}{M_{ik}^{E,0}} \tag{9}$$

where $E$ indicates the end cross-section under examination (left or right for the beams and top or bottom for the columns).

## STRUCTURAL MODELS

In order to investigate the influence of the cross-section overstrength on the inelastic behaviour of plan irregular structures subjected to seismic actions, a regularly asymmetric multi-storey building is subjected to the action of a set of thirty artificially generated accelerograms. The structural response is normalised with respect to that of the corresponding torsionally balanced system, so that results of both asymmetric and torsionally balanced structures are directly comparable. Furthermore, such data are analysed together with those related to the normalised response of the corresponding asymmetric single storey-system, in order to evidence differences between the response of asymmetric multi and single-storey models.

The multi-storey model considered in this study (Fig. 1) represents a six-storey asymmetric building characterised by one symmetry axis ($X$-axis). The model is composed by rigid decks supported by steel frames arranged along an orthogonal grid. The floor diaphragms present the same geometry and the same distribution of mass at each level. In particular, decks are rectangular in plan and have plan dimensions, denoted as $B$ and $L$ in Figure 1, equal to 12.5m and 29.5 m, respectively. Masses are considered lumped into the decks and equal to 187.3 t at each floor. Their plan distribution is characterised by a mass centre $C_M$ lying on the $X$-axis at a distance of 0.15 $L$ from the geometrical centre $C_G$ of the deck and by a radius of gyration $r_m$ about $C_M$ equal to 9.2 m. The structure consists of 12 frames, constituted of inextensible and massless elements, and considered to provide stiffness and strength in their plane only. The vertical resisting elements (4 frames along the longitudinal direction and 8 along the transversal one) are symmetric with respect to the geometrical centre of the deck and have stiffness matrices mutually proportional. The rigidity centres of the different storeys $C_R$ are therefore lined up on the vertical axis passing through $C_G$ and, consequently, the structural eccentricity $e_s$ is equal to 0.15 $L$ at each floor. Furthermore, the structure satisfies the hypotheses given by Hejal and Chopra [10] and is regularly asymmetric. The cross-sections of the elements of the frames are selected so that the uncoupled torsional to lateral frequency ratio $\Omega_\theta$ has a unity value, representative of many actual buildings. Fundamental lateral periods $T_{x1}$ and $T_{y1}$ are both equal to 1.0 s.

Analogously to the previously described multi-storey system, the reference asymmetric single-storey system is composed of a rigid deck supported by resisting elements arranged along an orthogonal grid. It has both the same plan dimensions and distributions of mass and stiffness as the above-mentioned multi-storey system. The values of the total mass and lateral stiffnesses along the $X$ and $Y$-axes have been determined from the conditions that the uncou-

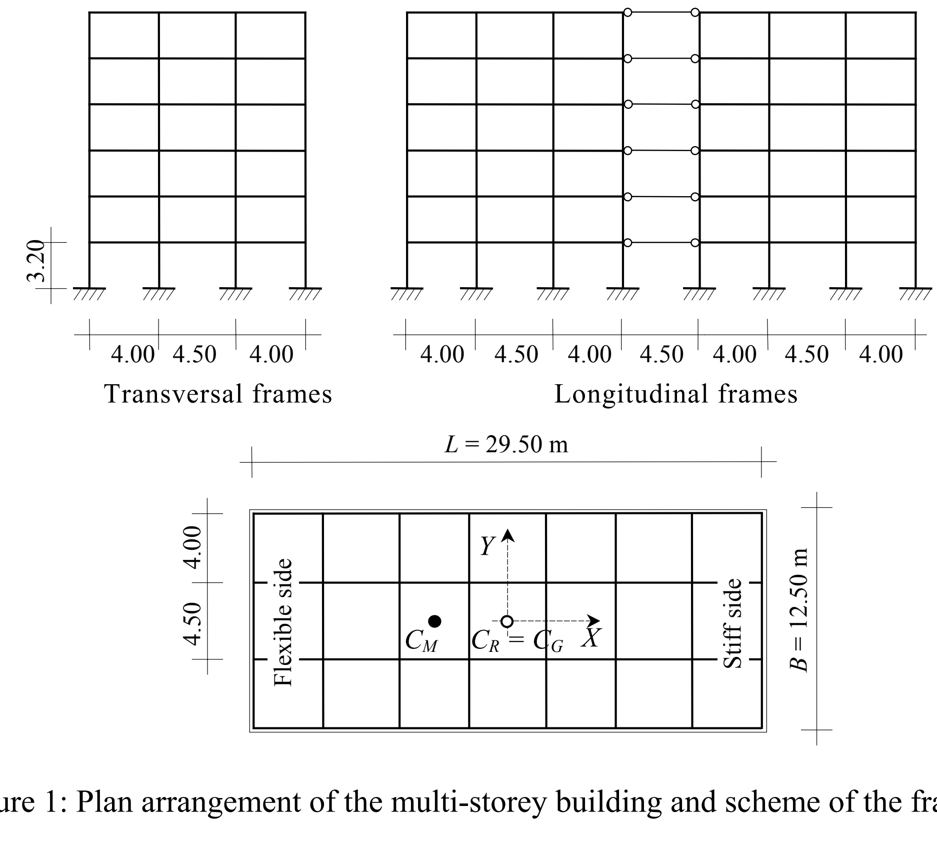


Figure 1: Plan arrangement of the multi-storey building and scheme of the frames.

pled torsional to lateral frequency ratio be equal to unity and that lateral periods $T_x$ and $T_y$ be equal to 1.0 s, as in the reference multi-storey building. The corresponding multi-storey and single-storey balanced systems are both obtained starting from the asymmetric systems, shifting the mass centres $C_M$ to the rigidity centres $C_R$.

## DESIGN OF ELEMENT STRENGTH

The strength of the resisting elements of the single-storey schemes (asymmetric and torsionally balanced) has been fixed by taking into account the effect of the seismic action only. Instead, for the multi-storey structures both the effects of vertical loads and seismic actions have been considered. In order to fulfil the *capacity design criterion,* yielding is allowed at the ends of the beams and at the bottom cross-sections of the first order columns only. The design bending moment of such cross-sections is determined as the maximum value corresponding to the two load conditions:

1. Vertical loads only. According to Eurocode 3 the design values of the permanent and variable loads (ultimate limit state) are obtained increasing the characteristic values by means of partial safety coefficients $\gamma_g$ and $\gamma_q$ equal to 1.4 and 1.5, respectively.
2. Seismic forces and reduced vertical loads.

Design seismic forces are represented by means of horizontal static forces having an inverted triangular distribution along the height of the building. The seismic base shear is evaluated as the product of the structure total mass by the spectral pseudo-acceleration related to the fundamental mode vibration of the corresponding balanced system. The pseudo-acceleration is obtained from the elastic spectrum proposed by Eurocode 8 for stiff soil, characterised by a peak ground acceleration equal to 0.35 g and reduced by a behaviour factor $q$ equal to 5. In the evaluation of the vertical loads assumed in the second load combination it is supposed that only 20% of the variable loads is present over the structure when the earthquake occurs.

The design of both single and multi-storey asymmetric systems is carried out in two different manners. In the first one, no torsional provision is imposed: consequently, seismic internal actions are evaluated by applying the horizontal forces in the floor mass centres $C_M$. In the second one, with the purpose of equating ductility demands of asymmetric buildings to those of the corresponding torsionally balanced systems, no reduction of strength is allowed with respect to that of the torsionally balanced structures: seismic internal actions are evaluated by means of two analyses in which the horizontal forces are applied at the mass centres $C_M$ of the decks and, subsequently, at the rigidity centres $C_R$.

### Plan-wise Distribution of the Overstrength in the Balanced Multi-Storey System

Although the flexural strength of the cross-sections has been fixed equal to the design bending moment, the vertical resisting elements of the multi-storey systems can resist seismic forces larger than those used in design. Indeed, owing to the use of different design load combinations, each frame has global and local overstrength. The value of these parameters depends on the importance of the intensity of the internal actions due to design seismic forces with respect to those due to design vertical loads. The overstrength increases as the effect of the vertical loads becomes more significant: being the flexural strength of the cross-section equal to the

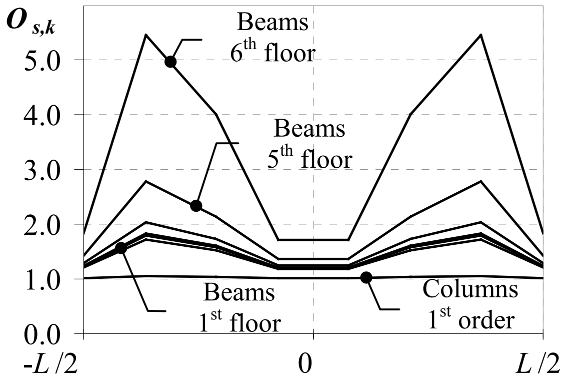


Figure 2. Plan-wise distribution of storey overstrength.

sum of the bending moments caused by vertical and seismic forces, Eq. (9) provides very large values when the effects of the vertical loads is large and that of the seismic actions is small.

By applying Eqs. (7) and (8) to the multi-storey torsionally balanced system, the storey overstrength $O_{S,k}$ of the vertical resisting elements acting along the $Y$-direction has been evaluated at each level. The obtained values are represented in Figure 2: information about the plan-wise distribution of the storey overstrength at k$^{th}$ level are described by the relevant curve, while information about the distribution of the overstrength along the height of the building is given by comparing the seven curves.

In the bottom cross-sections of the first order columns, the design vertical actions cause bending moments which are always slightly different from zero. Consequently, their flexural strength $M^{B}_{c,i1}$ are always close to the seismic bending moments $M^{B,0}_{c,i1}$ and, thus, their local overstrength, analytically represented by means of the ratio of the two above-mentioned bending moments, is substantially uniform and close to unity.

On the contrary, the design vertical loads and seismic actions induce similar internal actions in the beams of the examined structure and, thus, produce values of the local overstrength $O_{S,k}$, which are everywhere larger than unity. The plan-wise distribution of the overstrength $O_{S,k}$ is very variable. In particular, $O_{S,k}$ attains quite small values in the central and external frames, while larger values are reached in the other frames, later on named intermediate. Such trend is remarkably evident with reference to the top floor beams: indeed, for such elements the local overstrength is about 1.8 in the central and external frames while ranges from 5.4 to 4.0 in the intermediate ones.

The not uniform plan-wise distribution of the local overstrength of beams may be explained by examining the plan distribution of both vertical loads and frame stiffnesses. Due to the plan arrangement of frames (Figure 1), the vertical load of the beams, calculated by means of the tributary area concept, attains approximately the same values in the central and intermediate frames but about half the value in the external frames. The plan-wise distribution of the

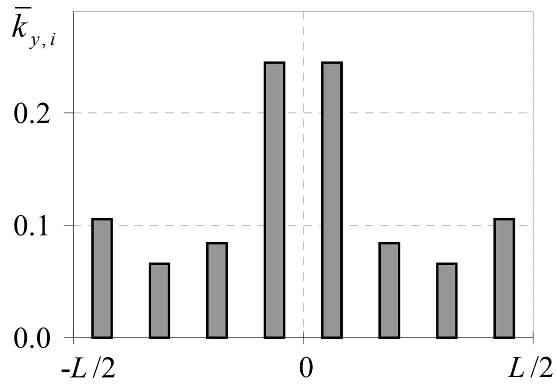


Figure 3. Plan-wise distribution of the stiffness.

lateral stiffness, instead, may be determined by considering that the stiffness matrix of the i$^{th}$ frame arranged along *Y*-direction may be written as:

$$\mathbf{K}_{y,i} = k_{y,i}\mathbf{K}, \qquad i = 1, 2, \ldots, 8 \tag{10}$$

where **K** is a reference stiffness matrix and $k_{y,i}$ a proportionality coefficient. The contribution of the frames to the stiffness of the building along Y-direction is shown in Figure 3 by means of the ratio, calculated as follows:

$$\bar{k}_{y,i} = \frac{k_{y,i}}{\sum_{n=1}^{8} k_{y,n}}, \qquad i = 1, 2, \ldots, 8 \tag{11}$$

On the basis of such results it is evident why the overstrength of the intermediate frames is much larger than that of the central frames (Figure 2). Indeed, while the bending moments produced by the design vertical loads are the same in both frames, the seismic internal actions caused by the seismic forces are higher in the central frames, characterised by larger lateral stiffness (Figure 3). On the contrary, the overstrength of the external frames is approximately equal to that of the central frames, in spite of their smaller stiffness (about the 10% of the total stiffness), because the design vertical loads of such frames are less than 50% of those acting on the other frames.

The comparison of the local overstrength related to different storeys points out that the parameter $O_{S,k}$ increases along the height of the building: while no significant overstrength is present in the first order columns, the local overstrength reaches the maximum value of 1.8 in the beams of the first floor and increases up to 5.0 at the top of the building. This trend may be explained if we observe that in the upper floors the seismic internal actions decrease, while no reduction is observed in the bending moments caused by the vertical loads. Consequently, the effect of the vertical loads becomes more important if compared to that of the seismic action, and therefore the local overstrength reaches larger values.

Strength of the Asymmetric Systems: torsional provisions not applied

If the seismic forces are applied at the mass centres of the asymmetric systems (single and multi-storey buildings), the resulting deck rotations modify the seismic internal actions of the resisting elements with respect to those of the corresponding torsionally balanced buildings. Such behaviour is evident from Figure 4a where, with reference to both single and multi-storey systems, the plan-wise distribution of the *normalised seismic moment* $\overline{M}_{S,i}$ is represented, i.e. the ratio of the seismic bending moment in the elements of the asymmetric system to that of the same elements in the corresponding torsionally balanced system. Along the plan dimension parallel to the $X$-axis, the normalised seismic bending moments vary according to a linear relationship and have the same values both in the single-storey system and at each floor of the multi-storey building. With respect to the reference torsionally balanced building, the seismic bending moments increase proportionally to the distance from the rigidity centres $C_R$ in the elements located on the flexible side of the building and decrease in those of the stiff side. The maximum increase due to the asymmetry, achieved in the outermost element of the stiff side, is about 77% of the seismic bending moment of the corresponding torsionally balanced system. Being the rigidity centres of the analysed building located in the midpoints of the decks, the maximum increase coincides with the maximum decrease.

If the design seismic bending moment of elements with no overstrength, e.g. belonging to the single-storey system, is increased, an equal increase in its flexural strength is obtained. But, when the same increase regards the design seismic bending moment of elements having already overstrength, e.g. elements of multi-storey buildings, the increase of the corresponding flexural strength will be less relevant than that of the design seismic internal action. Definitely, being the flexural strength provided by the sum of the seismic bending moment and that caused by the vertical loads, the larger is the rate of the gravitational actions, the larger is the overstrength and the smaller is the increase of the flexural bending moment. Obviously, such considerations may be repeated when a decrease of the seismic bending moment is imposed.

On the basis of such observations, the results shown in Figure 4b may be explained. In this figure, the plan-wise distribution of the *normalised plastic moment* $\overline{M}_{y,i}$ of the beams at each

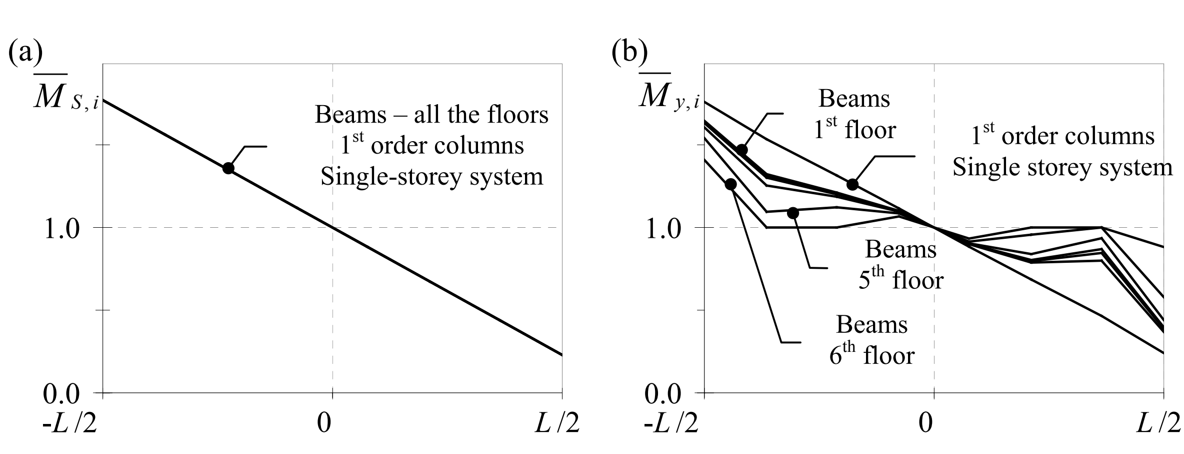


Figure 4. Normalised design bending moments: seismic (a) and plastic (b) bending moments.

floor and that of the bottom cross-sections of the first order columns is represented for both single and multi-storey asymmetric systems. The normalised plastic moment, defined as the ratio of the plastic moment of the elements of the asymmetric system to that of the same elements in the corresponding torsionally balanced system, determines the increase or decrease of the flexural strength of the elements because of asymmetry. The comparison between Figures 4a and 4b shows that the normalised plastic moment is everywhere equal to the normalised seismic moment in the single-storey system because there is no overstrength in the elements of such system. A quite different behaviour is instead observed in multi-storey systems where the trend of normalised plastic moment changes from one floor to the other. As the overstrength is substantially absent in the bottom cross-sections of the first order columns, the increase, as well as the decrease, of the flexural strength of these elements almost matches the one observed in the single-storey system (Figure 4b). In the beams, the variation of the plastic moment is no more linear along the *X*-direction and is always less relevant than that of the design seismic moment, particularly in the elements characterised by large overstrength. Apart from the central frames, in which the effect of the lateral-torsional coupling is obviously small because of their proximity to the rigidity centres axis, the smaller increases of the plastic bending moments are noticed in the beams of the intermediate frames located on the flexible side. In particular, at the sixth floor of such frames, in spite of the increase of the seismic internal actions, the flexural strength of the beams of the torsionally balanced and asymmetric structures is substantially the same.

Strength Distribution of the Asymmetric Systems: torsional provisions applied

As it is well known, the torsional component of the response developed by asymmetric structures in the occurrence of large inelastic deformations is less relevant than that developed in the elastic range of behaviour. Because of that, large ductility demands occur in the structural elements where, according to the elastic behaviour of the same structure, a strength reduction is allowed with respect to that of the corresponding torsionally balanced system. In order to avoid such unwanted effects, the asymmetric systems have been redesigned and a second analysis, in which the seismic design forces are applied in the rigidity centres of the structure, is carried out.

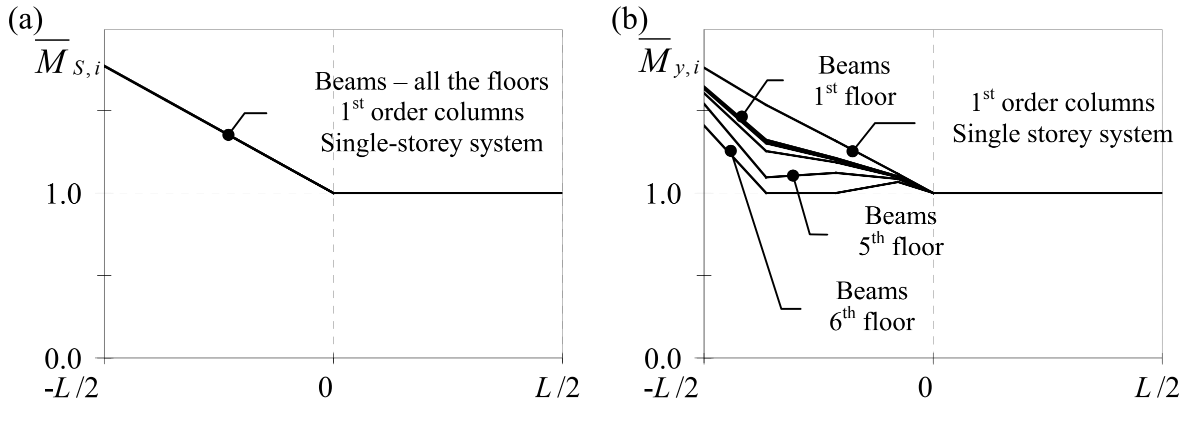


Figure 5. Normalised design bending moments: seismic (a) and plastic (b) bending moments.

The normalised design seismic bending moment, both in single and multi-storey asymmetric systems, still increases on the flexible side with the same trend observed when torsional provisions are not used. Instead, it is equal to one if referred to the elements on the stiff side of the structure (Figure 5a). As a consequence, no decrease is allowed in the strength of the resisting elements located on the stiff side of the structure (Figure 5b). Furthermore, since the second structural analysis does not influence the design seismic internal actions of the frames on the flexible side, the strength provided to such elements is the same as the previous case.

## NUMERICAL ANALYSES

Time-history analyses have been carried out for both asymmetric and torsionally balanced systems by means of the DRAIN-BUILDING computer program [14]. The inelastic response of such schemes has been evaluated under artificially generated accelerograms acting along the *Y*-direction. Each vertical resisting element has been idealised by means of one-dimensional members. Beams are modelled by plastic hinge elements. A very high strength is instead assigned to the cross-sections of the columns, apart from the ones at the bottom of the first order, where plastic hinges can develop. Consequently, yielding is allowed at all ends of the beams and at the bottom of the first order columns only. Such a choice, which implies the adoption of the capacity design criterion, also drastically reduces the computational cost. The interaction between bending moment and axial force in columns is neglected. Finally, a 5% viscous damping factor has been assumed in the analyses.

Input Ground Motions

Each system has been subjected along *Y*-direction to a set of thirty artificial accelerograms, matching the elastic response spectrum proposed by Eurocode 8 for stiff soil and characterised by a damping factor of 5%. The accelerograms, scaled 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.

Output parameters

As a result of the inelastic analyses, the maximum values of plastic rotations $\theta_p$ at the end cross-sections are obtained. Hence, member ductility demand *D*, which is defined as the ratio of maximum end rotation $\theta_{max}$ to yield rotation $\theta_y$, is evaluated as:

$$D = \frac{\theta_{max}}{\theta_y} = \frac{\theta_p + \theta_y}{\theta_y} \tag{12}$$

The yield rotation $\theta_y$ at the end cross-section of the generic element has been evaluated, with reference to a simply supported member having half element length, as the elastic rotation resulting from the application to the same end cross-section of a bending moment equal to yield moment. Finally, *normalised ductility demand d*, defined as the ratio of the member ductility demand *D* of the asymmetric system to that of the corresponding member of the torsionally balanced system, has been calculated. Such normalised parameter describes to what extent torsional response modifies inelastic behaviour of the system with respect to that of its symmetric counterpart. In order to obtain an estimate meaningful from statistical point of view, mean values $\overline{d}$ have been computed by averaging normalised ductility demand $d$ over the thirty considered records.

<u>Ductility demands for systems not designed with torsional provisions</u>
Plan-wise distribution of mean values $\overline{d}$ has been evaluated at each level of the multi-storey building (1st order columns, 1st floor beams, 2nd floor beams, etc.) and compared with that obtained for the single-storey scheme (Figures 6a and 6b).

Figure 6a shows that mean normalized ductility demand $\overline{d}_c$ of columns has the same trend for both single and multi-storey systems. In particular, $\overline{d}_c$ is smaller than unity for columns located at the flexible side and remarkably larger than unity for columns located at the building stiff side. Indeed, application of the seismic forces at the mass centres, eccentrically located with to respect to the rigidity centres, leads to an increase of the flexural strength of the columns disposed on the flexible side with a reduction in column ductility demands compared to those in the corresponding torsionally balanced systems. On the contrary, strength decrease is obtained in the stiff side elements, which give poor performance if they are well excited into the inelastic range.

A different behaviour, in terms of beams ductility demand, can be observed at each floor of the multi-storey structure (Figure 6b). In the lowest floors (from the $1^{st}$ to the $4^{th}$), plan-wise distribution of mean normalized ductility demand $\overline{d}_b$ has a similar trend, which slightly differs from that of the single-storey counterpart. On the contrary, substantial differences with respect to the results from the single storey system have been found for the upper floors ($5^{th}$ and $6^{th}$). Indeed, in the multi-storey system, the mean normalised ductility demand $\overline{d}_b$ of the stiff side beams shows values larger than unity, but considerably smaller than those of the single storey conterpart. The same parameter exceeds unity for the flexible side beams, contrary to results obtained from the single-storey model.

The above-described behaviour of the multi-storey asymmetric system may be explained if the overstrength distribution is examined. In the bottom cross-sections of the first order columns, because of the lack of overstrength, normalised ductility demand is close to that of the single storey system. In the $5^{th}$ and $6^{th}$ floor, application of lateral forces at the mass centres

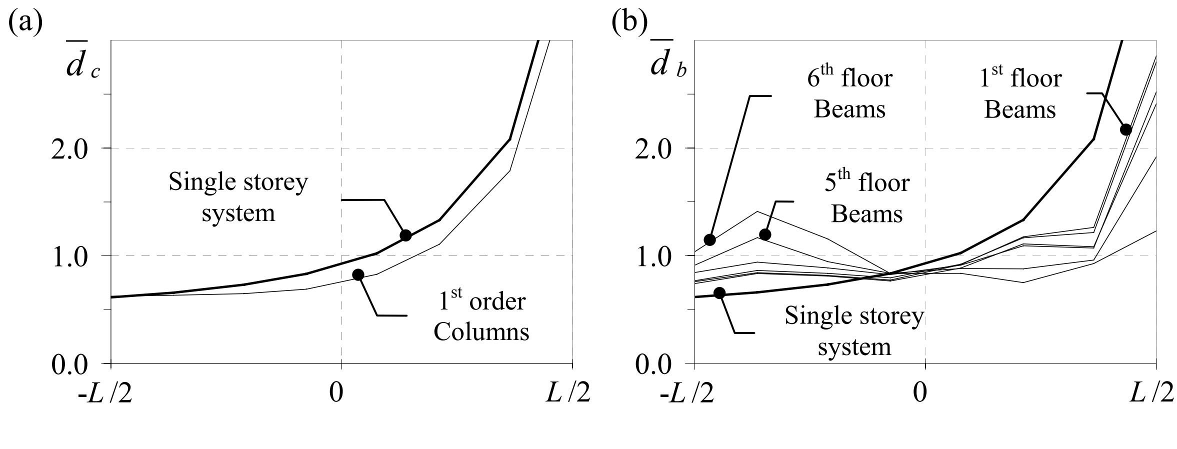


Figure 6. Normalised member ductility demand: (a) Columns; (b) Beams.

induce a reduction in design actions for the stiff side beams; however, because of the high overstrength of such elements (due to the vertical loads), reductions in their flexural strength is not significant. As a consequence, the poor performance characterising stiff side elements of the single-storey scheme is not observed in the upper floors of the multi-storey building. In the flexible side, instead, the presence of overstrength in the beams of the upper floors does not allow the needed increase in strength and, therefore, it induces large ductility demands in such elements, contrary to predictions from the single-storey scheme. Finally, the significant - but less important than that at the upper floors - overstrength of intermediate floors explains their intermediate behaviour.

Ductility demands for systems designed with torsional provisions

Torsional provisions are specified in all major seismic codes in order to obtain ductility demands in plan-asymmetric systems similar to those in their symmetric counterparts. Studies carried out on single-storey schemes show that this goal is usually achieved for elements on the stiff side, by precluding any reduction of their design seismic actions with respect to the values computed for the corresponding torsionally balanced systems (no-reduction rule).

If such torsional provisions are applied, ductility demands of asymmetric single-storey systems appear rather close to those of the corresponding torsionally balanced systems (Figure 7a and 7b). Analogously, first order columns of the multi-storey building benefit from the no-reduction rule. The obtained strength distribution avoids large values of ductility demand on the stiff side (Figure 6a), reducing the normalised parameter $\overline{d}_c$ everywhere to values lower than unity (Figure 7a). Similar considerations may be repeated for the beams of the lowest floors, while an unexpected behaviour characterises those of the upper floors. Due to the application of the no-reduction rule, the normalised ductility demand $\overline{d}_b$ is close to unity on the stiff side but still larger than unity in the elements of the flexible side (Figure 7b).

Results shown in Figures 7a and 7b confirm the importance of effects of the overstrength on the inelastic seismic behaviour of the plan irregular multi-storey buildings. Furthermore, such results demonstrate that overstrength can undermine effectiveness of code torsional provisions. It has been shown that the introduction of the additional second design analysis (trans-

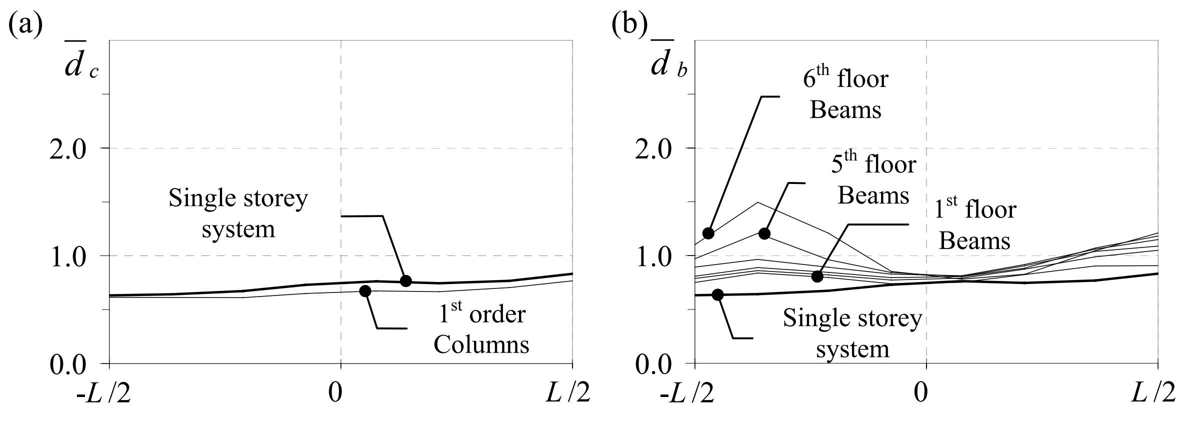


Figure 7. Normalised ductility demand: (a) Columns; (b) Beams.

lational analysis), which avoids any strength reduction in the stiff side elements, reduces their ductility demands up to values close to those of the torsionally balanced systems. However, it is almost unable to improve seismic performances of the beams of the upper floors located on the flexible side of the structure. Indeed, the poor behaviour exhibited by such elements is not related to the second design translational analysis, but it is due to the presence of a large overstrength, which does not allow the necessary increase of strength in the beams located on the flexible side of upper floors (Figure 5).

## CONCLUSIONS

In this paper, the effects of overstrength, always present in real multi-storey building structures, on the seismic behaviour of plan-irregular buildings have been investigated. In particular, the inelastic seismic response of a multi-storey asymmetric system, designed to sustain gravity loads and seismic forces, has been evaluated and compared to that of the corresponding single-storey scheme, in which no overstrength is present. The results show that in the analysed multi-storey asymmetric system, because of overstrength, ductility demands may become larger at unexpected locations. Namely, in the upper floors of the analysed asymmetric building, where overstrength reaches very large values, ductility demands attain the largest values on the flexible side, while not exceeding those characterising the corresponding balanced system on the stiff side elements, in contrast to predictions generally derived from the corresponding single-storey system. As a consequence, unless overstrength is properly accounted for, torsional provisions derived from studies of single-storey systems, aimed at limiting the normalised ductility demands to values close to unity, may fail their goal in multi-storey buildings.

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