

# Improved earthquake resistant design of torsionally stiff asymmetric steel buildings

M.T. Kyrkos and S.A. Anagnostopoulos\*

*Department of Civil Engineering, University of Patras, 26500 Patras, Greece*

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**Abstract.** In a companion paper as well as in earlier publications, it has been shown that in asymmetric frame buildings, designed in accordance with modern codes and subjected to strong earthquake excitations, the ductility demands at the so called “flexible” edges are consistently and substantially higher than the ductility demands at the “stiff” edges of the building. In some cases the differences in the computed ductility factors between elements at the two opposite building edges exceeded 100%. Similar findings have also been reported for code designed reinforced concrete buildings. This is an undesirable behavior as it indicates no good use of material and the possibility for overload of the “flexible” edge members with a consequent potential for premature failure. In the present paper, a design modification will be introduced that can alleviate the problem and lead to a more uniform distribution of ductility demands in the elements of all building edges. The presented results are based on the steel frames detailed in the companion paper. This investigation is another step towards more rational design of non-symmetric steel buildings.

**Keywords:** asymmetry; eccentricity; torsion; multistory steel buildings; braces; earthquake; inelastic response; plastic hinge model.

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## 1. Introduction

Code provisions for torsion have been largely based either on elastic analyses of single and multi story building models or on inelastic analyses of simplified, one story, shear beam type idealizations. In fact, more than 90% of the publications for inelastic response of non symmetric buildings to strong earthquake motions are based on oversimplified one-story building models of the shear beam type. Results from such studies have been used extensively to assess the produced designs in relation to the code provisions for torsion (Chandler and Duan 1991, 1997, Chopra and Goel 1991, Tso and Zhu 1992, Tso and Smith 1999, Tso and Myslimaj 2003, Correnza *et al.* 1995, De Stefano *et al.* 1998, Goel 1997, Humar and Kumar 1999, De-La-Colina 1999, Gherzi and Rossi 2006, Rutenberg 1998, 2002, De Stefano and Pintucchi 2008). However, in the past ten years more sophisticated multi-story inelastic models have been used to study this problem (Gherzi *et al.* 2000, Stathopoulos and Anagnostopoulos 2003, 2005, Fajfar *et al.* 2004, Marusic and Fajfar 2005). Results from such models were found to be considerably different from those based on the simplified ones. The differences were not just quantitative but qualitative as well. For example,

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\* Corresponding author, Professor, E-mail: [saa@upatras.gr](mailto:saa@upatras.gr)

while many studies with the simplified, one-story models indicated as the critical elements in non symmetric, code designed buildings the elements at their “stiff” edges, analyses of detailed, multi-story, plastic hinge type models gave the opposite results, i.e indicated as critical elements those at the “flexible” edges (Stathopoulos 2001, Stathopoulos and Anagnostopoulos 2005). In a more recent study (Anagnostopoulos *et al.* 2010) it was demonstrated that one of the sources of the observed differences between simplified and detailed models is in the strengths assigned to the elements of the simplified models. In almost all the publications using the simplified model, the element strengths have been determined only on the basis of the earthquake loading, neglecting gravity loads, interstory drift limitations, capacity design provisions and other code requirements applied in the design of real buildings and reflected in the properties of the realistic models. In the same study, it was shown that the simplified models could also predict the same trends, provided that their member properties were determined not in the traditional manner, i.e just from the earthquake loading alone, but from the member properties of the actual buildings.

The above cited studies were based on reinforced concrete (RC), eccentric frame buildings. To see if the behavior observed therein represents a design shortcoming, the same type of study was carried out using eccentric steel frames designed according to the Eurocodes 3 and 8 for steel and earthquake resistance (EC3, EC8, 2004). Details and results of this study are given in a companion paper (Kyrkos and Anagnostopoulos 2010a), where a behavior similar to that of the eccentric RC frame buildings is reported. It is obviously undesirable to have one side of the building experience consistently higher ductility demands than the other, not only because this may imply material waste but also because it points to the possibility for overload of the “flexible” edge members that can lead to premature failure. This suggests the need for a design modification that could alleviate this problem. Such a modification is investigated in the present paper for the buildings used in the companion paper by Kyrkos and Anagnostopoulos (2010a). Preliminary results for a slightly different set of steel buildings have been reported in Kyrkos and Anagnostopoulos (2010b). For the torsionally stiff buildings used in this study, the proposed modification improves substantially the building performance under strong earthquake motions and can be easily implemented into the existing codes.

## 2. Buildings and motions used

The buildings and earthquake motions used in this study are the same described in the companion paper (Kyrkos and Anagnostopoulos 2010a). For this reason, only a brief description will be given here for ease of reference. Three groups of buildings have been used: with one, 3 and 5 stories. In each group different designs were produced with initial mass eccentricities of 0.10 and 0.20, which led to buildings with biaxial mass and stiffness eccentricities as well. For comparisons, a torsionally balanced building was also designed in each group. The layout is the same for all floors of all three buildings and may be seen in Fig. 1. In the same figure the elevations of the two sides of the 3-story building are also shown. Each building is formed by 4 frames along the  $x$  axis, (FR-1X to FR-4X) and 6 frames along the  $y$  axis (FR-1Y to FR-6Y). The exterior frames on each side have braces either in the middle bay or in two bays symmetric about the middle. All buildings have a typical story height of 3.00 m and ground story height 4.00 m. The models used for both design and analyses were 3-D models with masses lumped at the joints.

All buildings were designed as spatial frames for gravity and earthquake loads using the dynamic,

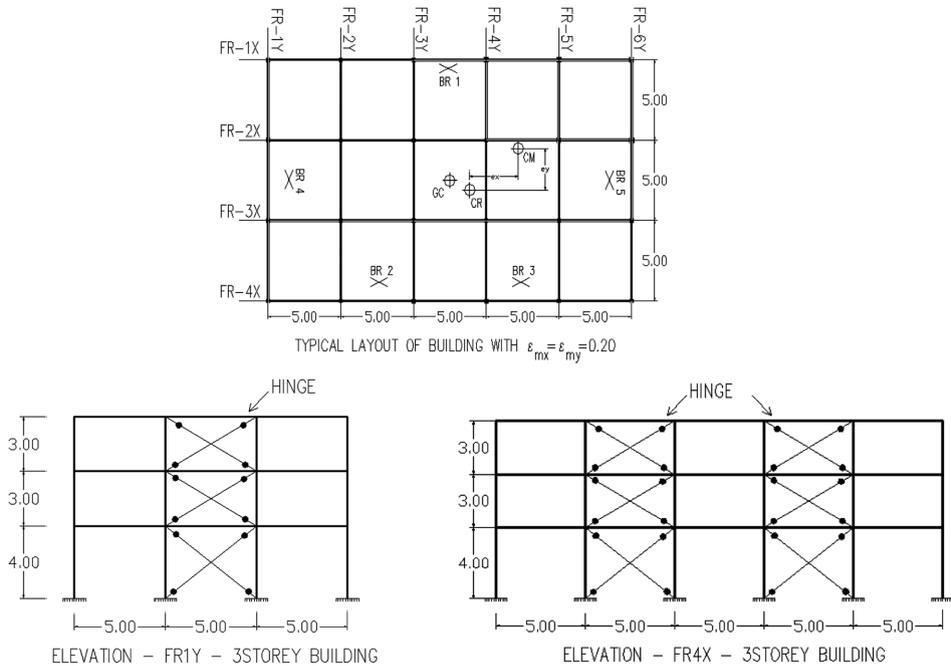


Fig. 1 Layout of 1, 3 and 5-storey steel buildings and x, y elevations of the 3-storey buildings

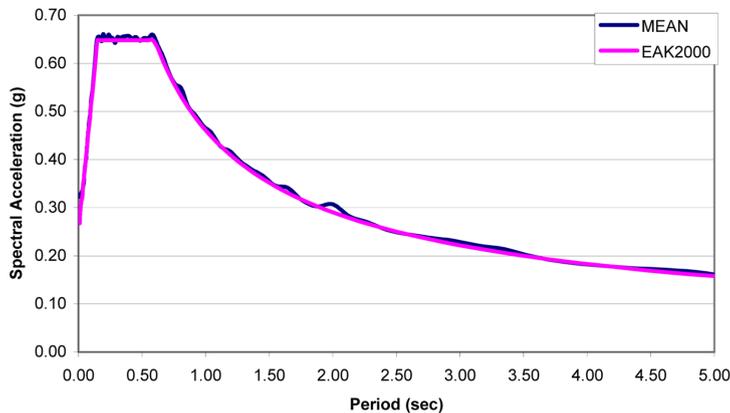


Fig. 2 Design spectrum for Greece and mean response spectrum of 10 semi artificial motions

response spectrum method, according to Eurocodes EC3 (steel structures), EC8 (earthquake resistant design) and the Greek Earthquake Resistant Design Code 2000. The design spectrum is shown in Fig. 2 for  $PGA = 0.24\text{ g}$ , along with the mean spectra of the motions used for subsequent evaluations.

The lowest, fundamental, periods of the symmetric building in each set are:  $T_y = 0.35\text{ s}$  (1-story),  $T_y = 0.58\text{ s}$  (3-story) and  $T_y = 0.94\text{ s}$  (5-story). The complete set of the lowest three periods and the resulting physical eccentricities  $\varepsilon_x$  and  $\varepsilon_y$  for all buildings have been presented in the companion paper. It is noted once again that for all models, the first torsional period is lower than the two translational periods, so all buildings are torsionally stiff.

The non linear analyses were carried out using the program RUAUMOKO (Carr 2005). Each of

the examined buildings, modeled as a non linear 3-D frame, was subjected to ten sets of two component semi-artificial motion pairs of biaxial motions. These motions were generated from a group of five, two-component, real earthquake records, to closely match the code design spectrum, using a method based on trial and error and Fourier transform techniques (Karabalis *et al.* 1994).

As Fig. 2 indicates, the mean response spectrum of the ten semi-artificial motions is quite close to the target design spectrum, a fact that eliminates the differences between design and applied actions as a potential source of any observed undesirable response. Each synthetic motion pair, derived from the two horizontal components of each historical record, was applied twice by mutually changing the components along the  $x$  and  $y$  system axes. Thus, each design case was analyzed for ten sets of 2-component motions and mean values of peak response indices were computed. In this manner, the effects of individual motions are smoothed and the conclusions become less dependent on specific motion characteristics.

### 3. Modification procedure

In general, a structural design for any loading case can be characterized as satisfactory, when the peak values of the controlling response parameters for a given performance level are within the specified limits and also do not exhibit wide variations in the group of structural members considered. Thus, for a no-collapse limit state, if the ductility demands of structural members in the same group of members remain within acceptable limits and the variation of their values is small, the design succeeds in both goals: structural safety and cost effectiveness. In the opposite case, sub optimal use of material may be present as well as a potentially higher risk of failure in cases of unexpected overloads.

The results reported in previous studies (Stathopoulos and Anagnostopoulos 2005, Kyrkos and Anagnostopoulos 2010a), indicate a consistent differences in ductility demands between “stiff” and “flexible” edges of the eccentric buildings. As an example, we reproduce here in Fig. 3 ductility demands (mean values of peak demands for the 10 earthquake pairs), from the companion paper by Kyrkos and Anagnostopoulos (2010a). These correspond to braces and beams at the stiff and flexible edges of the 5-story steel building with initial mass eccentricity of 0.20. The substantial differences in such demands between the two opposite edges of the building are apparent and point to the need for a design modification that would eliminate or reduce these differences. Such a modification is implemented in the present paper on the basis of results obtained from the elastic analyses of the buildings. It aims at increasing the strength of the members at the flexible edge and reducing the strength at the stiff edge without substantially affecting the yield deformations from which ductility factors are computed.

The first step for application of this modification is to obtain the top story displacements at the “flexible” and “stiff” edges of the buildings in both horizontal directions due to the earthquake loading considered and then compute the following factors in each horizontal direction

$$\left. \begin{aligned} f_{i,flex} &= \frac{u_{i,flex}}{(u_{i,flex} + u_{i,stiff})/2} \\ f_{i,stiff} &= \frac{u_{i,stiff}}{(u_{i,flex} + u_{i,stiff})/2} \end{aligned} \right\} \quad (1)$$

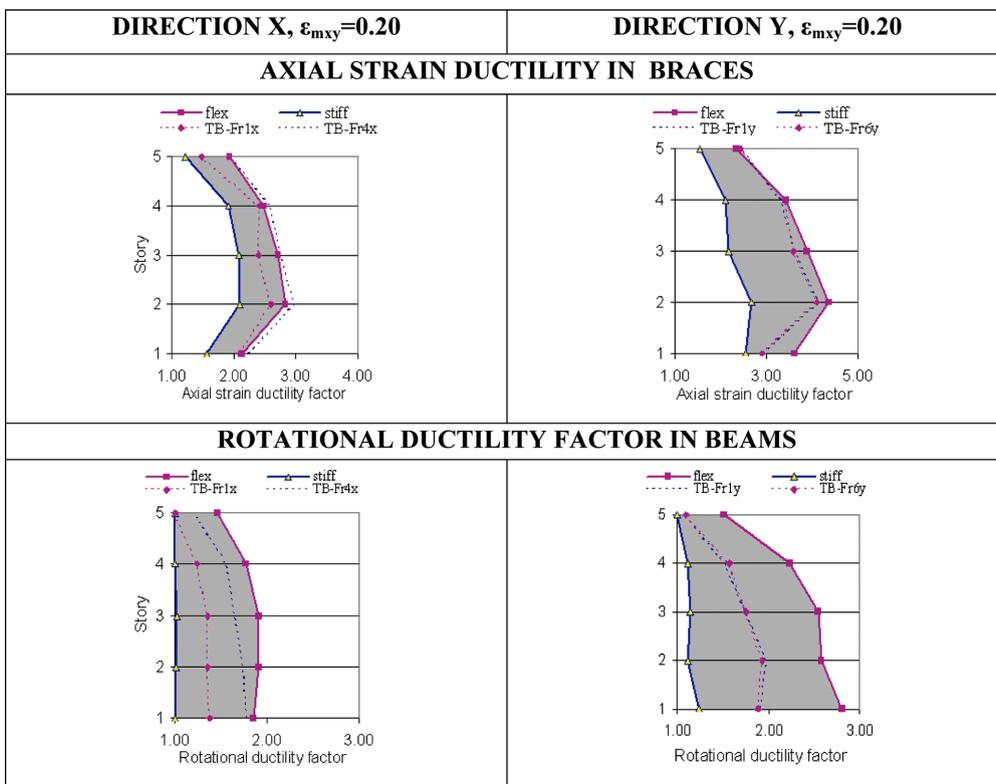


Fig. 3 Member ductility demands of 5-story buildings with  $\epsilon_m = 0.20$  and comparison with torsionally balanced (TB) building

where:  $u_{i,flex}$  is the top story displacement of the “flexible” edge in the  $i$ -direction and  $u_{i,stiff}$  is the top story displacement of “stiff” edge. These factors are the ratios of the top story displacements at the flexible and stiff edges in a given horizontal direction ( $x$  or  $y$ ), to their mean value.

The next step is to resize the bracing members at the edge frames by multiplying their axial areas by the corresponding factors in each direction. Here the modification is restricted to the bracing members since they provide most of the earthquake resistance. However, there is no reason not to include the beams and perhaps the columns of the edge frames in this modification. This is a topic still under investigation. Note that the value of  $f_{i,flex}$  is always greater than unity whereas the value of  $f_{i,stiff}$  is always less than unity. For the buildings used herein, the aforementioned factors vary from 1.10 to 1.40 for the flexible edges ( $f_{i,flex}$ ) and from 0.60 to 0.90 for the stiff edges ( $f_{i,stiff}$ ). Following this modification, new analyses are carried out and all the design checks and code required verifications are repeated. In general, only minor member resizing was required as a result of this modification. Note that as with the original buildings, the modified versions satisfy the code requirements of both the EC 8 and the Greek Code for Earthquake Resistant Design, in the very few cases that the two codes specify slightly different values for a design parameter.

Having now the new structures satisfying all the code requirements, inelastic dynamic analyses were carried out as described earlier to compute ductility demands of the buildings and to assess the effectiveness of the design modification just described.

Table 1 Eccentricities and fundamental periods of the modified buildings

Story number	Mass eccentricity	Mean natural eccentricity		Fundamental periods of buildings (sec)		
	$\varepsilon_{mx} = \varepsilon_{my}$	$\varepsilon_x$	$\varepsilon_y$	$T_y$	$T_x$	$T_\theta$
1	0.10	0.00	0.04	0.35	0.29	0.20
	0.20	0.01	0.10	0.34	0.29	0.20
3	0.10	0.00	0.05	0.56	0.49	0.33
	0.20	0.01	0.12	0.56	0.49	0.33
5	0.10	0.02	0.05	0.93	0.81	0.54
	0.20	0.02	0.11	0.92	0.80	0.52

#### 4. Results for the modified structural designs

The lowest, fundamental, periods of the modified building in each set are presented in Table 1, along with the new physical eccentricities  $\varepsilon_x$  and  $\varepsilon_y$  corresponding to the initial mass eccentricities  $\varepsilon_{mx} = \varepsilon_{my}$  (0.10 and 0.20). If we compare these numbers with the corresponding numbers of the original, unmodified, buildings in Kyrkos and Anagnostopoulos (2010a), we will see that the 3 lowest periods of each building have practically remained unaffected (changes on the order of 1%-2%) while the physical eccentricities have been reduced significantly. Obviously this is quite beneficial because it will decrease the torsional motion of each building accordingly.

The new modified structures were again subjected to the same two component earthquake groups and their responses were computed as before. The same indices used to assess the inelastic response of the original, unmodified, designs are also used here for direct comparisons (see Kyrkos and Anagnostopoulos 2010a). They are top story displacements, interstory drifts and ductility factors of beams and bracing members, all computed for the stiff and flexible edge frames as mean values of the peak response parameters over the ten pairs of applied motions. For braces the ductility factors are based on inelastic elongations and for beams on plastic hinge rotations. For interpreting the results we must note that due to the elongated shape of the building layouts ( $L_y/L_x = 0.60$ ), the end frames parallel to the  $y$  axis are affected more by torsion than the end frames along the  $x$  axis.

##### 4.1 One- story buildings

Results for the one story frames for both the initial structures and the modified models are presented in Tables 2-4. Ductility factors are considered only for braces and beams, given that columns remain essentially elastic. Looking into Table 2, we see that for both eccentricities, the modified buildings exhibit substantially smaller differences in the peak displacements between their “flexible” and “stiff” edges, compared to those in the original designs. In fact, in the new designs, the “flexible” edge displacements have decreased while those of the “stiff” edges increased as a result of reduced torsional motion. This is of course a consequence of the reduced physical eccentricities that the design modification produced. Similar improvement can be seen in the ductility factors of both braces and beams. The larger ductility demands at the flexible edges have decreased and those at the stiff edge members increased, thus making them more uniformly distributed throughout the building. The changes are more pronounced in the bracing members that are in the planes parallel to the  $y$  axis, since these members are affected more by torsion.

Table 2 Edge displacements for 1-story frames

INITIAL DESIGN		MODIFIED DESIGN		
DIRECTION Y				
ECCEN-TRICITY	FLEX-EDGE (FR6Y)	STIFF- EDGE (FR1Y)	FLEX-EDGE (FR6Y)	STIFF- EDGE (FR1Y)
$\varepsilon = 0.10$	<b>0.03573</b>	<b>0.02525</b>	<b>0.03016</b>	<b>0.02850</b>
$\varepsilon = 0.20$	<b>0.03622</b>	<b>0.01947</b>	<b>0.02860</b>	<b>0.02457</b>
DIRECTION X				
ECCEN-TRICITY	FLEX-EDGE (FR1X)	STIFF- EDGE (FR4X)	FLEX-EDGE (FR1X)	STIFF- EDGE (FR4X)
$\varepsilon = 0.10$	<b>0.01908</b>	<b>0.01530</b>	<b>0.01756</b>	<b>0.01606</b>
$\varepsilon = 0.20$	<b>0.01976</b>	<b>0.01299</b>	<b>0.01691</b>	<b>0.01308</b>

Table 3 Brace ductility factors for 1-story Frames

INITIAL DESIGN		MODIFIED DESIGN		
BRACES-DIRECTION Y				
ECCEN-TRICITY	FLEX-EDGE (FR6Y)	STIFF- EDGE (FR1Y)	FLEX-EDGE (FR6Y)	STIFF- EDGE (FR1Y)
$\varepsilon = 0.10$	<b>3.80</b>	<b>2.67</b>	<b>3.18</b>	<b>3.04</b>
$\varepsilon = 0.20$	<b>3.86</b>	<b>2.05</b>	<b>3.02</b>	<b>2.62</b>
BRACES-DIRECTION X				
ECCEN-TRICITY	FLEX-EDGE (FR1X)	STIFF-EDGE (FR4X)	FLEX-EDGE (FR1X)	STIFF- EDGE (FR4X)
$\varepsilon = 0.10$	<b>1.99</b>	<b>1.60</b>	<b>1.81</b>	<b>1.69</b>
$\varepsilon = 0.20$	<b>2.07</b>	<b>1.35</b>	<b>1.75</b>	<b>1.37</b>

Table 4 Beam ductility factors for 1-story Frames

INITIAL DESIGN		MODIFIED DESIGN		
BEAMS-DIRECTION Y				
ECCEN.	FLEX-EDGE (FR6Y)	STIFF- EDGE (FR1Y)	FLEX-EDGE (FR6Y)	STIFF-EDGE (FR1Y)
$\varepsilon = 0.10$	<b>1.08</b>	<b>1.00</b>	<b>1.03</b>	<b>1.00</b>
$\varepsilon = 0.20$	<b>1.58</b>	<b>1.00</b>	<b>1.47</b>	<b>1.00</b>
BEAMS-DIRECTION X				
ECCEN.	FLEX-EDGE (FR1X)	STIFF- EDGE (FR4X)	FLEX-EDGE (FR1X)	STIFF-EDGE (FR4X)
$\varepsilon = 0.10$	<b>1.06</b>	<b>1.02</b>	<b>1.03</b>	<b>1.02</b>
$\varepsilon = 0.20$	<b>1.25</b>	<b>1.00</b>	<b>1.16</b>	<b>1.00</b>

#### 4.2 Three story buildings

Total and interstory displacements for the “flexible” and “stiff” edge frames of the modified 3-story buildings are shown in Figs. 4 and 5 for biaxial mass eccentricities  $\varepsilon_m = 0.10$  and 0.20, respectively. In the same figures, the corresponding results of the original designs are also shown

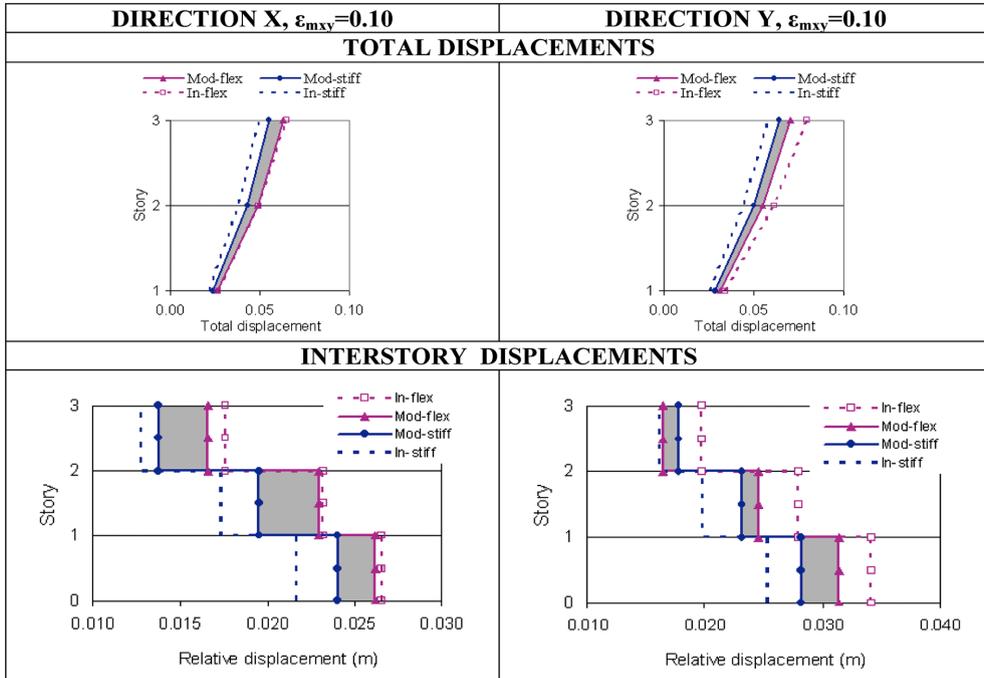


Fig. 4 Comparison of total displacements and interstory drifts of 3-story buildings with  $\epsilon_m = 0.10$ , for the initial and modified designs

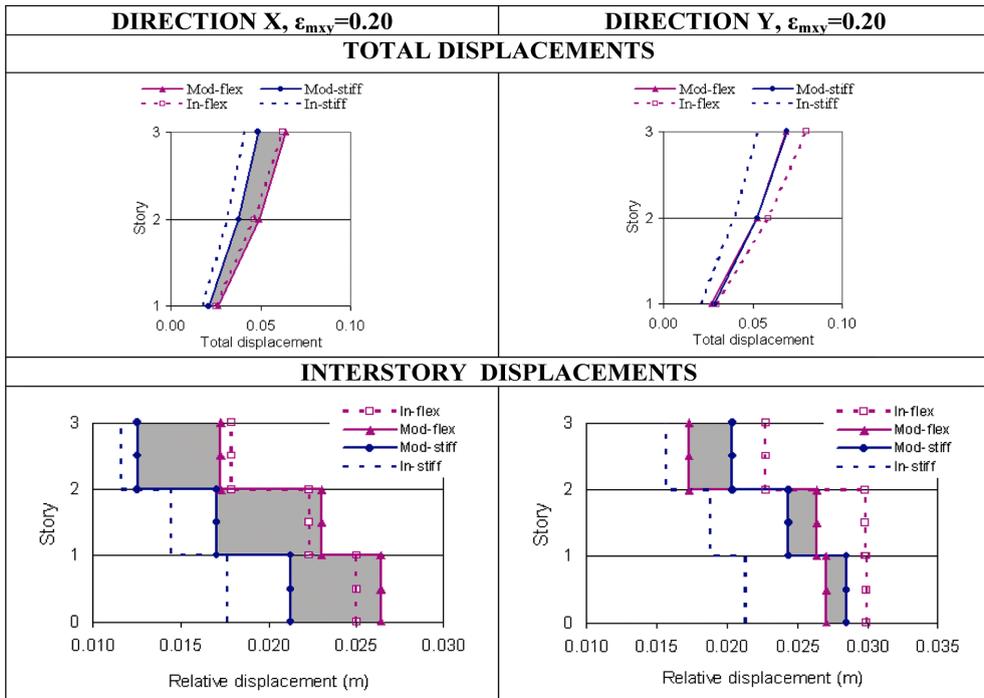


Fig. 5 Comparison of total displacements and interstory drifts of 3-story buildings with  $\epsilon_m = 0.20$ , for the initial and modified designs

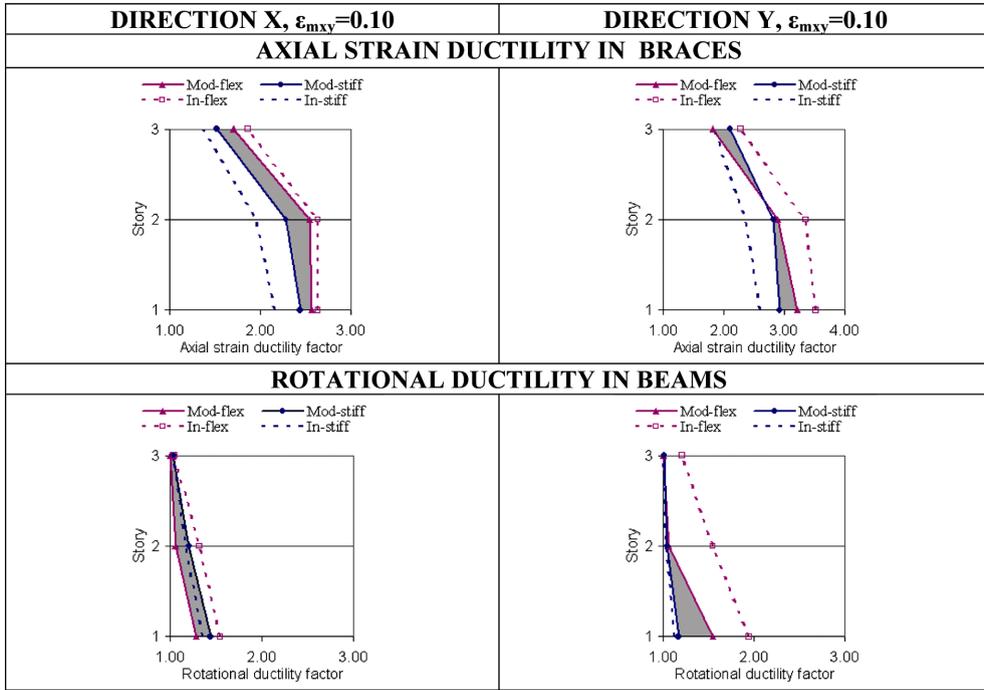


Fig. 6 Comparison of member ductility demands of 3-story buildings with  $\epsilon_m = 0.10$ , for the initial and modified designs

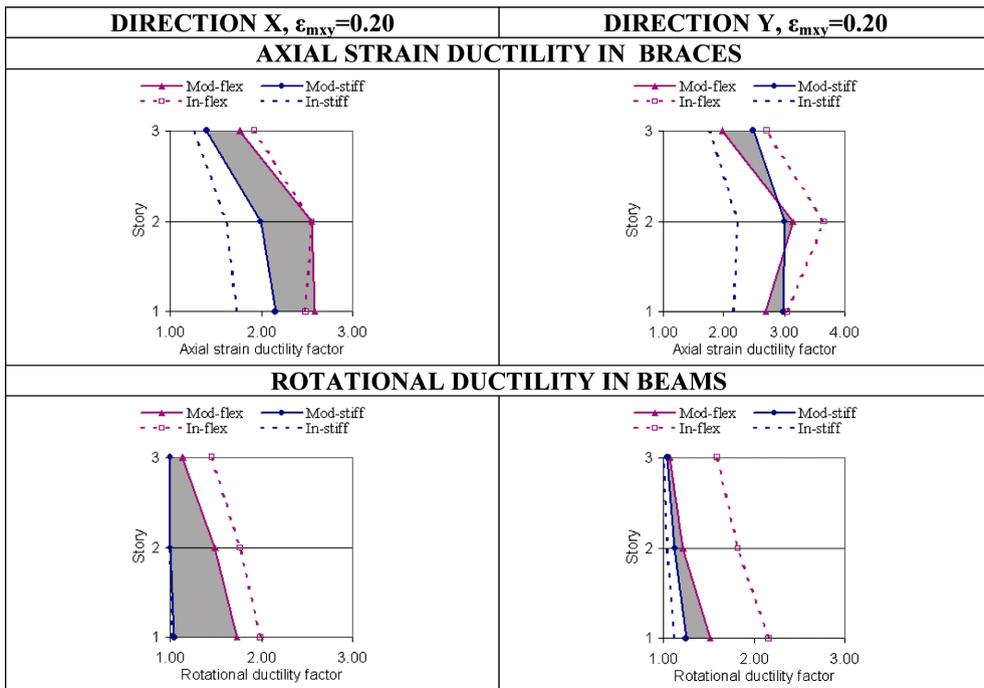


Fig. 7 Comparison of member ductility demands of 3-story buildings with  $\epsilon_m = 0.20$ , for the initial and modified designs

with dashed lines for comparison. In similar arrangement, ductility demands for braces and beams are also shown in Figs. 6 and 7. As with the one story buildings, the design modification has brought the approximate stiffness centers of the building floors closer to the mass centers, thus the physical eccentricities were reduced and so did the torsional motion. The result was, in general, a motion reduction at the flexible edge and an increase at the stiff edge, with a consequent effect on the interstory drifts and ductility demands, which now have become more uniform through substantial reductions in the “flexible” edge members. We note again that the effects of the applied design modification are much more pronounced in the edge frames parallel to the  $y$  axis direction because of their greater sensitivity to torsion due to the larger dimension of the building along the  $x$  axis.

### 4.3 Five story buildings

Results for the 5-story buildings are shown in Figs. 8-11, in the same arrangement as before. The improved behavior and the trends observed for the one and three story modified buildings are repeated here too, while comments and observations made for the one and three-story buildings apply to the five story buildings as well. In short, the design modification has also improved the inelastic behavior of the 5-story asymmetric buildings.

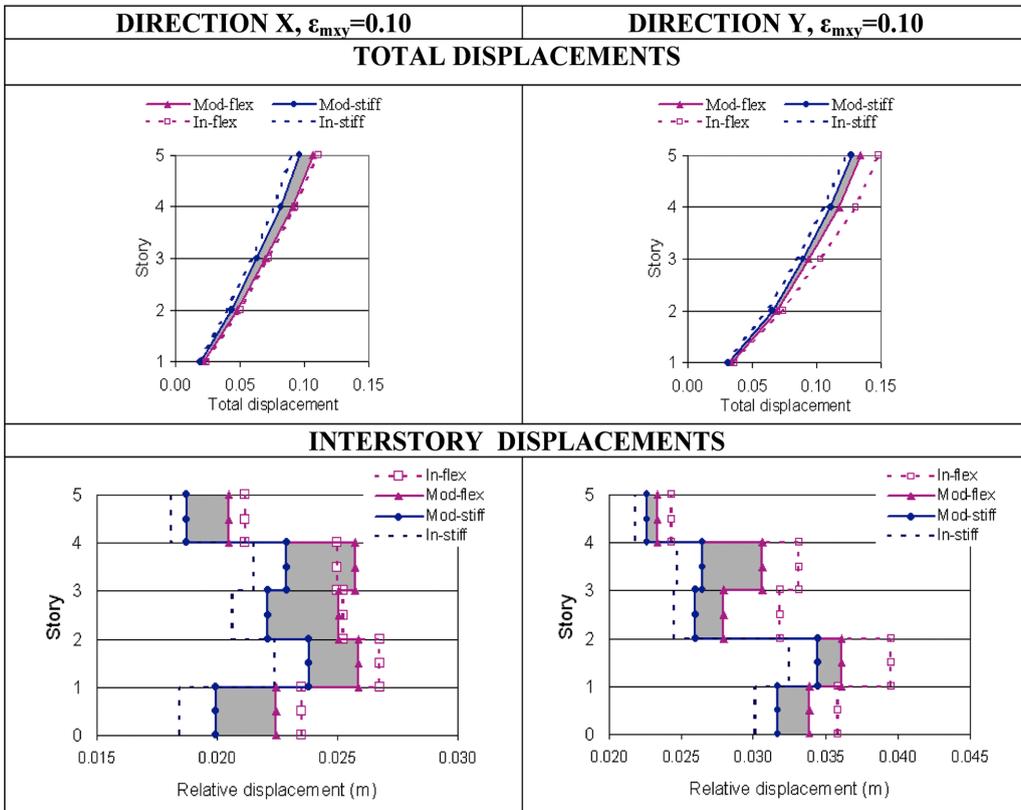


Fig. 8 Comparison of total displacements and interstory drifts of 5-story buildings with  $\epsilon_m = 0.10$ , for the initial and modified designs

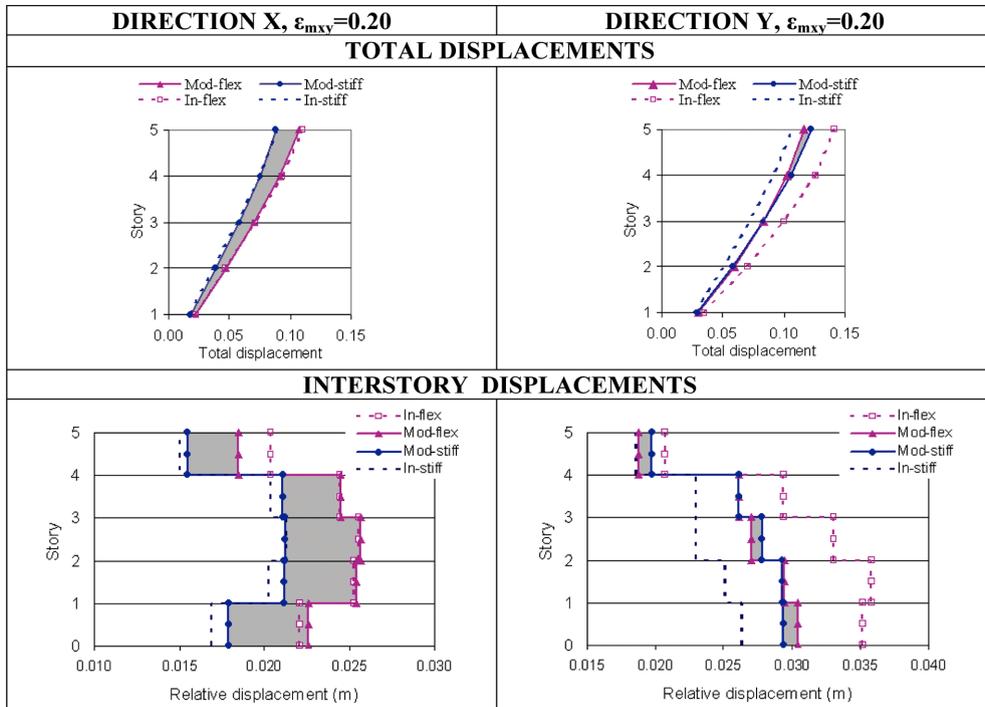


Fig. 9 Comparison of total displacements and interstory drifts of 5-story buildings with  $\epsilon_m = 0.20$ , for the initial and modified designs

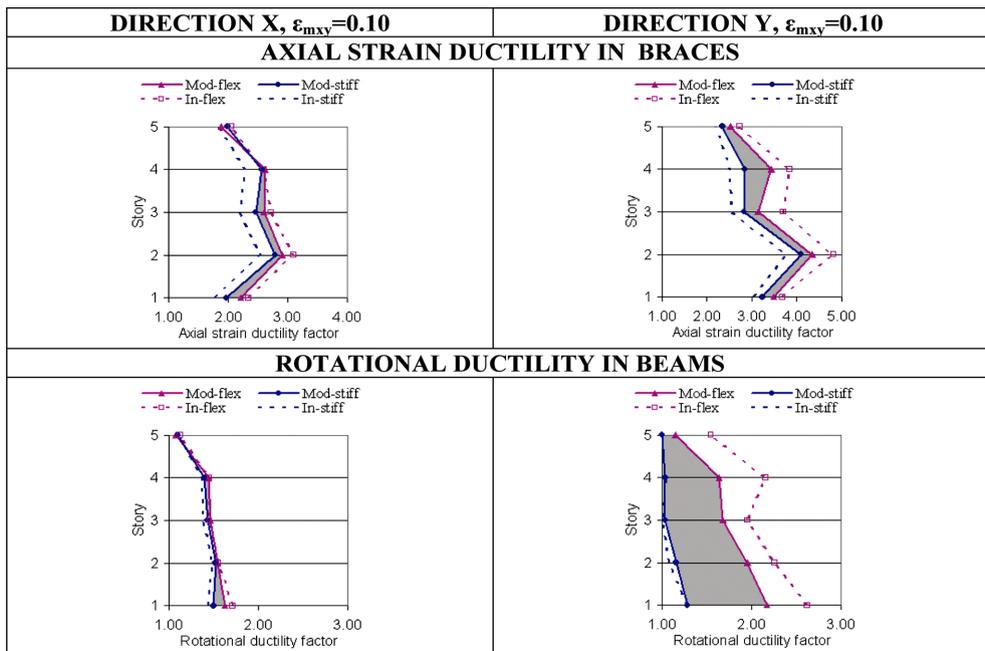


Fig. 10 Comparison of member ductility demands of 5-story buildings with  $\epsilon_m = 0.10$ , for the initial and modified designs

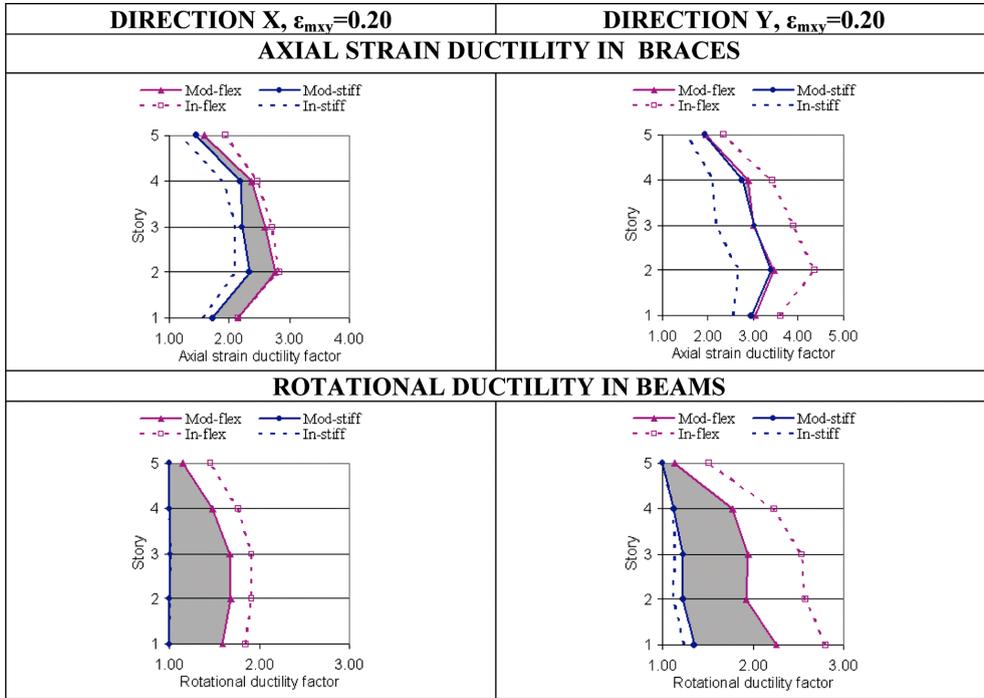


Fig. 11 Comparison of member ductility demands of 5-story buildings with  $\varepsilon_m = 0.20$ , for the initial and modified designs

#### 4.3.1 Two cycle design modification

The design modification suggested above may be applied repeatedly, i.e. after the first modification the new building is analyzed again, new factors are computed, a new modification is made and so on. Obviously, the final structure must be checked to satisfy all the code requirements. To get an idea of the potential improvement expected from this iterative process, the modified five story building with initial mass eccentricity  $\varepsilon_m = 0.20$ , was subjected to a second cycle of modification and the new design was analyzed again for the same set of motions. Results are shown in Figs. 12 and 13. We can see a small improvement in displacements and brace ductility demands and no noticeable change in the beam ductility demands. This might have been expected given that the modification was restricted to the bracing members only. For the buildings examined here the improvements from this iteration are considered marginal, but in other cases it might not.

#### 4.4 Effects of the proposed modification on the resisting plane frames

In order to have a better picture of the influence of the proposed design modification on the strength and stiffness distribution among the resisting plane frames of the original building designs, pushover analyses were carried out for all the plane frames along the  $x$  and  $y$  directions of the 3-story building having an original mass eccentricity of  $\varepsilon_m = 0.20$ . The analyses were carried out both for the original (as designed) and the modified buildings and the results are compared in Figs. 14 and 15. In these Figures we see that the effect of the modification was to make the “flexible” edge frames stiffer and stronger and the “stiff” edge frames softer and weaker, in comparison to the

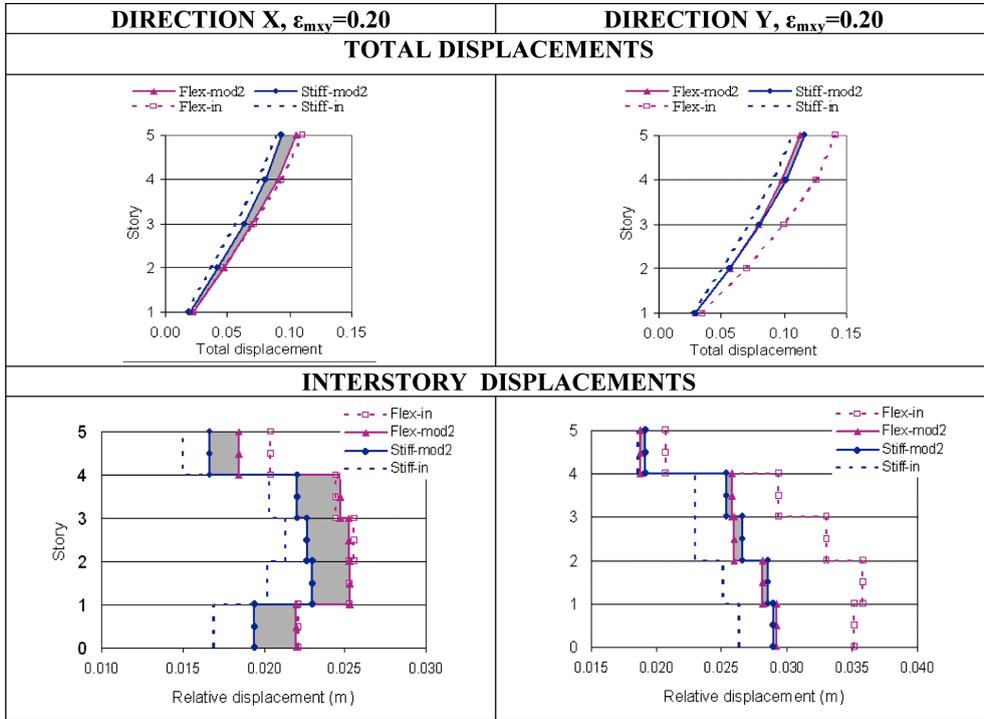


Fig. 12 Total displacements and interstory drifts of 5-story buildings with  $\epsilon_m = 0.20$  with a two-cycle design modification

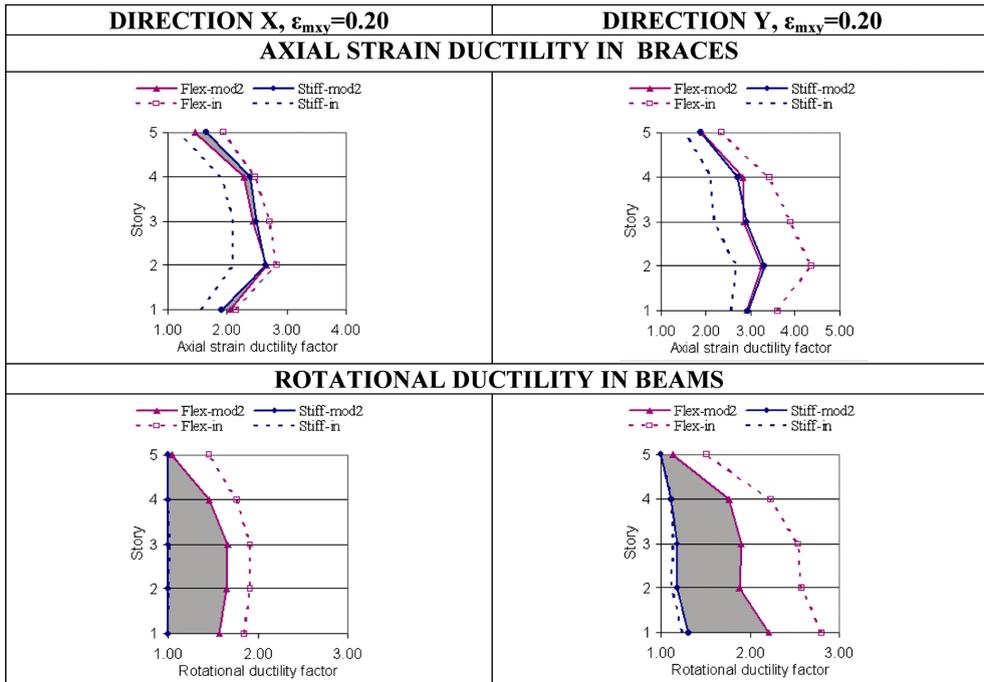


Fig. 13 Member ductility demands of 5-story buildings with  $\epsilon_m = 0.20$  with a two-cycle design modification

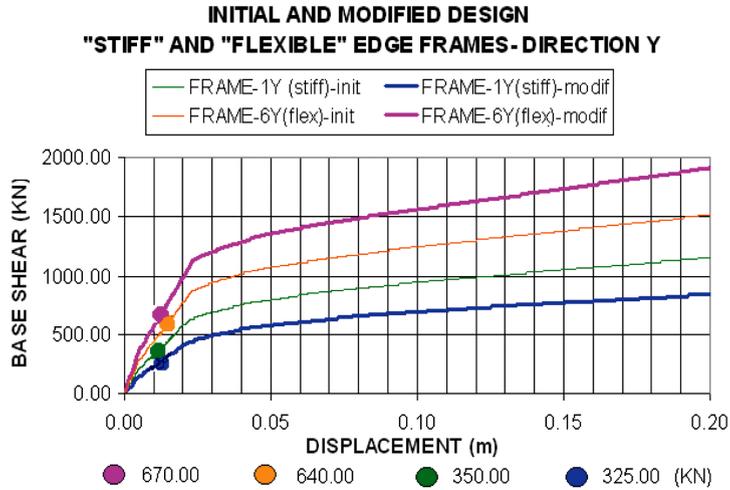


Fig. 14 Comparison of pushover curves of initial and modified 3-story building,  $\epsilon_m = 0.20$ , direction  $Y$ . Points on curves with values at bottom indicate the design base shears

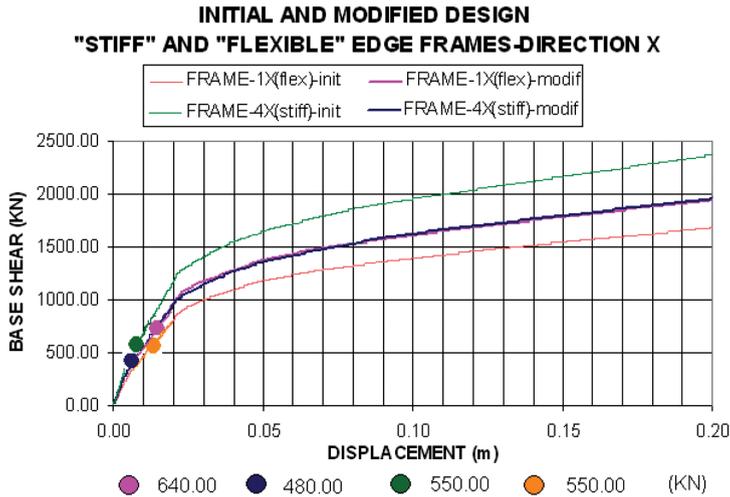


Fig. 15 Comparison of pushover curves of initial and modified 3-story building,  $\epsilon_m = 0.2$ , direction  $X$ . Points on curves with values at bottom indicate the design base shears

original design. This happened along both axes,  $y$  and  $x$ . In the same Figures the design base shears are also marked on each curve and their values are listed at the bottom of each graph, so one can have a clear picture of each frame’s overstrength. The design base shears are envelope values that result by applying a total of 32 earthquake loading combinations, arising from 8 possible loadings  $\pm(E_x + 0.3E_y)$ ,  $\pm(E_x - 0.3E_y)$ ,  $\pm(0.3E_x + E_y)$ ,  $\pm(0.3E_x - E_y)$  and four possible locations of the mass center due to the accidental design eccentricity, as specified by the code. The differences between design shear from earthquake loading alone and overstrength of a code designed frame, is a reason for which results on inelastic torsion, based on simplified models with element strengths determined only from the earthquake loading, can lead to erroneous conclusions and thus should not be used to assess a code as has often been done in the past (Anagnostopoulos *et al.* 2010).

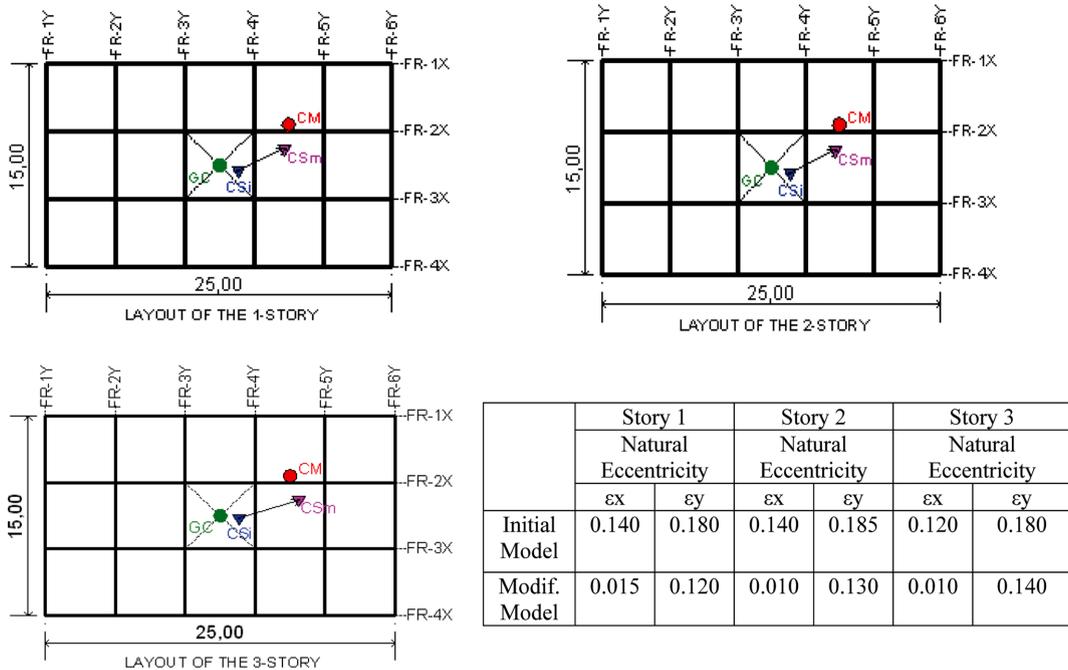


Fig. 16 Approximate stiffness centers and floor eccentricities of the original and modified building

To investigate further the consequences of the proposed modification, approximate stiffness centers were computed for each story of the initial and modified 3 story eccentric building using the approximations suggested by Kyrkos and Anagnostopoulos (2010a). Results are shown in Fig. 16, where in addition to the stiffness centers of the initial and modified building stories, the geometric and mass centers are also shown. Note that the mass centers remain unchanged. We can see that the proposed modification brings the approximate stiffness center of each story substantially closer to the mass center and thus the torsional motions are substantially reduced. This reduction is obviously greater for the “flexible” edge and hence the reduction in the observed differences of ductility demands between “flexible” and “stiff” edges. We must note here that bringing the stiffness center as close as possible to the mass center, in other words trying to minimize the physical eccentricity, is a well known design objective in earthquake engineering as it minimizes torsional motion. The proposed modification is thus a “blind” way of achieving this without significant extra effort.

#### 4.5 A case of overload

To complete this study, we have carried out analyses with increased levels of earthquake shaking, above the design level, in order to see the effects of overloading on members at the two edges, “stiff” and “flexible”, of the original and the redesigned buildings. Fig. 17 shows total displacements and interstory drifts and Fig. 18 shows ductility demands for the 5-story eccentric building with  $\epsilon_m = 0.20$  subjected to the same group of motions scaled up by 50%. All the response parameters indicate the more desirable behavior of the modified structure.

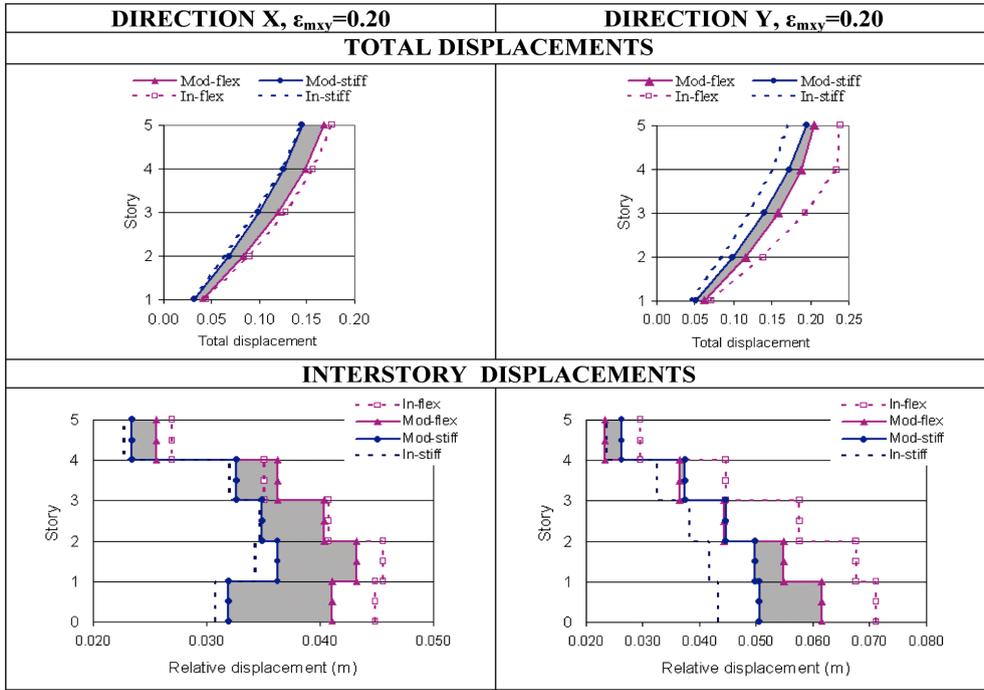


Fig. 17 Comparison of total displacements and interstory drifts of 5-story buildings with  $\epsilon_m = 0.20$ , for the initial and modified designs under 50% overload

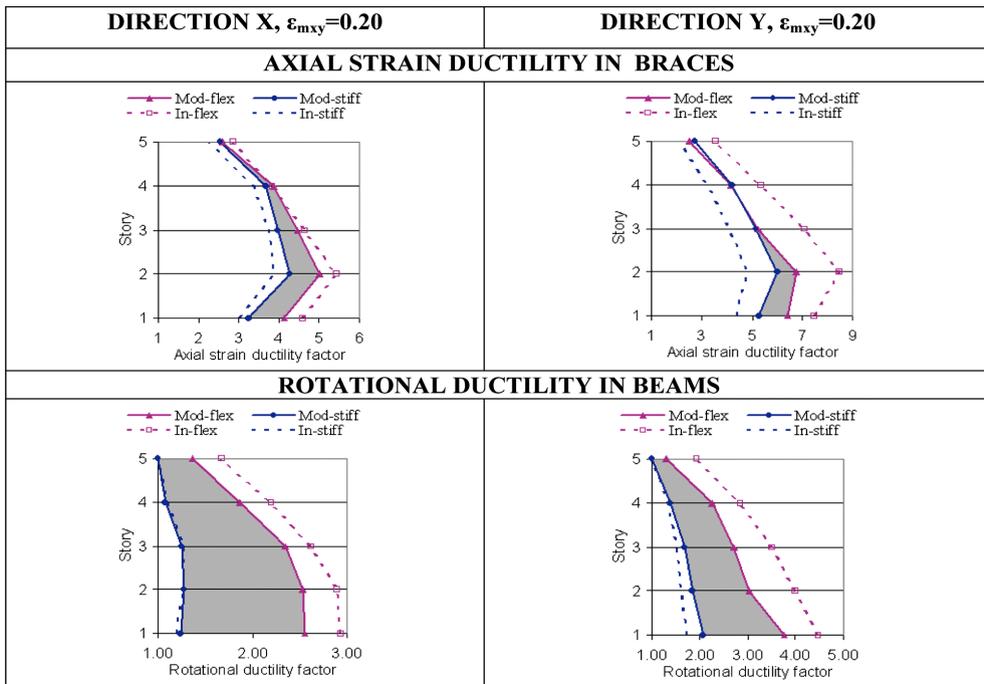


Fig. 18 Comparison of member ductility demands of 5-story buildings with  $\epsilon_m = 0.20$ , for the initial and modified designs under 50% overload

## 5. Conclusions

In recent publications it has been shown that eccentric frame buildings designed according to Eurocodes respond to strong earthquake excitations in a manner that produces substantially greater displacements and ductility demands in the “flexible” edge frames than in the “stiff” edge frames. The present paper has examined a design modification aiming at alleviating this shortcoming. The modification consists of strengthening the “flexible” edge and weakening the “stiff” edge by certain amounts that depend on the elastic design rotation of the top floor of the building. Elastic and inelastic dynamic analyses of the modified designs have shown substantial improvement of the inelastic response for all 3 groups of buildings examined. More specifically it was found that the modified designs:

1. Have substantially reduced physical eccentricities compared to the originally designed buildings.
2. As a result of the observation in (1) above, the modified building response has less torsional motion and the difference in maximum displacements between “flexible” and “stiff” edges is reduced. The reduced physical eccentricities bring the modified designs closer to becoming “torsionally balanced”.
3. Have improved ductility demand distributions: (a) the maximum ductility factors now are lower than those in the original designs, and, (b) the differences between “flexible” and “stiff” edge demands have been reduced.

Thus the modified designs appear to be preferable because both the maximum ductility demands are reduced and in addition the distribution of such demands is more uniform among the load bearing frames.

These findings are strictly applicable to the specific building types examined herein. However, given that the proposed modification was effective: (a) for all three groups of buildings - with one, three and five stories, (b) for another set of similar buildings having a different layout of their bracing system (see Kyrkos and Anagnostopoulos 2010b), one could expect that it might also be effective more generally, covering other cases of asymmetric buildings as well. This might warrant a pertinent code modification. However, more studies covering a wider range of building asymmetries, including geometric irregularities in layout and elevation, as well as a wider spectrum of parameters, will be required, before firm recommendations for code improvement could be made.

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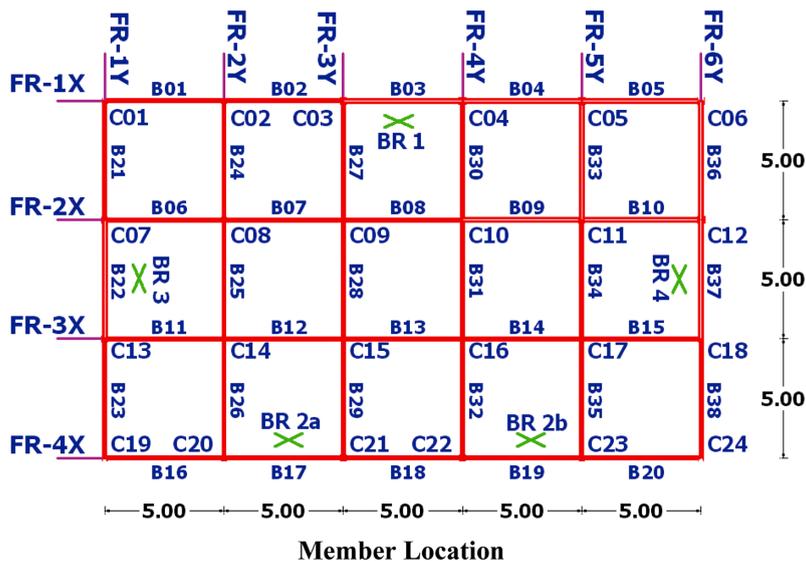
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CC

## Appendix

List of member sections of 3-story eccentric building with  $\varepsilon_m=0.20$



### COLUMN SECTIONS

COLUMNS	1 <sup>st</sup> story	2 <sup>nd</sup> story	3 <sup>rd</sup> story	COLUMNS	1 <sup>st</sup> story	2 <sup>nd</sup> story	3 <sup>rd</sup> story
C01	HEB 160	HEB 160	HEB 160	C13	HEB 260	HEB 240	HEB 220
C02	HEB 180w	HEB 180w	HEB 180w	C14	HEB 220	HEB 200	HEB 200
C03	HEB 240w	HEB 200w	HEB 160w	C15	HEB 220	HEB 200	HEB 200
C04	HEB 220w	HEB 200w	HEB 180w	C16	HEB 240	HEB 220	HEB 220
C05	HEB 280w	HEB 260w	HEB 260w	C17	HEB 240	HEB 220	HEB 200
C06	HEB 240	HEB 240	HEB 240	C18	HEB 320	HEB 300	HEB 260
C07	HEB 260	HEB 220	HEB 220	C19	HEB 200	HEB 180	HEB 180
C08	HEB 240	HEB 220	HEB 220	C20	HEB 220w	HEB 200w	HEB 200w
C09	HEB 240	HEB 220	HEB 220	C21	HEB 220w	HEB 200w	HEB 200w
C10	HEB 240	HEB 220	HEB 220	C22	HEB 220w	HEB 200w	HEB 200w
C11	HEB 300	HEB 300	HEB 280	C23	HEB 220w	HEB 200w	HEB 200w
C12	HEB 360	HEB 340	HEB 280	C24	HEB 240	HEB 240	HEB 220

w: denotes that the weak axis of the steel profile is parallel to the Y-axis

**BEAM SECTIONS**

<b>B01</b>	IPE 160	<b>B10</b>	IPE 450	<b>B19</b>	IPE 140	<b>B29</b>	IPE 200
<b>B02</b>	IPE 140	<b>B11</b>	IPE 200	<b>B20</b>	IPE 140	<b>B30</b>	IPE 240
<b>B03</b>	IPE 140	<b>B12</b>	IPE 200	<b>B21</b>	IPE 160	<b>B31</b>	IPE 200
<b>B04</b>	IPE 160	<b>B13</b>	IPE 200	<b>B22</b>	IPE 140	<b>B32</b>	IPE 200
<b>B05</b>	IPE 220	<b>B14</b>	IPE 200	<b>B23</b>	IPE 140	<b>B33</b>	IPE 360
<b>B06</b>	IPE 220	<b>B15</b>	IPE 330	<b>B24</b>	IPE 240	<b>B34</b>	IPE 300
<b>B07</b>	IPE 220	<b>B16</b>	IPE 140	<b>B25</b>	IPE 200	<b>B35</b>	IPE 220
<b>B08</b>	IPE 200	<b>B17</b>	IPE 140	<b>B26</b>	IPE 200	<b>B36</b>	IPE 240
<b>B09</b>	IPE 220	<b>B18</b>	IPE 140	<b>B27</b>	IPE 240	<b>B37</b>	IPE 220
				<b>B28</b>	IPE 200	<b>B38</b>	IPE 140

**BRACES (Circular Cross Sections)**

	Initial Model			Modified Model		
	1 <sup>st</sup> story	2 <sup>nd</sup> story	3 <sup>rd</sup> story	1 <sup>st</sup> story	2 <sup>nd</sup> story	3 <sup>rd</sup> story
<b>FR-1X</b>	193.7x6	139.7x6	88.9x5	193.7x8	168.3x6	88.9x6.3
<b>FR-4X</b>	168.3x5	139.7x5	88.9x4	114.3x6	114.3x5	88.9x3
<b>FR-1Y</b>	168.3x5	139.7x5	88.9x4	114.3x5	114.3x4	60.3x4
<b>FR-6Y</b>	193.7x6	139.7x6	88.9x5	193.7x8	139.7x8	139.7x4