Assessment of earthquake resistant design of eccentric, braced frame, steel buildings and proposal for possible improvements.

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SUMMARY:
Research in the past decade on the inelastic earthquake response of non-symmetric, multistory buildings, designed in accordance to the Eurocodes, has indicated that ductility demands under the action of design-level earthquakes, are not distributed evenly throughout the structure, as it would be desirable for a well balanced design. More specifically, it is found that elements at the so called "flexible" edges of the buildings exhibit substantially higher ductility demands than elements at the "stiff" edges. In the present paper the aforementioned observations are verified using two sets of three-story buildings: torsionally stiff and torsionally flexible. Subsequently a design modification for these buildings is proposed that improves substantially their inelastic earthquake response. The proposed modification reduces the natural eccentricity in all floors of the buildings and eliminates the differences in ductility demands between the "flexible" and "stiff" edges.

Keywords: torsion, eccentricity, inelastic response

1. INTRODUCTION

Earthquake response of irregular buildings in the inelastic range is an open research area that includes also assessment of code provisions pertaining to the design of such buildings (Rutenberg, 1998, 2002, De Stefano and Pintucchi, 2008). Most of the pertinent studies have been based on elastic analyses of idealized multistory buildings or on inelastic analyses of highly simplified, one-story, inelastic models of the shear beam type with 3 degrees of freedom (De La Colina, 2003, Stathopoulos and Anagnostopoulos, 2000, 2003). In the past decade, however, research on earthquake induced torsion started using more sophisticated, multi story, multi-degree of freedom inelastic models of the plastic hinge type (Ghersi et al, 2000, Stathopoulos and Anagnostopoulos, 2005, Anagnostopoulos et al, 2010, Kyrkos and Anagnostopoulos, 2011a, 2011b, 2012). These more recent studies have generated some interesting results that cast doubt about the adequacy of modern code provisions for frame type, multistory, asymmetric buildings. Additionally, it is shown that results based on the simplified shear beam model could lead to erroneous conclusions, unless model properties are very carefully selected to closely match key properties of the multistory building (Anagnostopoulos et al, 2010). Hence code provisions based, in part at least, on such results might be questionable. Moreover, a number of controversies had been generated from such studies and a few publications were devoted to them (Rutenberg, 1998, 2002, De Stefano and Pintucchi, 2008). One such controversy, that lingered for some time was the question whether in a code designed building the critical edge, where the term critical is used to mean “having the highest ductility demands”, is the so called “flexible” or “stiff” edge. A convincing answer to this has been presented recently, based on analyses using both detailed and simplified models (Anagnostopoulos et al, 2010). The recent studies of torsion with the detailed plastic hinge models also showed that, the ductility demand differences between the two edges, “flexible” and “stiff”, were often very large, with the demands in the “flexible” edge being always substantially greater than the demands in the “stiff” edge of the building. This was initially found for concrete buildings, where both rotational ductility factors and damage indices were used as measures of inelastic deformations (Stathopoulos and Anagnostopoulos, 2005). Subsequently the same was confirmed for eccentric, braced frame type, steel buildings and a design modification was proposed to alleviate this problem (Kyrkos and Anagnostopoulos, 2011a, 2011b, 2012).

In the present paper the aforementioned observations are verified using two sets of three-story buildings: torsionally stiff and torsionally flexible. In each set three buildings are designed: one symmetric and two eccentric variations with biaxial mass eccentricities: e_m=0.10L and e_m=0.20L, where L is the building's length along each direction. All buildings are designed as spatial frames for gravity and earthquake loads using the response spectrum method of analysis. For the inelastic analyses, the buildings are idealized with the well known plastic hinge model and are subjected to ten sets of two component semi-artificial motions that closely match the code design spectrum. Rotational ductility demands of beams and axial ductility demands of braces are used to evaluate the overall performance of the buildings. In earlier publications a design modification was introduced leading to more uniform ductility demands, i.e. to similar values in the “flexible” and “stiff” edges of the building. This modification, however, was slightly different for torsionally stiff and torsionally flexible buildings (Kyrkos and Anagnostopoulos, 2011a, 2011b, 2012). In the present paper the same modification is applied to both torsionally stiff and flexible buildings and results are presented indicating satisfactory inelastic response of both types of irregular buildings.

2. METHODOLOGY

The present investigation was carried out using two sets of 3-story, steel, braced frame buildings, the first torsionally stiff and the second torsionally flexible. The layout, same for all floors of the two building sets, can be seen in Figs. 2.1 and 2.2. Each building is formed by 4 frames along the x-axis (FR-X01 to FR-X04) and 4 frames along the y-axis (FR-Y01 to FR-Y04). In order to have torsionally stiff and torsionally flexible buildings just for the purpose of our work, braces were used to stiffen specific bays as shown in Figs. 1 and 2, respectively. Both buildings have a typical story height of 3.00m and ground story height 4.00m. Using appropriate distributions of the floor loads, e.g. through non-symmetric live load distribution, non-symmetric balconies (common causes of mass eccentricity in typical Greek buildings, not shown in the given layout), non-symmetric joint masses were assigned at each floor and thus biaxial mass eccentricities were introduced in all floors. In this manner, in addition to the symmetric layouts, eccentric variants were generated and designed with the following mass eccentricities: e_m=0.10L and e_m=0.20L, where L is the building length along each direction. Additionally the distribution of stiffness is not uniform as a result of the layout’s geometry and thus biaxial natural eccentricities were introduced in all floors.

The models used for both design and analyses are 3-D models with masses lumped at the joints and with floors acting as diaphragms. All buildings were designed as spatial frames for gravity and earthquake loads using the dynamic, response spectrum method, according to Eurocodes EC3 -steel structures- and EC8 -earthquake resistant design. Earthquake actions were described by the design spectrum specified by the Greek Code for peak ground acceleration PGA=0.24g and soil category II. As input for the nonlinear dynamic analyses, ten sets of two component semi-artificial motion pairs were used. They were generated from a group of five, two-component, real earthquake records, to closely match the code design spectrum (with a descending branch $\propto 1/T^{2/3}$), using a method based on trial and error and Fourier transform techniques (Karabalis et al, 2000). Results were excellent, as Fig. 2.3 indicates where the mean response spectrum of the ten semi-artificial motions is compared with the target design spectrum. 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. The design spectrum can be seen in Fig. 2.3, along with the mean spectra of the motions used for subsequent analyses. It should be noted that the dimensioning of the frame members took into account the uneven distribution of member forces due to the mass eccentricities and hence stiffness eccentricities were also generated, as it happens in actual practice.

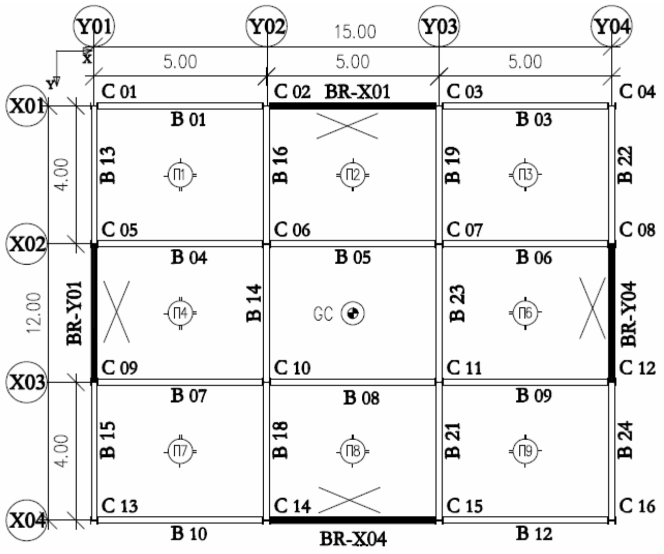

Figure 2.1. Typical layout of 3-story torsionally stiff steel buildings.

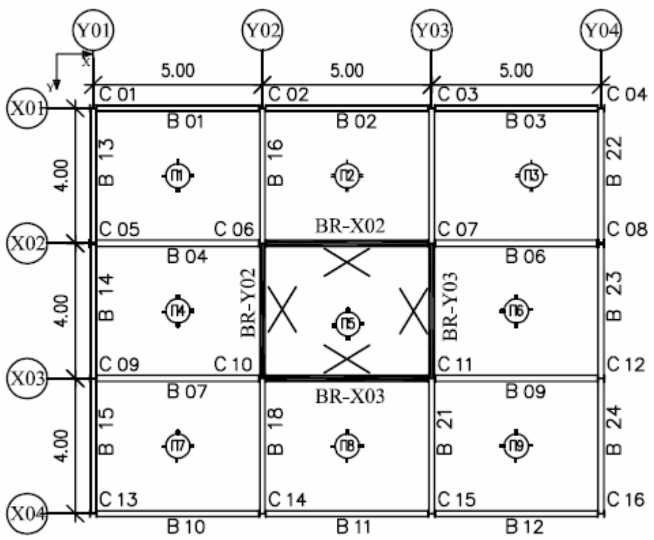

Figure 2.2. Typical layout of 3-story torsionally flexible steel buildings.

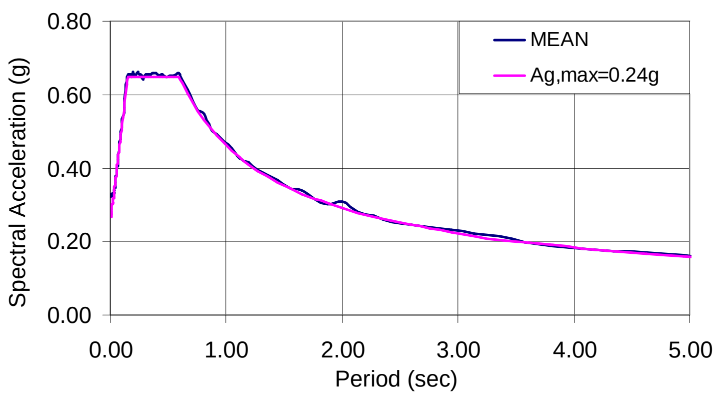

Figure 2.3. Design spectrum and mean spectrum of the ten semi-artificial motions.

The complete set of the lowest three periods of all building variants in each set is listed in Table 2.1, along with the initial mass eccentricities ε_{mx}=ε_{my}=0.10 and 0.20 and the resulting physical eccentricities ε_x= ε_y. The latter are the mean distances (for all stories) between the mass center (CM) and the approximate stiffness (or rigidity) center (CR) in each floor, normalized by the length of the corresponding building side. It is noted that in multistory buildings, the CR cannot be really defined, except under very restrictive conditions.

Table 2.1. Eccentricities and fundamental periods of the buildings.

	Mass eccentricity	Mean natural eccentricity		Fundamental periods of Buildings (sec)		
	ε_{mx}= ε_{my}	ε_x	ε_y	T_x	T_y	T_θ
STIFF	0.00	0.00	0.00	0.52	0.52	0.31
	0.10	0.045	0.050	0.54	0.53	0.32
	0.20	0.115	0.135	0.52	0.50	0.29
				T_θ	T_x	T_y
FLEXIBLE	0.00	0.00	0.00	0.82	0.56	0.55
	0.10	0.085	0.075	0.84	0.54	0.52
	0.20	0.165	0.145	0.88	0.53	0.46

3. NON-LINEAR DYNAMIC ANALYSES

The non linear analyses were carried out using the program RUAUMOKO (Carr, 2005). Frame beams and columns were modelled with the well-known plastic hinge model, in which yielding at member ends is idealized with plastic hinges of finite length having bilinear moment-curvature relationship and strain hardening ratio equal to 0.05. A moment-axial force interaction diagram was also employed for columns, giving the yield moment as a function of the applicable axial force on the column section. Bracing members, yielding in tension and buckling in compression, were modelled with a non-symmetric bilinear force-axial deformation relationship.
The basic measure used to assess the severity of inelastic response is the ductility factor of the various members. For bracing members the ductility factor is defined as:

$$\mu_u = 1 + \left(\frac{u_p}{u_y} \right) \tag{3.1}$$

where u_p is the maximum plastic member elongation and u_y the elongation at first yield.
In the present study, the rotational ductility factor used is based on the post yield plastic moment:

$$\mu = 1 + \left(\frac{\Delta M}{p \cdot M_y} \right) \tag{3.2}$$

where: ΔM=M_{max} - M_y , M_y = yield bending moment and $p=0.05$, the strain hardening ratio. It has been shown that this definition is essentially a rotational ductility factor, ratio of the maximum end rotation (including the elastic joint rotation plus the plastic hinge rotation of the member end), divided by a "yield rotation" reflecting the instantaneous boundary conditions of the considered beam (Anagnostopoulos, 1981).

4. RESULTS FROM NON-LINEAR ANALYSES OF "AS DESIGNED" BUILDINGS.

Results from time history analyses of the buildings are presented in terms of mean values of the peak response parameters over the ten pairs of applied motions. In the case of the beam ductility demands, the response parameter averaged over the ten pairs of motion is the maximum rotational ductility demand in any of the beams in the considered frame and floor. Following standard terminology based on static application of the lateral load in torsionally stiff buildings, the edge where the displacement from rotation is added to the pure floor translation is called "flexible" edge, while the opposite edge, where the displacement due to rotation is subtracted from the pure translation is called "stiff" edge. Since the examined buildings have biaxial eccentricity, the edge distinction just mentioned applies to both the *x* and *y* horizontal directions of the buildings. Thus, results are presented for each edge frame and each direction. In torsionally flexible buildings, however, it is not necessarily the "flexible" edge that experiences the largest translation but it could well be the "stiff" edge, depending on the relative values of the torsional and translational periods and on the input characteristics.

4.1. Three-story torsionally stiff buildings

Ductility demands for braces and beams of the "flexible" and "stiff" edges are presented in Figs. 4.1 and 4.2, for the torsionally stiff buildings with biaxial mass eccentricities $\varepsilon_{mx}=\varepsilon_{my}=0.10$ and 0.20, respectively, where in the same figures values for the symmetric case are also shown for comparison. Ductility demands are presented only for beams and brace members because the columns remained essentially elastic. We can see that the ductility demands at the "flexible" edges of the torsionally stiff eccentric building are substantially greater than those at the "stiff" edges due to the induced earthquake rotations, especially in the *y* direction.

4.2. Three-story torsionally flexible buildings

Ductility demands for braces and beams of the "flexible" and "stiff" edges are presented in Figs. 4.3 and 4.4, for the torsionally flexible buildings with biaxial mass eccentricities $\varepsilon_{mx}=\varepsilon_{my}=0.10$ and 0.20, respectively, where in the same figures values for the symmetric case are also shown for comparison. The differences in ductility demands in braces are quite small as the braces are placed near the core. The differences become large in the beams at the two edges of the buildings with the "flexible" edge experiencing substantially greater demands than the "stiff" edge, same as in torsionally "stiff" buildings.

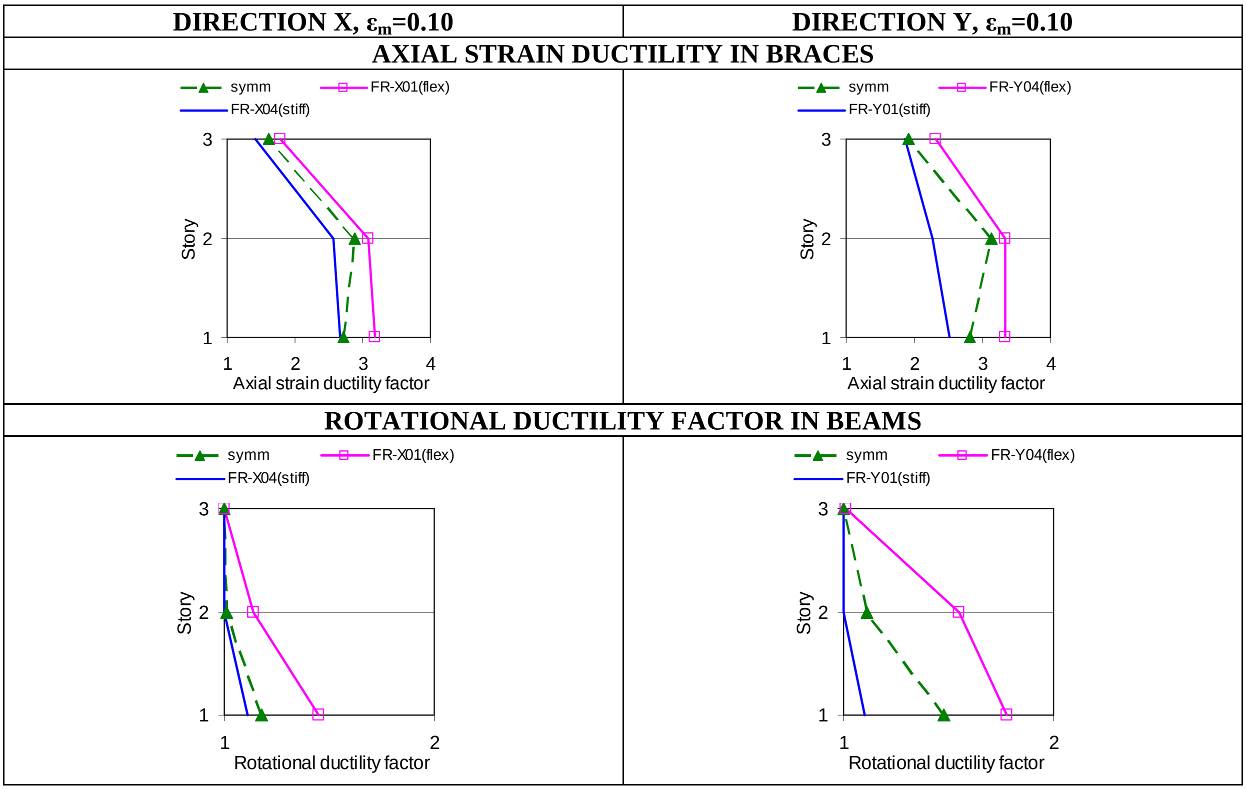

Figure 4.1. Member ductility demands of 3-story torsionally stiff building with ε_m=0.10 and comparison with the symmetric building. (FR-X01 & FR-Y04: "flexible" edges, FR-X04 & FR-Y01: "stiff" edges).

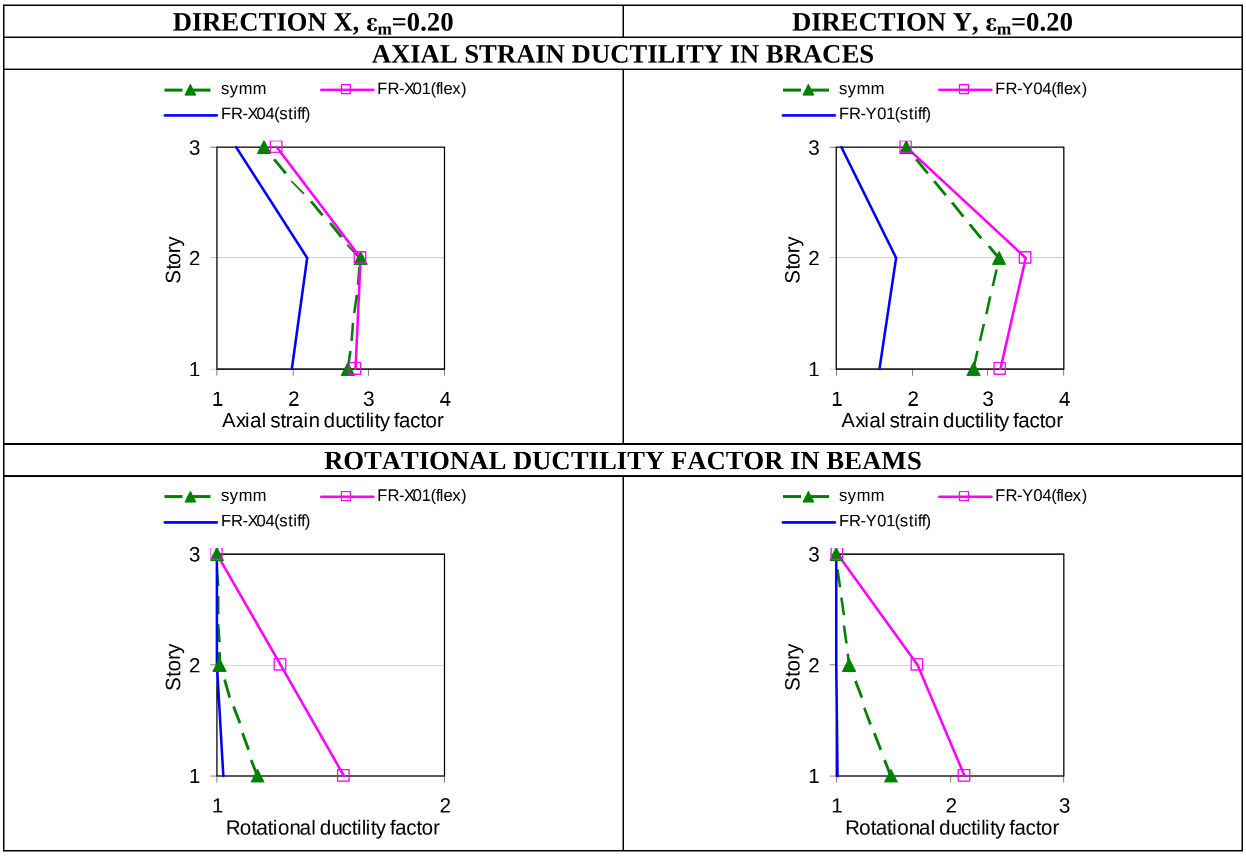

Figure 4.2. Member ductility demands of 3-story torsionally stiff building with ε_m=0.20 and comparison with the symmetric building. (FR-X01 & FR-Y04: “flexible” edges, FR-X04 & FR-Y01: “stiff” edges).

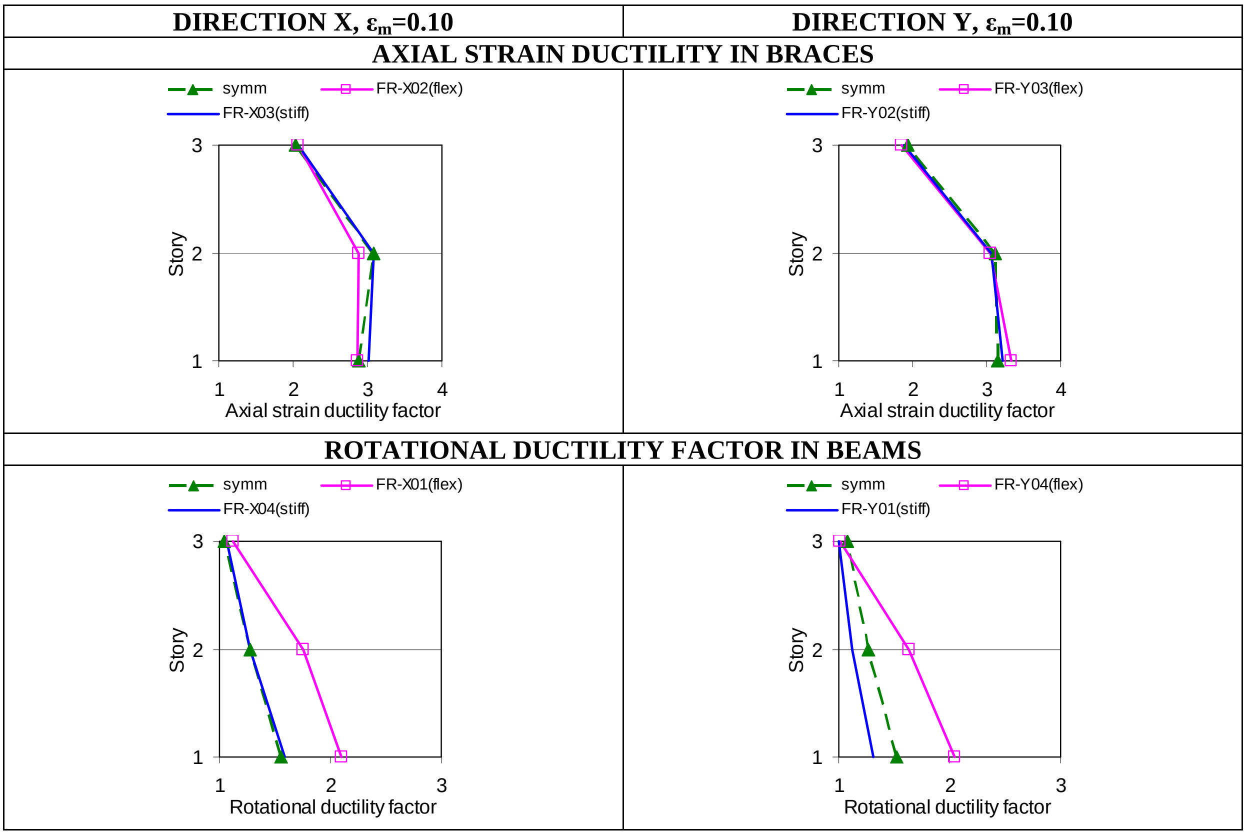

Figure 4.3. Member ductility demands of 3-story torsionally flexible building with ε_m=0.10 and comparison with the symmetric. (FR-X01,X02 & FR-Y03,Y04: “flexible” sides, FR-X03,X04 & FR-Y01,Y02: “stiff” sides).

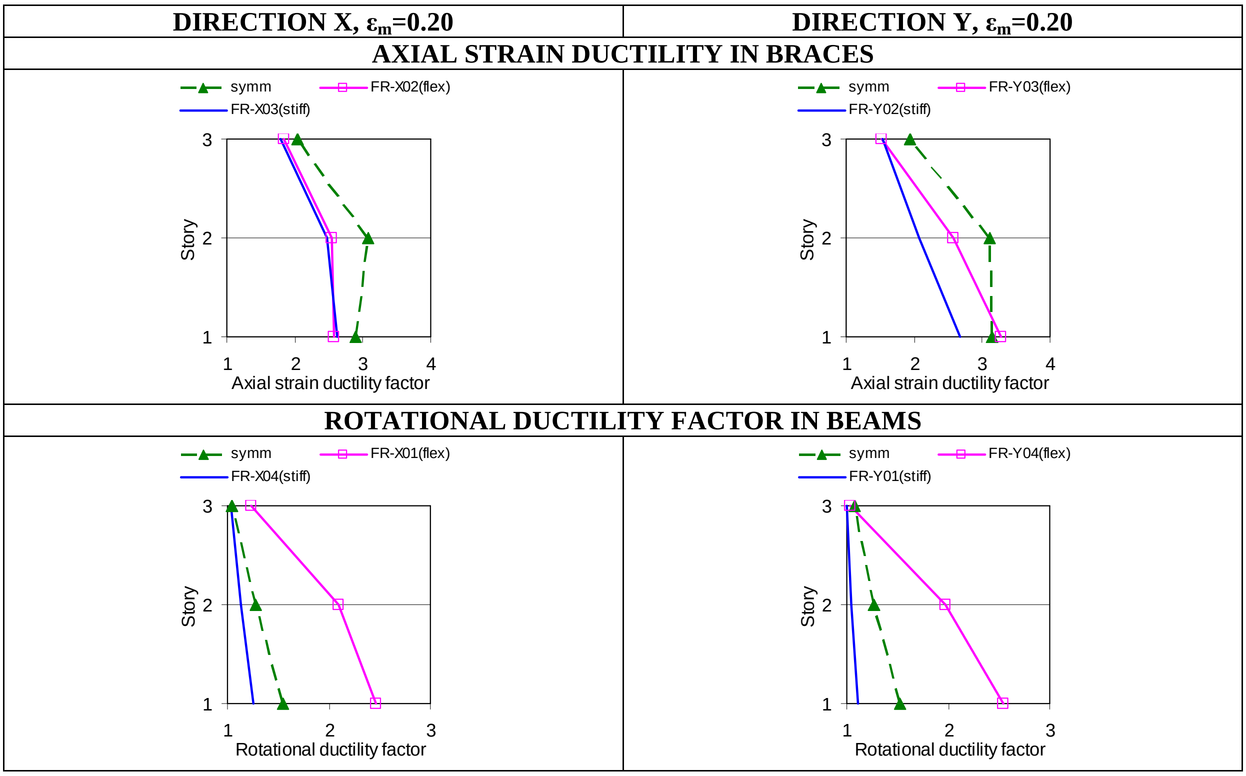

Figure 4.4. Member ductility demands of 3-story torsionally flexible building with ε_m=0.20 and comparison with the symmetric. (FR-X01,X02 & FR-Y03,Y04: "flexible" sides, FR-X03,X04 & FR-Y01,Y02: "stiff" sides).

5. MODIFICATION PROCEDURE

A structural design can be characterized as satisfactory when the limiting values of the controlling response parameters do not have wide variations within the groups of structural members to which they apply. In the opposite case, suboptimal use of material may be present as well as a potentially higher risk of failure in cases of unexpected overloads. Thus the observed substantial differences in ductility demands between the opposite edges of the examined buildings, 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 structural members (columns, beams and braces) at the "flexible" edges and reducing the strength of the braces at the "stiff" edges without affecting the strength of the other structural elements (columns, beams).

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 seismic combinations considered and then compute the following factors in each horizontal direction:

$$f_{i,flex} = 2 \cdot \frac{u_{i,flex}}{(u_{i,flex} + u_{i,stiff})} \qquad f_{i,stiff} = 2 \cdot \frac{u_{i,stiff}}{(u_{i,flex} + u_{i,stiff})} \tag{5.1}$$

where $u_{i,flex}$ is the top story displacement of the "flexible" edge in the i - direction and $u_{i,stiff}$ the top story displacement of the "stiff" edge also in the i - direction. These displacements are obtained by the equivalent static method for the seismic combinations considered. The factors are ratios of the top story displacements at the "flexible" and "stiff" edges in a given direction (x or y), to their mean values. The design modification that was subsequently applied was to multiply the axial areas of the bracing members in both the "stiff" and "flexible" edges by the corresponding factors in each direction and to do the same for the beam and column sections but only in the "flexible" edges to increase both stiffness and strength of the corresponding frames. The cross sections of columns and beams of the "stiff" edges are not reduced, as their strength is controlled mainly by gravity loads (loading combinations without earthquake). For the torsionally flexible buildings, these factors vary from 1.25-1.50 for the "flexible" edges and from 0.85 to 1.00 for the "stiff" edges, while for the torsionally stiff buildings these factors vary from 1.10-1.30 for the "flexible" edges and from 0.70 to 0.90 for the "stiff" edges. After this modification, each structure was checked again for full compliance with the applicable codes. The new, modified structures were again subjected to the same two component motion earthquake set and their responses were again computed as before.

The ductility demands for all buildings are presented in Figs. 5.1 to 5.4 for both the torsionally stiff and flexible buildings and for the initial and the modified designs. If we compare the results obtained from the modified designs with that of the original design, we see a substantial improvement of response in all cases. The proposed modification has the effect of reducing the natural eccentricity in all floors of the buildings and as a consequence it reduces substantially the differences in ductility demands between the "flexible" and "stiff" edges. We note that while in earlier publications on this subject (Kyrkos and Anagnostopoulos, 2011a, 2011b, 2012) a slightly different modification was used for torsionally stiff and torsionally flexible buildings, here the applied modification is identical for the two types of buildings. This makes it easier for introducing it into the code, after of course its effectiveness is verified with more case studies.

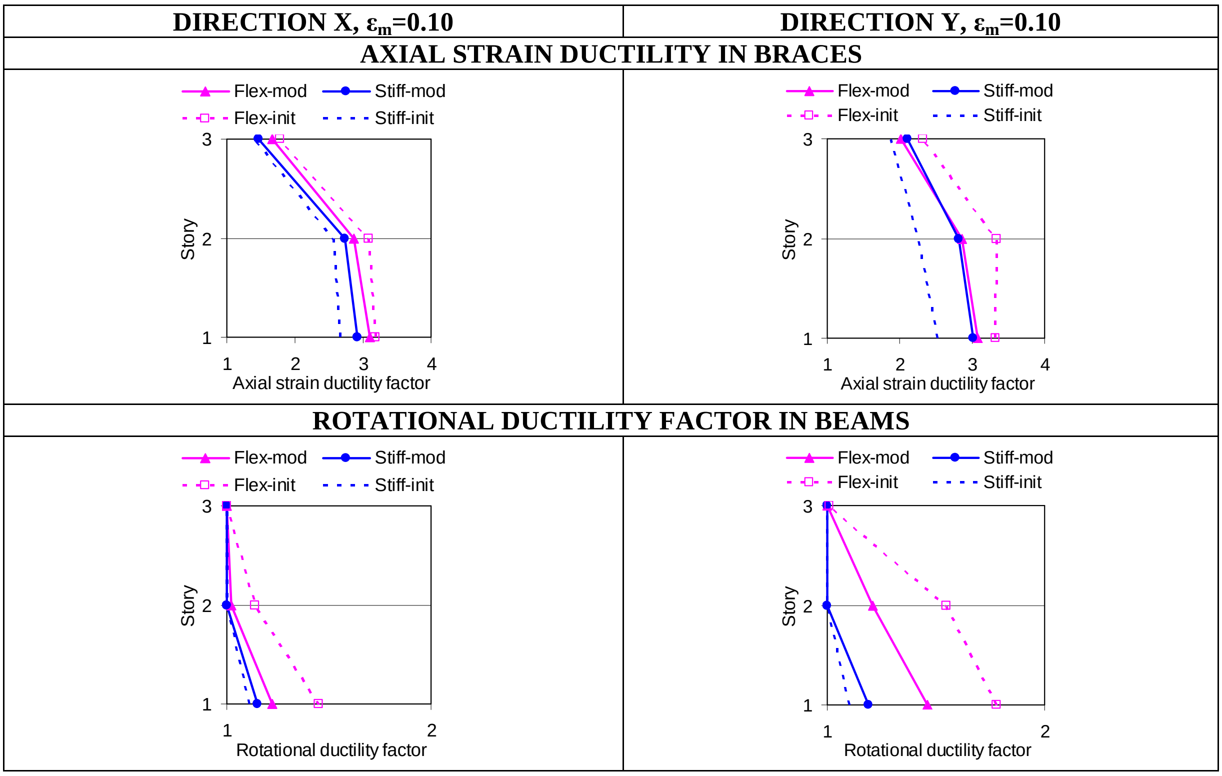

Figure 5.1. Member ductility demands of 3-story modified torsionally stiff building with ε_m=0.10 and comparison with the initial building.

6. CONCLUSIONS

In the present work the earthquake response of two sets of 3-story, steel, braced frame buildings, one torsionally stiff and the other torsionally flexible, both designed in accordance with Eurocodes EC3 and EC8 were examined and similar results were obtained compared to earlier findings for eccentric steel and reinforced concrete frame buildings. More specifically, it was found that under the action of two horizontal component earthquake loadings, compatible with the design spectra, ductility demands at the "flexible" edges were significantly greater than ductility demands at the "stiff" edges. Subsequently, the original designs were modified and it was found that the response of the new designs was improved: ductility demands at the "flexible" edges generally decreased, thus lowering the differences in demands between the two edges in each direction. Here the same modification has been used for both torsionally stiff and flexible buildings, thus making it easier for general application. On the basis of these findings, a code modification may appear desirable. However, additional studies covering other types of irregular buildings and a wider spectrum of parameters will be required, before any firm recommendation is put forward.

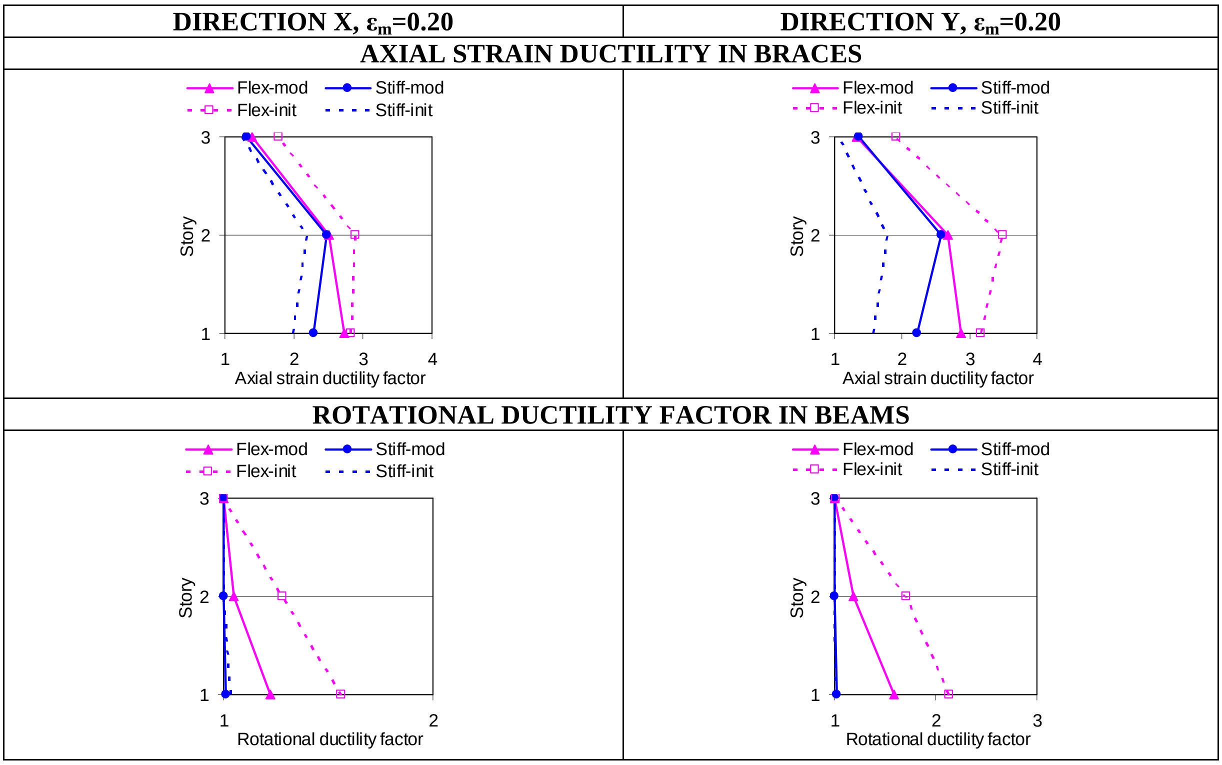

Figure 5.2. Member ductility demands of 3-story modified torsionally stiff building with ε_m=0.20 and comparison with the initial building.

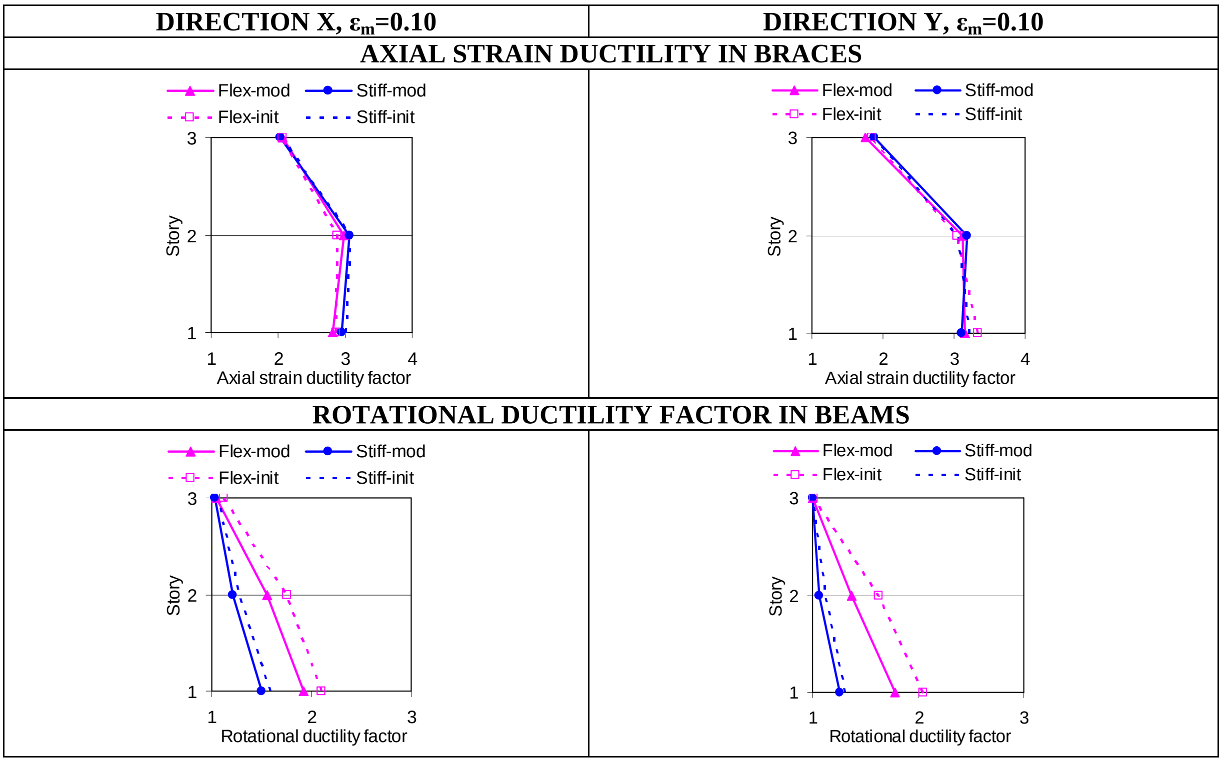

Figure 5.3. Member ductility demands of 3-story modified torsionally flexible building with ε_m=0.10 and comparison with the initial building.

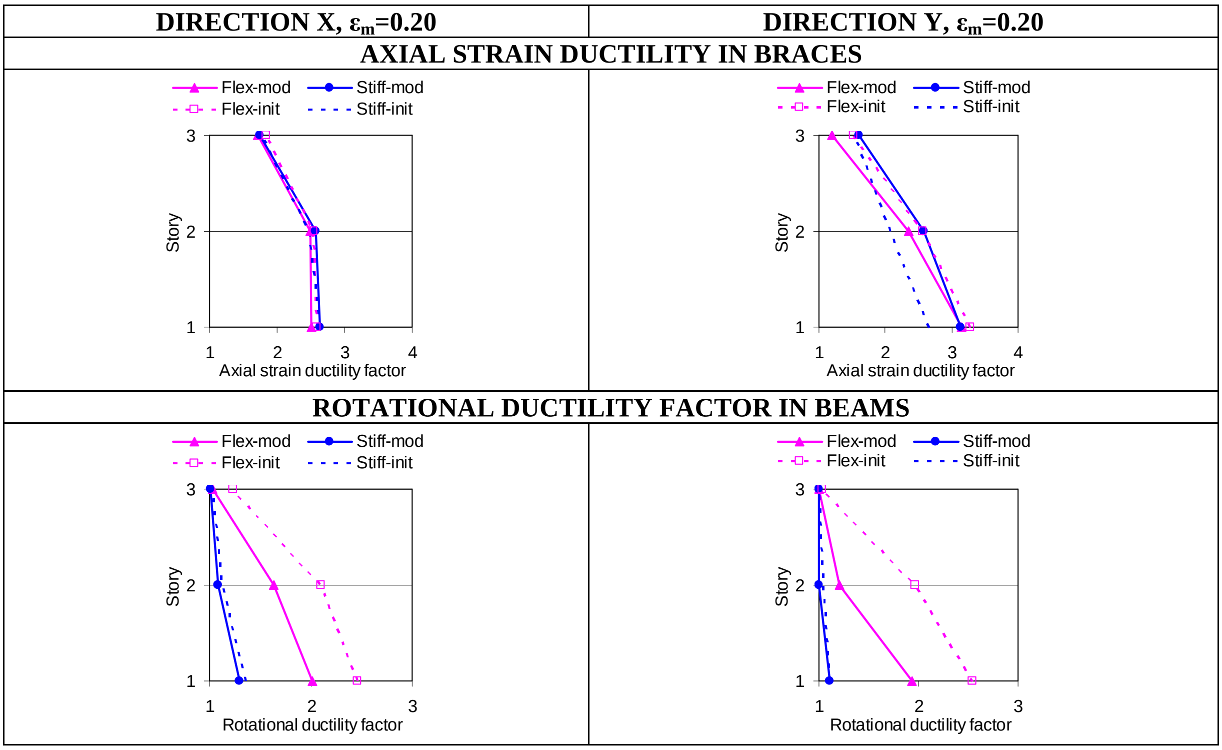

Figure 5.4. Member ductility demands of 3-story modified torsionally flexible building with ε_m=0.20 and comparison with the initial building.

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