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Should accidental eccentricity be eliminated from Eurocode 8?

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Abstract. Modern codes for earthquake resistant building design require consideration of the so-called accidental design eccentricity, to account for torsional response caused by several factors not explicitly considered in design. This provision requires that the mass centres in the building floor be moved a certain percentage of the building's dimension (usually 5%) along both the *x* and *y* axes and in both positive and negative directions. If one considers also the spatial combinations of the two component motion in a dynamic analysis of the building, the number of required analyses and combinations increases substantially, causing a corresponding work load increase for practicing structural engineers. Another shortcoming of this code provision is that its introduction has been based primarily on elastic results from investigations of oversimplified, hence questionable, one story building models.

This problem is addressed in the present paper using four groups of eccentric braced steel buildings, designed in accordance with Eurocodes 3 (steel) and 8 (earthquake design), with and without accidental eccentricities considered. The results indicate that although accidental design eccentricities can lead to somewhat reduced inelastic response demands, the benefit is not significant from a practical point of view. This leads to suggestions that accidental design eccentricities should probably be abolished or perhaps replaced by a simpler and more effective design provision, at least for torsionally stiff buildings that constitute the vast majority of buildings encountered in practice.

Keywords: accidental eccentricity; torsion; multistory steel buildings; inelastic response; plastic hinge model

1. Introduction

Under strong earthquake motions, non-symmetric buildings experience translational and torsional motion. The latter is caused mainly by non-symmetric distribution of element stiffness, strength and/or the building masses and since these properties are included in the building model used for the analysis, they are directly accounted for in design. However, there are also several other factors that are impossible or very difficult to quantify and hence quite difficult to be directly accounted in design. As such factors we may list: non uniform ground motion (due to wave

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travelling effects and motion incoherence) and consequent excitation differences at the support points, the presence of non-structural, yet stiffness and strength possessing elements not accounted for in design, unknown non-symmetric distributions of live loads or differences between actual and design distributions of mass, stiffness and strength. The torsional effects of such factors, which can also be present in fully symmetric buildings, have been idealized in modern codes by an extra mass eccentricity termed “accidental design eccentricity” (ADE). Eurocode 8 and the American code IBC specify this eccentricity equal to 0.05L while the New Zealand and Canadian codes equal to 0.10L, where L is the maximum dimension of the floor layout in the considered direction. For a specific class of torsionally irregular structures the American IBC code specifies an amplification factor A by which the sum of the static physical eccentricity and ADE (= 0.05L) is multiplied. The above codes specify that four additional loading conditions should be considered in design, by displacing the mass centers of all floors by amounts equal to ±ADE along each of the considered building directions *x* and *y*. The designer must also consider the spatial combination of the two component motion effects and thus the total number of load-structural model combinations for design checks increases dramatically causing a great increase in the work load of structural designers. For example considering linear spatial motion combinations of the form $\pm E_x \pm 0.30 E_y$, the number of loading conditions for the 5 locations of the mass centres climbs to 5×8=40. They are reduced if the SRSS combinations or an approximation for accidental eccentricity with a static torque are used, but the multiple design checks remain for member cross sections subjected to two or three gross forces peaking at different times.

Another aspect of the code accidental eccentricity provisions to be considered is the fact that they were introduced on the basis of elastic investigations carried out using often oversimplified building models. Subsequent application of this concept to realistic buildings responding in the inelastic range under design level earthquakes has been made rather intuitively and without the necessary supporting volume of research.

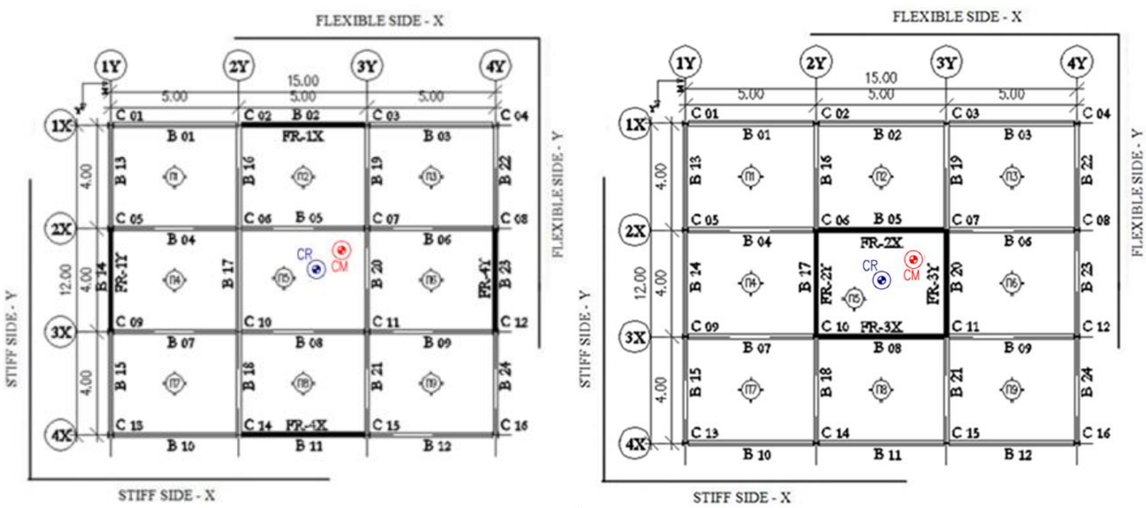
A detailed literature review of accidental design eccentricity and its causes may be found in the review paper on torsion by Anagnostopoulos *et al*. (2013, 2015). Here we will mention the early work of De La Llera and Chopra (1992, 1994 a, b, c, 1995), Wong and Tso (1994), Chandler *et al*. (1995), as well as more recent work on the same subject by Dimova and Alashki (2003), De la Colina and Almeida (2004), Aviles and Suarez (2006), Ramadan *et al*. (2008), De La Colina *et al*. (2011). In all these publications the problem of accidental eccentricity is studied under several simplifying assumptions for the building model and in most of them by addressing one source of it.

Investigations of how the code design eccentricity affects the inelastic response of realistic buildings have been reported by Stathopoulos and Anagnostopoulos (2005, 2006, 2010). They used inelastic dynamic analyses of detailed 3-D models of 1, 3 and 5-story concrete frame buildings, designed in accordance with Eurocodes 2 and 8 and concluded that although accounting for accidental eccentricity in design has some beneficial effects in terms of reducing ductility demands, such effects are not significant enough to justify the extra design efforts imposed on designers by the corresponding code provision. Thus they suggest reexamination of this provision with realistic models with the aim of removing it from the codes or limiting its application for special class buildings. More recently, DeBock *et al.* (2014) concluded that «accidental torsion provisions lead to significant changes in collapse capacity for buildings that are very torsionally flexible or asymmetric, while only inconsequential changes in collapse capacity are observed in the buildings that are both torsionally stiff and regular in plan”. As a result they conclude that accidental torsion provisions are not necessary for seismic design of buildings without excessive torsional flexibility or asymmetry. The results of both Stathopoulos and Anagnostopoulos (2005, 2006, 2010), DeBock *et al*. (2014) point all in the same direction, i.e., that the accidental torsion provisions in modern codes could probably be eliminated for the vast majority of torsionally stiff buildings.

The present paper extends the investigation of accidental design eccentricity by Stathopoulos and Anagnostopoulos (2005, 2006, 2010) to braced steel buildings, torsionally stiff and torsionally flexible, using inelastic dynamic analyses of detailed multistory models. The examined buildings are designed to fully comply with Eurocodes 3 and 8 (for steel and earthquake resistant design) and their inelastic response to a set of two component design level earthquake motions is compared to the response of another set of buildings, identical in geometry and masses to the original set but designed without considering the accidental eccentricity of the code (i.e., ADE is set =0.0). The investigation is completed by looking into the consequences to the response, when a real, "accidental" mass eccentricity (AME) is introduced to the already designed buildings through a shift of their masses by ±0.05L. It must be clarified here that this study does not intend to really check the importance of the various sources of accidental eccentricity but rather to check the effectiveness of the Code provision through the shifting by ±0.05L of the floor mass along the main diagonal.

2. The buildings used for the investigation

To carry out this study, realistic braced frame steel buildings with 3 and 5 stories have been used. Within each of these two groups, two subgroups were examined one torsionally stiff and another torsionally flexible. The layouts of the torsionally stiff and torsionally flexible buildings are shown in Fig. 1, where each layout is the same for the 3 and 5 stories. The difference between the two is the location of the braced bays: the torsionally stiff buildings have the braces in the



Torsionally stiff building Torsionally flexible building

Fig. 1 Layouts of the buildings used in the investigation middle bays of the perimeter sides, while the torsionaly flexible buildings have the bracing in the interior middle bays (core). Typical elevations of the unbraced and braced plane frames forming each 5-story building are given in Fig. 2. Since the results for the 3 and the 5-story buildings are qualitatively very similar, only results for the 5-story buildings will be presented here due to space limitations.

For both, 3 and 5-story, building groups and for both torsional categories - torsionally stiff and torsionally flexible- three building variants were generated: a fully symmetric (physical eccentricity 0.0) and two eccentric with initial bidirectional mass eccentricities of 0.10L and 0.20L. The latter were derived by appropriate distributions of the floor masses. Each of the buildings was subsequently designed as a real building would be, following Eurocodes 3 and 8 for earthquake resistant steel structures, using a more conservative *q* factor (behaviour or response reduction factor) of 3.0, instead of the code maximum 4.0. Accidental design eccentricities were accounted for by displacing the CM ±0.05L in the *x* and *y* directions.

For seismic actions, the response spectrum method was used with the design spectrum specified by the Greek code for PGA=0.24 g (zone II) and soil category B. This is shown by the smooth line in Fig. 3. Through the design process, the originally selected mass eccentricities led to uneven distributions of stiffness and strength in each building, so that the final natural eccentricities in each case, estimated for each floor as suggested by Stathopoulos and Anagnostopoulos (2005), were less than the original mass eccentricities and included stiffness eccentricity components.

In addition to these buildings, a variant set of similar buildings with the same geometry and mass distribution was designed having only one difference: the accidental design eccentricity (ADE) specified by the code was set to zero, while all other code checks and requirements were fulfilled. Thus, for each normally designed building there is its counterpart designed by assuming

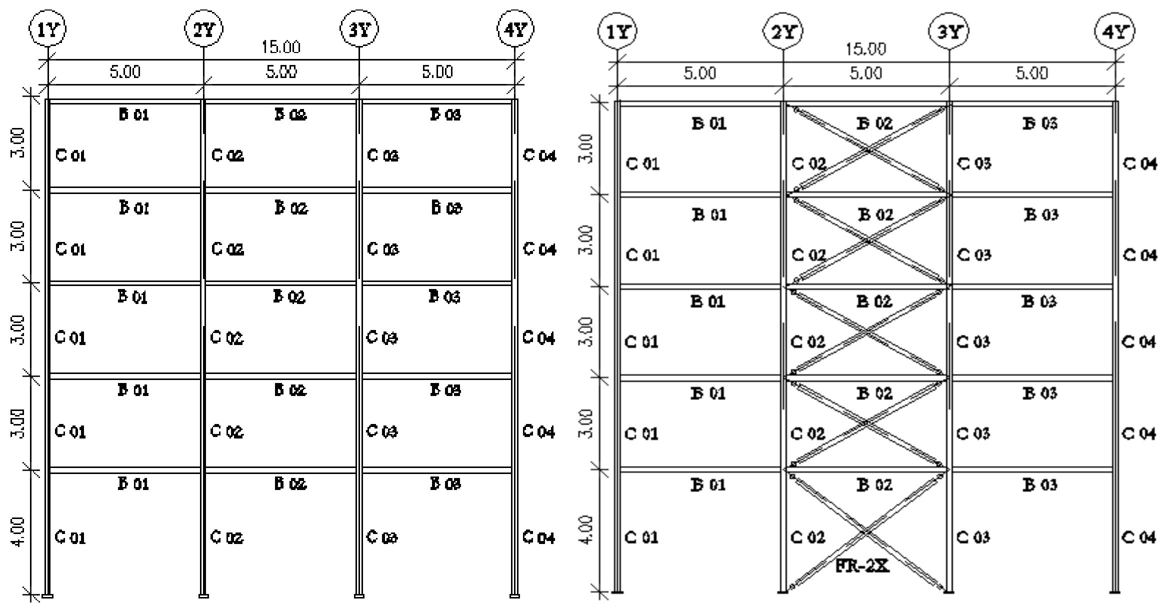


Fig. 2 Elevations of frames without and with cross braces

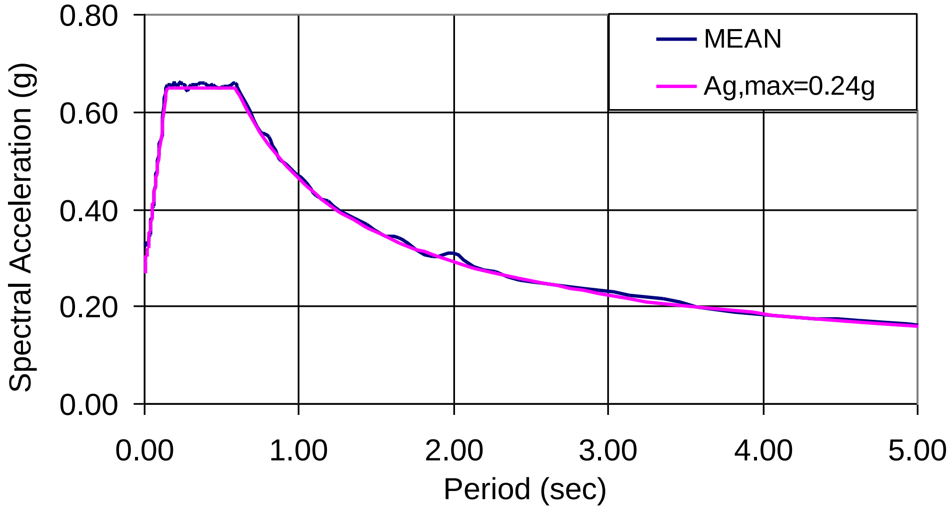


Fig. 3 Design spectrum and mean spectrum of the ten semi-artificial motions

Table 1 Physical eccentricities of the designed buildings with accidental eccentricities **ADE=0.00** and **ADE=0.05L**

Number of stories	ε_m	ADE =0.00				ADE =0.05L			
		Torsionally Stiff		Torsionally Flexible		Torsonally Stiff		Torsionally Flexible	
		ε_x	ε_y	ε_x	ε_y	ε_x	ε_y	ε_x	ε_y
3	0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.10	0.050	0.062	0.095	0.094	0.043	0.064	0.091	0.087
	0.20	0.114	0.110	0.165	0.146	0.110	0.105	0.160	0.140
5	0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.10	0.036	0.041	0.098	0.094	0.034	0.041	0.086	0.078
	0.20	0.110	0.104	0.168	0.153	0.105	0.100	0.160	0.145

ADE=0.0. The mean of these eccentricities for all building stories are listed in Table 1, while the relative positions of the stiffness and mass centres of the eccentric building with e_m=0.10L, are indicated in the layouts in Fig. 1. In the same figure, following typical terminology for torsional problems, the so called “stiff” and “flexible” edges are shown for both earthquake directions *X* and *Y*.

The effect of this design difference on the lower natural periods of the various building sets can be seen in Table 2 for the torsionally stiff buildings and in Table 3 for the torsionally flexible buildings. We see that the resulting differences in natural periods can in some cases be significant. In the same Tables, the Ω factors are also listed.

Finally, to test what happens when an actual, accidental mass eccentricity (AME) is introduced in an existing (already designed) building, the mass centres of all the above buildings were shifted by ±0.05L and the buildings were analysed for the earthquake set described above. This was done for both sets of buildings, i.e., those designed for ADE = 0.0 and those designed for ADE = 0.05L.

Table 2 Fundametal periods and ratio Ω of torsionally stiff buildings designed with accidental eccentricity **ADE=0.00** and **ADE=0.05L**

Number of stories	ε_m	ADE =0.00					ADE =0.05L				
		T_x	T_y	T_θ	Ω_x	Ω_y	T_x	T_y	T_θ	Ω_x	Ω_y
3	0.00	0.59	0.58	0.35	1.69	1.66	0.58	0.57	0.34	1.71	1.68
	0.10	0.58	0.58	0.34	1.71	1.71	0.55	0.55	0.33	1.67	1.67
	0.20	0.58	0.57	0.33	1.76	1.73	0.54	0.54	0.31	1.74	1.74
5	0.00	0.89	0.92	0.55	1.62	1.67	0.86	0.85	0.52	1.65	1.63
	0.10	0.90	0.90	0.54	1.67	1.67	0.80	0.80	0.48	1.67	1.67
	0.20	0.83	0.80	0.49	1.69	1.63	0.79	0.76	0.47	1.68	1.62

Table 3 Fundametal period and ratio Ω of torsionally flexible buildings designed with accidental eccentricity **ADE=0.00** and **ADE=0.05L**

Number of stories	ε_m	ADE =0.00					ADE =0.05L				
		T_x	T_y	$T_{\theta 0.77}$	Ω_x	Ω_y	T_x	T_y	T_θ	Ω_x	Ω_y
3	0.00	0.55	0.54	0.77	0.71	0.70	0.53	0.51	0.77	0.69	0.66
	0.10	0.56	0.60	0.90	0.62	0.67	0.51	0.48	0.78	0.65	0.62
	0.20	0.51	0.53	0.96	0.53	0.55	0.47	0.49	0.84	0.56	0.58
5	0.00	0.80	0.86	1.14	0.70	0.75	0.74	0.80	1.05	0.70	0.76
	0.10	0.80	0.86	1.24	0.65	0.69	0.71	0.75	1.10	0.65	0.68
	0.20	0.73	0.84	1.34	0.54	0.63	0.68	0.76	1.22	0.56	0.62

3. Nonlinear 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 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 (Fig. 4).

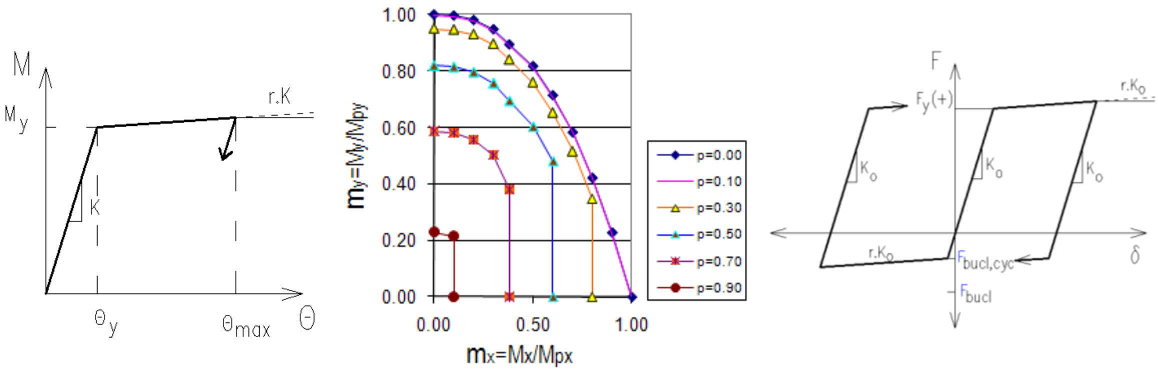
The basic measure used to assess the severity of inelastic response is the ductility factor of the various members. For bracing members the ductility factor is defined as

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

where u_p is the maximum plastic member elongation and u_y the elongation at first yield.

For beams and beam-columns the rotational ductility factor has traditionally been defined as

$$\mu_\theta = 1 + \left(\frac{\theta_p}{\theta_y} \right) \tag{2}$$



(a) (b) (c)

Fig. 4 (a) Nonlinear moment-rotation relations for beam-columns, (b) Column M-N interaction diagram and (c) nonlinear force deformation diagram for braces

where θ_p is the maximum plastic hinge rotation at either end of a member (beam or column) and θ_y is a normalizing "yield" rotation, typically set equal to θ_y=$M_y l$/6EI. For columns, the yield moment M_y is usually taken to correspond to the yield moment under the action of gravity loads. In the present study, an alternative definition of the rotational ductility factor, based on the post yield maximum plastic moment has been used (Anagnostopoulos 1981)

$$\mu = 1 + \left(\frac{\Delta M}{p \cdot M_y} \right) \tag{3}$$

where $\Delta M = M_{\max} - M_y$, M_y=yield bending moment and p=0.05, the strain hardening ratio. In addition to the above measures, peak floor displacements and interstory drifts are used to assess the inelastic behavior of the buildings.

As input for the nonlinear dynamic analyses, ten sets of two component semi-artificial motion pairs were applied. 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*. 1994). Results were excellent, as indicated in Fig. 3 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.

4. Torsionally stiff buildings

4.1 Response of the as designed buildings

Here we will present response comparisons of torsionally stiff buildings designed without and with accidental eccentricity (ADE=0.0 and =±0.05L). The masses of these buildings are identical in size and location as in the building design. This is noted to point the difference from the buildings in the next section, which are analyzed for different mass distributions compared to those used for the building design. The compared response parameters are mean values of the corresponding peaks of response from the 10 analyses described above. These comparisons are shown in Figs. 5 to 7 and all are for the edge frames of the buildings parallel to the *y* axis. The only graph in the top of each Figure is for the symmetric building (e_m=0.0), (same response of both edge frames), while the lower graphs are for eccentric buildings with e_m=0.20L**,** the left for frame 1-Y (stiff side) and the right for frame 4-Y (flexible side). The two lines in each graph, shown with different point marks and colors, correspond to designs with ADE=0.0 and ADE=±0.05L as per Eurocode 8 (ADE=Accidental Design Eccentricity). Fig. 5 shows the peak displacement profiles for the two designs, while Figs. 6 and 7 give the height variation of peak ductility demands for beams and brace members, respectively. The main observation here is that the response difference for the two design strategies, i.e., with ADE=0.0 and ADE=±0.05L (code) is really quite small, negligible we might say. This is especially so for the symmetric building, while contrary to what one may have expected intuitively, the differences increase for the eccentric building. But even in this case, the reduction of the peak ductility factor is negligible for the beams, becoming somewhat more noticeable e.g., for the braces of the eccentric building with e_m=0.20L**,** where the maximum value of about 3.8 in the 2^{nd} story of the flexible edge, reduces to about 3.4 as a consequence of considering ADE=±0.05L, a reduction of about 10%.

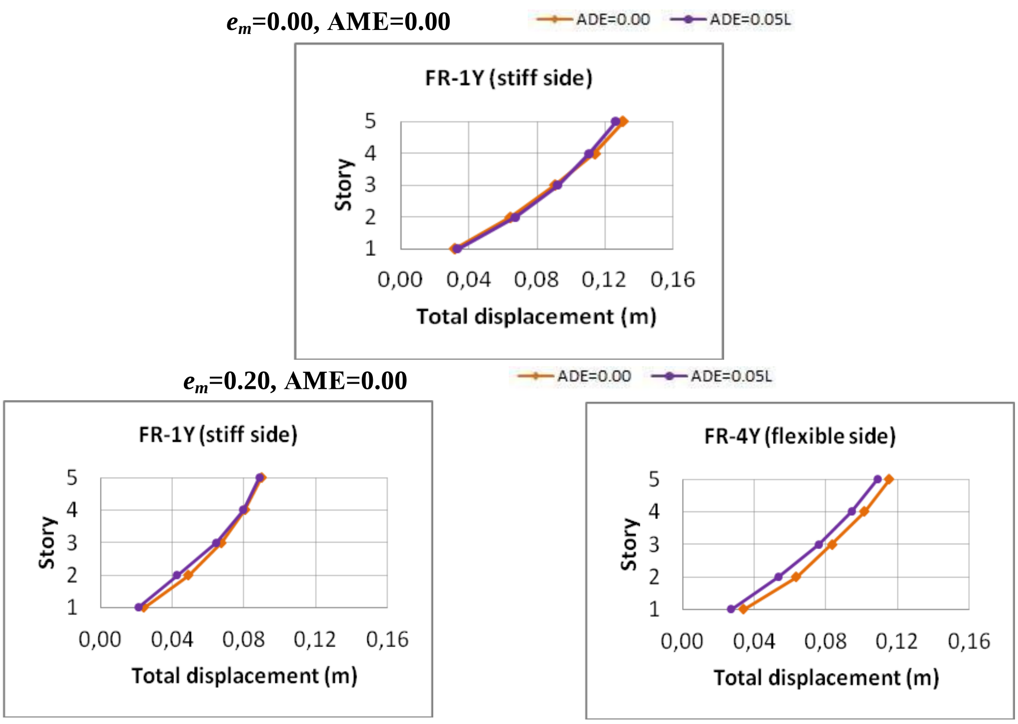


Fig. 5 Effects of ADE on peak, top story *Y* edge displacements of symmetric and eccentric, 5-story, torsionally stiff buildings

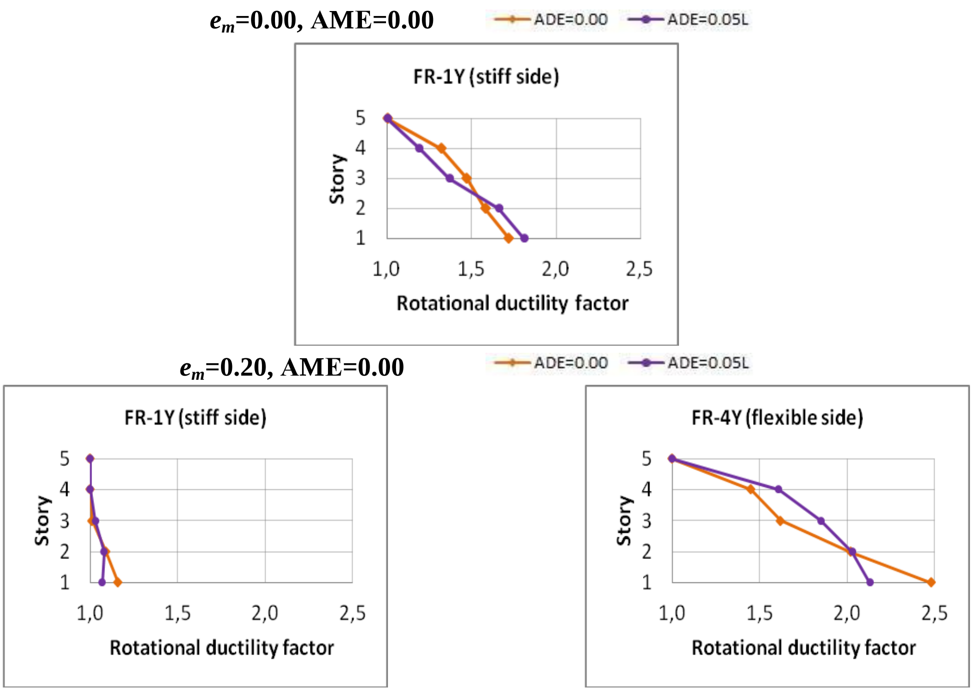


Fig. 6 Effects of ADE on beam ductility demands of symmetric and eccentric 5-story, torsionally stiff buildings

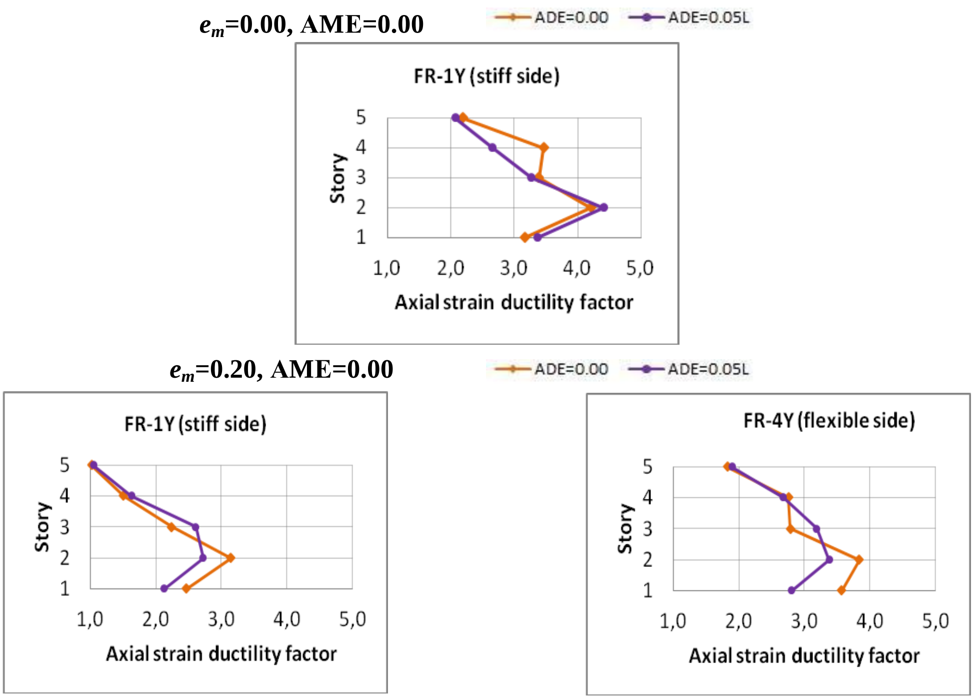


Fig. 7 Effects of ADE on brace ductility demands of, symmetric and eccentric 5-story, torsionally stiff buildings

4.2 Response of buildings with an induced accidental mass eccentricity (AME) of ±0.05L

In the previous section the comparisons were among “as designed” buildings, i.e., the buildings analyzed and compared were the same as designed, their difference being only in that one group was designed with ADE=0.0 and the other with ADE=±0.05L. In this section we will present was designed. Our first comparisons, shown in Figs. 8, 9 and 10, are again for the response of the buildings along the *y* axis and show total displacements, ductility demand factors for beams of the edge frames and ductility demand factors for bracings, respectively. The top graphs in all 3 Figures compare the responses of the stiff edge frame FR-1Y (left) and of the flexible edge frame FR-4Y (right) for a symmetric building (e_m=0.0) designed with ADE=0.0 and in which an accidental mass eccentricity AME=±0.05L is introduced to simulate a real, accidental, mass eccentricity.

In all these graphs, the blue line represents building response for AME=0.0, therefore the differences between this and the other two lines, represents the effects of an accidental mass eccentricity of ±0.05L on the corresponding response variable. We see that the largest such effect is on the ductility factors of the beams both on the stiff as well as the flexible edges. A peak mean, beam ductility factor of about 1.7 in the symmetric case with no design provision for accidental eccentricity (ADE=0.0), was increased to about 2.70 as a result of AME=±0.05L. Note that this AME has destroyed the building symmetry and now we observe the well-known difference reported in the past (e.g., see Stathopoulos and Anagnostopoulos 2006, 2010, Kyrkos and Anagnostopoulos 2011a, 2011b, 2012) in ductility demands between the stiff and flexible edges

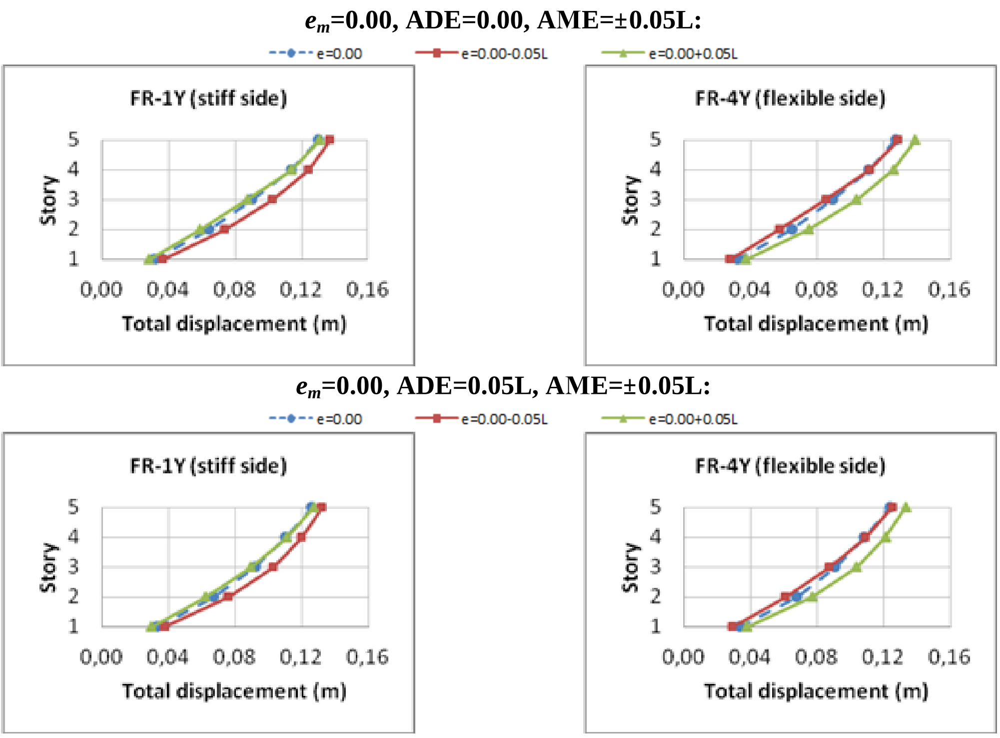
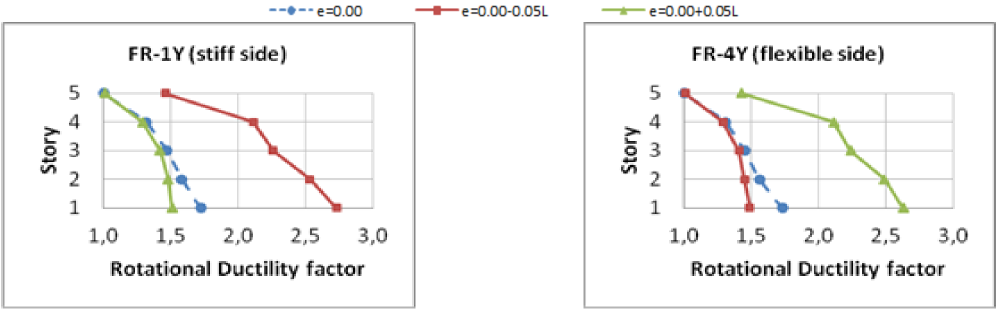


Fig. 8 Effects of induced accidental mass eccentricity, on the top story, *Y* edge displacement of 5-story symmetric, torsionally stiff buildings, designed without (top) and with (bottom) ADE

e_m=0.00, ADE=0.00, AME=±0.05L:



e_m=0.00, ADE=0.05L, AME=±0.05L:

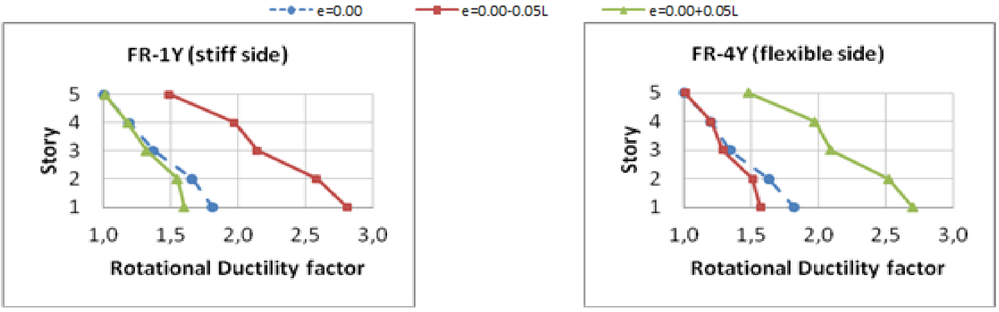
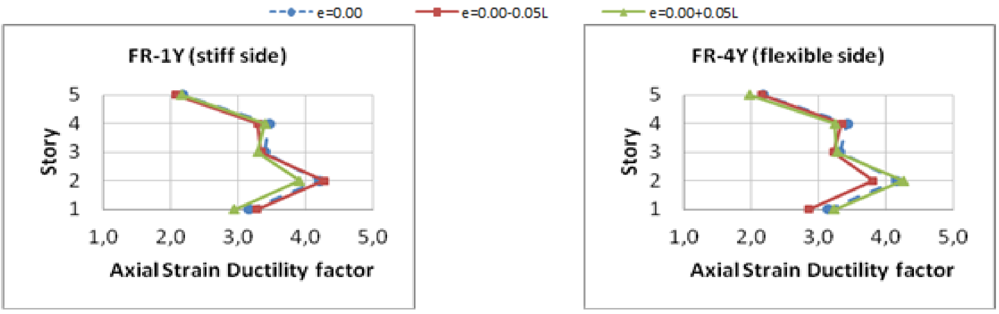


Fig. 9 Effects of induced accidental mass eccentricity, on beam ductility demands of 5-story symmetric, torsionally stiff buildings, designed without (top) and with (bottom) ADE

e_m=0.00, ADE=0.00, AME=±0.05L:



e_m=0.00, ADE=0.05L, AME=±0.05L:

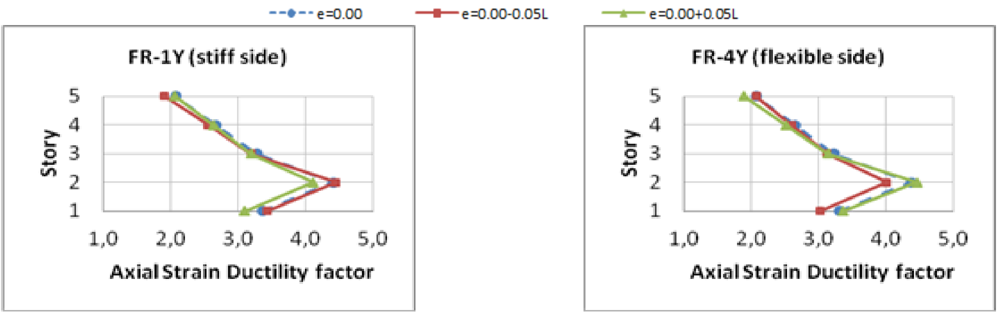


Fig. 10 Effects of induced accidental mass eccentricity, on brace ductility demands of 5-story symmetric, torsionally stiff buildings, designed without (top) and with (bottom) ADE

of an asymmetric building. To assess, however, the effectiveness of the code ADE provision, we must compare the upper with the lower graphs of Figs. 8 to 10. We see then that the observed differences are barely noticeable; indicating that in this case at least, an ADE of ±0.05L affects very little the structural peak inelastic response and could well be forgotten.

The next set of figures, Figs. 11, 12 and 13 are similar to the previous three with only one difference: here the examined buildings are not originally symmetric but eccentric with initial mass eccentricity e_m=0.20L**.**The behavior is similar to that observed for the symmetric buildings in Figs. 8 to 10. Here, for both cases of ADE, zero and ±0.05L, the effects of accidental mass eccentricity, AME=±0.05L , are negligible as far as top story displacements are concerned, while they cause a maximum increase in beam ductility demands from 2.5 to about 2.6 (Fig. 12) and in braces from 3.8 to 3.9 (Fig. 13) in the case of ADE=0.0. The corresponding changes when ADE =±0.05L are from 2.2 to 2.3 for beam ductility and from 3.4 to 3.5 for brace ductility. Clearly such differences can be neglected. In terms of the ADE effectiveness, a comparison of the top graphs to the lower graphs in all three Figs. show a slightly more beneficial effect of the ADE than in the case of symmetric buildings. Here, the largest ductility factor in the beams has been reduced from ~2.6 to ~2.3 and in the braces from ~3.9 to ~3.5 (Figs. 12 and 13 respectively). We observe again here the counter intuitive, lower effectiveness of the ADE for symmetric buildings compared to eccentric ones having a mass eccentricity of 0.20 L.

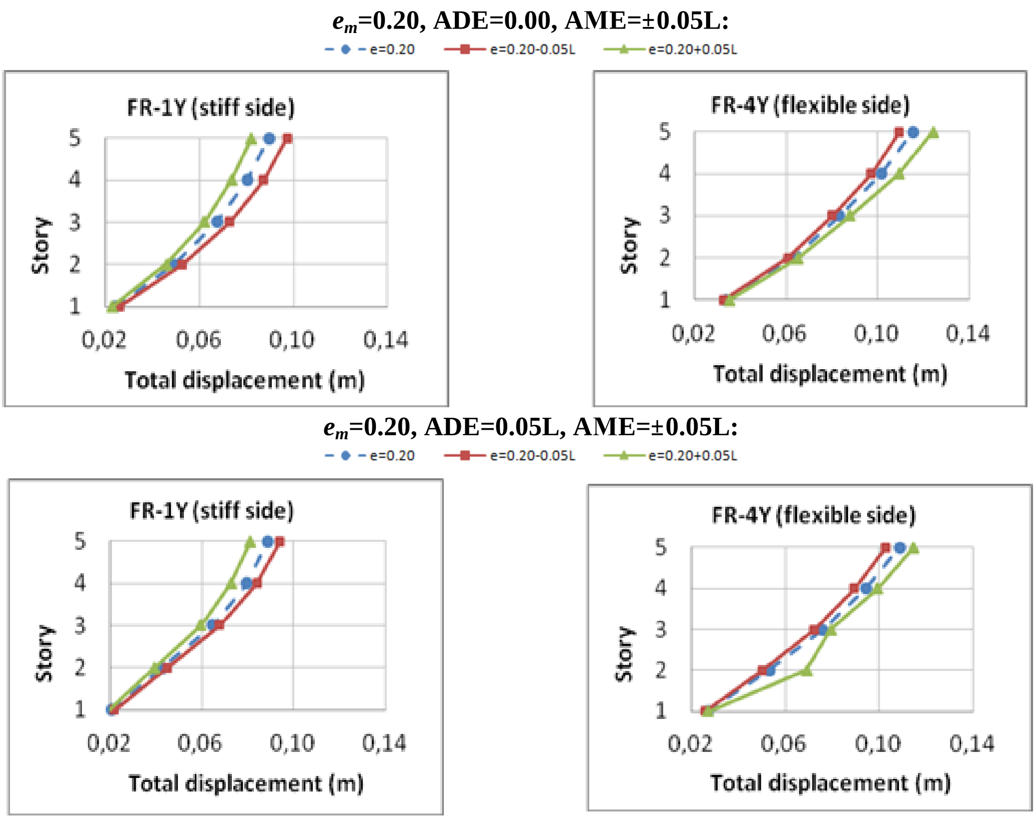


Fig. 11 Effects of induced accidental mass eccentricity, on the top story, *Y* edge displacement of 5-story eccentric, torsionally stiff buildings, designed without (top) and with (bottom) ADE

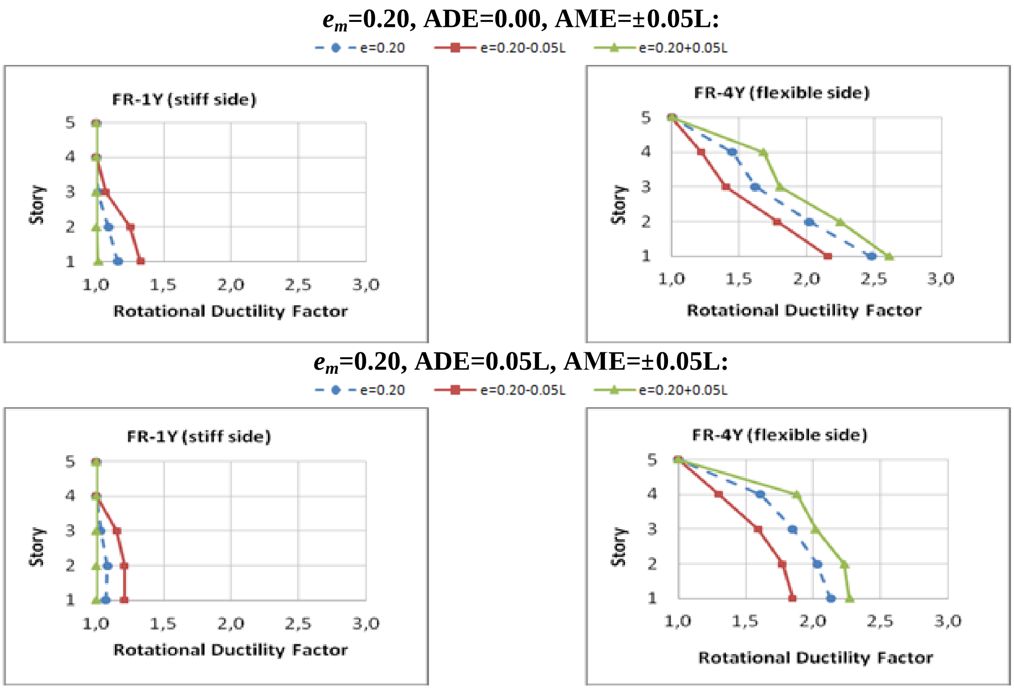


Fig. 12 Effects of induced accidental mass eccentricity, on beam ductility demands of 5-story eccentric, torsionally stiff buildings, designed without (top) and with (bottom) ADE

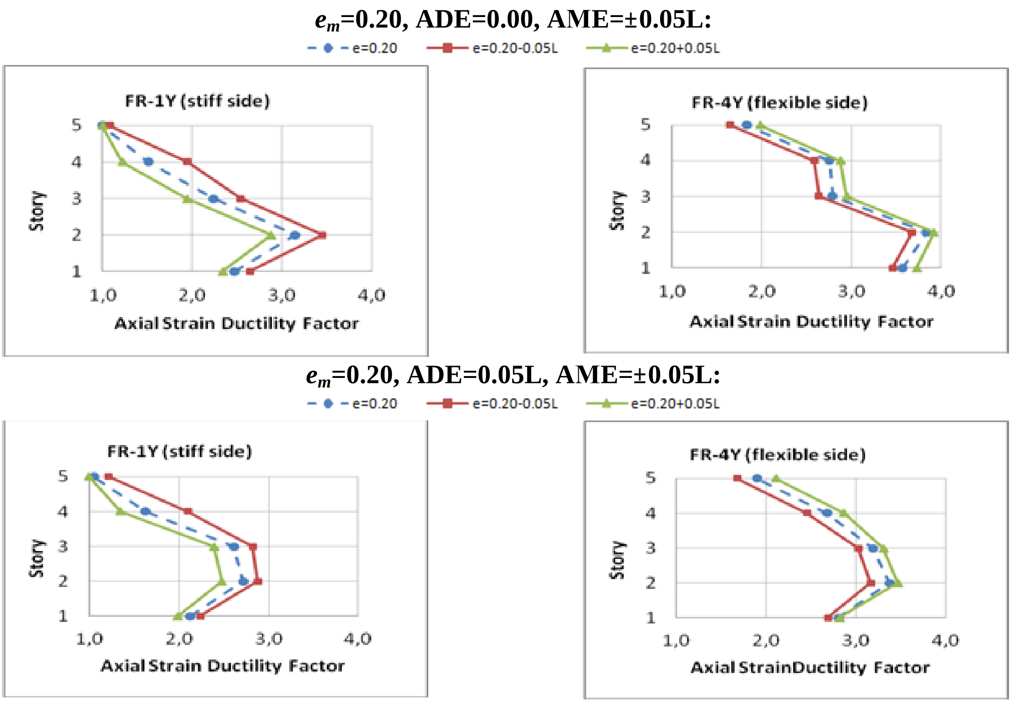


Fig. 13 Effects of induced accidental mass eccentricity, on brace ductility demands of 5-story eccentric, torsionally stiff buildings, designed without (top) and with (bottom) ADE

5. Torsionally flexible buildings

Torsionally flexible buildings are much rarer than torsionally stiff buildings, because the latter are inherently more earthquake resistant with better predictable earthquake response. For this reason, modern codes "penalize" torsionally flexible buildings through additional design and analysis requirements, a fact also contributing to the avoidance of such buildings, whenever possible. However, architectural requirements may lead to such buildings and therefore their earthquake design and response should be studied.

5.1 Response of the as designed buildings

Due to space limitations, here, as in the case of the torsionally stiff buildings, only the results from the 5-story frames will be presented and only for symmetric as well as eccentric with physical mass eccentricity of 0.20L. Figs. 14, 15 and 16 are the corresponding of Figs. 5, 6 and 7. They compare the seismic response of the edge frames in direction *Y* of the "as designed" buildings without and with accidental design eccentricity (ADE=0.0 and ADE=±0.05L). The top single graph is for the symmetric building (e_m=0.0, same response of both edge frames), while the lower graphs are for eccentric buildings with e_m=0.20L, the left for frame 1-Y (stiff side) and the right for frame 4-Y (flexible side). The two lines in each graph, shown with different point marks and colors, correspond to designs with ADE=0.0 and ADE=±0.05L as per Eurocode 8. Again, Fig. 14 shows the peak floor displacement profiles for the two designs, while Figs. 15 and 16 give the height variation of peak ductility demands for beams and brace members, respectively.

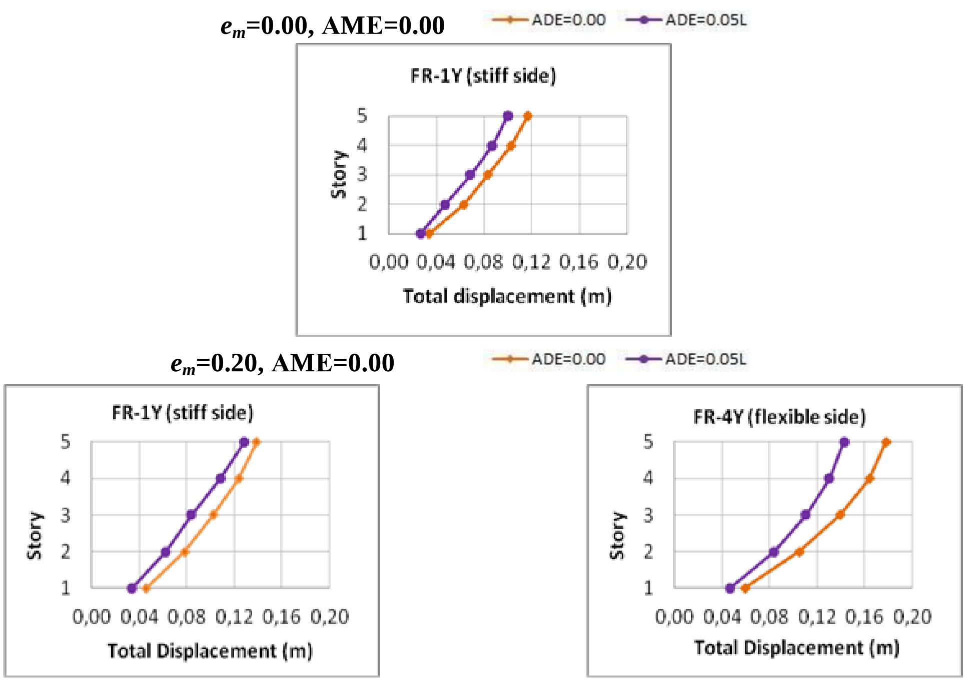


Fig. 14 Effects of ADE on peak, top story *Y* edge displacements of symmetric and eccentric, 5-story, torsionally flexible buildings

The influence of ADE is here somewhat greater than in the torsionally stiff buildings, although not much. As a result of using ADE=±0.05L, peak story displacements in the symmetric building are reduced from ~ 0.116 m to ~ 0.10 m and in the eccentric buildings from ~ 0.18 m to ~ 0.14 m at the flexible edge. In terms of beam ductility, in the symmetric building the reduction is from ~1.3 to ~1.2 while in the eccentric building it is from ~3.3 to ~2.7. Finally, in terms of brace ductility , in the symmetric case the maximum reduces from ~3.9 to ~2.7 , while in the eccentric case it goes from ~4.5 to ~3.3.

5.2 Response of buildings with an induced accidental mass eccentricity (AME) of ±0.05L

Here we repeat the "experiment" of introducing an accidental mass eccentricity, AME=±0.05L and we look at the effectiveness of ADE to reduce ductility requirements both in symmetric and eccentric buildings designed with and without the ±0.05L ADE. Figs. 17, 18, 19 are the corresponding to Figs. 8, 9, 10 that give results for the torsionally stiff buildings. They are for symmetric buildings (e_m=0.0) designed with ADE=0.0 and in which an accidental mass eccentricity AME=±0.05L is introduced to simulate a real, accidental, mass eccentricity. As before, in all these graphs, the blue line represents building response for AME=0.0, therefore the differences between this and the other two lines, represents the effects of an accidental mass eccentricity of±0.05L on the corresponding response variable. We see here that the effects are very similar to those observed earlier for the torsionally stiff buildings. In terms of peak top story displacements, an AME=±0.05L causes increases from 0.12 m to 0.13 m when ADE=0.0 and from 0.10 m to 0.13 m when ADE=±0.05L, suggesting a somewhat "detrimental" effect of ADE. In

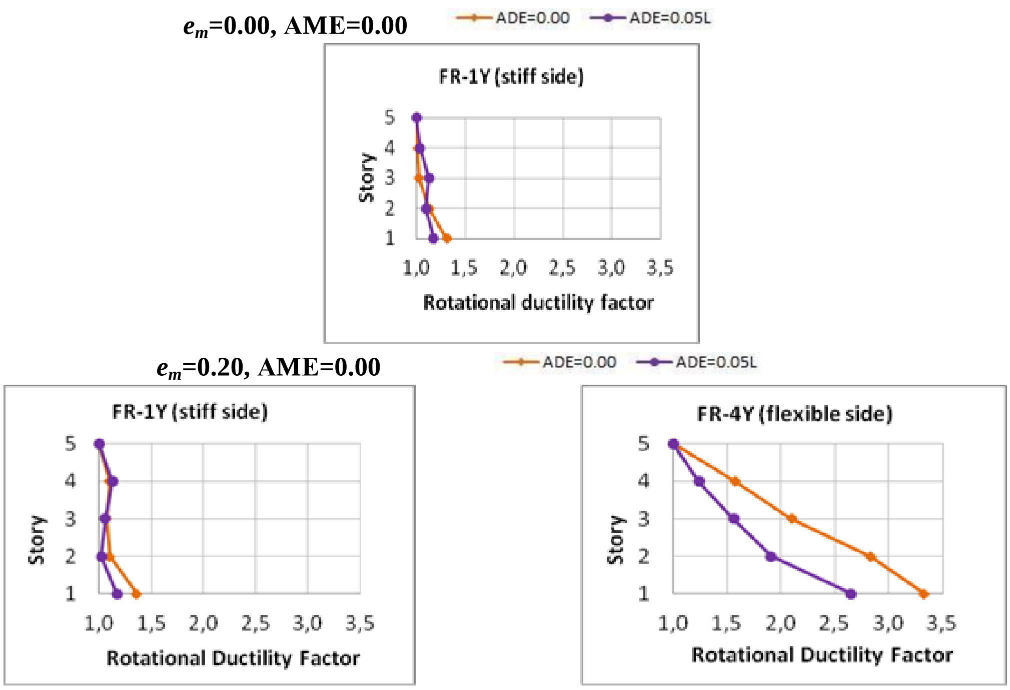


Fig. 15 Effects of ADE on beam ductility demands of symmetric and eccentric, 5-story, torsionally flexible buildings

terms of beam ductility, the AME=±0.05L causes an increase from 1.3 to 2.0 in the ADE=0.0 case and an increase from 1.2 to 2.0 in the ADE=±0.05L case. In the braces, the corresponding increases were from 3.8 to 4.0 (when ADE=0.0) and negligible (when ADE=±0.05L). The only noticeable difference between the torsionally stiff and torsionally flexible buildings can be seen by comparing Figures 10 and 19, where we will notice that for the torsionally flexible building there is a reduction of the peak brace ductility from ~ 4.1 to ~ 2.9 as a result of using ADE=±0.05L in the design, while no such reduction can be observed in the torsionally stiff building. The effect of ADE can be best seen in the upper parts of Figs. 5, 6, 7 and 14, 15, 16.

The last set of graphs are given in Figs. 20, 21, and 22 and compare the effects of an accidental mass eccentricity AME=±0.05L for eccentric buildings having an initial mass eccentricity e_m=0.20L**.** They are the equivalent of Figs. 11, 12 and 13, describing the same effect for the torsionally stiff, eccentric buildings. As expected, in terms of top story edge displacements, the torsionally flexible building exhibits somewhat greater response than the torsionally stiff building and somewhat greater spread in the displacements of the “flexible” edge in the case of ADE=0.0. Here, an accidental mass eccentricity of AME=±0.05L causes a top displacement increase from 0.18 m to 0.21 m when ADE=0.0 (no consideration of accidental design eccentricity) and from 0.14 m to 0.19 m when ADE=±0.05L is used in design. In terms of beam ductility, an AME=±0.05L causes an increase from 3.3 to 4.0 when ADE=0.0 and from 2.7 to 3.5 when ADE=±0.05L.

Finally, in terms of brace axial strain ductility, an AME=±0.05L causes an increase from 4.5 to 5.0 when ADE=0.0 and from 3.3 to 4.0 when ADE=±0.05L.

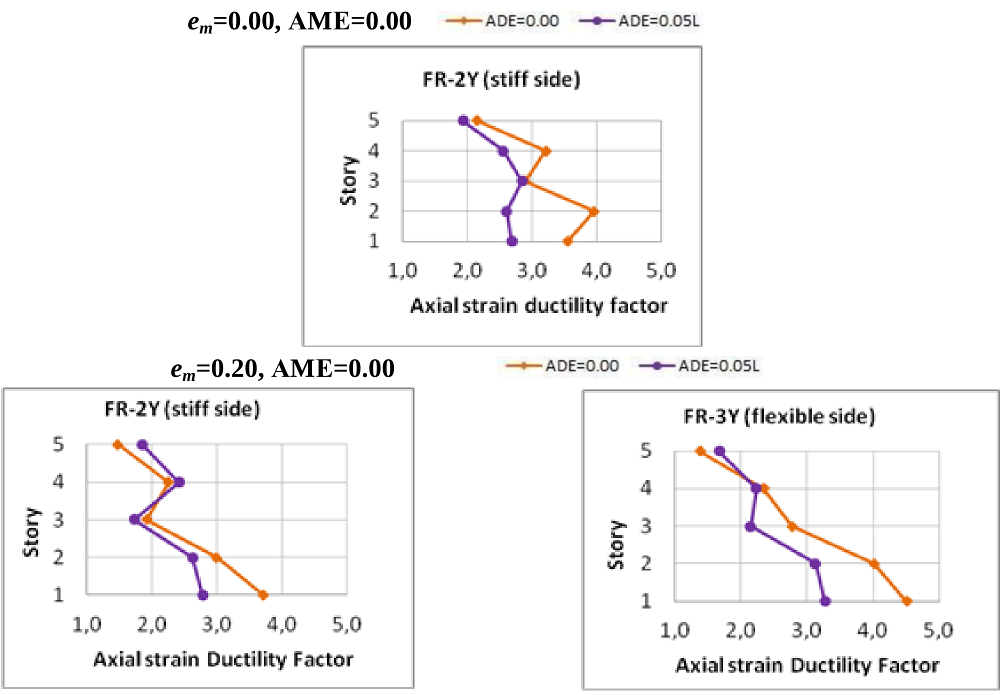


Fig. 16 Effects of ADE on brace ductility demands of symmetric and eccentric, 5-story, torsionally flexible buildings

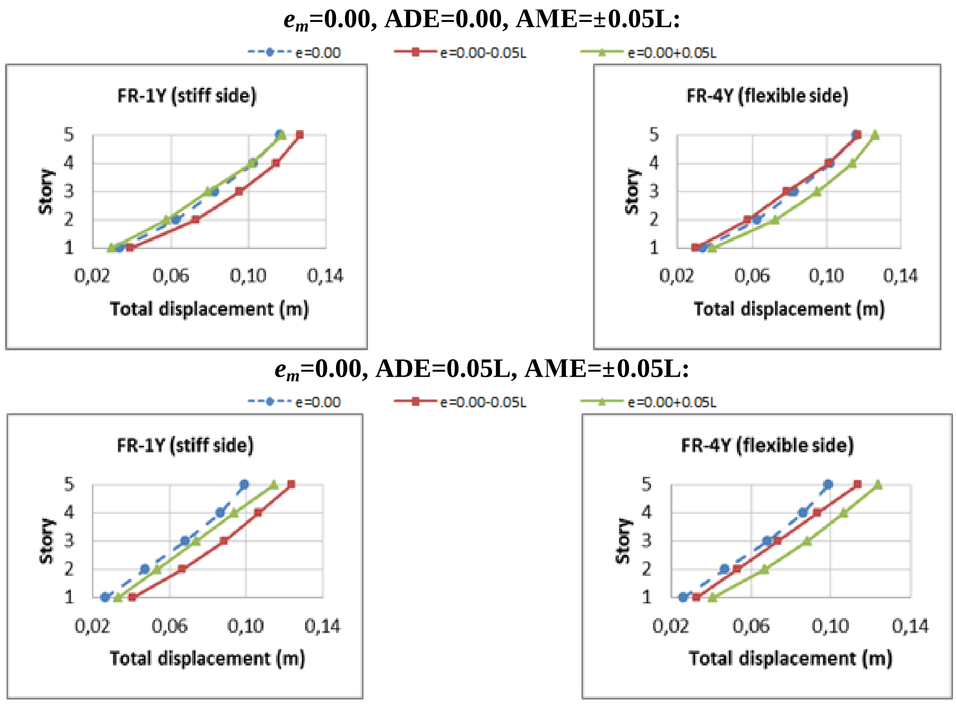


Fig. 17 Effects of induced accidental mass eccentricity, on the top story, *Y* edge displacement of 5-story symmetric, torsionally flexible buildings, designed without (top) and with (bottom) ADE

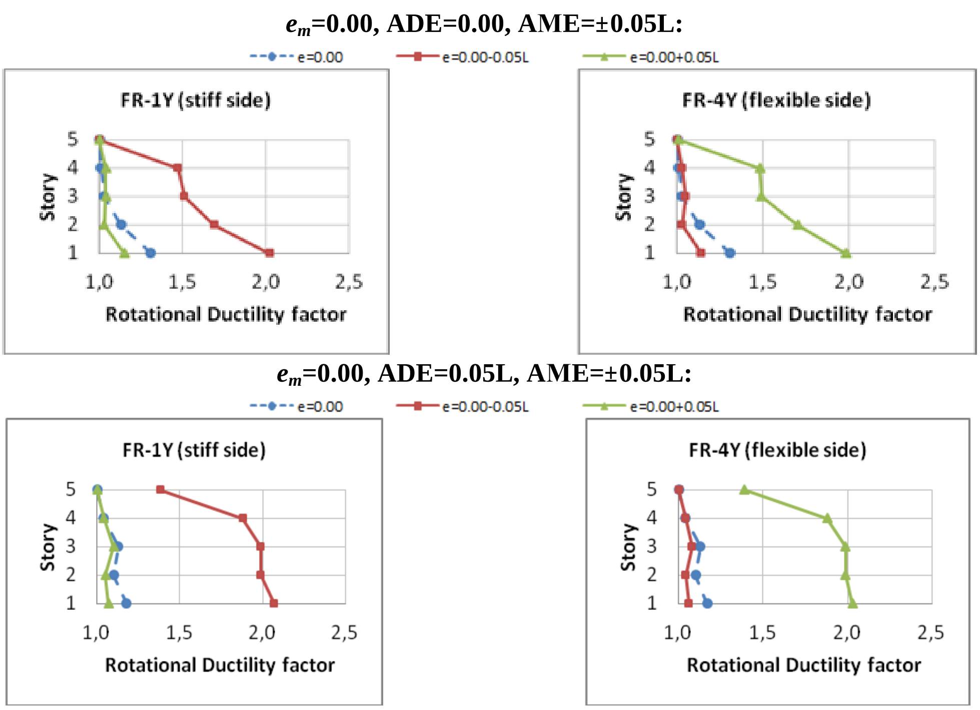


Fig. 18 Effects of induced accidental mass eccentricity, on beam ductility demands of 5-story symmetric, torsionally flexible buildings, designed without (top) and with (bottom) ADE

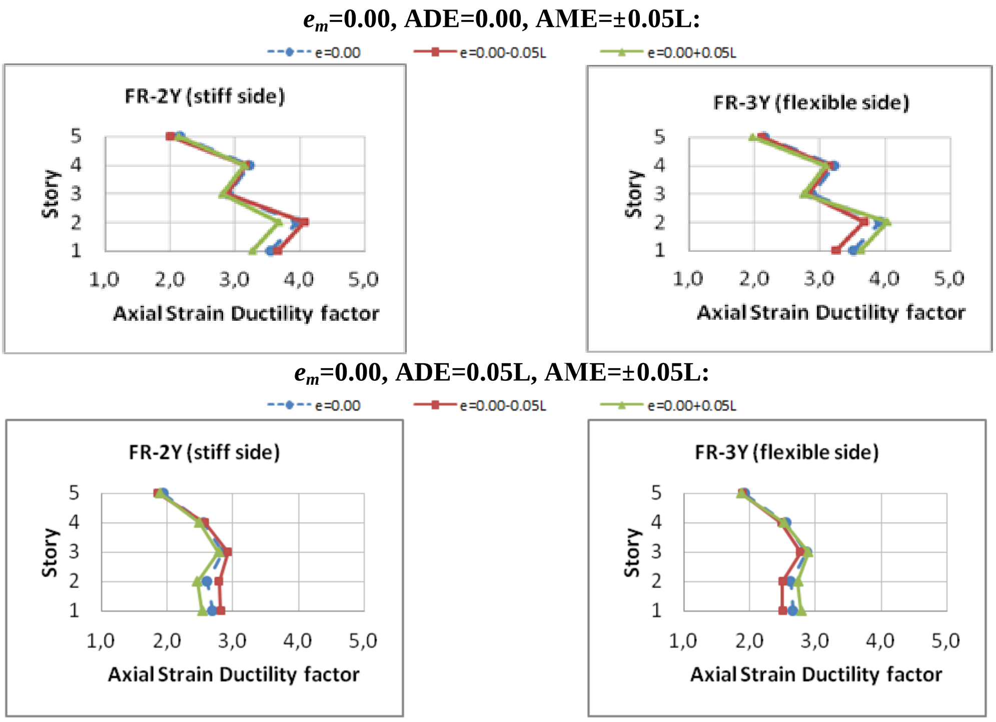


Fig. 19 Effects of induced accidental mass eccentricity, on brace ductility demands of 5-story symmetric, torsionally flexible buildings, designed without (top) and with (bottom) ADE

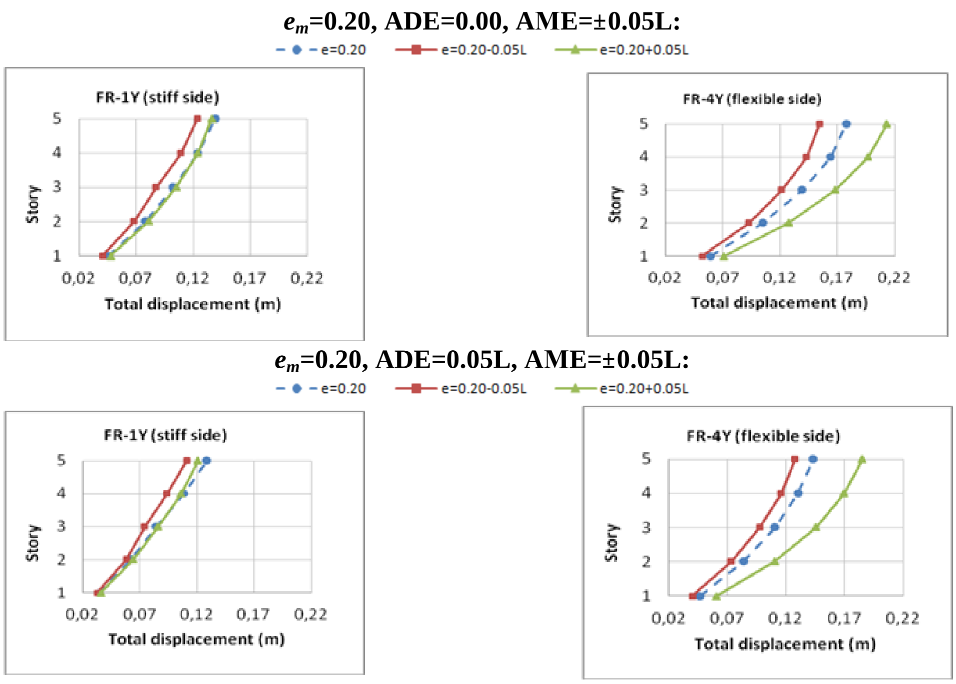
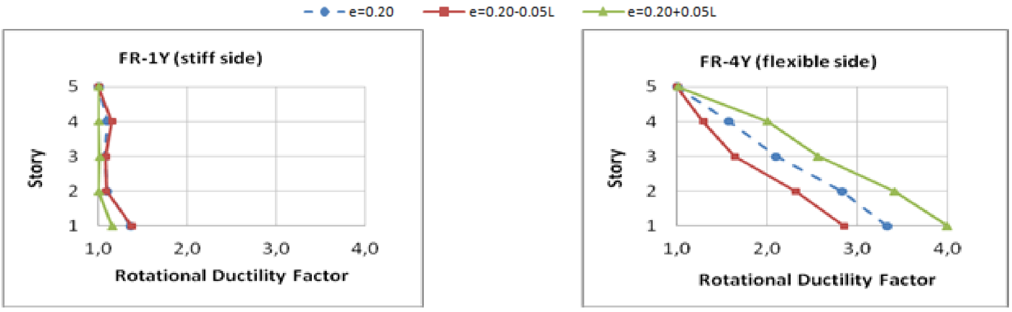


Fig. 20 Effects of induced accidental mass eccentricity, on the top story, *Y* edge displacement of 5-story eccentric, torsionally flexible buildings, designed without (top) and with (bottom) ADE

e_m=0.20, ADE=0.00, AME=±0.05L:



e_m=0.20, ADE=0.05L, AME=±0.05L:

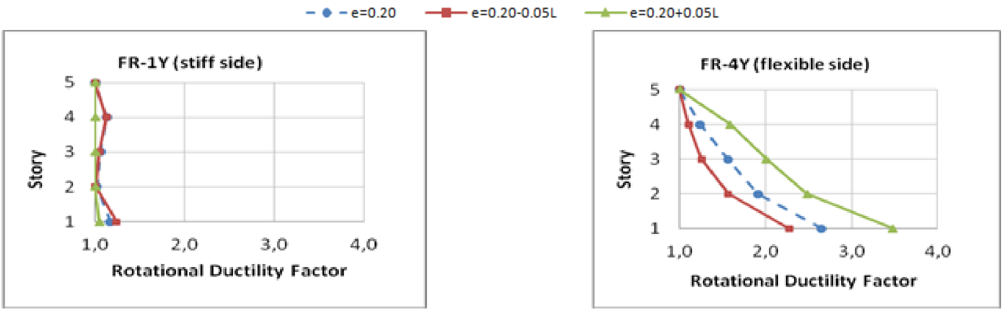
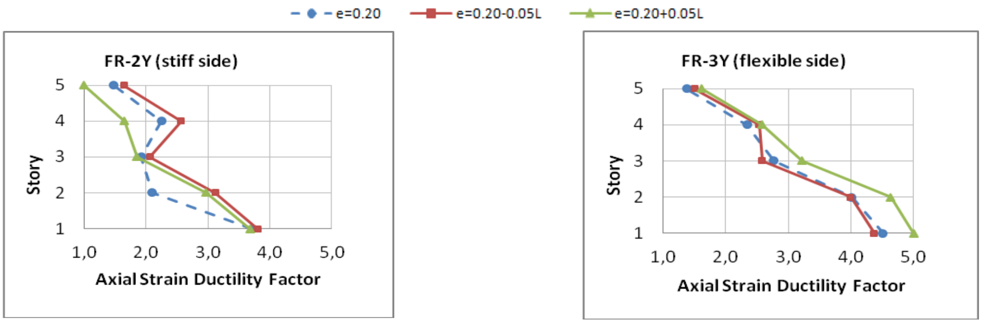


Fig. 21 Effects of induced accidental mass eccentricity, on beam ductility demands of 5-story eccentric, torsionally flexible buildings, designed without (top) and with (bottom) ADE

e_m=0.20, ADE=0.00, AME=±0.05L:



e_m=0.20, ADE=0.05L, AME=±0.05L:

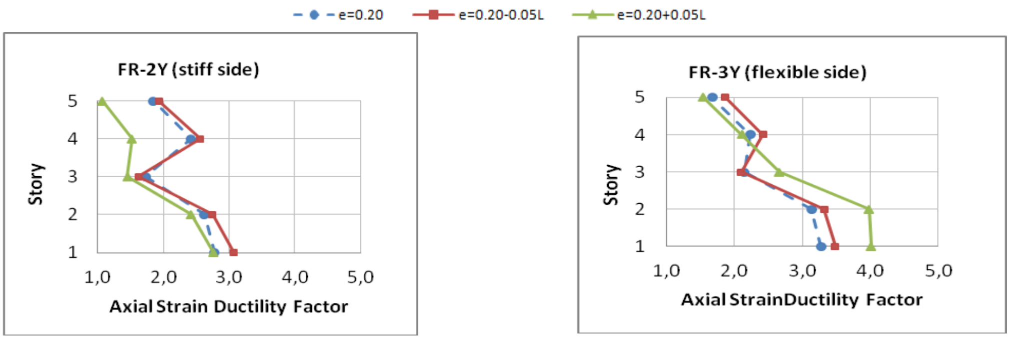


Fig. 22 Effects of induced accidental mass eccentricity, on brace ductility demands of 5-story eccentric, torsionally flexible buildings, designed without (top) and with (bottom) ADE

6. Summary and Conclusions

The results presented above are summarized in Tables 4 and 5, where we have also included results for the cases with e_m=0.10L, for which, due to space limitations, no graphs were presented earlier (similar to those for e_m=0.0.0 and e_m=0.20L). Table 4 presents the changes of peak inelastic response parameters, namely maximum top story displacement, beam ductility factors and brace ductility factors as a result of introducing an "accidental" mass eccentricity AME=±0.05L to the following sets of 5-story buildings: torsionally stiff and torsionally flexible, symmetric (e_m=0.0) and eccentric (e_m=0.10L and e_m=0.20L) and with each of them designed both for zero ADE and for ADE=0.05L. Overall there were small increases in the peak response parameter values, indicated by the arrows, where the left values are those before introducing the accidental mass eccentricity and the values at right after. These increases are greater for symmetric buildings because the accidental eccentricity makes them asymmetric and as it has been shown in the past (Stathopoulos and Anagnostopoulos 2005, Kyrkos and Anagnostopoulos 2011a) eccentric buildings designed by EC8 exhibit an increase in ductility demands at their flexible sides. Wherever the change was in the third significant digit it has been marked as NEGLIGIBLE. These increases were the result of the following factors: (a) change in the structural periods due to changes in member stiffness, (b) small changes in member strengths, and (c) physical eccentricity changes by ±0.05L. Overall they are small and, moreover, comparable for the cases of deign with ADE=0.0 and ADE=0.05L.

A more clear and direct assessment of the influence of ADE on the response of as designed buildings with no additional eccentricity introduced (to simulate the accidental) can be obtained from Table 5, where the same parameters as before are compared. Each column has two values

Table 4 Peak response parameter changes due to an introduced mass (accidental) eccentricity of 0.05L

Response parameter	**SYMMETRIC, e_m= 0.0**		**ECCENTRIC, e_m= 0.10L**		**ECCENTRIC, e_m= 0.20L**	
	ADE= 0.0	**ADE=0.05L**	**ADE= 0.0**	**ADE=0.05L**	**ADE= 0.0**	**ADE=0.05L**
	TORSIONALLY STIFF, 5-STORY					
δ_{top} **(m)**	~ 0.13 → 0.14	~ 0.12 → 0.13	~ 0.13 → 0.14	NEGLIGIBLE	0.115→ 0.125	0.11→ 0.12
μ_{beam}	1.7 → 2.7	1.8 → 2.8	2.0 → 2.45	1.9 → 2.2	2.5→ 2.6	2.2→ 2.3
μ_{brace}	NEGLIGIBLE	NEGLIGIBLE	4.3 → 4.6	NEGLIGIBLE	3.8→ 3.9	3.4→ 3.5
	TORSIONALLY FLEXIBLE, 5-STORY					
δ_{top} **(m)**	~ 0.11 → 0.13	~ 0.10 → 0.12	~ 0.15 → 0.18	~ 0.12 → 0.14	0.18→ 0.21	0.14→ 0.19
μ_{beam}	1.3 → 2.0	1.2 → 2.1	2.3 → 2.9	2.0 → 2.5	3.3→4.0	2.7→ 3.5
μ_{brace}	3.9 → 4.0	NEGLIGIBLE	NEGLIGIBLE	NEGLIGIBLE	4.5→ 5.0	3.3→ 4.0

Table 5 Peak response parameter changes due to design for an accidental eccentricity ADE = ± 0.05L

Response parameter	**TORSIONALLY STIFF, 5-STORY**			**TORSIONALLY FLEXIBLE, 5-STORY**		
	SYMMETRIC e_m=0.0	**ECCENTRIC e_m=0.10L**	**ECCENTRIC e_m=0.20L**	**SYMMETRIC e_m=0.0**	**ECCENTRIC e_m=0.10L**	**ECCENTRIC e_m=0.20L**
δ_{top} **(m)**	NEGLIGIBLE	~ 0.13 → 0.12	0.115 → 0.11	~ 0.11 → 0.10	~ 0.16 → 0.12	0.18 → 0.14
μ_{beam}	NEGLIGIBLE	NEGLIGIBLE	2.5 → 2.2	1.35 → 1.2	2.4 → 1.95	3.3 → 2.7
μ_{brace}	4.3 → 4.4	4.3 → 3.8	3.8 → 3.4	4.0 → 2.9	4.4 → 3.7	4.5 → 3.3

separated by an arrow where the values at left are for design with ADE=0.0 while the values at right are for designs with ADE=0.05L Here we see, reductions, as expected, of the peak parameters as a result of designing for ADE but in one case there was also a small increase (bottom line, first column). However they are generally small. The larger is in the brace ductility factor of the symmetric torsionally flexible building, where the value of 4.0 is reduced to 2.9, which represents a ~28% reduction. All other changes are smaller and some are negligible.

We note that very similar results were obtained with the 3-story building mentioned at the beginning and which are not presented here due to space limitations. Moreover, we must point out that care was exercised in designing all the above buildings to avoid overdesigns by using the minimum sections that would satisfy all code requirements. Therefore, the obtained results reflect clearly the effects of the code provision for ADE, at least for the specific buildings examined and the elastic design method used. These results are in agreement with those reported by Stathopoulos and Anagnostopoulos (2010) for R.C. buildings as well as with DeBock *et al*. (2014). The latter, however, are pertinent to the ADE provisions of the American ASCE norms and the IBC code.

This study indicates that designing torsionally stiff, steel braced frame buildings for an accidental mass eccentricity of 0.05L as per Eurocode 8, has small effects on their inelastic response or in protecting them in case that the "accident" occurs and the mass distribution varies from the design assumptions producing an extra eccentricity of 0.05L. In torsionally flexible buildings, an ADE=0.05L led to ductility demand reductions up to ~28% in braces and up to ~ 18% in beams. Inversely, omission of ADE, increased peak ductility demands in braces by ~ 35% and in beams by ~23% at most. Therefore, these results, similar to those referenced in the preceding paragraph, indicate that the benefits of the code specified ADE are insignificant for the vast majority of torsionally stiff buildings and its removal from the code or replacement by a simpler provision should be examined. On the other hand, ADE appears to be more effective in reducing ductility demands in torsionally flexible buildings. We must note, however, that for the buildings examined in the present study, the ~35% increase in ductility demands due to the omission of ADE from their design, did not raise the peak demands to unacceptable levels.

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