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EARTHQUAKE INDUCED INELASTIC TORSION IN ASYMMETRIC MULTISTORY BUILDINGS

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SUMMARY

The inelastic earthquake response of eccentric, multi-story, frame-type, reinforced concrete buildings is investigated using 3 and 5-story detailed structural models, as well as simplified one-story, shear beam type idealizations, subjected to a set of 10, two-component, semi-artificial motions, generated to match the design spectrum. The buildings were designed according to Eurocode 8 but alternative designs were also produced using the amplified accidental eccentricity of UBC97. It is found that contrary to what the simplified one-story, shear-beam models predict, the so-called "flexible" side frames exhibit higher ductility demands than the "stiff" side frames. The substantial differences in such demands between the two sides and further the relatively small influence of the design accidental eccentricity on the peak inelastic response indices, suggest a need for reassessment of the pertinent code provisions.

INTRODUCTION

Torsion in buildings during earthquake shaking may be caused from a variety of reasons, the most common of which are non-symmetric distributions of mass and stiffness. The torsional response may be intensified in the inelastic range due to increased eccentricities caused by potentially uneven yielding, especially in the perimeter of the structure.. Modern codes (e.g EC8[1], UBC97[2], Greek Code[3]) deal with torsion by placing restrictions on the design of buildings with irregular layouts and also through the introduction of an accidental eccentricity that must be considered in design.

Although the formulation for inelastic, asymmetric, multistory building response under earthquake motions has been around since the early 70ties (Anagnostopoulos[4], Anagnostopoulos et al [5]), most of the published work on this problem till today is based on simplified, one-story models, with simple shear-beam elements for lateral load resistance (e.g. Tso et al [6], Goel et al [7], Rutenberg [8], Stathopoulos et al [9]). On the basis of such models, which were initially of the simplest possible kind (e.g.Bozorgnia et al [10] but subsequently were improved by including biaxial eccentricities and two-component multiple earthquake input (e.g. Goel [11], Stathopoulos et al [12]}, a great number of parametric studies was

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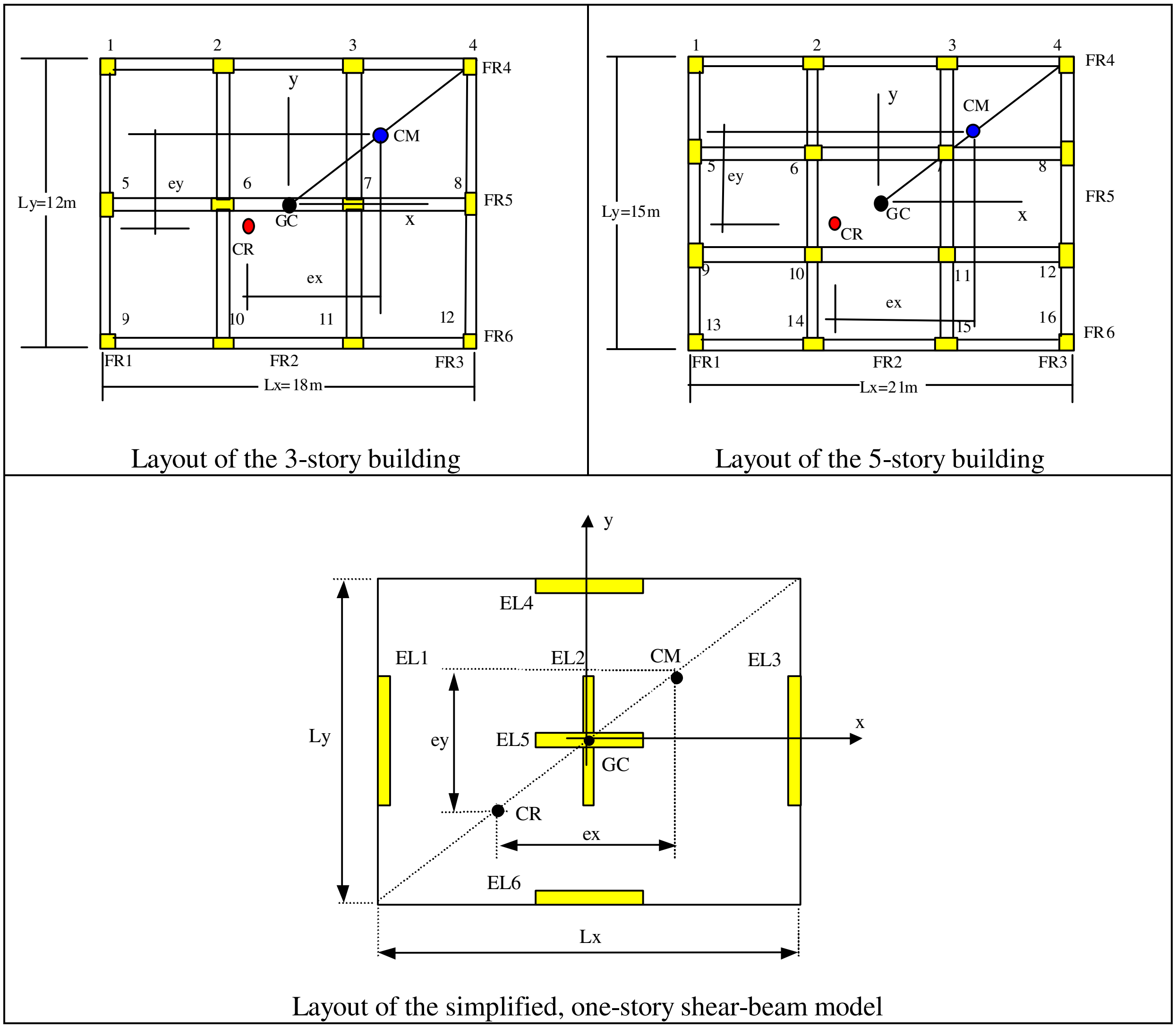
carried out and the adequacy of pertinent code provisions was investigated [e.g. Tso et al [13], Zhu et al [14], De La Llera et al [15], Chandler et al [16]). Due to a number of reasons, e.g. differences in underlying assumptions, conflicting conclusions have often been identified [e.g. Rutenberg [8], [18], Chandler et al [17]).

A more important issue, however, that seems to have escaped the attention of many researchers, has to do with the question of how appropriate is the simplified, one-story, shear beam model to approximate the inelastic earthquake response of real multistory buildings, through which the pertinent code provisions should be evaluated. Some recent comparisons have shown significant qualitative differences in the predicted responses between simplified and detailed models, thus raising strong doubts about code assessments based on the oversimplified one-story inelastic models (Stathopoulos et al [12,19,20,21,22]). These differences are attributed to the following: (a) The stiffness and strength of the resisting elements of the simplified shear-beam model are specified and calculated independent of each other and only for seismic loads. In real buildings, member stiffness, strength and yield deformation are related to each other directly in a way that a change in one parameter entails changes in the other two. This problem has been addressed recently, using again simplified one-story systems with two types of lateral load resisting elements, and interesting conclusions have been drawn (Tso et al, [24]). (b) In real buildings, members are designed for lateral and vertical loads and hence their stiffness and strength, in absolute or relative terms, are different from the corresponding values of the resisting elements of the simplified shear-beam models. Thus, the percentage changes of these quantities caused by applying Code provisions for torsion in real buildings are much smaller than the respective changes in the idealized one-story, shear-beam models and the same should be expected for the pertinent effects on the corresponding responses. (c) Yielding of an end-element of the simplified model implies the practical elimination of the stiffness in that position. A corresponding case in a real building would be the formation of a mechanism at the same side of the building, i.e. the simultaneous yield of all beam and column ends in a given floor of the corresponding frame, which modern codes prevent through capacity design provisions. In real buildings, the post-elastic stiffness of any given frame is a significant fraction of its elastic stiffness, as it is controlled by the substantial number of members, typically columns, that are elastic at any given instant. Thus, there are great differences in the post-elastic eccentricities between real buildings and the shear-beam models.

In view of the shortcomings of the simplified models even for one-story buildings, a need exists for an assessment of inelastic earthquake induced torsion in multistory, frame type, buildings, based on more realistic structural models (e.g. those using plastic hinges). To the best of the authors' knowledge, there are no systematic studies based on such models but only a handful of publications limited either to the investigation of an existing building (Boroschek et al [25]) or to mono-symmetric buildings under one-component motion (De Stefano et al [26], Ghersi et al [27]). The present paper presents results from 3 and 5-story, Reinforced Concrete, space frame buildings with biaxial eccentricity, designed according to EC8 and excited with a set of two-component motions. The effects of the amplified accidental eccentricity according to the American UBC97 (or according to the IBC2000) is also examined by producing a corresponding set of design variants using this instead of the EC8 pertinent specification. The space frame inelastic analyses are carried out using detailed plastic hinge type idealizations for the yielding members. Finally, comparisons are presented with results from simplified, one-story, shear beam type models.

SYSTEMS AND MOTIONS USED

The buildings selected for this study (Figure 1) are three and five-story reinforced concrete space frames, formed by three or four plane frames along the x and y directions. The two end frames parallel to the y direction are labeled Fr1 and Fr3, those parallel to the x direction are Fr4 and Fr6, while Fr2 and Fr5 indicate interior frames parallel to the y and x directions respectively. Note that for the y direction of the 3-story building and for both directions of the 5-story building, Fr2 and Fr5 designate pairs of interior frames. Both buildings have a typical story height of 3.0m and a ground story height of 4.0m. Using appropriate distributions of the floor masses, constant biaxial mass eccentricities were introduced in all the floors (CM on the diagonal, see Figure 1).



Layout of the 3-story building

Layout of the 5-story building

Layout of the simplified, one-story shear-beam model

Figure 1. Three and five-story reinforced concrete buildings

For both buildings, the cases of e_m=0 (symmetric), e_m=0.10d and 0.20d (e_m= physical eccentricity, d = length of diagonal) were examined while for the 3-story building the value e_m=0.30d was also included. Both buildings were designed for gravity and earthquake loads according to EC2 (reinforced concrete) and EC8 (earthquake resistant design), while in addition to the EC8 accidental eccentricity e_{acc}=0.05L alternative designs were produced for e_{acc}=A0.05L, as per UBC97. The dimensioning of the frame members took into account the mass eccentricities and thus stiffness eccentricities were also generated.

Since in multistory buildings, the so called center of rigidity (CR) cannot be really defined, except under very restrictive conditions, an approximate CR was computed herein for reference purposes, on a floor by floor basis as follows:

$$e_{sx} = \frac{\sum_{1}^{m} K_{f-iy}\, x_i}{\sum_{1}^{m} K_{f-iy}} \qquad e_{sy} = \frac{\sum_{i=1}^{n} K_{f-ix}\, y_i}{\sum_{1}^{n} K_{f-ix}} \tag{1}$$

$$\mathrm{K}_{\mathrm{f-i}} = \frac{24\mathrm{E}}{\mathrm{h}^2}\left[\frac{2}{\sum \mathrm{K_c}} + \frac{1}{\sum \mathrm{K_{ba}}} + \frac{1}{\sum \mathrm{K_{bb}}}\right]^{-1} \tag{2}$$

where: e_{sx}, e_{sy} are the x and y coordinates of the approximate center of rigidity CR, $K_{f\text{-}i}$ designates the approximate story stiffness of frame i, x and y the directions of the frame axis, m and n the number of frames along the y and x axes, respectively, E = modulus of elasticity, $K_c = EI_c / h$, $K_b = EI_b / \ell$, I_c, I_b = section moment of inertia of columns and beams, respectively, h = story height and ℓ = beam length. The indices a and b in the summations for the frame story stiffness designate the upper and lower beams of the frame in the considered story. Table 1 lists the total approximate eccentricities of each building (see Figure 1) corresponding to mass eccentricities of 0.10, 0.20 and 0.30 (ε_m=0.30 was not considered for the 5-story building).

Table 1: estimated physical eccentricities

ε_m	Floor 1		Floor 2		Floor 3		Floor 4		Floor 5	
	ε_x	ε_y	ε_x	ε_y	ε_x	ε_y	ε_x	ε_y	ε_x	ε_y
3-story building										
0.10	0.07	0.09	0.09	0.10	0.09	0.10				
0.20	0.13	0.20	0.17	0.20	0.16	0.20				
0.30	0.24	0.28	0.27	0.29	0.25	0.28				
5-story building										
0.10	0.09	0.09	0.09	0.09	0.10	0.10	0.10	0.10	0.10	0.10
0.20	0.15	0.16	0.16	0.18	0.18	0.18	0.19	0.18	0.19	0.18

The two sets of buildings, modeled as 3-D structures, were subjected to a group of semi-artificial, two-component, motions and were analyzed using a modified version of the ANSR program (Panagiotakos et al [28]). Beams and columns were idealized as nonlinear members by means of the well-known plastic hinge model, for which the modified moment – rotation relationship of Takeda (Otani [29]), with a strain hardening ratio of 0.05, was used. Bending-axial interaction in columns is treated approximately, through separate interaction diagrams in the two planes of bending. However, this approximation is not very important because the column axial loads are small fractions of the limit loads and also because most of the yielding takes place in the beams due to capacity design provisions. Two levels of member stiffness were considered: one corresponding to operational conditions as specified by the code for the design of the building and the second, a secant stiffness to the yield point based on anti-symmetric bending. The latter is computed as: $\mathrm{EI} = \mathrm{M_y}\ell / 6\theta_\mathrm{y}$, where M_y = yield moment, ℓ = member length and $è_y$ = yield rotation according to the semi-experimental equation by Park & Ang [30]. The secant stiffness at yield is quite lower than the code stiffness and leads to substantially softer models. This is seen from the three lower periods T_x, T_y, T_θ of the two buildings, listed in Table 2 for the various eccentricities.

Table 2: fundamental periods of buildings

ε_m	*Code Stiffness*			*Secant Stiffness to yield*		
	3–story buildings					
	T_y	T_x	T_θ	T_y	T_x	T_θ
0.00	0.69	0.61	0.61	1.10	1.01	0.98
0.10	0.70	0.62	0.53	1.05	1.01	0.88
0.20	0.67	0.58	0.45	0.94	1.00	0.78
0.30	0.70	0.54	0.36	0.90	1.00	0.65
	5–story buildings					
0.00	0.82	0.76	0.74	1.32	1.31	1.23
0.10	0.83	0.75	0.64	1.28	1.30	1.10
0.20	0.84	0.69	0.54	0.90	1.0	0.65

For this study, a group of five two-component, semi-artificial motions were used. These were generated from a group of five, two-component, real earthquake records, to match closely the EC8 design spectrum (spectrum with descending branch $\propto 1/T^{2/3}$). The method, based on trial and error and Fourier transform techniques (Karabalis et al [31]), gave excellent results as Figure 2 indicates. A good match is necessary so that the results are not masked by any effects of over or under-loading the buildings. Each motion pair 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.

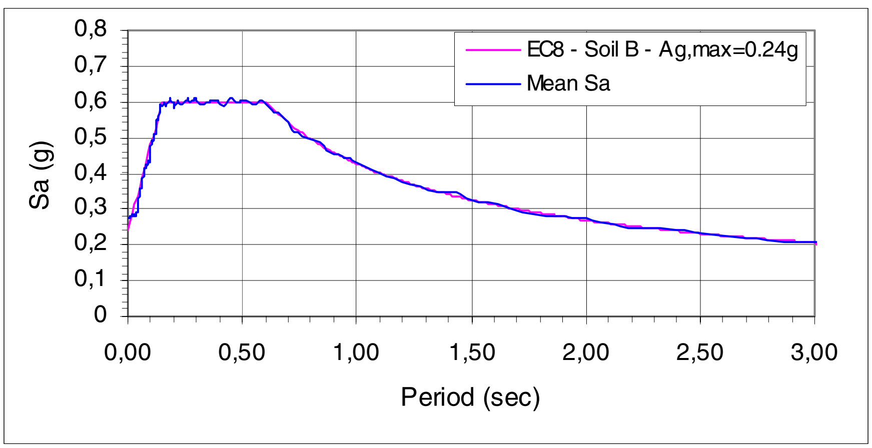


Figure 2: Design and mean spectrum of 10 semi-artificial motions

RESULTS

Four sets of results will be given: (a) Top story displacements and rotational ductility factors for the multistory buildings and (b) peak displacements and ductility factors of one-story simplified models that could be considered as approximations to the multistory models. The latter are from Stathopoulos [21] and are given here for comparison purposes.

Multistory, plastic hinge models

The results for the multistory buildings will be presented only for the secant to yield member stiffness, since the conclusions from the stiffer models (those with member stiffness according to the code), are for practical purposes almost the same. Listed displacements are mean values of the peak displacements from the 10 earthquake sets, while ductility factors are maximum values for all beams and columns (separately) in any story of a plane frame, selected from the set of values at both ends of all respective members in a story, where each value is the mean of the maxima for the 10 sets of earthquake motions.

The ductility factor used herein is the typical index, based on the maximum plastic hinge rotation at any of the two member ends. It is defined as:

$$\mu_{\vartheta} = 1 + \left(\frac{\theta_p}{\theta_y} \right) \tag{3}$$

where θ_p is the maximum plastic hinge rotation at the ends of a member (beam or column) and θ_y is a normalizing "yield" rotation, typically set equal to $\theta_y = M_y \ell / 6EI$.

Figure 3 compares the mean of the peak top story displacements of the two buildings, for accidental design eccentricities of 0.05L (EC8) and A.0.05L (UBC97), of their "stiff" and "flexible" edges, shown as functions of the natural eccentricity. The two upper graphs are for the 3-story buildings and the two lower graphs are for the 5-story buildings. Moreover, frames 1,3 are in the y direction and frames 4,6 are in the x direction (see Fig. 1). We see that the "stiff" sides in both directions are experiencing always less displacement than their "flexible" counterparts as a result of natural eccentricity, thus justifying the "stiff" vs. "flexible" terminology. However, the displacements of each frame do not exhibit a consistent, monotonic change, as functions of the natural eccentricity, although the same trend appears for corresponding frames of both buildings. We also see that the results for the two different accidental design eccentricities are almost identical.

Mean ductility demands for the 3 and 5-story buildings are shown in Figure 4 for the models with accidental eccentricity per EC8 (ε_{acc}=0.05L) and in Figure 5 for the models with accidental eccentricity per UBC97 (ε_{acc}= A.0.05L). The upper two sets of four graphs each, in both Figures, are for the 3-story frames and the lower two sets of 3 graphs each are for the 5-story frames, while the upper set for each building is for beams and the lower for columns. Each graph corresponds to a given physical eccentricity (0., 0.1, 0.2, 0.3 for the 3-story building and 0.0 , 0.1 , 0.2 for the 5-story building) and shows peak ductility demands of the stiff edge frame (frame-1, continuous line) and the flexible edge frame (frame 3, dashed line).

To comment briefly on the response of the symmetric buildings, we see that ductility demands in the beams are in the range of 2.0 to 4.0, while the columns remain essentially elastic, except for the top floors, where capacity design rules are not applied and also for the base, where yielding is typically expected. In the 5-story buildings, column yielding at the top and base is minimal.

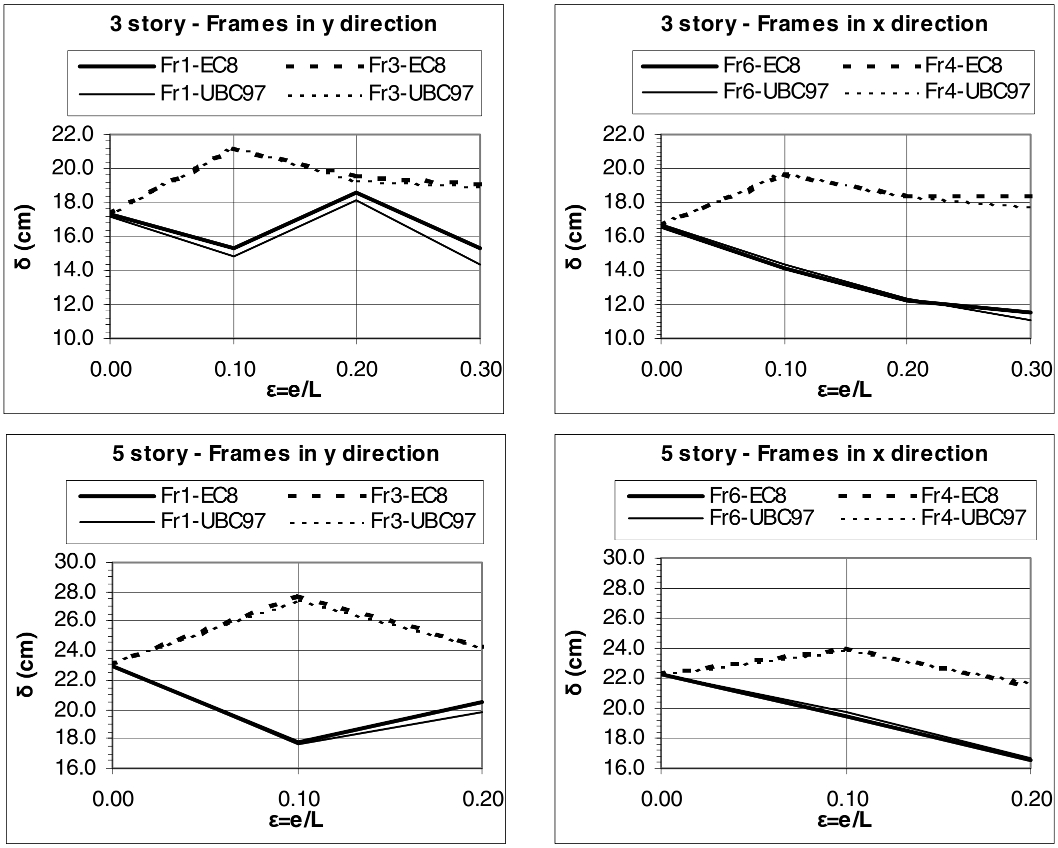


Figure 3: Displacements of the 3 and 5-story building designed according to EC8 and UBC97

The effect of eccentricity is seen by comparing beam and column ductility of the two end frames with corresponding values in the symmetric cases. As the eccentricity increases, we observe a substantial increase in beam ductility of the “flexible” side frames and a decrease, less pronounced, in the beam ductility of the “stiff” side frame. This is the case for both buildings and for both formulas of accidental design eccentricity, as well as the case with the frames in direction x (frames 4 and 6) not reported herein. At some levels the “flexible” side beams exhibit more than two times the ductility demands of the “stiff” side beams. Ductility factors of the beams in the middle frames have milder increases than the “flexible” side beams, compared to the symmetric case. The effects of eccentricity on column ductility are not as pronounced as on the beams, mainly because columns are designed to remain elastic, except for the top story and base. Nevertheless, we see that in the 3-story building, column ductilities of the “stiff” side have not been affected significantly by eccentricity: just a small reduction in the case of ε=0.30 and a small increase in the case of ε=0.20. In the “flexible” side, we see small yielding increases at the base while at the top the ductilities decrease so that columns turn out to remain elastic. In the x direction however (not shown here) column ductilities in the stiff side decrease and in the flexible side increase significantly at the top as the physical eccentricity increases. In the 5-story building, all columns at the “stiff” side have remained elastic, while in the “flexible” side there has been a substantial ductility increase at the top floor for ε=0.10.

Comparing results in Figures 4 and 5 we observe that they are very similar, almost identical, showing an unexpectedly small improvement when the UBC97 amplified accidental eccentricity is used, instead of the much simpler expression specified in EC8. These results suggest that the UBC97 requirement for amplifying the accidental eccentricity of 0.05L, may not be really necessary.

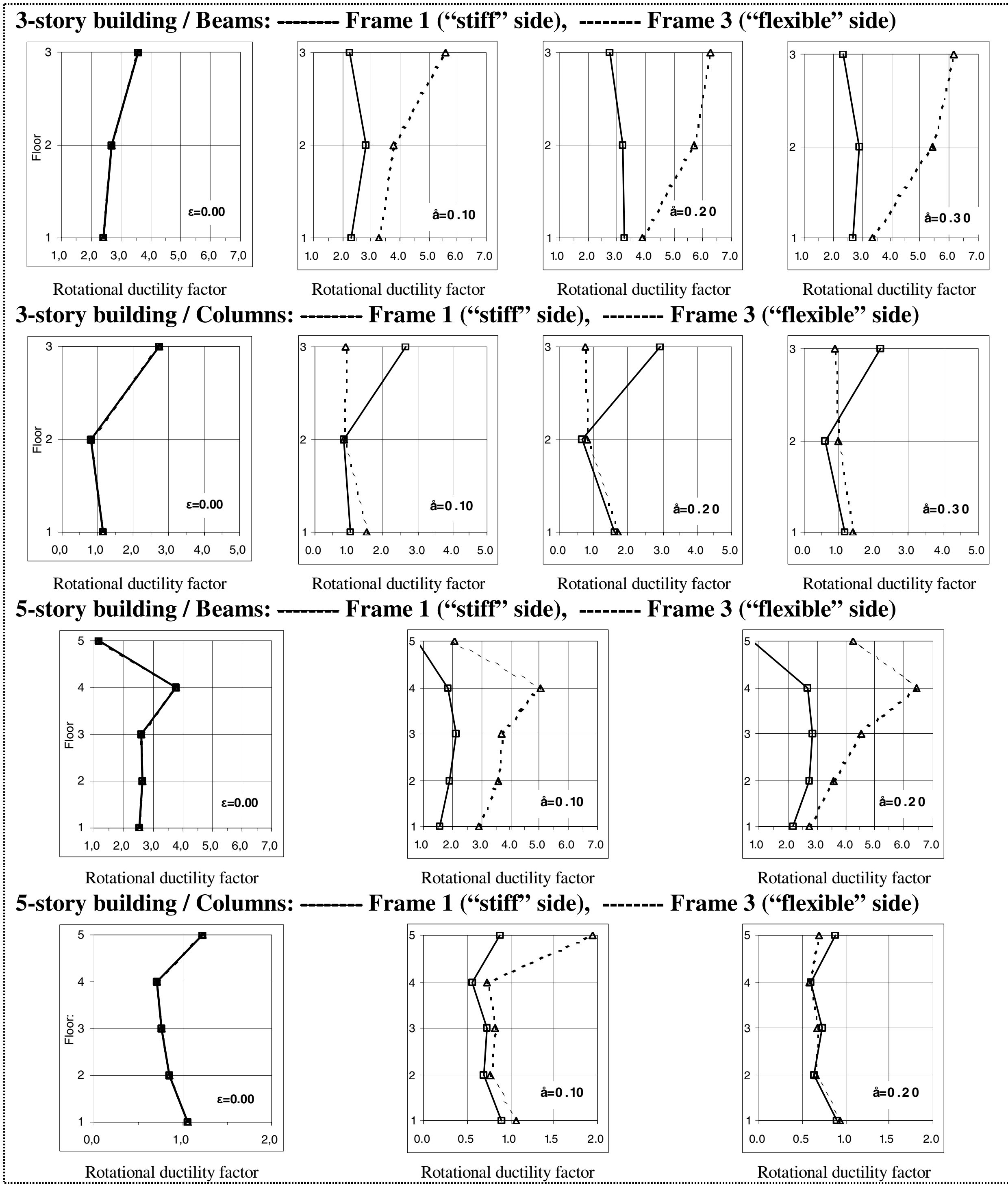


Figure 4: Ductility factors of the 3 and 5-story building designed according to EC8

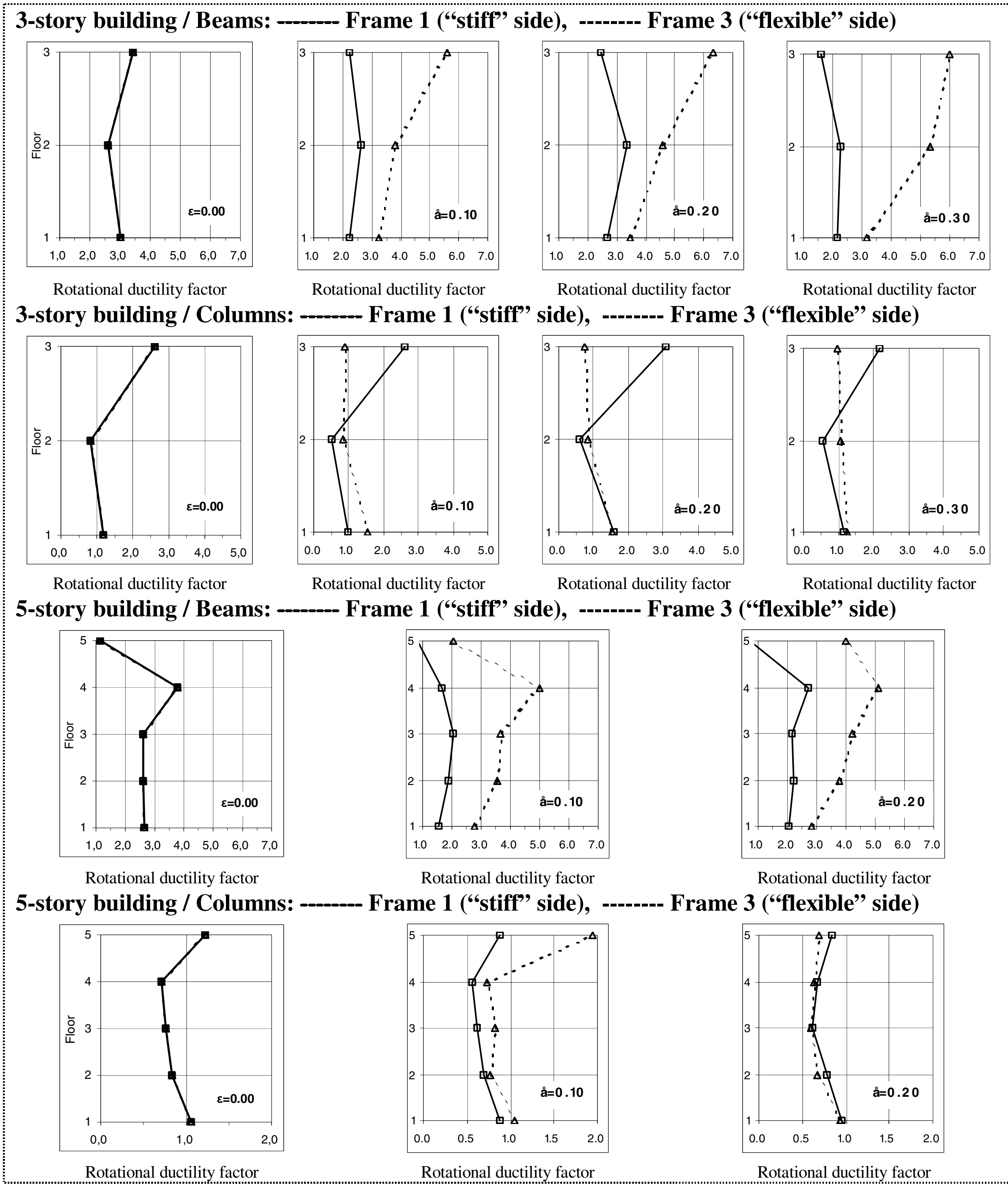


Figure 5: Ductility factors of the 3 and 5-story building designed according to UBC97

The above results, as well as others not reported herein (Stathopoulos et al [21,22]), indicate a real pattern of behavior that constitutes a code shortcoming, namely the uneven distribution of ductility demands in eccentric buildings between the “stiff” and the “flexible “ edges. The reason for that is that while the peak displacements of the “flexible” edge are, due to torsion, noticeably higher than the peak displacements of the “stiff “ side, as Figure 3 clearly indicates, the corresponding equivalent yield displacements that result from push over analyses are very close for both, stiff and flexible edge frames, and little affected by the physical eccentricities (Anagnostopoulos et al [23]). Higher global inelastic displacements imply even higher local inelastic effects, leading to the observed substantial differences in rotational ductility factors and damage indices at the two edges. So, if the objective for a well-balanced design would be to have similar level of inelastic action throughout the structure, thus avoiding pre-mature local failures, it seems that current design provisions in EC8 do not meet such an objective when applied to eccentric frame buildings. Similar results have been obtained and the same conclusions are reached with the stiffer models of the buildings based on the code specifies higher member stiffness for design (Stathopoulos [21]).

Simplified, one-story, shear beam type models

A basic purpose of the present study was to see if the trends in inelastic torsional response predicted by the simplified, one-story, shear beam model are correct, thus justifying its use for assessing code provisions for earthquake induced torsion. Therefore, a series of simplified models with biaxial mass eccentricities 0.00 (symmetric), 0.10, 0.20 and 0.30 were designed to approximate the corresponding 3 and 5-story buildings. These models have 3 elements in each direction to simulate the plane frames of the multistory buildings, as can be seen in the lower part of Figure 1. Only one middle element has been used to simulate the two middle frames in the y direction of the 3-story building and in both directions of the 5-story buildings. The three uncoupled periods of the symmetric model corresponding to the 3-story building are T_{υ}=1.2s, T_x=0.96s, T_{θ}=0.88s and those corresponding to the 5-story building are T_{υ}=1.6s, T_x=1.28s, T_{θ}=1.09s. According to usual practice for such models, their element strengths were computed only for earthquake loading, based on the design spectrum given earlier, and independent of their stiffness Additional details about these models may be found in Stathopoulos et al. [20]. We must note that the eight simplified systems were not designed to match specifically the corresponding multistory buildings but were selected from a parametric study in the thesis of the first author [21].

These models were subjected to the same group of earthquake motions and the results are summarized in Figures 6 and 7. Figure 6 gives peak mean displacements for the stiff and flexible edges of both buildings, both along the x and y directions. We see that the same general trend observed in Figure 3 for the multistory models is also observed here, namely the higher displacements of the “flexible” edge compared to the displacements of the “stiff” edge.

Figure 7 shows displacement ductility demands of the elements at the stiff and flexible sides for both buildings and in both directions. What we see here is that ductility demands at the “stiff” side are consistently higher than those at the “flexible” side. This is in agreement with what has been reported elsewhere in studies with the simplified model (Stathopoulos et al [20], Rutenberg et al [32], Chandler et al [33], Diaz- Molina [34]), but opposite to what the more detailed model predicts as can be seen in Figure 5. Such discrepancies must be attributed to the shortcomings of the simplified model stated in the beginning of this paper, especially when the extrapolation from a single story is made to multistory buildings. While one aspect of such shortcomings, namely the independence of stiffness and strength in the load bearing elements of the simplified, one-story models, has come into focus recently (Tso et al [24], [35]), others still remain, casting doubt on the conclusions from such models when extrapolated to frame type buildings.

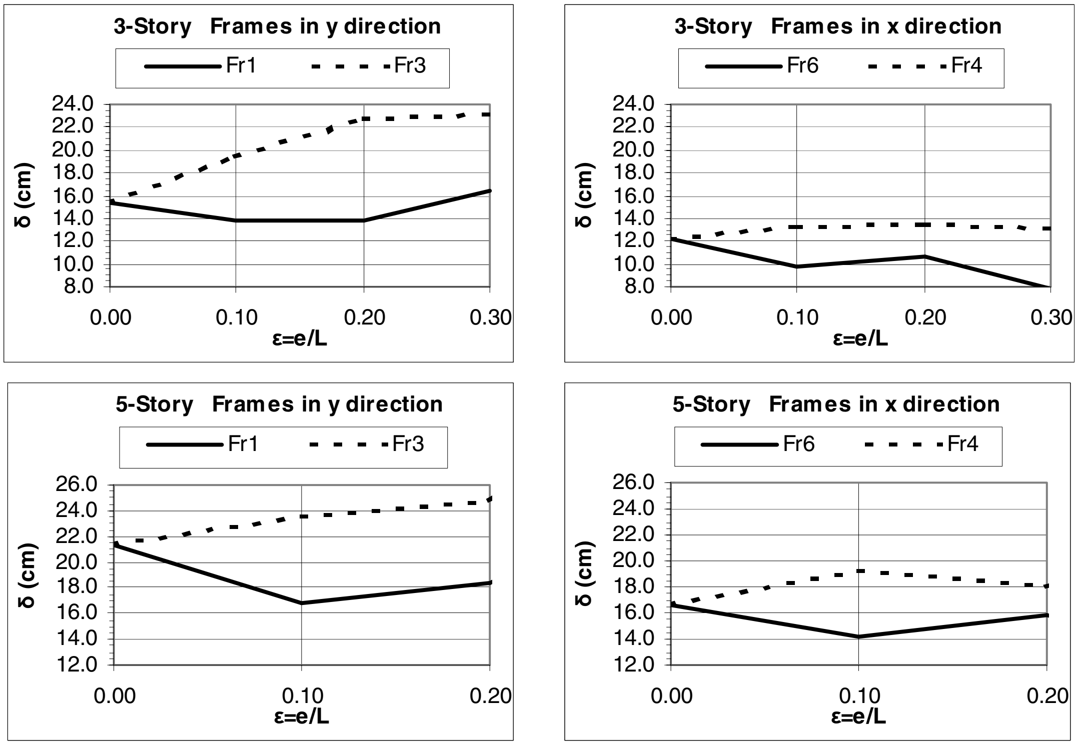


Figure 6: Displacements from the equivalent one-story shear-beam model

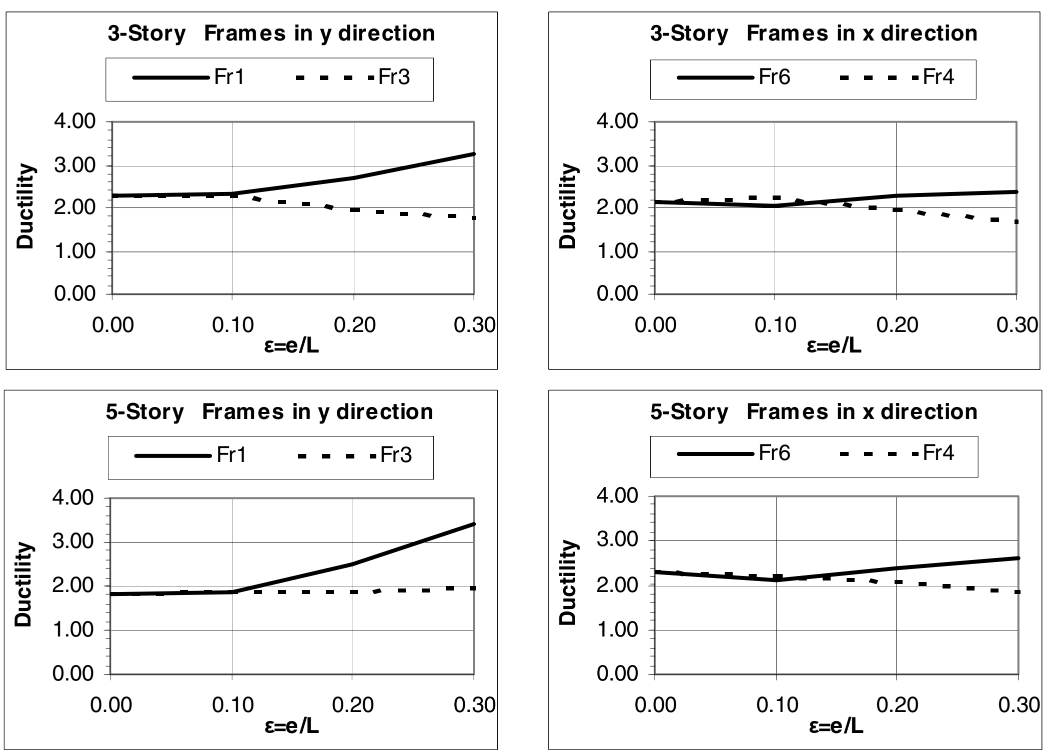


Figure 7: Ductility factors from the equivalent one-story shear-beam model

CONCLUSIONS

In the present paper, detailed structural models as well as simplified, one – story shear beam type systems are used to investigate earthquake induced inelastic torsion of multistory, eccentric, frame type buildings subjected to a group of ten earthquake motions. The results and comparisons reported herein provide a more reliable test of inelastic torsional response of multistory frame buildings, and consequently of the pertinent code provisions for torsion, indicating at the same time the inadequacy of the simplified one-story shear models when used for more complicated structural systems, such as multistory space frame buildings. Nevertheless, generalization should be made with care and any implications or recommendations about code provisions for torsion should be confirmed with further studies, including a wider sample of non-symmetric buildings. In summary then, the main conclusions are the following:

1. Eccentric frame-type buildings designed in accordance to EC8 and subjected to two-component earthquake excitations do not experience similar levels of inelastic deformation in all their members. Using the response of the associated symmetric building as the basis for comparison, it was found that frames at the "flexible" sides experience increased inelastic deformations and those at the "stiff" sides decreased deformations. As a result inelastic response measures, such as ductility factors at the "flexible" side have reached values more than twice those at the "stiff" side. Obviously, such uneven distributions are undesirable as they can lead to premature member failures. A desirable design for torsion ought to lead to members in which the design earthquake would generate ductility demands not significantly different than those in the corresponding symmetric building.

2. Results based on simplified, one-story, shear beam type systems show the opposite trend, i.e. indicate higher ductility demands in the "stiff" side of the building. For several important reasons listed in the paper, such systems are not adequate models of realistic multistory, frame type, buildings responding in the inelastic range.

3. It appears that the more complicated computations for the design accidental eccentricity required by the American UBC97 code, as compared to the simple expression of 0.05L specified by the European EC8 code, offers very little for improving the inelastic torsional response of multistory, space frame buildings. This suggests that a reexamination of the UBC97 provision for accidental eccentricities is warranted for buildings designed for high ductility (large q factors), in view of the additional design work it generates.

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