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INELASTIC EARTHQUAKE RESPONSE OF BUILDINGS SUBJECTED TO TORSION

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SUMMARY

The inelastic seismic torsional response of simple structures is examined by means of shear-beam type models as well as with plastic hinge idealization of a one-story building. Using mean values of peak ductility factors, obtained for groups of ten earthquake motions, as the basic index of post-elastic response, the following topics are examined with the shear-beam type model: mass eccentric versus stiffness eccentric structures, effects of different types of motions and effects of double eccentricities. In addition, results are presented for single story reinforced concrete frame buildings designed according to a modern code and modeled using the plastic hinge idealization for its members. The consequences of designing for different levels of accidental eccentricity is examined for the aforementioned frames in the case of full symmetry (ε=0) and in the case with a double eccentricity ε=0.20.

INTRODUCTION

The inelastic torsional response of buildings subjected to earthquake ground motions is still a subject attracting the interest of many researchers worldwide. Most of the work so far has been based on very simplified models of single story, shear-beam type systems with one axis of symmetry, subjected to a single component motion perpendicular to that axis [e.g. Bozorgnia & Tso, 1986, Goel & Chopra, 1991, Chandler & Duan, 1997]. In few papers, two component motions have been used, still with the simple 3-DOF mono-symmetric model [e.g. Correnza et al, 1994, Tso & Wong, 1995, Stathopoulos & Anagnostopoulos, 1998, Riddell & Santa-Maria, 1999], while systems with double eccentricities have rarely been considered [Goel, 1997]. Very little, if anything, has been done using more realistic models of inelastic structural behavior, such as the plastic hinge idealization. As far as the inelastic torsional response of multistory buildings is concerned, this has been restricted to either highly idealized systems [e.g. Anagnostopoulos et al, 1973, Duan & Chandler, 1993] or, more recently, to a class of wall-type systems, drawing from capacity design concepts [e.g. Paulay, 1997, 1998]. A key question that many published papers address, is whether the torsional provisions of various codes are adequate. The answers to this question vary considerably, depending upon the type of assumptions used by the authors in their respective studies. Thus, controversial conclusions have often been obtained and identified [e.g. Chandler et al, 1996]. The present paper presents some new results on this problem based on simplified systems, both of the shear beam and the plastic hinge type, with simple as well as double eccentricities.

SYSTEMS AND MOTIONS USED

In this work two types of systems have been used: (a) Shear beam systems with single as well as double eccentricity of the mass and of the stiffness type. (b) Mass eccentric systems with frames as resisting elements, whose inelastic behavior is modeled by means of plastic hinges forming at the ends of their columns and beams. This model provides a better approximation to the post-yield behavior of real buildings.

Two types of simple shear-beam models have been used, both derived from the system shown in Fig 1a: a mass eccentric model, in which the stiffness center CR coincides with the geometric center GC and a stiffness eccentric model, in which the center of mass CM coincides with CG. Moreover, for systems with single eccentricity the axis of symmetry is the x axis (i.e. $e_y=0$), while for systems with double eccentricity, their

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properties were determined so that $\varepsilon_x = e_x / L_x = \varepsilon_y = e_y / L_y = \varepsilon$, which implies that the center of mass CM and of stiffness CR lie on the diagonal. The mass of all these systems was assumed equal to 300t and the total stiffnesses K_x and K_y were determined so as to give the desired uncoupled periods T_x, T_y. For the mass eccentric systems, the following stiffness distribution among the six elements was assumed: $K_1=K_3=0.30K_y$, $K_2=0.40K_y$, $K_4=K_6=0.30K_x$ and $K_5=0.40K_x$. For the stiffness eccentric systems, the stiffness of the individual elements for the three eccentricities are as follows: **e=0.10:** $K_1/K_y=K_2/K_y=K_5/K_x=K_6/K_x=0.40$ and $K_3/K_y=K_4/K_x=0.20$, **e=0.20:** $K_1/K_y=K_6/K_x=0.55$, $K_2/K_y=K_5/K_x=0.30$ and $K_3/K_y=K_4/K_x=0.15$, **e=0.30:** $K_1/K_y=K_6/K_x=0.70$, $K_2/K_y=K_5/K_x= 0.20$ and $K_3/K_y=K_4/K_x=0.10$. In all cases the strength of each element was set equal to the design seismic shear, as determined from the design spectrum applicable to soil class B of Eurocode 8 [EC8, 1994], for ground acceleration PGA=0.30g and behavior factor q=2.5. In addition to the natural eccentricity e, the code specified accidental eccentricities $0.05L_x$ and $0.05L_y$ were also applied to each system, including the symmetric one (ε=0) used for reference and comparison. The effectiveness of this accidental eccentricity was investigated by also examining designs with $e_{acc}= 0.0$ and $e_{acc}= A_x\,(0.05L)$, where A_x in an amplification factor as required by the American and Greek codes [UBC, 1997, NEAK, 1995].

When actual buildings are designed, strength is not independent of stiffness, although for concrete structures their stiffness depends essentially on section dimensions and their strength on the reinforcement. Moreover, design for gravity loads plays an important role for the stiffness and strength of the resisting elements, but is typically ignored in shear beam type models. For these and other reasons, the more realistic model, shown in Fig. 1b was also used in our investigation. It is a mass-eccentric, one-story reinforced concrete building, fully designed according to NEAK and to EC8 as a space frame. Three plane frames can be distinguished therein parallel to each principal direction: FR1(C1,C4,C7), FR2(C2,C5,C8), FR3(C3,C6,C9) parallel to the y axis and FR4(C1,C2,C3), FR5(C4,C5,C6), FR6(C7,C8,C9) parallel to the x axis, in a complete analogy to elements EL1,EL2,EL3 and EL4,EL5,EL6 of the shear beam model. The uncoupled periods of the basic symmetric system used (based on the code specified section properties) are T_y=0.40s, T_x=0.32s, T_θ=0.36s. In addition to this, two other systems were examined with periods T_y=0.50s, T_x=0.40s, T_θ=0.46s and T_y=0.31s, T_x=0.25s, T_θ=0.28s. For most analyses, the ratio T_x / T_y was kept equal to 0.8.

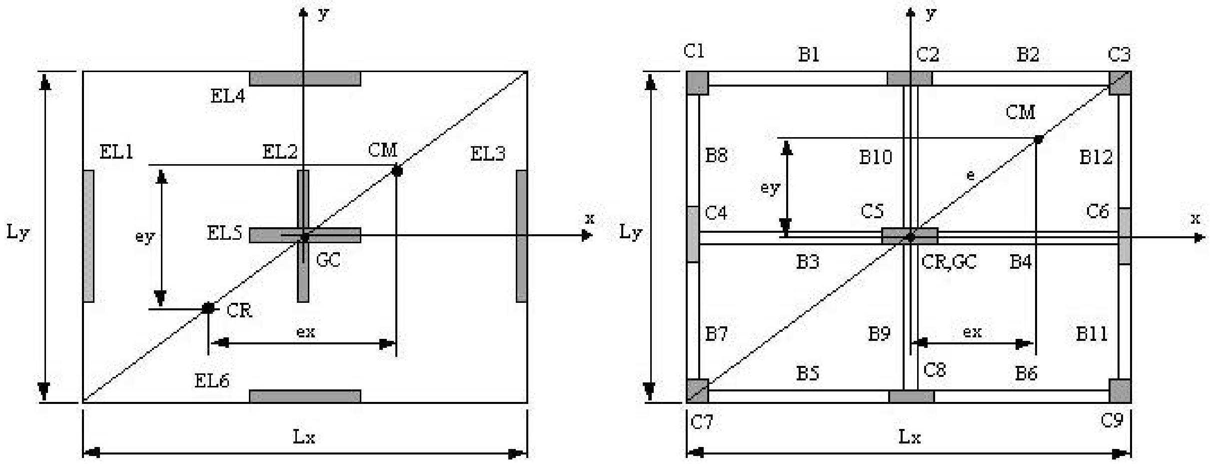

Figure 1. Geometry of systems analyzed: (a) Shear-beam model (b) Reinforced concrete building

The various systems were subjected to three groups of earthquake motions, selected so as to be compatible with the design spectrum. Each group includes five, two-component, motions, both applied simultaneously to the structure. Each motion pair was applied twice by mutually changing the components along the x and y system axes. Thus, for each group of motions, ten sets of analyses were carried out and the mean as well as the mean plus one standard deviation of the results were used to assess the effects of torsion.

The first two groups of motions, listed in Table 1, consist of real records from historical earthquakes, the first belonging to the wide-band type (group A) and the second to the pulse type (group B). Using as criterion a good match of the EC8 design spectrum by the mean response spectrum of the 10 components in each group, the wide band records were scaled to a PGA of 0.38g and the pulse-type records to a PGA of 0.43g. The scaling factor for each component pair was determined from the component with the higher PGA.

Table 1. Two groups of historical records used for the analyses

Wide-Band type Records (group A)					Pulse type Records (group B)				
Record		PGA(g) L	PGA(g) T	Scale	Record		PGA(g) L	PGA(g) T	Scale
El Centro	1940	0.35	0.21	1.092	Imperial Valley (#7)	1979	0.33	0.46	0.931
Loma Prieta (Hollister)	1989	0.29	0.28	1.324	Aigio	1995	0.50	0.54	0.792
Olympia (Wash. Hwy)	1949	0.17	0.28	1.357	Lefkada	1973	0.53	0.26	0.819
Corinthos	1981	0.24	0.30	1.284	Erzincan	1992	0.50	0.40	0.858
Thessaloniki	1978	0.14	0.14	2.639	Northridge (Sylmar)	1994	0.59	0.88	0.489

Figure 2 shows the elastic EC8 ground motion and design acceleration spectra for soil B at PGA=0.30g, as well as the mean spectra of the two groups of motion. We see that the pulse-type records match the EC8 ground motion spectrum very well in the long periods (sloping branch $\propto 1/T$), while the wide-band type of motions have a somewhat lower descending branch.

For a better match of the EC8 elastic design spectrum (spectrum with descending branch $\propto 1/T^{0.67}$) 10 semi-artificial records (group C) were also generated by modifying the 10 wide-band real motion components. The method, based on trial and error and Fourier transform techniques [Karabalis et al, 1994] gave excellent results as Fig. 2 indicates. In this case the maximum difference in spectral ordinates between the target design spectrum and any member in the group was below 15%. By matching closely the mean response spectrum to the target spectrum, any effects from over or under-designing the structure are removed.

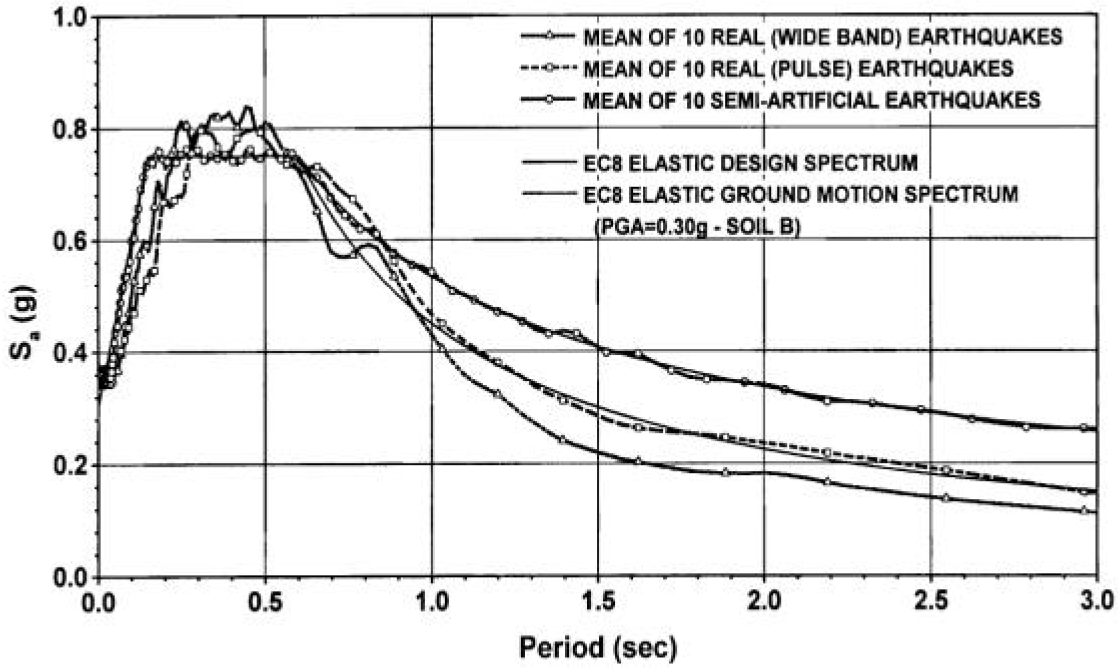

Figure 2. Elastic design spectra and mean spectra of motions used

SELECTED RESULTS

Due to paper length limitations only a few selected results are presented herein. These pertain to the mass versus stiffness eccentric models, the effects of earthquake motions with different characteristics, single versus double eccentricities and to the more sophisticated plastic hinge models.

Mass versus stiffness eccentric models

An ambiguity in the results on torsion by different researchers stems from the use of two different types of simplified shear beam models. In the so called mass eccentric model, the resisting element stiffnesses are symmetric with respect to the geometric center and the masses non symmetric. The opposite happens in the so called stiffness eccentric models. To see whether different conclusions are obtained from the two models, simply eccentric systems of both types with eccentricities ε_x = 0.0, 0.10, 0.20 and 0.30, were subjected to the C group of motions. Mean ductility demands for the end elements 1 and 3 (Fig. 1a) of the two types of models are shown in Fig. 3, as functions of the uncoupled system period T_y. The graphs on the left are for the mass eccentric systems (MES) and those at right for the stiffness eccentric (SES). In all cases, the uncoupled translational period T_x was set equal to $0.8T_y$. In the stiffness eccentric model, element 1 represents the so called flexible side and element 3 the stiff side. Comparisons of the corresponding graphs shows that: (a) all systems exhibit comparable ductility demands, independent of their natural eccentricity e, (b) both sides of the structure are equally penalized by the quake, (c) both mass eccentric and stiffness eccentric models exhibit very similar overall behavior.

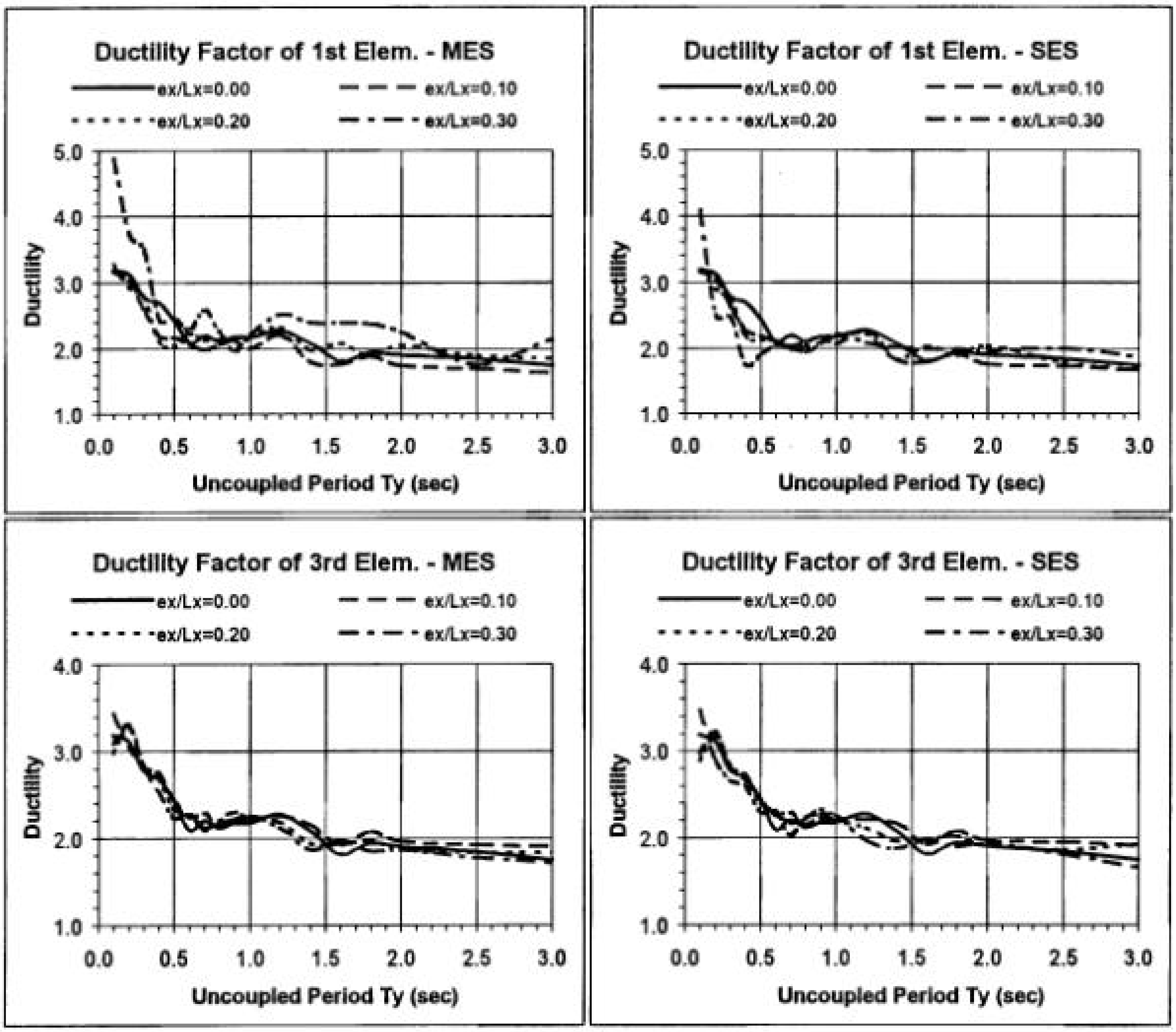

Figure 3. Comparisons of ductility demands between mass and stiffness eccentric systems (single eccentricity, motion group C)

Effects of motions with different characteristics

To see if there are qualitative differences in the results when motions with different characteristics are used, the various systems were analyzed also for motion groups A (wide-band type records) and B (pulse type records). Figure 4 shows mean ductility demands for element 1 of the mass eccentric systems subjected to motion groups A and B. The corresponding graph for group C is given in Fig. 3 (upper left). We observe that the behavior of the systems appears to be very similar for the three groups and, further, that the differences in ductility demands imposed by each group in the long period range, reflect the corresponding differences in the mean spectra of each group (see Fig. 2). It therefore suffices to work with one set of motions and as such, the group of semi-artificial motions has been selected. We must point out, however, that results from individual motions can exhibit substantially greater differences.

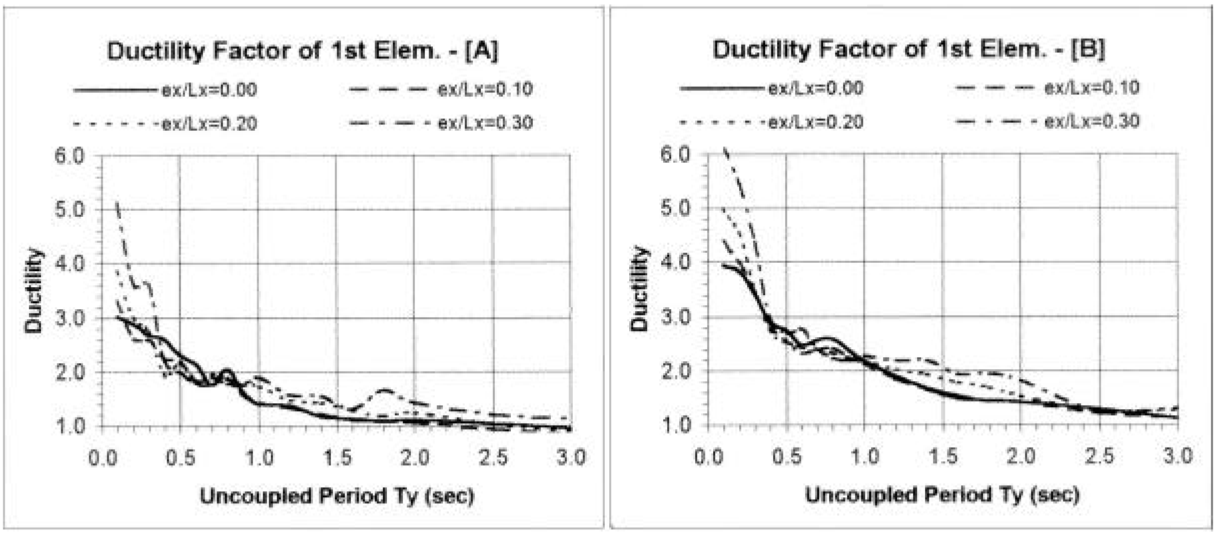

Figure 4. Results from motion groups A and B

Single vs. double eccentricities

Systems with double eccentricities are in principle no different than simply eccentric systems, except for the coupling of both translational modes with the torsional mode. A comparison between the two types of eccentric systems is given in Fig. 5, showing mean ductility demands in the end elements (1,2,4 and 6) of mass eccentric systems with single (left graphs) and double eccentricity (right graphs). The single eccentricity is along the x axis and the double eccentricity along the diagonal (see Fig. 1). We can see the overall similar behavior of all elements, except element 1, which appears to be penalized by large biaxial eccentricities ($\varepsilon > 0.20$) for periods greater than about 0.8s. We will also note that stiffness eccentric systems (not presented here) exhibit similar trends, except that we observe greater differences therein between curves corresponding to different values of ε, i.e. the various curves for the different elements are not as close as in the case of single eccentricities.

Results from plastic hinge models

The plastic hinge model of prismatic structural members (beams and columns) is the most widely used idealization for reliable predictions of the inelastic earthquake response of framed structures. Compared to the simplified shear beam models, typically used for investigating the effects of torsion, this model has the following advantages: (a) It provides a far better approximation of real structures. (b) Its member strengths, crucial for the inelastic response, are those of the actual structure as the code design requires, i.e. based also on gravity loads and on other code considerations. (c) Its post-yield stiffness and its time variation is far more realistic, because yielding of an end element in the shear beam model reduces the stiffness of that side to practically zero. This would be equivalent of mechanism formation, a condition that is highly unlikely in a well designed framed structure, where even if many plastic hinges form in a given frame, its stiffness is still substantial as it is controlled by its elastic members. For these reasons our investigation has been extended by using the structural model shown in Fig. 1b. This represents a single story reinforced concrete space frame, designed according to NEAK and to EC8, both as a symmetric structure with $\varepsilon_x = \varepsilon_y = 0.0$ and as a mass eccentric structure with double eccentricity $\varepsilon_x = \varepsilon_y = \varepsilon = 0.20$. The uncoupled periods in both cases are T_y=0.40s, T_x=0.32s, T_θ=0.36s, while the coupled periods of the eccentric structure are T_y=0.48s, T_x=0.35s, T_θ=0.27s.

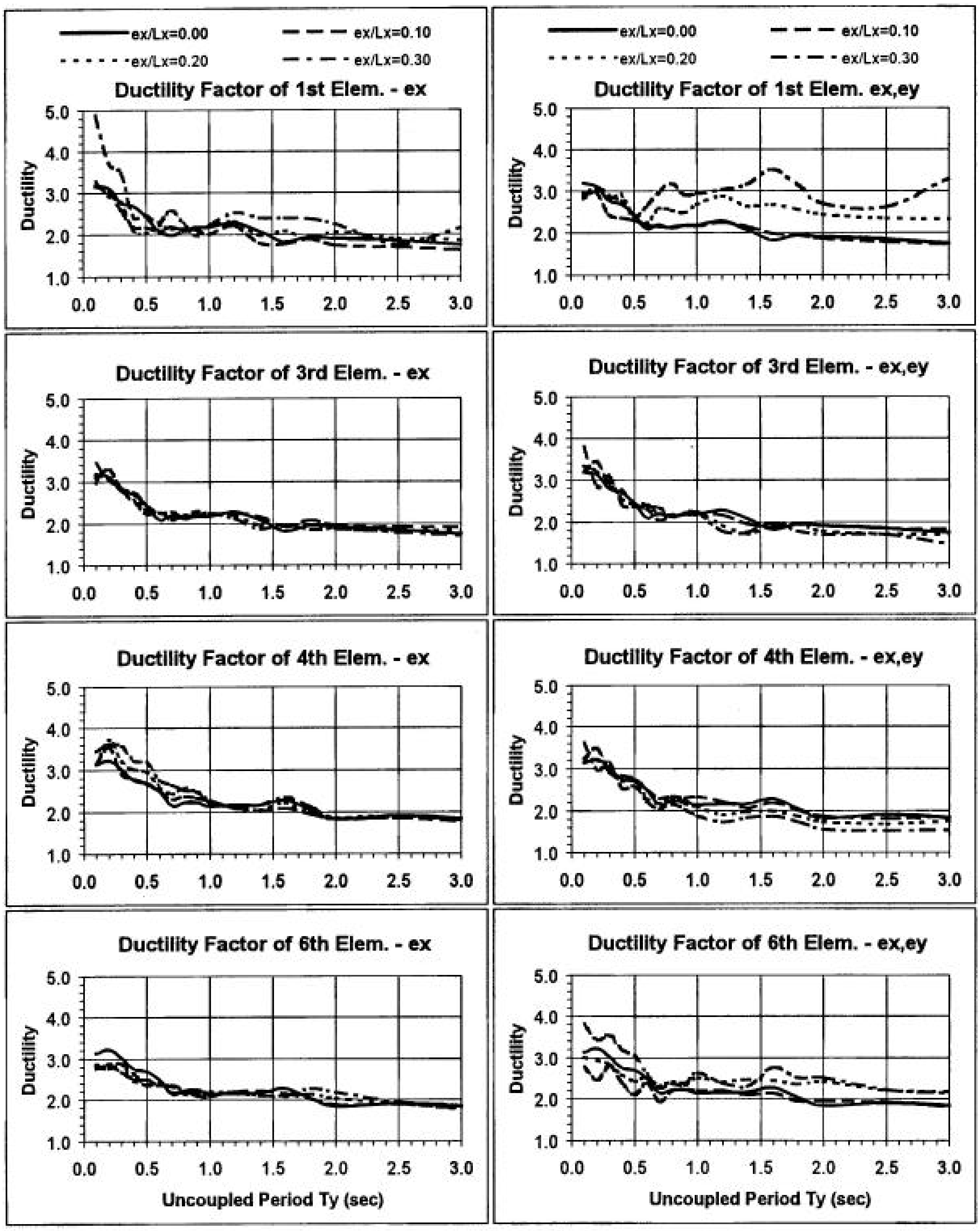

Figure 5. Results for mass eccentric systems with single and double eccentricity (motion group C)

Results in the form of mean ductility demands for the two systems are shown in Fig. 6 for the beams and columns (left and right graphs respectively), of each of the six plane frames that correspond to elements 1-6 of the shear beam model (see Fig. 1a,1b). The three values shown by bars for each frame correspond to three different designs: one (first bar, I) where the accidental design eccentricity in each of the two directions is e_{acc} = A(0.05L), where A is an amplification factor as specified by the Greek and UBC codes, the second (second bar, II) where e_{acc} = 0.05L as specified by EC8 and the third (third bar, III) where e_{acc} = 0.0. We see that the beams of all three designs show very large ductility demands that are somewhat reduced for the eccentric structure, as compared to the symmetric one (e=0) and that in both cases such demands are practically the same for all three designs. Columns also yield in all six elementary frames, but with much smaller ductility demands. Moreover, the amount of accidental design eccentricity is more significant for columns as it affects their ductility demands noticeably.

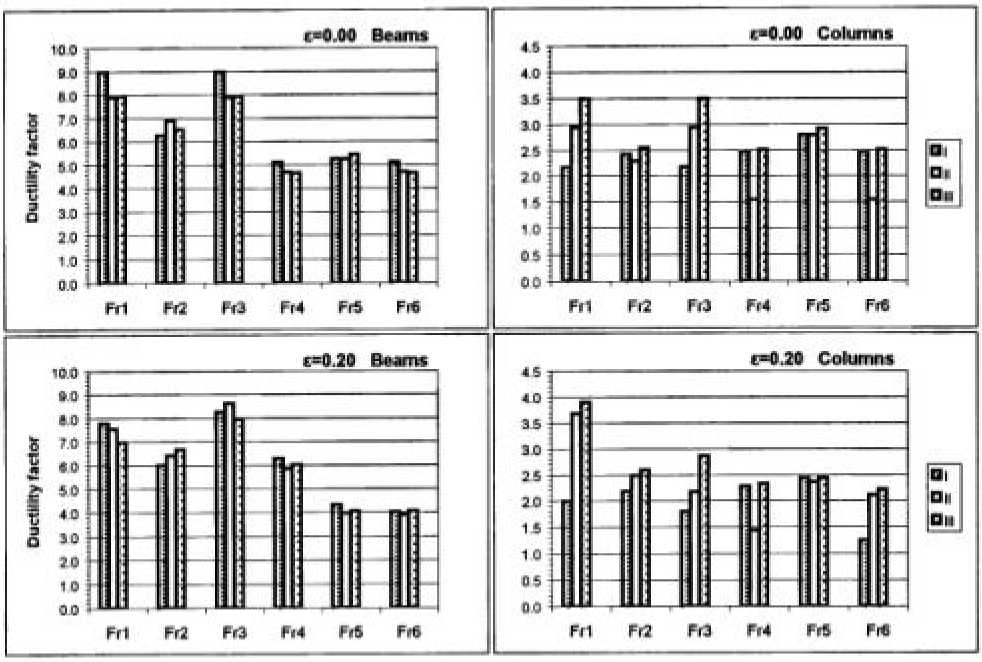

Figure 6. Mean ductility demands of symmetric and mass eccentric (e=0.20), one story, reinforced concrete buildings idealized with the plastic hinge model (design for three accidental eccentricities)

While ductility factors have been used routinely as the simplest measures of inelastic response for systems and members with simplified post-elastic behavior, for reinforced concrete structures the so-called damage indices have often been applied as more reliable indicators of the severity of inelastic action. One such index, suggested by Fardis (1995), was also used here and its values are shown in Fig. 7, for exactly the same cases of and in full correspondence to Fig. 6.

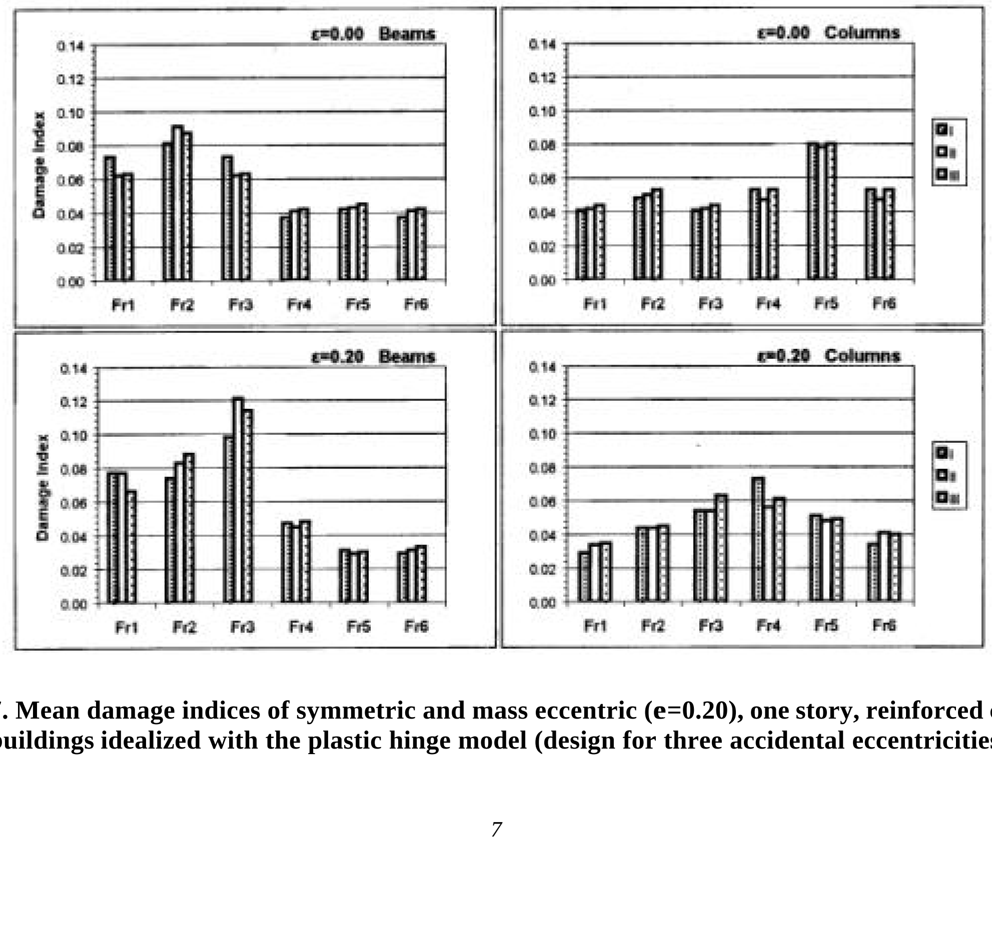

Figure 7. Mean damage indices of symmetric and mass eccentric (e=0.20), one story, reinforced concrete buildings idealized with the plastic hinge model (design for three accidental eccentricities)

We see here that the eccentric structure (ε=0.20) has very comparable damage indices with the symmetric structure, all below a value of 0.10, except for the beams of frame 3 designed with e_{acc} =0.0 and e_{acc} =0.05L that reach a damage index of about 0.12. On the basis of these graphs, one may conclude that at least for these one story concrete structures, the introduction of an accidental eccentricity in their design has no noticeable beneficial effect.

CONCLUSIONS

The results presented herein lead to the following tentative conclusions: (1) Code designed, simple shear-beam type, one-story systems, either mass or stiffness eccentric, exhibit similar overall ductility demands for natural eccentricities in the range of 0.0 to 0.30. Moreover, no noticeable difference in such demands is observed between the so-called flexible and stiff sides. (2) Motions with different characteristics cause similar post-elastic behavior of the systems examined. (3) With minor exceptions, occuring at large ε values, biaxial eccentricities do not appear to lead to substantially different results from those due to single eccentricities. The differences are essentially quantitative. (4) Use of realistic structural models of the plastic hinge type leads to results that would be directly applicable to real buildings. Ductility demands of systems with biaxial eccentricities up to e=0.30D, are very similar to those of the symmetric structure. This suggests that the code provisions for torsion are adequate. Moreover, if damage indices instead of ductility factors are used to assess the inelastic response of symmetric as well as eccentric concrete frames, it would appear that the design provision of the code for accidental eccentricities has little beneficial effect i.e. does not lead to lower damage indices compared to those for designs with zero accidental eccentricities.

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