Inelastic seismic response of an asymmetric multi-story R/C frame building

KILAR Vojko[1], FAJFAR Peter[2]

ABSTRACT

In the paper the inelastic torsional behaviour of a four-story asymmetric RC structure, representative for a typical frame-type building, is discussed. The symmetric variant of the building was designed according to Eurocode 8. Asymmetry was introduced by shifting the mass center in one direction. The influence of the magnitude of eccentricity and of the intensity of ground motion was studied. In addition, a torsionally flexible variant of the building was investigated. Each variant of the structure was subjected to a set of five ground motions. The nonlinear dynamic response under uni- and bi-directional input is compared in terms of mean values of envelopes of displacements and torsional rotations. The results demonstrate that, for the investigated building, uni-directional input yields results which are close to the results obtained for bi-directional loading.

INTRODUCTION

Inelastic seismic response of asymmetric structures is an important and popular research topic (see, for example, list of references in Rutenberg and De Stefano, 1997, and in Moghadam, 1998). Nevertheless, due to a very large number of parameters, which influence the response, the progress has been rather slow. The main objective of the research at the University of Ljubljana is development of a simplified nonlinear procedure for seismic analysis which is based on push-over analysis. In the case of an asymmetric structure, the problem arises how to combine the influence of the horizontal excitations in two directions. In order to better understand this problem, the response of structures subjected to uni- and bi-directional input has been studied. A parametric study has been made on simple single-story structures (Fajfar and Peruš, 1999). In this paper, the results of a study performed on a realistic building are presented.

DESCRIPTION OF THE BUILDING AND SEISMIC INPUT

Figure 1 presents the floor plan and the elevation of the analyzed building. The cross sections of the structural members are equal in all frames and in all stories. All columns have 0.3 x 0.6 m rectangular cross sections and are differently oriented in plan; all beams have height of 0.6 m and width of 0.3 m. The symmetric building variant was designed using the Eurocodes 2 and 8, considering an accidental eccentricity equal to 5% of the relevant plan dimension of the building. The design spectrum for soil class B scaled to the peak ground acceleration 0.35 g was used. The behaviour (reduction) factor q was equal to 3.75 (medium ductility class). The effects of the two horizontal components of the seismic action were considered by combining 100% of the effect in one direction with 30% of the effect due to the orthogonal component. Story masses amounted to 295 and 237 tons in bottom stories and at the roof, respectively. The design base shear was equal to 23% of the total weight.

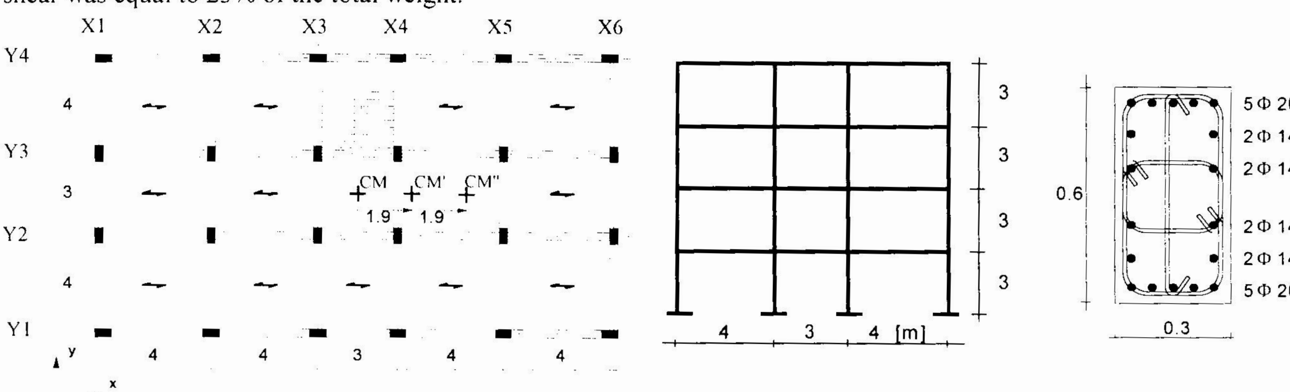


Figure 1. Analyzed building: typical floor plan, elevation and reinforcement of columns in the first and second story.

[1] Asst. Professor, University of Ljubljana, Faculty of Architecture, Zoisova 12, 1000 Ljubljana, Slovenia
[2] Professor, University of Ljubljana, Faculty of Civil and Geodetic Engineering, Jamova 2, 1000 Ljubljana, Slovenia

The required different strength levels were obtained by varying the amount of reinforcement. Reinforcement in the bottom two stories was different from that in the upper two stories. In order to achieve uniformity of structural elements, all columns in a story have equal reinforcement. All frames in Y direction (i.e. frames X1 to X6) are identical. Identical are also frames Y1 and Y4, as well as frames Y2 and Y3.

Nonlinear dynamic analyses of the 3-D mathematical model were performed by using the CANNY computer program (Li, 1996). The floor diaphragms of the structure were assumed to be rigid in their own planes and to have no out-of-plane stiffness. The columns were modeled as combined elements that consist of an elastic line element and two multi-spring elements (16 concrete and 18 steel springs) at the top and bottom of each column. The beams were modeled with usual elements composed of an elastic beam and two inelastic hinges, considering uniaxial bending and shear deformations. The inelastic flexural deformations, that were assumed to be concentrated at the beam ends, were modeled with the Takeda hysteretic model with trilinear envelope. A more detailed description of the design parameters and mathematical modeling of the structure can be find in (Faella and Kilar, 1998).

To examine the influence of mass eccentricity, the symmetric building was changed to an asymmetric building by shifting the center of masses CM in the +X direction for e_m=0.1·L (building S10) or e_m=0.2·L (building S20), L being the larger dimension of the floor plan. All other quantities remained the same as for the symmetric building. The influence of the intensity of the earthquake ground motion was studied by subjecting the building S10 to ground motions with lower intensity (accelerograms normalized to 0.35 g compared to 0.7 g used in other cases). This case is denoted as S10Low. The symmetric building as well as buildings S10 and S20 can be classified as torsionally stiff buildings (the first period of vibration is predominantly translational).

Torsionally flexible variant of the building S10 (denoted as building F10) was produced artificially by increasing the mass moment of inertia by a factor of 2. This case approximately corresponds to a theoretical situation when the whole mass is concentrated along the perimeter of the building. In this case the period of the predominately torsional vibration mode is larger than the periods of the predominantly translational vibration modes (see Table 1). All parameters, (except mass moment of inertia), remained the same as for the building S10.

Two horizontal components of five records were used to investigate the effects of the ground motion variation: Petrovac (Montenegro 1979), El Centro (1940), Kobe JMA (1995), and two records from the 1994 Northridge earthquake (Sylmar and Newhall). Normalized elastic acceleration spectra for all records are shown in Figure 2. The stronger components (i.e. components with larger peak ground acceleration) of all records were scaled to the peak ground acceleration 0.7 g, which equals to twice the design ground acceleration (the ratio between X and Y components of the accelerograms remained unchanged). The stronger components were applied in the building Y-direction. In the case S10Low, the records were scaled to 0.35 g.

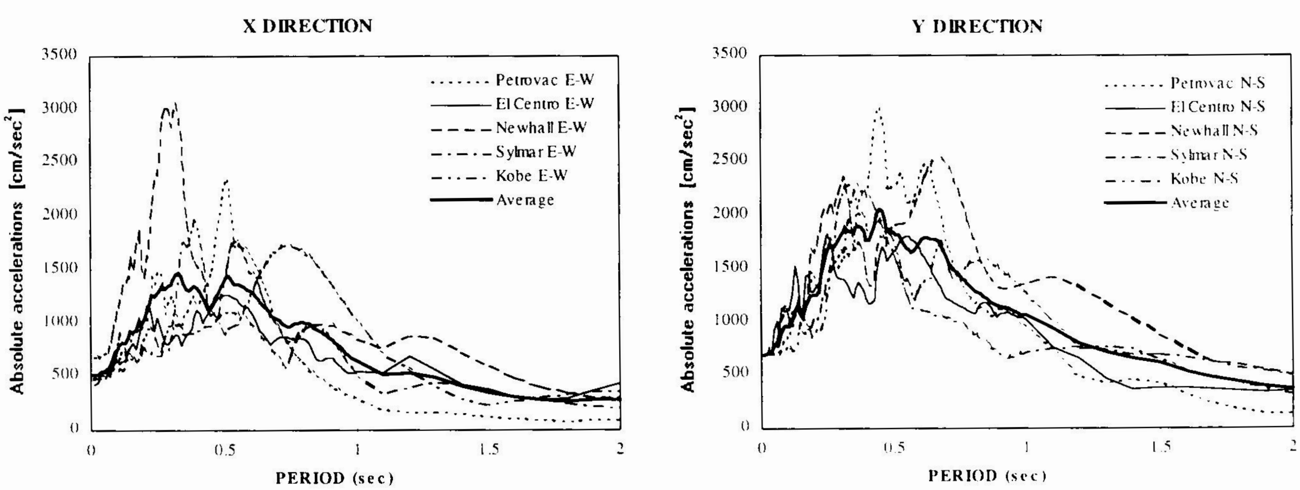


Figure 2. Elastic acceleration response spectra for components in X and Y direction for 5% damping.

Table 1. Periods and average values of maximum top displacements for loading in X and Y direction (values for loading in one direction only are in square brackets: [(loading in Y direction),(loading in X direction), ($\sqrt{Y^2+X^2}$)].

	Periods for the first 3 modes (s)			Average values of maximum top displacements (cm)			
Building	1	2	3	frame X1	CM	frame X6	max. frame (Y1,Y4)
Symmetric[1]	0.50 (Y)	0.50 (X)	0.40[2] (T)	19.4[20.5]	19.4[20.5]	19.4[20.5]	14.6[0.0, 16.8, 16.8]
S10	0.53 (Y)	0.50 (X)	0.36 (T)	14.6[12.5]	17.5[18.5]	24.1[26.3]	17.8[7.4, 16.8, 18.4]
S10Low	0.53 (Y)	0.50 (X)	0.36 (T)	5.6[5.6]	9.7[9.8]	15.2[14.4]	8.8[4.1, 7.4, 8.5]
S20	0.59 (Y)	0.50 (X)	0.33 (T)	13.1[11.1]	18.8[18.7]	27.1[26.7]	18.9[9.6, 16.8, 19.3]
F10	0.60 (T)	0.50 (X)	0.46 (Y)	20.7[20.0]	19.8[20.2]	20.9[22.6]	14.8[4.6, 16.8, 17.4]

[1] Ground motions scaled to 0.7 g [2] For torsionally stiff building. For torsionally flexible building the period of the torsional mode amounts to 0.57 s.

PUSHOVER ANALYSIS

This section describes some results obtained by pushover analysis. The load pattern was an inverted triangle. Figure 3 presents base shear - top displacement relationship for the symmetric and for two asymmetric building variants (S10 and S20). The loads were applied in the relevant center of masses CM, independently in X and Y direction (for symmetric building) and in Y direction (for asymmetric building). The curves are plotted up to a top displacement equal to 2% of the building height (0.24 m). For the symmetric structure it can be seen that the stiffness and the strength in Y direction are slightly larger than in X direction. The overstrength for the symmetric structure, defined as the maximum strength divided by the design base shear (2565 kN), amounts to about 1.7 for X direction and about 1.8 for Y direction. In the case of asymmetric buildings, both stiffness and strenght decrease. The decrease is larger for larger eccentricity.

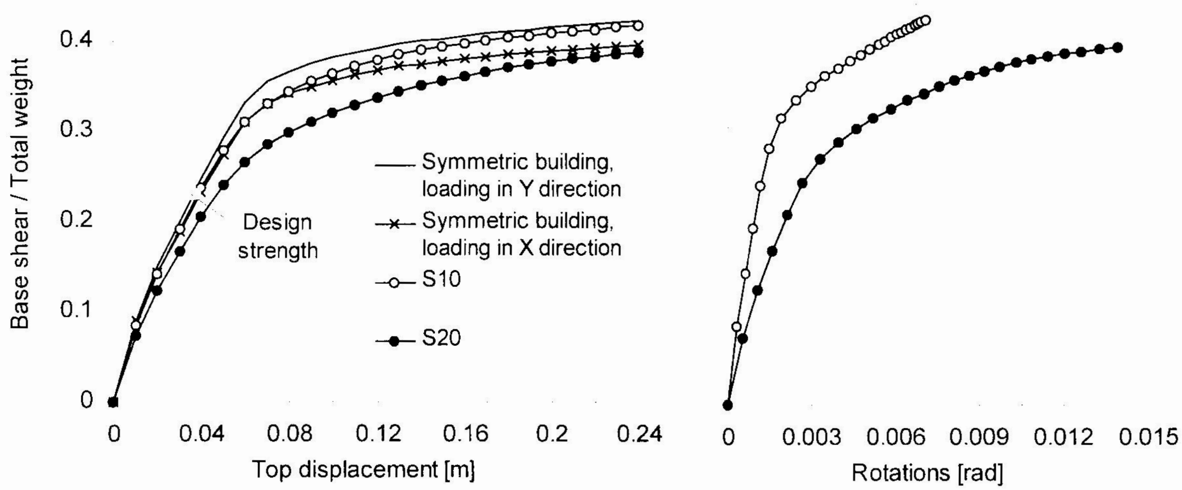


Figure 3. Base shear - top displacement and base shear - top torsional rotation relationships for the symmetric and two asymmetric building variants.

RESULTS OF DYNAMIC ANALYSES

Selected results of time-history analyses are shown in Table 1 and Figures 4-7. In Table 1 maximum top displacements (average values obtained for five ground motions) are presented. Displacements are given for frames X1 and X6, for mass center CM (in Y direction) and for the frame Y1 or Y4 (larger of the two values). Displacements in Y direction correspond to the bi-directional and to the uni-directional loading in Y direction (values in square brackets). For X direction (frame Y1 or Y4) four values are given which correspond to bi-directional loading, loading in Y direction, loading in X direction and SRSS combination of two uni-directional loading cases, respectively. In Figures 4 and 5 envelopes of displacements and torsional rotations for characteristic frames are presented. In Figure 4 the average values for five selected records are plotted for four investigated building variants. For a comparison the response of the symmetric building is also shown.

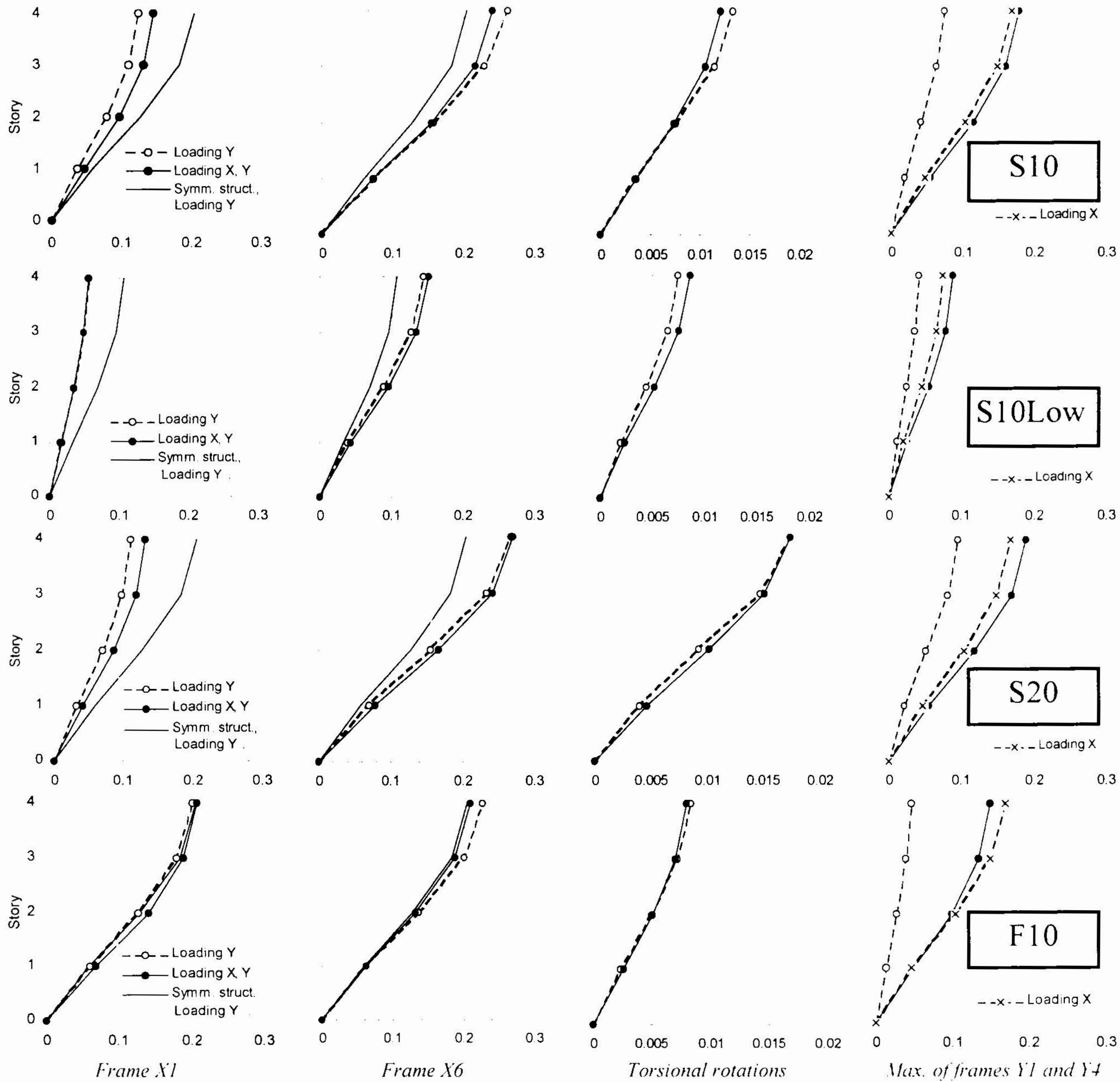


Figure 4: Envelopes of displacements [m] and torsional rotations [rad] for studied buildings (average values).

In Figure 5 the influence of different ground motions (scaled to the same peak ground acceleration) is shown. Ground motion was applied simultaneously in two directions. Plan views of envelopes of top displacements in +Y and -Y direction are shown in Figure 6. Results correspond to the torsionally stiff building S10 and to its torsionally flexible counterpart F10. In addition to average values, displacements obtained for three particular records are also shown. In Figure 7 rotations at beam and column ends are presented for frames X1 and X6 in the symmetric building and in the asymmetric building S20. Both buildings were subjected to El Centro ground motion in Y direction. Only rotations larger than yield rotations are indicated. Consequently, plastic mechanisms can be observed. Based on the results of the study the following observations can be made.

In the torsionally stiff building, as expected, eccentricity increases displacements at the flexible side and decreases displacements at the stiff side. The influence of torsion increases with the increase of eccentricity (but the relation is not linear) and with the decrease of intensity. The influence of ground motion, scaled to the same peak ground acceleration, is very large, mainly due to different frequency contents. The envelopes of displacements in plan are not linear. However, a line connecting maximum displacements at the flexible and stiff side is always on the safe side. As a consequence of increased and decreased displacements at the flexible and the stiff side, respectively, the rotations and related damage in different frames also change accordingly. In the case shown in Figure 7, the plastic mechanisms in frame X6 (at the flexible side) are similar in the symmetric and asymmetric building. However, the rotations in the asymmetric building are much larger. On the other hand, the rotations in the frame X1 (at the stiff side) of the asymmetric building are smaller and the plastic mechanism has not been formed.

In the case of the torsionally flexible building, the response of the asymmetric structure is very similar to that of the symmetric one. This is in striking contrast to the conclusions made by several other researchers, which observed large torsional influences in torsionally flexible buildings. However, in those cases torsional flexibility was produced by diminishing the torsional stiffness (and strenght). In the study, reported in this paper, a less realistic possibility of increasing the torsional moment of inertia was used. When using this option, the same phenomenon (i.e. very little influence of torsion) was observed within the research of our group on inelastic response of single-story structures (unpublished study by I. Peruš). Another observation on torsionally flexible buildings was made by De la Llera and Chopra (1995): "It has been observed that a reduction in torsional capacity of stiffness asymmetric systems may produce, at the expense of larger displacements, more uniform displacement demands among resisting planes". Additional research is needed in order to explore conflicting observations on the inelastic seismic response of torsionally flexible structures.

As regards the uni- versus bi-directional excitation, the difference is relatively small. In general, uni-directional ground motion slightly underestimates the response. However, there are some exceptions of this rule, e.g. the frame X6 of S10 (Figure 4). The correlation is improved if the results obtained by independent uni-directional input in two directions are combined by the SRSS (square root of sum of squares) combination rule (see Fajfar and Peruš, 1999). The investigated buildings are symmetric regarding X axis. Thus, the contribution of the loading in orthogonal direction applies only to frames in X direction. The results obtained by the SRSS combination rule are given in the last column of Table 1.

It should be noted that even for symmetric building there is some difference between the results of uni- and bi-directional loading. The reason is the multi-spring model used for modelling of columns. The response of this model depends on the axial force in columns. This effect introduces some asymmetry in the model of the building in the case of bi-directional input. Additional investigations of the behaviour of the multispring model in CANNY computer program are needed.

As a side product of the presented study, the seismic response of a building, designed according to Eurocodes 2 and 8, can be evaluated. The building was subjected to ground motions scaled to twice the design ground acceleration. The eccentricity was much larger than assumed in design. Nevertheless, the seismic demand was not excessive. In the case of such a ground motion, the building would be severely damaged, however, most probably it would not collapse.

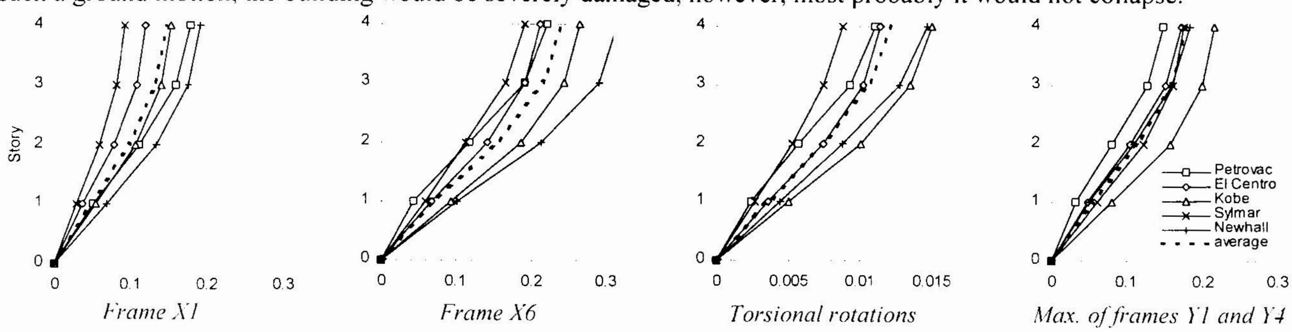


Figure 5: Envelopes of displacements [m] and torsional rotations [rad] for building S10 for different earthquake records (loading in two directions).

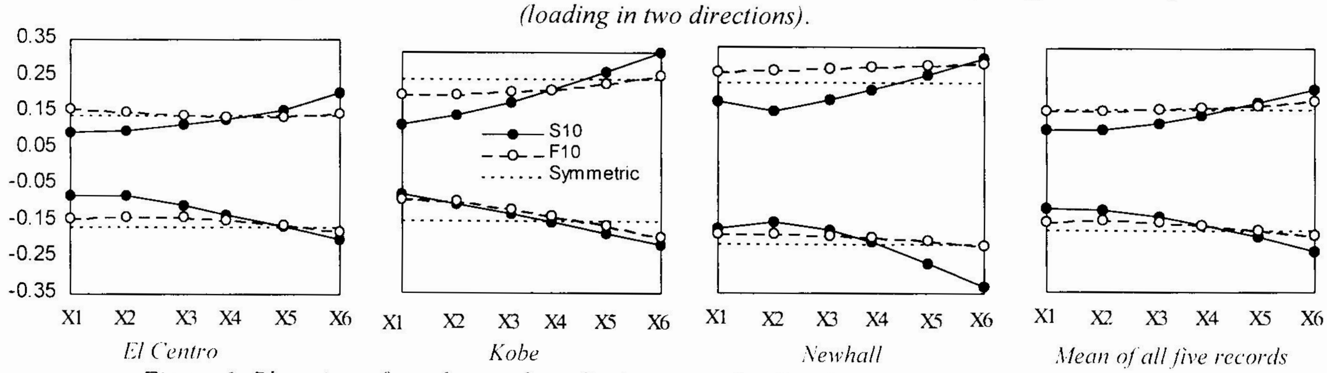
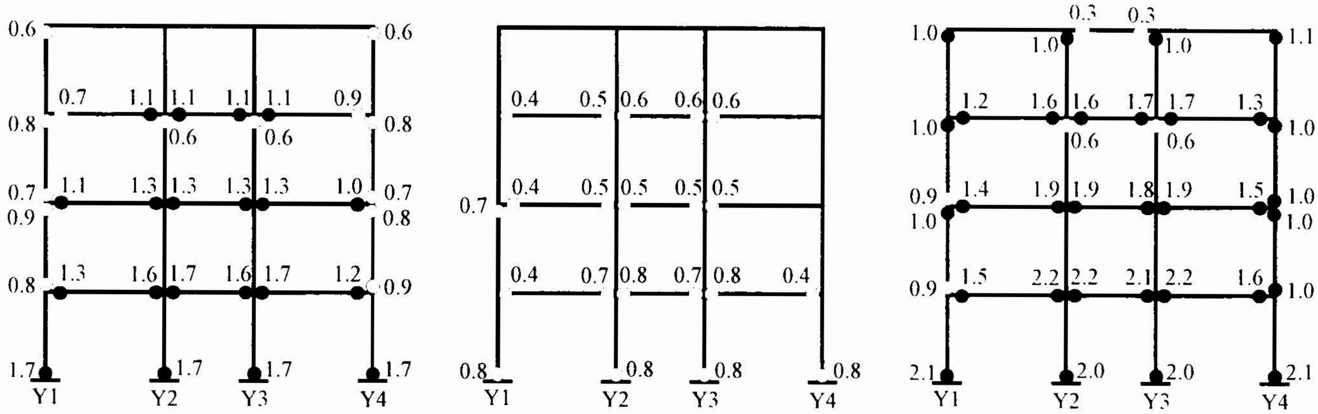


Figure 6: Plan view of envelopes of top displacements (scaling 0.7g, loading in Y direction only).



Symmetric structure (frames X1 and X6) *S20, frame X1* *S20, frame X6*

Figure 7: Maximum rotations ϕ for El Centro record scaled to 0.7 g, applied in Y direction (units 10^{-2} rad, O indicates yielding, $\phi < 1.0$; ● indicates yielding, $\phi \geq 1.0$;).

CONCLUSIONS

The inelastic seismic response of several variants of a building, conservatively designed according to Eurocodes 2 and 8, was studied. The conclusions apply to the investigated building. The behaviour of the torsionally stiff asymmetric variant was as expected. Due to torsion, an increase of displacements occurred at the flexible side of the building. The torsional effect increased with an increase in eccentricity. On the other hand it decreased with an increase in the intensity of ground motion, which causes larger inelastic deformations. Surprisingly, there was very small influence of torsion in the case of the torsionally flexible variant of the same building. However, it should be noted that this variant was produced by increasing the mass moment of inertia, whereas all other characteristics remained unchanged. Uni-directional ground motion in general underestimated the more realistic response obtained by bi-directional input. However, the difference was small, indicating that uni-directional pushover analysis may be a viable option for simplified nonlinear analysis.

ACKNOWLEDGEMENTS

The results presented in this paper are based on the work supported by the Ministry for Science and Technology of the Republic of Slovenia.

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