SEISMIC PERFORMANCE OF IRREGULAR 3D RC FRAMES

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ABSTRACT :

his paper deals with seismic performance of an irregular mass-eccentric 3D RC framed structure subjected to seismic actions. The sample structure has three double-span and six-storey plane frames and it is stiffness-regular both in plan and in elevation. A very detailed model has been set through the computer code Zeus^{NL}, which takes into account all the main characteristics of inelastic behaviour of rc structures.
The seismic input has been defined by considering seven ground motions reproducing the design specrum provided by EC8 and scaled in order to impose different values of PGA. Seismic response of the structure has been analyzed by performing a nonlinear dynamic analysis. In the analyses, mass center has been shifted from stiffness center at a distance going from zero to 15% of the relevant building plan dimension. Seismic responses, expressed in terms of top displacement and interstorey drift, have been evaluated for each selected value of PGA and eccentricity.

KEYWORDS: Irregular structures, mass irregularity, inelastic analysis, 3d rc frames.

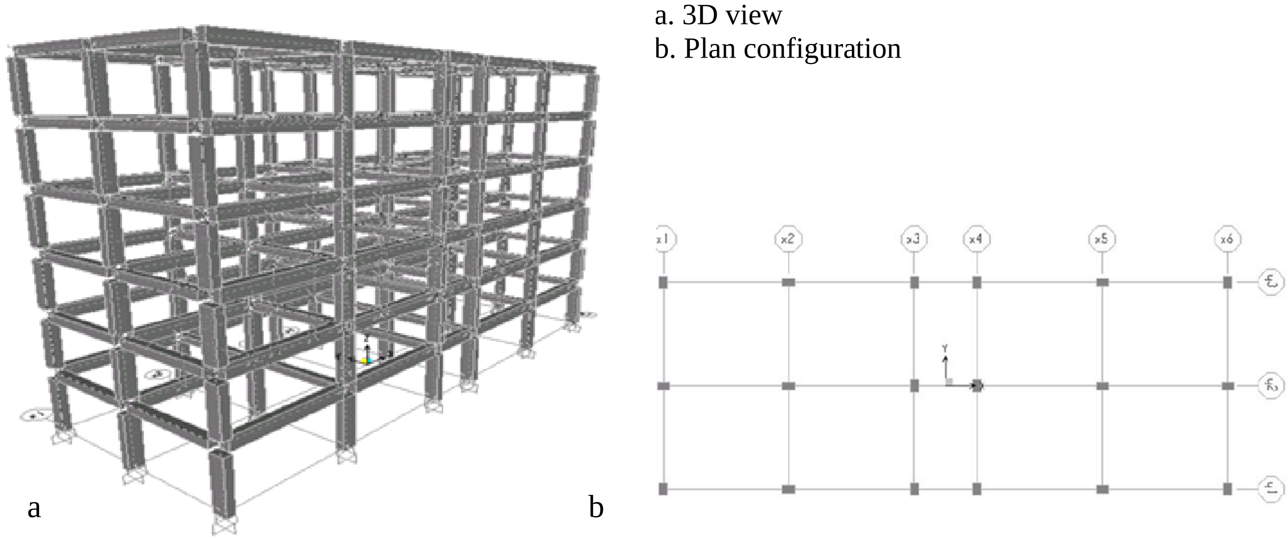
1. INTRODUCTION

As it is well known, structural regularity is an important issue for a good seismic response. Despite structural regularity is quite easy to obtain through a careful design; it is very common that, in the reality, different irregularities can occur, changing the seismic performance of the building. The sample structure studied in this paper is a frame RC building, stiffness symmetric in plan and regular in elevation. An eccentricity in the mass distribution, that is a non coincidence between mass and stiffness centers, has been introduced, as mass distribution is very easy to control during the design of the building, but not easily predictable during the life of the building, since it is related to its use, that is changeable during the time.
In this paper the effect of mass eccentricity on the seismic has been investigated. Two different eccentricities levels: low eccentricity (LE) equal to 5% and high eccentricity (HE) equal to 15% have been considered, in addition to the case of regularity (NE, i.e. no eccentricity). The regular sample frame has been designed according to the rules provided by Eurocode 8 for RC high ductility structures. The seismic response has been measured in terms of top displacement and interstorey drift, i.e. response parameters that are widely recognized to evaluate seismic performance. The observation of the obtained top displacements leads to evaluate the sensitivity of the seismic response to the mass eccentricity and the difficulties in predicting the seismic behavior of the building through simplified analyses. The response domains obtained for interstorey drift have been compared with the limit values provided by FEMA for the different limit states in order to evaluate the deterioration in seismic performance of the sample structure due to the mass eccentricity.

2. SAMPLE STRUCTURE

Sample structure is a six storey 3D RC frame. Its plan, shown in Figure 2.1, has two spans in y-direction (both length equal to 5 m) and five spans in x-direction (length, respectively, of 3 and 6 meters). The structure has been designed according to the rules provided by EC8 for framed high ductility structures, namely by applying the capacity design. A Peak Ground Aacceleration (PGA) equal to 0.35g, and a soil type B have been assumed for the design.

a. 3D view
b. Plan configuration



a b

Figure 2.1 Sample structure: 3D view and floor plan.

Concrete cubic strength has been assumed equal to 30 MPa and steel yield stress equal to 440 MPa. Both columns and primary beams have a rectangular cross section, 40x70 cm at the first 3 storeys and 40x60 cm at the upper three storeys. Secondary beams, in the y-direction, have a rectangular cross section 40x45 cm.
Reinforcement has been almost entirely made of rebars with 16 mm diameter, and only in few sections with bars having a larger diameter (20 mm). Columns have been equally oriented along the two main directions, as it can be seen in Figure 2.1.

3. SEISMIC INPUT

Seismic input consists of an ensemble of seven ground motions, listed in Table 3.1. As it can be seen in Figure 3.1, their mean elastic spectrum matches with a good approximations the elastic spectrum provided by EC8 for the soil type B.
Each record has been scaled so as its PGA equals three values: 0.20 g, 035 g and 0.50 g; in this manner, different levels of plastic excursions are investigated.

Table 3.1 Characteristics of record ensemble.

code	event name	country	date	station name
000196xa	Montenegro	Yugoslavia	15/04/1979	Petrovac (Hotel Oliva)
000199ya	Montenegro	Yugoslavia	15/04/1979	Bar (Skupstina Opstine)
000535ya	Erzincan	Turkey	13/03/1992	Erzincan
006263ya	South Iceland	Iceland	17/06/2000	Kaldarholt
006328ya	South Iceland (aftershock)	Iceland	21/06/2000	Kaldarholt
006334xa	South Iceland (aftershock)	Iceland	21/06/2000	Solheimar
006334ya	South Iceland (aftershock)	Iceland	21/06/2000	Solheimar

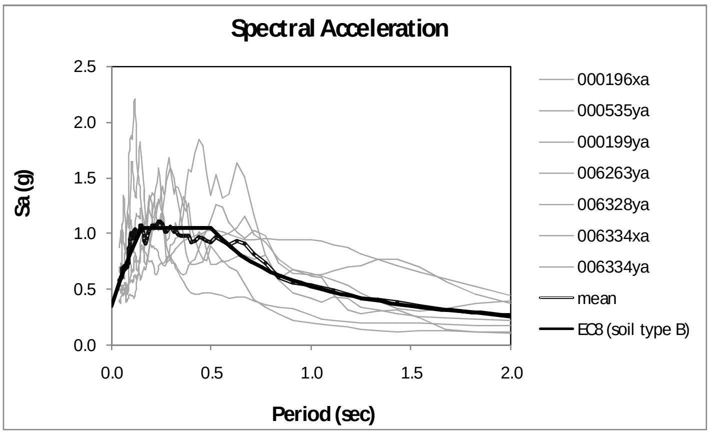


Figure 3.1 Comparison of EC8 elastic spectrum to those of the seven selected ground motions.

4. TIME HISTORY RESPONSE ANALYSIS

4.1. Computer model

Seismic response of the sample structure has been found by performing a time-history analysis through the computer code ZEUS^{NL} (Elnashai et al. 2002). The structure has been modeled with a fiber model, which leads to a very accurate description of geometrical and mechanical properties of each element. The confined concrete has

been described through the Mander et al. (1988) model, while behavior of concrete cover has been represented by a tri-linear model. Reinforcement behavior has been described by a bi-linear model including hardening. Each beam and column has been divided in four elements: the two elements close to the nodes, each having a length equal to 1/15 of the total length, and two longer elements in the central part. This detailed modeling of the elements leads to a precise description of the reinforcing rebars inside each element.
The contribution of the floor has been represented by introducing two diagonal beams for each floor span. Before starting the investigation, a modal analysis has been performed to determine the first shapes of the structures. The first mode, having a period of 0.77 sec, is related to the motion in y direction. Therefore, the following dynamic analysis has been carried out by selecting this direction.

4.2. Limit states

The seismic response of the sample structure has been assessed by comparing interstorey drifts to some conventional limit values provided by FEMA 356 (2000), as listed in Table 4.1

Table 4.1 Assumed limit states (see FEMA 356)

Limit states	Drifts (%)
Immediate Occupancy	1%
Life Safety	2%
Collapse Prevention	4%

5. RESULTS

A first investigation of seismic response has regarded the shape of the plan envelope of top lateral displacements, as input ground motion, PGA and mass eccentricity vary. Figure 5.1 shows the values of the maximum top displacements, averaged over the seven ground motions; curves show well known trends, i.e. increase in torsional effects with mass eccentricity, that are confirmed by Figure 5.2, showing mean top lateral displacements nondimensionalized to the corresponding top displacements at the plan center.
From the diagrams shown in Figure 5.1 it can be seen that, at the increase of the eccentricity, even the maximum top displacement increases. But, while for the lower value of PGA the increase in top displacements occurs only on the same side of eccentricity, for higher PGA, in the case of high eccentricity, there is an increase of seismic response even on the opposite side of the frame.
To better evaluate the effect of mass eccentricity on the distribution of maximum response parameters, the maximum top displacements have been normalized respect the value of the baricenter. The obtained curves, shown in Figure 5.2, evidenced the effect if inelastic involvement of the dynamic response of the frame. In fact the maximum sensitivity of the response to the mass eccentricity should be related to the elastic response of the structure, and therefore to the lower PGA value.

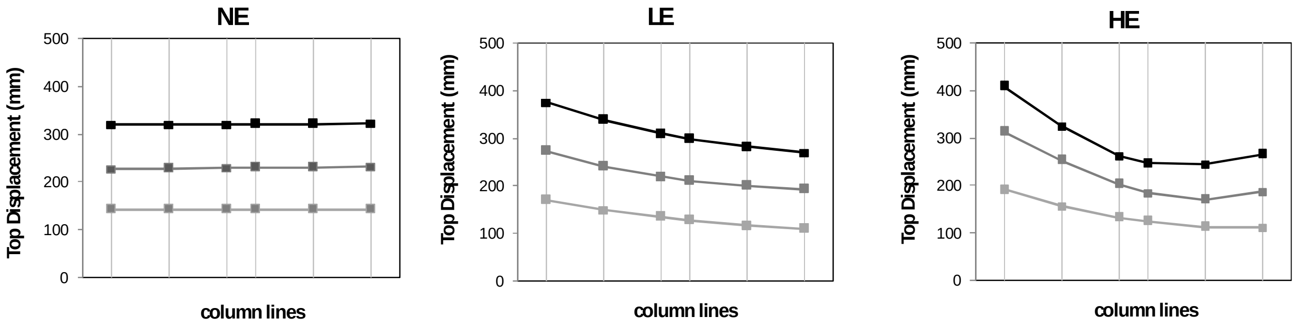


Figure 5.1 Mean top displacements. PGA = 0.20g PGA = 0.35g PGA = 0.50g

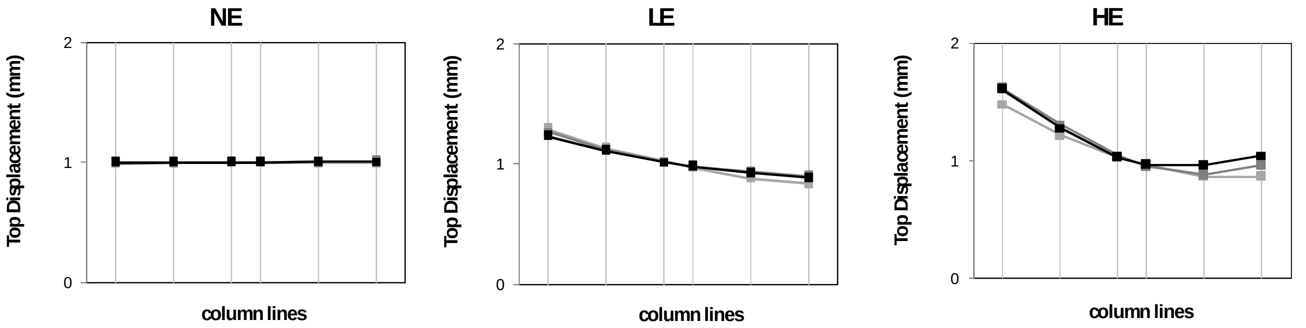


Figure 5.2 Normalized top displacements. PGA = 0.20g PGA = 0.35g PGA = 0.50g

However, it can be seen that top lateral displacements are affected by large scatters, as the input ground motion varies: Figure 5.3, indeed, shows that the coefficient of variation (*cov*) is very large, ranging from 0.70 and 0.80. As a consequence, it has been chosen to look at each earthquake response. In particular, Figure 5.4 compares top displacement plan envelopes obtained with the two following records: Erzincan (000535ya) and Kaldarholt (006263ya). It can be seen that the shape of the envelope varies significantly with the considered input ground motion: with the record 006263ya it recalls that of torsionally flexible systems, where displacements at both the stiff and the flexible sides are similar; conversely, with the 00535ya record the shape of top lateral displacements recalls that of torsionally stiff systems, with larger displacements at the flexible side. Of course, since results refer to the same structure, this trend is probably due to effects of different inelastic behavior induced by the two considered records.

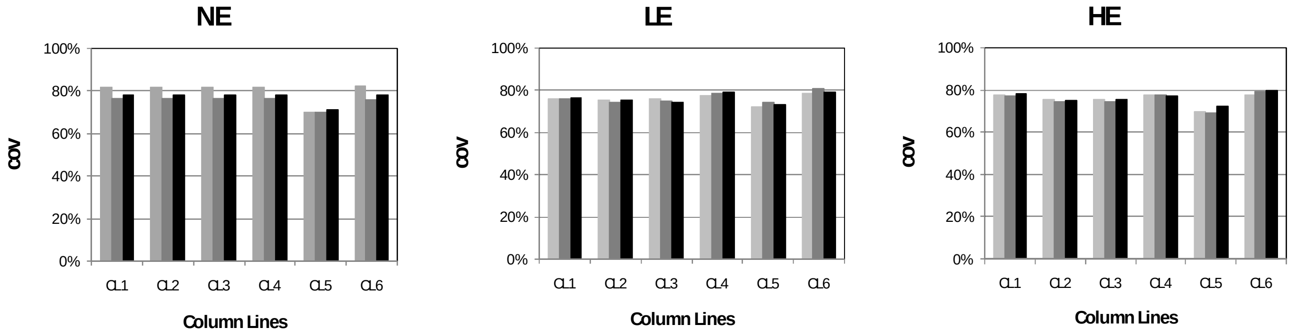


Figure 5.3 Coefficient Of Variation. PGA = 0.20g PGA = 0.35g PGA = 0.50g

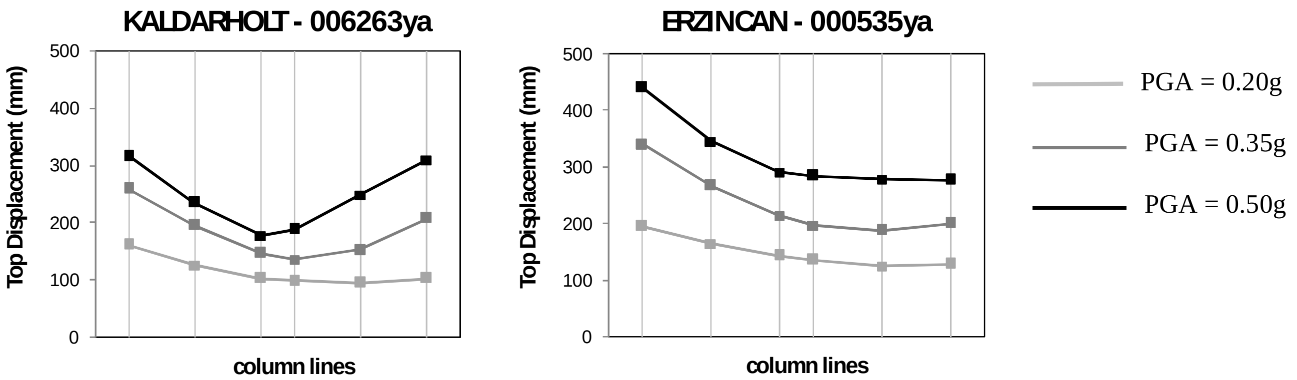


Figure 5.4 Top displacements from two ground motions.

Figure 5.5 shows the mean values of the maximum interstorey drift found for each column line. In this case, mean values are more representative since interstory drift *covs* of the response domains are definitely lower, ranging from 0.30 to 0.50 as shown in Figure 5.6. Interstorey drifts are compared to the limit values subscribed by FEMA 356 (2000) for the three performance levels, in order to better understand effect of mass eccentricity on damage sustained by the structure.

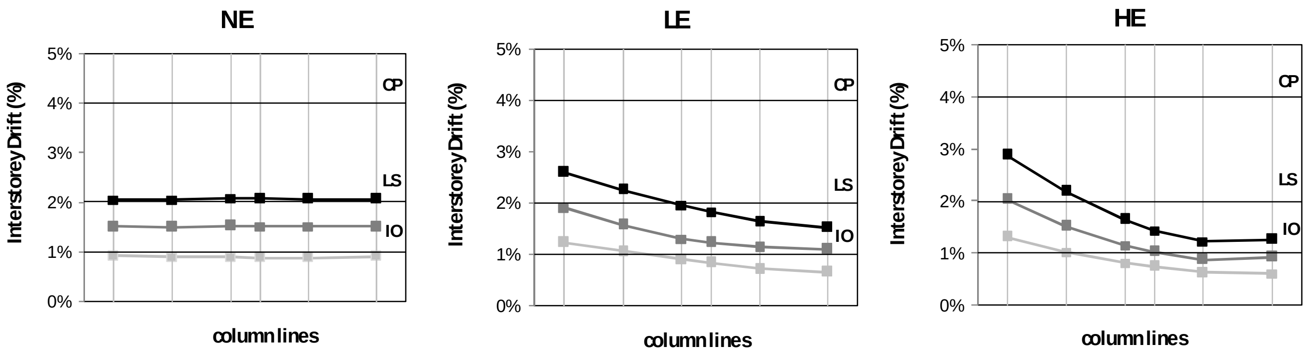


Figure 5.5 Mean interstorey drift. PGA = 0.20g PGA = 0.35g PGA = 0.50g

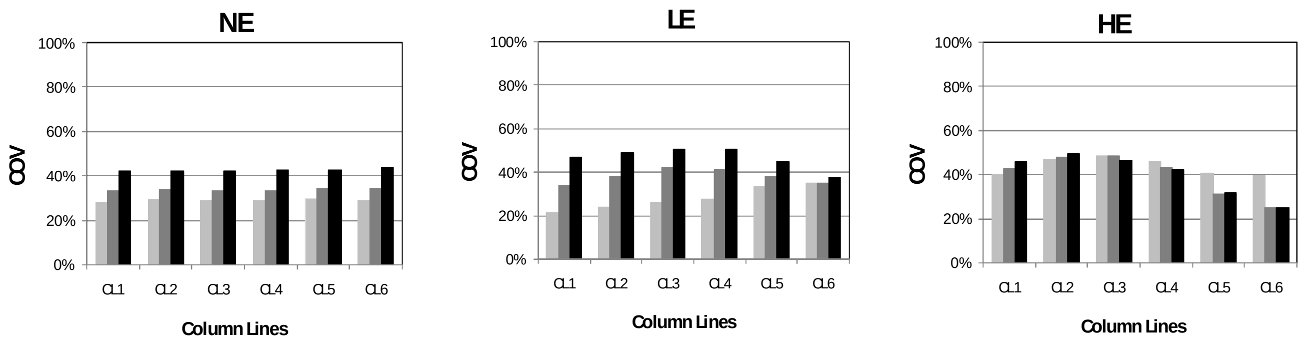


Figure 5.6 Coefficient Of Variation. PGA = 0.20g PGA = 0.35 g PGA = 0.50g

6. CONCLUSIONS

In this paper the incidence of mass eccentricity has been studied with reference to a framed six storey 3D building structure designed according to EC8. A detailed model has been set with the program ZEUS^{NL} and time-history analyses have been performed to obtain values of response parameters such as top displacements and interstorey drifts. As expected, the top displacements are very sensitive to the mass eccentricity, which induces an increase of about 50% at the flexible side, when it is equal to 15% of the relevant plan dimension. In any case, top lateral displacements are affected by large scatters as input ground motion varies; even the shape of plan envelope may vary significantly.

Maximum interstorey drift has a more predictable trend and a lower coefficient of variation, ranging between 0.20 and 0.50. The mass irregularity induces a significant increase in interstorey drift, up to 50% in the case of large eccentricity.

Further investigations must be done to better understand variation in the shape of the envelope of top lateral displacements with the earthquake record, which in turn influences the possibility to predict the seismic performance of plan irregular structures by adopting simplified procedures, such as pushover analysis.

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