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BIDIRECTIONAL ANALYSIS OF A REAL RC BUILDING ACCORDING TO THE ITALIAN CODE PROVISIONS

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Abstract. *The seismic assessment of existing buildings is an essential issue of seismic engineering. This work is focused on the effects of a bi-directional analysis on the evaluation of the seismic response of existing RC buildings according to the current Italian Technical Code. A nonlinear dynamic (time-history) analysis has been performed with reference to a case-study, that is a real RC hospital building. An accurate knowledge of the building has been achieved, as a result of a collaboration between the University of Florence and the Regional Government. Two different limit states, i.e. a serviceability (Damage Limitation) and a ultimate (Severe Damage) one, have been considered. Two ensemble of ground motions have been selected to represent the seismic action for the two limit states. Each set is made of 14 records, i.e. 7 ground motions with their two (N/S, E/W) components. The seismic response of the case-study has been studied in terms of displacements and interstorey drifts. The drift profile at each column-line of the case-study has been checked in order to evaluate the torsional effects arising by the eccentricities in the two directions.*

1 INTRODUCTION

The assessment of the seismic performance of existing buildings is a crucial issue of seismic engineering. In Italy, as well as in many other European countries, most part of RC buildings have been constructed in the 60s and 70s and therefore they often do not comply with the technical requirements provided by the Codes in force; a reliable evaluation of their seismic capacity is an essential step for a convenient choice about their use and maintenance.

According to the current Italian (NTC 2008, [1]) and European (Eurocode 8, [2]) Technical Codes, a seismic input acting along two different orthogonal directions must be considered for analysis. Usually the wave trains in the two directions are separately considered, and their effects are combined according to the Code' rules. Different combination criteria are provided by the Code, depending on the type of performed analysis.

In this work a bi-directional nonlinear dynamic analysis has been performed on a case-study, i.e. a real RC building, slightly irregular in plan, currently used as a hospital [3-5]. Its numerical modeling is based on a wide knowledge process, that is the result of a joint agreement with the Regional Government of Tuscany. A large number of experimental tests have been performed to evaluate material and ground mechanical properties [6-7].

The seismic response of the building is found with regards to two different limit states, i.e. a serviceability (*Damage Limitation, DL)* and a ultimate one (*Significant Damage, SD*).

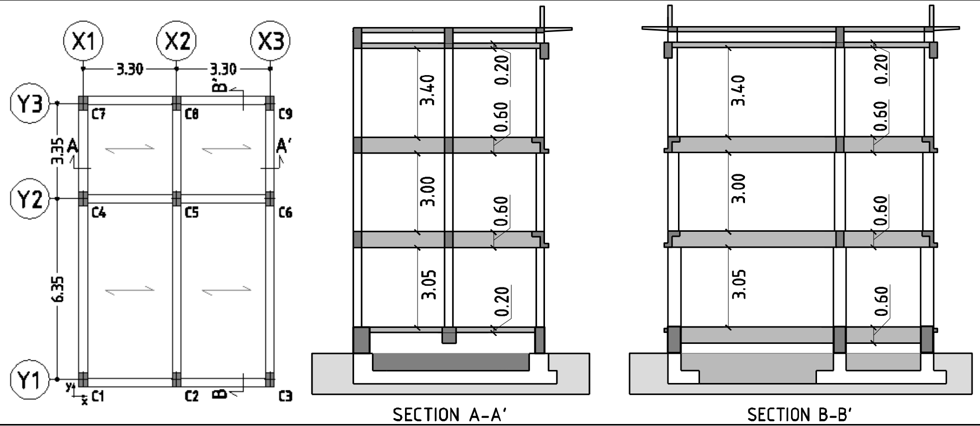
For each limit state an ensemble of seven ground motions, spectrum-compatible with the elastic seismic spectrum provided by the Italian Code, has been selected. The orthogonal components of each ground motion, which are also spectrum-compatible, are assumed to represent the ensemble in the orthogonal direction. For each selected seismic event, the two components of the ground motion have been applied in the two main directions of the building and then reversed. Five analyses have been run for each direction and limit state: one with the acceleration trains applied to the mass center and other four by introducing a ±5% eccentricity in the two directions according to the Code prescriptions.

Since the building is irregular the seismic action induces possible torsional effects [8-10]. Therefore the seismic response, expressed in terms of drift and displacement, has been checked both at the mass center and at the side column lines of the structure. Firstly, the results obtained by applying the assumed set of ground motions at the mass center only, have been preliminary discussed, in order to evaluate the influence of the direction of application of the two different components in terms of scatter of the response quantities. Then the enveloped results found by introducing the ±5% eccentricity have been found and shown, in terms of *mean* values only. Finally, the enveloped drift profiles of the building have been shown both at the mass center and at the side column lines, to investigate the torsional effects arising by the in-plan irregularities.

2 THE CASE STUDY

The case-study, shown in Figure 1, is a framed 3-storey RC building. It has been designed in 1976, i.e. just after the introduction of the first seismic Italian Technical Code, and it presents some efficient design criteria, such as column section reduction from foundation level to the top storey and solid joint connection at beam-column intersection, although it is far away from complying the current seismic design criteria. The dimensions of beams and columns are listed in Table1, while the reinforcements data can be find in La Brusco et al. [3-5]. It should be noted that the third floor of the building consists of two different structural layers, partially coinciding. In fact, two different floors, 40 cm far away each other, constitute the last storey of the building. In some alignments (*X3, Y1* and *Y3*) two layers of beams separately support the two different floors, whilst in the other alignments (*X1*, *X2* and *Y2*) a single beam supports both floors. Therefore the 3rd floor and the related beams will be in the following distinguished with a subscript *a* or *b*, depending on weather they refer to the lower or upper layer respectively. All floors are made by deck and concrete, and they have a total height of 20 cm. The infill panels of the RC frames have a double layer, with an inside casing. A in-depth investigation, including destructive and non-destructive tests [3-5], has been made to characterize the material mechanical properties; the obtained *mean* and design strengths for concrete and steel, together with the achieved Knowledge Level (*KL*), have been listed in Fig.1. Even the ground properties have been investigated through seismic refraction and down hole techniques; on the basis of such investigations, the soil has been classified as B-type, according to NTC2008 (Sec. 3.2.2, Tab. 3.2.II) and EC8 (Part 1, sec. 3.1.2, Tab. 3.1).

The case-study building is rectangular in plan, structurally symmetrical about the *Y*-direction, while in the *X*-direction the two spans have different lengths. In order to quantify the in-plan irregularity, the mass-stiffness eccentricity has been determined in the two main directions. At each storey, the mass center has been found by considering the mass of the floors and of the infill panels, while the center of stiffness has been determined by applying the simplified relationship proposed by Anagnastopoulos [11-13].



concrete:
class: Rck 250
$f_{c,mean}$ = 10.2 MPa
KL = 2
CF = 1.20
$f_{c,d}$ 8.5 MPa

Steel:
Class: FeB32K
$f_{s,mean}$ = 385.7 MPa
KL = 1
CF = 1.35
$f_{c,d}$ = 285.7 MPa

Figure 1. Plan and sections of the RC case-study building.

Table 1. Cross section data of beams and columns.

	beams						columns
	x_1,2; x_2,3; x_7,8; x_8,9	x_4,5; x_5,6	y_1,4; y_2,5	y_3,6	y_4,7; y_5,8	y_6,9	c1-c9
1st st.	Z-shape	30x60	30x60	Z-shape	30x60	Z-shape	30 x 50
2nd st.	Z-shape	30x60	30x60	Z-shape	30x60	Z-shape	30 x 40
3rd st.(a)	30x60	30x80	30x 80	30x60	30x80	30x60	30 x 35
3rd st.(b)	30x20	30x20	-	30x20	-	30x20	

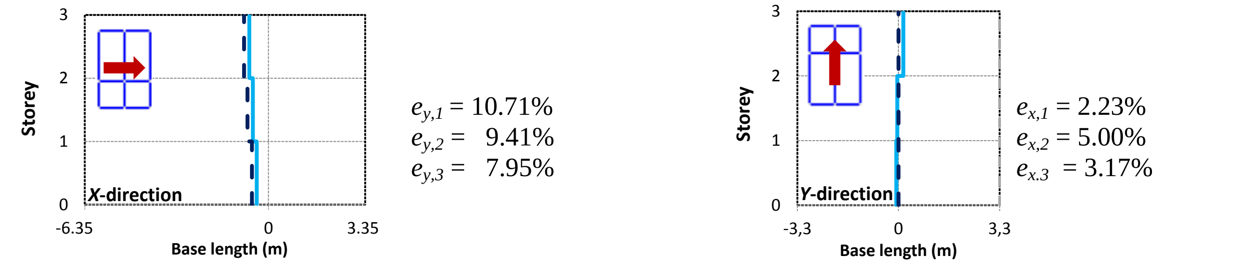


Figure 2. Stiffness eccentricity of the case-study.

Figure 2 shows the positions of the mass and stiffness centers at each storey in the two directions, together with the corresponding eccentricities. It can be noted that, despite the building has a symmetric structure about the *Y*-direction, it presents non-zero eccentricity even in this direction, confirming the role of infill panels distribution on structural regularity [14].

3 ASSUMED GROUND MOTIONS AND SEISMIC ANALYSIS

3.1 The seismic input

A soil-type B has been assumed for the case-study ground, with a maximum PGA equal to 0.32 g and 0.15 g respectively for the two limit states. To perform the nonlinear dynamic analysis, two different sets of ground motions (see Table 2) have been considered, whose average spectra closely approaches the NTC 2008 ones (for both assumed limit states).

Table 2. Ground motions data for the assumed Limit States (soil-type B)

l.s.	Ensemble code	Record code	Event	Station Name	Date YYYY-MM-DD	PGA [g]	scale factor
Damage Limitation (DL)	*DL_E/W DL_N/S*	FRC_HN	FRIULI_EARTHQUAKE_4TH_SHOCK	FORGARIA CORNINO	1976-09-15	0.332	0.45
		SRC0_HN	FRIULI_EARTHQUAKE_4TH_SHOCK	S. ROCCO	1976-09-15	0.249	0.60
		AQK_HN	L_AQUILA_MAINSHOCK	L'AQUILA - V. ATERNO	2009-04-06	0.330	0.45
		CSO1_HN	L_AQUILA_MAINSHOCK	CARSOLI1	2009-04-06	0.018	7.99
		MRT_HN	L_AQUILA_MAINSHOCK	MONTEREALE	2009-04-06	0.044	3.40
		AQK_HN	L_AQUILA_EARTHQUAKE	L'AQUILA - V. ATERNO	2009-04-07	0.090	1.65
		SSU_HN	EMILIA_2ND_MAINSHOCK	SASSUOLO	2012-05-29	0.017	8.54
Severe damage (SD)	*SD_E/W SD_N/S*	SRC0_HN	FRIULI_EARTHQUAKE_4TH_SHOCK	S. ROCCO	1976-09-15	0.249	1.29
		PTT1_HN	PATTI_GULF_EARTHQUAKE	PATTI	1978-04-15	0.163	1.98
		AQK_HN	L_AQUILA_MAINSHOCK	L'AQUILA - V. ATERNO	2009-04-06	0.330	0.98
		MDN_HN	EMILIA_1ST_MAINSHOCK	MODENA	2012-05-20	0.037	8.73
		CNT_HN	EMILIA_2ND_MAINSHOCK	CENTO	2012-05-29	0.224	1.44
		BUI_HN	FRIULI_EARTHQUAKE_3RD_SHOCK	BUIA	1976-09-15	0.093	3.47
		MDN_HN	EMILIA_2ND_MAINSHOCK	MODENA	2012-05-29	0.030	10.69

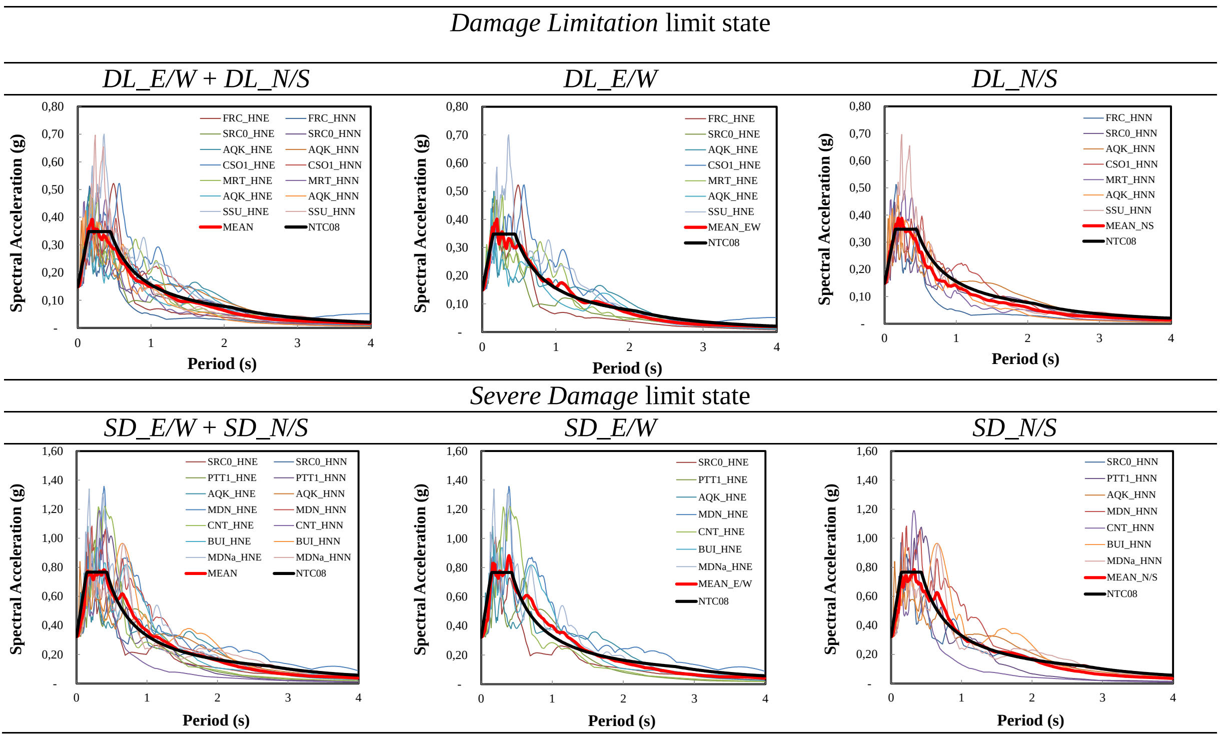


Figure 3 Comparison between the elastic spectra of the assumed ensembles and the NTC 2008 ones.

For each limit state, 7 ground motions, with their two directional components (N/S, E/W) have been selected. Each ensemble has been identified with a code defining the limit state and the direction of application (see Table 2).

The ground motions have been taken by the Italian Accelerometric Archive [15] through the adoption of the software REXEL [16-18], on the basis of a soil-type B, a nominal life of 50 years, a magnitude between 5.5 and 6.5, and a *coefficient of use* equal to 2.0, as required for strategic buildings. In Figure 3 the two ensembles of elastic spectra assumed to represent the seismic input for the two limit states have been shown and compared to the ones provided by NTC 2008.

3.1 The seismic analysis

All analyses have been performed by using the computer code SAP2000 [19]. The two floors of the third level have been modeled according to the real geometry as regards the stiffness and strength distribution, while the mass (both translational and rotational) of the storey has been considered applied at the center of the storey package. The effect of the joint stiffness has been considered by introducing a rigid offset at each element end. A reduced value of the Young modulus of the concrete E_c, i.e. $E_{c,red}$ = 0.5 E_c, has been assumed, as allowed by NTC 2008 (4.1.1.1) and EC8 (part 1, 4.3.1(7)). The floor stiffness has been introduced by assigning the diaphragm constraint to all nodes belonging to the same floor. A lumped plasticity model has been adopted at each end of all structural members, with mono-directional flexural hinges. Plastic hinges have an almost constant bending moment until the ultimate rotation is achieved (hardening ratio between 2% and 6%), and a residual bending moment equal to 20% of the yield one. Limit values of bending moment and chord rotation have been evaluated according to NTC 2008, which are identical to those provided by EC8. The bi-directional analysis has been performed by applying, alternatively reversing the direction of application, 7 ground motions along one direction and the other 7 in the orthogonal direction (see Table 3).

Table 3. Performed analyses.

	Damage Limitation		Severe Damage	
Analysis Code	*DL_1*	*DL_2*	*SD_1*	*SD_2*
Seismic input	*X*-dir: *DL_E/W* *Y*-dir: *DL_N/S*	*X*-dir: *DL_N/S* *Y*-dir: *DL_E/W*	*X*-dir: *SD_E/W* *Y*-dir: *SD_N/S*	*X*-dir: *SD_N/S* *Y*-dir: *SD_E/W*

According to EC8 prescriptions, a ± 5% mass eccentricity has been introduced in both directions. The maximum response in each direction, therefore, is given by the envelope of 5 different cases: besides the one with no eccentricity, where the seismic input is applied to the mass center (*MC*), there are two cases with the 5% introduced along the *X*-direction (*X_E+* and *X_E-*) and two other cases with the 5% eccentricity introduced along the *Y*-direction (*Y_E+* and *Y_E-*).

4. SEISMIC RESPONSE OF THE CASE-STUDY

Figures 4 and 5 show, for the two main directions, the maximum displacements found at the mass center of each storey for the *DL* and *SD* limit states respectively. The diagrams report the maximum responses produced by each record, and their *mean* value, obtained by applying the assumed sets of ground motions to the mass center only. In the *X*-direction the building presents a more scattered response than in the *Y*-direction. This occurrence is expectable, since the building is asymmetric around the *X*-axis; therefore, when the seismic response is checked in the *X*-direction, some torsional effect can occur, affecting the global response of the structure. Reversing the direction of application of the two components of the ground motions, the displacement patterns show a slight variation. Figures 6 and 7 show the results of the same analyses in terms of interstorey drifts. When the response in the *X*-direction is checked, i.e. at the occurring of torsional effects, the shape of the drift profile is sensitive to the considered limit state: while the drift profile of the *DL* limit state is almost constant, the one obtained for the *SD* limit state achieves its maximum value at the first storey, regularly decreasing along the building height. Even in this case the different scatter associated to each set of ground motions can be noted.

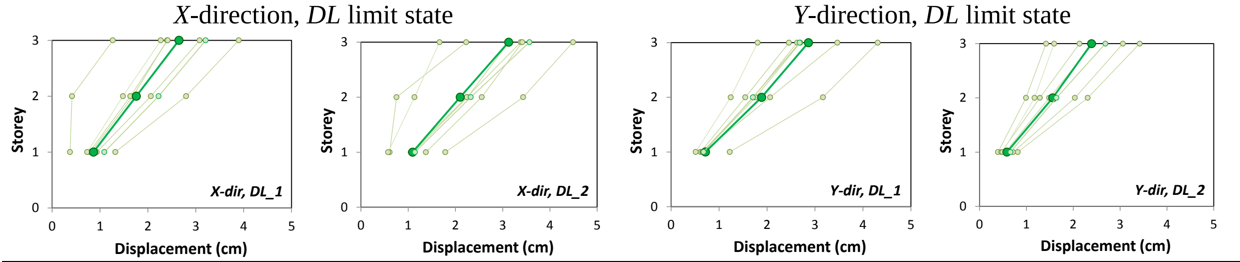


Figure 4. *DL* limit state: maximum displacement at the mass center.

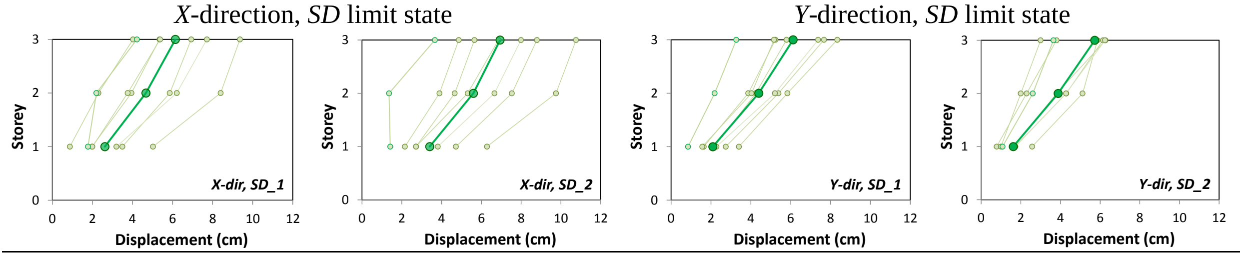


Figure 5. *SD* limit state: maximum displacement at the mass center.

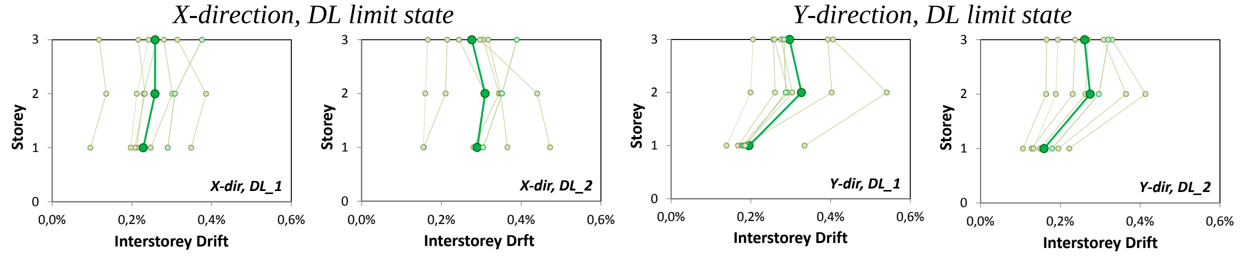


Figure 6. *DL* limit state: maximum Interstorey Drift at the mass center.

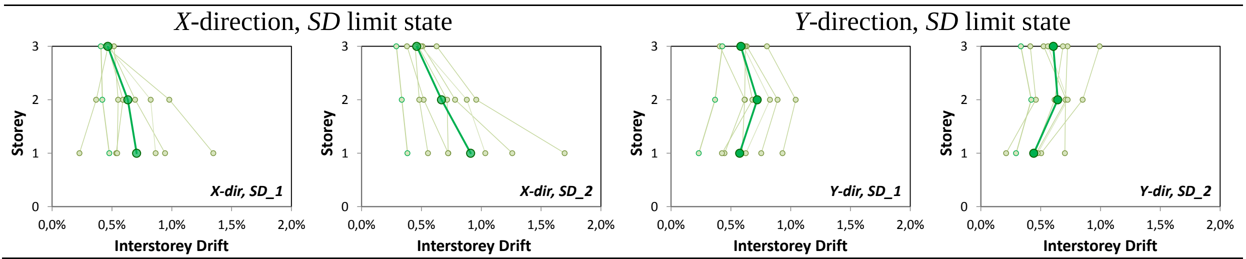


Figure 7. *SD* limit state: maximum Interstorey Drift at the mass center.

Figure 8 shows the response of the structure at the mass center in terms of maximum displacement for all the seismic analyses, comprehensive of the introduced 5% eccentricity in the two directions. As can be noted, the results coming from all the performed analyses are almost coincident with each other, showing that the introduced 5% eccentricity does not affect the displacement profile at the mass center for the present case study. The effects of the introduced 5% eccentricity is more pronounced in the displacement at the two ends of the structures, as can be seen in Figure 9, where the ratio between the maximum displacement experienced by the structure at the side column-lines and the one experienced at the mass center (D_{SIDE}/D_{MC}) is shown.

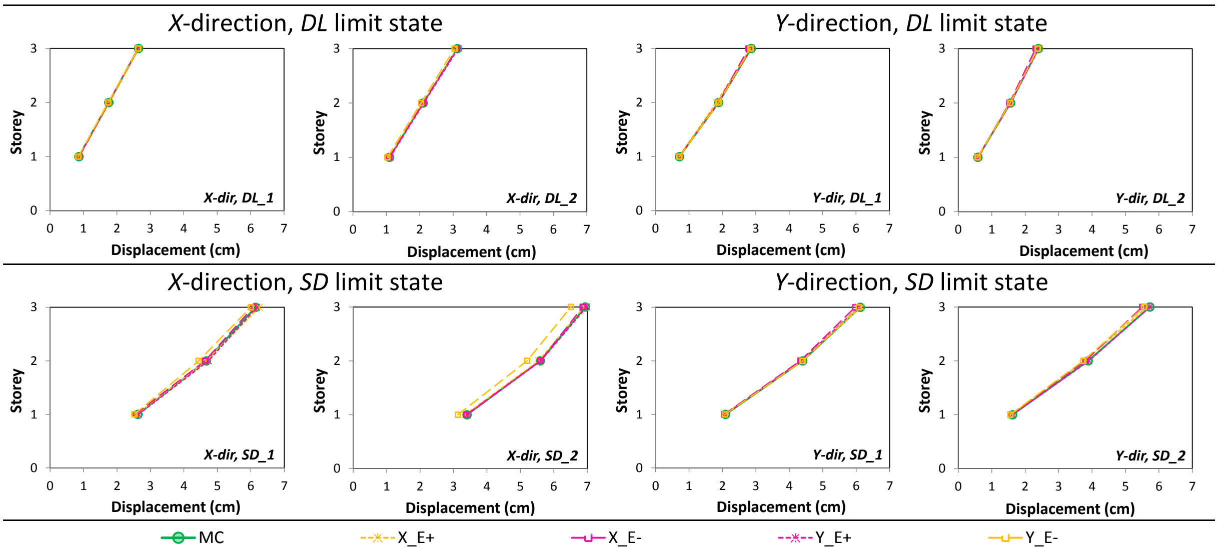


Figure 8. Maximum displacements at the mass center provided by all the performed seismic analyses.

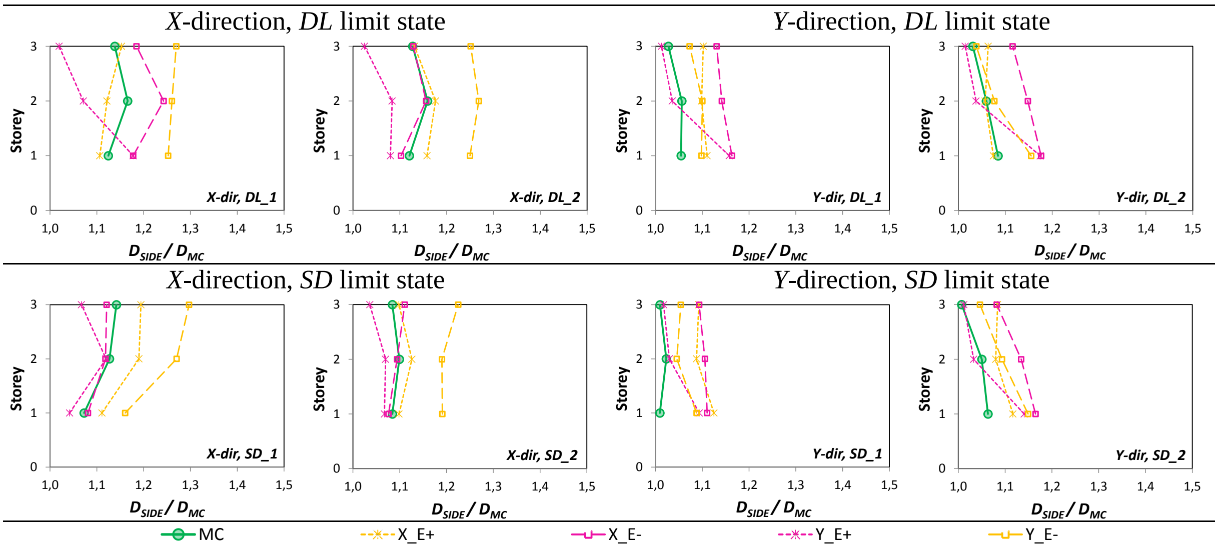


Figure 9. Ratio between the maximum displacement at the building flexible side and the one at the mass center.

Finally, Figures 10 and 11 show the enveloped drift profile related to the 5 analyses, at each column line of the building. As evidenced by the diagrams shown in Figure 9, in fact, the building, under the seismic excitation, exhibits a torsional behavior due to the in-plan eccentricity. The drift occurring at each column line, therefore, can differ from the one observed at the mass center of the building. Diagrams in Figs 10 and 11 show the enveloped drift profile, i.e. the maximum by the 5 analyses, provided by each set of ground motions. The scatter between the maximum drift at the side column lines is more pronounced in the *X*-direction, due the geometrical non-symmetry of the building along the *X* direction. As already observed in Figs 6 and 7, when the behavior in the *X*-direction is checked, the drift profile is different for the two limit states: in the *DL* limit state, in fact, the maximum drift occurs at the second level, while in the *SD* one it occurs at the first level. To better understand the sensitivity of each performed analysis to the torsional effects, the percentage increase of interstorey drift at the side columns has been found with respect to the one observed at the mass center (Figure 12).

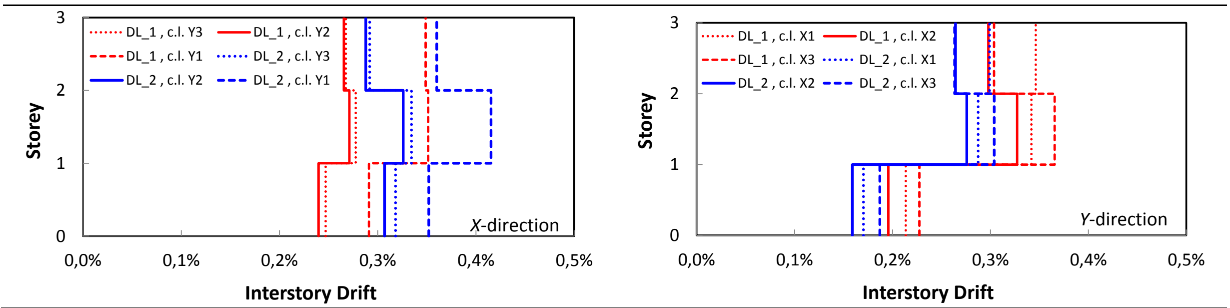


Figure 10. Interstorey Drift envelope at each column line (*DL* limit state).

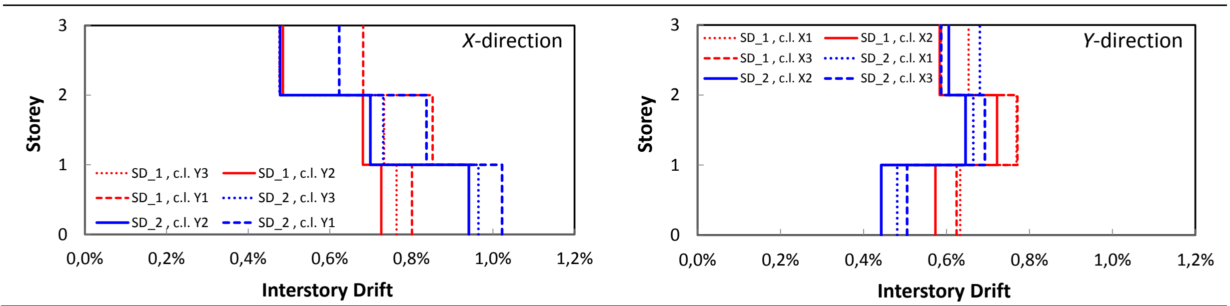


Figure 11. Interstorey Drift envelope at each column line (*SD* limit state).

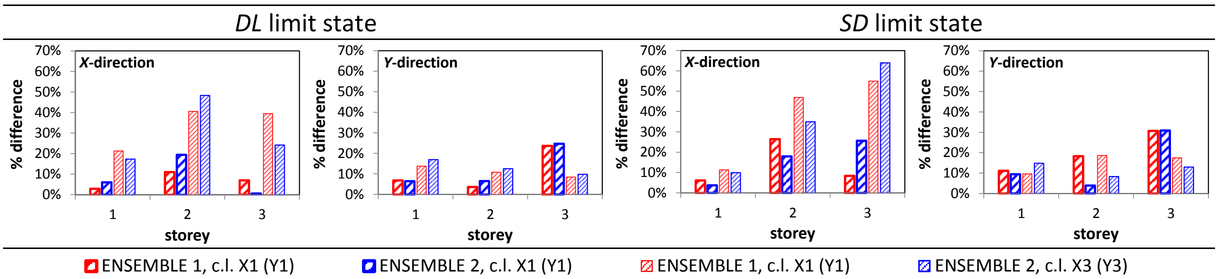


Figure 12. Percentage increase of Interstorey Drift at the side column line with respect to the one at the mass center.

5. CONCLUSIVE REMARKS

In this work a bi-directional nonlinear dynamic analysis has been performed on a real RC framed structure, i.e. a 3-storey existing building currently used as a hospital. Two ensembles of 7 ground motions each, whose both components (N/S, E/W) are compatible with the elastic spectrum provided by the Italian Code for the case-study site, have been selected and used as seismic input for the *Damage Limitation* and *Significant Damage* limit states verification, respectively. Each ground motion has been applied with 2 components oriented on both the main direction of the building, alternatively, and by introducing the ±5% eccentricity prescribed EC8 and NTC 2008. The structural response has been expressed in terms of displacement and interstorey drift.

Different evaluations have been made on the obtained results. First of all the effects of the choice of the assumed ensembles of ground motions have been checked. The amount and the scatter related to each considered ensemble direction have been shown and compared. The two components of the assumed ground motions, slightly differing each other, produce almost identical results in terms of *mean* response, despite providing a different amount of scatter. The case-study, which is not symmetric around the *X*-axis, shows a different response in the two main directions, evidencing a larger response along the *X*-direction.

The effects of the introduced eccentricity have been checked in terms of maximum displacements only. The maximum displacements found at the mass center by performing the five analyses for each set of ground motions are almost coincident, showing a negligible effect of the introduced 5% eccentricity for the present case study. If the introduced eccentricity does not affect the maximum translational response, it increases the torsional effects arising in the structure as a consequence of the seismic excitation. To evaluate the role of the 5% eccentricity in the torsional response of the structure, the ratio between the maximum displacement experienced by the structure at the side column lines and the one at the mass center has been checked. The increase in displacement due to the 5% eccentricity achieves 30% along the *X*-direction.

Finally, the torsional effects have been observed by checking the maximum enveloped interstorey drift at each column-line of the structure. The comparison among the maximum drift experienced by the building at the flexible side and at the mass center one has evidenced not negligible torsional effects in both the main directions, with an expectable prevalence in the non-symmetric *X*-direction.

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