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THE EFFECTS OF THE SEISMIC INPUT ON THE SEISMIC RESPONSE OF RC BUILDINGS

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Abstract. *This work is aimed at investigating the variation of the seismic response of RC structures subjected to different ensembles of ground motions in the nonlinear dynamic analysis for the seismic assessment of existing RC buildings. At the current time indeed, all the main International Seismic Codes provide a soil classification which is based on the shear wave velocity, the soil morphology and the assumed distance from the fault source. Depending on the soil properties, a suitable elastic spectrum is provided, defined on the basis of average properties assumed for the soil. To perform a nonlinear dynamic analysis, an ensemble of ground motions compatible to the elastic spectrum must be selected. The ensemble can be made by ground motions compatible to the Code spectrum for the assumed soil-type, or, alternatively, by assuming the ground motions to be consistent with the bed rock Code spectrum and by filtering them according with the specific stratigraphy of the site soil. In this work a comparison among these different approaches, all compatible to the European (Eurocode 8, EC8) and Italian (NTC 2008) Code provisions, has been made on a case-study, i.e. a real irregular RC Italian building. The comparison has been made in terms of seismic response, i.e. by checking the maximum displacement and interstory drift induced by the assumed ensembles of ground motions.*

1 INTRODUCTION

The evaluation of the seismic performance of existing buildings is affected by many uncertainties, both aleatoric or epistemic ones. The seismic input is considered to be the most random quantity involved in such evaluation [1,2,3]. It is commonly accepted, indeed, that the properties of ground motions which will occur in a certain area - i.e. intensity, frequency contents, distance from the source fault, etc. - are not possible to be exactly predicted. In the past years many studies have been made to limit the aleatoric uncertainty, by carefully mapping the entire area of countries affected by seismic hazard and collecting all types of information useful for prediction.

Even the epistemic uncertainty has been strongly reduced. At the current time, all the main International Seismic Codes provide a soil classification which is based on the shear wave velocity *Vs*, the soil morphology and the assumed distance from the fault source. In Italy in the past decades the available information about the seismic hazard of the territory has been largely increased achieving, at the current time, a detailed mapping [4,5,6]. Depending on the soil properties, a suitable elastic spectrum is provided by the seismic codes [7,8], defined on the basis of average properties assumed for the soil. To perform a nonlinear dynamic analysis, an ensemble of ground motions compatible to the elastic spectrum must be selected [9]. The choice of a seismic input as close as possible to the code spectrum is a controversial matter [10]: it helps in limiting the randomness of the response quantities induced by the seismic event, but it introduces a “discretional” limitation of the response itself, which could eventually result to be unconservative [11]. Another controversy about the spectral matching is about the most significant range of periods to check. In fact, while usually the fundamental period characterizing the dynamic response of the structure is assumed to be the one corresponding to the first vibrational mode, alternative assumptions can be made [12], referring to the “non-linear period” of the structure, defined as the one corresponding to the initial branch of the bilinear idealized capacity curve obtained from the non-linear static (pushover) analysis, according to Eurocode 8, which, in turn, depends on the distribution of lateral loads.

This work is focused on the effects of the ground motions selection on the seismic response of real buildings. According to the Code specification, in fact, different choices can be made about the selection of spectrum-compatible ground motions. Moreover, when a real building is considered for analysis, the soil properties can be not so easy to be classified according to the Code criteria, and consequently the ground motions selection should be even more careful.

According to EC8 and NTC 2008, for soil-types different from the bed rock, the ensemble of ground motions to be used for the analysis can be made *i)* by ground motions selected to be consistent to the Code spectrum for the assumed soil-type, or, alternatively, *ii)* by assuming a set of ground motions consistent with the bed rock Code spectrum and by filtering them according with the specific stratigraphy of the site soil. In the first case, the designer can chose natural ground motions from a standard database, or artificial ones, appositely generated to fit the Code-spectrum for the selected soil-type. In the second case, natural and – when it is possible – local ground motions are usually preferred, and the filtering procedure should provide more reliable results in terms of signal representativeness. The two alternative options, whose choice is up to the engineer in charge of the analysis, can induce very different results in terms of seismic performance, depending on the specific soil-type.

In this work, a comparison among these different approaches, all compatible to the EC8 provisions, has been made on a case-study, i.e. a real irregular RC building currently used as a Hospital. A fully satisfactory knowledge of the building has been achieved; all the documents describing the original design and the foundation soil have been found, and a soil-type B has been assumed for the building, according to the Italian Technical Code NTC 2008.

In the first part of the paper three different selections of ground motions have been described and compared for two different limit states, i.e. a serviceability (Damage Limitation, *DL*) and an ultimate (Significant Damage, *SD*) one, respectively. The adopted ensembles are characterized as follows:

a) A set of 7 artificial accelerograms, all compatible to the elastic spectrum provided by NTC 2008 for the soil-type B, generated by the software SIMQKE [13].
b) A set of 7 real ground motions, taken by the Italian Accelerometric Archive [14] through the adoption of the software REXEL [15,16,17], for a soil-type B.
c) A set of 7 "local" real ground motions chosen through the software SCALCONA 2.0 [18], which selects spectrum-compatible accelerograms by the data-base ASCONA [19] taking into account the position of the case-study area. The elastic spectra have been chosen to fit the NTC 2008 elastic spectrum for a soil-type A; subsequently the time-histories have been filtered by the software Strata [20] on the basis of the specific mechanical properties of the site soil, to obtain a "customized" seismic input compatible with the site.

In the second part of the work the seismic response of the case-study has been found by performing a nonlinear dynamic analysis, for the two considered limit states, for each of the three assumed ground motions ensembles. The results found by the three ensembles, expressed in terms of maximum displacement and interstory drift, have been compared in order to quantify the effects of the different ground motions selection criteria.

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 criterions, like column section reduction from foundation level to the top storey, or solid connection of the beam-column joints, although it is far away from complying the current seismic design criteria.

The dimensions of beams and columns are listed in Table 1, while the reinforcements data can be find in [21,22]. 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.

An accurate investigation, including destructive and non-destructive tests [21,22], has been made to characterize the material; the obtained *mean* and design stresses for concrete and steel, together with their Knowledge Level (*KL*), has been listed in Fig.1.

Soil mechanical properties have been determined through a geophysical site investigation, through seismic refraction and down hole techniques. The results, processed using both Generalized Reciprocal Method [23] and Tomographic one, have provided detailed information on the distribution and thicknesses of subsurface layers, rock dynamics and geo-mechanical properties, seismic *S* waves velocity profiles (see Fig. 2) and the consequential value of the average shear wave velocity $V_{s,30}$ [24], which is equal to 390 m/s. Therefore 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).

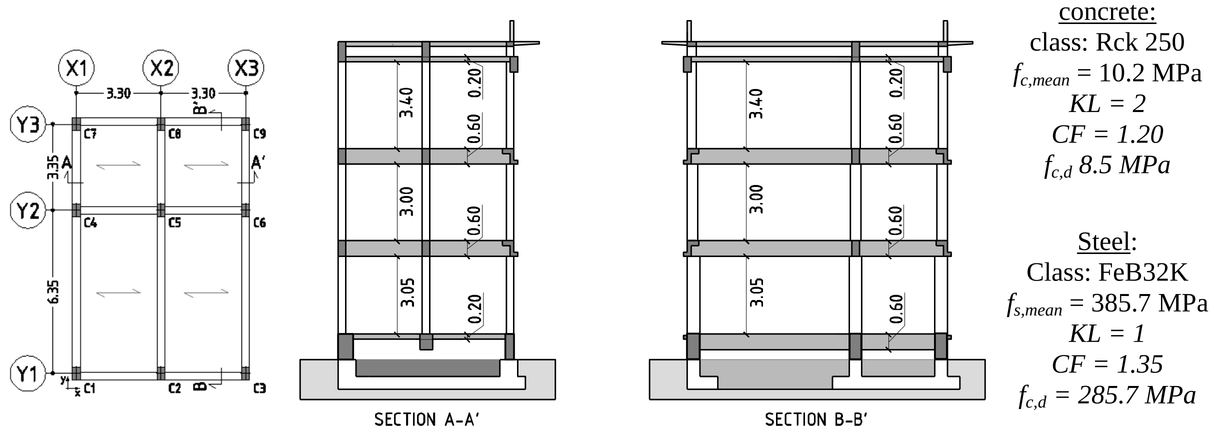


Figure 1. Plan and sections of the RC case-study building.

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	

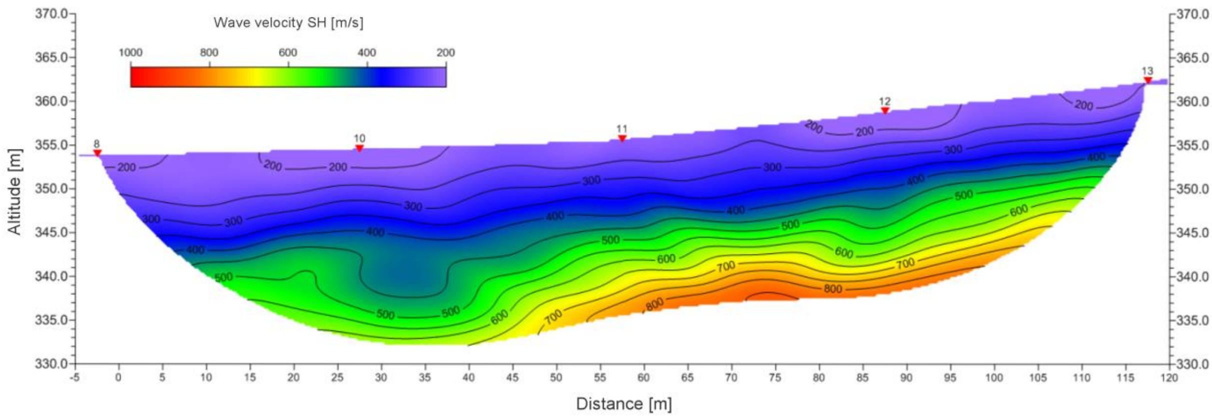
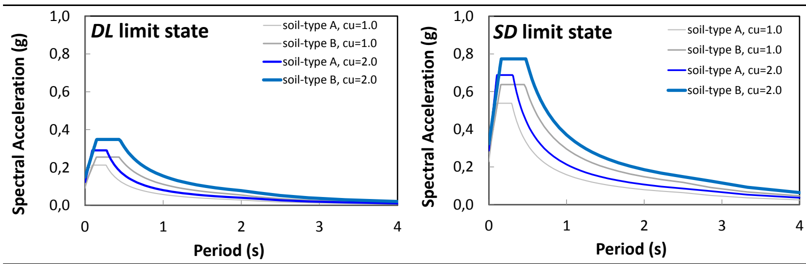


Figure 2. Tomographic section representing the *S* waves profile of the case-study site.

3 THE ASSUMED SEISMIC INPUT

3.1 The elastic spectrum provided by the Code (NTC 2008)

The expected maximum seismic intensity of the area, measured in terms of Peak Ground Acceleration (PGA), has been defined according to NTC 2008. Two Limit States, a serviceability (*DL*) and an ultimate (*SD*) ones, are assumed for analysis. Since the building is currently used as a Hospital, according to NTC 2008, a *coefficient of use* (c_U) equal to 2.0 is assumed, with a consequent doubling of the return period. The main information about the maximum seismic intensity assumed for the two limit states is listed in Tab.2, while the plots of the elastic spectra are shown in Fig. 2.



limit state	*DL*	*SD*
probability of exceedance in 50 years	63%	10%
return period (c_U=1) (in years)	50	475
return period (c_U=2) (in years)	100	950
PGA (g)	0.107	0.227

Figure 2. Elastic spectra provided by NTC 2008.

3.2 Ensemble #1, made of artificial ground motions fitting the B soil-type Code spectrum

A set of 7 artificially generated spectrum-fitting accelerograms, with a duration of 25 s and compatible with the *DL* and *LS* demand spectra respectively provided by the Italian code NTC 2008, have been defined. The time-histories functions have been produced by the software Simqke [13]. The software generates statistically independent artificial ground motions matching a specified response spectrum, through the superposition of sinusoids having random phase angles and amplitudes derived from a stationary power spectral density function. The refinement of the spectral match is done through an iterative procedure. In the present case, ten cycles have been used in the iterative procedure to smoothen the generated response spectra and make them closer to the Code one. The match has been checked in the period range 0-4 s. The generated accelerograms present a stationary part of 10 s, preceded and followed by two branches, increasing and decreasing respectively, of 7.5 s.

Serviceability limit state, *DL* — Ultimate limit state, *SD*

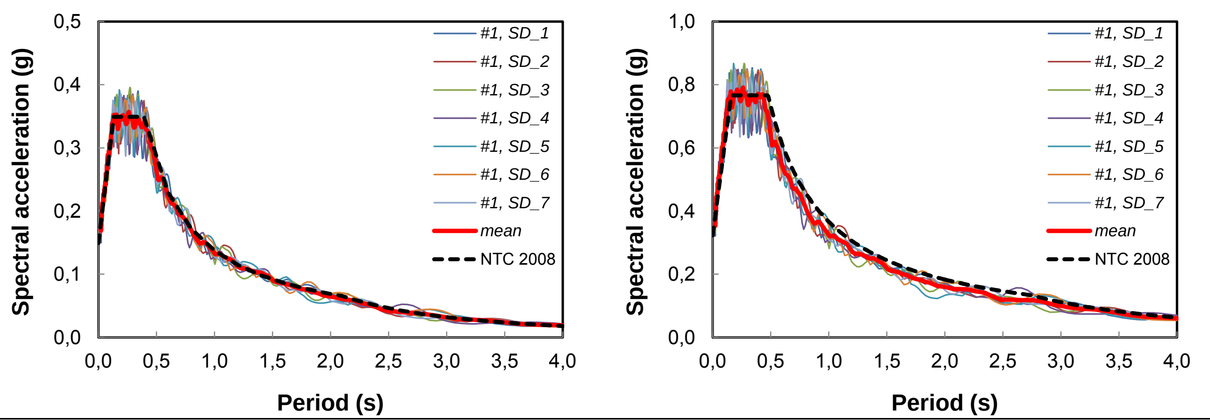


Figure 3. Comparison between the ensemble # 1 elastic spectra and the Code one (soil-type B).

3.3 Ensemble #2, made of natural ground motions fitting the B soil-type Code spectrum

The assumed natural ground motions have been selected, for each limit state, by the Italian Accelerometric Archive [14] through the adoption of the software REXEL [15,16,17], on the basis of the elastic spectrum found for the soil-type B. All ground motions have been used without introducing any scale-factor, i.e. by assuming their effective PGAs. In Figure 4 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.

Table 2. Description of the ground motions of the ensemble #2.

	Code	Event	Station Name	Date YYYY-MM-DD	PGA g
DL limit state	#2, DL_1	Friuli Earthquake 3rd Shock	Forgaria Cornino	1976-09-15	0.215
	#2, DL_2	Irpinia Earthquake	Mercato S. Severino	1980-11-23	0.141
	#2, DL_3	Irpinia Earthquake	Mercato S. Severino	1980-11-23	0.107
	#2, DL_4	Umbria-Marche 3rd Shock	Norcia	1997-10-14	0.095
	#2, DL_5	Irpinia Earthquake	Rionero in Vulture	1980-11-23	0.096
	#2, DL_6	Irpinia Earthquake	Rionero in Vulture	1980-11-23	0.099
	#2, DL_7	Irpinia Earthquake	Sturno	1980-11-23	0.225
SD limit state	#2, SD_1	L'Aquila Mainshock	L'aquila - V. Aterno - Colle Grilli	2009-04-06	0.446
	#2, SD_2	L'Aquila Mainshock	L'aquila - V. Aterno - Colle Grilli	2009-04-06	0.489
	#2, SD_3	L'Aquila Mainshock	L'aquila - V. Aterno - Aquil Park Ing.	2009-04-06	0.354
	#2, SD_4	L'Aquila Mainshock	L'aquila - V. Aterno - Centro Valle	2009-04-06	0.657
	#2, SD_5	Irpinia Earthquake	Sturno	1980-11-23	0.316
	#2, SD_6	Irpinia Earthquake	Sturno	1980-11-23	0.225
	#2,SD_7	Friuli Earthquake 1st Shock	Tolmezzo Centrale - Diga Ambiesta 1	1976-05-06	0.346

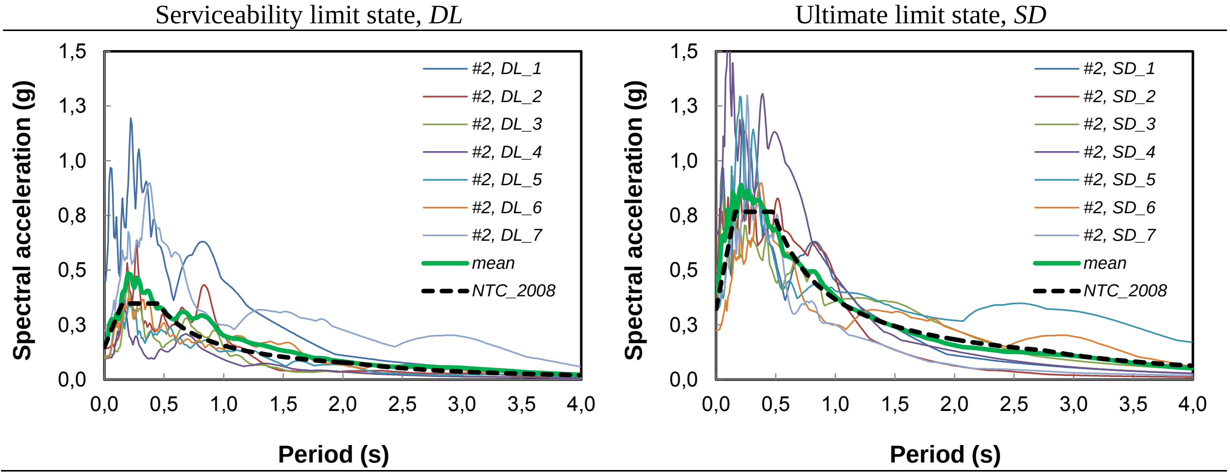


Figure 4. Comparison between the ensemble # 2 elastic spectra and the Code one (soil-type B).

3.4 Ensemble #3, made of natural ground motions fitting the A soil-type Code spectrum

The ensemble #3 is made of seven ground motions for each limit state, fitting the Code spectrum related to the soil-type A (see Fig. 5). The ground motions, whose data are listed in Tab. 3, have been selected through SCALCONA 2.0 [18], which selects natural ground motions, complying to intensity and spectrum-compatibility requirements, taking into account the position (location or geographic coordinates) of the considered site. The time-histories, scaled by two different scale factors to comply the requirements [25], are selected by the database ASCONA [19], which includes ESD (http://www.isesd.hi.is/), PEER-NGA (http://peer.berkeley.edu/nga/) and Itaca [14] databases. The ground motion is applied to the bedrock and the filtering process is performed through the soil, represented as a column of individual layers. Each layer has been characterized according to parameters found by experimental laboratory and *in situ* tests.

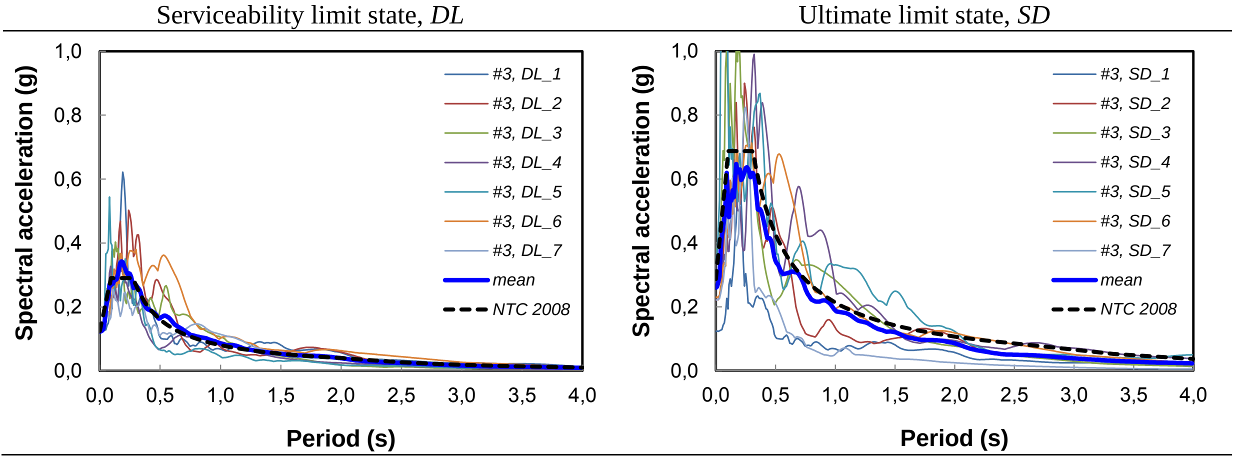


Figure 5. Comparison between the ensemble # 3 elastic spectra and the Code one (soil-type A).

Table 3. Description of the ground motions of the ensemble #3.

	Source file name	Magnitude [Mw]	Epic. Dist. [km]	Total S.F.
DL limit state	ESD 000783ya.cor	5.30	37.00	2.51
	ESD 000234ya.cor	6.20	32.00	1.58
	ESD 000944xa.cor	5.71	37.00	1.48
	NGA 0146y.txt	5.74	12.56	0.92
	NGA 0455x.txt	6.19	38.63	1.78
	ITACA 19971014_152309ITDPC_CSC__WEC.DAT	5.60	22.00	1.94
	ITACA 20090406_013239ITDPC_CLN__NSC.DAT	6.30	31.60	1.35
SD limit state	ESD 000182xa.cor	6.87	11.00	1.16
	ESD 000234ya.cor	6.20	32.00	2.84
	NGA 0146y.txt	5.74	12.56	2.87
	NGA 0804y.txt	6.93	83.53	2.81
	KNET1 SAG0010503201053.NS	6.60	36.18	2.78
	ITACA 19971014_152309ITDPC_CSC__WEC.DAT	5.60	22.00	3.64
	ITACA 20090407_174737ITDPC_AQP__WEC.DAT	5.60	14.40	2.36

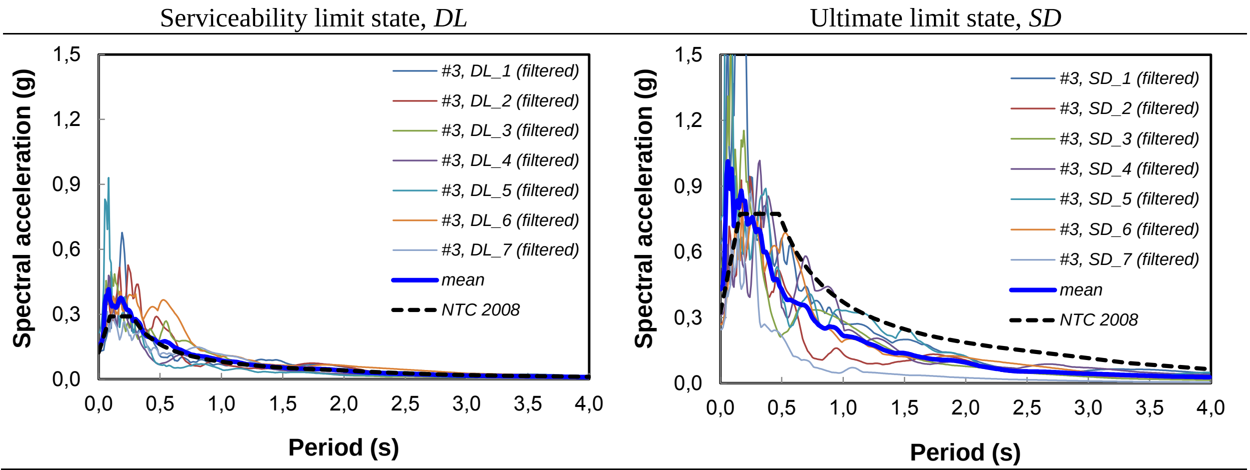


Figure 6. Comparison between the ensemble # 3 elastic spectra and the Code one (soil-type B).

The ground motions constituting the ensemble #3 have been filtered, by assuming the specific soil parameters of the building site, in order to find the expected seismic input as a function of the effective soil properties. The software Strata [20] has been adopted for the filtering phase, by obtaining the elastic spectra shown in Fig. 6.

4 THE STRUCTURAL RESPONSE

All analyses have been performed by using the computer code SAP2000. 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 recommended 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 for all RC elements, with one-directional flexural hinges at each element end. After the attainment of the yielding moment, plastic hinges have an almost constant bending moment (hardening ratio between 2% and 6%) till the ultimate rotation is achieved, with a drop of the bending capacity to 20% of the yielding one. Limit values of bending moment and chord rotation have been made according to NTC 2008, which are identical to those provided by EC8.

The seismic response of the building has been measured in terms of displacement and drift, The displacements have been measured at the mass center of each storey of the building, while the drifts have been checked at each column line. Despite being regular both in plan and in elevation, the building evidences some eccentricity along the two main directions. In fact the column distribution is not symmetric in the *Y*-direction, and the infill panels distribution is irregular in both directions. The in-plan eccentricity of the building, expressed in terms of stiffness [23,24,25], is equal to 10.71% (1^{st} st.), 9.41% (2^{nd} st.) and 7.95% (3^{rd} st.) in the *X*-direction and to 2.23% (1^{st} st.), 5.00 (2^{nd} st.) and 3.17% (3^{rd} st.) in the *Y*-direction respectively.

Figs 7 and 8 show, for the two limit states and the three selected ensembles, both the displacements provided by each single ground motion and their *mean*. In this way both the amount of the seismic response found for each ensemble and its dispersion around the *mean* value can be read. The maximum displacements found by using the ensemble #3 are much lower than the other ones, for both the two limit states and direction. The scatter of the results differs very much for the three considered ensembles. The ensemble #1, which is made by artificial ground motions, has the smallest dispersion of the response domain. On the contrary, the response domain found by the #2, made of real non-scaled ground motions taken by [14], has the larger dispersion, especially in the *DL* limit state.

In Fig. 9 the mean values of the maximum displacements found by the three ensembles are compared for the two limit states, in order to evidence the mutual differences.

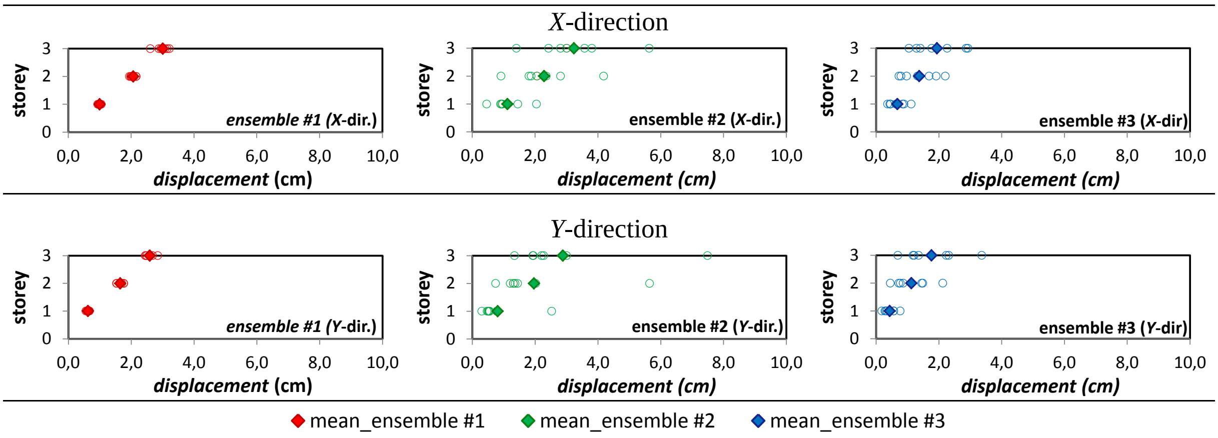


Figure 7. *DL* limit state: displacements profiles provided by the three ensembles.

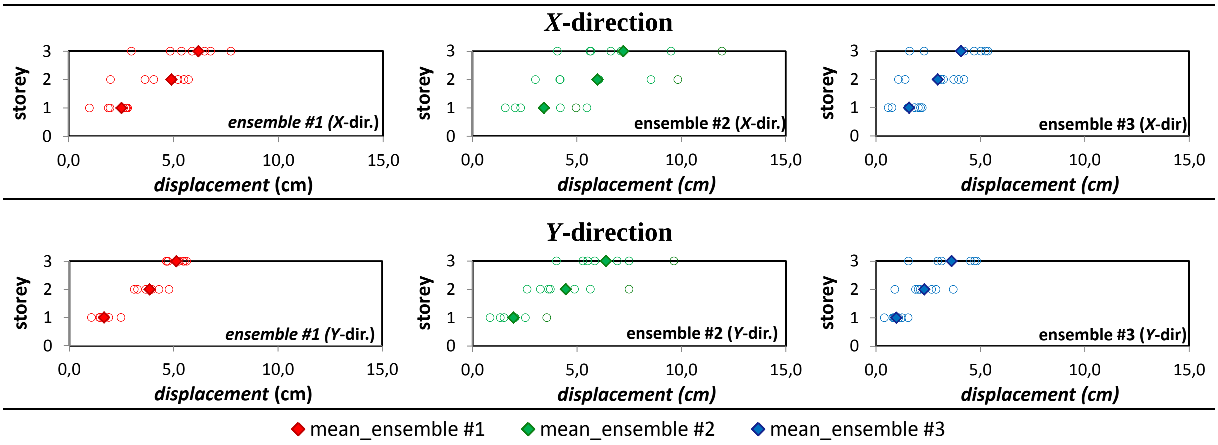


Figure 8. *SD* limit state: displacements profiles provided by the three ensembles.

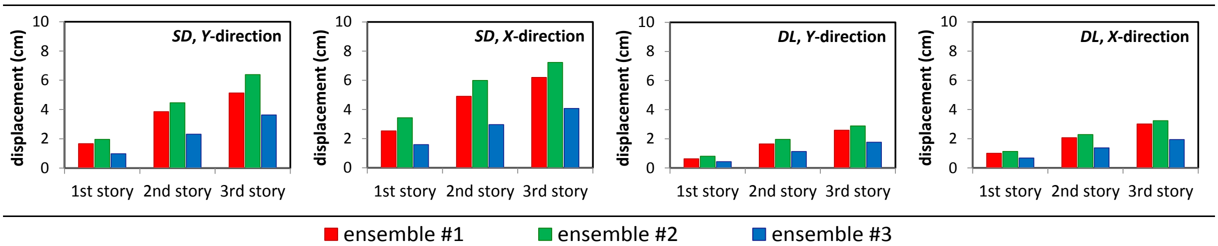


Figure 9. Comparison between the maximum displacement found by the three ensembles.

Figs 10 and 11 show the maximum drift found at each column line of the case-study. The trend found for the drift confirm the displacement one. The ensembles #1 and #2 provide similar results, with the ensemble #2 being the more conservative of the two. The drift found by adopting the ensemble #3, instead, provides the lowest drift, which is even lower than the one provided by the ensemble #1, and up to 54% lower than the one provided by the ensemble #2.

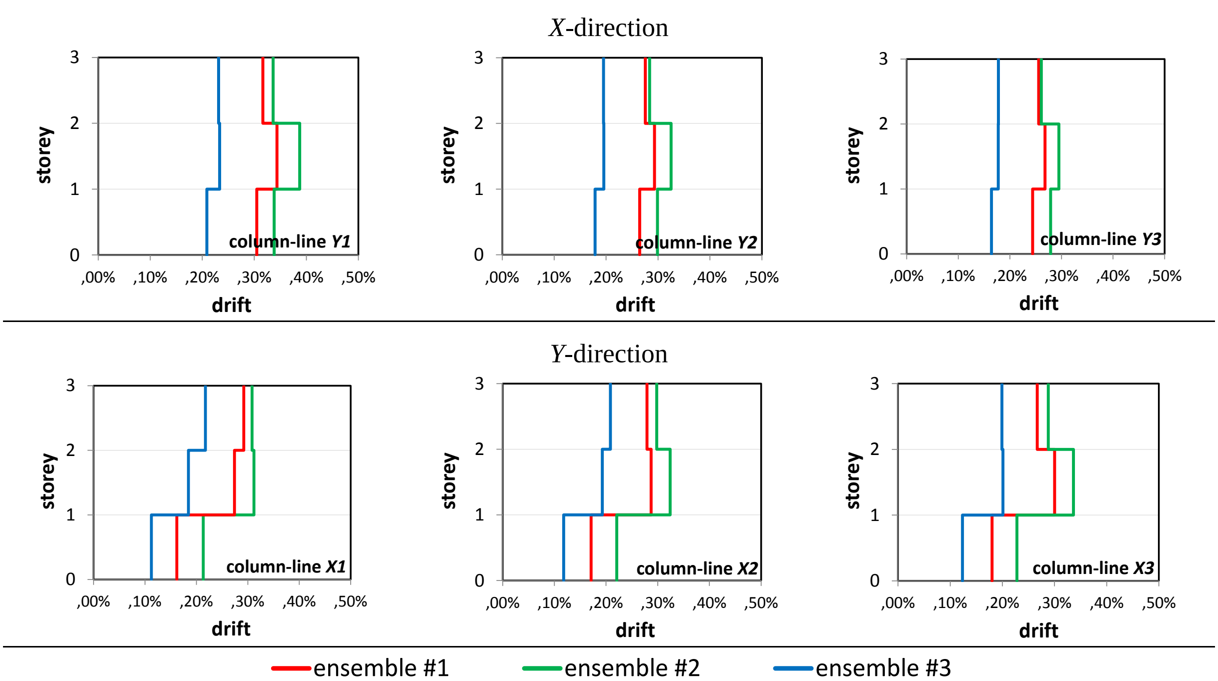


Figure 10. *DL* limit state: maximum drift at each column-line.

The torsional effects due to the eccentricities in the two directions can be evaluated by comparing the maximum drift found at the side column line. As it was expectable, since the only non-negligible (> 5%) eccentricity is along the *Y*-direction, the torsional effects are larger when the seismic analysis in is performed the *X*-direction. In the results found for both the limit states, in fact, the drifts experienced by the structure at the column line *Y1* are larger than the ones at the column line *Y3*. The effects of the different ground motions assumed as seismic input can be read even in the drift profile, especially when the Significant Damage limit state is considered.

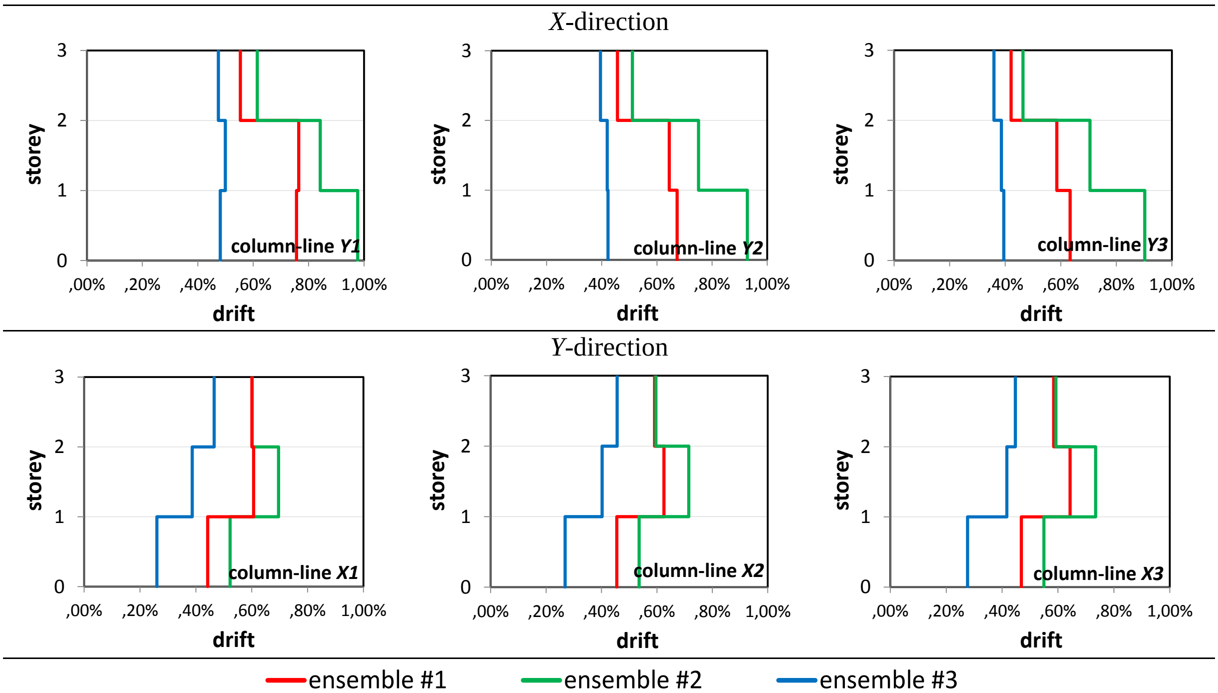


Figure 11. *SD* limit state: maximum drift at each column-line.

5 CONCLUSIONS

In this work the effects of the selection of ground motions to be used as seismic input for nonlinear dynamic analysis have been studied on a case-study, i.e. an existing irregular RC building. A very detailed knowledge has been achieved on the building, including both structural and geological aspects. The site ground have been extensively described, and a soil-type B has been consequently assumed. Three different ensembles of ground motions, all compatible to the Italian Technical Code (NTC 2008), have been selected and compared. Two of them, respectively made of artificial ground motions (#1) and natural ones selected by a National archive (#2), have been set directly on the elastic spectrum provided by the Code for the soil-type B, while the third one (#3), made of natural ground motions taken by a regional database, has been set on the bed rock soil, and then filtered on the base of the specific ground properties.

The three selected ensembles differ very much each other both in terms of dispersion and spectral ordinates. The ensemble #1, in fact, can be generated by imposing different amounts of scatter, all compatible to the Code requirements. The ensemble adopted in the analysis presents a much lower scatter than the other ones. Its spectral ordinates are close enough to the ones of the ensemble #2, which is characterized by a very high scatter. The ensemble #3, instead, differ very much from the other two in terms of spectral ordinates, providing lower values of Spectral Acceleration. These differences among the three considered ensembles can be found even when the seismic response of the building has been checked. Both the results in terms of maximum displacement and drift, in fact, confirm these observations, as regards both the amount of the response and the dispersion of the response domain.

According to this work, the three assumed ensembles of ground motions, despite being consistent to the provisions of the current Technical Code, provide very different results. Further investigations are needed to understand which selection of ground motions can be more appropriate as a function of the soil-type and of the purpose of the analysis.

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