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Seismic assessment of existing RC buildings under alternative ground motion ensembles compatible to EC8 and NTC 2008

Marco Tanganelli[1], Stefania Viti[1], Valentina Mariani[2], Maria Pianigiani[1]

[1] *Department of Architecture (DiDA), University of Florence, Italy*
[2] *Department of Structural Engineering Delft University of Technology, Delft, Netherlands*

Summary

This work investigates the effects of the choice of different ensembles of ground motions on the seismic assessment of existing RC buildings through nonlinear dynamic analysis. Nowadays 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 as target, defined on the basis of average properties assumed for the soil. An ensemble of ground motions, compatible to the target one, must be selected to perform a nonlinear dynamic analysis. The ensemble can be made by artificial or natural ground motions, compatible with the Code spectrum for the assumed soil-type. Alternatively, the set of ground motions can be assumed as compatible with the bedrock Code spectrum and, subsequently, subjected to site response analysis, i.e. filtered through 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 RC Italian building. The seismic response of the case-study under the assumed seismic inputs, expressed in terms of chord rotation and shear force, has been found by performing a nonlinear dynamic analysis under the different assumed seismic excitations. The comparison has been made in terms of seismic performance, expressed as the ratio between the seismic response found for each structural element and the corresponding capacity. The comparison among the seismic performance found by the application of the different ground motion ensembles pointed out significant differences, which underline the importance of the seismic input choice in the seismic assessment of RC buildings.

1. INTRODUCTION

The earthquake is one of the most intense action that a building may have to face during its service life and it is characterized by being a non-deterministic event, that is why seismic design and assessment of structures necessarily involves a probabilistic approach (Kappos 2002, Porter et al. 2002; Lee and Mosalam 2005, Faggella et al. 2013). Of course, it is commonly accepted that the properties of ground motions which will occur in a certain area can not be exactly foreseen. In the past years a remarkable effort has been devoted to limit the inevitable uncertainty related to seismic phenomena, through a detailed mapping of the seismic prone areas, aimed at characterizing the seismic hazard (Gupta 2002) and at ensuring the most reliable prediction of the seismic action. Once an exhaustive knowledge of the site seismic risk is achieved, the seismic assessment of buildings by means of nonlinear dynamic analysis requires to choose, or create, a proper ensemble of ground motions, on the basis of the available information.
In these years many contributions have been dedicated to the selection of ground motions to use for analysis. A comprehensive classification of the records databases has been made after each of the parameters of utmost importance, like the Magnitude (Shome et al. 1998), the distance from the rupture zone (Stewart et al. 2001) and the soil profile (Bommer and Acevedo 2004) through a disaggregation approach (Bazzurro and Cornell 1999). Once the role of each parameter was known, the problem of how to combine the statistical information of each of them had to be faced. To this purpose, different procedures for Ground Motions Selection and Modification (GMSM) have been developed (Katsanos 2010, Tarbali and Bradley 2015, Iervolino et al. 2011, Al Atik and

Abrahmson 2010, Seifried and Baker 2014, Bradley 2010), based on Seismic Hazard Analysis (Bazzurro and Cornell 1999, Baker 2011), which attributes a multivariate distribution to the considered classification accounting for the marginal probability of the optimization function. Depending on the assumed hypotheses about the seismic hazard, different target spectra can be defined (Lin et al. 2013a, 2013b).

Most of the International Technical Codes, as Eurocode 8 (EC8), ASCE standards 7-05 (ASCE 2006) and 4-98 (ASCE 2000), as well as the Italian NTC 2008, provide uniform hazard spectra, which are made of spectral accelerations with equal probabilities of exceedance at all periods. The soil classification is usually based on the uppermost 30 m shear-wave velocity ($v_{s,30}$) of the site. Therefore, the ground motions to be assumed in order to perform a nonlinear dynamic analysis are selected to be spectrum-compatible with the Code spectrum (Gascot et al. 2014, Amara et al. 2014, Dhakal et al. 2013). The prescriptions of the main seismic Codes are not identical, in terms of period range for spectrum-fitting, number of ground motions to be assumed in the analysis (D'Ambrisi et al. 2009, 2014), and possibility to alternatively use scaled, spectrum-matched or artificial ground motions. EC8, which almost coincides with the Italian NTC 2008, admits the adoption of both natural and artificial ground motions, and fixes at 7 the minimum number of ground motions in order to be allowed to assume the *mean* response as reference value for the seismic design/assessment. Many contributions (Iervolino et al. 2008, Baker 2011) have been given in these years as regards the selection of ground motion ensembles spectrum-compatible with the Codes ones. Iervolino et al. (2006, 2008) focused their attention on the elastic spectra provided by NTC2008 for the different soil classes, providing suitable criteria and a proper software (Iervolino et al. 2009) to select un-scaled natural ground motions whose spectra closely fit the Code ones. For very soft soils (classes D and E according to NTC 2008 classification), as well as at the occurring of specific phenomena, e.g. liquefaction (Know at al. 2008), the records selection is not so easy to do. Most part of the available ground motions, in fact, refers to soil categories A, B and C. Moreover, some of these classes, like the B one, covers a large range of $v_{s,30}$ values, possibly leading to a record selection not so consistent with the soil features at the specific site. In these cases, therefore, an alternative solution is to select the records with reference to the bedrock (i.e. compatible with soil-A spectrum), and to perform a Site Response Analysis (SRA), i.e. a filtering procedure through the soil layers from the bedrock to the building foundation.

In this wide framework, this work is aimed at investigating the effects of the assumption of different ground motions ensembles, all consistent with the NTC 2008 prescriptions, on the seismic performance of a case-study.

The ground motions have been selected to be spectrum-compatible, respectively, *i)* with the soil-type of the case study, and *ii)* with its bedrock (by performing a further SRA through the soil stratigraphy). In the first case, two different ensembles have been considered, respectively made of natural ground motions and artificial accelerograms.

The three record ensembles, all complying with the NTC 2008 provisions, have been compared and applied to the seismic assessment of a case-study, i.e. a real irregular RC building currently used as a hospital, built on class B soil. EC8 specifies that ground classification scheme and shape of the elastic spectra can be defined according to National Annexes. Since the Italian territory is provided with a detailed seismic hazard map (NTC 2008, Ord. PCM 3274, Ord. PCM 3529), the elastic spectrum assumed for the record selection is the one provided by the Italian Code NTC 2008.

The adopted ensembles are characterized as follows:

a) a set of 7 artificial accelerograms, all compatible with the elastic spectrum provided by NTC 2008 for the soil-type B, generated by the software SIMQKE (Gasparini and Vanmarcke 1976) referred as "ensemble#1" in the following;
b) a set of 7 natural ground motions, selected within the Italian Accelerometric Archive (ITACA 2008) through the adoption of the software REXEL (Iervolino et al. 2009, Smerzini e al. 2014), for a soil-type B (referred as "ensemble#2" in the following);
c) a set of 7 natural ground motions chosen through the software SCALCONA 2.0 (Zuccolo et al. 2014), which selects spectrum-compatible records by the data-base

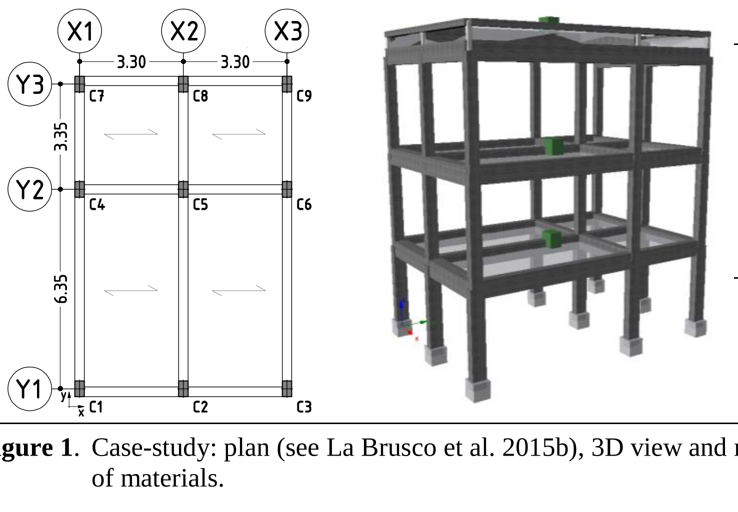
ASCONA (Corigliano et al., 2012) taking into account the position of the case-study area. The records have been selected to fit the NTC 2008 elastic spectrum for a soil-type A; subsequently the time-histories have been subjected to a SRA, by filtering the signals through the software Strata (Kottke and Rathje 2008) on the basis of the specific mechanical properties of the site soil, so as to obtain a “customized” seismic input compatible with the site (referred as “ensemble#3” in the following).

This work follows on a previous research, made by the Authors on the same case-study (Pianigiani et al. 2015), which had compared the seismic response of the building under alternative selections of seismic input, partially differing from the current ones. In this work, a more refined numerical model has been assumed to represent the seismic response of the building, and a different ensemble of artificial accelerograms, having a larger scatter, has been adopted as “ensemble #1”. Moreover, the capacity of each member has been found, according to EC8 prescriptions, and the seismic performance of the building has been consequently checked by the ratio D/C between demand (D) and capacity (C), with reference to two limit states, i.e. a serviceability (Damage Limitation, *DL*) and an ultimate (Significant Damage, *SD*) ones. The assumed response quantities are the chord rotation and the shear force for the *SD* limit state and the columns chord rotation only for the *DL* limit state. The response corresponding to each ground motion ensemble has been assumed as the average of the maximum responses associated to each ground motion composing the ensamble, as prescribed by NTC 2008.
The comparison among the seismic performance found by the application of the three different ground motion ensembles pointed out significant differences, which underline the importance of the seismic input choice in the seismic assessment of RC buildings.

2. THE ANALYSIS

2.1 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.



Concrete (class Rck 250)	
$f_{c,mean}$ (MPa)	10.2
$f_{c,d}$ (MPa)	8.5
KL	2
CF	1.20

Steel (class FeB32K)	
$f_{s,mean}$ (MPa)	385.7
$f_{s,d}$ (MPa)	285.7
KL	1
CF	1.35

Figure 1. Case-study: plan (see La Brusco et al. 2015b), 3D view and main mechanical properties of materials.

The dimensions of beams and columns are listed in Table 1, while the reinforcements data and further information about the structural regularity can be found in La Brusco et al. (2015). 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 (La Brusco et al. 2015), has been made to characterize the materials; the obtained *mean* and design strengths for concrete and steel, together with their Knowledge Level (*KL*) and Confidence Factor (*CF*), defined according to EC8 prescriptions, have been listed in Fig.1.

Table 1. Cross section dimensions 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	

2.2 The soil properties

Soil mechanical properties have been determined through a geophysical site investigation, using seismic refraction. The results, processed with both Generalized Reciprocal Method (Palmer 1981) and Tomographic one, have provided detailed information on the distribution and thicknesses of subsurface layers (see Fig. 2b), rock dynamics and geo-mechanical properties. Seismic *P* and *S* waves velocity profiles have been found and consequently the shear wave velocity v_s (Sirles and Viksne 1990) has been determined.

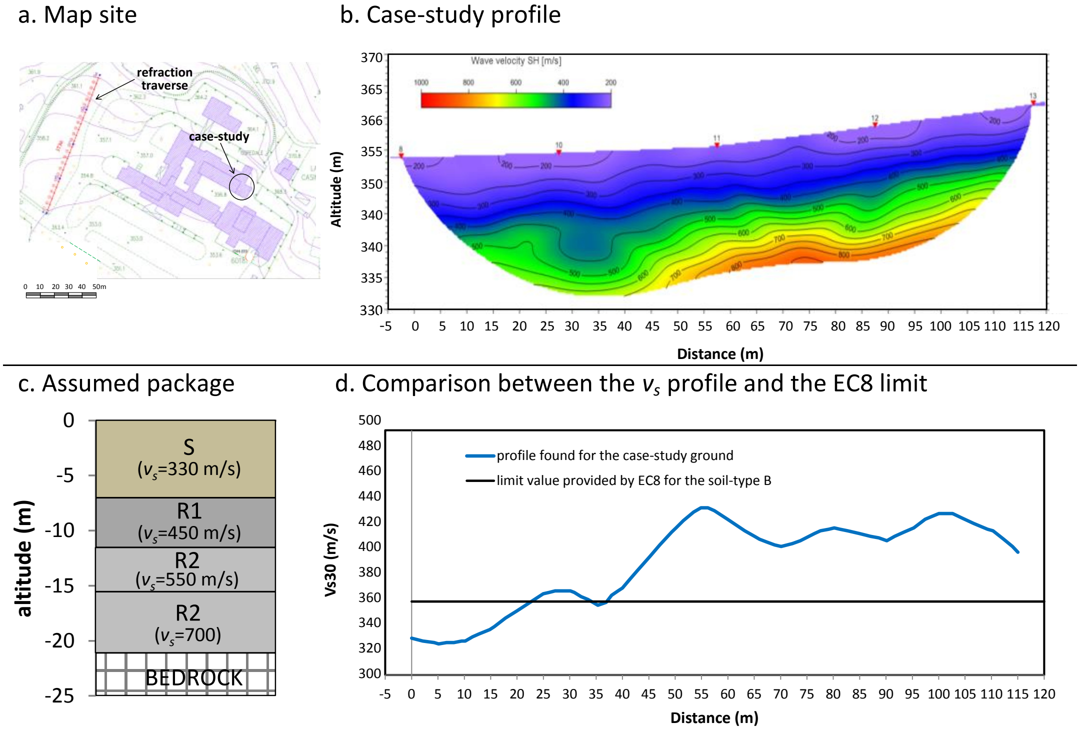


Figure 2. Soil profile of the case-study.

Both EC8 (Part 1, sec. 3.1.2, Tab. 3.1) and NTC 2008 (Sec. 3.2.2, Tab. 3.2.II) provide a classification of the soil (see Table 2) based on $v_{s,30}$, i.e. the average shear wave velocity over the first 30 m of soil in depth (EC8. SEC. 3.1.2, eq. 3.1). The $v_{s,30}$ of the case-study soil has been found over 21 m only, since such quote corresponds to the beginning of the bedrock.

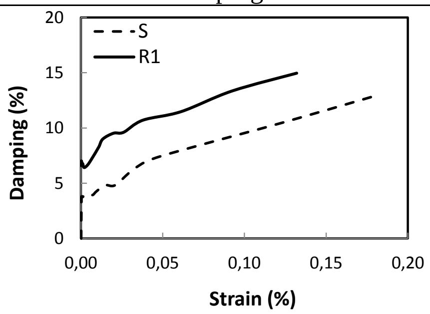
Table2. NTC (2008) soil classification.

Ground type	Description of stratigraphic profile	$v_{s,30}$
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800
B	Deposits of very dense sand, gravel, or very stiff clay, at least 30 meters in thickness, characterized by a gradual increase of mechanical properties with depth.	360-800
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from 30 meters	180-360
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers) with thickness from 30 m, or of predominantly soft-to-firm cohesive soil	<180
E	Surface alluvium layer with vs values of type C or D and a maximum thickness of 20 m, underlain by stiffer material with v_s > 800 m/s.	

Figure 2 shows the main data collected through the geophysical investigation, that allow a proper characterization of the site soil. Figure 2d shows the obtained values of $v_{s,30}$, which range between 320 and 450 m/s, with a *mean* value equal to 390 m/s, that would lead to a soil type B, according to the $v_{s,30}$ ranges provided by both EC8 and NTC 2008, the latter ones reported in Table 2. Moreover, the case-study building is located in the central part of the investigated area (see Figure 2a), where the $v_{s,30}$ values are higher. It should be noted that NTC 2008 would provide a minimum thickness of 30 m on which the $v_{s,30}$ should be evaluated, whereas EC8 prescribes a surface dense soil layer "several tens of meters" tick. Since no other soil class of NTC 2008 resulted representative of the site soil, the class B, complying with EC8, has been evaluated to be the most appropriate to describe the subsoil stratigraphy. The controversial definition of the soil category makes the use of SRA very suitable in this case, giving the possibility to take into account the real site-specific soil layers.

Figure 2c shows the assumed soil profile, characterized by three different soil layers, above the bedrock (*BR*): silt (*S*) and two types of softer rock (*R1*, *R2*). Each layer has been characterized by proper values of normalized shear modulus (G/G_{max} ratio) and percentage of damping. *BR* and *R2* have been characterized according to the data provided by Idriss and Sun (1992) and Seed and Idriss (1970), since *BR* is the standard rock soil, whilst *R2* is very similar to the Morello Mountain ground. The other two soil types, *S* and *R1*, instead, have been characterized after proper laboratory investigations made on some extracted samples. The obtained values of damping and G/G_{max} ratio have been shown in Figure 3.

damping



G/G$_{max}$

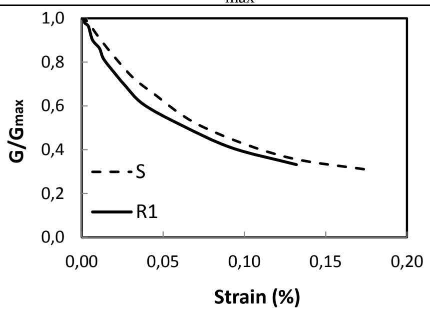


Figure 3. Assumed data for soil characterization.

2.3 The finite element model

All analyses have been performed by using the computer code Seismostruct (Seismosoft 2006). The cross sections of each structural member has been represented through a fiber model (200 fibers for cross section), based on the assumed constitutive relationship of the materials, i.e. the Mander et al. (1988) model for the concrete - with a confinement coefficient assumed accordingly to the effective stirrups size and spacing - and a bilinear model, with a hardening ratio equal to 5%, for the reinforcement steel. All members have been divided in four branches and represented through an inelastic model (infrmFB), by adopting a force-based approach with 6 control points.

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 assumed as 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. The floor stiffness has been introduced by assigning the diaphragm constraint to all nodes belonging to the same floor.

The first vibrational period of the building, T_1, corresponding to a translational motion along the X-direction, is equal to 0.677 s. The second period (T_2 = 0.533 s) corresponds to a translational motion along the Y-direction, while the third one (T_3 = 0.237 s) is again a translational mode in the X-direction.

2.4 The assumed limit states

Two different limit states have been assumed in the analysis, the *DL* and the *SD*. The ultimate capacity of each member has been expressed in terms of chord rotation and shear force, according to EC8 provisions. More precisely, the limit chord rotation assumed for the *SD* limit state, θ_{SD}, has been defined as 2/3 of the ultimate chord rotation, θ_u (EC8-3, Annex A eq. A1) while the ultimate shear force, V_u has been quantified by the equation A.12 of Annex A (EC8-3). Conversely, for the *DL* limit state, the only condition assumed for verification is the column chord rotation (assumed equal to the interstorey drift), to be below the yield rotation θ_y (EC8-3, Annex A, eq. A5).

3. THE SEISMIC INPUT

Three different ensembles of ground motions, all complying with the NTC 2008 requirements, have been selected to represent the seismic input. The ensemble #1 is made of artificial ground motions. This choice, simple to be pursued, is the rougher one: the accelerograms are generated without taking into account any site-specific feature, except the mean spectrum and the magnitude; the frequency content is usually – as well as in the current analysis – completely neglected. Moreover, the scatter in amplitude of the acceleration histories can be arbitrary chosen and, in case, contained. The ensemble #2 is made of real ground motions. Usually this criterion of selection assures to the ensemble a larger variety under all the above mentioned aspects. In the selection, the unscaled and – whenever it is possible – local accelerograms are usually preferred. The ensemble #3, made of real ground motions selected to be compatible with the seismic input at the bedrock and filtered through the local layout of soil layers, represents the most advanced selection criterion. The choice of filtering the ground motions through the real soil, indeed, assures a better representativeness in terms of dynamic contents, leading to a more "customized" seismic input. In the following sections, each ensemble is presented and compared to the NTC 2008 spectra and a comparison among the three selected approaches is presented.

3.1 The elastic spectra 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 for an assigned probability of occurrence of a seismic event in a 50-years period, which is the assumed nominal life (*NL*) of a standard building, and it is equal to 0.107g and 0.227g, respectively for the *DL* and *SD* limit states (see Tab. 3).

Table 3. Seismic intensities provided by NTC 2008 for the two assumed limit states.

limit state	prob. of exceedance in 50 years	T_R (c_U=1)	T_R (c_U=2)	PGA (*NL*=50 years)	PGA (*NL*=100 years)
	(%)	years	years	(g)	(g)
DL	63%	50	101	0.107	0.124
SD	10%	475	949	0.227	0.286

Table 4. Assumed parameters to set the elastic NTC 2008 spectra (soil-type B, c_U=2.0).

l.s.	T_R	a_g	*Fo*	*T*c*	S_T	S_S	*Cc*	T_C	T_B	T_D
DL	101	0,124	2,338	0,280	1	1,200	1,4189	0,3973	0,1324	2,096
SD	949	0,286	2,375	0,309	1	1,128	1,3912	0,4299	0,1433	2,744

T_R = return period, a_g = ground acceleration, F_0 = amplification factor on the rock-site, T^*_C = beginning period of the velocity-constant branch, S_T = topographic amplification factor, S_S = stratigraphic amplification factor, *Cc* = soil-type amplification, T_C = period at the beginning of the velocity-constant branch for the assumed soil-type, T_B = period at the beginning of the constant-acceleration branch, T_D = period at the beginning of the displacement constant branch.

For each limit state, an elastic spectrum is provided as a function of the assumed soil-type, the topographic properties of the soil and the class of use of the building. On the base of the geological data (see Section 2.2), a soil-type B has been assumed. Moreover, since the case-study is currently used as a hospital, a *coefficient of use* c_U equal to 2.0 is assumed, (see NTC 2008, Section 2.4.3), with a consequent doubling of *NL*, and, in turn, of the return period (T_R).
In Tab. 3 the seismic intensities provided by NTC 2008 for the two assumed limit states have been shown, while Table 4 summarizes all the necessary parameters to set the two target elastic spectra.
The elastic spectra obtained for the two limit states can be found in the next sections, compared to the spectra of the assumed ensembles of ground motions.

3.2 Ensemble #1: artificial ground motions fitting the B soil-type Code spectrum

The ensemble of each limit state is made of 7 accelerograms having a duration of 25 s, which have been artificially generated by the software Simqke (Gasparini et al. 1976).
The software generates statistically independent artificial ground motions, whose *mean* spectrum is compatible with the target 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 ones. The match has been checked in the period range 0-4 s, within which all the relevant frequencies of the building are included. The generated accelerograms present a stationary part of 10 s, preceded and followed by two branches of 7.5 s, increasing and decreasing respectively, so as to match the total duration of 25 s prescribed by NTC 2008.
In Figure 4 the comparison between the spectra of the assumed ground motion ensemble and the corresponding elastic spectra provided by NTC 2008 for the soil-type B has been shown.

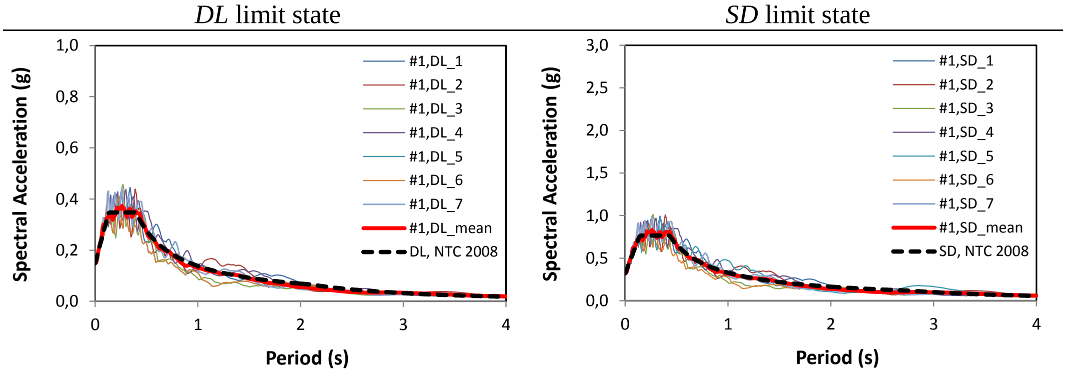


Figure 4. Comparison between the ensemble #1 and the elastic spectra provided by NTC 2008 (soil-type B).

3.3 Ensemble #2: natural ground motions fitting the B soil-type Code spectrum

The ensembles #2 is made of natural ground motions; they have been selected within the Italian Accelerometric Archive (Itaca 2008) through the adoption of the software REXEL (Iervolino et al. 2009; Smerzini e al. 2014), on the basis of the elastic spectrum related to the soil-type B. All ground motions have been used without introducing any scale-factor, i.e. by assuming their effective PGAs.

In Table 5 the main data of the ground motions, selected to represent the seismic input of the two limit states, have been listed, while in Figure 5 their elastic spectra have been shown and compared to the ones provided by NTC 2008.

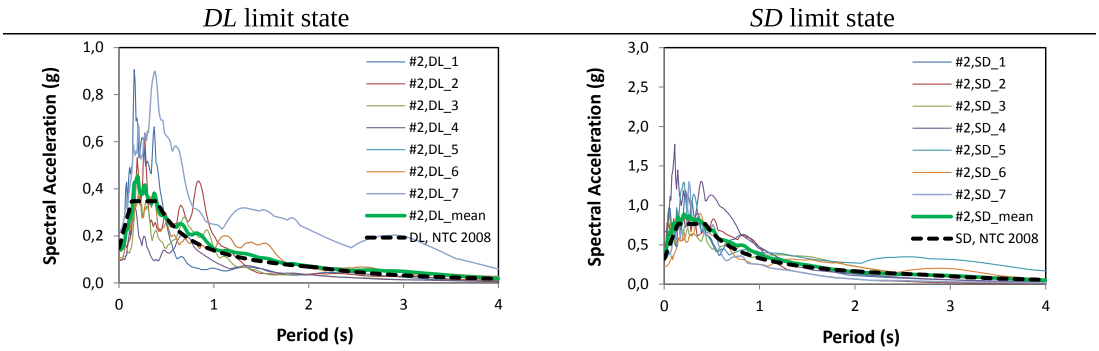


Figure 5. Comparison between the ensemble #2 and the elastic spectra provided by NTC 2008 (soil-type B).

Table 5. Baseline data of the ground motions of the ensemble #2.

	Code	Event	Date YYYY-MM-DD	PGA g
DL limit state	SLD_FRC_HNE	Friuli Earthquake 3rd Shock	1976-09-15	0.215
	SLD_MRT_HNE	Irpinia Earthquake	1980-11-23	0.141
	SLD_MRT_HNN	Irpinia Earthquake	1980-11-23	0.107
	SLD_NRC_HNN	Umbria-Marche 3rd Shock	1997-10-14	0.095
	SLD_RNR_HNE	Irpinia Earthquake	1980-11-23	0.096
	SLD_RNR_HNN	Irpinia Earthquake	1980-11-23	0.099
	SLD_STR_HNN	Irpinia Earthquake	1980-11-23	0.225
SD limit state	SLV_AQG_HNE	L'Aquila Mainshock	2009-04-06	0.446
	SLV_AQG_HNN	L'Aquila Mainshock	2009-04-06	0.489
	SLV_AQK_HNN	L'Aquila Mainshock	2009-04-06	0.354
	SLV_AQV_HNE	L'Aquila Mainshock	2009-04-06	0.657
	SLV_STR_HNE	Irpinia Earthquake	1980-11-23	0.316
	SLV_STR_HNN	Irpinia Earthquake	1980-11-23	0.225
	SLV_TLM1_HNN	Friuli Earthquake 1st Shock	1976-05-06	0.346

3.4 Ensemble #3: natural ground motions fitting the A soil-type Code spectrum and filtered through the site soil profile

The ensemble #3 is made of seven local ground motions selected to fit the seismic input at the bedrock (soil-type A) and filtered through the real site stratigraphy, according to the specific geological features. The ground motions, listed in Tab. 6, have been chosen through SCALCONA 2.0 (Zuccolo et al. 2014), which selects natural ground motions, complying with intensity and spectrum-compatibility requirements, taking into account the position (location or geographic coordinates) of the building site. The time-histories, scaled by two different scale factors to comply with the requirements (Zuccolo et al. 2013), are selected by the database ASCONA (Corigliano et al. 2012), which includes ESD (http://www.isesd.hi.is/), Itaca (Itaca, 2008), and PEER-NGA (http://peer.berkeley.edu/nga/) databases.

The comparison between the spectra of the selected ground motions and the elastic spectrum provided by NTC 2008 for the soil-type A is shown in Figure 6. The ensemble of ground motions assumed to be representative of the case-study site has been found by applying each ground motion at the bedrock level and filtering the dynamic response through the specific soil stratigraphy, modelled as a column of individual layers, whose mechanical properties have been described in Section 2.2.

Figure 7 shows the obtained elastic spectra of the records belonging to the ensemble #3 compared to the one provided by NTC 2008 for the soil-type B. As can be noted, the *mean* spectrum is much higher than the Code one for low periods (around 0.2 sec), while it is lower than the Code one for larger periods.

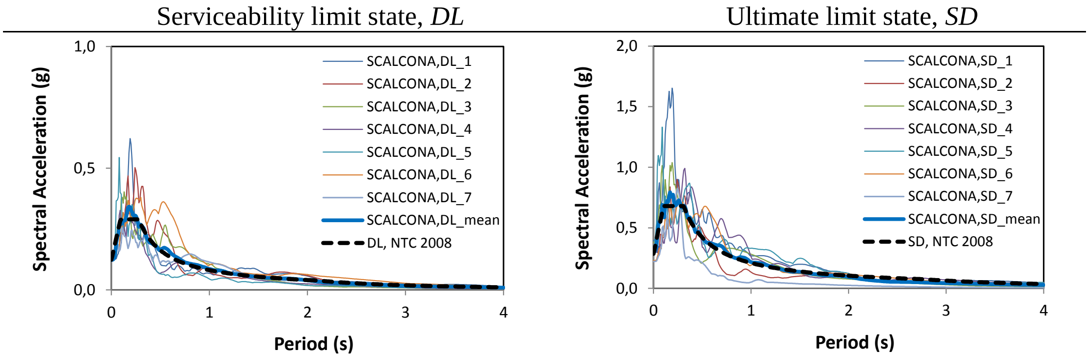


Figure 6. Comparison between the SCALCONA ensemble and the elastic spectra provided by NTC 2008 (soil-type A).

Table 6. Baseline data of the ground motions of the ensemble #3.

	Source file name	Magnitude [Mw]	Epic. Dist. [km]	Total scale factor
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

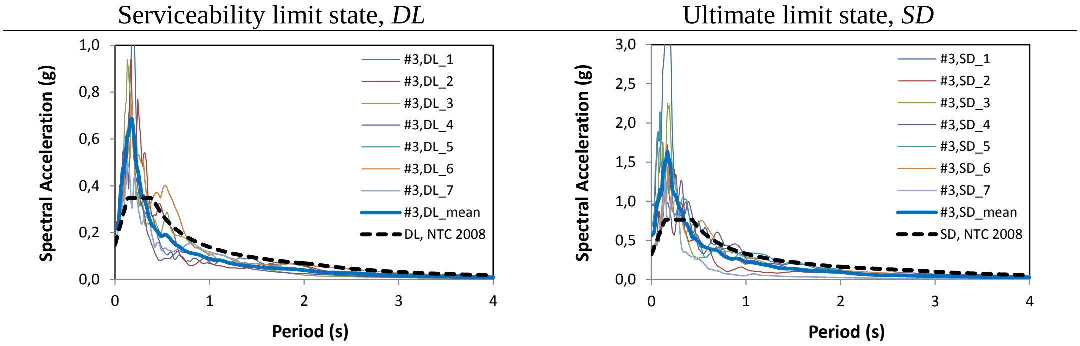


Figure 7. Comparison between the ensemble #3 and the elastic spectra provided by NTC 2008 (soil-type B).

3.5 Comparison of the three assumed ensembles

In Figure 8 the *mean* elastic spectra of the three assumed ensembles for the two limit states have been compared. It can be observed that, while the ensembles #1 and #2 spectra are very close to each other, the one of the ensemble #3 presents a higher peak for low periods, while it is much lower for increasing periods. At this regard, it should be reminded that all the ensembles, despite presenting very different scattering, are all valid and consistent with code requirements, therefore each of them can be assumed for analysis. No prescriptions are given neither by NTC 2008 and EC8 on the scattering of the ensembles and no mandatory need to scale the PGA of the accelerograms is given in the NTC 2008, provided that compatibility in the significant range of periods is assured. With regard to this latter requirement, the gap between the spectra of each of the assumed ensemble and the Code ones have been shown in Figure 9 as a function of the period. For ensembles #1 and #2 the gap has been evaluated with reference to the soil-B elastic spectrum. For ensemble #3 the comparison has been made between the average spectrum of the Scalcona set of accelerograms and the soil-A elastic spectrum, since the choice of accelerograms has been done with reference to the bedrock and subsequently filtered through the real site soil layers. This procedure led to a higher deviation of the average spectrum of ensemble #3 with respect to the soil-B elastic spectrum (see dashed blue line in Figure 9), highlighting how the definition of the soil category for this building is quite controversial, as described in Section 2.2.

In Figure 9 the first two periods of the case study have been evidenced in each graph; despite the ensembles #1 and #2 are very close to the Code spectra for the range of periods predominant of the structure, the ensemble #2 progressively drifts apart from the Code spectrum for higher periods. When the structure undergoes the inelastic range, its vibrational period necessary increases and the differences between the ensembles #1 and #2 possibly becomes more relevant. The mean spectra of ensembles #3, instead, present a gap of about 40% comparing to the Code ones for the range of periods within which is included most of the dynamic response of the structure.

Another important difference among the assumed ensembles is related to their scattering. Figure 10 shows the Coefficient of Variation (*CoV*) of each ensemble as a function of the period. As can be seen, for the period range within which most of the dynamic response of the structure is included, the ensemble #2 and #3 have similar *CoV*s, while the ensemble #1 presents a lower scattering. The level of scattering of the ensemble is important especially when inelastic analyses are performed. Due to nonlinear effects, in fact, the *mean* response of very scattered ground motion ensembles is likely to be larger to the *mean* response of more gathered ones. The comparison among the three selected ensemble has been made by considering their capability to reproduce the spectral shape provided by NTC 2008.

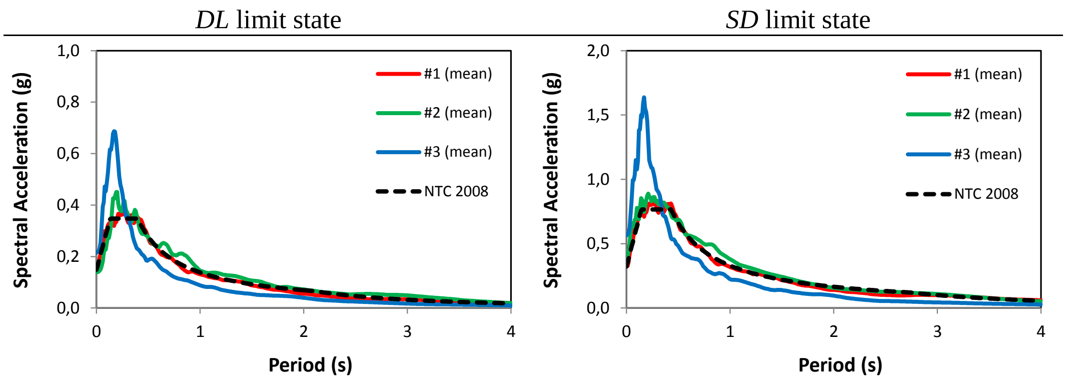


Figure 8. Comparison among the three ensembles and the elastic spectra provided by NTC 2008 (soil-type B).

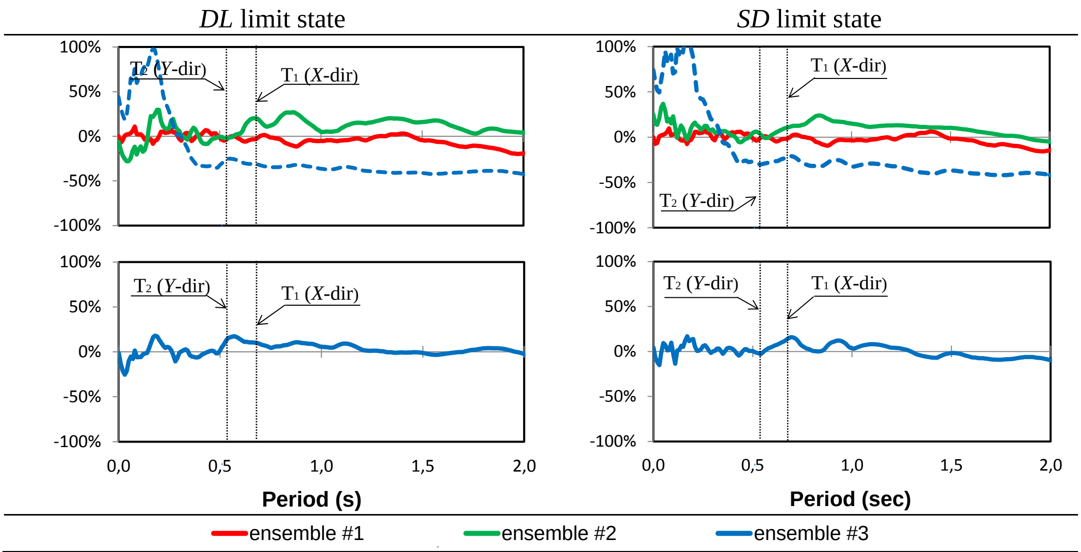


Figure 9. Percentage difference between the spectra of the 3 ensembles and the corresponding NCT 2008 ones.

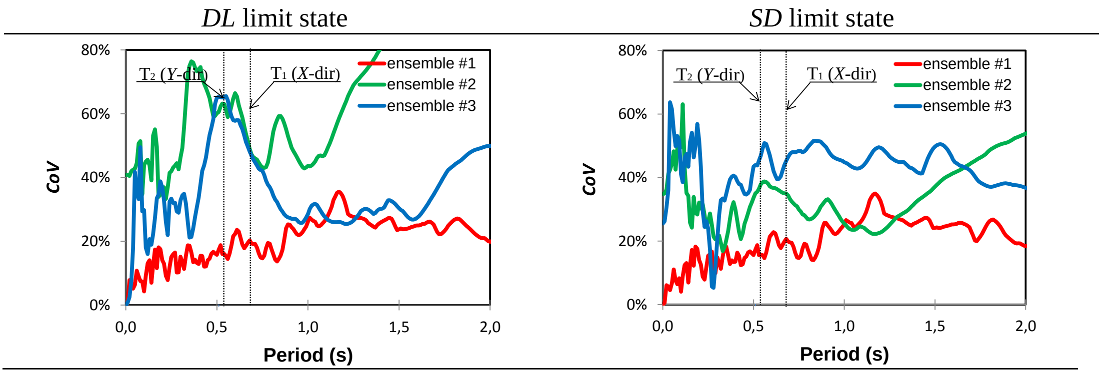


Figure 10. *CoV* of the three ensembles as a function of *T*.

4. THE SEISMIC ASSESSMENT

In this section the seismic assessment of the case-study under the alternative ensembles of ground motions have been shown and compared. In Section 4.1 the global response, in terms of displacement and drift, has been presented, while in Section 4.2 the seismic performance, expressed as the ratio between demand (D) and capacity (C), has been shown. The seismic performance has been found according to EC8 prescriptions, i.e. by checking the columns drifts for the *DL* limit state, and chord rotation and shear force for the *SD* limit state. The limit values have been assumed according to the criteria expressed in Section 2.4.

4.1 The global response

4.1.1 DL limit state

Figure 11 shows the displacement profile found at the mass center (*MC*) of the case study under the three alternative ensembles. The graphs illustrate both the displacement provided by each record and their *mean* (solid circles). Therefore they show the difference among the obtained displacements both in terms of average values and dispersion of results.

Figure 12 shows the drift profile along the case-study height. The maximum drift occurs at the second storey in both directions. In the diagrams the 0.5% drift limit provided by EC8 for the non-structural components has been evidenced. Despite the drift values obtained through the three ensembles are, in all cases, lower than the 0.5% limit, they are very different from each other. The difference between the results obtained by the three ensembles has been analyzed in terms of Percentage Difference (*DIFF%)* with respect to the results provided by the ensemble #1, which is the one that most closely approaches the elastic spectra provided by the Code.

Figure 13 shows the results of this comparison both for the displacements and drifts at each storey. As can be noted, the ensemble #2 induces, comparing to the results provided by the ensemble #1, a systematic increase in the response, (with a maximum of about 33%), whilst the ensemble #3 provides a reduction in the response ranging between 11% and 33%.

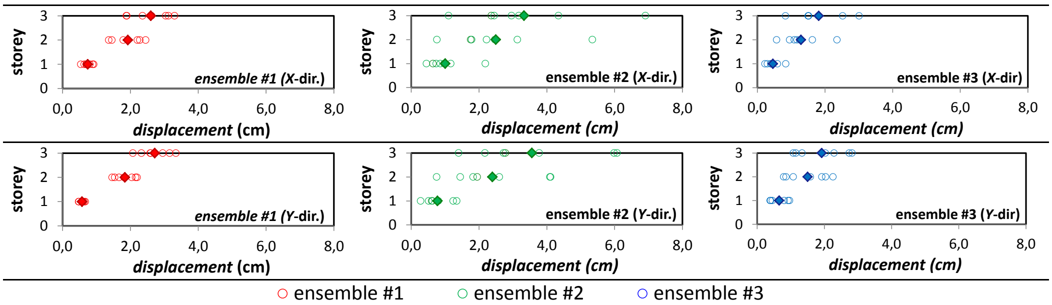


Figure 11. *DL* limit state: maximum displacement at the *MC* for the 3 ensembles.

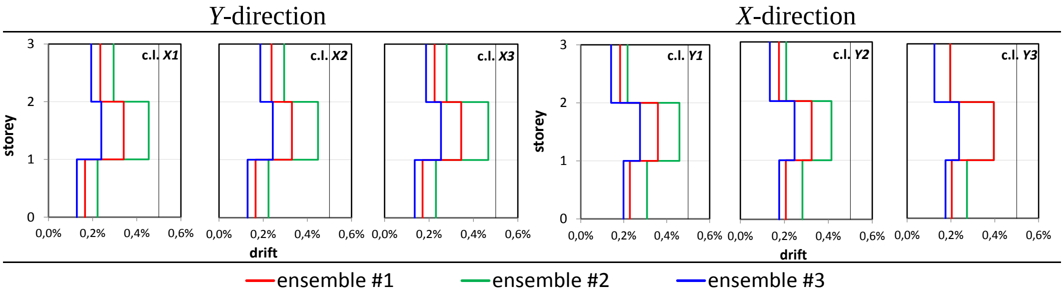


Figure 12. *DL* limit state: drift profiles for the 3 ensembles.

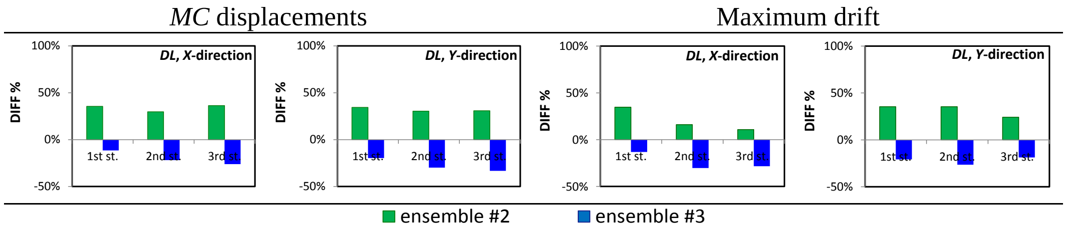


Figure 13. *DL* limit state: comparison among the maximum response found by the three ensembles

4.1.2 SD limit state

Figures 14 and 15 show the displacement and drift profiles provided by the three assumed ensembles. Drift and displacement profiles have a similar trend with respect to the ones found for the *DL* limit state. The displacement domains provided by the ensemble #1 are the less scattered, while the ones provided by the ensembles #2 and 3 present a larger dispersion. These two response domains, in turn, differ very much in terms of mean values, being one about double than the other.

The drift profiles at each column line, shown in Figure 15, illustrate the torsional response due to the in-plan irregularity of the case-study. As can be noted, the analysis along the *Y*-direction provides similar response in all column lines, while in the analysis along the *X*-direction, different values of drift are observed for each column line. The three drift domains evidence a similar sensitiveness to the torsional effects, with larger responses at the column line *Y1* (analysis along the *X*-direction).

Figure 16 shows the comparison among the results provided by the three ensembles in terms of *DIFF%*. The comparison evidences similar trends than the ones observed for the *DL* limit state; the maximum increase in the response found by adopting the ensemble #2 reaches, in this case, 50%, while the maximum decrease related to the ensemble #3 reaches 29%.

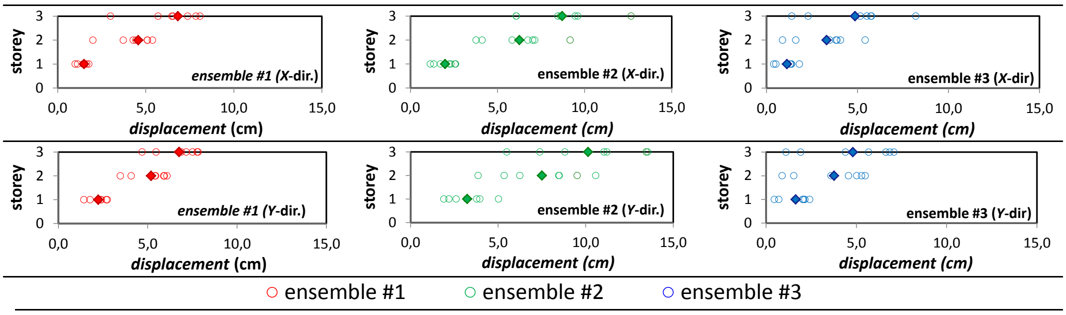


Figure 14. *SD* limit state: maximum displacement at the *MC* for the 3 ensembles.

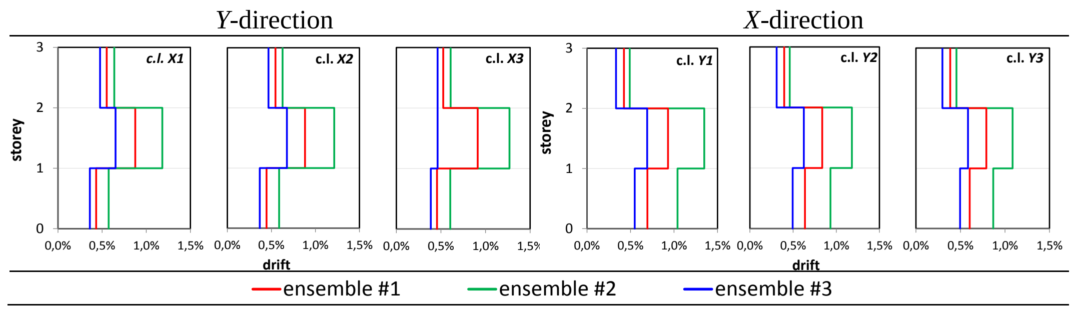


Figure 15. *SD* limit state: drift profiles for the 3 ensembles.

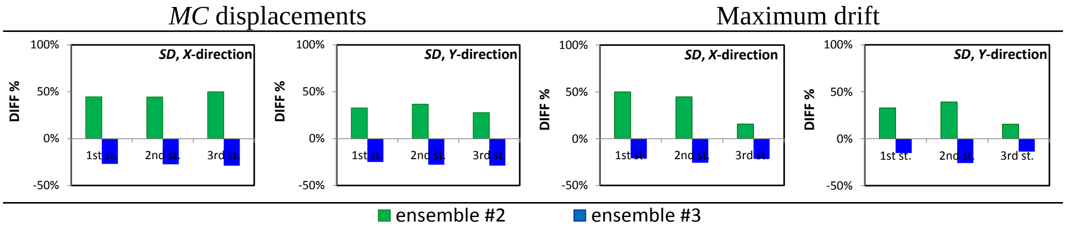


Figure 16. *SD* limit state: comparison among the maximum *MC* displacements found by the three ensembles.

4.2 The seismic performance

4.2.1 DL limit state

The structural performance has been expressed as D/C ratio and reported in the following for the columns of the two lower storeys only, since the structural elements of the third level were always largely complying the code prescriptions, presenting D/C values always below 0.1.

Figure 17 shows the obtained D/C values at the application of the different ground motion ensembles: despite being well below the limit (D/C=1), they show a significant difference, depending on the adopted ground motion ensemble.

The comparison among the results provided by each ensemble, expressed in terms of *DIFF%* with regard to the results coming from the ensemble #1, is shown in Figure 18.

X-direction	*Y-direction*

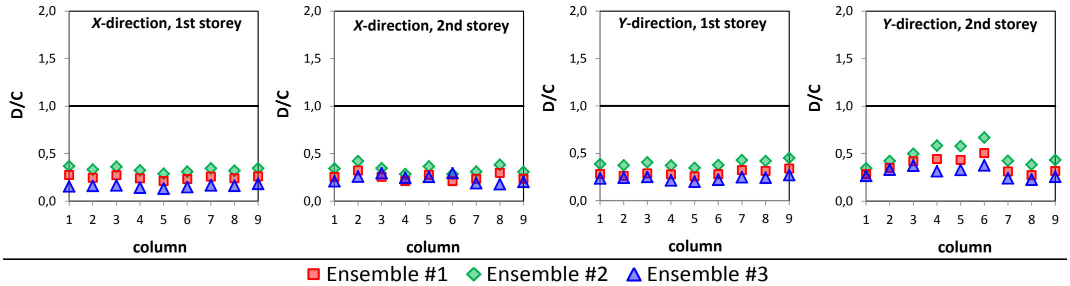


Figure 17. *DL* limit state: D/C ratio in the columns for the 3 ensembles.

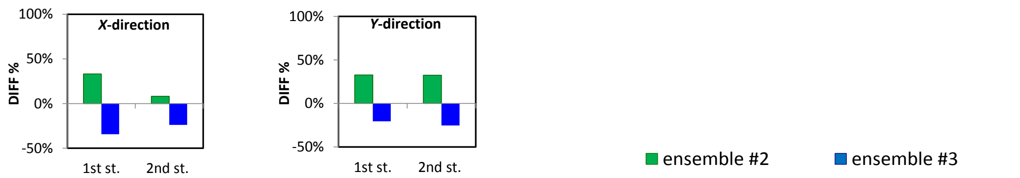


Figure 18. *DL* limit state: comparison among the D/C ratios found by the three ensembles

4.2.2 SD limit state

The *SD* limit state has been checked both in terms of chord rotation and shear force. Figs 19 and 20 show the obtained D/C ratios for columns and beams of the first and second storeys. The columns are more critical with respect to the beams, showing some values of the D/C approaching the unity, both at the first and at the second storey.

Figure 21 shows the comparison among the results obtained through the three assumed ensembles, expressed in terms of *DIFF%*. The increase in the chord rotation induced by the ensemble #2, with respect to the ensemble #1, reaches 50% in both beams and columns at the first storey, when the analysis is performed along the *X*-direction, while in the *Y*-direction, the maximum increase reaches 38% in the columns (first storey) and 23% in the beams (first storey). The *DIFF%* in the chord rotation found by adopting the ensemble #3 shows a decrease ranging between 14% and 25% in all cases.

X-direction	*Y-direction*

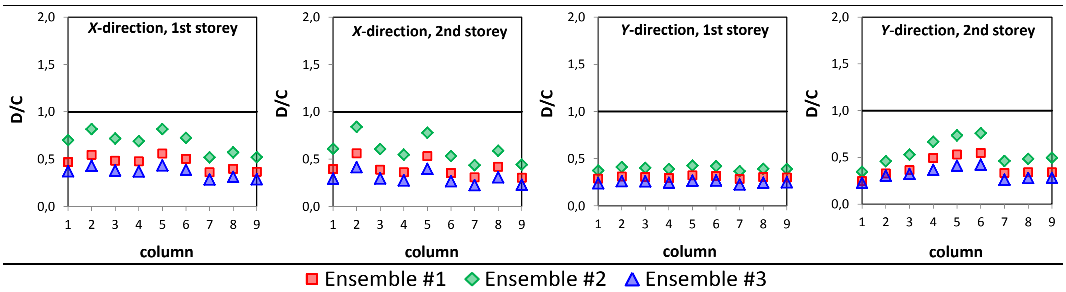


Figure 19. *SD* limit state**:** chord rotation. D/C ratio in the columns for the 3 ensembles.

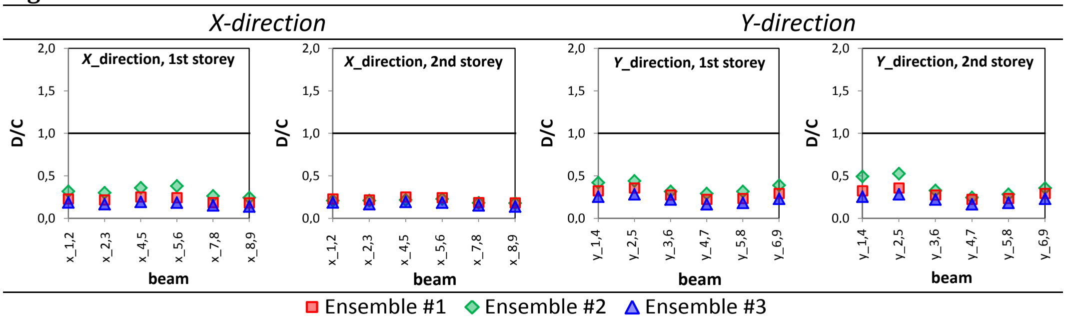


Figure 20. *SD* limit state**:** chord rotation. D/C ratio in the beams for the 3 ensembles.

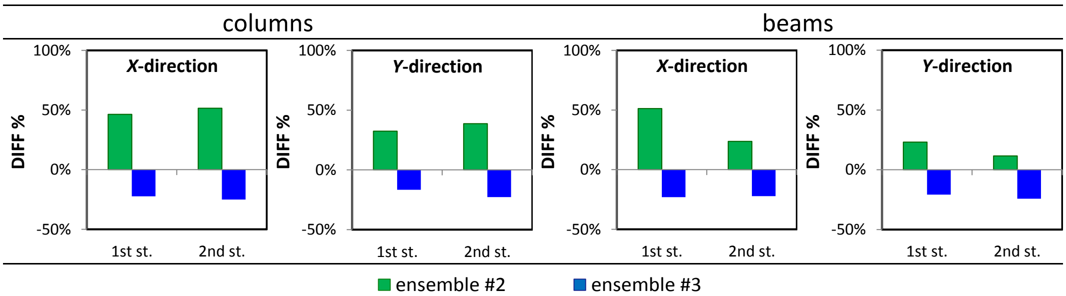


Figure 21. *SD* limit state (chord rotation): comparison among the D/C ratios found by the three ensembles.

Figs 22 and 23 show the same D/C ratios related to the shear force verification. While the chord rotation shows higher values for the seismic input acting along the *X*-direction, the shear force shows larger values in the *Y*-direction. The case-study, indeed, symmetric along the *Y*-direction, has all the columns with the larger dimension oriented along the *Y*-direction. Due to the slight asymmetry, therefore, the case-study experiences an increase in drift and – consequently – in chord rotation at the side column lines, when the seismic input is applied in the *X*-direction, whilst the distribution of the shear force demand in the members is related to the orientation of the columns, which have the larger dimension oriented in the *Y*-direction.

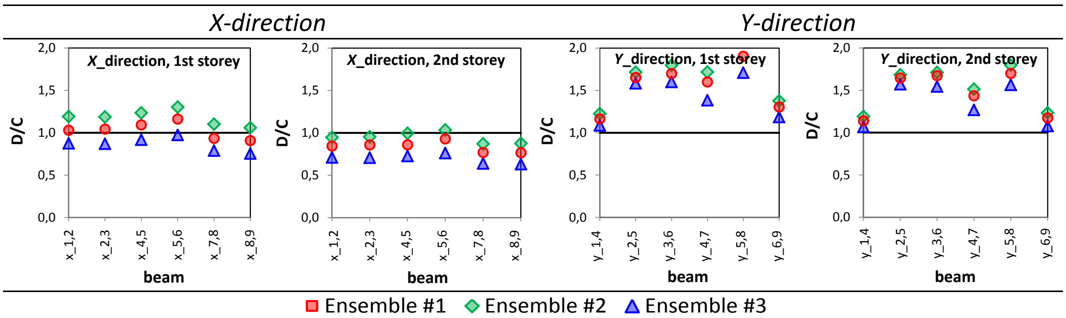


Figure 22. *SD* limit state**:** shear force. D/C ratio in the beams for the 3 ensembles.

X-direction *Y-direction*

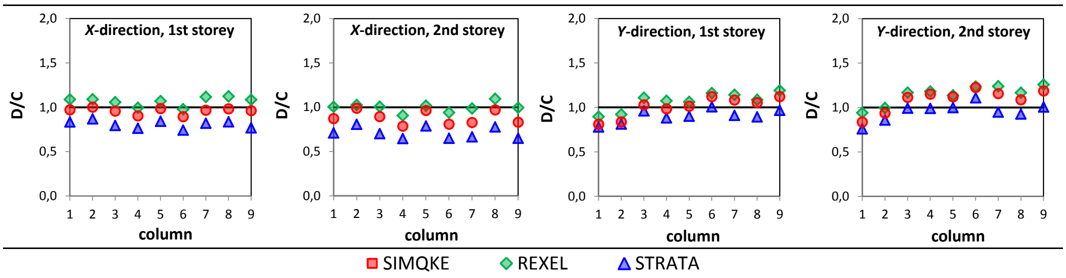


Figure 23. *SD* limit state**:** shear force. D/C ratio in the columns for the 3 ensembles.

The beams running along the *Y*-direction do not comply the unity check as regards the shear force, for the seismic input applied in the *Y*-direction, whichever ensemble is adopted as seismic input. Conversely, the columns and the beams running along the *X*-direction overcome their shear limit depending on which ensemble is adopted. When the ensemble #2 is assumed as seismic input, the shear force limit is exceeded in both beams and columns in the two directions at the first and at the second storey.

Figure 24 shows the comparison among the D/C values concerning the shear force, found through the three ensembles, in terms of *DIFF%*. The increase in the D/C ratio related to the ensemble #2 ranges between 3% and 12%, while the decrease related to the ensemble #3 ranges between 5% and 12%.

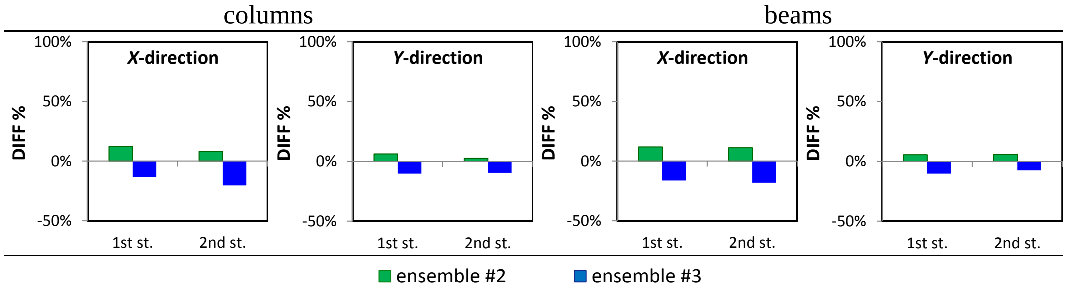


Figure 24. *SD* limit state (shear force): comparison among the D/C ratios found by the three ensembles.

5. CONCLUSIVE REMARKS

This work deals with the seismic assessment of an existing RC building and presents a comparison of the results obtained by nonlinear dynamic analysis, by applying 3 different ground motion ensembles, all complying with the Italian Code NTC 2008 requirements. The aim is to check the effect of the choice of different ground motion types (natural or artificial) and different approaches (soil-B spectrum compatible or soil-A spectrum compatible accelerograms post-processed through a site response analysis) on the results of the seismic assessment.

A very detailed knowledge has been achieved on the building, including both structural and geological aspects. The site ground have been extensively investigated, leading to the assumption of a soil-type B. Three different ensembles of ground motions have been assumed. Two of them, respectively consisting 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), consisting of natural ground motions taken by a regional database, has been set on the bed-rock and then subjected to a site response analysis, by filtering the signals through the layout of soil layers detected on site.

The three selected ensembles differ very much each other both in terms of dispersion and spectral ordinates. The ensemble #1, indeed, presents a mean spectrum very close to the one of the ensemble #2, but a lower scattering. The ensemble #3, instead, has lower values of Spectral Acceleration than the other two, in the period range within which is included most of the dynamic response of the structure. It does not comply the NTC 2008 requirements about the spectral matching if the B-class of soil is considered. Nevertheless in this case, the matching is made on the bedrock spectrum and therefore the ensemble must be considered reliable according to the NTC 2008 prescriptions.

The seismic response of the case-study reflects the differences underlined in the three ensembles. The response domains, in fact, differ each other both in their *mean* value and in their dispersion. The difference among the mean value of each response domain has been measured in non-dimensional terms, by comparing the *mean* response found by the ensembles #2 and #3 to the one found through the ensemble #1. The ensemble #2, provides the highest results, with a larger drift up to 33% than the one provided by the ensemble #1.

The structural performance of the case-study, expressed as the D/C ratio between demand and capacity, has been checked as a function of the three selected ensembles. The case study largely complies with the limit values provided by EC8 for the *DL* limit state, as well as with the ones referred to the chord rotation for the *SD* limit state, whilst it does not respect the limit conditions concerning the shear force. In this last case, the choice of an ensemble with respect to another one can lead the code requirements to be fulfilled or not, completely changing the outcome of the verification. In fact, when the ensemble #3 is adopted, only the beams running along the *Y*-direction overcome the limit shear conditions, while all the other members comply the unity check. When the ensemble #2 is adopted, instead, almost all members exceed their limit values. The ensemble #1 provides results included between the ones coming from the ensembles #2 and #3.

The effects related to the choice of the assumed ensemble has been measured in percentage terms (*DIFF%*), by comparing the results in terms of seismic performance found by the ensembles #2 and #3 to the ones coming from the ensemble #1. The ensemble #2 has resulted to be the more conservative of the three, with a DIFF% up to 50% higher in both beams and columns in the *X*-direction analysis and up to 38% in the *Y*-direction analysis, when the performance is checked in terms of chord rotation. When the performance is checked in terms of shear force, instead, the overestimation never exceed 12%. The reduction in the seismic response coming from the adoption of the ensemble #3 is less sensitive to the assumed response quantity and direction of analysis. The related DIFF% ranges between -25% and -15% for the performance expressed in terms of chord rotation and slightly lower (-20% ÷ -6%) for the performance expressed in terms of shear force. In all cases, i.e. whichever limit state and response quantity, the choice of ground motion ensembles proved to largely affect the results of the seismic assessment of the case-study.

According to this work, therefore, the three assumed ensembles of ground motions, despite all complying with the provisions of NTC 2008, provide very different results, possibly affecting the acceptance evaluation of the seismic assessment of the case-study. Since the existing buildings are very sensitive to performance requirements hardly to be fulfilled, a careful evaluation of the seismic input is a crucial aspect of the seismic assessment. As a conclusion, therefore, an accurate investigation of the site soil should be made in order to select the more reliable as possible seismic input to adopt in the seismic assessment.

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