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SENSITIVITY STUDY ON THE DISCRETIONARY NUMERICAL MODEL ASSUMPTIONS IN THE SEISMIC ASSESSMENT OF EXISTING BUILDINGS

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Abstract: In this work the main assumptions to be made in the seismic assessment of existing RC buildings have been assessed, in order to evaluate how much each of them can affect the obtained evaluation of the building safety, with reference to a case-study structure. Eurocode 8 provides all the information needed to perform the seismic assessment of existing buildings. Even if all the provided prescriptions are followed, in several assumptions a large subjectivity is left to the engineer in charge of the analysis. In this work the scatter related to such choices is evaluated with reference to a case-study building: a real RC Italian building currently used as a hospital. The contemplated assumptions concern the material characterization, the seismic input definition, the type of performed analysis and the set-up of the numerical model adopted to be representative of the building behaviour. The effects of each of these assumptions have been assessed in terms scatter induced in the assumed response parameters, in order to evaluate the role of the discretionary choices in the seismic assessment. The results presented in this paper are representative of the analysed case-study building only, but aims at putting a spotlight to the effect that some too discretional prescriptions of the technical codes can have on the seismic assessment of existing structures.

1. INTRODUCTION

The seismic assessment of existing buildings is one of the most topical issues of seismic engineering. In these years many European countries are dealing with the problem of assuring the due structural safety to the buildings population. Many of their current buildings, in fact, have been constructed in the 60s and 70s, and consequently they do not comply with the technical requirements, included the seismic ones, provided by the Codes in force. The choice of how to reduce the seismic vulnerability of existing buildings is related to both the safety level that needs to be ensured, combined to their capacity to provide an overall satisfactory efficiency (functional, energetic, technological etc.), and to the available economic resources. If the economic aspect does not influence the evaluation, existing buildings - not complying with the current standards - could be demolished and replaced. In this case the safety of the buildings population could be easily achieved, and their performance could be kept at the desired level. More often, instead, the lack of money induces a careful evaluation of the structural safety of the buildings. If the seismic performance of the buildings is adequate, or if it can become adequate with a limited amount of improvements, the choice of retrofitting the buildings is often preferred to the more radical and expensive replacement.

A reliable evaluation of the seismic capacity of existing buildings is, therefore, a crucial issue for the management of the building population of the European countries. The seismic assessment of a building is performed through the comparison between its ability to withstand a seismic event with a given (conventional) intensity and some performance thresholds provided by EC 8-3 [1] and integrated by the National Annex of each country. While the thresholds values are well defined by the codes, the seismic capacity of the buildings is possibly affected by the choices made by the engineer in charge of the assessment. The subjectivity of the evaluation cannot be completely avoided; in order to determine the seismic capacity of the building, in fact, the engineer must take a number of decisions regarding: *i)* the assumptions about the mechanical properties of materials, *ii)* the seismic input representation, *iii)* the type of analysis to reproduce the seismic response, and *iv)* the numerical model to adopt.

Each of these choices can be made according to a range of possibilities provided by [1]. However, even when the choices are made within the Code frame, they can largely differ with each other, affecting the obtained results. As a conclusion, depending on the choices made by the engineer in charge of the assessment, different conclusions about the seismic performance of existing buildings can be possibly achieved. In this paper, the most relevant "subjective" factors affecting the seismic assessment of existing RC buildings have been checked with reference to a case-study, i.e. an existing RC building, located in Tuscany (Italy) and currently used as a hospital. The structure has been the object of a wide knowledge process, that is the result of a joint agreement with the Regional Government of Tuscany. This case-study has been the object of some previous papers by the author, focused, respectively, on the seismic performance of the building by means of alternative types of analysis [2], and on the soil characterization [3,4,5]. These papers have shown as, even when a wide amount of information is collected, the numerical representation of the building is still very controversial. Both the modelling of the building and of the soil have evidenced different possible - and plausible – choices, which provide a relevant scatter in the assessed seismic behaviour of the building.

This paper is focused on the effects of the possible assumptions made to perform the seismic assessment of existing buildings as regards the type of analysis, the seismic input representation, the materials characterization and the numerical model set-up. In Section 2 the case-study building is briefly described and in Section 3 the main assumptions made to perform its seismic analysis are described and discussed. Section 4 shows the obtained results, in terms of scatter in the selected response parameters due to the contemplated assumptions. The obtained results evidence the relevant variability in the seismic assessment of the case-study building due the subjective choices made by the engineer in charge of the analysis.

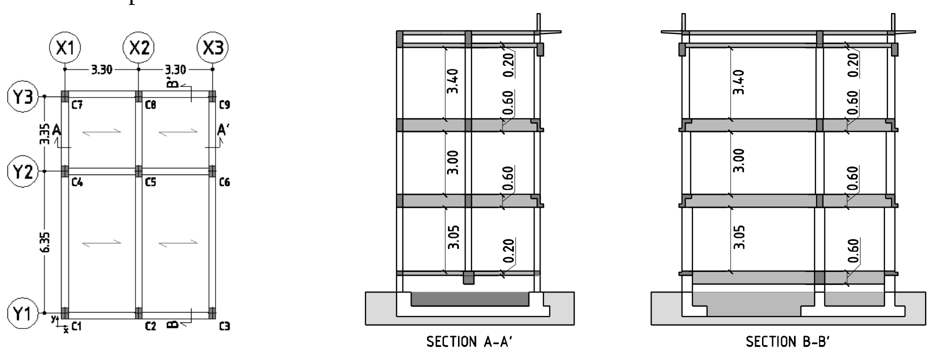
2. THE CASE-STUDY

The assumed case-study is an existing RC building, located in Tuscany (Italy) and currently used as a hospital; it has been object of a wide investigation [2], which has provided many information about its geometry and structural features, mechanical properties of soil and materials. A brief description of the building and its foundation soil is provided in the next paragraphs, although further information can be found in the quoted references.

2.1 The building

The building, shown in Figure 1, has a 3-storey RC structure and it has been designed in 1976, before the introduction of the current seismic Italian legislation [6]. The building presents some efficient design criteria, like column section reduction from foundation level to the top storey and solid connection of the beam-column joints, although it is far away from complying the current seismic design criteria.

a. plan b. sections c. eccentricities



X-direction

e_{str}	e_{stiff}
9.0%	10.7%
7.5%	9.4%
6.1%	7.9%

Y-direction

e_{str}	e_{stiff}
3.3%	2.2%
5.6%	5.0%
0.8%	3.2%

d. Views of the building

Figure 1. Case-study building (values in m in Figures 1a-b).

The building has a regular plan (Fig. 1a), with a structural symmetry around the y-axis, despite the infill panels distribution is not symmetric in any direction. The eccentricity at each storey has been determined in terms of strength (e_{str}) and stiffness (e_{stiff}), by comparing the mass center (MC), given by the mass of the floors and of the infill panels, to the center of strength (C_{str}) of the columns and to the center of stiffness (C_{stiff}), found according to the simplified relationship proposed by Anagnastopoulos [7]. The values obtained for the eccentricities are listed in Fig. 1c; further information about geometry, reinforcement and structural details can be found in [2].

2.2 The soil

The case-study building is located in Sansepolcro, one of the most seismic areas in Tuscany (Italy). According to the National soil classification [8], the area has a Peak Ground Acceleration (PGA) equal to 0.227g for a Return Period of 475 years. A careful geological investigation has been performed [3], in order to assess the soil stratigraphy and to determine the value of the uppermost 30 m shear-wave velocity ($v_{s,30}$) of the site, that represents the key-parameter for the soil classification according to [1].

Four different types of test, based on the correlation between the propagation velocities of seismic waves through the soil and its mechanical properties [9], have been performed to assess the soil properties. More specifically, a down-hall test (DHT), a seismic refraction test (SRT), a Multi-channel Analysis of Surface Waves, integrated by an Extended Spatial Auto-Correlation test (MASW/ESAC) and a number of single station scanning to seismic noise area (HVSR) have been performed. Each test has provided a different profile of shear velocity. Figure 2 shows the location of the performed tests in the area, together with the contour lines and the position of the case-study. Since each test has been made on a different depth, both the mean value of the shear velocity ($v_{s,mean}$) and the $v_{s,30}$ have been found. The values have been listed in Table 1, together with the consequent soil classification according to [1]. A more detailed description of the soil information and the resulting soil classification can be found in [4].

The position of the case-study building is between the sites on which the DHT and the MASW/ESAC have been performed. A further information is provided by a HVSR scanning, made few meters north of the case-study; such investigation can provide qualitative information only, but it indicates the presence of surface rock. The soil classification, therefore, is affected by the type of performed investigation and by the position of the test, that often is bound to technical reasons.

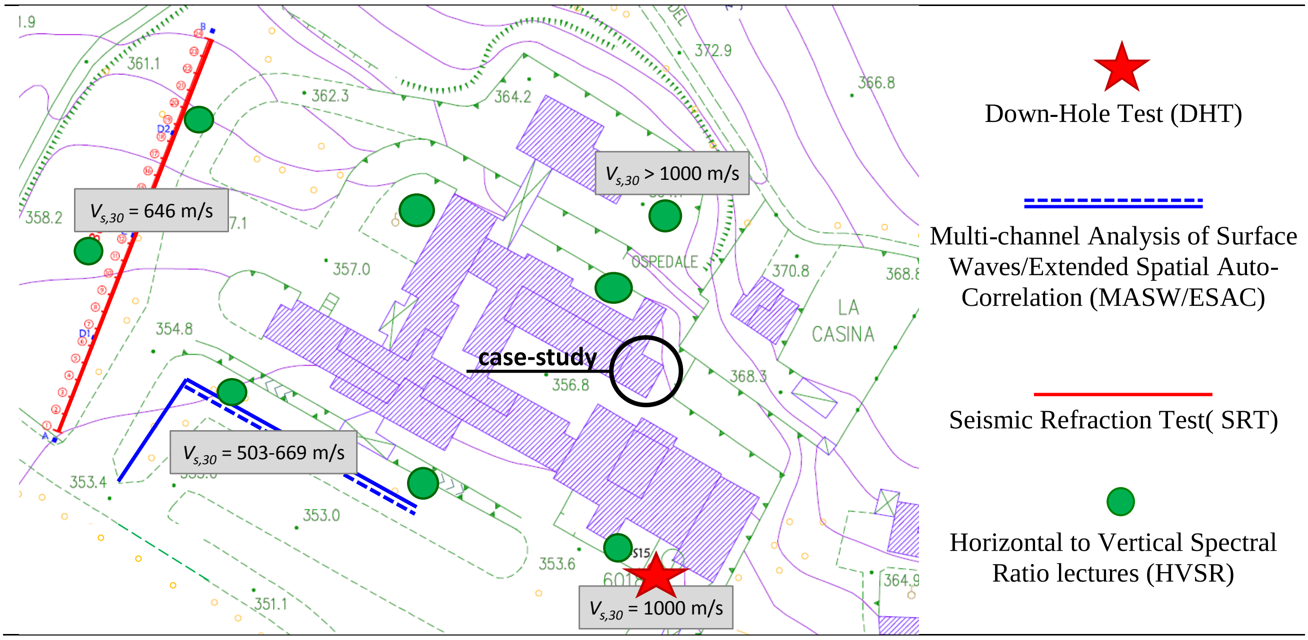


Figure 2. Investigations made on the foundation soil and obtained shear-wave velocities.

Table 1. Shear-wave velocity values (in m/2) provided by the tests on the soil and resulting soil classification.

	$v_{s,\ mean}$	$v_{s,\ 30}$	EC8 (NTC2008) classification
DHT	1000	867	A-soil
SRT	646	646	B-soil
MASW/ESAC	394-514	503-669	B-soil

3. THE ASSUMPTIONS MADE FOR ANALYSIS

3.1 Mechanical properties of materials

The codes [1,6,10] agree in assuming the mechanical properties of materials after an accurate *in-situ* investigation [11]. For material samples having a Coefficient of Variation (CoV) over 14%, however, the American and European Codes provide different approaches. In case of *CoV* values exceeding the 14% limit, the American code [10] prescribes to assume a design strength equal to the *mean* value reduced by the *standard deviation*. In [1], instead, it is assumed a design strength value as a function of the quality of the investigation only, neglecting the level of dispersion of the experimental data. Depending on the Knowledge Level (*KL*), therefore, a different value of Confidence Factor (*CF*), respectively equal to 1.00, 1.20 and 1.35 can be assumed. The approach presented in [1] is certainly safe enough for materials with low or moderate dispersion. In existing RC buildings, however, concrete often presents very poor and scattered mechanical properties [12-13]. When the concrete presents a high variability, there are two possible different problems: *i)* single members with a strength value largely lower than the assumed one, and ii) an irregular distribution of the seismic energy, both in plan [14,15] and in elevation [16], due to torsional effects [17] related to the strength variability itself. The combination of the two above mentioned factors can represent a vulnerability source for the building [18,19], whose seismic performance can result much lower than the one provided following the approach in [1]. The assumption of a uniform strength in all the members of the building, therefore, does not always assure a safe evaluation of its seismic performance. Since the prescriptions followed in the present study are the ones of the EC 8-3 [1], a uniform strength distribution will been assumed for the case-study, but the sensitivity of the results to the *KL* will be assessed.

The mechanical properties of materials have been extrapolated from an extensive investigation. As regards the concrete, both SonReb [20,21,22] and destructive [23] tests have been performed and subsequently the results have been combined by adopting an *ad hoc expression* [24], which has provided a final cylindrical strength, $f_{c,mean}$, equal to 10.2 MPa. According to the structural design, the reinforcement steel belongs to the FeB32K class, having a yield stress over 32 MPa and a ultimate strength over 50 MPa. By a visual inspection, two different types of steel have been identified, respectively ribbed and not. Three destructive tests, one for each storey, have been done on rebars samples, according to the standard procedure [25], returning a *mean* value, $f_{s,mean}$, equal to 385.7 MPa.

According to [1], the number and quality of performed tests returned a *KL1* for steel and *KL2* for concrete. In the current study, all the possible design values compatible to the experimental investigation, listed in Tab. 3, have been evaluated, in order to evaluate the effects of the material assumptions on the seismic performance of the case-study.

Table 2: Design values assumed for materials (values in MPa).

materials	*mean* strength	Knowledge Levels			Values assumed in La Brusco et al. (2015)
		KL3 (CF = 1.00)	KL2 (CF = 1.20)	KL1 (CF = 1.35)	
concrete	10,2	10,2	8,5	7,6	8,5
steel	385,7	385,7	321,4	285,7	285,7

3.2 Type of analysis

EC 8-3 [1], as well as the Italian NTC 2008 [6], provides the possibility to perform alternative analyses to determine the seismic response of buildings, according to some criteria. Despite the choice of the numerical procedure to adopt is partially left to the designer, it is well known that the obtained seismic performance is sensitive to the type of analysis [26, 27, 28]. The pseudo-dynamic linear analysis is usually more conservative than the inelastic ones. The inelastic analyses are assumed to be more accurate, especially in representing the seismic response of RC buildings, which present relevant non-linearity even for low seismic excitation [26,29,30].

The comparison in terms of seismic assessment among different analysis types is not trivial, since elastic and inelastic analyses provide different response parameters, which must be checked by following different criteria. In the inelastic analyses the response quantities required by the Code for verification in the Significant Damage (*SD*) limit state are the chord rotation and the shear force, which are respectively representative of ductile and brittle collapse mechanisms. For the same limit state instead, the elastic analysis is checked in terms of bending moment and shear force. The ductile and brittle mechanisms are taken into account by adopting different seismic spectra, reduced respectively by a q-factor equal to 3.0 (ductile mechanisms) and 1.5 (brittle mechanisms).

In order to compare the results of the seismic assessment provided by different types of analysis, the ratio between structural capacity (C) and the seismic demand (D) has been assumed as response quantity. The three assumed analysis types, i.e. the linear pseudo-dynamic, the nonlinear static and the nonlinear dynamic ones, have been performed according to the EC 8-3 [1] prescriptions. Table 3 resumes all the performed analyses.

Table 3. Performed analyses.

Nonlinear static (pushover) analysis (*P*)

X-direction						Y-direction					
e=0		e = +5%		e = -5%		e=0		e = +5%		e = -5%	
+way	-way	+way	-way	+way	-way	+way	-way	+way	-way	+way	-way
PMX+E0	*PMX-E0*	*PMX+E+*	*PMX-E+*	*PMX+E-*	*PMX-E-*	*PMY+E0*	*PMY-E0*	*PMY+E+*	*PMY-E+*	*PMY+E-*	*PMY-E-*
P1X+E0	*P1X-E0*	*P1X+E+*	*P1X-E+*	*P1X+E-*	*P1X-E-*	*P1Y+E0*	*P1Y-E0*	*P1Y+E+*	*P1Y-E+*	*P1Y+E-*	*P1Y-E-*

Nonlinear dynamic (time-history) analysis (*D*)

X-direction							Y-direction						
record 1	*record 2*	*record 3*	*record 4*	*record 5*	*record 6*	*record 7*	*record 1*	*record 2*	*record 3*	*record 4*	*record 5*	*record 6*	*record 7*
D1_X	*D2_X*	*D3_X*	*D4_X*	*D5_X*	*D6_X*	*D7_X*	*D1_Y*	*D2_Y*	*D3_Y*	*D4_Y*	*D5_Y*	*D6_Y*	*D7_Y*

Linear analysis (*L*)

X-direction						Y-direction					
e=0		e = +5%		e = -5%		e=0		e = +5%		e = -5%	
+way	-way	+way	+way	-way	+way	+way	-way	+way	+way	-way	+way
LX+E0	*LX-E0*	*LX+E+*	*LX+E0*	*LX-E0*	*LX+E+*	*LY+E0*	*LY-E0*	*LY+E+*	*LY+E0*	*LY-E0*	*LY+E+*

In the pseudo-dynamic linear analysis (*L*), the seismic response has been found by combining - according to the participation factors - the spectral accelerations corresponding to the main five periods of the structure, preliminary found through an eigenvalue analysis. The elastic spectrum assumed as seismic input has been scaled by a q-factor assumed respectively equal to 1.5 and 3.0 for brittle and ductile behaviour. The structural response in each direction is the maximum of 6 different analyses, where the lateral force has been applied in both ways at the mass center, with no eccentricity (*E0*) and with an eccentricity equal to +/- 5% (*E*, *E-*). In all cases the response in one direction has been combined with that coming from the application of the 30% of the seismic action in the orthogonal direction.

The nonlinear static (*P*) analysis has been performed by assuming two different force distributions, respectively proportional to the 1st mode (*P1*) and to the masses (*PM*). The determination of the maximum displacement experienced by the case-study under the assumed seismic input has been made by assuming the inelastic displacement to be equal to the elastic one, since in all the analyses the intersection between the demand and the capacity spectra, corresponded to periods larger than T_C. For each direction of analysis, an eccentricity equal to +/- 5% has been assumed, beside the case with no eccentricity. The structural response in each direction is the maximum of 12 different analyses.

To perform the nonlinear dynamic analysis (*D*) an ensemble of seven ground motions, spectrum-compatible to the elastic spectrum provided by the Code, has been selected (see Section 3.3). The seismic response in each direction has been found by averaging the maximum response parameters provided by the seven ground motions. According to EC 8-3 [1] prescriptions, the seismic response to be assumed in the verification is the maximum one for pseudo-dynamic and pushover analyses, and the mean one for nonlinear dynamic analysis.

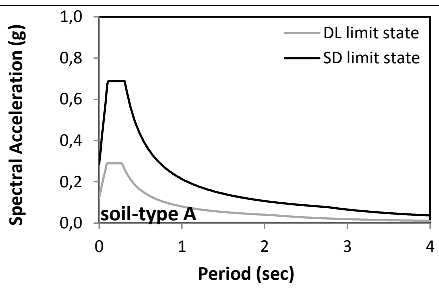
The capacities of each member have been found by assuming, for concrete and steel, the strength values reported in the last column of Table 3, and by assuming the effective geometry and reinforcement distribution (see [2]). As regards the *SD* limit state, the ultimate shear force has been assumed for brittle failure, whereas the ultimate bending moment and the ultimate chord rotation have been assumed for ductile failure. Regarding the Damage Limitation (*DL*) Limit State, instead, the maximum storey drift, to be below 5‰, has be checked.

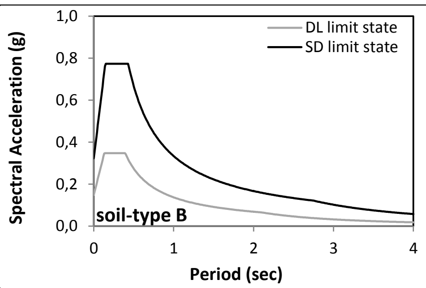
3.3 The seismic input

According to the main International Codes, as EC 8-3 [1], ASCE standards 7-05 [31] and 4-98 [32], FEMA regulations [33], as well as the Italian NTC 2008 [6], the seismic input is quantified according to a soil classification which is a function of the soil properties and of the site seismic hazard. Such classification is usually based on the uppermost 30 m shear-wave velocity ($v_{s,30}$) of the site, even if further parameters, like the bedrock depth, the fundamental period of the soil and the shear velocity in the surface layers would help to achieve a more refined description [34].

In this study, the performed investigation has provided controversial information, which does not drive to a univocal classification. In the previous contributions by the authors [2-5], the soil has been classified according to the safest assumption, and therefore it has been assumed to belong to the B-type, according to [1]. It should be noted that, usually, only one investigation is performed, and the soil classification is consequently made. If the DHT investigation would have been the only available one, therefore, the soil would have been classified as a A-type. In the current work, therefore, both the A and B soil have been evaluated as possible assumptions.

Since the case-study is located in Italy, the Italian Technical Code [6], has been taken into account to integrate the EC 8-3 [1] instructions and adopted to define the seismic spectra. Figure 3 shows the elastic spectra provided by [6] for two different limit states, i.e. *DL* and *SD* for the two alternative soil types.





Parameters for spectra definition

	Limit State	
	DL	*SD*
a_g	0,124	0,287
F_0	2,340	2,396
T^*_C	0,277	0,310
S_T	1,000	1,000

a_g = ground acceleration (g), F_0 = amplification factor on the rock-site, T^*_C = beginning period of the velocity-constant branch (s), S_T = topographic amplification factor

Figure 3. Elastic Spectra representing the seismic input.

The sensitivity analysis has been performed by carrying out the nonlinear static analysis, where the seismic input has been represented through the Code elastic spectrum only. When the dynamic analysis has been performed, an ensemble of 7 ground motions has been assumed to represent the seismic input. The ground motions have been selected by the Italian Accelerometric Archive [35], in order to be spectrum-compatible to the B-soil elastic spectrum. They have been chosen through the adoption of the software REXEL [36,37,38], on the basis of a PGA respectively equal to 0.287g and 0.124g for the *SD* and *DL* limit states, 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 NTC 2008 [6]. The main information of the ensemble of ground motions selected for the dynamic analysis are listed in Table 4.

Table 4. Ground motions data for the assumed Limit States (soil B).

	Name	Location	Date (dd/mm/yyyy)	PGA [g]	Duration [s]
SD	TLM1 HNN	TOLMEZZO	06/05/1976	0.346	36.385
	STR HNN	STURNO	23/11/1980	0.225	70.755
	STR HNE	STURNO	23/11/1980	0.316	70.755
	AQV HNE	L'AQUILA	06/04/2009	0.656	100.000
	AQK HNN	L'AQUILA	06/04/2009	0.354	100.000
	AQG HNN	L'AQUILA	06/04/2009	0.489	100.000
	AQG HNE	L'AQUILA	06/04/2009	0.446	100.000
DL	STR HNN	STURNO	23/11/1980	0.225	70.755
	FRC HNE	FORGARIA	15/09/1976	0.215	21.990
	MRT HNN	MERCATO S.S.	23/11/1980	0.107	79.8500
	MRT HNE	MERCATO S.S.	23/11/1980	0.141	79.850
	NRC HNN	NORCIA	14/09/1997	0.095	39.120
	RNR HNN	RIONERO IN V.	23/11/1980	0.099	79.995
	RNR HNE	RIONERO IN V.	23/11/1980	0.096	79.995

3.4 The finite element model set-up

The finite element model set-up is a crucial step to perform any structural analysis. The structural behaviour of the assessed building must be represented by means of numerical models and it is acknowledged that different models, as well as different softwares, can possibly provide very different results [39]. In this work the attention has focused on some assumptions which are left to the choice of the analysis maker and that can substantially affect the obtained results. There are multiple - all reasonable - modelling choices that can be done by different analysts while building up a finite element model. In the present study only a couple of very simple assumptions that need to be done by modelling an existing building have been analysed, with the aim of pointing out the potential effect that this kind of choices can have on the results of the seismic assessment.

The first assumption concerns the Young modulus (E_c) of the concrete. NTC 2008 [6] suggests to assume a reduced value of the stiffness, and therefore of E_c, to take into account for cracking of brittle materials. The level of reduction is left to the freedom of the analyst even if a maximum reduction to half of the initial value is suggested. This appears to be quite a vague suggestion, affecting the obtained results, first of all in terms of fundamental period of the structure and, consequently, of adsorbed seismic energy.

Another crucial assumption, which does not respond to any prescription, is the behaviour of the joint panels. Each member of a framed RC structure, in fact, is assumed to have fixed ends, which are assumed to be completely deformable in the structural models. To take into account the effect of the joint panel in terms of stiffness, and the consequent reduction of the flexible length of the member, proper rigid arms can be introduced at the ends of beams and columns. This assumption, completely left to the analyst, affects very much the global stiffness of the structure and its seismic response. In [2] the Young modulus has been assumed equal to 50% of E_c, and rigid arms have been introduced in each member.

4. THE SENSITIVITY ANALYSIS

This Section reports the results of the sensitivity analysis to the highlighted "discretional" assumptions. The investigation of the effects of all possible modelling choices would be very interesting, but too wide to be faced in this study.

Table 5 resumes the performed analyses and the hypotheses adopted in each case. Each line of the table represents the focus on a single choice and the bold text highlight the different assessed assumptions.

Each choice has been investigated with reference to different response parameters, pertinent to the type of analysis.

Table 5. Performed sensitivity analysis.

Checked choice	Response parameters	Assumptions				
		Type of analysis	Material strength (MPa)		Soil	Model set-up
			concrete	steel		
Material	*θ_u* *V_u* *M_u*	-	**7.6**	**285.7**	-	-
			8.5	**321.4**		
			10.2	**385.7**		
Type of analysis	*Drift (DL)* *C/D (SD)*	**Elastic**	8.5	285.7	B-soil	0.5 E_c, rigid arms
		Nonlinear static				
		Nonlinear dynamic				
Seismic Input	*SA* *TD* *BS*	Nonlinear static	8.5	285.7	**A-soil**	0.5 E_c, rigid arms
					B-soil	
Model setting	*TD* *BS*	Nonlinear static	-	-	B-soil	**1.0 E_c, rigid arms**
						0.5 E_c, no rigid arms

4.1 Effects related to the material characterization

In the assessment of existing buildings, the seismic response of the structure related to a *DL* seismic excitation is checked in terms of maximum interstorey drift, compared to a limit drift equal to 5‰. In the *SD* limit state, instead, the capacity of each member, in terms of chord rotation Θ_u, bending moment M_u and shear force V_u, must be checked in order to evaluate the seismic performance of the building. The capacity of each member is found on the basis of the assumed strength of the materials, which, in turn, depends on the assumed *CF* values.

In the columns, the limit values of bending moment, shear force and chord rotation have been found by assuming the axial load provided by the dead loads, i.e. neglecting the axial load variation occurring during the seismic response, despite it can also affect the structural capacity [40]. As a consequence of the static axial load, all the columns of each storey, despite having the same geometry and reinforcement (1st storey: 30 x 50, 10 ϕ 16, 2nd storey: 30 x 40, 8 ϕ 16, 3rd storey: 30 x 35, 10 ϕ 14) have a different capacity. The capacities found for the columns of the building for the considered *CF* are shown in Fig. 4; as should be noted, the bending moment M_u of the columns in the *Y*-direction presents the largest variation related to the assumption of different *CF* values; the shear capacity V_u has almost the same value in each column of the same storey, since it is not affected by the axial load level, depending mostly on the stirrups contribution.

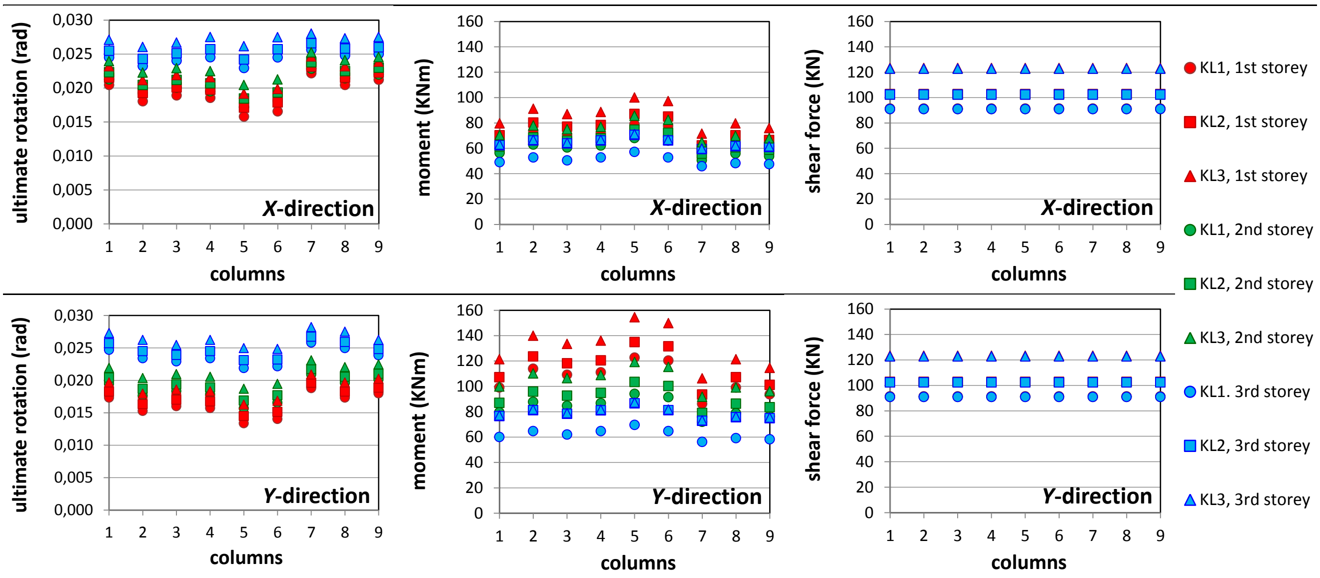


Figure 4. Capacities of the columns for the assumed *CF*.

The beams are assumed to have a negligible axial load, therefore their capacities depend on the structural geometry and reinforcement only; Table 6 shows the obtained capacities found for the beams as a function of the three *CFs*, and the consequent Percentage Difference (*PD*). In case of different reinforcements at the two beam ends, the minimum capacity is reported. *PD* is found as the ratio between the difference of the two extreme values and the maximum one. In Fig. 5 the *PD* found for columns and beams is shown for each storey and direction. As it can be seen, the scatter in the chord rotation ranges between 10% and 15%, while the one in shear force exceeds 20% in many columns.

Table 6. Chord rotation θ_u, Shear force V_u and Bending moment M_u capacities of the beams for the assumed CF (expressed in rad, KN and KN m, respectively)

dir.	code	storey	geometry	reinforcement	quantity	CF = 1.00	CF = 1.20	CF = 1.35	*PD*
X-direction	B1x	1,2	Z-shape	ends: 6ϕ14; 7ϕ14	θ_u	0.0264	0.0242	0.023	12,9%
					V_u	200	167	148	26,0%
					M_u	175	146	130	25,7%
	B2x	1,2	30 x 60	ends: 4ϕ14; 3ϕ14	θ_u	0.0198	0.019	0.0185	6,6%
					V_u	200	167	148	26,0%
					M_u	130	108	96	26,2%
	B3x	3	30 x 60	ends: 3ϕ16; 3ϕ16	θ_u	0.0172	0.0164	0.016	7,0%
					V_u	200	167	148	26,0%
					M_u	247	206	183	25,9%
	B4x	3	30 x 80	ends: 4ϕ14; 3ϕ14	θ_u	0.0212	0.0203	0.0198	6,6%
					V_u	200	167	148	26,0%
					M_u	127	106	94	26,0%
Y-direction	B1y	1,2	Z-shape	end (l) 11ϕ16; 8ϕ16 end (r) 10ϕ16; 8ϕ16	θ_u	0.0195	0.0188	0.0184	5,6%
					V_u	200	167	148	26,0%
					M_u	415	345	307	26,0%
	B2y	1,2	30 x 60	ends: 5ϕ14 + 4ϕ16; 4ϕ16	θ_u	0.0199	0.0191	0.0186	6,5%
					V_u	200	167	148	26,0%
					M_u	203	170	151	25,6%
	B3y	1,2	Z-shape	end (l) 7ϕ16; 4ϕ16 end (r) 11ϕ16; 4ϕ16	θ_u	0.0215	0.0207	0.020	7,0%
					V_u	200	167	148	26,0%
					M_u	322	268	238	26,1%
	B4y	3	30 x 80	ends: 5ϕ14 + 4ϕ16; 4ϕ16	θ_u	0.0202	0.0194	0.0189	6,4%
					V_u	270	225	200	25,9%
					M_u	177	148	131	26,0%
	B5y	3	30 x 80	ends:5ϕ14 + 1ϕ16; 4ϕ16	θ_u	0.0261	0.025	0.0244	6,5%
					V_u	270	225	200	25,9%
					M_u	443	370	328	26,0%

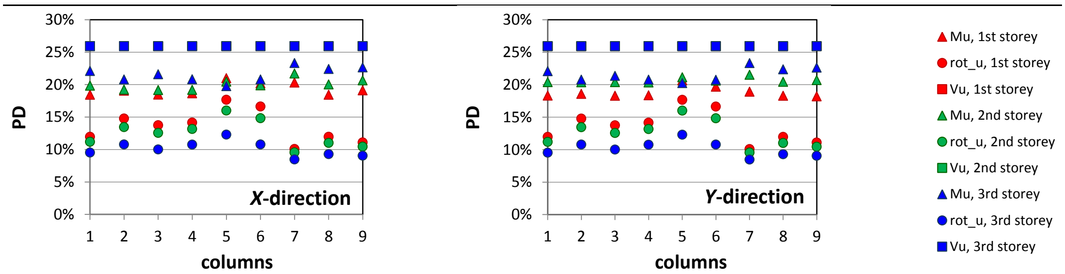


Figure 5. *PD* capacity in the columns due to the different assumed *CFs*.

4.2 Effects related to the type of analysis

4.2.1 DL limit state

The response parameter indicated by EC 8-3 [1], in order to check the seismic performance of existing buildings related to the *DL* limit state, is the drift, regardless the adopted type of analysis. In this section, therefore, the drift has been used as response parameter. Figure 6 shows the maximum drift values provided by the three analysis types, linear (L), pushover (P) and nonlinear dynamic (D),for each building storey. In the same Figure, the percentage difference (*PD*) related to the use of alternative types of analysis can be seen.

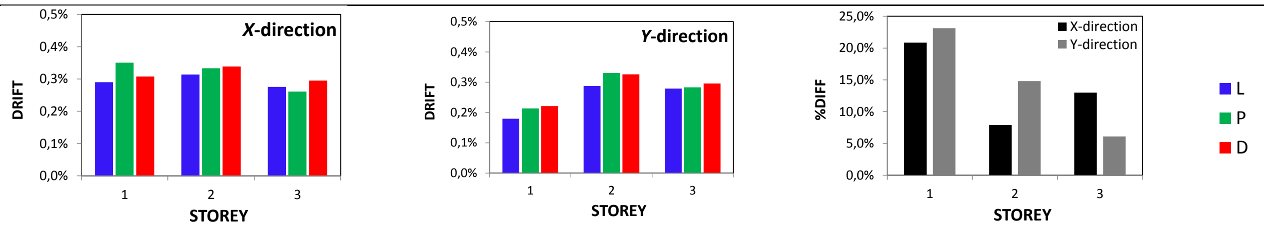


Figure 6. *DL* limit state: maximum drift and percentage difference due to the type of analysis.

4.2.2 SD limit state

The response quantities required by EC 8-3 [1], in order to check the seismic performance of existing buildings related to the *SD* limit state, are the chord rotation and shear force for inelastic analyses and the bending moment and shear force for elastic analysis. In order to compare the results of different analysis types, that refer to different response quantities, the performance index *C/D* has been assumed as control parameter. The capacities have been found for each structural element of the building, by assuming for the materials the strength values specified in Tab. 6.

Figure 7 shows the *C/D* values obtained through the three performed analyses for ductile and brittle mechanisms in the beams (B_duc and B_brit) and in the columns (C_duc and C_brit), respectively. It can be seen that the inelastic dynamic analysis provides values of *C/D* more than twice the ones found through the linear analysis in almost all cases. The two inelastic analyses provide similar results in some cases (brittle mechanisms in the columns at the 1st and 2nd levels, in both directions), whereas they largely differ each other in other cases (brittle mechanism in the beams), especially in the case of seismic excitation along X-direction, due to the lack of symmetry of the building in this direction.

Figure 8 shows the values obtained for *PD*. As can be seen, *PD* exceeds 100% at each storey in some cases both for beams and columns and it almost overcomes 300% in the beams of the second storey.

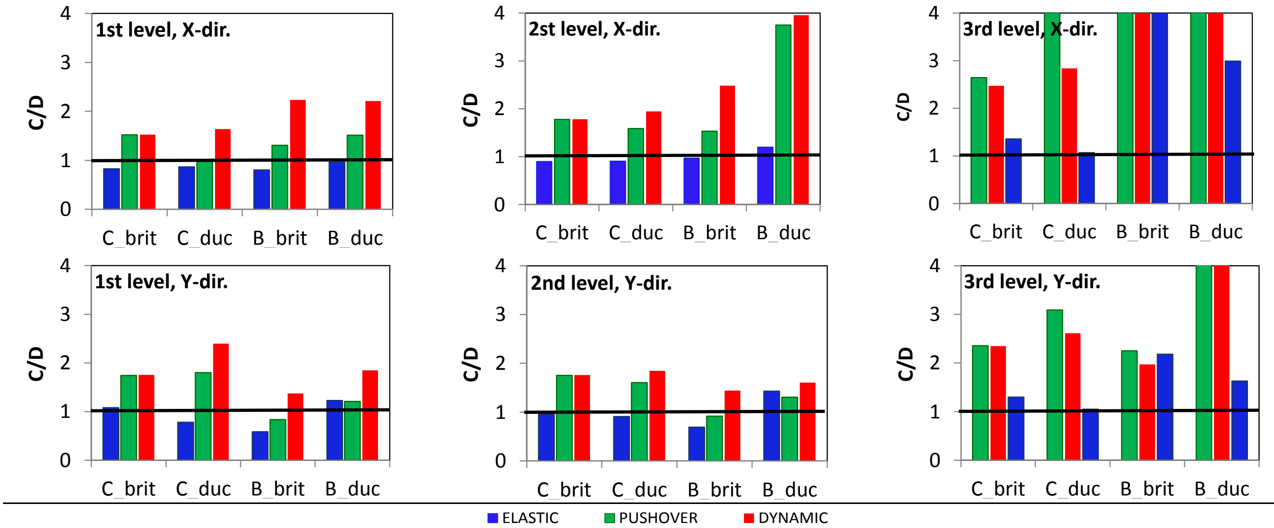


Figure 7. *SD* limit state: minimum *C/D* values found for the three analysis types.

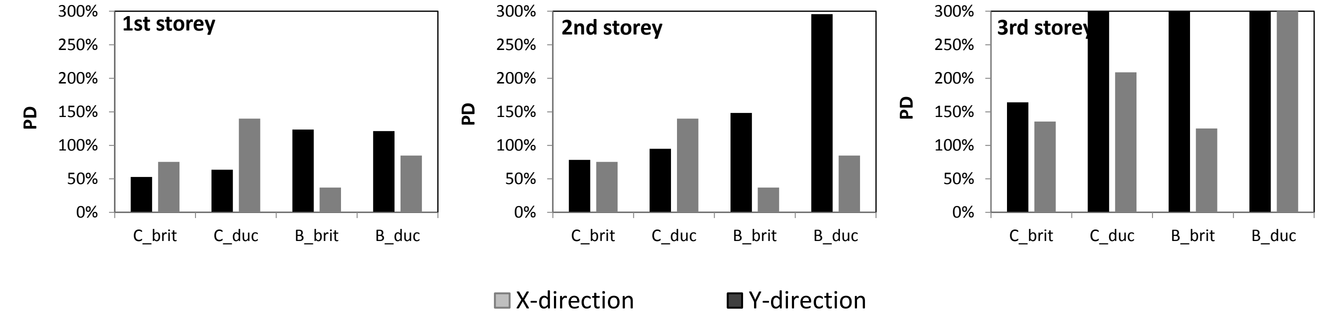


Figure 8. *SD* limit state: *PD* in the *C/D* ratio due to the type of analysis.

4.3 Effects related to the seismic input

The effect of the soil classification on the seismic response of the building has been checked both in terms of spectral acceleration (*SA*) related to the fundamental period of the building and in terms of global response, i.e. Top Displacement (*TD*) and Base Shear (*BS*). For sake of brevity, the results reported in the next sections refer to the global response of the structure related to one capacity curve only, i.e. by assuming a forces distribution proportional to the first vibrational mode without any eccentricity. The other model assumptions are specified in Tab. 5.

4.3.1 DL limit state

Table 7 shows the values of the *SA* of the elastic spectra provided by NTC 2008 for the A and B soil types along the two main directions. Table 8 reports the values of *TD* and *BS* found by intersecting the capacity curve of the building to the two elastic spectra assumed as possible seismic input. It can be seen that the *PD* related to the soil assumption is almost identical – around 40% - for all the three analysed response quantities.

Table 7. Difference in the Spectral Acceleration due to the soil assumption for the DL limit state.

	Fundamental Period sec	SA (A-soil) g	SA (B-soil) g	PD %
X-direction	0.63	0.1272	0.2171	41%
Y-direction	0.49	0.1622	0.2768	41%

Table 8. Difference in the global response of the building due to the soil assumption for the DL limit state.

	A-soil		B-soil		PD	
	TD (mm)	BS (KN)	TD (mm)	BS (KN)	TD (%)	BS (%)
X-direction	14.69	177.7	25.21	303.6	42%	41%
Y-direction	13.27	200.2	22.64	341.7	41%	41%

4.3.2 SD limit state

Tables 9 and 10 show, respectively, the differences related to the soil class assumption in terms of *SA* and global response. In this case, the three selected response quantities evidence a different sensitivity to the checked assumption. In fact, whilst the *PD* found for *SA* is equal to 36%, the one found for *TD* is around 59% in the two directions. No variation in terms of *BS* is observed and this is related to the assumption of pushover curves with an elastic-perfect-plastic behaviour, that provides no *BS* variations once the elastic response is exceeded.

Table 9. Difference in the Spectral Acceleration due to the soil assumption for the SD limit state.

	Fundamental Period sec	SA (A-soil) g	SA (B-soil) g	PD %
X-direction	0.63	0.3383	0.5291	36%
Y-direction	0.49	0.4315	0.6749	36%

Table 10. Difference in the global response of the building due to the soil assumption for the SD limit state.

	A-soil		B-soil		PD	
	TD (mm)	BS (kN)	TD (mm)	BS (kN)	TD (%)	BS (%)
X-direction	47.71	380.2	116.81	380.2	59%	-
Y-direction	35.01	526.3	86.02	526.3	59%	-

4.4. Effect related to the numerical model assumptions

The choices related to the model set-up are possibly the most discretional. It is commonly accepted, in fact, that different finite element softwares could provide some differences in the results even when the same assumptions are selected [41]. Numerical results, indeed, are sensitive not only to the subjective assumptions, but even to the selected numerical control parameters, e.g. type of analysis control (displacement-based vs force-base), convergence norms and related assumed limits, adopted numbers of points/sections of control, and so on.

Moreover, there are several choices concerning the modelling of constituent materials among a number of scientifically accepted approaches. These choices for example concern the use of a fiber approach instead of a plastic hinge approach in the modelling of reinforced concrete and the definition of all the related nonlinear parameters. For what concerns the steel reinforcements, the extracted samples confirmed the use of ribbed bars in the case-study building. Nevertheless, while dealing with existing structures, sometimes there is the need to adopt specific models in case of smooth bars, e.g. taking into account slipping of the longitudinal reinforcements and consequent pinching effect. The evaluation of the effects related to each of the possible modelling choices is well beyond the scope of this work. In the following the effects of two assumptions only have been checked, i.e. the concrete Young modulus and the joint panels behaviour. It is well known that each of these two assumptions affects a lot the estimated stiffness of the structure. The variation of E_c is dealt in this Section rather than in Section 4.1, because it does not belong to the potential variability related to the mechanical characterization of the materials, but it is due to a modelling assumption aiming at accounting for the cracking of brittle materials in the nonlinear range of behaviour.

In the following, the global response of the case-study building is evaluated in case of two different model set-ups: (1) a rigid model, characterized by a full E_c and rigid joint panels and (2) a flexible model, with 50% E_c and no rigid joint panels. In both cases, the seismic response of the case-study has been found through a nonlinear static analysis. Fig. 9 shows the pushover curves obtained for the two compared models. As can be noted, the capacity curves provided by the two numerical models are substantially different, both in terms of stiffness and ductility.

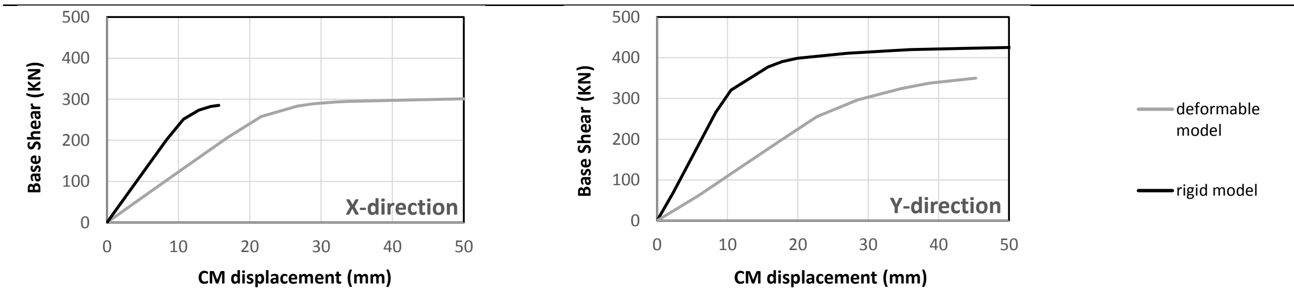


Figure 9. Pushover curves for two different model set-ups: rigid model and flexible model.

4.4.1 DL limit state

In Table 11 the global response of the case-study provided by the two models for the *DL* limit state has been reported in terms of *TS* and *BS*. In the same table, the *PD*, normalized to the maximum response, has been shown for each response parameter. As can be seen, the scatter between the two models is very high both in terms of displacement and shear, resulting around 30% along the *X*-direction and achieving 45% along the *Y*-direction.

Table 11. Difference in the global response of the building due to different choices in the model set-up for the DL limit state.

	RIGID		DEFORMABLE		PD	
	TD (mm)	BS (KN)	TD (mm)	BS (KN)	TD (%)	BS (%)
X-direction	17.74	429.4	24.12	300.2	26%	30%
Y-direction	14.58	439.9	26.67	280.4	45%	36%

4.4.2 SD limit state

Table 12 shows the response in terms of *TD* and *BS*, and the consequent *PD* associated to the *SD* limit state, provided by the two numerical models. The structural response has been found by the bilinear pushover curves represented in the ADRS plane, therefore the inelastic response of the structure, in terms of shear force, coincides to the ordinate of the horizontal inelastic branch. The scatter in the shear force between the two models is relatively low. The percentage difference, expressed as a function of the obtained TD, is equal to 32% in the X-direction and to 50% in the Y-direction.

Table 12. Difference in the global response of the building due to different choices in the model set-up for the SD limit state.

	RIGID		DEFORMABLE		PD	
	TD (mm)	BS (KN)	TD (mm)	BS (g)	TD (%)	BS (%)
X-direction	40.13	363.8	58.60	381.8	32%	5%
Y-direction	32.43	547.6	64.73	444.6	50%	19%

4.5. Synthesis of the performed analyses

In Figure 10 the *PD* values found for each assessed model assumption have been shown. The average *PD* values found for the two directions of analysis and for each limit state have been reported in the diagrams. As remarked at the beginning of this work, due to the inherent differences in the numerical procedures, different response parameters have been checked for each model assumption sensitivity study. The obtained results, therefore, are not completely comparable to each other, nevertheless the differences that will be reported give a qualitative idea of the potential effect that different modelling choices could have on the results of the numerical analysis. Moreover, the effects of single choices could combine together, possibly inducing an even larger variability in the results of the seismic assessment of the case-study building.

An overview of all the obtained results is reported in Figure 10 and, albeit with the specifications above, some conclusions can be drawn.

Firstly, the checked response parameters result to be strongly affected by the assumed modelling choices. Each model assumption, anyway, has a different impact in the seismic assessment of the case-study building. As concerns the *SD* limit state, the most relevant assumption is the type of analysis, which induces a *PD* over 70% in terms of *C/D*; the soil classification induces a *PD* around 40% in the *SA* and around 60% in the *TD*. The *PD* due to the variation of the materials design strength, related to the assumed Confidence Factors, ranges between 10% and 25%, depending on the assumed capacity parameter. Finally, the evaluated choices on the model set-up induce values of *PD* ranging from 20% to 5%, depending on the assumed parameter and direction of analysis.

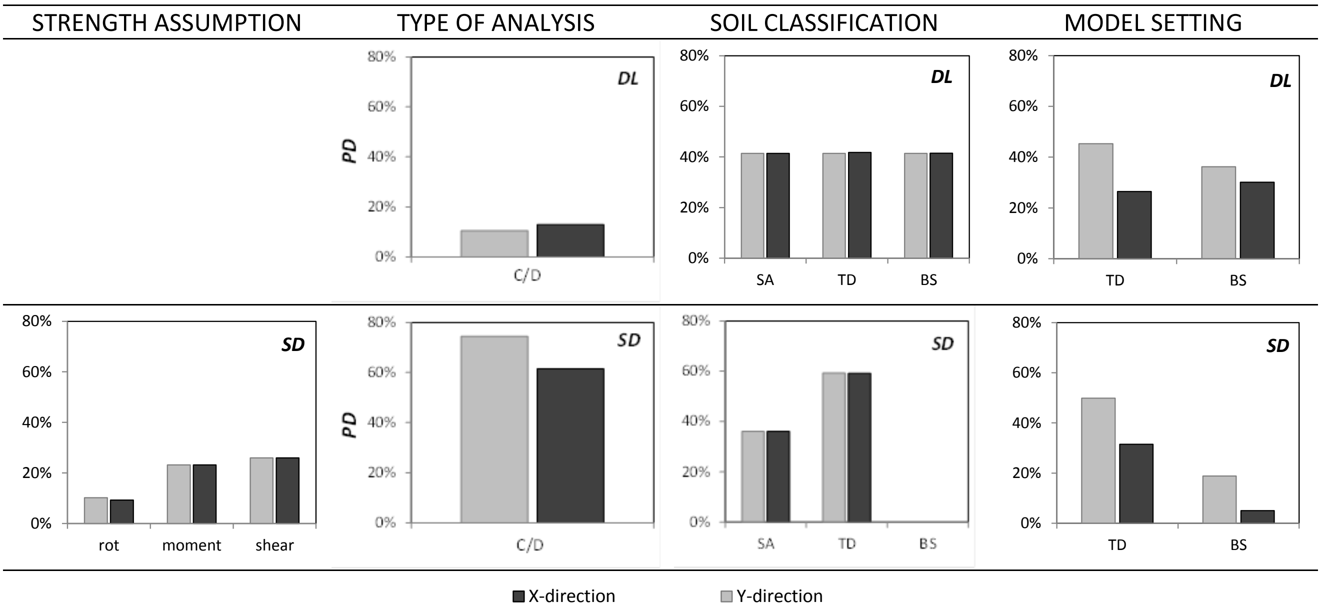


Figure 10. Average values of *PD* related to the different assumptions.

As regards the *DL* limit state, the type of analysis induces a *PD* around 10%, not resulting, therefore, a relevant choice for the seismic assessment of the case-study building. This results was to be expected, since in the *DL* limit state the seismic response is assumed to be elastic, while the type of analysis mostly affects the inelastic response of structures. The *PD* related to the soil assumption is equal to 40% for all the response quantities, due again to the assumed elastic behaviour. The *PD* related to the model set-up assumptions is equal to 26% and 30%, for the *TD* and *BS* respectively in the *X*-direction, and equal to 45% and 36% in the *Y*-direction.

5. CONCLUSIVE REMARKS AND RECOMMENDATIONS

In this work the effects of the possible modelling choices related to the seismic assessment of existing buildings has been checked. The study reports the results related to a case-study RC framed building, located in Italy and currently used as a hospital. Some of the most important model assumptions have been identified and subjected to sensitivity studies. More precisely, the design strength assumed for the materials, the type of performed numerical analysis, the seismic classification of the soil and the set-up of the numerical model have been taken into account.

The work was aimed at evaluating the potential effects on the results of the seismic assessment, of some of the most common modelling and analysis assumptions that need to be made by the engineer in charge of the assessment. There is of course a multitude of possible assumptions, all compatible to the EC 8-3 [1] instructions, that can be made and would need such sensitivity studies. The present work does not claim to give a comprehensive overview of the effects of all the possible assumptions, but aims at putting a spotlight to potential variations that some too discretional prescriptions of the technical codes can induce in the seismic assessment of existing structures. The performed sensitivity studies did not include any specious assumption: all the assumed parameters were supported by the collected experimental data and consistent to the effective information on the case-study building. Moreover all the assumptions resulted to be consistent with the EC 8-3 [1] prescriptions.

The major result of this work is that the sensitivity studies on the assumed parameters, all reported a not-negligible, and in some cases relevant, effect on the results of the seismic assessment.

As regards the *SD* limit state, the *PD* obtained in the performance of the case-study due to the type of analysis, measured as the ratio between the capacity, *C*, and the demand, *D*, exceeds 70%. The soil classification, in turn, induces a *PD* around 40% in the *BS* and around 60% in the *TD*. The *PD* related to the variation of the materials design strength, associated to the assumed Confidence Factors, ranges between 10% and 25% depending on the assumed capacity parameter. Lastly, the scatter related to some choices in the model set-up is very high, with a consequent *PD* ranging between 20% and 5%, depending on the assumed response parameter and direction of analysis.

Concerning the *DL* limit state, as it was expectable, the effect of the variations reported a lower sensitivity. The type of analysis induces a *PD* around 10%, not resulting, therefore, a relevant choice for the seismic assessment of the case-study building. In the *DL* limit state, in fact, the seismic response is assumed to be elastic, whereas the type of analysis affects mostly the inelastic response of structures. The *PD* related to the soil assumption is equal to 40% for all the considered response quantities, while the *PD* related to the model set-up assumptions is equal to 26% and 45%, for the *TD* and *BS* respectively in the *X*-direction, and equal to 30% and 36% in the *Y*-direction.

The results presented in this paper are representative of the analysed case-study building only, but point out the effect that some too discretional prescriptions of the technical codes can have on the seismic assessment of an existing building. The evaluation of the safety of the building can potentially be largely affected by the assumptions selected by the analyst, even if they are all in compliance with the prescriptions of the EC 8-3 [1]. In order to achieve a safe evaluation of the seismic response of the building, all the choices should be made on the safe side. However, it’s worth observing that selecting all the safest assumptions could lead to over-conservative results, having of course administrative and economic consequences. Especially when dealing with strategic buildings or buildings that have public functions, a reliable and effective assessment of their structural safety, rather than a too conservative one, would be desirable. Therefore the results reported in the current study highlight a very important issue to be further explored and assessed. Moreover, due to the discretion left in several points of the technical codes, this study points out the importance of an experienced engineering judgment, able to properly evaluate the more suitable model assumptions, while performing the seismic assessment of an existing building. The performance of some support analyses with variation studies is highly recommended to get some understanding of the upper and lower bounds of the results. This approach would help the analyst to make the best choice in the final set of assumptions, finding a balance between accuracy of the results, complexity of the analysis and aim of the specific assessment.

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