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THE ROLE OF STRUCTURAL MODELING ON THE SEISMIC ASSESSMENT OF EXISTING RC BUILDINGS ACCORDING TO EUROCODE 8

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ABSTRACT

In these years, due to the construction peak in the past decades and to the unfavorable economic situation, the choice to rehabilitate, restore and retrofit existing buildings is preferred to their reconstruction whenever it is possible. The seismic assessment of existing buildings, therefore, is an essential issue of the current Technical Codes.

This work is focused on the evaluation of the seismic performance of existing RC buildings, and more precisely, with the role of their modeling on such evaluation. The work is made on a case-study, that is a real, RC hospital building. An accurate knowledge of the building has been achieved, as a result of a collaboration between the University of Florence and the Regional Government. All the main information about material, soil, design and architectural features have been collected. Two different analytical procedures provided by the Eurocode 8, integrated to the Italian Technical Code (NTC 2008) have been applied and compared. The analysis has shown the scatter in seismic assessment referring to the possible modeling choices.

INTRODUCTION

The evaluation of the seismic performance of existing buildings is a crucial issue of the seismic engineering. In fact, due to the current economic crisis and to the intense construction activity of the last decades, the current building heritage includes many buildings that do not comply with the technical requirements provided by the Codes in force.

In many cases, demolition and successive reconstruction would be the easier and more logical choice, since existing buildings, mostly done in the 60s and 70s, are inadequate under different aspects beyond structural safety. In most cases, however, the lack of money prevents this choice. Retrofitting and rehabilitation, in these cases, would be the most feasible solution. Nevertheless, it is important to quantify the eventual inadequacy of the structure, in order to evaluate the potential success of the rehabilitation, and to properly predict the required investment.

Due to its intrinsic complexity, the evaluation of seismic performance is affected by many uncertain factors, whose precise definition is very hard to achieve, like the intrinsic variability of the quantities involved in the seismic response of the building (material, soil, acting loads) and the analytical procedure adopted in analysis. The current Technical Codes, in fact, let the designer to

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choose the type of analysis, according to some guidelines. It is well known that the choice about material behavior (linear or nonlinear) and seismic analysis modeling (static or dynamic) affects very much the performance prediction; nevertheless, the Codes admit any choice as acceptable, while respecting certain requirements, so that different evaluations can be equally admitted for the same building.

This work deals with the effects of modeling choices on the seismic performance evaluation of existing buildings. It is focused on a case-study, i.e. a real RC building, currently used as an hospital. The evaluation of its seismic performance is based on a wide knowledge process, that is the result of a joint agreement with the Regional Government of Tuscany. A fully satisfactory knowledge of the building has been achieved; all the documents describing the original design and the foundation soil have been studied and a large number of experimental tests have been performed to evaluate material mechanical properties.

The evaluation of the seismic performance has been made according to the Eurocode 8 (EC8) prescriptions. EC8 is the European Code which rules seismic assessment of existing buildings, and it is integrated by specification introduced by the National Codes, which eventually introduce some changes in detail aspects. In Italy the current Technical Code is NTC 2008, which is in this work assumed as reference as regards procedures and assessment evaluation; the main differences between the Italian (NTC 2008) and the European (EC8) Codes about seismic assessment of existing buildings have been evidenced. An ultimate limit state, which has the same probability of occurrence according to the two Codes, has been assumed for analysis. The results obtained for the seismic assessment of the case-study by performing two different analytical procedures are shown and compared. The comparison has evidenced the influence of the applied analyses on the seismic assessment of the case-study.

SEISMIC PERFORMANCE EVALUATION ACCORDING TO EC8 (AND NTC 2008)

The seismic performance of a building can be defined as its capacity to satisfy assumed safety conditions after a ground motion of assigned intensity. The correct evaluation of the seismic performance, therefore, involves 1) the seismic hazard, i.e. the level of seismic intensity expected in a specific area, 2) the Limit States, with the consequent quantification of safety requirements, and 3) the structural capacity, that is a function of the geometrical and mechanical features of the considered building.

The Italian Technical Code, NTC 2008, has been developed according to the European guidelines and it basically coincides with the corresponding Sections of Eurocodes. Nevertheless, it presents, comparing to Eurocodes, some minor differences (De Luca et al. 2011 a,b) which will be underlined in the next paragraphs. Special attention has been paid to the specifications regarding existing buildings, which require additional attention concerning to two main aspects, i.e. the lack of ductility and the uncertain material properties definition.

Ductility, achieved in new buildings through the “capacity design”, is one of the main key-factors in the modern performance-based design. Existing buildings usually have been designed without any attention to the strength hierarchy and therefore, they can experience brittle failures more than ductile ones. The evaluation of the seismic performance of existing RC buildings, therefore, must be pursued by checking, separately, ductile and brittle failure mechanisms.

As regards the material properties definition, it is well known that existing buildings can exhibit very variable mechanical properties, even within a single building (De Stefano et al. 2013a,b; 2014). EC8 underlines the importance of the knowledge process of the building as preliminary and essential step of the analysis. Depending on the level of the knowledge, a different value of safety factor (Confidence Factor, *CF*) is assumed to reduce the *mean* value of material strength to use in the subsequent verification process.

Seismic input and Limit States. The expected maximum seismic intensity of the area is provided by a proper seismic hazard analysis conducted by each country and measured in terms of Peak Ground Acceleration (PGA). The elastic spectrum of the ground motion, i.e. the trend of maximum acceleration as a function of the vibrational period of the structure, can be defined – as a function of the building and soil properties – for each considered local seismic hazard and limit state.

EC8 and NTC 2008 define similar, but not identical, limit states. The probabilities of occurrence of the seismic event in a 50-years reference period related to each limit state, for the two Codes, are listed in Table 1. As can be seen, only the less severe Ultimate Limit State (*SLV* according to NTC 2008, and *SD* according to EC8) coincides for the two Codes.

Table 1. Probability of exceedance related to each Limit State

EC8		NTC 2008	
Limit State	Prob. of exceedance	Limit State	Prob. of exceedance
-	-	Operational	81% in 50 years
Damage Limitation	20% in 50 years	Damage Limitation	63% in 50 years
Significant Damage	10% in 50 years	Life Safety	10% in 50 years
Near Collapse	2% in 50 years	Collapse Prevention	5% in 50 years

Another difference between EC8 and NTC 2008 concerns the elastic spectrum definition. In fact, while in EC8 the elastic spectra of the same building for the different limit states can be simply scaled, NTC 2008 provides (slight) different shapes of the spectra for each limit state. In the Italian Technical Code, indeed, even the period at the beginning of the constant-velocity branch is related, together to the peak ground acceleration and the amplification factor, to the building geographical site.

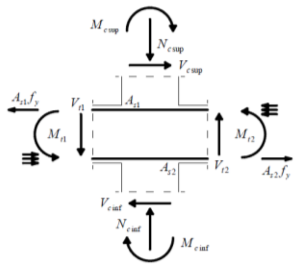
Seismic response. The seismic response can be found by performing a structural analysis, whose reliability can largely vary depending on the hypothesis concerning the material behavior (elastic or inelastic) and the type of analysis (static or dynamic) performed on a finite element model of the structural system. According to both EC8 and NTC 2008, the *mean* value of concrete strength must be used in analysis, while a reduced (until 50%) value of the Young modulus, to take into account the possible viscous phenomena, can be assumed.

Structural capacity. In existing buildings ductile and brittle members and mechanisms must be distinguished and checked separately. The ductile mechanisms are related to the flexural behavior and they are checked in terms of bending moment in case of elastic analysis, and chord rotation in case of inelastic analysis. The limit values provided by the Code for the chord rotation are fixed for each limit state (see Table 2). The brittle mechanisms are related to shear failure. Verification, made in terms of force, must be done for ultimate limit state only and it involves each structural element, including the joints. The joint verification, in particular, is a crucial issue, since it must be performed for all not-confined joints. The tensile and compressive capacity of the joint can be determined (see Fig. 1) according to eq. 8.7.2.2 and 8.7.2.3 (Circ. No. 617, Section C.8.7.2.5), as a function of the joint geometry, the amount of axial load in the top column, the total shear in the joint and the stress level of the longitudinal reinforcement of the beams adjacent to the joint.

While EC8 lets the designer to choose the ultimate limit state to be checked (between Life Safety and Collapse Prevention), NTC 2008 defines the *SLV* as the mandatory ultimate limit state to be verified. Both Codes indicates a verification procedure to be used in the joints based on the equilibrium between the forces of the structural elements converging in the joint. The structural capacity of each member is evaluated by assuming, for concrete and steel strength, a conventional (reduced) value, in general not coincident to the *mean* value coming from the *in-situ* experimental tests. EC8 (and NTC 2008), in fact, gives a central role to the knowledge process of the building. All information concerning its construction, including documents, architectural and structural design, eventual changes made during the building life, must be considered and integrated by direct observation and experimental tests on structural materials. Depending on the achieved Knowledge Level (*KL*) of the building, a different value must be assumed for the safety factor (Confidence Factor, *CF*) to reduce the *mean* strength value coming from the *in situ* tests. When an exhaustive knowledge level (*KL*=3) is achieved, a *CF*=1.00 can be assumed, and therefore the *mean* strength value can be adopted. When *KL*=2 (good knowledge), *CF* is assumed equal to 1.20, while for a moderate knowledge (*KL*=1) a *CF*=1.35 must be assumed. Depending on the type of verification (brittle or ductile mechanisms), the strength value must be reduced by the Confidence Factor only, or by the combination of *CF* with the safety coefficient γ_c=1.5 and γ_s=1.15 for concrete and steel respectively, adopted to find the conventional strength for new buildings.

Table 2. Limit chord rotations for each limit state.

	EC8			NTC 2008			
Limit state	*DL*	*SD*	*NC*	*SLO*	*SLD*	*SLV*	*SLC*
Limit condition	θ_y	3/4 *NC*	θ_u	2/3 *SLD*	θ_y *or 0.5h*	3/4 θ_u	θ_u



$$\sigma_{nt} = \left|\frac{N}{2A_g} - \sqrt{\left(\frac{N}{2A_g}\right)^2 + \left(\frac{V_n}{A_g}\right)^2}\right| \le 0.3\sqrt{f_{cd}} = \sigma_{lim,t}$$

$$\sigma_{nc} = \frac{N}{2A_g} + \sqrt{\left(\frac{N}{2A_g}\right)^2 + \left(\frac{V_n}{A_g}\right)^2} \le 0.5 f_{cd} = \sigma_{lim,c}$$

A_g = gross area of the joint;
N = axial load in the top column;
V_n = total shear in the joint;
f_{cd} = mean strength reduced by *CF* and γ_c.

Figure 1. Scheme of the joint verification.

CASE-STUDY

The evaluation of the seismic performance of the building belongs to an agreement between the University of Florence and the Tuscan Regional Government. A wise and comprehensive knowledge of the building has been achieved; all original documents of the project have been found and compared to the building as it currently is. Many experimental tests, destructive and not, have been made to evaluate the mechanical properties of the materials. In this Chapter the main information of the building have been reported.

General features and geometrical description. The building, having a RC skeleton, has been deigned in 1976, i.e. just after the introduction of the first seismic Italian Technical Code. In Italy the first general Code regarding reinforced concrete buildings was issued in 1971, while the first seismic code was issued in 1974. The building presents some efficient design criterions, like column section reduction from foundation level to the top storey, or solid connection of the beam-column joints, but it is far away from the current seismic design criteria. The 3-storey building, shown in Figure 2, has a regular (rectangular) plan, with one symmetric axis only. In Table 3 the main dimensions of beams and columns are listed. Some beams, as can be seen even in Fig. 2, have a Z-shape; they have been represented by assuming an equivalent area, with an equivalent moment of inertia. It should be noted that the third floor of the building consists of two different structural layers, partially coinciding. In fact two different floors, 40 cm far away each other, constitute the last storey of the building. Such floors are supported by two different order of beams (*x*-direction: column lines *x3*, *y*-direction: column lines *y1* and *y3*), and by one beam only (*x*-direction: column lines *x1* and *x2*, *y*-direction, column lines *y2*) depending on the different in-plan position. Therefore, beams of the 3rd storey have been indicated with *a* and *b* depending of the layer they belongs to.

The effective reinforcements have been considered both at the end and at the mid-span of each beam, therefore different capacities have been obtained for different sections of the same member. All floors are made by deck and concrete, and they have a total depth of 20 cm. The infill panels have a double layer, with an inside casing.

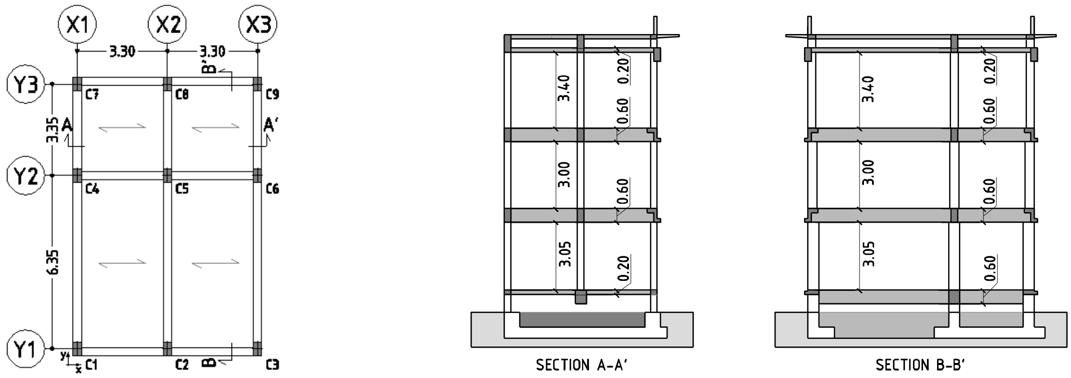


Figure 2. Plan and sections.

Table 3. Columns and beams dimensions.

	COLUMNS	BEAMS						
		X1	X2		X3	Y1	Y2	Y3
1st st.	30x50	30x60	30x60		Z-shape	Z-shape	30x60	Z-shape
2nd st.	30x40	30x60	30x60		30x60	Z-shape	30x60	Z-shape
3rd st.	30x35	30x80	30x80	a)	30x60	30x60	30x80	30x60
				b)	30x20	30x20		30x20

Materials. It is well known that existing buildings can exhibit material with very different mechanical properties with respect to those assumed in the original project (Cristofaro 2009, 2012, De Stefano et al. 2013a, b, 2014). Mechanical properties of structural materials, i.e. concrete and steel, have been determined through destructive and not-destructive tests, according to the Code provisions (see Circ. n. 617 del 02-02-2009 – Tab. C8A.1.3).

Concerning the concrete characterization, 3 destructive tests have been performed on the columns, which have been integrated by other SonReb (sclerometric + ultrasonic methods) tests (see Tab. 4), which have been extended even to the beams, in order to obtain an overall evaluation of the concrete strength. Even the structural details have been properly investigated, since 11 destructive and 10 not destructive tests (see Tab. 4) have been carried out. Results of the experimental tests have been separately found for SonReb (R.Giacchetti e L.Lacquaniti,1980; J.Gasparik, 1984; A.Di Leo e G.Pascale, 1994) and destructive (BS 1881 Part.120, 1983; Concrete Society, 1976; Cestelli-Guidi e Morelli 1981, Masi,2005) tests. The final compressive strength has been found by combining both destructive and Sonreb results, by adopting an *ad hoc expression* (Cristofaro, 2009) based on formulation (Masi, 2008), i.e. a non-linear regression which has provided a final cylindrical strength, $f_{c,mean}$ equal to 10.2 MPa. The global knowledge level achieved for the concrete structural elements has been conservatively evaluated as *KL2* (*CF*=1.20), according to EC8 standard, so that a design (reduced) value of strength, f_{cd}, equal to 8.5 MPa has been found (see Tab. 4). The elastic modulus assumed in analysis has been found by the eq. 11.2.5 (NTC2008, Section 11.2.10.3), based on the value of $f_{c,mean}$. The reinforcement steel, according to the structural design, belongs to the FeB32K class, having a yield stress over 32 MPa and a ultimate stress over 50 MPa. By a visual inspection, two different types of steel have been used, respectively ribbed and not. Three destructive tests, one for each storey, have been done on the rebars, according to the standard procedure (UNI EN ISO 6892-1, 2009), returning a *mean* value, $f_{s,mean}$, equal to 10.2 MPa. For the steel, a Klowledge Level *KL1* has been assumed, with a consequent Confidence Factor equal to 1.35. Information about the steel tests are listed in Tab. 5.

Table 4. Number of visual inspections and testings for concrete.

storey	OBSERVATION			nondestructive test (SonReb)		destructive test		stress value (KN/mm^2)			
	columns	beams	nodes	columns	Beams	columns	beams	test	$f_{c,\,mean}$	*KL*	$f_{c,d}$
1st st.	2	4	2	2	2	1	-	11.5	10.2	*KL2*	8.5
2nd st.	2	5	3	2	2	1	-	64.2			
3rd st.	2	6	6	2	-	1	-	7.6			

Table 5. Number of visual inspections and tests for steel.

storey	Yield load (KN)	Ultimate load (KN)	Yield stress (KN/mm^2)	*mean* yield stress (KN/mm^2)	*KL*	Design stress (KN/mm^2)
1^{st} st.	571	857	391.23			
2^{nd} st.	806	1158	392.91	385.72	*KL1*	285.72
3^{rd} st.	765	1071	373.02			

The soil. Ground behavior has been determined through a geophysical site investigation. In order to gain an accurate estimate of the subsoil structure, seismic refraction and down hole techniques have been carried out. The results, processed using both Generalized Reciprocal Method (Palmer, 1981) and Tomographic one, have provided detailed information on the distribution and thicknesses of subsurface layers, rock dynamics and geo-mechanical properties, seismic *P* and *S* waves velocity profiles and the consequential value of the average shear wave velocity $V_{s,30}$ (Sirles and Viksne, 1990), which is equal to 390 m/s. Therefore the soil has been assumed as B-type.

The structural model. The analyses have been performed by using the computer code SAP2000 (http://www.csi-italia.eu/software/sap2000/). The two floors of the third levels have been modeled according to the real geometry as regards the stiffness and strength distribution, while the mass (both translational and rotational) of the storey has been considered applied at the center of the storey package. The two levels are named 3a and 3b in the following. The effect of the joint stiffness has been considered by introducing a rigid offset at each element end. A reduced value of the Young modulus of the concrete E_c, i.e. $E_{c,red} = 0.5\ E_c$, has been assumed, as suggested by NTC 2008. The floor stiffness has been introduced by assigning the diaphragm constraint to all nodes belonging to the same floor. Each member as an elastic element with terminal plastic hinges, whose properties have been defined by assigning a simplified (bi-linear) moment-rotation relationship, defined by the yield and the ultimate points. Limit values of bending moment and rotation (eq. C8.7.2.1.a and eq. C8A.6.1 in Circ. No 716) have been made according to the Code prescriptions.

The structural capacity. In this work the ductile capacity has been expressed in terms of ultimate moment and chord rotation, depending on the type of analysis. Fig 3 shows the values found for beams and columns ultimate moment, together with the limit values of the chord rotation for the considered limit state (*SLV*).

The ultimate shear of beams and columns is shown in Fig 3. For joints (see symbols recalled in Fig.1), two limit values, respectively referred to tensile $\sigma_{lim,t}$ and compressive $\sigma_{lim,c}$ stress, have been considered, defined as:

$$\sigma_{lim,\,t} = 0.3\ (f_c)^{0.5} = 0.78\ \text{MPa} \tag{1}$$

$$\sigma_{lim,\,c} = 0.5\ f_c\ = 3.41\ \text{MPa} \tag{2}$$

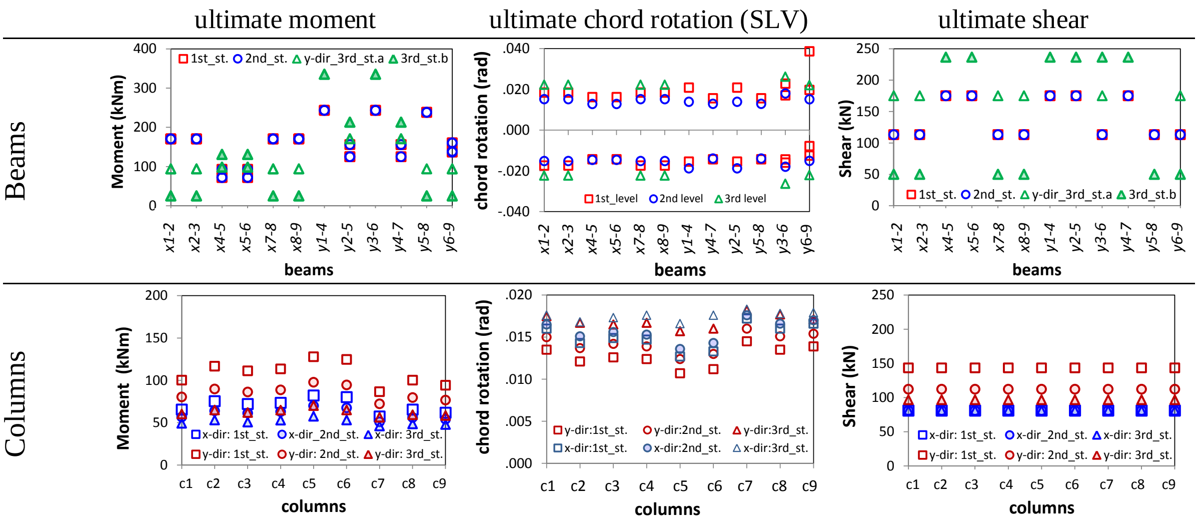


Figure 3. Capacity of beams and columns.

The seismic input. The seismic input has been defined according to the site seismic classification (NTC 2008, All. B). The parameters assumed to find the elastic spectrum refer to the soil-type B. In this work only the spectrum representing the *SLV* limit state has been considered. Both the elastic spectrum and the scaled one have been considered. For the scaled spectrum two different q-factors have been used, for ductile (q=3.0) and brittle (q=1.5) mechanisms.

SEISMIC PERFORMANCE OF THE CASE-STUDY

The seismic performance has been assessed, with reference to the *LS* (i.e. *SVL*) limit state, as the ratio between the capacity (C) of each component (beams, column and joints) of the case-study and the corresponding demand (D). Both ductile and brittle mechanisms have been considered, according to the Code prescriptions. The demand has been quantified by performing two different analyses, respectively elastic ("pseudo-dynamic" elastic) and inelastic (nonlinear static analysis). Both the *x* and *y* directions have been considered in analysis. In the pseudo-dynamic analysis, the effects induced by the seismic action in the considered direction have been combined to 30% of the one coming from the other direction.

A preliminary modal analysis has been performed to determine the dynamic properties of the structure. In Table 6 the main information related to the first 5 modes are listed.

Table 6. Dynamic properties of the considered

mode	Period (sec)	Participant Mass *x*-direction (%)	Participant Mass *y*-direction (%)
1	0.628	≅0	87.20
2	0.494	80.72	≅0
3	0.380	0.18	0.51
4	0.226	≅0	9.45
5	0.182	11.05	≅0
1+2+3+4+5		92.95	97.18

The pseudo-dynamic linear analysis. The seismic response of the case-study has been evaluated by considering the values of the spectral acceleration for the five periods coming from the modal analysis, and then combining the consequent effects according to their participation factors.

The acceptance conditions of the linear analysis have been checked both by considering the elastic and the reduced spectrum; it has been found that such conditions are satisfied only for reduced spectra, which have, therefore, selected for analysis; a *q*-factor equal to 3.0 has been used for ductile mechanisms, while a *q*-value equal to 1.5 has been used for brittle ones. The seismic performance of the case-study has been found by comparing the maximum value of force achieved in each member to the corresponding capacity. The shear values have been used to find the maximum tensile and compressive stress in the joints. The ductile mechanisms, i.e. the flexural ones, have been checked in terms of bending moment.

Figure 4 shows, both for *x* and *y* direction, the comparison between capacity and demand of the checked parameters, i.e. bending moment (ductile mechanism) and shear (brittle mechanism) for beams and columns. The joints have been verified by comparing the maximum tensile and compressive stress with the corresponding resistances. In all cases the verification is satisfied when the ratio D/C is over the unity.

When the brittle mechanisms are considered in beams and columns, the structure is almost verified in all cases. Moreover the capacity, in some cases, exceeds so much the corresponding demand that for the D/C ratio a logarithmic scale has been chosen.

The joint verification, instead, evidences different results. The joint capacity, in fact, is much lower than the corresponding demand in many cases. All joints of the case-study have been considered as not-confined. In fact, most part of them were side joints, and even in the internal one (Joint 5) the required geometrical conditions are not verified.

Nonlinear static analysis. The nonlinear static analysis has been performed by intersecting the capacity curve to the design spectra. Two different height-wise patterns, respectively proportional to the first vibration mode and to mass distribution along the building height (1^{st} mode and mass proportional distributions) have been considered, according to the Code provisions (§7.3.4.1). Here below, for sake of brevity, only the results referring to the 1st mode distribution have been shown.

Each capacity curve has been modified by representing the multi degree of freedom system by a bi-linear one-degree of freedom system, according to the EC8 prescriptions. Each limit condition considered for the *LS* limit state, referring both to ductile and brittle mechanisms, has been considered as ultimate limit for the structure, and represented on the pushover curve. When one member (beam, column or joint) attains its limit condition, the whole structure has assumed to have achieved the assumed limit state. The achievement of the limit state can, in this way, be indicated in the pushover curve, in order to have a global control of the structural behavior.

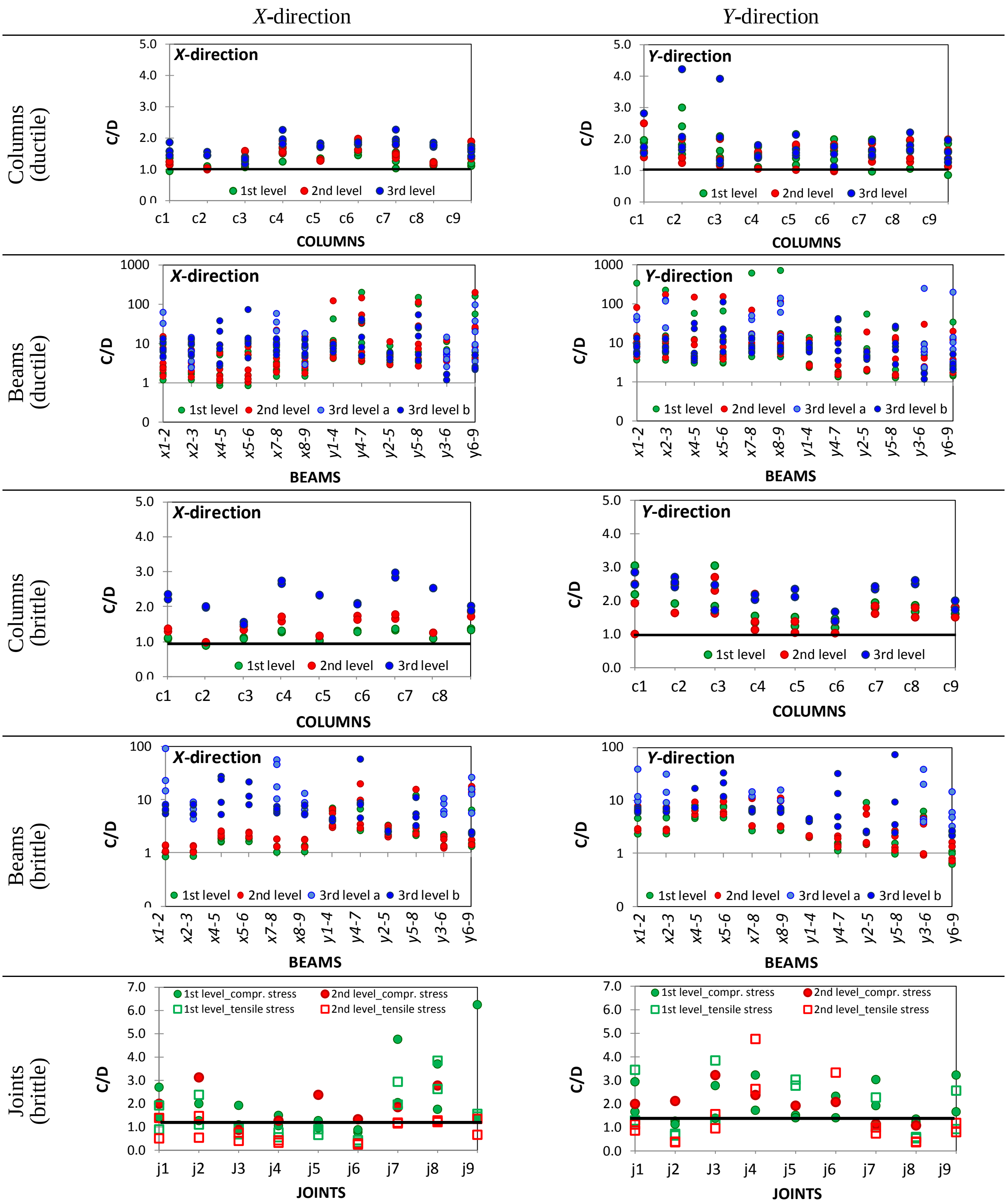


Figure 4. Linear analysis: seismic performance (C/D) for the checked quantities.

Fig 5 shows the capacity curves of the case-study along the two directions, intersected to the SLV spectrum. On the capacity curves, the steps corresponding to the attainment of each limit condition have been evidenced.

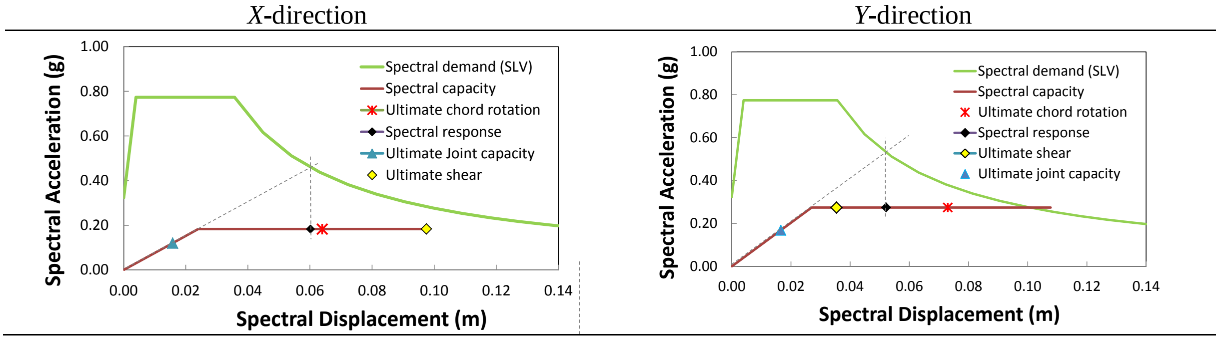


Figure 5. Capacity curves and seismic response for the *LS* limit state.

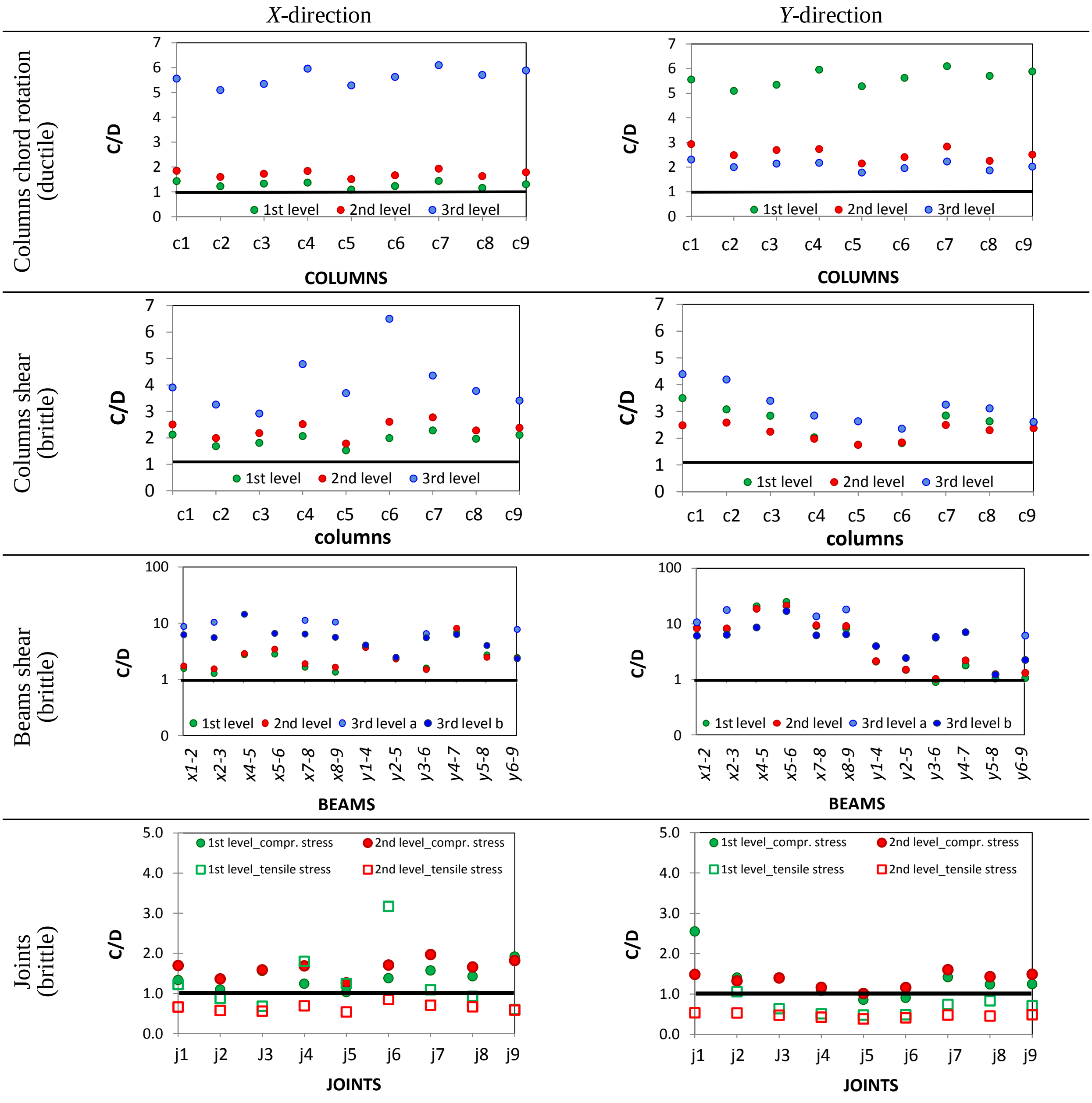


Figure 6. Non-linear analysis: seismic performance (C/D) for the checked quantities.

The structural response is given by the abscissa of the intersecting point between capacity curve and design spectrum. In the example shown in Fig. 5, the structural capacity satisfies the demand with regards to the ductile mechanisms, while it does not, for what concerns the joints verification. Concerning the shear capacity, the brittle mechanism is verified in the *x*-direction, while it is not verified in the *y*-direction. It should be noted that the joints limit condition occurs much earlier than the other ones. The limit conditions imposed to the joints, in fact, are extremely more severe than the ones referring to beams and columns.

Figure 6 shows the seismic performance, expressed in terms of the ratio of capacity over demand (C/D) found for ductile and brittle mechanisms. The ductile mechanisms have been checked in terms of chord rotation. The seismic performance, therefore, has been expressed as the ratio between the maximum chord rotation attained in the member and the corresponding limit value, as defined for the *LS* limit state. It should be noted that different scale of representation have been used for the C/D values in the different diagrams, since the clearness of the representation has been preferred to the scale uniformity.

As regards the brittle mechanisms, the maximum amount of shear in beams and columns has been shown. Verification of members results to be satisfied in almost all cases; only some beams experience values of shear lightly over the corresponding limit, for the analysis in the *y*- direction.

Even in this case, as in the results obtained from the linear analysis, the maximum stress found in the joints largely exceeds the corresponding limit. The found C/D values, in fact, are largely below the unity in many cases. It should be noted that the tensile stress, in the joints, exceeds the limit condition much more than the compression one.

Figure 7 shows the comparison between the minimum C/D ratios found by elastic and pushover analyses in each considered collapse mechanism. Higher values of C/D evidence a more conservative estimation of the seismic performance. As can be observed in Fig. 7, both brittle and ductile mechanisms in columns are largely affected by the type of analysis. As regards the final evaluation of the seismic assessment of the case-study, anyway, the type of analysis does not plays a fundamental role. In fact the seismic assessment is mostly conditioned by the joints verification, which provides similar results when performed by the two analyses.

The pseudo-dynamic linear analysis, anyway, has resulted to be more conservative than the nonlinear static analysis in the evaluation of the seismic performance. Especially for columns and joints. The difference, expressed in not-dimensional terms, between the minimum C/D values provided by the nonlinear and the linear analyses is shown in Figure 8.

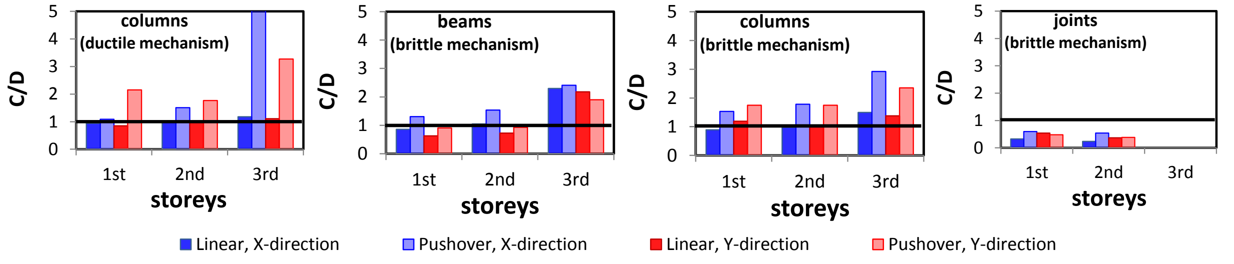


Figure 7. Comparison between the seismic performance found by the two performed analyses.

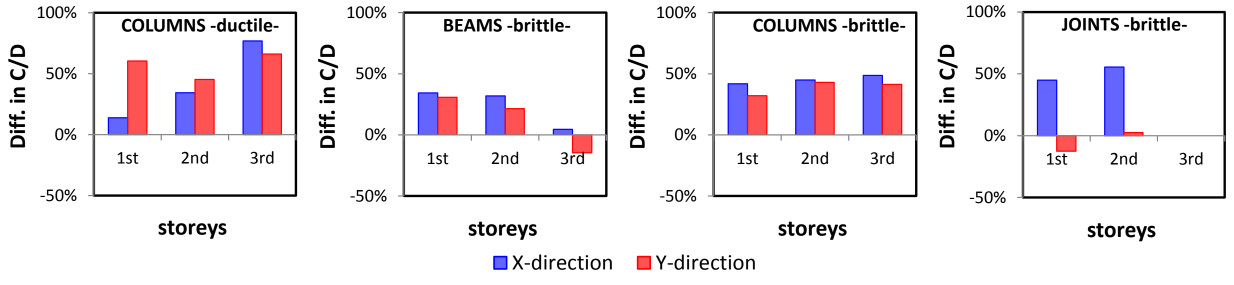


Figure 8. Not-dimensional difference (DIFF in C/D) between the C/D values found with the two analyses.

CONCLUSIVE REMARKS

In this work the seismic assessment of an existing RC case-study building has been carried out, by performing two different analyses, both admitted by the current European Technical Code (EC8): the pseudo-dynamic elastic analysis and the static non-linear analysis. The case-study is a real RC existing building, currently used as an hospital. An exhaustive knowledge process has been performed on the case-study; all documents regarding the original project have been collected and a survey on material, including a large number of tests (destructive and not) has been done. The Knowledge Level of the case-study has been assumed, conservatively, equal to 1 for reinforcement and equal to 2 for the concrete.

The Life Safety (*LS*) limit state has been considered in the seismic assessment, and the structural capacity has been consequently found for each member, both referring to ductile and brittle mechanisms, according to EC8 prescriptions. The seismic input has been represented thorough the *LS* spectrum, on the base of the seismicity of the area and assuming a soil-type B.

The seismic response of the case-study has been found by performing the two selected analyses. The demand in each member has been compared to the corresponding capacity and therefore the seismic performance, expressed in terms of capacity over demand (C/D), has been found for each member, both for ductile and brittle mechanisms.

The obtained results have shown two main evidences: i) the limit conditions regarding the not-confined joints proved to be more strict than the ones imposed to the members, and ii) the pseudo-dynamic linear analysis is more conservative than the pushover analysis, especially as regards the seismic performance of columns and joints.

As regards the seismic verification of the case-study, very similar C/D values have been found for ductile and brittle mechanisms, both for beams and columns. Values of C/D found for joints, instead, are at least one order of magnitude lower than the ones found for beams and columns. From the results obtained on the case-study, the limit conditions imposed to joints, therefore, are more severe than the ones assumed for beams and columns.

As regards the role played by the type of analysis on the evaluation of the seismic assessment, the two analytical approaches have provided the same evaluation, i.e. the building to be not respectful of the safety conditions as provided by the Code. Nevertheless, the difference in C/D values provided by the two analyses is not negligible, resulting on the order of 50% in columns (brittle and ductile mechanisms) and in joints.

Further analyses should be needed to achieve more general results about the relationship between the safety conditions of the joints compared to the ones of the other structural members. Besides, since the selected analytical procedure proved to largely affect the seismic assessment of the case-study, a more comprehensive investigation should be done, to include the nonlinear dynamic analysis in the comparison, and to consider different case-studies in the evaluation.

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