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Seismic performance sensitivity to concrete strength variability: a case-study

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Abstract. Existing building structures can easily present material mechanical properties which can largely vary even within a single structure. The current European Technical Code, Eurocode 8, does not provide specific instructions to account for high variability in mechanical properties. As a consequence of the high strength variability, at the occurrence of seismic events, the structure may evidence unexpected phenomena, like torsional effects, with larger experienced deformations and, in turn, with reduced seismic performance. This work is focused on the reduction in seismic performance due to the concrete strength variability. The analysis has been performed on a case-study, i.e., a 3D RC framed 4 storey building. A Normal distribution, compatible to a large available database, has been taken to represent the concrete strength domain. Different plan layouts, representative of realistic strength distributions, have been considered, and a statistical analysis has been performed on the induced reduction in seismic performance. The obtained results have been compared to the standard analysis as provided by Eurocode 8 for existing buildings. The comparison has shown that the Eurocode 8 provisions are not conservative for existing buildings having a large variability in concrete strength.

Keywords: RC framed structures; plan irregularity; torsional effects; concrete mechanical properties; RC existing buildings; seismic performance

1. Introduction

The evaluation of the seismic reliability of existing structures is one of the most important issues of current effort in earthquake engineering. In recent years, many Authors have investigated the concrete strength characterization, focusing their attention both on the choice of the parameters to assume in the characterization (D'Ambrisi *et al*. 2013b, Masi *et al*. 2009) and on the variability of the strength (De Stefano *et al*. 2013a), with the consequent assumption of a suitable value for analysis (Franchin *et al*. 2007, 2009, Fardis 2009, Masi *et al*. 2008, Rajeev *et al*. 2010, Jalayer *et al*. 2008, Cosenza *et al*. 2009, Marano *et al*. 2008, Monti *et al*. 2007, D'Ambrisi *et al*. 2013a, b).

In this paper the concrete strength variability has been investigated on a case-study as source of in-plan irregularity. The case-study is a 4-storey RC building, representing a simple and typical.

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example of pre-seismic normative structure, designed to vertical loads only, according to the allowable stress criterion. The concrete strength has been characterized by using a database provided by the Regional Government, which is very large and includes buildings made in different decades, ranging between 1950 and 1980, in a homogeneous area. Three different amounts of concrete strength variability, as represented by the Coefficient of Variation (*CoV*) equal to 15%, 30% and 45%, are introduced at the columns belonging to first storey only, while the other columns, as well as all the beams, are characterized by the *mean* value of the strength domain. A sample of 180 different in-plan layouts has been considered to represent the possible strength distributions, which has been shown in previous paper (De Stefano *et al*. 2015a).

Due to the introduced strength variability, a stiffness and strength eccentricity arises at the first storey of the structure. In De Stefano *et al*. (2015a) the amount of eccentricity arising from each layout has been studied and related to the consequent torsional effects. This work, instead, investigates the effect of the strength variability on the seismic performance. While the seismic response of the case-study is related to the strength variability through the associated eccentricity only, the seismic performance is affected by the combination of such eccentricity and the capacity of each column, which, in turn, depends on its concrete strength.

The results presented in this work have been organized in three different sections. In the first one the seismic response of the case study to different intensities of ground motions is presented. Due to the considered concrete strength variability, different amounts of eccentricity are introduced in the model, and the structure experiences a torsional behavior. The seismic response of the 180 layouts representing the analyzed domain is compared to the Eurocode 8 predictions, which assume the *mean* strength for all members and introduce an accidental eccentricity of 5% in the analysis.

In the second part the capacity of each column of the case study, expressed in terms of chord rotation and shear, is presented as a function of the introduced strength variability. The limit values have been determined according to the EC8 prescriptions, by using both the values of the assumed strength domain and the conventional (reduced) values provided by EC8.

In the third part the performance of the case-study has been investigated by comparing capacity and demand, both in terms of chord rotation and shear. While the chord rotation is the reference quantity for all limit states, shear must be considered only for one of the ultimate conditions, whose choice is up to the designer. In this case the *Near Collapse* limit state has been selected for shear verification, while the acceptance criteria in terms of chord rotation have been checked in all limit states, i.e., *Damage Limitation*, *Severe Damage* and *Near Collapse*. As regards the chord rotation, the seismic performance found by introducing the strength variability is much worse than the one coming from the standard EC8 approach in all cases. At the end of the section, therefore, percentiles equal to 84% have been found for the seismic performance domains in terms of chord rotation, in order to neglect the most unfavorable in-plan layouts considered in analysis. The 84% percentile values of performance have been compared to the conventional values found according to EC8 instructions.

The comparison evidenced that the EC8 approach for the seismic performance evaluation, including the 5% accidental eccentricity, does not cover the effects due to the considered strength variability.

2. The analysis

2.1 Concrete characterization

2.1.1 The EC8 prescriptions

According to EC8, the *mean* value of the concrete strength (f_c) should be used in the seismic analysis, while a reduced strength, equal to the *mean* divided by a Confidence Factor (*CF*) should be adopted for verification. The value of *CF* depends on the knowledge level (*KL*) of the structure; *CF* is equal to 1.00 when a fully satisfactory knowledge (*KL*3) of the structure is achieved, to 1.20 for normal knowledge (*KL*2) and 1.35 (Italian National Annex), for partial or unsatisfactory knowledge level (*KL*1). EC8 implicitly assumes that an high variability of the concrete strength in existing buildings depends on the unsatisfactory size of the tested sample, as it recommends to increase the amount of *in situ* tests to reach the *KL*3, thus assuming *CF* equal to 1.00. The adoption of a conventional (reduced) value of f_c is supposed to compensate the variability of the strength, which is neglected in the analysis.

2.1.2 The assumed strength domain

In this work the concrete strength has been statistically modeled according to a database (Cristofaro 2009), collected by the Seismic Agency of Tuscan Government, which encompasses about 300 buildings, and over 1000 destructive and not destructive tests (Cristofaro *et al*. 2012, 2015). The statistical parameters have been found by considering buildings which have a minimum of three test results. The Coefficient of Variation (*CoV*) of specimens taken from each single building ranges in a large interval, reaching 50% within all the three (60s, 70s, 80s) considered decades.

The assumed strength domain has a Gaussian distribution. It has a *mean* equal to 19.36 MPa and different values of *CoV*, according to the experimental data (Cristofaro 2009), respectively equal to 15%, 30% and 45%. The domain consists of 7 values, corresponding to the percentiles of 5%, 10%, 20%, 50%, 80%, 90% and 95% respectively.

2.2 The case study

2.2.1 Geometrical and mechanical features

The sample structure (De Stefano *et al*. 2015a) is a 4-story 3D reinforced concrete frame, symmetric along both *x* and *y* directions, with two 4.5 m long bays in the *y*-direction and 5 bays 3.5 m long in the *x*-direction, as shown in Fig. 1. In the figure the columns have been numbered according to the results presented in the following sections.

All the columns have cross section dimensions of 30×30 cm, and their longitudinal

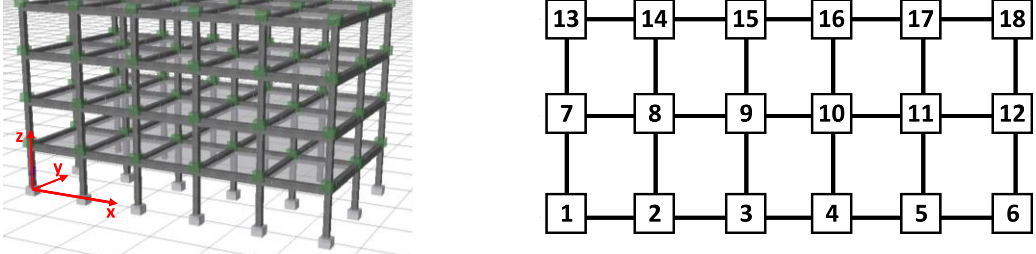

Fig. 1 Case-study: 3D view and plan configuration

Table 1 Strength distribution in the columns of the first storey

percentile	K_{05}	K_{10}	K_{20}	K_{50}	K_{80}	K_{90}	K_{95}
No. of columns	1	2	3	6	3	2	1

reinforcement consists of 8 ϕ14 rebars. The stirrups have been assumed to have a diameter of 6 mm and a spacing of 20 cm. The joints are not confined, according to the standard of 70s. Longitudinal beams have constant cross section dimensions of 30×50 cm in both directions. The concrete strength, having a *mean* value equal to 19.36 MPa, has been described according to the statistical description shown in Section 2.1.1 (see De Stefano *et al*. 2015a), while the reinforcement is assumed to have the same mechanical properties as the Italian FeB38k steel (yield stress over 375 MPa, ultimate stress over 430 MPa), which are compatible with those of the steel used in the 70s. The building is designed for vertical loads only, ignoring seismic loads. Vertical loads consist of dead loads equal to 5.9 KN/m^2 and live loads equal to 2 KN/m^2.

The analysis has been performed along the *y* direction only. The vibrational mode of the structure along the *y* direction (see De Stefano *et al*. 2015a), which is the first one of the structure, has a period equal to 0.777 sec.

2.2.2 The considered layouts

Previous investigations made on a similar case-study (De Stefano *et al*. 2013b, 2015b) proved that considering strength variability at the first storey only leads to similar effects as considering such variability at all floors, since the maximum response occurs at the 1st storey. In this work, therefore, the in-plan strength variability has been introduced at the first storey only. The seven values constituting the assumed strength domain have been given to the columns of the first storey in order to obtain the expected variability. Since the structure has 18 columns, in each model the strength values are assorted according to the specifications listed in Table 1.

Since the seismic analysis has been performed in the *y*-direction only, the variability has been given along the *x*-direction; all the considered plan layouts are symmetric around the central horizontal axis. Six groups of 30 schemes each, corresponding to 180 layouts, have been considered. Each group is characterized by the position of the weakest column (f_c=f_{k05}), which covers all the possible positions of the central axis (see De Stefano *et al*. 2015a). For each position of the weakest column, all the most significant strength combinations have been considered. Special attention has been paid to the “extreme” considered strength values, i.e., the values corresponding to the percentiles k05, k10, k90 and k95, that have been exhaustively combined.

Due to the introduced strength variability, each considered plan layout presents both a strength and a stiffness irregularity. In concrete structures, in fact, the Young modulus E_c is defined as a function of f_c (see EC2, Table 3.1).

Both strength and stiffness eccentricities arising in the case-study from the introduced variability have been found; the strength eccentricity, e_V, has been expressed in terms of ultimate shear of the columns, (see EC8-3 Eq. (A.12)) while the stiffness eccentricity, e_K, has been expressed through a simplified relationship (Anagnastopoulos *et al*. 2009, 2013) based on the yield moment and rotation, i.e., $K=(M_yH)/(6\theta_y)$.

In Table 2 the ranges obtained for the two eccentricities are listed as a function of the assumed *CoV*. The maximum stiffness eccentricity related to the introduced strength variability achieves 6%, exceeding the 5% provided by EC8 to account for accidental irregularities.

Table 2 Ranges of eccentricities (e_K, e_{str_Vu}) due to the introduced strength variability

	CoV=15%		*CoV*=30%		*CoV*=45%	
	min	max	min	max	min	max
e_K	-1.7%	1.7%	-3.6%	3.6%	-6.1%	+6.1%
e_V	-0.6%	+0.6%	-1.4%	+1.4%	-2.8%	+2.8%

2.3 Nonlinear static analysis

The analysis has been performed by using the computer code Seismostruct (Seismosoft 2006). A fiber model has been adopted to describe the cross sections, and each member has been subdivided into four segments. The Mander *et al*. model (Mander *et al*. 1988) has been assumed for the core concrete, a three-linear model has been assumed for the unconfined concrete, and a bilinear model has been assumed for the reinforcement steel. Contribution of floor slabs has been considered by introducing a rigid diaphragm.

Nonlinear static analysis has been performed to find the inelastic response of the case-study; in recent years pushover analysis has been extensively adopted to investigate inelastic seismic response of building structures, and different improvements have been introduced (Fajfar *et al*. 2005, D'Ambrisi *et al*. 2009, Chopra and Goel 2002, Bosco *et al*. 2009, 2013a, b, Magliulo *et al*. 2012, Shakeri *et al*. 2012, Bhatt and Bento 2014, Stathopoulos *et al*. 2010) to account for structural irregularities. In the current work, anyway, the standard N2 method, as provided by EC8, has been applied.

The inelastic response of the structure has been found by considering different seismic intensities, with PGAs ranging between 0.10 g and 0.25 g. The seismic input has been represented by the elastic spectrum provided by EC8 for a soil-type *B*. In (De Stefano *et al*. 2014) has been found that the assumed horizontal pattern distribution does not significantly affect the results. In this work, therefore, only one force pattern, proportional to the first vibration mode, has been considered.

4. Results

4.1 Seismic response

In this section the response domains of the case-study after a seismic intensity ranging between 0.10 g and 0.25 g are shown. It has to be underlined that the limit value of the pushover curve, i.e., a 20% shear drop respect to the shear peak, corresponds to a PGA between 0.20 g and 0.25 g, depending on the considered model. Results obtained for a PGA equal to 0.25 g, therefore, have to be considered beyond the ultimate possible response of the structure.

4.1.1 Chord rotation

The seismic response at the first storey of the case study has been studied in terms of drift and shear at each column. In Fig. 2 the results obtained from analysis and from EC8 standard approach, including the 5% eccentricity, are shown for four different PGAs. It should be noted that the 1st storey drift is the same at each column belonging to the same column-line in the *x*-direction, since the hypothesis of diaphragmatic floor has been assumed. As should be noted in

Fig. 2, the variability in the drift response increases both with the seismic intensity and the strength variability as represented by *CoV*. Moreover, at the PGA increasing, the torsional effects arising from the introduced eccentricity increase as well, achieving their maximum amount at the side columns (columns No. 1, 6, 7, 12, 13, 18). It can be seen that the structural demand provided by EC8, when the 5% eccentricity is introduced, seems to adequately include the increase in the side drift due to torsional effects.

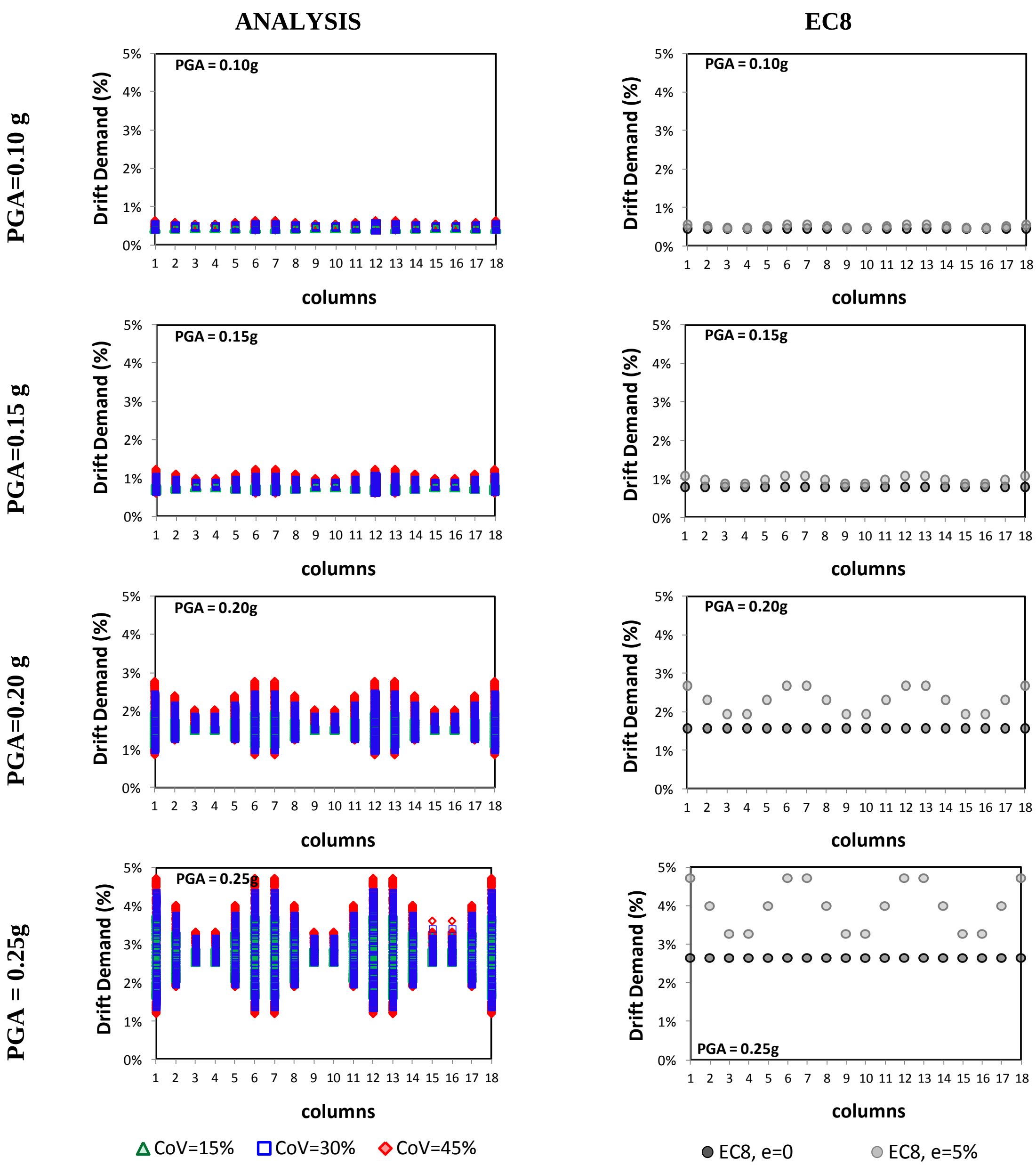

Fig. 2 Drift demand at the first storey

4.1.2 Shear

The shear demand at the 1st storey columns is shown in Fig. 3.

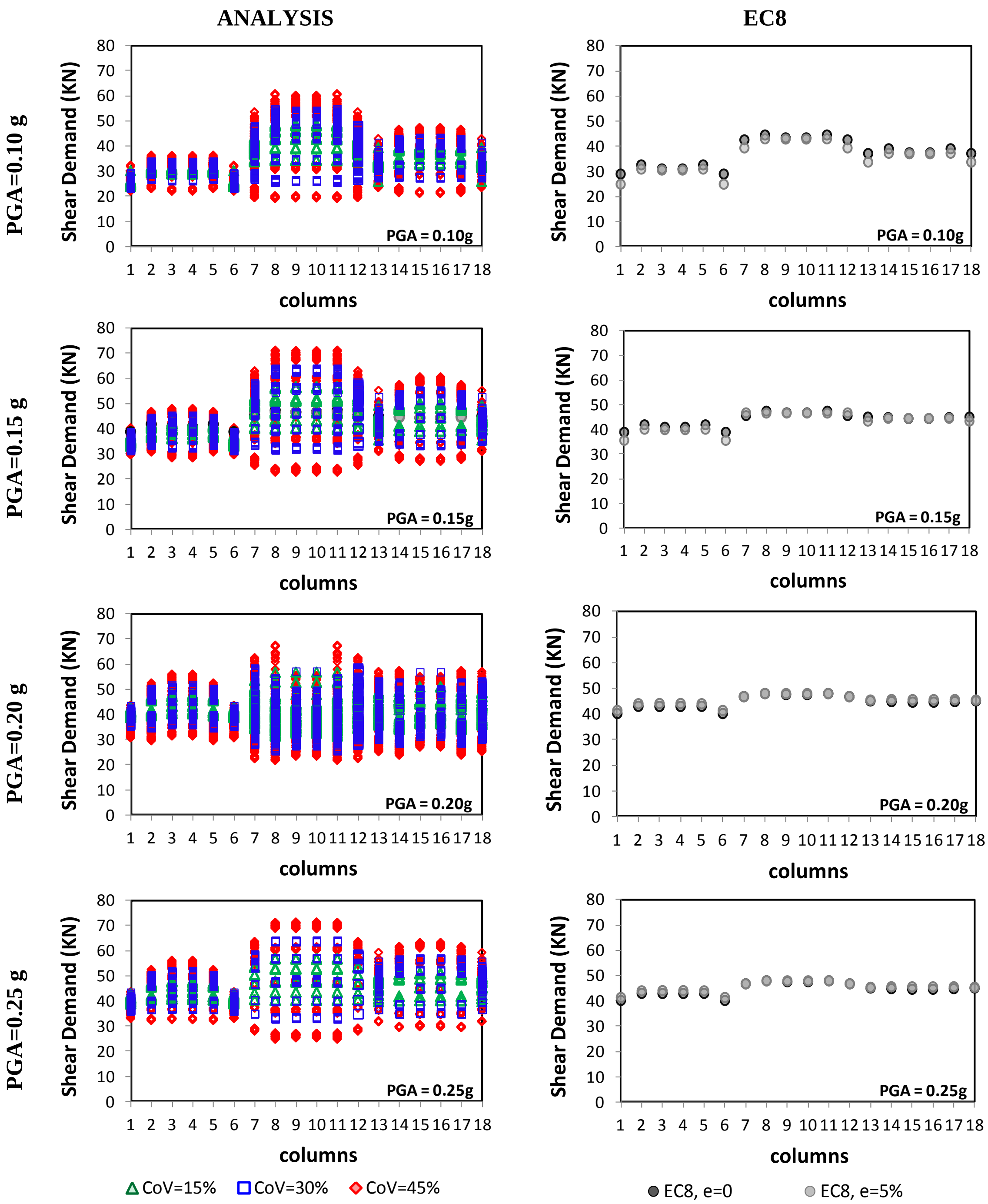

Fig. 3 Shear demand at the columns of the first storey

The results found from analysis are shown in the left column of Fig. 3, while those provided by EC8 are shown in the right column. It can be noted that the shear distribution in the columns is not affected by torsional effects. The shear response, instead, has a large scatter due to the introduced strength variability. The scatter in the shear domains increases with the PGA until the maximum shear value is achieved (PGA between 0.15 g and 0.25 g, depending on the columns). It should be reminded, in fact, that the pushover curves have been cut at 20% drop of shear. Therefore, when the structural response belongs to the softening branch of the pushover, the maximum value of shear has been taken to describe the structural response, with a consequent reduction in the shear scatter.

4.2 Seismic capacity

The seismic capacity of the columns has been found both in terms of chord rotation and shear force, according to the EC8 provisions. In the following paragraphs the two limit quantities have been separately presented for each column.

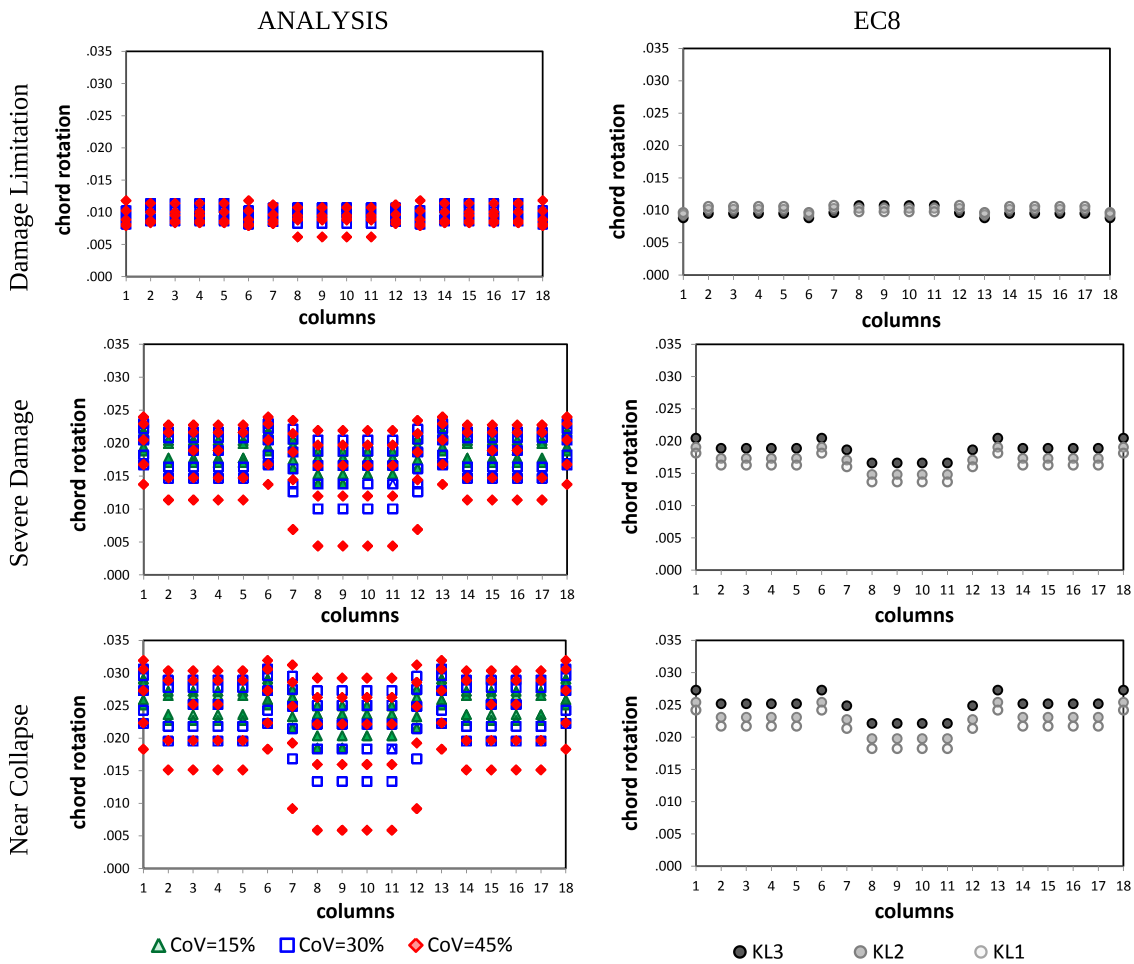

Fig. 4 Chord rotation limits

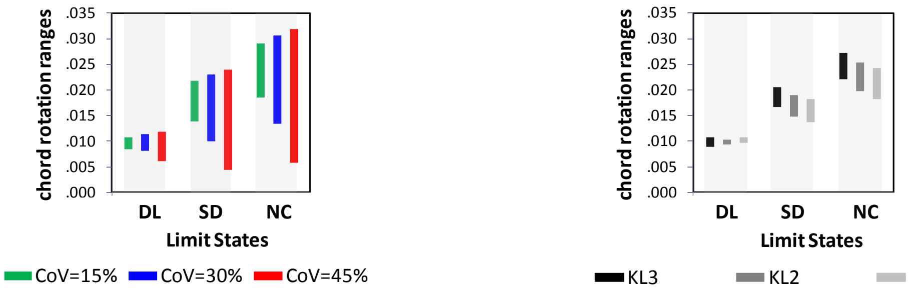

Fig. 5 Ranges of limit values related to the strength variability and to the EC8 Knowledge levels

4.2.1 Limit chord rotation

The limit chord rotation of each column has been found for all the considered limit states, i.e., Damage Limitation (*DL*), Severe Damage (*SD*) and Near Collapse (*NC*). For the *DL* limit state, the yield chord rotation, θ_y, defined according to Eq. (A5) (EC8, Annex A), has been assumed as limit value. The limit values assumed for the ultimate limit states (*SD*, *NC*), instead, have been found as a function of the ultimate chord rotation, θ_u, as provided by EC8 in Eq. (A1) of EC8, Annex A. More precisely, the limit value assumed for the *NC* limit state has been taken equal to θ_u, while the one assumed for the *SD* limit state has been assumed to be ¾ of θ_u.

Both θ_y and θ_u are a function of the column material, geometry, reinforcement and axial load. Therefore, since all columns of the case-study have the same geometry, material and reinforcement, when the conventional EC8 approach is adopted, the limit chord rotations found for the columns differ from each other for the assumed *CF* and the amount of axial load only. When the strength variability is introduced, instead, the limit values depend even on the considered concrete strength.

Fig. 4 shows the limit values of the 18 columns of the first storey for three considered limit states found from f_c values constituting the assumed sample and from EC8 instructions. It can be noted that the chord rotation limits are sensitive to the amount of the assumed axial load, that has been taken as the static one under gravity loads.

When the strength variability is considered, the obtained values of the chord rotation have a larger scatter, especially for the ultimate limit states (*SD*, *NC*). As it should be observed in Fig. 4, when the *CoV* exceeds 30%, the limit values obtained for the *SD* and *NC* can overlap each other. In Fig. 5 the ranges obtained for each limit states by adopting the conventional values provided by EC8 and the assumed strength domains are shown. As it should be noted from the diagrams in Fig. 5, the minimum values of the limit domains are more affected by the strength variability than the maximum ones.

4.2.2 Limit shear

According to EC8, only one limit value is provided for the shear, and it has to be adopted for one of the ultimate limit states. The choice of the limit state to considered is left to the designer, therefore, the *NC* limit state is in the following considered for verification. The ultimate shear force of each column has been defined according to Eq. (A12) (EC8, Annex A).

Fig. 6 shows the values obtained for the limit shear by adopting the strength values found from the assumed strength domain and from EC8 approach, by introducing the Confidence Factors. The limits found by applying the EC8 procedure are scarcely sensitive to the considered *CF* values.

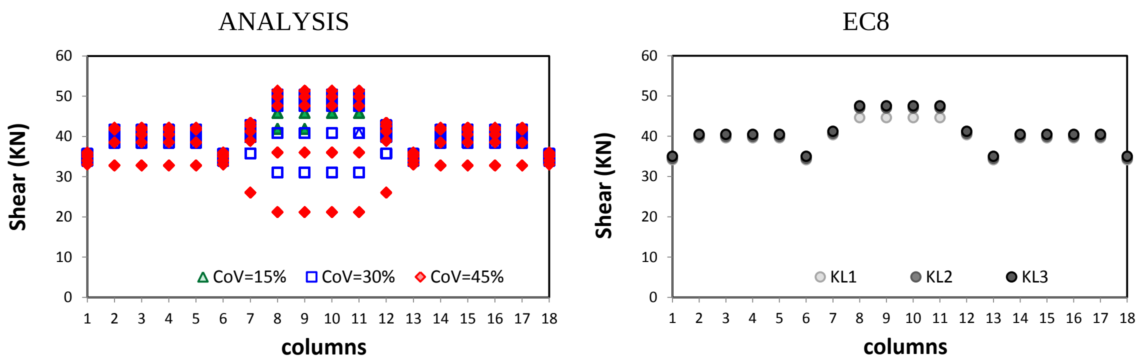

Fig. 6 Limit values of the shear (*NC* Limit State)

Table 3 Seismic performance according to EC8

	DL	*SD*	*NC*
Probability of exceedance	22% in 50 yrs	10% in 50 yrs	2% in 50 yrs
PGA ranges in the assumed area	0.10 g - 0.17 g	0.15 g - 0.24 g	0.20 g - 0.40 g
PGA considered in analysis	0.10 g, 0.15 g	0.15 g, 0.20 g	0.20 g, 0.25 g

When the strength variability is included in the analysis, the columns belonging the central frame (columns No. 7, 8, 9, 10, 11, 12), due to their larger amount of axial load, present larger scatters than the other ones. Due to the strength variability, in fact, central columns can present a limit shear much smaller than the one found by applying the EC8 approach.

4.3 Seismic performance

4.3.1 Performance in terms of chord rotation

The seismic performance is the most relevant index for evaluating the seismic reliability of existing structures. In this work the seismic performance is measured as the ratio between the demand (*D*), i.e., the maximum response of the structure under the assumed seismic input, and the capacity (*C*), i.e., the limit value of the chord rotation for each considered limit state. The seismic intensity to be considered for each limit state is provided by EC8 in terms of Probability Of Exceedance (POE) in a period of 50 years (see Table 3). The PGA values to be associated to each POE depend on the seismicity of the area. As specified in Section 2.2.1, the case-study of this investigation is not a real structure. Its localization has been assumed in Tuscany, since the experimental data adopted to characterize the strength domain are referred to such region.

Therefore, for each Limit State, a range of PGA values, compatible with the seismic classification of different Tuscan areas, have been found. In Table 3 the PGA ranges, together to the PGA values considered in the analysis, are listed for each limit state.

It should be noted that the Near Collapse Limit State is only partially investigated, since the ultimate PGA sustained by the structure is between 0.20 g and 0.25 g, depending on the assumed strength distribution.

The ratio D/C has been assumed in this section as a damage index; when it is lower than unity the structure respects the limit provisions, while for values over unity the structure exceeds them,

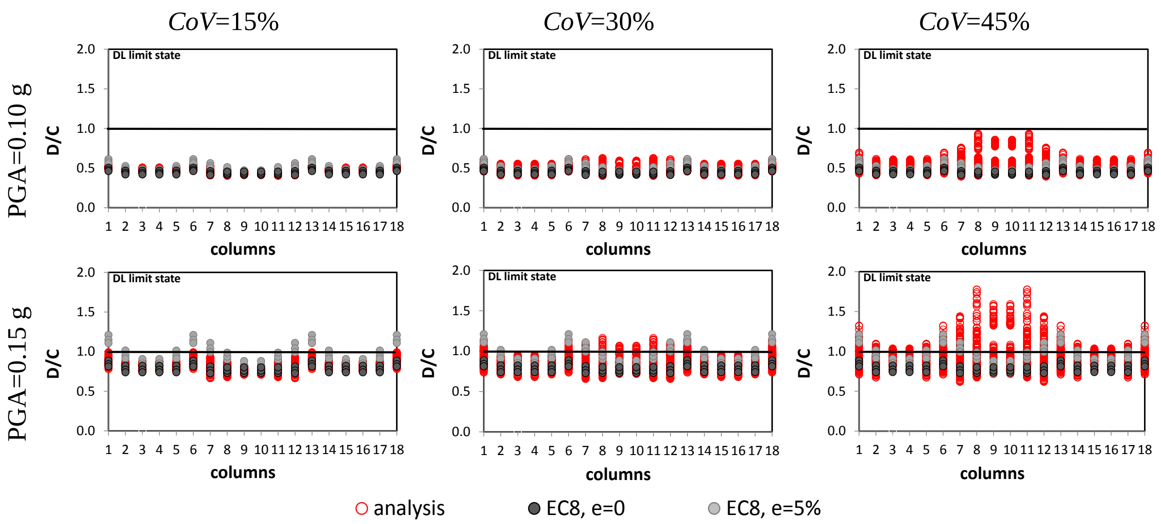

Fig. 7 *DL* limit state: seismic performance of the 1st storey columns

resulting to be not compatible with the safety requirements. In Figs. 7-9 the obtained values of D/C found by the chord rotation are shown for the considered limit states and seismic intensities. Both the D/C values obtained by considering the strength variability and the EC8 procedure, by accounting for the 5% eccentricity and not, are shown for each column. As it should be noted from the diagrams, the D/C in each column depends both on the assumed PGA and on *CoV*.

The serviceability limit state, *DL*, is satisfied only for PGA=0.10 g (see Fig. 7). When a PGA=0.15 g is considered, the ratio D/C found by considering the strength variability is below the unity only for *CoV*=15%. For higher values of *CoV* the limit value is exceeded, approaching the value of 2 for *CoV*=45%. It should be noted that the EC8 approach provides D/C always below the unity if the 5% eccentricity is not considered, while it achieves 1.25 when the eccentricity is taken into account.

As regards the *SD* limit state (see Fig. 8), for PGA=0.15 g, the limit value is exceeded only for the highest value (*CoV*=45%) of the strength variability. For PGA=0.20 g, instead, the performance limits are not respected in any case. It should be noted, anyway, that the maximum D/C value coming from the higher strength variability is 3.5 times higher than the corresponding value provided by EC8, even when the 5% eccentricity is considered. Similar observations can be done for the *NC* limit state, shown in Fig. 9. In this case the limit values are exceeded even for the lower considered seismic intensity (PGA=0.20). In all the considered limit states, the most conservative results provided by EC8, i.e., obtained by assuming *KL*=3 (*CF*=1.35) and the 5% eccentricity, are conservative, compared to the ones found by analysis, only for *CoV* below 30%.

The introduced strength variability proved to affect very much the seismic performance of the case-study, leading to an evaluation of its structural assessment substantially different from the one found by the conventional EC8 approach. In order to exclude the effect of the worst layouts, the drift-performance D/C provided by EC8 has been also compared to the 84% percentile of the D/C domain found through the nonlinear static analysis. Figs. 10-12 show the comparison between the EC8 results and the 84% percentiles for the three limit states.

As regards the *DL* limit state, the EC8 appraoch provides a reliable evaluation of the seismic

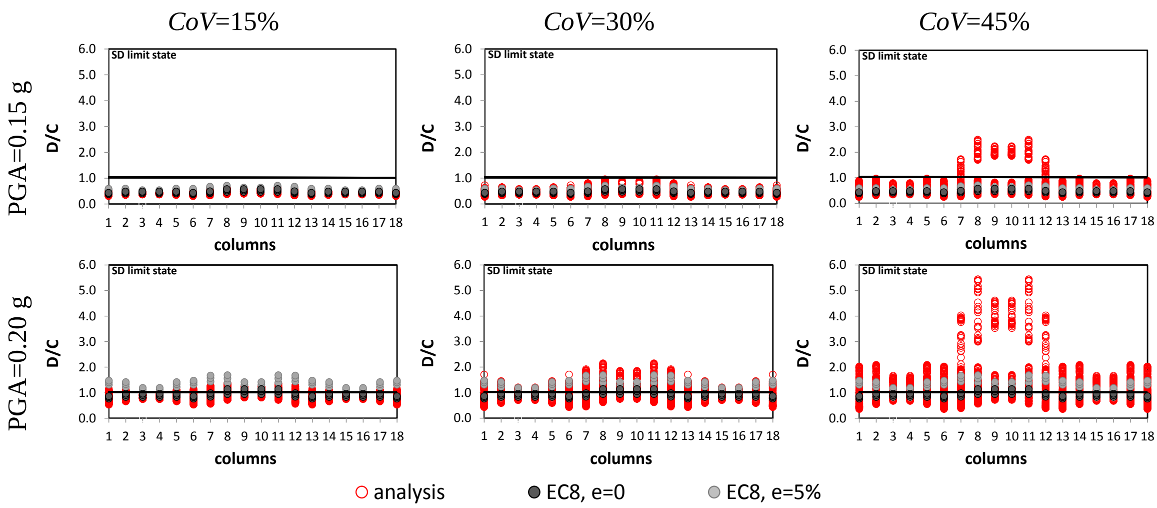

Fig. 8 *SD* limit state: seismic performance of the 1st storey columns

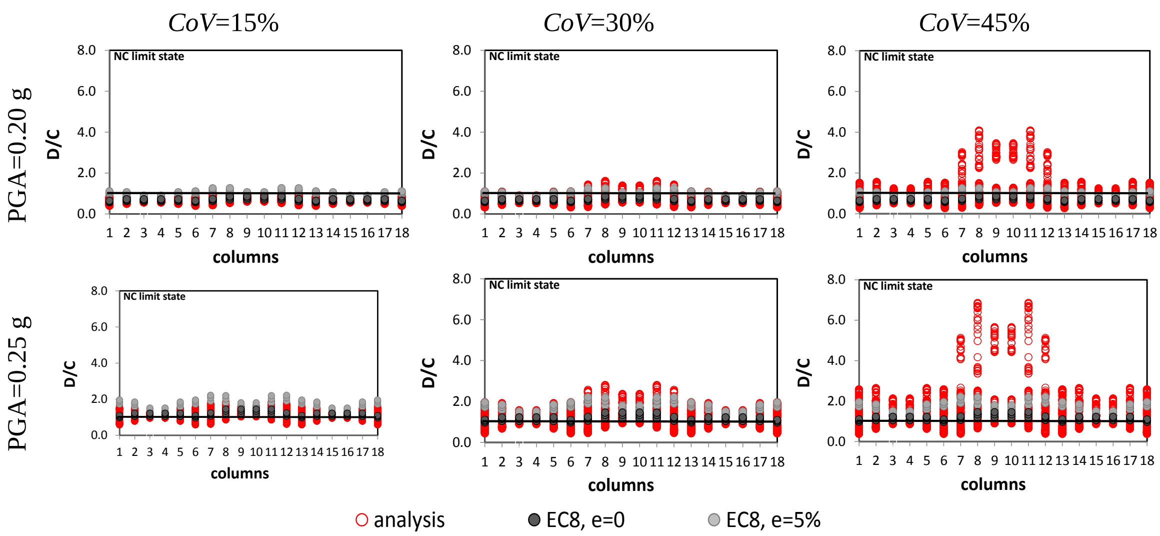

Fig. 9 *NC* limit state: seismic performance of the 1st storey columns

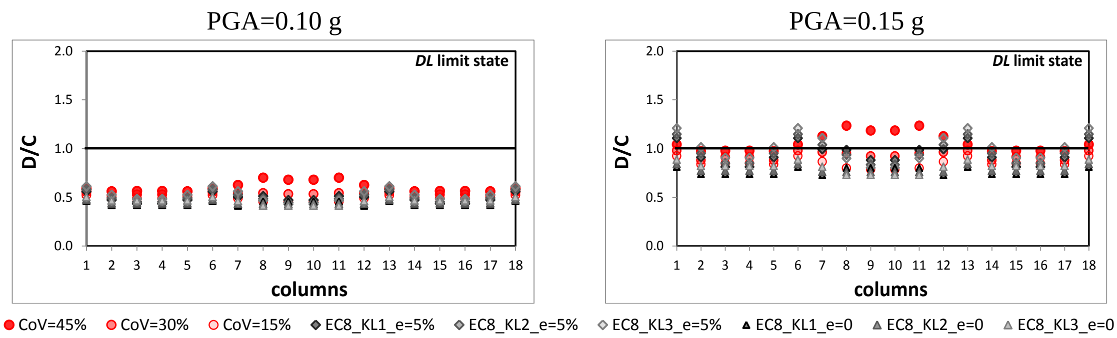

Fig. 10 *DL* limit state. Comparison between the *K_84* percentile of D/C and EC8 provisions

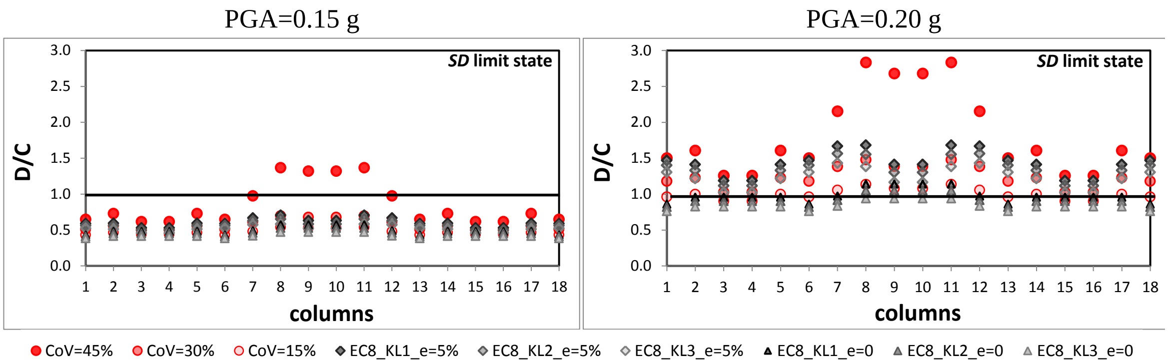

Fig. 11 *SD* limit state. Comparison between the *K_84* percentile of D/C and EC8 provisions

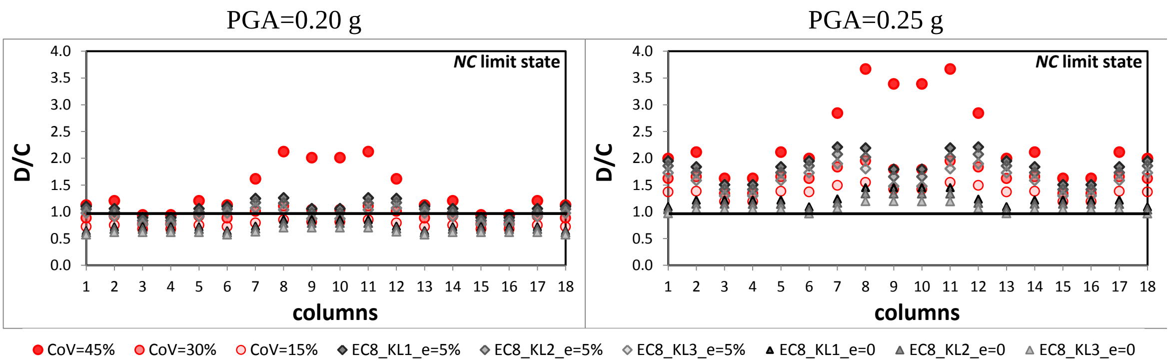

Fig. 12 *NC* limit state. Comparison between the *K_84* percentile of D/C and EC8 provisions

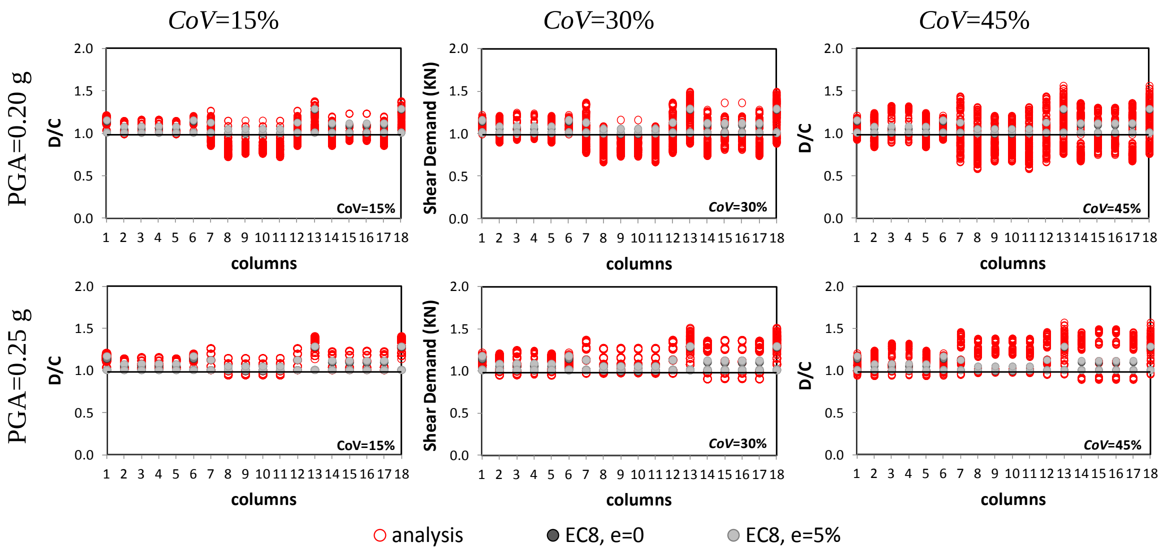

Fig. 13 *NC* limit state: seismic performance of the 1st storey columns

performance. In fact, the *K_84* values found for D/C are very similar than the ones found by applying the standard EC8 provisions. Even the 5% eccenricity is not relevant in this case, since the obtained D/C values are scarcely affected by its introduction.

When the ultimate limit states *SD* and *NC* are considered, instead, the *K_84* percentile of the D/C index found by considering the strength variability is still much higher than the values found by applying the standard EC8 approach. In the columns belonging to the central frame, in particular, the difference between EC8 provisions and the *K_84* percentile is significant. Even when the most conservative assumptions of EC8 are adopted (*CF*=1.35, e=5%), in fact, values of D/C found through the analysis (*K_84*) are almost twice than those provided by EC8.

4.3.2 Performance in terms of shear

Fig. 13 shows the shear performance found for the *NC* limit state, measured as the ratio D/C, at each column of the first storey. The shear is less sensitive to the strength variabiliy than the chord rotation. At the increasing of the strength variability the obtained D/C ranges increase, but the final assessment of the building safety found from the analysis is similar to the EC8 one.

5. Conclusions

This work deals with the effects of the concrete strength variability on the seismic response of existing RC building structures. Such effects have been evaluated in terms of seismic demand, capacity and performance on a case-study, i.e., a 4-storey RC building, designed to vertical loads only, without any specific attention to seismic and wind actions. The concrete strength variability has been represented by assuming a Gaussian distribution, according to a large available database, and it is supposed to affect the columns belonging to first storey only, representing a source of stiffness and strength eccentricity. A sample of 180 different in-plan layouts has been considered to represent the possible strength distributions.

In the first part of the work the seismic response of the case-study has been investigated. Due to the introduced eccentricity, the building experiences a torsional response, with a consequent increase in the maximum drift demand. Such increase is maximum at the building flexible side, where it reaches almost 50%. The seismic demand found by considering the strength variability has been compared to the one provided by the standard EC8 approach, which assumes the *mean* strength value at all members, and introduces a 5% eccentricity to take into account accidental irregularities. The EC8 previsions, in terms of maximum drift, cover the maximum response found by the analysis. As regards the shear demand, the increase related to the strength variability is not influenced significantly by torsional effects, since it affects all the columns in the same way. In this case the EC8 prevision does not cover the maximum values of the shear domain found by the analysis.

In the second part of the work the seismic capacity of the 1st storey columns has been found, in terms of chord rotation and shear, according to the EC8 instructions, and compared to the values provided by EC8 by assuming the three possible values of Confidence Factors. The scatter found in the capacity of the columns due to the strength variability is much larger than the one due to the different *CF* values. As a consequence, due to the introduced strength variability, each column can experience a capacity much lower than the standard one. The comparison with the limit values found by assuming the conventional strength values as provided by EC8 shows similar results both for chord rotation and shear.

In the third part of the paper the seismic performance, i.e., the ratio between demand (*D*) and capacity (*C*), of the case-study has been investigated. Three different limit states, *DL*, *SD* and *NC*, have been considered for the performance measured in terms of chord rotation, while only one limit state, *NC*, has been considered for the shear performance. As regards the chord rotation performance, the comparison to the EC8 previsions shows different results depending on the considered limit states. For the serviceability limit state (*DL*), in fact, the EC8 evaluation provides similar results to those from the analysis. For a strength variability below 30%, the maximum D/C values are close with the two approaches; when the highest value of *CoV* (*CoV*=45%) is considered, the analysis provides D/C values higher than the EC8 approach, but the final evaluation on the building safety does not change. When the ultimate limit states (*SD*, *NC*) are considered, instead, the analysis provides D/C values even three times larger than the EC8 previsions, with a consequent different evaluation of the performance. Since the strength variability proved to largely affect the performance of the case-study, a more careful evaluation has been done, by comparing the D/C values provided by EC8 with the 84% percentile of the D/C domain found by analysis, in order to cut the most unfavorable considered in-plan layouts. The comparison has shown that, even when the 84% is considered, the strength variability significantly affects the performance evaluation, and the found D/C values are not covered by EC8 previsions. As regards EC8 previsions, the 5% role introduced by EC8 to take into account of accidental eccentricity proved to be essential for response and performance evaluation, while the adoption of different Confidence Factor values does not significantly affects the response and the performance of the case-study. When the seismic performance is evaluated in terms of shear, the strength variability still affects the response domains, but without a significant impact on the performance evaluation.

As a conclusion, the performed analysis proved the strength variability to be a significant source of in-plan irregularity for the case-study, affecting its seismic performance, and leading to an evaluation of its seismic assessment different from the one provided by the standard EC8 approach, even when all the most conservative assumptions have been considered (i.e., 5% eccentricity and *CF* equal to 1.35). When the concrete presents an high variability, therefore, the EC8 approach does not provide a safe evaluation of the seismic performance of the case-study. Since the strength variability proved to significantly affect the seismic performance of the case-study, and it is affected by many parameters, a more extended investigation should be conducted to get more general results.

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