SECOND EUROPEAN CONFERENCE ON EARTHQUAKE ENGINEERING
AND SEISMOLOGY, ISTANBUL AUG. 25-29, 2014

THE İNFLUENCE OF THE AXİAL LOAD VARİATİON ON THE SEİSMİC CAPACİTY OF EXİSTİNG RC BUİLDİNGS

Mario DE STEFANO[1], Valentina MARIANI[2], Marco TANGANELLI[3], Stefania VITI[4]

ABSTRACT

The paper deals with the role of axial load on the seismic performance of existing RC buildings. The axial load variability is often neglected in seismic analysis, despite it can largely affect the capacity of RC columns. The limit chord rotation of columns, in fact, is a function of the amount of axial load. The sensitiveness of the rotational capacity of a column to axial load depends even by its concrete strength. If concrete presents poor mechanical properties, in fact, the sensitiveness of the columns' capacity to the axial load is larger.

In this work the axial load variation has been considered both in the seismic demand and capacity on a case-study, i.e. a 3D RC framed structure designed for vertical loads only. As regards the seismic demand, the relationship between axial load variation and type of performed analysis has been checked, by performing both a nonlinear static and a time-history analyses, in order to evaluate the effect of the dynamic response of the case-study on the axial load variation. As regards the seismic capacity, it has been found by considering the effective amount of axial load in each column and different assumptions about the concrete strength characterization.

INTRODUCTION

It is well known that the axial load plays an important role in the evaluation of the structural performance of RC columns (Abbasnia et al. 2011, Saadeghvaziri 1997). In fact the axial load largely affects both the seismic demand of RC buildings and their capacity.

As regards the seismic demand, in fact, due to its dynamic response, the structure necessary experiences a variation in the axial load in its vertical elements. At the occurring of severe ground motion, in particular, some columns can be subjected to axial load reduction, experiencing in extreme cases traction, or conversely, they can experience a large increase in compression.

Even the capacity is affected by the axial load. In fact, it is common knowledge that the capacity of a RC section is maximum for a level of axial load corresponding to its "balanced" ultimate bending condition. If the section is subjected to traction its flexural and shear capacity is largely reduced, like when the section is subjected to high levels of compression.

The seismic performance, being the ratio between demand (D) and capacity (C) should therefore be carefully checked at the axial load variation. In existing buildings the sensitivity of the seismic performance to axial load can be even more relevant than in new ones, since the material can easily present poor and uncertain mechanical properties.

---

[1] Full Professor, DiDA,University of Florence, Italy, mario.destefano@unifi.it
[2] PhD, DiDA,University of Florence, Italy, valentina.mariani@unifi.it
[3] Researcher, DiDA,University of Florence, Italy, marco.tanganelli@unifi.it
[4] Researcher, DiDA,University of Florence, Italy, stefania.viti@unifi.it

In this work an investigation is made on a case-study to check the role of the axial load variation on the seismic capacity, evaluated in terms of limit chord rotation, of RC columns. The case-study is a doubly symmetric 4-storey RC building, representing a typical example of pre-seismic code structure. Special attention has been paid to check i) the role played by the type of analysis (static vs dynamic) on the axial load variation in the structural demand and ii) the influence of the effective concrete strength on the sensitivity of members capacity to the axial load variation.

Concerning the seismic demand, the relationship between axial load variation and type of performed analysis has been checked, by performing both a nonlinear static and a time-history analyses, in order to evaluate the effect of the dynamic response of the case-study on the axial load variation.

As regards the seismic capacity, the limit chord rotations of the assumed limit states have been found for each column of the case study by assuming the current axial load found by the two analyses. Two different approaches for concrete characterization have been considered. The first one is the EC8 conventional approach, consisting in scaling the mean value of concrete strength by proper reduction factors (Confidence Factors, *CF*) depending on the knowledge level of the building. The second approach consists in representing the concrete strength as a variable quantity. As a consequence it is represented by a Gaussian distribution, having the assumed *mean* strength, and different amount of Coefficient of Variation (*CoV*), consistent to experimental observation. Both approaches have been adopted and compared, in order to evaluate the effect of concrete strength on the sensitivity of the seismic performance to axial load variation.

THE CASE-STUDY

The sample structure (De Stefano et al. 2014) 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.

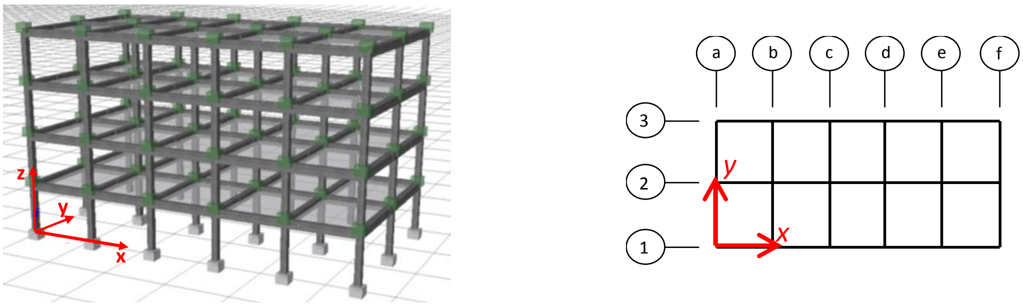


Fig. 1 Case-study: 3D view and plan configuration.

All the columns have cross section dimensions of 30x30 cm, and their longitudinal 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 the 70s. Longitudinal beams have constant cross section dimensions of 30x50 cm in both directions.

The concrete has been assumed to have a *mean* strength equal to 19.36 MPa, 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). The building is designed for vertical loads only, ignoring seismic loads. Vertical loads consist of dead loads and live loads equal to 2 KN/m^2.

The analysis has been performed along the *y* direction only. The first vibrational mode of the structure is along the *y* direction (see De Stefano et al. 2013, 2014), and has a period equal to 0.778 sec.

THE ANALYSIS

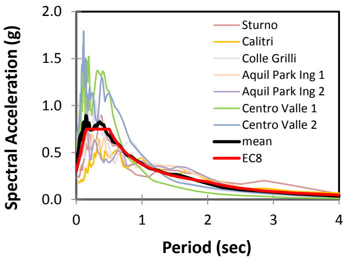
Both a static nonlinear (pushover) and a nonlinear dynamic (time-history) analyses have been performed. In recent years, several improvements have been made on the pushover analysis (Fajfar et al. 2005, D'Ambrisi et al. 2009, Chopra and Goel 2002, Bosco et al. 2009, Magliulo et al. 2012), but they have not yet been implemented in technical Codes. Therefore, in the current work the standard N2 method, as provided by EC8, has been applied. Since in (De Stefano et al. 2014) has been found that the assumed horizontal pattern distribution does not significantly affect the results, in this work, only one force pattern, proportional to the first vibration mode, has been considered.

Both analyses have 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.

The seismic input. The seismic input has been defined by assuming the elastic spectrum provided by EC8 for a soil-type B. The nonlinear static analysis have been performed by using directly the EC8 spectrum, while for the dynamic analysis, a set of 7 ground motions have been assumed, whose average spectrum closely approaches the EC8 one. The assumed set of ground motions was provided by Working Group Itaca (Itaca 2008), on the basis of a PGA equal to 0.25g, a nominal life of the structure of 50 years and a magnitude between 5.5 and 6.5. The adopted selection criterion aims to optimize the match with the elastic design spectrum for a period range between 0.5 sec and 1.0 sec. Table 1 lists the main information for the adopted ground motions, together with their elastic spectra compared to that from EC8. Both the spectrum and the records have been scaled to represent different seismic intensities, with PGAs ranging between 0.05g and 0.25g.

Table 1. Ground motions data.

Name	Location	Date dd/mm/yyyy	PGA (g)	Duration (sec)
Irpinia	Sturno	23/11/1980	0.225	70.75
Irpinia	Calitri	23/11/1980	0.174	85.99
L'Aquila	Colle Grilli	06/04/2009	0.446	100.00
L'Aquila	Aquil Park Ing 1	06/04/2009	0.353	100.00
L'Aquila	Aquil Park Ing 2	06/04/2009	0.330	100.00
L'Aquila	Centro Valle 1	06/04/2009	0.545	100.00
L'Aquila	Centro Valle 2	06/04/2009	0.657	100.00



The considered limit states. In this work two different limit states have been considered: one serviceability limit state, i.e. *Damage Limitation* (*DL*) limit state, and one ultimate limit state, i.e. *Life Safety* (*LS*). EC8 provides, for each limit state, a limit value of the chord rotation, which represents the structural capacity to check.

EC8 provides, for the *DL* limit state, a limit value equal to the yield chord rotation (EC8, eq. A.10a), and for the *LS* limit state, a limit value based on the ultimate rotation (EC8, eq. A.1). Both (*DL* and *SD*) limit values depend on the amount of axial load in the member. On varying of the concrete strength, the influence of the axial load changes, resulting negligible or significant in the different cases. The influence of axial load, therefore, results to be related to the concrete strength. In existing buildings, the evaluation of the concrete strength is a crucial issue (Masi et al. 2008; Jalayer et al. 2008, 2009; Cosenza et al. 2009; D'Ambrisi et al. 2013). An improper characterization of material, in fact, can compromise the effectiveness of the model representation. The European technical Code, Eurocode 8 (EC8) indicates the *mean* value of the strength domain, whose size is dependent on the achieved 'knowledge level', as the strength value to be used in the analysis, while it prescribes a reduced strength value for verification. The reduction is made by introducing a Confidence Factor (*CF*), ranging between 1.00 and 1.35 (Italian Annex) depending on the knowledge level of the structure.

Recent studies made by some of the Authors (De Stefano et al. 2013, 2014) have shown that existing buildings can easily present material mechanical properties which can largely vary even within a single structure. According to a large 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), the Coefficient of Variation (*CoV*) of specimens taken from each single building ranges in a large interval, possible reaching 50%. In De Stefano et al., 2014 a wide analysis has been performed, referring to the concrete strength values of the available database. A *mean* strength equal to 19.36 MPa has been assumed, and three different *CoV*, respectively equal to 15%, 30% and 45% have been considered. For each *CoV*, a sample of seven values, corresponding to the percentiles of 5%, 10%, 20%, 50%, 80%, 90% and 95% has been assumed. İn this work the strength domains introduced in De Stefano et al. 2014 have been considered and compared to conventional values provided by EC8 according to the three possible Knowledge Levels.

THE STRUCTURAL RESPONSE AS A FUNCTION OF THE PERFORMED ANALYSIS

Nonlinear static analysis. Figure 2a shows the maximum drift obtained by performing the pushover analysis. Due to the horizontal forces, an axial load variation is experienced by each column of the case-study. In Figure 2b the maximum axial load variation experienced by all columns of the case-study is shown.

In Figure 3 the axial load values in each column of the case-study at the increasing of the considered PGA are shown for horizontal forced directed in one way only. For each of the three frames the axial load values experienced by each column under the nonlinear static analysis are represented; as it is expectable, the central frame (frame *2*) is almost not affected by the horizontal action in terms of axial load variation; the side frames, instead, are sensitive to the horizontal forces. In the three frames, it can be observed a different amount of axial load in the side columns (column lines *a* and *f*) comparing to the internal ones. Since the building is symmetric, for horizontal forces directed in the opposite way, the axial load distribution in each frame would be mirrored.

Figure 2. Drift at the 1st storey and axial load variation (pushover analysis).

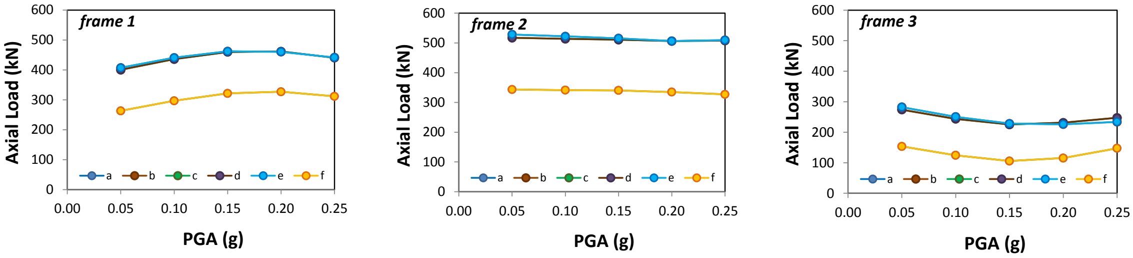


Figure 3. Axial Load distribution in each frame of the case-study.

The axial load variation for horizontal forces directed in the two ways can be better observed in Figure 4, where it is expressed in a not-dimensional form, as the ratio between the maximum axial load of each column under the horizontal forces and the corresponding static one. In the diagram the axial load variation has been distinguished for horizontal forces acting in the two ways (positive and negative). It can be noted that the sensitivity to the axial load variation is the same for the horizontal forces in the two ways, since the case-study is symmetric. The maximum amount of axial load increase is experienced by the columns of the side frame, and it achieves 50% in the corner columns.

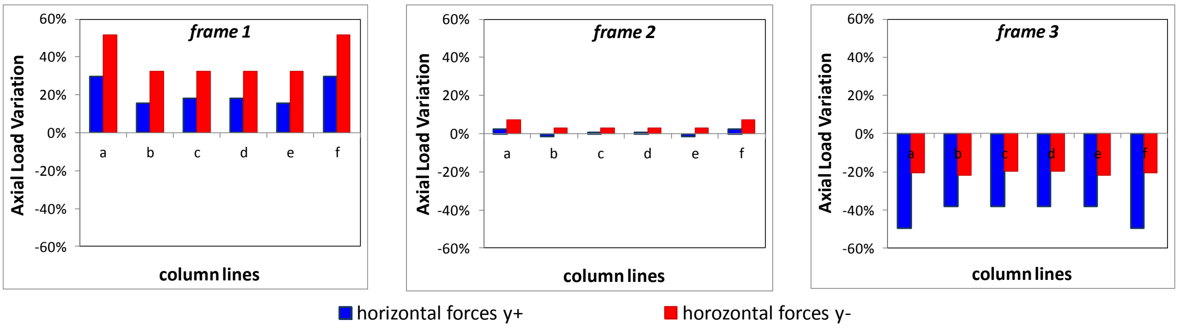


Figure 4. Axial load variation in each column.

Nonlinear dynamic analysis. Figure 5 shows the structural response, in terms of top displacement and interstorey drift at the first storey, obtained by performing the dynamic analysis. The mean response has been found by averaging, for each PGA, the maximum values of the seven analyses.

Figure 6 shows the maximum variation in axial load due to the seismic excitation. The increase and the decrease in axial load have been shown separately. It should be noted that the axial load increase clearly differs by the axial load decrease.

Figure 7 shows the maximum and minimum axial load experienced by each column for the different considered PGAs. The values plotted in Fig. 7 are the mean values averaged over the seven ground motions. The main difference from the results coming from the static analysis is the not symmetric response of the structure as regards the axial load in each column. In fact, it should be noted that the curves representing the axial load variation are different (but not mirrored) for the frames 1 and 3. Moreover, even the central frame (frame 2) experiences a significant amount of axial load variation, whereas in the static analysis the effect was negligible.

In Figure 8 the axial load variation in each column is expressed in a not-dimensional form, by dividing the maximum and minimum axial load values got from the dynamic analysis by the static one. From the diagrams shown in Fig. 8 the different sensitivity to axial load of the two side frames can be clearly noted.

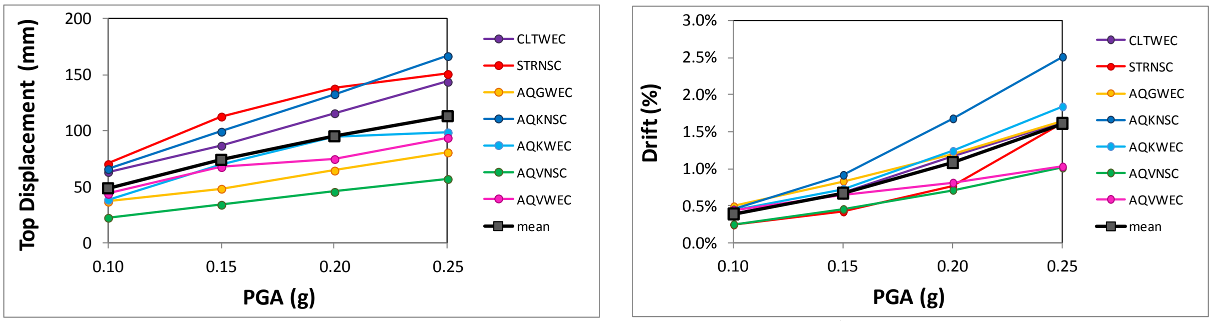


Figure 5. Maximum Top Displacement and Drift (1st storey).

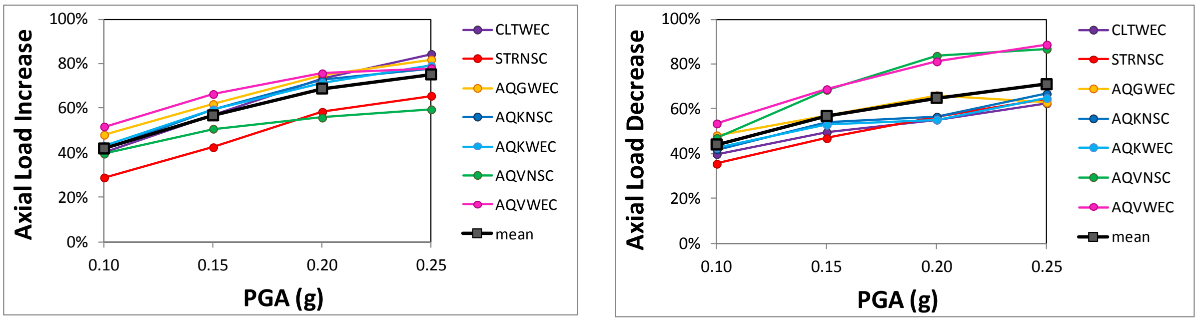


Figure 6. Maximum increase and decrease in axial load.

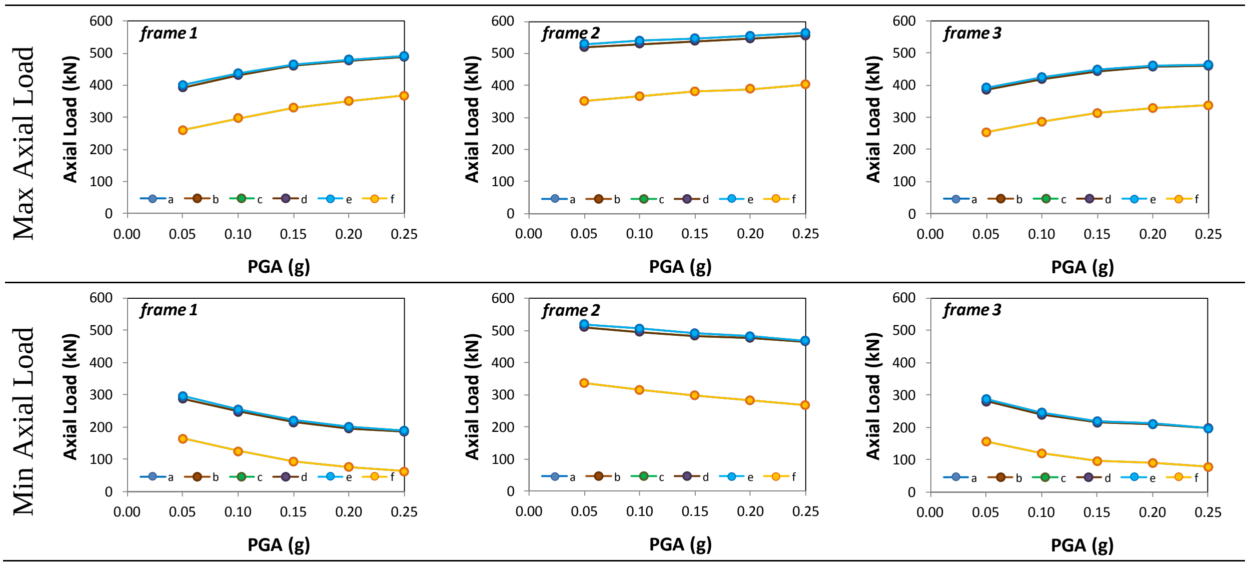


Figure 7. Axial Load distribution in each frame of the case-study (dynamic analysis).

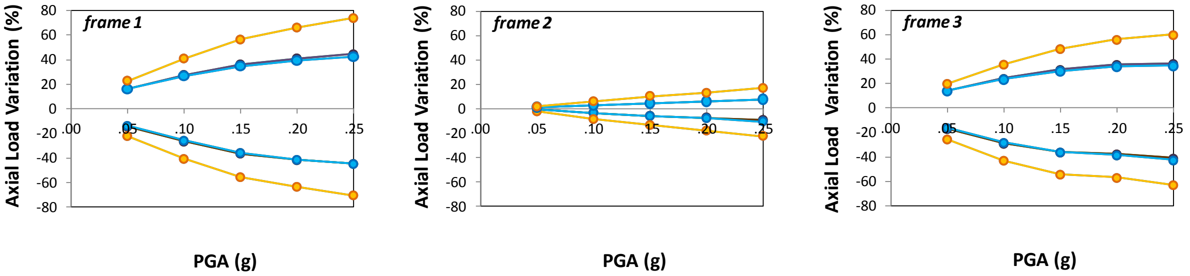


Figure 8. Axial Load variation in each frame of the case-study (dynamic analysis).

The effect of the dynamic response on the axial load variation. In order to measure the effect of the type of analysis on the axial load variation, the maximum and minimum axial load in each column of the case-study have been compared. Figure 9 shows, for each considered PGA, the results of the comparison. The difference has been expressed in a not-dimensional way, by dividing the difference between the dynamic and static results by the corresponding static value. It should be noted that the difference between the results provided by the two analyses, in the negative variation (axial load reduction) is much larger than the one in the positive variation (axial load increase).

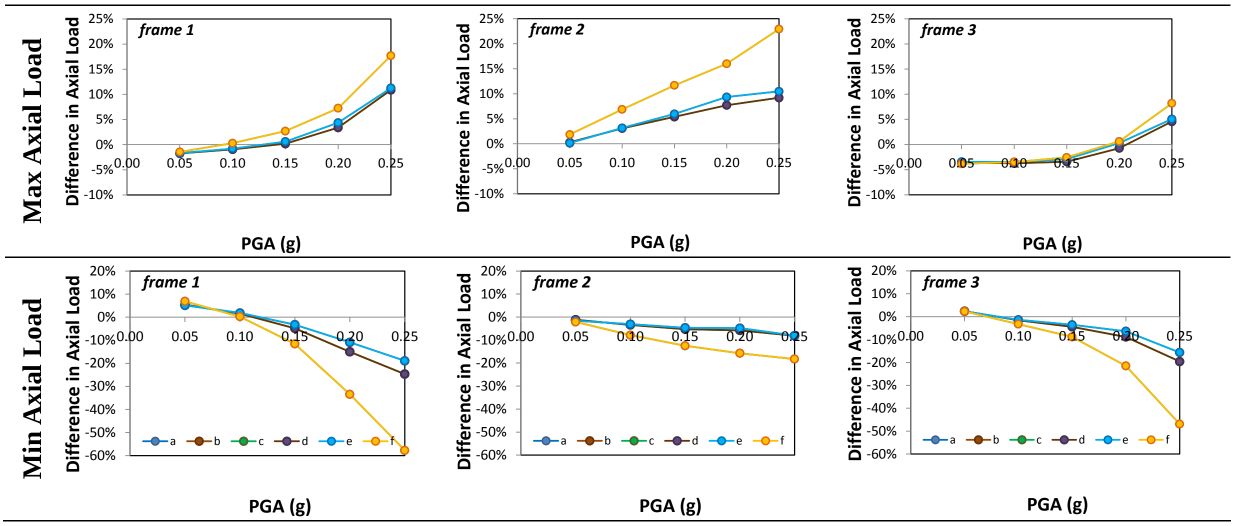


Figure 9. Difference in maximum and minimum axial load due to the type of analysis.

The results referred to the central frame are normalized by considering a static value of the axial load that is double than the other ones. Consequently, the effect of the type of analysis on the variation does not appear as large as it is. It should be noted, in fact, that the axial load variation in Frame 2 is almost negligible according to the nonlinear static analysis, while it is relevant when the dynamic analysis is performed.

THE STRUCTURAL CAPACITY AS A FUNCTION OF THE CONCRETE STRENGTH

The structural capacity of existing structure, as regards ductile mechanisms, is quantified in terms of chord rotation. Different chord rotation limit values are provided for the different limit states.

In Figs 10 and 11 the limit values of chord rotations for the two considered limit states have been shown, on varying of axial load, for both the conventional strength values provided by EC8 and for the assumed concrete strength domains. As can be noted, the range of capacities corresponding to the conventional strength values provided by EC8 (identified by the grey area in the graphs of Fig. 10) is smaller than the one obtained when the strength variability is considered even for the lowest assumed *CoV* (*CoV* = 15%). For higher values of *CoV*, more likely to be found in existing buildings (see De Stefano et al. 2013, 2014), the difference between the two ranges is even more relevant. It should be noted the different limit chord rotation trend at the axial load variation for the two limit states. In fact, the limit chord rotation of serviceability limit state (*DL*) increases for low amount of axial load (between 200 kN and 800 kN depending on the concrete mechanical properties), and it decreases when the peak axial load is overcome. The limit value of ultimate limit state (*LS*), instead, regolarly decreases at the increase of axial load.

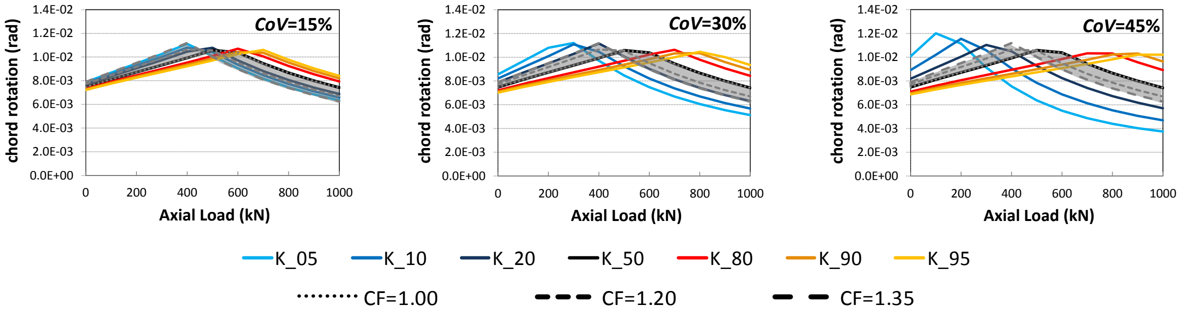


Figure 10. Limit chord rotation: *DL* limit state

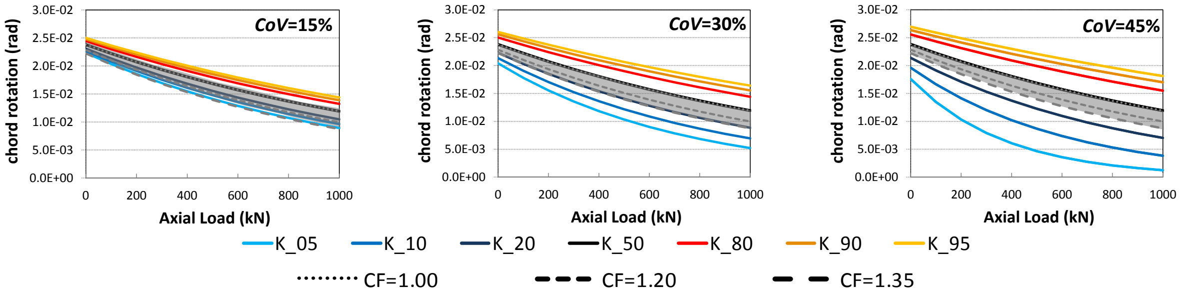


Figure 11: Limit chord rotation; *LS* limit state

THE SEISMIC PERFORMANCE

In this section the effect of the axial load variation on the seismic performance of the case study has been investigated, both for the static and the dynamic analyses. Both the limit values found for the columns, according to EC8 prescriptions and to the assumed strength variability, have been considered and compared.

Figs 12 and 13 shows, for all the considered PGAs, the ratio between the demand (D) found by performing the two analysis (static and dynamic) and capacity (C) defined by considering the conventional values provided by EC8. The quantity D/C has been assumed as a measure of the seismic performance of the building, since for D/C value below the unity the structure respects the seismic prescriptions. For sake of brevity, only results obtained for four columns have been shown. The selected columns are a border and an internal one for a side frame (frame 1) and the corresponding ones for the internal frame (frame 2). It should be noted that, even if the frame 3 is not shown in the figure, its behaviour is not exactly the same than the one of frame 1.

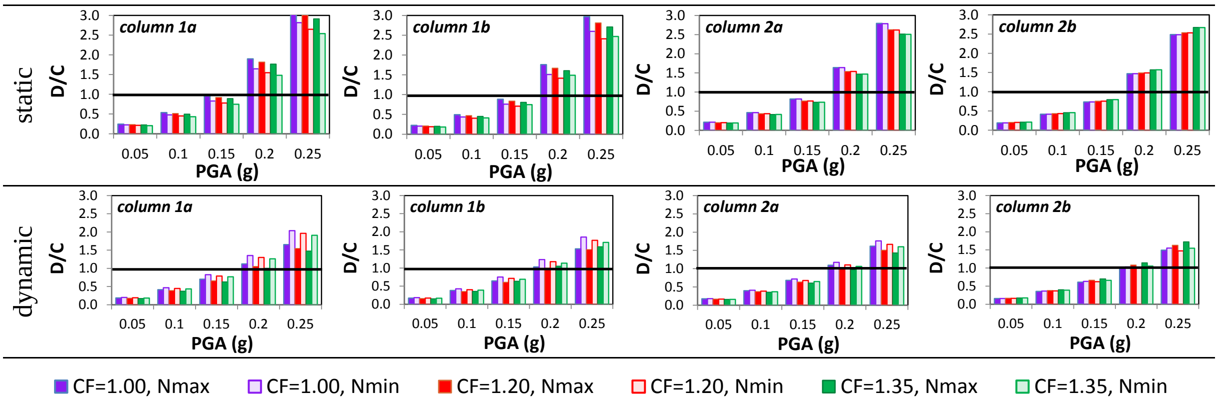


Figure 12. Seismic performance (*DL* limit state) by considering the conventional values provided by EC8.

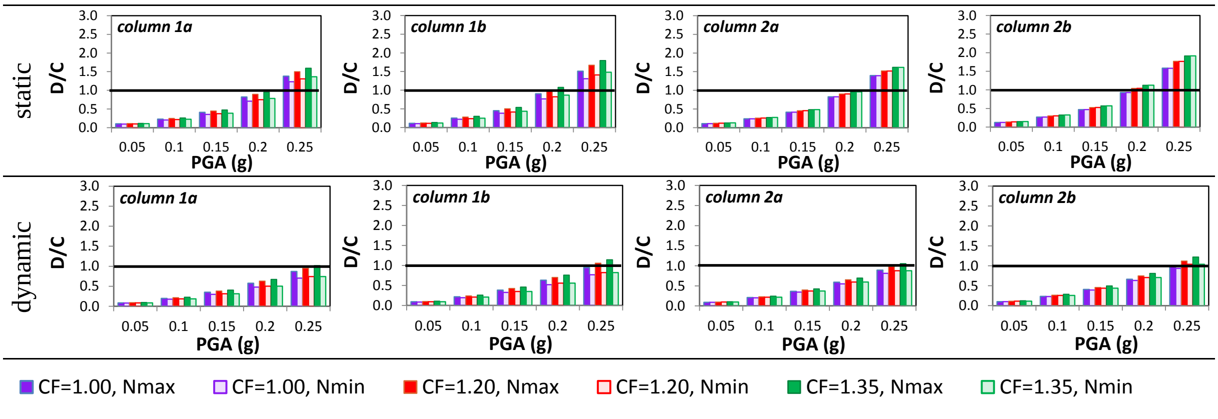


Figure 13. Seismic performance (*LS* limit state) by considering the conventional values provided by EC8.

In the performance evaluation the capacity has been measured with both the limit values of axial load (i.e. minimum and maximum) in each column, and both the performance values have been shown. As it was expectable, the D/C values found by performing the static analysis are higher than the ones coming from the dynamic analysis. The scatter between the minimum and maximum values of axial load is similar in the two analysis for the frame 1, while in the frame 2 there is not scatter when the static analysis is considered, since it does not provide axial load variation in the central frame.

Figs 14-17 show the same performance index (D/C) values found by considering for the concrete the strength values representing the assumed strength samples, with three different *CoV* values. Figs 14 and 15 show the performance index (D/C) found by performing the two analyses, static and dynamic, for the *DL* limit state, while in Figs 16 and 17 the same index is shown for the *SL* limit state.

In the figures the results corresponding to the mean strength value (K50), which is included in the sample representation, are not shown, since they coincide to those found for *CF*=1.00.

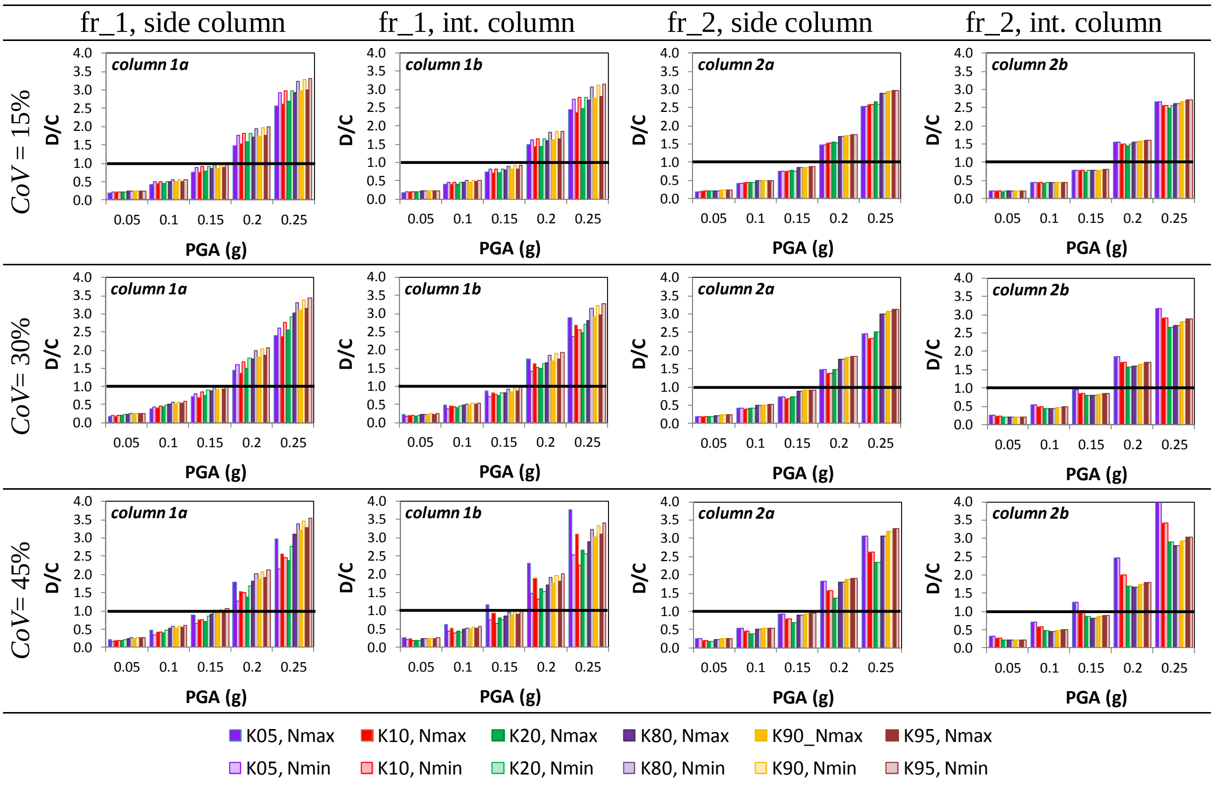


Figure 14. Seismic performance according to the static analysis: *DL* limit state.

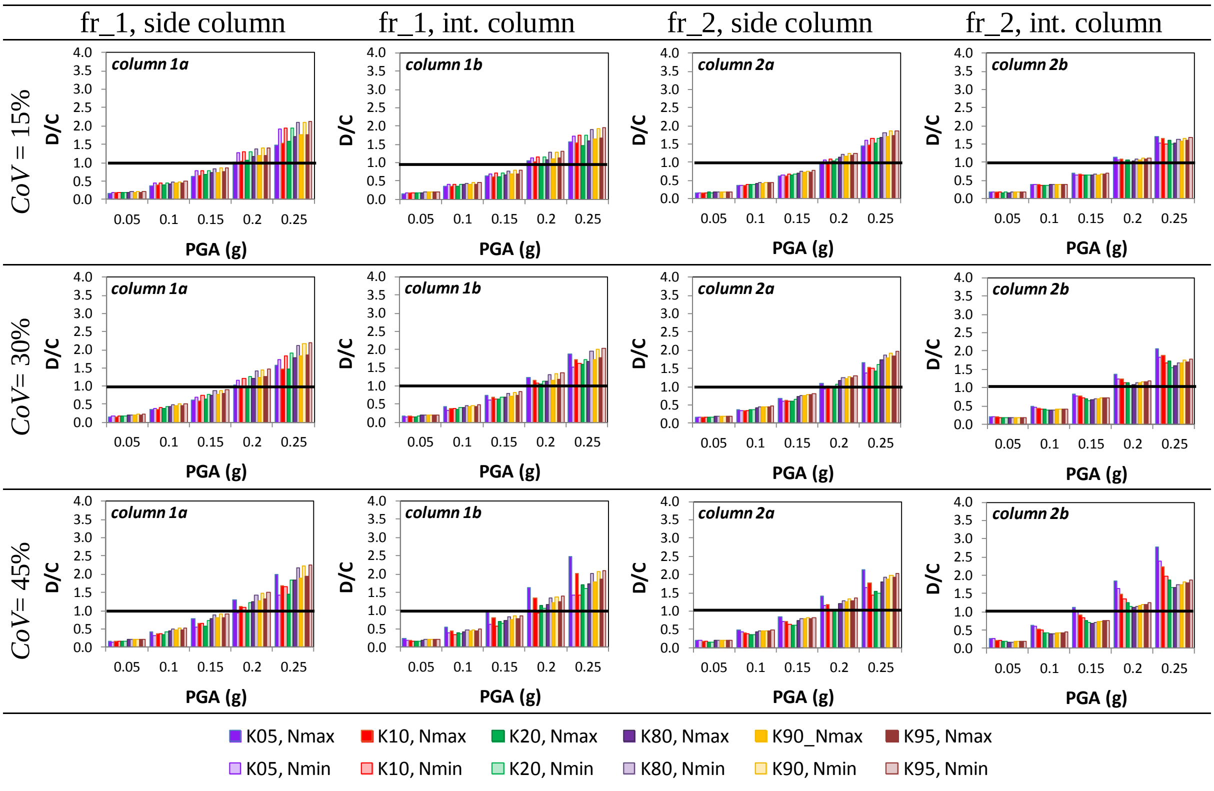


Figure 15. Seismic performance according to the dynamic analysis: *DL* limit state.

As can be noted, when the concrete strength is assumed to be variable, and it is represented by a sample of percentile values, the seismic performance of the structure decreases as a function of the considered percentile.

As regards the *LS* limit state, when the conventional strength values provided by EC8 are considered, the value of D/C is below 2 when the static analysis is performed, and it just overcomes the unity when found by the dynamic analysis.

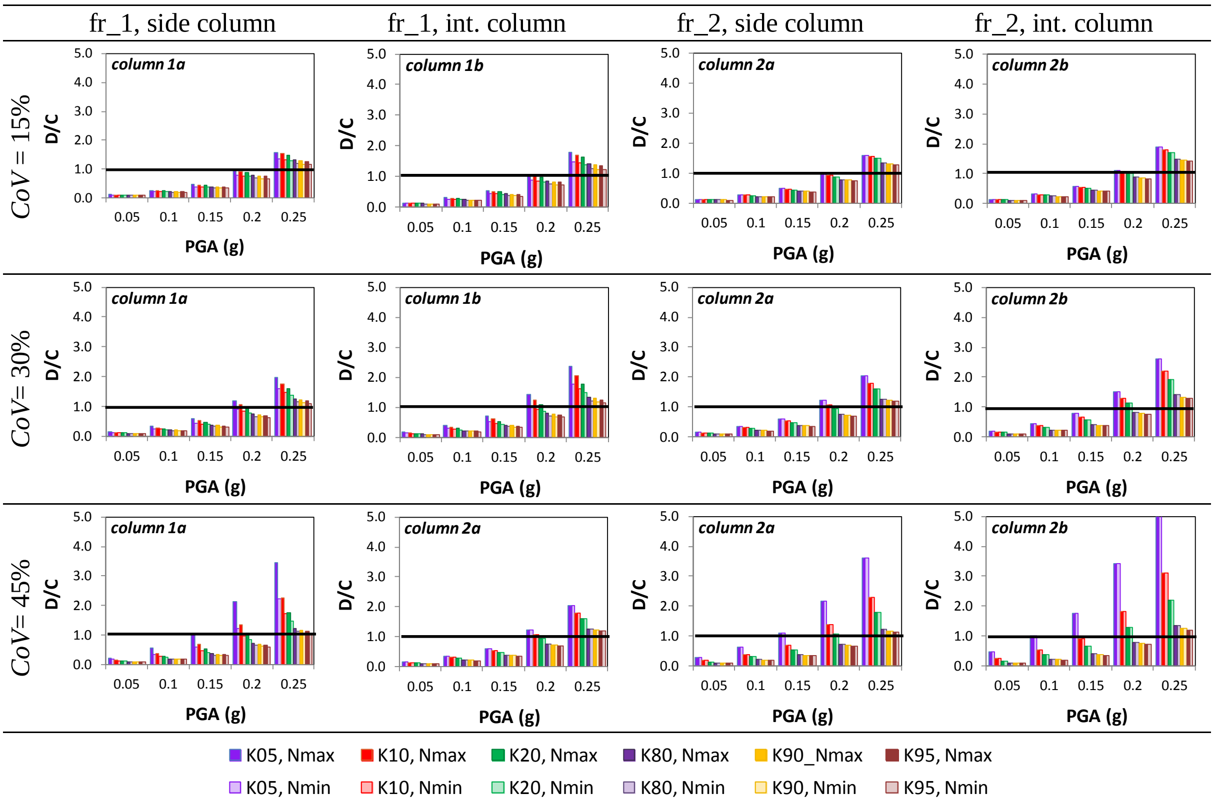


Figure 16. Seismic performance according to the static analysis: *LS* limit state.

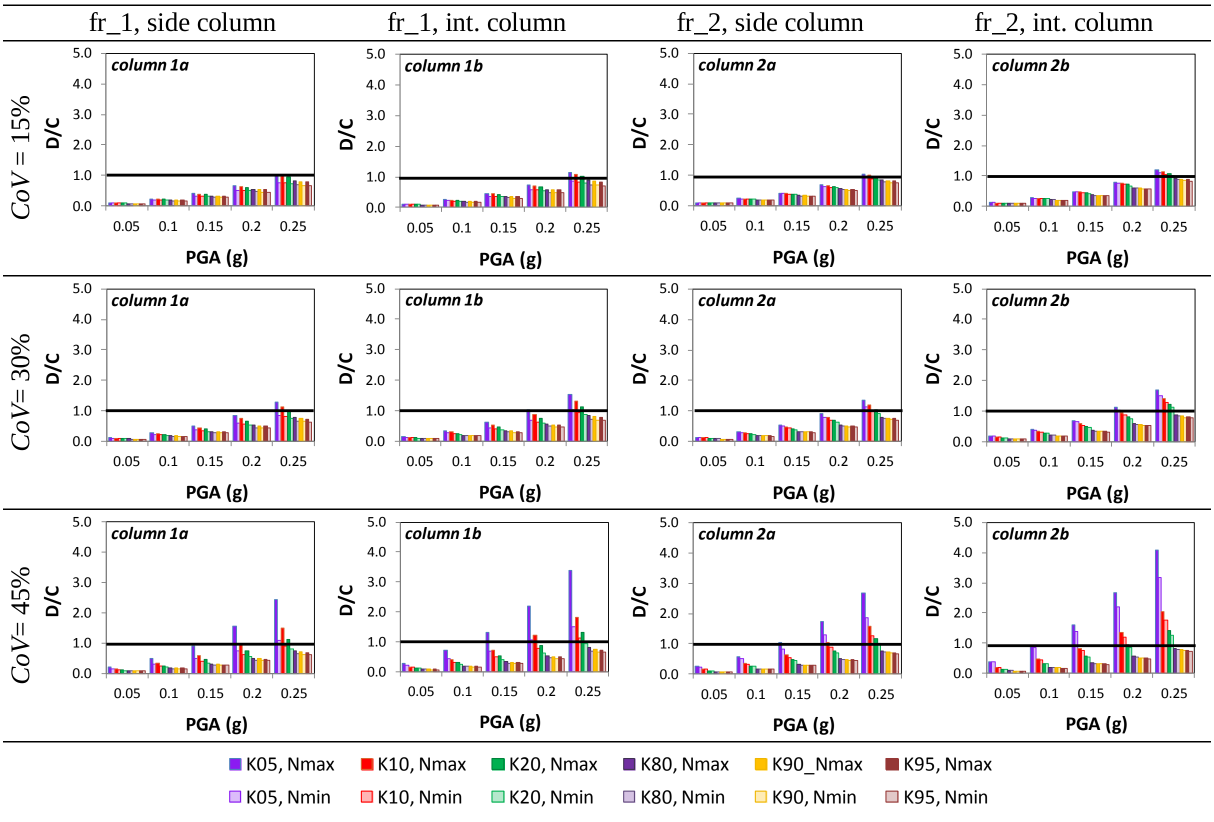


Figure 17. Seismic performance according to the dynamic analysis: *LS* limit state.

When a variable concrete strength is considered, D/C achieves the values of 5 (static) and 4 (dynamic) with static and dynamic analysis respectively. In both cases the performance index shows to be very sensitive to the concrete strength characterization. Moreover, when a high CoV (45%) is considered for the concrete strength, the scatter in performance index values due to the axial load variation is large, suggesting a relevant correlation between the sensitivity to axial load and the effective strength of the column.

When the *DL* limit state is considered, instead, the scatter in the quantity D/C due to the axial load variation does not seem to be sensitive to the concrete strength characterization. In fact, the scatter in performance index is much smaller than the one found for the *LS* limit state. Even in this case, anyway, by introducing the concrete strength variability, an increase in values assumed by the index D/C can be noted.

CONCLUSIONS

In this paper the effects of the axial load variation on the seismic performance have been investigated on a case-study, i.e. a 4-storey RC framed building. The effect of the axial load variation has been investigated with regards to two different aspects: the role of the type of analysis, to check how the dynamic amplification can increase the axial load amount in each column, and the role of the concrete strength characterization, since the assumed strength value can affect very much the sensitivity of the columns to the axial load amount. The investigation has shown that the nonlinear dynamic analysis has introduced an axial load variation, comparing to the one coming from the nonlinear static analysis, ranging between 20% and 60%, depending on the position of the column and its initial (static) amount of axial load. In particular, while the static analysis has provided a mirrored axial load variation in the two side frames, so that the increase in one side frame corresponds to the decrease in the other one (since the case-study is symmetric), the dynamic analysis has provided a not symmetric axial load variation in the structure. As a results of such irregular variation, the amount of axial load increase and decrease coming from the two analyses is, quantitatively, very different. Moreover, the static analysis does not provide any axial load variation in the central frame, that is the most loaded, while the dynamic analysis shows a 20% variation even in such columns.

The second aspect investigated in the work is the role of the concrete strength on the sensitivity to axial load variation. A *mean* concrete strength equal to 19.36 MPa has been assumed for the concrete, and two different approaches, the first one consistent to EC8, the second one based on a wise experimental campaign, have been applied. When the EC8 approach is considered, three different conventional strength values have been considered for the concrete, found by scaling the assumed strength to the *CF* values as provided by EC8. Alternatively, the concrete strength is described as a variable quantity, by means of a normal distribution made of 7 values with different percentiles. Three different *CoV*, consistent to the experimental observation, have been considered. The capacity of each column for different amounts of axial load has been found for each strength value, i.e. the three values consistent to EC8 and the 7 values constituting the sample, and the scatter in capacity due to the axial load variation has been measured in all cases.

The effect of axial load variation has been finally evaluated in terms of seismic performance, by considering two limit states: Damage Limitation (*DL*) and Life Safety (*LS*). Even if the dynamic analysis is more sensitive to axial load variation, the D/C provided by the dynamic analysis is smaller than the one provided by the static analysis, since the static analysis, as it was expectable, provides a larger response of the case-study than the dynamic one. When the concrete strength variability is introduced in the analysis, the values found for the D/C index increases very much, both when coming from static and dynamic analyses. A different sensitivity to the axial load variation has been observed for the two limit state, being the ultimate (*LS*) limit state more sensitive to the axial load variation than the serviceability (*DL*) one, especially when the concrete strength variability is considered. In all cases, anyway, the performance estimation obtained by considering an high strength variability (*CoV*=45%) is much lower than the one found by assuming the conventional strength values provided by EC8. The D/C, in fact, is 3.5 times (static) and 4 times (dynamic) higher when the strength variability is introduced. When the lowest percentile (K05) is considered, together to the highest *CoV* (*CoV*=45%), the scatter in D/C due to the axial load variation achieves, for high PGAs, 100%.

REFERENCES

Abbasnia, R., Mirzadeh, N., Kildashti, K. Assessment of axial force effect on improved damage index of confined RC beam-column members. International Journal of Civil Engineering, Vol. 9, No. 3, September 2011.

Bosco, M., Ghersi, A. Marino, E. (2013b). Corrective eccentricities for assessment by the nonlinear static method of 3D structures subjected to bidirectional ground motions. Earthquake Eng. Struct. Dyn. 2012; 41:1751–1773.

Bosco, M., Ghersi, A., Marino, E.M. (2009). On the Evaluation of Seismic Response of Structures by Nonlinear Static Methods. Earthquake Engineering & Structural Dynamics, John Wiley & sons, Ltd., Vol. 38/13, pp. 1465-1482.

Bosco, M., Marino, E., Rossi, P.P. (2013a). An analytical method for the evaluation of the in-plan irregularity of non-regularly asymmetric buildings. Bull Earthquake Eng. DOI 10.1007/s10518-013-9438-3.

Chopra, A.K. and Goel. R.K. 2002. A modal pushover analysis procedure for estimating seismic demand for buildings. Earth. Eng. Struct. Dyn. 31(3):561-582.

Cosenza, E., Monti, G. (2009). Assessment and reduction of the vulnerability of existing reinforced concrete buildings. Atti Convegno finale ReLuis, pp. 51-110.

Cristofaro, M.T. (2009). Metodi di valutazione della resistenza a compressione del calcestruzzo di strutture in c.a. esistenti. PhD Dissertation, Università di Firenze.

Cristofaro, M.T., D'Ambrisi, A., De Stefano, M., Pucinotti, R., Tanganelli, M., (2012). Studio sulla dispersione dei valori di resistenza a compressione del calcestruzzo di edifici esistenti. Il giornale delle prove non distruttive, monitoraggio, diagnostica., vol. 2/2012, pp. 32-39, ISSN: 1721-7075.

D'Ambrisi, A., De Stefano, M., Tanganelli, M. (2009). Use of pushover analysis for predicting seismic response of irregular buildings: a case study. Journal of Earthquake Engineering, Taylor & Francis, Vol. 13, pp. 1089-1100.

D'Ambrisi, A., De Stefano, M., Tanganelli, M., Viti, S. (2013). Influence of the variability of concrete mechanical properties on the seismic response of existing RC framed structures.. In: M. De Stefano, O. Lavan. Seismic behaviour and design of irregular and complex civil structures. p. 59-72, Dordrecht, Heidelberg, New York, London:Springer, ISBN: 9789400753761, doi: 10.1007/978-94-007-5377-8

De Stefano, M., Tanganelli, M. Viti, S. (2104). Variability in concrete mechanical properties as a source of in-plan irregularity for existing RC framed structures. Engineering Structures Volume 59, February 2014, Pages 161–172

De Stefano, M., Tanganelli, M., Viti, S. (2013). Effect of the variability in plan of concrete mechanical properties on the seismic response of existing RC framed structures. BULLETIN OF EARTHQUAKE ENGINEERING, vol. 11 Issue 4, p. 1049-1060, ISSN: 1573-1456, doi: 10.1007/s10518-012-9412-5

De Stefano, M., Tanganelli, M., Viti, S. (2013). On the variability of concrete strength as a source of irregularity in elevation for existing RC buildings: a case study. BULLETIN OF EARTHQUAKE ENGINEERING, vol. 11 Issue 5, p. 1711-1726, ISSN: 1573-1456, doi: 10.1007/s10518-013-9463-2

Fajfar, P. Marusic, D. and Perus, I. 2005. Torsional effects in the pushover-based seismic analysis of buildings. J. Earthquake Engineering 9(6): 831-854.

Faramarz Khoshnudian and Danoush Motamedi. Seismic Response of Asymmetric Steel Isolated Structures Considering Vertical Component of Earthquakes. KSCE Journal of Civil Engineering (2013) 17(6):1333-1347 DOI 10.1007/s12205-013-0115-5

Itaca. (2008). Database of the Italian strong motions data. http://itaca.mi.ingv.it.

Jalayer F., Iervolino I., Manfredi G. (2008). Structural Modeling: uncertainties and their Influence on Seismic Assessment of Existing RC Structures. Structural Safety, 2008.

Magliulo, G., Maddolani, G., Cosenza, E. 2012. Extension of N2 method top plan irregular buildings considering accidental eccentricity", Soil Dyn. Earth. Eng. 43:69-84.

Magliulo, G., Ramasco, R., Capozzi, V. (2012). Seismic performance of R/C frames with overstrength discontinuities in elevation, Bull Earthquake Eng. , vol. 10, pp. 679–694.

Mander, J.B, Priestley, M.J.N, Park, R. (1988). Theoretical stress-strain model for confined concrete. Journal of the Structural Division ASCE, vol. 114, pp. 1804-1826.

Masi, A. Vona, M. Manfredi, V. (2008). A parametric study on RC existing buildings to compare different analysis methods considered in the European seismic code (EC8-3), Proc. 14th WCEE, Beijing, China.

Saadeghvaziri, M. A. (1997). Nonlinear response and modelling of RC columns subjected to varying axial load. Engineering Structures, Vol. 19, No. 6, pp. 417 424.

Seismosoft 2006. Seismostruct Version 5.2.2 - A computer program for static and dynamic nonlinear analysis of framed structures. Available online from URL: www.seismosoft.com.