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# Torsional effects due to concrete strength variability in existing buildings

M. De Stefano[a], M. Tanganelli[b] and S. Viti[*]

*Department of Architecture (DiDA), University of Florence, Italy*

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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 torsional effects related to the irregular stiffness and strength distribution 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 stiffness distributions, have been considered, and a statistical analysis has been performed on the induced torsional effects. The obtained results have been compared to the standard analysis as provided by Eurocode 8 for existing buildings, showing that the Eurocode 8 provisions, despite not allowing explicitly for material strength variability, are conservative as regards the estimation of structural demand.

**Keywords:** RC framed structures; plan irregularity; torsional effects; concrete mechanical properties; existing buildings; stiffness center; strength center

## 1. Introduction

In Europe, current years are characterized by a strong reduction in the building activity due to the peak occurred in the last decades, after the end of WWII. Moreover, many buildings, due to the lack of technical regulations and controls during construction, present poor material mechanical characteristics with large scatters even within a single building.

One of the most crucial technical issues is, therefore, the evaluation of the seismic safety of existing buildings, which involves a suitable characterization of actual material properties.

Especially in RC buildings, the homogeneity of the material within each structure cannot be assumed; as an example, an experimental campaign performed by the Regional Government of Tuscany proved that strength variability inside single buildings can result in values of Coefficient

∗Corresponding author, Ph.D., E-mail: viti@unifi.it
[a]Professor, E-mail: mario.destefano@unifi.it
[b]Ph.D., E-mail: marco.tanganelli@unifi.it


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of Variation (*CoV*) over 30%. Furthermore, an exhaustive characterization of the concrete mechanical properties is hard to achieve, since it would require a large number of destructive and non-destructive tests (Cristofaro *et al*. 2012, 2015).

International technical legislation provides different criteria to identify the required mechanical properties of concrete. The European technical Code, Eurocode 8 (EC8) indicates the *mean* strength value obtained from the *in-situ* tests as the one to use for 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 (EC8, NTC 2008) depending on the knowledge level of the structure. The safest approach provided by EC8, therefore, consists of evaluating the structural response by using the *mean* concrete strength, and to compare member forces and deformations to their limit values found by assuming a *CF*=1.35. The amount of strength variability, as represented by *CoV*, does not affect anyway the analysis.

The American FEMA 356 (Applied Technology Council 2000), instead, takes into account explicitly the strength variability of concrete. When the *CoV* of the concrete strength exceeds the limit value of 14%, in fact, the strength value to use both in analysis and in verification is assumed as the difference between the *mean* and the *standard deviation* of the strength domain.

In these last years, many Authors have been investigating about concrete strength characterization, focusing their attention both on the parameters to be assumed in the characterization (Masi and Vona 2009) and on the variability of the strength, with the consequent assumption of a value for analysis (Franchin *et al*. 2007, 2009, Fardis 2009, Masi *et al*. 2008, Rajeev *et al*. 2010, Jalayer *et al*. 2008, Cosenza and Monti 2009, Marano *et al*. 2008, Monti *et al*. 2007, D'Ambrisi *et al*. 2013a, 2013b).

In this paper the concrete strength variability has been investigated on a case-study as possible source of in-plan irregularity, and the torsional effects related to such irregularity have been evaluated. The case-study is a 4-storey RC building, which represents a simple and typical example of pre-seismic normative structure. Each member has been designed to vertical loads only, according to the allowable stress criterion. The concrete strength has been characterized on the base of a large database provided by the Regional Government of Tuscany (Cristofaro 2009), since it is very large and it includes buildings made in different decades, ranging between 1950 and 1980, in a homogeneous area. The strength variability has been considered in 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. This investigation follows some previous works made by the authors (De Stefano *et al*. 2013a, 2014) on a similar case-study. From the previous studies the strength variability proved to largely affect both the seismic demand and performance of existing buildings. In those previous studies, however, only two "extreme", very skewed, in-plan strength distributions have been considered. Therefore, only the largest possible effects related to the assumed strength variability have been found, while no information was found on the probability of occurrence of such effects. In this work, instead, a significant number (180) of different in-plan distributions have been considered, qualitatively representing all the possible combinations among the values of the strength domain. Depending on the considered in-plan strength distributions, different values of eccentricity have been found; both strength and stiffness eccentricities (Bosco *et al.* 2012, 2013) have been considered in analysis. The torsional effects induced by the considered strength distributions have been found by performing both a modal elastic analysis and a nonlinear static (pushover) one, to check the role of inelasticity on the torsional effects (Bugeja *et al*. 1999, Peruš and Fajfar 2005). Special attention has been paid to the variability of the structural response, measured both in terms of maximum drift and torsional effects, as a function of the variability assumed for the concrete strength.

## 2. A case-study with concrete strength variability

### *2.1 Case-study*

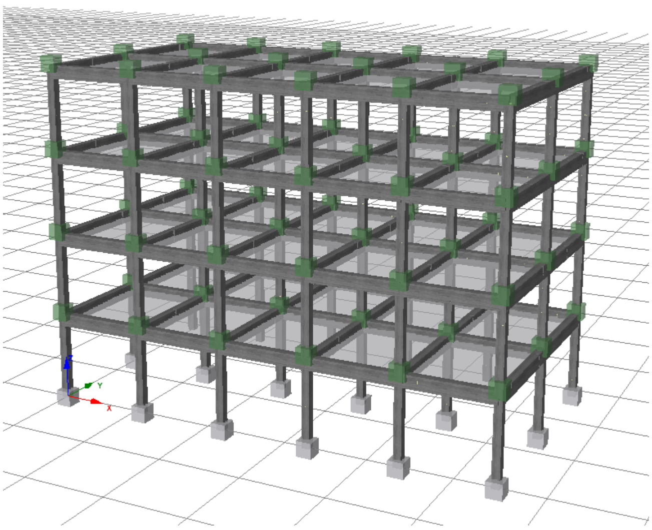
The sample structure is a 4 story 3D reinforced concrete frame with two 4.5 m long bays in the *y*-direction and 5 bays 3.5 m long in the *x*-direction, as shown in Fig. 1. The building is symmetric along both *x* and y directions.

All the columns are 3.20 meters long; they have a cross dimension of 30×30 cm, while their longitudinal reinforcement consists of 8 $\phi$14 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 a dimension of 30×50 cm in both directions.

The concrete strength ($f_c$), having a mean strength equal to 19.36 MPa, has been described according to the statistical properties shown in the next section, 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 all horizontal actions, like seismic and wind loads. Vertical loads consist of dead loads equal to 5.9 KN/m$^2$ and of live loads equal to 2

Table 1 Periods and participating masses of the structure

| | Period (sec) | mode | participating mass |
|---|---|---|---|
| 1$^{st}$ period | 0.777 | Translational, *y* | 87.42% |
| 2$^{nd}$ period | 0.735 | Translational, *x* | 88.14% |
| 3$^{rd}$ period | 0.694 | Torsional, *z* | 87.87% |



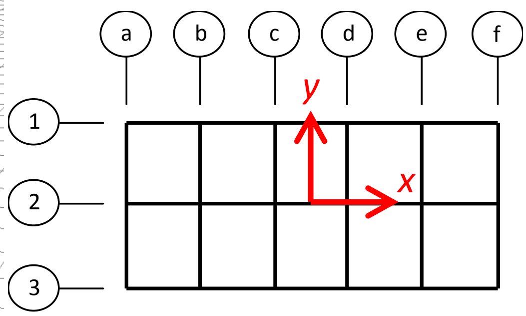



Fig. 1 Case-study: 3D view and plan configuration

KN/m$^2$. The structure has a fundamental period equal to 0.777 sec (see Table 1), related to a pure translational mode in the *y* direction. The dynamic properties of the case-study have been found by assuming a rigid diaphragm at each storey, while the stiffness of each member has been found by considering the integer section, since the not-reduced Young modulus has been adopted.

## 2.2 Concrete characterization

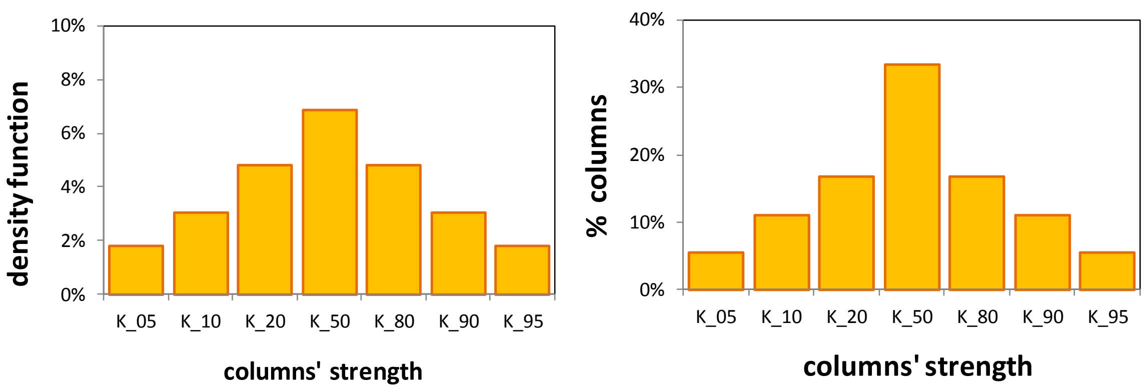
### 2.2.1 Concrete strength properties according to experimental data

According to EC8, the *mean* value of $f_c$ 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 of the structure, ranging between 1.00, when a fully satisfactory knowledge of the structure is achieved, and 1.35 (Italian National Annex), for partial or unsatisfactory knowledge level. 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* test to assume the *CF* equal to1.00. 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.

In this work the concrete strength has been statistically modeled on the basis of a database (Cristofaro 2009), collected by the Seismic Agency of Tuscan Government, which encompasses about 300 buildings, and over 1000 destructive and non-destructive tests (Cristofaro *et al*. 2012). The statistical parameters have been found by considering buildings which have a minimum of three test results. The *CoV* of specimens taken from each building ranges in a large interval, reaching 50% within all the three (60s, 70s, 80s) considered decades.

### 2.2.2 The assumed strength domain

The assumed strength domain, shown in Fig. 2(a), 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), ranging between 15% and 45%. The domain consists of 7 values, corresponding to the percentiles of 5%, 10%, 20%, 50%, 80%, 90% and 95% respectively.




(a) Density function representing the concrete strength

(b) Percentage of columns with the different strength values

Fig. 2 The assumed concrete strength distribution

Such strength values have been given to each of the columns of the first storey, in order to obtain, globally, the assumed mean and *CoV* values. Fig. 2(a) shows the probability of occurrence of each percentile value for the 1st storey columns, while Fig. 2(b) represents the percentage of columns having the assumed strength values.

#### *2.2.3 The assumed strength distribution models*

In the current investigation, it has been chosen to introduce strength plan variability along the *x*-direction only, thus considering one-way plan asymmetric building models. Therefore, seismic analysis has been performed in the *y*-asymmetric direction only. For the sake of simplicity, the concrete strength variability has been assumed to affect the columns, while for beams the *mean* value of $f_c$ has been taken. Besides, the strength variability has been introduced at the columns of the 1st storey only, which is the level which evidences a higher seismic response. In fact, previous investigations (De Stefano *et al*. 2013a, 2013b) have shown that the maximum interstorey drift found by introducing the strength variability at all storeys is very similar to that found by introducing it at the first storey only. Six groups of 30 schemes each, corresponding to 180 layouts, have been considered. Each group is named after the position of the weakest column ($f_c = f_{k05}$), which is assumed for each of the six column-lines (a, b, c, d, e, f, g) of the case study, and it covers all the possible positions along the central *x*-axis (column line No. 2, according to the scheme in Fig. 1), i.e., 2a, 2b, 2c, 2d, 2e and 2f, as it is shown in Fig. 3 for the layouts of the 1st group (weakest column in 2a, models A01-A30). For each position of the weakest column, all the most significant strength combinations have been considered. Special attention has been paid to

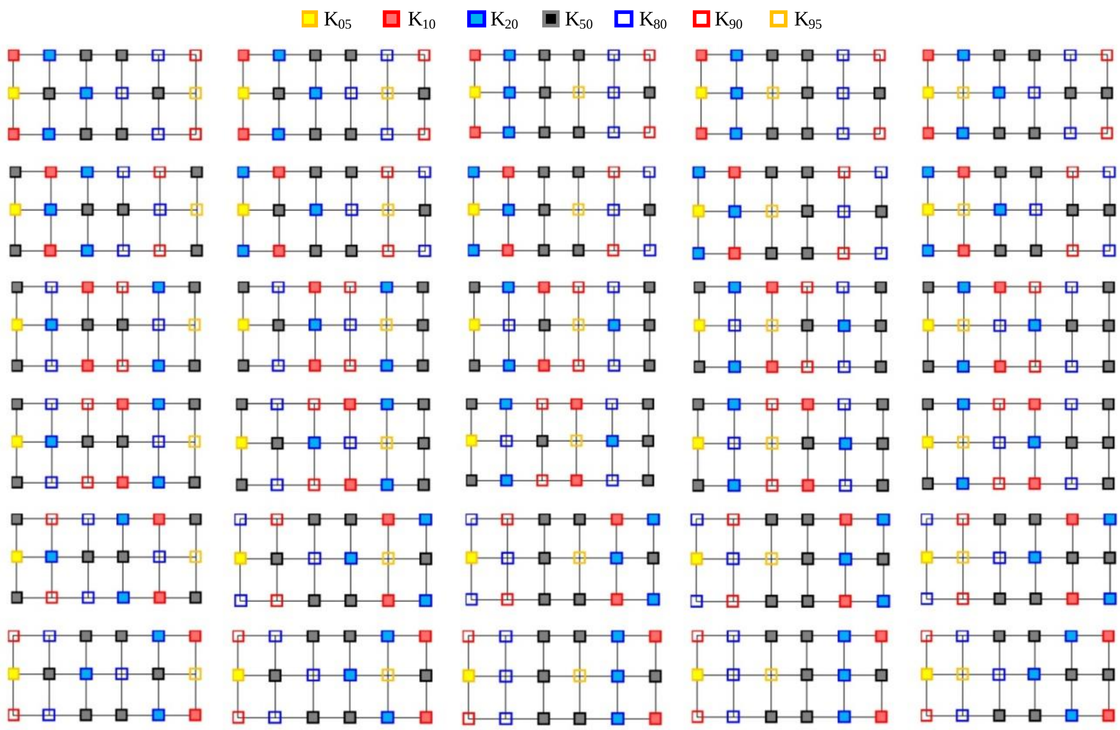



Fig. 3 Models A01-A30: weakest column in the a2 position

Table 2 The considered layouts (first 90 models)

| | $K_{05}$ | $K_{10}$ | $K_{10}$ | $K_{20}$ | $K_{20}$ | $K_{20}$ | $K_{50}$ | $K_{50}$ | $K_{50}$ | $K_{50}$ | $K_{50}$ | $K_{50}$ | $K_{80}$ | $K_{80}$ | $K_{80}$ | $K_{90}$ | $K_{90}$ | $K_{95}$ | | $K_{05}$ | $K_{10}$ | $K_{10}$ | $K_{20}$ | $K_{20}$ | $K_{20}$ | $K_{50}$ | $K_{50}$ | $K_{50}$ | $K_{50}$ | $K_{50}$ | $K_{50}$ | $K_{80}$ | $K_{80}$ | $K_{80}$ | $K_{90}$ | $K_{90}$ | $K_{95}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| A01 | A2 | A1 | A3 | B1 | B3 | C2 | B2 | C1 | C3 | D1 | D3 | E2 | D2 | E1 | E3 | F1 | F3 | F2 | B16 | B2 | D1 | D3 | B1 | B3 | D2 | A1 | A2 | A3 | E2 | F1 | F3 | C2 | E1 | E3 | C1 | C3 | F2 |
| A02 | A2 | A1 | A3 | B1 | B3 | C2 | B2 | C1 | C3 | D1 | D3 | F2 | D2 | E1 | E3 | F1 | F3 | E2 | B17 | B2 | D1 | D3 | C2 | E1 | E3 | A1 | A2 | A3 | F1 | F2 | F3 | B1 | B3 | D2 | C1 | C3 | E2 |
| A03 | A2 | A1 | A3 | B1 | B3 | B2 | C2 | C1 | C3 | D1 | D3 | F2 | E2 | E1 | E3 | F1 | F3 | D2 | B18 | B2 | D1 | D3 | B1 | B3 | F2 | A1 | A3 | C2 | E2 | F1 | F3 | A2 | E1 | E3 | C1 | C3 | D2 |
| A04 | A2 | A1 | A3 | B1 | B3 | B2 | C1 | C3 | D1 | D2 | D3 | F2 | E2 | E1 | E3 | F1 | F3 | C2 | B19 | B2 | D1 | D3 | B1 | B3 | F2 | A1 | A3 | D2 | E2 | F1 | F3 | A2 | E1 | E3 | C1 | C3 | C2 |
| A05 | A2 | A1 | A3 | B1 | B3 | C2 | B2 | C1 | C3 | D1 | D3 | E2 | D2 | E1 | E3 | F1 | F3 | F2 | B20 | B2 | D1 | D3 | B1 | B3 | C2 | A1 | A3 | E2 | F1 | F2 | F3 | C2 | E1 | E3 | C1 | C3 | A2 |
| A06 | A2 | B1 | B3 | B2 | C1 | C3 | A1 | A3 | C2 | D2 | F1 | F3 | D1 | D3 | E2 | E1 | E3 | F2 | B21 | B2 | E1 | E3 | D2 | F1 | F3 | A2 | C1 | C3 | D1 | D3 | E2 | A1 | A3 | C2 | B1 | B3 | F2 |
| A07 | A2 | B1 | B3 | A1 | A3 | C2 | B2 | C1 | C3 | D1 | D3 | F2 | F1 | F3 | D2 | E1 | E3 | E2 | B22 | B2 | E1 | E3 | D1 | D3 | F2 | A1 | A3 | C2 | D2 | F1 | F3 | A2 | C1 | C3 | B1 | B3 | E2 |
| A08 | A2 | B1 | B3 | A1 | A3 | B2 | C1 | C2 | C3 | D1 | D3 | F2 | F1 | F2 | E2 | E1 | E3 | D2 | B23 | B2 | E1 | E3 | D1 | D3 | F2 | A1 | A3 | C2 | E2 | F1 | F3 | A2 | C1 | C3 | B1 | B3 | D2 |
| A09 | A2 | B1 | B3 | A1 | A3 | B2 | C1 | C3 | D1 | D2 | D3 | F2 | F1 | F2 | E2 | E1 | E3 | C2 | B24 | B2 | E1 | E3 | D1 | D3 | F2 | A1 | A3 | D2 | E2 | F1 | F3 | A2 | C1 | C3 | B1 | B3 | C2 |
| A10 | A2 | B1 | B3 | A1 | A3 | C2 | C1 | C3 | D1 | D3 | E2 | F2 | F1 | F2 | D2 | E1 | E3 | B2 | B25 | B2 | E1 | E3 | D2 | F1 | F3 | C1 | C3 | D1 | D3 | E2 | F2 | A1 | A3 | C2 | B1 | B3 | A2 |
| A11 | A2 | C1 | C3 | B2 | E1 | E3 | A1 | A3 | C2 | D2 | F1 | F3 | B1 | B3 | E2 | D1 | D3 | F2 | B26 | B2 | F1 | F3 | D2 | E1 | E3 | C1 | C3 | D1 | D3 | A2 | E2 | B1 | B3 | C2 | A1 | A3 | F2 |
| A12 | A2 | C1 | C3 | C2 | E1 | E3 | A1 | A3 | B2 | F1 | F2 | F3 | B1 | B3 | D2 | D1 | D3 | E2 | B27 | B2 | F1 | F3 | E1 | E3 | F2 | C1 | C2 | C3 | D1 | D2 | D3 | A2 | B1 | B3 | A1 | A3 | E2 |
| A13 | A2 | C1 | C3 | B1 | B3 | E2 | A1 | A3 | C2 | F1 | F2 | F3 | B2 | E1 | E3 | D1 | D3 | D2 | B28 | B2 | F1 | F3 | E1 | E3 | F2 | C1 | C2 | C3 | D1 | D3 | E2 | A2 | B1 | B3 | A1 | A3 | D2 |
| A14 | A2 | C1 | C3 | B1 | B3 | E2 | A1 | A3 | D2 | F1 | F2 | F3 | B2 | E1 | E3 | D1 | D3 | C2 | B29 | B2 | F1 | F3 | E1 | E3 | F2 | C1 | C3 | D1 | D2 | D3 | E2 | A2 | B1 | B3 | A1 | A3 | C2 |
| A15 | A2 | C1 | C3 | B1 | B3 | D2 | A1 | A3 | E2 | F1 | F2 | F3 | C2 | E1 | E3 | D1 | D3 | B2 | B30 | B2 | F1 | F3 | D2 | E1 | E3 | C1 | C3 | D1 | D3 | E2 | F2 | B1 | B3 | C2 | A1 | A3 | A2 |
| A16 | A2 | D1 | D3 | B2 | E1 | E3 | A1 | A3 | C2 | D2 | F1 | F3 | B1 | B3 | E2 | C1 | C3 | F2 | C01 | C2 | A1 | A3 | B1 | B2 | B3 | A2 | C1 | C3 | D1 | D2 | D3 | E1 | E2 | E3 | F1 | F3 | F2 |
| A17 | A2 | D1 | D3 | C2 | E1 | E3 | A1 | A3 | B2 | F1 | F2 | F3 | B1 | B3 | D2 | C1 | C3 | E2 | C02 | C2 | A1 | A3 | A2 | B1 | B3 | B2 | C1 | C3 | D1 | D2 | D3 | E1 | E3 | F2 | F1 | F3 | E2 |
| A18 | A2 | D1 | D3 | B1 | B3 | E2 | A1 | A3 | C2 | F1 | F2 | F3 | B2 | E1 | E3 | C1 | C3 | D2 | C03 | C2 | A1 | A3 | A2 | B1 | B3 | B2 | C1 | C3 | D1 | D3 | E2 | E1 | E3 | F2 | F1 | F3 | D2 |
| A19 | A2 | D1 | D3 | B1 | B3 | E2 | A1 | A3 | D2 | F1 | F2 | F3 | B2 | E1 | E3 | C1 | C3 | C2 | C04 | C2 | A1 | A3 | A2 | B1 | B3 | C1 | C3 | D1 | D2 | D3 | E2 | E1 | E3 | F2 | F1 | F3 | B2 |
| A20 | A2 | D1 | D3 | B1 | B3 | D2 | A1 | A3 | E2 | F1 | F2 | F3 | C2 | E1 | E3 | C1 | C3 | B2 | C05 | C2 | A1 | A3 | B1 | B2 | B3 | C1 | C3 | D1 | D2 | D3 | F2 | E1 | E2 | E3 | F1 | F3 | A2 |
| A21 | A2 | E1 | E3 | D1 | D3 | E2 | A1 | A3 | C2 | D2 | F1 | F3 | B2 | C1 | C3 | B1 | B3 | F2 | C06 | C2 | B1 | B3 | A1 | A3 | B2 | A2 | C1 | C3 | D1 | D2 | D3 | E2 | F1 | F3 | E1 | E3 | F2 |
| A22 | A2 | E1 | E3 | D2 | F1 | F3 | B2 | C1 | C3 | D1 | D3 | F2 | A1 | A3 | C2 | B1 | B3 | E2 | C07 | C2 | B1 | B3 | A1 | A2 | A3 | B2 | C1 | C3 | D1 | D2 | D3 | F1 | F2 | F3 | E1 | E3 | E2 |
| A23 | A2 | E1 | E3 | E2 | F1 | F3 | C1 | C2 | C3 | D1 | D3 | F2 | A1 | A3 | B2 | B1 | B3 | D2 | C08 | C2 | B1 | B3 | A1 | A3 | B2 | A2 | C1 | C3 | D1 | D3 | F2 | E2 | F1 | F3 | E1 | E3 | D2 |
| A24 | A2 | E1 | E3 | E2 | F1 | F3 | C1 | C3 | D1 | D2 | D3 | F2 | A1 | A3 | B2 | B1 | B3 | C2 | C09 | C2 | B1 | B3 | A1 | A2 | A3 | C1 | C3 | D1 | D2 | D3 | E2 | F1 | F2 | F3 | E1 | E3 | B2 |
| A25 | A2 | E1 | E3 | D2 | F1 | F3 | C1 | C3 | D1 | D3 | E2 | F2 | A1 | A3 | C2 | B1 | B3 | B2 | C10 | C2 | B1 | B3 | A1 | A3 | B2 | C1 | C3 | D1 | D2 | D3 | F2 | E2 | F1 | F3 | E1 | E3 | A2 |
| A26 | A2 | F1 | F3 | D2 | E1 | E3 | C1 | C3 | D1 | D3 | E2 | B2 | B1 | B3 | C2 | A1 | A3 | F2 | C11 | C2 | C1 | C3 | B1 | B3 | E2 | A1 | A2 | A3 | D2 | F1 | F3 | B2 | E1 | E3 | D1 | D3 | F2 |
| A27 | A2 | F1 | F3 | D2 | E1 | E3 | B2 | C1 | C3 | D1 | D3 | F2 | B1 | B3 | C2 | A1 | A3 | E2 | C12 | C2 | C1 | C3 | B1 | B3 | F2 | A1 | A3 | B2 | D2 | F1 | F3 | A2 | E1 | E3 | D1 | D3 | E2 |
| A28 | A2 | F1 | F3 | E1 | E2 | E3 | C1 | C2 | C3 | D1 | D3 | F2 | B1 | B2 | B3 | A1 | A3 | D2 | C13 | C2 | C1 | C3 | B1 | B3 | E2 | A1 | A2 | A3 | F1 | F2 | F3 | B2 | E1 | E3 | D1 | D3 | D2 |
| A29 | A2 | F1 | F3 | E1 | E2 | E3 | C1 | C3 | D1 | D2 | D3 | F2 | B1 | B2 | B3 | A1 | A3 | C2 | C14 | C2 | C1 | C3 | B1 | B3 | F2 | A1 | A3 | D2 | E2 | F1 | F3 | A2 | E1 | E3 | D1 | D3 | B2 |
| A30 | A2 | F1 | F3 | D2 | E1 | E3 | C1 | C3 | D1 | D3 | E2 | F2 | B1 | B3 | C2 | A1 | A3 | B2 | C15 | C2 | C1 | C3 | B1 | B3 | E2 | A1 | A3 | D2 | F1 | F2 | F3 | B2 | E1 | E3 | D1 | D3 | A2 |
| B01 | B2 | A1 | A3 | B1 | B3 | C2 | A2 | C1 | C3 | D1 | D3 | E2 | D2 | E1 | E3 | F1 | F3 | F2 | C16 | C2 | D1 | D3 | B1 | B3 | E2 | A1 | A2 | A3 | D2 | F1 | F3 | B2 | E1 | E3 | C1 | C3 | F2 |
| B02 | B2 | A1 | A3 | A2 | B1 | B3 | C1 | C2 | C3 | D1 | D2 | D3 | E1 | E3 | F2 | F1 | F3 | E2 | C17 | C2 | D1 | D3 | B1 | B3 | F2 | A1 | A3 | B2 | D2 | F1 | F3 | A2 | E1 | E3 | C1 | C3 | E2 |
| B03 | B2 | A1 | A3 | A2 | B1 | B3 | C1 | C2 | C3 | D1 | D3 | E2 | E1 | E3 | F2 | F1 | F3 | D2 | C18 | C2 | D1 | D3 | B1 | B3 | E2 | A1 | A2 | A3 | F1 | F2 | F3 | B2 | E1 | E3 | C1 | C3 | D2 |
| B04 | B2 | A1 | A3 | A2 | B1 | B3 | C1 | C3 | D1 | D2 | D3 | E2 | E1 | E3 | F2 | F1 | F3 | C2 | C19 | C2 | D1 | D3 | B1 | B3 | F2 | A1 | A3 | D2 | E2 | F1 | F3 | A2 | E1 | E3 | C1 | C3 | B2 |
| B05 | B2 | A1 | A3 | B1 | B3 | C2 | C1 | C3 | D1 | D3 | E2 | F2 | D2 | E1 | E3 | F1 | F3 | A2 | C20 | C2 | D1 | D3 | B1 | B3 | E2 | A1 | A3 | D2 | F1 | F2 | F3 | B2 | E1 | E3 | C1 | C3 | A2 |
| B06 | B2 | B1 | B3 | A1 | A3 | C2 | A2 | C1 | C3 | D1 | D3 | E2 | D2 | F1 | F3 | E1 | E3 | F2 | C21 | C2 | E1 | E3 | E2 | F1 | F3 | A2 | C1 | C3 | D1 | D2 | D3 | A1 | A3 | B2 | B1 | B3 | F2 |
| B07 | B2 | B1 | B3 | A2 | C1 | C3 | A1 | A3 | C2 | D2 | F1 | F3 | D1 | D3 | F2 | E1 | E3 | E2 | C22 | C2 | E1 | E3 | F1 | F2 | F3 | B2 | C1 | C3 | D1 | D2 | D3 | A1 | A2 | A3 | B1 | B3 | E2 |
| B08 | B2 | B1 | B3 | A2 | C1 | C3 | A1 | A3 | C2 | E2 | F1 | F3 | D1 | D3 | F2 | E1 | E3 | D2 | C23 | C2 | E1 | E3 | E2 | F1 | F3 | A2 | C1 | C3 | D1 | D3 | F2 | A1 | A3 | B2 | B1 | B3 | D2 |
| B09 | B2 | B1 | B3 | A2 | C1 | C3 | A1 | A3 | D2 | E2 | F1 | F3 | D1 | D3 | F2 | E1 | E3 | C2 | C24 | C2 | E1 | E3 | F1 | F2 | F3 | C1 | C3 | D1 | D2 | D3 | E2 | A1 | A2 | A3 | B1 | B3 | B2 |
| B10 | B2 | B1 | B3 | A1 | A3 | C2 | C1 | C3 | D1 | D3 | E2 | F2 | D2 | F1 | F3 | E1 | E3 | A2 | C25 | C2 | E1 | E3 | E2 | F1 | F2 | C1 | C3 | D1 | D2 | D3 | F2 | A1 | A3 | B2 | B1 | B3 | A2 |
| B11 | B2 | C1 | C3 | B1 | B3 | D2 | A1 | A2 | A3 | E2 | F1 | F3 | C2 | E1 | E3 | D1 | D3 | F2 | C26 | C2 | F1 | F3 | E1 | E2 | E3 | A2 | C1 | C3 | D1 | D2 | D3 | B1 | B2 | B3 | A1 | A2 | F2 |
| B12 | B2 | C1 | C3 | C2 | E1 | E3 | A1 | A2 | A3 | F1 | F2 | F3 | B1 | B3 | D2 | D1 | D3 | E2 | C27 | C2 | F1 | F3 | E1 | E3 | F2 | B2 | C1 | C3 | D1 | D2 | D3 | A2 | B1 | B3 | A1 | A3 | E2 |
| B13 | B2 | C1 | C3 | B1 | B3 | F2 | A1 | A3 | C2 | E2 | F1 | F3 | A2 | E1 | E3 | D1 | D3 | D2 | C28 | C2 | F1 | F3 | E1 | E3 | F2 | B2 | C1 | C3 | D1 | D3 | E2 | A2 | B1 | B3 | A1 | A3 | D2 |
| B14 | B2 | C1 | C3 | B1 | B3 | F2 | A1 | A3 | D2 | E2 | F1 | F3 | A2 | E1 | E3 | D1 | D3 | C2 | C29 | C2 | F1 | F3 | E1 | E3 | F2 | C1 | C3 | D1 | D2 | D3 | E2 | A2 | B1 | B3 | A1 | A3 | B2 |
| B15 | B2 | C1 | C3 | B1 | B3 | D2 | A1 | A3 | E2 | F1 | F2 | F3 | C2 | E1 | E3 | D1 | D3 | A2 | C30 | C2 | F1 | F3 | E1 | E2 | E3 | C1 | C3 | D1 | D2 | D3 | F2 | B1 | B2 | B3 | A1 | A3 | A2 |

the "extreme" considered strength values, i.e., the values corresponding to the percentiles k05, k10, k90 and k95, that have been exhaustively combined.

The considered layouts intend to cover all the possible strength distributions, not facing the possible effects of constructive choices. In real buildings the effective strength distribution can be affected by many factors, like the construction timing, the exposure of each column or the specific conditions (weather, organization, etc.) occurring when the building was made. Since these factors would be very hard to assume, especially in a simulated design, they have been completely neglected in the study.

The last three groups of models (F, E, D, with the weakest column in 2f, 2e and 2d respectively) are mirrored comparing to the first three groups (A, B, C, with the weakest column in 2a, 2b and 2c respectively). A complete description of the strength distribution in the first 3 groups of layouts (90 models) is provided in Table 2, where the strength values, in terms of assumed percentiles, are reported in the columns, while the lines describe the positions, specified through the two interested frames, assumed by each column.

### *2.3 The eccentricity related to the strength variability*

It is well known that stiffness and strength eccentricities, in both elastic and inelastic systems, are not independent from each other (Peruš and Fajfar 2005, Tso and Myslimay 2003, Myslimaj and Tso 2005, Sommer and Bachman 2005, De Stefano and Pintucchi 2008). In the current analysis, due to the introduced in-plan strength irregularity, each considered plan layout presents a different amount of strength eccentricity. Furthermore, as concrete strength can be related to the concrete Young modulus (see EC2, Table 3.1), strength plan variability results in stiffness plan variability and, in turn, stiffness eccentricity. Both eccentricities have been found as the distance between the strength and stiffness centers to the mass one, assumed to coincide with the geometrical center of the plan. Both strength and stiffness centers have been evaluated in two alternative ways. The strength center has been expressed both in terms of the concrete strength, $c_{str\_fc}$, and ultimate shear of the columns as defined according to EC8 (see EC8-3 equation A.12), $c_{str\_Vu}$. The stiffness center has been expressed by considering the mechanical stiffness of columns, based on the Young modulus, $c_{stiff,\ Ec}$, and their shear-type stiffness, $c_{stiff,\ K}$, expressed through a simplified relationship (Anagnostopoulos *et al*. 2009) based on the yield moment and rotation, i.e.,

Table 3 Considered response parameters and symbols

| Response parameter | symbol |
|---|---|
| drift at the 1$^{st}$ storey | *drift_1st* |
| drift at the 4$^{th}$ storey | *drift_4th* |
| normalized displacement at the 1$^{st}$ storey | *ND_1st* |
| normalized displacement at the 4$^{th}$ storey | *ND_4th* |
| response strength eccentricity | $e_{resp,\ str}$ |
| response stiffness eccentricity | $e_{resp,\ siff}$ |
| *CoV* in the 1$^{st}$ storey drift | $CoV_{drift\_1st}$ |
| *CoV* in the 4$^{th}$ storey drift | $CoV_{drift\_4th}$ |
| *CoV* in the normalized 1$^{st}$ storey displacement | $CoV_{ND\_1st}$ |
| *CoV* in the normalized 4$^{th}$ storey displacement | $CoV_{ND\_4th}$ |

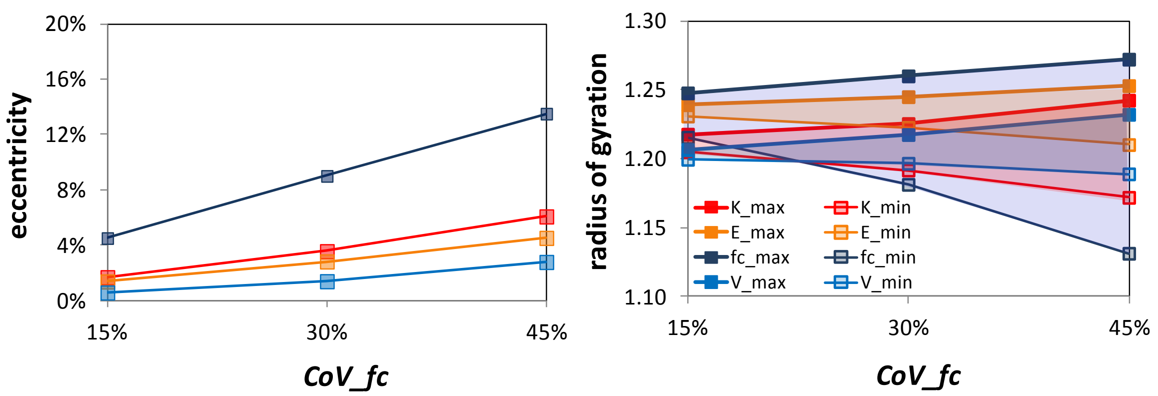
$K=(M_y\ H)/(6\ \theta_y)$, where $M_y$ is the column yield moment, $H$ is the column height and $\theta_y$ is the yield rotation. Since all columns have the same dimensions and reinforcement, the stiffness $K$ is affected by the assumed concrete strength only. $M_y$ and $\theta_y$ have been found by neglecting the section cracking.

Another quantity adopted to characterize strength plan layout is the mass-normalized radius of gyration of the floor slabs (Anagnostopoulos *et al.* 2013, De Stefano and Pintucchi 2010), which gives a measure of strength centrifugation. Since four different eccentricities have been considered, even four radii of gyration have been found, i.e., based on concrete strength ($\rho_{str,\ fc}$),ultimate shear ($\rho_{str,\ Vu}$), Young modulus ($\rho_{stiff,\ Ec}$), and simplified shear type behavior ($\rho_{stiff,\ K}$). Fig. 4(b) reports their values, nondimensionalized with respect to the mass radius.

For each considered strength *CoV* both the strength and the stiffness eccentricities have been found for each model. The maximum values of eccentricity have been shown in Fig. 4(a) as a function of the *CoV*. Since the last 90 considered layouts are mirrored with respect to the first 90, the consequent eccentricity varies symmetrically around the 0. In the diagrams shown in Fig. 4(a) only the positive values of eccentricity have been shown. A mirrored diagram could be obtained in the negative region of the ordinate. It should be noted that each group of considered layouts induces both positive and negative eccentricities.

As shown in Fig. 4(a), the two stiffness eccentricities are very close each other, while the two strength eccentricities differ remarkably. Except for $e_{str,\ fc}$, the introduced eccentricities are below 5%, that is the accidental eccentricity introduced by EC8 to take into account the possible unexpected plan-irregularities of the structure.

Values of non-dimensionalized radii of gyration, being larger than the unity, show that the building models can be classified as moderately torsionally stiff and strong.




*a. eccentricity* | *b. radius of gyration*

*strength eccentricity, estr, fc* | *radius of gyration based on strength, ρstr, fc*

*shear eccentricity, estr, V* | *radius of gyration based on shear, ρstr, V*

*Young modulus eccentricity, estiff, Ec* | *radius of gyration based on Young modulus, ρstiff, Ec*

*stiffness eccentricity, estiff, K* | *radius of gyration based on stiffness, ρstr, K*

Fig. 4 Eccentricity and radius of gyration for the considered *CoV* strength.

## 3. The analysis

### 3.1 The performed analyses

Both elastic modal response spectrum analysis and standard N2 nonlinear static one 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.

Modal response spectrum analysis has been performed to study how the introduced strength plan variability, which results in stiffness plan variability, affects the dynamic behavior of the structure.

Nonlinear static analysis has been performed to obtain information about the structural response and capacity; it should be underlined that 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, 2013, Magliulo *et al*. 2012, Shakeri *et al*. 2012, Bhatt and Bento 2014) to account for structural irregularities. In the current work, anyway, the standard N2 method, as provided by EC8, has been applied in order to discuss the obtained results according to the EC8 provisions.

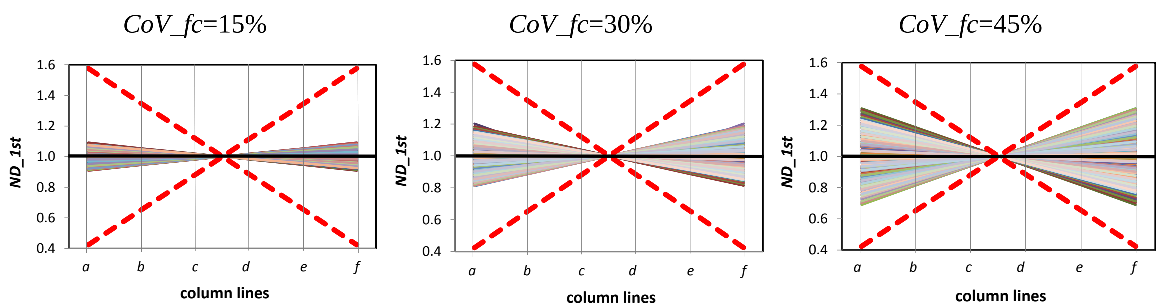



Fig. 5 Normalized displacement (*ND_1st*) at each column line of the 1st storey

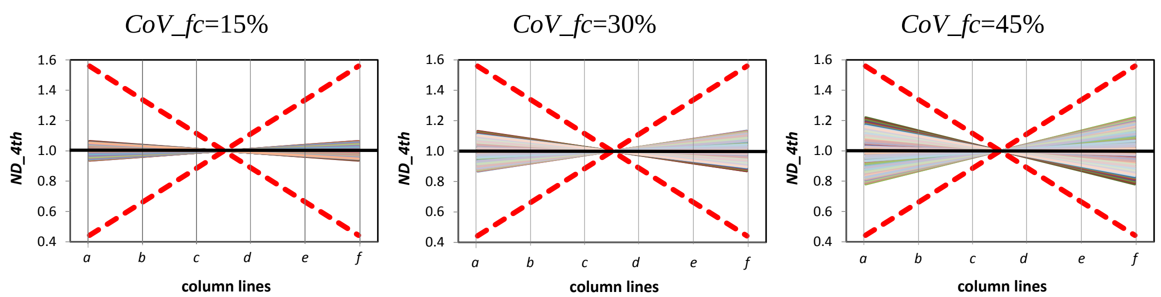



Fig. 6 Normalized Drift (ND) at each column line of the 4st storey

The seismic response of the structure has been found by considering different seismic intensities as torsional effects may vary as the structure experiences increasingly larger plasticity. Therefore, different values of PGA ranging between 0.05 g and 0.25 g, with a step equal to 0.05 g have been considered. The seismic input has been assumed to be represented by the elastic spectrum provided by EC8 (spectrum type 2, soil-type *B*, 5% viscous damping). In De Stefano *et al*. (2014, 2015) it has been found that the assumed vertical pattern of lateral forces does not affect significantly the results. In this work, therefore, only one force pattern, proportional to the first vibration mode, has been assumed.

### *3.2 Response parameters*

Special attention has been paid to measure the torsional effects and to relate such effects to the strength variability and the consequent eccentricity. The torsional response of the structure, being related to a source irregularity introduced at the first storey only, has been measured both at the first storey (*ND_1st*) and at the top level (*ND_4th*), in terms of normalized displacement, i.e., the value of the displacement at each column line divided by the displacement at the center of mass.

Since the eccentricity is one of the main key parameters of this work, it has been evaluated even on the structural response (Bosco *et al*. 2012). Therefore both the strength and the stiffness response eccentricities have been found, by considering the distribution of base shear and drift at the $1^{st}$ storey column. Since the response domains are related to the strength variability, the *CoV* of each response domain has been found and related to the concrete strength *CoV*. The assumed response parameters have been listed in Table 3.

## 4. Results

### *4.1 Modal response spectrum analysis*

#### *4.1.1 Normalized displacements*

Torsional effects from modal response spectrum analysis have been expressed, at each column-line, in terms of *ND*. In Figs. 5 and 6 the *ND* profiles along the structure *x*-axis at the $1^{st}$ storey and at the top storey are shown. In the same diagrams the normalized displacement profile obtained by introducing in the regular model (all columns having a $f_c$ strength equal to the *mean* value) the 5% mass eccentricity, as required by EC8 to cover all possible sources of eccentricity arising is a real buildings. As it should be noted, the torsional effects related to the 5% mass eccentricity, obtained by applying the horizontal forces to a distance equal to 5% of the building side to the hass center and represented by the red dashed line in the figure, largely cover those due to all the cases considered in the analysis. It should be observed, anyway, that the 5% eccentricity provided by EC8 is aimed to cover all the accidental irregularities and not the one related to the strength variability only.

#### *4.1.2 CoV of the response domains*

Fig. 7 shows the relationship between the strength and the *ND CoV*s in the performed analysis to check the propagation of strength variability to torsional response as measured by *ND_1st* and *ND_4th*. Two different values of *ND CoV* have been found, respectively obtained by considering all the columns of the storey and the columns belonging to the flexible column-line only. It is

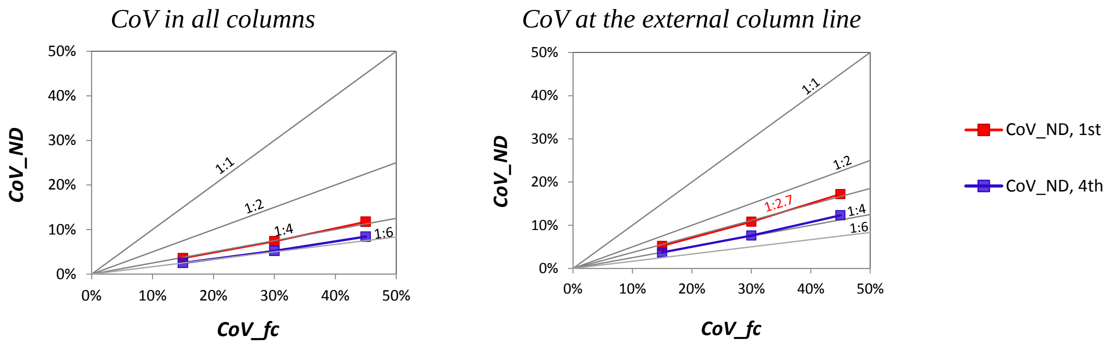



Fig. 7 Relationship between *CoV* in the strength and the consequent torsional effects

obvious, in fact, that the columns at the flexible side are the most affected by the torsional effects. The *CoV* in the response domains is much smaller than the one assumed for the correspondent strength variability. But, since the concrete strength has such a high variability, even the *CoV* of the consequent response domains is not negligible at all. The ratio between the *CoV* of concrete strength and the one of the elastic response parameter at the first storey is about 4 when all the columns are considered, while it is about 2.7 when evaluated on the side columns only.

### *4.2 Nonlinear analysis*

In this section the response parameter domains after a seismic intensity ranging between 0.05 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.

In the analysis only the global response parameters, which are drift (Section 4.2.1) and normalized displacement (Section 4.2.2) at the first storey have been checked, while the development of the inelastic deformation in the single cross section and members has not been studied. To better understand the correspondence between the eccentricity of each scheme and the consequent torsional effects, the eccentricity of the response has been found (Section 4.2.3) and related to the one of the capacity. Finally, to evaluate the relationship between the input and response variability, the correspondence between the concrete strength *CoV* and the *CoV* in each response domain has been checked (Section 4.2.4).

#### *4.2.1 Drift domains*

Figs. 8 and 9 show the response domains in terms of drift at the first storey of all three column lines (the other three would be mirrored) along the symmetric *x*-axis (column line 2) of the structure.

It has to be underlined that the main differences in terms of seismic response are along the *x*-axis, since the strength variability has been introduced along the *x*-direction only; due to the effect of axial loads in the columns, however, the response domain slightly varies even along the *y*-axis. Fig. 8 shows the density function of each response domain, while in Fig. 9 the cumulative probability functions are shown.

Diagrams of Figs. 8 and 9 show that the variability in the response domains increases with the seismic intensity in addition to the strength variability. In fact the wider distributions in the

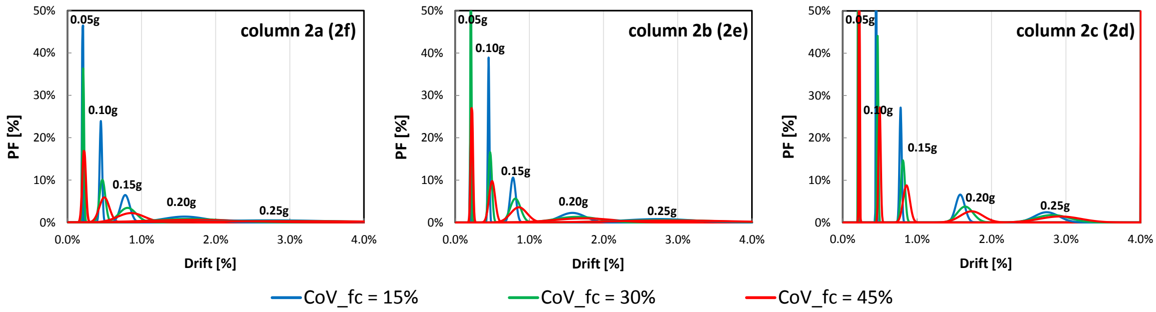



Fig. 8 Drift domains: Probability Functions of the drift at the 1st storey

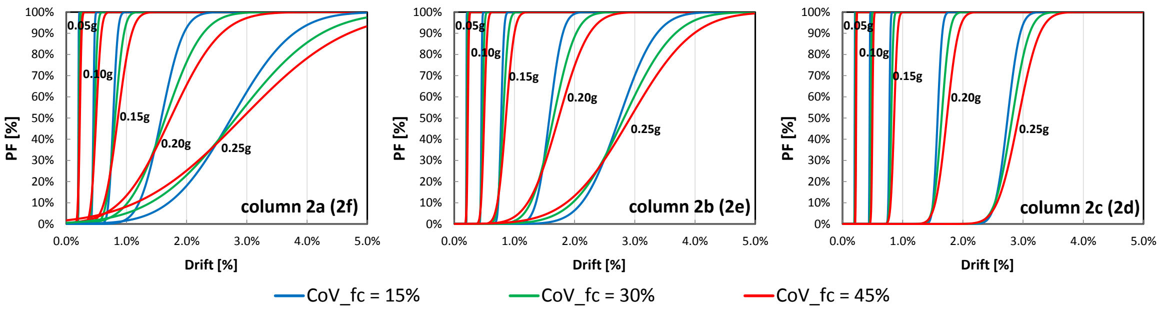



Fig. 9 Drift domains: Cumulative Probability of the drift at the 1st storey

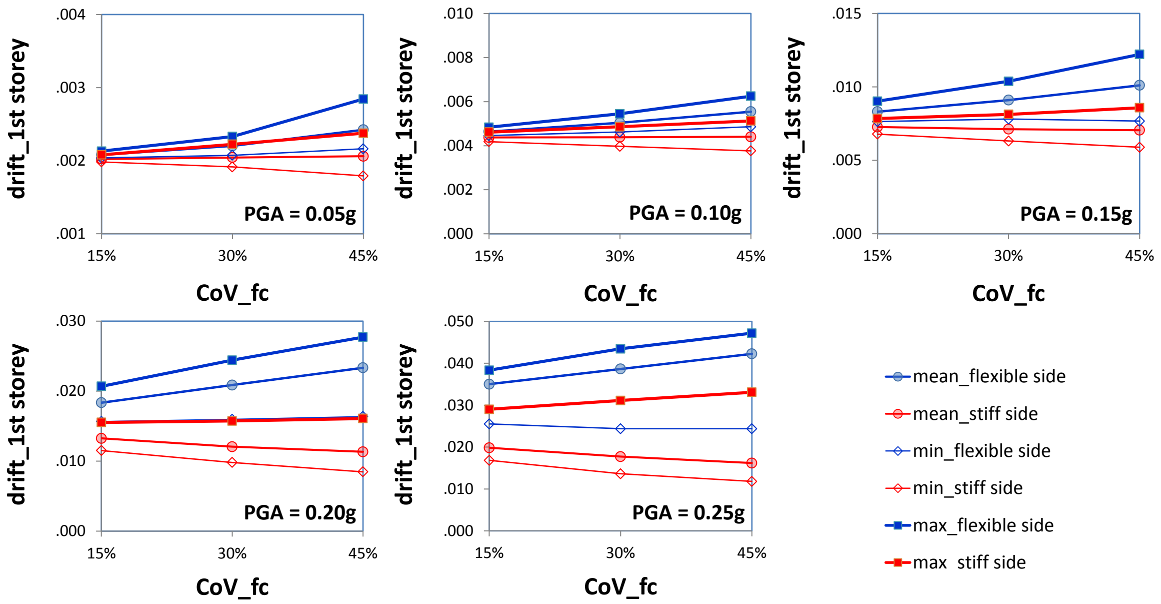



Fig. 10 Maximum drift at the stiff and soft side (90 models).

probability function of Fig. 8 and the lower slope in the cumulative probability diagrams of Fig. 9 reflect smaller variability. Moreover, as the strength *CoV* increases, the *mean* value of the drift slightly increases as well (≈10%), due to nonlinearity in the response.

As it is expected, the dispersion in the response domain is larger at the side column lines (*a* and *f* column lines), since, due to their position in plan, they are more affected by torsional effects.

Fig. 10 shows the relationship obtained, for the different PGA levels, between the *minimum*, *mean* and *maximum* drifts obtained at the stiff and flexible sides of the building. The results refer to 90 models only, since otherwise the response domains would be mirrored, and the response values would coincide. Diagrams in Fig. 10 show that the torsional effects are larger on the flexible side of the building than on the rigid one. This evidence was largely expected, since the building is torsionally stiff. It can be noted that the mean drift at the stiff side is not sensitive to the assumed *CoV* for low PGAs, i.e., for the elastic response of the case study, while the *mean* drift at the flexible side increases at the increasing of the assumed strength *CoV* for all considered PGAs. Maximum and minimum drifts at both sides evidence a regular -almost linear- increase at the increasing of *CoV* for all considered PGAs, with the exception of the maximum drift, whose trend varies for the different seismic intensities.

#### *4.2.2 Normalized displacement*

Figs. 11 and 12 show the normalized displacement, previously defined in Section 2.3, at the first and at the top storey respectively, found at each column line for the different values of PGA. For each considered *CoV* and PGA, the torsional effects have been compared to the ones(represented by the dashed red line) obtained by assuming the *mean* strength value in each column together with the introduction, at the first storey only, of a 5% mass eccentricity. By comparing the two figures, it can be observed that the most relevant torsional effects occur at the first storey of the structure. In fact the torsional effects at the top storey never exceed 40%, while at the first storey they exceed 60%.

Like it was observed for the elastic response, torsional effects increase both with the PGA and the concrete strength *CoV*. When the inelastic response is considered, however, the torsional effects related to the highest considered strength variability (*CoV* equal to 45%) slightly exceed the ones found by introducing the 5% eccentricity as provided by EC8.

It should be noted that the response of the case-study for high PGAs, which can be assumed to be mostly inelastic, results to be more sensitive to torsional effects than one found for low PGAs, where the behavior is mostly elastic. This evidence is not surprising, since strength eccentricity is larger than the stiffness one.

In Fig. 13 the correspondence between the four eccentricities introduced in Section 2.3 *ND_1st* is shown. As it can be seen, the relationship between each of the considered eccentricities and the torsional effects does not seem to be sensitive to the amount of strength variability; the trend of the obtained diagrams, in fact, is the same for each considered *CoV*.

The considered seismic intensity, instead, plays an important role in the relationship and the values of the normalized displacement increase, approaching a maximum value equal to 60%.

#### *4.2.3 Response eccentricity*

The response eccentricity has been evaluated both in terms of strength, $e_{resp,\,str}$, and of stiffness, $e_{resp,\,stiff}$. The response strength eccentricity has been found by considering the in-plan distribution of the base shear arising after the applied seismic action; the response stiffness eccentricity, instead, has been found as the in-plan distribution of lateral stiffness, i.e., the ratio between the shear and the drift at the first storey.

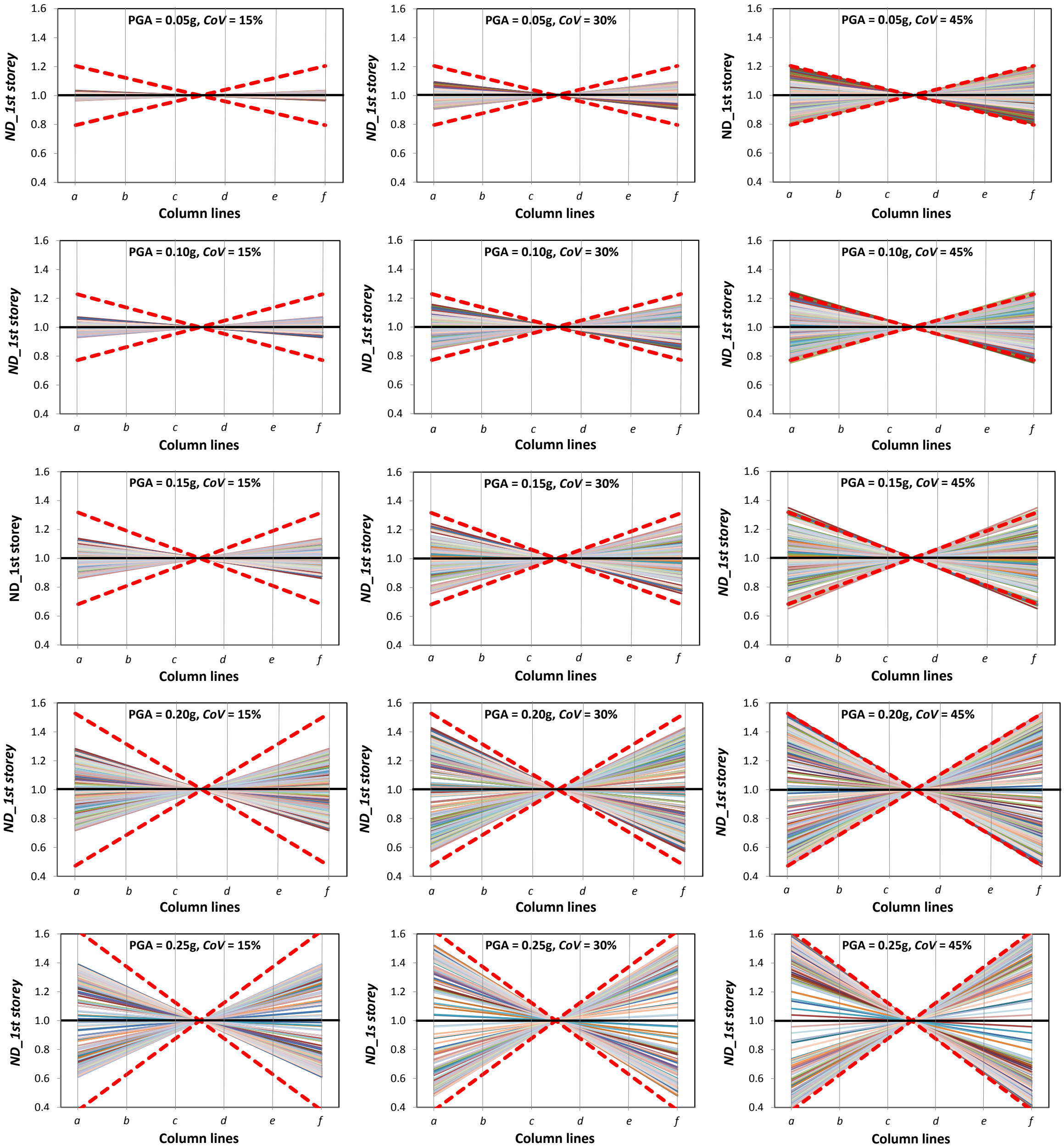



Fig. 11 Normalized displacement at the first storey

Fig. 14 shows the comparison between the response eccentricities and the initial ones ($e_{str,\,fc}$, $e_{str,V}$, $e_{stiff,\,Ec}$, $e_{stiff,\,K}$), as described in section 2.3. It should be noted that the response stiffness eccentricities for the three *CoV*s coincide with the initial ones when the system has an elastic behavior (PGA=0.05 g); conversely, as PGA and, in turn, inelasticity increases, the response strength eccentricities tend to the values found for the initial strength eccentricity. For PGA equal

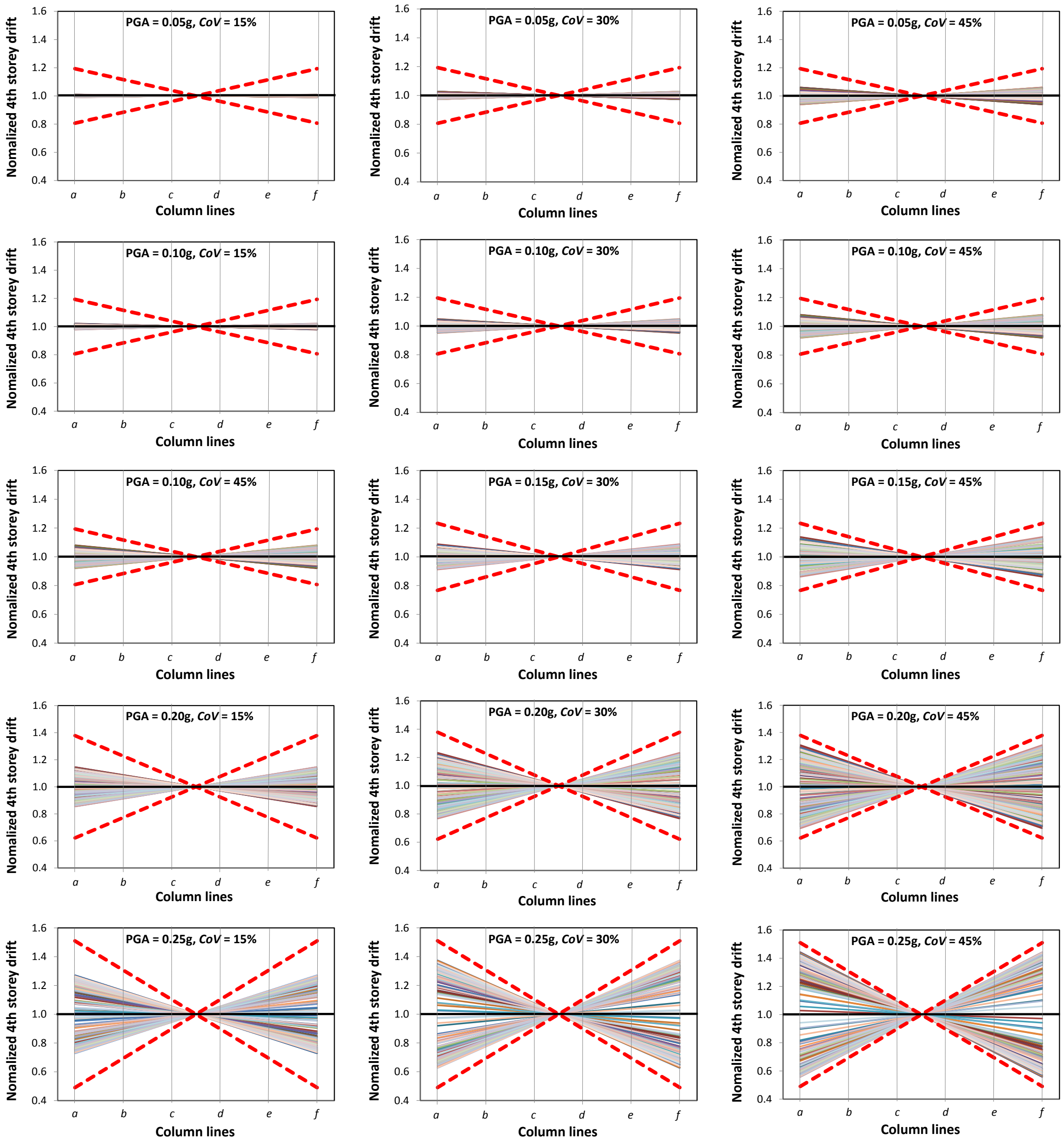



Fig. 12 Normalized displacement at the top storey

to 0.25 g, in fact, $e_{resp,\,str}$ is approaching the correspondent values of $e_{str,\,fc}$ for the three considered *CoV*s. It should be reminded that for PGA=0.25 g the structure has already overcome its ultimate capacity. Its response, anyway, cannot be assumed to be “perfectly” inelastic, since the columns at the different levels achieve their yielding point in different steps of the loading, and therefore the most involved members achieve their ultimate capacity when some columns are still in the elastic range.

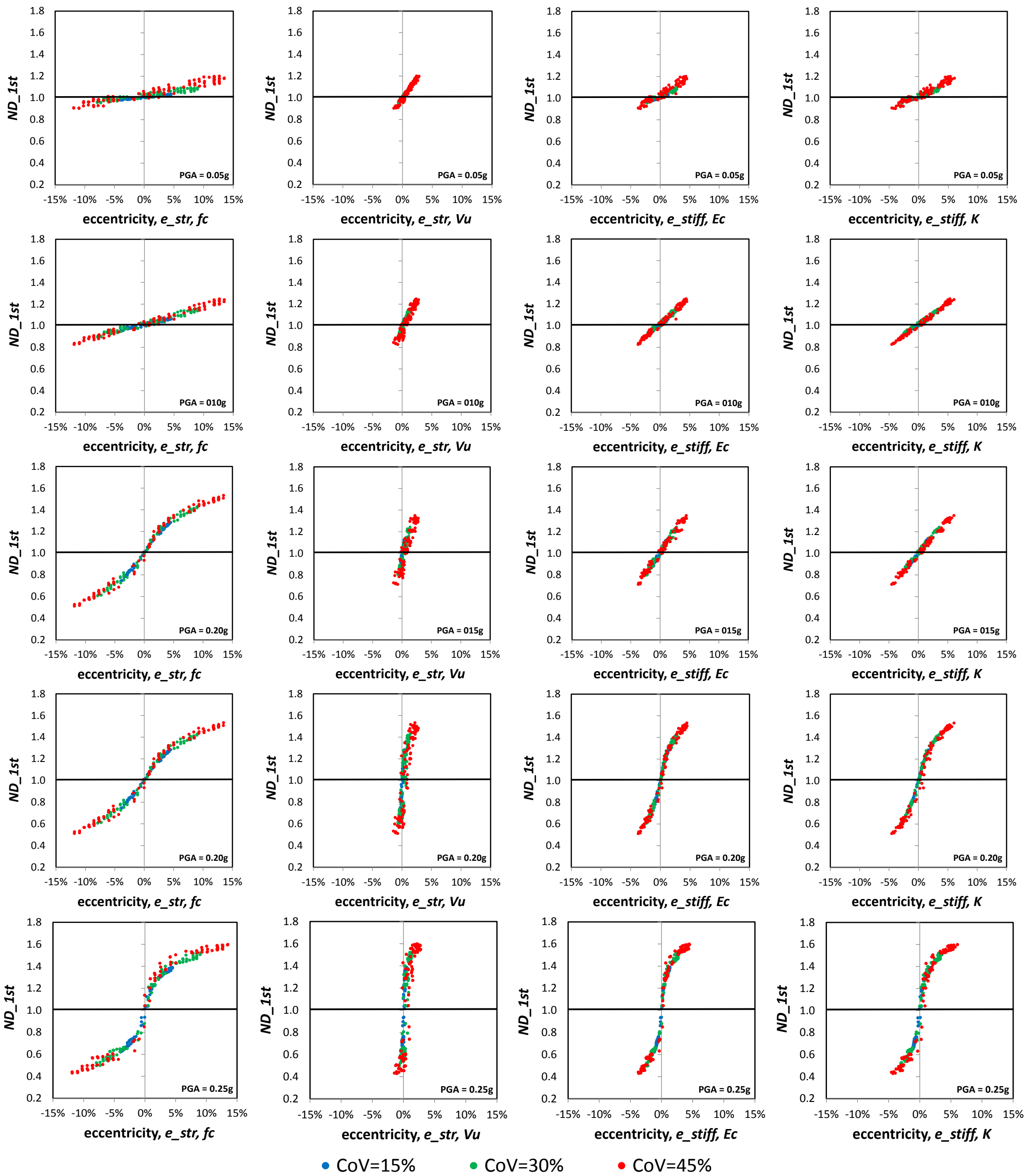



Fig. 13 Relationship between eccentricity and normalized displacement at the 1st storey (column-line 2a)

### *4.2.4 CoV of the response domains*

In Fig. 15 the relationship between the *CoV* of the concrete strength and of the normalized displacement at the first (Fig. 15 (a), (b)) and at the top (Fig. 15 (c), (d)) storeys is shown. The *ND*

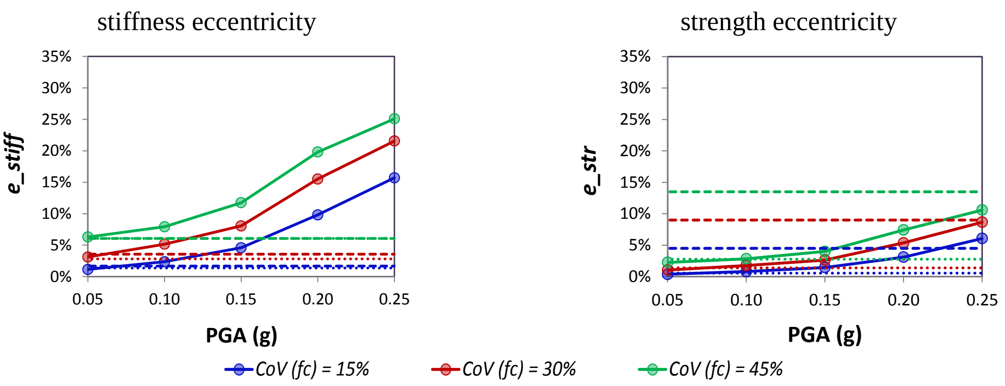
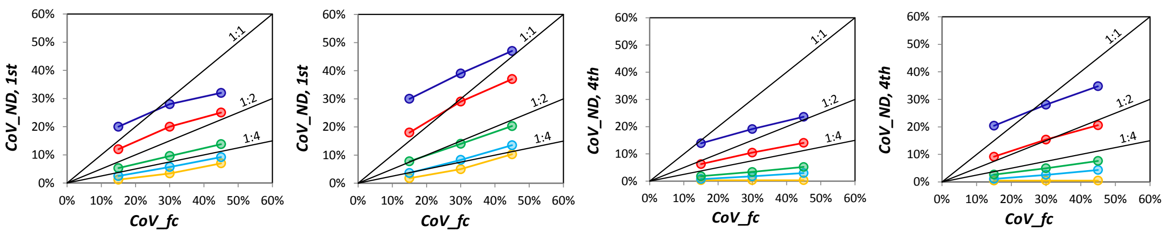



Fig. 14 Comparison between capacity and response eccentricity




(a) 1st storey, all columns (b) 1st storey, side columns (c) top storey, all columns (d) top storey, side columns

Fig. 15 Relationship between the *CoV* in concrete strength and in the normalized displacement

variability has been determined by considering all columns and the columns belonging to the flexible side column line.

As it was expected, the maximum *CoV* in the torsional response occurs at the side columns of the first storey. In this case the response *CoV* is larger than the input one for high seismic excitation (PGA=0.20 g, PGA=0.25 g). Even when all columns are considered, the output *CoV* remains significant, despite being lower than the input one.

The considered variability, therefore, proved to largely affect the inelastic response of the case-study under seismic excitation.

## 5. Conclusions

This paper is focused on concrete strength variability in existing buildings as a source of in-plan irregularity, inducing unexpected stiffness and strength eccentricity in the structure. The concrete strength has been described through a 7-sample domain, having three different amounts of variability (*CoV*=15%, 30%, 45%), consistent with the experimental results found for existing buildings in Tuscan (Italy). 180 in-plan layouts, comprehending all the most significant strength combinations, have been considered to represent the strength distribution at the columns of the first storey. The eccentricity arising from the represented strength variability has been found, both in terms of stiffness and strength, and compared to the one (eccentricity equal to 5%) provided by

EC8 to take into account of accidental irregularity. Both a modal and a nonlinear static analyses have been performed, to check the seismic response of the case-study.

From the modal response spectrum analysis, the maximum torsional effects, measured as normalized displacement, result in displacement increase of 30% at the flexible side of the first storey and of 20% at the top storey for the higher considered variability (*CoV*=45%). Both increments are largely covered by applying the 5% mass eccentricity rule subscribed by major seismic codes, such as EC8, to cover accidental eccentricity (due to all sources of irregularities, of course, and not to those related to concrete strength only).

The inelastic response of the case-study has been obtained by pushover analysis by considering, as seismic input, the response spectrum provided by EC8, with a PGA ranging between 0.05 g and 025 g. Different response domains, measured as 1$^{st}$ storey drift and normalized displacement, have been found for each seismic intensity and strength *CoV*.

Torsional effects found for high PGAs by pushover analysis are much larger than those coming from the modal response spectrum analysis. When the larger strength *CoV* is considered, in fact, the normalized displacement at the flexible side of the first storey reaches, and even exceeds, 50% for PGA=0.20 g and 60% for the highest PGA value (0.25 g), slightly exceeding the ones obtained by the 5% eccentricity rule provided by EC8. The inelastic response of the case study is more sensitive to the concrete strength variability than the elastic one, where strength variability influences response indirectly through the concrete Young modulus. In the considered models, being the columns at the extreme flexible side also the weakest ones, the attainment of the limit strength by a column during inelastic excursions results in the instantaneous loss of its contribution to the floor torsional stiffness, thus amplifying the torsional effects. Even the variability in the response domains found by the inelastic static analysis is larger than one found in the elastic analysis. The *CoV* found in the normalized displacement at the side first storey columns exceeds, for high PGA values, the one of the concrete strength.

The current EC8 approach does not specifically consider the possible high variability in concrete strength, despite it is very common to find such a situation in existing buildings, as it always assumes the *mean* value to be introduced in the analysis. Therefore, the introduction of the 5% accidental eccentricity provided by EC8 is essential to achieve a conservative evaluation of the seismic response of existing buildings. The eccentricity related to the considered strength variability is lower than 5% in almost all considered cases; therefore application of the 5% accidental eccentricity leads to larger amplifications of displacements at the flexible side than those resulting from the assumed plan strength layouts. It should be noticed, though, that the accidental eccentricity provided by EC8 is not specifically introduced to cover the strength variability, but it is supposed to cover all sources of accidental irregularity.

Further analyses, including buildings with different geometry and features, should be conducted in order to obtain more general results.

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