# Seismic Capacity of EC8 Compliant RC Frames with Irregular Vertical Distribution of Stiffness

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## 1 Introduction

The awareness that structural regularity deeply affects the seismic response of structures dates back to the beginning of the XX century. However, the first studies on the effects of the so called "set-backs" in structures appeared only in the second part of the past century. The main goal of those studies was to evaluate if the seismic response of irregular structures could be predicted accurately by lateral force method of analysis. Due to the limited computation capacity at that time, the structural models were quite simple and used mainly shear-type frames [1]. Such an approach is not coherent with the present design criteria that aim at avoiding soft story mechanisms and is ineffective for evaluating collapse mechanism and ductility demand of a structure. The first criteria for the evaluation of the vertical regularity of structures were developed at the end of the XX century and seemed to be based more on "common sense" rather than on rigorous analyses.

The knowledge acquired in the earlier studies reflected on both Uniform Building Code [2] and International Building Code [3], which provided the definition of

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vertical regularity/irregularity. Eurocode 8 [4] introduced similar criteria to judge whether a structure was regular in elevation or not. These criteria were also used to reduce the value of the behavior factor $q$ adopted in the structural design, even though this provision was not supported by specific scientific studies.

In the first two decades of the XXI century, scientific research devoted increasing efforts to the topic of vertical irregularity [5, 6]. These new studies mainly aimed at judging the effectiveness of nonlinear static procedures in evaluating the seismic response of irregular structures [7]. Due to the higher computational capacity, the adopted structural models became more realistic. Nonetheless, the vertical irregularity is obtained by varying stiffness and/or strength without a specific correlation between them. Furthermore, the structural performance is still evaluated in terms of interstory drifts, even though this response parameter becomes less significant in the case of global collapse mechanisms. Only few studies available in literature aims at analyzing the seismic performance instead of the effectiveness of the nonlinear procedures [8, 9], while the study by Athanassiadou and Bervanakis [10] shows that capacity design criteria are able to lead to a global collapse mechanism even in the case of irregular structures.

Most of the current seismic codes define a structure as regular in elevation based on the vertical distribution of mass, lateral stiffness and strength and introduce penalties for irregular structures. However, the parameters adopted to classify structures as regular or irregular vary according to the different seismic codes. This paper is focused on lateral stiffness distribution, which has been determined as the ratio of the story shear to the interstory drift.

Eurocode 8 classifies a structure as vertically regular when the lateral stiffness is constant in elevation or, at most, reduces gradually without abrupt reductions from the base to the top. Note that, even if the stiffness of a story is slightly larger than that of the story below the structure is considered as vertically irregular. The Italian seismic code NTC2018 [11] requires that lateral stiffness of regular structures be constant along the height or vary gradually, without abrupt changes. It quantifies the limits of the increase/reduction of lateral stiffness in elevation, specifying that lateral stiffness of the lower story must not be lower than 30% or larger than 10% of the stiffness of the upper story. In the case of irregular structures, both the European and Italian code prescribes 20% reduction of the behavior factor.

This research focuses on the prescriptions provided by seismic codes for the definition of vertically regular Vs. irregular structures. Indeed, criteria given by the European and the Italian codes lead to classify a large number of new buildings as irregular. This has been confirmed by an extended survey, conducted by one of the authors (Ghersi), that showed that the interviewed engineers classified 75% of their designed buildings as irregular in elevation, mainly due to the change of stiffness between two consecutive stories. This tendency may diminish the motivation and the efforts devoted by professional engineers in designing actual regular structures. This paper examines the seismic response of RC framed structures designed in compliance with code prescriptions, i.e. structures are designed according to the capacity design approach and structural members have good local ductility. The investigated structures were designed assuming the same value of the behavior factor $q$, but differed because of the level of variation of lateral stiffness along the height: vertically regular structures, wherein the value of lateral stiffness is constant or gradually changes in elevation; vertically irregular structures, wherein the value of lateral stiffness drastically varies and this change occurs in a few stories. The goal is to investigate (i) if the variation along the height of the lateral stiffness affects the seismic response of structures and (ii) whether the reduction of the behavior factor imposed by seismic codes for irregular structures can be considered appropriate or too conservative.

## 2 Description of Case Study Frames

The set of case study frames aimed to be representative of a wide range of real structures. To this end, 54 frames were designed to include the typical features of RC framed structures (presence of flat beam and deep beam, columns orientated along their strong axis or weak axis). Furthermore, 27 frames were 4-story high and the other 27 were 7-story high. Figure 1 shows the part of plan layout weighing on the designed frame and the vertical configuration of the 7-story frame. The 54 case study frames were all designed to be regular according to the NTC2018 and differ because of the following aspects:

- Beam span: three lengths of beam spans were considered (4.00 m, 5.00 m, 6.00 m) to have a different amount of bending moment due to gravity loads on beams.
- Ratio of the size of cross sections of columns and beams: the depth of cross sections of deep beams was assumed equal or reduced of 10–20 cm with respect to that of columns.

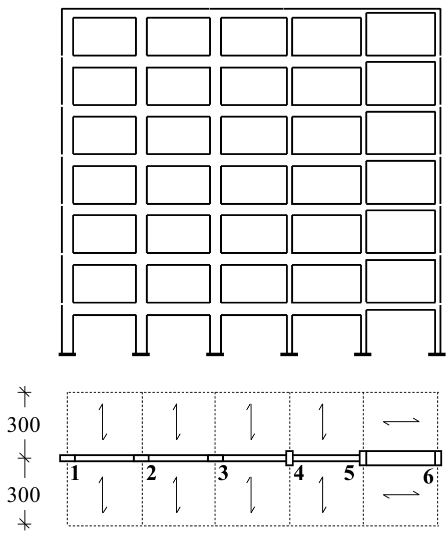


**Fig. 1** Vertical view and plan layout of the analysed frame

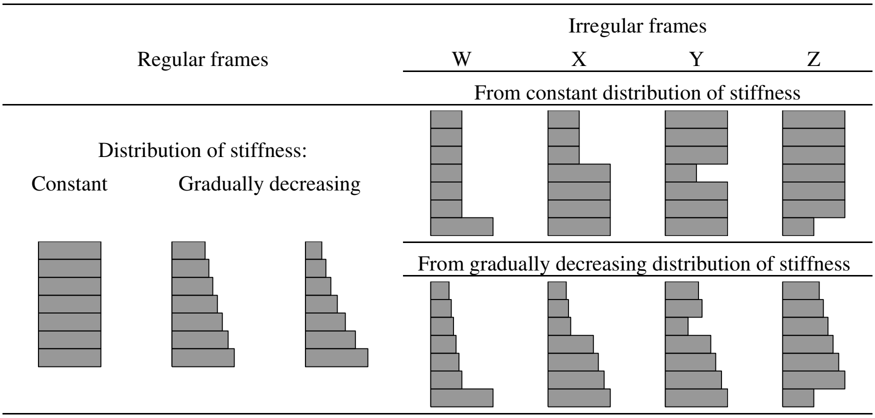


**Fig. 2** Distribution of lateral stiffness along the height of case study frames

– Stiffness of columns: three distributions of story lateral stiffness were considered, as shown in Fig. 2. First, the stiffness of columns was set constant along the height; second, it was assumed to reduce from the bottom to the top by 10% at each story; third, it was assumed to reduce from the bottom to the top by 20% at each story.

To design structures that could be classified as (rigorously) regular, the first inter-story height was increased compared to other stories. This approach permitted to mitigate the increase of stiffness of the first story due the rigid restraint at column base that simulates the foundation. The regular 4-story frames and 7-story frames have fundamental period of vibration equal to 0.50 s and 0.85 s, respectively.

The set of case study frames was extended to include also irregular structures. The irregular frames were derived from the regular ones by reducing/increasing the lateral stiffness. The elastic modulus of all members was slightly scaled in order to maintain the same fundamental period of vibration. The following cases of irregularity were considered:

W- Decrease of the lateral stiffness from the first to the second story, that considers the stiffening effect of foundation.

X- Decrease of the lateral stiffness from the bottom part of the frame to the top part (reduced stiffness from third story for 4-story frames and fifth story for 7-story frames). Indeed, the lower part of the structure is usually stiffer than the upper one due to the design practice that tends to reduce the cross sections of structural members at higher stories.

Y- Abrupt decrease of the lateral stiffness at a single intermediate story (second story for 4-story frames and fourth story for 7-story frames). This configuration is less usual but could be caused by a specific need related to the destination of use of the relevant story.

Z- Abrupt increase of stiffness from the first to the second story. This configuration simulates the lack of infill panels that characterises buildings with *pilotis*.

Figure 2 shows the considered distributions of stiffness for the irregular structures. The variation of stiffness $\Delta k$ between two consecutive stories was evaluated as follows:

$$\Delta k = \frac{k_{\text{inf}} - k_{\text{sup}}}{k_{\text{inf}}} \tag{1}$$

where $k_{inf}$ and $k_{sup}$ are the lateral stiffnesses of the lower and upper stories, respectively.

This parameter was assumed as the reference to evaluate the regularity/irregularity of structures and it was increased in step of 0.1. The value of stiffness was modified by changing both the dimensions (depth and width) of the cross sections of deep beams and columns. The rate of variation to be applied to the dimensions of the cross sections was determined according to a preliminary estimation of the stiffness. Then, the obtained stiffness was analytically evaluated by lateral force method of analysis and resulted to be close enough to the predicted one.

The set of regular frames included 27 frames with 4 stories and 27 frames with 7 stories. Each set of 27 frames generated 378 irregular frames, having up to 80% variation of stiffness between consecutive stories: 405 (27 + 378) 4-story frames and 405 7-story frames.

## 3 Research Methodology

To design the case study frames (regular or irregular), the internal forces are determined by a modal response spectrum analysis and for each of the 405 + 405 frames four values of behavior factor $q$ were considered:

- $q = 5.85$, that is the maximum value allowed by the Italian seismic code for high ductility structures (ductility class A);
- $q = 4.68$ ($5.85 \times 0.8$), that is the previous value of $q$ reduced by the coefficient $K_R = 0.8$;
- $q = 3.90$, that is the maximum value allowed for low ductility structures (ductility class B);
- $q = 3.12$ ($3.90 \times 0.8$), that is the previous value of $q$ reduced by the coefficient $K_R = 0.8$.

Hence, all the 405 frames with four stories and the 405 frames with 7 stories, having a stiffness distribution of type A, B, C and W, were designed four times assuming the four aforementioned values of $q$. All the case studies were supposed to be located in Messina (peak ground acceleration $a_g = 0.250$ g, $F_o = 2.410$, $T_C{}^* = 0.360$) on soil type C.

A member-by-member modelling with beam elements is adopted for beams and columns. The intersection between column and beam is modelled separately from the rest of the element and assumed to remain elastic. Hence, plastic hinge develops at the external face of the column. The same cross section and the same moment of inertia is assigned to the two parts of beam elements (internal and external to the node). The moment of inertia is assumed equal to its nominal value because the stiffness reduction caused by the cracking of the concrete may be assumed as included in the considered variation of stiffness.

The case study frames are designed using concrete C25/30 and steel B450C, following the capacity design principles of the Italian seismic code. Rebars are sized based on the results of the modal response spectrum analyses. The amount of rebars of beams is assumed exactly equal to that calculated to resist the bending moment at the external face of the column, or equal to the minimum value required by the code. The stirrups are determined to sustain a shear force determined by equilibrium conditions and assuming that at both the ends of the beam the bending moment is equal to the flexural plastic resistance, increased by the coefficient $\gamma_{Rd}$. The longitudinal rebars of columns are designed so that in each node the summation of the resisting bending moment of columns equals the summation of the resisting bending moment of beams increased by $\gamma_{Rd} = 1.3$. The minimum longitudinal reinforcement area is set equal to 0.25% on each side of the cross section. The stirrups of columns are determined based on the shear resistance of columns, following the modifications introduced by NTC2018. The reinforcement of the nodes is not specifically determined, but it is assumed sufficient to avoid shear failure. Each value of the behavior factor $q$ leads to a structure with a different lateral resistance.

The seismic response of each designed frame is evaluated by nonlinear static analysis. The numerical model uses beam elements with concentrated elastic perfectly plastic hinges, which can develop only outside the beam-column node. According to the prescriptions of seismic codes for the seismic assessment of existing structures, the resisting bending moment of columns and beams should be evaluated considering the average value of the material strength. For this research, it was considered more adequate to determine the resisting bending moment of columns and beams using the characteristic values of the material strength, i.e. cylinder strength $f_c = 25$ MPa for concrete and yielding strength $f_y = 450$ MPa for steel. In particular, the resisting bending moment of columns was determined assuming the value of axial force equal to that calculated considering gravity loads acting in seismic combination. The shear resistance of both columns and beams was evaluated considering the partial safety coefficients $\gamma_c$ e $\gamma_s$.

The nonlinear static analysis was performed assuming a distribution of lateral forces proportional to the first mode of vibration. The displacement demand of each step of the pushover analyses was associated with the corresponding value of $a_g$ according to the procedure suggested by the Italian seismic code. This analysis allowed the assessment of the seismic response of each structure subjected to seismic inputs with increasing magnitude, i.e. increasing values of $a_g$. A preliminary evaluation of the seismic behavior and collapse mechanism of the case study frames was conducted by observing the distribution of plastic hinges in structural members.

However, the seismic assessment of structures must be based on numerical evidences, i.e. the development of shear failure or the attainment of the ultimate value of plastic rotation. The nonlinear static analyses showed that none of the case study frame developed shear failures, which means that the capacity design approach was effective in avoiding brittle behavior of buildings. Hence, in the following, only the plastic rotation of members will be observed.

Both the Italian seismic code [11] and the European seismic code (EC8—part 3 [4]) determine the value of the chord rotation corresponding to the attainment of Near Collapse limit state $\theta_{u,NC}$ according to the equation provided by Panagiotakos e Fardis [12]. The value of the chord rotation at the attainment of the Significant Damage limit state (SD) $\theta_{u,SD}$ is derived from that of $\theta_{u,NC}$. The numerical model with concentrated plastic hinges allowed the determination of the plastic rotation demand $\varphi$. The limit value of the plastic rotation corresponding to the SD limit state $\varphi_{u,SD}$ was determined subtracting the elastic part of the rotation from $\theta_{u,SD}$:

$$\varphi_{u,SD} = \theta_{u,SD} - \frac{V\ L_V^2}{3\,E\,I} \tag{2}$$

where $V$ is the shear force corresponding to the attainment of the limit value of the chord rotation, $L_v$ is the shear length and $EI$ is the stiffness of the section. For each cross section where plastic hinge develops, the damage is estimated as the ratio $D$ of the plastic rotation to the plastic rotation corresponding to the DL limit state:

$$D = \frac{\varphi}{\varphi_{u,SD}} \tag{3}$$

The value of peak ground acceleration $a_{g(D=1)}$ corresponding to $D = 1$, i.e. the attainment of the SD, was evaluated for both columns and beams of each structure. In addition, the damage index $D$ corresponding to the reference value of $a_g = 0.25$ g, i.e. the peak ground acceleration corresponding to the SD limit state, was evaluated and the average value $D_{av,SD}$ of $D$ was calculated for columns and beams. These response parameters are used to assess the seismic response of each case study frame and to compare the seismic performances of regular and irregular structures.

## 4 Results of the Numerical Analysis

### *4.1 Case A: Stiffness Reduction from First to Second Story*

Figure 3a, b show, for increasing values of the rate of stiffness variation $\Delta k$, the average damage $D_{av,SD}$ corresponding to the SD limit state and the peak ground acceleration $a_g$ leading to the attainment of the SD limit state, respectively. The results refer to the frames with seven stories designed by $q = 5.85$. The damage is mainly concentrated in beams (black diamond), as required by the capacity design approach, and it is not significantly related to the stiffness variation. The continuous

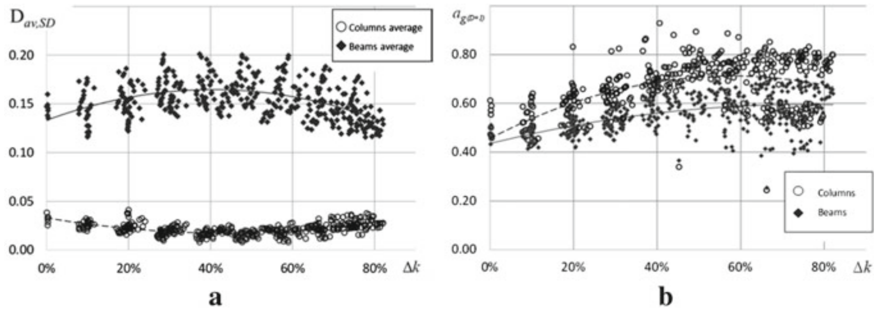


**Fig. 3** Case A, 7-story frames, $q = 5.85$: **a** Average damage at SD limit state, **b** PGA corresponding to the SD limit state

line and the dashed line represent the trend equation of the average damage for beams and columns, respectively. The trend line shows that for values of $\Delta k$ lower than 40%, the average damage of columns slightly decreases with $\Delta k$, while the average damage tends to increase if $\Delta k$ is larger than 40%. However, the damage attained for structures with strong irregularities (large values of $\Delta k$) does not differ significantly from that occurred in regular structures ($\Delta k = 0$). Figure 3b shows that almost all case study frames attains the SD limit state for peak ground accelerations that are larger than the value used to design the frames (0.25 g). The values of $a_g$ leading to D = 1 in columns (white circles in the Figure) are generally larger than those leading to D = 1 in beams. The values of $a_{g(D=1)}$ of beams are more scattered compared to columns. However, they increase for larger irregularity, with a slight reduction in case of $\Delta k$ larger than 60%. Even if not shown in figure, the results obtained for frames with 4 stories are analogous to those of frames with 7 stories.

The trend lines in Fig. 4 show for increasing values of irregularity the values of $a_{g(D=1)}$, determined as the minimum of the two values of peak ground acceleration corresponding to the attainment of SD limit state in columns and beams. Each line refers to a different value of $q$ and shows a seismic response almost independent of

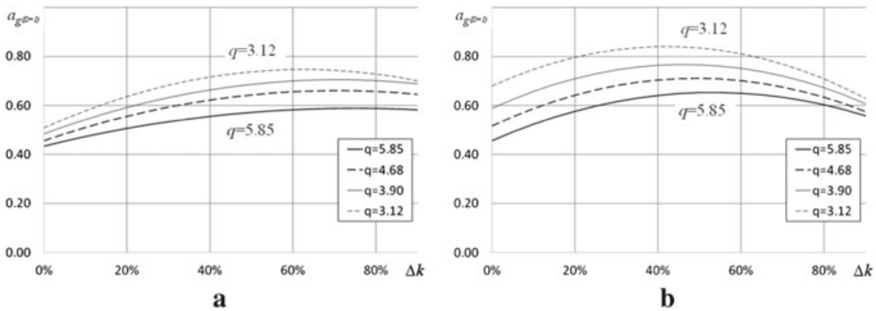


**Fig. 4** Values of $a_g$ corresponding to the SD limit states for increasing values of $\Delta k$ and different values of $q$: **a** Case A, 7 stories, **b** Case A, 4 stories

the value of $\Delta k$. Lower values of $q$ reduce the damage in beams and the structural collapse is attained for significantly larger peak ground accelerations.

It can be observed that large values of $\Delta k$ lead the structures to the SD limit state for lower values of peak ground accelerations. However, the value of $a_g$ at the SD limit state is still larger than, or at least comparable, to that of regular frames.

## 4.2 Case B: Stiffness Reduction from the Lower to the Upper Part of the Frame

The stiffness variation considered as Case B corresponds to structures where the cross sections of columns and beams is drastically reduced in the upper part of the building. Figure 5a shows the average damage experienced by structural members of 7-story frames designed by $q = 5.85$, at the SD limit state. In these cases, the value of $\Delta k$ quantifies the difference of stiffness between the fourth and fifth story. The results are close to those obtained for case A: the damage mainly occurs in beams, as result of the capacity design, and it is not significantly affected by the increase of irregularity. The value of $a_g$corresponding to the SD limit state of columns and beams is reported in Fig. 5b. The majority of the case study frames collapse for a peak ground acceleration larger than that used in design (0.25 g) and the capacity of structural members is not influenced by $\Delta k$.

The design of frames belonging to Case B was conducted assuming different values of $q$. Figure 6a, b show the trend lines of $a_g$ at the SD limit state for increasing values of $\Delta k$, for different values of $q$, for 7-story frames and 4-story frames, respectively. The trend lines show results similar to those of Case A, that is the value of the $a_{g,D=1}$ is not affected by the increase of $\Delta k$.

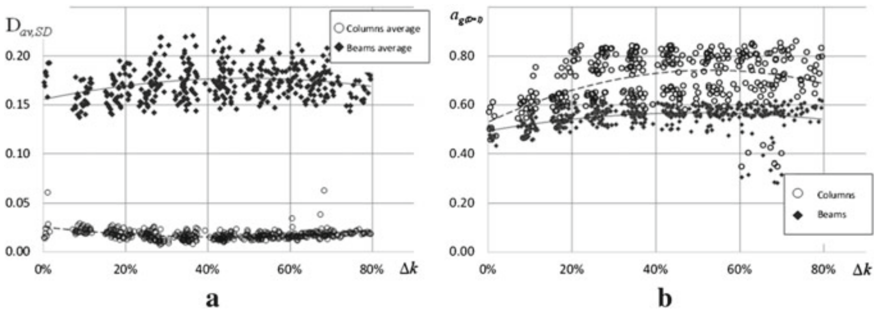


**Fig. 5** Case B, 7-story frames, $q = 5.85$: **a** Average damage at SD limit state, **b** PGA corresponding to the SD limit state

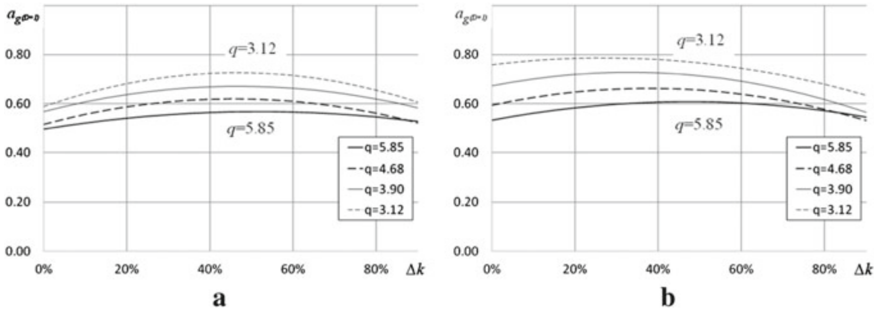


**Fig. 6** Values of $a_g$ corresponding to the SD limit states for increasing values of $\Delta k$ and different values of $q$: **a** Case B, 7 stories, **b** Case B, 4 stories

## 4.3 *Case C: Stiffness Reduction in an Intermediate Story*

The abrupt reduction of lateral stiffness at an intermediate story (Case C) could be representative of real buildings where one of the stories is dedicated to a different destination of use and requires an interstory height larger than in other stories.

The average damage cumulated at the SD limit state in columns and beams of 7-story frames designed by $q = 5.85$ is displayed in Fig. 7a. In this case, the parameter $\Delta k$ is determined as the difference of lateral stiffness between the third and fourth story, that is followed by an increase of stiffness between the fourth and the fifth story. The results are close to those obtained for Case A or B. In fact, the damage mainly occurs in beams. However, it can be noted that, in case of strong irregularity (large values $\Delta k$), the damage in columns has a non-negligible increase. Nevertheless, for all the case study frames, the peak ground acceleration corresponding to the SD limit state is larger than the reference value (0.25 g) (Fig. 7b). However, compared to Case A and B, the values of $a_{g,D=1}$ of columns are more scattered and show a more significant dependency from $\Delta k$.

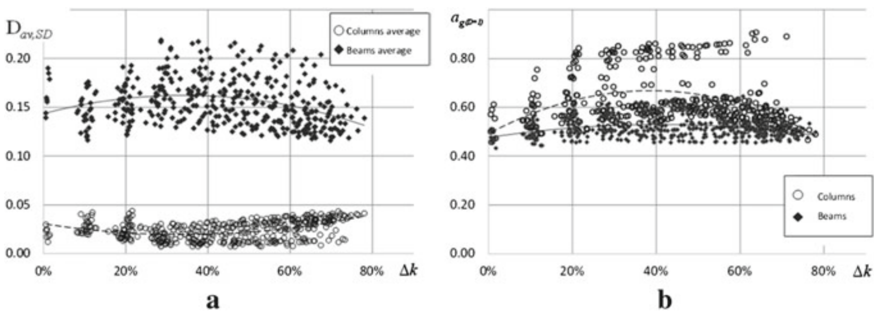


**Fig. 7** Case C, 7-story frames, $q = 5.85$: **a** Average damage at SD limit state, **b** PGA corresponding to the SD limit state

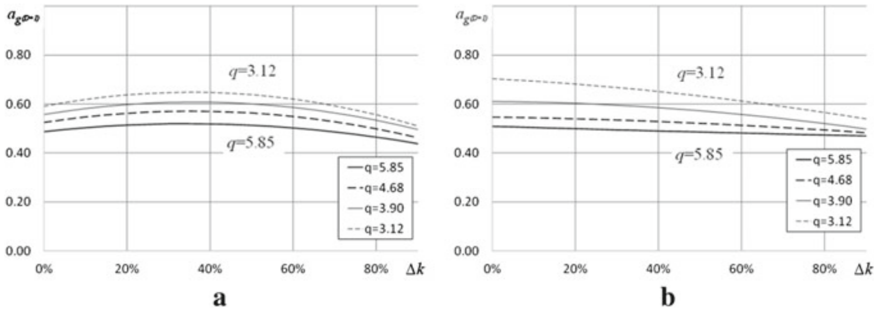


**Fig. 8** Values of $a_g$ corresponding to the SD limit states for increasing values of $\Delta k$ and different values of $q$, **a** Case C, 7 stories, **b** Case C, 4 stories

Although different values of $q$ were adopted for the design, the seismic capacity 7-story frames resulted to be almost independent from the entity of vertical irregularity, as shown in Fig. 8a. On the contrary, the seismic capacity of 4-story frames was affected by $\Delta k$. Indeed, the value of $a_g$ leading these frames to the SD limit state tends to decrease with $\Delta k$, as shown in Fig. 8b.

## 4.4 Case W: Stiffness Increase from the First to the Second Story

The increase of lateral stiffness from the first to the second story, named Case W, can be found in case of buildings having the first interstory height larger than other stories, or in case of the presence of a *pilotis.*

The average damage of structural members at SD limit state is showed in Fig. 9a for 7-story frames designed by $q = 5.85$. Note that, in this case, the parameter $\Delta k$ defined by Eq. 1 assumes negative values. However, the absolute value of $\Delta k$ is

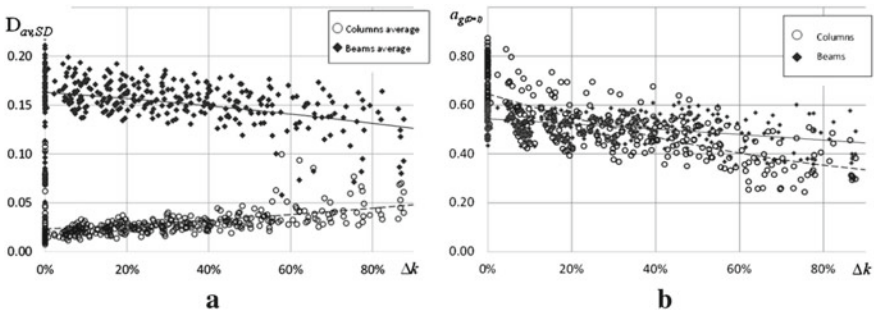


**Fig. 9** Case W, 7-story frames, $q = 5.85$: **a** Average damage at SD limit state, **b** PGA corresponding to the SD limit state

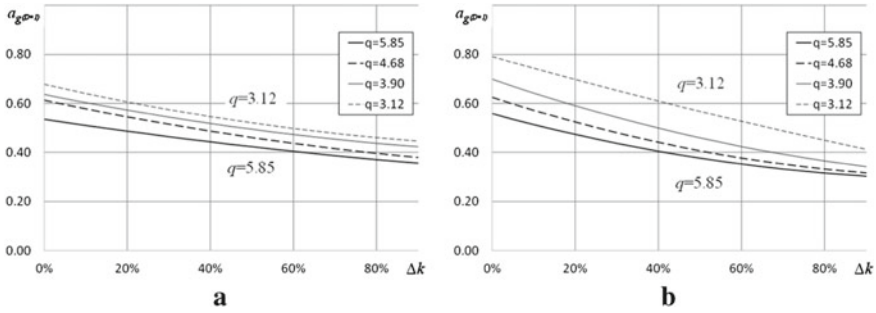


**Fig. 10** Values of $a_g$ corresponding to the SD limit states for increasing values of $\Delta k$ and different values of $q$: **a** Case W, 7 stories, **b** Case W, 4 stories

reported in Fig. 9. The results obtained for structures belonging to Case W are in line with the previous cases. In fact, damage is mainly concentrated in beams. However, differently from Case A, B or C, the damage in columns increases almost linearly with the parameter $\Delta k$, while that in beams tends to decrease. As a consequence (Fig. 9b), for larger values of $\Delta k$, the plastic rotation capacity is attained in columns and the peak ground acceleration corresponding to the SD limit state decreases almost linearly with increasing structural irregularity. This result is confirmed by the trend lines reported in Fig. 10 for both the 7-story and 4-story frames.

It is noteworthy that in this case the presence of vertical irregularity strongly affects the seismic capacity of structures. Hence, a proper reduction of the behavior factor $q$ is necessary to ensure that the seismic capacity of irregular structures is not penalized by the variation of stiffness. In fact, a value of the reductive coefficient $K_R = 0.8$, suggested by code for $\Delta k > 0.1$, appeared to be insufficient. Based on the obtained results, the value of $K_R$ should be determined as function of $\Delta k$:

$$K_R = 1 + 0.6\ \Delta k \tag{4}$$

where $\Delta k$ is determined by Eq. 1 considering its sign, and $K_R = 1$ for $|\Delta k| < 0.1$.

## 5 Conclusions

The seismic response of case study frames belonging to category A, B or C was basically independent of the variation of stiffness along the height, even in the case of significant values of stiffness reduction. These results show that the design criteria stipulated by seismic codes provide the structures with a good seismic capacity and guarantee a satisfying seismic performance even in the case of significant reduction of lateral stiffness. Hence, it is too restrictive to classify the structures with stiffness reduction as irregular, even though avoiding abrupt reduction of lateral stiffness along the height remains a valid design indication.

Differently, the increase of lateral stiffness along the height showed a significant impact on the seismic response of structures. Increasing values of the rate of variation of stiffness $\Delta k$ from the first to the second story led to a larger concentration of damage at first story and lower values of the peak ground acceleration $a_g$ corresponding to the attainment of the SD limit state. In particular, large values of the rate of stiffness variation almost nullify the effect of the parameter $K_R = 0.8$ that Italian and European seismic codes introduce to reduce the behavior factor $q$ in the case of irregular structures. Based on the obtained results, it seems reasonable that the reduction of the behavior factor should be evaluated as a function of the rate of stiffness $\Delta k$, as proposed by Eq. 4. The values of $q$ so reduced should provide irregular structures with a suitable seismic response. Note that, this study has to be intended as a preliminary investigation. Further investigations are of interest and studies with more realistic numerical models with distributed plasticity members and nonlinear dynamic analysis are in progress. The goal is to verify whether and the extent to which the results presented in this paper are confirmed or not.

## References

1. Valmundsson E, Nau JM (1997) Seismic response of building frames with vertical structural irregularities. J Struct Eng 123(1):30–41
2. ICBO (1997) Uniform building code, Whittier, California
3. ICC (2000) International building code
4. CEN Eurocode 8 (2004) Design of structures for earthquake resistance. Part 1: general rules, seismic actions and rules for buildings. EN 1998–1:2004. Brussels, Belgium
5. De Stefano M, Pintucchi B (2008) A review of research on seismic behaviour of irregular building structures since 2002. Bull Earthq Eng 6:285–308
6. Magliulo G, Ramasco R, Realfonzo R (2001) Sul comportamento sismico di telai piani in c.a. caratterizzati da irregolarità in elevazione. Proceedings of 10° Convegno Nazionale "L'ingegneria Sismica in Italia", Potenza-Matera, 9–13 September 2001
7. Chintanapakdee C, Chopra AK (2004) Seismic response of vertically irregular frames—response history and modal pushover analyses. J Struct Eng 130(8):1177–1185
8. Bhosale AS, Davis R, Sarkar P (2017) Vertical irregularity of buildings: regularity index versus seismic risk. ASCE-ASME J Risk Uncert Eng Syst Part A: Civil Eng 3(3)
9. Dya AFC, Oretaa AWC (2015) Seismic vulnerability assessment of soft story irregular buildings using pushover analysis. Procedia Engineering 125:925–932
10. Athanassiadou C, Bervanakis S (2005) Seismic behaviour of R/C buildings with setbacks designed to EC8. Proceedings of the 4th European workshop on the seismic behaviour of irregular and complex structures. CD ROM. Thessaloniki, August 2005
11. D.M. 17/1/2018, 2018. Aggiornamento delle "Norme Tecniche per le Costruzioni"
12. Panagiotakos TB, Fardis MN (2001) Deformations of reinforced concrete members at yielding and ultimate. ACI Struct J 98(2):135–148