Chapter 4
The Effect of Common Irregularities on the Seismic Performance of Existing RC Framed Buildings

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Abstract This chapter deals with the seismic performance of irregular 3D RC existing framed structures subjected to seismic actions. More specifically, the effect of the noncoincidence between the mass and the stiffness centers on the seismic response of these structures is investigated. The analysis is performed on a 4-story 3D framed sample structure designed for vertical loads only. A very detailed nonlinear model of the structure is implemented under the computer code SeismoStruct. The seismic response of the structure is analyzed performing a nonlinear incremental dynamic analysis. The obtained response domain is compared with the limit values provided by FEMA 356 for the different limit states. The effect of the introduced irregularities on the seismic performance of the structure is not negligible despite the low value of the eccentricity. The performed analysis evidences that a particular attention has to be paid to the seismic behavior of RC buildings realized in the 1960s and 1970s, before the adoption of seismic codes, since even light irregularities can consistently affect their seismic performance.

4.1 Introduction

A significant part of the Italian building heritage is constituted by reinforced concrete buildings designed according to inadequate rules and realized with concrete having poor mechanical properties during the intense constructive activity experienced by the country in the in the 1960s and 1970s, before the adoption in 1971 of the first code regarding reinforced concrete buildings (Legge 5/11/1971 n 1086) and

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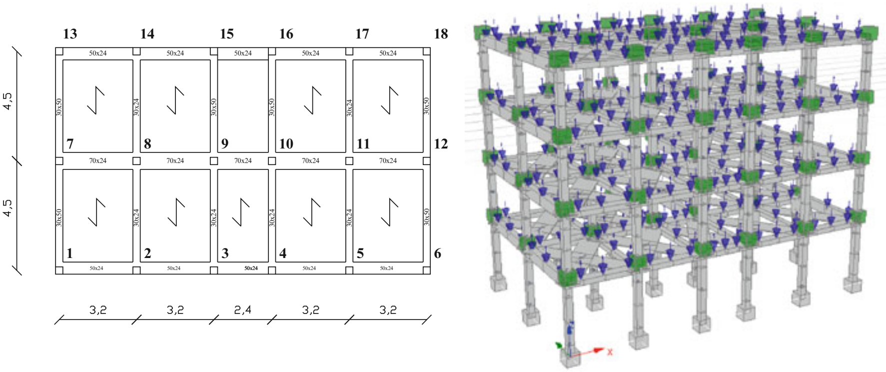

Fig. 4.1 Plan and 3D view of the considered sample structure

in 1974 of the first seismic code (Legge 2/2/1974 n. 64 1974). The evaluation of the seismic performance level of these buildings is therefore very important.

The present work deals with the evaluation of the safety of the RC buildings constructed in the 1960s and 1970s and tries to quantify the reduction in their seismic performance due to some of the typical loss of regularity.

One of the most common problems affecting such buildings is the noncoincidence between the mass and the stiffness centers and the consequent torsional effects. Usually the torsion is not considered in the design; it can be related to an asymmetry of the plan, to an irregular distribution of internal walls or balconies, or to a different use of a portion of the building. The torsional effects induced by these irregularity factors can be amplified by a nonhomogeneous distribution of the mechanical properties of the concrete very frequent in these kind of structures, poorly controlled during their construction phase.

The effect of all these irregularities on the seismic performance of existing RC framed buildings is evaluated herein with reference to a 4-story 3D framed sample structure designed for vertical loads only. The structure represents a typical existing RC building realized in Italy in the 1960s, before the introduction of seismic codes. The performed study focuses on the effect of the noncoincidence between the mass and the stiffness centers. The concrete mechanical properties and their variability are calibrated on the results of an extensive survey performed by the authors on a large sample of RC framed buildings realized before the adoption of seismic codes.

4.2 Sample Structure

The sample structure is a 4-story 3D reinforced concrete frame with two bays of 4.5 m in the *y* direction and 5 bays in the *x* direction, four of 3.2 m and one of 2.4 m, as shown in Fig. 4.1. The building is symmetric along the *x* and the *y* direction

Table 4.1 Mechanical properties of material

Materials	Class	Strength	Model
Concrete	C25/30	25/30 MPa	Mander
Steel	FeB38k	380 MPa	Bilinear

Table 4.2 Beam and column cross sections and reinforcement

Elements	*B* (cm)	*H* (cm)	Reinforcement
Beams – *x*	50	24	3Φ16 – 3Φ16
Beams – *x*	70	24	4Φ16 – 4Φ16
Beams – *y*	30	50	2Φ16 – 2Φ16
Beams – *y*	30	24	2Φ16 – 2Φ16
Columns	30	30	2Φ16 – 2Φ16

Table 4.3 Periods and modal masses of the sample structure

Period	Time (s)	% Participant mass (dir *x*)	% Participant mass (dir *y*)
1	0.927	0.0	88.1
2	0.891	83.4	0.0
3	0.806	4.7	0.0
4	0.303	0.0	8.9
5	0.289	8.8	0.0

and regular along its height. A C20/25 concrete and a FeB38k steel are assumed as materials, since they have mechanical characteristics (Table 4.1) compatible with those of the materials used in the 1960s.

The building is designed for vertical loads only, ignoring seismic loads. Vertical loads consist in dead loads and in live loads equal to 2 KN/m^2. The beam and column dimensions and reinforcement are summarized in Table 4.2.

The sample structure has been modeled with a fiber model using the computer code SeismoStruct (http://seismosoft.com) (Seismostruct v5.2.1 released 2011). The constitutive models used for the materials are that of Mander et al. (Mander et al. 1988) for the confined concrete, the three-linear model for the unconfined concrete, and the bilinear model for the reinforcing steel. Each structural element is modeled using four subelements, two external and two internal; the external subelements have a length equal to 1/15 of the total span. The contribution of the floors is computed introducing a rigid diaphragm at each floor bay. A preliminary modal analysis is performed to calculate the periods and the participation mass of the sample structure (Table 4.3).

4.3 Considered Irregularities

Some of the most common irregularities have been introduced in the sample structure to evaluate how much a not-computed torsional behavior can affect the seismic response of the building. The irregularities considered in the analysis are an asymmetric plan, an irregular distribution of balconies, a change of destination, and

Table 4.4 Values characterizing the assumed domain of f_c

Nominal value, f_{ck}	25.00 MPa
Mean value, f_{cm}	24.40 MPa
Reduced mean value, f'_{cm}	17.25 MPa
5% percentile value, $f_{ck,05}$	8.35 MPa

Table 4.5 Eccentricity of the "irregular" structure at each story

Level	Eccentricity (%)
0	1.90
1	4.90
2	5.09
3	5.09
4	0.31

a nonhomogeneity of the concrete mechanical characteristics. To obtain a building with an asymmetric plan, a further span having the same length of the others has been introduced along the *x* direction. An irregular distribution of balconies is obtained introducing a balcony on the right-hand side of the building. The changes of destination in old buildings are quite common. In the present case, it is assumed that part of the first story is changed from a residential destination to an office destination, with a consequent variation in the live loads.

In structures built in the 1960s, concrete can present mechanical properties significantly lower than the design properties due to the viscous phenomena, the degradation, but mainly to the low-quality level of the constructions realized in those years (Cristofaro 2009; D'Ambrisi et al. 2010). As a result, it is common to observe a strong nonhomogeneity in the distribution of concrete strength in the different parts of a building, with *coefficients of variation* up to 50% (Cristofaro 2009) and, consequently, an irregular distribution of the stiffness, different from that assumed in the analysis.

In the present work, a *coefficient of variation* of 40% has been assumed for the strength domain of concrete, while its mean value has been evaluated as the difference between the *mean value* and the *standard deviation* (Manfredi et al. 2007; American Society of Civil Engineering 2000). The nominal characteristic value of strength (f_{ck}), the average value including viscous effects (f_{cm}), the reduced value proposed by Manfredi et al. (f'_{cm}) (Manfredi et al. 2007), and the value corresponding to a 5% percentile of the distribution having f'_{cm} as mean value are reported in Table 4.4.

The effect of the irregularity in the distribution of the mechanical properties of concrete has been evaluated in a simplified way assuming that in the lower story the three columns belonging to the right column line have a strength equal to $f_{ck,\,05}$, while all the other columns have a strength equal to f'_{cm}.

All the above-described irregularities have been introduced in the sample building to obtain an irregular building whose seismic response can be compared with that of the sample structure. The eccentricities of each story of the irregular structure are summarized in Table 4.5.

4.4 Seismic Response

4.4.1 Incremental Dynamic Analysis

An incremental dynamic analysis has been performed on both the regular and the irregular structure with the computer code SeismoStruct (2011) considering as response parameters the top story displacement (*TD*) and the interstory drift (*ID*). The analysis has been performed considering a set of ground motions scaled to represent different seismic intensities, equal to 0.10, 0.15, 0.20, 0.25, 0.30, 0.35, and 0.40 g, respectively. For each value of PGA, the response of the structures has been defined elaborating the response domains from the assumed ensemble of ground motions.

4.4.2 The Seismic Input

The considered seismic input is constituted by an ensemble of seven ground motions extracted by a database of strong motions recorded in Italy (Itaca 2008) considering a PGA equal to 0.25 g, a nominal life of the structure of 50 years, and a magnitude ranging between 5.5 and 6.5. The data relative to the utilized ground motions are reported in Table 4.6. The selected ground motions have an average spectrum that approaches very well the one provided by Eurocode 8 (2005) for a soil of type B, as it is evident from Fig. 4.2.

4.4.3 Assumed Limit States

The maximum *ID* obtained for both the regular and the irregular sample structure has been compared with the standard values provided by FEMA 356 (American Society of Civil Engineering 2000) for the limit states *immediate occupancy* (*IO*), *life safety* (*LS*), and *collapse prevention* (*CP*) reported in Table 4.7.

Table 4.6 Ground motions data

Name	Location	Date (gg/mm/yyyy)	PGA (g)	Magnitude (L-S-W)	Duration (s)
Irpinia	Sturno	23/11/1980	0.225	6.5 – 6.8 – 6.9	70.75
Irpinia	Calitri	23/11/1980	0.174	6.5 – 6.8 – 6.9	85.99
L'Aquila	Colle Grilli	06/04/2009	0.446	5.8 –··· – 6.3	100.00
L'Aquila	Aquil Park Ing	06/04/2009	0.353	5.8 –··· – 6.3	100.00
L'Aquila	Aquil Park Ing	06/04/2009	0.330	5.8 –··· – 6.3	100.00
L'Aquila	Centro Valle	06/04/2009	0.545	5.8 –··· – 6.3	100.00
L'Aquila	Centro Valle	06/04/2009	0.657	5.8 –··· – 6.3	100.00

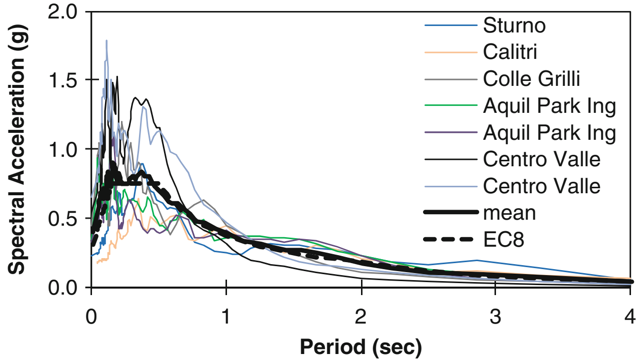

Fig. 4.2 Elastic response spectra of the considered ground motions

Table 4.7 Considered limit states

Limit states	Drifts (%)
Immediate occupancy	1
Life safety	2
Collapse prevention	4

4.5 Obtained Results

Figure 4.3 shows the results in terms of top story displacements (*TD*) obtained from the performed analyses for both the regular and the irregular structure. Each response domain refers to the values of the maximum *TD* obtained from the dynamic analysis in each column line, perpendicularly to the considered seismic excitation. Indeed, due to the introduced eccentricities, the response of the irregular structure varies along the longitudinal axis of the building.

As it can be observed from Fig. 4.3, the *TD* of the irregular structure at the column line 2, that is, the flexible side of the structure, is consistently larger than the corresponding value of the regular structure.

To better evidence the difference in the seismic response of the two considered structures, the values of the *TD* at each column line have been normalized with respect to the *TD* of the center of mass of the building. The obtained normalized displacements are shown in Fig. 4.4. As it can be observed from the figure, both at the stiff and at the flexible side of the structure, there is a difference of about 10% in the *TD* as a consequence of the introduced irregularities.

The increase in the *TD* due to the considered eccentricity is larger for low and medium values of PGA, for which the structure evidences an elastic response. With the increase of the PGA, indeed, the inelastic involvement of the structure increases and the sensitivity of its response to the torsional effects in terms of *TD* decreases (De Stefano and Pintucchi 2010).

The maximum values of the interstory drift obtained at each story are compared with the limit values provided by FEMA (Fig. 4.5) (American Society of Civil

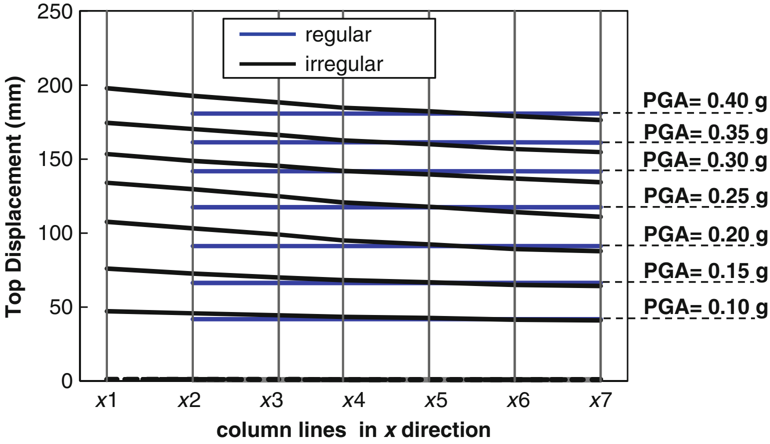

Fig. 4.3 Top story displacement at each column line

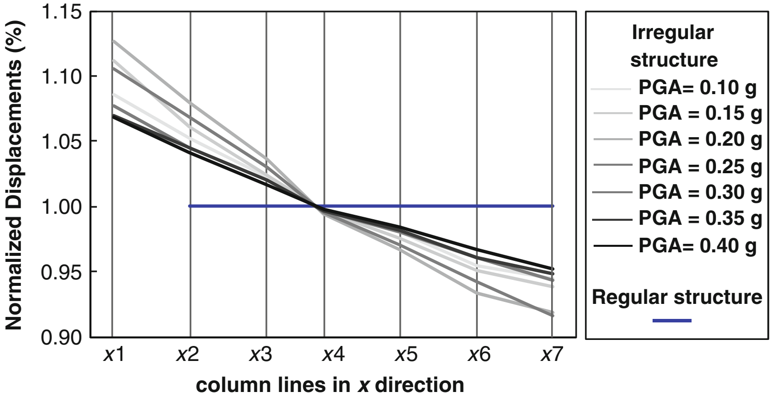

Fig. 4.4 Normalized top story displacement at each column line

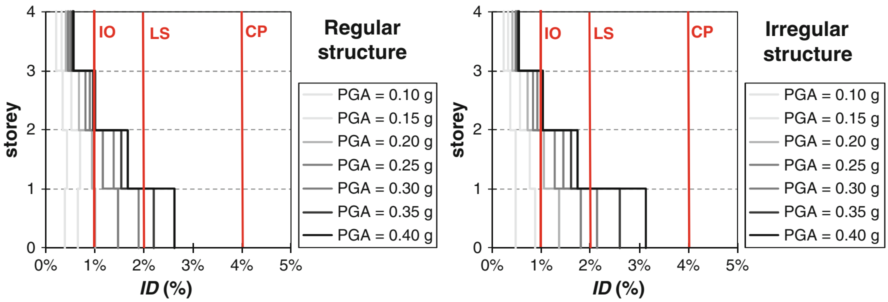

Fig. 4.5 Maximum interstory drift (*ID*)

Table 4.8 Statistical indexes of the response domains in terms of interstory drifts

	Regular structure			Irregular structure		
	Mean (%)	*Standard deviation* (%)	*Cov* (%)	*Mean* (%)	*Standard deviation* (%)	*Cov* (%)
PGA = 0.10 g	0.44	0.08	17.94	0.49	0.10	21.28
PGA = 0.15 g	0.66	0.14	20.8	0.89	0.24	26.9
PGA = 0.20 g	1.01	0.26	25.73	1.36	0.42	31.29
PGA = 0.25 g	1.47	0.44	29.7	1.80	0.63	35.2
PGA = 0.30 g	1.89	0.60	31.99	2.13	0.82	38.70
PGA = 0.35 g	2.21	0.75	33.7	2.60	0.98	37.7
PGA = 0.40 g	2.62	0.89	34.04	3.12	1.11	35.64

Table 4.9 Exceedance probability of the considered limit states

	Regular structure			Irregular structure		
	IO (%)	LS (%)	CP (%)	IO (%)	LS (%)	CP (%)
PGA = 0.10 g	0.0	0.0	0.0	0.0	0.0	0.0
PGA = 0.15 g	0.8	0.0	0.0	32.3	0.0	0.0
PGA = 0.20 g	51.5	0.0	0.0	80.4	6.4	0.0
PGA = 0.25 g	85.7	11.4	0.0	89.8	37.5	0.0
PGA = 0.30 g	93.1	42.7	0.0	91.6	56.3	1.1
PGA = 0.35 g	94.7	61.0	0.9	94.9	73.0	7.7
PGA = 0.40 g	96.6	75.7	6.1	97.2	84.4	21.4

Engineering 2000). The influence of the introduced irregularity on the seismic response of the considered structure can be observed also in terms of *ID*. The distribution of the *ID* is similar for the two structures; in both cases, the maximum *ID* occurs at the first story, which evidences an exceedance of the *LS* limit state. The difference between the two obtained response domains, in terms of *ID*, is evaluated comparing the exceedance probability of the considered limit states of the two structures. The exceedance probability of the limit values has been determined assuming a normal distribution for the two response domains; the two distributions are characterized by the statistical parameters reported in Table 4.8. It can be observed that the irregular structure has larger *mean* values but also a greater *coefficient of variation*, with a consequent larger probability to exceed the limit values. The exceedance probabilities of the considered limit states obtained for the two structures for the different PGA are reported in Table 4.9. As it can be noticed from the table, the probability to exceed the *collapse prevention* limit state is almost the same for the two structures in the considered range of PGA. As regards the other limit states, instead, the probability to exceed the limit values increases due to the eccentricity. The maximum increase in the exceedance probability of the *immediate occupancy* limit state occurs for a PGA = 0.15 g (exceedance probability from 0.8 to 32%), while for the *life safety* limit state, it occurs for a PGA equal to 0.25 g (exceedance probability from 11 to 37%).

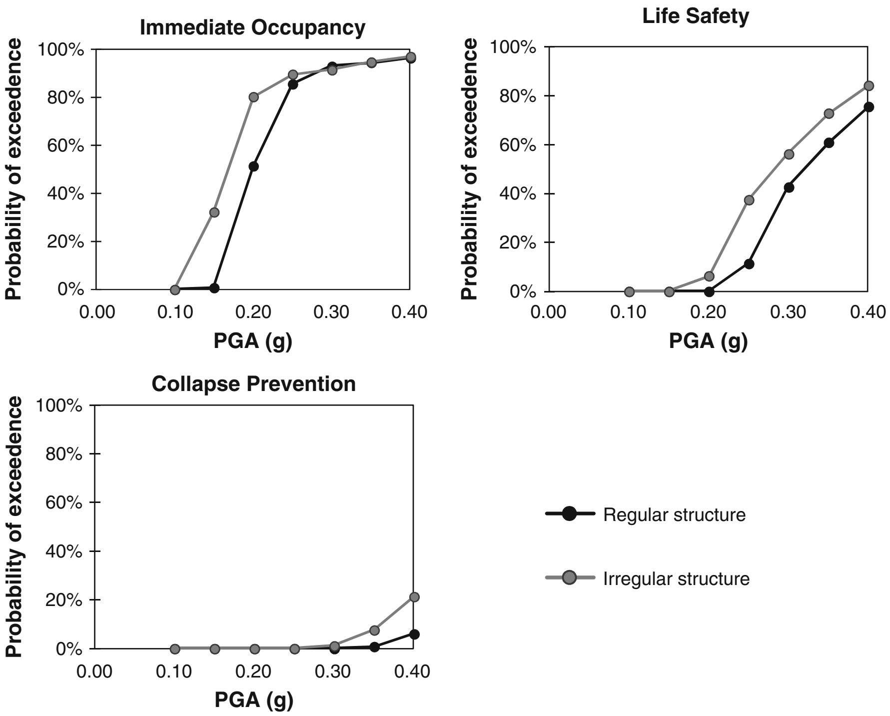

Fig. 4.6 Linearized fragility curves

The high sensitivity of the structure to the introduced eccentricity can be related to the low quality of the concrete and, therefore, to the reduced strength and stiffness of the structure.

In Fig. 4.6, the probabilities of the structures to exceed the considered limit states are plotted versus the PGA. The curves represent, in a linearized form, the fragility curves of the two structures with regard to the considered limit states. Each point of the curves represents the probability (P) of the response parameters (r) of the structure to exceed the limit value (lv) corresponding to the assumed performance level under a seismic input of given intensity (PGA) (Barron Corvera 2000; Reinhorn and Barron 1999), according to the following expression:

$$\text{Exceedance probability curve} = p\left[r > \frac{\text{lv}}{\text{PGA}}\right] \tag{4.1}$$

It can be noticed that the introduced irregularities induce a significant increase of the exceedance probability of the considered limit states. This increase depends on the range of considered PGA. In the range of interest for the considered buildings, that is, medium-low seismicity with a PGA ranging between 0.10 g and 0.20 g, the fragility curves diverge for the *IO* limit state only. For PGA larger than 0.20 g,

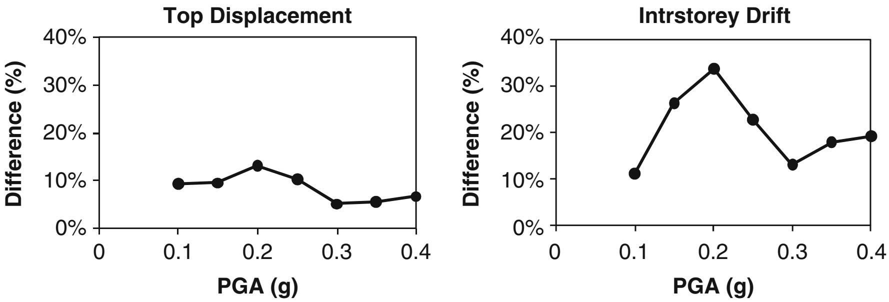

Fig. 4.7 Percentage difference due to the introduced irregularities

the fragility curves relative to the *LS* limit state show significant differences. The fragility curves related to *CP* limit state start to diverge only for PGA values larger than 0.30 g, that is, a seismic hazard not compatible with the considered areas.

The comparison between the seismic responses of the two examined structures is reported in Fig. 4.7 both in terms of *TD* and *ID*. The curves do not have a very regular trend due to the complexity of the nonlinear seismic response of the structures.

Both Figs. 4.6 and 4.7 evidence a significant effect of the introduced irregularities on the considered response parameters. As already mentioned above, such effect decreases for high values of the PGA. The reduction of torsional effects with the increase of the PGA is related to the inelastic involvement of the structure. In the case of strong earthquakes, the seismic response of the structure depends on the ductility available at local level; therefore it is important to check both the global and the local response parameters of the performed analysis.

To check the local ductility demand of the two sample structures, the values of the rotation at the chord, ϑ, at each beam end have been detected. Such values have been compared with the ultimate value of ϑ_u provided by Eurocode 8 (2005) and other national codes (NTC 2008). The exceedance of ϑ_u is suggested by EC8 as an acceptance conditions; therefore, if such limit is exceeded even in only one element, the structure does not satisfy the safety requirement. To check the inelastic involvement of the structures, a damage index is introduced, measuring the percentage of elements in which ϑ_u is exceeded. Damage index versus the PGA curves of the regular and irregular structures is reported in Fig. 4.8. The regular structure remains under the limit value of rotation in all cases, while in the irregular structure, a number of elements, increasing with the increase of the PGA (PGA larger than 0.20 g), exceed the limit. From these results, it is possible to assume that the irregular structure for high values of PGA experiences a large inelastic involvement, evidencing a distribution of damage that exceeds the acceptance level defined by the EC8.

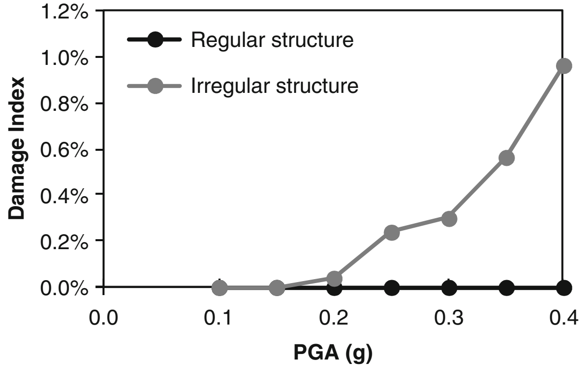

Fig. 4.8 Percentage of elements exceeding ϑ_u

4.6 Conclusions

In this chapter, the seismic performance of a typical RC framed structure built in Italy in the 1960s, designed for horizontal loads only, has been studied. The effect of common irregularities such as asymmetric plan, irregular distribution of balconies, different live loads, and nonhomogeneous mechanical properties of the concrete has been investigated, by introducing at each storey the equivalent eccentricity, that resulted to be around 5%. A seismic input consistent with the seismic hazard of the area has been considered assuming a set of seven ground motions, spectrum compatible with the EC8 provisions.

The introduced eccentricity, despite being very low, leads to non-negligible torsional effects. An increase larger than 10% has been found for the top story displacement, while the increase in the first-story interstory drift ranges between 7 and 25% depending on the considered PGA.

The evaluation of the exceedance probability of the assumed limit states has evidenced a significant sensitivity of the structure to the introduced irregularities. As regards the *life safety* limit state, an increase of the exceedance probability from 11 to 37% has been found (PGA = 0.25%), while for *immediate occupancy* limit state, the exceeding probability goes from 0.8 to 32% (PGA = 0.15 g).

The seismic performance level of the two structures has been studied comparing their fragility curves for each considered limit state. The comparison has evidenced a significant sensitivity of the seismic response to the introduced irregularities, depending, for each limit state, on the intensity of the seismic input.

The obtained results show that common irregularities significantly affect the seismic response of RC buildings realized with concrete having poor mechanical properties. To obtain more general results, further analyses should be performed on sample structures characterized by different geometrical and mechanical features.

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