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Influence of earthquake direction on the seismic response of irregular plan RC frame buildings

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Abstract: The nonlinear response of structures is usually evaluated by considering two accelerograms acting simultaneously along the orthogonal directions. In this study, the influence of the earthquake direction on the seismic response of building structures is examined. Three multi-story RC buildings, representing a very common structural typology in Italy, are used as case studies for the evaluation. They are, respectively, a rectangular plan shape, an L plan shape and a rectangular plan shape with courtyard buildings. Nonlinear static and dynamic analyses are performed by considering different seismic levels, characterized by peak ground acceleration on stiff soil equal to 0.35 g, 0.25 g and 0.15 g. Nonlinear dynamic analyses are carried out by considering twelve different earthquake directions, and rotating the direction of both the orthogonal components by 30° for each analysis (from 0° to 330°). The survey is carried out on the L plan shape structure. The results show that the angle of the seismic input motion significantly influences the response of RC structures; the critical seismic angle, i.e., the incidence angle that produces the maximum demand, provides an increase of up to 37% in terms of both roof displacements and plastic hinge rotations.

Keywords: plan irregularity; nonlinear analyses; reinforced concrete buildings; seismic performance; ground motion directionality

1 Introduction

Irregular structures often exhibit unfavorable seismic behavior, characterized by the concentration of plastic demand in a limited portion of the structure; this issue can cause early collapse under strong seismic motion. All modern building codes provide rules to verify the regularity of structures either in plan or in elevation; in case the rules are not followed, some "penalties" in the design are provided (CEN, 2004). The evidence from past earthquakes clearly denoted that the irregularity in plan, which can be caused by asymmetric distributions of mass, stiffness and strength, is one of the most frequent sources of severe damage, since it causes torsional floor rotations localizing the seismic demand in small portions of the building (e.g., De Stefano and Pintucchi, 2008). In past years, large research efforts were devoted to the study of the seismic response of irregular structures, both in plan (e.g., Athanatopoulou *et al*., 2006; Roy and Chakroborty, 2013) and in elevation (e.g., Magliulo *et al*., 2012a). The presence of nonstructural components, such as infill systems, strongly influences the lateral stiffness of the portion of the structure in which they are inserted (Magliulo *et al*., 2012c; Magliulo *et al*., 2013) and they may also affect the regularity of the structural system.

The modern seismic codes, such as Eurocode 8 (CEN, 2004), allow using different analysis methodologies: 1) lateral force, 2) modal response spectrum, 3) static nonlinear and 4) dynamic nonlinear analyses. Their level of reliability decreases from 4) to 1) and, consequently, the safety margin with respect to the same limit state should increase according to the same order (Magliulo *et al*., 2007). The choice of a method depends on the structural characteristics (regularity, fundamental periods, presence of isolation systems) and the importance of the structure. Certainly, among the different approaches, nonlinear dynamic analysis is able to provide the best prediction of the structural response. The influence of the seismic analysis on the vulnerability assessment of buildings is also discussed in Magliulo *et al*. (2008) focusing on existing precast industrial buildings.

The nonlinear response of 3D structures is usually determined considering the two horizontal seismic components acting simultaneously (e.g., Di Sarno *et al*., 2011; Magliulo and Ramasco, 2007), that simulate the realistic condition to which a building is subjected during a seismic event. Many years ago, MacRae and Mattheis

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(2000) concluded that the assessment of building drifts for bidirectional shaking effects is dependent on the chosen reference axes. Athanatopoulou (2005) proposed analytical formulas for the determination of the critical angle of seismic incidence, i.e., the incidence angle of the seismic input that produces the maximum demand, and the corresponding maximum value of a response quantity of structures considering either two or three seismic components. The developed analytical formulas are derived by assuming that the structure behaves linearly. Rigato and Medina (2007) demonstrated that the inelastic peak deformation demands are underestimated if bi-directional ground motions are applied only along the principal axes of an inelastic building when compared to those obtained at other angles of incidence. They proposed a procedure that takes into account the critical seismic angle and demonstrated that this procedure is the most efficient for the design of RC frame elements. Reyes and Kalkan (2012) examined the influence that the angle of incidence of the ground motion has on several engineering demand parameters (EDPs). The study focused on both symmetric and asymmetric buildings in near fault sites, in which the records are typically rotated to fault-normal/fault-parallel directions. It was found that nonlinear dynamic analyses for ground motions oriented in the FN/FP axes could underestimate the peak value of median-displacement overall orientations. Such an underestimation was less than 20% when a large set of ground motions was adopted.

The aim of the study presented here is to analyze the seismic behavior of three typical Italian RC buildings, both regular and irregular in plan, with different nonlinear analysis methods, in order to evaluate the differences, if any, among the analysis methodologies. In particular, the results of nonlinear dynamic analysis, nonlinear static analysis with "modal" load pattern and nonlinear static analysis with "uniform" load pattern are compared. Furthermore, the influence of the incidence angle of the seismic action is also investigated by performing twelve nonlinear dynamic analyses, rotating the direction of both the orthogonal components by 30° for each analysis (from 0° to 330°).

The analyses are performed both at the Significant Damage Limit State, assuming as reference the EC8 elastic spectrum characterized by a return period of 475 years, corresponding to a 10% probability of exceedance in 50 years, and at the Near Collapse Limit State, characterized by a return period of 2,475 years, assuming as reference the same spectrum amplified by a factor equal to 1.5 (Ordinanza del Presidente del Consiglio dei Ministri n. 3274 del 20/3/2003, 2003).

The results of the nonlinear analysis methodologies are compared in terms of the ratio between the required rotation and the capacity at the top and bottom of each column in the two directions, and of roof displacements. Some observations on the influence of the incidence angle of the seismic action are also made; they aim at the evaluation of a critical angle for which the seismic demand attains the maximum values.

2 Methodology

In this study, three different RC buildings are considered, and their geometry is shown in Fig. 1. The selected buildings are representative of typical Italian RC structures, both regular (Building No. 1) and irregular in plan (Buildings No. 2 and 3). The chosen geometry and longitudinal and horizontal reinforcement are commonly found throughout Italy.

Building No. 1 (Fig. 1(a)) is a "regular" doubly symmetric four-story building with 3.20 m interstory

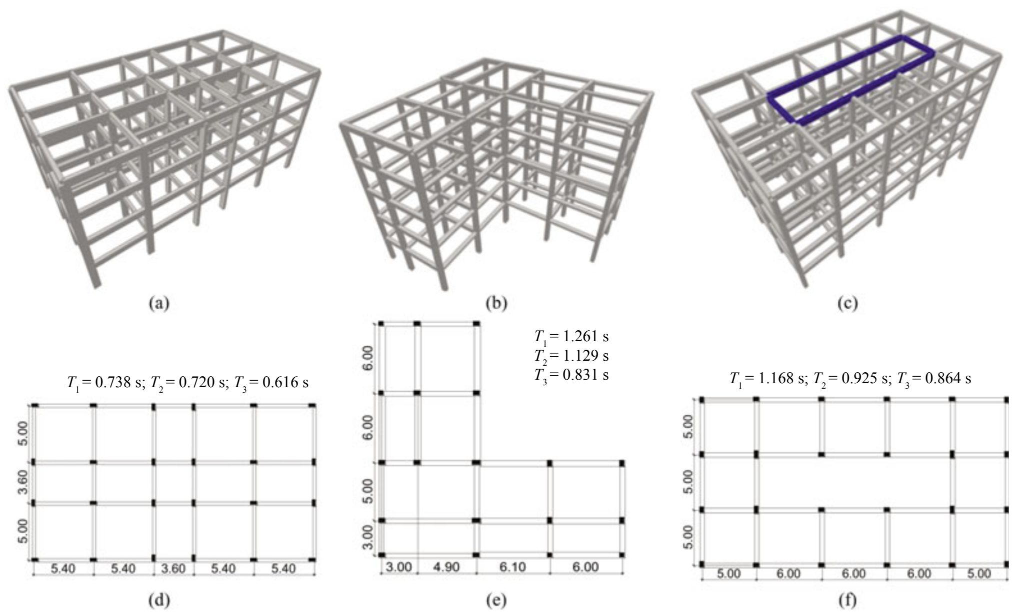


Fig. 1 3D view of the analyzed buildings: (a) Building No. 1; (b) Building No. 2; (c) Building No. 3 Plan view of the considered buildings and the first three natural periods: (d) Building No. 1; (e) Building No. 2; (f) Building No. 3

height for each level; the dimensions of the sections of all the columns and beams, expressed in cm, are 30×60. Concrete average cubic strength, equal to 33.5 N/mm^2, and steel average strength, equal to 500 N/mm^2, is adopted.

Building No. 2 (Fig. 1(b)) is an L plan shape five-story structure with an interstory height equal to 3.50 m for each level. The dimensions of the column sections remain constant for the entire height of the building and are represented by three different typologies (in cm): 40×40, 40×50, 40×70. The beams also have different section dimensions (in cm): 40×60, 40×50, 25×50. Concerning the materials, concrete average cubic strength, equal to 25 N/mm^2, and steel average strength, equal to 400 N/mm^2, is adopted.

Building No. 3 (Fig. 1(c)) has a rectangular shape with a central courtyard; it is a five-story building, with the interstory height equal to 3.50 m for each level. The columns have two different typologies of section dimensions maintained constant for all the stories (in cm): 40×55 and 40×70. The beams also have different section dimensions assembled in three typologies (in cm): 40×60, 40×50, 25×40. As in Building No. 2, concrete average cubic strength, equal to 25 N/mm^2, and steel average strength, equal to 400 N/mm^2, is used.

The structures were defined and selected by a wide research group, in the framework of the ReLUIS-DPC 2005-2008 project, in order to be representative of the current Italian RC building stock. Further details on the geometry of the elements are included in Bianchi *et al.* (2007) and Maddaloni *et al.* (2008).

The buildings are assumed to be founded on a "B" soil type, according to EC8 classification, i.e., soil with an average shear wave velocity in the range between 360 m/s and 800 m/s.

Modal analyses of the structures are performed considering a reduction of the 50% in the inertia moment of the section of all the elements, according to the prescriptions included in Eurocode 8 (CEN, 2004); Eurocode 8 allows reducing the flexural stiffness properties of concrete elements up to one-half of the corresponding stiffness of the uncracked elements, in order to take into account the effect of cracking. Building No. 1 and Building No. 3 are characterized by a first translational mode along the smallest plan dimension direction, with periods equal to 0.74 s and 1.17 s, respectively. Building No. 2 is characterized by a first torsional mode, with a period equal to 1.26 s.

Nonlinear analyses are performed by means of two computer programs, i.e., SAP2000 (CSI Computer & Structures Inc., 2004) and CANNY99 (Li, 1996). Both beams and columns are characterized by lumped plasticity models. For the columns, two independent nonlinear springs are assigned, one for each orthogonal direction. No axial force-bending moment interaction is considered in the plastic hinges. Bending moment springs are characterized by a bilinear skeleton curve, defined by the yielding and ultimate moment and the corresponding rotations. Yielding and ultimate moments are defined by the bilinear envelope of the moment-curvature diagram using the MC program (Li, 1996). The yielding θ_y and the ultimate rotations θ_u of the plastic hinges are evaluated as provided by EC8 (CEN, 2005). The hysteretic model is Takeda type; the pinching effect is also taken into account.

The moments and corresponding curvatures are computed considering a parabola-rectangle diagram for concrete under compression, with a strain value at the end of the parabola equal to 0.2% and an ultimate strain equal to 0.35%. An elastic-perfectly plastic steel stress-strain diagram is considered for the ribbed longitudinal bars. Bond-slip relationship is not directly taken into account. The above mentioned mean strength values are assigned to concrete (f_c) and steel (f_y) strength.

3 Nonlinear analyses

For each building, pushover analyses are performed in both the horizontal directions according to the N2 method (Fajfar, 2000). Two different distributions of horizontal forces are considered: a "uniform" pattern, based on lateral forces that are proportional to mass and a "modal" pattern, consistent with the lateral force distribution of the first mode. Furthermore, the analyses consider three different seismic levels, i.e., the demand is represented by three elastic spectra obtained considering three values of design peak ground accelerations on stiff soil: 0.15 g, 0.25 g and 0.35 g. The safety of the structure is assessed for the different limit states (Significant Damage Limit State and Near Collapse Limit State) and the three seismic intensities.

Both the horizontal components of three sets of seven earthquakes, i.e., 42 natural recorded events, are used for nonlinear dynamic analyses and the results are shown herein. The three sets are selected according to the EC8 rules from the European Strong Motion Database (Fig. 2) (Ambraseys *et al.*, 2002). Arbitrary accelerogram magnitude and distance bins are considered in the preliminary search. They are selected in order to be compatible (i.e., Iervolino *et al.*, 2008) with the EC8 elastic spectrum for a design ground acceleration on stiff soil equal to 0.35 g, 0.25 g and 0.15 g (thick black line in Fig. 2). A 1.2 scale factor is applied to three accelerograms in the set that matches the spectrum with a design ground acceleration equal to 0.35 g. According to the selection procedures presented in the literature (Iervolino *et al.*, 2008; Iervolino *et al.*, 2010; Maddaloni *et al.*, 2012), they satisfy the EC8 rule for spectrum matching; in the range of periods between 0.2 T_1 and 2 T_1, where T_1 is the fundamental period of the structure, the mean 5% damping elastic spectrum (thick red line in Fig. 2) of the selected accelerograms is not less than 90% of the corresponding value of the 5% damping elastic response spectrum (black line in Fig. 2). Then, nonlinear dynamic analyses, considering different hazard levels, i.e., $a_g = 0.15$ g, $a_g = 0.25$ g, $a_g = 0.35$ g, for the Significant Damage Limit State, are performed.

The main characteristics of the selected earthquakes are listed in Tables 1, 2 and 3. Instead, for the Near Collapse Limit State, characterized by a return period of 2,475 years, the above mentioned analyses are repeated with the same earthquakes amplified by a factor equal to 1.5 according to OPCM 3274 (ordinanza del Presidente del Consiglio dei Ministri n. 3274 del 20/3/2003, 2003).

The nonlinear analyses are performed for each earthquake, by simultaneously applying the two horizontal components. The top displacements and rotational ductility demand evaluated at the column ends are considered as response parameters. Such a demand

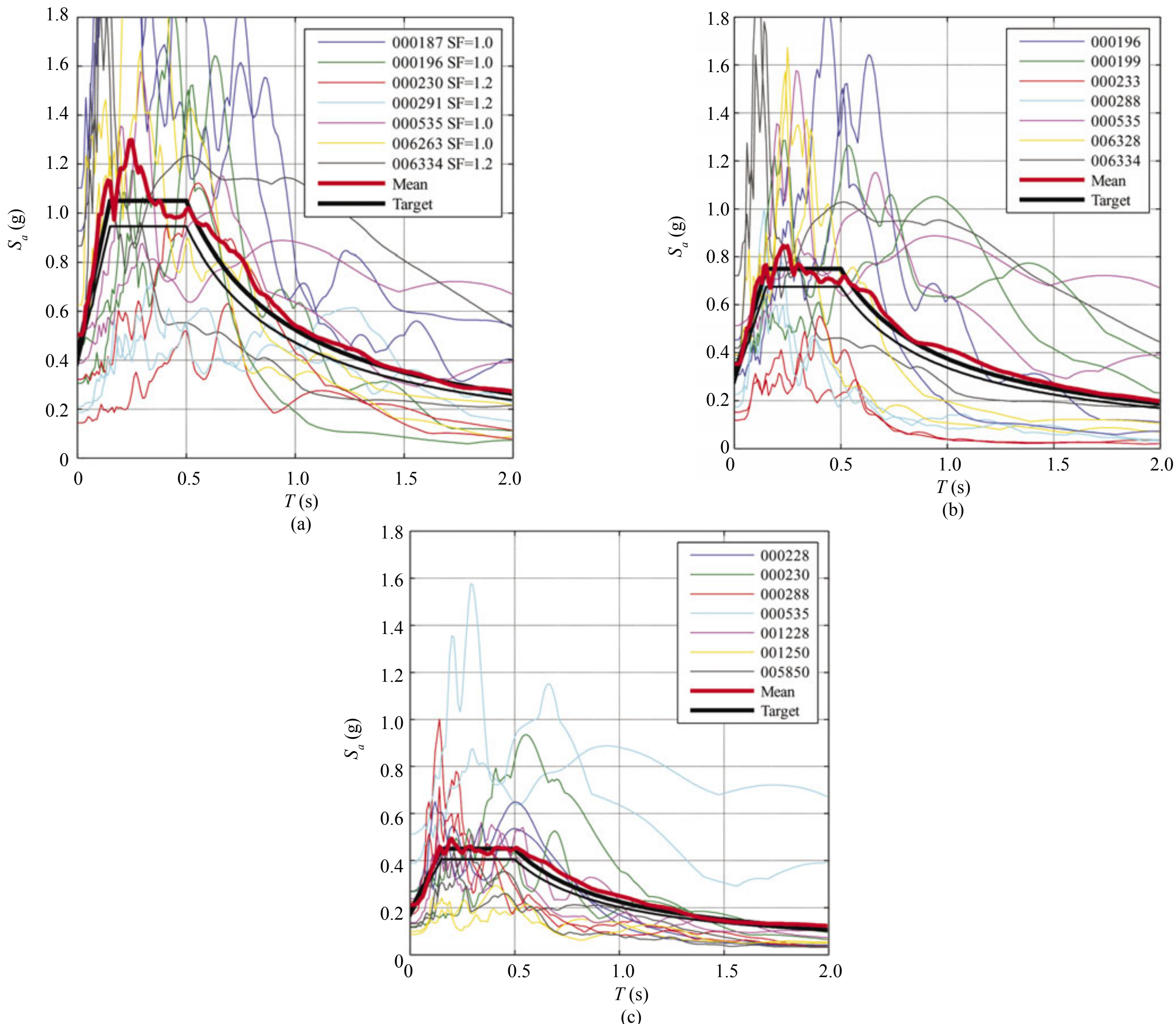


Fig. 2 Comparison of the acceleration response spectra of the selected accelerograms, their mean 5% damping elastic spectrum (thick red line), EC8 target elastic response spectrum (thick black line) and the 90% of the target response spectrum (black line) for design peak ground acceleration on stiff soil equal to (a) 0.35 g, (b) 0.25 g and (c) 0.15 g

Table 1 Set of accelerograms selected in order to match the EC8 elastic spectrum for a design ground acceleration on stiff soil equal to 0.35 g

Earthquake code	Earthquake name	Moment magnitude	Epicentral distance (km)	Date (dd/mm/yy)	PGA_X (g)	PGA_Y (g)	SF
000187	Tabas	7.3	57	16/09/1978	0.93	1.10	1.0
000196	Montenegro	6.9	25	15/04/1979	0.45	0.31	1.0
000230	Montenegro (aftershock)	6.2	8	24/05/1979	0.12	0.27	1.2
000291	Campano Lucano	6.9	16	23/11/1980	0.16	0.18	1.2
000535	Erzincan	6.6	13	13/03/1992	0.39	0.51	1.0
006263	South Iceland	6.5	7	17/06/2000	0.63	0.51	1.0
006334	South Iceland (aftershock)	6.4	11	21/06/2000	0.42	0.72	1.2

Table 2 Set of accelerograms selected in order to match the EC8 elastic spectrum for a design ground acceleration on stiff soil equal to 0.25 g

Earthquake code	Earthquake name	Moment magnitude	Epicentral distance (km)	Date (dd/mm/yy)	PGA_*X* (g)	PGA_*Y* (g)	SF
000196	Montenegro	6.9	25	15/04/1979	0.45	0.31	1.0
000199	Montenegro	6.9	16	15/04/1979	0.38	0.36	1.0
000233	Montenegro (aftershock)	6.2	22	24/05/1979	0.12	0.15	1.0
000288	Campano Lucano	6.9	43	23/11/1980	0.23	0.17	1.0
000535	Erzincan	6.6	13	13/03/1992	0.39	0.51	1.0
006328	South Iceland (aftershock)	6.4	12	21/06/2000	0.33	0.39	1.0
006334	South Iceland (aftershock)	6.4	11	21/06/2000	0.42	0.72	1.0

Table 3 Set of accelerograms selected in order to match the EC8 elastic spectrum for a design ground acceleration on stiff soil equal to 0.15 g

Earthquake code	Earthquake name	Moment magnitude	Epicentral Distance (km)	Date (dd/mm/yy)	PGA_*X* (g)	PGA_*Y* (g)	SF
000228	Montenegro (aftershock)	6.2	33	24/05/1979	0.20	0.27	1.0
000230	Montenegro (aftershock)	6.2	8	24/05/1979	0.12	0.27	1.0
000288	Campano Lucano	6.9	43	23/11/1980	0.23	0.17	1.0
000535	Erzincan	6.6	13	13/03/1992	0.39	0.51	1.0
001228	Izmit	7.6	47	17/08/1999	0.24	0.14	1.0
001250	Izmit	7.6	172	17/08/1999	0.09	0.10	1.0
005850	Strofades	6.6	38	18/11/1997	0.13	0.12	1.0

is compared to the capacity at the Significant Damage Limit State, which corresponds to the attainment at elements ends of a rotation equal to 3/4 θ_u, mentioned above.

The comparison is also performed at the Near Collapse Limit State; in such a case, the demand is evaluated considering the accelerograms amplified by 1.5 and the capacity corresponding to the attainment of the ultimate rotation θ_u.

3.1 Comparison between nonlinear static analyses and nonlinear dynamic analyses

The results of the nonlinear static analyses are compared with those obtained by nonlinear dynamic analyses. The average of the seven maximum demand values obtained applying the seven earthquakes is considered to be the result of nonlinear dynamic analysis, following the EC8 rule (Magliulo *et al*., 2012b). In particular, the ratios (*R*) between the required rotation and the capacity at the top and bottom of each column in the two directions are compared for the three hazard seismic levels; such a comparison is performed considering the maximum ratios among all the columns of the building, as well as the average values at each story level. The evaluations are performed both at the Significant Damage Limit State and at the Near Collapse Limit State. In Fig. 3, for sake of brevity, only the maximum ratios *R*, considering both the *X* and *Y* direction, in the columns at the 1st story at Significant Damage Limit State are shown; indeed, the damage is concentrated at the first story, i.e., the maximum *R* values are recorded in the column at the 1st story. For the nonlinear static analysis, the two different distributions of horizontal forces ("uniform" and "modal" pattern) are both considered.

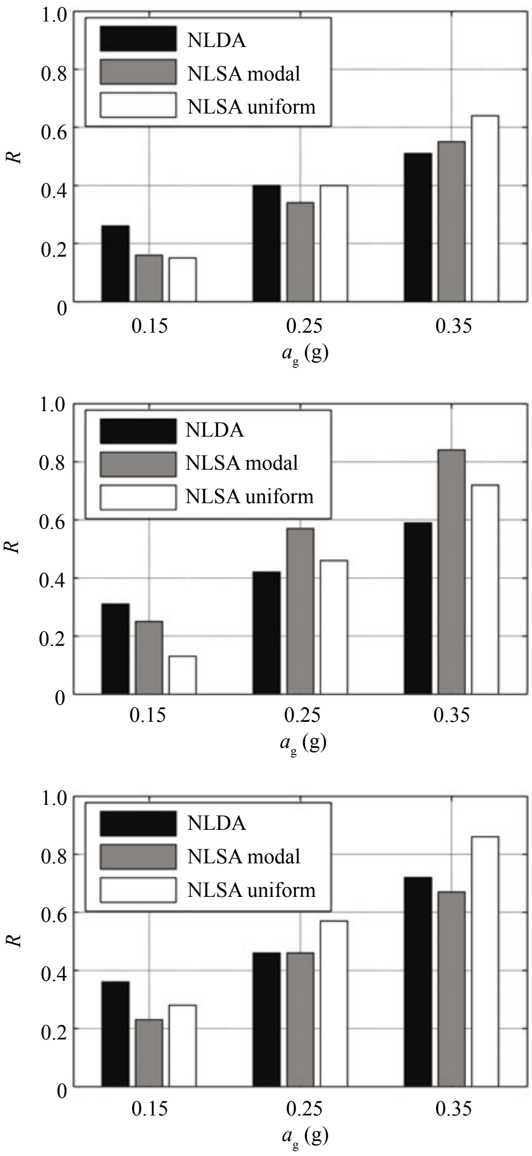
Magliulo *et al*. (2007) concluded that the safety assessment performed according to the static nonlinear analysis is generally more conservative, i.e., it yields a smaller safety factor with respect to the safety assessment performed according to nonlinear dynamic analysis. Figure 3 shows that such a conclusion, generally valid for new structures designed according to EC8, is not always valid for the analyzed buildings. In particular, it is not verified for low intensity earthquakes.

As shown in Fig. 3, the *R* values resulting from nonlinear dynamic analyses are smaller than those resulting from nonlinear static analysis for low seismic intensity levels. As the seismic hazard level increases, the latter comparison is inverted, i.e., the pushover *R* values are larger than the dynamic nonlinear analysis *R* values. Note that the maximum value between the *R* values resulting from modal and uniform load patterns has to be taken into account.

Comparing the *R* values among the different structures, the unfavorable behavior of irregular structures is also confirmed. Indeed, the ratios between the demand and the capacity in Buildings No. 2 and No. 3 are larger than those recorded in Building No. 1.

The results related to the Near Collapse Limit State (NCLS) follow the same trend as shown in Fig. 3. However, larger values of the ratio *R* are provided, as expected. In fact, the seismic input is 50% more intense than the Significant Damage Limit State (SDLS), while the rotation capacity is 33% larger than SDLS.

In Figs. 4-6, for each floor, the average values of the



Building No. 1

a_g (g)	Nonlinear dynamic analysis	Pushover "modal" pattern	Pushover "uniform" pattern
0.15	0.26	0.16	0.15
0.25	0.40	0.34	0.40
0.35	0.51	0.55	0.64

Building No. 2

a_g (g)	Nonlinear dynamic analysis	Pushover "modal" pattern	Pushover "uniform" pattern
0.15	0.31	0.25	0.13
0.25	0.42	0.57	0.46
0.35	0.59	0.84	0.72

Building No. 3

a_g (g)	Nonlinear dynamic analysis	Pushover "modal" pattern	Pushover "uniform" pattern
0.15	0.36	0.23	0.28
0.25	0.46	0.46	0.57
0.35	0.72	0.67	0.86

Fig. 3 Comparison between maximum values of the ratio *R* between the required rotation and the capacity in the columns at the 1st story for nonlinear dynamic analysis (NLDA) and nonlinear static analysis with "modal" load pattern (NLSA modal) and "uniform" load pattern (NLSA uniform) at Significant Damage Limit State for the three considered buildings

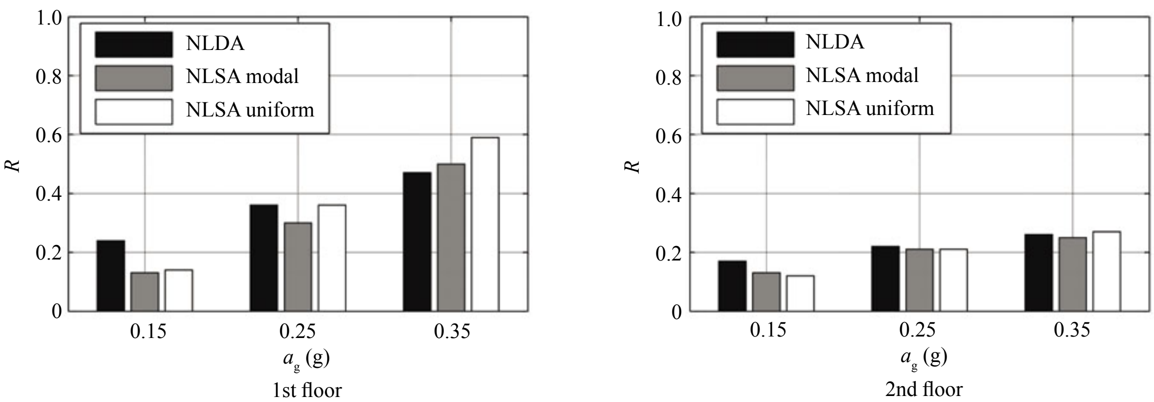


Fig. 4 Comparison between average values of the ratios *R* between the required rotation and the capacity in the columns at the four stories for nonlinear dynamic analysis (NLDA) and the ratios *R* resulting from nonlinear static analysis with "modal" load pattern (NLSA modal) and "uniform" load pattern (NLSA uniform) at Significant Damage Limit State for Building No. 1

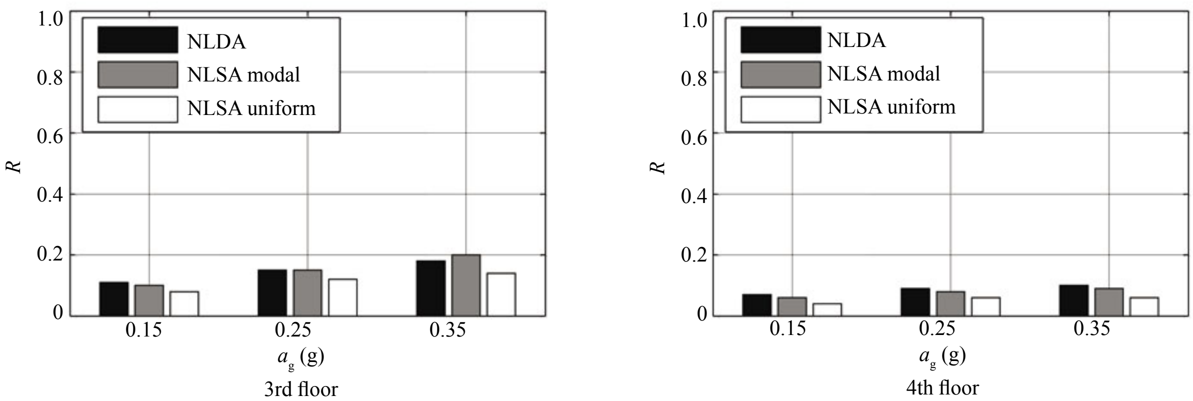


Fig. 4 Continued

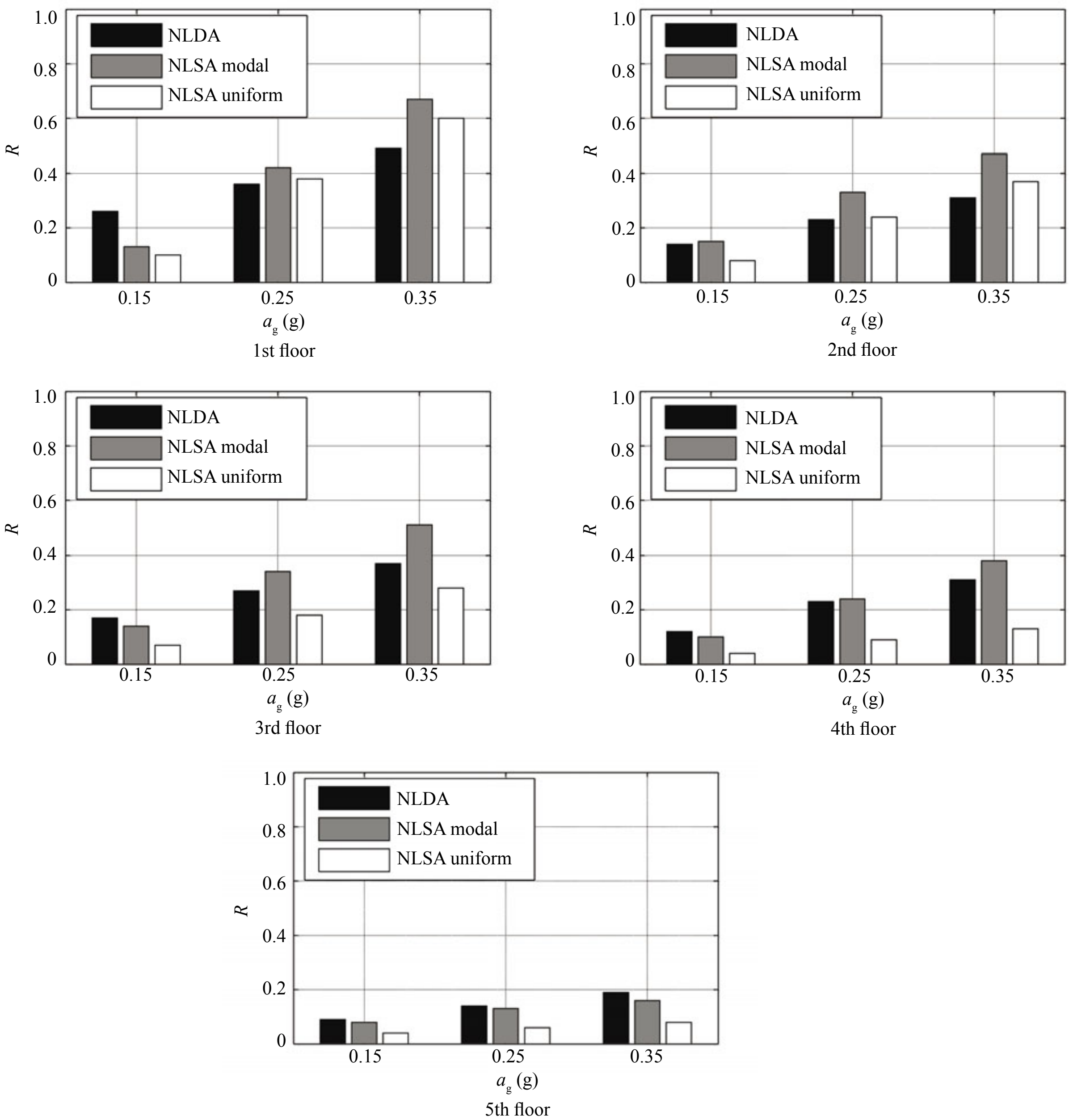


Fig. 5 Comparison between average values of the ratios *R* between the required rotation and the capacity in the columns at the four stories for nonlinear dynamic analysis (NLDA) and the ratios *R* resulting from nonlinear static analysis with "modal" load pattern (NLSA modal) and "uniform" load pattern (NLSA uniform) at Significant Damage Limit State for Building No. 2

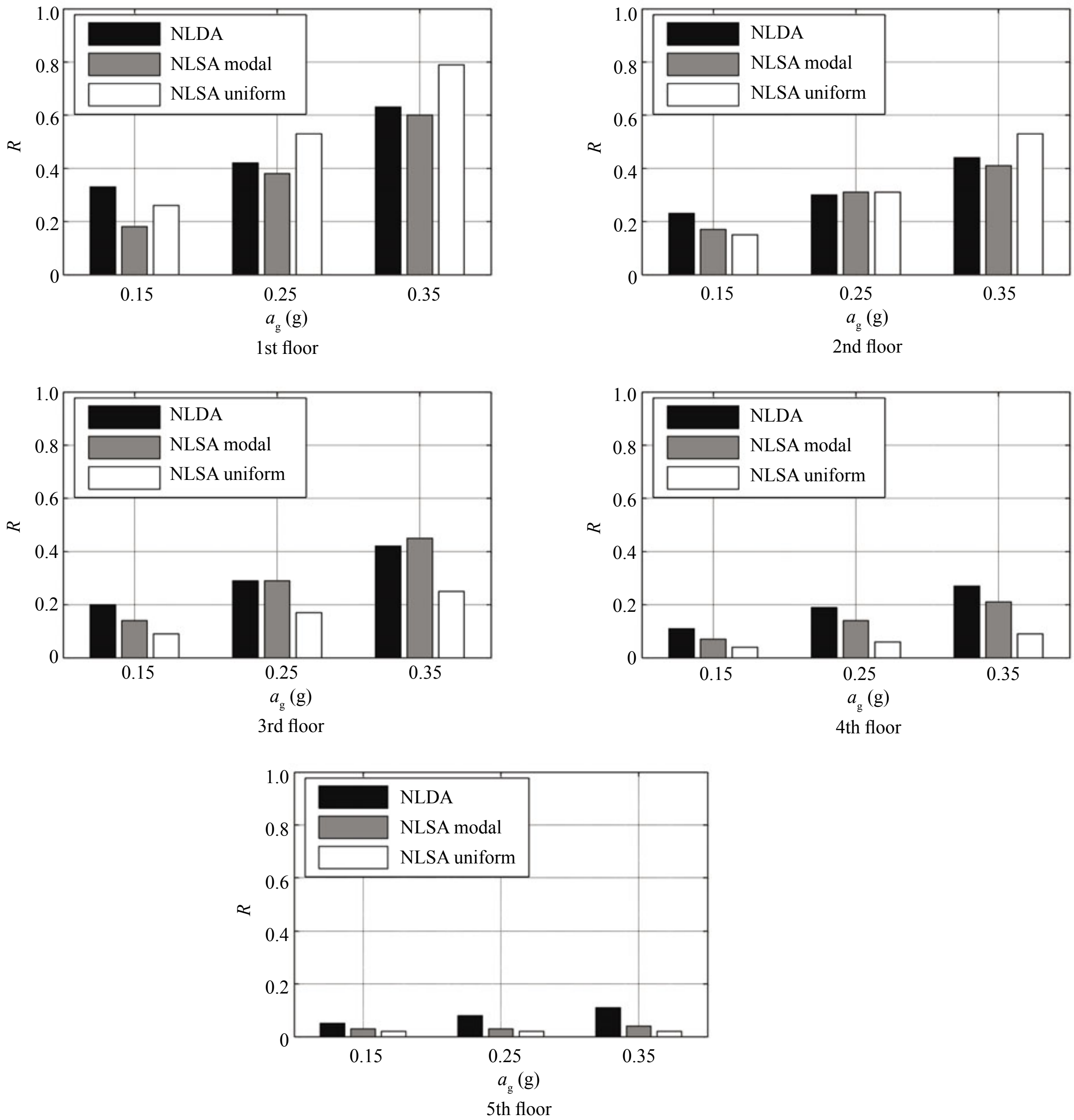


Fig. 6 Comparison between average values of the ratios *R* between the required rotation and the capacity in the columns at the four stories for nonlinear dynamic analysis (NLDA) and the ratios *R* resulting from nonlinear static analysis with "modal" load pattern (NLSA modal) and "uniform" load pattern (NLSA uniform) at Significant Damage Limit State for Building No. 3

ratio R between the required rotation and the capacity are shown for the three buildings at the Significant Damage Limit State. The ratio R is computed as the average of the R values in the columns resulting from the X and Y orthogonal directions.

The results in Figs. 4-6 show that the pushover analysis performed by the uniform force pattern provides a larger demand at the bottom floors than the one provided by the modal force pattern. They also confirm that nonlinear dynamic analysis gives larger R values than nonlinear static analysis for low intensity seismic actions.

The average values of R in Building No. 1 (Fig. 4) are comparable to the maximum values shown in Fig. 3. Instead, for Building No. 2 and No. 3, a large discrepancy between average (Fig. 5 and Fig. 6) and maximum values is provided. This clearly denotes that in irregular structures, the demand is concentrated in certain location and the seismic demand is not well divided among the different elements.

The results provided by nonlinear dynamic analyses are larger than those provided by pushover analyses at the top floors. The main reason for the underestimation of the demand in pushover analyses is the fact that they do not take into account the effect of higher-modes, whose influence is significant especially in the top floors. The discrepancy between the two analysis methodologies increases with larger seismic intensities. This tendency confirms that the more the structure goes in the inelastic range, the influence of higher modes on the seismic response becomes greater (Fischinger *et al*., 2011; Rejec *et al*., 2012). However, the absolute values of the required rotations are far enough from the capacity; hence, this issue does not compromise the structural assessment for the considered buildings.

3.2 Influence of the incidence angle on the seismic response

Building No. 2 is irregular in plan according to the EC8 rules. The influence of the incidence angle on the demand is evaluated on this structure in order to further investigate its seismic performance, which is known to exhibit an unusual dynamic behavior. Twelve different earthquake directions are considered, rotating the orthogonal components of the selected accelerograms by 30 degrees for each analysis (from 0° to 330°).

The results in terms of both the top displacements in the two orthogonal directions and the vectorial top displacement (i.e., the Square Root of the Sum of the Squares, namely the SRSS, of the orthogonal displacements) are shown in Fig. 7 considering the three selected sets of accelerograms at the Significant Damage Limit State. Since the maximum top displacements in the two horizontal directions are not attained at the same time instant, the maximum vectorial top displacement is not directly obtained by composing the maximum X and Y displacements.

In Fig. 8, the influence of the seismic input angle in terms of the ratio R between the required rotation and the capacity in the column plastic hinges is also shown for the three hazard seismic zones at the Significant Damage Limit State.

The incidence angles that induce the largest values of the considered engineering demand parameters are highlighted in Fig. 7 and Fig. 8 with a larger marker.

Some of the numerical values plotted in Fig. 7 and Fig. 8 are shown in Table 4 and Table 5. The variations of the seismic demand are evaluated with respect to the response obtained by applying the input without any rotation. The mean value of the recorded displacement, the standard deviation and the relative coefficient of variation are also evaluated in order to assess the

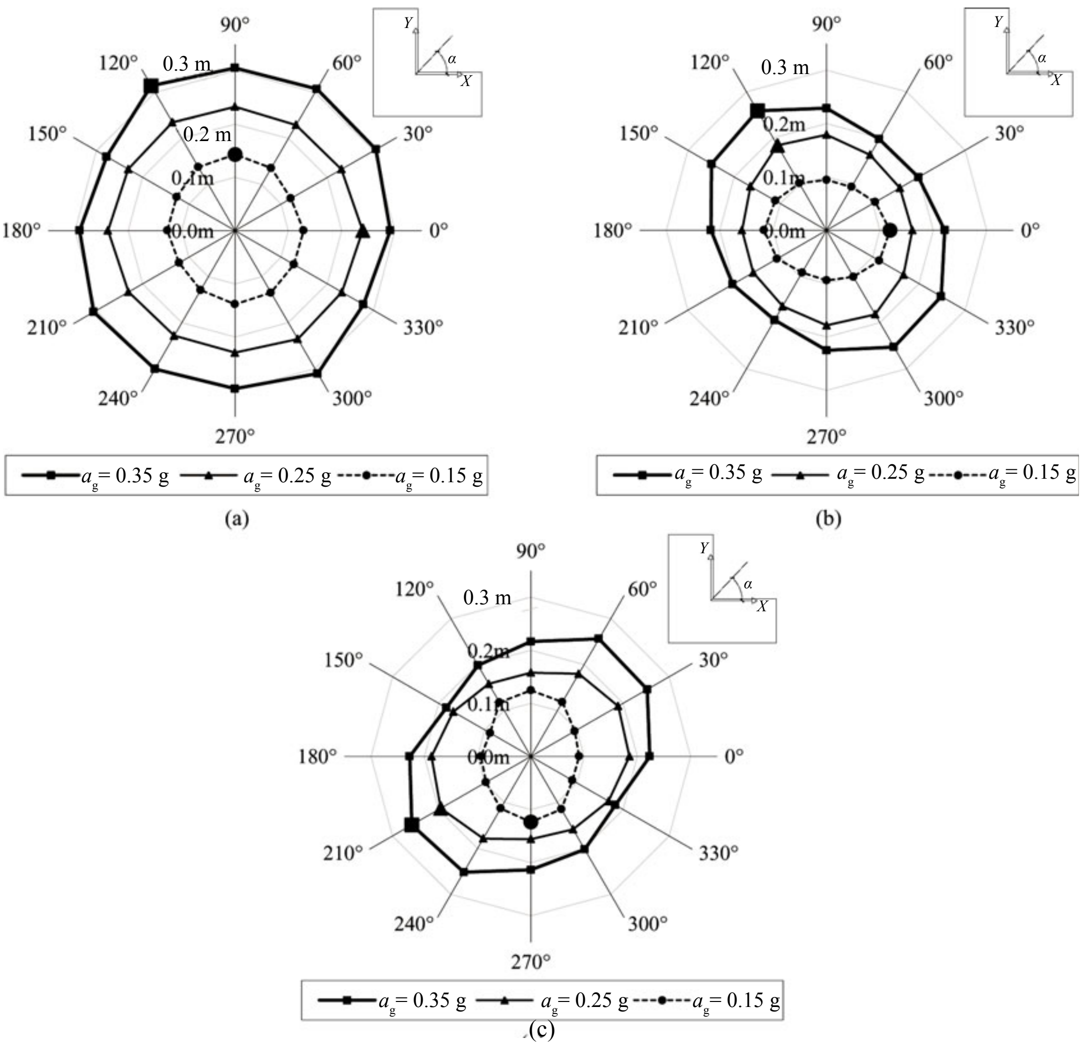


Fig. 7 Influence of the incidence angle of the seismic input on the response of Building No. 2 in terms of (a) top vectorial displacement, (b) X direction top displacement and (c) Y direction top displacement for 0.35 g, 0.25 g and 0.15 g seismic input

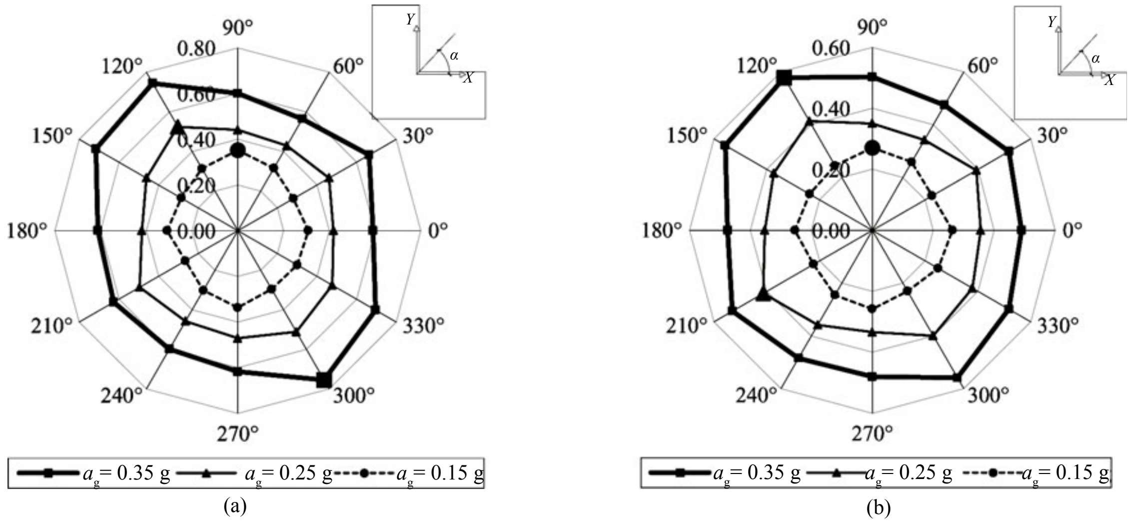


Fig. 8 Influence of the incidence angle *α* of the seismic input on the response of Building No. 2 in terms of the ratio *R* between the required rotation and the capacity considering the (a) maximum attained rotation and (b) the average of the rotations in columns at the first story

Table 4 Influence of the incidence angle of the seismic input on the response of Building No. 2 in terms of *X* direction (Δ_X) and *Y* direction top displacement (Δ_Y) for the three considered seismic intensities. The mean value (μ) of the recorded displacements, the standard deviation (σ) and the relative coefficient of variation (CoV) are also evaluated

	a_g = 0.35 g				a_g = 0.25 g				a_g = 0.15 g			
	X (m)	Var. (%)	*Y* (m)	Var. (%)	*X* (m)	Var. (%)	*Y* (m)	Var. (%)	*X* (m)	Var. (%)	*Y* (m)	Var. (%)
0°	0.222		0.223		0.161		0.185		0.119		0.091	
30°	0.199	-10.2	0.252	13.0	0.159	-1.2	0.189	2.3	0.106	-11.4	0.095	5.0
60°	0.198	-10.8	0.256	14.5	0.163	1.5	0.180	-3.0	0.094	-21.2	0.118	30.5
90°	0.229	3.3	0.216	-3.1	0.179	11.5	0.158	-14.8	0.094	-20.8	0.124	37.3
120°	0.259	16.7	0.198	-11.2	0.184	14.4	0.158	-14.7	0.102	-14.5	0.118	29.7
150°	0.249	12.2	0.183	-17.9	0.165	2.4	0.168	-9.2	0.111	-6.9	0.088	-2.5
180°	0.217	-2.2	0.228	2.0	0.158	-1.5	0.187	0.8	0.117	-1.7	0.093	3.1
210°	0.203	-8.3	0.258	15.7	0.159	-1.3	0.196	5.6	0.107	-10.2	0.097	7.5
240°	0.194	-12.3	0.252	12.8	0.165	2.4	0.179	-3.5	0.092	-23.1	0.113	24.8
270°	0.225	1.6	0.214	-4.3	0.178	11.0	0.156	-16.0	0.094	-21.0	0.124	36.3
300°	0.253	14.0	0.202	-9.5	0.182	13.3	0.158	-14.4	0.101	-15.4	0.115	26.8
330°	0.248	12.0	0.183	-17.9	0.167	3.7	0.169	-9.0	0.114	-4.2	0.090	-0.5
MAX	0.259	16.7	0.258	15.7	0.184	14.4	0.196	5.6	0.119	0.0	0.124	37.3
μ	0.225		0.222		0.168		0.173		0.104		0.106	
σ	0.023		0.028		0.010		0.014		0.010		0.014	
CoV		10.3		12.4		5.8		8.2		9.3		13.4

variability of the demand.

In Fig. 9, the variation of the ratio *R* with the input seismic incidence angle is shown for the different seismic intensities.

The following conclusions can be drawn.

(1) The incidence angle of the seismic input motion causes a significant variation of the response of RC structures; the variation with respect to the case in which the ground motion is applied without any rotation is about 15% for the horizontal top displacements. Only in a single case, i.e., the *Y* direction top displacement for a_g = 0.15g, the variation reaches values close to 40%. The top displacement variations, excluding the *Y* top displacement case for a_g = 0.15g, are in the same range as the outcomes included in Reyes and Kalkan (2012), even if far field records are considered in this study. If the rotations in plastic hinges are analyzed, the discrepancy due to a different incidence angle may be up to 30% (Fig. 9). For this reason, the use of a single analysis without considering the uncertainty due to the incidence angle may be questioned.

(2) The critical incidence angle, i.e., the incidence angle that produces the maximum demand, depends both on the chosen engineering demand parameter, e.g., displacements or plastic hinge rotations, and on the intensity level of the seismic input. This issue clearly denotes the large uncertainty related to the definition of a critical incidence angle.

Table 5 Influence of the incidence angle of the seismic input on the response of Building No. 2 in terms of the ratio *R* between the required rotation and the capacity considering the maximum rotation attained in columns for the three considered seismic intensities. The mean value (μ) of the recorded values, the standard deviation (σ) and the relative coefficient of variation (CoV) are also evaluated

	a_g = 0.35 g		a_g = 0.25 g		a_g = 0.15 g	
	R	Var. (%)	*R*	Var. (%)	*R*	Var. (%)
0°	0.59		0.42		0.31	
30°	0.66	12.2	0.46	10.1	0.28	-9.0
60°	0.56	-4.6	0.43	2.5	0.32	2.2
90°	0.60	1.5	0.44	5.3	0.35	14.0
120°	0.74	26.0	0.53	25.4	0.31	1.5
150°	0.72	21.0	0.46	10.2	0.29	-7.5
180°	0.61	3.5	0.42	0.1	0.31	0.3
210°	0.63	5.9	0.50	18.4	0.26	-14.1
240°	0.60	1.2	0.46	9.3	0.30	-2.2
270°	0.62	4.4	0.47	12.5	0.34	9.4
300°	0.75	27.6	0.51	22.2	0.30	-4.0
330°	0.70	17.6	0.48	14.0	0.30	-2.5
MAX	0.75	27.6	0.53	25.4	0.35	14.0
μ	0.65		0.46		0.31	
σ	0.06		0.03		0.02	
CoV		9.9		7.4		7.7

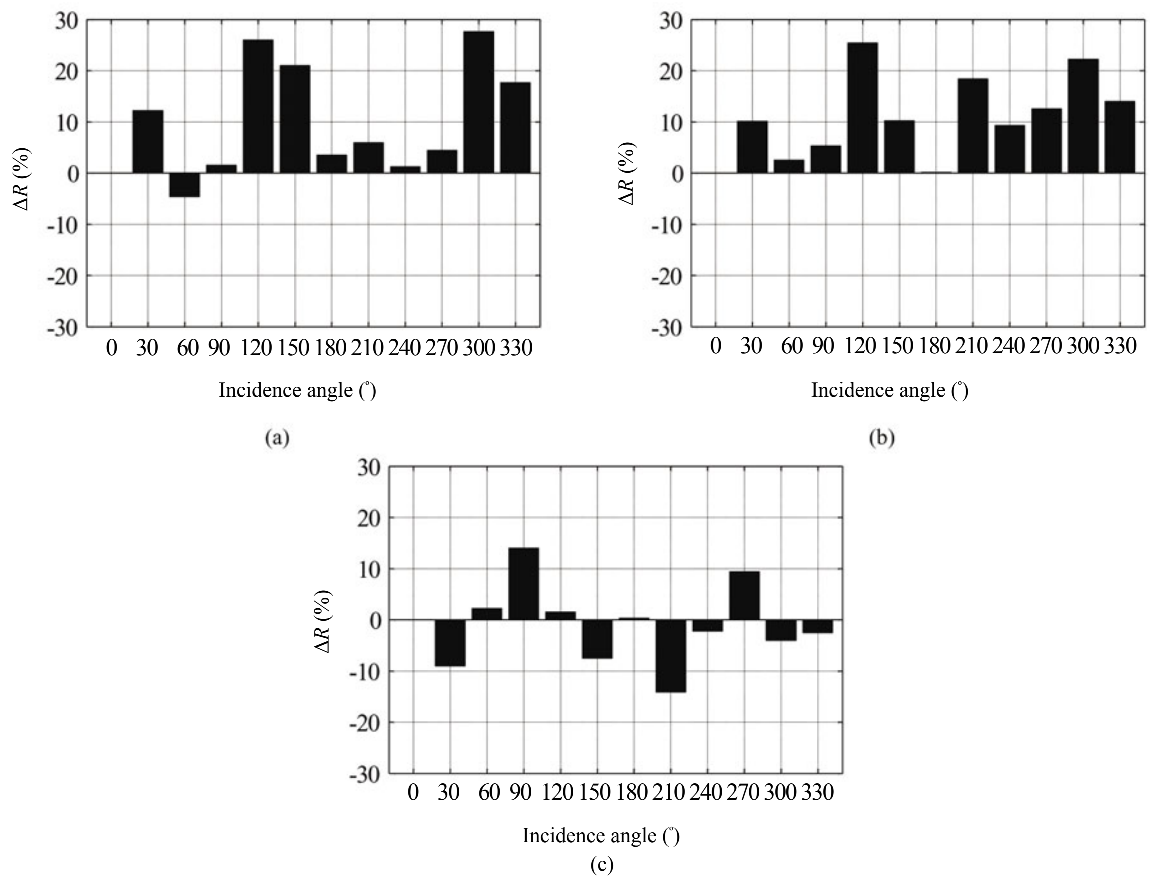


Fig. 9 Influence of the incidence angle of the seismic input on the response of Building No. 2 in terms of the variation of the ratio *R* (ΔR) between the required rotation and the capacity considering the maximum rotation attained in columns for (a) a_g = 0.35 g, (b) a_g = 0.25 g and (c) a_g = 0.15 g

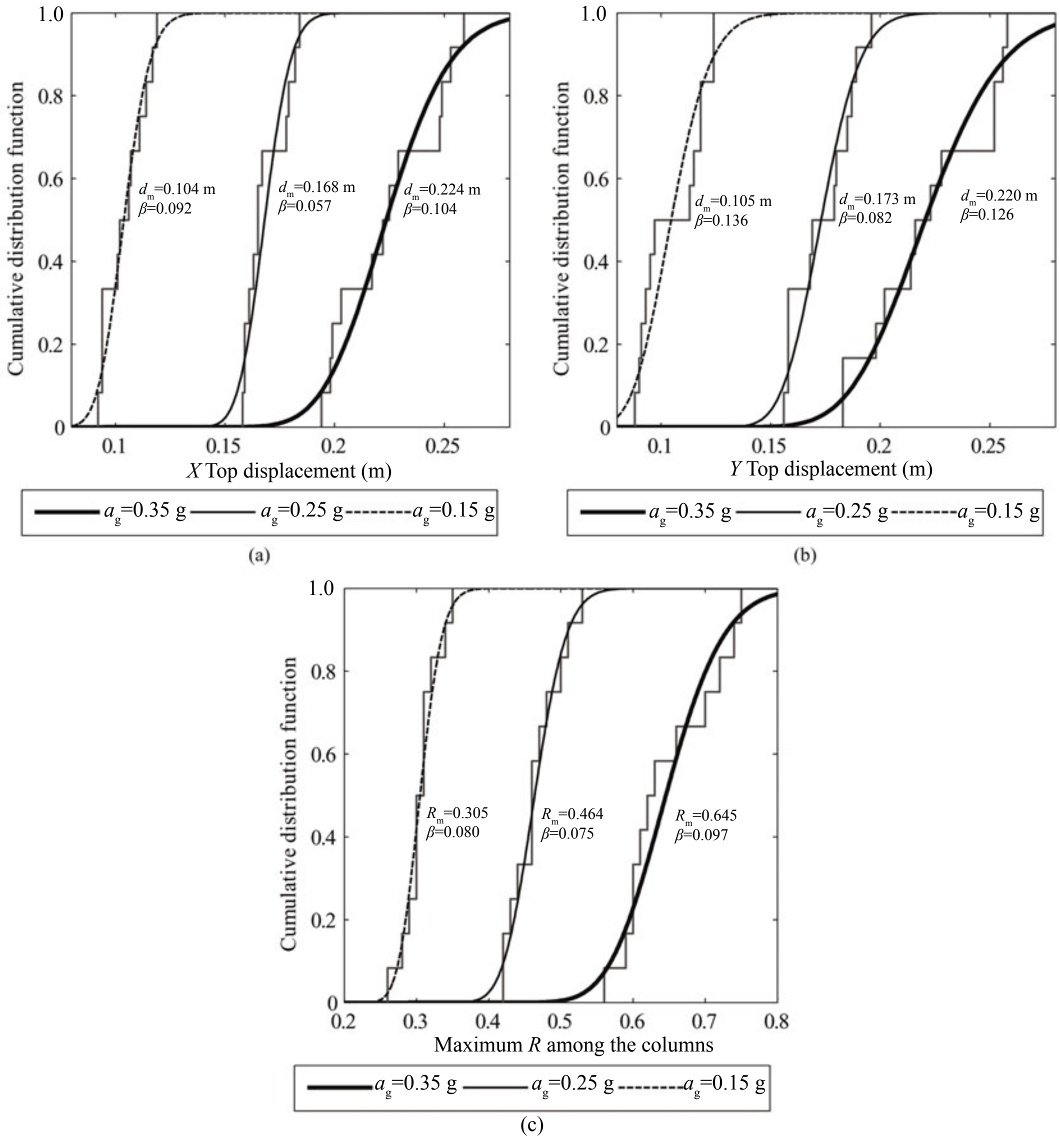


Fig. 10 Empirical cumulative distribution function of the (a) *X* and (b) *Y* top displacements and (c) the maximum ratio *R* among the different columns, fitted with a lognormal distribution, defined upon the median value of the demand parameter (i.e., d_m and R_m) and the standard deviation β

(3) The uncertainty in the seismic demand can be represented by the coefficient of variation, i.e., the ratio between the standard deviation and the mean demand value. The coefficient of variation of the required top displacement ranges from 0.06 and 0.13, while the coefficient of variation of the maximum plastic hinge rotation demand ranges from 0.07 to 0.10, confirming the non-negligible variation of the seismic demand due to the incidence angle of the seismic input.

(4) An attempt to quantify the uncertainty related to the variability in the incidence angle is also provided. In particular, in Fig. 10 the empirical cumulative distribution functions of the three considered demand parameters are shown for the different intensity levels. The empirical distributions are fitted with a lognormal distribution, defined upon the median value of the demand parameter (i.e., d_m and R_m) and the standard deviation β, in order to numerically assess the uncertainty. Indeed, the slope of the lognormal distribution is measured by β, and reflects the variability (uncertainty) in the results (FEMA P695, 2009).

The uncertainty due to the incidence angle variability for three selected engineering demand parameters is in the range of 0.057 – 0.136. It can be compared to the β values considered by FEMA P695 (2009), i.e., the (1) record-to-record uncertainty, (2) design requirements-related uncertainty, (3) test data-related uncertainty and (4) modelling uncertainty. The uncertainty due to the seismic incidence angle is not negligible with respect to the values considered by FEMA P695, which considers β ranging from 0.1 to 0.4 for the uncertainties (2), (3) and (4).

4 Conclusions

In this study, nonlinear methods of analysis (static and dynamic), according to Eurocode 8, are compared by considering three different reinforced concrete buildings that are representative of typical RC buildings in Italy: a rectangular plan shape, an L plan shape and a rectangular plan shape with a courtyard building.

By comparing the assessments performed with pushover and dynamic analyses, it is found that the static nonlinear analysis is more conservative, i.e., it yields smaller safety factors with respect to nonlinear dynamic analysis for high intensity seismic levels. This conclusion is inverted, i.e., dynamic nonlinear analysis yields lower safety factors for low intensity seismic levels.

Building No. 2 is irregular in plan according to the EC8 rules. The influence of the incidence angle on the demand is evaluated in order to better explore the seismic performance of this structure, which exhibits an unusual dynamic behavior. Twelve different earthquake directions are considered, rotating the direction of both the orthogonal components of the selected accelerograms by 30 degrees for each analysis (from 0° to 330°).

The results show that the incidence angle of the seismic input motion significantly influences the response of RC structures; the critical seismic angle, i.e., the incidence angle that produces the maximum demand, provides an increase in the demand larger than 15% in both displacements and in the ratio R between the required rotation and the capacity. The underestimation of code compliant dynamic analyses could be up to 37% if the incidence angle variation is not taken into account. If the rotations in plastic hinges are considered, the discrepancy due to the earthquake incidence angle may be up to 30%; for this reason, such a critical angle should be taken into account in the seismic structural analyses. The critical incidence angle depends on both the engineering demand parameter, e.g. displacements or plastic hinge rotations, and on the intensity of the seismic input. This issue clearly denotes the large uncertainty related to the definition of a critical incidence angle. The coefficient of variation of the required top displacement ranges from 0.06 and 0.13, while the coefficient of variation of the ratio R between the required rotation and the capacity ranges from 0.07 to 0.10.

This study has proven the significant influence of the seismic incidence angle on plan-asymmetric structures. Such an influence is not currently included in the building codes; the seismic demand can be significantly underestimated even when "refined" dynamic nonlinear code-compliant analyses are used. A wide parametric study is required in order to include the influence of the seismic incidence angle in the current building codes.

An attempt to quantify the uncertainty related to the variability in the incidence angle is also provided. In particular, assuming that the seismic demand is lognormally distributed, the uncertainty is quantitatively defined as the lognormal standard deviation β. The uncertainty due to the incidence angle variability for three selected engineering demand parameters is in the range of 0.057 – 0.136, and is not negligible with respect to the values considered by FEMA P695.

Note that the conclusion is related and limited to a single plan asymmetric case study building; a wide parametric study is needed in order to quantify the uncertainty related to the variability in the incidence angle and to achieve more general results. For example, a larger sample of buildings having different material strengths and plan-wise configurations should be considered.

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