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STATIC AND DYNAMIC NON LINEAR ANALYSIS OF PLAN IRREGULAR EXISTING R/C FRAME BUILDINGS

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ABSTRACT:

The paper deals with the topic of seismic response of three existing structures. They are multi-storey r/c frame buildings, a very spread typology in Italy; in particular, the first has a rectangular plane shape, the second has a L plane shape and the third has a rectangular plane shape with courtyard. The second and the third are irregular in plane according to the rules proposed by the Italian seismic code OPCM n.3431 and by the Eurocode 8 (EC8). Nonlinear static (pushover) and dynamic analyses are performed considering three different seismic zones, with PGA (peak ground acceleration) equal to 0.35 g, 0.25 g and 0.15 g. These are performed using sets of seven earthquakes (each with both the horizontal components), fully satisfying the EC8 provisions. Twelve different earthquake directions are considered, rotating the direction of both the orthogonal components by 30° for each analysis (from 0° to 330°). The analyses have been performed at Significant Damage Limit State, with earthquakes fitting the EC8 elastic spectrum characterised by a return period of 475 years; furthermore, the Near Collapse Limit State is also considered, amplifying the previous earthquakes by a factor equal to 1.5. The results of non linear static analyses are compared to the ones obtained by nonlinear time-history analyses, in terms of demanded / available rotation ratio at the top and at the bottom of each column in the two directions and in terms of maximum frame top displacements.

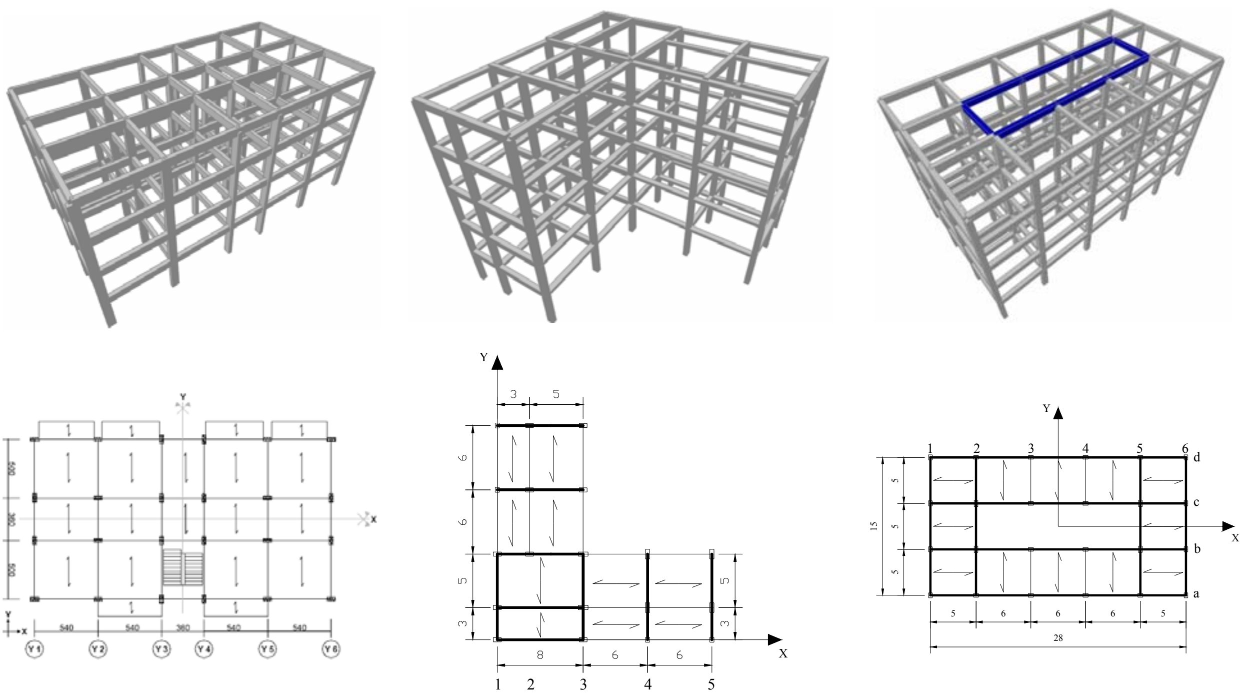
KEYWORDS: Plan irregularity, existing buildings, non linear analyses, reinforced concrete buildings, Italian seismic code.

1. INTRODUCTION

The modern seismic codes, as EC8 (CEN, 2003) and the Italian one (OPCM n.3431, 2005), allow to use different analysis methodologies, in particular: 1) lateral force and 2) multi-modal elastic ones and 3) static and 4) dynamic non linear ones. Their level of reliability decreases from n.4) to n.1) and, consequently, the safety margin with respect to the same limit state should increase according to the same order.
The choice of a method depends from the characteristics (regularity, fundamental periods) and the importance of the structure. Certainly, among the different approaches, non linear dynamic analysis is able to provide the best prediction of the structural response.
In the paper the seismic response of three existing structures is analysed. They are multi-storey r/c frame buildings, a very spread typology in Italy; in particular, the first has a rectangular plane shape, the second has a L plane shape and the third has a rectangular plane shape with courtyard.
The structures are analyzed with non linear static (pushover) and dynamic analyses varying a certain number of factors. According to the OPCM n.3431 seismic hazard, three different seismic zones with PGA (peak ground acceleration) equal to 0.35 g, 0.25 g and 0.15 g have been considered.
In particular, for non linear static analyses 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, are considered.
For non linear dynamic analyses twelve different earthquake directions are considered, rotating the direction of both the orthogonal components by 30° for each analysis (from 0° to 330°). These have been performed using sets of seven earthquakes (each one with both the horizontal components), fully satisfying the EC8 provisions. Records are natural accelerograms, selected from European Strong-motion Database (ESD) so that their average elastic spectrum does not underestimate the code’s spectrum, with a 10% tolerance, in a range of periods depending on the structure’s dynamic properties.
The analyses are performed at Significant Damage Limit State, with earthquakes fitting the OPCM n.3431 elastic spectrum characterised by a return period of 475 years, corresponding to a probability of exceedance of 10% in 50 years; furthermore, the Near Collapse Limit State is also considered, amplifying the previous earthquakes by a factor equal to 1.5. The results of non linear static analyses are compared to the ones obtained by nonlinear time-history analyses, in terms of demanded / available rotation ratio at the top and at the bottom of each column in the two directions and in terms of maximum frame top displacements.

2. GEOMETRY OF THE BUILDING AND MODELLING

The geometry of the r/c frame buildings is reported in Figure 1. The building n.1 is a doubly symmetrical four-storey building with the interstorey height equal to 3.20 m 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 f_{cm}=33.5 N/mm^2 and steel average strength equal to f_{ym}=500 N/mm^2 are adopted.
The building n.2 is an L plane shape five-storey building with an interstorey height equal to 3.50 m for each level. The dimensions of the columns sections remain constant for the whole height of the building and are represented by three different typologies: 40×40, 40×50, 40×70. Also the beams have different section dimensions: 40×60, 40×50, 25×50. About the materials, concrete average cubic strength equal to f_{cm}=25 N/mm^2 and steel average strength equal to f_{ym}=400 N/mm^2 are adopted.
The building n.3 has a rectangular shape and a central courtyard; it is a five-storey building, with the interstorey height equal to 3.50 m for each level. The columns have two different typologies of section dimensions maintained constant for all the storeys: 40×55 and 40×70. Also the beams have different section dimensions assembled in three typologies: 40×60, 40×50, 25×40. As for building n.2, concrete average cubic strength equal to f_{cm}=25 N/mm^2 and steel average strength equal to f_{ym}=400 N/mm^2 are adopted.
Non linear analyses are performed by means of two computer programs, SAP2000 (CSI, 2004) and CANNY99 (Li, 1996). Non linearity regards flexural rotations, while all the other deformations are assumed linear. Both beams and columns are characterised by lumped plasticity models; in the latter case for each section two independent non linear springs are assigned, one for each orthogonal direction. No axial force-bending moment interaction is considered at the plastic hinge.



Building n.1 Building n.2 Building n.3

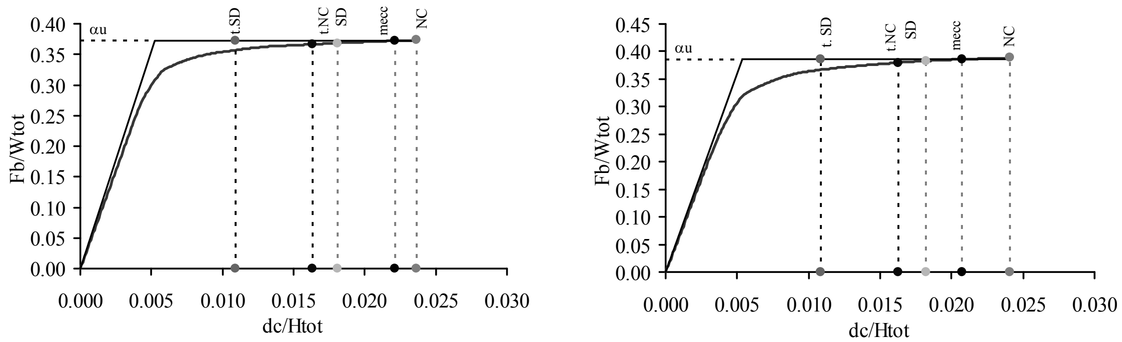
Figure 1 3D view and floor plane of the analyzed buildings

Bending moment springs are characterised by bi-linear skeleton curve, defined by yielding moment and corresponding rotation; the cracking is also taken into account reducing by 50% the Young modulus. The moments and the 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.
The yielding and the ultimate rotations are evaluated as provided by OPCM n.3431 equations (11.1.a) and (11.A.1) respectively, where the already cited medium values are assigned to concrete (f_c) and steel (f_y) strength. The hysteretic model is Takeda type; the pinching effect is also taken into account.

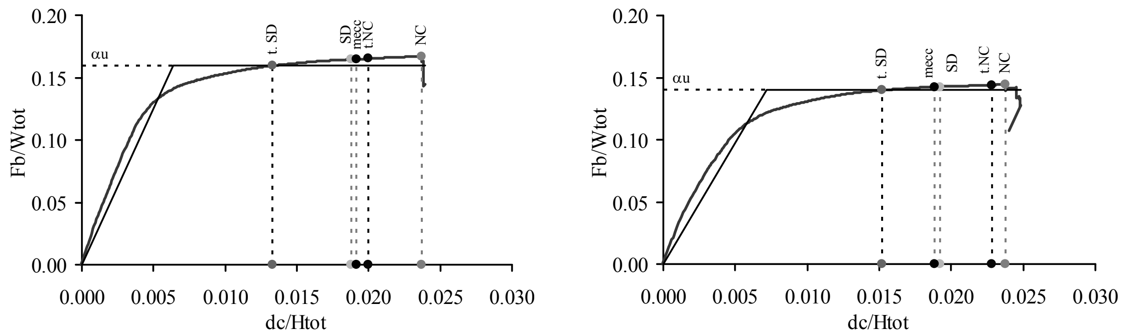
3. NON LINEAR STATIC ANALYSES

For each building, 24 pushover analyses are performed; in particular ,for each building, 2 opposite signs for 4 different positions of the centre of mass are considered.
The OPCM n.3431, as EC8, provides non linear static analysis according to 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 analysis is repeated considering the three different seismic zones provided by the OPCM n.3431, i.e. the demand is represented by three elastic spectra obtained considering three values of PGA: 0.15 g, 0.25 g and 0.35 g.
In Figure 2, only two pushover curves (in terms of normalised top displacement vs base shear) for each building are shown. They refer to the ones obtained applying the modal pattern; the demand, evaluated for a_g=0.35 g, is also reported. The N2 method bi-linear capacity curve is presented along with capacity and demand points: “mecc” indicates the mechanism of the structure (and α_u the corresponding strength), “SD” (Significant Damage Limit State) and “NC” (Near Collapse Limit State) the attainment in at least one hinge of the rotation value 3/4 Θ_u and Θ_u respectively; “t.SD” and “t. NC” the demand corresponding to the OPCM n.3431 elastic design acceleration spectrum and 1.5 times the elastic design acceleration spectrum respectively; Θ_u is the total chord rotation capacity computed according to OPCM n.3431 empirical formula (11.A.1).

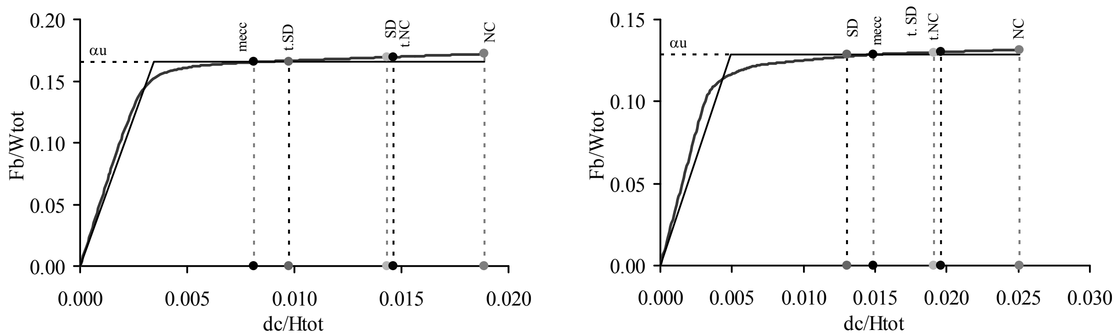
Figure 2 shows that the structure is not always verified. In the case of building 3 direction Y, the demand (t.SD) is larger than the capacity (SD).



Building n.1



Building n.2



Building n.3

Figure 2 Buildings top displacement-base shear capacity curves (forces applied in direction X (left) and Y (right))

4. NON LINEAR DYNAMIC ANALYSES

Both the horizontal components of a set of 7 earthquakes, i.e. 14 natural records, are used for non linear dynamic analyses whose results are shown herein; according to the selection procedure presented in (Iervolino et al., 2008), they satisfy the OPCM n.3431 provisions: in the range of periods between 0.15 T_1 and 2 T_1, where T_1 is the fundamental period of the structure, no value of the mean 5% damping elastic spectrum calculated from all time histories should be less than 90% of the corresponding value of the 5% damping elastic response spectrum; if the response is obtained from at least 7 non linear time history analyses, the average of response quantities should be used as the design value of the action effect E_d in relevant verifications.

The used accelerograms are downloaded by site www.reluis.it. An incremental non linear dynamic analysis (IDA) is performed, considering different hazard levels according to OPCM n.3431: a_g=0.15 g (zone 3), a_g=0.25 g (zone 2), a_g=0.35 g (zone 1).
The analyses are performed applying, for each earthquake, the two horizontal components simultaneously and considering top centre of mass displacements and rotational ductility demand evaluated at beam and column ends as response parameters. Such demand is compared to capacity at the Significant Damage Limit State, which corresponds to the attainment at elements ends of 3/4 Θ_u as already written.
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.
Only for the building n.2, twelve different earthquake directions are considered, rotating the direction of both the orthogonal components by 30° for each analysis (from 0° to 330°).
The results in terms of displacements in the two orthogonal directions X and Y and as vectorial displacements, considering the set of accelerograms satisfying OPCM n.3431 provisions for the seismic zone 1 (a_g=0.35 g), are shown in Figure 3 (left).

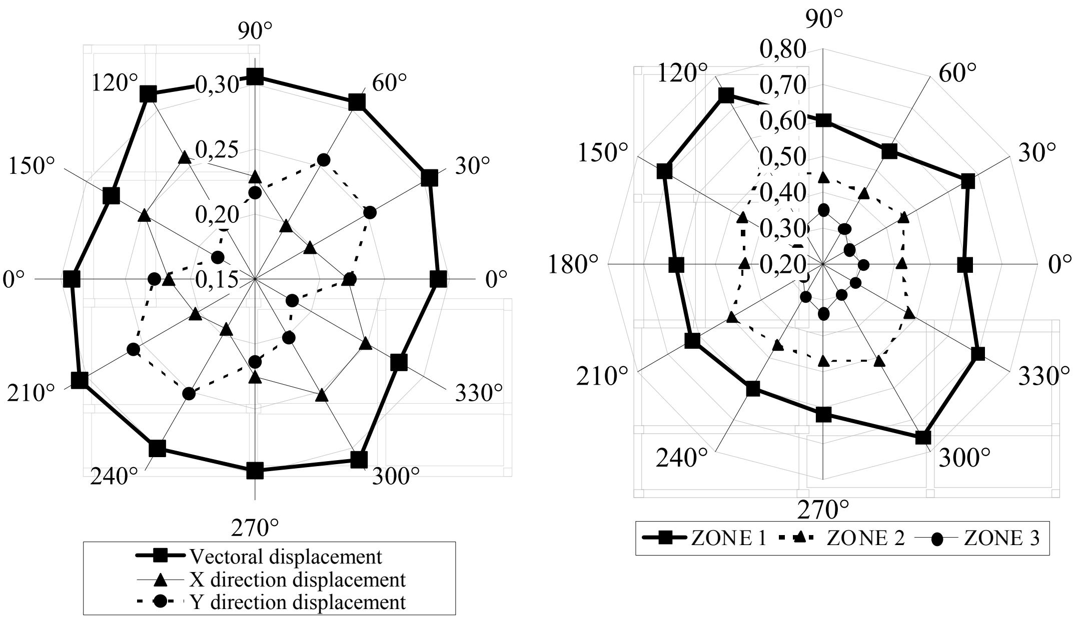


Figure 3 Influence of the seismic input angle on the response: displacements (left) and demanded / available rotation ratios (right).

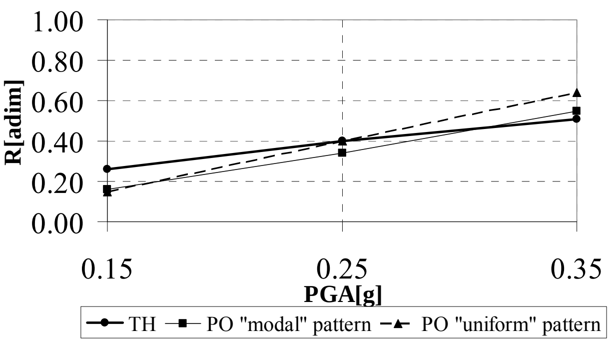
In the same Figure 3 (right) the influence of the seismic input angle in terms of demanded / available rotation ratio at the top and at the bottom of each column in the two directions for the three hazard seismic zones is also shown. In Table 1, the numerical values plotted in Figure 3 and its variations in comparison to the seismic response obtained applying the input without any rotation are shown.

5. COMPARISONS OF THE RESULTS

The results of non linear static analyses are compared with the ones obtained by non linear dynamic analyses. In particular, the demanded / available rotation ratios (R) at the top and at the bottom of each column in the two directions for the three hazard seismic levels are compared; such comparison is performed considering the maximum ratios among all the columns of the building, as well as the average of the values at each level. Such comparisons are performed both at the Significant Damage Limit State and at the Near Collapse Limit State. In Figure 4, for sake of brevity, only the maximum demanded / available rotation ratios at Significant Damage Limit State are shown; for the non linear static analysis both the two different distributions of horizontal forces ("uniform" and "modal" pattern) are considered.

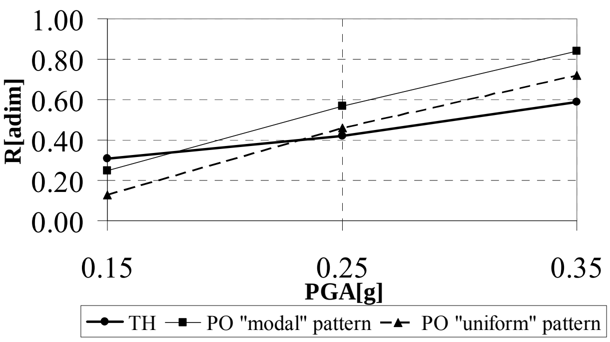
Table 1 Numerical values of the displacements and demanded / available rotation ratios plotted in Figure 3

	Displacement						Rotational ductility					
	Zone 1						Zone 1		Zone 2		Zone 3	
	Δ_{vect}	var.	Δ_x	var.	Δ_y	var.	θ_d/θ_c	var.	θ_d/θ_c	var.	θ_d/θ_c	var.
	(m)	%	(m)	%	(m)	%	(-)	%	(-)	%	(-)	%
0°	0.29		0.22		0.22		0.59		0.42		0.31	
30°	0.31	4.9	0.20	-10.2	0.25	13.0	0.66	12.2	0.46	10.1	0.28	-9.0
60°	0.31	5.4	0.20	-10.8	0.26	14.5	0.56	-4.6	0.43	2.5	0.32	2.2
90°	0.31	5.0	0.23	3.3	0.22	-3.1	0.60	1.5	0.44	5.3	0.35	14.0
120°	0.31	7.9	0.26	16.7	0.20	-11.2	0.74	26.0	0.53	25.4	0.31	1.5
150°	0.28	-4.6	0.25	12.2	0.18	-17.9	0.72	21.0	0.46	10.2	0.29	-7.5
180°	0.29	0.0	0.22	-2.2	0.23	2.0	0.61	3.5	0.42	0.1	0.31	0.3
210°	0.31	5.1	0.20	-8.3	0.26	15.7	0.63	5.9	0.50	18.4	0.26	-14.1
240°	0.30	3.2	0.19	-12.3	0.25	12.8	0.60	1.2	0.46	9.3	0.30	-2.2
270°	0.30	2.1	0.23	1.6	0.21	-4.3	0.62	4.4	0.47	12.5	0.34	9.4
300°	0.31	6.6	0.25	14.0	0.20	-9.5	0.75	27.6	0.51	22.2	0.30	-4.0
330°	0.28	-4.5	0.25	12.0	0.18	-17.9	0.70	17.6	0.48	14.0	0.30	-2.5
MAX	0.31	7.9	0.26	16.7	0.26	15.7	0.75	27.6	0.53	25.4	0.35	14.0



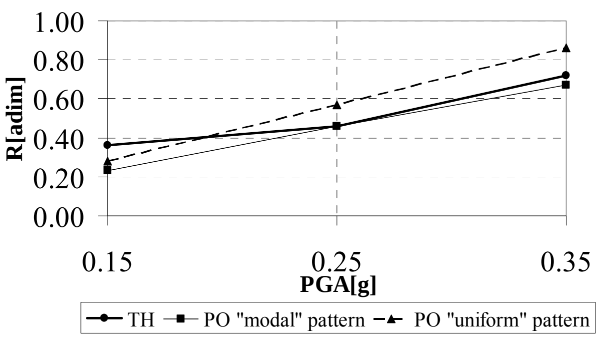
PGA (g)	Time History	Pushover "modal" pattern	Pushover "uniform" pattern
0.15	0.26	0.16	0.15
0.25	0.40	0.34	0.40
0.35	0.51	0.55	0.64

Building n.1



PGA (g)	Time History	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 n.2



PGA (g)	Time History	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

Building n.3

Figure 4 Maximum values of demanded / available rotation ratio at Significant Damage Limit State

Figure 4 shows that, for the analysed existing buildings, the following principle, generally valid for the buildings of new design (Magliulo et al., 2007), is not always respected: the safety level associated to different analyses performed according to OPCM n.3431 is correlated to their accuracy, i.e. the safety verification performed according to static non linear analysis is more conservative with respect to the one performed according to non linear dynamic analysis, which is the most accurate.
Indeed, as shown by Figure 4, non linear dynamic analysis provides some results more conservative than those provided by non linear static analysis, mainly as the seismic hazard level decreases.
In the Figure 5, for each floor, the average of the columns maximum demanded/available rotation ratios are shown for building n.1 at the Near Collapse Limit State. For each column, the maximum demanded/available rotation ratio is computed as maximum considering both the X and Y orthogonal directions and both the column ends.
The results in Figure 5 show, as expected, that the pushover analysis performed by the uniform force pattern provides at the bottom storeys a demand larger than the one provided by the modal force pattern; the contrary happens at the top floors.

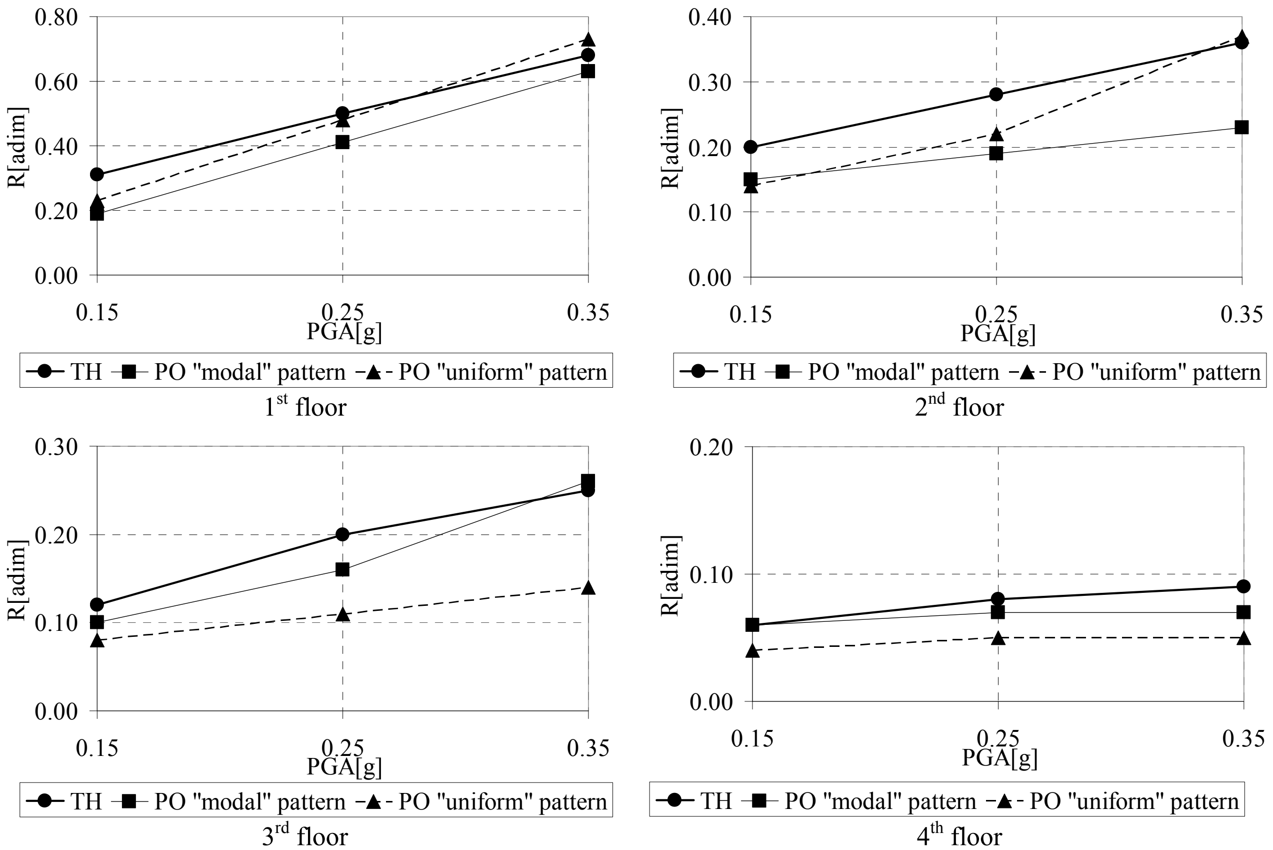


Figure 5 Average values of demanded / available rotation ratios for each floor at Near Collapse Limit State (Building n.1)

6. CONCLUSION

In the paper non linear methods of analysis (static and dynamic), according OPCM n.3431, are performed considering three different typologies of reinforced concrete buildings, selected as very spread typologies in Italy. In particular, the first has a rectangular plane shape, the second has an L plane shape and the third has a rectangular plane shape with courtyard.
The second and the third building are irregular in plane according to the rules proposed by the Italian seismic code OPCM n.3431 and by the Eurocode 8.
The accelerograms used for the non linear dynamic analyses are downloaded from the web site www.reluis.it; the sets of 7 natural earthquakes satisfy the OPCM n.3431 provisions.

Pushover analyses have been carried out by “SAP 2000” computer program, while non linear dynamic ones by CANNY99. Three different seismic zones have been considered, with PGA (peak ground acceleration) equal to 0.35 g, 0.25 g and 0.15 g. In particular, for non linear static analysis 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, are considered.
The analyses have been performed at Significant Damage Limit State and at Near Collapse Limit State. The results of non linear static analyses are compared to the ones obtained by nonlinear time-history analyses, in terms of demanded / available rotation ratio evaluated at the top and at the bottom of each column in the two directions and in terms of the maximum frame top displacements.
Only in the case of building n.2, non linear dynamic analyses are performed considering twelve different earthquake directions, obtained by rotating the direction of both the orthogonal components by 30° for each analysis (from 0° to 330°).
From non linear static analyses results it is possible to deduce that, generally, as expected, pushover analysis performed by the uniform force pattern provides at the bottom storeys a demand larger than the one provided by the modal force pattern; the contrary happens at the top storeys.
Furthermore the performed non linear analyses show that, for the analysed existing buildings, the following principle, generally valid for the buildings of new design, is not always respected: the safety level associated to different analyses performed according to OPCM n.3431 is correlated to their accuracy, i.e. the safety verification performed according to static non linear analysis is more conservative with respect to the one performed according to non linear dynamic analysis, which is the most accurate. Indeed, non linear time-history analysis provides some results more conservative than those provided by non linear static analysis, mainly as the seismic hazard level decreases.
Finally, time-history analyses on building n.2 show that a *critical angle of seismic incidence*, i.e. the angle of application of the records providing the maximum structural response, exists and the increment of response parameters is not negligible; consequently, such critical angle should be taken into account in the seismic structural analyses.

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