Structures 7 (2016) 1–13

Contents lists available at ScienceDirect

Structures

journal homepage: www.elsevier.com/locate/structures

Seismic behavior of RC building structures designed according to current codes

A. Mosleh [a], H. Rodrigues [b,*], H. Varum [c], A. Costa [a], A. Arêde [c]

[a] RISCO, Civil Engineering Department, University of Aveiro, Aveiro, Portugal
[b] RISCO, School of Technology and Management, Polytechnic Institute of Leiria, Portugal
[c] CONSTRUCT-LESE, Faculty of Engineering (FEUP), University of Porto, Porto, Portugal

ARTICLE INFO

Article history:
Received 9 September 2015
Received in revised form 5 February 2016
Accepted 6 April 2016
Available online xxxx

Keywords:
Reinforced concrete buildings
Seismic vulnerability
Push-over analysis
Non-linear dynamic analysis
Codes

ABSTRACT

Earthquakes which recently occurred in highly populated regions show that existing buildings constructed without appropriate seismic resisting characteristics may constitute as an important source of risk and may cause economical loses and casualties. It is recognized the progress of the knowledge in earthquake engineering in the last decades. In this paper, two 6 irregular storey buildings were studied consisting of frame structures, representative of the common practice in Portugal, i.e. designed without considering earthquake actions. Push-over and non-linear time history analyses were done, with non-linear 3-D models in longitudinal and transverse directions. The building responses were analyzed in two different levels: global and local. For the global response analyses: max displacement, inter-storey drift (IS drift), floor rotation for each storey and base shear were compared. For local response four columns were chosen and the variation of axial load in terms of base shear and drift as well as the biaxial demand was considered. The result shows that most variation of axial load happens in corner, facade-X, facade-Y and centre column respectively. It is noteworthy that by increasing the initial axial load the biaxial demand decreases. The seismic vulnerability was analyzed for earthquake of different return periods, and the seismic demands were compared with limit proposed in international codes and conclusion are drafted in terms of safety. The vulnerability assessment based on seismic codes clearly shows that the building 2 presented a better performance with low inter-storey drifts. The main goal of this study is considering the application and methodology for the seismic assessment of existent real buildings. In fact this is an important topic, to understand the seismic vulnerability of certain particularities in existing buildings to assure that the common observation can be applied for a prototype building, especially irregular ones. Also one of the major observations in this study is the comprehension of the effect and importance of biaxial loading in columns and the influence of the axial load variation, relating the position of the columns in plan and in height.

1. Introduction

Strong ground motions in the past decade in the densely populated area made great impacts on many buildings specially those designed according to older codes, and revealed that these structures are seismically vulnerable. Several devastating earthquakes, particularly the 1989 Loma Prieta and the 1994 Northridge earthquakes in California, the 1995 Kobe earthquake in Japan, the 2009 L'Aquila and the 2012 Emilia Romagna in Italy, and the 2011 Lorca earthquake in Spain have caused significant damage on the buildings. There are some reasons that show why the structures are practically vulnerable during past earthquakes such as: inadequacy of previous seismic codes and guidelines [1], low standards of construction due to inattention to local detailing [2] and quality control with high variation in material properties [3]. The capacity of the columns is one of the important factors to evaluate the seismic performance of reinforced concrete (RC) buildings. Recent investigation shows that the response of RC members subjected to axial loads combined with biaxial bending moment is recognized as a research topic among researchers for buildings [4]. To achieve this goal, non-linear analyses could be used to evaluate the safety of a structure designed according to the existing design codes. Previous researches have illustrated the trend of seismic performance of reinforce concrete (RC) buildings. Kim and Kim [5] evaluated the seismic demand of reinforce concrete special moment-resisting frame according to IBC 2003. The performance of RC building according to Eurocode 8 was investigated by Panagiotakos and Fardis [6]. Chaulagain et al. [7] conducted a numerical investigation on the seismic performance of four-storey RC buildings. Rodrigues et al. proposed an experimental and numerical simulation to represent the non-linear response of reinforced concrete members due to biaxial bending combined with a constant axial load [8–10]. Varum et al. [11] evaluated numerical tools for the assessment and redesign of concrete buildings capable of estimating the optimum distribution of strengthening needs for a specific performance objective.

* Corresponding author.
E-mail address: hugo.fp.rodrigues@ipleiria.pt (H. Rodrigues).

http://dx.doi.org/10.1016/j.istruc.2016.04.001

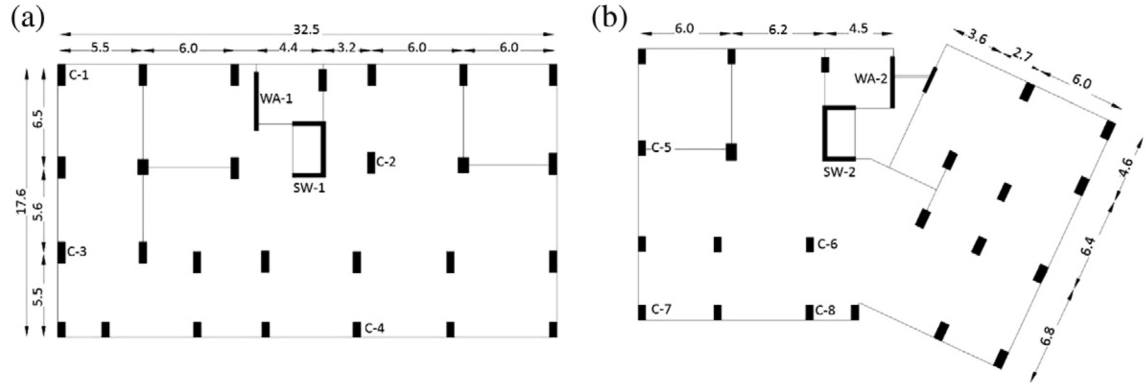


Fig. 1. Geometry of building structure. (a): building 1, (b): building 2.

Table 1
Dimension of column cross-section (dimensions are in cm).

Storey	C-1	C-2	C-3	C-4	C-5	C-6	C-7	C-8
1st	30 × 60	30 × 80	25 × 80	30 × 60	25 × 80	30 × 80	60 × 30	30 × 65
	14 Φ 20	16 Φ 18	18 Φ 20	12 Φ 20	16 Φ 22	14 Φ 20	12 Φ 20	12 Φ 18
2nd	25 × 60	25 × 60	25 × 60	25 × 60	25 × 60	25 × 70	25 × 50	25 × 60
	12 Φ 18	12 Φ 18	12 Φ 18	12 Φ 18	12 Φ 18	14 Φ 18	12 Φ 18	12 Φ 18
4th	25 × 50	25 × 60	25 × 60	25 × 50	25 × 60	25 × 70	25 × 50	25 × 50
	12 Φ 16	12 Φ 16	12 Φ 16	12 Φ 16	12 Φ 16	14 Φ 16	12 Φ 16	12 Φ 16

Kueht and Hueste [12] evaluated a numerical modeling on the seismic performance of a four-storey RC frame designed by the 2003 International Building Code (IBC). Kotronis et al. [13] proposed a strategy to simulate the non-linear behavior of two RC wall specimens designed according to the French code PS92 and the Eurocode 8, respectively. A constitutive model for predicting the cyclic response of RC structures using a smeared crack approach with orthogonal fixed cracks was studied by Ile and Reynouard [14]. Mazza [15,16] conducted a numerical investigation and structural testing to evaluate the seismic vulnerability and retrofitting of the town hall of Spilinga with an L-shape plan built in 1960. The structural safety assessment procedures based on Eurocode 8 in RC structures are proposed in the study of Romao et al. [17]. There have been several investigations on the seismic performance of RC frames in other countries. The research interest in the 3-D earthquake actions in building irregularities subjected to biaxial bending combined with axial force in the columns is well recognized. The effects of the biaxial loading and its importance in the column response, in terms of the strength degradation and reduction of the ductility capacity are proposed by previous researchers [18]; nevertheless, further studies have to be addressed. For simulation of the biaxial cyclic behavior of RC members with axial load, several modeling processes are proposed, however it is obvious that the available biaxial models are not developed enough to be utilized in practice.

In this research two existing irregular RC buildings which are designed with the previous codes of Eurocode 8 are selected and proposed for non-linear analyses. The main objective of this research is focused on the performance of the existing building in two different levels: global and local. The building responses are analyzed in terms of max

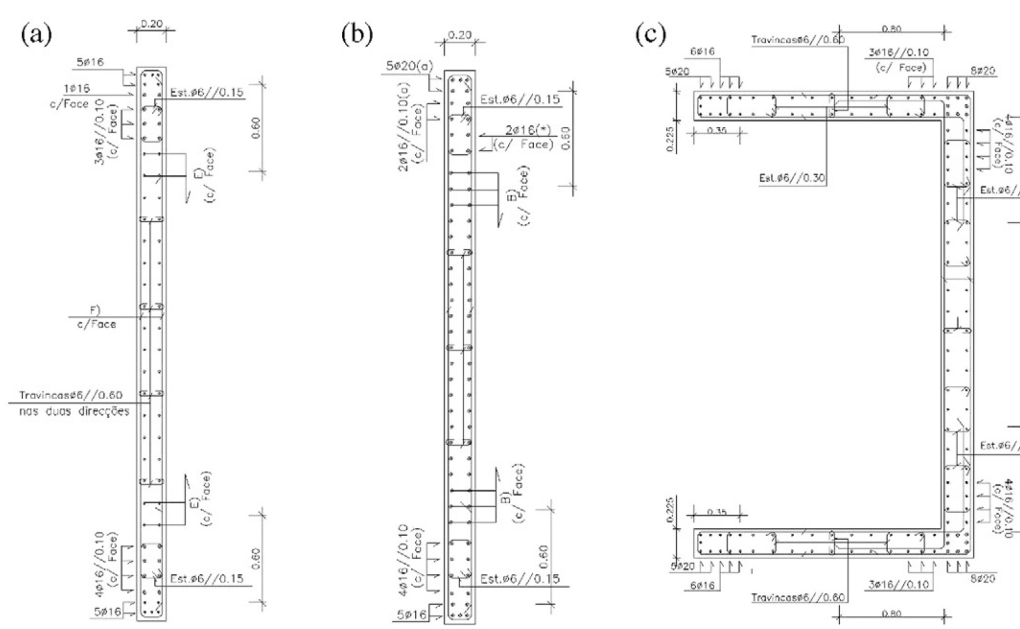


Fig. 2. Shear walls details: (a): WA-1, (b): WA-2, (c): SW-1.

Table 2
Periods and frequency for building 1 and building 2.

Mode	Building 1		Building 2	
	Time (s)	Frequency (Hz)	Time (s)	Frequency (Hz)
1st mode(X direction)	1.62	0.61	1.68	0.59
2nd mode (rotation)	1.21	0.82	1.2	0.83
3rd mode (Y direction)	1.04	0.96	1.02	0.98

displacement, inter-storey drift, and floor rotation for each storey as well as base shear due to global response. However, for local response four columns are selected as representative of corner, center and facade columns. Subsequently, the variation of axial load in terms of base shear and drift as well as the biaxial demand was considered. Finally the seismic vulnerability was analyzed for earthquake of different return periods, and the seismic demands were compared with limit proposed in international codes and conclusions are drafted in terms of safety.

2. Case study

2.1. Building description

Two six-storey irregular RC buildings were considered in this study. Building 1 has six bays in longitudinal direction (32.5 m) and three bays in transverse direction (17.6 m). Building 2 has five bays in longitudinal direction and three bays in transverse direction (17.6 m). Geometry of building structures is presented in Fig. 1. The buildings consist of 6 floors high and 3 underground floors. The total building height is 20.8 m from ground, with the first storey height of 5.4 m, 3.06 m storey height for the middle stories and 3.16 m for the upper storey. For two buildings, floors −3, −2 and −1 are considered as a parking, which are surrounded by shear walls, therefore the structure is modeled from the upper stories. The first storey is designed for the commercial and stories 1 to 5 are residential occupancy. The cross-section of largest column which is located in the first storey is $(0.8 \times 0.4)\ m^2$ with 18 Φ 25. The smallest column is located in the 6th storey with the damnation $(0.5 \times 0.25)\ m^2$ with 12 Φ 16. It should be noticed that due to the diversity in column number and sections in 6 floors, existence of all columns, beams and shear walls is unpractical. Therefore column sections and longitudinal reinforcement for columns which are selected for local response are presented in Table 1. The cross-section for shear walls is presented in Fig. 2 as well.

2.2. Material properties

The building is design to be located in an urban area in the center of Portugal. The design of the structural elements was carried out using the methods and calculation rules advised by the national codes and other applicable technical documents, including: Safety Regulations and Actions for Buildings and Structures Bridges, Regulation of Structures Concrete and Pre-stressed, Regulation of Steel Structures for Buildings, Eurocode 2 [19], Reinforced Concrete — Efforts normal and bending (REBAP-83)-LNEC [20], CEB-FIP Model Code 1990 [21]. The wind action considered that the building was in zone B, and that the surface roughness was type I. The structure corresponding to the management area to seismic action was considered a type III ground, coefficient performance $\eta = 2.5$ and seismicity coefficient $\alpha = 0.5$, corresponding to zone C, in accordance with the Portuguese National code for load (RSA, 1983).

The material properties assumed for the construction are the following: concrete compressive strength, $f_c = 30$ MP and reinforcing steel yield strength, $f_s = 400$ MPa. Note that the materials provided for the construction are as follows: Concrete C35/45 XC4/XS3 (EN 206-1) in foundations, bottom slab and peripheral containment; concrete C30/37/XC4 (EN 206-1) in the other structural elements and steels: A400NR Steel. For the first storey: live load = 4 kN/m^2 (30% for earthquake), dead load = 1.5 kN/m^2, for 1–5 stories: live load = 2 kN/m^2 (30% for earthquake), dead load = 2.5 kN/m^2, roof live load = 0.7 kN/m^2 (Nil for earthquake), roof dead load = 1.5 kN/m^2.

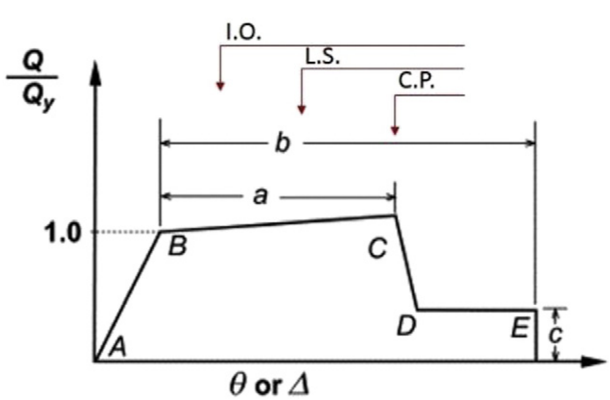


Fig. 3. Schematic diagram of force-deformation in concrete elements.

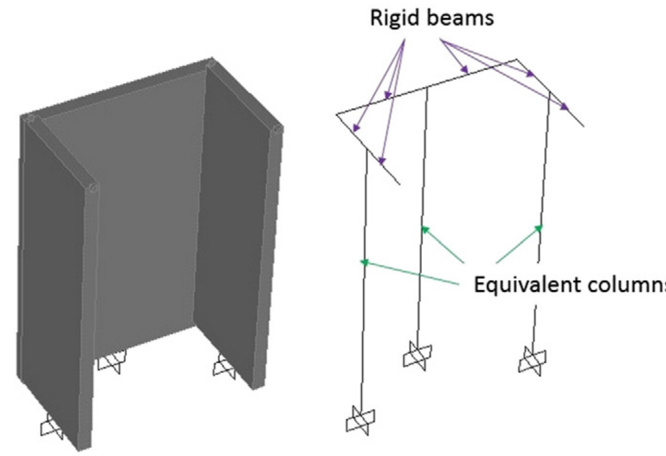


Fig. 4. Equivalent frame model considered and description for shear wall modeling.

3. Structural analysis

In order to assess the seismic capacity of the structures considered, several non-linear analyses were performed. In a first study it was developed a push-over analysis for directions of each building. In the second study and to complement this analysis a series of non-linear dynamic time history analysis with different earthquake records was also performed. In the following section the assumption considered for the analysis is briefly described. Time period and frequency corresponding to modal analysis are presented in Table 2. As can be seen in Table 2, first and third modes are in longitudinal (X) and transverse (Y) directions respectively, however second mode is corresponding to rotation.

Damage assessments due to past earthquakes indicate that irregular plan-form buildings have more drastic damage related to torsional response than regular buildings [22–23]. Eccentricity is one of the critical parameters as for plan irregularity. For these two buildings, since the center of mass does not coincide with the center of stiffness, the torsional forces cause the structure to rotate around the center of stiffness. This indicates that both buildings are irregular. Although building 1 is regular in plan, both of the buildings show irregularities due to the position of the mass and stiffness centers.

3.1. Non-linear modeling

The finite element (FE) structural analysis program SAP2000 [24] was used to develop the non-linear analyses in two directions: longitudinal

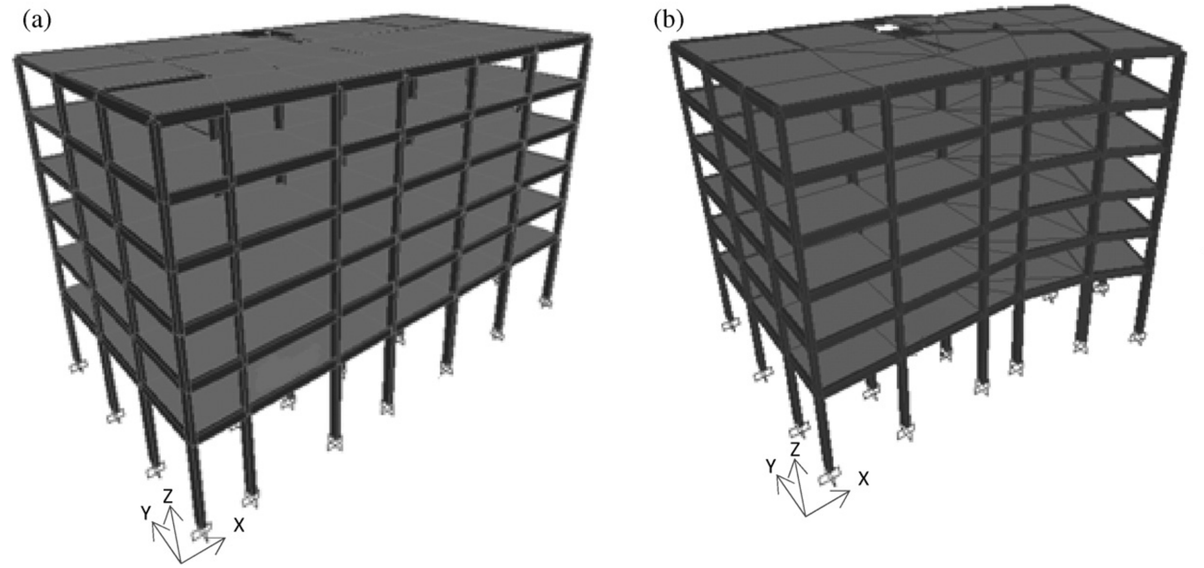
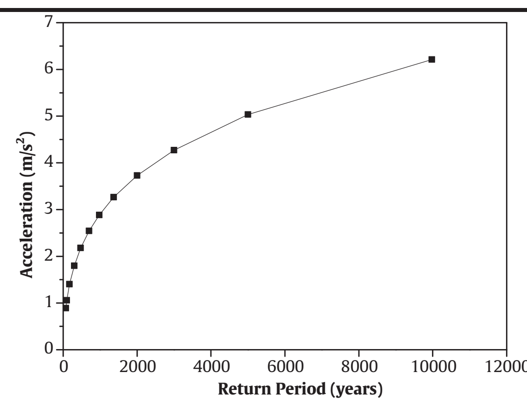


Fig. 5. 3-D model of the building: (a) building 1, (b) building 2.

Table 3
Hazard curves for the moderate-high European scenario [39].

PGA (g)	PGA (m/s^2)	Return period (years)
0.09g	0.889	73
0.11g	1.06	100
0.14g	1.402	170
0.18g	1.796	300
0.22g	2.18	475
0.26g	2.543	700
0.29g	2.884	975
0.33g	3.265	1370
0.38g	3.728	2000
0.44g	4.273	3000



and transverse. This software has been used successfully to describe the seismic vulnerability of concrete structures considering the geometric non-linearity and material properties. The non-linear analysis was conducted by using the numerical implicit Wilson-θ time integration method. The effects of (P–Δ) were considered within all non-linear analyses. The mass and stiffness proportional Rayleigh damping coefficients were determined for the response-history analysis of the buildings considering the first two modal periods assuming a 5% viscous damping ratio. Additional hysteretic damping was developed through the yielding of building components, which are known to experience inelastic deformations. Both buildings are considered on hard soil, thus soil flexibility for the building foundations are not taken in the analytical models.

To assign plastic hinges, since the structure is modeled with the loads, section properties and steel content, therefore default hinges are assigned to column as PMM, and to the beams as M3 in order to FEMA-356 [25].

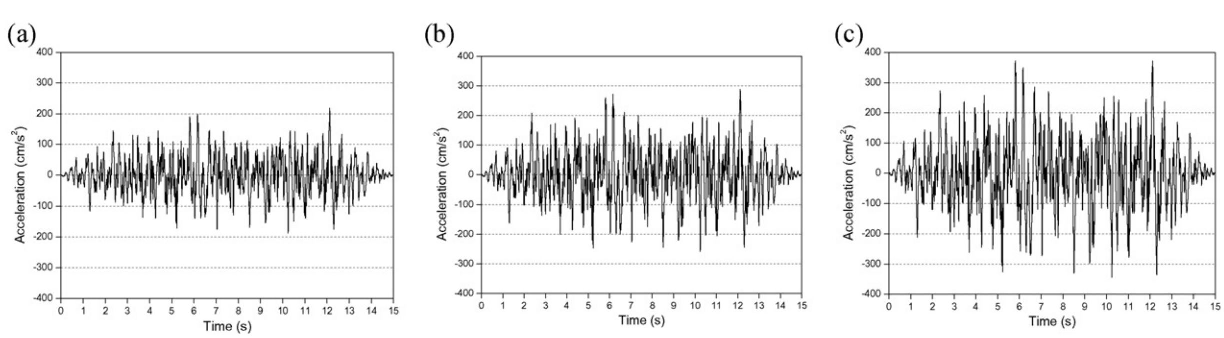


Fig. 6. Time versus acceleration for (a): 475, (b): 975 and (c): 2000-yrp.

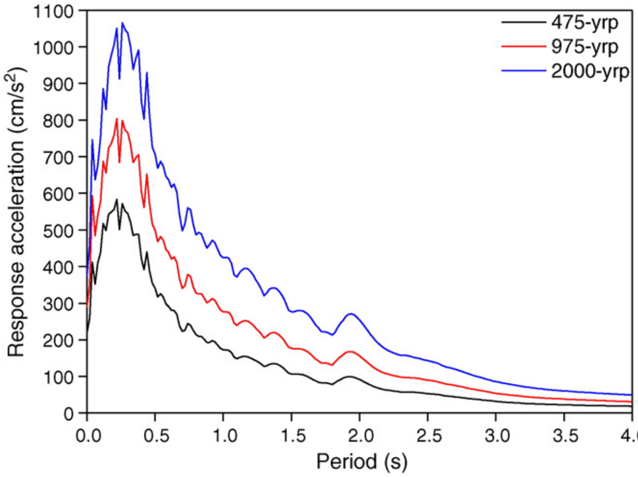


Fig. 7. Response spectra for 475, 975 and 2000-yrp (5% damping).

A schematic diagram of the plastic hinge behavior is shown in Fig. 3, based on the parameters specified in FEMA-356 [25]. The building performance levels are defined as Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). Note that point A is the origin, B is the yield and C is the end state of the plastic hinge range, D presents the residual strength and E is the failure point. The parameters are adopted from FEMA-356 [25] for columns, beams and shear walls respectively. For column the mentioned parameters are upon on the ratio of $\frac{P}{A_g.f_c'}$, present of confined or unconfined concrete for transfer reinforcement and the ratio of $\frac{V}{b_w.d.\sqrt{f_c'}}$. For beams the ratio of $\frac{\rho-\rho'}{\rho_{bal}}$ should be checked, however for shear walls the ratio of $\frac{(A_s-A_s').f_y+P}{t_w.l_w.f_c'}$ and $\frac{V}{t_w l_w.\sqrt{f_c'}}$ have to be considered to determine the mentioned parameters.

where P: axial force, A_g: the gross cross-section, V: the shear force, b_w: width of the section, d: distance from extreme compression fiber to centroid of tension reinforcement, f_c': compression strength of concrete, A_s: reinforcement area, A_s': compression reinforcement area, f_y: yield strength of tension reinforcement, t_w: thickness of wall and l_w is length of wall.

Default hinge properties are recommended by SAP2000, in terms of PMM (with axial force-moment interaction) for column and M3 for beams. The PMM plastic hinges were assigned at the wall ends and shear hinges were assigned at the mid-height level of the walls [26]. To model shear wall, previous researchers have shown different methods namely: (*i*) Equivalent Frame Model [27–31], (*ii*) Braced Frame Analogy [32] and (*iii*) Two-Column Analogy [33], in this study the first method is used. In this research every shear wall is modeled as an equivalent frame structure, which is also known as wide column analogy with rigid beams at the floor levels. On the other word each shear wall is exchanged by an idealized frame structure involving of a column and rigid beams located at floor levels. Note that the column is located at the center of wall axis and wall's inertia and axial area are assigned to the equivalent element. In each fame level rigid beams join the equivalent element to adjacent columns. Large values are assigned for the inertia values of rigid arms compared to other frame elements. A sample model is presented in Fig. 4.

Different studies have proposed their experiences to estimate the hinges length (L_P). In this study (L_P) is consider a high value between

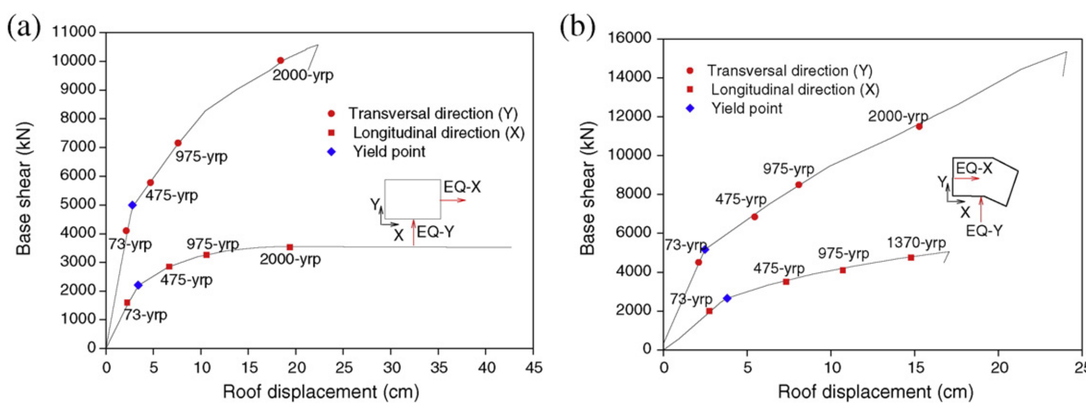


Fig. 8. Capacity curves in longitudinal and transverse directions: (a) building 1, (b) building 2.

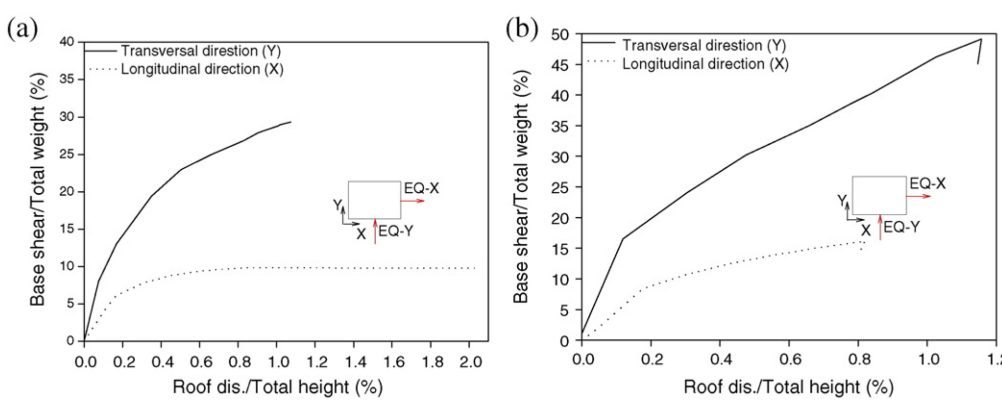


Fig. 9. Comparison of the capacity curves in longitudinal and transverse directions: (a) building 1, (b) building 2.

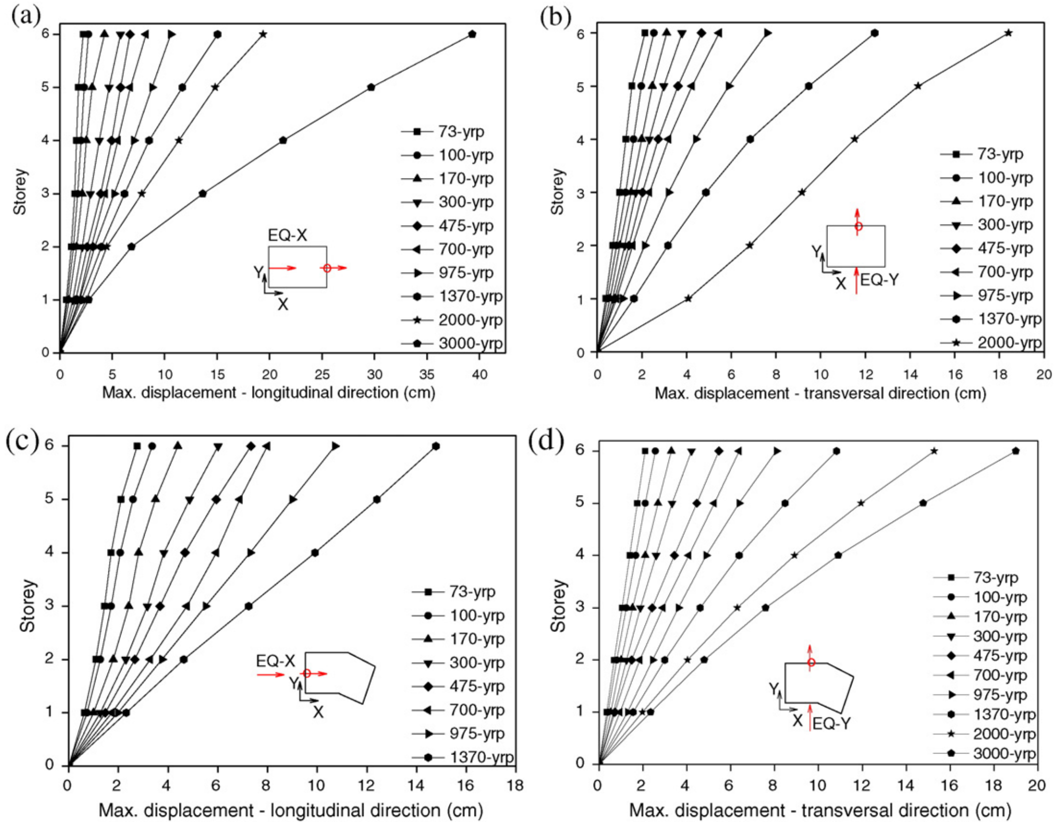


Fig. 10. Maximum displacement profile for earthquakes in: (a) building 1 — longitudinal direction, (b) building 1 — transverse direction, (c) building 2 — longitudinal direction, (d) building 2 — transverse direction.

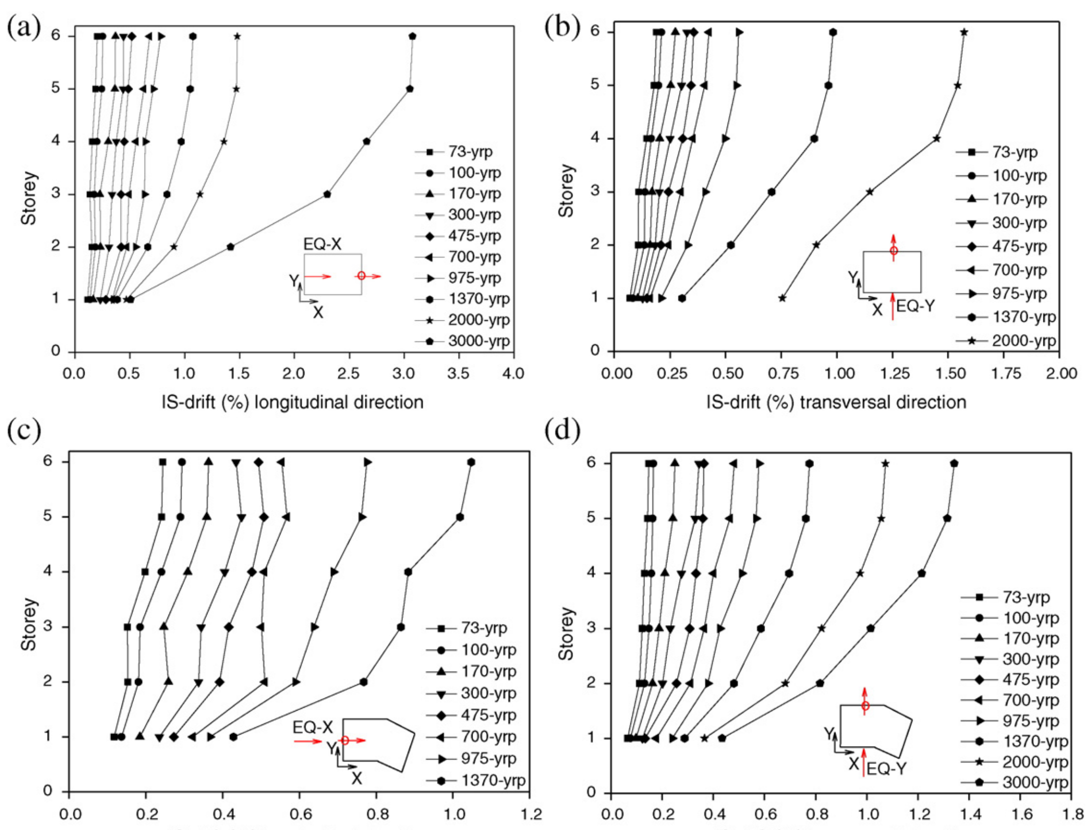


Fig. 11. Max. IS-drift in: (a) building 1 — longitudinal direction, (b) building 1 — transverse direction, (c) building 2 — longitudinal direction, (d) building 2 — transverse direction.

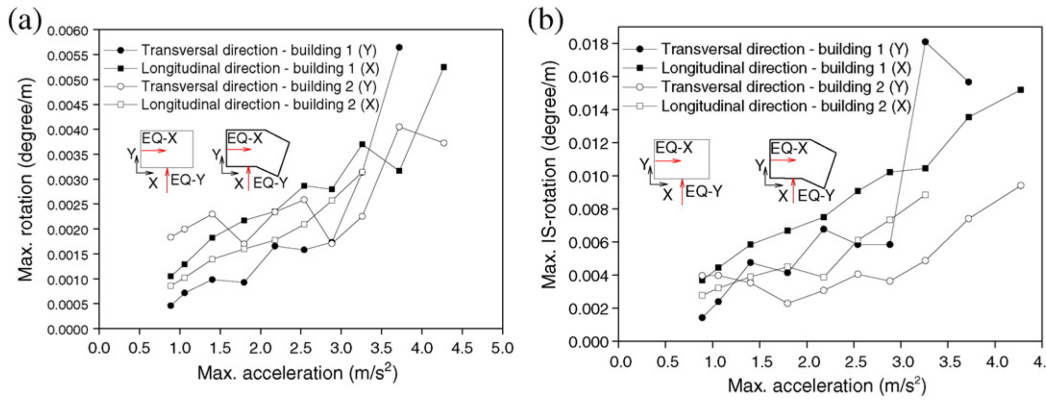


Fig. 12. (a) Maximum rotation, (b) max. IS-rotation in different earthquakes.

Expression (1), which is recommend by Park and Paulay [34] and Eq. (2), which is proposed by Priestley et al. [35], both of these recommendations have been used in some guidelines (i.e. ATC-32 [36]):

$$L_p = 0.5H \tag{1}$$

$$L_p = 0.08L + 0.022\ f_{ye}d_{bl} \geq 0.044\ f_{ye}d_{bl} \tag{2}$$

in Eqs. (1) and (2), L_p: plastic hinge length, H: section depth, L: critical distance from the critical section of the plastic hinge to the point of contra flexure, f_{ye}: expected yield strength and d_{bl}: diameter of longitudinal reinforcement. Fig. 5 shows the 3-D model which is analyzed in this study.

3.2. Static push-over analysis

One of the powerful seismic tools in civil engineering is using nonlinear static procedure to predict seismic demands and the horizontal capacity in structures [37]. Push over method becomes more popular for nonlinear analysis of structures due to simplicity in operation and time efficiency [36]. It could be performed as either displacement controlled or force controlled. The First approach is used when the building loses its strength, or when specified drifts are examined, where the amount of the applied load is not known [25]. However, the second one is used when the load is known and the structure expected to support the load [38]. In this paper the first method is used.

3.3. Dynamic analysis

The mentioned buildings have been analyzed by time history analysis using direct integration method. Twelve artificial records were scaled to different peak ground acceleration (PGA) levels from 0.09 g to 0.44 g. An earthquake with 0.09 g is corresponding to the earthquake which has 73-year return period (yrp), however a PGA of 0.44 g is related to 3000-yrp. The input seismic motions were defined in order to be representative of a moderate-high European seismic hazard scenario [39]. To increase return periods hazard based on time series of acceleration (with 15 s duration) were artificially generated yielding a set of twelve uniform hazard response spectra. For generating the record 0.01 s as the time increment was considered to give input accelerograms with 1500 points. Table 3 presents the return periods and the corresponding values of peak acceleration. The acceleration time histories for 475, 975 and 2000 years return periods are depicted in Fig. 6. In Fig. 7, the 5% damped acceleration response spectra for 475, 975 and 2000 years return periods are plotted.

4. Discussion of the results

4.1. Push-over results

Fig. 8 presents the results of the push-over analysis for both directions: longitudinal and transverse for the initial assessment of structures and general behavior of the buildings. Note that the capacity curve which is shown in terms of top displacement versus base shear represents the envelope of the structural behavior under inelastic incursions. The capacity curve is performed for corresponded top floor nodes in longitudinal and transverse directions. Fig. 9 considers base shear and top displacement normalized with respect to the total height = 20.8 m and total weight = 36,075 and 31,266 kN for buildings 1 and 2 respectively. Fig. 10 plots the results from time history analyses of two buildings subjected to 12 artificially seismic records. The absolute of maximum values of roof displacement obtained from nonlinear

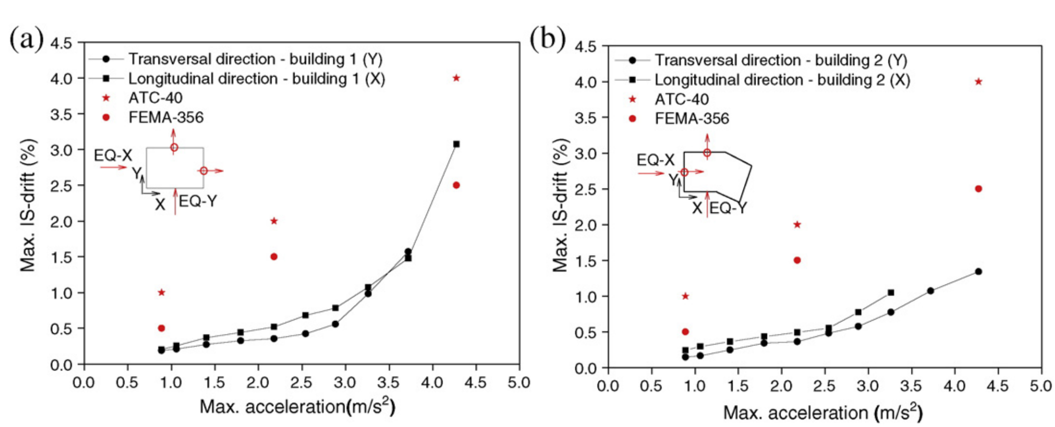


Fig. 13. Max-IS-drift in different earthquakes: (a) building 1, (b) building 2.

Table 4
IS-drift limits in FEMA-356 [25] and ATC-40 [38].

Performance level according to FEMA-356				
	Fully operational	Operational	Life safe	Near collapse
Drift limit	0.2%	0.5%	1.5%	2.5%
Performance level according to ATC-40				
	Immediate occupancy	Damage control	Life safety	Structural stability
Drift limit	1%	1–2%	2%	4%

analyses is presented in Fig. 8 for 73, 475, 975 and 200 yrp to compare strength behavior of two structures. The push-over curve for transverse direction shows a higher initial stiffness and strength. By applying the rectangular load pattern, in building 1 (Fig. 8a) the first exceedance of yield displacement occurred at base shear 2208 kN and 4996 kN for longitudinal and transverse direction respectively. The corresponding PGA is between 0.11 g–0.14 g for both directions. Before 0.11 g the structure remains in linear region, however after 0.14 g the building follows a non-linear behavior in both directions.

For building 2 (Fig. 8b) the first yield displacement for longitudinal direction happened in 3.8 cm, and the corresponding PGA is between 0.11 g–0.14 g. While for transverse direction it is fall out in 2.47 cm. The PGA is between 0.09 g–0.11 g.

The push-over analyses show the higher initial stiffness and strength for transverse direction in both building, however in longitudinal direction the buildings are more flexible.

Not that the total weight for building 1 and 2 is 36,075 kN and 31,266 kN respectively. The effective mass associated with a mode decreases as the mode number increases. In the analyses the number of modes should be large enough, hence, the modes used in this study represent 90% of the total structure mass.

Fig. 9 considers base shear and top displacement normalized with respect to the total height = 20.8 m and total weight = 36,075 and 31,266 kN for buildings 1 and 2 respectively.

4.2. Global response

Using twelve earthquake records developed in dynamic non-linear analysis. Time history analyses with increasing ground motion intensity, were promoted for each case study buildings. Maximum displacement, inter-storey drift as well as rotation were examined in this study. Summary of the results from maximum displacement and inter-storey drift and inter-storey rotation (IS rotation) which are obtained in different time history analyses is presented and compared in Figs. 4–13.

Fig. 10 provides the maximum displacement from each time history analyses for both buildings in longitudinal and transverse directions. The displacement profiles resulting from each non-linear analysis are compared to the responses of the push-over analysis in Fig. 8. The building models in two directions have been pushed until the response resulting of the structure becomes equal to roof displacement from the time history analysis. Consequently the target displacements in the different procedures are shown that the buildings are more flexible in longitudinal direction, while in transverse direction it is stiffer. Figures present that building 2 shows poor performance in terms of strength, tangent stiffness and deformation as compared with building 1.

For the purpose of general evaluation, in Fig. 11 the inter-storey drifts resulting from all of the records for the time history analyzes are shown. One of the most important factors to evaluate the seismic performance is inter-storey drift ratio (IDR), because it is related to level of structural damage which is determined from Eq. (3).

$$IDR = (\delta_i\text{–}\delta_{i\_1})/h_i \tag{3}$$

where, δ_i is the lateral displacement corresponding to level i, δ_{i_1} states the lateral displacement corresponding to adjacent floor level $_{i\text{-}1}$, h_i: storey height. For both buildings in transverse direction, the maximum IS-drift is presented in 6th storey with value around 1.6% and 1.38% respectively. Fig. 11a and c provides, the IS-drift distribution which is almost uniform when the building drift is lower 0.3%, since the structure behavior is elastic. As shown in Fig. 11b and d, for transverse direction the structures remain in elastic zone when the IS-drift is below 2.5% and 1.5% for both buildings respectively. The 6th storey shows significant IS-drifts compared to the lower stories.

For building 1 the Fig. 11a illustrates that in earthquake with value of 0.38 g the IS-drift is 1.48%, however in 0.44 g the IS-drift is jumped to 3.07%. Building 2 follows the same pattern, in 0.33 g the IS-drift is 1.05%, but in 0.38 g the IS drift increased from 4.44% in 14.90 s, to 7.77% in 14.93 s, and finally collapse.

As mentioned before two buildings are irregular in plan, because the center of mass does not coincide with the center of stiffness. So it causes structures to rotate around the center of stiffness because of the torsion. However in elevation, even the first storey is larger comparing to the other stories, the graphs show the linear behavior in elevation in terms of inter-storey drift. But in building 2 in longitudinal direction in 2000-yrp the nonlinear response in the 1st storey is shown.

Fig. 12 provides the maximum inter-storey rotation and maximum rotation of the buildings in longitudinal and transverse directions. Fig. 12a provides the rotation of two buildings in two directions. The maximum rotation happened in building 1 in transverse direction with the value of 0.0057 (°/m). The corresponding PGA for the mentioned earthquake is 0.38 g. Since the structure shows the non-linear behavior, therefore the maximum jump occurred between 0.29 g–0.38 g. For the longitudinal direction in building 1 the maximum rotation happened between 0.38 g–0.44 g with the values 0.003–0.0052 (°/m) respectively. For building 2 in longitudinal direction the trend is linear, while in transverse direction the graph does not follow the specific pattern. The maximum jump is between 0.33 g–0.38 g.

From Fig. 12b it is shown that by comparing longitudinal and transverse directions, for both buildings maximum IS-rotation is in transverse direction. It also illustrates that the behavior in building 1 in longitudinal direction is linear. As it is expected this building in X direction is more flexible and it runs until 3000-yrp with a value of 0.015. For the transverse direction the maximum IS-rotation is in 1375-yrp with a value of 0.0181. In building 2 the structure in transverse direction is stiffer than the other ones. It goes to 3000-yrp with the value = 0.095. However, in longitudinal direction it runs until 1375-yrp. As it remanded before, between 1375 and 2000-yrp earthquakes, there is a high jump in displacement and rotation. For building 2 in 2000-yrp until 14.89 s rotation is 0.034°. Since the structure is in non-linear zone, after 4 s until 14.93 s, rotation increases and the structure is destroyed.

The main goal of this section is considered the application and methodology for the seismic assessment of existent real buildings. In fact this is an important topic, to understand the seismic vulnerability of certain particularities in existing buildings to assure the common observation

Table 5
Performance objective in FEMA-356 [25].

Earthquakes design level	Fully operational	Operational	Life safe	Near collapse
43-yrp-frequent				
73-yrp-occasional		X		
475-yrp-rare			X	
(975–2000)-yrp-very rare				X

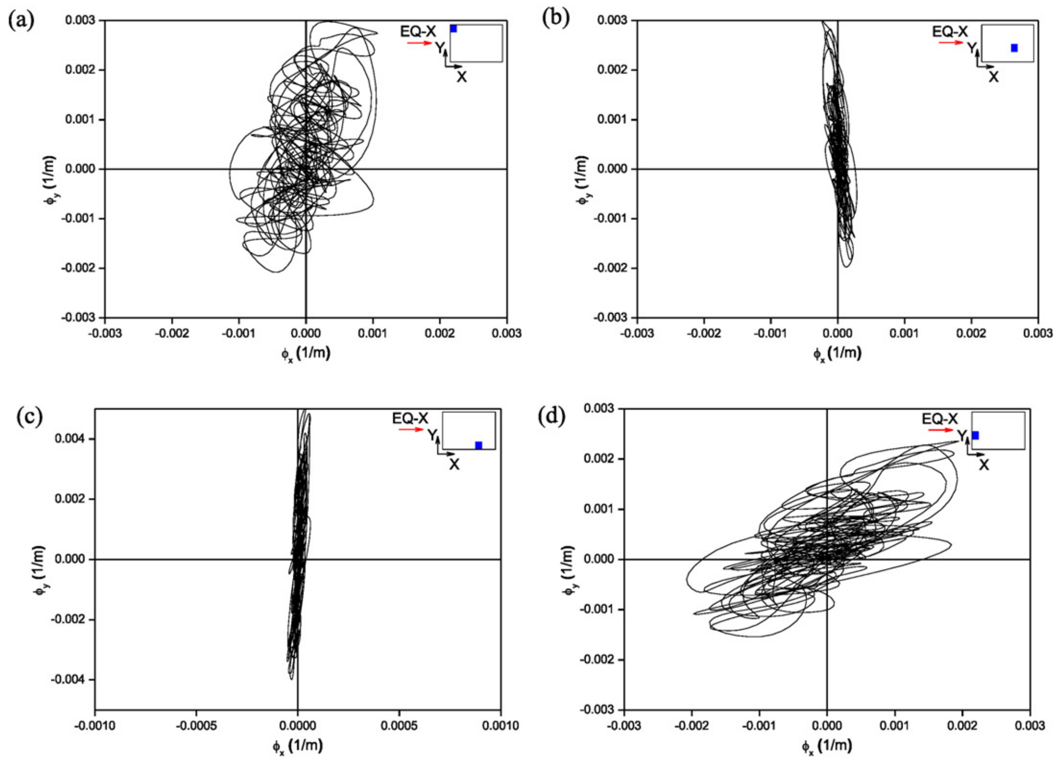


Fig. 14. The biaxial demand in: (a) corner, (b) centre, (c) facade-X, (d) facade-Y columns for $a_g = 0.38$ g, building 1.

can be applied for prototype building, especially irregular ones. Also one of the major observations of this study is the comprehension of the effect and importance of biaxial loading in columns and the influence of the axial load variation, relating the position of the columns in plan and in high, highlights the importance of this topic.

4.3. Seismic safety assessment of the building

In this paper, the result of time history analysis in terms of maximum drift and maximum displacement are compared with international guidelines: FEMA-356 [25], ATC-40 [38], in order to

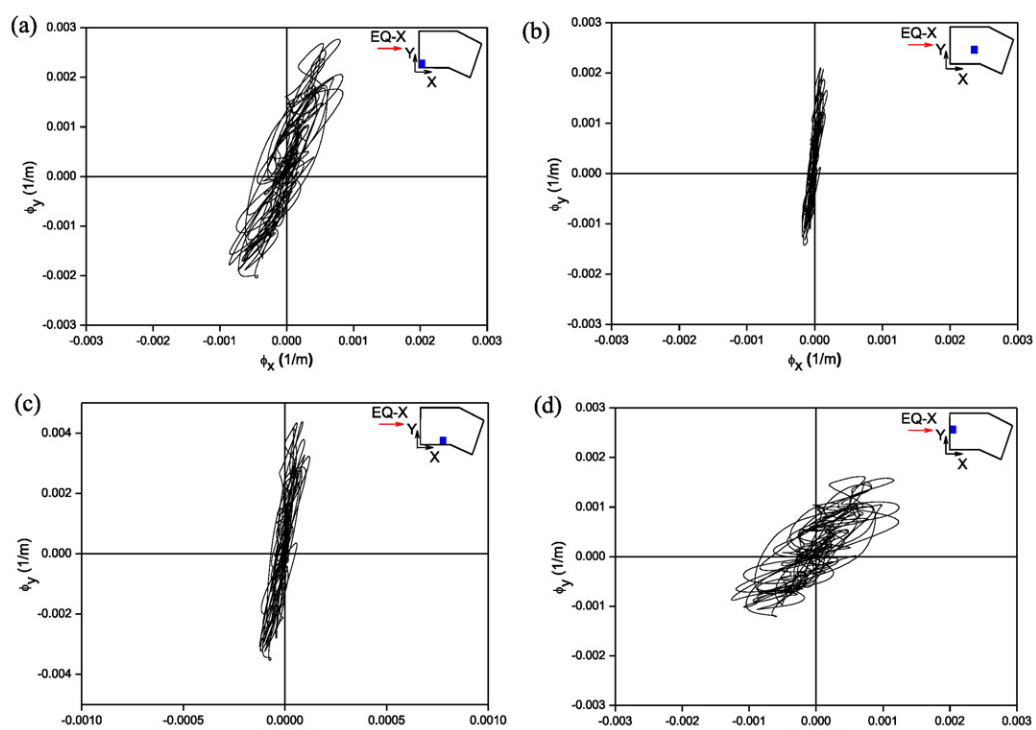


Fig. 15. The biaxial demand in: (a) corner, (b) center, (c) facade-X, (d) facade-Y columns for $a_g = 0.29$ g, building 2.

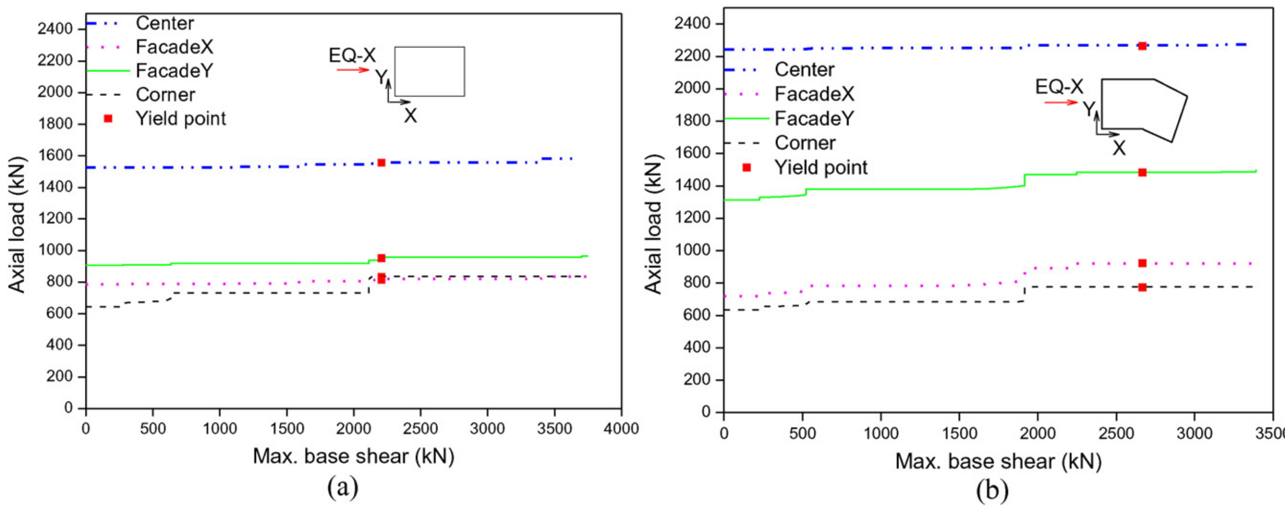


Fig. 16. Variation of axial load — base shear: (a) building 1, (b) building 2.

assess vulnerability of six-storey buildings (Tables 4 and 5). Although, other researchers follow the same concept with differences in values [39–40], these two guidelines are very popular among civil engineers to assess the seismic vulnerability of buildings. Also at that time there are no drift limits imposed by the Eurocodes to evaluate the seismic safety of existent buildings. The FEMA and ATC proposed limits for well design RC structures that are in accordance with the experimental observation of several authors around the world.

As can be seen in the Tables 4, 5 and Fig. 13, the IS-drift values for life safety and near collapse for both buildings in transverse direction are more than the global-level limit.

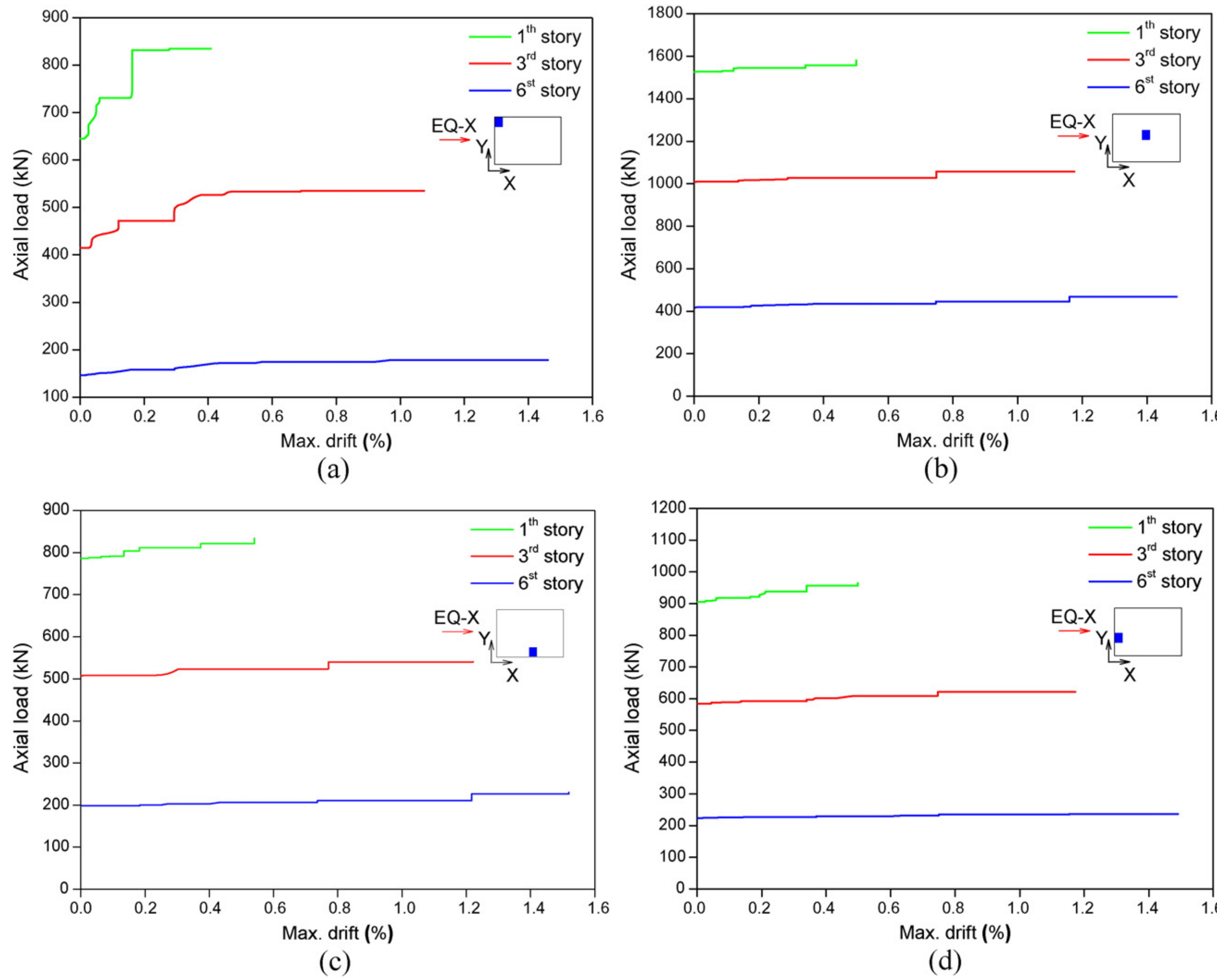


Fig. 17. Variation of axial load — drift in 1st, 3rd, 6th stories: (a) corner, (b) centre, (c) facade-X, (d) facade-Y columns for $a_g = 0.38$ g, building 1.

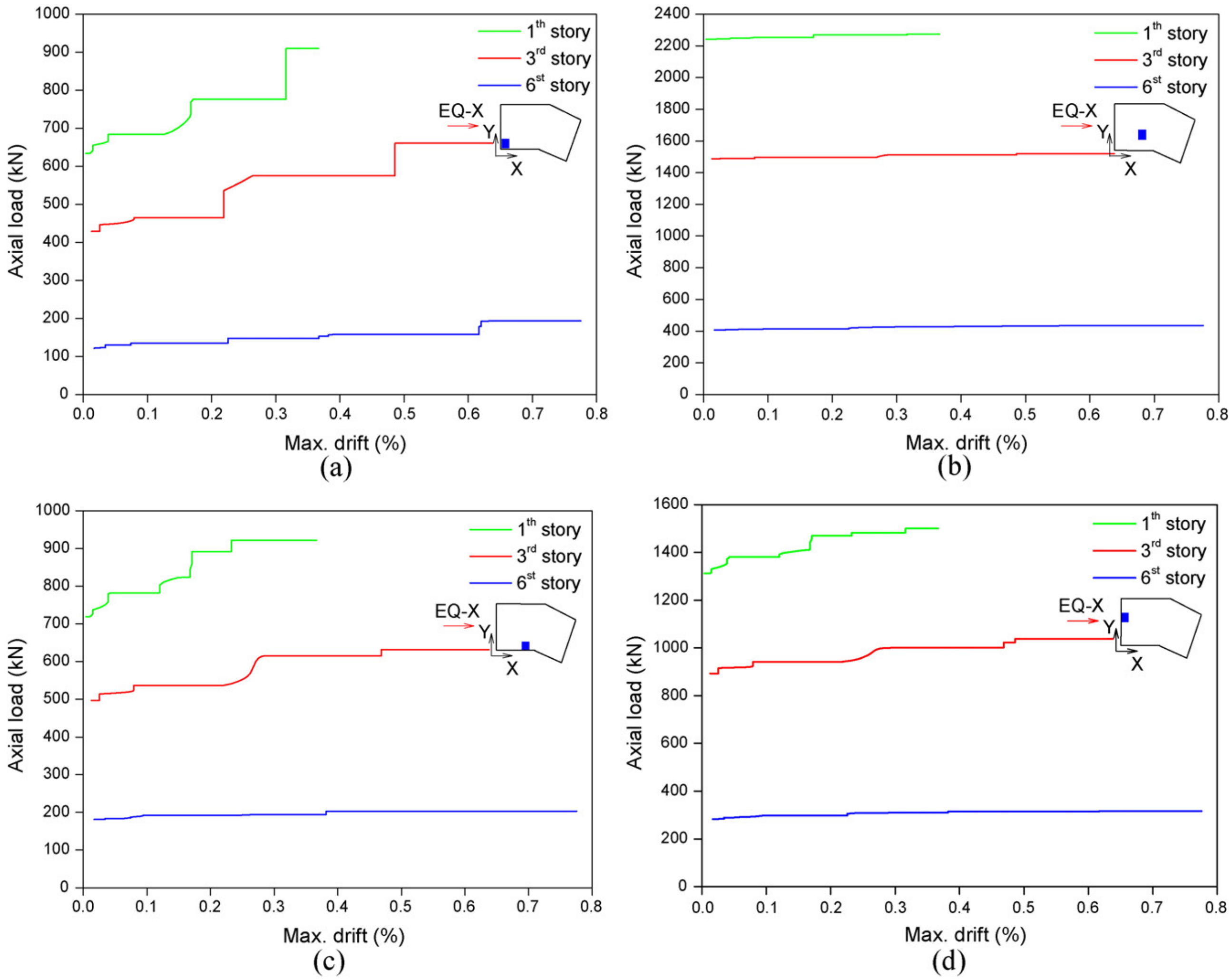


Fig. 18. Variation of axial load — drift in 1st, 3rd, 6th stories: (a) corner, (b) centre, (c) facade-X, (d) facade-Y columns for $a_g = 0.29$ g, building 2.

For building 1 in collapse prevention, the structure loses the strength and becomes unstable in 3000-yrp in longitudinal direction with drift value more than 2.5% according to FEMA-356, however it would be stable in order to ATC-40. In building 2, in longitudinal direction the structure is stable up to 2000-yrp, with the drift value 1.04%, but in 2000-yrp the IS-drift value is more than the FEMA and ATC global-level. Therefore, the case study building is stiffer in transverse direction, however in longitudinal direction it is vulnerable for the rare event based on a general global-level evaluation using the suggested drift limits.

4.4. Local response

For local response, 2000-yrp earthquake corresponding to PGA = 0.38 g for building 1 is selected. Since the structure in 2000-yrp loses its resistance and collapse, so the 975-yrp earthquake corresponding to PGA = 0.29 g for building 2 is chosen. Four columns which are located in the 1st storey are selected as a representative of corner, center and facade columns. For member level performance the biaxial demand, the variation of axial load versus base shear and drift and the variation of axial load for all the columns in each first floor are studied.

As can be seen in Figs. 14 and 15, for both buildings the most important biaxial demand happens in the corner columns while for the center columns the lower demand is represented. In facade columns, since it is assumed that the earthquake force is applied in the longitudinal direction, so it is expected that Φ_x should be greater than Φ_y, but from the analysis results, for the facade column in X direction, it is reverse. Hence, it could be concluded that torsion happened in the buildings.

Fig. 16 illustrates the variation of axial load in terms of base shear. For both buildings, it is obvious that the minimum initial axial load belongs to the corner column. It increases in facade column in X direction, facade column in Y direction and center column. It should be noted that by increasing the initial axial load the biaxial demand decreases.

For building 1, the maximum interaction happens in the corner column with the values between 645 and 834 kN (around 30%), while center and facade columns represent a lower values (around 6%).

For building 2, as can be seen all the graphs follow the same pattern. It is noted that the maximum variation of axial load versus base shear is in corner and facade-X columns with values around 25%. The facade column in Y direction shows the 14% changes, and center column presents only 1% variation. The yield point which is obtained from Fig. 8 is shown on both graphs. It is noted that for building 1 the yield point is near the second jump of graph, while in building 2 the yield point does not follow the specific pattern.

Figs. 17 and 18, present the variation of axial load in terms of drift for four representative columns in 1st, 3rd and 6th stories. In building 1 the maximum variation of axial load happens in the corner column in the first storey, while in building 2 corner and facade-X columns show the higher demands. It is obvious that the initial axial load is decreasing from lower floors to upper one, while drift is reverse. As noted above by increasing the initial axial load the variation of axial load decreases. This section is not only to highlight the torsion, but also the importance

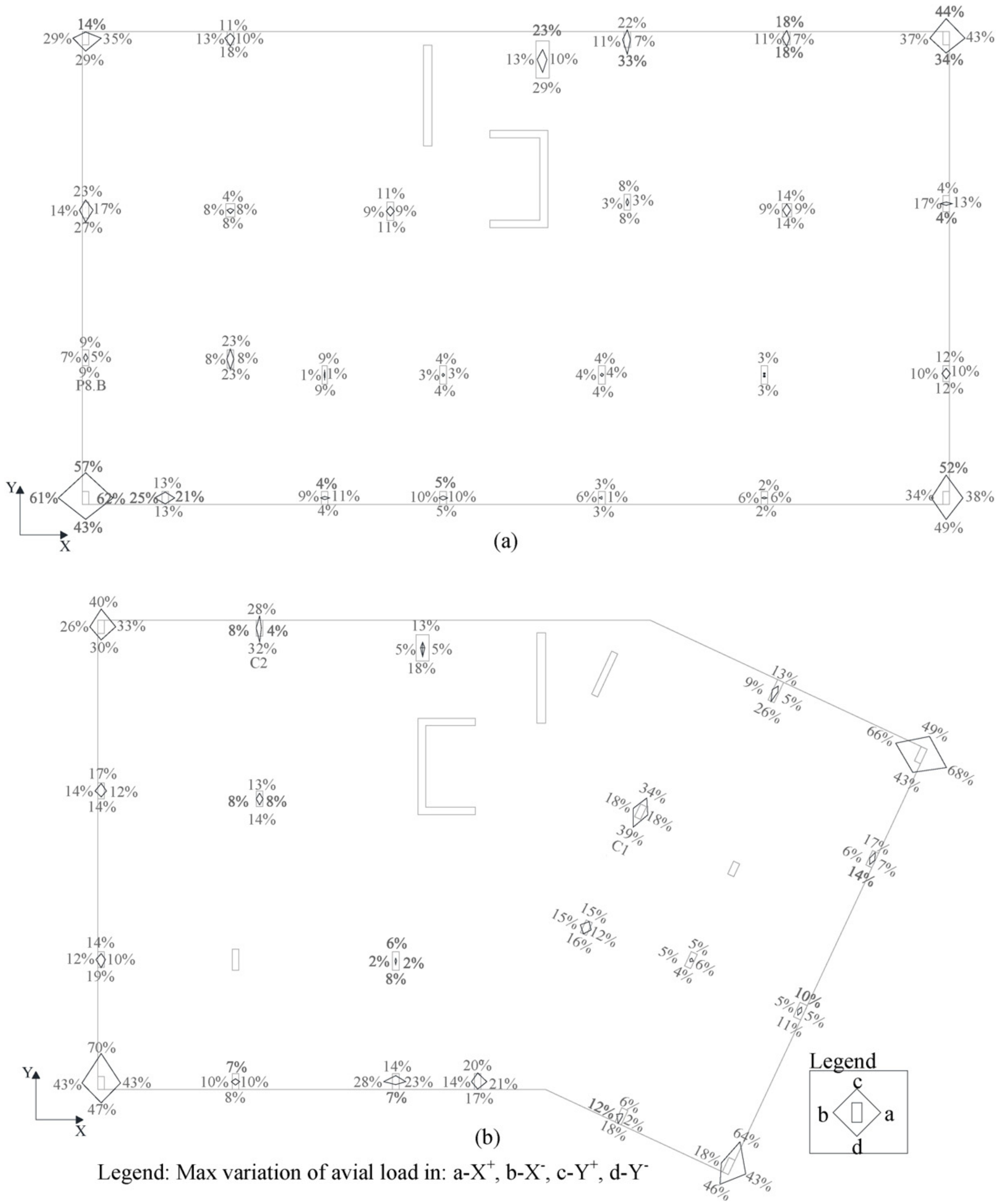


Fig. 19. Variation of axial load for columns in: (a) building 1, (b) building 2.

of biaxial demand in the columns, in particular in the corner columns is considered. As stated by several authors [1,7,8] the biaxial demand can change significantly the behavior of a column, increasing the seismic vulnerability of the buildings.

Fig. 19 shows the variation of axial loads for two buildings. Since the other stories follow the same pattern, only the first storey is considered. For building 1 in Fig. 19a it is obvious that, as it is expected in corner columns the variation of axial load is high (30–60%), in center columns is neglegible (0–10%) and facade columns the lower demand is presented (10–30%).

For building 2 in Fig. 19b these variations are the same as the other one. It is clear that for corner column the variation is between 30–70%, for facade columns is 10–30% and for center columns is between 0–15% except a few columns like C1 and C2. Although C1 is located in center but the variation is high. In the case of column C2 in longitudinal direction the variation is low, while in transverse direction this value is high.

5. Summary and conclusion

In this research two existing concrete buildings which were designed with older codes were chosen and proposed for push-over and time history analyses in longitudinal and transverse directions. The building responses were analyzed in two different levels: global and local. For global response, max displacement, IS-drift, and IS-rotation were considered. While, for local member, four columns were selected which were located at the 1st storey namely: corner, center, facade-X and facade-Y, and the variation of axial load in terms of base shear and drift for 1st, 3rd and 6th stories and biaxial demand was studied. The conclusions of the study were summarized as follows:

- For these two buildings, since the center of mass does not coincide with the center of stiffness, the torsional forces cause the structure to rotate around the center of stiffness. This indicates that both buildings are irregular. Although building 1 is regular in plan, but both of the buildings show irregularities due to the position of the mass and stiffness centers.
- In elevation, both buildings are regular and the response of the buildings shows that even the first storey is larger than the other stories, but the behavior is linear. The nonlinear behavior is shown only in building 2 in longitudinal direction in 2000-yrp.
- The vulnerability assessment based on FEMA-356 and ATC-40 clearly shows that building 1 is vulnerable in longitudinal direction according to FEMA-356, however it would be stable in order to ATC-40. However, building 2 is stable based on both guidelines. Comparing the obtained results with the drift limits proposed by FEMA-356 ATC-40, the building is safe, with the exception of the last limit state proposed by ATC40. To have a full conclusion about the seismic safety of this particular building more analysis should be performed including more earthquakes to consider the full response of the RC building for different earthquakes.
- An abrupt change in inter-storey rotation is observed in building 1 in transverse direction between 0.29 g–0.33 g which shows the large rotation in the 6th storey. It is because the non-linear behavior of the structure. The main reason is that the building is behaving almost in an elastic range until this point. On the other hand after this acceleration the structure is subjected to sever nonlinearity and incursions are more severe for the structure.
- For local response two buildings show that, the most variation of axial load happens in corner, facade-X, facade-Y and center column respectively. It is noteworthy that by increasing the initial axial load the biaxial demand decreases.
- In both buildings facade column in X direction presents only uniaxial behavior in longitudinal and transverse directions. Since the direction of earthquake is in longitudinal direction, so it is expected that the Φ_x should be more than Φ_y, but the analysis presents in X direction for longitudinal direction it is reverse. Hence, it could be concluded that torsion happened in the building.

Acknowledgments

This paper reports research developed under financial support provided by “FCT — Fundação para a Ciência e Tecnologia,” Portugal, of the first author through the research project PTDC/ECM/102221/2008.

References

[1] Rodrigues H, Arêde A, Varum H, Costa A. Damage evolution in reinforced concrete columns subjected to biaxial loading. Earthq Eng Struct Dyn 2013;42:239–59.
[2] Sorace S, Terenzi G. Dissipative bracing-based seismic retrofit of R/C school buildings. Open Constr Build Technol J 2012;6:334–45.
[3] Masi A. Seismic vulnerability assessment of gravity load designed R/C frames. Bull Earthq Eng 2003;1:371–95.
[4] De Stefano M, Tanganelli M, Viti S. Effect of the variability in plan of concrete mechanical properties on the seismic response of existing RC framed structures. Bull Earthq Eng 2012. http://dx.doi.org/10.1007/s10518-012-9412-5.
[5] Kim T, Kim J. Seismic demand of an RC special moment frame building. Struct Des Tall Spec Build 2009;18(10):137–47.
[6] Panagiotakos TB, Fardis MN. Seismic performance of RC frames designed to Eurocode8 or Greek code 2000. Bull Earthq Eng 2004;2(2):221–59.
[7] Chaulagain H, Rodrigues H, Jara J, Spacone E, Varum H. Seismic response of current RC buildings in Nepal: a comparative analysis of different design/construction. Eng Struct 2013;49:284–94.
[8] Rodrigues H, Romao X, Andrade-Campos A, Varum H, Arêde A, Costa AG. Simplified hysteretic model for the representation of the biaxial bending response of RC columns. Eng Struct 2012;44:146–58.
[9] Rodrigues H, Arêde A, Varum H, Costa AG. Experimental evaluation of rectangular reinforced concrete column behaviour under biaxial cyclic loading. Earthq Eng Struct Dyn 2013;42:239–59.
[10] Rodrigues H, Varum H, Arêde A, Costa A. Comparative efficiency analysis of different non-linear modeling strategies to simulate the biaxial response of RC columns. Earthq Eng Eng Vib 2012;11:553–66.
[11] Varum H, Pinto A, Costa A, Vila RP. Simplified models for assessment and optimal redesign of irregular planar frames. Eng Struct 2012;42:245–57.
[12] Kueht E, Hueste MD. Impact of code requirements in the Central United States: seismic performance assessment of a reinforced concrete building. ASCE J Struct Eng 2009;135(4):404–13.
[13] Kotronis P, Ragueneau F, Mazars JA. Simplified modeling strategy for R/C wall satisfying PS92 and EC8 design. Eng Struct 2005;27(8):1197–208.
[14] Ile N, Reynouard JM. Non-linear analysis of reinforced concrete shear wall under earthquake loading. Earthq Eng 2000;4(2):183–213.
[15] Mazza F. Modelling and nonlinear static analysis of reinforced concrete framed buildings irregular in plan. Eng Struct J 2014;80:98–108.
[16] Mazza F. Comparative study of the seismic response of RC framed buildings retrofitted using modern techniques. Earthq Struct 2015;9(1):29–48.
[17] Romão X, Delgado R, Costa A. Practical aspects of demand and capacity evaluation of RC members in the context of EC8-3. Earthq Eng Struct Dyn 2010;39:73–499.
[18] Rodrigeus H, Varum H, Arede A. Biaxial seismic behaviour of reinforced concrete columns. 2nd ECCOMAS young investigators conference (YIC 2013); Sep 2013 [Bordeaux, France. <hal00855921>].
[19] Eurocode 2: design of concrete structures. Part 2: concrete Bridges. 2ndDraft, Bruxelas; January, 2002.
[20] REBAP — Regulamento de Estruturas de Betão Armado e Pré-Esforçado. Dec.-Lei N.° 349-C/83 de 30 de Julho; 1983.
[21] CEB-FIP model code 1990 design code. Comité Euro-International du Béton; 1993.
[22] FEMA. State of the art report on past performance of steel moment-frame buildings in earthquakes-FEMA 355E: Washington DC; 2000.
[23] Otani S. Disaster mitigation engineering — the Kobe earthquake disaster-. JSPS seminar on engineering in Japan at the Royal Society, London; 1999.
[24] CSI. SAP2000 V-14. Integrated finite element analysis and design of structures basic analysis reference manual. Berkeley (CA, USA): Computers and Structures Inc.; 2009.
[25] FEMA356. Pre-standard and commentary for the seismic rehabilitation of buildings. Washington, DC: Federal Emergency Management Agency; 2000.
[26] Inel M, Ozmen HB. Effects of plastic hinge properties in non-linear analysis of reinforced concrete buildings. Eng Struct 2006;28:1494–502.
[27] MacLeod IA. General frame element for Shear Wall analysis. Proc Inst Civ Eng Part 2 1976;61:785–90.
[28] MacLeod IA. Structural analysis of Wall systems. Struct Eng 1977:487–95.
[29] MacLeod IA, Hosny HM. Frame analysis of Shear Wall cores. J Struct Div ASCE 1977; 103(10).
[30] Lew IP, Narov F. Three-dimensional equivalent frame analysis of shear walls. Concrete International; 1983.
[31] Smith BS, Girgis A. Deficiencies in the wide column analogy for shear wall Core analysis. Concrete International; 1986.
[32] Smith BS, Girgis A. Simple analogous frames for Shear Wall analysis. J Struct Div ASCE 1984;110(11).
[33] Smith BS, Jesien W. Two-column model for static analysis of mono symmetric Thin Wall beams. Structural engineering report no: 88–3. Department of Civil Engineering and Applied Mechanics, McGill University; 1988.
[34] Park R, Paulay T. Reinforced concrete structures. New York: John Wiley & Sons; 1975[769 pages].
[35] Priestley MJN, Seible F, Calvi GMS. Seismic design and retrofit of bridges. New York: John Wiley & Sons; 1996.
[36] Applied Technology Council, ATC-32. Improved seismic design criteria for California bridges: provisional recommendations; 1996[Redwood City (CA)].
[37] Nicknam A, Mosleh A, Hamidi H. Seismic performance evaluation of urban Bridge using static nonlinear procedure, case study: Hafez Bridge. Protein Eng 2011;14: 2350–7.
[38] ATC-40. Seismic evaluation and retrofit of concrete buildings. Applied technical council, California seismic safety commission, report no. SSC 96–01 (two volumes); 1996 [Redwood City, California, US].
[39] Carvalho EC, Coelho E, Campos-Costa A. Preparation of the full-scale tests on reinforced concrete frames — characteristics of the test specimens, materials and testing conditions. ICONS report, innovative seismic design concepts for new and existing structures. Lisbon: European TMR Network – LNEC; 1999.
[40] SEAOC. Vision 2000. Performance based seismic engineering of buildings, vols. I and II: conceptual framework. Sacramento (CA): Structural Engineers Association of California; 1995.