Estimation of the Seismic Demand and Capacity of RC Buildings Using Nonlinear Analysis Procedures

Conference Paper · October 2017

2 authors:

Huseyin Bilgin
Epoka University

Marsed Leti
Epoka University

Estimation of the Seismic Demand and Capacity of RC Buildings Using Nonlinear Analysis Procedures

Marsed Leti[1], Hüseyin Bilgin[1]

1Department of Civil Engineering, Epoka University, Albania

Abstract

In this paper the inelastic response of existing reinforced concrete (RC) buildings designed without seismic details is investigated, presenting the results from more than 80 nonlinear analysis. The seismic performance evaluation is conducted for two building types representing low-rise and a mid-rise residential buildings. The overall objective of the current study is to investigate the structural performance of RC buildings under different seismic loadings. For this purpose two performance earthquake engineering (PBEE) procedures are utilized: static and dynamic non-linear analysis. Zeus-NL, a finite element analysis program, is employed for the numerical analysis phase using a set of 20 natural ground motion records. In addition, a methodology is presented for the process and strategies followed up to conduct the mathematical model in Zeus-NL software. Afterwards, the structural performance under different loading conditions is investigated using nonlinear procedures. Results are interpreted based on the FEMA 356 guidelines. Moreover, a comparison between static and dynamic pushover curves is accomplished. The interstory drift results show that 50% of the ground motion records forced both structural models to exceed the life safety (LS) performance level. Due to the sudden changes in columns cross-section and reinforcement at the second and third stories, both structures possessed higher amount of interstory drift at these story levels.

Keywords: *Low and Mid-Rise RC Buildings; Nonlinear Analysis; Performance Assessment; Zeus-NL*

1 INTRODUCTION

Earthquakes are one of the most destructive phenomena that cause yearly excessive loss of life and livelihood. Around 10,000 people die each year caused by severe ground motions, while the economic losses are in the billions of dollars, affecting the gross national product of the state [1]. Past studies have shown that reinforced concrete buildings designed with older building codes are prone to seismic actions [2-3]. Earthquake engineering has follow up a long and challenging way since in its beginning, and still appears to improve quickly as we face consequences of the earthquake hazards. The proper design of the structures to resist the severe ground motions, causing as less as possible losses, whether they are human or material, has been the main attention of both researchers and professional engineers. Therefore, the Performance-Based Earthquake Engineering (PBEE) was born as a new but innovative and fast growing idea. The aim of the PBEE is to design building to meet accurate performance objectives under the strong or rare ground motion forces that the structure may experience during its lifespan, following various analysis procedures. In this study the seismic response of two reinforced concrete buildings representatives of low- and mid-rise RC structures is investigated. A three and seven story building is considered from existing structures in Albanian building stock, designed with old building codes. The mathematical model is conducted using Zeus-NL, a software for 2D/3D finite element modeling developed at the Mid-America Earthquake Center, University of Illinois at Urbana-Champaign [4]. Moreover, two analyses procedures are utilized such as static pushover and nonlinear time history analysis. For the static pushover analysis there are applied two types of load patterns, the inverse triangular pattern and uniform or rectangular load pattern. For the dynamic time-history analysis there are considered twenty real ground motion records having different peak ground accelerations and showing no directivity influenced, not to influence the intensity measure. Based on previous studies the structural elements are studied and finally it is verified if structure is considered as special moment frame. Moreover, distribution of the interstory drift for each moment resisting frame is evaluated. The obtained results are compared with life safety (LS) performance level based on FEMA 356 guidelines [5]. In addition, the most critical regions alongside the structure which are forced by the ground motions showing huge amount of drift accumulation are highlighted. Finally, useful conclusions are drawn at the end of this paper.

2 DESCRIPTION OF THE BUILDING AND MATHEMATICAL MODELLING

Both buildings have the same plan dimensions as 23 m long and 14 m wide. They are composed of 5 bays and 4 frames in x- and y-direction respectively as shown in the Figure 1. The story height is 3 m same for each story elevation of the low- and mid-rise buildings. The plan areas are symmetrical in both directions, therefore there will develop no torsional effect due to structural irregularities. Both buildings are modeled as reinforced concrete structures with a concrete strength of fc = 30 MPa and steel class fy = 355 MPa. To model beams and columns, a cubic elasto-plastic type 3D element was used. The bilinear elasto-plastic material model with kinematic strain hardening (stl1) was used for the steel reinforcement and rigid links modeling, and the uniaxial constant confinement concrete material model (conc2) was used for the concrete [4].

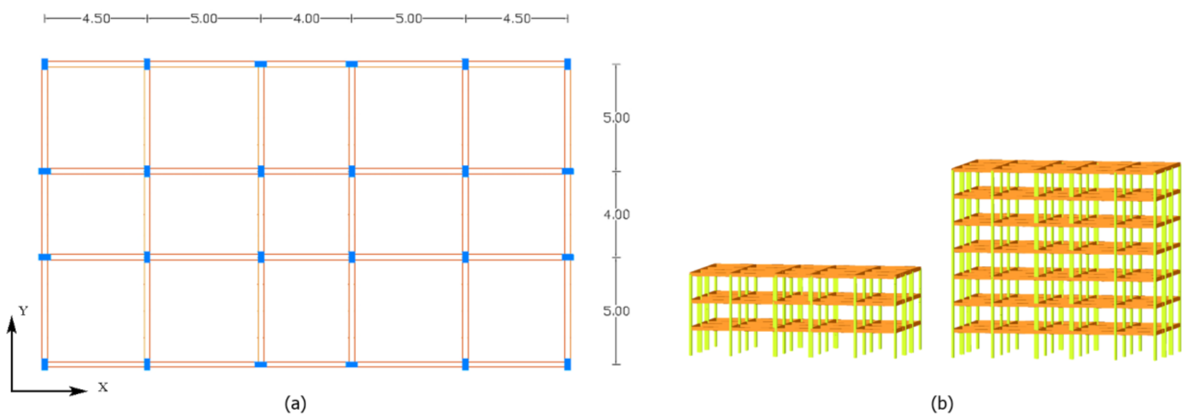

Figure 1: a) Structural plan (units in m); b) Elevation view of the frames

The low-rise model is composed of two types of columns and two types of beams, while the mid-rise building is composed of four types of columns and two types of beams as presented also in the Table 1. The element types change from each other according to their cross-sectional size and reinforcement. The infill walls have a thickness of 20 cm while slab thickness is considered 15 cm.

Table 1: Column and Beam details

Column Type	Column size	Longitudinal reinforcement (No. of bars / bar size)	Beam Type	Beam size	Longitudinal reinforcement (No. of bars / bar size)	Structure
Type 1	40 * 70 cm	12 Ø18	Type 1	30 * 50 cm	8 Ø14	7-Story
Type 2	40 * 70 cm	8 Ø18	Type 2	30 * 50 cm	8 Ø12	7-Story
Type 3	30 * 70 cm	12 Ø16	---	---	---	7-Story
Type 4	30 * 70 cm	8 Ø16	---	---	---	7-Story
Type 5	25 * 50 cm	8 Ø16	Type 3	25 * 40 cm	8 Ø12	3-Story
Type 6	25 * 50 cm	6 Ø16	Type 4	25 * 40 cm	6 Ø12	3-Story

Buildings are modeled both as moment resisting frames by employing Zeus-NL program, a platform which uses finite element analyses facility developed especially for earthquake engineering applications [4]. The software uses a fiber approach for the nonlinear analyses, monitoring the cross section into several fibers such as confined concrete fibers, unconfined concrete cover and reinforcement fiber. All the frame elements (Beams and Columns) are modeled in four sections for each member to increase the accuracy of the results. The self-weight of the structural members are calculated and assigned as distributed load in the horizontal elements and as point load in the vertical elements. Since there is no slab or infill wall member type in the Zeus-NL library, the self-weight, dead loads and live loads are calculated and assigned over the beams as distributed load. At the base nodes all the degrees of freedom are restrained.

To accelerate the modeling stage a new methodology is followed while modeling the structural elements. Using Microsoft excel 2013 the allocation of the steel bars in each member is calculated in common with reinforcement area aiming to minimize the calculation mistakes. On the other hand the structural elements are renamed with a proper “prefix” in an Excel sheet to adopt in Zeus-NL platform so the automatic allocation of the elements are generated by the software.

3 RESULTS

3.1 Pushover Analysis

Nonlinear static analysis is deployed using rectangular (uniform) and triangular (inversed triangular) load distribution considering the existence of gravity loads acting on buildings. For each structural frame two capacity curves are plotted using the results obtained from pushover analysis as shown in the Figure 2.

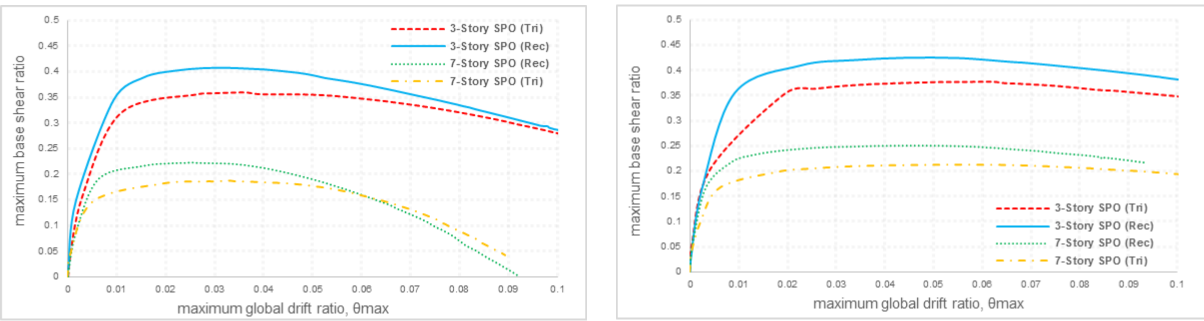

Figure 2: Comparison of pushover results for the 3-story and 7-story buildings (x-direction left, y-direction right)

The results show that rectangular load pattern curve show higher strength capacity than the triangular pattern in all frames considered in this study. As the story height increases, the lateral load bearing capacity of the structure decreases. Similar trends has been observed from past researchers [6-7].

3.2 Time-History Analysis

The non-linear dynamic time history analysis are performed using a set of twenty real ground motion records. The records are selected with different peak ground acceleration values showing no directivity influenced as shown in the Table 2. Ground motion records are taken from the Pacific Earthquake Engineering Research Centre (PEER) [8] and from the U.S Geological Survey (USGS) [9]. Each frame is subjected to twenty dynamic time history analyses. The obtained interstory drift results, are plotted as a function of building height.

Table 2: The suite of twenty ground motion records used for this study

No	Event	Year	Station	Ø°	Soil	M	R	PGA (g)
1	Erzincan	1992	Turkey, Erzincan	90	C	6.7	8.9	0.488
2	Imperial Valley	1979	Westmoreland Fire Station	90	C,D	6.5	15.1	0.074
3	Loma Prieta	1989	Agnews State Hospital	90	C,D	6.9	28.2	0.159
4	Loma Prieta	1989	Coyote Lake Dam Downstr.	285	B,D	6.9	22.3	0.179
5	Loma Prieta	1989	Hollister South & Pine	0	D	6.9	28.8	0.371
6	Loma Prieta	1989	Sunnyvale Colton Ave	270	C,D	6.9	28.8	0.207
7	Imperial Valley	1979	Chihuahua	282	C,D	6.5	28.7	0.254
8	Imperial Valley	1979	Plaster City	45	C,D	6.5	31.7	0.042
9	San Fernando	1971	LA, Hollywood Stor. Lot	180	C,D	6.6	21.2	0.174
10	Northridge	1994	LA, Hollywood Storage FF	360	C,D	6.7	25.5	0.358
11	San Fernando	1971	LA, Hollywood Stor. Lot	90	C,D	6.6	21.2	0.210
12	Spitak	1988	Armenia, Gukasian	90	C	6.8	36.1	0.207
13	Sup.erstition	1987	Wildlife Liquefaction Array	360	C,D	6.7	24.4	0.200
14	Tabas	1978	Iran, Dayhook	280	B	7.4	20.6	3.500
15	Loma Prieta	1989	WAHO	0	D	6.9	16.9	0.370
16	Loma Prieta	1989	WAHO	90	D	6.9	16.9	0.638
17	Northridge	1994	LA, Baldwin Hills	90	B	6.7	31.3	0.239
18	Friuli	1976	Italy, Tolmezo	270	B	6.5	20.2	0.345
19	Corinth	1981	Greece, Corinth	0	C	6.6	19.9	0.264
20	Kocaeli	1999	Turkey, Duzce	180	C	7.1	1.6	0.427

3.2.1 Interstory Drift

FEMA 356 [5] provides suggestions on the interstory drift values that should be considered while evaluating the structural performance. For the structures designed properly for seismic loadings and with sufficient member detailing defined as special moment frames (SMF), the suggested values for the interstory drift are 1% for the immediate occupancy (IO) performance level, 2% for the life safety (LS) and 4% for the collapse prevention (CP) performance level for the concrete frame structures. Based on the member detailing provisions, the spacing of the hoops near the plastic hinges zones is required to be less than ¼ of the distance between compression face of the RC section and tension reinforcement. On the other hand, the column width must be 20 times greater than the largest diameter of the longitudinal rebar. According to the specified rules, the case building cannot be considered as SMF, as the column width does not fulfill the required criteria. Table 3 presents a summary for all members which satisfy the abovementioned condition and for the others which does not satisfy.

Table 3. Column details and code provisions

Column Type	Column width	Largest Longitudinal reinforcement diameter	Satisfying code provisions FEMA 356	Structure
Type 1	40 mm	Ø18	**Yes**	7-Story
Type 2	40 mm	Ø18	**Yes**	7-Story
Type 3	30 mm	Ø16	**No**	7-Story
Type 4	30 mm	Ø16	**No**	7-Story
Type 5	25 mm	Ø16	**No**	3-Story
Type 6	25 mm	Ø16	**No**	3-Story

For intermediate moment frames it has been proposed that interstory drift limits should be reduced to 0.5% for the IO performance level, 1% for the LS and 2% for the CP performance level [10]. Based on this observation the limit state for the current study is set to 1% for the LS performance level while evaluating the buildings performance.

Results are presented for both buildings in x- and y-direction in Table 4. Each case is illustrated with ground motion records, which are presented on the left of the table. Median values are calculated and presented at the end of the table. The interstory drift results which exceed the median value, are highlighted for each earthquake. In this way it can be easier to differentiate the most risky earthquakes among those we have selected. The capacity curves for both buildings are obtained by nonlinear static pushover analyses. Nonlinear time history analysis is used to estimate the displacement demands of the representative building models.

Table 4: Displacement demands for 3- and 7-story RC buildings

No	Earthquake (Event, Station)	3-Story Building		7-Story Building	
		x-direction	y-direction	x-direction	y-
		Interstory Drift (%)		Interstory Drift (%)	
1	Erzincan (Turkey)	2.650	2.144	4.545	3.683
2	Imperial Valley Westmoreland Fire	0.414	0.405	0.400	0.370
3	Loma Prieta Agnews State Hospital	0.423	0.497	0.497	0.517
4	Loma Prieta Coyote Lake Dam	0.739	0.650	0.526	0.469
5	Loma Prieta Hollister South & Pine	3.292	3.467	1.957	2.038
6	Loma Prieta Sunnyvale Colton Ave	0.878	0.855	1.428	1.259
7	Imperial Valley Chihuahua	1.624	1.561	1.097	0.943
8	Imperial Valley Plaster City	0.117	0.139	0.094	0.110
9	San Fernando LA, Hollywood Stor. Lot	0.368	0.376	0.407	0.615
10	Northridge LA, Hollywood Storage FF	1.611	1.349	1.076	0.898
11	San Fernando LA, Hollywood Stor. Lot	0.735	0.733	1.430	1.207
12	Spitak Armenia, Gukasian	0.988	1.191	1.321	1.168
13	Superstition Hills Wildlife Liquefaction	1.935	1.527	1.721	1.719
14	Tabas Iran, Dayhook	1.294	0.696	0.965	0.719
15	Loma Prieta WAHO	1.733	1.666	1.026	0.969
16	Loma Prieta WAHO	2.129	2.148	1.388	0.966
17	Northridge LA, Baldwin Hills	0.073	0.079	0.072	0.047
18	Friuli Italy, Tolmezo	0.802	0.925	0.712	0.512
19	Corinth (Greece)	0.857	1.084	1.040	0.604
20	Kocaeli (Turkey), Duzce	2.958	3.083	2.527	2.128
--	**Median**	0.933	0.933	1.058	0.906

As shown from the Table 4, around 50% of earthquake records forced both structures to exceed their median values in x- and y-direction. Interstory drift results are presented also graphically for both buildings in Figures 3 and 4 together with life safety (LS) performance level. As shown from the figures, some ground motion records express excessive displacement demands on both structures, however the median values do not exceed the limit level in none of frames. In addition it is observed that the maximum interstory drift values are located at the 2nd and 3rd story level for low-rise and mid-rise buildings respectively.

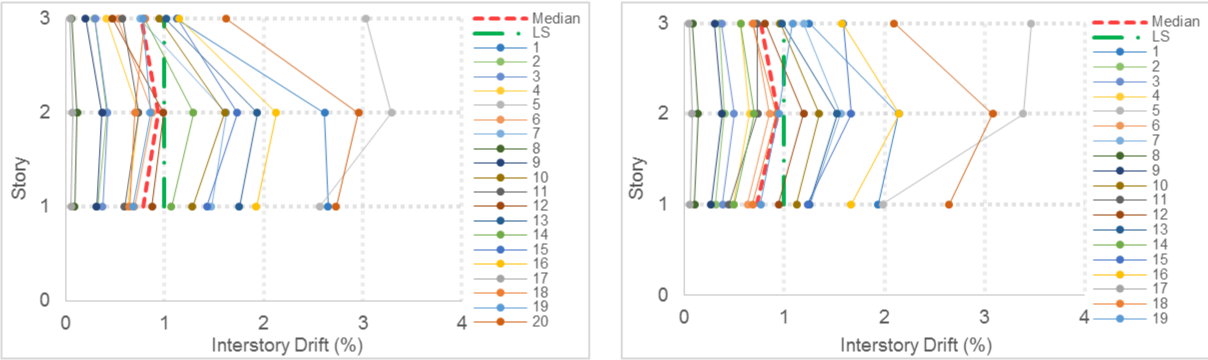

Figure 3: Interstory drifts for the 3-story building (x-direction left, y-direction right)

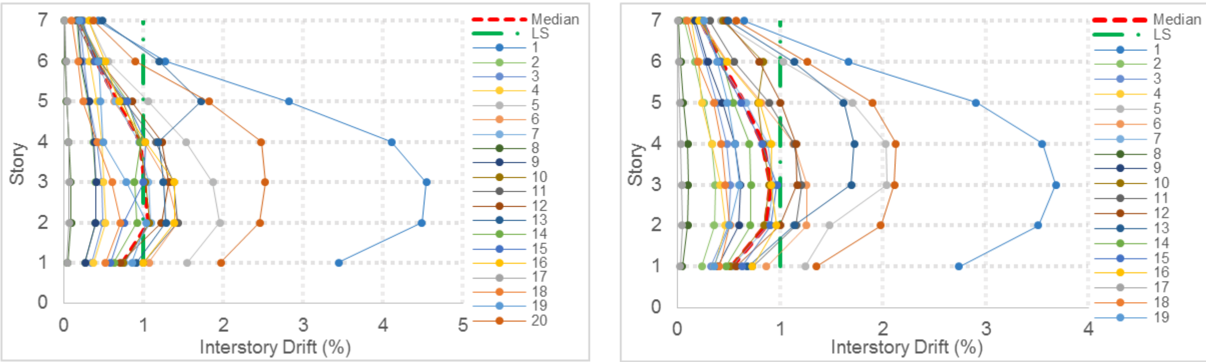

Figure 4: Interstory drifts for the 7-story building (x-direction left, y-direction right)

3.2.2 Comparison of Pushover and Time History Analyses results

In addition pushover curves are plotted in 2D graphs as maximum global drift ratio and maximum base shear ratio. Figures 5 and 6 show the plotted time history analysis results and the pushover curves for each frame. The dynamic results are represented by data points, each representative of one ground motion record as shown in Table 2. On the other hand, pushover curves are represented by curves for both rectangular and triangular patterns. The median of the non-linear dynamic values is shown with a plus "+" sign in the same graph with pushover curves.

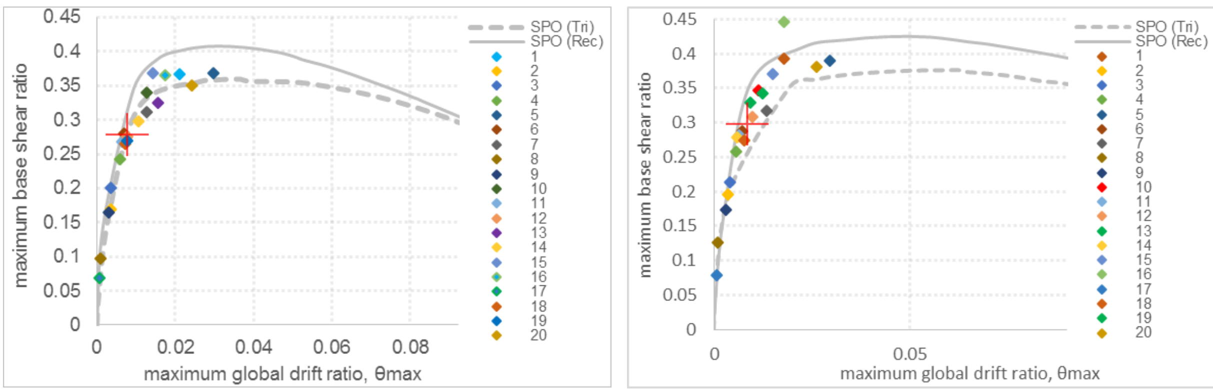

Figure 5: Comparison of pushover and time-history results for the 3-story building (x-direction left, y-direction right)

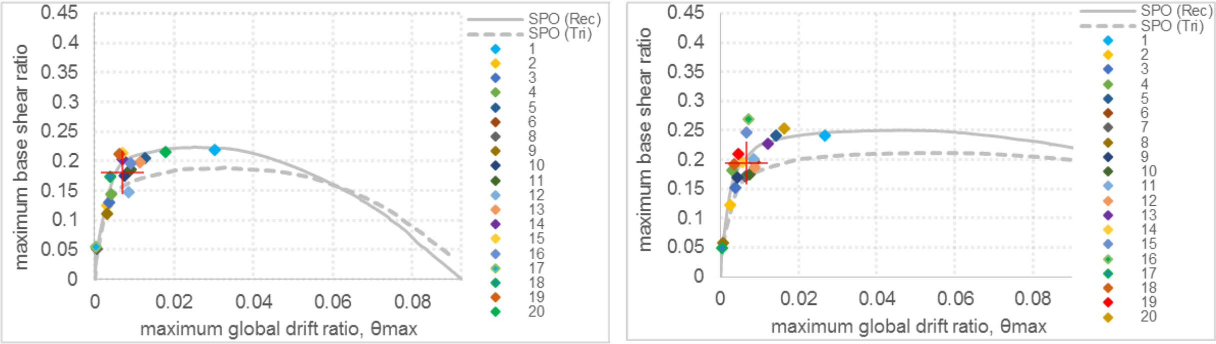

Figure 6: Comparison of pushover and time-history results for the 7-story building (x-direction left, y-direction right)

It can be easily observed that the results from the time history analyses generally follow the pushover curves. Most of the data points are located between two static pushover curves. A better indicator is also the median of the time history values, which falls between two pushover curves for all frames. On the other hand, a few dynamic analyses values exceeded the pushover curves with rectangular pattern, showing a higher values of shear demand. This phenomena is usually observed in the 7-story frames.

In the x-direction of the 3-story frame, most of the dynamic analyses results follow the pushover curve with triangular pattern, while the median is located closely to this curve. This shows the overestimation of the pushover curve with rectangular pattern alongside the dynamic results.

4 CONCLUSIONS

This study investigates the capacity assessment of a low and mid-rise reinforced concrete building using nonlinear static and dynamic analysis. Based on the values gathered from the analyses, the following conclusions could be drawn:

- From static pushover analyses it is observed that SPO curve which belongs to rectangular loading pattern shows higher strength capacity than the curve which represents the triangular loading pattern. The behavior is observed for all frames considered in this study. This phenomena is detected also in other studies [6-7]. Thus, it can be concluded that the rectangular loading pattern in pushover analysis dominates from the triangular pattern.

- The maximum interstory drift for each structure in both directions were compared. The results show that 50% of the interstory drifts values exceeded the life safety performance level for the low and mid-rise buildings. However it’s important to note that median gives satisfactory results since does not go beyond the limit state.

- It is observed that for both buildings the highest interstory drift results are located at the second story level and for the mid-rise building just a few at the third story level. The higher drift values correspond near the region where reductions in column reinforcements are observed. Therefore, it can be concluded that reductions in column size and reinforcement may attract the accumulation of drift demands at this level of the structure.

- ❖ The interstory drift results are compared with their median values to estimate the most hazardous earthquakes for both structures. In the end it is concluded that the most destructive earthquake records, which simultaneously forced both structures in x- and y-directions are: Erzincan, Loma Prieta Hollister South and Pine, Superstition Hills Wild Life Liquefaction Array and Düzce.

- ❖ Static pushover curves with rectangular and triangular pattern were plotted together with dynamic time history analysis results in the same graph. It is observed that for most of the cases, dynamic analysis results are located between two pushover curves. However, it is important to mention that in the three-story building most of the dynamic analysis results fall in the curve with triangular pattern, while in the seven-story structure mostly in the curve with rectangular pattern. This is illustrated also by the median value for both building in x- and y-directions.

REFERENCES

[1] Elnashai, A.S.; Sarno L.D.: Fundamentals of Earthquake Engineering, London: Wiley, 2008

[2] Bilgin, H.: Fragility-based assessment of public buildings in Turkey, Engineering Structures, vol. 56, pp. 1283-1294, 2013

[3] Mehmet, I.; Bilgin, H.; Özmen, H. B.: Orta Yükseklikteki Betonarme Binalarin Deprem Performanslarinin Afet Yönetmeliğine Göre Tayini, Pamukkale Üniversitesi Mühendislik Bilimleri Dergisi, vol. 13, pp. 81-89, 2007

[4] Elnashai, A.S.: Zeus NL - A System for Inelastic Analysis of Structures, Urbana–Champaign: Creating a Multi-hazard Approach to Engineering, 2002

[5] FEMA, 356.: Prestandard and Commentary for The Seismic Rehabilitation of Buildings, Federal emergency management agency, Washington, 2000

[6] Bento R.; Falcão S.; Rodrigues F.: Non-Linear Static Procedures In Performance Based Seismic Design, 13th World Conference on Earthquake Engineering, Vancouver, B.C., Canada, 2004

[7] Frangu I.; Bilgin H.: Seismic Performance Evaluation of a RC Public Building: Case Study From Albanian Practice, Epoka University, Tirana, 2013

[8] PEER.: Pacific Earthquake Engineering Research Center, University of California, Berkeley, [Online]. Available: http://peer.berkeley.edu/. [Accessed 13 July 2016]

[9] USGS.: U.S. Geological Survey, U.S. Department of the Interior, [Online]. Available: https://www.usgs.gov/. [Accessed 14 July 2016]

[10] Ramamoorthy S.; Gardoni P.; Bravcci J.: Probabilistic Demand Models and Fragility Curves for Reinforced Concrete Frames, Journal of Structural Engineering, pp. 1563-1572, 2006