Seismic Capacity Assessment Of Existing Irregular Reinforced Concrete (RC) Buildings By An Adaptive Three - Dimensional Pushover Procedure

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ABSTRACT:
Structural capacity assessment of an existing structure under an earthquake excitation is a phenomenon in earthquake engineering. Since torsion is assessed by the fundamental mode shape under an earthquake excitation, most conventional pushover programs are usually designed for two dimensionally analysis neglecting torsional effects. In this study, the aforementioned three-dimensional adaptive pushover procedure, which is represented in PEER 2011, is implemented on three existing irregular RC buildings, one of which is the SPEAR building. The other two buildings are selected from the database of Istanbul Earthquake Master Plan 2003 project with in a concept of Zeytinburnu Pilot Region, for previously determined 7 earthquake records. As a result, it can be stated that, the conventional pushover analysis overestimates the capacity results of irregular RC buildings. Studies have shown that adaptive results of the drift profiles are much closer to the nonlinear time history analysis results for these types of buildings.

Keywords: 3-D adaptive pushover, capacity assessment, existing RC buildings, torsional effects

1. INTRODUCTION

In recent years, performance-based design methods which rely on nonlinear static analysis procedures have found wide use among the structural engineers. As a well-known fact, nonlinear static procedures are based on converting the multi degree of freedom system (MDOF) to an equivalent single degree of freedom system (SDOF). They produce estimates of the maximum displacement, story drifts and other structural components. Structural capacity is determined by the pushover or capacity curve that was used to generate the equivalent SDOF model. In pushover analysis the static forces are distributed along the height of the structure until a predefined target displacement is reached. If the lateral load pattern is kept constant through the analysis, the method is called as conventional pushover and if the load pattern is constantly updated through each analysis step in the inelastic range, then the analysis method is called as adaptive pushover method.

Modern standards and guidelines, such as FEMA 273, FEMA 356, ATC-40, Eurocode 8, FEMA 440 (ATC-55) and ASCE 41 proposed solution methods to determine the inelastic performance under an earthquake excitation. In fact, all the mentioned procedures are based on the assumption that the inelastic response of a multi degree of freedom system (MDOF) may be determined in terms of an equivalent single degree of freedom system (SDOF). Estimation of the target displacement and determination of the height-wise lateral force distribution are the key points of nonlinear static analysis. Both quantities are based on the assumption that the structure vibrates predominantly by the fundamental mode and the mode shape remains the same after yielding occurs.

Originally, nonlinear static methods are limited to planar models. However, buildings are often asymmetric and irregular in plan due to architectural reasons. When plan-asymmetry of a building is the issue, then the torsional effects should be taken into account. Recent studies showed that, most of the structural damage during an earthquake excitation is due to plan irregularities, such as asymmetric distributed mass, stiffness and strength. It is shown by many researchers that, conventional pushover analysis procedures underestimate the seismic torsional response of a plan-asymmetric building. It is admitted that, 3-dimensionel models should be used instead of planar frame models while determining the seismic response of plan-asymmetric buildings.

In order to overcome the mentioned deficiencies, researchers developed different analysis strategies. Recent studies that rely on adaptive pushover procedures, update the load pattern at instantaneous states of inelasticity. Shakeri et al. (2010) proposed a Story Shear Based Adaptive Procedure (SSAP) for nonlinear static analysis. It is based on the story shear forces. According to the proposed procedure, reversal of sign changes and higher mode effects are taken into account. At each step, the load pattern is derived from the modal story shears of the instantaneous step. Using the energy concept, multi degree of freedom system (MDF) is converted to an equivalent single degree of freedom system (SDOF), and the target displacement is determined. However, adaptive pushover procedures are still need to be developed. Irregularity and torsional effects should be considered in the adaptive analysis.

2. SEISMIC EVALUATION OF EXISTING IRREGULAR BUILDINGS

In this study, it is aimed to show the importance of torsional effects. For this a computer code which is capable of three-dimensional modelling is developed basing on OpenSees (McKenna, et al., 2006) modules and is validated by a well-known structural analysis program, Perform 3-D (Computers & Engineering, 2004).

The aforementioned three-dimensional adaptive pushover procedure, which is represented in PEER 2011 and 2012 (Oyguc and Boduroglu), is implemented on three existing irregular reinforced concrete (RC) buildings. The input signal is consisted of seven semi-artificial series obtained by the modification of the North-South (NS) and West-East (WE) components of Herceg-Novi record of 1979 Montenegro earthquake, given in Table 2.1. Each of the records is modified to be compatible with the EC8 Type 1 design spectrum, soil Type C and 5% damping. They are normalized to peak ground acceleration (PGA) of 1.0g on rock site, which means that PGA is 1.15g on soil type C.

Table 2.1. List of the selected earthquakes

No	Earthquakes	Stations	PGA (g)
1	Montenegro 1979	Ulcinj	1.15
2	Montenegro 1979	Herceg Novi	1.15
3	Friuli 1976	Tolmezzo	1.15
4	Imperial Valley 1940	El Centro Array 9	1.15
5	Kalamata 1986	Prefecture	1.15
6	Loma Prieta 1989	Capitola	1.15
7	Imperial Valley 1979	Bonds Corner	1.15

Two of the buildings are selected from the database of Istanbul Earthquake Master Plan 2003 project with in a concept of Zeytinburnu Pilot Region and named as Z1 and Z2. The other selected building is a well-known test building called SPEAR (Fardis and Negro, 2005) building. Pseudo-dynamic test results of SPEAR are also compared with the drift profiles that are gathered from the adaptive pushover and nonlinear dynamic analysis.

The 3-D software package used in the present work is called as “NASAP” coded by Ozcitak and the narrator in 2010. This is a tool for finite element analysis of structural elements, meaning “Nonlinear Adaptive Structural Analysis Program”. It has friendly user menus. The cross-sections of the buildings are determined using Xtract (Thao, 2006).

Since, Goel and Chopra (2005) stated that, CQC will give better estimates when the modal responses are closer, modal story-shear quantities are combined using CQC. The initial target displacement values are calculated by the formula given in FEMA 440. After the analysis completed the exact target displacement values are interchanged with the initial ones. Second order effects are omitted in the implemented analysis. Lateral loads are applied through the center of mass. The gathered pushover curves represent the roof displacements at the center of mass versus total base shear in each direction.

2.1. SPEAR Building

The SPEAR structure was designed by Fardis in 2002. It is a representative of an existing irregular three-story reinforced concrete (RC) building constructed in Greece. It has been designed using the design code criteria in Greece between 1954 and 1995, with the knowledge and materials of early 70's for only gravity loads.

The 3-dimensional model view of the SPEAR building is given in Figure 2.1. The story height is 3 m, with 2.5 m clear height of columns between the beams. The specified design strength of concrete is fc=25 MPa, and the design yield strength of reinforcement is f_y=320 MPa. Design gravity loads on slabs are 0.5 kN/m^2 for dead loads and 2 kN/m^2 for live loads. Slab thickness is 150 mm and total beam depth is 500 mm. The slab is reinforced with 8 mm bars at 200 mm intervals. Columns longitudinal reinforcement is composed of 12 mm plain bars. Column stirrups are 8 mm plain bars, closed with 90° hooks with 250 mm intervals. Beam longitudinal reinforcement is designed as two 12 mm bars at the top, anchored with 180° hooks at the end of the column. The bottom beam reinforcement consists of two 12 mm bars anchored at the end of the column with 180° hooks. Beam stirrups are 8 mm bars at 200 mm intervals, anchored with 90° hooks.

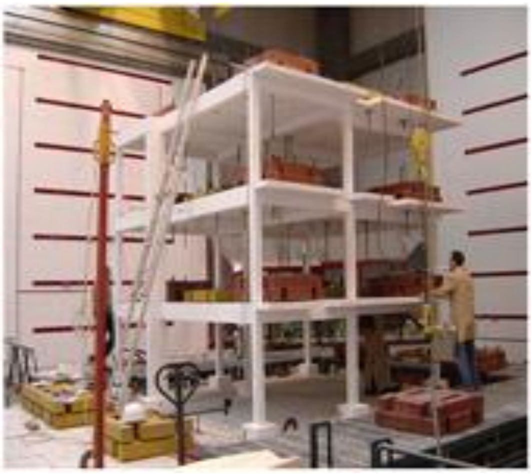


Figure 2.1. Model view of the SPEAR building

Story masses and the modulus of inertia of SPEAR is given in Table 2.2. The calculated modal participation factors and the period values are given in Table 2.3 and Table 2.4 respectively.

Table 1.2. Center of Mass and Mass properties of SPEAR building

	Centre of Mass (m)	Mass (KNs^2/m)	Modulus of Inertia [KNm^2/(m/s^2)]
FLOOR 1&2	X = 4.53	66.57	1249
	Y = 5.29		
ROOF	X = 4.57	64.43	1170
	Y = 5.33		

Table 2. 2. Calculated Modal Participation Factors

	X Direction	Y Direction	Around Z Direction
1	12.02	-3.14	-20.53
2	4.76	11.07	21.50
3	2.71	-5.56	52.46
4	3.83	-0.86	-6.38
5	1.55	3.53	10.36
6	-0.22	3.02	-14.94

Table 2.3. Period and Mass ratio values for both directions

Mode	Period (s)	Long. M. Ratio	Trans. M. Ratio	Torsional M. Ratio
1	0.61	0.74	0.05	0.11
2	0.55	0.11	0.62	0.11
3	0.44	0.03	0.15	0.67
4	0.21	0.07	0.01	0.01
5	0.17	0.01	0.06	0.02
6	0.14	0	0.04	0.05

Load patterns for both the longitudinal and the transverse directions are determined implementing the stated procedures. The gathered load pattern, which is used in the adaptive pushover analyses are given in Figure 2.2 and Figure 2.3 for the longitudinal and transverse directions respectively.

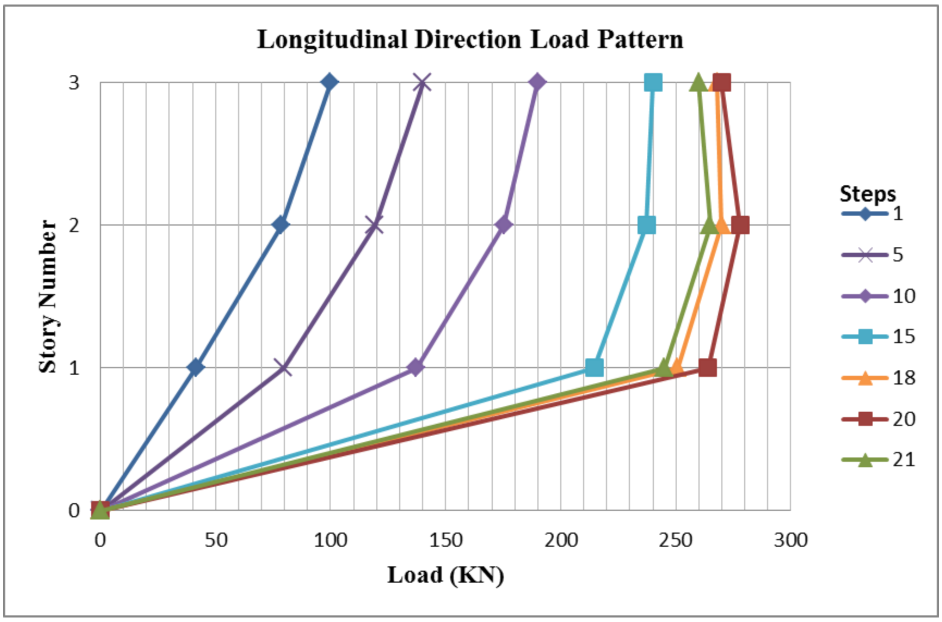


Figure 2.2. Longitudinal direction load pattern.

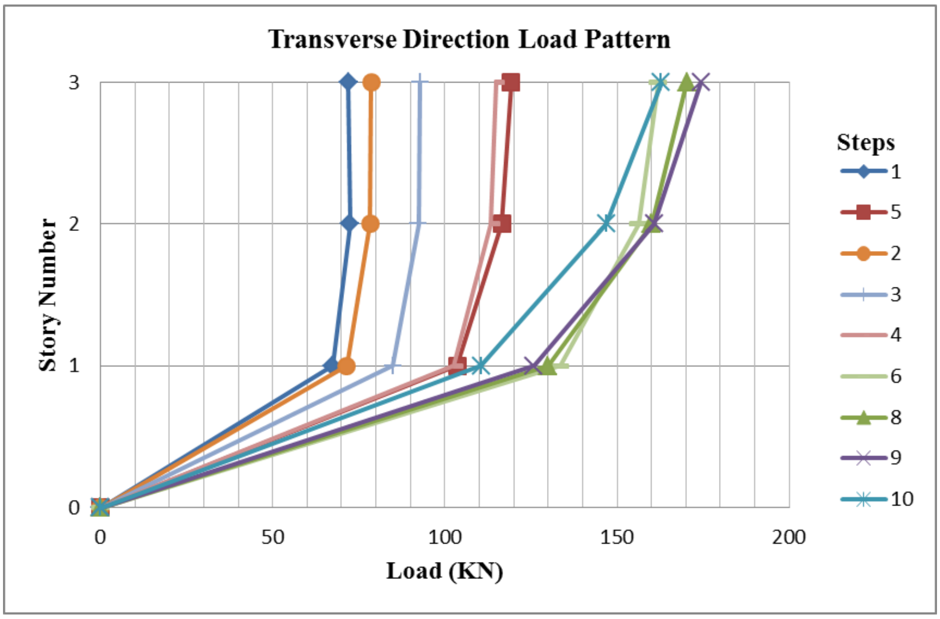


Figure 2.3. Transverse direction load pattern.

The target displacement values are calculated using FEMA for the longitudinal direction and the transverse direction. The detailed results of the 3-dimensional adaptive pushover analysis are given in PEER Report (Oyguc and Boduroglu, 2011). The drift comparisons and the pushover curves are represented in the same report for both directions. It is also stated in the report that, for this specific irregular concrete building model, the conventional pushover procedures overestimate the capacity approximately by %20.

2.2. Zeytinburnu Pilot Region Z1 and Z2 buildings

As stated previously, the data of the Z1 and Z2 buildings are gathered from Istanbul Earthquake Master Plan 2003 project. Z1 building is a 5-storey irregular reinforced concrete building whereas Z2 is a 4-storey one. The 3-dimensional models of the mentioned buildings are shown in Figure 2.4.

Gravity loads for the analytical models are calculated by summing parts of the design gravity loads on slabs and the self-weight of the structure itself. Total dead loads and 30% of live loads are taken in the analysis. 0.5 kN/m^2 is assumed for slabs, and 2 kN/m^2 for live loads. As stated before, the concrete self-weight is taken as 25 kN/m^3. The mass is calculated by dividing the gravity loads by the acceleration (9807 mm/sec^2). Calculated gravity loads are distributed to beams and columns. Gravity loads on slabs and self-weight of slabs are distributed to the nearest beams. The mass properties, the calculated modal participation factors, the periods and the mass ratio values for both directions are determined in Table 2.5 for both of the mentioned buildings.

The specified design strength of concrete for Z1 building is fc=10 MPa, and the design yield strength of reinforcement is f_y=220 MPa; whereas fc=13 MPa and f_y=220 MP for the Z2 building. Slab thickness is 100 mm for Z1 and Z2 buildings. Total beam depth is 500 mm for Z1 building and 550 mm for Z2 building. Columns longitudinal reinforcement is composed of 14 mm plain bars for Z1 building and 12 mm for Z2 building. Column stirrups are 8 mm plain bars, closed with 90° hooks with 350 mm intervals for Z1 building and 300 mm for Z2 building. Beam longitudinal reinforcement is designed as two 12 mm bars at the top, anchored with 180° hooks at the end of the column for both of the buildings. Both buildings bottom beam reinforcement consists of two 12 mm bars anchored at the end of the column with 180° hooks. Beam stirrups are 8 mm bars at 200 mm intervals, anchored with 90° hooks.

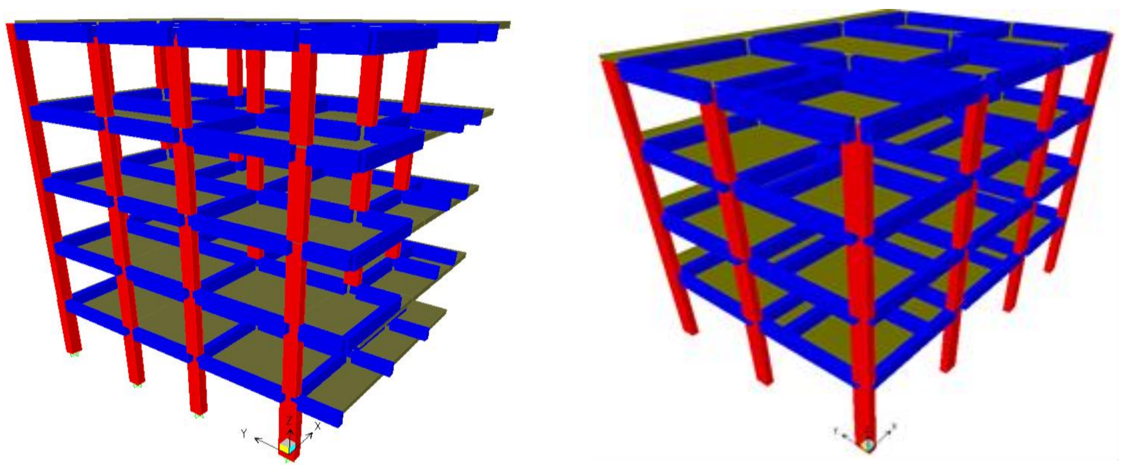


Figure 2.4. 3-Dimensional Models of Z1 and Z2 buildings

Table 2. 4. Mass, modal properties and periods of Z1 and Z2 buildings

Building	Story Number	Mass (KNs^2/m)	Modal Participation Factors		Period (sec)	Modal Mass Factors	
			X	Y		X	Y
Z1	1	109.27	20.33	20.13	0.99	81.44	79.90
	2	106.75	-7.46	-7.54	0.32	10.96	11.22
	3	109.38	4.71	5.06	0.19	4.37	5.06
	4	109.41	-3.36	-3.64	0.14	2.22	2.61
	5	70.98	2.23	2.46	0.12	0.98	1.20
Z2	1	116.07	20.87	20.80	0.75	91.30	90.65
	2	117.86	5.82	-5.99	0.23	7.11	7.53
	3	118.90	2.53	2.69	0.13	1.34	1.52
	4	124.45	-1.05	-1.15	0.09	0.23	0.27

As stated before, the cross-sections of the buildings are determined using Xtract (Thao, 2006). Rigid diaphragm action is considered at the floor levels. The target displacement values are calculated using FEMA-356 as 0.24 m in the longitudinal direction and 0.25 m in the transverse direction for Z1 building. The target value of Z2 building is calculated as 0.12 m in the longitudinal direction and 0.12 m in the transverse direction. The determined 3-dimensional adaptive and conventional pushover curves in X and Y directions are represented in Figure 2.5 for both buildings.

It is obvious that, lower pushover graphs are obtained when the mentioned adaptive pushover procedure is implemented on the buildings for two directions. In order to validate the results, using the previously determined earthquakes in Table 2.1, time history analyses are conducted. The calculated drift results are checked with the adaptive pushover results. The determined drift comparisons of Z1 and Z2 buildings for both X and Y directions are given in Figure 2.6.

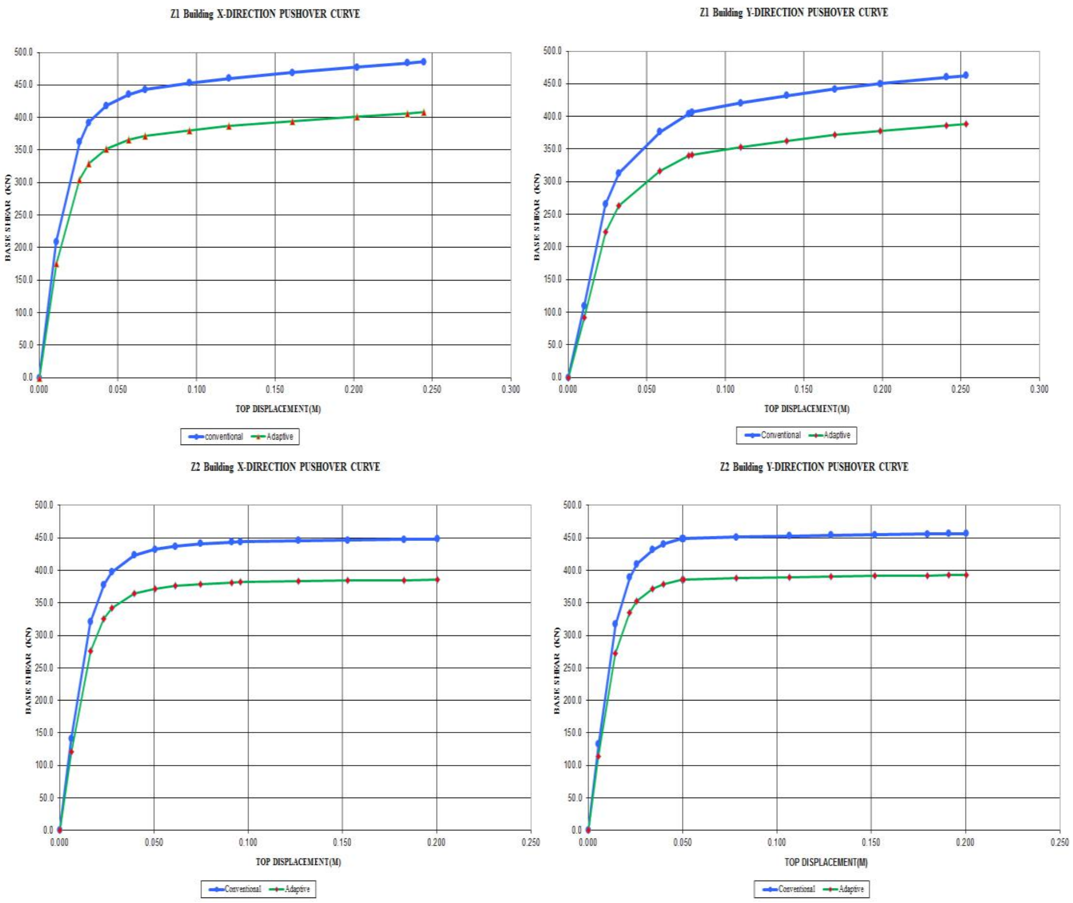
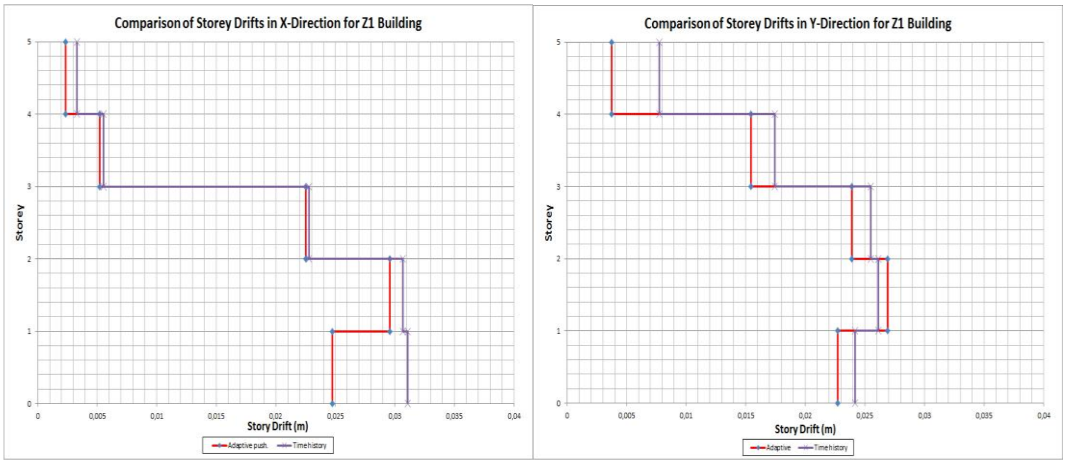


Figure 2.5. 3-Dimensional Models of Z1 and Z2 buildings



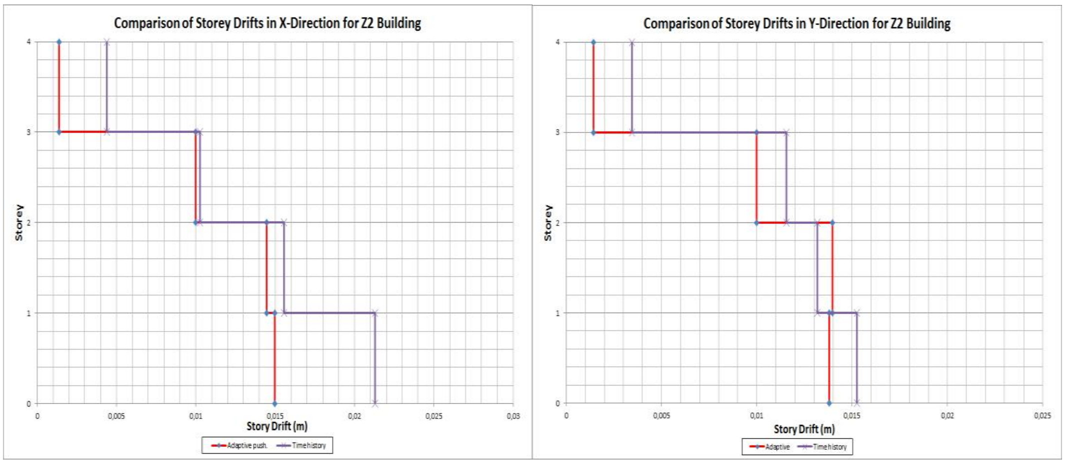


Figure 2.6. Drift comparisons of Z1 and Z2 buildings with time history results

3. CONCLUSIONS

The aforementioned adaptive pushover procedure has been adapted to three existing irregular buildings. P-Δ effects are omitted during the analysis. The results are compared for both directions with adaptive and conventional pushovers. Seismic capacity was evaluated by the inelastic dynamic analysis. Seven artificial recorded bidirectional ground motions were scaled to match the EC8 spectra for soil type C.

As mentioned before, it is a fact that the conventional pushover analysis overestimates the results. When Figure 2.5 is investigated, it will be seen that, the adaptive pushover results are approximately 16% lower than the conventional ones when the irregular buildings are the issue. This misleads the structural engineering during structural modeling.

It can be stated that, adaptive procedure is more accurate than the conventional one while determining the capacity and the drift profiles of the irregular structures. Figure 2.6 shows that the calculated drift results of the 3-dimensional adaptive analyses are adequate with the time history results. This should be added that; the accuracy in drift is increased when the higher modes are significant as in the upper stories.

REFERENCES

ATC 40 (1996). “Seismic Evaluation and Retrofit of Concrete Buildings”, Applied Technology Council, Washington, D.C.

ASCE 41 (2007). “Seismic Rehabilitation and Evaluation of Existing Buildings”, ASCE Standard No. ASCE/SEI 41-06, American Society of Civil Engineers, Reston, VA, 2007.

Computers & Engineering, (2004). “Nonlinear Analysis and Performance Assessment for 3-Dimensional Structures”, http://www.comp-engineering.com/PERFORM3D/.

Eurocode 8 (2004). “Design of structures for earthquake resistance”, British Standards, Washington, D.C.

Fardis, M., Negro, P. A., (2005). “*Seismic Performance Assessment and Rehabilitation of Existing Buildings (SPEAR)*”. Proceedings of the International Workshop, An event to honour the memory of Prof. Jean Donea, Institute for the Protection and Security of the Citizen European Laboratory for Structural Assessment (ELSA) **I-21020** Ispra, Italy, 4-5 April.

FEMA 273 (1997). “NEHRP Guidelines For The Seismic Rehabilitation Of Buildings”, Federal Emergency Management Agency, Washington, D.C.

FEMA 356 (2000). “Prestandard and commentary for the seismic rehabilitation of buildings”, Federal Emergency Management Agency, Washington, D.C.

FEMA 440 (2005). “Improvement of Nonlinear Static Seismic Analysis Procedures”, Federal Emergency Management Agency, Washington, D.C.

Goel, R. K., and Chopra, A. K., (2005). “Role of Higher-“Mode” Pushover Analyses in Seismic Analysis of Buildings”. *Earthquake Spectra*, **21**, no. 4, pp. 1027-1041.
Istanbul Earthquake Master Plan (2003). Istanbul Greater Municipality, Istanbul.
McKenna, F., Fenves, G. L., Filippou, F. C., & Mazzoni, S. (2006). Open system for earthquake engineering simulation (OpenSees). Pacific Earthquake Engineering Research Center, Berkeley, http://opensees.berkeley.edu/OpenSees/home/about.php.
McKenna, F., Fenves, G. L., Filippou, F. C., & Mazzoni, S. (2006). OpenSees Command Language Manual.
Oyguç, R. A., Boduroğlu, H., (2011). “*Seismic Risk Management in Urban Areas*”. Proceedings of a U.S.-Iran-Turkey Seismic Workshop, December 14-16, 2010, Istanbul, Turkey, *PEER Report* **2011/07**, pp 303-314.
Oyguç, R. A., Boduroğlu, H., (2012). “*The International Workshop: "Role of research infrastructures in seismic rehabilitation*" 8-9 February, 2012, Istanbul, Turkey.
Shakeri, K., Shayanfar, M. A., and Kabeyasawa, T., (2010). “A story shear-based adaptive pushover procedure for estimating seismic demands of buildings”. *Engineering Structures*, **32**, no. 1, pp. 174-183.
Thao, Y., (2006). XTRACT, Imbsen, from http://www.imbsen.com/xtract.htm.