Static and Dynamic Analyses of Asymmetric Reinforced Concrete Frames

15 WCEE
LISBOA 2012

T. Mahdi
Building and Housing Research Centre, Tehran, Iran

V. Soltangharaie
Faculty of Building and Housing, Tehran, Iran

SUMMARY:

In this paper, the seismic behavior of three concrete intermediate moment-resisting space frames with unsymmetrical plan in five, seven and ten stories are evaluated. In each of these three cases, plan configurations of the structure contain reentrant corners. Nonlinear static and linear dynamic procedures have been used to analyze these structures. To measure the accuracy of these two methods, the non-linear dynamic analysis has been used. Although the differences between the results of these two methods with the nonlinear dynamic procedure are quite wide, the linear dynamic analysis has shown slightly better results than nonlinear static analysis.

Keywords: RC frame, irregular building, pushover, nonlinear time history, linear time history

1. INTRODUCTION

The selection of a suitable procedure to evaluate performance of structures under seismic loads is one of the most sensitive issues that structural engineers face. This would be especially important when dealing with irregular structures since the wrong choice of a procedure would lead to results that are far away from the correct solution. One of the most common types of irregularities that found in most buildings is the plan irregularities. The existence of an asymmetry in the plan is usually leading to an increase in stresses of certain elements that consequently results in a significant destruction.

Several methodologies involving nonlinear pushover analysis using an invariant height-wise lateral force distribution, such as the FEMA356 (ASCE 2000), FEMA440 (ATC 2005), and ASCE 41 (ASCE 2007), have routinely been used in structural engineering practice for estimation of global and local structural deformations. As shown by many authors (Kunnath & Kalkan, 2004; Themelis, 2008), these methods have proven their superiority over existing elastic force-based procedures. For structures responding primarily in the first mode, they represent reliable options to estimate inelastic demands. However, many studies have shown that conventional pushover methods are ineffective in dealing with some types of plan irregularities. Based on this and by using three-dimensional models, new pushover methods are proposed (Moghadam & Tso, 2000; Chopra & Goel, 2004; Fajfar et. al, 2005; Kalkan & Kunnath, 2006).

To evaluate the seismic behavior of complex tall asymmetric buildings with significant higher mode effects, the nonlinear dynamic analysis methods generally provide more realistic models of structural response and, thereby, provide more reliable assessment of earthquake performance than other methods. However, this method is not feasible for complex and large buildings. Thus, it is the purpose of this paper to strike a balance between practicality on one hand and accuracy on the other. The most logical alternatives in this respect are the nonlinear static analysis (pushover) and the linear dynamic analysis. The pushover analysis can be an effective design tool to investigate aspects of the analysis model and the nonlinear response that are difficult to do by nonlinear dynamic analysis (Deierlein et. al., 2010).

2. DESCRIPTION OF ANALYZED BUILDING

In this paper, five, seven and ten stories residential buildings are considered. In each of these three cases, plan configurations of the structure contain reentrant corners, where both projections of the structure beyond a reentrant corner are greater than 33 percent of the plan dimension of the structure in the given direction, as shown in Figure 2.1. However, in all these cases, the differences between center of mass and rigidity are less than 2.4% of the corresponding dimension of the building. The structural system used for these buildings is taken as concrete intermediate moment-resisting space frames (IMRSF). Soil type is considered as type two that is equivalent to type (B) in the USGS classification and a soil profile (A) spectrum according to Eurocode classification (CEN, 2004). Furthermore, the peak ground acceleration is assumed equal to 0.35 g. which corresponds to that used for very high seismic zone in the Seismic Iranian Code of Practice (BHRC, 2005). The 28-day strength of concrete, yield strength of steel, elasticity modules of concrete and elasticity modules of steel are 24.5 x 10^6, 392.4 x 10^6, 24.5 x 10^9 and 196.2 x 10^9 N/m^2 respectively. All the floors are considered to be subjected to dead loads equal to 5591.7 N/m^2 and to live loads equal to 1962 N/m^2. At the roof, loads of 5689.8 N/m2 and 1471.5 N/m^2 are considered respectively.

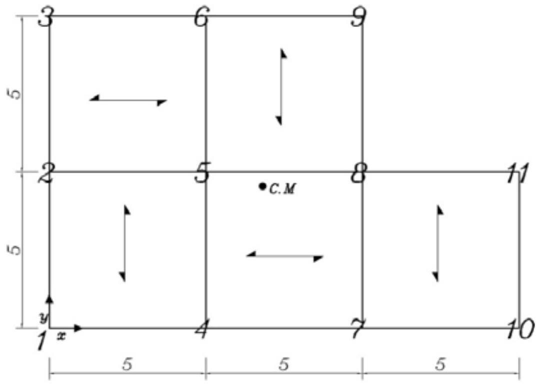


Figure 2.1. A typical plan (all dimensions in meters).

3. BUILDING MODELING

The analyses of the three space frames have been performed using the finite element software, SAP 2000, Version 14.1 (computer & Structures, 2009). All these three buildings have been analyzed according to the Iranian Code of Practice for Seismic Resistant Design of Buildings (BHRC, 2005). Each of the three structures have been designed by considering an accidental eccentricity equal to 5% of the relevant orthogonal plan dimension of the building and the sections of the structural elements have been controlled by ACI (ACI, 2005). A sample of the 7-story frame is shown in Figure 3.1.

In all dynamic analyses, the buildings considered have been subjected to seven selected records of the 1994 Northridge Earthquakes (PEER website), as given by Table 3.1.

The modal properties, in terms of periods and effective modal mass percentages, of the 10-story building are reported in Table 3.2. The results show that the main modes of the structure are transitional ones.

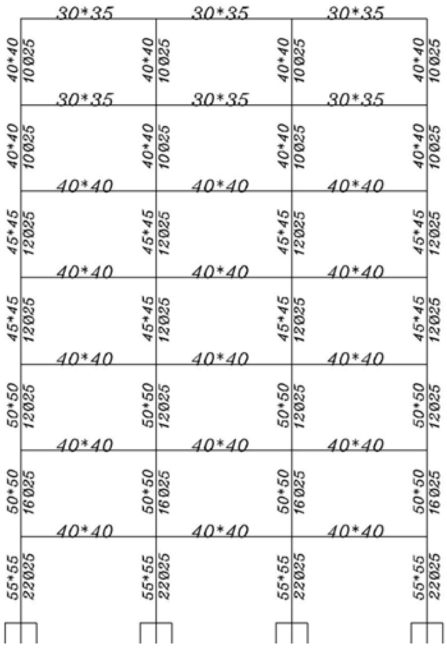


Figure 3.1. Designed elements of the 2-5-8-11 frame in the 7-story building
(Dimensions of frame's members are in centimetres and dimensions of steel reinforcements are in millimetres)

Table 3.1. The earthquake records used in this paper

Station	Component	Epicentre Distance (km)	PGA (g)	PGV (Cm/s)	PGD (Cm)
24088 Pacoima Kagel Canyon	PKC360	19.28	0.433	51.5	7.21
24157 LA - Baldwin Hills	BLD090	28.2	0.239	14.9	6.17
24278 Castaic - Old Ridge Route	ORR090	40.68	0.568	52.1	9
24396 Malibu – Point Dume Sch	MAL090	31.21	0.13	8.5	2.11
90015 LA - Chalon Rd	CHL070	14.92	0.225	16.6	3.39
90049 Pacific Palisades - Sunset Blvd	SUN190	18.22	0.469	31	5.26
90059 Burbank - Howard Rd	HOW330	23.18	0.163	8.5	1.81

Table 3.2. Mass participation ratios for different modes in the 10-story building

Mode Number	Period (sec)	UX (%)	UY (%)	Sum UX (%)	Sum UY (%)	RZ (%)	Sum RZ (%)
1	2.3	0.04	72.50	0.04	72.50	34.00	34.00
2	2.2	73.20	0.04	73.24	72.54	17.00	51.00
3	1.9	0.14	0.27	73.38	72.81	22.00	73.00
4	0.8	0.00	12.00	73.38	84.81	6.00	79.00
5	0.8	11.80	0.00	85.11	84.81	3.00	81.00
6	0.7	0.02	0.03	85.13	84.84	4.00	85.00
7	0.5	0.00	5.10	85.13	89.94	2.00	87.00
8	0.5	5.10	0.00	90.23	89.94	1.00	88.00
9	0.4	0.01	0.01	90.24	89.95	2.00	90.00

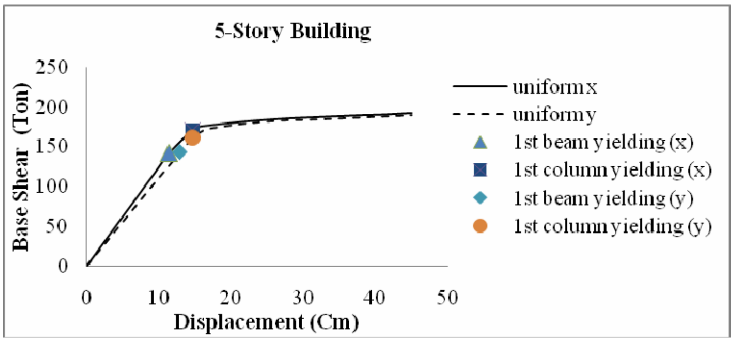
4. PUSH OVER ANALYSIS

This section describes some results obtained by pushover analysis. The performance of the designed structures is assessed at a target performance level that represents a ground motion with a 10%/50 year exceedance probability (ASCE, 2000). The two gravity (vertical) loads used in this paper are 1.1(DL+LL) and 0.9DL.

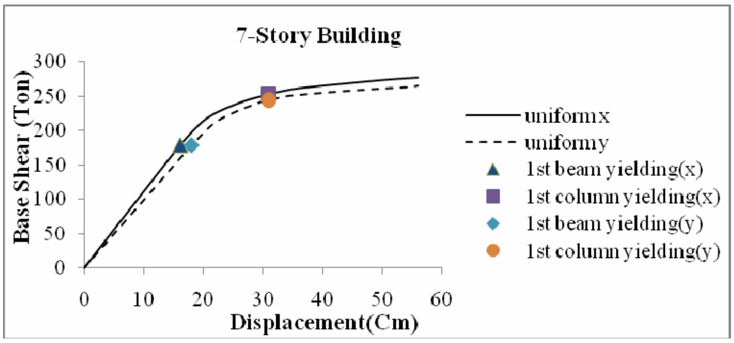
For lateral seismic loads, the analysis was performed by assuming two types of lateral loads distributions; triangular and rectangular ones. The target displacements for these buildings are calculated according to the FEMA 356 formula and are given in Table 4.1 (ASCE, 2000). To obtain the capacity curves, combinations of lateral loads and gravity loads are used. Some examples of the resulting capacity curves for the three buildings are shown in Figure 4.1. In this figure, the locations of the first plastic hinges occurred in beams and columns are displayed. It can be seen that all curves display similar patterns of behavior. They are linear initially but start to deviate from linearity when inelastic actions start to take place. Furthermore, it can be concluded that the curves obtained for the two gravity loads are approximately similar to each other while they are more sensitive to the type of lateral loads, as shown in Figure 4.1. Moreover, it can be noted that the choice of a push direction has a little influence on capacity curves.

Table 4.1. Target displacements

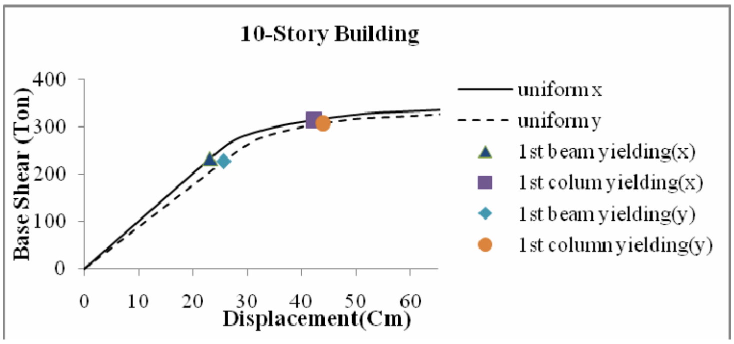
Building	Lateral load pattern	Displacement (X) (Cm)	Displacement (Y) (Cm)
5-story	Uniform	19.5	21
	Triangular	21	22.45
7-story	Uniform	26.7	28.7
	Triangular	28.7	31.3
10-story	Uniform	36.8	39.77
	Triangular	39.86	43.1



(a)



(b)



(c)

Figure 4.1. Pushover curves for (a) a five-story building (b) a seven-story building and (c) a ten-story building

Sample of results of the inter-story drift ratio for the 10-story building with both lateral load patterns are presented in Figure 4.2. The results obtained indicate that all the frames in both directions are within the life safety performance level

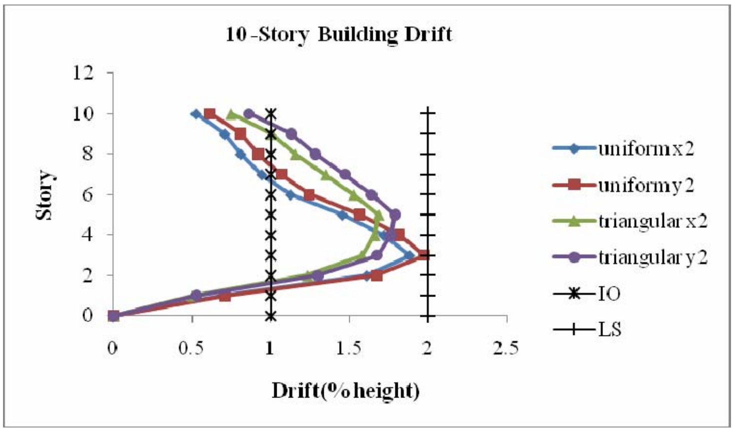


Figure 4-2- Performance of 10-story RC frame based on maximum inter-story drifts

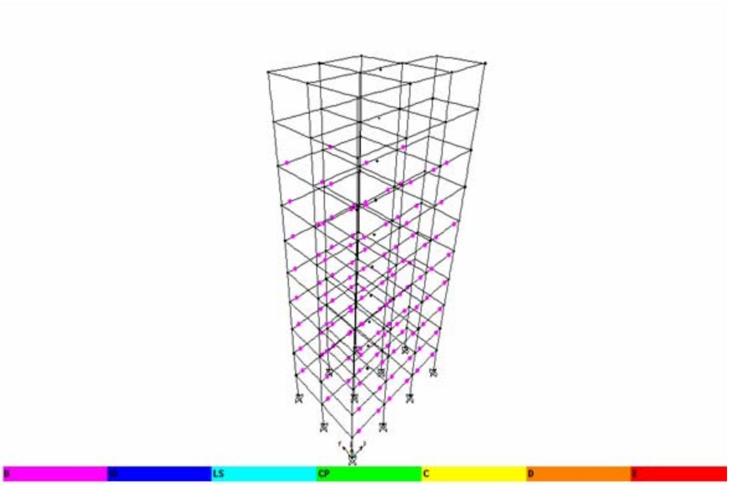


Figure 4.3. Plastic hinges in 10-story building caused by uniform load pattern in X direction

From studying the distribution of hinges along the height, it has been found that the use of different lateral load patterns could lead to different results. By using a triangular pattern, the hinges are distributed more uniformly along the height of the building whereas the use of a rectangular pattern, the hinges are concentrated in the lower parts of the building with some presence in the middle part. The formation of hinges; for the 10-story building at the life safety target displacement level along the x-axis, is shown in Figure 4.3. The plastic hinge pattern shows that no plastic hinges are formed in the y-direction. This can be considered as an additional proof on the ineffectiveness of the torsional moments in these structures.

5. LINEAR TIME HISTORY ANALYSIS

By using all the seven records given in Table1, the three buildings have been analyzed. The results obtained for all the buildings, with the exception of very rare cases, show that the drifts are within the life safety performance level. The exceptional cases are related to records with high PGA. Results of drifts at different floors of the 10-story building are shown in Figure 5.1.

To make detailed investigations on the effect of the plan asymmetry on torsion, the most critical record; i.e. Castaic-Old Ridge Route, has been used. Figure 5.2. shows the maximum drift differences between different points and the centre of mass of the 10th floor in the 10-story building in the Y-direction. Numbering of these points is shown in Figure 2.1. The largest differences are related to points 10 and 11 that approximately equal to 0.2% of the height of the building. However, these differences are smaller than those given by seismic codes for torsionally irregular structures (BCCS, 2009).

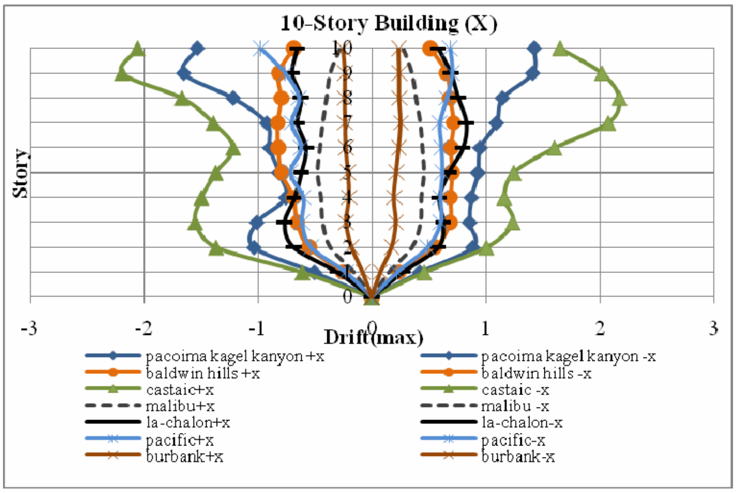


Figure 5.1. Maximum drifts occurred in the 10-Story building

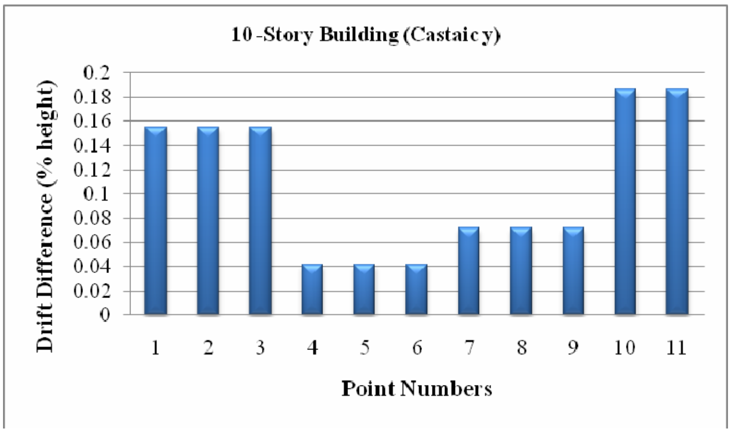


Figure 5.2. Maximum drift differences at the 10th floor of the 10-Story building

6. COMPARISON OF PUSHOVER AND LINEAR TIME HISTORY ANALYSIS WITH NONLINEAR TIME HISTORY ANALYSIS

In this section, nonlinear time history analysis has been performed using the most critical records; Castaic-Old Ridge Route and Pacoima Kagel Canyon, given in Table 3.1. Results of the pushover using rectangular and triangular pattern of loads and linear time history analysis have been compared with the results of nonlinear time history analysis. This has been done by equating the top displacement measured at the center of mass for linear and nonlinear dynamic analyses to the target displacement given by the pushover analysis.

6.1. Comparison of maximum drift results

Maximum drifts of the three structures estimated by linear time history and nonlinear static analyses are compared with those given by the nonlinear time history analysis. It can be seen that the results of linear time history analysis are relatively closer to the corresponding results of nonlinear time history analysis. Percentage of the drift error for the 10^{th} floor of the 10-Story building using the Pacoima Kagel Canyon record is shown in Figure 6.1.

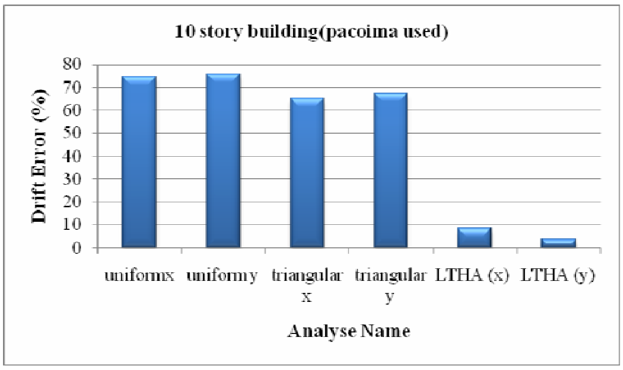


Figure 6.1. Maximum drift comparison of 10-story building using Pacoima Kagel Canyon record

6.2. Displacement comparison

In order to assess various analysis methods, the variations of displacements along the height of the buildings using pushover and linear time history analyses are obtained and compared with the nonlinear time history analysis. Displacements for the 10-story building are shown in Figure 6.2. As shown in this figure, the linear time history results are closer to the nonlinear time history results than those obtained by the pushover analysis.

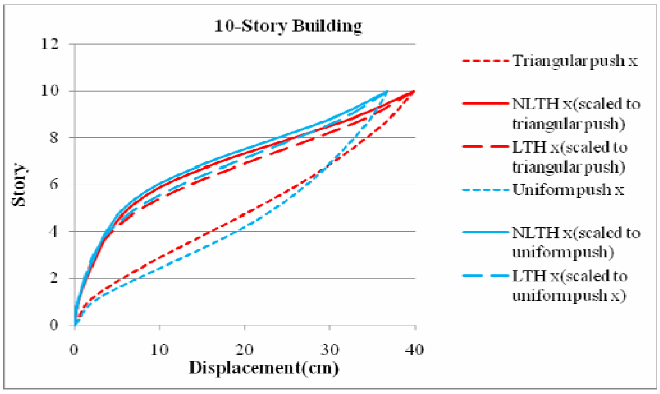


Figure 6.2. Displacement comparison for the 10-story building using different analysis methods

6.3. Base shear comparison

To have further assessment of the analysis methods, base shears for the three types of buildings are calculated. For the five story building, the pushover analysis with a uniform pattern load has shown better results than others. However, for the seven and ten story buildings, and as shown in Figure 10, the pushover analysis with a triangular pattern load gives closer results.

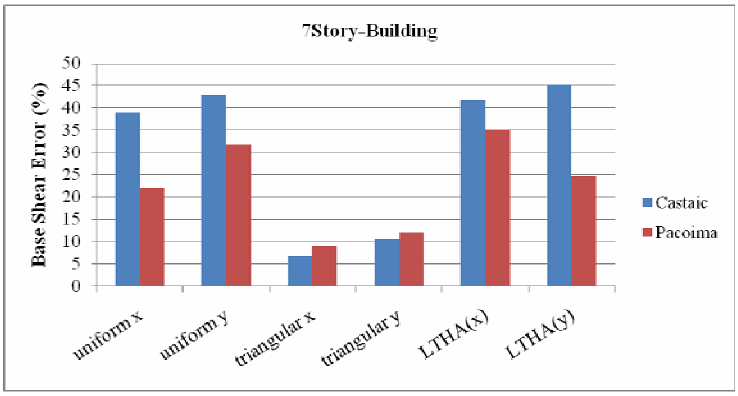


Figure 6.3. Base shear- comparison of pushover and linear time history analyses with nonlinear time history analysis

7. CONCLUSIONS

In this paper, different analysis method such as nonlinear static analysis, linear time history analysis and nonlinear time history analysis, were utilized for mid-rise concrete moment frame structures to investigate plan irregularity. The results obtained show that:

- The results of pushover analysis confirm the validity of designs based on existing codes. This, most probably, due to the conservative nature of the existing codes.
- Since the first two periods of vibration are predominantly translational, the investigated buildings can be classified as torsionally stiff buildings.
- The results obtained for the three frames show that pushover techniques still require further refinement in order to provide reliable estimates of the dynamic response of 3D asymmetrical structures. Accordingly, for irregular buildings similar to those tested in this paper, nonlinear time history analysis need to be performed.
- To evaluate the seismic behavior of complex tall asymmetric buildings with significant higher mode effects, the nonlinear dynamic analysis methods generally provide more reliable assessment of earthquake performance than other methods. In this paper, the nonlinear dynamic procedure produces results that are different from those given by the nonlinear static and linear time history analyses. However, and since the linear dynamic analysis has shown closer results to the nonlinear dynamic procedure than the nonlinear static analysis, it can be concluded that the linear dynamic analysis are more reliable than the nonlinear static analysis in tackling irregular concrete frames similar to those discussed in this paper.

REFERENCES

ACI. (2005). Building Code Requirements for Structural Concrete, ACI Committee 318, American Concrete Institute, Detroit, USA.

ASCE, (2000). FEMA356, Prestandard and Commentary for the Seismic Rehabilitation of Buildings, Reston, Virginia, USA.

ASCE, (2007). Seismic rehabilitation of existing buildings, ASCE/SEI Standard 41-06 with supplement 1, American Society of Civil Engineers, Reston, VA, USA.

ATC, (2005). Improvement of Nonlinear Static Seismic Analysis Procedures, FEMA440 Report, Federal Emergency Management Agency, Redwood City, CA., USA.

BHRC. (2005). Iranian Code of Practice for Seismic Resistant Design of Buildings, 3rd Edition, Tehran, Iran.

BSSC. (2009). NEHRP recommended seismic provisions for new buildings and other structures (FEMA P-750), Washington, D.C.,USA.

CEN. (2004). Eurocode 8 - Design of Structures for Earthquake Resistance - Part 1: General Rules, UNI EN 1998-1:2005, Brussels, 2004.

Chopra, A.K., and Goel, R.K. (2004). A modal pushover analysis procedure to estimate seismic demands for unsymmetric-plan buildings. *Earthquake Engineering and Structural Dynamics* **33**, 903-927.

Computers and Structures, INC. (2009). CSI Analysis Reference Manual for ASP2000, ETABS, and SAFE, Berkeley, Calif., USA.

Deierlein, G.G., Reinhorn, A. M. and Willford, M.R. (2010). Nonlinear structural analysis for seismic design, a guide for practicing engineers, NEHRP Seismic Design Technical Brief No. 4, NIST GCR 10-917-5.

Fajfar, P., Marusic, D. and Perus, I. (2005). Torsional effects in the pushover-based seismic analysis of buildings. *Journal of Earthquake Engineering* **9**, 831-854.

Kalkan, E. and Kunnath, S.K. (2006). Adaptive modal combination procedure for nonlinear static analysis of building structures. *ASCE Journal of Structural Engineering* **132,** 1721-1731.

Kunnath, S.K. and Kalkan, E. (2004). Evaluation of seismic deformation demands using nonlinear procedures in multistory steel and concrete moment frames. *ISET Journal of Earthquake Technology,* Paper No. 445, **41:1**, 159-181.

Moghadam, A.S. and Tso, W.K. (2000). Pushover analysis for asymmetric and set-back multi-story buildings. *The 12th World Conference on Earthquake Engineering,* Paper No. 1093.

PEER website, peer. Berkely. edu/ smcat /data .html

Themelis, S. (2008). Pushover analysis for seismic assessment and design of structures, PhD Thesis, Heriot-Watt University, School of the Built Environment.