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Seismic demand evaluation of medium ductility RC moment frames using nonlinear procedures

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Abstract: Performance-based earthquake engineering is a recent focus of research that has resulted in widely developed design methodologies due to its ability to realistically simulate structural response characteristics. Precise prediction of seismic demands is a key component of performance-based design methodologies. This paper presents a seismic demand evaluation of reinforced concrete moment frames with medium ductility. The accuracy of utilizing simplified nonlinear static analysis is assessed by comparison against the results of time history analysis on a number of frames. Displacement profiles, drift demand and maximum plastic rotation were computed to assess seismic demands. Estimated seismic demands were compared to acceptance criteria in FEMA 356. The results indicate that these frames have sufficient capacity to resist interstory drifts that are greater than the limit value.

Keywords: seismic demand evaluation; pushover analysis; time history analysis; plastic rotations; FEMA 356

1 Introduction

In recent years, performance-based design methods have been proposed as new concepts and have been extensively used in the seismic design and evaluation of structures. Existing problems of force-based design methods has led to greater interest in performance-based design. This approach enables engineers to design structures with predictable performance against earthquakes. The three main steps of performance-based design are as follows: performance objectives, determination of seismic demands, and seismic performance evaluation. Determination of seismic demand for structural and nonstructural components according to earthquake loads is the most important and difficult step in evaluating performance and requires accurate modeling and analysis. Therefore, evaluation of the response of structures designed by current design codes can be achieved by inelastic analysis. Although nonlinear dynamic time-history analysis is the most accurate analytical procedure to evaluate the seismic demands, the application of nonlinear static pushover (NSP) analysis procedures is generally considered to be more appropriate for seismic design in engineering practice. Simplified NSP analysis procedures proposed in FEMA 356, which have become common in engineering practice, are based on increasing predefined lateral load patterns until target displacement is achieved.

Given the importance of seismic demands, many studies have been conducted to resolve the ambiguities in this field. Some of these studies have evaluated the seismic demands of different structural systems using nonlinear procedures, whereas others have investigated prevalent methods of seismic demand determination. In this paper, only a few important studies are mentioned in the following summary.

Prediction of the seismic deformation demands of structures has been the main attempt of several researchers (Fajfar and Fischinger, 1988; Seneviratna and Krawinkler, 1997; Miranda, 1997). Iwan (1999) investigated the inaccuracy of pushover approaches to estimate demands during pulse-like near fault ground motions. Kunnath and Gupta (2000) and Kunnath and John (2000) studied different lateral load patterns suggested in FEMA 356 and recognized inconsistencies in various pushover methods.

The drawbacks in pushover approaches based on lateral load patterns recommended in FEMA 356 have brought attention to alternative pushover procedures. Chopra and Goel (2002) introduced modal pushover analysis (MPA) as a new analytical method. In this method, participation of higher modes is considered in the behavior of a structure that leads to accurate results rather than other load distributions. Barros and Almeida (2005) proposed a new multi-mode load pattern, according to the relative participation of each mode of vibration in the elastic range of response of a structure excited by an earthquake. Kunnath and Kalkan (2004)

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investigated a group of steel and concrete buildings using nonlinear static analysis with three different load patterns and compared the seismic demands to the results of time history analysis. They found that selection of load pattern has a considerable effect on seismic demands. Kalkan (2006) investigated the specifications of near fault excitations in relation to seismic demands to obtain new methods of determining seismic deformation demands. The validation of the adaptive modal combination (AMC) approach introduced by Kalkan and Kunnath (2006) as a new adaptive pushover scheme was investigated and employed for regular moment frames (Kalkan and Kunnath, 2006, 2007). Han *et al.* (2007) investigated the consequences of different sets of earthquakes on seismic demands and concluded that they can have a considerable effect on seismic demands even though all the sets had similar soil characteristics. In addition, an improved modal pushover analysis (IMPA) approach was suggested by Mao *et al.* (2008). This method considers the redistribution of inertia forces after the structure yields. The IMPA procedure uses the product of the time variant floor displacement vector and the structural mass matrix as the lateral force distribution at each applied load step beyond the yield point of the structure.

Furthermore, Mass Proportional Pushover (MPP) was introduced by Kim and Kurama (2008) to predict the peak seismic lateral displacement of structures responding in the nonlinear range. The superiority of the MPP is that the consequences of higher modes on the lateral displacement demands are incorporated in a single invariant lateral force distribution that is proportional to the total seismic masses at the floor and roof levels.The demands estimated by MPP were compared with the results of MPA. The comparisons demonstrated that the MPP procedure surpasses the MPA approach and provides more accurate demand predictions of roof and floor lateral displacement.

A study by Kim and Kim (2009) investigated the seismic demands of an RC special moment frame using nonlinear static and dynamic procedures and showed that the inelastic behavior of the studied frame satisfied the design drift limit. Furthermore, Huang and Kuang (2010) studied the applicability of pushover analysis for seismic assessment of medium-to-high-rise shear wall structures and demonstrated that pushover analysis underestimates interstory drifts and rotations, particularly those at the upperstories of buildings, and overestimates the peak roof displacement at the inelastic range. Mortezaei *et al*. (2011) investigated the effectiveness of a modified pushover analysis procedure for seismic demand estimation of RC special moment frames excited with near-fault ground motions characterized with forward directivity and proposed a new pushover method. They showed that the proposed pushover method yielded better results than all the pushover approaches proposed in FEMA 356 and provided a close prediction to dynamic results. Ruiz-Garcia and Miranda (2010) presented a probabilistic approach to estimate residual drift demands during the seismic performance evaluation of existing multi-story buildings. Nguyen *et al*. (2010) compared the accuracy of modal, MPA, IMPA and MPP procedures for seismic evaluation of buckling-restrained braced frame (BRBF) buildings. The results showed that the MPP approach tends to inaccurately estimate seismic demands of lower stories whereas the MPA and IMPA methods result in precise estimation of maximum interstory drift over all stories of the studied buildings.Vamvatsikos and Fragiadakis (2010) investigated seismic performance sensitivity and uncertainty of a steel moment-resisting frame using incremental dynamic analysis. Seismic vulnerability assessment of RC moment frame buildings in moderate seismic zones was studied by ElHowary and Mehanny (2011).

Although many researchers have investigated seismic demands, there is still vagueness in seismic demands limitations. In FEMA356, four analytical methods are presented to assess the seismic demands of buildings. In this study, the nonlinear static and dynamic analyses are utilized. The emphasis is on demand predictions for regular medium ductility RC moment frames designed according to the Iranian Code of Practice for Seismic Resistant Design of Buildings (Standard No. 2800, 3rd edition). Additionally, interstory drifts and plastic rotations and their limiting values based on FEMA 356 are investigated to evaluate the seismic demands and capacity of the frames designed according to the Iranian Seismic Code to endure higher drift demands. In order to assess the accuracy of pushover analysis in predicting the nonlinear response of structures, the results were compared to seismic demand estimations obtained from dynamic analysis. The estimated demands are evaluated at the global, story and local levels using nonlinear approaches. At the global level, the displacement profile of the building is assessed and at the story level, interstory drift values are compared. Eventually, plastic rotations at the ends of beam and column members are evaluated as local demands. Three different lateral load patterns proposed in FEMA 356 are employed in the pushover analyses. Four, eight, and 12-story RC frames with medium ductility were designed and analyzed using nonlinear procedures, and finally, the seismic demands of RC frames with medium ductility were estimated.

2 Performance-based evaluation

The FEMA 356 criteria were used to evaluate the seismic demands based on nonlinear analysis. FEMA 356 provides analytical approaches for the seismic performance evaluation of existing buildings. During the ground motion, performance levels limit the maximum damage while performance objectives describe the target performance level for a particular intensity of ground motion. Determination of performance levels in FEMA 356 is the first step in performance-based evaluation; these include Immediate Occupancy (IO), LifeSafety (LS), and Collapse Prevention (CP). At the IO performance level, structures experience minor damage. At the LS performance level, structures may experience damage while collapse resistance is still significant. Structures at CP should remain standing, but have little resistance against collapse. In FEMA 356, the Basic Safety Objective (BSO) is defined as LS performance for the Basic Safety Earthquake 1 (BSE-1) earthquake hazard level and CP performance for the BSE-2 earthquake hazard level. BSE-1 is specified as the smaller of an event corresponding to 10% probability of exceedance in 50 years (10% in 50 years) and 2/3 of BSE-2, which is 2% probability of exceedance in 50 years (2% in 50 years). The case study frames were designed based on the Iranian seismic design code, which has a design spectrum for seismic hazard levels that corresponds to 10% in 50 years (BSE-1). The next step, which is the most critical, is prediction of deformation demands by the nonlinear analytical procedures prescribed in FEMA 356. As the final step of the evaluation, the obtained demands are compared to acceptance criteria related to the three performance levels.

3 Case study frames and analytical assumptions

This study is limited to reinforced concrete moment frames with medium ductility. The three RC frames adopted in this investigation are 4, 8 and 12-story frames, which are all assumed to be located in a highly seismic region, with soil type II. The frames were designed in accordance with the Iranian Seismic Code. The height of the stories and length of the bays are 3.2 m and 5 m, respectively. An elevation view of the frames is shown in Fig. 1(a). The compressive strength of the concrete material is equal to 21 MPa. The yield stress of the longitudinal reinforcing steel is assumed to be 400 MPa. The uniformly distributed design dead load is 6.5 kN/m^2. The live load of each floor and roof is 2 kN/m^2 and 1.5 kN/ m^2, respectively, based on the Iranian National Building Codes (Part 6: Structural Loadings). The floor load width of each frame is also 5 m. Consequently, the structural weights are 51.75 kN for the stories and 51 kN for the roof. The natural periods of vibration are 0.93, 1.83 and 2.37 for 4, 8 and 12-story frames, respectively. The frames were predesigned using the program ETABS and were then modeled using DRAIN2DX computer software (Prakash *et al*, 1993) to perform nonlinear static and dynamic analysis. Beam and column sections are presented in Table 1. The moment-rotation relationship used in the modeling of the frame members is shown in Fig. 1(b). $M_y^{\pm}$ is the yield moment in the positive and negative directions and $\varphi_y^{\pm}$ is the yield rotation in the positive and negative directions.

4 Nonlinear static analysis

Application of pushover analysis for seismic assessment and design has increased appreciably in recent years. Pushover analysis can be used to assess lateral capacity and overall stability as well as evaluate plastic deformation mechanisms. The advantage of pushover analysis is mainly related to its simplicity in modeling and computational demands, in comparison with nonlinear dynamic analysis (Krawinkler and Seneviratna, 1998). In the case of static procedures, the following lateral load patterns were used:

Push 1: The inverse triangular distribution in which a linear lateral load is applied to the structure throughout the building height is based on the following equations:

$$F_x = C_{vx}V, \quad C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{1}$$

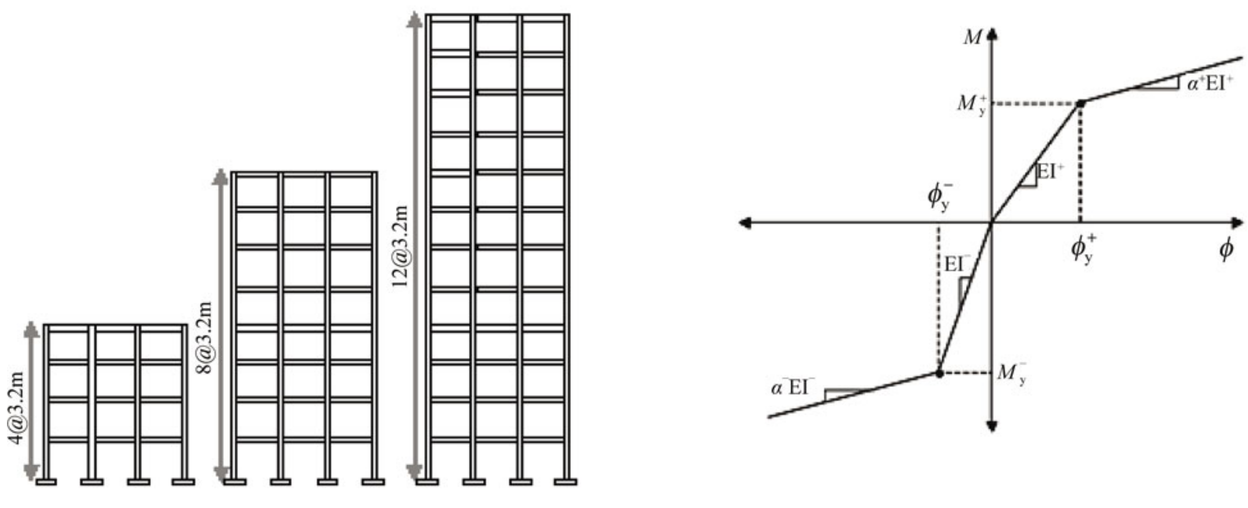


Fig. 1 (a) Configuration of the frames; (b) Moment-rotation relationship used in the modeling of the frame members

Table 1 Beam and column sections of the frames

Frame	Story	Beam	Column	Frame	Story	Beam	Column
4-story	1st	B35×50	C50×50-12φ20	12-story	1st	C35×60	C80×80-16φ28
	2nd	B35×50	C45×45-12φ20		2nd	B35×60	C80×80-12φ25
	3rd	B35×50	C40×40-12φ18		3rd	B35×60	C80×80-12φ25
	4th	B35×50	C40×40-12φ18		4th	B35×60	C55×55-12φ22
8-story	1st	B35×50	C80×80-12φ25		5th	B35×60	C55×55-12φ22
	2nd	B35×50	C55×55-12φ22		6th	B35×60	C50×50-12φ22
	3rd	B35×50	C50×50-12φ22		7th	B35×50	C50×50-12φ22
	4th	B35×50	C50×50-12φ22		8th	B35×50	C50×50-12φ20
	5th	B35×50	C45×45-12φ18		9th	B35×50	C45×45-12φ18
	6th	B35×50	C45×45-12φ18		10th	B35×50	C45×45-12φ18
	7th	B35×50	C40×40-12φ18		11th	B35×50	C40×40-12φ18
	8th	B35×50	C40×40-12φ18		12th	B35×50	C40×40-12φ18

where F_x is the applied lateral force at level 'x', C_{vx} is the vertical distribution factor and V is the base shear. k = 2 for $T \geq 2.5$ and k =1 for $T \leq 0.5$, so that values of k for intermediate values of T can be interpolated. w_i is a portion of the total building weight W related to floor level i, w_x is a portion of the total building weight W related to floor level x, h_i is the height from the base to floor level i and h_x is the height from the base to floor level x.

Push 2: The uniform distribution in which a constant distribution of the lateral forces is applied to the structure.

Push 3: The modal load pattern related to distribution of forces proportional to the first mode of vibration.

In order to verify the applicability of pushover procedures for estimating the overall seismic demands, the pushover results from the frames are compared with the dynamic analysis reported in the following sections. In addition, to provide a realistic basis for this comparison, the selection and scaling of the ground motions used are carefully assessed with regard to the design spectrum.

4.1 Target displacement

The capacity spectrum method (CSM) and displacement coefficient method are promising procedures for estimating demands that have been commonly used by many researchers. The pushover analyses were conducted for each frame until the roof displacement of the frame attained a specified target displacement as a measure of seismic demands. The target displacements were calculated based on the provisions in FEMA 356 for BSE-1. The target displacement is obtained based on the displacement coefficient method from the following equation:

$$\delta_t = c_0 c_1 c_2 c_3 s_a \frac{T_e^2}{4\pi^2} g \tag{2}$$

where c_0 is the first modal participation factor (at the level of the control node), c_1 is the system's inelastic displacement modification factor, c_2 is the coefficient for the influence of stiffness degradation upon displacement, c_3 is the post yield stiffness coefficient, T_e is the effective period and s_a is the response spectrum acceleration, at the effective fundamental period that is calculated here based on the Iranian Seismic Code design response spectrum corresponding to 10% probability of exceedance in 50 years (10% in 50 years). Note that 10% in 50 years hazard level ground motions (or site response spectra) are commonly used for seismic design of regular structures. The target displacements were determined as 16 cm, 38 cm, and 57 cm for 4, 8, and 12-story frames, respectively.

5 Selection of ground motions for time history analysis

As previously noted, the validity of pushover approaches is verified based on the results of nonlinear dynamic analyses. The set of ground motions used contains eight selective ground motions, which were developed for the SAC project (Somerville *et al.*, 1997). All ground motions were recorded on sites that are classified as soil type II, in accordance with the Iranian Seismic Code, with a magnitude range of 6.6 to 7.4. Details of these records are tabulated in Table 2. Furthermore, the earthquakes were chosen to have seismic hazard levels corresponding to 10% in 50 years. The accelerograms need to be modified to be compatible with design response spectrum in performance assessments using time history analysis. Therefore, the records were adjusted in the frequency domain so that their mean response spectrum matched the Iranian Seismic Code design spectrum, which is plotted in Fig. 2 for a damping of 5%.

Table 2 Ground motion details

EQ No.	Record	Station	Magnitude	Distance (km)	PGA (g)
Eq-1	Loma Prieta	1028 Hollister City Hall	7.1	28.2	0.247
Eq-2	Loma Prieta	Gilroy Array#2	7.1	12.7	0.367
Eq-3	Landers	22170Joshua Tree	7.4	11.6	0.284
Eq-4	Landers	Barstow	7.4	36.1	0.135
Eq-5	Imperial Valley	El Centro Array #9	7.2	8.3	0.313
Eq-6	Imperial Valley	El Centro Array #12	6.9	18.2	0.143
Eq-7	San Fernando	135 LA Hollywood StorLot	6.6	21.2	0.210
Eq-8	Northridge	La Hollywood Storage FF	6.7	25.5	0.358

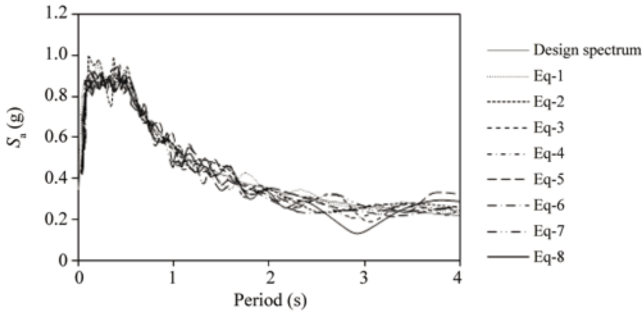


Fig. 2 Response spectrum of selected records

6 Evaluation of seismic demands

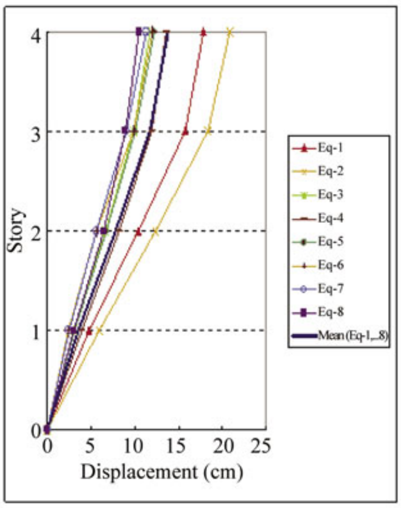
In order to evaluate seismic demands, the results of the pushover analysis with different lateral load patterns are compared to the results of nonlinear time history analysis conducted with the selected ground motions, at global, story and local levels including the displacement profile, interstory drifts and plastic rotations, and location of their formation.

6.1 Displacement profile

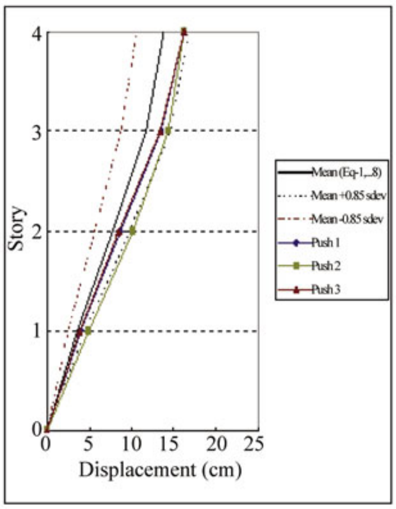
Global demands include the displacement profile of the building at the peak roof displacement. Displacement profiles for each frame using nonlinear time history analysis are plotted in Fig. 3(a) through 5(a) for each record. To enable direct comparison with estimated demands of time history analysis, results of pushover analysis using three lateral load patterns and the mean value of time history analysis obtained by averaging results from selective ground motions, along with mean ± 0.85 standard deviation, are plotted in Fig. 3(b) through 5(b). In all cases, the results of the pushover analysis with Push 1 and Push 3 load patterns yielded similar results and reasonably accurate estimates of the peak displacement. These patterns also have better overall compatibility with the dynamic results and are closely related to the average of the dynamic analyses. They slightly overestimate the displacement in the upper stories. The uniform load pattern, Push 2, tends to grossly overestimate demands at the lower stories since this pattern results in higher loads being applied at the lower stories.

6.2 Interstory drifts

Interstory drift is an important index in performance evaluation. The interstory displacements vary with time

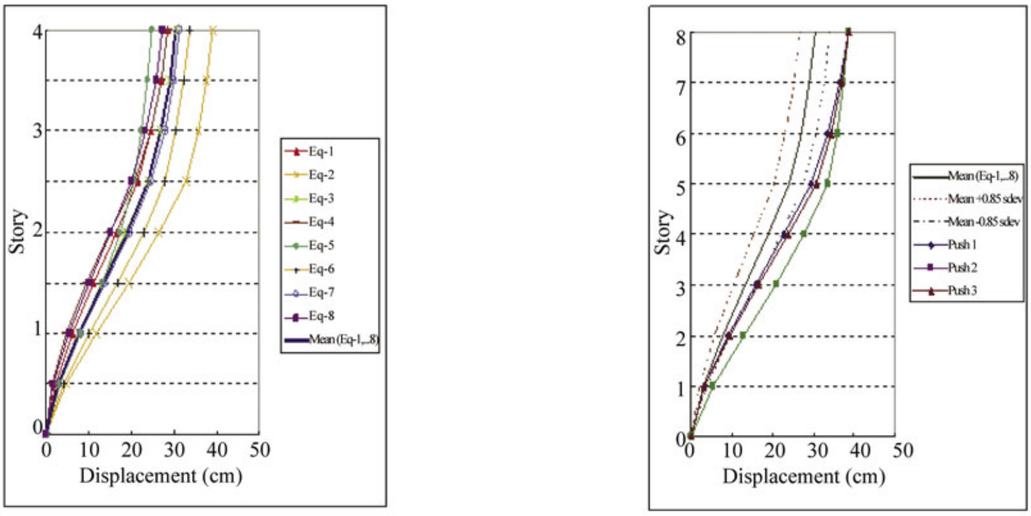


(a)



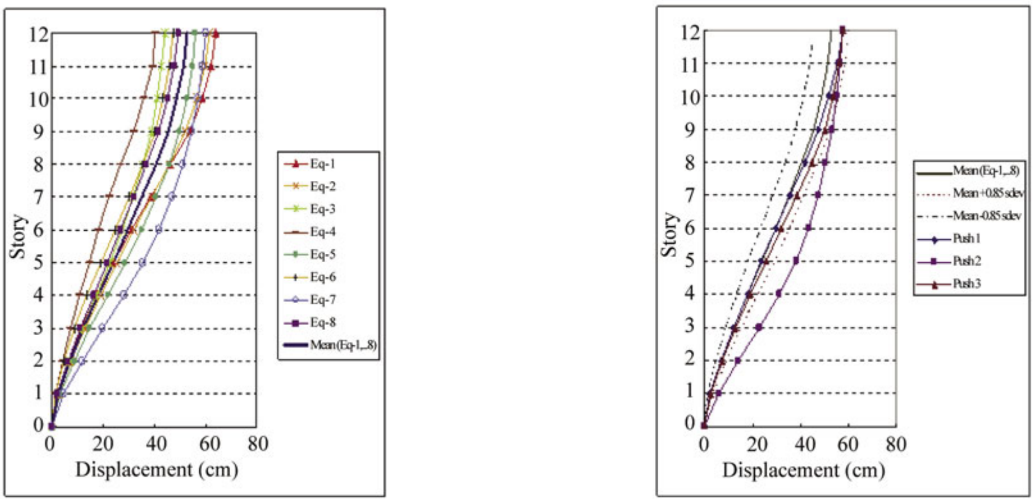
(b)

Fig. 3 Maximum displacement profiles for 4-story RC frame



(a) (b)

Fig. 4 Maximum displacement profiles for 8-story RC frame



(a) (b)

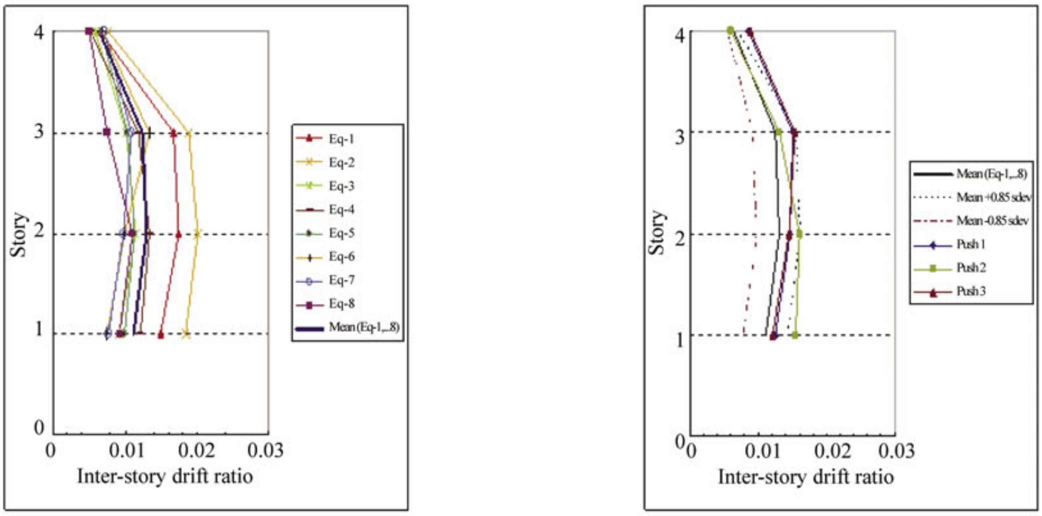
Fig. 5 Maximum displacement profiles for 12-story RC frame

based on the dominant modes of vibration. Invariant load patterns used in pushover methods lead to a consistent pattern of interstory demand up to the elastic range, following which the story demands become localized and depend on the story level to experience first excursion beyond the elastic state.

The maximum interstory drift was also obtained to specify the damage assessment of the frames. Figures 6(a) through 8(a) provide interstory drift demands estimated by time history analysis for each frame. The results of the pushover analysis are depicted in Fig. 6(b) through 8(b) in which the mean value of the time history analysis along with mean ± 0.85 standard deviation is also specified for comparison.

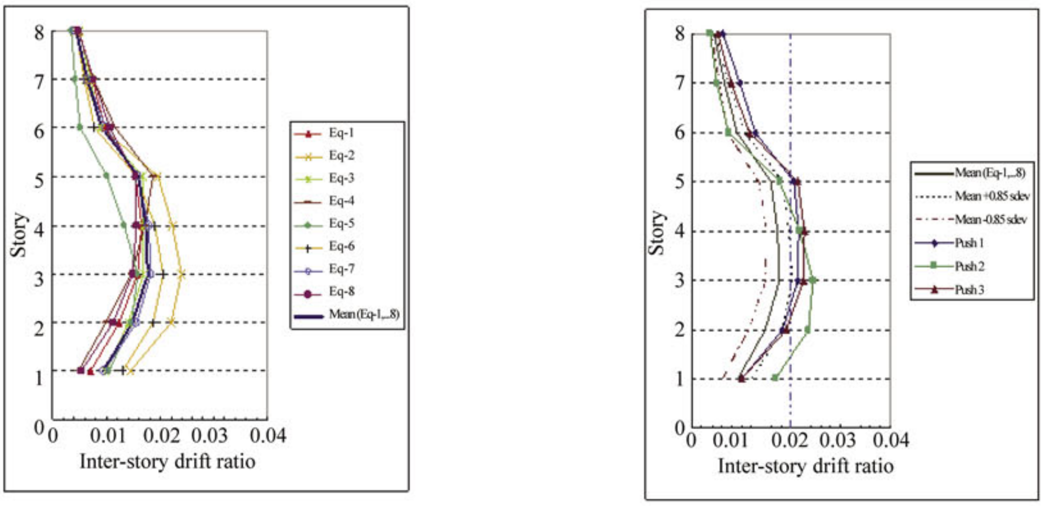
They are also compared to limiting interstory drift values provided by the provisions of the Iranian seismic Code and FEMA 356 as 2% for the BSE-1 hazard level. The results of the time history analysis for the 4-story frame indicate that the interstory drifts obtained from all the records do not exceed the limiting value of 2%. The interstory drifts related to the 8-story frame do not exceed the limiting value with the exception of Eq-2. For the 12-story frame, Eq-1, Eq-2, Eq-5 and Eq-7 pass the limiting drift value; however, the mean value obtained by averaging results from selective ground motions for all three frames implies that estimated interstory demands do not exceed the limiting interstory value.

For all three frames, the results of the pushover analysis with the Push 1 load pattern and Push 3 load pattern both result in similar estimates. Although interstory drifts estimated with Push 1 and Push 3 are found to be closest to the average dynamic estimates, in some cases, they overestimate the drifts. For the 4-story frame, from Fig. 6 it is obvious that the overestimation of the drifts is apparent not only in the upper stories, but also in the lower stories. In the 8-story frame, these patterns tend to overestimate the drifts in the lower stories while drifts are slightly overestimated in the upper stories. For the 12-story frame, the Push 1 load pattern tends to slightly overestimate the drifts in the



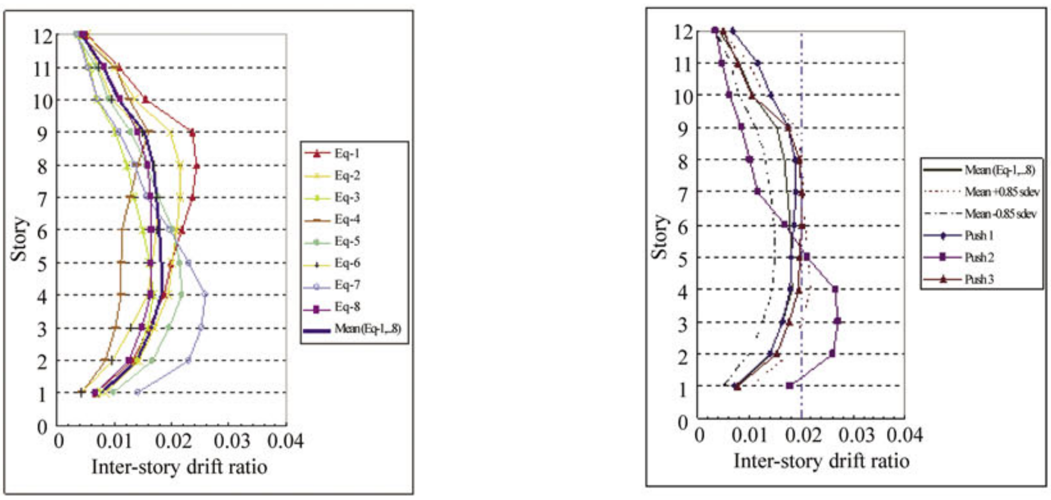
(a) (b)

Fig. 6 Interstory demands in 4-story RC frame



(a) (b)

Fig. 7 Interstory demands in 8-story RC frame



(a) (b)

Fig. 8 Interstory demands in 12-story RC frame

upper stories while the Push 3 load pattern tends to slightly overestimate the drifts in the lower stories. On the other hand, uniform load pattern Push 2 grossly over estimates interstory drifts at the lower stories and underestimates them at the upper stories in all three frames. The discrepancy becomes more apparent for the 8 and 12-story frames that exhibit significant interstory drifts in the lower stories compared to the upper levels, which is due to higher load being applied at the lower floors in uniform distribution. Therefore, triangular and modal load patterns result in a more realistic estimation of the demands compared to the uniform load pattern.

6.3 Estimation of plastic rotations

Global level demands are determined to evaluate the overall structural response for a given hazard or performance level, while the local level demands are utilized to assess structural elements according to the member's details and other specifications that influence structural behavior. Determination of the distribution of plastic hinges, obtained from pushover and dynamic analysis, results in an accurate seismic performance evaluation. In fact, the distribution of plastic hinges enables the identification of overloaded members and the global mechanism of collapse.

For evaluation of member-level seismic demands, inelastic rotations at the ends of beam and column elements are computed based on nonlinear static and time history analysis. Maximum plastic rotations at the end of beams and columns based on pushover analysis under three load patterns and time history analysis with FEMA 356 limits are presented in Table 3 through Table 5, for the 4, 8 and 12-story frames, respectively. The location of hinge formations obtained from pushover analysis with the three load patterns along with the results of time history analysis are illustrated in Fig. 9 through 11, respectively. For 8 and 12-story frames, the results show that nonlinear static analysis with Push 1 and Push 3 generally provide better estimates of plastic rotations and show good agreement with the results of the time history analysis. In the uniform load pattern, the location of hinges is not accurately determined and this pattern is unable to identify plastic hinging and capture the inelastic demands in the upper stories. Plastic rotations obtained from Push 2 are also overestimated in the lower stories.

According to the results from all the frames, all of the plastic rotations of the beams and columns due to both nonlinear static (three load patterns) and time history analysis remain within the LS performance level.

For the 4-story frame, another significant observation is related to the formation of plastic hinges at the lower ends of all columns at the first floor and at the upper ends of all columns at the third floor, based on pushover and time history analysis. Thus, the first and third stories are much more vulnerable than the second floor 4-story. Formation of plastic hinges at the lower ends of all the columns at the first and second floor and at the upper

Table 3 Maximum plastic rotations of 4-story frame

Level	Max. plastic rotation (rad), Push 1, Push 3		Max. plastic rotation (rad), Push 2		Max. plastic rotation (rad), time history		FEMA 356 limits (rad), LS level	
	Beams	Columns	Beams	Columns	Beams	Columns	Beams	Columns
1	0.008	0.010	0.011	0.013	0.006	0.009	0.019	0.014
2	0.007	0.003	0.007	0.004	0.005	0.004	0.019	0.014
3	0.004	0.006	0.002	0.006	0.001	0.007	0.020	0.015
4	0.006	-	0.002	-	0.002	0.002	0.020	0.015

Table 4 Maximum plastic rotations of 8-story frame

Level	Max. plastic rotation (rad), Push 1		Max. plastic rotation (rad), Push 2		Max. plastic rotation (rad), Push 3		Max. plastic rotation (rad), time history		FEMA 356 limits (rad), LS level	
	Beams	Columns	Beams	Columns	Beams	Columns	Beams	Columns	Beams	Columns
1	0.005	0.006	0.010	0.012	0.005	0.006	0.006	0.008	0.019	0.013
2	0.011	0.004	0.013	0.001	0.011	0.004	0.009	0.003	0.017	0.013
3	0.013	-	0.011	0.001	0.013	-	0.011	0.001	0.017	0.013
4	0.012	0.001	0.007	0.002	0.011	0.001	0.007	0.003	0.017	0.014
5	0.007	0.007	0.002	0.006	0.005	0.008	0.004	0.008	0.018	0.015
6	0.002	0.006	-	0.001	0.001	0.005	0.001	0.005	0.019	0.015
7	0.001	0.003	-	-	-	0.002	-	0.003	0.020	0.015
8	0.001	-	-	-	-	-	-	-	0.020	0.015

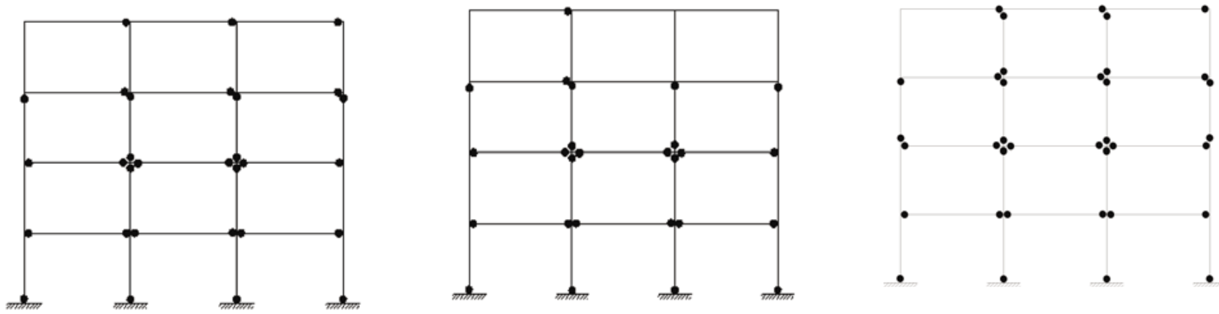


Fig. 9 Location of plastic hinges in pushover and time history analysis for 4-story frame: (a) Push 1 and Push 3; (b) Push 2; and (c) Time history

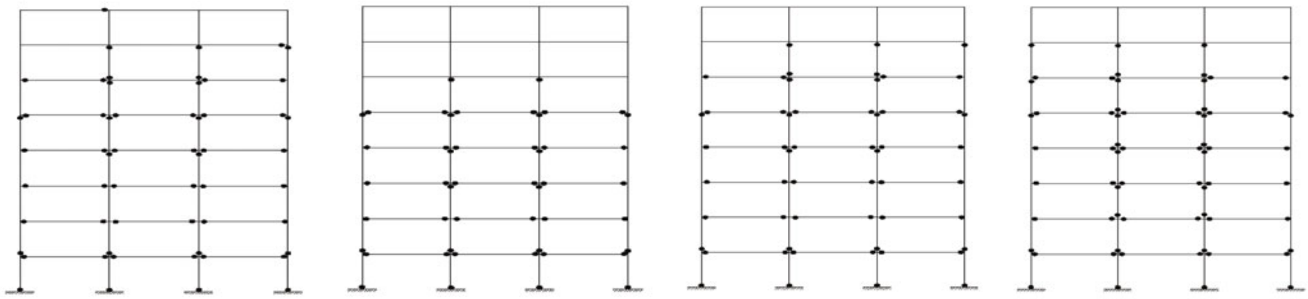


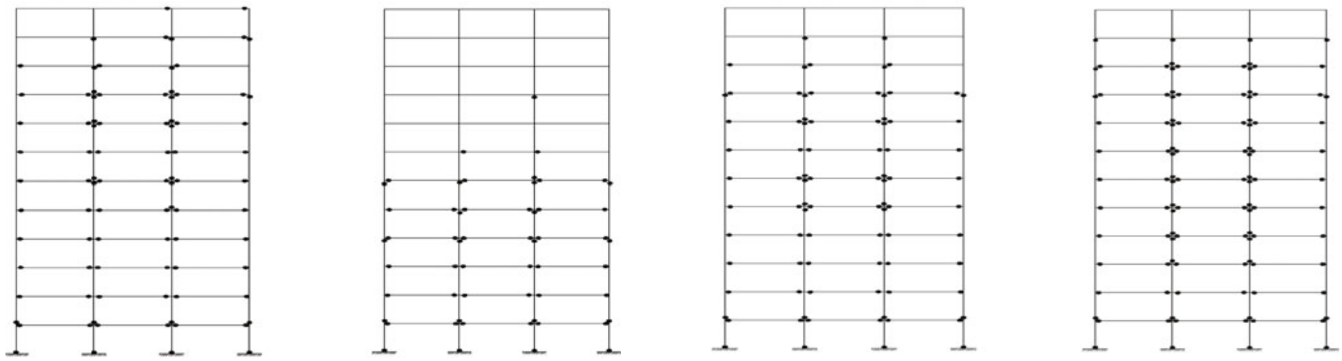
Fig. 10 Location of plastic hinges in pushover and time history analysis for 8-story frame: (a) Push 1; (b) Push 2; (c) Push 3; and (d) Time history

Table 5 Maximum plastic rotations of 12-story frame

Level	Max. plastic rotation (rad), Push 1		Max. plastic rotation (rad), Push 2		Max. plastic rotation (rad), Push 3		Max. plastic rotation (rad), time history		FEMA 356 limits (rad), LS level	
	Beams	Columns	Beams	Columns	Beams	Columns	Beams	Columns	Beams	Columns
1	0.005	0.004	0.011	0.010	0.006	0.004	0.005	0.005	0.019	0.012
2	0.011	0.003	0.017	0.001	0.013	0.003	0.011	0.003	0.017	0.012
3	0.014	-	0.017	-	0.015	-	0.012	-	0.017	0.012
4	0.014	-	0.014	0.002	0.017	-	0.011	0.002	0.017	0.012
5	0.014	-	0.009	0.003	0.016	0.001	0.009	0.003	0.017	0.013
6	0.011	0.005	0.003	0.003	0.012	0.006	0.006	0.007	0.017	0.013
7	0.012	0.006	-	-	0.013	0.006	0.007	0.007	0.017	0.013
8	0.009	0.003	-	-	0.009	0.004	0.005	0.005	0.017	0.014
9	0.005	0.006	-	-	0.004	0.007	0.003	0.007	0.018	0.014
10	0.010	0.003	-	-	0.004	0.002	0.004	0.004	0.018	0.015
11	0.002	0.004	-	-	-	0.005	-	0.004	0.019	0.015
12	0.001	-	-	-	-	0.005	-	0.005	0.020	0.015

ends of all the columns at the 5th floor represent the most vulnerable stories in the 8-story frame while formation of plastic hinges at the lower ends of all the columns at the first and second floor indicates a significant amount of plastic deformation in the 12-story frame.

Since the interstory drift limit (2%) was satisfied in the design process and according to plastic rotation demands determined based on time history analysis, which are much smaller than the limit value provided by FEMA 356, the frames seemto have sufficient capacity to undergo interstory drifts of more than 2%. In other words, a higher drift limit value can be considered



(a) (b) (c) (d)

Fig. 11 Location of plastic hinges in pushover and time history analysis for 12-story frame: (a) Push 1; (b) Push 2; (c) Push 3; (d) Time history

for BSE-1 hazard levels in RC frames with medium ductility.

The results of the uniform load pattern in comparison with the mean value of the dynamic analyses show that the pushover procedure can markedly overestimate demands (displacement, drift and plastic rotation) and can thus considerably affect seismic evaluations. However, favorable comparisons were obtained between pushover analysis using the triangular and modal load pattern and the mean dynamic data.

7 Conclusions

In this study, the nonlinear static behavior of RC frames with medium ductility was studied by comparing the responses obtained from pushover loading patterns. The validity of adopting nonlinear static procedures was investigated by comparing the results with those obtained from nonlinear time history analysis. Consequently, eight earthquake acceleration records, which were selected and scaled for compatibility with the design spectrum, were utilized. The following conclusions were made based on the results of the nonlinear static and time history analysis:

(1) Compared to the Push 2 load pattern, Push 1 and Push 3 load patterns provide displacement and story drift estimates that are generally much closer to the mean time history estimates. In particular, for the 12-story frame, interstory drifts predicted by triangular lateral load pattern in the lower stories, and interstory drifts predicted by the modal lateral load pattern in the upper stories showed good agreement with nonlinear time history analysis in the prediction of the global structural response.

(2) Using the uniform load pattern in nonlinear static analysis leads to a significant overestimation of interstory drifts in the lower stories. This is more prominent in taller frames since in uniform distribution, higher lateral load is being applied in the lower stories. Interstory drift demands at the upper stories were underestimated using the uniform load pattern.

(3) Nonlinear static analysis using triangular and modal lateral load patterns predicts the location of plastic hinge formation more accurately than the uniform load pattern. The uniform load pattern fails to identify the location of hinges in the upper stories and overestimates the plastic rotations in the lower stories.

(4) A comparison between the results of the plastic rotation demands for time history analysis and FEMA 356 limits for the LS limit state indicate that while the frames reach the interstory drift value limit, the plastic rotations are much smaller than the FEMA 356 limits. Actually, the frames seem to be able to resist interstory drifts that are greater than the limiting value; thus, a 2% and higher drift limit value could be considered for the BSE-1 hazard level in the design of RC frames with medium ductility. The location of plastic hinge formation and plastic rotation values at the end of the beams and columns should be considered in the analysis and design of structures when using the Iranian Seismic Code. These recommendations could be very useful to the design community and users.

(5) The sensitivity of the demand predictions to different load patterns made in nonlinear static analysis becomes significant as the height of a structure increases. In fact, neglecting the effect of the higher modes in the evaluation process may lead to significant underestimation of the seismic demands of the structure.

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