4st International Conference on
Structural Engineering
17 - 18 February 2018, Olympic Hotel, Tehran, Iran

Paper ID: 4011-IRAST(R1)

Behavior Factor of RCMRFs Based on Incremental Dynamic Analysis

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Abstract

The seismic codes usually extract the design forces by using a behaviour factor that modifies the 'linear' force system to an equivalent one to account approximately for the nonlinear effects. The issue of the behavior factor is of great interest in earthquake-resistant design, as the design forces are inversely proportional to the value adopted for this parameter. The factor that reflects the capability of a structure to dissipate energy through inelastic behavior is the ratio of the strength required to maintain the structure elastic to the inelastic design strength of the structure. The objective of this study is to determine the behavior factor of Reinforced Concrete Moment Resisting Frames (RCMRFs) by employing nonlinear incremental dynamic analysis. First, several sample frames are selected and designed based on the ACI regulations. Then nonlinear incremental dynamic analysis is performed on the structures and the behavior factors are determined. The results are compared with those from pushover analyses. It is shown that the behavior factors resulting from the nonlinear incremental dynamic analysis are generally larger than those obtained from pushover analysis and were similar to the values given in the design code.

Keywords:

Reinforced Concrete Moment Resisting Frames; Incremental Dynamic Analysis; Behavior Factor

1- INTRODUCTION

The behavior factor, which was first proposed in the ATC report(1978), was selected through committee consensus based on the observed performance of buildings during past earthquakes and on the estimates of system overstrength and damping, etc (ATC.(1995)). This factor is determined by the product of three factors including overstrength factor, ductility factor, and redundancy factor (ATC.(1995)). Various studies have already dealt with matters relating to the behavior factor for structures. For example, Hwang and Jav (1989) presented a statistical evaluation for the behavior factor of reinforced concrete structures. Based on their empirical formula, the response modification factor is as a function of the maximum ductility ratio, the ratio of viscous damping and ratio of the structural period to the dominant period of earthquake. Borzi and Elnash (2000) i employed a controlled and evenly distributed earthquake data-set to derive values for force reduction factors needed for the structure to reach, and not exceed, a pre-determined level of ductility. They observed that the force modification factors were only slightly influenced by the shape of the hysteretic model used in their derivation and even less sensitive to strong motion characteristics. Chryssanthopoulos et al. (2000) presented a methodology for the probabilistic assessment of behavior factors in EC8-designedreinforced concrete frames . The variability in the actual behavior factor of the frames was estimated and the appropriateness of the EC8 specified value was assessed. Maheri and Akbari (2003) evaluated the seismic behavior factor for steel X-braced and knee-braced RC buildings. They proposed tentative values for behavior factor of steel-braced moment-resisting RC frame dual systems for different ductility demands. Kim and Choi (2005) evaluated the overstrength, ductility, and the response modification factors of special concentric braced frames and ordinary concentric braced frames by performing nonlinear analysis of model structures with various stories and span lengths. According to the results, the response modification factors of model structures computed from pushover analysis were generally smaller than the values given in the design codes.

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Hatzigeorgiou (2007) evaluated behavior factors for nonlinear structures subjected to multiple near-fault earthquakes. A comprehensive nonlinear regression analysis was carried out to provide simple and unique empirical expressions for the behavior factor. The results showed that these expressions provide a good estimation of mean behavior factors. In this study, it was also shown that frequent/smaller earthquakes

necessitate similar behavior factors while seismic sequences lead to smaller behavior factors in comparison with the "design earthquake". Castiglioni and Zambrano (2010) presented a method for the definition of the behavior factor for multi-storey steel frames accounting for cumulative damage in structural components. Their proposed approach can be useful in performance-based design, since the linear procedure allows the definition of the behavior factor corresponding to different level of damage or collapse prevention. Mahmoudi and Zaree (2010) evaluated the response modification factors of conventional concentric braced frames (CBFs) as well as buckling restrained braced frames (BRBFs). They showed that the response modification factors for BRBFs were higher than the CBFs one. Izadinia et al. (2012) evaluated response modification factor for steel moment-resisting frames by different pushover analysis methods. They reported that the maximum relative difference for response modification factors was about 16% obtained by the methods of conventional and adaptive pushover analyses. Mandel et al. (2013) evaluated performance-based behavior factor of reinforced concrete frames. The results showed that the behavior factors are smaller than the value considered in the seismic codes. Studying the previous researches reveals that most of previous studies related to response factor have been done based on the nonlinear static analysis results and no research has yet been conducted for evaluation of behavior factors of RCMRFs by using incremental dynamic analysis. Fanaie and Ezzatshoar (2014) Studied the seismic behavior of gate braced frames by incremental dynamic analysis (IDA). They suggested values of 3.5 and 5 for response modification factor in ultimate limit state and allowable stress methods, respectively. They used the incremental dynamic analysis to plot the curves of failure. Maheri and Akbari (2003) investigated the effects of some parameters influencing the value of seismic behaviour factor, R, for steel X-braced and knee-braced RC buildings, including the height of the frame, share of bracing system from the applied load and the type of bracing system. The results showed that the height of this type of lateral load-resisting system has a profound effect on the R factor, as it directly affects the ductility capacity of the dual system.

The present study focuses on the evaluation of overstrength, ductility, and response modification factors of four RCMRFs, designed in accordance with Iranian code 2800 and AISC seismic provisions. Nonlinear static pushover analyses are also carried out to obtain behavior factors, and the results are compared with those resulting from nonlinear incremental dynamic analyses and then are evaluated.

2- BEHAVIOR FACTOR

The behavior factor is generally defined as the ratio of the elastic strength demand to the inelastic strength demand. The value of the behavior factor mainly depends on the ductility of the structure, on the strength reserves that normally exist in a structure, and on the damping of the structure. all these factors directly affect the energy dissipation capacity of a structure. Several theoretical approaches have been proposed to compute the response modification factor, such as the maximum plastic deformation approach, the energy approach, and the low-cycle fatigue approach (Mazzolani FM, Piluso V.(1996)). An appropriate definition of the behavior factor has been suggested by Yong (1991) and is used in this study. According to this method, the response modification factor, R, is determined as the product of the three parameters that influence the seismic response of structures:

$$R = R_o R_\mu R_r \qquad (1)$$

where R_o is the overstrength factor, R_μ is a ductility factor, and R_r is the allowable stress factor. The overstrength factor accounts for the effect that the maximum lateral strength of a structure generally exceeds its design strength. Three components of overstrength factors including design overstrength, material overstrength, and system overstrength can be defined (ISIRI 2800 (2005)). The ductility factor is defined as a measure of the global nonlinear response of a structure.

The allowable stress factor is determined using the ratio of formation limit of the first plastic hinge to the force at the allowable stress limits. According to eq. (1) the response modification factor is determined as the product of the overstrength factor, the allowable stress factor and the ductility factor. Fig.1 represents the capacity curve for a structure, which is developed by a nonlinear incremental dynamic analysis in this study. In this figure, V_e is the maximum seismic demand for elastic response, V_d is the design base shear, V_y is the base shear corresponding to the maximum displacement, Δ_e is the displacement of a corresponding elastic structure, Δ_{max} is the Maximum displacement of a structure, and Δ_y is the yield displacement of a structure.

The ductility factor R_μ and the overstrength factor R_o are determined from the following relations:

$$R_\mu = \frac{Ve}{Vy} \qquad (2)$$

$$R_o = \frac{Vy}{Vd} \quad (3)$$

Based on the design codes, Vs (the shear of corresponding to the first plastic hinge) is reduced to Vw for designing using allowable stress method. Hence the allowable stress factor is defined as follows:

$$R_{r=} = \frac{Vs}{Vw} \quad (3)$$

It should be noted that response modification factors are dependent on the building performance, which is a combination of both structural and nonstructural components and is expressed in terms of building performance levels. These building performance levels are discrete damage states selected from among the infinite spectrum of possible damage states that buildings could experience as a result of earthquake. There are a number of building performance levels (or particular damage states) defined in the literature such as Operational (OP), Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) levels. In this study, LS is considered to compute the response modification factors. The maximum inter-story drift ratio of a RCMRF is limited to 2.0% for this performance level [18].

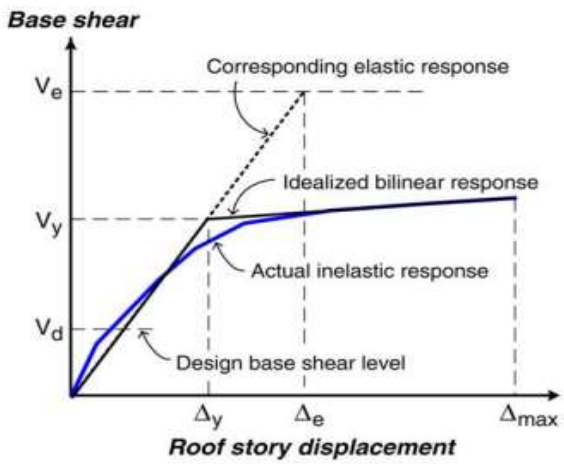


Fig. 1. Capacity curve for a structure

3- STUDIED STRUCTURES

Four two-dimensional reinforced concrete moment resisting frames, as shown in Figure 2 (the number of beams and columns is from left to right and from top to bottom), were designed according to the requirements of Iranian national building code, and Iranian seismic code (Standard 2800 (2005), The ninth issue of the National Building Regulations(2013)), with soil type B (rock site) and the peak ground acceleration (PGA) of 0.35g. The height of each story is 3.2 meters and the length of each bay is 4 meters in all the frames. Based on the code 2800, the response modification factor of 10 was used for design of the frames. The dead and live loads applied on the stories were assumed to be 20 and 8 kN/m, respectively. The beams and columns were designed to resist all lateral seismic loads, and the beam–column joints were assumed to be rigid. The structural design was carried out using the ETABS software. The design details for 3- & 6-story frames are given in Tables 1-4.

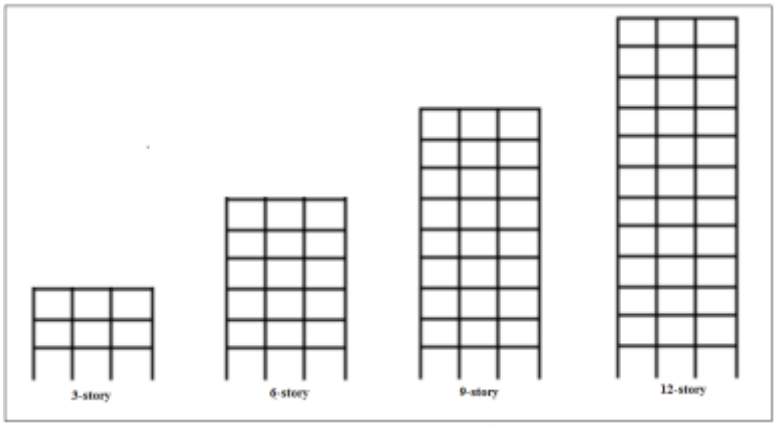


Fig. 2. Configuration of studied structures

Table 1: Details of beam sections designed for 3-storey frame

Beam number	Type	H (mm)	B (mm)	Longitudinal Reinforcement Area (mm^2)				Shear reinforcement
				Left		Right		
				Bot	Top	Bot	Top	
1,4	1	300	250	508.9	1017.9	508.9	763.4	2 ø 8 @ 60
2,5	2	300	250	508.9	1017.9	508.9	1017.9	2 ø 8 @ 60
3,6	3	300	250	508.9	763.4	508.9	1017.9	2 ø 8 @ 60
7	4	300	250	508.9	763.4	254.5	508.9	2 ø 8 @ 60
8	5	300	250	508.9	763.4	508.9	763.4	2 ø 8 @ 60
9	6	300	250	254.5	508.9	508.9	763.4	2 ø 8 @ 60

Table 2: Details of column sections designed for 3-storey frame

column number	Type	B (mm)	H (mm)	Reinforcement on one face	Longitudinal Reinforcement Area (mm^2)	Shear reinforcement
1,4	1	300	300	2 ø 18	509	2 ø 8 @ 75
2,3	2	400	400	3 ø 18	763.5	2 ø 8 @ 100
5,8	3	300	300	2 ø 18	509	2 ø 8 @ 75
6,7	4	400	400	3 ø 18	763.5	2 ø 8 @ 100
9,12	5	300	300	2 ø 18	509	2 ø 8 @ 75
10,11	6	400	400	3 ø 18	763.5	2 ø 8 @ 100

Table 3: Details of beam sections designed for 6-storey frame

Beam number	Type	H (mm)	B (mm)	Longitudinal Reinforcement Area (mm^2)				Shear reinforcement
				Left		Right		
				Bot	Top	Bot	Top	

1,2,3,4,5,6,7,8,9, 10,11,12,13,14,15	1	500	300	628.3	942.5	628.3	942.5	2 Ø 8 @ 110
16	2	450	250	628.3	942.5	628.3	628.3	2 Ø 8 @ 95
17	3	450	250	628.3	942.5	628.3	942.5	2 Ø 8 @ 95
18	4	450	250	628.3	628.3	628.3	942.5	2 Ø 8 @ 95
19,20,21,23	5	450	250	628.3	628.3	628.3	628.3	2 Ø 8 @ 95
22	6	450	250	628.3	628.3	314.2	628.3	2 Ø 8 @ 95
24	7	450	250	314.2	628.3	628.3	628.3	2 Ø 8 @ 95
25,26,27	8	300	250	314.2	628.3	314.2	628.3	2 Ø 8 @ 60

Table 4: Details of column sections designed for 6-storey frame

column number	Type	B (mm)	H (mm)	reinforcement on one face	Longitudinal Reinforcement Area (mm^2)	Shear reinforcement
1,4	1	400	400	3Ø20	942.48	2Ø8@100
2,3	2	450	450	3Ø20	942.48	2Ø8@110
5,8,22,23	3	400	400	3Ø20	942.48	2Ø8@100
6,7	4	450	450	3Ø20	942.48	2Ø8@110
9,12	5	400	400	3Ø20	942.48	2Ø8@100
10,11	6	450	450	3Ø20	942.48	2Ø8@110
13,16,26,27	7	400	400	3Ø20	942.48	2Ø8@100
14,15	8	450	450	3Ø20	942.48	2Ø8@110
17,20	9	350	350	2Ø20	628.32	2Ø8@85
18,19	10	400	400	3Ø20	942.48	2Ø8@100
21,24,30,31	11	350	350	2Ø20	628.32	2Ø8@85
25,28	12	350	350	2Ø20	628.32	2Ø8@85
33,36	13	300	300	2Ø20	628.32	2Ø8@75
29,32,34,35	14	350	350	2Ø20	628.32	2Ø8@85

4- NONLINEAR ANALYSIS OF STUDIED STRUCTURES

Pushover analysis, which is a simple method to estimate component and system deformation demands, has been generally used to determine the behavior factors of structures. Despite its capabilities, the application of pushover analysis is not limitation free. For example, the procedure is an approximate method, and is not suitable for buildings in which higher mode effects are significant (Mwafy AM, Elnashai AS.(2001)). Incremental Dynamic Analysis (IDA), which is a more accurate and suitable procedure, is a parametric analysis method that has emerged in several different forms to estimate more thoroughly structural performance under seismic loads. In this study, the incremental dynamic analysis is employed to obtain the behavior factors of the studied structures, and the results are compared with those obtained from pushover analysis. The selected earthquake records for incremental dynamic analysis have been given in Table 5 and design spectrum and response spectra of the earthquakes have been shown in Fig. 3.

Table 5: Earthquake selected for incremental dynamic analysis

Number	Record	Component	Station	PGA	Distanceof Fault	Magnitude	Spectrum Acceleration
1	**Cape Mendocino 1**	**East**	**Fortuna Blvd**	**0.116**	**23.6**	**7.1**	0.098
2	**Cape Mendocino 2**	**North**	**Fortuna Blvd**	**0.114**	**23.6**	**7.1**	0.101
3	**Loma Prieta**	**East**	**Mission San Jose**	**0.124**	**43**	**6.9**	0.092
4	**Northridge 1**	**North**	**Fremont School**	**0.079**	**35.7**	**6.7**	0.086
5	**Northridge 2**	**East**	**Saran**	**0.076**	**34.2**	**6.7**	0.072
6	**Chi-Chi**	**East**	**ALS**	**0.183**	**15.29**	**7.6**	0.138

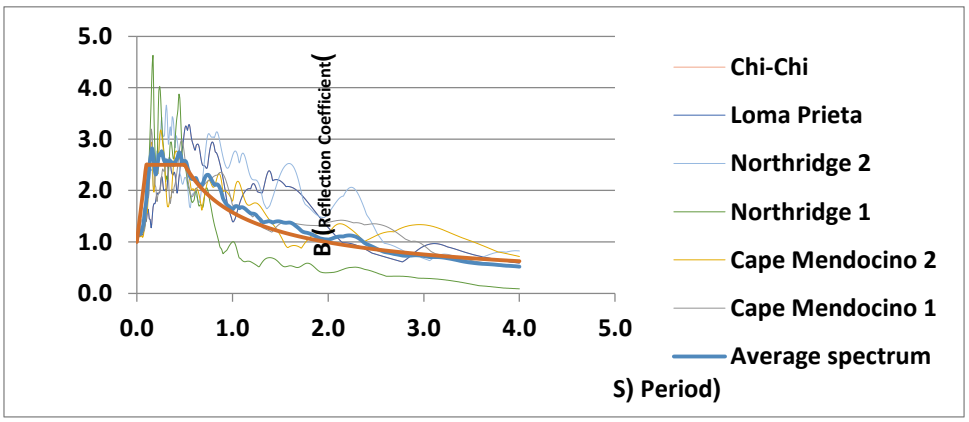


Fig. 3. Design spectrum and response spectra of selected earthquake records

Frequency analyses are first carried out by using the program IDARC (2006) to determine the elastic natural periods and mode shapes of the structures. Then pushover and IDA analyses by using the program SeismoStruct softwar (2013), are carried out to evaluate the global yield limit state and the structural capacity. The P–Δ effect is considered in the analysis.

The capacity curves of all the studied frames have been determined using both IDA and pushover analyses. As a sample, Fig. 4 shows the capacity curves of the 9-story frame. In this figure, it can be observed that the stiffness of the frame decreases slightly by the global yielding of structures. This is due to the damping and hardening effects. The maximum strength is about three times as high as the design base shear. Fig. 5 shows the inter-story drift ratio of the nine-story structure. It can be observed that large drift occurs in lower stories where first yielding occurs in beams and column.

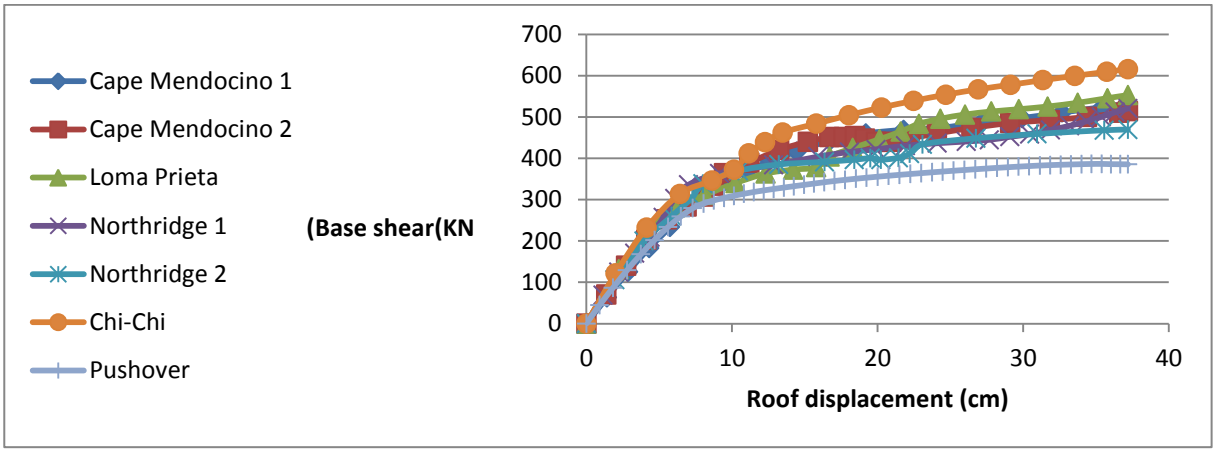


Fig. 4. Capacity curves of the nine-story structure

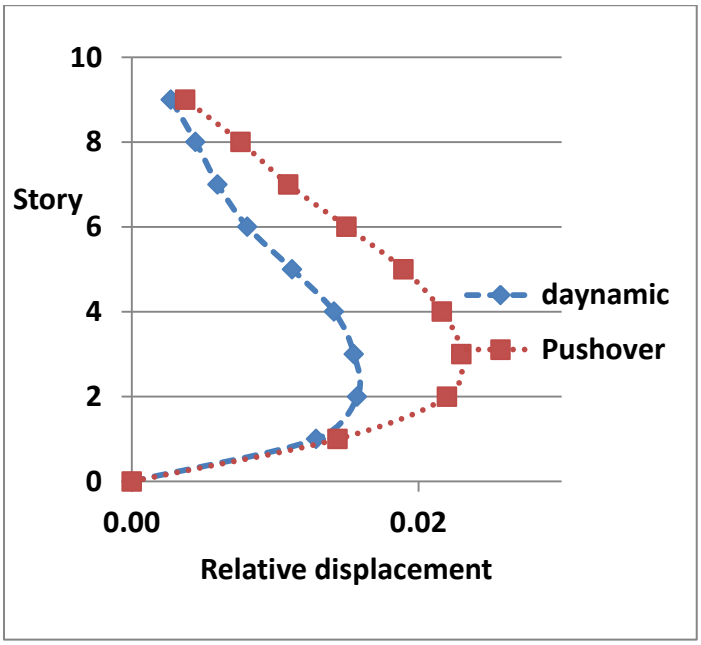
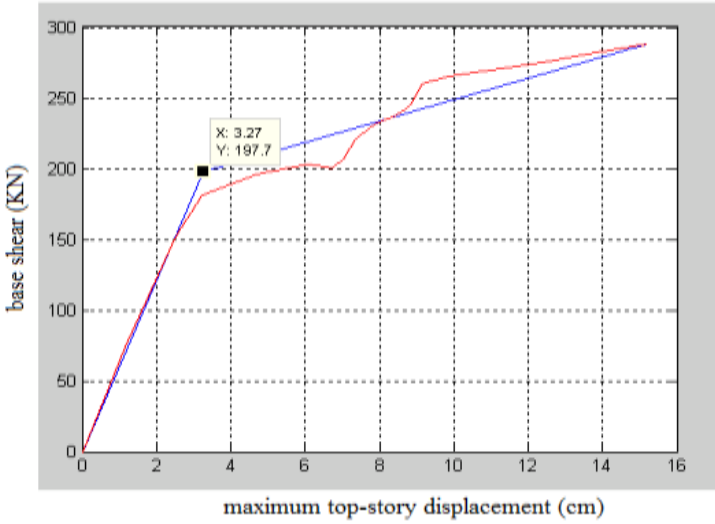


Fig. 5. Drift ratios of the nine-story structure

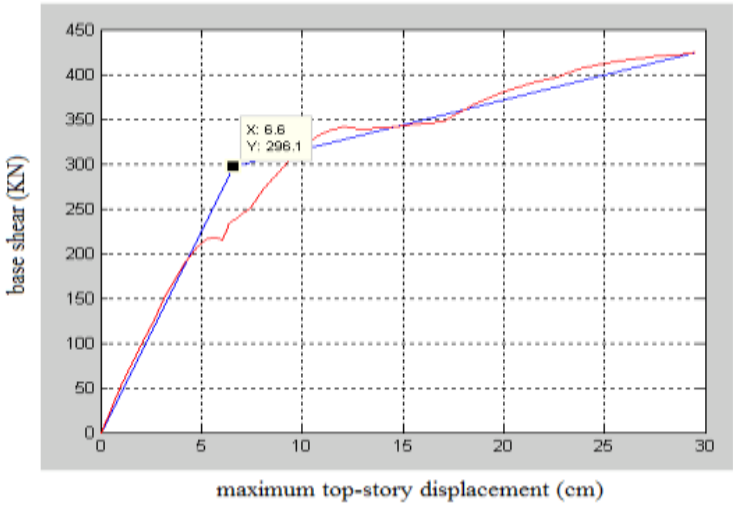
5- EVALUATION OF RESPONSE MODIFICATION FACTORS

5.1. Overstrength factors

The capacity curves resulting from incremental dynamic analysis and pushover analysis are utilized to evaluate overstrength factors. The yield points are determined based on the recommended criteria in FEMA-356 (200) (Fig. 6.a & b). The overstrength factors are plotted in Fig. 7. It can be observed that the overstrength factors of RCMRFs increase as the number of stories decreases. The overstrength factors obtained from the static pushover method are generally larger than the ones obtained from nonlinear incremental dynamic analysis. All the overstrength factors resulting from both IDA and pushover methods are smaller than 2 and greater than 1.5. These results are in agreement with the predicted factors in the FEMA-369 report (2001) .



(a) 3-story frame



(b) 6-story frame

Fig.6: Bilinear capacity curve of 3- & 6-story frames subjected to Northridge earthquake

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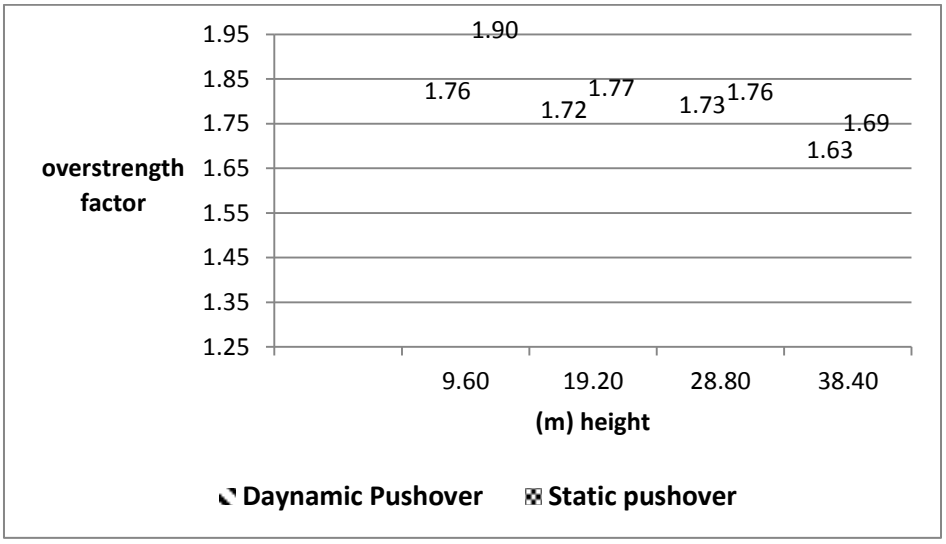

Fig. 7. Overstrength factors of model structures

5.2. Ductility factors

The ductility factor Rμ was obtained using the system ductility ratio μ by the procedure proposed by Newmarkand Hall (1982) and Miranda and Bertero (1994). Newmark and Hall proposed the following equation for the ductility factor:

$$R\mu = \begin{cases} 1 & T < 0.03\ s \\ \sqrt{2\mu - 1} & 0.12 < T < 0.03\ s \\ \mu & T > 1.0\ s \end{cases} \qquad (5)$$

where T is the natural period of the structure. Miranda and Bertero developed the following relationship to determine the ductility factor using 124 ground motions recorded on a wide range of soil conditions:

$$R\mu = \frac{\mu - 1}{\Phi} + 1 \qquad (6)$$

where Φ is a coefficient reflecting a soil condition and is determined as follows:

$$\Phi = 1 + \frac{1}{10T - \mu T} - \frac{1}{2T} e^{-1.5(ln(T)-0.6)2}$$

The system ductility ratio μ is obtained by dividing the roof displacement at the limit state by the system yield displacement. Figs. 8 & 9 show the ductility factor Rμ of the studied structures when the roof displacement reaches the target displacement corresponding to the life safety performance level. In most cases, the factors computed by Newmark and Hall's method are larger than those resulting from Miranda and Bertero's method. It can be observed that the mean ductility factors increase as the period increases.

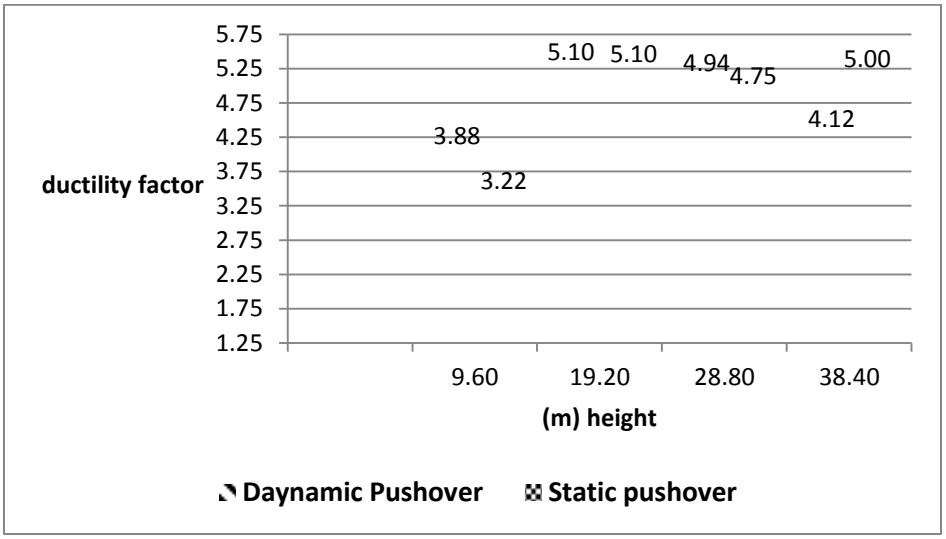

Fig. 8. Mean ductility factors of model structures

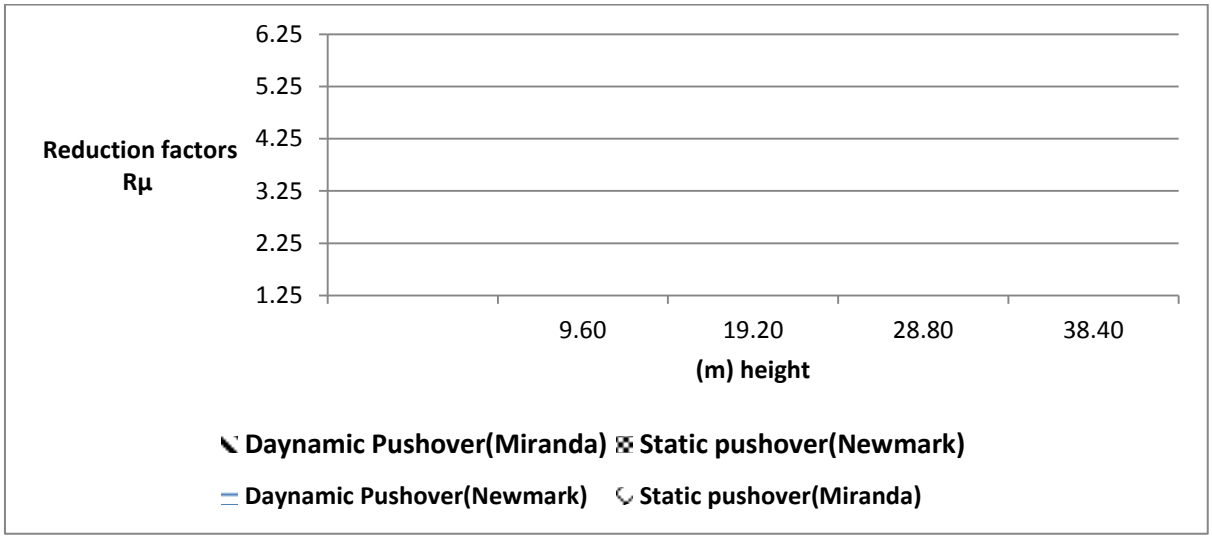

Fig. 9. Ductility factors of model structures based on Miranda's and Newmark methods

5.3 Allowable stress factor

The allowable stress factor Rr was obtained using the equation 3. In both the incremental dynamic and pushover analyses, this factor was specified at the step in which the first plastic hinge was formed. The factor was obtained by dividing the shear corresponding to the first plastic hinge to the design shear. To increase the accuracy of results, a large number load step was considered. The Fig. 11 shows that the factors obtained from the IDA are greater than the ones resulting from the pushover analysis in the 3- & 12-story frames and smaller than those in the 6- & 9-story frames. These factors are ranged between 1.34 to 1.52 for nonlinear incremental dynamic analysis and 1.32 to 1.52 for pushover analysis.

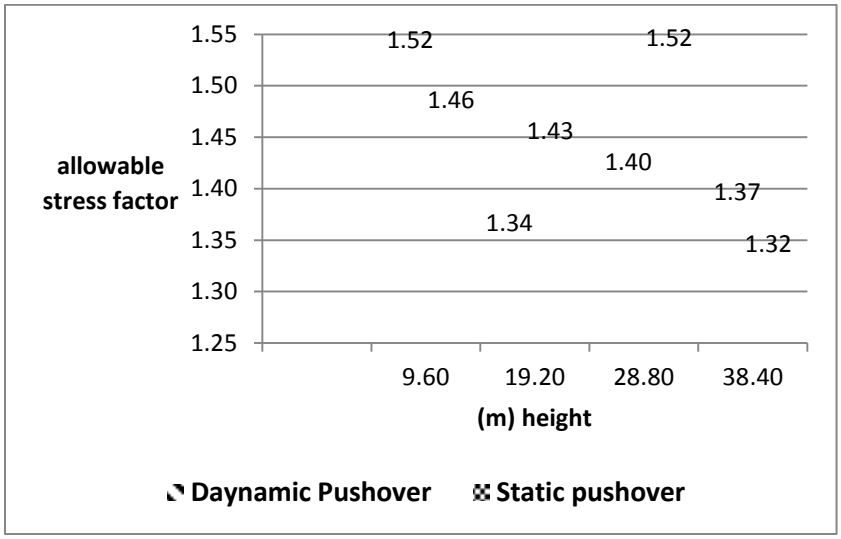

Fig. 11. Allowable stress factors of model structures

5.4. Response modification factors

The response modification factors, presented in Fig. 12, are computed by multiplying the allowable stress, overstrength and the ductility factors, obtained from the IDA or the pushover analysis. For the IDA, the intensities of the time history records were varied by multiplying appropriate scaling factors. The response modification factors were obtained when the roof floor displacement reached the target displacement. To calculate behavior factors, the six dynamic capacity curves were averaged and the average curve was fitted into a bi-linear curve(Somerville P. et al. (1997)). In the six-, nine- and twelve-story RCMRFs, the mean values of the response modification factors resulting from IDA are larger than 10 which is prescribed in code 2800 while the factor of the three-story frame obtained by the IDA is smaller than 10. In the six- & nine-story RCMRFs, the mean values of the response modification factors evaluated by pushover analysis are larger than 10 while the three- & twelve-story structures have the factors smaller than 10. The mean values of the response modification factors for RCMRFs obtained in this study by performing the IDA are somewhat larger than those obtained by pushover analysis. These values are 9.55, 10.56, 11.12, and 10.66 for 3-, 6-, 9-, and 12-story RCMRFs, respectively. These differences can be related to the damping, dynamic nature of the load and higher modes effects.

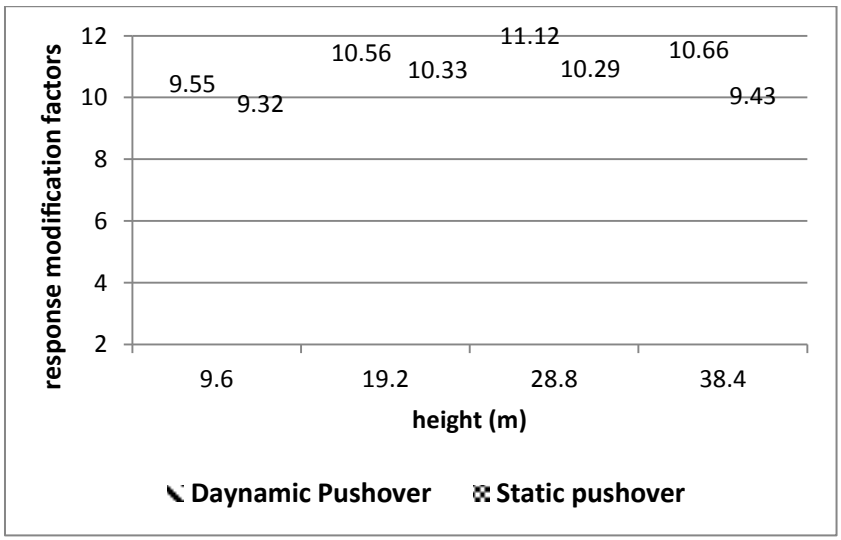

Fig. 12. Response modification factors of model structures

6- CONCLUSIONS

In this study, the overstrength, ductility, allowable stress, and the response modification factors of reinforced concrete moment-resisting frames were evaluated by performing both nonlinear incremental dynamic analysis and static pushover analysis. The results showed that overstrength factors are in agreement with the predicted factors in the FEMA-369. It was shown that the ductility factors increase as the period increases. The response modification factors of RCMRFs resulting from the IDA are somewhat larger than those obtained by pushover analysis.

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