Earthquakes and Structures, *Vol. 16, No. 6 (2019) 655-664*
DOI: https://doi.org/10.12989/eas.2019.16.6.655

Behavior factor of vertically irregular RCMRFs based on incremental dynamic analysis

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(Received February 11, 2019, Revised March 15, 2019, Accepted March 28, 2019)

Abstract. Behavior factor of a structure plays a crucial role in designing and predicting the inelastic responses of it. Recently, irregular buildings have been interested in many designers. To design irregular structures, recognizing the inelastic behavior of them is necessary. The main objective of this study is to determine the behavior factor of irregular Reinforced Concrete Moment Resisting Frames (RCMRFs) via nonlinear Incremental Dynamic Analysis (IDA). To do so, first, several frames are designed according to the regulations of the Iranian national building code. Then the nonlinear incremental dynamic analysis is performed on these structures and the behavior factors are achieved. The acquired results are compared with those obtained using pushover analysis and it is shown that the behavior factors acquired from the nonlinear incremental dynamic analysis are somewhat larger than those obtained from pushover analysis. Eventually, two practical relations are proposed to predict the behavior factor of irregular RCMRFs. Since these relations are based on the simple characteristics of frames such as: irregularity indices, the height and fundamental period, the behavior factor of irregular RCMRFs can be achieved efficiently using these relations. The proposed relations are applied to design of four new irregular RCMRFs and the outcomes confirm the accuracy of the aforementioned relations.

Keywords: behavior factor; irregular frames; incremental dynamic analysis; pushover analysis; reinforce concrete

1. Introduction

In seismic codes, the structure is typically designed for much less lateral forces than would be required to remain the structure completely elastic during seismic ground motions. The philosophy of the seismic codes to reduce the lateral base shear forces is based on this fact that ductile framing systems could tolerate large inelastic deformation without collapse and expand lateral strength more than their design strength as well. The reduced design seismic forces are achieved by the use of response modification factor (behavior factor). This factor reflects the capability of a structure to dissipate energy through inelastic behavior and it is proportional to the ratio of shear force that must be resisted by the building if it remains fully elastic to shear force corresponding to the formation of a first plastic hinge. In many of seismic codes like Iranian seismic code (code 2800), the behavior factors were not separately presented for regular and irregular frames, therefore, not all behavior factors of regular frames may be appropriate for irregular frames. In Eurocode 8, the factor accounting for irregular frames ranges from 80 to 100 percent of regular ones. So there is a maximum reduction of 20% for the structural irregularity. In this research, the behavior factors of irregular RCMRFs are investigated.

The issue of the response modification factor is of great interest in earthquake-resistant design, as the design forces are inversely proportional to the value adopted for this parameter (Elnashai and Di Sarno 2008). Various procedures have been proposed to achieve the response modification factor of structures. Two well-known approaches to calculate the response modification factor of structures are illustrated as follows. In FEMA P695, a statistical approach is presented to calculate the response modification factor. This method has been applied in some studies to acquire the behavior factor of structures (Lee and Kim 2015, Yavarian and Ahmad 2016). In ATC approach, the behavior factor is determined by the product of three factors, including an overstrength factor, a ductility factor, and a redundancy factor (ATC-19 and ATC-34). Some studies such as Whittaker *et al*. (1999), Aliakbari and Shariatmadar (2019), Zerbin *et al*. (2019) have used this approach to obtain the response modification factor of structures. In the present study, the ATC approach will be used to obtain the response modification factor of vertically irregular RCMRFs.

The behavior factor, first presented in the ATC-3-06, based on the observed performance of some buildings during past earthquakes. Various studies have already dealt with matters relating to the behavior factor for structures such as Hwang and Jav (1989), Borzi and Elnashai (2000), Chryssanthopoulos *et al*. (2000), Maheri and Akbari (2003)

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ISSN: 2092-7614 (Print), 2092-7622 (Online)

and Aliakbari and Shariatmadar (2019). Hwang and Jav (1989) presented a statistical evaluation of the behavior factor of reinforced concrete structures. According to their empirical formula, the response modification factor is as a function of the maximum ductility ratio, the ratio of viscous damping and the ratio of the structural period to the dominant period of the earthquake. Borzi and Elnashai (2000) 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 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 probabilistic assessment of behavior factors in EC8-designed reinforced concrete frames. In this study, the variability in the actual behavior factor of the frames was estimated and the appropriateness of the value of EC8 was assessed. 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 structures computed from pushover analysis were generally smaller than the values given in the design codes except in low-rise special concentric braced frames. Hatzigeorgiou (2010) evaluated behavior factors for nonlinear structures subjected to multiple near-fault earthquakes. In this research, 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. Furthermore, it was also shown that frequent/smaller earthquakes necessitated 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-story steel frames accounting for cumulative damage to structural components. Their proposed approach can be useful in performance-based design. Di Sarno and Manfredi (2012) assessed the behavior factors of some existing RC framed structures. 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 the response modification factor for steel moment-resisting frames by using different pushover methods. They reported that the maximum difference between the obtained response modification factors by various pushover methods and the conventional and adaptive pushover analyses was around 16%. Mandal *et al*. (2013) evaluated the 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 research reveals that most of the 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. 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 behavior factor, R, for steel X-braced and knee-braced RC buildings. These parameters consist of the height of the frame, the share of a bracing system from the applied load and the type of a 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. Kim *et al*. (2009) evaluated the behavior factors of a framed structure with chevron-type buckling restrained braces using pushover and incremental dynamic analysis. They proposed that the response modification factors should be more than what is presented in the provision in low-rise structures, and a little lower than the proposal of the provision in the medium-rise structures. Mohammadi *et al*. (2015) numerically assessed the reliability index and the behavior factor of three-dimensional RCMRFs using both deterministic and probabilistic approaches. They showed that the changes in the reliability index and the behavior factor do not always have the same manner because of the increasing of the structural redundancy. Moreover, they illustrated that concerning any increase in the structural redundancy result in increasing the reliability index of the structure. Gomez-Martinez *et al*. (2016) evaluated the response modification factor of wide-beam and deep-beam reinforced concrete frames. They proved when wide beam frames are designed based on serviceability limit states show frames the same seismic capacity with deep beam frames. Zerbin *et al*. (2019) presented an alternative formulation for computing force reduction factors for reinforced concrete wall and frame structures. Their method relies on an analytical model encompassing a single linear elastic cantilever beam with a rotational plastic hinge at the base for wall and a linear elastic 1-story/1-column shear frame with two rotational plastic hinges.

Recently, evaluating the behavior of irregular frames has been interested in many studies, such as Asteris *et al*. (2017), Landi *et al*. (2014), Nezhad and Poursha (2015) and Bosco *et al*. (2015). The present study focuses on the evaluation of overstrength, ductility, and response modification factors of twenty-one irregular RCMRFs, designed in accordance with the Iranian national building code and Iranian seismic code. The research results are limited to the structures with vertical irregularity and plan irregularity is not considered in this study. The behavior factors of the frames are calculated through both nonlinear static analysis and incremental nonlinear dynamic analysis and the outcomes are compared. After that, two relations are proposed to predict the behavior factor of vertically irregular RCMRFs. Since these relations are based on

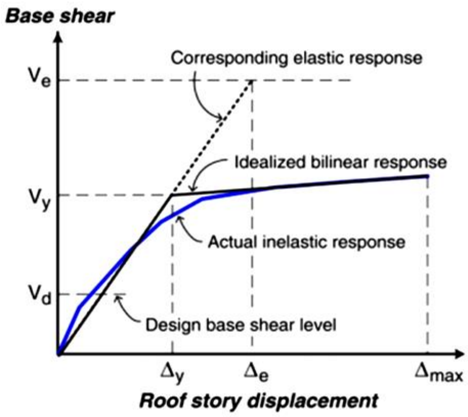


Fig. 1 The capacity curve for a structure

fundamental properties of frames, determining the behavior factor via these relations is so beneficial. Finally, to evaluate the validity of the aforementioned relations, four new vertically irregular RCMRFs are designed once via the proposed behavior factor of these relations and again using the proposed behavior factors of code 2800. Comparing the results illustrated that the performance of these relations is more appropriate than that of Iranian seismic code.

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 and Piluso 1996). An appropriate definition of the behavior factor has been suggested by Yong (1991) and is used in the present study. According to Yong (1991), the response modification factor is determined as the product of the three parameters that influence the seismic response of structures

$$R = R_o\, R_\mu\, R_r \tag{1}$$

where R is the response modification factor, 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 in code 2800. 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 the formation limit of the first plastic hinge to the force at the allowable stress limits (for structures designed based on allowable stress design procedure (in this study, allowable stress factor is assumed to equal to 1). 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 as follows.

$$R_\mu = \frac{Ve}{Vy} \tag{2}$$

$$R_o = \frac{Vy}{Vd} \tag{3}$$

Based on the design codes (like code 2800), *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 using Eq. (4).

$$R_{r=} = \frac{Vs}{Vw} \tag{4}$$

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 with regard to building performance levels. These building performance levels are discrete damage states selected from among the infinite spectrum of possible damage states that buildings likely experience as a result of an 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 (FEMA 695). The maximum inter-story drift ratio of a RCMRF is limited to 2.0% for this performance level.

3. Studied structures

Twenty-one two-dimensional special reinforced concrete moment resisting frames which are ranged from 3 to 12 stories, as shown in Fig. 2, are designed according to the requirements of Iranian national building code and Iranian seismic code, with soil type B (rock site) and the peak ground acceleration (PGA) of 0.35 g. The height of each story is 3.2 meters and the length of each bay is 4 meters. The concrete is assumed to have a cylinder strength of 25 MPa, a modulus of elasticity of 23797.9 MPa, a strain of 0.002 at maximum strength, and an ultimate strain of 0.0035. The modulus of rupture measured from three- or four-point bending tests on plain concrete beams is often used to assess the uniaxial tensile strength of concrete. The value of this modulus for the studied structures is assumed to be 3.00 MPa based on Iranian national building code. The steel has a yield strength of 400 MPa and a modulus of elasticity of 200,000 MPa. The dead and live loads applied on the stories are assumed to be 20 and 8 kN/m, respectively. The beams and columns are designed to resist

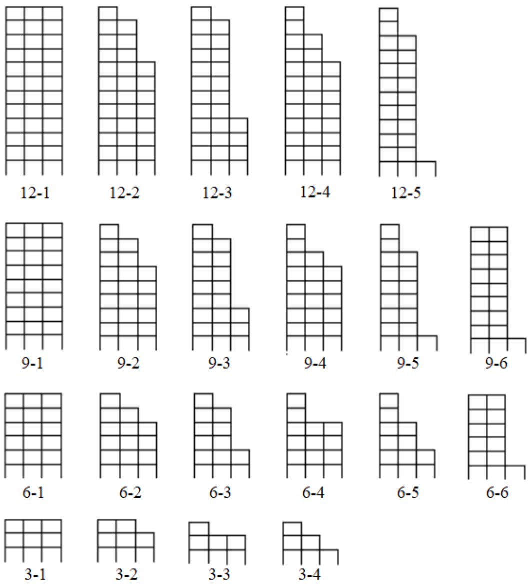


Fig. 2 Configuration of studied structures

Table 1 Characteristics of the designed frames

Frame number	Period	Height (m)	Irregularity Index	
			Vertical	Horizontal
12-1	1.47	38.4	1	1
12-2	1.37	38.4	1.136	1.232
12-3	1.38	38.4	1.136	1.92
12-4	1.35	38.4	1.136	2.46
12-5	1.4	38.4	1.136	5.6
9-1	1.2	28.8	1	1
9-2	1.1	28.8	1.187	1.229
9-3	1.08	28.8	1.187	1.895
9-4	1.06	28.8	1.187	1.226
9-5	1.21	28.8	1.06	5
9-6	1.11	28.8	1.187	4.14
6-1	1.04	19.2	1	1
6-2	0.8	19.2	1.3	1.225
6-3	0.84	19.2	1.3	1.85
6-4	0.79	19.2	1.4	1.25
6-5	0.81	19.2	1.3	1.75
6-6	0.92	19.2	1.1	3.5
3-1	0.68	9.6	1	1
3-2	0.61	9.6	1.25	1.25
3-3	0.54	9.6	2	1.25
3-4	0.54	9.6	1.75	1.75

all lateral seismic loads, and the beam-column joints are assumed to be rigid. Some characteristics of these frames have been summarized in Table 1. It is worth emphasizing that the irregularity indices in Table. 1, are acquired based on the proposed relations by Karavasilis *et al*. (2008). The sectional properties of these frames have been presented by Gholami (2014).

4. Nonlinear analysis of studied structures

Pushover analysis, which is a simple procedure 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 has some limitations. For example, the procedure is an approximate method and is not suitable for buildings in which higher mode effects are significant (Mwafy and Elnashai 2001). Incremental dynamic analysis, which is a more general 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, IDA 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 2 and the design spectrum and response spectra of the earthquakes have been shown in Fig. 3.

Frequency analyses are first carried out by using the program IDARC (2009) to determine the elastic natural periods and the mode shapes of the structures as detailed in (Ghasem Fam 2014). Then pushover and IDA analyses using the program SeismoStruct software are carried out to evaluate the global yield limit state and the structural capacity. The inelastic force-based frame element type-infrmFB is used to simulate the structural elements. Of the four frame element types of SeismoStruct, the aforementioned model is the excellent one because it can capture the nonlinear behavior along the entire length of a

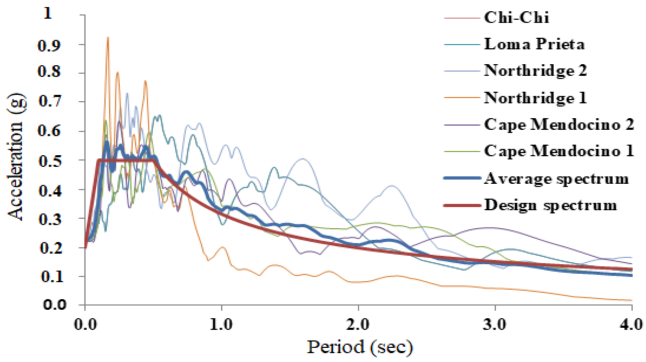


Fig 3 Design spectrum and response spectra of selected earthquake records

Table 2 Earthquake selected for incremental dynamic analysis

Number	Record	Component	Station	PGA	Distance of Fault	Magnitude
1	Cape Mendocino 1	East	Fortuna Blvd	0.116	23.6	7.1
2	Cape Mendocino 2	North	Fortuna Blvd	0.114	23.6	7.1
3	Loma Prieta	East	Mission San Jose	0.124	43	6.9
4	Northridge 1	North	Fremont School	0.079	35.7	6.7
5	Northridge 2	East	Saran	0.076	34.2	6.7
6	Chi-Chi	East	ALS	0.183	15.29	7.6

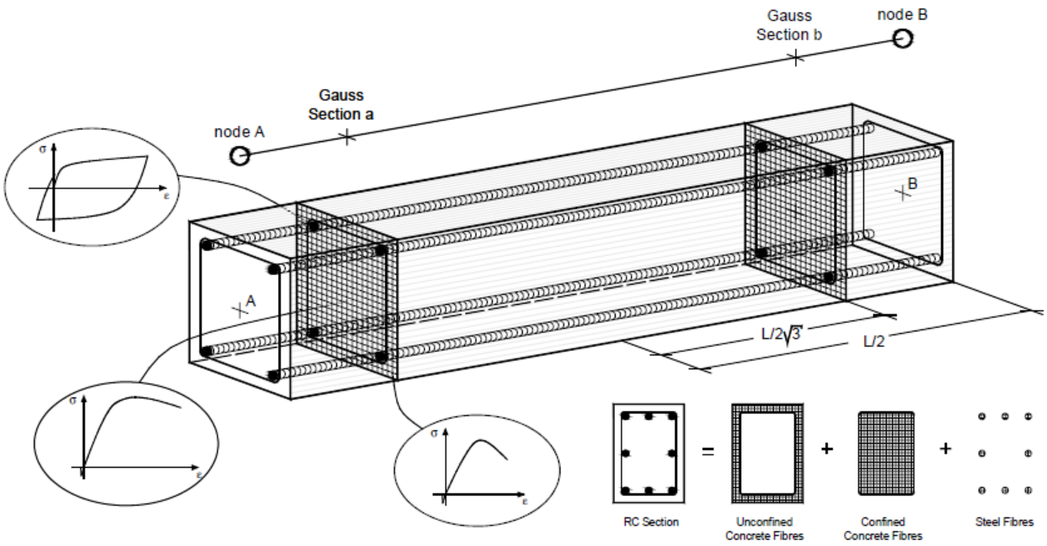


Fig. 4 Discretization of a typical reinforced concrete cross-section (SeismoStruct 2016)

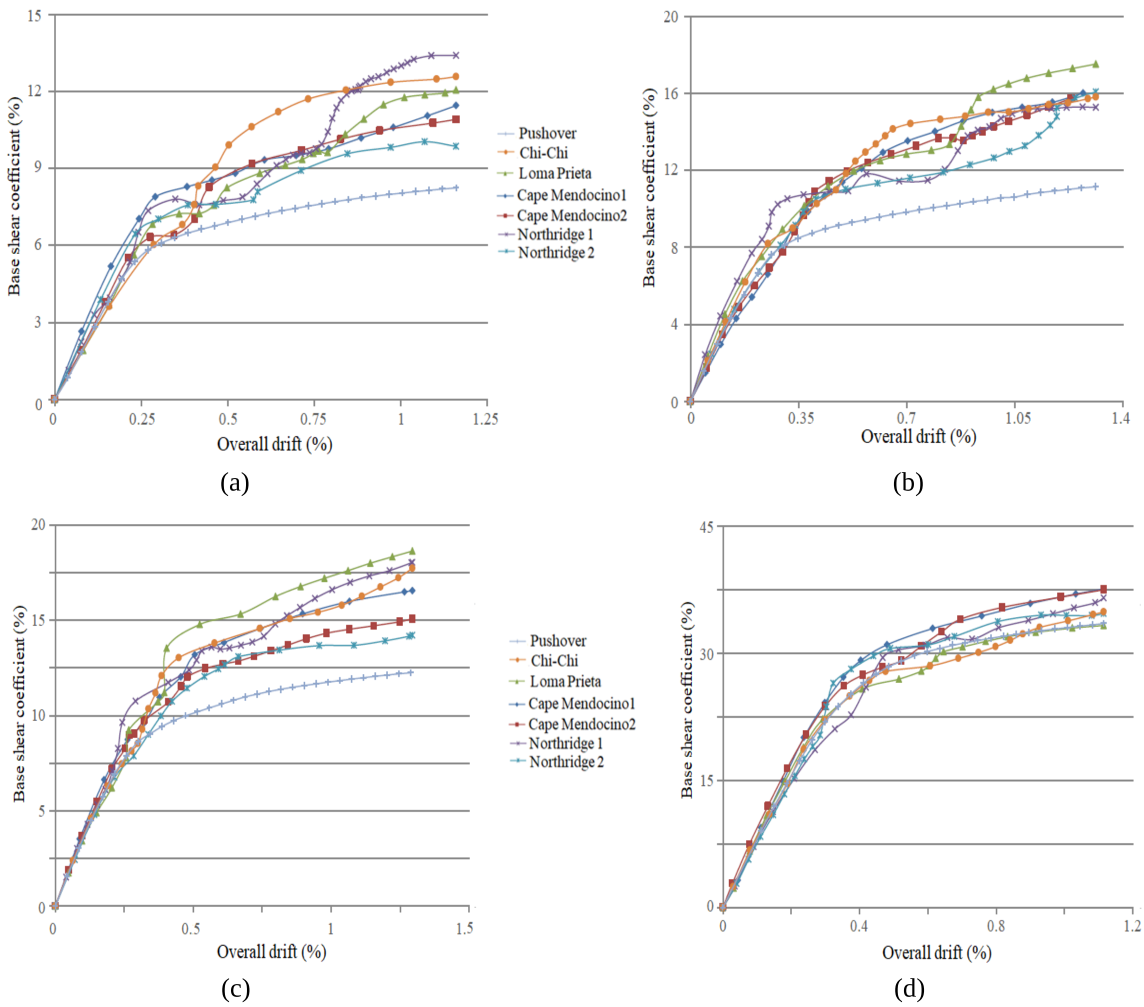
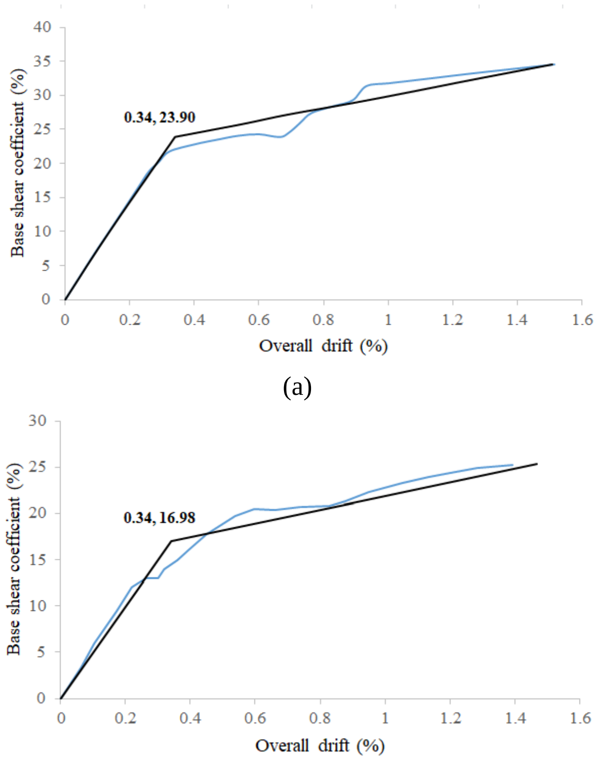


Fig. 5 Capacity curves of the frames (a) 12-5 (b) 9-5 (c) 6-6 (d) 3-4

structural member, even when employing a single element per member (SeismoStruct 2016). Ten element's integration sections are considered in each section and 200 section fibers are used in equilibrium computations performed at each of the element's integration sections (Fig 4). More details about the effect of the number of element's integration sections have been presented by Habibi and Izadpanah (2017).



(a)

(b)

Fig. 6 The bilinear capacity curve of frame (a) 3-1 (b) 6-1, subjected to Northridge earthquake

The *P*-Δ effect is considered in all analyses. The capacity curves of all the studied frames have been determined through both IDA and pushover analyses. The capacity curves of frames 12-5, 9-5, 6-6 and 3-4 are shown in Fig. 5. It should be noticed that in these frames, earthquake loads are applied incrementally on the structures until the target displacement is reached.

As it is shown in Fig. 5, the pushover capacity curves of all frames except frame 3-4 are significantly lower than those of IDA. For frame 3-4, the capacity curve resulted from pushover analysis is close to the average capacity curve of IDAs.

5. Evaluation of response modification factors

5.1 Overstrength factors

The capacity curves resulted from IDA and pushover analysis are utilized to evaluate overstrength factors. The yield points are determined based on the recommended criteria in FEMA-356 (Fig. 6(a) and (b)). The overstrength factors resulted from IDA and pushover analyses are compared in Fig. 7. It is worth emphasizing that 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.

It is clear that the overstrength factors of RCMRFs increase when the number of stories decreases. Although the overstrength factors obtained from pushover method are generally larger than those obtained from the nonlinear incremental dynamic analysis for regular RCMRFs, the situation is entirely reverse for irregular frames. Majority of the overstrength factors acquired from both IDA and pushover methods are smaller than 2 and greater than 1.5. These outcomes are in compliance with the predicted factors in the FEMA-369.

5.2 Ductility factors

The ductility factor is obtained based on the ductility ratio proposed by Miranda and Bertero (1994). Newmark and Hall proposed the following equation for the ductility factor

$$R_\mu = \begin{cases} 1 & T < 0.03\ \mathrm{s} \\ \sqrt{2\mu - 1} & 0.12 < T < 0.03\ \mathrm{s} \\ \mu & T > 1.0\ \mathrm{s} \end{cases} \tag{5}$$

Where R_μ is the ductility factor, μ is the ductility ratio and 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

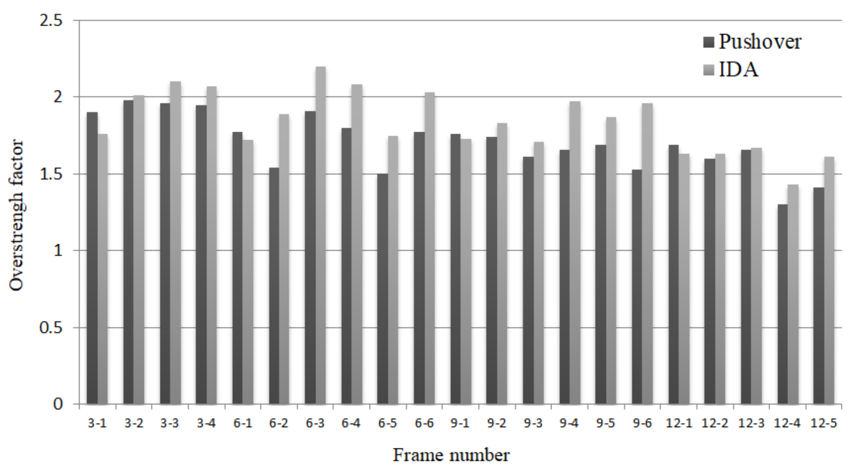


Fig. 7 Overstrength factors of considered frames

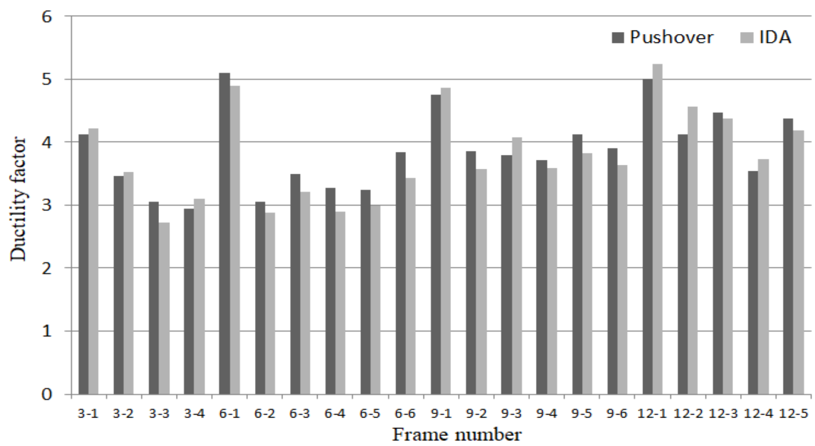


Fig. 8 Ductility factors of considered frames

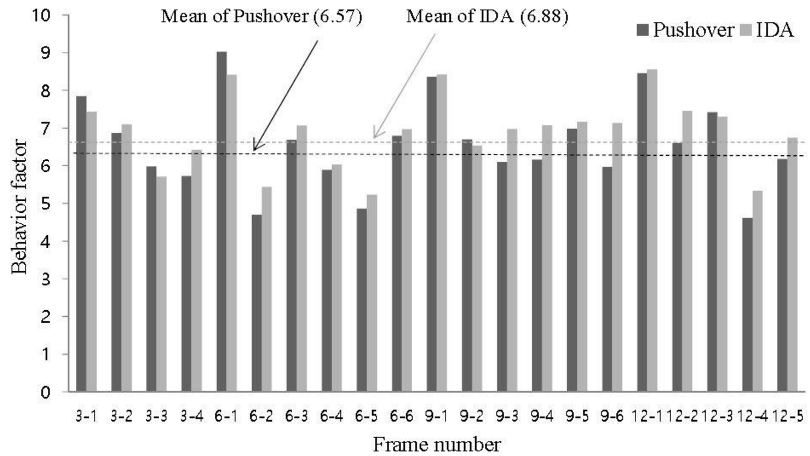


Fig. 9 Behavior factors of studied structures

wide range of soil conditions

$$R_\mu = \frac{\mu - 1}{\Phi} + 1 \tag{6}$$

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

$$\Phi = 1 + \frac{1}{10T - \mu T} - \frac{1}{2T} e^{-1.5(\ln(T) - 0.6)2} \tag{7}$$

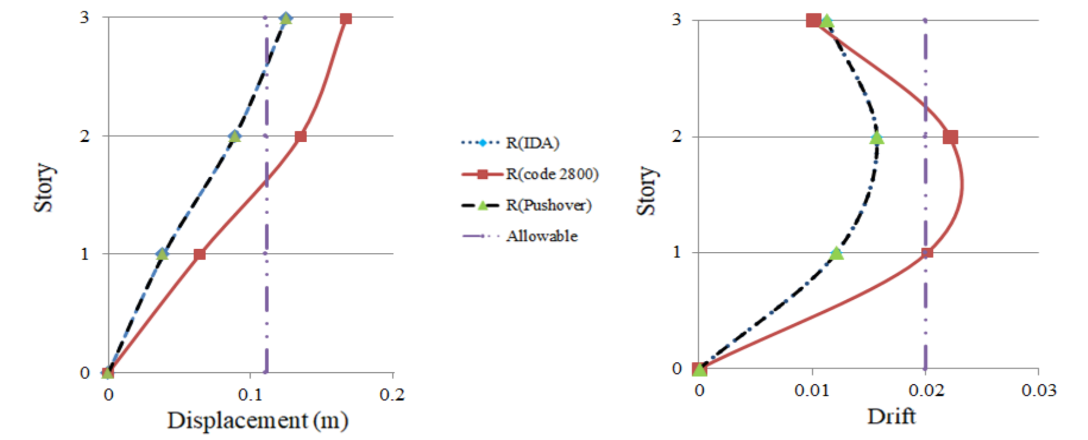
The ductility factor (μ) is obtained by dividing the roof displacement by the yield displacement. The ductility factor (R_μ) of the studied structures when the roof displacement reaches the target displacement corresponding to the life safety performance level is depicted in Fig. 8.

Fig. 8 shows that the ductility factors of regular three-story frame resulting from the pushover and IDA analyses are 4.1 and 4.2, respectively. These values are greater than ones of all the irregular three-story frames. Similar results are observed for six-story, nine-story and twelve-story frames. Accordingly, the ductility factor of regular RCMRFs is more than irregular ones. Furthermore, it can be seen that the ductility factor increase when rising the height of the frame.

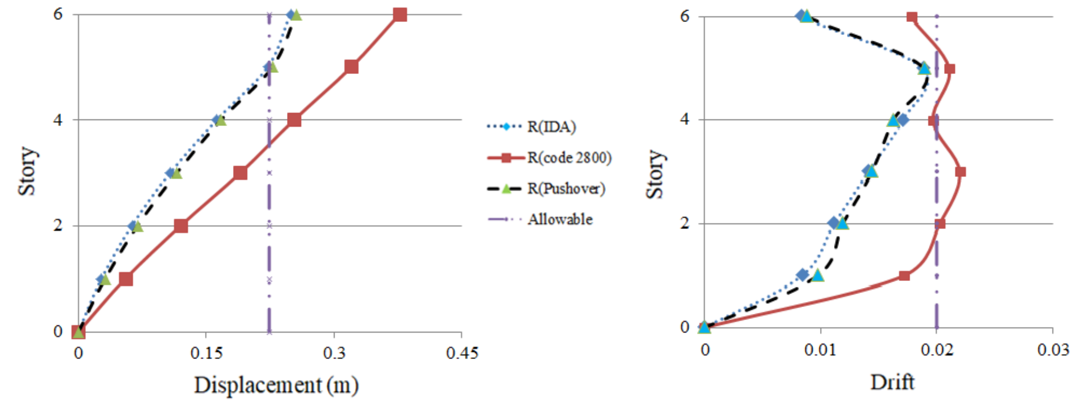
5.3 Response modification factors

In this section, first, the behavior factors of studied frames are calculated by multiplying the allowable stress, overstrength and the ductility factors obtained in previous sections in both IDA and pushover analyses. After that based on obtained behavior factors, two practical relations are presented that using them, the behavior factor of RCMRFs whether regular or irregular can be acquired easily. In Fig. 9, the calculated behavior factors using IDA and pushover analyses are shown.

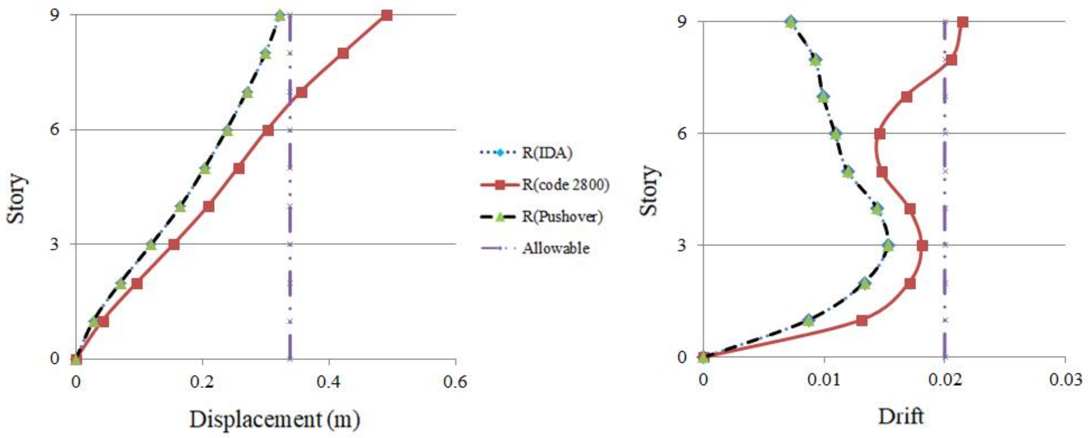
As it is evident, the obtained response modification factors of all irregular frames, whether via pushover or IDA analysis are lower than 7.5 which is prescribed in code 2800. The regular frames (3-1, 6-1, 9-1 and 12-1) account for the highest behavior factors in each category of frames (3-story, 6-story, 9-story, and 12-story). In the frames 3-1, 3-3, 6-1, 9-2 and 12-3 the behavior factors acquired by pushover analysis are larger than IDA, yet in other frames, the behavior factors of IDA are moderately more that pushover analysis. It is clear that for higher frames, the obtained response modification factors through IDA are more than Pushover analysis. The highest difference between pushover and IDA behavior factors belongs to frame 9-6 (20% approximately). It seems, as a rule of thumb, the behavior factor will dramatically drop when irregularity factor increases. It is due to fall of the ductility factor for irregular RCMRFs. The average of the calculated behavior factors by pushover analysis is equal to 6.57 and



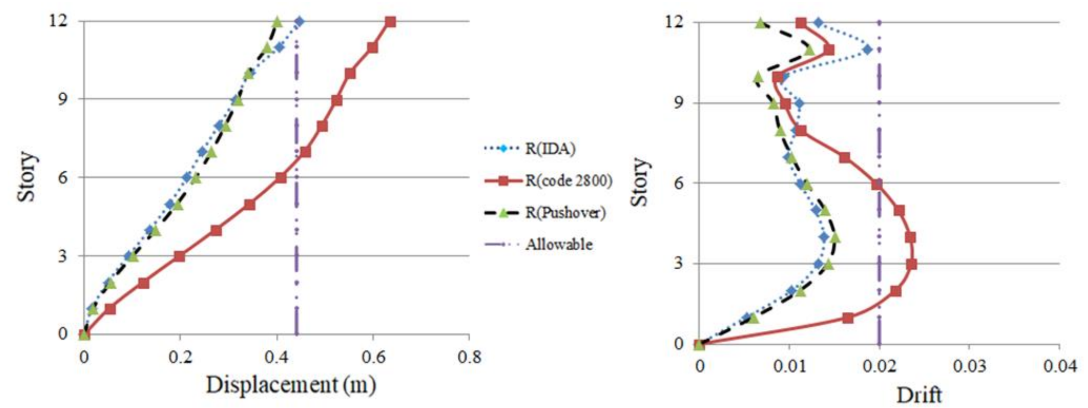
(a)



(b)



(c)



(d)

Fig. 10 Story displacements and interstory drifts of frame (a) 3-3 (b) 6-4 (c) 9-5 (d) 12-5 that of IDA is 6.88. It is vivid that the behavior factors of regular frames are more than average ones in both pushover and IDA analyses, whereas those of irregular ones fluctuate around average behavior factors. Among obtained behavior factors of considered irregular frames, the lowest gap with the average figures belongs to 9-story frames and the largest difference belongs to the 6-story category.

The hands-on proposed relations to compute the behavior factor of irregular RCMRFs are as follows.

$$R_s = 99.28 \times T^{1.63} \times b^{-0.063} \times s^{0.129} \times H^{-0.873} \quad (8)$$

$$R_d = 50.43 \times T^{1.03} \times b^{-0.029} \times s^{-0.097} \times H^{-0.683} \quad (9)$$

Where R_s and R_d are the behavior factor resulted in pushover and incremental dynamic analyses. T is the fundamental period of the frame, b and s are the horizontal and vertical irregularity indices respectively (Karavasilis *et al*. 2008) and H is the height of the frame. In the next section, the validity of the proposed relations is assessed.

5.4 Verification of the proposed relations

To evaluate the accuracy of the presented relations, the frames 3-4, 6-3, 9-6 and 12-5 are redesigned using the acquired behavior factors from Eqs. (8) and (9) (*R*(Pushover) and R(IDA)). Then nonlinear dynamic analyses of these redesigned frames subjected to six seismic ground motions (Table 2) are performed and the floor displacements, as well as inter-story drifts are compared with those of frames 3-3, 6-4, 9-5 and 12-5 which are designed according to the proposed behavior factors of Iranian seismic code 2800 (*R*(code 2800)) (Fig. 10).

It is noticeable in Fig. 10 that the displacements of higher stories for all frames which are designed based on the behavior factors of code 2800 are more than the allowable permitted limitation of code 2800 for life safety level whereas in the cases that are designed according to the presented relations of this research, the figures are almost close to restrictions values. In terms of inter-story drifts, the values of drifts in all frames that are designed according to behavior factors of code 2800 exceed from allowable ones while the figures for all frames that are designed based on proposed relations of this study are in allowed range.

6. Conclusions

In this study, the overstrength, ductility and the response modification factors of irregular reinforced concrete moment-resisting frames were evaluated performing both nonlinear incremental dynamic analysis and static pushover analyses. It was shown that the proposed behavior factor of Iranian seismic code for RCMRFs is not reliable for irregular ones; therefore, two new relations based on fundamental period, irregularity indices and height of the frame to acquire the response modification factors of RCMRFs were presented. According to obtained results, the following outcomes can be outlined:

• The obtained values for behavior factors of regular RCMRFs are more than 7.5 which is prescribed in code 2800 while those of irregular ones are less than 7.5.

• For some frames, the behavior factors of IDA are lower than pushover analysis, whereas the mean value of behavior factor resulting from IDA (6.88) is more than the one from pushover results (6.57). The highest gap between IDA and pushover response modification factors is around 20 percent.

• The ductility factors of regular three-story frame resulting from the pushover and IDA analyses are 4.1 and 4.2, respectively. These values are greater than ones of all the irregular three-story frames. Similar results have been achieved for six-story, nine-story and twelve-story frames. Accordingly, the ductility factor of regular RCMRFs is more than irregular ones.

• The irregular RCMRFs designed based on the behavior factors of Iranian seismic code do not satisfy the limitation of allowable inter-story drift for life safety level, whereas those which are designed based on the proposed response modification factors of this study show appropriate seismic performance.

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