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Deflection amplification factor for estimating seismic lateral deformations of RC frames

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Abstract: Different values have been assigned to the ratio of the deflection amplification factor (C_d) to the response modification factor (R) for a specified force-resisting system in the seismic design provisions while the same application is defined for it. An analytical study of the seismic responses of several reinforced concrete frames subjected to a suite of earthquake records performed in this research indicate that the stories' overstrength and stiffness distribution along the structural height can affect local deflections more than global ones. Therefore, the C_d/R ratio is calculated based on the ratio of both maximum inelastic to maximum elastic displacements and interstory drifts. Due to damage concentration in some specific stories, the deflection amplification factor calculated based on inelastic interstory drifts was larger than that of the inelastic displacements. Consequently, a minimum value of 1.0 is recommended for the C_d/R ratio in order to estimate maximum inelastic drifts. The ratio of inelastic to elastic displacement was generally found to increase slightly along the structural height for the studied RC models. In addition, it was detected that the story damage indices of the studied RC frames decrease when the inverted value of inelastic interstory drift ratios are increased through a (negative) power form.

Keywords: deflection amplification factor; nonlinear dynamic analysis; inelastic displacement; interstory drift; RC frame

1 Introduction

Reserving significant strength (overstrength, Ω) and ductility are the two main characteristics of structures, which can result in a reduction in design loads by seismic design provisions. These properties are realized in structural design through a response modification factor (R) (Kim and Choi, 2005). A direct relationship between response modification factor and lateral capacity of the structures has been demonstrated by Elnashai and Mwafy (2002). On the other hand, a structure designed based on these reduced (design) forces should be capable of sustaining inelastic deformations. For estimating the maximum inelastic deflection that might occur during an earthquake, the design deflections computed from an elastic analysis are amplified by a deflection amplification factor (C_d) (Uang, 1991; Karami Mohammadi, 2002). This process is referred to as force-based design method. Many researchers have investigated the methods of estimating the inelastic deformations of buildings. The deflection amplification factor was used for this purpose in some of the following cases.

The deflection amplification factor of RC buildings has been evaluated through a statistical procedure by Hwang and Jaw (1989). The ratio of the corresponding nonlinear and linear displacements was defined as the C_d factor. The obtained structural response data from dynamic analyses was used to extract an empirical equation for the C_d factor as a function of the maximum story ductility ratio. They concluded that generally, the calculated design story drifts based on specified C_d factors in NEHRP provisions are overestimated. The ratio of C_d/R was also calculated for two types of steel structures and two special reinforced concrete moment frames (Uang and Maarouf, 1994). This ratio was evaluated as a function of ductility-reduction-factor (R_d). It was observed that the ratio of inelastic to elastic roof displacement increases with an increase in R_d factor. They indicated that within the practical range of ductility reduction factors for the analyzed frames, the ratio of inelastic to elastic interstory drifts varies from 1.0 to 1.5, which can be even higher for the structures with a weak first story. From an analytical study and experimental test outputs of three instrumented reinforced concrete

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building frames, they (Uang and Maarouf, 1995) completed the results by showing that drifts estimated based on UBC or NEHRP provisions are very unconservative than those developed in major ground motions. Another approximate method was presented by Miranda (2002) for investigating the maximum roof and interstory drifts in a simplified model of multistory buildings with non-uniform lateral stiffness. It was a continuation of his research (1999) in which some amplification factors were defined to estimate inelastic deformations of these models with uniform stiffness as functions of number of stories and displacement ductility ratio. However, only the first mode contribution was considered for the models during an earthquake. Miranda (2002) concluded that the difference between the spectral displacement and the maximum roof displacement increases with the number of stories as well as overall deflections developed by flexural behavior. He came to the point that lateral stiffness reduction along the structural height generally would result in decreasing the ratio of the maximum interstory drift to the roof drift. Gupta and Krawinkler (2000) evaluated another process for estimating seismic roof and story drift demands of steel frame buildings. In this study, three modification factors for calculating multi-degree-of-freedom (MDOF), inelasticity, and *P*-delta effects were applied to the first mode spectral displacement demand in order to estimate the roof drift demand of the MDOF structure. Further, a new story-capacity factor was introduced to represent how the distribution of story drift along the building height is affected by the contribution of the story strength and stiffness, taking into account some practical yielding mechanisms (Lu *et al*., 2009). The results demonstrated that generally, the inverse of the story capacity factor correlated well with the actual drift distributions for MDOF frames. Seismic design response factors (Ω, R, C_d) of some concrete wall buildings were assessed by Mwafy (2011) in another research effort. It was concluded that equal values for C_d and R factors can provide an adequate safety margin due to the lower collapse-to-yield interstory drift ratios when compared to PGA ones.

Some other approximate methods have been proposed for estimating the maximum inelastic displacement demand of single-degree-of-freedom (SDOF) systems. However, because of developing local effects such as interstory drifts and local distortions, SDOF models cannot be used to assess the story and local seismic demands in multistory structures (Mollaioli *et al*., 2007). In some of these approximate methods, the maximum displacement of the inelastic SDOF system is estimated as a product of the maximum deformation of a linear elastic system times a displacement modification factor, considering the fact both elastic and inelastic systems have the same damping coefficient and same lateral stiffness (Miranda and Ruiz-Garcia, 2002). The well-known Newmark and Hall relations (1982) for estimating inelastic response spectra from elastic ones can be classified in this group. In their method, the displacement modification factor varies depending on the spectral region where the SDOF system initial period is located. In addition, a detailed statistical study for estimating inelastic displacement demands of a SDOF system has been presented (Miranda, 2000) to compute the ratios of maximum inelastic to elastic displacements (C_μ) from time-history analyses with a large number of earthquake records on firm sites. It was concluded that in the short period spectral region, maximum elastic displacement demands are expected, on average, to be smaller than maximum inelastic demands. While, in the medium and long period spectral regions, the ratios of the maximum inelastic and elastic displacements are, on average, equal to one. Miranda (2000) observed that the coefficient of variation of inelastic displacement ratios increase as the level of inelastic deformation significantly increases.

Other approximate methods use the concept of equivalent viscous damping and effective period through the displacement-based design procedure. In these methods, the maximum displacement of an inelastic system is assumed to be equal to that of an equivalent linear elastic SDOF system having lower stiffness and higher damping in proportion to the main system characteristics. This topic was the issue of research by Gulkan and Sozen (1974), Iwan (1980), Priestley *et al*. (1996), Guyader and Iwan (2006), Browning *et al*. (2008) and Su *et al*. (2012). Vidot-Vega and Kowalsky (2013) showed that maximum interstory drifts of RC frames can be accurately controlled using both force and displacement based design methods. DBD methods are not directly related to the present study and are not discussed here.

There has not been a detailed study to date which evaluates the inelastic displacement profile of designed reinforced concrete frames considering their corresponding elastic displacement from linear dynamic analysis. The purpose of this study is to investigate the correlation between the deflection amplification factor and response modification factor of intermediate RC moment frames based on the ratio of maximum inelastic to elastic both story and roof drifts as the local and global seismic responses. This study also identifies which factors might have a greater affect on developing maximum seismic story drifts in intermediate RC frames and consequently which stories can be susceptible to maximum inelastic story distortions. Therefore, nonlinear and linear time-history analyses are performed on the 24 models of reinforced concrete moment frames to determine the maximum seismic deflections at all the stories when subjected to the seven scaled earthquake records. A constant value for the ratio of C_d/R is proposed and compared with the seismic design provisions. Modified Park & Ang damage index (Park *et al*., 1984; Valles *et al*., 1996) is one of the most popular combined damage indices, which is formed from a linear combination of ductility and energy absorption capacity indices. It is utilized for estimating a quantitative measure of structural damage in this study. Regression analysis is performed to develop an empirical correlation between the computed local (story) damage indices and the corresponding interstory drift ratios for the studied frames.

2 Structural modelling

To assess the inelastic and elastic lateral deformations, 24 intermediate reinforced concrete moment frames were designed based on the Iranian Seismic Design Code (Standard No. 2800-2005), and ACI 318 (2002) provisions. To cover a wide range of building geometries, several frame models with two, three, and four bays were selected. They ranged in height from 2 to 12 stories, in 1-story increments except for the frames with more than 6 stories, which had 2-story increments. The typical bay span and story height were 5 and 3.2 meters, respectively.

The gravity dead and live loads were 6.5 kN/m^2 and 2 kN/m^2, respectively, at the floors and 5.5 kN/m^2 and 1.5 kN/m^2, respectively, at the roof. Figure 1 illustrates plan of the buildings and frames selected for the study. The compressive strength and modulus of elasticity of concrete were considered equal to 30 MPa and 27.4 GPa, respectively. Yield stress of reinforcing steel and its modulus of elasticity were 400 MPa and 200 GPa, respectively. The seismic design base shear was computed in accordance with the Iranian Seismic Design Code (2005) using importance factor of I = 1, seismic zone factor of A = 0.35, Soil Type II and the response modification factor R (R_w) = 7. The Iranian Seismic Design Code (2005) provisions are in accordance with the allowable stress design method.

The beams and columns dimensions and reinforcement details of two frames having 3 bays with 5 and 8 stories are listed in Tables 1 and 2 after the optimized design and controlling the story drifts according to the Iranian Seismic Design Code (2005). In all these frames, the column sections are square with different dimensions while the beams have rectangular cross sections. The depth of the beams is 100 millimeters more than their width to satisfy the design provisions and to have adequate stiffness to help limit lateral drifts. Reinforcement ratios in the proportioned members fluctuated from almost 1% to 2.6% in the first floor columns up to 1% in the upper levels columns. This ratio in girders averaged about 0.79%.

3 Nonlinear seismic analysis of models

Linear and nonlinear time-history analyses were used to estimate elastic and inelastic displacements and interstory drifts of the selected RC frames. Time-history analysis for each building was performed using seven input earthquake records shown in Table 3. For these analyses, IDARC 2D program (Reinhorn *et al*., 2006) was used. To evaluate the hysteretic behavior of structural members, the three parameter "Park hysteretic model" (Park *et al*., 1987; Valles *et al*., 1996) was used in the nonlinear time-history analyses. The model incorporates stiffness degradation (HC), strength deterioration (HBD, HBE) and pinching (HS) parameters. HC, HBD, HBE and HS are considered to be equal to 10, 0.15, 0.08 and 1 (no pinching effect) for columns, and equal to 4, 0.3, 0.15, 1 for beams, respectively. The values are selected based on the specified typical ranges for hysteretic parameters and their controlling effect on the hysteretic behavior of the structural components in the IDARC User's Guide (Reinhorn *et al*., 2006) and its technical report (Valles *et al*., 1996) including some case studies. The Newmark-Beta integration method was utilized for time-history analysis and the Rayleigh damping including the first two modes of vibration was considered.

The earthquake records for the time-history analysis were selected from the database of the Pacific Earthquake Engineering Research Center (PEER, 2010) such that their average acceleration response spectrum (5% of damping) was in accordance with design spectrum of Soil Type II of Standard No. 2800 (2005). Time-history analyses were carried out for the 24 frames under the seven records, which were scaled based on Iranian Seismic Design Code (2005) criteria. First, each record was normalized by its peak ground acceleration (PGA) to 1 g. Then the earthquake records were scaled in such a way that between the range of $0.2T_i$ to $1.5T_i$, the intensity of their 5% damped average acceleration spectrum was not less than that of the Iranian Seismic Design Code

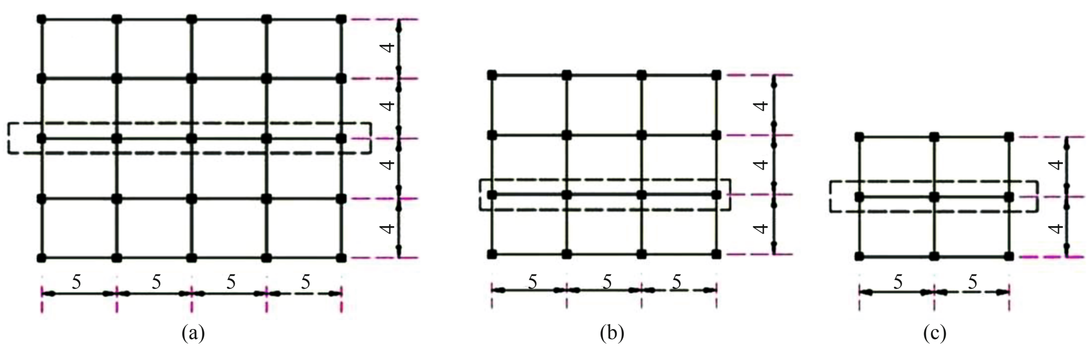

Fig. 1 Plan of structural models with (a) 4 bays, (b) 3 bays and (c) 2 bays (unit: m)

Table 1 Section and reinforcement details for 5-story RC frame with 3 bays

Story No.	Beam						Column				
		Exterior			Interior			Exterior		Interior
	Dim.[a]	Longitudinal Rein. (%)		Transverse Rein.	Longitudinal Rein. (%)		Dim.[a]	Longitudinal Rein. (%)	Transverse Rein.	Longitudinal Rein. (%)
		top	bottom		top	bottom				
1	350×450	1.08	0.53	ϕ8 @ 90	1.04	0.47	450×450	2.17	ϕ8 @ 130	1.99
2	350×450	1.22	0.65	ϕ8 @ 90	1.19	0.60	450×450	1.00	ϕ8 @ 130	1.19
3	300×400	1.36	0.54	ϕ8 @ 90	1.34	0.50	450×450	1.00	ϕ8 @ 130	1.00
4	300×400	1.12	0.45	ϕ8 @ 90	1.08	0.45	400×400	1.18	ϕ8 @ 130	1.51
5	300×400	0.70	0.38	ϕ8 @ 90	0.71	0.38	400×400	1.26	ϕ8 @ 130	1.00

[a] All dimensions (width×depth) are in mm.

Table 2 Section and reinforcement details for 8-story RC frame with 3 bays

Story No.	Beam						Column				
		Exterior			Interior			Exterior		Interior	
	Dim.[a]	Longitudinal Rein. (%)		Transverse Rein.	Longitudinal Rein. (%)		Dim.[a]	Longitudinal Rein. (%)	Transverse Rein.	Transverse Rein.	Longitudinal Rein. (%)
		top	bottom		top	bottom					
1	400×500	0.88	0.86	ϕ8 @ 90	0.51	0.49	550×550	1.85	ϕ 8 @ 130	ϕ 8 @ 130	1.33
2	400×500	1.05	1.02	ϕ8 @ 90	0.68	0.66	550×550	1.00	ϕ 8 @ 130	ϕ 8 @ 130	1.00
3	400×500	1.11	1.08	ϕ8 @ 90	0.73	0.69	550×550	1.00	ϕ 8 @ 130	ϕ 8 @ 130	1.00
4	350×450	1.27	1.27	ϕ8 @ 90	0.72	0.70	500×500	1.00	ϕ 8 @ 130	ϕ 8 @ 130	1.00
5	350×450	1.18	1.19	ϕ8 @ 90	0.62	0.62	500×500	1.00	ϕ 8 @ 130	ϕ 8 @ 130	1.00
6	350×450	1.00	0.99	ϕ8 @ 90	0.47	0.46	500×500	1.00	ϕ 8 @ 130	ϕ 8 @ 130	1.00
7	350×450	0.75	0.77	ϕ8 @ 90	0.37	0.38	450×450	1.00	ϕ 8 @ 130	ϕ 8 @ 85	1.00
8	350×450	0.48	0.50	ϕ8 @ 90	0.29	0.29	450×450	1.00	ϕ 8 @ 130	ϕ 8 @ 85	1.00

[a] All dimensions (width×depth) are in mm.

Table 3 Records summaries before scaling

Earthquake record	Station	PGA (g)	Magnitude (M)
Loma Prieta (1)	57504 Coyote Lake Dam (Downst)	0.484	6.9
Loma Prieta (2)	57217 Coyote Lake Dam (SW Abut)	0.484	6.9
Loma Prieta (3)	58378 APEEL 7 - Pulgas	0.156	6.9
Northridge (1)	14403 LA - 116th St School	0.208	6.7
Northridge (2)	24538 Santa Monica City Hall	0.370	6.7
Tabas	9102 Dayhook	0.406	7.4
Kern County	1095 Taft Lincoln School	0.178	7.4

corresponding to the selected soil type, where T_i is the fundamental period of the structure. T_i is calculated by the following formula:

$$T_i = 0.07H^{0.75} \tag{1}$$

where H is the height of the building. The scale factors of the records for the frames with 2, 3, 4, 5, 6, 8, 10, and 12 stories were calculated equal to 0.620, 0.580, 0.571, 0.532, 0.552, 0.692, 0.749, and 0.749, respectively.

4 Evaluating the ratio of inelastic to elastic displacements

The design of all parts of the structure should provide an integral unit behavior capable of resisting seismic lateral forces unless a structural separation distance sufficient to prevent damaging contact under total deflection is considered for the design earthquake (ASCE 7, 2006). Furthermore, for a quick evaluation of seismic performance of existing buildings, or essentially for initial design of new buildings, it is required to estimate the maximum displacement, which can deflect the building laterally during different ground motions. Accordingly, in the force-based seismic design method, the maximum inelastic (actual) displacement of the structure at each story should not exceed the allowable limits of the design code. This control is applied through the deflection amplification factor. Estimating inelastic displacement of a structure at different height levels can then be used for evaluating the required distance of separation joint.

In most seismic design codes, the deflection amplification factor (C_d) is presented as a ratio of the response modification factor (R). Figure 2 shows global inelastic behavior of a building (the ratio of base shear to the structural weight, C, versus roof or story drift relation), which can be developed by nonlinear static analysis. In this figure, real inelastic behavior is idealized by a bilinear elasto-plastic relation. Relying on this figure and according to the definition, which has been presented by FEMA P695 (2009) and by different researchers such as Uang and Maarouf (1994), the C_d/R ratio is calculated as follows:

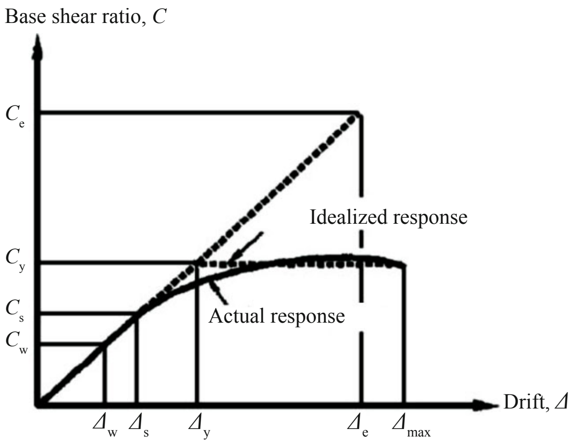

Fig. 2 General structural response

$$\frac{C_d}{R} = \frac{\Delta_{max}}{\Delta_e} \quad , \quad \left(\frac{C_d}{R}\right)_i = \frac{(\Delta_{max})_i}{(\Delta_e)_i} \tag{2}$$

In this equation, the term $(C_d/R)_i$ represents the ratio of deflection amplification factor to response modification factor when calculated based on the ratio of maximum inelastic to elastic displacement of the ith story of the structure. $(\Delta_e)_i$ and $(\Delta_{max})_i$ are the maximum elastic displacement and the maximum inelastic displacement of the ith story, respectively, which were calculated by means of linear and nonlinear time-history analyses. The average of the obtained responses for the results of selected seven earthquake records at roof level is presented in Table 4 (Δ_e, Δ_{max}) for all the studied RC frames. The ratios of averaged inelastic to elastic roof drifts are also listed in the aforementioned table. Note that buildings and other structures should be designed based on the provisions of the allowable stress or strength method. According to Eq. (2) and also the relations (Uang and Maarouf, 1994) that have led to this equation, it can be demonstrated that the ratio of deflection amplification factor to response modification factor is a comparable value in different seismic codes regardless of which seismic design method is used.

The average of the obtained ratios for the deflection amplification factor to the response modification factor in all the frames is equal to 1.0 (with the standard deviation of 0.07) when they were calculated based on roof drifts. From the comparison of the obtained values for C_d/R

Table 4 Average of maximum inelastic and elastic roof displacements with their ratios

RC Frame		$(\Delta_{max})_{roof}$ (mm)	$(\Delta_e)_{roof}$ (mm)	$(\Delta_{max}/\Delta_e)_{roof}$	RC Frame		$(\Delta_{max})_{roof}$ (mm)	$(\Delta_e)_{roof}$ (mm)	$(\Delta_{max}/\Delta_e)_{roof}$
Number of bays	Number of stories				Number of bays	Number of stories			
2	2	93.12	87.88	1.060	3	6	167.56	175.65	0.954
2	3	115.71	95.91	1.206	3	8	242.01	234.14	1.034
2	4	137.13	128.05	1.071	3	10	287.40	265.58	1.082
2	5	140.26	126.17	1.112	3	12	361.51	313.96	1.151
2	6	162.40	175.32	0.926	4	2	98.65	88.80	1.111
2	8	245.08	235.77	1.040	4	3	120.63	102.50	1.177
2	10	294.07	269.89	1.090	4	4	137.96	126.89	1.087
2	12	371.07	324.37	1.144	4	5	144.90	141.93	1.021
3	2	97.24	88.73	1.096	4	6	160.35	171.68	0.934
3	3	117.42	98.88	1.188	4	8	240.66	233.57	1.030
3	4	136.71	127.54	1.072	4	10	288.89	263.76	1.095
3	5	144.26	140.31	1.028	4	12	354.62	306.47	1.157

based on roof drift for intermediate reinforced concrete frames (IRCF) in this study with the one specified in the ASCE 7 provisions (2006), 0.9, it is concluded that the presented value for the deflection amplification factor in this study would result in a slightly more conservative roof drifts.

The Δ_{max}/Δ_e ratio at each story of the frame can be different from that of the roof level. In all the studied frames, the Δ_{max}/Δ_e ratio was averaged over the stories equal to 1.0 similar to those that were determined based on roof drifts but with the standard deviation of 0.2. Therefore, a deflection amplification factor, which is equal to response modification factor, can be used for estimating both maximum inelastic roof and stories displacements for intermediate reinforced concrete moment frames considering that a constant value is presented for C_d in the current seismic design provisions.

It is clear that the normalized displacement profile of a building is different from the interstory drift ratio profile along the structural height. Here, normalized elastic and inelastic displacement profiles (total displacement of each story divided by the structural height at the intended story level from the base) of some studied RC frames, which were calculated from time-history analyses, are compared to Standard No. 2800 (2005) and ASCE 7 (2006) provisions in Fig. 3. The values of 0.7 and 0.9 have been dedicated to the ratio of deflection amplification factor to response modification

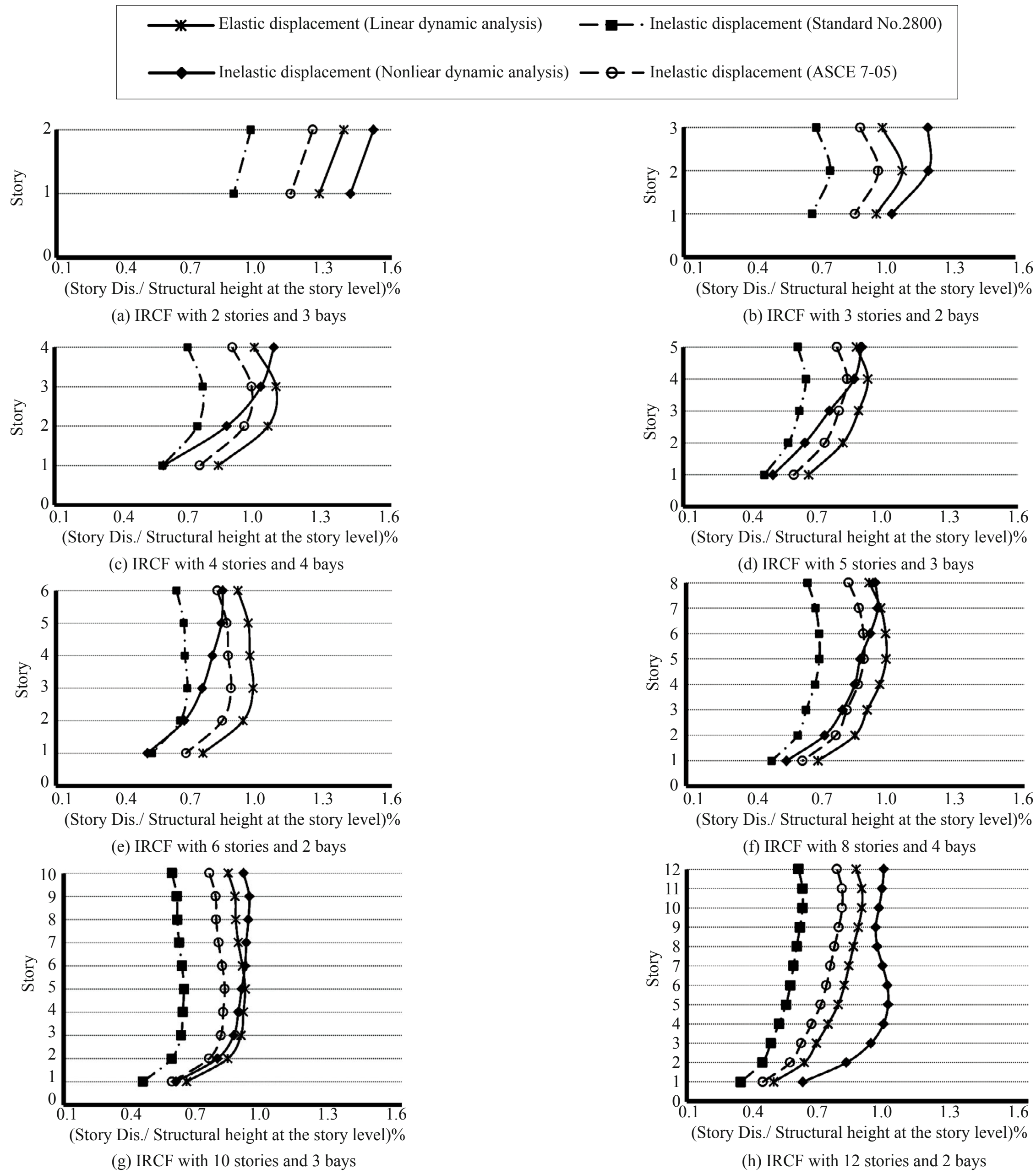

Fig. 3 Comparing normalized inelastic displacement profile of some IRCFs with that calculated based on deflection amplification factors specified in seismic design codes, ((a)-(h))

factor in these seismic design codes, respectively, for the intermediate reinforced concrete moment frames. The ratio of C_d/R equal to one will result in coinciding inelastic displacement with the elastic one. It was observed that generally, the difference between the maximum inelastic displacement and the corresponding maximum elastic displacement profiles is decreased by increasing the number of frame stories, especially at lower story levels. Furthermore, it can be concluded that inelastic displacement profiles are closer to the ASCE 7 (2006) results when compared to the Standard No. 2800 (2005). In all the studied cases, the inelastic displacement profile is approximated unconservative using a C_d equal to 0.7, although the results show less differences at lower stories than the upper ones regardless of which factor is applied.

Since the inelastic seismic roof drift of a building might only reflect the global behavior of the structure, and there are different factors that have a greater effect on story deflections during an earthquake. It is of interest to determine how the maximum inelastic displacement to the corresponding maximum elastic displacement ratio changes at different stories along the structural height. Figure 4 illustrates this trend for intermediate RC frames with 3, 5, 8, and 10 stories as examples. It was observed that generally the ratio of maximum inelastic to maximum elastic displacement increases along the height of the building frames. More structural damage is sustained by the stories that coincide with upper levels of changes in sectional dimensions. This may explain the results shown in in Fig. 4, although the maximum interstory drift profile is more clearly affected by this factor. From this comparison of investigated frames, it was also found that the above-obtained graphs represent a smoother ascending trend when the number of frame stories increase, whereas changing the numbers of frame bays did not have much of an effect on the values of the intended ratios. In other words, as the number of frame stories increased, the differences between the values of Δ_{max}/Δ_e ratios in adjacent stories decreased and generally, the coefficients of variation (CoV) of these ratios were larger in mid-rise frames compared to high-rise ones for frames with more than 3 stories. This coefficient is equal to the ratio of standard deviation of Δ_{max}/Δ_e ratios to their average over the frame stories. It is an indicator of data scattering. Figure 5 represents the percentage of CoV in all the studied RC models.

5 Evaluating the ratio of inelastic to elastic interstory drifts

Estimation of the maximum interstory drift ratio (ratio of the maximum interstory drift to the interstory height (h)) and of the maximum roof displacement is necessary for recognizing the capacities required to attain adequate seismic performance of the building, particularly the lateral stiffness demand (Miranda, 1999). Shear-type buildings have larger concentrations of interstory drifts than flexural-type buildings (Miranda, 1999). Since structural damage in multistory frames tends to concentrate in a limited number of stories, it is expected that the C_d/R ratio will be greater than one when it is calculated based on interstory drift ratios.

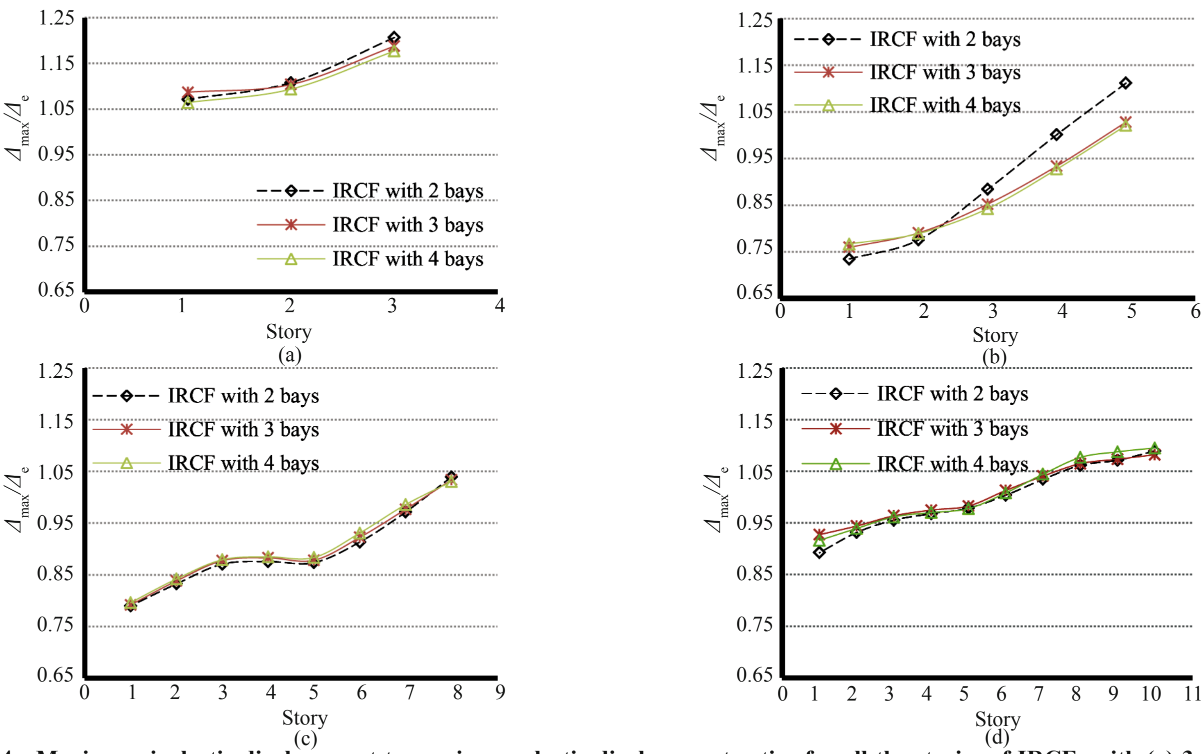

Fig. 4 Maximum inelastic displacement to maximum elastic displacement ratios for all the stories of IRCFs with (a) 3 stories, (b) 5 stories, (c) 8 stories, (d) 10 stories

Therefore, it seems worthwhile to assess the C_d/R ratio when calculated from the ratio of inelastic to elastic interstory drift ratios. The following equation is used for calculating the C_d/R ratio based on the interstory drift ratio of the ith story:

$$\frac{C_d}{R} = \frac{\delta_{max}}{\delta_e} \quad , \quad \left(\frac{C_d}{R}\right)_i = \frac{(\delta_{max})_i}{(\delta_e)_i} \tag{3}$$

where $(\delta_{max})_i$ and $(\delta_e)_i$ are the maximum inelastic interstory drift and the maximum elastic interstory drift of the ith story of the RC frame, respectively, which were calculated from averaging the results of nonlinear and linear time-history analyses performed on the model building frames when subjected to seven scaled earthquake records. The coefficients of variation of δ_{max}/δ_e ratios in all the studied frames defined as the standard deviation divided by the mean are presented in Fig. 6.

Since the location of maximum elastic interstory drifts are different from those of the inelastic time-history analyses for most of the RC frames, and damage concentration is observed in a limited number of stories, different δ_{max}/δ_e ratio over the structural height are expected as illustrated in Fig. 7. It can be observed that the ratio of maximum inelastic interstory drift to maximum elastic interstory drift is increased by increasing the story number. From comparing the trend of the δ_{max}/δ_e ratio of the studied RC frames, it can be concluded that by increasing the frame height, the slope of the graphs is decreased along the structural height towards the top stories. In addition, it is observed that the ratio of δ_{max}/δ_e is not significantly affected by the number of bays as the Δ_{max}/Δ_e ratio.

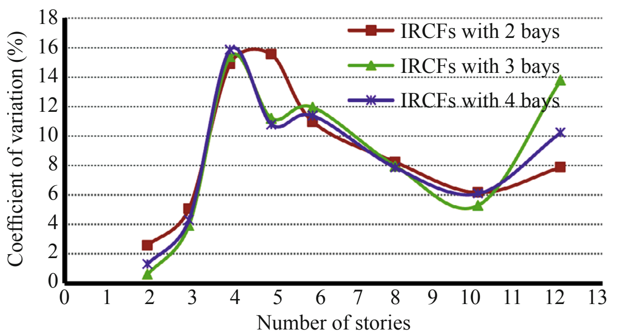

Fig. 5 Variation coefficient of inelastic to elastic displacement ratios for all of the stories in each frame

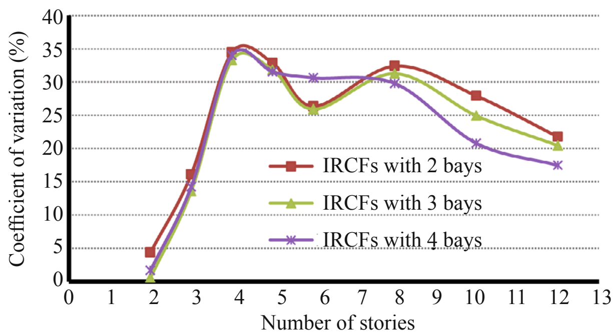

Fig. 6 Variation coefficient of inelastic to elastic interstory drift ratios for all of the stories in each frame

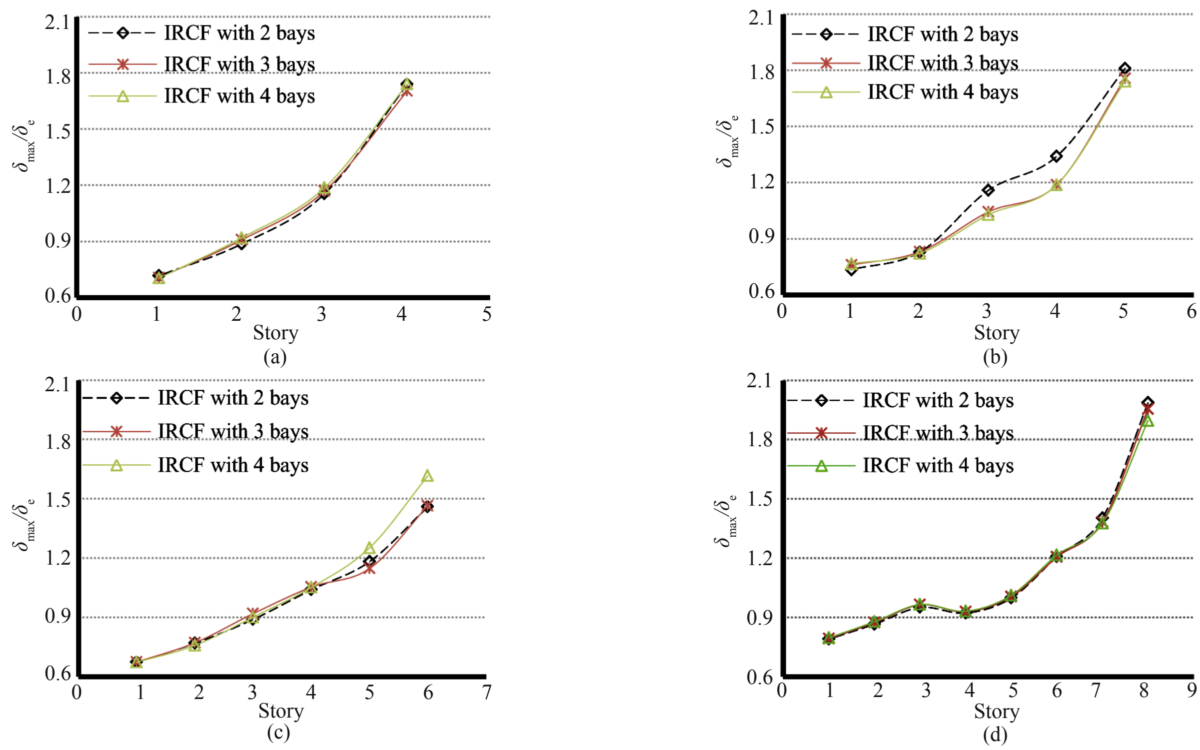

Fig. 7 Maximum inelastic interstory drift to maximum elastic interstory drift ratios for all of the stories of IRCFs with (a) 4 stories, (b) 5 stories, (c) 6 stories, (d) 8 stories

The average value of the δ_{max}/δ_e ratios for all the reinforced concrete frames was calculated to be equal to 1.2 with the standard deviation of 0.3, which leads to a general point of view. It was observed that stories sustaining more damage would experience larger interstory drifts when compared to others of the same structure. The relation between story damage index (as an indicator of local damage scale) and interstory drift is thoroughly investigated in the next section. More cracks and plastic hinges appeared in the elements of the stories in which sectional dimension changes have occurred. It is remarkable that in some cases, stiffness distribution along the height of the structures changed more than once as can be seen in the frames with 8, 10, and 12 stories. Stories that coincided with the location of upper levels of cross-sectional dimension changes for columns were more significantly damaged, which was due to the fact that they possess lesser overstrength when compared to the adjacent stories below. Consequently, the maximum inelastic interstory drift ratios, generally, were observed at these stories in the analyzed RC frames. Browning and his colleagues came to an equivalent point that, on average, the location of maximum story drift of the reinforced concrete building frames obtained from nonlinear analyses is 1.6 times higher than the location determined by linear analysis (Browning *et al.*, 2008). The reason for this observation can be defined under two conditions. In some cases, the longitudinal reinforcement ratio (area of longitudinal reinforcing steel divided by the cross-sectional area of the concrete) of the columns belong to the story that coincides with the uppermost level of changing sectional dimension is larger than that of the lower story columns (typically the minimum value of 1%). Therefore, the overstrength of this story of RC frame is less than that of those situated directly below it. Even in the case of equal longitudinal reinforcing steel ratios for the two adjacent stories, the upper one in which the cross-sectional dimension has changed, has less overstrength than the one below. The latter case can be explained through the following example.

In the case of the RC frame with 10 stories and 4 bays, the required longitudinal reinforcement ratio of the central columns of the eighth and ninth stories to resist the design forces according to the applied load combinations were calculated to be equal to 0.4% and 0.8% respectively. Considering the ACI (2002) provisions, the minimum required longitudinal reinforcing steel ratio of 1% was considered for these columns. From comparing the required longitudinal reinforcement ratio for satisfying design force demands to the allocated value of 1%, it can be concluded that the ninth story has lesser reserve strength than the eighth story. This trend was observed in most of the studied RC frames with more than six stories. Figure 8 illustrates the dimensions associated with the 10-story frame with 4 bays.

6 Local story damage index assessment by considering interstory drift ratio

Different features of structural response can be taken into account using damage indices, which result in developing a quantitative measure of structural damage. The modified Park and Ang damage index ($\mathrm{DI}_{\text{M-P\&A}}$) (Park *et al.*, 1984; Valles *et al.*, 1996) has been incorporated in the IDARC program (Reinhorn *et al.*, 2006). It is used to estimate the accumulated damage sustained by the components of the structure, by each story level ($\mathrm{DI}_{\text{story}}$), and the entire building ($\mathrm{DI}_{\text{global}}$). The story and overall damage indices are computed using weighting factors based on the dissipated energy at component and story levels, respectively. Modified Park and Ang damage indices (Park *et al.*, 1984; Valles *et al.*, 1996) are calculated as follow

$$\mathrm{DI}_{\text{M-P\&A}} = \frac{\phi_{\mathrm{m}} - \phi_{\mathrm{r}}}{\phi_{\mathrm{u}} - \phi_{\mathrm{r}}} + \frac{\beta E_{\mathrm{h}}}{M_{\mathrm{y}} \phi_{\mathrm{u}}} \tag{4}$$

$$\mathrm{DI}_{\text{story}} = \sum (\lambda_i)_{\text{component}} (\mathrm{DI}_i)_{\text{component}}; \quad (\lambda_i)_{\text{component}} = \left(\frac{E_i}{\sum E_i} \right)_{\text{component}} \tag{5}$$

$$\mathrm{DI}_{\text{global}} = \sum (\lambda_i)_{\text{story}} (\mathrm{DI}_i)_{\text{story}}; \quad (\lambda_i)_{\text{story}} = \left(\frac{E_i}{\sum E_i} \right)_{\text{story}} \tag{6}$$

where φ_{m} is the maximum rotation obtained during the loading history; φ_{u} is the ultimate rotation capacity of the section; φ_{r} is the recoverable rotation during an unloading;

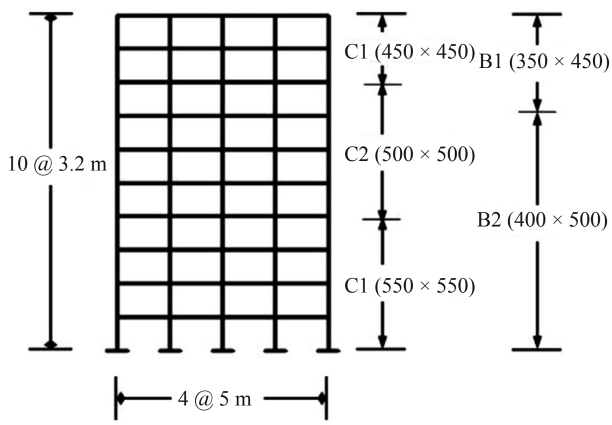

Fig. 8 Beams (B) and columns (C) dimensions of 10-story RC frame with 4 bays

β is the constant parameter of the model; M_y is the yield moment and E_h is the section dissipated energy. In Eqs. (5) and (6), λ_i is the energy weighting factor; and E_i is the total absorbed energy by the component or the story "i". The program default value of 0.1 for β is considered.

Different levels of damage have emerged in each of the structures' stories due to their main structural elements plastic deformation (ductility) and energy absorption capacities. Interstory drift as a macroscopic quantity is an appropriate indicator of both local and global damage sustained by the regular structures (Kappos and Manafpour, 2001). Therefore, a key point in assessing the damage scales of the stories can be their drift ratios. In other words, more significant damage is expected for the stories with larger lateral interstory drifts. Consequently, it was investigated how the (local) stories' damage indices vary with inelastic interstory drift ratios. Investigating unsteady rate of story damage index development within the practical range of interstory drifts showed that a direct relation between these two variables could not be justified. Therefore, the other possible relations were examined to find the best estimate for the story damage index, which allows an actual rate of development of the DI_{story} vs. interstory drift ratio to be considered. As shown in Fig. 9, it can be observed that the story damage indices of the studied RC frames decrease by increasing the inverted value of the inelastic interstory drift ratios through a (negative) power form. This figure illustrates the best-fitted curve, which was calculated by regression analysis performed on these parameters. These parameters have been computed by means of nonlinear time-history analyses of seven ground motion records. The following relation was calculated for estimating local stories' damage indices considering their inelastic interstory drift ratios for the range of story drift ratios obtained here.

$$DI_{story} = 0.076\left(\frac{1}{\left(\frac{\delta_{max}}{h}(\%)\right)}\right)^{-0.78} \tag{7}$$

Offering a rather simple relation for engineers to estimate the story damage index of RC frames based on the expected (or the specified) interstory drift was the main purpose for derivation of Eq. (7), while it was not taken into consideration significantly in the past. Although this relation is not related to the global response of the structure and consequently the redundancy will be greatly decreased, more study is recommended before using it for RC frames, which are different from those investigated in this study.

It is observed that the rate of the story damage index increases as the interstory drift ratios increase up to approximately 1% to 1.5%, whereas in lower drift ratios, a milder trend is visible. Seismic design codes and guidelines, such as ASCE 7 (2006), control structural

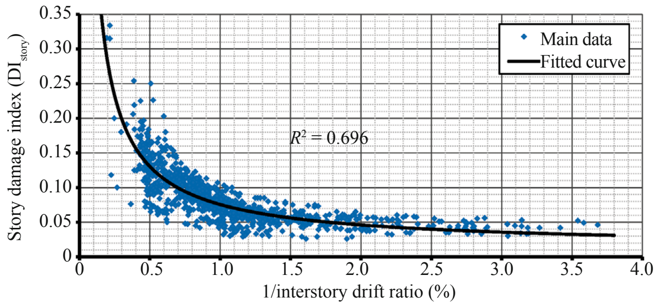

Fig. 9 The correlation between story damage index and interstory drift ratio

deformations and consequently eventual damage to the building components by limiting the calculated interstory drifts. Allowable story drift of a reinforced concrete frame might vary up to 2.5% of the story height in accordance with the occupancy category of the building. Considering the inelastic responses of all the studied RC frames, it can be concluded that 76.86%, 91.14% and 98.67% of interstory drift ratios do not exceed 1.5%, 2%, and 2.5%, respectively. In addition, the most condensation of inelastic story drift ratio of the intermediate reinforced concrete frames has been obtained in the range of 0.7% to 2%. Therefore, generally, the specified limits for story drifts of intermediate RC frames in seismic design codes are acceptable; however, more restriction is required if a structure may be subjected to impulse-type earthquakes.

7 Conclusions

Estimating global and local seismic drifts of reinforced concrete frames through the deflection amplification factor is evaluated in this study. The ratio of the inelastic displacement to elastic displacement was calculated for all the stories of studied RC frames and its inclination was investigated along the structural height. Furthermore, the maximum interstory drift magnitude and its location were assessed according to the results of linear analyses. Seven scaled earthquake records were used for linear and nonlinear dynamic time-history analyses. Based on the analysis of the proportioned frames and selected earthquake records, the following conclusions are summarized as follows:

- The proposed value for the C_d/R ratio, which is calculated based on the ratio of maximum inelastic to elastic roof drift for intermediate RC frames, is equal to 1.0 when computed according to the stories' maximum inelastic to elastic displacement ratio.
- It was observed that generally, the ratio of maximum inelastic to elastic displacements increases along the height of RC frames and shows a milder ascending trend when the number of frame stories increases, whereas changing the number of bays does not significantly affect this ratio.
- By increasing the number of frame stories, the difference between the values of Δ_{max}/Δ_e ratios in

adjacent stories are decreased and generally, the variation coefficient of these ratios is computed larger for the low to mid-rise structures than for the high-rise frames when the number of stories is more than three.

- The location of the maximum inelastic story drifts in the studied RC frames is almost observed in the stories that coincide with the last change of the sectional dimensions along the height of structure. It can be concluded that usually, these stories should be designated as having larger drift ratios. These stories have less overstrength when compared to the adjacent stories below, which can be due to their reinforcement details and dimensions.
- For all of the studied RC frames, on average, the magnitude of the maximum interstory drift calculated using nonlinear time-history analysis is a factor of 1.2 larger than that estimated using linear time-history analysis. The standard deviation is 0.3 for these obtained ratios.
- The presented relation for estimating local damage indicates that as the rate of the story damage index increases, the story drift ratios also increase up to approximately 1% to 1.5%, whereas in lower drift ratios, a milder trend is observed. The most condensation of the inelastic story drift of the reinforced concrete frames with moderate ductility has been found in the range of 0.7% to 2% of the story height.

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