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Investigation of Efficiency of Different Ground Motion Scaling Methods for Estimating Seismic Demands of Steel Moment Frames

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ABSTRACT

There are different methods in seismic design codes for scaling earthquake ground motion records for the purpose of performing nonlinear time history analysis. The main objective of this study is to evaluate how different scaling approaches affect the results of nonlinear time history analysis. Reduced the scatter in estimated drift angles was the main criteria to measure the effectiveness of different scaling methods. For this purpose, three steel moment resisting frames with different height (3, 6, and 10 stories) were subjected to Incremental Dynamic Analysis under a suite of twenty near-fault and far-fault ground motions. The results indicate that the scaling method based on UBC 97 causes a more significant dispersion in the responses compared to the other considered methods.

Keywords - ground motion scaling method, incremental dynamic analysis, steel moment frame, near-fault, far-fault ground motion

1. INTRODUCTION

Earthquake ground motions have caused severe casualties and damage to structures for over the past decades. Many attempts have been made to design the structure under seismic loads with appropriate safety margins. Time history and pushover analyses are the two conventional numerical methods for simulating the seismic loads on the structures. These methods can be employed for the design of new structures and seismic evaluation of existing structures. Pushover analysis is a nonlinear static analysis, which lateral loads are monotonically increased from zero to failure level to evaluate possible plastic hinge formation and damages in the structure. In many previous studies, pushover analysis has been used to numerically estimate seismic parameters of structures and compare them with suggested value in the seismic codes or evaluate the seismic performance of existing building under earthquake ground motion records [1-12]. Although pushover analysis is more practical for engineering offices and less time consuming than time history analysis, it does not explicitly address the real performance of structures under seismic loads. In pushover analyses, lateral load patterns cannot simulate the dynamic characteristic of loading correctly when an element undergoes non-linear behavior. Therefore, in some cases performing a time history analysis is unavoidable to find the real nonlinear behavior of structures under strong earthquake excitations.

In codes and standards related to seismic design and evaluation of seismic performance of structures, some specifications are mentioned for conducting a time history analysis. Some of these specifications are related to criteria for selecting appropriate earthquake ground motion records and scaling methods. Considering the importance of ground motion scaling, there are many researches in this field aim to increase the efficiency of the scaling method. In this paper, besides studying the different scaling methods, their accuracy has been evaluated to estimate the seismic demands of steel moment frames.

Time history analyses are utilized in the design of new structures and the evaluation of seismic performance of existing structures [13-21]. In ATC-58-1 [22], three different methods are specified to evaluate seismic performance: intensity-based method, scenario-based method, and time-based method. Choosing the right method for selecting and scaling earthquake ground motion records depends on the type of assessment or design target. The most common method for evaluating structural performance is the intensity-based method. In this method, the response of the structure and its components is evaluated in term of the intensity of earthquakes. The intensity of earthquake depends on factors such as ground motion magnitude, distance from the seismic source, and site condition. To determining criteria of effectiveness in the intensity of earthquakes, parameters such as peak ground acceleration (PGA), peak ground velocity (PGV), and peak ground displacement (PGD) are used. Another parameter to measure the intensity of an earthquake is the acceleration spectrum value for the first vibration mode of the structure for a specific ground motion. In this paper, the intensity-based method was employed as a scaling evaluation method.

Scaling of earthquake ground motion records is a significant step for seismic design and evaluation of structures when nonlinear time history analysis is employed. In general, record scaling can be conducted in both frequency and time domain. Time-domain scaling is conducted by changing in the acceleration amplitudes of an earthquake ground motion record without any change in its phase and frequency content. In this method, all the values of acceleration are multiplied by a specific coefficient. In frequency-domain scaling method, the frequency and phase content of the earthquake are adjusted in a way that the resulted spectrum approaches the target spectrum. In the seismic codes and standards, time-domain scaling is recommended [23].

In time-domain scaling method, the phase and frequency contents of the records are not changed through the scaling procedure. Moreover, using this method, two primary goals of scaling, i.e., appropriate and accurate estimation of seismic response quantities and high efficiency can be satisfied [24]. In the first method peak ground acceleration was used to scale ground motion accelerations in the time domain. This method is not very efficient and causes a large dispersion in values of structural response (under various earthquake ground motion records) [25-27].

In this regard, other intensity measures such as maximum effective ground motion velocity were also used, but the results were not convincing [28]. The reason is that the seismic characteristics of structures are not considered in these measures. To consider the vibrational properties of the structure, a scaling method was proposed based on the values of the elastic spectrum and the first vibration mode. This method promotes the accuracy of ground motion scaling, especially for the structures with the first dominant vibration mode [29]. On the other hand, this scaling method does not have enough accuracy for the structures which are expected to suffer high nonlinearity, i.e., moment frames with high ductility or with the significant higher vibration mode, i.e., irregular structures [30, 31]. To consider the effect of higher modes, a scalar intensity measure was proposed as a combination of acceleration spectrum values of the first and second structure modes (S_a (T_1) and S_a (T_2)) as well as vector quantities containing acceleration spectrum values of the first mode and the ratio $S_a(T_1)/S_a(T_2)$ [32, 33]. Although this method improves the accuracy of scaling, it still does not have a sufficient accuracy for near fault earthquakes [34]. To consider the structure period elongation due to the structure nonlinearity, a combined intensity measure including $S_a(T_1)$ and $S_a(cT_1)$ ($c>1$) was recommended [35, 36]. In the other proposed method, the difference between the elastic earthquake and the target spectrum (or design spectrum) is minimized [37, 38].

In all the mentioned methods, only the vibration period of structure determines the scale factor, and other criteria such as ultimate strength and nonlinear behavior of the structure were not considered. Although these methods do not have sufficient accuracy for near-fault earthquakes and the structures in the nonlinear region, they are still widely utilized as the standard methods in the structural design codes. Recently, many efforts have been made to solve the problems related to these scaling methods. To overcome the problems with the traditional scaling methods, normalization before scaling is proposed in FEMA 695 [39]. Moreover, some methods based on the nonlinear displacement response spectrum have been proposed [40]. These methods increase the accuracy of scaling method and reduce the scattering in time history analysis results and might be included in the design codes very soon. The other method was proposed by Chopra, which is based on the results of modal pushover analyses. Despite having a decent accuracy, it requires a high skill for a structural engineer to perform nonlinear analyses and it is time-consuming. [24].

The purpose of scaling earthquake records is that a scaled record can apply a specific intensity from earthquake to the structure. Therefore, one of the most important factors in determining a scaling method is to define a quantity for the earthquake intensity. The design codes scale records based on a comparison between earthquake spectra and the target design spectrum by considering acceleration spectrum values as the earthquake intensity measures. In these methods, earthquake records are scaled in a way that the values of a scaled record spectrum (or their average spectrum values) are not less than the target spectrum within a specific period range. This interval is defined between $0.2T_1$ and $1.5T_1$ in most design codes such as UBC 97 [41] and ASCE 7-10 [42], where T_1 is the first mode of vibration. According to the recommendations of American Institute for Standardization and Technology

(NIST) [23], for moment frame structures, the interval ($0.2T_1$,$3T_1$), and for structures with shear wall system or braced frames, the interval ($0.2T_1$,$2T_1$) are recommended, respectively.

1.1 Scaling Method Based on Uniform Building Code (UBC 97)

According to UBC 97, the following method is used to scale accelerations:

1. All accelerations are scaled to their maximum amplitude, i.e., the maximum accelerations are equal to the gravity acceleration;

2. The acceleration response spectra of each scaled acceleration pair is determined by considering a 5% damping ratio;

3. Response spectra of each acceleration pair are combined with each other using square root of the sum of the squares (SRSS) and a single compound spectrum for each pair is constructed;

4. The combined response spectra for the pair of all the accelerations (derived from step 3) are averaged and compared with the design response spectrum. The scale factors are determined in a way that the averaged spectrum within the period range of 0.2T to 1.5T is not less than 1.4 times of the design response spectrum values. At the end, the accelerations are multiplied by the caculated scale factors. In this method, the same scale factor is obtained for all accelerations, and this scale factor should be applied to the normalized acceleration (normalized to the gravity acceleration). In time history analyses, in addition to introducing the scaled accelerations, according to the seismicity of the area, the peak accelerations are also considered [41].

1.2. Scaling Method Based on ASCE 07-10 Regulations

In ASCE 07-10 [42], scaling earthquake records is slightly different from UBC 97. According to ASCE, in the 3D analysis of structures, non-normalized records are used. The acceleration spectra values S_a are compared with the design spectra. Also, there is no need to apply the same scale factor to the records. In this method, the acceleration spectra of the horizontal components of the record pairs are combined using square root of sum of squares without any normalization. At the end, the calculated scale factors are multiplied to the pair accelerations. The scale factors are determined in a way that the average spectrum obtained from the acceleration pairs in the time interval of 0.2T to 1.5T should not be less than the values for the design spectrum.

1.3. Scaling Method Based on FEMA P695

In order to perform an incremental dynamic analysis to estimate collapse in structures, a scaling method for earthquake records was recommended in FEMA 695 [39]. This method consists of two steps. First, normalizing the earthquake records to their peak ground velocity (PGV_{PEER}), which is done before scaling the records. Peak ground velocity (PGV_{PEER}) is defined as an average geometry of the peak ground velocities of two horizontal earthquake components considering different applying angles. Normalization of accelerations is performed to eliminate the effect of factors such as magnitude, distance from fault, and site conditions. Therefore, a correction factor (normalization coefficient) is obtained for each record and the orthogonal earthquake components are multiplied by it. The correction factor is obtained from Eq. (1).

$$NM_i = \text{Median}\,(PGV_{PEER,i})/PGV_{PEER,i} \qquad (1)$$

In this equation, NM_i is normalization coefficient related to the record i, which is multiplied by acceleration values of both components. The term Median ($PGV_{PEER,i}$) refers to the median values of PGV_{PEER} for all records and $PGV_{PEER,i}$ is the value of peak ground velocity (PEER) for i^{th} record.

In FEMA 695, it has been stipulated that the record for near-fault earthquakes must be rotated in parallel and perpendicular to the fault before using in analyses.

In next step, the records are scaled to a specific intensity criterion, in a way that the average value of the acceleration spectra in the fundamental period of the structure becomes greater than the target spectrum.

2. EARTHQUAKE AND SCALING METHOD SELECTION CRITERIA

Two important indicators in determining the proper performance of the record selecting and scaling methods are accuracy and efficiency. The purpose of accuracy criterion is reaching the expected target response, and the purpose of efficiency is to minimize the amount of scattering in structural responses in terms of ground motion records [43]. Parameters such as standard deviation (SD), mean square error (MSE), and mean relative error (MRE) are used to determine the dispersion of results. The standard deviation (SD) is calculated according to Eq. (2). Also, MSE and MRE are obtained according to Eq. (3) and Eq. (4), respectively [43].

$$\delta = \sqrt{\frac{1}{m}\sum_{i=1}^{m}\left(\frac{EDP(i)-\overline{EDP}}{\overline{EDP}}\right)^2} \qquad (2)$$

$$MSE = \frac{1}{m}\sum_{i=1}^{m}(EDP(i)-\overline{EDP})^2 \qquad (3)$$

$$MRE = \frac{1}{m}\sum_{i=1}^{m}\left|\frac{EDP(i)-\overline{EDP}}{\overline{EDP}}\right| \qquad (4)$$

In these equations, m is the number of acceleration record pairs. $EDP(i)$ and $\overline{EDP}$ are the seismic demand quantities obtained from the i^{th} record and the average seismic response quantities respectively.

3. RESEARCH OBJECTIVES

In this paper, the efficiency of various scaling methods for evaluating the seismic responses of steel moment frames has been evaluated. For this purpose, two-dimensional models and non-linear time history analysis have been used. The studied models included three special steel moment frames of 3, 6, and 10-story. The structural configurations are shown in Fig. 1 and the designed sections of the structural elements are presented in Table 1.

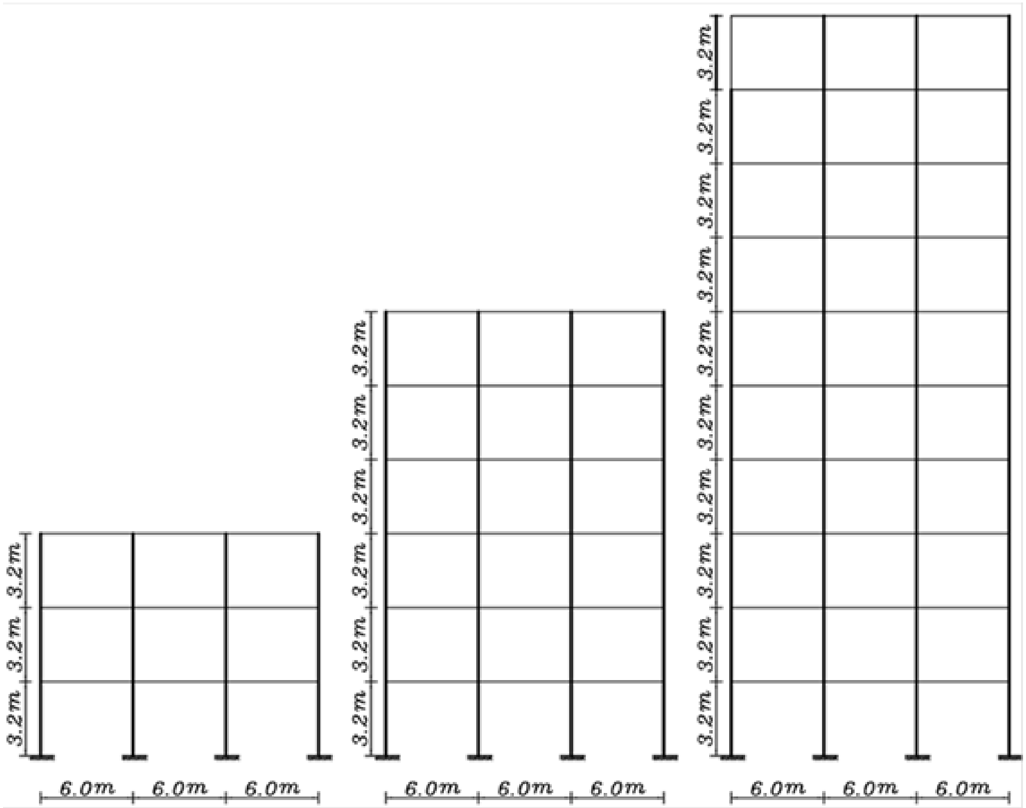

Fig. 1 Configuration of structures

Incremental dynamic analysis has been used for evaluating the seismic responses of the structures [14]. In this method, the seismic responses of the structures are estimated by using different earthquake intensities. Two sets of earthquake records of near-fault and far-fault were utilized for the analyses. The record suits are presented in Table 2. First, the records are scaled using the following methods:

1. Scaling method based on the value of acceleration spectrum for the first vibration mode;
2. Scaling method based on UBC 97;
3. Scaling method based on FEMA 695.

In the following, the efficiency of different scaling methods has been evaluated. The results are presented separately for far and near-fault records.

Table 1. Steel moment frames sections

Frame structure	Fundamental period (s)	Story number	Column section (mm)	Beam section (mm)
3-story	0.86	1-3	Box 280*280*10	H300*200*10*14
6-story	1.22	1-3	Box 280*280*15	H300*200*10*14
		3-6	Box 280*280*10	H260*180*10*14
10-story	1.69	1-3	Box 380*380*20	H380*200*12*20
		4, 5	Box 340*340*20	H380*200*12*20
		6	Box 340*340*20	H340*200*12*20
		7, 8	Box 320*320*16	H340*200*12*20
		9, 10	Box 260*260*16	H300*160*10*20
		Story number	Column section (mm)	Beam section (mm)
		1-3	Box 280*280*10	H300*200*10*14
		1-3	Box 280*280*15	H300*200*10*14
		3-6	Box 280*280*10	H260*180*10*14
		1-3	Box 380*380*20	H380*200*12*20
		4, 5	Box 340*340*20	H380*200*12*20
		6	Box 340*340*20	H340*200*12*20
		7, 8	Box 320*320*16	H340*200*12*20
		9, 10	Box 260*260*16	H300*160*10*20

3.1. Earthquake Records

In this paper, twenty earthquake records have been utilized to evaluate the different scaling methods using incremental dynamic analysis. Ten records have velocity pulses and the others are ordinary ground motions. All earthquake ground motion records used in dynamic analysis and scaling are based on soil type C in the USGS classification. The earthquake magnitudes are taken in the range of 6.1 to 7.6. The record specifications are presented in Table 2.

The criterion for discriminating the near-fault records with a pulse from ordinary earthquake records are chosen based on Baker's research [44].

3.2. Designing Structures and Finite Element Model

The office buildings of 3, 6, and 10-story, with steel special moment frame systems, are selected for investigation (Fig. 1). Loading and designing of structures were carried out in accordance with ASCE 7-10 and AISC 2010 [42, 45]. Load resistance factor design (LRFD) method has been used to design the steel structures. The response modification coefficient of 8 is adopted in accordance with ASCE 7-10 [42] recommendation to calculate design base shear. Soil condition is assumed to be site class C. Buildings are in a high seismic region (PGA = 0.35). The static method was used for analyzing structures against earthquakes. Live and dead loads are 500 kg / m2 and 250 kg / m2, respectively. The dead load of roof floor is similar to other floors, but the live load is 150 kg / m2. The height of all floors is 3.2 meters. ST37 steel type with mechanical characteristics such as yield strength, ultimate strength and elastic modulus of 2.1*106, 2400, and 3700 kg/cm2 have been used, respectively. Linear modeling and designing of structures were performed by using ETABS software version 9.5 and nonlinear modeling and analyzing was conducted by Seismo Struct software version 6 [46]. The software is able to consider geometrical nonlinearity (including P-Delta effects) in addition to material nonlinearity. The plastic hinges of beams and columns are modeled based on the fiber elements [47]. Steel hysteresis curve is considered as a bilinear curve with a hardening rate of 3%. Rayleigh damping of 2% and 5% are considered for the first and second vibration modes of the structures. The fundamental periods of the frames are 0.86, 1.22, and 1.69 seconds, respectively.

Table 2. Earthquake record specifications

Earthquake records	Magnitude	Distance from epicenter (km)	PGA (g)
1-Far-fault			
Chi-Chi, Taiwan, 1999	7.6	11.14	0.35
Imperial Valley, 1979	6.5	10.4	0.32
Loma Prieta, 1989	6.9	12.7	0.32
Loma Prieta, 1989	6.9	14.4	0.37
Northridge, 1994	6.7	15.8	0.42
Northridge, 1994	6.7	13	0.41
Tabas, 1978	7.4	26.1	0.09
Kobe, 1995	6.9	95.72	0.143
N. Palm Springs, 1986	6.06	64.8	0.121
Manjil, 1990	7.37	64.67	0.1
2-Near-fault			
Chi-Chi, 1999	7.6	0.24	0.42
Chi-Chi, 1999	7.6	1.09	0.57
Erzincan, 1992	6.9	2	0.50
Northridge, 1994	6.7	7.1	0.84
Landers, 1992	7.3	23.6	0.25
Loma, 1989	6.9	11.1	0.32
Imperial Valley, 1979	6.5	10.4	0.16
Kobe, 1995	6.9	0.3	0.69
Cape Mendocino, 1992	7	8.2	0.59

3.3. Scaling Evaluation Method

In this study, minimizing dispersion of structural responses for different records is contemplated as the efficiency criterion for scaling methods. For this purpose, the coefficient of variation (CV) of the seismic responses is employed. This criterion can provide the variance of structural responses for different records in the normalized form. This coefficient is obtained according to Eq. 5 by dividing the standard deviation (SD) on the mean value.

$$CV = \delta_i(i = 1{:}n)/Mean \qquad (5)$$

In this equation, δ is the standard deviation (SD), n is the number of variables, and the mean is the average values of variables.

The fewer values of CV show smaller dispersion in the seismic responses and more efficiency of the scaling methods.

4. ANALYSIS AND RESULT

Incremental dynamic analysis has been used to determine the values of seismic responses of the structures under different earthquake intensities. The analyses were carried out using far and near-fault earthquakes and the responses of structures for the various earthquake intensities and for each of the scaling methods are estimated. A relative inter-story displacement (drift angle) is one of the most critical damage criteria for structures. Therefore, this parameter was considered as a response quantity in this study. The maximum drift angle curves in terms of different PGA of the records are presented in Fig. 2. The acceleration spectrum values for each structure and scaling method were calculated, and by using IDA curves in Fig. 2, maximum inter-story drift angles corresponding to each acceleration spectrum were calculated. The values of standard deviation (SD) and dispersion coefficients for each scaling method and for each structure under different sets of earthquake excitation were obtained and presented in Table 3 to 5.

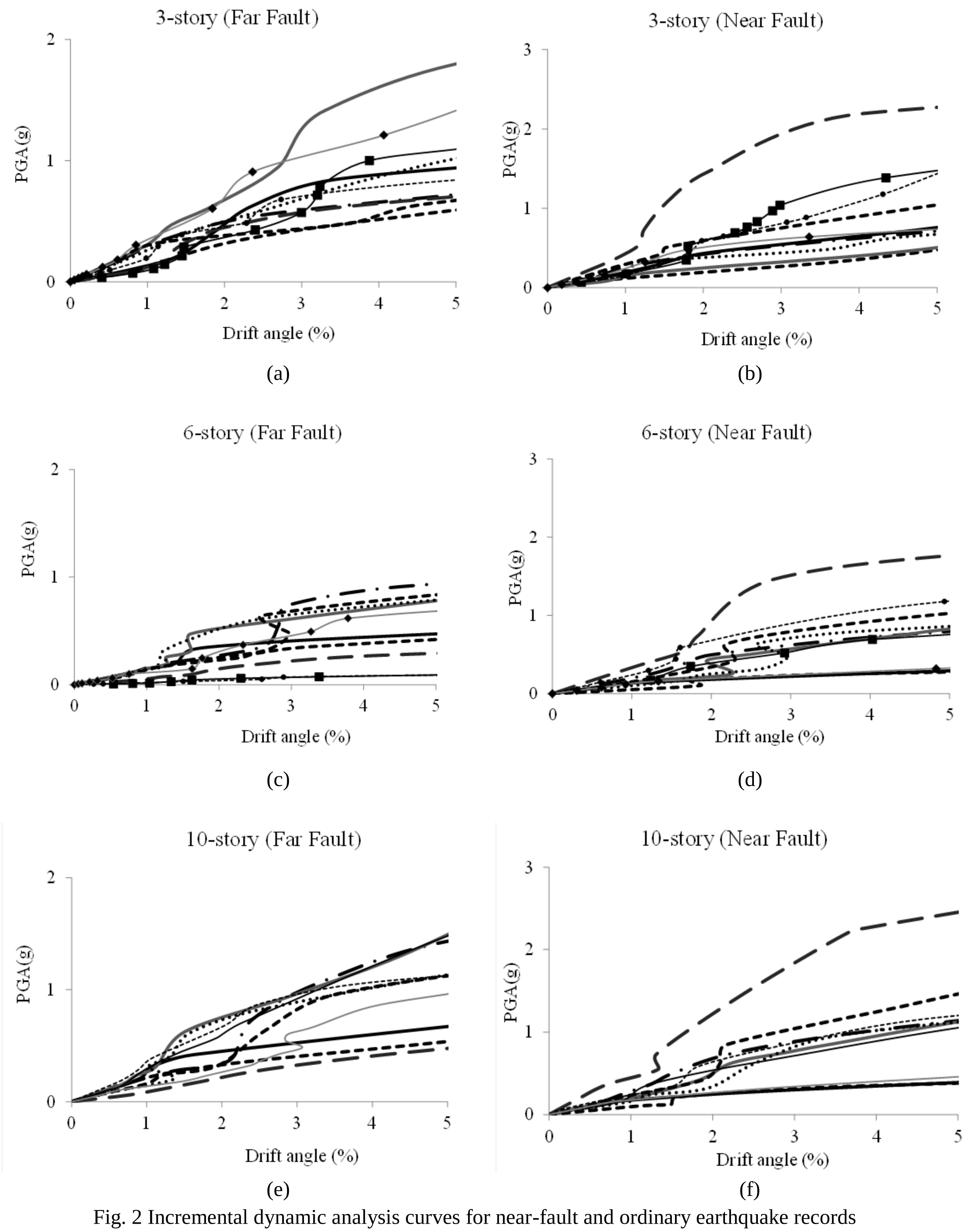

Fig. 2 Incremental dynamic analysis curves for near-fault and ordinary earthquake records for 3, 6, and 10-story steel moment frames

Table 3 is related to UBC 97 scaling method, Table 4 is related to the FEM P695 method, and Table 6 is related to the scaling method based on absolute values of acceleration spectrum. In these tables, the standard deviation (SD) and coefficient of variation (CV) are related to the seismic response values obtained from different record sets and are separately presented for far and near-fault records and for three considered structures. The dispersion criterion has been used as an efficiency indicator for comparing the scaling methods.

Table 3. Precision indicators of UBC 97 scaling method for far and near-fault earthquakes

Structure type	3-story	6-story	10-story
Far-fault records			
SD	1.814	6.853	4.392
CV	0.483	0.965	0.923
Near-fault records			
SD	1.6`31	3.781	6.614
CV	0.483	1.741	1.189

Table 4. Precision indicators of FEMA 695 scaling method for far and near-fault earthquakes

Structure type	3-story	6-story	10-story
Far-fault records			
SD	0.687	0.913	0.61
CV	0.251	0.271	0.277
Near-fault records			
SD	1.124	1.009	0.962
CV	0.421	0.351	0.379

Table 5. Precision indicators of scaling method based on absolute value of acceleration spectrum for first vibrational mode for far and near-fault earthquakes

Structure type	3-story	6-story	10-story
Far-fault records			
SD	1.169	1.84	1.334
CV	0.380	0.608	0.508
Near-fault records			
SD	0.940	1.671	0.339
CV	0.312	0.531	0.200

5. CONCLUSION

The efficiencies and precisions of different scaling methods in estimating the seismic responses of moment steel frames for far and near-fault earthquake ground motion records have been investigated. For this purpose, three special steel moment frames with three different heights (3, 6, and 10-story) and twenty earthquake records (ten far-fault and ten near-fault records) were selected. Incremental dynamic analyses were performed to estimate seismic responses for different scaling methods on the studied structures. The analysis results show that, scaling by UBC 97 method causes a large dispersion of the response results, while the normalization based on the peak ground velocity significantly reduces the dispersion of the results, and increases efficiency. The average coefficient of variations for scaling methods of UBC 97, absolute acceleration spectrum value of the first mode and FEMA 695 are 0.96, 0.42 and 0.32, respectively. In another word, the average variation coefficient by FEMA method is about 66% less than the equivalent value for UBC 97 scaling method. Furthermore, by comparing scaling methods for near and far-fault records, it is observed that variations of the responses with different scaling methods for near-fault earthquakes are more than the results obtained from far-fault earthquakes for UBC 97 and FEMA 695 methods

REFERENCE

[1] D. Yahmi, T., Branci,A. Bouchaïr, and E., Fournely, Evaluation of behaviour factors of steel moment-resisting frames using standard pushover method. *Procedia Engineering, 199,* 2017, .397-403.

[2] R., Taghinezhadbilondy, *Extending Use of Simple for Dead Load and Continuous for Live Load (SDCL) Steel Bridge System to Seismic Areas.*", Doctoral diss., Florida International University, Miami, FL, 2016

[3] S., Soleimani, A., Aziminejad, and A.S., Moghadam, *Approximate two-component incremental dynamic analysis using a bidirectional energy-based pushover procedure*. Engineering Structures, 157, 2018, 86-95.

[4] V., Soltangharaei, M., Zarean, V., Mahdavifar, R., Taghinezhad, and A., Taghinezhad, Response Modification Factor for Cold-Formed Steel Structures Using Pushover Analysis., *International Journal of Engineering Science, 7(12),* 2017, 15875-15880.

[5] B., Kissi, Y., Riyad, I., Mrani, M.A., Parron, N., Labjar, A., El Haouzi, and C., Guemimi, Influence of zone type on performance of retrofitted Reinforced Concrete buildings by using Pushover Analysis. *Materials Today: Proceedings*, *5(1)*, 2018, 22-29.

[6] R., Taghinezhad, A., Taghinezhad, V., Mahdavifar, and V., Soltangharaei, Numerical Investigation of Deflection Amplification Factor in Moment Resisting Frames Using Nonlinear Pushover Analysis. *International Journal of Innovations in Engineering and Science, 2(12)*, 2017, 1-7.

[7] H.N., Faal, and M., Poursha, Applicability of the N2, extended N2 and modal pushover

analysis methods for the seismic evaluation of base-isolated building frames with lead rubber bearings (LRBs). *Soil Dynamics and Earthquake Engineering, 98,* 2017, 84-100.

[8] R., Taghinezhad, A., Taghinezhad, V., Mahdavifar, and V., Soltangharaei, Seismic Vulnerability Assessment of Coupled Wall RC Structures. *International Journal of Science and Engineering Applications, 7 (2),* 2018, 1-7.

[9] A., Fiore, G., Spagnoletti, and R., Greco, On the prediction of shear brittle collapse mechanisms due to the infill-frame interaction in RC buildings under pushover analysis. *Engineering Structures, 121*, 2016, 147-159.

[10] A., Azizinamini, A., Yakel, A., Sherafati, R., Taghinezhad, and J.H., Gull. "Flexible Pile Head in Jointless Bridges: Design Provisions for H-Piles in Cohesive Soils." *Journal of Bridge Engineering 21(3),* 2016.

[11] F., Minghini, E., Bertolesi A., Del Grosso, G., Milani, and Tralli, A., Modal pushover and response history analyses of a masonry chimney before and after shortening. *Engineering Structures, 110,* 2016, 307-324.

[12] P., Huy, R., Taghinezhad, and A., Azizinamini. Experimental Investigation of Redundancy of Twin Steel Box-Girder Bridges under Concentrated Load, *Transportation Research Board 96th Annual Meeting, Washington DC, United States, No. 17-03649,* 2017.

[13] S., Li, Z., Zuo, C., Zhai, and L., Xie, Comparison of static pushover and dynamic analyses using RC building shaking table experiment. *Engineering Structures, 136,* 2017, 430-440.

[14] V., Soltangharaei, M., Razi, and M., Gerami, Comparative evaluation of behavior factor of SMRF structures for near and far fault ground motions. *Periodica Polytechnica. Civil Engineering, 60(1),* 2016, 75.

[15] X., Tian, M., Su, M., Lian, F., Wang, and S., Li, Seismic behavior of K-shaped eccentrically braced frames with high-strength steel: Shaking table testing and FEM analysis. *Journal of Constructional Steel Research, 143,* 2018, 250-263.

[16] P., Chomchuen, and V., Boonyapinyo, Incremental dynamic analysis with multi-modes for seismic performance evaluation of RC bridges. *Engineering Structures, 132,* 2017, 29-43.

[17] N., Fanaie, and S., Ezzatshoar, Studying the seismic behavior of gate braced frames by incremental dynamic analysis (IDA). *Journal of Constructional Steel Research, 99,* 2014, 111-120.

[18] P.A., Bońkowski, Z., Zembaty, and M.Y., Minch, Time history response analysis of a slender tower under translational-rocking seismic excitations. *Engineering Structures, 155*, 2018, 387-393.

[19] N., Nakamura, Time history response analysis using extended Rayleigh damping model. *Procedia Engineering, 199,* 2017, 1472-1477.

[20] S.W., Liu, R., Bai, and S.L., Chan, Dynamic time-history elastic analysis of steel frames using one element per member. *Elsevier Structures, 8,* 2016, 300-309.

[21] R., Taghinezhad, A., Taghinezhad, V., Mahdavifar, and V. Soltangharaei, Evaluation of Story Drift under Pushover Analysis in Reinforced Concrete Moment Frames. *International Journal of Research and Engineering, 5(1),* 2018, 296-302.

[22] FEMA P-58-1: *Seismic performance assessment of buildings. Volume 3–Supporting* Electronic Materials and Background Documentation, 2012.

[23] NIST, G-2011. GCR 11-917-15. *Selecting and scaling earthquake ground motions for performing response history analyses*. National Institutes of Standards and Technology & NEHRP, 2011.

[24] E. Kalkan, and A.K., Chopra, Practical guidelines to select and scale earthquake records for nonlinear response history analysis of structures. *US geological survey open-file report, 1068,* 2010, 126.

[25] J.M., Nau, and W.J., Hall, Scaling methods for earthquake response spectra. *Journal of Structural Engineering, 110(7),* 1984, 1533-1548.

[26] E., Miranda, Evaluation of site-dependent inelastic seismic design spectra. *Journal of Structural Engineering, 119(5),* 1993, 1319-1338.

[27] N., Shome, and C.A., Cornell, *Normalization and scaling accelerograms for nonlinear structural analysis. Proc. 6th US National Conference on Earthquake Engineering,* Seattle, WA, 1998, 1-12.

[28] Y.C., Kurama, and K.T., Farrow, Ground motion scaling methods for different site conditions and structure characteristics. Earthquake engineering & structural dynamics, 32(15), 2003, 2425-2450.

[29] N., Shome, C.A., Cornell, P., Bazzurro, and J.E., Carballo, Earthquakes, records, and nonlinear responses. *Earthquake Spectra, 14(3),* 1998, 469-500.

[30] B., Alavi, and H., Krawinkler, *Consideration of near-fault ground motion effects in seismic design. Proc. 12th World Conference on Earthquake Engineering (8),* Auckland, New Zealand, 2000.

[31] Y.C., Kurama, and KT., Farrow, Ground motion scaling methods for different site conditions and structure characteristics. *Earthquake engineering & structural dynamics, 32(15),* 2003, 2425-2450.

[32] P., Bazzurro, *Probabilistic Seismic Demand Analysis*, Docotral Diss., Stanford University, CA,. 1998.

[33] N., Shome, *Probabilistic seismic demand analysis of nonlinear structures*. Doctoral diss., Stanford University, CA. 1999.

[34] J.W., Baker, and A., Cornell, Spectral shape, epsilon and record selection. *Earthquake Engineering & Structural Dynamics, 35(9),* 2006, 1077-1095.

[35] S.F.F, Mehanny, and G.G., Deierlein, *Modeling and assessment of seismic performance of composite frames with reinforced concrete columns and steel beams,* Doctoral diss., Stanford University, CA.,1999.

[36] P.P., Cordova, G.G., Deierlein, S.S., Mehanny, and C.A., Cornell, *Development of a two-parameter seismic intensity measure and probabilistic assessment procedure. Proc. 2nd US-Japan Workshop on Performance-Based Earthquake Engineering Methodology for Reinforced Concrete Building Structures, Berkeley, CA,* 2000, 187-206.

[37] B., Alavi, and H., Krawinkler, Behavior of moment- resisting frame structures subjected to near- fault ground motions. *Earthquake engineering & structural dynamics, 33(6),* 2004, 687-706.

[38] R., Youngs, M., Power, G., Wang, F., Makdisi, and C.C., Chin, *Design ground motion library (DGML)–tool for selecting time history records for specific engineering applications. Proc. SMIP07 Seminar on Utilization of Strong-Motion Data*, Sacramento, CA: California Geological Survey, 2007, 109-110.

[39] FEMA-P695, *Quantification of Building Seismic Performance Factors.* Federal Emergency Management Agency (FEMA), 2009.

[40] N., Luco, and C.A., Cornell, Structure-specific scalar intensity measures for near-source and ordinary earthquake ground motions. *Earthquake Spectra*, *23(2),* 2007, 357-392.

[41] UBC-1997, Uniform *Building Code (UBC), International Code Council;* Whittier, CA, USA, 1997.

[42] ASCE/SEI 7-10, Minimum Design Loads for Buildings and Other Structures, *American Society of Civil Engineers (ASCE),* Reston, Virginia, 2010.

[43] A., Catalán, A., Benavent-Climent, and X,.Cahís, Selection and scaling of earthquake records in assessment of structures in low-to-moderate seismicity zones. *Soil dynamics and earthquake engineering, 30(1-2),* 2010, 40-49.

[44] S.K., Shahi, and J.W., Baker, NGA-West2 models for ground motion directionality. *Earthquake Spectra*, *30(3),* 2014, 1285-1300.

[45] ANSI/AISC 360-16, Specification for structural steel buildings. *AISC,* 2010.

[46] Seismo Soft. Seismo Struct, *A computer program for static* and dynamic analysis for framed structures. version 6.0, Available from URL: www.seismosoft.com(online), 2012.

[47] M.H., Scott, and G.L., Fenves, Plastic hinge integration methods for force-based beam–column elements. *Journal of Structural Engineering, 132(2),* 2006, 244-252.