Evaluation of strong ground motion record selection & scaling methods in case of regular and irregular generic frames

M. Molavi[1], M. Ghafory-Ashtiany [2], S. Arian-Moghaddam [3]

[1] *MSc in earthquake engineering, IIEES, Tehran, Iran, m.molavi@iiees.ac.ir*
[2] *Professor, IIEES, Tehran, Iran, ashtiany@iiees.ac.ir*
[3] *PhD student, IIEES, Tehran, Iran, s.arianmoghaddam@iiees.ac.ir*

Abstract

In performance based earthquake engineering, Nonlinear Time History Analysis (NLTHA) is recognized as an efficient tool for reliable estimation of seismic demands. Knowing the fact that nonlinear structural response can be highly sensitive to the selection and scaling of input Strong Ground Motions (SGMs), many SGM selection and modification methods have been proposed in the literature, however, most of them are evaluated quantitatively just in case of several typical regular structures. Despite the importance of this issue, few studies have focused on the comprehensive evaluation of available methods to assess their performance by using a series of benchmark structures and find the applicability scope of each approach. This paper presents an evaluation of three well-known SGM scaling methods (PGA-based, $S_a(T_1)$-based and the conventional code-based scaling approach) as well as a recently introduced method known as modal pushover based scaling and selection. To have a comprehensive assessment, forty of 2-D one bay vertically regular frames designed with general features as: five different heights (N= 3, 6, 9, 12 and 15 stories), two types of seismic behavior i.e. stiff and flexible frames with periods of 0.1 and 0.2 time of the number of stories, respectively and four ductility capacities (μ =1, 2, 4 and 6). Furthermore, for all frames three types of irregularity (stiffness, strength, and combined stiffness and strength) have been applied in four different locations along the height (top story, middle story, first story and lower half of the frame) using four modification factors (0.2, 0.5, 2 and 5). A set of 22 pairs of horizontal far-field SGMRs which was used in the ATC63 project is selected for response history analysis of total 1960 frames. Engineering Demand Parameters (EDPs) considered in this study are: Maximum Inter-story Drift Ratio over building height, absolute acceleration of each story, base shear, and etc. The results illustrate that the efficiency of each method is affected by properties of the structure that is subjected to NLTHA. Ductility capacity is a property that may significantly threaten the accuracy of predicted EDPs if it is not involved in the method of selection and/or scaling. On the other hand, the results indicate that although vertical irregularities are important, but they may not produce the meaningful bias in the results. It is shown that the efficiency of each method depends on the desired EDP selected to be controlled, i.e. the proper performance of any method in the prediction of a specified EDP does not guarantee its acceptable efficiency for predicting other EDPs keeping the same level of accuracy.

Keywords: Nonlinear time history analysis, Record selection and scaling, performance based earthquake engineering

1. Introduction

Nonlinear Time History Analysis (NLTHA) is recommended as an ultimate method for analysis of the irregular structures and structures with multi-mode contribution to its response. The reason can be attributed to the application of fewer simplifying assumptions in the structural modeling procedure as well as considering input seismic loads as realistic as possible. However due to the sensitivity of NLTHA to the selection and scaling of input Strong Ground Motions (SGMs) [1, 2], it does not always result in an acceptable precision for estimated Engineering Demand Parameters (EDPs). Many researchers have been proposed methods for selection and scaling of the input SGMs, while there is non- negligible deviation among both the overall methodology and consequent results. Despite the fairly large interest in the proposing new methods, few studies have concentrated on the comprehensive evaluation of these methods to provide engineers with impartial guidance for practical NLTHA application. So, the engineers are left to make an important decision on SGMR without having enough understanding of its nature [3]. Most judgments in this field are qualitatively, while available quantitative studies do not cover a wide enough range of methods and are limited to several typical regular structures.

This paper presents an evaluation of three well-known SGM scaling methods (PGA-based, $S_a(T_1)$-based and the conventional code-based scaling approach); as well as a recently introduced method known as modal pushover based scaling and selection [4]. The scaling a SGM to the target values such as PGA and $S_a(T_1)$ are the conventional methods in earthquake engineering, while the routine code-based methods can be considered as the generalization of spectrum-based ones so that they cover the effect of nonlinearity as well as higher modes contribution. The recently introduced modal pushover based scaling scheme is proposed to improve the reliability of the estimated EDPs by involving the nonlinear behavior of structure obtained through modal push over analysis as shown in Fig.1 [4]. Focusing on the quantitative assessment of the mentioned methods, a set of 2-D steel frames are designed and analyzed considering nonlinear modeling in the following sections. EDPs considered in this study are: Maximum Inter-story Drift Ratio (MIDR) over building height, absolute acceleration of each story, base shear, base moment, story displacement, and inter story drift ratio.

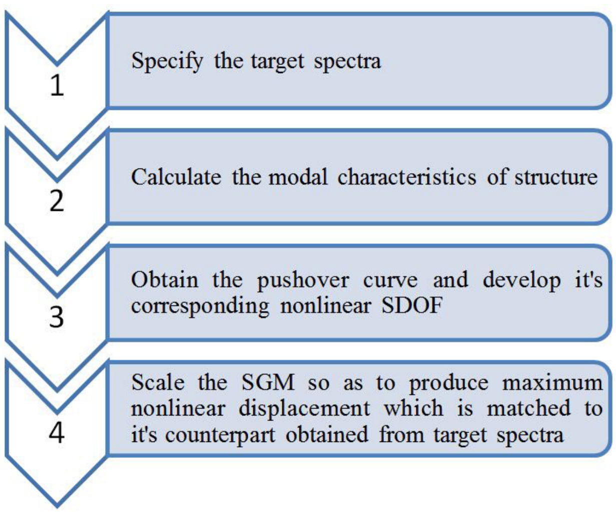


Fig. 1 – General steps of modal pushover methods [4]

2. Ground Motions

A set of 22 pairs of horizontal SGMRs selected from a specific far-field set, which was used in the FEMA P. 695 [5] as the suggested SGMs for NLTHA. Detailed information about the selected SGMRs and selection criteria, are provided in [6]. Fig.2 shows the magnitude–distance distribution, as well as the 5% damped acceleration response spectra of the general set.

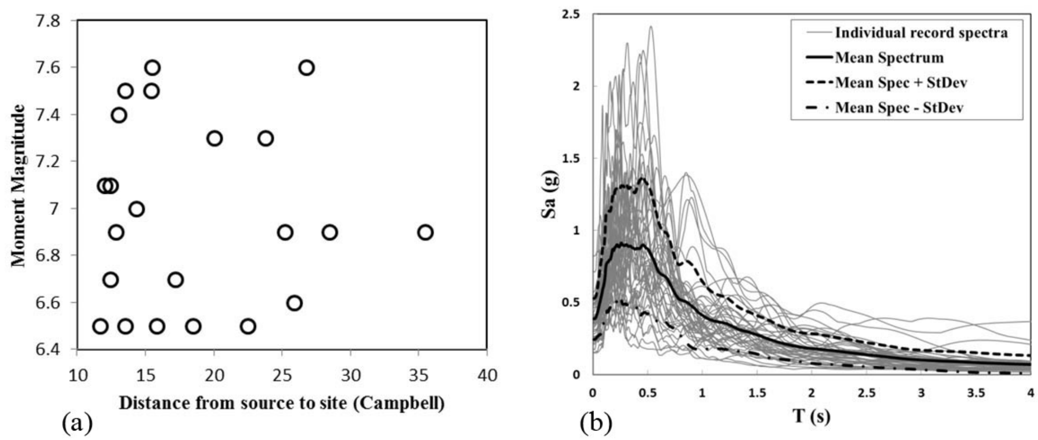


Fig. 2 – a) The magnitude–distance distribution; and b) the 5% damped acceleration response spectra of the general set of SGMRs

3. Structural Models

3.1 Regular frames

For comprehensive study, a group of 2-D one bay vertically regular frames of five different heights (i.e. 3, 6, 9, 12, and 15 stories) has been used in this study. The height-wise distribution of stiffness was tuned to achieve equal drifts in all stories that are calculated using the Iranian code of practice for seismic resistant design of buildings forces (Standard 2800) [7]. The overall procedure that is relatively similar to all force-based seismic code can be summarize as:

$$F_i = V_b \frac{W_i h_i^{\,k}}{\sum_{j=1}^{N} W_i h_i^{\,k}} \qquad \text{Where:} \quad k = \begin{cases} 1 & T_1 \le 0.5\,\text{sec} \\ \dfrac{T_1 + 1.5}{2} & 0.5 < T_1 < 2.5\,\text{sec} \\ 2 & T_1 \ge 2.5\,\text{sec} \end{cases} \tag{1}$$

Where F_i, V_b, W_i, and h_i, respectively, are the lateral inertia force, the total base shear, story weight, and elevation of the i^{th} floor. The lumped mass at each floor is 20 tons; the story height (h) is 3.5m and the beam span (L) is 7m. Assuming that the second moment of cross sectional area for each beam and its supporting columns in the lower story are the same, two types of seismic behavior i.e. stiff and flexible frames considered with periods of 0.1 and 0.2 time of the number of stories that correspond to the lower band period (T_l) and upper band period (Tu), respectively, and is shown in Fig.3. The frames were designed according to the strong-column/weak-beam philosophy, therefore, plastic hinges form only at beam ends and the base of the columns in the first story; the columns in other stories were assumed to behave elastic. The yield strength distribution was chosen such that yielding is observed almost simultaneously at all plastic hinges under the lateral force distribution described in Eq. (1). The yield base shear is:

$$V_{by} = (A_y / g) W \tag{2}$$

Where W is the total seismic weight and A_y is the design spectral value at the first modal period T_1.

To cover a variety of ductility values, nonlinear response spectra with constant ductility equal to 1, 2, 4, and 6 have been utilized as the design spectra. Thus, eight different designs (two values of T_1 and four values of μ) are considered for each frame height, leading to a total of 40 frames. For the NLTHA, 5% Rayleigh damping is assigned to the first mode and the mode at which the cumulative mass participation exceeds 95%. Furthermore, global P-delta effect is included. The development of the structural database follows similar principles as those can be found in [8, 9, 10].

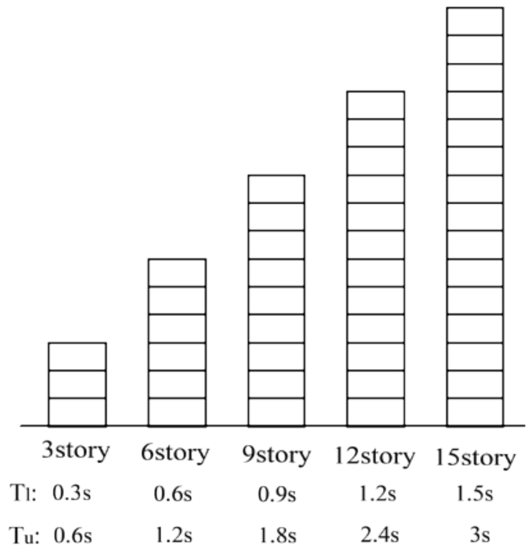


Fig. 3 – Vertically regular one-bay 3, 6, 9, 12, and 15 frames, including two values, TL or TU, for the fundamental vibration period, T_1.

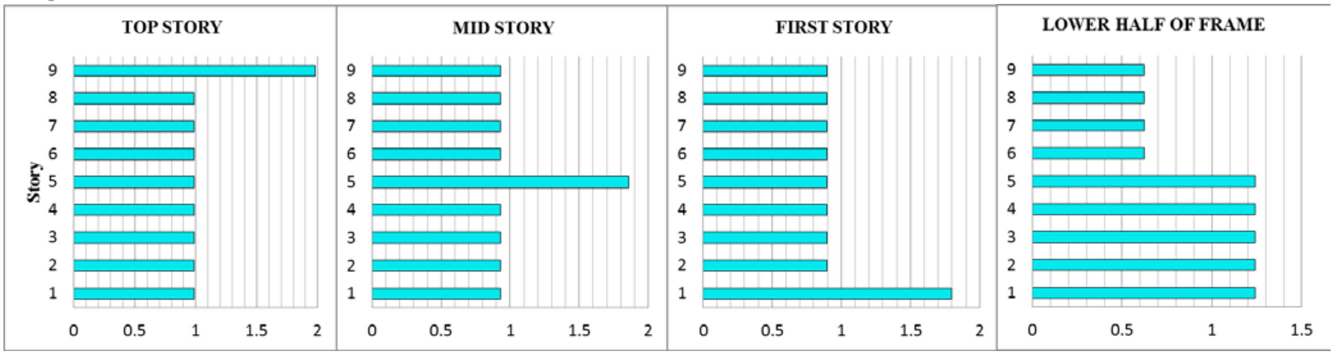
3.2 Vertically irregular frames

Corresponding to each regular frame introduced in the previous section, 48 vertically irregular frames were extracted by modifying stiffness and/or strength in four different locations along the height, i.e. top story, middle story, first story, and lower half of the frame. To obtain a soft or stiff story, the story stiffness was divided or multiplied by a modification factor; and to obtain a weak or strong story, the story strength was divided or multiplied by the modification factor. Tow modification factors which are considered in this study are 2 and 5. Considering that applying stiffness and/or strength vertical irregularities by using a modification factor as large as 5, may result in an unrealistic structural system, these frames were grouped based on their modification factor as: 1) frames with possible level of irregularities and modification factor of 2 (C1); and 2) frames with exaggerated level of irregularities and modification factor of 5 (C2).

To design "Stiffness Irregular Frames-Km"; the stiffness of the intended story (or stories) of the corresponding regular frame was modified by changing the stiffness of the columns and the beams that are supported by them. To maintain the period of the irregular structure equal to the period of regular counterpart frame, all story stiffness values were scaled uniformly. The irregularity patterns used along the height are presented in Fig.4.

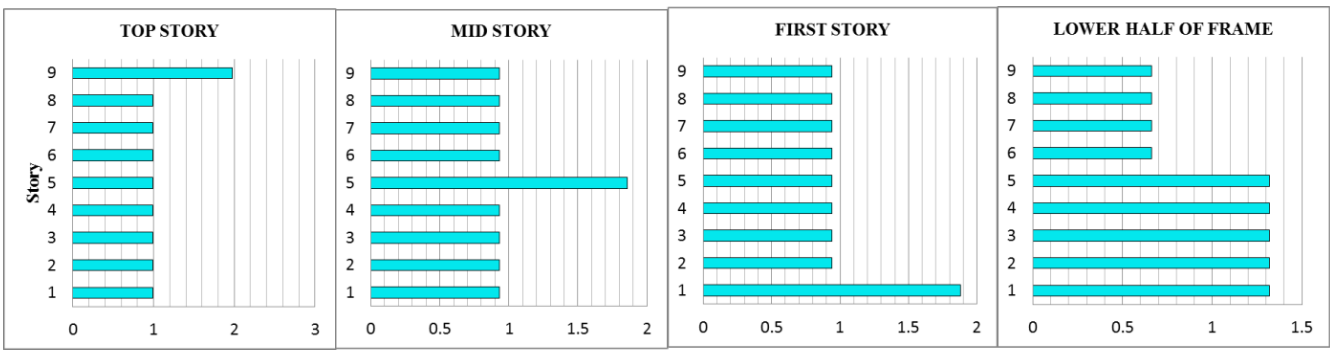
Similarly to design "Strength Irregular Frames-Sm"; an intended story (or stories) of any regular frame was modified by a modification factor to produce its irregular case. As it is assumed that the columns remain elastic during the NLTHA, the strength of a story was modified by changing only the strength of the beam at the top of the story. However, if the intended irregularity is applied in the first story, the strength of the columns is also changed. The irregularity patterns used along the height are presented in Fig.5.

For studying the effect of simultaneous irregularity in stiffness and strength (KSm); both values of the intended story (or stories) of the corresponding regular frame were modified respectively.



Ratio of Story stiffness of irregular frame to Story stiffness of reference 'regular' frame

Fig. 4 – Sample of stiffness irregularity patterns used along the height.



Ratio of Story strength of irregular frame to Story strength of reference 'regular' frame

Fig. 5 – Sample of strength irregularity patterns used along the height.

4. Efficiency Assessment of the SGMs Scaling Methods

Efficiency of a SGM scaling method is a factor that plays an important role in the evaluation of the method's robustness. Less dispersion in the estimated EDPs can be interpreted to more efficiency of an applied SGM scaling method. Coefficient of Variation (COV) that shows the extent of variability relative to the mean of the population [11], is used as an efficiency index for a method. Figs.6 to 9 show COV of six different investigated EDPs of all regular frames that have been estimated under four different SGMs scaling methods. As it can be seen, the efficiency of these methods may change according to the dynamic characteristics of the frames. It is shown that proper involvement of these characteristics such as ductility, heights, and etc. should be included by SGM scaling methods. On the other hand the efficiency of the scaling methods strongly depends on the type of desired EDP. In the other words, effectiveness of any method in the prediction of a specified EDP does not guarantee its acceptable efficiency for predicting other EDPs keeping the same level of accuracy. It also indicates that focusing only on a separate set of EDPs classified in to the main groups of force or displacement –based demands, may not give a good coverage of all facts.

Although the MPS based scaling method somehow considers the nonlinear characteristics, but the ductility still is an influencing parameter on the efficiency of this method as it can be seen in Fig.8. On the other hand, the performance of MPS method in the estimation of EDPs may not be the same for all frames with same periods but different heights. This observation can be attributed to the fact that simplifying a multi degree of freedom structure to a single degree of freedom system ignores some part of effective dynamic characteristics.

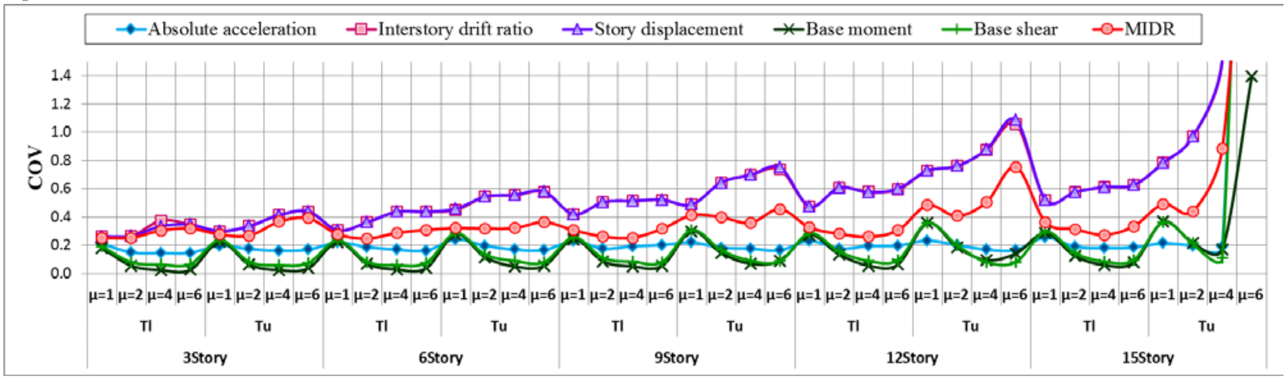


Fig. 6 – Estimated C.O.V values for different EDPs in case of regular frames using PGA-based scaling

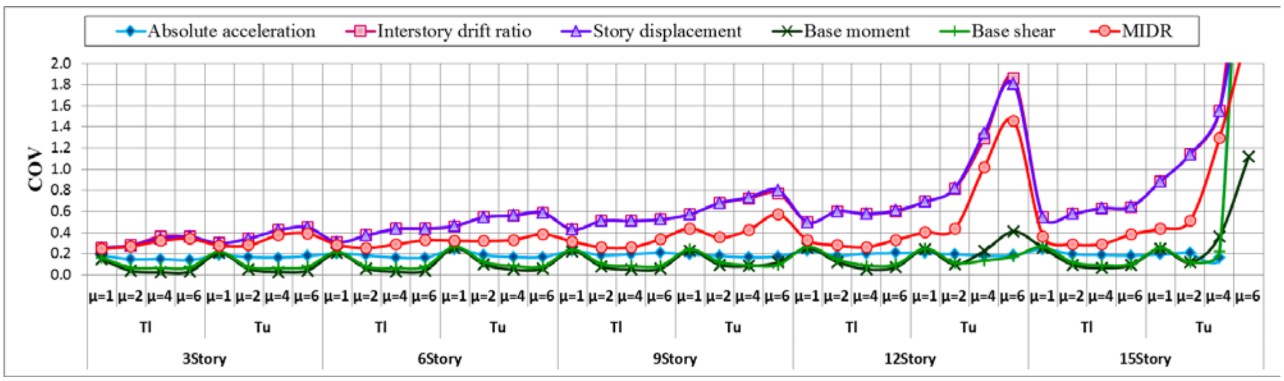


Fig. 7 – Estimated C.O.V values for different EDPs in case of regular frames using Code-based scaling

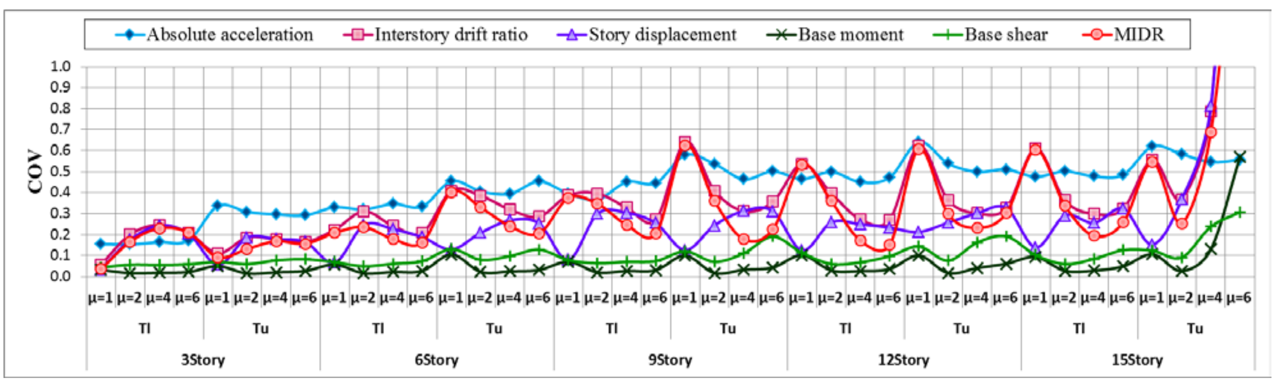


Fig. 8 – Estimated COV values for different EDPs in case of regular frames using MPS

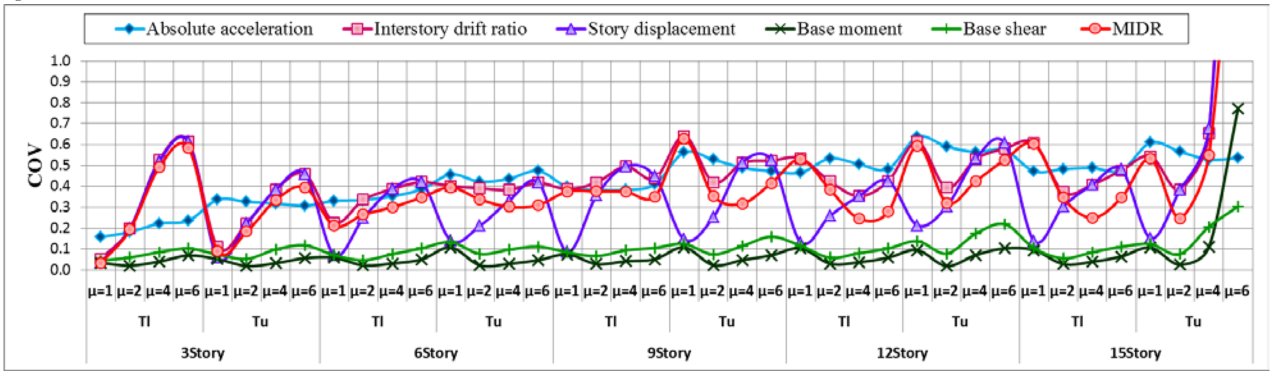


Fig. 9 – Estimated COV values for different EDPs in case of regular frames using $S_a(T_1)$-based scaling

To compare the scaling methods with each other in case of the vertically irregular frames, Eq. (3) demonstrates the COV of the estimated EDP by scaling method I, divided by corresponding estimated value by method II.

$$\beta = \frac{COV\ (EDP_{Method\ I})}{COV\ (EDP_{Method\ II})} \tag{3}$$

Empirical probability cumulative distribution curve of these β values calculated for MPS relative to three other cases has been calculated for all EDPs and the results are shown in Fig.10 for C1 irregular frames. According to this figure, the type of desired EDP plays a key role in the relative efficiency of the scaling methods. On the other hand, fairly wide range of the β values shows that any method (for example MPS in current case) is not always superior to others. In other words, dynamic characteristics of the analyzed structure may significantly affect the performance of a method against others. Also, according to these observations proper performance of any method in the prediction of a specified EDP cannot be verified by its evaluation by using of a limited number of structures. In fact, the acceptable performance of a method cannot be generalized for predicting same EDP in all other structures keeping the same level of accuracy.

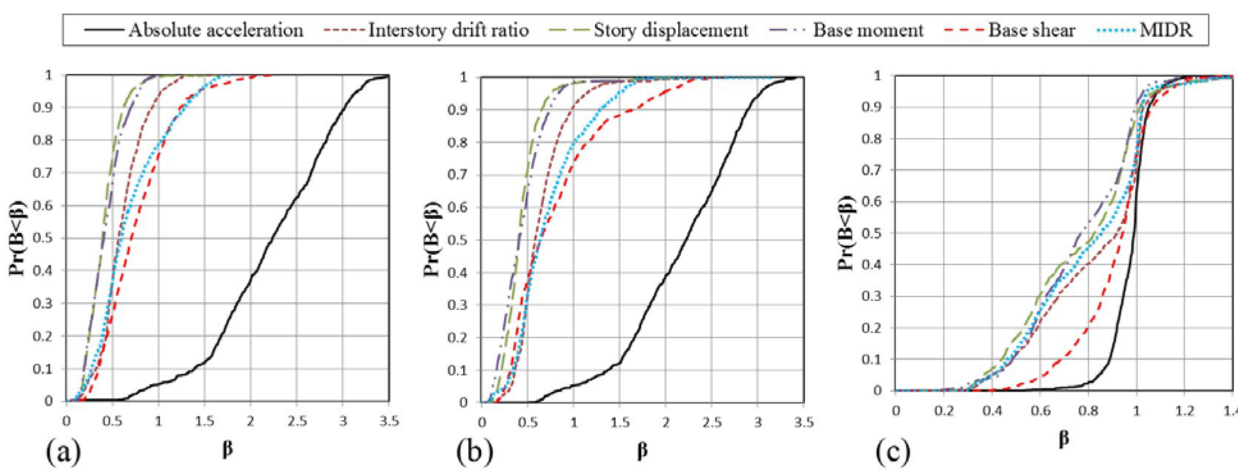


Fig. 20 – Empirical probability cumulative distribution curves for β in case of: a) MPS/Code; b) MPS/PGA; and c) MPS/$S_a(T_1)$

To have a closer look at the MPS method vs. $S_a(T_1)$- based method, Fig.10.c for MIDR has been de-aggregated into Fig.11 to separately present the stiff or flexible frames, story numbers, and ductility capacity. As it can be seen in Fig.11.a, there is no tangible difference between cumulative distribution curves of β values in stiff or flexible irregular frames. According to Fig.11.b although MPS method has a relative superiority in most of 3 story frames, but it is evident that by increasing the number of stories (especially in case of 15 story frames) this observation diminishes in such a way that in a large number of 15 story irregular frames, there is no differences between the results of MPS or $S_a(T_1)$ scaling method. Furthermore, according to the Fig.11.c with increasing ductility, the efficiency of the MPS is improved compared to $S_a(T_1)$-based method. The latter observation can be justified by noting to the ignorance of the ductility in the $S_a(T_1)$-based scaling method.

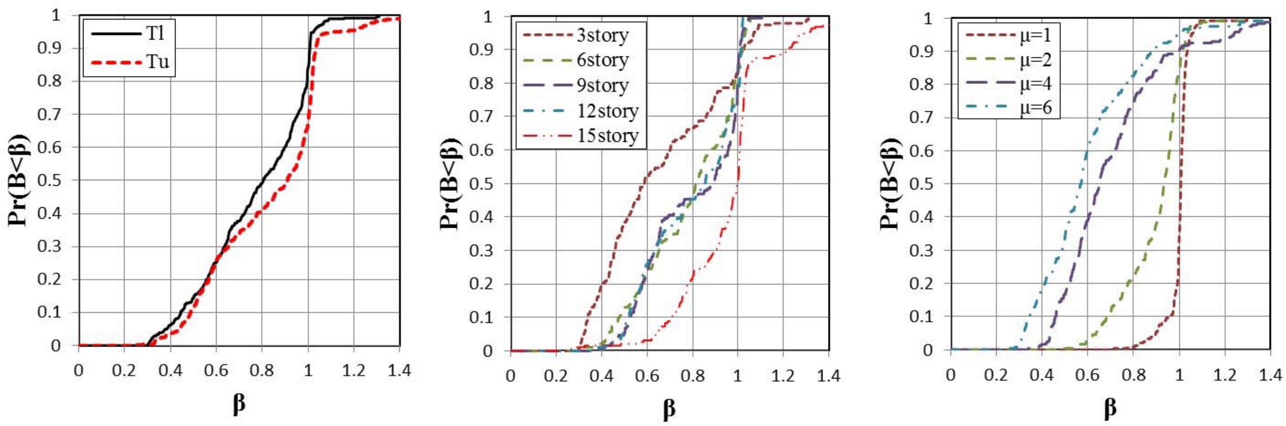


Fig. 31 – Empirical probability cumulative distribution curves for β in case of MIDR values disaggregated to compare MPS against S_a (T_1)

5. Vertically Irregularities Effects

The effects of vertical irregularities on the efficiency of the scaling methods have been studied in this section. For this purpose, γ index is introduced to measure the sensitivity of C.O.V to the irregularity as:

$$\gamma = \frac{COV_{Irregular}}{COV_{regular}} \tag{4}$$

Empirical probability cumulative distribution curve of these γ values has been calculated for all EDPs and the results are shown in Fig.12 for C1 irregular frames. The results indicate the amount of vertical irregularity on the efficiency of all scaling methods depends on the desired EDP; and the base moment and base shear are, respectively, ranked as the most sensitive EDPs against irregularities.

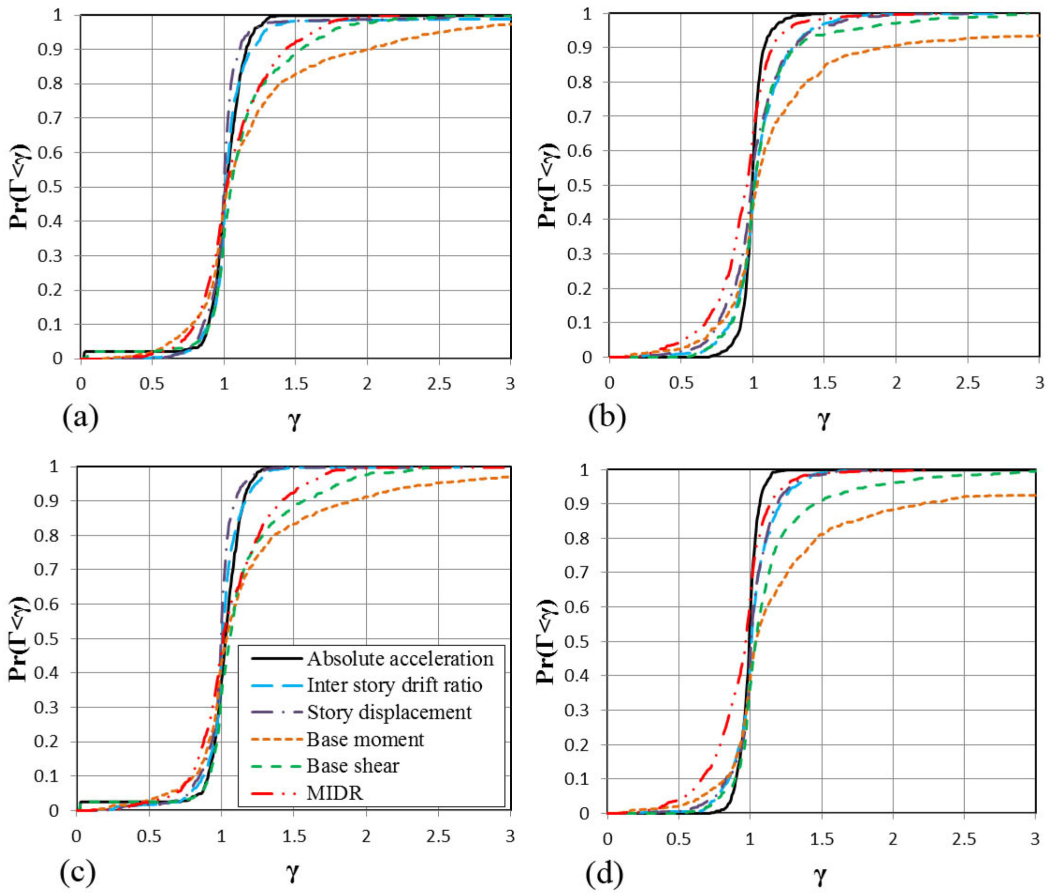


Fig. 42 – Empirical probability cumulative distribution curves for γ in case of estimated EDPs by using: a)Code; b)MPS; c)PGA; d)$S_a(T_1)$

For the evaluation of the sensitivity of different investigated scaling methods relative to each other, cumulative distribution curve of γ values for C1 irregular frames is plotted in Fig.13. As it is clear this figure, the sensitivity of all four scaling methods to the vertical irregularities does not show significant differences compared to each other. According to Fig.13.f, there are no meaningful differences between MPS and $S_a(T_1)$ methods in case of computed MIDR. In other words, MPS method despite its complexity in comparison with the $S_a(T_1)$, does not offer more advantages in case of vertical irregularity effects. For a closer look at this observation cumulative distribution curve of the γ values for the estimation of MIDR for C1 irregular frames are plotted in a disaggregated format based on the irregularity types in Fig.14. As it can be seen in this figure, in all types of vertical irregularities, MPS does not show a sensitivity to vertical irregularities lesser than $S_a(T_1)$. These observations can be attributed to the weakness of MPS method in taking into account vertical irregularity characteristics in a simplified nonlinear SDOF system equivalent to the first mode of analyzed structure.

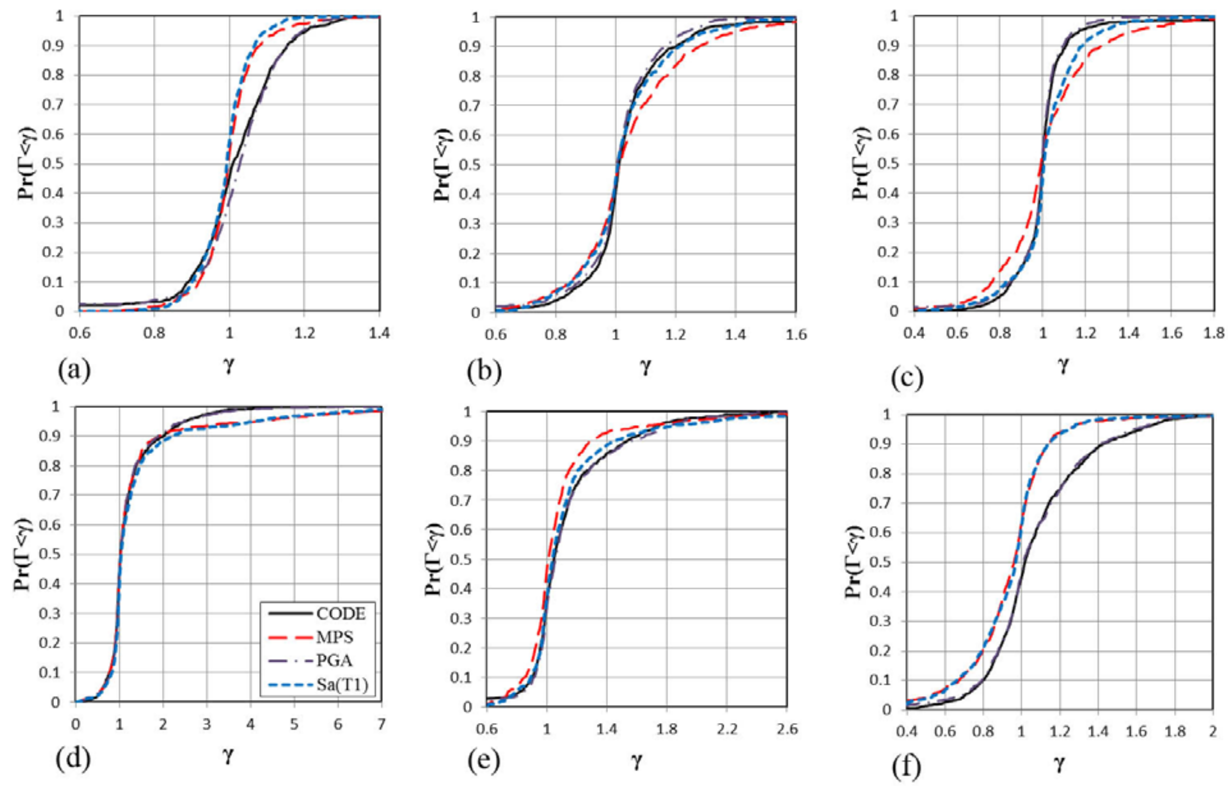


Fig. 53 – Empirical probability cumulative distribution curves for γ in case of different methods for: a) Absolute acceleration; b) Inter story drift ratio; c) Story displacement; d) Base moment; e) Base shear; f)MIDR

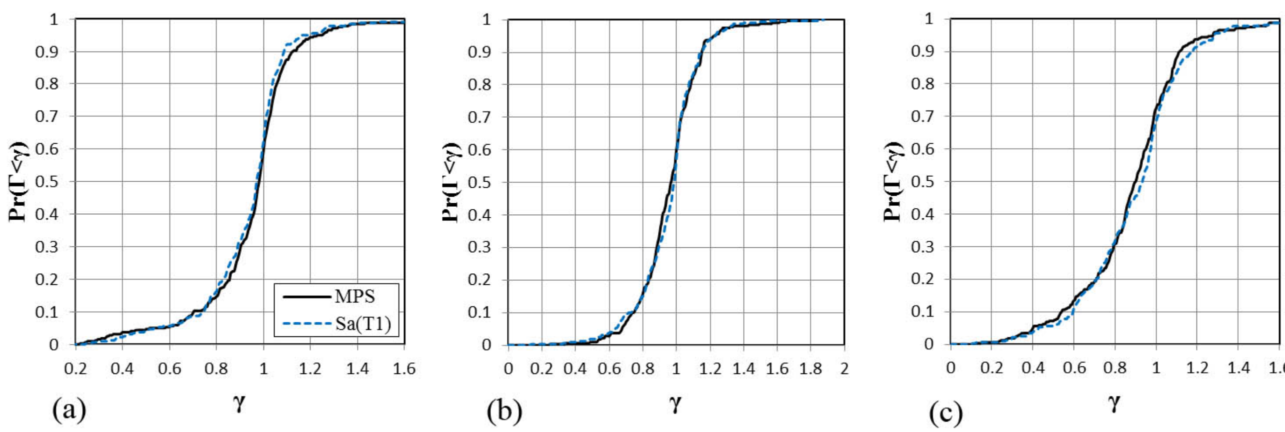


Fig. 64 – Empirical probability cumulative distribution curves for γ in case of MIDR values disaggregated to compare MPS against Sa (T_1); a)Km; b)Sm; c)KSm

As previously mentioned, two level of vertical irregularities (C1 and C2) is applied to the regular frames, and sofar here only the results of C1 irregular frames are presented. In Fig.15, C2 irregular frames including frames with exaggerated levels of irregularity in comparison with C1 irregular frames in terms of all six investigated EDPs by MPS method. According to this figure, sensitivity of the MPS method against irregularities in C2 frames does not change dramatically compared to the results obtained for C1 frames. These observations are highlighted especially in the case of estimated MIDR and inter story drift ratio values.

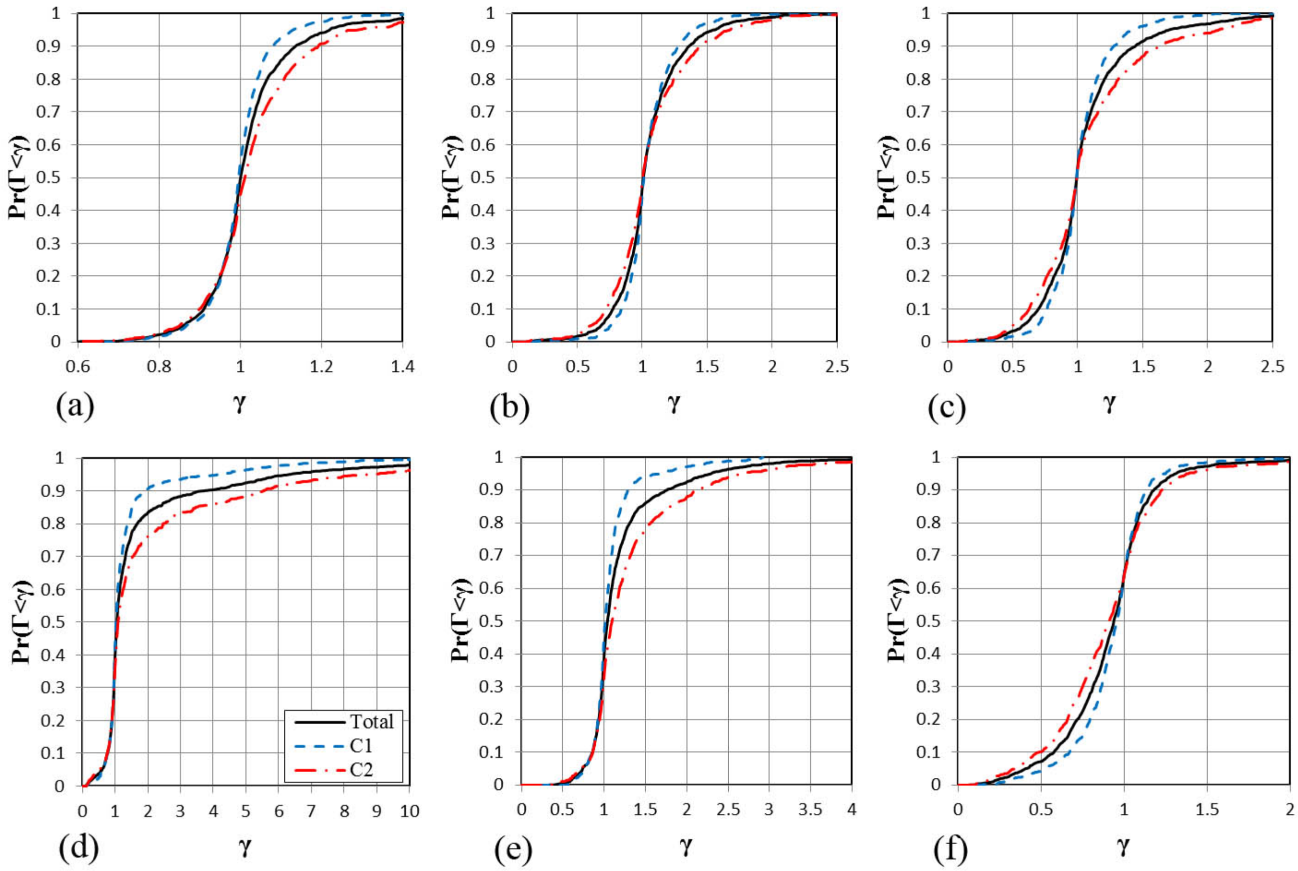


Fig. 15 – Empirical probability cumulative distribution curves for γ in case of irregular frames for: a)Absolute acceleration; b)Inter story drift ratio; c)Story displacement; d)Base moment; e)Base shear; f)MIDR

6. Conclusion

The objective of this study was to evaluate the efficiency of several of SGM scaling methods in estimating different EDPs for a comprehensive database of steel generic frames. The most important concluding remarks can be listed as:

- Dynamic characteristics of the selected structural system can play an important role in the performance of the ground motion scaling methods.
- Considering the structural ductility in the SGMs selection and/or scaling procedure may increase the accuracy of the NLTHA outputs.
- Although vertical irregularities affect the efficiency of the investigated SGMs scaling methods, but they may not cause the meaningful bias in the results.
- The efficiency of each method strongly depends on the desired EDP selected to be controlled, i.e. the proper performance of any method in the prediction of a specified EDP does not guarantee its acceptable efficiency for predicting other EDPs keeping the same level of accuracy. Also the proper performance of any method in the prediction of a specified EDP cannot be verified by its evaluation by using of a limited number of structures. In fact, the acceptable performance of a method cannot be generalized for predicting same EDP in all other structures keeping the same level of accuracy.

7. References

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