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# An Energy Based Adaptive Pushover Analysis for Nonlinear Static Procedures

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**ABSTRACT:** Nonlinear static procedure (NSP) is a common technique to predict seismic demands on various building structures by subjecting a monotonically increasing horizontal loading (pushover) to the structure. Therefore, the pushover analysis is an important part of each NSP. Accordingly, the current paper aims at investigating the efficiency of various algorithms of lateral load patterns applied to the structure in NSPs. In recent years, fundamental advances have been made in the NSPs to enhance the response of NSPs toward nonlinear time history analysis (NTHA). Among the NSPs, the philosophy of "adaptive procedures" has been focused by many researchers. In the case of utilizing adaptive procedures, the use of incremental force vector considering the effects of higher modes of vibration and stiffness deteriorations is possible and seems that it can lead to a good prediction of seismic response of structures. In this study, a new adaptive procedure called energy-based adaptive pushover analysis (EAPA) is implemented based on the work done by modal forces in each level of the structure during the analysis and is examined for steel moment resisting frames (SMRFs). EAPA is inspired by force-based adaptive pushover (FAP) and story shear-based adaptive pushover (SSAP). FAP has applied modal forces directly into load patterns; SSAP, on the other hand, has implemented the energy method in system`s capacity curve for measuring the equivalent movement. EAPA has enforced the concept of energy directly in load pattern; so that by using the modal forces-movements an energy-based adaptive algorithm is obtained. Hence, the effects of higher modes, deterioration in stiffness and strength, and characteristics of a specific site are incorporated and reflected in applied forces on the structure. Results obtained from the method proposed a desirable accordance with the extracted results from NTHA over the height of the structure.

**Keywords**: Adaptive Pushover, EAPA Procedure, Seismic Response, SMRF Structures, Stiffness Deterioration.

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## INTRODUCTION

Nonlinear time history analysis (NTHA) is known as the most accurate method to evaluate the response of the structures subjected to earthquake excitations; though, all nonlinear static procedures (NSPs) suffer invariably from some limitations due to their inherent static assumptions (Krawinkler and Seneviratna, 1998). Nevertheless, some of them are still popular for assessing the seismic capacity of structures due to their simplicity and application (Jiang et al. 2010; Amini and Poursha, 2016; Izadinia et al., 2012).

Considering an invariant load pattern during the analysis is a drawback of conventional pushover, leading to ignoring the effects of higher modes of vibration. In order to overcome this drawback, multi-run modal pushover procedures such as the well-known modal pushover analysis (MPA) (Chopra and Goel, 2002) subject the system to separate lateral loads corresponding to the considered elastic mode shapes. Successively, the total seismic response of the system is estimated through the combination of the responses due to each modal load (Chopra and Goel, 2002; Shakeri et al., 2010). Also, a modified modal pushover analysis (MMPA) based on elastic spectral responses has been proposed by Chopra et al. (Chopra et al., 2004) whereby the problem with potential reversal displacement of roof was solved (Hernandez-Montes et al., 2004). Moreover, Hernández-Montes proposed an energy based capacity curve method in which displacements of all floors were involved (Hernandez-Montes et al., 2004; Shakeri and Ghorbani, 2015). Using the method proposed by Hernández-Montes can possibly define the capacity curve of the system corresponding to higher modes than the first mode, removing the concerns about reversal displacement of roof. In other words, an energy-based modal pushover analysis was proposed which hereafter referred to as EMPA.

Based on what mentioned above and the importance of NSPs in engineering practice to predict seismic demands on building structures, future attempts are needed to improve the pushover analysis procedures. This need is confirmed by the fact that the simplified procedures based on invariant load patterns are partially inadequate to predict inelastic seismic demands in buildings when the issues such as effects of higher modes, inelastic effects, and cumulative damages are significant (Shakeri et al., 2012; Abbasnia et al., 2013; Kunnath and Kalkan, 2004). In recent years, so as to overcome some of these drawbacks, a number of enhanced procedures have been proposed considering the effects of instantaneous state of the system related to equivalent seismic loads at each pushover step (Poursha et al., 2011, Malekzadeh, 2013; Belejo and Bento, 2016; Shakeri et al., 2010; Izadinia et al., 2012).

However, there are still issues of controversy such as selecting a suitable load vectors, variations in nonlinear response associated with record-to-record variability, difficulty in selecting appropriate load vectors, and the convergence of the analysis related to sudden drops in component strength (NIST, 2010). For this reason, proposed method namely energy-based adaptive pushover analysis (EAPA) aims to use the energy concepts to provide an appropriate adaptive load pattern to approach the responses of the NTHA. Also, in this load pattern, it is possible to implement an incremental process that may lead to the good responses compatible with drop in system strength in inelastic deformation ranges; in other words, removing the concerns about the convergence of the analysis during the inelastic ranges of deformation. Also, equivalent forces of a given earthquake can be computed and practically consider the variations related to record-to-record variability. Moreover, it depends on engineering design decisions to simply use the design earthquake spectral response recommended in seismic codes such as ASCE 7-05 (S.E. Institute, 2006) for a particular site. This study shows that by using the concept of energy to illustrate the lateral load vector in nonlinear static methods is closer to reality and gives better answers.

Pushover analysis as an important part of each NSP, is a static technique that directly involves the nonlinear properties of materials (Mazza, 2014; Poursha et al. 2014) investigated by many researchers for various structures (e.g. Nguyen et al., 2010; Khoshnoudian and Kashani, 2012; Malekzadeh, 2013; Panyakapo, 2014). Conventional pushover methods apply an increasingly single direction predetermined load pattern which is kept constant throughout the analysis (EN, 2004; FEMA, 2005; Camara and Astiz, 2012; Manoukas et al., 2012; Giorgi and Scotta, 2013; Beheshti-Aval and Keshani, 2014). Although choosing a constant load pattern is simple, it may lead to uncertain predictions of responses in high-raise structures; since, for example, the modal characteristics of the structure can be varied during the analysis.

One of the most important assumptions of multi-run modal pushover procedures (Chopra, 2001) can be related to the independent analysis of system in each mode by pushing the structure with the corresponding modal load patterns. In order to define the system's overall responses in most well-known modal pushover analysis (MPA), the obtained results of each mode are combined using an appropriate modal combination rule such as square root of sum of squares (SRSS) or complete quadratic combination (CQC) (Chopra and Goel, 2002, 2007). Also, a consecutive modal pushover (CMP) procedure is proposed by Poursha et al. (2009) considering the effects of higher modes. In the same way, a modified version of CMP is proposed and examined in braced frames (Khoshnoudian and Kashani, 2012). Even though modal non-adaptive approaches offer improvements over conventional methods, the limitations about ignoring changes in structural properties of the system during the analysis are existent yet (Krawinkler and Seneviratna, 1998; Tarta and Pintea, 2012). Using these patterns, some of seismic behaviors of system such as material accumulated strain, reducing stiffness, and subsequent increase in the structure period of vibration are not taken into account. Therefore, it seems that non-adaptive NSPs which do not consider these changes present unreliable responses.

Based on the above-mentioned issues and the influence of inelastic deformations on seismic behavior of system, many studies have been carried out to consider the changes of modal characteristics in their analysis which are known as adaptive pushover analysis (Araújo et al., 2014; Tarbali and Shakeri, 2014; Beheshti-Aval and Jahanfekr, 2015). In an adaptive procedure, the load pattern as shown in Figure 1 is updated at each step of the analysis and the progress of structural stiffness deterioration is reflected during the inelastic deformation stages.

In recent years, many researchers have proposed different lateral load patterns to improve the compatibility of time-varying inertial forces with new modal characteristics of the system. Reinhorn (1997), and Bracci et al. (1997) utilized the concepts of adaptive pushover to improve the NSPs. In these procedures the analysis is started with the assumption of an initial lateral load distribution, and continued with the changing instantaneous patterns in accordance with floor shear strength in the next steps. Lefort (2000) extended this work by applying a scaled additional force that was associated with considering the participation of higher modes.

Albanesi et al. (2002) proposed another pushover method where the analysis is defined based on the concept of energy. The proposed adaptive energy-based pushover analysis (AEPOA) not only include the internal structural properties in lateral force that is applied at each step, but also the expected kinetic energy of the motion of the system under earthquake loading is taken into account. Antoniou and Pinho (2004) proposed a force-based adaptive pushover (FAP) algorithm in which the load vector is updated based on modal forces. They also offered an adaptive load vector scheme based on drift and displacement and verified through multi-ground motion incremental dynamic analyses (Antoniou and Pinho, 2004; Ferracuti et al., 2009).

Recently, a pushover method based on the developed story shear force in each level of structure is proposed by Shakeri et al. (2010), namely story shear-based adaptive pushover (SSAP) that uses the energy concepts to define the capacity curve of structure. In several cases, this method leads to underestimation of responses especially in lower stories. Therefore, the author (Shakeri et al., 2010) proposed that the maximum response of SSAP and the conventional pushover approach be selected to obtain better prediction of responses. The combined procedure was called “SSM1”.

To attain an adaptive approach that incorporates the effect of specific-site spectrums, contribution of higher modes of vibration, change in local resistances and structural modal characteristics due to cumulative damages, and predicting maximum seismic response of system by a reliable accuracy, the current paper will focus on EAPA where lateral load pattern is updated in each step based on concepts of energy.

The other feature of this load pattern can be noted as involving the effects of modal forces and displacements, simultaneously. Thus, this adaptive load pattern not only considers the instantaneous state of the system under deterioration in stiffness and strength, but also incorporates the movement of structure in updating the applied load pattern. In other words, the novelty of EAPA method is related to entering the concept of energy to define the incremental adaptive load pattern to achieve better prediction of responses. As a result, in this load pattern, the sign and amount of modal forces, as well as the story displacements are directly involved to define and update the load pattern. In addition, the effects of higher modes and stiffness deteriorations are directly reflected to update the load pattern.

For assessing the accuracy of EAPA with respect to NTHA, a parametric study on seismic response prediction of 3, 9, and 20-story steel moment resistant buildings which designed by consulting structural engineers for the Phase II of the SAC project is presented under a range of non-elastic responses. These models are exposed to a suitable collection of natural earthquakes with 10% probability of being exceed in 50 years. In addition, the performance of some common NSPs along with EAPA is evaluated to predict the peak inter-story drift, peak story shear, and peak floor overturning moment (OTM) profiles as the basic engineering design parameters.

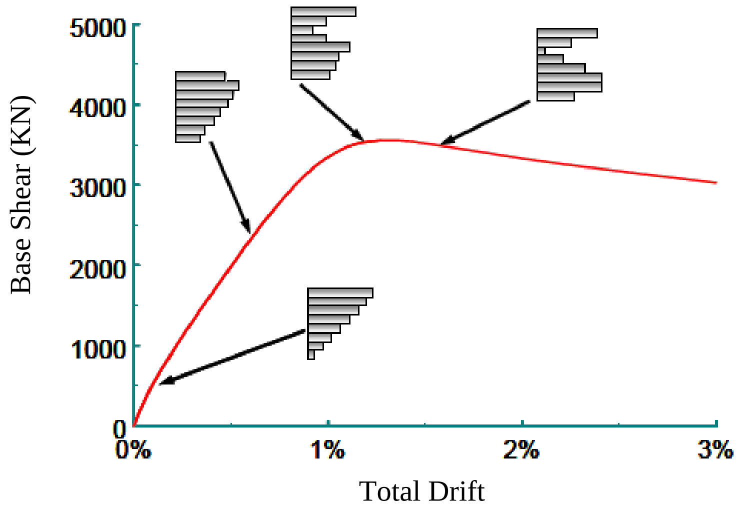


**Fig. 1.** The capacity curve obtained from an adaptive pushover analysis along with the applied load vectors that are updated during the analysis (Elnashai and Di Sarno, 2008)

## MATERIALS AND METHODS

To conduct the analysis, a mathematical model of the building is made up of all original laterally resistance members which is subjected to an incremental lateral force loading up to reaching a predetermined displacement value or structural collapse threshold. This algorithm can be summarized in four main steps as follows.

### The Initial Vector

The fundamental mode shape vector is used in the first step of analysis to determine the initial load distribution. This vector is then automatically updated with the progress of the analysis algorithm.

### Calculate the Scaled Load Vector

Scaled load vector, $\bar{E}$, determines the shape of increasing lateral load vector taking a real stiffness state of structure in each step of the analysis. For this aim, the eigenvalue analysis is performed in each step of the pushover analysis and the obtained results at the end of previous time step are implemented to determine the modal characteristics of the system. The results are used to determine the inter-story drifts, $\Delta_{ij}$, [given by Eq. (1)] and modal forces, $F_{ij}$, [given by Eq. (2)] in each floor corresponding to each mode (Antoniou and Pinho, 2004). Where $i$ indicates the floor level, and $j$ shows the number of considered modes. Spectral amplification factors are defined for adoption of a weighted story force and inter-story drift using the value of response spectrums corresponding to $j^{th}$ vibrational mode period, i.e. $S_{ij}$ and $S_{dj}$. It is thought that this idea leads to considerable improvement in prediction of both capacity curve and drift profile (Antoniou and Pinho, 2004a,b).

$$\Delta_{ij} = S_{dj} \Gamma_j \left( \phi_{i,j} - \phi_{i-1,j} \right) \quad (1)$$

$$F_{ij} = \Gamma_j \phi_{i,j} m_i Sa_j \quad (2)$$

where $S_{ij}$ and $S_{dj}$ are spectral acceleration and spectral displacement, respectively, corresponding to period of system in $j^{th}$ mode, $\phi_{i,j}$ is the component of mode shape for the $i^{th}$ story and the $j^{th}$ mode, and $m_i$ is the lumped mass in level $i$. $\Gamma_j$ is the modal participation factor of mode $j$ which is obtained from Eq. (3). In this equation, $[m]$ is the mass matrix of structure, and $\{\Phi_j\}$ is referred to the component of structural mode shape matrix corresponding to $j^{th}$ vibration mode.

$$\Gamma_j = \frac{\{\Phi_j\}^T .[m].\{1\}}{\left(\{\Phi_j\}^T .[m].\{\Phi_j\}\right)} \tag{3}$$

Then, the modal drifts and modal forces are combined with a suitable combination rule to achieve the inter-story drift and inertia force at each level by means of Eqs. (4) and (5), respectively. In these equations, $m$ is the number of considered vibration modes. Generally, estimated responses in any modal method are practically affected by assumed modal combination rule. However, it is assumed that frequencies of various vibration modes are enough far from each other to simply use SRSS combination rule. Therefore, SRSS combination rule is in the current paper used in all of implemented modal analyses.

Displacement value of $i^{th}$ level, $D_i$ is computed through summation of combined modal drifts from lower levels up to $i^{th}$ level [given by Eq. (6)] (Antoniou and Pinho, 2004). Consequently, the work done in level $i$, $E_i$ can be easily calculated from the product of the force by the corresponding displacement as denoted by Eq. (7) where $f_i$ is effective force in level $i$ defined by Eq. (8), and $\Delta D_i$ is incremental displacement in level $i$. In Eq. (8), $dF_i^{(t)}$ is the incremental applied force in the level $i$ at step t, and $F_i^{(t-1)}$ is the existing force in the level $i$ at the end of step $t$-1 of the analysis (Shakeri et al., 2010).

$$\Delta_i = \sqrt{\sum_{j=1}^{m} \Delta_{ij}^2} \tag{4}$$

$$F_i = \sqrt{\sum_{j=1}^{m} F_{ij}^2} \tag{5}$$

$$D_i = \sum_{k=1}^{i} \Delta_k \tag{6}$$

$$E_i = f_i .\Delta D_i \tag{7}$$

$$f_i^{(t)} = \left(F_i^{(t-1)} + \frac{1}{2} dF_i^{(t)}\right) \tag{8}$$

Now, scaled load vector, $\bar{E}$, can be defined via dividing the work done in each floor by the maximum of floor displacements as shown by following expression:

$$\overline{E_i} = E_i / \max D \tag{9}$$

### New Increment of Loads

New increment of loads, $\Delta V$, at each step is obtained from product of scaled load vector, $\bar{E}$, and incremental base shear, $\Delta V_b$, as introduced in Eq. (10).

$$\Delta \mathrm{V} = \Delta \mathrm{V}_b .\overline{E} \tag{10}$$

### Updating Force-Based Load Vector

When both of the scaled load vector, $\bar{E}$, and incremental base shear, $\Delta V_b$, are determined, the force vector, $V_t$, at each step of the analysis can be updated by the evolution pattern shown in Figure 2. Eq. (11) (Antoniou and Pinho, 2004) demonstrates the new load vector applied at step $t$, $V_t$, where $V_{t-1}$ is the previous load vector, $\Delta V_{b,t}$ is the current incremental base shear, and $\bar{E}_t$ is the current scaled load vector.

$$V_t = V_{t-1} + \Delta V_{b,t} \overline{E_t} \tag{11}$$

### EAPA in Sequence

The main stages of the nonlinear static procedure can be as follows:

- Stage 1: Perform a pushover analysis based on the concepts of energy.
- Stage 2: Convert the capacity curve of multi-degree of freedom (MDOF) system to an equivalent single degree of freedom (SDOF) system.
- Stage 3: Estimate the total displacement demand of the equivalent SDOF system and obtain the corresponding pushover analysis response.

Accordingly, the sequence of NSP based on EAPA is as follows:

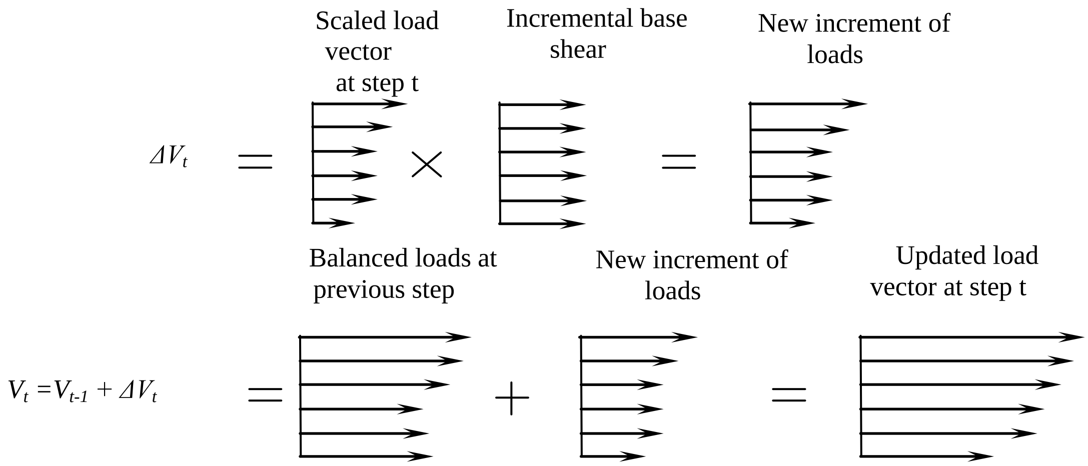


**Fig. 2.** Updating the load vector

***Stage 1***

1. Establishing the structural model in which the non-linear properties of the material have been considered.
2. Performing eigenvalue analysis in order to calculate the natural instantaneous frequencies, $\{\omega\}$, and the mode shapes, $[\Phi]$, of the system.
3. Calculating the modal drifts at each level for considered modes, $\Delta_{ij}$, via Eq. (1).
4. Calculating the modal forces at each level for considered modes, $F_{ij}$, using Eq. (2).
5. Combining the obtained modal drifts of step 3 using a suitable combination rule to define total drift, $\Delta_i$, [Eq. (4)] at each level.
6. Combining the obtained modal forces of step 4 to define the total force, $F_i$, [Eq. (5)] at each level.
7. Determining the displacement of the $i^{th}$ level, $D_i$, by summation of story drifts from base up to level $i$ as shown in Eq. (5).
8. Calculating the work done in each level by multiplying the modal force and the corresponding displacement given by Eq. (7).
9. Dividing the work done at each level, $E_i$, by the largest displacement of all floors, max$D$, at each step to obtain the scaled load vector, $\bar{E}$, using Eq. (9).
10. Calculating the new increment of loads using the predetermined incremental base shear, $\Delta V_b$, via Eq. (10).
11. Calculating the updated load scheme from Eq. (11) (as shown in Figure 2) and applying it to structural model.
12. Returning to step 2 and repeating the process until a predefined assumed control point value is achieved.

***Stage 2***

This stage is carried out based on force-deformation relationship ($F^*$-$D^*$ curve). For this aim, the assumed equivalent fundamental mode shape (EFMS) (Shakeri et al., 2010) is used to define an inelastic single degree of freedom (SDOF) system with unit mass. EFMS is defined in Eq. (12) as follows:

$$\{\phi\}^{(t)} = [m]^{-1} \times \{f\}^{(t)} \tag{12}$$

where, $\{\phi\}^{(t)}$ is the assumed EFMS at step $t$, $[m]^{-1}$ is the inverse of structure mass matrix, and $\{f\}^{(t)}$ is the vector of the total forces applied to the structure at step $t$ (Shakeri et al., 2010) from the database of the analysis.

Consequently, the roof displacement-base shear curve of the MDOF system (Chopra and Goel, 2002) is converted to the $F^*$-$D^*$ curve of the SDOF system based on the assumed EFMS. The vertical axis values of the curve, $F^*$, are calculated by Eq. (13) based on the assumed EFMS.

$$F^* = \frac{V_b}{M^*} \tag{13}$$

where, $V_b$ and $M^*$: are the base shear and effective mass of the MDOF system, respectively. As presented in Eq. (14) $M^*$ can be obtained using mass matrix, $[m]$, and the assumed EFMS of the structure, $\{\phi\}$ (Chopra and Goel, 2002; Shakeri et al., 2010).

$$M^* = \left(\{\phi\}^T .[m].\{1\}\right)^2 \Big/ \left(\{\phi\}^T .[m].\{\phi\}\right) \tag{14}$$

Also, the horizontal axis values of the curve, $D^*$, is defined by means of dividing the roof's displacement by the product of component of the $\phi$ in the roof level and mode participation factor, $\Gamma$, [given by Eq. (15)], at each step (Chopra and Goel, 2002; Shakeri et al., 2010).

$$D^* = \frac{u_r}{\Gamma \phi_r} \tag{15}$$

Hence:

13. Calculating the assumed equivalent fundamental mode shape using Eq. (12).
14. Creating the force-displacement curve ($F^*$-$D^*$ curve) of the equivalent inelastic SDOF system with unit mass, based on the values calculated via Eqs. (13) and (15); and then idealizing this curve in terms of an equivalent bilinear curve.

***Stage 3***

15. Determining the target displacement of SDOF system by a reliable method. In this study, the same procedure is used for all considered methods to test the validity of these methods. As such, after performing bilinear idealization of $F^*$-$D^*$ curve, the maximum displacement of the SDOF system is obtained directly from NTHA.
16. Converting the maximum displacement of the SDOF system to expected roof displacement of MDOF system by means of Eq. (16).

$$u_r = \Gamma \phi_r \tag{16}$$

17. Finding the corresponding step of pushover analysis in which roof displacement is equal to the expected roof displacement of MDOF system, and then extracting the seismic demands of the system such as inter-story drift, internal force of members, etc.

## DESCRIPTION OF MODELS

### Analysis

Among the methods which incorporate the concepts of energy in their works, the accuracy of the SSM1, EMPA, and EAPA to predict the maximum structural responses of SMRFs are evaluated by comparing with the NTHA as a benchmark. SSM1 and EMPA use the work done to define the capacity curve of the system, while the EAPA uses it to produce and update the required lateral load pattern. Considering the distribution of work done to establish the applied load pattern over the height of the structure, it may be possible to monitor the instantaneous state of the system under deterioration in stiffness and strength incorporating the movement of structure in updating the load pattern. Moreover, single-run FAP and conventional pushover approach which hereafter referred to as M1 methods are examined with respect to NTHA. In all multi-mode procedures, only the first three modes of vibration were considered.

The response profiles over the height of the structure along with their error values with respect to NTHA are demonstrated.

Also, a statistical approach is implemented using the mean, and mean +/- one standard deviation of the peak demands of system from NTHA, referred to as "NTHA +/- StD", along with the mean response of each NSP incorporating all of the earthquake excitations.

So as to investigate the accuracy of these pushover algorithms for predicting the peak responses of system under various ground motions in relatively wide range of multi-story buildings composed of SMRFs is the focus of this research, total error criterion, proposed by López-Menjivar (2004) was used (Eq. (17)).

$$\text{Total Error (\%)} = 100 \times \frac{1}{n}\sqrt{\sum_{i=1}^{n}\left(\frac{\Delta_{i-NSP}-\Delta_{i-NTHA}}{\Delta_{i-NTHA}}\right)^2} \quad (17)$$

Also, as it is preferred in ATC-76-6 (NIST, 2010), the error profiles of obtained responses of each NSP are computed by Eq. (18) and then plotted over the height of the structures.

$$\text{Error}_i = \frac{\Delta_{i-NSP}}{\Delta_{i-NTHA}} \quad (18)$$

In these equations, $n$ represents the number of floors, $\Delta_{i\text{-}NTHA}$ shows the maximum response of nonlinear time history analysis in level $i$, and $\Delta_{i\text{-}NSP}$ is the corresponding response obtained from nonlinear static procedure at that level.

### Structural Models

The structures analyzed in the current research consist of three steel moment resisting frames designed as a part of the FEMA-funded SAC joint venture project (Krawinkler, 2000) denoted as SAC-3, SAC-9, and SAC-20 buildings. These structures were designed and detailed in accordance with the design requirements in 1994 UBC (Krawinkler, 2000; Gupta and Krawinkler, 1999). These models may be representative of low-rise, mid-rise and high-rise SMRF buildings, respectively. For this study one frame of each building in N-S direction is considered to be analyzed and schematically is shown in Figure 3. More detailed information about the model buildings can be found in Krawinkler (2000).

### Ground Motions

A total of three frames, representative of 3-, 9-, and 20-story steel moment resistant frames under different analyses, were subjected to lateral excitations representing seismic hazard level of 10% probability of being exceed in 50 years for downtown Los Angeles. These excitations contain twenty numbers of well-known strong ground motions LA01 to LA20, based on the SAC steel project (Somerville and Venture, 1997).

## RESULTS AND DISCUSSIONS

To confirm the efficiency of the proposed method, nonlinear time history analyses were carried out through subjecting 20 strong ground motion loadings to these models. The mean and the range of mean +/- one standard deviation of each response have been calculated for NTHA and compared with the mean results of NSP analyses through these imposed excitations to the structures. Engineering design parameters of peak story drift, peak story shear, and peak floor overturning moment is selected to assess the efficiency of each NSP to predict seismic response of structures. Peak story drift is presented in terms of percent; as such drift response of each two adjacent floors is divided by the height of the corresponding story and shown in terms of percent. Also, the results of peak story shears are normalized by the total weight of the structure, $W$, and presented for each nonlinear analysis. Similarly, the responses of peak overturning moments are presented to evaluate each NSP, only with the difference that the results of overturning moment are normalized by the product of total weight, *W*, and total height, *H*, of the structure as preferred in ATC 76-6 (NIST, 2010). Moreover, the mean of maximum responses for considered parameters using each NSP over the mean of those extracted from the results of NTHA (Eq. (18)) are shown over the height of the structures.

**Three-Story SMRF**

Figure 4 illustrates the results of 3-story archetype. The left diagram of this figure shows the profile of mean peak inter-story drifts over the height of the structure; and the right diagram shows the mean error profile or in the other words, ratios of mean response predictions to mean results of NTHA for each NSP given by Eq. (18). Figure 4 indicates that EAPA algorithm has the smallest error compared with other NSPs. It is assumed that the inter-story drift error is largest for M1 method, while it is underestimated. As can be seen in the figure, maximum errors are related to M1 and EMPA methods. Since the capacity curve obtained through the second and third mode shape algorithms are large, corresponding target displacement of SDOF system is very small; therefore, EMPA coincides to the M1 results. FAP and SSM1 methods, as can be observed in Figure 4 and 5, did not give reliable responses in the lower stories. Although all considered pushover analyses offered the results in the range of “NTHA +/- StD”, however, it seems that response scattering of NTHA is high for this structure. As clear in Figure 4, SSM1 method seems to have utilized the M1 method to estimate the peak response of the system in the first and second floor. However, SSM1 method resulted in the underestimated results yet.

Figure 5 shows the predicted results of maximum shear developed in each story which were obtained by different pushover methods with respect to NTHA. The results represented that the estimated peak story shear of the EAPA is more efficient than other considered methods. So that, only the predicted maximum story shear profile of EAPA could stand in the range of “NTHA +/- StD” over the height of the structure. This behavior may raise the reliability of the method to estimate the local responses of the system. However, in all of the considered methods the accuracy of predicted response is decreased in upper floor. To estimate the peak story shear of SAC3, the profile of the response resulted from EAPA is closer to one resulting from NTHA than other method.

As shown in Figure 6, regarding the estimation of peak floor overturning moment of SAC3, all considered methods are in the proximity of responses for this parameter. However, only EAPA could stand in the range of “NTHA +/- StD” resulted from NTHA and other methods have been outside this range all along the height of the structure. The left graph of the figure represents the average results obtained by each NSP over the average response obtained by the NTHA method. The error on the upper floors, shown in the graph, is somewhat higher than the lower floor; this behavior is thought to be due to the small measure of OTM in the upper floor resulting in the sensitivity of the response to be considered as error criterion. In this case, also, EAPA indicates the closest profile to the response extracted from NTHA.

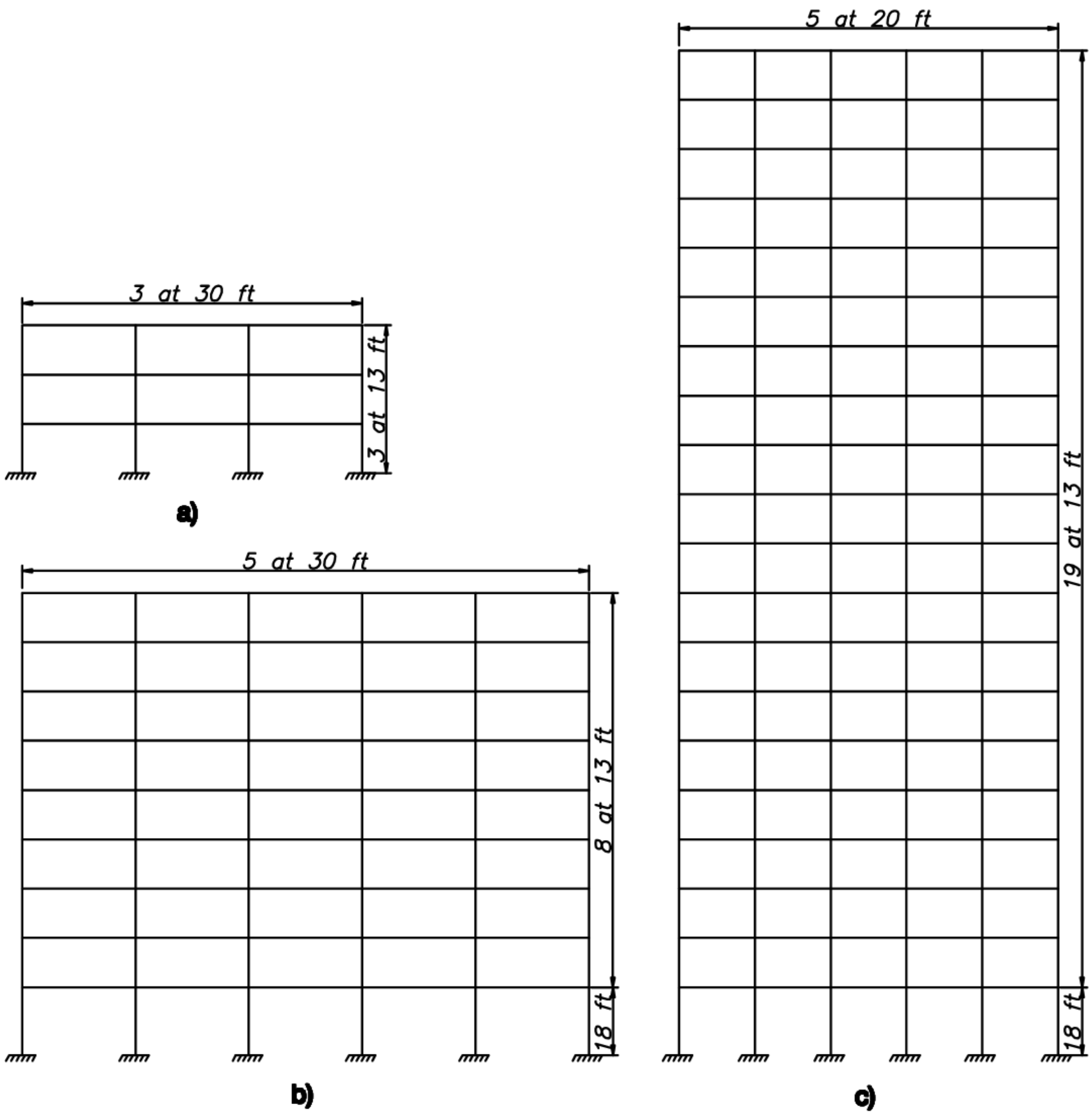


**Fig. 3.** Frame geometries of a) SAC3, b) SAC9, and c) SAC20 buildings

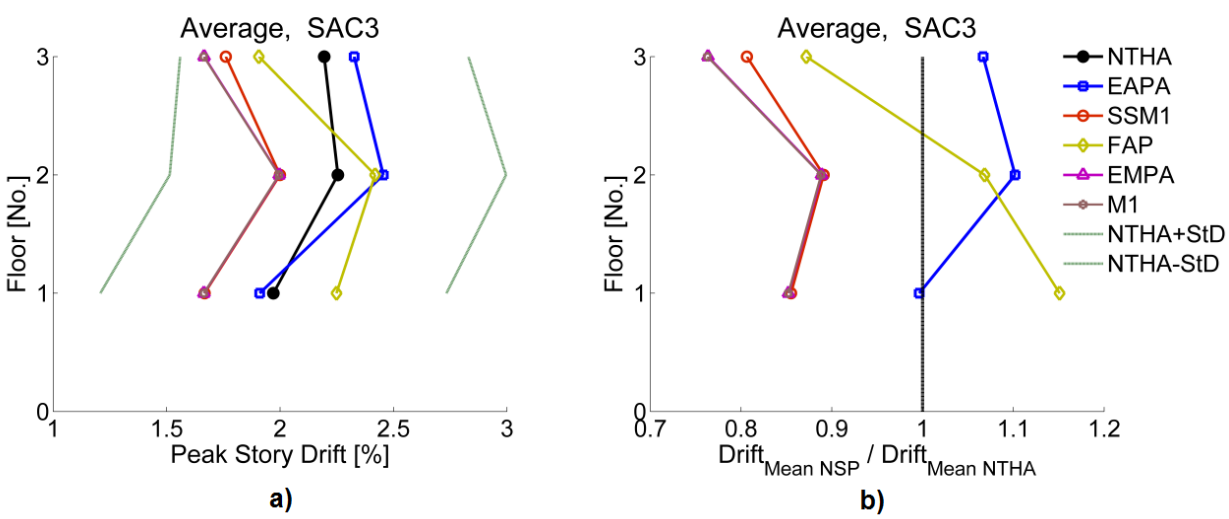


**Fig. 4.** a) The mean peak story drift profiles and b) the peak story drift error profiles for SAC3

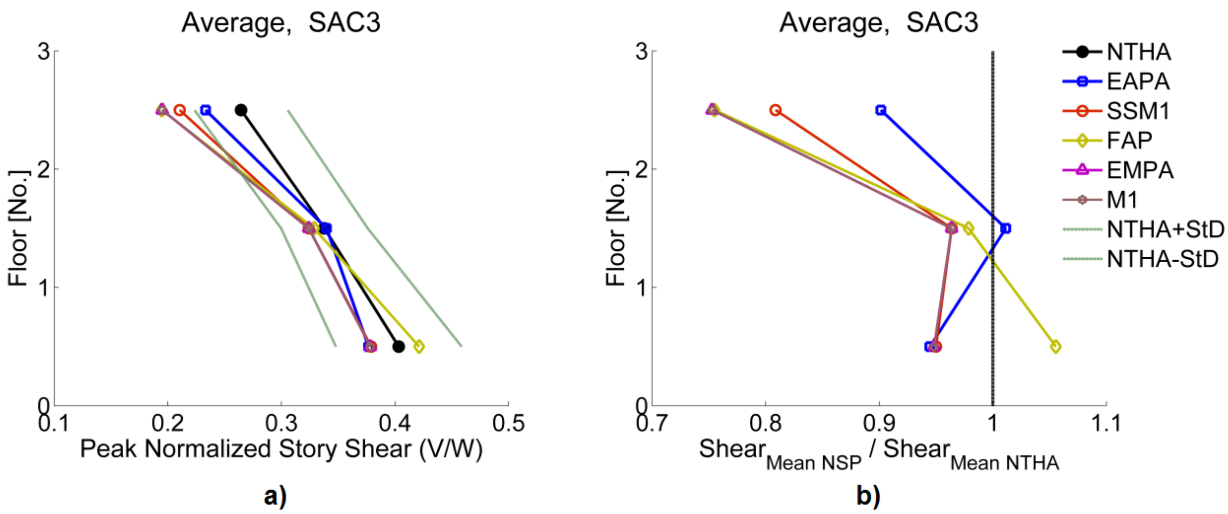


**Fig. 5.** a) The mean peak normalized story shear profiles and b) the mean peak story shear error profiles for SAC3

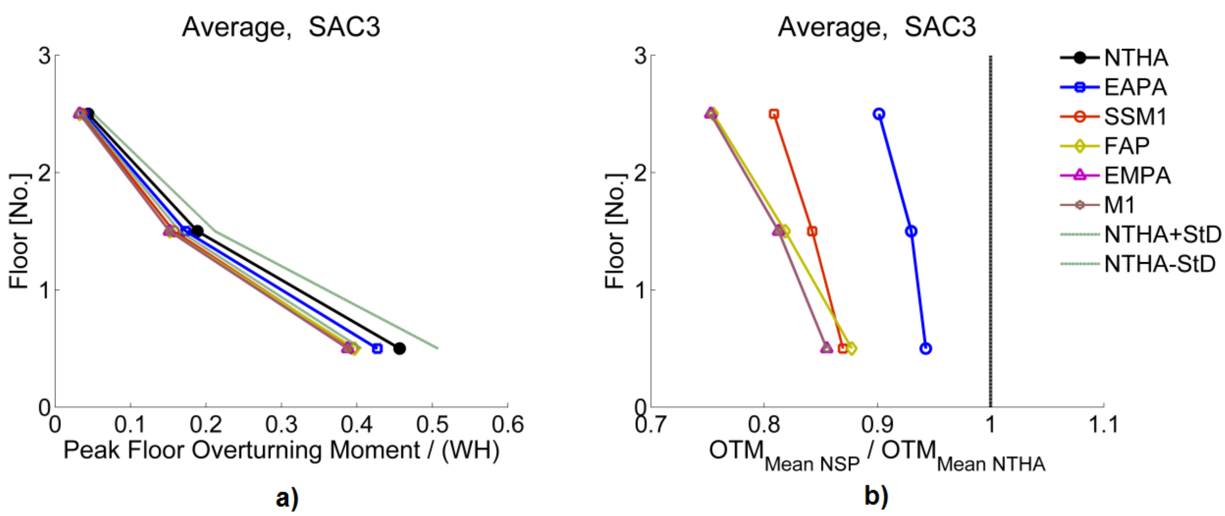


**Fig. 6.** a) The mean peak normalized floor overturning moment profiles and b) the mean peak floor overturning moment error profiles for SAC3

Figure 7 provides an overview on the conducted analyses by indicating the total story drift errors in response prediction of all considered earthquakes computed by means of Eq. (17) for SAC3 structural responses. Moreover, the means and the ranges of mean +/- one standard deviation (referred to as $\mu \pm \sigma$) of the total errors for all methods to predict all of the considered response parameters are illustrated. As such, error scattering of results is also presented in this figure partially. As it is evident, the majority of the total errors of drift fall within a standard deviation of about 4% for EAPA method. Furthermore, mean total inter-story drift error in the proposed method is calculated approximately 9%, having the least error.

It is possible to note from the figure that the mean of total errors associate with peak story shears is obtained about 6% by EAPA and have the lowest value among the considered methods. As evident, the value of the mean total error resulted from EAPA in estimation the response of the OTM parameter is less than other considered methods. Moreover, except for the total error obtained from the results of 3 earthquakes of 20 applied earthquakes, the errors of EAPA are less than the mean total error of M1, FAP, and EMPA methods.

**Nine-Story SMRF**

The Figures 8-11 show the results obtained from various considered nonlinear methods same as those were presented earlier related to SAC3. As previously presented in Figure 4, the validity of EAPA is confirmed in Figure 8 to predict inter-story drifts. In the figure, it is indicated that EAPA offer smaller mean total inter-story drift than other considered methods. For 14 excitations of 20 cases, EAPA have smaller total inter-story drift error than mean total error in predicting the results of 20 cases. As a result, in most cases the total errors can be shifted to lower error side. As Figure 8 shows, most considered methods provide non-conservative responses. The profile of response obtained from EAPA method approximately well predicts the expected response of NTHA throughout the height of the structure, except in stories 8 and 9 which are located outside of the range of “NTHA +/- StD” with relatively small difference. Predicted trend of the maximum story drifts by EAPA is acceptable. However, estimation of the drift response by this method in the upper floors is less accurate than lower floors.

Figure 9 shows the results of conducted analyses on SAC9 associated with the parameter of peak story shear. The results obtained from shear responses have presented the same behavior for estimation of drift responses. It also is indicated in the figure that EAPA provides closest prediction with respect to other methods.

As noticeable in Figure 10, EMPA method has presented the best answers for estimating the maximum floor OTM. So that EMPA is the only method that its predicted profile could stand in the range of “NTHA +/- StD”. Other methods indicate non-conservative results with relatively more errors. However, EMPA presents larger errors in the upper floors than lower floors; this behavior is partly related to the sensitivity of the error criterion in small response values (close to zero). Also the estimated profile by EAPA shows the relatively near responses to the results of NTHA.

Figure 11 indicates the total errors obtained by each method for the parameters of drift, story shear, and OTM, respectively, under all of the imposed earthquakes. As seen in these figures, the mean total error corresponding to each of three intended responses resulted from EAPA is less than those obtained from other methods. Also obtained scattering in estimation of the responses is relatively in acceptable ranges.

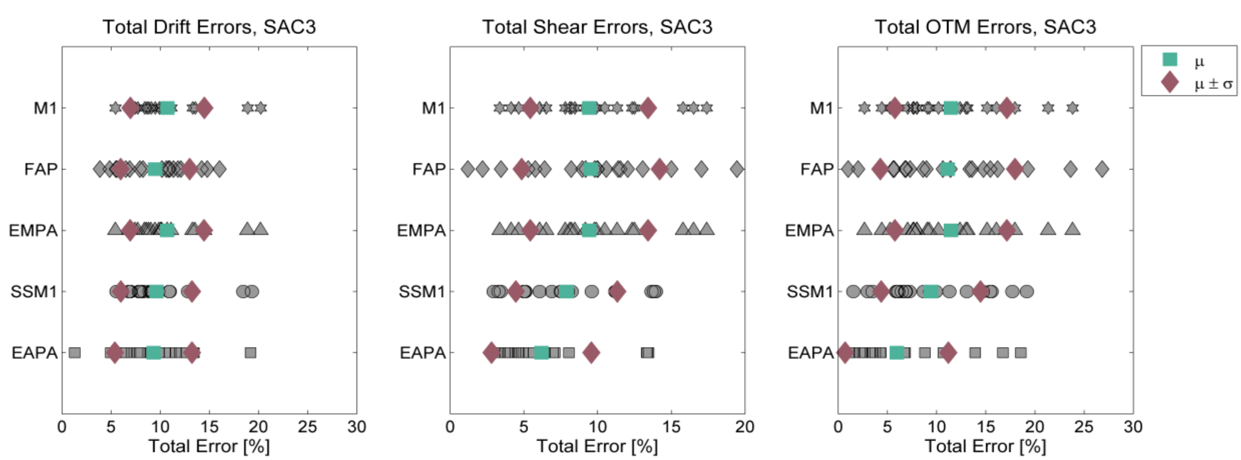


**Fig. 7.** Total errors for SAC3 in each NSP under all of the earthquake excitations along with the mean and mean +/- one standard deviation of them

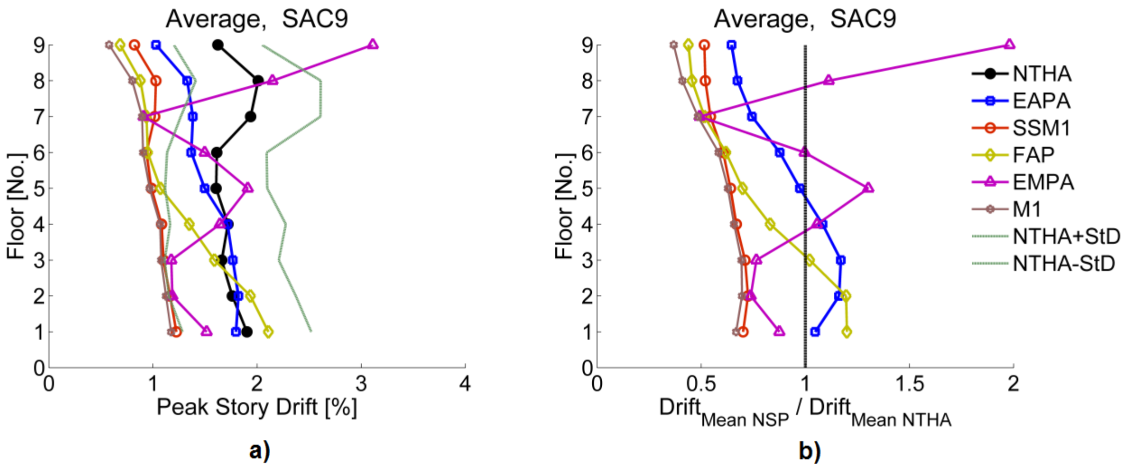


**Fig. 8.** a) The mean peak story drift profiles and b) the mean peak story drift error profiles for SAC9

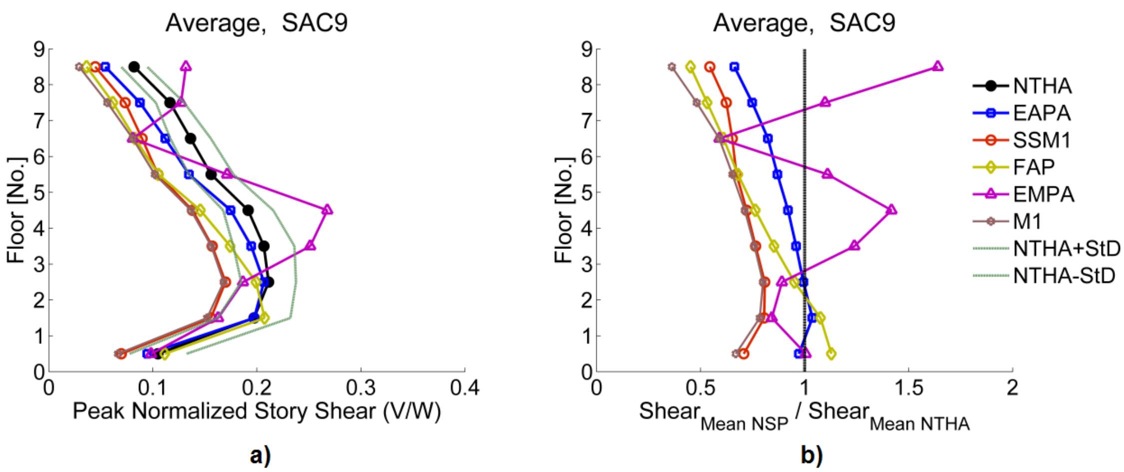


**Fig. 9.** a) The mean peak normalized story shear profiles and b) the peak story shear error profiles for SAC9

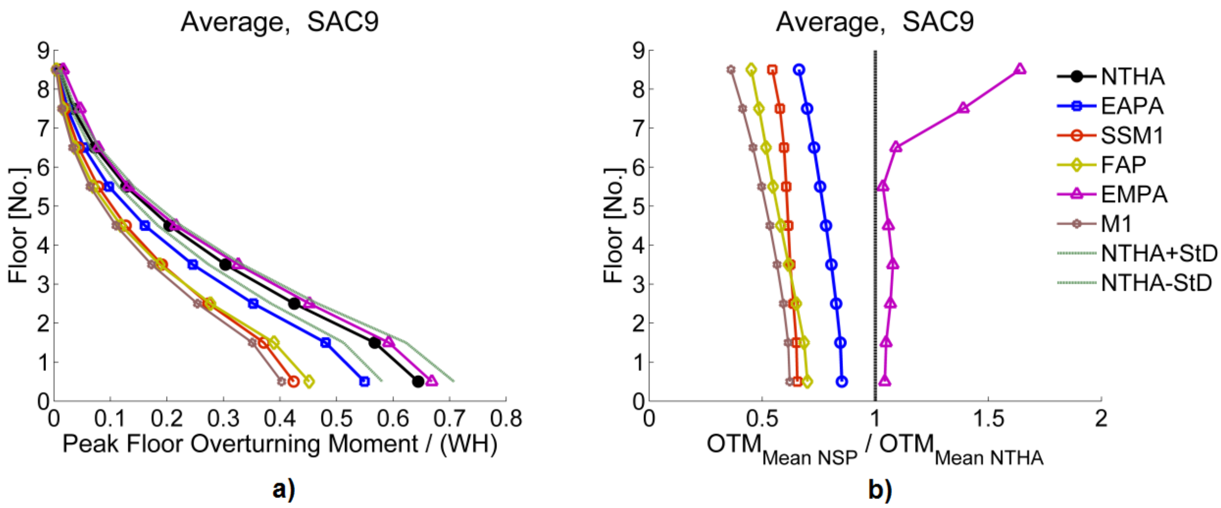


**Fig. 10.** a) The mean peak normalized floor overturning moment profiles and b) the peak floor overturning moment error profiles for SAC9

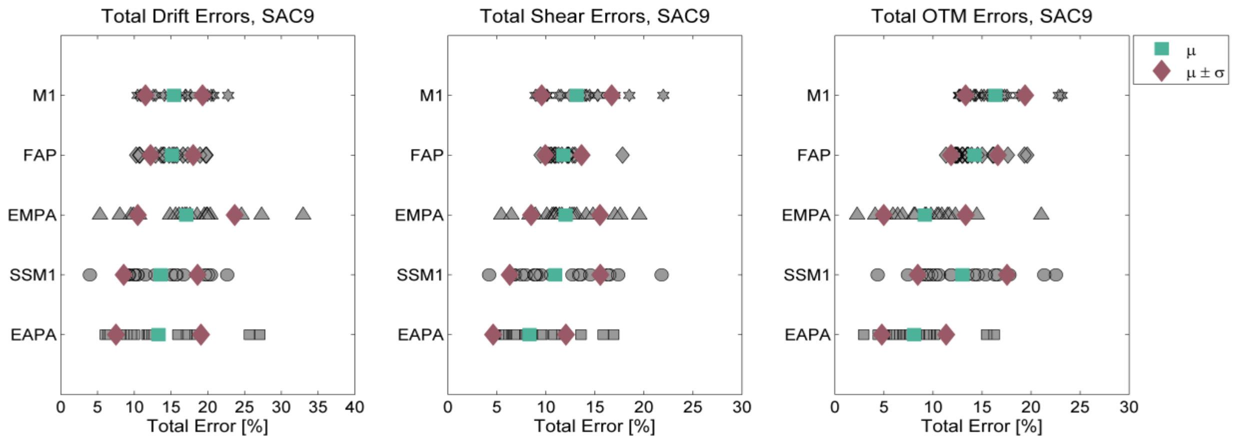


**Fig. 11.** Total errors for SAC9 in each NSP under all of the earthquake excitations along with the mean and mean +/- one standard deviation of them

**Twenty-Story SMRF**

Estimation of the responses of 20-story SMRF structure is shown in Figures 12-15. It may be understood from the figures that the behavior of different methods for estimating the maximum response of 20-story buildings somewhat differs from those were occurred in SAC3 and SAC9 structures which is discussed following on. Fig. 12 shows the maximum inter-story drifts resulted from different methods over the height of the structure. Also to evaluate the considered NSPs, the estimated result of each NSP over the response of NTHA is presented. As can be seen, FAP and EAPA methods give conservative estimations on the lower floors of the structure, but the prediction of response on upper floors are still non-conservative. SSM1 method provided non-conservative responses over the height of the structure. The shape of profiles predicted by intended NSPs for the drift response over the height of the structure is pretty good, except for EMPA.

EAPA is located within the range of “NTHA +/- StD” entire height of the structure except for the three top floors of the structure with an average partially difference of about 0.2%. Although the response of EAPA tent to differ from the NTHA response in upper floors of this structure (which may be resulted from the fact that the NSPs always suffer from their inherent static basis with the limitations and weaknesses), the obtained results of EAPA can be used to predict the maximum dynamic response of the structures with a good approximation.

Figure 13 shows the estimation of the maximum shear response of the structure which is normalized by weight of the structure. As the figure suggests, EAPA method offers nearest response to results of NTHA among the considered methods. The range of standard deviation for parameter of story shear is relatively small for this structure resulted from that the maximum shear responses of the structure are near in various earthquakes. However, predicted profile of EAPA could locate in this range from base to mid-height of the structure.

The results presented in Figure 14 are related to the estimation of maximum floor OTM in SAC20. As seen in the figure, it is compatible with the results of 9-story structure that the estimated response of floor OTM using EMPA method leads to high accuracy predictions. So with a small conservative difference, maximum expected floor OTM response in the 20-story structure is predicted. Also, EAPA provide the results with acceptable accuracy.

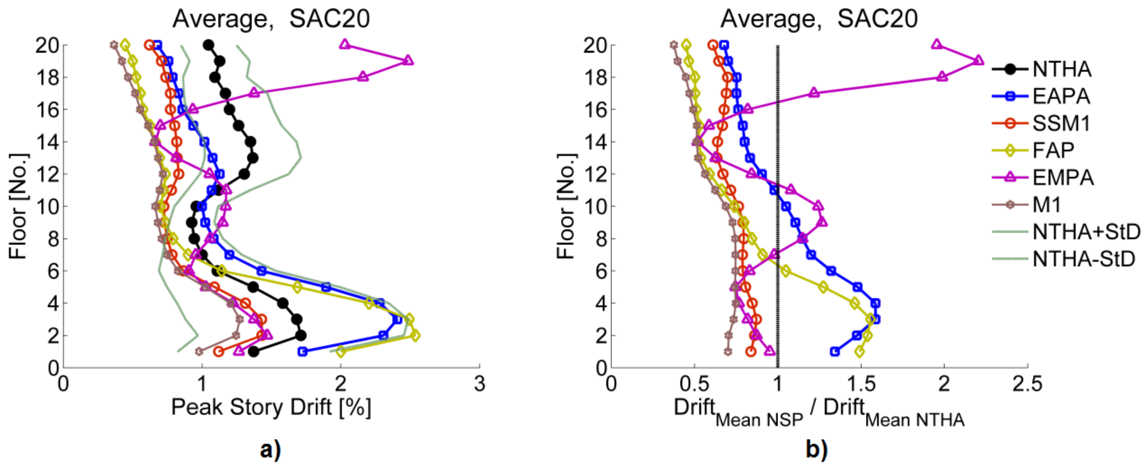


**Fig. 12.** a) The mean peak story drift profiles and b) the mean peak story drift error profiles for SAC20

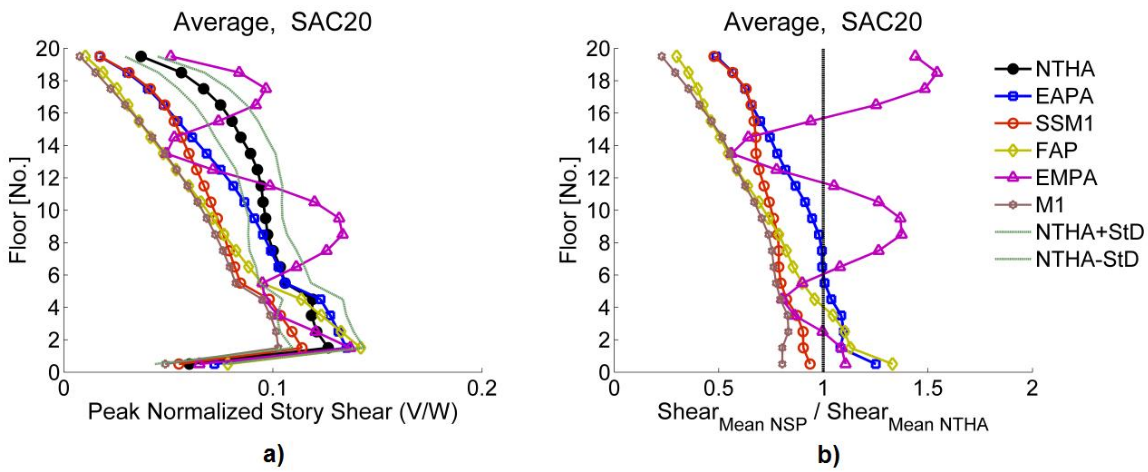


**Fig. 13.** a) The mean peak normalized story shear profiles and b) the mean peak story shear error profiles for SAC20

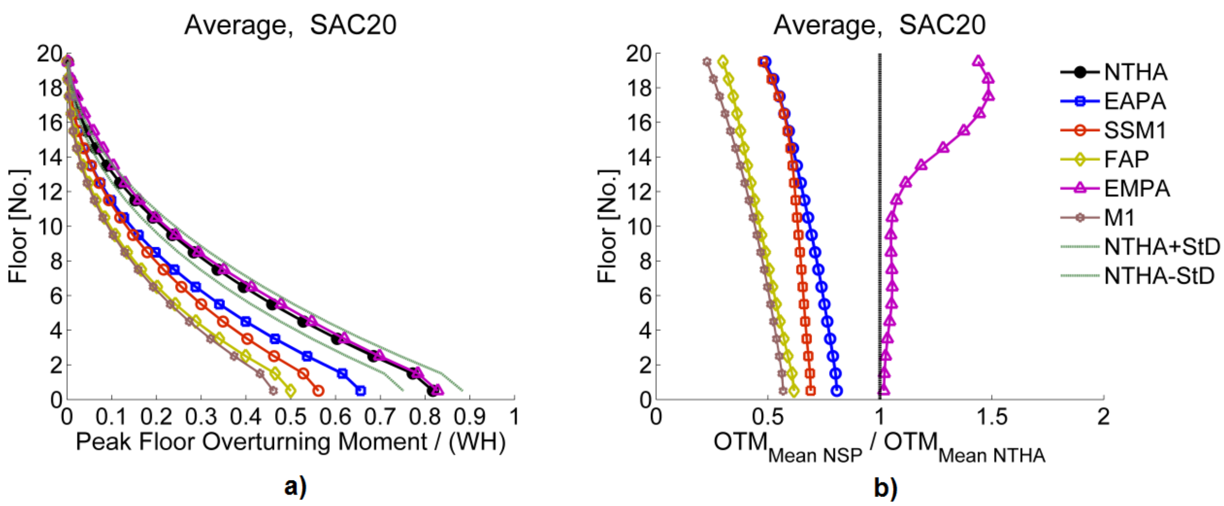


**Fig. 14.** a) The mean peak normalized floor overturning moment profiles and b) the mean peak floor overturning moment error profiles for SAC20

Figure 15 shows the total errors associated with the results obtained from each method for each of the earthquakes. It is noticeable that SSM1 method is able to offer better responses than other intended methods for predicting the peak story drifts in SAC20. Also, in most cases EAPA provides good results with the total errors less than about 9% (16 from 20 imposed ground motions), however, the results of four earthquakes have more total error in prediction of drift responses that increase the scattering of the responses and subsequent mean total error. As it is obvious, EAPA, SSM1, EMPA, FAP, and M1 have in an ascending order the lowest average error in estimation of peak story shear. EAPA has provided relatively accurate responses for estimating the peak story shear so that its worst total error is less than the average total error of FAP and M1 procedures. Regarding to the floor OTM, although the scattering of EMPA results is more than other methods; however, in most cases its error is less than others; so that its average total error is less than that obtained by other considered methods. Except the EMPA, EAPA, SSM1, FAP, and M1methods have the lowest average error respectively.

In order to understand how SSM1 and EMPA differ from the M1 method, the responses of the M1 is displayed alongside those obtained by SSM1 and EMPA. As a result, if the M1 diagram attaches the diagram of SSM1, it may be concluded that SSAP method is underestimated and the results of M1 method has been used.

Figure 16 provides an overview on the results of considered methods to predict the maximum responses of inter-story drift, story shear, and floor OTM. In this figure the total error, computed by means of the Eq. (17), in estimating the structural response for each parameter is shown as a bar. As it is suggested, EAPA indicates the lowest average error for all three considered parameters under these earthquakes for 3-story, and 9-story SMRF structures. Therefore, the validity of this approach for evaluating these types of the structures may be confirmed. In the case of 20-story structure, the least value of mean total error drift is resulted from the SSM1 method. EAPA method provides the least amount of mean total error in estimation of peak story shear and EMPA method predictions provide the responses of floor OTM with lowest mean total error. In general, EAPA method provides good seismic responses, but some shortcomings are evident in these profiles. This problem is relatively attributed to consideration of limited number of higher modes, the fact that all NSPs suffer from their inherently static concepts, and also partially is related to the features of ground motion such as PGA and frequency content.

As mentioned earlier, about EAPA method with updating the load pattern in terms of work done, the developed modal forces and floor displacements are directly involved in incremental load vector at each load step. Therefore, the sensitivity of the system to the force or displacement is reduced by considering both of them to update the applied load pattern. With this capability, considering the effects of acceleration and displacement response spectrum of a specific site is possible simultaneously; and seems that it leads to a better prediction of seismic response.

As it seems that the behavior of three-story structure can be dominated by the first mode vibration of the structure and this structure be categorized in class of short structures, it is expected that M1 method offer acceptable prediction of responses. However, the estimations of maximum responses of the structure using M1 is not satisfactory, so that the maximum error values for the three considered parameters of seismic responses (inter-story drift, story shear and floor OTM) are obtained from M1.

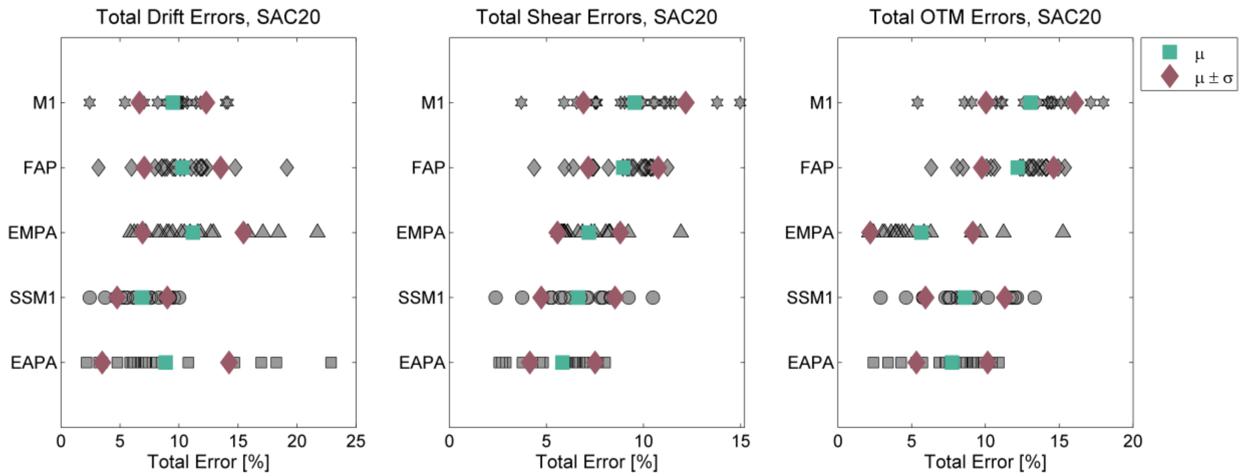


**Fig. 15.** Total errors for SAC20 in each NSP under all of the earthquake excitations along with the mean and mean +/- one standard deviation of them

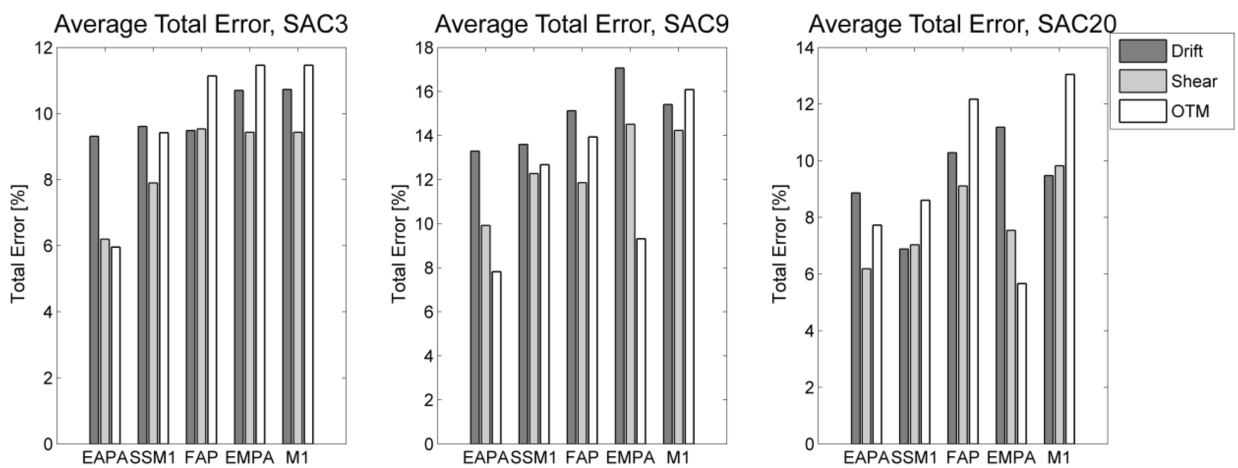


**Fig. 16.** Mean of total errors from each NSP method to predict the considered demand parameters

Obtained results from EMPA technique in short structures seems to be near the M1 method. This behavior is rather attributed to the definition of the capacity curve of the structure. With increase in height of the structure, as is apparent in the results of SAC9, the influence of higher modes is important; therefore, maximum structural response profiles estimated by EMPA differ from the results of M1. Response profiles obtained using EMPA associated with inter-story drift and story shear have not good agreement with the expected response of NTHA over the height of the structure. However, this method provides accurate estimations on the maximum floor OTM. With respect to obtained results of 3-, 9-, and 20-story structures, it is found that the behavior of this method is improved by increasing the height of the structure.

FAP is an adaptive force base pushover analysis which solves several drawbacks of conventional pushover methods such as site effects, progressive of stiffness and strength deterioration. However, this method could not provide reliable response with respect to NTHA since the results of the methods is near to M1 in many cases. This behavior may be attributed to this fact that modal forces corresponding to each mode are combined with the SRSS method for updating the lateral load pattern and not taking the sign of forces into account. This method resulted in non-conservative responses in many cases, especially in upper levels of the structures.

SSM1 method consists of two methods of M1 and SSAP whereby maximum response of these methods is considered as the response of SSM1. The decision is mainly adopted for underestimated response of SSAP in lower stories of the structure. However, SSM1 in many cases of conducted analysis presents underestimated responses in prediction of considered engineering design parameters yet.

Estimated profiles of maximum responses using EAPA will be in the range of "NTHA +/- StD" in most cases. This method estimates the seismic response of the structure well in the short to medium structures. As a result, the profiles of maximum responses of SAC3 and SAC9 are close to the expected responses of NTHA and give least value of total error in most cases. Also with increase in the height of the structure, EAPA could maintain its efficiency and provide response profiles of story drift, story shear, and floor OTM with acceptable accuracy, especially for the estimation of shear and OTM parameters.

## CONCLUSIONS

A new energy-based adaptive pushover analysis was proposed and examined for low-, mid-, and high-rise buildings through a series of nonlinear analyses under strong ground motions. The steel moment resisting frame structural models have been investigated in this study including SAC-3, SAC-9, and SAC-20 models.

The proposed method namely EAPA used the concepts of work done in its definition and updating the lateral load pattern. The key parameters that used to investigate the seismic responses of the different models are peak inter-story drift, peak story shear, and peak overturning moment profiles.

The following points summarize the key response observations:

As noted above, EAPA suggests the least error among all of other considered methods in low- to mid-raise structures; moreover, while the height of structure is increased, EAPA keeps the good performance and gives the acceptable result, especially for story shear profiles. As a result, this method reduced the error of predictions in maximum seismic responses and the shape of response profiles as well.

Definition of load pattern based on energy concepts can be more efficient than using this concept for establishing the capacity curve of the system, as it can be understood with comparing the results of SSM1 and EAPA.

It should be noted that these results are obtained for SMRF systems in the hazard level of 10% probability of being exceed in 50 years, and not include a widely research with many structure, proportioning, detailing, etc. Thus, supplemental studies are needed to assess the efficiency of the proposed method in various structures as a reliable replacement for NTHA to evaluate the global and local responses of a system.

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