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Evaluation of a Modified-SSAP in estimating seismic demands of a torsionaly flexible building

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Abstract. *Storey Shear-Based Adaptive Pushover (SSAP) procedure has originally been developed for estimating seismic demands of planar frames. An extension and modification of the SSAP, has recently been proposed to estimate seismic demands of asymmetric buildings, represented by a 3D structural model. In order to investigate the capability of the proposed Modified-SSAP in severe mass asymmetric buildings, in the current paper, the method is evaluated for a 12-story asymmetric reinforced concrete moment resisting building with a torsionaly dominant fundamental natural mode, representing torsionaly flexible buildings. Results of the storey drift predictions of the Modified-SSAP are compared with Nonlinear Response History Analysis (NRHA) as well as estimations of the well-known Modal Pushover Analysis (MPA) procedure, subjected to a set of seven near-field ground motions. Alternative correction factors for the target displacement to improve the results are also examined. An acceptable compatibility of storey drift estimations of pushover procedures is observed, while Modified-SSAP is more compatible with the results of rigorous NRHA when proper correction factors for the target displacement are applied.*

1 INTRODUCTION

Conventional pushover analysis procedures are based on the assumption that the structure predominantly vibrates in a single mode. This assumption is not straightforward particularly for tall and irregular buildings. In recent years many attempts has been made worldwide to take the higher modes effects and the changes of modal characteristics into consideration while structure responds in inelastic range. Many of the proposed methods enjoy from strong theoretical background and acceptable degree of accuracy. However, they have merely been introduced to estimate seismic demands of planar frames. In this respect, development of the nonlinear static methods, originally for planar frames, so that are able to analysis of irregular building represented by 3D structural model has been considered. Backed by an extensive range of studies N2 method [1] has been extended for asymmetric buildings based on the assumption that torional effects in the elastic and inelastic domains are very close and mostly overestimated by an elastic analysis [2]. The well-known modal pushover analysis [3] has also been developed for asymmetric building [4] by extending the formulations in accordance with dynamics of structures theories. An alternative of conventional capacity spectrum method in an adaptive framework [5] has also been proposed another indication of development of conventional methods.

Storey Shear-Based Adaptive Pushover (SSAP) originally has been proposed for planar frames taking into account reversal signs of applied load pattern as well as changes in modal attribute within inelastic range [6]. In this paper an extension of the SSAP introduced first in [7] has been summarized. The extension has been done through extending the formulations based on structural dynamics theory. Modifications for load pattern and target displacement have also been proposed [7] which are currently in their premature stages of development. In the current study, a torsionaly flexible 12 storey building subjected to uni-directional excitation is validated by extended SSAP. Proposed Modifications of load pattern and target displacement are further evaluated for the example building when standard design spectrum is utilized in the algorithm of the method. Therefore the accuracy of the method is investigated by using standard design spectrum instead of individual spectrum.

2 SUMMARY OF THE EXTENDED SSAP METHOD

The overall trend of the procedure is similar to its original for planar frames developed by Shakeri et al. [6], except the fact that formulations have been converted to three dimensional based on the concepts of structural dynamic. Extended story shear-based adaptive pushover analysis to asymmetric plan buildings under uni-directional excitation can be summarized in the following steps:

Step 1: Create a 3D structural model incorporating nonlinear material characteristics.
Step 2: Perform a nonlinear static analysis to consider gravity loads on the structure.
Step 3: Compute instantaneous natural frequencies, ω_n and mode shapes, $\emptyset_n$ by performing an eigenvalue analysis using the current stiffness characteristics of the structure.
Step 4: For a selected number of modes considered, compute modal story forces and torques associated with the global x,y, and z directions using the following equation:

$$\boldsymbol{f}_{nx} = \Gamma_n \boldsymbol{\phi}_{nx} \boldsymbol{m} A_n \qquad \boldsymbol{f}_{ny} = \Gamma_n \boldsymbol{\phi}_{ny} \boldsymbol{m} A_n \qquad \boldsymbol{f}_{n\theta} = \Gamma_n \boldsymbol{\phi}_{n\theta} \boldsymbol{I}_o A_n \tag{1}$$

Where $\boldsymbol{f}_{nx}$and $\boldsymbol{f}_{ny}$ are the vectors of modal story forces, $\boldsymbol{f}_{n\theta}$ is the vector of torsional moments at each floor diaphragm. $\boldsymbol{\phi}_{nx}$, $\boldsymbol{\phi}_{ny}$, and $\boldsymbol{\phi}_{in\theta}$ are three sub vectors of the n-th instantaneous translational and torsional natural mode shapes associated with x, y, and z directions, respectively. A_n is the ordinate $A(T_n, \xi_n)$ of the earthquake pseudo-acceleration response spectrum for the n-th mode single degree of freedom system. $\boldsymbol{m}$ is the diagonal sub matrix of the global mass matrix, $\boldsymbol{M}$, with $m_{ii} = m_i$, the lumped mass at the i-th floor diaphragm:

$$\boldsymbol{M} = \begin{Bmatrix} \boldsymbol{m} \\ \boldsymbol{m} \\ \boldsymbol{I}_o \end{Bmatrix} \tag{2}$$

$\boldsymbol{I}_o$ is a diagonal matrix with $I_{ii} = I_{oi}$, the polar moment of inertia of the i-th floor diaphragm about a vertical axis through the center of mass. Γ_n is the modal participation factor obtained as the following:

$$\Gamma_n = \frac{L_n}{M_n} \qquad M_n = \boldsymbol{\phi}_n^T \mathbf{M} \boldsymbol{\phi}_n \qquad L_n = \begin{cases} \boldsymbol{\phi}_{xn}^T \mathbf{m1} & for\ \ddot{u}_{gx}(t) \\ \boldsymbol{\phi}_{yn}^T \mathbf{m1} & for\ \ddot{u}_{gy}(t) \end{cases} \tag{3}$$

Step 5: Calculate modal story shears of x and y directions and summation of torsional moments of the considered story and all upper storeys:

$$SS_{inx} = \sum_{k=i}^{N} f_{knx} \qquad SS_{iny} = \sum_{k=i}^{N} f_{kny} \qquad SM_{in\theta} = \sum_{k=i}^{N} f_{kn\theta} \tag{4}$$

Step 6: Combine the story shear forces and torsions obtained by Eq. (4) using modal combination rules such as SRSS and CQC to define forces and torsional moment of each floor diaphragm. In this study the SSRS rule has been used because of its simplicity as the following:

$$SS_{ix} = \sqrt{\sum_{n=1}^{m} {SS_{inx}}^2} \qquad SS_{iy} = \sqrt{\sum_{n=1}^{m} {SS_{iny}}^2} \qquad SM_{i\theta} = \sqrt{\sum_{n=1}^{m} {SM_{in\theta}}^2} \tag{5}$$

Step 7: Define the amount and sign of the incremental load pattern, including forces in translational directions and torsional moments about a vertical axis through the center of mass, by subtracting the consecutive story shears and moments:

$$\begin{aligned} &F_{ix} = SS_{ix} - SS_{(i+1)x} \quad F_{iy} = SS_{iy} - SS_{(i+1)y} \quad F_{i\theta} = SM_{i\theta} - SM_{(i+1)\theta} \qquad i = 1,2, \dots, (i-1) \\ &F_{Nx} = SS_{Nx} \qquad F_{Ny} = SS_{Ny} \qquad F_{N\theta} = SM_{N\theta} \qquad i = N \end{aligned} \tag{6}$$

Step 8: Use any appropriate quantity to normalize the lateral load patterns. As an alternative lager value of the base shear associated with x and y directions can be taken for normalizing the lateral loads pattern using the following equations:

$$V = \max\left(\sum F_{ix}\ , \sum F_{iy}\right) \tag{7}$$

$$\bar{F}_{ix} = \frac{F_{ix}}{V} \qquad \bar{F}_{iy} = \frac{F_{iy}}{V} \qquad \bar{F}_{i\theta} = \frac{F_{i\theta}}{V} \tag{8}$$

Step 9: Consider an incremental base shear and multiply it to the normalized lateral loads pattern to obtain forces and torques which are applied in each step to each floor diaphragm:

$$\Delta F_{ix} = \Delta V_b \times \bar{F}_{ix} \qquad \Delta F_{iy} = \Delta V_b \times \bar{F}_{iy} \qquad \Delta F_{i\theta} = \Delta V_b \times \bar{F}_{i\theta} \tag{9}$$

Where ΔV_b is the incremental base shear and ΔF_{ix} , ΔF_{iy} , $\Delta F_{i\theta}$ are the i-th components of the incremental applied loads at each step.
Step 10: Apply the scaled incremental load profile (Eq. 9) to the structural model considering $P - \Delta$ effects; compute displacements, inter-story drifts, element forces, etc.
Step 11: compute the assumed fundamental mode shape vector for step k employing the following equation:

$$\emptyset^{\mathrm{k}} = \mathbf{M^{-1}}\boldsymbol{F}^k \tag{10}$$

Where k denotes the step number, $\emptyset^{\mathrm{k}}$ is the assumed mode shape at step k, $\mathbf{M^{-1}}$ is the inverse of the global mass matrix, and $\boldsymbol{F}^k$ is vector of total forces applied up to the step k, consisting of three sub vectors, as it is shown by Eq. 11.

$$\boldsymbol{F^k} = \begin{Bmatrix} \boldsymbol{F}_x^k \\ \boldsymbol{F}_y^k \\ \boldsymbol{F}_{\boldsymbol{\theta}}^k \end{Bmatrix} \tag{11}$$

Step 12: Based on the assumed fundamental mode shape (Eq. 10), convert base shear of the Multi Degree Of Freedom (MDOF) system to the equivalent force of the Single Degree Of Freedom (SDOF) system using the following equation:

$$F^* = S_a = \frac{V_b}{M^*} \tag{12}$$

Where F^* is the equivalent force of the SDOF system, V_b is the base shear of the applied forces in the direction of excitation, and M^* is the effective modal mass considering the mode shape acquired by Eq. (10).

Step 13: Calculate the equivalent displacement of the SDOF system by means of an energy concept using Eqs. (13) to (15).

$$\sum_{i=1}^{N}\left(\left(F_i^{(k-1)} + \frac{1}{2}dF_i^{(k)}\right) \times \Delta d_i^k\right) = \left(\sum_{i=1}^{N}\left(F_i^{(k-1)} + \frac{1}{2}dF_i^{(k)}\right)\right) \times \Delta D^k \tag{13}$$

$$\Delta D^{(k)} = \sum_{i=1}^{N}\left(\left(F_i^{(k-1)} + \frac{1}{2}dF_i^{(k)}\right) \times \Delta d_i^k\right) / \left(\sum_{i=1}^{N}\left(F_i^{(k-1)} + \frac{1}{2}dF_i^{(k)}\right)\right) \tag{14}$$

$$D^{(k)} = D^{(k-1)} + \Delta D^k \tag{15}$$

where $F_i^{(k-1)}$: the existing story force in the direction of excitation at the end of step $k - 1$, $dF_i^{(k)}$: incremental applied force in the story i at step k, Δd_i^k: incremental displacement of the story i in the direction of the excitation due to increased applied load at step k, ΔD^k: incremental displacement of the equivalent SDOF system at step k, $D^{(k)}$ displacement of the equivalent SDOF system at step k.

Step 14: Go back to step 3 and repeat the process until an extreme value of base shear is achieved or the structure fails.
Step 15: Develop the Force-Displacement curve, ($F^* - D$ curve) of equivalent inelastic SDOF system with unit mass based on the computed values in steps 12 and 13 in the previous cycles and idealize it as a bilinear curve.
Step 16: compute the peak inelastic displacement of the equivalent SDOF system by performing a nonlinear time history analysis to obtain the target displacement. Alternatively, target displacement can be computed by plotting the capacity curve (i.e. $F^* - D$ curve) against the inelastic acceleration-displacement response spectra.
Step 17: Determine the corresponding step to the target displacement in the pushover procedure and obtain the interested pseudo seismic demands.

3 DESCRIPTION OF PARAMETRIC STUDIES

3.1 Structural system

A twelve-storey moment resisting concrete frame has been selected with a structural plan as shown in Fig. 1. A distance equal to 10% of the plan dimension has been considered between the centre of mass (CM) and centre of stiffness (CS). Since the first natural vibration mode of the structure is dominantly torsional the building is considered as a torsionaly flexible structure. Fig. 2 demonstrates the natural vibration modes of the structure. The building has been loaded as majority of common residential buildings are loaded in Iran. However, polar moment of inertia has been multiplied to represent a torsinaly flexible structure. Nonlinear behaviour of the structure occurs at discrete hinges at both ends of beam and column elements which were defined in accordance with provisions of FEMA356 [8]. First 10 natural vibration modes of the structure have been considered to implement the SSAP method.

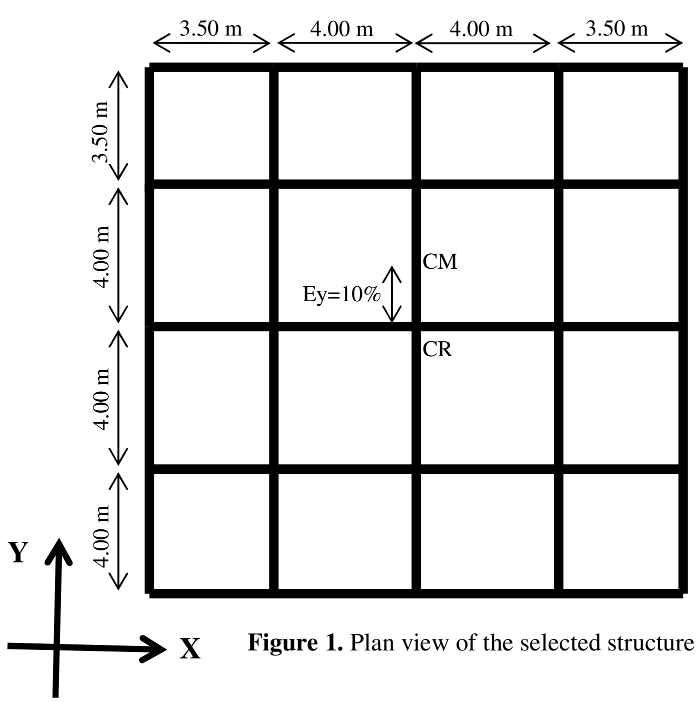

Figure 1. Plan view of the selected structure

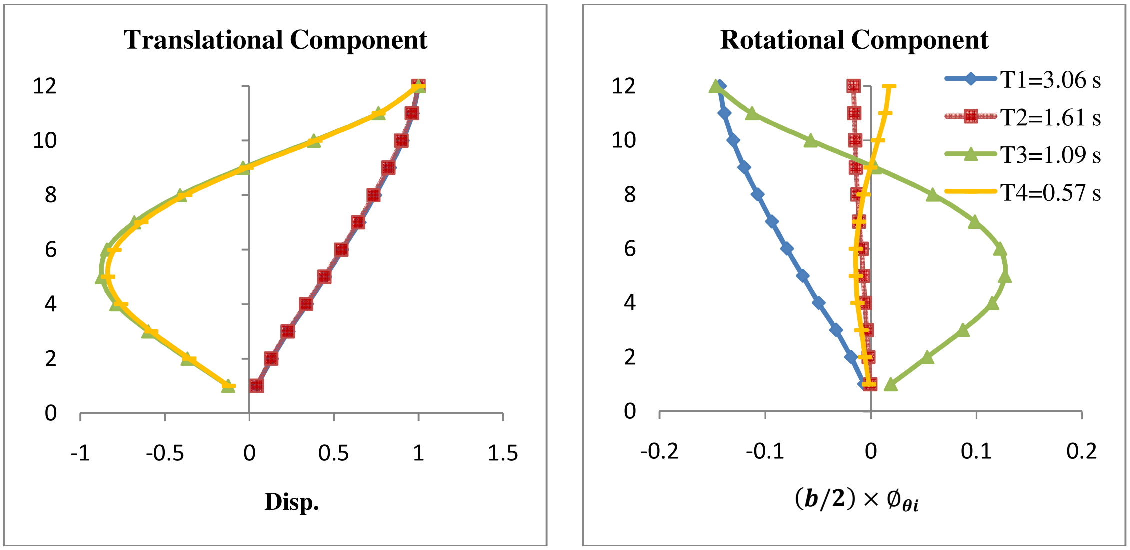

Figure 2. Natural periods and modes of vibration of the selected torsionaly flexible structure

3.2. Ground motions

The building is subjected to a set of seven near-field seismic actions in the X-direction where torsion occurs throughout the analysis. Table 1 depicts the ensemble of selected motions. In addition, Fig.3 depicts pseudo-acceleration response spectrum of the records along with the design response spectrum of FEMA356 [8] for site class D in a high seismic region having short period response spectral acceleration (S_s) equal to 2.05g and spectral acceleration response at a period of 1 (S_1) equal to 0.82g. In order to ensure that the building responds within the inelastic range, records have been scaled such that the acceleration response spectrum of individual records are identical to pseudo acceleration response spectrum of FEMA356 at the structural fundamental period.

Table-1. List of selected ground motions

Eq#	Earthquake	Magnitude (Mw)	Station name	Distance (km)	PGA (g)
1	Cape Mendocino 1992/04/25	7.1	89156 Petrolia	9.5	0.662
2	Coyote Lake 1979/08/06	5.7	47380 Gilroy Array #2	7.5	0.339
3	Erzincan, Turkey 1992/03/13	6.9	95 Erzincan	2.0	0.515
4	Northridge 1994/01/17 12:31	6.7	24279 Newhall - Fire Sta	7.1	0.59
5	Kobe 1995/01/16 20:46	6.9	0 Nishi-Akashi	11.1	0.509
6	N. Palm Springs 1986/07/08 09:20	6.0	5071 Morongo Valley	10.1	0.218
7	Gazli, USSR 1976/05/1	6.8	9201 Karakyr	Hypocentral (3.0)	0.718

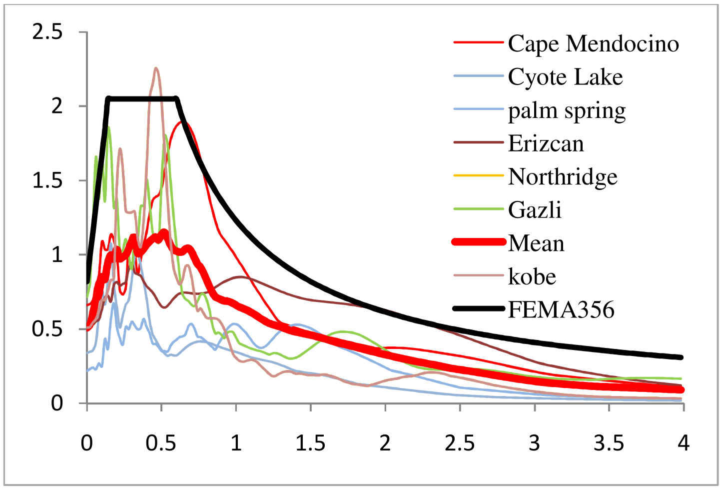

Figure 3. Pseudo-acceleration response spectrum of selected records along with the mean spectrum and FEMA356 design spectrum

4 RESULTS OF NUMERICAL STUDIES

4.1. SSAP method via FEMA356 standard spectrum

Previous studies conducted to validate the SSAP procedure and its extended form (e.g. [6] and [7]) have been done by utilizing response spectrum of individual records, i.e. response spectrum of each record is used for A_n in eq. 2.1. One the main purposes of the current paper is to make the SSAP more practical. In this regard, the response spectrum of FEMA356 [8] has been used for the SSAP method instead of utilising individual spectrum for each record. Therefore, seven NLRHA have been performed and storey drifts were compared to the results of SSAP. To compute the target displacement, NLRHA of equivalent single degree of freedom system has been done. Since SSAP is performed once for seven earthquakes, mean of maximum displacement response of the equivalent single degree of freedom system from NLRHA of seven ground motions is taken as the target displacement. Fig.4. demonstrates the storey drift predictions of the extension of SSAP method versus mean of NLRHA obtained from seven ground motions selected for the study. As it is indicated in the figure, SSAP estimates are fairly acceptable at lower storeys while results are markedly under estimated at the upper storeys up to a level of roughly 60 percent at both flexible and stiff edges of the plan.

4.2. Modified-SSAP method via FEMA356 standard spectrum

Modifications already been defined in [7] to improve results of the extended form of SSAP are evaluated in this section while design spectrum is used in the method. To find the best matched storey drift profiles between pushover analysis and NLRHA, correction factor of the load pattern is applied and target displacement amplification is then evaluated such that best results for the pushover are obtained. To modify the load pattern, applied force to the top level is amplified based on the instantaneous period of first natural vibration mode, i.e. eq. (9) is replaced by the following equation:

$$\begin{cases} \Delta F_{ix} = \Delta V_b \bar{F}_{ix},\ \Delta F_{iy} = \Delta V_b \bar{F}_{iy},\ \Delta F_{i\theta} = \Delta V_b \bar{F}_{i\theta} & i = 1,..,N-1 \\ \Delta F_{ix} = \Delta V_b \bar{F}_{ix}(1+0.9T_1),\ \Delta F_{iy} = \Delta V_b \bar{F}_{iy}(1+0.9T_1), \Delta F_{i\theta} = \Delta V_b \bar{F}_{i\theta}(1+0.9T_1) & i = N \end{cases} \quad (16)$$

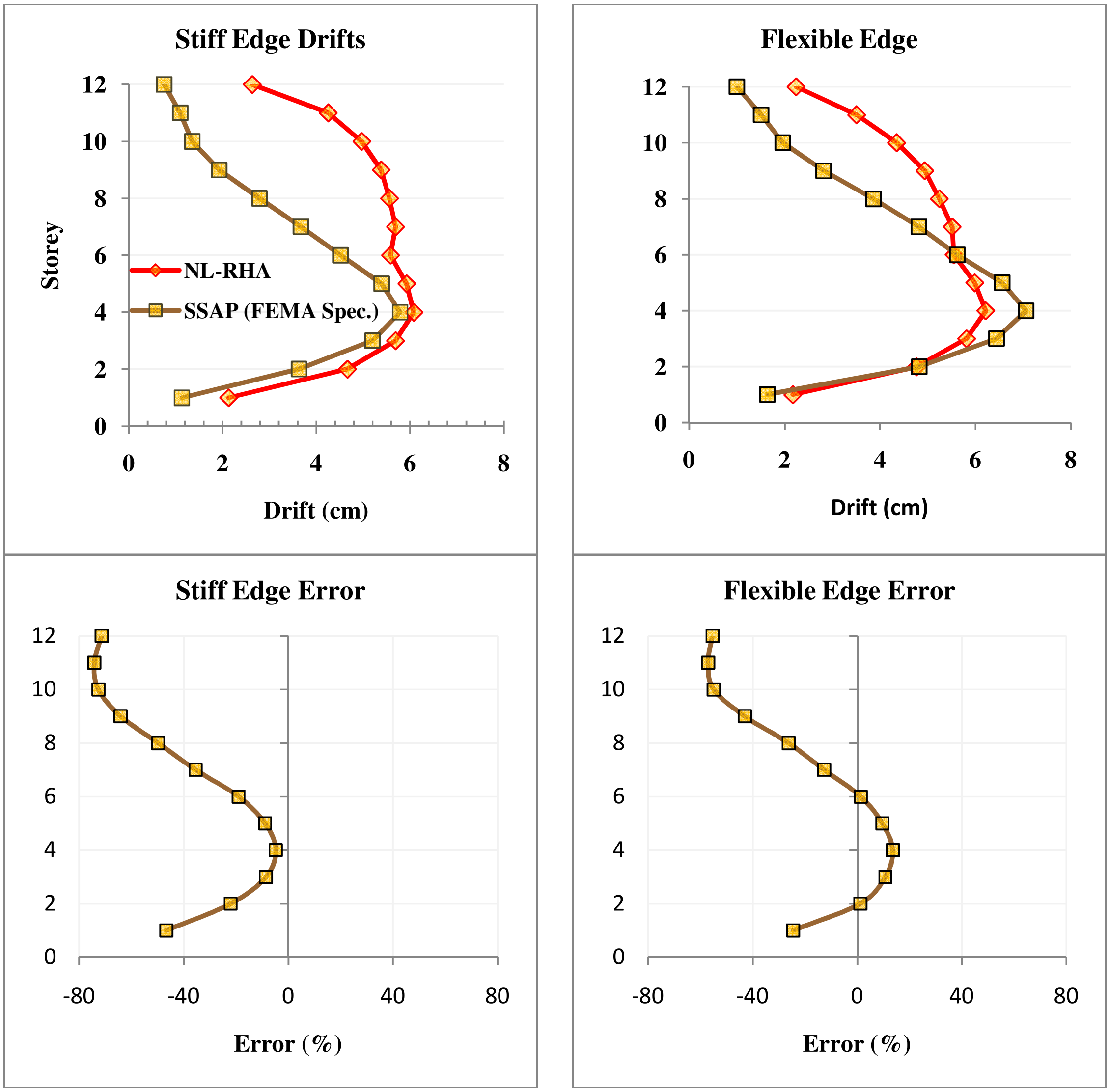

Figure 4. Height-wise storey drift predictions and errors of extended SSAP in comparision with mean of NLRHA from seven ground motions

where T_1 is instantaneous period of the first mode at each step obtained by stiffness characteristics of the structure at the end of the previous step.

As in the case of extended SSAP method described earlier, pushover analysis is performed once and the storey drift predictions of Modified-SSAP is compared with the mean of the results of NLRHA from seven ground motions. Figure 5 illustrates height-wise distribution of storey drifts in both flexible and stiff edges of the plan for SSAP when the modification factor of load pattern has been implemented in the algorithm. In order to be able to investigate the influence of the response spectrum shape in accuracy of the applied load pattern, depicted also in Figure 5 is a comparison of Modified-SSAP using FEMA356 spectrum as well as mean spectrum of the seven selected records. In both cases, target displacement correction factors have been multiplied which can lead to the best possible compatibility of the result compared to NLRHA. In this respect, correction factor 1.45 has been multiplied to target displacement

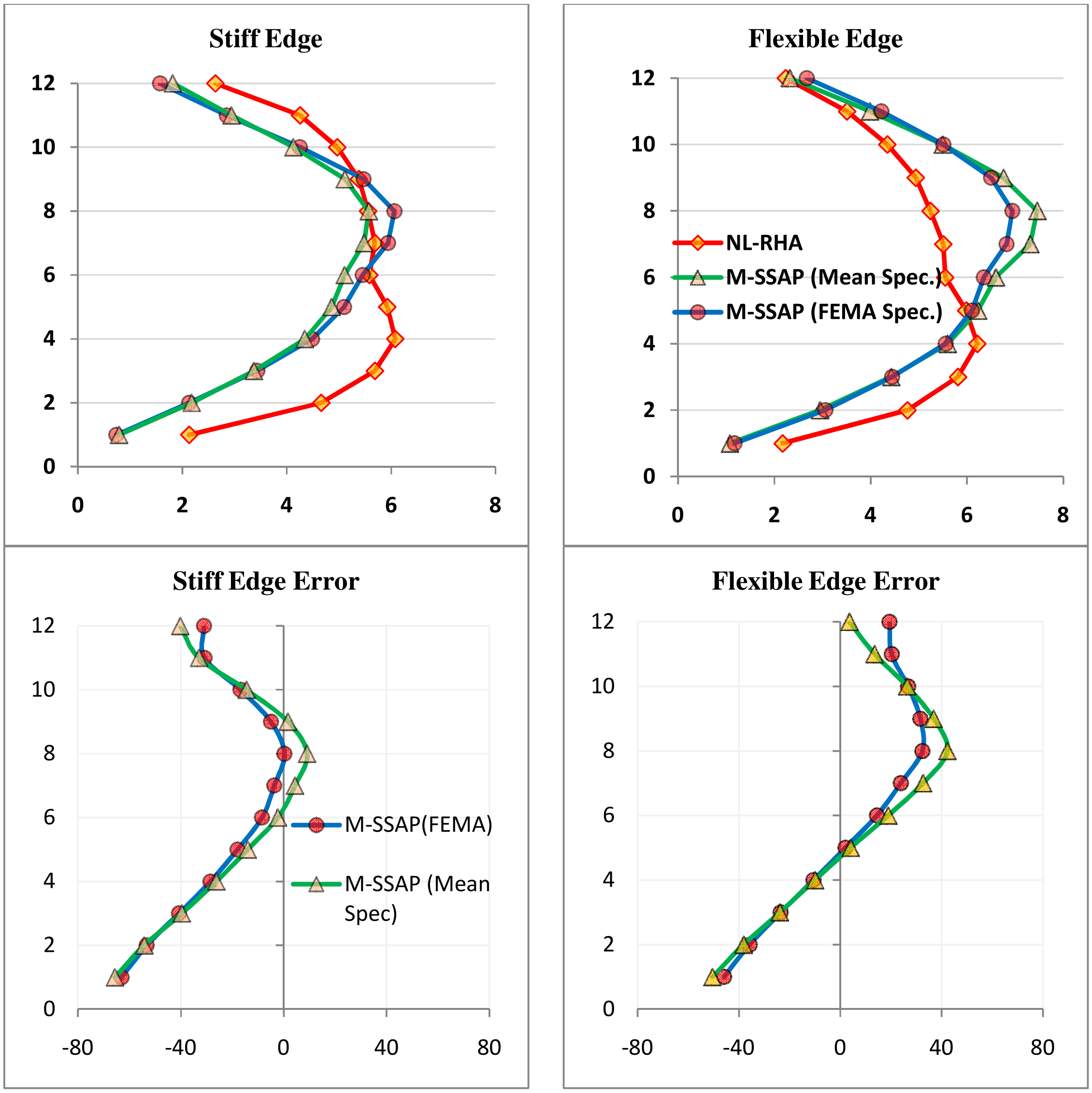

Figure 5. Height-wise storey drift predictions and errors of Modified- SSAP using mean as well as FEMA spectra in comparision with mean of NLRHA from seven ground motions

when the mean spectrum is used, and correction factor 1.5 has been used for the FEMA356 spectrum. Errors of the pushover analyses with respect to NLRHA have also been depicted in the figure. Clearly, it is expected to see much better results in the case where mean spectrum is utilised in the method. However a small difference is observed using two different spectra. This implies that design spectra can be used in the SSAP method while errors do not dramatically increase. The observation is important as far as the applicability of SSAP practically is concerned.

Comparison of figure 4 and 5 indicates that implication of modification of load pattern has been effective to improve the storey drift profile of the example building. Nevertheless, errors in the lower levels have been increased particularly in stiff edge. As it can be seen in the figure the little quantities of storey drifts in lower storey makes the errors larger while the differences are not very big. Therefore, definition of the error percentage as the ratio of drifts difference between pushover analysis and NLRHA should be taken into consideration.

5 CONCLUSIONS

A twelve storey building with torsinaly dominant fundamental natural mode was evaluated in the paper by extended SSAP and NLRHA. FEMA356 design spectrum was used in SSAP to investigate the applicability of design spectra in the procedure algorithm instead of utilizing individual records spectra. Relatively acceptable degree of compatibility of results was observed in lower levels while results deteriorated in upper storeys. Mutliplying the load pattern at the top storey by a factor related to the instantaneous period of the structure appeared to be effective to improve the compatibility of SSAP and NLRHA for the example building. However, modification factor leads to the underestimation of drift responses in lower storeys. Also, flexible edge estimates were better compatible to those of stiff edge. The impact of the selection of response spectrum on the accuracy of procedure seemed to be negligible.
Following the results of the example building studied in the paper, modifications of the original algorithm of the SSAP for planar frames looks to be a promising approach to improve the results of SSAP method. Needless to say that a very wide range of study is required to find most suitable correction factors for load pattern as well as target displacement.

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