Assessment of current nonlinear static procedures For seismic evaluation of special moment-resisting Frames

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SUMMARY:
Nonlinear static procedures are now used in engineering practice to predict seismic demands in building structures. This paper aims to investigate comparatively the bias and accuracy of modal pushover analysis (MPA) and Mass proportional pushover (MPP) and Upper-bound pushover analysis (UBPA) and Consecutive modal pushover(CMP) procedures when they are applied to special steel moment frame buildings .Three- 5- 7- 9 Story buildings were analyzed due to these procedures. The assessment is based on comparing seismic displacement demands such as peak floor/roof displacements and Peak story drift angle demands. The NSP estimates are compared to results from nonlinear response history analysis (NL-RHA). The response statistics presented show that the MPP procedure tends to inaccurately estimate seismic demands of lower stories of tall buildings considered in this study while MPA and CMP procedures provide reasonably accurate results in estimating maximum story drift over all stories of studied special steel moment frame systems.

Keyword: Consecutive Modal Pushover, Mass proportional pushover analysis, Upper-bound pushover analysis

1. INTRODUCTION

In performance assessment and design verification of building structures, approximate nonlinear static procedures (NSPs) are becoming commonplace in engineering practice to estimate seismic demands. In fact, some seismic codes have begun to include them to aid in performance assessment of structural systems (e.g., Eurocode 8 (2001); Japanese Design Code). Although seismic demands are best estimated using nonlinear response history analysis (NL-RHA), NSPs are frequently used in ordinary engineering applications to avoid the intrinsic complexity and additional computational effort required by the former. As a result, simplified NSPs recommended in ATC-40 (1996) and FEMA-356 (2000) have become popular. These procedures are based on monotonically increasing predefined load patterns until some target displacement is achieved. However, it is now well-known that these simplified procedures based on invariant load patterns are inadequate to predict inelastic seismic demands in buildings when modes higher than first mode contribute to the response and inelastic effects alter the height-wise distribution of inertia forces (e.g., Gupta and Kunnath(2000); Kunnath and Kalkan(2004); Kalkan and Kunnath(2004); Goel and Chopra (2004)). In order to overcome some of these drawbacks, a number of enhanced procedures considering different loading vectors (derived from mode shapes) were proposed. These procedures attempt to account for higher mode effects and use elastic modal combination rules while still utilizing invariant load vectors. The modal pushover analysis (MPA) of Chopra and Goel(2002), and the upper-bound pushover analysis (UBPA) procedure of Jan et al. (2004) are examples of this approach. Another class of enhanced pushover methods is the adaptive pushover procedures, where the load vectors are progressively updated to consider the change in system modal attributes during inelastic phase. Gupta and Kunnath(2000) proposed an adaptive algorithm utilizing an elastic demand spectrum. In this procedure, equivalent seismic loads are calculated at each pushover step using the instantaneous mode shapes. The corresponding elastic spectral accelerations are used for scaling of the lateral loads which are applied to the structure in each mode independently. Several other force-based or displacement based pushover procedures utilizing adaptive load patterns have also been proposed (e.g., Elnashai(2000);

Antoniou and Pinho(2004)). More recently, a new adaptive modal combination (AMC) procedure, whereby a set of adaptive mode-shape based inertia force patterns is applied to the structure, has been developed (Kalkan and Kunnath,2006).

2. VARIOUS METHOD

Pushover Analysis method can be categorized into three major groups based on how they analyze the structure (lateral load vector). The groups are Single Mode Pushover Analysis Method (SMPAM), Simple Pushover Analysis Method (SPAM), and Multi-mode Pushover Analysis Method (MMPAM).

2.1. Single Mode Pushover Analysis (SMPAM)

Pushover Analysis can be done using the mode shape of the structure as the pattern of its lateral loading vector. The most common mode shape that is used as lateral loading vector for analysis is the elastic first mode shape (fundamental mode shape) of the structure. However, these single mode load distributions are inadequate when higher-mode (mode higher than the fundamental mode) of the structure significantly contributes to the response of the structure.

2.2. Simple Pushover Analysis (SPAM)

Simple Pushover Analysis uses simple lateral load vector distribution. Two types of this loading vector are uniform distribution and equivalent lateral force. FEMA-273 determined the uniform distribution as $s^*_i=m_i$, in which m_i is the mass at *i*-th floor, and s^*_I is the lateral force at *i*-th floor. The second type is the equivalent lateral force (ELF) distribution. Based on FEMA-273, ELF can be obtained as $s^*_i = \frac{m_x h^k{}_x}{\sum_{i=1}^{N} m_i h^k{}_i}$ where the exponent $k=1$ for $T_1 < 0.5$ sec (fundamental period), $k=2$ for $T_1 > 2.5$ sec and linear interpolation shall be used for values in between, h_i and h_x are heights from the base to *i*-level floor and *x*, respectively.

2.3. Multi-mode Pushover Analysis (MMPAM)

In order to solve the problem of the structure which higher-mode effects are important, a number of enhanced procedures which account for the higher-mode effects have been proposed. Most of the current proposed methods have similarity with each other. They can be categorized into four groups based on how they determine the lateral loading vector and also how they obtain the seismic demands of the structures (results).

3.METHODOLOGY

This study will explore the ability of the current enhanced pushover procedure to simulate the structure's performance and seismic demands. The effectiveness of each method in the particular type of structure will be investigated in this study. The methods that will be compared are Modal Pushover Analysis (MPA), Consecutive Modal Pushover (CMP), Upper-bound Pushover Analysis (UBPA) and also the Mass Proportional Pushover procedure (MPP). All of those methods will be compared with the "benchmark" solution from NL-RHA.

3.1. Modal Pushover Analysis (MPA)

Modal Pushover Analysis (MPA) that has been recently developed by Chopra and Goel(2002) is an improved pushover procedure which takes into account higher-mode effect in analyzing seismic demand while retaining the simplicity of the loading pattern in pushover analysis. MPA utilizes the concept of modal combination through several pushover analyses using invariant loading distribution vector based on the elastic modes of the structure. The response of each pushover analysis is being combined use SRSS or CQC combination rule to get the total response of the structure.

3.2. Consecutive Modal Pushover (CMP)

Consecutive Modal Pushover (CMP) has been proposed by Pourshaet.al(2009). This method uses a single-stage and multi-stage pushover analysis. The multi-stage pushover analysis is conducted using an advantage of the consecutive implementation of MPA procedure. When one stage of the modal pushover analysis has been performed, then the next stage (another modal pushover analysis) begins with an initial state (stresses and deformations) from the end state condition of the previous stage. CMP procedure is carried out with the various loading pattern based on the modal properties of the linearly elastic structure.

3.3. Upper-bound Pushover Analysis (UBPA)

Upper-bound Pushover Analysis (UBPA) first proposed by Jan et.al, (2004) determined the distribution vector of the lateral loads over the height of the building by combining effects of the first and second mode.

Unlike the MMPA where the response is obtained from the combination of individual analyses using different mode shapes, the upper-bound pushover analysis is based on utilizing a single load vector obtained as the combination of the first mode shape and a factored second mode shape. The spectral displacements (D_n)corresponding to elastic first and second mode periods are estimated from the elastic spectrum of the considered ground motion and the upper-bound contribution of the second mode is established using modal participation factors (Γ_n), as follows:

$$\left(\frac{q_2}{q_1}\right) = \left| \ (\Gamma_2 D_2)/(\Gamma_1 D_1) \ \right| \tag{3.1}$$

The invariant load vector (F) is then computed as the combination of first and second mode shapes:

$$F = \omega^2{}_1 m\varphi_1 + \omega^2{}_2 m\varphi_2 \left(\frac{q_2}{q_1}\right) \tag{3.2}$$

3.4. Mass proportional pushover (MPP) procedure

An alternative pushover analysis procedure, called the mass proportional pushover (MPP), was proposed by Kim and Kurama (2008) to estimate the peak seismic lateral displacement demands for buildings. The main advantage of the MPP procedure over other approximate procedures is the use of a single pushover analysis for the structure with no need to conduct a modal analysis to capture the effect of higher "modes". A summary of the mass proportional pushover procedure, whose details can be found in Kim and Kurama, is as follows:

(1) Determine the multi-degree-of-freedom (MDOF) base shear force versus the roof displacement (V_b-u_r) relationship using the force distribution given by $mg_L = w_L$ where m is the mass matrix and w is weight matrix.

(2) Idealize the pushover curve as a bilinear curve.

(3) Convert the idealized pushover curve to the pseudo-acceleration versus the displacement (A-D) relationship of an equivalent SDF system using:

$$A = \frac{V_b}{M} \tag{3.3}$$

$$D = \frac{u_r}{\Gamma} \tag{3.4}$$

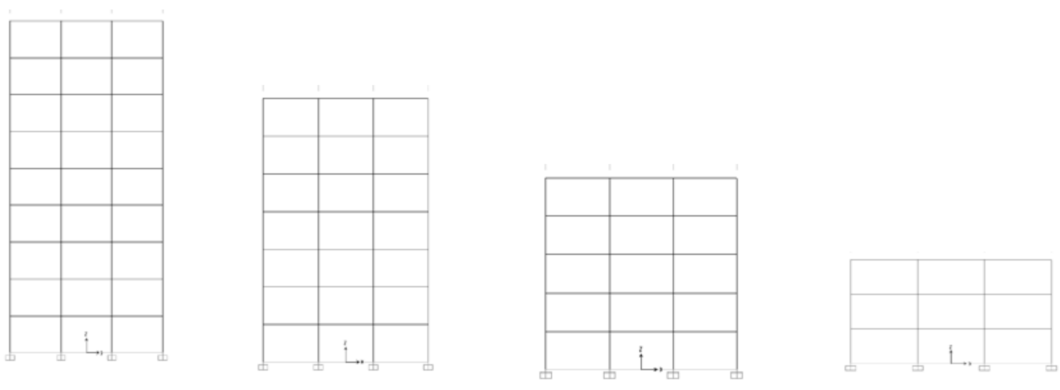
Where M is the total mass and Γ is the participation factor calculated as: $\Gamma = \frac{u^T{}_e mL}{u^T{}_e m u_e}$; u_e is the lateral floor displacement vector (normalized with respect to the roof) obtained from the Linear elastic response range of the pushover analysis using the $mg_L=w_L$ force distribution which is the same as uniform distribution of FEMA-356.
(4) Determine the maximum SDOF displacement, D_{max}
(5) Calculate the maximum MDOF roof and floor displacements of structure as:

$$u_{max}=D_{max}\,\Gamma\,u_e \qquad (3.5)$$

4. STRUCTURAL SYSTEM AND ANALYTICAL MODEL

The structural systems that will be used in this study are 3-5-7-9 buildings. The frames elevations were 9, 15, 21and 27m, respectively. The floor elevation views of the structural systems in this study are shown in Figure 1. The structural models were prepared in Open Sees 2.1.0 (2005) for nonlinear dynamic time history analysis subjected to different bidirectional earthquake loadings. Hardening material type having a bilinear Hysteretic behavior with strain hardening ratio of 3% was used for steel members. All the members were modeled as Nonlinear Beam-Column fiber elements with five integration points along each one. This type of element has the capability of considering distributed plasticity so that the initiation points of plastic behavior are not predetermined by the user and it can start in any of the fibers along the length of element at any time. The fiber sections can simultaneously take into account the effects of axial forces and bending moments but shear effects are ignored.

In order to obtain the “benchmark” result, Nonlinear Time History Analysis was conducted. Ten ground motions were selected from the strong ground motion database of the Pacific Earthquake Engineering Research(PEER) Centre (http://peer.berkeley.edu). The selected ground motions were far-field records, and corresponded to locations which were at least 15 km from a rupturing fault. Also, the soil at the site corresponds to NEHRP site class D. To ensure that the structures respond into the inelastic range when subjected to ground motions, all records are scaled with the Iranian code 2800 standard response spectrum for soil type III. More characteristics of the ground motion records used are given in Table 1. The elastic pseudo-acceleration, together with the corresponding the mean spectra, are presented, for a 5% damping ratio, in Figure. 2. The mean spectra are shown by a black line.



$(d)=9-story$ $T_1=1.51\sec$

$(c)=7-story$ $T_1=1.25\sec$

$(b)=5-story$ $T_1=1.15\sec$

$(a)=3-story$ $T_1=0.58\sec$

Figure 1. Frame elevation view of the structures (3-5-7-9 story buildings)

Table 1. Characteristics of considered far fault earthquake motions

No	Earthquake	date	Station name	Magnitude (MS)	PGA (g)
1	SAN FERNANDO	1971/02/09	LA HOLLYWOOD STOR LOT	6.61	0.21
2	NORTHRIDGE	1994/01/17	LA - HOLLYWOOD STORAGE FF	6.69	0.231
3	SIERRA MADRE	1991/06/28	LOS ANGELES - OBREGON PARK	5.61	0.221
4	COYOTE LAKE	1979/08/06	HALLS VALLEY	5.74	0.0391
5	NORTHRIDGE	1994/01/17	LA - FARING RD	6.69	0.273
6	MORGAN HILL	1984/04/24	APEEL 1E HAYWARD	6.19	0.0406
7	TRINIDAD	1980/11/08	RIO DEL OVERPASS FF	7.2	0.0614
8	IMPERIAL VALLEY	1979/10/15	COACHELLA CANAL	6.53	0.115
9	VICTORIA, MEXICO	1980/06/09	CHIHUAHUA	6.33	0.15
10	LOMA PRIETA	1989/10/18	HALLS VALLEY	6.93	0.134

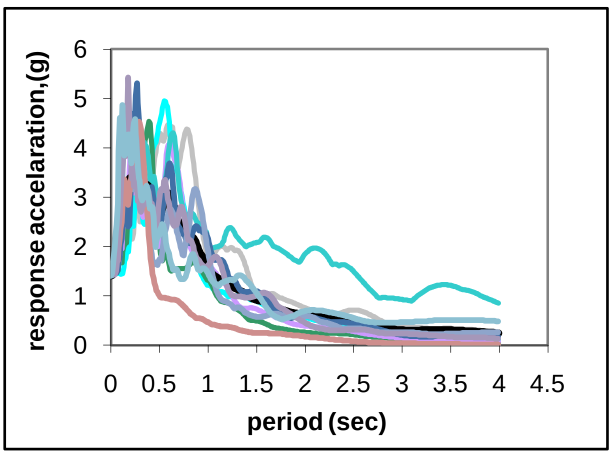
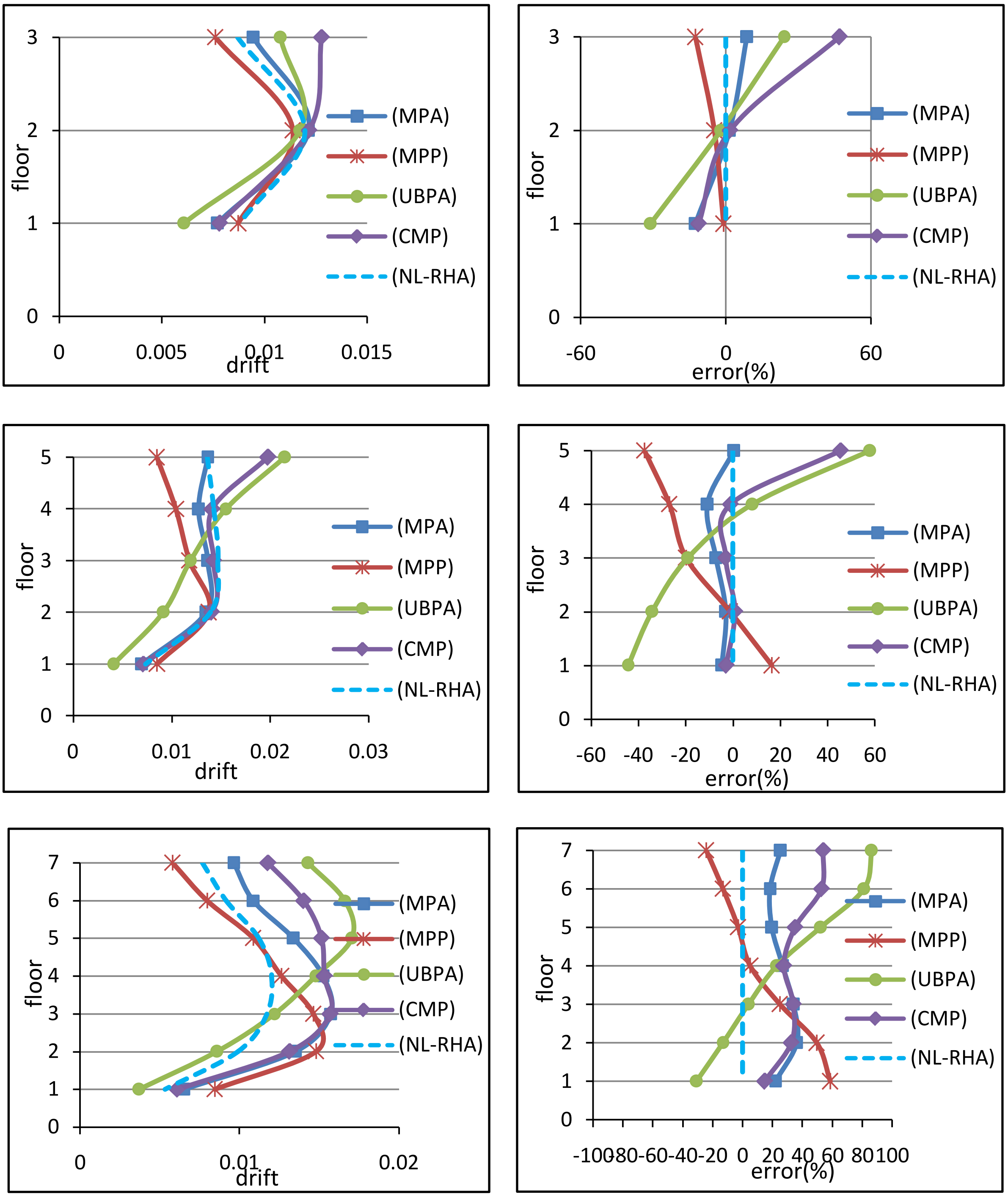


Figure 2. (a) Response-acceleration spectra of the set of far-field records of ground motions, damping ratio =5%

5. EVALUATION OF NONLINEAR STATIC PROCEDURES

The performance of the structure can be investigated through some parameters which will be obtained after the analysis. To validate the methods, they will be compared to NL-RHA method as the "benchmark" results. The response evaluation parameters which were selected to analyze the structural system are as follows:

(1) **Story drift of the structure**. Story drift is the lateral displacement that occurs in a single story of multistory building. Story drift can be obtained as the relative drift between two consecutive stories normalized by corresponding story height. Figure 3 shows the story drift obtained with each method.



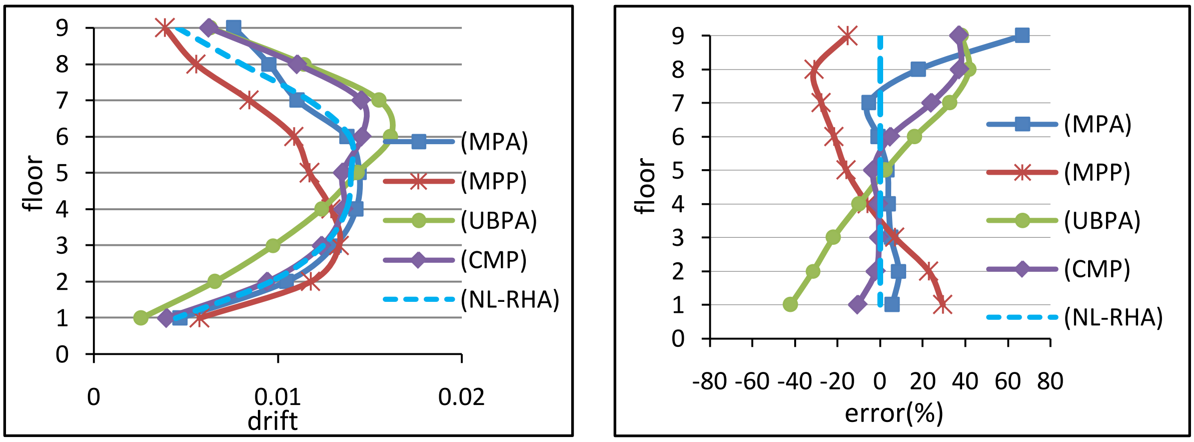
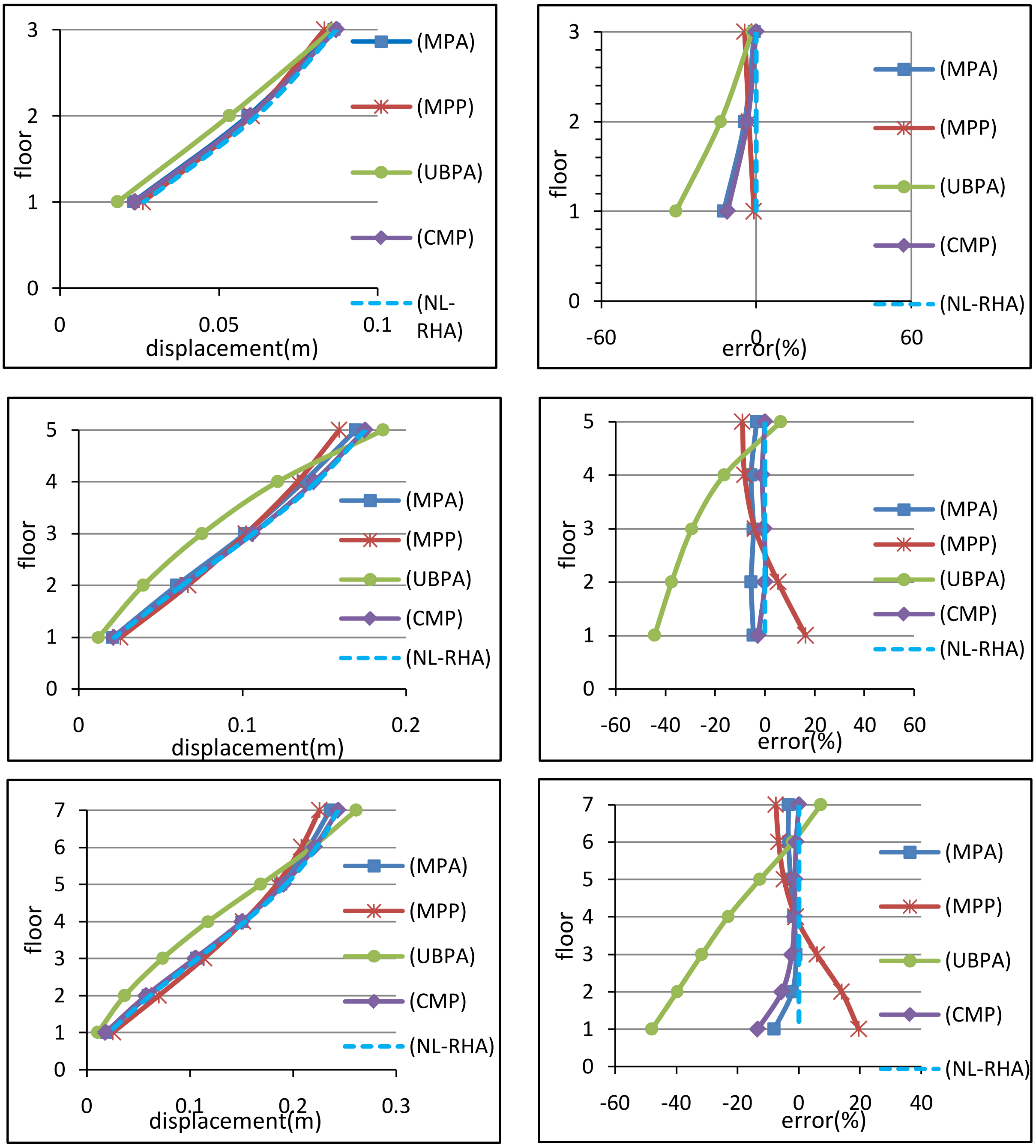
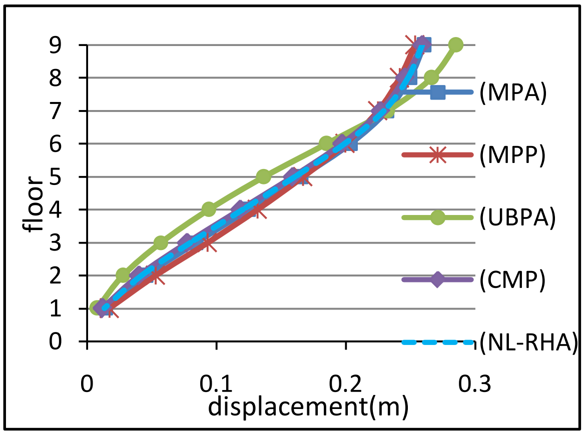


Figure 3. Story drift profile

(2) **Lateral displacement of the structure**. To find the behavior of the structure when it is subjected to the lateral loading can be observed from its lateral displacement. Figure 4 shows the story displacement obtained with each method.





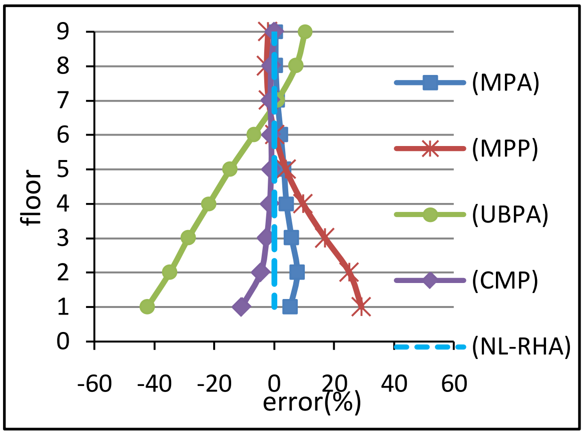


Figure 4. Displacement profile

6. CONCLUSIONS

In the needs of Performance Based Design (PBD) “tools”, attempts have been made to include higher mode effect to the Pushover Analysis method so that it can obtain reliable results. Some parameters have been investigated in this study, and compared to validate the methods. This study compared all 4 methods (CMP, MPA, MPP and UBPA) that include higher-mode effects to the analysis and compared to the “benchmark” result from NL-RHA.
Research has been conducted and led to the following conclusions:

(1) Enhanced Pushover Analysis methods that have been studied in this research tend to be conservative in giving displacement value and story drift ratio of the structure compared to the NL-RHA.
(2) To obtain total base demands, only MPA and CMP can give reasonable results compared with NL-RHA.
(3) The UBPA procedure has a tendency to overestimate the seismic demands at the upper stories and underestimate them at the lower stories and among considered procedures this is the most inaccurate one.
(4) The MPP procedure has a tendency to underestimate the seismic demands at the upper stories and overestimate them at the lower stories.
(5) The CMP procedure benefits from consecutive implementation of modal pushover analysis and uses limited number of modes to develop. The CMP procedure estimates the height-wise distribution of drift ratio well, and their results are similar to results obtained by NL-RHA.
(6) Among considered procedures, UBPA overestimates the target displacements.

Based on the results and analysis of the investigated methods, it can be concluded that enhanced pushover analysis methods is able to be used to evaluate particular response (displacement, drift) of the structure although some of the results cannot give conservative results compared to NL-RHA.

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