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Verification of Consecutive Modal Pushover Procedure for Estimating the Seismic Performances of Steel Plate Shear Walls

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ABSTRACT

In this paper, we evaluated seismic performance properties of steel plate shear wall (SPSW) using Consecutive Modal Pushover Procedure (CMPP). This method is performed on 3, 6 and 9-story SPSW frames subjected to seven earthquake records which are scaled according to ASCE/SEI 7-05 provisions. We conducted nonlinear time history analysis (THA) to verify extracted outputs. The SPSW models indicate a relatively accurate estimation in nonlinear story drift and story displacement response of pushover procedures compared to that of the THA with respect to responses like shear story; while, in the high-rise model in specific, the deformation parameters are more accurate through an increase in the height of the models.

Keyword:
Steel plate shear wall;
nonlinear time history analysis;
Consecutive Modal Pushover Procedure;
Story shear;
drift

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INTRODUCTION

In recent years, many attentions have been paid on high spectrum performance by many digital radio system

During recent decades, major developments are being made in earthquake engineering and analysing structures under seismic loads. A pseudo-capacity resists against the pre-mentioned lateral force based on elastic methods of seismic design. It is been assumed that the structure could resist against imposed load of earthquake by the means of yielding into the plastic range, energy dissipation and ductility (Seifi et al., 2008). Major earthquakes such as Northridge, California (1994), Kobe, Japan (1995) lead to significant demolitions of buildings designed based on old methods and consequently massive human and economic losses. Therefore it is proved that such methods could have deficiencies to make buildings safe during an earthquake. Since these methods did not inform us about how the structure would actually act under severe seismic conditions. Using performance-based methods it is possible to recognize actual behaviour of structure. There are some guidelines entitled as "Performance-Based Design Engineering" (PBDE) that include performance-oriented methods instruction. These methods are frequently highlighted in this field of study (Krawinkler and Seneviratna, 1998; Behforooz et al., 2014).

This new analytical procedure has two more advantages in comparison with the conventional perspectives on earthquake engineering: first, the direct relation between structural design and the performance of structure; second, ability to have a multiple performance design. The performance targets could be the level of stresses that not to be exceeded. This level could also be controlled by defining load or displacement target, limit states or damage state target (Bracci et al., 1997).

The performance of structure subjected to earthquake loads could be predicted using nonlinear time history analysis (NTHA). Nonlinear dynamic procedures as the most exhaustive analysing methods apply ground motion records to a detailed structural model. These methods might have a relatively low uncertainty to design structures precisely.

The extracted outputs from structural model are very susceptible to variations of specifications of the individual ground motion which is used as input. Consequently, several analyses must be implemented to apply different ground motion records (Bracci et al., 1997; FEMA, 2000).

A mathematical tool which works properly under tight time constraints could handle all analyses and often intrudes on the capabilities of a design office. Trying to find a reliable method with less computational costs, results in apparition of Nonlinear Static Pushover method.

When one structure is subjected to strong motions, its actual response at any given time could be realistically described by implementation of a non-linear dynamic analysis. Crack order and members yielding as well as evaluation of structure response, could be determined using this analysis method .

At present, many seismic codes in different countries all around the world provide non-linear dynamic analysis methods to analyze many special and complex structures. Obviously, this method has two major drawbacks: its computational costs and uncertainties that it is required to have many inputs to reduce them. Although the non-linear dynamic analysis is now common in the theoretical studies, it is worth devising a simplified analyzing method for seismic performance evaluation of structures. For this purpose Push over Analysis (POA) is a competent method. Important outputs could be resulted from POA method that is much simpler than dynamic analysis.

Pushover analysis could be used to analyze Steel Plate Shear Wall (SPSW) system. SPSW is a lateral-load-resisting system which includes vertical steel plate infills. Steel plates fill whole space surrounded by beams and columns in each bay. These plates are connected to the frame and they might be installed in one or more bays of the frame. SPSW has some significant advantages over many other lateral-load-resisting systems. It has lower costs and better performance as well as ease in design. Lateral loads applied to SPSW system could lead to appear diagonal tension in the plate and overturning forces in the adjoining columns. Therefore SPSW system could resist lateral loads and it might be a good choice for retrofitting seismically-weak structures. It could be added to existing buildings that they have not sufficient strength and stiffness against major earthquakes.

Systems designed for high-seismic loading are expected to undergo multiple cycles of loading. Multiple load cycles could lead systems to have inelastic responses with the controlled damage accepted as a means of dissipating the energy of the earthquake. Web plates in SPSW consume a large amount of the system energy by plasticity, which ends up in the ductility causing the ductility of the existing system. The high-seismic design of SPSW is based on confining ductility demands to the web plate and to plastic hinges in the Horizontal Boundary Element (HBE) at the Vertical Boundary Element (VBE) face. In this respect, SPSW could be designed using capacity-design concept according to AISC 341 guidelines. Based on this method boundary elements would be designed using forces corresponding to the full yield strength of the web plate (Memarzadeh et al., 2010).

In this paper the accuracy of the Consecutive Modal Pushover Procedure (CMPP) (Poursha et al., 2009) is evaluated. For this purpose, seismic performance of typical SPSW systems are estimated subjected to the different earthquake records (Behforooz et al., 2014). In this study 3, 6 and 9-story SPSW frames (Behforooz et al., 2014; Behforooz et al., 2013) are investigated subjected to seven earthquake records which are scaled according to ASCE/SEI 7-05 provisions. Evaluated Frames are designed based on AISC requirements (AISC, 2005). Moreover, nonlinear time history analysis (THA) is conducted to verify extracted outputs.

1. Consecutive modal pushover procedure

The consecutive modal pushover (CMP) procedure could be implemented on the structures with inelastic behavior to estimate their peak response subjected to earthquake excitation. This method is based on conducting different pushover analysis successively. Extracted results from various analyses must be enveloped to find the maximum values as final result in CMP method. This method is performed through multi-stage and single-stage pushover analyses. The multi-stage pushover analysis benefits from consecutive implementation of modal pushover analyses, including a limited number of modes. In this method, outputs of one stage would be used as inputs for next stage. Each stage includes a modal pushover analysis that must be completely performed individually. In each stage, some properties derived from previous stage such as stress and deformation would be used as initial structural properties. Lateral force distribution in consecutive modal pushover analyses is based on mode shapes obtained from Eigen-analysis of the linear elastic structure. Changes in the modal properties of the structure are ignored when the structure experiences nonlinear yielding under increasing lateral loads during pushover analysis. The number of modes in the consecutive modal pushover analyses depends on the fundamental period, T, of the building structure. When the fundamental period of the building is less than 2.2 s, the multi-stage pushover analysis is carried out in two stages (Lopez-Menjivar and Pinho, 2004).

For buildings with fundamental periods of 2.2 s or more, both two- and three-stage pushover analyses are used. In multi-stage analysis, it is required to calculate the displacement increment at the roof for each stage. Displacement increment, u_{ri}, at the roof in the i^{th} stage could be obtained by multiplication of total target displacement in the roof floor and a factor which is determined from the initial modal properties of the structure:

$$u_{ri} = \beta_i \delta_i \quad (1)$$

in which $\beta_i = \alpha_i$ stages before the last stage, and

$$\beta_i = 1 - \sum_{j=1}^{N_S - 1} \alpha_j \quad (2)$$

where δ_t is the total target displacement at the roof, and Ns is the number of stages included in the multi-stage pushover analysis. Also, α_i is the effective modal mass ratio for the ith mode. Several different approaches can be used to establish the total target displacement at the roof level. This displacement can be determined by using the displacement coefficient approach or dynamic analysis of the structure (Poursha et al., 2009).

As demonstrated previously, in addition to multi-stage pushover analysis, single-stage pushover analysis could be used in CMP method too. Triangular or uniform load distribution is used to perform single-stage pushover analysis separately. Finally, multi-stage and single-stage pushover analyses responses would be enveloped to derive peak responses and consequently seismic demands. The details of the CMP procedure are expressed as a sequence of the following five steps:

- To calculate the natural frequencies, and the mode-shapes. These properties are determined by Eigen-analysis of the linearly elastic structure for the first three modes.
- To derive $S_n^* = m\phi_n$ based on mode shapes as a lateral load pattern.
- To compute the total target displacement of the structure at the roof.
- The CMP procedure includes multi-stage and single-stage pushover analyses. Gravity loads must be applied firstly and displacement-control pushover analyses would be conducted according to the following sub-steps :
  - To perform the single-stage pushover analysis using proper load distribution. Inverted triangular load pattern and uniform force distribution should be used for medium-rise and high-rise buildings respectively. Analysis ends when the displacement of control node at the roof reaches to the predefined total target displacement.
  - In this step a two-stage pushover analysis will be conducted. In the first stage, nonlinear static analysis must be performed. Incremental lateral forces $S_1^* = m\phi_1$ are used in the analysis until the displacement increment at the roof sways to ur1 (Eq.1). In second stage lateral forces $S_2^* = m\phi_2$ would be incremented until the displacement increment at the roof reaches to $u_{r2} = \beta_2 \delta_t$ where $\beta_2 = 1 - \alpha_1$. It is worth noting that state of structure at the last step of analysis in the first stage must be used as initial condition in the second stage of the two-stage pushover analysis.
  - The third step is a pushover analysis that must be performed in three stages. It is only performed for buildings having a fundamental period of 2.2 s or more.
- Calculate the peak values of the desired responses, such as displacements, story drifts, and hinge plastic rotations, for the pushover analyses described above. The peak values resulting from the one-, two-, and three-stage pushover analyses are used for estimating the seismic of structure. It is shown by Poursha et al. (2009) that the seismic demand of the inelastic structure in the CMP procedure is obtained by enveloping the peak responses resulting from the single- and multi-stage pushover Analyses.

2. SPSW Structure

In this study, three SPSW frames are considered to be analyzed. Figure 1 shows the specifications assumed for these frames such as geometry and section properties. SPSW frames are designed (Sabelli and Bruneau, 2007) based on AISC 341 (AISC, 2005). Load distributions applied on these structures are chosen according to lateral forces specified by ASCE 7 (Lopez-Menjivar and Pinho, 2004). Sabelli and Bruneau (2007) implemented an analysis using these equivalent forces. They conducted this procedure for 3, 6 and 9-story 2D frames with three bays that steel plates are placed in the middle bay as shear wall (see figure 1). There are reduced beam steel sections at both ends of Horizontal Boundary Elements (HBE). Plastic section modulus of reduced beam section must be two-thirds of plastic modulus of corresponding HBE section. Mechanical properties of the SPSW frames are provided in Table 1.

Table 1. Mechanical properties of typical bi-linear kinematic hardening model

E_1 (GPa)	E_2 (GPa)	υ	ρ (ton/m^2)	F_{yp} (MPa)	F_{yb} (MPa)
200	200	0.3	7.8	248	345

where ρ, E and υ are the density, modulus of elasticity and Poison's ratio of steel material, respectively. F_{yp} and F_{yb} are the yield strength of steel material of plate and boundary member. Material properties of beams and columns are the same as the boundary element properties. Section name and dimensions of the beams, columns and plates are shown in figure 1.

Moreover, finite elements (FE) procedure is conducted to model SPSW frames and to validate results of our model. For a time interval of 5 seconds, we applied a sinusoidal load at the roof level of frames with natural frequency of first mode. Similar to research conducted by Memarzadeh et al. (2010), loads were removed and SPSW frames were allowed to oscillate freely. Furthermore, degree of the meshing efficiency was checked by some analyses. By comparing results, it could be found that FE results such as various energy quantities, shear forces as well as displacements and accelerations are validated properly.

It is very important to choose suitable element in finite elements modeling. For beams and columns of SPSW frames, the "B31" beam elements with I-shape cross sections are selected. "S4" shell element is also chosen for shear wall steel plates. These elements are available in the ABAQUS library of elements (ABAQUS user manual, 2007). "B31" element is a 2-node beam element that its formulation is derived by linear interpolation in three-dimensional space. Although, transverse shear for deformation is allowed in “B31” element, the additional flexibility corresponded to it is ignored in this study. Moreover, “B31” element uses lumped mass assumption in its formulation (ABAQUS user manual, 2007).

Transverse shear deformation is also allowed in "S4" element which is a 4-node general-purpose doubly curved shell element. When shell thickness decreases, this deformation will become very small. Based on thick shell theory, "S4" element will become discrete Kirchhoff thin shell element when the shell thickness increases. Furthermore, linear interpolation is used in formulation of this element and it is taken into account for arbitrarily large rotations and finite membrane strains (ABAQUS user manual, 2007).

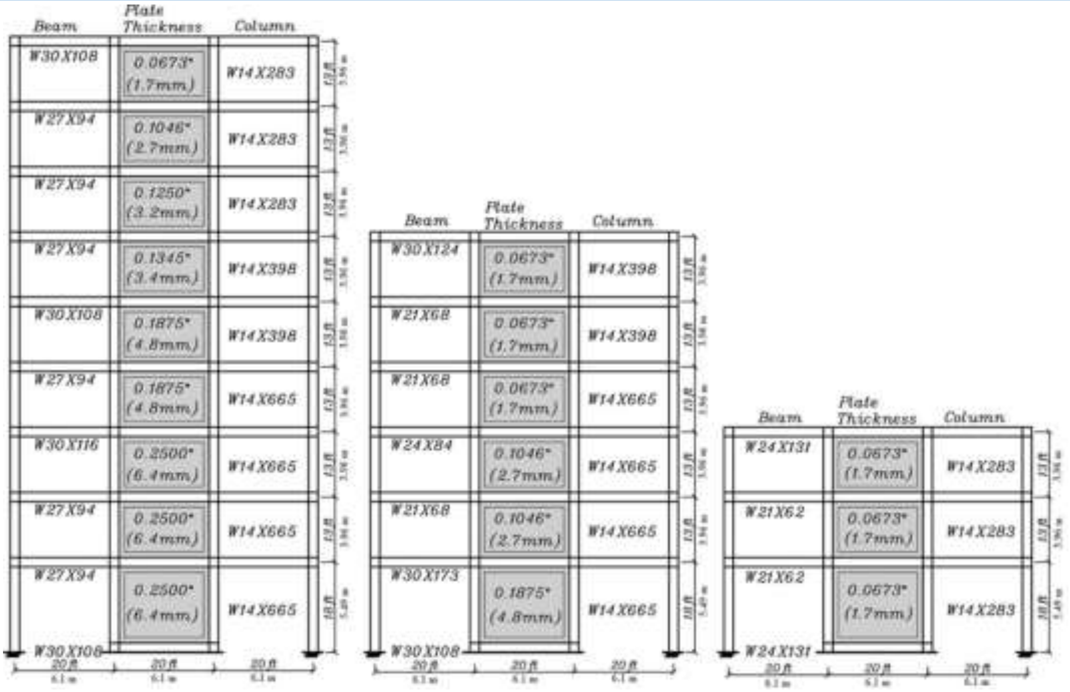

Figure 1. Dimensions and properties of SPSW buildings used for analysis (Behforooz et al., 2014)

3. Ground Motion Scaling

Seven ground motion records are applied on the SPSW frames for the sake of evaluating degree of accuracy for chosen methods subjected to different ground motions. Various earthquake records are available on the Pacific Earthquake Engineering Research (PEER) website (http://peer.berkeley.edu/smcat). Ground motion records used in this study are also extracted from PEER website. There are instructions for time history analysis (THA) in design codes such as the International Building Code (IBC) (ICBO, 2006) and California Building Code (CBC) (ICBO, 2007). Based on these design codes, earthquake records must be scaled according to the ASCE/SEI 7-05 provisions (ASCE, 2005). In time history analysis, ground motion must be scaled in such a way that average value of elastic response spectra becomes greater than design response spectra over the period in range $0.2T_1$ through $1.5T_1$ while damping ratio is considered as 5%. It could be observed clearly in figure 2 that all analyzed SPSW structures are scaled properly, so that their elastic response spectra are not less than ASCE spectra in specified range in design codes. In Table 2 properties of seven ground motion earthquake records and their scaled response spectra are presented.

Table 2. Ground motion properties

Name	Date	Station	Component (deg)	Vs30 (m/s)	PGA	PGV	Soil Type (UBC97)	Site Class in NEHRP
Cape Mendocino	4/25/1992	Rio Dell Overpass-FF	270	311.8	0.385	43.92	SD	D
Loma Prieta	10/18/1989	Capitola	0	288.6	0.529	35.001	SD	D
Duzce	11/12/1999	Duzce	180	276	0.348	60.024	SD	D
Erzincan	3/13/1992	Erzincan	NS	274.5	0.515	83.956	SD	D
Imperial Valley	10/15/1979	El Centro Array #11	E	196.3	0.364	34.366	SD	D
Northridge	1/17/1994	Pardee - SCE	L	345.4	0.657	75.209	SD	D
Kobe	1/16/1995	Takatori	0	256	0.611	127.191	SD	D

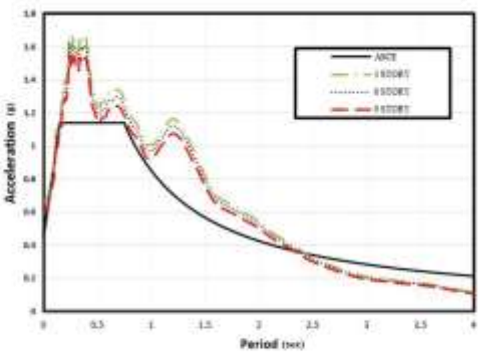

Figure 2. Scaled Response Spectra for each frame and ASCE spectra

4. Evaluation OF Consecutive modal pushover procedure

Outputs extracted from models evaluated by CMP method are compared to nonlinear THA results. Roof drift ratio, inter-story drift ratio and story shear values for different frames are calculated using CMP and THA method which are shown in figures 3 through 5. Part a of figures 3 to 5 shows roof drift ratios which is computed using normalizing maximum roof displacement by building height. Part b of figures 3 to 5 indicates inter-story drift ratio where it could be calculated using normalizing relative displacement between two adjacent stories by story height. Story shear values are shown in part c of figures 3 to 5. In these figures the time history results are computed based on a set of seven scaled earthquake records and both the mean THA and four-estimated load patterns.

Lopez-Menjivar and Pinho (2004) proposed an error index to compare the accuracy of the different NSPs parameters which is applied in this project:

$$ERORR_{(NSPs)} = 100 \times \sqrt{(\frac{1}{n}) \times \sum_{i=q}^{n} (\frac{NSP_{Parameters} - THA_{parameters}}{THA_{parameters}})^2} \quad (3)$$

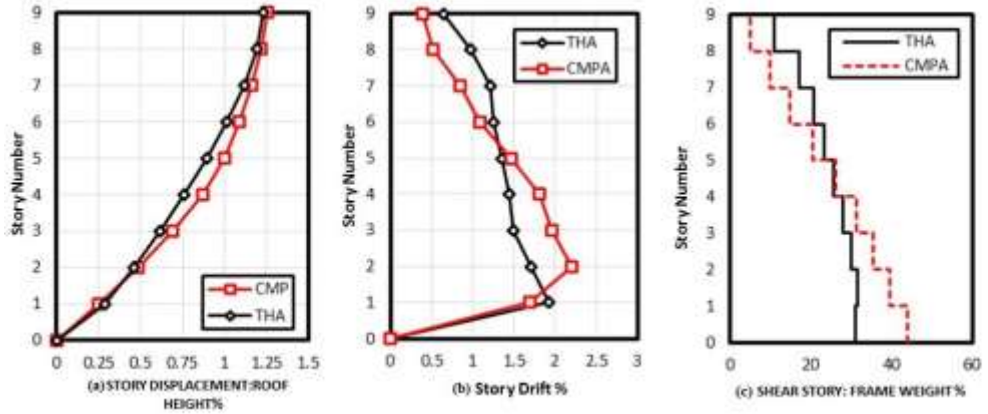

Figure 3. Evaluation of different seismic parameters in 9-story SPSW frame

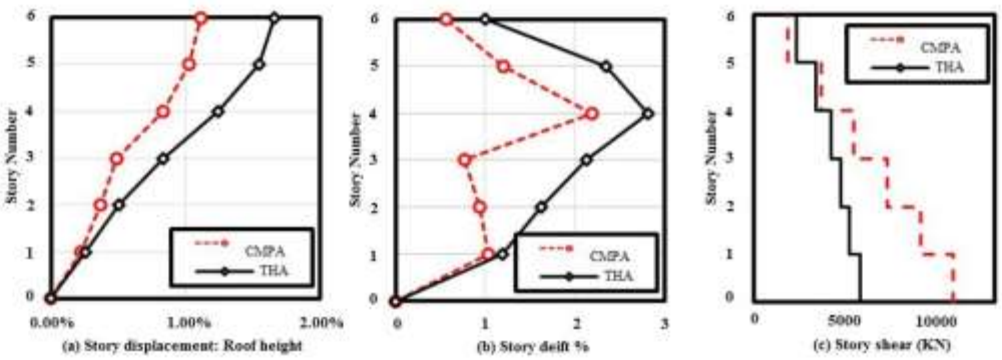

Figure 4. Evaluation of different seismic parameters in 6-story SPSW frame

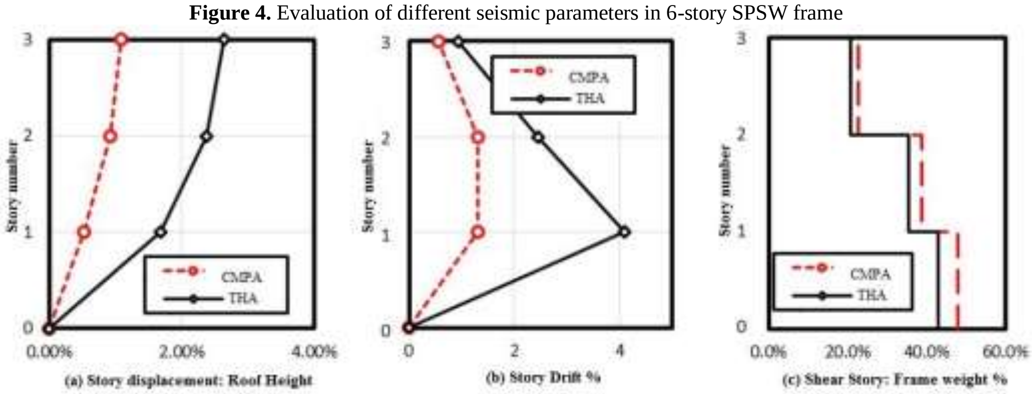

Figure 5. Evaluation of different seismic parameters in 3-story SPSW frame

5. Discussion and Results

Some dynamical properties such as roof drift ratios, story drift ratios and story shear are computed using CMP and THA methods which are shown in figures 3 through 6 for different SPSW frames. CMP and THA analysis outputs are the approximated and accurate data respectively. As it could be found from figure 3-a, the peak floor displacement for 9-story SPSW frame is nearly accurately approximated using CMP method. Moreover, according to figure 3-b results obtained by CMP methods have a good agreement with THA results, so that this method could be accurately applicable to estimate inter-story drift ratio in high-rise buildings. Although CMP method has accurate results for drift ratios, extracted story shear data is not accurate for 9-story SPSW frame. Generally, if we compare all seismic data computed by CMP method we can conclude that CMP is one of the most accurate methods to analyze SPSW structures .

In spite of good results obtained for high-rise SPSW buildings, data computed by CMP method for low-rise and mid-rise buildings have not a good accuracy. As it is shown in figure 4 and figure 5, peak floor displacements have relatively low accuracy so this method could not be a good option for estimating inter story drift and story shear for 3-story and 6-story SPSW frames.

Figure 6 shows that error index is relatively low for 9-story SPSW building, while it is totally not acceptable for 3-story and 6 story SPSW frames. Consequently, this method is not sufficiently accurate to estimate seismic performance of low-rise and medium-rise SPSW buildings and it is almost accurate method for high-rise SPSW buildings.

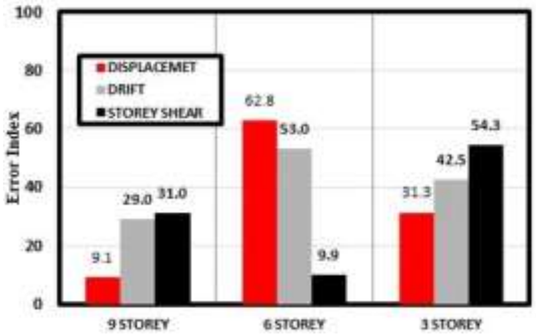

Figure 6. Comparison of accuracy of CMP parameters for SPSW buildings, using proposed error index

6. Conclusion

In this paper, seismic performance properties of steel plate shear wall (SPSW) are assessed using Consecutive Modal Pushover Procedure (CMPP). This method is performed on low-rise, mid-rise and high-rise SPSW frames. So, 3, 6 and 9 story frames are analyzed under seven earthquake records which are scaled according to ASCE/SEI 7-05 provisions. Nonlinear time history analysis (THA) is conducted to verify extracted outputs and selected structures are designed to meet seismic code criteria. The accuracy of CMP method is evaluated and these conclusions are drawn:

- Peak roof displacement can be accurately estimated using nonlinear static methods determined by equivalent bilinear SDF systems for 9-story SPSW, while results are not accurate for 3 and 6-story SPSW models.
- Only the first mode could be approximately adequate by itself to construct mode-based load pattern for 3-story and 6-story SPSWs buildings. Since, higher modes have no meaningful statistical content and higher modes are not required to be extracted from CMP method for low-rise, and mid-rise SPSW buildings.
- Maximum story drift and roof displacement extracted by suggested CMP procedure are generally not accurate in low-rise and mid-rise SPSW models across all stories. However, this method might be suitably applied to estimate maximum story drift at an individual story for certain cases.
- It could be found by comparison CMP results with THA outputs that CMP results have good degree-of-accuracy to evaluate seismic performance of high-rise SPSW frames. So, CMP method is good choice for high-rise SPSW buildings to estimate floor displacements, story drift ratios and story shears.

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