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A new lateral load pattern for pushover analysis in structures

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Abstract. Some conventional lateral load patterns for pushover analysis, and proposing a new accurate pattern was investigated in present research. The new proposed load pattern has load distribution according weight and stiffness variation in height and mode shape of structure. The assessment of pushover application with mentioned pattern in X type braced steel frames and steel moment resisting frames, with stiffness and mass variation in height, was studied completely and the obtained results were compared with nonlinear dynamic analysis method (including time history analysis). The methods were compared from standpoints of some basic parameters such as displacement, drift and shape of lateral load pattern. It is concluded that proposed load pattern results are closer to nonlinear dynamic analysis (NDA) compared to other pushover load patterns especially in tall and medium-rise buildings having different stiffness and mass during the height.

Keywords: pushover load pattern; nonlinear dynamic analysis; steel frames; tall and medium-rise buildings; load distribution

1. Introduction

Nonlinear static methods are simplified procedures in which the problem of evaluating the maximum expected response of a MDOF system for a specified level of earthquake motion is replaced with response evaluation of its equivalent SDOF system. The common features of these procedures are the application of pushover analysis to characterize the structural system.

The nonlinear static procedures (NSPs) using the lateral force distributions recommended in ATC-40 (1996) and the FEMA-356 (2000) documents are now widely used as a means of seismic demands estimation and buildings assessment in engineering practice. The nonlinear static procedure in these documents is based on the capacity spectrum method (ATC-40) and displacement coefficient method (FEMA-356), which assumes that the lateral force distribution for the pushover analysis is based on the fundamental vibration mode of the elastic structure. Consequently, these NSPs based on invariant load patterns provide accurate seismic demand estimates only for low- and medium-rise moment-frame buildings where contributions of higher modes response are not significant (Chintanapakdee *et al.* 2003, Chopra *et al.* 2002, Bobadilla *et*

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al. 2007, Krawinkler *et al.* 1999, Gupta 1999, Kunnath *et al.* 2004, Cosenza *et al.* 2006, Gonzales 2012, Duan 2012, Carvalho 2013, Bayat *et al.* 2011a, 2011b). Correcting the mentioned drawbacks, a promoted pushover procedure, called modal pushover analysis (MPA), was successfully applied by Chopra *et al.* (2002) to take into account higher modes contributions. An improved modal pushover analysis (IMPA) procedure has been recently proposed by Jianmeng *et al.*, to consider the redistribution of inertia forces after the structure yields (Jianmeng *et al.* 2008). The IMPA procedure uses the product of time variant floor displacement vector (as the displacement shape vector) and the structural mass matrix as the lateral force distribution at each applied-load step beyond the yield point of structure.

An alternative pushover analysis method to estimate seismic displacement demands, as the mass proportional pushover (MPP) procedure, was proposed by Kim and Kurama (2008). The main advantage of the MPP is that the effects of higher modes on the lateral displacement demands are lumped into a single invariant lateral force distribution which is proportional to the total seismic masses at the floor and roof levels. However, the accuracy of MPA, IMPA and MPP procedures is not appropriate in some cases especially in tall buildings. Although the modal Pushover, improved modal pushover and mass proportional analysis results are widely used nowadays, these results are not accurate in tall and medium-rise buildings having different stiffness and mass during the height of structure. In this study, a new efficient and accurate load pattern is proposed for pushover analysis in mentioned structure types and the obtained results are compared with other pushover analysis and nonlinear dynamic results.

2. Pushover procedure

In Nonlinear Static Procedure, the basic demand and capacity parameter for the analysis is the lateral displacement of the building. The generation of a capacity curve (base shear vs roof displacement) defines the capacity of the building uniquely for an assumed force distribution and displacement pattern depending upon no specific seismic shaking demand and replaces the base shear capacity of conventional design procedures as is shown in Fig. 1 (FEMA-274 1997). If the building displaces laterally, its response must lie on this capacity curve. A point on the curve defines a specific damage state for the structure, since the deformation for all components can be related to the global displacement of the structure. By correlating this capacity curve to the seismic demand generated by a specific earthquake or ground shaking intensity, a point can be found on the capacity curve which estimates the maximum displacement of the building caused by an earthquake. This defines the performance point or target displacement. The location of this performance point relative to the performance levels defined by the capacity curve indicates whether or not the performance objective is met (FEMA-274 1997).

Based on the standard Pushover assumptions, a recently proposed method known as Modal Pushover Analysis (MPA) includes the effects of higher modes to the response of the structure and in most cases decreases bias to the prediction of above parameters (Chopra *et al.* 2001, 2002). According to MPA, which is based on structural dynamics theory, the structure is subjected to monotonically increasing lateral forces based on a specific mode, until a target roof displacement is reached (Fig. 2). In order to simplifying, these forces were kept invariant during the time. Chintanapakdee and Chopra (2003), Lignos and Gantes (2005) have evaluated MPA using vertically regular and irregular structures with respect to mass and stiffness and it was concluded that MPA can accurately predict engineering demand parameters in comparison to nonlinear time

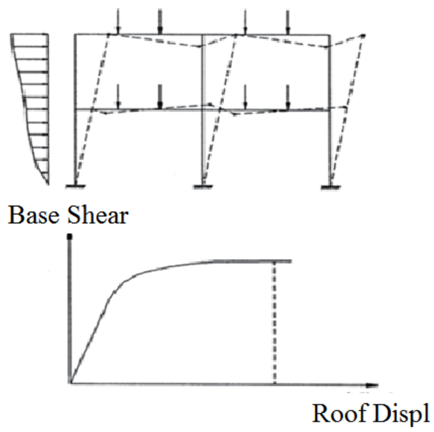


Fig. 1 Pushover illustration

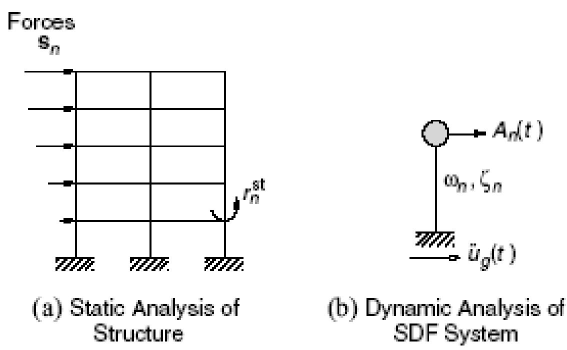


Fig. 2 Conceptual explanation of modal RHA of elastic MDF systems

history analysis. Different lateral load patterns of FEMA-273 are as follows (a-d), (FEMA-273 2000)

a. Uniform distribution:

$$S_J = \frac{m_j}{\sum_{i=1}^{N} m_i} \tag{1}$$

b. Equivalent lateral force (ELF) distribution:

$$S_J = \frac{m_j h_j^k}{\sum_{i=1}^{N} m_i h_j^k} \tag{2}$$

c. Mode1 deformed shape pattern

d. SRSS distribution: A vertical distribution Proportional to the storey shear distribution calculated by combining modal responses of a response spectrum analysis of the building, Including sufficient modes to capture at least 90% of the total building mass, and using the appropriate ground motion spectrum.

e. Modal Pushover: MPA procedure is an improved pushover method proposed to estimate seismic demands of buildings taking into account the higher mode effects and retaining the simplicity of invariant load distributions. Modal pushover analysis (MPA) utilizes the concept of modal combinations through several pushover analysis using invariant load patterns based on elastic mode shapes where the total response is determined with combination of each mode at the end.

2.1 New proposed pattern (SM)

New Proposed pattern including Stiffness, weight and mode shape of structure

$$S_i = \frac{m_i \times \psi_i^n \times h_i \times k_i}{\sum_{k=1}^{N} m_k \times \psi_k^n \times h_k \times k_k} \tag{3}$$

Where: S_i is lateral load at Storey i , m_i is weight at storey i, ψ_i^n is Modal displacement of storey i at mode n, h_i is the height of storey i, k_i is Storey Stiffness i and N is the number of building stories.

For this investigation, number of modes for this approach is equal to the number of modal pushover modes. (Number of modes that have 90% mass participating ratio)

For moment resisting frames

$$k_i = \sum_{m=1}^{Q} \frac{E \times I_{im}}{12 \times h_i^3} \tag{4}$$

Where E is Elastic module, I_{im}^n is moment of inertia of columns at storey i and Q is the Number of columns in storey i.

For X type braced frames

$$k_i = \sum_{m=1}^{Q} \frac{E \times A_{im}}{l_{im}} Cos^2 \theta_m \tag{5}$$

Where A_{im} is Area of bracings at storey i, l_{im} is length of bracings at Storey i and θ is the angle of bracings.

For example, using mode n, the obtained lateral load pattern is named SM_n. For different numbers of modes, it is named SM pattern. Figs. 3, 6, 9 and12 show mode shape 1 and its changes when SM approach (named SM1) is used.

3. Modeling and analysis

Nonlinear response history analysis (NL-RHA) and nonlinear static analysis are applied for 12 and 18 Storey buildings for two cases (Table 1). The first case is when frames are using x type bracings for lateral resistance and Second one is when frames are using moment resisting connections.

Table 1 Characteristics of analytical cases

Frame Name	Lateral Resistance Type	Storey Numbers	Period (second)			Modal Participating Mass Ratio		
			T_1	T_2	T_3	U_1	U_2	U_3
12B	X type bracing	12	0.836	0.241	0.16	0.648	0.215	0.0001
18B	X type bracing	18	1.433	0.386	0.224	0.626	0.220	0.00002
12M	Moment resistance connection	12	1.322	0.497	0.297	0.75	0.08	0.083
18M	Moment resistance connection	18	2.166	0.95	0.59	0.6075	0.196	0.04823

Table 2 Frames height and weight

No.	Storey Name	Storey height (m)	Frame Storey weight (Ton)
1	Storey 1~ Storey 5	3	24
2	Storey 6	6	48
3	Storey 7 ~ Storey 11	3	24
4	Storey 12	6	48
5	Storey 13 ~ Storey 17	3	24
6	Storey 18	6	48

Table 3 Frame section properties

Frame Name	Item	Frame Sections				
		Story 1~4	Story 5~8	Story 9~12	Story 13~15	Story 15~18
12B	Column	Box 500x12	Box 500x12	Box 400x10	-	-
	Beam	PG2	PG2	PG2	-	-
	Brace	2UNP 180	2UNP 140	2UNP 120	-	-
12M	Column	Box 500x12	Box 500x12	Box 400x10	-	-
	Beam	PG1	PG2	PG2	-	-
	Brace	-	-	-	-	-
18B	Column	Box 500x20	Box 500x15	Box 500X12	Box 400X12	Box 400X10
	Beam	PG1	PG1	PG1	PG2	PG2
	Brace	2UNP 240	2UNP 220	2UNP 200	2UNP 180	2UNP 140
18M	Column	Box 500x20	Box 500x15	Box 500X12	Box 400X12	Box 400X10
	Beam	PG1	PG1	PG2	PG2	PG2
	Brace	-	-	-	-	-

All dimensions are in millimeter
PG 1(Plate girder 1) - Flange : PL. 280X20 Web: PL.500x10
PG 2(Plate girder 2)- Flange : PL. 200X20 Web: PL.400x10

Frames have different storey weights, and have stories with variable height which is needed to be concerned in order to weight and stiffness variation considerations. For all models, 6th, 12th and18th stories have 2 m weight and 2 h storey height when other storey weight is m and storey height is h (Table2). All buildings are designed according AISC 360-05. All frame sections including beams, columns and braces are specified in Table 3.

SAP2000 software is employed to monitor nonlinear static and dynamic analysis methods. Lateral load patterns for nonlinear static method is according to Uniform (U), Equivalent lateral force (ELF), mode 1 deformed shape (M1) ,Modal pushover (MP) and proposed stiffness and weight combining modal shapes (SM). For exact comparison, number of modes for SM is assumed equal to modal pushover approach.

Two types of analysis are performed including nonlinear response history analysis (NL-RHA) and pushover analysis (using the Gravity loads) and P-δ effects are taken into account in all cases. Pushover analysis is performed by first applying gravity loads followed by monotonically increasing lateral forces with a specified distribution.

Each structural frame model is subjected to an ensemble of seven ground motion acceleration records, as shown in Table 4. The earthquake inputs have been chosen from pacific earthquake engineering center in such a way that a wide range of frequency contents as well as the effects of both near and far fault distances are taken into account in the analysis (Peer, Berkeley).

Table 4 Ground motion data set considered in this study

No.	Earthquake Name	Magnitude	Station	PGA(g)	Fault distance to station(Km)
1	San Fernando	6.6	Pacoima Dam	0.366	3
2	Mexico	6.9	Corro prieto	0.587	12
3	Northridge	6.7	Rinaldi	0.474	7.1
4	Loma perita	7	Gilroy Historic	0.450	12.7
5	Kobe	6.9	Takatori	0.821	4.3
6	Imperial Valley	6.5	El Centro	0.341	0.3
7	Chichi	7.6	TCU065	0.808	2.5

All analyzes are performed to achieve the target point of structures. Pushover and NL-RHA analysis load combination based on FEMA 273 and 356 is

$$1.1\times(\text{Dead Load} + \text{Live Load}) + \text{Lateral Load} \tag{6}$$

The nonlinear force-displacement or moment-rotation behavior occurs in discrete hinges for nonlinear static and nonlinear response history analysis. Hinges are introduced into frame elements and assigned at end location along the frame elements. Also, hinge properties are introduced based on FEMA-273 criteria. Coupled P-M3 hinge based on the interaction of axial force and bending moments, M3 hinge based on only bending moment and P hinge based on only axial force are assigned at the hinge location of columns, beam elements and bracings.

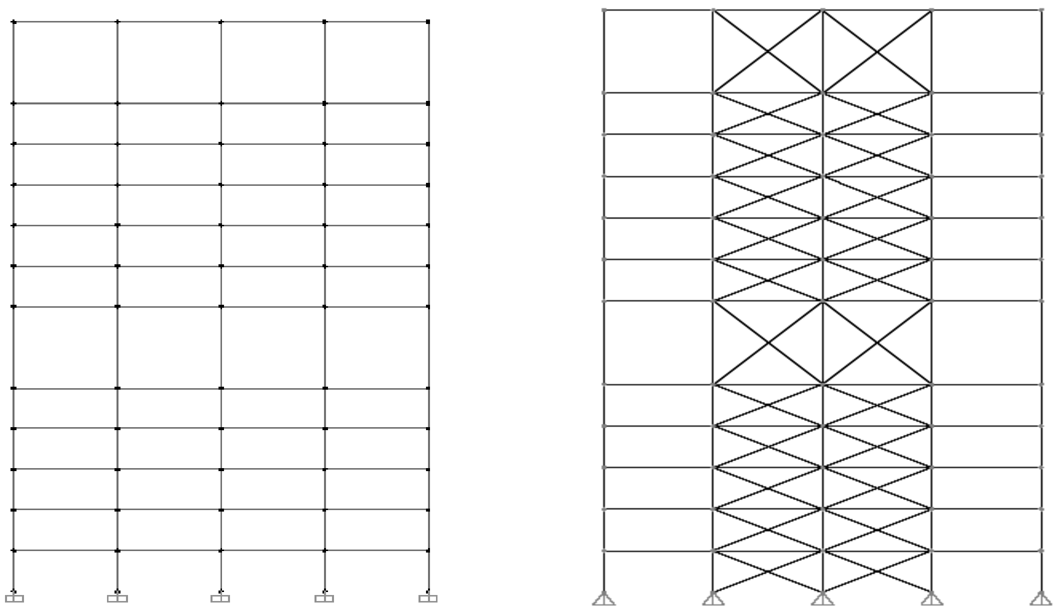
The results of all analyzes are briefly presented. Shape of new proposed pattern, Floor displacements, Story drift ratios (story drifts / height of story) are calculated by mentioned procedure.

The results of pushover analysis are obtained at the target displacements. Also, the errors of pushover analysis relative to exact solutions (benchmark results) obtained by NL-RHA, are extracted.

To establish the target displacement, either a capacity spectrum approach [ATC-40] or a displacement coefficient approach (FEMA 273/356) is utilized. Also, target displacement is assumed equal to the maximum dynamic roof displacement.

12B model frame, M1 and SM1 shapes and results are shown in Figs. 3 to 6. Similarly, Figs. 6 to 8, 9 to 11 and 12 to 14 are for 12M, 18B and 18M models respectively.

Having a more scrutinized comparison between all load pattern distributions in frame height, the Tables 5 to 8 show the load acting on frames 12B, 12M, 18B and 18M in all load patterns respectively.



(a) (b)

Fig. 3 The scheme of the 12 story structure: (a) 12M and (b) 12 B

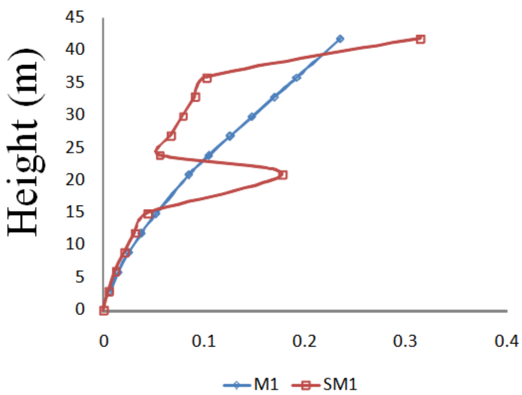


Fig. 4 Comparison of the load patterns of Mode 1 shape and SM1 for 12B structure

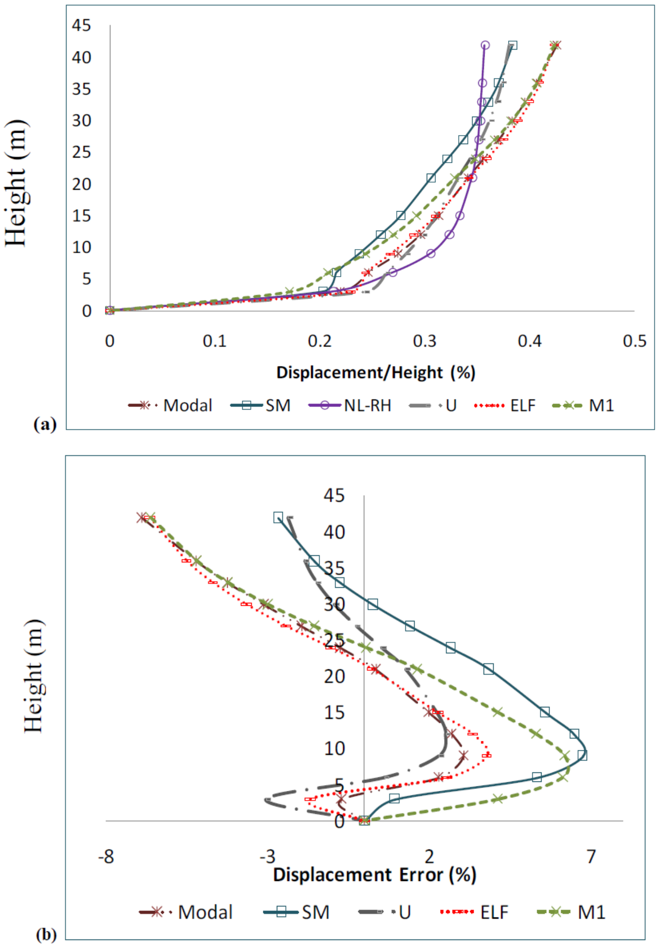


Fig. 5 Variation of displacement and displacement error for 12B building versus height

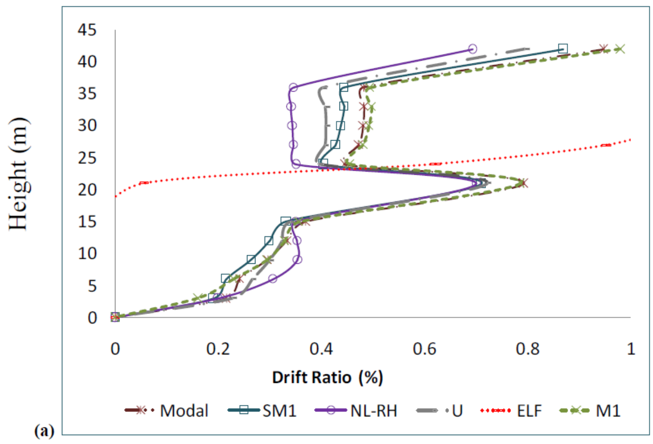
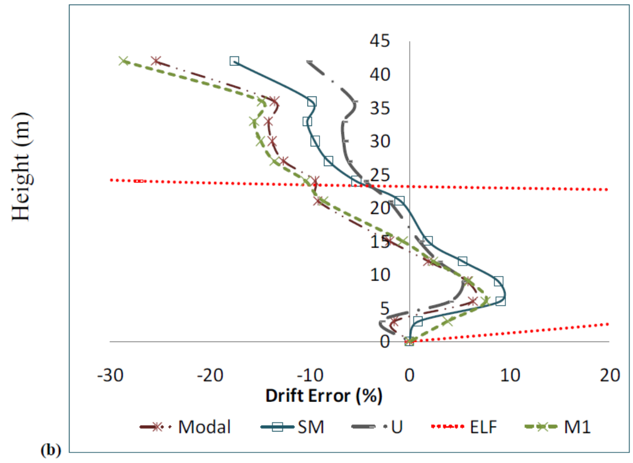


Fig. 6 Variation of drift & drift error for 12B building versus height



(b)

Fig. 6 Continued

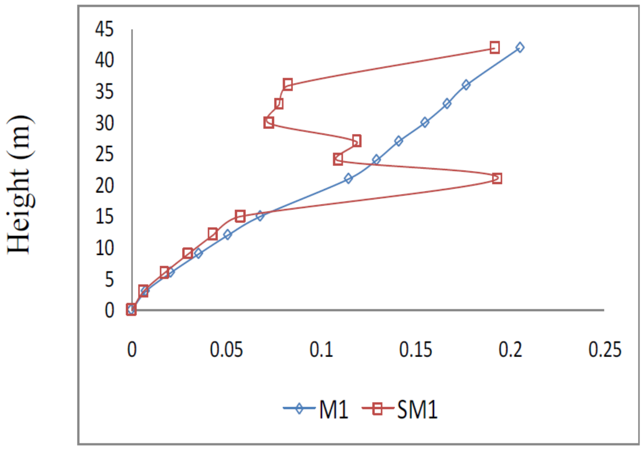
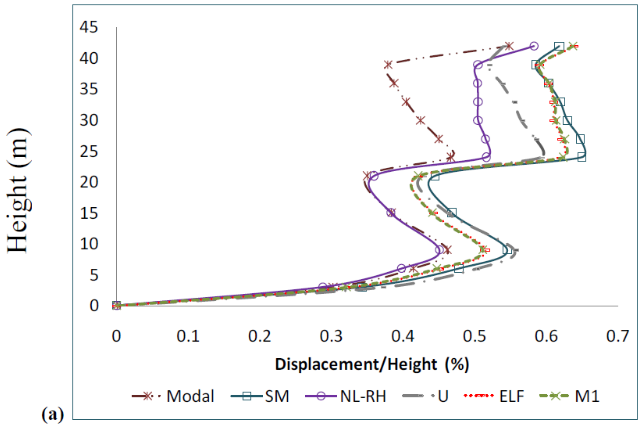


Fig. 7 Comparison of the load patterns of Mode 1 shape and SM1 for 12M structure



(a)

Fig. 8 Variation of displacement & displacement error for 12M building versus height

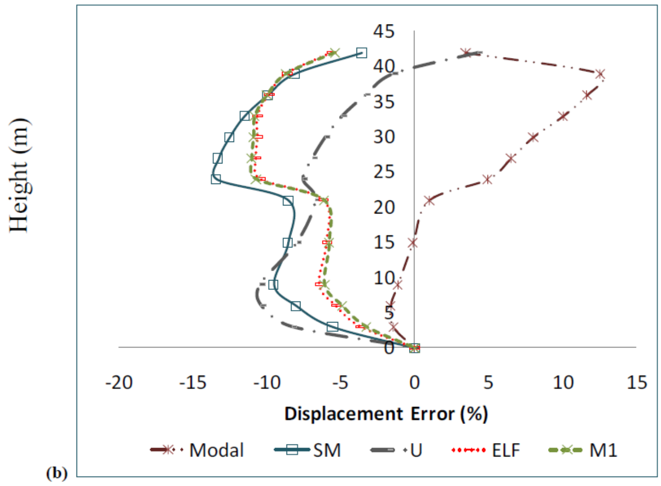


Fig. 8 Continued

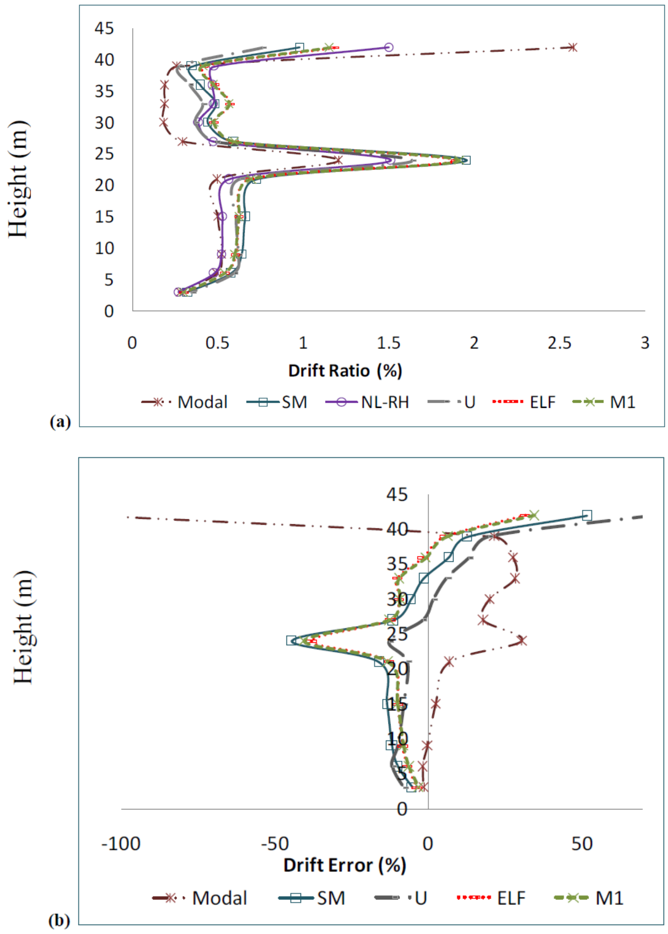
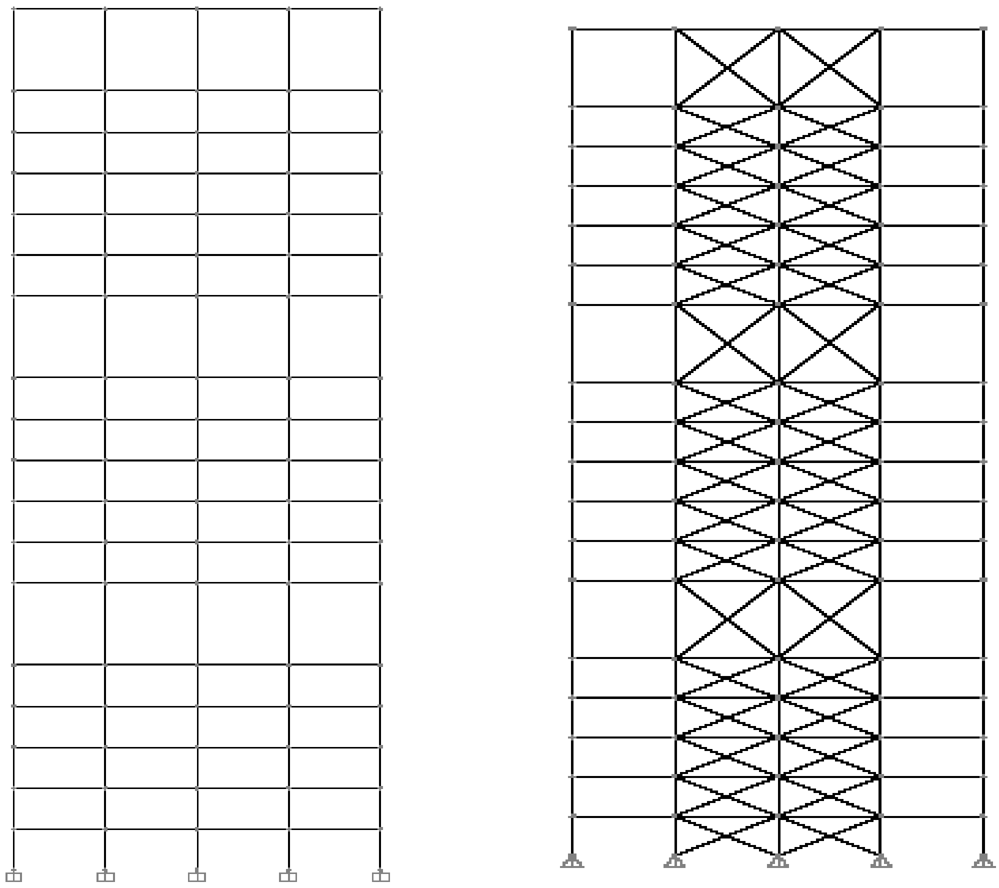


Fig. 9 Variation of drift & drift eror for 12M building versus height



(a) (b)

Fig. 10 The scheme of the 18 story structure: (a) 18M and (b) 18 B

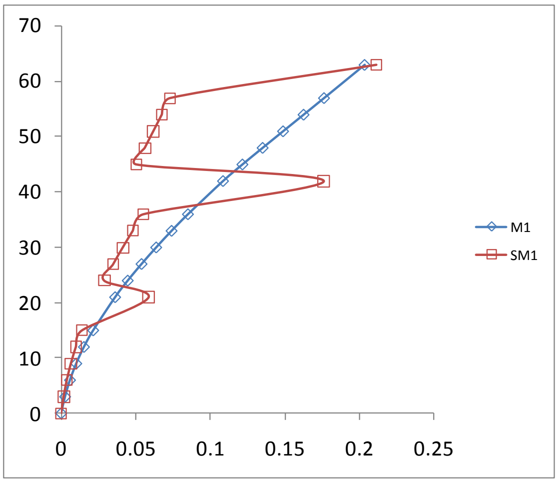


Fig. 11 Comparison of the load patterns of Mode 1 shape and SM1 for 18B structure

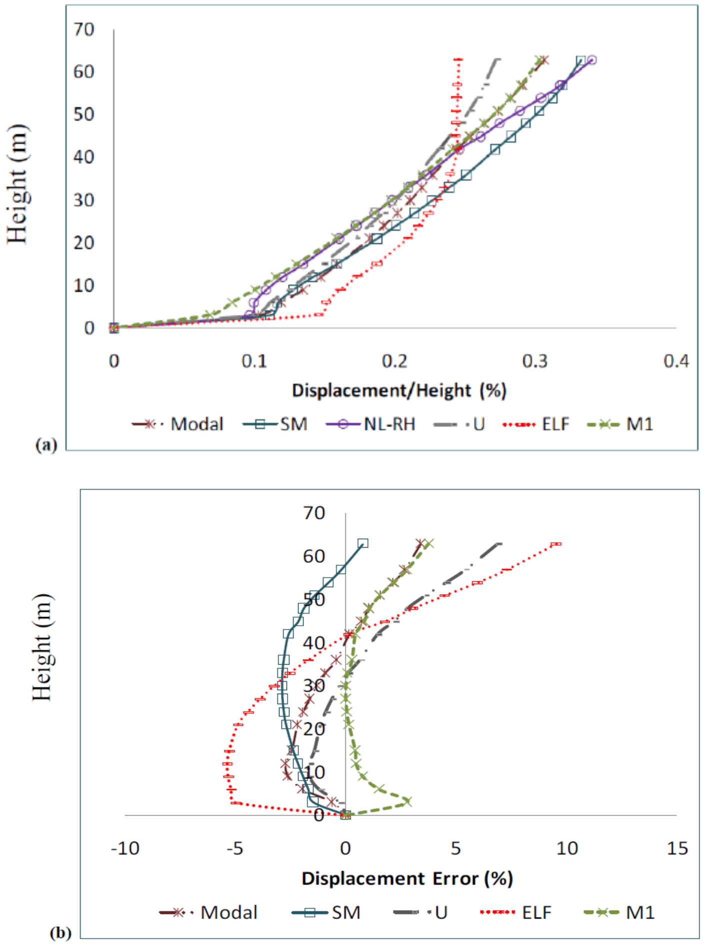


Fig. 12 Variation of displacement & displacement error for 18B building versus height

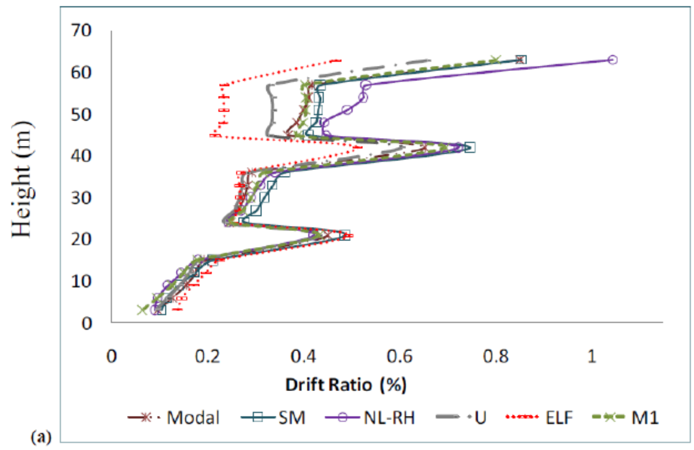
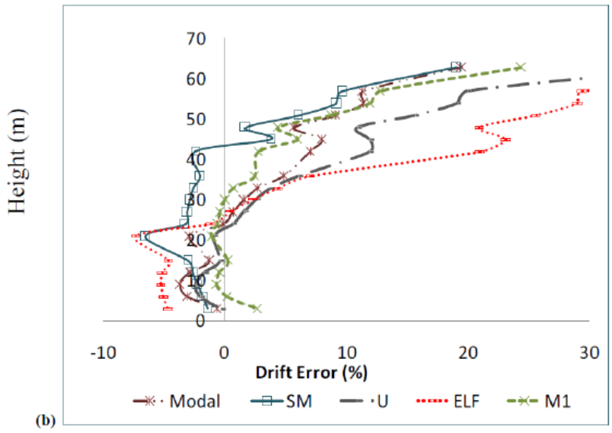


Fig. 13 Variation of drift & drift error for 18B building versus height



(b)

Fig. 13 Continued

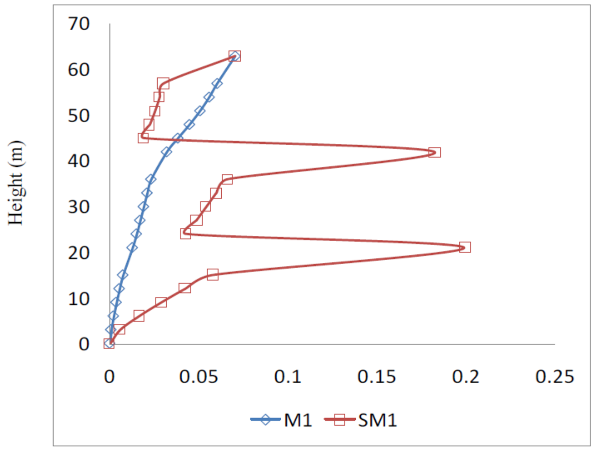
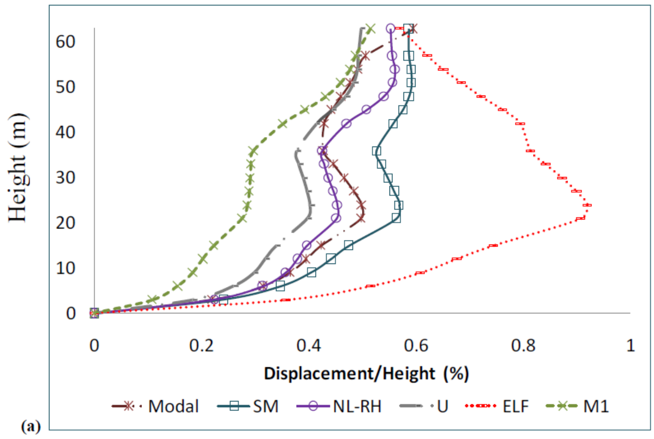


Fig. 14 Comparison of the load patterns of Mode 1 shape and SM1 for 18M structure



(a)

Fig. 15 Variation of displacement & displacement error for 18M building versus height

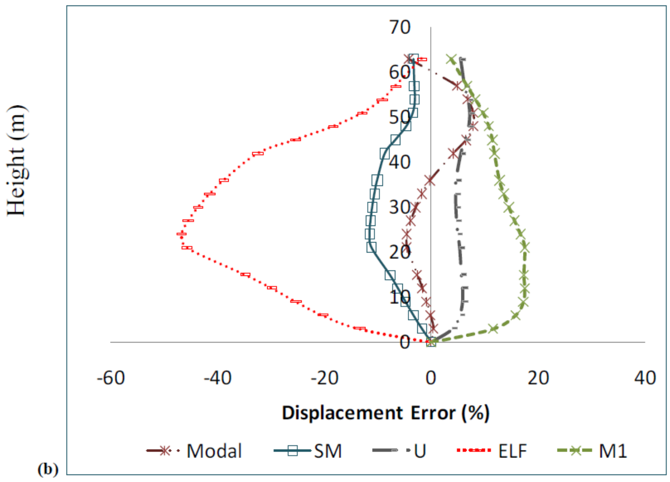
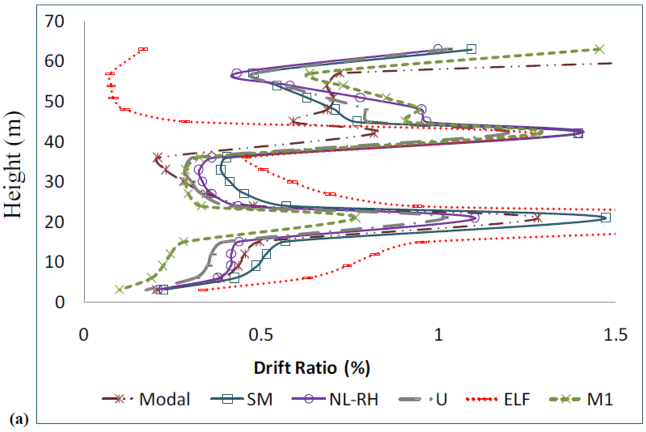


Fig. 15 Continued



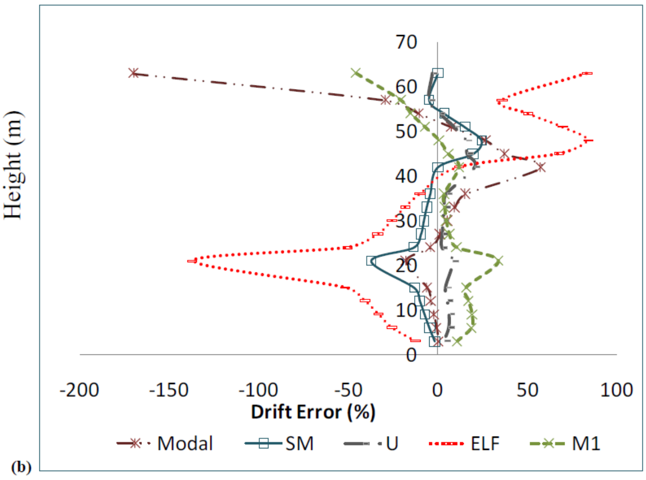


Fig. 16 Variation of drift & drift error for 18M building versus height

Table 5 Load acting frame height 12B in all load patterns

		12B			
Height (m)	ELF	MODAL	U	SM1	M1
42	0.273790959	0.27458354	0.092	0.313885	0.235282
36	0.19131757	0.14708299	0.092	0.102356	0.19181
33	0.155240345	0.13019693	0.092	0.090605	0.169789
30	0.122758201	0.1132503	0.092	0.078812	0.147689
27	0.094151165	0.09650993	0.092	0.067162	0.125858
24	0.06960418	0.08026647	0.092	0.055858	0.104675
21	0.049502004	0.13121311	0.092	0.177347	0.085079
15	0.021764813	0.04801806	0.092	0.043666	0.05237
12	0.012589596	0.03471934	0.092	0.031573	0.037866
9	0.006296543	0.02315264	0.092	0.021054	0.025251
6	0.002443876	0.01347935	0.092	0.012258	0.014701
3	0.000540775	0.00596535	0.092	0.005425	0.006506
0	0	0	0	0	0

Table 6 Load acting on frame height 12M in all load patterns

		12M			
Height (m)	ELF	MODAL	U	SM1	M1
42	0.235782485	0.17878609	0.0978	0.191913	0.205389
36	0.174094134	0.11681454	0.0978	0.08266	0.176928
33	0.150469032	0.11014086	0.0978	0.077937	0.16682
30	0.127205215	0.10242334	0.0978	0.072476	0.155131
27	0.104256929	0.11709355	0.0978	0.118936	0.141272
24	0.084975455	0.1073678	0.0978	0.109057	0.129538
21	0.065866506	0.1385864	0.0978	0.193218	0.114752
15	0.027900426	0.05640419	0.0978	0.057292	0.068051
12	0.016705406	0.04221508	0.0978	0.042879	0.050932
9	0.008739983	0.02944827	0.0978	0.029912	0.035529
6	0.003400809	0.0171879	0.0978	0.017458	0.020737
3	0.000609823	0.00616417	0.0978	0.006261	0.007437
0	0	0	0	0	0

Table 7 Load acting on frame height 18B in all load patterns

		18B			
Height	ELF	MODAL	U	SM1	M1
63	0.191072155	0.16602376	0.077	0.21135	0.203709
57	0.149763901	0.09988574	0.077	0.073238	0.176476
54	0.130850304	0.09211963	0.077	0.067544	0.162755

Table 7 Continued

51	0.113126718	0.08432691	0.077	0.06183	0.148987
48	0.096665188	0.07655967	0.077	0.056135	0.135264
45	0.081535391	0.06888186	0.077	0.050506	0.121699
42	0.067951273	0.11393238	0.077	0.176163	0.108668
36	0.045623212	0.0561269	0.077	0.055196	0.085121
33	0.036418279	0.04887572	0.077	0.048065	0.074124
30	0.028492286	0.04206237	0.077	0.041365	0.063791
27	0.021686716	0.03557278	0.077	0.034983	0.053949
24	0.015963649	0.02945836	0.077	0.02897	0.044676
21	0.011324066	0.03797362	0.077	0.058715	0.036219
15	0.004797027	0.01416344	0.077	0.013929	0.02148
12	0.002728499	0.01007002	0.077	0.009903	0.015272
9	0.001356835	0.00667686	0.077	0.006566	0.010126
6	0.000528209	0.0038989	0.077	0.003834	0.005913
3	0.000117558	0.00173548	0.077	0.001707	0.002632
0	0	0	0	0	0

Table 8 Load acting on frame height 18M in all load patterns

			18M		
Height (m)	ELF	MODAL	U	SM1	M1
63	0.200882679	0.07738032	0.025	0.070383	0.070308
57	0.155607941	0.04967864	0.025	0.03013	0.060195
54	0.136426111	0.04597445	0.025	0.027883	0.055707
51	0.117010987	0.04175123	0.025	0.025322	0.05059
48	0.097257605	0.03687186	0.025	0.022362	0.044677
45	0.077884931	0.03149588	0.025	0.019102	0.038163
42	0.060804336	0.11799575	0.025	0.182616	0.031922
36	0.037796337	0.04915193	0.025	0.066217	0.02315
33	0.031300231	0.0444045	0.025	0.059821	0.020914
30	0.025793449	0.04025145	0.025	0.054226	0.018958
27	0.020810919	0.03608451	0.025	0.048613	0.016995
24	0.016289812	0.03177593	0.025	0.042808	0.014966
21	0.012054904	0.11663688	0.025	0.19941	0.012658
15	0.004995345	0.03585197	0.025	0.057842	0.007343
12	0.002926948	0.02625866	0.025	0.042365	0.005378
9	0.001494532	0.01787728	0.025	0.028843	0.003662
6	0.000565989	0.01015537	0.025	0.016384	0.00208
3	9.79802E-05	0.00351606	0.025	0.005673	0.00072
0	0	0	0	0	0

4. Conclusions

(a) - Comparison of several load patterns for moment resisting frames with weight and stiffness variation in height indicates that displacements and drift ratios of SM pattern are the nearest to NL-RH results range compared to other lateral load patterns. The SM results are closer to NL-RH as the number of mode shapes is increased and exact stiffness of storey is taken into account.

Considering obtained results, applied lateral load patterns and the time history analysis are listed below in results accuracy order from 1 to 6

1-SM (New Proposed Method)

2- Modal Pushover

3-Uniform (U)

4-Mode1 (M1)

5-Equivalent lateral force (ELF)

6- Non-Linear Response History Analysis (NL-RHA)

Differences of SM and modal pushover are indicated in upper stories; it is observed that for upper stories of 18M, SM drift ratio tolerances are smaller than modal pushover.

(b) - For x type braced steel frames, drift ratios of pushover are conservative for upper stories in 12B building while a reverse pattern is observed in 18B buildings. Thus, it is clear that for braced frames, pushover displacements are Conservative but drifts are underestimated toward NL-RH results because of pushover loads direction. For these frames, applied lateral load patterns are listed according results accuracy order

1-SM & modal pushover

2-Uniform (U)

3-Mode1 (M1)

4-Equivalent lateral force (ELF)

Considering above results, SM method can be employed as one of accurate lateral load patterns of pushover approach.

The mentioned pattern considers Uniform and mode shape patterns in m_i and ψ_i^n parameters. Storey height and stiffness variation are inserted in term K and can be employed specially for tall buildings that have stiffness and weight variation in height.

Results show that ELF has no parameters of frame stiffness and weight variation resulting in big tolerances especially for moment resisting frames. Uniform (U) and Mode shape 1 (M1) lateral load patterns have more reasonable results than ELF while results have tolerances for stories including mass variation. In modal pushover, drift ratios for upper stories of 12M and 18M have tolerances with NL-RH results. SM pattern considers U and M patterns, modified modal pushover and stiffness variation in height. An advantage of SM pattern is the number of modes that should be used in such a way that the optimum accuracy is reached. In this pattern, for 18B and 18M, using only 2 modes can reach accuracy while 3 modes are needed in Modal pushover.

The SM can be applied as a proper method for all Mass participating ratios especially when mass participation ratio of first modes is smaller than 61%while other pushover methods can’t reach reasonable results.

References

ATC Seismic evaluation and retrofit of concrete buildings (1996), Volumes 1-2, ATC-40, Applied Technology Council, Redwood City, California.

ASCE Prestandard and commentary for the seismic rehabilitation of building (2000), FEMA-356, American Society of Civil Engineers (ASCE), Federal Emergency Management Agency, Washington, DC.

Bayat, M. and Abdollahzade, G.R. (2011a), "Analysis of the steel braced frames equipped with ADAS devices under the far field records", *Lat. Am. J. Solids Struct*., **8**, 163-181.

Bayat, M. and Abdollahzadeh, G.R. (2011b), "On the effect of the near field records on the steel braced frames equipped with energy dissipating devices", *Lat. Am. J. Solids Struct*., **8**, 429-443.

Bobadilla, H. and Chopra, A.K. (2007), "Modal pushover analysis for seismic evaluation of reinforced concrete special moment resisting frame buildings", Report no. EERC 2007-01, Earthquake Engineering Research Center, University of California, Berkeley, California.

Building Seismic safety Council (BSSC) (1997), NEHRP guidelines for the seismic rehabilitation of buildings, FEMA-274, Federal Emergency Management Agency, Washington , D.C.

Building Seismic Safety Council (BSSC) (2000), Pre standard and commentary for the seismic rehabilitation of buildings, FEMA-356, Federal Emergency Management Agency, Washington, D.C.

Carvalho, G., Bento, R. and Bhatt, C. (2013), "Nonlinear static and dynamic analyses of reinforced concrete buildings – comparison of different modelling approaches", *Earthq. Struct*., **4**(5), 451-470.

Cosenza, E., Manfredi, G. and Verderame, G. (2006), "A fibre model for push-over analysis of underdesigned reinforced concrete frames", *Comput. Struct*., **84**(13-14), 904-916.

Chopra, A.K. and Goel, R.K. (2001), "A modal pushover analysis procedure to estimating seismic demands for buildings: theory and preliminary evaluation", Report No. PEER 2001/03, Pacific Earthquake Engineering Research Center, University of California, Berkeley.

Chintanapakdee, C. and Chopra, A.K. (2003), "Evaluation of modal pushover analysis using generic frames", *Earthq. Eng.Struct. Dyn*., **32**(3), 417-442.

Chopra, A.K. and Goel, R.K. (2002), "A modal pushover analysis procedure for estimating seismic demands for buildings", *Earthq. Eng. Struct. Dyn*., **31**(3), 561-582.

Duan, H. and Hueste, M.B. (2012), "Seismic performance of a reinforced concrete frame building in China", *Eng. Struct*., **41**, 77-89.

FEMA 273 (1997), "NEHRP Guidelines and commentary for the seismic rehabilitation of buildings", Technical Report FEMA. 273 (Guidelines), Federal Emergency Management Agency, Washington, D.C.

Gonzales, H. and López-Almansa, F. (2012), "Performance of buildings with thin RC bearing walls", *Eng. Struct.*, **34**, 244-258.

Gupta, A. and Krawinkler, H. (1999), "Seismic demands for performance evaluation of steel moment resisting structures (SAC Task 5.4.3)", Report No. 132, John A. Blume Earthquake Engineering Center, Stanford University, California.

Guner, S. and Vecchio, F.J. (2010), "Pushover analysis of shear-critical frames verification and application", *ACI Struct. J*., **107**(1), 72-81.

Jianmeng, M., Changhai, Z. and Lili, X. (2008), "An improved modal pushover analysis procedure for estimating seismic demands of structures", *Earthq. Eng. Eng. Vib*., 7(1), 25-31.

Krawinkler, H. and Seneviratna, G.D.P.K. (1999), "Pros and cons of a pushover analysis of seismic performance evaluation", *Eng. Struct*., **20**(4-6), 452-464.

Kunnath, S.K. and Kalkan, E. (2004), "Evaluation of seismic deformation demands using nonlinear procedures in multistory steel and concrete moment frames", *ISET J. Earthq. Technol*., **41**(1), 159-182.

Kim, S.P. and Kurama, Y.C. (2008), "An alternative pushover analysis procedure to estimate seismic displacement demands", *Eng. Struct*., **30**(12), 3793-3807.

Kyrkos, M.T. and Anagnostopoulos, S.A. (2011), "Improved earthquake resistant design of torsionally stiff asymmetric steel buildings", *Earthq. Struct*., **2**(2), 127-147.

Lignos, D.G. and Gantes, C.J. (2005), "Modal pushover analysis as a tool for practical design of structures",

Proc., 3rd conference on Mechanics and Solids, MIT, Paper no. 078-271.
Pacific Earthquake Engineering Research Center (PEER), http://peer.berkeley.edu/.
Triantafyllou, S. and Koumousis, V. (2013), "Hysteretic finite elements for the nonlinear static and dynamic analysis of structures", *J. Eng. Mech*., 10.1061/(ASCE)EM.1943-7889.0000699.

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