Assessment of conventional nonlinear static procedures with FEMA load distributions and modal pushover analysis for high-rise buildings

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Abstract: *The nonlinear static pushover analysis technique is mostly used in the performance-based design of structures and it is favored over nonlinear response history analysis. However, the pushover analysis with FEMA load distributions losses its accuracy in estimating seismic responses of long period structures when higher mode effects are important. Some procedures have been offered to consider this effect. FEMA and Modal pushover analysis (MPA) are addressed in the current study and compared with inelastic response history analysis. These procedures are applied to medium high-rise (10 and 15 storey) and high-rise (20 and 30 storey) frames; efficiency and limitations of them are elaborated. MPA procedure present significant advantage over FEMA load distributions in predicting storey drifts, but the both are thoroughly unsuccessful to predict hinge plastic rotations with acceptable accuracy. It is demonstrated that the seismic demands determined with MPA procedure will be unsatisfactory in nonlinear systems subjected to individual ground motions which inelastic SDF systems related to significant modes of the buildings respond beyond the elastic limit. Therefore, it's inevitable to avoid evaluating seismic demands of the buildings based on individual ground motion with MPA procedure.*

Keywords: *Modal pushover analysis, FEMA load distributions, nonlinear response history analysis, high-rise buildings*

Introduction

Nonlinear static analysis is a simplified analysis procedure that can be useful for estimating seismic demands and providing valuable information about the locations of structural weaknesses and failure mechanisms in the inelastic range.[[Krawinkler , H., and seneviratna G.D.P.K., 1998]. Also pushover analysis has the advantage that it is capable of considering a response spectrum of codes as demand diagram to estimate the earthquake induced response of structures [Chopra and Goel, 1999]. This method is recommended as a standard tool for assessment proposes and design verification [FEMA 273].

However this procedure contains several principal limitations [Kim S. and D'Amore, E., 1999]. One of the momentous deficiencies of this procedure is that it is based on the assumption that the response quantity of interest is driven primarily by response in a single mode and inelastic action is uniformly distributed over the height of the building [Krawinkler , H., and seneviratna G.D.P.K., 1998]. This assumption is valid only for low-rise buildings. Then several authors have been developing pushover-based methods, which do represent a major step forward in comparison with standard equivalent elastic procedures and take into account the influence of higher modes. The method in which higher modes are taken into account varies from procedure to procedure.

Adaptive pushover procedures consider progressive stiffness degradation and consequently changes in the modal properties at each step on the lateral story forces. This provides improved response predictions [Bracci et al.1997; Gupta, B., and Kunnath, S. K. _2000]. However, it was demonstrated that force-based adaptive pushover is not more preferable over current non-adaptive pushover analyses [Antoniou , S. and Pinho , R., 2004a]. A displacement-based adaptive pushover procedure was proposed that lateral displacements instead of forces are monotonically applied to the structure. It was concluded that this procedure results in better estimates of seismic demands in relation to force-based analyses [Antoniou , S. and Pinho , R., 2004b]

Multi-mode pushover (MMP) was identified, but the higher mode effects wasn't quantified [Sasaki, K.K., Freeman, S.A. and Paret, T.F., 1988]. Pushover results combinations (PRC) was proposed that estimates the maximum seismic responses by combining the results of pushover analyses and utilizing a mode shape as its load pattern in each analysis [Moghadam, A.s. , 2002].

Incremental response spectrum analysis (IRSA) has been developed. The IRSA method addresses the issue of higher modes using piecewise elastic (spectral) modal analysis with constant structural and dynamic properties between the occurrences of the two successive plastic hinges, using consistent theoretical approach [Aydinoglu, M.N., 2003].

In other studies modal pushover analysis (MPA) was proposed that pushover analyses are carried out separately for each significant mode, and the contributions from individual modes to calculated are combined using an appropriate combination rule (SRSS or CQC) to calculate response quantities (displacements, drifts, etc.). Of course, the rule of superposition of modal responses is not reasonable in the inelastic range of the response because modes are not uncoupled anymore. It was shown that MPA procedure is capable of estimating displacements and storey drifts with acceptable accuracy but it fails to provide satisfactory results for plastic rotations of the hinges [Chopra , A.K. and Goel, R.K., 2002]. In another research, it was demonstrated that MPA procedure is more reliable than FEMA load distributions in computing seismic demands of SAC (9 and 20 storey) buildings [Goel , R.K., and Chopra, A.K., 2004].

Also, a modified version of MPA (MMPA) has been proposed in which the inelastic response obtained from first-mode pushover analysis has been combined with the elastic contribution of higher modes [Chopra, A. K., Goel, R. K., and Chintanapakdee, C. 2004].

The above-mentioned discussion clarifies the importance of higher mode effects in pushover analyses of long period structures in the recent studies. In view of this matter, MPA has achieved a preference over current procedures due to simplicity and attractiveness. So it's necessary to be verified for a wide range of buildings and ensembles of ground motions. The principle objective of this paper is to appraise modal pushover analysis (MPA) and elaborate attentively its effectiveness and limitations. Additionally the degree of accuracy of FEMA force distributions in pushover analysis will be investigated in this paper. For that reason, MPA and FEMA force distributions will be applied to four steel special moment-resisting high-rise frames possessing various heights. It is noted that the seismic demands produced by nonlinear response history analyses are treated as benchmark results and will be compared with those obtained by approximate pushover procedures.

Details of MPA procedure

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

Details of MPA procedure are illustrated as a series of following steps [Chopra , A.K. and Goel R.K., 2002]:

1. Compute the natural frequencies, w_n , and mode shapes, ϕ_n. These properties are determined with eigen analysis of the linearly-elastic

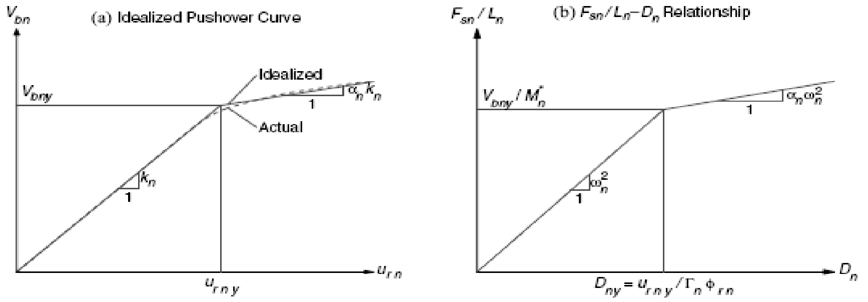

Fig.1 Properties of the nth-"mode" inelastic SDF system from the pushover Curve [Chopra , A.K. and Goel , R.K., 2002]

structure for the first three modes. It is necessary to be normalized mode-shape ϕ_n so that the roof component of ϕ_n equals to unity (ϕ_n=1).

2. Develop the base-shear – roof displacement ($V_{bn}-u_{rn}$) pushover curve for the nth-mode employing the load distribution $S_n^*=M\phi_n$

3. Idealize the pushover curve as a bilinear curve (Figure 1).

4. Transform the idealized pushover curve into the $F_{sn}/L_n - D_n$ relation (Figure 1) by utilizing formulas as follow:

$$\frac{F_{sny}}{L_n}=\frac{V_{bny}}{M_n^*} \quad , \quad D_{ny}=\frac{u_{rny}}{\Gamma_n \phi_{rn}} \tag{1}$$

where

$$\Gamma_n=\frac{\phi_n^T mi}{\phi_n^T m\phi_n} \tag{2}$$

$$L_n=\phi_n^T mi \tag{3}$$

$$M_n^*=L_n\Gamma_n \tag{4}$$

5. Compute the peak deformation, D_n , of the nth-mode inelastic SDF system (Figure 2) with force-deformation relation of Figure 1 by solving Equation :

$$\ddot{D}_n+2\zeta_n w_n \dot{D}_n+\frac{F_{sn}}{L_n}=-\ddot{u}_g(t) \tag{5}$$

It can be calculated from the inelastic response (or design) spectrum.

6. Calculate the peak roof displacement u_{rno} related to the nth-"mode" inelastic SDF system from:

$$u_{rno}=\Gamma_n\phi_{rn}D_n \tag{6}$$

7. Compute other favorite responses, r_{no} ,at u_{rno}

8. Iterate Steps 3 to 7 for as many modes as required for sufficient accuracy.

9. Calculate the total response by combining the contribution of peak "modal" responses using pertinent combination rule such as SRSS and CQC.

FEMA load distributions

Three lateral load distributions stipulated in FEMA-273 are as follows [Building Seismic Safety Council, 2000]:

1. Uniform distribution: $s_i^*=m_i$, in which m_i is the mass and s_I^* is the lateral force at i-th floor. (where the floor number i=1,2…N)

2. Equivalent lateral force (ELF) distribution: "A vertical distribution proportional to the values of $s^*_i=(m_x h_x^k)/(\sum_{i=1}^{N} m_i h_i^k)$ where the exponent k = 1 for fundament period $T_1 \leqq$ 0.5sec, k = 2 for $T_1 \geqq$ 2.5sec; and Linear interpolation shall be used in between. h_i , h_x are heights from the base to floor level i and x, respectively."

3. SRSS distribution: ''A vertical distribution proportional to the story shear distribution calculated by combining modal responses from 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. This distribution shall be used when the period of the

Table 1 Characteristics of analytical cases

No.	No. of stories	H (m)	b (m)	Seismic mass of floors (kg-sec^2/m)	Periods		
					T_1 (Sec)	T_2 (Sec)	T_3 (Sec)
B1	10	32	15	5440	1.697	0.605	0.347
B2	15	48	15	5546	2.338	0.854	0.493
B3	20	64	15	5600	3.092	1.135	0.670
B4	30	96	15	5650	3.866	1.381	0.798

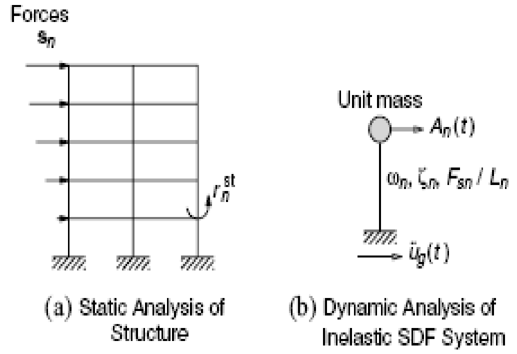

Fig.2 Conceptual explanation of uncoupled modal RHA of inelastic MDF systems [Chopra , A.K. and Goel , R.K., 2002]

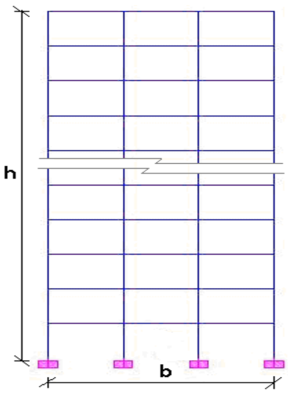

Fig.3 Configuration of selected two dimensional buildings

fundamental mode exceeds 1.0 second.”

Structural models

The structures considered are the 10, 15, 20 and 30-story two dimensional buildings, designed according to AISC-ASD89. Seismic effects determined in accordance with the requirements of Iranian code of practice for seismic resistant design of buildings. An amplification factor of 0·7R was applied to the drifts obtained from elastic analysis to obtain the maximum inelastic displacements. The results satisfied allowable drift criteria of code after some iteration [Standard No. 2800-05]. All buildings are assumed to be founded on firm soil type ‘II’ of Iranian seismic code (class C of NEHRP) and located in the region of highest seismicity. The buildings lateral load-resisting system is steel special moment-resisting frame (SMRF). It’s noted that Configuration of these buildings is shown in figure 3. All buildings are 15 m in width. The bays are 5 m on center with three bays. Story heights of all buildings are 3.2 m. The seismic mass of all levels of the each structure are assumed to be equal and the values of them are given in table 1. More details of these buildings are listed in table 1. The sections of beam and column elements are considered plate girder and box, respectively.

This model is based on centerline dimensions of the bare frame in which beams and columns extend from centerline to centerline. The strength, dimension, and shear distortion of panel zones are neglected but P-Δ (second order) effects are included.

The first three periods for linearly elastic vibration resulted from eigen analysis of the structures are presented in table 1.

The nonlinear force-displacement or moment-rotation behavior occurs in discrete hinges for nonlinear static and nonlinear response history analyses. 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 and M3 hinge based on only bending moment are considered at the hinge location of column and beam elements,

Table 2 List of used ground motions

No.	Earthquake Name	Date	Magnitude	Station Name	Station Number	Component (deg)	PGA (g)
1	Duzce, Turkey	1999/11/12	Ms (7.3)	Lamont	1061	E	0.134
2	Northridge	1994/01/17	Ms (6.7)	LA - Baldwin Hills	24157	90	0.239
3	Trinidad, California	1980/11/08	Ms (7.2)	Rio Dell Overpass, FF	1498	270	0.147
4	Victoria, Mexico	1980/06/09	Ms (6.4)	Cerro Prieto	6604	45	0.621
5	Hollister	1986/01/26	Ml (5.5)	SAGO South - Surface	47189	3295	0.09
6	Imperial Valley	1979/10/15	Ms (6.9)	Parachute Test Site	5051	315	.204
7	Morgan Hill	1984/04/24	Ms (6.1)	Corralitos	57007	310	0.109
8	Landers	1992/06/28	Ms (7.4)	Boron Fire	33083	0	0.119

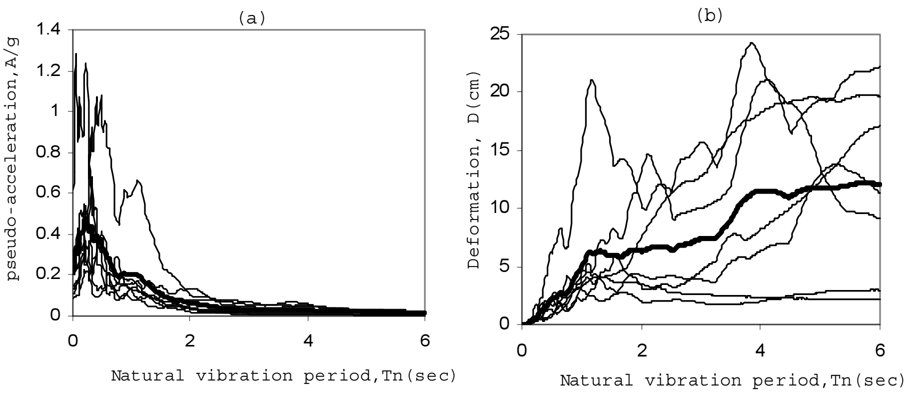

Fig.4 (a) Pseudo acceleration spectra and (b) deformation spectra of far field records set of ground motions, damping ratio=5%.

respectively.

Ground-Motion Scaling

Seven ground motions were selected from the Pacific Earthquake Engineering Research (PEER) Center strong ground motion database (http://peer.berkeley.edu). Ground motions were intended to be far at least 12 km from a fault rupturing. Also the Soil at the site corresponds to NEHRP Site Class C. To ensure that the structures respond well into the inelastic range when subjected to ground motions, the records were scaled up to 0.7g. More Characteristics of the used records are given in table 2. The elastic pseudo acceleration and deformation and the median spectra for 5% damping ratio are presented in Figure 4. The median spectrum is shown by a thicker line.

Analyses and assumptions

In order to deliberate MPA procedure and FEMA load distributions and to put forward some points about them, several types of analysis are performed: nonlinear response history analysis (NL-RHA), pushover analysis using the three force distributions in FEMA-273 and Modal pushover analysis (MPA) considering three modes. MPA is performed for each ground motion to obtain its seismic demands and the median values are determined over ensembles of seven ground motions. The computer program SAP2000, nonlinear version was employed to perform nonlinear static and dynamic analyses [Computers and Structures, 2004].

Table 3 Modal properties of inelastic SDF systems in modal pushover analyses

buildings	Properties	L_n (kg)	Γ_n	M_n^* (kg)	F_{sny} / Ln (cm/sec^2)	D_{ny} (cm)	$F_{sno}\ L_n$ (cm/sec^2)	D_{no} (cm)	T_n (sec)	ζ_n (%)
B1	Mode 1	302446.6	1.357	410473.4	220.946	16.196	252.776	36.841	1.697	5
	Mode 2	-116712	-.544	63519.96	1492.444	13.702	1767.61	41.341	0.605	3.86
	Mode 3	76243.2	0.296	22556.84	4275.864	13.025	5063.95	42.250	0.347	5
B2	Mode 1	443560.9	1.387	615252.8	148.776	20.709	172.696	46.859	2.338	5
	Mode 2	-156249	-.595	92922.94	954.33	17.342	1137.27	42.037	0.854	3.85
	Mode 3	100061.9	.335	33548.81	2494.873	15.168	3104.77	44.738	0.493	5
B3	Mode 1	577620.6	1.412	815768.6	113.524	27.687	125.042	54.875	3.092	5
	Mode 2	-204590	-.646	132090.2	741.917	23.651	845.711	54.210	1.135	3.93
	Mode 3	123552.8	0.390	48221.01	1806.267	20.246	2152.23	64.055	0.670	5
B4	Mode 1	827751.7	1.459	1207721	123.604	47.128	133.201	82.246	3.866	5
	Mode 2	-329979	-0.707	233249.5	572.348	26.899	684.357	67.198	1.381	3.87
	Mode 3	170719.1	0.413	70568.92	1639.815	26.149	1951.71	78.623	0.798	5

Gravity loads and P-Δ effects were included in all analyses. Pushover analyses are performed by first applying gravity loads, followed by monotonically increasing lateral forces with a specified height-wise distribution. The P-Δ effects due to gravity loads are considered for the first mode in MPA procedure. [Chopra , A.K. and Goel , R.K., 2001]. The responses resulted from pushover analyses are compared with the median of maximum seismic demands computed by rigorous NL-RHA analyses for an ensemble of ground motions described earlier. Nonlinear response history analysis is performed using numerical implicit *wilson-θ* time integration method in which parameter θ determines the stability and accuracy characteristics of the method and the value of 1.4 is allocated for this parameter. A damping ratio of 5% is considered for the first and third modes of vibration to define the Rayleigh damping matrix.

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 would be assumed equal to the maximum dynamic roof displacement [Tso WK, Moghadam AS., 1998; Mwafy A.M. and Elnashai A.S., 2001; Moghadam, A.s., 2002]. In this research, the target displacement at roof in pushover analyses is assumed as the median values of maximum top floor displacements of the structures, resulted from seven NL-RHA analyses. The values of target displacements are 26.92, 33.05, 38.27, 61.26 cm for B1, B2, B3 and B4 buildings, respectively.

The earthquake-induced demands are computed for the modeling buildings introduced earlier. For a yielding structure, the occurrence of structural damage is more closely related to story drift [Chopra , A.K. and Goel , R.K., 2001]. Therefore, in performance-based seismic evaluation, inter-story drift is an important attribute for damage control. Also, in order to evaluate performance of structural component, it is necessary to compute hinge plastic rotations and compare them with acceptability criteria based on FEMA-273/356.

Results and Discussion

Results of all analyses are briefly presented here. Floor displacements, Story drift ratios (story drifts / height of story), hinge plastic rotations of internal beam at all floor levels are calculated by aforementioned procedures. The results of pushover analyses are obtained at the target displacements. Also the errors of pushover analyses relative to exact solutions (benchmark results) obtained by NL-RHA are extracted.

MPA procedure is executed according to details expressed earlier. Properties of modal inelastic SDF systems are summarized in Table 3. This table provides evidence that yielding deformation, D_{ny} of inelastic SDF system becomes less for higher frequency modes, but instead there is a consequential increase in yielding force deformation, F_{sny} / L_n. This means that the slope of first branch of force-displacement diagram of inelastic SDF system increases for higher modes. It is rational due to

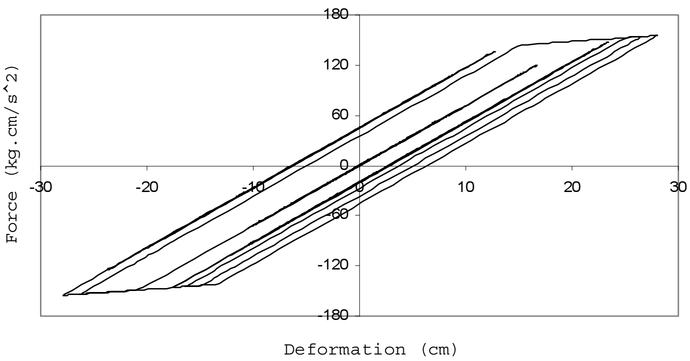

Fig.5 Force-deformation (F_{sn}/ L_n - D_n) diagram of an inelastic SDF system subjected to Northridge scaled up to 0.7g for the first mode of MPA in 15 storey building

Table 4 Peak deformation of inelastic SDF systems and target displacements of roof, u_{mo}

BUILDING	Ground Motion	Mode 1			Mode 2			Mode 3		
		D_n(cm)	μ	u_{mo}	D_n(cm)	μ	u_{mo}	D_n(cm)	μ	u_{mo}
B1	Northridge	22.44	1.385	30.455	8.27	0.604	4.501	3.93	0.302	1.163
	Duzce	20.76	1.281	28.175	5.67	0.414	3.086	9.94	0.763	2.941
	Trinidad	17.09	1.055	23.194	9.94	0.726	5.410	2.696	0.207	0.798
	Victoria	15.78	0.974	21.416	8.74	0.638	4.757	2.83	0.217	0.837
	Hollister	21.64	1.336	29.369	15.66	1.143	8.523	2.63	0.202	0.778
	Imperial Valley	16.12	0.995	21.878	4.45	0.325	2.422	2.94	0.226	0.870
	Morgan Hill	17.39	1.073	23.601	12.29	0.897	6.689	3.65	0.280	1.080
B2	Northridge	28.02	1.353	38.866	10.41	0.600	6.191	7.19	0.474	2.411
	Duzce	17.76	0.858	24.635	9.74	0.562	5.792	7.04	0.464	2.360
	Trinidad	19.29	0.931	26.757	13.85	0.799	8.237	3.95	0.260	1.324
	Victoria	13.5	0.652	18.726	13.05	0.753	7.761	6.97	0.459	2.337
	Hollister	22.65	1.094	31.417	22.7	1.309	13.500	5.17	0.341	1.733
	Imperial Valley	33.4	1.613	46.328	5.29	0.305	3.146	6.78	0.447	2.273
	Morgan Hill	12.71	0.614	17.630	19.82	1.143	11.787	11	0.725	3.688
B3	Northridge	37.39	1.350	52.806	17.67	0.747	11.408	6.92	0.342	2.701
	Duzce	25.8	0.932	36.437	15.52	0.656	10.020	6.86	0.339	2.677
	Trinidad	17.7	0.639	24.998	12.4	0.524	8.005	8.19	0.404	3.197
	Victoria	11.45	0.414	16.171	25.55	1.080	16.495	9.2	0.454	3.591
	Hollister	19.43	0.702	27.441	27.6	1.167	17.819	12.21	0.603	4.766
	Imperial Valley	43.39	1.567	61.280	14.79	0.625	9.548	5.01	0.247	1.955
	Morgan Hill	11.43	0.413	16.143	27.15	1.148	17.528	14.07	0.695	5.492
B4	Northridge	61.14	1.297	89.203	22.4	0.833	15.832	11.52	0.441	4.762
	Duzce	39.99	0.849	58.345	27.9	1.038	19.720	6.88	0.263	2.844
	Trinidad	27.95	0.593	40.779	9.67	0.360	6.835	9.89	0.378	4.088
	Victoria	22.06	0.468	32.186	21.07	0.784	14.892	8.14	0.311	3.365
	Hollister	17.73	0.376	25.868	31.33	1.165	22.144	18.61	0.712	7.693
	Imperial Valley	53.16	1.128	77.560	14.28	0.531	10.093	4.7	0.180	1.943
	Landers	43.73	0.928	63.802	31.01	1.153	21.918	33.7	1.289	13.930

the fact that higher modes have higher frequency and the structure exhibits more stiffness in higher modes. It's notable that the initial slope of force-deformation diagram related to nth mode is equal to the square of frequency of nth mode. Force-deformation (F_{sn} / L_n − D_n) diagram of an inelastic SDF system subjected to Northridge scaled up to 0.7g is shown in figure 5. This system is related to the first mode of MPA procedure in 15 storey building.

Target displacements of roof, u_{mo} are individually calculated from peak deformation of inelastic SDF systems subjected to scaled ground motions and results are presented in Table 4. Figures 6 to 9 show height-wise distribution of floor displacements, storey drift ratios computed with MPA procedure and nonlinear response history analyses for buildings B1 to B4, respectively. Also, the errors of the results estimated with MPA are presented in relation to benchmark solution (NL-RHA) in these figures. The figures provide results individually for each of the ground motion records. Several important observations emanate from figures 6 to 9 in conjunction with table 4. If SDF systems related to significant modes (usually 3 modes) of the buildings remain elastic(μ <1), the error of responses determined with MPA procedure will reach by up to 23%, 32%, 17% and 34% for B1, B2 ,B3 and B4 buildings, respectively. This is evident from aforementioned table and figures that whenever inelastic SDF systems respond beyond the elastic limit (μ <1), the seismic demands obtained by MPA will be less accurate. The bias of responses increase by up to 60%, 37%, 60% and 39% compared to the value from NL-RHA for the foregoing buildings, respectively. As a result, the error of MPA will be larger in the event that inelastic SDF system deforms well into the nonlinear region.

It is worth noting that the MPA procedure proposes to estimate peak dynamic response quantities of inelastic structures based on a combination of nonlinear responses obtained independently for each mode. In view of the fact that the application of modal combination rules to inelastic systems obviously lacks a theoretical basis (modes are not uncoupled anymore), thus MPA contains errors inherent in modal combination rules [Chopra, A.K. and Goel , R.K., 2002; Goel , R.K., and Chopra, A.K., 2004]. The foregoing figures demonstrate to some extent that modal uncoupling approximation would have small bias unless the structure responds far into inelastic range.

Figures 10, 11 and 12, 13 show the median of MPA and NL-RHA results over an ensemble of seven ground motion. As seen in these figures, the median values of floor displacements and story drift ratios resulted from MPA provide almost satisfactory estimates compared to NL-RHA and they are accurate enough for the engineering profession. According to figures 10 and 11, floor displacements obtained by MPA are usually more agreeable to NL-RHA results, but they are not able to indicate the damage of structures well. The MPA procedure leads to the bias in storey drift ratios by up 22%, 25%, 12% and 21% for the buildings in the same order mentioned earlier. Therefore, the errors produced by MPA become less if the median results determined over a series of ground motions are compared to NL-RHA. As a result it's notable that the seismic demands estimated by MPA procedure would be unreliable in nonlinear systems due to individual ground motions. Consequently, it's inevitable to avoid evaluating seismic demands of the buildings based on individual ground motion.

Figures 14 and 15 illustrate that the all FEMA load distributions entirely underestimate storey drift ratios in upper storeys of the buildings contrary to good predictions of floor displacements. Uniform load distribution underestimates the storey drifts by up to 66%, 73%, 74% and 63% for the buildings in the same order as they are declared in table 1. This load pattern overestimates seismic demand at lower storeys of the buildings that overestimations become less in high-rise buildings (e.g. 20 and 30 storey buildings) in relation to medium high-rise buildings(e.g. 10 and 15 storey buildings). Although SRSS and ELF load patterns take into account of higher mode effects, but they entirely fail to estimate with acceptable accuracy the storey drifts at top storeys. The discussion above demonstrates MPA superiority over all FEMA load distributions in predicting storey drift ratios.

Figures 16 and 17 provide evidence that not only all FEMA load patterns but MPA procedure moreover are incapable of estimating beam plastic rotations. These seismic demands are grossly underestimated in upper storeys of all buildings. According to figures 18 and 19 uniform load distribution gives the errors by up to 100% at top storeys of all buildings without exception. The errors of MPA reach by up to 100% for the first three buildings and 93% for the last one. It's noted that the contribution of higher modes other than first three modes will not improve the plastic rotations of hinges.

The locations of plastic hinges for 15 and 20 story buildings shown in figures 20 and 21 were obtained by four pushover analyses and NL-RHA. As shown in these figures, FEMA load

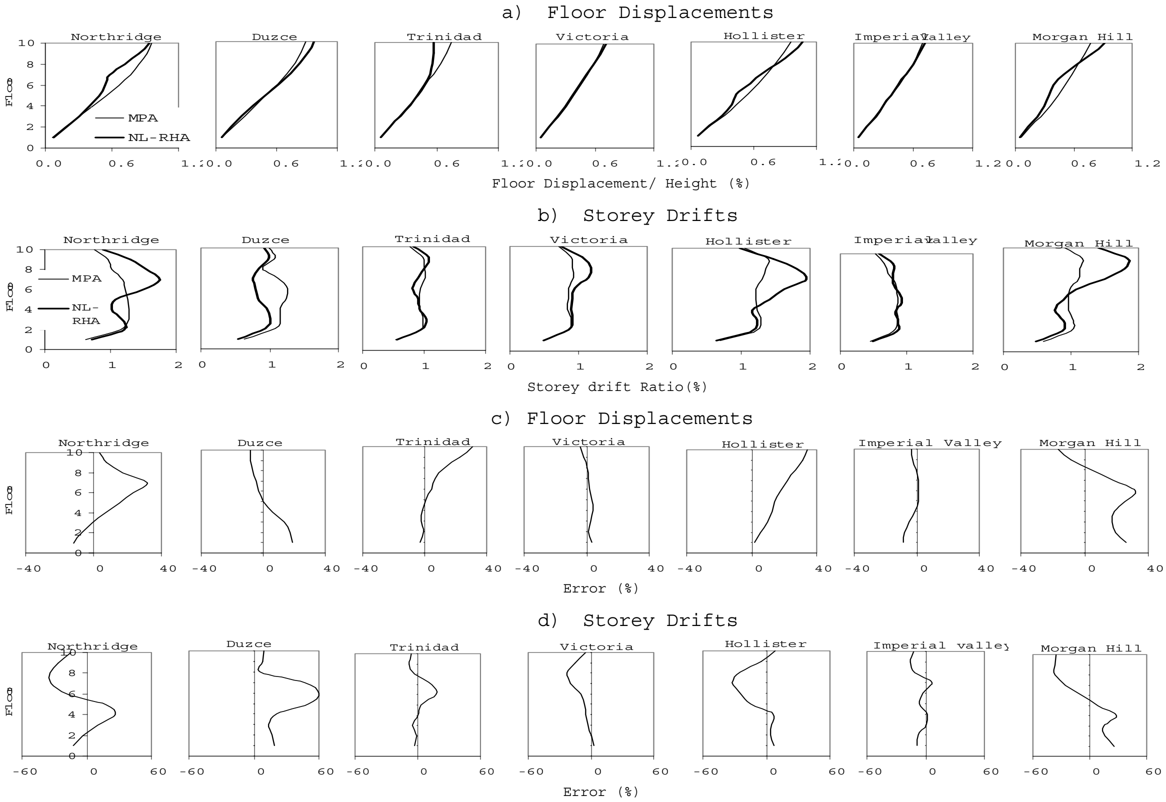

Fig.6 Height-wise variation of floor displacements and storey drift ratios and Error of MPA for 10 storey Building

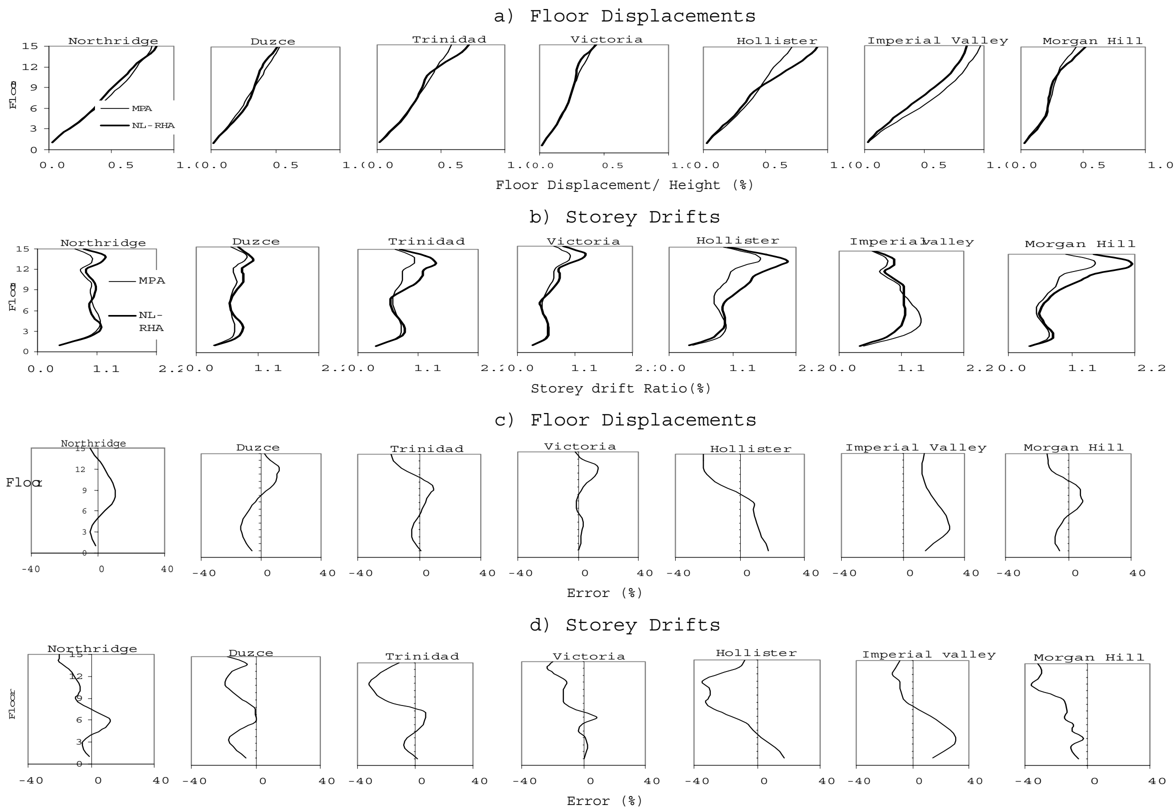

Fig.7 Height-wise variation of floor displacements and storey drift ratios and Error of MPA for 15 storey Building

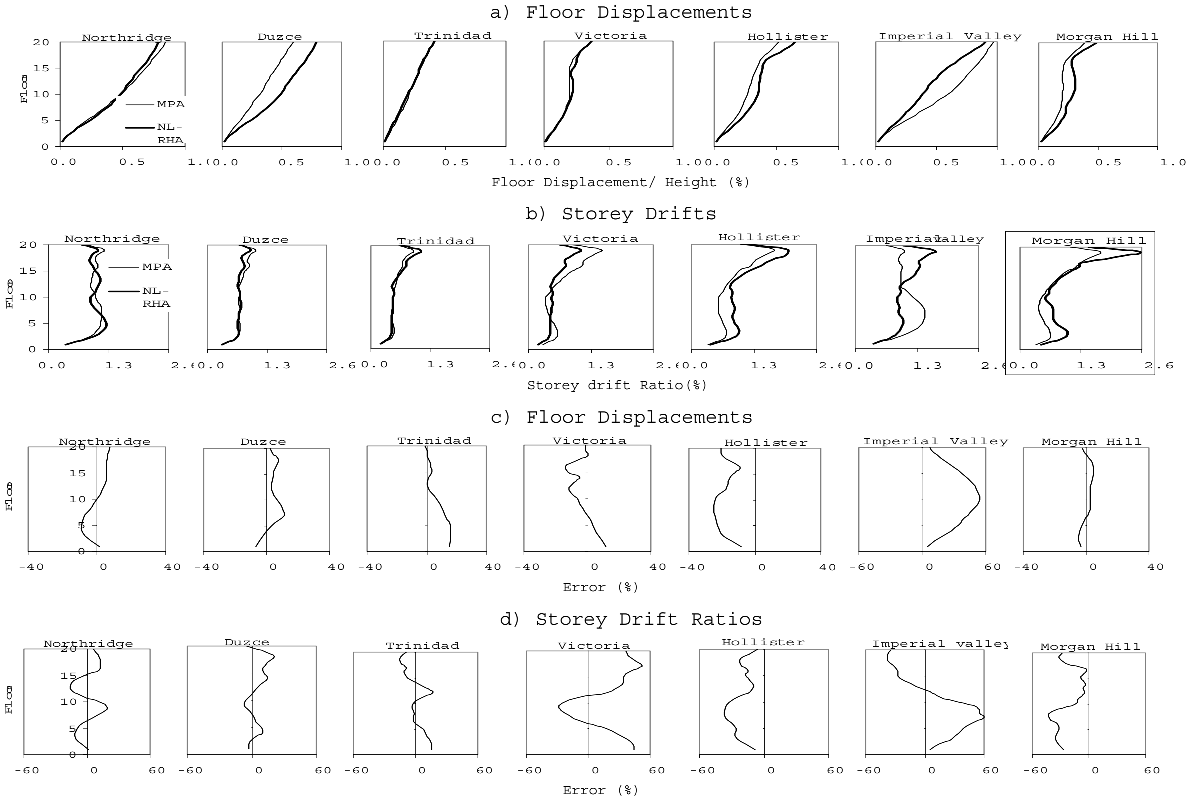

Fig.8 Height-wise variation of floor displacements and storey drift ratios and Error of MPA for 20 storey Building

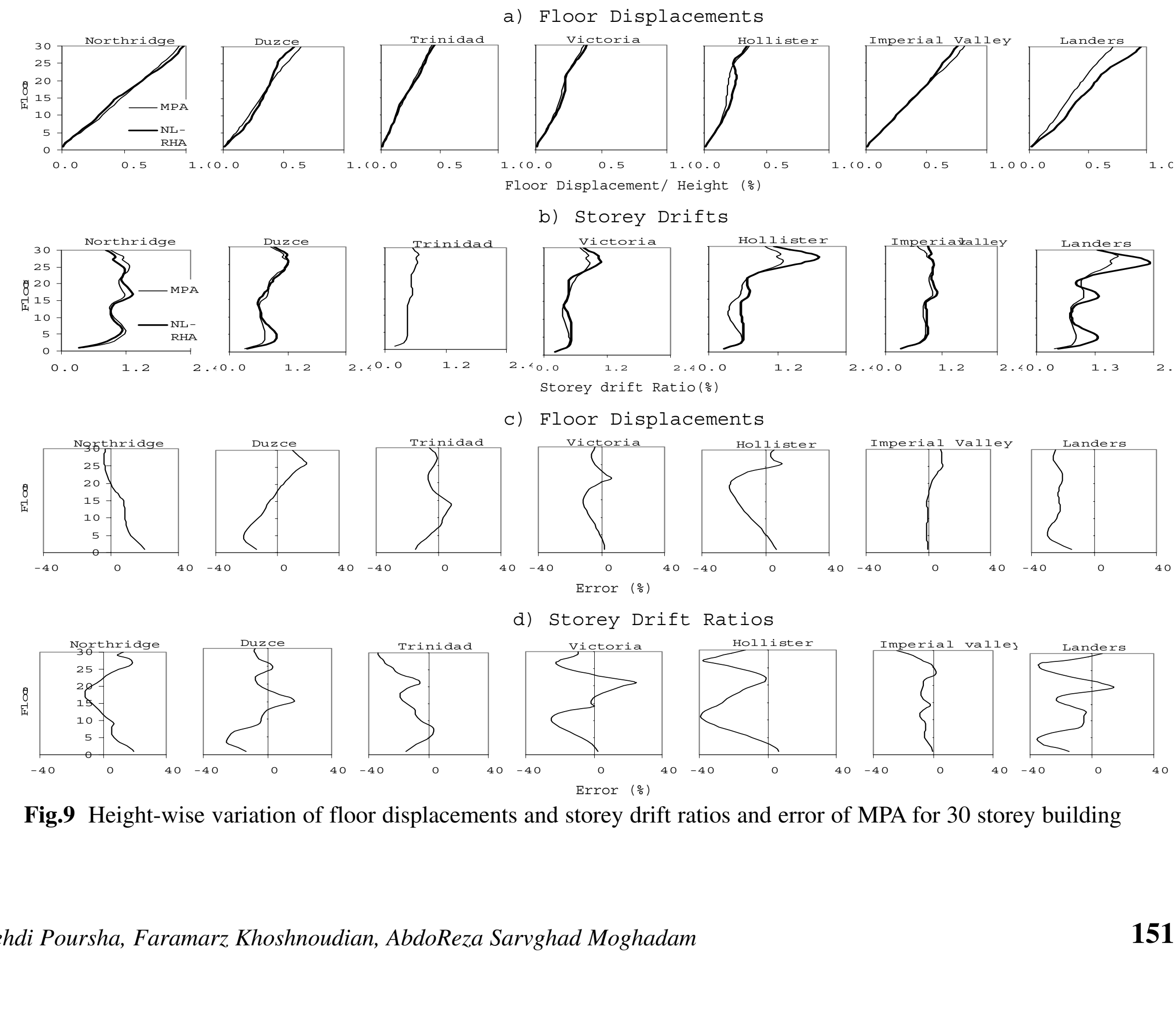

Fig.9 Height-wise variation of floor displacements and storey drift ratios and error of MPA for 30 storey building

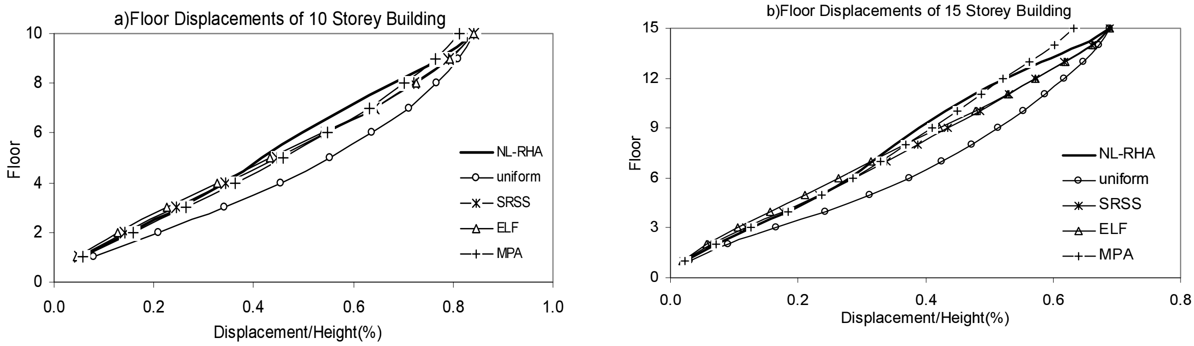

Fig.10 Height-wise variation of floor displacements of 10 and 15 storey buildings

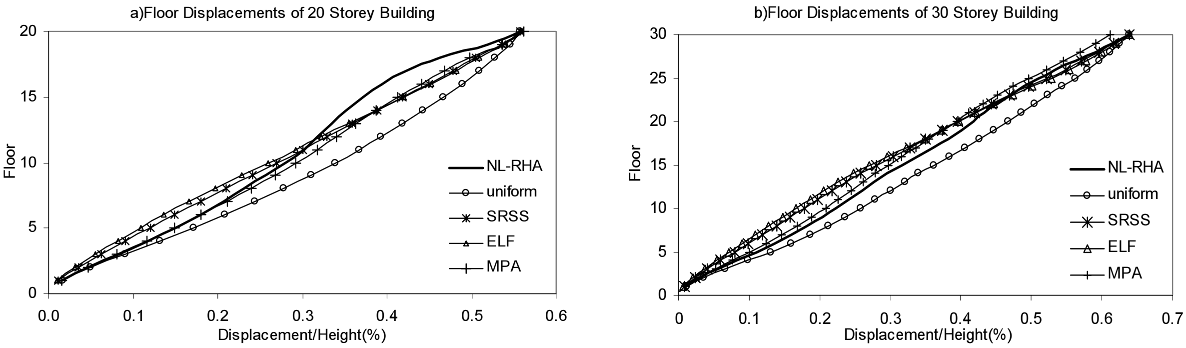

Fig.11 Height-wise variation of floor displacements of 20 and 30 storey buildings

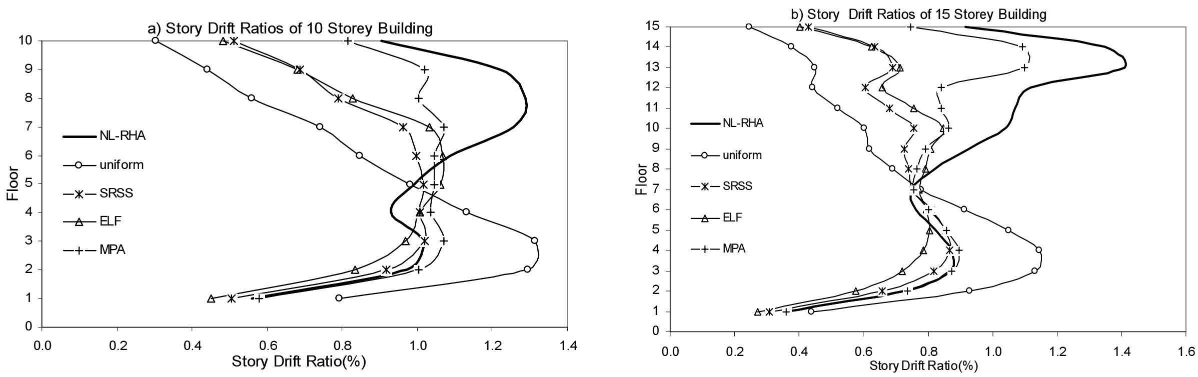

Fig.12 Heightwise variation of story drifts of 10 and 15 storey buildings

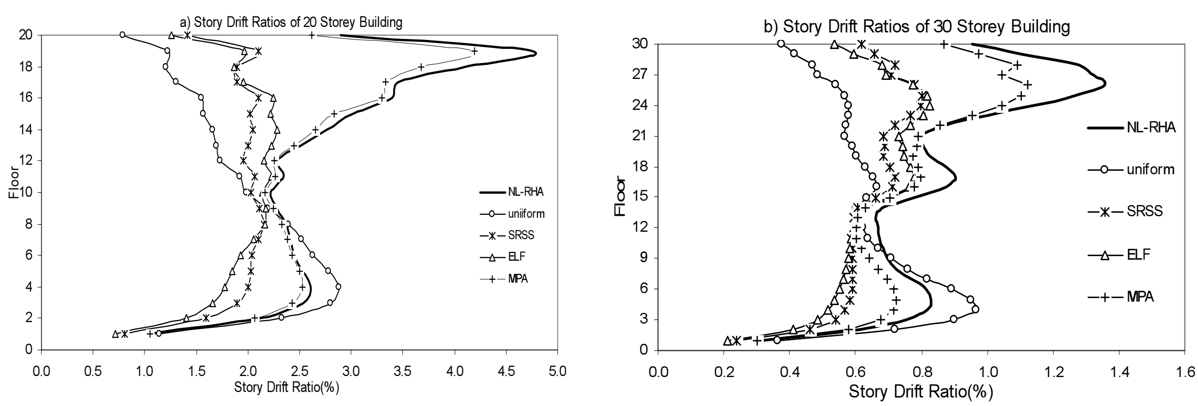

Fig.13 Heightwise variation of story drifts of 20 and 30 storey buildings

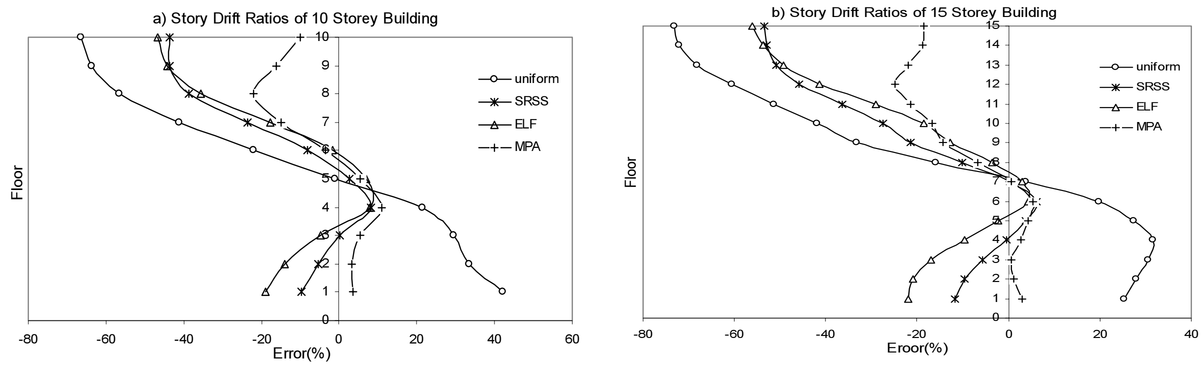

Fig.14 Errors in story drifts of 10 and 15 storey buildings

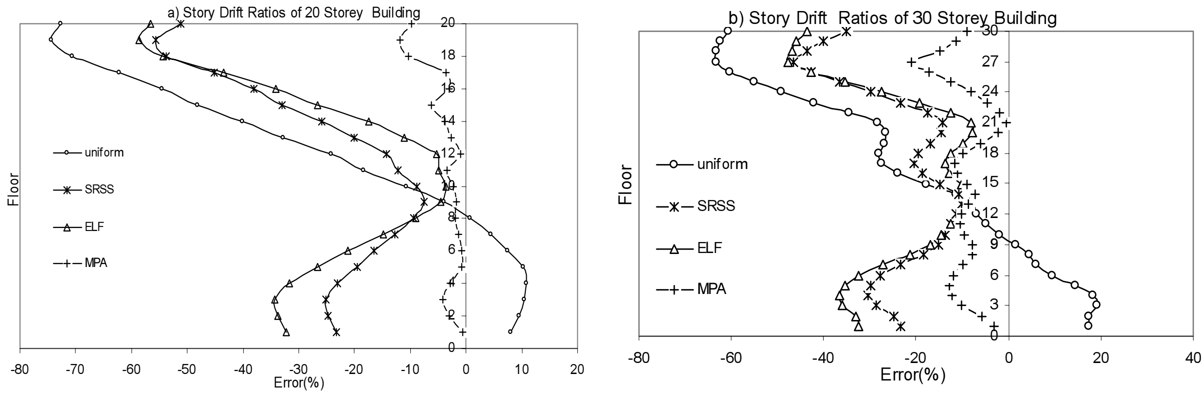

Fig.15 Errors in story drifts of 20 and 30 storey buildings

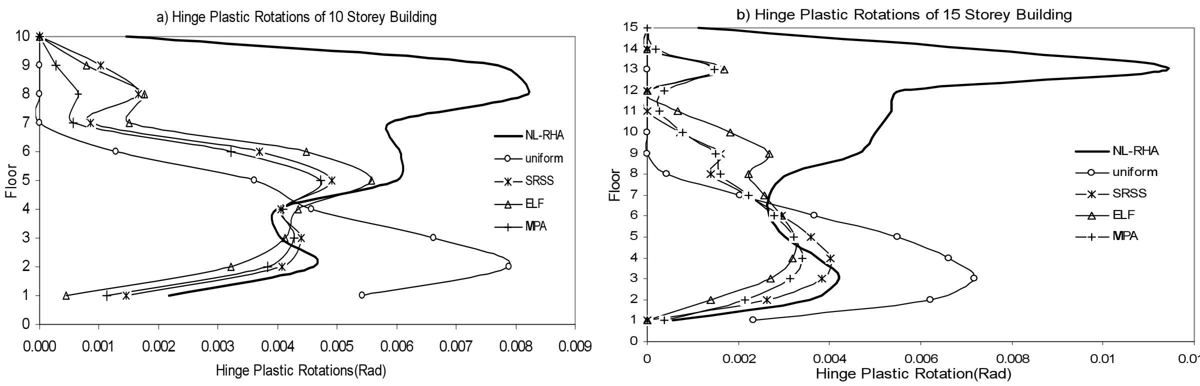

Fig.16 Height-wise variation of hinge plastic rotations of 10 and 15 storey buildings

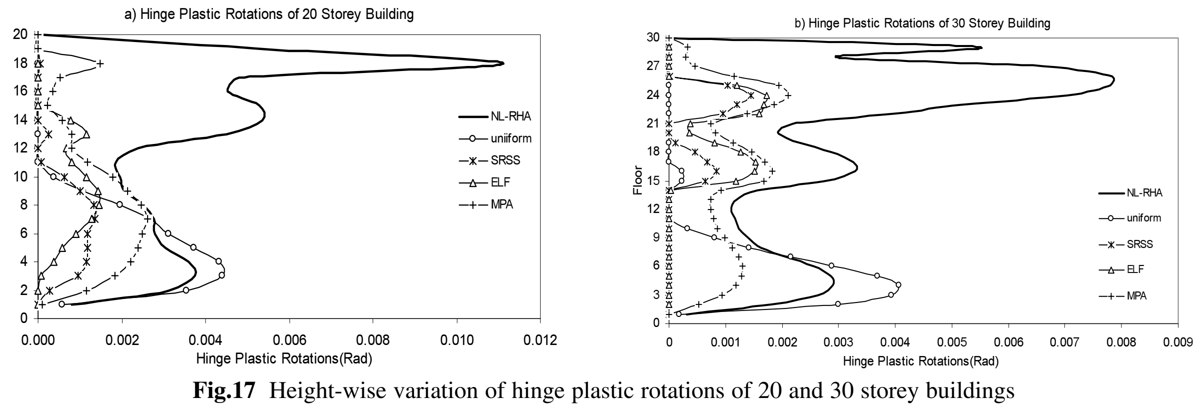

Fig.17 Height-wise variation of hinge plastic rotations of 20 and 30 storey buildings

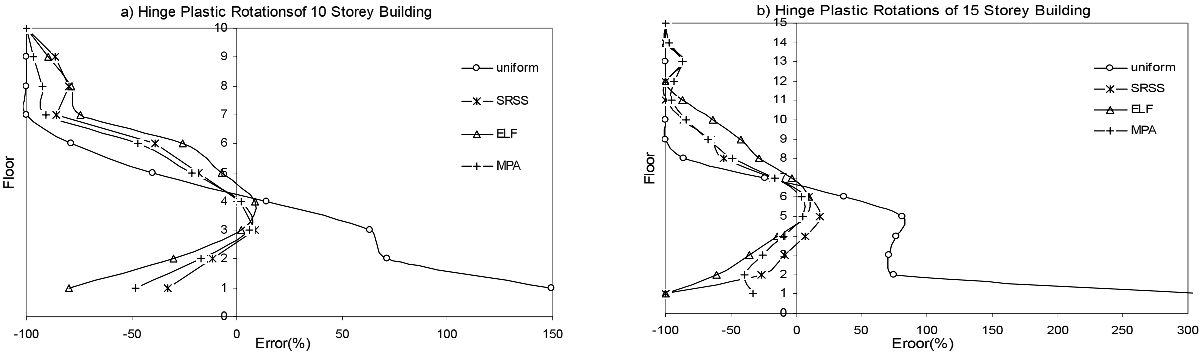

Fig.18 Errors in hinge plastic rotations of 10 and 15 storey buildings

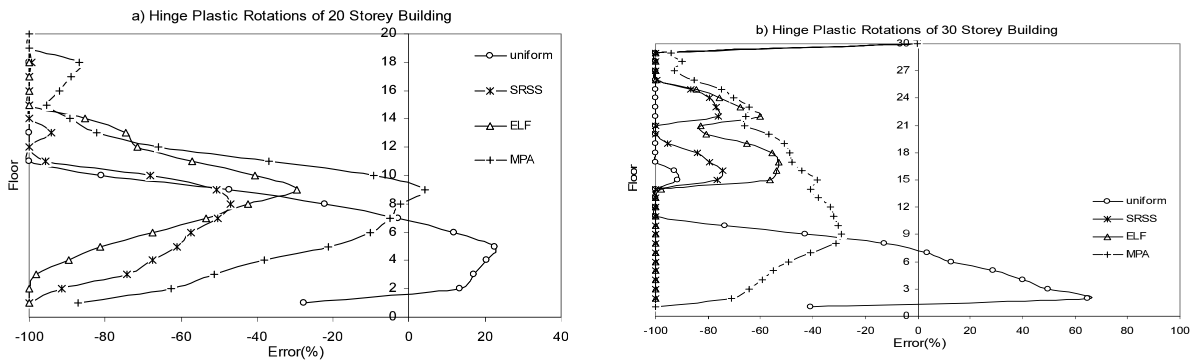

Fig.19 Errors in hinge plastic rotations of 20 and 30 storey buildings

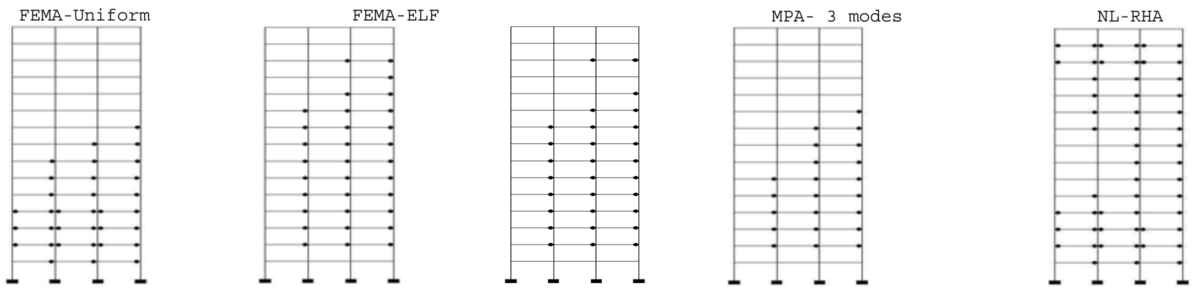

Fig.20 Locations of plastic hinges yielded by several analyses for 15 storey building

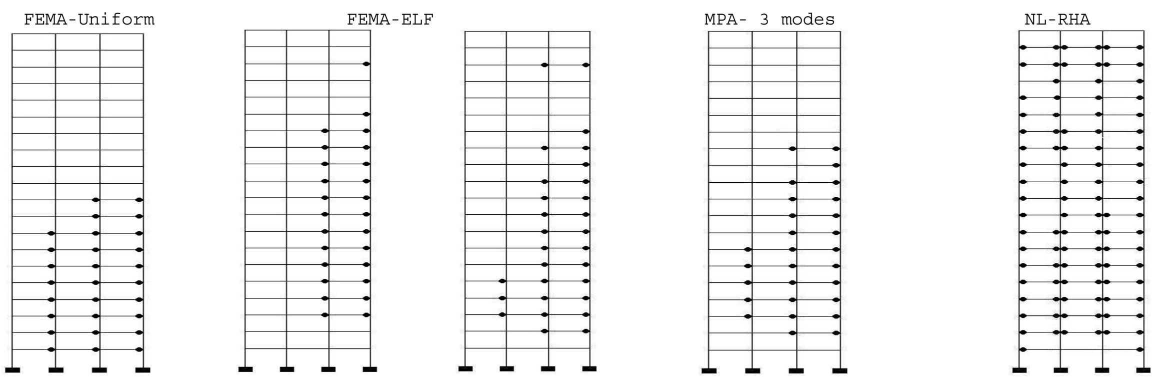

Fig.18 Errors in hinge plastic rotations of 10 and 15 storey buildings

distributions and MPA procedure are unable to accurately predict the hinge location in medium high-rise buildings and high-rise ones; Both FEMA and modal pushover analyses are unsuccessful to identify yielding of the beams at upper floor levels and several other locations. (The results of other buildings aren't presented here for the sake of briefness.)

As a matter of fact, these deficiencies point towards the necessity of another improvement in pushover analysis to overcome the shortcomings, compute hinge plastic rotations more reliably than current pushover procedures and result in more time savings compared to MPA.

Although MPA procedure conceptually is more attractive, it needs additional effort associated with calculating the peak value of deformation history, D_n, idealizing the pushover curve as a bilinear curve ,converting it into force-deformation relation for the first n modes and implementing modal pushover analyses individually for each of ground motions. Therefore, this procedure will run into a lot of time if a variety of ground motion patterns is used.

Pushover analysis suffers from some limitations and the cumulative rotation of the plastic hinge is not considered in pushover analysis [Chopra , A.K. and Goel , R.K., 2001]. Maybe it is possible to propose in future a procedure whereby reversal of lateral forces is provided at some top floor levels for the sake of higher mode effects during a single pushover analysis. Reversal of lateral loads may help to cumulative measure of response to some extent and consequently result in satisfactory predictions of hinge plastic rotations. This issue needs to more investigation.

Conclusions

The MPA is an enhanced pushover procedure developed to provide improved seismic demand predictions when higher mode effects are significant. The seismic demands obtained by MPA procedure will be unreliable in nonlinear systems subjected to individual ground motions which inelastic SDF systems related to significant modes of the buildings respond beyond the elastic limit (m >1). Therefore, it's necessary to avoid evaluating seismic demands of the buildings based on individual ground motion. Hence, the assessment is accomplished based on the median values obtained by an ensemble of ground motions. The MPA procedure results in significant superiority in computing the storey drift ratios in relation to FEMA load distributions and it almost yields acceptable estimates for professional practice. FEMA load distributions all grossly underestimate storey drift ratios at upper storeys of medium high-rise (e.g. 10 and 15 storey) and high-rise (e.g. 20 and 30 storey) buildings.

Indeed, both MPA procedure and FEMA load distributions fail to predict with convincing accuracy plastic rotations of hinges and to indicate locations of plastic hinges at top floor levels. The underestimation by these procedures is very large and even reaches by up to 100% occasionally.

Therefore, it is necessary to be developed another improved pushover procedure considering higher mode effects to overcome the shortcomings, compute hinge plastic rotations more reliably than current procedures, resolve the issue of cumulative measure of responses to some extent and result in more time savings compared to MPA. Research in this respect continues.

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