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An Investigation on the Accuracy of Nonlinear Static Procedures for Seismic Evaluation of Buckling-restrained Braced Frames

An Hong Nguyen, Chatpan Chintanapakdee, and Toshiro Hayashikawa

***Abstract*—**Presented herein is an assessment of current nonlinear static procedures (NSPs) for seismic evaluation of buckling-restrained braced frames (BRBFs) which have become a favorable lateral-force resisting system for earthquake resistant buildings. The bias and accuracy of modal, improved modal pushover analysis (MPA, IMPA) and mass proportional pushover (MPP) procedures are comparatively investigated when they are applied to BRBF buildings subjected to two sets of strong ground motions. The assessment is based on a comparison of seismic displacement demands such as target roof displacements, peak floor/roof displacements and inter-story drifts. The NSP estimates are compared to 'exact' results from nonlinear response history analysis (NL-RHA). The response statistics presented show that the MPP procedure tends to significantly overestimate seismic demands of lower stories of tall buildings considered in this study while MPA and IMPA procedures provide reasonably accurate results in estimating maximum inter-story drift over all stories of studied BRBF systems.

***Keywords*—**Buckling-restrained braced frames, nonlinear response history analysis, nonlinear static procedure, seismic demands.

I. Introduction

TO estimate seismic demands in design and evaluation of buildings, the nonlinear static procedures (NSPs) using the lateral force distributions recommended in ATC-40 [1] and the FEMA-356 [2] documents are now widely used 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), and 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 [3]-[8]. To overcome these drawbacks, an improved pushover procedure, called modal pushover analysis (MPA), was proposed by Chopra and Goel [4] to include contributions of higher 'modes'. The MPA procedure has been demonstrated to increase accuracy of seismic demand estimation in taller moment-frame buildings, e.g., 9- and 12-story tall, compared to the conventional pushover analysis [9], [10].

Recently, an improved modal pushover analysis (IMPA) procedure was proposed by Jianmeng et al. [11] to consider the redistribution of inertia forces after the structure yields. The IMPA procedure uses the product of the 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. However, to avoid a large computation, only two phase lateral load distribution was recommended. In the first phase, the pushover analysis is performed by using the first few elastic natural 'modes' of structure, i.e., similar to the MPA. In the second phase, only for the first 'mode' the lateral load distribution is based on assumption that the floor displacement vector at the initial yielding point is the displacement shape vector.

An alternative pushover analysis method to estimate seismic displacement demands, referred to as the mass proportional pushover (MPP) procedure, was proposed by Kim and Kurama [12]. 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 that is proportional to the total seismic masses at the floor and roof levels. However, the accuracy of both IMPA and MPP procedures has been verified for a limited number of cases.

With the increase in the number of alternative pushover analysis procedure proposed in recent years, it is useful to assess the accuracy and classify the potential limitations of these methods. An assessment on accuracy of MPA and FEMA pushover analyses for moment resisting frame buildings was investigated by Chopra and Chintanapakdee [9]. Then, an investigation on accuracy of improved nonlinear static procedures in FEMA-440 was carried out by Akkar and Metin [13].

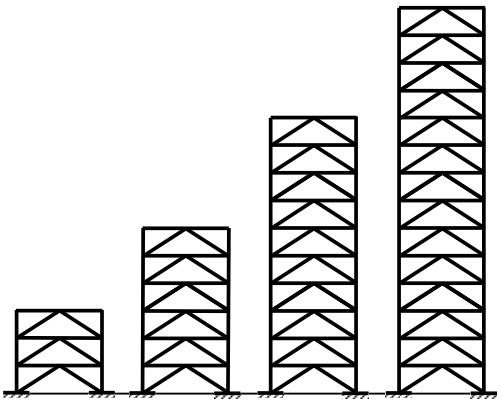
To assess the ability of current procedures, this paper aims to investigate the bias and accuracy of MPA, IMPA and MPP procedures when applied to buckling-restrained braced frames (BRBFs), which have become a favorable lateral-force resisting system for earthquake resistant buildings as its

An Hong Nguyen is with the Department of Civil Engineering, Chulalongkorn University, Bangkok 10330, Thailand (e-mail: annguyenbk@gmail.com).
Chatpan Chintanapakdee is with the Department of Civil Engineering, Chulalongkorn University, Bangkok 10330, Thailand (e-mail: fceccn@eng.chula.ac.th).
Toshiro Hayashikawa is with the Graduate School of Engineering, Hokkaido University, Sapporo, Japan (phone: +81-11-706-6170; fax: +81-11-757-8159; e-mail: toshiroh@eng.hokudai.ac.jp).

hysteretic behavior is non-degrading and much hysteretic energy can be dissipated. BRBF is an innovative structural system that prevents buckling of the braces by using a steel core and an outer casing filled with mortar for the brace. Brace axial force is resisted only by the steel core, which is restrained from buckling by the outer shell and infill mortar. The system is considered to have favorable seismic performance over traditional braced frames, making it an attractive option to structural engineers. More comprehensive background on this system can be found in [14], [15].

II. Structural Systems

Analyses of 3-, 6-, 10-, and 14-story BRBF buildings, which were designed to meet seismic code criteria, are presented to evaluate the bias and accuracy of MPA, IMPA and MPP procedures. Building designs for the BRBF system in both the 3-story and 6-story cases adhered to the criteria for the 3vb2 and 6vb2 model cases studied by Sabelli et al. [16] while the characteristics of the 10- and 14-story buildings were adopted from Asgarian and Shokrgozar [17]. Elevation view of all BRBF systems are shown in Fig. 1. Analytical properties of these BRBF buildings can be found in Chintanapakdee et al. [18]. Rayleigh damping model was used with 5% critical damping ratios for the first two modes, according to common practice for code designed steel structures [16]. *P*–Δ effect was also considered for this study. Nonlinear static and dynamic analyses were carried out using the computer program DRAIN-2DX [19]. The natural periods of all models are shown in Table I.



(a) 3-story (b) 6-story (c) 10-story (d) 14-story
$T_1 = 0.50\,\text{sec}$ $T_1 = 0.80\,\text{sec}$ $T_1 = 0.98\,\text{sec}$ $T_1 = 1.27\,\text{sec}$

Fig. 1 Frame elevations of 3-, 6-, 10-, and 14-story buildings

TABLE I
NATURAL PERIODS OF BUILDING MODELS IN THIS STUDY

Mode	Modal natural periods T_n (sec)			
	3-story	6-story	10-story	14-story
1	0.504	0.797	0.982	1.274
2	0.197	0.296	0.338	0.423
3	0.120	0.174	0.187	0.230

III. Ground Motions and Response Statistics

Two sets of ground motions used in this study, referred as LA2/50 and LA10/50, correspond to 2% and 10% probability of exceedence in a 50-year period [20]. These acceleration time histories were derived from historical recordings or from simulations of physical fault rupture processes. Each set of ground motions consists of 20 records which are the fault-normal and fault-parallel components of 10 recordings. The records in these suites include both near-fault and far-fault records. The pseudo-acceleration spectra for the two sets of ground motions are shown in Fig. 2 together with the median spectra (black solid lines).

To determine seismic demands of a building due to a set of ground motions, each record was scaled such that the spectral acceleration at the fundamental natural period of the building is equal to the median spectral acceleration for that period. This method of scaling helps reduce the dispersion of results [21]. More details of these scaling ground motions can be found in [18].

The response of each building to each set of the ground motions was determined by nonlinear response history analysis (NL-RHA), and nonlinear static procedure (NSP), e.g., MPA, IMPA and MPP. The peak value of inter-story drift, Δ, determined by NL-RHA is denoted by Δ_{NL-RHA} , and from NSP by Δ_{NSP}. From these data for each ground motion, a response ratio was determined from the following equation: $\Delta^*_{NSP} = \Delta_{NSP} / \Delta_{NL-RHA}$. The median values, $\hat{x}$, defined as the geometric mean, of n observed values (x_i) of Δ_{NSP}, Δ_{NL-RHA} and Δ^*_{NSP}; and the dispersion measure δ of Δ^*_{NSP} defined as the standard deviation of logarithm of the n observed values were calculated:

$$\hat{x} = \exp\left[\frac{\sum_{i=1}^{n} \ln x_i}{n}\right] \quad (1)$$

$$\delta = \sqrt{\frac{\sum_{i=1}^{n} (\ln x_i - \ln \hat{x})^2}{n-1}} \quad (2)$$

IV. Evaluation of Selected Nonlinear Static Procedures

The bias and accuracy of MPA, IMPA and MPP procedures are evaluated by comparing the target roof displacements, peak floor (or roof) displacements and inter-story drifts compare to 'exact' results from nonlinear response history analysis (NL-RHA).

A. Target roof displacements

Pushover curves, which show the relationship between base shear force and roof displacement, for the 3- and 6-story BRBF buildings due to first 'mode' load pattern (MPA), variable lateral force distribution (IMPA) and seismic mass (or

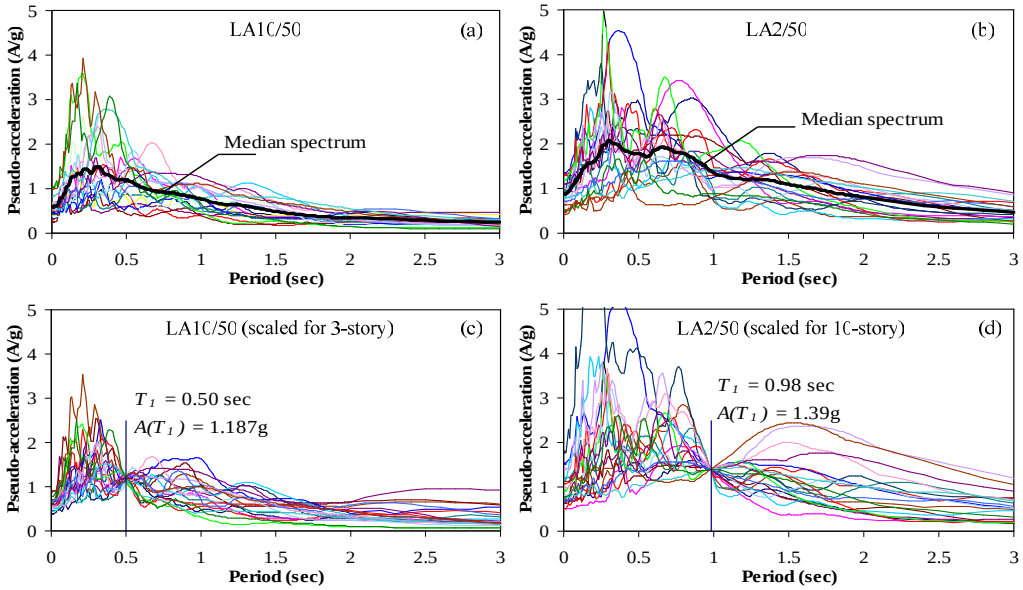


Fig. 2 Pseudo-acceleration spectra of (a) LA10/50, and (b) LA2/50 set of ground motions. (c) Pseudo-acceleration spectra of scaled LA10/50 ground motions for analyzing 3-story building, (d) Pseudo-acceleration spectra of scaled LA2/50 ground motions for analyzing 10-story building

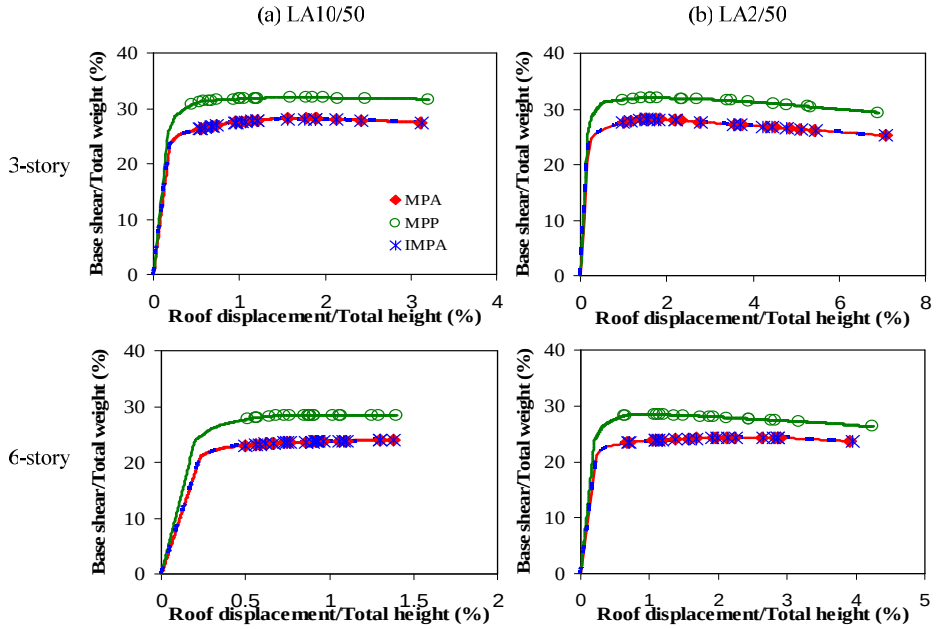


Fig. 3 First 'mode' pushover curves of 3- and 6-story buildings due to (a) LA10/50 and (b) LA2/50 ground motions

weight) distribution (MPP) are plotted in Fig. 3. It shows that the pushover curve of IMPA is similar to MPA. This results in nearly identical estimates of target roof displacements of both procedures. It implies that the changes of lateral load distribution of IMPA procedure are not significant whereas the force distribution of MPP leads to different results. Pushover curves of MPP are always higher and stiffer than both MPA's and IMPA's for all cases.

The accuracy of target maximum roof displacements predicted by displacement of equivalent SDF systems: $(u_r)_{SDF} = \Gamma_1 \phi_{r1} D_1$ for MPA and IMPA (where Γ_1, ϕ_{r1} and D_1 are participation factor, roof value and peak deformation of SDF system of the first 'mode', respectively) or $(u_r)_{SDF} = \Gamma D_{\max}$ for MPP are examined by calculating the ratio between SDF system estimate and roof displacement determined from NL-RHA: $(u_r^*)_{SDF} = (u_r)_{SDF} / (u_r)_{NL-RHA}$. The

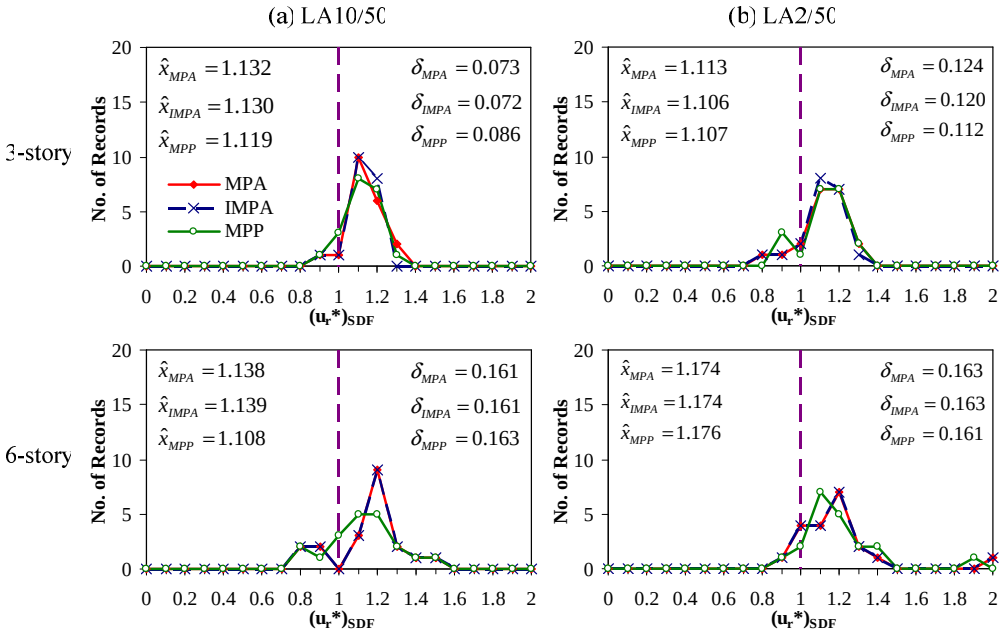


Fig. 4 Histograms of ratio $\left(u_r^*\right)_{SDF}$ for 3- and 6-story due to (a) LA10/50 and (b) LA2/50 ground motions

ratio $\left(u_r^*\right)_{SDF}$ being close to 1 indicates good accuracy. The histograms of these ratios of the 3- and 6-story buildings are shown in Fig. 4. The median and dispersion of the peak roof displacements are also noted. Fig. 4 shows that the SDF systems of these nonlinear static procedures slightly over-estimate the maximum roof displacements but the bias of MPA, IMPA and MPP is no larger than 14% for set of LA10/50 ground motions and 18% for stronger ground motions LA2/50.

B. Peak floor/roof displacements

The responses of buildings to the two sets of ground motions were determined by MPA, IMPA, MPP nonlinear static procedures and by nonlinear response history analysis (NL-RHA). The MPA and IMPA were considered as many modes as to include participating mass at least 95% of total mass. The combined values of floor displacements and story drifts were computed by using SRSS modal combination rule.

The peak floor/roof displacement demands from four methods are compared in Fig. 5; the results from modal pushover analysis (MPA) including only the fundamental 'mode' are also shown in dashed line. These results lead to the following observations for BRBF system. The contributions of higher 'modes' of MPA and IMPA procedures to floor displacements are not significant. One 'mode' pushover analysis, MPA, and IMPA can estimate the peak floor displacements reasonably well with a tendency to slightly overestimate floor/roof displacement compared to NL-RHA whereas the MPP tends to significantly overestimate peak floor displacements of lower stories (Fig. 5).

C. Story drift demands

Unlike the floor/roof displacements, the contributions of higher 'modes' in estimating the story drifts of MPA and IMPA procedures are more significant, especially in upper stories of tall buildings. Fig. 6 shows that the story drift demands of 10- and 14-story buildings predicted by MPA are able to follow the nonlinear RHA results whereas the first 'mode' alone is inadequate. With three or four 'modes' included, the story drifts estimated by MPA is generally similar to the 'exact' results from nonlinear RHA. However, the MPA story drift results including two 'modes' for 3-story and three 'modes' for 6-story buildings are close to one 'mode' results indicating that the contributions of higher 'modes' are not significant for these buildings. Both one 'mode' pushover analysis and MPA can estimate the response of structures reasonably well, although their results differ from NL-RHA results at some stories. Similar to investigation of peak floor/roof displacements, IMPA estimates tends to overlap the MPA estimates in estimating story drift demands. The MPP excessively overestimates story drifts in lower stories but underestimates story drifts in upper stories in these cases. Moreover, the story drifts predicted by MPP procedure seem to be uniform in upper stories, especially for 10- and 14-story buildings.

To verify a building design or to evaluate an existing structure, building codes usually require the maximum story drift in any stories to be less than its allowable value. Fig. 7 plots the maximum story drifts over all stories determined by NL-RHA and NSP as abscissa and ordinate, respectively. The MPA and IMPA data points are clustered along the diagonal line indicating that the maximum story drifts over all stories estimated by MPA and IMPA are close to the 'exact' value from NL-RHA. The median and dispersion of story-drift ratio

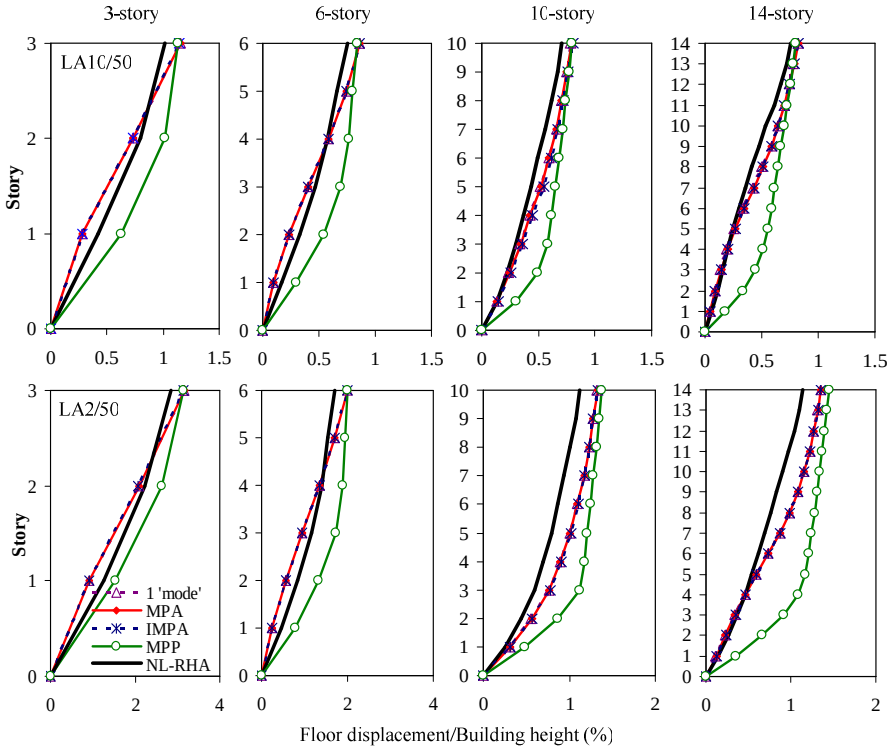


Fig. 5 Median floor displacements of 3-, 6-, 10- and 14-story buildings determined by one 'mode' pushover analysis, MPA, IMPA, MPP and NL-RHA due to LA10/50 (first row), and LA2/50 (second row) ground motions

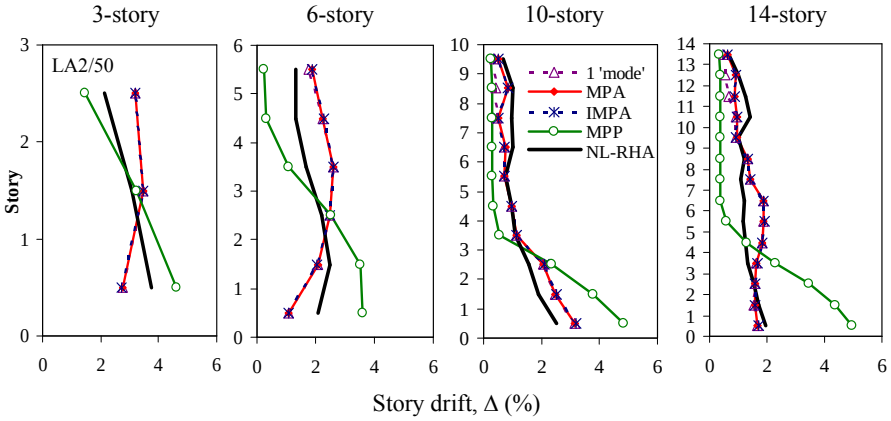


Fig. 6 Median story drifts of 3-, 6-, 10- and 14-story buildings determined by one 'mode' pushover analysis, MPA, IMPA, MPP and NL-RHA due to LA2/50 ground motions

Δ^*_{NSP} considering maximum story drift over all stories are also shown in Table II. The median story-drift ratios of MPA, Δ^*_{MPA}, range from 0.93 to 1.14 while the median story-drift ratios of IMPA, Δ^*_{IMPA}, from 0.92 to 1.16 indicating that both MPA and IMPA procedures predict maximum story drifts over all stories with bias less than 14% and 16% for these buildings, respectively. On the contrary, the bias in estimating maximum story drifts over all stories of MPP can be considerable in the range from 1.22 to 2.26. This implies that MPP significantly overestimates maximum story drift over all stories.

Thus, MPA and IMPA can be a useful analysis tool to estimate peak story drift over all stories in evaluating existing buildings or design of new buildings using BRBFs. Both of these procedures provide practically the same results but MPA is simpler and more practical than IMPA because it involves invariant load pattern. On the contrary, MPP method is simple with no need to conduct a modal analysis to capture the effects of higher 'modes' but it may be inaccurate in estimating seismic demands for BRBF tall buildings due to strong ground motions.

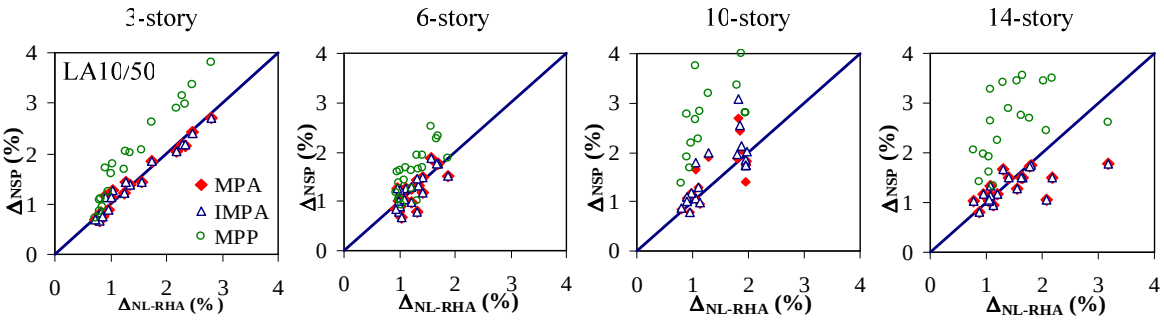


Fig. 7 Maximum story drifts over all stories determine by NSP, Δ_{NSP}, versus 'exact' values Δ_{NL-RHA}, for 3-, 6-, 10- and 14-story buildings due to LA10/50 ground motions

TABLE II
MEDIAN AND DISPERSION OF MAXIMUM STORY DRIFT RATIOS OVER ALL STORIES DETERMINE BY MPA, IMPA AND MPP, Δ_{NSP}, VERSUS 'EXACT' VALUES, Δ_{NL-RHA}

Set of records	3-story	6-story	10-story	14-story
LA10/50	Δ^*_{MPA} = 0.982; δ = 0.119	Δ^*_{MPA} = 0.949; δ = 0.205	Δ^*_{MPA} = 1.058; δ = 0.214	Δ^*_{MPA} = 0.986; δ = 0.248
	Δ^*_{IMPA} = 0.983; δ = 0.117	Δ^*_{IMPA} = 0.952; δ = 0.206	Δ^*_{IMPA} = 1.101; δ = 0.220	Δ^*_{IMPA} = 0.987; δ = 0.249
	Δ^*_{MPP} = 1.353; δ = 0.163	Δ^*_{MPP} = 1.244; δ = 0.209	Δ^*_{MPP} = 2.154; δ = 0.284	Δ^*_{MPP} = 1.831; δ = 0.317
LA2/50	Δ^*_{MPA} = 0.926; δ = 0.132	Δ^*_{MPA} = 1.013; δ = 0.203	Δ^*_{MPA} = 1.143; δ = 0.226	Δ^*_{MPA} = 1.046; δ = 0.298
	Δ^*_{IMPA} = 0.922; δ = 0.128	Δ^*_{IMPA} = 1.015; δ = 0.202	Δ^*_{IMPA} = 1.161; δ = 0.227	Δ^*_{IMPA} = 1.048; δ = 0.297
	Δ^*_{MPP} = 1.225; δ = 0.149	Δ^*_{MPP} = 1.422; δ = 0.212	Δ^*_{MPP} = 1.839; δ = 0.364	Δ^*_{MPP} = 2.256; δ = 0.287

V. Conclusions

The following conclusions are based on a comparison of NSP estimates of seismic demands and corresponding 'exact' values determined by nonlinear RHA for 3-, 6-, 10-, and 14-story BRBF buildings which were designed to meet seismic code criteria.

The story drift demands predicted by MPA and IMPA are able to follow the nonlinear RHA results. However, the higher 'modes' contributions of these procedures in response of 3-, and 6-story buildings are generally not significant, so the first 'mode' alone may be adequate.

Despite considering the redistribution of inertia forces after structure yields, the pushover curve of IMPA is similar to MPA, resulting in nearly identical estimates of target roof displacements by both procedures. The IMPA tends to overlap the MPA in estimating story drifts with slight differences.

The MPP tends to significantly overestimate seismic demands for lower stories but underestimates story drifts for upper stories with increasing bias when the building height increases.

The bias of MPA and IMPA procedures in estimating the maximum story drift over all stories is generally small; however, the bias of these procedures in estimating peak story drift at an individual story can be considerable for certain cases. On the contrary, the bias in estimating maximum story drifts over all stories of MPP can be large.

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