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EVALUATION OF MODAL PUSHOVER ANALYSIS (MPA) FOR TALL BUILDINGS SUBJECTED TO TWO COMPONENTS OF GROUND MOTION

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ABSTRACT

A recent extension of the modal pushover analysis procedure to estimate seismic demands of buildings subjected to two horizontal components of ground motion, simultaneously, is evaluated in this paper. The median seismic demands for existing 48- and 62-story buildings with ductile concrete core walls—designed to comply with current codes—due to an ensemble of 30 ground motions is computed by MPA and nonlinear RHA, and compared. We demonstrate that MPA procedure shows an acceptable degree of accuracy that should make it useful for practical application in estimating seismic demands for tall buildings subjected to two horizontal components of ground motion.

Introduction

This investigation evaluates the accuracy of the modal pushover analysis (MPA) procedure for estimating seismic demands for tall buildings with ductile concrete shear walls. Seismic demands are computed for two existing buildings subjected to 30 ground motions acting simultaneously in the longitudinal and transverse directions. We demonstrate that MPA provides useful estimates of seismic responses of these buildings.

Structural Systems

The structural systems considered are 48- and 62-story buildings taken from the Tall Building Initiative project at the Pacific Earthquake Engineering Research Center (PEER) (http://peer.berkeley.edu/~yang/). The buildings are identified by the letters CW (concrete wall) followed by the number of stories. The lateral resisting system of the buildings is a ductile concrete core wall consisting of two channel-shaped walls connected to rectangular walls by coupling beams, as shown in Figs. 1 and 2. The channel-shaped walls of the CW48 building have openings creating L-shaped walls at some stories (Figs. 1a and 2a). The typical floors are 8-inch-thick post-tensioned slabs spanning between the core and perimeter concrete columns. The total height of the CW48 and CW62 buildings is 471 and 630 feet, respectively. The height-to-width aspect ratio of the core of the 62-story building is 12:1 in the long direction (*x*-

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direction) and 18:1 in the short direction (y-direction), as shown in Fig. 2. To increase the aspect ratio and stiffness of the system in the short direction, concrete outrigger columns are included. The outrigger columns are connected to the core with buckling restrained braces (BRBs) at the 28th and 51st floors. Additionally, this building has a tuned liquid mass damper at the roof to help reduce sway during strong wind to acceptable comfort levels (Post 2008).

Modeling of these buildings in the PERFORM-3D computer program (CSI 2006) is described in Reyes (2009).

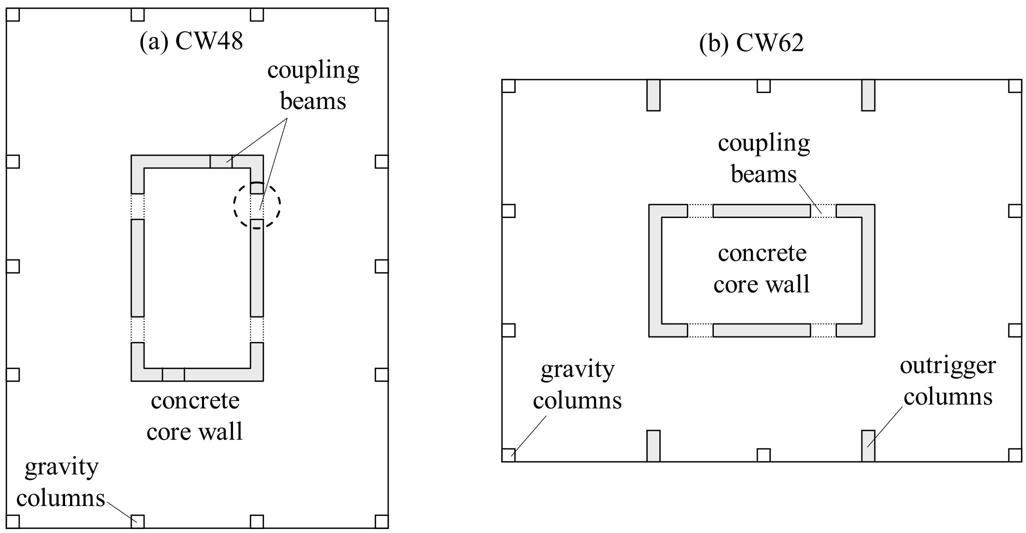


Figure 1. Schematic plan view of: (a) the CW48 building, and (b) the CW62 building

Ground Motions

A total of 30 ground acceleration records from nine different earthquakes with magnitudes ranging from 7.3 to 7.9 were selected; these are listed in Reyes (2009).

All the 30 records were scaled to represent the same seismic hazard defined by $A(T_1)$, the pseudo-acceleration at the fundamental vibration period T_1 of the structure. Both components of a record were scaled by the same factor selected to match their geometric mean to the selected seismic hazard. The geometric mean is defined as $A(T_1) = \sqrt{A_a(T_1) \times A_b(T_1)}$ where $A_a(T_1)$ and $A_b(T_1)$ are the $A(T_1)$ values for the two horizontal components of the record.

The selected seismic hazard spectrum was determined as the average of three uniform hazard spectra for 2% probability of exceedance in 50 years (return period of 2475 years) obtained using attenuation relationships developed by Campbell and Bozorgnia (2008), Boore and Atkinson (2008), and Abrahamson and Silva (2008) as part of the “Next Generation of Ground-Motion Attenuation Models” (NGA) project.

Table 1 lists the values of $A(T_1)_{2\%/50}$ selected to define ground motion ensembles. Fig. 3 shows the 5%-damped median response spectra for the ensemble of 30 ground motions along the a and b directions scaled to match $A(T_1)_{2\%/50}$, and the 5%-damped seismic hazard spectrum defined earlier, corresponding to 2% probability of exceedance in 50 years. As imposed by the scaling criterion, the median pseudo-acceleration of the ensemble at the fundamental period is matched to the seismic hazard spectrum; because T_1 differs for each structure, the scaling factors for ground motions and hence their median spectrum varies with the building.

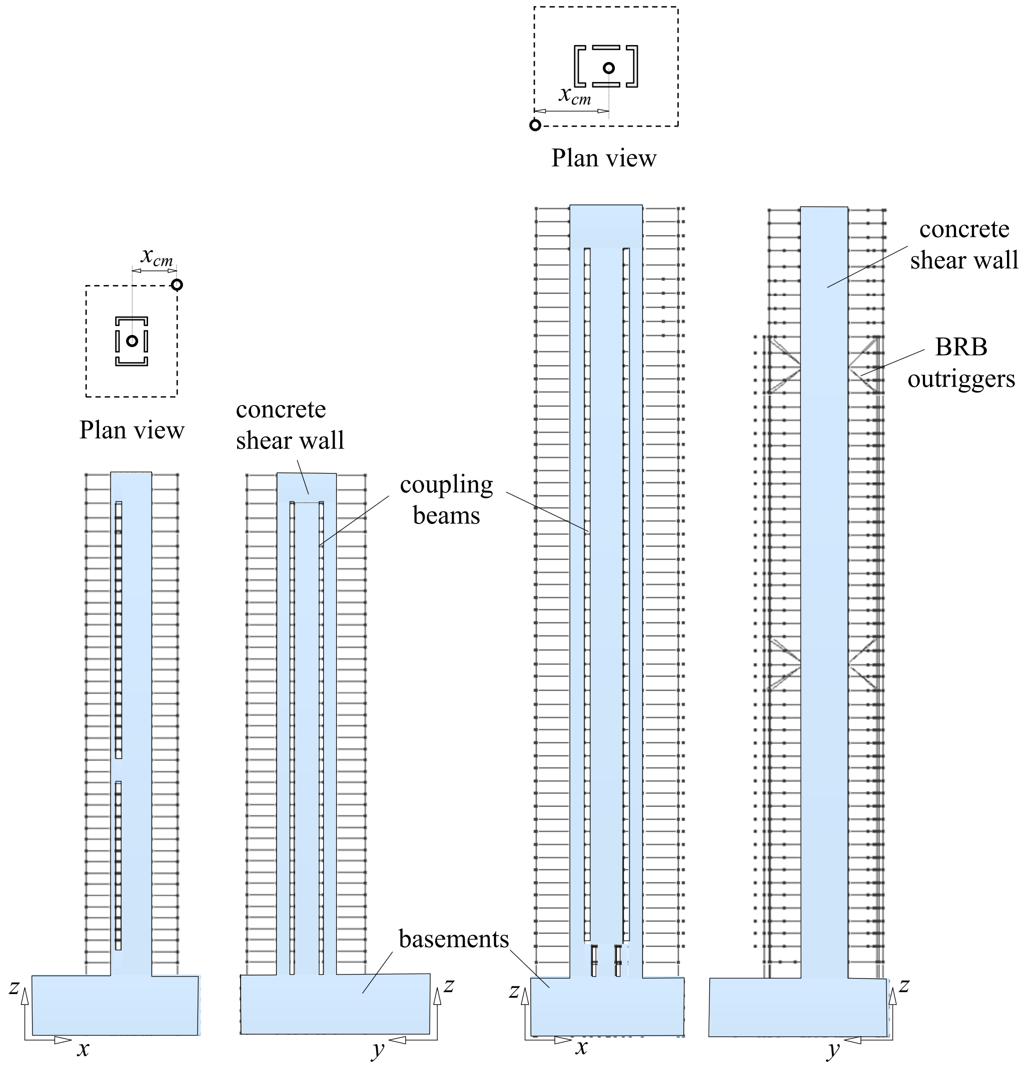


Figure 2. Lateral view of: (a) the CW48 building, and (b) the CW62 building

Table 1. Selected Values of $A(T_1)$

Building	$A(T_1)_{2\%/50}$ (g)
CW48	0.148
CW62	0.137

Modal Pushover Analysis for Two Components of Ground Motion

Three versions of MPS are considered in this investigation: (1) previously developed MPA for one component of ground motion extended to two components; (2) MMPA wherein the system is treated as linearly elastic in estimating higher mode contributions to seismic demands; and (3) PMPA wherein the median value of peak deformation D_n of the *n*th "mode" inelastic SDF system is estimated directly from the design spectrum using an empirical equation for the inelastic deformation ratio. These procedures are described in Reyes (2009).

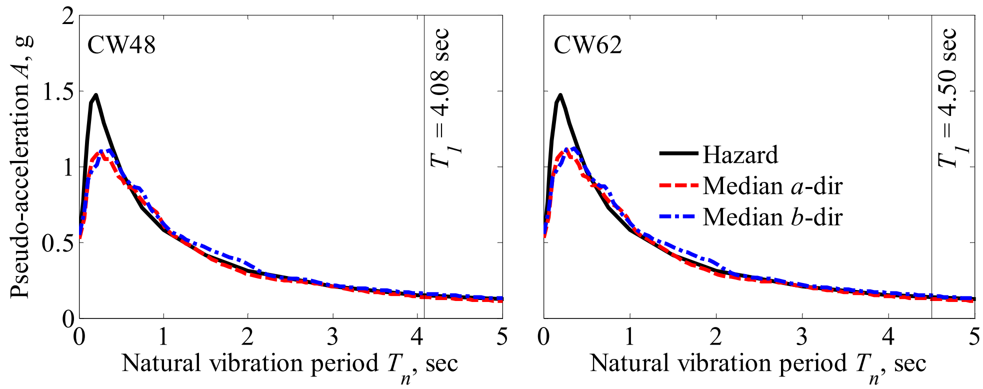


Figure 3. Seismic hazard spectrum for building site corresponding to 2% probability of exceedance in 50 years (solid line), and the median response spectra of 30 scaled ground motions in the *a* and *b* directions (dashed lines): CW48 and CW62 buildings

Evaluation of MPA Procedure

Higher Mode Contribution in Seismic Demands

Fig. 4 shows the median values of floor displacements and story drifts in the *x* and *y* directions at the center of mass (C.M.) including a variable number of "modes" in MPA superimposed with the "exact" result from nonlinear RHA, for each building subjected to both components of ground motion, simultaneously. The variation of story drifts over building height is typical of tall buildings except for the rapid fluctuations near the top of the CW62 building; such fluctuations are due to reduction of the core wall stiffness and presence of a tuned liquid mass damper, electromechanical equipment and architectural setbacks at those floors. The observations presented in this section will exclude the upper three stories of the CW62 building and basements of both buildings.

The first pair of "modes" alone is adequate in estimating floor displacements; including higher "modes" does not significantly improve this estimate (Fig. 4a). Although the first pair of "modes" alone is inadequate in estimating story drifts, with the second pair of "modes" (fourth- and fifth-"mode") included, story drifts estimated by MPA are much better, and resemble nonlinear RHA results (Fig. 4b). However, notable discrepancies between MPA and nonlinear RHA results remain for both buildings; these discrepancies are discussed in the following section.

Evaluation of Modal Pushover Analysis

MPA underestimates the *x* and *y* components of floor displacements for both buildings; the roof displacement is underestimated by about 16% and 12% for the CW48 and CW62 buildings, respectively. The height-wise largest underestimation of story drift is about 18% and 21% in the *x* and *y* directions, respectively, of the CW48 building; story drifts in the *x*-direction of the CW62 building are underestimated by 24%. Notable discrepancies remain for *y*-direction drifts in the upper part of the CW62 building where the underestimation is around 30%; in this direction, the core wall of the building interacts with BRBs and outrigger columns developing plastic hinges at various locations over building height.

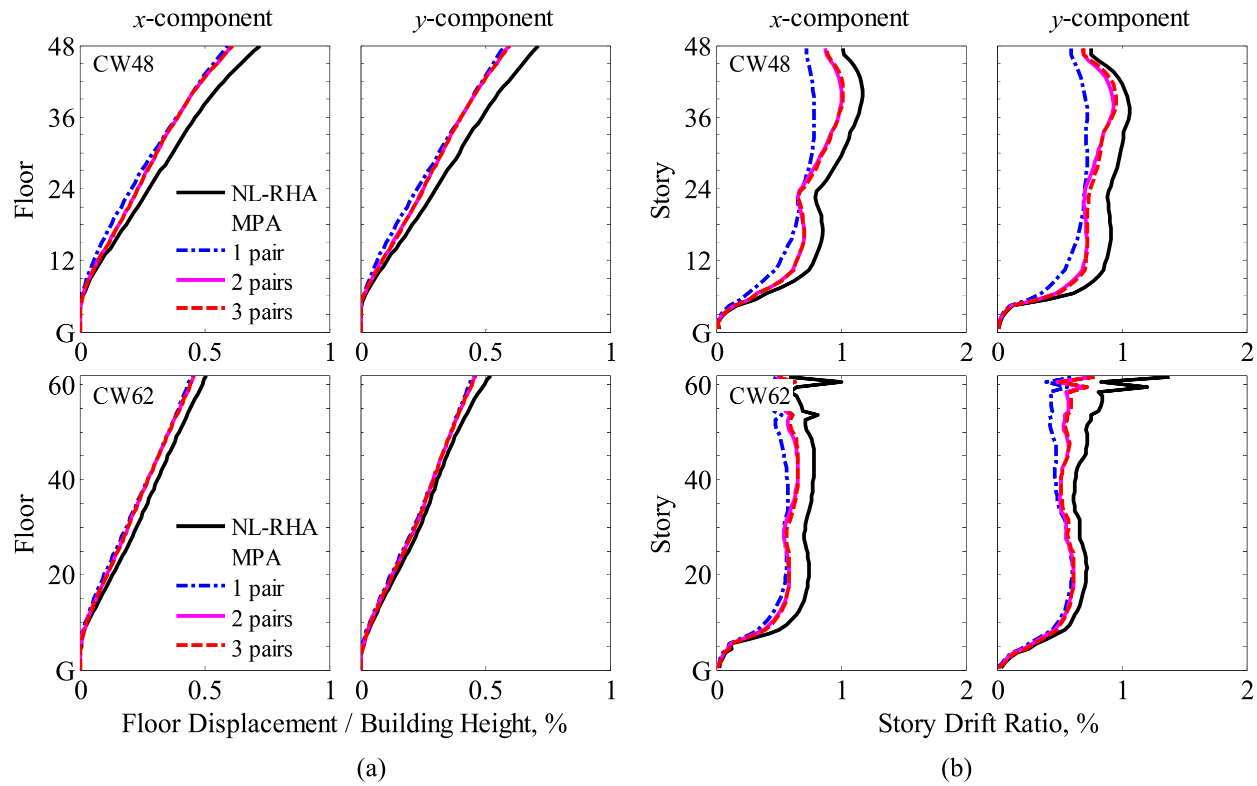


Figure 4. (a) Median floor displacements and (b) story drifts at the C.M. of the CW48 (row 1) and CW62 (row 2) buildings determined by nonlinear RHA and MPA, with a variable number of "modes".

Evaluation of Modified MPA (MMPA)

Results presented in Reyes (2009) and their interpretation suggested that the buildings could be treated as linear in estimating contributions of modes higher than the first triplet of "modes" (or first pair of "modes" for a symmetric-plan building) to seismic demands. This observation is utilized in the Modified MPA (MMPA) procedure, which was implemented to estimate seismic demands for the CW48 and CW62 buildings; torsional modes are ignored because their effective modal mass is negligible for the *x*- and *y*-direction excitations.

Fig. 5 shows the median values of *x* and *y* components of floor displacements and story drifts at the C.M., determined by nonlinear RHA, MPA and MMPA for both buildings. The seismic demands estimated by MMPA and MPA are very close (Fig. 5) implying that it is valid to treat the buildings as linearly elastic in estimating higher-mode contributions to seismic demands.

Evaluation of Practical MPA (PMPA)

Floor Displacements and Story Drifts at the C.M.

In implementing the Practical MPA (PMPA) procedure, the median value of the peak deformation $\hat{D}_n$ of the *n*th-mode inelastic SDF system was estimated by multiplying the median peak deformation $\hat{D}_{no}$ of the corresponding linear system by an inelastic deformation ratio C_{Rn} taken from Chopra and Chintanapakdee (2004); $\hat{D}_{no}$ was determined from the median response spectrum for the ensemble of 30 ground motions. For all modes of these long-period buildings, C_{Rn} was essentially equal to 1.0.

Fig. 5 shows the median values of the *x* and *y* components of floor displacements and story drifts at the C.M., determined by nonlinear RHA, MPA, and PMPA for both buildings. In general, PMPA provides a larger estimate of seismic demands compared to MPA because $(\hat{D}_n)_{\text{PMPA}}$ determined as $(\hat{D}_n)_{\text{PMPA}} = C_{Rn}\hat{D}_{no}$ in PMPA is larger than the exact $(\hat{D}_n)_{\text{MPA}}$ determined in MPA; the ratio of the two for the first pair of "modes" of both buildings is shown in Table 2. This is to be expected for some long-period systems because the empirical equation for C_{Rn} does not permit values below 1.0, whereas the exact data does fall below 1.0 (Chopra and Chintanapakdee 2004). Despite this overestimation in one step of PMPA, the method underestimates the seismic demands (Fig. 5). The height-wise maximum underestimation of story drifts is about 26% and 27% for the CW48 and the CW62 buildings, respectively.

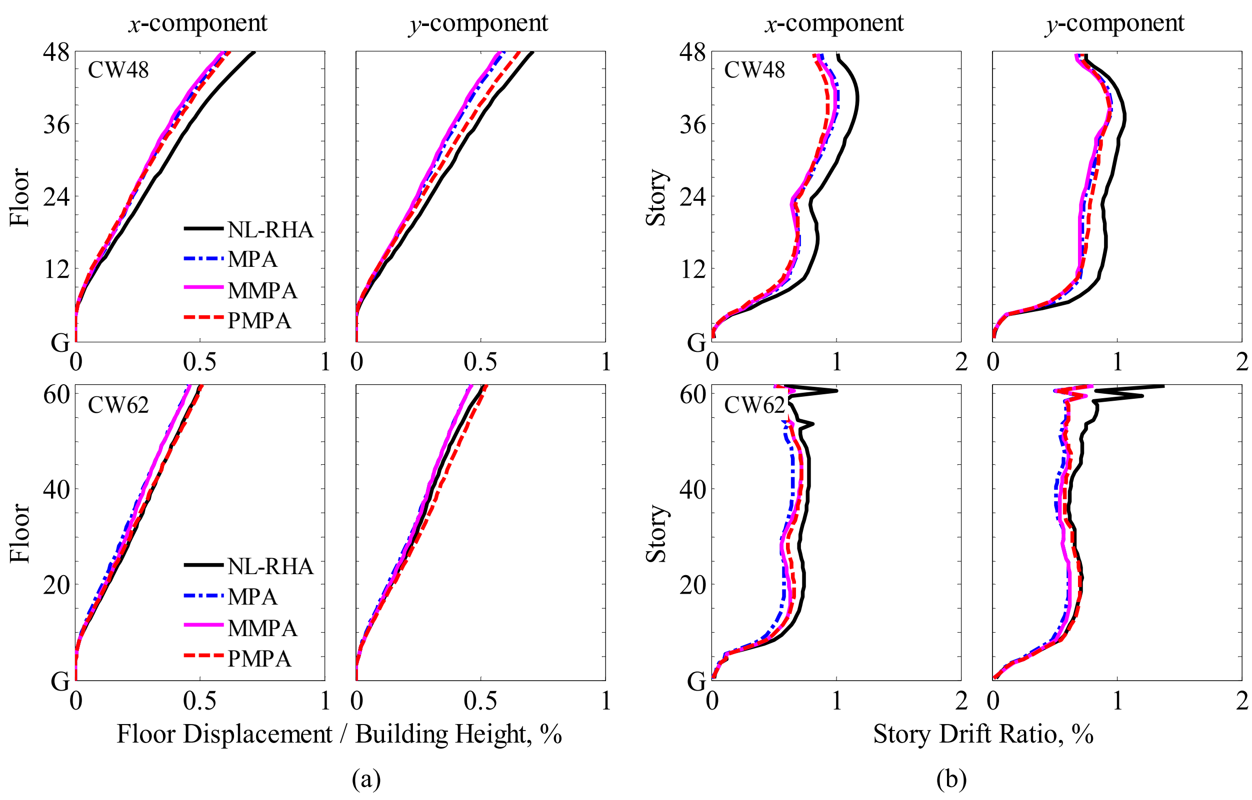
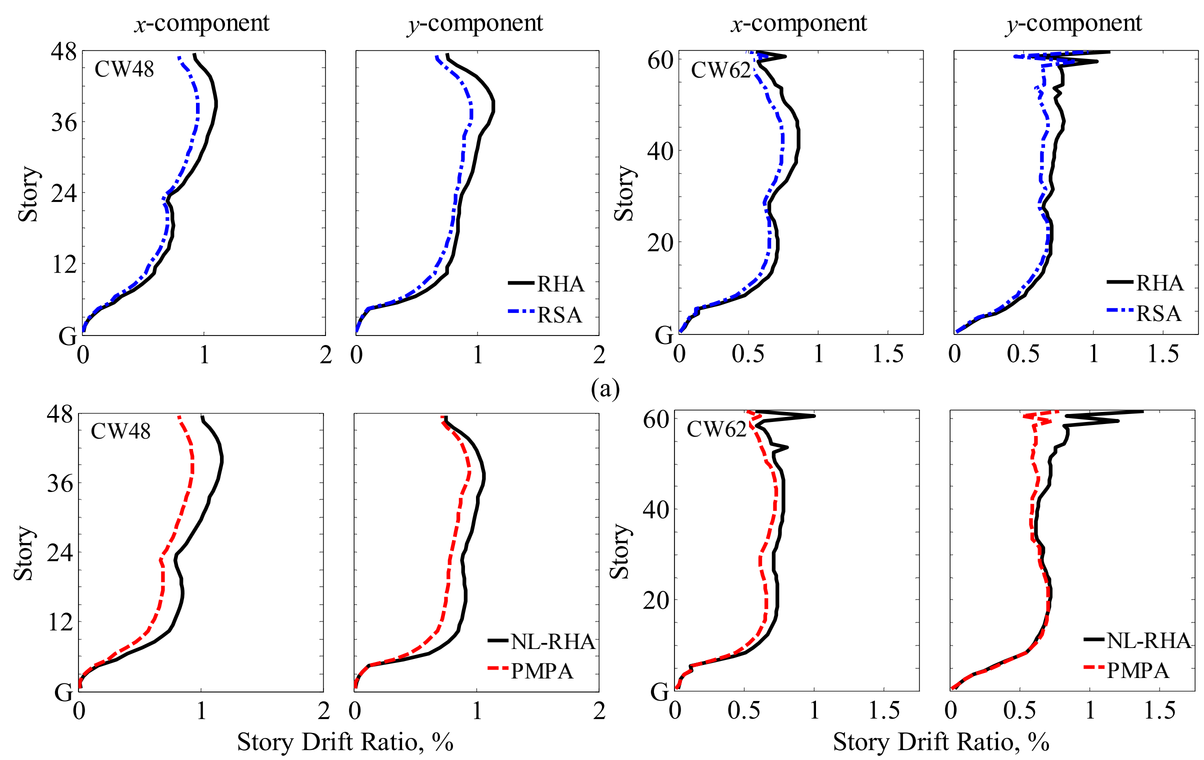


Figure 5. Median (a) floor displacements and (b) story drifts at the C.M. of the CW48 (row 1) and CW62 (row 2) buildings determined by nonlinear RHA, MPA, MMPA and PMPA.

Fig. 6 compares the accuracy of PMPA in estimating the response of nonlinear systems with that of RSA in estimating the response of linear systems. For each of the two buildings, the results for story drifts at the C.M. are organized in two parts: (a) story drift demands for these buildings treated as linearly elastic systems determined by RSA and RHA procedures, and (b) demands for nonlinear systems determined by PMPA and nonlinear RHA. In implementing the RSA and PMPA procedures, three pairs of "modes" were included for each building.

Table 2. Peak deformation ratio $(\hat{D}_n)_{\mathrm{PMPA}} \div (\hat{D}_n)_{\mathrm{MPA}}$ for the first pair of modes of the CW48 and CW62 buildings

Building	$(\hat{D}_n)_{\mathrm{PMPA}} \div (\hat{D}_n)_{\mathrm{MPA}}$	
	a-dir (n=1)	b-dir (n=2)
CW48	1.08	1.16
CW62	1.13	1.27



(b)

Figure 6. Median story drifts at the C.M. for: (a) linearly elastic systems determined by RSA and RHA procedures, and (b) nonlinear systems determined by PMPA and nonlinear RHA procedures. Results are for CW48 (columns 1 and 2) and CW62 (columns 3 and 4) buildings

Observe that the RSA procedure underestimates the median response for both buildings. This underestimation tends to be greater in the upper stories of the buildings, consistent with the height-wise variation of contribution of higher modes to response (Chopra 2007). The height-wise largest underestimation in story drifts is 20% and 22% for the CW48 and CW62 buildings, respectively. The additional errors introduced by estimating $\hat{D}_n$ using empirical equations and by neglecting modal coupling and cyclic stiffness degradation in the PMPA procedure, which are apparent by comparing parts (a) and (b) of Fig. 6, are small. PMPA underestimates the story drifts by 26% and 27% for the CW48 and the CW62 buildings, respectively; these errors are only slightly larger than those observed in RSA.

For the CW48 building, the principal source of errors in PMPA is due to the underestimation of roof displacement (Fig. 6). Suppose we eliminate this underestimation by scaling the PMPA values of D_n to obtain roof displacements equal to the exact values from nonlinear RHA, as shown in Fig. 7a. Then, the height-wise maximum error is reduced from 25% to 15% for the x-component of story drifts, and from 26% to 21% for the y-component. Thus, it would be useful to develop improved methods to estimate roof displacement.

The PMPA procedure leads to a slightly larger estimate of seismic demand, thus reducing the unconservatism (relative to nonlinear RHA) of MPA results; therefore, PMPA is a useful alternative to estimate seismic demands for tall buildings due to two horizontal components of ground motion applied simultaneously.

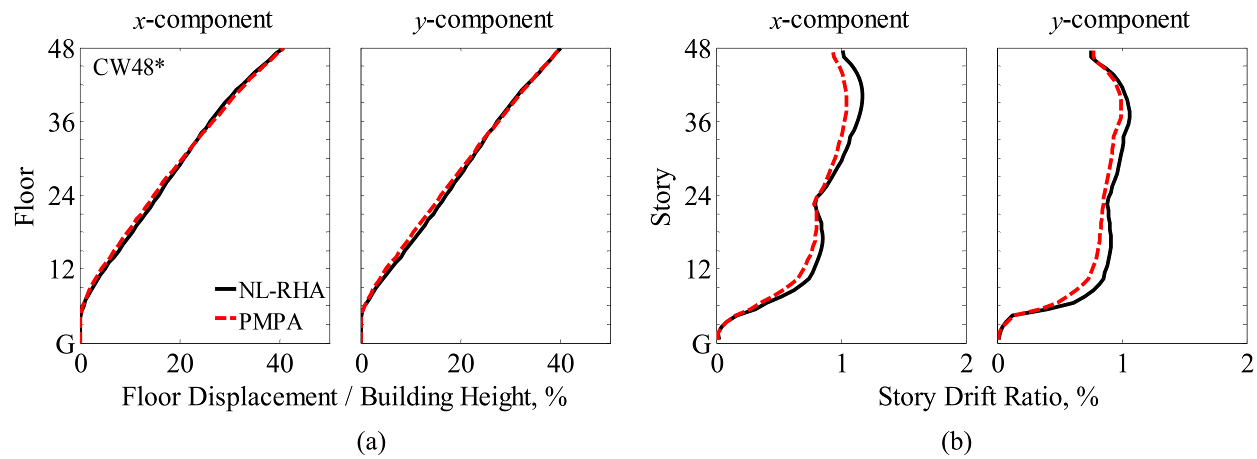


Figure 7. (a) Median floor displacements and (b) story drifts at the C.M. of the CW48 building determined by nonlinear RHA and PMPA with D_n values scaled to obtain roof displacements equal to those from NLRHA.

Other Response Quantities

The member forces and total hinge rotations corresponding to the story drifts calculated by PMPA (Fig. 6b) were estimated by the PMPA procedure. Fig. 8 presents results for the CW48 building including, shear forces and end rotations in the coupling beams highlighted in Fig. 1a, determined by the PMPA and nonlinear RHA procedures. It is apparent that internal forces are estimated accurately, whereas total rotations are underestimated just as the story drifts were underestimated. The error in internal forces is generally smaller than the error in hinge rotations because internal forces vary slowly with hinge rotation for members that deform beyond the elastic limit at both ends. As a result, even a large error in the hinge rotation leads to only small error in the computed internal forces; these observations are consistent with Goel and Chopra (2005).

The forces in the core wall can be estimated to a useful degree of accuracy by the PMPA procedure. Fig. 9 presents the height-wise variation of shear forces (V_x and V_y) in the core wall including a variable number of "modes" in PMPA superimposed with the results from nonlinear

RHA. Fig. 9 shows that the first pair of "modes" alone is grossly inadequate in estimating internal forces for the CW48 building; however, with the second and third pair of "modes" included, internal forces estimated by PMPA resemble nonlinear RHA results. By including the contributions of all significant modes of vibration, PMPA is able to adequately capture the height-wise variation of shear forces and bending moments in the core shear wall. Thus, PMPA overcomes a well known limitation of current pushover procedures with invariant force distribution which are unable to consider higher mode effects after formation of local mechanisms (Krawinkler and Seneviratna 1998).

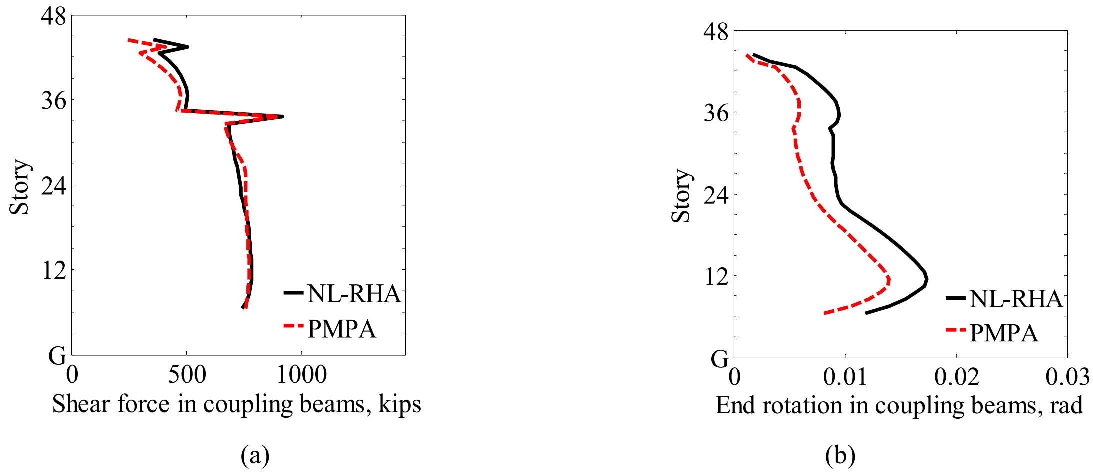


Figure 8. (a) Shear forces and (b) end rotations for the coupling beam highlighted in Fig. 1a, determined by nonlinear RHA (NL-RHA) and Practical MPA (PMPA). Results are for the CW48 building.

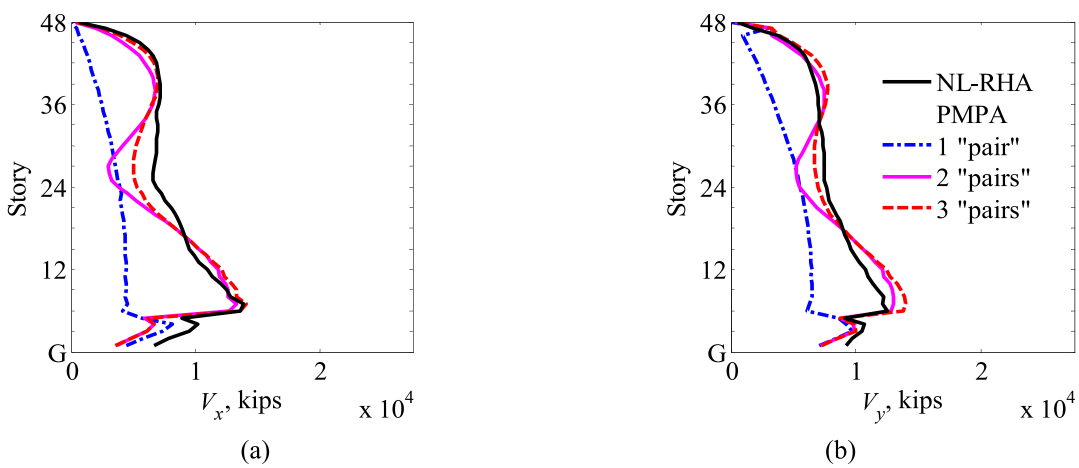


Figure 9. Shear forces V_x and V_y for the core wall of the CW48 building, determined by nonlinear RHA and Practical MPA (PMPA).

Conclusions

It has been demonstrated that the MPA procedure offers a sufficient degree of accuracy that should make it useful for practical application in estimating seismic demands—floor displacements, story drifts, rotations and internal forces—for tall buildings due to two horizontal components of ground motion applied simultaneously.

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