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An Investigation on the Adaptive Modal Combination Procedure for Seismic Evaluation of Buildings Considered Soil Structure Interaction

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Abstract— **Nonlinear response history analysis (NL-RHA) is a powerful tool for seismic analysis of structures. The seismic performance of structures can be estimated accurately due to a set of ground motions. However, NL-RHA is an onerous task to estimate seismic demands due to its complexity for practical design applications. Therefore, nonlinear static procedures (NSPs), which are rooted in structural dynamic theory, are developed as an alternative to rigorous NL-RHA. NSPs are now widely used in engineering practice to evaluate seismic responses of structure in the inelastic range. The main objective of this study is to assess the bias and accuracy of the adaptive modal combination (AMC) procedure, which is modified from the modal pushover analysis (MPA) procedure, for seismic evaluation of buildings. Three-, 9-, 18-storey buildings were analyzed due to two sets of strong ground motions having 2% and 10% probability of being exceeded in 50 years. The influence of soil-structure interaction in the analysis was also considered by using Beam-on-Nonlinear-Winkler-Foundation modeling. The assessment is based on comparing seismic displacement demands such as target roof displacements, peak floor displacements and inter-storey drifts. The AMC estimates are compared to results from nonlinear response history analysis (NL-RHA) and results from AMC procedure considered only first mode (AMC- 1ˢᵗ mode). The obtained results show that the AMC procedure provides reasonably accurate results in estimating seismic demands of studied buildings.**

Keywords— *Nonlinear static procedure, adaptive modal combination procedure, seismic demand, modal pushover analysis, soil structure interaction.*

I. INTRODUCTION

Although nonlinear response history analysis (NL-RHA) is considered to be the most rigorous method to determine responses of a structure due to a strong earthquake excitation, it can only be undertaken by highly qualified engineers and may be too time-consuming for typical structural design and evaluation projects. The nonlinear static procedure, or pushover analysis, thus became a popular method to estimate building responses considering inelastic behavior, although the results are not as accurate as those from NL-RHA. Nonlinear static procedures (NSPs) using the lateral force distributions recommended in ATC-40, FEMA-356 and FEMA-440 documents are now widely used in engineering practice [1][2]. 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 and inadequate to predict inelastic seismic demands in buildings when the higher 'modes' contribute to the response. To overcome these limitations, a multi-mode pushover procedure called modal pushover analysis (MPA) was proposed by Chopra and Goel (2002) [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 compared to conventional pushover analysis (Chopra and Goel, 2002; Chintanapakdee and Chopra, 2003; Chopra and Chintanapakdee, 2004; and Chopra *et al.*, 2004; and Nguyen *et al.*, 2010) [4-8].

One of critical tasks of nonlinear static procedures is to predict the target displacement of inelastic multi-degree-of-freedom structures due to a ground motion. Several approaches were proposed to estimate the target displacement by using an equivalent single-degree-of-freedom (SDF) system. The methods described in the ATC-40 and FEMA-356 guidelines are now commonly used in practice. According to the nonlinear static procedure described in ATC-40 and FEMA-356 documents, seismic demands are computed by nonlinear static analysis of a structure subjected to monotonically increasing lateral loads representing inertia forces in an earthquake (pushover analysis) with a specified, usually invariant height-wise distribution until a pre-determined target displacement is reached. The target displacement of these procedures is estimated from the deformation *D* of an equivalent inelastic SDF system which is derived from relationship between base shear force and roof displacement known as the pushover curve.

The ATC-40 presents an approach, called Capacity Spectrum Method (CSM), to estimate seismic response of inelastic SDF systems where the deformation *D* of an inelastic SDF system is determined by an iterative method which requires analysis of a sequence of equivalent linear systems with successively updated values of period and damping ratio. This method is

typically implemented graphically. However, the accuracy and convergence of the ATC-40 iterative procedure can be considerable (Chopra and Goel, 2000) [9]. The ATC-40 tends to underestimate the deformation over a wide range of periods. Unlike the ATC-40 Capacity Spectrum Method, FEMA-356 presents an alternative approach, known as Displacement Coefficient Method (DCM), in which the deformation D of an inelastic SDF system is estimated by multiplying the deformation of the corresponding linear system. The limitations on accuracy of ATC-40 Capacity Spectrum Method such as lack of convergence or large errors in some case (Chopra and Goel, 2000) [9], and the lack on research results of coefficient C_1 of FEMA-356 Displacement Coefficient Method are specified and rectified in FEMA-440 report (ATC, 2005). Then, an investigation on accuracy of improved nonlinear static procedures in FEMA-440 was carried out by Akkar and Metin, (2007) [10]. It was found that both of these improved procedures provide fairly good deformation demand estimations.

To overcome the limitation of previous mentioned methods, several improved methods were proposed and investigated. An improved Capacity Demand Diagram (CDD) method, which is originally developed by Freeman *et al.* (1975) and Freeman (1978) [11,12], based on inelastic design spectrum for estimating seismic deformation of inelastic structures using SDF systems was developed and illustrated by Chopra and Goel, (1999) [13]. Subsequently, Chopra *et al.* (2001) [14], and Chopra and Goel, (2002) [4] suggested that the contribution of the *n*th vibration mode to the target roof displacement of an inelastic MDF system in MPA procedure is calculated from the peak value of deformation of an equivalent inelastic SDF system of *n*th 'mode', determined rigorously by nonlinear response history analysis by solving the uncoupled modal response history analysis (UMRHA) governing equation for the *n*th 'mode'. For planar analysis of symmetric-plan building, the peak modal responses, each determined by pushover analysis for modal force distribution and dynamic analysis of the *n*th 'mode' inelastic SDF system, may be combined using an appropriate modal combination rule such as the square-root-of-sum-of-squares (SRSS) rule to obtain a MPA estimate of the total roof displacement.

Then, an investigation on accuracy of single-degree-of-freedom estimate of displacement for pushover analysis of buildings was carried out by Chopra *et al.* (2003) [15]. The statistics presented show that the roof displacement of a multi-story building can be determined from the deformation of the SDF system. The estimation considering first 'mode' SDF system overestimates the median roof displacement for systems subjected to large ductility demand μ, but underestimates for small μ. The bias and dispersion of this method tend to increase for longer period system for every value of μ and increasing when *P-Δ* effects are included due to gravity loads. On the other hand, considering the contribution of higher 'modes' of MPA procedure helps reduce the dispersion in the roof displacement, but it increases slightly the overestimate of roof displacement of buildings

responding far into the inelastic range (Chopra *et al.*, 2003) [15].

An alternative pushover method is the adaptive pushover procedure in which the load pattern distributions are redefined. The loading pattern is determined by modal combination rules (e.g. SRSS of modal loads) at each stage of the response during which the dynamic characteristics of the structure change, usually at each step when a new plastic hinge forms in inelastic range (Fajfar and Fischinger, 1988; Bracci *et al.*, 1997; and Gupta and Kunnath, 2000) [16-18]. In this procedure, equivalent seismic loads are calculated at each pushover step using the immediate 'mode' shape. Recently, a new adaptive pushover method, called Adaptive Modal Combination (AMC) procedure, has been developed by Kalkan and Kunnath, (2006) [19] where a set of adaptive mode-shape based inertia force patterns is applied to the structure.

When a structure is subjected to inertial loading due to earthquake ground motion, the foundation may undergo sliding, settling and rocking movements. Therefore, the soil structure interaction (SSI) has a significant effect on the vibrational properties and dynamic responses of the structural system. The calculation of structures subjected to earthquake loads is often overlooked the influence of the SSI and is often based on the assumption that the foundation is a solid block. This assumption is true if the structural system is placed on good foundation. However, there will be significant differences if the ground is weak. If the capacity of the foundation is mobilized, the soil-foundation interface will dissipate significant amounts of vibrational energy, resulting in a reduction in structural force demand [20]. Due to the above effects, it is necessary to mention the role of soil foundation in the SSI model when analyzing earthquake resistant works.

The problem of interaction between soil and structure has been paid much attention in recent years by the accuracy and reasonableness of the model. It can simulate the simultaneous operation of the structure and soil during subjected to earthquake loads. As a result, the models which have calculation to the impact of SSI have working very close to the fact. In many cases, the soil is assumed to be elastic material. This assumption does not accurately reflect the properties of the soil in fact. Presently, with the appearance of soil characterization models and the development of analytical methods have allowed simulations of soil - structure behavior under the effect of earthquake ground motion.

The main objective of this study is to assess the bias and accuracy of the adaptive modal combination procedure, which is adapted from MPA procedure, for seismic evaluation of buildings. Three-, 9-, 18-storey buildings were analyzed due to two sets of strong ground motions having 2% and 10% probability of being exceeded in 50 years. The influence of soil-structure interaction in the analysis was also considered by using Beam-on-Nonlinear-Winkler-Foundation modeling [20]. The assessment is based on comparing seismic displacement demands such as target roof displacements, peak floor displacements and inter-storey drifts. The peak modal responses are combined by using Square-Root-of-Sum-of-Squares (SRSS rules. The AMC

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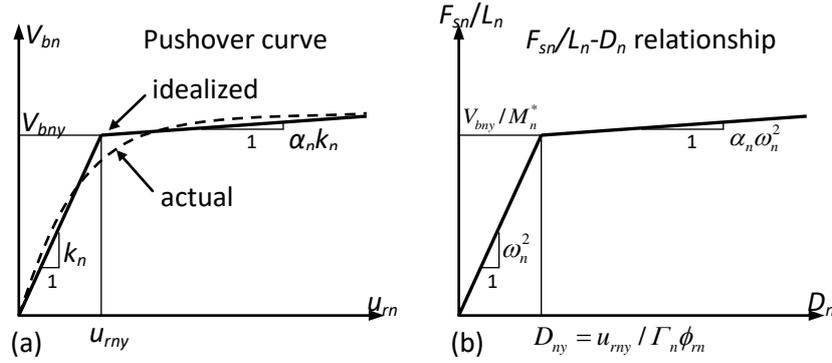
Figure 1. (a) Pushover curve and (b) Force and deformation relationship of SDF system.

estimates are compared to results from nonlinear response history analysis (NL-RHA) and results from AMC procedure considered only first mode (AMC- 1ˢᵗ mode).

II. METHOD

The Adaptive Modal Combination (AMC), which has been proposed by Kalkan and Kunnath (2006) [19], is an extension of Modal Pushover Analysis procedure where a set of adaptive mode-shape based inertia force patterns is applied to the structure. A step-by-step summary of the AMC procedure used in this study to estimate the seismic demands for building is presented as a sequence of steps:

(1) Compute the natural frequencies, ω_n, and mode shape vectors of the building, ϕ_n, for each step of pushover analysis.

(2) For the nth-'mode', develop the base-shear—roof-displacement ($V_{bn} - u_{rn}$) pushover curve by nonlinear static analysis of the building using the force distribution $\mathbf{s}_n^* = \mathbf{m}\phi_n$ where $\mathbf{m}$ is the mass matrix and ϕ_n is adaptive mode shape at each step of pushover analysis.

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

(4) Convert the idealized pushover curve to the force–deformation ($F_{sn}/L_n - D_n$) relation of the nth-'mode' inelastic SDF system and determine the elastic modal frequency ω_n, and yield deformation D_{ny}. The nth-'mode' inelastic SDF system is defined by the force–deformation curve of Figure 1b (with post-yield stiffness ratio α_n) and damping ratio ζ_n specified for the nth 'mode'. Where $M_n^* = \Gamma_n L_n$ is the effective modal mass,

$$L_n = \phi_n^T \mathbf{m}\iota, \quad \Gamma_n = \frac{\phi_n^T \mathbf{m}\iota}{\phi_n^T \mathbf{m}\phi_n}, \text{ and each element of the}$$

influence vector ι is equal to unity.

(5) Compute the peak deformation, $D_n \equiv \max_{\forall t} |D_n(t)|$, of the nth-'mode' inelastic SDF system with force–deformation relation of Fig. 1b due to ground excitation $\ddot{u}_g(t)$ by solving:

$$\ddot{D}_n + 2\zeta_n\omega_n\dot{D}_n + \frac{F_{sn}(D_n, \dot{D}_n)}{L_n} = -\ddot{u}_g(t) \tag{1}$$

(6) Calculate the peak roof displacement u_{rno} associated with the nth-'mode' inelastic SDF system from

$$u_{rno} = \Gamma_n \phi_{rn} D_n \tag{2}$$

(7) Extract other desired responses, r_{no}, from the pushover database when roof displacement equal to u_{rno}.

(8) Repeat Steps 2–7 for as many 'modes' as required for sufficient accuracy; usually the first two or three 'modes' will suffice for buildings shorter than 10 stories.

(9) Determine the total response r_{MPA} by combining the peak 'modal' responses using appropriate modal combination rule, e.g., Square-Root-of-Sum-of-Squares (SRSS) as shown by Equation (3) or Complete Quadratic Combination (CQC) rule:

$$r_{MPA} = \sqrt{\sum_{n=1}^{j} r_{no}^2} \tag{3}$$

where j is the number of 'modes' included.

III. STRUTURAL SYSTEMS

Analyses of 3-, 9-, and 18-story buildings are presented to evaluate the bias and accuracy of AMC procedure. These building systems are model cases studied by Goel and Chopra (1997) [21]. They represent the lateral load resisting systems of buildings with plan symmetric about two orthogonal directions and subjected to earthquake ground motion along an axis of symmetry. All the generic frames used in this study are designed according to strong-column weak beam concept. This results in beam-sway type of plastic mechanism and ensures optimal performance of the structure under extreme seismic loading thereby ensuring premature-failure of the structure due to formation of story mechanism. In other words, the beams and columns are designed such that inelastic deformations occur only at the beam ends and the base of the ground floor columns. All other columns are assumed to remain elastic.

Inelasticity in the generic frames being considered is modeled by concentrated plastic hinges. The two ends of the beams of all the stories and the base of the column in the ground floor are each assigned a concentrated rotational plastic hinge. The moment-rotation

relationship of the plastic hinges is represented by symmetric, non-degrading rigid-plastic model with 3% strain hardening. A schematic diagram of the moment-rotation relationship of a typical plastic hinge is shown in Figure 2.

The frames considered are a set of generic one-bay frames of three different heights: 3, 9, and 18 stories. Each story has a height equal to 144 inches and the bay length of each frame is equal to 288 inches. In all the 3 generic frames considered in this study, mass equal to 200 kips is assigned to each story. The mass at each storey is considered to be lumped mass. The elevation view of all systems is shown in Figure 3. Analytical models were created to analyze these buildings whose details can be found in Goel and Chopra (1997) [21]. Rayleigh damping model was used with 5% critical damping ratios for the first two modes, according to common practice for code designed steel structures [22]. Nonlinear static and dynamic analyses were carried out using the computer program Opensees [23].
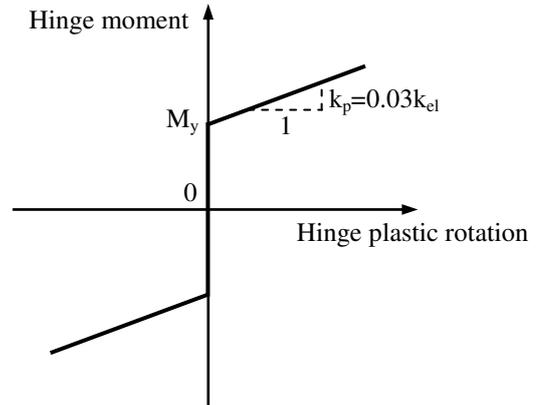
Figure 2. Moment – rotation relationship of a rigid-plastic hinge with 3% strain hardening ratio.
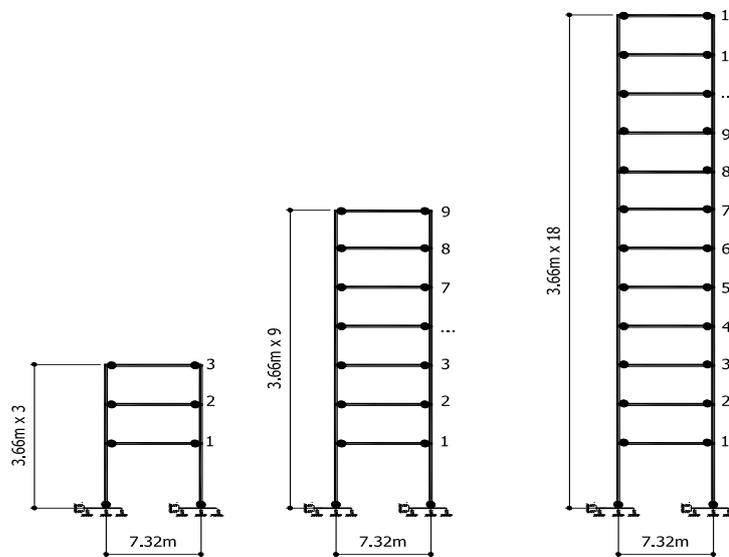
Figure 3. The elevation view of all systems.
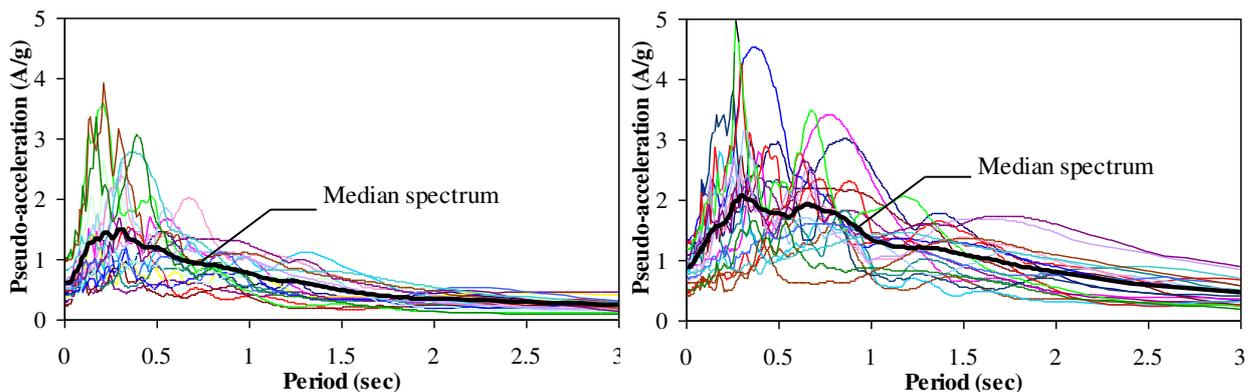
Figure 4. Pseudo-acceleration spectra of (a) LA10/50, and (b) LA2/50 set of ground motions.

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Table 1. Set of ground motions having 2% probability of being exceeded in 50 years (LA2/50)

Record	Earthquake/Recording station	Earthquake magnitude	Distance (km)	Scaling factor	PGA (cm/sec²)
LA21	1995 Kobe	6.9	3.4	1.15	1258
LA22	1995 Kobe	6.9	3.4	1.15	903
LA23	1989 Loma Prieta	7.0	3.5	0.82	410
LA24	1989 Loma Prieta	7.0	3.5	0.82	464
LA25	1994 Northridge	6.7	7.5	1.29	854
LA26	1994 Northridge	6.7	7.5	1.29	925
LA27	1994 Northridge	6.7	6.4	1.61	909
LA28	1994 Northridge	6.7	6.4	1.61	1304
LA29	1974 Tabas	7.4	1.2	1.08	793
LA30	1974 Tabas	7.4	1.2	1.08	973
LA31	Elysian Park (simulated)	7.1	17.5	1.43	1271
LA32	Elysian Park (simulated)	7.1	17.5	1.43	1164
LA33	Elysian Park (simulated)	7.1	10.7	0.97	767
LA34	Elysian Park (simulated)	7.1	10.7	0.97	668
LA35	Elysian Park (simulated)	7.1	11.2	1.10	973
LA36	Elysian Park (simulated)	7.1	11.2	1.10	1079
LA37	Palos Verdes (simulated)	7.1	1.5	0.90	698
LA38	Palos Verdes (simulated)	7.1	1.5	0.90	761
LA39	Palos Verdes (simulated)	7.1	1.5	0.88	491
LA40	Palos Verdes (simulated)	7.1	1.5	0.88	613

Table 2. Median ductility factors for building models calculated from NL-RHA estimate of peak roof displacement

Set of records	Building model		
	3-story	9-story	18-story
LA10/50	2.35	2.31	2.04
LA2/50	3.73	3.63	3.50

In order to consider the influence of soil-structure interaction in this study, a numerical model based on the Beam-on-Nonlinear-Winkler Foundation (BNWF) concept is used to capture the above mentioned foundation behavior (Raychowdhury, P. 2008) [20]. The BNWF model is selected due to its relative simplicity, ease of calibration, and acceptance in engineering practice. The soil-foundation interface is assumed to be an assembly of discrete, nonlinear elements composed of springs, dashpots and gap elements. Inelastic nonlinear springs are used to describe the resistance of soil foundation in horizontal and vertical directions.

The foundation model used in this study is considered as an elastic beam element and is declared with the "elasticBeamColumn" element in the OPENSEES software. Each node of the element has three degrees of freedom representing loads and deformations in horizontal, vertical, rotational angle. This element is supported by separate nonlinear springs. The springs are declared with the element "zeroLength Element". For the interaction problem, Boulanger *et al.* [24] has proposed a model of QzSimple1, PySimple1 and TzSimple1 material to simulate the response of the soil to the pile during the load bearing process. These materials are attached to the spring element to create a nonlinear behavior of the spring. Raychowdhury (2008) [20] proposed models of QzSimple2, PySimple2, TzSimple2 materials that have been modified from the QzSimple1,

PySimple1, and TzSimple1 material models by the result of experiments in fact. This correction aims to create a soil response that impacts to the foundation structure exactly during the load bearing process. The BNWF model can simulate the behavior of soil-foundation structures through nonlinear behavior of ground (geometrical and material nonlinearities).

IV. GROUND MOTIONS AND RESPONSE STATISTICS

Two sets of ground motions, referred as LA2/50 and LA10/50, corresponding to 2% and 10% probabilities of exceedence in a 50-year period are used in this study. These ground motions were compiled by the SAC Phase II Steel Project for a site in Los Angeles, California (Somerville *et al.*, 1997) [25]. 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 near-fault and far-fault records. The pseudo-acceleration spectra for the two sets of ground motions are shown in Figure 4 together with the median spectra (black solid lines). Table 1 provides the information of LA2/50 set of records including: recording station, earthquake

magnitude, distance, scaling factor, and peak ground acceleration (PGA).

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

$$\hat{x} = \exp\left[\frac{\sum_{i=1}^{n} \ln x_i}{n}\right] \tag{4}$$

$$\delta = \sqrt{\frac{\sum_{i=1}^{n}\left(\ln x_i - \ln \hat{x}\right)^2}{n-1}} \tag{5}$$

V. EVALUATION OF ADAPTIVE MODAL COMBINATION PROCEDURE

The bias and accuracy of the AMC procedure applied to buildings are evaluated by comparing the target roof displacements, peak floor (or roof) displacements and inter-story drifts compared to more accurate results from nonlinear response history analysis (NL-RHA) and results from AMC procedure considered only first mode (AMC- 1st mode).

1. Target Roof Displacements

Pushover curves, which show the relationship between the base shear force and the roof displacement, for the 3-, 9-, and 18-story buildings due to the first 'mode' load pattern are plotted in Figure 5.

On each pushover curve, diamond markers show the peak roof displacements of buildings determined by NL-RHA of the equivalent single-degree-of-freedom (SDF) system due to 20 records in each set of ground motions. The ductility factors of the first 'mode', defined here as the ratio between median of peak roof displacements
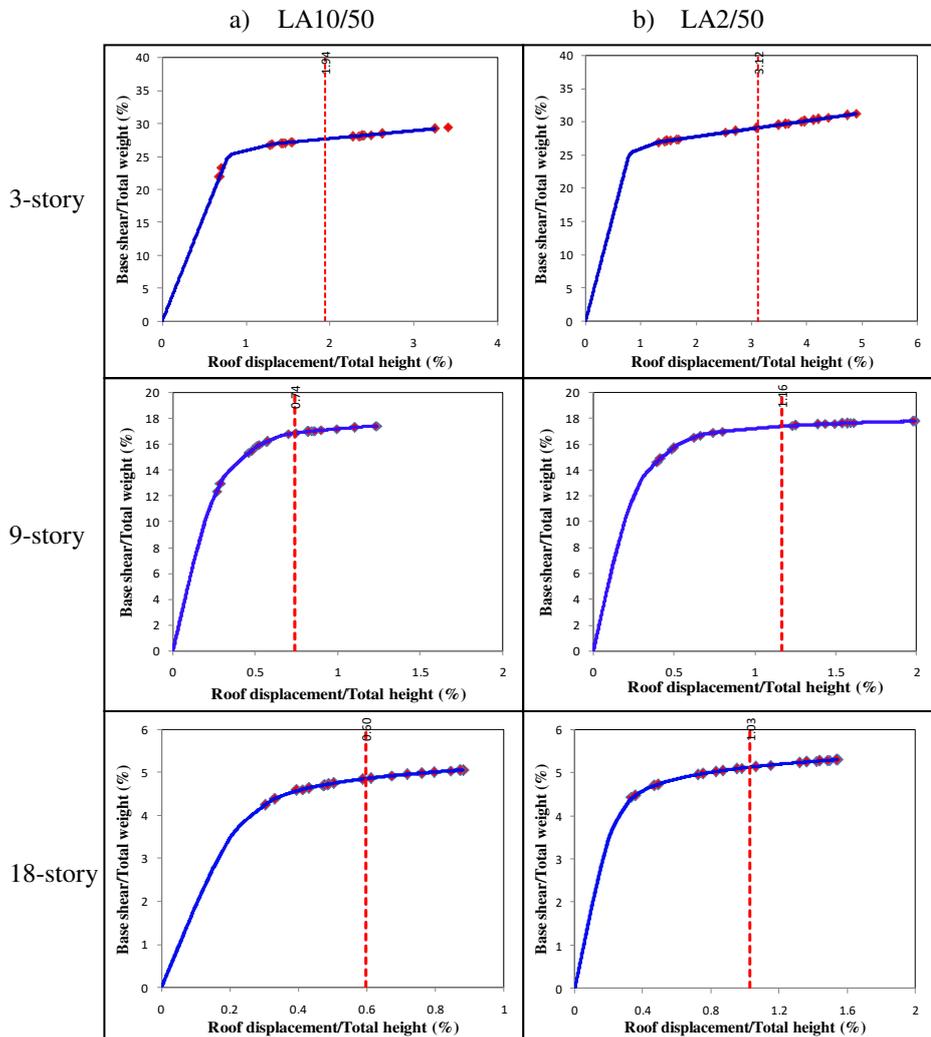
a) LA10/50   b) LA2/50

Figure 5. First 'mode' pushover curves of 3-, 9-, and 18-story buildings due to (a) LA10/50 and (b) LA2/50 ground motions.

determined by NL-RHA and yield roof displacement estimated by first 'mode' load pattern, are about 2.04 to 2.35 for LA10/50 and 3.50 to 3.73 for LA2/50 ground motions, respectively. Table 2 shows the median ductility factors for these buildings calculated from NL-RHA estimate. The median ductility factor decreases when the building height increases.

2. Peak floor/roof displacements

The responses of studied buildings to the two sets of ground motions were determined by AMC-1ˢᵗ mode, AMC procedure and by nonlinear response history analysis (NL-RHA). The AMC was considered as many modes as to include participating mass at least 90% of total mass. For the structures in this study, the contribution of the first two 'modes' for 3-story building, three 'modes' for 9-story building, and five 'modes' for 18-story building were considered to estimate the seismic demands. The combined values of floor displacements and story drifts were computed by using SRSS modal combination rule.

The peak floor/roof displacement demands from three methods are compared in Figure 6. These results lead to the following observations for building system. The AMC-1ˢᵗ mode and AMC can estimate the peak floor displacements reasonably well for 3-story and 9-story buildings with a tendency to slightly overestimate

floor/roof displacements for upper stories of 9-story building due to LA2/50 set of ground motions. The contributions of higher 'modes' of AMC procedure to floor displacements are significant for 9-story building due to the two sets of ground motions. The AMC procedure can well predict floor displacements of upper and lower stories of 18-story building compared to NL-RHA whereas the AMC-1ˢᵗ mode procedure tends to underestimate peak floor displacements of 18-story building considered in this study.

3. Story drift demands

The contributions of higher 'modes' in estimating the story drifts of AMC procedure are more significant, especially in lower and upper stories of tall buildings. Figure 7 shows that the story drift demands of 3-, 9- and 18-story buildings predicted by AMC are able to follow the NL-RHA results whereas the AMC first 'mode' alone is inadequate. With three or five 'modes' included, the story drifts estimated by AMC is generally similar to the results from nonlinear RHA. However, the AMC story drift results including two 'modes' for 3-story are close to AMC—1ˢᵗ mode results indicating that the contributions of higher 'modes' are not significant for this building. Both one 'mode' pushover analysis and AMC can estimate the response of structures reasonably well. The AMC and AMC – 1ˢᵗ mode procedures tend to
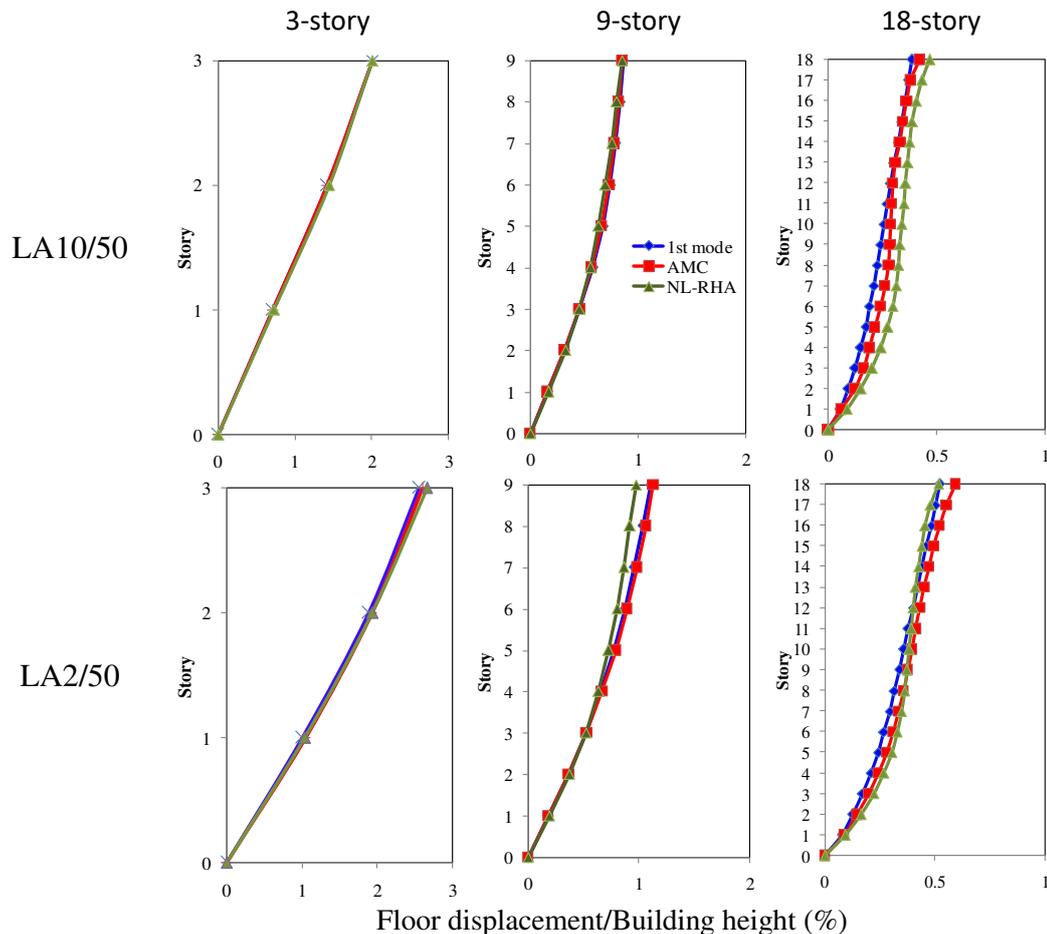
Figure 6. Median floor displacements of 3-, 9-, and 18-story buildings determined by one 'mode' pushover analysis, AMC and NL-RHA due to LA10/50 (first row), and LA2/50 (second row) ground motions.

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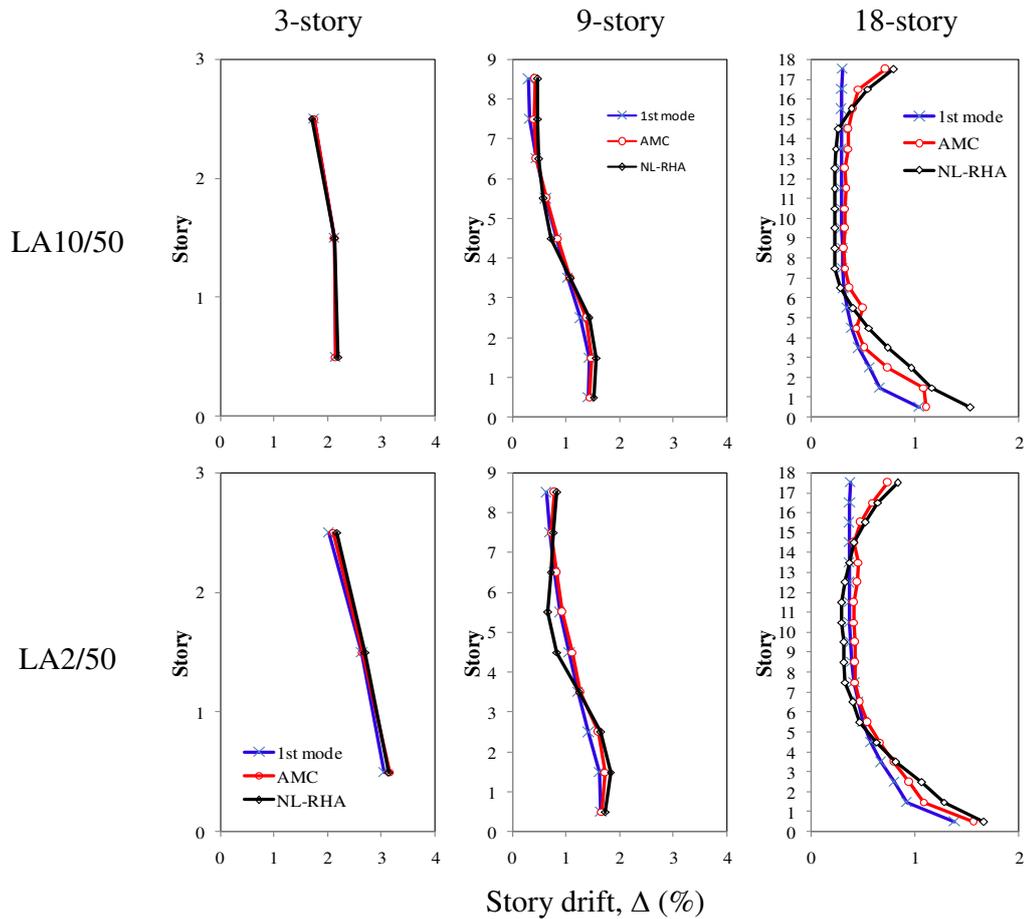
Figure 7. Median story drifts of 3-, 9-, and 18-story buildings determined by AMC- 1ˢᵗ mode, AMC and NL-RHA due to LA10/50 (first row), and LA2/50 (second row) ground motions.

underestimate story drifts in lower and upper stories, and overestimate story drifts in middle stories of 9- and 18-story buildings due to the two sets of ground motions.

The bias of AMC nonlinear static procedures in estimating seismic demands tends to increase for stronger excitations and the variation of AMC bias in estimating seismic demands along building height primarily depends on the building height rather than the intensity of ground motions. The bias of AMC and AMC-1ˢᵗ mode in estimating peak story drifts at an individual story can be as large as 50% at certain locations for LA10/50 and LA2/50 ground motions.

VI. CONCLUSIONS

The following conclusions are obtained from the accuracy assessment of Adaptive Modal Combination procedure in estimating seismic demands of buildings considered soil structure interaction using LA10/50 and LA2/50 sets of intense ground motions. These conclusions are based on a comparison of AMC estimates of seismic demands and corresponding values determined by NL-RHA for 3-, 9-, and 18-story buildings which were designed to meet seismic code criteria.

(1) The equivalent bilinear SDF systems of AMC procedure can estimate peak roof displacement quite accurately with bias no larger than 11% and 15.6% for LA10/50 and LA2/50 sets of ground motions, respectively.

(2) The story drift demands predicted by AMC are able to follow the NL-RHA results. However, the higher 'modes' contributions of this procedure in response of 3-story building are generally not significant, so the first 'mode' alone may be adequate.

(3) The bias and dispersion of AMC procedure in estimating seismic demands tends to increase for taller buildings and stronger excitations. The height-wise variation of bias primarily depends on structural properties, e.g., building height, rather than the intensity of ground motions.

(4) The bias of AMC and AMC-1st mode in estimating peak story drifts at an individual story can be as large as 50% at certain locations for LA10/50 and LA2/50 ground motions.

VII. ACKNOWLEDGEMENTS

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