

Lateral load distribution in nonlinear static procedures for seismic design

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Abstract

Nonlinear static analysis using pushover procedures are becoming increasingly common in engineering practice for seismic evaluation of building structures. Various invariant distributions of lateral forces are recommended in FEMA-356 (2000) to perform a pushover analysis. However, the use of these invariant force distributions does not adequately represent the effects of varying dynamic characteristics during the inelastic response or the influence of higher modes. More recently, new approaches to combining lateral load distributions have been proposed to overcome some of the drawbacks in FEMA procedures. In this paper the validity and applicability of several lateral load configurations are assessed by comparison of the pushover response of eight and twelve story steel moment frame buildings with benchmark solutions based on nonlinear time history analyses. The study reveals the suitability of using unique modal combinations to determine lateral load configurations that best approximate the inter-story demands in multistory frame buildings subjected to seismic loads.

Introduction

Although current seismic design practice is still governed by force-based design principles, a common trend in structural earthquake engineering practice is to use performance-based seismic evaluation methods for the estimation of inelastic deformation demands in structural members. A widely used and popular approach to establish these demands is a “pushover” analysis in which a model of the building structure is subjected to an inverted triangular distribution of lateral forces. While such a load distribution may be adequate for regular and low-rise structures whose response is primarily in their fundamental mode, it can produce misleading results for structures with significant higher mode contributions. This accentuates the need for improved procedures that addresses current drawbacks in the lateral load patterns used in pushover analyses. New lateral load configurations using modal combinations originally proposed by Kunnath (2004) and some variations of the approach are investigated in this paper. In all cases, the computed peak response is compared to FEMA-based patterns and to results from nonlinear time-history analyses.

Description of Buildings Used in Evaluation

Two special moment resisting steel frame buildings were selected as representative case studies to carry out the evaluation of different later load distributions. The building designs are based on a configuration presented in the SEAOC Seismic Design Manual (SEAOC, 2000). The original design presented in the manual pertains to a four-story building. The building's lateral force resisting system is composed of steel perimeter moment resisting frames (MRF).

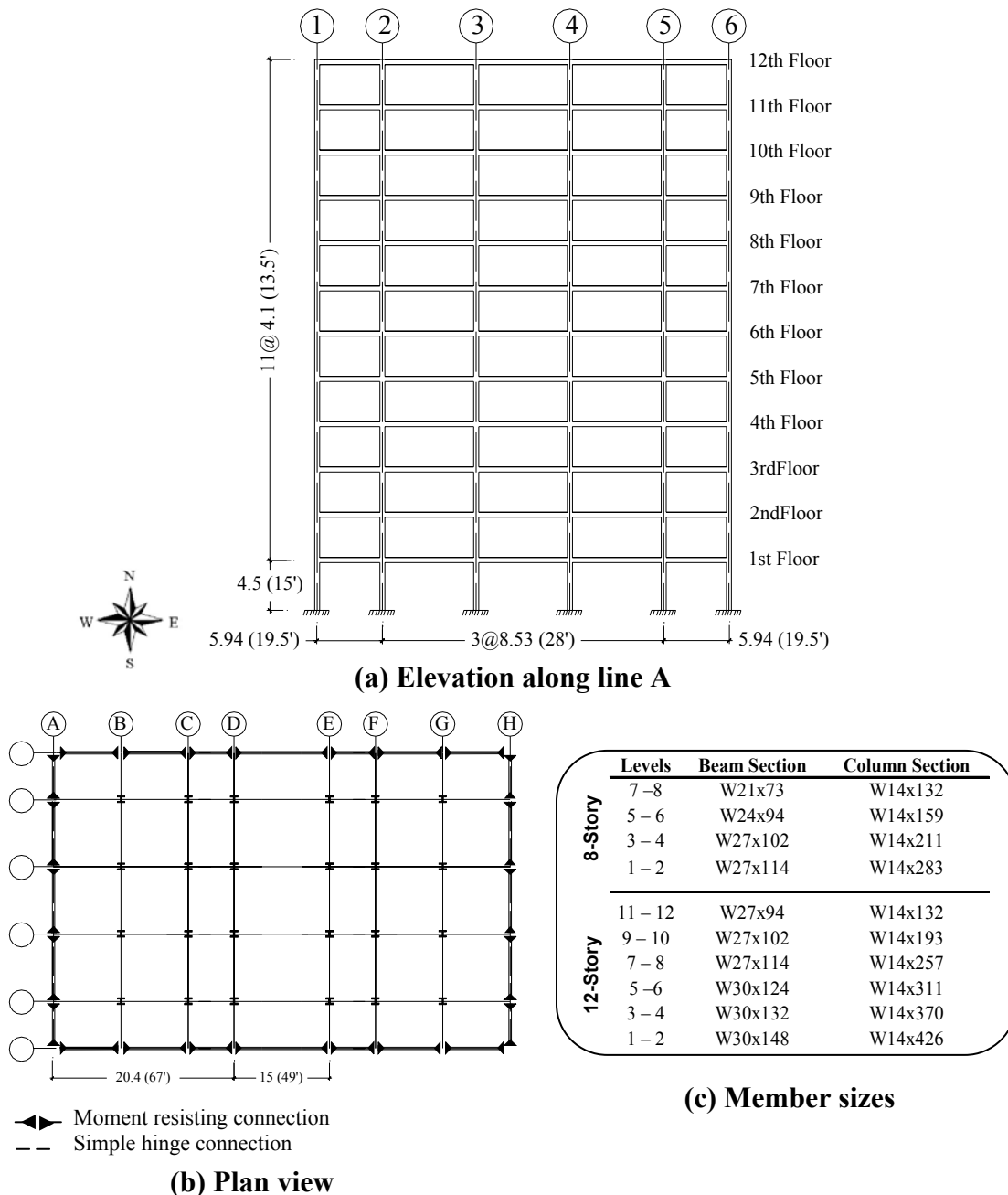


Figure 1. Structural details of 8 and 12 story buildings (units in meters)

In this study, the same floor plan was extended to eight and twelve stories. These buildings are 37.5 m (123') by 56 m (200') in plan and 33.4 m (109.5') and 50 m (163.5') in elevation for eight and twelve story cases, respectively. In the north-south (N-S) direction the interior bays are 8.5 m (28') and exterior bays are 6m (19.5') with a total of five bays. The floor-plan and elevation view of the building are illustrated in Figure 1. Also shown in this figure are the member sizes for the 8 and 12 story buildings.

The design roof dead load is 939 tons (2066 kips) and the dead load of each of the remaining floors is 1016 tons (2235 kips). The yield strength of steel is assumed to be $f_y = 345$ Mpa (50 ksi) for all structural members. Since the plan of structures is essentially symmetric, only a single frame in the transverse direction (line A in Figure 1) is analyzed. Composite action of floor slabs is not taken into consideration. Since the intent of the SEAOC design manual is to simply illustrate the design process, the final design presented in the manual is not optimized. In the present study, the SEAOC design was modified to result in optimized member sizes that conform to the requirements of the UBC (1997) provisions. The design base shears for the eight and twelve story structures are approximately 4.8% of their respective building weights.

Ground Motions

In order to establish a benchmark response to examine the validity of the different pushover procedures based on invariant load distributions, nonlinear time-history analyses were performed on the same set of buildings. The seismic excitation used for nonlinear time history evaluations is defined by a set of seven strong ground motions. These ground motion records are recommended by ATC-40 (1996). All ground motions were recorded from California earthquakes having a magnitude range of 6.6 to 7.5 at soil sites and at distances of 4.5 to 31 km. Details of these records are presented in Table 1, and their five-percent damped elastic acceleration and displacement response spectra along with their median spectra are presented in Figure 2.

Table 1. Details of ground motion recordings

Eq. No	Magnitude	Year	Earthquake	Recording Station	PGA (g)	Distance (km) *
1	6.6	1971	San Fernando	Station 241	0.26	16.5
2	6.6	1971	San Fernando	Station 458	0.12	18.3
3	7.1	1989	Loma Prieta	Hollister, South & Pine	0.18	17.2
4	7.1	1989	Loma Prieta	Gilroy #2	0.32	4.5
5	7.5	1992	Landers	Yermo	0.15	31.0
6	7.5	1992	Landers	Joshua Park	0.28	10.0
7	6.7	1994	Northridge	Century City LACC North	0.26	23.7

* Closest distance to fault

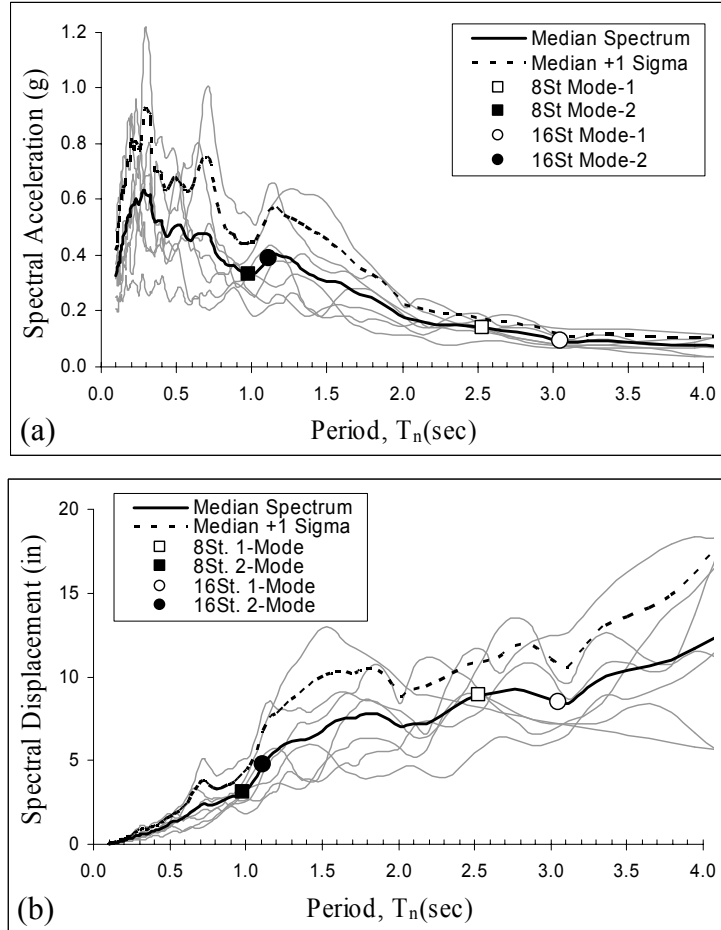


Figure 2. (a) Pseudo spectral acceleration spectra, and (b) Spectral displacement spectra

Lateral Load Configurations

Several lateral load cases were evaluated in this study. The first set of three patterns is derived from FEMA-356. The following notations are used to describe these patterns:

NSP-1: The buildings are subjected to a lateral load distributed across the height of the building based on the following formula specified in FEMA-356:

$$F_x = \frac{W_x * h_x^k}{\sum W_i * h_i^k} V \quad (1)$$

In the above expression, F_i is the applied lateral force at level 'x', W is the story weight, h is the story height and V is the design base shear. This results in an inverted

triangular distribution of the lateral load when the period-dependent power k is set equal to unity.

NSP-2: A uniform lateral load distribution consisting of forces that are proportional to the story masses at each story level.

NSP-3: A lateral load distribution that is proportional to the story shear distribution determined by combining modal responses from a response spectrum analysis of the building using the appropriate ground motion spectrum. Herein, we utilized the BSE-2 hazard level design spectrum (Figure 3) for calculation of story shear forces.

The next set of lateral load configurations involved a combination of modes as proposed in Kunnath (2004). The modal combination procedures involve identifying appropriate modes to include in the analysis and the manner in which the combination will be carried out. Two different methods of combination were used in this study. These methods require an eigenvalue analysis of the structure to be carried out at the initial elastic state and possibly again in their inelastic states.

Modal Combination Procedure 1, (MCP-1): In this procedure the spatial variation of applied forces can be determined from Equation-1.

$$F_j = \sum \alpha_n \Gamma_n \mathbf{m} \Phi_n S_a(\zeta_n, T_n) \quad (1)$$

where α_n is a modification factor that can assume positive or negative values; Φ_n is the mode shape vector corresponding to mode n ; S_a is the spectral acceleration at the period corresponding to mode n ; and

$$\Gamma = ([\Phi]^T [m] \{t\}) / M_n \text{ in which } M_n = [\Phi]^T [m] [\Phi] \quad (2)$$

If only the first two modes were combined, then Equation 1 would have the following form:

$$F_j = \alpha_1 \Gamma_1 \mathbf{m} \Phi_1 S_a(\zeta_1, T_1) \pm \alpha_2 \Gamma_2 \mathbf{m} \Phi_2 S_a(\zeta_2, T_2) \quad (3)$$

Therefore the procedure requires multiple pushover analyses wherein a range of modal load patterns are applied. In order to arrive at estimates of deformation and force demands, it is necessary to consider peak demands at each story level and then establish an envelope of demand values for use in performance based-evaluation.

Modal Combination Procedure 2, (MCP-2): An alternative approach to the above combination scheme is also investigated. Here, the lateral forces are determined in a similar manner to the previous technique for each independent mode and then combined using an appropriate combination rule such as SRSS. The individual invariant load pattern is computed from the following expression:

$$F_{ij} = \Gamma_j \phi_{ij} W_i S_a(j) \quad (4)$$

where i is the floor number and j is the mode number. The key aspect of this procedure is the varying target displacement for each lateral loading case. Since the first mode behavior of a structure is generally more dominant than the higher modes, the calculated target displacement is kept constant for the first mode and scaled for the other modes based on their corresponding spectral displacement values obtained from response spectrum analysis (Figure 2b). Then the pushover procedure is applied considering each individual invariant load distribution based on mode shapes. Further details on this procedure are given elsewhere (Kalkan et al., 2004) and only representative results are presented in this paper.

Details of Evaluation Procedure

For the pushover analyses, two-dimensional computer models of each building were developed for use in OpenSees (2003). The program utilizes the layered ‘*fiber*’ approach for inelastic frame analysis. It has also the feature of representing the spread of inelasticity along the member length as well as section level. In the finite element domain, all members were simulated as nonlinear beam-column elements that accounts for axial-moment interaction. The target displacement for each building model was computed using the provisions in FEMA-356. Accordingly the site-specific spectrum corresponding to BSE-2 hazard level was used (Figure 3).

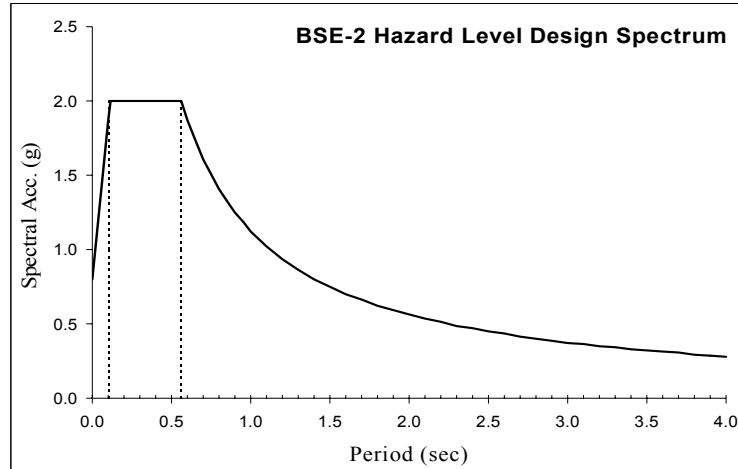


Figure 3. Response spectrum for BSE-2 hazard level

The target displacements of 1.07m (42.5”) and 1.30m (51.0”) were computed using Equation (3-15) in FEMA-356 for the 8 and 12-story buildings, respectively. Each building model was subjected to the different lateral load configurations outlined above. These lateral loads were incrementally applied to structures till the roof node reached the specified target displacement.

Summary of Results

Figure 4a-4b and 7a-7b show the peak displacement and peak inter-story drift profiles obtained from nonlinear time history analyses of the seven ground motions for 8 and 12 storey structures, respectively. For transient analyses, records were scaled so that their peak displacements are comparable to target displacements used in pushover analyses. The median and 84 percentile curves of peak displacement and inter-story drift profiles are presented in Figures 5a-5b and 8a-8b for two of the buildings. These results are next used as benchmark solutions for comparing pushover analyses results.

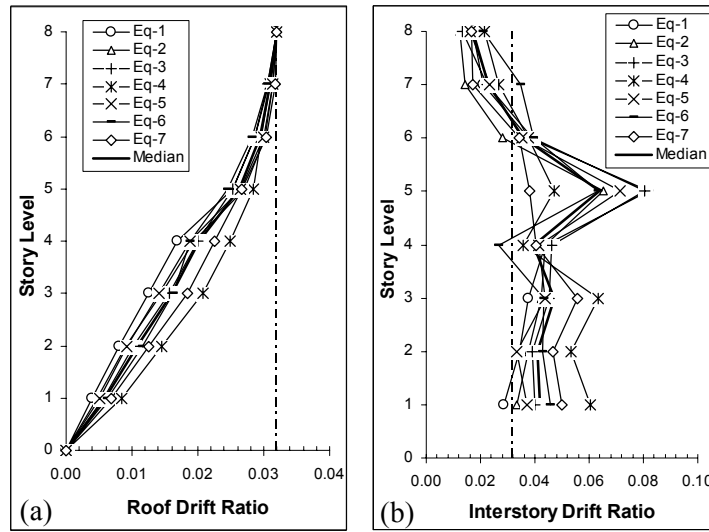


Figure 4. Nonlinear time history analysis results for 8-story building; (a) Roof drift ratio; (b) Inter-story drift ratio

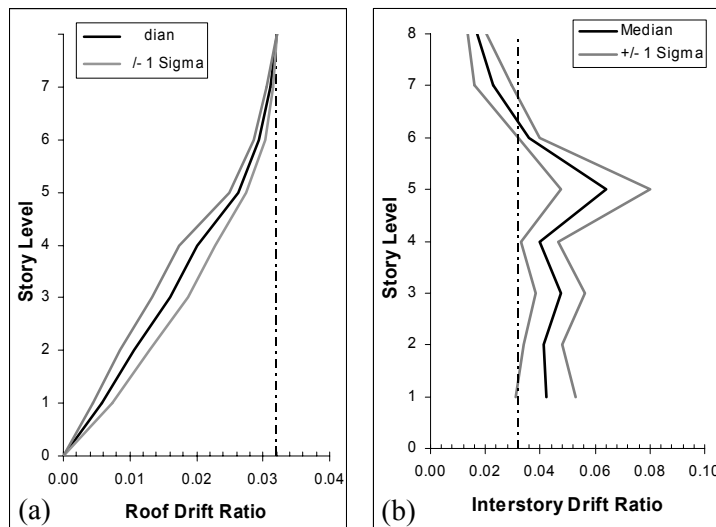


Figure 5. Nonlinear time history analysis results for 8-story building; Median and 84 percentile curves of (a) Roof drift ratio; (b) Inter-story drift ratio

Comparison of results (Figures 6a and 9a) reveal that peak displacements are generally well represented by FEMA NSP-1 procedure that is closely analogous to the elastic first mode loading pattern. The remaining two patterns (NSP-2 and 3) overestimated the peak displacement at almost all levels. The plot of peak inter-story drift, on the other hand, clearly highlights the inability of FEMA load patterns to predict this critical design parameter (Figures 6b and 9b). An important consideration in evaluating a pushover method, therefore, is its ability to predict inter-story drifts rather than roof displacements. Consequently, the concept of a roof drift ductility factor is not meaningful in the design or assessment of structures since the controlling parameter may be local story failure mechanism.

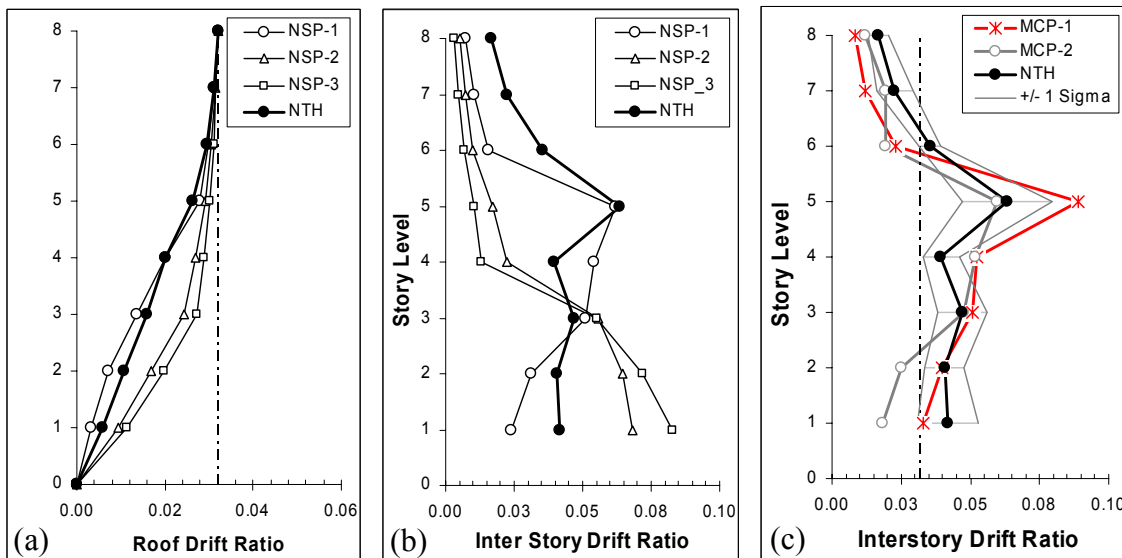


Figure 6. Nonlinear static pushover analysis results for 8-story building; (a-b) FEMA procedures; (c) Modal combination procedures

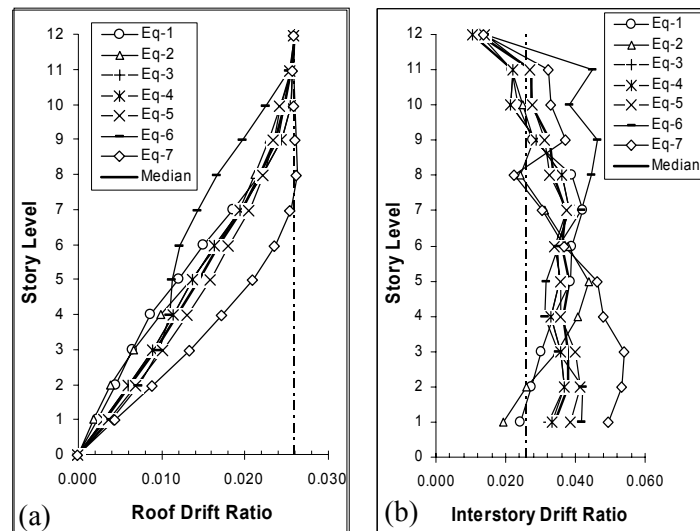


Figure 7. Nonlinear time history analysis results for 12-story building; (a) Roof drift ratio; (b) Inter-story drift ratio

It is clear from the findings reported here that the proposed modal-combination procedures for capturing the inter-story drifts are significantly better than those of single mode nonlinear static procedures (Figures 6c and 8c).

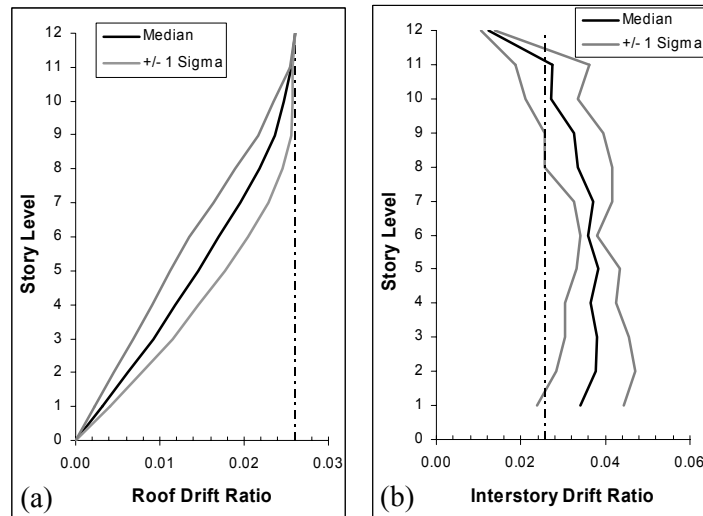


Figure 8. Nonlinear time history analysis results for 12-story building; Median and 84 percentile curves of (a) Roof drift ratio; (b) Inter-story drift ratio

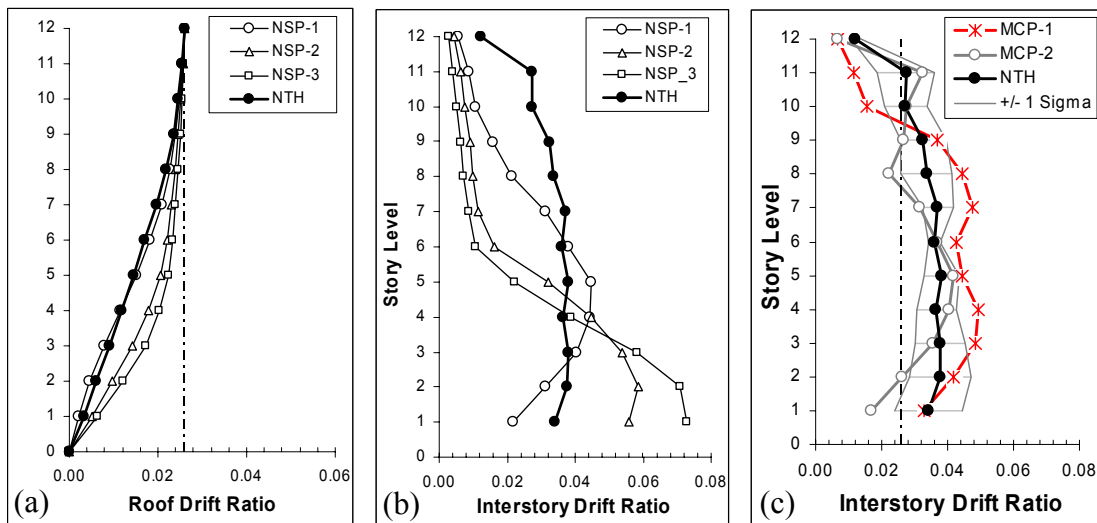


Figure 9. Nonlinear static pushover analysis results for 12-story building; (a-b) FEMA procedures; (c) Modal combination procedures

Conclusions

Given the increasing use of nonlinear static pushover analysis in engineering practice, the aim of the present paper is to develop alternative multi-mode pushover analysis procedures in an attempt to better estimate the critical inelastic response quantities such as inter-story drift and plastic hinge rotations. Various practically applicable

modal combination techniques proposed for that purpose, and these procedures intentionally avoid the complexity of adaptive methods by using invariant load patterns.

This study also indicates that simple pushover methods such as those recommended in FEMA-356 are incapable of predicting the story level at which the critical demands occur. On the other hand, the results of modal combination procedures based on our ongoing research appear to be promising in terms of better estimating the peak values of the major inelastic response quantities such as lateral displacement, inter-story drifts, and plastic hinge rotations (not reported here). The validity of the proposed procedures should be evaluated through statistical studies considering various structural systems and ground motions.

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