EVALUATION OF NONLINEAR STATIC PROCEDURES USING STRONG-MOTION BUILDING RECORDS

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Abstract

The objective of this investigation is to evaluate the FEMA-356 Nonlinear Static Procedure (NSP) and a recently developed Modal Pushover Analysis (MPA) procedure using recorded motions of buildings that were damaged during the 1994 Northridge earthquake. It is found the FEMA-356 NSP typically underestimates the drifts in upper stories and overestimates them in lower stories. The MPA procedure provides much-improved estimates of the response compared to the FEMA-356 NSP. In particular, the MPA procedure, unlike the FEMA-356 NSP, is able to capture the effects of higher modes.

Introduction

Estimating seismic demands at low performance levels, such as life safety and collapse prevention, requires explicit consideration of inelastic behavior of the structure. While nonlinear response history analysis (RHA) is the most rigorous procedure to compute seismic demands, current civil engineering practice prefers to use the nonlinear static procedure (NSP) or pushover analysis specified in the FEMA documents. In early version of FEMA NSP procedure (FEMA, 1997), the seismic demands are computed by nonlinear static analysis of the structure subjected to monotonically increasing lateral forces with an invariant height-wise distribution until a predetermined target displacement is reached. Both the force distribution and target displacement are based on the assumption that the response is controlled by the fundamental mode and that the mode shape remains unchanged after the structure yields.

Obviously, after the structure yields both assumptions are approximate, but investigations (Fajfar and Gaspersic, 1996; Gupta and Krawinkler, 1999; Maison and Bonowitz, 1999; Skokan and Hart, 2000) have led to good estimates of seismic demands. However, such satisfactory predictions of seismic demands are mostly restricted to low- and medium-rise structures for which higher mode effects are likely to be minimal and the inelastic action is distributed throughout the height of the structure (Krawinkler and Seneviratna, 1998).

None of the invariant force distributions can account for a redistribution of inertia forces because of structural yielding and the associated changes in the vibration properties of the structure. To overcome this limitation, several researchers have proposed adaptive force distributions that attempt to follow more closely the time-variant distributions of inertia forces (Bracci et al., 1997; Gupta and Kunnath, 2000). The most recent version of the FEMA NSP (FEMA, 2000), denoted as FEMA-356 NSP, includes one adaptive distribution in the list of lateral load pattern from which two are selected (details are provided latter). While these adaptive force distributions may provide better estimates of seismic demands (Gupta and

Kunnath, 2000), they are conceptually complicated, computationally demanding for routine application in structural engineering practice, and require special purpose computer program to carry out the step-by-step analysis.

Attempts have also been made to consider more than the fundamental vibration mode in pushover analysis. While the Multi-Mode Pushover (MMP) procedure (Paret et al., 1996; Sasaki et al., 1998) provided information on possible failure mechanisms due to higher modes, which may be missed by the standard NSP analyses, other information of interest in the design process, such as story drifts and plastic rotations, could not be computed by this procedure. The "sum-difference" method (Matsumori et al., 1999; Kunnath and Gupta, 2000) provided "useful" information (Kunnath and Gupta, 2000) but lacks theoretical basis.

Recently, a modal pushover analysis (MPA) procedure was developed based on structural dynamics theory that includes the contribution of several modes of vibration (Chopra and Goel, 2001, 2002). This procedure was systematically evaluated (Goel and Chopra, 2002) using six buildings, each analyzed for 20 ground motions. The selected buildings represented two building heights – 9-story and 20-story – and three different seismic regions of the United States – Boston, Seattle, and Los Angeles. The median value of story drifts obtained from the MPA procedure and nonlinear response history analysis (RHA) were compared. It was found that with sufficient number of "modes" included, the height-wise distribution of story drifts estimated by MPA is generally similar to trends noted from nonlinear RHA. Furthermore, the additional error (or bias) in the MPA procedure applied to inelastic structures is small to modest compared to the bias in response spectrum analysis (RSA) applied to elastic structures – the standard analytical tool for the structural engineering profession – unless the building is deformed far into the inelastic region with significant stiffness and strength deterioration.

It is clear from the above review of literature that previous work on development and evaluation of the NSP and improved procedures are based on response of analytical models subjected to recorded and/or simulated earthquake ground motions. Recorded motions of buildings, especially those deformed into the inelastic range, provide a unique opportunity to evaluate such procedures. Therefore, the principal objective of this investigation is to evaluate the FEMA-356 NSP and the MPA procedures using recorded motions of buildings that were deformed beyond the yield limit.

Selected Buildings and Recorded Motions

Recorded motions of buildings that were deformed beyond the yield limit (or damaged) during the earthquake are required for this investigation. For this purpose, four buildings have been identified (Table 1) for which the motions were recorded during the 1994 Northridge earthquake. Of these four buildings, three haven been extensively instrumented by California Strong Motion Instrumentation Program (CSMIP) and one has been nominally instrumented in accordance to the code requirements. The responses of first three of these fours buildings – Van Nuys 7-Story Hotel, Woodland Hills 13-Story, and Sherman Oaks 13-Story – are presented in this paper; the work is in progress on the last building. Following is a brief description of each of these three buildings.

Table 1. Selected buildings, and peak ground and structure accelerations recorded during the 1994 Northridge earthquake.

Buildings name	CSMIP Station	Number of Stories	Peak accelerations (g)	
			Ground	Structure
Van Nuys 7-Story Hotel	24386	7	0.47	0.59
Woodland Hills 13-Story	C246	12/2	0.44	0.33
Sherman Oaks 13-Story	24322	13/2	0.46	0.65
Los Angeles 19-Story	24643	19/4	0.32	0.65

Van Nuys 7-Story Hotel

This 7-story reinforced concrete building (Fig. 1a) was designed in 1965 and constructed in 1966. The vertical load carrying system consists of 8 to 10-inch (20.3 to 25 cm) concrete slabs supported by concrete columns and spandrel beams at the perimeter (Naeim, 1997, 2000). The lateral load resisting system consists of interior column-slab frames and exterior column-spandrel beam frames.

This building is instrumented to measure horizontal accelerations at the base, 2nd floor, 3rd floor, 6th floor, and the roof (Figure 1b). Motions of this building have been recorded during several earthquakes in the past. The motions that are of interest are the ones recorded during the 1994 Northridge earthquake. The peak horizontal accelerations were 0.47 at the base and 0.57g in the structure. This building was heavily damaged during the 1994 Northridge earthquake and subsequently closed for repair and retrofit. Several columns between the fourth and fifth floors failed in shear at the top just below the spandrel beam. Most damage was observed in the longitudinal perimeter frames, with south perimeter suffering more damage than the north perimeter. This building has been extensively analyzed in the past (Naeim, 1997; Islam et al., 1998; Li and Jirsa, 1998; Goel et al., 2000; Naeim, 2000).

Woodland Hills 13-Story Building

The 13-story welded special moment frame building was constructed in 1975. Its lateral load resisting system consists of four identical steel frames along the building perimeter. The typical floor is square with 160-ft (48.8 m) sides (Fig. 2). At the first floor above ground, the plan broadens on three sides to form a plaza level while the fourth side abuts a landscape berm. These conditions provide a high degree of lateral restraint at this level. Basement perimeter walls are reinforced concrete and the foundation system consists of piles, pilecaps, and grade beams.

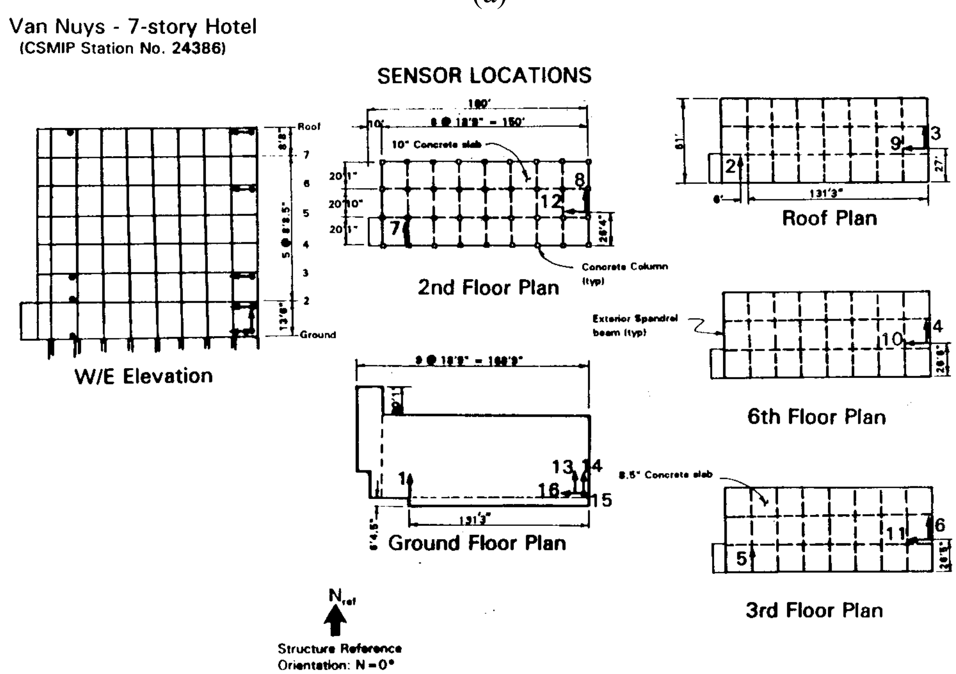
Denoted as Code-Instrumented building, this structure is nominally instrumented as required by the local building code. Motions were recorded during the 1994 Northridge earthquake at three levels: basement, 6th floor, and roof (Darragh et al., 1994). The peak horizontal accelerations were 0.44g at the base and 0.33g in the structure. This building was damaged during the 1994 Northridge earthquake. The damage consisted of local fracture at the beam-to-column welded joints (Uang et al., 1997).

Sherman Oaks 13-Story Commercial Building

This office building has 13 stories above and two floors below the ground. Designed in 1964, its vertical load carrying system consists of 2.4 inch (6 cm) thick one-way slabs supported by concrete beams, girders, and columns. The lateral load system consists of moment resisting concrete frames in the upper stories and concrete shear walls in the basements. The foundation system consists of concrete piles.

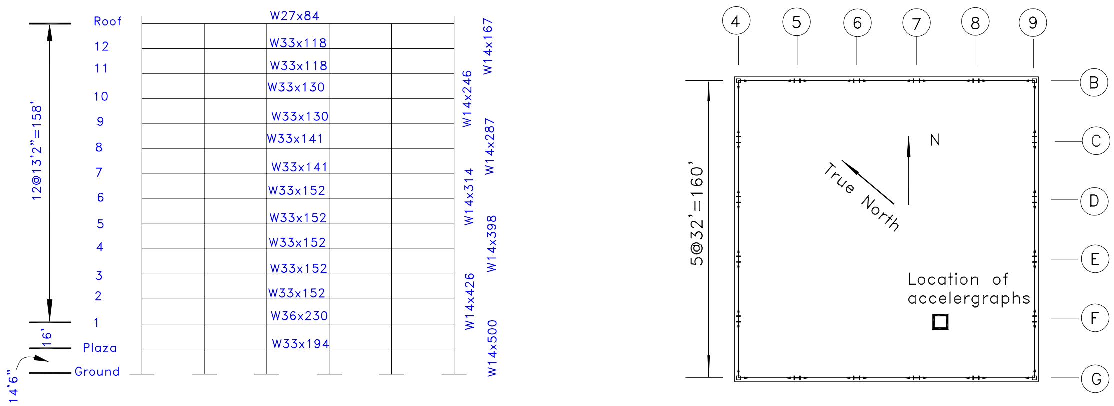
This building is instrumented to measure horizontal accelerations at the 2nd sub-basement level, ground level, 2nd floor, 8th floor, and roof level. The peak horizontal accelerations recorded during the 1994 Northridge earthquake were 0.46g at the basement and 0.65g in the structure. The building is reported to suffer cracks at many beam-column joints (Shakal et al., 1994).

(a)



(b)

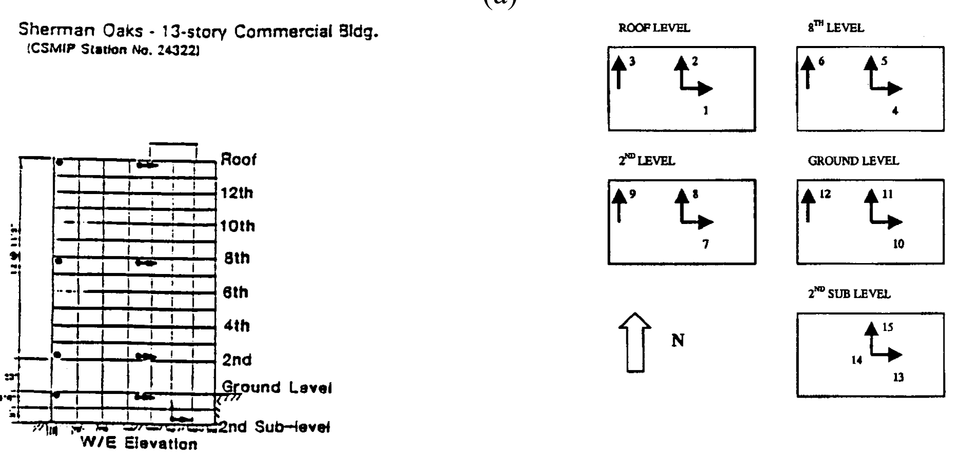
Figure 1. (a) Photograph (Naeim, 1997) and (b) sensor location for 7-Story Hotel buildings in Van Nuys.



(a) (b)

Figure 2. (a) Schematic elevation, and (b) plan of the 13-Story building in Woodland Hills (Uang et al., 1997).

(a)



(b)

Figure 3. (a) Photograph (Naeim, 1997), and (b) sensor location for 13-story building in Sherman Oaks.

Analysis Methods

Inter-Story Drifts Derived from Recorded Motions

Since buildings are typically instrumented at a limited number of floors, the motions of non-instrumented floors must be inferred from the instrumented floors for calculations of inter-story drifts in all stories. For this purpose, cubic spline interpolation procedure developed earlier by others (Naeim, 1997; De la Llera and Chopra, 1998) is used. The cubic spline interpolation procedure is preferred over the parametric model procedure because it automatically accounts for nonlinearities and time variance of the building parameters. This procedure has been tested (De la Llera and Chopra, 1998) and found to he highly accurate in estimating the motions of non-instrumented floors.

The cubic spline interpolation is performed on the building deformation (relative to the base) instead of the floor accelerations as traditionally done. This is because splines satisfy conditions of continuity and differentiability of second order at the interpolation points (i.e., instrumented floors in this case) and hence provide smooth shapes, as it should be, for the displacement field of the building.

Once the time variation of deformations of all floors have been developed using the cubic spline interpolation procedure, inter-story drifts at each time instant is computed from

$$\delta_j(t) = u_j(t) - u_{j-1}(t) \tag{1}$$

in which $\delta_j(t)$ is the inter-story drift in the jth story, and $u_j(t)$ and $u_{j-1}(t)$ are the deformations at the jth and j-1th floor levels at time *t*. Once the time histories of the inter-story drifts have been developed, peak values in the jth story, δ_{jo}, is be computed as the absolute maximum value over time. These values, denoted as "derived" inter-story drifts, would be used to evaluate the FEMA-356 NSP and MPA procedures.

Modal Decomposition of Recorded Motions

The contributions of various natural modes of vibration of the building to the total displacement can be extracted from the recorded (or interpolated) motions by using the standard modal analysis method (Chopra, 2001); the procedure would lead to exact modal contributions for buildings that remain elastic but approximate for inelastic buildings. This procedure has been used in our previous research (Chopra and Goel, 2001, 2002) to investigate the contributions of higher modes in inelastic buildings.

The contribution of the nth mode to total deformation at floor level *j* and time instant *t* is given by:

$$u_{jn}(t) = \frac{\boldsymbol{\phi}_n^T \boldsymbol{m} \boldsymbol{u}(t)}{\boldsymbol{\phi}_n^T \boldsymbol{m} \boldsymbol{\phi}_n} \phi_{jn} \tag{2}$$

in which ϕ_n is the nth mode shape of the elastic building, $\boldsymbol{m}$ is the mass matrix, $\boldsymbol{u}$(t) is the vector of displacements at all floor levels at time *t*, and ϕ_{jn} is the nth mode shape component at the jth floor level. Once the contribution of the nth mode to the floor displacements have been computed, its contribution to inter-story drift, $\delta_{jn}(t)$, can be computed using Eq. (1).

FEMA-356 NSP

The nonlinear static procedure (NSP) specified in the FEMA-356 (FEMA, 2000) document may be used for any structure and any rehabilitation objective except for structures with significant higher mode effects. To determine if higher mode effects are present, two linear response spectrum analyses must be performed: (1) using sufficient modes to capture 90% of the total mass, and (2) using only the fundamental mode. If shear in any story from the first analysis exceeds 130% of the corresponding shear from the second analysis, the higher mode effects are deemed significant. In case the higher mode effects are present, the NSP analysis needs to be supplemented by the Linear Dynamic Procedure (LDP); acceptance criteria for the LDP are relaxed but remain unchanged for the NSP.

The FEMA-356 NSP requires development of a pushover curve, which is defined as the relationship between the base shear and lateral displacement of a control node, ranging between zero and 150% of the target displacement. The control node is located at the center of mass at the roof of a building. For buildings with a penthouse, the floor of the penthouse (not its roof) is regarded as the level of the control node. Gravity loads are applied prior to the lateral load analysis required to develop the pushover curve.

The pushover curve is developed for at least two vertical distributions of lateral loads. The first pattern is selected from one of the following: (1) Equivalent lateral force (ELF) distribution: $s_j^* = m_j h_j^k$ (the floor number $j = 1,2,\ldots$N) where s_j^* is the lateral force and m_j the mass at *j*th floor, h_j is the height of the *j*th floor above the base, and the exponent $k = 1$ for fundament period $T_1 \le 0.5$ sec, $k = 2$ for $T_1 \ge 2.5$ sec; and varies linearly in between; (2) Fundamental mode distribution: $s_j^* = m_j \phi_{j1}$ where ϕ_{j1} is the fundamental mode shape component at the *j*th floor; and (3) SRSS distribution: $\mathbf{s}^*$ is defined by the lateral forces back-calculated from the story shears determined by linear response spectrum analysis of the structure including sufficient number of modes to capture 90% of the total mass. The second pattern is selected from either "Uniform" distribution: $s_j^* = m_j$ in which m_j is the mass and s_j^* is the lateral force at *j*th floor; or Adapted distribution that changes as the structure is displaced. This distribution should be modified from the original distribution by considering properties of the yielded structure.

The target displacement is computed from

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{2\pi^2} g \tag{3}$$

where T_e = Effective fundamental period of the building in the direction under consideration, S_a = Response spectrum acceleration at the effective fundamental vibration period and damping ratio of the building under consideration and g is the acceleration due to gravity, C_0 = Modification factor that relates the elastic response of an SDF system to the elastic displacement of the MDF building at the control node, C_1 = Modification factor that relates the maximum inelastic and elastic displacement of the SDF system, C_2 = Modification factor to represent the effects of pinched hysteretic shape, stiffness degradation, and strength deterioration, and C_3 = Modification factor to represent increased displacement due to P-delta effects.

The deformation/force demands in each structural element is computed at the target displacement and compared against acceptability criteria set forth in the FEMA-356 document. These criteria depend on the material (e.g., concrete, steel), type of member (e.g., beam, column, panel zones, connections etc.), importance of the member (e.g., primary, secondary) and the structural performance levels (e.g., immediate occupancy, life safety, collapse prevention).

The FEMA-356 NSP procedure contains several approximations. These include those in estimating the target displacement from Eq. 3, and using the pushover curve to estimate the member demands imposed by the earthquake. In this investigation, the focus is primarily on the second source of approximation; the first approximation is a focus of numerous other investigations. For this purpose, the following analysis method is employed.

The target displacement is selected to be equal to that of the roof level recorded during the earthquake, as opposed to calculating it according to the FEMA-356 document (Eq. 3). The structure is pushed to this target displacement using the FEMA-356 lateral load patterns and inter-story drifts are computed. These computed inter-story drifts are then compared with the "derived" inter-story drifts, i.e., those computed directly from the recorded motions using the procedure described in the preceding section. Such an analysis enables evaluation of the adequacy of various lateral load patterns in the FEMA-356 NSP, in particular, if the FEMA-356 NSP is able to capture the higher mode effects, which are likely to be present in the selected buildings.

MPA Procedure

Recently a MPA procedure has been developed to account for the higher mode effects and analytically tested for SAC buildings and ground motions (Chopra and Goel, 2001, 2002). This procedure has been found to be highly accurate unless the building is deformed far into the region of stiffness and strength deterioration (Goel and Chopra, 2002). Following is a brief summary of this procedure.

1. Compute the natural frequencies, ω_n and modes, ϕ_n, for linearly elastic vibration of the building.
2. For the nth-mode, develop the base shear-roof displacement, $V_{bn} - u_{rn}$, pushover curve for force distribution, $s_n^* = m\phi_n$, where m is the mass matrix of the structure. Gravity loads, including those present on the interior (gravity) frames, are applied before the first-"mode"

pushover analysis. The resulting P-delta effects may lead to negative post-yielding stiffness in the pushover curve. The gravity loads are not included in the higher-"mode" pushover curves.

3. Idealize the pushover curve as a bilinear curve. If the pushover curve exhibits negative post-yielding stiffness, the second stiffness (or post-yield stiffness) of the bilinear curve would be negative.
4. Convert the idealized $V_{bn} - u_{rn}$ pushover curve to the force-displacement, $F_{sn} / L_n - D_n$, relation for the nth -"mode" inelastic SDF system by utilizing $F_{sny} / L_n = V_{bny} / M_n^*$ and $D_{ny} = u_{rny} / \Gamma_n \phi_{rn}$ in which M_n^* is the effective modal mass, ϕ_{rn} is the value of $\boldsymbol{\phi}_n$ at the roof, and $\Gamma_n = \boldsymbol{\phi}_n^T \mathbf{m} \mathbf{1} / \boldsymbol{\phi}_n^T \mathbf{m} \boldsymbol{\phi}_n$.
5. Compute the peak deformation D_n of the nth-"mode" inelastic single-degree-of-freedom (SDF) system defined by the force-deformation relation developed in Step 4 and damping ratio ζ_n. The elastic vibration period of the system is $T_n = 2\pi \left(L_n D_{ny} / F_{sny} \right)^{1/2}$. For an SDF system with known T_n, ζ_n and a selected earthquake excitation, D_n can be computed by either by nonlinear RHA of the SDF system or from inelastic design spectrum.
6. Calculate peak roof displacement u_{rn} associated with the nth-"mode" inelastic SDF system from $u_{rn} = \Gamma_n \phi_{rn} D_n$.
7. From the pushover database (Step 2), extract values of desired responses r_n: floor displacements, story drifts, plastic hinge rotations, etc.
8. Repeat Steps 3-7 for as many modes as required for sufficient accuracy. Typically, the first two or three "modes" will suffice.
9. Determine the total response (demand) by combining the peak "modal" responses using the SRSS rule: $r \approx \left(\sum_n r_n^2 \right)^{1/2}$.

Steps 3 to 6 of the MPA procedure described above are used to compute the peak roof displacement associated with the nth-"mode" inelastic SDF system. However, these steps are not necessary for analysis of a building for which recorded motions are available. The contribution of the nth-"mode" to the total roof displacement, u_{rn}, can be computed from modal decomposition of recorded motion using Eq. (2).

In the MPA procedure, total floor displacements and story drifts can be computed within sufficient degree of accuracy by combining the values obtained from "modal" pushover analysis (Step 9). However, this procedure may not lead to accurate estimates of localized demands such as plastic rotations and member forces. For this purpose, improved procedure are being developed.

Analytical Models

The computer program DRAIN-2DX (Prakash et al., 1993) was used for analysis of the selected buildings. For this purpose, analytical models were developed and calibrated as follows. First, the fundamental mode period from eigen analysis of the analytical model was compared with the "elastic" period obtained from system-identification analysis of the record segment during which the structure is expected to remain elastic. Such analysis involves plotting the ratio of the absolute values of Fast Fourier Transform of the displacement at the roof and base and identifying the peak corresponding to the fundamental mode. The system identification analysis is also performed using the entire record leading to "apparent" fundamental mode period. Value of the "apparent" period significantly longer than the "elastic" period is indicative of inelastic action in the building during the ground motions. The periods from eigen analysis, and the "elastic" and the "apparent" periods identified from the recorded motions are presented in Table 2.

Second, the time history of the displacement response is computed from the analytical model using the acceleration recorded at the base as the input motion. The computed motions are then compared with the recorded motions to verify that the response from the analytical model correlates reasonably with the recorded motions.

Table 2. Vibration periods of fundamental mode from eigen analysis and system identification.

Building	Period (sec)		
	Eigen	"Elastic"	"Apparent"
Van Nuys 7-Story Hotel	1.50	1.59	2.05
Woodland Hills 13-Story	3.05	N/A	3.90
Sherman Oaks 13-Story	2.47	2.28	2.93

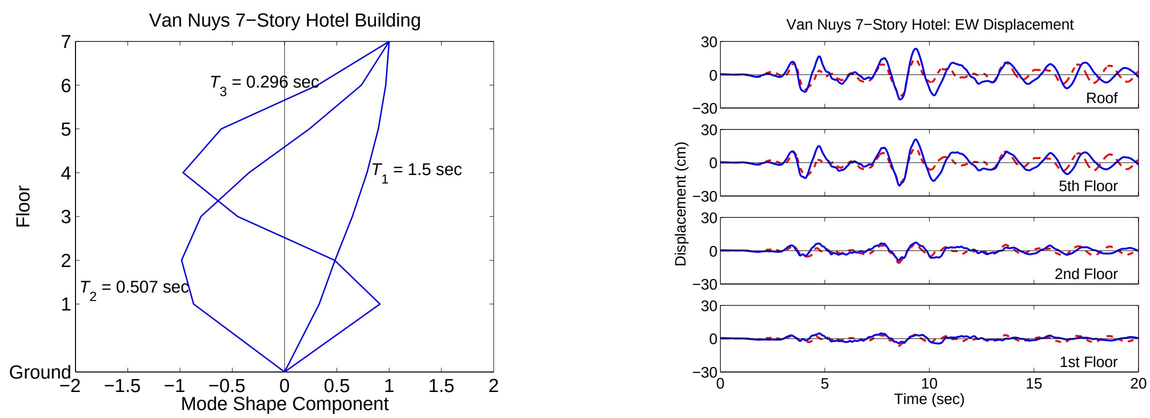
Van Nuys 7-Story Hotel Building

The DRAIN-2DX model used in earlier investigations (Browning et al., 2000; Goel et al., 2000) was modified to develop a model for the south frame of this building; this frame is of interest in this study because it sustained significant damage during the 1994 Northridge earthquake. The frame is modeled using beam-columns elements with center-line dimensions. Initial stiffness was equal to 0.5 and 0.7 times the gross cross-sectional stiffness for beams and columns, respectively. The beams were modeled without P-M interaction while P-M interaction relationship for reinforced-concrete sections was used for the columns. The moment yield strengths were computed using conventional procedures (Browning et al., 2000). The mass equal to one-third of the total building mass was assigned to this frame, and Rayleigh damping of 10% was used for the first and third mode of vibration.

Figure 4a shows the first three vibration modes and periods obtained from elastic eigen-value analysis of the model. The fundamental period of 1.5 sec (Fig. 4a) correlated reasonable well with the "elastic" period of 1.59 sec (Table 2) identified from recorded motions. The "apparent" period of 2.05 sec (Table 2) is much longer than the "elastic" period or the period from eigen analysis, indicating significant inelastic response during the ground motion; the damage reported to this building during the 1994 Northridge earthquake (Naeim, 1997; Islam et al., 1998; Li and Jirsa, 1998; Browning et al., 2000) clearly supports this observation.

The displacement response history of the analytical model was calculated using the east-west component of the motion recorded at the base during the 1994 Northridge earthquake. The comparison of displacements from the response history analysis with the recorded motions in the east-west direction at the center of the building, shown in Fig. 4b, indicates a reasonable match between the two. This implies that the simple model used in this study is adequate in representing the recorded motions. It may be possible to further improve the accuracy of the model by using more appropriate force-deformation relationships (Li and Jirsa, 1998; Browning et al., 2000).

It must be noted that the model used in this investigation, as well as those used by others (Li and Jirsa, 1998; Browning et al., 2000), are two-dimensional in nature. There is strong evidence from recorded motions that this building exhibited significant torsional motions during the 1994 Northridge and other earthquakes. Therefore, only a three-dimensional model would be able to capture the true behavior of this building.



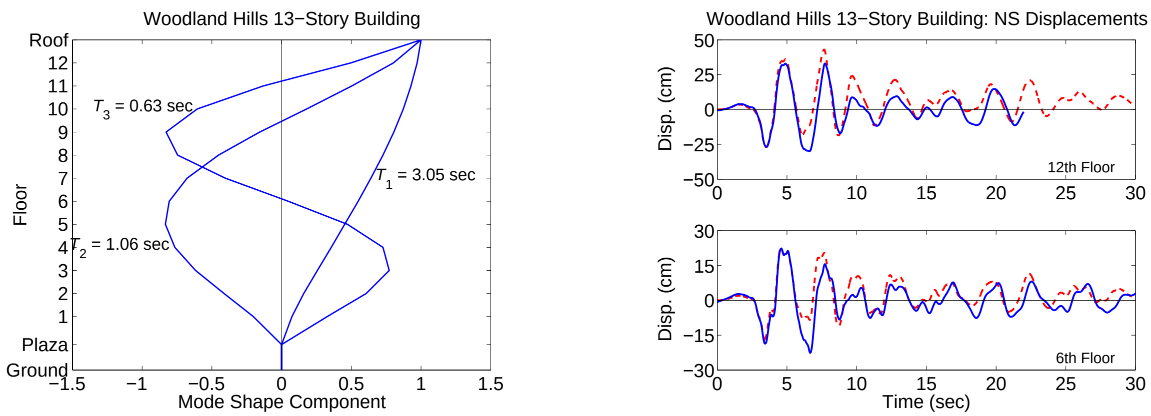
(a) (b)

Figure 4. (a) Elastic modes and periods of analytical model; (b) Comparison of displacements computed from analytical model (dashed line) with recorded displacement (solid line) of Van Nuys 7-Story Hotel building.

Woodland Hills 13-Story Building

The DRAIN-2DX model developed earlier (Uang et al., 1997) was adopted for analysis of this building. The moment frame in the north-south direction is modeled because it experienced significant damage, in the form of connection failures, during the 1994 Northridge earthquake (Uang et al., 1997). The two-dimensional model consisted of beams and columns modeled by DRAIN-2DX Element 2, 100% rigid-end offsets, 2% strain hardening for the beams, steel section P-M interaction curve for columns, panel zones modeled as semi-rigid with DRAIN-2DX Element, and Rayleigh damping of 5% for the first and third modes. The expected yield stress for steel members equal to 47.3 ksi is used, which is about 30% higher than the nominal value of 36 ksi. Further details of the model are available elsewhere (Uang et al., 1997). The two-dimensional model for this building is reasonable as the building plan is quite symmetric.

The displacement response of above described model computed to the north-south component of the motions recorded at the base matched reasonably well with the recorded motions in this direction (Uang et al., 1997). However, when this model is pushed during the pushover analysis (presented later in this paper) to the peak roof displacement recorded during the 1994 Northridge earthquake, none of its elements yield. This behavior of the model is contrary to the physical observation during the post-earthquake inspection, which revealed numerous beam-column connection failures. Therefore, the model was modified by reducing the strengths of beams and panel zone elements by 25% compared to the original model. This brings the expected yield stress close to the nominal yield stress of 36 ksi. Furthermore, the Rayleigh damping was increased from 5% to 7% in the first and third modes.



(a) (b)

Figure 5. (a) Elastic modes and periods of analytical model; (b) Comparison of displacements computed from analytical model (dashed line) with recorded displacement (solid line) of Woodland Hills 13-Story building.

The fundamental period of this building from the eigen analysis is 3.05 sec (Fig. 5a). The system identification could not identify the true "elastic" period for this building because long-enough initial time segment of the recorded motions during which the building behaved elastically could not be selected. The "apparent" period of 3.9 sec (Table 2) is much longer than the period from eigen analysis, indicating inelastic response during the ground motion; the damage reported to this building during the 1994 Northridge earthquake (Uang et al., 1997) clearly supports this observation.

The displacement response history of the analytical model was calculated using the north-south component of the motion recorded at the base during the 1994 Northridge earthquake. The comparison of displacements from the response history analysis with the recorded motions in the north-south direction at the center of the building, shown in Fig. 5b, indicates a reasonable match between the two. This implies that the simple model used in this study is adequate in representing the recorded motions. It may be possible to further improve the accuracy of the model by using more appropriate connection behavior.

Sherman Oaks 13-Story Commercial Building

The DRAIN-2DX model was developed for the exterior frame in the east-west direction for this building. The model was developed based on the structural plans and additional information available in an earlier study (John A. Martin & Associates, 1973). The frame is modeled using beam-columns elements with center-line dimensions. Initial stiffness was equal to 0.5 and 0.7 times the gross cross-sectional stiffness for beams and columns respectively. Rigid end offsets equal to 50% of the joint dimensions were assumed. The beams were modeled without P-M interaction while P-M interaction relationship for reinforced-concrete sections was used for the columns. The moment yield strengths were computed using moment-curvature analysis. The nominal strength of beams were increased by 25% for a better match with the recorded motions. The mass equal to one-third of the total building mass was assigned to this frame, and Rayleigh damping of 10% was assigned to the first and third mode of vibration.

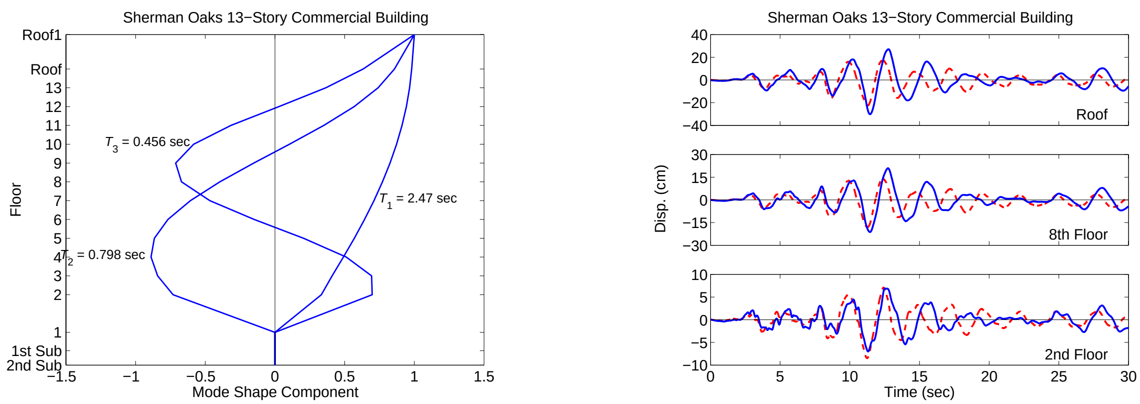


Figure 6. (a) Elastic modes and periods of analytical model; (b) Comparison of displacements computed from analytical model (dashed line) with recorded displacement (solid line) of Sherman Oaks 13-Story Commercial building.

Figure 6a shows the first three vibration modes and periods obtained from elastic eigen-value analysis of the model. The fundamental period of 2.47 sec (Fig. 6a) is slightly longer than the "elastic" period of 2.28 sec (Table 2) identified from recorded motions. The "apparent" period of 2.93 sec (Table 2) is much longer than the "elastic" period or the period from eigen analysis, indicating some inelastic response during the ground motion; the post earthquake investigation indicates minor cracking at the beam columns joints after the 1994 Northridge earthquake (Naeim, 1997).

The displacement response history of the analytical model was calculated using the east-west component of the motion recorded at the base during the 1994 Northridge earthquake. The comparison of displacements from the response history analysis with the recorded motions in the east-west direction at the center of the building, shown in Fig. 6b, indicates a reasonable match between the two. This implies that the simple model used in this study is reasonable in representing the recorded motions. As mentioned previously for the Van Nuys building, it may be possible to further improve the accuracy of the model by using more appropriate force-deformation relationships.

Evaluation of Nonlinear Static Procedures

The FEMA-356 NSP and MPA procedures are evaluated in this section using recorded motions of selected buildings. Presented for each selected buildings are the pushover curves for the four FEMA-356 distributions for the FEMA NSP analysis and the first three "modes" for the

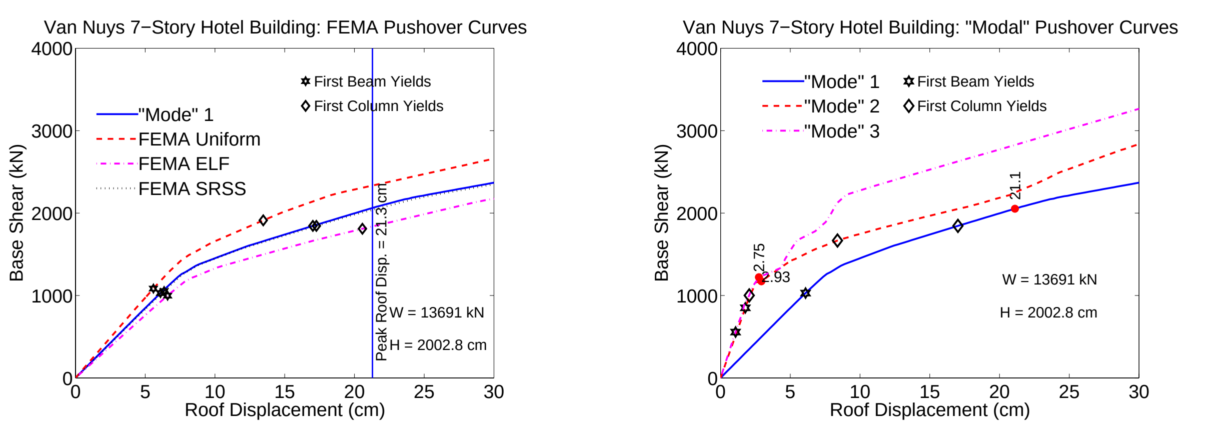
MPA analysis. Shown on each pushover curve are the peak roof displacement – total value for the FEMA-356 curves and the modal component for the "modal" pushover curves – during the 1994 Northridge earthquake; and locations of first yielding of beam, columns, or connection. Subsequently, story drifts from the four FEMA analyses and MPA procedure are compared with the "derived" values from the recorded motions. It is useful to emphasis again that two-dimensional models have been used in this investigation. Therefore, the motions of the frame were taken as equal to those recorded at the center for the selected buildings.

Van Nuys 7-Story Hotel Building

The pushover curves for the longitudinal frame on the south face of the Van Nuys 7-Story Hotel building are presented in Fig. 7. These results lead to the following observations. The characteristic – elastic stiffness, and yield strength and displacement – of the pushover curve depend on the lateral force distribution (Fig. 7a). The "Uniform" distribution generally leads to pushover curve with higher elastic stiffness, higher yield strength, and lower yield displacement compared to all other distributions. The ELF distribution, on the other hand, leads to pushover curve with lower elastic stiffness, lower yield strength, and higher yield displacement. The "Mode" 1 and SRSS distribution give pushover curves that are essentially identical and are bounded by the pushover curves due to "Uniform" and ELF distributions.

The first beam yields at much lower force level compared to the first column (Fig. 7a). This building was deformed well into the inelastic range during the 1994 Northridge earthquake, as apparent from the peak roof displacement being much larger than the yield displacement. This is consistent with the post-earthquake observations that indicated cracking in several beams and fracture in columns just below the 5th floor (Li and Jirsa, 1998).

The "modal" pushover curves (Fig. 7b) indicate the significant yielding in the first "mode". The building is deformed nearly to the elastic limit of the pushover curve in the second and third modes. However, yielding has been initiated in some beams and columns, indicating that modes higher than the fundamental mode also contributed to the inelastic behavior of this building.



(a) (b)

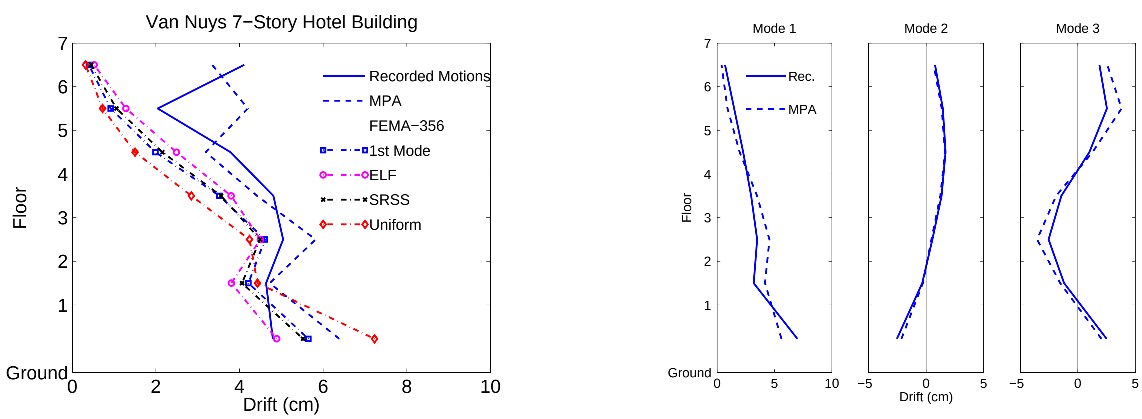
Figure 7. Pushover curves for Van Nuys 7-Story Hotel for (a) four FEMA-356 distributions; and (b) modal distributions corresponding to first three modes in the MPA procedure.

The results presented for story drift (Fig. 8a) indicate that among the four FEMA-356 distributions, the “Uniform” distribution always leads to the largest drifts in the lower stories and smallest drifts in upper stories. Comparing the drift demands from the FEMA-356 distributions with those from recorded motions demonstrates the serious limitations of the FEMA-356 NSP: the FEMA-356 force distributions lead to gross underestimation of story drifts in the upper stories and gross overestimation in the lower stories (Fig. 8a).

Among the four FEMA-356 distributions, the “Uniform” force distribution leads to the worst estimates of story drifts. For example, this distribution leads to underestimation of the drift at 7th story more a factor of more than 13: the story drifts from recorded motions and FEMA-356 “Uniform” distributions are 4.11 cm and 0.32 cm, respectively. On the other hand, the drift in the first story is overestimated by a factor of about 1.5: the story drifts from recorded motions and FEMA-356 “Uniform” distributions are 4.80 cm and 7.23 cm, respectively. Therefore, this distribution seems unnecessary in the FEMA-356 NSP, an observation which is consistent with that based on an earlier analytical study (Goel and Chopra, 2002).

The presented results for story drifts also demonstrate another serious limitation of the FEMA-356 NSP. The higher mode effects for this building were deemed not to be significant based on the FEMA-356 criterion. Therefore, it may be expected that the FEMA-356 would lead to reasonable estimates of drifts in upper stories. Yet the drifts are significantly underestimated (Fig. 8a). It is well known that the larger drifts in upper stories tend to occur due to higher modes. Therefore, the FEMA-356 criterion for significant higher mode effects should be re-examined.

The MPA procedure for this building provides much better estimates of story drifts throughout the building height. In particular, the match between the story drifts from MPA and recorded motions is excellent in upper stories indicating that the MPA procedure is able to capture the higher mode effects for this building.



(a) (b)

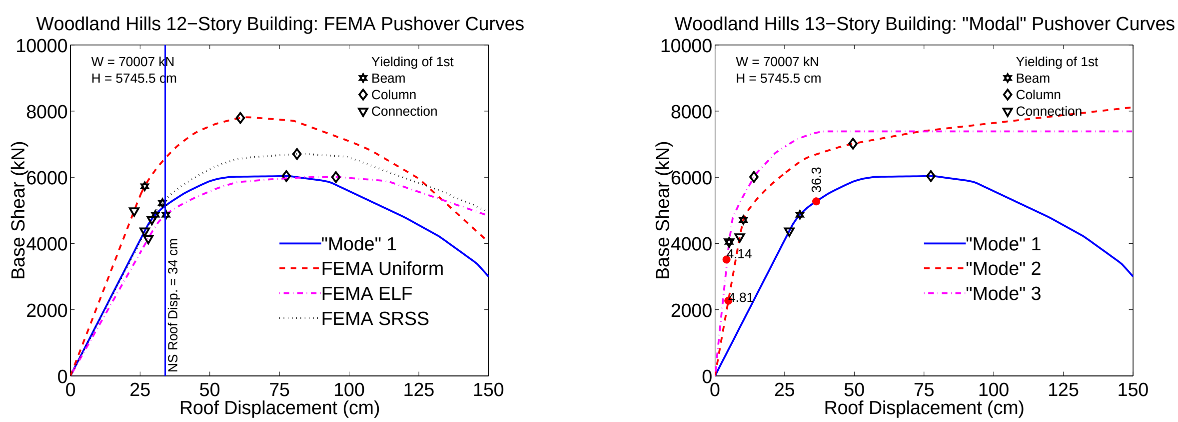
Figure 8. Comparison of (a) displacements and (b) story drifts from recorded motions, MPA procedure, and four FEMA-356 NSP analyses for Van Nuys 7-Story Hotel.

While the estimates of story drifts from the MPA procedure are much better compared to the FEMA NSP, minor differences exist, such as drift in the sixth story (Fig. 8a). In order to understand the source of this discrepancy, peak displacement and drifts in each mode of the

MPA procedure are compared with those obtained from modal decomposition of recorded motions (Fig. 8b). This comparison shows that the match between the two is reasonably good. Therefore, the prime source of discrepancy appears to be from modal combination procedure. The modal combination rule was found to be deficient in an earlier study (Goel and Chopra, 2002) even for elastic buildings. Furthermore, the SRSS combination rule is likely to be inaccurate for individual ground motion as it was developed to work well with smooth design spectrum. The research is currently underway by others to develop improved modal combination rules. With improved modal combination rules, the accuracy of the MPA procedure may be expected to further improve.

Woodland Hills 13-Story Building

The pushover curves for the longitudinal frame on in the north-south direction of the Woodland Hills 13-Story building are presented in Fig. 9. These results lead to the following observations. The characteristic – elastic stiffness, yield strength and displacement, and post-yield strength decay – of the pushover curve depend on the lateral force distribution (Fig. 9a). The “Uniform” distribution generally leads to pushover curve with higher elastic stiffness, higher yield strength, lower yield displacement, and more rapid decay in post-yield strength compared to all other distributions. The ELF distribution, on the other hand, leads to pushover curve with lower elastic stiffness, lower yield strength, and higher yield displacement. The “Mode” 1 and SRSS distribution give pushover curves that are essentially identical up to the elastic limit. Thereafter, the strength is higher for the SRSS distribution compared to the “Mode” 1 distribution. The first yielding occurs in the connection followed soon after by the first yielding of the beam (Fig. 9a). The columns yielding occurs at much higher deformation level and is soon followed by rapid decay in the strength. This building is deformed only slightly beyond the elastic limit during the 1994 Northridge earthquake. The “modal” pushover curves (Fig. 9b) also indicate the slight yielding in the first “mode”. The building remains elastic in all higher modes.

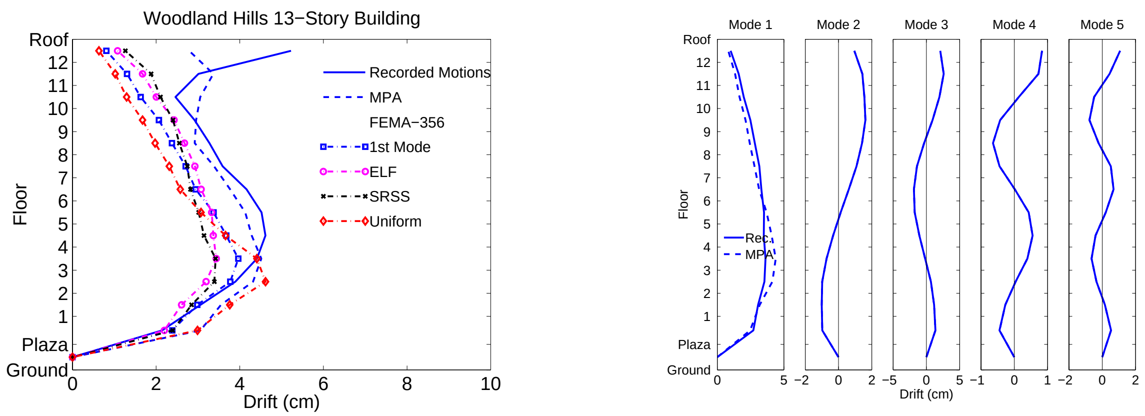


(a) (b)

Figure 9. Pushover curves of Woodland Hills 13-Story building for: (a) four FEMA-356 distributions; and (b) modal distributions corresponding to first three modes in the MPA procedure.

The results presented for story drift (Fig. 10a) indicate that the FEMA-356 distributions provide reasonable estimates at lower stories. However, the FEMA-356 force distributions lead to gross underestimation of story drifts in the upper stories (Fig. 10a). As noted earlier, among the four FEMA-356 distributions, the "Uniform" force distribution leads to the worst estimates of story drifts. For example, this distribution leads to underestimation of the drift at 12th story more a factor of about 3: the story drifts from recorded motions and FEMA-356 "Uniform" distributions are 3.01 cm and 1.02 cm, respectively.

It must be noted that higher mode effects are likely to be significant for this building. Therefore, the higher drifts noted in upper stories from the recorded motions are due to higher modes. Clearly, FEMA-356 distributions are unable to adequately represent the drifts in upper stories due to higher modes.



(a) (b)

Figure 10. Comparison of (a) displacements and (b) story drifts from recorded motions, MPA procedure, and four FEMA-356 NSP analyses for Woodland Hills 13-Story building.

The MPA procedure for this building in general provides excellent estimates of the story drifts (Fig. 10a), except for the 13th story, indicating that the MPA procedure is able to capture the effects of higher modes. The comparison of drifts from MPA and those from modal decomposition of recorded motions for each mode (Fig. 10b) shows an excellent match between the two. Therefore, the slight discrepancy between the results from MPA and recorded motions are due to the modal combination procedure, which are likely to be inaccurate for individual ground motions. With improved modal combination rules, the accuracy of the MPA procedure may be expected to further improve.

Sherman Oaks 13-Story Commercial Building

The pushover curves for the longitudinal frame on in the east-west direction of the Sherman Oaks 13-Story building are presented in Fig. 11. As noted previously, the characteristic – elastic stiffness, yield strength and displacement, and post-yield strength decay – of the pushover curve depend on the lateral force distribution (Fig. 11a). The "Uniform" distribution generally leads to pushover curve with higher elastic stiffness, higher yield strength, lower yield displacement, and more rapid decay in post-yield strength compared to all other distributions. The ELF distribution, on the other hand, leads to pushover curve with lower elastic stiffness, lower yield strength, and higher yield displacement. The "Mode" 1 and SRSS distribution give pushover curves that are essentially identical and bounded by the "Uniform" and ELF curves. The first yielding occurs in the beam followed soon after by the first yielding of the column (Fig. 11a). This building is deformed significantly beyond the elastic limit during the 1994 Northridge earthquake. The “modal” pushover curves (Fig. 11b) also indicate significant yielding in the first “mode”. The building remains elastic in all higher modes. However, the yield strength appears to be much lower in higher mores compared to the first mode.

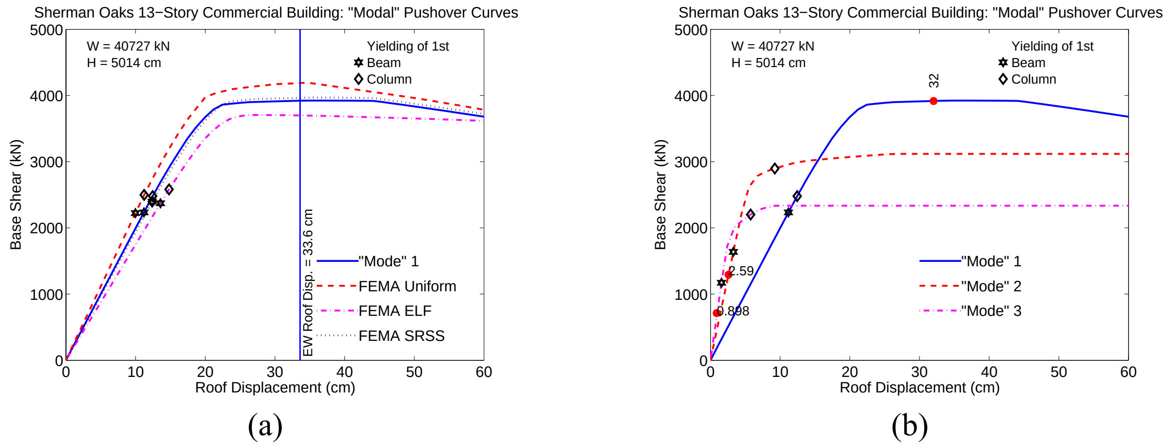
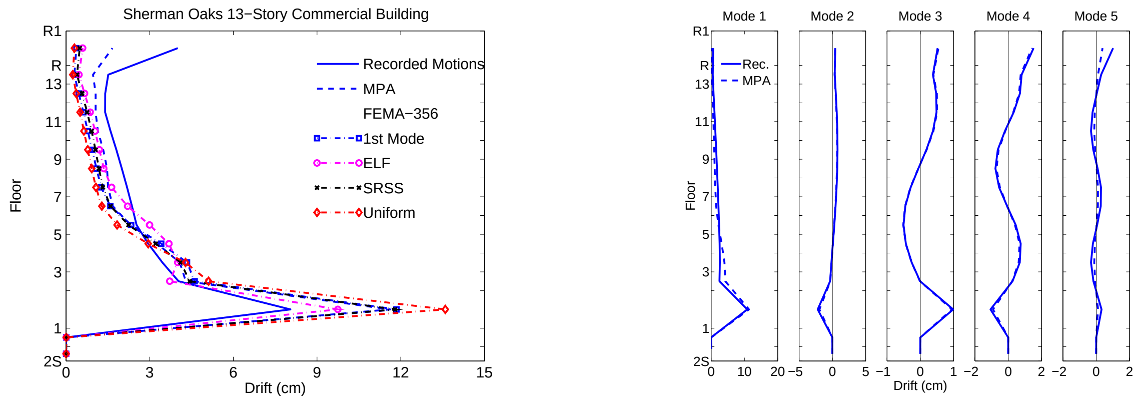


Figure 11. Pushover curves of Sherman Oaks 13-Story Commercial building for: (a) four FEMA-356 distributions; and (b) modal distributions corresponding to first three modes in the MPA procedure.

The results presented for story drift (Fig. 12a) indicate that among the four FEMA-356 distributions, the “Uniform” distribution always leads to the largest drifts in the lower stories and smallest drifts in upper stories. Comparing the drift demands from the FEMA-356 distributions with those from recorded motions shows that the FEMA-356 force distributions lead to gross underestimation of story drifts in the upper stories and gross overestimation in the lower stories (Fig. 12a). Among the four FEMA-356 distributions, the “Uniform” force distribution leads to the worst estimates of story drifts. For example, this distribution leads to underestimation of the drift at 13th story more a factor of more than 6: the story drifts from recorded motions and FEMA-356 “Uniform” distributions are 1.51 cm and 0.24 cm, respectively. On the other hand, the drift in the first story is overestimated by a factor of more than 1.5: the story drifts from recorded motions and FEMA-356 “Uniform” distributions are 8.05 cm and 13.60 cm, respectively. As noted for other buildings, this distribution seems unnecessary in the FEMA-356 NSP.

It must be noted that higher mode effects are significant for this building, as apparent from higher drifts noted in upper stories from the recorded motions (Fig. 12a). Comparison of drifts from recorded motions and the FEMA-356 distributions show that the FEMA-356 distributions are unable to adequately capture the effects of higher modes.

The MPA procedure for this building in general provides excellent estimates of the story drifts (Fig. 12a), except for the 1st story, indicating that the MPA procedure is clearly able to capture the effects of higher modes; the MPA overestimates the drifts in the first story. The comparison of drifts from MPA and those from modal decomposition of recorded motions for each mode (Fig. 12b) shows an excellent match between the two. Therefore, the slight discrepancy between the results from MPA and recorded motions are due to the modal combination procedure, which are likely to be inaccurate for individual ground motions. With improved modal combination rules, the accuracy of the MPA procedure may be expected to further improve.



(a) (b)

Figure 12. Comparison of (a) displacements and (b) story drifts from recorded motions, MPA procedure, and four FEMA-356 NSP analyses for Sherman Oaks 13-Story Commercial building.

Conclusions

This research investigation on evaluation of the FEMA-356 NSP and MPA procedure using recorded motions of buildings that were damaged during the 1994 Northridge earthquake has led to the following conclusions.

1. The FEMA-356 NSP leads to significant underestimation of drifts in upper stories of the selected buildings. The underestimation ranges by a factor of 13 for the Van Nuys building to 3 for the Sherman Oaks building.
2. The FEMA-356 NSP is unable to account for higher mode effects, which typical contribute significantly to the drifts in upper stories.
3. The FEMA-356 NSP leads to significant overestimation – by a factor of 1.5 – of drift in lower stories for Van Nuys and Sherman Oaks buildings.
4. Among the four FEMA-356 distributions, the “Uniform” force distribution leads to the most excessive underestimation in upper stories and overestimation in the lower stories. Therefore, this distribution seems unnecessary in the FEMA-356 NSP.
5. The FEMA-356 NSP is expected to provide reasonable estimate of the response if the higher mode effects are deemed not to be significant based on the FEMA-356 criterion. Although the FEMA-356 criterion is satisfied for the Van Nuys building, the drifts in upper stories are significantly underestimated indicating the need to re-examine the FEMA-356 criterion for evaluating significant higher mode effects.
6. The MPA procedure provides much better estimates of drifts compared to the FEMA-356 NSP, and is able to account for the higher mode effects.
7. The response for each mode in the MPA procedure matched closely with the modal response obtained from decomposition of the recorded motions, indicating the observed discrepancy between the response from MPA and recorded response is due to limitations in the

combination procedure. With development of the improved combination procedures, accuracy of the MPA procedure is likely to improve.

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