SEISMIC EVALUATION OF HIGH-RISE BUILDINGS BY MODIFIED DYNAMIC INELASTIC ANALYSIS METHOD

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SUMMARY

It has been common practice to design tall buildings in a way that the reserved strength available after the formation of plastic hinges is called on to resist earthquakes, avoiding the collapse of structures. Pushover analysis is fast becoming an accepted and simple method for the seismic evaluation of high-rise buildings. The popularity of this approximate, nonlinear static analysis method is due to its conceptual simplicity and ability to graphically describe a structure's capacity and demand. In this method, the lateral load is increased with the same profile to find the inelastic capacity of structure. Although the lateral load distribution is selected considering the elastic response, it is clear that as building enters into inelastic range, the elastic-based lateral load distribution may not be applicable anymore.

This paper describes Modified Dynamic Inelastic Analysis (MODIA) - the method which may help to account for the dynamic characteristics of structures using the mode shapes during the inelastic analysis. MODIA extends the basic pushover analysis to address dynamic characteristics yet maintains the simplicity of the basic pushover analysis. The steps to perform MODIA are described and the proposed method is applied to seven- and fifteen-story example buildings. At each loading step, the system ductility, base shear capacity, overturning moment, inter-story drift, and other design parameters can be calculated as a function of lateral deflection. The location and sequence of hinge formation are also evaluated.

MODIA uses the Spectrum Method(SM) to graphically compare the pushover curve to the earthquake demand. Once the pushover and response spectra curves are plotted on the same graph, the capacity of the structure can be easily compared to the earthquake demand. The intersections of the pushover curves with the appropriate response spectrum curve represent peak response. Since MODIA has been developed for design, we believe it can be useful in identifying failure mechanisms due to dynamic inelastic characteristics as well as ground motions that may cause these failure mechanisms.

INTRODUCTION

It has been common practice to design tall buildings in a way that the reserved strength available after the formation of plastic hinges is called on to resist earthquakes, avoiding the collapse of structures[8]. The pushover analysis is the commonly used for static inelastic analysis of structure[1]. In this method, the lateral load is increased with the same profile to find the inelastic capacity of structure. At each lateral load increment, the sequence of plastic hinge and the elastic-plastic behavior of each member are found without dynamic consideration of structure[3].

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Although the elastic-based lateral load distribution is selected as the initial load profile, it is clear that it may not be applicable anymore in inelastic range. Inelastic time history analysis is used for considering the dynamic inelastic effects of structure. But it is used only for the evaluation of structure for specific earthquakes and is impractical for engineers[4].

This paper describes Modified Dynamic Inelastic Analysis (abbreviated as MODIA) - the method which may help to account for the dynamic characteristics of structures using the mode shapes during the inelastic analysis. MODIA extends the basic pushover analysis to address dynamic characteristics yet maintains the simplicity of the basic pushover analysis. MODIA uses the Spectrum Method (SM) to graphically compare the capacity curve to the earthquake demand. Two example frames are analyzed for comparison. Since MODIA has been developed for design, we believe it can be useful in identifying collapse mechanism due to dynamic inelastic characteristics as well as ground motions that may cause these failure mechanisms.

SELECTION OF LATERAL LOAD DISTRIBUTION

In considering the different variations of the story accelerations, different modes of deformations and the influence of higher modes, a power distribution of lateral loads is used. The lateral load increment at floor "i" is calculated as following [11];

$$\Delta P_i = \frac{W_i h_i^k}{\sum_{j=1}^{N} W_j h_j^k} \Delta V_b \tag{1}$$

in which ΔV_b is base shear increment and W_i, h_i, ΔP_i are weight, story height, story shear at floor "i" respectively. k is the parameter that controls the shape of the lateral load distribution and a function of the fundamental period of structure[7].

These lateral load distributions increase constantly along the height of building. As the structure enters into inelastic ranges, initial lateral load distribution may not be applicable anymore[9]. The typical lateral load profile may not be consistent with the internal resistance of the structure especially if story mechanisms develop under the imposed load. The limitation of using such a lateral load distribution has been pointed out by Bertero et al.[10]. Recently Reinhorn et al.[11] proposed an adaptive pushover method that uses a self-correcting loading, but it is impractical for tall building, which requires a great amount of computation time.

Dynamic inelastic characteristics of structures mainly depend on the stiffness and the mass. When plastic hinges form as the results of the inelastic analysis, the adjusted force response of structures must be established since the natural period of structures with new plastic hinges is lengthened. The lateral load that structures must resist lies in the natural periods and associated mode shapes. At each moment of forming new plastic hinge mechanism, MODIA calculates the lateral load profile based on mode shapes. The newly determined distribution of lateral load is applied to obtain dynamic behavior of structures as well as new hinges. The base shear due to the initial horizontal load is kept constant during the inelastic analysis.

MODIFIED DYNAMIC INELASTIC ANALYSIS

Procedure:

The first step in the proposed method is to evaluate the residual strength factor. The residual strength factor λ_1 is defined as the ratio of ultimate strength to corresponding force in members.

$$\lambda_{k,i} = \frac{\pm\{F_r\}_{k,i}}{\{F_o\}_{k,i}} \tag{2}$$

where, k, i, F_r and F_o are the lateral loading step, the number of members, residual strength, and the corresponding force, respectively. The required additional load for the first hinge and the associated location are determined by finding $\lambda_{min,1}$, the minimum of λ_1 among all members as calculated in Equation (2).

As shown in Figure 1, the lateral load, deflection and the member force at the time of first hinge formation can be simply calculated as following;

lateral load : $\{P_p\}_1 = \lambda_{min,1}\{P\}$ (3)

member force : $\{F_m\}_1 = \lambda_{min,1}\{F_o\}_{1,i}$ (4)

lateral deflection : $\{d_p\}_1 = \lambda_{min,1}\{d\}_1$ (5)

Subscripts p, m are the node, the member and {P} is the initial lateral load imposed on the building while $\{d\}_1$, $\{F_o\}_1$ are the corresponding lateral deflection and the force, respectively. After first hinge occurs, stiffness of members with plastic hinge should be modified so that this member cannot resist the moment bigger than plastic moment. The residual strength factor of members without plastic hinges, is then calculated as in Equation (6).

$$\{F_r\}_{2,i} = \{F_u\}_i - \lambda_{min,1}\{F_o\}_{1,i} \tag{6}$$

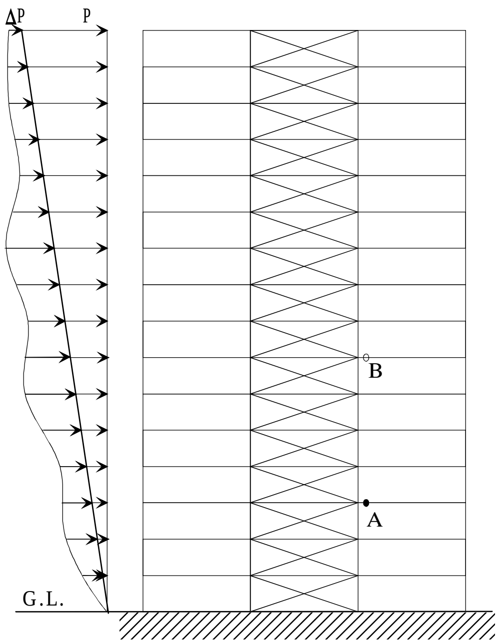


Figure 1: Variation of lateral load profile

For the second hinge, the load increment is found by considering the mode shapes and the initial lateral load {P}. Since the inelastic response of the structure changes the stiffness matrix, thus the mode shapes, lateral distribution must be derived to be proportional to the changing mode shapes to capture the inelastic deformation. Mode shapes are combined using SRSS method and the load increment at story "i" is calculated according to

$$\Delta P_i = \lambda_{min,1}\sqrt{\sum_{j=1}^{N} m(\phi_{i,j}(u)\Gamma_j(u))^2}\,\{P\}_i - \{P\}_i^{old} \tag{7}$$

where $\phi_{i,j}$ (u) : the value of the mode shape "j" at story "i"

T_j (u) : the modal participation factor for same mode "j"

P_i^{old} : the force at story "i" in the previous loading step

As the lateral load increment is continued, the residual strength of member is decreased. Eventually the plastic hinge forms in the member of zero residual strength. The same procedure is iterated until the formation of collapse mechanism and the corresponding base shear is the ultimate capacity of structures. If the collapse mechanism of the structure is obtained at the *n*th step, the residual strength factors are found in Equation (8).

$$\lambda_n = \frac{\pm\{F_u\}_{n,i} - \{F_m\}_{n,i}}{\{F_o\}_{n,i}} \tag{8}$$

where, $\{F_m\}_{n,i} = \lambda_{min,1}\{F_0\}_{1,i} + \lambda_{min,2}\{F_0\}_{2,i} + ... + \lambda_{min,n-1}\{F_0\}_{n-1,i}$. Therefore, the corresponding lateral load, deflection, and the member strength are obtained through Equations (9), (10), and (11).

Lateral load $$\{P_p\}_n = \lambda_{min,1}\{P\} + \lambda_{min,2}\{P\}\sqrt{\sum_{j=1}^{N} m(\phi_j(u)\Gamma_j(u))^2} + \cdots + \lambda_{min,n}\{P\}\sqrt{\sum_{j=1}^{N} m(\phi_j(u)\Gamma_j(u))^2} \tag{9}$$

Member strength $$\{F_m\}_n = \lambda_{min,1}\{F_0\}_1 + \lambda_{min,2}\{F_0\}_2 + ... + \lambda_{min,n}\{F_0\}_n \tag{10}$$

Lateral deflection $\{d_p\}_n = \lambda_{min,1}\{d\}_1 + \lambda_{min,2}\{d\}_2 + ... + \lambda_{min,n}\{d\}_n$
(11)

In MODIA, the stiffness of structures are gradually reduced as hinges are forming up to mechanism, instead of increasing lateral force at each load step. Consequently, the same level of initial load intensity, but with different profile, is used at all steps. The summation of load profile of all steps up to the mechanism is the inelastic ultimate load that structures can demonstrate.

Yield Surface of Members:

We assumed the material as the elastic-perfectly plastic. A stress path to the yield for columns is simplified as shown in Figure 2. The first possibility is obtained when a stress path follows Line (c) and hits the yield surface, the region of which is influenced by the axial load. The second situation is expected to occur when a stress path meets the vertical Line (b) where the influence of the axial force is negligible. The residual strength factor is then obtained from Equations (12) and (13). Similarly, the residual strength factor for beams and braces can be simply calculated at each iteration from Figure 3 and Equation (14) [7].

$$\text{¥ë}_{2,\text{column}} = \frac{CX_1}{CE} = \frac{OX_1 - OC}{OE - OC} \tag{12}$$

$$\text{¥ë}'_{2,\text{column}} = \frac{CX_2}{CE'} = \frac{OX_2 - OC}{OE' - OC} \tag{13}$$

$$\text{¥ë}_{2,\text{beam,brace}} = \frac{GX_1}{GH} = \frac{OX_1 - OG}{OH - OG} \tag{14}$$

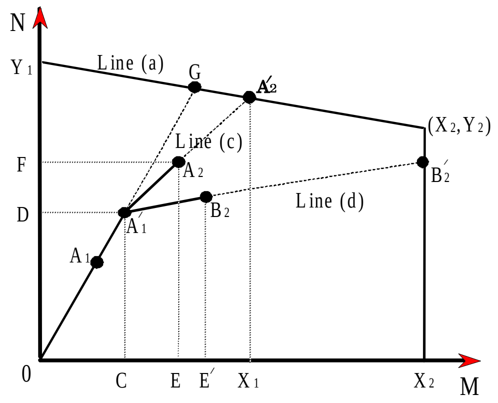


Figure 2: Yield surface for column

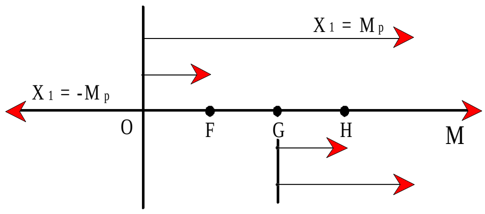


Figure 3: Stress path for beam and brace

Spectral Displacement:

The ratio of roof displacement Δ_{roof} to the spectral displacement S_d is the modal participation factor for the first mode at the roof level[2]. m_n is the mass at the *n*th story and ϕ_{nj} is the mode shape at the corresponding story for the *j*th mode. Therefore, the modal participation factor PF_{nj} can be calculated in Equation (15). The spectral displacement is the function of the roof displacement and modal participation factor, calculated in Equation (16). The spectral displacement S_d for multi-story system is calculated by using the SRSS combination method.

$$PF_{nj} = \frac{\sum_{i=1}^{n} m_i \phi_{ij}}{\sum_{i=1}^{n} m_i \phi_{ij}^{\ 2}} \phi_{nj} \tag{15}$$

$$S_d = \frac{\Delta_{roof}}{PF_{nj}} \tag{16}$$

Spectral Acceleration:

For the *j*th mode, the formula for calculating the effective mass coefficient α_m is given in Equation (17)[5]. The spectral acceleration is the function of the base shear *V*, weight of building *W*, and the effective mass coefficient. It can be calculated in Equation (18). The spectral acceleration S_a for multi-story system is also calculated by using the SRSS combination method.

$$\alpha_j = \frac{(\sum_{i=1}^{n} m_i \phi_{ij})^2}{(\sum_{i=1}^{n} m_i)(\sum_{i=1}^{n} m_i \phi_{ij}^{2})} \tag{17}$$

$$S_a = \frac{V}{W \cdot \alpha_j} \tag{18}$$

SEVEN-STORY EXAMPLE FRAME

Frame Analyzed:
In this study, 7-story steel frame is used to evaluate the inelastic capacity based on the proposed design approach. El-Centro earthquake is applied to the frame. The structural descriptions are shown in Figure 4, Tables 1, and 2. The initial lateral load profile is based on the story shear distribution by elastic spectrum analysis.

Table 1: Properties of members

Index	A (in^2)	I (in^4)	Z_p (in^3)	M_p (ksi)	P_y (kips)	Section
C1	51.8	2,140	320	11,520	1,864.8	W14×176
C2	62.0	2,660	390	14,040	2,232.0	W14×211
C3	68.5	3,010	436	15,696	2,466.0	W14×233
C4	83.3	3,840	542	19,512	2,998.8	W14×283
B1	30.6	3,100	289	10,404	-	W24×104
B2	38.5	4,020	370	13,320	-	W24×131
B3	47.7	5,170	468	16,848	-	W24×162

Table 2: Properties of frame

Young's Modulus	29,500 ksi	Story Mass	0.49 kip-sec^2/in
Yield Strength	36 ksi	Damping	0.05%

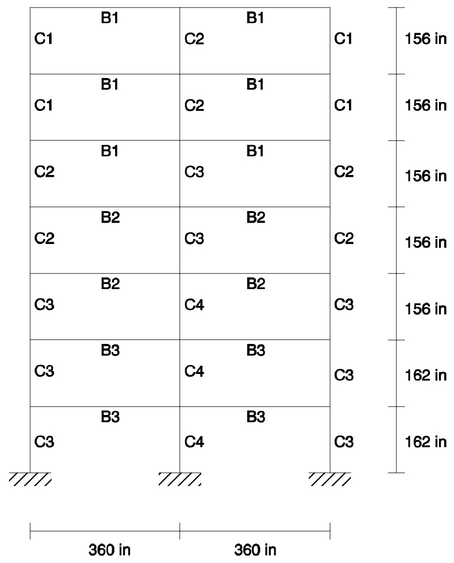


Figure 4: 7-story steel frame

Determination of Ultimate Capacity:
The ultimate capacity of structure may be obtained among when : 1) the formation of total collapse mechanism,

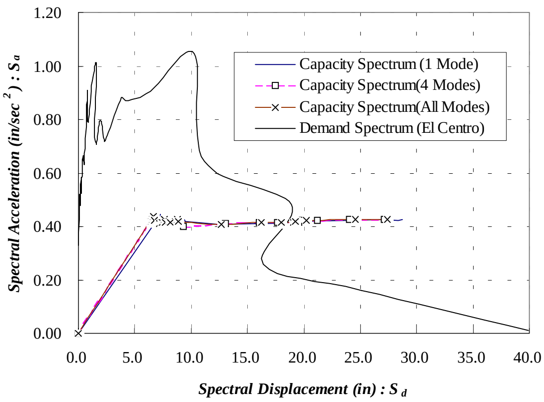


Figure 5: Spectral acceleration-deflection relation

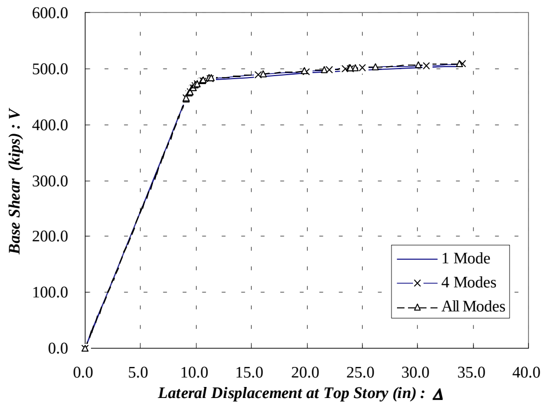


Figure 6: Base shear-displacement relationship

2) the formation of side sway mechanism due to local collapse of story and 3) the formation of combined collapse mechanism. In this study the side-sway mechanism is adopted for the ultimate capacity of structure. The limit of drift ratio of 1/50 is specified for the side-sway failure mechanism. Number of modes is selected as the parameter for the capacity spectrum. 4 Mode in Figure means that the first mode to fourth mode are considered in the inelastic analysis. Demand spectrum is obtained by using the ADRS method[2].

Seismic Evaluation of Frame:

The base shear-displacement relationship is an important characteristic for inelastic dynamic analysis. As shown in Figure 5, when all modes are considered in the inelastic analysis, it shows the largest ultimate capacity of structure. And the capacity of structure is larger than the demand for structure in which the demand depends on seismic force. The capacity spectrum considering only the first mode results in larger spectral displacement. This means that the commonly used pushover analysis with constant lateral load profile must be modified.

Variation of Inelastic Characteristics:

Figures 7 and 8 show the variation of lateral deflection and ductility for the structure in the inelastic range. At Step 1 the first plastic hinge is formed while Step 18 means the last plastic hinge where the ultimate state is achieved. As the structure enters into the inelastic range, the lateral deflection is increased.

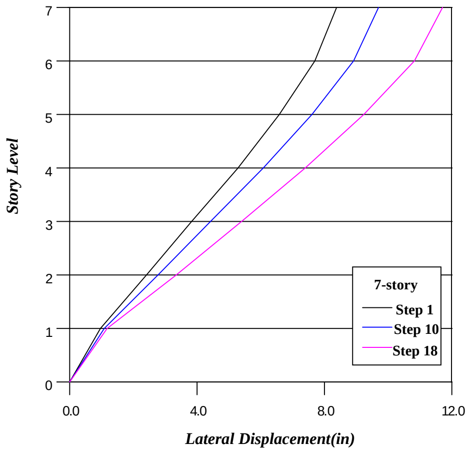


Figure 7: Variation of lateral deflection

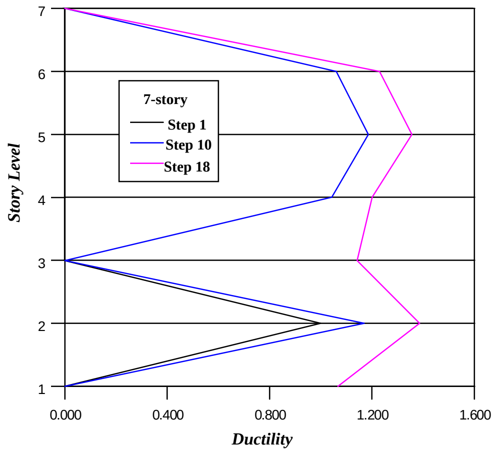


Figure 8: Variation of ductility

One of the important characteristics of inelastic analysis is to find the ductility as shown in Figure 8. This ductility ratio can be also used for the evaluation of inelastic spectrum of structure

Variation of Effective Mass Coefficient and Modal Participation Factor:

As the structure behaves in the inelastic range, the effective mass coefficient(EMC) α and modal participation factor(MPF) PF are changed. The range of variation is from 0.75 to 0.92 for EMC while from 1.15 to 1.45 for modal participation factor. In case of first mode only, the EMC and MPF show the lower values.

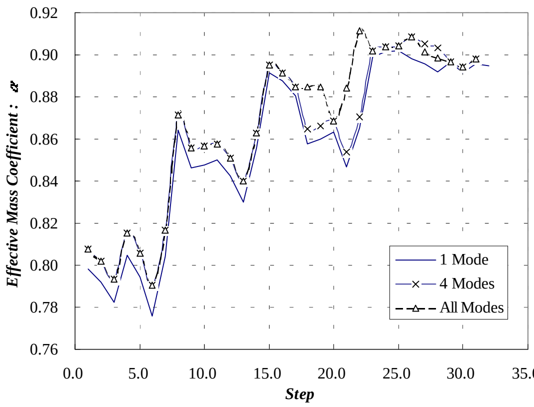


Figure 9: Variation of effective mass coefficient

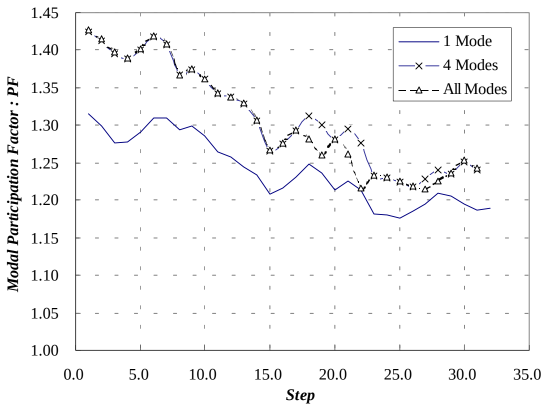


Figure 10: Variation of modal participation factor

INELASTIC ANALYSIS OF HIGH-RISE BUILDING

Frame Analysed:

A 15-story building(Figure 11) designed by Englekirk [6] is analyzed based on the methodology proposed in this study. The system is designed for earthquake load and the initial distribution of lateral loads for inelastic analysis is based on the earthquake inertial load calculated from response spectral analysis.

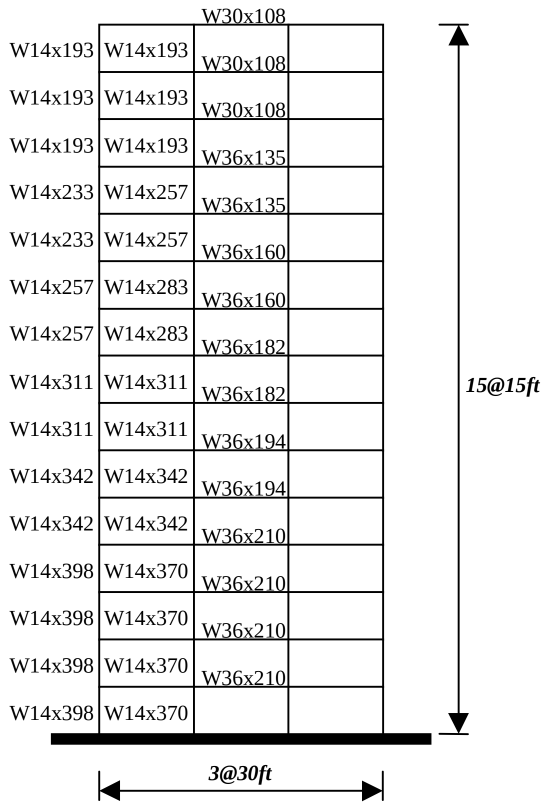
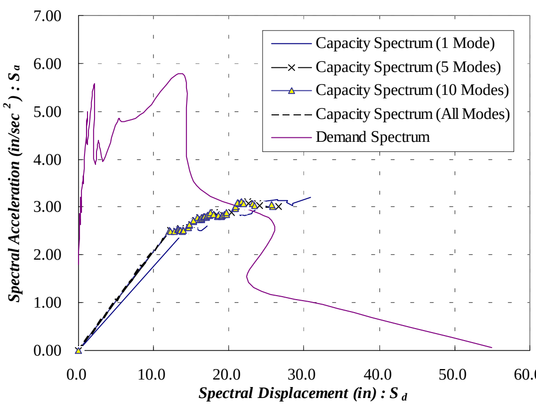


Figure 11: Elevation of steel frame

Seismic evaluation of structure:

As shown in Figures 12 and 13, the capacity of structure is larger than the demand for structure. Similar to seven-story frame, the capacity spectrum considering only one mode results in largest spectral displacement.



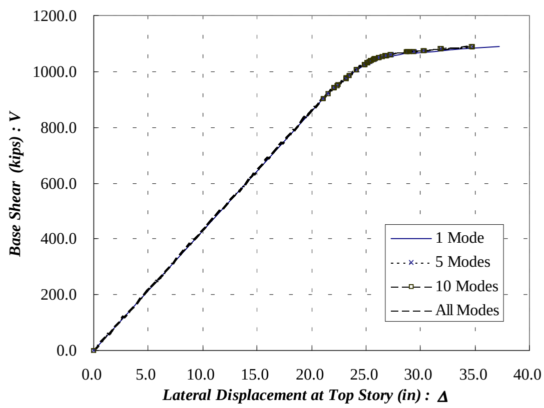


Figure 12: Spectral acceleration-deflection relation **Figure 13: Base shear-displacement relationship**

Variation of Effective Mass Coefficient and Modal Participation Factor:

For the fifteen-story frame, the range of variation is from 0.72 to 0.82 for effective mass coefficient(EMC) while from 1.1 to 1.8 for modal participation factor(MPF). In case of fundamental mode only, the EMC and MPF show the lower results.

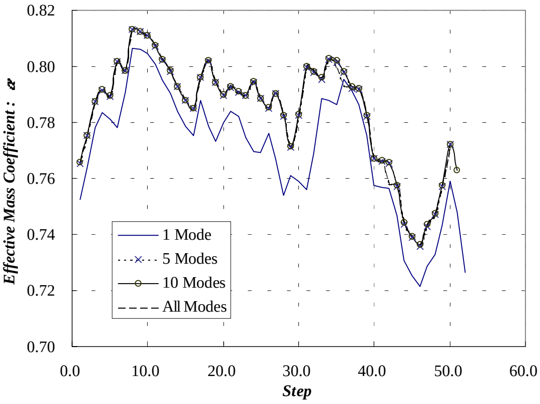


Figure 14: Variation of effective mass coefficient

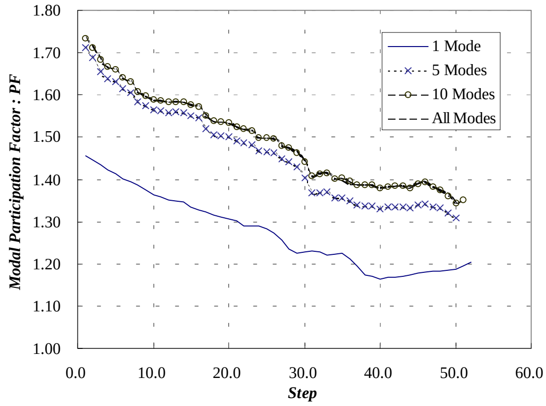


Figure 15: Variation of modal participation factor

CONCLUSIONS

Modified dynamic inelastic analysis technique based on mode shapes has been introduced to evaluate the seismic capacity of structures, which would lead to the following summaries.

1. The seismic evaluation of structure can be obtained by the proposed method. For two frame analyzed, the capacity of structure is larger that the demand required.
2. The capacity of structure considering the first mode only is lower than those of structure considering more modes.
3. It was possible to predict the inelastic load-deflection relationship and can check the built-in inelastic moment redistribution capability.
4. The range of effective mass coefficient is from 0.7 to 0.9 while that of modal participation factor is from 1.1 to 1.8. In case of first mode only, the EMC and MPF show the lower values.

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