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An investigation on the accuracy of pushover analysis for estimating the seismic deformation of braced steel frames

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Abstract

This paper investigates the potentialities of the pushover analysis to estimate the seismic deformation demands of concentrically braced steel frames. Reliability of the pushover analysis has been verified by conducting nonlinear dynamic analysis on 5, 10 and 15 story frames subjected to 15 synthetic earthquake records representing a design spectrum. It is shown that pushover analysis with predetermined lateral load pattern provides questionable estimates of inter-story drift. To overcome this inadequacy, a simplified analytical model for seismic response prediction of concentrically braced frames is proposed. In this approach, a multistory frame is reduced to an equivalent shear-building model by performing a pushover analysis. A Conventional shear-building model has been modified by introducing supplementary springs to account for flexural displacements in addition to shear displacements. It is shown that modified shear-building models have a better estimation for the nonlinear dynamic response of real framed structures compare to nonlinear static procedures.

Keywords: pushover analysis; concentrically braced frames; shear building; nonlinear dynamic analysis; seismic demands

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1- Introduction

Both structural and nonstructural damage sustained during earthquake ground motions is primarily produced by lateral displacements. Thus, the estimation of lateral displacement demands is of primary importance in performance based earthquake resistant design; specially, when damage control is the main quantity of interest. Nonlinear time history analysis of a detailed analytical model is perhaps the best option for the estimation of deformation demands. However, there are many uncertainties associated with the generation of site-specific input and with the analytical models presently employed to represent structural behavior. In many cases, the effort associated with detailed modeling and analysis may not be feasible; therefore, it is prudent to have a simpler analysis tool in order to assess the seismic performance of a frame structure.

The estimation of seismic deformation demands for multi-degree-of-freedom (MDOF) structures has been the subject of many studies [1-4]. Although studies differ in their approach, commonly an equivalent SDOF system is first established as the reduced model with which the inelastic displacement demands of the full model are estimated. Consequently, the inelastic displacement demands are translated into local deformation demands, either through multiplicative conversion factors, derived from a large number of non-linear analyses of different types of structural systems, or through building specific relationships between global displacements and local deformations, developed through a pushover analysis. Miranda [5-6] and Miranda et al. [7] have incorporated a simplified model of a building based on an equivalent continuum structure consisting of a combination of a flexural and a shear cantilever beams to develop an approximate method to estimate deformation demands in multistory buildings subjected to earthquakes. Although in this method the effect of nonlinear behavior is considered by using some amplification factors, the flexural and shear cantilever beams can only behave in elastic range of vibration.

In the non-linear static procedure (NSP), or pushover analysis, in the recent NEHRP guidelines [8,9], the seismic demands are computed by non-linear 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. However, after the structure yields, both assumptions are approximate. Therefore, deformation estimates obtained from a pushover analysis may be very inaccurate for structures in which higher mode effects are significant and in which the story shear forces vs. story drift relationships are sensitive to the applied load pattern [10]. None of the invariant force distributions can account for the contributions of higher modes to response, or for a redistribution of inertia forces because of structural yielding and the associated changes in the vibration properties of the structure. To overcome these limitations, several researchers have proposed adaptive force distributions that attempt to follow more closely the time-variant distributions of inertia forces [2, 11, 12]. While these adaptive force distributions may provide better estimates of seismic demands, they are conceptually complicated and computationally demanding for routine application in structural engineering practice. For practical applications, modal pushover analysis has been developed by Chopra and Goel [13, 14]. In this method, the seismic demand of the effective earthquake forces is determined by a pushover analysis using the inertia force distribution for each mode. Combining these 'modal' demands due to the first two or three terms of the expansion provides an estimate of the total seismic demand on inelastic systems. However, this approximate method is intended to provide rough estimates of maximum lateral deformations and it is not accurate enough to be a substitute for more detailed analyses, which are appropriate during the final evaluation of the proposed design of a new building or during the detailed evaluation of existing buildings.

In the present study, the accuracy of pushover analysis for estimating the seismic deformation of concentrically braced steel frames is investigated. It is shown that pushover analysis could never be a perfect substitute of dynamic time-history analysis. To overcome this inadequacy, a conventional shear-building model has been modified by introducing supplementary springs to account for flexural displacements in addition to shear drifts. Reliability of this modified shear-building model is then investigated by conducting nonlinear dynamic analysis on 5, 10 and 15 story concentrically braced steel frames subjected to 15 different synthetic earthquake records representing a design spectrum. It is shown that the proposed modified shear-building models more accurately estimate the nonlinear dynamic response of the corresponding concentrically braced frames compare to nonlinear static procedures.

2- Modeling and Assumptions

In the present study, three concentric braced steel frames, as shown in Figure 1, with 5, 10 and 15 stories have been considered. The buildings are assumed to be located on a soil type S_D and a seismically active area, zone 4 of the UBC 1997 [15] category, with PGA of 0.44 g. All connections are considered to be simple. The frame members were sized to support gravity and lateral loads determined in accordance with the minimum requirements of UBC 1997 [15]. In all models, the top story is 25% lighter than the others. IPB, IPE and UNP sections, according to DIN standard, are chosen for columns, beams and bracings, respectively. To eliminate the over strength effect, auxiliary sections have been artificially developed by assuming a continuous variation of section properties. In the code type design, once the members were seized, the entire design was checked for the code drift limitations and if necessary refined to meet the requirements. For static and nonlinear dynamic analysis, computer program Drain-2DX [16] was used to predict the frame responses. The Rayleigh damping is adopted with a constant damping ratio 0.05 for the first few effective modes. A two-dimensional beam-column element that allows for the formation of plastic hinges at concentrated points near its ends was employed to model the columns. The bracing elements are assumed to have an elastic-plastic behavior in tension and compression. The yield capacity in tension is set equal to the nominal tensile resistance, while the yield capacity in compression is set to be 0.28 times the nominal compressive resistance as suggested by Jain et al. [17].

To investigate the accuracy of different methods to predict the seismic response of concentrically braced steel frames, fifteen seismic motions are artificially generated using the SIMQKE program [18], having a close approximation to the elastic design response spectra of UBC 1997 [15] with a PGA of 0.44g. Therefore, these synthetic earthquake records are expected to be representative of the design spectra. The comparisons between artificially generated spectra and the UBC 1997 [15] design spectra are shown in Figure 2.

3- Nonlinear Static Procedure

In the Nonlinear Static Procedure (NSP), or pushover analysis, monotonically increasing lateral forces are applied to a nonlinear mathematical model of the building until the displacement of the control node at the roof level exceeds the target displacement. The lateral forces should be applied to the building using distributions or profiles that bound, albeit approximately, the likely distribution of inertial forces in the design earthquake. The recent NEHRP guidelines [8, 9] indicate that for a specific earthquake, the building should have enough capacity to withstand a specified roof displacement. This is called the target displacement and is defined as an estimate of the likely building roof displacement in the design earthquake. The Guidelines give an indication on how to estimate the target displacement using the following expression:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} \tag{1}$$

where C_0= modification factor to relate the spectral displacement and expected maximum inelastic displacement at the roof level; C_1= modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response; C_2= modification factor to represent the effects of stiffness degradation, strength deterioration, and pinching on the maximum displacement response; C_3= modification factor to represent increased displacements due to dynamic second-order effects; T_e= effective fundamental period of the building in the direction under consideration calculated using the secant stiffness at a base shear force equal to 60% of the yield force; and S_a= response spectrum acceleration at the effective fundamental period and damping ratio of the building. The factors C1, C2, and C3 serve to modify the relation between mean elastic and mean inelastic displacements where the inelastic displacements correspond to those of a bi-linear elastic-plastic system. The effective stiffness, K_e, the elastic stiffness, K_i, and the secant stiffness at maximum displacement, K_s, are identified in Figure 3. To calculate the effective stiffness, K_e, and yield strength, V_y, line segments on the force-displacement curve were located using an iterative procedure that approximately balanced the area above and below the curve [8, 9].

A nonlinear static procedure is used to evaluate the seismic performance of 5, 10 and 15 story concentrically braced frames shown in Figure 1. To accomplish this, target displacement corresponding to the UBC 1997 [15] design spectra is estimated in accordance with equation (1). Subsequently, the pushover analysis is performed under a predetermined load pattern to achieve the target displacement. Story demands computed at this stage are considered as estimates of the maximum demands experienced by the structure in the design earthquake. For all pushover analyses, three vertical distributions of lateral load are considered. A vertical distribution proportional to the shape of the fundamental mode of vibration; a uniform distribution proportional to the total mass at each level; and a vertical distribution proportional to the values of C_{vx} given by the following equation [8, 9]:

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{2}$$

where C_{vx} is the vertical distribution factor; w_i and h_i are the weight and height of the i^{th} floor above the base, respectively; n is the number of stories; and k is an exponent increases from 1 to 2 as period varies from 0.5 to 2.5 second.

In order to demonstrate the validity of the nonlinear static procedure to predict the displacement demands of concentrically braced frames, nonlinear dynamic analyses have been performed for all 15 synthetic earthquakes records representing UBC design spectra. The maximum roof displacements suggested by the nonlinear static procedure are compared with mean and mean plus one standard deviation of the results for all earthquakes in Figure 4. It is shown that the results obtained by this method are slightly underestimates. However, the accuracy of nonlinear static procedure to predict the maximum roof displacement caused by the design ground motion seems to be acceptable for practical applications. Similar conclusions are reported by Gupta and Krawinkler [4] for regular SMRF structures.

In order to evaluate the relative accuracy of pushover analysis for prediction of maximum story drift demands in individual stories, for a given target roof displacement, the results are compared with average of those of 15 synthetic earthquakes. As shown in Figure 5, nonlinear static procedure provides questionable estimates of inter story drift demands for concentrically braced frames investigated in this study. The results illustrated in this figure are obtained by using a vertical distribution of lateral loads proportional to the values of C_{vx} given by the equation (2).

Using different distribution patterns, the effects of pre-assumed lateral load on the results of pushover analysis have been investigated. Maximum story displacement and maximum drift distribution of 5-story frame suggested by nonlinear static procedures with different vertical distributions of lateral load are compared with average of those of 15 synthetic earthquakes in Figure 6. One can clearly observe from this figure that the results are very sensitive to the choice of lateral load pattern and there is very large scatter in the observations, particularly for the maximum drift distribution. Similar results have been obtained for 10 and 15-story models.

Accordingly, an acceptable estimation of story drift demands over the height of the structure is difficult to accomplish by using nonlinear static procedure because of the dependence to multitude factors such as relative strength and stiffness of the stories, effects of higher mode, pre-assumed lateral load pattern and characteristics of the ground motions. To overcome this inadequacy, a new simplified model for prediction of nonlinear dynamic response of concentrically braced frames is introduced in the sequel.

4- Shear and Flexural Deformations

Recent design guidelines, such as FEMA 273 [8], FEMA 356 [9] and SEAOC Vision 2000 [19], place limits on acceptable values of response parameters; implying that exceeding of these limits is a violation of a performance objective. Among various response parameters, the inter-story drift is considered as a reliable indicator of damage to nonstructural elements, and is widely used as a failure criterion because of the simplicity and convenience associated with its estimation. Considering the 2-D frame shown in Figure 7-a, the axial deformation of the columns results in increase of lateral story and inter-story drifts. In each story, the total inter-story drift (Δ_t) is a combination of the shear deformation (Δ_{sh}) due to shear flexibility of the story, and the flexural deformation (Δ_{ax}) due to axial flexibility of the lower columns. Hence, inter-story drift could be expressed as:

$$\Delta_t = \Delta_{sh} + \Delta_{ax} \tag{3}$$

Flexural deformation does not contribute to the damage imposed to the story, though it may impair the stability due to P-Δ effects. For a single panel, as shown in Figure 7-b, shear deformation could be calculated using the following approximate equation [20]:

$$(\Delta_{sh}) = (\Delta_t) + \frac{H}{2L}\left(U_6 + U_8 - U_2 - U_4\right) \tag{4}$$

Where, U_6, U_8, U_2 and U_4 are vertical displacements, as shown in Figure 7-b, and H and L are height of the story and span length, respectively. The axial deformation of beams is neglected in equation (4). The derivation of equation (4) is described in detail in Moghaddam et al. [21]. For multi-span models, the maximum value of the shear drift in different panels would be considered as the shear story drift.

5- Modified Shear Building Model

The modeling of engineering structures usually involves a great deal of approximation. Among the wide diversity of structural models that are used to estimate the non-linear seismic response of building frames, the shear building is the one most frequently adopted. In spite of some drawbacks, it is widely used to study the seismic response of multi-story buildings because of its simplicity and low computational expenses [22], which might be considered as a great advantage for a design engineer to deal with. Lai et al. [23] have investigated the reliability and accuracy of such shear-beam models.

Lateral deformations in buildings are usually a combination of lateral shear-type deformations and lateral flexural-type deformations. In ordinary shear building models, the effect of column axial deformations is usually neglected, and therefore, it is not possible to calculate the nodal displacements caused by flexural deformation, while it may have a considerable contribution to the seismic response of most frame-type structures. In the present study, the shear-building model has been modified by introducing supplementary springs to account for flexural displacements in addition to shear displacements. According to the number of stories, the structure is modeled with *n* lumped masses, representing the stories. Only one degree of freedom of translation in the horizontal direction is taken into consideration and each adjacent mass is connected by two supplementary springs as shown in Figure 8. As shown in this figure, the modified shear-building model of a frame condenses all the elements in a story into two supplementary springs, thereby significantly reduces the number of degrees of freedom. The stiffnesses of supplementary springs are equal to the shear and bending stiffnesses of each story, respectively. These stiffnesses are determined by enforcing the model to undergo the same displacements as those obtained from a pushover analysis on the original frame model. As shown in Figure 8, the material nonlinearities may be incorporated into stiffness and strength of supplementary springs. In Figure 8, m_i represents the mass of i^{th} floor; and V_i and S_i are, respectively, the total shear force and yield strength of the i^{th} story obtained from the pushover analysis. $(k_t)_i$ is the nominal story stiffness corresponding to the relative total drift at i^{th} floor (Δ_t in Figure 7). $(k_{sh})_i$ denotes the shear story stiffness corresponding to the relative shear drift at i^{th} floor (Δ_{sh} in Figure 7). $(k_{ax})_i$ represents the bending story stiffness corresponding to the flexural deformation at i^{th} floor (Δ_{ax} in Figure 7), and $(\alpha_t)_i$, $(\alpha_{sh})_i$ and $(\alpha_{ax})_i$ are over-strength factors for nominal story stiffness, shear story stiffness and bending story stiffness at i^{th} floor, respectively. $(k_t)_i$ and $(\alpha_t)_i$ are determined from a pushover analysis taking into account the axial deformation of columns. In this study, the nonlinear force-displacement relationship between the story shear force (V_i) and the total inter-story drift $(\Delta_t)_i$ has been replaced with an idealized bilinear relationship to calculate the nominal story stiffness $(k_t)_i$ and effective yield strength (S_i) of each story as shown in Figure 8. Line segments on the idealized force-displacement curve were located using an iterative procedure that approximately balanced the area above and below the curve. The nominal story stiffness $(k_t)_i$ was taken as the secant stiffness calculated at a story shear force equal to 60% of the effective yield strength of the story [8, 9].

Using equation (4), shear story drift corresponding to each step of pushover analysis could be calculated and consequently $(k_{sh})_i$ and $(\alpha_{sh})_i$ are determined. As the transmitted force is equal in two supplementary springs, equation (3) could be rewritten as:

For $V_i \leq S_i$,

$$\frac{V_i}{(k_t)_i} = \frac{V_i}{(k_{sh})_i} + \frac{V_i}{(k_{ax})_i} \tag{5}$$

Hence,

$$\frac{1}{(k_t)_i} = \frac{1}{(k_{sh})_i} + \frac{1}{(k_{ax})_i} \tag{6}$$

For $V_i > S_i$ we have

$$\frac{S_i}{(k_t)_i} + \frac{V_i - S_i}{(\alpha_t)_i (k_t)_i} = \frac{S_i}{(k_{sh})_i} + \frac{V_i - S_i}{(\alpha_{sh})_i (k_{sh})_i} + \frac{S_i}{(k_{ax})_i} + \frac{V_i - S_i}{(\alpha_{ax})_i (k_{ax})_i} \tag{7}$$

Substituting equation (6) in (7), $(k_{ax})_i$ and $(\alpha_{ax})_i$ are obtained as follows:

$$(k_{ax})_i = \frac{(k_{sh})_i (k_t)_i}{(k_{sh})_i - (k_t)_i} \tag{8}$$

$$(\alpha_{ax})_i = \frac{(\alpha_{sh})_i (\alpha_t)_i \left[(k_{sh})_i - (k_t)_i\right]}{(\alpha_{sh})_i (k_{sh})_i - (\alpha_t)_i (k_t)_i} \tag{9}$$

Numerical experiments show that $(\alpha_{ax})_i$ is almost equal to 1 when columns are designed to prevent buckling against earthquake loads. According to the foregoing discussion, all of the parameters required to define a modified shear-building model corresponding to a given frame model, could be determined by performing one pushover analysis.

The shear inter-story drift, that causes damage to the structure, can be separated from the flexural deformation by using the modified shear-building model. The modified shear-building model also takes into account both the higher mode contribution to (elastic) structural response as well as the effects of material non-linearity. Therefore, this modified model represents the behavior of frame models more realistically as compared with other conventional approaches. Figure 9,– illustrates the response of 15 story frame model and its corresponding modified shear-building model under Imperial Valley 1979 earthquake. It is shown, in this figure, that modified shear-building model has a good capability to estimate the seismic response parameters of braced frames, such as roof displacement, total inter story drifts and shear inter-story drifts. This conclusion has been confirmed by further analyses on different models and ground motions. To verify the reliability of modified shear building model to estimate the seismic response parameters of concentrically braced frames, non-linear time history analyses have been performed for 5, 10 and 15 story frames and their corresponding modified shear-building models subjected to 15 synthetic earthquakes. Average of the results for frame models and modified shear building models are compared in Figure 10. This Figure indicates that, in average, displacement demands estimated by modified shear-building models agree very well with the 'exact' values from full-frame models. Hence, it can be concluded that modified shear-building model is very reliable and has a good capability to estimate the seismic response parameters of concentrically braced frames.

For each synthetic excitation, the errors in displacement demands computed by modified shear-building model relative to the 'exact' response were determined. Consequently, average of the errors was calculated for every story. Maximum errors corresponding to 5, 10 and 15 story frames are shown in Figure 11. As it is depicted for modified shear-building models, the errors are slightly larger in drift than in displacement, but still the maximum errors in all response quantities are only a few percent. Therefore, displacement demands estimated by modified shear-building models are effectively equivalent to those based on typical frame models of the same structure.

Table 1 compares fundamental period and total computational time for 5, 10 and 15 story braced frames and their corresponding modified shear-building model under 15 synthetic earthquakes. As shown in Table 1, the relatively small number of degrees of freedom for modified shear-building model results in significant computational savings as compare to the corresponding frame model. According to the results of this study, total computational time for modified shear-building models are less than 4% of those based on typical frame models. Therefore, having acceptable accuracy, using the modified shear-building model makes the structural analysis of concentrically steel braced frames to a large extend simple.

In summary, evaluating the deformation demands using modified shear-building models is demonstrated to be about the same as using the corresponding full-frame models, which are significantly more time-consuming to analyze. In the practical applications, the computational savings associated with the modified shear-building model makes it possible to consider more design alternatives and earthquake ground motions. This modified model has been used efficiently for optimum seismic design of concentrically braced steel frames [21].

Based on the outcomes of this study, the modified shear-building model has obvious superiority: it is reliable; it is simple and timesaving compared with other models; incorporation of non-linearity is easy; and the shear inter-story drift, which causes damage to the structure, can be easily determined. Thus, the proposed modified shear-building model is appropriate for practical application in building evaluation and design.

6- Conclusion

1. The ability of nonlinear static procedures to predict the maximum roof displacement caused by the design ground motion is emphasized for concentrically braced steel frames. It is shown that nonlinear static procedures with predetermined lateral load pattern are very sensitive to the choice of load pattern and provide questionable estimates of inter-story drift demands for concentrically braced steel frames.
2. A conventional shear-building model has been modified by introducing supplementary springs to account for flexural displacements in addition to shear drifts. Using this modified shear-building model, the mechanical properties of each story are condensed into two supplementary springs; therefore, the number of degrees of freedom is significantly decreased. All parameters required to define a modified shear-building model corresponding to the given full-frame model are determined by performing a pushover analysis.
3. Evaluating the deformation demands using modified shear-building models is demonstrated to be about the same as using the corresponding full-frame models, which are significantly more time-consuming to analyze. Therefore, making the structural analysis of concentrically braced steel frames to a large extend simple, the

proposed model is accurate enough for practical application in building evaluation and design.

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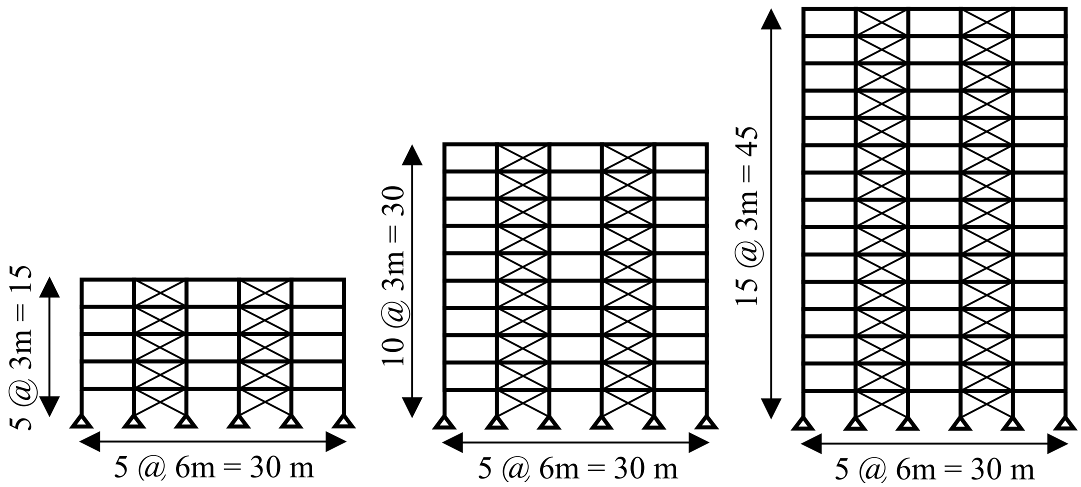


Figure 1. Typical geometry of concentric braced frames.

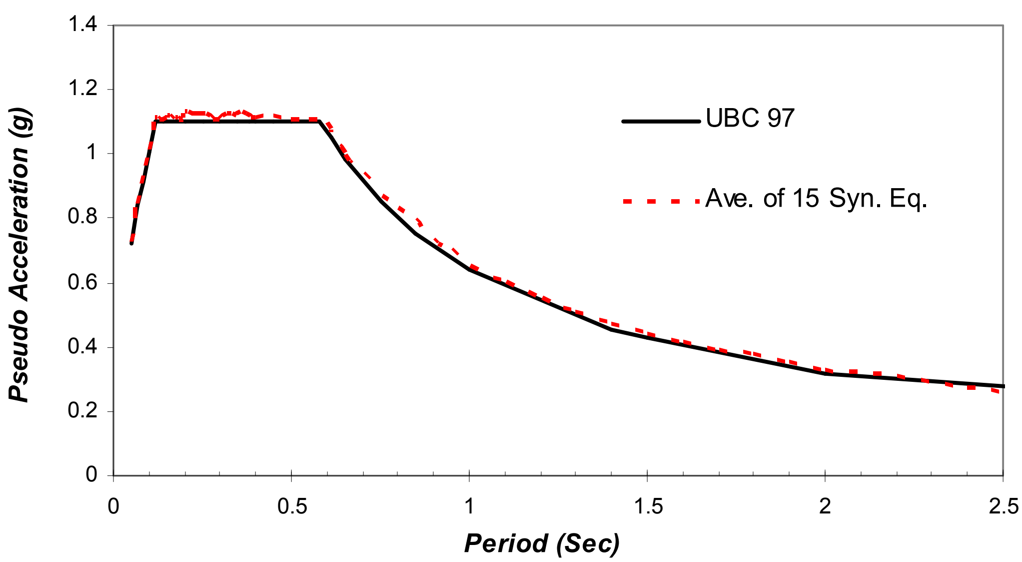


Figure 2. A comparison of UBC design spectrum with the average of 15 synthetic earthquakes.

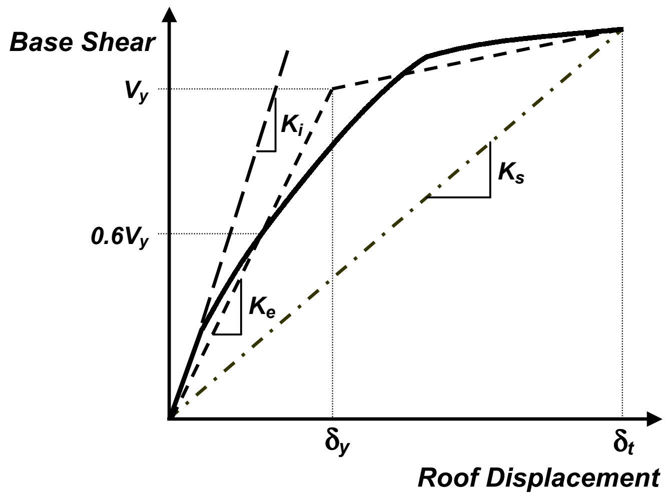


Figure 3. Idealized force-displacement curves.

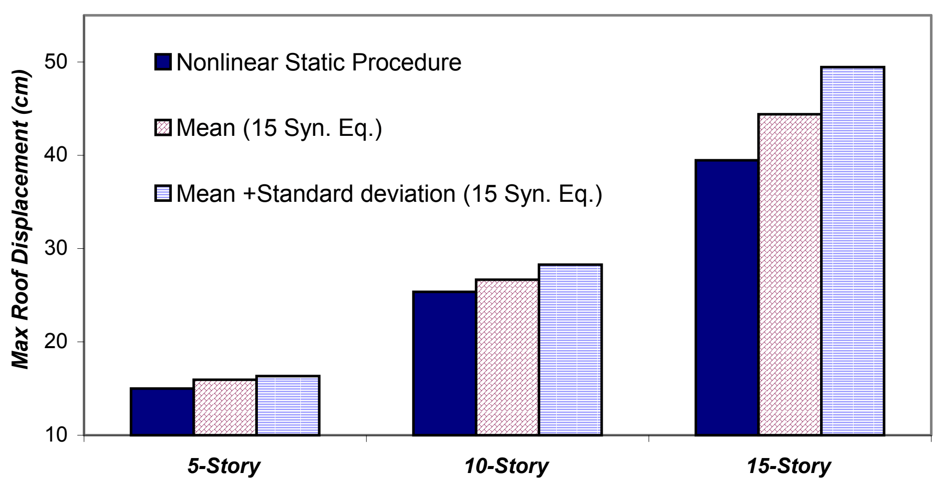
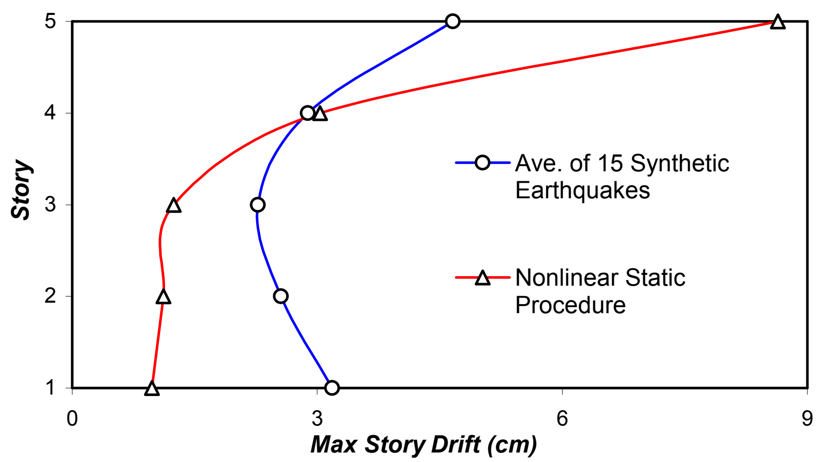
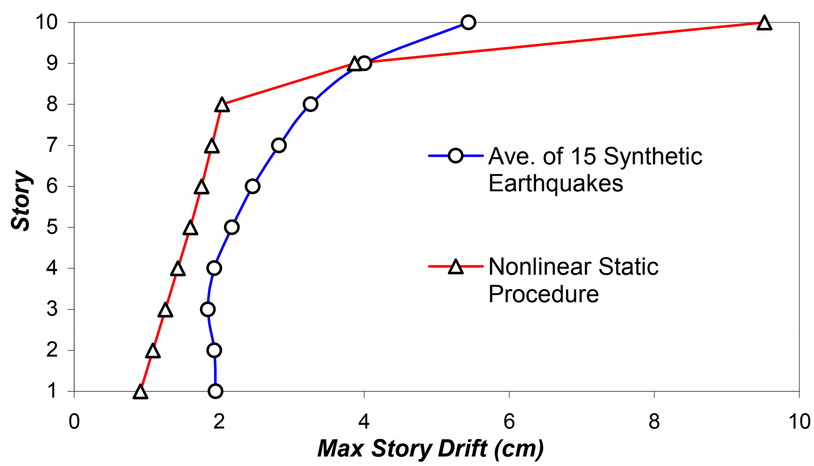


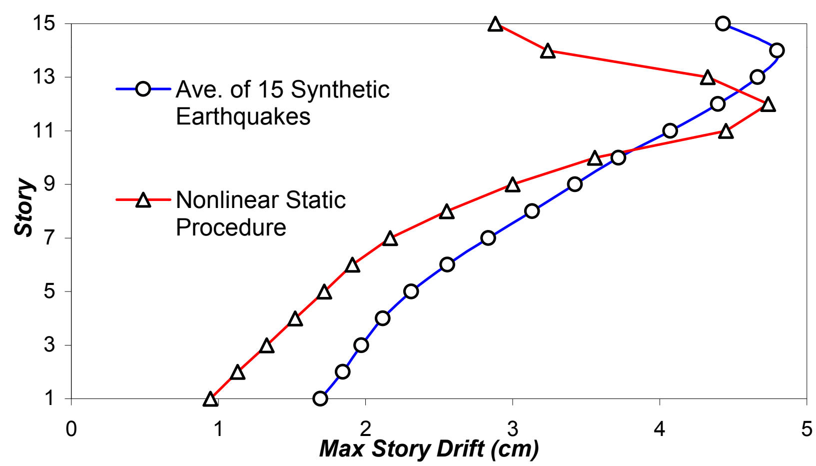
Figure 4. Comparisons of maximum roof displacement suggested by the NSP with mean and mean plus one standard deviation of the results for 15 synthetic earthquakes.



(a)

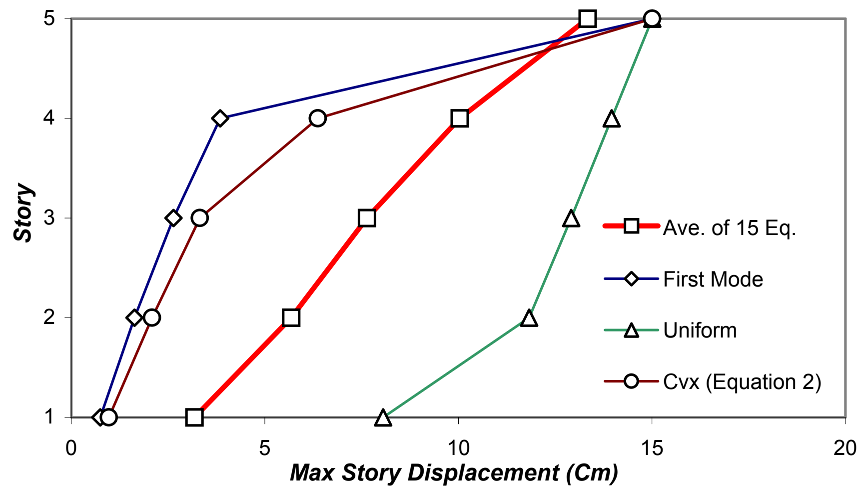


(b)

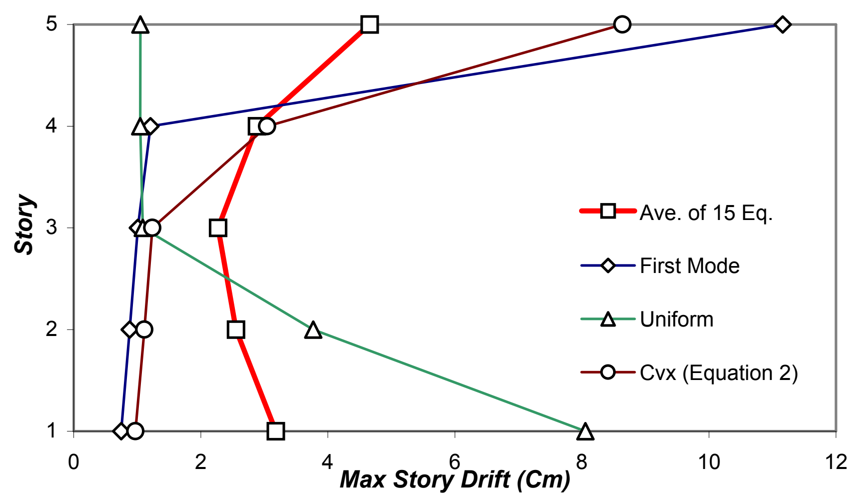


(c)

Figure 5. Comparison of the results obtained from a nonlinear static procedure with the average of 15 synthetic earthquakes; (a) 5-story model, (b) 10-story model, (c) 15-story model.



(a)



(b)

Figure 6. The Effect of vertical distribution of lateral load on the results of the nonlinear static procedure; (a) Max story displacement, (b) Max story drift.

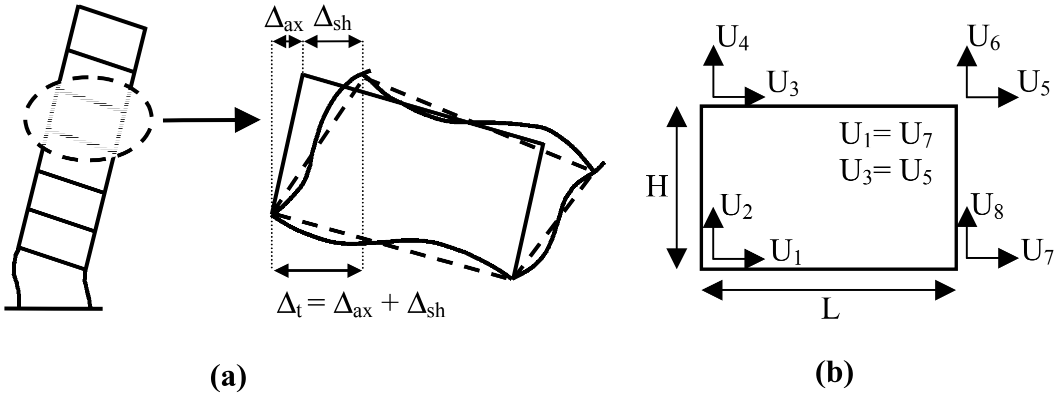


Figure 7. (a) Definitions of total inter-story drift (Δ_t), shear inter-story drift (Δ_{sh}) and the effect of axial flexibility of columns (Δ_{ax}), (b) Displacement components of a single panel.

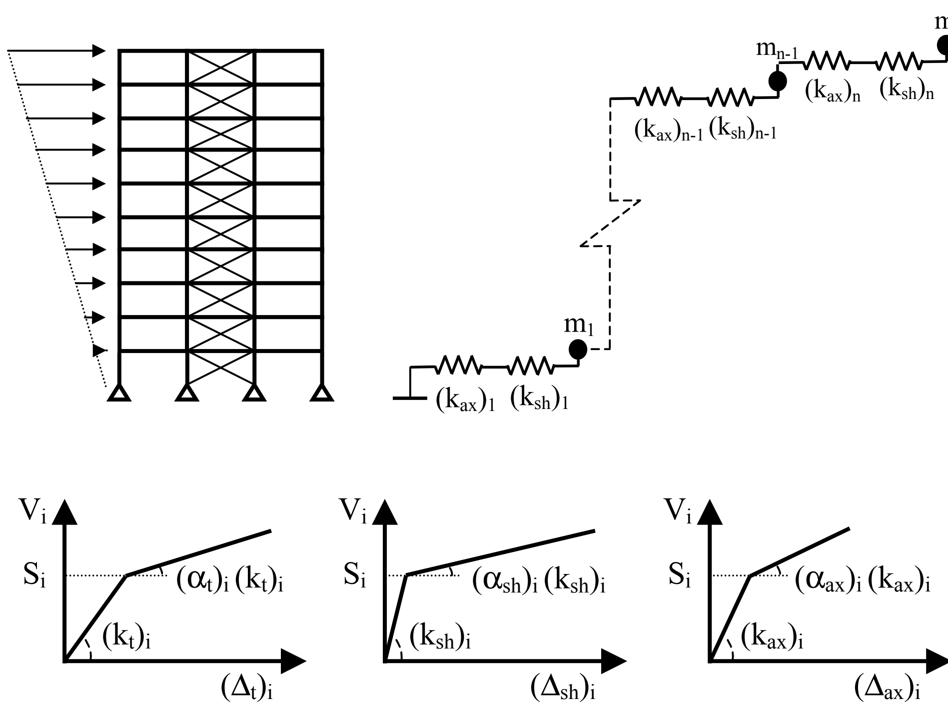


Figure 8. Using pushover analysis to define equivalent modified shear-building model.

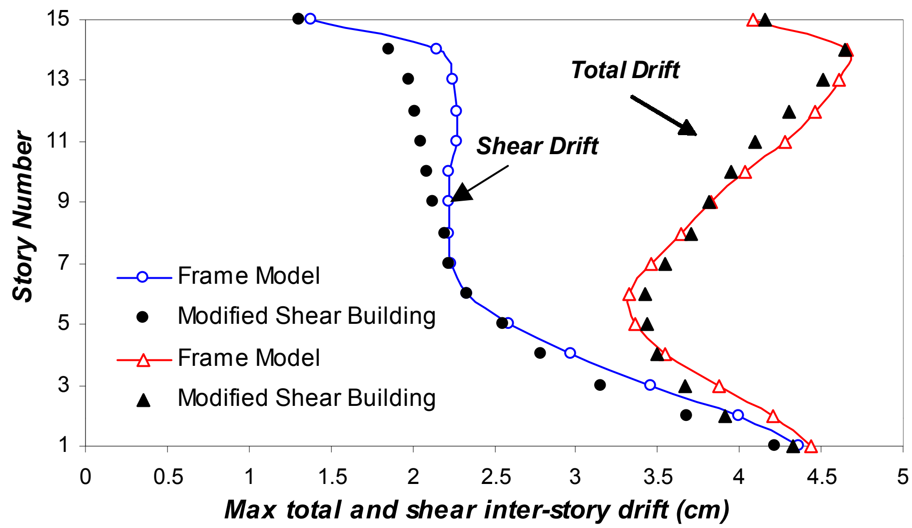


Figure 9. A comparison of frame model and modified shear-building model for 15-story model subjected to Imperial Valley 1979.

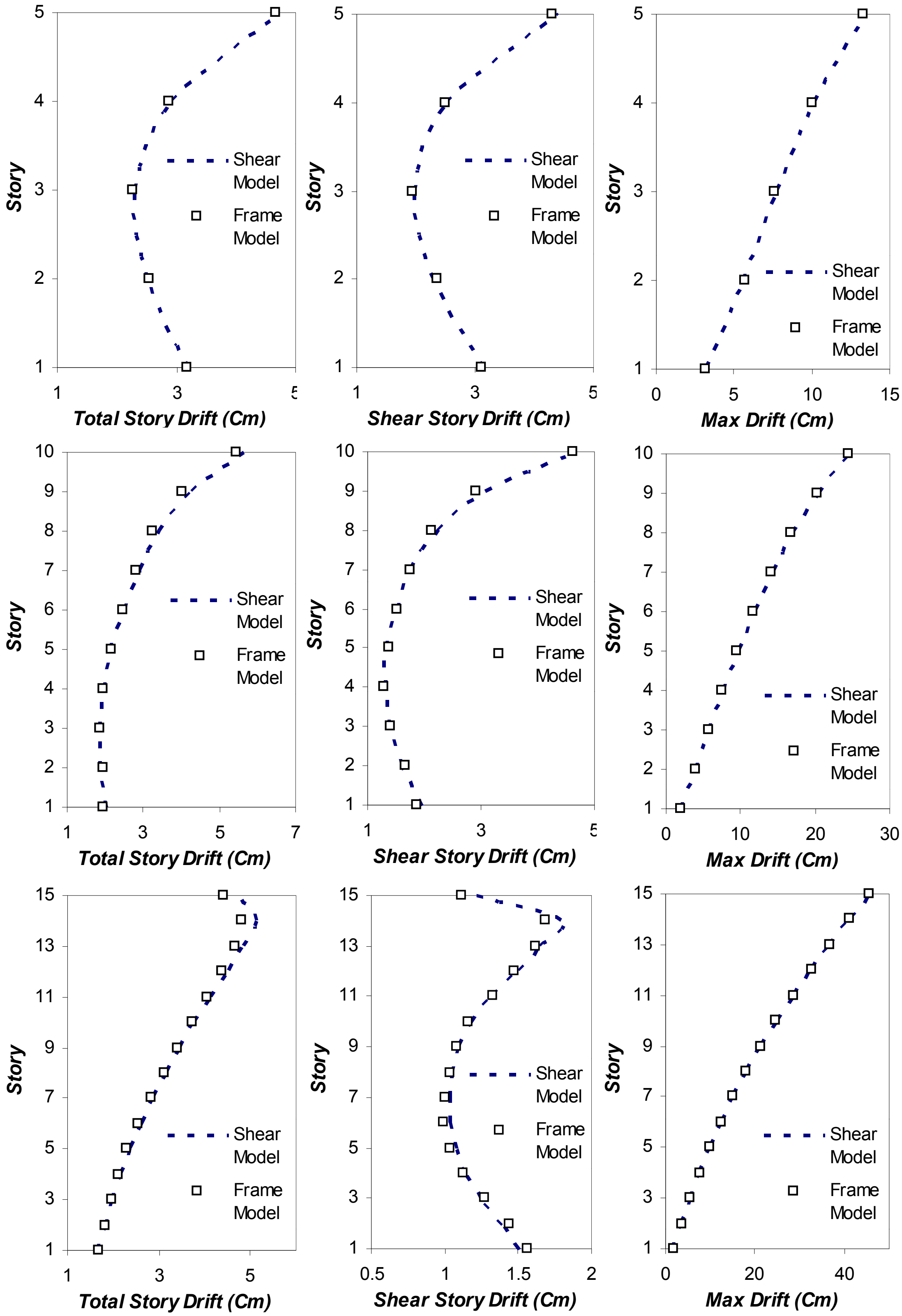


Figure 10. Comparison of the frame model and the corresponding modified shear-building model for 5, 10 and 15-story braced frames, Average of 15 synthetic earthquakes.

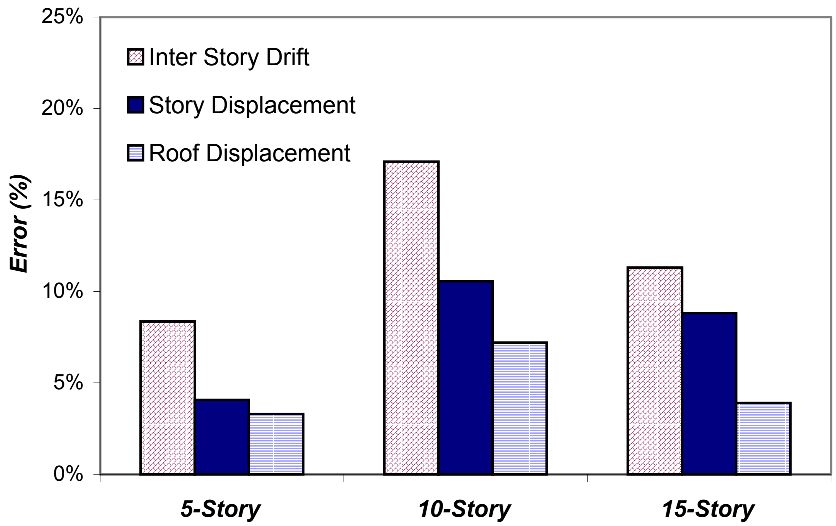


Figure 11. Maximum errors in displacement demands estimated by modified shear-building models, Average of 15 synthetic earthquakes.

Table 1. Fundamental period and total computational time for frame model and the corresponding modified shear-building model

	Fundamental Period (Sec)		***Total Computational Time (Sec)***	
	Frame Model	Modified Shear-Building	Frame Model	Modified Shear-Building
5-Story	0.6208	0.6209	906.0	36.3
10-Story	1.1127	1.1131	1616.1	52.0
15-Story	1.7705	1.7713	4915.4	67.7