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A simplified model for seismic response prediction of concentrically braced frames

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Abstract

This paper proposes a simplified analytical model for seismic response prediction of concentrically braced frames. In the proposed approach, a multistory frame model is reduced to an equivalent shear-building one by performing a static pushover analysis. The conventional shear-building model has been improved by introducing supplementary springs to account for flexural displacements in addition to shear displacements. The adequacy of the modified model has been verified by conducting nonlinear dynamic analysis on 5, 10 and 15 story concentrically braced frames subjected to 15 synthetic earthquake records representing a design spectrum. It is shown that the proposed improved shear-building models provide a better estimate of the nonlinear dynamic response of the original framed structures, as compared to the conventional models. While simplifying the analysis of concentrically braced frames to a large extend, and thus reducing the computational efforts significantly, the proposed method is accurate enough for practical applications in performance assessment and earthquake-resistant design.

Keywords: concentrically braced frames; shear buildings; non-linear dynamic analysis; seismic demands; pushover analysis; cumulative damage

1- Introduction

Both structural and nonstructural damages observed during earthquake ground motions are primarily produced by lateral displacements. Thus, the estimation of lateral displacement demands is of significant importance in performance-based design methods; specially, when damage control is the main quantity of interest. Most structures experience inelastic deformations when subjected to severe earthquake ground motions. Therefore, nonlinear behaviour of structures should be taken into account to have accurate estimation of deformation demands. Nonlinear time history analysis of a detailed analytical model is perhaps the best option for the estimation of deformation demands. However, due to many uncertainties associated with the site-specific excitation as well as uncertainties in the parameters of analytical models, in many cases, the effort associated with detailed modeling and analysis may not be justified and feasible. Therefore, it is prudent to have a reduced model, as a simpler analysis tool, to assess the seismic performance of a frame structure. Construction of such reduced model is the main goal of the present study.

The estimation of seismic deformation demands for multi-degree-of-freedom (MDOF) structures has been the subject of many studies [1-8]. Although those studies differ in their approach, they commonly establish an equivalent single-degree-of-freedom (SDOF) system as the reduced model with which the inelastic displacement demands of the full model are estimated. Consequently, the inelastic displacement demands are converted 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 using a pushover analysis. These approximate methods are particularly intended to provide rough estimates of maximum lateral deformations and are 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.

For the purpose of preliminary design and analysis of structures, many studies have been carried out to construct reduced nonlinear models that feature both accuracy and low computational cost. 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 series of flexural and shear cantilever beams to estimate deformation demands in multistory buildings subjected to earthquakes. Although in that 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. Some researchers [2, 8, 9] have attempted to develop analytical models to predict the inelastic seismic response of reinforced concrete shear-wall buildings, including both the flexural and shear failure modes. Lai et al. [10] developed a multi-rigid-body theory to analyze the earthquake response of shear-type structures. In that work, material non-linearity can be incorporated into the multi-rigid-body discrete model; however, it is not possible to calculate the nodal displacements caused by flexural deformations, which in most cases has a considerable contribution to the seismic response of frame-type structures.

Among the wide variety of structural models that are used to estimate the non-linear seismic response of building frames, the conventional shear building model is the most frequently utilized reduced model. In spite of some of its drawbacks, the conventional shear building model is widely used to study the seismic response of multi-story buildings mainly due to its excessive simplicity and low computational expenses. This model has been developed several decades ago and has been successfully employed in preliminary design of many high-rise buildings [11-13]. The reliability of conventional shear-building models to predict non-linear dynamic response of moment resistance frames is investigated by Diaz et al. [14]. It has been shown, there, that conventional shear building models overestimate the ductility demands in the lower stories, as compared with more accurate frame models. This is mainly due to inability of shear building models to distribute the inelastic deformations among the members of adjacent stories. To overcome this issue, in the present study, the conventional shear-building model has been improved by introducing supplementary springs to account for flexural displacements in addition to shear drifts. The construction of such reduced model is based on a static pushover analysis. Reliability of this modified shear-building model is investigated by conducting nonlinear dynamic analysis on 5, 10 and 15 story concentrically steel braced 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 the conventional shear-building models.

2- Modeling and assumptions

In the present study, three steel concentric braced frames with 5, 10 and 15 stories have been selected (Fig. 1). 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. Simple beam to column connections are considered to prevent the transmission of any moment from beams to the supporting columns. The frame members are 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. All joint nodes at the same floor were constrained together in the horizontal direction of the input ground motion. Once the structural members are seized, the entire design is checked for the code drift limitations and if necessary refined to meet the requirements.

For the static and nonlinear dynamic analysis, the computer program Drain-2DX [16] is used. The Rayleigh damping is adopted with a constant damping ratio 0.05 for the first few effective modes. The columns were modelled using a fibre-type element with distributed plasticity (element 15) in which the location of non-linearity within the elements is computed during the analysis. The brace members are assumed to have elastic-plastic behaviour 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 equal to 0.28 times the nominal compressive resistance as suggested by Jain et al. [17].

To investigate the accuracy of different methods for prediction of 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 Fig. 2.

3- Conventional shear building model

The conventional shear building model is an assembly of structural members connected along horizontal interfaces, which coincide with the floor levels and, therefore, with the levels where the building mass is assumed to be concentrated. These members can only undergo shear deformations when subjected to lateral forces as shown in Fig. 3.

The conventional shear building model has n degrees of freedom where n is the number of stories. The lateral stiffness $(k_t)_i$, yield strength S_i and over-strength factor $(\alpha_t)_i$ of the structural element representing the mechanical properties of the i^{th} floor, are computed on the basis of adequate assumptions regarding the deformed shape of the original frame. To accomplish this, a pushover analysis is conducted on the full-model framed structure and the relationship between the story shear force (V_i) and the total inter-story drift $(\Delta_t)_i$ is extracted. The nonlinear force-displacement relationship has been replaced with an idealized relationship to calculate the nominal story stiffness $(k_t)_i$ and effective yield strength (S_i) of each story as shown in Fig. 4. Line segments on the idealized force-displacement curve have been located using an iterative procedure that approximately balances the area above and below the curve. The nominal story stiffness $(k_t)_i$ is then taken as the secant stiffness calculated at a story shear force equal to 60% of the effective yield strength of the story [19, 20].

It is well known that deformation estimates obtained from a pushover analysis may be very inaccurate for structures in which higher vibration modes have significant contribution to the overall response. Also for situations where the resulting story shear forces, caused by the story drifts, are sensitive to the applied load pattern the application of the pushover analysis seems questionable [21, 22]. None of the invariant force distributions can account for the contributions of higher modes to the overall structural response or even the redistribution of inertia forces. This is due to yielding of structural components and the resulting changes in the vibration characteristics of the structure. This problem can be mitigated to some extend by applying more than one lateral load pattern which includes those that excite elastic higher mode effects.

In this study, pushover analyses are performed under different lateral load patterns to investigate the effects of pre-assumed load pattern on computed mechanical properties of each story. For all pushover analyses four different vertical distribution of lateral load are considered; a vertical distribution proportional to the shape of the fundamental mode of vibration; a triangular distribution according to UBC 97 [15]; a uniform distribution proportional to the total mass at each level; and finally a vertical distribution proportional to the values of C_{vx} given by following equation [19, 20]:

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} , \quad (1)$$

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. Also, 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.

The lateral stiffness and yield strength distributions corresponding to each case are compared in Fig. 5 for a 10-story concentrically braced frame. As shown in this figure, mechanical properties of the stories are rather insensitive to the predetermined lateral load pattern used for pushover analyses. It is particularly true if a rational lateral load distribution is used.

To evaluate the reliability of conventional shear-building models to estimate the displacement demands of concentrically braced frames, time history analyses have been performed on 5, 10 and 15 story full-frame models and their corresponding conventional shear-building models subjected to 15 synthetic earthquakes. For each seismic excitation, the errors in prediction of roof displacements, story displacements and inter-story drifts have been determined. Subsequently, for each story, the average value of the errors corresponding to 15 synthetic earthquakes has been calculated. Table 1 summarizes the maximum errors corresponding to 5, 10 and 15 story concentrically braced frames. As indicated in this table, using modified shear building models, maximum errors in estimation of roof and story displacements are small (less than 16 percent). However, maximum roof and story displacements are not good indicators of seismic performance of a structure as compared with story drifts. The results presented in Table 1 show that the errors in estimation of story drifts are much larger (2.5 times higher) compared to story displacements. Therefore, conventional shear-building models are not reliable enough to estimate the maximum story drifts of concentrically braced frames for the case of large non-linear deformations which is observed in sever earthquakes.

In Fig. 6, the maximum story displacement and maximum drift distribution of the 10-story frame obtained using conventional shear-building models are compared with the average of actual values for 15 synthetic earthquakes. This figure shows that, on average, conventional shear building models provide reasonable estimates of maximum roof and story displacements; however, estimated story drifts are not accurate enough. The errors are especially large for the case of the maximum drift estimated at the level of top stories where the estimated drift is 40% higher than the actual value. Although seismic forces in top stories may not control the overall design of the structure, inter-story drifts at the top floors could govern the seismic design of multi-story frames, especially for high-rise buildings where the higher mode effects are considerable.

As described very briefly, in the present study, the conventional shear-building model has been modified in order to achieve a better estimation of nonlinear dynamic response of real framed structures. More details of such extension are presented next.

4- Shear and flexural deformations

Recent design guidelines, such as FEMA 273 [19], FEMA 356 [20] and SEAOC Vision 2000 [23], 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 the convenience associated with its estimation.

Considering the 2-D frame shown in Fig. 7-a, the axial deformation of 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 can be expressed as:

$$\Delta_t = \Delta_{sh} + \Delta_{ax} \ . \tag{2}$$

Flexural deformation does not contribute in the damage imposed to the story, though it may impair the stability due to the P-Δ effects. Neglecting the axial deformation of beams, the shear deformation for a single panel, as shown in Fig. 7-b, is determined by [24],

$$\Delta_{sh} = \Delta_t + \frac{H}{2L}\left(U_3 + U_6 - U_2 - U_5\right) . \tag{3}$$

where, U_5, U_6, U_2 *and* U_3 are vertical displacements, as shown in Fig. 7-b. *H* and *L* are the height of the story and the span length, respectively. The derivation of Equation (3) is described in detail in Moghaddam et al. [25]. For multi-span models, the maximum value of the shear drift in different panels is considered as the shear story drift.

5- Modified shear building model

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. 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 Fig. 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 Fig. 8, the material nonlinearities may be incorporated into stiffness and strength of supplementary springs. In Fig. 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 Fig. 7). $(k_{sh})_i$ denotes the shear story stiffness corresponding to the relative shear drift at i^{th} floor (Δ_{sh} in Fig. 7). $(k_{ax})_i$ represents the bending story stiffness corresponding to the flexural deformation at i^{th} floor (Δ_{ax} in Fig. 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 Fig. 8. Line segments on the idealized force-displacement curve have been located using an iterative procedure that approximately balanced the area above and below the curve. The nominal story stiffness $(k_t)_i$ is then taken as the secant stiffness calculated at a story shear force equal to 60% of the effective yield strength of the story [19, 20].

Using Equation (3), shear story drift corresponding to each step of pushover analysis can be calculated and consequently $(k_{sh})_i$ and $(\alpha_{sh})_i$ are determined. As the transmitted force is equal in two supplementary springs, Equation (2) can be rewritten as:

For $V_i \leq S_i$,

$$\frac{V_i}{(k_t)_i} = \frac{V_i}{(k_{sh})_i} + \frac{V_i}{(k_{ax})_i} \ . \tag{4}$$

Hence,

$$\frac{1}{(k_t)_i} = \frac{1}{(k_{sh})_i} + \frac{1}{(k_{ax})_i} . \tag{5}$$

For $V_i > S_i$ we have

$$\frac{S_i}{(k_t)_i} + \frac{V_i - S_i}{(\alpha_t)_i (k_t)_i} = \frac{S_i}{(k_{sh})_i} + \frac{V_i - S_i}{(\alpha_{sh})_i (k_{sh})_i} + \frac{S_i}{(k_{ax})_i} + \frac{V_i - S_i}{(\alpha_{ax})_i (k_{ax})_i} . \tag{6}$$

Substituting Equation (5) in (6), $(k_{ax})_i$ and $(\alpha_{ax})_i$ are obtained as follows:

$$(k_{ax})_i = \frac{(k_{sh})_i (k_t)_i}{(k_{sh})_i - (k_t)_i} \ . \tag{7}$$

$$(\alpha_{ax})_i = \frac{(\alpha_{sh})_i (\alpha_t)_i \left[(k_{sh})_i - (k_t)_i \right]}{(\alpha_{sh})_i (k_{sh})_i - (\alpha_t)_i (k_t)_i} \ . \tag{8}$$

Calculations show that $(\alpha_{ax})_i$ is almost equal to 1 when columns are designed to prevent buckling against earthquake loads, thus implying that the spring which represents the axial deformation always remains in the elastic deformation range. As will be described in the sequel, for each frame model, all the required parameters of the modified shear-building can be determined by performing only one pushover analysis. By considering P-Δ effects in this pushover analysis, the modified model will be capable to account for P-Δ effects as well.

The shear inter-story drift, which causes damage to the structure, can be separated from the flexural deformation by using the modified shear-building model. The modified shear-building model takes into account both the higher mode contribution to (elastic) structural response as well as the effects of material non-linearity; therefore, it represents the behavior of frame models more realistically as compared to the conventional shear-building model.

To investigate the reliability of the proposed modified model in estimating 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. It is shown in Fig.6 that the modified model is capable to estimate the nonlinear seismic response of the 10 story concentrically braced frame more accurately compare to the conventional shear-building model.

Average of the displacement demands for 5, 10 and 15 story frame models and their corresponding modified shear building models are compared in Fig. 9. This Figure indicates that on average, modified shear-building models are capable to predict story displacement, total inter-story drift and shear inter-story drift of concentrically braced frames very accurately.

For each synthetic excitation, the errors in prediction of displacement demands between the modified shear-building model analysis and the original frame are determined. Consequently, the average of these errors is calculated for every story. Maximum errors corresponding to 5, 10 and 15 story frames are summarized in Table 1. It is shown that maximum errors associated with the modified shear building model are significantly less than the corresponding values for the conventional shear-building model, particularly for story drifts where the errors are almost one third of those estimated by conventional models. The errors are slightly larger for prediction of drift than for estimation of displacement. However, for modified shear building models, the maximum errors in all response quantities are only a few percent (less than 16%).

Based on the above discussion, displacement demands estimated by modified shear-building models proved to be good representatives of those obtained based on typical non-linear frame models of the same structure. Next, it is investigated how the errors in displacement demands obtained by modified shear-building models vary with the deformation demands imposed by the ground motion and in particular with the degree to which the system deforms beyond its elastic limit. For this purpose, displacement demands for the 10-story frame model and its corresponding modified shear-building model are obtained for ground motions of different intensity. These excitations are scaled El Centro 1940 ground motions with scaling coefficients 0.15, 0.25, 0.5, 0.75, 1.0, 1.5, 2.0, 2.5, and 3.0. For each excitation, the errors in response quantities obtained by the modified shear-building model compared to the corresponding original frame response quantities are determined. Fig. 10 summarizes the maximum errors in displacement demands estimated by modified shear-building models as a function of ground motion intensity, indicated by the ground motion scale coefficient, and maximum story ductility. One can observe from this figure that these errors are larger in story drifts compared to story displacements; however, maximum errors are less than 20% even for very intense ground motions. This is further illustrated in Fig. 10 that the errors are almost independent to the ground motion intensity and maximum story ductility. Therefore, it can be concluded that the modified shear-building model estimates the seismic response of buildings experienced high inelastic deformations (i.e. story ductility more than 10) with the same degree of accuracy as it predicts the response of elastic systems. The same observations have been made with other models and under different ground motions.

As mentioned before, the behavior of modified shear building model is idealized by a bilinear force-displacement curve. For the concentrically braced frames, the nominal story stiffness in the equivalent modified shear building model is very close to the initial tangent stiffness of the typical full-frame model. Therefore, modified shear building model has a good capability to estimate the natural periods of the corresponding full-frame model. The close prediction of the natural periods in full frame models and their corresponding modified shear building models for 5, 10 and 15 story braced frames are illustrated in Table 2. It is shown that using modified shear-building model, the period of the first three vibration modes agree very well with the natural periods of the full-frame model. This is particularly true for the fundamental period (1st mode) where the predicted values are almost identical with the actual values.

Total computational time for 5, 10 and 15 story braced frames and their corresponding modified shear-building model under 15 synthetic earthquakes are compared in Table 2. As it is illustrated, the relatively small number of degrees of freedom for modified shear-building model results in significant computational savings, while maintaining the accuracy, as compared to the corresponding frame model. According to the results, total computational time for modified shear-building models are less than 4% of those based on typical frame models.

6- Cumulative damage

The peak shear story drift may not always be the best performance criterion for performance base design as it occasionally fails in predicting the state of structural damage in earthquakes. To investigate the extent of cumulative damage, the damage criterion proposed by Baik et al. [26] based on the classical low-cycle fatigue approach has been adopted. The story inelastic shear deformation is chosen as the basic damage quantity, and the cumulative damage index after N excursions of plastic deformation is calculated as:

$$D_i = \sum_{j=1}^{N} \left(\frac{\Delta\delta_{pj}}{\delta_{yi}} \right)^c \cdot \tag{9}$$

Where D_i is the cumulative damage index at ith story, ranging from 0 for undamaged to 1 for severely damaged stories, $\Delta\delta_{pj}$ is the plastic deformation of ith story in jth excursion, δ_{yi} is the nominal yield deformation, and c is a parameter that accounts for the effect of magnitude of plastic deformation taken to be 1.5 [27]. To assess the damage experienced by the whole structure, the global damage index is obtained as a weighted average of the damage indices at the story levels, with the energy dissipated being the weighting function given by:

$$D_g = \frac{\sum_{i=1}^{n} D_i W_{pi}}{\sum_{i=1}^{n} W_{pi}}, \tag{10}$$

where D_g is the global damage index, W_{pi} is the energy dissipated at ith story, D_i is the damage index at ith story, and n is the number of stories.

Using this equation, the global damage index has been calculated for 5, 10 and 15 story concentrically braced frames and their corresponding modified shear-building models subjected to 15 synthetic earthquakes. As an example, the global damage index of 10-story frame obtained by modified shear-building model is compared with those obtained by full-frame model in Fig. 11. The results suggest that, from low level (less than 20%) to thigh level (more than 70%) of damage intensity, the global damage experienced by the concentrically braced frames can be estimated utilizing modified shear-building models up to an acceptable accuracy for practical applications.

Estimation of peak inelastic deformation demands is a key component of any performance-based procedure for earthquake-resistant design of new structures or for seismic performance evaluation of existing structures. The modified shear building models proved to be capable to account for contribution of several modes of vibration, P-Δ effects and characteristics of the ground motions. Therefore, evaluating the deformation demands and cumulative damages using modified shear-building models is demonstrated to be reasonably close to those of the full-frame models. This makes it an appropriate model to be utilized in seismic performance-based design softwares. In practical applications, due to significantly low computational efforts associated with the proposed modified shear-building model, one can possibly consider more design alternatives and earthquake ground motions as opposed to designs based on the full-frame model. Therefore, the modified shear-building model can be efficiently used for optimum seismic design of structures where many nonlinear dynamic analyses would be required to get to the optimum solution [25].

7- Conclusions

1. It is shown that, in general, conventional shear building models provide accurate estimates of maximum roof and story displacements of concentrically braced frames; but are not able to provide good estimates of inter-story drifts. While the maximum errors in the estimation of maximum roof and story displacements are usually less than

15%, they are particularly large for the maximum drift at top stories where the estimated drift could be more than 40% higher than the actual value.

2. The conventional shear-building model has been modified by introducing supplementary springs to account for flexural displacements in addition to shear drifts. It is shown that the accuracy of modified shear building models to predict story displacements and peak inter-story drifts is significantly higher than conventional models.
3. It is shown that the modified shear-building model is not sensitive to the ground motion intensity and maximum story ductility; and therefore, could be utilized to estimates the seismic response of concentrically braced frames from elastic to highly inelastic range of behaviour. The results indicate that the proposed model is also capable to estimate the global damage experienced by the concentrically braced frames from low (less than 20%) to high (more than 70%) level of damage intensity.

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List of symbols

The following symbols are used in this paper:

$(\alpha_{ax})_i$ = Over-strength factors for bending story stiffness at i^{th} floor

$(\alpha_{sh})_i$ = Over-strength factors for shear story stiffness at i^{th} floor

$(\alpha_t)_i$ = Over-strength factors for nominal story stiffness at i^{th} floor

$\Delta\delta_{pj}$ = Plastic deformation of i^{th} story in j^{th} excursion

δ_{yi} = Nominal yield deformation of i^{th} story

Δ_t = Total inter-story drift

Δ_{sh} = Shear inter-story drift

Δ_{ax} = Flexural inter-story drift

C_{vx}= Vertical distribution factor for lateral loads

c = Parameter that accounts for the effect of magnitude of plastic deformation

D_g = Global damage index

D_i = Cumulative damage index at i^{th} story

H = Height of the story

h_i = Height of i^{th} story

k = Positive number as a power

$(k_t)_i$ = Nominal story stiffness of i^{th} story

$(k_{ax})_i$ = Bending story stiffness of i^{th} story

$(k_{sh})_i$ = Shear story stiffness of i^{th} story

L = Span length

N = Number of plastic excursions

n = Number of stories

S_i = Shear yield strength of i^{th} story

V_i = Total shear force of i^{th} story

U_1 = Horizontal displacement at the bottom line of the panel

U_2, U_3 = Vertical displacements at the bottom line of the panel

U_4 = Horizontal displacement at the top line of the panel

U_5, U_6 = Vertical displacements at the top line of the panel

w_i = Weight of i^{th} story

W_{pi} = Energy dissipated at i^{th} story

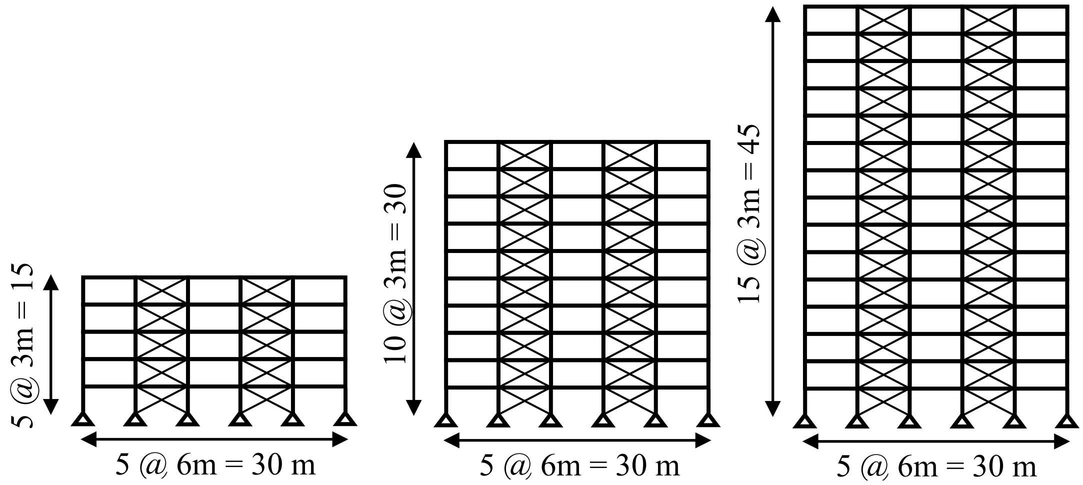

Fig. 1. Typical geometry of concentric braced frames

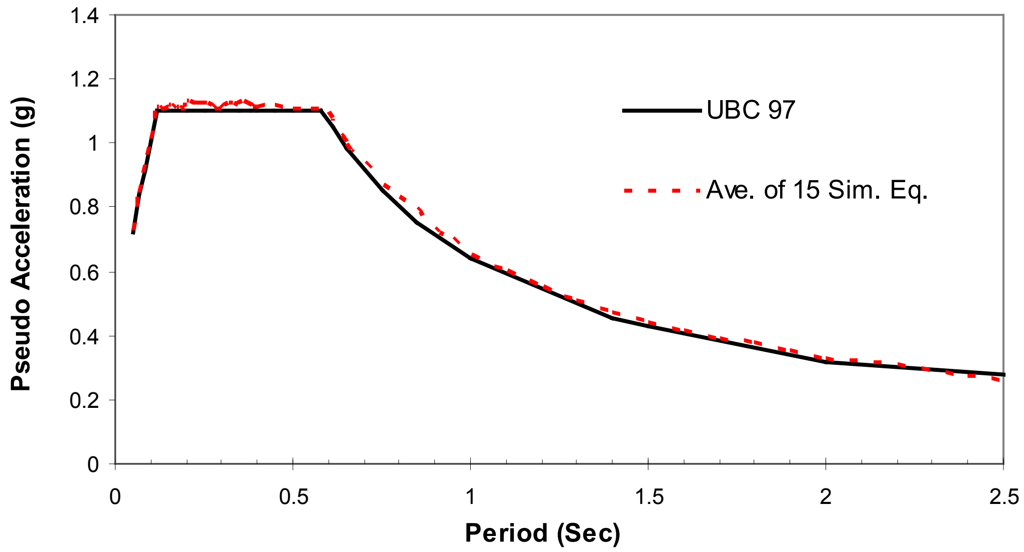

Fig. 2. UBC design spectrum and average response spectra of 15 synthetic earthquakes (5% damping)

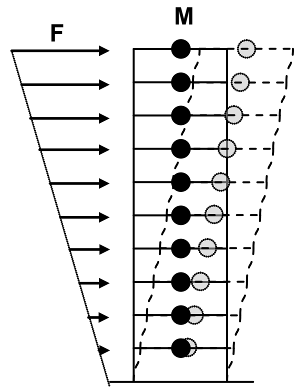

Fig. 3. Conventional shear-building model

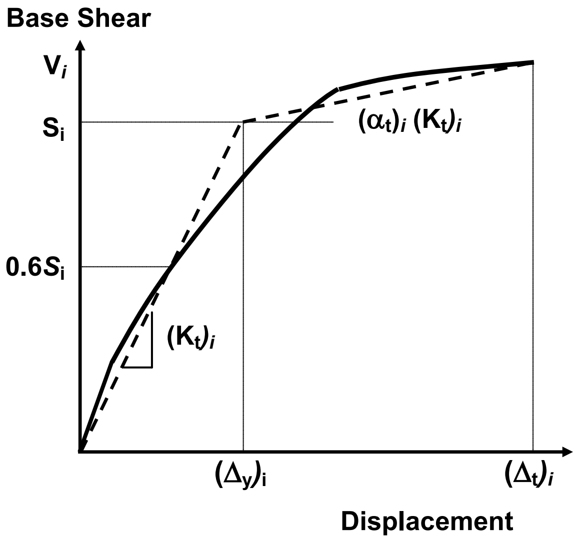

Fig. 4. Idealized force-displacement curves

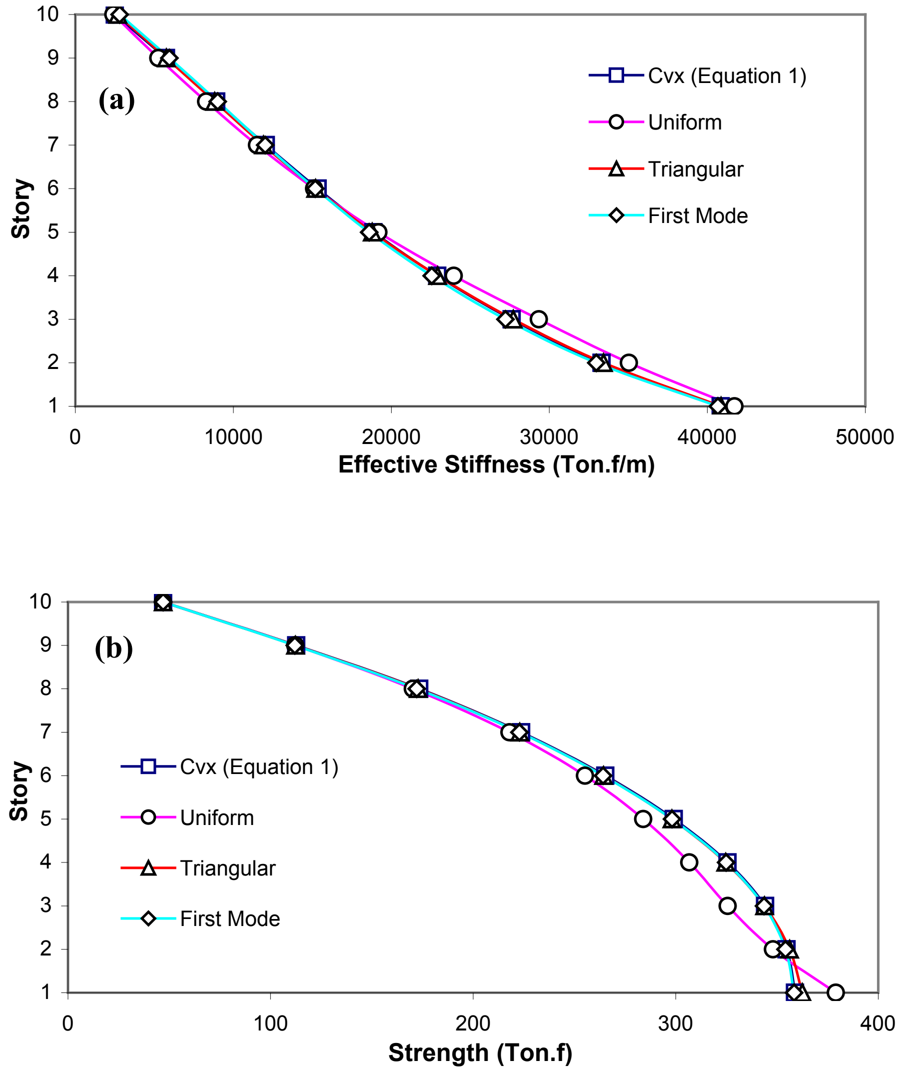

Fig. 5. The effect of vertical distribution of lateral loads on computed mechanical properties; (a) Story stiffness, (b) Story strength

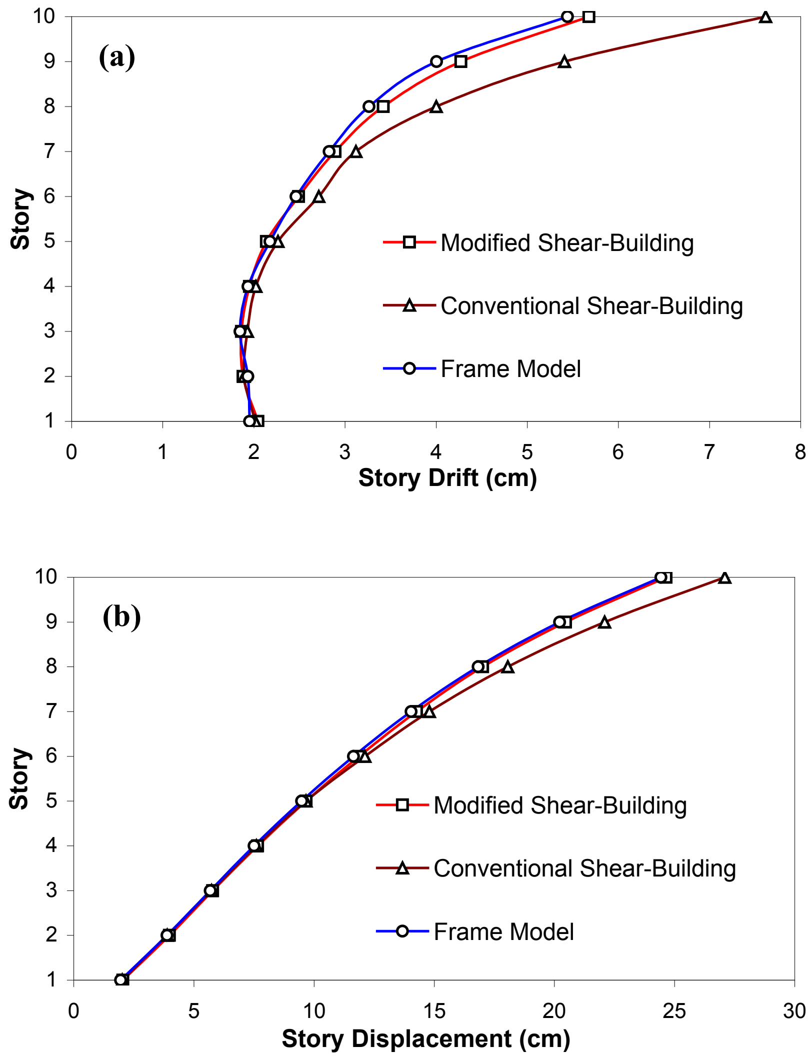

Fig. 6. Comparison of frame model, conventional shear-building model and modified shear-building model for 10-story braced frame, Average of 15 synthetic earthquakes; (a) Story drift, (b) Story displacement

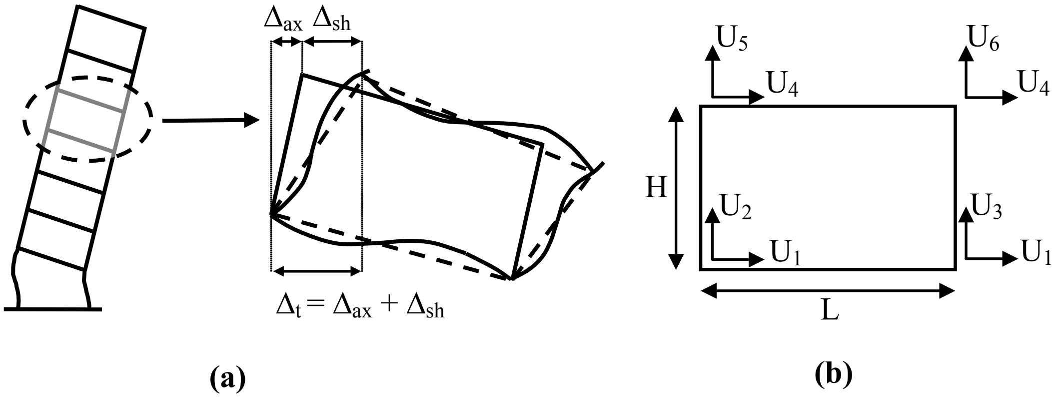

Fig. 7. (a) Definitions of total inter-story drift (Δ_t), shear inter-story drift (Δ_{sh}) and the effect of axial flexibility of columns (Δ_{ax}), (b) Displacement components of a single panel.

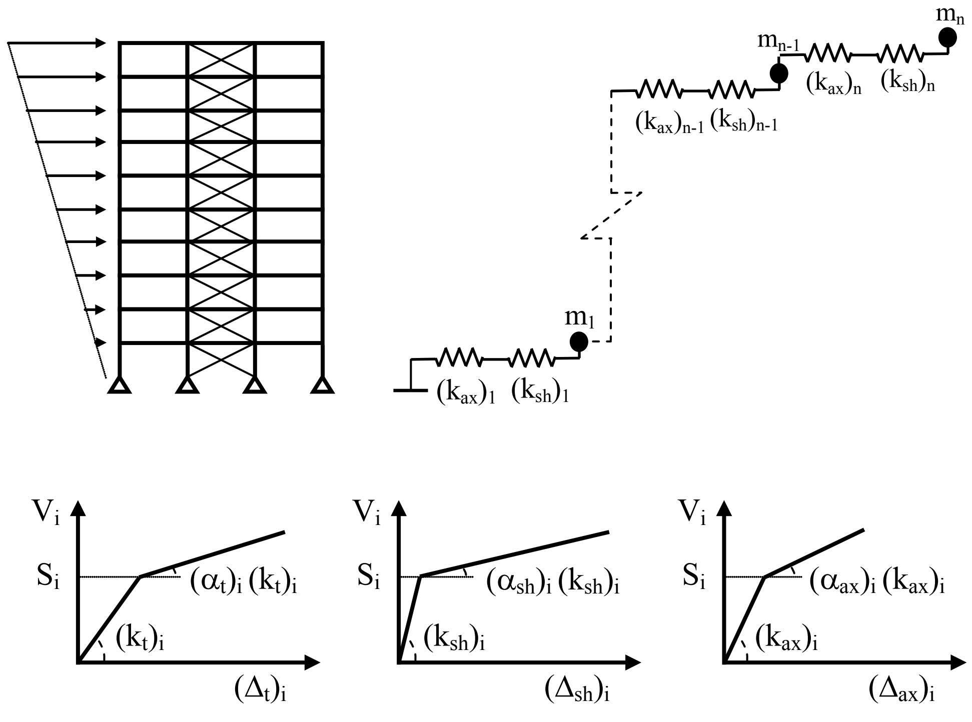

Fig. 8. Using push over analysis to define equivalent modified shear-building model

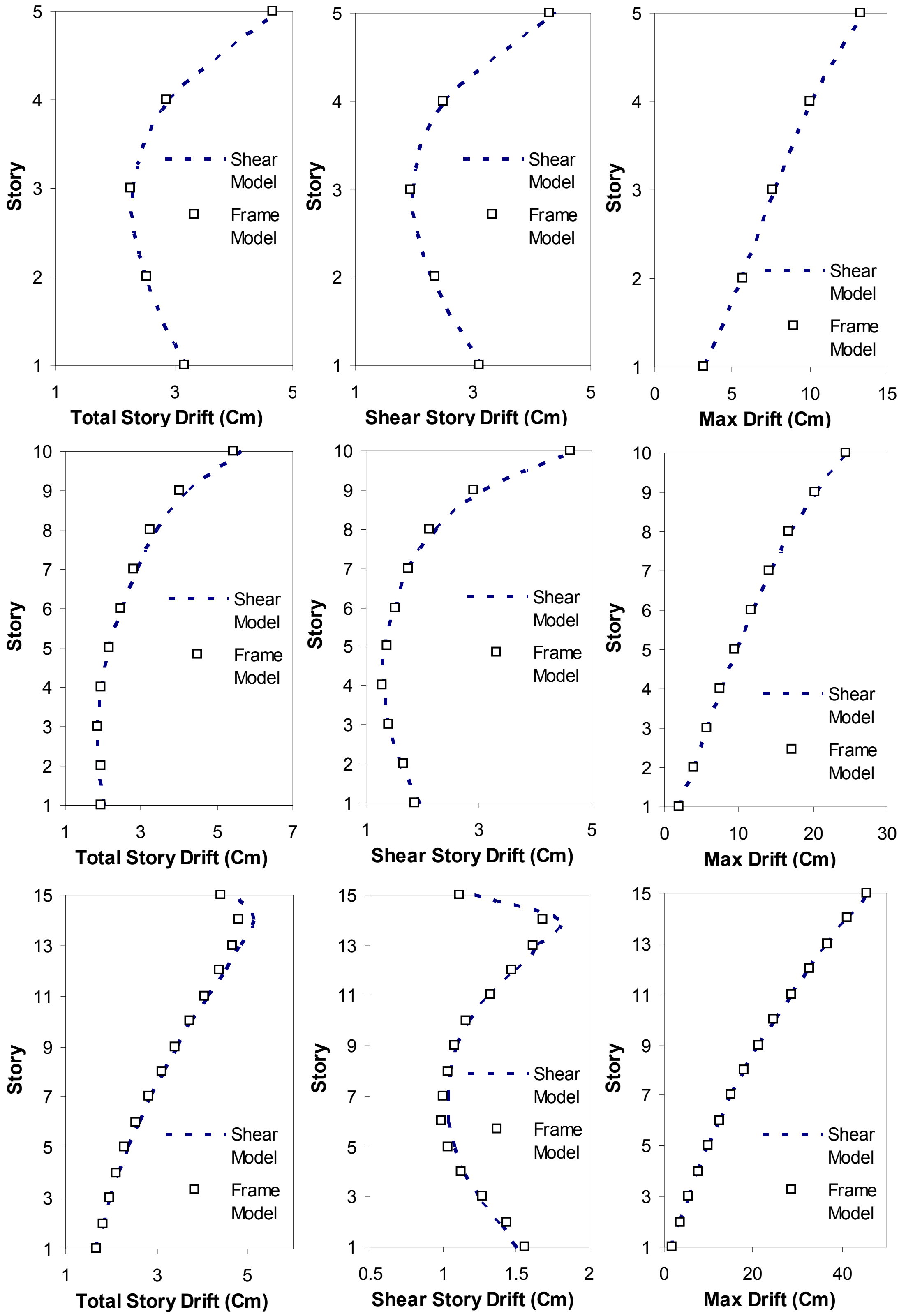

Fig. 9. Comparison of the full-frame model and the corresponding modified shear-building model for 5, 10 and 15-story braced frames, Average of 15 synthetic earthquakes

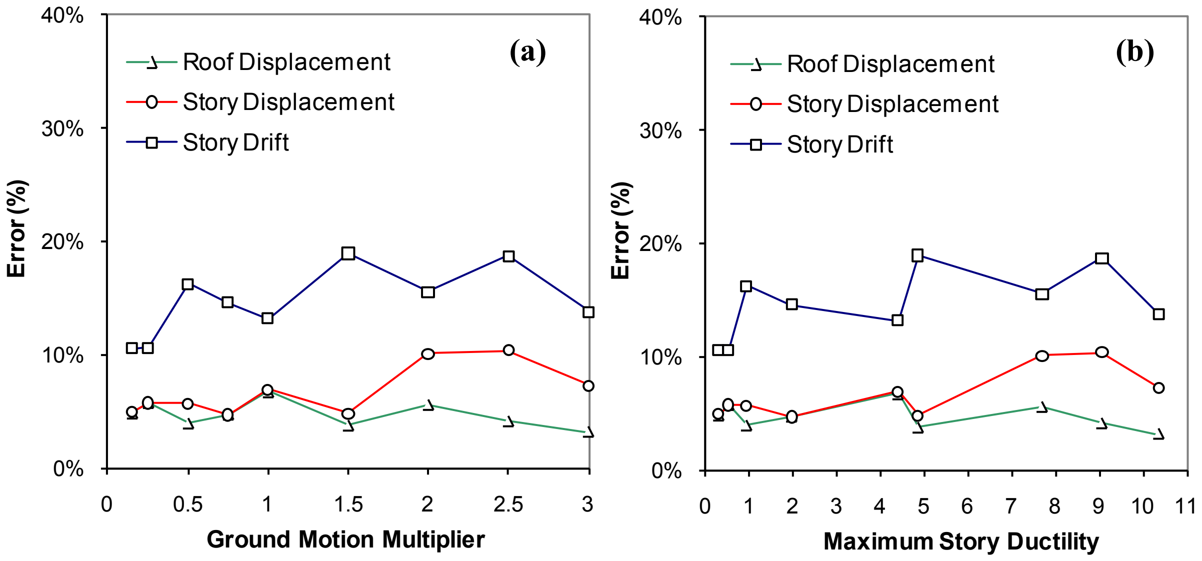

Fig. 10. Errors in displacement demands obtained by modified shear-building models as a function of (a) ground motion intensity; (b) maximum story ductility, 10-story model subjected to El Centro 1940

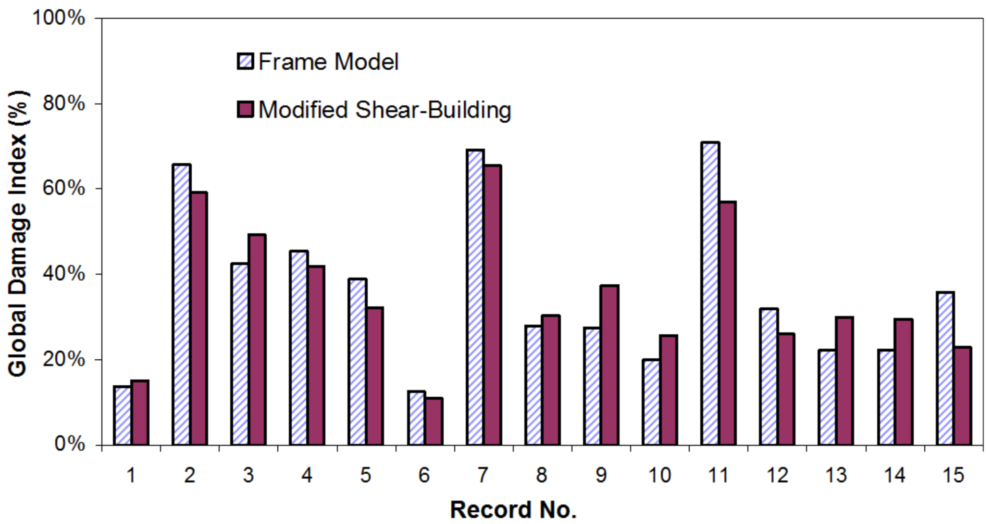

Fig. 11. Comparison of the global damage index of 10-story frame obtained by modified shear-building model and full-frame model subjected to 15 synthetic earthquakes

Table 1. Maximum errors in estimated displacement demands using conventional and modified shear-building models, Average of 15 synthetic earthquakes

		Max error in roof displacement (%)	Max error in story displacement (%)	Max error in story drift (%)
5-Story	Conventional Model	7.5%	7.5%	20.4%
	Modified Model	3.3%	4.1%	8.4%
10-Story	Conventional Model	12.0%	15.6%	45.9%
	Modified Model	6.9%	9.6%	16.1%
15-Story	Conventional Model	6.3%	15.1%	38.6%
	Modified Model	3.9%	7.8%	11.3%

Table 2. Natural periods and total computational time for full-frame model and the corresponding modified shear-building model

		5-Story		10-Story		15-Story	
		Frame Model	Modified Shear-Building	Frame Model	Modified Shear-Building	Frame Model	Modified Shear-Building
Period (sec)	1st Mode	0.62	0.62	1.11	1.11	1.77	1.77
	2nd Mode	0.25	0.27	0.41	0.46	0.66	0.71
	3rd Mode	0.15	0.17	0.23	0.28	0.41	0.45
Total Computational Time (sec)		906	36	1616	52	4915	68