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Optimum seismic design of concentrically braced steel frames: Concepts and design procedures

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ABSTRACT:

A methodology is presented for optimization of dynamic response of concentrically braced steel frames subjected to seismic excitation, based on the concept of uniform distribution of deformation. In order to obtain the optimum distribution of structural properties, an iterative optimization procedure has been adopted. In this approach, the structural properties are modified so that inefficient material is gradually shifted from strong to weak areas of a structure. This process is continued until a state of uniform deformation is achieved. It is shown that the seismic performance of such a structure is optimal, and behaves generally better than those designed by conventional methods. In order to prevent the cumbersome analysis of the frame models, an equivalent procedure is introduced to perform the optimization procedure on the modified reduced shear-building model of the frames, which is shown to be accurate enough for design purposes.

Keywords: optimal strength pattern, performance-based design, braced frames, seismic loading

Introduction

The preliminary design of most buildings is normally based on equivalent static forces specified by the governing building code. The height wise distribution of these static forces (and therefore, stiffness and strength) seems to be based implicitly on the elastic vibration modes [1]. However, structures do not remain elastic during severe earthquakes and they are expected to undergo large nonlinear deformations. Therefore, the employment of such arbitrary height wise distribution of seismic forces may not lead to the optimum utilization of structural materials. Many experimental and analytical studies have been carried out to investigate the validity of the distribution of lateral forces according to seismic codes. Lee and Goel [2] analyzed a series of 2 to 20 story frame models subjected to various earthquake excitations. They showed that in general there is a discrepancy between the earthquake induced shear forces and the forces determined by assuming distribution patterns. The consequences of using the code patterns on seismic performance have been investigated during the last decade [3, 4]. Chopra [5] evaluated the ductility demands of several shear-building models subjected to the El- Centro Earthquake of 1940. The relative story yield strength of these models was chosen in accordance with the distribution patterns of the earthquake forces specified in the Uniform Building Code [6]. It was concluded that this distribution pattern does not lead to equal ductility demand in all stories, and that in most cases the ductility demand in the first story is the largest of all stories. The first author [7, 8] proportioned the relative story yield strength of a number of shear building models in accordance with some arbitrarily chosen distribution patterns as well as the distribution pattern suggested by UBC1997 [6]. It has been concluded that: (a) the pattern suggested by code guidelines does not lead to a uniform distribution of ductility, and (b) a rather uniform distribution of ductility with a relatively smaller maximum ductility demand can be obtained from other patterns. These findings have been confirmed by further investigations [9- 11], and led to the development of a new concept: optimum distribution pattern for seismic performance that is discussed in this paper.

Concept of Optimum Distribution Pattern for seismic performance

As discussed before, the use of distribution patterns for lateral seismic forces suggested by codes does not guarantee the optimum performance of structures. Current studies indicate that during strong earthquakes the deformation demand in structures does not vary uniformly [1- 4]. Therefore, it can be concluded that in some parts of the structure, the deformation demand does not reach the allowable level of seismic capacity, and therefore, the material is not fully exploited. If the strength of these strong parts decreases, the deformation would be expected to increase [12]. Hence, if the strength decreases incrementally, we should eventually obtain a status of uniform deformation. At this point the material capacity is fully exploited. As the decrease of strength is normally obtained by the decrease of material, a structure becomes relatively lighter as deformation is distributed more uniformly. Therefore, in general it can be concluded that a status of uniform deformation is a direct consequence of the optimum use of material. This is considered as the *Theory of Uniform Deformations* [10, 11]. This theory is the basis of the studies presented in this paper.

Shear and Flexural Deformation

Recent design guidelines, such as FEMA 356 [13] and SEAOC Vision 2000 [14], 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 1-a, the axial deformation of the columns results in increased 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:

$$\Delta_t = \Delta_{sh} + \Delta_{ax} \tag{1}$$

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, rotation at the top and bottom levels of the panel shown in Figure 1-b are given by:

$$\theta_1 = \frac{U_5 - U_6}{L} \tag{2}$$

$$\theta_2 = \frac{U_2 - U_3}{L} \tag{3}$$

where, U_5, U_6, U_2 *and* U_3 are vertical displacements, as shown in Figure 1-b. H is the story height, and L is the span length. The rotation of the panel, α, could be approximated by averaging θ_1 and θ_2 as follows:

$$\alpha \approx \frac{\theta_1 + \theta_2}{2} = \frac{(U_2 + U_5 - U_3 - U_6)}{2L} \tag{4}$$

Hence, as indicated in Figure 1-b, the flexural deformation Δ_{ax} is calculated as:

$$\Delta_{ax} = \alpha \times H = \frac{H}{2L}(U_2 + U_5 - U_3 - U_6) \tag{5}$$

Considering equations (5) and (1), the shear inter-story drift can be determined as follows [15]:

$$\Delta_{sh} = \Delta_t + \frac{H}{2L}(U_3 + U_6 - U_2 - U_5) \tag{6}$$

For multi-span models, the maximum value of the shear drift in different panels would be considered as the shear story drift.

Modeling and Assumptions

In the present study, three steel concentric braced frames, as shown in Figure 2, with 5, 10 and 15 stories have been selected. The buildings are assumed to be located on a soil type S_D and a seismically active area, zone 4 of the UBC 1997 [6] 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 [6]. In all models, the top story is 25% lighter than the others. IPB, IPE and UNP sections, according to DIN, are chosen for columns, beams and bracings, respectively. To eliminate the over strength effect, conceptual auxiliary sections have been 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 brace members are assumed to have an elasto-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 equal to 0.28 times the nominal compressive resistance as suggested by Jain et al [17].

Four strong ground motion records were used to evaluate and compare the seismic performance of the frames, (1) The 1994 Northridge earthquake NWH360 component with a PGA of 0.59g, (2) The 1979 Imperial Valley earthquake H-E06230 component with a PGA of 0.44g, (3) The 1992 Cape Mendocino earthquake PET090 component with a PGA of 0.66g, and (4) A synthetic earthquake record generated to have a target spectrum close to that of the UBC 1997 [6] code with a PGA of 0.44g. All of these excitations correspond to the sites of soil profiles similar to the design site soil, S_D, of UBC.

Optimum Design of Bracings for Seismic Excitations

The theory of uniform deformation can be employed for evaluation of optimum strength distribution in concentrically braced frames. As an example, a 15-story model designed in accordance to UBC code [6], as shown in Figure 2, is considered to be laterally loaded for the Northridge earthquake 1994 (NWH360). The question is how to proportion and arrange the bracings to minimize the maximum shear story drift. The following conditions are also stipulated:

a) The cross sections of beams and columns are not regarded as variables in the optimization procedures, and therefore they remain unchanged.

b) All columns are checked for stability under the combination of gravitational loads and the dynamic seismic forces (resulting from seismic excitation) according to ASD [18], and are resized if necessary to meet:

$$\frac{f_a}{F_a}+\frac{c_{mx}f_{bx}}{(1-f_a/F'_{ex})F_{bx}}\leq 1 \tag{7}$$

where f_a and f_{bx} are the calculated axial and bending stresses; F_a and F_{bx} are the allowable compressional and bending stresses, respectively. c_{mx} is a coefficient dependent on the column end moments, and F'_{ex} is the elastic buckling stress divided by a factor of safety as given in [18].

To obtain the optimum distribution of bracings, the evolutionary optimization method is adapted as follows:

1. The model already designed for gravitational and any arbitrary lateral load pattern, that of UBC97 [6] here, is regarded as a primary pattern for distribution of structural properties. Here, the cross section area of bracings is assumed to be the only key parameter controlling the structural seismic behavior. However, as mentioned before, the columns have to be checked for stability. This is indeed a stipulating condition for the optimization program.

2. The structure is subjected to the given excitation, the peak values of shear story drifts, $(\Delta_{sh})_i$, and the average of those values, Δ_{avg} , are determined. Consequently, the *COV*, coefficient of variation, of shear story drifts is calculated. If *COV* is small enough, distribution of bracing strength in each story can be considered as practically optimum. The *COV* of the first pattern is determined as 0.42. It is decided that the *COV* is high, and the analysis should be continued.

3. At this step the distribution of bracing cross section areas, as a parameter monotonically proportion to the shear strength of each story and hence to the total strength of the story, is modified. Using the theory of uniform deformations, the inefficient material should be shifted from strong parts to the weak parts to obtain an optimum structure. To accomplish this, the cross section of bracings should be increased in the stories with peak shear story drift greater than the average of peak drifts, Δ_{avg}, and should be decreased in the stories where peak shear drift is less than the average. The total cross section areas of the all bracings in the frame is kept unchanged in order for the structural weight of the frame to be constant. This alteration should be applied incrementally to obtain convergence in numerical calculations. Hence, the following equation was used in the present work

$$[(A_b)_i]_{n+1} = [(A_b)_i]_n \left[\frac{(\Delta_{sh})_i}{\Delta_{avg}} \right]^{\alpha} \tag{8}$$

where $(A_b)_i$ is the total cross section area of bracings at i^{th} story, n denotes the step number. α is the convergence coefficient ranging from 0 to 1. For the above example, an acceptable convergence has been obtained for a value of α equals 0.2. Consequently, cross section areas of the bracings are scaled so that the total structural weight remains constant. Using these modified cross sections; the procedure is repeated from step 2. It is expected that the *COV* of peak shear story drifts for this pattern is smaller than the corresponding *COV* for the previous pattern. This procedure is iterated until *COV* becomes small enough, and a state of rather uniform shear story drift prevails.

Figure 3, illustrates the evolution of shear story drift distribution from the UBC 97 [6] model toward the final optimum distribution. As it is shown in this Figure, peak shear story drifts in the final step have become remarkably uniform and the maximum peak shear story drift has been decreased from 4.7 *cm* to 2.4 *cm*.

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 simplicity and low computational expenses [19], which might be considered as a great advantage for a design engineer to deal with. Lai et al. [20] have investigated the reliability and accuracy of such shear-beam models. In the present study, the shear-building model has been modified to have a better estimation for the nonlinear dynamic response of real framed structures.

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 4. The stiffnesses of these 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 frame model. As shown in Figure 4, the material nonlinearities may be incorporated into stiffness and strength of supplementary springs. In Figure 4, 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 1). $(k_{sh})_i$ denotes the shear story stiffness corresponding to the relative shear drift at i^{th} floor (Δ_{sh} in Figure 1). $(k_{ax})_i$ represents the bending story stiffness corresponding to the flexural deformation at i^{th} floor (Δ_{ax} in Figure 1), 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. Using equation (6), shear story drift corresponds to each step of previous push over analysis could be calculated and consequently $(k_{sh})_i$ and $(\alpha_{sh})_i$ are determined. As transmitted force is equal in two supplementary springs, equation (1) could be rewritten as:

For $V_i \leq S_i$ we have

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

hence

$$\frac{1}{(k_t)_i} = \frac{1}{(k_{sh})_i} + \frac{1}{(k_{ax})_i} \tag{9}$$

For $V_i > S_i$ we have

$$\frac{S_i}{(k_t)_i} + \frac{V_i - S_i}{(\alpha_t)_i (k_t)_i} = \frac{S_i}{(k_{sh})_i} + \frac{V_i - S_i}{(\alpha_{sh})_i (k_{sh})_i} + \frac{S_i}{(k_{ax})_i} + \frac{V_i - S_i}{(\alpha_{ax})_i (k_{ax})_i} \tag{10}$$

Substituting Equation (9) in (10), $(k_{ax})_i$ and $(\alpha_{ax})_i$ are obtained as follows:

$$(k_{ax})_i = \frac{(k_{sh})_i (k_t)_i}{(k_{sh})_i - (k_t)_i} \tag{11}$$

$$(\alpha_{ax})_i = \frac{(\alpha_{sh})_i (\alpha_t)_i \left[(k_{sh})_i - (k_t)_i\right]}{(\alpha_{sh})_i (k_{sh})_i - (\alpha_t)_i (k_t)_i} \tag{12}$$

Calculations show that $(\alpha_{ax})_i$ is almost equal to 1 when columns are designed to prevent buckling against earthquake loads. 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. Moreover, this modified model represents the behavior of frame models more realistically as compared with the ordinary shear-building model. Figure 5, illustrates the response of 15 story frame model and its corresponding modified shear-building model under Imperial Valley 1979. 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.

Stiffness and Strength Relationship

In the absence of over-strength, a specific relation exists between stiffness and strength of a story. This relation depends on the type of structural members, and the frame geometry, and can be simply determined by using a pushover analysis. Numerous analyses were conducted using the frames designed for different seismic load patterns. These analyses show that for each story, the ratios $(k_{ax})_i/(k_{sh})_i$ and $S_i/(k_{sh})_i$ are not dependant on the type of strength distribution pattern (Figure 6), hence:

$$\begin{aligned} (k_{ax})_i &= a_i \cdot (k_{sh})_i \\ S_i &= b_i \cdot (k_{sh})_i \end{aligned} \tag{13}$$

where a_i and b_i are constant multipliers and S_i is the shear strength of the i^{th} story, respectively. These parameters depend on the type of structural members as well as the frame geometry, and can be simply determined by using a pushover analysis. These observations are fundamental and similar assumptions about the stability of the member yield displacement have been adopted by others [21, 22].

Optimum Seismic Design using Modified Shear Building Model

As described in previous sections, the theory of uniform deformation can be employed directly to evaluate optimum lateral loading patterns for braced frames. However, nonlinear dynamic analysis of frame models needs a great deal of computational effort, and therefore, it would be desirable to employ the shear building model for such analysis. This can be accomplished by using the aforementioned modified shear-building model. The procedure is as follows:

1. An arbitrary lateral load pattern (such as that of UBC 97 [6]) is chosen and used for design of structure.

2. Bilinear spring parameters and constant multipliers of equation (13) are determined for each story by conducting a pushover analysis on the designed frame, as discussed in previous sections. The corresponding modified shear- building model is defined accordingly.

3. Nonlinear time history analysis under the design earthquake is carried out on the modified shear- building model. Arbitrary values for strengths, shear and flexural stiffness satisfying equation (13) are considered in analysis. The average and peak values of shear story drifts, (Δ_{avg}) and ($\Delta_{sh})_i$, are determined, and the corresponding coefficient of variation (*COV*) is calculated. The procedure continues until *COV* decreases down to an acceptable level.

4. According to the theory of uniform deformation, the shear strength, shear stiffness, and flexural stiffness of the stories with shear story drifts greater than the average drift, (Δ_{avg}), should be increased proportionally. On the contrary, these parameters should decrease where the drifts are less than average. As a result, a rather uniform distribution of story drifts prevails. The following relationship has been employed to modify the strength parameters:

$$[(k_{sh})_i]_{n+1} = [(k_{sh})_i]_n \left[\frac{(\Delta_{sh})_i}{\Delta_{avg}} \right]^{\alpha} \tag{14}$$

where, α is the convergence coefficient chosen as equal to 0.2 in this work. After modifying the story shear stiffness, for each story, the flexural stiffness and strength are modified according to equation (13). In order to keep the weight of the model constant, the parameters $[(k_{sh})_i]_{n+1}$ and $[(k_{ax})_i]_{n+1}$ are scaled so that the dominant period of the structure remains unchanged. The procedure continues until the *COV* of peak shear story drifts decreases down to a target value. At this stage, the strength distribution is regarded as the optimum.

5. Considering the safety factors incorporated in the design of lateral resistant system of the frame, the optimum lateral load can be calculated from the foregoing optimum strength pattern. Now, the constant multipliers of equation (13) are recalculated, and the procedure is repeated. However, the present study shows that these multipliers are nearly constant for each story, and the optimum solution is not sensitive to small variation of those multipliers.

Figure 7 illustrates the steps of this approach from the UBC 97 [6] designed model toward the final design for a 10-story building subjected to the Imperial Valley 1979. The convergence efficiency of the proposed method to the optimum design is emphasized in Figure 7. It is shown in this Figure, having the same structural weight, maximum shear story drift is reduced almost 50% after only five steps. Figure 7 shows that reduction *COV* is always accompanied with reduction of maximum shear story drift. These results are in agreement with the *Theory of Uniform Deformation*.

As mentioned before, by using modified shear-building model, optimization procedure can be adapted on simple nonlinear spring elements and there is no need to perform any nonlinear dynamic analysis on a full frame models. In Figure 8, final results of two proposed methods are compared with UBC 97 [6] design for 15-story braced frame subjected to Northridge earthquake 1994. As it is shown in this Figure, using modified shear-building model is both simple and accurate enough for design purposes. According to these results, the procedure introduced in this paper seems to be a practical alternative to current design procedures for steel braced frames.

Optimum Seismic Design Load Pattern

The foregoing procedure has been used for optimum design of 5, 10 and 15-story braced frames, shown in Figure 2, subjected to different strong ground motions. The results indicate that optimum structures suffer relatively less damage as compared with structures designed for conventional seismic loadings. Figure 9, shows the lateral seismic design loads for the fifteen-story conventionally designed and that of the optimum model under the Northridge 1994 earthquake. The results indicate that to improve the performance under this specific earthquake, the frame should be designed in compliance with a new load pattern different from the conventional UBC pattern.

Effect of the Initial Pattern on the Optimum Load Pattern

As described before, an initial height wise strength distribution is necessary to begin the optimization algorithm. In order to investigate the effect of this initial strength distribution pattern on the final optimum load pattern, the design base shear was distributed as follows; (1) A concentrated load on the roof level, (2) Triangular distribution according to UBC 97 [6], (3) Rectangular distribution over the height of the frame, (4) An inverted triangular distribution with the maximum lateral load on the first floor and the minimum lateral load on the roof floor. For each case, the optimum lateral load pattern was derived for the Imperial Valley earthquake 1979. The comparison of the optimum lateral load pattern of each case is depicted in Figure 10. As shown in this Figure, the optimum load pattern is unique and does not depend on the initial strength pattern; however, the speed of convergence is to some extent dependant on the initial strength pattern. This conclusion has been confirmed by further analyses on different models and ground motions.

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. [23] 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:

$$D_i = \sum_{j=1}^{N} \left(\frac{\Delta\delta_{pj}}{\delta_y} \right)^c \tag{15}$$

where D_i is the cumulative damage index at i^{th} story, ranging from 0 for undamaged to 1 for severely damaged stories, N is the number of plastic excursions, $\Delta\delta_{pi}$ is the plastic deformation of i^{th} story in j^{th} excursion, δ_y 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 [24]. 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.

$$D_g = \frac{\sum_{i=1}^{n} D_i W_{pi}}{\sum_{i=1}^{n} W_{pi}} \tag{16}$$

where D_g is the global damage index, W_{pi} is the energy dissipated at i^{th} story, D_i is the damage index at i^{th} story, and n is the number of stories. Using this equation, the global damage index of each frame designed according to UBC97 [6] and the optimum lateral loading related to each earthquake has been calculated and presented in Figure 11. The results suggest that the damages experienced by the optimum frames are significantly less than those of the UBC's.

Conclusion

1. This paper presents a new method for optimization of dynamic response of concentrically braced steel frames subjected to seismic excitation. This method is based on the concept of uniform distribution of deformation.
2. It is shown that it is possible to improve the seismic performance of a structure by shifting the material from strong to weak parts. This eventually leads to an optimum distribution of material, correlated with optimum performance of the structure during the given earthquake. It has been shown that at this stage, a state of uniform distribution of deformation prevails. Therefore, in general it may be concluded that we need to reach a status of uniform deformation for optimum use of material. This is considered as the *Theory of Uniform Deformation*.
3. The *Theory of Uniform Deformation* has been employed for evaluation of optimum strength distribution in concentrically braced frames. It is shown that deformation demand is reduced for optimum model compare to conventional models.
4. The shear-building model has been modified by introducing supplementary springs to account for flexural displacements in addition to shear drifts. It is shown that this model can be used for estimating the seismic response of braced frames with acceptable accuracy. Instead of a direct employment of the theory of uniform deformation, it is shown that the modified shear-building model can be used to accomplish the optimum seismic design of braced frames.
5. It has been demonstrated that there is generally a unique optimum distribution of structural properties, which is independent of the seismic load pattern used for initial design.

6. The cumulative damage has been calculated for both optimum and conventional models in different earthquakes. It has been concluded that optimum structures suffer relatively less damage as compared with conventional structures.

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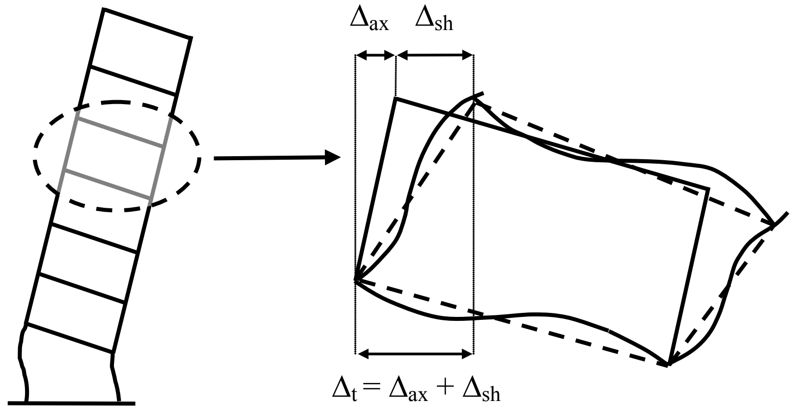
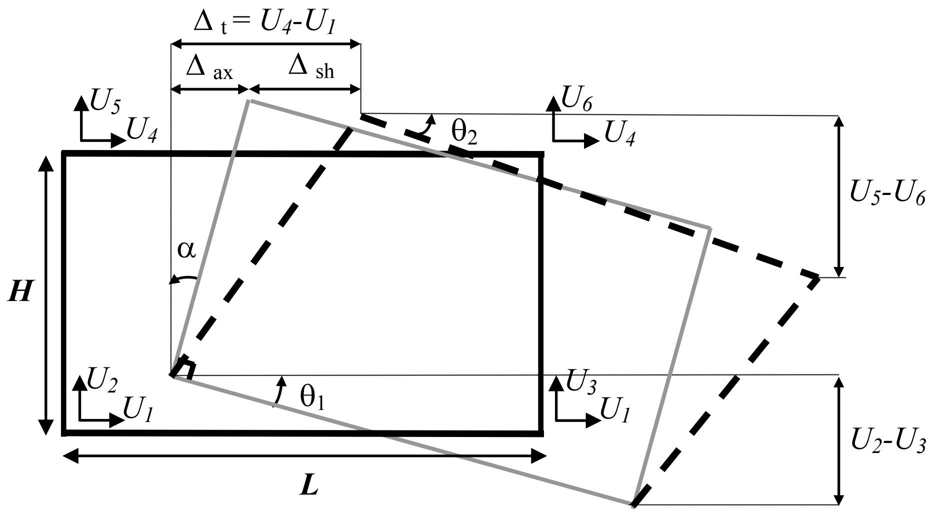


(a)



(b)

Figure 1. Definitions of total inter-story drift (Δ_t), shear inter-story drift (Δ_{sh}) and the effect of axial flexibility of columns (Δ_{ax})

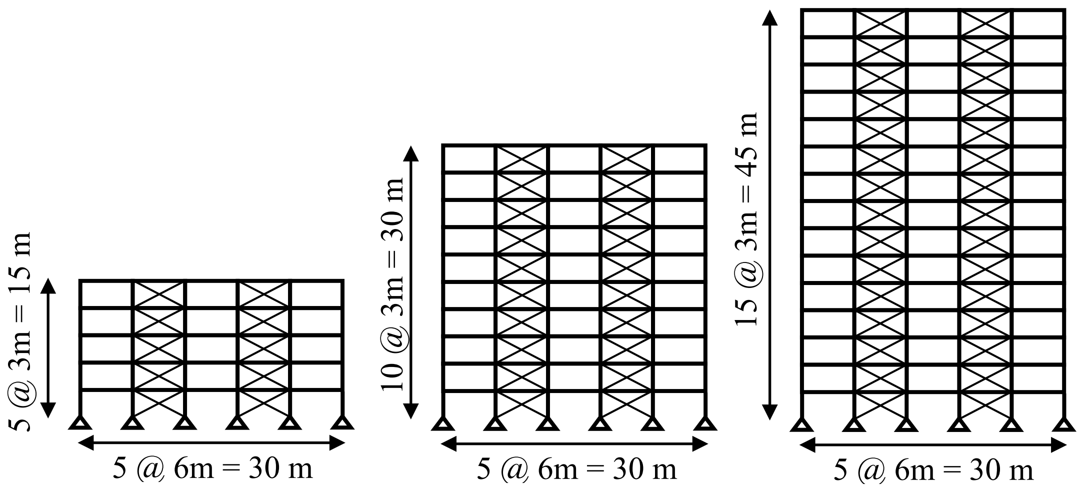


Figure 2. Typical geometry of concentric braced frames

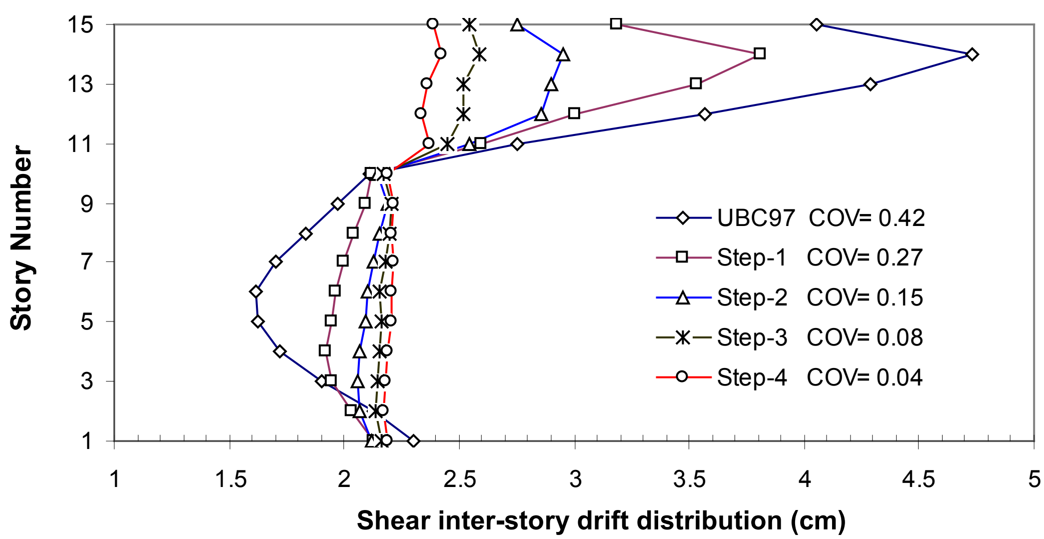


Figure 3. Shear story drift distribution from UBC97 designed model toward the Final answer, 15 story braced frame, Northridge 1994 (NWH360)

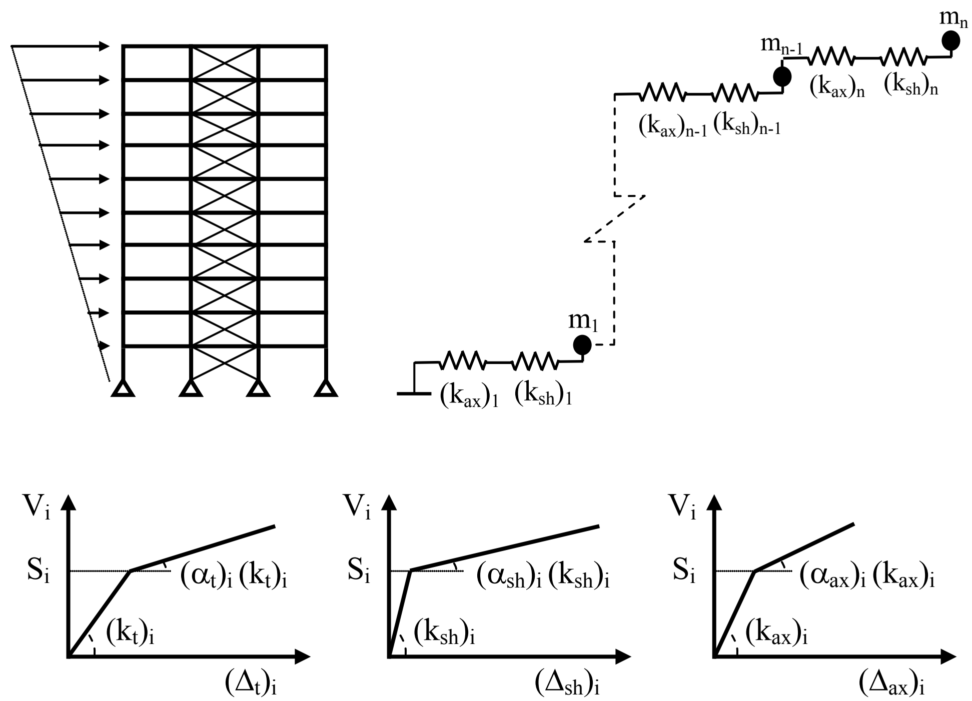


Figure 4. Using push-over analysis to define equivalent modified shear-building model

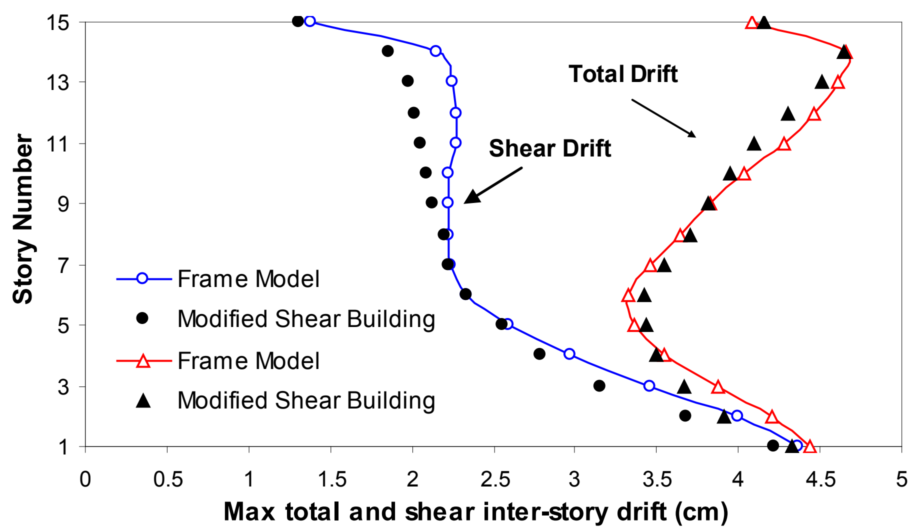


Figure 5. : A comparison of frame model and modified shear-building model for 15-story model subjected to Imperial Valley 1979

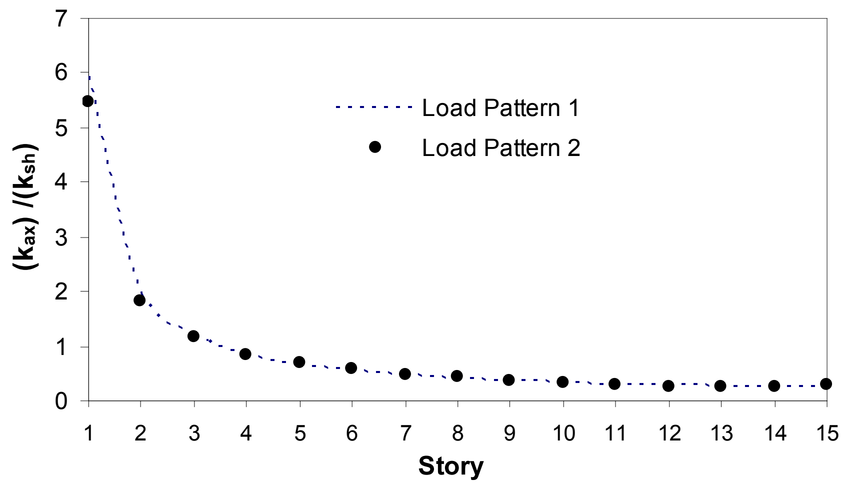


Figure 6. A comparison of $(k_{ax})/(k_{sh})$ ratio for two 15 story braced frames designed for different seismic load patterns

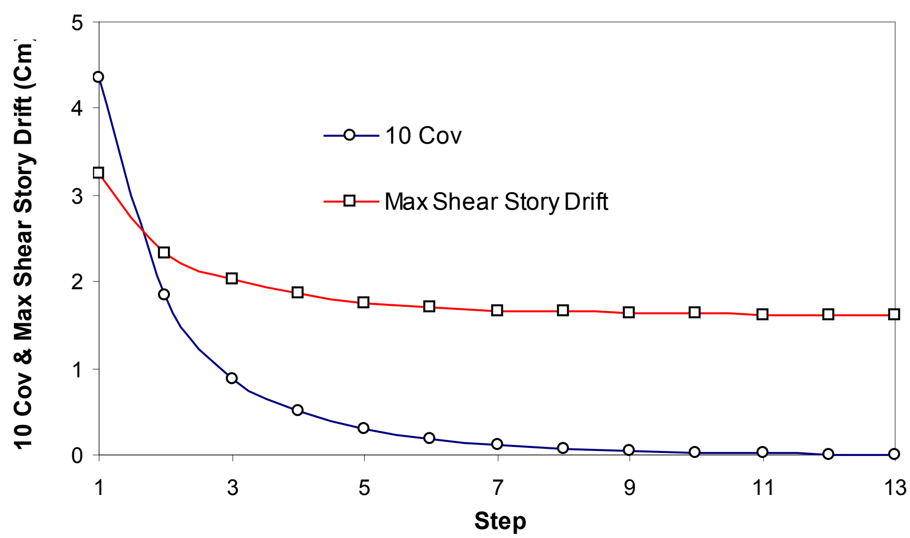


Figure 7. COV of shear story drifts and maximum shear story drifts from UBC97 designed model toward the Final answer, 10story braced frame, Imperial Valley 1979.

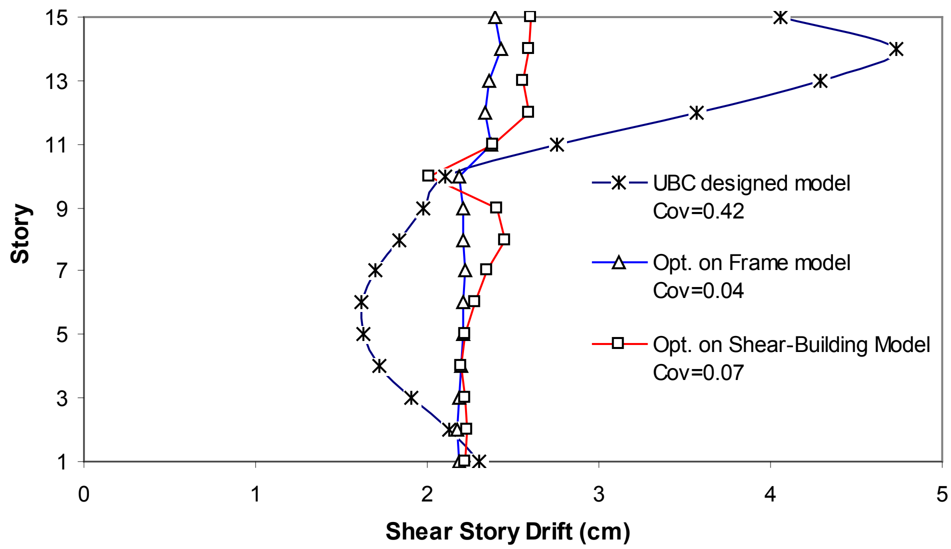


Figure 8. Optimization on frame model and shear-building model compare to UBC designed for 15-story model subjected to Northridge earthquake 1994

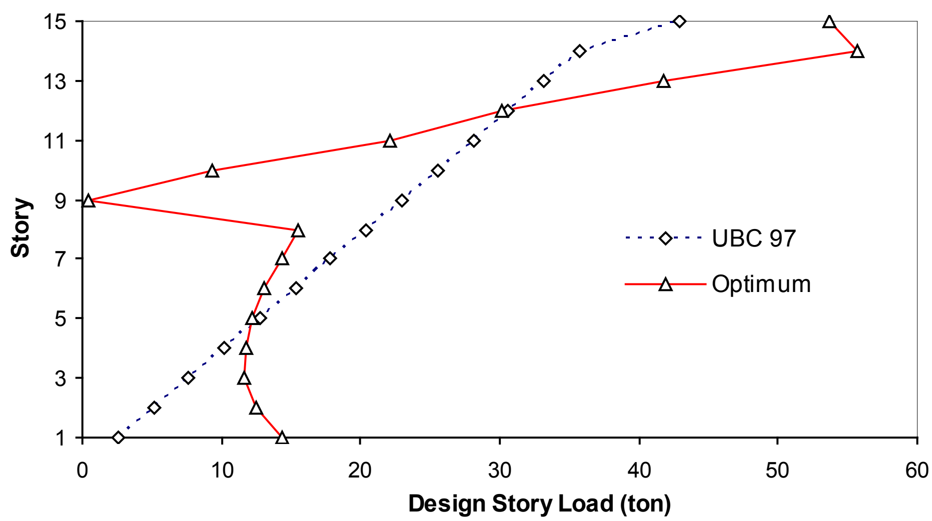


Figure 9. Optimum and conventional design loads for 15-story braced frame subjected to Northridge earthquake 1994

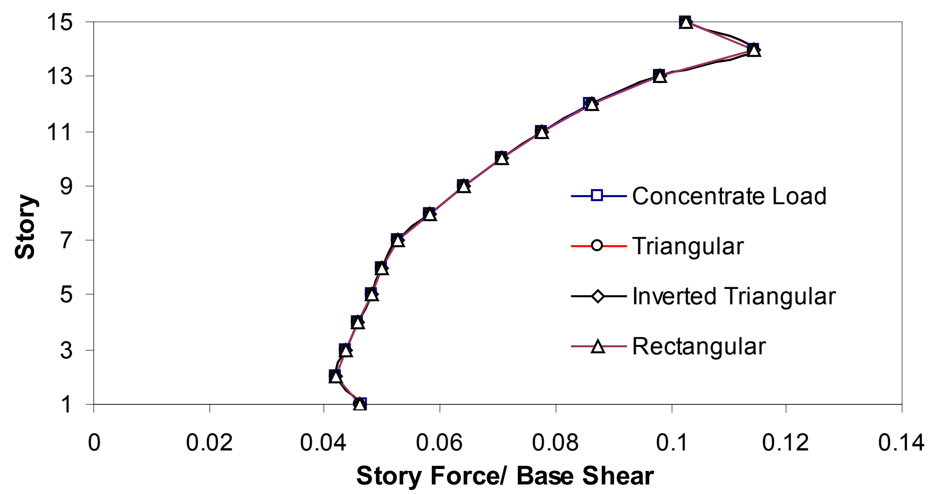


Figure 10. Optimum lateral load pattern for different initial strength patterns, 15 story braced frame subjected to Imperial Valley 1979

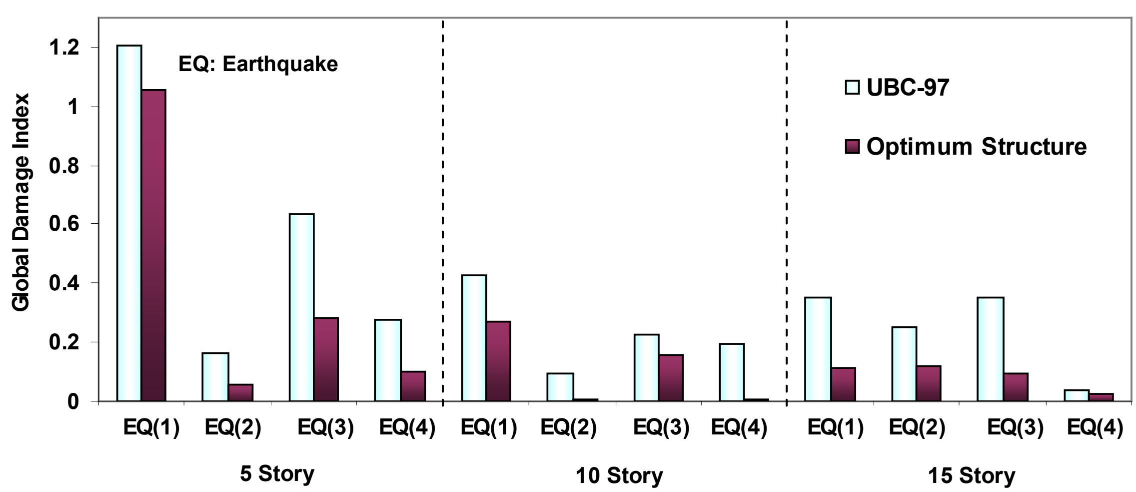


Figure 11. Global damage indices calculated for different models designed with optimum and code-type load pattern under different earthquakes