

Performance-Based Seismic Design of Eccentrically Braced Frames Using Target Drift and Yield Mechanism as Performance Criteria

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Eccentrically braced steel frames are very efficient structures for resisting earthquakes as they combine the ductility that is characteristic of moment frames and the stiffness associated with braced frames. In EBFs the inelastic activity is confined to a small length of the floor beams which yields mostly in shear (therefore called the shear link). A capacity design approach is followed in an attempt to limit the inelastic activity to the shear links only while all other frame members are designed to behave elastically. Research work carried out during the 1970s and 1980s led to the formulation of design code provisions. However, current design practice generally follows elastic analysis procedures for proportioning the frame members. Therefore, it is possible that yielding may not be entirely confined in the shear links and may not be uniformly distributed along the height of the structure as the frame responds in the inelastic range.

Serious efforts have been undertaken to develop the framework for performance-based earthquake engineering (PBEE) in the United States, especially after the 1994 Northridge earthquake. In current practice, the performance-based seismic design for a new structure is carried out in somewhat indirect manner. It usually starts with an initial design according to conventional elastic design procedure using applicable design codes, followed by a nonlinear static pushover assessment (ATC, 1996; ASCE, 2000). Usually, an iterative process between design and assessment is inevitable. Moreover, as mentioned in FEMA 440 (ATC, 2004), this procedure is still not quite satisfactory in predicting reasonably accurate structural behavior during a major earthquake when compared with the results from a nonlinear dynamic analysis.

While further refinement is needed in the current practice to move toward more reliable performance-based design methodologies, this paper presents a direct performance-based design approach that basically requires no assessment

such as nonlinear static or dynamic analysis after initial design. Based on an energy-balance criterion (Leelataviwat, Goel, and Stojadinović, 1999; Lee, Goel, and Chao, 2004), the proposed approach gives a design base shear by using the code-specified elastic design spectral value for a given hazard level, a preselected global structural yield mechanism, and a predesignated target drift. In addition, the design lateral force distribution employed in the proposed method is based on nonlinear dynamic analysis results using a number of ground motions. The shear links are designed according to a plastic design approach (Chao and Goel, 2005), while the members outside the links are designed using a capacity design concept. The entire design procedure can be easily computerized. It should be noted that use of plastic design concepts for EBFs was also proposed by researchers in the past (for example, Roeder and Popov, 1977; Manheim, 1982; Kasai and Popov, 1986). However, use of a preselected yield mechanism with criteria for strength distribution along the height, and determination of design forces based on energy equations are new in the methodology presented herein. A more detailed review of those methods can be found elsewhere (Chao and Goel, 2005).

Four EBFs were investigated in this study: two EBFs (three-story and 10-story) were designed in accordance with current practice using the IBC 2000 procedure; the other two were obtained by redesigning those frames by using the proposed performance-based plastic design method. A method to take care of stiffness irregularity in the EBF was also presented. In order to validate the proposed design method, extensive nonlinear dynamic analyses were conducted using eight 10% in 50 years and four 2% in 50 years SAC Los Angeles region ground motions. The seismic performance of the four study frames was examined in terms of location of yielding, maximum link plastic rotations, maximum relative story shear distributions, maximum interstory drifts, and peak floor accelerations.

PROPOSED PERFORMANCE-BASED DESIGN PROCEDURE

Design Lateral Forces

Unlike in the current design codes, the design lateral force distribution in the proposed method is determined by using a shear distribution factor, β_s , which is obtained from and calibrated by extensive nonlinear time-history analyses of

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EBF. This lateral force distribution accounts for inelastic behavior of EBF when subjected to major earthquakes and can be expressed as (Lee et al., 2004; Chao and Goel, 2005):

$$\beta_i = \frac{V_i}{V_n} = \left(\frac{\sum_i^n w_i h_i}{w_n h_n} \right)^{0.75T^{-0.2}} \quad (1)$$

$$F_n = V \left(\frac{w_n h_n}{\sum_{j=1}^n w_j h_j} \right)^{0.75T^{-0.2}} \quad (2)$$

$$F_i = (\beta_i - \beta_{i+1})F_n \text{ when } i = n, \beta_{n+1} = 0 \quad (3)$$

where

- β_i = shear distribution factor at level i
- V_i, V_n = story shear forces at level i and at the top (n th) level, respectively
- w_i, w_j = seismic weights at level i and j , respectively
- h_i, h_j = heights of levels i and j from the ground, respectively
- w_n = seismic weight of the structure at the top level
- h_n = height of roof level from ground
- T = fundamental structure period obtained by code-specified methods or elastic dynamic analysis
- F_i, F_n = lateral forces applied at level i and top level n , respectively
- V = design base shear

Design Base Shear

Design base shears in current seismic codes are obtained by reducing the elastic strength demands to the inelastic strength demands using the response modification factors. These inelastic strength demands are further increased according to the importance of specific structures using an occupancy importance factor. Generally, the design base shear is determined from the prescribed design acceleration spectrum and expressed in the following form

$$V = C_s W = C_e \left(\frac{I}{R} \right) W \quad (4)$$

where

- C_s = seismic response coefficient
- C_e = normalized design pseudo-acceleration
- I = occupancy importance factor
- R = response modification factor (= 8 for EBFs with a back-up moment frame; = 7 without back-up moment frame)
- W = total seismic weight

After selecting the member sizes for required strengths (which is generally done by elastic analysis), the calculated drift using elastic analysis is multiplied by a deflection amplification factor, C_d , given in the building code, and kept within specified drift limits (on the order of 2%). It should be noted that the response modification factors, R , specified in design codes for various structural systems are set based on a number of factors including engineering judgment. Moreover, conventional design procedures are based on elastic force-based analysis methods rather than displacement-based methods; thus the inelastic response beyond the elastic limit for a structure cannot be predicted with good precision.

A more rational design approach to overcome the above-mentioned shortcomings was proposed based on energy and plastic design concepts (Leelataviwat et al., 1999; Lee et al., 2004). In this approach the design base shear is determined by pushing the structure monotonically up to a target drift after the formation of a selected yield mechanism. The amount of external work needed to do that is assumed as a factor, γ , times the elastic input energy, $E (= \frac{1}{2} MS_v^2)$. It should be noted that this amount of work assumes no relationship with the energy dissipated during earthquake excitation. In the proposed method, it is simply used as a means to calculate the required design base shear by establishing a tie between the intended yield mechanism, target drift, force-displacement characteristics of the structure, and elastic input energy from the design ground motion. The modification factor, γ , is dependent on the natural period of the structure, which has significant influence on the earthquake input energy as observed by many investigators [for example, Uang and Bertero (1988)]. Thus, the energy balance equation can be written as

$$\gamma E = \gamma \left(\frac{1}{2} MS_v^2 \right) = (E_e + E_p) \quad (5)$$

where

- E_e, E_p = elastic and plastic components, respectively, of the energy needed as the structure is pushed up to the target drift
- S_v = design pseudo-velocity
- M = total mass of the system

The modification factor, γ , depends on the structural ductility factor, μ_s , and the ductility reduction factor, R_μ , which is related to the structure's period. Figure 1 shows the relationship between the base shear (CW) and the corresponding drift (Δ) of an elastic system and an elastic-plastic system. From the geometric relationship, Equation 5 can be written as

$$\gamma \left(\frac{1}{2} C_{eu} W \Delta_{eu} \right) = \frac{1}{2} C_y W (2\Delta_{\max} - \Delta_y) \quad (6)$$

Using the expression for drifts (Δ), Equation 6 can be rewritten as

$$\gamma \frac{\Delta_{eu}}{\Delta_y} = \frac{(2\Delta_{max} - \Delta_y)}{\Delta_{eu}} \quad (7)$$

where

$$\begin{aligned} \Delta_{eu} &= R_\mu \Delta_y \\ \Delta_{max} &= \mu_s \Delta_y \end{aligned}$$

Substituting these terms into Equation 7, the energy modification factor, γ , can be determined as

$$\gamma = \frac{2\mu_s - 1}{R_\mu^2} \quad (8)$$

where

- μ_s = structural ductility factor which is equal to target drift divided by yield drift = Δ_{max} / Δ_y
- R_μ = ductility reduction factor which can be determined based on the μ_s value

It can be seen from Equation 8 that the energy modification factor, γ , is a function of the ductility reduction factor and the structural ductility factor. In this study, the method proposed by Newmark and Hall (1982) is adopted to relate the ductility reduction factor and the structural ductility factor as shown in Figure 2. Plots of calculated energy modification factor, γ , from Equation 8 are shown in Figure 3.

The design elastic energy demand, E , can be determined from the elastic design pseudo-acceleration spectra as typically given in the building codes. The design pseudo-acceleration based on the selected design spectrum for elastic systems can be specified as

$$A = C_e g \quad (9)$$

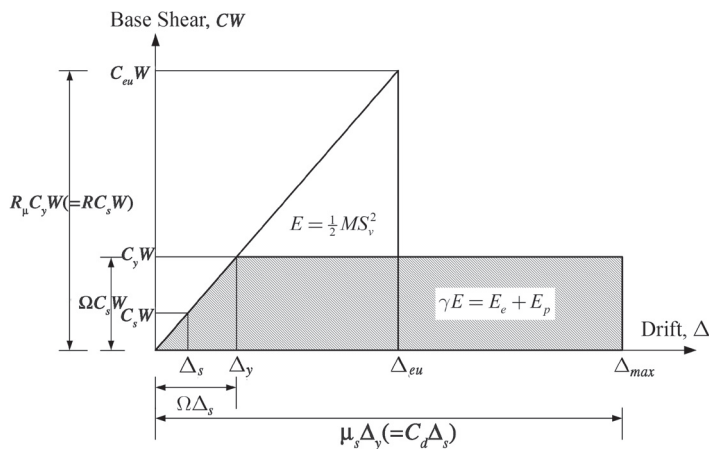


Fig. 1. Structural idealized response and energy balance concept.

where

- A = design pseudo-acceleration
- g = acceleration due to gravity
- C_e = normalized design pseudo-acceleration as defined in Equation 4

Note that no occupancy importance factor is included in the design pseudo-acceleration for the proposed approach. The occupancy importance factor, I , raises the design force level in an attempt to decrease the drift and ductility demand for the structure for a given level of ground shaking (SEAOC, 1999; NEHRP, 2001). However, that cannot be considered as a direct method to achieve the intended purpose such as damage control. The reduction of potential damage should better be handled by using appropriate drift limitations. In this regard, the approach of calculating the design base shear proposed in this study uses the target drift as the governing parameter. It is assumed that the selected target drift will

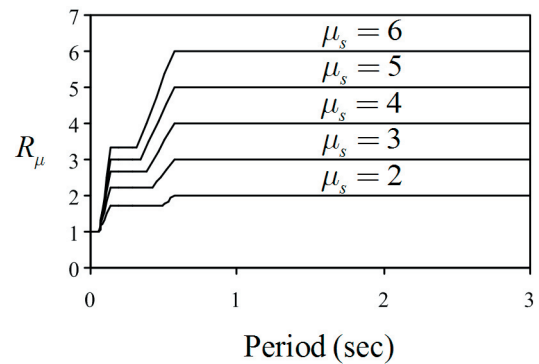


Fig. 2. Ductility reduction factors proposed by Newmark and Hall (1982).

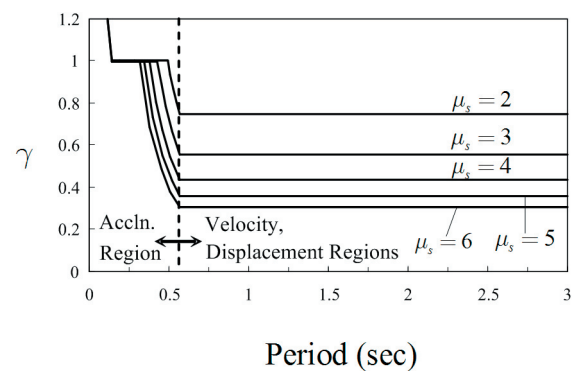


Fig. 3. Modification factors for energy equation versus period.

have the occupancy importance factor built into it. In any case, C_e can be modified or increased in order to account for factors such as near-fault effect, redundancy consideration, or possible torsion in the global structural system. Pending further research on these issues, guidance given in current codes can be used.

The energy balance equation can be rewritten as

$$(E_e + E_p) = \gamma \left(\frac{1}{2} M S_v^2 \right) = \frac{1}{2} \gamma M \left(\frac{T}{2\pi} C_e g \right)^2 \quad (10)$$

Akiyama (1985) showed that the elastic vibrational energy, E_e , can be calculated by assuming that the entire structure can be reduced into a single-degree-of-freedom system. In other words,

$$E_e = \frac{1}{2} M \left(\frac{T}{2\pi} \frac{V}{W} g \right)^2 \quad (11)$$

where

V = yield base shear

W = total seismic weight of the structure = Mg

Substituting Equation 11 into Equation 10 and rearranging the terms gives

$$E_p = \frac{WT^2g}{8\pi^2} \left(\gamma C_e^2 - \left(\frac{V}{W} \right)^2 \right) \quad (12)$$

Using a preselected yield mechanism for various EBF configurations shown in Figure 4 and equating the plastic energy term E_p to the external work done by the design lateral forces (Equation 3) gives

$$E_p = \sum_{i=1}^n F_i h_i \theta_p \quad (13)$$

where θ_p is the global inelastic drift of the structure, which is the difference between the preselected target drift, θ_u , and yield drift, θ_y . The yield drift of an EBF was assumed as 0.5% for design purposes. Substituting Equations 3 and 12 into Equation 13, and solving for V/W gives

$$\frac{V}{W} = \frac{-\alpha + \sqrt{\alpha^2 + 4\gamma C_e^2}}{2} \quad (14)$$

where

V = design base shear

α = dimensionless parameter, which depends on the stiffness of the structure, the modal properties, and the intended drift level

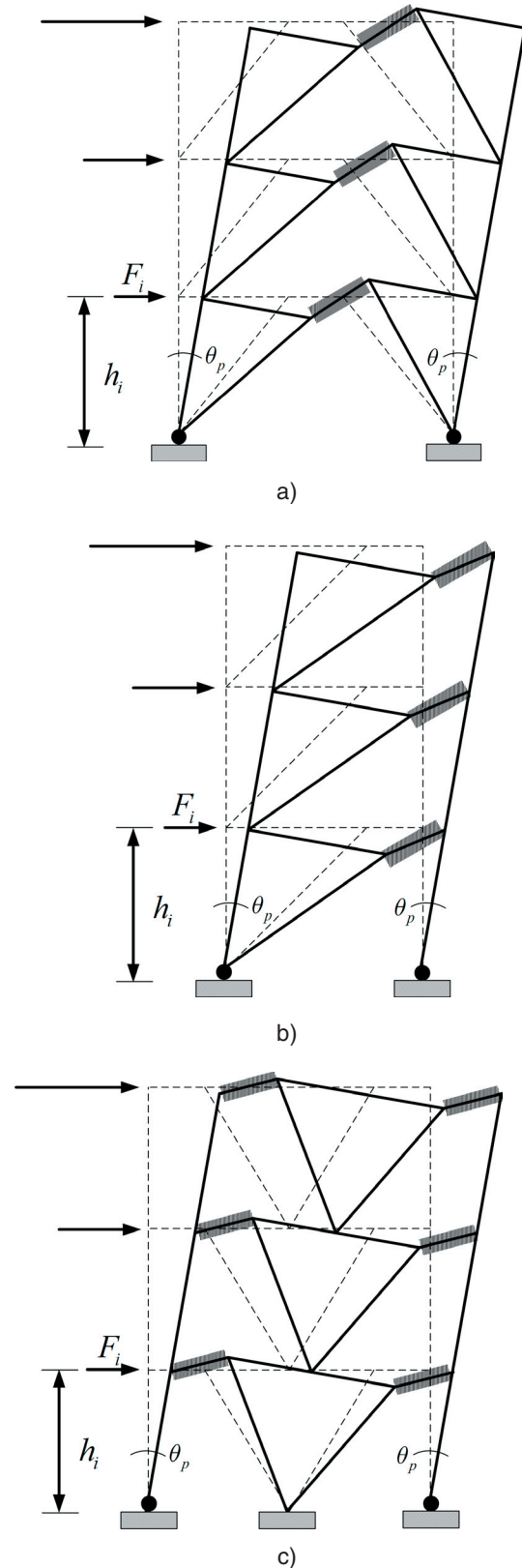


Fig. 4. Preselected yield mechanism of EBFs with various geometries.

$$\alpha = \left(\sum_{i=1}^n (\beta_i - \beta_{i+1}) h_i \right) \left(\frac{w_n h_n}{\sum_{j=1}^n w_j h_j} \right)^{0.75T^{-0.2}} \left(\frac{\theta_p 8\pi^2}{T^2 g} \right) \quad (15)$$

It should be noted that the required design base shear given by Equation 14 is related to the lateral force distribution, the intended target drift, θ_p , and a preselected yield mechanism. Also note that in Equation 15 when $i = n$, $\beta_{n+1} = 0$.

It can be seen that the proposed method for determining lateral design forces is based on principles of structural dynamics, while ensuring formation of selected yield mechanism and drift control simultaneously. There is no need for using a response modification factor, R , an occupancy importance factor, I , or a displacement amplification factor (such as C_d), which are required in current practice and are largely based on engineering judgment. It should also be noted that the design base shear in the proposed method as given by Equation 14 represents the ultimate yield force level (in other words, $C_y W$ in Figure 1) at which the complete mechanism is expected to form. In contrast, the code design base shear as given by Equation 4 represents the required strength at the first significant yield point for use in design by elastic methods.

Preselected Yield Mechanism

Figure 4 shows EBF subjected to design lateral forces and pushed to its drift limit state under design ground motions. All the inelastic deformations are intended to be confined within the shear links in the form of shear yielding (for short link). Since the plastic hinges developed at the column bases are almost inevitable in a major earthquake, the desired global yield mechanism of an EBF is formed by shear yielding of the links plus flexural plastic hinges at the column bases.

Proportioning of Link Beams

The proposed lateral force distribution, obtained and calibrated by nonlinear dynamic analyses (Chao and Goel, 2005), helps to distribute the yielding more uniformly along the height, thereby preventing excessive yielding from concentrating at a few levels. Referring to an EBF with the link connected to the column in Figure 5, only one bay of the frame and the corresponding portion of lateral forces are shown for illustration of the design procedure. The gravity loads (dead and live) are assumed uniformly distributed and no pattern loading is considered for live loads.

By using the principle of virtual work and equating the external work to the internal work, as is commonly done for plastic analysis by the mechanism method and knowing that the link plastic rotation angle is $\gamma_p = (L/e) \theta_p$ for the EBF shown in Figure 5 (AISC, 2005), the required shear strength

of shear links, $\beta_i V_{pr}$, for the EBF with fixed column bases and uniformly distributed gravity loading at any level i can be determined as

$$\beta_i V_{pr} = \beta_i \frac{\left(\sum_{i=1}^n F_i h_i + \frac{1}{2} L(L-e) \sum_{i=1}^n w_{iu} - 2M_{pc} \right)}{L \sum_{i=1}^n \beta_i} \quad (16)$$

where

- L = span length
 - e = length of shear link
 - V_{pr} = required shear strength of links at the top level and the only unknown variable in Equation 16
 - M_{pc} = required plastic moment of columns in the first story as shown in Figure 5, and is chosen in such a way that no soft-story mechanism would occur when a factor of 1.1 times the design lateral forces are applied on the frame (Leelataviwat et al., 1999)
- $$= \frac{1.1V'h_1}{4} \quad (17)$$
- V' = base shear for one bay only, which is equal to V divided by the number of braced bays
 - h_1 = height of the first story

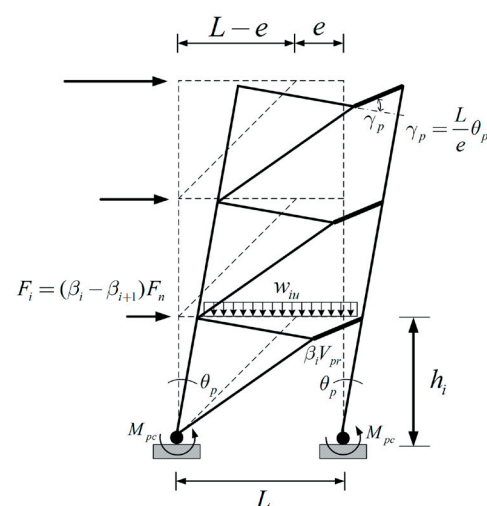


Fig. 5. One-bay EBF with preselected yield mechanism for calculating required strength of shear links. Note that the values of F_i and F_n are for one bay only.

The factor 1.1 is the overstrength factor accounting for strain-hardening and uncertainty in material strength. For EBFs with other configurations [Figures 4(a) and 4(c)], $\beta_i V_{pr}$ can be easily derived using the above principles (Chao and Goel, 2005).

The design of shear links is then performed as follows

$$\begin{aligned}\phi V_n &= 0.9V_p = 0.9(0.6F_y A_w) \\ &= 0.9[0.6F_y (d_b - 2t_f)t_w] \geq \beta_i V_{pr}\end{aligned}\quad (18)$$

Link sections should also satisfy the width-thickness limitations given in the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2005) Table I-8-1; in other words, $0.38\sqrt{E_s/F_y}$.

Design of Members Outside the Shear Links

The design of elements outside the shear links, including beam segments, braces, and columns, is performed based on the capacity design approach. That is, elements outside the shear links should have a design strength to resist the combination of factored gravity loads and the maximum expected shear forces as well as moments developed in the shear links. As specified in the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2005), the maximum expected shear force for short links ($V_u = 1.25R_y V_p$) can be considerably higher than the plastic shear strength (V_p) resulting primarily from material overstrength, strain-hardening, and the development of shear resistance in the link flanges. By the same reasoning, maximum link moments at the column side and brace side for shear links (which means short links in this study; that is, $e \leq 1.6M_p/V_p$) are $M_C = R_y M_p$ and $M_B = [e(1.25R_y V_p) - R_y M_p] \geq 0.75R_y M_p$, respectively. It is noted that, for the design of the beam segment outside the link, the required strength based on only 1.1 times the link expected shear strength is allowed by the *Provisions*; namely, link shear $= 1.1R_y V_p$.

Once the maximum expected link shear forces and moments are determined, the frame can be cut into several free bodies, in other words, columns with associated beam segments and braces. An example three-story EBF and the corresponding column free body diagrams are shown in Figure 6. (Figures 6 through 19 begin on page 183.)

Design of Columns with Associated Beam Segments and Braces

The design procedure is illustrated by using an example three-story EBF shown in Figure 6. Due to the presence of pin connections, the exterior columns are designed using gravity loads only, whereas the interior columns need to be designed based on the capacity approach. The free bodies of interior columns with and without braces and beam segments

are shown schematically in Figures 6a and 6b, respectively. When the frame reaches its target drift, the shear forces and end moments in shear links at all levels are assumed to reach the maximum expected shear strength, V_w and maximum expected moments, M_C and M_B . The column strength at the lowermost level is also assumed to have reached its maximum capacity, M_{pc} . At this stage, the required balancing lateral forces applied on this free body are assumed to maintain the distribution as used earlier and can be easily calculated by using moment equilibrium of the free bodies. The lateral forces should be updated based on the expected strength of shear links because they have significant influence on the internal forces of members outside the shear links.

For the interior columns shown in Figure 6a; when the lateral forces are acting to the left, the sum of the applied lateral forces, F_L , can be obtained from moment equilibrium at column bases

where

$$F_L = \frac{(L-e)\sum_{i=1}^n (V_u)_i + \sum_{i=1}^n (M_B)_i + \frac{(L-e)^2}{2}\sum_{i=1}^n w_{iu} + M_{pc}}{\sum_{i=1}^n \alpha_i h_i} \quad (19)$$

w_{iu} = factored uniformly distributed gravity load

$$= 1.2 DL + 0.5 LL$$

$$\alpha_i = \frac{F_i}{\sum_{i=1}^n F_i} = \frac{(\beta_i - \beta_{i+1})}{\sum_{i=1}^n (\beta_i - \beta_{i+1})} \quad (20)$$

when $i = n$, $\beta_{n+1} = 0$

For interior columns shown in Figure 6b, the design is governed by the lateral forces acting to the right. The sum of lateral forces, F_R , can be calculated as

$$F_R = \frac{\sum_{i=1}^n (V_u)_i \frac{(d_c)_i}{2} + \sum_{i=1}^n (M_C)_i + M_{pc}}{\sum_{i=1}^n \alpha_i h_i} \quad (21)$$

where

d_c = depth of the column section and may be assumed as constant at this stage of design

After the lateral forces have been calculated as described above, the required strength of outside elements (beam segments, braces, and columns) can be easily computed by using any elastic structural analysis program to model the column free bodies shown in Figure 6. The terms $\alpha_i F_r$ ($\alpha_i F_L$), $(V_u)_i$, $(M_B)_i$, $(M_C)_i$, w_{iu} , and $(P_u)_i$ represent applied loads. The

term $(P_u)_i$ represents axial force in the columns resulting from tributary gravity loads. Design of these elements is performed in accordance with the AISC LRFD conventional elastic design procedures. Flowcharts are given in Figures 7 and 8 to illustrate the entire proposed design procedure.

DESIGN AND ANALYSIS OF STUDY FRAMES

Two eccentrically braced frames were investigated, one with three stories and the other with 10 stories. Both frames (hereinafter called IBC frames) were originally designed by a conventional elastic design method in accordance with IBC 2000 criteria (Richards, 2004). In order to prevent concentration of large inelastic link deformation from occurring at some levels, the capacity-to-demand ratio of link sections in the IBC frames were optimized later based on $\phi_v V_p / V_u \geq 1.0$ [suggested to be kept uniform for all links in Popov, Ricles, and Kasai (1992)], where V_u is the link shear force demand, which is obtained from elastic analysis; ϕ_v is the resistance factor; and V_p is the nominal shear strength.

With slightly different design parameters but with the same frame configurations, these two frames were redesigned by using the proposed performance-based plastic design procedure (hereinafter PBPD frames). Rather than amplifying the design base shear, the design of the two PBPD frames was controlled by choosing an intended yield mechanism [Figure 4(b)] and a maximum allowable interstory drift of 2%. The resulting design base shears were 1,374 kips and 1,358 kips for the three-story and 10-story PBPD frames, respectively. Preliminary analyses suggested that change of height at the second level of the 10-story frame resulted in a link deformation concentration at that level. A remedy for the stiffness irregularity was employed based on the observation that link rotation at a story is proportional to the lateral story drift. Therefore, the link rotation can be controlled by adjusting the link strength based on the ratio of target story stiffness to the current story stiffness. That is,

$$V'_p = V_p \frac{K_t}{K_{story}} \quad (22)$$

where

- V_p = design link shear strength obtained according to the proposed design method, without taking the stiffness irregularity into consideration
- V'_p = modified link shear strength
- K_{story} = elastic story stiffness

In general, for a regular building, story stiffness is maximum at the bottom story and gradually decreases as we move up. Therefore, the target story stiffness K_t for the second floor should be somewhat in between the stiffness of the first and third stories. The final member sizes for the IBC and

PBPD frames are shown in Figures 9 and 10. In general, the IBC frames have heavier link sections and lighter column sections than those of the corresponding PBPD frames.

Nonlinear time-history analyses were conducted to investigate the behavior of the IBC frames and the PBPD frames. The analyses were performed using the Perform-3D (RAM, 2003) program, which has a built-in shear hinge model for short shear links. The leaning columns with vertical loading from gravity frames were not included in this study since the interstory drifts were not large enough to induce significant $P-\Delta$ effect. Also, the beneficial effect of the leaning columns to provide additional lateral strength was ignored. Eight 10% in 50 years and four 2% in 50 years SAC Los Angeles ground motions were selected as the input earthquake records. For comparison purposes, the design gravity loading for the PBPD frames was applied to both IBC and PBPD frames when conducting nonlinear analyses, even though the IBC frames were designed for larger gravity loading. Therefore, some response parameters in the IBC frames, such as maximum interstory drift and link plastic rotation, may have been slightly underestimated.

PERFORMANCE EVALUATION OF STUDY FRAMES

Location of Yield Activity

To achieve the goal of damage control in PBEE, it is important to have predictable and known locations of damage. In the proposed design method a target yield mechanism with defined locations of yielding is preselected. In an EBF, the inelastic activity is expected to occur at first level column bases and in shear links only. Some selected analysis results are shown in Figures 11 and 12. As can be seen, all the inelastic activity was limited in shear links and first story column bases in PBPD frames. On the other hand, the inelastic activity in the IBC frames was scattered, which also varied with different earthquake events. Such seismic behavior was also observed in analyses carried out by Richards and Uang (2003) on the same IBC frames. The maximum column plastic hinge rotation is 0.014 radian in IBC frames, which may have been somewhat underestimated as explained earlier. As mentioned earlier, the column moments in PBPD frames stayed below their yield capacity except at the base. For instance, seven out of the 10 interior columns [first interior column line, left side of the frame, see Figure 10(b)] reached 70% of the yielding capacity of the columns when subjected to LA 01 ground motion; while eight out of the 10 interior columns reached 90% of the yielding capacity of the columns when subjected to LA 09 ground motion.

Maximum Plastic Rotation in Shear Links

As indicated in Figures 13 and 14, the maximum link plastic rotations in PBPD frames are generally within the AISC de-

sign limitation of 0.08 radian, and well below the maximum rotation capacity achieved in University of Texas–Austin tests using the new AISC loading protocol (Okazaki, Arce, Ryu, and Engelhardt, 2004). It is also noticed that, especially for the 10-story frame, the PBPD frames showed more evenly distributed plastic rotations along the height. This can be attributed to the proposed lateral force distribution used for the design. In contrast, as shown in Figure 14a, the link plastic rotation angles are not evenly distributed among floors in the IBC frame. Although most maximum link plastic rotation angles at each floor are well below the AISC limitation, the plastic rotation at upper floors tended to increase suddenly and exceeded the 0.08 radian limitation for some ground motions. The top floor links exhibited only minor yielding, which means the top floor links were not fully utilized to dissipate energy. Higher plastic rotations in the upper floor (except for the top floor), shear links in EBF designed according to the elastic method, and code specified lateral forces have also been observed in past studies (Popov et al., 1992).

Maximum Interstory Drifts

Figure 15 shows the maximum interstory drifts resulting from selected 10% in 50 years ground motions for the three-story and 10-story EBF. It is seen that all the maximum interstory drifts of the PBPD frames were within the 2% pre-selected target drift, signifying that the seismic performance of the deformation-sensitive components (such as cladding, partitions, interior veneers, and glazing systems), as well as the link damage can be controlled by the proposed design procedure. It is also indicated in these figures that, the PBPD frames generally exhibited more uniform interstory drifts than the IBC frames.

Maximum Absolute Floor Accelerations

As seen in Figures 16 and 17, the maximum absolute floor accelerations are generally below the code-specified floor design acceleration, indicating that the seismic performance of the acceleration-sensitive components (such as mechanical equipment, piping systems, and storage vessels) can also be expected to be satisfactory. It is also evident that the acceleration-sensitive components in a low-rise EBF are more vulnerable than in a high-rise EBF.

Maximum Relative Story Shear Distributions

The maximum relative story shear distributions (namely, the story shear in any story divided by the story shear in the top story) obtained from nonlinear dynamic analyses using 10% in 50 years ground motions for the three-story and 10-story PBPD EBF are shown in Figure 18. It can be seen that the proposed design story shear distribution well represents the envelope story shear distribution of the structure due to the

ground motion records used in this study. It is also seen that while the IBC 2000 and proposed distributions have minor difference in the three-story EBF, the difference becomes more significant in the 10-story EBF. The results suggest that the code-specified distribution, which is generally based on the first mode elastic story shear distribution including some higher mode effect, cannot represent the realistic maximum story shear distribution during a major earthquake. The proposed design force distribution does it much better because it is based on consideration of inelastic response. Based on the observation from all nonlinear analyses performed in this study, it is also noted that the maximum story shear at each level generally occurred at approximately the same time.

Multilevel Seismic Design

Performance objectives in PBEE typically include multiple goals. This multilevel design can be easily carried out by the proposed performance-based approach and is demonstrated in the following for the 10-story PBPD frame.

In this study, the first performance level was that under 10% in 50 years hazard (design basis earthquake, DBE = $\frac{2}{3}$ maximum considered earthquake, MCE) the maximum interstory drift should be within an assumed target drift of 2%. For 2% in 50 years hazard (MCE), the maximum target interstory drift was assumed as 3%, to be consistent with the ratio of MCE and DBE spectrum intensity. Accordingly, the design seismic coefficient, C_s , and the design pseudo-acceleration, C_e , were amplified by a factor of 1.5 to determine the MCE design base shear coefficient. The resulting design base shear for the second performance level was 1,313 kips, which is less than the design base shear for the first performance level (1,358 kips); hence, the first level governed the design. The results of nonlinear dynamic analyses using four 2% in 50 year SAC ground motions, as presented in Figure 19, showed that the interstory drifts indeed remained within 3% for the ground motions used.

Material Weight Comparison

Tables 1 and 2 (tables begin on page 198) compare the material weight used in the IBC frames and PBPD frames. Although the design base shears are different for the IBC and PBPD frames, these tables give a general idea how the material difference exists in the two frames. As seen from the two tables, while the IBC frames have heavier total beam weight and lighter total column weight, the PBPD frames are just the opposite. The total material weights of the two frames are almost equal, but the PBPD frames showed better performance, as described above. It should be mentioned that, in practice, column sizes can be changed every two or three floors instead of every floor as done in this study. This would somewhat increase the material weight but reduce the fabrication cost such as column splices.

DESIGN EXAMPLE

The proposed performance-based plastic design procedure is illustrated by following the flowcharts given in Figures 7 and 8 using the 10-story PBPD frame.

- The corresponding design parameters are first obtained in accordance with IBC 2000 as summarized in Table 3a.
- The design period is revised based on preliminary analysis. A target drift of 2% is selected. Then the modified design parameters are calculated and listed in Table 3b. The design base shear is obtained according to Equations 1, 14, and 15. After knowing the design base shear, the lateral forces are calculated by using Equation 2 as shown in Table 4.
- The shear link sections are then designed by using Equations 16 and 18. Compactness needs to be checked. ASTM A992 steel with 50-ksi nominal yield strength is used. The final design sections are listed in Table 5. Note that the link section at the second level was modified according to Equation 22 to account for the story stiffness irregularity. The original section at the first level was W16×67, which was reduced to W10×39 for optimization of responses. The maximum expected shear force, the maximum link moment at the column side, and the maximum link moment at the brace side are then calculated to design the frame members outside the shear links.
- Design of members outside the shear links is illustrated by using two interior columns similar to those in Figures 6(a) and 6(b). The applied forces on the columns with braces and beam segments [Figure 6(a)] are updated lateral forces $\alpha_i F_L$ (or $\alpha_i F_R$); axial forces in the columns resulting from tributary gravity loadings (P_u)_i; distributed gravity loading, w_{ui} ; maximum link moment at brace side (M_B)_i; and maximum expected shear forces (V_u)_i. For columns without braces and beam segments [Figure 6(b)], the applied forces are $\alpha_i F_R$, (V_u)_i, (M_C)_i, (ΔM)_i, and (P_u)_i. Corresponding values are summarized in Tables 6 and 7.
- The free bodies shown in Figures 6(a) and 6(b) are then analyzed using elastic analysis programs to calculate the elastic internal forces, which in turn are used to design the columns and braces according to the AISC LRFD specification.

SUMMARY AND CONCLUSIONS

A direct performance-based design approach that basically requires no assessment such as nonlinear static or dynamic analysis after initial design has been presented. Based on energy-balance criterion, the proposed approach gives the design base shear by using the elastic design spectral value for a given hazard level, a preselected global structural yield

mechanism, and a predesignated target drift. In addition, the design lateral force distribution employed in the proposed method is based on nonlinear dynamic analysis results using a number of SAC ground motions. The shear links are designed according to the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2005), while the members outside the links are designed by using a capacity design concept. The following conclusions can be drawn from this study:

1. Overall, the EBFs designed by the proposed method can be expected to satisfy the required performance objectives when subjected to a major earthquake. That is because the selected performance objectives in terms of the yield mechanism and maximum drift are explicitly built into the determination of design lateral forces and design of the frame members.
2. All the inelastic activity was confined to the shear links and the column bases in PBPD frames as intended. On the other hand, inelastic activity in the IBC frames occurred in a somewhat less controlled manner among the frame members including columns, depending on the ground motion used.
3. The maximum link plastic rotations in PBPD frames were generally within the AISC limitation, and well below the maximum rotations reached in the University of Texas–Austin tests. Moreover, the 10-story PBPD frame showed more evenly distributed plastic rotations along the height. On the other hand, in the IBC frame, although most maximum link plastic rotations were below the AISC limitation, the plastic rotations at upper floors tended to be higher and exceeded the 0.08 radian limit during some of ground motions. The roof level links exhibited only minor yielding.
4. The maximum interstory drifts in the PBPD frames were within the 2% preselected target drift for all the selected 10% in 50 years ground motions, signifying that the seismic performance of the deformation-sensitive components can be controlled by the proposed design procedure.
5. The maximum absolute floor accelerations were generally below the code-specified floor design acceleration, indicating that the seismic performance of the acceleration-sensitive components can also be assumed to be satisfactory. It was also evident that the acceleration-sensitive components in a low-rise EBF are more vulnerable than in a high-rise EBF.
6. Generally, the proposed design story shear distribution represents the envelope story shear distribution of the structure due to the ground motion records used in this

study very well, because it is based on inelastic behavior. On the contrary, the code-specified force distribution, although it includes higher mode effect, does not represent realistic maximum story shear distribution during major earthquakes which causes the structures to respond inelastically.

7. It was shown that the proposed procedure can be used to achieve the multilevel design goals as required in PBEE.
8. The results also showed that the EBFs can be designed by using the proposed methodology to determine the preselected target performance without increase in the material weight.

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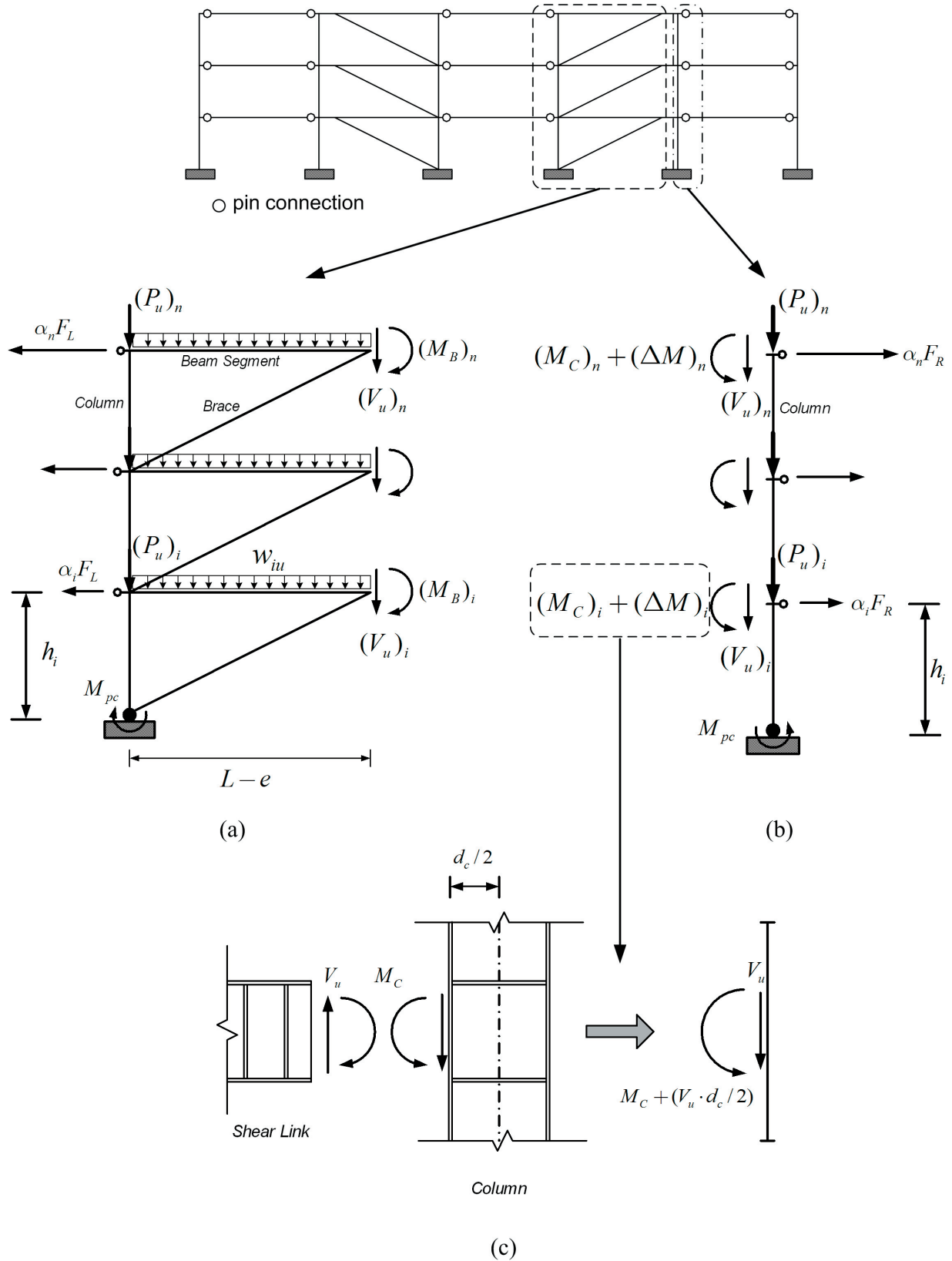


Fig. 6. Free body diagram of interior columns and associated beam segments and braces:
(a) lateral forces acting toward left; (b) lateral forces toward right; (b) illustration showing $(\Delta M)_i$ in the figure.

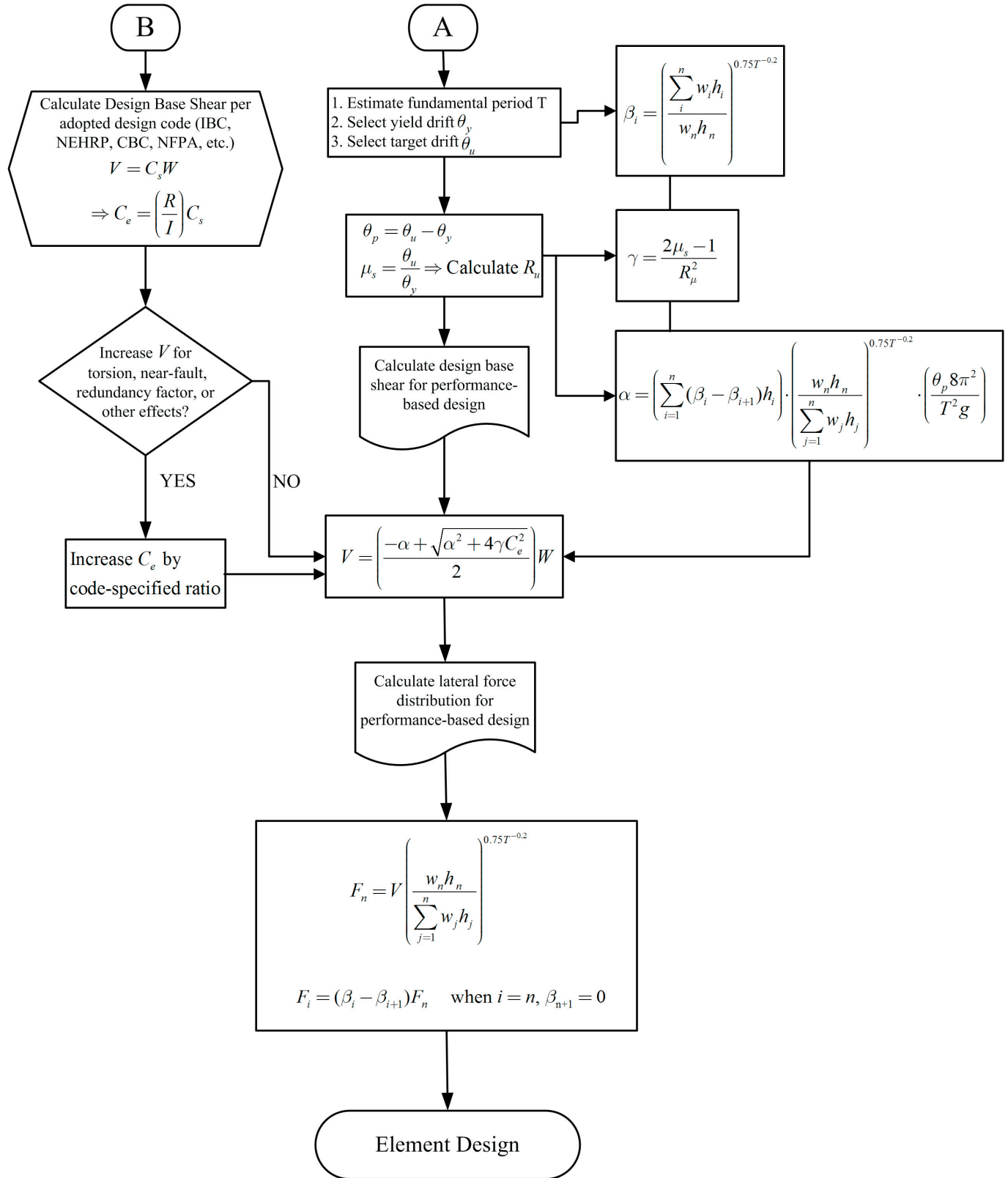


Fig. 7. Performance-based plastic design flowchart: design base shear and lateral force distribution.

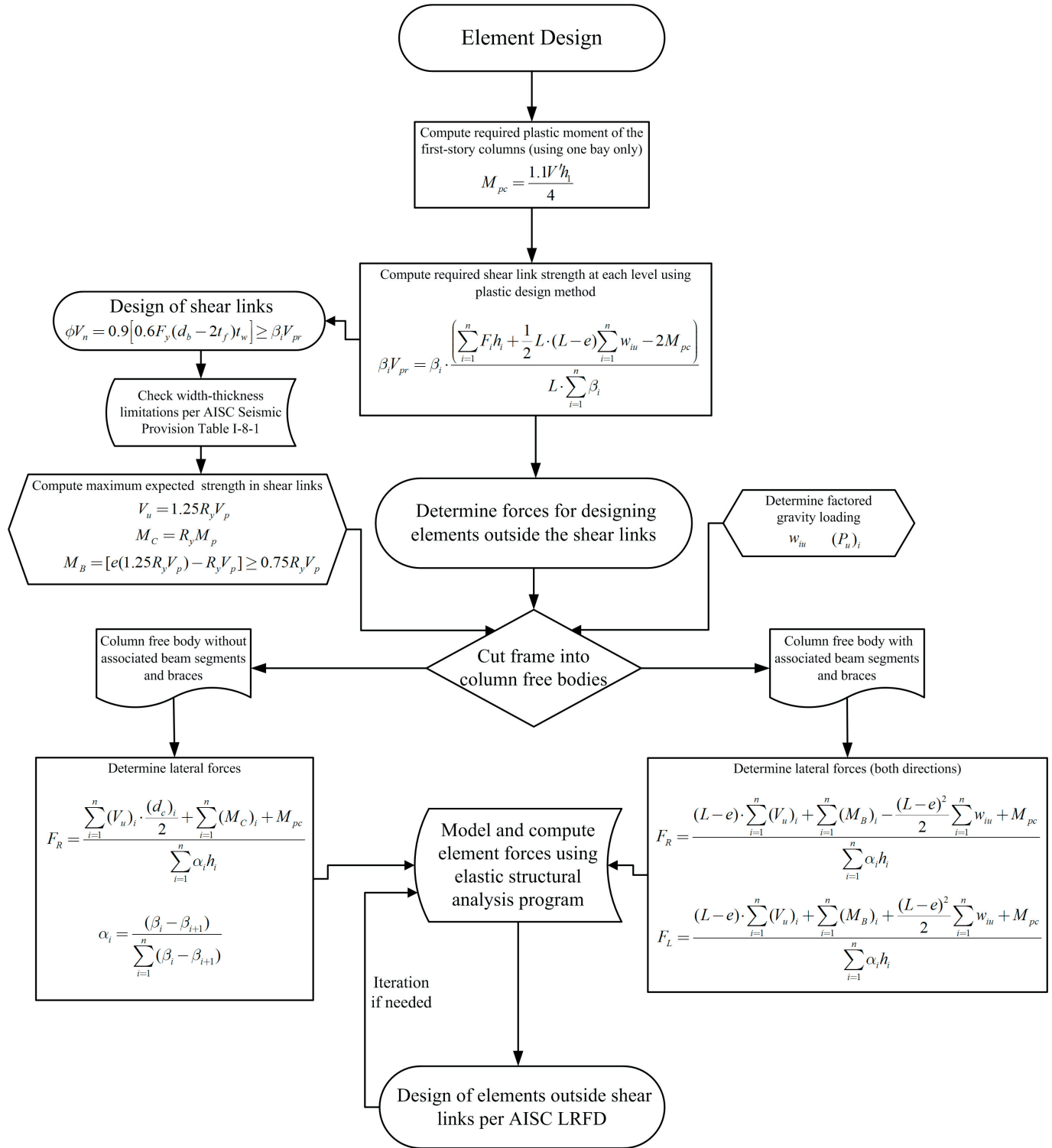


Fig. 8. Performance-based plastic design flowchart for EBFs: element design.

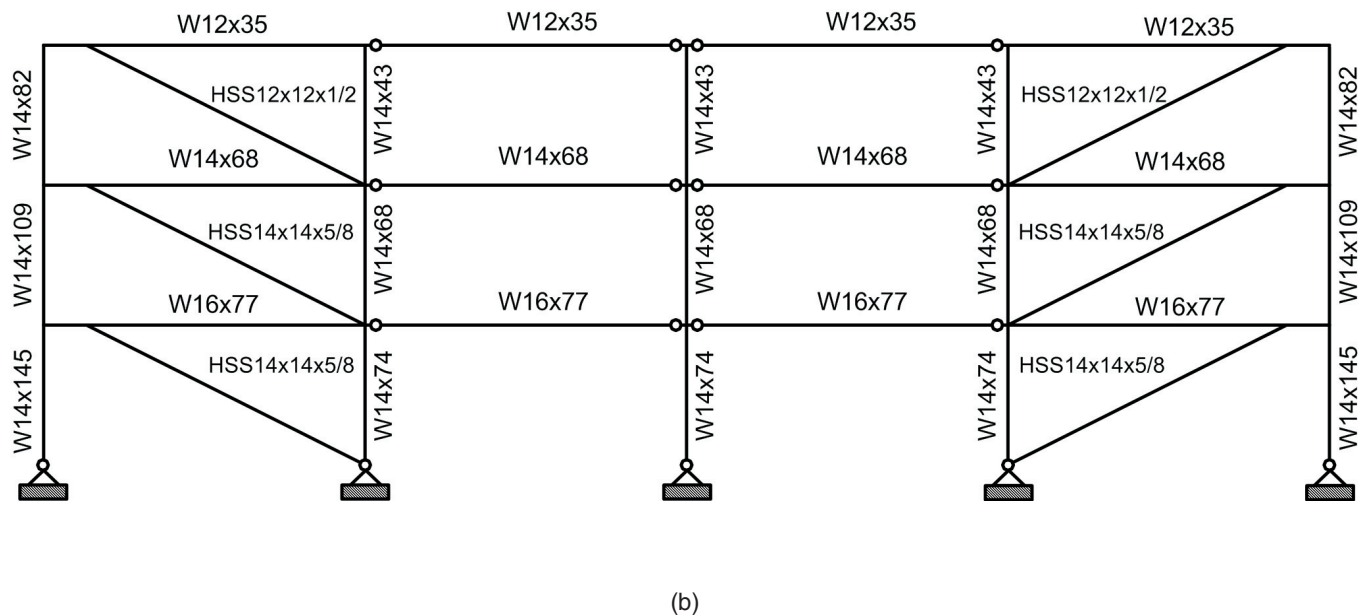
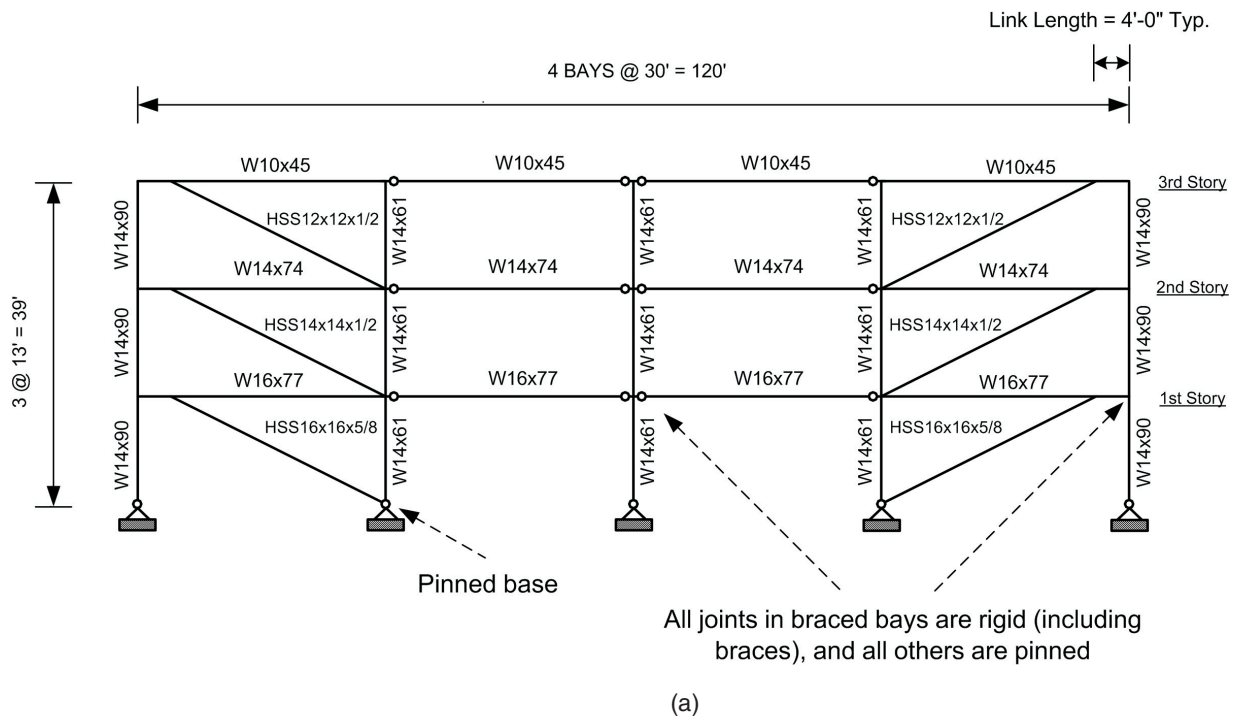
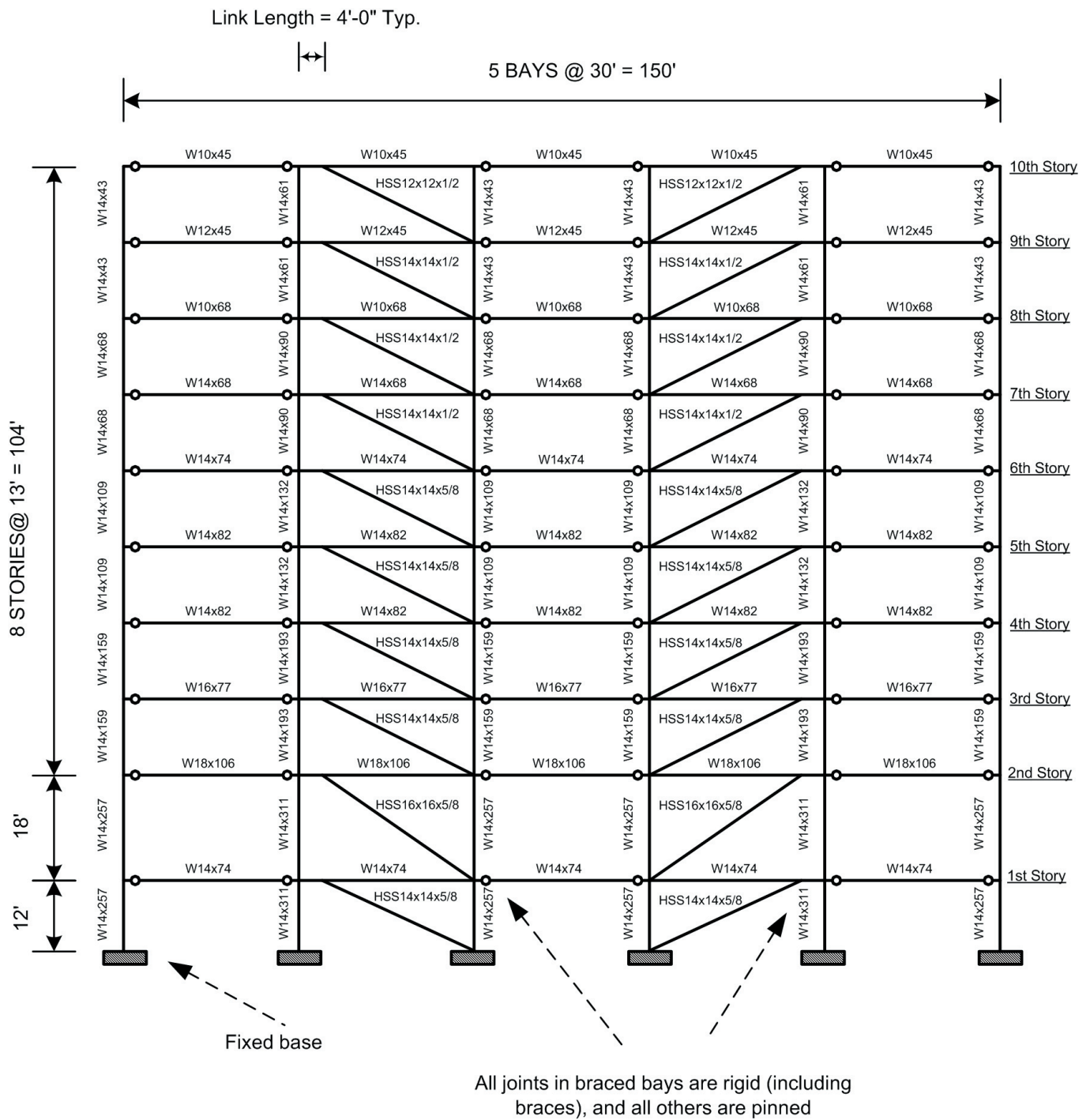
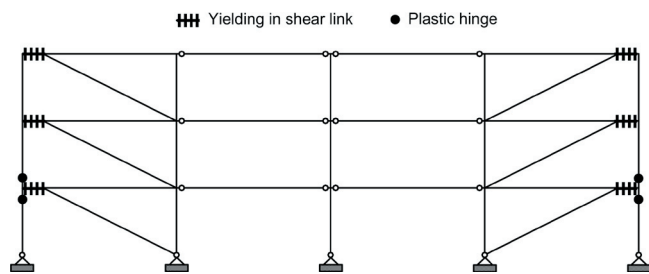


Fig. 9. Member sections of three-story EBFs designed based on (a) IBC 2000 approach and (b) proposed performance-based design approach.

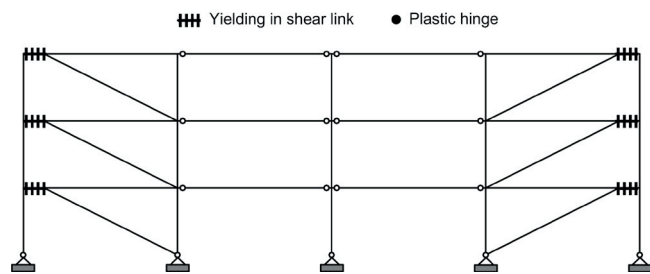


(a)

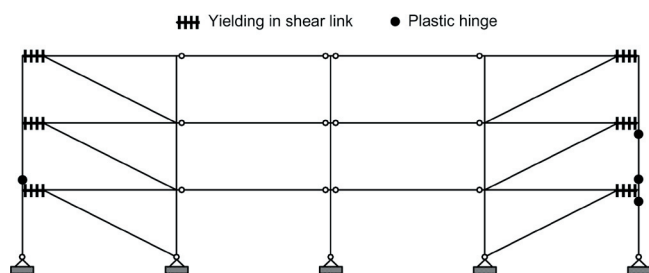
Fig. 10(a). Member sections of 10-story EBF designed based on IBC 2000 approach.



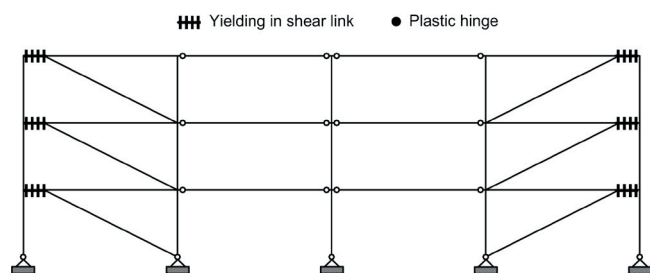
(a)



(b)



(c)



(d)

Fig. 11. Inelastic activity in three-story: (a) IBC EBF and (b) PBPD EBF during LA01 event (Imperial Valley, 1940, El Centro); (c) IBC EBF and (d) PBPD EBF during LA12 event (Loma Prieta, 1989, Gilroy).

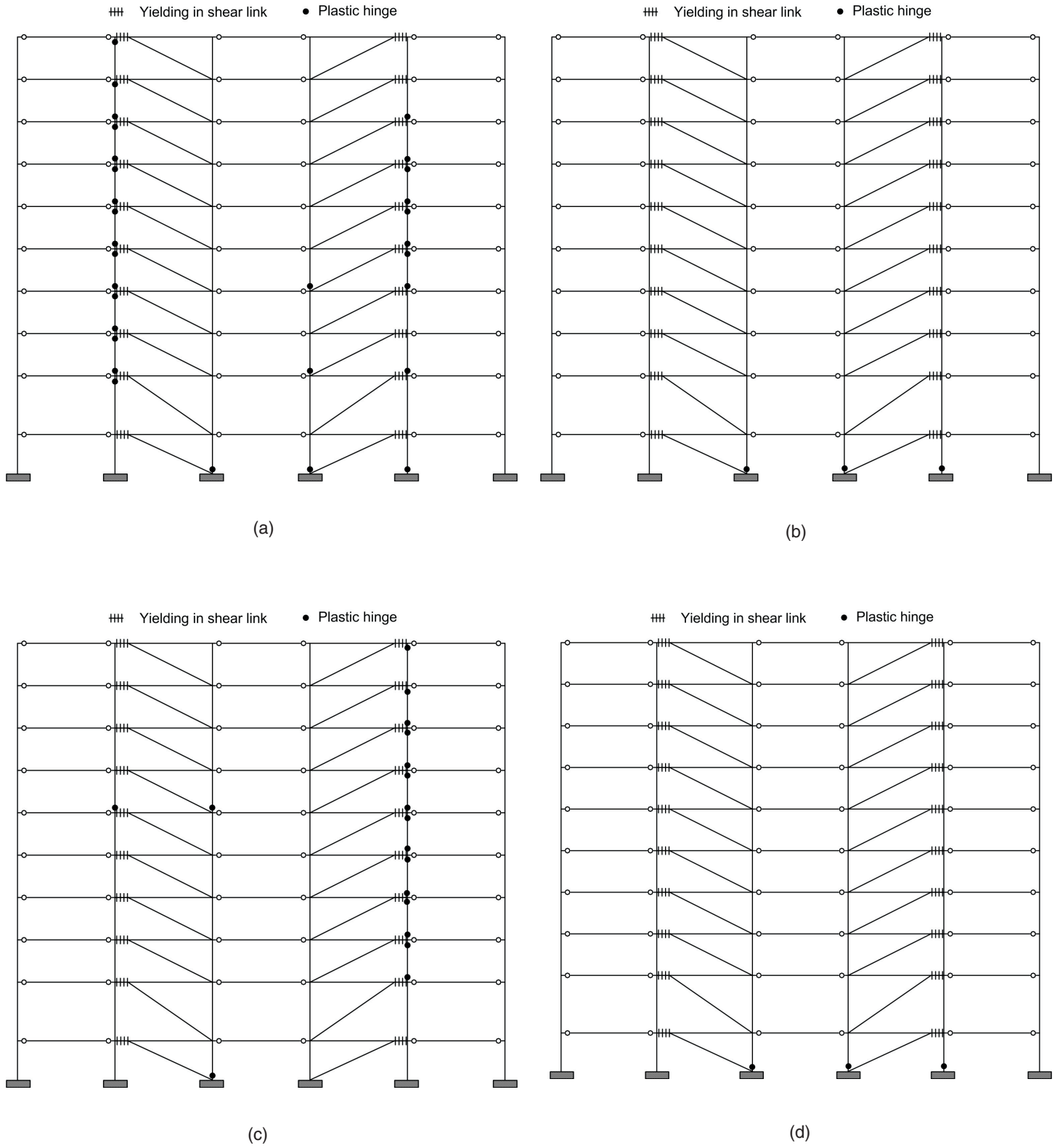
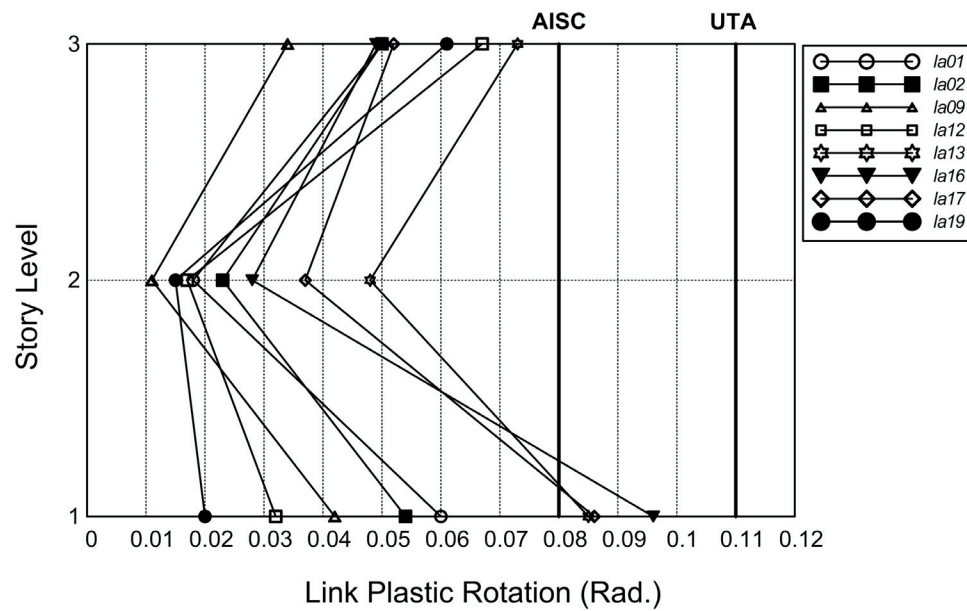
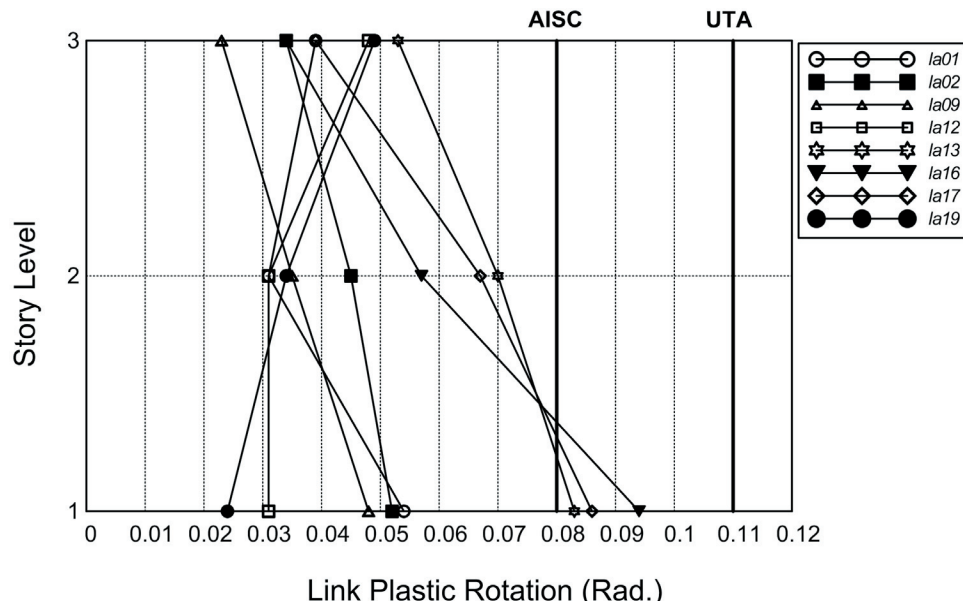


Fig. 12. Inelastic activity in 10-story: (a) IBC EBF and (b) PBPD EBF during LA09 event (Landers, 1992, Yermo);
c) IBC EBF and (d) PBPD EBF during LA17 event (Northridge, 1994, Sylmar).

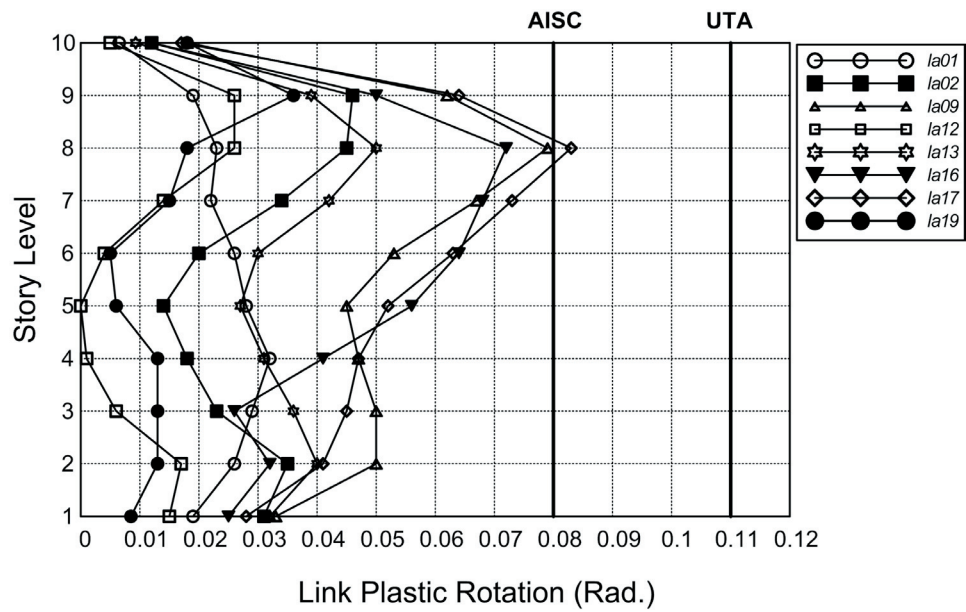


(a) IBC Frame

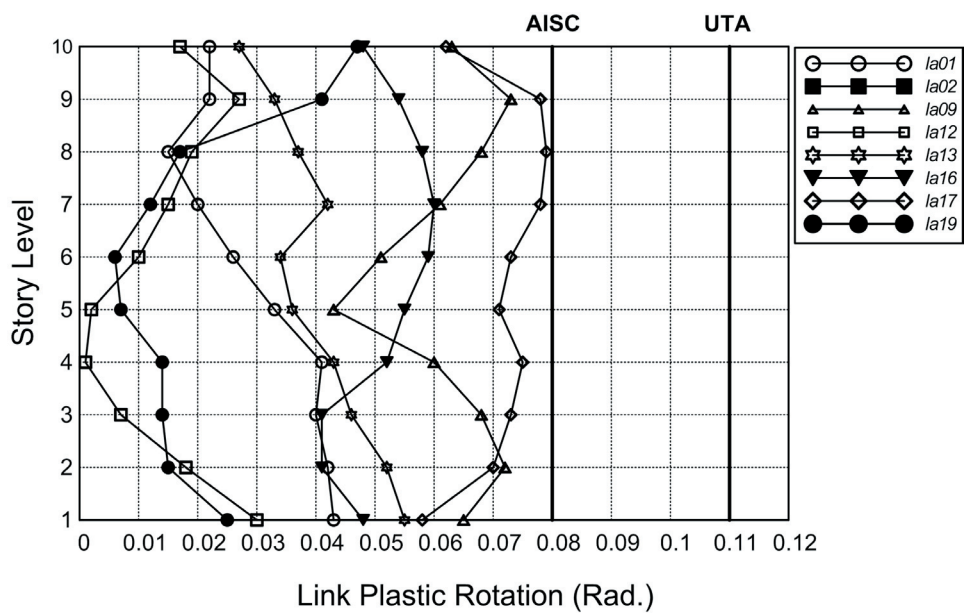


(b) PBPD Frame

Fig. 13. Maximum link plastic rotations in three-story (a) IBC EBF and (b) PBPD EBF.

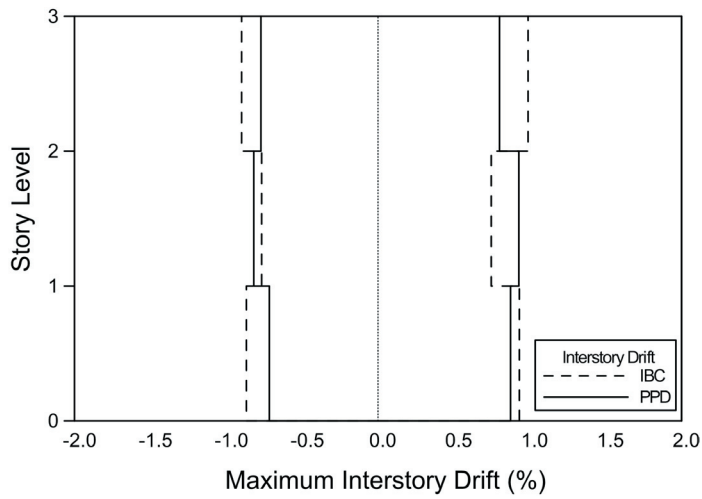


(a) IBC Frame

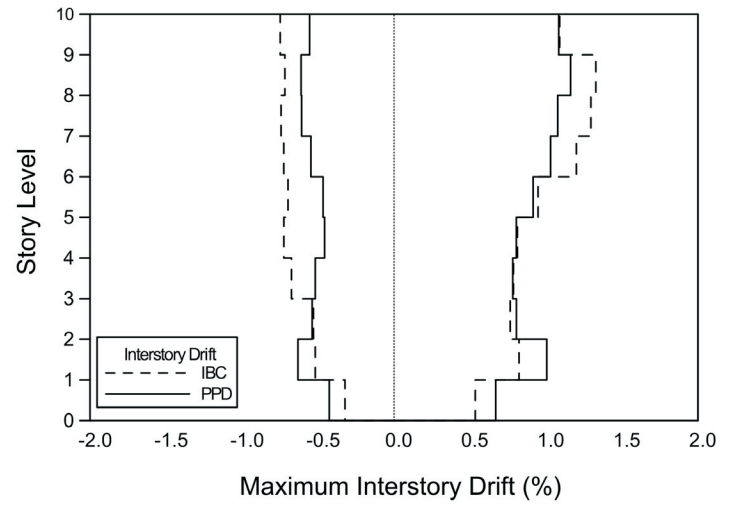


(b) PBPD Frame

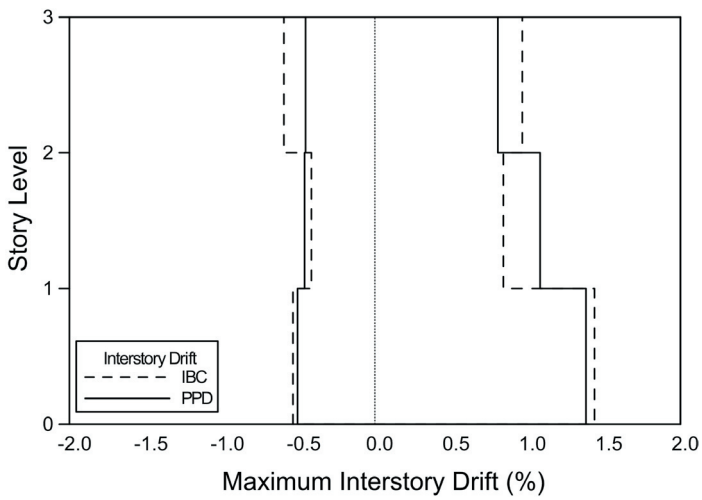
Fig. 14. Maximum link plastic rotations in 10-story (a) IBC EBF and (b) PBPD EBF.



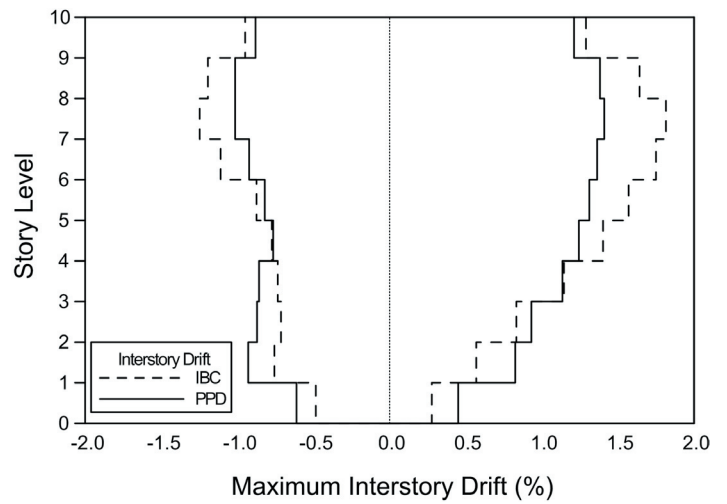
(a)



(b)

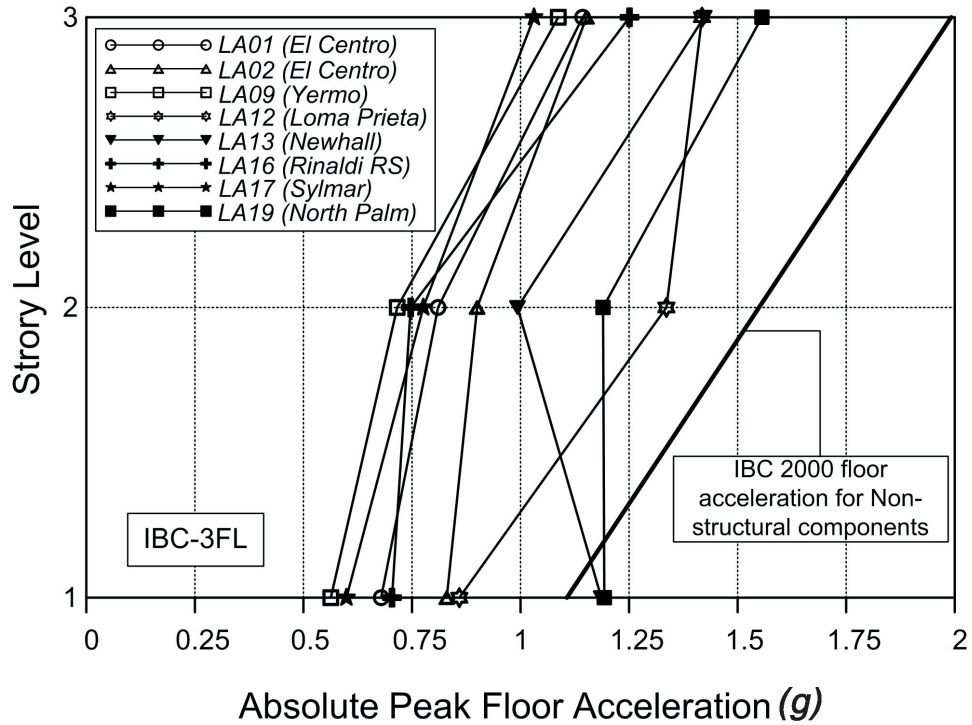


(c)

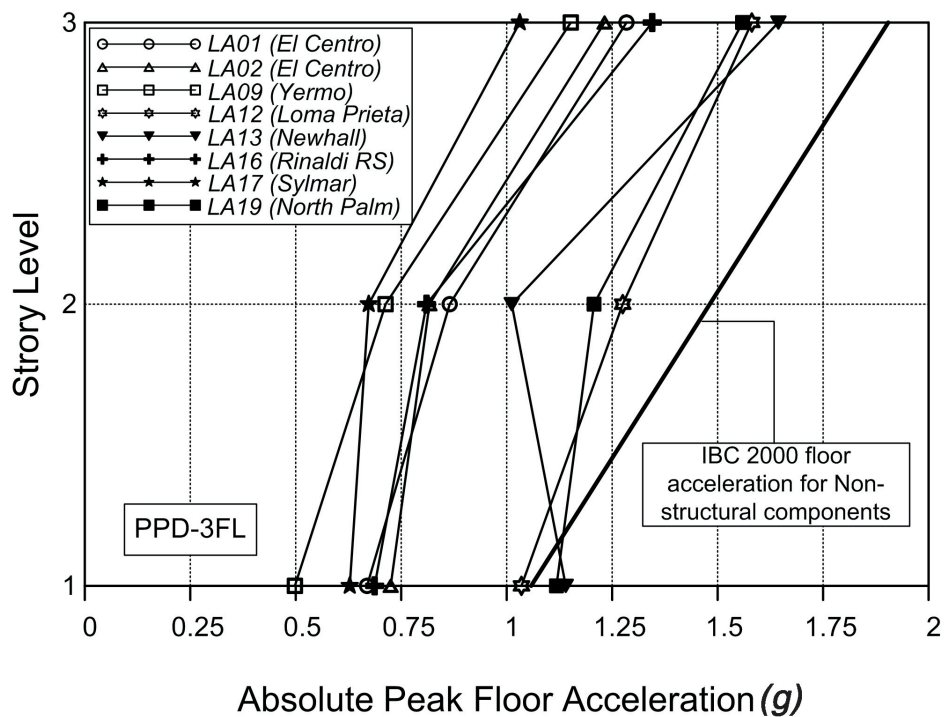


(d)

Fig. 15. Maximum interstory drift: (a) three-story frames; (b) 10-story frames during LA 02 event; (c) three-story frames; (d) 10-story frames during LA 16 event.

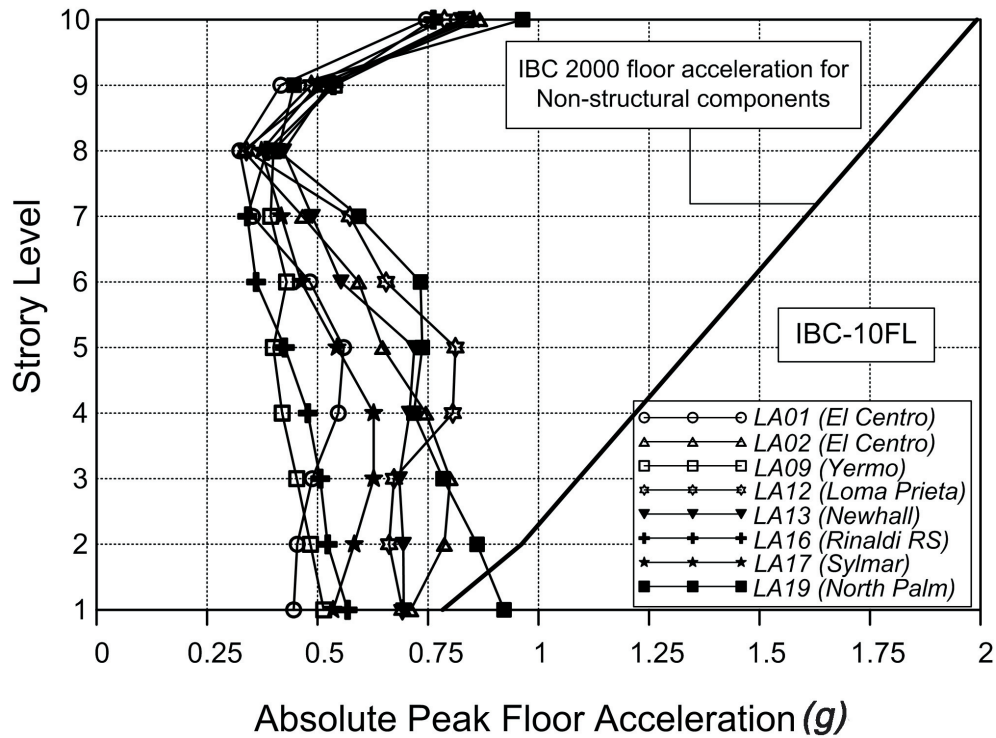


(a)

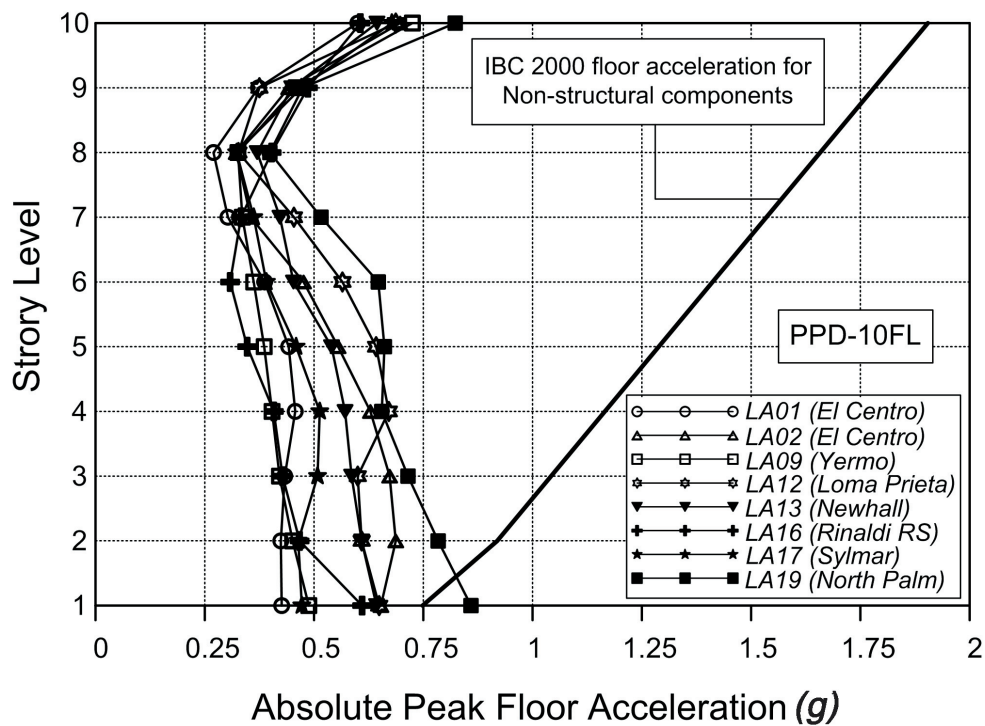


(b)

Fig. 16. IBC 2000 design acceleration for acceleration-sensitive nonstructural components and peak floor acceleration (10% in 50 years) occurred: (a) three-story IBC frame; (b) three-story PBPD frame.

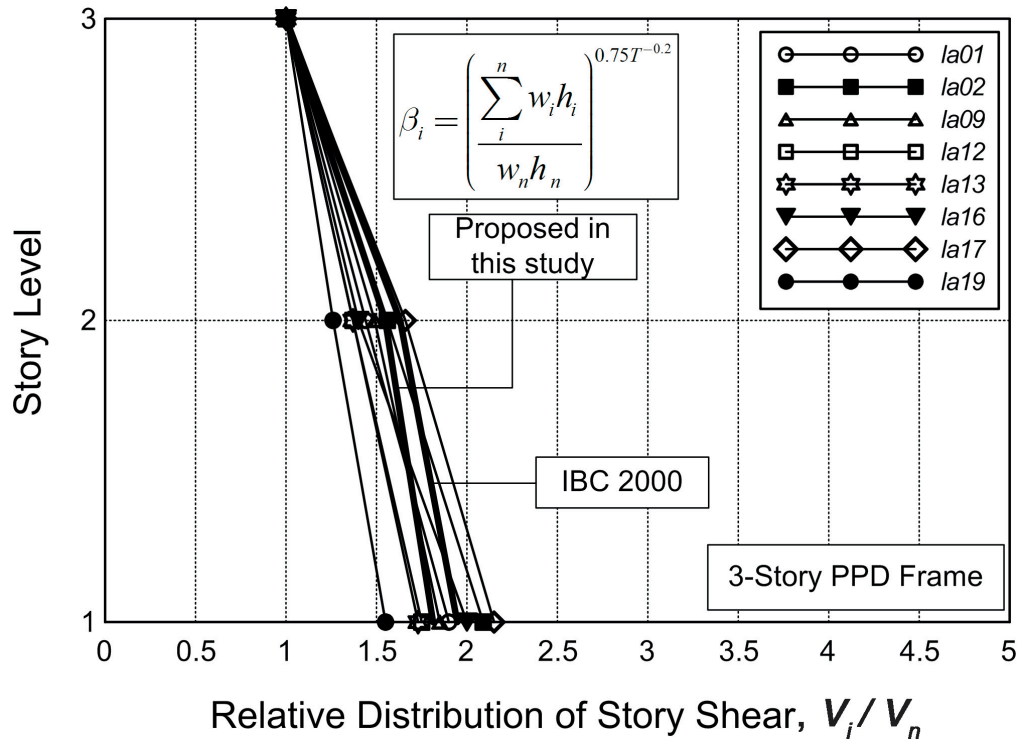


(a)

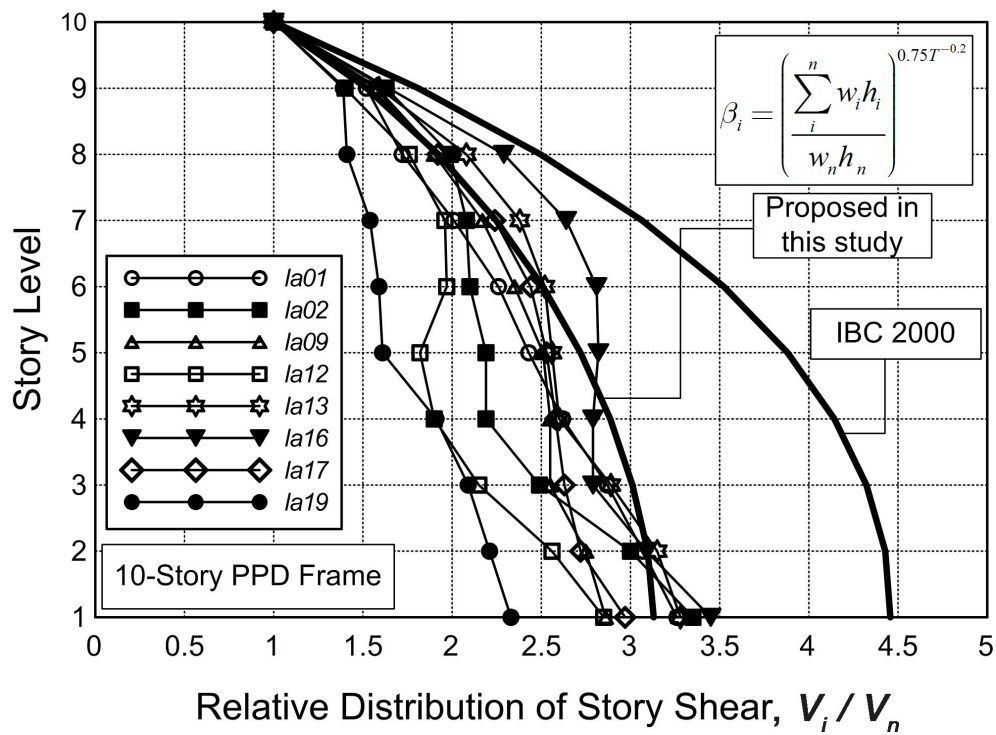


(b)

Fig. 17. IBC 2000 design acceleration for acceleration-sensitive nonstructural components and peak floor acceleration (10% in 50 years) occurred: (a) 10-story IBC frame; (b) 10-story PBPD frame.

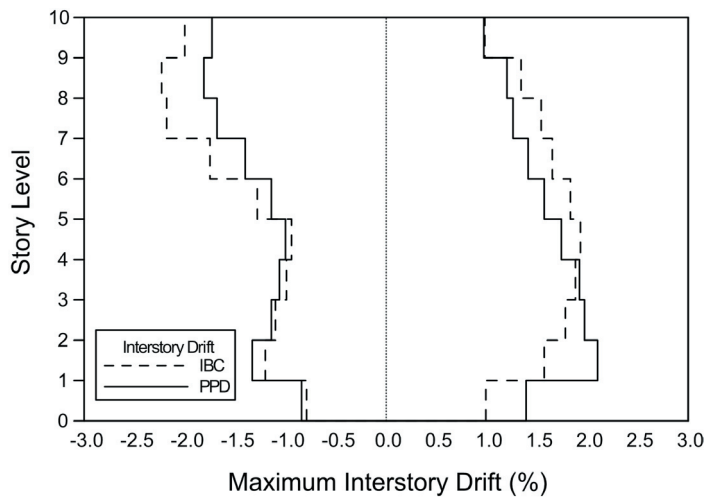


(a)

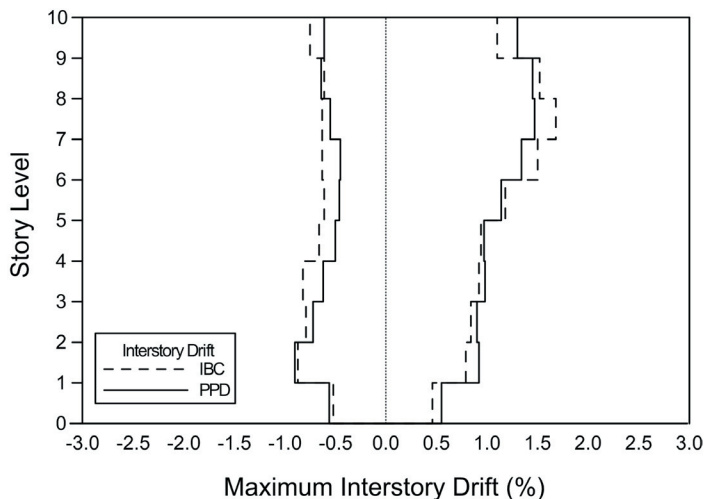


(b)

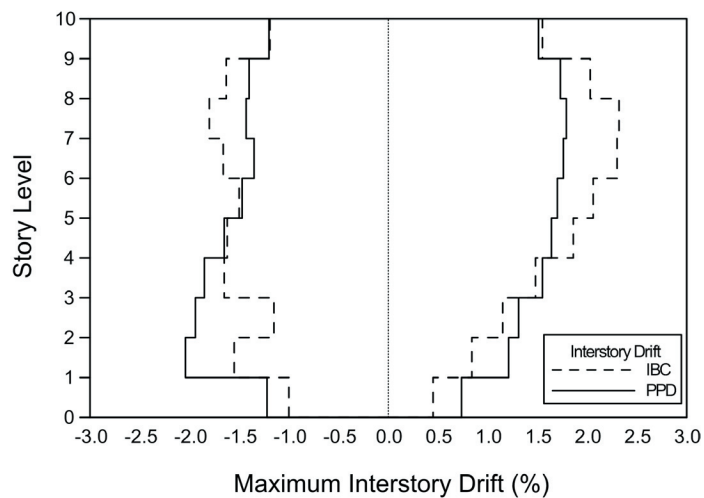
Fig. 18. Relative story shear distributions based on nonlinear dynamic analyses: (a) three-story PPD frame; (b) 10-story PPD frame.
Note: V_i is the story shear at i 'th level, and V_n is the story shear at top level.



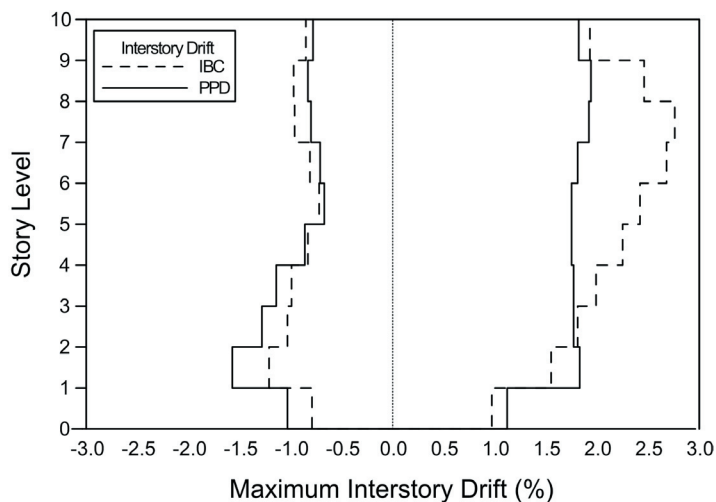
(a) LA21 event



(b) LA23 event



(c) LA26 event



(d) LA27 event

Fig. 19. Maximum interstory drift occurred in 10-story EBFs during 2% in 50 years earthquakes:
(a) LA21 event; (b) LA23 event; (c) LA26 event; (d) LA27 event.

Table 1. Comparison of Material Weight Between Two Three-Story Frames			
	IBC	PBPD	PBPD/IBC
Beam Weight (lb)	23,520	21,600	0.92
Column Weight (lb)	14,157	15,951	1.13
Brace Weight (lb)	17,005	17,202	1.01
Total Weight (lb)	54,682	54,753	1.00

Table 2. Comparison of Material Weight Between Two 10-Story Frames			
	IBC	PBPD	PBPD/IBC
Beam Weight (lb)	108,150	89,700	0.83
Column Weight (lb)	113,668	137,966	1.21
Brace Weight (lb)	59,951	57,963	0.97
Total Weight (lb)	281,769	285,629	1.01

Table 3a. Original Design Parameters for 10-Story PBPD Frames Based on IBC 2000	
Parameters	10-Story
S_s	2.380 g
S_1	0.840 g
F_a	1.000
F_v	1.500
S_{DS}	1.587 g
S_{D1}	0.840 g
Site Class	D
Seismic Use Group	I
Seismic Design Category	E
Building Height	134 ft
T_a	1.182
C_U	1.200
T	1.418
Importance Factor	1.00
Total Building Weight, W	15,787.83 kips
Design Base Shear, V	1,169.18 kips
$C_s = V/W$	0.074

Table 3b. Modified Design Parameters for PBPD Frames Using the Proposed Performance-Based Design Method	
Parameters	10-Story
C_e	0.559
T	1.600
Yield Drift, θ_y	0.5%
Target Drift, θ_u	2%
$\mu_s = \theta_u/\theta_y$	4
R_μ	4
γ	0.4375
α	1.491
V/W	0.086
Design Base Shear, V	1,358 kips

Note: All parameters were obtained based on Figure 7.

Table 4. Design Parameters for 10-Story PBPD Frame		
Level	Floor Weight (kips)	Applied Story Forces (kips)
10	1,660	434
9	1,560	226
8	1,560	174
7	1,560	140
6	1,560	114
5	1,560	92
4	1,560	72
3	1,560	54
2	1,560	37
1	1,560	15

Table 5. Design Link Sections and Corresponding Design Parameters					
Level	Link Section	$(V_u)_i = 1.25R_y(V_p)_i$ (kips)	Link End Moment at Column, $(M_B)_i$ (kip-ft)	Link End Moment at Brace, $(M_C)_i$ (kip-ft)	$\frac{b_f}{2t_f}^*$
10	W8×31	83.7	139.3	146.8	9.19
9	W10×45	127.9	251.6	188.7	6.47
8	W12×50	166.7	329.5	247.2	6.31
7	W14×53	191.8	399.2	299.4	6.11
6	W14×68	214.9	527.1	395.3	6.97
5	W14×74	234.3	577.5	433.1	6.41
4	W14×74	234.3	577.5	433.1	6.41
3	W16×67	243.8	605.0	453.8	7.70
2	W18×97	372.1	967.1	725.3	6.41
1	W10×39	115.1	214.5	178.7	7.53

* $0.38\sqrt{E_s / F_y} = 9.2$ per AISC (2005); all h/t_w values meet AISC requirements and are not shown here.

Table 6. Design Example of Columns with Beam Segments and Braces			
Level	Columns with Beam Segments and Braces		
	Gravity Loads on Beam Segment, w_{iu} (kip/ft)	Column Axial Force, $(P_u)_i$ (kips)	Updated Lateral Force, $\alpha_i F_L$ (kips)
10	0.877	17.84	182.89
9	0.957	23.71	95.32
8	0.957	23.71	73.50
7	0.957	23.71	59.14
6	0.957	23.71	48.02
5	0.957	23.71	38.66
4	0.957	23.71	30.37
3	0.957	23.71	22.78
2	0.957	23.71	15.64
1	0.957	25.51	6.20

Table 7. Design Example of Columns Without Beam Segments and Braces			
Level	Columns with Beam Segments and Braces		
	$(\Delta M)_i$ (kip-ft)*	Column Axial Force, $(P_u)_i$ (kips)	Updated Lateral Force, $\alpha_i F_R$ (kips)
10	48.85	30.99	21.13
9	74.59	38.06	11.01
8	97.21	38.06	8.49
7	111.89	38.06	6.83
6	125.37	38.06	5.55
5	136.68	38.06	4.47
4	136.68	38.06	3.51
3	142.21	38.06	2.63
2	217.04	38.06	1.81
1	67.13	39.86	0.72

*See Figure 6(b).