Performance-Based Plastic Design of Special Truss Moment Frames

SHIH-HO CHAO and SUBHASH C. GOEL

The special truss moment frame (STMF) is a relatively new type of steel framing system suitable for high seismic areas. The frames dissipate earthquake energy through ductile special segments located near the mid-span of truss girders. STMFs generally have higher structural redundancy compared to other systems because four plastic hinges can form in the chords of one truss girder. The redundancy can be further enhanced if web members are used in the special segments. Simple connection details are needed for the girder-to-column connections. One other advantage of using STMF systems is that the truss girders can be used over longer spans, and greater overall structural stiffness can be achieved by using deeper girders. In addition, the open webs can easily accommodate mechanical and electrical ductwork. As a consequence, this system is gaining popularity in the United States, especially for hospital and commercial buildings. Research work carried out at the University of Michigan led to the formulation of design code provisions (Goel and Itani, 1994; Basha and Goel, 1995; AISC, 2005a). Current design practice generally follows elastic analysis procedures to proportion the frame members. Therefore, it is possible that story drifts and yielding in the special segments may not be uniformly distributed along the height of the structure and may be concentrated in a few floors causing excessive inelastic deformations at those levels. Thus, the intended deformation limits and yield mechanism may not be achieved when an STMF is subjected to strong earthquakes.

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In recent years, seismic design has been gradually moving toward a more direct performance-based design approach, which is intended to produce structures with predictable and controlled seismic performance. To achieve this goal, knowledge of the ultimate structural behavior, such as nonlinear relationships between forces and deformations, and the yield mechanism of structural systems are essential. Therefore, the desired global yield mechanism needs to be built into the design process. In current practice, the performance-based seismic design for a new structure is carried out in a somewhat indirect manner. It usually starts with an initial design according to conventional elastic design procedures using applicable design codes, followed by a nonlinear static pushover assessment (ATC, 1996; ASCE, 2000). Usually, an iterative process between design and assessment follows. Moreover, as mentioned in FEMA 440 (ATC, 2004), this procedure still has difficulty in predicting reasonably accurate structural behavior during a major earthquake when compared with the results from nonlinear response history analyses.

While further refinement is needed in 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 (pushover) or dynamic analysis after design. Based on an energy (work) concept (Leelataviwat, Goel and Stojadinović, 1999; Lee, Goel and Chao, 2004; Chao and Goel, 2005), the proposed approach determines the 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 target drift. In addition, the design lateral force distribution employed in the proposed method is based on nonlinear dynamic analysis results using a large number of ground motions (Somerville, Smith, Punyamurthula and Sun, 1997). The chord members in the special segments are designed according to the plastic design method (Chao and Goel, 2006), while the members outside the special segments are designed by using a capacity design approach. A complete detailed design procedure was developed and the design steps are summarized in a flowchart in Figures 10 and 11, which could be easily applied to develop a computer program.

PROPOSED PERFORMANCE-BASED PLASTIC DESIGN PROCEDURE

Design Lateral Forces

Unlike in the current design codes, the design lateral force distribution in the proposed method is determined based on maximum story shears that are obtained from extensive nonlinear time-history analyses. The proposed design lateral force distribution was found suitable for moment frames (MFs), eccentrically braced frames (EBFs), concentrically braced frames (CBFs), and special truss moment frames (STMFs) (Chao, Goel and Lee, 2007). This lateral force distribution is expressed as:

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

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

$$\beta_i = \frac{V_i}{V_n} = \left(\frac{\sum_{j=i}^{n} w_j h_j}{w_n h_n}\right)^{0.75T^{-0.2}} \tag{3}$$

where

β_i = shear distribution factor at level i
V_i, V_n = story shear forces at level i and at the top (nth) level, respectively
w_i = seismic weight at level j
h_j = height of level j from the ground
w_n = seismic weight of the structure at the top level
h_n = height of roof level from ground
T = fundamental natural period
F_i, F_n = lateral forces applied at level i and top level n, respectively
V = design base shear

Design Base Shear

Design base shear in current seismic codes is commonly obtained by reducing the elastic strength demands to the inelastic strength demands by using 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 code prescribed design acceleration spectrum and expressed in the following form:

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

where

C_s = seismic response coefficient calculated based on specified design spectrum
S_a = spectral response acceleration
I = occupancy importance factor
R = response modification factor [= 7.0 for STMF (NEHRP, 2000)]
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 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 reliably predicted.

A more rational design approach to overcome the aforementioned shortcomings was proposed based on energy equation 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 choice of target drift will generally be based on acceptance criteria, such as damage or life safety. For example, a maximum transient story drift ratio of 2.5% is specified in FEMA 356 (FEMA, 2000) to meet the life safety performance level for steel moment frames. The amount of work needed to do that is assumed as a factor γ times the elastic input energy $E = ½MS_v^2$ for an equivalent single degree of freedom (SDOF). 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. Thus, the energy equation can be written as:

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

where

E_e, E_p = elastic and plastic components of the energy (work) needed to push the structure up to the target drift, respectively
S_v = design spectral 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 period of the structure. Figure 1 shows the relationship between the base shear, CW, and the corresponding drift, Δ, of the elastic and corresponding elastic-plastic

SDOF systems. From the geometric relationship, Equation 5 can be written as:

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

where

C_{eu} = seismic response coefficient corresponding to the maximum base shear when the structure behaves essentially elastically

C_y = seismic response coefficient corresponding to the maximum base shear when the structure demonstrates elastic-perfect-plastic behavior

Δ_y = yield drift

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

$$\gamma\,\frac{\Delta_{eu}}{\Delta_y} = \frac{\left(2\Delta_{max} - \Delta_y\right)}{\Delta_{eu}} \tag{7}$$

where

Δ_{eu} = $R_\mu\Delta_y$ from Figure 1

Δ_{max} = $\mu_s\Delta_y$ from Figure 1

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

$$\gamma = \frac{2\mu_s - 1}{R_\mu^2} \tag{8}$$

where

μ_s = ductility factor equal to target drift divided by yield drift = Δ_{max}/Δ_y

R_μ = ductility reduction factor = C_{eu}/C_y

It can be seen from Equation 8 that the energy modification factor, γ, is a function of the ductility reduction factor, R_μ, and the structural ductility factor, μ_s. In this study, the method 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:

$$A = S_a g \tag{9}$$

where

A = design pseudo-acceleration

g = acceleration due to gravity

S_a = spectral response 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, increases the design force level in an attempt to lower the drift and ductility demands for the structure for a given level of ground shaking (SEAOC, 1999; NEHRP, 2000). However, that cannot be considered as a direct method to achieve the intended purpose such as

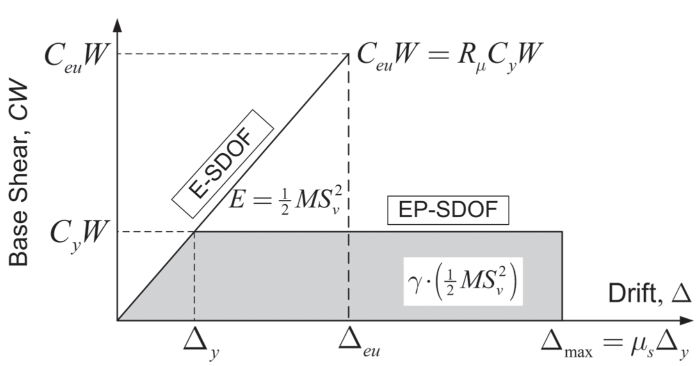


Fig. 1. Structural idealized response and energy balance concept.

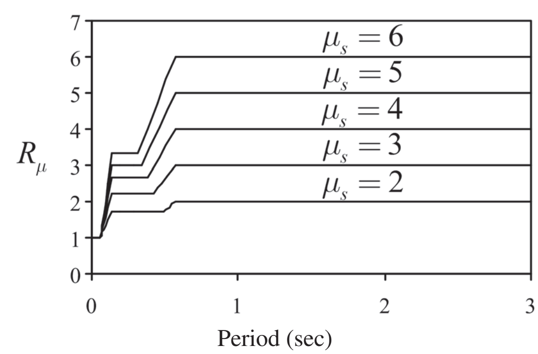


Fig. 2. Ductility reduction factors proposed by Newmark and Hall (1982).

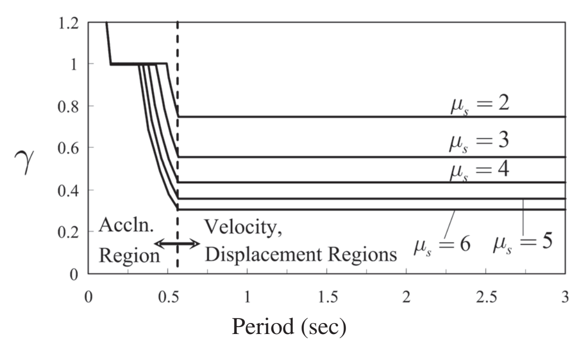


Fig. 3. Modification factors for energy equation vs. period.

damage control. The reduction of potential damage could 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 have the occupancy importance factor built into it. In any case, S_a 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.

The energy (work) equation can be rewritten as:

$$\left(E_e + E_p\right) = \gamma\left(\frac{1}{2}MS_v^2\right) = \frac{1}{2}\gamma M\left(\frac{T}{2\pi}S_a g\right)^2 \tag{10}$$

Akiyama (1985) showed that the elastic vibrational energy, E_e, can be calculated by assuming that the entire structure can be reduced into an SDOF system, in other words,

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

where

V = yield base shear

W = total seismic weight of the structure = Mg

Substituting Equation 11 into Equation 10 and rearranging the terms gives:

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

Using a preselected yield mechanism for the STMF configurations shown in Figure 4 and equating the plastic energy term, E_p, to the external work done by the design lateral forces gives:

$$E_p = \sum_{i=1}^{n} F_i h_i \theta_p \tag{13}$$

where

θ_p = inelastic drift of the global structure, which is the difference between the selected target drift, θ_u, and yield drift, θ_y

Based on nonlinear analyses, the yield drift for STMF can be taken as 0.75% for design purposes (Chao and Goel, 2006). Substituting Equations 1 and 12 into Equation 13, and solving for V/W gives:

$$\frac{V}{W} = \frac{-\alpha + \sqrt{\alpha^2 + 4\gamma S_a^2}}{2} \tag{14}$$

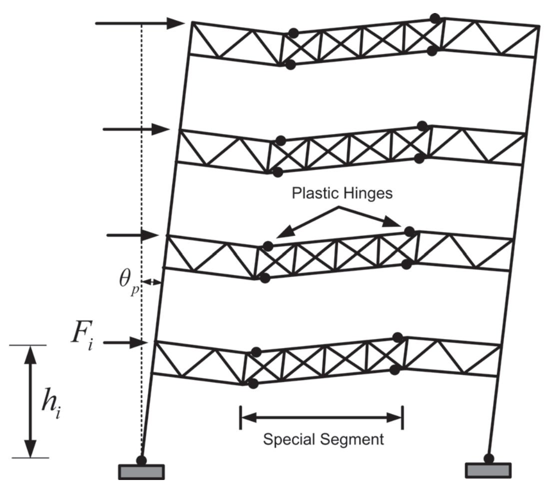
where

V = design base shear

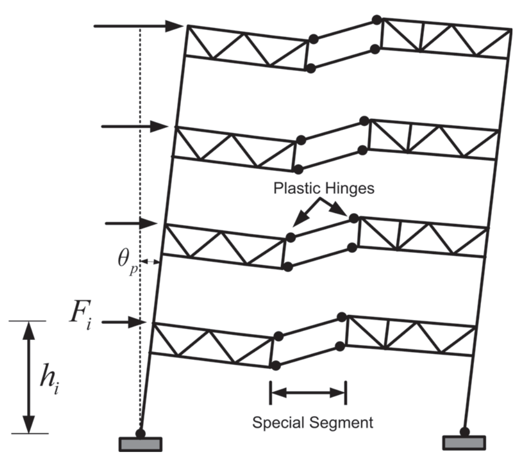
α = dimensionless parameter, which depends on the stiffness of the structure, the modal properties, and the intended drift level, and is given by:

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

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



(a)



(b)

Fig. 4. Preselected yield mechanism of STMF with various geometries: (a) with X-diagonal web members in special segment; (b) without web members in special segment (Vierendeel type).

It can be seen that the proposed method for determining the 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), as required in current practice and 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_yW in Figure 1) at which a 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 STMFs subjected to design lateral forces and pushed to its target drift limit state. All the inelastic deformations are intended to be confined within the special segments in the form of plastic hinges (and yielding and buckling of the diagonal web members when they are used). Because the plastic hinges at column bases generally form during a major earthquake, the desired global yield mechanism of STMF is achieved through yielding (due to shear force) of the special segments plus the plastic hinges at the column bases.

Proportioning of Chord Members of the Special Segments

In this study, STMFs with a Vierendeel-type special segment (see Figure 4b) are used to demonstrate the proposed plastic design approach. The primary aim of using a plastic design procedure is to ensure the formation of the intended yield mechanism. For STMFs, the plastic hinges are intended to occur only in the special segments and column bases. Earlier studies have shown that it is desirable to have the distribution of chord member strength along the building height closely follow the distribution of story shears, in other words, the shear distribution factor, β_i, which was obtained and calibrated by nonlinear response history analysis results. This helps to distribute the yielding more evenly along the height, thereby, preventing yielding from concentrating at a few levels. Referring to Figure 5, only one bay of an STMF is shown for illustration of the design procedure. It should be noted that using all the bays gives the same required chord strength.

By using the principle of virtual work and equating the external work to the internal work, as is commonly done in plastic analysis by the mechanism method, the required chord member strength at each level can be determined as:

$$\sum_{i=1}^{n} F_i h_i \theta_p = 2M_{pc}\theta_p + 4\sum_{i=1}^{n} \beta_i M_{pbr} \frac{L}{L_s}\theta_p \tag{16}$$

where

L = span length of truss girders

L_s = length of special segment (see Figure 5)

M_{pbr} = required plastic moment of chord members at the top level and the only unknown variable in Equation 16

The required chord member strength (plastic moment capacity) at level i can be determined by multiplying M_{pbr} by the shear distribution factor β_i at level i, namely, $\beta_i M_{pbr}$. M_{pc} is the required plastic moment of columns in the first story as shown in Figure 5. Note that due to the selected deformation mechanism of the truss girder (as shown in Figures 4 and 5) and assuming uniformly distributed gravity loads, the external work done by the gravity loads is zero, and thus not included in Equation 16.

M_{pc} is the required plastic moment of columns in the first story, which can be selected 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), as shown in Figure 6:

$$M_{pc} = \frac{1.1V'h_1}{4} \tag{17}$$

where

V' = base shear (for an equivalent one-bay frame), which is equal to V divided by the number of bays

h_1 = height of the first story

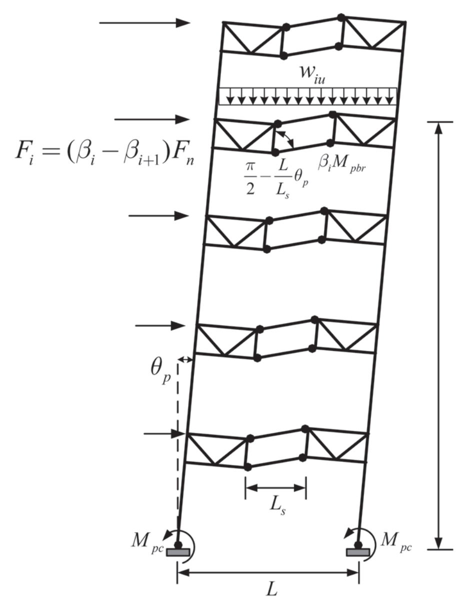


Fig. 5. One-bay STMF with preselected yield mechanism for calculating required strength of chord member. Note that the values of F_i and F_n are for one bay only.

The factor 1.1 is the overstrength factor accounting for possible overloading due to stain hardening and uncertainty in material strength.

By using Equations 16 and 17, the required chord member strength at level i can be determined as:

$$\beta_i M_{pbr} = \beta_i \frac{\left(\sum_{i=1}^{n} F_i h_i - 2M_{pc}\right)}{4\frac{L}{L_s}\sum_{i=1}^{n}\beta_i} \tag{18}$$

The design of the chord member is performed using:

$$\phi M_{nci} = \phi Z_i F_y \geq \beta_i M_{pbr} \tag{19}$$

where

ϕ = resistance factor = 0.90
Z_i = plastic section modulus
F_y = specified minimum yield stress (taken equal to 50 ksi in this study)

Chord members should also satisfy the specified width-thickness limitations as listed in the AISC *Seismic Provisions for Structural Steel Buildings* Table I-8-1 (AISC, 2005a), hereafter referred to as the AISC Seismic Provisions. Design recommendations on Vierendeel special segments with intermediate vertical members can be found elsewhere (Chao and Goel, 2006).

Design of Members Outside the Special Segments

The design of elements outside the special segments, including truss members and columns, is performed based on the capacity design approach. That is, elements outside the special segments should have a design strength to resist the combination of factored gravity loads and the maximum expected vertical shear strength, V_{ne}, developed at the midpoint of the special segments. The expression for V_{ne}, as specified in the AISC Seismic Provisions [Equation 12-1, (AISC, 2005a)], was originally derived based on a somewhat conservative chord deformation assumption. For members with a larger moment of inertia, such as the double-channel sections employed in this study, the AISC expression generally leads to quite a conservative design for members outside the special segments. In view of this, a modification has been proposed by Chao and Goel (2006) and was used in this study. For STMF with a special segment of the Vierendeel configuration:

$$V_{ne} = \frac{3.6R_y M_{nc}}{L_s} + 0.036E_s I_s \frac{L}{L_s^3} \tag{20}$$

where

R_y = yield overstrength factor (taken as 1.1)
M_{nc} = nominal flexural strength of the chord members of the special segment
E_s = Young's modulus (= 29,000 ksi)
I_s = moment of inertia of the chord members of the special segment

Once the maximum expected vertical shear strength is determined, the frame can be broken into free-body diagrams of exterior and interior columns with associated truss girders as shown in Figures 7, 8 and 9.

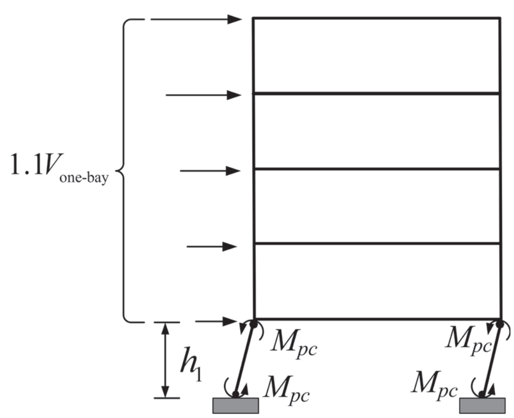


Fig. 6. One-bay equivalent frame with soft-story mechanism (Leelataviwat et al., 1999).

Design of Columns with Associated Truss Girders

Referring to Figure 7a, when the frame reaches its target drift, the vertical shear force in the middle of the special segment at all levels is assumed to reach the maximum expected strength, $(V_{ne})_i$. The column at the base 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 equilibrium of the free body. For the case when the lateral forces are acting to the right, the sum of those forces, $(F_R)_{ext}$, can be obtained as:

$$(F_R)_{ext} = \frac{\frac{L}{2}\sum_{i=1}^{n}(V_{ne})_i - \frac{L^2}{8}\sum_{i=1}^{n} w_{iu} + M_{pc}}{\sum_{i=1}^{n}\alpha_i h_i} \tag{21}$$

where

w_{iu} = factored uniformly distributed gravity load on the truss girders = 1.2*DL* + 0.5*LL* in this study

and

$$\alpha_i = \frac{F_i}{\sum_{i=1}^{n} F_i} = \frac{(\beta_i - \beta_{i+1})}{\sum_{i=1}^{n}(\beta_i - \beta_{i+1})} \text{ when } i = n,\ \beta_{n+1} = 0 \tag{22}$$

For the case when the lateral forces are directed to the left (Figure 7b), the sum of the applied lateral forces, $(F_L)_{ext}$, can be obtained as:

$$(F_L)_{ext} = \frac{\frac{L}{2}\sum_{i=1}^{n}(V_{ne})_i + \frac{L^2}{8}\sum_{i=1}^{n}w_{iu} + M_{pc}}{\sum_{i=1}^{n}\alpha_i h_i} \tag{23}$$

For the case where the gravity loading on truss girders consists of concentrated loads, Equation 21 can be replaced by Equation 24 (see Figure 8):

$$(F_R)_{ext} = \frac{\frac{L}{2}\sum_{i=1}^{n}(V_{ne})_i - L_1\sum_{i=1}^{n}p_{iu} + M_{pc}}{\sum_{i=1}^{n}\alpha_i h_i} \tag{24}$$

where

p_{iu} = factored concentrated gravity loads on the truss girders = $1.2DL + 0.5LL$, as shown in Figure 8

Also Equation 23 can be replaced by:

$$(F_L)_{ext} = \frac{\frac{L}{2}\sum_{i=1}^{n}(V_{ne})_i + L_1\sum_{i=1}^{n}p_{iu} + M_{pc}}{\sum_{i=1}^{n}\alpha_i h_i} \tag{25}$$

It should be mentioned that although the column design has been carried out by using column trees, the overturning of the entire frame is accounted for through beam shear forces applied on the column trees.

For the case of interior columns with associated truss girders, both directions of lateral forces lead to the same result;

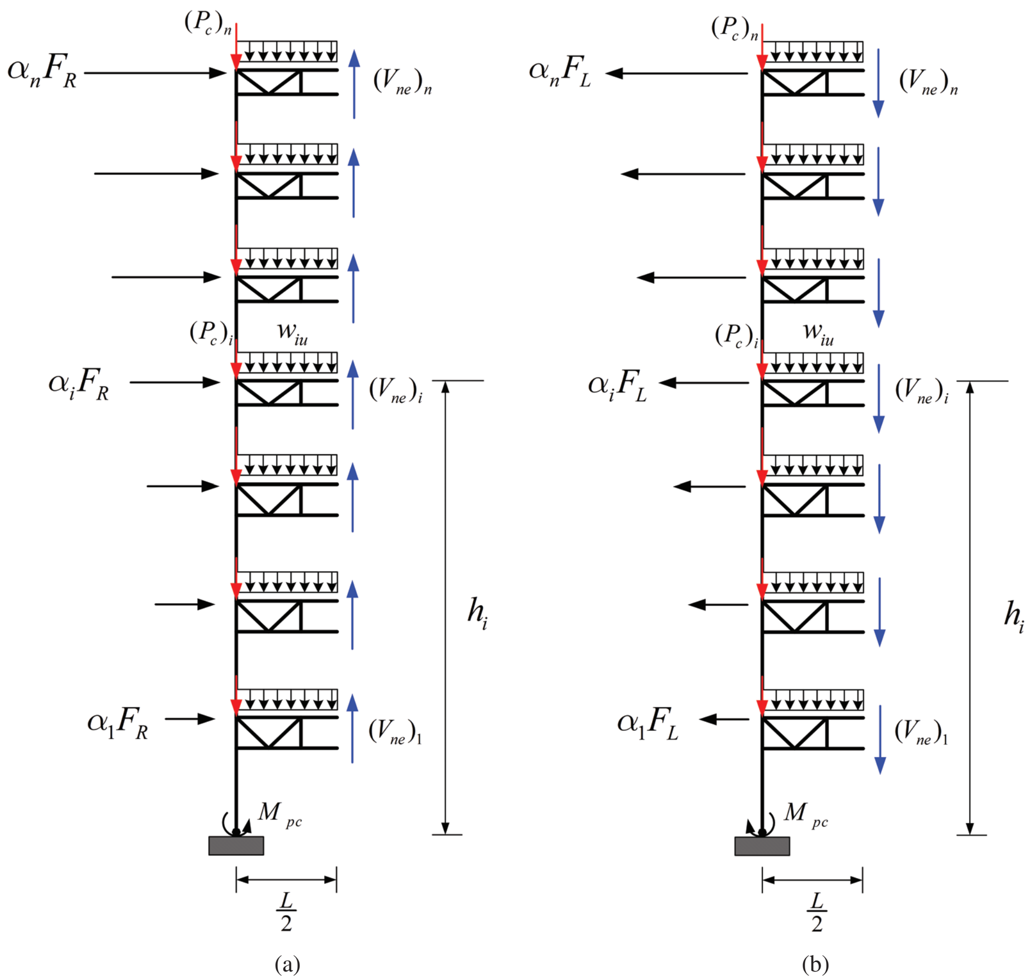


Fig. 7. Free-body diagram of an exterior column and associated truss girder branches: (a) lateral forces acting to right side; (b) lateral forces acting to left side.

hence only the lateral forces acting to the right are shown in Figure 9. The sum of lateral forces, $(F_R)_{int}$, can be calculated as:

$$(F_R)_{int} = \frac{L\sum_{i=1}^{n}(V_{ne})_i + 2M_{pc}}{\sum_{i=1}^{n}\alpha_i h_i} \tag{26}$$

After the lateral forces required for equilibrium are calculated as described earlier, the required strength of individual members (diagonals, chord members, columns and vertical members, if any) can be easily computed by using an elastic structural analysis program. Preliminary sections can be assumed in the beginning and revised later. The terms $\alpha_i F_R$, $\alpha_i F_L$, $(V_{ne})_i$, $(P_c)_i$, p_{iu} and w_{iu}, which appear in the preceding equations and in Figure 9, are treated as applied loads. Design of these elements is performed in accordance with the AISC *Specification for Structural Steel Buildings*, ANSI/AISC 360-05 (AISC, 2005b) through conventional elastic design procedures. For STMF with hinged bases, the column-truss models may be structurally unstable when loaded because they are basically determinate cantilever beams. Nevertheless, the hinge support can be replaced with a fixed support for computing the element forces without affecting the results because all the external forces are already balanced and moment at the column base will automatically be zero (because of hinged supports). The vertical members adjacent to the special segment are recommended to have the same section as the chord member in the special segment without performing any design calculations (Basha and Goel, 1994). However, a stronger section can be used, if needed. Flowcharts are given in Figures 10 and 11 to illustrate the entire design procedure.

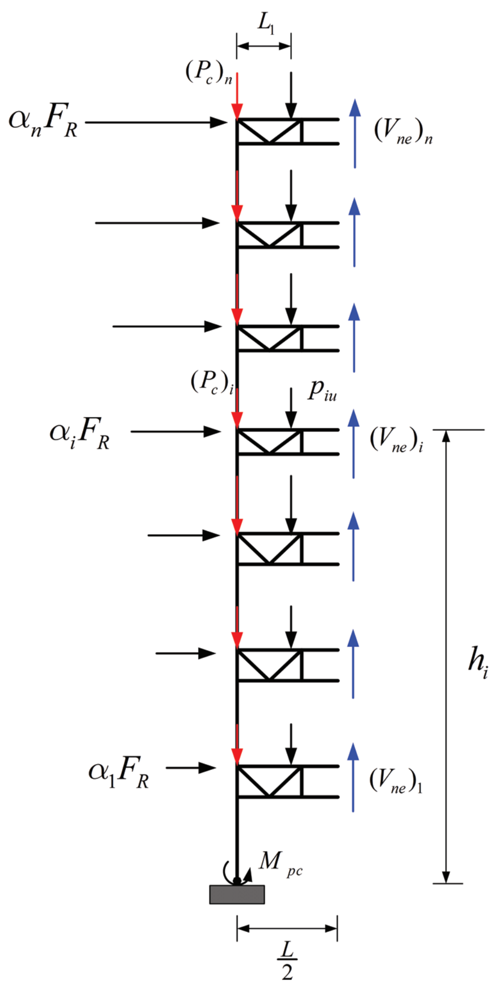


Fig. 8. Free-body diagram of an exterior column and associated truss girder branches with concentrated gravity loadings on truss girders (lateral forces acting to right side).

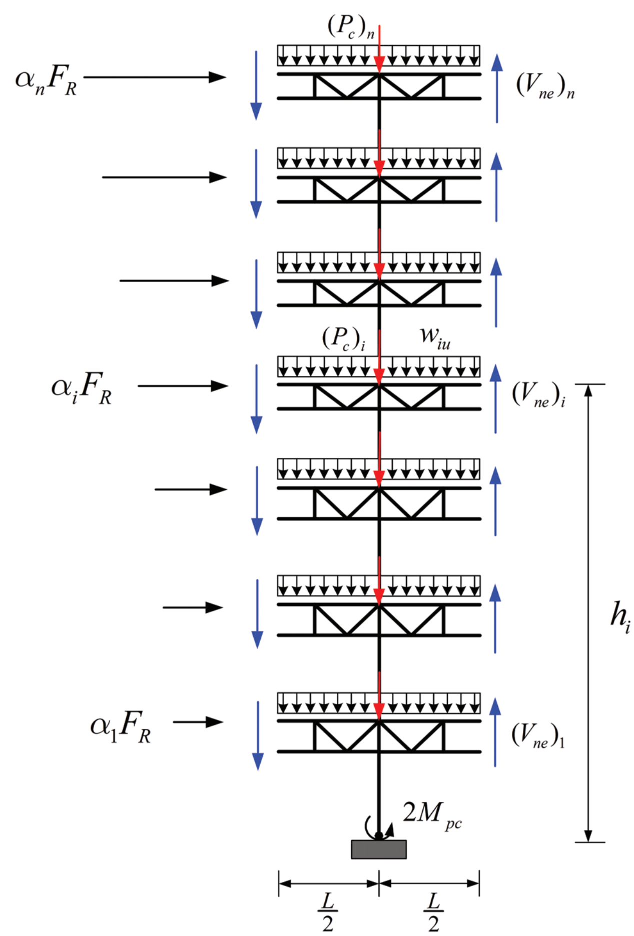


Fig. 9. Free-body diagram of an interior column and associated truss girder branches subjected to lateral forces to right.

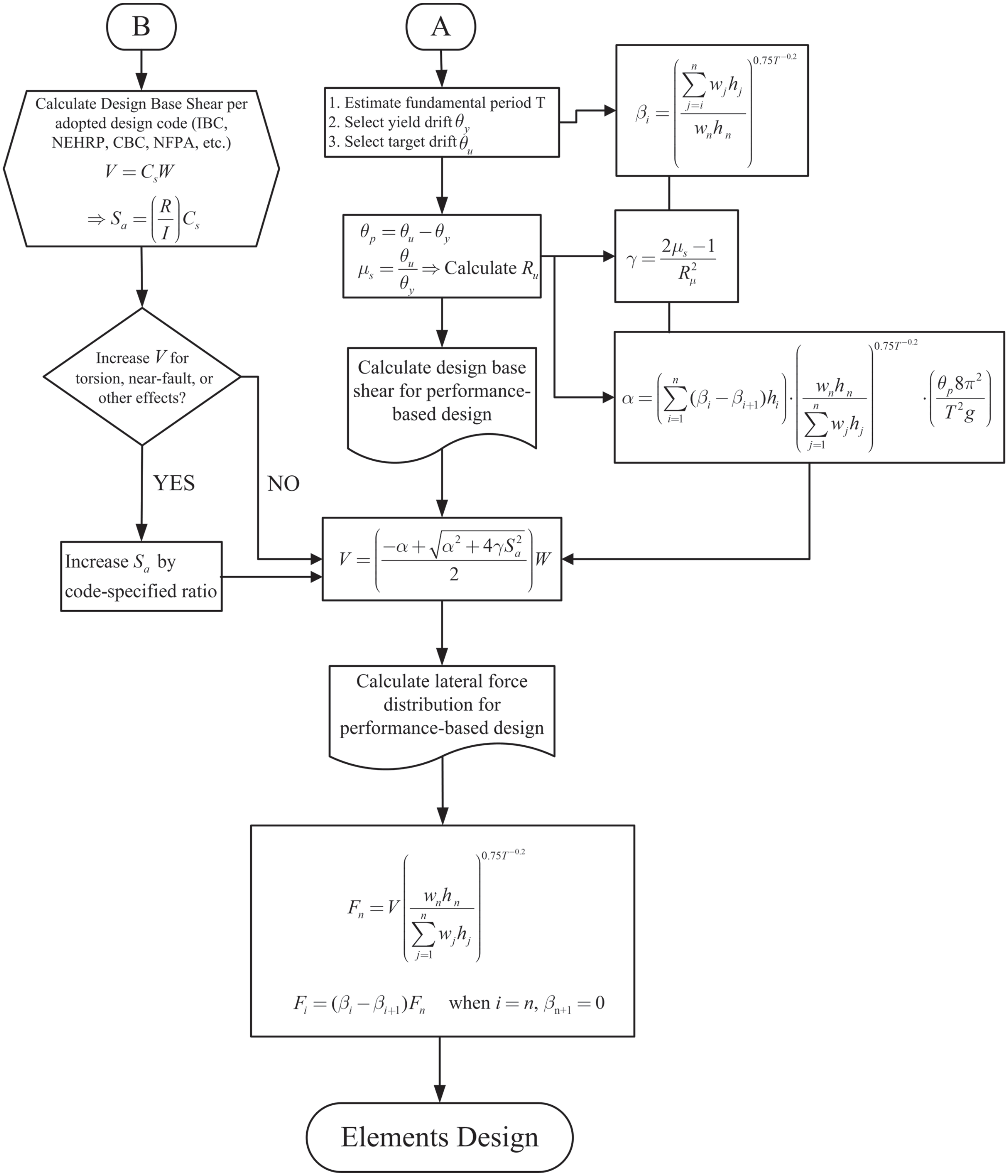


Fig. 10. Performance-based plastic design flowchart for STMF: determine design base shear and lateral force distribution.

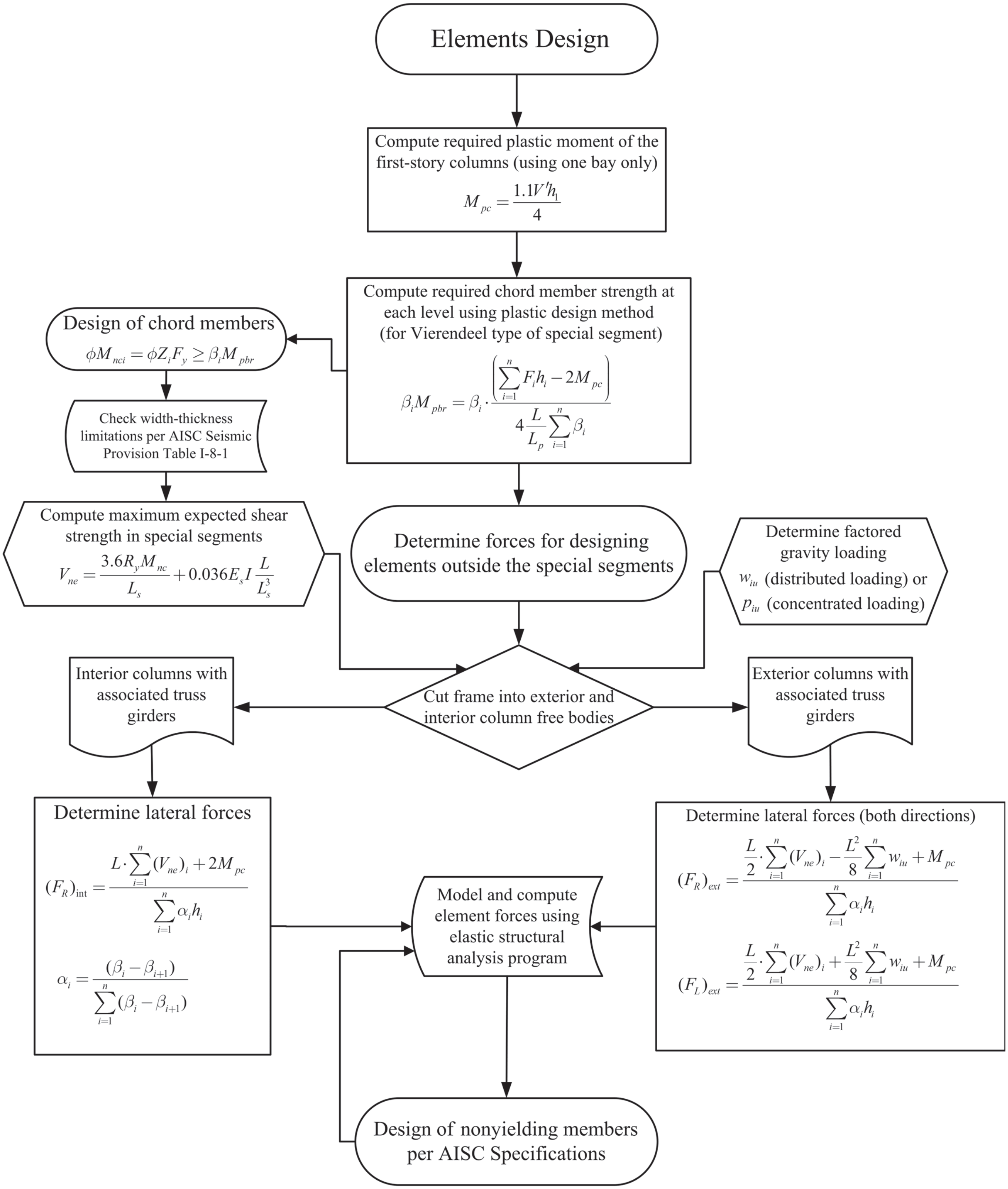
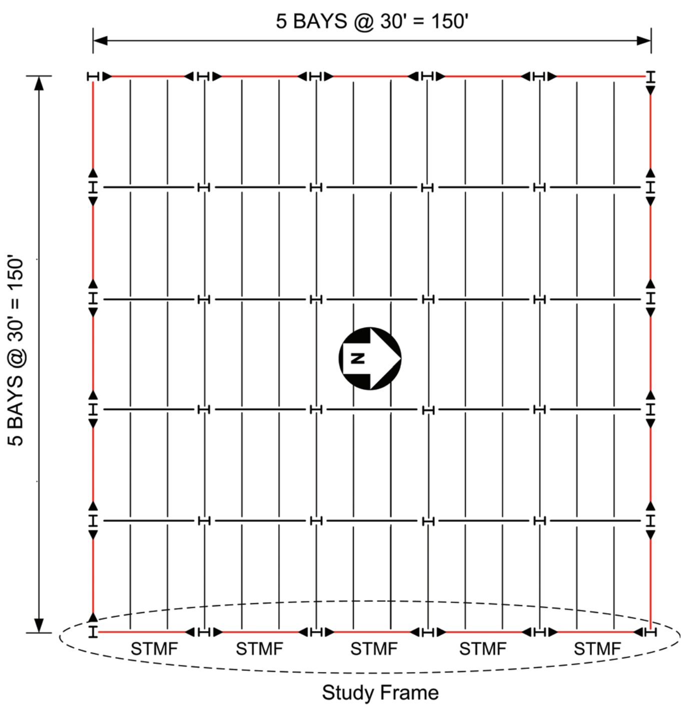


Fig. 11. Performance-based plastic design flowchart for STMF: element design.



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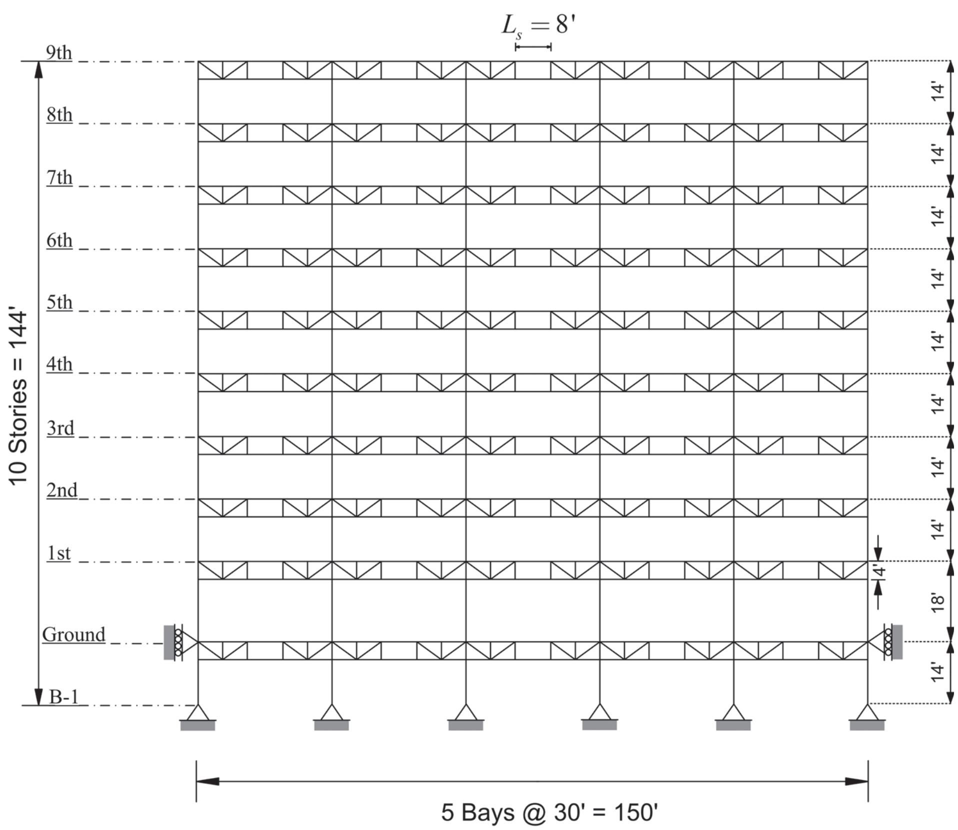


(b)

Fig. 12. Building plan and elevation of study STMFs.

DESIGN AND ANALYSIS OF THE STUDY FRAMES

Two nine-story STMFs, representing essential facilities (in other words, hospital buildings) and ordinary occupancy (office/residential) type, were designed by using the proposed procedure. The building plan and elevation are shown in Figure 12. For an ordinary occupancy building, the target drifts of 2% and 3% for 10% in 50 years [2/3 maximum considered earthquake (MCE)] and 2% in 50 years (MCE) design hazard levels, respectively, were chosen. These values were chosen to be consistent with the current design practice. The corresponding numbers for essential buildings were 1.5% and 2.25%. Design spectral values were based on NEHRP Provisions for the San Francisco site (NEHRP, 2000). The final member sections for the two study frames are shown in Figures 13 through 16. In this study, the column sizes were changed at every floor for study purposes. In practice, column sizes can be changed every two or three floors instead of every floor. This would somewhat increase the material weight but reduce the fabrication cost, due to the reduction of column splices. It is also noted that, to avoid biaxial bending in the exterior columns, the lower chord member adjacent to the exterior column bending about the weak axis is not connected to the column (see Figures 13 and 15).

After the final design work was completed, nonlinear response history analyses were conducted to study the response and ductility demands of the frames. Nine 10% in 50 years,

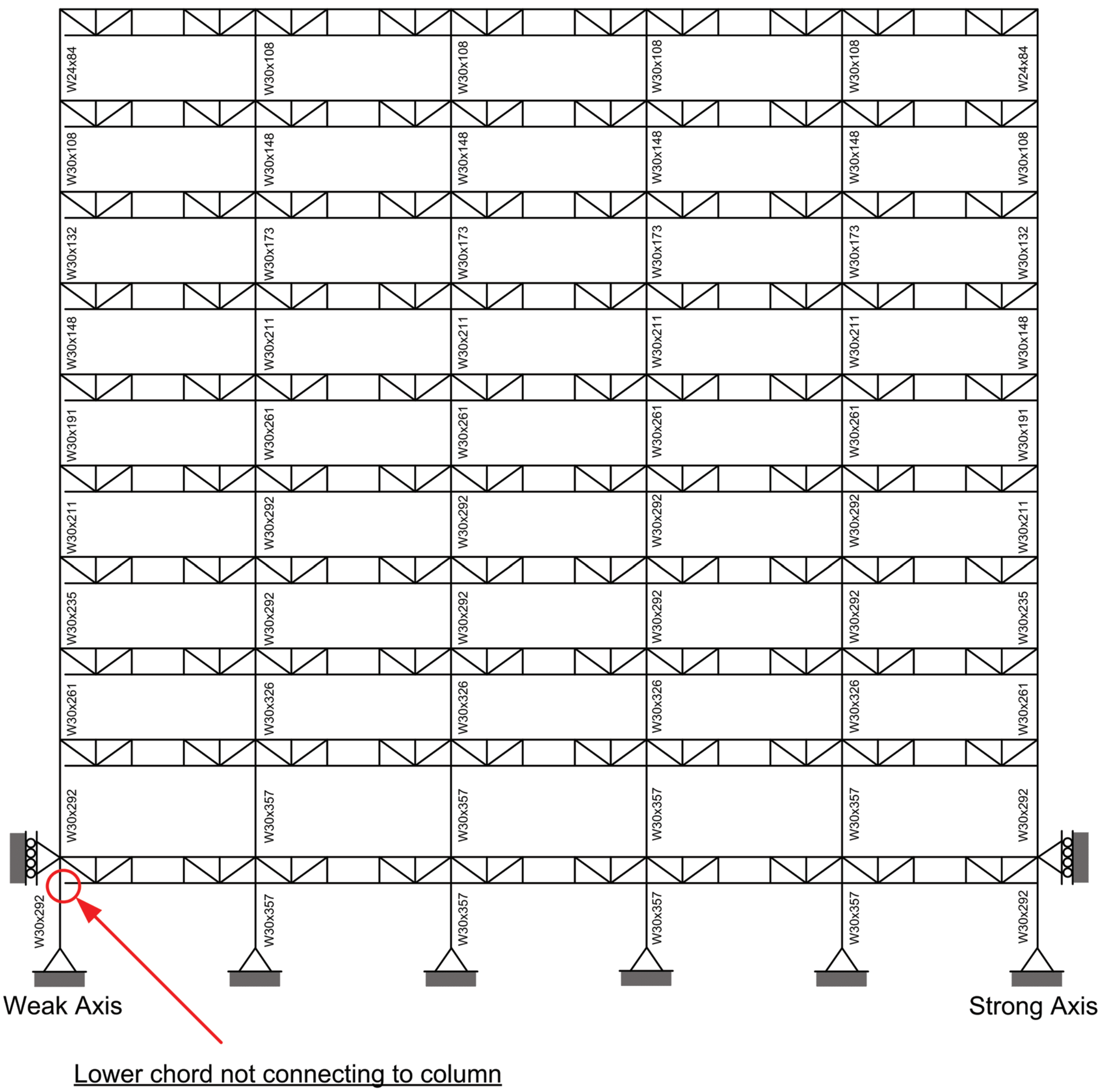
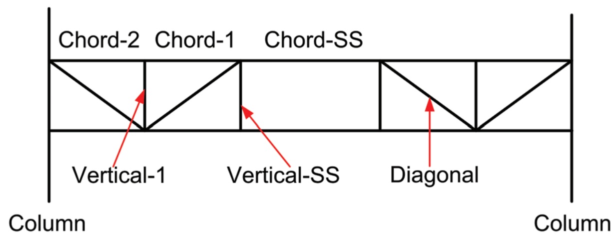


Fig. 13. Design column sections for the nine-story ordinary STMF.



FLR	Chord-SS	Chord-1	Chord-2	Vertical-SS	Vertical-1	Diagonal
9	C7×12.25	C7×12.25	C7×12.25× (0.25-in. plate)	C7×12.25	MC6×12	MC6×12
8	C8×18.75	C8×18.75	C8×18.75× (0.5-in. plate)	C8×18.75	MC6×12	MC6×16.3
7	C9×20	C9×20	C9×20× (0.75-in. plate)	C9×20	MC6×12	MC7×22.7
6	C10×20	C10×20	C10×20× (1.0-in. plate)	C10×20	MC6×12	MC7×22.7
5	C10×25	C10×25	C10×25× (1.25-in. plate)	C10×25	MC6×12	MC9×25.4
4	C10×25	C10×25	C10×25× (1.25-in. plate)	C10×25	MC6×12	MC9×25.4
3	C10×25	C10×25	C10×25× (1.25-in. plate)	C10×25	MC6×12	MC9×25.4
2	C10×30	C10×30	C10×30× (1.5-in. plate)	C10×30	MC6×16.3	M10×C25
1	C10×30	C10×30	C10×30× (1.5-in. plate)	C10×30	MC6×16.3	M10×C25
B-1	C10×30	C10×30	C10×30× (1.5-in. plate)	C10×30	MC6×16.3	M10×C25

Note 1: All sections are double channels.
Note 2: B-1 Level uses the same sections as at Level 1.
Note 3: Web plates are extended over the whole length of the corresponding panel, i.e., 5.5 ft.

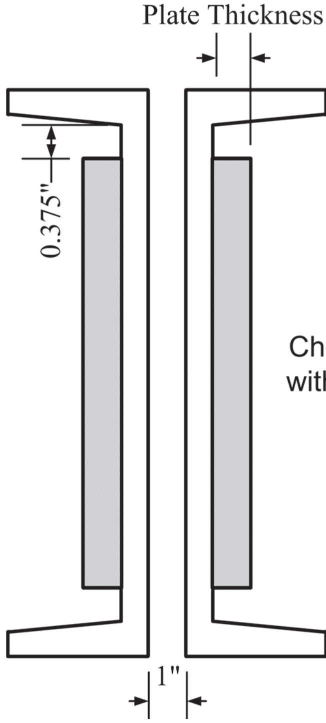


Fig. 14. Design special segment and truss member sections for the nine-story ordinary STMF.

and five 2% in 50 years SAC* Los Angeles region ground motions representing the two design hazard levels were used (Somerville et al., 1997). The results of the analyses were studied to validate the design procedure, and to compare the chord member ductility demands with the capacities as determined from the testing work on built-up double-channel specimens (Parra-Montesinos, Goel and Kim, 2006). The seismic performance of the study frames was examined in terms of location of yielding, maximum plastic rotation in chord members, maximum interstory drift, peak floor accelerations, and maximum relative story shear distribution. The analyses were performed using the Perform-2D (RAM, 2003) program. $P - \delta$ effect was accounted for in the analysis. However, the "leaning columns" with vertical loading from gravity frames were not included in this study because the interstory drifts were not large enough to induce significant $P–\Delta$ effect. At the same time, the beneficial effect of the "leaning columns" to provide additional lateral strength was ignored. These two assumptions should be offsetting each other's influence on the response. Detailed modeling for the nonlinear analyses can be found elsewhere (Chao and Goel, 2006).

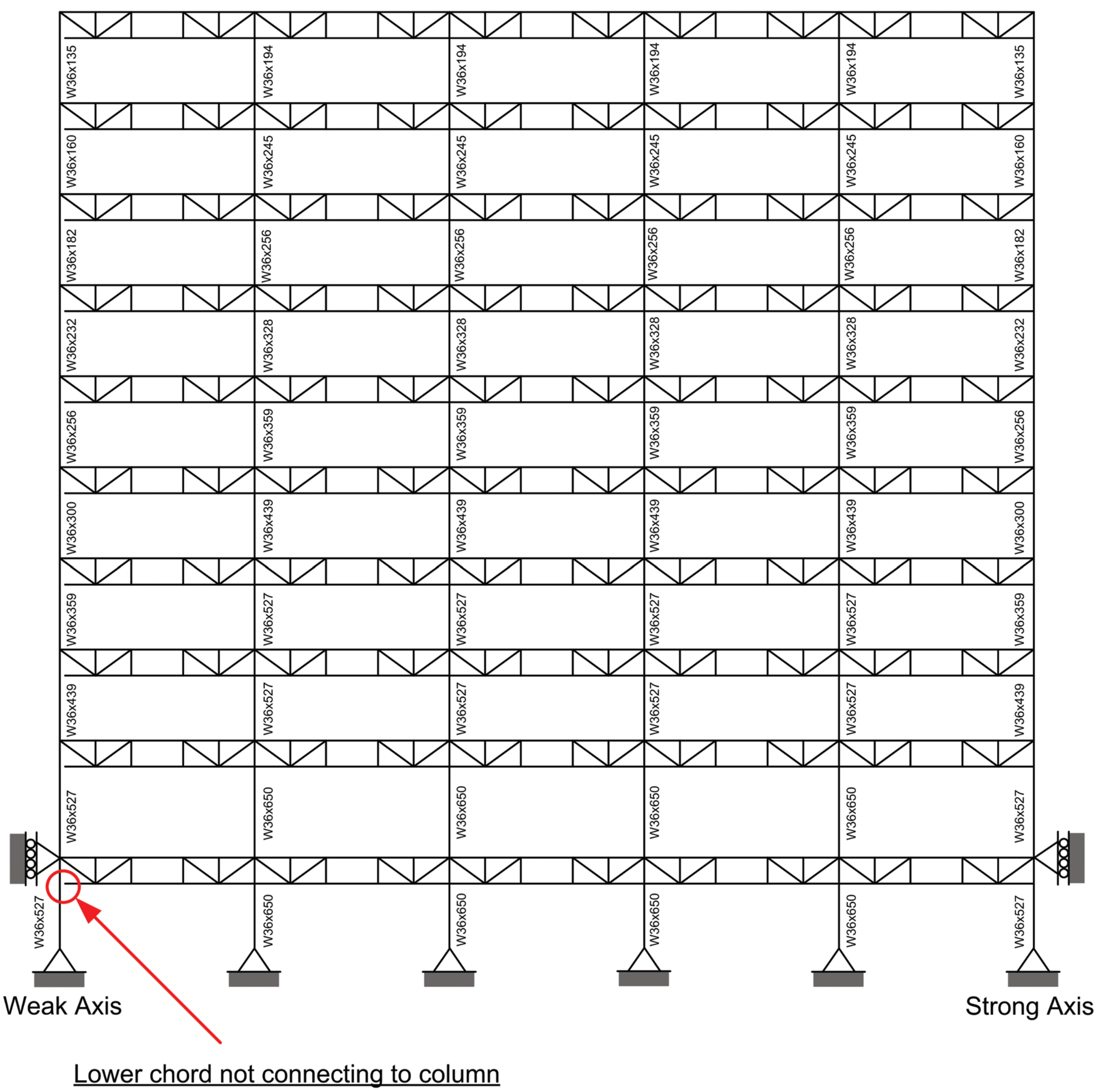
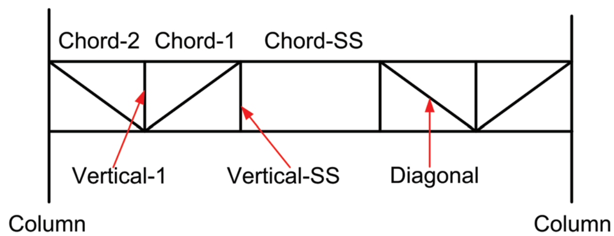


Fig. 15. Design column sections for the nine-story essential STMF.

*A joint venture partnership of Structural Engineers Association of California, Applied Technology Council, and California Universities for Research in Earthquake Engineering



FLR	Chord-SS	Chord-1	Chord-2	Vertical-SS	Vertical-1	Diagonal
9	C9×20	C9×20	C9×20× (0.25-in. plate)	C9×20	MC6×12	MC9×25.4
8	C10×25	C10×25	C10×25× (0.5-in. plate)	C10×25	MC6×12	MC9×25.4
7	C10×30	C10×30	C10×30× (0.75-in. plate)	C10×30	MC6×12	MC9×25.4
6	C12×30	C12×30× (0.25-in. plate)	C12×30× (1.0-in. plate)	C12×30	MC6×12	MC10×41.1
5	C12×30	C12×30× (0.25-in. plate)	C12×30× (1.0-in. plate)	C12×30	MC7×22.7	MC10×41.1
4	MC12×31	MC12×31× (0.25-in. plate)	MC12×31× (1.25-in. plate)	MC12×31	MC7×22.7	MC10×41.1
3	MC12×31	MC12×31× (0.25-in. plate)	MC12×31× (1.25-in. plate)	MC12×31	MC7×22.7	MC10×41.1
2	MC12×35	MC12×35× (0.25-in. plate)	MC12×35× (1.5-in. plate)	MC12×35	MC7×22.7	MC10×41.1
1	MC12×35	MC12×35× (0.25-in. plate)	MC12×35× (2.0-in. plate)	MC12×35	MC8×22.8	MC10×41.1
B-1	MC12×35	MC12×35× (0.25-in. plate)	MC12×35× (2.0-in. plate)	MC12×35	MC8×22.8	MC10×41.1

Note 1: All sections are double channels.
Note 2: B-1 Level uses the same sections as at Level 1.
Note 3: Web plates are extended over the whole length of the corresponding panel, i.e., 5.5 ft.

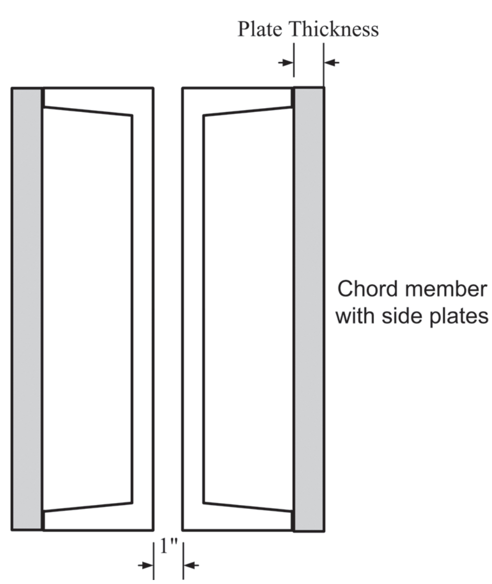


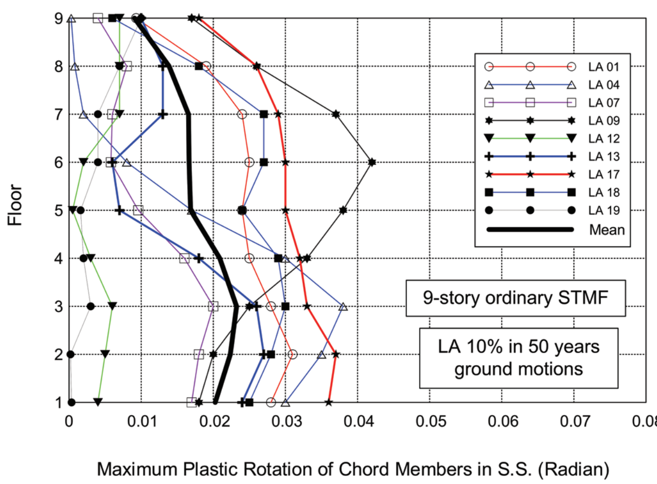
Fig. 16. Design special segment and truss member sections for the nine-story essential STMF.

PERFORMANCE EVALUATION OF THE STUDY FRAMES

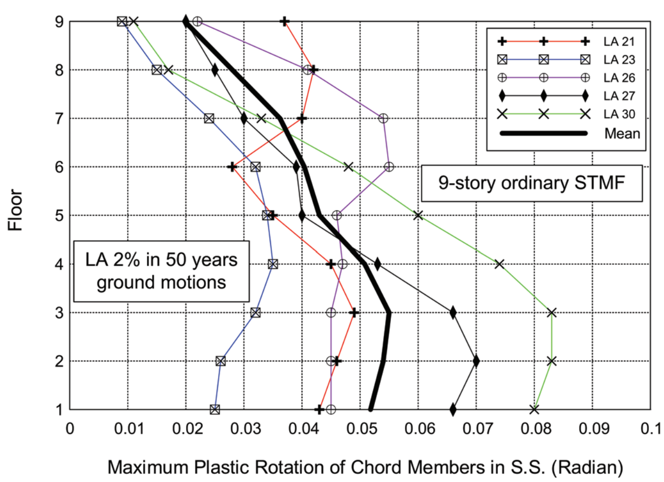
Maximum Plastic Rotation in the Special Segments

Analysis results showed that plastic hinges occurred mainly at the ends of the chord members in the special segments, while the other elements remained elastic. It can be concluded that STMFs designed by the proposed performance-based plastic design (PBPD) approach resulted in the formation of a mechanism as intended. Maximum plastic hinge rotations at all levels for the two study frames are shown in Figures 17 and 18, along with their mean values from all ground motions.

As can be seen, the maximum plastic hinge rotation is about 0.04 rad when subjected to 10% in 50 years ground motions and 0.08 rad when subjected to 2% in 50 years ground motions. The peak mean values of the rotations are about 0.022 rad and 0.055 rad for 10% in 50 years and 2% in 50 years ground motions, respectively. It should be mentioned that tests of double-channel sections for STMF showed plastic rotation capacity on the order of 0.07 rad (Parra-Montesinos et al., 2006). The plastic rotations are generally uniformly distributed along the height of the frames, which is thought to be primarily due to the design lateral force distribution used in this study.

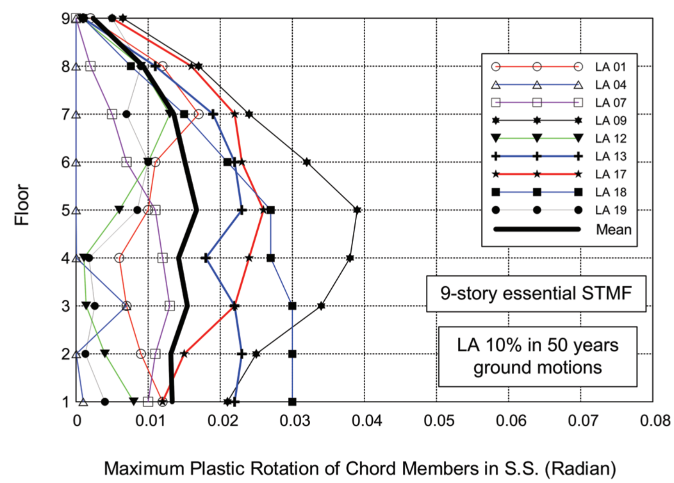


(a)

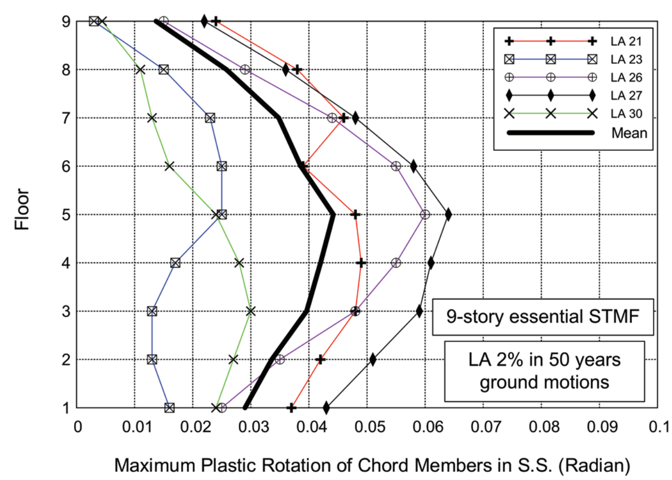


(b)

Fig. 17. Maximum plastic hinge rotations in chord members for nine-story ordinary STMF subjected to 10%/50 and 2%/50 ground motions.



(a)



(b)

Fig. 18. Maximum plastic hinge rotations in chord members for nine-story essential STMF subjected to 10%/50 and 2%/50 ground motions.

Maximum Interstory Drifts

Figures 19 and 20 show the maximum interstory drifts for the ordinary and essential STMFs, respectively. The mean values of maximum story drifts and corresponding target drifts are also shown. It can be noticed that all the maximum interstory drifts of the PBPD frames were within the preselected target drift limits for the hazard levels, signifying that the seismic performance of the deformation-sensitive components (such as cladding, partitions, interior veneers, and glazing systems), as well as the chord member damage can be kept within target limits by the proposed design procedure.

Maximum Absolute Floor Accelerations

As seen in Figure 21, the maximum absolute floor accelerations of the two study STMFs are generally below the code-specified floor design acceleration except at a few lower levels, 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. Higher floor accelerations in the essential STMF are primarily the result of higher stiffness of the essential STMF, which is known to cause greater floor accelerations (Mayes, Goings, Naguib, Harris, Lovejoy, Fanucci, Bystricky and Hayes, 2004).

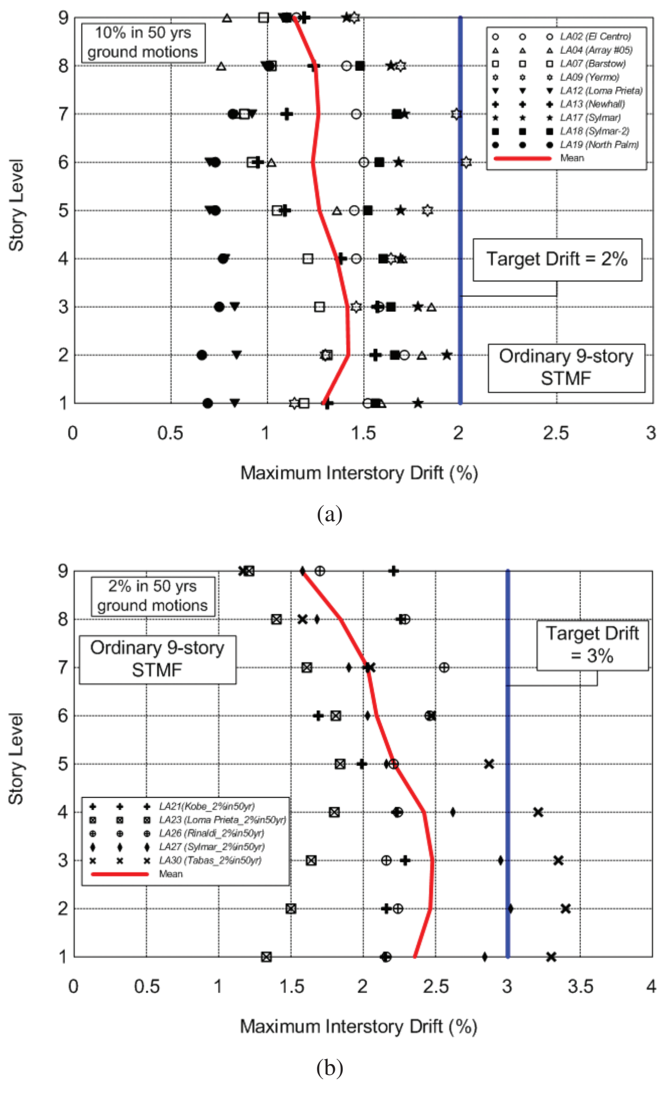


Fig. 19. Mean value of maximum interstory drifts and corresponding target drifts for the nine-story ordinary STMF.

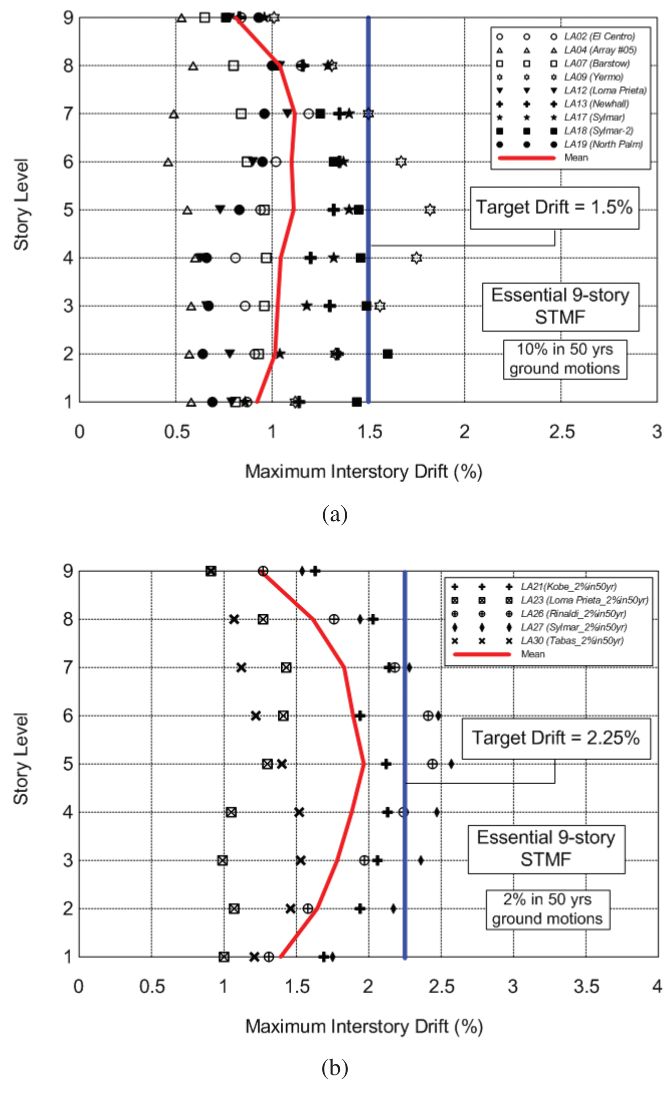
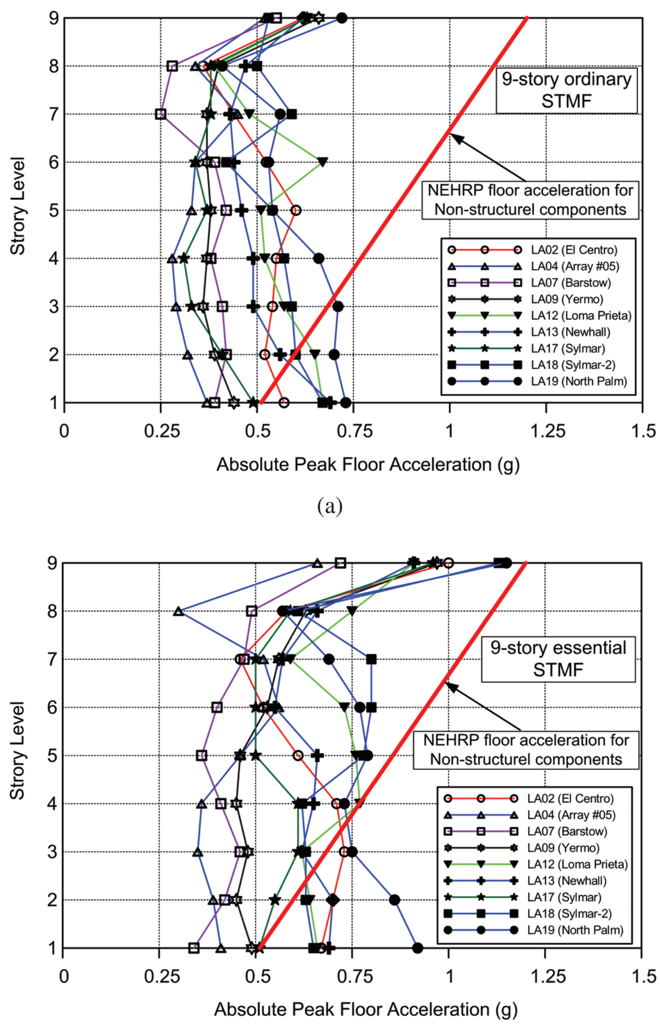


Fig. 20. Mean value of maximum interstory drifts and corresponding target drifts for the nine-story essential STMF.

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 nine 10% in 50 years, and five 2% in 50 years response histories for the nine-story ordinary STMF are shown in Figure 22. It can be seen that the proposed design story shear distribution well represents the mean story shear distribution of the structure due to the ground motion records used in this study. Relative story shear distributions according to NEHRP (NEHRP, 2000) and the *Uniform Building Code* (UBC 97) (ICBO, 1997) expressions are also plotted. It can be seen that while the NEHRP distribution shows significant deviation from those obtained from nonlinear dynamic analyses the UBC distribution gives better prediction. This may be attributed to the additional concentrated force applied at the top floor by using the UBC expression. The results suggest that the NEHRP distribution (also the distribution used by current IBC Provisions), which is generally based on the first mode elastic story shear distribution including some higher mode effects, does not predict the realistic maximum story shear distribution during major earthquakes (Chao et al., 2007). The proposed design force distribution does it much better because it is based on inelastic response results.



(a)

(b)

Fig. 21. Peak floor accelerations (10% in 50 years) and the NEHRP-specified design acceleration for acceleration-sensitive nonstructural components: (a) ordinary STMF; (b) essential STMF.

DESIGN EXAMPLE

The proposed performance-based plastic design (PBPD) procedure is summarized in the flowcharts given in Figures 10 and 11 using the nine-story ordinary STMF example. The main steps in the procedure are:

1. The basic design parameters are first obtained from NEHRP 2000 as summarized in Table 1.

2. The design base shear is determined for a two-level performance criteria: (a) a 2% maximum story drift for a ground motion hazard with 10% probability of exceedance in 50 years (10/50 and 2/3 MCE) and (b) 3% maximum story drift for 2/50 event (MCE). Then the modified design parameters are calculated and listed in Table 2. The design base shear is obtained from Equations 3, 14, and 15. After knowing the design base shear, the lateral forces are calculated by using Equations 1, 2 and 3 as shown in Tables 3 and 4. It is seen from Tables 1 and 2 that for this example the PBPD design base shear is 1.77 times that obtained from NEHRP 2000. This is explained as follows: (1) the drift control is built into the PBPD design base shear expression, and (2) the base shear in the proposed method corresponds to the global yield mechanism while that from NEHRP is intended for use with elastic design methods.

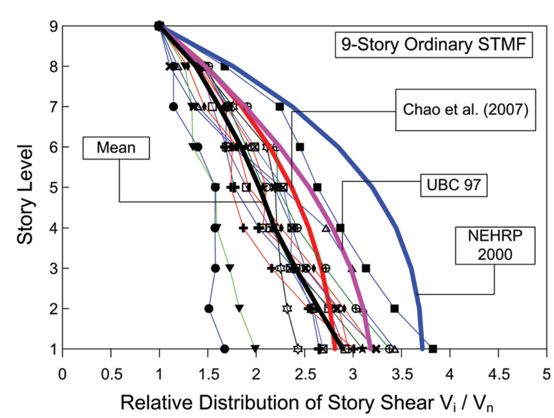


Fig. 22. Relative story shear distributions obtained from nonlinear dynamic analyses.

Table 1. Design Parameters for Nine-Story Ordinary STMF Calculated According to NEHRP 2000	
Parameters	**Nine-Story STMF**
S_S	1.50 g
S_1	0.78 g
S_{MS}	1.50 g
S_{M1}	1.01 g
F_a	1.000
F_v	1.3
S_{DS}	1.00 g
S_{D1}	0.68 g
Site Class	*C*
Occupancy Importance Factor, *I*	1.0 (Ordinary Building)
Seismic Design Category	*E*
Building Height	130 ft (above the base)
T_a	1.375 sec
C_U	1.4
T	1.925 sec
Response Modification Factor, *R*	7
Total Building Weight, *W*	19839 kips
$C_s = V/W$	0.056

Table 2. Design Parameters for the PBPD Procedure		
Parameters	**10% in 50-Year Hazard**	**2% in 50-Year Hazard**
S_a	0.39 g	0.525 g
T	1.925	1.925
Yield Drift, θ_y	0.75%	0.75%
Target Drift, θ_u	2%	3%
Inelastic Drift, θ_p	1.25%	2.25%
$\mu_s = \theta_u/\theta_y$	2.67	4
R_μ	2.67	4
γ	0.609	0.438
α	0.841	1.515
V/W	0.099	0.076
Design Base Shear, *V*	1956.1 kips (governs)	1504.3 kips

Table 3. Shear Distribution Factor for the Nine-Story Ordinary STMF						
Floor	h_j (ft)	w_j (kips)	$w_j h_j$ (kip-ft)	$\sum_{j=i}^{n} w_j h_j$	β_i (= V_i / V_n)	$(\beta_i - \beta_{i+1})h_i$
9	130	2357	306410	306410	1.000	130.00
8	116	2180	252880	559290	1.486	56.34
7	102	2180	222360	781650	1.852	37.34
6	88	2180	191840	973490	2.139	25.31
5	74	2180	161320	1134810	2.367	16.80
4	60	2180	130800	1265610	2.543	10.57
3	46	2180	100280	1365890	2.673	6.02
2	32	2180	69760	1435650	2.762	2.85
1	18	2222	39996	1475646	2.813	0.91

Table 4. Design Lateral Forces for the Nine-Story Ordinary STMF (for Two Frames)		
Floor	$\beta_i - \beta_{i+1}$	F_i, kips
9	1.000	695.4
8	0.486	337.8
7	0.366	254.6
6	0.288	200.1
5	0.227	157.9
4	0.176	122.5
3	0.131	91.0
2	0.089	61.9
1	0.050	35.1

Table 5. Required Chord Member Strength and Selected Chord Member Sections						
Floor	Require Moment Strength $\beta_i M_{pbr}$, kip-ft	Required Z, in.^{3}	Section* (Double Channels)	Z, in.^{3}	M_{nc}, kip-in.	I_x, in.^{4}
9	61.0	16.3	C7×12.25	16.92	846	48.4
8	90.6	24.2	C8×18.75	27.8	1390	87.8
7	112.9	30.1	C9×20	33.8	1690	121.8
6	130.5	34.8	C10×20	38.8	1940	157.8
5	144.3	38.5	C10×25	46.2	2310	182.2
4	155.1	41.4	C10×25	46.2	2310	182.2
3	163.0	43.5	C10×25	46.2	2310	182.2
2	168.5	44.9	C10×30	53.4	2670	206
1	171.6	45.7	C10×30	53.4	2670	206

*Section sizes taken from the 3rd Ed. AISC *Load and Resistance Factor Design Manual of Steel Construction*.

Table 6. Compactness Check for Chord Member Sections per AISC *Seismic Provisions* Table I-8-1				
Floor	Width-Thickness Ratio $\frac{b_f}{t_f}$	Limiting Width-Thickness Ratio $0.3\sqrt{\frac{E_s}{F_y}}$	Width-Thickness Ratio $\frac{d}{t_w}$	Limiting Width-Thickness Ratio* $1.12\sqrt{\frac{E_s}{F_y}}\left(2.33-\frac{P_u}{\phi_b P_y}\right)$
9	5.98	7.22	22.3	35.87
8	6.49	7.22	16.4	35.87
7	6.42	7.22	20.1	35.87
6	6.28	7.22	26.4	35.87
5	6.63	7.22	19.0	35.87
4	6.63	7.22	19.0	35.87
3	6.63	7.22	19.0	35.87
2	6.95	7.22	14.9	35.87
1	6.95	7.22	14.9	35.87

*Note: Conservatively $P_u = \phi_b P_y$ was assumed even though the chord members in the special segment are generally subjected to small axial forces.

3. The required plastic moment of the first-story columns is computed as (Equation 17):

$$M_{pc} = \frac{1.1V'h_1}{4}$$
$$= \left(1.1 \times \frac{1956.1 \text{ kips}}{(2)(5)} \times 18 \text{ ft}\right)/4$$
$$= 968.3 \text{ kip-ft}$$

   Note that V' is the base shear for one bay (in each direction of the building there are two STMFs and each STMF has five bays, see Figure 12).

4. The required chord member strength at each level is determined by using Equation 18. ASTM A992 steel with 50-ksi nominal yield strength is used. The selected chord sections for each level are given in Table 5 and the compactness check is shown in Table 6.

5. Design of members (chords, verticals, diagonals and columns) outside the special segment is based on the capacity design approach (see Figure 11 for design flowchart). It should be noted that the lateral forces at each level are those needed to develop the expected ultimate strength of the special segments, in other words, V_{ne}. The applied forces on the interior and exterior column free-bodies are shown in Table 7. Figure 23 shows the forces acting on the interior column free-body. The required elastic moment and axial force for each element outside the special segment can be easily obtained by using an elastic structural analysis. All the elements are designed as beam-column elements, according to the AISC *Specification for Structural Steel Buildings* (AISC, 2005b). The final sections are shown in Figures 13 and 14.

SUMMARY AND CONCLUSIONS

A direct performance-based plastic design (PBPD) approach, which requires no iterative evaluation or refinement such as by nonlinear static (pushover) or dynamic analysis after the initial design, has been presented. Based on an energy (work) concept and plastic design method, 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 preselected target drift. The design lateral force distribution employed in the proposed method is based on nonlinear response history analysis results using a number of ground motions. The chord members are designed according to the AISC Seismic Provisions, while the members outside the special segment are designed by using a capacity design approach. The following conclusions can be drawn from this study:

1. All inelastic activity was confined to the special segments and the column bases in the study frames; that is, STMFs designed by the proposed PBPD method resulted in the formation of a yield mechanism as intended.

Table 7. Design Forces for Elements Outside Special Segments					
Floor	$(V_{ne})_i$ kips	Concentrated Factored Gravity Loading @ 10 ft Spacing in Each Bay, kips	Lateral Forces at Ultimate Drift Level for Exterior Column Free Body, kips		Lateral Forces at Ultimate Drift Level for Interior Column Free Body, kips
			$\alpha_i F_R$	$\alpha_i F_L$	F_i
9	55.5	16	68.5	78.0	146.4
8	94.6	15	33.3	37.9	71.1
7	121.5	15	25.1	28.5	53.6
6	147.1	15	19.7	22.4	42.1
5	172.7	15	15.5	17.7	33.2
4	172.7	15	12.1	13.7	25.8
3	172.7	15	9.0	10.2	19.2
2	197.6	15	6.1	6.9	13.0
1	197.6	15	3.4	3.9	7.4
∑	1332.0		192.6	219.3	411.9

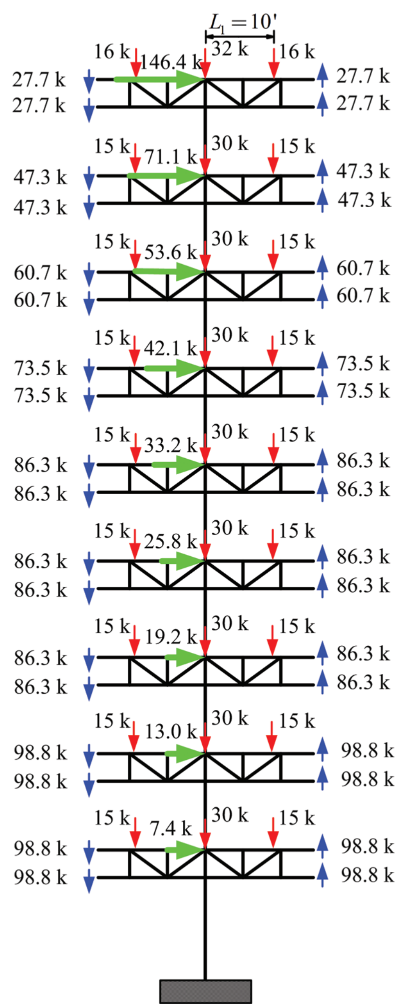


Fig. 23. Forces acting on the interior column free body (nine-story ordinary STMF).

2. The maximum plastic hinge rotations of chord members in the study frames were within the available rotation capacity for all 10% in 50 years ground motions used in this study. Moreover, the study frames generally showed quite uniformly distributed plastic rotations along the height, due to the use of the proposed lateral force distribution. This also leads to more even distribution of dissipated energy along the height.

3. It was observed that all interstory drifts of the study frames were within the 1.5% and 2% preselected target drift when the structure was subjected to the 2/3 MCE ground motions (first level hazard) for the ordinary and essential buildings, respectively. Also, most interstory drifts were within the 2.25% and 3% preselected target drift when the structure was subjected to the MCE ground motions (second level hazard) for the ordinary and essential buildings, respectively. This suggests that satisfactory seismic performance of the deformation-sensitive nonstructural components can also be achieved by the proposed design procedure.

4. The maximum absolute floor accelerations were generally within the code-specified values, suggesting that the seismic performance of the acceleration-sensitive components can also be assumed to be satisfactory.

5. Based on 14 time-history nonlinear dynamic analyses, the suggested design story shear distribution (lateral force distribution) represents the mean story shear distributions of the structure very well because it is based on inelastic behavior. On the contrary, the NEHRP (IBC) force distribution does not represent the expected maximum story shear distributions during strong earthquakes. The UBC expression, by providing an additional force at the top level, gave a more realistic relative story shear distribution than the NEHRP expression.

6. Overall, STMFs designed by the proposed performance-based plastic design (PBPD) method can be expected to satisfy the required performance objectives when subjected to a major earthquake. This 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.

ACKNOWLEDGMENTS

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