6

Structural Steel Design

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This chapter illustrates how the 2009 NEHRP Recommended Provisions (the Provisions) is applied to the design of steel framed buildings. The following three examples are presented:

1. An industrial warehouse structure in Astoria, Oregon
2. A multistory office building in Los Angeles, California
3. A mid-rise hospital in Seattle, Washington

The discussion examines the following types of structural framing for resisting horizontal forces:

- Ordinary concentrically braced frames (OCBF)
- Special concentrically braced frames
- Intermediate moment frames
- Special moment frames
- Buckling-restrained braced frames, with moment-resisting beam-column connections

The examples cover design for seismic forces in combination with gravity they are presented to illustrate only specific aspects of seismic analysis and design—such as lateral force analysis, design of concentric and eccentric bracing, design of moment resisting frames, drift calculations, member proportioning detailing.

All structures are analyzed using three-dimensional static or dynamic methods. ETABS (Computers & Structures, Inc., Berkeley, California, v.9.5.0, 2008) is used in Examples 6.1 and 6.2.

In addition to the 2009 NEHRP Recommended Provisions, the following documents are referenced:

Chapter 6: Structural Steel Design


The symbols used in this chapter are from Chapter 11 of the Standard, the above referenced documents, or are as defined in the text. U.S. Customary units are used.

6.1 INDUSTRIAL HIGH-CLEARANCE BUILDING, ASTORIA, OREGON

This example utilizes a transverse intermediate steel moment frame and a longitudinal ordinary concentric steel braced frame. The following features of seismic design of steel buildings are illustrated:

- Seismic design parameters
- Equivalent lateral force analysis
- Three-dimensional analysis
- Drift check
- Check of compactness and spacing for moment frame bracing
- Moment frame connection design
- Proportioning of concentric diagonal bracing

6.1.1 Building Description

This building has plan dimensions of 180 feet by 90 feet and a clear height of approximately 30 feet. It includes a 12-foot-high, 40-foot-wide mezzanine area at the east end of the building. The structure consists of 10 gable frames spanning 90 feet in the transverse (north-south) direction. Spaced at 20 feet on center, these frames are braced in the longitudinal (east-west) direction in two bays at the east end. The building is enclosed by nonstructural insulated concrete wall panels and is roofed with steel decking covered with insulation and roofing. Columns are supported on spread footings.

The elevation and transverse sections of the structure are shown in Figure 6.1-1. Longitudinal struts at the eaves and at the mezzanine level run the full length of the building and therefore act as collectors for the distribution of forces resisted by the diagonally braced bays and as weak-axis stability bracing for the moment frame columns.

The roof and mezzanine framing plans are shown in Figure 6.1-2. The framing consists of a steel roof deck supported by joists between transverse gable frames. The mezzanine represents both an additional load and additional strength and stiffness. Because all the frames resist lateral loading, the steel deck functions as a diaphragm for distribution of the effects of eccentric loading caused by the mezzanine floor when the building is subjected to loads acting in the transverse direction.

The mezzanine floor at the east end of the building is designed to accommodate a live load of 125 psf. Its structural system is composed of a concrete slab over steel decking supported by floor beams spaced at 10 feet on center. The floor beams are supported on girders continuous over two intermediate columns spaced approximately 30 feet apart and are attached to the gable frames at each end.
The member sizes in the main frame are controlled by serviceability considerations. Vertical deflections due to snow were limited to 3.5 inches and lateral sway due to wind was limited to 2 inches.

![Framing elevation and sections](image)

**Figure 6.1-1** Framing elevation and sections  
(1.0 ft = 0.3048 m; 1.0 in. = 25.4 mm)

Earthquake rather than wind governs the lateral design due to the mass of the insulated concrete panels. The panels are attached with long pins perpendicular to the concrete surface. These slender, flexible pins isolate the panels from acting as shear walls.

The building is supported on spread footings based on moderately deep alluvial deposits (i.e., medium dense sands). The foundation plan is shown in Figure 6.1-3. Transverse ties are placed between the footings of the two columns of each moment frame to provide restraint against horizontal thrust from the moment frames. Grade beams carrying the enclosing panels serve as ties in the longitudinal direction as well as across the end walls. The design of footings and columns in the braced bays requires consideration of combined seismic loadings. The design of foundations is not included here.
Figure 6.1-2  Roof framing and mezzanine framing plan
(1.0 ft = 0.3048 m; 1.0 in. = 25.4 mm)
6.1.2 Design Parameters

6.1.2.1 Ground motion and system parameters. See Section 3.2 for an example illustrating the determination of design ground motion parameters. For this example the parameters are as follows.

- $S_{DS} = 1.0$
- $S_{DI} = 0.6$
- Occupancy Category II
- Seismic Design Category D

Note that *Standard* Section 12.2.5.6 permits an ordinary steel moment frame for buildings that do not exceed one story and 65 feet tall with a roof dead load not exceeding 20 psf. Intermediate steel moment frames with stiffened bolted end plates and ordinary steel concentrically braced frames are used in this example.

- North-south (N-S) direction:
  
  Moment-resisting frame system = intermediate steel moment frame (*Standard* Table 12.2-1)  
  $R = 4.5$

Figure 6.1-3 Foundation plan
(1.0 ft = 0.3048 m; 1.0 in. = 25.4 mm)
$\Omega_0 = 3$
$C_d = 4$

- East-west (E-W) direction:

  Braced frame system = ordinary steel concentrically braced frame (Standard Table 12.2-1)
  
  $R = 3.25$
  
  $\Omega_0 = 2$
  
  $C_d = 3.25$

### 6.1.2.2 Loads

- Roof live load ($L$), snow = 25 psf
- Roof dead load ($D$) = 15 psf
- Mezzanine live load, storage = 125 psf
- Mezzanine slab and deck dead load = 69 psf
- Weight of wall panels = 75 psf

Roof dead load includes roofing, insulation, metal roof deck, purlins, mechanical and electrical equipment that portion of the main frames that is tributary to the roof under lateral load. For determination of the seismic weights, the weight of the mezzanine will include the dead load plus 25 percent of the storage load (125 psf) in accordance with Standard Section 12.7.2. Therefore, the mezzanine seismic weight is $69 + 0.25(125) = 100$ psf.

### 6.1.2.3 Materials

- Concrete for footings: $f_c' = 2.5$ ksi
- Slabs-on-grade: $f_c' = 4.5$ ksi
- Mezzanine concrete on metal deck: $f_c' = 3.0$ ksi
- Reinforcing bars: ASTM A615, Grade 60
- Structural steel (wide flange sections): ASTM A992, Grade 50
- Plates (except continuity plates): ASTM A36
- Bolts: ASTM A325
- Continuity Plates: ASTM A572, Grade 50

### 6.1.3 Structural Design Criteria

#### 6.1.3.1 Building configuration. Because there is a mezzanine at one end, vertical weight irregularities might be considered to apply (Standard Sec. 12.3.2.2). However, the upper level is a roof and the
Standard exempts roofs from weight irregularities. There also are no plan irregularities in this building (Standard Sec. 12.3.2.1).

6.1.3.2 Redundancy. In the N-S direction, the moment frames do not meet the requirements of Standard Section 12.3.4.2b since the frames are only one bay long. Thus, Standard Section 12.3.4.2a must be checked. A copy of the three-dimensional model is made, with the moment frame beam at Gridline A pinned. The structure is checked to make sure that an extreme torsional irregularity (Standard Table 12.3-1) does not occur:

\[
1.4 \left( \frac{\Delta_k + \Delta_A}{2} \right) \geq \Delta_A
\]

\[
1.4 \left( \frac{4.17 \text{ in.} + 6.1 \text{ in.}}{2} \right) = 7.19 \text{ in.} \geq 6.1 \text{ in.}
\]

where:

\( \Delta_A \) = maximum displacement at knee along Gridline A, in.

\( \Delta_K \) = maximum displacement at knee along gridline K, in.

Thus, the structure does not have an extreme torsional irregularity when a frame loses moment resistance.

Additionally, the structure must be checked in the N-S direction to ensure that the loss of moment resistance at Beam A has not resulted in more than a 33 percent reduction in story strength. This can be checked using elastic methods (based on first yield) as shown below, or using strength methods. The original model is run with the N-S load combinations to determine the member with the highest demand-capacity ratio. This demand-capacity ratio, along with the applied base shear, is used to calculate the base shear at first yield:

\[
V_{yield} = \left( \frac{1}{(D/C)_{max}} \right) V_{base}
\]

\[
V_{yield} = \left( \frac{1}{0.89} \right) (223 \text{ kips}) = 250.5 \text{ kips}
\]

where:

\( V_{base} \) = base shear from Equivalent Lateral Force (ELF) analysis

A similar analysis can be made using the model with no moment resistance at Frame A:

\[
V_{yield, MFremoved} = \left( \frac{1}{0.951} \right) (223 \text{ kips}) = 234.5 \text{ kips}
\]

\[
\frac{V_{yield, MFremoved}}{V_{yield}} = \frac{234.5 \text{ kips}}{250.5 \text{ kips}} = 0.94
\]
Thus, the loss of resistance at both ends of a single beam only results in a 6 percent reduction in story strength. The moment frames can be assigned a value of $\rho = 1.0$.

In the E-W direction, the OCBF system meets the prescriptive requirements of *Standard* Section 12.3.4.2a. As a result, no further calculations are needed and this system can be assigned a value of $\rho = 1.0$.

### 6.1.3.3 Orthogonal load effects.
A combination of 100 percent seismic forces in one direction plus 30 percent seismic forces in the orthogonal direction must be applied to the columns of this structure in Seismic Design Category D (*Standard* Sec. 12.5.4).

### 6.1.3.4 Structural component load effects.
The effect of seismic load (*Standard* Sec. 12.4.2) is:

$$ E = \rho Q_E \pm 0.2 S_{DS} D $$

$S_{DS} = 1.0$ for this example. The seismic load is combined with the gravity loads as shown in *Standard* Sec. 12.4.2.3, resulting in the following:

$$ 1.4D + 1.0L + 0.2S + \rho Q_E $$

$$ 0.7D + \rho Q_E $$

Note that $1.0L$ is for the storage load on the mezzanine; the coefficient on $L$ is 0.5 for many common live loads.

### 6.1.3.5 Drift limits.
For a building assigned to Occupancy Category II, the allowable story drift (*Standard* Table 12.12-1) is:

- $\Delta = 0.025h_{ss}$ in the E-W direction
- $\Delta / \rho = 0.025h_{ss} / 1.0$ in the N-S direction

At the roof ridge, $h_{ss} = 34$ ft-3 in. and $\Delta = 10.28$ in.

At the knee (column-roof intersection), $h_{ss} = 30$ ft-6 in. and $\Delta = 9.15$ in.

At the mezzanine floor, $h_{ss} = 12$ ft and $\Delta = 3.60$ in.

Footnote c in *Standard* Table 12.12-1 permits unlimited drift for single-story buildings with interior walls, partitions, etc., that have been designed to accommodate the story drifts. See Section 6.1.4.3 for further discussion. The main frame of the building can be considered to be a one-story building for this purpose, given that there are no interior partitions except below the mezzanine. (The definition of a story in building codes generally does not require that a mezzanine be considered a story unless its area exceeds one-third the area of the room or space in which it is placed; this mezzanine is less than one-third of the footprint of the building.)

### 6.1.3.6 Seismic weight.
The weights that contribute to seismic forces are:
The weight associated with the main frames accounts for only the main columns, because the weight associated with the remainder of the main frames is included in the roof dead load above. The computed seismic weight is based on the assumption that the wall panels offer no shear resistance for the structure but are self-supporting when the load is parallel to the wall of which the panels are a part. Additionally, snow load does not need to be included in the seismic weight per Standard Section 12.7.2 because it does not exceed 30 psf.

### 6.1.4 Analysis

Base shear will be determined using an ELF analysis.

#### 6.1.4.1 Equivalent Lateral Force procedure

In the longitudinal direction where stiffness is provided only by the diagonal bracing, the approximate period is computed using Standard Equation 12.8-7:

\[
T_a = C_s h_n^e = \left(0.02\right)\left(34.25^{0.75}\right) = 0.28 \text{sec}
\]

where \(h_n\) is the height of the building, taken as 34.25 feet at the mid-height of the roof. In accordance with Standard Section 12.8.2, the computed period of the structure must not exceed the following:

\[
T_{max} = C_u T_a = \left(1.4\right)\left(0.28\right) = 0.39 \text{sec}
\]

The subsequent three-dimensional modal analysis finds the computed period to be 0.54 seconds. For purposes of determining the required base shear strength, \(T_{max}\) will be used in accordance with the Standard; drift will be calculated using the period from the model.

In the transverse direction where stiffness is provided by moment-resisting frames (Standard Eq. 12.8-7):

\[
T_a = C_s h_n^e = \left(0.028\right)\left(34.25^{0.8}\right) = 0.47 \text{sec}
\]

and

\[
T_{max} = C_u T_a = \left(1.4\right)\left(0.47\right) = 0.66 \text{sec}
\]

Also note that the dynamic analysis finds a computed period of 1.03 seconds. As in the longitudinal direction, \(T_{max}\) will be used for determining the required base shear strength.

The seismic response coefficient \((C_s)\) is computed in accordance with Standard Section 12.8.1.1. In the longitudinal direction:
\[ C_s = \frac{S_{DS}}{R/I} = \frac{1.0}{3.25/1.0} = 0.308 \]

but need not exceed:

\[ C_s = \frac{S_{DS}}{T(R/I)} = \frac{0.6}{0.39(3.25/1.0)} = 0.473 \]

Therefore, use \( C_s = 0.308 \) for the longitudinal direction.

In the transverse direction:

\[ C_s = \frac{S_{DS}}{R/I} = \frac{1.0}{4.5/1} = 0.222 \]

but need not exceed:

\[ C_s = \frac{S_{DS}}{T(R/I)} = \frac{0.6}{(0.66)(4.5/1)} = 0.202 \]

Therefore, use \( C_s = 0.202 \) for the transverse direction.

In both directions the value of \( C_s \) exceeds the minimum value (Standard Eq. 12.8-5) computed as:

\[ C_s = 0.044JS_{DS} \geq 0.01 = (0.044)(1)(1.0) = 0.044 \]

The seismic base shear in the longitudinal direction (Standard Eq. 12.8-1) is:

\[ V = C_s W = (0.308)(889 \text{kips}) = 274 \text{kips} \]

The seismic base shear in the transverse direction is:

\[ V = C_s W = (0.202)(1102 \text{kips}) = 223 \text{kips} \]

*Standard* Section 12.8.3 prescribes the vertical distribution of lateral force in a multilevel structure. Even though the building is considered to be one story for some purposes, it is clearly a two-level structure. Using the data in Section 6.1.3.6 of this example and interpolating the exponent \( k \) as 1.08 for the period of 0.66 second, the distribution of forces for the N-S analysis is shown in Table 6.1-1.

<table>
<thead>
<tr>
<th>Level</th>
<th>Weight ( (w_x) ) (kips)</th>
<th>Height ( (h_x) ) (ft)</th>
<th>( w_x h_x^k )</th>
<th>( C_{ux} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} )</th>
<th>( F_x = C_{ux} V ) (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>707</td>
<td>32.375</td>
<td>30,231</td>
<td>0.84</td>
<td>187</td>
</tr>
</tbody>
</table>
Table 6.1-1  ELF Vertical Distribution for N-S Analysis

<table>
<thead>
<tr>
<th>Level</th>
<th>Weight ((w_x)) (kips)</th>
<th>Height ((h_x)) (ft)</th>
<th>(w_xh_x^k)</th>
<th>(C_{ux} = \frac{w_xh_x^k}{\sum_{i=1}^{a}W_{ih_i^k}})</th>
<th>(F_x = C_{ux}V) (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mezzanine</td>
<td>395</td>
<td>12</td>
<td>5,782</td>
<td>0.16</td>
<td>36</td>
</tr>
<tr>
<td>Total</td>
<td>1,102</td>
<td>36,013</td>
<td></td>
<td></td>
<td>223</td>
</tr>
</tbody>
</table>

It is not immediately clear whether the roof (a 22-gauge steel deck with conventional roofing over it) will behave as a flexible, semi-rigid, or rigid diaphragm. For this example, a three-dimensional model was created in ETABS including frame and diaphragm stiffness.

6.1.4.2 Three-dimensional ELF analysis. The three-dimensional analysis is performed for this example to account for the following:

- The differing stiffness of the gable frames with and without the mezzanine level
- The different centers of mass for the roof and the mezzanine
- The flexibility of the roof deck
- The significance of braced frames in controlling torsion due to N-S ground motions

The gabled moment frames, the tension bracing, the moment frames supporting the mezzanine and the diaphragm chord members are explicitly modeled using three-dimensional beam-column elements. The tapered members are approximated as short, discretized prismatic segments. Thus, combined axial bending checks are performed on a prismatic element, as required by AISC 360 Chapter H. The collector at the knee level is included, as are those at the mezzanine level in the two east bays. The mezzanine diaphragm is modeled using planar shell elements with their in-plane rigidity being based on actual properties and dimensions of the slab. The roof diaphragm also is modeled using planar shell elements, but their in-plane rigidity is based on a reduced thickness that accounts for compression buckling phenomena and for the fact that the edges of the roof diaphragm panels are not connected to the wall panels. SDI’s *Diaphragm Design Manual* is used for guidance in assessing the stiffness of the roof deck. The analytical model includes elements with one-tenth the stiffness of a plane plate of 22 gauge steel.

The ELF analysis of the three-dimensional model in the transverse direction yields an important result: the roof diaphragm behaves as a rigid diaphragm. Accidental torsion is applied at the center of the roof as a moment whose magnitude is the roof lateral force multiplied by 5 percent of 180 feet (9 feet). A moment is also applied to the mezzanine level in a similar fashion. The resulting displacements are shown in Table 6.1-2.

Table 6.1-2  ELF Analysis Displacements in N-S Direction

<table>
<thead>
<tr>
<th>Grid</th>
<th>Roof Displacement (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>4.98</td>
</tr>
<tr>
<td>B</td>
<td>4.92</td>
</tr>
</tbody>
</table>
The average of the extreme displacements is 4.45 inches. The displacement at the centroid of the roof is 4.51 inches. Thus, the deviation of the diaphragm from a straight line is 0.06 inch, whereas the average frame displacement is approximately 75 times that. Clearly, then, the diaphragm flexibility is negligible and the deck behaves as a rigid diaphragm. The ratio of maximum to average displacement is 1.1, which does not exceed the 1.2 limit given in Standard Table 12.3-1 and torsional irregularity is not triggered.

The same process needs to be repeated for the E-W direction.

The demands from the three-dimensional ELF analysis are combined to meet the orthogonal combination requirement of Standard Section 12.5.3 for the columns:

- E-W: \((1.0)(\text{E-W direction spectrum}) + (0.3)(\text{N-S direction spectrum})\)
- N-S: \((0.3)(\text{E-W direction spectrum}) + (1.0)(\text{N-S direction spectrum})\)

### 6.1.4.3 Drift.

The lateral deflection cited previously must be multiplied by \(C_d = 4\) to find the transverse drift:

\[
\delta_x = \frac{C_d \delta_c}{I} = \frac{(4)(4.51)}{1.0} = 18 \text{ in.}
\]
This exceeds the limit of 10.28 inches computed previously. However, there is no story drift limit for single-story structures with interior wall, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts. Detailing for this type of design may be problematic.

In the longitudinal direction, the lateral deflection is much smaller and is within the limits of Standard Section 12.12.1. The deflection computations do not include the redundancy factor.

6.1.4.4 P-delta effects. The P-delta effects on the structure may be neglected in analysis if the provisions of Standard Section 12.8.7 are followed. First, the stability coefficient maximum should be determined using Standard Equation 12.8-17. \( \beta \) may be assumed to be 1.0.

\[
\theta_{\text{max}} = \frac{0.5}{\beta} C_d \leq 0.25
\]

\[
\theta_{\text{max}, N-S} = \frac{0.5}{(1.0)(4)} = 0.125
\]

\[
\theta_{\text{max}, E-W} = \frac{0.5}{(1.0)(3.25)} = 0.154
\]

Next, the stability coefficient is calculated using Standard Equation 12.8-16. The stability coefficient is calculated at both the roof and mezzanine levels in both orthogonal directions. For purposes of illustration, the roof level check in the N-S direction will be shown as:

\[
\theta = \frac{P_i \Delta I}{V_x h_{xx} C_d} \quad \Delta = \frac{C_d (\delta_{c2} - \delta_{c1})}{I}
\]

\[
\theta = \frac{P_x (\delta_{c2} - \delta_{c1}) C_d I}{V_x h_{xx} C_d I} = \frac{P_x (\delta_{c2} - \delta_{c1})}{V_x h_{xx}}
\]

\[P_{\text{roof}} = \text{Roof LL} + \text{Roof DL} + \text{Panels} + \text{Frames}\]

\[P_{\text{roof}} = (180 \text{ ft} \times 90 \text{ ft}) (15 \text{ psf} + 25 \text{ psf}) + 437 \text{ kips} + 224 \text{ kips} + 27 \text{ kips}\]

\[P_{\text{roof}} = 1336 \text{ kips}\]

\[
\theta = \frac{(1,336 \text{kips})(4.51 \text{ in.})}{(187 \text{kips})(32.375 \text{ ft})} = 0.083 < 0.1
\]

The three other stability coefficients were all determined to be less than \( \theta_{\text{max}} \), thus allowing P-delta effects to be excluded from the analysis.

6.1.4.5 Force summary. The maximum moments and axial forces caused by dead, live and earthquake loads on the gable frames are listed in Tables 6.1-3 and 6.1-4. The frames are symmetrical about their
ridge and the loads are either symmetrical or can be applied on either side on the frame because the forces are given for only half of the frame extending from the ridge to the ground. The moments are given in Table 6.1-4 and the axial forces are given in Table 6.1-5. The moment diagram for the combined load condition is shown in Figure 6.1-4. The load combination is $1.4D + L + 0.2S + \rho Q_e$, which is used throughout the remainder of calculations in this section, unless specifically noted otherwise.

The size of the members is controlled by gravity loads, not seismic loads. The design of connections will be controlled by the seismic loads.

Forces in the design of the braces are discussed in Section 6.1.5.5.

### Table 6.1-4 Moments in Gable Frame Members

<table>
<thead>
<tr>
<th>Location</th>
<th>$D$ (ft-kips)</th>
<th>$L$ (ft-kips)</th>
<th>$S$ (ft-kips)</th>
<th>$Q_e$ (ft-kips)</th>
<th>Combined* (ft-kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 - Ridge</td>
<td>61</td>
<td>0</td>
<td>128</td>
<td>0</td>
<td>112 (279)</td>
</tr>
<tr>
<td>2 - Knee</td>
<td>161</td>
<td>0</td>
<td>333</td>
<td>162</td>
<td>447 (726)</td>
</tr>
<tr>
<td>3 - Mezzanine</td>
<td>95</td>
<td>83</td>
<td>92</td>
<td>137</td>
<td>79</td>
</tr>
<tr>
<td>4 - Base</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

* Combined Load = $1.4D + L + 0.2S + \rho Q_e$ (or $1.2D + 1.6S$). Individual maxima are not necessarily on the same frame; combined load values are maximum for any frame. 1.0 ft = 0.3048 m, 1.0 kip = 1.36 kN-m.

### Table 6.1-5 Axial Forces in Gable Frame Members

<table>
<thead>
<tr>
<th>Location</th>
<th>$D$ (ft-kips)</th>
<th>$L$ (ft-kips)</th>
<th>$S$ (ft-kips)</th>
<th>$\rho Q_e$ (ft-kips)</th>
<th>Combined* (ft-kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 - Ridge</td>
<td>14</td>
<td>3.5</td>
<td>25</td>
<td>0.8</td>
<td>39</td>
</tr>
<tr>
<td>2 - Knee</td>
<td>16</td>
<td>4.5</td>
<td>27</td>
<td>7.0</td>
<td>37</td>
</tr>
<tr>
<td>3 - Mezzanine</td>
<td>39</td>
<td>39</td>
<td>23</td>
<td>26</td>
<td>127</td>
</tr>
<tr>
<td>4 - Base</td>
<td>39</td>
<td>39</td>
<td>23</td>
<td>26</td>
<td>127</td>
</tr>
</tbody>
</table>

* Combined Load = $1.4D + L + 0.2S + \rho Q_e$. Individual maxima are not necessarily on the same frame; combined load values are maximum for any frame. 1.0 ft = 0.3048 m, 1.0 kip = 1.36 kN-m.
6.1.5 Proportioning and Details

The gable frame is shown schematically in Figure 6.1-5. Using the load combinations presented in Section 6.1.3.4 and the loads from Tables 6.1-4 and 6.1-5, the proportions of the frame are checked at the roof beams and the variable-depth columns (at the knee). The mezzanine framing, also shown in Figure 6.1-1, was proportioned similarly. The diagonal bracing, shown in Figure 6.1-1 at the east end of the building, is proportioned using tension forces determined from the three-dimensional ELF analysis.
Figure 6.1-5  Gable frame schematic: Column tapers from 12 in. at base to 36 in. at knee; roof beam tapers from 36 in. at knee to 18 in. at ridge; plate sizes are given in Figure 6.1-7  (1.0 in. = 25.4 mm)

Additionally, the bolted, stiffened, extended end-plate connections must be sized correctly to conform to the pre-qualification standards. AISC 358 Table 6.1 provides parametric limits on the beam and connection sizes. Table 6.1-6 shows these limits as well as the values used for design.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Minimum (in.)</th>
<th>As Designed (in.)</th>
<th>Maximum (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td><em>t</em>&lt;sub&gt;p&lt;/sub&gt;</td>
<td>3/4</td>
<td>1 1/4</td>
<td>2 1/2</td>
</tr>
<tr>
<td><em>b</em>&lt;sub&gt;p&lt;/sub&gt;</td>
<td>9</td>
<td>9</td>
<td>15</td>
</tr>
<tr>
<td><em>g</em></td>
<td>5</td>
<td>5</td>
<td>6</td>
</tr>
<tr>
<td><em>p</em>&lt;sub&gt;fl&lt;/sub&gt;, <em>p</em>&lt;sub&gt;fl0&lt;/sub&gt;</td>
<td>1 3/4</td>
<td>1 3/4</td>
<td>2</td>
</tr>
<tr>
<td><em>p</em>&lt;sub&gt;b&lt;/sub&gt;</td>
<td>3 1/2</td>
<td>3 1/2</td>
<td>3 3/4</td>
</tr>
<tr>
<td><em>d</em></td>
<td>18 1/2</td>
<td>36</td>
<td>36</td>
</tr>
<tr>
<td><em>t</em>&lt;sub&gt;bf&lt;/sub&gt;</td>
<td>19/32</td>
<td>5/8</td>
<td>1</td>
</tr>
<tr>
<td><em>b</em>&lt;sub&gt;bf&lt;/sub&gt;</td>
<td>7 3/4</td>
<td>8</td>
<td>12 1/4</td>
</tr>
</tbody>
</table>
6.1.5.1 Frame compactness and brace spacing. According to Standard Section 14.1.3, steel structures assigned to Seismic Design Category D, E, or F must be designed and detailed per AISC 341. For an intermediate moment frame (IMF), AISC 341, Part I, Section 1, “Scope,” stipulates that those requirements are to be applied in conjunction with AISC 360. Part I, Section 10 of AISC 341 itemizes a few additional items from AISC 360 for intermediate moment frames, but otherwise the intermediate moment frames are to be designed per AISC 360.

AISC 341 requires IMFs to have compact width-thickness ratios per AISC 360, Table B4.1.

All width-thickness ratios are less than the limiting \( \lambda_p \) from AISC 360, Table B4.1. All P-M ratios (combined compression and flexure) are less than 1.00. This is based on proper spacing of lateral bracing.

Lateral bracing is provided by the roof joists. The maximum spacing of lateral bracing is determined using beam properties at the ends and AISC 341, Section 10.8:

\[
L_{b,\text{max}} \leq 0.17 r_y \frac{E}{F_y}
\]

\[
L_{b,\text{max}} \leq 0.17 \left(1.46\text{in.}\right) \left( \frac{29000\text{ksi}}{50\text{ksi}} \right) = 148\text{ in.}
\]

\( L_b \) is 48 inches; therefore, the spacing is OK.

Also, the required brace strength and stiffness are calculated per AISC 360, Equations A-6-7 and A-6-8:

\[
P_{br} = \frac{0.02 r_y C_d}{h_o}
\]

\[
\beta_{br} = \frac{1}{\phi} \left( \frac{10 M_r C_d}{L_b h_o} \right)
\]

where:

\( M_r = R_y Z F_y \)

\( C_d = 1.0 \)

\( h_o = \) distance between flange centroids, in.

\( L_b = \) distance between braces or \( L_p \) (from AISC 360 Eq. F2-5), whichever is greater, in.

\[
M_r = (1.1) \left(309 \text{ in}^3\right)\left(50 \text{ ksi}\right) = 16,992 \text{ in.-kip} = 1,416 \text{ ft-kip}
\]

\[
P_{br} = \frac{0.02 \left(16,992 \text{ in.-kip}\right)(1.0)}{\left(36 \text{ in.} - 5/8 \text{ in.}\right)}
\]
\[ P_{br} = 9.61 \text{ kips} \]

\[ L_p = 1.76r_y \sqrt{\frac{E}{F_y}} \]

\[ L_p = 1.76(1.46 \text{ in.}) \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} = 62 \text{ in.} \]

\[ \beta_{br} = \frac{1}{(0.75)} \left( \frac{10(16,992 \text{ in.-kip})(1.0)}{(62 \text{ in.})(35.375 \text{ in.})} \right) \]

\[ \beta_{br} = 104 \text{ kips/in.} \]

Adjacent to the plastic hinge regions, lateral bracing must have additional strength as defined in AISC 341 10.8

\[ P_u = \frac{0.06M_u}{h_o} \]

\[ P_u = \frac{0.06(16992 \text{ in.-kip})}{(35.375 \text{ in.})} \]

\[ P_u = 28.8 \text{kips} \]

The C-joists used in this structure likely are not adequate to brace the moment frames. Instead, tube brace members will be used, but they are not analyzed in this example.

At the negative moment regions near the knee, lateral bracing is necessary on the bottom flange of the beams and inside the flanges of the columns (Figure 6.1-6).
6.1.5.2 **Knee of the frame.** The knee detail is shown in Figures 6.1-6 and 6.1-7. The vertical plate shown near the upper left corner in Figure 6.1-6 is a gusset providing connection for X-bracing in the longitudinal direction. The beam-to-column connection requires special consideration. The method of AISC 358 for bolted, stiffened end plate connections is used. Refer to Figure 6.1-8 for the configuration. Highlights from this method are shown for this portion of the example. Refer to AISC 358 for a discussion of the entire procedure.
Figure 6.1-7  Bolted stiffened connection at knee
(1.0 in. = 25.4 mm)

The AISC 358 method for bolted stiffened end plate connection requires the determination of the maximum moment that can be developed by the beam. The steps in AISC 358 for bolted stiffened end plates follow:

Step 1. Determine the maximum moment at the plastic hinge location. The end plate stiffeners at the top and bottom flanges increase the local moment of inertia of the beam, forcing the plastic hinge to occur away from the welds at the end of beam/face of column. The stiffeners should be long enough to force the plastic hinge to at least \( d/2 \) away from the end of the beam. With
the taper of the section, the depth will be slightly less than 36 inches at the location of the hinge, but that reduction will be ignored here. The probable maximum moment, $M_{pe}$, at the plastic hinge is computed using AISC 358 Equation 6.9-2 as follows:

$$M_{pe} = C_{pr} R_y F_y Z_x$$

Where, per AISC 358 Equation 2.4.3-2:

$$C_{pr} = \frac{F_y + F_u}{2F_y} \leq 1.2$$

$$C_{pr} = \frac{50 + 65}{2(50)}$$

$$C_{pr} = 1.15$$

where:

$R_y = 1.1$ from AISC 341 Table I-6-1

$Z_x = 309 \text{ in.}^3$

$F_y = 50 \text{ ksi}$

Therefore:

$$M_{pe} = (1.15)(1.1)(50 \text{ ksi})(309 \text{ in.}^3) = 19,541 \text{ in.-kip} = 1,628 \text{ ft-kip}$$

The moment at the column flange, $M_f$, which drives the connection design, is determined from AISC 358 Equation 6.9-2 as follows:

$$M_f = M_{pe} + V_u S_h$$

where:

$V_u =$ shear at location of plastic hinge

$L' =$ distance between plastic hinges

$S_h =$ distance from the face of the column to the plastic hinge, ft.

$$S_h = L_{st} + t_p$$

where:

$L_{st} =$ length of end-plate stiffener, as shown in AISC 358 Figure 6.2.
$t_p =$ thickness of end plate, in.

$$L_{st} = \frac{h_{st}}{\tan 30^\circ}$$

where:

$h_{st} =$ height of the end-plate from the outside face of the beam flange to the end of the end-plate

$$L_{st} = \frac{(7 \text{ in.})}{\tan 30^\circ}$$

$$L_{st} \approx 12.1 \text{ in.}$$

Use $L_{st} = 13 \text{ in.}$

$$S_h = 13 \text{ in.} + 1.25 \text{ in.} = 14.3 \text{ in.}$$

$$L' = L_{out} - 2d_e - 2S_h$$

$$L' = (90 \text{ ft}) - 2(36 \text{ in.}) - 2(14.3 \text{ in.}) = 81.63 \text{ ft}$$

$$V_u = \frac{2M_{pe}}{L'} + V_{gravity}$$

$$V_u = \frac{2(1628 \text{ ft-kip})}{81.63 \text{ ft}} + 18.9 \text{ kips} = 58.8 \text{ kips}$$

$$M_f = 19,541 \text{ in.-kip} + (58.8 \text{ kips})(14.3 \text{ in.})$$

$$M_f = 1,698 \text{ ft-kip} = 20,379 \text{ in.-kip}$$

Step 2. Find bolt size for end plates. For a connection with two rows of two bolts inside and outside the flange, AISC 358 Equation 6.9-7 indicates the following:

$$d_{b \text{ req'd}} = \sqrt{\frac{2M_f}{\pi \phi_n F_{nt} (h_1 + h_2 + h_3 + h_4)}}$$

where:

$F_{nt} =$ nominal tensile stress of bolt, ksi

$h_i =$ distance from the centerline of the beam compression flange to the centerline of the $i^{th}$ tension bolt row, in.

Try A490 bolts. See Figure 6.1-7 for bolt geometry.
Use 1 in. diameter A490N bolts.

Step 3. Determine the minimum end-plate thickness from AISC 358 Equation 6.9-8.

\[ d_{b\text{req'd}} = \sqrt{\frac{2(20,379 \text{ in.-kip})}{\pi(0.9)(113 \text{ ksi})(29.81 \text{ in.} + 33.31 \text{ in.} + 37.44 \text{ in.} + 40.94 \text{ in.})}} = 0.95 \text{ in.} \]

\[ f_{p\text{req'd}} = \sqrt{\frac{1.11M_f}{\phi_d F_{y_p} Y_p}} \]

where:

\[ F_{y_p} = \text{specified minimum yield stress of the end plate material, ksi} \]

\[ Y_p = \text{the end-plate yield line mechanism parameter from AISC 358 Table 6.4} \]

\[ \phi_d = \text{resistance factor for ductile limit states, taken as 1.0} \]

From AISC 358 Table 6.4:

\[ s = \frac{1}{2} \sqrt{b_p g} \]

where:

\[ b_p = \text{width of the end plate, in.} \]

\[ g = \text{horizontal distance between bolts on the end plate, in.} \]

\[ s = \frac{1}{2} \sqrt{(9 \text{ in.})(5 \text{ in.})} = 3.35 \text{ in.} \]

\[ d_e = 7 \text{ in. (see Figure 6.1-7)} \]

Use Case 1 from AISC 358 Table 6.4, since \( d_e > s \)

\[ Y_p = \frac{b_p}{2} \left[ h_1 \left( \frac{1}{2d_e} \right) + h_2 \left( \frac{1}{p_{fo}} \right) + h_3 \left( \frac{1}{p_{fi}} \right) + h_4 \left( \frac{1}{s} \right) \right] + \frac{2}{g} \left[ h_1 \left( d_e + \frac{p_b}{4} \right) + h_2 \left( p_{fo} + \frac{3p_b}{4} \right) + h_3 \left( p_f + \frac{p_b}{4} \right) + h_4 \left( s + \frac{3p_b}{4} \right) + p_b^2 \right] + g \]

where:

\[ p_{fo} = \text{vertical distance between beam flange and the nearest outer row of bolts, in.} \]
\( \textit{p}_b = \text{vertical distance between beam flange and the nearest inner row of bolts, in.} \)

\( \textit{p}_b = \text{distance between the inner and our row of bolts, in.} \)

\[
Y_p = \frac{\left( \text{9 in.} \right)}{2} \left[ \left( \frac{\text{40.94 in.}}{3.35 \text{ in.}} \right) + \left( \frac{\text{37.44 in.}}{1.75 \text{ in.}} \right) \right]
\]

\[
+ \left( \frac{\text{33.31 in.}}{1.75 \text{ in.}} \right) + \left( \frac{\text{29.81 in.}}{3.35 \text{ in.}} \right)
\]

\[
+ \frac{2}{\left( \text{5 in.} \right)} \left[ \left( \text{40.94 in.} \right) + \left( \frac{\text{3.5 in.}}{4} \right) \right] + \left( \frac{\text{37.44 in.}}{1.75 \text{ in.} + \frac{3\left( \text{3.5 in.} \right)}{4}} \right)
\]

\[
+ \left( \text{33.31 in.} \right) + \left( \frac{\text{3.5 in.}}{4} \right) + \left( \frac{\text{29.81 in.}}{3.35 \text{ in.} + \frac{3\left( \text{3.5 in.} \right)}{4}} \right)
\]

\[
+ \left( \text{5 in.} \right)
\]

\( Y_p = 499 \text{ in.} \)

\[
t_{p,req}d = \sqrt{\frac{1.11 \left( \text{20,379 in.-kip} \right)}{\left( 1.0 \right) \left( 36 \text{ ksi} \right) \left( 499 \text{ in.} \right)}} = 1.12 \text{ in.}
\]

Use 1.25-inch thick end-plates.


\[
F_{fu} = \frac{M_f}{d - t_{bf}}
\]

where:

\( d = \text{depth of the beam, in.} \)

\( t_{bf} = \text{thickness of beam flange, in.} \)

\[
F_{fu} = \frac{20,379 \text{ in.-kip}}{36 \text{ in.} - 5/8 \text{ in.}} = 576 \text{ kips}
\]


\[
t_s \geq t_{bw} \left( \frac{F_{sh}}{F_{ys}} \right)
\]

where:

\( t_{bw} = \text{thickness of the beam web, in.} \)
$F_{yb}$ = specified minimum yield stress of beam material, ksi

$F_{ys}$ = specified minimum yield stress of stiffener material, ksi

$$t_s \geq \left( \frac{50}{36} \text{ ksi} \right) \left( \frac{7}{16} \text{ in.} \right) = 0.61 \text{ in.}$$

Use 5/8-inch plates.

The stiffener width-thickness ratio must also comply with AISC 358 Equation 6.9-14.

$$\frac{h_{st}}{t_s} \leq 0.56 \sqrt{\frac{E}{F_{ys}}}$$

$$h_{st} \leq 0.56 \sqrt{\left( \frac{29000 \text{ ksi}}{36 \text{ ksi}} \right) \left( \frac{5/8 \text{ in.}}{\text{ksi}} \right)}$$

$$h_{st} \leq 9.93 \text{ in.}$$

$h_{st} = 7 \text{ in.}$ OK


$$V_u < \phi_n R_n = \phi_n \left( n_b \right) F_v A_b$$

where:

$\phi_n$ = resistance factor for non-ductile limit states, taken as 0.9

$n_b$ = number of bolts at compression flange

$F_v$ = nominal shear stress of bolts from AISC 360 Table J3.2, ksi

$A_b$ = nominal bolt area, in.

$$\phi_n R_n = (0.9)(8)(60)\left( \frac{\pi^2}{4} \right) = 339 \text{ kips} > 58.8 \text{ kips}$$ OK

Step 7. Check bolt bearing/tear-out of the end-plate and column flange by AISC 358 Equation 6.9-17.

$$V_u < \phi_n R_n = \phi_n \left( n_i \right) r_{ni} + \phi_n \left( n_o \right) r_{no}$$

where:

$n_i$ = number of inner bolts
\( n_o = \text{number of outer bolts} \)

\[
\begin{align*}
    r_{ni} &= 1.2L_c t F_u < 2.4d_b t F_u \text{ for each inner bolt} \\
    r_{no} &= 1.2L_c t F_u < 2.4d_b t F_u \text{ for each outer bolt}
\end{align*}
\]

\( L_c = \text{clear distance, in the direction of force, between the edge of the hole and the edge of the adjacent hole or edge of the material, in.} \)

\( t = \text{end-plate or column flange thickness, in.} \)

\( F_u = \text{specified minimum tensile strength of end-plate or column flange material, ksi} \)

\( d_b = \text{diameter of bolt, in.} \)

\[
L_{ci} = p_b - d_e
\]

\[
L_{co} = L_c - \frac{d_e}{2}
\]

where:

\( d_e = \text{effective area of bolt hole, in.} \)

\( L_c = \text{edge spacing of the bolts, in.} \)

\[
L_{ci} = 3.5 \text{ in.} - 1 \frac{3}{8} \text{ in.} = 2.38 \text{ in.}
\]

\[
L_{co} = 1.75 \text{ in.} - \frac{1 \frac{3}{8} \text{ in.}}{2} = 1.19 \text{ in.}
\]

\[
\begin{align*}
    r_{ni} &= 1.2 \left( 2.38 \text{ in.} \right) \left( 1.25 \text{ in.} \right) (58 \text{ ksi}) < 2.4 \left( 1 \text{ in.} \right) (1.25 \text{ in.}) (58 \text{ ksi}) \\
    r_{ni} &= 207 \text{ kips} < 174 \text{ kips} \\
    r_{ni} &= 174 \text{ kips}
\end{align*}
\]

\[
\begin{align*}
    r_{no} &= 1.2L_c t F_u < 2.4d_b t F_u \\
    r_{no} &= 1.2 \left( 1.19 \text{ in.} \right) \left( 1.25 \text{ in.} \right) (58 \text{ ksi}) < 2.4 \left( 1 \text{ in.} \right) (1.25 \text{ in.}) (58 \text{ ksi}) \\
    r_{no} &= 103 \text{ kips} < 174 \text{ kips} \\
    r_{no} &= 103 \text{ kips}
\end{align*}
\]
\[ V_u < \phi_{n} R_u = (1)(4)(174 \text{ kips}) + (1)(4)(103 \text{ kips}) \]

\[ \phi_{n} R_u = 998 \text{ kips} > 58.8 \text{ kips} \quad \text{OK} \]

Step 8. Check the column flange for flexural yielding by AISC 358 Equation 6.9-20.

\[ t_{cf \; 	ext{req'd}} = \sqrt{\frac{1.11 M_f}{\phi_{n} F_{yc} Y_c}} \leq t_{cf} \]

where:

- \( F_{yc} \) = specified minimum yield stress of column flange material, ksi
- \( Y_c \) = stiffened column flange yield line from AISC 358 Table 6.6
- \( t_{cf} \) = column flange thickness, in.

\[ Y_c = \frac{b_{cf}}{2} \left[ h_1 \left( \frac{1}{s} \right) + h_2 \left( \frac{1}{P_{so}} \right) + h_3 \left( \frac{1}{P_{si}} \right) + h_4 \left( \frac{1}{s} \right) \right] \]

\[ + \frac{2}{g} \left[ h_1 \left( s + \frac{P_b}{4} \right) + h_2 \left( P_{so} + \frac{3P_b}{4} \right) + h_3 \left( P_{si} + \frac{P_b}{4} \right) + h_4 \left( s + \frac{3P_b}{4} \right) + P_b^2 \right] + g \]

where:

- \( b_{cf} \) = column flange width, in.
- \( P_{si} \) = distance from column stiffener to inner bolts, in.
- \( P_{so} \) = distance from column stiffener to outer bolts, in.

\[ s = \frac{1}{2} \sqrt{b_{cf} g} \]

\[ s = \frac{1}{2} \sqrt{(8 \text{ in.})(5 \text{ in.})} = 3.16 \text{ in.} \]
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\[
Y_c = \frac{8 \text{ in.}}{2} \left[ (40.94 \text{ in.}) \left( \frac{1}{3.16 \text{ in.}} \right) + (37.44 \text{ in.}) \left( \frac{1}{1.75 \text{ in.}} \right) \\
+ (33.31 \text{ in.}) \left( \frac{1}{1.75 \text{ in.}} \right) + (29.81 \text{ in.}) \left( \frac{1}{3.16 \text{ in.}} \right) \right] \\
+ \frac{2}{(5 \text{ in.})} \left[ (40.94 \text{ in.}) \left( 3.16 \text{ in.} + \frac{3.5 \text{ in.}}{4} \right) + (37.44 \text{ in.}) \left( 1.75 \text{ in.} + \frac{3(3.5 \text{ in.})}{4} \right) \\
+ (33.31 \text{ in.}) \left( 1.75 \text{ in.} + \frac{3.5 \text{ in.}}{4} \right) + (29.81 \text{ in.}) \left( 3.16 \text{ in.} + \frac{3(3.5 \text{ in.})}{4} \right) + (3.5 \text{ in.})^2 \right] \\
+ (5 \text{ in.}) \\
Y_c = 497 \text{ in.}
\]

\[
t_{cf\text{ req'd}} = \frac{1.11(20,379 \text{ in.-kip})}{\sqrt{(1)(50 \text{ ksi})(497 \text{ in.})}} \leq 2 \text{ in.}
\]

\[
t_{cf\text{ req'd}} = 0.95 \text{ in.} \leq 2 \text{ in.}
\]

Column flange of 2 inches is OK.


\[
\phi_d M_{cf} = \phi_d F_y M_{yc} t_{cf}^2
\]

\[
\phi_d M_{cf} = (1)(50 \text{ ksi})(497 \text{ in.})(2 \text{ in.})^2 = 99,344 \text{ in.-kip}
\]

The equivalent column flange design force used for stiffener design by AISC 358 Equation 6.9-22.

\[
\phi_d R_n = \frac{\phi_d M_{cf}}{(d - t_{bf})}
\]

\[
\phi_d R_n = \frac{(99,690 \text{ in.-kip})}{(36 \text{ in.}) - (5 \text{ in.})/8} = 2,808 \text{ kips}
\]

2,808 kips > 576 kips

OK


\[
\phi_d R_n \geq F_{fu}
\]
\[
\phi_d R_n = \phi_d C_t \left( 6k_c + t_{bf} + 2t_p \right) F_{yc} t_{cw}
\]

where:

\( C_t = 0.5 \) if the distance from the column top to the top of the beam flange is less than the depth of the column; otherwise 1.0

\( k_c = \) distance from outer face of the column flange to web toe of fillet weld, in.

\( t_p = \) end-plate thickness, in.

\( F_{yc} = \) specified yield stress of the column web material, ksi

\( t_{cw} = \) column web thickness, in.

\( t_{bf} = \) beam flange thickness, in.

\[
\phi_d R_n = (1.0)(0.5)(6(2 \text{ in.} + \frac{0.5}{6} \text{ in.}) + (0.5 \text{ in.}) + 2(1.25 \text{ in.}))(50 \text{ ksi})(0.5 \text{ in.})
\]

\[
\phi_d R_n = 212 \text{ kips} \leq 576 \text{ kips}
\]

The design is not acceptable. Column stiffeners need to be provided.

Step 11. Check the unstiffened column web buckling strength at the beam compression flange by AISC 358 Equations 6.9-25 and 6.9-27.

\[
\phi R_n \geq F_{fu}
\]

\[
\phi R_n = \phi \frac{12t_{cw}^3 \sqrt{EF_{yc}}}{h}
\]

where:

\( h = \) clear distance between flanges when welds are used for built-up shapes, in.

\[
\phi R_n = (0.75) \frac{12(0.5 \text{ in.})^3 \sqrt{(29,000 \text{ ksi})(50 \text{ ksi})}}{(36 \text{ in.} - 2 \text{ in.} - 0.75 \text{ in.})} = 42 \text{ kips}
\]

42 kips \( < \) 576 kips NOT OK

Step 12. Check the unstiffened column web crippling strength at the beam compression flange by AISC 358 Equation 6-30.

\[
\phi R_n \geq F_{fu}
\]
\[ \phi R_n = \phi 0.40 t_{cw}^2 \left[ 1 + 3 \left( \frac{N}{d_c} \right) \left( \frac{t_{cw}}{t_{cf}} \right)^{1.5} \right] \sqrt{\frac{EF_{ye} t_{cf}}{t_{cw}}} \]

where:

\( N \) = thickness of beam flange plus 2 times the groove weld reinforcement leg size, in.

\( d_c \) = overall depth of the column, in.

\( R_n = (0.75)(0.80)(0.5 \text{ in.})^2 \left[ 1 + 3 \left( \frac{3(5/8 \text{ in.})}{36 \text{ in.}} \right) \left( \frac{0.5 \text{ in.}}{2 \text{ in.}} \right)^{1.5} \right] \sqrt{\frac{29,000 \text{ ksi}}{0.5 \text{ in.}}} \]

\[ R_n = 184 \text{ kips} \]

184 kips < 576 kips \hspace{1cm} \text{NOT OK}

Step 13. Check the required strength of the stiffener plates by AISC 358 Equation 6-32.

\[ F_{su} = F_{fu} - \min \phi R_n = 576 \text{ kips} - 42 \text{ kips} = 534 \text{ kips} \]

where:

\( \min \phi R_n \) = the minimum design strength value from column flange bending check, column web yielding, column web buckling and column web crippling check

Although AISC 358 says to use this value of 534 kips to design the continuity plate, a different approach will be used in this example. In compression, the continuity plate will be designed to take the full force delivered by the beam flange, \( F_{su} \). In tension, however, the compressive limit states (web buckling and web yielding) are not applicable and column web yielding will control the design instead. The tension design force can be taken as follows:

\[ F_{su} = F_{fu} - \phi R_{n,web \ yielding} = 576 \text{kips} - 212 \text{ kips} = 364 \text{kips} \]

Step 14. Design the continuity plate for required strength by AISC 360 Section J10.

Find the cross-sectional area required by the continuity plate acting in tension:

\[ A_{s, \text{reqd}} = \frac{F_{su}}{\phi F_y} \]

\[ A_{s, \text{reqd}} = \frac{364 \text{ kips}}{(0.9)(50 \text{ ksi})} = 8.1 \text{ in.}^2 \]
Use a 1-3/8-inch continuity plate. As it will be shown later, net section rupture (not gross yielding) will control the design of this plate.

From AISC Section J10.8, calculate member properties using an effective length of $0.75h$ and a column web length of $12t_w = 6$ in.:

$$I_x = \frac{t_{cw} 12 t_{cw}^3}{12} + \frac{(b_{st} - t_{cw}) t_{st}^3}{12}$$

$$I_x = \frac{(6 \text{ in.})^3}{12} + \frac{(8 \text{ in.} - 0.5 \text{ in.})(1.375 \text{ in.})^3}{12} = 10.6 \text{ in.}^4$$

$$I_y = \frac{t_{st} b_{st}^3}{12} + \frac{(12t_{cw} - t_{st}) t_{cw}^3}{12}$$

$$I_y = \frac{(1.375 \text{ in.})(8 \text{ in.})^3}{12} + \frac{(6 \text{ in.} - 1.375 \text{ in.})(0.5 \text{ in.})^3}{12} = 58.7 \text{ in.}^4$$

$$J \approx \frac{b_{st} t_{st}^3}{3} + \frac{(12t_{cw} - t_{st}) t_{cw}^3}{3}$$

$$J \approx \frac{(8 \text{ in.})(1.375 \text{ in.})^3}{3} + \frac{(6 \text{ in.} - 1.375 \text{ in.})(0.5 \text{ in.})^3}{3} = 7.13 \text{ in.}^4$$

$$A = b_{st} t_{st} + (12t_{cw} - t_{st}) t_{cw}$$

$$A = (8 \text{ in.})(1.375 \text{ in.}) + (6 \text{ in.} - 1.375 \text{ in.})(0.5 \text{ in.}) = 13.3 \text{ in.}^2$$

$$r_y = \sqrt{\frac{I_y}{A}}$$

$$r_y = \sqrt{\frac{58.7 \text{ in.}^4}{13.3 \text{ in.}^2}} = 2.1 \text{ in.}$$

$$L = 0.75h = 0.75 \left( d_e - t_{f1} - t_{f2} - 2t_{weld} \right)$$

$$L = 0.75 \left( 36 \text{ in.} - 2 \text{ in.} - 0.75 \text{ in.} - 2(5/16 \text{ in.}) \right) = 24.5 \text{ in.}$$
Check the continuity plate in compression from AISC 360 Equation J4.4:

\[
\frac{KL}{r_y} = \frac{(1.0)(24.5 \text{ in.})}{2.1 \text{ in.}} = 11.65 \leq 25
\]

Strength in the other direction does not need to be checked because the cruciform section will not buckle in the plane of the column web.

Since \(KL/r\) is less than 25, use AISC 360 Equation J4-6 to determine compression strength:

\[
\phi P_n = \phi F_y A_g
\]

\[
\phi P_n = (0.9)(50 \text{ ksi})(13.3 \text{ in.}^2) = 599 \text{ kips}
\]

However, torsional buckling may control. Therefore, check flexural-torsional buckling using AISC 360 Equation E4-4:

\[
F_e = \frac{GJ}{I_x + I_y}
\]

\[
F_e = \frac{(11,200 \text{ ksi})(7.13 \text{ in.}^4)}{10.6 \text{ in.}^4 + 58.7 \text{ in.}^4} = 1,150 \text{ ksi}
\]

\[
F_{e,x} = \left[ 0.658 \left( \frac{F_e}{F_y} \right)^{\frac{50}{1350 \text{ ksi}}} \right] F_y
\]

\[
F_{e,x} = \left[ 0.658 \left( \frac{1150}{50 \text{ ksi}} \right)^{\frac{50}{1350 \text{ ksi}}} \right] 50 \text{ ksi} = 49.1 \text{ ksi}
\]

\[
\phi P_n = (0.9)(49.1 \text{ ksi})(13.3 \text{ in.}^2) = 588 \text{ kips}
\]

Check the continuity plate in tension. The continuity plate had been previously sized for adequacy to tensile yielding of the gross section. Now tensile rupture of the net section must be checked using AISC 360 Section D2-2. The critical section will be analyzed where the continuity plates are clipped adjacent to the k-region of the column.

\[
A_e = (t_u)(b_w - 2(t_{weld} + \text{clip}))
\]

\[
A_e = (1.375 \text{ in.})(8 \text{ in.} - 2(5/16 \text{ in.} + 0.5 \text{ in.})) = 8.77 \text{ in.}^2
\]

\[
\phi_i P_n = \phi_i F_u A_e = 381 \text{ kips} > 364 \text{ kips}
\]

OK
Step 15. Check the panel zone for required strength per AISC 341 Equation J10-9.

\[ P_c = F_y A \]

where:

A = column cross sectional area, in\(^2\).

\[ P_c = (50 \text{ ksi})(38.6 \text{ in.}) = 1,931 \text{ kips} \]

\[ P_r = F_{fu} = 576 \text{ kips} \]

\[ \frac{P_r}{P_c} = \frac{576 \text{ kips}}{1,931 \text{ kips}} = 0.3 < 0.75 \]

Therefore, use AISC 360 Equation J10-111. Note that panel zone flexibility was accounted for in the ETABS model.

\[ \phi R_n = \phi 0.60 F_y d_c t_{cw} \left(1 + \frac{3b_c t_{cf}}{d_b d_c t_{cw}}\right) \]

\[ \phi R_n = (0.9)(0.60)(50 \text{ ksi})(36 \text{ in.})(0.5 \text{ in.}) \left(1 + \frac{3(8 \text{ in.})(2 \text{ in.})^2}{(36 \text{ in.})(36 \text{ in.})(0.5 \text{ in.})}\right) \]

\[ \phi R_n = 558 \text{ kips} < 576 \text{ kips} \quad \text{NOT OK} \]

The column web is not sufficient to resist the panel zone shear. Although doubler plates can be added to the panel zone to increase strength, this may be an expensive solution. A more economical solution would be to simply upsize the column web to a sufficient thickness, such as 5/8 inch.

\[ \phi R_n = \phi 0.60 F_y d_c t_{cw} \left(1 + \frac{3b_c t_{cf}}{d_b d_c t_{cw}}\right) \]

\[ \phi R_n = (0.9)(0.60)(50 \text{ ksi})(36 \text{ in.})(5/8 \text{ in.}) \left(1 + \frac{3(8 \text{ in.})(2 \text{ in.})^2}{(36 \text{ in.})(36 \text{ in.})(5/8 \text{ in.})}\right) \]

\[ \phi R_n = 680 \text{ kips} > 576 \text{ kips} \]

Note that changing the column member properties might affect the analysis results. In this example, this is not the case, although the slight difference in web thickness would result in
marginally different values for some of the end-plate connection calculations. For simplicity, these changes are not undertaken in this example.

6.1.5.3 Frame at the ridge. The ridge joint detail is shown in Figure 6.1-8. An unstiffened bolted connection plate is selected.

This is an AISC 360 designed connection, not an AISC 358 designed connection because there should not be a plastic hinge forming in this vicinity. Lateral seismic forces produce no moment at the ridge until yielding takes place at one of the knees. Vertical accelerations on the dead load do produce a moment at this point; however, the value is small compared to all other moments and does not appear to be a concern. Once seismic loads produce a plastic hinge at one knee, further lateral displacement produces positive moment at the ridge. Under the condition on which the AISC 358 design is based (a full plastic moment is produced at each knee), the moment at the ridge will simply be the static moment from the gravity loads less the horizontal thrust times the rise from knee to ridge. Analyzing this frame under the gravity load case $1.2D + 0.2S$, the static moment is 406 ft-kip and the reduction for the thrust is 128 ft-kip, leaving a net positive moment of 278 ft-kip, coincidentally close to the design moment for the factored gravity loads.

6.1.5.4 Design of mezzanine framing. The design of the framing for the mezzanine floor at the east end of the building is controlled by gravity loads. The concrete-filled 3-inch, 20-gauge steel deck of the mezzanine floor is supported on steel beams spaced at 10 feet and spanning 20 feet (Figure 6.1-2). The steel beams rest on three-span girders connected at each end to the portal frames and supported on two intermediate columns (Figure 6.1-1). The girder spans are approximately 30 feet each. Those lateral forces that are received by the mezzanine are distributed to the frames and diagonal bracing via the floor diaphragm. A typical beam-column connection at the mezzanine level is provided in Figure 6.1-9. The design of the end plate connection is similar to that at the knee, but simpler because the beam is horizontal and not tapered. Also note that demands on the end-plate connection will be less because this connection is not at the end of the column.
6.1.5.5 Braced frame diagonal bracing

Although the force in the diagonal X-braces can be either tension or compression, only the tensile value is considered because it is assumed that the diagonal braces are capable of resisting only tensile forces.

See AISC 341 Section 14.2 for requirements on braces for OCBFs. The strength of the members and connections, including the columns in this area but excluding the brace connections, must be based on Standard Section 12.4.2.3:

\[
1.4D + 1.0L + 0.2S + \rho Q_E
\]

\[
0.7D + \rho Q_E
\]

Recall that a 1.0 factor is applied to \( L \) when the live load is greater than 100 psf (Standard Sec. 2.3.2). For the case discussed here, the “tension only” brace does not carry any live or dead load, so the load factor does not matter.

For simplicity, we can assume that the lateral force is equally divided among the roof level braces and is slightly amplified to account for torsional effects. Thus the brace force can be approximated using the following equation:
\( P_u = 0.55V \times \frac{1}{2} \times \frac{1}{\cos \theta} \) \[6-37\]

\[ P_u = 0.55 \times 211 \text{kips} \times \frac{1}{2} \times \frac{1}{\cos 47^\circ} = 85 \text{kips} \]

All braces at this level will have the same design. Choose a brace member based on tensile yielding of the gross section by AISC 360 Equation D2-1:

\[ \phi L P_n = \phi F_y A_g \]

\[ A_{g,\text{reqd}} = \frac{85 \text{kips}}{(0.9)(36 \text{ksi})} = 2.62 \text{ in.}^2 \]

This also needs to be checked for tensile rupture of the net section. Demand will be taken as either the expected yield strength of the brace or the amplified seismic load. Try a 2L3\(\frac{3}{8}\)x3x 7/16, which is the smallest seismically compact angle shape available.

\[ A_g = 5.34 \text{ in.}^2 \]

The \(Kl/r\) requirement of AISC 341 Section 14.2 does not apply because this is not a V or an inverted V configuration.

Check net rupture by AISC 360 Equation D2-2 and D3-1:

\[ \phi L P_n = \phi F_y A_e \]

\[ A_e = A_n U \]

Determine the shear lag factor, \(U\), from AISC 360, Table D3.1, Case 2. In order to calculate \(U\), the weld length along the double angles needs to be determined.

\[ U = 1 - \frac{x}{L} \]

Brace connection demand is given as the expected yield strength of the brace in tension per AISC 341 Section 14.4.

\[ R_y F_y A_g = (1.5)(36 \text{ksi})(5.34 \text{in.}^2) = 288 \text{kips} \]

Expected yield strength of the brace is 288 kips. However, AISC 341 Section 14.4b limits the brace connection design force to the amplified seismic load.

\[ \Omega_0 P_{QE} = (2)(85 \text{kips}) = 170 \text{kips} \]
Use four fillet welds, two on each angle. Try 1/4-inch welds using AISC 360 Equation J2-3:

\[
\phi R_n = \phi F_w A_w
\]

\[
\phi R_n = \phi \left(0.60 F_{Exx}\right) \left(0.707 t_w\right) (L)
\]

\[
L = \frac{170 \text{ kips}}{4(0.75)(0.6)(70 \text{ ksi})(0.707)(0.25 \text{ in.})} = 7.63 \text{ in.}
\]

Use four 1/4-inch fillet welds 8 inches long.

Check the base metal:

\[
\phi R_n = \phi F_{BM} A_{BM}
\]

Shear yielding from AISC 360 Equation J4-3:

\[
\phi R_n = \phi 0.6 F_y A_g
\]

\[
\phi R_n = (1.0) \left(0.6\right) \left(36 \text{ ksi}\right) \left(0.25 \text{ in.}\right) (8 \text{ in.}) = 173 \text{kips}
\]

OK

Shear rupture from AISC 360 Equation J4-4:

\[
\phi R_n = \phi 0.6 F_u A_{nv}
\]

\[
\phi R_n = (0.75) \left(0.6\right) \left(58 \text{ ksi}\right) (0.25 \text{ in.}) (8 \text{ in.})
\]

OK

Calculate the shear lag factor and the effective net area:

\[
U = 1 - \frac{(0.846 \text{ in.})}{(9 \text{ in.})} = 0.89
\]

\[
A_e = \left(5.34 \text{ in.}^2\right) (0.89) = 4.78 \text{ in.}^2
\]

Calculate the tensile rupture strength:

\[
\phi P_n = (0.9) \left(58 \text{ ksi}\right) \left(6.1 \text{ in.}^2\right) = 207 \text{kips} > 170 \text{kips}
\]

OK

Additionally, the capacity of the eave strut at the roof must be checked. The eave strut, part of the braced frame, also acts as a collector element and must be designed using the overstrength factor per Standard Section 12.10.2.1.

6.1.5.6 Roof deck diaphragm. Figure 6.1-10 shows the in-plane shear force in the roof deck diaphragm for both seismic loading conditions. There are deviations from simple approximations in both directions. In the E-W direction, the base shear is 274 kips (Section 6.1.4.2) with 77 percent or 211 kips at the roof. Torsion is not significant, so a simple approximation is to take half the force to each side and divide by
the length of the building, which yields \((211,000/2)/180\) feet = 586 plf. The plot shows that the shear in the edge of the diaphragm is significantly higher in the two braced bays. This is a shear lag effect; the eave strut in the three-dimensional model is a HSS 6x6x1/4. In the N-S direction, the shear is generally highest in the bay between the mezzanine frame and the first frame without the mezzanine. This is expected given the significant change in stiffness. There is no simple approximation to estimate the shear here without a three-dimensional model. The shear is also high at the longitudinal braced bays because they tend to resist the horizontal torsion. However, the shear at the braced bays is lower than observed for the E-W motion.

**Figure 6.1-10** Shear force in roof deck diaphragm; upper diagram is for E-W motion and lower is for N-S motion
6.2 SEVEN-STORY OFFICE BUILDING, LOS ANGELES, CALIFORNIA

Two alternative framing arrangements for a seven-story office building are illustrated.

6.2.1 Building Description

6.2.1.1 General description. This seven-story office building of rectangular plan configuration is 177 feet, 4 inches long in the E-W direction and 127 feet, 4 inches wide in the N-S direction (Figure 6.2-1). The building has a penthouse. It is framed in structural steel with 25-foot bays in each direction. The typical story height is 13 feet, 4 inches; the first story is 22 feet, 4 inches high. The penthouse extends 16 feet above the roof level of the building and covers the area bounded by Gridlines C, F, 25 in Figure 6.2-1. Floors consist of 3-1/4-inch lightweight concrete over composite metal deck. The elevators and stairs are located in the central three bays.

6.2.1.2 Alternatives. This example includes two alternatives—a steel moment-resisting frame and a concentrically braced frame:

- Alternative A: Seismic force resistance is provided by special moment frames (SMF) with prequalified Reduced Beam Section (RBS) connections located on the perimeter of the building (on Gridlines A, H, 16 in Figure 6.2-1, also illustrated in Figure 6.2-2). There are five bays of moment frames on each line.

- Alternative B: Seismic force resistance is provided by four special concentrically braced frames (SCBF) in each direction. They are located in the elevator core walls between Columns 3C and 3D, 3E and 3F, 4C and 4D, 4E and 4F in the E-W direction and between Columns 3C and 4C, 3D and 4D, 3E and 4E, 3F and 4F in the N-S direction (Figure 6.2-1). The braced frames are in a two-story X configuration. The frames are identical in brace size and configuration, but there are some minor differences in beam and column sizes. Braced frame elevations are shown in Figures 6.2-10 through 6.2-12.

6.2.1.3 Scope. The example covers:

- Seismic design parameters (Sec. 6.2.2.1)
- Analysis of perimeter moment frames (Sec. 6.2.4.1)
- Beam and column proportioning (Sec. 6.2.4.2.3)
- Moment frame connection design (Sec. 6.2.4.2.5)
- Analysis of concentrically braced frames (Sec. 6.2.5.1)
- Proportioning of braces (Sec. 6.2.5.2.1)
- Braced frame connection design (Sec. 6.2.5.2.5)
Figure 6.2-1 Typical floor framing plan and building section
(1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m)
6.2.2 Basic Requirements

6.2.2.1 Provisions parameters. See Section 3.2 for an example illustrating the determination of design ground motion parameters. For this example, the parameters are as follows:

- $S_{DS} = 1.0$
- $S_{DI} = 0.6$
- Occupancy Category II
- Seismic Design Category D

For Alternative A, Special Steel Moment Frame *(Standard Table 12.2-1)*

- $R = 8$
- $\Omega_0 = 3$
- $C_d = 5.5$
For Alternative B, Special Steel Concentrically Braced Frame (Standard Table 12.2-1):

- $R = 6$
- $\Omega_0 = 2$
- $C_d = 5$

### 6.2.2.2 Loads.

- Roof live load ($L$): 25 psf
- Penthouse roof dead load ($D$): 25 psf
- Exterior walls of penthouse: 25 psf of wall
- Roof $DL$ (roofing, insulation, deck beams, girders, fireproofing, ceiling, mechanical, electrical plumbing): 55 psf
- Exterior wall cladding: 25 psf of wall
- Penthouse floor $D$: 65 psf
- Penthouse Equipment: 39 psf
- Floor $L$: 50 psf
- Floor $D$ (deck, beams, girders, fireproofing, ceiling, mechanical electrical, plumbing, partitions): 68 psf
- Floor $L$ reductions: per the IBC

### 6.2.2.3 Basic gravity loads.

- Penthouse roof:
  
  - Roof slab = $(0.025 \text{ ksf})(25 \text{ ft})(75 \text{ ft}) = 47 \text{ kips}$
  - Walls = $(0.025 \text{ ksf})(8 \text{ ft})(200 \text{ ft}) = 40 \text{ kips}$
  - Columns = $(0.110 \text{ ksf})(8 \text{ ft})(8 \text{ ft}) = 7 \text{ kips}$
  - Total = 94 kips

- Lower roof:
  
  - Roof slab = $(0.055 \text{ ksf})(127.33 \text{ ft})(177.33 \text{ ft}) - (25 \text{ ft})(75 \text{ ft}) = 1139 \text{ kips}$
  - Penthouse floor = $(0.065 \text{ ksf})(25 \text{ ft})(75 \text{ ft}) = 122 \text{ kips}$
  - Walls = 40 kips + $(0.025 \text{ ksf})(609 \text{ ft})(6.67 \text{ ft}) = 142 \text{ kips}$
  - Columns = 7 kips + $(0.170 \text{ ksf})(6.67 \text{ ft})(48 \text{ ft}) = 61 \text{ kips}$
  - Equipment = $(0.039 \text{ ksf})(25 \text{ ft})(75 \text{ ft}) = 73 \text{ kips}$
  - Total = 1,537 kips
Typical floor:

- Floor = (0.068 ksf)(127.33 ft)(177.33 ft) = 1,535 kips
- Walls = (0.025 ksf)(609 ft)(13.33 ft) = 203 kips
- Columns = (0.285 ksf)(13.33 ft)(48 ft) = 182 kips
- Total = 1,920 kips

Total weight of building = 94 kips + 1,537 kips + 6 (1,920 kips) = 13,156 kips

6.2.2.4 Materials

- Concrete for floors: $f'_{c} = 3$ ksi, lightweight (LW)
- All other concrete: $f'_{c} = 4$ ksi, normal weight (NW)
- Structural steel:
  - Wide flange sections: ASTM A992, Grade 50
  - HSS: ASTM A500, Grade B
  - Plates: ASTM A36

6.2.3 Structural Design Criteria

6.2.3.1 Building configuration. The building has no vertical irregularities despite the relatively tall height of the first story. The exception of Standard Section 12.3.2.2 is taken, in which the drift ratio of adjacent stories are compared rather than the stiffness of the stories. In the three-dimensional analysis, the first story drift ratio is less than 130 percent of that for the story above. Because the building is symmetrical in plan, plan irregularities would not be expected. Analysis reveals that Alternative B is torsionally irregular, which is not uncommon for core-braced buildings.

6.2.3.2 Orthogonal load effects. A combination of 100 percent of the seismic forces in one direction with 30 percent of the seismic forces in the orthogonal direction is required for structures in Seismic Design Category D for certain elements—namely, the shared columns in the SCBF (Standard Sec. 12.5.4). In using modal response spectrum analysis (MRSA), the bidirectional case is handled by using the square root of the sum of the squares (SRSS) of the orthogonal spectra.

6.2.3.3 Structural component load effects. The effect of seismic load is defined by Standard Section 12.4.2 as:

$$E = \rho Q_E + 0.2S_{DS}D$$

Using Standard Section 12.3.4.2, $\rho$ is 1.0 for Alternative A and 1.3 for Alternative B. (For simplicity, $\rho$ is taken as 1.3; the design does not comply with the prescriptive requirements of Standard Sec. 12.3.4.2. It is assumed that the design would fail the calculation-based requirements of Standard Sec. 12.3.4.2.) Substitute for $\rho$ (and for $S_{DS} = 1.0$).

- For Alternative A:
  $$E = Q_E \pm 0.2D$$
- Alternative B:
\[ E = 1.3Q_E \pm 0.2D \]

### 6.2.3.4 Load combinations

Load combinations from ASCE 7-05 are as follows:

- \(1.4D\)
- \(1.2D + 1.6L + 0.5L_r\)
- \(1.2D + L + 1.6L_r\)
- \((1.2 + 0.2S_{D0})D + 0.5L + \rho Q_E\)
- \((0.9 - 0.2S_{D0})D + \rho Q_E\)

For each of these load combinations, substitute \(E\) as determined above, showing the maximum additive and minimum subtractive. \(Q_E\) acts both east and west (or north and south):

- **Alternative A:**
  
  1.4\(D\)
  
  1.2\(D\) + 1.6\(L\) + 0.5\(L_r\)
  
  1.2\(D\) + \(L\) + 1.6\(L_r\)
  
  1.4\(D\) + 0.5\(L\) + \(Q_E\)
  
  0.7\(D\) + \(Q_E\)

- **Alternative B**
  
  1.4\(D\)
  
  1.2\(D\) + 1.6\(L\) + 0.5\(L_r\)
  
  1.2\(D\) + \(L\) + 1.6\(L_r\)
  
  1.4\(D\) + 0.5\(L\) + 1.3\(Q_E\)
  
  0.7\(D\) + 1.3\(Q_E\)

For both cases, six scaled response spectrum cases are used:

1) Spectrum in X direction

2) Spectrum in X direction with 5 percent eccentricity

3) Spectrum in Y direction

4) Spectrum in Y direction with 5 percent eccentricity

5) SRSS combined spectra in X and Y directions

6) SRSS combined spectra in X and Y directions with 5 percent eccentricity.

### 6.2.3.5 Drift limits

The allowable story drift per Standard Section 12.12.1 is \(\Delta_a = 0.02h_{cr}\).
The allowable story drift for the first floor is \( \Delta_a = (0.02)(22.33 \text{ ft})(12 \text{ in./ft}) = 5.36 \text{ in.} \)

The allowable story drift for a typical story is \( \Delta_a = (0.02)(13.33 \text{ ft})(12 \text{ in./ft}) = 3.20 \text{ in.} \)

Adjust calculated story drifts by the appropriate \( C_d \) factor from Standard Table 12.2-1.

### 6.2.4 Analysis and Design of Alternative A: SMF

#### 6.2.4.1 Modal Response Spectrum Analysis.

Determine the building period \( (T) \) per *Standard Equation 12.8-7*:

\[
T_a = C_T h_a^n = (0.028)(102.3)^{0.8} = 1.14 \text{ sec}
\]

where \( h_a \), the height to the main roof, is conservatively taken as 102.3 feet. The height of the penthouse will be neglected since its seismic mass is negligible. \( C_u T_a \), the upper limit on the building period, is determined per *Standard Table 12.8-1*:

\[
T = C_u T_a = (1.4)(1.14) = 1.596 \text{ sec}
\]

It is assumed that the calculated period will exceed \( C_u T_a \); this is verified after member selection. The seismic response coefficient \( (C_s) \) is determined from *Standard Equation 12.8-2* as follows:

\[
C_s = \frac{S_{DS}}{R/I} = \frac{1}{8/1} = 0.125
\]

However, *Standard Equation 12.8-3* indicates that the value for \( C_s \) need not exceed:

\[
C_s = \frac{S_{DI}}{T(R/I)} = \frac{0.6}{(1.596 \text{sec})(8/1)} = 0.047
\]

and the minimum value for \( C_s \) per *Standard Equation 12.8-5* is:

\[
C_s = 0.044 IS_{DS} = 0.01 = (0.044)(1)(1) = 0.044
\]

Therefore, use \( C_s = 0.047 \).

Seismic base shear is computed per *Standard Equation 12.8-1* as:

\[
V = C_s W = (0.047)(13,156 \text{ kips}) = 618 \text{ kips}
\]

where \( W \) is the seismic mass of the building as determined above.

In evaluating the building in ETABS, twelve modes are analyzed, resulting in a total modal mass participation of 97 percent. The code requires at least 90 percent participation for strength. A scaling factor is used to take the response spectrum to 85 percent of the base shear, with a minimum scale factor for strength calculations of \( I/R \). Typical software utilizes a spectrum presented as a coefficient of \( g \), thus
requiring scaling by $g$, thus the scaling factor used here is $g/(R/I) = 386/(8/1) = 48.3$. For drift, results are scaled by $C_d(R/I)$; for a spectrum using a coefficient times $g$, this factor is $gC_d(R/I)$.

### 6.2.4.2 Size members

The method used is as follows:

1. Select preliminary member sizes
2. Check deflection and drift *(Standard Sec. 12.12)*
3. Check the column-beam moment ratio rule (AISC 341 Sec. 9.6)
4. Check beam strength
5. Check connection design (AISC 341 Sec. 9.7)
6. Check shear requirement at panel-zone (AISC 341 Sec. 9.3; AISC 358 Sec. 5.4)

After the weight and stiffness have been modified by changing member sizes, the response spectrum must be rescaled for strength. The most significant criteria for the design are drift limits, relative strengths of columns and beams the panel-zone shear. Member strength must be checked but rarely governs for this system.

1. Select Preliminary Member Sizes: The preliminary member sizes are shown for the moment frame in the X-direction in Figure 6.2-3 and in the Y direction in Figure 6.2-4. These sections are selected from AISC SDM Table 1-2, ensuring that they are seismically compact. Members are sized to meet the prequalification limits of AISC 358 Section 5.3 for span-depth ratios, weight flange thickness. Members are also sized for drift limitations and to satisfy strong column–weak beam requirements by using a target ratio of:

$$\sum_{c} \frac{Z_c}{Z_b} \geq 1.25$$

This proportioning does not guarantee compliance with AISC 341 Section 9.6, but is a useful target that makes conformance likely. Using a ratio of 2.0 may save on detailing costs, such as continuity plates, doublers bracing.

The software used accounts explicitly for the increase in beam flexibility due to the RBS cuts. For every beam, RBS parameters were chosen as follows:

$$a = 0.625b_f, b = 0.75d_b, c = 0.20b_f$$

In accordance with AISC 341 Table I-8-1, beam flange slenderness ratios are limited to $0.3\sqrt{E/F_y}$ (7.22 for $F_y = 50$ ksi) beam web height-to-thickness ratios are limited to $2.45\sqrt{E/F_y}$ (59.0 for $F_y = 50$ ksi). Since all members selected are seismically compact per AISC SDM Table 1-2, they conform to these limits.

For columns in special steel moment frames such as this example, AISC 341 Table I-8-1 Footnote b requires that where the ratio of column moment strength to beam moment strength is less than or
equal to 2.0, the more stringent $\lambda_p$ requirements apply for $b/t$ (given above) when $P_u/\phi_b P_y$ is greater than or equal to 0.125, the more stringent $h/t$ requirements apply.

Per AISC 341 Table I-8-1, consider the W14x132 column at Gridline B:

$$h/t_w \leq 1.49 \sqrt{E/F_y} = 22.8 \leq 35.9$$

Therefore, the column is seismically compact.

Strength checks are performed using ETABS; all members are satisfactory for strength.

---

2. Check Drift: Check drift is in accordance with Standard Section 12.12.1. The building is modeled in three dimensions using ETABS. Displacements at the building corners under the 5 percent accidental torsion load cases are used here. Calculated story drifts, response spectrum scaling factors $C_d$ amplification factors are summarized in Table 6.2-1 below. P-delta effects are included.

---

Figure 6.2-3 SMRF frame in E-W direction (penthouse not shown)
All story drifts are within the allowable story drift limit of $0.020h_s$ per Standard Section 12.12 and Section 6.2.3.6 of this chapter.

Figure 6.2-4 SMRF frame in N-S direction (penthouse not shown)
FEMA P-751, *NEHRP Recommended Provisions: Design Examples*

### Table 6.2-1 Alternative A (Moment Frame) Story Drifts under Seismic Loads

<table>
<thead>
<tr>
<th>Level</th>
<th>(\delta_{\text{E-W}}) (in.)</th>
<th>(\delta_{\text{N-S}}) (in.)</th>
<th>(\delta_{\text{E-W}}) (in.)</th>
<th>(\delta_{\text{N-S}}) (in.)</th>
<th>(\Delta_{\text{E-W}}/h) (%)</th>
<th>(\Delta_{\text{N-S}}/h) (%)</th>
<th>(\Delta/h) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level 7</td>
<td>2.92</td>
<td>3.18</td>
<td>16.0</td>
<td>17.5</td>
<td>1.2</td>
<td>1.2</td>
<td>2.0</td>
</tr>
<tr>
<td>Level 6</td>
<td>2.66</td>
<td>2.89</td>
<td>14.7</td>
<td>15.9</td>
<td>1.4</td>
<td>1.7</td>
<td>2.0</td>
</tr>
<tr>
<td>Level 5</td>
<td>2.33</td>
<td>2.47</td>
<td>12.8</td>
<td>13.6</td>
<td>1.6</td>
<td>2.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Level 4</td>
<td>1.91</td>
<td>1.95</td>
<td>10.5</td>
<td>10.7</td>
<td>1.9</td>
<td>2.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Level 3</td>
<td>1.41</td>
<td>1.40</td>
<td>7.76</td>
<td>7.70</td>
<td>1.8</td>
<td>1.8</td>
<td>2.0</td>
</tr>
<tr>
<td>Level 2</td>
<td>0.90</td>
<td>0.88</td>
<td>4.96</td>
<td>4.85</td>
<td>1.2</td>
<td>1.2</td>
<td>2.0</td>
</tr>
<tr>
<td>Level 1</td>
<td>0.55</td>
<td>0.52</td>
<td>3.04</td>
<td>2.89</td>
<td>1.1</td>
<td>1.1</td>
<td>2.0</td>
</tr>
</tbody>
</table>

1.0 in. = 25.4 mm.

3. **Check the Column-Beam Moment Ratio**: Check the column-beam moment ratio per AISC 341 Section 9.6. The expected moment strength of the beams is projected from the plastic hinge location to the column centerline per the requirements of AISC 341 Section 9.6. This is illustrated in Figure 6.2-5. For the columns, the moments at the location of the beam flanges are projected to the column-beam intersection as shown in Figure 6.2-6.

![Figure 6.2-5 Projection of expected moment strength of beam](image)

(1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m)
The column-beam strength ratio calculation is illustrated for the lower level in the E-W direction, Level 2, at Gridline D (W24x146 column and W21x73 beam).

For the beams:

$$M_{pb}^* = M_{pr} + V_e \left( S_h + \frac{d_e}{2} \right) \pm V_g \left( S_h + \frac{d_e}{2} \right)$$

where:

$$M_{pr} = C_{pr}R_y F_y Z_e = (1.15)(1.1)(50)(122) = 7,361 \text{ in.-kips}$$

$$R_y = 1.1 \text{ for Grade 50 steel}$$

$$Z_e = Z_x - 2ctbf(d - tbf) = 172 - 2(1.659)(0.74)(21.24 - 0.74) = 122 \text{ in.}^3$$

$$S_h = \text{Distance from column face to centerline of plastic hinge (see Figure 6.2-9)} = a + b/2 = 13.2 \text{ in.}$$

for the RBS

$$V_e = 2M_{pr} / L' \quad V_g = w_g L' / 2$$

$$L' = \text{Distance between plastic hinges} = 248.8 \text{ in.}$$

$$w_g = \text{Factored uniform gravity load along beam}$$

$$= 1.4D + 0.5L = 1.4[(0.068 \text{ ksf})(12.5 \text{ ft})+(0.025)(13.3 \text{ ft})] + 0.5(0.050 \text{ ksf})(12.5 \text{ ft})$$

$$= 2.42 \text{ klf}$$
The shear at the plastic hinge (Figure 6.2-7) is computed as:

\[ V_p = V_c + V_g \]

where:

\( V_p = \) Shear at plastic hinge location

Figure 6.2-6  Moment in the column
**Figure 6.2-7** Free body diagram bounded by plastic hinges

**Figure 6.2-8** Forces at beam-column connection
Therefore:

\[ V_e = 2M_{psr} / L = 2 \left( \frac{7361 \text{ in} \cdot \text{kips}}{248.8 \text{ in.}} \right) = 59.2 \text{kips} \]

\[ V_g = w_a L / 2 = \left( \frac{2.42 \text{ klf}}{12} \right) \left( \frac{248.8 \text{ in.}}{2} \right) = 25.1 \text{kips} \]
\[ V_p = 59.2 \text{kips} + 25.1 \text{kips} = 84.3 \text{kips} \]

For the beam on the right, with gravity moments adding to seismic:

\[
M_{pb,r}^* = M_{pr} + V_c \left( S_h + \frac{d_c}{2} \right) + V_g \left( S_h + \frac{d_c}{2} \right) = (7,361) + (59.2) \left( \frac{13.2}{2} + \frac{24.74}{2} \right) = 9,517 \text{ in.-kips}
\]

For the beam on the left, with gravity moments subtracting from seismic:

\[
M_{pb,l}^* = M_{pr} + V_c \left( S_h + \frac{d_c}{2} \right) - V_g \left( S_h + \frac{d_c}{2} \right) = (7,361) + (59.2) \left( \frac{13.2}{2} + \frac{24.74}{2} \right) = 8,233 \text{ in.-kips}
\]

\[
\sum M_{pb}^* = M_{pb,r}^* + M_{pb,l}^* = 9,517 + 8,233 = 17,749 \text{ in.-kips}
\]

Note that in most cases, the gravity moments cancel out and can be ignored for this check.

For the columns, the sum of the moments at the top and bottom flanges of the beam is:

\[
\sum M_{BF} = \sum Z_c \left( F_{yc} - \frac{P_{w,c}}{A_g} \right)
\]

\[
\sum M_{BF} = 2 \left[ 418 \text{ in.}^3 \left( \frac{50 \text{ ksi} - \frac{228 \text{ kips}}{43 \text{ in.}^2}}{2} \right) \right] = 37,367 \text{ in.-kips}
\]

where:

\[ M_{BF} = \text{column moment at beam flange elevation} \]

Referring to Figure 6.2-6, the moment at the beam centerline is:

\[
\sum M_{pc}^* = \sum M_{BF} + V_c^* \frac{d_c}{2}
\]

where:

\[ V_c^* = \left[ M_{BF} + M_{BF,c} \right]/h_c, \text{ based on the expected yielding of the spliced column assuming an inflection point at column mid-height (e.g., a portal frame) and not the expected shear when the mechanism forms, which is:} \]
\[ V_c = \frac{1}{2} \left( \sum M_{ph}^* + \frac{1}{2} \sum M_{phv}^* \right) / h, \text{ where } h \text{ is the story height} \]

\[ h_c = \text{clear column height between beams} = (13.33 \text{ ft})(12 \text{ in./ft}) - 21.24 \text{ in.} = 139 \text{ in.} \]

\[ V_c^* = \frac{(18,683 \text{ in.-kips}) + (7,964 \text{ in.-kips})}{(139 \text{ in.})} = 192 \text{ kips} \]

\[ V_c = \left[ \frac{1}{2} \sum M_{ph}^* + \frac{1}{2} \sum M_{phv}^* \right] / h = \left[ \frac{1}{2} (17749) + \frac{1}{2} (14976) \right] / ((13.33)(12)) \]

\[ = 102 \text{ kips} \]

Thus:

\[ \sum M_{pc}^* = 37,367 \text{ in.-kips} + (192 \text{ kips}) \left( \frac{21.24 \text{ in.}}{2} \right) = 39,400 \text{ in.-kips} \]

The ratio of column moment strengths to beam moment strengths is computed as:

\[ \text{Ratio} = \frac{\sum M_{pc}^*}{\sum M_{pb}^*} = \frac{39,400 \text{ in.-kips}}{17,749 \text{ in.-kips}} = 2.22 > 1 \]

OK

Since the ratio is greater than 2, bracing is only required at the top flange per AISC 341 Section 9.7a.

4. Check the Beam Strength: Per AISC 358 Equation 5.8-4, the beam strength at the reduced section is:

\[ \phi M_{pb} = \phi F_y Z_w = (0.9)(50 \text{ kpsi})(122 \text{ in.}^3) = 5,490 \text{ in.-kips} \]

From analysis, \( M_u = 4072 \text{ in.-kips} \). Therefore, \( \phi M_{pb} \geq M_u \); the beam has adequate strength.

The moment at the column face is:

\[ M_f = M_{pc} + V_e S_h \pm V_g S_h \]

\[ M_{f,v} = 7,361 \text{ in.-kips} + (59.2 \text{ kips})(13.2 \text{ in.}) + (25.1 \text{ kips})(13.2 \text{ in.}) = 8,474 \text{ in.-kips} \]

\[ M_{f,x} = 8,474 \text{ in.-kips} \leq \phi_d R_y F_y Z_w = (1.0)(1.1)(50)(172) = 9,460 \text{ in.-kips} \]

OK

To check the shear in the beam, first the appropriate equation must be selected:

\[ 2.24 \sqrt{\frac{E}{F_{yw}}} = 2.24 \sqrt{\frac{(29,000 \text{ ksi})}{(50 \text{ ksi})}} = 53.9 \]
\[ \frac{h}{t_w} = 46.6 \leq 53.9 \]

Therefore:

\[ V_n = 0.6F_yA_wC_v \]

where \( C_v = 1.0 \).

\[ V_n = 0.6(50 \text{ksi})(21.2 \text{in.})(0.455 \text{in.})(1.0) = 289 \text{kips} \]

Comparing this to \( V_p \):

\[ \phi V_n = 289 \geq 84.3 = V_p \quad \text{OK} \]

Check the beam lateral bracing. Per AISC 341 Section 9.8, the maximum spacing of the lateral bracing is:

\[ L_b \leq 0.086 \frac{r_y E}{F_y} = 0.086(1.81 \text{in.})(29,000 \text{ksi})/(50 \text{ksi}) = 90 \text{ in.} = 7' - 6" \]

The braces near the plastic hinges are required to have a minimum strength of:

\[ P_{br} = \frac{0.06M_u}{h_o} \]

\[ = \frac{0.06(1.1)(50 \text{ksi})(172 \text{in.}^3)}{21.24 \text{ in.}^2 - 0.74 \text{ in.}^2} = 27.7 \text{kips} \]

where:

\[ M_u = R_yF_yZ \]

\[ h_o = \text{the distance between flange centroids} \]

The required brace stiffness is:

\[ \beta_{br} = \frac{10M_uC_d}{\phi L_b h_o} \]

\[ = \frac{10(1.1)(50 \text{ksi})(172 \text{in.}^3)(1.0)}{0.75(6.39 \text{ft})(12 \text{ in./ft})(21.24 \text{ in.}^2 - 0.74 \text{ in.}^2)} = 80.2 \text{kips/in.} \]

\( L_b \) is taken as \( L_p \). These values are for the typical lateral braces. No supplemental braces are required at the reduced section per AISC 358 Section 5.3.1.

5. Check Connection Design:
Check the need for continuity plates. Continuity plates are required per AISC 358 Section 2.4.4 unless:

\[
t_{cf} \geq 0.4 \sqrt[1.8]{b_{bf} t_{bf}} \left( \frac{F_{yc} R_{vb}}{F_{yc} R_{yc}} \right)
\]

\[
\geq 0.4 \sqrt{1.8(8.30 \text{ in.})(0.74 \text{ in.}) \left( \frac{50 \text{ ksi}}{50 \text{ ksi}} \right)^{1.1} = 1.33 \text{ in.}}
\]

And:

\[
t_{cf} \geq \frac{b_{bf}}{6} = \frac{8.30 \text{ in.}}{6} = 1.38 \text{ in.}
\]

Since \( t_{cf} = 1.09 \text{ inches} \), continuity plates are required. See below for the design of the plates.

Checking web crippling per AISC 360 Section J10.3:

\[
R_u = \frac{M_{f,c}}{d_h - t_f} = \frac{8,474 \text{ in.-kips}}{(21.24 \text{ in.}) - (0.74 \text{ in.})} = 413 \text{ kips}
\]

\[
R_n = 0.40r_w^2 \left[ 1 + 3 \left( \frac{N}{d} \right) \left( \frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{EF_{yw} t_f \over t_w}
\]

\[
R_n = 0.80(0.65)^2 \left[ 1 + 3 \left( \frac{(0.74 + 5/16)}{24.74} \right) \left( \frac{(0.65)}{(1.09)} \right)^{1.5} \right] \sqrt{29,000(50)(1.09) \over (0.65)} = 558 \text{ kips}
\]

\[
\phi R_n = 0.75(558 \text{ kips}) = 419 \text{ kips} \geq 413 \text{ kips} = R_u
\]

Checking web local yielding per Specification Section J10.2:

\[
R_u = 413 \text{ kips}
\]

\[
\phi R_n = \phi(5k + N) F_{yw} t_w
\]

\[
\phi R_n = (1.00) \left( 5 \left( 5.19 \text{ in.} + (0.74 \text{ in.} + \frac{5}{16} \text{ in.}) \right) (50 \text{ ksi}) (0.65 \text{ in.}) \right) = 293 \text{ kips}
\]

Therefore, since \( \phi R_n \leq R_u \), as well as due to the check above, continuity plates are required. The force that the continuity plates must take is \( 413 - 293 = 120 \text{ kips} \). Therefore, each plate takes 60 kips. The minimum thickness of the plates is the thickness of the beam flanges, 0.74 inch. The minimum width of the plates per AISC 341 Section 7.4 is:
\[ b_{pl} = f_{pl} - 2(k_{1,h} + \frac{\gamma}{2}) \text{ in.} \]

\[ = 8.31 \text{ in.} - 2(0.875 \text{ in.} + 0.5 \text{ in.}) = 5.56 \text{ in.} \]

Checking the strength of the plate with minimum dimensions:

\[ \phi R_n = \phi t_{pl} b_{pl} F_y \]

\[ = (1.0)(0.74 \text{ in.})(5.56 \text{ in.})(50 \text{ ksi}) = 206 \text{ kips} \]

Therefore, since \( \phi R_n = 206 \text{ kips} > 60 \text{ kips} \), the minimum continuity plates have adequate strength. Alternatively, a W24x192 section will work in lieu of adding continuity plates.

6. Check Panel Zone: The Standard defers to AISC 341 for the panel zone shear calculation.

The panel zone shear calculation for Story 2 of the frame in the E-W direction at Grid C (column: W24x176; beam: W21x73) is from AISC 360 Section J10.6. Check the shear requirement at the panel zone in accordance with AISC 341 Section 9.3. The factored shear \( R_u \) is determined from the flexural strength of the beams connected to the column. This depends on the style of connection. In its simplest form, the shear in the panel zone \( (R_u) \) is as follows for W21x73 beams framing into each side of a W24x146 column (such as Level 2 at Grid C):

\[ R_u = \sum \frac{M_f}{d_h - t_{fb}} = \frac{16,285}{21.24 - 0.74} = 794 \text{ kips} \]

\( M_f \) is the moment at the column face determined by projecting the expected moment at the plastic hinge points to the column faces (see Figure 6.2-5):

\[ M_f = M_{pr} + V_e S_h \pm V_g S_h \]

\[ M_{f,r} = 7,361 \text{ in.-kips} + (59.2 \text{ kips})(13.2 \text{ in.}) + (25.1 \text{ kips})(13.2 \text{ in.}) = 8,474 \text{ in.-kips} \]

\[ M_{f,d} = 7361 \text{ in.-kips} + (59.2 \text{ kips})(13.2 \text{ in.}) - (25.1 \text{kips})(13.2 \text{ in.}) = 7811 \text{ in.-kips} \]

Note that in most cases, the gravity moments cancel out and can be ignored for this check. The total moment at the column face is:

\[ \sum M_f = M_{f,r} + M_{f,d} = 8,474 \text{ in.-kips} + 7,811 \text{ in.-kips} = 16,285 \text{ in.-kips} \]

The shear transmitted to the joint from the story above, \( V_c \), opposes the direction of \( R_u \) and may be used to reduce the demand. Previously calculated, this is 102 kips at this location. Thus the frame \( R_u = 794 - 102 = 692 \text{ kips} \).

The column axial force (Load Combination: \( 1.2D + 0.5L + \Omega_oE \)) is \( P_f = 228 \text{ kips} \).
0.75 \( P_c = 0.75 F_y A_g = 0.75 (50 \text{ ksi})(43 \text{ in.}^2) = 1,613 \text{ kips} \)

Since \( P_c \leq 0.75 P_r \), using AISC 360 Equation J10-11:

\[
R_n = 0.60 F_y d_c t_w \left( 1 + \frac{3b_{cf}f_{cf}^2}{d_c d_c t_w} \right)
\]

\[
R_n = 0.60 (50)(24.74)(0.65) \left( 1 + \frac{3(12.90)(1.09)^2}{(21.24)(24.74)(0.65)} \right) = 547 \text{ kips}
\]

Since \( \phi_r = 1 \), \( \phi_r R_n = 547 \text{ kips} \).

Therefore, doubler plates are required. The required additional strength from the doubler plates is \( 692 - 547 = 145 \text{ kips} \). The strength of the doubler plates is:

\[
\phi_r R_n = 0.6 t_{doub} d_c F_y
\]

Therefore, to satisfy the demand the doubler plate must be at least 1/4 inch thick. Plug welds are required as:

\[
t = 0.25 \text{ in.} < (d_z + w_z) / 90 = [21.24 + 24.74 - (1.09)] / 90 = 0.49 \text{ in.}
\]

Use four plug welds spaced 12 inches apart. Alternatively, the use of a W24x192 column will not require doubler plates (\( \phi_r R_n = 737 \text{ kips} \)).

6.2.5 Analysis and Design of Alternative B: SCBF

6.2.5.1 Modal Response Spectrum Analysis. As with the SMF, find the approximate building period (\( T_a \)) using Standard Equation 12.8-7:

\[
T_a = C_h h_n = (0.02)(102.3)^{0.75} = 0.64 \text{ sec}
\]

\( C_u T_u \), the upper limit on the building period, is determined per Standard Table 12.8-1:

\[
T = C_u T_u = (1.4)(0.64) = 0.896 \text{ sec}
\]

It is assumed that the calculated period will exceed \( C_u T_u \); this is verified after member selection. The seismic response coefficient (\( C_s \)) is determined from Standard Equation 12.8-2 as follows:

\[
C_s = \frac{S_{DS}}{R / I} = \frac{1}{6 / 1} = 0.167
\]
However, Standard Equation 12.8-3 indicates that the value for \( C_s \) need not exceed:

\[
C_s = \frac{S_{DI}}{T(R/I)} = \frac{0.6}{(0.896\text{sec})(6/1)} = 0.112
\]

and the minimum value for \( C_s \) per Standard Equation 12.8-5 is:

\[
C_s = 0.044/S_{DS} \geq 0.01 = (0.044)(1)(1) = 0.044
\]

Use \( C_s = 0.112 \).

Seismic base shear is computed using Standard Equation 12.8-1 as:

\[
V = C_s W = (0.112)(13,156\text{kips}) = 1,473\text{kips}
\]

where \( W \) is the seismic mass of the building as determined above.

In evaluating the building in ETABS, twelve modes are analyzed, resulting in a total modal mass participation of 99 percent. The Standard Sec. 12.9.1 requires at least 90 percent participation. As before with Alternative A, strength is scaled to 85 percent of the equivalent lateral force base shear and drift is scaled by \( gC_d/(R/I) \).

**6.2.5.2 Size members.** The method used to size members is as follows:

1. Select brace sizes based on strength
2. Select column sizes based on special seismic load combinations (Standard Sec. 12.4.3.2)
3. Select beam sizes based on the load imparted by the expected strength of the braces
4. Check drift (Standard Sec. 12.12)
5. Design the connection

Reproportion member sizes as necessary after each check. After the weight and stiffness have been modified by changing member sizes, the response spectrum must be rescaled. Torsional amplification is a significant consideration in this alternate.

1. Select Preliminary Member Sizes and Check Strength: The preliminary member sizes are shown for the braced frame in the E-W direction (seven bays) in Figure 6.2-10 and in the N-S direction (five bays) in Figures 6.2-11 and 6.2-12.
Figure 6.2-10 Braced frame in E-W direction
Figure 6.2-11 Braced frame in N-S direction on Gridlines C and F
Figure 6.2-12 Braced frame in N-S direction on Gridlines D and E

Check slenderness and width-to-thickness ratios—the geometrical requirements for local stability. In accordance with AISC 341 Section C13.2a, bracing members must satisfy the following:

\[
\frac{kl}{r} \leq 200
\]
All members are seismically compact for SCBF per AISC SDM Table 1-2, thus satisfying slenderness requirements.

Columns: Wide flange members must comply with the width-to-thickness ratios contained in AISC 341 Table I-8-1. Flanges must satisfy the following:

\[ \frac{b}{t} \leq 0.30 \sqrt{\frac{E}{F_y}} = 7.23 \]

Webs in combined flexural and axial compression (where \( \frac{P_u}{\phi_b P_y} = 0.385 > 0.125 \)) must satisfy the following:

\[ \frac{h_c}{t_w} \leq 1.12 \sqrt{\frac{E}{F_y}} \left( 2.33 - \frac{P_u}{\phi_b P_y} \right) = 52.5 \]

Braces: Rectangular HSS members must satisfy the following:

\[ \frac{b}{t} \leq 0.64 \sqrt{\frac{E}{F_y}} = 16.1 \]

Using a redundancy factor of 1.3 on the earthquake loads, the braces are checked for strength using ETABS and found to be satisfactory.

2. Select Column Sizes: Columns are checked using special seismic load combinations; \( \rho \) does not apply in these combinations (see Standard Sec. 12.3.4.1 Item 6). The columns are then checked for strength using ETABS and found to be satisfactory.

3. Select Beam Sizes: The beams are sized to be able to resist the expected plastic and post-buckling capacity of the braces. In the computer model, the braces are removed and replaced with forces representing their capacities. These loads are applied for four cases reflecting earthquake loads applied both left and right in the two orthogonal directions (\( T_{1x}, T_{2x}, T_{1y}, T_{2y} \)). For instance, in \( T_{1x} \), the earthquake load is imagined to act left to right; the diagonal braces expected to be in tension under this loading are replaced with the force \( R_y F_y A_g \) and the braces expected to be in compression are replaced with the force \( 0.3 P_n \). For \( T_{2x} \), the tension braces are now in compression and vice versa. \( T_{1y} \) and \( T_{2y} \) apply in the other orthogonal direction.

The load cases applied are as follows:

\[ (1.2 + 0.2 S_{DS}) D + 0.5L + T \text{ (four combinations; use all four } T's) \]

\[ (0.9 - 0.2 S_{DS}) D + T \text{ (four combinations; use all four } T's) \]

Beam strength is checked for each of these eight load combinations using ETABS and found to be satisfactory.

4. Check Story Drift: After designing the members for strength, the ETABS model is used to determine the design story drift. The results are summarized in Table 6.2-2.
### Table 6.2-2 Alternative B Story Drifts under Seismic Load

<table>
<thead>
<tr>
<th>Level</th>
<th>Δ_e-W (in.)</th>
<th>Δ_e-N-S (in.)</th>
<th>Δ_E-W (in.)</th>
<th>Δ_N-S (in.)</th>
<th>Δ-E-W/h (%)</th>
<th>Δ-N-S/h (%)</th>
<th>Δ/h (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level 7</td>
<td>1.63</td>
<td>1.75</td>
<td>8.14</td>
<td>8.76</td>
<td>0.72</td>
<td>0.93</td>
<td>2.0</td>
</tr>
<tr>
<td>Level 6</td>
<td>1.41</td>
<td>1.48</td>
<td>7.07</td>
<td>7.38</td>
<td>0.74</td>
<td>0.94</td>
<td>2.0</td>
</tr>
<tr>
<td>Level 5</td>
<td>1.19</td>
<td>1.20</td>
<td>5.97</td>
<td>5.99</td>
<td>0.76</td>
<td>0.84</td>
<td>2.0</td>
</tr>
<tr>
<td>Level 4</td>
<td>0.96</td>
<td>0.94</td>
<td>4.80</td>
<td>4.72</td>
<td>0.81</td>
<td>0.85</td>
<td>2.0</td>
</tr>
<tr>
<td>Level 3</td>
<td>0.71</td>
<td>0.69</td>
<td>3.56</td>
<td>3.43</td>
<td>0.72</td>
<td>0.71</td>
<td>2.0</td>
</tr>
<tr>
<td>Level 2</td>
<td>0.49</td>
<td>0.47</td>
<td>2.44</td>
<td>2.33</td>
<td>0.60</td>
<td>0.59</td>
<td>2.0</td>
</tr>
<tr>
<td>Level 1</td>
<td>0.30</td>
<td>0.28</td>
<td>1.49</td>
<td>1.40</td>
<td>0.56</td>
<td>0.52</td>
<td>2.0</td>
</tr>
</tbody>
</table>

1.0 in. = 25.4 mm.

All story drifts are within the allowable story drift limit of 0.020\(h_v\) in accordance with Standard Section 12.12 and the allowable deflections for this building from Section 6.2.3.6 above. As shown in the table above, the drift is far from being the governing design consideration.

5. Design the Connection: Figure 6.2-13 illustrates a typical connection design at a column per AISC 341 Section 13.
Figure 6.2-13 Bracing connection detail (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m)

The connection designed in this example is at the fourth floor on Gridline C. The required strength of the connection is to be the nominal axial tensile strength of the bracing member. For an HSS6x6x5/8, the expected axial tensile strength is computed using AISC 341 Section 13.3a:

\[ R_u = R_y F_y A_g = (1.4)(46 \text{ ksi})(11.7 \text{ in.}^2) = 753 \text{ kips} \]

The area of the gusset is determined using the plate thickness and section width (based on geometry). See Figure 6.2-13 for the determination of this dimension. The thickness of the gusset is chosen to be 1 inch.

For tension yielding of the gusset plate:

\[ \phi T_u = \phi F_y A_g = (0.90)(50 \text{ ksi})(1 \text{ in.} \times 17 \text{ in.}) = 765 \text{ kips} > 753 \text{ kips} \]

OK
For fracture in the net section:

\[
\phi T_n = \phi F_u A_n = (0.75)(65 \text{ ksi })(1 \text{ in. } \times 17 \text{ in.}) = 829 \text{ kips} > 753 \text{ kips} \quad \text{OK}
\]

For a tube slotted to fit over a connection plate, there will be four welds. The demand in each weld will be 753 kips/4 = 188 kips. The design strength for a fillet weld per AISC 360 Table J2.5 is:

\[
\phi F_w = \phi(0.6F_{exx}) = (0.75)(0.6)(70 \text{ ksi}) = 31.5 \text{ ksi}
\]

For a 1/2-inch fillet weld, the required length of weld is determined to be:

\[
L_w = \frac{188}{(0.707)(0.5 \text{ in.})(31.5 \text{ ksi})} = 16.9 \text{ in.}
\]

Therefore, use 17 inches of weld.

In accordance with the exception of AISC 341 Section 13.3b, the design of brace connections need not consider flexure if the gusset can accommodate the inelastic rotation associated with brace post-buckling deformations. This is typically done by providing a “hinge zone”; the gusset plate is detailed such that it can form a plastic hinge over a distance of 2t (where t = thickness of the gusset plate) from the end of the brace. The gusset plate must be permitted to flex about this hinge, unrestrained by any other structural member. See also AISC 341 Section C13.3b. With such a pinned-end condition, the compression brace tends to buckle out-of-plane. During an earthquake, there will be alternating cycles of compression and tension in a single bracing member and its connections. Proper detailing is imperative so that tears or fractures in the steel do not initiate during the cyclic loading.

While the gusset is permitted to hinge, it must not buckle. To prevent buckling, the gusset compression strength must exceed the expected brace strength in compression per AISC 341 Section 13.3c. Determine the nominal compressive strength of the brace member. The effective brace length (kL) is the distance between the hinge zones on the gusset plates at each end of the brace member. This length is somewhat dependent on the gusset design. For the brace being considered, kL = 161 inches the expected compressive strength is determined using expected (not specified minimum) material properties per AISC 360 Section E3:

\[
P_n = F_{cr} A_g
\]

where:

\[A_g = \text{ gross area of the brace}\]

\[F_{cr} = \text{ flexural buckling stress, determined as follows}\]

When:

\[\frac{kL}{r} \leq 5.18 \sqrt{E / R_y F_y} = 119\]
\begin{align*}
F_{cr} & \leq \left[ 0.692 \left( \frac{R_{y}F_{y}}{F_{e}} \right) \right] R_{y}F_{y} \\
\text{Otherwise,} & \\
F_{cr} & \leq F_{e}
\end{align*}

where:

\[
F_{e} = \frac{\pi^{2}E}{\left( \frac{kL}{r} \right)^{2}}
\]

The equations have been recalibrated to use the expected stress rather than the specified minimum yield stress. Note that the 0.877 factor, which represents out-of-straightness, is not used here in order to calculate an upper bound brace strength and thereby ensure adequate gusset compression strength. Here, \( kL/r = (1)(161)/(2.17) = 74.2 \), thus:

\[
F_{e} = \frac{\pi^{2}(29,000 \text{ ksi})}{(74.2)^{2}} = 52.0 \text{ ksi}
\]

\[
F_{cr} = \left[ 0.692 \left( \frac{52 \text{ ksi}}{46 \text{ ksi}} \right) \right] (1.4)(46 \text{ ksi}) = 40.8 \text{ ksi}
\]

\[
P_{n} = (40.8 \text{ ksi}) (11.7 \text{ in.}^{2}) = 478 \text{ kips}
\]

Now, using the expected compressive load from the brace of 449 kips, check the buckling capacity of the gusset plate using the section above. By this method, illustrated by Figure 6.2-13, the compressive force per unit length of gusset plate is \( (478 \text{ kips}/23.5 \text{ in.}) = 20.3 \text{ kips/in.} \).

Try a plate thickness of 1 inch:

\[
f_{a} = \frac{P}{A} = 20.3 \text{ kips}/(1 \text{ in.} \times 1 \text{ in.}) = 20.3 \text{ ksi}
\]

The gusset plate is modeled as a 1-inch-wide by 1-inch-deep rectangular section, fixed at both ends. The length, from geometry, is 17.2 inches. The effective length factor, \( k \), for this “column” is 1.2 per AISC 360 Table C-C2.2. The radius of gyration, \( r \), for a plate is \( t/\sqrt{12} \).

Per AISC 360 Section E3:

\[
\frac{kL}{r} = \frac{(1.2)(17.2 \text{ in.})}{(0.29 \text{ in.})} = 71.2
\]
Next, check the reduced section of the tube, which has a 1-1/8-inch-wide slot for the gusset plate (the thickness of the gusset plus an extra 1/8 inch for ease of construction). The reduction in HSS6x6x5/8 section due to the slot is (0.581 in. × 1.125 in. × 2) = 1.31 in.² the net section, $A_{net} = (11.7 - 1.31) = 10.4$ in.²

To ensure gross section yielding governs, reinforcement is added over the area of the slot. The shear lag factor is computed per AISC 360 Table D3.1:

$$U = 1 - \frac{x}{l}$$

where:

$$x = \frac{B^2 + 2BH}{4(B + H)} = \frac{(6 \text{ in.})^2 + 2(6 \text{ in.})(6 \text{ in.})}{4(6 \text{ in.} + 6 \text{ in.})} = 2.25 \text{ in.}$$

and $l$ is the length of the weld as determined above.

$$U = 1 - \frac{(2.25 \text{ in.})}{(17 \text{ in.})} = 0.867$$

Thus, the effective area of the section is:

$$A_e = UA_{net} = (0.867)(10.4 \text{ in.}^2) = 9.02 \text{ in.}^2$$

Try a reinforcing plate 1/2 inch thick and 3-1/2 inches wide on each side of the brace. (The necessary width can be computed from the effective area, but that calculation is not performed here.) Grade 50 material is used in order to match or exceed the brace material strength, thus allowing for treatment of the material as homogenous. The area of the section is $(2 \times 0.5 \text{ in.} \times 3.5 \text{ in.}) = 3.5$ in.². The distance of its center of gravity from the center of gravity of the slotted brace is:

$$x = \frac{B}{2} + \frac{t_{reinf}}{2} = \frac{(6 \text{ in.})}{2} + \frac{(0.5 \text{ in.})}{2} = 3.25 \text{ in.}$$

Thus, the area of the reinforced section is:

$$A = A_n + A_{reinf} = 10.4 \text{ in.}^2 + 3.5 \text{ in.}^2 = 13.9 \text{ in.}^2$$
The weighted average of the \( x \)'s is 2.53 inches. Thus, the shear lag factor for the reinforced section is:

\[
U = 1 - \frac{(2.53 \text{ in.})}{(16.9 \text{ in.})} = 0.850
\]

Thus, the effective area of the section is:

\[
A_e = UA_{net} = (0.850)\left(13.9 \text{ in.}^2\right) = 11.8 \text{ in.}^2
\]

Now, check the effective area of the reinforced section against the original section of the brace per AISC 341 Section 13.2b:

\[
\frac{A_e}{A_c} = \frac{(11.7 \text{ in.}^2)}{(11.8 \text{ in.}^2)} = 0.99 \leq 1 \quad \text{OK}
\]

The reinforcement is attached to the brace such that its expected yield strength is developed.

\[
R_u = A_{reinf}F_y = 3.5 \text{ in.}^2 \left(1.1\right)(50 \text{ ksi}) = 193 \text{ kips}
\]

The plate will be developed with two 5/16-inch fillet welds, 14 inches long:

\[
R_u = 2\phi 0.6F_{eex} \sqrt{\frac{L}{2}} sL = 2(0.75)(0.6)(70 \text{ ksi})\sqrt{\frac{2}{2}} \left(\frac{5}{16} \text{ in.}\right)(14 \text{ in.}) = 195 \text{ kips}
\]

The force must be developed into the plate, carried past the reduced section developed out of the plate. To accomplish this, the reinforcement plate will be 33 inches: 14 inches on each side of the reduced section, 2 inches of anticipated over slot, plus 1 inch to provide erection tolerance.

The complete connection design includes the following checks (which are not demonstrated here):

- Attachment of reinforcement to brace
- Brace shear rupture
- Brace shear yield
- Gusset block shear
- Gusset yield, tension rupture, shear rupture weld at both the column and the beam
- Web crippling and yielding for both the column and the beam
- Gusset edge buckling
- Beam-to-column connection
6.2.6 Cost Comparison

For each case, the total structural steel was estimated. The takeoffs are based on all members, but do not include an allowance for plates and bolts at connections. The result of the material takeoffs are as follows:

- Alternative A, Special Steel Moment Resisting Frame: 640 tons
- Alternative B, Special Steel Concentrally Braced Frame: 646 tons

The higher weight of the systems with bracing is primarily due to the placement of the bracing in the core, where resistance to torsion is poor. Torsional amplification and drift limitations both increased the weight of steel in the bracing. The weight of the moment-resisting frame is controlled by drift and the strong column rule.

6.3 TEN-STOREY HOSPITAL, SEATTLE, WASHINGTON

This example features a buckling-restrained braced frame (BRBF) building. The example covers:

- Analysis issues specific to buckling-restrained braced frames
- Proportioning of buckling-restrained braces
- Capacity design principles
- Nonlinear response history analysis
- Buckling-restrained brace connections

6.3.1 Building Description

This ten-story hospital includes a two-story podium structure beneath an eight-story tower, as shown in Figures 6.3-1 and 6.3-2. The podium is 211.3 feet by 121.3 feet in plan, while the tower’s floorplate is square with 91.3-foot sides. Story heights are 18 feet in the podium and reduce to 15 feet throughout the tower, bringing the total building height to 156 feet. As the tower is centered horizontally on the podium below, the entire building is symmetric about a single axis. Both the podium and the tower have large roof superimposed dead loads due to heavy HVAC equipment located there.
The structure exemplifies a common situation for hospital facilities. The combination of a stiff podium structure beneath a more flexible tower results in significant force transfer at the floor level between them.
The vertical-load-carrying system consists of lightweight concrete fill on steel deck floors supported by steel beams and girders that span to steel columns. The bay spacing is 30 feet each way. There are three floor beams per bay. All beams and girders are composite.

BRBFs have been selected for this building because they provide high stiffness paired with a high degree of ductility and stable hysteretic properties. The building has a thick mat foundation. The foundation soil is representative of Site Class C conditions identical to those discussed in Section 3.2. The design of foundations is not included here.

6.3.1.1 Design method. Seismic forces, rather than wind forces, govern the building’s lateral design (in part due to the mass of the thick concrete-filled decks). The lateral force-resisting system throughout the tower consists of BRBFs in the middle bay along each side of the perimeter—Gridlines 3, 6, A, D, as can be seen in the representative elevation of Figure 6.3-3. These BRBFs deliver lateral loads to the collectors and diaphragm at the third floor where both in-plane and out-of-plane discontinuities exist. This transfer occurs in-plane along Gridlines A and D to two braced bays nearer the ends of the podium and out-of-plane from a single braced bay in the tower along Gridlines 3 and 6 to braces in the two adjacent bays along Gridlines 2 and 7 in the podium below. The podium bracing configuration is illustrated in Figure 6.3-2. A typical bracing elevation in the transverse direction of the podium (illustrating the out-of-plane offset) is shown in Figure 6.3-4.

Figure 6.3-3 Longitudinal elevation at Gridline D
Chapter 6: Structural Steel Design

An ELF analysis is first performed to scale the base shear for the subsequent MRSA used for strength design of the buckling-restrained braces (BRB). Each BRB is designed for its share of 100 percent of the horizontal component of the earthquake lateral load without considering additional tributary vertical loads. This is done to encourage distributed yielding of braces up the height of the structure and is justified because the braces will shed any gravity load upon first yield and transfer it to the connecting beams and columns, which are designed to accommodate gravity loads without support provided by the braces. Beams, columns collectors are preliminarily sized using capacity design principles considering plastic mechanisms that develop based on the brace sizes determined using elastic MRSA. Finally, a
nonlinear response history analysis (NRHA) is executed to verify BRB strains remain at acceptable levels, check that story drifts do not exceed allowable limits possibly reduce column sizes from what the plastic mechanism analyses require. The details of this design procedure are summarized in Table 6.3-1.

<table>
<thead>
<tr>
<th><strong>Table 6.3-1</strong> Design Philosophy</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Element</strong></td>
</tr>
<tr>
<td>BRBs</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Columns</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Diaphragms</td>
</tr>
<tr>
<td>Collector beams, braced-bay beams, etc.</td>
</tr>
</tbody>
</table>

6.3.2 Basic Requirements

6.3.2.1 Provisions parameters. Section 3.2 illustrates the determination of design ground motion parameters for this example. They are as follows:

- $S_{DS} = 0.859$
- $S_{D1} = 0.433$
- Occupancy Category IV
- Seismic Design Category D

For BRBFs (Standard Table 12.2-1):

- $R = 8$
- $\Omega_o = 2.5$
- $C_d = 5$

These values are representative of a seismic force-resisting system that is a unification of two previously different BRBF system classifications: those with non-moment-resisting beam-column connections and those with moment-resisting beam-column connections. Modifications to Section 15.7 of AISC 341 now require the beam-to-column connections for a building frame system either to meet the requirements for fully restrained (FR) moment connections as specified in AISC 341 Section 11.2a or to possess sufficient rotation capacity to accommodate the rotation required to achieve a story drift of 2.5 percent. The later compliance path is selected for this design example. This requirement effectively amounts to a braced frame with simple beam-to-column connections per AISC 360 Section B3.6a with rotation specified at 2.5 percent. A standard detail illustrating this connection is presented in Figure 6.3-5.
6.3.2.2 Loads.

- Roof live load, $L_r$: 25 psf
- Roof dead load, $D$: 135 psf
- Exterior wall cladding: 300 plf of spandrel beams
- Floor live load, $L$: 60 psf
- Partitions: 10 psf
- Floor dead load, $D$: 104 psf
- Floor live load reductions: per the IBC

Roof dead load includes roofing, insulation, lightweight concrete-filled metal deck, concrete ponding allowance, framing, mechanical and electrical equipment, ceiling fireproofing. Floor dead load includes lightweight concrete-filled metal deck, ponding allowance, framing, mechanical and electrical equipment, ceiling and fireproofing. Due to potential for rearrangement, partition loads are considered live loads per Standard Section 4.2.2 but are also included in the effective seismic weight in accordance with Standard Section 12.7.2. Therefore, the seismic weight of a typical tower floor, whose footprint is a square 8,342 ft$^2$ in area, is $104 + 10 + 300(30)(3)(4)/8,342 = 127$ psf.
6.3.2.3 Materials

- Concrete for drilled piers: $f'_{c} = 5$ ksi, normal weight (NW)
- Concrete for floors: $f'_{c} = 3$ ksi, lightweight (LW)
- All other concrete: $f'_{c} = 4$ ksi, NW
- Structural steel:
  - Wide flange sections: ASTM A992, Grade 50
  - Plates: ASTM A36

6.3.3 Structural Design Criteria

6.3.3.1 Building configuration. The hospital building does not possess any stiffness, strength, or weight irregularities despite the relatively tall height of the podium stories. At the podium levels, the two braced bays corresponding to each line of single-bay chevron bracing in the tower above provide more than enough additional strength to compensate for the slight increase in floor-to-floor height. The story drift ratio increases up the full height of the structure, meeting the exception of Standard Section 12.3.2.2 for assessing vertical stiffness and weight irregularities. However, the structure does possess both a vertical geometric irregularity (Type 3) and an in-plane discontinuity in vertical lateral force-resisting element irregularity (Type 4) since the lateral force-resisting system transitions from a single chevron braced bay in the tower to two chevron braced bays at the podium levels. The two chevron braced bays in the podium occur two bays away from the tower braced bay in the longitudinal direction. The Type 4 vertical irregularity triggers an increase in certain design forces per Standard Sections 12.3.3.3 and 12.3.3.4. Together, the Type 3 and Type 4 vertical irregularities preclude the use of an equivalent lateral force analysis as defined in Standard Section 12.8 based on the permissions in Standard Table 12.6-1. Note that this analysis prohibition is also triggered by the flexibility of the structure, as its fundamental period (see Sec. 6.3.4.1) exceeds $3.5T_S = 3.5(S_{DI}/S_{DS})$ seconds = $3.5 \times (0.433/0.859)$ seconds = 1.76 seconds. Nevertheless, the design base shear still must be determined using the equivalent lateral force analysis procedures to ensure that the design base shear for a modal response spectrum analysis meets the requirements of Standard Section 12.9.4.

Due to the building’s symmetry and the strong torsional resistance provided by the layout of the vertical lateral force-resisting elements, numerous plan irregularities are not expected. Analysis reveals that the structure is torsionally regular the only horizontal structural irregularity present is an out-of-plane offset irregularity (Type 4) triggered by the shift in the vertical lateral force-resisting system from Gridlines 3 and 6 in the tower to Gridlines 2 and 7 in the podium structure below. The only additional provisions triggered by the Type 4 horizontal structural irregularity relate to three-dimensional modeling requirements.

6.3.3.2 Redundancy. The limited number of braced bays in each direction of the tower require the redundancy factor ($\rho$) to be taken as 1.3 per Standard Section 12.3.4.2 Item a and Table 12.3-3. Because there are only two BRBF chevrons in each direction throughout the tower, removal of a single brace would dramatically increase flexural demands in the beam at that location and would certainly result in at least a 33 percent reduction in story strength even if the resulting system does not have an extreme torsional irregularity. The 1.3 redundancy factor ($\rho$) is incorporated as a load factor on the seismic loads used in the design of the braces.
6.3.3.3 Orthogonal load effects. *Standard* Section 12.5.4 stipulates a combination of 100 percent of the seismic forces in one direction plus 30 percent of the seismic forces in the orthogonal direction, at a minimum, for structures in Seismic Design Category D. However, it has been shown (Wilson, 2004) that use of the 100/30 percentage combination rule can result in member designs that are not equally resistant to earthquake ground motions originating from different directions. Instead, a SRSS combination of seismic forces from two full-magnitude response spectra analyses conducted along each principal axis of the building is performed to ensure the design forces remain independent of the selected reference coordinate system (in this case, the building’s main orthogonal axes).

In the context of NRHA, orthogonal pairs of ground motion acceleration histories are applied simultaneously in accordance with the requirements of *Standard* Section 12.5.4 for structures in Seismic Design Category D.

6.3.3.4 Structural component load effects. The effect of seismic load as defined by *Standard* Section 12.4.2 is as follows:

\[
E = \rho Q_E \pm 0.2SDS \Delta
\]

In this example, \( SD_S = 0.859 \). The seismic load is combined with the gravity loads in elastic analyses as shown in *Standard* Section 12.4.3.2, resulting in the following load combinations:

\[
1.37D + 0.5L + 0.2S + \rho Q_E
\]

\[
0.73D + 1.6H + \rho Q_E
\]

The 0.5 coefficient on \( L \) is permitted for all occupancies in which \( L_o \) in *Standard* Table 4.1 is less than or equal to 100 psf per Exception 1 to *Standard* Section 2.3.2. The braces are designed without considering additional tributary vertical loads to encourage distributed yielding up the height of the structure. However, the surrounding beams and columns that are part of the lateral force-resisting system are designed for the above gravity loads in conjunction with the earthquake effect as specified in AISC 341 Section 16.5b. Again, the redundancy factor, \( \rho \), is taken as 1.3 for design of the braces themselves.

In a NRHA, the structure is analyzed for the effects of the scaled pairs of ground motions simultaneously with the effects of dead load and 25 percent of the required live loads per *Standard* Section 16.2.3.

6.3.3.5 Drift limits. For a building assigned to Occupancy Category IV, the allowable story drift (*Standard* Sec. 12.12.1 and Table 12.12-1) is \( \Delta_a = 0.010h_s \).

The allowable story drift for a typical podium floor is \( \Delta_a = (0.01)(18 \text{ ft})(12 \text{ in.}/\text{ft}) = 2.16 \text{ in.} \).

The allowable story drift for a typical tower floor is \( \Delta_a = (0.01)(15 \text{ ft})(12 \text{ in.}/\text{ft}) = 1.80 \text{ in.} \).

The calculated design story drifts are amplified by the appropriate \( C_d \) factor from *Standard* Table 12.2-1 in elastic analysis procedures that employ seismic response coefficients reduced by the appropriate response modification factor, \( R \).

*Standard* Section 16.2.4.3 permits the allowable story drift obtained from a nonlinear response history analysis to be increased by 25 percent relative to the drift limit specified in Section 12.12.1.
The maximum allowable value of story drifts summed to the roof of the ten-story hospital building (156 feet) obtained from an elastic analysis is 18.72 inches. This same figure extracted from a nonlinear response history analysis cannot exceed 1.25(18.72 in.) = 23.40 in.

6.3.3.6 Seismic weight. The area of the tower floorplate is approximately equal to \[ [(3)(30 \text{ ft}) + (2)(8 \text{ in.})(1 \text{ ft/12 in.})]^2 = 8,342 \text{ ft}^2 \], while the area of the podium floorplate is approximately \[ [(7)(30 \text{ ft}) + (2)(8 \text{ in.})(1 \text{ ft/12 in.})] \times [(4)(30 \text{ ft}) + (2)(8 \text{ in.})(1 \text{ ft/12 in.})] = 25,642 \text{ ft}^2 \]. Thus, the weights that contribute to seismic forces are as follows:

- **Tower roof:**
  - Roof \( D \) = \( 0.135)(8,342) \) = 1,126 kips
  - Cladding = \( 4)(3)(30)(0.300) \) = 108 kips
  - Total = 1,234 kips

- **Tower floor:**
  - Floor \( D \) = \( 0.104)(8,342) \) = 868 kips
  - Partitions = \( 0.010)(8,342) \) = 83 kips
  - Cladding = \( 4)(3)(30)(0.300) \) = 108 kips
  - Total = 1,059 kips

- **Podium roof:**
  - Roof \( D \) = \( 0.135)(25,642 - 8,342) \) = 2,336 kips
  - Floor \( D \) = \( 0.104)(8,342) \) = 868 kips
  - Partitions = \( 0.010)(8,342) \) = 83 kips
  - Cladding = \( 2)(11)(30)(0.300) \) = 198 kips
  - Total = 3,485 kips

- **Podium floor:**
  - Floor \( D \) = \( 0.104)(25,642) \) = 2,667 kips
  - Partitions = \( 0.010)(25,642) \) = 256 kips
  - Cladding = \( 2)(11)(30)(0.300) \) = 198 kips
  - Total = 3,121 kips

Total effective seismic weight of building = 1,234 + 7(1,059) + 3,485 + 3,121 = 15,253 kips

6.3.4 Elastic Analysis

The base shear is determined using an ELF analysis; the base shear so computed is needed later when evaluating the scaling of the base shears obtained from the modal response spectrum analysis.

In a subsequent section (Section 6.3.6.3.3), columns are designed using forces obtained from nonlinear response history analyses that are intended to represent the maximum force that can develop in these elements per the exception to *Standard* Section 12.4.3.1. Compliance with story drift limits is also evaluated using the results of the nonlinear response history analyses.
6.3.4.1 Equivalent Lateral Force procedure. First, the ELF base shear will be determined, followed by its vertical distribution up the height of the building.

6.3.4.1.1 ELF base shear. Compute the approximate building period, \( T_a \), using Standard Equation 12.8-7:

\[
T_a = C_i h_n^x = \left(0.03\right)\left(156^{0.75}\right) = 1.32 \text{ sec}
\]

In accordance with Standard Section 12.8.2, the building period used to determine the design base shear must not exceed the following:

\[
T_{max} = C_u T_a = \left(1.4\right)\left(1.32\right) = 1.85 \text{ sec}
\]

The subsequent three-dimensional modal analysis finds the computed period to be approximately 2.30 seconds in each principal direction. Thus the upper limit on the fundamental period \( T_{max} \) applies.

The seismic response coefficient, \( C_s \), is computed in accordance with Standard Section 12.8.1.1. Equation 12.8-2 provides the value of \( C_s \) that generally governs at short periods:

\[
C_s = \frac{S_{DS}}{R/I} = \frac{0.859}{8/1.5} = 0.161
\]

However, Standard Equation 12.8-3 indicates that the value for \( C_s \) need not exceed the following:

\[
C_s = \frac{S_{DI}}{T(R/I)} = \frac{0.433}{(1.85)(8/1.5)} = 0.044
\]

and the minimum value for \( C_s \) per Standard Equation 12.8-5 is:

\[
C_s = 0.044/S_{DS} \geq 0.01 = \left(0.044\right)(1.5)(0.859) = 0.057
\]

Therefore, use \( C_s = 0.057 \).

The seismic base shear is computed per Standard Equation 12.8-1 as follows:

\[
V = C_s W = \left(0.057\right)(15,253) = 865 \text{ kips}
\]

The redundancy factor (\( \rho \)) is accounted for by setting the coefficient on the horizontal seismic load effect to 1.3 in all earthquake load combinations used for strength design of the BRBs. The redundancy factor is not applicable to the determination of deflections.

6.3.4.1.2 Vertical distribution of ELF seismic forces. Standard Section 12.8.3 prescribes the vertical distribution of lateral force in a multilevel structure. The floor force, \( F_x \), is calculated using Standard Equation 12.8-11 as:

\[
F_x = C_{yx} V
\]
where (per *Standard* Eq. 12.8-12):

$$C_{\text{vr}} = \frac{w_i h_i^k}{\sum w_i h_i^k}$$

Using the data in Section 6.3.3.5 of this example and interpolating the exponent $k$ as 1.68 for the period of 1.85 seconds, the vertical distribution of forces for the ELF analysis is shown in Table 6.3-2. The seismic design shear in any story is computed as follows (per *Standard* Eq. 12.8-13):

$$V_x = \sum_{i=x}^n F_i$$

**Table 6.3-2  ELF Vertical Seismic Load Distribution**

<table>
<thead>
<tr>
<th>Level</th>
<th>Weight ($w_x$)</th>
<th>Height ($h_x$)</th>
<th>$w_x h_x^k$</th>
<th>$C_{\text{vr}}$</th>
<th>$F_x$</th>
<th>$V_x$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>1,234 kips</td>
<td>156 ft</td>
<td>5,818,337</td>
<td>0.24</td>
<td>210 kips</td>
<td>210 kips</td>
</tr>
<tr>
<td>Story 10</td>
<td>1,059 kips</td>
<td>141 ft</td>
<td>4,215,392</td>
<td>0.18</td>
<td>152 kips</td>
<td>362 kips</td>
</tr>
<tr>
<td>Story 9</td>
<td>1,059 kips</td>
<td>126 ft</td>
<td>3,491,537</td>
<td>0.15</td>
<td>126 kips</td>
<td>488 kips</td>
</tr>
<tr>
<td>Story 8</td>
<td>1,059 kips</td>
<td>111 ft</td>
<td>2,823,657</td>
<td>0.12</td>
<td>102 kips</td>
<td>590 kips</td>
</tr>
<tr>
<td>Story 7</td>
<td>1,059 kips</td>
<td>96 ft</td>
<td>2,214,116</td>
<td>0.09</td>
<td>80 kips</td>
<td>670 kips</td>
</tr>
<tr>
<td>Story 6</td>
<td>1,059 kips</td>
<td>81 ft</td>
<td>1,665,744</td>
<td>0.07</td>
<td>60 kips</td>
<td>730 kips</td>
</tr>
<tr>
<td>Story 5</td>
<td>1,059 kips</td>
<td>66 ft</td>
<td>1,182,040</td>
<td>0.05</td>
<td>43 kips</td>
<td>772 kips</td>
</tr>
<tr>
<td>Story 4</td>
<td>1,059 kips</td>
<td>51 ft</td>
<td>767,496</td>
<td>0.03</td>
<td>28 kips</td>
<td>800 kips</td>
</tr>
<tr>
<td>Story 3</td>
<td>3,485 kips</td>
<td>36 ft</td>
<td>1,409,320</td>
<td>0.06</td>
<td>51 kips</td>
<td>851 kips</td>
</tr>
<tr>
<td>Story 2</td>
<td>3,121 kips</td>
<td>18 ft</td>
<td>395,253</td>
<td>0.02</td>
<td>14 kips</td>
<td>865 kips</td>
</tr>
<tr>
<td>Total</td>
<td>15,253 kips</td>
<td>23,982,891</td>
<td>1.00</td>
<td>865 kips</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1.0 kip = 4.45 kN
1.0 ft = 30.5 cm

All floor decks, including the roofs, are constructed of 3¾ in. lightweight concrete over 3 in. metal deck. *Standard Sec. 12.3.1.2* allows for such diaphragms to be modeled as rigid as long as their span-to-depth ratios do not exceed three. Since the floor span-to-depth ratio is a maximum of 1.75 at the podium levels, the hospital diaphragms meet these conditions. However, *Standard Sec. 12.3.1.2* also requires the structure to have no horizontal irregularities for its diaphragms to be modeled as rigid. Due to the out-of-plane offsets irregularity (Type 4) in the transverse lateral frames at the podium-to-tower interface, the hospital does not meet this restriction. As such, the effect on the vertical lateral force distribution of explicitly considering the stiffness of the diaphragm at the podium roof level was examined in a three-dimensional computer model of the structure and found to be insignificant (i.e., results closely matching a model with rigid diaphragms) due to the stiff nature of the thick concrete floor. Thus, it was deemed acceptable to model all diaphragms as rigid for subsequent analyses. The rigid diaphragm assumption is especially helpful in the context of nonlinear response history analysis, where the additional degrees of freedom needed to model diaphragm stiffness explicitly can render analysis times prohibitive.
Assessment of load transfer in the level 3 diaphragm would require a separate, explicit analysis. Another reasonable approach to the primary model would be to include the level 3 diaphragm explicitly and model all other diaphragms as rigid.

### 6.3.4.2 Three-dimensional static and Modal Response Spectrum Analysis

The three-dimensional analysis is performed for this example to accurately account for the following:

- The different centers of mass for the podium and tower levels
- The varying stiffness of the braced frames as they transition from their wide configuration at the podium levels to a single-bay arrangement throughout the tower
- The effects of bi-directional frame interaction on the columns engaged by orthogonal braced frames in the podium
- The ability of the braced frames to control torsion

The braced frames and diaphragm chords and collectors, together with all gravity system beams and columns, are explicitly modeled using three-dimensional beam-column elements. The floor diaphragms are modeled as rigid.

As mentioned previously, the ELF analysis procedure of Standard Section 12.8 is not admissible for this structure due to the restrictions on fundamental period in Standard Table 12.6-1. However, the ELF analysis of the three-dimensional model is still useful in assessing whether torsional irregularities are present. The ELF seismic forces derived in Table 6.3-2 above are applied to each diaphragm at 5 percent eccentricity orthogonal to the direction of loading. The maximum and average story drifts along an edge transverse to the direction of loading for the critical direction of eccentricity at each level are then compared. This ratio of $\frac{\delta_{\text{max}}}{\delta_{\text{avg}}}$ never exceeds 1.11, which is below the 1.2 limit that defines torsional irregularity. For this torsionally regular structure, the accidental torsion amplification factor, $A_t$, is equal to 1.0.

A three-dimensional modal response spectrum analysis is performed per Standard Section 12.9 using the three-dimensional computer model. The design response spectrum is based on Standard Section 11.4.5 and is shown in Figure 6.3-6.
Within this model, the first twelve modes of vibration and the corresponding mode shapes of the structure were determined. Twelve modes provide more than enough participation to capture 90 percent of the actual mass in each direction of response as required by Standard Section 12.9.1.

The design value for modal base shear, $V_t$, is determined by combining the individual modal values for base shear after dividing the design response spectrum by the quantity $R/I = 8/1.5 = 5.33$ as prescribed by Standard Section 12.9.2. The complete quadratic combination (CQC) modal combination rule was selected for this task to account for coupling of closely-spaced modes that are likely present in symmetrical structures. Five percent modal damping in all modes is specified for the response spectrum analysis to match the assumption used in deriving the design response spectrum of Figure 6.3-6. Base shears thus obtained from the model having an effective seismic weight of 15,253 kips are as follows:

- Longitudinal: $V_t = 627$ kips
- Transverse: $V_t = 637$ kips

In accordance with Standard Section 12.9.4, the design values of modal base shear are compared to the base shear determined by the ELF method. If the design value for modal base shear is less than 85 percent of the ELF base shear calculated using a period of $C_T$ (see Sec. 6.3.4.1.1 above), a factor greater than unity must be applied to the design forces to raise the modal base shear up to this minimum ELF comparison value. Accordingly:

- Multiplier: $0.85 \left( \frac{V}{V_t} \right)$
- Longitudinal multiplier: $0.85 \left( \frac{865 \text{ kips}}{627 \text{ kips}} \right) = 1.17$
- Transverse multiplier: $0.85 \left( \frac{865 \text{ kips}}{637 \text{ kips}} \right) = 1.15$

In a typical elastic analysis, it is recommended to examine lateral displacements early in the design process, as seismic (or wind) drift often controls the design of taller structures. For this building,
compliance with drift limits will ultimately be checked using a nonlinear response history analysis. However, lateral displacements are still examined in the elastic analysis to ensure they remain reasonably close to the limits derived in Section 6.3.3.5 (that is, in order to prevent wasting analysis effort on a design that is unlikely to meet the drift limits). This check is illustrated in Section 6.3.4.3 below.

To obtain elastic design forces for the BRBs, the results from the two orthogonal MRSAs in the three-dimensional model are combined via SRSS, exceeding the requirements of Standard Section 12.5.4.

6.3.4.3 Preliminary drift assessment. Seismic drift is examined in accordance with Standard Section 12.12.1. The design story drift in each translational direction was extracted from the three-dimensional ETABS model corresponding to the response spectrum case, including 5 percent accidental torsion, exciting that same direction. Although only strictly required for structures possessing torsional irregularities as defined by Standard Table 12.3-1, story drift was nonetheless examined at the building corners rather than the centers of mass for this structure because the location of cladding attachment is the most critical location for this check.

The lateral deflections obtained from the response spectrum analysis must be multiplied by $C_d/I(R) = C_d/I = 5/8 = 0.625$ to find the design story drift. However, the response spectrum used in the analysis has already been scaled twice. The spectrum was first scaled by $R/I = 8/1.5 = 5.33$ to obtain design-level forces; thus the resulting displacements can be amplified by $C_d/I = 5/1.5 = 3.33$ to obtain expected drifts. The second scaling was by a factor (in each direction) to ensure that the design base shear forces in each direction to meet the minimum 85 percent of ELF base shear. This latter scaling does not apply to drifts, per Standard Section 12.9.4. Thus, the 1.17 and 1.15 scale factors applied in the longitudinal and transverse directions, respectively, must be divided back out of the drifts extracted from the model used to obtain design forces for the braces. The resulting scale factors applied to the results of the scaled spectra are $3.33/1.17 = 2.85$ and $3.33/1.15 = 2.90$ applied in the longitudinal and transverse directions, respectively. (It would also be possible to simply perform an additional response spectrum analysis with the design spectrum multiplied by 0.625 and use the resulting story drifts directly.)

Story drifts in all ten stories of the hospital building are within the allowable story drift limit of $0.010h_{ex}$ per Standard Section 12.12.1 and Section 6.3.5.5 of this chapter. Although story drifts calculated using MRSA reach a maximum value at the roof level that is just 89 percent of the $0.010h_{ex}$ limit (and hence acceptable), a nonlinear response history analysis is nevertheless used to confirm that all story drifts indeed remain within prescribed limits.

A comparison of story drift ratios also confirms that no story drift ratio is more than 130 percent of that for the story above, as required to prove certain vertical irregularities are not present in the structure via the exception to Standard Section 12.3.2.2.

6.3.4.4 Second-order (P-delta) effects. AISC 360 requires consideration of second order effects. Such effects were investigated by conducting a three-dimensional P-delta analysis, which determined that secondary P-delta effects on the frame accounted for less than 10 percent of the primary demand. Furthermore, Standard Section 12.8.7 gives a different means of determining the significance of P-delta effects through the stability coefficient, $\theta$, defined in Standard Equation 12.8-6. In either case, P-delta effects were found to be insignificant for this particular braced frame structure.

6.3.4.5 Brace design force summary. The maximum axial forces in each level’s individual BRBs caused by horizontal earthquake loads are listed in Table 6.3-3. Again, each brace is designed for its share of 100 percent of the horizontal earthquake load effect times the redundancy factor ($\rho$) of 1.3 without considering additional vertical loads to encourage distributed yielding of braces up the height of
the structure. Nonetheless, the braces are also checked to ensure they do not yield under maximum live load (i.e., the load combination of $1.2D + 1.6L + 0.5L_r$).

Because the length of the yielding segment of a BRB is significantly less than its workpoint-to-workpoint length (see Sec. 6.5.3.1.1 below), the axial stiffness of the brace elements in the three-dimensional elastic analysis model must be adjusted to account for the non-prismatic nature of these elements. The modulus of elasticity of the steel in the brace elements was increased by a factor of 1.51 for single-diagonal and chevron bracing throughout the tower and 1.45 for chevron or V bracing configurations in the podium to match the true elastic stiffness of these elements as they are defined in the nonlinear response history analysis.

The sizes of the BRBF members are controlled by seismic loads, always bi-directional and with eccentricity, rather than wind loads. Standard Section 12.8.4.2 only requires that the 5 percent displacement of the center of mass associated with accidental torsion be applied in the direction that generates the greater effect when earthquake forces are applied simultaneously in two orthogonal directions. However, due to the intricacies of SRSS directional combination of response spectra in ETABS, the 5 percent offset is applied in both orthogonal directions at the same time, which is slightly conservative for torsional response (and is not a significant penalty for this particular building due to its regular nature in plan). The design of connections will be governed by the seismic requirements of AISC 341.

### Table 6.3-3 Design Axial Forces in Buckling-Restrained Bracing Members

<table>
<thead>
<tr>
<th>Location</th>
<th>Gridline A (kips)</th>
<th>Gridline D (kips)</th>
<th>Gridline 2, 3, 6, or 7 (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>139</td>
<td>144</td>
<td>142</td>
</tr>
<tr>
<td>Story 10</td>
<td>117</td>
<td>127</td>
<td>126</td>
</tr>
<tr>
<td>Story 9</td>
<td>135</td>
<td>136</td>
<td>133</td>
</tr>
<tr>
<td>Story 8</td>
<td>142</td>
<td>141</td>
<td>141</td>
</tr>
<tr>
<td>Story 7</td>
<td>152</td>
<td>151</td>
<td>151</td>
</tr>
<tr>
<td>Story 6</td>
<td>167</td>
<td>175</td>
<td>170</td>
</tr>
<tr>
<td>Story 5</td>
<td>193</td>
<td>195</td>
<td>197</td>
</tr>
<tr>
<td>Story 4</td>
<td>212</td>
<td>214</td>
<td>210</td>
</tr>
<tr>
<td>Story 3</td>
<td>152</td>
<td>197d</td>
<td>187</td>
</tr>
<tr>
<td>Story 2</td>
<td>162</td>
<td>233</td>
<td>195</td>
</tr>
</tbody>
</table>

---

\(a\)Individual maxima are not necessarily on the same frame; values are maximum for any frame.  
\(b\)All braces are oriented in the chevron configuration except for single diagonal or V.  
1.0 kip = 4.45 kN

### 6.3.5 Initial Proportioning and Details

The BRBFs occur on Gridlines 3, 6, A D in the tower and transfer their loads at the third floor to two BRBFs per line on Gridlines 2, 7, A D in the podium. These frames are shown schematically in plan in Figures 6.3-1 and 6.3-2 and in elevation in Figures 6.3-3 and 6.3-4. Using the horizontal component of the seismic load (amplified by the redundancy factor) as determined by response spectrum analysis and the loads from Table 6.3-3, the proportions of the braces are checked for adequacy. Then, initial sizes for
the lateral columns, beams collectors are determined from the three-dimensional elastic analysis model using capacity design principles and relevant plastic mechanism analyses. In the preliminary elastic design stage, generic BRB properties are used to derive expected brace strengths. This allows for a specific BRB supplier to be selected further downstream in the project schedule, as is usually done in traditional project delivery methods. Design forces for columns are obtained from a summation of the vertical component of the adjusted brace strengths above the level of interest, while those for horizontal elements are derived from two different plastic mechanisms, always using adjusted brace strengths in tension and compression as required by AISC 341 Section 16.5b. All lateral columns are then subject to resizing according to the force and displacement demands determined using nonlinear response history analysis.

6.3.5.1 Buckling-Restrained Brace sizes

6.3.5.1.1 Buckling-Restrained Brace mechanics. The mechanics of BRBs are such that compression buckling need not be considered in their selection. Their required strength is controlled by yielding of the steel core material only. A BRB consists of a steel core that resists imposed axial stresses together with a mortar-filled sleeve that resists buckling. The steel core has both a yielding portion and two non-yielding portions at its ends where the cross-section enlarges to facilitate connection to a gusset plate. A debonding agent, often proprietary, decouples the axial behavior of the core from the buckling behavior of the sleeve. In compression, a BRB acts as a sleeved column—the steel core is able to achieve the full magnitude of its squash load while, at the same time, the sleeve can provide its full Euler buckling resistance without taking on any axial load. From a performance standpoint, such a component produces very desirable balanced hysteretic behavior that exhibits both isotropic and kinematic (cyclic) strain-hardening. Unlike conventional bracing, BRB behavior is much more symmetric with respect to tension and compression and is not subject to strength and stiffness degradation.

6.3.5.1.2 Steel core area. According to AISC 341 Section 16.2a, the steel core must resist the entire axial force in the brace. This force is tabulated in Table 6.3-3. The brace design axial strength, \( \phi P_{\text{ysc}} \), in either tension or compression, as controlled by the limit state of yielding, is equal to the following:

\[
\phi P_{\text{ysc}} = \phi F_{\text{ysc}} A_{\text{sc}}
\]

where:

\( \phi = 0.90 \)

\( F_{\text{ysc}} = \) specified minimum yield stress (or actual from coupon test) of the steel core

\( A_{\text{sc}} = \) net area of steel core

Setting \( \phi P_{\text{ysc}} \) equal to the \( P_u \) values in Table 6.3-3 and rearranging terms, the required net area of steel core can be expressed as follows:

\[
A_{\text{sc}} = \frac{P_u}{\phi F_{\text{ysc}}}
\]

This required area, together with the actual steel core area provided, is shown for each brace in Table 6.3-4. Rarely do designers know the actual yield stress of the steel core during the design phase;
hence, a minimum yield stress \( (F_{yse}) \) of the steel core equal to 38 ksi is assumed for this example. This minimum yield stress value would be specified on the design drawings.

### Table 6.3-4 Steel Core Areas for Buckling-Restrained Bracing Members

<table>
<thead>
<tr>
<th>Location</th>
<th>Gridline A (kips)</th>
<th>Gridline D (kips)</th>
<th>Gridline 2, 3, 6, or 7 (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( A_{sc} ) req’d (in.²)</td>
<td>( A_{sc} ) (in.²)</td>
<td>( A_{sc} ) req’d (in.²)</td>
</tr>
<tr>
<td>Roof</td>
<td>4.06</td>
<td>4.5</td>
<td>4.21</td>
</tr>
<tr>
<td>Story 10</td>
<td>3.42</td>
<td>3.5</td>
<td>3.71</td>
</tr>
<tr>
<td>Story 9</td>
<td>3.95</td>
<td>4.0</td>
<td>3.98</td>
</tr>
<tr>
<td>Story 8</td>
<td>4.15</td>
<td>4.5</td>
<td>4.12</td>
</tr>
<tr>
<td>Story 7</td>
<td>4.44</td>
<td>4.5</td>
<td>4.42</td>
</tr>
<tr>
<td>Story 6</td>
<td>4.88</td>
<td>5.0</td>
<td>5.12</td>
</tr>
<tr>
<td>Story 5</td>
<td>5.64</td>
<td>6.0</td>
<td>5.70</td>
</tr>
<tr>
<td>Story 4</td>
<td>6.20</td>
<td>6.5</td>
<td>6.26</td>
</tr>
<tr>
<td>Story 3</td>
<td>4.44</td>
<td>4.5</td>
<td>5.76c</td>
</tr>
<tr>
<td>Story 2</td>
<td>4.74</td>
<td>5.0</td>
<td>6.81</td>
</tr>
</tbody>
</table>

\( ^a \)All braces are oriented in the chevron configuration except for single diagonal \( ^b \) or V \( ^c \)

1.0 in = 25.4 mm

#### 6.3.5.2 Lateral force-resisting columns. To design the frame containing the BRBs, unless using a nonlinear analysis, the designer should assume a plastic mechanism in which all BRBs are yielding in tension or compression and have reached their strain-hardened adjusted strengths, including all sources of overstrength. These adjusted brace strengths per AISC 341 Section 16.2d are as follows:

- **Compression:** \( \beta \omega R_y F_{yse} A_{sc} \)
- **Tension:** \( \omega R_y F_{yse} A_{sc} \)

The adjusted brace strength values represent the yield strength of the steel core adjusted for material overstrength \( (R_y) \), strain-hardening \( (\omega) \) compression overstrength \( (\beta) \). Whereas conventional bracing usually buckles in compression well before reaching its yield strength, BRBs are often slightly stronger in compression than in tension. The strain-hardening \( (\omega) \) and compression overstrength \( (\beta) \) factors traditionally are provided by BRB manufacturers and are calculated from cyclic sub-assemblage testing to a brace deformation equivalent to twice the design story drift per AISC 341 Appendix T.

For the initial proportioning of braced-frame columns and beams using the three-dimensional elastic analysis model, generic values of \( \beta = 1.05, \omega = 1.36 \) \( R_y = 1.21 \) are assumed. A material overstrength factor, \( R_y \), of 1.21 is selected to bring the design yield strength of the steel core, \( F_{yse} = 38 \) ksi, up to the typical maximum specified steel core yield strength of 46 ksi. Note that all of these values are subject to revision for use in the nonlinear response history analysis once the BRB calibration has been performed in Section 6.3.6.2.
This capacity design methodology can easily be implemented to design lateral columns in a BRBF once the three-dimensional analysis model is constructed. The designer simply needs to generate an axial force in each brace corresponding to its adjusted brace strength as defined above, either by deleting the braces from the model and replacing them with their associated forces directly or by some other method that achieves the same result (for example, hand calculations or spreadsheets). Two different load cases must be examined for each principal direction: one with all braces in either tension or compression based on lateral load originating from one side of the frame a second with the brace forces determined by lateral load originating from the other side of the frame.

Appropriate consideration should be given to bi-directional combination of the resulting brace loads on columns engaged by two orthogonal frames. While AISC 341 is unwavering in its requirement to design columns for the full adjusted brace strengths, the displacement corresponding to the adjusted brace strength should remain constant in any direction. Thus, such a displacement imposed at 45 degrees to the principal building axes will cause yielding of all braced frames, but not full strain hardening. This reduction factor for bi-directional loading is dependent on the brace’s post-yield behavior and will not be much less than one for a ductile system such as a BRBF. The 100%/30% orthogonal combination procedure defined in Standard Section 12.5.3 is not applicable in the context of capacity design as mandated for columns by AISC 341 Section 16.5b.

To complete the preliminary column design, the column axial loads resulting from the maximum expected brace forces defined above are substituted for the earthquake load effect and combined with vertical loads as specified in Section 5.3.3.4. The translation and twist of all diaphragms should be locked when performing the column design for stability of the model. This will ensure each column is designed for the axial force equal to the summation of the vertical components of the adjusted brace strengths of all braces above it. Column flexural forces are not considered in their design, consistent with AISC 341 Section 8.3 (1). Such an approximation is valid because localized flexural yielding of a column at locations where it receives a brace is deemed acceptable from a performance standpoint. The preliminary column designs can be seen in Figures 6.3-3 and 6.3-4.

To illustrate this process, a detailed calculation of the column design forces for the column at D4 can be seen in Table 6.3-5. Because column strengths are governed by compression buckling rather than yielding, brace actions that induce compression on the columns are considered critical. For the uppermost column below the tower roof, the 228-kip design force is equal to the sum of the design load due to gravity/vertical earthquake effect of 102 kips and the vertical component of the adjusted brace strength in tension at that level, equal to 126 kips. The adjusted brace strength in tension is used here because tension in the single diagonal brace at the roof will impose compressive forces on the column below, as can be seen in the elevation of Figure 6.3-3. At lower levels with chevron bracing, the design axial load in the column is calculated as follows:

1. Start with the vertical component of the roof brace in tension (126 kips).
2. Add the associated design gravity and vertical earthquake effect loading at that level.
3. Add the sum of the vertical components of the adjusted brace strengths in compression of all chevron bracing at levels above.
4. Subtract half of the sum of the unbalanced vertical loads (difference in vertical components of adjusted brace strengths in compression and tension) on the beams intersecting all chevron bracing at that level and above.
This procedure recognizes that for levels with chevron bracing, the adjusted brace strength in compression will always control over that in tension and will enter the column below at the base of that level. Additionally, the unbalanced vertical load from the chevron transmits shear to the beam above and ultimately the column that works to alleviate the downward gravity and brace compression forces.

**Table 6.3-5 Determination of Column Design Forces for Column at Gridline D4**

<table>
<thead>
<tr>
<th>Level</th>
<th>Brace Area (in²)</th>
<th>(1.2+0.2SD)D + 0.5L + 0.2S</th>
<th>Brace Angle α</th>
<th>Vertical Component of Adjusted Brace Strengths</th>
<th>Column Required Strength Pu</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>4.5</td>
<td>102 kips</td>
<td>26.6°</td>
<td>126 kips, 132 kips</td>
<td>228 kips</td>
</tr>
<tr>
<td>Story 10</td>
<td>4.0</td>
<td>196 kips</td>
<td>45°</td>
<td>177 kips, 186 kips</td>
<td>318 kips</td>
</tr>
<tr>
<td>Story 9</td>
<td>4.0</td>
<td>293 kips</td>
<td>45°</td>
<td>177 kips, 186 kips</td>
<td>596 kips</td>
</tr>
<tr>
<td>Story 8</td>
<td>4.5</td>
<td>390 kips</td>
<td>45°</td>
<td>199 kips, 209 kips</td>
<td>873 kips</td>
</tr>
<tr>
<td>Story 7</td>
<td>4.5</td>
<td>486 kips</td>
<td>45°</td>
<td>199 kips, 209 kips</td>
<td>1174 kips</td>
</tr>
<tr>
<td>Story 6</td>
<td>5.5</td>
<td>583 kips</td>
<td>45°</td>
<td>243 kips, 255 kips</td>
<td>1474 kips</td>
</tr>
<tr>
<td>Story 5</td>
<td>6.0</td>
<td>681 kips</td>
<td>45°</td>
<td>265 kips, 279 kips</td>
<td>1821 kips</td>
</tr>
<tr>
<td>Story 4</td>
<td>6.5</td>
<td>780 kips</td>
<td>45°</td>
<td>288 kips, 302 kips</td>
<td>2191 kips</td>
</tr>
<tr>
<td>Story 3</td>
<td>none</td>
<td>947 kips</td>
<td>-</td>
<td>-</td>
<td>2660 kips</td>
</tr>
<tr>
<td>Story 2</td>
<td>none</td>
<td>1106 kips</td>
<td>-</td>
<td>-</td>
<td>2819 kips</td>
</tr>
</tbody>
</table>

*aAll braces are oriented in the chevron configuration except for single diagonal at the roof level.

*bMeasured from the horizontal.

*cωRyFyscAsc sin α

dβωRyFyscAsc sin α

1.0 kip = 4.45 kN

1.0 in = 25.4 mm

Columns are spliced at every other level to simplify erection. Column sizes are subject to revision due to results from nonlinear response history analysis. While tension forces in the columns may not control their design, tension demands can certainly affect the design of base plates, anchor rods drilled piers. The design tension force at the base of this same column calculated in a similar manner (using the appropriate load combination) is equal to 1,222 kips.

**6.3.5.3 Lateral force-resisting beams.** The braced frame beams were designed for gravity loads corresponding to $1.37D + 0.5L + 0.2S$, without accounting for any mid-span support provided by chevron bracing, together with earthquake loads extracted from two different plastic mechanism analyses. The first of these mechanisms assumes both the brace(s) above and below the beam of interest have reached their full adjusted brace strengths. The beam is then designed for its share (depending on its location along the line of framing) of the horizontal component of the resulting story force together with the largest drag force from the brace(s) above. In the second mechanism, the brace(s) below the beam of interest is assumed to have reached their full adjusted brace strengths at the same time the diaphragm reaches its design force (this mechanism requires a rough diaphragm analysis to derive collector forces). The beam must resist the same share of the horizontal component of the diaphragm force at that line of framing plus the largest drag force from the brace(s) above, determined by the difference between the force in the brace(s) below minus the diaphragm force at that line of framing. In either mechanism, the
braced frame beam is also designed for any unbalanced upward component of the BRBs that would arise in a chevron or V-bracing configuration. These mechanisms and their associated forces are illustrated in Figure 6.3-7.

\[
\omega R_y F_{yu,a_a} (\beta - 1) \sin \alpha = (1.36)(1.21)(38)(4)(1.05 - 1)(\sin 45^\circ) = 9 \text{ kips}
\]

This unbalanced upward component reduces the shear demand in the beam by \( P/2 = 9/2 = 5 \text{ kips} \) and the moment by \( PL/4 = (9)(30)/4 = 66 \text{ kip-ft} \). Hence the moment demand that will eventually be combined with the critical axial demand determined by plastic mechanism analysis is:

\[
128 \text{ kip-ft} - 66 \text{ kip-ft} = 62 \text{ kip-ft}
\]

The first plastic mechanism shown in Figure 6.3-7 considers both the brace(s) above and below the beam of interest to have reached their adjusted strengths. Hence, the plastic story shear below the tenth floor is equal to:

\[
\omega R_y F_{yu,a_a}(1 + \beta) \cos \alpha = (1.36)(1.21)(38)(4)(1 + 1.05)(\cos 45^\circ) = 363 \text{ kips}
\]

The plastic story shear above the tenth floor is:

\[
\beta \omega R_y F_{yu,a_a} \cos \alpha = (1.05)(1.36)(1.21)(38)(4.5)(\cos 26.6^\circ) = 264 \text{ kips}
\]

(The brace at the tenth story is a single-diagonal.)
The story force corresponding to this yielding mechanism is equal to the difference between these two values, or 99 kips. Since the chevron braces below the tenth floor beam receive the lateral force at the midpoint of the associated line of framing, half of this story force (attributed to inertial mass) is presumed to come from each half of the braced frame beam, or approximately 50 kips. This 50-kip force is added to the 264-kip horizontal component from the yielding single-diagonal brace above that must be dragged through the braced frame beam to the chevron braces below, resulting in a design axial force of 314 kips for the first plastic mechanism.

The second plastic mechanism illustrated in Figure 6.3-7 requires estimation of the force entering the braced frame beam from the diaphragm at that floor in accordance with Standard Section 12.10. As shown in Figure 6.3-7 this is a collector force thus the value of \( F_{px} \) calculated using Standard Equation 12.10-1 requires amplification by the overstrength value, \( \Omega_{0} \), of 2.5; this gives 2.5 \times 142 kips = 355 kips. This value cannot be taken as less than the minimum of 0.2\( S_{DS}\omega_{wpx} \), which gives a value of 270 kips. (Note that the overstrength factor does not apply to this minimum, even for collectors.) Thus, the 355 kips governs the corresponding frame force can be taken as 55 percent of this force (that is taking 1/2 adding 10 percent to account for accidental eccentricity): 195 kips. The chevron braces below the tenth-floor braced frame beam are still assumed to have reached their yield strength, resulting in the same 363-kip frame shear at that level. Hence the statically consistent force in the (now elastic) single-diagonal brace above the tenth floor is equal to the difference between these two values, or 168 kips. Just as is done in the calculation of the design axial force resulting from the first plastic mechanism, half of the 195-kip story force (the maximum from the diaphragm) is added to the 168-kip horizontal component from the single-diagonal brace above that must be dragged through the braced frame beam to the chevron braces below, resulting in a design axial force of 265 kips for the second plastic mechanism.

The 314 kips obtained from the first plastic mechanism (both braces above and below the beam at their adjusted strengths) controls. Thus, the braced frame beam along Gridline D above the ninth story must be designed for a 62 kip-ft moment in combination with a 314 kip axial force. The axial strength is typically determined without accounting for the benefits of composite action with the concrete-filled deck above. Although some minor benefit can be obtained from considering the composite contribution to flexural strength, this is often neglected for simplicity; flexural forces due to brace unbalanced loading are typically small in the inverted-V configuration they oppose gravity forces. The axial capacity of braced frame beams is often controlled by flexural-torsional buckling (as opposed to buckling about the weak axis).

6.3.5.4 Third floor/low roof collector forces. Collector elements that transfer forces between the single bay of chevron bracing in the tower to the multiple, offset bays of chevron bracing in the podium are sized in a manner identical to the lateral force-resisting beams. The same two plastic mechanisms—one involving braces above and below the level of interest reaching their full adjusted strengths the second involving the braces below the level of interest reaching their full adjusted strengths in conjunction with the diaphragm delivering its maximum force to the framing in line with the braces—are assumed the forces are traced from the single bays of chevron bracing above through the collector lines to the chevron bracing below. Story forces accumulate in the collectors and braced frame beams based on the fraction of the full length of the line of framing represented by the particular beam section of interest. In the case of the out-of-plane offset that occurs between Gridlines 2 and 3 (and 6 and 7), the horizontal component of the adjusted chevron brace strengths above the third floor must be distributed into the diaphragm via the adjacent collector elements, then collected by collector elements along the outer line of framing (at Gridlines 2 or 7) to be channeled to the chevron braces in the podium levels below. As with the braced frame beams, the axial capacity of these collector elements is based on that of the bare steel section and usually is governed by flexural-torsional buckling. It is acceptable to calculate the flexural capacity of the collector elements considering composite action with the concrete-filled deck above.
Chapter 6: Structural Steel Design

6.3.5.5 Connection design. According to AISC 341 Section 16.3, gussets and beam-column connections must be designed for \(1.1 \times C_{\text{max}}\), where \(C_{\text{max}}\) is the adjusted brace strength in compression as defined in AISC 341 Section 16.2d. Connection design is not illustrated here since this topic is more thoroughly treated elsewhere and is not unique to BRBF systems. The connection of the 4.5 in.\(^2\) single diagonal brace to the frame beam below it at the tenth floor would need to be designed for \(1.1 \beta \omega R_{fy} F_{\text{yy}} A_{sc} = 1.1(1.05)(1.36)(1.21)(38)(4.5) = 325\) kips in tension and compression based the full expected and strain-hardened brace capacity. However, it should be mentioned that a designer might consider using NRHA to design for potentially reduced connection force demands; such a reduction would likely be limited to establishing a lower value for the strain-hardening factor \(\omega\) for each brace.

6.3.6 Nonlinear Response History Analysis

After completing a preliminary design using three-dimensional modal response spectrum analysis, nonlinear response history analysis is performed to:

- Establish brace deformation demands (and verify the adequacy of specified braces for the application)
- Determine expected drifts
- Re-examine the required strength of column members in the BRBF

The braced frames and diaphragm chords and collectors, together with all primary gravity system beams and columns, are explicitly modeled using three-dimensional beam-column elements. Secondary gravity framing is omitted from the nonlinear model for simplicity. The floor diaphragms are still modeled as rigid.

The specific goals of the nonlinear response history analysis are threefold. First, even though the original elastic design is found to comply with the drift limitations of Standard Section 12.12.1, the nonlinear response history analysis can more accurately predict results such as story drift. Hence, the acceptability of the design in satisfying story drift requirements is evaluated using the procedures of Standard Section 16.2.4. Second, the ability of the BRBs to perform at the Immediate Occupancy (IO) performance level in a design basis earthquake (DBE) event will be verified explicitly. Third, the required strength of column elements in the BRBF system is re-evaluated according to the “maximum force that can be developed by the system” as permitted by AISC 341-05 Section 16.5b. The use of a nonlinear response history analysis to determine this maximum force for individual elements is justified in the provisions of Standard Section 12.4.3.1. Specifically, the exception to this section permits the determination of “the maximum force that can develop in the element … by a rational, plastic mechanism analysis or nonlinear response analysis utilizing realistic expected values of material strengths.” Standard Section 16.2 then defines the requirements for nonlinear response history analysis. In this design example, only the columns in the BRBF system are examined. This is due to the limited savings potential of economizing the relatively small number of BRBF beam and collector elements in the structure that have already been reasonably sized (attributable to the absence of large BRB elements) using a rational plastic mechanism analysis. The designer of a taller building with bulky BRB elements should probably consider using the same procedure to potentially reduce design force demands on the BRBF beams and collectors as well as the columns.
A second, separate three-dimensional building model is assembled in the PERFORM program its similarity to the ETABS model used for the elastic design is confirmed by comparing fundamental periods and loads.

6.3.6.1 Design ground motions. In Chapter 3, risk-targeted maximum considered earthquake (MCE) response spectra are determined in accordance with Standard Section 11.4 using NGA attenuation relations. Seven pairs of time histories are selected and scaled to be consistent with the event magnitudes, fault distances source mechanisms controlling the MCE spectrum for the Seattle, Washington, hospital site. The base ground motions in the suite are scaled to the MCE response spectrum so as to satisfy the requirements of Standard Section 16.1.3.2 for periods between 0.18 and 4.95 seconds. The translational structural periods for the hospital facility are found to be approximately 2.3 seconds, so the period range of interest is from $0.2 \times 2.3 = 0.46$ seconds to $1.5 \times 2.3 = 3.45$ seconds, which is narrower than the preliminary range used in the ground motion selection and scaling. Design-level ground motions are obtained by multiplying MCE motions by $2/3$ per Standard Section 16.2.3.

6.3.6.2 Basis of nonlinear design. In keeping with the intent of the building code to protect essential facilities in a seismic event, the hospital should perform at the IO performance level under ground shaking corresponding to the DBE. As is traditionally the case in elastic designs, an explicit performance check at the MCE is not done here. The building is assumed to meet Life Safety (LS) performance objectives at the MCE if it meets IO performance criteria at the DBE.

At the present time, there are three international providers of BRBs: CoreBrace and StarSeismic in the United States Nippon Steel in Japan. In the preliminary design stage, it is often the case that any one of these three brace manufacturers may ultimately be chosen as the supplier. Thus, generic BRB properties may be assumed until the later stages of a project. However, a specific BRB supplier must be selected to accurately model actual brace behavior in the PERFORM NRHA model.

BRBs in the NRHA are modeled using expected properties based on test results provided by CoreBrace; acceptance criteria are based on Section 2.8.3 of ASCE 41 for deformation-controlled elements. There are three different types of brace-to-gusset connections used for BRBs: bolted, welded pinned. However, generic “ACME” braces are used throughout the structure hence the nature of the brace-to-gusset connection is not considered. All other structural elements are modeled using expected properties. Because the lateral beams and columns are connected by simple pin connections in this example (see Section 6.3.2.1), the beams will not require consideration of inelastic behavior. Unless some column element sizes are reduced based on NRHA results, inelastic column actions are not likely, although it is possible that uneven (or higher-mode) story drifts will result in inelastic flexural demands.

The initial gravity load condition is $1.0D + 0.25L$ per Standard Section 16.2.3.

6.3.6.3 Buckling-Restrained Brace calibration. In order to capture the nonlinear BRB behavior as accurately as possible in the NRHA computer model, brace properties are calibrated to match test results provided by CoreBrace. Typically, a number of calibrations are performed on a range of different brace sizes. Critical modeling parameters are then interpolated between (or extrapolated from) those established for a brace or braces of similar size. For illustration purposes, one calibration is shown and modeling parameters are matched to corresponding values for the flat core plate test specimen. (Note that the specific series of BRBs tested possess a post-yield modulus of elasticity that is unusually stiff.)

Data to which critical brace parameters are calibrated is obtained from a standard cyclic testing protocol that is in line with the requirements of AISC 341 Appendix T and ASCE 41 Section 2.8.3. Elastic displacement components in both the connection region and the non-yielding brace segments (the larger “elastic bar” segment in PERFORM) of the BRB specimens are subtracted out of measured displacements.
in the test data. As a result, the inelastic core plate’s force-displacement behavior is isolated to facilitate calibration. The result of the calibration is a backbone curve that reasonably envelopes the observed cyclic test hysteretic behavior.

BRB elements typically are modeled in nonlinear computer analysis software as three components in series: a stiff connection region, an elastic bar segment an inelastic yielding segment. The workpoint-to-workpoint length of the BRB element in the three-dimensional computer model must be divided reasonably into these three components. Thus, the stiff connection zone size and relative stiffness are determined by a rough gusset plate design based on a representative brace size and geometry in the building. The elastic bar segment’s length and cross-sectional area are set such that their proportions relative to the inelastic yielding segment remain identical to those of the test specimen. The remainder of this section is devoting to developing modeling parameters for the inelastic yielding portion of the BRB element.

One key modeling parameter for the inelastic portion of the BRB elements is its initial elastic stiffness, $K_o$. This value is simply set to equal $A_{se}E / L_y$, where $L_y$ is the length of the yielding segment set to equal the same proportion of the brace length outside the connection region as the test specimen. Next, the designer must select a post-yield stiffness, $K_f$, for the inelastic portion of the backbone curve. This value is chosen such that the post-yield slope of the hysteresis loops obtained from the computer model is similar to that in the test data is typically a percentage of the initial stiffness, $K_o$. For the hospital building in this example, $K_f = 1.5$ percent of $K_o$ to match the observed hysteretic behavior of the test specimen.

Other key modeling inputs are the strain-hardening and compression overstrength factors that define the ultimate strength of the BRB elements. Each brace in the hospital computer analysis model is assigned strain-hardening ($\omega$) and compression overstrength ($\beta$) factors equal to 1.63 and 1.05, respectively. These values are selected to match the full hardened strength of the test specimen in tension and compression and are different from the generic values assumed earlier for the plastic mechanism analysis used to design beams in columns in the BRBF. Because the compression overstrength factor does not equal 1.0, the BRB behavior is not symmetric in tension and compression. The material strength of the braces in the model is set to equal the minimum specified strength for this project (i.e., $R_y F_y = 38$ ksi). Alternatively, one could use the actual material strength as determined by coupon test of BRB specimens tested specifically for that particular project.

All nonlinear analysis software programs require an ultimate deformation or strain value corresponding to the BRB having reached its fully-hardened strength as an input in the inelastic BRB component’s properties. This figure was set such that the strain matched that observed in the test at full compression hardening. Additionally, the program requires information about the rate of isotropic hardening between cycles. In the nonlinear analysis software used for this design example, the rate of isotropic hardening is captured by inputting the maximum brace deformation corresponding to the average of the initial BRB yield strength and its strength after full hardening (this cyclic hardening parameter can also be defined in terms of accumulated deformations but is not done so here). To match the vertical progression of hysteretic behavior (i.e., hardening between progressive loading cycles) observed in the test specimen, this hardening parameter was set to 2/3 of the deformation from initial yield to fully-hardened strength. Finally, the software must know at what point the deformation in the nonlinear BRB component has exceeded its capacity. Since the BRBs in this example are expected to perform well below their capacity and this value was never reached during the standard testing protocol, an artificially high number was selected for this input individual brace performance was subject to review as outlined in Section 6.3.6.3.1.

The above parameters are sufficient to define a bilinear elastic-plastic backbone curve for the BRB elements in the computer analysis model. The hysteretic behavior of the BRB component matching the test specimen extracted from the computer model is compared with the experimental test data in
Figure 6.3-8.  One can see that the backbone curve modeling parameters accurately capture the observed experimental hysteresis properties for this size component.  With some additional effort, a trilinear backbone curve can also be calibrated to the test data to better capture the “rounding” of the actual BRB hysteretic loops.  The trilinear curve considers a higher component stiffness value just after yield before the ultimate post-yield stiffness value, $K_f$, prevails.  This third stiffness value, together with the force-deformation point at which the ultimate post-yield stiffness ($K_f$) “takes over,” must be calibrated to the test data as well.  Figure 6.3-9 shows the inelastic BRB component’s hysteretic behavior as modeled using the trilinear backbone curve together with the experimental test data to which it is calibrated.  Subsequent analysis results are based on BRB inelastic components modeled using the trilinear backbone curve because the additional accuracy of the tri-linear backbone curve can be realized without causing excessive analysis run times for this example building.

**Figure 6.3-8**  Bilinear BRB calibration ($A_c = 12$ in$^2$)  
(1.0 in. = 25.4 mm; 1.0 kip = 4.45 kN)
6.3.6.4 Results.

6.3.6.4.1 BRB strains. One goal of the NRHA is to confirm the ability of the BRBs to perform at the IO performance level in a DBE event. The acceptance criteria for BRB strain, as dictated by ASCE 41 Section 2.8.3, can be seen in Figure 6.3-10 (overlaid on the corresponding test results). No permanent, visible damage was observed during the standard experimental testing protocol used to calibrate the BRB elements the test was terminated at or near Point 2 as defined in the Type 1 and Type 2 component force versus deformation curves for deformation-controlled actions in ASCE 41 Section 2.4.4.3. Consequently, the IO, LS Collapse Prevention (CP) acceptance criteria are equal to $0.67 \times 0.75, 0.75 \times 1.0$ times the deformation at Point 2 on the component force versus deformation curves, respectively. Observe that the limiting BRB strains for IO performance in tension and compression are $6.64 \Delta_y / L_y = 6.64 \times 0.00131 = 0.008704$ and $6.47 \Delta_y / L_y = 6.47 \times 0.00131 = 0.008482$, in turn.
The ratio of maximum inelastic deformation demand $\Psi = \Delta / \Delta_y$ observed along a subset of the 92 total BRB elements during each of the seven ground motion time histories to the relevant IO performance point in tension or compression is shown in Table 6.3-6. Braces for which results are presented are chosen to represent BRB elements in all ten levels of the structure. As permitted by Standard Section 16.2.4 for analyses including at least seven ground motion pairs, the average BRB $\Psi_{max} / \Psi_{IO}$ value across the seven earthquake records is calculated for each brace. In assessing the performance of the entire structure, the maximum of these 92 average BRB inelastic deformation demand values is extracted and compared with the relevant IO performance point in tension or compression. Table 6.3-6 shows that this critical $\Psi_{max} / \Psi_{IO}$ value for the hospital structure is equal to 0.566, indicating acceptable performance of the BRB elements in the DBE event according to the methodology set forth in Standard Section 16.2.4. The procedure set forth in the Standard may lead to designs that fail a criterion in some element or measure for every ground motion but still pass the criteria on average.

Table 6.3-6  Ratio of Maximum BRB Inelastic Deformation Demand $\Psi$ to Immediate Occupancy (IO) Performance Limit ($\Psi_{max} / \Psi_{IO}$)

<table>
<thead>
<tr>
<th>BRB ID</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>Average</th>
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<tbody>
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<td>0.350</td>
<td>0.173</td>
<td>0.159</td>
<td>0.125</td>
<td>0.250</td>
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<td>0.255</td>
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<td>0.179</td>
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<td>0.201</td>
<td>0.282</td>
<td>0.732</td>
<td>0.351</td>
</tr>
<tr>
<td>16</td>
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<td>0.189$^a$</td>
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<td>0.185</td>
<td>0.266</td>
<td>0.294</td>
<td>0.215</td>
<td>0.212</td>
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</tbody>
</table>
Table 6.3-6 Ratio of Maximum BRB Inelastic Deformation Demand $\Psi$ to Immediate Occupancy (IO) Performance Limit ($\Psi_{\text{max}}/\Psi_{\text{IO}}$)

<table>
<thead>
<tr>
<th>BRB ID</th>
<th>Record ID</th>
<th>Average</th>
</tr>
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<tbody>
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</tr>
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<td>90</td>
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<tr>
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</tr>
<tr>
<td>92</td>
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<td>0.366</td>
</tr>
<tr>
<td>Max</td>
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<td>1.286</td>
</tr>
</tbody>
</table>

$^a$ All $\Psi_{\text{max}}/\Psi_{\text{IO}}$ values for the individual braces are controlled by the compression strain limit except those denoted by the superscript $^a$ (for which the tension strain limit controls).

$^b$ Note that the 0.566 structure “usage ratio” is equal to the maximum of the average $\Psi_{\text{max}}/\Psi_{\text{IO}}$ values across the seven ground motion pairs for each BRB element, not the average of the maximum $\Psi_{\text{max}}/\Psi_{\text{IO}}$ values for each of the seven ground motion pairs across all 92 BRB elements.

6.3.6.4.2 Drift assessment. Although seismic drift was preliminarily examined in Section 6.3.4.3 to ensure the design possessed reasonable stiffness before modeling its nonlinear behavior, conformance with story drift limits is assessed using the procedures of Standard Section 16.2. The maximum story drift ratio in each translational direction was extracted from the three-dimensional PERFORM model for each of the seven ground motion pairs. As before, story drift was examined at the building corners rather than the center of mass.

Once the maximum story drift ratio at every story and corner of the floorplate is identified in each of the seven time histories, the resulting seven values for each story are averaged (as allowed by Standard Section 16.2.4) the maximum average story drift ratio is identified to assess compliance with the allowable story drift, which Standard Section 16.2.4.3 permits to be increased by 25 percent relative to the drift limit specified in Section 12.12.1 in the context of nonlinear response history analysis. Thus, the relevant story drift limit for nonlinear response history analysis is $(1 + 0.25)(0.010h_{\text{c}}) = 0.0125h_{\text{c}}$. Figure 6.3-11 shows, for each story, the story drift ratios in the longitudinal direction for each time history, the average of all seven the allowable story drift ratio. As before, story drift was examined at the building corners rather than the center of mass.
Table 6.3-7  Maximum Longitudinal Story Drift Ratio at Critical Corner of Building Floorplate

<table>
<thead>
<tr>
<th>Level</th>
<th>Record ID</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td></td>
<td>0.0084</td>
<td>0.0088</td>
<td>0.0076</td>
<td>0.0108</td>
<td>0.0062</td>
<td>0.0060</td>
<td>0.0083</td>
<td>0.0080</td>
</tr>
<tr>
<td>Story 10</td>
<td></td>
<td>0.0064</td>
<td>0.0079</td>
<td>0.0079</td>
<td>0.0108</td>
<td>0.0067</td>
<td>0.0052</td>
<td>0.0062</td>
<td>0.0073</td>
</tr>
<tr>
<td>Story 9</td>
<td></td>
<td>0.0095</td>
<td>0.0164</td>
<td>0.0082</td>
<td>0.0073</td>
<td>0.0077</td>
<td>0.0056</td>
<td>0.0062</td>
<td>0.0087</td>
</tr>
<tr>
<td>Story 8</td>
<td></td>
<td>0.0109</td>
<td>0.0212</td>
<td>0.0108</td>
<td>0.0069</td>
<td>0.0084</td>
<td>0.0063</td>
<td>0.0055</td>
<td>0.0100</td>
</tr>
<tr>
<td>Story 7</td>
<td></td>
<td>0.0110</td>
<td>0.0225</td>
<td>0.0145</td>
<td>0.0090</td>
<td>0.0116</td>
<td>0.0085</td>
<td>0.0045</td>
<td>0.0117a</td>
</tr>
<tr>
<td>Story 6</td>
<td></td>
<td>0.0096</td>
<td>0.0166</td>
<td>0.0110</td>
<td>0.0070</td>
<td>0.0109</td>
<td>0.0072</td>
<td>0.0041</td>
<td>0.0095</td>
</tr>
<tr>
<td>Story 5</td>
<td></td>
<td>0.0084</td>
<td>0.0098</td>
<td>0.0068</td>
<td>0.0049</td>
<td>0.0108</td>
<td>0.0066</td>
<td>0.0042</td>
<td>0.0074</td>
</tr>
<tr>
<td>Story 4</td>
<td></td>
<td>0.0063</td>
<td>0.0062</td>
<td>0.0047</td>
<td>0.0050</td>
<td>0.0131</td>
<td>0.0047</td>
<td>0.0039</td>
<td>0.0063</td>
</tr>
</tbody>
</table>

Figure 6.3-11  Longitudinal story drift ratios (and drift limits)

Table 6.3-7  Maximum Longitudinal Story Drift Ratio at Critical Corner of Building Floorplate

<table>
<thead>
<tr>
<th>Level</th>
<th>Record ID</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td></td>
<td>0.0084</td>
<td>0.0088</td>
<td>0.0076</td>
<td>0.0108</td>
<td>0.0062</td>
<td>0.0060</td>
<td>0.0083</td>
<td>0.0080</td>
</tr>
<tr>
<td>Story 10</td>
<td></td>
<td>0.0064</td>
<td>0.0079</td>
<td>0.0079</td>
<td>0.0108</td>
<td>0.0067</td>
<td>0.0052</td>
<td>0.0062</td>
<td>0.0073</td>
</tr>
<tr>
<td>Story 9</td>
<td></td>
<td>0.0095</td>
<td>0.0164</td>
<td>0.0082</td>
<td>0.0073</td>
<td>0.0077</td>
<td>0.0056</td>
<td>0.0062</td>
<td>0.0087</td>
</tr>
<tr>
<td>Story 8</td>
<td></td>
<td>0.0109</td>
<td>0.0212</td>
<td>0.0108</td>
<td>0.0069</td>
<td>0.0084</td>
<td>0.0063</td>
<td>0.0055</td>
<td>0.0100</td>
</tr>
<tr>
<td>Story 7</td>
<td></td>
<td>0.0110</td>
<td>0.0225</td>
<td>0.0145</td>
<td>0.0090</td>
<td>0.0116</td>
<td>0.0085</td>
<td>0.0045</td>
<td>0.0117a</td>
</tr>
<tr>
<td>Story 6</td>
<td></td>
<td>0.0096</td>
<td>0.0166</td>
<td>0.0110</td>
<td>0.0070</td>
<td>0.0109</td>
<td>0.0072</td>
<td>0.0041</td>
<td>0.0095</td>
</tr>
<tr>
<td>Story 5</td>
<td></td>
<td>0.0084</td>
<td>0.0098</td>
<td>0.0068</td>
<td>0.0049</td>
<td>0.0108</td>
<td>0.0066</td>
<td>0.0042</td>
<td>0.0074</td>
</tr>
<tr>
<td>Story 4</td>
<td></td>
<td>0.0063</td>
<td>0.0062</td>
<td>0.0047</td>
<td>0.0050</td>
<td>0.0131</td>
<td>0.0047</td>
<td>0.0039</td>
<td>0.0063</td>
</tr>
</tbody>
</table>

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Chapter 6: Structural Steel Design

Table 6.3-7  Maximum Longitudinal Story Drift Ratio at Critical Corner of Building Floorplate

<table>
<thead>
<tr>
<th>Level</th>
<th>Record ID</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Story 3</td>
<td>0.0057</td>
<td>0.0054</td>
</tr>
<tr>
<td>Story 2</td>
<td>0.0048</td>
<td>0.0032</td>
</tr>
</tbody>
</table>

Note that the 0.0117 structure story drift ratio is equal to the maximum of the average story drift ratio values across the seven ground motion pairs for each story, not the average of the maximum story drift ratio values for each of the seven ground motion pairs across all ten stories.

The maximum story drift calculated earlier using MRSA reached a maximum value at the roof level equal to 89 percent of the 0.010h_{sx} limit prescribed in Standard Section 12.12.1. The magnitude of this maximum was similar in both principal building axes. The design story drifts obtained from the MRSA were also found to increase from story to story up the height of the structure. Contrast the preliminary elastic MRSA drift results with those just presented from the NRHA. First, the maximum story drift no longer always occurs at the uppermost level of the building; the specific location of the maximum varies depending on the ground motion. Second, the maximum story drift is no longer consistent between the two primary structural axes. This is an important point and is a direct result of the way the ground motion pairs were applied to the structure. While not required by the Standard, some engineers elect to re-analyze the structure with the ground motions rotated, in order to investigate sensitivity to ground-motion orientation; this results in a total of 14 analyses, with a corresponding increase in effort. However, a detailed discussion of this and other ground motion issues is beyond the scope of this design example. Finally perhaps most importantly, the maximum story drift from the NRHA, 0.0117h_{sx}, is 31 percent higher than that from the MRSA, 0.89 \times 0.010h_{sx} = 0.0089h_{sx}.

The results in this section highlight an important misconception of NRHA in general. There is no guarantee of economizing a design with respect to the required strength or stiffness of a frame simply by performing a NRHA. Rather, when executed correctly, a NRHA simply assures a more accurate representation of actual structural performance in a particular seismic event. This increase in the accuracy of seismic response parameters can actually increase the required frame strength or stiffness in some instances.

6.3.6.4.3 Column design forces. All BRBF columns were initially designed in Section 6.3.5.2 to resist the vertical component of the adjusted strengths of any braces above, using capacity design principles and generic values of the brace material overstrength (R_{c}), strain-hardening (\omega) compression overstrength (\beta) parameters. Thus, the lateral columns are expected to remain nominally elastic in the DBE event. More realistic expected BRB behavior specific to a particular BRB product line and supplier is modeled in Section 6.3.6.2 based on experimental test data. The required strengths of the columns in the BRBF as specified in AISC 341 Section 16.5b are subject to revision based on results from the NRHA, as is permitted in the exception to Standard Section 12.4.3.1. To this end, Table 6.3-8 shows the maximum compressive axial force attained at every story in the BRBF column at Gridline D/4 during each of the seven time history analyses. This force represents the summation of the gravity load prescribed in Standard Section 16.2.3 (1.0D + 0.25L) plus the additional force imposed by the relevant earthquake time history pair. As above, compression forces are the most critical for column design because column strengths are governed by compression buckling rather than yielding.
The model included column potential hinges with axial-moment interaction relationships determined from ASCE 41. Inelastic rotations were limited to IO values per that standard. However, virtually no inelastic rotation was recorded in the analyses.

Table 6.3-8 also identifies the overall maximum compression force in the column at every story across all seven ground motion pairs, together with the column design force determined by plastic mechanism analysis in Section 6.3.5.2. While Standard Section 16.2.4 permits the use of average member forces in determining design values with at least seven ground motions, maximum member force values are selected for design of the columns due to their critical role in sustaining the vertical load-carrying capacity of the structure. Even when using maximum values of member forces extracted from the time histories, substantial savings in column design forces and hence steel tonnage are facilitated by NRHA in this example.

### Table 6.3-8 Comparison of Column Design Forces from NRHA and Plastic Analysis for Column at Gridline D/4

<table>
<thead>
<tr>
<th>Level</th>
<th>Maximum Compression Force (kips)</th>
<th>Design Axial Force $Q_e$ (kips)</th>
<th>Percent Reduction in Design Force</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Record ID</td>
<td>NRHAa</td>
<td>Plasticb</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Roof</td>
<td>152</td>
<td>154</td>
<td>152</td>
</tr>
<tr>
<td>Story 10</td>
<td>214</td>
<td>216</td>
<td>216</td>
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<tr>
<td>Story 9</td>
<td>404</td>
<td>412</td>
<td>393</td>
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<td>Story 8</td>
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<td>616</td>
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<tr>
<td>Story 5</td>
<td>1198</td>
<td>1320</td>
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<tr>
<td>Story 4</td>
<td>1414</td>
<td>1566</td>
<td>1504</td>
</tr>
<tr>
<td>Story 3</td>
<td>1750</td>
<td>1851</td>
<td>1780</td>
</tr>
<tr>
<td>Story 2</td>
<td>1861</td>
<td>1965</td>
<td>1894</td>
</tr>
</tbody>
</table>

1. Maximum of individual maxima from the seven time histories.
2. As determined in Table 6.3-5.
3. Relative to that determined using plastic analysis.
4. 1.0 kip = 4.45 kN 1.0 in = 25.4 mm

In addition to these significant reductions in column design forces, the same procedure reveals a design tension uplift force at this same column of 336 kips, which is just 28 percent of the comparable 1,222-kip force obtained from plastic analysis. This force reduction enables tremendous savings in the design of foundation elements such as base plates, anchor rods drilled piers.

With the possible exception of the foundation elements just mentioned, despite the fact that the column design forces have been sharply reduced relative to those obtained from the plastic mechanism analysis in Section 6.3.5.2, none of the column sizes are actually reduced in this example. This is because the structure is currently very close to the allowable story drift limit. In other words, the NRHA reveals that stiffness, not strength, governs the design of the BRBF examined here. A prudent designer might consider enlarging select braces in conjunction with reducing column sizes as allowed by the NRHA.
design forces in Table 6.3-8 to increase cost savings on the project, depending on the relative prices of BRBs and structural steel. However, the tradeoff between BRB and column stiffness would have to preserve the overall stiffness of the frame in order to ensure the allowable drift limit is still satisfied. In particular, it may be possible to increase the brace size in Story 7 (and possibly Stories 6 and 8) while reducing the column size considerably.

6.3.6.4.4 Brace-to-gusset connection design. As mentioned earlier, brace-to-gusset connections for BRBs take on three different forms: bolted, welded pinned. The specific nature of this connection is not considered in this design example. However, the possible additional benefit of economizing on material by using NRHA to determine connection design forces will be demonstrated. As shown in Section 6.3.5.5, the connection of the single diagonal brace below the roof to its gusset would need to be designed for 325 kips in tension and compression. Using the maximum BRB force results from NRHA (obtained in the same manner that column design forces were determined), this design value can be reduced to 210 kips in tension and 213 kips in compression, representing a reduction of approximately 35 percent in both cases. This corresponds to utilizing a lower strain-hardening factor, $\omega$, corresponding to a more refined method of establishing deformation demands. It should be noted that this reduction is not explicitly allowed by AISC 341; therefore, it would constitute “alternate means and methods” and be subject to approval by the building official.

6.3.6.4.5 Summary of NRHA goals. Before concluding, the exact extent to which NRHA was used in this design example merits emphasis one last time. Based on the fundamental period of the structure, the minimum level of sophistication required for its seismic lateral analysis is an elastic MRSA. Thus, in keeping with code requirements, a three-dimensional model of the structure was created the BRBs were designed to accommodate force demands determined by MRSA. BRBF beams, columns collectors were then designed using a rational plastic mechanism analysis with the assumption that any earthquake load effect is determined from the full adjusted brace strengths in tension and compression. This example then goes beyond elastic analysis and relies on a NRHA for three additional aspects of the design. First, NRHA is used to verify that the strains in the BRBs designed to MRSA forces indeed satisfy the IO performance requirements in DBE-level shaking using the methodology in ASCE 41 Section 2.8.3. Second, compliance with the allowable story drift limit was evaluated in the context of NRHA and the provisions of Standard Section 16.2.4. Finally, as a demonstration, NRHA was used to establish reduced design axial forces for the BRBF columns using the exception to Standard Section 12.4.3.1 to justify the use of NRHA in determining their required strength as stipulated in AISC 341 Section 16.5b. However, because the structure was found to be at the allowable drift limit as designed using plastic mechanism analysis, the potential savings offered by these reduced column design forces went unrealized.

Designers could elect to place an even greater emphasis on NRHA during the design of such a structure. For example, design forces for BRBF beams and collectors might also be set using NRHA, as was illustrated for the columns, rather than by plastic mechanism analysis. Depending on the relative prices of BRBs and structural steel, designers could economize the design by enlarging select BRB elements (relative to what MRSA finds is necessary) in order to reduce some column sizes. Justifying such a design would require iterative NRHA runs to ensure the structure’s overall stiffness is such that allowable drift limits are not exceeded. Finally, the entire structure might be designed using NRHA exclusively. BRB sizes would be established not by MRSA but instead by NRHA and hence a different force distribution that takes into account the building’s inelastic characteristics.