5

STRUCTURAL STEEL DESIGN

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This chapter illustrates how the 2000 *NEHRP Recommended Provisions* (hereafter the *Provisions*) is applied to the design of steel framed buildings. The three examples include:

1. An industrial warehouse structure in Astoria, Oregon;
2. A multistory office building in Los Angeles, California; and
3. A low-rise hospital facility in the San Francisco Bay area of California.

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

1. Concentrically braced frames,
2. Intermediate moment frames,
3. Special moment frames,
4. A dual system consisting of moment frames and concentrically braced frames, and
5. Eccentrically braced frames.

For determining the strength of steel members and connections, the 1993 [1999] *Load and Resistance Factor Design Specification for Structural Steel Buildings,* published by the American Institute of Steel Construction, is used throughout. In addition, the requirements of the 1997 [2002] *AISC Seismic Provisions for Structural Steel Buildings* are followed where applicable.

The examples only cover design for seismic forces in combination with gravity, and 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, and detailing.

All structures are analyzed using three-dimensional static or dynamic methods. The SAP2000 Building Analysis Program (Computers & Structures, Inc., Berkeley, California, v.6.11, 1997) is used in Example 5.1, and the RAMFRAME Analysis Program (RAM International, Carlsbad, California, v. 5.04, 1997) is used in Examples 5.2 and 5.3.

In addition to the 2000 *NEHRP Recommended Provisions*, the following documents are referenced:

AISC LRFD — American Institute of Steel Construction. 1999. *Load and Resistance Factor Design Specification for Structural Steel Buildings*.

AISC Manual	American Institute of Steel Construction. 2001. *Manual of Steel Construction, Load and Resistance Factor Design*, 3rd Edition.
AISC Seismic	American Institute of Steel Construction. 2000. [2002] *Seismic Provisions for Structural Steel Buildings*, 1997, including Supplement No. 2.
IBC	International Code Council, Inc. 2000. *2000 International Building Code*.
FEMA 350	SAC Joint Venture. 2000. *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*.
AISC SDGS-4	AISC Steel Design Guide Series 4. 1990. *Extended End-Plate Moment Connections*, 1990.
SDI	Luttrell, Larry D. 1981. S*teel Deck Institute Diaphragm Design Manual*. Steel Deck Institute.

The symbols used in this chapter are from Chapter 2 of the *Provisions*, the above referenced documents, or are as defined in the text. Customary U.S. units are used.

Although the these design examples are based on the 2000 *Provisions*, it is annotated to reflect changes made to the 2003 *Provisions*. Annotations within brackets, [ ], indicate both organizational changes (as a result of a reformat of all of the chapters of the 2003 *Provisions*) and substantive technical changes to the 2003 *Provisions* and its primary reference documents. While the general concepts of the changes are described, the design examples and calculations have not been revised to reflect the changes to the 2003 *Provisions*.

The most significant change to the steel chapter in the 2003 *Provisions* is the addition of two new lateral systems, buckling restrained braced frames and steel plate shear walls, neither of which are covered in this set of design examples. Other changes are generally related to maintaining compatibility between the *Provisions* and the 2002 edition of AISC Seismic. Updates to the reference documents, in particular AISC Seismic, have some effects on the calculations illustrated herein.

Some general technical changes in the 2003 *Provisions* that relate to the calculations and/or design in this chapter include updated seismic hazard maps, changes to Seismic Design Category classification for short period structures and revisions to the redundancy requirements, new Simplified Design Procedure would not be applicable to the examples in this chapter.

Where they affect the design examples in this chapter, other significant changes to the 2003 *Provisions* and primary reference documents are noted. However, some minor changes to the 2003 *Provisions* and the reference documents may not be noted.

It is worth noting that the 2002 edition of AISC Seismic has incorporated many of the design provisions for steel moment frames contained in FEMA 350. The design provisions incorporated into AISC Seismic are similar in substance to FEMA 350, but the organization and format are significantly different. Therefore, due to the difficulty in cross-referencing, the references to FEMA 350 sections, tables, and equations in this chapter have not been annotated. The design professional is encouraged to review AISC Seismic for updated moment frame design provisions related to the examples in this chapter.

5.1 INDUSTRIAL HIGH-CLEARANCE BUILDING, ASTORIA, OREGON

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

1. Seismic design parameters,
2. Equivalent lateral force analysis,
3. Three-dimension (3-D) modal analysis,
4. Drift check,
5. Check of compactness and brace spacing for moment frame,
6. Moment frame connection design, and
7. Proportioning of concentric diagonal bracing.

5.1.1 Building Description

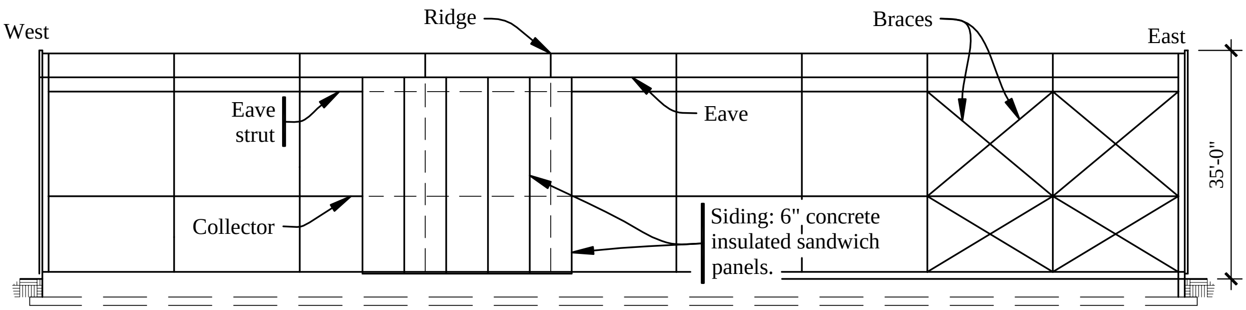
This industrial building has plan dimensions of 180 ft by 90 ft and a clear height of approximately 30 ft. It includes a 12-ft-high, 40-ft-wide mezzanine area at the east end of the building. The structure consists of 10 gable frames spanning 90 ft in the transverse (N-S) direction. Spaced at 20 ft o.c., these frames are braced in the longitudinal (EW) 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 5.1-1. Longitudinal struts at the eaves and 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.

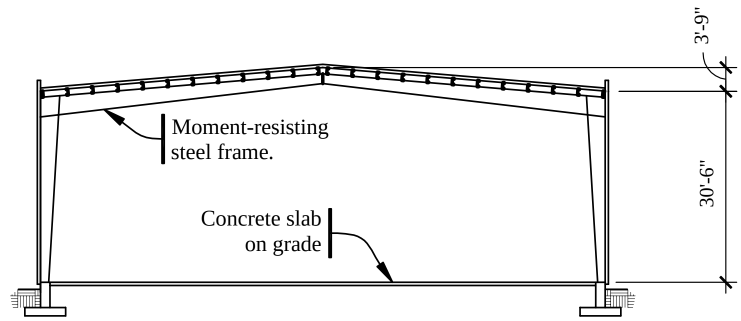
The roof and mezzanine framing plans are shown in Figure 5.1-2. The framing consists of a steel roof deck supported by joists between transverse gable frames. Because the frames resist lateral loading at each frame position, the steel deck functions as a diaphragm for distribution of the effects of eccentric 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 10 ft o.c. The floor beams are supported on girders continuous over two intermediate columns spaced approximately 30 ft 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 in. and lateral sway due to wind was limited to 2 in. (which did not control). These serviceability limits are not considered to control any aspect of the seismic-resistant design. However, many aspects of seismic design are driven by actual capacities so, in that sense, the serviceability limits do affect the seismic design.



(a)



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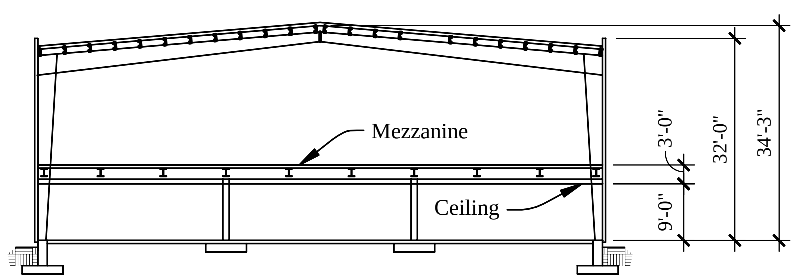


(c)

Figure 5.1-1 Framing elevation and sections (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

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 avoid shear resistance by the panels. (This building arrangement has been intentionally contrived to illustrate what can happen to a tapered-moment frame building if high seismic demands are placed on it. More likely, if this were a real building, the concrete panels would be connected directly to the steel frame, and the building would tend to act as a shear wall building. But for this example, the connections have been arranged to permit the steel frame to move at the point of attachment in the in-plane direction of the concrete panels. This was done to cause the steel frame to resist lateral forces and, thus, shear-wall action of the panels does not influence the frames.)

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 5.1-3. 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.

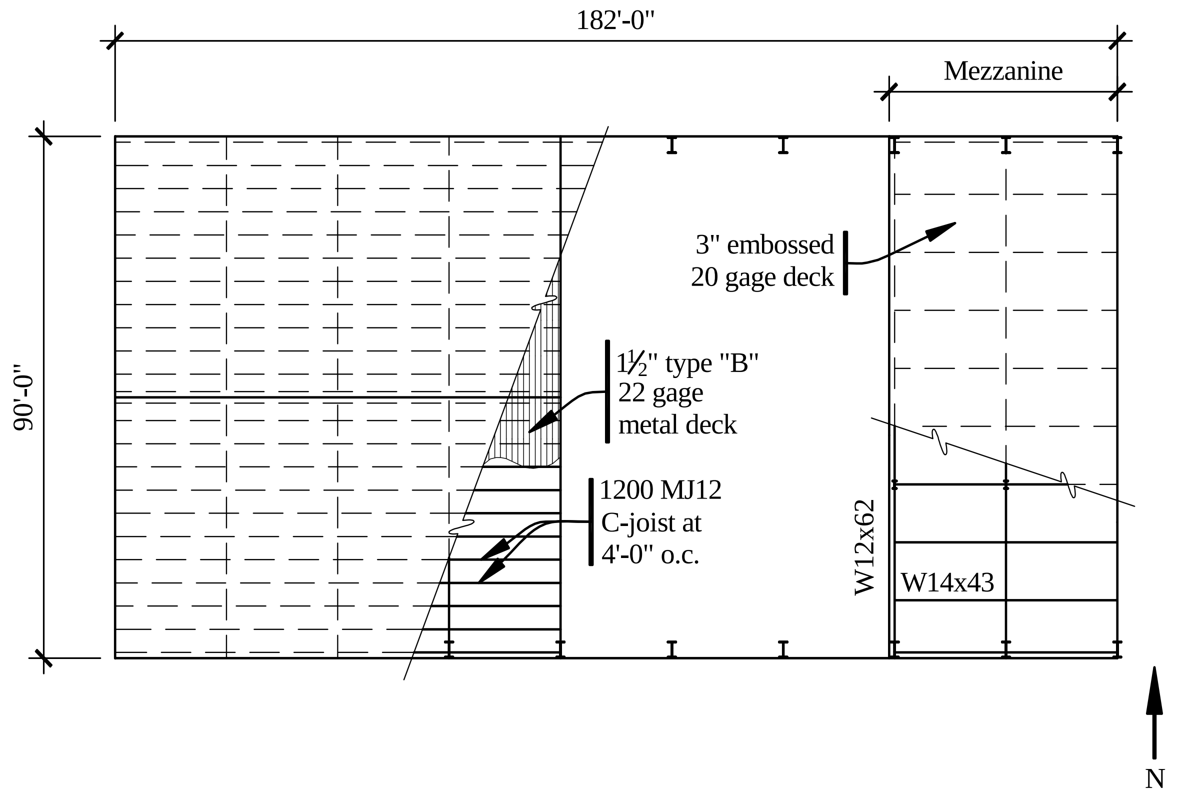


Figure 5.1-2 Roof framing and mezzanine framing plan (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

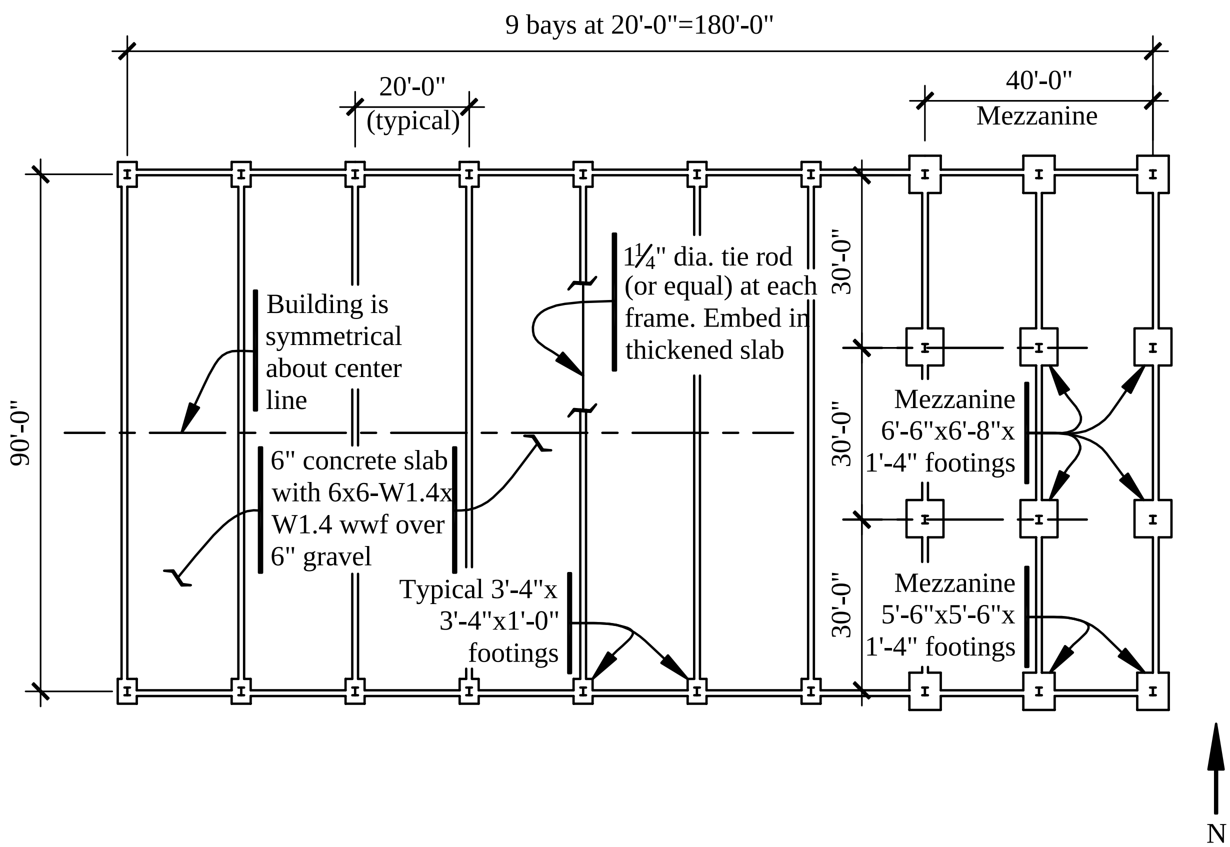


Figure 5.1-3 Foundation plan (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

5.1.2 Design Parameters

5.1.2.1 *Provisions* Parameters

Site Class	= D (*Provisions* Sec. 4.1.2.1[3.5])
S_S	= 1.5 (*Provisions* Map 9 [Figure 3.3-1])
S_1	= 0.6 (*Provisions* Map 10 [Figure 3.3-2])
F_a	= 1.0 (*Provisions* Table 4.1.2.4a [3.3-1])
F_v	= 1.5 (*Provisions* Table 4.1.2.4b [3.3-2])
$S_{MS} = F_a S_S$	= 1.5 (*Provisions* Eq. 4.1.2.4-1 [3.3-1])
$S_{M1} = F_v S_1$	= 0.9 (*Provisions* Eq. 4.1.2.4-2 [3.3-2])
$S_{DS} = 2/3\ S_{MS}$	= 1.0 (*Provisions* Eq. 4.1.2.5-1 [3.3-3])
$S_{D1} = 2/3\ S_{M1}$	= 0.6 (*Provisions* Eq. 4.1.2.5-2 [3.3-4])
Seismic Use Group	= I (*Provisions* Sec. 1.3 [1.2])
Seismic Design Category	= D (*Provisions* Sec. 4.2.1 [1.4])

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package).]

Note that *Provisions* Table 5.2.2 [4.3-1] permits an ordinary moment-resisting steel frame for buildings that do not exceed one story and 60 feet tall with a roof dead load not exceeding 15 psf. This building would fall within that restriction, but the intermediate steel moment frame with stiffened bolted end plates is chosen to illustrate the connection design issues.

[The height and tributary weight limitations for ordinary moment-resisting frames have been revised in the 2003 *Provisions*. In Seismic Design Category D, these frames are permitted only in single-story structures up to 65 feet in height, with field-bolted end plate moment connections, and roof dead load not exceeding 20 psf. Refer to 2003 *Provisions* Table 4.3-1, footnote h. The building in this example seems to fit these criteria, but the presence of the mezzanine could be questionable. Similarly, the limitations on intermediate moment-resisting frames in Seismic Design Category D have been revised. The same single-story height and weight limits apply, but the type of connection is not limited.]

N-S direction:

Moment-resisting frame system	= intermediate steel moment frame
R	= 4.5 (*Provisions* Table 5.2.2 [4.3-1])
Ω_0	= 3 (*Provisions* Table 5.2.2 [4.3-1])
C_d	= 4 (*Provisions* Table 5.2.2 [4.3-1])

E-W direction:

Braced frame system	= ordinary steel concentrically braced frame (*Provisions* Table 5.2.2 [4.3-1])
R	= 5 (*Provisions* Table 5.2.2 [4.3-1])[1]
Ω_0	= 2 (*Provisions* Table 5.2.2 [4.3-1])
C_d	= 4.5 (*Provisions* Table 5.2.2 [4.3-1])

[1] R must be taken as 4.5 in this direction, due to *Provisions* Sec. 5.2.2.2.1 [4.3.1.2], which states that if the value of R in either direction is less than 5, the smaller value of R must be used in both directions. If the ordinary steel moment frame were chosen for the N-S direction, this R factor would change to 3.5.

5.1.2.2 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, and 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 *Provisions* Sec. 5.3 [5.2.1].) Therefore, the mezzanine seismic weight is 69 + 0.25(125) = 100 psf.

5.1.2.3 Materials

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

5.1.3 Structural Design Criteria

5.1.3.1 Building Configuration

Because there is a mezzanine at one end, the building might be considered vertically irregular (*Provisions* Sec. 5.2.3.3 [4.3.2.3]). However, the upper level is a roof, and the *Provisions* exempts roofs from weight irregularities. There also are plan irregularities in this building in the transverse direction, again because of the mezzanine (*Provisions* Sec. 5.2.3.2 [4.3.2.2]).

5.1.3.2 Redundancy

For a structure in Seismic Design Category D, *Provisions* Eq. 5.2.4.2 [not applicable in the 2003 *Provisions*] defines the reliability factor (ρ) as:

$$\rho = 2 - \frac{20}{r_{max_x}\sqrt{A_x}}$$

where the roof area (A_x) = 16,200 sq ft.

To checking ρ in an approximate manner. In the N-S (transverse) direction, there are (2 adjacent columns)/(2 x 9 bays) so:

$$r_{max_x} = 0.11 \text{ and } \rho = 0.57 < 1.00 \,.$$

Therefore, use ρ = 1.00.

In the E-W (longitudinal) direction, the braces are equally loaded (ignoring accidental torsion), so there is (1 brace)/(4 braces) so

$$r_{max_x} = 0.25 \text{ and } \rho = 1.37 \text{ .}$$

Thus, the reliability multiplier is 1.00 in the transverse direction and 1.37 in the longitudinal direction. The reliability factor applies only to the determination of forces, not to deflection calculations.

[The redundancy requirements have been substantially changed in the 2003 *Provisions*. For a building assigned to Seismic Design Category D, ρ = 1.0 as long as it can be shown that failure of beam-to-column connections at both ends of a single beam (moment frame system) or failure of an individual brace (braced frame system) would not result in more than a 33 percent reduction in story strength or create an extreme torsional irregularity. Therefore, the redundancy factor would have to be investigated in both directions based on the new criteria in the 2003 *Provisions*.]

5.1.3.3 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 structures in Seismic Design Category D (*Provisions* Sec. 5.2.5.2.3 and 5.2.5.2.2 [4.4.2.3 and 4.4.2.2, respectively]).

5.1.3.4 Structural Component Load Effects

The effect of seismic load (*Provisions* Eq. 5.2.7-1 and 5.2.7-2 [4.2-1 and 4.2-2, respectively]) is:

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

Recall that S_{DS} = 1.0 for this example. The seismic load is combined with the gravity loads as follows:

$$1.2D + 1.0L + 0.2S + E \;=\; 1.2D + 1.0L + \rho Q_E + 0.2\mathrm{D} \;=\; 1.4\mathrm{D} + 1.0\mathrm{L} + 0.2\mathrm{S} + \rho Q_E.$$

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

$$0.9D + E = 0.9D + \rho Q_E \;-0.2D = 0.7D + \rho Q_E.$$

5.1.3.5 Drift Limits

For a building in Seismic Use Group I, the allowable story drift (*Provisions* 5.2.8 [4.5-1]) is:

$$\Delta_a = 0.025\, h_{sx}.$$

At the roof ridge, h_{sx} = 34 ft-3 in. and Δ_α = 10.28 in.

At the hip (column-roof intersection), h_{sx} = 30 ft-6 in. and Δ_a = 9.15 in.

At the mezzanine floor, h_{sx} = 12 ft and Δ_a = 3.60 in.

Footnote b in *Provisions* Table 5.2.8 [4.5-1, footnote c] permits unlimited drift for single-story buildings with interior walls, partitions, etc., that have been designed to accommodate the story drifts. See Sec. 5.1.4.3 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 the footprint of the building.)

5.1.3.6 Seismic Weight

The weights that contribute to seismic forces are:

	E-W direction	N-S direction
Roof D and L = (0.015)(90)(180) =	243 kips	243 kips
Panels at sides = (2)(0.075)(32)(180)/2 =	0 kips	437 kips
Panels at ends = (2)(0.075)(35)(90)/2 =	224 kips	0 kips
Mezzanine slab = (0.100)(90)(40) =	360 kips	360 kips
Mezzanine framing =	35 kips	35 kips
Main frames =	27 kips	27 kips
Seismic weight =	889 kips	1,102 kips

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 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.

5.1.4 Analysis

Base shear will be determined using an equivalent lateral force (ELF) analysis; a modal analysis then will examine the torsional irregularity of the building. The base shear as computed by the ELF analysis will be needed later when evaluating the base shear as computed by the modal analysis (see *Provisions* Sec. 5.5.7 [5.3.7]).

5.1.4.1 Equivalent Lateral Force Procedure

In the longitudinal direction where stiffness is provided only by the diagonal bracing, the approximate period is computed using *Provisions* Eq. 5.4.2.1-1 [5.2-6]:

$$T_a = C_r h_n^x = (0.02)(34.25^{0.75}) = 0.28 \text{ sec}$$

In accordance with *Provisions* Sec. 5.4.2 [5.2.2], the computed period of the structure must not exceed:

$$T_{max} = C_u T_a = (1.4)(0.28) = 0.39 \text{ sec.}$$

The subsequent 3-D modal analysis finds the computed period to be 0.54 seconds.

In the transverse direction where stiffness is provided by moment-resisting frames (*Provisions* Eq. 5.4.2.1-1 [5.2-6]):

$$T_a = C_r h_n^x = (0.028)(34.25^{0.8}) = 0.47 \text{ sec}$$

and

$$T_{max} = C_u T_a = (1.4)(0.47) = 0.66 \text{ sec.}$$

Also note that the dynamic analysis found a computed period of 1.03 seconds.

The seismic response coefficient (C_s) is computed in accordance with *Provisions* Sec. 5.4.1.1 [5.2.1.1]. In the longitudinal direction:

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

but need not exceed

$$C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.6}{(0.39)(4.5/1)} = 0.342$$

Therefore, use C_s = 0.222 for the longitudinal direction.

In the transverse direction (*Provisions* Eq. 5.4.1.1-1 and 5.4.1.1-2 [5.2-2 and 5.2-3, respectively]):

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

but need not exceed

$$C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.6}{(0.66)(4.5/1)} = 0.202$$

Therefore, use C_s = 0.202 for the transverse direction.

In both directions the value of C_s exceeds the minimum value (*Provisions* Eq. 5.4.1.1-3 [not applicable in the 2003 *Provisions*]) computed as:

$C_s = 0.044I\,S_{DS} = (0.044)(1)(1.0) = 0.044$

[This minimum C_s value has been removed in the 2003 *Provisions*. In its place is a minimum C_s value for long-period structures, which is not applicable to this example.]

The seismic base shear in the longitudinal direction (*Provisions* Eq. 5.4.1 [5.2-1]) is:

$V = C_sW$ = (0.222)889 kips) = 197 kips.

The seismic base shear in the transverse direction is:

$V = C_sW$ = (0.202)(1,102 kips) = 223 kips.

The seismic force must be increased by the reliability factor as indicated previously. Although this is not applicable to the determination of deflections, it is applicable in the determination of required strengths. The reliability multiplier (ρ) will enter the calculation later as the modal analysis is developed. If the ELF method was used exclusively, the seismic base shear in the longitudinal direction would be increased by ρ now:

$V = \rho\,(197)$
V = (1.37)(197) = 270 kips

[See Sec. 5.1.3.2 for discussion of the changes to the redundancy requirements in the 2003 *Provisions*.]

Provisions Sec. 5.4.3 [5.2.3] 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 Sec. 5.1.3.6 of this example and interpolating the exponent k as 1.08 for the period of 0.66 sec, the distribution of forces for the N-S analysis is shown in Table 5.1-1.

Table 5.1-1 ELF Vertical Distribution for N-S Analysis

Level	Weight (w_x)	Height (h_x)	$w_x h_x^k$	C_{vx}	F_x
Roof	707 kips	30.5 ft.	28340	0.83	185 kips
Mezzanine	395 kips	12 ft.	5780	0.17	38 kips
Total	1102 kips		34120		223 kips

It is not immediately clear as to whether the roof (a 22-gauge steel deck with conventional roofing over it) will behave as a flexible or rigid diaphragm. If one were to assume that the roof were a flexible diaphragm while the mezzanine were rigid, the following forces would be applied to the frames:

Typical frame at roof (tributary basis) = 185 kips / 9 bays = 20.6 kips
End frame at roof = 20.6/2 = 10.3 kips
Mezzanine frame at mezzanine = 38 kips/3 frames = 12.7 kips

If one were to assume the roof were rigid, it would be necessary to compute the stiffness for each of the two types of frames and for the braced frames. For this example, a 3-D model was created in SAP 2000.

5.1.4.2 Three-Dimension Static and Modal Response Spectrum Analyses

The 3-D analysis is performed for this example to account for:

1. The significance of differing stiffness of the gable frames with and without the mezzanine level,
2. The significance of the different centers of mass for the roof and the mezzanine,
3. The relative stiffness of the roof deck with respect to the gable frames, and
4. 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 3-D beam-column elements. The collector at the hip 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 3-D model in the transverse direction yields two important results: the roof diaphragm behaves as a rigid diaphragm and the displacements result in the building being classified as torsionally irregular. The forces at the roof are distributed to each frame line in a fashion that offsets the center of force 5 percent of 180 ft (9 ft) to the west of the center of the roof. The forces at the mezzanine are similarly distributed to offset the center of the mezzanine force 5 percent of 40 ft to the west of the center of the mezzanine. Using grid locations numbered from west to east, the applied forces and the resulting displacements are shown in Table 5.1-2.

Table 5.1-2 ELF Analysis in N-S Direction

Grid	Roof Force, kips	Mezzanine Force, kips	Roof Displacement, in.
1	13.19		4.56
2	25.35		4.45
3	23.98		4.29
4	22.61		4.08
5	21.24		3.82
6	19.87		3.53
7	18.50		3.21
8	17.13	14.57	2.86
9	15.76	12.67	2.60
10	7.36	10.77	2.42
Totals	184.99	38.01	

The average of the extreme displacements is 3.49 in. The displacement at the centroid of the roof is 3.67 in. Thus, the deviation of the diaphragm from a straight line is 0.18 in. whereas the average frame displacement is about 20 times that. Clearly then, the behavior is as a rigid diaphragm. The ratio of maximum to average displacement is 1.31, which exceeds the 1.2 limit given in *Provisions* Table 5.2.3.2 [4.3-2] and places the structure in the category "torsionally irregular." *Provisions* Table 5.2.5.1 [4.4-1] then requires that the seismic force analysis be any one of several types of dynamic analysis. The simplest of these is the modal response spectrum (MRS) analysis.

The MRS is an easy next step once the 3-D model has been assembled. A 3-D dynamic design response spectrum analysis is performed per *Provisions* Sec. 5.5 [5.3] using the SAP 2000 program. The design response spectrum is based on *Provisions* Sec. 4.1.2.6 [3.3.4] and is shown in Figure 5.1-4. [Although it has no affect on this example, the design response spectrum has been changed for long periods in the 2003 *Provisions*. See the discussion in Chapter 3 of this volume of design examples.]

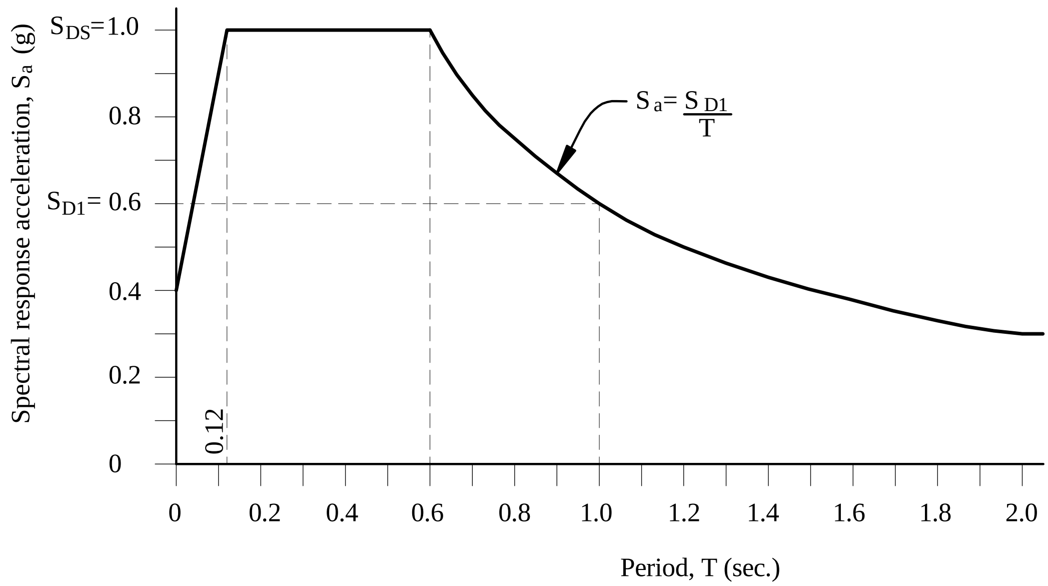


Figure 5.1-4 Design response spectrum.

The modal seismic response coefficient (*Provisions* Eq. 5.5.4-3 [5.3-3]) is $C_{sm} = \dfrac{S_{am}}{R/I}$. The design response spectra expressed in units of g and ft/sec^2 are shown in Table 5.1-3.

Table 5.1-3 Design Response Spectra

T (sec)	S_a (g) $S_{am} = S_a$ (g)	$C_{sm} = \dfrac{S_{am}}{(R/I)}$ $R = 4.5$	C_{sm} (ft/sec^2)
0.0	0.4	0.089	2.862
0.12	0.9	0.222	7.155
0.6	1.0	0.222	7.155
0.7	0.857	0.190	6.132
0.8	0.750	0.167	5.367
0.9	0.666	0.148	4.766
1.0	0.600	0.133	4.293
1.1	0.545	0.121	3.900
1.2	0.500	0.111	3.578
1.3	0.461	0.102	3.299
1.4	0.429	0.095	3.070

1.0 ft = 0.3048 m.

With this model, the first 24 periods of vibration and mode shapes of the structure were computed using the SAP2000 program. The first mode had a period of vibration of 1.03 seconds with predominantly transverse participation. The third mode period was 0.54 seconds with a predominantly longitudinal participation. The first 24 modes accounted for approximately 98 percent of the total mass of the structure in the transverse direction and approximately 93 percent in the longitudinal direction, both of which are is greater than the 90 percent requirement of *Provisions* Sec. 5.5.2 [5.3.2].

The design value for modal base shear (V_t) is determined by combining the modal values for base shear. The SAP 2000 program uses the complete quadratic combination (CQC) of the modal values, which accounts for coupling of closely spaced modes. In the absence of damping, the CQC is simply the square root of the sum of the squares (SRSS) of each modal value. Base shears thus obtained are:

Longitudinal V_t = 159.5 kips
Transverse V_t = 137.2 kips

In accordance with *Provisions* Sec. 5.5.7 [5.3.7], compare the design values of modal base shear 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_uT_a, a factor to bring the modal base shear up to this comparison ELF value must be applied to the modal story shears, moments, drifts, and floor deflections. According to *Provisions* Eq. 5.5.7.1 [5.3-10]:

Modification factor = 0.85 (V/V_t)

E-W modification factor = 0.85(V/V_t) = (0.85)(197 kips/159.5 kips) = 1.05
N-S modification factor = 0.85(V/V_t) = (0.85)(223 kips/137.2 kips) = 1.38

The response spectra for the 3-D modal analysis is then revised by the above modification factors:

E-W (1.0)(1.05)(x-direction spectrum)
N-S (1.0)(1.38)(y-direction spectrum)

The model is then run again.

The maximum lateral displacements at the ridge due to seismic loads (i.e., design response spectra as increased by the modification factors above) from the second analysis are:

E-W deflection δ_{xe} = 0.84 in.
N-S deflection δ_{ye} = 2.99 in. at the first frame in from the west end

where δ_{xe} and δ_{ye} are deflections determined by the elastic modal analysis. Those frames closer to the mezzanine had smaller N-S lateral deflections in much the same fashion as was shown for the ELF analysis. Before going further, the deflections should be checked as discussed in Sec. 5.1.4.3 below.

The response spectra for the 3-D modal analysis are combined to meet the orthogonality requirement of *Provisions* Sec. 5.2.5.2.2a [4.4.2.3]:

E-W (1.0)(E-W direction spectrum) + (0.3)(N-S direction spectrum)
N-S (0.3)(E-W direction spectrum) + (1.0)(N-S direction spectrum)

Finally, the design response spectra for the 3-D modal analysis is again revised by increasing the E-W direction response by the reliability factor, ρ = 1.37. Note that ρ is equal to unity in the N-S direction. Thus, the factors on the basic spectrum for the load combinations become:

E-W (1.0)(1.05)(1.37)(E-W direction spectrum) + (0.3)(1.38)(1.00)(N-S direction spectrum)
N-S (0.3)(1.05)(1.37)(E-W direction spectrum) + (1.0)(1.38)(1.00)(N-S direction spectrum)

and the model is run once again to obtain the final result for design forces, shears, and moments. From this third analysis, the final design base shears are obtained. Applying the ρ factor (1.37) is equivalent to increasing the E-W base shear from (0.85 x 197 kips) = 167.5 kips to 230 kips.

5.1.4.3 Drift

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

$$\delta_x = \frac{C_d \delta_{xe}}{I} = \frac{4.0(2.99)}{1.0} = 12.0 \text{ in.}$$

This exceeds the limit of 10.28 in. 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. (The heavy wall panels were selected to make an interesting example problem, and the high transverse drift is a consequence of this. Some real buildings, such as refrigerated warehouses, have heavy wall panels and would be expected to have high seismic drifts. Special attention to detailing the connections of such features is necessary.)

In the longitudinal direction, the lateral deflection was much smaller and obviously is within the limits. Recall that the deflection computations do not consider the reliability factor. This value must be multiplied by a C_d factor to find the transverse drift. The tabulated value of C_d is 4.5, but this is for use when design is based upon R = 5. The *Provisions* does not give guidance for C_d when the system R factor is overridden by the limitation of *Provisions* Sec. 5.2.2.1 [4.3.1.2]. The authors suggest adjusting by a ratio of R factors.

5.1.4.4 P-delta

The AISC LRFD Specification requires P-delta analyses for frames. This was investigated by a 3-D P-delta analysis, which determined that secondary P-delta effect on the frame in the transverse direction was less than 1 percent of the primary demand. As such, for this example, P-delta was considered to be insignificant and was not investigated further. (P-delta may be significant for a different structure, say one with higher mass at the roof. P-delta should always be investigated for unbraced frames.)

5.1.4.5 Force Summary

The maximum moments and axial forces caused by dead, live, and earthquake loads on the gable frames are listed in Tables 5.1-2 and 5.1-3. 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 5.1-4 and the axial forces are given in Table 5.1-5. The moment diagram for the combined load condition is shown in Figure 5.1-5. 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 and design of the braces are discussed in Sec. 5.1.5.5 of this chapter.

Table 5.1-4 Moments in Gable Frame Members

Location	*D* (ft-kips)	*L* (ft-kips)	*S* (ft-kips)	Q_E (ft-kips)	Combined***** (ft-kips)
1- Ridge	61	0	128	0	112 (279)
2- Knee	161	0	333	162	447 (726)
3- Mezzanine	95	83	92	137	79
4- Base	0	0	0	0	0

* Combined Load = $1.4D + L + 0.2S + \rho Q_E$ (or $1.2D + 1.6S$)**.** Individual maximums 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 5.1-5 Axial Forces in Gable Frames Members

Location	*D* (ft-kips)	*L* (ft-kips)	*S* (ft-kips)	ρQ_E (ft-kips)	Combined***** (ft-kips)
1- Ridge	14	3.5	25	0.8	39
2- Knee	16	4.5	27	7.0	37
3- Mezzanine	39	39	23	26	127
4- Base	39	39	23	26	127

* Combined Load = $1.4D + L + 0.2S + \rho Q_E$. Individual maximums 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.

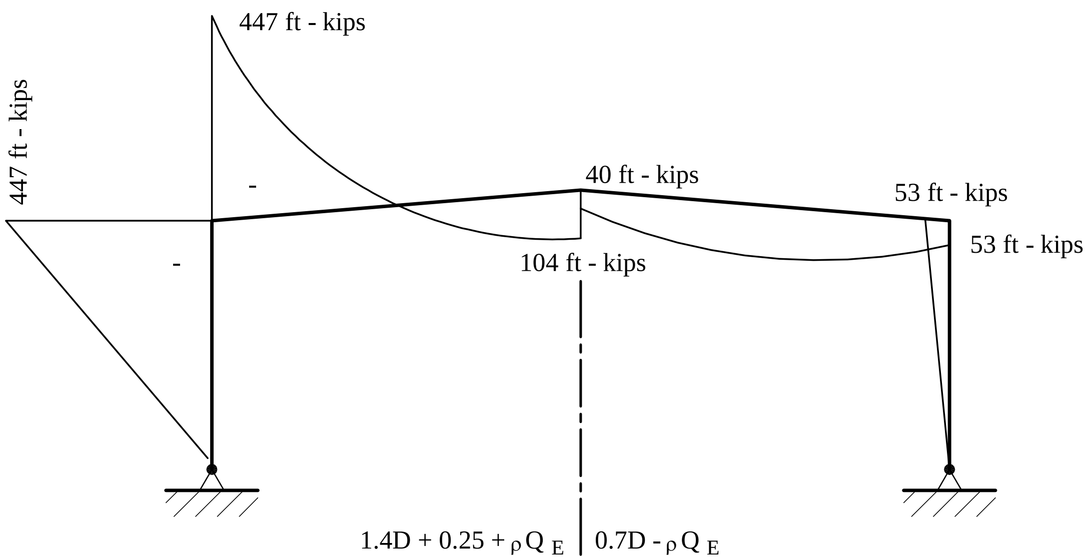


Figure 5.1-5 Moment diagram for seismic load combinations (1.0 ft-kip = 1.36 kN-m).

5.1.5 Proportioning and Details

The gable frame is shown schematically in Figure 5.1-6. Using load combinations presented in Sec. 5.1.3.4 and the loads from Tables 5.1-2 and 5.1-3, 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 5.1-1, was proportioned similarly. The diagonal bracing, shown in Figure 5.1-1 at the east end of the building, is proportioned using tension forces determined from the 3-D modal analysis.

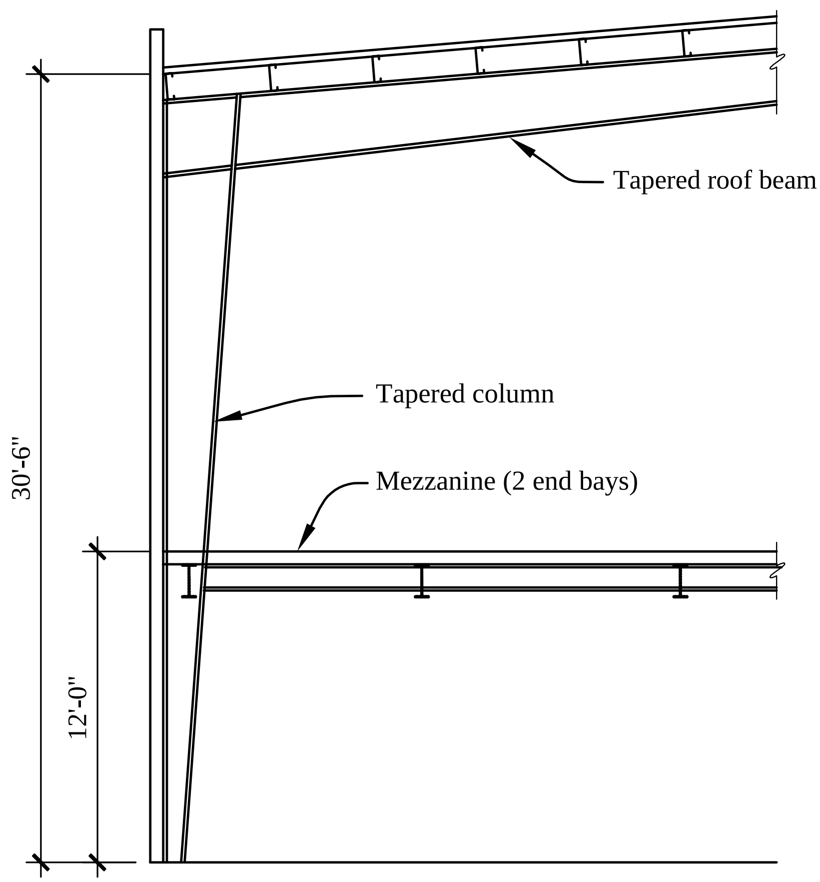


Figure 5.1-6 Gable frame schematic: Column tapers from 12 in. at base to 36 in. at knee; roof beam tapers from 36 in. at knee to 18 in. at ridge; plate sizes are given in Figure 5.1-8 (1.0 in. = 25.4+ mm).

5.1.5.1 Frame Compactness and Brace Spacing

According to *Provisions* Sec. 8.4 [8.2.2], steel structures assigned to Seismic Design Categories D, E, and F must be designed and detailed (with a few exceptions) per AISC Seismic. For an intermediate moment frame (IMF), AISC Seismic Part I, Section 1, "Scope," stipulates that those requirements are to be applied in conjunction with AISC LRFD. Part I, Section 10 of AISC Seismic itemizes a few exceptions from AISC LRFD for intermediate moment frames, but otherwise the intermediate moment frames are to be designed per the AISC LRFD Specification.

Terminology for moment-resisting frames varies among the several standards; Table 5.1-6 is intended to assist the reader in keeping track of the terminology.

Table 5.1-6 Comparison of Standards

Total Rotation (story drift angle)	Plastic Rotation	AISC Seismic (1997)	FEMA 350	AISC Seismic (Supplement No. 2)	*Provisions*
0.04	0.03	SMF	SMF	SMF	SMF
0.03	0.02	IMF	Not used	Not used	Not used
0.02	0.01*	OMF	OMF	IMF	IMF
Not defined	Minimal	Not used	Not used	OMF	OMF

*This is called “limited inelastic deformations” in AISC Seismic.
SMF = special moment frame.
IMF = intermediate moment frame.
OMF = ordinary moment frame.

For this example, IMF per the *Provisions* corresponds to IMF per AISC Seismic.

[The terminology in the 2002 edition of AISC Seismic is the same as Supplement No. 2 to the 1997 edition as listed in Table 5.1-6. Therefore, the terminology is unchanged from the 2000 *Provisions*.]

Because AISC Seismic does not impose more restrictive width-thickness ratios for IMF, the width-thickness ratios of AISC LRFD, Table B5.1, will be used for our IMF example. (If the frame were an SMF, then AISC Seismic would impose more restrictive requirements.)

The tapered members are approximated as short prismatic segments; thus, the adjustments of AISC LRFD Specification for web-tapered members will not affect the results of the 3-D SAP 2000 analysis.

All width-thickness ratios are less than the limiting λ_p from AISC LRFD Table B5.1. All P-M ratios (combined compression and flexure) were less than 1.00. This is based on proper spacing of lateral bracing.

Lateral bracing is provided by the roof joists and wall girts. The spacing of lateral bracing is illustrated for the high moment area of the tapered beam near the knee. The maximum moment at the face of the column under factored load combinations is less than the plastic moment, but under the design seismic ground motion the plastic moment will be reached. At that point the moment gradient will be higher than under the design load combinations (the shear will be higher), so the moment gradient at design conditions will be used to compute the maximum spacing of bracing. The moment at the face of the column is 659 ft.-kip, and 4.0 ft away the moment is 427 ft.-kip. The member is in single curvature here, so the sign on the ratio in the design equation is negative (AISC LRFD Eq. F1-17):

$$L_{pd} = \left[0.12 + 0.076\left(\frac{M_1}{M_2}\right)\right]\left(\frac{E}{F_y}\right)r_y$$

$$L_{pd}\left[0.12 + 0.076\left(\frac{-488}{659}\right)\right]\left(\frac{29{,}000}{50}\right)(1.35) = 49.9 \text{ in.} > 48 \text{ in.} \qquad \text{OK}$$

Also, per AISC LRFD Eq. F1-4:

$$L_p = 300 r_y / \sqrt{F_{yf}}$$

$$L_p = (300)(1.35)/\sqrt{50} = 57 \text{ in.} > 48 \text{ in.} \qquad \text{OK}$$

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 5.1-7).

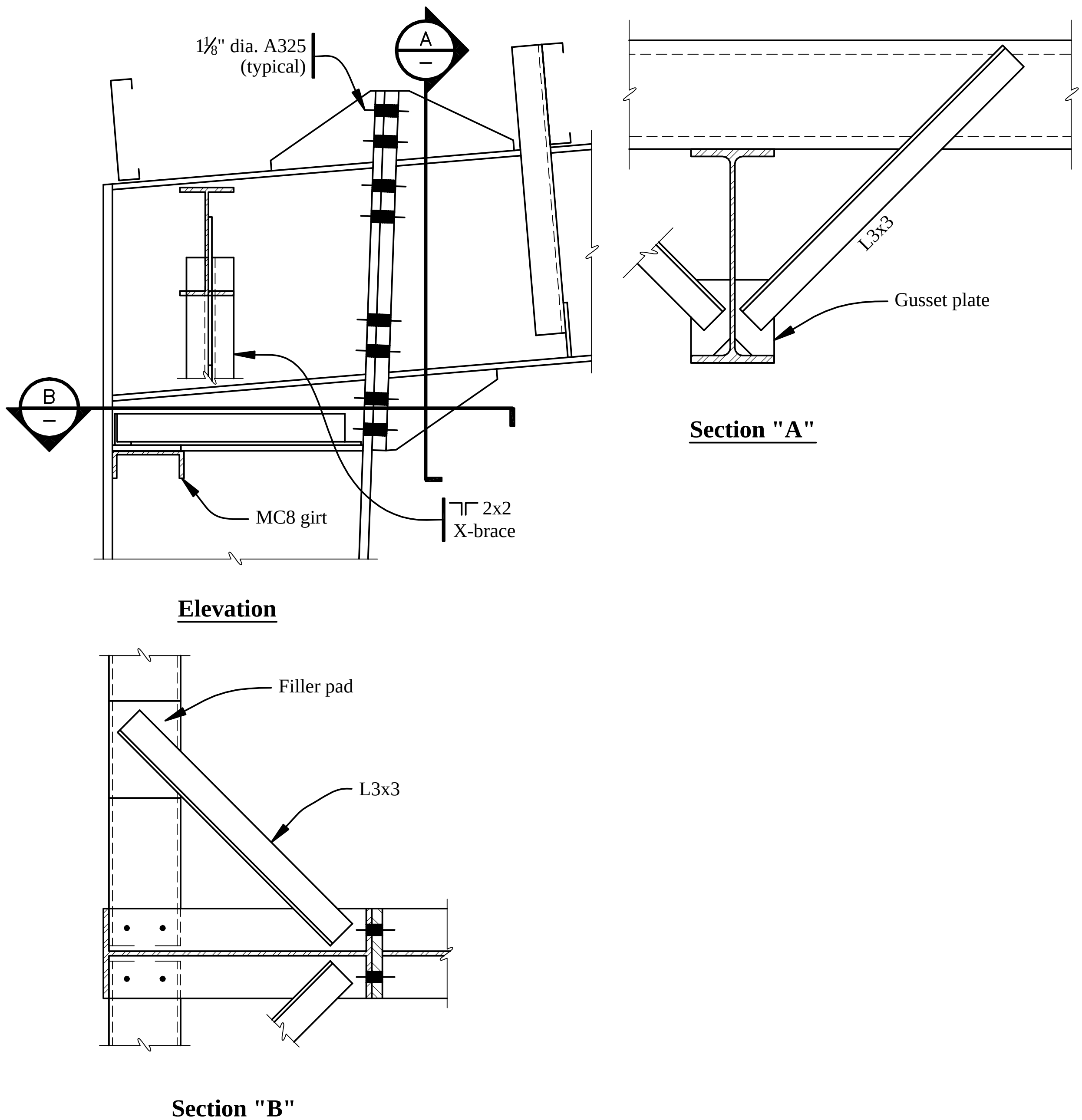


Figure 5.1-7 Arrangement at knee (1.0 in. = 25.4 mm).

5.1.5.2 Knee of the Frame

The knee detail is shown in Figures 5.1-7 and 5.1-8. The vertical plate shown near the upper left corner in Figure 5.1-7 is a gusset providing connection for X-bracing in the longitudinal direction. The beam to column connection requires special consideration. The method of FEMA 350 for bolted, stiffened end plate connections is used for a design guide here. (FEMA 350 has design criteria for specific connection details. The connection for our moment frame, which has a tapered column and a tapered beam is not one of the specific details per FEMA 350. However, FEMA 350 is used as a guide for this example because it is the closest design method developed to date for such a connection.) Refer to Figure 5.1-8 for configuration. Highlights from this method are shown for this portion of the example Refer to FEMA 350 for a discussion of the entire procedure. AISC SDGS-4 is also useful.

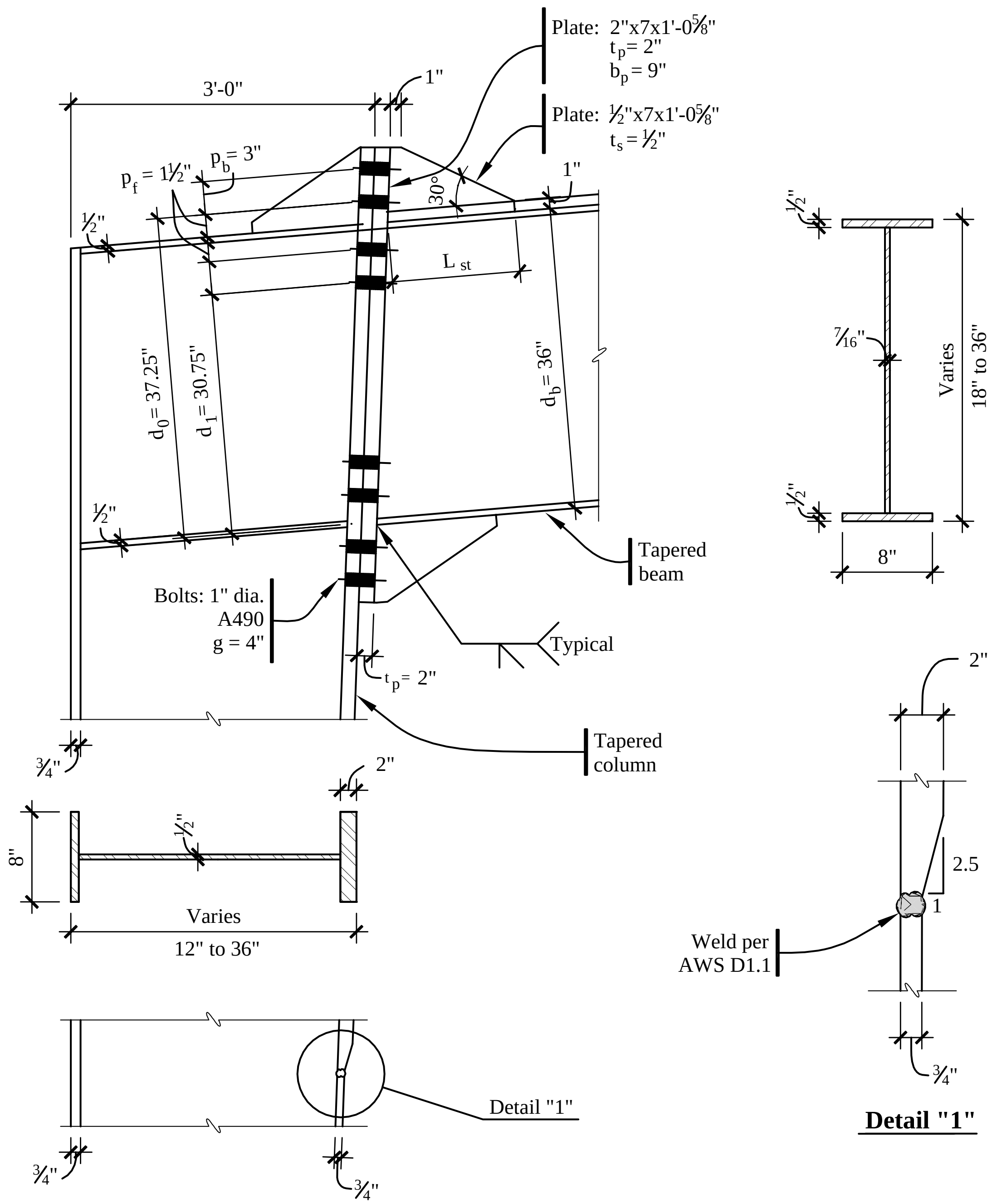


Figure 5.1-8 Bolted stiffened connection at knee (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

The FEMA 350 method for bolted stiffened end plate connection requires the determination of the maximum moment that can be developed by the beam. The steps in FEMA 350 for bolted stiffened end plates follow:

Step 1. The location of the plastic hinge is distance x from the face of the column. 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_{pr}) at the plastic hinge is computed (FEMA 350 Eq. 3-1) as follows:

$$M_{pr} = C_{pr}R_yZ_eF_y.$$

Per FEMA 350 Eq. 3-2:

$$C_{pr} = \frac{F_y + F_u}{2F_u} = \frac{(50+65)}{(2)(50)} = 1.15 \quad .$$

AISC Seismic Table I-6-1 indicates:

R_y = 1.1
Z_e = 267 in.^{3} at d/2 from the end plate (the plastic hinge location)
F_y = 50 ksi

Therefore, M_{pr} = (1.15)(1.1)(267)(50) = 16,888 in.-kips. = 1,407 ft-kips.

The moment at the column flange, M_f , which drives the connection design, is determined from FEMA 350 Figure 3-4 as:

$$M_f = M_{pr} + V_px$$

where

V_p = Shear at location of plastic hinge, assuming the frame has formed two hinges, one near each column.

$$V_p = w_g\frac{l}{2} + \frac{M_{pr1} + M_{pr2}}{l} = (0.52\text{ klf})\left(\frac{81\text{ ft}}{2}\right) + \frac{1407 + 1407\text{ ft-k}}{81\text{ft}} = 55.8\text{ kips}$$

l = 81 ft comes from the 90 ft out-to-out dimension of the frame, less the column depth and distance to the hinge at each end. Where the gravity moments are a large fraction of the section capacity, the second hinge to form, which will be in positive moment, may be away from the column face, which will reduce l and usually increase V_p. That is not the circumstance for this frame.

x = $d_b/2$ = 18 in. = 1.5 ft

Thus, M_f = 1407 + (55.8)(1.5) = 1491 ft-kips

In a like manner, the moment at the column centerline is found:

$$M_c = M_{pr} + V_p\left(x + \frac{d_c}{2}\right) = 1407 + 55.8(1.5 + 1.5) = 1574 \text{ ft-kips}$$

Step 2. Find bolt size for end plates. For a connection with two rows of two bolts inside and outside the flange, FEMA 350 Eq. 3-31 indicates:

$M_f < 3.4\ T_{ub}(d_o + d_i)$
$(1491)(12) < 3.4\ T_{ub}(37.25 \text{ in.} + 30.75 \text{ in.})$
$77.38 < T_{ub}$
$77.38 < 113\ A_b$ (for A490 bolts)
$0.685 \text{ in.}^2 < A_b$

Use 1 in. Diameter A490 bolts.

Now confirm that T_{ub} satisfies FEMA 350 Eq. 3-32:

$$T_{ub} \geq \frac{0.00002305 p_f^{\ 0.591} F_{fu}^{\ 2.583}}{t_p^{\ 0.895} d_{bt}^{\ 1.909} t_s^{\ 0.327} b_p^{\ 0.965}} + T_b$$

where:

p_f = dimension from top of flange to top of first bolt = 1.5 in.
t_p = end plate thickness = 2 in. (Trial t_p)
d_{bt} = bolt diameter = 1 in.
t_s = thickness of stiffener plate = 0.44 in.
b_p = width of end plate = 9 in.
T_b = bolt pretension per AISC LRFD Table J3.1
T_{ub} = 113 A_b = (113)(0.785) = 88.7 kips

$$T_{ub} \geq \frac{(0.00002305)(1.5)^{0.591}(504)^{2.583}}{(2)^{0.895}(1)^{1.909}(0.44)^{0.327}(9)^{0.965}} + 64$$

T_{ub} = 88.7 kips > 87.5 kips OK

Therefore, a 2-in.-thick end plate is acceptable.

Step 3. Check the bolt size to preclude shear failure. This step is skipped here because 16 bolts will obviously carry the shear for our example.

Step 4. Determine the minimum end plate thickness necessary to preclude flexural yielding by comparing the thickness determined above against FEMA 350 Eq. 3-34:

$$t_p \geq \frac{0.00609 p_f^{\ 0.9} g^{0.6} F_{fu}^{\ 0.9}}{d_{bt}^{\ 0.9} t_s^{\ 0.1} b_p^{\ 0.7}}$$

$$t_p \geq \frac{(0.00609)(1.5)^{0.9}(4)^{0.6}(504)^{0.9}}{(1)^{0.9}(0.44)^{0.1}(9)^{0.7}}$$

2 in. > 1.27 in. OK

and against FEMA 350 Eq. 3-35:

$$t_p \geq \frac{0.00413 {p_f}^{0.25} g^{0.15} F_{fu}}{{d_{bt}}^{0.7} {t_s}^{0.15} {b_p}^{0.3}}$$

$$t_p \geq \frac{(0.00413)(1.5)^{0.25}(4)^{0.15}(504)}{(1)^{0.7}(0.44)^{0.15}(9)^{0.3}}$$

2 in. > 1.66 in. OK

Therefore, use a 2-in.-thick end plate.

Step 5. Determine the minimum column flange thickness required to resist beam flange tension using FEMA 350 Eq. 3-37:

$$t_{cf} > \sqrt{\frac{\alpha_m F_{fu} C_3}{0.9 F_{yc}(3.5 p_b + c)}}$$

where

$$C_3 = \frac{g}{2} - d_{bt} - k_1 = \frac{4}{2} - \frac{1}{4} - 0.75 = 1.00 \text{ in.}$$

(For purposes of this example, k_1 is taken to be the thickness of the column web, 0.5 in. and an assumed 0.25 in. fillet weld for a total of 0.75 in.).

Using FEMA 350 Eq. 3-38:

$$\alpha_m = C_a \left(\frac{A_f}{A_w}\right)^{\frac{1}{3}} \frac{C_3}{(d_{bt})^{\frac{1}{4}}} = (1.48)\left(\frac{(2)(8)(0.5)}{(35)(0.44)}\right)^{\frac{1}{3}} \frac{1}{(1)^{0.25}} = 1.19$$

$$t_{cf} > \sqrt{\frac{(1.19)(504)(1.00)}{(0.9)(50)[(3.5)(3) + (3.5)]}} = 0.95 \text{ in.}$$

Minimum t_{cf} = 0.95 in. but this will be revised in Step 7.

Step 6. Check column web thickness for adequacy for beam flange compression. This is a check on web crippling using FEMA 350 Eq. 3-40:

$$t_{wc} = \frac{M_f}{(d_b - t_{fb})(6k + 2t_p + t_{fb})F_{yc}} = \frac{(1491)(12)}{(36 - 0.5)[(6)(0.75) + (2)(2) + (0.5)](50)} = 1.44 \text{ in.}$$

$t_{wc\ reqd}$ = 1.44 in. > 0.5 in. = t_{wc} OK

Therefore, a continuity plate is needed at the compression flange. See FEMA 350 Sec. 3.3.3.1 for continuity plate sizing. For one-sided connections, the necessary thickness of the continuity plate is $0.5(t_{bf} + t_{bf})$ = 0.5 in.

Step 7. Because continuity plates are required, t_{cf} must be at least as thick as the end plate thickness t_p. Therefore, t_{cf} = 2 in. For this column, the 2-in.-thick flange does not need to be full height but must continue well away from the region of beam flange compression and the high moment portion of the column knee area. Some judgment is necessary here. For this case, the 2-in. flange is continued 36 in. down from the bottom of the beam, where it is welded to the 0.75-in.- thick flange. This weld needs to be carefully detailed.

Step 8. Check the panel zone shear in accordance with FEMA 350, Sec. 3.3.3.2. For purposes of this check, use d_b = 35.5 + 1.5 + 3 + 1.5 = 41.5 in. Per FEMA 350 Eq. 3-7:

$$t \geq \frac{C_y M_c \left(\frac{h-d_b}{h}\right)}{(0.9)(0.6F_y)R_{yc}d_c(d_b - t_{fb})}$$

where, according to FEMA 350 Eq. 3-4:

$$C_y = \frac{1}{C_{pr}\frac{Z_{be}}{S_b}} = \frac{1}{1.15\left(\frac{267}{218}\right)} = 0.71$$

$$t_{cw} \geq \frac{(0.71)(1574 \text{ x } 12)\left(\frac{366-41.5}{366}\right)}{(0.9)(0.6)(50)(1.1)(36)(36-0.5)} = 0.31 \text{ in.}$$

$t_{cw\ required}$ = 0.31 in. < 0.50 in. = t_{cw} OK

5.1.5.3 Frame at the Ridge

The ridge joint detail is shown in Figure 5.1-9. An unstiffened bolted connection plate is selected.

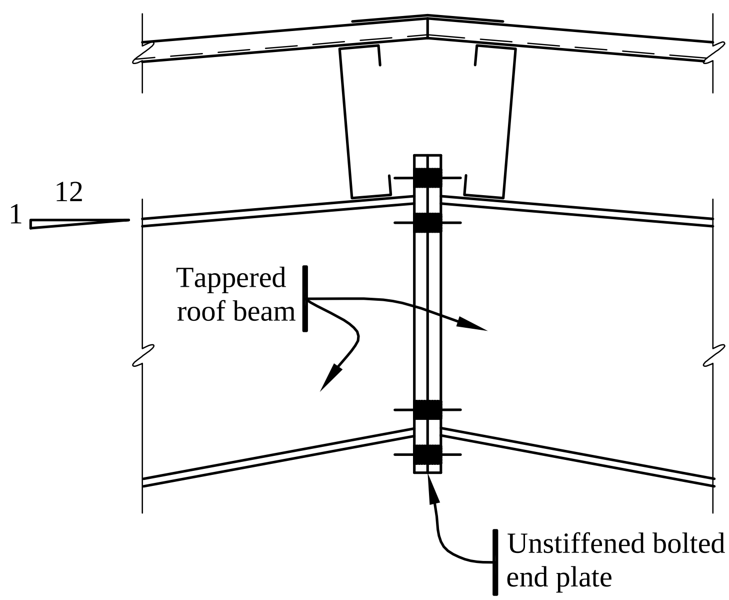
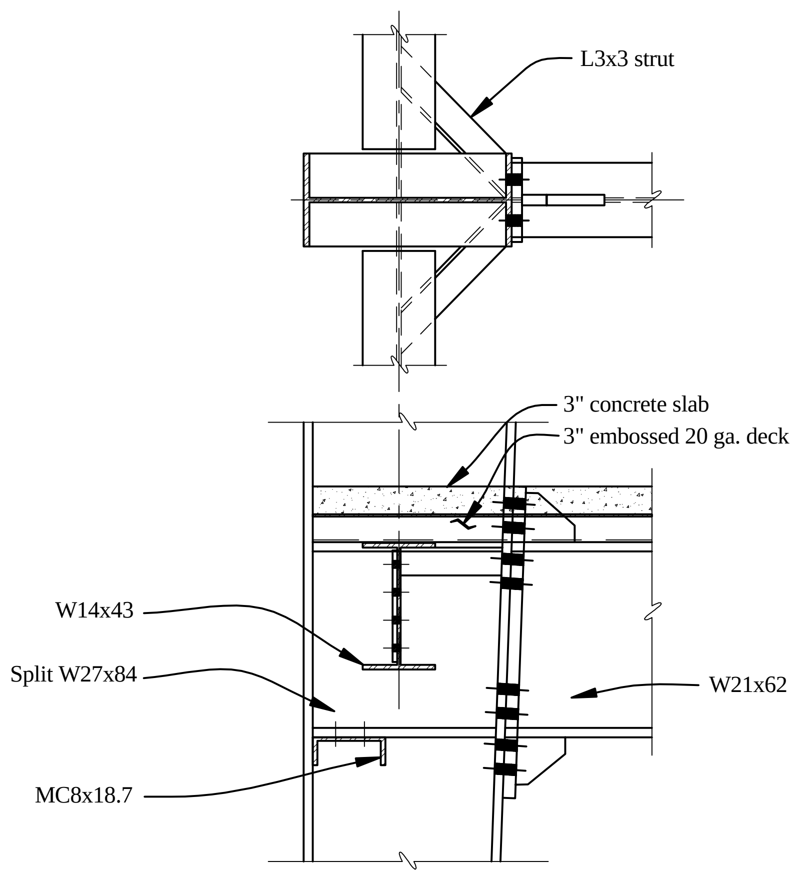


Figure 5.1-9 End plate connection at ridge.

This is an AISC LRFD designed connection, not a FEMA 350 designed connection because there should not be a plastic hinge forming in this vicinity. Lateral seismic force produces 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 lateral seismic loads produce yielding at one knee, further lateral displacement produces some positive moment at the ridge. Under the condition on which the FEMA 350 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. If one uses $1.2D + 0.2S$ as the load for this scenario, 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.

5.1.5.4 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-in., 20-gauge steel deck of the mezzanine floor is supported on steel beams spaced at 10 ft and spanning 20 ft (Figure 5.1-2). The steel beams rest on three-span girders connected at each end to the portal frames and supported on two intermediate columns (Figure 5.1-1). The girder spans are approximately 30 ft each. The design of the mezzanine framing is largely conventional as seismic loads do not predominate. 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 5.1-10. The design of the end plate connection is similar to that at the knee, but simpler because the beam is horizontal and not tapered.



(b)

Figure 5.1-10 Mezzanine framing (1.0 in. = 25.4 mm).

5.1.5.5 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 Seismic Sec. 14.2 (November 2000 Supplement) for requirements on braces for OCBFs. The strength of the members and connections, including the columns in this area but excluding the brace connections, shall be based on AISC Seismic Eq. 4-1.

$$1.2D + 0.5L + 0.2S + \Omega_0 Q_E$$

Recall that a 1.0 factor is applied to L when the live load is greater than 100 psf (AISC Seismic Sec. 4.1). For the case discussed here, the "tension only" brace does not carry any live load so the load factor does not matter. For the braced design, $\Omega_0 = 2$.

However, *Provisions* Sec. 5.2.7.1, Eq. 5.2.7.1-1 and -2 [4.2-3 and 4.2-4, respectively] requires that the design seismic force on components sensitive to overstrength shall be defined by:

$$E = \Omega_0 Q_E \pm 0.2S_{DS}D$$

Given that the *Provisions* is being following, the AISC Seismic equation will be used but E will be substituted for Q_E. Thus, the load combination for design of the brace members reduces to:

$$1.4D + 0.5L + 0.2S + \Omega_0 Q_E$$

[The special load combinations have been removed from the 2002 edition of AISC Seismic to eliminate inconsistencies with other building codes and standards but the design of ordinary braced frames is not really changed because there is a reference to the load combinations including "simplified seismic loads." Therefore, 2003 *Provisions* Eq. 4.2-3 and 4.2-4 should be used in conjunction with the load combinations in ASCE 7 as is done here.]

From analysis using this load combination, the maximum axial force in the X brace located at the east end of the building is 66 kips computed from the combined orthogonal earthquake loads (longitudinal direction predominates). With the Ω_0 factor, the required strength becomes 132 kips. All braces will have the same design. Using A36 steel for angles:

$$T_n = \phi F_y A_g$$

$$A_g = \frac{P_n}{\phi F_y} = \frac{132}{(0.9)(36)} = 4.07 \text{ in.}^2$$

Try (2) L4 ×3 × 3/8:

$$A_g = (2)(2.49) = 4.98 \text{ in.}^2 > 4.07 \text{ in.}^2 \qquad \text{OK}$$

AISC Seismic Sec. 14.2 requires the design strength of the brace connections to be based on the expected tensile strength:

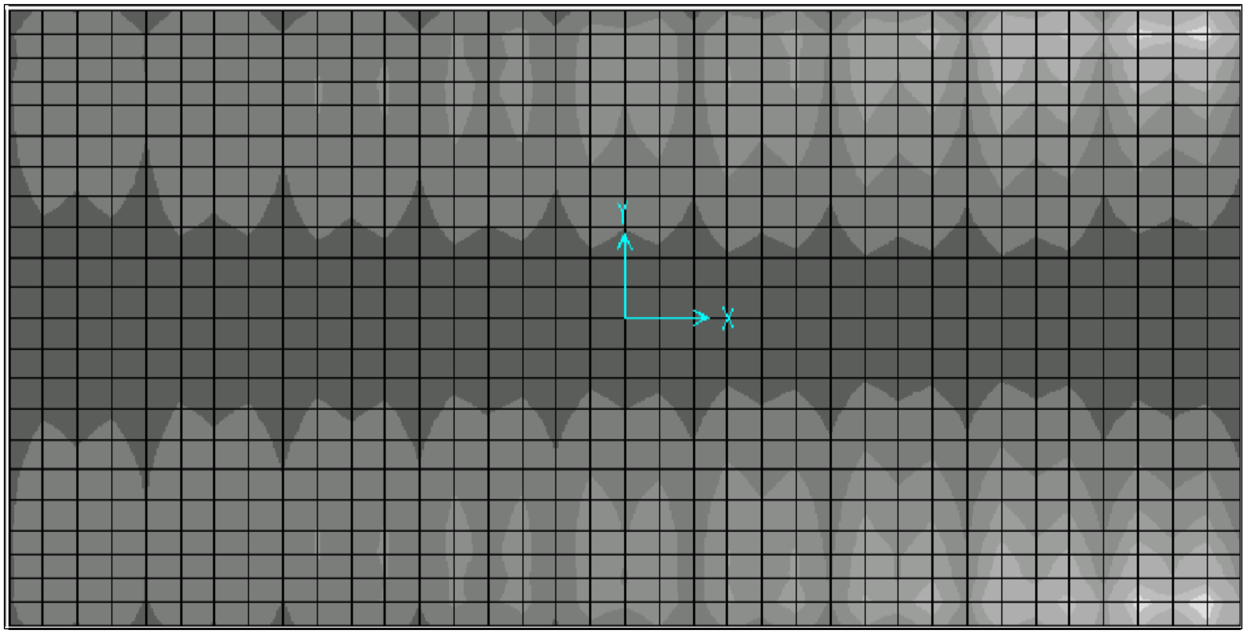
$$R_y F_y A_g = (1.5)(36 \text{ ksi})(4.98 \text{ in.}^2) = 269 \text{ kips.}$$

Also be sure to check the eave strut at the roof. The eave strut, part of the braced frame, has to carry compression and that compression is determined using the overstrength factor.

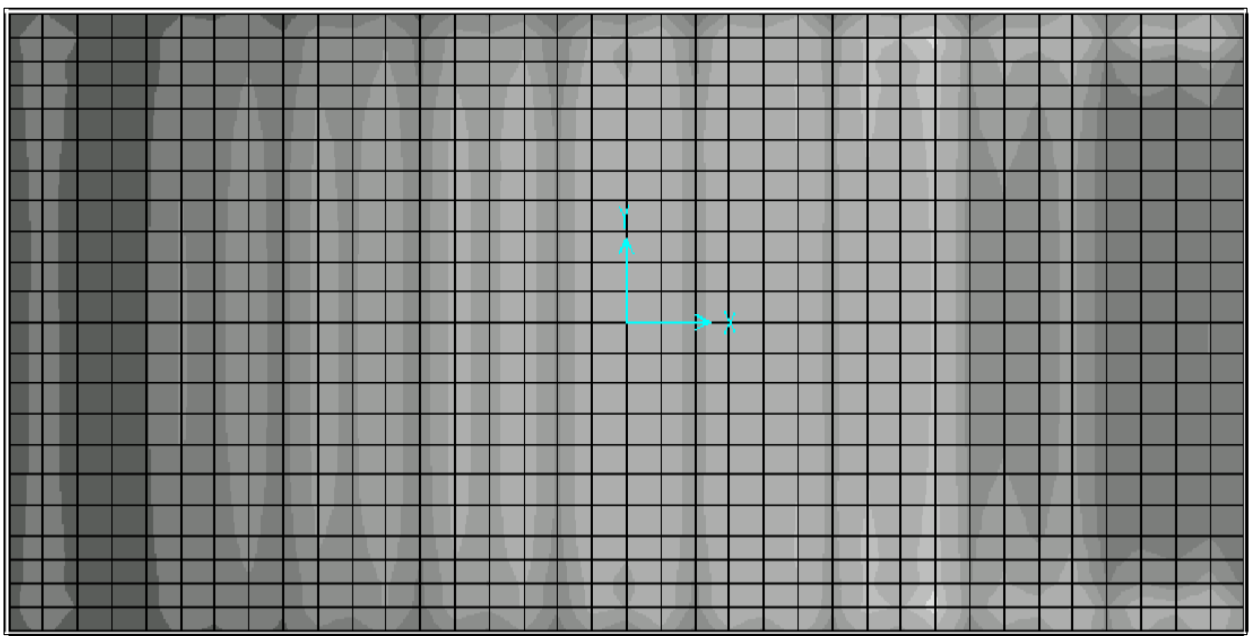
The *kl/r* requirement of AISC Seismic Sec. 14.2 does not apply because this is not a V or an inverted V configuration.

5.1.5.6 Roof Deck Diaphragm

Figure 5.1-11 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 230 kips (Sec. 5.1.4.2) with 83 percent or 191 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 (191,000/2)/180 ft. = 530 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 3-D 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 might be expected given the significant change in stiffness. There does not appear to be any particularly good simple approximation to estimate the shear here without a 3-D model. The shear is also high at the longitudinal braced bays because they tend to resist the horizontal torsion. The shear at the braced bays is lower than observed for the E-W motion, however.



Roof diaphragm shear, East-West motion, pound per foot.



Roof diaphragm shear, North-South motion, pound per foot.

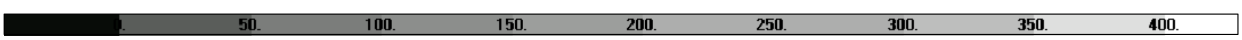


Figure 5.1-11 Shear force in roof deck diaphragm; upper diagram is for E-W motion and lower is for N-S motion (1.0 lb. /ft. = 14.59 N/M).

5.2 SEVEN-STORY OFFICE BUILDING, LOS ANGELES, CALIFORNIA

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

5.2.1 Building Description

5.2.1.1 General Description

This seven-story office building of rectangular plan configuration is 177 ft, 4 in. long in the E-W direction and 127 ft, 4 in. wide in the N-S direction (Figure 5.2-1). The building has a penthouse. It extends a total of 118 ft, 4 in. above grade. It is framed in structural steel with 25-ft bays in each direction. The story height is 13 ft, 4 in. except for the first story which is 22 ft, 4 in. high. The penthouse extends 16 ft above the roof level of the building and covers the area bounded by gridlines C, F, 2, and 5 in Figure 5.2-1. Floors consist of 3-1/4 in. lightweight concrete placed on composite metal deck. The elevators and stairs are located in the central three bays. The building is planned for heavy filing systems (350 psf) covering approximately four bays on each floor.

5.2.1.2 Alternatives

This example features three alternatives – a steel moment-resisting frame, concentrically braced frame, and a dual system with a moment-resisting frame at the perimeter and a concentrically braced frame at the core area – as follows:

1. Alternative A – Seismic force resistance is provided by special moment frames located on the perimeter of the building (on lines A, H, 1, and 6 in Figure 5.2-1, also illustrated in Figure 5.2-2).

2. Alternative B – Seismic force resistance is provided by four special concentrically braced frames in each direction. They are located in the elevator core walls between columns 3C and 3D, 3E and 3F, 4C and 4D, and 4E and 4F in the E-W direction and between columns 3C and 4C, 3-D and 4D, 3E and 4E, and 3F and 4F in the N-S direction (Figure 5.2-1). The braced frames in an X configuration are designed for both diagonals being effective in tension and compression. The braced frames are not identical, but are arranged to accommodate elevator door openings. Braced frame elevations are shown in Figure 5.2-3.

3. Alternative C – Seismic force resistance is provided by a dual system with the special moment frames at the perimeter of the building and a special concentrically braced frames at the core. The moment frames are shown in Figure 5.2-2 and the braced frames are shown in Figure 5.2-3.

5.2.1.3 Scope

The example covers:

1. Seismic design parameters
2. Analysis of perimeter moment frames
3. Beam and column proportioning
4. Analysis of concentrically braced frames
5. Proportioning of braces
6. Analysis and proportioning of the dual system

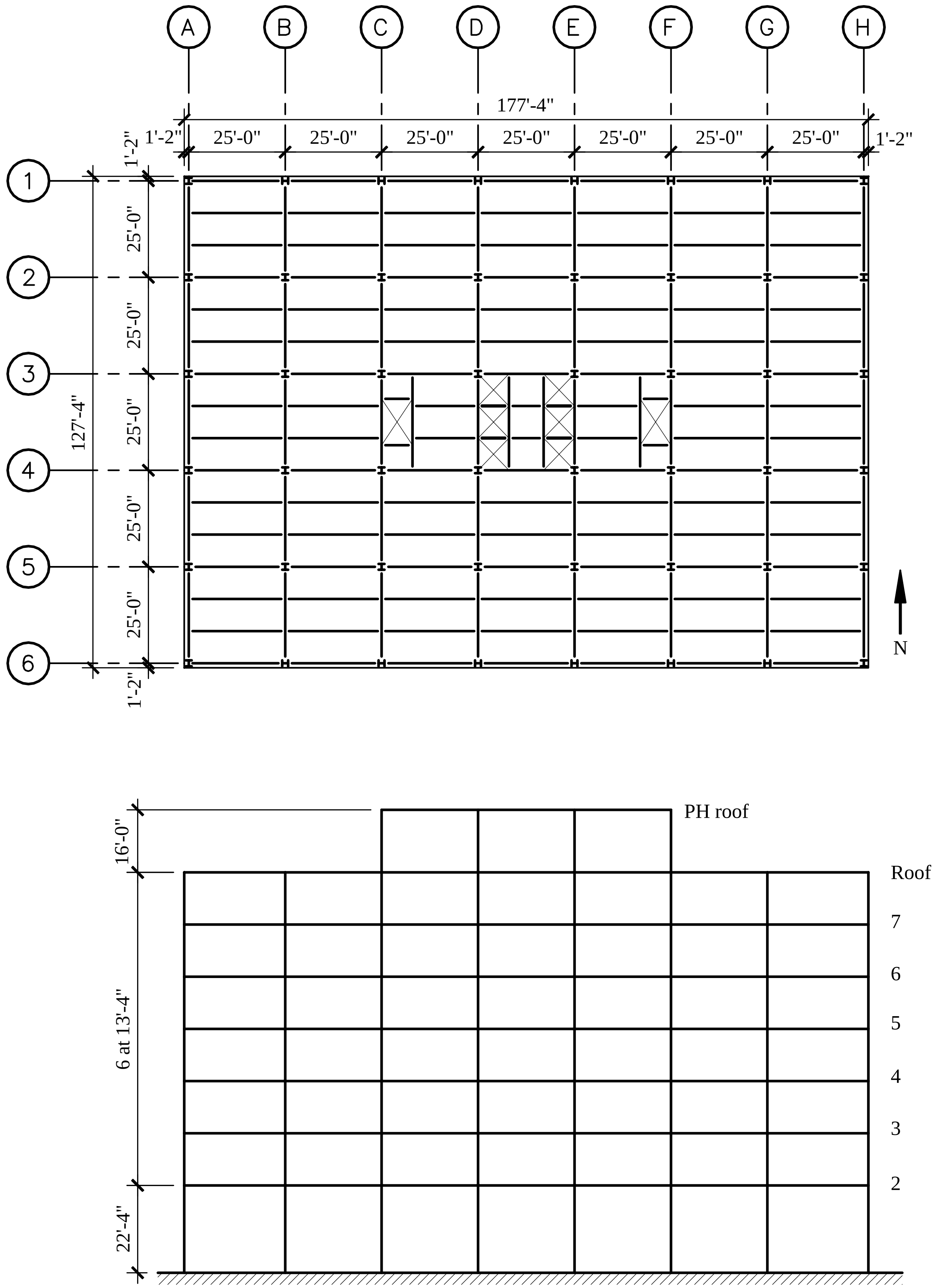


Figure 5.2-1 Typical floor framing plan and building section (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

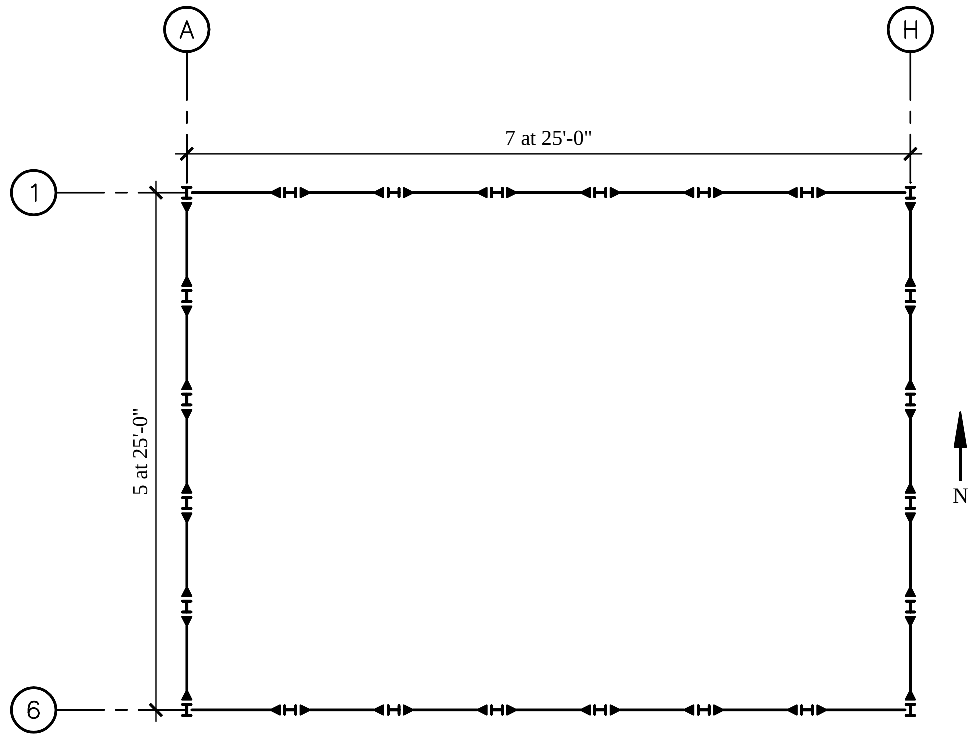


Figure 5.2-2 Framing plan for special moment frame (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

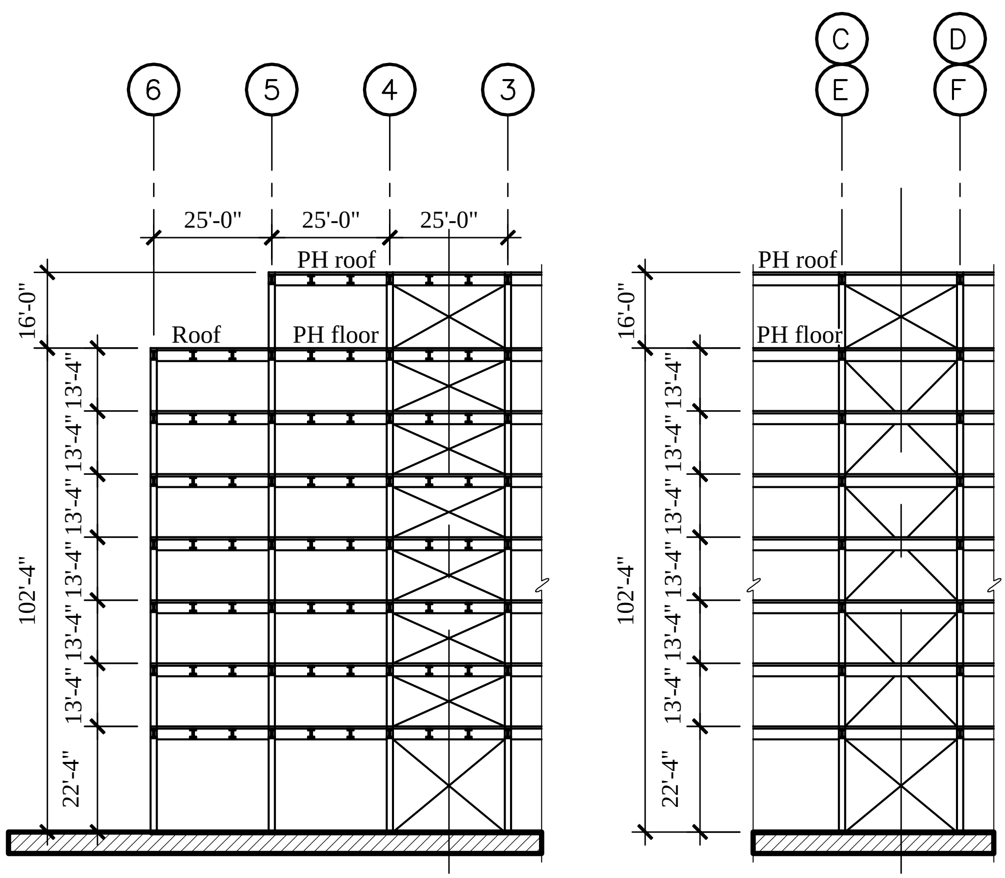


Figure 5.2-3 Concentrically braced frame elevations (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

5.2.2 Basic Requirements

5.2.2.1 *Provisions* Parameters

Site Class = D	(*Provisions* Sec. 4.1.2.1 [3.5])
S_S = 1.5	(*Provisions* Map 9 [Figure 3.3-3])
S_1 =0.6	(*Provisions* Map 10 [Figure 3.3-4])
F_a = 1.0	(*Provisions* Table 4.1.2.4a [3.3-1])
F_v = 1.5	(*Provisions* Table 4.1.2.4b [3.3-2])
$S_{MS} = F_aS_S$ = 1.5	(*Provisions* Eq. 4.1.2.4-1 [3.3-1])
$S_{M1} = F_vS_1$ = 0.9	(*Provisions* Eq. 4.1.2.4-2 [3.3-2])
S_{DS} = 2/3 S_{MS} = 1.0	(*Provisions* Eq. 4.1.2.5-1 [3.3-3])
S_{D1} = 2/3 S_{M1} = 0.6	(*Provisions* Eq. 4.1.2.5-2 [3.3-4])
Seismic Use Group = I	(*Provisions* Sec. 1.3 [1.2])
Seismic Design Category = D	(*Provisions* Sec. 4.2.1 [1.4])

Alternative A, Special Steel Moment Frame (*Provisions* Table 5.2.2 [4.3-1])

R = 8
Ω_0 = 3
C_d = 5.5

Alternative B, Special Steel Concentrically Braced Frame (*Provisions* Table 5.2.2 [4.3-1])

R = 6
Ω_0 = 2
C_d = 5

Alternative C, Dual System of Special Steel Moment Frame Combined with Special Steel Concentrically Braced Frame (*Provisions* Table 5.2.2 [4.3-1])

R = 8
Ω_0 = 2.5
C_d = 6.5

5.2.2.2 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, M&E)	= 55 psf
Exterior wall cladding	= 25 psf of wall
Penthouse floor *D*	= 65 psf
Floor *L*	= 50 psf
Floor *D* (deck, beams, girders, fireproofing, ceiling, M&E, partitions)	= 62.5 psf
Floor *L* reductions per the IBC	

5.2.2.3 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
HSS	ASTM A500, Grade B
Plates	ASTM A36

5.2.3 Structural Design Criteria

5.2.3.1 Building Configuration

The building is considered vertically regular despite the relatively tall height of the first story. The exception of *Provisions* Sec. 5.2.3.3 [4.3.2.3]is taken in which the drift ratio of adjacent stories are compared rather than the stiffness of the stories. In the 3-D analysis, it will be shown that the first story drift ratio is less than 130 percent of the story above. Because the building is symmetrical in plan, plan irregularities would not be expected. Analysis reveals that Alternatives B and C are torsionally irregular, which is not uncommon for core-braced buildings.

5.2.3.2 Redundancy

According to *Provisions* Sec. 5.2.4.2 [not applicable in the 2003 *Provisions*], the reliability factor, (ρ) for a Seismic Design Category D structure is:

$$\rho = 2 - \frac{20}{r_{max_x}\sqrt{A_x}}$$

In a typical story, the floor area, A_x = 22,579 ft.^{2}

The ratio of the design story shear resisted by the single element carrying the most shear force in the story to the total story shear is r_{max_x} as defined in *Provisions* Sec. 5.2.4.2.

Preliminary ρ factors will be determined for use as multipliers on design force effects. These preliminary ρ factors will be verified by subsequent analyses.

[The redundancy requirements have been substantially changed in the 2003 *Provisions*. For a building assigned to Seismic Design Category D, ρ = 1.0 as long as it can be shown that failure of beam-to-column connections at both ends of a single beam (moment frame system) or failure of an individual brace (braced frame system) would not result in more than a 33 percent reduction in story strength or create an extreme torsional irregularity. Alternatively, if the structure is regular in plan and there are at least two bays of perimeter framing on each side of the structure in each orthogonal direction, it is permitted to use ρ = 1.0. Per 2003 *Provisions* Sec. 4.3.1.4.3, special moment frames in Seismic Design Category D must be configured such that the structure satisfies the criteria for ρ = 1.0. There are no reductions in the redundancy factor for dual systems. Based on the preliminary design, ρ = 1.0 for Alternative A because it has a perimeter moment frame and is regular. The determination of ρ for Alternatives B and C (which are torsionally irregular) requires the evaluation of connection and brace failures per 2003 *Provisions* Sec. 4.3.3.2.]

5.2.3.2.1 Alternative A (moment frame)

For a moment-resisting frame, r_{max_x} is taken as the maximum of the sum of the shears in any two adjacent columns divided by the total story shear. The final calculation of ρ will be deferred until the building frame analysis, which will include the effects of accidental torsion, is completed. At that point, we will know the total shear in each story and the shear being carried by each column at every story. See Sec. 5.2.4.3.1.

Provisions Sec. 5.2.4.2 requires that the configuration be such that ρ shall not exceed 1.25 for special moment frames. [1.0 in the 2003 *Provisions*] (There is no limit for other structures, although ρ need not be taken larger than 1.50 in the design.) Therefore, it is a good idea to make a preliminary estimate of ρ, which was done here. In this case, ρ was found to be 1.11 and 1.08 in the N-S and E-W directions, respectively. A method for a preliminary estimate is explained in Alternative B.

Note that ρ is a multiplier that applies only to the force effects (strength of the members and connections), not to displacements. As will be seen for this moment-resisting frame, drift, and not strength, will govern the design.

5.2.3.2.2 Alternative B (concentrically braced frame)

Again, the following preliminary analysis must be refined by the final calculation. For the braced frame system, there are four braced-bay braces subject to shear at each story, so the direct shear on each line of braces is equal to $V_x/4$. The effects of accidental torsion will be estimated as:

The torsional moment $M_{ta} = (0.05)(175)(V_x) = 8.75V_x$.
The torsional force applied to either grid line C or F is $V_t = M_{ta}Kd / \Sigma Kd^2$.

Assuming all frame rigidity factors (K) are equal:

$$V_t = \frac{M_{ta}(37.5)}{\left[(2)(37.5)^2 + (6)(12.5)^2\right]} = 0.01M_{ta}$$

$$V_t = (0.01)(8.75\ V_x) = 0.0875V_x$$

The amplification of torsional shear (A_x) must be considered in accordance with *Provisions* Sec. 5.4.4.1.3 [5.2.4.3]. Without dynamic amplification of torsion, the direct shear applied to each line of braces is $V_x/4$ and the torsional shear, $V_t = 0.0875\ V_x$. Thus, the combined shear at Grid C is $0.25V_x - 0.0875V_x = 0.1625V_x$, and the combined shear at Grid F is $0.25V_x + 0.0875V_x = 0.3375V_x$. As the torsional deflections will be proportional to the shears and extrapolating to Grids A and H, the deflection at A can be seen to be proportional to $0.250V_x + (0.0875V_x.)(87.5/37.5) = 0.454V_x$. Likewise, the deflection at H is proportional to $0.250V_x - (0.0875V_x)(87.5/37.5) = 0.046V_x$. The average deflection is thus proportional to $[(0.454 + 0.046)/2]V_x = 0.250V_x$. These torsional effects are illustrated in Figure 5.2-4.

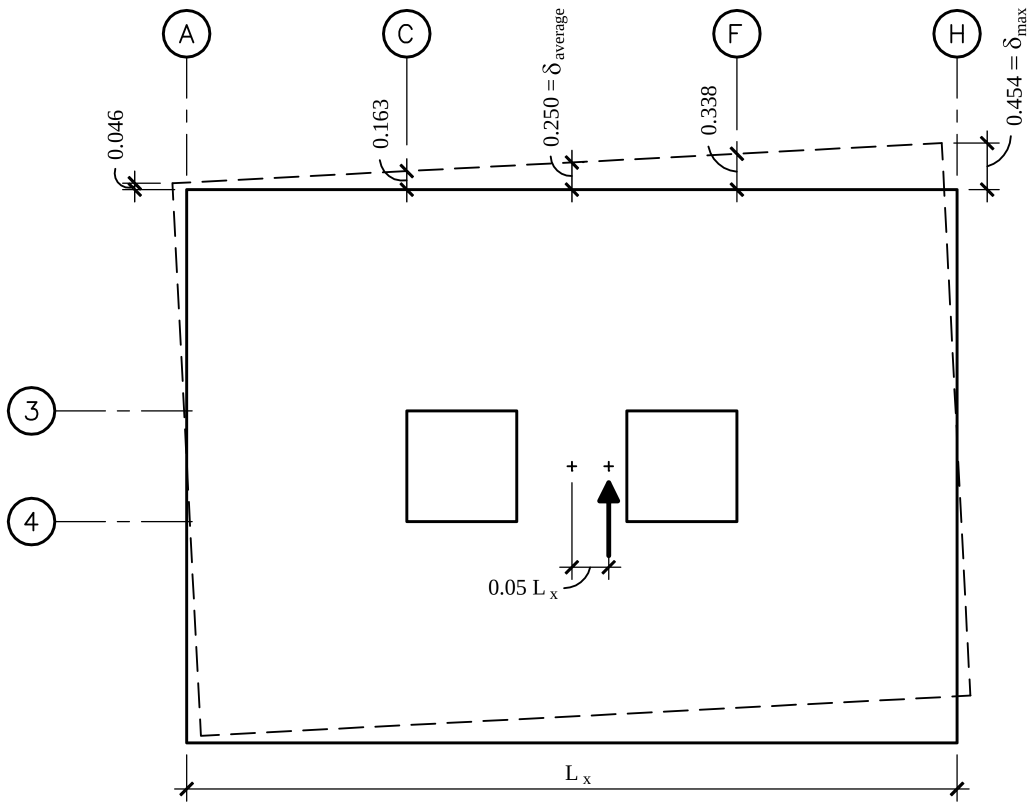


Figure 5.2-4 Approximate effect of accidental of torsion (1.0 in. = 25.4 mm).

From the above estimation of deflections, the torsional amplification can be determined per *Provisions* Eq. 5.4.4.1.3.1 [5.2-13] as:

$$A_x = \left(\frac{\delta_{max}}{1.2\ \delta_{avg}}\right)^2 = \left(\frac{0.454}{(1.2)(0.250)}\right)^2 = 2.29$$

The total shear in the N-S direction on Gridlines C or F is the direct shear plus the amplified torsional shear equal to:

$$V_x/4 + A_x V_t = [0.250 + (2.29)(0.0875)]V_x = 0.450V_x$$

As there are two braces in each braced bay (one in tension and the other in compression):

$$r_{max_x} = \frac{0.450}{2} = 0.225$$

and

$$\rho = 2 - \frac{20}{r_{max_x}\sqrt{Ax}} = 2 - \frac{20}{(0.225)\sqrt{22,579}} = 1.41$$

Therefore, use ρ = 1.41 for the N-S direction. In a like manner, the ρ factor for the E-W direction is determined to be ρ = 1.05. These preliminary values will be verified by the final calculations.

5.2.3.2.3 Alternative C (dual system)

For the dual system, the preliminary value for ρ is taken as 1.0. The reason for this decision is that, with the dual system, the moment frame will substantially reduce the torsion at any story, so torsional amplification will be low. The combined redundancy of the braced frame combined with the moment frame (despite the fact that the moment frame is more flexible) will reduce ρ from either single system. Finally, *Provisions* Sec. 5.2.4.2 [not applicable in the 2003 *Provisions*] calls for taking only 80 percent of the calculated ρ value when a dual system is used. Thus, we expect the final value to fall below 1.0, for which we will take ρ = 1.0. This will be verified by analysis later.

5.2.3.3 Orthogonal Load Effects

A combination of 100 percent of the seismic forces in one direction with 30 percent seismic forces in orthogonal direction is required for structures in Seismic Design Category D (*Provisions* Sec. 5.2.5.2.3 and 5.2.5.2.2 [4.4.2.2]).

5.2.3.4 Structural Component Load Effects

The effect of seismic load is be defined by *Provisions* Eq, 5.2.7-1 [4.2-1] as:

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

Recall that S_{DS} = 1.0. As stated above, ρ values are preliminary estimates to be checked and, if necessary, refined later.

For Alternative A

N-S direction $E = (1.11)Q_E \pm (0.2)D$
E-W direction $E = (1.08)Q_E \pm (0.2)D$

Alternative B

N-S direction $E = (1.41)Q_E \pm (0.2)D$
E-W direction $E = (1.05)Q_E \pm (0.2)D$

Alternative. C

N-S direction $E = (1.00)Q_E \pm (0.2)D$
E-W direction $E = (1.00)Q_E \pm (0.2)D$

5.2.3.5 Load Combinations

Load combinations from ASCE 7 are:

$$1.2D + 1.0E + 0.5L + 0.2S$$

and

$$0.9D + 1.0E + 1.6H$$

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

Alternative A

N-S	$1.4D + 1.11Q_E + 0.5L$ and $0.7D + 1.11Q_E$
E-W	$1.4D + 1.08Q_E + 0.5L$ and $0.7D + 1.08Q_E$

Alternative B

N-S	$1.4D + 1.41Q_E + 0.5L$ and $0.7D + 1.41Q_E$
E-W	$1.4D + 1.05Q_E + 0.5L$ and $0.7D + 1.05Q_E$

Alternative C

N-S	$1.4D + Q_E + 0.5L$ and $0.7D + Q_E$
E-W	$1.4D + Q_E + 0.5L$ and $0.7D + Q_E$

5.2.3.6 Drift Limits

The allowable story drift per *Provisions* Sec. 5.2.8 [4.5-1] is $\Delta_a = 0.02h_{sx}$.

The allowable story drift for the first floor is Δ_a = (0.02)(22.33 ft)(12 in./ft) = 5.36 in.
The allowable story drift for a typical story is Δ_a = (0.02)(13.33 ft)(12 in./ft) = 3.20 in.

Remember to adjust calculated story drifts by the appropriate C_d factor from Sec. 5.2.2.1.

Consider that the maximum story drifts summed to the roof of the seven-story building, (102 ft-4 in. main roof/penthouse floor) is 24.56 in.

5.2.3.7 Basic Gravity Loads

Penthouse roof

Roof slab = (0.025)(75)(75)	=	141 kips
Walls = (0.025)(8)(300)	=	60 kips
Columns = (0.110)(8)(16)	=	14 kips
Total	=	215 kips

Lower roof

Roof slab = (0.055)[(127.33)(177.33) - (75)^2]	=	932 kips
Penthouse floor = (0.065)(75)(75)	=	366 kips
Walls = 60 + (0.025)(609)(6.67)	=	162 kips
Columns = 14 + (0.170)(6.67)(48)	=	68 kips
Equipment (allowance for mechanical equipment in penthouse)	=	217 kips
Total	=	1,745 kips

Typical floor

Floor = (0.0625)(127.33)(177.33)	= 1,412 kips
Walls = (0.025)(609)(13.33)	= 203 kips
Columns = (0.285)(13.33)(48)	= 182 kips
Heavy storage = (0.50)(4)(25 x 25)(350)	= 438 kips
Total	= 2,235 kips

Total weight of building = 215 + 1,745 + 6(2,235) = 15,370 kips

Note that this office building has heavy storage in the central bays of 280 psf over five bays. Use 50 percent of this weight as effective seismic mass. (This was done to add seismic mass to this example thereby making it more interesting. It is not meant to imply that the authors believe such a step is necessary for ordinary office buildings.)

5.2.4 Analysis

5.2.4.1 Equivalent Lateral Force Analysis

The equivalent lateral force (ELF) procedure will be used for each alternative building system. The seismic base shear will be determined for each alternative in the following sections.

5.2.4.1.1 ELF Analysis for Alternative A, Moment Frame

First determine the building period (T) per *Provisions* Eq. 5.4.2.1-1 [5.2-6]:

$$T_a = C_r h_n^x = (0.028)(102.3)^{0.8} = 1.14\,\text{sec}$$

where h_n, the height to the main roof, is conservatively taken as 102.3 ft. The height of the penthouse (the penthouse having a smaller contribution to seismic mass than the main roof or the floors) will be neglected.

The seismic response coefficient (C_s,) is determined from *Provisions* Eq. 5.4.1.1-1 [5.2-2] as:

$$C_S = \frac{S_{DS}}{R/I} = \frac{1}{(8/1)} = 0.125$$

However, *Provisions* Eq. 5.4.1.1-2 [5.2-3] indicates that the value for C_s need not exceed:

$$C_S = \frac{S_{D1}}{T(R/I)} = \frac{0.6}{1.14(8/1)} = 0.066$$

and the minimum value for C_s per *Provisions* Eq. 5.4.1.1-3 [not applicable in the 2003 *Provisions*] is:

$$C_s = 0.044 I S_{DS} = (0.044)(1)(1) = 0.044$$

Therefore, use C_s = 0.066.

Seismic base shear is computed per *Provisions* Eq. 5.4.1 [5.2-1] as:

$$V = C_S W = (0.066)(15{,}370) = 1014 \text{ kips}$$

5.2.4.1.2 ELF Analysis for Alternative B, Braced Frame

As above, first find the building period (*T*) using *Provisions* Eq. 5.4.2.1-1 [5.2-6]:

$$T_a = C_r h_n^x = (0.02)(102.3)^{0.75} = 0.64 \text{ sec}$$

The seismic response coefficient (C_s) is determined from *Provisions* Eq. 5.4.1.1-1 [5.2-2] as:

$$C_S = \frac{S_{DS}}{R/I} = \frac{1}{(6/1)} = 0.167$$

However, *Provisions* Eq. 5.4.1.1-2 [5.2-3] indicates that the value for C_s need not exceed:

$$C_S = \frac{S_{D1}}{T(R/I)} = \frac{0.6}{(0.64)(6/1)} = 0.156$$

and the minimum value for C_s per *Provisions* Eq. 5.4.1.1-3 [not applicable in 2003 *Provisions*] is:

$$C_s = 0.044 I S_{DS} = (0.044)(1)(1) = 0.044$$

Use C_s = 0.156.

Seismic base shear is computed using *Provisions* Eq. 5.4.1 [5.2-1] as:

$$V = C_S W = (0.156)(15{,}370) = 2{,}398 \text{ kips}$$

5.2.4.1.3 ELF Analysis for Alternative C, Dual System

The building period (*T*) is the same as for the braced frame (*Provisions* Eq. 5.4.2.1-1 [5.2-6]):

$$T_a = C_r h_n^x = (0.02)(102.3)^{0.75} = 0.64 \text{ sec}$$

The seismic response coefficient (C_s) is determined as (*Provisions* Eq. 5.4.1.1-1 [5.2-2]):

$$C_S = \frac{S_{DS}}{R/I} = \frac{1}{(8/1)} = 0.125$$

However, the value for C_s need not exceed (*Provisions* Eq. 5.4.1.1-2 [5.2-3]):

$$C_S = \frac{S_{D1}}{T(R/I)} = \frac{0.6}{(0.64)(8/1)} = 0.117$$

and the minimum value for C_s is (*Provisions* Eq. 5.4.1.1-3 [not applicable in the 2003 *Provisions*]):

$$C_s = 0.044 I S_{DS} = (0.044)(1)(1) = 0.044$$

Therefore, use C_s = 0.117.

Seismic base shear is computed as (*Provisions* Eq. 5.4.1 [5.2-1]):

$V = C_s W = (0.117)(15{,}370) = 1{,}798$ kips

5.2.4.2 Vertical Distribution of Seismic Forces

Provisions Sec. 5.4.3 [5.2.3] provides the procedure for determining the portion of the total seismic load that goes to each floor level. The floor force F_x is calculated using *Provisions* Eq. 5.4.3-1 [5.2-10] as:

$$F_x = C_{vx} V$$

where (*Provisions* Eq. 5.4.3-2 [5.2-11])

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

For Alternative A

$T = 1.14$ secs, thus $k = 1.32$

For Alternatives B and C

$T = 0.64$ sec, thus $k = 1.07$

Using *Provisions* Eq. 5.4.4 [5.2-12], the seismic design shear in any story is computed as:

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

The story overturning moment is computed from *Provisions* Eq. 5.4.5 [5.2-14]:

$$M_x = \sum_{i=x}^{n} F_i (h_i - h_x)$$

The application of these equations for the three alternative building frames is shown in Tables 5.2-1, 5.2-2, and 5.1-3.

Table 5.2-1 Alternative A, Moment Frame Seismic Forces and Moments by Level

Level (x)	W_x (kips)	h_x (ft)	$W_x h_x^k$ (ft-kips)	C_{vx}	F_x (kips)	V_x (kips)	M_x (ft-kips)
PH Roof	215	118.33	117,200	0.03	32	32	514
Main roof	1,745	102.33	785,200	0.21	215	247	3,810
Story 7	2,235	89.00	836,500	0.23	229	476	10,160
Story 6	2,235	75.67	675,200	0.18	185	661	18,980
Story 5	2,235	62.33	522,700	0.14	143	805	29,710
Story 4	2,235	49.00	380,500	0.10	104	909	41,830
Story 3	2,235	35.67	250,200	0.07	69	977	54,870
Story 2	2,235	22.33	134,800	0.04	37	1,014	77,520
Σ	15,370		3,702,500	1.00	1,014		

1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m.

Table 5.2-2 Alternative B, Braced Frame Seismic Forces and Moments by Level

Level (x)	W_x (kips)	h_x (ft)	$W_x h_x^k$ (ft-kips)	C_{vx}	F_x (kips)	V_x (kips)	M_x (ft-kips)
PH Roof	215	118.33	35,500	0.03	67	67	1,070
Main roof	1,745	102.33	246,900	0.19	463	530	8,130
Story 7	2,235	89.00	272,300	0.21	511	1,041	22,010
Story 6	2,235	75.67	228,900	0.18	430	1,470	41,620
Story 5	2,235	62.33	186,000	0.15	349	1,819	65,870
Story 4	2,235	49.00	143,800	0.11	270	2,089	93,720
Story 3	2,235	35.67	102,400	0.08	192	2,281	124,160
Story 2	2,235	22.33	62,000	0.05	116	2,398	177,720
Σ	15,370		1,278,000	1.00	2,398		

1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m.

Table 5.2-3 Alternative C, Dual System Seismic Forces and Moments by Level

Level (x)	W_x (kips)	h_x (ft)	$W_x h_x^k$ (ft-kips)	C_{vx}	F_x (kips)	V_x (kips)	M_x (ft-kips)
PH Roof	215	118.33	35,500	0.03	50	50	800
Main roof	1,745	102.33	246,900	0.19	347	397	6,100
Story 7	2,235	89.00	272,350	0.21	383	781	16,500
Story 6	2,235	75.67	228,900	0.18	322	1,103	31,220
Story 5	2,235	62.33	186,000	0.15	262	1,365	49,400
Story 4	2,235	49.00	143,800	0.11	202	1,567	70,290
Story 3	2,235	35.67	102,386	0.08	144	1,711	93,120
Story 2	2,235	22.33	62,000	0.05	87	1,798	133,270
Σ	15,370		1,278,000	1.00	1,798		

1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m.

Be sure to note that the seismic mass at any given level, which includes the lower half of the wall above that level and the upper half of the wall below that level, produces the shear applied at that level and that shear produces the moment which is applied at the top of the next level down. Resisting the overturning moment is the weight of the building above that level combined with the moment resistance of the framing at that level. Note that the story overturning moment is applied to the level below the level that receives the story shear. (This is illustrated in Figure 9.2-4 in the masonry examples.)

5.2.4.3 Size Members

At this point we are ready to select the sizes of the framing members. The method for each alternative is summarized below.

Alternative A, Special Moment Frame:

1. Select preliminary member sizes
2. Check deflection and drift (*Provisions* Sec. 5.2.8 [5.4.1])
3. Check torsional amplification (*Provisions* Sec. 5.4.4.1.3 [5.2.4.3])
4. Check the column-beam moment ratio rule (AISC Seismic Sec. 9.6)
5. Check shear requirement at panel-zone (AISC Seismic Sec. 9.3; FEMA 350 Sec. 3.3.3.2)
6. Check redundancy (*Provisions* Sec. 5.2.4.2 [5.3.3])
7. Check strength

Reproportion member sizes as necessary after each check. The most significant criteria for the design are drift limits, relative strengths of columns and beams, and the panel-zone shear.

Alternative B, Special Concentrically Braced Frame:

1. Select preliminary member sizes
2. Check strength
3. Check drift (*Provisions* Sec. 5.2.8 [4.5.1])
4. Check torsional amplification (*Provisions* Sec. 5.4.4.1 [5.2.4.3])
5. Check redundancy (*Provisions* Sec. 5.2.4.2 [4.3.3])

Reproportion member sizes as necessary after each check. The most significant criteria for this design is torsional amplification.

Alternative C, Dual System:

1. Select preliminary member sizes
2. Check strength of moment frame for 25 percent of story shear (*Provisions* Sec. 5.2.2.1 [4.3.1.1])
3. Check strength of braced frames
4. Check drift for total building (*Provisions* Sec. 5.2.8 [4.5.1])
5. Check torsional amplification (*Provisions* Sec. 5.4.4.1 [5.2.4.3])
6. Check redundancy (*Provisions* Sec. 5.2.4.2 [4.3.3])

Reproportion member sizes as necessary after each check.

5.2.4.3.1 Size Members for Alternative A, Moment Frame

1. Select Preliminary Member Sizes – The preliminary member sizes are shown for the moment frame in the X-direction (7 bays) in Figure 5.2-5 and in the Y direction (5 bays) in Figure 5.2-6.

   Check Local Stability – Check beam flange stability in accordance with AISC Seismic Table I-9-1 [I-8-1] (same as FEMA 350 Sec. 3.3.1.1) and beam web stability in accordance with AISC Seismic Table I-9-1 [I-8-1]. (FEMA 350 Sec.3.3.1.2.is more restrictive for cases with low $P_u/\phi_b P_y$, such as in this example.) Beam flange slenderness ratios are limited to $52/\sqrt{F_y}$ and beam web height-to-thickness ratios are limited to $418/\sqrt{F_y}$.

   [The terminology for local stability has been revised in the 2002 edition of AISC Seismic. The limiting slenderness ratios in AISC Seismic use the notation λ_{ps} ("seismically compact") to differentiate them from λ_p in AISC LRFD. In addition, the formulas appear different because the elastic modulus, E_s, has been added as a variable. Both of these changes are essentially editorial, but Table I-8-1 in the 2002 edition of AISC Seismic has also been expanded to include more elements than in the 1997 edition.]

   Be careful because certain shapes found in the AISC Manual will not be permitted for Grade 50 steel (but may have been permitted for Grade 36 steel) because of these restrictions. For Grade 50, *b/t* is limited to 7.35.

   Further note that for columns in special steel moment frames such as this example, AISC Seismic 9.4b [I-8-1] requires that when the column moment strength to beam moment strength ratio is less than or equal to 2.0, the more stringent λ_p requirements apply for *b/t*, and when $P_u/\phi_b P_y$ is less than or equal to 0.125, the more stringent *h/t* requirements apply.

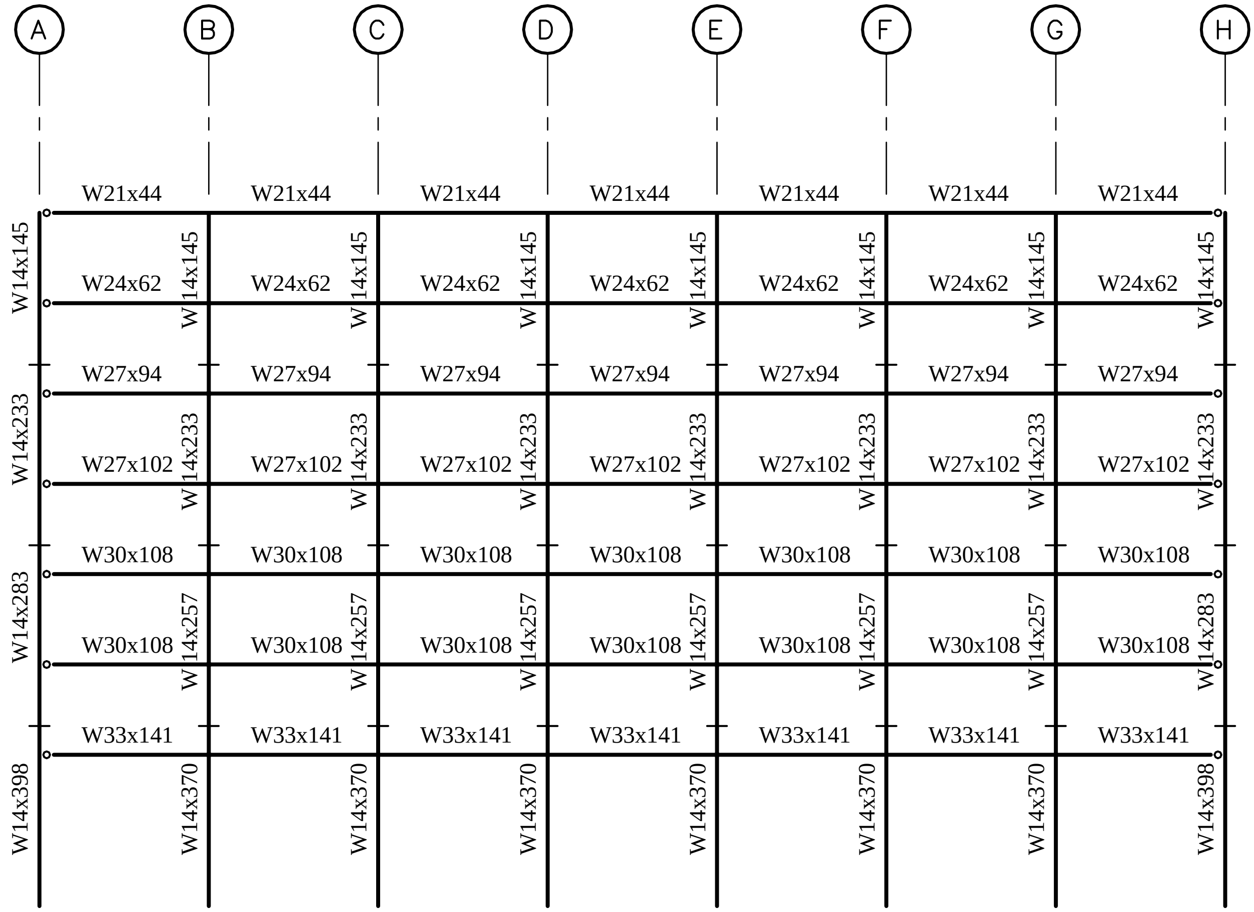


Figure 5.2-5 SMRF frame in E-W direction (penthouse not shown).

2. Check Drift – Check drift in accordance with *Provisions* Sec. 5.2.8 [4.5.1]. The building was modeled in 3-D using RAMFRAME. Displacements at the building centroid are used here because the building is not torsionally irregular (see the next paragraph regarding torsional amplification). Calculated story drifts and C_d amplification factors are summarized in Table 5.2-4. P-delta effects are included.

   All story drifts are within the allowable story drift limit of $0.020h_{sx}$ per *Provisions* Sec. 5.2.8 [4.5.1] and Sec. 5.2.3.6 of this chapter.

   As indicated below, the first story drift ratio is less than 130 percent of the story above (*Provisions* Sec. 5.2.3.3 [4.3.2.3]):

$$\frac{C_d \Delta_{x \text{ story } 2}}{C_d \Delta_{x \text{ story } 3}} = \frac{\left(\dfrac{5.17 \text{ in.}}{268 \text{ in.}}\right)}{\left(\dfrac{3.14 \text{ in.}}{160 \text{ in.}}\right)} = 0.98 < 1.30$$

   Therefore, there is no vertical irregularity.

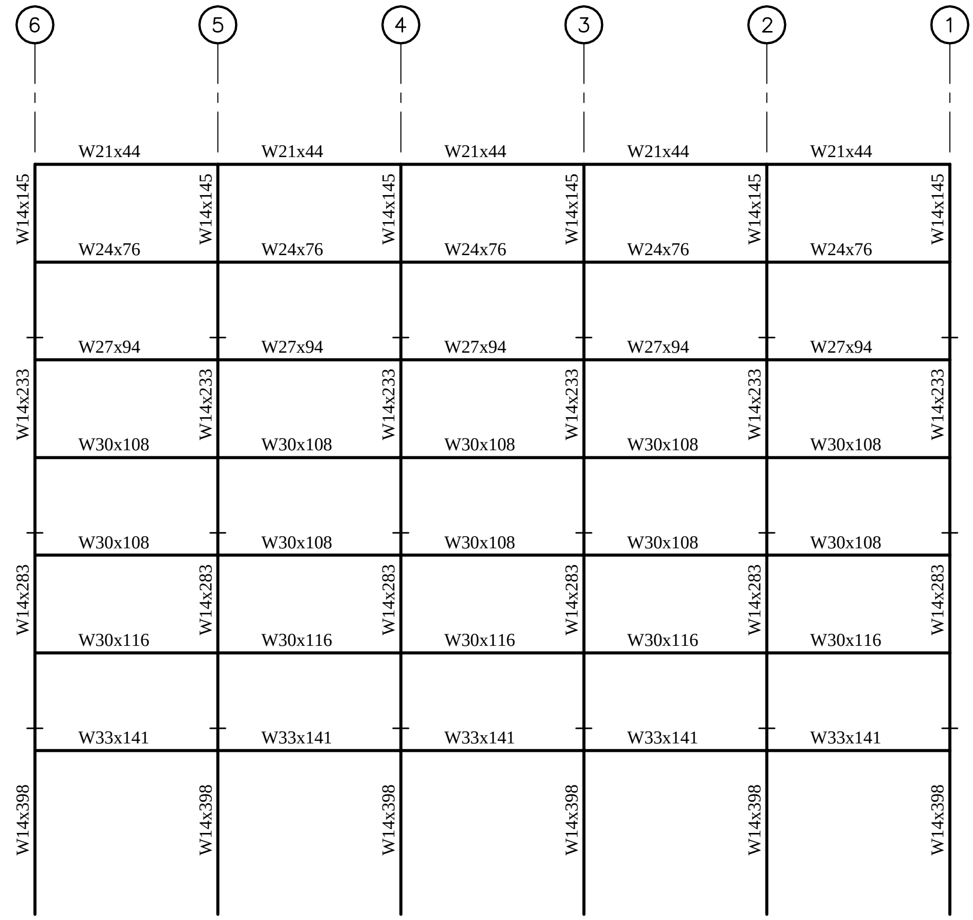


Figure 5.2-6 SMRF frame in N-S direction (penthouse not shown).

Table 5.2-4 Alternative A (Moment Frame) Story Drifts under Seismic Loads

	Total Displacement at Building Centroid (86.5, 62.5)		Story Drift from 3-D Elastic Analysis at Building Centroid		C_d	(C_d) x (Elastic Story Drift)		Allowable Story Drift
	δE-W (in.)	δN-S (in.)	ΔE-W (in.)	ΔN-S (in.)	5.5	ΔE-W (in.)	ΔN-S (in.)	Δ (in.)
Roof	4.24	4.24	0.48	0.47	5.5	2.64	2.59	3.20
Floor 7	3.76	3.77	0.57	0.58	5.5	3.14	3.19	3.20
Floor 6	3.19	3.19	0.54	0.53	5.5	2.97	2.92	3.20
Floor 5	2.65	2.66	0.57	0.58	5.5	3.14	3.19	3.20
Floor 4	2.08	2.08	0.57	0.58	5.5	3.14	3.19	3.20
Floor 3	1.51	1.50	0.57	0.57	5.5	3.14	3.14	3.20
Floor 2	0.94	0.93	0.94	0.93	5.5	5.17	5.12	5.36

1.0 in. = 25.4 mm.

3. Check Torsional Amplification – The torsional amplification factor per *Provisions* Eq. 5.4.4.1.3-1 [5.2-13] is:

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}} \right)^2$$

If $A_x < 1.0$, then torsional amplification need not be considered. It is readily seen that if the ratio of $\delta_{max}/\delta_{avg}$ is less that 1.2, then torsional amplification will not be necessary.

The 3-D analysis provided the story deflections listed in Table 5.2-5. Because none of the ratios for $\delta_{max}/\delta_{avg}$ exceed 1.2, torsional amplification of forces is not necessary for the moment frame alternative.

Table 5.2-5 Alternative A Torsional Analysis

	Torsion Checks			
	$\delta_{EW_{max}}$ (in.) (175,0)	$\delta_{NS_{max}}$ (in.) (125,0)	$\delta_{EW_{max}} / \delta_{EW_{avg}}$	$\delta_{NS_{max}} / \delta_{NS_{avg}}$
Roof	4.39	4.54	1.04	1.07
Story 7	3.89	4.04	1.04	1.07
Story 6	3.30	3.42	1.04	1.07
Story 5	2.75	2.85	1.03	1.07
Story 4	2.16	2.23	1.04	1.07
Story 3	1.57	1.62	1.04	1.08
Story 2	0.98	1.00	1.04	1.08

1.0 in. = 25.4 mm.

Member Design Considerations – Because $P_u/\phi P_n$ is typically less than 0.4 for the columns (re: AISC Seismic Sec. 8.2 [8.3]), combinations involving Ω_0 factors do not come into play for the special steel moment frames. In sizing columns (and beams) for strength we will satisfy the most severe value from interaction equations. However, this frame is controlled by drift. So, with both strength and drift requirements satisfied, we will check the column-beam moment ratio and the panel zone shear.

4. Check the Column-Beam Moment Ratio – Check the column-beam moment ratio per AISC Seismic Sec. 9.6. For purposes of this check, the plastic hinge was taken to occur at $0.5d_b$ from the face or the column in accordance with FEMA 350 for WUF-W connections (see below for description of these connections). The expected moment strength of the beams were projected from the plastic hinge location to the column centerline per the requirements of AISC Seismic Sec. 9.6. This is illustrated in Figure 5.2-7. For the columns, the moments at the location of the beam flanges is projected to the column-beam intersection as shown in Figure 5.2-8.

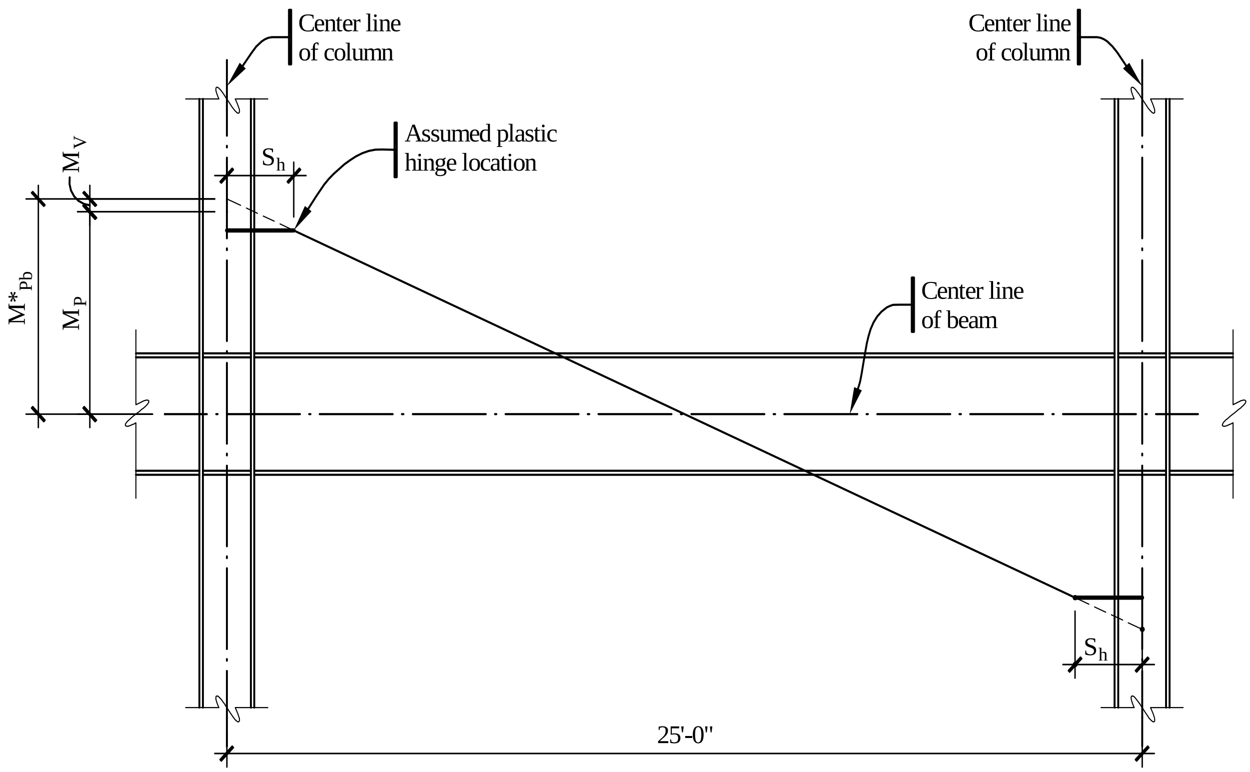


Figure 5.2-7 Projection of expected moment strength of beam (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

The column-beam strength ratio calculation is illustrated for the lower level in the E-W direction, Level 2, at gridline G (W14×370 column and W33×141 beam). For the columns:

$$\Sigma M^*_{pc} = \Sigma Z_c \left(F_{yc} - \frac{P_{uc}}{A_g} \right)$$

$$\Sigma M^*_{pc} = 2\left[736 \text{ in.}^3 \left(50 \text{ ksi} - \frac{500 \text{ kips}}{109 \text{ in.}^2} \right) \right] = 66{,}850 \text{ ft-kips}$$

Adjust this by the ratio of average story height to average clear height between beams, or (268 + 160)/ (251.35 + 128.44) = 1.13. Therefore, ΣM^*_{pc} = (1.13)(66,850) = 75,300 ft-kips. For the beams,

$$\Sigma M^*_{pb} = \Sigma(1.1 R_y M_p + M_v)$$

where

R_y = 1.1 for Grade 50 steel
$M_p = F_y Z$ = (50) (514) = 25,700 in.-kips
$M_v = V_p S_h$
S_h = Distance from column centerline to plastic hinge = $d_c/2 + d_b/2$ = 25.61 in.
V_p = Shear at plastic hinge location

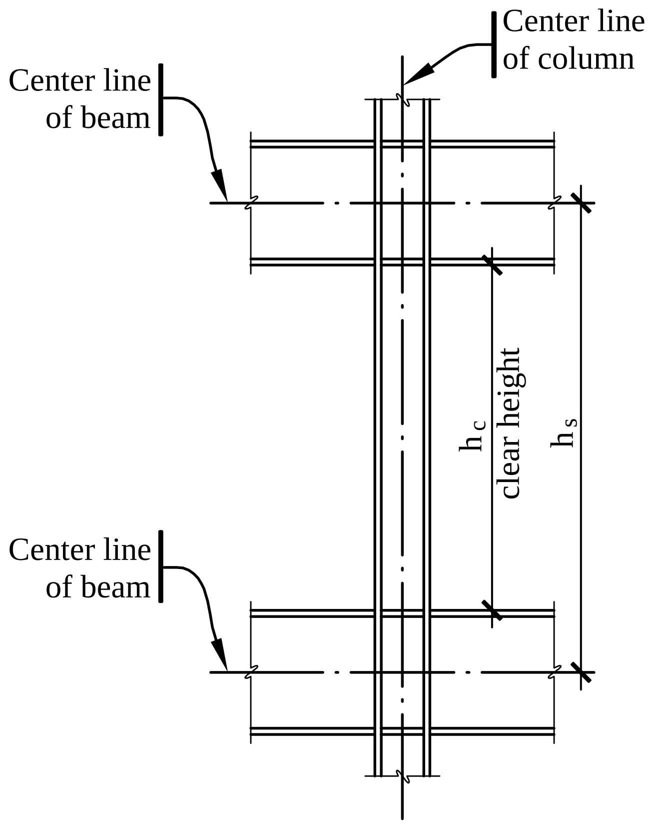


Figure 5.2-8 Story height and clear height.

The shear at the plastic hinge (Figure 5.2-9) is computed as:

$$V_p = [2M_p + (wL'^2/2] / L'$$

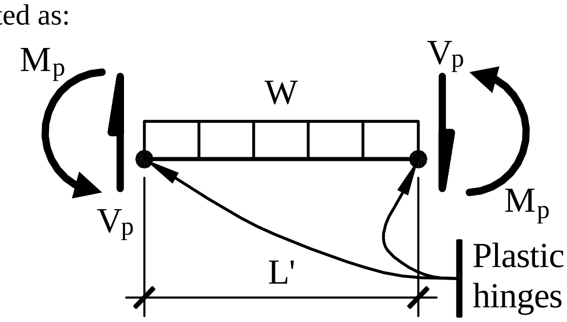


Figure 5.2-9 Free body diagram bounded by plastic hinges.

where

L' = Distance between plastic hinges = 248.8 in.
w = Factored uniform gravity load along beam
w = 1.4D + 0.5L = 1.4(0.0625 ksf)(12.5 ft)
+ 0.5(0.050 ksf)(12.5 ft) = 1.406 klf

Therefore,

$$V_p = \frac{2M_p + \dfrac{wL'^2}{2}}{L'} = \frac{(2)(25,700) + \left(\dfrac{(1.406)}{12}\dfrac{(248.8)^2}{2}\right)}{248.8} = 221.2 \text{ kips}$$

and

$$M_v = V_p S_h = (221.2)(25.61) = 5,665 \text{ in.-kips}$$

Finally, $\Sigma M^*_{pb} = \Sigma(1.1R_yM_p + M_v) = 2[(1.1)(1.1)(25,700) + 5,665] = 73,500$ in.-kips.

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

$$\text{Ratio} = \frac{\Sigma M^*_{pc}}{\Sigma M^*_{pb}} = \frac{76{,}900}{73{,}500} = 1.05 > 1.0 \qquad \text{OK}$$

The column-beam strength ratio for all the other stories is determined in a similar manner. They are summarized in Table 5.2-4 for the E-W direction (seven-bay) frame and in Table 5.2-5 for the N-S direction (five-bay) frame. All cases are acceptable because the column-beam moment ratios are all greater than 1.00.

Table 5.2-4 Column-Beam Moment Ratios for Seven-Bay Frame (N-S Direction)

Story	Member		ΣM^*_{pc} (in.-kips)	ΣM^*_{pb} (in.-kips)	Column-Beam Ratio
7	column beam	W14×145 W24×62	29,000	21,300	1.36
5	column beam	W14×233 W27×102	40,000	42,600	1.15
3	column beam	W14×257 W30×108	53,900	48,800	1.11
2	column beam	W14×370 W33×141	75,300	73,500	1.02

For levels with the same size column, the one with the larger beam size will govern; only these are shown. 1.0 in.-kip = 0.113 kN-m.

Table 5.2-5 Column-Beam Moment Ratios for Five-Bay Frame (N-S Direction)

Story	Member		ΣM^*_{pc} (in.-kips)	ΣM^*_{pb} (in.-kips)	Column-Beam Ratio
7	column beam	W14×145 W24×76	29,400	27,700	1.06
5	column beam	W14×233 W30×108	50,700	48,700	1.04
3	column beam	W14×283 W30×116	63,100	53,900	1.17
2	column beam	W14×398 W33×141	85,900	74,100	1.16

For levels with the same size column, the one with the larger beam size will govern; only these are shown. 1.0 in.-kip = 0.113 kN-m.

5. Check Panel Zone – The *Provisions* defers to AISC Seismic for the panel zone shear calculation. Because the two methods for calculating panel zone shear (AISC Seismic and FEMA 350) differ, both are illustrated below.

AISC Seismic Method

Check the shear requirement at the panel zone in accordance with AISC Seismic Sec. 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

$$R_u = \Sigma \frac{M_f}{d_b - t_{fb}}$$

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 5.2-10).

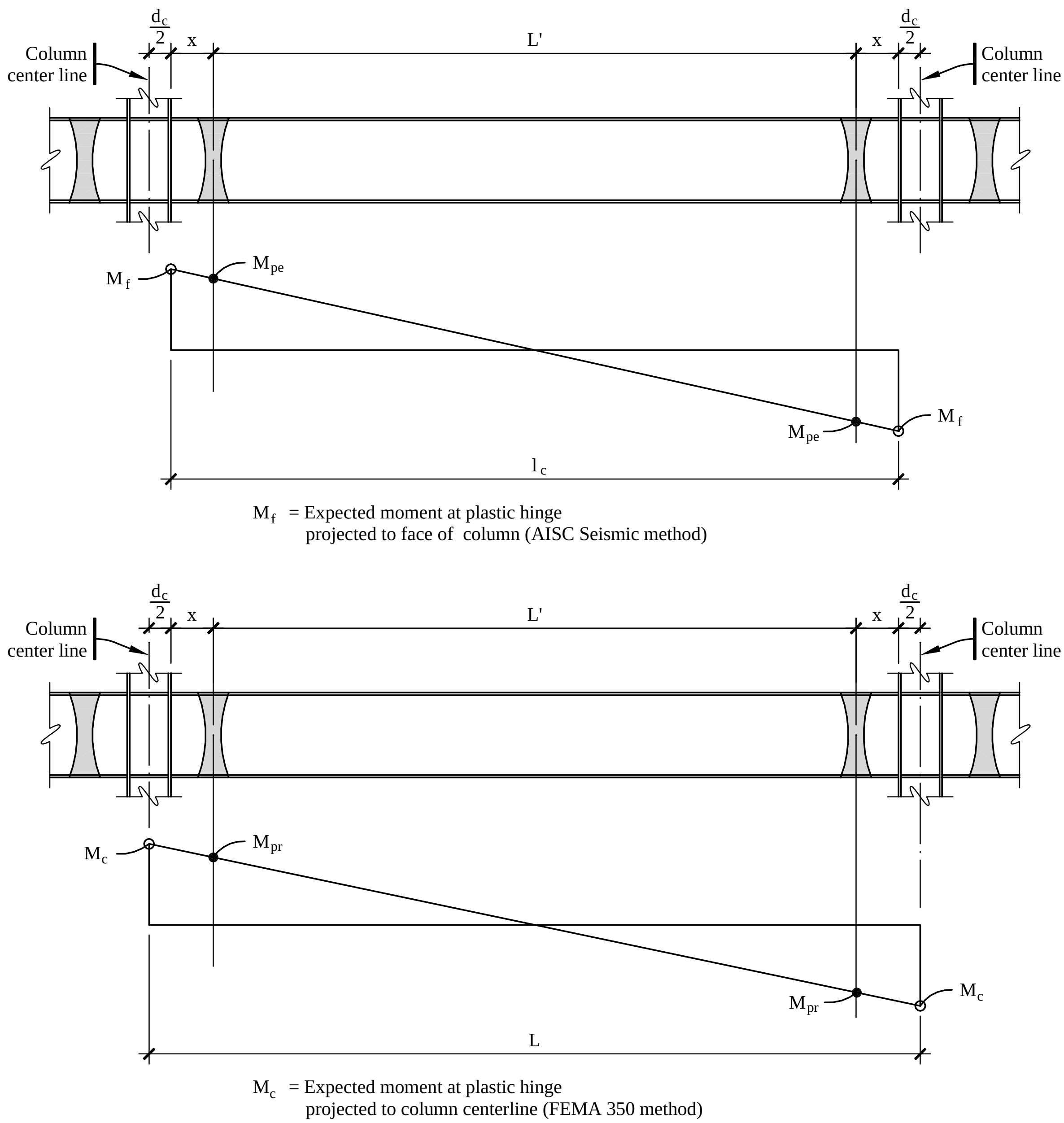


Figure 5.2-10 Illustration of AISC Seismic vs. FEMA 350 methods for panel zone shear.

For a column with equal beams of equal spans framing into opposite faces (such as on Grids C, D, E, F, 2, 3, 4, and 5), the effect of gravity loads offset, and

$$\Sigma M_f = 2R_y F_y Z_x \left[\frac{l_c}{l_c - 2x}\right]$$

where l_c = the clear span and x = distance from column face to plastic hinge location.

For Grids 1 and 6, only one beam frames into the column; at Grids B and G, the distance x is different on one side; at Grids A and H, there is no moment because the beams are pin-connected to the corner columns. For all these cases, the above relationship needs to be modified accordingly.

For W33×141 beams framing into each side of a W14×370 column (such as Level 2 at Grid F):

$$\Sigma M_f = (2)(1.1)(50)(514)\left[\frac{282.1}{282.1-(2)(16.55)}\right] = 64{,}056 \text{ in.-kips}$$

$$R_u = \frac{64{,}056}{33.30 - 0.96} = 1{,}981 \text{ kips}$$

The shear transmitted to the joint from the story above opposes the direction of R_u and may be used to reduce the demand. From analysis, this is 98 kips at this location. Thus the frame R_u = 1,981 - 98 = 1,883 kips.

The panel zone shear calculation for Story 2 of the frame in the E-W direction at Grid F (column: W14×370; beam: W33×141) is from AISC Seismic Eq. 9-1:

$$R_v = 0.6F_y d_c t_p \left[1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_p}\right]$$

$$R_v = (0.6)(50)(17.92)(t_p)\left[1 + \frac{(3)(16.475)(2.660)^2}{(33.30)(17.92)(t_p)}\right]$$

$$R_v = 537.6t_p\left[1 + \frac{0.586}{t_p}\right]$$

$$R_v = 537.6t_p + 315$$

The required total (web plus doubler plate) thickness is determined by:

$R_v = \phi R_u$

Therefore, $537.6t_p + 315 = (1.0)(1883)$ and $t_p = 2.91$ in.

Because the column web thickness is 1.655 in., the required doubler plate thickness is 1.26 in. Use a plate thickness of 1-1/4 in.

Both the column web thickness and the doubler plate thickness are checked for shear buckling during inelastic deformations by AISC Seismic Eq. 9-2. If necessary, the doubler plate may be plug-welded to the column web as indicated by AISC Seismic Commentary Figure C-9.2. For this case, the minimum individual thickness as limited by local buckling is:

$$t \geq (d_z + w_z)/90$$

$$t \geq \frac{(31.38+12.6)}{90} = 0.49 \text{ in.}$$

Because both the column web thickness and the doubler plate thicknesses are greater than 0.49 in., plug welding of the doubler plate to the column web is not necessary.

In the case of thick doubler plates, to avoid thick welds, two doubler plates (each of half the required thickness) may be used, one on each side of the column web. For such cases, buckling also must be checked using AISC Seismic Eq. 9-2 as doubler plate buckling would be a greater concern. Also, the detailing of connections that may be attached to the (thinner) doubler plate on the side of the weld needs to be carefully reviewed for secondary effects such as undesirable out-of-plane bending or prying.

FEMA 350 Method

For the FEMA 350 method, see FEMA 350 Sec. 3.3.3.2, "Panel Zone Strength," to determine the required total panel zone thickness (t):

$$t = \sum \frac{C_y M_c \left[\dfrac{h-d_b}{h}\right]}{(0.9)(0.6)F_{yc}R_{yc}d_c(d_b - t_{fb})}$$

(Please note the Σ; its omission from FEMA 350 Eq. 3-7 is an inadvertent typographical error.)

The term M_c refers to the expected beam moment projected to the centerline of the column; whereas AISC Seismic uses the expected beam moment projected to the face of the column flange. (This difference is illustrated in Figure 5.2-10.) The term $\left[\dfrac{h-d_b}{h}\right]$ is an adjustment similar to reducing R_u by the direct shear in the column, where h is the average story height. C_y is a factor that adjusts the force on the panel down to the level at which the beam begins to yield in flexure (see FEMA 350 Sec. 3.2.7) and is computed from FEMA 350 Eq. 3-4:

$$C_y = \frac{1}{C_{pr}\dfrac{Z_{be}}{S_b}}$$

C_{pr}, a factor accounting for the peak connection strength, includes the effects of strain hardening and local restraint, among others (see FEMA 350 Sec. 3.2.4) and is computed from FEMA 350 Eq. 3-2:

$$C_{pr} = \frac{(F_y + F_u)}{2F_y}$$

For the case of a W33×141 beam and W14×370 column (same as used for the above AISC Seismic method), values for the variables are:

Distance from column centerline to plastic hinge, $S_h = d_c/2 + d_b/2 = 17.92/2 + 33.30/2 = 25.61$ in.

Span between plastic hinges, $L' = 25$ ft - 2(25.61 in.)/12 = 20.73 ft

$M_{pr} = C_{pr}R_yZ_eF_y$ (FEMA 350 Figure 3-4)
M_{pr} = (1.2)(1.1)(514)(50) = 33,924 in.-kips (FEMA 350, Figure 3-4)

$$V_p = \frac{\left[2M_{pr} + \left(\frac{wL'^2}{2}\right)\right]}{L'}$$

$$V_p = \frac{\left[(2)(33{,}924) + \left(\frac{(1.266)(20.73)^2}{(12)(2)}\right)\right]}{(20.73)(12)} = 273 \text{ kips}$$

$M_c = M_{pr} + V_p(x + d_c/2)$ (FEMA 350 Figure 3-4)

M_c = 33,924 + (273)(17.92/2 + 25.61/2) = 40,916 in.-kips

$$C_y = \frac{1}{C_{pr}\frac{Z_{be}}{S_b}} = \frac{1}{(1.2)\frac{514}{448}} = 0.73$$

Therefore,

$$t = 2\left[\frac{(0.73)(40{,}916)\left[\frac{(214)-(33.30)}{(214)}\right]}{(0.9)(0.6)(50)(1.1)(17.92)(33.30-0.96)}\right] = 2.93 \text{ in.}$$

The required doubler plate thickness is equal to $t - t_{cw}$ = 2.93 in. - 1.655 in. = 1.27 in. Thus, the doubler plate thickness for 1.27 in. by FEMA 350 is close to the thickness of 1.26 by AISC Seismic.

6. Check Redundancy – Return to the calculation of r_x for the moment frame. In accordance with *Provisions* Sec. 5.2.4.2 [not applicable in the 2003 *Provisions*], r_{max_x} is taken as the maximum of the sum of the shears in any two adjacent columns in the plane of a moment frame divided by the story shear. For columns common to two bays with moment resisting connections on opposite sides of the column at the level under consideration, 70 percent of the shear in that column may be used in the column shear summation (Figures 5.2-11 and 5.2-12).

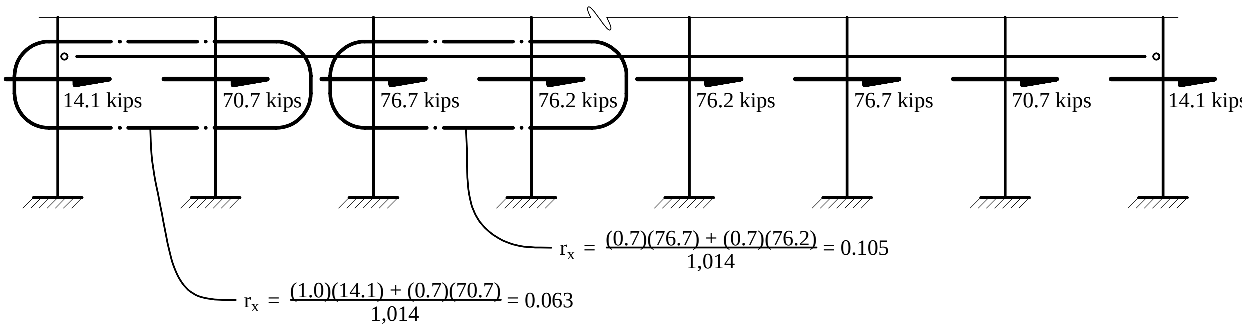


Figure 5.2-11 Column shears for E-W direction (partial elevation, Level 2) (1.0 kip = 4.45 kN).

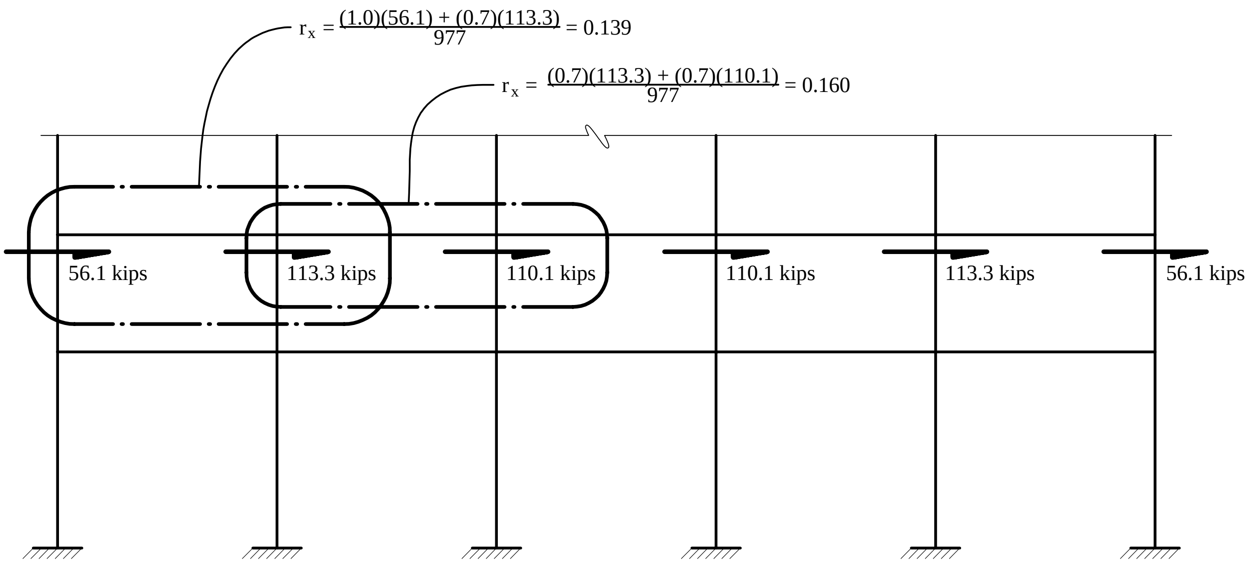


Figure 5.2-12 Column shears for N-S direction (partial elevation, Level 3) (1.0 kip = 4.45 kN).

For this example, r_x was computed for every column pair at every level in both directions. The shear carried by each column comes from the RAMFRAME analysis, which includes the effect of accidental torsion. Selected results are illustrated in the figures. The maximum value of r_{max_x} in the N-S direction is 0.160, and ρ is now determined using *Provisions* Eq.5.2.4.2 [not applicable in the 2003 *Provisions*]:

$$\rho = 2 - \frac{20}{r_{max_x}\sqrt{A_x}}$$

$$\rho = 2 - \frac{20}{0.160\sqrt{21{,}875 \text{ ft}^2}} = 1.15$$

Because 1.15 is less than the limit of 1.25 for special moment frames per the exception in the *Provisions* Sec. 5.2.4.2 [not applicable in the 2003 *Provisions*], use ρ =1.15. (If ρ > 1.25, then the framing would have to be reconfigured until ρ < 1.25.)

In the E-W direction, r_{max_x} = 0.105 and ρ = 0.71, which is less than 1.00, so use ρ = 1.00. All design force effects (axial force, shear, moment) obtained from analysis must be increased by the ρ factors. (However, drift controls the design in this example. Drift and deflections are not subject to the ρ factor.)

7. Connection Design – One beam-to-column connection for the moment-resisting frame is now designed to illustrate the FEMA 350 method for a prequalified connection. The welded unreinforced flanges-welded web (WUF-W) connection is selected because it is prequalified for special moment frames with members of the size used in this example. FEMA 350 Sec. 3.5.2 notes that the WUF-W connection can perform reliably provided all the limitations are met and the quality assurance requirements are satisfied. While the discussion of the design procedure below considers design requirements, remember that the quality assurance requirements are a vital part of the total requirements and must be enforced.

   Figure 5.2-13 illustrates the forces at the beam-to-column connection.

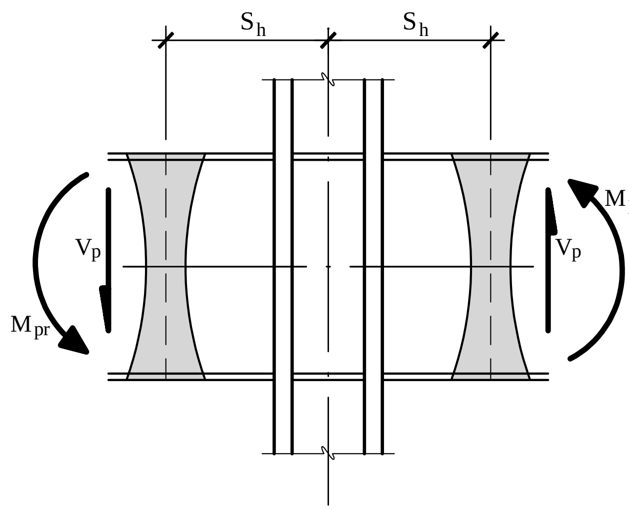


Figure 5.2-13 Forces at beam/column connection.

   First review FEMA 350 Table 3-3 for prequalification data. Our case of a W36×135 beam connected to a W14×398 column meets all of these. (Of course, here the panel zone strength requirement is from FEMA 350, not the AISC Seismic method.)

   The connection, shown in Figure 5.2-14, is based on the general design shown in FEMA 350 Figure 3-8. The design procedure outlined in FEMA 350 Sec. 3.5.2.1 for this application is reviewed below. All other beam-to-column connections in the moment frame will be similar.

   The procedure outlined above for the FEMA 350 method for panel zone shear is repeated here to determine S_h, M_{pr}, V_p, M_c , C_y and the required panel zone thickness.

   Continuity plates are required in accordance with FEMA 350 Sec. 3.3.3.1:

$$t_{cf} < 0.4\sqrt{1.8 b_f t_f \frac{F_{yb} R_{yb}}{F_{yc} R_{yc}}}$$

$$t_{cf} < 0.4\sqrt{(1.8)(11.950)(0.790)\frac{(50)(1.1)}{(50)(1.1)}} = 1.65 \text{ in. required}$$

$$t_{cf} = 2.845 \text{ in.} > \text{actual} \qquad \text{OK}$$

Therefore, continuity plates are not necessary at this connection because the column flange is so thick. But we will provide them anyway to illustrate continuity plates in the example. At a minimum, continuity plates should be at least as thick as the beam flanges. Provide continuity plates of 7/8 in. thickness, which is thicker than the beam flange of 0.79 in.

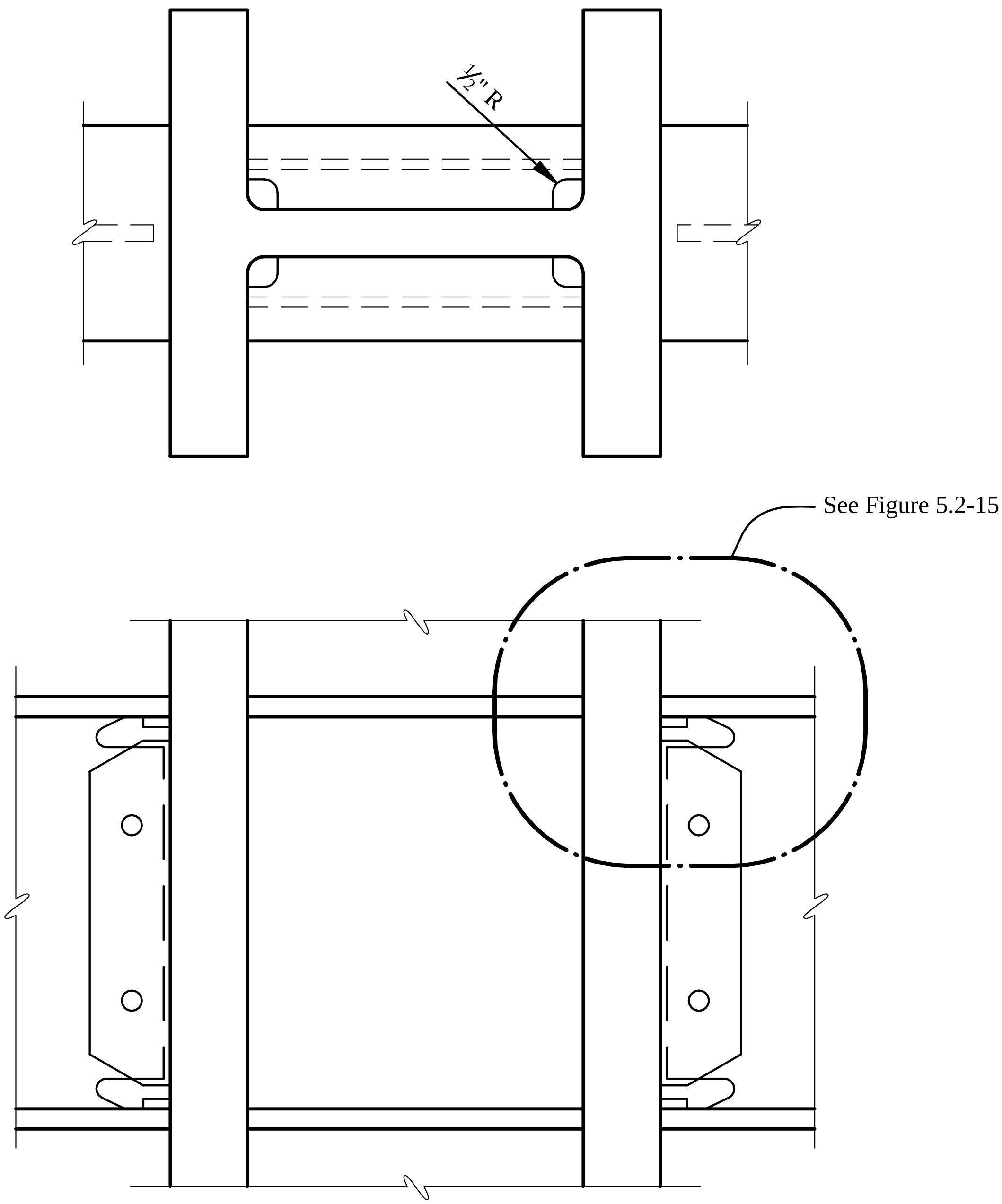


Figure 5.2-14 WUF-W connection, Second level, NS-direction (1.0 in. = 25.4 mm).

Check AISC LRFD K1.9:

Width of stiffener $+\dfrac{t_{cw}}{2} \geq \dfrac{b_{bf}}{3}$

$\left(5+\dfrac{1.77}{2}\right) = 5.88 \text{ in.} > 3.98 \text{ in.} = \dfrac{11.950}{3}$ OK

$t_{stiffener} \geq \dfrac{b_f}{2}$

$0.875 \text{ in.} > 0.395 \text{ in.} = \dfrac{0.79}{2}$ OK

$t_{stiffener} > w_{stiffener}\dfrac{\sqrt{F_y}}{95}$

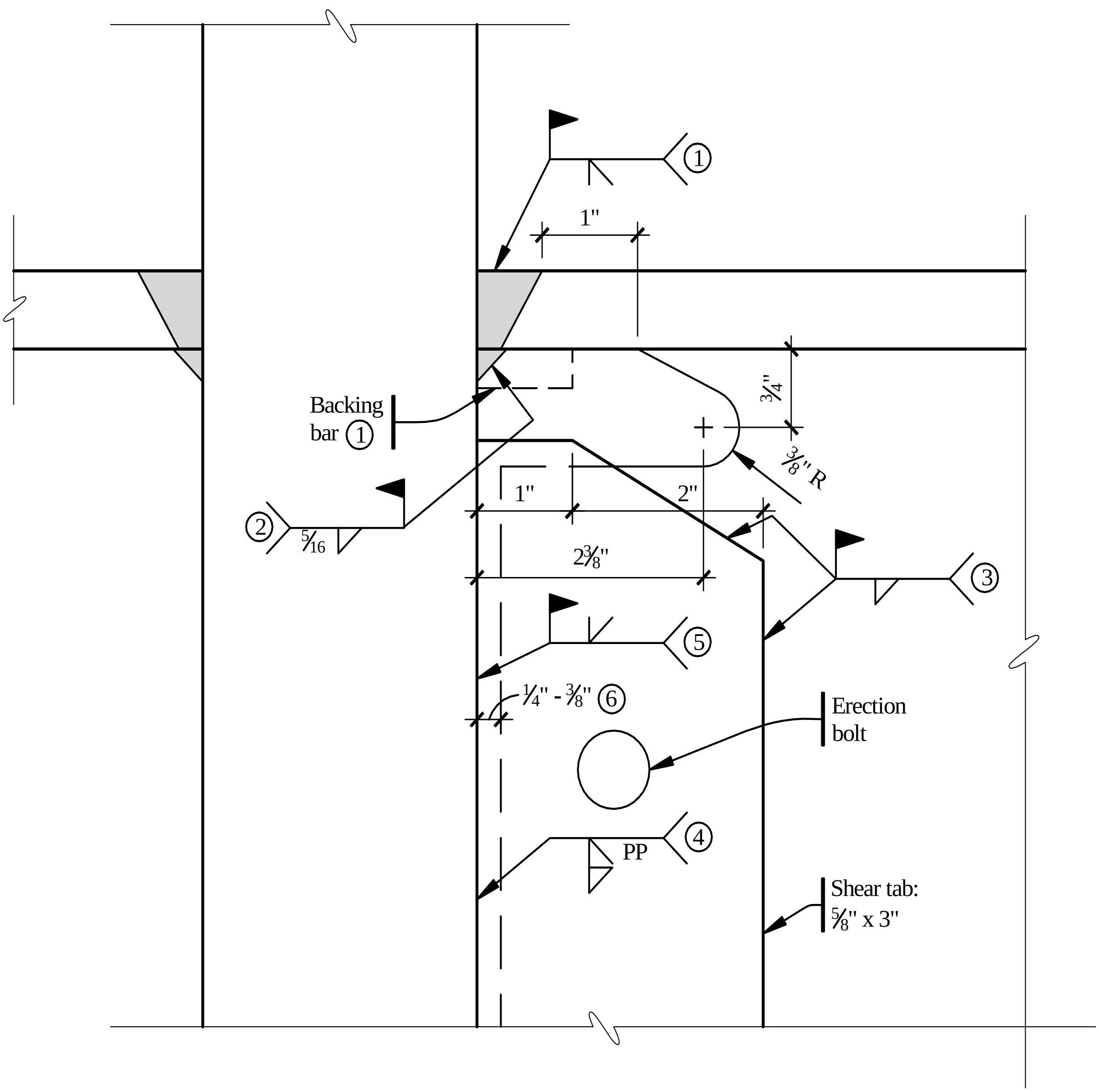


Figure 5.2-15 WUF-W weld detail (1.0 in. = 25.4 mm).

$$0.875 \text{ in.} > 0.37 \text{ in.} = (5)\left(\frac{\sqrt{50}}{95}\right) \qquad \text{OK}$$

The details shown in Figures 5.2-14 and 5.2-15 conform to the requirements of FEMA 350 for a WUF-W connection in a special moment frame.

Notes for Figure 5.2-15 (indicated by circles in the figure) are:

1. CJP groove weld at top and bottom flanges, made with backing bar.
2. Remove backing bar, backgouge, and add fillet weld.
3. Fillet weld shear tab to beam web. Weld size shall be equal to thickness of shear tab minus 1/16 in. Weld shall extend over the top and bottom third of the shear tab height and extend across the top and bottom of the shear tab.
4. Full depth partial penetration weld from far side. Then fillet weld from near side. These are shop welds of shear tab to column.
5. CJP groove weld full length between weld access holes. Provide non-fusible weld tabs, which shall be removed after welding. Grind end of weld smooth at weld access holes.
6. Root opening between beam web and column prior to starting weld 5.

See also FEMA 350 Figure 3-8 for more elaboration on the welds.

5.2.4.3.2 Size Members for Alternative B, Braced Frame

1. Select Preliminary Member Sizes – The preliminary member sizes are shown for the braced frame in the E-W direction (seven bays) in Figure 5.2-16 and in the N-S direction (five bays) in Figure 5.2-17. The arrangement is dictated by architectural considerations regarding doorways into the stairwells.

2. Check Strength – First, check slenderness and width-to-thickness ratios – the geometrical requirements for local stability. In accordance with AISC Seismic Sec. 13.2, bracing members must satisfy

$$\frac{kl}{r} \le \frac{1000}{\sqrt{F_y}} = \frac{1000}{\sqrt{50}} = 141$$

The columns are all relatively heavy shapes, so *kl/r* is assumed to be acceptable and is not examined in this example.

Wide flange members and channels must comply with the width-to-thickness ratios contained in AISC Seismic Table I-9-1 [I-8-1]. Flanges must satisfy:

$$\frac{b}{2t} \le \frac{52}{\sqrt{F_y}} = \frac{52}{\sqrt{50}} = 7.35$$

Webs in combined flexural and axial compression (where $P_u/\phi_b P_y < 0.125$, which is the case in this example) must satisfy:

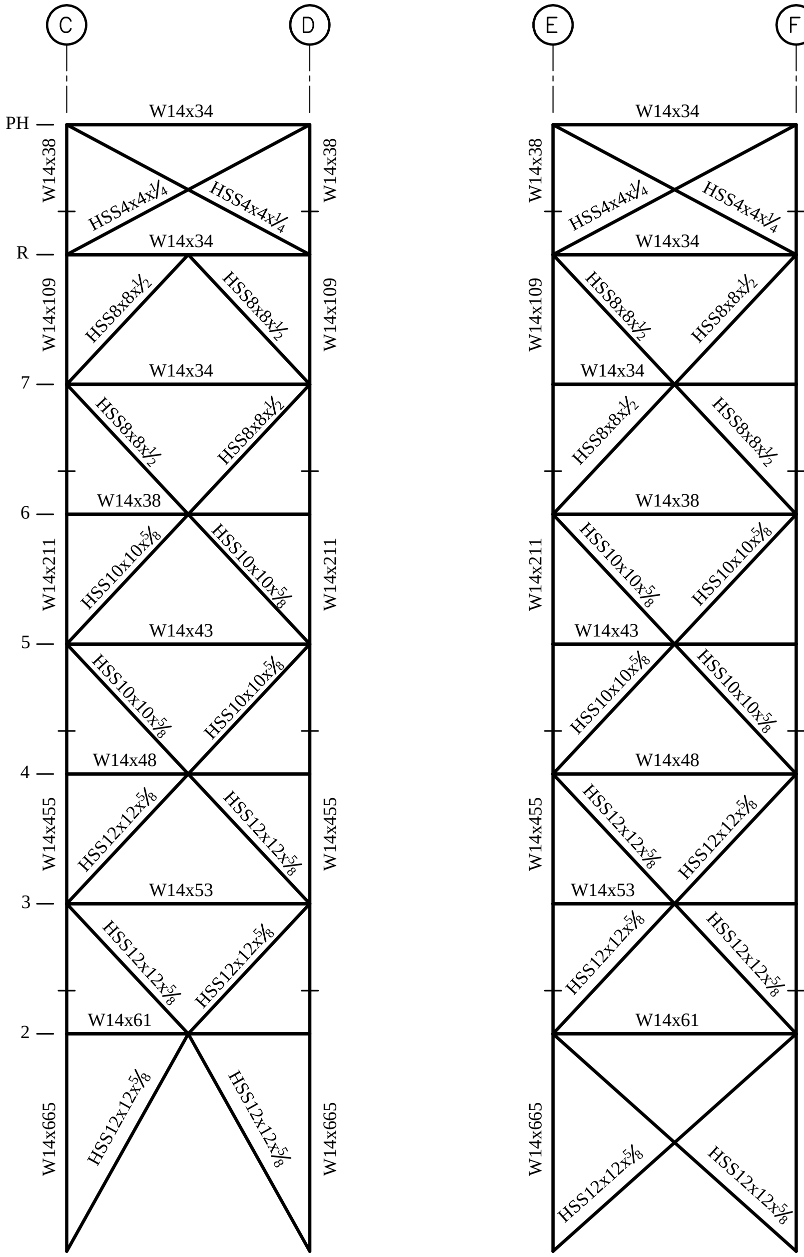


Figure 5.2-16 Braced frame in E-W direction.

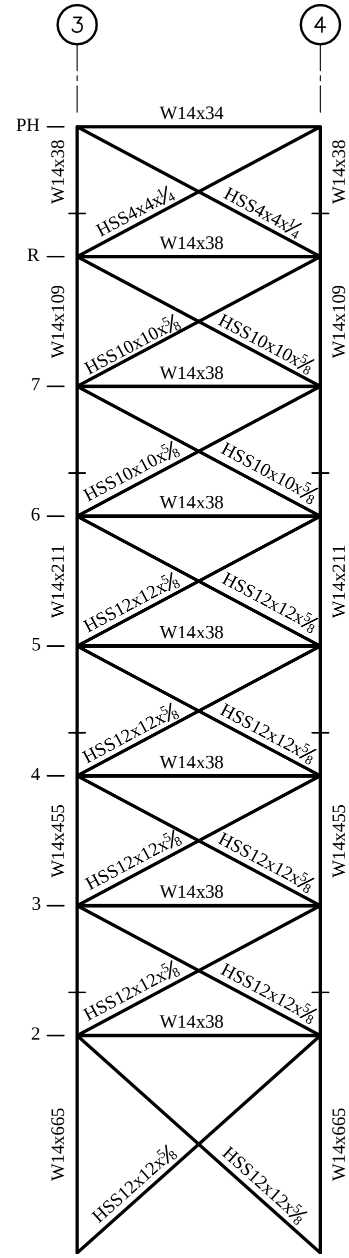


Figure 5.2-17 Braced frame in N-S direction.

$$\frac{h_c}{t_w} \leq \frac{520}{\sqrt{F_y}}\left[1-1.54\frac{P_u}{\phi_b P_y}\right]$$

Rectangular HSS members must satisfy:

$$\frac{b}{t} \leq \frac{110}{\sqrt{F_y}} = \frac{110}{\sqrt{46}} = 16.2$$

Selected members are checked below:

W14×38: $b/2t = 6.6 < 7.35$ OK

W14×34: $b/2t = 7.4 > 7.35$, but is acceptable for this example. Note that the W14×34 is at the penthouse roof, which is barely significant for this braced frame.

HSS12×12×5/8: $\dfrac{kl}{r} = \dfrac{(1)\left(\dfrac{28.33 \times 12}{2}\right)}{4.62} = 36.8 < 141$ OK

$$\frac{b}{t} = \frac{9.4}{0.581} = 16.17 < 16.2$$ OK

Also note that t for the HSS is actual, not nominal. The corner radius of HSS varies somewhat, which affects the dimension b. The value of b used here, 9.40 in., depends on a corner radius slightly larger than $2t$, and it would have to be specified for this tube to meet the b/t limit.

3. Check Drift – Check drift in accordance with *Provisions* Sec. 5.2.8 [4.5]. The building was modeled in 3-D using RAMFRAME. Maximum displacements at the building corners are used here because the building is torsionally irregular. Displacements at the building centroid are also calculated because these will be the average between the maximum at one corner and the minimum at the diagonally opposite corner. Use of the displacements at the centroid as the average displacements is valid for a symmetrical building. Calculated story displacements are used to determine A_x, the torsional amplification factor. This is summarized in Table 5.2-6. P-delta effects are included.

Table 5.2-6 Alternative B Amplification of Accidental Torsion

	Average Elastic Displacement = Displacement at Building Centroid (in.)		Maximum Elastic Displacement at Building Corner* (in.)		$\frac{\delta_{max}}{\delta_{avg}}$ **		Torsional Amplification Factor = $A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2$		Amplified Eccentricity = A_x(0.05 *L*)*** (*ft*)	
	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S
R	2.38	2.08	3.03	3.37	1.28	1.62	1.13	1.82	7.08	15.95
7	2.04	1.79	2.62	2.93	1.29	1.64	1.15	1.88	7.20	16.41
6	1.65	1.47	2.15	2.44	1.30	1.67	1.18	1.93	7.37	16.86
5	1.30	1.16	1.70	1.96	1.32	1.69	1.2	1.99	7.52	17.41
4	0.95	0.86	1.27	1.48	1.33	1.72	1.23	2.06	7.71	17.99
3	0.66	0.59	0.89	1.03	1.34	1.75	1.25	2.14	7.80	18.70
2	0.39	0.34	0.53	0.60	1.35	1.79	1.26	2.23	7.89	19.57

* These values are taken directly from the analysis. Accidental torsion is not amplified here.
** Amplification of accidental torsion is required because this term is greater than 1.2 (*Provisions* Table 5.2.3.2 Item 1a [4.3-2, Item 1a]). The building is *torsionally irregular* in plan. *Provisions* Table 5.2.5.1 [4.4-1] indicates that an ELF analysis is “not permitted” for torsionally irregular structures. However, given rigid diaphragms and symmetry about both axes, a modal analysis will not give any difference in results than an ELF analysis insofar as accidental torsion is concerned unless one arbitrarily offsets the center of mass. The *Provisions* does not require an arbitrary offset for center of mass in dynamic analysis nor is it common practice to do so. One reason for this is that the computed period of vibration would lengthen, which, in turn, would reduce the overall seismic demand. See Sec. 9.2 and 9.3 of this volume of design examples for a more detailed examination of this issue.
*** The initial eccentricities of 0.05 in the E-W and N-S directions are multiplied by A_x to determine the amplified eccentricities. These will be used in the next round of analysis.
1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m.

4. Check Torsional Amplification – A second RAMFRAME 3-D analysis was made, using the amplified eccentricity for accidental torsion instead of merely 0.05L for accidental torsion. The results are summarized in Table 5.2-7.

Table 5.2-7 Alternative B Story Drifts under Seismic Load

	Max. Elastic Displacement at Building Corners (in.)		Elastic Story Drift at Location of Max. Displacement (at corners) (in.)		C_d	(C_d) × (Elastic Story Drift) (in.)		Allowable Story Drift (in.)
	E-W	N-S	ΔE-W	ΔN-S		ΔE-W	ΔN-S	Δ
R	3.14	4.50	0.42	0.55	5	2.10	2.75	3.20
7	2.72	3.95	0.49	0.64	5	2.46	3.19	3.20
6	2.23	3.32	0.45	0.63	5	2.27	3.16	3.20
5	1.77	2.68	0.45	0.64	5	2.25	3.18	3.20
4	1.32	2.05	0.40	0.61	5	1.98	3.07	3.20
3	0.93	1.43	0.38	0.59	5	1.89	2.93	3.20
2	0.55	0.85	0.55	0.85	5	2.75	4.24	5.36

1.0 in. = 25.4 mm

All story drifts are within the allowable story drift limit of $0.020h_{sx}$ in accordance with *Provisions* Sec. 5.2.8 [4.5-1] and the allowable deflections for this building from Sec. 5.2.3.6 above. This a good point to reflect on the impact of accidental torsion and its amplification on the design of this core-braced structure. The sizes of members were increased substantially to bring the drift within the limits (note how close the N-S direction drifts are). For the final structure, the elastic displacements at the main roof are:

At the centroid = 2.08 in.
At the corner with accidental torsion = 3.37 in.
At the corner with amplified accidental torsion = 4.50 in.

The two effects of torsional irregularity (in this case, it would more properly be called torsional flexibility) of amplifying the accidental torsion and checking the drift limits at the corners combine to create a demand for substantially more stiffness in the structure. Even though many braced frames are controlled by strength, this is an example of how the *Provisions* places significant stiffness demands on some braced structures.

5. Check Redundancy – Now return to the calculation of r_x for the braced frame. Per *Provisions* Sec. 5.2.4.2 [not applicable in the 2003 *Provisions*], r_{max_x} for braced frames is taken as the lateral force component in the most heavily loaded brace element divided by the story shear (Figure 5.2-18).

A value for r_x was determined for every brace element at every level in both directions. The lateral component carried by each brace element comes from the RAMFRAME analysis, which includes the effect of amplified accidental torsion. Selected results are illustrated in the figures. The maximum r_x was found to be 0.223 below Level 7 in the NS-direction. The reliability factor (ρ) is now determined using *Provisions* Eq. 5.2.4.2 [not applicable in the 2003 *Provisions*]:

$$\rho = 2 - \frac{20}{r_{max_x}\sqrt{A_x}} = 2 - \frac{20}{0.223\sqrt{21{,}875\ \text{ft}^2}} = 1.39$$

In the N-S direction, all design force effects (axial forces, shears, moments) obtained from analysis must be increased by the ρ factor of 1.39. Similarly, for the E-W-direction, r_{max_x} and ρ are found to be 0.192 and 1.26, respectively. (However drift controls the design for this problem. Drift and deflection are not subject to the ρ factor.)

[See Sec. 5.2.3.2 for a discussion of the significant changes to the redundancy requirements in the 2003 *Provisions*.]

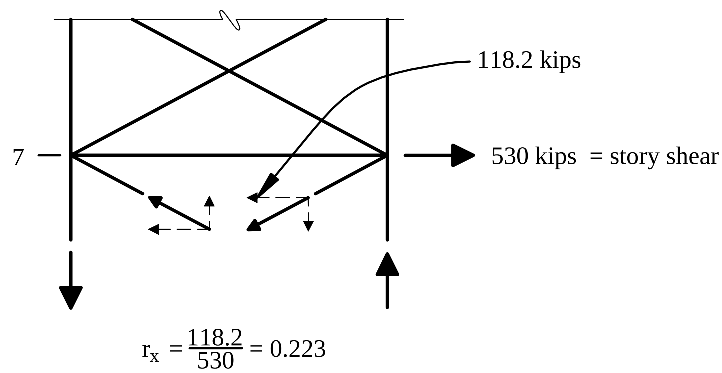


Figure 5.2-18 Lateral force component in braces for N-S direction – partial elevation, Level 7 (1.0 kip = 4.45 kN).

6. Braced Frame Member Design Considerations – The design of members in the special concentrically braced frame (SCBF) needs to satisfy AISC Seismic Sec. 13 and columns also need to satisfy AISC Seismic Sec. 8. When $P_u/\phi P_n$ is greater than 0.4, as is the predominant case here, the required axial strength needs to be determined from AISC Seismic Eq. 4-1 and 4-2 [*Provisions* Eq. 4.2-3 and 4.2-4]. These equations are for load combinations that include the Ω_0, or overstrength, factors. Moments are generally small for the braced frame so load combinations with Ω_0 can control column size for strength considerations but, for this building, drift controls because of amplified accidental torsion. Note that ρ is not used where Ω_0 is used (see *Provisions* Sec. 5.2.7 [4.2.2.2]).

   Bracing members have special requirements as well, although Ω_0 factors do not apply to braces in a SCBF. Note in particular AISC Seismic Sec. 13.2c, which requires that both the compression brace and the tension brace share the force at each level (as opposed to the "tension only" braces of Example 5.1). AISC Seismic Sec. 13.2 also stipulates a *kl/r* limitation and local buckling (width-thickness) ratio limits.

   Beams in many configurations of braced frames have small moments and forces, which is the case here. V and inverted V (chevron) configurations are an exception to this. There is a panel of chevron bracing at the top story of one of the braced frames (Figure 5.2-16). The requirements of AISC Seismic Sec. 13.4 should be checked although, in this case, certain limitations of AISC Seismic do not apply because the beam is at the top story of a building. (The level above in Figure 5.2-16 is a minor penthouse that is not considered to be a story.) If the chevron bay were not at the top story, the size of the braces must be known in order to design the beam. The load combination for the beam is modified using a Q_b factor defined in AISC Seismic Sec. 13.4a. Basically, the beam must be able to carry a concentrated load equal to the difference in vertical force between the post-buckling strength of the compression brace and the yield strength of the tension brace (i.e., the compression brace has buckled, but the tension brace has not yet yielded). The prescribed load effect is to use $0.3\phi_c P_n$ for the compression brace and P_y for the tension brace in order to determine a design vertical force to be applied to the beam.

7. Connection Design – Figure 5.2-19 illustrates a typical connection design at a column per AISC Seismic Sec. 13. First, check the slenderness and width-to-thickness ratios (see above). The bracing members satisfy these checks.

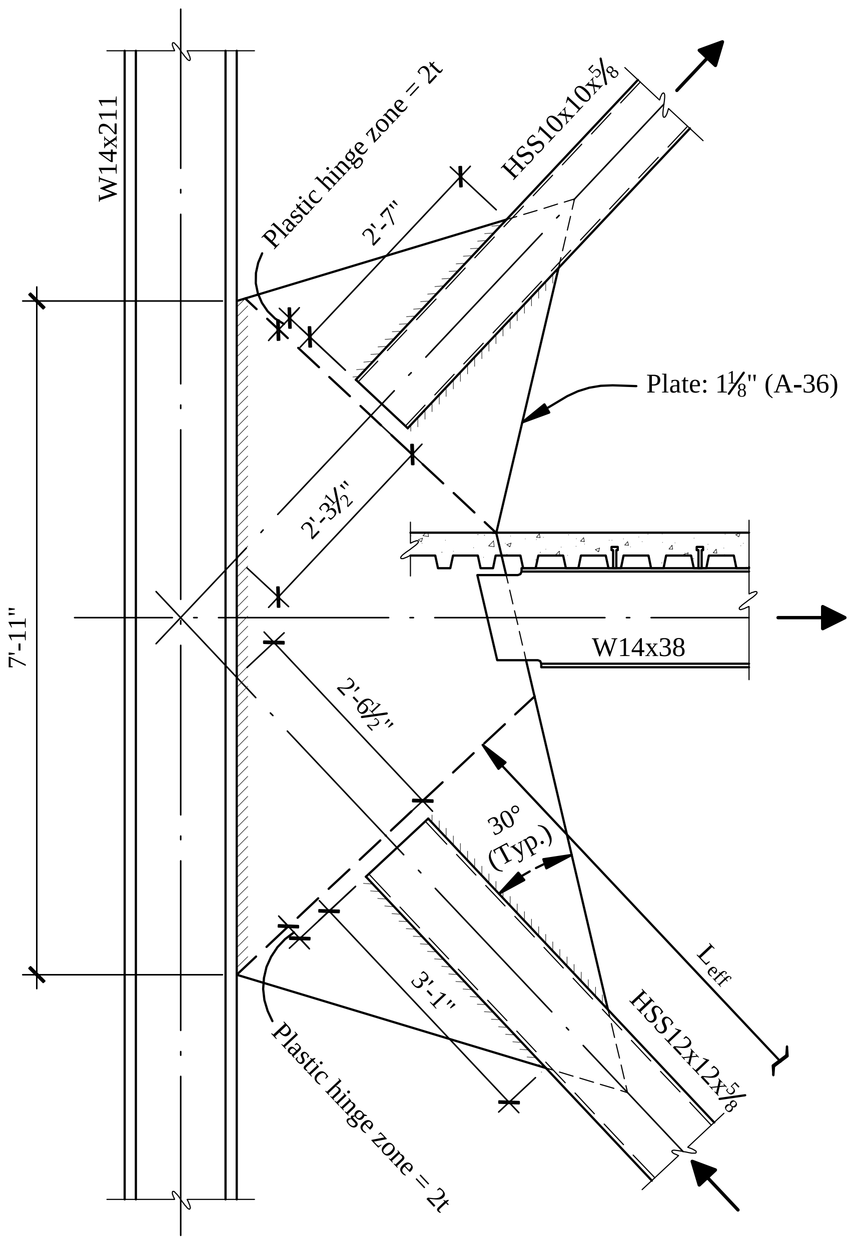


Figure 5.2-19 Bracing connection detail (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

Next, design the connections. The required strength of the connection is to be the nominal axial tensile strength of the bracing member. For an HSS12×12×5/8, the nominal axial tensile strength is computed using AISC Seismic Sec. 13.3a:

$P_n = R_y F_y A_g$ = (1.3)(46 ksi)(27.4 in.^{2}) = 1,639 kips

The area of the gusset is determined using the plate thickness and the Whitmore section for effective width. See Figure 5.2-20 for the determination of this dimension.

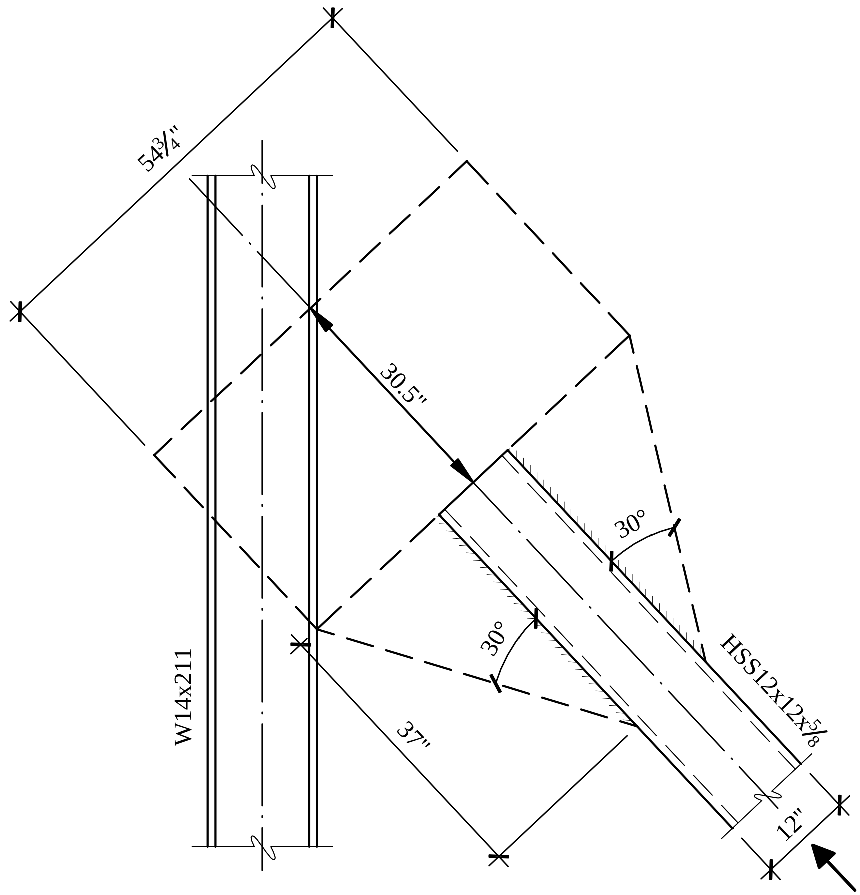


Figure 5.2-20 Whitmore section (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

For tension yielding of the gusset plate:

$\phi T_n = \phi F_y A_g =$ (0.90)(36 ksi)(1.125 in. × 54.7 in.) = 1,993 kips > 1,639 kips OK

For fracture in the net section:

$\phi T_n = \phi F_u A_n$ = (0.75)(58 ksi)(1.125 in. × 54.7 in.) = 2,677 kips > 1,639 kips OK

Since 1,933 kips is less than 2,677 kips, yielding (ductile behavior) governs over fracture.

For a tube slotted to fit over a connection plate, there will be four welds. The demand in each weld will be 1,639 kips/4 = 410 kips. The design strength for a fillet weld per AISC LRFD 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 in. fillet weld, the required length of weld is determined to be:

$$L_w = \frac{410 \text{ kips}}{(0.707)(0.5 \text{ in.})(31.5 \text{ ksi})} = 37 \text{ in.}$$

In accordance with the exception of AISC Seismic Sec. 13.3c, the design of brace connections need not consider flexure if the connections meet the following criteria:

a. Inelastic rotation associated with brace post-buckling deformations: The gusset plate is detailed such that it can form a plastic hinge over a distance of 2*t* (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 Seismic C13.3c. With such a plastic hinge, the compression brace may buckle out-of-plane when the tension braces are loaded. Remember that during the earthquake, there will be alternating cycles of compression to 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.

b. The connection design strength must be at least equal to the nominal compressive strength of the brace.

Therefore, the connection will be designed in accordance with these criteria. First, determine the nominal compressive strength of the brace member. The effective brace length (L_{eff}) is the distance between the plastic hinges on the gusset plates at each end of the brace member. For the brace being considered, L_{eff} = 169 in. and the nominal compressive strength is determined using AISC LRFD Eq. E2-4:

$$\lambda_c = \frac{kl}{r\pi}\sqrt{\frac{F_y}{E}} = \frac{(1)(169)}{(4.60)\pi}\sqrt{\frac{46}{29{,}000}} = 0.466$$

Since $\lambda_c < 1.5$, use AISC LRFD Eq. E2-2:

$$F_{cr} = (0.658^{\lambda_c^2})F_y = (0.658^{0.217})(46) = 42.0 \text{ ksi}$$

$P_{cr} = A_g F_{cr} = (27.4)(42.0) = 1{,}151 \text{ kips}$

Now, using a design compressive load from the brace of 1,151 kips, determine the buckling capacity of the gusset plate using the Whitmore section method. By this method, illustrated by Figure 5.2-20, the compressive force per unit length of gusset plate is (1,151 kips/54.7 in.) = 21.04 kips/in.

Try a plate thickness of 1.125 in.

$f_a = P/A = 21.04 \text{ kips}/(1 \text{ in.} \times 1.25 \text{ in.}) = 18.7 \text{ ksi}$

The gusset plate is modeled as a 1 in. wide by 1.125 in. deep rectangular section, pinned at one end (the plastic hinge) and fixed at the other end where welded to column (see Whitmore section diagram). The effective length factor (*k*) for this "column" is 0.8.

Per AISC LRFD Eq. E2-4:

$$\lambda_c = \frac{kl}{r\pi}\sqrt{\frac{F_y}{E}} = \frac{(0.8)(30.5)}{(0.54)\pi}\sqrt{\frac{36}{29,000}} = 0.51$$

Since λ_c < 1.5, use AISC LRFD Eq. E2-2:

$$F_{cr} = (0.658^{\lambda_c^2})F_y = (0.658^{0.257})(36) = 32.3 \text{ ksi}$$

ϕF_{cr} = (0.85)(32.3) = 27.4 ksi

ϕF_{cr} = 27.4 ksi > 18.7 ksi OK

Now consider the brace-to-brace connection shown in Figure 5.2-21. The gusset plate will experience the same tension force as the plate above, and the Whitmore section is the same. However, the compression length is much less, so a thinner plate may be adequate.

Try a 15/16 in. plate. Again, the effective width is shown in Figure 5.2-20. For tension yielding of the gusset plate:

$\phi T_n = \phi F_y A_g$ = (0.90)(36 ksi)(0.9375 in. × 54.7 in.) = 1,662 kips > 1,639 kips OK

For fracture in the net section:

$\phi T_n = \phi F_y A_g$ = (0.75)(58 ksi)(0.9375 in. × 54.7 in.) = 2,231 kips > 1,639 kips OK

Since 1,662 kips is less than 2,231 kips, yielding (ductile behavior) governs over fracture.

For compression loads, the plate must be detailed to develop a plastic hinge over a distance of 2*t* from the end of the brace. The effective length for buckling of this plate will be *k*[12" + (2)(2*t* + weld length)]. For this case, the effective length is 0.65[12 + (2)(2 × 15/16 + 5/16)] = 9.2 in. Compression in the plate over this effective length is acceptable by inspection and will not be computed here.

Next, check the reduced section of the-tube, which has a 1 1/4 in. wide slot for the gusset plate (at the column). The reduction in HSS12×12×5/8 section due to the slot is (0.581 × 1.25 × 2) = 1.45 in.^{2}, and the net section, A_{net} = (25.7 - 1.45) = 24.25 in.^{2}

Compare yield in the gross section with fracture in the net section:

Yield = $F_y A_g$ =(46 ksi)(25.7 in.^{2}) = 1,182 kips OK

Fracture = $F_u A_n$ =(58 ksi)(24.25 in.^{2}) = 1,406 kips OK

AISC Seismic 13.3b could be used to require design fracture strength (0.75 x 1,406 = 1,055 kips) to exceed probable tensile yield (1,639 kips), but this is clearly impossible, even if the net area equaled the gross area. This design is considered satisfactory.

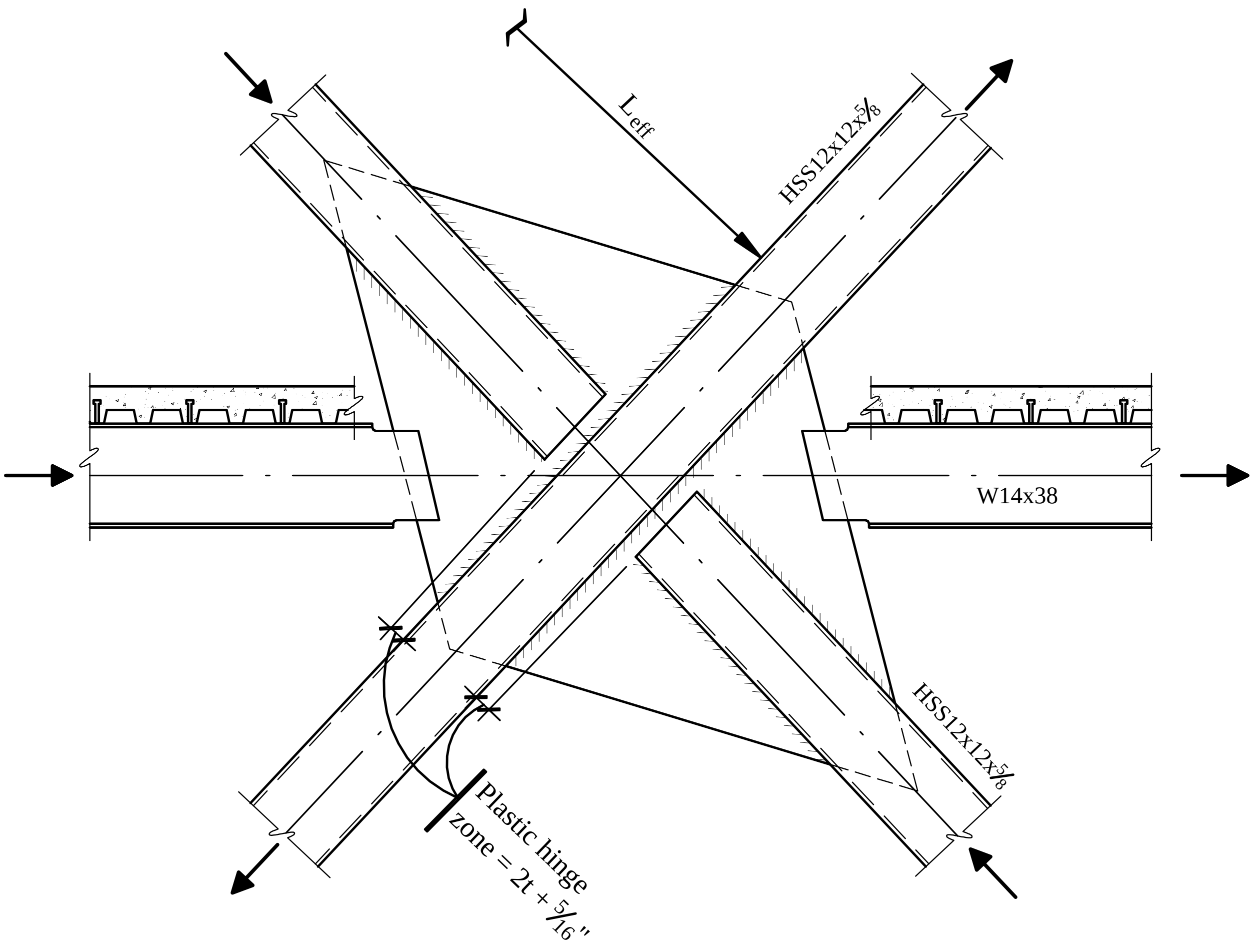


Figure 5.2-21 Brace-to-brace connection (1.0 in. = 25.4 mm).

5.2.4.3.3 Size Members for Alternative C, Dual System

1. Select Preliminary Member Sizes – A dual system is a combination of a moment-resisting frame with either a shear wall or a braced frame. In accordance with the building systems listed in *Provisions* Table 5.2.2 [4.3-1], a dual system consisting of special moment frames at the perimeter and special concentrically braced frames at the core will be used.

2. Check Strength of Moment Frame – The moment frame is required to have sufficient strength to resist 25 percent of the design forces by itself (*Provisions* Sec. 5.2.2.1 [4.3.1]). This is a good place to start a design. Preliminary sizes for the perimeter moment frames are shown in Figures 5.2-22 and 5.2-23. It is designed for strength using 25 percent of the design lateral forces. All the design requirements for special moment frames still apply (flange and web width-to-thickness ratios, column-beam moment ratio, panel zone shear, drift, and redundancy) and all must be checked; however, it may be prudent to defer some of the checks until the design has progressed a bit further. The methodology for the analysis and these checks is covered in Sec. 5.2.4.3.1, so they will not be repeated here.

   For some buildings this may present an opportunity to design the columns without doubler plates because the strength requirement is only 25 percent of the total. However, for the members used in this example, doubler plates will still be necessary. The increase in column size to avoid doubler plates is substantial, but feasible. The sequence of column sizes that is shown is W 14×132 - 82 - 68 -

53 and would become W14×257 - 233 - 211 - 176 to avoid doubler plates. The beams in Figures 5.2-22 and 5.2-23 are controlled by strength because drift is not a criterion.

Note that $P_u / \phi P_n > 0.4$ for a few of the columns when analyzed without the braced frame so the overstrength requirements of AISC Seismic Sec. 8.2 [8.3] apply to these columns. Because the check using $\Omega_0 E$ is for axial capacity only and the moment frame columns are dominated by bending moment, the sizes are not controlled by the check using $\Omega_0 E$.

3. Check the Strength of the Braced Frames – The next step is to select the member sizes for the braced frame. Because of the presence of the moment frame, the accidental torsion on the building will be reduced as compared to a building with only a braced core. In combination with the larger R factor (smaller design forces), this should help to realize significant savings in the braced frame member sizes. A trial design is selected, followed by analysis of the entire dual system. All members need to be checked for width-thickness ratios and the braces need to be checked for slenderness. Note that columns in the braced frame will need to satisfy the overstrength requirements of AISC Seismic Sec. 8.2 [8.3] because $P_u/\phi P_n > 0.4$. This last requirement causes a significant increase in column sizes, except in the upper few stories.

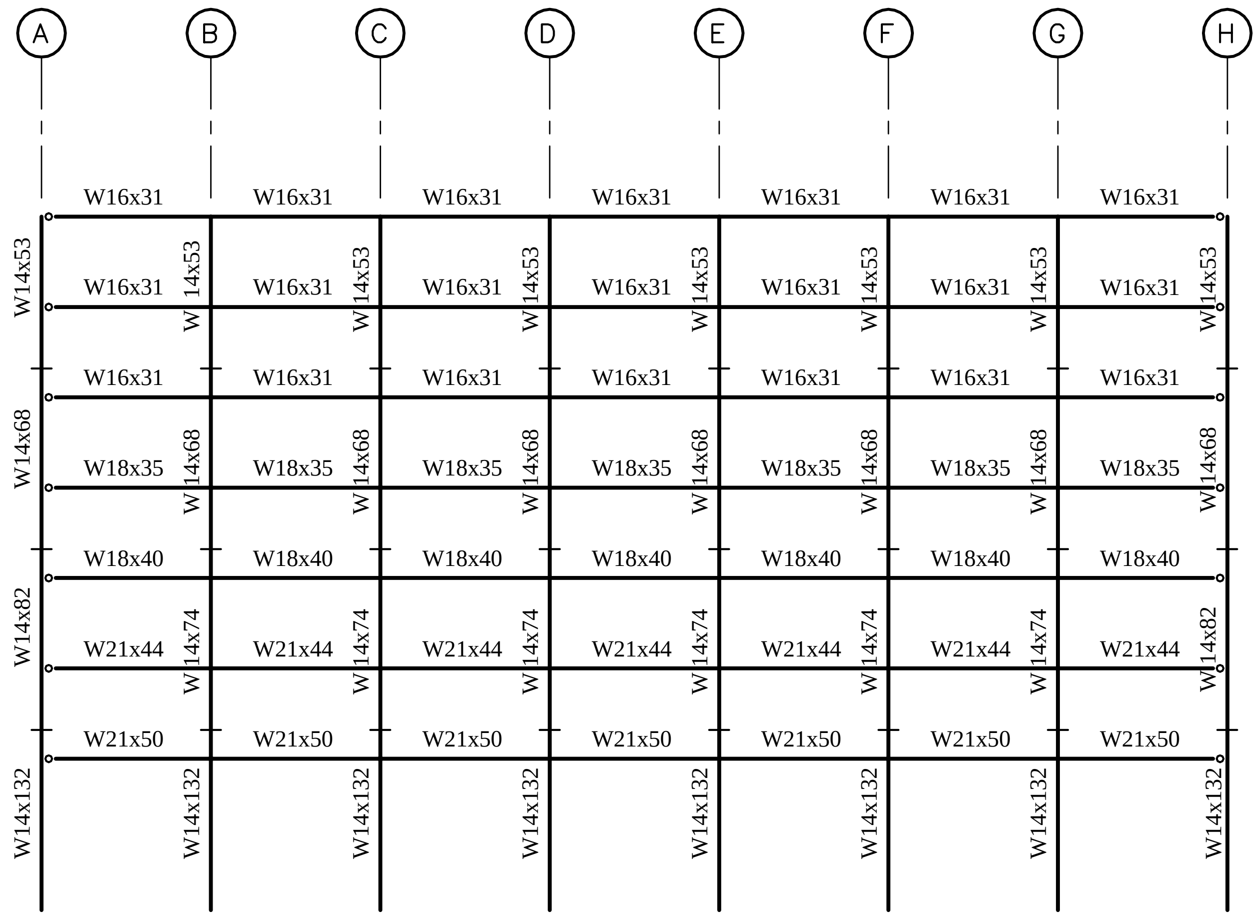


Figure 5.2-22 Moment frame of dual system in E-W direction.

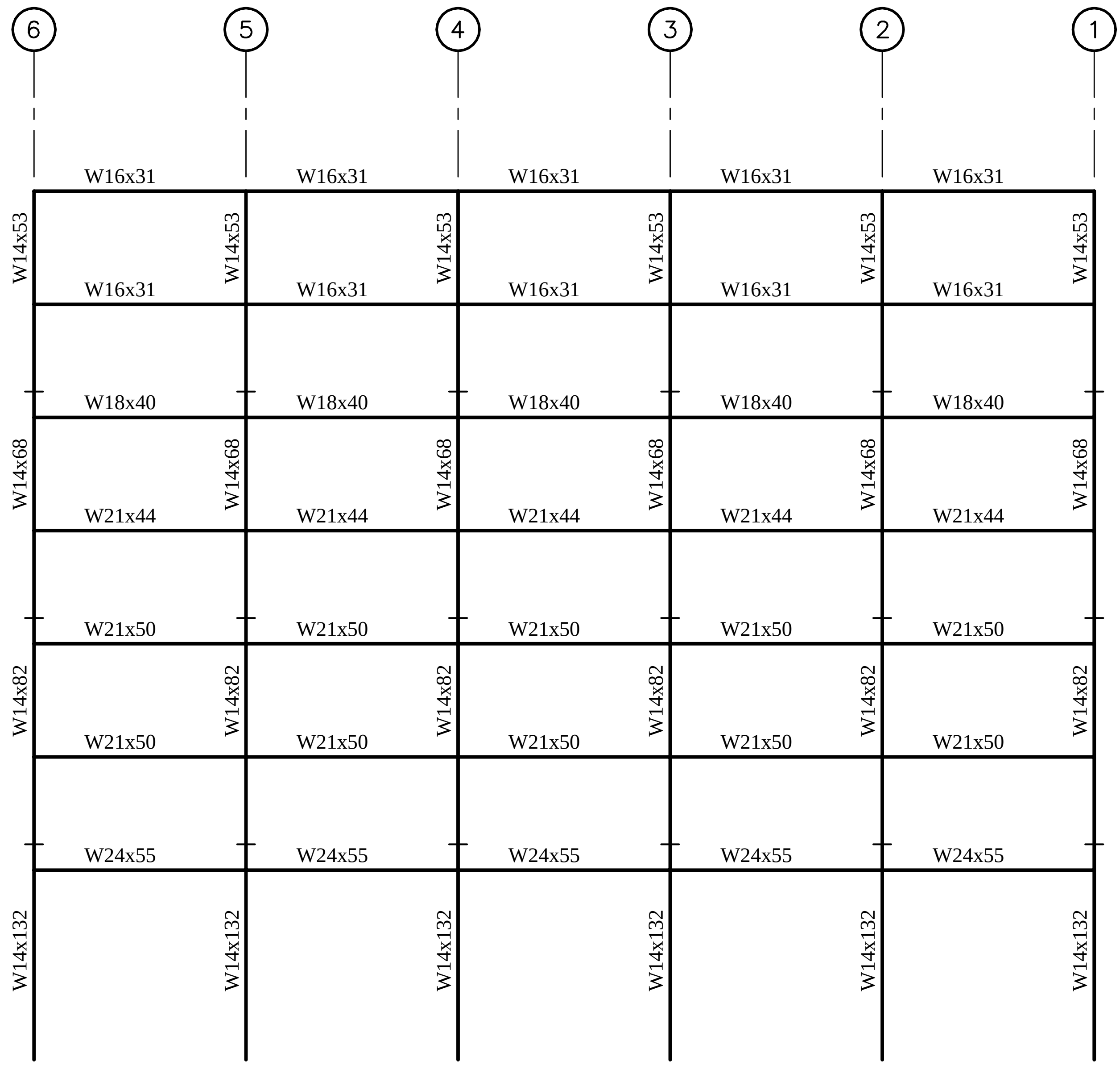


Figure 5.2-23 Moment frame of dual system in N-S direction.

4. Check Drift – Check drift in accordance with *Provisions* Sec. 5.2.8 [4.5]. The building was modeled in three dimensions using RAMFRAME. Maximum displacements at the building corners are used here because the building is torsionally irregular. Displacements at the building centroid are also calculated because these will be the average between the maximum at one corner and the minimum at the diagonally opposite corner. Use of the displacements at the centroid as the average displacements is valid for a symmetrical building.

5. Check Torsional Amplification – Calculated story drifts are used to determine A_x, the torsional amplification factor (Table 5.2-8). P-delta effects are included.

Table 5.2-8 Alternative C Amplification of Accidental Torsion

	Average Elastic Displacement = Displacement at Building Centroid (in.)		Maximum Elastic Displacement at Building Corner* (in.)		$\dfrac{\delta \max}{\delta \text{avg}}$ **		Torsional Amplification Factor = $A_x = \left(\dfrac{\delta_{\max}}{1.2\delta_{\text{avg}}}\right)^2$		Amplified Eccentricity = $A_x(0.05\ L)$*** (ft.)	
	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S
R	2.77	2.69	3.57	3.37	1.29	1.25	1.15	1.09	7.19	9.54
7	2.45	2.34	3.15	3.00	1.28	1.28	1.14	1.14	7.14	10.01
6	2.05	1.91	2.63	2.50	1.28	1.31	1.13	1.20	7.07	10.46
5	1.64	1.51	2.10	2.01	1.28	1.33	1.13	1.23	7.08	10.8
4	1.22	1.11	1.56	1.50	1.28	1.35	1.14	1.27	7.13	11.15
3	0.81	0.75	1.05	1.03	1.29	1.38	1.16	1.31	7.25	11.50
2	0.43	0.41	0.57	0.57	1.32	1.40	1.20	1.37	7.52	11.98

* These values are directly from the analysis. Accidental torsion is not amplified here.

** Amplification of accidental torsion is required because this term is greater than 1.2 (*Provisions* Table 5.2.3.2, Item 1a [4.3-2, Item 1a). The building is *torsionally irregular* in plan. See discussion in footnote ** of Table 5.2.6.

*** The initial eccentricities of 0.05L in the E-W and N-S directions are multiplied by A_x to determine the amplified eccentricities. These will be used in the next round of analysis.

1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m.

The design that yielded the displacements shown in Table 5.2-8 does not quite satisfy the drift limits, even without amplifying the accidental torsion. That design was revised by increasing various brace and column sizes and then re-analyzing using the amplified eccentricity instead of merely 0.05L for accidental torsion. After a few iterations, a design that satisfied the drift limits was achieved. These member sizes are shown in Figures 5.2-24 and 5.2-25. That structure was then checked for its response using the standard 0.05L accidental eccentricity in order to validate the amplifiers used in design. The amplifier increased for the E-W direction but decreased for the N-S direction, which was the controlling direction for torsion. The results are summarized in Table 5.2-9.

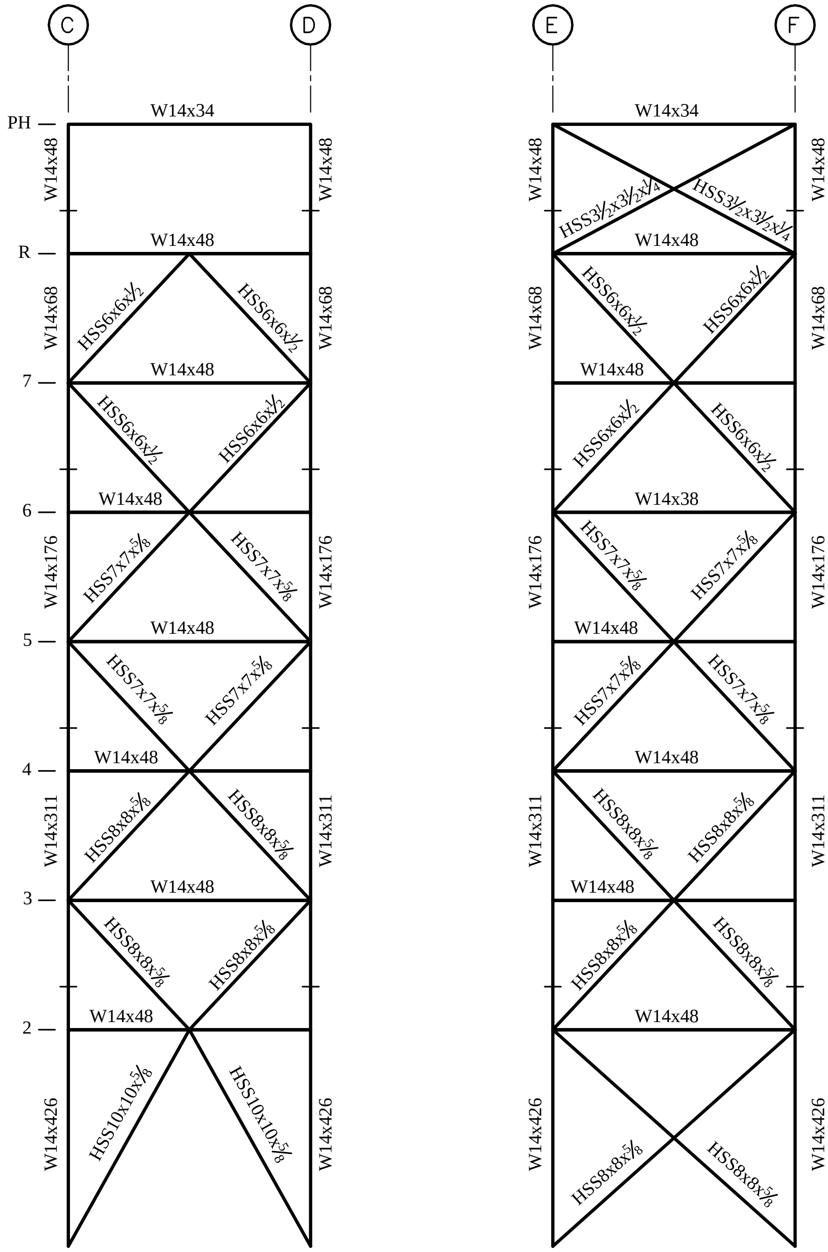


Figure 5.2-24 Braced frame of dual system in E-W-direction**.**

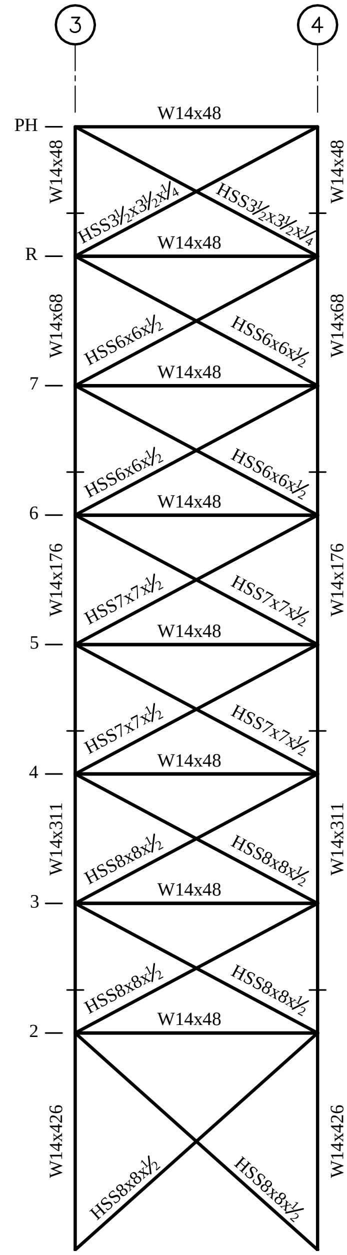


Figure 5.2-25 Braced frame of dual system in N-S direction.

Table 5.2-9 Alternative C Story Drifts under Seismic Load

	Max. Elastic Displacement at Building Corners (in.)		Elastic Story Drift at Location of Max. Displacement (at corners) (in.)		C_d	(C_d) x (Elastic Story Drift) (in.)		Allowable Story Drift (in.)
	E-W	N-S	ΔE-W	ΔN-S		ΔE-W	ΔN-S	Δ
R	3.06	3.42	0.37	0.37	6.5	2.43	2.42	3.20
7	2.69	3.05	0.45	0.47	6.5	2.94	3.05	3.20
6	2.24	2.58	0.45	0.49	6.5	2.89	3.17	3.20
5	1.79	2.09	0.45	0.51	6.5	2.93	3.30	3.20
4	1.34	1.58	0.41	0.48	6.5	2.66	3.09	3.20
3	0.93	1.11	0.39	0.46	6.5	2.55	3.01	3.20
2	0.54	0.64	0.54	0.64	6.5	3.52	4.17	5.36

1.0 in. = 25.4 mm

The story drifts are within the allowable story drift limit of $0.020h_{sx}$ as per *Provisions* Sec. 5.2.8 [4.5.1]. Level 5 has a drift of 3.30 in. > 3.20 in. but the difference of only 0.1 in. is considered close enough for this example.

6. Check Redundancy – Now return to the calculation of r_x for the braced frame. In accordance with *Provisions* Sec. 5.2.4.2 [not applicable in the 2003 *Provisions*], $r_{\max_x}$ for braced frames is taken as the lateral force component in the most heavily loaded brace element divided by the story shear. This is illustrated in Figure 5.2-18 for Alternative B.

For this design, r_x was determined for every brace element at every level in both directions. The lateral component carried by each brace element comes from the RAMFRAME analysis, which includes the effect of amplified accidental torsion. The maximum value was found to be 0.1762 at the base level in the N-S direction. Thus, ρ is now determined to be (see Sec. 5.2.4.2):

$$\rho = 0.8\left[2 - \frac{20}{r_{\max_x}\sqrt{A_x}}\right] = 0.8\left[2 - \frac{20}{0.1762\sqrt{21{,}875\ \text{ft.}^2}}\right] = 0.986$$

The 0.8 factor comes from *Provisions* Sec. 5.2.4.2 [not applicable in the 2003 *Provisions*]. As ρ is less than 1.0, $\rho = 1.0$ for this example.

In the E-W direction, r_{max} is less; therefore, ρ will be less, so use $\rho = 1.0$ for both directions.

[See Sec. 5.2.3.2 for a discussion of the significant changes to the redundancy requirements in the 2003 *Provisions*.]

7. Connection Design – Connections for both the moment frame and braced frames may be designed in accordance with the methods illustrated in Sec. 5.2.4.3.1 and 5.2.4.3.2.

5.2.5 Cost Comparison

Material takeoffs were made for the three alternatives. In 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:

Alternative A, Special Steel Moment Resisting Frame	593 tons
Alternative B, Special Steel Concentrically Braced Frame	640 tons
Alternative C, Dual System	668 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.

5.3 TWO-STORY BUILDING, OAKLAND, CALIFORNIA

This example features an eccentrically braced frame (EBF) building. The following items of seismic design of steel-framed buildings are illustrated:

1. Analysis of eccentrically braced frames
2. Design of bracing members
3. Brace connections

5.3.1 Building Description

This two-story hospital, 120 ft by 140 ft in plan, is shown in Figure 5.3-1. The building has a basement and two floors. It has an unusually high roof load because of a plaza with heavy planters on the roof.

The vertical-load-carrying system consists of concrete fill on steel deck floors supported by steel beams and girders that span to steel columns and to the perimeter basement walls. The bay spacing is 20 ft each way. Floor beams are spaced three to a bay. The beams and girders on the column lines are tied to the slabs with stud connectors.

The building is founded on a thick mat. The foundation soils are deep stable deposits of sands, gravels, and stiff clays overlying rock.

The lateral-force-resisting system for Stories 1 and 2 consists of EBFs on Gridlines 1, 8, B, and F as

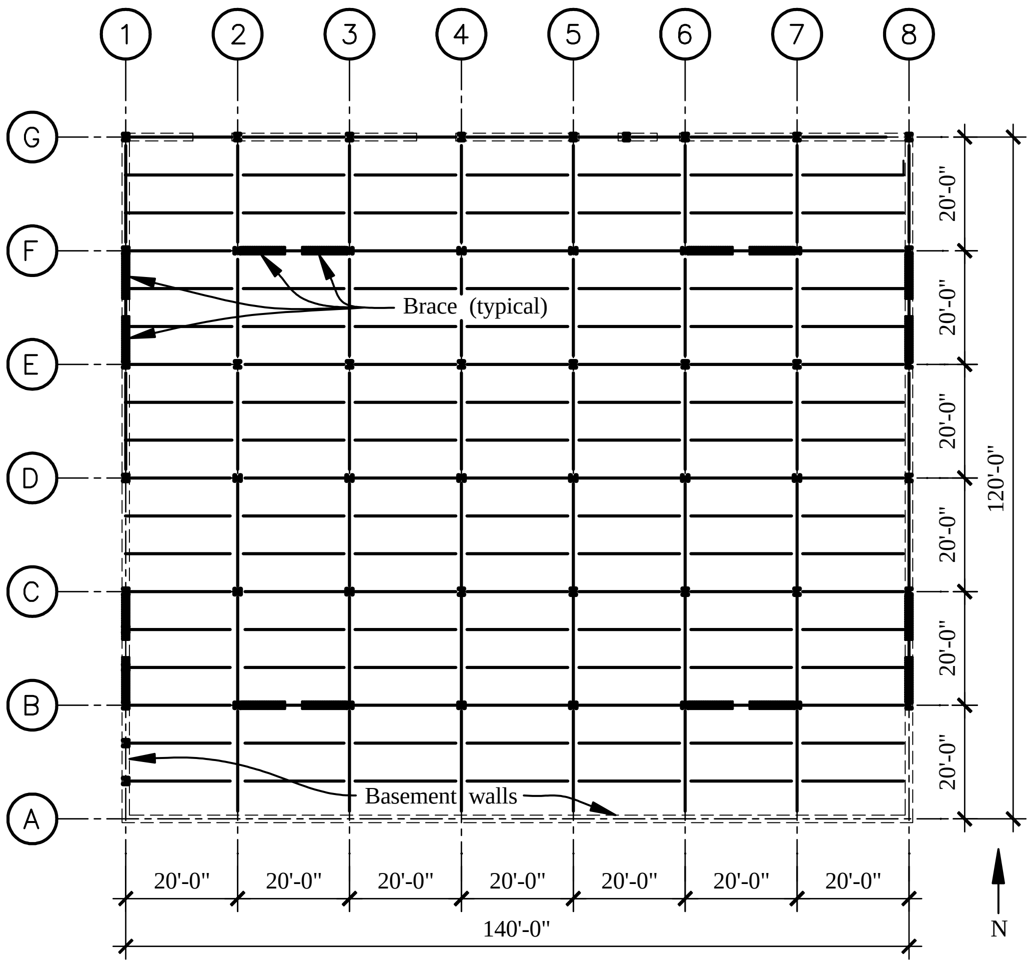


Figure 5.3-1 Main floor framing plan (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

shown in Figure 5.3-1. A typical bracing elevation is shown in Figure 5.3-2. These EBFs transfer lateral loads to the main floor diaphragm. The braced frames are designed for 100 percent of lateral load and their share of vertical loads. EBFs have been selected for this building because they provide high stiffness and a high degree of ductility while permitting limited story-to-story height.

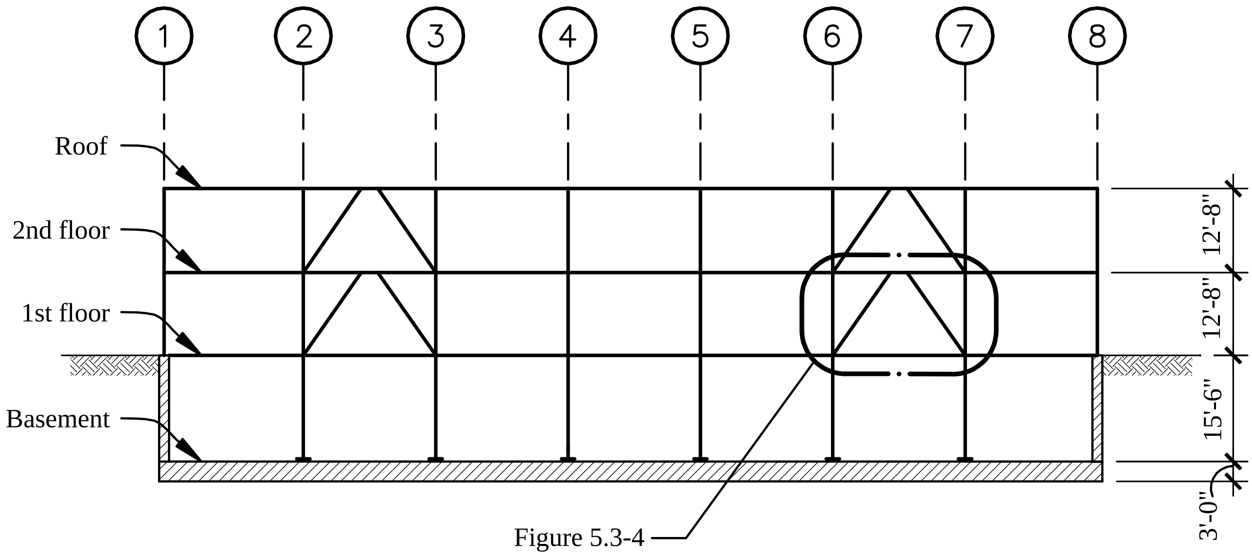


Figure 5.3-2 Section on Grid F (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

The structure illustrates a common situation for low-rise buildings with basements. The combination of the basement walls and the first floor diaphragm is so much stiffer that the superstructure that the *base* (see *Provisions* Chapter 2 [4.1.3] for definition) of the building is the first floor, not the foundation. Therefore, the diagonal braces do not extend into the basement because the horizontal force is in the basement walls (both in shear parallel to the motion considered and in direct pressure on perpendicular walls). This has a similarity to the irregularity Type 4 "out-of-plane offsets" defined in *Provisions* Table 5.2.3.2 [4.3-2], but because it is below the base that classification does not apply. However, the columns in the basement that are part of the EBFs must be designed and detailed as being the extension of the EBF that they are. This affects width-thickness ratios, overstrength checks, splice requirements, and so on. Column design for an EBF is illustrated later in this example.

5.3.2 Method

The method for determining basic parameters is similar to that for other examples. It will not be repeated here; rather the focus will be on the design of a specific example of an EBF starting with the forces in the frame as obtained from a linear analysis. Keep in mind that the load path is from the floor diaphragm to the beam to the braces. The fundamental concept behind the eccentric braced frame is that seismic energy is absorbed by yielding of the link. Yielding in shear is more efficient than yielding in flexure, although either is permitted. A summary of the method is as follows:

1. Select member preliminary sizes.

2. Perform an elastic analysis of the building frame. Compute elastic drift (δ_e) and forces in the members.

3. Compute the inelastic displacement as the product of C_d times δ_e. The inelastic displacement must be within the allowable story drift from *Provisions* Table 5.2.8 [4.5-1].

4. Compute the link rotation angle (α) and verify that it is less than 0.08 radians for yielding dominated by shear in the link or 0.02 radians for yielding dominated by flexure in the link. (See Figure 5.3-4 for illustration of α). The criteria is based on the relationship between M_p and V_p as related to the length of the link.

5. To meet the link rotation angle requirement, it may be necessary to modify member sizes, but the more efficient approach is to increase the link length. (The trade-off to increasing the link length is that the moment in the link will increase. Should the moment become high enough to govern over shear yielding, then α will have to be limited to 0.02 radians instead of 0.08 radians.)

6. The braces and building columns are to remain elastic. The portions of the beam outside the link are to remain elastic; only the link portion of the beam yields.

7. For this case, there are moment-resisting connections at the columns. Therefore from *Provisions* Table 5.2.2 [4.3-1], R = 8, C_d = 4, and Ω_0 =2. (Neither the *Provisions* nor AISC Seismic offer very much detailed information about requirements for moment-resisting connections for the beam to column connection in an EBF. There are explicit requirements for the connection from a link to a column. The EBF system will not impose large rotational demands on a beam to column connection; the inelastic deformations are confined to the link. Therefore, without further detail, it is the authors' interpretation that an ordinary moment resisting frame connection will be adequate).

5.3.3 Analysis

Because the determination of basic provisions and analysis are so similar to those of other examples, they will not be presented here. An ELF analysis was used.

5.3.3.1 Member Design Forces

The critical forces for the design of individual structural elements, determined from computer analysis, are summarized in Table 5.3-1.

Table 5.3-1 Summary of Critical Member Design Forces

Member	Force Designation	Magnitude
Link	P_{link}	5.7 kips
	V_{link}	85.2 kips
	M_{left}	127.9 ft-kips
	M_{right}	121.3 ft-kips
Brace	P_{brace}	120.0 kips
	M_{top}	15.5 ft-kips
	M_{bot}	9.5 ft-kips

1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m.

The axial load in the link at Level 2 may be computed directly from the second-floor forces. The force from the braces coming down from the roof level has a direct pass to the braces below without affecting the link. The axial forces in the link and brace may be determined as follows:

Total second-story shear (determined elsewhere) = 535.6 kips

Second-story shear per braced line = 535.6/2 = 267.8 kips
Second-story shear per individual EBF = 267.8/2 = 133.9 kips
Second-story shear per brace = 133.9/2 = 66.95 kips
Axial force per brace = 66.95 (15.25 ft/8.5 ft) = 120.0 kips

Second-story shear per braced line = 267.8 kips
Second-story shear per linear foot = 267.8 kips/140 ft = 1.91 klf
Axial force in link = (1.91 klf)(3 ft) = 5.7 kips

5.3.3.2 Drift

From the linear computer analysis, the elastic drift was determined to be 0.247 inches. The total inelastic drift is computed as:

$C_d\delta_c$ = (4)(0.247) = 0.99 in.

The link rotation angle is computed for a span length, L = 20 ft, and a link length, e = 3 ft as follows:

$$\alpha = \left(\frac{L}{e}\right)\theta = \left(\frac{20\text{ ft}}{3\text{ ft}}\right)\left(\frac{0.99\text{ in.}}{(12.67\text{ ft})(12)}\right) = 0.043\text{ radians}$$

The design is satisfactory if we assume that shear yielding governs because the maximum permissible rotation is 0.08 radians (AISC Seismic Sec. 15.2g [15.2]). For now, we will assume that shear yielding of the link governs and will verify this later.

5.3.4 Design of Eccentric Bracing

Eccentric bracing adds two elements to the frame: braces and links. As can be seen in Figure 5.3-3, two eccentric braces located in one story of the same bay intersect the upper beam a short distance apart, thus creating a link subject to high shear. In a severe earthquake, energy is dissipated through shear yielding of the links while diagonal braces and columns remain essentially elastic.

The criteria for the design of eccentric bracing are given in AISC Seismic Sec. 15. All section sizes and connection details are made similar for all braced bays. The following sections have been selected as a preliminary design:

Typical girders	W16×57
Typical columns	W14×132
Typical braces	HSS 8×8×5/8

Since all members of the braced frames are to be essentially the same, further calculations deal with the braced frames on Line F, shown in Figures 5.3-3 and 5.3-4.

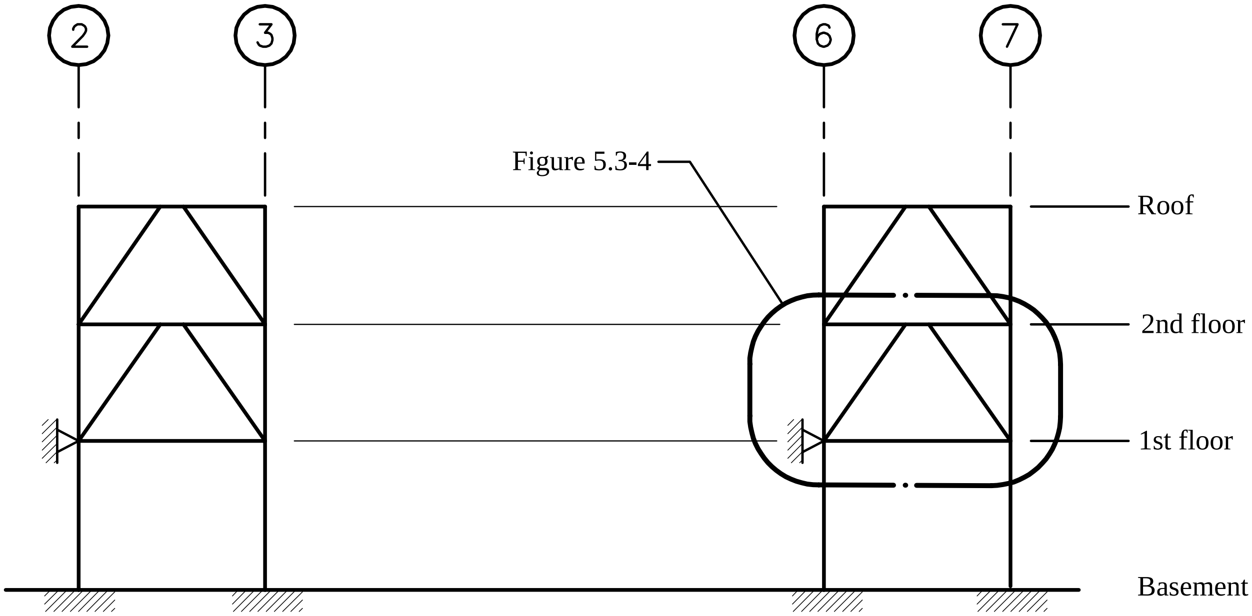


Figure 5.3-3 Diagram of eccentric braced frames on Grid F.

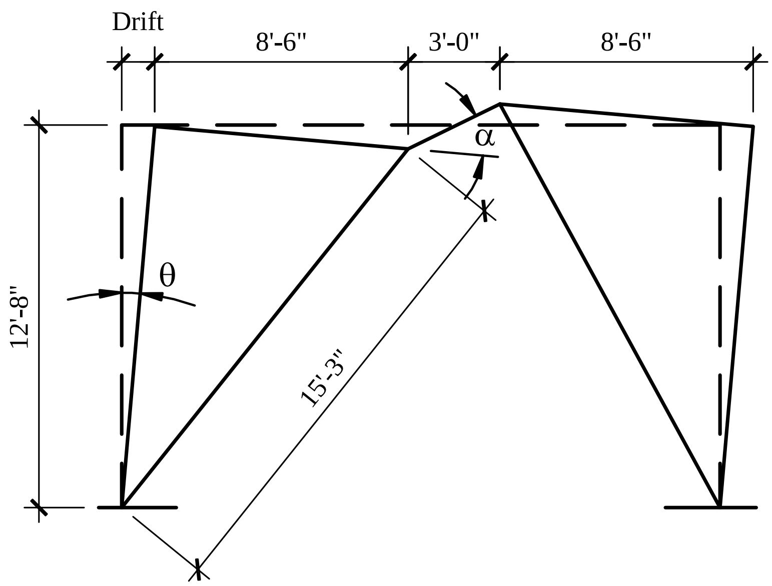


Figure 5.3-4 Typical eccentric braced frame
(1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

5.3.4.1 Link Design

The first-story eccentric braced frame (identified in Figure 5.3-2) is examined first. The shear force and end moments in the link (W16x57 beam section) are listed in Table 5.3-1 and repeated below:

P_{link} = 5.7 kips
V_{link} = 85.2 kips
M_{left} = 127.9 ft-kips
M_{right} = 121.3 ft-kips

5.3.4.1.1 Width-Thickness Ratio

The links are first verified to conform to AISC Seismic Sec. 15.2a [15.2], which refers to AISC Seismic Table I-9-1 [I-8-1].

First, check the beam flange width-thickness ratio. For the selected section, b/t = 4.98, which is less than the permitted b/t ratio of :

$$\frac{52}{\sqrt{F_y}} = \frac{52}{\sqrt{50}} = 7.35 \qquad \text{OK}$$

The permitted web slenderness is dependent on the level of axial stress. The level of axial stress is determined as:

$$\frac{P_u}{\phi_b P_y} = \frac{5.8}{(0.9)(16.8 \times 50)} = 0.008$$

It is less than 0.125; therefore, the ratio t_w/h_c = 33.0 for the selected section is less than the limiting width-to-thickness ratio computed as:

$$\frac{253}{\sqrt{F_y}} = \frac{253}{\sqrt{50}} = 35.7 \qquad \text{OK}$$

5.3.4.1.2 Link Shear Strength

The forces V_{link}, M_{left}, and M_{right} must not exceed member strength computed from AISC Seismic Sec. 15.2d [15.2]. That section specifies that the required shear strength of the link (V_u) must not exceed the design shear strength $\phi V_{n,}$ where $V_u = V_{link}$ = 85.2 kips and V_n is the nominal shear strength of link. The nominal shear strength of the link is defined as the lesser of:

$$V_p = (0.60F_y)(d\text{-}2t_f)t_w$$

and

$$\frac{2M_p}{e}$$

For the W16×57 section selected for the preliminary design:

$$V_p = (0.60)(50)[16.43 - (2)(0.715)](0.430) = 193.5 \text{ kips}$$

and

$$M_p = \phi M_n = 0.9F_y\, Z_x = (0.9)(50)(105) = 4725 \text{ in.-kips}$$

$$\frac{2M_p}{e} = \frac{(2)(4725)}{(3 \times 12)} = 262.5 \text{ kips}$$

Therefore,

V_n = 193.5 kips

ϕV_n = (0.9)(193.5) = 174.2 ft-kips > 85.2 kips OK

5.3.4.1.3 Link Axial Strength

In accordance with AISC Seismic Sec. 15.2e [15.2], the link axial strength is examined:

P_y of the link = $F_y A_g$ = (50 ksi)(16.8 in) = 840 kips

0.15P_y of the link = (0.15)(840) = 126 kips

Since the axial demand of 5.7 kips is less than 126 kips, the effect of axial force on the link design shear strength need not be considered. Further, because $P_u < P_y$, the additional requirements of AISC Seismic Sec. 15.f [15.2] do not need to be invoked.

5.3.4.1.4 Link Rotation Angle

In accordance with AISC Seismic Sec. 15.2g [15.2], the link rotation angle is not permitted to exceed 0.08 radians for links 1.6M_p/V_p long or less. Therefore, the maximum link length is determined as:

1.6M_p/V_p = (1.6)(4725)/(193.5) = 39.1 in.

Since the link length (e) of 36 in. is less than 1.6M_p/V_p, the link rotation angle is permitted up to 0.08 radians. From Sec. 5.3.3.2, the link rotation angle, α, was determined to be 0.043 radians, which is acceptable.

5.3.4.1.5 Link Stiffeners

AISC Seismic Sec. 15.3a [15.3] requires full-depth web stiffeners on both sides of the link web at the diagonal brace ends of the link. These serve to transfer the link shear forces to the reacting elements (the braces) as well as restrain the link web against buckling.

Because the link length (e) is less than 1.6M_p/V_p, intermediate stiffeners are necessary in accordance with AISC Seismic Sec. 15.3b [15.3]. Interpolation of the stiffener spacing based on the two equations presented in AISC Seismic Sec. 15.3b.1 [15.3] will be necessary. For a link rotation angle of 0.08 radians:

Spacing = (30t_w - d/5) = (30 × 0.430 - 16.43/5) = 9.6 in.

For link rotation angle of 0.02 radians:

Spacing = (52t_w - d/5) = (52 × 0.430 - 16.43/5) = 19.1 in.

For our case the link rotation angle is 0.043 radians, and interpolation results in a spacing requirement of 15.4 in. Therefore, use a stiffener spacing of 12 in. because it conforms to the 15.4 in. requirement and also fits nicely within the link length of 36 in.

In accordance with AISC Seismic Sec. 15.3a [5.3], full depth stiffeners must be provided on both sides of the link, and the stiffeners must be sized as follows:

Combined width at least (b_f - $2t_w$) = (7.120 - 2 × 0.430) = 6.26 in. Use 3.25 in. each.
Thickness at least $0.75t_w$ or 3/8 in. Use 3/8 in.

5.3.4.1.6 Lateral Support of Link

The spacing of the lateral bracing of the link must not exceed the requirement of AISC LRFD Eq. F1-17, which specifies a maximum unbraced length of:

$$L_{pd} = \frac{[3,600 + 2,200(M_1/M_2)]r_y}{F_y} = \frac{[3,600 + 2,200(121.3/127.9)](1.60)}{50} = 182 \text{ in.}$$

Accordingly, lateral bracing of beams with one brace at each end of the link (which is required for the link design per AISC Seismic Sec. 15.5) is sufficient.

In accordance with AISC Seismic Sec. 15.5, the end lateral supports must have a design strength computed as:

$0.06R_yF_yb_ft_f$ = (0.06)(1.1)(50)(7.120)(0.715) = 16.8 kips

While shear studs on the top flange are expected to accommodate the transfer of this load into the concrete deck, the brace at the bottom flange will need to be designed for this condition. Figure 5.3-5 shows angle braces attached to the lower flange of the link. Such angles will need to be designed for 16.8 kips tension or compression.

5.3.4.2 Brace Design

For the design equations used below, see Chapter E. of the AISC LRFD Specification. The braces, determined to be 8×8×5/8 in. tubes with F_y = 46 ksi in the preliminary design, are subjected to a calculated axial seismic load of 120 kips (from elastic analysis in Table 5.3-1). Taking the length of the brace conservatively as the distance between panel points, the length is 15.26 feet. The slenderness ratio is

$$\frac{kl}{r} = \frac{(1)(15.26)(12)}{2.96} = 61.9$$

(*k* has been conservatively taken as 1.0, but is actually lower because of restraint at the ends.)

Using AISC LRFD E2-4 for F_y = 46 ksi:

$$\lambda_c = \frac{kl}{r\pi}\sqrt{\frac{F_y}{E}} = \frac{(1)(15.26 \times 12)}{2.96\pi}\sqrt{\frac{50}{29,000}} = 0.817$$

$$F_{cr} = (0.658^{\lambda_c^2})F_y = (0.658^{0.817^2})(46) = 34.8 \text{ ksi}$$

The design strength of the brace as an axial compression element is:

$P_{br} = \phi_c A_g F_{cr}$ = (0.85)(17.4)(34.8) = 514 kips

AISC Seismic Sec. 15.6a [15.6] requires that the design axial and flexural strength of the braces be those resulting from the expected nominal shear strength of the link (V_n) increased by R_y and a factor of 1.25.

Thus, the factored V_n is equal to (193.5 kips)(1.1)(1.25) = 266 kips. The shear in the link, determined from elastic analysis, is 85.2 kips. Thus, the increase is 266/85.2 = 3.12. Let us now determine the design values for brace axial force and moments by increasing the values determined from the elastic analysis by the same factor:

Design P_{brace} = (3.12)(120) = 374 kips
Design M_{top} = (3.12)(15.5) = 48.4 ft-kips
Design M_{bot} = (3.12)(9.5) = 29.6 ft-kips

The design strength of the brace, 514 kips, exceeds the design demand of 374 kips, so the brace is adequate for axial loading. However, the brace must also be checked for combined axial and flexure using AISC LRFD Chapter H. For axial demand-to-capacity ratio greater than 0.20, axial and flexure interaction is governed by AISC LRFD H1-1a:

$$\frac{P_u}{\phi P_n} + \frac{8}{9}\left(\frac{M_u}{\phi_b M_n}\right) \le 1.0$$

where

P_u = 374 kips
P_n = 514 kips
$M_n = ZF_y$ = (105)(50) = 5250 in.-kips

The flexural demand, M_u, is computed in accordance with AISC LRFD Chapter C and must account for second order effects. For a braced frame only two stories high and having several bays, the required flexural strength in the brace to resist lateral translation of the frame only (M_{lt}) is negligible. Therefore, the required flexural strength is computed from AISC LRFD C1-1 as:

$$M_u = B_1 M_{nt}$$

where M_{nt} = 48.4 ft-kips as determined above and, per AISC LRFD C1-2:

$$B_1 = \frac{C_m}{1 - P_u / P_e} \ge 1.0$$

$$P_e = \frac{A_g F_y}{\lambda_c^2} = \frac{(17.4)(46)}{(0.817)^2} = 1{,}199 \text{ kips}$$

$$Cm = 0.6 - 0.4\left(\frac{M_1}{M_2}\right) = 0.6 - 0.4\left(\frac{29.6}{48.4}\right) = 0.36$$

Therefore,

$$M_u = B_1 M_{nt} = \frac{C_m M_{nt}}{1 - \dfrac{P_u}{P_e}} = \frac{(0.36)(48.4)}{1 - \dfrac{374}{1{,}199}} = (0.52)(48.4) = 25.3 \text{ ft-kips}$$

and

$$\frac{P_u}{\phi P_n} + \frac{8}{9}\left(\frac{M_u}{\phi_b M_n}\right) = \frac{374}{(0.85)(514)} + \frac{8}{9}\left(\frac{(25.3)(12)}{(0.9)(4{,}830)}\right) = 0.92 < 1.00 \qquad \text{OK}$$

The design of the brace is satisfactory.

5.3.4.3 Brace Connections at Top of Brace

AISC Seismic Sec. 15.6 requires that, like the brace itself, the connection of the brace to the girder be designed to remain elastic at yield of the link. The required strength of the brace-to-beam connection must be at least as much as the required strength of the brace. Because there is a moment at the top of the brace, the connections must also be designed as a fully restrained moment connection. The beam, link, and brace centerlines intersect at a common work point, and no part of this connection shall extend over the link length.

The tube may be attached to the girder with a gusset plate welded to the bottom flange of the girder and to the tube with fillet welds. The design of the gusset and connecting welds is conventional except that cutting the gusset short of the link may require adding a flange. (Such a flange is shown ine Figure 5.3-5.) Adding a similar flange on the other side of the brace will keep the joint compact. In such a case, it may be required, or at least desirable, to add another stiffener to the beam opposite the flange on the gusset. It also should be remembered that the axial force in the brace may be either tension or compression reflecting the reversal in seismic motions.

In addition to the design of the gusset and the connecting welds, a check should be made of stiffener requirements on the beam web opposite the gusset flanges (if any) and the panel zone in the beam web above the connection. All of these calculations are conventional and need no explanation here. Details of the link and adjacent upper brace connection are shown in Figure 5.3-5.

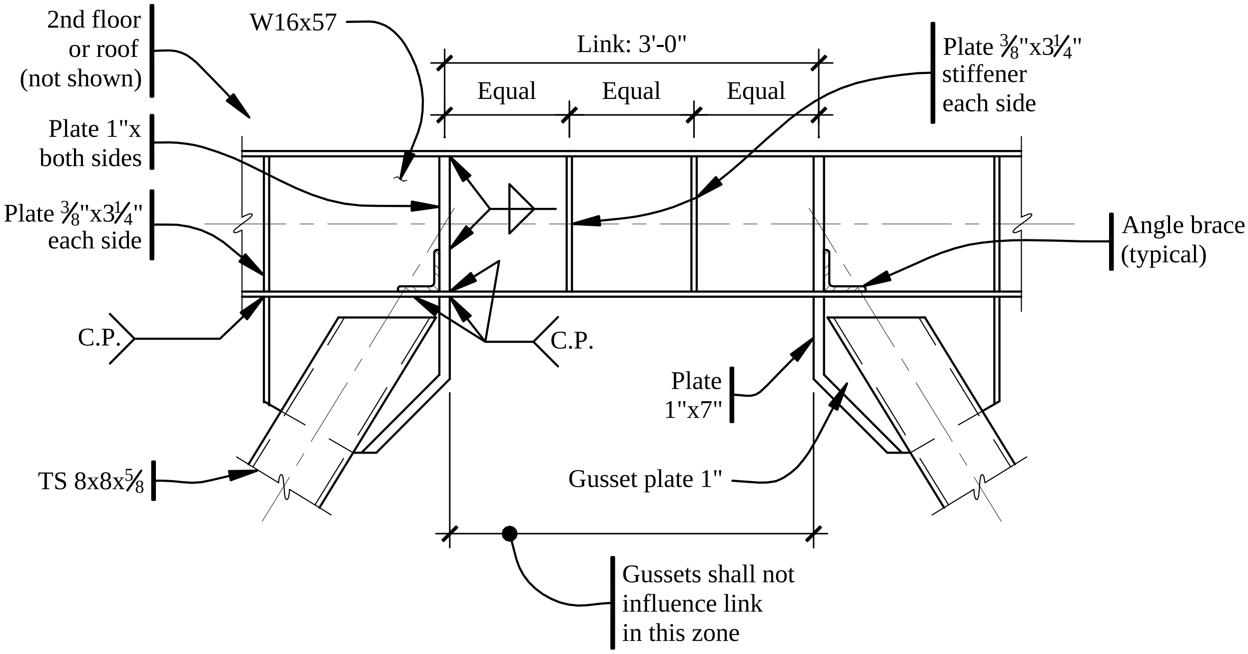


Figure 5.3-5 Link and upper brace connection (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

5.3.4.4 Brace Connections at Bottom of Brace

These braces are concentric at their lower end, framing into the column-girder intersection in a conventional manner.

The design of the gusset plate and welds is conventional. Details of a lower brace connection are shown in Figure 5.3-6. In order to be able to use R = 8, moment connections are required at the ends of the link beams (at the roof and second floor levels). Moment connections could be used, but are not required, outside of the EBF (e.g., the left beam in Figure 5.3-6) or at the bottom of the brace at the first floor (e.g., the right beam in Figure 5.3-6 if it is at the first floor level) . The beam on the left in Figure 5.3-6 could be a collector. If so, the connection must carry the axial load (force from floor deck to collector) that is being transferred through the beam to column connection to the link beam on the right side, as well as beam vertical loads.

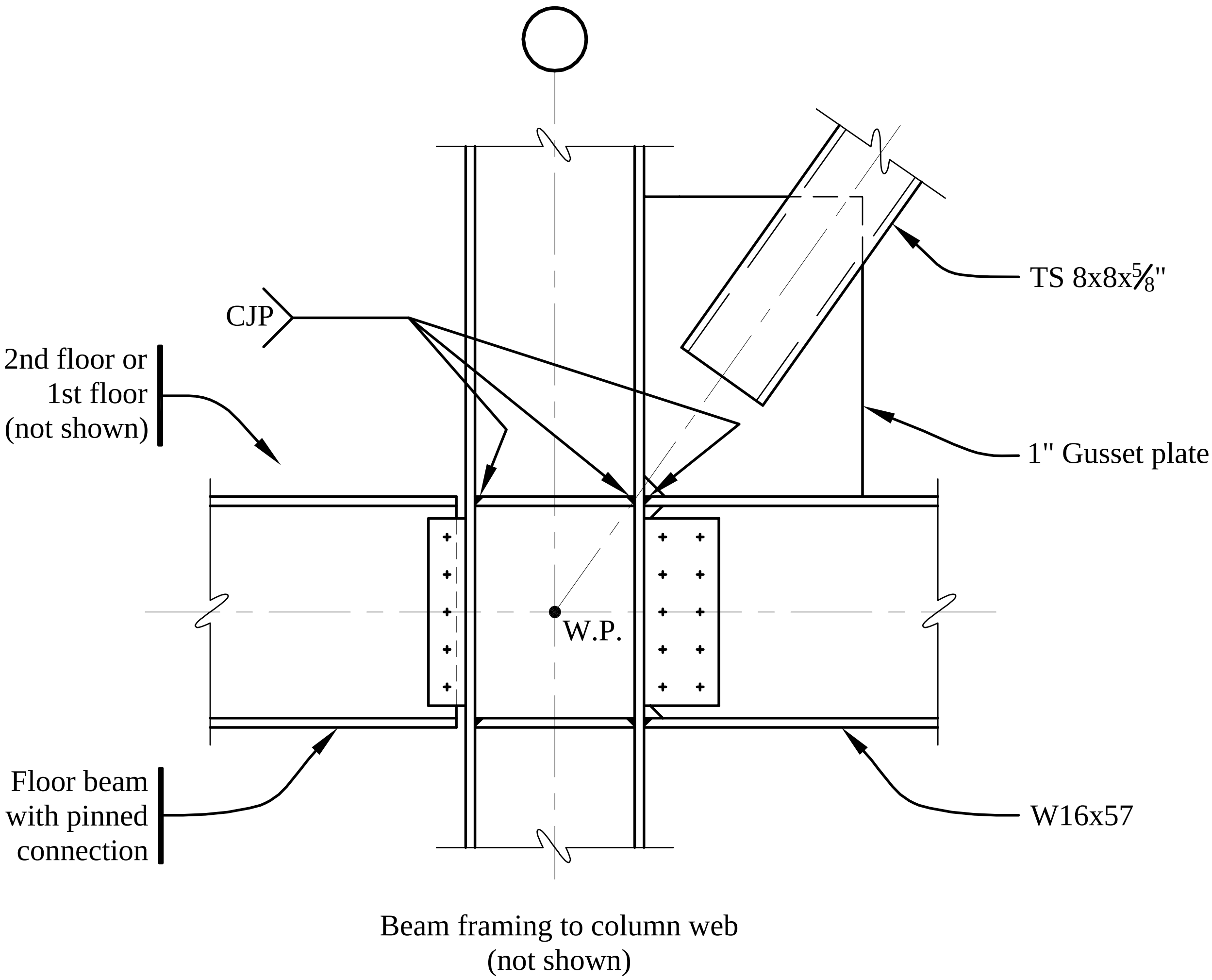


Figure 5.3-6 Lower brace connections (1.0 in. = 25.4 mm).

5.3.4.5 Beam and Column Design

Refer to AISC Seismic Sec. 15.6 for design of the beam outside the link and AISC Seismic Sec. 15.8 for design of the columns. The philosophy is very similar to that illustrated for the brace: the demand becomes the forces associated with expected yield of the link.. Although the moment and shear are less in the beam than in the link, the axial load is substantially higher.