Design Example 1B
Ordinary Concentric Braced Frame

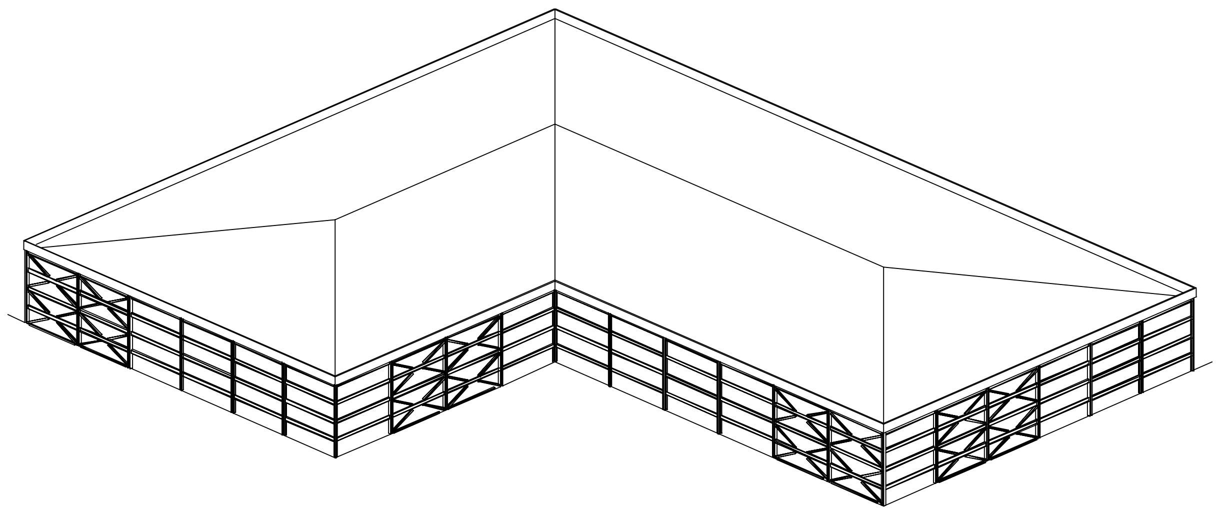

Figure 1B-1. Four-story steel frame office building with ordinary concentric braced frames (OCBF)

Overview

This Design Example illustrates the differences in design requirements for an ordinary concentric braced frame (OCBF) and a special concentric braced frame (SCBF) (illustrated in Design Example 1A). The same four-story steel frame structure from Example 1A is used in this Design Example (Figure 1B-1).

Building weights, dimensions, and site seismicity are the same as Example 1A.

Coefficients for seismic base shear are revised as required for the OCBF. The "typical design bay" is revised for the OCBF, and the results compared to those for the SCBF structure.

It is recommended that the reader first review Design Example 1A before reading this Design Example. Refer to Example 1A for plans and elevations of the structure (Figures 1A-1 through 1A-4).

In the Blue Book Commentary (C704.12), OCBFs are not recommended for areas of high seismicity or for essential facilities and special occupancy structures. SCBFs are preferred for those types of structures, since SCBFs are expected to perform better in a large earthquake due to their ductile design and detailing. OCBFs are considered more appropriate for use in one-story light-framed construction, non-building structures and in areas of low seismicity.

Outline

This Design Example illustrates the following parts of the design process:

1. **Design base shear.**
2. **Distribution of lateral forces.**
3. **Interstory drifts.**
4. **Braced frame member design.**
5. **Bracing connection design.**

Calculations and Discussion — Code Reference

1. Design base shear.

1a. Classify the structural system. §1629.6

The structure is a building frame system with lateral resistance provided by ordinary braced frames (System Type 2.4.a of Table 16-N). The seismic factors are:

$$R = 5.6$$

$$\Omega = 2.2$$

$$h_{max} = 160\,\text{ft}$$ Table 16-N

1b. Select lateral force procedure. §1629.8.3

The static lateral force procedure will be used, as permitted for irregular structures not more than five stories or 65 feet in height.

1c. Determine seismic response coefficients. §1629.4.3

For Zone 4 and Soil Profile Type S_D:

$$C_a = 0.44(N_a) = 0.44(1.0) = \underline{\underline{0.44}}$$ Table 16-Q

$$C_v = 0.64(N_v) = 0.64(1.08) = \underline{\underline{0.69}}$$ Table 16-R

1d. Evaluate structure period T.

From Design Example 1A:

$$T_B = \underline{\underline{0.57\,\text{sec}}}$$ §1630.2.2

1e. Determine design base shear. §1630.2.1

$$V = \frac{C_v I}{RT} W = \frac{0.69(1.0)}{5.6(0.57)} W = 0.216W \quad (30\text{-}4)$$

Base shear need not exceed:

$$V = \frac{2.5 C_a I}{R} W = \frac{2.5(0.44)(1.0)}{5.6} = 0.196W \quad (30\text{-}5)$$

For Zone 4, base shear shall not be less than:

$$V = \frac{0.8 Z N_v I}{R} W = \frac{0.8(0.4)(1.08)(1.0)}{5.6} = 0.062W \quad (30\text{-}7)$$

Equation 30-5 governs base shear.

$$\therefore V = \underline{\underline{0.196W}}$$

1f. Determine earthquake load combinations. §1630.1

Reliability/redundancy factor $\rho = 2 - \dfrac{20}{r_{max}\sqrt{A_b}}$ (30-3)

From Design Example 1A, use $\rho = 1.0$.

For the load combinations of §1630.1:

$$E = \rho E_h + E_v = 1.0(V) \quad (30\text{-}1)$$

$$E_m = \Omega E_h = 2.2(V) \quad (30\text{-}2)$$

2. Distribution of lateral forces.

2a. Building weights and mass distribution.

The weight and mass distribution for the building is shown in Table 1B-1. These values are taken from Design Example 1A.

Table 1B-1. Mass properties summary

Level	*W (kips)*	*X_{cg} (ft)*	*Y_{cg} (ft)*	*M (kip·sec²/in.)*	*MMI $(\text{kip}\cdot\text{sec}^2\cdot\text{in.})$*
Roof	6,687	161.1	1,80.9	17.3	316,931
4th	6,840	161.1	1,80.9	17.7	324,183
3rd	6,840	161.1	1,80.9	17.7	324,183
2nd	6,840	161.1	1,80.9	17.7	324,183
Total	27,207			70.4	

2b. Determine total base shear.

As noted above, Equation (30.5) governs, and

$$V = 0.196W = 0.196(27207) = \underline{\underline{5{,}333\,\text{kips}}} \quad (30\text{-}5)$$

2c. Determine vertical distribution of force. §1630.5

For the Static lateral force procedure, vertical distribution of force to each level is applied as follows:

$$F_x = \frac{(V - F_t)W_x h_x}{\sum W_i h_i} = V\left(\frac{W_x h_x}{\sum W_i h_i}\right) \tag{30-15}$$

Table 1B-2. Distribution of base shear

Level	w_x *(kips)*	h_x *(ft)*	$w_x h_x$ *(ft)*	$\frac{w_x h_x}{\Sigma w_x h_x}$	F_x *(kips)*	ΣV *(kips)*
Roof	6,687	62	414,594	0.39	2,064	
4th	6,840	47	321,480	0.30	1,600	2,064
3rd	6,840	32	218,880	0.20	1,090	3,665
2nd	6,840	17	116,280	0.11	579	4,754
Total	27,207		1,071,234	1.00	5,333	5,333

3. Calculate interstory drift.

3a. Determine Δ_M.

The maximum inelastic response displacement, Δ_M, is determined per §1630.9.2 as:

$$\Delta_M = 0.7(R)\Delta_S = 0.7(5.6)\Delta_S = 3.92\Delta_S \tag{30-17}$$

3b. Check story drift.

The maximum interstory drift (obtained from a computer analysis and summarized in Table 1A-7 of Design Example 1A) occurs in the north-south direction at the second story, and is 0.36 inches with $R = 5.6$. This value must be adjusted for the $R = 6.2$ used for OCBF systems.

$$\Delta_S \text{ drift } = \left(\frac{6.2}{5.6}\right)(0.36") = 0.40 \text{ in.}$$

$$\Delta_M \text{ drift } = 0.40(3.92) = 1.57 \text{ in.}$$

$$\text{Drift ratio } = \frac{1.57}{180} = 0.009 < 0.025 \quad o.k.$$ 1630.10.2

Comment: The elastic story displacement is greater for the SCBF than the OCBF, but the maximum inelastic displacement (Δ_M) is equivalent to the SCBF. Drift limitations rarely, if ever, govern braced frame designs. And, as a design consideration, there is essentially no difference in the calculated maximum drifts for OCBFs and SCBFs.

4. Braced frame member design.

Braced frame member design will be done using the same typical design bay as shown in Example 1A. SCBF member seismic forces are increased proportionally for the OCBF using a ratio of the R values. Member axial forces and moments for dead load and seismic loads are shown below (Figure 1B-2). All steel framing is designed per Chapter 22, Division V, Allowable Stress Design. Requirements for braced frames, except SCBF and EBF, are given in §2213.8.

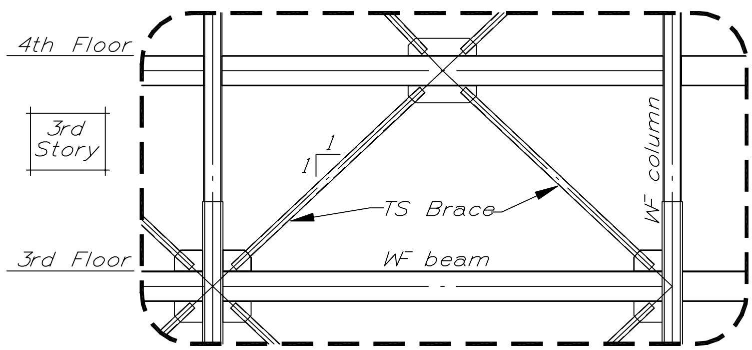

Figure 1B-2. Typical braced bay

TS brace @ 3rd story:

$$P_{DL} = 24 \text{ kips}$$

$$P_{LL} = 11\text{kips}$$

$$P_E = 400\text{kips}$$

WF beam @ 3rd floor:

$M_{DL} = 1600$ kip-in.

$M_{LL} = 1193$ kip-in.

$V_{DL} = 14.1$ kips

$V_{LL} = 10.3$ kips

$P_E = 83$ kips

WF column @ 3rd story:

$P_{DL} = 67$ kips

$P_{LL} = 30$ kips

$P_E = 130$ kips

$M_E \approx 0$

4a. Diagonal brace design at the 3rd story.

The basic ASD load combinations of §1612.3.1 with no one-third increase will be used.

$$D + \frac{E}{1.4}: \; P_1 = 24 + \frac{400}{1.4} = 310 \text{ kips (compression)} \qquad (12\text{-}9)$$

$$0.9D \pm \frac{E}{1.4}: \; P_2 = 0.9(24) - \frac{400}{1.4} = -264 \text{ kips (tension)} \qquad (12\text{-}10)$$

$$D + 0.75\left[L + \left(\frac{E}{1.4}\right)\right]: \; P_3 = 24 + 0.75\left[11 + \frac{400}{1.4}\right] = 246 \text{ kips (compression)} \qquad (12\text{-}11)$$

The compressive axial load of Equation (12-9) controls.

The unbraced length, l_w, of the TS brace is 18.5 feet.

The effective length $kl = 1.0(18.5) = 18.5$ feet .

Maximum slenderness ratio:

$$\frac{kl}{r} \le \frac{720}{\sqrt{F_y}} \qquad \text{§2213.8.2.1}$$

For a tube section:

$$F_y = 46\,\text{ksi}$$

$$\therefore \frac{720}{\sqrt{46}} = 106$$

Minimum $r = \dfrac{kl}{106} = \dfrac{12(18.5)}{106} = 2.09\,\text{in.}$

Maximum width-thickness ratio $\left(\dfrac{b}{t}\right) \le \dfrac{110}{\sqrt{F_y}} = 16.2$ §2213.8.2.5

Try TS $10 \times 10 \times 5/8$.

$$r = 3.78 > 2.09" \qquad o.k.$$

$$\frac{b}{t} = \frac{10}{0.625} = 16.0 < 16.2 \qquad o.k.$$

For an OCBF, the capacity of bracing members in compression must be reduced by the stress reduction factor “B” per §2213.8.2:

$$F_{as} = BF_a \qquad (13\text{-}4)$$

$$B = 1/\{1 + [(K\,l/r)/(2C_c)]\} \qquad (13\text{-}5)$$

where:

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}} \qquad \text{AISC-ASD §E2}$$

$$(Kl)/r = \frac{1.0(12)(18.5)}{3.78} = 58.7$$

$$B = \frac{1}{1 + [58.7/2(111.6)]} = 0.79$$

For $kl = 18.5\,\text{ft}$

$P_{allow} = 482\,\text{kips}$ AISC-ASD, pp. 3-41

$P_{as} = (0.79)(482) = 380 > 310\,\text{kips}$ *o.k.*

$\therefore$ Use TS $10 \times 10 \times 5/8$

4b. Girder design at the 3rd story.

From a review of Design Example 1A, the vertical load moment governs the girder design. With only a nominal increase in axial force from seismic loading, the girder is okay by inspection.

4c. Column design at the 3rd story.

The columns will be designed using the basic ASD load combinations with no one-third increase.

$$D + L: \quad P_1 = 67 + 30 = 97\,\text{kips (compression)} \tag{12-8}$$

$$D + \frac{E}{1.4}: \quad P_1 = 67 + \frac{130}{1.4} = 160\,\text{kips (compression)} \tag{12-9}$$

$$0.9D \pm \frac{E}{1.4}: \quad P_2 = 0.9(67) - \frac{130}{1.4} = 33\,\text{kips (tension)} \tag{12-10}$$

$$D + 0.75\left[L + \left(\frac{E}{1.4}\right)\right]: \quad P_3 = 67 + 0.75\left[30 + \frac{130}{1.4}\right] = 159\,\text{kips (compression)} \tag{12-11}$$

For the columns, ASTM A36 steel with $F_y = 36\,\text{ksi}$. The unbraced column height is:

$h = 15 - 1 = 14\,\text{ft}$

Per AISC-ASD manual, p. 3-30, select a $W10 \times 49$ column with $kl = 14\,\text{ft}$.

$P_{allow} = 242 > 160\,\text{kips}$ *o.k.* AISC-ASD pp. 3-30

$\therefore$ Use $W10 \times 49$ column

Note that without the local buckling compactness requirement of §2213.9.2.4, the $W10\times 49$ works in the OCBF, where a $W10\times 54$ is required for the SCBF of Example 1A. Also note that the special column strength requirements of §2213.5.1 do not apply to the OCBF. The relaxation of ductility requirements for the OCBF reflects lesser inelastic displacement capacity than the SCBF, hence the greater seismic design forces for the OCBF.

5. Braced connection design. §2213.8.3

The design provisions for OCBF connections are nearly identical to those for SCBF connections, with one significant difference. The SCBF requirements for gusset plates do not apply to OCBF connections. Therefore, the minimum "2*t*" setback, as shown in Figure 1A-19(a) of Design Example 1A for the SCBF, may be eliminated. This allows the end of the tube brace to extend closer to the beam-column intersection, thereby reducing the size of the gusset plate.

Under the requirements of §2213.8.3.1, the OCBF connections must be designed for the lesser of:

1. $P_{ST} = F_y A = 46(22.4) = 1030\text{ kips}$ §2213.8.3.1

2. $P_M = P_D + P_L + \Omega_M P_E = (24+11) + 2.2(400) = 915\text{ kips}$

3. Maximum force that can be transferred to brace by the system.

The remainder of the connection design follows the same procedure as for Design Example 1A, with all components designed for the $\underline{\underline{915\text{ kip}}}$ force derived above.