OPE

e Seismic Provisions requirements and other design considerations summarized in this rt apply to the design of the members and connections in moment frames that utilize a ponse modification factor, R, greater than 3.

In the Seismic Provisions, moment frames are separated into three categories: Ordinary oment Frame (OMF) systems, Intermediate Moment Frame (IMF) systems, and Special oment Frame (SMF) systems.

RDINARY MOMENT FRAME (OMF) SYSTEMS

dinary Moment Frame (OMF) systems resist lateral forces through the flexural and shear engths of the beams and columns. Lateral displacement is resisted primarily through the xural stiffness of the framing members and the restraint of relative rotation between the ams and columns at the connections, or "frame action." OMF systems must be capable of oviding an interstory drift angle of at least 0.01 radian. Note that the use of this system is rmitted within the limits of the applicable building code.

OMF systems tend to have larger and heavier beam and column sizes than in braced-me systems, as the beams and columns are often sized for drift control rather than ength. The increase in member sizes and related costs, however, may be acceptable based the increased flexibility in the architectural and mechanical layout in the structure. The sence of diagonal bracing members can provide greater freedom in the configuration of lls and in the routing of mechanical ductwork and piping. As with other moment frame stems, OMF systems are usually positioned at the perimeter of the structure, allowing ximum flexibility of the interior spaces. The flexible nature of the frames, however, does rrant some additional consideration of the interaction between the steel frame and more id architectural cladding systems.

The Seismic Provisions provide for two connection types when designing OMF sys-ns — fully restrained (FR) and partially restrained (PR). For the purposes of discussion, is defined as having sufficient rigidity in the connection to maintain the angles ween members upon application of loads, while PR is defined as having insufficient idity in the connection to maintain these angles upon load application.

FR moment connections are required to be designed to reach a required flexural strength,

$$M_u = 1.1 R_y M_p$$

ere,

R_y = ratio of the expected yield strength, F_{ye}, to the minimum specified yield strength, F_y (Seismic Provisions Table I–6-1)

M_p = nominal plastic flexural strength for either the beam or the girder

ernatively, the connections may be designed for the maximum force that can be delivered he frame by the surrounding components in the structural system, if this force is less than $R_y M_p$ of the beam or girder; refer to Seismic Provisions Commentary Section C11.2a. FR ment connections also require the removal of backing bars in complete-joint-penetration ove welds, except at top flange connections where the backing bar is connected to the umn with a continuous fillet weld. Upon removal of the backing material, the weld is

required to be back-gouged and provided with a reinforcing-fillet weld. If the connection utilizes weld-access holes, they are required to conform with the requirements of Seismic Provisions Section 11.2a(2). The required weld-access-hole configuration is shown in Seismic Provisions Figure 11–1. Finally, for FR moment connections that use double-sided partial-joint-penetration groove welds or double-sided fillet welds, such welds must be designed to support a required strength equal to $1.1R_yF_yA_g$ of the connected part. The Seismic Provisions also stipulate that single-sided partial-joint-penetration groove welds and single-sided fillet welds are not to be used to transmit tensile forces.

PR moment connections are required to develop similar strength limits as FR moment connections. In addition, it is noted that PR moment connections must have a nominal flexural strength no less than $0.50M_p$ of the connected beam or column. It is also noted that the strength and flexibility of the connection must be considered in the design, including the effects on overall frame stability.

Both FR and PR moment connections are required to have an available shear strength greater than the required strength, V_u, as determined by the load combination of $1.2D + 0.5L + 0.2S$ plus shear resulting from the application of a moment equaling,

$$2\left(\frac{1.1R_yM_p}{L_h}\right)$$

where L_h is the distance between plastic hinges.

A lesser value is permitted if justified by analysis. PR moment connections are further required to develop, in addition to the load combination above, the shear strength to resist the maximum end moment that can be resisted by the connections.

Continuity plates are required for FR moment connections when the connection utilizes a welded flange or a welded flange plate. Special requirements for the attachment of the continuity plates to the column are outlined in the Seismic Provisions. See Section 11.5.

OMF systems are not required to have any special detailing of the panel zones, and have no special requirements for the relationship between beam and column strength. This is indicative of the overall OMF system, where the detailing requirements are reduced and the seismic forces are larger than moment frame systems intended to provide higher amounts of ductility. This basic design philosophy for OMF systems allows for their use as an economical moment frame system when OMF systems are permitted by the applicable building code.

OMF DESIGN EXAMPLES

Example 4.1. OMF Story Drift and Stability Check

Given: Refer to the roof plan shown in Figure 4–1 and the OMF elevation shown in Figure 4–2. Determine if the frame satisfies the drift and stability requirements based on the following loading. The Applicable Building Code specifies the use of ASCE 7 for calculation of loads.

$D = 15$ psf	$S = 20$ psf	$L_r = 20$ psf
W12×35	$Z_x = 51.2$ in.^{3}	$A_g = 10.3$ in.^{2}
W18×40	$Z_x = 78.4$ in.^{3}	

From ASCE 7, the Seismic Use Group is I, the Seismic Design Category is D, $R = 3.5$, $C_d = 3$, $I = 1.0$, $\rho = 1.3$, and $S_{DS} = 0.533$.

$$0.2S_{DS} = 0.2(0.533) = 0.107$$

Solution: *Check drift*

From an elastic analysis of the structure that includes second-order effects and panel-zone deformations, the drift is,

$$\delta_{xe} = 1.20 \text{ in.}$$

Per Seismic Provisions Section 3, the Design Story Drift and the story drift limits are those stipulated by the Applicable Building Code. From ASCE 7, the allowable story drift, Δ_a, is $0.025h_{sx}$, where h_{sx} is the story height below level x.

$$\Delta_a = 0.025h_{sx} = 0.025(17 \text{ ft})(12 \text{ in./ft}) = 5.10 \text{ in.}$$

ASCE 7 defines the Design Story Drift as δ_x, the deflection of Level x at the center of mass.

$$\delta_x = \frac{C_d \delta_{xe}}{I} \qquad \text{(ASCE 7)}$$

$$\delta_x = \frac{3(1.20 \text{ in.})}{1.0}$$

$$= 3.60 \text{ in.} < 5.10 \text{ in.}$$

$\delta_x < \Delta_a$ **o.k.**

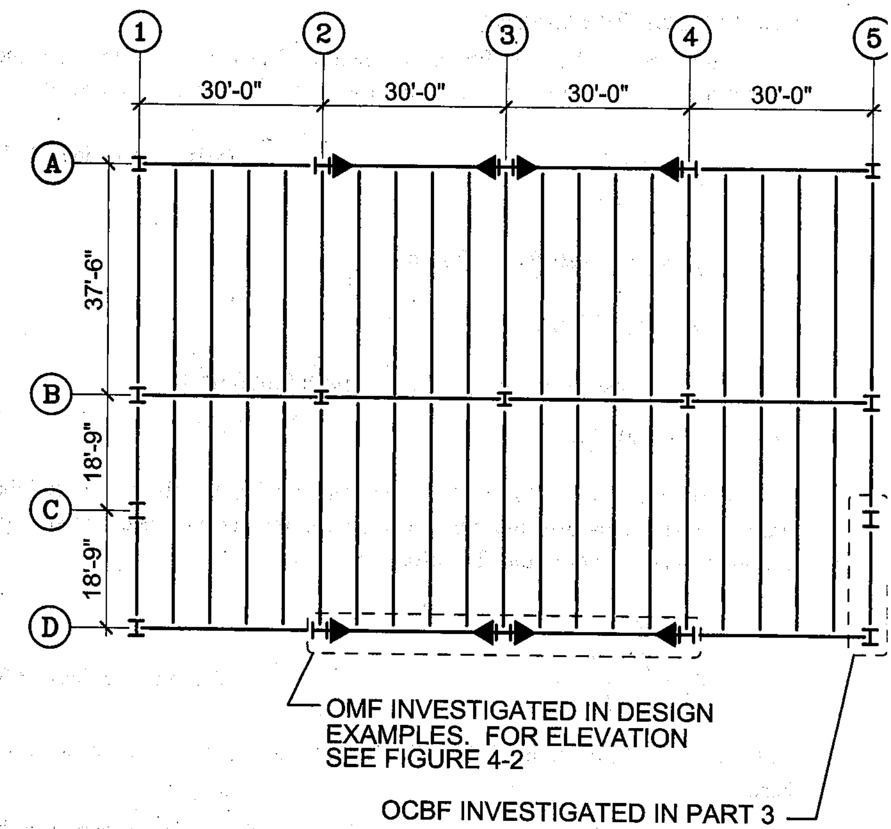

Figure 4–1. OMF and OCBF roof plan for Examples 3.1, 3.2, 3.3, 4.1, 4.2, and 4.3.

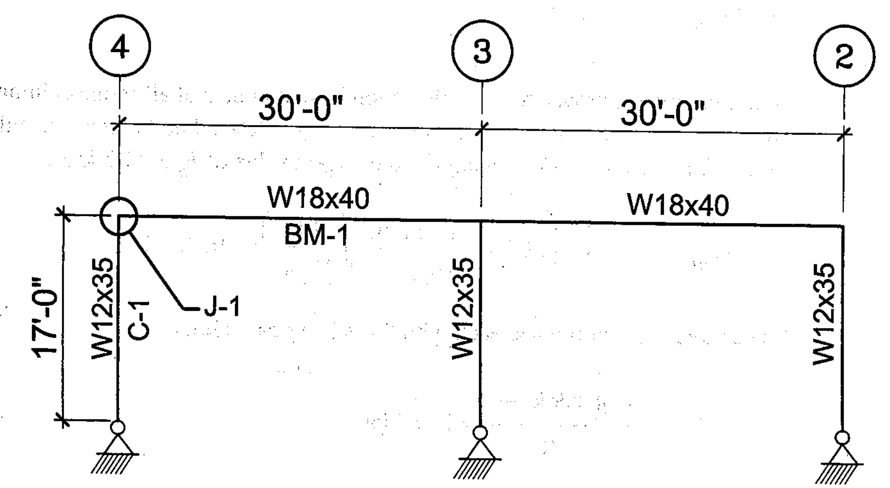

Figure 4–2. OMF elevation for Examples 4.1, 4.2, 4.3, and 4.4.

Check frame for instability

The commentary to Seismic Provisions Section 3 provides a method for the evaluation of the *P*-Δ effects on moment frames based on a drift index, Ψ_i.

$$\Psi_i = \frac{P_i R \Delta_i}{V_{yi} H} \qquad \text{(Seismic Provisions C3-1)}$$

$$A_{roof} = 75\ \text{ft}(120\ \text{ft}) = 9{,}000\ \text{ft}^2$$

$$\begin{aligned} P_i &= P_D + 0.25 P_{L_r} \\ &= 15\ \text{psf}\left(9{,}000\ \text{ft}^2\right) + 0.25\left(20\ \text{psf}\right)\left(9{,}000\ \text{ft}^2\right) \\ &= 180\ \text{kips} \end{aligned}$$

Since the columns have a lower plastic section modulus than the girders and the frame is continuous over multiple bays, all columns do not meet the strong-column weak-beam criterion. Therefore,

$$V_{yi} = \frac{\sum_{k=1}^{m} M_{pC_k}}{H} \qquad \text{(Seismic Provisions C3-3)}$$

Note that half of Equation C3-3 applies because the columns are pinned at the base.

The calculation of M_{pC_k} requires the consideration of the axial loads present in each column. Seismic Provisions Section 9.6 allows this to be calculated as,

$$M_{pC_k} = Z_x\left(F_y - \frac{P_u}{A_g}\right)$$

A conservative approach to this calculation is to assume that all frame columns have the same axial load and that this load is equal to load on the most heavily loaded frame column. From analysis, the largest value of P_u is 11.3 kips.

$$M_{pC_k} = 51.2\ \text{in.}^3\left(50\ \text{ksi} - \frac{11.3\ \text{kips}}{10.3\ \text{in.}^2}\right)\left(\frac{1\ \text{ft}}{12\ \text{in.}}\right) = 209\ \text{kip-ft}$$

Since there are two frames, each with three identical columns,

$$V_{yi} = \frac{2(3)(209\ \text{kip-ft})}{17\ \text{ft}} = 73.8\ \text{kips}$$

Therefore,

$$\Psi = \frac{180\ \text{kips}(3.5)(1.20\ \text{in.})}{73.8\ \text{kips}(17\ \text{ft})(12\ \text{in./ft})} = 0.0502$$

Per Seismic Provisions Commentary Section C3, since Ψ < 0.3, the struc[t] is considered stable.

Alternatively, a more detailed analysis can be completed to reflect the axial [l] on each column. Such an analysis would result in an increase in the plastic s[tory] shear quantity, V_{yi}, and thus a reduction in the drift index.

Example 4.2. OMF Column Design

Given: Refer to Column C-1 in Figure 4–2. Determine the adequacy of the ASTM A[…] wide-flange section (F_y = 50 ksi, F_u = 65 ksi) for the following loading. Applicable Building Code specifies the use of ASCE 7 for calculation of loa[…]

$P_D = 9$ kips $P_{L_r} = 7$ kips $P_S = 12$ kips $P_{Q_E} = \pm 2$ kip[…]

$V_D = 0.49$ kips $V_{L_r} = 0.40$ kips $V_S = 0.65$ kips $V_{Q_E} = \pm 2.23$

$M_{D\,top} = 8$ kip-ft $M_{L_r\,top} = 7$ kip-ft $M_{S\,top} = 11$ kip-[…]

$M_{Q_E\,top} = \pm 36.3$ kip-ft

W12×35	$d = 12.5$ in.	$t_w = 0.300$ in.	$I_x = 285$ in.
	$A_g = 10.3$ in.^{2}	$r_x = 5.25$ in.	$r_y = 1.54$ in
	$S_x = 45.6$ in.^{3}	$Z_x = 51.2$ in.^{3}	$r_{ts} = 1.79$
	$\dfrac{Jc}{S_x h_o} = 0.00135$		
W18×40	$I_x = 612$ in.^{4}		

From ASCE 7, Seismic Design Category is D, ρ = 1.3, and $S_{DS} = 0.533$.

$$0.2S_{DS} = 0.2(0.533) = 0.107$$

Assume there is no transverse loading between the column supports in plane of bending and that the beam framing into the column's weak axis [pro]duces a negligible moment out of plane of the frame.

Solution: *Check column element slenderness*

The width-thickness ratio for the flanges is

$$\lambda_f = \frac{b_f}{2t_f} = 6.31 \quad \text{(Specification B4.1)}$$

For flexure, the limiting width-thickness ratio for compact flanges is,

$$\lambda_p = 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}} = 9.15 \quad \text{(Specification Table B4.1)}$$

Since $\lambda_f < \lambda_p$, the flanges are compact for flexure.

The width-thickness ratio for the web is,

$$\lambda_w = \frac{h}{t_w} = 36.2 \quad \text{(Specification B4.2)}$$

For flexure, the limiting width-thickness ratio for a compact web is,

$$\lambda_p = 3.76\sqrt{\frac{E}{F_y}} \quad \text{(Specification Table B4.1)}$$

$$\lambda_p = 3.76\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}} = 90.6$$

Since $\lambda_w < \lambda_p$, the web is compact for flexure.

Therefore, the W12×35 is compact for flexure. For compression, similar checks can be made. Alternatively, the listing for a W12×35 in Manual Table 1–1 has a footnote indicating that the web is slender. Therefore, the column strength may be reduced by the web slenderness.

Check unbraced length

From Manual Table 3–2,

$L_p = 5.44$ ft $\qquad L_r = 16.7$ ft

$L_b = 17.0 \text{ ft} > L_r$

Determine K

For the X-X axis,

$$G = \frac{\sum\left(I_c/L_c\right)}{\sum\left(I_g/L_g\right)} \quad \text{(Specification Figure C–C2.4)}$$

With one lateral-frame beam and one lateral-frame column at the connect located at the column top,

$$G_{top} = \frac{285 \text{ in.}^4/17 \text{ ft}}{612 \text{ in.}^4/30 \text{ ft}} = 0.822$$

From the notes for Commentary Figures C–C2.3 and C–C2.4, G = 10 fc pinned-base connection. Using Specification Figure C–C2.4, K_x = 1.9 (note t it may be possible to determine a reduced value by accounting for inela column behavior through the use of the stiffness reduction factor, τ_a, in calculation of G).

The leaning column amplifier is (Lim and McNamara, 1972),

$$\sqrt{1+\frac{\sum P_{leaning}}{\sum P_{stability}}}$$

With 3,375 ft² tributary to the stability columns and 5,625 ft² tributary to leaning columns, and a uniform load over the entire area,

$$\sqrt{1+\frac{5{,}625 \text{ ft}^2}{3{,}375 \text{ ft}^2}} = 1.63$$

Therefore,

$$K_x = 1.63 \times 1.9 = 3.10$$

From Specification Section C1.3a and Commentary Table C–C2.2, K_y = 1.0

Determine the compression strength of the column

$$\frac{K_x L_x}{r_x} = \frac{3.10(17 \text{ ft})\left(12 \text{ in.}/\text{ft}\right)}{5.25 \text{ in.}} = 120$$

$$\frac{K_y L_y}{r_y} = \frac{1.0(17.0 \text{ ft})\left(12 \text{ in.}/\text{ft}\right)}{1.54 \text{ in.}} = 132$$

Using Manual Table 6–1 with K_yL_y = 17 ft,

$$p = 0.00751 \text{ kips}^{-1}$$

$$\phi_c P_n = \frac{1}{p} = 133 \text{ kips}$$

Determine the flexural strength

From Specification Section F2, with compact flanges and web and $L_b > L_r$, the applicable limit states are yielding and lateral-torsional buckling.

$$M_n = F_{cr}S_x \le F_yZ_x \quad \text{(Specification F2-3)}$$

$$F_{cr} = \frac{C_b\pi^2 E}{\left(L_b/r_{ts}\right)^2}\sqrt{1+0.078\left(\frac{Jc}{S_xh_o}\right)\left(\frac{L_b}{r_{ts}}\right)^2} \quad \text{(Specification F2-4)}$$

Since this column has no intermediate loads, the moment diagram is a straight line, and the alternative C_b equation from the Commentary to the Specification can be used.

$$C_b = 1.75+1.05\left(\frac{M_1}{M_2}\right)+0.3\left(\frac{M_1}{M_2}\right)^2 \quad \text{(Specification C-F1-1)}$$

With $M_1 = 0$,

$$C_b = 1.75$$

$$F_{cr} = \frac{1.75\pi^2 29{,}000}{\left(\frac{17\times12}{1.79}\right)^2}\sqrt{1+0.078\left(0.00135\right)\left(\frac{17\times12}{1.79}\right)^2}$$

$$= 59.3 \text{ ksi}$$

$$M_n = 59.3 \text{ ksi}\left(45.6 \text{ in.}^3\right)\left(\frac{1 \text{ ft}}{12 \text{ in.}}\right) \le 50 \text{ ksi}\left(51.2 \text{ in.}^3\right)\left(\frac{1 \text{ ft}}{12 \text{ in.}}\right)$$

$$= 225 \text{ kip-ft} \le 213 \text{ kip-ft}$$

$$= 213 \text{ kip-ft}$$

$$\phi_b M_n = 0.90\left(213 \text{ kip-ft}\right) = 192 \text{ kip-ft}$$

Consider second-order effects

$$B_1 = \frac{C_m}{1-\frac{\alpha P_r}{P_{e1}}} \ge 1 \quad \text{(Specification C2-2)}$$

For the calculation of B_1,

$$P_r = P_{nt} + P_{lt}$$

$$\alpha = 1.0$$

$$C_m = 0.6-0.4\left(\frac{M_1}{M_2}\right) \quad \text{(Specification C2}$$

With $M_1 = 0$,

$$C_m = 0.6$$

Considering the load combinations given in ASCE 7, it was determined that t governing load combination for the column is,

$$1.2D + 1.0E + 0.5L + 0.2S \quad \text{(ASCE}$$

Therefore, for the axial load,

$$P_u = (1.2+0.2S_{DS})P_D + \rho P_{Q_E} + 0.5P_L + 0.2P_S$$

$$= 1.31P_D + 1.3P_{Q_E} + 0.5P_L + 0.2P_S$$

$$P_{nt} = 1.31(9 \text{ kips})+1.3(0 \text{ kips})+0.5(0 \text{ kips})+0.2(12 \text{ kips})$$

$$= 14.2 \text{ kips}$$

$$P_{lt} = 1.31(0 \text{ kips})+1.3(2 \text{ kips})+0.5(0 \text{ kips})+0.2(0 \text{ kips})$$

$$= 2.60 \text{ kips}$$

$$P_{e1} = \frac{\pi^2 EI}{(KL)^2} = \frac{\pi^2\left(29{,}000 \text{ ksi}\right)\left(285 \text{ in.}^4\right)}{\left[1.0\left(17 \text{ ft}\right)\left(12 \text{ in./ft}\right)\right]^2} = 1{,}960 \text{ kips} \quad \text{(Specificat C2}$$

Therefore,

$$B_1 = \frac{0.6}{1-\left[\frac{\left(1.0\right)16.8 \text{ kips}}{1{,}960 \text{ kips}}\right]} \ge 1.0$$

$$= 0.605 \ge 1.0$$

$$= 1.0$$

$$B_2 = \frac{1}{1-\left(\frac{\alpha\Sigma P_{nt}}{\Sigma P_{e2}}\right)} \ge 1 \quad \text{(Specification C2}$$

Assuming no translation due to gravity loads and all translation is due to seismic load,

$$\Sigma P_{nt} = 1.31\left(15 \text{ psf}\right)\left(9{,}000 \text{ ft}^2\right)+1.3\left(0\right)$$

$$+0.5\left(0\right)+0.2\left(20 \text{ psf}\right)\left(9{,}000 \text{ ft}^2\right)$$

$$= 213 \text{ kips}$$

$$P_{e2} = \frac{\pi^2 EI}{(KL)^2} = \frac{\pi^2 (29{,}000 \text{ ksi})(285 \text{ in.}^4)}{\left[3.10(17 \text{ ft})(12 \text{ in./ft})\right]^2} = 204 \text{ kips}$$

Since there are two frames, and ignoring the potential reduction in K_x for the middle column,

$$\Sigma P_{e2} = 2(3P_{e2}) = 6(204 \text{ kips}) = 1{,}220 \text{ kips}$$

Therefore,

$$B_2 = \frac{1}{1 - \frac{(1.0)(213 \text{ kips})}{1{,}220 \text{ kips}}} = 1.21$$

$$\begin{aligned} P_r &= P_{nt} + B_2 P_{lt} \\ &= 14.2 \text{ kips} + 1.21(2.60 \text{ kips}) \\ &= 17.3 \text{ kips} \end{aligned}$$

$$\begin{aligned} M_{nt} &= 1.31(8 \text{ kip-ft}) + 1.3(0 \text{ kip-ft}) + 0.5(0 \text{ kip-ft}) + 0.2(11 \text{ kip-ft}) \\ &= 12.7 \text{ kip-ft} \end{aligned}$$

$$\begin{aligned} M_{lt} &= 1.31(0 \text{ kip-ft}) + 1.3(36.3 \text{ kip-ft}) + 0.5(0 \text{ kip-ft}) + 0.2(0 \text{ kip-ft}) \\ &= 47.2 \text{ kip-ft} \end{aligned}$$

$$M_r = B_1 M_{nt} + B_2 M_{lt} \qquad \text{(Specification C2-1a)}$$

$$\begin{aligned} M_r &= 1.0(12.7 \text{ kip-ft}) + 1.21(47.2 \text{ kip-ft}) \\ &= 69.8 \text{ kip-ft} \end{aligned}$$

$$V_u = (1.2 + 0.2S_{DS})V_D + \rho V_{Q_E} + 0.5V_L + 0.2V_S \qquad \text{(ASCE 7)}$$

$$\begin{aligned} V_u &= 1.31(0.49 \text{ kips}) + 1.3(2.23 \text{ kips}) + 0.5(0 \text{ kips}) + 0.2(0.65 \text{ kips}) \\ &= 3.67 \text{ kips} \end{aligned}$$

Check combined loading

$$\frac{P_r}{P_c} = \frac{17.3 \text{ kips}}{133 \text{ kips}} = 0.130$$

Since $P_r/P_c < 0.2$,

$$\frac{P_r}{2\phi P_c} + \frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \le 1.0 \qquad \text{(Specification H1-1b)}$$

$$\frac{0.130}{2} + \frac{69.8 \text{ kip-ft}}{192 \text{ kip-ft}} + 0 = 0.429$$

$0.429 < 1.0$ **o.k.**

Check the shear strength of the column

$$2.24\sqrt{\frac{E}{F_y}} = 2.24\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}} = 53.9$$

Since $h/t_w = 36.2 < 53.9$,

$$V_n = 0.6F_y A_w C_v \qquad \text{(Specification G2-1)}$$

$$C_v = 1.0 \qquad \text{(Specification G2-2)}$$

$$\begin{aligned} \phi_v V_n &= 1.0(0.6)(50 \text{ ksi})(12.5 \text{ in.})(0.300 \text{ in.})(1.0) \\ &= 113 \text{ kips} > 3.67 \text{ kips} \end{aligned}$$

> Alternatively, using Table 4–2 ($\phi = 1.00$) for the W12×35 column,
>
> $$\phi_v V_n = \phi R_{v1} = 113 \text{ kips}$$
>
> Note that for shapes with $h/t_w > 2.24\sqrt{E/F_y}$, $\phi_v = 0.90$ must be used in the shear strength check.

$V_u < \phi_v V_n$ **o.k.**

The W12×35 is adequate to resist the loads given for Column C-1.

Example 4.3. OMF Beam Design

Given: Refer to Beam BM-1 in Figure 4–2. Determine the adequacy of the ASTM A992 wide-flange section ($F_y = 50$ ksi, $F_u = 65$ ksi) for the following loading. The Applicable Building Code specifies the use of ASCE 7 for calculation of loads. The governing load combination, which includes seismic effects, is,

$$1.2D + 1.0E + 0.5L + 0.2S \qquad \text{(ASCE 7)}$$

The beam required strengths are,

$M_u = -77.2$ kip-ft $\qquad V_u = 9.5$ kips

The factored beam moments at the quarter points are,

$M_{1/4} = -10.9$ kip-ft $M_{1/2} = 29.9$ kip-ft $M_{3/4} = -45.1$ kip-ft

W18×40 $d = 17.9$ in. $t_w = 0.315$ in. $r_y = 1.27$ in.

$S_x = 68.4$ in.^{3} $Z_x = 78.4$ in.^{3}

$\frac{Jc}{S_x h_o} = 0.000681$ $r_{ts} = 1.56$ in.

Assume that the beam flanges are braced at the columns.

Solution: *Check beam element slenderness*

The width-thickness ratio for the flanges is,

$$\lambda_f = \frac{b_f}{2t_f} = 5.73$$

The limiting width-thickness ratio for compact flanges is,

$$\lambda_p = 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}} = 9.15 \quad \text{(Specification Table B4.1)}$$

Since $\lambda_f < \lambda_p$, the flanges are compact.

The width-thickness ratio for the web is,

$$\lambda_w = \frac{h}{t_w} = 50.9$$

The limiting width-thickness ratio for a compact web is,

$$\lambda_p = 3.76\sqrt{\frac{E}{F_y}} = 3.76\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}} = 90.6 \quad \text{(Specification Table B4.1)}$$

Since $\lambda_w < \lambda_p$, the web is compact.

Check unbraced length

From Manual Table 3-2,

$L_p = 4.49$ ft $L_r = 13.1$ ft

$L_b = 30.0$ ft $> L_r$

Note that the infill beams are not described in this example. If the actual framing were suitable to brace the flanges of the beam being designed, L_b could be reduced to 6 ft.

Determine the flexural strength

From Specification Section F2, with compact flanges and web and $L_b > L_r$, the applicable limit states are yielding and lateral-torsional buckling.

$$M_n = F_{cr}S_x \le F_y Z_x \quad \text{(Specification F2-3)}$$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2} \quad \text{(Specification F2-4)}$$

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \quad \text{(Specification F1-1)}$$

where,

$M_{max} = |M_u|$ $M_A = |M_{1/4}|$ $M_B = |M_{1/2}|$ $M_C = |M_{3/4}|$

$$C_b = \frac{12.5(77.2 \text{ kip-ft})}{2.5(77.2 \text{ kip-ft}) + 3(10.9 \text{ kip-ft}) + 4(29.9 \text{ kip-ft}) + 3(45.1 \text{ kip-ft})}$$
$$= 2.01$$

$$F_{cr} = \frac{(2.01)\pi^2(29{,}000 \text{ ksi})}{\left[\frac{(30 \text{ ft})(12 \text{ in./ft})}{1.56 \text{ in.}}\right]^2}\sqrt{1 + 0.078(0.000681)\left[\frac{(30 \text{ ft})(12 \text{ in./ft})}{1.56 \text{ in.}}\right]^2}$$

$$F_{cr} = 21.1 \text{ ksi}$$

$$M_n = F_{cr}S_x \le F_y Z_x$$
$$= \frac{21.1 \text{ ksi}\left(68.4 \text{ in.}^3\right)}{12 \text{ in./ft}} \le \frac{50 \text{ ksi}\left(78.4 \text{ in.}^3\right)}{12 \text{ in./ft}}$$
$$= 120 \text{ kip-ft} \le 327 \text{ kip-ft}$$
$$= 120 \text{ kip-ft}$$

$$\phi_b M_n = 0.90(120 \text{ kip-ft}) = 108 \text{ kip-ft} > 77.2 \text{ kip-ft}$$

$M_u < \phi M_n$ **o.k.**

Check shear strength

$$2.24\sqrt{\frac{E}{F_y}} = 2.24\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}} = 53.9$$

Since $h/t_w < 2.24\sqrt{E/F_y}$,

$V_n = 0.6F_yA_wC_v$ (Specification G2-1)

$C_v = 1.0$ (Specification G2-2)

$\phi V_n = 1.00(0.6)(50\text{ ksi})(17.9\text{ in.})(0.315\text{ in.})(1.0)$

$= 169\text{ kips} > 9.5\text{ kips}$

> Alternatively, using Table 4–2 ($\phi = 1.00$) for the W18×40 beam,
>
> $\phi V_n = \phi R_{v1} = 169$ kips
>
> Note that for shapes with $h/t_w > 2.24\sqrt{E/F_y}$, $\phi_v = 0.90$ must be used in the shear strength check.

$V_u < \phi V_n$ **o.k.**

The W18×40 is adequate to resist the loads given for Beam BM-1.

Note that load combinations that do not include seismic effects must also be investigated. For example, considering the load combination $1.2D + 1.6S$,

$M_u = 94.0$ kip-ft $\quad V_u = 14.4$ kips

$C_b = 2.14 \quad \phi_bM_n = 115$ kip-ft

$M_u < \phi_bM_n$ **o.k.**

$V_u < \phi V_n$ **o.k.**

Example 4.4. OMF Beam-Column Connection Design

Given: Refer to Joint J-1 in Figure 4–2. Design a directly welded flange FR moment connection for the joint shown in Figure 4–3. The beam and column are ASTM A992 wide-flange sections ($F_y = 50$ ksi, $F_u = 65$ ksi). Use 70-ksi electrodes.

W12×35	$d = 12.5$ in.	$t_w = 0.300$ in.	$b_f = 6.56$ in.
	$A_g = 10.3$ in.^{2}	$k_{des} = 0.820$ in.	$k_{det} = 1^3/_{16}$ in.
	$T = 10^1/_8$ in.	$t_f = 0.520$ in.	$Z_x = 51.2$ in.^{3}
W18×40	$d = 17.9$ in.	$t_w = 0.315$ in.	$t_f = 0.525$ in.
	$b_f = 6.02$ in.	$Z_x = 78.4$ in.^{3}	$k_1 = {}^{13}/_{16}$ in.

$V_D = 3.40$ kips $\quad V_S = 4.50$ kips $\quad V_{L_r} = 4.50$ kips

Solution: *Determine the expected shear force at the plastic hinge*

For this framing arrangement, the plastic hinge will form in the column, so the maximum moment that can be delivered to the connection is the column moment strength, which should be taken as $1.1R_yM_{pcol}$.

Assuming plastic hinges form at the top of each column in the frame, per Seismic Provisions Section 11.2a, the required shear strength is,

$$V_{beam} = \frac{3(1.1R_yM_{pcol})}{2L}$$

For ASTM A992, $R_y = 1.1$.

$$1.1R_yM_p = 1.1R_yF_yZ_x = 1.1(1.1)(50\text{ ksi})(51.2\text{ in.}^3) = 3{,}100\text{ kip-in.}$$

$$\begin{aligned} V_{gravity} &= 1.2V_D + 0.5V_L + 0.2V_S \\ &= 1.2(3.4\text{ kips}) + 0.5(0\text{ kips}) + 0.2(4.5\text{ kips}) \\ &= 4.98\text{ kips} \end{aligned}$$

$$\begin{aligned} V_{u\,conn} &= V_{gravity} + V_{beam} \\ &= 4.98\text{ kips} + \frac{3(3{,}100\text{ kip-in.})}{2(30\text{ ft} \times 12\text{ in./ft})} \\ &= 17.9\text{ kips} \end{aligned}$$

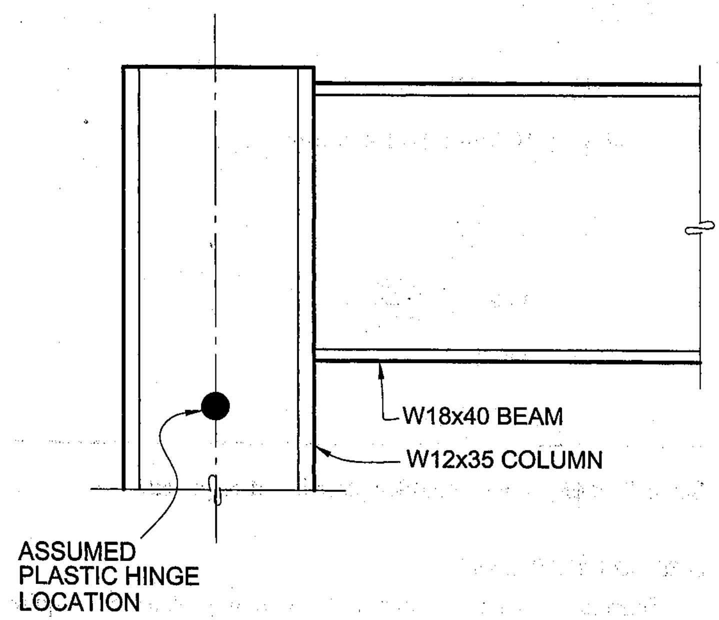

Figure 4–3. Connection given in Example 4.4.

Check column panel zone shear strength

Using the factored moment given in Example 4.3, the required panel zone strength for this single-sided moment connection is,

$$R_u = \frac{M_u}{d_b - t_f} = \frac{77.2 \text{ kip-ft}(12 \text{ in./ft})}{17.9 \text{ in.} - 0.525 \text{ in.}} = 53.3 \text{ kips}$$

From Example 4.1, panel zone deformations were included in the analysis of the structure and from Example 4.2, $P_{uc} = 16.8$ kips.

$$0.75P_y = 0.75(50 \text{ ksi})(10.3 \text{ in.}^2) = 386 \text{ kips}$$

When $P_u < 0.75P_y$, the panel zone shear strength is,

$$R_n = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_w}\right) \qquad \text{(Specification J10-11)}$$

$$\phi R_n = 0.9(0.60)(50 \text{ ksi})(12.5 \text{ in.})(0.300 \text{ in.})\left(1 + \frac{3(6.56 \text{ in.})(0.520 \text{ in.})^2}{17.9 \text{ in.}(12.5 \text{ in.})(0.300 \text{ in.})}\right)$$

$$= 109 \text{ kips} > 53.3 \text{ kips}$$

> Alternatively, using Table 4–2 ($\phi = 1.00$) for the W12×35 column,
>
> $$0.75P_y = 386 \text{ kips}$$
>
> $$\phi R_{v1} = (113 \text{ kips}) = 113 \text{ kips}$$
>
> $$\phi R_{v2} = (160 \text{ kip-in.}) = 160 \text{ kip-in.}$$
>
> $$\phi R_n = \phi R_{v1} + \frac{\phi R_{v2}}{d_b}$$
>
> $$= 113 \text{ kips} + \frac{160 \text{ kip-in.}}{17.9 \text{ in.}}$$
>
> $$= 122 \text{ kips}$$
>
> For $\phi = 0.9$, $\phi R_n = 109$ kips.

Since $R_u < \phi R_v$, a web doubler plate is not required.

Size continuity plates

Per Seismic Provisions Section 11.5, continuity plates are required because $t_{cf} < b_f/6$. A thickness greater than or equal to one-half of the beam flange thickness is required. With $t_f = 0.525$ in., a continuity plate thickness of $^5/_{16}$ in. will work. The minimum stiffener width to match the beam flange width is,

$$\frac{b_{fb} - t_{wc}}{2} = \frac{6.02 \text{ in.} - 0.300 \text{ in.}}{2} = 2.86 \text{ in.}$$

Use two pairs of $^5/_{16}$-in. × 3-in. continuity plates in the column at the locations of the top and bottom beam flanges.

Design connection of continuity plates to column flange

Assume that plate material with a minimum yield strength of 36 ksi is used for the continuity plates. Seismic Provisions Section 11.5 specifies that the required strength of this connection must be greater than or equal to the design strength of the contact area of the continuity plate with the column flange. To develop the contact area in tension and using the increase in strength for transversely loaded fillet welds allowed by Specification Section J2.4, the minimum required double-sided fillet weld size is,

$$D_{min} = \frac{\phi F_y t}{2(1.5)(1.392 \text{ kips/in.})} = \frac{0.90(36 \text{ ksi})(5/16 \text{ in.})}{2(1.5)(1.392 \text{ kips/in.})} = 2.42$$

Use double-sided, $^3/_{16}$-in. fillet welds to connect the continuity plates to the column flanges, which satisfies the minimum size of fillet weld permitted by Specification Table J2.3.

Design connection of continuity plates to column web

To satisfy the requirements in Seismic Provisions Section 7.5, the inner corners of the continuity plates along the web must be clipped approximately,

$$k_{det} - t_f + 1\tfrac{1}{2} \text{ in.} = 1\tfrac{3}{16} \text{ in.} - \tfrac{1}{2} \text{ in.} + 1\tfrac{1}{2} \text{ in.} = 2\tfrac{3}{16} \text{ in.}$$

Use a $2^1/_4$-in. clip dimension along the web.

Seismic Provisions Section 11.5 specifies that the required strength of the connection of the continuity plates to the column web be the least of the following four forces:

1. The sum of the design strengths of the connections of the continuity plate to the column flanges. With a corner clip along the flange of $^3/_4$ in.,

$$R_u = 2\phi F_y A_{gt}$$
$$= 2(0.90)(36 \text{ ksi})(3 \text{ in.} - \tfrac{3}{4} \text{ in.})(\tfrac{5}{16} \text{ in.})$$
$$= 45.6 \text{ kips}$$

2. The design shear strength of the contact area of the plate with the column web. With a corner clip along the web of 2¼ in.,

$$\begin{aligned} L &= d_c - 2t_{fc} - 2(2.25\text{ in.}) \\ &= 12.5\text{ in.} - 2(0.520\text{ in.}) - 2(2.25\text{ in.}) \\ &= 6.96\text{ in.} \end{aligned}$$

$$\begin{aligned} R_u &= \phi 0.6 F_y A_{gv} \\ &= 1.00(0.6)(36\text{ ksi})(6.96\text{ in.})(5/16\text{ in.}) \\ &= 47.0\text{ kips} \end{aligned}$$

3. The weld design strength that develops the design shear strength of the column panel zone.

$$R_u = \frac{122\text{ kips}}{2} = 61.0\text{ kips}$$

4. The actual force transmitted by the stiffener. This is determined as half of the difference between the beam flange force and the least design strength from Specification Section J10. The column strength is based on the flange force being delivered less than d from the top of the column. The length of bearing is,

$$N = t_{bf} = 0.525\text{ in.}$$

The column web yielding strength is,

$$R_n = (2.5k + N)F_{yw}t_w \quad \text{(Specification J10-3)}$$

$$\begin{aligned} \phi R_n &= 1.00(2.5k + N)F_{yw}t_w \\ &= 1.00[2.5(0.820\text{ in.}) + 0.525\text{ in.}](50\text{ ksi})(0.300\text{ in.}) \\ &= 38.6\text{ kips} \end{aligned}$$

The column web crippling strength with $N/d < 0.2$ and the load applied at less than $d/2$ from the member end is,

$$R_n = 0.40t_w^2\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \quad \text{(Specification J10-5a)}$$

$$\begin{aligned} R_n &= 0.40(0.300)^2\left[1 + 3\left(\frac{0.525}{12.5}\right)\left(\frac{0.300}{0.520}\right)^{1.5}\right]\sqrt{\frac{29{,}000(50)(0.520)}{0.300}} \\ &= 60.2\text{ kips} \end{aligned}$$

The column local flange bending strength with the load applied at less than $10t_f$ from the member end is,

$$R_n = \tfrac{1}{2}\left(6.25t_f^2F_{yf}\right) \quad \text{(Specification J10-1)}$$

$$\begin{aligned} \phi R_n &= \tfrac{1}{2}\left[0.90(6.25)(0.520\text{ in.})^2(50\text{ ksi})\right] \\ &= 38.1\text{ kips} \end{aligned}$$

The actual force transmitted by the continuity plates is the load resulting from the beam flange force minus the least design strength based on limit states evaluated above. Therefore, for each continuity plate,

$$R_u = \tfrac{1}{2}(53.3\text{ kips} - 38.1\text{ kips}) = 7.60\text{ kips}$$

Therefore, the required strength of the connection of the continuity plates to the column web is 7.60 kips. The minimum required single-sided fillet weld size is,

$$\begin{aligned} D_{min} &= \frac{R_u}{1.392\text{ kips/in.}(l_w)} \\ &= \frac{7.60\text{ kips}}{1.392\text{ kips/in.}(6.96\text{ in.})} \\ &= 0.784 \end{aligned}$$

Checking Specification Table J2.4, with the column web thickness = 0.300 in., the minimum fillet-weld size is 3/16 in.

Use single-sided, 3/16-in. fillet welds to connect the continuity plates to the column web.

Design beam flange-to-column flange connection

Per Seismic Provisions Section 11.2a(2), if weld-access holes are provided, they must comply with Seismic Provisions Figure 11–1.

Use a complete-joint-penetration groove weld to connect the beam flanges to the column flange. The weld-access-hole geometry must comply with Seismic Provisions Figure 11–1.

It is important to note that Seismic Provisions Sections 11.2a(1) and (2) have additional requirements for treatment of weld backing and surface roughness of weld access holes.

Design beam web-to-column flange connection

Select a single-plate connection to support erection loads. With the single plate as backing, use a CJP groove weld to connect the beam web to the column flange. Note that other connection details that are capable of providing for 0.01 radian interstory drift angle are permitted as an alternative.

Check beam web strength

Assume a reduced web depth of 12 in. for shear to account for the required weld access holes.

$$\begin{aligned}\phi R_n &= \phi 0.6 F_y d t_w \\ &= 1.00(0.6)(50\text{ ksi})(12\text{ in.})(0.315\text{ in.}) \\ &= 113\text{ kips} > 17.9\text{ kips}\end{aligned}$$

$$R_u < \phi R_n \quad \textbf{o.k.}$$

The final connection design and geometry is shown in Figure 4–4.

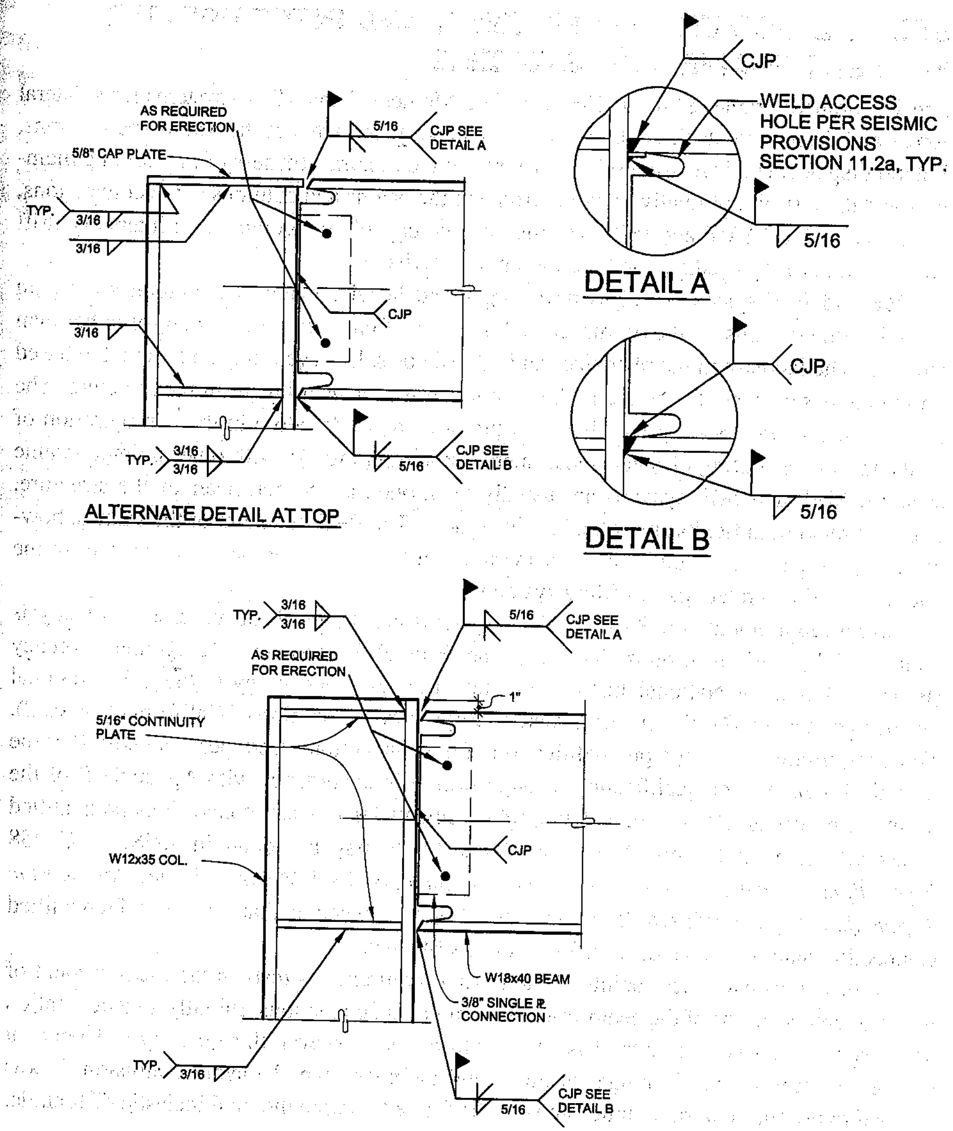

Figure 4–4. Connection as designed in Example 4.4.