Solution:

Calculate the transverse stiffener forces and web doubler plate shear force:
From Equation 4.2-1, the required strength for the transverse stiffeners is

$$R_{u\,st} = P_{uf} - \phi R_{n\,\min} = 224 \text{ kips} - 142 \text{ kips}$$
$$= 82 \text{ kips}$$

From Equation 4.2-2, the required strength for the two web doubler plates is

$$V_{u\,dp} = V_u - \phi R_{v\,cw} = 355 \text{ kips} - 167 \text{ kips}$$
$$= 188 \text{ kips}$$

Design the web doubler plates and their associated welding:
For strength, from Equation 4.4-1, the total thickness of web doubler plates required is

$$t_p \geq \frac{V_{udp}}{0.9 \times 0.6 F_y d_c}$$
$$\geq \frac{188 \text{ kips}}{0.9 \times 0.6(36 \text{ ksi})(14.02 \text{ in.})}$$
$$\geq 0.690 \text{ in. (or 0.345 in. per plate)}$$

Check minimum thickness required to prevent shear buckling of the web doubler plate. From Equation 4.4-5,

$$t_{p\,\min} = \frac{h\sqrt{F_y}}{418} = \frac{[14.02 \text{ in.} - 2(0.710 \text{ in.})]\sqrt{36 \text{ ksi}}}{418}$$
$$= 0.181 \text{ in.}$$

The thickness required for strength governs.

The web doubler plate width and depth are selected based upon the dimensions of the panel-zone and the edge details. Transverse to the axis of the column, the web doubler plate dimension is selected equal to the T-dimension of the column, plus twice the permissible encroachment from LRFD Manual Table 9-1 (page 9-12), which is 11¼ in. + 2(¼ in.) = 11¾ in. Parallel to the axis of the column, the web doubler plate dimension is selected equal to the beam depth plus two times the flange-plate thickness minus two times the transverse stiffener thickness minus two times the root opening for the CJP groove weld that will be used to connect the web doubler plate along the top and bottom edges. Assuming ½-in. transverse stiffener thickness and a ⅜-in. root opening for the CJP groove weld, 17.99 in. + 2(¾-in.) − 2(½-in.) − 2(⅜-in.) = 17¾ in., nominally.

Use 2 PL ⅜ in. × 11¾ in. × 1'-5¾.

The column-flange edges are to be CJP groove welded. **Use ⅜-in. CJP groove welds to connect the web doubler plates to the column flanges.**

The top and bottom edges of the web doubler plates are welded to the column web and transverse stiffeners with CJP groove welds. **Use ⅜-in. CJP groove welds to connect the top and bottom edges of the web doubler plate to the column web.**

Design the transverse stiffeners and their associated welding:
From Equation 4.3-1, the minimum required cross-sectional area for the transverse stiffeners at each flange is

$$A_{st\,\min} = \frac{R_{ust}}{\phi F_{yst}} = \frac{82 \text{ kips}}{0.9(36 \text{ ksi})} = 2.53 \text{ in.}^2$$

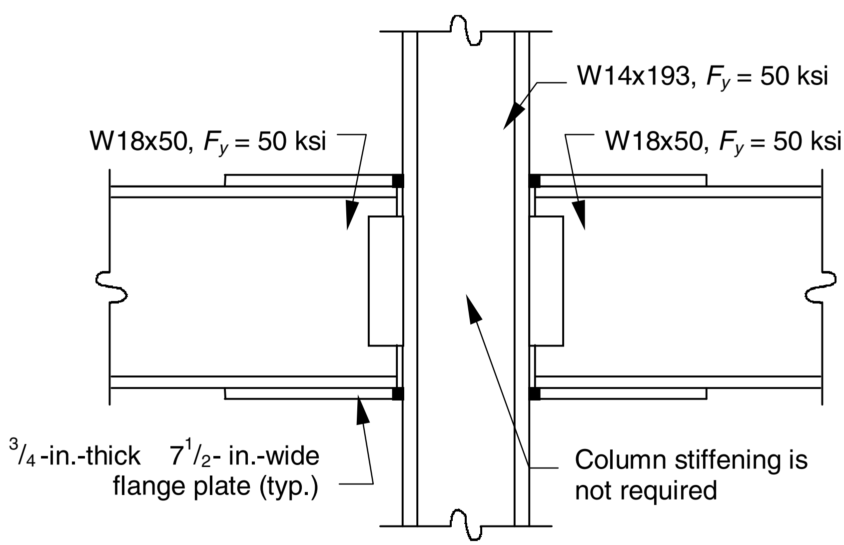

Figure 6-6 Framing arrangement for Example 6-5.

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From Equation 4.3-2, the minimum width of each transverse stiffener is

$$b_{s\,\min} = \frac{b}{3} - \frac{t_{pz}}{2} = \frac{7\tfrac{1}{2}\text{ in.}}{3} - \frac{0.440\text{ in.}}{2} = 2.28\text{ in.}$$

Try a pair of 3½-in.-wide transverse stiffeners at each beam flange with ¾-in. × ¾-in. corner clips. From Equation 4.3-3, the minimum thickness is

$$\begin{aligned} t_{s\,\min} &= \frac{t}{2} \geq \frac{b_s\sqrt{F_{yst}}}{95} \\ &= \frac{\tfrac{3}{4}\text{ in.}}{2} \geq \frac{(3\tfrac{1}{2}\text{ in.})\sqrt{36\text{ ksi}}}{95} \\ &= 0.375\text{ in.} \geq 0.221\text{ in.} \end{aligned}$$

Try a ½-in. transverse stiffener thickness.

$$\begin{aligned} A_{st} &= 2(\tfrac{1}{2}\text{ in.})(3\tfrac{1}{2}\text{ in.} - \tfrac{3}{4}\text{ in.}) \\ &= 2.75\text{ in.}^2 > \mathrm{A}_{st\,\min} = 2.53\text{ in.}^2 \quad \textbf{o.k.} \end{aligned}$$

The length of the transverse stiffeners is selected equal to the depth of the column minus two times the column flange thickness, which is 14.02 in. − 2(0.710 in.) = 12⅝ in.

Check the shear strength of the transverse stiffener to transmit the unbalance force in the transverse stiffener to the column panel-zone. Neglecting the effects of story shear, the worst-case unbalanced force in the transverse stiffener is that due to the combined effects of the two 250 ft-kip moment due to lateral load (in reverse curvature), the 100 ft-kip moment due to total gravity load on one side (adding) and the 45 ft-kip moment due to dead load only on the other side (subtracting). The unbalanced force in the transverse stiffener is

$$\begin{aligned} (R_{ust})_1 + (R_{ust})_2 &= (P_{uf} - \phi R_{n\,\min})_1 + (P_{uf} - \phi R_{n\,\min})_2 \\ &= (224\text{ kips} - 142\text{ kips}) \\ &\quad + (131\text{ kips} - 168\text{ kips}) \\ &= 82\text{ kips} + 0\text{ kips} \\ &= 82\text{ kips} \end{aligned}$$

From Equation 4.3-5,

$$\begin{aligned} t_s &\geq \frac{(R_{ust})_1 + (R_{ust})_2}{0.9 \times 0.6F_{yst}(l - 2 \times clip) \times 2)} \\ &\geq \frac{82\text{ kips}}{0.9 \times 0.6(36\text{ ksi})(12.6\text{ in.} - 2 \times \tfrac{3}{4}\text{ in.}) \times 2} \\ &\geq 0.190\text{ in.} \end{aligned}$$

Therefore, a ½-in. transverse stiffener thickness is **o.k.**

Use 2 PL ½-in. × 3½ in. × 1'-0⁹⁄₁₆ with two ¾-in. × ¾-in. corner clips each at each flange plate.

The double-sided fillet welds connecting the transverse stiffeners to the column flanges are sized to develop the strength of the welded portion of the transverse stiffener. From Equation 4.3-6, the weld size required for strength is

$$\begin{aligned} w_{\min} &= \frac{0.943F_{yst}t_s}{F_{EXX}} = \frac{0.943(36\text{ ksi})(\tfrac{1}{2}\text{ in.})}{70\text{ ksi}} \\ &= 0.242\text{ in.} \sim \tfrac{1}{4}\text{ in.} \end{aligned}$$

From LRFD Specification Table J2.4, with ½-in.-thick transverse stiffeners and 0.710-in.-thick column flanges, the minimum weld size is ¼ in. **Use ¼-in. double-sided fillet welds to connect the transverse stiffeners to the column flange.**

The transverse stiffeners are to be connected to the column panel zone with a detail that combines two fillet welds and two CJP groove weld as illustrated in Figure 4-12a. From Equation 4.3-10, the fillet weld size required for strength is

$$\begin{aligned} w &\geq \frac{(R_{ust})_1 + (R_{ust})_2}{0.75 \times 0.6F_{EXX}(l - 2 \times clip) \times 2 \times \sqrt{2}} \\ &\geq \frac{82\text{ kips}}{0.75 \times 0.6(70\text{ ksi})(12.6\text{ in.} - 2 \times \tfrac{3}{4}\text{ in.}) \times 2 \times \sqrt{2}} \\ &\geq 0.0829\text{ in.} \end{aligned}$$

From LRFD Specification Table J2.4, the minimum weld size for the ½-in.-thick transverse stiffener, ⅜-in-thick web doubler plate and 0.440-in.-thick column web is ³⁄₁₆ in. **Use ³⁄₁₆-in. fillet welds.**

Each ⅜-in. CJP groove weld must transmit one-quarter of the 82-kip unbalanced force in the transverse stiffeners (20.5 kips). From LRFD Specification Table J2.5, the shear strength is the lesser of that on the effective area in the transverse stiffener base metal and that in the weld itself. For the transverse stiffener base metal,

$$\begin{aligned} \phi R_n &= 0.9 \times 0.6F_{yst}w'(l - 2 \times clip) \\ &= 0.9 \times 0.6(36\text{ ksi})(\tfrac{3}{8}\text{ in.})(12.6\text{ in.} - 2 \times \tfrac{3}{4}\text{ in.}) \\ &= 80.9\text{ kips} > 20.5\text{ kips} \quad \textbf{o.k.} \end{aligned}$$

For the weld metal,

$$\begin{aligned} \phi R_n &= 0.8 \times 0.6F_{EXX}w'(l - 2 \times clip) \\ &= 0.8 \times 0.6(70\text{ ksi})(\tfrac{3}{8}\text{ in.})(12.6\text{ in.} - 2 \times \tfrac{3}{4}\text{ in.}) \\ &= 140\text{ kips} > 20.5\text{ kips} \quad \textbf{o.k.} \end{aligned}$$

The column web and web doubler plate thicknesses must also be checked for shear strength to transmit the unbalanced force in the transverse stiffeners to the panel-zone. For this detail, one-half of the unbalanced force (41 kips, the shear transmitted by the fillet welds) can be assigned to the column web with one-quarter (20.5 kips, the shear transmitted by each CJP groove weld) assigned to

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each web doubler plate. For the column web, the design shear strength is

$$\begin{aligned}\phi R_n &= 0.9 \times 0.6F_y d_c t_w \\ &= 0.9 \times 0.6(50 \text{ ksi})(14.02 \text{ in.})(0.440 \text{ in.}) \\ &= 167 \text{ kips} > 41 \text{ kips} \quad \textbf{o.k.}\end{aligned}$$

For the web doubler plate, the design shear strength is

$$\begin{aligned}\phi R_n &= 0.9 \times 0.6F_{ydp} d_c t_{pl} \\ &= 0.9 \times 0.6(36 \text{ ksi})(14.02 \text{ in.})(3/8 \text{ in.}) \\ &= 102 \text{ kips} > 20.5 \text{ kips} \quad \textbf{o.k.}\end{aligned}$$

Therefore, the column web and web doubler plates are of adequate thickness to provide for proper force transfer of the unbalance force in the transverse stiffener to the panel-zone. If either the column web or the web doubler plate thickness were inadequate in the above calculations, shear transfer between these elements on the effective area of the CJP groove weld root area can be utilized as a load path. Note, however, that if force is to be transferred from the column web to the web doubler plate in this manner, the maximum force transfer may be limited by the design shear strength on the effective area at the juncture between the CJP groove weld and the web doubler plate.

Summary:

The use of a W14×90 column requires the use of a pair of web doubler plates and a pair of transverse stiffeners at the location of each beam flange plate. The web doubler plates required are 2 PL 3/8 in. × 11 3/4 in. × 1'-5 3/4. They are welded to the column flanges along the column-flange edges and to the column web and transverse stiffeners along the top and bottom edges with 3/8-in. CJP groove welds. The transverse stiffeners required are 4 PL 1/2-in. × 3 1/2 in. × 1'-0 9/16 with two 3/4-in. × 3/4-in. corner clip each. Each transverse stiffener is welded to the column flange with 1/4-in. double-sided fillet welds and to the column web and web doubler plates with a combination of a 3/16-in. single-sided fillet weld and 3/8-in. CJP groove weld. This column-stiffening configuration is illustrated in Figure 6-7.

Example 6-7

Given:

Repeat Example 6-1 using a four-bolt extended end-plate moment connection as illustrated in Figure 6-8 instead of a directly welded flange moment connection. For the end-plate thickness, use 3/4 in. For the beam-flange-to-end-plate welds, use 1/2-in. fillet welds on both sides of the beam flange.

Use the following end-plate parameters in the calculations (see Section 2.2.2):

$$\begin{aligned}p_f &= 1\tfrac{1}{2} \text{ in.} \\ g &= 5\tfrac{1}{2} \text{ in.} \\ d_b &= 1 \text{ in.}\end{aligned}$$

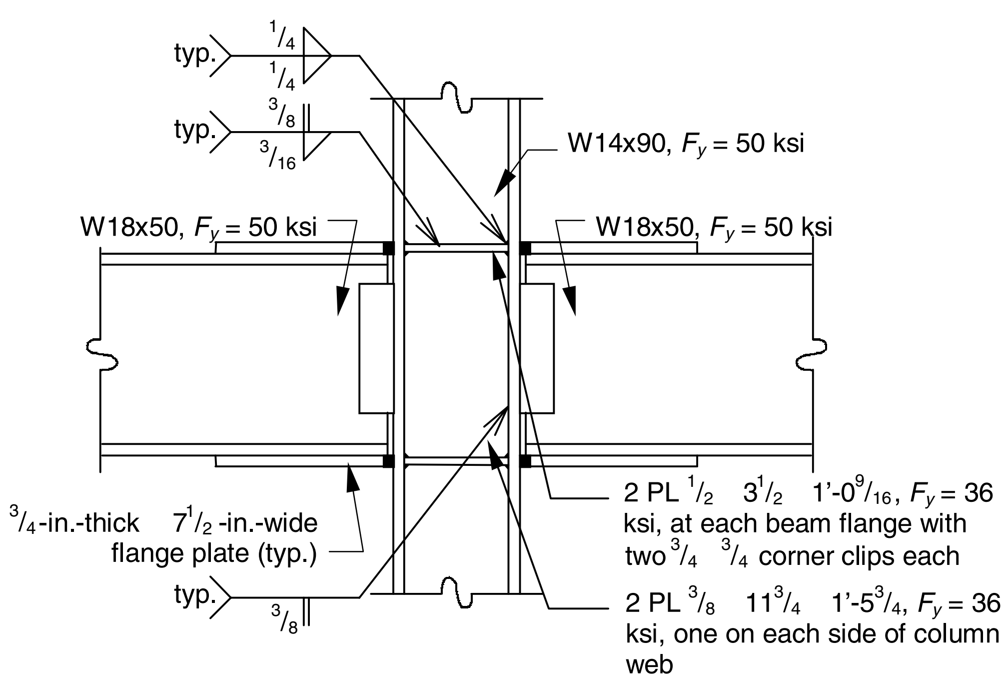

Figure 6-7 Framing arrangement for Example 6-6.

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Solution:

Calculate the flange forces and panel-zone shear force:
From Example 6-1,

$$P_{uf} = 172 \text{ kips}$$
$$V_u = 172 \text{ kips}$$

Determine the design panel-zone web shear strength:
From Example 6-1,

$$\phi R_v = 139 \text{ kips} < V_u = 172 \text{ kips} \quad \textbf{n.g.}$$

Therefore, the web of the W14×53 is inadequate to resist the panel-zone web shear without reinforcement.

Determine the design strength of the flange and web to resist the flange forces in tension:
For a tensile flange force, the limit states of local flange bending and local web yielding must be checked. For local flange bending, from Equation 2.2-9,

$$\begin{aligned} b_s &= 2.5(2p_f + t_{fb}) \\ &= 2.5(2 \times 1\tfrac{1}{2} \text{ in.} + 0.570 \text{ in.}) \\ &= 8.93 \text{ in.} \end{aligned}$$

$$\begin{aligned} p_e &= \frac{g}{2} - \frac{d_b}{4} - k_1 \\ &= \frac{5\tfrac{1}{2} \text{ in.}}{2} - \frac{1 \text{ in.}}{4} - \tfrac{15}{16} \text{ in.} = 1.56 \text{ in.} \end{aligned}$$

$$\alpha_m = 1.36\left(\frac{p_e}{d_b}\right)^{1/4} = 1.36\left(\frac{1.56 \text{ in.}}{1 \text{ in.}}\right)^{1/4} = 1.52$$

$$\begin{aligned} \phi R_n &= 0.9 \times \left(\frac{b_s}{\alpha_m p_e}\right) t_f^2 F_y \times C_t \\ &= 0.9 \times \left(\frac{8.93 \text{ in.}}{(1.52)(1.56 \text{ in.})}\right)(0.660 \text{ in.})^2(36 \text{ ksi}) \times 1 \\ &= 53.2 \text{ kips} < P_{uf} = 172 \text{ kips} \quad \textbf{n.g.} \end{aligned}$$

Note that F_y has been conservatively taken as 36 ksi as recommended in Section 2.2.2. For local web yielding, from Equation 2.2-11,

$$\begin{aligned} \phi R_n &= 1.0 \times [C_t(6k + 2t_p) + N]F_y t_w \\ &= 1.0 \times [(1)(6 \times 1\tfrac{7}{16} \text{ in.} + 2 \times \tfrac{3}{4} \text{ in.}) \\ &\quad + 0.570 \text{ in.}](50 \text{ ksi})(0.370 \text{ in.}) \\ &= 198 \text{ kips} > P_{uf} = 172 \text{ kips} \quad \textbf{o.k.} \end{aligned}$$

Therefore, while the web thickness is adequate, the flange of the W14×53 is inadequate to resist the tensile flange force without reinforcement.

Determine the design strength of the web to resist the flange forces in compression:
For a compressive flange force, the limit states of local web yielding, web crippling, and compression buckling of the web must be checked. In this case, the compression buckling limit state does not apply because there is a moment connection to one flange only. For local web yielding, as determined previously,

$$\phi R_n = 198 \text{ kips} > P_{uf} = 172 \text{ kips} \quad \textbf{o.k.}$$

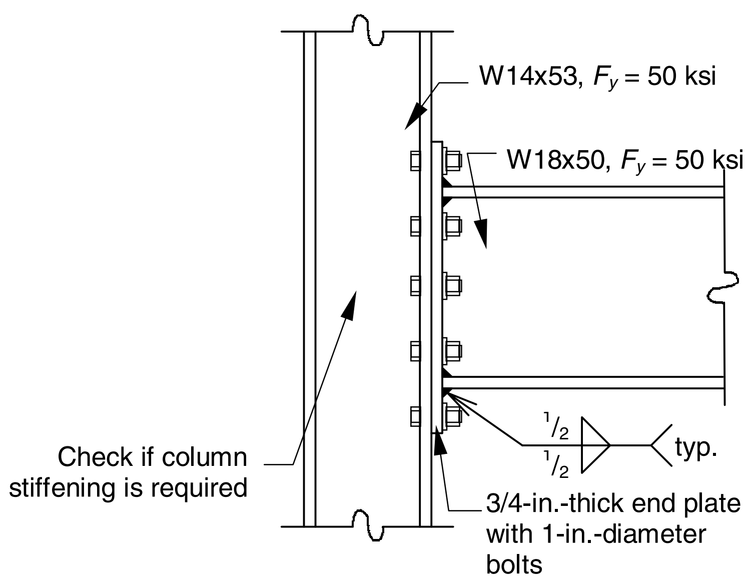

Figure 6-8 Framing arrangement for Example 6-7.

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For web crippling, from Equation 2.2-12,

$$N = 2w + 2t_p = 2(\tfrac{1}{2}\text{ in.}) + 2(\tfrac{3}{4}\text{ in.}) = 2.50\text{ in.}$$

$$N_d = \frac{3N}{d_c} = \frac{3(2.50\text{ in.})}{13.92\text{ in.}} = 0.539$$

$$\phi R_n = 0.75 \times 135 C_t t_w^2 \left[1 + N_d \left(\frac{t_w}{t_f}\right)^{1.5}\right] \times \sqrt{\frac{F_y t_f}{t_w}}$$

$$= 0.75 \times 135(1)(0.370\text{ in.})^2 \times \left[1 + (0.539)\left(\frac{0.370\text{ in.}}{0.660\text{ in.}}\right)^{1.5}\right]\sqrt{\frac{(50\text{ ksi})(0.660\text{ in.})}{0.370\text{ in.}}}$$

$$= 161\text{ kips} < P_{uf} = 172\text{ kips} \quad \textbf{n.g.}$$

Therefore, the web of the W14×53 is inadequate to resist the compressive flange force without reinforcement.

Summary:

The W14×53 is inadequate to resist the local forces that are induced without column stiffening. For the selection of a column that is adequate without stiffening, refer to Example 6-8. Although the design of stiffening for the W14×53 is not illustrated with an example problem for this case, it can be accomplished in a manner that is similar to that illustrated in Example 6-3.

Example 6-8

Given:

For the framing arrangement given in Example 6-7, reselect a column size that will eliminate the need for stiffening.

Solution:

As determined in Example 6-7, the flange thickness must be increased to increase the local flange bending strength and the web thickness must be increased to increase the web crippling strength and the panel-zone web shear strength. The required flange thickness is determined using a rearranged form of Equation 2.2-9 as

$$t_{f\ \text{req}} = \sqrt{\frac{P_{uf} p_e \alpha_m}{\phi F_y b_s C_t}} = \sqrt{\frac{(172\text{ kips})(1.56\text{ in.})(1.52)}{0.9(36\text{ ksi})(8.93\text{ in.})(1.0)}}$$

$$= 1.19\text{ in.}$$

Note that F_y has been conservatively taken as 36 ksi as recommended in Section 2.2.2. A W14×159 has a flange thickness equal to 1.19 in.

Check the web thickness of the W14×159 for web crippling. From Equation 2.2-12,

$$Nd = \frac{3N}{d} = \frac{3(0.570\text{ in.} + 2 \times \tfrac{1}{2}\text{ in.} + 2 \times \tfrac{3}{4}\text{ in.})}{14.98\text{ in.}}$$

$$= 0.615$$

$$\phi R_n = 0.75 \times 135 C_t t_w^2 \left[1 + N_d \left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{F_y t_f}{t_w}}$$

$$= 0.75 \times 135(1)(0.745\text{ in.})^2 \times \left[1 + (0.615)\left(\frac{0.745\text{ in.}}{1.19\text{ in.}}\right)^{1.5}\right]\sqrt{\frac{(50\text{ ksi})(1.19\text{ in.})}{0.745\text{ in.}}}$$

$$= 655\text{ kips} > P_{uf} = 172\text{ kips} \quad \textbf{o.k.}$$

Check the web thickness of the W14×159 for panel-zone web shear. Assuming the behavior of the panel-zone remains nominally within the elastic range,

$$P_y = F_y A = (50\text{ ksi})(46.7\text{ in.}^2) = 2{,}340\text{ kips}$$

$$\frac{P_u}{P_y} = \frac{300\text{ kips}}{2{,}340\text{ kips}} = 0.128$$

Since this ratio is less than 0.4, Equation 2.2-1 is applicable.

$$\phi R_v = 0.9 \times 0.6 F_y d_c t_w$$

$$= 0.9 \times 0.6(50\text{ ksi})(14.98\text{ in.})(0.745\text{ in.})$$

$$= 301\text{ kips} > V_u = 172\text{ kips} \quad \textbf{o.k.}$$

Summary:

As illustrated in Figure 6-9, a W14×159 column (F_y = 50 ksi) can be used without stiffening. This column-weight increase of 106 lb/ft (= 159 − 53) is within the range identified as economical in Chapter 3 for the elimination of two pairs of partial-depth transverse stiffeners and a web doubler plate.

Example 6-9

Given:

Repeat Example 6-1, except with a column that ends 2 in. above the top of the beam as illustrated in Figure 6-10.

Solution:

Calculate the flange forces and panel-zone shear force: From Example 6-1,

$$P_{uf} = 172\text{ kips}$$

$$V_u = 172\text{ kips}$$

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Determine which column-end criteria apply and if they apply at the near flange only or at both flanges of the beam:
The column-end criteria apply for local flange bending within $10t_f = 6.60$ in.; for local web yielding, within $d_c = 13.92$ in.; and for web crippling and compression bucking of the web within $d_c/2 = 6.96$ in. Thus, for a W18×50 beam, with $d = 17.99$ in., the column-end criteria apply for all limit states at the near (top) flange only.

Determine the design panel-zone web shear strength:
From Example 6-1,

$$\phi R_v = 139 \text{ kips} < V_u = 172 \text{ kips} \quad \textbf{n.g.}$$

Therefore, the web of the W14×53 is inadequate to resist the panel-zone web shear without reinforcement.

Determine the design strength of the flange and web to resist the flange forces in tension:
For a tensile flange force, the limit states of local flange bending and local web yielding must be checked. At the bottom flange force, from Example 6-1, for local flange bending,

$$\phi R_n = 123 \text{ kips} < P_{uf} = 172 \text{ kips} \quad \textbf{n.g.}$$

and for local web yielding,

$$\phi R_n = 144 \text{ kips} < P_{uf} = 172 \text{ kips} \quad \textbf{n.g.}$$

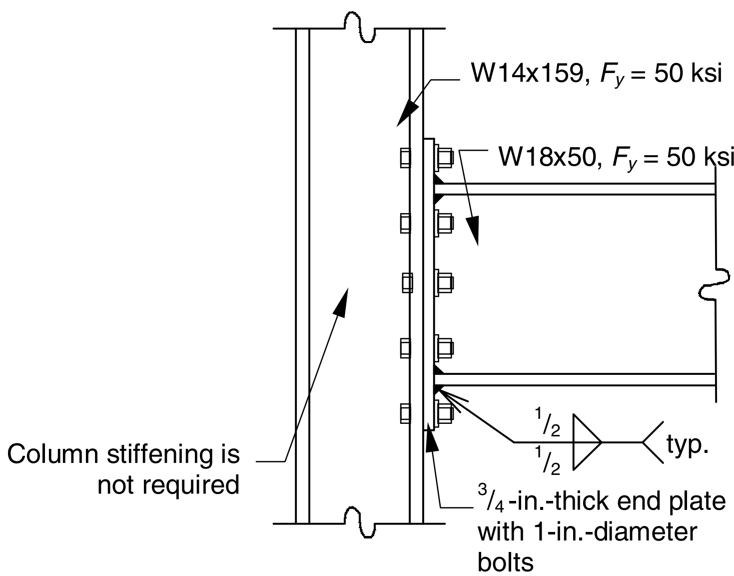

Figure 6-9 Framing arrangement for Example 6-8.

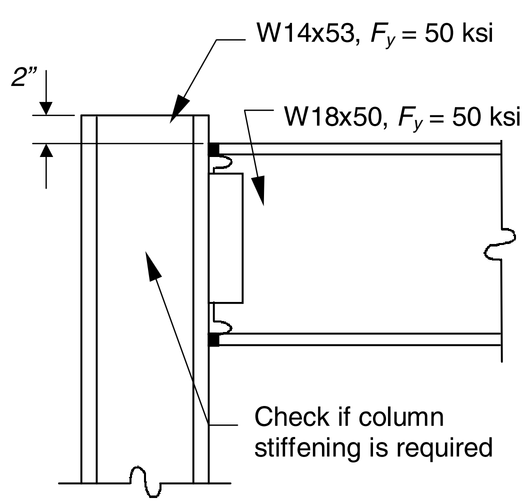

Figure 6-10 Framing arrangement for Example 6-9.

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At the top flange force, for local flange bending, from Equation 2.2-8,

$$\begin{aligned}\phi R_n &= 0.9 \times 6.25 t_f^2 F_y \times C_t \\ &= 0.9 \times 6.25(0.660\text{ in.})^2(50\text{ ksi}) \times 0.5 \\ &= 61.3\text{ kips} < P_{uf} = 172\text{ kips} \quad \textbf{n.g.}\end{aligned}$$

and for local web yielding, from Equation 2.2-10,

$$\begin{aligned}\phi R_n &= 1.0 \times [C_t(5k) + N]F_y t_w \\ &= 1.0 \times [0.5(5)(1\tfrac{7}{16}\text{ in.}) \\ &\qquad + 0.570\text{ in.}](50\text{ ksi})(0.370\text{ in.}) \\ &= 77.0\text{ kips} < P_{uf} = 172\text{ kips} \quad \textbf{n.g.}\end{aligned}$$

Therefore, the flange and web of the W14×53 are inadequate to resist the tensile flange force without reinforcement at both the top and bottom flanges.

Determine the design strength of the web to resist the flange forces in compression:
For a compressive flange force, the limit states of local web yielding, web crippling, and compression buckling of the web must be checked. In this case, the compression buckling limit state does not apply because there is a moment connection to one flange only. At the bottom flange force, as determined previously, for local web yielding,

$$\phi R_n = 144\text{ kips} < P_{uf} = 172\text{ kips} \quad \textbf{n.g.}$$

and for web crippling,

$$\phi R_n = 138\text{ kips} < P_{uf} = 172\text{ kips} \quad \textbf{n.g.}$$

At the top flange force, for local web yielding, as determined previously,

$$\phi R_n = 77.0\text{ kips} < P_{uf} = 172\text{ kips} \quad \textbf{n.g.}$$

and for web crippling, from Equation 2.2-12,

$$N_d = \frac{3N}{d_c} = \frac{3(0.570\text{ in.})}{13.92\text{ in.}} = 0.123$$

$$\phi R_n = 0.75 \times 135 C_t t_w^2 \left[1 + N_d\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{F_y t_f}{t_w}}$$

$$\begin{aligned}&= 0.75 \times 135(0.5)(0.370\text{ in.})^2 \\ &\quad \times \left[1 + (0.123)\left(\frac{0.370\text{ in.}}{0.660\text{ in.}}\right)^{1.5}\right]\sqrt{\frac{(50\text{ ksi})(0.660\text{ in.})}{0.370\text{ in.}}} \\ &= 68.8\text{ kips} < P_{uf} = 172\text{ kips} \quad \textbf{n.g.}\end{aligned}$$

Therefore, the web of the W14×53 is inadequate to resist the compressive flange force without reinforcement at both the top and bottom flanges.

Summary:

The W14×53 is inadequate to resist the local forces that are induced without column stiffening. For the selection of a column that is adequate without stiffening, refer to Example 6-10.

Comments:

The foregoing solution can be determined more expediently using the design aids in Appendices A, B, and C. The design panel-zone web shear strength is determined from Table A-1 where, for a W14×53 with $P_u/P_y \leq 0.4$,

$$\phi R_v = 139\text{ kips} < V_u = 172\text{ kips} \quad \textbf{n.g.}$$

The design strength of the flange and web to resist the flange force in tension is determined from Tables B-1 and C-1 where, for a W14×53, with $N = 0.570$ in. and reading from the **T** column,

$$\begin{aligned}\phi R_n &= 123\text{ kips at the bottom flange (Table B-1)} \\ &\qquad < P_{uf} = 172\text{ kips} \quad \textbf{n.g.} \\ \phi R_n &= 61.3\text{ kips at the top flange (Table C-1)} \\ &\qquad < P_{uf} = 172\text{ kips} \quad \textbf{n.g.}\end{aligned}$$

by interpolation between the values that are tabulated for $N = \tfrac{1}{2}$ in. and $N = \tfrac{3}{4}$ in. The design strength of the web to resist the flange force in compression is also determined from Tables B-1 and C-1 where, for a W14×53, with $N = 0.570$ in. and reading from the C column,

$$\begin{aligned}\phi R_n &= 138\text{ kips at the bottom flange (Table B-1)} \\ &\qquad < P_{uf} = 172\text{ kips} \quad \textbf{n.g.} \\ \phi R_n &= 69.3\text{ kips at the top flange}^{21}\text{ (Table C-1)} \\ &\qquad < P_{uf} = 172\text{ kips} \quad \textbf{n.g.}\end{aligned}$$

by interpolation between the values that are tabulated for $N = \tfrac{1}{2}$ in. and $N = \tfrac{3}{4}$ in.

Example 6-10

Given:

For the framing arrangement given in Example 6-9, reselect the column size to eliminate the need for stiffening:

A) entirely.
B) except the transverse stiffeners at the top flange force (near the column end).

[21] The slight discrepancy between the calculated value (68.8 kips) and the value determined by linear interpolation (69.3 kips) results because the equations used to generate the tabulated values are not linear.

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Solution A:

Try a W14×159 with $F_y = 50$ ksi:

$$P_y = F_y A = (50 \text{ ksi})(46.7 \text{ in.}^2) = 2{,}340 \text{ kips}$$

$$= \frac{300 \text{ kips}}{2{,}340 \text{ kips}} = 0.128$$

From Table A-1, with $P_u/P_y \leq 0.4$,

$$\phi R_v = 301 \text{ kips} = V_u = 172 \text{ kips} \quad \textbf{o.k.}$$

At the bottom flange force (away from the column end), from Table B-1, with $N = 0.570$ in.,

$$\phi R_n = 371 \text{ kips}(\mathbf{T}) > P_{uf} = 172 \text{ kips} \quad \textbf{o.k.}$$

$$= 371 \text{ kips}(\mathbf{C}) > P_{uf} = 172 \text{ kips} \quad \textbf{o.k.}$$

by interpolation between the values that are tabulated for $N = 1/2$ in. and $N = 3/4$ in. At the top flange force (near the column end), from Table C-1, with $N = 0.570$ in.,

$$\phi R_n = 194 \text{ kips}(\mathbf{T}) > P_{uf} = 172 \text{ kips} \quad \textbf{o.k.}$$

$$= 195 \text{ kips}(\mathbf{C}) > P_{uf} = 172 \text{ kips} \quad \textbf{o.k.}$$

by interpolation between the values that are tabulated for $N = 1/2$ in. and $N = 3/4$ in.

Solution B:

From Example 6-2, a W14×74 can be used without a web doubler plate and without transverse stiffeners at the bottom flange force. At the top flange force (near the column end), either a pair of partial-depth transverse stiffeners can be provided or a detail such as that illustrated in Figure 6-12 can be used.

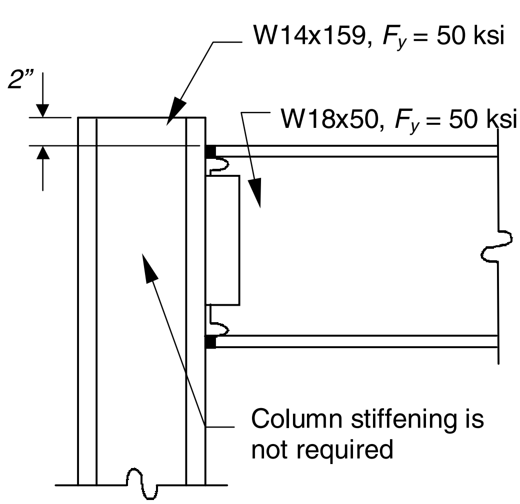

Figure 6-11 Framing arrangement for Example 6-10 (Solution A).

Summary A:

As illustrated in Figure 6-11 W14×159 column ($F_y = 50$ ksi) can be used without stiffening. This column-weight increase of 106 lb/ft (= 159 − 53) is within the range identified as economical in Chapter 3 for the elimination of two pairs of partial-depth transverse stiffeners and a web doubler plate.

Summary B:

A W14×74 column ($F_y = 50$ ksi) can be used without stiffening, except the transverse stiffeners at the top flange force (near the column end). This column-weight increase of 21 lb/ft (= 74 − 53) is well within the range identified as economical in Chapter 3 for the elimination of one pair of partial-depth transverse stiffeners and a web doubler plate.

Example 6-11

Given:

For a pair of ½-in.-thick full-depth transverse stiffeners ($F_y = 36$ ksi) that transmit an unbalanced force of 82 kips to a 0.440-in.-thick column web ($F_y = 50$ ksi) with a single ⅜-in.-thick web doubler plate ($F_y = 36$ ksi), proportion the welds and check shear in the column web and web doubler plate. The transverse stiffeners are 1'-0$\frac{9}{16}$-in. long and have two ¾-in. × ¾-in. corner clips each. They are used with a W14×90 column. Use a joint detail as illustrated in:

A) Figure 4-11a.
B) Figure 4-11b.
C) Figure 4-11c.
D) Figure 4-11d.

Solution A:

The transverse stiffeners are to be connected to the column panel zone with a detail that combines three fillet welds and one CJP groove weld as illustrated in Figure 4-11a. From Equation 4.3-10, the fillet weld size required for strength is

$$w \geq \frac{(R_{u\,st})_1 + (R_{u\,st})_2}{0.75 \times 0.6F_{EXX}(l - 2 \times clip) \times 2 \times \sqrt{2}}$$

$$\geq \frac{82 \text{ kips}}{0.75 \times 0.6(70 \text{ ksi})(12.6 \text{ in.} - 2 \times 3/4 \text{ in.}) \times 2 \times \sqrt{2}}$$

$$\geq 0.0829 \text{ in.}$$

From LRFD Specification Table J2.4, the minimum weld size for the ½-in.-thick transverse stiffener, ⅜-in-thick web doubler plate, and 0.440-in.-thick column web is $\frac{3}{16}$ in. **Use $\frac{3}{16}$-in. fillet welds.**

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The $3/8$-in. CJP groove weld must transmit one-quarter of the 82-kip unbalanced force in the transverse stiffeners (20.5 kips). From LRFD Specification Table J2.5, the shear strength is the lesser of that on the effective area in the transverse stiffener base metal and that in the weld itself. For the transverse stiffener base metal,

$$\begin{aligned}\phi R_n &= 0.9 \times 0.6 F_{yst} w'(l - 2 \times clip) \\ &= 0.9 \times 0.6(36 \text{ ksi})(3/8 \text{ in.})(12.6 \text{ in.} - 2 \times 3/4 \text{ in.}) \\ &= 80.9 \text{ kips} > 20.5 \text{ kips} \quad \textbf{o.k.}\end{aligned}$$

For the weld metal,

$$\begin{aligned}\phi R_n &= 0.8 \times 0.6 F_{EXX} w'(l - 2 \times clip) \\ &= 0.8 \times 0.6(70 \text{ ksi})(3/8 \text{ in.})(12.6 \text{ in.} - 2 \times 3/4 \text{ in.}) \\ &= 140 \text{ kips} > 20.5 \text{ kips} \quad \textbf{o.k.}\end{aligned}$$

The column web and web doubler plate thicknesses must also be checked for shear strength to transmit the unbalanced force in the transverse stiffeners to the panel-zone. For this detail, three-quarters of the unbalanced force (61.5 kips, the shear transmitted by the fillet welds) can be assigned to the column web with the remaining one-quarter (20.5 kips, the shear transmitted by the CJP groove weld) assigned to the web doubler plate. For the column web, the design shear strength is

$$\begin{aligned}\phi R_n &= 0.9 \times 0.6 F_y d_c t_w \\ &= 0.9 \times 0.6(50 \text{ ksi})(14.02 \text{ in.})(0.440 \text{ in.}) \\ &= 167 \text{ kips} > 61.5 \text{ kips} \quad \textbf{o.k.}\end{aligned}$$

For the web doubler plate, the design shear strength is

$$\begin{aligned}\phi R_n &= 0.9 \times 0.6 F_{ydp} d_c t_{pl} \\ &= 0.9 \times 0.6(36 \text{ ksi})(14.02 \text{ in.})(3/8 \text{ in.}) \\ &= 102 \text{ kips} > 20.5 \text{ kips} \quad \textbf{o.k.}\end{aligned}$$

Therefore, the column web and web doubler plate are of adequate thickness to provide for proper force transfer of the unbalance force in the transverse stiffeners to the panel-zone. If either the column web or the web doubler plate thickness were inadequate in the above calculations, shear transfer between these elements on the effective area of the CJP groove weld root area can be utilized as a load path. Note, however, that if force is to be transferred from the column web to the web doubler plate in this manner, the maximum force transfer may be limited by the design shear strength on the effective area at the juncture between the CJP groove weld and the web doubler plate.

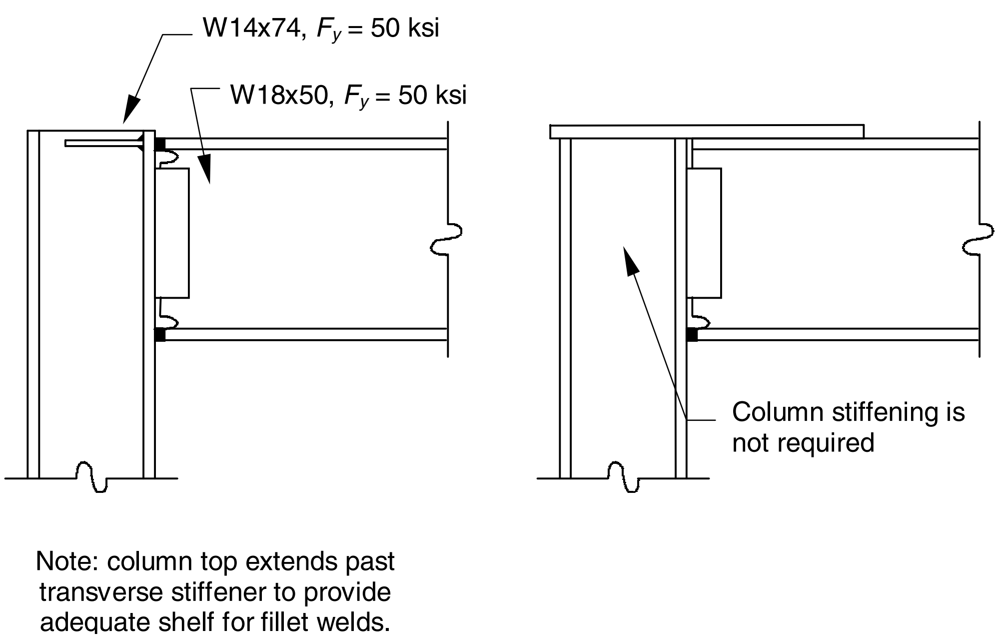

Figure 6-12 Framing arrangement for Example 6-10 (Solution B).

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Solution B:

The solution for this example and the joint detail illustrated in Figure 4-11b is identical to Solution A.

Solution C:

The transverse stiffeners are to be connected to the column panel zone with a detail that combines three fillet welds and one CJP groove weld as illustrated in Figure 4-11c. For the fillet welds on the side of the web without a web doubler plate, from Equation 4.3-10, the fillet weld size required for strength is

$$w \geq \frac{(R_{u\,st})_1 + (R_{u\,st})_2}{0.75 \times 0.6F_{EXX}(l - 2 \times clip) \times 2 \times \sqrt{2}}$$

$$\geq \frac{82 \text{ kips}}{0.75 \times 0.6(70 \text{ ksi})(12.6 \text{ in.} - 2 \times 3/4 \text{ in.}) \times 2 \times \sqrt{2}}$$

$$\geq 0.0829 \text{ in.}$$

From LRFD Specification Table J2.4, the minimum weld size for the ½-in.-thick transverse stiffener, ⅜-in-thick web doubler plate, and 0.440-in.-thick column web is 3/16 in. **Use 3/16-in. fillet welds.**

The ½-in. CJP groove weld must transmit one-half of the 82-kip unbalanced force in the transverse stiffeners (41 kips). From LRFD Specification Table J2.5, the shear strength is the lesser of that on the effective area in the transverse stiffener base metal and that in the weld itself. For the transverse stiffener base metal,

$$\phi R_n = 0.9 \times 0.6F_{yst}w'(l - 2 \times clip)$$
$$= 0.9 \times 0.6(36 \text{ ksi})(1/2 \text{ in.})(12.6 \text{ in.} - 2 \times 3/4 \text{ in.})$$
$$= 108 \text{ kips} > 41 \text{ kips} \quad \textbf{o.k.}$$

For the weld metal,

$$\phi R_n = 0.8 \times 0.6F_{EXX}w'(l - 2 \times clip)$$
$$= 0.8 \times 0.6(70 \text{ ksi})(1/2 \text{ in.})(12.6 \text{ in.} - 2 \times 3/4 \text{ in.})$$
$$= 186 \text{ kips} > 41 \text{ kips} \quad \textbf{o.k.}$$

For this detail, either the entire unbalanced force can be transmitted to the column web (through the two fillet welds on the side of the column web without a web doubler plate and the CJP groove weld) or the fillet weld between the web doubler plate and transverse stiffener can be sized to transmit a portion of this force to the web doubler plate.[22] In the former case, the fillet weld between the web doubler plate and the transverse stiffener is selected as a minimum-size fillet weld per LRFD Specification Table J2.4 (3/16-in.). For the column web, the design shear strength is

$$\phi R_n = 0.9 \times 0.6F_y d_c t_w$$
$$= 0.9 \times 0.6(50 \text{ ksi})(14.02 \text{ in.})(0.440 \text{ in.})$$
$$= 167 \text{ kips} > 82 \text{ kips} \quad \textbf{o.k.}$$

Therefore, the column web is adequate to transfer the entire unbalanced load to the panel zone without additional strength from the web doubler plate. The fillet weld between the web doubler plate and the transverse stiffener is selected as minimum size per LRFD Specification Section J2.4. **Use a 3/16-in. fillet weld.**

Solution D:

The transverse stiffeners are to be connected to the column panel zone with a detail that combines three fillet welds to the column web and one fillet weld to the web doubler plate as illustrated in Figure 4-11d. From Equation 4.3-10, the fillet weld size required for strength is

$$w \geq \frac{(R_{u\,st})_1 + (R_{u\,st})_2}{0.75 \times 0.6F_{EXX}(l - 2 \times clip) \times 2 \times \sqrt{2}}$$

$$\geq \frac{82 \text{ kips}}{0.75 \times 0.6(70 \text{ ksi})(12.6 \text{ in.} - 2 \times 3/4 \text{ in.}) \times 2 \times \sqrt{2}}$$

$$\geq 0.0829 \text{ in.}$$

From LRFD Specification Table J2.4, the minimum weld size for the ½-in.-thick transverse stiffener, ⅜-in-thick web doubler plate, and 0.440-in.-thick column web is 3/16 in. **Use 3/16-in. fillet welds.**

The column web and web doubler plate thicknesses must also be checked for shear strength to transmit the unbalanced force in the transverse stiffeners to the panel-zone. For this detail, three-quarters of the unbalanced force (61.5 kips, the shear transmitted by the fillet welds) can be assigned to the column web with the remaining one-quarter (20.5 kips, the shear transmitted by the CJP groove weld) assigned to the web doubler plate. For the column web, the design shear strength is

$$\phi R_n = 0.9 \times 0.6F_y d_c t_w$$
$$= 0.9 \times 0.6(50 \text{ ksi})(14.02 \text{ in.})(0.440 \text{ in.})$$
$$= 167 \text{ kips} > 61.5 \text{ kips} \quad \textbf{o.k.}$$

For the web doubler plate, the design shear strength is

$$\phi R_n = 0.9 \times 0.6F_{ydp} d_c t_{pl}$$
$$= 0.9 \times 0.6(36 \text{ ksi})(14.02 \text{ in.})(3/8 \text{ in.})$$
$$= 102 \text{ kips} > 20.5 \text{ kips} \quad \textbf{o.k.}$$

[22] As in Solution A, the shear strength of the effective area at the root of the CJP groove weld can be used for force transfer to the web doubler plate, if necessary.

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