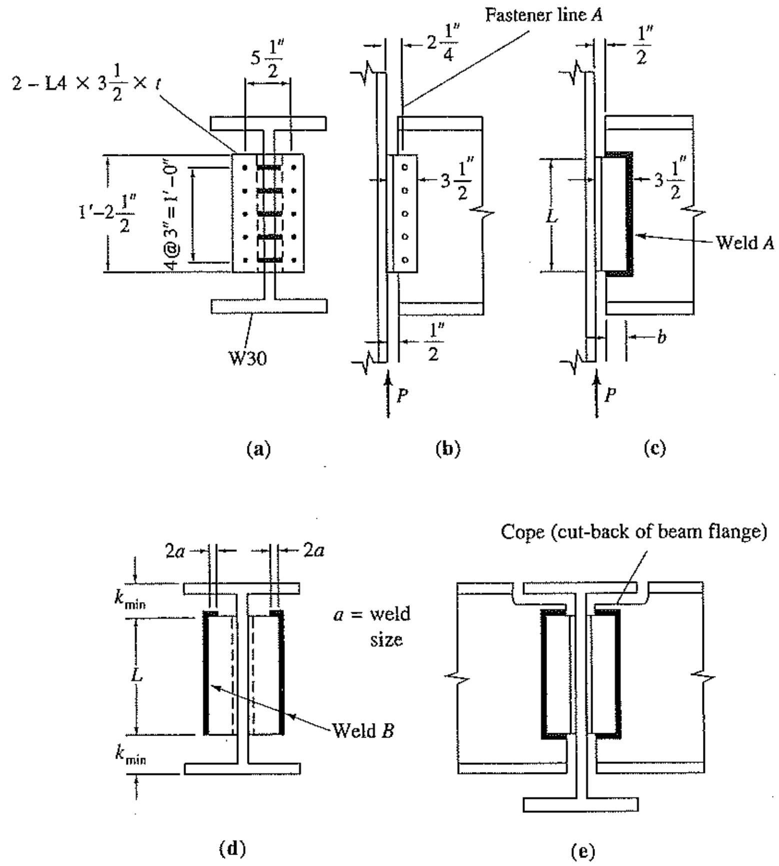

Figure 13.2.1 Simple shear double-angle connections.

Fig. 13.2.1b, while field fastener holes are shown as solid black dots. There have been many studies [13.19–13.28] of simple shear double-angle connections.

In today's fabrication practice, the shop connection is usually welded, while the field connection may be either bolted or welded; thus, any combination in Fig. 13.2.1 of (a) with (b) or (c); or (d) with (b) or (c) may be used.

The *single-plate framing connection* is a modification where a single plate (instead of the pair of angles) is bolted flat against the beam web and then is welded perpendicular to the beam web or column flange or web to which it is attached. The design of single plate framing connections has been studied by Richard, Gillett, Kriegh, and Lewis [13.29], Young and Disque [13.30]. Richard, Kriegh, and Hormby [13.31], Hormby, Richard, and Kriegh [13.32], and Astaneh, Call, and McMullin [13.33].

Another type of simple shear connection is the *tee framing connection* as studied by Astaneh and Nader [13.35, 13.36], where the tee flange attaches to the supporting column (or beam) and the tee web laps against the loaded beam to transmit its shear.

Another single-plate framing connection, studied by Kennedy [13.34], uses the plate in the vertical position welded flat against the end of the beam with the connection to the beam or column made with bolts.

For the angles connected to the W24,

$$0.75(0.6)(65)[18.0 - 6(\tfrac{13}{16} + \tfrac{1}{16})]2t \geq 210$$

$$t \geq \frac{210}{0.75(0.6)65[18.0 - 5.25]2} = 0.28 \text{ in.}$$

Occasionally, the angle strength might be controlled by gross shear yield (the angles under LRFD-J5.3; in this case, the thickness required is 0.22 in., le than the 0.28 in. required above. The bearing value ϕR_n for $\frac{5}{16}$-in. angles 27.4 kips/bolt, which is more than adequate to carry the factored load of 17 kips/bolt from the beams (a coincidence that the same factored load per bolt contributed by both the W10 and the W24); therefore, $\frac{5}{16}$-in. angles are satisfa tory as connecting angles.

Use 2—L4×3½×$\frac{5}{16}$×0′–6″ for W10×68.

Use 2—L4×3½×$\frac{5}{16}$×1′–6″ for W24×104.

The length of angle should not exceed the dimension T, which is $7\frac{5}{8}$ in. f the W10×68. The girder flange thickness is such that a cope is required on t beams that encroaches on the T dimension. ■■

Weld Capacity in Eccentric Shear on Angle Connections

Since no initial tension is involved with welded connections, the eccentricity loading, even though small, is considered. The principles of Chapter 5 (See. 5.18) a used with the welds treated as lines.

EXAMPLE 13.2.4

Compute the factored load P_u capacity for weld A on the angle connection shown Fig. 13.2.1. The beam is a W30×99 and the weld is $\frac{1}{4}$ in. with E70 electrodes. T angles are 4×3½×$\frac{5}{16}$×1′-2½″ in length. Use A36 steel and Load and Resistance Fact Design.

Solution Analysis of this eccentric shear situation may be done using streng analysis as presented in Sec. 5.17 or the elastic (vector) method presented Sec. 5.18.

(a) Elastic (vector) method. Using I_p from Table 5.18.1 and referring Fig. 13.2.1c,

$$I_p = \frac{8(3)^3 + 6(3)(14.5)^2 + (14.5)^3}{12} - \frac{(3)^4}{2(3) + 14.5} = 583.5 \text{ in.}^3$$

Using the moment of inertia computed with a 1-in. effective throat, the force p unit length at critical locations can be computed.

$$R_v = \frac{P_u}{2(20.5)} = 0.0244P_u \quad \text{(direct shear component)} \downarrow$$

$$\bar{x} = \frac{(3)^2}{2(3) + 14.5} = 0.44 \text{ in.}$$

The x and y components of force due to torsional moment are

$$R_y = \frac{P_u(3.50 - 0.44)(3.50 - 0.44 - 0.50)}{2(583.5)} = 0.00671P_u \downarrow$$

$$R_x = \frac{P_u(3.50 - 0.44)(7.25)}{2(583.5)} = 0.0190P_u \rightarrow$$

$$R_u = P_u\sqrt{(0.0244 + 0.0067)^2 + (0.0190)^2} = 0.0364P_u$$

The design strength ϕR_{nw} per inch of weld is

$$\begin{aligned}\phi R_{nw} &= \phi(0.707a)(0.60F_{\text{EXX}})\\ &= 0.75(0.707)(\tfrac{1}{4})(0.60)70 = 5.57 \text{ kips/in.}\end{aligned}$$

Check base metal shear fracture for the beam web and the angles,

$$\begin{aligned}(\phi R_{nw})_{\text{base metal}} &= 0.75(0.60F_u)t = 0.45F_u t\\ (\phi R_{nw})_{\text{angle}} &= 0.45(58)0.3125 = 8.16 \text{ kips/in.}\\ (\phi R_{nw})_{\text{web}} &= 0.45(58)(0.520/2) = 6.79 \text{ kips/in.}\end{aligned}$$

Weld strength controls; $\phi R_{nw} = 5.57$ kips/in.

$$P_u = \frac{5.57}{0.0364} = 153 \text{ kips}$$

(b) Strength analysis. Use *LRFD Manual* [1.18], Table 8–42, "Coefficients C for Eccentrically Loaded Weld Groups" with $\theta = 0°$. For $\frac{1}{4}$-in. weld using E70 electrodes,

$$\begin{aligned}a &= (e - xL)/L = (3.5 - 0.44)/14.5 = 0.211\\ k &= kL/L \quad = 3.0/14.5 = 0.207\end{aligned}$$

	$k = 0.2$	0.207	0.3	
$a = 0.2$	1.98		2.33	
0.211	1.958	1.982	2.306	$C = 1.982$
0.25	1.88		2.22	

Table value $= \phi P_n$

$$\phi P_n = CC_1DL = 1.982(1.0)(4)14.5 = 115 \text{ kips}$$

where C_1 = coefficient for electrode = (Electrode used)/70
D = number of $\frac{1}{16}$s of an inch in weld size
L = length of vertical weld, in.

Since there are two angles, the factored load reaction capacity is

$$P_u = 2(115) = 230 \text{ kips}$$

As expected, the strength analysis gives the higher value. ■■

Tests of welded angle connections by Johnston and Green [13.19] and Johnston and Diets [13.38] have demonstrated that performance of web angles agrees generally with assumptions.

Weld Capacity in Tension and Shear on Angle Connections

This is the field-welded connection shown in Fig. 13.2.1d. There is no agreeme regarding the strength analysis for this situation. Blodgett [13.2] considers t strength as an eccentric shear situation in the plane of the welds. With the eccent load as in Fig. 13.2.5b, the angles bear against themselves for a distance of $L/6$ frc the top, and the torsional stress over the remaining $\frac{5}{6}$ of the length L is resisted by t weld. Neglecting the effects of the returns at the top, the horizontal component R_x c be obtained from moment equilibrium. Equilibrium in the plane of the load P a weld leg B requires

$$\underbrace{\frac{1}{2}R_x\left(\frac{5}{6}L\right)}_{\text{force}}\underbrace{\frac{2}{3}L}_{\text{arm}} = \frac{P}{2}e_2 \tag{13.2.}$$

$$R_x = \frac{9Pe_2}{5L^2}\text{ force/unit length} \tag{13.2.}$$

The direct shear component is

$$R_v = \frac{P}{2L}\text{ force/unit length} \tag{13.2.}$$

$$\text{Actual } R = \sqrt{\left(\frac{P}{2L}\right)^2 + \left(\frac{9}{5}\frac{Pe_2}{L^2}\right)^2}$$

$$R = \frac{P}{2L^2}\sqrt{L^2 + 12.9e_2^2}\text{ force/unit length} \tag{13.2.}$$

Equation 13.2.8 neglects eccentricity e_1, which tends to cause tension at the t of the weld lines. The authors believe it is more appropriate to consider the flexu stress distribution of Fig. 13.2.5c to be a more appropriate approach. The flexu tension component R_x at the top of the weld B is

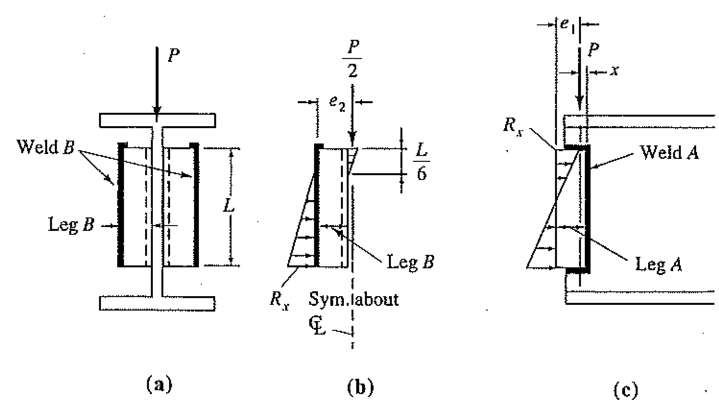

Figure 13.2.5 Field-welded connection for web framing angles.

$$R_x = \frac{Mc}{I} = \frac{Pe_1(L/2)}{2L^3/12} = \frac{3Pe_1}{L^2} \tag{13.2.9}$$

when the returns at the tops of the angles are neglected. The direct shear component R_v is

$$R_v = \frac{P}{2L} \text{ force/unit length} \tag{13.2.10}$$

$$\text{Actual } R = \sqrt{\left(\frac{P}{2L}\right)^2 + \left(\frac{3Pe_1}{L^2}\right)^2}$$

$$R = \frac{P}{2L^2}\sqrt{L^2 + 36e_1^2} \text{ force/unit length} \tag{13.2.11}$$

Or, if returns are considered (distance b of Fig. 13.2.6) the expression becomes complicated. The *LRFD Manual* p. 9-89 indicate the returns to be twice the weld size. The returns have the greatest effect when the angle length L is short. It may be reasonable to consider the returns to be $L/12$ (2 times $\frac{1}{4}$ in. weld for $L = 6$ in.).

Using, from Table 5.18.1 (Case 4), $S = I/\bar{y}$ referred to the tension fiber at the top of the configuration,

$$S = 2\left(\frac{4bd + d^2}{6}\right) \tag{13.2.12}$$

which for $d = L$ and $b = L/12$ becomes

$$S = \frac{4L^2}{9} \tag{13.2.13}$$

The flexural component, as shown in Fig. 13.2.6, is

$$R_x = \frac{M}{S} = \frac{Pe_1}{S} = \frac{Pe_1}{4L^2/9} = \frac{9Pe_1}{4L^2} \tag{13.2.14}$$

Since little of the shear is carried by the returns, they are neglected for the direct shear component, giving

$$R_v = \frac{P}{2L} \tag{13.2.15}$$

$$\text{Actual } R = \sqrt{\left(\frac{P}{2L}\right)^2 + \left(\frac{9Pe_1}{4L^2}\right)^2}$$

$$R = \frac{P}{2L^2}\sqrt{L^2 + 20.25e_1^2} \text{ kips/in.} \tag{13.2.16}$$

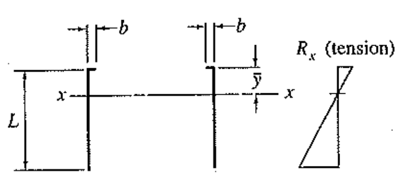

Figure 13.2.6 Weld configuration for web angles and beam seats.

EXAMPLE 13.2.5

Determine the factored load capacity P_u of weld B on Fig. 13.2.5 if $\frac{5}{16}$-in. weld is used and $L = 20$ in. E70 electrodes are used in shielded metal arc welding (SMAW). $4\times3\times\frac{3}{8}$ angles are used. Assume base material is thick enough to preclude shear fracture as the controlling limit state; i.e., the fillet weld strength controls. Use Load and Resistance Factor Design.

Solution

(a) Best procedure, Eq. 13.2.16

$$\phi R_{nw} = 0.75(0.707)(\tfrac{5}{16})(0.60)70 = 6.96 \text{ kips/in.}$$

$$\text{Actual } R_u = \frac{P_u}{2L^2}\sqrt{L^2 + 20.25e_1^2}$$

$$e_1 = 3.00 - \bar{x} = 3.00 - 0.25 = 2.75 \text{ in.}$$

$$\bar{x} = \frac{2(2.5)(1.25)}{2(2.5) + 20} = 0.25 \text{ in.}$$

$$\text{Actual } R_u = \frac{P_u}{2(20)^2}\sqrt{(20)^2 + 20.25(2.75)^2} = 0.0294P_u$$

$$P_u = \frac{6.96}{0.0294} = 237 \text{ kips}$$

(b) Neglecting returns entirely, Eq. 13.2.11,

$$\text{Actual } R_u = \frac{P_u}{2(20)^2}\sqrt{(20)^2 + 36(2.75)^2} = 0.0324P_u$$

$$P_u = \frac{6.96}{0.0324} = 215 \text{ kips}$$

(c) Using Ref. 13.2 equation, Eq. 13.2.8,

$$\text{Actual } R_u = \frac{P_u}{2(20)^2}\sqrt{(20)^2 + 12.9e_2^2}$$

$$e_2 = 4\text{-in. leg}$$

$$\text{Actual } R_u = 0.0308P_u$$

$$P_u = \frac{6.96}{0.0308} = 226 \text{ kips}$$

The authors believe method (a) to be appropriate, $P_u = 237$ kips. ■■

Note that when the elastic (vector) method is used, the same formulas apply for Allowable Stress Design and for Load and Resistance Factor Design. In ASD, P is the service load connection capacity, R is the service load resultant force per unit length at the most highly stressed weld segment, and the allowable resistance is R_w. In LRFD P_u and R_u represent factored loads, and the design strength ϕR_{nw} is used.