3.3-5

Gross section:

$$\phi_t P_n = 0.9 F_y A_g = 0.9(36)(5.86) = 190 \text{ kips}$$

Net section:

$$\text{Hole diameter} = 1 + \tfrac{1}{8} = 1\tfrac{1}{8} \text{ in.}$$

$$A_n = A_g - A_{hole} = 5.86 - 2(1.125)(5/8) = 4.454 \text{ in.}^2$$

$$U = 1 - \frac{\bar{x}}{L} = 1 - \frac{1.03}{9} = 0.8856$$

$$A_e = UA_n = 0.8856(4.454) = 3.944 \text{ in.}^2$$

$$\phi_t P_n = 0.75 F_u A_e = 0.75(58)(3.944) = 172 \text{ kips}$$

Factored load:

Combination (A4-2): $1.2D + 1.6L = 1.2(50) + 1.6(100) = 220$ kips

Combination (A4-4): $1.2D + 1.3W + 0.5L = 1.2(50) + 1.3(45) + 0.5(100) = 168$ kips

Combination A4-2 controls: 220 kips > 172 kips (N.G.) <u>Member is not adequate</u>

3.5-4

Gross section strength:

$$\phi_t P_n = 0.9 F_y A_g = 0.9(50)(3.60) = 162 \text{ kips}$$

Net section strength:

$$A_n = 3.60 - 2(7/8)(0.314) = 3.050 \text{ in.}^2$$
$$A_e = UA_n = 0.85(3.050) = 2.593 \text{ in.}^2$$
$$\phi_t P_n = 0.75 F_u A_e = 0.75(65)(2.593) = 126 \text{ kips}$$

Block shear strength of tension member:

Shear areas:

$$A_{gv} = 0.314(1.5 + 3 + 3) \times 2 = 4.710 \text{ in.}^2$$
$$A_{nv} = 0.314[1.5 + 3 + 3 - 2.5(7/8)] \times 2 = 3.336 \text{ in.}^2$$

(2.5 hole diameters in each line of shear)

Tension areas:

$$A_{gt} = 0.314(3) = 0.9420 \text{ in}^2$$
$$A_{nt} = 0.314[3 - (0.5 + 0.5)(7/8)] = 0.6672 \text{ in.}^2 \quad (0.5 + 0.5 \text{ hole diameters})$$

AISC Eq. J4-3a:

$$\begin{aligned}\phi R_n &= \phi[0.6 F_y A_{gv} + F_u A_{nt}] \\ &= 0.75[0.6(50)(4.710) + 65(0.6672)] = 0.75(141.3 + 43.37) = 139 \text{ kips}\end{aligned}$$

AISC Eq. J4-3b:

$$\begin{aligned}\phi R_n &= \phi[0.6 F_u A_{nv} + F_y A_{gt}] \\ &= 0.75[0.6(65)(3.336) + 50(0.9420)] = 0.75(130.1 + 47.1) = 133 \text{ kips}\end{aligned}$$

Eq. J4-3b has the larger fracture term, so it controls.

$\phi R_n = 133$ kips for tension member

Gusset plate:

Shear areas:

$$A_{gv} = \tfrac{3}{8}(1.5 + 3 + 3) \times 2 = 5.625 \text{ in.}^2$$
$$A_{nv} = \tfrac{3}{8}[1.5 + 3 + 3 - 2.5(7/8)] \times 2 = 3.984 \text{ in.}^2$$

(2.5 hole diameters in each line of shear)

Tension areas:

$$A_{gt} = \tfrac{3}{8}(3) = 1.125 \text{ in}^2$$
$$A_{nt} = \tfrac{3}{8}[3 - (0.5 + 0.5)(7/8)] = 0.7969 \text{ in.}^2 \quad (0.5 + 0.5 \text{ hole diameters})$$

AISC Eq. J4-3a:

$$\begin{aligned}\phi R_n &= \phi[0.6 F_y A_{gv} + F_u A_{nt}] \\ &= 0.75[0.6(36)(5.625) + 58(0.7969)] = 0.75(121.5 + 46.22) = 126 \text{ kips}\end{aligned}$$

AISC Eq. J4-3b:

$$\phi R_n = \phi[0.6F_u A_{nv} + F_y A_{gt}]$$

$$= 0.75[0.6(58)(3.984) + 36(1.125)] = 0.75(138.6 + 40.5) = 134 \text{ kips}$$

Eq. J4-3b has the larger fracture term, so it controls: $\phi R_n = 134$ kips for gusset plate

Member block shear strength is smaller and controls; block shear strength is

$$\phi R_n = 133 \text{ kips}$$

Net section strength controls overall:

$$\underline{P_u = \phi R_n = 126 \text{ kips}}$$

<u>3.6-1</u>

$$P_u = 1.2D + 1.6L = 1.2(28) + 1.6(84) = 168 \text{ kips}$$

$$\text{Required } A_g = \frac{P_u}{0.9F_y} = \frac{168}{0.9(36)} = 5.19 \text{ in.}^2$$

$$\text{Required } A_e = \frac{P_u}{0.75F_u} = \frac{168}{0.75(58)} = 3.86 \text{ in.}^2$$

$$\text{Required } r_{\min} = \frac{L}{300} = \frac{18(12)}{300} = 0.72 \text{ in.}$$

Try L7 × 4 × ½:

$$A_g = 5.25 \text{ in.}^2 > 5.19 \text{ in.}^2 \quad \text{(OK)} \qquad r_{\min} = r_z = 0.872 \text{ in.} > 0.72 \text{ in.} \quad \text{(OK)}$$

$$A_n = 5.25 - 1.125(0.5) = 4.668 \text{ in.}^2$$

Use average value of U from the Commentary:

$$A_e = UA_n = 0.85(4.688) = 3.98 \text{ in.}^2 > 3.86 \text{ in.}^2 \quad \text{(OK)}$$

$$\underline{\text{Use L7} \times 4 \times \tfrac{1}{2}}$$

3.6-3

$$\text{Required } A_g = \frac{P_u}{0.9F_y} = \frac{180}{0.9(50)} = 4.0 \text{ in.}^2$$

$$\text{Required } A_e = \frac{P_u}{0.75F_u} = \frac{180}{0.75(65)} = 3.69 \text{ in.}^2$$

$$\text{Required } r_{\min} = \frac{L}{300} = \frac{25(12)}{300} = 1.0 \text{ in.}$$

The angle leg must be at least 5 in. long to accommodate two lines of bolts (See usual gages for angles, Fig. 3.22).

Try $2L6 \times 3\frac{1}{2} \times \frac{5}{16}$:

$$A_g = 5.74 \text{ in.}^2 > 4.0 \text{ in.}^2 \quad \text{(OK)} \qquad r_{min} = r_y = 1.38 \text{ in.} > 1.0 \text{ in.} \quad \text{(OK)}$$

$$A_n = 5.74 - 4\left(\tfrac{7}{8} + \tfrac{1}{8}\right)(5/16) = 4.49 \text{ in.}^2$$

Use average value of U from the Commentary:

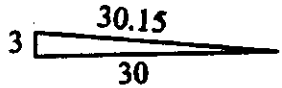
$$A_e = UA_n = 0.85(4.49) = 3.82 \text{ in.}^2 > 3.69 \text{ in.}^2 \quad \text{(OK)}$$

Use $2L6 \times 3\frac{1}{2} \times \frac{5}{16}$

3.8-1

Interior joint load:

Snow:	20(10)(12.5)	= 2500 lb
Roofing:	12(10)(30.15/30)(12.5)	= 1508 lb
Purlins:	8.5(12.5)	= 106.2 lb
Truss weight:	1000/3	= 333.3 lb



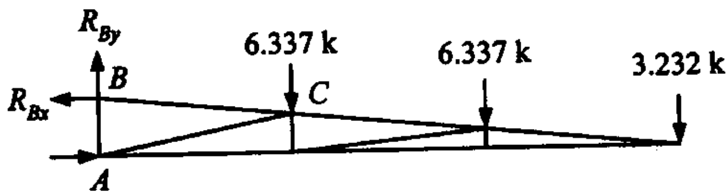
(The assumption that the truss weight is distributed equally to the joints is approximate but is consistent with the approximate nature of the estimate of total truss weight.)

Load combination A4-3 controls:

$$1.2D + 1.6S = 1.2(1.508 + 0.1062 + 0.3333) + 1.6(2.5) = 6.337 \text{ kips}$$

Exterior joint load. Use half of the above loads except for the purlin weight, which is the same:

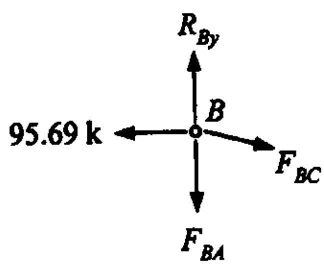
$$1.2D + 1.6S = 1.2\left(\frac{1.508}{2} + 0.1062 + \frac{0.3333}{2}\right) + 1.6\left(\frac{2.5}{2}\right) = 3.232 \text{ kips}$$



$$\sum M_A = 6.337(10) + 6.337(20) + 3.232(30) - R_{Bx}(3) = 0$$

$$R_{Bx} = 95.69 \text{ kips} \leftarrow$$

Joint B:



$$\sum F_x = -95.69 + \frac{30}{30.15} F_{BC} = 0, \quad F_{BC} = 96.17 \text{ kips}$$

$$\text{Required } A_g = \frac{F_{BC}}{0.9F_y} = \frac{96.17}{0.9(36)} = 2.97 \text{ in.}^2$$

$$\text{Required } A_e = \frac{F_{BC}}{0.75F_u} = \frac{96.17}{0.75(58)} = 2.21 \text{ in.}^2$$

$$L = 10\left(\frac{30.15}{30}\right) = 10.05 \text{ ft}$$

$$\text{Required } r_{min} = \frac{L}{300} = \frac{10.05(12)}{300} = 0.402 \text{ in.}$$

Try WT5 × 11:

$A_g = 3.24 \text{ in.}^2 > 2.97 \text{ in.}^2$ (OK) $\quad r_{min} = 1.33 \text{ in.} > 0.402 \text{ in.}$ (OK)

Use average value of U from the Commentary:

$A_e = UA_g = 0.85(3.24) = 2.75 \text{ in.}^2 > 2.21 \text{ in.}^2$ (OK)

Use WT5 × 11