# CHAPTER 3 - TENSION MEMBERS

**3.2-1**

For yielding of the gross section,

$$A_g = 7(3/8) = 2.625 \text{ in.}^2, \qquad P_n = F_y A_g = 36(2.625) = 94.5 \text{ kips}$$

For rupture of the net section,

$$A_e = (3/8)\left(7 - \left(1 + \frac{3}{16}\right)\right) = 2.180 \text{ in.}^2$$

$$P_n = F_u A_e = 58(2.108) = 122.3 \text{ kips}$$

a) The design strength based on yielding is

$$\phi_t P_n = 0.90(94.5) = 85.05 \text{ kips}$$

The design strength based on rupture is

$$\phi_t P_n = 0.75(122.3) = 91.73 \text{ kips}$$

The design strength for LRFD is the smaller value: $\underline{\phi_t P_n = 85.1 \text{ kips}}$

b) The allowable strength based on yielding is

$$\frac{P_n}{\Omega_t} = \frac{94.5}{1.67} = 56.59 \text{ kips}$$

The allowable strength based on rupture is

$$\frac{P_n}{\Omega_t} = \frac{122.3}{2.00} = 61.15 \text{ kips}$$

The allowable service load is the smaller value: $\underline{P_n/\Omega_t = 56.6 \text{ kips}}$

Alternate solution using allowable ***stress***: For yielding,

$$F_t = 0.6F_y = 0.6(36) = 21.6 \text{ ksi}$$

and the allowable load is $F_t A_g = 21.6(2.625) = 56.7$ kips

For rupture,

$$F_t = 0.5F_u = 0.5(58) = 29.0 \text{ ksi}$$

and the allowable load is $F_t A_e = 29.0(2.180) = 63.22$ kips

The allowable service load is the smaller value $= 56.7$ kips

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**3.2-2**

For yielding of the gross section,

$$A_g = 6(3/8) = 2.25 \text{ in.}^2$$

$$P_n = F_y A_g = 50(2.25) = 112.5 \text{ kips}$$

For rupture of the net section,

$$A_e = A_g = 2.25 \text{ in.}^2$$

$$P_n = F_u A_e = 65(2.25) = 146.3 \text{ kips}$$

a) The design strength based on yielding is

$$\phi_t P_n = 0.90(112.5) = 101 \text{ kips}$$

The design strength based on rupture is

$$\phi_t P_n = 0.75(146.3) = 110 \text{ kips}$$

The design strength for LRFD is the smaller value: $\underline{\phi_t P_n = 101 \text{ kips}}$

b) The allowable strength based on yielding is

$$\frac{P_n}{\Omega_t} = \frac{112.5}{1.67} = 67.4 \text{ kips}$$

The allowable strength based on rupture is

$$\frac{P_n}{\Omega_t} = \frac{146.3}{2.00} = 73.2 \text{ kips}$$

The allowable service load is the smaller value: $\underline{P_n/\Omega_t = 67.4 \text{ kips}}$

Alternate solution using allowable ***stress***: For yielding,

$$F_t = 0.6F_y = 0.6(50) = 30.0 \text{ ksi}$$

and the allowable load is

$$F_t A_g = 30.0(2.25) = 67.5 \text{ kips}$$

For rupture,

$$F_t = 0.5F_u = 0.5(65) = 32.5 \text{ ksi}$$

and the allowable load is

$$F_t A_e = 32.5(2.25) = 73.1 \text{ kips}$$

The allowable service load is the smaller value $= 67.5$

kips

**3.2-3**

For yielding of the gross section,

$$P_n = F_y A_g = 50(3.37) = 168.5 \text{ kips}$$

For rupture of the net section,

$$A_n = A_g - A_{holes} = 3.37 - 0.220\left(\frac{7}{8} + \frac{1}{8}\right) \times 2 \text{ holes} = 2.930 \text{ in.}^2$$

$$A_e = 0.85A_n = 0.85(2.930) = 2.491 \text{ in.}^2$$

$$P_n = F_e A_e = 65(2.491) = 161.9 \text{ kips}$$

a) The design strength based on yielding is

$$\phi_t P_n = 0.90(168.5) = 152 \text{ kips}$$

The design strength based on rupture is

$$\phi_t P_n = 0.75(161.9) = 121.4 \text{ kips}$$

The design strength is the smaller value: $\phi_t P_n = 121.4$ kips

Let $P_u = \phi_t P_n$

$$1.2D + 1.6(3D) = 121.4, \text{ Solution is: } \{D = 20.23\}$$

$$P = D + L = 20.23 + 3(20.23) = 80.9 \text{ kips}$$ $\underline{P = 80.9 \text{ kips}}$

b) The allowable strength based on yielding is

$$\frac{P_n}{\Omega_t} = \frac{168.5}{1.67} = 100.9 \text{ kips}$$

The allowable strength based on rupture is

$$\frac{P_n}{\Omega_t} = \frac{161.9}{2.00} = 80.95 \text{ kips}$$

The allowable load is the smaller value = 80.95 kips $\underline{P = 81.0 \text{ kips}}$

Alternate computation of allowable load using allowable ***stress***: For yielding,

$$F_t = 0.6F_y = 0.6(50) = 30.0 \text{ ksi}$$

and the allowable load is

$$F_t A_g = 30.0(3.37) = 101.1 \text{ kips}$$

For rupture,

$$F_t = 0.5F_u = 0.5(65) = 32.5 \text{ ksi}$$

and the allowable load is

$$F_t A_e = 32.5(2.491) = 80.96 \text{ kips}$$

**3.2-4**

For A242 steel and $t = ½$ in., $F_y = 50$ ksi and $F_u = 70$ ksi. For yielding of the gross section,

$$A_g = 8(1/2) = 4 \text{ in.}^2$$

$$P_n = F_y A_g = 50(4) = 200 \text{ kips}$$

For rupture of the net section,

$$A_n = A_g - A_{holes} = 4 - (1/2)\left(1 + \frac{3}{16}\right)(2) = 2.813 \text{ in.}^2$$

$$A_e = A_n = 2.813 \text{ in.}^2$$

$$P_n = F_u A_e = 70(2.813) = 196.9 \text{ kips}$$

a) The design strength based on yielding is

$$\phi_t P_n = 0.90(200) = 180 \text{ kips}$$

The design strength based on rupture is

$$\phi_t P_n = 0.75(196.9) = 147.7 \text{ kips}$$

The design strength for LRFD is the smaller value: $\underline{\phi_t P_n = 148 \text{ kips}}$

b) The allowable strength based on yielding is

$$\frac{P_n}{\Omega_t} = \frac{200}{1.67} = 120 \text{ kips}$$

The allowable strength based on rupture is

$$\frac{P_n}{\Omega_t} = \frac{196.9}{2.00} = 98.45 \text{ kips}$$

The allowable service load is the smaller value: $\underline{P_n/\Omega_t = 98.5 \text{ kips}}$

Alternate solution using allowable ***stress***: For yielding,

$$F_t = 0.6F_y = 0.6(50) = 30 \text{ ksi}$$

and the allowable load is

$$F_t A_g = 30(4) = 120 \text{ kips}$$

For rupture,

$$F_t = 0.5F_u = 0.5(70) = 35 \text{ ksi}$$

and the allowable load is

$$F_t A_e = 35(2.813) = 98.5 \text{ kips}$$

The allowable service load is the smaller value = $98.5$ kips

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<u>**3.2-5**</u>

For a thickness of $t = 3/8$ in., $F_y = 50$ ksi and $F_u = 70$ ksi. First, compute the nominal strengths. For the gross section,

$$A_g = 7.5(3/8) = 2.813 \text{ in.}^2$$

$$P_n = F_y A_g = 50(2.813) = 140.7 \text{ kips}$$

Net section:

$$A_n = 2.813 - \left(\frac{3}{8}\right)\left(1\frac{1}{8} + \frac{3}{16}\right)(2) = 1.829 \text{ in.}^2$$

$$A_e = A_n = 1.829 \text{ in.}^2$$

$$P_n = F_u A_e = 70(1.829) = 128.0 \text{ kips}$$

a) The design strength based on yielding is

$$\phi_t P_n = 0.90(140.7) = 127 \text{ kips}$$

The design strength based on rupture is

$$\phi_t P_n = 0.75(128.0) = 96.0 \text{ kips}$$

The design strength is the smaller value: $\phi_t P_n = 96.0$ kips

Factored load:

Combination 1: $1.4D = 1.4(25) = 35.0$ kips

Combination 2: $1.2D + 1.6L = 1.2(25) + 1.6(45) = 102$ kips

The second combination controls; $P_u = 102$ kips.

Since $P_u \; \phi >_t P_n$, (102 kips >96.0 kips), <u>The member is unsatisfactory.</u>

b) For the gross section, the allowable strength is

$$\frac{P_n}{\Omega_t} = \frac{140.7}{1.67} = 84.3 \text{ kips}$$

Alternately, the allowable stress is

$$F_t = 0.6F_y = 0.6(50) = 30 \text{ ksi}$$

and the allowable strength is $F_t A_g = 30(2.813) = 84.4$ kips

For the net section, the allowable strength is

$$\frac{P_n}{\Omega_t} = \frac{128.0}{2.00} = 64.0 \text{ kips}$$

Alternately, the allowable stress is

$F_t = 0.5F_u = 0.5(70) = 35$ ksi and the allowable strength is

$$F_t A_e = 35(1.829) = 64.02 \text{ kips}$$

The smaller value controls; the allowable strength is 64.0 kips. When the only loads are dead load and live load, ASD load combination 2 will always control:

$$P_a = D + L = 25 + 45 = 70 \text{ kips}$$

Since 70 kips > 64.0 kips, The member is unsatisfactory.

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**3.2-6**

Compute the strength for one angle, then double it. For the gross section,

$$P_n = F_y A_g = 36(1.20) = 43.2 \text{ kips}$$

For two angles, $P_n = 2(43.2) = 86.4$ kips

Net section:

$$A_n = 1.20 - \left(\frac{1}{4}\right)\left(\frac{3}{4} + \frac{1}{8}\right) = 0.9813 \text{ in.}^2$$

$$A_e = 0.85A_n = 0.85(0.9813) = 0.8341 \text{ in.}^2$$

$$P_n = F_u A_e = 58(0.8341) = 48.38 \text{ kips}$$

For two angles, $P_n = 2(48.38) = 96.76$ kips

a) The design strength based on yielding is

$$\phi_t P_n = 0.90(86.4) = 77.76 \text{ kips}$$

The design strength based on rupture is

$$\phi_t P_n = 0.75(96.76) = 72.57 \text{ kips}$$

The design strength is the smaller value: $\phi_t P_n = 72.6$ kips

$$P_u = 1.2D + 1.6L = 1.2(12) + 1.6(36) = 72.0 \text{ kips} < 72.6 \text{ kips} \qquad \text{(OK)}$$

The member has enough strength.

b) For the gross section, the allowable strength is

$$\frac{P_n}{\Omega_t} = \frac{86.4}{1.67} = 51.74 \text{ kips}$$

Alternately, the allowable stress is

$F_t = 0.6F_y = 0.6(36) = 21.6$ ksi

and the allowable strength is $F_tA_g = 21.6(2 \times 1.20) = 51.84$ kips

For the net section, the allowable strength is

$$\frac{P_n}{\Omega_t} = \frac{96.76}{2.00} = 48.38 \text{ kips}$$

Alternately, the allowable stress is

$F_t = 0.5F_u = 0.5(58) = 29$ ksi

and the allowable strength is $F_tA_e = 29(2 \times 0.8341) = 48.38$ kips

The net section strength controls; the allowable strength is 48.4 kips. When the only loads are dead load and live load, ASD load combination 2 will always control:

$P_a = D + L = 12 + 36 = 48 \text{ kips} < 48.4 \text{ kips}$ (OK)

The member has enough strength.

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**3.3-1**

(a) $U = 1 - \frac{\bar{x}}{\ell} = 1 - \frac{1.47}{5} = 0.7060$

$A_e = A_gU = 5.86(0.7060) = 4.14 \text{ in.}^2$ $\qquad$ $A_e = 4.14 \text{ in.}^2$

(b) Plate with longitudinal welds only:

$$U = \frac{3\ell^2}{3\ell^2 + w^2}\left(1 - \frac{\bar{x}}{\ell}\right) = \frac{3(5)^2}{3(5)^2 + (4)^2}\left(1 - \frac{(3/8)/2}{5}\right) = 0.7933$$

$A_e = A_gU = \left(\frac{3}{8} \times 4\right)(0.7933) = 1.19 \text{ in.}^2$

$A_e = 1.19 \text{ in.}^2$

(c) $U = 1.0$

$A_e = A_gU = \left(\frac{5}{8} \times 5\right)(1.0) = 3.13 \text{ in.}^2$ $\qquad$ $A_e = 3.13 \text{ in.}^2$

(d) $U = 1.0$

$A_g = 0.5(5.5) = 2.750 \text{ in.}^2$

$A_n = A_g - A_{holes} = 2.750 - \frac{1}{2}\left(\frac{3}{4} + \frac{1}{8}\right) = 2.313 \text{ in.}^2$

$A_e = A_nU = 2.313(1.0) = 2.313 \text{ in.}^2$ $\qquad$ $A_e = 2.31 \text{ in.}^2$

(e) $U = 1.0$

$$A_g = \frac{5}{8} \times 6 = 3.750 \text{ in.}^2$$

$$A_n = A_g - A_{holes} = 3.750 - \frac{5}{8}\left(\frac{7}{8} + \frac{1}{8}\right) = 3.125 \text{ in.}^2$$

$$A_e = A_n U = 3.125(1.0) = 3.125 \text{ in.}^2 \qquad \underline{A_e = 3.13 \text{ in.}^2}$$

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**3.3-2**

$$A_n = A_g - A_{holes} = 3.31 - \frac{7}{16}\left(\frac{7}{8} + \frac{1}{8}\right) = 2.873 \text{ in.}^2$$

$$U = 1 - \frac{\bar{x}}{\ell} = 1 - \frac{1.15}{3} = 0.6167$$

$$A_e = A_n U = 2.873(0.6167) = 1.772 \text{ in.}^2$$

$$P_n = F_u A_e = 70(1.772) = 124 \text{ kips} \qquad \underline{P_n = 124 \text{ kips}}$$

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**3.3-3**

$$U = 1 - \frac{\bar{x}}{\ell} = 1 - \frac{0.775}{8} = 0.9031$$

$$A_e = A_g U = 2.49(0.9031) = 2.249 \text{ in.}^2$$

$$P_n = F_u A_e = 58(2.249) = 130.4 \text{ kips} \qquad \underline{P_n = 130 \text{ kips}}$$

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**3.3-4**

For A588 steel, $F_y = 50$ ksi and $F_u = 70$ ksi

For yielding of the gross section,

$$P_n = F_y A_g = 50(4.79) = 239.5 \text{ kips}$$

For rupture of the net section,

$$A_n = A_g - A_{holes} = 4.79 - \frac{1}{2}\left(\frac{3}{4} + \frac{1}{8}\right) = 4.353 \text{ in.}^2$$

From AISC Table D3.1, Case 8, $U = 0.80$

$$A_e = A_n U = 4.353(0.80) = 3.482 \text{ in.}^2$$

$$P_n = F_u A_e = 70(3.482) = 243.7 \text{ kips}$$

a) The design strength based on yielding is

$$\phi_t P_n = 0.90(239.5) = 215.6 \text{ kips}$$

The design strength based on rupture is

$\phi_t P_n = 0.75(243.7) = 182.8$ kips

The design strength is the smaller value: $\phi_t P_n = 182.8$ kips

Let $P_u = \phi_t P_n$

$1.2D + 1.6(2D) = 182.8$, Solution is: 41.55

$P = D + L = 41.55 + 2(41.55) = 125$ kips $\qquad$ $\underline{P = 125 \text{ kips}}$

b) The allowable strength based on yielding is

$$\frac{P_n}{\Omega_t} = \frac{239.5}{1.67} = 143.4 \text{ kips}$$

The allowable strength based on rupture is

$$\frac{P_n}{\Omega_t} = \frac{243.7}{2.00} = 121.9 \text{ kips}$$

The allowable load is the smaller value = 121.9 kips $\qquad$ $\underline{P = 122 \text{ kips}}$

Alternate computation of allowable load using allowable ***stress***: For yielding,

$F_t = 0.6F_y = 0.6(50) = 30.0$ ksi

and the allowable load is

$F_t A_g = 30.0(4.79) = 143.7$ kips

For rupture,

$F_t = 0.5F_u = 0.5(70) = 35$ ksi

and the allowable load is

$F_t A_e = 35(3.482) = 121.9$ kips

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**3.3-5**

Gross section: $P_n = F_y A_g = 36(5.86) = 211.0$ kips

Net section: $A_n = 5.86 - \left(\frac{5}{8}\right)\left(1 + \frac{3}{16}\right)(2) = 4.376 \text{ in.}^2$

$$U = 1 - \frac{\bar{x}}{\ell} = 1 - \frac{1.03}{(3+3+3)} = 0.8856$$

$A_e = A_n U = 4.376(0.8856) = 3.875 \text{ in.}^2$

$P_n = F_u A_e = 58(3.875) = 224.8$ kips

(a) The design strength based on yielding is

$\phi_t P_n = 0.90(211.0) = 190$ kips

The design strength based on rupture is

$$\phi_t P_n = 0.75(224.8) = 168.6 \text{ kips}$$

The design strength is the smaller value: $\phi_t P_n = 169$ kips

Load combination 2 controls:

$$P_u = 1.2D + 1.6L = 1.2(50) + 1.6(100) = 220 \text{ kips}$$

Since $P_u > \phi_t P_n$, (220 kips > 169 kips), The member is not adequate.

(b) For the gross section, The allowable strength is $\dfrac{P_n}{\Omega_t} = \dfrac{211.0}{1.67} = 126$ kips

For the net section, the allowable strength is $\dfrac{P_n}{\Omega_t} = \dfrac{224.8}{2.00} = 112.4$ kips

The smaller value controls; the allowable strength is 112 kips.

Load combination 6 controls:

$$P_a = D + 0.75L + 0.75(0.6W) = 50 + 0.75(100) + 0.75(0.6)(45) = 145.3 \text{ kips}$$

Since 145 kips > 112 kips, The member is not adequate.

Alternate ASD solution using allowable stress:

$$F_t = 0.6F_y = 0.6(36) = 21.6 \text{ ksi}$$

and the allowable strength is $F_t A_g = 21.6(5.86) = 127$ kips

For the net section, $F_t = 0.5F_u = 0.5(58) = 29.0$ ksi

and the allowable strength is $F_t A_e = 29.0(3.875) = 112.4$ kips

The smaller value controls; the allowable strength is 112 kips. From load combination 6,

Since 145 kips > 112 kips, the member is not adequate.

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**3.3-6**

For yielding of the gross section,

$$A_g = 5(1/4) = 1.25 \text{ in.}^2$$

$$P_n = F_y A_g = 36(1.25) = 45.0 \text{ kips}$$

For rupture of the net section, from AISC Table D3.1, case 4,

$$U = \frac{3\ell^2}{3\ell^2 + w^2}\left(1 - \frac{\bar{x}}{\ell}\right) = \frac{3(7)^2}{3(7)^2 + (5)^2}\left(1 - \frac{0.25/2}{7}\right) = 0.8394$$

$A_e = A_g U = 1.25(0.8394) = 1.049 \text{ in.}^2$

$P_n = F_u A_e = 58(1.049) = 60.84 \text{ kips}$

a) The design strength based on yielding is

$\phi_t P_n = 0.90(45.0) = 40.5 \text{ kips}$

The design strength based on rupture is

$\phi_t P_n = 0.75(60.84) = 45.63 \text{ kips}$

The design strength for LRFD is the smaller value: $\underline{\phi_t P_n = 40.5 \text{ kips}}$

b) The allowable strength based on yielding is

$$\frac{P_n}{\Omega_t} = \frac{45.0}{1.67} = 27.0 \text{ kips}$$

The allowable strength based on rupture is

$$\frac{P_n}{\Omega_t} = \frac{60.84}{2.00} = 30.42 \text{ kips}$$

The allowable service load is the smaller value: $\underline{P_n/\Omega_t = 27.0 \text{ kips}}$

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**3.3-7**

Gross section: $P_n = F_y A_g = 50(10.3) = 515.0 \text{ kips}$

Net section: $A_n = 10.3 - 0.520\left(\frac{7}{8} + \frac{1}{8}\right)(4) = 8.220 \text{ in.}^2$

Connection is through the flanges with four bolts per line.

$$\frac{b_f}{d} = \frac{6.56}{12.5} = 0.525 < \frac{2}{3} \qquad \therefore\ U = 0.85$$

$A_e = A_n U = 8.220(0.85) = 6.987 \text{ in.}^2$

$P_n = F_u A_e = 65(6.987) = 454.2 \text{ kips}$

(a) The design strength based on yielding is

$\phi_t P_n = 0.90(515.0) = 464 \text{ kips}$

The design strength based on rupture is

$\phi_t P_n = 0.75(454.2) = 341 \text{ kips}$

The design strength is the smaller value: $\underline{\phi_t P_n = 341 \text{ kips}}$

(b) For the gross section, The allowable strength is $\dfrac{P_n}{\Omega_t} = \dfrac{515.0}{1.67} = 308 \text{ kips}$

For the net section, the allowable strength is $\dfrac{P_n}{\Omega_t} = \dfrac{454.2}{2.00} = 227$ kips

The smaller value controls; $\underline{\dfrac{P_n}{\Omega_t} = 227 \text{ kips}}$

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**3.3-8**

Gross section: $P_n = F_y A_g = 50(5.17) = 258.5$ kips

Net section:

$$U = 1 - \frac{\bar{x}}{\ell} = 1 - \frac{1.30}{10} = 0.87$$

$$A_e = A_g U = 5.17(0.87) = 4.498 \text{ in.}^2$$

$$P_n = F_u A_e = 70(4.498) = 314.9 \text{ kips}$$

(a) The design strength based on yielding is

$$\phi_t P_n = 0.90(258.5) = 233 \text{ kips}$$

The design strength based on rupture is

$$\phi_t P_n = 0.75(314.9) = 236 \text{ kips}$$

The design strength is the smaller value: $\phi_t P_n = 233$ kips

Load combination 3:

$$P_u = 1.2D + 1.6S + 0.5W = 1.2(75) + 1.6(50) + 0.5(70) = 205.0 \text{ kips}$$

Load combination 4:

$$P_u = 1.2D + 1.0W + 0.5L + 0.5S = 1.2(75) + 1.0(70) + 0.5(50) = 185.0 \text{ kips}$$

Load combination 3 controls. Since $P_u < \phi_t P_n$, (205 kips < 233 kips),

The member is adequate.

(b) For the gross section, The allowable strength is $\dfrac{P_n}{\Omega_t} = \dfrac{258.5}{1.67} = 155$ kips

For the net section, the allowable strength is $\dfrac{P_n}{\Omega_t} = \dfrac{314.9}{2.00} = 157$ kips

The smaller value controls; the allowable strength is 155 kips.

Load combination 3: $P_a = D + S = 75 + 50 = 125$ kips

Load combination 6:

$$P_a = D + 0.75(0.6W) + 0.75S = 75 + 0.75(0.6)(70) + 0.75(50) = 144.0 \text{ kips}$$

Load combination 6 controls. Since 144 kips < 155 kips, The member is adequate.

**3.4-1**

Gross section: $A_g = 10(1/2) = 5 \text{ in.}^2$

Net section: Hole diameter $= \dfrac{7}{8} + \dfrac{1}{8} = 1$ in.

Possibilities for net area:

$$A_n = A_g - \sum t \times (d \text{ or } d') = 5 - (1/2)(1)(2) = 4.0 \text{ in.}^2$$

or $A_n = 5 - (1/2)(1) - (1/2)\left[1 - \dfrac{(2)^2}{4(3)}\right] - (1/2)\left[1 - \dfrac{(2)^2}{4(3)}\right] = 3.833 \text{ in.}^2$

or $A_n = 5 - (1/2)(1)(3) = 3.5 \text{ in.}^2$, but because of load transfer,

use $A_n = \dfrac{9}{6}(3.5) = 5.25 \text{ in.}^2$ for this possibility.

The smallest value controls. Use $A_n = 3.833 \text{ in.}^2$

$$A_e = A_n U = A_n(1.0) = 3.833 \text{ in.}^2$$

$$P_n = F_u A_e = 58(3.833) = 222 \text{ kips}$$

The nominal strength based on the net section is $\underline{P_n = 222 \text{ kips}}$

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**3.4-2**

Compute the strength of one plate, then double it.

Gross section: $A_g = 10(1/2) = 5.0 \text{ in.}^2$

Net section: Hole diameter $= \dfrac{3}{4} + \dfrac{1}{8} = \dfrac{7}{8}$ in.

Possibilities for net area:

$$A_n = A_g - \sum t \times (d \text{ or } d') = 5 - (1/2)(7/8)(2) = 4.125 \text{ in.}^2$$

or $A_n = 5 - (1/2)(7/8) - (1/2)\left[\dfrac{7}{8} - \dfrac{(5)^2}{4(6)}\right] = 4.646 \text{ in.}^2$

Because of load transfer, use $A_n = \dfrac{10}{9}(4.646) = 5.162 \text{ in.}^2$ for this possibility.

or $A_n = 5 - (1/2)(7/8) - (1/2)\left[\dfrac{7}{8} - \dfrac{(2)^2}{4(3)}\right] - (1/2)\left[\dfrac{7}{8} - \dfrac{(2)^2}{4(3)}\right] = 4.021 \text{ in.}^2$

Because of load transfer, use $A_n = \dfrac{10}{8}(4.021) = 5.026 \text{ in.}^2$ for this possibility.

The smallest value controls. Use $A_n = 4.125 \text{ in.}^2$

$$A_e = A_n U = 4.125(1.0) = 4.125 \text{ in.}^2$$

$$P_n = F_u A_e = 58(4.125) = 239.3 \text{ kips}$$

For two plates, $P_n = 2(239.3) = 478.6$ kips

The nominal strength based on the net section is $\underline{P_n = 479 \text{ kips}}$

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**3.4-3**

Gross section: $A_g = 8(3/8) = 3.0 \text{ in.}^2$, $P_n = F_y A_g = 36(3.0) = 108 \text{ kips}$

Net section: Hole diameter $= \dfrac{1}{2} + \dfrac{1}{8} = \dfrac{5}{8}$ in.

$$A_n = A_g - \sum t_w \times (d \text{ or } d') = 3 - (3/8)(5/8) = 2.766 \text{ in.}^2$$

or $A_n = 3 - (3/8)(5/8) - (3/8)\left[5/8 - \dfrac{(3)^2}{4(2)}\right] = 2.954 \text{ in.}^2$

or $A_n = 3 - (3/8)(5/8) - (3/8)\left[5/8 - \dfrac{(3)^2}{4(2)}\right] \times 2 = 3.141 \text{ in.}^2$

or $A_n = [3 - (3/8)(5/8)(2)] \times \dfrac{6}{5} = 3.038 \text{ in.}^2$

or $A_n = \left(3 - (3/8)(5/8) - (3/8)\left[5/8 - \dfrac{(2.5)^2}{4(2)}\right](2)\right) \times \dfrac{6}{5} = 3.460 \text{ in.}^2$

Use $A_e = A_n = 2.766 \text{ in.}^2$

$$P_n = F_u A_e = 58(2.766) = 160.4 \text{ kips}$$

(a) Gross section: $\phi_t P_n = 0.90(108) = 97.2$ kips

Net section: $\phi_t P_n = 0.75(160.4) = 120$ kips $\underline{\phi_t P_n = 97.2 \text{ kips}}$

(b) Gross section: $\dfrac{P_n}{\Omega_t} = \dfrac{108}{1.67} = 64.7$ kips

Net section: $\dfrac{P_n}{\Omega_t} = \dfrac{160.4}{2.00} = 80.2$ kips $\underline{P_n/\Omega_t = 64.7 \text{ kips}}$

---

**3.4-4**

Gross section: $A_g = 5.87 \text{ in.}^2$, $P_n = F_y A_g = 50(5.87) = 293.5 \text{ kips}$

Net section: Hole diameter $= 1\dfrac{1}{8} + \dfrac{3}{16} = 1.313$ in.

$$A_n = A_g - \sum t_w \times (d \text{ or } d') = 5.87 - 0.448(1.313) = 5.282 \text{ in.}^2$$

or $A_n = 5.87 - 0.448(1.313) - 0.448\left(1.313 - \dfrac{(1.5)^2}{4(4)}\right) = 4.757 \text{ in.}^2$

$$U = 1 - \frac{\bar{x}}{\ell} = 1 - \frac{0.583}{6(1.5)} = 0.9352$$

$$A_e = A_n U = 4.757(0.9352) = 4.449 \text{ in.}^2$$

$$P_n = F_u A_e = 70(4.449) = 311.4 \text{ kips}$$

(a) Gross section: $\phi_t P_n = 0.90(293.5) = 264$ kips

Net section: $\phi_t P_n = 0.75(311.4) = 234$ kips (controls)

$P_u = 1.2D + 1.6L = 1.2(36) + 1.6(110) = 219 \text{ kips} < 234 \text{ kips}$ (OK)

Since $P_u < \phi_t P_n$ (219 kips < 234 kips), The member has enough strength.

(b) Gross section: $\frac{P_n}{\Omega_t} = \frac{293.5}{1.67} = 176$ kips

Net section: $\frac{P_n}{\Omega_t} = \frac{311.4}{2.00} = 156$ kips (controls)

$P_a = D + L = 36 + 110 = 146 \text{ kips} < 156 \text{ kips}$ (OK)

Since $P_a < \frac{P_n}{\Omega_t}$ (146 kips < 156 kips), The member has enough strength.

---

**3.4-5**

For A572 Grade 50 steel, $F_y = 50$ ksi and $F_u = 65$ ksi.

Compute the strength for one angle, then multiply by 2.

Gross section: $A_g = 4.00 \text{ in.}^2$, $P_n = F_y A_g = 50(4.00) = 200.0$ kips

For two angles, $P_n = 2(200.0) = 400.0$ kips

Net section: Hole diameter $= \frac{7}{8} + \frac{1}{8} = 1$ in.

$$A_n = A_g - \sum t \times (d \text{ or } d') = 4.00 - (3/8)(1) = 3.625 \text{ in.}^2$$

or $A_n = 4.00 - (3/8)(1) - (3/8)\left(1 - \frac{(3)^2}{4(1.5)}\right) = 3.813 \text{ in.}^2$

or $A_n = 4.00 - (3/8)(1) - (3/8)\left(1 - \frac{(3)^2}{4(1.5)}\right) \times 2 = 4.0 \text{ in.}^2$

or $A_n = 4.00 - (3/8)(1) \times 2 = 3.25 \text{ in.}^2$, but because of load transfer,

use $A_n = \frac{7}{6}(3.25) = 3.792 \text{ in.}^2$ for this possibility.

$$U = 1 - \frac{\bar{x}}{\ell} = 1 - \frac{0.861}{3 + 3 + 3} = 0.9043$$

$$A_e = A_n U = 3.625(0.9043) = 3.278 \text{ in.}^2$$

$P_n = F_u A_e = 65(3.278) = 213.1$ kips

For two angles, $P_n = 2(213.1) = 426.2$ kips

(a) LRFD Solution

Gross section: $\phi_t P_n = 0.90(400) = 360$ kips

Net section: $\phi_t P_n = 0.75(426.2) = 320$ kips (controls)

$\underline{\phi_t P_n = 320 \text{ kips}}$

(b) ASD Solution

Gross section: $\dfrac{P_n}{\Omega_t} = \dfrac{400.0}{1.67} = 240$ kips

Net section: $\dfrac{P_n}{\Omega_t} = \dfrac{426.2}{2.00} = 213$ kips

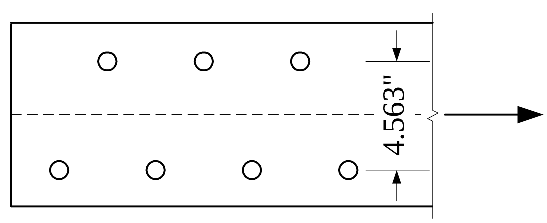
$$\underline{\frac{P_n}{\Omega_t} = 213 \text{ kips}}$$

---

**<u>3.4-6</u>**

Gross section: $P_n = F_y A_g = 36(3.30) = 118.8$ kips

Net section: Use a gage distance of $2.5 + 2.5 - \dfrac{7}{16} = 4.563$ in.




Hole diameter = $\dfrac{3}{4} + \dfrac{1}{8} = \dfrac{7}{8}$ in.

$$A_n = A_g - \sum t \times (d \text{ or } d')$$

$$= 3.30 - (7/16)(7/8) = 2.917 \text{ in.}^2$$

or $A_n = 3.30 - (7/16)(7/8) - (7/16)\left(\dfrac{7}{8} - \dfrac{(2)^2}{4(4.563)}\right) = 2.63 \text{ in.}^2$

Use $A_e = A_n = 2.63 \text{ in.}^2$, and $P_n = F_u A_e = 58(2.63) = 152.5$ kips

(a) Gross section: $\phi_t P_n = 0.90(118.8) = 106.9$ kips

Net section: $\phi_t P_n = 0.75(152.5) = 114.4$ kips

Gross section controls. $\underline{\phi_t P_n = 107 \text{ kips}}$

(b) Gross section: $\dfrac{P_n}{\Omega_t} = \dfrac{118.8}{1.67} = 71.14 \text{ kips}$

Net section: $\dfrac{P_n}{\Omega_t} = \dfrac{152.5}{2.00} = 76.25 \text{ kips}$

Gross section controls. $\underline{P_n/\Omega_t = 71.1 \text{ kips}}$

---

**3.5-1**

Shear areas:

$$A_{gv} = \frac{7}{16}(4.5) = 1.969 \text{ in.}^2$$

$$A_{nv} = \frac{7}{16}[4.5 - 1.5(1.0)] = 1.313 \text{ in.}^2$$

Tension area $= A_{nt} = \dfrac{7}{16}[1.75 - 0.5(1.0)] = 0.5469 \text{ in.}^2$

For this type of connection, $U_{bs} = 1.0,$ and from AISC Equation J4-5,

$$R_n = 0.6F_uA_{nv} + U_{bs}F_uA_{nt}$$

$$= 0.6(65)(1.313) + 1.0(65)(0.5469) = 86.8 \text{ kips}$$

with an upper limit of

$$0.6F_yA_{gv} + U_{bs}F_uA_{nt} = 0.6(50)(1.969) + 1.0(65)(0.5469) = 94.6 \text{ kips}$$

$\underline{R_n = 86.8 \text{ kips}}$

---

**3.5-2**

Shear areas:

$$A_{gv} = \frac{1}{2}(2 + 4) \times 2 = 6 \text{ in.}^2$$

$$A_{nv} = \frac{1}{2}(2 + 4 - 1.5(1 + 3/16)) \times 2 = 4.219 \text{ in.}^2$$

Tension area $= A_{nt} = \dfrac{1}{2}(7.5 - 2 - 2 - (0.5 + 0.5)(1 + 3/16)) = 1.156 \text{ in.}^2$

For this type of connection, $U_{bs} = 1.0,$ and from AISC Equation J4-5,

$$R_n = 0.6F_uA_{nv} + U_{bs}F_uA_{nt}$$

$$= 0.6(58)(4.219) + 1.0(58)(1.156) = 214 \text{ kips}$$

with an upper limit of

$$0.6F_yA_{gv} + U_{bs}F_uA_{nt} = 0.6(36)(6) + 1.0(58)(1.156) = 197 \text{ kips}$$

$\underline{R_n = 197 \text{ kips}}$

---

**3.5-3**

Tension member:

The shear areas are $A_{gv} = \frac{7}{16}(3.5 + 1.5) \times 2 = 4.375 \text{ in.}^2$

$$A_{nv} = \frac{7}{16}\left[3.5 + 1.5 - 1.5\left(\frac{3}{4} + \frac{1}{8}\right)\right] \times 2 = 3.227 \text{ in.}^2$$

The tension area is $A_{nt} = \frac{7}{16}\left[3.0 - (0.5 + 0.5)\left(\frac{3}{4} + \frac{1}{8}\right)\right] = 0.9297 \text{ in.}^2$

For this type of connection, $U_{bs} = 1.0$, and from AISC Equation J4-5,

$$R_n = 0.6F_uA_{nv} + U_{bs}F_uA_{nt}$$

$$= 0.6(58)(3.227) + 1.0(58)(0.9297) = 166.2 \text{ kips}$$

with an upper limit of

$$0.6F_yA_{gv} + U_{bs}F_uA_{nt} = 0.6(36)(4.375) + 1.0(58)(0.9297) = 148.4 \text{ kips}$$

The nominal block shear strength of the tension member is therefore 148.4 kips.

Gusset Plate:

$$A_{gv} = \frac{3}{8}(3.5 + 2.5) \times 2 = 4.5 \text{ in.}^2$$

$$A_{nv} = \frac{3}{8}[3.5 + 2.5 - 1.5(7/8)] \times 2 = 3.516 \text{ in.}^2$$

$$A_{nt} = \frac{3}{8}[3.0 - (0.5 + 0.5)(7/8)] = 0.7969 \text{ in.}^2$$

From AISC Equation J4-5,

$$R_n = 0.6F_uA_{nv} + U_{bs}F_uA_{nt}$$

$$= 0.6(58)(3.516) + 1.0(58)(0.7969) = 168.6 \text{ kips}$$

with an upper limit of

$$0.6F_yA_{gv} + U_{bs}F_uA_{nt} = 0.6(36)(4.5) + 1.0(58)(0.7969) = 143.4 \text{ kips}$$

The nominal block shear strength of the gusset plate is therefore 143.4 kips

The gusset plate controls, and the nominal block shear strength of the connection is 143.4 kips

(a) The design strength is $\phi R_n = 0.75(143.4) = 108 \text{ kips}$ $\underline{\phi R_n = 108 \text{ kips}}$

(b) The allowable strength is $\frac{R_n}{\Omega} = \frac{143.4}{2.00} = 71.7$ kips $\underline{R_n/\Omega = 71.7 \text{ kips}}$

---

**3.5-4**

Gross section nominal strength:

$$P_n = F_y A_g = 50(3.59) = 179.5 \text{ kips} \qquad (A_g = 3.59 \text{ in.}^2 \text{ for a C7 x 12.25})$$

Net section nominal strength:

$$A_n = 3.59 - 0.314(7/8)(2) = 3.041 \text{ in.}^2$$

$$U = 1 - \frac{\bar{x}}{\ell} = 1 - \frac{0.525}{(3+3)} = 0.9125$$

$$A_e = A_n U = 3.041(0.9125) = 2.775 \text{ in.}^2$$

$$P_n = F_u A_e = 65(2.775) = 180.4 \text{ kips}$$

Block shear strength of tension member:

The shear areas are $A_{gv} = 0.314(1.5+3+3) \times 2 = 4.710 \text{ in.}^2$

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

The tension area is

$$A_{nt} = 0.314[3.0 - (0.5+0.5)(7/8)] = 0.6673 \text{ in.}^2$$

For this type of connection, $U_{bs} = 1.0$, and from AISC Equation J4-5,

$$R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt}$$

$$= 0.6(65)(3.336) + 1.0(65)(0.6673) = 173.5 \text{ kips}$$

with an upper limit of

$$0.6F_y A_{gv} + U_{bs} F_u A_{nt} = 0.6(50)(4.710) + 1.0(65)(0.6673) = 184.7 \text{ kips}$$

The nominal block shear strength of the tension member is therefore 173.5 kips.

Block shear strength of gusset plate:

$$A_{gv} = \frac{3}{8}(1.5+3+3) \times 2 = 5.625 \text{ in.}^2$$

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

$$A_{nt} = \frac{3}{8}[3 - (0.5+0.5)(7/8)] = 0.7969 \text{ in.}^2$$

From AISC Equation J4-5,

$$R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt}$$

$$= 0.6(58)(3.984) + 1.0(58)(0.7969) = 184.9 \text{ kips}$$

with an upper limit of

$$0.6F_yA_{gv} + U_{bs}F_uA_{nt} = 0.6(36)(5.625) + 1.0(58)(0.7969) = 167.7 \text{ kips}$$

The nominal block shear strength of the gusset plate is therefore 167.7 kips. The gusset plate controls, and the nominal block shear strength of the connection is 167.7 kips

(a) Design strength for LRFD:

For tension on the gross area, $\phi_t P_n = 0.90(179.5) = 162$ kips

For tension on the net area, $\phi_t P_n = 0.75(180.4) = 135$ kips

For block shear, $\phi R_n = 0.75(167.7) = 126$ kips

Block shear controls. <u>Maximum factored load = design strength = 126 kips</u>

(b) Allowable strength for ASD:

For tension on the gross area, $\dfrac{P_n}{\Omega_t} = \dfrac{179.5}{1.67} = 108$ kips

For tension on the net area, $\dfrac{P_n}{\Omega_t} = \dfrac{180.4}{2.00} = 90.2$ kips

For block shear, $\dfrac{R_n}{\Omega} = \dfrac{167.7}{2.00} = 83.9$ kips

Block shear controls. <u>Maximum service load = allowable strength = 83.9 kips</u>

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**<u>3.6-1</u>**

(a) $P_u = 1.2D + 1.6L = 1.2(28) + 1.6(84) = 168$ kips

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

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

Required $r_{min} = \dfrac{L}{300} = \dfrac{18 \times 12}{300} = 0.72$ in.

Try L5 × 3½ ×¾

$A_g = 5.85 \text{ in.}^2 > 5.19 \text{ in.}^2$ (OK)

$r_{min} = r_z = 0.744 \text{ in.} > 0.72 \text{ in.}$ (OK)

$A_n = 5.85 - 0.75(1 + 3/16) = 4.959 \text{ in.}^2$

$A_e = A_nU = 4.959(0.80) = 3.97 \text{ in.}^2 > 3.86 \text{ in.}^2$ (OK)

Use an L5 × 3½ × ¾

(b) $P_a = D + L = 28 + 84 = 112$ kips

$$\text{Required } A_g = \frac{P_a}{F_t} = \frac{P_a}{0.6F_y} = \frac{112}{0.6(36)} = 5.19 \text{ in.}^2$$

$$\text{Required } A_e = \frac{P_a}{0.5F_u} = \frac{112}{0.5(58)} = 3.86 \text{ in.}^2$$

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

Try L5 × 3½ ×¾

$A_g = 5.85 \text{ in.}^2 > 5.19 \text{ in.}^2$ (OK)

$r_{\min} = r_z = 0.744 \text{ in.} > 0.72 \text{ in.}$ (OK)

$A_n = 5.85 - 0.75(1 + 3/16) = 4.959 \text{ in.}^2$

$A_e = A_n U = 4.959(0.80) = 3.97 \text{ in.}^2 > 3.86 \text{ in.}^2$ (OK)

Use an L5 × 3½ × ¾

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**3.6-2**

(a) $P_u = 1.2D + 1.6L = 1.2(100) + 1.6(50) = 200.0$ kips

$$\text{Required } A_g = \frac{P_u}{0.9F_y} = \frac{200}{0.9(36)} = 6.17 \text{ in.}^2$$

$$\text{Required } A_e = \frac{P_u}{0.75F_u} = \frac{200}{0.75(58)} = 4.60 \text{ in.}^2$$

$$\text{Required } r_{\min} = \frac{L}{300} = \frac{20 \times 12}{300} = 0.8 \text{ in.}$$

Try C12 × 25

$A_g = 7.34 \text{ in.}^2 > 6.17 \text{ in.}^2$ (OK)

$r_{\min} = r_y = 0.779 \text{ in.} < 0.8 \text{ in.}$ (N.G.)

(Although this value for the radius of gyration does not quite satisfy the AISC recommendation for maximum slenderness, tensile strength is not affected by slenderness, so some leeway is permitted. Therefore, we will consider this value acceptable.)

$A_n = 7.34 - 0.387(1 + 3/16)(2) = 6.421 \text{ in.}^2$

$$U = 1 - \frac{\bar{x}}{\ell} = 1 - \frac{0.674}{6} = 0.8877$$

$A_e = A_nU = 6.421(0.8877) = 5.70 \text{ in.}^2 > 4.60 \text{ in.}^2$ (OK)

Use a C12 × 25

(b) $P_a = D + L = 100 + 50 = 150$ kips

Required $A_g = \dfrac{P_a}{F_t} = \dfrac{P_a}{0.6F_y} = \dfrac{150}{0.6(36)} = 6.94 \text{ in.}^2$

Required $A_e = \dfrac{P_a}{0.5F_u} = \dfrac{150}{0.5(58)} = 5.17 \text{ in.}^2$

Required $r_{\min} = \dfrac{L}{300} = \dfrac{20 \times 12}{300} = 0.8$ in.

Try C12 × 25

$A_g = 7.34 \text{ in.}^2 > 6.94 \text{ in.}^2$ (OK)

$r_{\min} = r_y = 0.779 \text{ in.} < 0.8 \text{ in.}$ (N.G.)

(Although this value for the radius of gyration does not quite satisfy the AISC recommendation for maximum slenderness, tensile strength is not affected by slenderness, so some leeway is permitted. Therefore, we will consider this value acceptable.)

$A_n = 7.34 - 0.387(1 + 3/16)(2) = 6.421 \text{ in.}^2$

$U = 1 - \dfrac{\bar{x}}{\ell} = 1 - \dfrac{0.674}{6} = 0.8877$

$A_e = A_nU = 6.421(0.8877) = 5.70 \text{ in.}^2 > 5.17 \text{ in.}^2$ (OK)

Use a C12 × 25

---

**3.6-3**

(a) $P_u = 1.2D + 1.6L = 1.2(30) + 1.6(90) = 180.0$ kips

Required $A_g = \dfrac{P_u}{0.9F_y} = \dfrac{180}{0.90(50)} = 4.00 \text{ in.}^2$

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

Required $r_{\min} = \dfrac{L}{300} = \dfrac{25 \times 12}{300} = 1.0$ in.

The angle leg must be at least 5 in. long to accommodate two lines of bolts (See workable gages for angles, *Manual* Table 1-7A).

Try 2L5 × 5 × $^5/_{16}$

$A_g = 6.14 \text{ in.}^2 > 4.00 \text{ in.}^2$ (OK) $r_{\min} = r_x = 1.56 \text{ in.} > 1.0 \text{ in.}$ (OK)

$$A_n = 6.14 - 4(7/8 + 1/8)(5/16) = 4.89 \text{ in.}^2$$

From AISC Table D4.1, for 4 or more bolts per line, $U = 0.80$

$$A_e = A_n U = 4.89(0.80) = 3.91 \text{ in.}^2 > 3.69 \text{ in.}^2 \quad \text{(OK)}$$

<u>Use 2L5 × 5 × $^5/_{16}$</u>

(b) $P_a = D + L = 30 + 90 = 120$ kips

$$\text{Required } A_g = \frac{P_a}{0.6F_y} = \frac{120}{0.6(50)} = 4.00 \text{ in.}^2$$

$$\text{Required } A_e = \frac{P_a}{0.5F_u} = \frac{120}{0.5(65)} = 3.69 \text{ in.}^2$$

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

The angle leg must be at least 5 in. long to accommodate two lines of bolts (See workable gages for angles, *Manual* Table 1-7A).

Try 2L5 × 5 × $^5/_{16}$

$$A_g = 6.14 \text{ in.}^2 > 4.00 \text{ in.}^2 \quad \text{(OK)} \qquad r_{\min} = r_x = 1.56 \text{ in.} > 1.0 \text{ in.} \quad \text{(OK)}$$

$$A_n = 6.14 - 4(7/8 + 1/8)(5/16) = 4.89 \text{in.}^2$$

From AISC Table D4.1, for 4 or more bolts per line, $U = 0.80$

$$A_e = A_n U = 4.89(0.80) = 3.91 \text{ in.}^2 > 3.69 \text{ in.}^2 \quad \text{(OK)}$$

<u>Use 2L5 × 5 × $^5/_{16}$</u>

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**<u>3.6-4</u>**

(a) Load combination 2 controls:

$$P_u = 1.2D + 1.6L = 1.2(54) + 1.6(80) = 192.8 \text{ kips}$$

$$\text{Required } A_g = \frac{P_u}{0.90F_y} = \frac{192.8}{0.90(50)} = 4.28 \text{ in.}^2$$

$$\text{Required } A_e = \frac{P_u}{0.75F_u} = \frac{192.8}{0.75(65)} = 3.96 \text{ in.}^2$$

$$\text{Required } r_{\min} = \frac{L}{300} = \frac{17.5 \times 12}{300} = 0.7 \text{ in.}$$

Try C10 × 20:

$$A_g = 5.87 \text{ in.}^2 > 4.28 \text{ in.}^2 \quad \text{(OK)}$$

$$r_{\min} = r_y = 0.690 \text{ in.} \approx 0.7 \text{ in.} \quad \text{(say OK)}$$

$A_e = A_g U = 5.87(0.85) = 4.99 \text{ in.}^2 > 3.96 \text{ in.}^2$ (OK) Use a C10 × 20

(b) Load combination 6 controls:

$$P_a = D + 0.75L + 0.75(0.6W) = 54 + 0.75(80) + 0.75(0.6 \times 75) = 147.8 \text{ kips}$$

$$\text{Required } A_g = \frac{P_a}{0.6F_y} = \frac{147.8}{0.6(50)} = 4.93 \text{ in.}^2$$

$$\text{Required } A_e = \frac{P_a}{0.5F_u} = \frac{147.8}{0.5(65)} = 4.55 \text{ in.}^2$$

$$\text{Required } r_{\min} = \frac{L}{300} = \frac{17.5 \times 12}{300} = 0.7 \text{ in.}$$

Try C12 × 25:

$A_g = 7.34 \text{ in.}^2 > 4.93 \text{ in.}^2$ (OK)

$r_{\min} = r_y = 0.779 \text{ in.} > 0.7 \text{ in.}$ (OK)

$A_e = A_g U = 7.34(0.85) = 6.24 \text{ in.}^2 > 4.55 \text{ in.}^2$ (OK) Use a C12 × 25

---

**3.6-5**

$$\text{Required } A_g = \frac{P_u}{0.9F_y} = \frac{180}{0.9(36)} = 5.56 \text{ in.}^2$$

$$\text{Required } A_e = \frac{P_u}{0.75F_u} = \frac{180}{0.75(58)} = 4.14 \text{ in.}^2$$

$$\text{Required } r_{\min} = \frac{L}{300} = \frac{15 \times 12}{300} = 0.6 \text{ in.}$$

Try C10 × 20

$A_g = 5.87 \text{ in.}^2 > 5.56 \text{ in.}^2$ (OK)

$r_{\min} = r_y = 0.690 \text{ in.} > 0.6 \text{ in.}$ (OK)

$A_n = 5.87 - 0.379(1.0)(2) = 5.112 \text{ in.}^2$

$A_e = A_n U = 5.112(0.85) = 4.35 \text{ in.}^2 > 4.15 \text{ in.}^2$ (OK)

Use a C10 × 20

---

**3.6-6**

From Part 1 of the *Manual*, $F_y = 50$ ksi and $F_u = 70$ ksi.

$$P_u = 1.2D + 1.6L = 1.2(175) + 1.6(175) = 490.0 \text{ kips}$$

$$\text{Required } A_g = \frac{P_u}{0.9F_y} = \frac{490}{0.9(50)} = 10.9 \text{ in.}^2$$

$$\text{Required } A_e = \frac{P_u}{0.75F_u} = \frac{490}{0.75(70)} = 9.33 \text{ in.}^2$$

$$\text{Required } r_{\min} = \frac{L}{300} = \frac{30 \times 12}{300} = 1.2 \text{ in.}$$

Try W10 × 49

$A_g = 14.4 \text{ in.}^2 > 10.9 \text{ in.}^2$ (OK)

$r_{\min} = r_y = 2.54 \text{ in.} > 1.2 \text{ in.}$ (OK)

$A_n = 14.4 - 0.560(1.25 + 3/16)(4) = 11.18 \text{ in.}^2$

$\frac{b_f}{d} = \frac{10.0}{10.0} > \frac{2}{3}$ $\Rightarrow$ From AISC Table D3.1, Case 7, $U = 0.90$

$A_e = A_n U = 11.18(0.90) = 10.1 \text{in.}^2 > 9.33 \text{ in.}^2$ (OK)

Use a W10 × 49

---

**3.7-1**

(a) LRFD: Load combination 1 controls: $P_u = 1.4(45) = 63.00$ kips

$$\text{Required } A_b = \frac{P_u}{0.75(0.75F_u)} = \frac{63.00}{0.75(0.75)(58)} = 1.931 \text{ in.}^2$$

Let $\frac{\pi d^2}{4} = 1.931,$ $d = 1.568$ in.

Required $d = 1.57$ in. Use $1^5/8$ in.

(b) ASD: Load combination 2 controls: $P_a = D + L = 45 + 5 = 50$ kips

$F_t = 0.375F_u = 0.375(58) = 21.75$ ksi

$$\text{Required } A_b = \frac{P_a}{F_t} = \frac{50}{21.75} = 2.299 \text{ in.}^2$$

Let $\frac{\pi d^2}{4} = 2.299,$ $d = 1.71$ in.

Required $d = 1.71$ in. Use $d = 1¾$ in.

---

**3.7-2**

(a) Dead load = beam weight = 0.048 kips/ft

$$w_u = 1.2w_D + 1.6w_L = 1.2(0.048) = 0.0576 \text{ kips/ft}$$

$$P_u = 1.2P_D + 1.6P_L = 1.6(20) = 32.0 \text{ kips}$$

Because of symmetry, the tension is the same in both rods.

$$T_u = \frac{1}{2}[0.0576(30) + 32] = 16.86 \text{ kips}$$

$$\text{Required Area} = A_b = \frac{T_u}{0.75(0.75F_u)} = \frac{16.86}{0.75(0.75)(58)} = 0.5168 \text{ in.}^2$$

From $A_b = \dfrac{\pi d^2}{4}$, required $d = \sqrt{\dfrac{4(0.5168)}{\pi}} = 0.811$ in.

Required $d = 0.811$ in., use $d = 7/8$ in.

(b) Maximum force in rod occurs when live load is an *A* or *D*. Entire live load is taken by one rod.

$$T_u = \frac{0.0576(30)}{2} + 32 = 32.86 \text{ kips}$$

$$\text{Required } A_b = \frac{T_u}{0.75(0.75F_u)} = \frac{32.86}{0.75(0.75)(58)} = 1.007 \text{ in.}^2$$

Let $\dfrac{\pi d^2}{4} = 1.007$, $d = 1.13$ in. Required $d = 1.13$ in., use $d = 1¼$ in.

---

**3.7-3**

(a) Dead load = beam weight = 0.048 kips/ft

Because of symmetry, the tension is the same in both rods.

$$T_a = \frac{1}{2}[0.048(30) + 20] = 10.72 \text{ kips}$$

$$F_t = 0.375F_u = 0.375(58) = 21.75 \text{ ksi}$$

$$\text{Required } A_b = \frac{T_a}{F_t} = \frac{10.72}{21.75} = 0.4929 \text{ in.}^2$$

Let $\dfrac{\pi d^2}{4} = 0.4929$, $d = 0.792$ in. Required $d = 0.792$ in., use $d = 13/16$ in.

(b) Maximum force in rod occurs when live load is an *A* or *D*. Entire live load is taken by one rod.

$$T_a = \frac{0.048(30)}{2} + 20 = 20.72 \text{ kips}$$

$$\text{Required } A_b = \frac{T_a}{F_t} = \frac{20.72}{21.75} = 0.9526 \text{ in.}^2$$

$$\text{Let } \frac{\pi d^2}{4} = 0.9526, \qquad d = 1.10 \text{ in.} \quad \underline{\text{Required } d = 1.10 \text{ in., use } d = 1^1/8 \text{ in.}}$$

---

**3.7-4**

All members are pin-connected, and all loads are applied at the joints; therefore, all members are two-force members (either tension members or compression members). Load combination 4 controls.

$$1.0W = 1.0(10) = 10 \text{ kips}$$

At joint $C$,

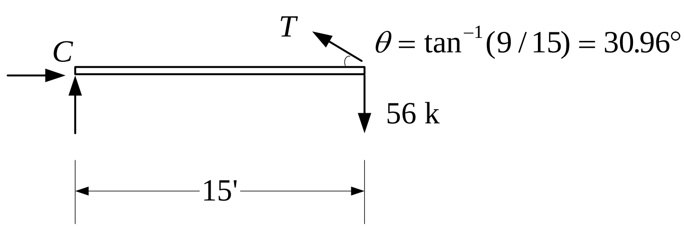
$$\sum F_x = 10 - T_u \cos 26.57° = 0 \qquad \Rightarrow \qquad T_u = 11.18 \text{ kips}$$

$$\text{Required } A_b = \frac{T_u}{0.75(0.75F_u)} = \frac{11.18}{0.75(0.75)(58)} = 0.3427 \text{ in.}^2$$

$$\text{Let } \frac{\pi d^2}{4} = 0.3427 \qquad d = 0.661 \text{ in.} \qquad \underline{\text{Required } d = 0.661 \text{ in., use } d = 7/8 \text{ in.}}$$

**3.7-5**

(a) LRFD: $P_u = 1.2D + 1.6L = 1.6(35) = 56$ kips

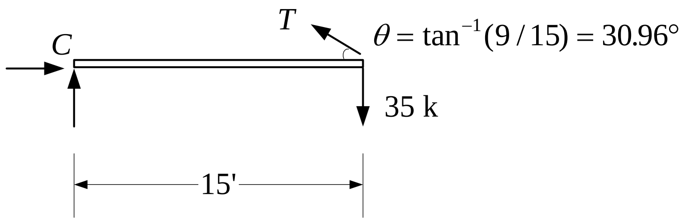



$$\sum M_C = 56(15) - [T\sin(30.96°)](15) = 0, \qquad T = 108.9 \text{ kips}$$

$$\text{Required } A_b = \frac{T}{0.75(0.75F_u)} = \frac{108.9}{0.75(0.75)(58)} = 3.338 \text{ in.}^2$$

$$\text{Let } \frac{\pi d^2}{4} = 3.338, \qquad d = 2.062 \text{ in.} \qquad \underline{\text{Required } d = 2.06 \text{ in. Use } 2\ ^1/_8 \text{ in.}}$$

(b) ASD: $P_a = D + L = 35$ kips




$$\sum M_C = 35(15) - [T\sin(30.96°)](15) = 0, \qquad T = 68.04 \text{ kips}$$

$$F_t = 0.375F_u = 0.375(58) = 21.75 \text{ ksi}$$

$$\text{Required } A_b = \frac{T}{F_t} = \frac{68.04}{21.75} = 3.128 \text{ in.}^2$$

$$\text{Let } \frac{\pi d^2}{4} = 3.128, \qquad d = 2.00 \text{ in.} \qquad \underline{d = 2 \text{ in.}}$$

---

**3.7-6**

From Part 1 of the *Manual*, the inside diameter is $d = 10.0$ in.

$$\text{Volume of water per foot of length} = \frac{\pi d^2}{4} \times 12 = \frac{\pi(10.0)^2}{4} \times 12 = 942.5 \text{ in.}^3$$

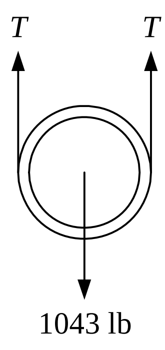
The total weight per foot is

$$\text{weight of water + weight of pipe} = \frac{942.5}{(12)^3}(62.4) + 40.5 = 74.53 \text{ lb/ft}$$

where the density of water has been taken as 62.4 lb/ft$^3$

(a) Treat the load as 100% dead load: $w_u = 1.4(74.53) = 104.3$ lb/ft

The load at each support is 104.3 lb/ft ×10 ft = 1043 lb




$$\Sigma F_y = 2T - 1043 = 0$$

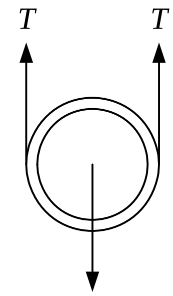
$$T = 521.5 \text{ lb}$$

$$\text{Required } A_b = \frac{T}{0.75(0.75F_u)} = \frac{0.5215}{0.75(0.75)(58)} = 1.598 \times 10^{-2} \text{ in.}^2$$

$$\text{Let } \frac{\pi d^2}{4} = 0.01598, \qquad d = 0.143 \text{ in.}$$

Required $d = 0.143$ in. Use $^5/_8$ in.minimum

(b) The load at each support is 74.53 lb/ft ×10 ft = 745.3 lb.




$$\Sigma F_y = 2T - 745.3 = 0$$

$$T = 372.7 \text{ lb}$$

$$F_t = 0.375F_u = 0.375(58) = 21.75 \text{ ksi}$$

$$\text{Required } A_b = \frac{T}{F_t} = \frac{0.3727}{21.75} = 1.714 \times 10^{-2} \text{ in.}^2$$

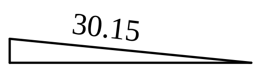
$$\text{Let } \frac{\pi d^2}{4} = 0.01714, \qquad d = 0.148 \text{ in.}$$

Required $d = 0.148$ in. Use $^5/_8$ in.minimum

---

**3.8-1**

Interior joint load:




Snow: $20(10)(12.5) = 2500$ lb

Roofing: $12(10)(30.15/30)(12.5) = 1508$ lb

Purlins: $8.5(12.5) = 106.3$ lb

Truss weight: $1000/3 = 333.3$ lb

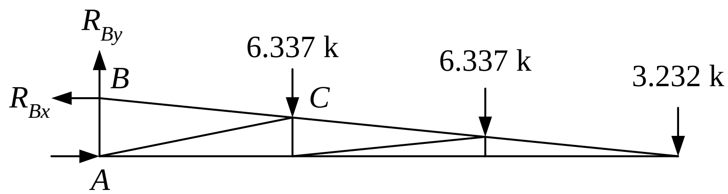
(The assumption that the truss weight is distributed equally to the joints is approximate but is consistent with the approximate nature of the estimate of total truss weight.)

(a) Load combination 3 controls:

$$1.2D + 1.6S = 1.2(1.508 + 0.1063 + 0.3333) + 1.6(2.5) = 6.337 \text{ kips}$$

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

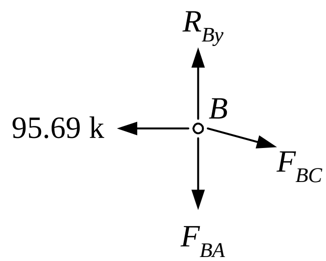
$$1.2D + 1.6S = 1.2\left(\frac{1.508}{2} + 0.1063 + \frac{0.3333}{2}\right) + 1.6\left(\frac{2.5}{2}\right) = 3.232 \text{ kips}$$




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

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

Joint *B*:




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

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

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

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

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

Try WT5 × 11

$$A_g = 3.24 \text{ in.}^2 > 2.79 \text{ in.}^2 \quad \text{(OK)} \qquad r_{\min} = 1.33 \text{ in.} > 0.402 \text{ in.} \quad \text{(OK)}$$

$$U = 1 - \frac{\bar{x}}{\ell} = 1 - \frac{1.07}{11} = 0.9027$$

$$A_e = A_gU = 3.24(0.9027) = 2.93 \text{ in.}^2 > 2.21 \text{in.}^2 \text{ (OK)}$$

<u>Use WT5 × 9.5</u>

(b) Load combination 3 controls:

$$D + S = 1.508 + 0.1063 + 0.3333 + 2.5 = 4.448 \text{ kips}$$

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

$$D + S = \frac{1.508}{2} + 0.1063 + \frac{0.3333}{2} + \frac{2.5}{2} = 2.277 \text{ kips}$$

For a free-body diagram of the entire truss,

$$\sum M_A = 4.448(10) + 4.448(20) + 2.277(30) - R_{Bx}(3) = 0$$

$$R_{Bx} = 67.25 \text{ kips } \leftarrow$$

For a free body of joint $B$:

$$\sum F_x = -67.25 + \frac{30}{30.15} F_{BC} = 0, \qquad F_{BC} = 67.59 \text{ kips}$$

$$\text{Required } A_g = \frac{F_{BC}}{0.6F_y} = \frac{67.59}{0.6(36)} = 3.13 \text{ in.}^2$$

$$\text{Required } A_e = \frac{F_{BC}}{0.5F_u} = \frac{67.59}{0.5(58)} = 2.33 \text{ in.}^2$$

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

Try WT5 × 11

$$A_g = 3.24 \text{ in.}^2 > 3.13 \text{ in.}^2 \quad \text{(OK)} \qquad r_{\min} = 1.33 \text{ in.} > 0.402 \text{ in.} \quad \text{(OK)}$$

$$U = 1 - \frac{\bar{x}}{\ell} = 1 - \frac{1.07}{11} = 0.9027$$

$$A_e = A_g U = 3.24(0.9027) = 2.93 \text{ in.}^2 > 2.33 \text{ in.}^2 \text{ (OK)} \qquad \underline{\text{Use WT5} \times 9.5}$$

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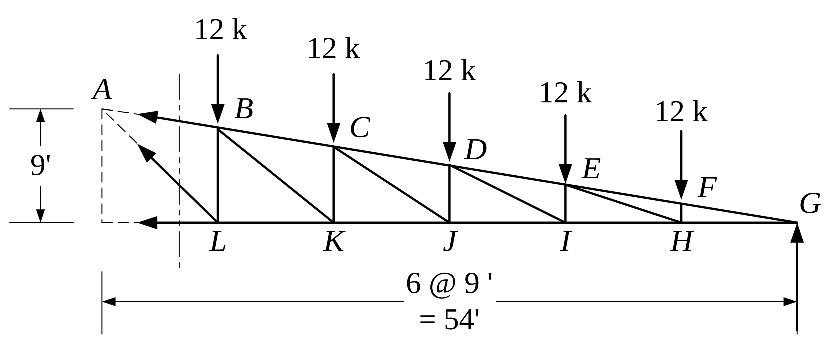
**3.8-2**

The diagonal web members are the tension members, and member $AL$ has the largest force.




Using the method of sections and considering the force in member $AL$ to act at $L$,

$$\sum M_G = 45(F_{AL} \sin 45°) - 12(45 + 36 + 27 + 18 + 9) = 0$$

$$F_{AL} = 50.91 \text{ kips}$$

$$\text{Required } A_g = \frac{F_{AL}}{0.9F_y} = \frac{50.91}{0.9(50)} = 1.13 \text{ in.}^2$$

$$\text{Required } A_e = \frac{F_{AL}}{0.75F_u} = \frac{50.91}{0.75(65)} = 1.04 \text{ in.}^2$$

$$L = \sqrt{(9)^2 + (9)^2} = 12.73 \text{ ft}$$

$$\text{Required } r_{\min} = \frac{L}{300} = \frac{12.73 \times 12}{300} = 0.509 \text{ in.}$$

Try $L3\frac{1}{2} \times 3 \times \frac{1}{4}$

$A_g = 1.58 \text{ in.}^2 > 1.13 \text{ in.}^2$ (OK) $r_{\min} = 0.628 \text{ in.} > 0.509 \text{ in.}$ (OK)

$$A_n = 1.58 - \left(\frac{3}{4} + \frac{1}{8}\right)\left(\frac{1}{4}\right) = 1.36 \text{ in.}^2$$

From AISC Table D3.1, Case 8, use a value of $U = 0.80$

$A_e = A_n U = 1.36(0.80) = 1.09 > \text{in.}^2 > 1.04 \text{ in.}^2$ (OK)

Use $L3\frac{1}{2} \times 3 \times \frac{1}{4}$ for member $AL$

This shape can be used for all of the web tension members. Although each member could be a different size, this would not usually be practical. The following table shows the relatively small difference in requirements for all the web tension members.

| Member | Force (kips) | Req'd $A_g$ (in.$^2$) |
|---|---|---|
| $AL$ | 50.91 | 1.13 |
| $BK$ | 46.86 | 1.04 |
| $CJ$ | 43.27 | 0.962 |
| $DI$ | 40.25 | 0.894 |
| $EH$ | 37.95 | 0.843 |

---

**3.8-3**

Use load combination 3: $1.2D + 1.6S$.

Tributary surface area per joint $= 15\sqrt{(9)^2 + (9/6)^2} = 136.9 \text{ ft}^2$

Roofing: $1.2D = 1.2(12)(136.9) = 1971 \text{ lb}$

Snow: $1.6S = 1.6(18)(9 \times 15) = 3888 \text{ lb}$

Truss weight: $1.2D = 1.2(5000)/12 = 500 \text{ lb}$

Purlin weight: $1.2D = 1.2(33 \times 15) = 594.0 \text{ lb}$

Interior joint: $1971 + 3888 + 500 + 594 = 6953$ lb = 6.95 kips

At peak: $1971 + 3888 + 500 + 2(594) = 7547$ lb = 7.55 kips

Load = 7.55 kips at peak, 6.95 kips elsewhere

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**3.8-4**

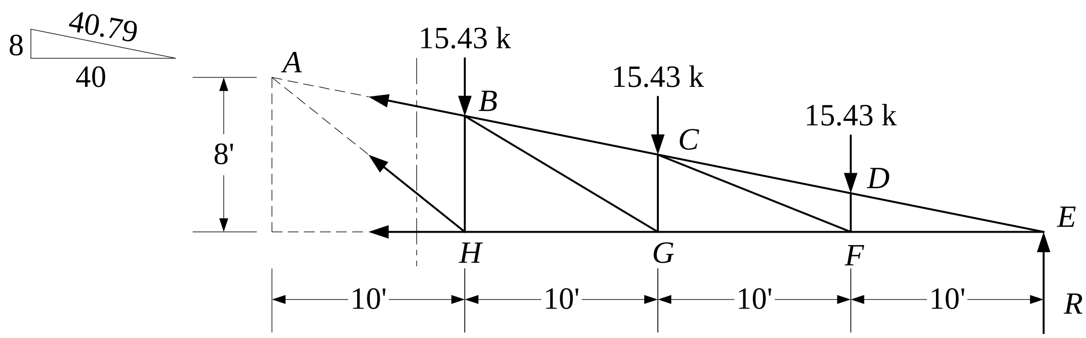
Dead load per truss = $(4+12+6)(40.79\times 2)(25)+5(80)(25) = 54,870$ lb

Snow load per truss = $18(80)(25) = 36,000$ lb

$D = 54870/8 = 6859$ lb/joint, $\quad S = 36000/8 = 4500$ lb/joint

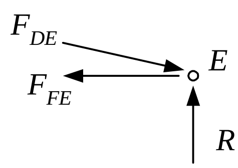
(a) Load combination 3 controls:

Factored joint load = $1.2D + 1.6S = 1.2(6.859)+1.6(4.500) = 15.43$ kips




Bottom chord: Member *FE* has the largest tension force.

Use a free body of joint *E*:




$R$ = Reaction $=7(15.43)/2 = 54.01$ kips

$\sum F_y = 54.01 - \dfrac{8}{40.79}F_{DE} = 0, \qquad F_{DE} = 275.4$ kips

$\sum F_x = 275.4\left(\dfrac{40}{40.79}\right) - F_{FE} = 0, \qquad F_{FE} = 270.1$ kips

Required $A_g = \dfrac{F_{FE}}{0.9F_y} = \dfrac{270.1}{0.9(50)} = 6.002 \text{ in.}^2$

Required $A_e = \dfrac{F_{FE}}{0.75F_u} = \dfrac{270.1}{0.75(65)} = 5.541 \text{ in.}^2$

From $A_e = A_gU$, $\qquad$ Required $A_g = \dfrac{\text{Required } A_e}{U} = \dfrac{5.541}{0.85} = 6.52 \text{ in.}^2$

(This controls the gross area requirement.)

Required $r_{min} = \dfrac{L}{300} = \dfrac{10\times 12}{300} = 0.4$ in.

Try 2L $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$

$A_g = 6.50 \text{ in.}^2 < 6.52 \text{ in.}^2$ (But say OK)

$r_x = 1.05 \text{ in.}, \ r_y = 1.63 \text{ in.}, \quad \therefore \ r_{min} = 1.05 \text{ in.} > 0.4 \text{ in.}$ (OK)

Use 2L $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$ for bottom chord

Web members: Design for the maximum tensile force, which occurs in member *AH*, and use one shape for all tension web members (the diagonal web members). Using the method of sections (see figure), consider the force in member *AH* to act at *H*.

$$\text{Length} = \sqrt{(8)^2 + (10)^2} = 12.81 \text{ ft.}$$

$$\sum M_E = \frac{8}{12.81} F_{AH}(30) - 15.43(10 + 20 + 30) = 0, \qquad F_{AH} = 49.41 \text{ kips}$$

$$\text{Required } A_g = \frac{F_{AH}}{0.9F_y} = \frac{49.41}{0.9(50)} = 1.098 \text{ in.}^2$$

$$\text{Required } A_e = \frac{F_{AH}}{0.75F_u} = \frac{49.41}{0.75(65)} = 1.014 \text{ in.}^2$$

From $A_e = A_g U$, $\quad \text{Required } A_g = \dfrac{\text{Required } A_e}{U} = \dfrac{1.014}{0.85} = 1.19 \text{ in.}^2$ (controls)

$$\text{Required } r_{min} = \frac{L}{300} = \frac{12.81 \times 12}{300} = 0.512 \text{ in.}$$

Try 2L $2 \times 2 \times \frac{3}{16}$

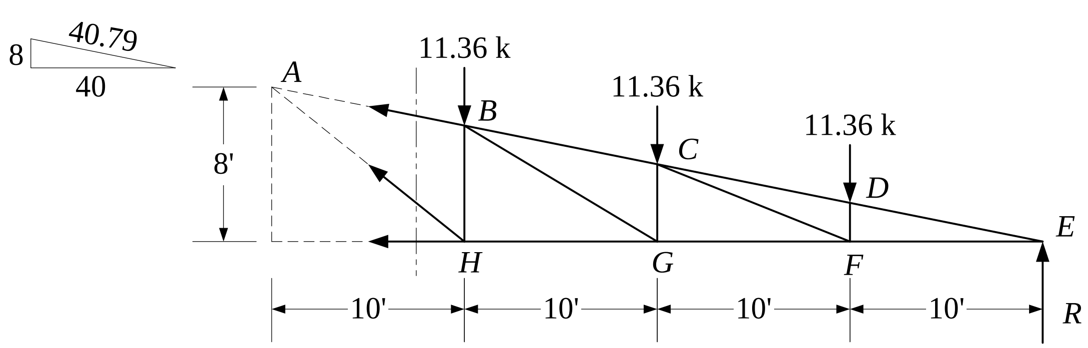
$A_g = 1.44 \text{ in.}^2 > 1.19 \text{ in.}^2$ (OK)

$r_x = 0.612 \text{ in.}, \ r_y = 0.967 \text{ in.}, \quad \therefore \ r_{min} = 0.612 \text{ in.} > 0.512 \text{ in.}$ (OK)

Use 2L $2 \times 2 \times \frac{3}{16}$ for diagonal web members

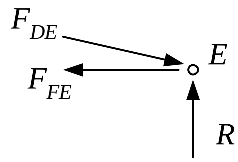
(b) Load combination 3 controls:

Joint load $= D + S = 6859 + 4500 = 11,360$ lb




Bottom chord: Member *FE* has the largest tension force.

Use a free body of joint $E$:




$$R = \text{Reaction} = 7(11.36)/2 = 39.76 \text{ kips}$$

$$\sum F_y = 39.76 - \frac{8}{40.79} F_{DE} = 0, \qquad F_{DE} = 202.7 \text{ kips}$$

$$\sum F_x = 202.7\left(\frac{40}{40.79}\right) - F_{FE} = 0, \qquad F_{FE} = 198.8 \text{ kips}$$

$$\text{Required } A_g = \frac{F_{FE}}{0.6F_y} = \frac{198.8}{0.6(50)} = 6.627 \text{ in.}^2$$

$$\text{Required } A_e = \frac{F_{FE}}{0.5F_u} = \frac{198.8}{0.5(65)} = 6.117 \text{ in.}^2$$

From $A_e = A_g U$, $\quad \text{Required } A_g = \dfrac{\text{Required } A_e}{U} = \dfrac{6.117}{0.85} = 7.20 \text{ in.}^2$

(This controls the gross area requirement.)

$$\text{Required } r_{\min} = \frac{L}{300} = \frac{10 \times 12}{300} = 0.4 \text{ in.}$$

Try 2L $5 \times 5 \times \frac{3}{8}$

$$A_g = 7.30 \text{ in.}^2 > 7.20 \text{ in.}^2 \qquad \text{(OK)}$$

$$r_x = 1.55 \text{ in.}, \; r_y = 2.20 \text{ in.}, \quad \therefore \; r_{\min} = 1.55 \text{ in.} > 0.4 \text{ in.} \qquad \text{(OK)}$$

<u>Use 2L $5 \times 5 \times \frac{3}{8}$ for bottom chord</u>

Web members: Design for the maximum tensile force, which occurs in member *AH*, and use one shape for all tension web members (the diagonal web members). Using the method of sections (see figure), consider the force in member *AH* to act at *H*.

$$\text{Length} = \sqrt{(8)^2 + (10)^2} = 12.81 \text{ ft.}$$

$$\sum M_E = \frac{8}{12.81} F_{AH}(30) - 11.36(10 + 20 + 30) = 0, \qquad F_{AH} = 36.38 \text{ kips}$$

$$\text{Required } A_g = \frac{F_{AH}}{0.6F_y} = \frac{36.38}{0.6(50)} = 1.213 \text{ in.}^2$$

$$\text{Required } A_e = \frac{F_{AH}}{0.5F_u} = \frac{36.38}{0.5(65)} = 1.119 \text{ in.}^2$$

From $A_e = A_g U$, $\quad \text{Required } A_g = \dfrac{\text{Required } A_e}{U} = \dfrac{1.119}{0.85} = 1.32 \text{ in.}^2$ (controls)

Required $r_{\min} = \dfrac{L}{300} = \dfrac{12.81 \times 12}{300} = 0.512$ in.

Try 2L $2 \times 2 \times \frac{3}{16}$

$A_g = 1.44 \text{ in.}^2 > 1.32 \text{ in.}^2$ (OK)

$r_x = 0.612$ in., $r_y = 0.967$ in., $\therefore\ r_{\min} = 0.612 \text{ in.} > 0.512$ in. (OK)

Use 2L $2 \times 2 \times \frac{3}{16}$ for diagonal web members

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**3.8-5**

Use sag rods at midspan of purlins.

Top Chord length $= \sqrt{(40)^2 + (8)^2} = 40.79$ ft

Tributary area $= 40.79(25/2) = 509.9 \text{ ft}^2$

(a) Total vertical load $= 6(509.9) = 3059$ lb

Component parallel to roof $= 3059\left(\dfrac{8}{40.79}\right) = 600.0$ lb

Since the design is for dead load only, use load combination 1:

$P_u = 1.4D = 1.4(600) = 840$ lb

Required $A_g = \dfrac{P_u}{\phi_t(0.75F_u)} = \dfrac{0.840}{0.75(0.75)(58)} = 0.02575 \text{ in.}^2$

Let $\dfrac{\pi d^2}{4} = 0.02575$: $d = 0.181$ in.

Required $d = 0.181$ in., Use $\frac{5}{8}$ in. minimum

(b) $P_a = 600.0$ lb

$F_t = 0.375F_u = 0.375(58 = )21.75$ ksi

Required $A_b = \dfrac{T}{F_t} = \dfrac{0.6000}{21.75} = 0.02759 \text{ in.}^2$

Let $\dfrac{\pi d^2}{4} = 0.02759$, $d = 0.187$ in.

Required $d = 0.187$ in., Use $\frac{5}{8}$ in. minimum