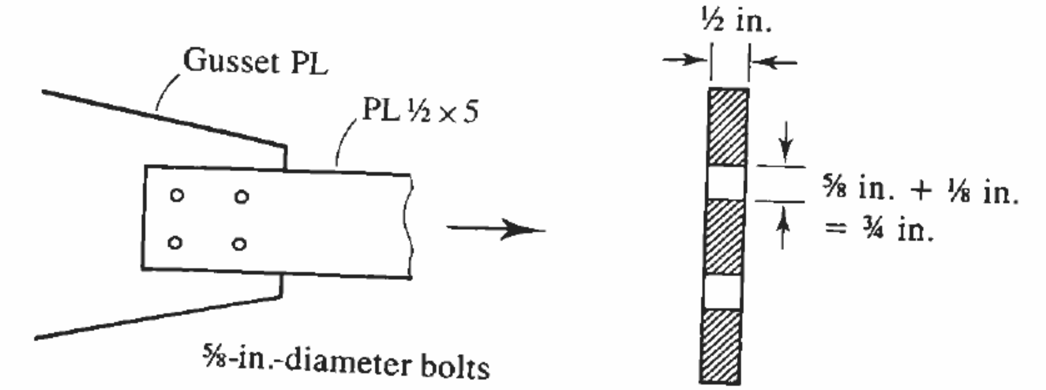
Example 1 (LRFD Steel Design, Third Edition by William T. Segui, page 37-38.)



A ½ × 5 plate of A36 steel is used as a tension member. It is connected to a gusset plate with four ⅝-inch-diameter bolts, as shown in Figure 3.3. Assume that the effective net area A_e equals the actual net area A_n and compute the design strength.

For yielding of the gross section,

$$A_g = 5\left(\frac{1}{2}\right) = 2.5 \text{ in.}^2$$

The *nominal* strength is

$$P_n = F_y A_g = 36(2.5) = 90 \text{ kips}$$

and the *design* strength is

$$\phi_t P_n = 0.90(90) = 81 \text{ kips}$$

For fracture of the net section,

$$\begin{aligned} A_n &= A_g - A_{\text{holes}} \\ &= 2.5 - \left(\frac{1}{2}\right)\left(\frac{3}{4}\right) \times 2 \text{ holes} \\ &= 2.5 - 0.75 = 1.75 \text{ in.}^2 \end{aligned}$$

$A_e = A_n = 1.75 \text{ in.}^2$ (This is true for this example, but A_e does not always equal A_n)

The *nominal* strength is

$$P_n = F_u A_e = 58(1.75) = 101.5 \text{ kips}$$

and the *design* strength is

$$\phi_t P_n = 0.75(101.5) = 76.1 \text{ kips}$$

The smaller value controls.

Design strength = 76.1 kips.

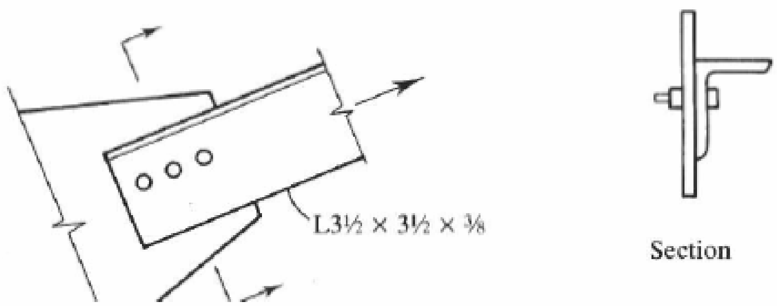
Example 2 (LRFD Steel Design, Third Edition by William T. Segui, page 38-39.)

A single-angle tension member, an L3½ × 3½ × ⅜, is connected to a gusset plate with ⅞-inch-diameter bolts, as shown in Figure 3.4. A36 steel is used. The service loads are 35 kips dead load and 15 kips live load. Investigate this member for compliance with the AISC Specification. Assume that the effective net area is 85% of the computed net area (we cover computation of effective net area in Section 3.3).

When only dead load and live load are present, the only load combinations with a chance of controlling are combinations 1 and 2.

Combination 1: $1.4D = 1.4(35) = 49$ kips

Combination 2: $1.2D + 1.6L = 1.2(35) + 1.6(15) = 66$ kips



The second combination controls; $P_u = 66$ kips.

The design strengths are

Gross section: $A_g = 2.50\text{ in.}^2$ (from Part 1 of the *Manual*)

$$\phi_t P_n = \phi_t F_y A_g = 0.90(36)(2.50) = 81 \text{ kips}$$

Net section:

$$A_n = 2.50 - \left(\frac{3}{8}\right)\left(\frac{7}{8} + \frac{1}{8}\right) = 2.125\text{ in.}^2$$

$$A_e = 0.85A_n = 0.85(2.125) = 1.806\text{ in.}^2 \quad \text{(in } \textit{this} \text{ example)}$$

$$\phi_t P_n = \phi_t F_u A_e = 0.75(58)(1.806) = 78.6 \text{ kips} \quad \text{(controls)}$$

Since $P_u < \phi_t P_n$ (66 kips < 78.6 kips), the member is satisfactory.

Example 3 (LRFD Steel Design, Third Edition by William T. Segui, page 39-40.)

Determine the tensile design strength of the double-angle shape shown in Figure 3.5. The steel is A36, and the holes are for ½-inch-diameter bolts. Assume that $A_e = 0.75A_n$.

Figure 3.5 illustrates the notation for unequal-leg double-angle shapes. The notation LLBB means "long-legs back-to-back," and SLBB indicates "short-legs back-to-back."

When a double-shape section is used, two approaches are possible: (1) Consider a single shape and double everything, or (2) consider two shapes from the outset. (Properties of the double-angle shape are given in Part 1 of the *Manual*.) In this example, we consider one angle and double the result. For one angle, the design strength based on the gross area is

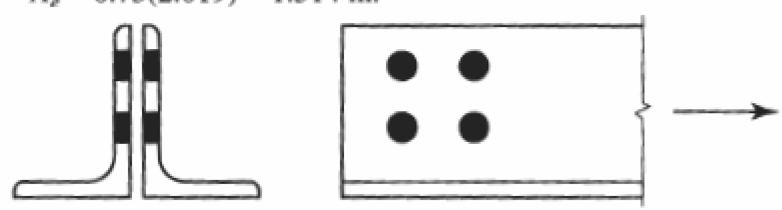
$$\phi_t P_n = \phi_t F_y A_g = 0.90(36)(2.41) = 78.08 \text{ kips}$$

There are two holes in each angle, so the net area of one angle is

$$A_n = 2.41 - \left(\frac{5}{16}\right)\left(\frac{1}{2} + \frac{1}{8}\right) \times 2 = 2.019 \text{ in.}^2$$

The effective net area is

$$A_e = 0.75(2.019) = 1.514 \text{ in.}^2$$



The design strength based on the net area is

$$\phi_t P_n = \phi_t F_u A_e = 0.75(58)(1.514) = 65.86 \text{ kips}$$

Because 65.86 kips < 78.08 kips, fracture of the net section controls, and the design strength for the two angles is $2 \times 65.86 = 132$ kips. ■

Example 4 (LRFD Manual, page 3-5, 3-6)

W-shape tension member design.

Determine the design strength of an ASTM A992 W8×24 with four lines of 3/4-in.-diameter bolts in standard holes, two per flange, as illustrated in Figure 3–1.

$F_y = 50$ ksi $A_g = 7.08$ in.^{2} $\bar{y} = 0.695$ (for WT4×12)
$F_u = 65$ ksi $t_f = 0.400$ in.
$r_y = 1.61$ in.

a) Assume the holes are located at the member end and the connection length is 9 in. Also, calculate at what length this tension member would cease to satisfy the slenderness limitation in LRFD Specification Section B7.
b) Assume the holes are located away from the member end so that shear-lag effects do not apply ($U = 1$).

For tension yielding, per LRFD Specification Section D1(a),

$$\begin{aligned}\phi_t P_n &= \phi_t F_y A_g \\ &= 0.9(50\text{ ksi})(7.08\text{ in.}^2) \\ &= 319\text{ kips}\end{aligned}$$

For tension rupture, per LRFD Specification Section D1(b),

$$\begin{aligned}U &= 1 - \frac{\bar{x}}{\ell} \le 0.9 \\ &= 1 - \frac{0.695\text{ in.}}{9\text{ in.}} \le 0.9 \\ &= 0.923 \le 0.9 \\ &= 0.9\end{aligned}$$

$$\begin{aligned}A_n &= A_g - 4(d_h + 1/16\text{ in.})t_f \\ &= 7.08\text{ in.}^2 - 4(13/16\text{ in.} + 1/16\text{ in.})(0.400\text{ in.}) \\ &= 5.68\text{ in.}^2\end{aligned}$$

$$\begin{aligned}A_e &= UA_n \\ &= 0.9(5.68\text{ in.}^2) \\ &= 5.11\text{ in.}^2\end{aligned}$$

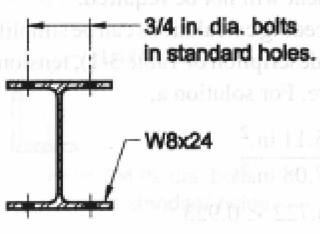


Fig. 3–1. Illustration for Example 3.1.

$$\phi_t P_n = \phi_t F_u A_e$$
$$= 0.75(65 \text{ ksi})(5.11 \text{ in.}^2)$$
$$= 249 \text{ kips}$$

Thus, the W8×24 tension member design strength is controlled by the tension rupture limit-state, where

$$\phi_t P_n = 249 \text{ kips}$$

Per LRFD Specification Section B7,

$$L_{\max} = 300r$$
$$= \frac{300\ (1.61 \text{ in.})}{12 \text{ in./ft}}$$
$$= 40.3 \text{ ft}$$

Thus, the W8×24 tension member satisfies the slenderness requirements up to a 40.3-ft length.