Chapter Two: Tension Members

2.1 Tension members are structural elements that are subjected to axial tensile forces, as shown in the figure below

They are used in various types of structures and include truss members, bracing for buildings and bridges, cables in suspended roof systems, and cables in suspension and cable-stayed bridges.

Steel Truss

Cable-stayed bridge

Suspension bridge

The stress in an axially loaded tension member is given by

$$f = \frac{P}{A}$$

Where P is the magnitude of the load and A is the cross-sectional area (the area normal to the load).

The most common built-up configuration is probably the double-angle section, shown in Figure 2.1, along with other typical cross sections.

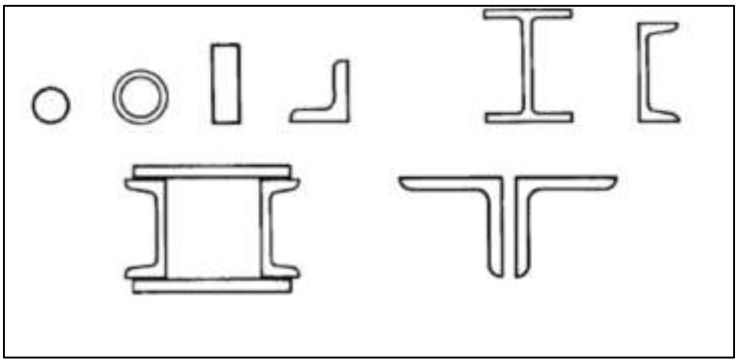


Figure 2.1

Tension members are frequently connected at their ends with bolts, as illustrated in Figure 2.2.

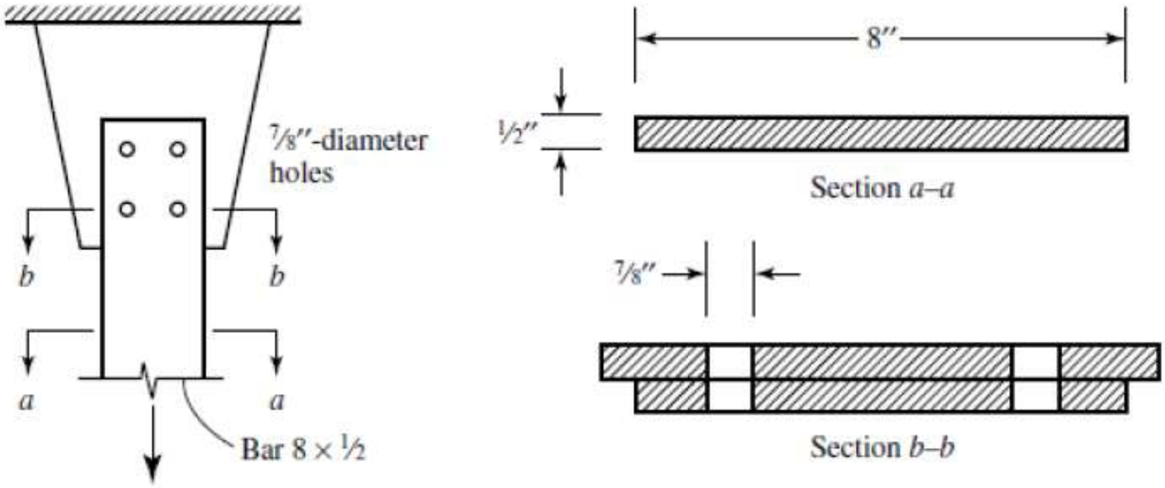


Figure 2.2

The tension member shown, a 1⁄2 × 8 plates, is connected to a gusset plate, which is a connection element whose purpose is to transfer the load from the member to a support or to another member. The area of the bar at section a–a is (1⁄2)(8) = 4 in^2, but the area at section b–b is only 4 – (2)(1⁄2)(7⁄8) = 3.13 in2 and will be more highly stressed. This reduced area is referred to as the net area, or net section, and the unreduced area is the gross area.

2.2 Tensile Strength

A tension member can fail by reaching one of two limit states: excessive deformation or fracture. To prevent excessive deformation, initiated by yielding, the load on the gross section must be small enough that the stress on the gross section is less than the yield stress F_y. To prevent fracture, the stress on the net section must be less than the tensile strength Fu. In each case, the stress P/A must be less than a limiting stress F or

$$\frac{P}{A} < F$$

Thus, the load P must be less than FA, or

$$P < FA$$

The *nominal* strength in yielding is

$$P_n = F_y A_g$$

and the nominal strength in fracture is

$$P_n = F_u A_e$$

Where A_e is the effective net area, which may be equal to either the net area or, in some cases, a smaller area.

Although yielding will first occur on the net cross section, the deformation within the length of the connection will generally be smaller than the deformation in the remainder of the tension member.

LRFD: In load and resistance factor design, the factored tensile load is compared to the design strength. The design strength is the resistance factor times the nominal strength.

Ru = φRn

Can be written for tension members as $P_u \leq \varphi_t P_n$ where P_u is the governing combination of factored loads. The resistance factor ft is smaller for fracture than for yielding, reflecting the more serious nature of fracture.

For yielding, $\varphi_t = 0.90$

For fracture, $\varphi_t = 0.75$

Because there are two limit states, both of the following conditions must be satisfied:

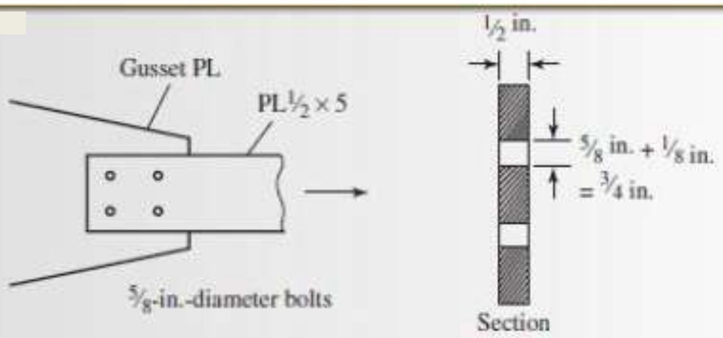
$P_u \leq 0.90\ F_y\ A_g$

$P_u \leq 0.75\ F_u\ A_e$

Example 2.1:

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 (we cover computation of effective net area in Section 3.3).

a. What is the design strength for LRFD?



SOLUTION For yielding of the gross section,

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

and the nominal strength is

$$P_n = F_y A_g = 36(2.5) = 90.0 \text{ kips}$$

For fracture of the net section,

$$A_n = A_g - A_{holes} = 2.5 - (\tfrac{1}{2})(\tfrac{3}{4}) \times 2 \text{ holes}$$
$$= 2.5 - 0.75 = 1.75 \text{ in.}^2$$

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

The nominal strength is

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

a. The design strength based on yielding is

$$\phi_t P_n = 0.90(90) = 81.0 \text{ kips}$$

The design strength based on fracture is

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

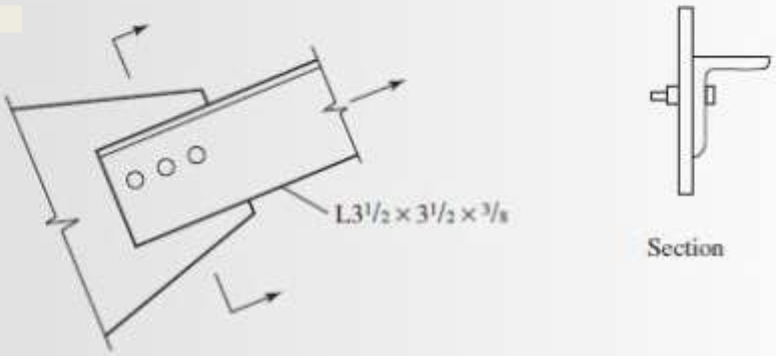
ANSWER The design strength for LRFD is the smaller value: $\phi_t P_n = 76.1$ kips.

EXAMPLE 3.2

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.

a. Use LRFD.

b. Use ASD.



SOLUTION First, compute the nominal strengths.

Gross section:

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

$P_n = F_y A_g = 36(2.50) = 90 \text{ kips}$

Net section:

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

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

$P_n = F_u A_e = 58(1.806) = 104.7 \text{ kips}$

a. The design strength based on yielding is

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

The design strength based on fracture is

$\phi_t P_n = 0.75(104.7) = 78.5 \text{ kips}$

The design strength is the smaller value: $\phi_t P_n = 78.5$ kips

Factored load:

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: $1.4D = 1.4(35) = 49 \text{ kips}$

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

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

(When only dead load and live load are present, combination 2 will always control when the dead load is less than eight times the live load. In future examples, we will not check combination 1 [$1.4D$] when it obviously does not control.)

ANSWER Since $P_u < \phi_t P_n$, (66 kips < 78.5 kips), the member is satisfactory.