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### Chapter 3.1 – Introduction

- **Tension members** are structural elements that are subjected to axial tensile forces.




- 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.

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### Chapter 3.1 – Introduction

- **Tension members** are structural elements that are subjected to axial tensile forces.

Hunter Harrison Bridge



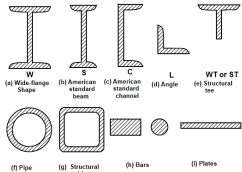
Kings Crossing Bridge



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### Chapter 3.1 – Introduction

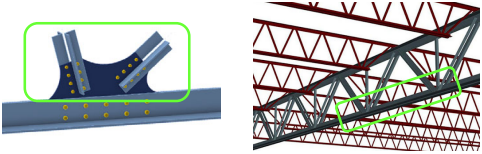
- Any **cross-sectional configuration** may be used, because the primary determinant of the strength of a tension member is the **cross-sectional area**.
- Circular rods and rolled angle shapes are frequently used.
- **Built-up shapes**, either from plates, rolled shapes, or a combination of plates and rolled shapes, are sometimes used when large loads must be resisted.



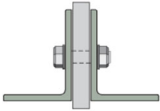
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### Chapter 3.1 – Introduction

- The most common **built-up** configuration is probably the **double-angle section**.



- Tables of properties of various combinations of angles are included in the **AISC Steel Construction Manual**.



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### Chapter 3.1 – Introduction

- 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 **stress** as given by this equation is exact, provided that the cross section under consideration is **not adjacent** to the point of application of the load, where the **distribution of stress is not uniform**.

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### Chapter 3.1 – Introduction

- The **stress** in an axially loaded tension member is given by

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

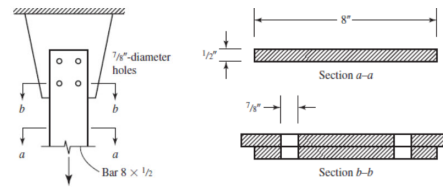
where **P** is the magnitude of the load, and **A** is the cross-sectional area (the area normal to the load)

- If the **cross-sectional area varies** along its length, the stress is a function of the section under consideration.
- The presence of **holes** in a member will influence the **stress** at a cross-section through the hole or holes.

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### Chapter 3.1 – Introduction

- Tension members are frequently connected at their ends with bolts.

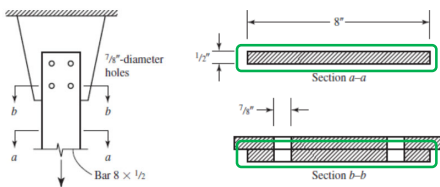


- The 8 x 1/2 plate is connected to a gusset plate, which is a connection element whose purpose is to **transfer the load** from the member to a support or to another member.

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### Chapter 3.1 – Introduction

- Tension members are frequently connected at their ends with bolts.

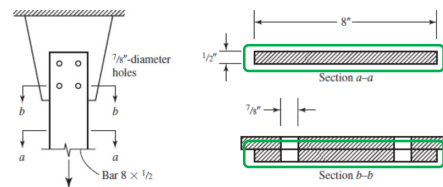


- The area of the plate at section **a-a** is (1/2 in.)(8 in.) = 4 in.<sup>2</sup>
- The area at section **b-b** is 4 in.<sup>2</sup> - (2)(1/2 in.)(7/8 in.) = 3.13 in.<sup>2</sup>

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### Chapter 3.1 – Introduction

- Tension members are frequently connected at their ends with bolts.



- This reduced area at **b-b** is referred to as the **net area**, or **net section**.
- The unreduced area at **a-a** is the **gross area**.

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### Chapter 3.1 – Introduction

- The typical **design problem** is to select a member with sufficient cross-sectional area to resist the loads.
- A closely related problem is that of the **analysis** of a given member, wherein the strength is computed and compared with the load.
- In general, analysis is a direct procedure, but design is an **iterative process** and may require some **trial and error**.
- Tension members are covered in **Chapter D of the Specification (16.1-32)**.
- Requirements that are common with other types of members are covered in **Chapter B (16.1-15)**.

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### Chapter 3.2 – Tensile Strength

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

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

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### Chapter 3.2 – Tensile Strength

- The nominal strength in **yielding** is:

$$P_n < F_y A_g$$

- The nominal strength in **rupture** is:

$$P_n < F_u A_e$$

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

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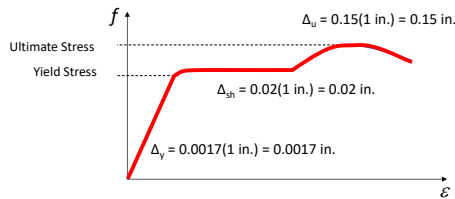
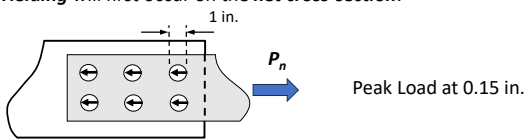
### Chapter 3.2 – Tensile Strength

- **Yielding** will first occur on the **net cross-section**.
- The deformation within the length of the **connection** will generally be less than the deformation in the **remainder of the tension member**.
- The reason is that the **net section** exists over a relatively small length of the member, and the **total elongation** is a product of the **length** and the **strain** (a function of the stress).

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### Chapter 3.2 – Tensile Strength

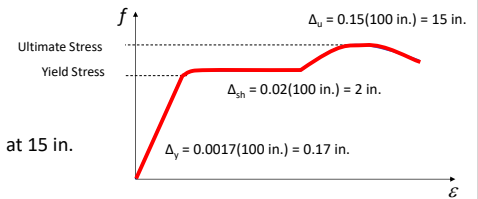
- **Yielding** will first occur on the **net cross-section**.



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### Chapter 3.2 – Tensile Strength

- Most of the members will have an **unreduced cross-section**, so attainment of the **yield stress on the gross area** will result in larger total elongation.
- Let's assume a length of 100 in. instead of 1 in. for the bolts.

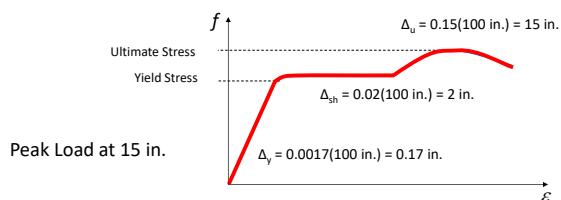


Peak Load at 15 in.

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### Chapter 3.2 – Tensile Strength

- Most of the members will have an **unreduced cross-section**, so attainment of the **yield stress on the gross area** will result in larger total elongation.
- This **larger deformation**, rather than the first yield, constitutes the **limit state**.

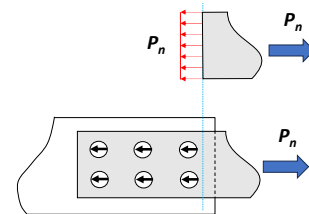


Peak Load at 15 in.

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### Chapter 3.2 – Tensile Strength

- Consider **rupture** on the **net cross-section** of a 6-bolt connection



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### Chapter 3.2 – Tensile Strength

➤ Consider *rupture* on the *net cross-section* of a 6-bolt connection

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### Chapter 3.2 – Tensile Strength

➤ Consider *rupture* on the *net cross-section* of a 6-bolt connection

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### Chapter 3.2 – Tensile Strength

➤ **LRFD:** the factored tensile load is compared to the design strength.

➤ The design strength is the **resistance factor** times the **nominal strength**.

$$P_u \leq \phi P_n$$

where  $P_u$  is the governing combination of factored loads, and  $P_n$  is the nominal tensile strength.

➤ The resistance factor  $\phi_t$  is smaller for rupture than for yielding, reflecting the more serious nature of rupture.

$\phi_t = 0.90$  for yielding       $\phi_t = 0.75$  for rupture

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### Chapter 3.2 – Tensile Strength

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

$$P_u \leq 0.90 F_y A_g \quad P_u \leq 0.75 F_u A_e$$

➤ The smaller of these is the design strength of the member.

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### Chapter 3.2 – Tensile Strength

➤ **ASD:** In allowable strength design, the total **service load** is compared to the **allowable strength** (allowable load):

$$P_o \leq \frac{P_n}{\Omega_t}$$

where  $P_o$  is the required strength (applied load), and  $P_n/\Omega_t$  is the allowable strength.

➤ For **yielding of the gross section**, the safety factor  $\Omega_t$  is 1.67, and the allowable load is

$$\frac{P_n}{\Omega_t} = \frac{F_y A_g}{1.67} = 0.6 F_y A_g$$

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### Chapter 3.2 – Tensile Strength

- **ASD:** In allowable strength design, the total **service load** is compared to the **allowable strength** (allowable load):

$$P_o \leq \frac{P_n}{\Omega_t}$$

where  $P_o$  is the required strength (applied load), and  $P_n/\Omega_t$  is the allowable strength.

- For **rupture of the net section**, the safety factor is 2.00 and the allowable load is

$$\frac{P_n}{\Omega_t} = \frac{F_u A_e}{2.00} = 0.5F_u A_e$$

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### Chapter 3.2 – Tensile Strength

- **ASD:** Alternatively, the service load stress can be compared to the allowable stress. This can be expressed as:

$$f_t \leq F_t$$

where  $f_t$  is the applied stress and  $F_t$  is the allowable stress.

- For the yielding of the gross section,

$$f_t = \frac{P_o}{A_o} \quad F_t = \frac{P_n/\Omega_t}{A_o} = \frac{0.6F_y A_g}{A_o} = 0.6F_y$$

- For the rupture of the net section,

$$f_t = \frac{P_o}{A_e} \quad F_t = \frac{P_n/\Omega_t}{A_e} = \frac{0.5F_u A_e}{A_e} = 0.5F_u$$

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### Chapter 3.2 – Tensile Strength

- You can find values of  $F_y$  and  $F_u$  for various structural steel shapes in **Table 2-4 (2-56)** in the **Manual**.

- All of the steels that are available for various **hot-rolled shapes** are indicated by shaded areas.

- The **black** areas correspond to preferred materials, and the **gray** areas represent other steels that are available.

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### Chapter 3.2 – Tensile Strength

- You can find values of  $F_y$  and  $F_u$  for various structural steel shapes in **Table 2-4 (2-56)** in the **Manual**.

- Under the **W** heading, **A992** is the preferred material.
- Other materials are available, but usually at a higher cost.

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### Chapter 3.2 – Tensile Strength

- Values of  $F_y$  and  $F_u$  for plates are given in **Table 2-5 (2-58)** for fasteners in **Table 2-6 (2-60)**

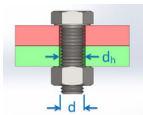
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### Chapter 3.2 – Tensile Strength

- The diameter of a bolt hole or the width of a slotted bolt hole **must be made slightly larger** than the bolt diameter.
- For bolts less than 1-in.-diameter, the additional amount for standard holes (i.e., not oversized) is  $1/16$  inch.
- For bolts greater than 1 inch in diameter, the additional amount for standard holes (i.e., not oversized) is  $1/8$  inch.

**TABLE J3.3**  
**Nominal Hole Dimensions, in.**

Bolt Diameter	Hole Dimensions			
	Standard (Dia.)	Oversize (Dia.)	Short-Slot (Width x Length)	Long-Slot (Width x Length)
3/8	3/8	3/8	3/8 x 1/8	3/8 x 1/4
1/2	1/2	1/2	1/2 x 1/8	1/2 x 1/4
5/8	5/8	5/8	5/8 x 1	5/8 x 1/4
3/4	3/4	3/4	3/4 x 1 1/2	3/4 x 1/4
7/8	7/8	7/8	7/8 x 1 1/2	7/8 x 1/4
1	1	1	1 x 1 1/2	1 x 1/4
>1 1/8	d + 1/8	d + 1/8	(d + 1/8) x (d + 1/8)	(d + 1/8) x 2.5d



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## Chapter 3.2 – Tensile Strength

- Details related to standard, oversized, and slotted holes can be found in **Section J3.3** of the AISC Specification “Size and Use of Holes” in Chapter J “Design of Connections.”
- AISC Section B4.3.3b** requires an additional  $\frac{1}{16}$  inch to account for possible roughness around the edge of the hole.
- The effective hole diameter is therefore

$$d_{hole} = d_{bolt} + \frac{1}{16} + \frac{1}{16} = d_{bolt} + \frac{1}{8} \quad \text{for } d_{bolt} < 1 \text{ in.}$$

$$d_{hole} = d_{bolt} + \frac{1}{8} + \frac{1}{16} = d_{bolt} + \frac{3}{16} \quad \text{for } d_{bolt} \geq 1 \text{ in.}$$

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## Chapter 3.2 – Tensile Strength

**CHAPTER D**  
**DESIGN OF MEMBERS FOR TENSION**

This chapter applies to members subjected to axial tension.

The chapter is organized as follows:

- D1. Slenderness Limitations
- D2. Tensile Strength
- D3. Effective Net Area
- D4. Bolted Members
- D5. Pin-Connected Members
- D6. Eyebars

**Use Note:** For cases not included in this chapter, the following sections apply:

- B3.1 Members subjected to fatigue
- Chapter H Members subjected to combined axial tension and flexure
- J1 Threaded rods
- J4.1 Connecting elements in tension
- J4.3 Block shear rupture strength at end connections of tension members

**D1. SLENDERNESS LIMITATIONS**

There is no maximum slenderness limit for members in tension.

**Use Note:** For members designed on the basis of tension, the slenderness ratio of the member or fabricated section is the fabricated length of the member divided by the least radius of gyration of the section—probably should not exceed 300. This restriction does not apply to rods.

**D2. TENSILE STRENGTH**

The design tensile strength,  $\phi_t P_n$ , and the allowable tensile strength,  $P_n/1.67$ , of tension members shall be the lesser value obtained according to the limit states of tensile yielding in the gross section and tensile rupture in the net section.

(a) For tensile yielding  $\phi_t P_n = F_y A_g$  (D2-1)  
 $\phi_t = 0.90$  (LRFD)  $\phi_t = 1.67$  (ASD)

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## Chapter 3.2 – Tensile Strength

**TABLE D3.1**  
**Shear Lag Factors for Connections to Tension Members**

Case	Description of Element	Shear Lag Factor, U	Examples
1	1. All tension members welded to the tension flange in a moment-resisting joint, in which the longitudinal axis of the member is perpendicular to the longitudinal axis of the flange.	U = 1.0	—
2	2. All tension members, welded (E80), which are welded to the tension flange of a moment-resisting joint, in which the longitudinal axis of the member is perpendicular to the longitudinal axis of the flange. Cases 1 and 2 are permitted for the design of plates and angles if permitted by Section C6.1.	$U = 1 - \frac{e}{l}$	—
3	3. All tension members welded to the tension flange in a moment-resisting joint, in which the longitudinal axis of the member is parallel to the longitudinal axis of the flange.	U = 1.0 and U = 0.85, depending on the connection geometry.	—
4	4. Plates, angles, channels with wide flanges, and other shapes welded to the tension flange of a moment-resisting joint, in which the longitudinal axis of the member is perpendicular to the longitudinal axis of the flange. Cases 1 and 2 are permitted for the design of plates and angles if permitted by Section C6.1.	$U = 1 - \frac{e}{l}$	—
5	5. Plates, angles, channels with wide flanges, and other shapes welded to the tension flange of a moment-resisting joint, in which the longitudinal axis of the member is parallel to the longitudinal axis of the flange.	U = 1.0 and U = 0.85, depending on the connection geometry.	—
6	6. Plates, angles, channels with wide flanges, and other shapes welded to the tension flange of a moment-resisting joint, in which the longitudinal axis of the member is perpendicular to the longitudinal axis of the flange. Cases 1 and 2 are permitted for the design of plates and angles if permitted by Section C6.1.	$U = 1 - \frac{e}{l}$	—
7	7. Plates, angles, channels with wide flanges, and other shapes welded to the tension flange of a moment-resisting joint, in which the longitudinal axis of the member is parallel to the longitudinal axis of the flange.	U = 1.0 and U = 0.85, depending on the connection geometry.	—

**TABLE D3.1 (continued)**  
**Shear Lag Factors for Connections to Tension Members**

Case	Description of Element	Shear Lag Factor, U	Examples
8	8. All tension members welded to the tension flange of a moment-resisting joint, in which the longitudinal axis of the member is perpendicular to the longitudinal axis of the flange. Cases 1 and 2 are permitted for the design of plates and angles if permitted by Section C6.1.	$U = 1 - \frac{e}{l}$	—
9	9. All tension members welded to the tension flange of a moment-resisting joint, in which the longitudinal axis of the member is parallel to the longitudinal axis of the flange.	U = 1.0 and U = 0.85, depending on the connection geometry.	—

**D3. PIN-CONNECTED MEMBERS**

**1. Tensile Strength**

The design tensile strength,  $\phi_t P_n$ , and the allowable tensile strength,  $P_n/1.67$ , of pin-connected members, shall be the lesser value determined according to the limit states of tensile yielding, shear rupture, bearing, and yielding.

(a) For tensile yielding  $\phi_t P_n = F_y A_g$  (D3-1)  
 $\phi_t = 0.90$  (LRFD)  $\phi_t = 1.67$  (ASD)

(b) For shear rupture  $\phi_t P_n = 0.65 F_u A_n$  (D3-2)  
 $\phi_t = 0.75$  (LRFD)  $\phi_t = 2.00$  (ASD)

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## Chapter 3.2 – Tensile Strength

**2. Dimensional Requirements**

(a) The pin hole shall be located midway between the edges of the member in the direction normal to the axial force.

(b) When the pin is expected to provide for relative movement between connected parts, which shall fall back the distance of the pin hole shall not be more than 1/16 in. (1 mm) greater than the diameter of the pin for pins less than 1 in. (25 mm) in diameter and not more than 1/8 in. (3 mm) greater than the diameter of the pin for pins of 1 in. (25 mm) or diameter or greater.

(c) The width of the plate at the pin hole shall not be less than  $2d_p + d$  and the minimum extension,  $a$ , beyond the bearing end of the pin hole, parallel to the axis of the member, shall not be less than  $1.5d_p$ .

(d) The corners beyond the pin hole are permitted to be cut at  $45^\circ$  to the axis of the member, provided the cut does not pass the pin hole, on a plane perpendicular to the cut, is not less than that required beyond the pin hole parallel to the axis of the member.

**D6. EYEBARS**

**1. Tensile Strength**

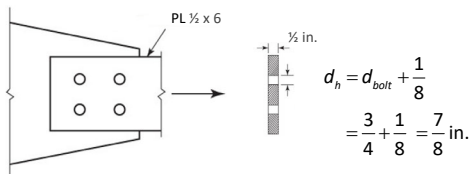
The available tensile strength of eyebars shall be determined in accordance with Section D2, with  $A_n$  taken as the gross area of the eyebars body.

For calculation purposes, the width of the body of the eybar shall not exceed eight times its thickness.

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## Chapter 3.2 – Tensile Strength

- Example 3-1:** A  $\frac{1}{2}$  x 6 plate of A36 steel is used in tension.
- It is connected to a gusset plate with four  $\frac{3}{4}$ -inch bolts.
- Assume that the effective net area  $A_e$  equals the actual net  $A_n$  area (we cover the computation of effective in section 3.3).



- What is the design strength for **LRFD**?

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## Chapter 3.2 – Tensile Strength

- Example 3-1:** A  $\frac{1}{2}$  x 6 plate of A36 steel is used in tension.

For A36 steel,  $F_y = 36$  ksi  
 $F_u = 58$  ksi

**Table 3-4**  
**Applicable ASTM Specifications for Various Structural Shapes**

Steel Type	ASTM Spec	$F_y$ (ksi)	$F_u$ (ksi)	Min. Thickness (in.)	Max. Thickness (in.)	Max. Length (ft)	Max. Weight (lb/ft)
Flat	A36	36	58	1/8	1/2	120	120
	A572	50	68	1/8	1/2	120	120
	A572	50	68	1/8	1/2	120	120
	A572	50	68	1/8	1/2	120	120
Channel	A36	36	58	1/8	1/2	120	120
	A572	50	68	1/8	1/2	120	120
	A572	50	68	1/8	1/2	120	120
	A572	50	68	1/8	1/2	120	120
I-Beam	A36	36	58	1/8	1/2	120	120
	A572	50	68	1/8	1/2	120	120
	A572	50	68	1/8	1/2	120	120
	A572	50	68	1/8	1/2	120	120

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### Chapter 3.2 – Tensile Strength

➤ **Example 3-1:** A ½ x 6 plate of **A36** steel is used in tension.

For the **yielding** of the gross section:  $A_g = 6 \text{ in} \left( \frac{1}{2} \text{ in} \right) = 3 \text{ in}^2$

The nominal strength is:  $P_n = F_y A_g = (36 \text{ ksi})(3 \text{ in}^2) = 108 \text{ k}$

For rupture of the net section:  $A_n = A_g - A_{\text{holes}}$   
 $= 3 \text{ in}^2 - \left( \frac{1}{2} \text{ in} \right) \left( \frac{7}{8} \text{ in} \right) \times 2 \text{ holes}$   
 $= 2.125 \text{ in}^2$

For this example,  $A_e = A_n$

The nominal strength for **rupture** is  $P_n = F_u A_e = (58 \text{ ksi})(2.125 \text{ in}^2)$   
 $= 123.25 \text{ k}$

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### Chapter 3.2 – Tensile Strength

➤ **Example 3-1:** A ½ x 6 plate of **A36** steel is used in tension.

The design strength based on **yielding** is  $\phi_t P_n = 0.90(108 \text{ k})$   
 $= 97.20 \text{ k}$

The design strength based on **rupture** is  $\phi_t P_n = 0.75(123.3 \text{ k})$   
 $= 92.44 \text{ k}$

The design strength for **LRFD** is the smaller value:  $\phi_t P_n = 92.44 \text{ k}$

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### Chapter 3.2 – Tensile Strength

➤ **Example 3-1:** A ½ x 6 plate of **A36** steel is used in tension.

Solution using **ASD**:

The design strength based on **yielding** is  $\frac{P_n}{\Omega_t} = \frac{108 \text{ k}}{1.67} = 64.67 \text{ k}$

The design strength based on **rupture** is  $\frac{P_n}{\Omega_t} = \frac{123.25 \text{ k}}{2.00} = 61.63 \text{ k}$

The allowable service load is the smaller value:  $\frac{P_n}{\Omega_t} = 61.63 \text{ k}$

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### Chapter 3.2 – Tensile Strength

➤ **Example 3-1:** A ½ x 6 plate of **A36** steel is used in tension.

Alternative solution using allowable stress:

For **yielding**,  $F_t = 0.6F_y = 0.6(36 \text{ ksi}) = 21.60 \text{ ksi}$

The allowable load is:  $F_t A_g = 21.6 \text{ ksi}(3 \text{ in}^2) = 64.80 \text{ k}$

For **rupture**,  $F_t = 0.5F_u = 0.5(58 \text{ ksi}) = 29.00 \text{ ksi}$

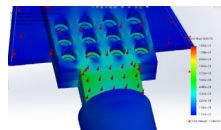
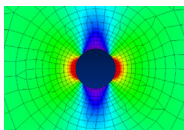
The allowable load is:  $F_t A_e = 29 \text{ ksi}(2.125 \text{ in}^2) = 61.63 \text{ k}$

The allowable service load is the smaller value:  $F_t A_e = 61.63 \text{ k}$

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### Chapter 3.2 – Tensile Strength

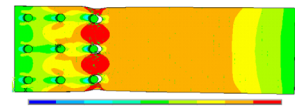
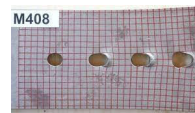
- The effects of **stress concentrations at holes** appear to have been overlooked.
- Stresses at holes can be as high as **three times** the average stress on the net section, and at fillets of rolled shapes, they can be more than **twice the average**.
- Because of the ductile nature of structural steel, the usual design practice is to **neglect such localized overstress**.



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### Chapter 3.2 – Tensile Strength

- After **yielding begins at a point** of stress concentration, additional stress is transferred to adjacent areas of the cross-section.
- This stress redistribution is responsible for the **“forgiving”** nature of structural steel.
- Its ductility permits the initially yielded zone to deform without rupture as the stress on the remainder of the cross section continues to increase.



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### Chapter 3.2 – Tensile Strength

➤ **Example 3-2:** Use *LRFD* and compute the nominal strengths

The gross section from Part 1 of the *Manual*,  $A_g = 3.75 \text{ in}^2$

The nominal strength is:  $P_n = F_y A_g = (50 \text{ ksi})(3.75 \text{ in}^2) = 187.5 \text{ k}$

The net section:  $A_n = A_g - A_{\text{holes}}$   
 $= 3.75 \text{ in}^2 - (\frac{1}{2} \text{ in})(\frac{3}{4} + \frac{1}{8}) \text{ in} = 3.313 \text{ in}^2$

The effective area:  $A_e = 0.90 A_n = 0.90(3.313 \text{ in}^2) = 2.98 \text{ in}^2$

The nominal strength for rupture is:  $P_n = F_u A_e = (65 \text{ ksi})(2.98 \text{ in}^2) = 193.81 \text{ k}$

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### Chapter 3.2 – Tensile Strength

➤ **Example 3-2:** Use *LRFD* and compute the nominal strengths

The design strength based on *yielding* is:  $\phi_t P_n = 0.90(187.5 \text{ k}) = 168.8 \text{ k}$

The design strength based on *rupture* is:  $\phi_t P_n = 0.75(193.81 \text{ k}) = 145.36 \text{ k}$

The design strength for *LRFD* is the smaller value:

$$\phi_t P_n = 145.36 \text{ k}$$

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### Chapter 3.2 – Tensile Strength

➤ **Example 3-2:** Use *LRFD* and compute the nominal strengths

**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.

The design strength based on yielding is:

1.  $1.4D = 1.4(35 \text{ k}) = 49 \text{ k}$
2.  $1.2D + 1.6L + (0.5L_r \text{ or } 0.3S \text{ or } 0.5R)$   
 $= 1.2(35 \text{ k}) + 1.6(15 \text{ k}) = 66 \text{ k}$

Since,  $P_u < \phi_t P_n \Rightarrow 66 \text{ k} < 145.36 \text{ k}$  the member is *satisfactory*.

O.K.

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### Chapter 3.2 – Tensile Strength

➤ **Example 3-2:** Use *ASD* and compute the nominal strengths.

For the gross area, the allowable strength is:

$$\frac{P_n}{\Omega_t} = \frac{187.5 \text{ k}}{1.67} = 112.3 \text{ k}$$

For the net area, the allowable strength is:

$$\frac{P_n}{\Omega_t} = \frac{193.81 \text{ k}}{2.00} = 96.89 \text{ k}$$

The allowable strength for *ASD* is the smaller value:

$$\frac{P_n}{\Omega_t} = 96.91 \text{ k}$$

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### Chapter 3.2 – Tensile Strength

➤ **Example 3-2:** Use *ASD* and compute the nominal strengths.

When the only loads are dead load and live load, *ASD* load combination 2 will always control:

2.  $D + L$   
 $= 35 \text{ k} + 15 \text{ k} = 50 \text{ k}$

Since,  $P_a < \frac{P_n}{\Omega_t} \Rightarrow 50 \text{ k} < 96.91 \text{ k}$  the member is *satisfactory*.

O.K.

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### Chapter 3.2 – Tensile Strength

➤ **Example 3-2:** Use *ASD* and compute the nominal strengths.

Alternative solution using allowable stress:

$$f_t = \frac{P_a}{A_g} = \frac{50 \text{ k}}{3.75 \text{ in}^2} = 13.3 \text{ ksi}$$

The allowable stress is:  $F_t = 0.6 F_y = 0.6(50 \text{ ksi}) = 30.0 \text{ ksi}$

For the limit state  $f_t < F_t$ : O.K.

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## Chapter 3.2 – Tensile Strength

- **Example 3-2:** Use **ASD** and compute the nominal strengths.

Alternative solution using allowable stress:

$$\text{The net section: } f_t = \frac{P_o}{A_e} = \frac{50k}{2.98 \text{ in}^2} = \boxed{16.78 \text{ ksi}}$$

$$\text{The allowable stress is: } F_t = 0.5F_u = 0.5(65 \text{ ksi}) = \boxed{32.5 \text{ ksi}}$$

For the limit state  $f_t < F_t$ : **O.K.**

Since  $f_t < F_t$  for both limit states, the member is **satisfactory**.

**O.K.**

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## Chapter 3.2 – Tensile Strength

- *What is the difference in computational effort for the two different approaches?*
- Regardless of the method used, the two nominal strengths must be computed (if a stress approach is used with **ASD**, an equivalent computation must be made).
- With **LRFD**, the nominal strengths are multiplied by resistance factors.
- With **ASD**, the nominal strengths are divided by load factors.
- Up to this point, the **number of steps is the same**.

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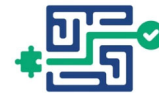
## Chapter 3.2 – Tensile Strength

- *What is the difference in computational effort for the two different approaches?*
- The difference in effort between the two methods involves the load side of the relationships.
- In **LRFD**, the loads are factored before adding.
- In **ASD**, in most cases, the loads are added.
- Therefore, for tension members, **LRFD** requires slightly more computation.

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## Chapter 3.2 – Tensile Strength

Let's work on some problems



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## Chapter 3.2 – Tensile Strength

Any questions?



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