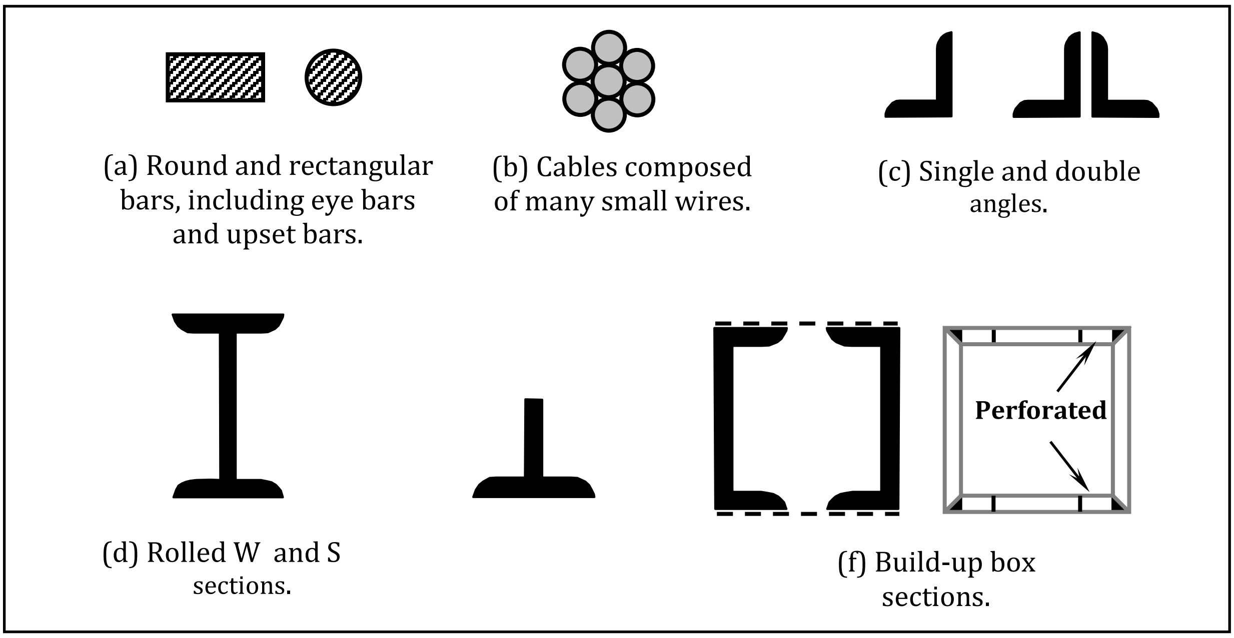
CHAPTER TWO

TENSION MEMBERS

1.1 Overview

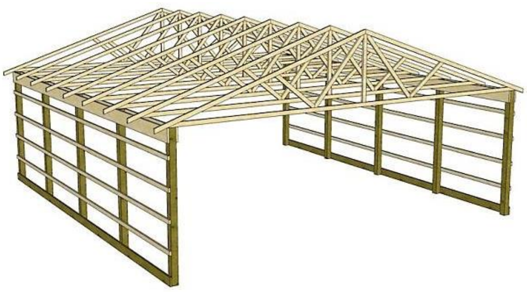
Tension member: is a structural elements which subjected to axial tensile forces. Steel shapes, which are used as tension members, are shown in the figure below.



Steel shapes use as tension members

Generally the used in:

1- ***Trusses in Frames & Bridges***



2- bracing for building & bridge

3- cables such as: suspended roof systems, suspension & bridges

The stress in an axially loaded tension member is given by:

$$f = P/A$$

Where,

P is the magnitude of load, and
A is the cross-sectional area normal to the load.

The stress in tension member is uniform throughout the cross-section except

- ✓ Near the point of application of load, and
- ✓ At the cross-section with holes for bolts.

- ➢ The cross-sectional area will be reduced by amount equal to the area removed by holes.

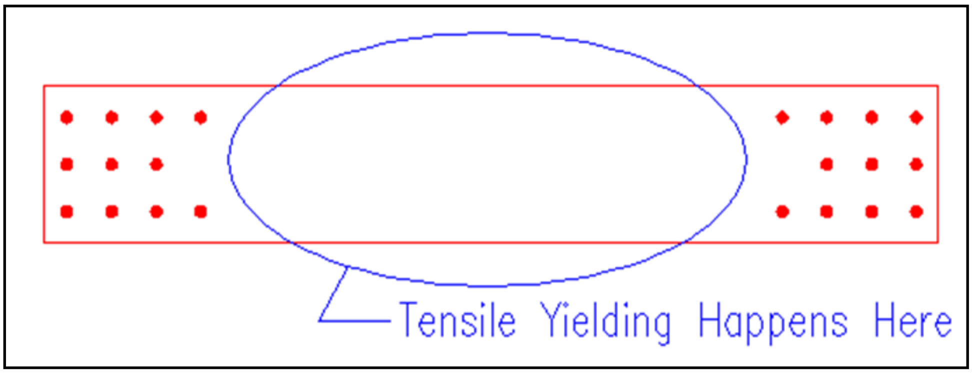
1.2 Controlling Limit States

There are three limit states that relate to the member itself. These limit states that will be considered are:

- ✓ Tensile yielding
- ✓ Tensile rupture
- ✓ Slenderness

1.2.1 Tension yielding:

Tensile yielding is considered away from the connections in the mid part of the member & excessive deformation can occur due to the yielding of the gross section. The figure shows the general region of concern for a flat plate member.



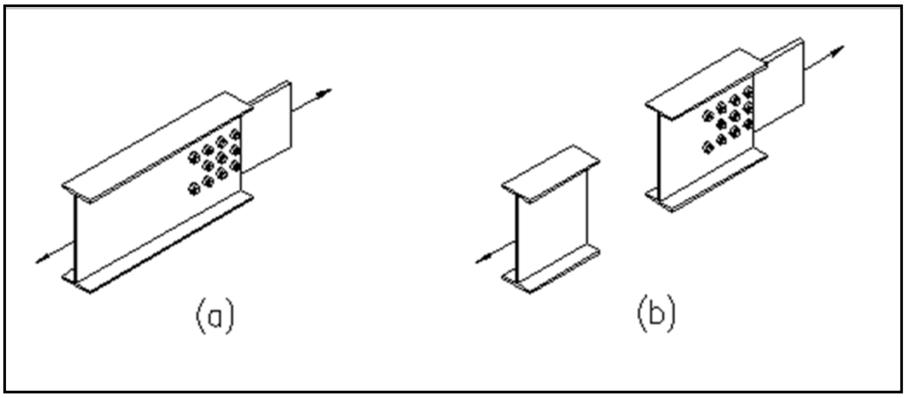
Tensile Yielding Region

Tensile yielding is illustrated in Figure (b). This failure mode looks at yielding on the gross cross sectional area, $\mathbf{A_g}$, of the member under consideration. Consequently, the critical area is located away from the connection as shown in Figure a.

To prevent excessive deformation, the stress at gross sectional area must be smaller than yielding strength:

$$f < \mathbf{F_y} \qquad \text{i.e.} \qquad \mathbf{P/A < F_y}$$

The nominal strength in yielding is: $\mathbf{P_{n1} = F_y * A_g}$



Tensile Strength Limit States

The statement of the limit states and the associated reduction factor and factor of safety are given here:

$$P_{u1} \leq \varphi_t P_{n1}$$
$$\varphi_t = 0.90$$

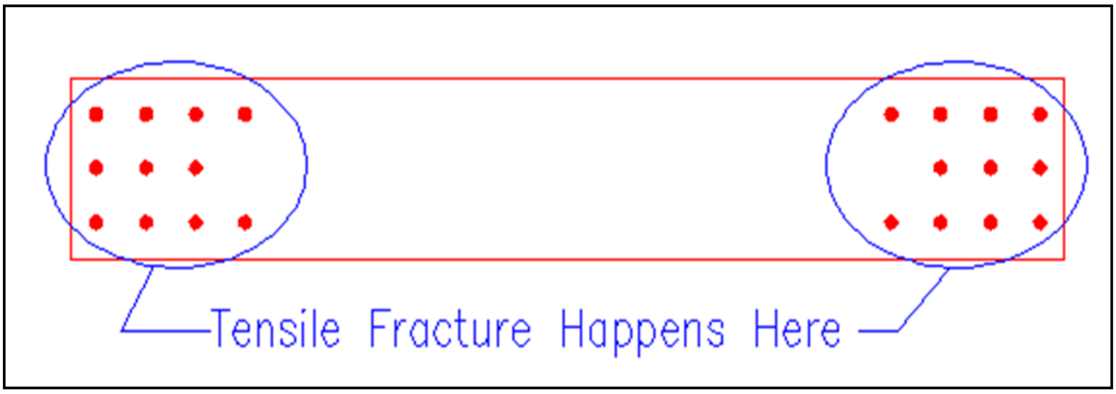
The values of P_{u1} and P_{n1} are the **LRFD** factored load and nominal tensile yielding strength of the member, respectively, applied to the member.

1.2.2 Tensile Rupture of a Member:

Tensile rupture is a strength based limit state similar to the tensile yielding limit state that we just considered. When the cross section is reduced by holes or if not all the cross sectional elements of a particular section (such as the flanges on a W section) are transferring force to a connection, then less of the section is effective in supporting the applied forces. Stress concentrations will also cause localized yielding. Local yielding to relieve stress concentrations is not a major problem for ductile materials so the yielding limit state is not considered where the connections are made. The concern at these locations is actual rupture so the applied forces are compared against the rupture strength in the region of reduced effective section. The figure illustrations where the concern is for sample flat bar member with bolted end connections. To prevent fracture, the stress at the net sectional area must be smaller than ultimate strength:

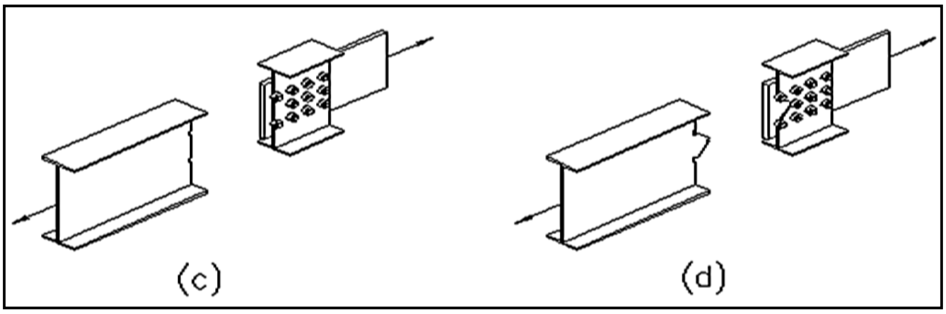
$$f < F_u \quad \text{i.e.} \quad P/A < F_u$$

The nominal strength in yielding is: $P_{n2} = F_u * A_e$



Tensile Yielding Region

In this case we have two potential failure paths that see the full force of the member. These are shown in Figures (c) and (d). Tensile rupture is complicated by the need to get the forces out of the flanges, through the web, and into the bolts. This means that we need to account for the stress concentrated in and around the bolts.



Tensile Strength Limit States

The statement of the limit states and the associated reduction factor and factor of safety are given here:

$$P_{u2} \leq \varphi_t P_{n2}$$

$$\varphi_t = 0.75$$

The values of $\mathbf{P_{u2}}$ and $\mathbf{P_{n2}}$ are the **LRFD** factored load and nominal tensile rupture strength of the member, respectively, applied to the member.

1.2.3 Block Shear Rupture:

Block shear is, in some ways, similar to tensile rupture in that the main part of the member tears away from the connection i.e. the tension member can fail due to 'tear-out' of material at the connected end. The difference is that there is now a combination of tension and shear on the failure path. Like tensile rupture, there frequently is more than one failure path. The figure shows three possible block shear failure paths for a **WT** section. Block shear strength is determined as the **sum** of the **shear strength** on a **failure path** and the **tensile strength** on a **perpendicular segment**:

Block shear strength = gross yielding strength of the shear path
+ gross yielding strength of the tension path

Or

Block shear strength= gross yielding strength of the shear path
+ net section fracture strength of the tension path

When:

- $F_u A_{nt} \geq 0.6 F_u A_{nv}$:

$$\varphi_t R_{n3} = \varphi_t (0.6 F_y A_{gv} + F_u A_{nt}) \leq \varphi_t (0.6 F_u A_{nv} + F_u A_{nt}) \quad P_{u3} \leq \varphi_t R_{n3}$$

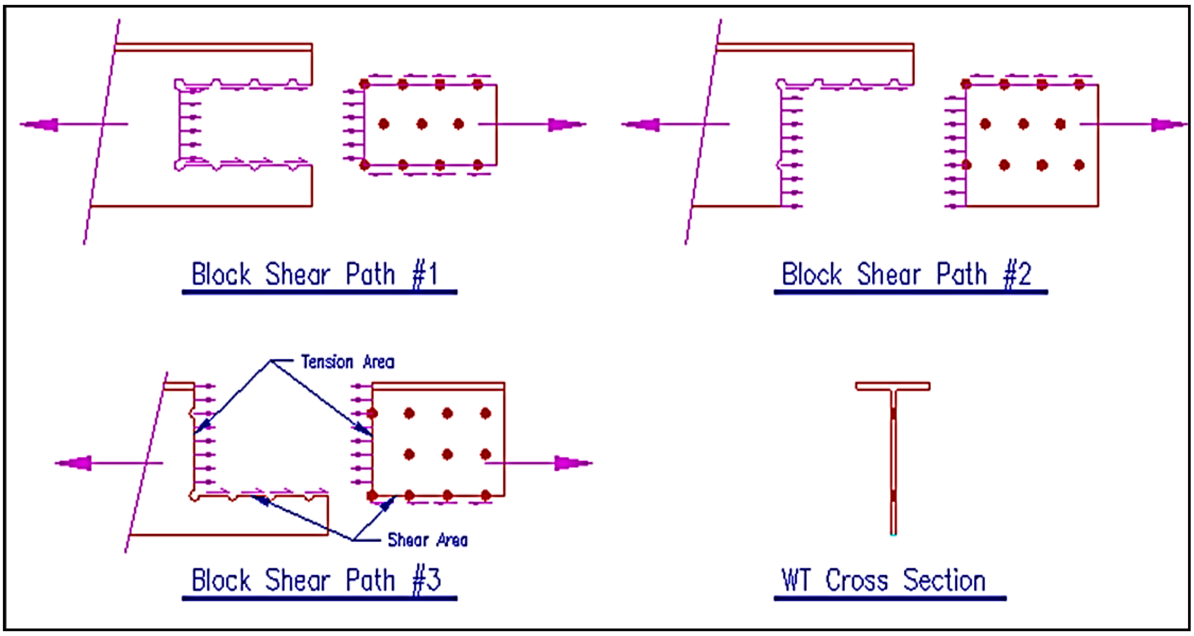
- $F_u A_{nt} < 0.6 F_u A_{nv}$:

$$\varphi_t R_{n3} = \varphi_t (0.6 F_u A_{nv} + F_y A_{gt}) \leq \varphi_t (0.6 F_u A_{nv} + F_u A_{nt})$$

$$P_{u3} \leq \varphi_t R_{n3}$$

Where: $\varphi_t = 0.75$
A_{gv} = gross area subjected to shear
A_{gt} = gross area subjected to tension
A_{nv} = net area subjected to shear
A_{nt} = net area subjected to tension

and values of P_{u3} and R_{n3} are the LRFD factored load and nominal resistance or strength associated with block shear of the member, respectively.



Block Shear Failure Paths

1.2.4 Slenderness Limits:

Slenderness is a *serviceability* limit state, not a strength limit state, so failure to adhere to the suggestion is unlikely to cause an unsafe condition.
The limit state is written as:

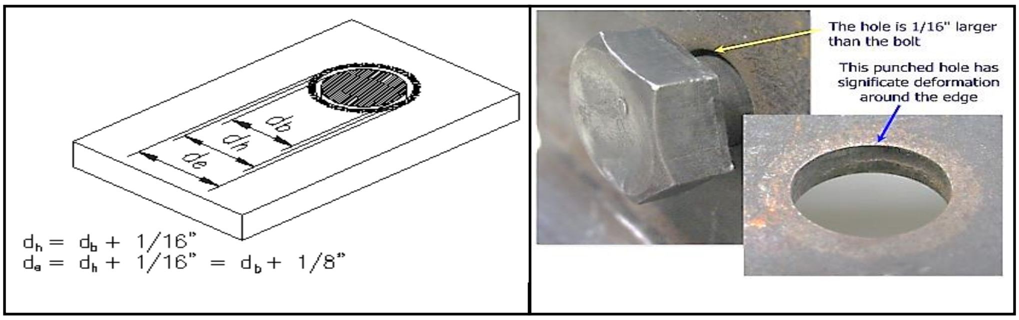
$$L/r_{min.} \leq 300$$

Where "$\mathbf{r_{min}}$" is the least radius of gyration. "**r**" is a *section property that equals the square root of the moment of inertia divided by the cross section area*. Every member has an "**r**" for each of the principle axes.

1.3 Area Determination

1.3.1 Net Area (An):

The net area computation requires computation of a reduced section due to *holes* made in the member as well a *failure path* for the rupture surface. The figure shows a typical standard hole and the dimensions that are related to it.



Bolt Holes

For A_n calculations, is to be taken as **1/8"** larger than the bolt (i.e. **1/8" = 1/16"** *for the actual hole diameter plus an additional* **1/16"** *for damage related to punching or drilling.)* So, if you specify **3/4"** bolts in standard holes, the effective width of the holes is **7/8"** (i.e. **3/4"** for the bolt diameter + **1/16"** for the hole diameter **+1/16"** damage allowance.).

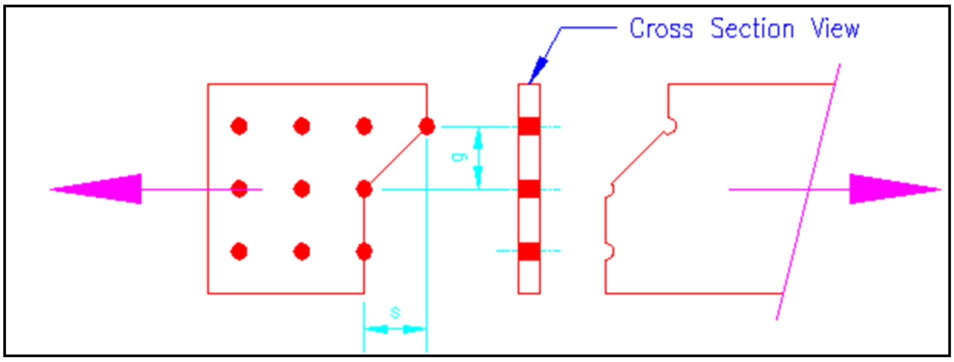
The next concept that needs discussing is the concept of failure paths. Failure paths are the approximate locations where a fracture may occur. For bolted tension member, maximum net area can be achieved if the bolts are

Placed in a single line. The connecting bolts can be staggered for several reasons:

1- To get more capacity by increasing the effective net area
2- To achieve a smaller connection length
3- To fit the geometry of the tension connection itself

The figure shows a failure path that has a component that is not perpendicular to the line of action for the force. The stagger is characterized by a "**pitch**" of **s** and a "**gage**" of **g** as shown.

$$A_n = A_g - (\sum d + \sum s^2/4g) * t$$



Failure Path with Staggered Bolts

1.3.2 Effective Net Area, A_e:

In cases where *SOME BUT NOT ALL* of the cross sectional elements are used to transfer force to/from the member at the connection, then not all the net area is really effective for tensile rupture. This is the result of a phenomena called shear lag. Shear lag affects both bolted and welded connections. Therefore, the effective net area concept applied to both type of connections.

- ✓ For bolted connection, the effective net area is $A_e = UA_n$
- ✓ For welded connection, the effective net area is $A_e = UA_g$

Where, the reduction factors U is given by: $U = 1 - x/L$

Where, **x** is the distance from the centriod of the connected area to the plane of the connection, and **L** is the length of the connection.

1.3.3 Reduction Coefficient "U":

The AISC manual also gives values of **U** that can be used instead of calculating **x/L** as follow:

1.3.3.1 Bolted Members

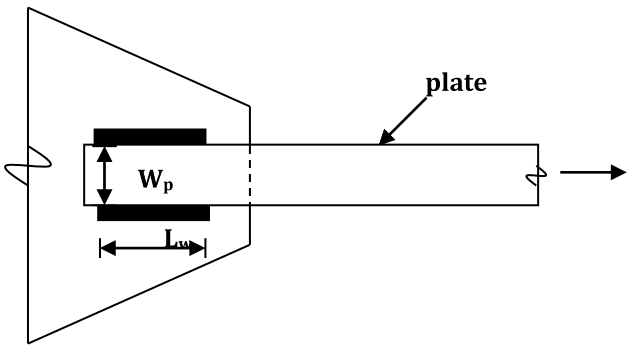
- For W, M, I, and S shapes with $b_f/d \geq 2/3$ with at least three fasteners per line in the direction of applied load …….U=0.9
- For T – shape with $b_f/d \geq 4/3$ with at least three fasteners per line in the direction of applied load …….U= 0.9
- For I- & T- shapes not meeting the above conditions & all other shapes including build up section ...U=0.85
- For all other shapes section with only two fasteners per line ...U=0.75
- When the load is transmitted through all of the cross section, U=1

1.3.3.2 Welded Members

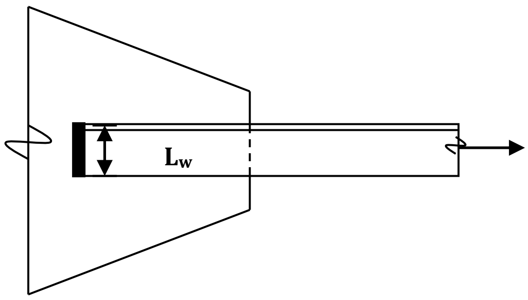
- When a plate is connected by only longitudinal weld to all
  - U = 0.75 when $1.0 \leq (L_w/W_p) < 1.5$
  - U = 0.87 when $1.5 \leq (L_w/W_p) < 2.0$
  - U = 1.00 when $(L_w/W_p) \geq 2.0$

Where L_w = length of longitudinal weld, in

W_p = plate width, in



- When tensile load is transmitted by transverse welds only $A_n=A_e$ & U=1.0



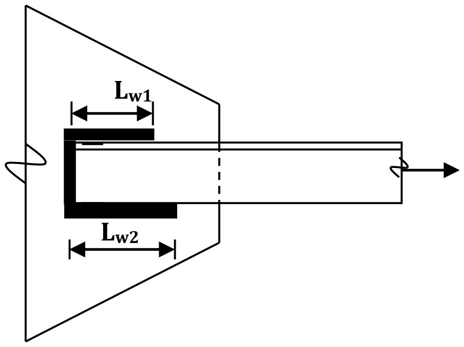
– When tensile load is transmitted only by longitudinal weld to a member other than plate, or longitudinal welds in combination with transverse welds:

$$A_n = A_g \qquad \& \qquad U = \min[(1 - x_{con}/L_{con}), 0.9]$$

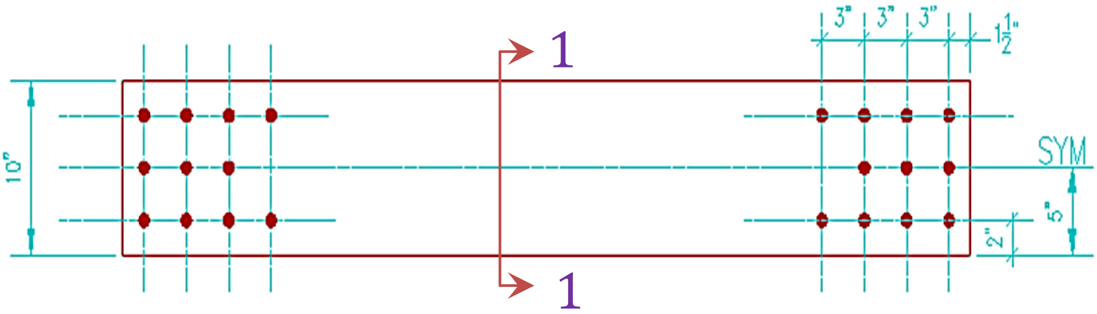
Where A_g = gross area of the members, in^2

L_{con}= connection length, taken as the length of longer longitudinal weld, in

= max. [L_{w1}, L_{w2}]



<u>Example 2-1</u>: **A 3/4" x 10"** plate of **Gr. 36** steel have span of **5 ft** long and has standard holes for **3/4"** bolts at each end for attachment to other structural members. The figure shows a face view of the plate. The service level loads that the member will be subject to are **140 kips** of dead load and **30 kips** of live load. Determine the axial tension capacity of the member.



Solution:

The problem solution is pursued in the following steps:

Determine the demand on the member.

P_u = 1.2D + 1.6L = 1.2(140 k) + 1.6(30 k) = 216 kips

Check size based on the slenderness limit state.

Our member is 5 feet long and the least value of r is computed as:

$$r = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{(10")(0.75")^3/12}{(10")(0.75")}} = 0.217in$$

The correct computation of L/r = (5 ft)(12 in/ft) / (0.217 in)= 277 < 300 ... The limit state is satisfied

Determine the capacity of the member based on the

- ✓ tensile yielding limit state

$$P_{n1} = F_y * A_g = (36\ ksi)(7.500\ in^2) = 270\ kips$$

$$\phi_t P_{n1} = 0.9*270 = 243\ k > P_u \quad ....Ok.$$

- ✓ tensile rupture limit state

First let's compute the net area A_n for each of the two failure paths identified in Figure 2-5-1.

Path #2

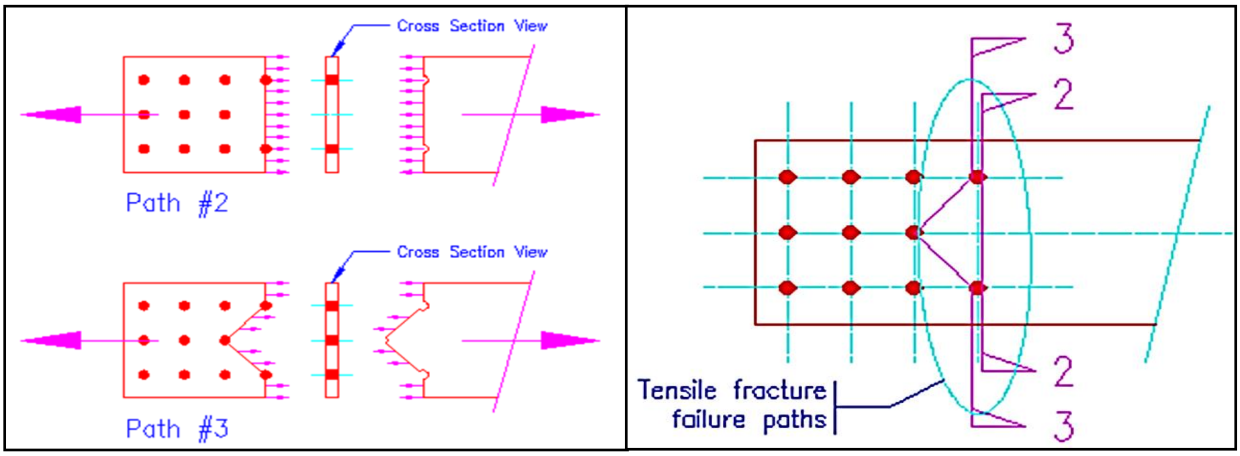
$A_{n2} = A_g$ - hole area + gage area
$= A_g$ - (num holes)
$*(d_b+1/16"+1/16")(t_{pl})$
$= 7.50\ in^2$ - (2 holes)
$*(0.75\ in + 1/8")(0.75\ in)$

$A_{n2} = 6.19\ in^2$

Path #3

$A_{n3} = A_g$ - hole area + gage area
$= A_g$ - (num holes)$(d_b+1/16"+1/16")(t_{pl})$
$+ (t_{pl})(s^2/4g)_1 + (t_{pl})(s^2/4g)_2$
$= 7.50\ in^2$ - (3 holes)$(0.75\ in + 1/8")(0.75\ in)$
$+ (0.75\ in)(3\ in)^2/(4*(3\ in))$
$+ (0.75\ in)(3\ in)^2/(4*(3\ in))$

$A_{n3} = 6.66\ in^2$



Tensile Rupture Failure Paths

The controlling net area is $\mathbf{A_{n2}}$ as it has the smaller value. This means that, if tensile rupture were to actually occur, this is the path that it would take. Therefore, for this problem:

$$\mathbf{A_n = 6.19\ in^2}$$

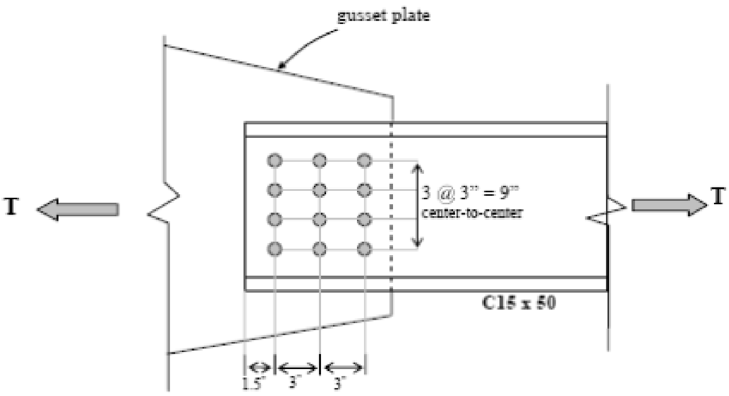
In this problem we have only one cross sectional element (i.e. one plate element in the cross section) and it is attached to the bolts leading us to **U = 1.0.** This means that there is no shear lag for this problem.

$$A_e = UA_n = (1)(6.19\ in^2) = 6.19\ in^2$$

$$P_{n2} = F_u * A_e = (58\ ksi)(6.19\ in^2) = 359\ kips$$

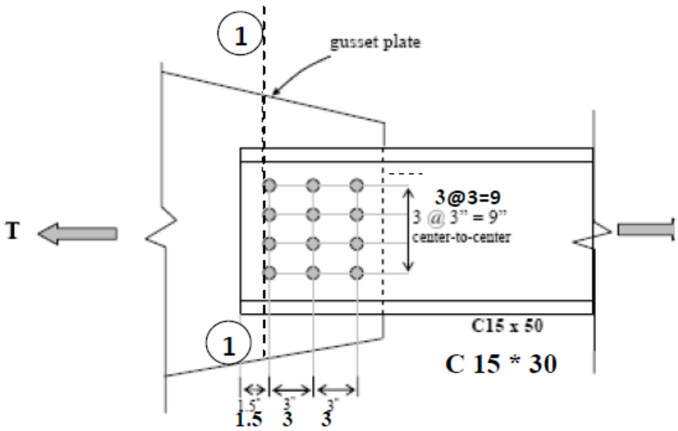
$$\phi_t\ P_{n2} = 0.75*359 = 269\ k > P_u \quad \text{.......Ok.}$$

Example 2-2: Determine the design tension strength for a single channel C15 x 50 connected to a 0.5 in. thick gusset plate as shown in Figure. Assume that the holes are for 3/4 in. diameter bolts and that the plate is made from structural steel with yield stress (Fy) equal to 50 ksi and ultimate stress (Fu) equal to 65 ksi.



Solution:

From AISCM for C15*50 $A_g = 14.7\ in^2$,
$t_w = 0.716"$, $\bar{x} = 0.798"$



- **Yielding Due to Tension**

$$\phi_t P_n = \phi_t\ F_y\ A_g = 0.9*50*14.7 = 662\ kips$$

- **Fracture Due to Tension**

- For section 1-1

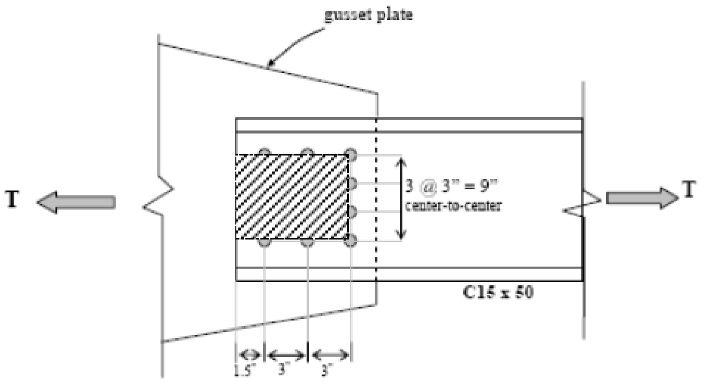
$$A_n = 14.7 - 4*(\frac{3}{4}+\frac{1}{8})*0.716 = 12.19\ in^2$$

$$U = 1 - \frac{\bar{x}}{L} = 1 - \frac{0.798}{6} = 0.867"$$

$$A_e = U\ A_n = 0.867 * 12.19 = 10.57$$

$$\phi_t P_n = \phi_t\ F_u\ A_e = 0.75*65*10.57 = 515\ kips$$

- Block Shear Rupture



- $A_{nt} = 9*0.716 - 3*(\frac{3}{4}+\frac{1}{8})*0.716 = 4.565\,in^2$

$A_{gv} = 2*[(3+3+1.5)*0.716] = 10.74\,in^2$

$A_{nv} = 2*[5.37 - 2.5*(\frac{3}{4}+\frac{1}{8})*0.716] = 7.61\,in^2$

$\phi_t P_n = \phi_t\,[0.6\,F_u\,A_{nv} + U_{bs}\,F_u\,A_{nt}]$
$= 0.75[0.6*65*7.61 + 1.0*65*4.565] = 445.13$ kips

OR $\phi_t\,P_n = \phi_t\,[0.6\,F_y\,A_{gv} + U_{bs}\,F_u\,A_{nt}]$
$= 0.75[0.6*50*10.74 + 1.0*65*4.565] = 464.19$ kips

$\therefore \phi_t P_n = 445.13$