Lecture 4 – Tension members

Steel tension members are perhaps the simplest members to design. There is no compressive, bending, shear or other stresses involved. Typical structural members that are under tension loads are:

- Trusses
- Bracing
- Hangers

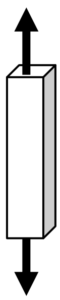
The design of steel tension members is found in the following locations in the LRFD Manual:

- AISC Part 5
- AISC SPEC Chapter D p. 16.1-26
- AISC SPEC Chapter D p. 16.1-282

There are 2 types of failure mechanisms for tension members. The first is ***yielding on the gross area*** and the second is ***fracture on the net section***.

Yielding on Gross Area

Yielding on the gross area refers to "stretching" of the gross cross-sectional area of the member:



The **LRFD available strength** for yielding on gross section in tension = $\phi_t P_n$

The **ASD available strength** for yielding on gross section in tension = $\dfrac{P_n}{\Omega}$

where: $\phi_t = 0.90$ (LRFD)

$\Omega = 1.67$ (ASD)

P_n = nominal strength of member

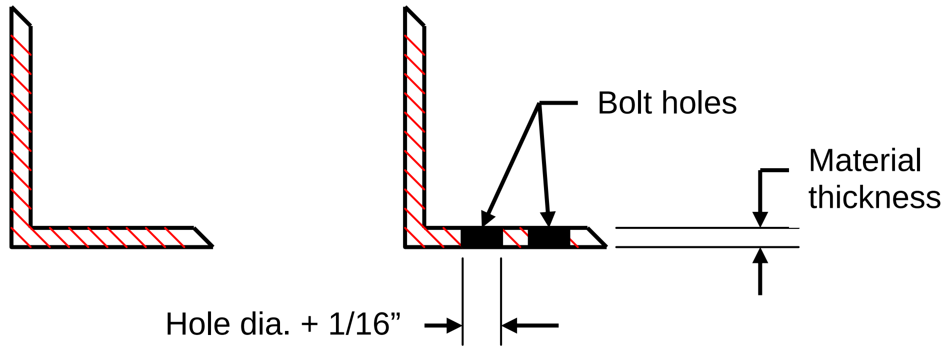
$= F_y A_g$

F_y = yield stress of steel (see AISC p. 2-48)

A_g = gross cross-sect. area (see AISC Part 1)

Fracture on Net Section

Fracture on the net section refers to "breaking" the section perpendicular from the direction of force through the reduced cross-sectional area of a member, typically across the bolt holes.



A_g = Gross area of angle **A_n = Net area of angle**

The **LRFD available strength** for fracture on net section in tension = $\phi_t P_n$

The **ASD available strength** for fracture on net section in tension = $\dfrac{P_n}{\Omega}$

where: ϕ_t = 0.75 (LRFD)
Ω = 2.00 (ASD)
P_n = nominal strength of member
= $F_u A_e$

F_u = ultimate stress of steel (see AISC p. 2-48)
A_e = effective net area
= $A_n U$

A_n = net area (see AISC p. 16.1-18)
= A_g – [No. of holes{(hole dia. + 1/16")(matl. thk.)}]

U = reduction factor considering "shear lag"
= See AISC Table D3.1 p. 16.1-28
= 1.0 if tension load is transmitted directly to each element by means of fasteners or welds
= $1-\left(\dfrac{\bar{x}}{l}\right)$ if tension load is transmitted to some but not all of the elements by use of fasteners or welds

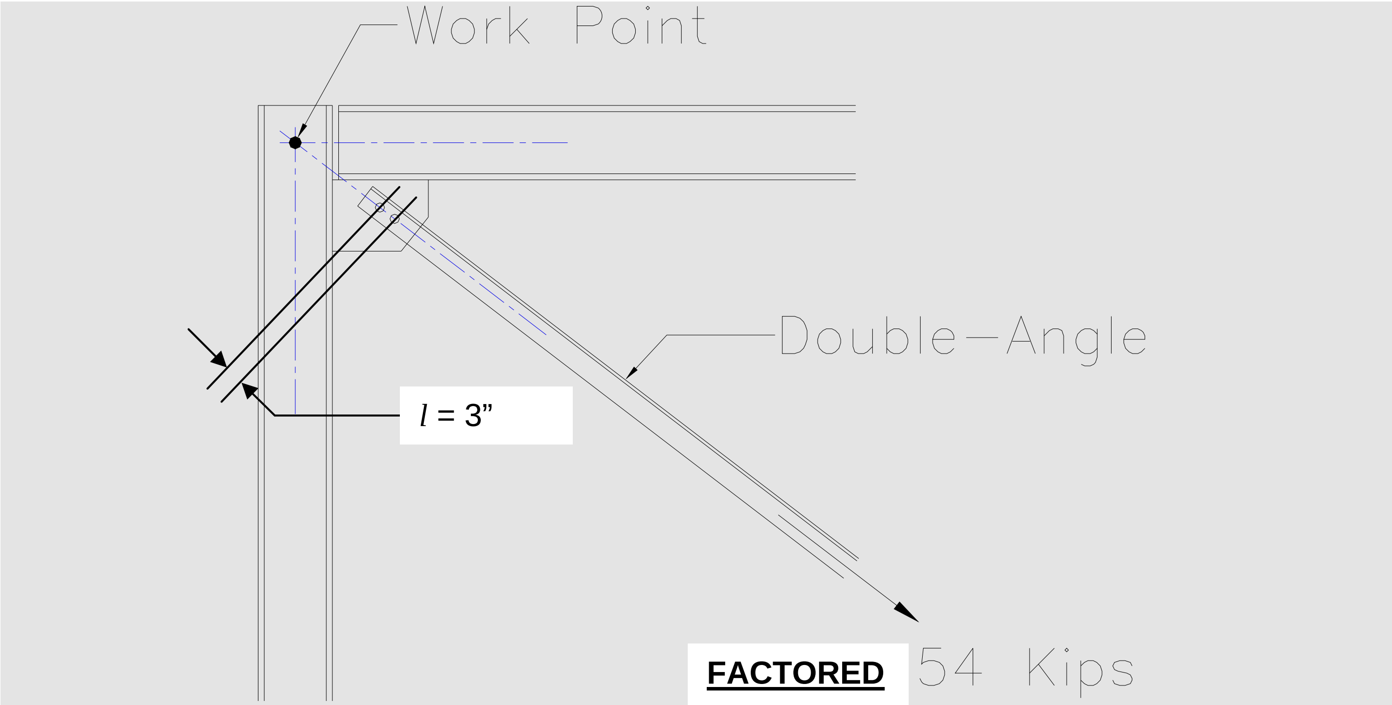
where: $\bar{x}$ = connection eccentricity, inch
l = length of connection in the direction of loading, inch

Example 1 **(LRFD)**

GIVEN: The double-angle L4x4x¼ A36 bracing member as shown below is under a **FACTORED tensile load** of 54 kips. The angles are bolted to the steel gusset plate using 2 – ¾" diameter bolts.

REQUIRED:

1) Determine if the angles are acceptable based on yielding on gross area.
2) Determine if the angles are acceptable based on fracture on net area.



Step 1 – Determine LRFD available strength of member considering "yielding":

Available strength = $\phi_t P_n$

$= 0.90(F_y A_g)$

$= 0.90(36\text{ KSI})[2\text{ angles}(1.94\text{ in}^2\text{ per angle})]$

$= 125.7\text{ KIPS}$

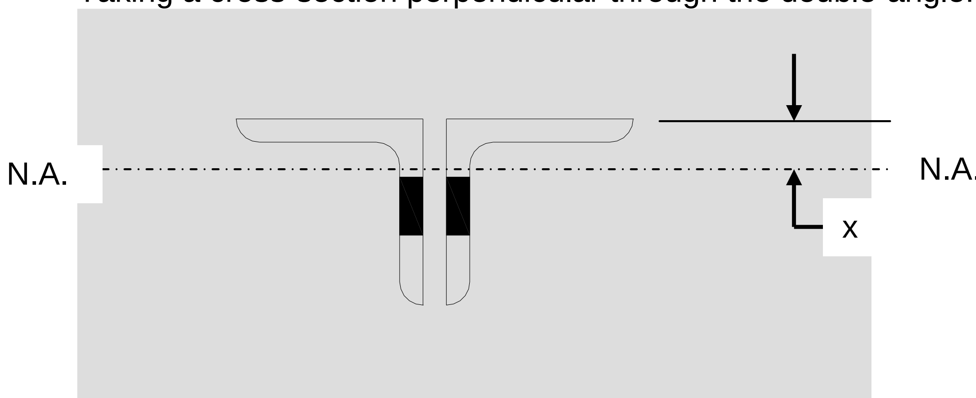
Since 125.7 KIPS > 54 KIPS → member is acceptable

<u>Step 2 – Determine LRFD available strength considering "fracture" on net area</u>:

Available strength = $\phi_t P_n$
$= 0.75(F_u A_e)$

$A_e = A_n U$

Taking a cross-section perpendicular through the double-angle:



$A_n = A_g$ – [No. of holes{(hole dia. + 1/16")(mat. th)}]
= (2)(1.94 in^2) – [2 holes{(¾" + 1/16")(¼")}]
= 3.47 in^2

$$U = 1 - \left(\frac{\bar{x}}{l}\right)$$

where $\bar{x}$ = 1.08" see properties AISC p. 1-45
l = Connection length
= 3" (see sketch above)

$$= 1 - \left(\frac{1.08''}{3''}\right)$$

= 0.64

$A_e = A_n U$
= 3.47 in^2(0.64)
= 2.22 in^2

Available strength = 0.75(58 KSI)(2.22 in^2)
= 96.6 KIPS

<u>Since 96.6 KIPS > 54 KIPS → member is acceptable</u>

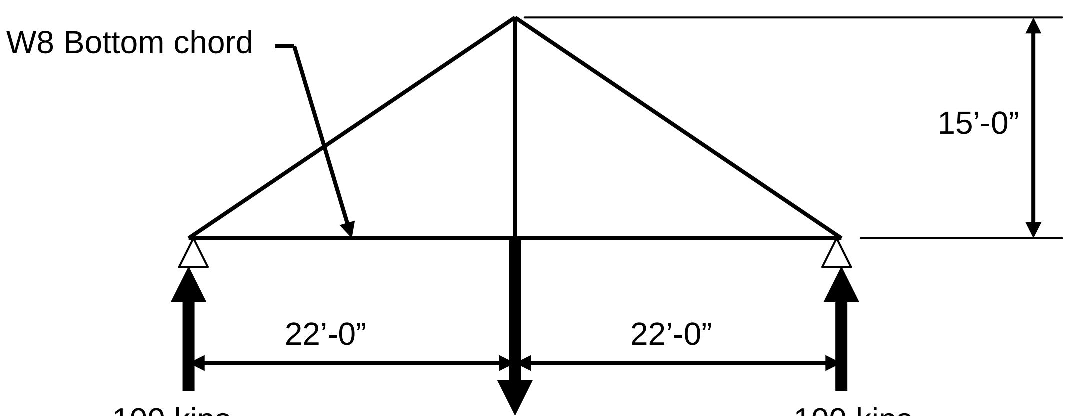
In the design of steel tension members the AISC recommends a maximum permissible **slenderness ratio,** $\frac{L}{r_{\min}}$ **$\leq$ 300** for members other than rods, straps or HSS members (see AISC p. 16.1-26). This is not so much for structural strength – rather to provide some stiffness to reduce the undesirable effects of lateral movement. Also, since truss members and other tension members may see some load-reversal and experience some compressive loads, it is to reduce the likelihood of buckling.

Example 2 (ASD)

GIVEN: The bottom chord of the truss is to be designed using a single A992 W8 member. The truss has welded connections, so there is no need to check for fracture on the net area.

REQUIRED: Design the lightest weight W8 member checking:

1) Slenderness ratio is not exceeded
2) Yielding on gross area



SERVICE load = 200 Kips

Step 1 – Determine r_{min} such that slenderness ratio is not exceeded:

$$\frac{L}{r_{\min}} \leq 300$$

L = 22'-0"(12" per ft)
  = 264"

Rearranging and solve for r_{min}:

$$\frac{264''}{300} \leq r_{\min}$$

Required $r_{min} \geq$ 0.88 in. $\rightarrow$ looking at AISC p. 1-29, the smallest W8 that will work is W8x18 with r_y = 1.23 in. $\geq r_{min}$ of 0.88 in.

<u>Step 2 – Determine SERVICE tensile load on member</u>:

By simple truss analysis, the force in the member is:

$$\frac{VertForce}{VertLength} = \frac{HorzForce}{HorzLength}$$

$$\frac{100KIPS}{15'\text{–}0''} = \frac{HorzForce}{22'\text{–}0''}$$

Horz Force = 146.7 KIPS

<u>Step 3 – Design lightest W8 member considering yielding on gross area</u>:

ASD Available strength for yielding = $\frac{F_y A_g}{\Omega} \geq$ 146.7 KIPS

where: Ω = 1.67
F_y = 50 KSI (see AISC p. 2-48)

Solve for A_g:

$$A_g \geq \frac{146.7kips(\Omega)}{F_y}$$

$$\geq \frac{146.7kips(1.67)}{50KSI}$$

$A_g \geq 4.9\ in^2$

AISC p. 1-28

<u>Use W8x18 → Area = 5.26 in² > 4.9 in²</u>

<u>Step 4 – Determine the maximum available tensile yielding load on W8x18:</u>

ASD Available strength for yielding = $\frac{F_y A_g}{\Omega}$

$$= \frac{(50KSI)(5.26in^2)}{1.67}$$

<u>ASD Available strength for yielding = 157.5 Kips</u>

Step 5 – Determine the available tensile yielding load on W8x18 using AISC Table 5-1:

From AISC p. 5-13:

Table 5–1 (continued)
Available Strength in Axial Tension
W Shapes

F_y = 50 ksi
F_u = 65 ksi

W8

ASD

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding, kips		Rupture, kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
W8×67	19.7	14.8	590	887	481	722
×58	17.1	12.8	512	770	416	624
×48	14.1	10.6	422	634	345	517
×40	11.7	8.78	350	527	285	428
×35	10.3	7.73	308	463	251	377
×31	9.12	6.84	273	410	222	333
W8×28	8.24	6.18	247	371	201	301
×24	7.08	5.31	212	319	173	259
W8×21	6.16	4.62	184	277	150	225
×18	5.26	3.94	157	237	128	192
W8×15	4.44	3.33	133	200	108	162
×13	3.84	2.88	115	173	93.6	140
×10	2.96	2.22	88.6	133	72.2	108

W8x18

Available yielding **SERVICE** tensile load = **157 Kips**

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.