2.4 Design of Tension Members

In design problems, the required tensile strength of member, $\mathbf{P_u}$, is known. The design task then consist of selecting a section and end connection such that the design tensile strength of member, $\boldsymbol{\phi}\mathbf{P_n}$, is greater than or equal to the required strength $\mathbf{P_u}$. thus for design:

$$\phi P_n = \min[\phi P_{n1}, \phi P_{n2}] \geq P_u \quad \text{or} \quad \phi P_{n1} \geq P_u \quad \& \quad \phi P_{n2} \geq P_u$$

$\mathbf{P_u}$, $\mathbf{P_{n1}}$ and $\mathbf{P_{n2}}$ are the LRFD factored load (or required tensile strength of member), nominal tensile yielding strength of the member, and nominal tensile rupture strength of the member, respectively. To satisfy the limit state of yielding in the gross section, the gross area must satisfy the relation:

$$A_{g1} \geq P_u /(0.9*F_y)$$

While to satisfy the limit state of fracture in the net section, the net area must satisfy the relation:

$$A_n \geq P_u /(0.75*F_y*U)$$

Then $$A_{g2} \geq P_u /(0.75*F_y*U) + \text{estimated loss in area due to bolt holes}$$

$$A_g \geq \max.[A_{g1}, A_{g2}]$$

So, only section that satisfy the these relation are retained for further consideration in design.

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

2.5 Tables for the Design of Tension Members

Part 5 of the Manual contain tables to assist the design of tension members of various cross-sectional shapes. The AISC manual tabulates the tension design strength of standard steel sections - Include: wide flange shapes, angles, tee sections, and double angle sections.

- The net section fracture strength is tabulated for an assumed value of U = 0.75, obviously because the precise connection details are not known.

- For all W, Tee, angle and double-angle sections, A_e is assumed to be = 0.75 Ag

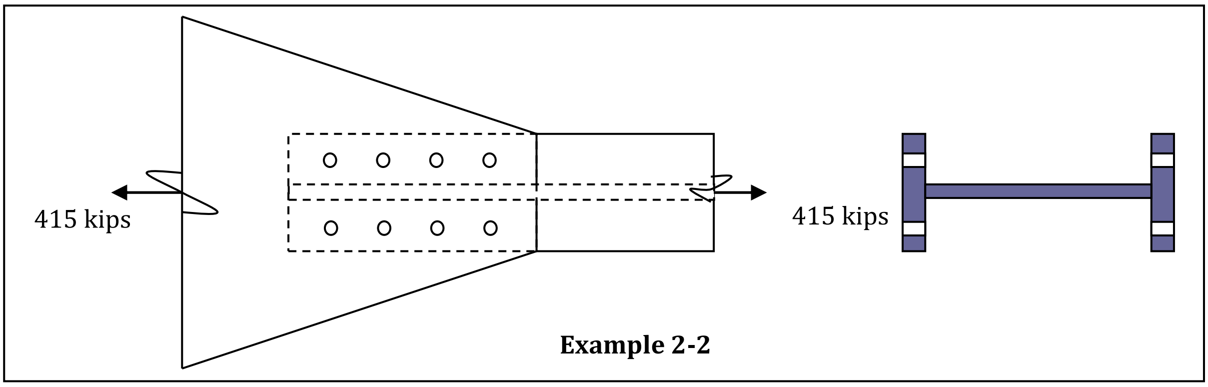
- The engineer can first select the tension member based on the tabulated gross yielding and net section fracture strengths, and then check the net section fracture strength and the block shear strength using the actual connection details.

Example 2-3: Select the lightest **W16*?** Shown in the figure, as a member of truss to transmit a factored tensile load of **415 kips**, the member is 30 ft long. **A588 Grade 50** shapes are available. Use ⅞-in bolt in two line in each flange.

Solution:

A588 Grade 50 steel; $\mathbf{F_y = 50\ ksi}$ and $\mathbf{F_u = 70\ ksi}$. (**Table 2-3, page 2-39**)

Required member strength $\mathbf{P_u = 415\ kips}$



Example 2-2

$$P_{n1} = F_y * A_g \geq P_u$$

$$A_{g1} \geq P_u /(0.9*F_y) = 415/(0.9*50) = 9.22 \text{ in}^2$$

$$r_{min} \geq L/300 = (3*12)/300 = 1.2 \text{ in}$$

from the LRFD Manual, W16×36 **(Page 1-20)** satisfy the two requirements

$$A_g = 10.6 \text{ in}^2 > 9.22 \text{ in}^2 \quad \& \quad r_y = 1.52 > 1.2 \text{ in} \quad \text{O.K.}$$

$b_f = 6.99$ in, $d = 15.9$ in., $b_f /d = 0.439 < ⅔$ ……. $U = 0.85$

$$d_e = d_b + 1/8 = 7/8 + 1/8 = 8/8 = 1 \text{ in}$$

$$A_n = A_g - 4d_e t_f = 10.6 - 4(1)(0.43) = 8.88 \text{ in}^2$$

$$\phi_t P_{n2} = 0.75 F_u * A_e = (0.75)(70 \text{ ksi})(0.85*8.88) = 396 \text{ kips} < P_u = 415 \quad \text{…….N.G.}$$

Select the next heavier section, a W16×40:

$$A_g = 11.8 \text{ in}^2 > 9.22 \text{ in}^2 \quad \& \quad r_y = 1.57 > 1.2 \text{ in} \quad \text{O.K.}$$

$b_f = 7.00$ in, $d = 16$ in., $b_f /d = 0.4375 < ⅔$ ……. $U = 0.85$

$$d_e = d_b + 1/8 = 7/8 + 1/8 = 8/8 = 1 \text{ in}$$

$$A_n = A_g - 4d_e t_f = 11.8 - 4(1)(0.505) = 9.78 \text{ in}^2$$

$$\phi_t P_{n2} = 0.75 F_u * A_e = (0.75)(70 \text{ ksi})(0.85*9.78) = 436 \text{ kips} > P_u = 415 \quad \text{.......O.K.}$$

Example 2-4 : A tension member with a length of 5 feet 9 inches must resist a service dead load of 18 kips and a service live load of 52 kips. Select a member with rectangular cross section plate. Use A36 steel and assume connection with one line of 7/8 inch diameter bolts.

Solution:

$$P_u = 1.2D + 1.7L = 1.2 * 18 + 1.6*52 = 104.8 \text{ kips}$$

A_g For tensile yielding

$$\text{Required } A_g = \frac{P_u}{0.9 F_y} = \frac{104.8}{0.9*36} = 3.235 \text{ in}^2$$

A_g For tensile rupture

U=1.0 for bolted plate

$$\text{Required } A_g = \frac{P_u}{0.75 F_u U} + \sum A_{holes}$$

$$= \frac{104.8}{0.75*58*1.0} + (\frac{7}{8} + \frac{1}{8})*t$$

$$= 2.41 + t$$

$$r_{min} = \frac{L}{300} = \frac{5.75*12}{300} = 0.23 \text{ in}$$

$$\text{Let } t = \frac{1}{2}'' \rightarrow A_g = 2.91 < 3.235 \rightarrow A_g = 3.235$$

Use PL 1/2 *7

$$A = 7*1/2 = 3.5 > 3.235,\ I_{min} = \frac{7*(1/2)^3}{12} = 0.073$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{0.073}{3.5}} = 0.14 \text{in} < 0.23 \rightarrow \text{NG}$$

Increase t

$$\text{Let } t = \frac{5}{8}'' \rightarrow A_g = 3.035 < 3.235 \rightarrow A_g = 3.235$$

Use PL 5/8 *5.5

$$A = 3.44 > 3.235,\ I_{min} = 0.111, r_{min} = 0.18\text{in} < 0.23 \rightarrow \text{NG}$$

$$\text{Let } t = 1'' \rightarrow A_g = 3.41 > 3.235 \rightarrow A_g = 3.41$$

Use PL 1 * 3.5

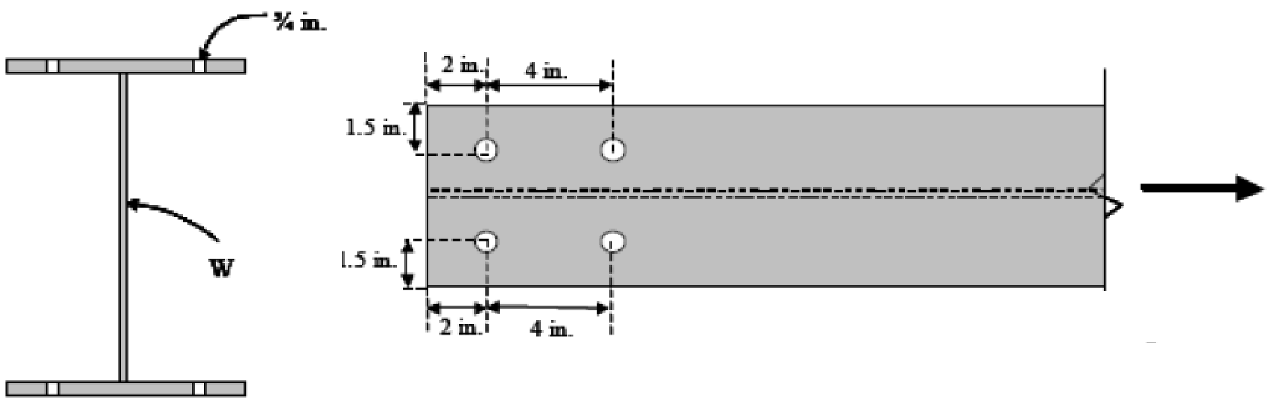
$$A = 3.5 > 3.41,\ I_{min} = 0.29\ ,\ r_{min} = 0.29\text{in} > 0.23 \rightarrow \text{OK}$$

Choose PL 1*3.5"

The results can be summarizes as follow:

Thickness	provided Width	A_g Required		A_g Available	r	Remarks
		Yielding	Fracture			
1/2"	7	3.235	2.91	3.5	0.14	**A_g Available < A_g Required ---- OK** **r < r_{min}**----------NG
5/8"	5.5	3.235	3.035	3.44	0.18	**A_g Available < A_g Required ---- OK** **r < r_{min}**----------NG
1"	3.5	3.235	3.41	3.5	0.29	**A_g Available < A_g Required ---- OK** **r > r_{min}**----------OK

Example 2-5: Design a member to carry a factored maximum tension load of 100 kips. Assume that the member is a wide flange connected through the flanges using eight ¾ in. diameter bolts in two rows of four each as shown in the figure below. The center-to-center distance of the bolts in the direction of loading is 4 in. The edge distances are 1.5 in. and 2.0 in. as shown in the figure below. Steel material is A992.



Solution:

For steel A992

$F_y = 50$ ksi and $F_u = 65$ ksi

$P_u = 100$ kips

A_g For tensile yielding

$$\text{Required } A_g = \frac{P_u}{0.9\,F_y} = \frac{100}{0.9 * 50} = 2.22\,\text{in}^2$$

A_g For tensile rupture

Shear lag factor, *U,* is assumed 0.75 for W shape due to flange have bolts less than three thus alternative values of Table D3.3 not applicable.

$U = 0.75$

$$\text{Required } A_g = \frac{P_u}{0.75\,F_u\,U} + \sum A_{holes}$$
$$= \frac{100}{0.75 * 65 * 0.75} + 2 * 2 * (\frac{3}{4} + \frac{1}{8}) * t$$
$$= 2.74 + 3.5\,t$$

Go to the Table 5.1 of AISC manual, with Pu=100 and A_g = 2.96 and select W 8*10, ***see the table***

For W 8*13 the gross yielding strength = 173 kips, and

net section fracture strength=140 kips

Check selected section for net section fracture

- A_g = 3.84 in2
- A_n = 3.84 – 4*(7/8+1/8) 0.255 = 2.95 in^2
- From dimensions of WT4 x 6.5 $\bar{x} = 1.03"$

$$U = 1 - \frac{\bar{x}}{L} = 1 - \frac{1.03}{4} = 0.74"$$

- $A_e = U A_n = 0.74 * 2.95 = 2.19$

$$\therefore \phi_t P_n \text{ for fracture} = 0.75 * 65 * 2.19 = 106.7 > P_u = 100 \rightarrow \text{OK}$$

Check the block shear rupture strength

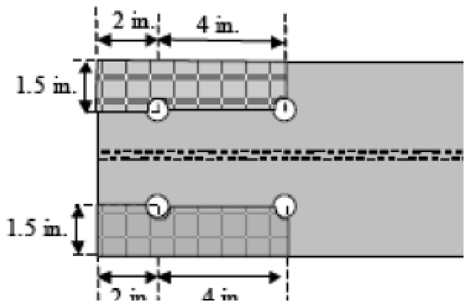
Identify block shear path

- $$A_{nt} = 4[1.5*0.255 - 0.5*(\frac{3}{4}+\frac{1}{8})*0.255] = 1.084\,in^2$$
$$A_{gv} = 4*[(2+4)*0.255] = 6.12\,in^2$$
$$A_{nv} = 4*[6.12 - 1.5*(\frac{3}{4}+\frac{1}{8})*0.255] = 4.78\,in^2$$

$$\phi_t P_n = \phi_t [0.6\ F_u\ A_{nv} + U_{bs}\ F_u\ A_{nt}]$$
$$= 0.75[0.6*65*4.78 + 1.0*65*1.084] = 192.66 \text{ kips}$$

OR $$\phi_t P_n = \phi_t [0.6\ F_y\ A_{gv} + U_{bs}\ F_u\ A_{nt}]$$
$$= 0.75[0.6*50*6.12 + 1.0*65*1.0844.565] = 190.55 \text{kips}$$
$$\therefore \phi_t P_n = 190.55 > P_u = 100 \rightarrow \text{OK}$$

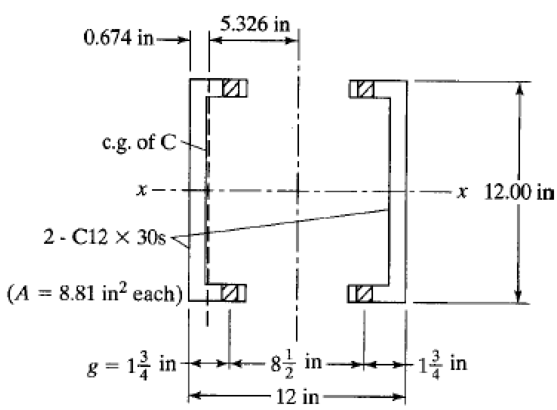
Therefore, W 8 x 13 is acceptable.



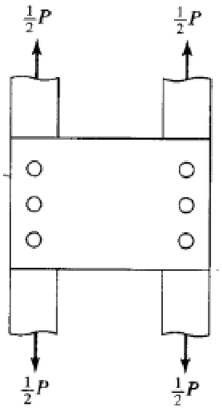
The results can be summarizes as follow:

Section	t_f	A_g Required		A_g Available	Slenderness effect	Remarks
		Yielding	Fracture			
W 8*10	0.205	2.22	3.46	2.96	N/A	**A_g Available < A_g Required---- NG**
W 8*13	0.255	2.22	3.63	3.84	N/A	**A_g Available > A_g Required---- OK**

Example 2-6: The two C12 * 30 have been selected to support a dead tensile load of 120 k and a 240 k working load. The member is 30 ft long, consists of A36 steel, and has one line of three 7/8" bolts in each channel flange 3in on center. Determine whether the member is satisfactory. Assume centers of bolts holes are 1.75 in from the backs of the channels.



Solution:



From AISC Manual for C15*30

$A_g = 8.81 in^2$, $t_f = 0.501, I_x = 162 in^4$

$I_y = 5.12 in^4$, $r_y = 0.762,$

$y - axis\ 0.674\ from\ back\ of\ C$

For A36 steel

$F_y = 50\ ksi$ and $F_u = 65\ ksi$

$P_u = 1.2D + 1.7L = 1.2*120 + 1.6*240 = 528\ kips$

- **Tensile Yielding Strength**

$\phi_t P_n = \phi_t F_y A_g = 0.9*36*(2*8.81) = 570.9 > 528 \rightarrow OK$

- **Tensile Rupture Strength**

$$A_n = 2*[8.81 - 2*(\frac{7}{8} + \frac{1}{8})*0.501 = 15.62 in^2$$

$$U = 1 - \frac{\overline{x}}{L} = 1 - \frac{0.674}{(3+3)} = 0.89"$$

$A_e = U\ A_n = 0.89*15.62 = 13.9$

$\phi_t P_n = \phi_t F_u A_e = 0.75*58*13.9 = 604.7 > 528 \rightarrow OK$

Slenderness Effect

$I_x = 2*162 = 324\ in^4$

$I_y = 2*[5.12 + 8.81*5.326^2] = 510\ in^4$

$$r_x = \sqrt{\frac{324}{2*8.81}} = 4.29\ in$$

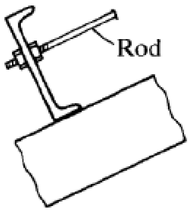
$$r_y = \sqrt{\frac{510}{2*8.81}} = 5.38\ in$$

$r_x < r_y \rightarrow r_{min} = r_x = 4.29$

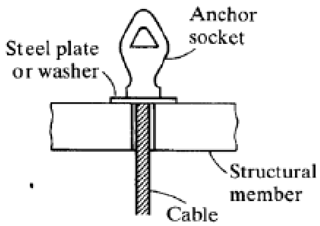
$$\frac{L}{r_x} = \frac{30*12}{4.29} = 83.9 < 300 \rightarrow OK$$

2.6 TENSION RODS

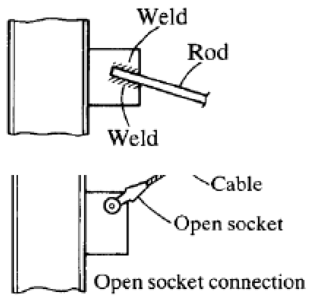
Rods with a circular cross section are commonly used as tension members when slenderness is not consideration. Tension rods might be referred to as hanger rods or sag rods. Hangers are tension members that are hung from one member to support other members. Sag rods are often provided to prevent a member from deflecting (or sagging) under its own self-weight. Tension rods are also commonly used as diagonal bracing in combination with a clevis and turnbuckle to support lateral loads.



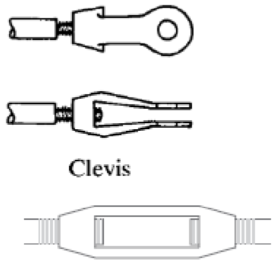
Sag rod



Hanger

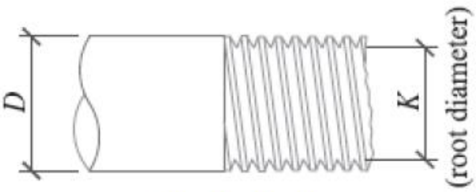


Bracing



b. turnbuckle

The more commonly used threaded rods is a rod where the nominal diameter is greater than the root diameter. The tensile capacity is based on the available cross-sectional area at the root where the threaded portion of the rod is the thinnest.



Threaded road

The AISC specification does not limit the size of tension rods, but the practical minimum diameter of the rod should not be less than 5⁄8 in. since smaller diameter rods are more susceptible to damage during construction.

The design strength of a tension rod is given in the AISC specification as

$$\phi_t P_n \geq P_u$$

$$P_n = F_n A_b$$

$$\phi_t = 0.75 \ , \ F_n = 0.75 F_u$$

A_b = Nominal unthreaded body area.

$$\therefore \ \phi_t P_n = \phi_t (0.75 F_u) A_b$$

For design

$$A_b = \frac{P_u}{0.75(0.75 F_u)}$$

The F_u term in the above equations is the minimum tensile stress of the threaded rod. There are several acceptable grades of threaded rods that are available in AISCM , the most common of which are summarized in Table below.

Grades of threaded rods

Material Specification		Diameter Range, in.	F_y, ksi	F_u, ksi
ASTM A36		Up to 10	36	58–80
ASTM A193 Gr. B7 *(corrosion-resistant)*		4 to 7	–	100
		2.5 to 4	–	115
		2.5 and under	–	125
ASTM F1554	Grade 36	0.25 to 4	36	58–80
	Grade 55	0.25 to 4	55	75–95
	Grade 105	0.25 to 3	105	125–150

Example 2-7: A threaded rod is to be used as a bracing member that must resist a service tensile load 2of 2 kips dead load and 6 kips live load. What size rod is required if A 36 steel is used?

Solution:

$$P_u = 1.2D + 1.7L = 1.2 * 2 + 1.6 * 6 = 12 \text{ kips}$$

$$A_b \text{ required} = \frac{P_u}{0.75(0.75\ F_u)}$$

$$= \frac{12}{0.75(0.75 * 58)} = 0.3678 \text{ in}$$

$$A_b = \frac{\pi}{4} d^2$$

$$\text{Re quired , } d = \sqrt{\frac{4 * 0.3678}{\pi}} = 0.684$$

Use a 3/4 in diameter rod

$$A_b = 0.442 > 0.3678 \rightarrow OK$$