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Steel Tension Member Design by ASD/LRFD Steel Construction Manual

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Steel Tension Member Design by

ASD / LRFD Steel Construction Manual

13th Edition

Tension members are found in trusses, towers, bracing members, hangers, tie rods and sag rods. Tension members can be structural steel sections, plates, built-up members, eyebars, pin-connected members, rods, bars, wire rope and steel cables. Tension members are one of the simplest to design. Tension members are now connected by bolts or welds. There a few limit states for tension members. Section D of the specifications covers *Design of Members for Tension.*

- D1. Slenderness Limitations
- D2. Tensile Strength
- D3. Area Determination
- D4. Built-up Members
- D5. Pin-connected Members
- D6 Eyebars

We will look at the first three, since they apply to most tension members.

D1. SLENDERNESS LIMITATIONS

There is no required slenderness limit but it is suggested the L/r should be less than 300. Rods and hangers in tension are exempt.

Example 1

Let's say we have an angle, L4X4X1/2 that is under tension and 26 feet long, what is it's L/r ratio?

	Shape	Weight	Area	Depth	b_f	t_w	t_f	k_{des}	$b_f/2t_f$	h/t_f
2		w	A	d						
3		lbs/ft	in^2	in	in	in	in	in		
469	L4X4X3/4	18.5	5.44	4.00	0.00	0.00	0.00	1.13	0.00	0.00
470	L4X4X5/8	15.7	4.61	4.00	0.00	0.00	0.00	1.00	0.00	0.00
471	L4X4X1/2	12.8	3.75	4.00	0.00	0.00	0.00	0.875	0.00	0.00
472	L4X4X7/16	11.3	3.31	4.00	0.00	0.00	0.00	0.813	0.00	0.00
473	L4X4X3/8	9.80	2.86	4.00	0.00	0.00	0.00	0.750	0.00	0.00
474	L4X4X5/16	8.20	2.40	4.00	0.00	0.00	0.00	0.688	0.00	0.00

	Shape	I_x	Z_x	S_x	r_x	I_y	Z_y	S_y	r_y	r_z	J
2											
3		in^4	in^3	in^3	in	in^4	in^3	in^3	in	in	in^4
469	L4X4X3/4	7.62	5.02	2.79	1.18	7.62	5.01	2.79	1.18	0.774	1.0
470	L4X4X5/8	6.62	4.28	2.38	1.20	6.62	4.28	2.38	1.20	0.774	0.61
471	L4X4X1/2	5.52	3.50	1.96	1.21	5.52	3.50	1.96	1.21	0.776	0.32
472	L4X4X7/16	4.93	3.10	1.73	1.22	4.93	3.10	1.73	1.22	0.777	0.22
473	L4X4X3/8	4.32	2.69	1.50	1.23	4.32	2.68	1.50	1.23	0.779	0.14
474	L4X4X5/16	3.67	2.26	1.27	1.24	3.67	2.26	1.27	1.24	0.781	0.08

L/r_z =(26 ft X 12 in/ft)/0.776 in=402 > 300 This would be no good.

Be sure to watch your units so they cancel.

D2. TENSILE STRENGTH

The *design tensile strength*, $\phi_t P_n$, and the *allowable tensile strength*, P_n/Ω_t, of tension members, shall be the lower 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 in the gross section:

$$P_n = F_y A_g \tag{D2-1}$$

$$\phi_t = 0.90 \text{ (LRFD)} \qquad \Omega_t = 1.67 \text{ (ASD)}$$

(b) For tensile rupture in the net section:

$$P_n = F_u A_e \tag{D2-2}$$

$$\phi_t = 0.75 \text{ (LRFD)} \qquad \Omega_t = 2.00 \text{ (ASD)}$$

where

A_e = *effective net area*, in.^{2} (mm^2)
A_g = gross area of member, in.^{2} (mm^2)
F_y = *specified minimum yield stress* of the type of steel being used, ksi (MPa)
F_u = *specified minimum tensile strength* of the type of steel being used, ksi (MPa)

Steel tension member selection tables are in part 5 of the Steel Construction Manual. The tables list the available strength for different sections. The tensile rupture strength is based on $A_e = 0.75A_g$.

- Table 5-1. W-Shapes
- Table 5-2. Single Angles
- Table 5-3. WT-Shapes
- Table 5-4. Rectangular HSS
- Table 5-5. Square HSS
- Table 5-6. Round HSS
- Table 5-7. Steel Pipe
- Table 5-8. Double Angles

These tables can be easily produced in Excel. Table 5-1

D3. AREA DETERMINATION

Gross Area

The gross area, A_g, of a member is the total cross-sectional area.

Net Area

The *net area*, A_n, of a member is the sum of the products of the thickness and the net width of each element computed as follows:

In computing net area for tension and shear, the width of a bolt hole shall be taken as $^1/_{16}$ in. (2 mm) greater than the *nominal dimension* of the hole.

For a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part shall be obtained by deducting from the gross width the sum of the diameters or slot dimensions as provided in Section J3.2, of all holes in the chain, and adding, for each *gage* space in the chain, the quantity $s^2/4g$

where

s = longitudinal center-to-center spacing (*pitch*) of any two consecutive holes, in. (mm)

g = transverse center-to-center spacing (gage) between *fastener* gage lines, in. (mm)

Effective Net Area

The effective area of tension members shall be determined as follows:

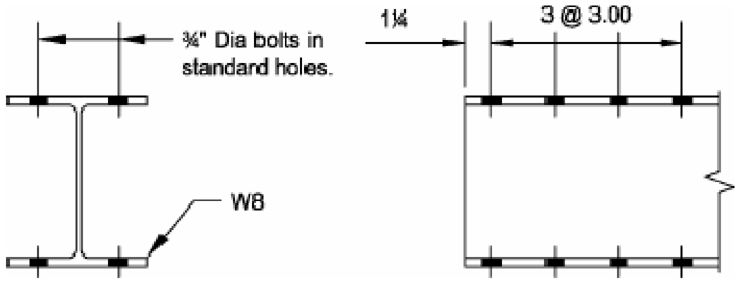
$$A_e = A_n U \qquad \text{(D3-1)}$$

where U, the shear lag factor, is determined as shown in Table D3.1.

Note: The *nominal dimension* of the hole for a standard hole is the bolt diameter plus 1/16 inch so the bolt hole shall be taken as this plus 1/16 inch or the bolt diameter plus 1/8 inch. That is ($d_{hole}+{}^1/_{16}$) or ($d_{bolt}+{}^1/_8$).

Example 2

Select an 8 inch W-shape, ASTM A992, to carry a dead load of 40 kips and a live load of 100 kips in tension. The member is 26 feet long. There are four lines of 3/4 in diameter bolts in the flanges.



Since the material is ASTM A992 then F_y=50 ksi and F_u=65 ksi.

LRFD

P_u=(1.2 X 40 kips)+(1.6 X 100 kips)

P_u=208 kips

ASD

P_a=40 kips + 100 kips

P_a=140 kips

Try a W8 X 21

Shape	Weight	Area	Depth	b_f	t_w	t_f	k_{des}	$b_f/2t_f$	h/t_f	I_x	Z_x	S_x	r_x	I_y	Z_y	S_y	r_y	r_z
	w	A	d															
	lbs/ft	in^2	in	in	in	in	in			in^4	in^3	in^3	in	in^4	in^3	in^3	in	in
W8X35	35.0	10.3	8.12	8.02	0.310	0.495	0.889	8.10	20.5	127	34.7	31.2	3.51	42.6	16.1	10.6	2.03	0.00
W8X31	31.0	9.12	8.00	8.00	0.285	0.435	0.829	9.19	22.3	110	30.4	27.5	3.47	37.1	14.1	9.27	2.02	0.00
W8X28	28.0	8.24	8.06	6.54	0.285	0.465	0.859	7.03	22.3	98.0	27.2	24.3	3.45	21.7	10.1	6.63	1.62	0.00
W8X24	24.0	7.08	7.93	6.50	0.245	0.400	0.794	8.12	25.9	82.7	23.1	20.9	3.42	18.3	8.57	5.63	1.61	0.00
W8X21	21.0	6.16	8.28	5.27	0.250	0.400	0.700	6.59	27.5	75.3	20.4	18.2	3.49	9.77	5.69	3.71	1.26	0.00
W8X18	18.0	5.26	8.14	5.25	0.230	0.330	0.630	7.95	29.9	61.9	17.0	15.2	3.43	7.97	4.66	3.04	1.23	0.00
W8X15	15.0	4.44	8.11	4.01	0.245	0.315	0.615	6.37	28.1	48.0	13.6	11.8	3.29	3.41	2.67	1.70	0.876	0.00
W8X13	13.0	3.84	7.99	4.00	0.230	0.255	0.555	7.84	29.9	39.6	11.4	9.91	3.21	2.73	2.15	1.37	0.843	0.00
W8X10	10.0	2.96	7.89	3.94	0.170	0.205	0.505	9.61	40.5	30.8	8.87	7.81	3.22	2.09	1.66	1.06	0.841	0.00

$\bar{y} = 0.831\ in\ (for\ WT4\ X\ 10.5)$

LRFD

$$\phi P_n = 0.90 F_y Ag = 0.90(50\ ksi)(6.16\ in^2) = 277\ kips$$

ASD

$$\frac{P_n}{\Omega} = \frac{F_y Ag}{1.67} = \frac{(50\ ksi)(6.16\ in^2)}{1.67} = 184\ kips$$

These are the yielding limit states. These are the same values found in my version of Table 5-1 of the Steel Manual.

Table 5-1						
F_y=	50	ksi				
F_u=	65	ksi				
			Yielding		Rupture	
			ASD	LRFD	ASD	LRFD
Shape	Ag	A_e=0.75Ag	P_n/Ω	ϕP_n	P_n/Ω	ϕP_n
	in^2	in^3	kips	kips	kips	kips
W8X40	11.7	8.78	350	527	285	428
W8X35	10.3	7.73	308	64	251	377
W8X31	9.12	6.84	273	10	222	333
W8X28	8.24	6.18	247	71	201	301
W8X24	7.08	5.31	212	319	173	259
W8X21	6.16	4.62	184	277	150	225
W8X18	5.26	3.95	157	237	128	192
W8X15	4.44	3.33	133	200	108	162
W8X13	3.84	2.88	115	173	93.6	140
W8X10	2.96	2.22	88.6	133	72.2	108
W6X25	7.34	5.51	220	330	179	268
W6X20	5.87	4.40	176	264	143	215
W6X15	4.43	3.32	133	199	108	162
W6X16	4.74	3.56	142	213	116	173
W6X12	3.55	2.66	106	160	86.5	130
W6X9	2.68	2.01	80.2	121	65.3	98
W6X8.5	2.52	1.89	75.4	113	61.4	92

Case 2, Table D3.1 says:

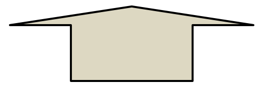
All tension members, except plates and HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or longitudinal welds (Alternatively, for W, M, S and HP, Case 7 may be used.)

The *shear lag factor*, $U = 1 - \frac{\bar{x}}{l}$

$\bar{x} = \bar{y} = 0.831\ in. for\ WT4\ X\ 10.5\ and\ l = 9\ inches, the\ ouside\ dimension\ of\ the\ bolts.$

$$U = 1 - \frac{0.831\ in}{9.00\ in} = 0.908$$

Case 7, Table D3.1 says:

W, M, S or HP Shapes or Tees cut from these shapes. (If U is calculated per Case 2, the larger value is permitted to be used)	with flange connected with 3 or more fasteners per line in direction of loading	$b_f \geq 2/3d \ldots U = 0.90$ $b_f < 2/3d \ldots U = 0.85$ 
	with web connected with 4 or more fasteners in the direction of loading	$U = 0.70$

For the W8X 21 section we have 4 bolts in the flange in the direction of loading, b_f=5.27 inches and d=8.28 inches so

$^2/_3$d = 5.52 inches which is > b_f so U=0.85. Use the larger of 0.908 or 0.85.

Calculate A_{net}

A_n=Ag-[4 holes X (d_{bolt}+$^1/_8$ inch) t_f] =6.16 in^2-[(4)($^3/_4$ in+$^1/_8$ in)(0.40 in)]

A_n=4.76 in^2

Calculate $A_{effective}$

$A_e=A_nU$=4.76 in^2 X 0.908=4.32 in^2

A_e/Ag=4.32 in^2/6.16 in^2=0.702<0.75 **so tabulated values in Table 5-1 for rupture are not applicable.**

$P_n=F_uA_e$=(65 ksi)(4.32 in^2)=281 kips

LRFD

$$\phi P_n = 0.75(281\ kips) = 211\ kips > 208\ kips = P_u\ so\ it\ is\ OK$$

ASD

$$\frac{P_n}{\Omega} = \frac{281\ kips}{2.00} = 140\ kips > 140\ kips = P_a\ \ close\ but\ it\ is\ OK$$

Check slenderness L/r<300

L = 26 feet and r_y=1.26 inches so

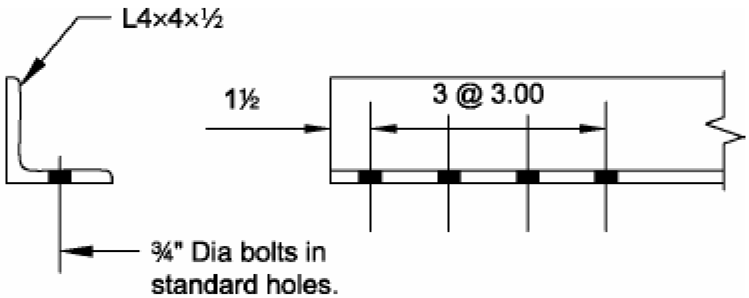
L/r=(26 ft X 12 in/ft)/1.26 in =248<300 so that is OK

Remember to watch the units so they cancel.

So the tensile rupture limit state determined the strength of this member. It is less than the yielding strength.

Example 3

Analyze the strength of an L4X4X$^{1}/_{2}$, ASTM A36 with (four) ¾ inch diameter bolts in standard holes. The member is in tension and carries a dead load of 30 kips and carries a live load of 70 kips. The member is 16 feet long.



Since it is ASTM A36, F_y=36 ksi and F_u=58 ksi

Shape	Weight	Area	Depth	k_{des}	x	y	I_x	Z_x	S_x	r_x	I_y	Z_y	S_y	r_y	r_z	J	C_w
	w	A	d														
	lbs/ft	in^2	in	in	in	in	in^4	in^3	in^3	in	in^4	in^3	in^3	in	in	in^4	in
L5X3X1/4	6.60	1.94	5.00	0.688	0.648	1.64	5.09	2.68	1.51	1.62	1.41	1.05	0.600	0.853	0.652	0.0438	0.0606
L4X4X3/4	18.5	5.44	4.00	1.13	1.27	1.27	7.62	5.02	2.79	1.18	7.62	5.01	2.79	1.18	0.774	1.02	1.12
L4X4X5/8	15.7	4.61	4.00	1.00	1.22	1.22	6.62	4.28	2.38	1.20	6.62	4.28	2.38	1.20	0.774	0.610	0.680
L4X4X1/2	12.8	3.75	4.00	0.875	1.18	1.18	5.52	3.50	1.96	1.21	5.52	3.50	1.96	1.21	0.776	0.322	0.366
L4X4X7/16	11.3	3.31	4.00	0.813	1.15	1.15	4.93	3.10	1.73	1.22	4.93	3.10	1.73	1.22	0.777	0.220	0.252
L4X4X3/8	9.80	2.86	4.00	0.750	1.13	1.13	4.32	2.69	1.50	1.23	4.32	2.68	1.50	1.23	0.779	0.141	0.162
L4X4X5/16	8.20	2.40	4.00	0.688	1.11	1.11	3.67	2.26	1.27	1.24	3.67	2.26	1.27	1.24	0.781	0.0832	0.0963
L4X4X1/4	6.60	1.94	4.00	0.625	1.08	1.08	3.00	1.82	1.03	1.25	3.00	1.82	1.03	1.25	0.783	0.0438	0.0505
L4X3-1/2X1/2	11.9	3.50	4.00	0.875	0.994	1.24	5.30	3.46	1.92	1.23	3.76	2.69	1.50	1.04	0.716	0.301	0.302

Ag=3.75 in^2, r_z=0.766 inches, $\bar{x} = \bar{y} = 1.18\, inches$

LRFD

P_u=(1.2 X 30 kips)+(1.6 X 70 kips)=148 kips

ASD

P_a=30 kips + 70 kips=100 kips

Calculate the nominal tensile yield strength

P_n=F_yAg=(36 ksi)(3.75 in^2)=135 kips

LRFD

$\phi P_n = 0.90(135\ kips) = 122\ kips\ wich\ is < Pu = 148\ kips$

$When\ \phi P_n < P_u$,we need to select a different member. The design strength is less than the required strength. Let’s select an L4X4X$^3/_4$.

Ag=5.44 in^2, r_z=0.744 in, $\bar{x} = \bar{y} = 1.27\ in.$

$$P_n = F_y Ag = (36\ ksi)(5.44\ in^2) = 196\ kips$$

LRFD

$\phi P_n = 0.9(196\ kips) = 176\ kips > P_u = 148\ kips\quad so\ this\ is\ OK$

ASD

$\frac{P_n}{\Omega} = \frac{196\ kips}{1.67} = 117\ kips > P_a = 100\ kips\quad so\ this\ is\ OK$

This agrees with my version of Table 5-2

Calculate U as the larger of the value from Table D3.1 case 2 or case 8.

Case 2

$U = 1 - \frac{\bar{x}}{l} = 1 - \left(\frac{1.27\ in}{9.00\ in}\right) = 0.859$

Case 8

Single angles (If U is calculated per Case 2, the larger value is permitted to be used)	with 4 or more fasteners per line in direction of loading	$U = 0.80$
	with 2 or 3 fasteners per line in the direction of loading	$U = 0.60$

So, U=0.80

Use U=0.859 since it is larger.

$A_n = Ag - \left[\left(d_{bolt} + {}^1/_8\,in\right)(t)\right] = 5.44\ 10^2 - \left[\left({}^3/_4\,in + {}^1/_8\,in\right)\left({}^3/_4\,in\right)\right] = 4.784\ in^2$

$A_e = A_n U = 4.784\ in^2(0.859) = 4.109\ in^2$

$P_n = F_u A_e$=(58 ksi)(4.109 in^2)=238 kips

LRFD

$\varphi P_n = 0.75(238 \text{ kips}) = 179 \text{ kips} > P_u = 148 \text{ kips}$ so this is OK

ASD

$P_n/\Omega = (238 \text{ kips})/2.00 = 119 \text{ kips} > P_a = 100 \text{ kips}$ so this is OK

For this tension member, the yielding strength was less than the rupture strength so the yielding limit state controls the strength of this member.

Next calculate L_{max}

$L_{max} = 300 r_z = 300(0.744 \text{ in})(\text{ft}/12 \text{ in}) = 18.6 \text{ ft}$

Now we have looked at:

D1. Slenderness Limitations

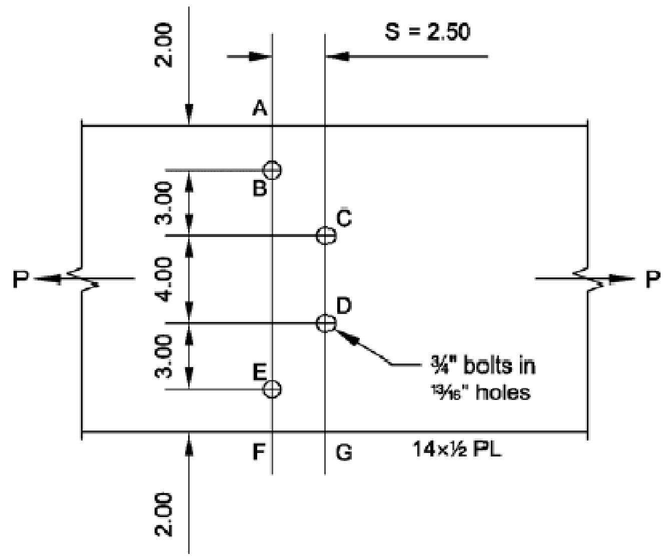
D2. Tensile Strength

a. Tensile yielding in the gross section
b. Tensile rupture in the net section

We looked at how this specification applied to a wide flange member and an angle. Specification D3, Area Determination tells what we do with *staggered holes*. We are to add $s^2/4g$, to determine the net area.

> For a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part shall be obtained by deducting from the gross width the sum of the diameters or slot dimensions as provided in Section J3.2, of all holes in the chain, and adding, for each *gage* space in the chain, the quantity $s^2/4g$
>
> where
>
> s = longitudinal center-to-center spacing (*pitch*) of any two consecutive holes, in. (mm)
>
> g = transverse center-to-center spacing (gage) between *fastener* gage lines, in. (mm)

Example 4



The *pitch*, s, is 2.50 inches. The *gage*, g, is 3.00 inches between A-C and D-E but it is 4.00 inches between C-D. The d_{hole} = $^3/_4$ in. + $^1/_8$ in = 0.875 in. Ag=14 in X ½ in = 7.0 in^2. To compute the net area, A_n, compute the net width, w_n, and multiply it by the thickness.

The net width is:

$$w_{net} = 14.0\ in - \sum d_{holes} + \sum \frac{s^2}{4g}$$

Line A-B-E-F: $w = 14.0\ in - (2\ holes\ X\ 0.875\ in) = 12.2\ in$

Line A-B-C-D-E-F:

$$w = 14.0\ in - (4\ holes\ \times 0.875\ in) + \left(2 \times \left(\frac{(2.50\ in)^2}{4 \times 3.00\ in}\right)\right) = 11.5\ in\ \ (\boldsymbol{use\ smallest})$$

Line A-B-C-D-G: $w = 14.0\ in - (3\ holes\ \times 0.875\ in) + \left(\frac{(2.50\ in)^2}{4 \times 3.00\ in}\right) = 11.9\ in$

Line A-B-D-E-F:

$$w = 14.0\ in - (3 \times 0.875\ in) + \left(\frac{(2.50\ in)^2}{4 \times 7.00\ in}\right) + \left(\frac{(2.50\ in)^2}{4 \times 3.00\ in}\right) = 12.1\ in$$

A_n= 11.5 in X 0.500 in=5.77 in^2

Table D3.1, Case 1 says U=1.0 so A_e=A_nU=5.77 in^2 x 1.0=5.77 in^2

Now, if the material is ASTM A36, F_y=36 kis and F_u=58 ksi.

Yielding

$P_n=F_yAg=(36\ ksi)(7.0\ in^2)=252\ kips$

LRFD

$\varphi P_n=0.9(252\ kips)=226\ kips$

ASD

$P_n/\Omega=(252\ kips)/1.67=151\ kips$

Rupture

$P_n=F_uA_e=(58\ ksi)(5.77\ in^2)=335\ kips$

LRFD

$\varphi P_n=0.75(335\ kips)=251\ kips$

ASD

$P_n/\Omega=(335\ kips)/2.00=167\ kips$

In both LRFD and ASD, yielding controls the strength of this connection. By LRFD, the strength is 226 kips and by ASD, the strength is 151 kips. *The designer should decide which, LRFD or ASD is to me used on the entire project and should not hop back and forth between the two. I am working out all problems by both methods for illustration purposes.*

Now we have looked at only bolted connections in tension and not welded connections in tension. There are three other modes of failure or limit states for bolted connections in tension. Since the connection is in tension, the bolts could shear so we could have a *shear failure* of the bolts. Also the bolts bear on the tension member so there could be a *bearing failure* of the bolts on the tension member. There could also be a *block shear failure* where a block of the tension member tears away. Some portions of this block are in tension and some are in shear so this is a combination of shear and tension that result in this type of failure.

In part J3 of the specification it states:

Bolts are permitted to be installed to only the snug-tight condition when used in

(a) *bearing-type connections.*

(b) tension or combined shear and tension applications, for ASTM A325 or A325M bolts only, where loosening or *fatigue* due to vibration or *load* fluctuations are not design considerations.

The snug-tight condition is defined as the tightness attained by either a few impacts of an impact wrench or the full effort of a worker with an ordinary spud wrench that brings the connected plies into firm contact. Bolts to be tightened only to the snug-tight condition shall be clearly identified on the design and erection drawings.

Table J3.2 says that an A325 N bolt, when the threads are not excluded from the shear plane, the *nominal shear stress*, F_{nv}=48 ksi and for an A325 X, when the threads are excluded from the shear plane, the *nominal shear stress*, F_{nv}=60 ksi.

J3.6

Tension and Shear Strength of Bolts and Threaded Parts

The *design tension* or *shear strength*, ϕR_n, and the *allowable tension* or *shear strength*, R_n/Ω, of a snug-tightened or pretensioned high-strength bolt or threaded part shall be determined according to the *limit states* of *tensile rupture* and *shear rupture* as follows:

$$R_n = F_n A_b \quad \text{(J3-1)}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

where

F_n = nominal tensile stress F_{nt}, or shear stress, F_{nv} from Table J3.2, ksi (MPa)

A_b = nominal unthreaded body area of bolt or threaded part (for upset rods, see footnote d, Table J3.2), in.^{2} (mm^2)

Section J3.3 of the specifications list minimum spacing requirements of bolts.

Minimum Spacing

The distance between centers of standard, oversized, or slotted holes, shall not be less than 2 $^2/_3$ times the nominal diameter, d, of the *fastener*; a distance of $3d$ is preferred.

Table J3.4, list minimum edge distances in inches from the center of a standard hole the edge of the connection part.

Bolt Diameter (in)	At Sheared Edge (in)	At Rolled Edge (in)
1/2	7/8	3/4
5/8	1 1/8	7/8
3/4	1 1/4	1
7/8	1 1/2	1 1/8
1	1 3/4	1 1/4
1 1/8	2	1 1/2
1 1/4	2 1/4	1 5/8
Over 1 1/4	1 3/4 X d	1 1/4 X d

J3.10

Bearing Strength at Bolt Holes

The *available bearing strength*, ϕR_n and R_n/Ω, at bolt holes shall be determined for the *limit state* of *bearing* as follows:

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

(a) For a bolt in a *connection* with standard, oversized, and short-slotted holes, independent of the direction of loading, or a long-slotted hole with the slot parallel to the direction of the bearing *force*:

(i) When deformation at the bolt hole at *service load* is a design consideration

$$R_n = 1.2\, L_c t F_u \leq 2.4\, d t F_u \qquad \text{(J3-6a)}$$

(ii) When deformation at the bolt hole at service load is not a design consideration

$$R_n = 1.5\, L_c t F_u \leq 3.0\, d t F_u \qquad \text{(J3-6b)}$$

where

d = nominal bolt diameter, in. (mm)
F_u = *specified minimum tensile strength* of the connected material, ksi (MPa)
L_c = clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, in. (mm)
t = thickness of connected material, in. (mm)

J4.3

Block Shear Strength

The *available strength* for the *limit state* of *block shear rupture* along a shear failure path or path(s) and a perpendicular tension failure path shall be taken as

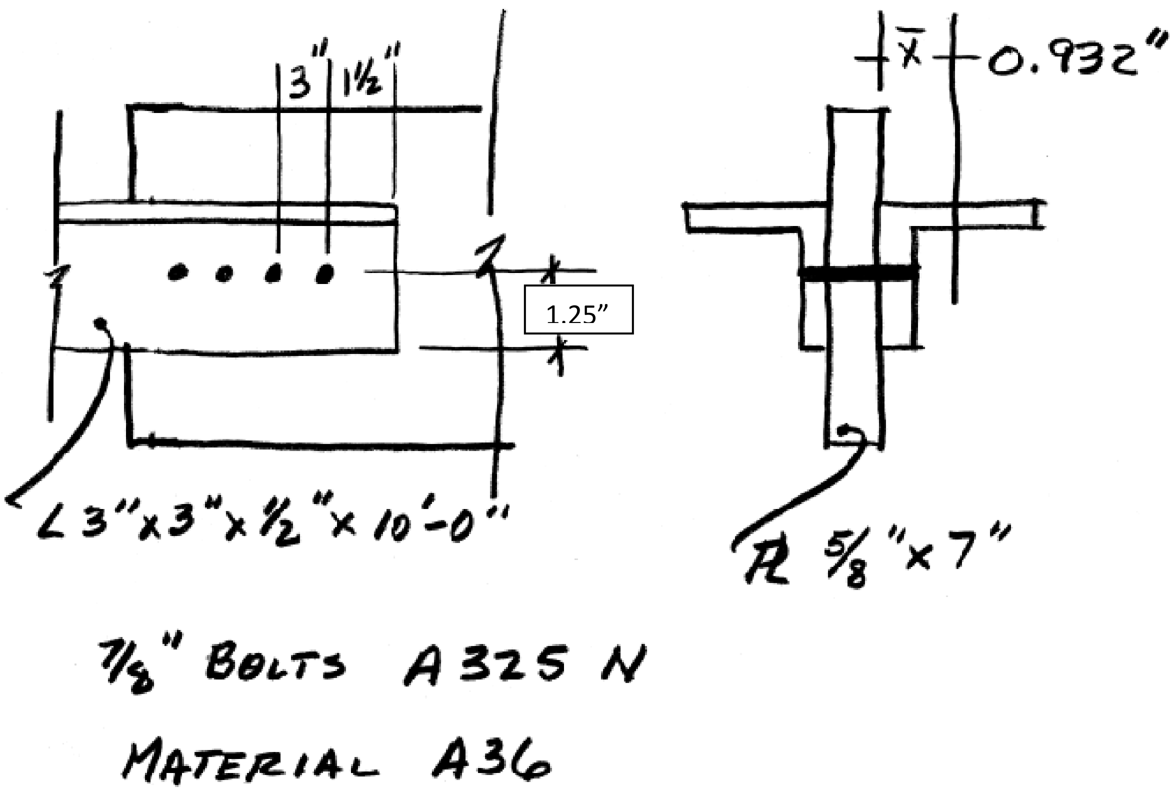
$$R_n = 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} \qquad \text{(J4-5)}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

where

A_{gv} = gross area subject to shear, in.^{2} (mm^2)
A_{nt} = *net area* subject to tension, in.^{2} (mm^2)
A_{nv} = net area subject to shear, in.^{2} (mm^2)

Example 5



We will look at each of the following:

- **Gross Section Yielding**
- **Tensile Rupture Strength**
- **Shearing Strength of Bolts**
- **Bearing Strength of Bolts**
- **Block Shear**

Gross Section Yielding

Plate

$A_g = {}^{5}/_{8}$ in X 7 in=4.375 in^2

$P_n = F_y A_g = (36 \text{ ksi})(4.375 \text{ in}^2) = 158$ kips

LRFD

φP_n=0.9(158 kips)=142 kips

ASD

P_n/Ω=158 kips/1.67=94.6 kips

Gross Section Yielding

Angles

Shape	Weight	Area	Depth	b_f	t_w	t_f	k_{des}	$b_f/2t_f$	h/t_f	I_x	Z_x	S_x	r_x	I_y	Z_y	S_y	r_y	r_z
	w	A	d															
	lbs/ft	in^2	in	in	in	in	in			in^4	in^3	in^3	in	in^4	in^3	in^3	in	in
L3-1/2X2-1/2X1/2	9.40	2.75	3.50	0.00	0.00	0.00	0.875	0.00	0.00	3.24	2.52	1.41	1.08	1.36	1.39	0.756	0.701	0.532
L3-1/2X2-1/2X3/8	7.20	2.11	3.50	0.00	0.00	0.00	0.750	0.00	0.00	2.56	1.96	1.09	1.10	1.09	1.07	0.589	0.716	0.535
L3-1/2X2-1/2X5/16	6.10	1.78	3.50	0.00	0.00	0.00	0.688	0.00	0.00	2.20	1.67	0.925	1.11	0.937	0.900	0.501	0.723	0.538
L3-1/2X2-1/2X1/4	4.90	1.44	3.50	0.00	0.00	0.00	0.625	0.00	0.00	1.81	1.36	0.753	1.12	0.775	0.728	0.410	0.731	0.541
L3X3X1/2	9.40	2.75	3.00	0.00	0.00	0.00	0.875	0.00	0.00	2.20	1.91	1.06	0.895	2.20	1.91	1.06	0.895	0.580
L3X3X7/16	8.30	2.43	3.00	0.00	0.00	0.00	0.813	0.00	0.00	1.98	1.70	0.946	0.903	1.98	1.70	0.946	0.903	0.580
L3X3X3/8	7.20	2.11	3.00	0.00	0.00	0.00	0.750	0.00	0.00	1.75	1.48	0.825	0.910	1.75	1.48	0.825	0.910	0.581
L3X3X5/16	6.10	1.78	3.00	0.00	0.00	0.00	0.688	0.00	0.00	1.50	1.26	0.699	0.918	1.50	1.25	0.699	0.918	0.583
L3X3X1/4	4.90	1.44	3.00	0.00	0.00	0.00	0.625	0.00	0.00	1.23	1.02	0.569	0.926	1.23	1.02	0.569	0.926	0.585
L3X3X3/16	3.71	1.09	3.00	0.00	0.00	0.00	0.563	0.00	0.00	0.948	0.774	0.433	0.933	0.948	0.774	0.433	0.933	0.586
L3X2-1/2X1/2	8.50	2.50	3.00	0.00	0.00	0.00	0.875	0.00	0.00	2.07	1.86	1.03	0.910	1.29	1.34	0.736	0.718	0.516

$A_g=2.75\ in^2$ X 2 angles=$5.50\ in^2$

$P_n=F_yA_g$=(36 ksi)($5.50\ in^2$)=198 kips

LRFD

φP_n=0.9(198 kips)=178 kips

ASD

P_n/Ω=(198 kips)/1.67=119 kips

These values check with Table 5-8 in the Steel Construction manual.

So far, the plate should fail before the two angles. While we are at it, let's check L/r. L=10 feet and r_z=0.580 inches.

L/r_z=(10 ft X 12 in/ft)/0.580 in=207<300 so this is OK. Now let's check the tensile rupture strength.

Tensile Rupture Strength

Plate

A_n=Ag-hole=4.375 in^2-[($^7/_8$ in-$^1/_8$ in)($^5/_8$ in)]=3.75 in^2

U=1.00

$P_n=F_uA_nU$=(58 ksi)(3.75 in^2)(1.00)=218 kips

LRFD

φP_n=0.75(218 kips)=164 kips

ASD

P_n/Ω=(218 kips)/2.00=109 kips

Tensile Rupture Strength

Angles

A_n=Ag-2 holes=5.50 in^2-[(2 holes)($^7/_8$ in +$^1/_8$ in)($^1/_2$ in thick)=4.50 in^2

$$\bar{x} = 0.929\ in$$

$$U = 1 - \frac{\bar{x}}{L} = 1 - \frac{0.929\ in}{9.00\ in} = 0.897$$

$A_e=A_nU$=4.50 in^2(0.897)=4.03 in^2

$P_n=F_uA_e$=(58 ksi)(4.03 in^2)=234 kips

LRFD

φP_n=0.75(234 kips)=175 kips

ASD

P_n/Ω=(234 kips)/2.00=117 Kips

Again, the plate is the weaker member. However, gross section yielding (LRFD of 142 kips and ASD of 94.6 kips) is less than the tensile rupture strength. Next, let’s check the shear strength of the bolts.

Shearing Strength of Bolts

A325 N, F_{nv}=48 ksi per Table J3.2

R_n=$F_{nv}A_b$(N, number of shear planes)=$48\ ksi\left(\frac{\pi\left(\frac{7}{8}in\right)^2}{4}\right)(2) = 57.7\ kips/bolt$

LRFD

φR_n=0.75(57.7 kips)(4bolts)=173 kips

ASD

R_n/Ω=(57.7 kips)(4 bolts)/2.00=115 kips

Still, the tensile rupture strength of the plate controls the strength of this connection. Next let's look at the bearing strength of the bolts when the deformation at the bolt hole at *service load* is a design consideration.

Bearing Strength of Bolts on Plate

L_c is the clear distance in the direction of the force, between the edge of the hole and the edge of the adjacent hole or the edge of the material, whichever is less.

L_c=center to center dimension of holes – a hole

L_c=3.00 in – ($^7/_8$ in +$^1/_8$ in)=2.00 in

L_c=edge to center dimension – half a hole

L_c=1.50 in-[0.5 X 1.00 in]=1.00 in which is less so we use 1.00 in.

R_n=1.2 $L_c t F_u N \leq 2.4\ d t F_u N$ N is number of bolts

R_n=(1.2)(1.00 in)($^5/_8$ in)(58 ksi)(4 bolts)=174 kips *use this value*

R_n=(2.4)($^7/_8$ in)($^5/_8$ in)(58 kis)(4 bolts)=304 kips

LRFD

φP_n=0.75(174 kips)=130 kips

ASD

P_n/Ω=(174 kips)/2.00=87 kips

Now, the bearing strength of the bolts on the plate control the strength of this connection. Next we will look at *block shear*.

Block Shear

$R_n=0.6\ F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.6\ F_yA_{gv} + U_{bs}F_uA_{nt}$

A_{nt} = [1.25 in-(half hole)] X (thickness)

A_{nt} = [1.25 in-(.5)($^7/_8$ in+$^1/_8$ in)] X ($^1/_2$ in)=0.375 in^2

A_{nv} = [10.5 in – (3.5 holes)] X thickness

A_{nv} = [10.5 in – (3.5 X 1.00 in)] X 0.50 in=3.50 in^2

A_{gv} = (10.5 in)(0.50 in)=5.25 in^2

U_{bs} = 1.00

$R_n = 0.6\ F_uA_{nv} + U_{bs}F_uA_{nt}$

R_n = [0.6(58 ksi)(3.50 in^2)]+[1.00(58 ksi)(0.375 in^2)]=144 kips

$R_n = 0.6\ F_yA_{gv} + U_{bs}F_uA_{nt}$

R_n = [0.6(36 ksi)(5.25 in^2)] + [1.00(58 ksi)(0.375 in^2)]=135 kips   *use this*

LRFD

φP_n=2 angles(0.75)(135 kips)=203 kips

ASD

P_n/Ω=(2 angles X 135 kips)/2.00=135 kips

Block shear is really not a problem for this connection. So the strength of this connection is governed by *bearing strength of the bolts on the plate*. By LRFD, the strength is 130 kips and by ASD, the strength is 87 kips. So, how would we increase the strength of this connection? We could use A325 X which excludes the threads from the shear plane. That would increase F_{nv}=60 ksi or we could increase the diameter of the bolt. If *gross section yielding* was a problem, then increase the gross area. If *tensile rupture strength* was the problem, then increase the net area. If *bearing strength* was the problem, then increase thickness, diameter of the bolts, dimensions and number of bolts. If *block shear* was the problem, then increase the number of bolts and dimensions.

SUMMARY

		LRFD	ASD
Gross section yielding			
	Plate		
		142 k	94.6 k
	Angles		
		178 k	119 k
L/r	OK		
Tensile rupture strength			
	Plate		
		164 k	109 k
	Angles		
		175 k	117 k
Shear strength of bolts			
		173 k	115 k
Bearing strength of bolts on plate			
		130 k	87 k
Block shear strength			
		203 k	135 k

To design tension members, you will need to choose the type of members to be used for the connection like W-shapes, double angles, WT-shapes, etc. You will also need to choose to bolt or weld the connection. Compute the min Ag, min A_e, min A_n and the min r. Use these values to select a member. Check the limit states to verify that the members and connection will work.

LRFD

min $Ag=P_u/\varphi F_y$

min $A_e=P_u/\varphi F_u$

min $A_n=P_u/\varphi F_u U$

min $r=L/300$

ASD

min $Ag=\Omega P_a/F_y$