Design of steel structure

References: -

1- "Steel Design" 6th edition byWilliam T. Segui
2- Bresler and lin: "design of steel structures".
3- J McCormac: "structural steel design".
4- J Bowels: "structural steel Design".
5- Charles G. Salmon: "steel structures" design and behavior.
6- AISC manual: "manual of steel construction".

Syllabus: -

1- Introduction, material and properties.
2- Tension member.
3- Compression member.
4- Truss, Purlins, Joint construction.
5- Columns.
6- Column base plates and beam bearing plates.
7- Anchor bolts.

Why do we design steel structures?

1- It takes less time to construction.
2- It is easier to construct.
3- It can be constructed under any weather conditions.
4- Used for long spans.
5- Has high strength.
6- Performance.
7- Ductile.
8- Occupy less area.
9- Less dead loads.
10- Used for structures subjected to vibration.

Disadvantages: -

1- Cost of maintenance.
2- Cost of fire proofing.
3- Stiffening against buckling.

Design procedure: -

1- Functional planning.
2- Preliminary structural arrangement.
3- Establishment of loads.
4- Preliminary member selection.
5- Structural analysis to ascertain whether member selected are safe but not excessively so.
6- Evaluation: are all requirements are satisfied?
7- Redesign.

Structural loads:-

1. D.L = wt. of the structure.
2. L.L = human occupants, furniture's, movable loads.
3. Impact = dynamic effect.
4. Snow load = $20\frac{\mathbf{Ib}}{\mathbf{ft^2}}$
5. Wind load.
6. Earthquake.
7. Temperatures.

1- Dead loads (DL):-

It is a fixed position gravity load, the weight of the structure is considered dead load, as well as attachments such as pipes, electric conduits …….. .

2- Live loads (LL):-

Gravity loads acting when the structure is in service, but varying in magnitude & location.
The (LL) is prescribed by state & local building codes.

3- Wind load (WL):-

All structures are subject to wind load, which is usually considered for buildings more than 3-4 stories high and large bridge.
It is in the form of pressure on the wind word and suction on the Leeward side.

$q = \frac{1}{2}\rho v^2$

$$q = \text{dynamic pressure on the object (Psf)}$$
$$\rho = \text{mass desity of air}$$
$$V = \text{wind velocity (mile per hour)}$$

$q == 0.0026\,(Kz)(Kzt)(Kd)V^2\,I$ Ib/ft^2 ASCE 7-02

"q" is converted into equivalent static pressure "p":-

Snow load, gravity load:-

- For steeper slopes – Less snow can accumulate.
- Full or partial snow load can be considered.
- Basic snow Load 30-40 Psf.

Earthquake Load:-

Consists of horizontal & vertical ground motions.

Impact:-

Refers to dynamic effect of a suddenly applied load.
Like crane & various types of machinery.

Temperate effort:-

Coef. of expansion $=11.25 \times 10^{-6}$ per ℃

$$\Delta L = \alpha(\pm \Delta T)L$$

Structural steel: -

- ❖ Steel is an alloy of primarily iron, carbon and small amount of other components (manganese, nickel……..).
- ❖ Carbone contributes to strength but reduce ductility.
- ❖ **The important characteristics of steel for design are: -**

1- Yield stress F_y.
2- F_u.
3- Modulus of elasticity E_s.
4- Coefficient of thermal expansion. (**α**)

Steel tests: -

1- Tensile test.

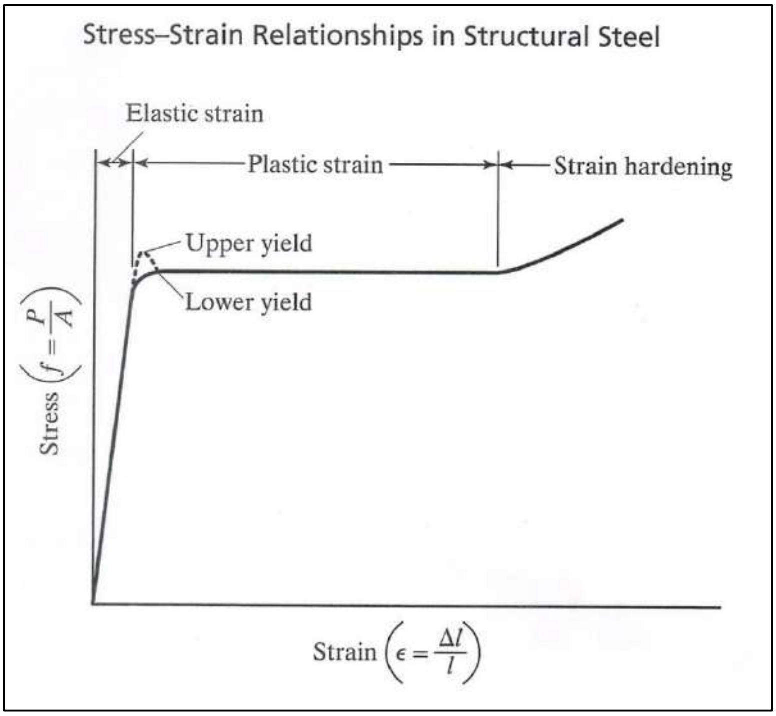
2- Chemical test.
3- Impact test.
4- Hardness test.
5- X-ray test.

❖ Tensile test: -

Stress = $\frac{\mathbf{P}}{\mathbf{A}}$

Strain = $\frac{l_u - l_0}{l_0}$

l_u is the new length.



ASTM designation:

❖ Structural steel is classified as: -

1- Carbon steel.
2- High strength low alloy steels.
3- Corrosion resistance, high strength, low alloy.

Table 2–3
Applicable ASTM Specifications for Various Structural Shapes

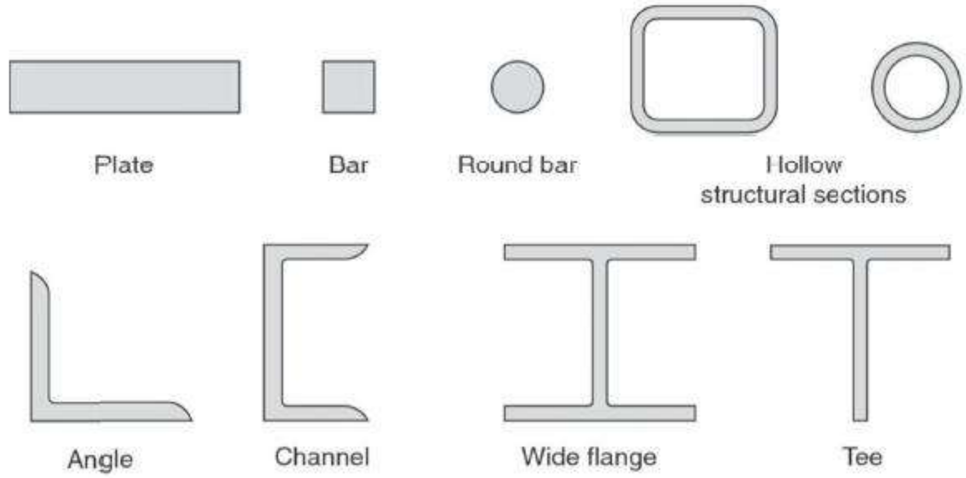
Steel Type	ASTM Designation		F_y Min. Yield Stress (ksi)	F_u Tensile Stress[a] (ksi)	Applicable Shape Series: W	M	S	HP	C	MC	L	HSS Rect.	HSS Round	Pipe
Carbon	A36		36	58-80[b]	▒	■	■	▒	■	■	■			
	A53 Gr. B		35	60										■
	A500	Gr. B	42	58									■	
			46	58								■		
		Gr. C	46	62									▒	
			50	62								▒		
	A501		36	58								▒	▒	
	A529[c]	Gr. 50	50	65-100	▒	▒	▒	▒	▒	▒	▒			
		Gr. 55	55	70-100	▒	▒	▒	▒	▒	▒	▒			
High-Strength Low-Alloy	A572	Gr. 42	42	60	▒	▒	▒	▒	▒	▒	▒			
		Gr. 50	50	65[d]	▒	▒	▒	■	▒	▒	▒			
		Gr. 55	55	70	▒	▒	▒	▒	▒	▒	▒			
		Gr. 60[e]	60	75	▒	▒	▒	▒	▒	▒	▒			
		Gr. 65[e]	65	80	▒	▒	▒	▒	▒	▒	▒			
	A618[f]	Gr. I & II	50[g]	70[g]								▒	▒	
		Gr. III	50	65								▒	▒	
	A913	50	50[h]	60[h]	▒	▒	▒	▒	▒	▒	▒			
		60	60	75	▒	▒	▒	▒	▒	▒	▒			
		65	65	80	▒	▒	▒	▒	▒	▒	▒			
		70	70	90	▒	▒	▒	▒	▒	▒	▒			
	A992		50-65[i]	65[i]	■	▒	▒	▒	▒	▒	▒			
Corrosion Resistant High-Strength Low-Alloy	A242		42[j]	63[j]	▒									
			46[k]	67[k]	▒			▒			▒			
			50[l]	70[l]	▒	▒	▒	▒	▒	▒	▒			
	A588		50	70	▒	▒	▒	▒	▒	▒	▒			
	A847		50	70								▒	▒	

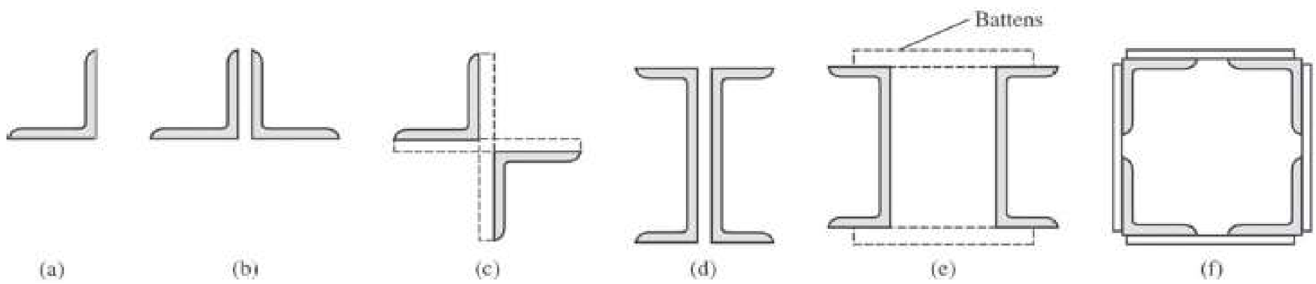
■ = Preferred material specification.
▒ = Other applicable material specification, the availability of which should be confirmed prior to specification.
☐ = Material specification does not apply.

[a] Minimum unless a range is shown.
[b] For shapes over 426 lb/ft, only the minimum of 58 ksi applies.
[c] For shapes with a flange thickness less than or equal to 1½ in. only. To improve weldability a maximum carbon equivalent can be specified (per ASTM Supplementary Requirement S78). If desired, maximum tensile stress of 90 ksi can be specified (per ASTM Supplementary Requirement S79).
[d] If desired, maximum tensile stress of 70 ksi can be specified (per ASTM Supplementary Requirement S91).
[e] For shapes with a flange thickness less than or equal to 2 in. only.
[f] ASTM A618 can also be specified as corrosion-resistant; see ASTM A618.
[g] Minimum applies for walls nominally ¾-in. thick and under. For wall thicknesses over ¾ in., F_y = 46 ksi and F_u = 67 ksi.
[h] If desired, maximum yield stress of 65 ksi and maximum yield-to-tensile strength ratio of 0.85 can be specified (per ASTM Supplementary Requirement S75).
[i] A maximum yield-to-tensile strength ratio of 0.85 and carbon equivalent formula are included as mandatory in ASTM A992.
[j] For shapes with a flange thickness greater than 2 in. only.
[k] For shapes with a flange thickness greater than 1½ in. and less than or equal to 2 in. only.
[l] For shapes with a flange thickness less than or equal to 1½ in. only.

Shape	Preferred Steel
Angles	A36
Plates	A36
S, M, C, MC	A36
HP	A572 Grade 50
W	A992
Pipe	A53 Grade B (only choice)
HSS	A500 Grade C, F_y = 46 ksi (round) or A500 Grade C, F_y = 50 ksi (rectangular)

Structural shapes:



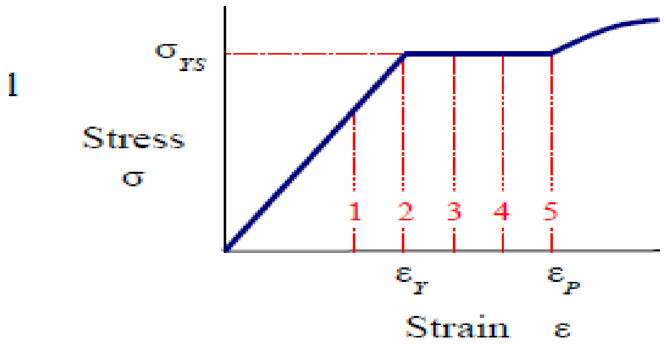


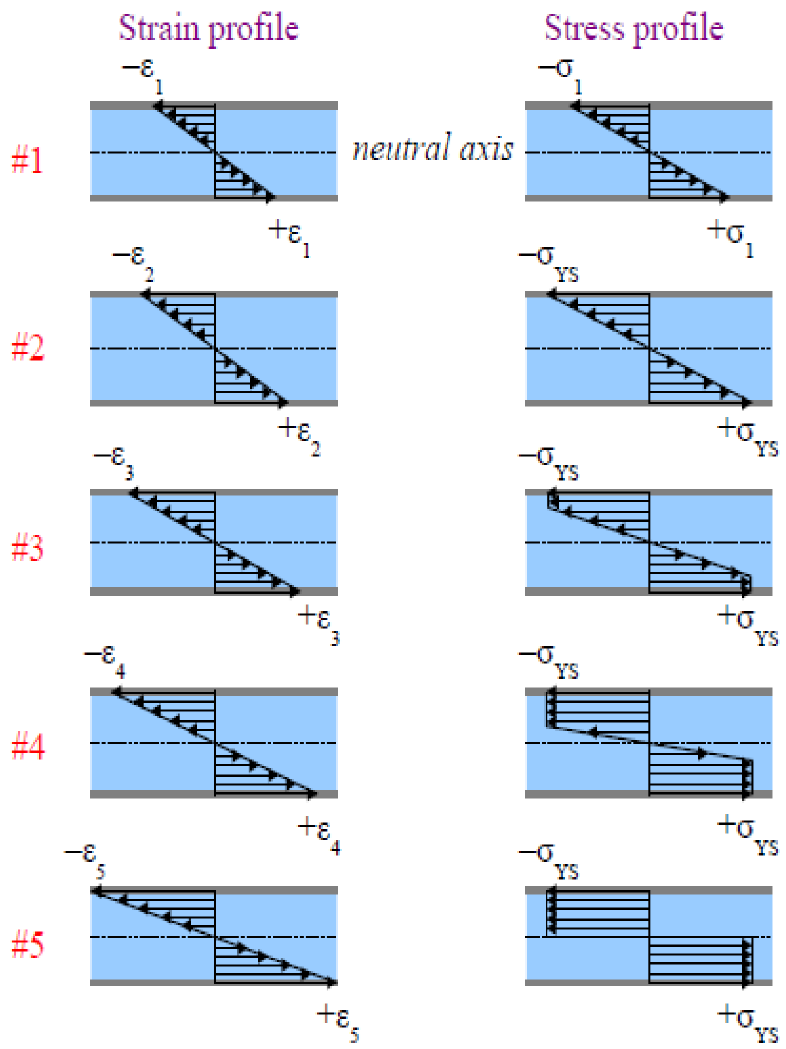
Design criteria:

1- Elastic method. (Allowable strength design (ASD))

2- Load and resistance factor design LRFD.

3- Plastic method.





1- **Elastic design method (working stress design W.S.D):** Safety is obtained by specifying that the effect of loadings should produces stresses that is a fraction of yield stress.

Max stress due to working loads does not exceed allowable stress.

$$fb = \frac{mc}{I} \leq F_b = \frac{F_y}{f.s} \quad \rightarrow \text{AISC } 1989 \rightarrow F.s = 1.67$$

calculated stress allowable stress

So, $F_b = 0.6F_y$ allowable stress $M = F_b \frac{I}{C}$

$$M = F_b\ S$$

Factor of safety: -

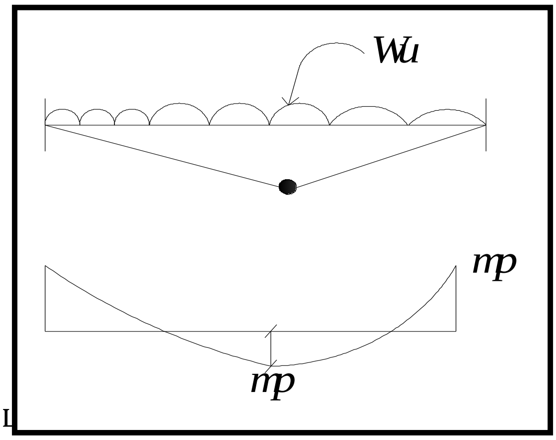
Strength of any member > the expected force carried by the member.

$$F.s = \frac{Fy}{\text{Pemissible stress}}$$

2- **Plastic design:** The strength or safety is dealt with applying factors to the loading.

$M(F_s) \leq M_u$

actual plastic strength can be achieved



$$\therefore M_p = z\ fy$$

Z = plastic section modulus

$\frac{bd^2}{4}$ for rectangular shape.

See part 2 of “AISC” 1989 uses Fs = 1.7 or 1.3

$$w_u = 1.7\ (W_{D.L} + W_{L.L}$$

$$w_u = 1.3\ (W_{D.L} + W_{L.L} + W_{W.L} \text{ or } W_E)$$

LRFD: The basic LRFD provision is provided in Section B3.1 of the Specification as

Ru ≤ φRn (AISC B3-1)-

Factored strength > factored loads

Σ (Loads *factors) ≤ resistance *R factor

Each load has a different factor.

- ❖ Resistance factor is introduced to account for uncertainties in materials strengths, dimensions, and workmanship.

$U1 \ = \ 1.4\ D$

$U\ 2 = \ 1.2\ D + 1.6\ L + 0.5(L_r \text{ or } S \text{ or } R)$

$U3 \ = \ 1.2\ D + 1.6\ (L_r \text{ or } S \text{ or } R) + 0.5\ L \text{ or } 0.5\ w$

$U\ 4 = \ 1.2\ D + W + \ \ 0.5(L_r \text{ or } S \text{ or } R)$

$U5 \ = \ 0.9\ D + W$

In combinations 3 and 4, the load factor on L should be increased from 0.5 to 1.0 if L is greater than 100 pounds per square foot and for garages or places of public assembly.

In combinations with wind load, you should use a direction that produces the worst effect.

In all load combinations, the load factor for a certain load effect is not the same. For example, the load factor for the live load L in combination 2 is 1.6, but it is 0.5 in combination 3. The reason for this is that in combination 2, the live load is assumed to be the major effect, whereas in combination 3, one of the three effects, Lr, S, or R, will be dominant.

<u>Example :</u>

A column (compression member) in the upper story of a building is subject to the following loads:

Dead load:	109 kips compression
Floor live load:	46 kips compression
Roof live load:	19 kips compression
Snow:	20 kips compression

Determine the controlling load combination for LRFD and the corresponding factored load.

Combination 1: 1.4D = 1.4(109) 5 152.6 kips

Combination 2: 1.2D + 1.6L + 0.5(Lr or S or R). Because S is larger than Lr and R = 0, we need to evaluate this combination only once, using S.

1.2D + 1.6L + 0.5S = 1.2(109) + 1.6(46) + 0.5(20) = 214.4 kips

Combination 3: 1.2D 1 1.6(Lr or S or R) 1 (0.5L or 0.5W). In this combination, we use S instead of Lr , and both R and W are zero.

1.2D + 1.6S + 0.5L = 1.2(109) + 1.6(20) + 0.5(46) = 185.8 kips

Combination 4: 1.2D + 1.0W + 0.5L + 0.5(Lr or S or R). This expression reduces to 1.2D + 0.5L + 0.5S, and by inspection, we can see that it produces a smaller result than combination 3.

Combination 5 does not apply in this example, because there is no wind load to counteract the dead load.

<u>Combination 2 controls, and the factored load is 214.4 kip</u>

Both ASD, LRFD procedures are based on limit states design principles which provide the boundaries of structural usefulness.

- ❖ The term limit state is used to describe a condition of which a structure or a part of a structure ceases to perform its intended function.
- ❖ **There are two categories of limit states: -**

1. Strength.
2. Serviceability.

Strength →load caring capacity, yielding, fracture… .

Serviceability → deflection – cracking - ……….. .

- ❖ All limit states should be prevented.

- ❖ Computation of loads for LRFD & ASD.

1. LRFD: -

(Reduction factor ϕ)(nominal strength of member ≥ computed factored force in member R_u .

$$\phi R_n \geq R_u$$

where R_n is nominal strength (theoretical strength)with no safety factors or resistance factors.

(ϕ) is a resistance factor or reduction factor, varies as follow:

Limit state of	ϕ	*AISC 2005*
Tensile yielding	0.9	D-2
Tensile rupture	0.75	D-2
Compression	0.9	E1
Beam- flexure	0.9	F1 &E1
Beam- shear	0.9	F1 &E1
Weld	Same as for type of action	J2-5
Fastener	0.75	J3-6 J3-7

2. ASD: -

$$\frac{\text{strength of member}}{\text{Factor of safety } \Omega} \geq \text{largest computed force } R_a$$

$$\frac{R_n}{\Omega} \geq R_a$$

❖ Steel designations: -

W 24-305 → wt. of beam in$\frac{\mathbf{lb}}{\mathbf{ft}}$

S 20 – 150 → wt.

Standard section

❖ Specifications:-

1. AISC 2005
2. AREA
3. AASHTO

- We shall use (AISC 2005)

Chapter one

Tension members

- ❖ Tension members are encountered in most steel structures.
- ❖ Tension members may consist of a single structural shape or they may be built up from a number of structural shapes.

Built up section will be required when: -

a. Tensile capacity of a single rolled section is not sufficient.
b. The slenderness ratio does not provide sufficient rigidity.

Strength as a design criterion: -

- ❖ Tension member without hole (such as welded connection) achieves its strength when all fibers of the cross section have yielded.

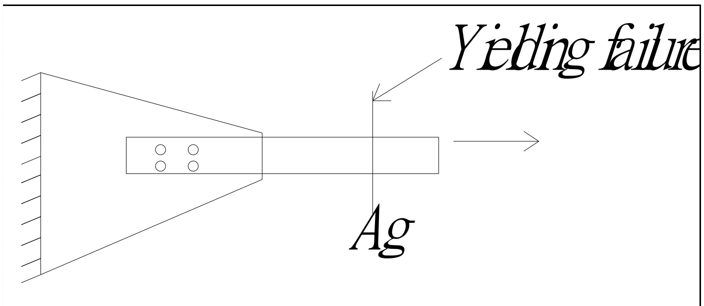
i.e. the tensile steel distribution is uniform at ultimate strength.

$$T_u = Fy\ Ag$$

Ag → gross section area.

- ❖ For tension member having holes such as rivets, the reduced cross section is referred to as the "Net area".

- ❖ A ductile steel member without holes and subjected to a tensile load can resist without fracture a load larger than its gross cross – sectional area × Fy because of strain hardening.
- ❖ With holes → fails by fracture at the net section through the holes.



Design strength: A tension member may fail in:

1- Excessive deformation that can occur due to yielding of the gross area.

2-Fracture of the net area can occur if the stresses in the net section reach F_u .

the object is to prevent the two fractures.

■ The design strength will be the lesser of two values.

For the limit state of yielding in the gross section.

LRFD	ASD
$\phi t\, P_n = \phi t\, Fy\, Ag$ $\phi t = 0.9$	$P_n = Fy\, Ag$ $\frac{P_n}{\Omega t} = \frac{Fy\, Ag}{\Omega t}$ $\Omega = 1.67$

❖ For tensile rupture, net section is used: -

LRFD	ASD
$\phi\, P_n = \phi\, Fu\, Ae$ $\phi t = 0.75$	$P_n = Fu\, Ae$ $\frac{P_n}{\Omega t} = \frac{Fu\, Ae}{\Omega}$ $\Omega = 2.0$

Net area: -

The term "net cross – section area" refers to the gross cross sectional area of member, minus any holes.

❖ $A_n = A_g$ – area of holes.

❖ there are four types of holes (standard holes, oversized, short and long slotted). So for standard holes:

❖ The hole is drilled as 1/16 in larger than the diameter of the bolt ,

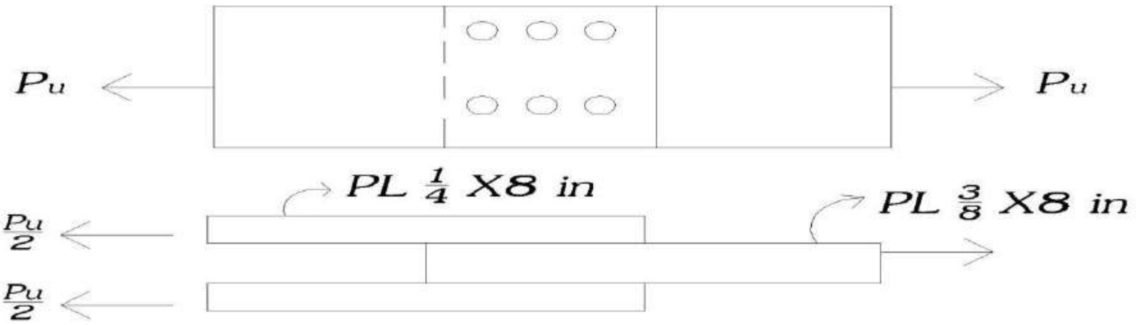
❖ Nominal hole diameter = dia. of bolt + $\frac{1}{16}$

❖ Due to punching of a hole, the material damage will extend to $\frac{1}{16}$ around the hole.

❖ So, the area of the hole = $b_d + \frac{1}{8}\, inch$ (for standard holes only)

❖ Holes causes stress concentration.

❖ ***Ex.1***:- Determine the net area of the $\frac{3}{8}$ × 8 inch plate, connected by two lines of 3/4-in bolts.(standard holes)



Net area = $\mathbf{A_n}=(\frac{3}{8}\times 8)-2(\frac{3}{4}+\frac{1}{8})(\frac{3}{8})=2.34\ in^2$

Specification: AISC chapter 5

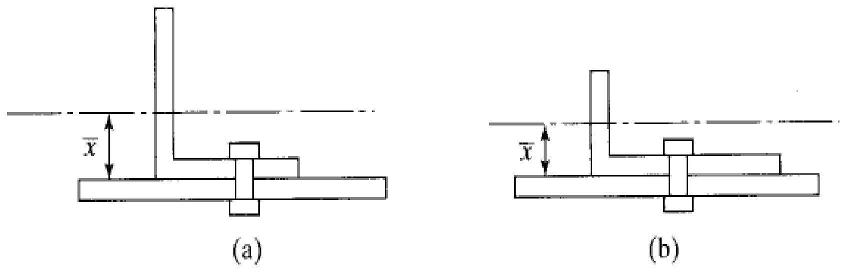
Effective net area: (A_e) (B-3)

* The connection is a region of weakness in tension member.

*For a tension member, if the forces are not transferred uniformly across a member (not connected through the whole section), there will be a transition region of uneven stresses from the connection into the member. In the transition zone, shear transfer has "lagged" called "shear lag".

For example: for the below figure:

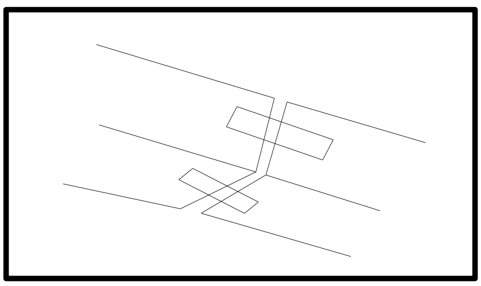
Reducing shear lag by reducing length of unconnected leg and and thus $\bar{x}$.



*the connected part to the gusset plate is fully stressed while the unconnected part is not so.

*net effective area will be used $=U A_n$ *AISC 2005 Equation* D3-1

Ae = An when the load is transmitted directly to each of the cross – sectional elements by connectors.

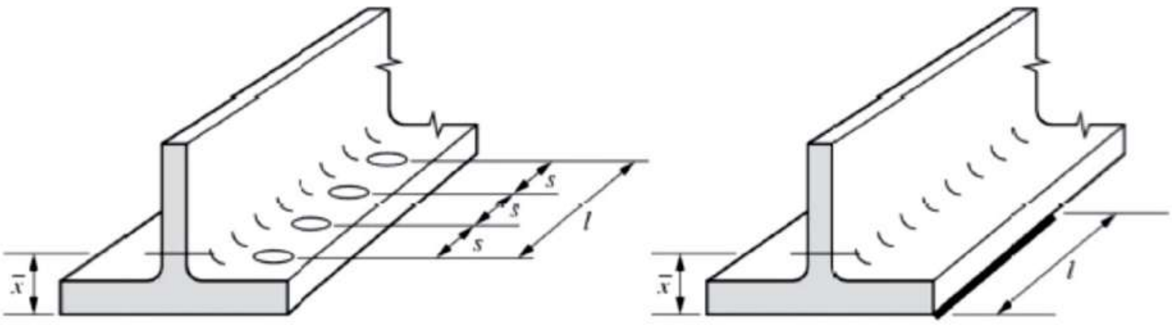


$U = 1 - x/L \leq 0.9$

where L =distance between the first and last bolt in the line.

- When there are two lines of bolts , L is the length of the line with maximum number of bolts.

x = the distance measured from the plan of connection to the centroid of the whole section.



(a) Bolted (b) Welded

bolted or welded member:

a) When the load is transmitted by *bolts or rivets* through some but not all of the cross-sectional elements of the member, then: -

Ae = ⊔An (B3 – 1) AISC

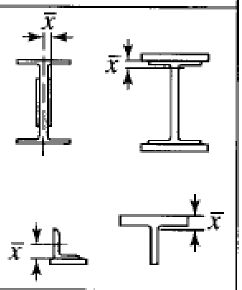
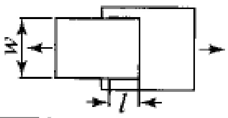
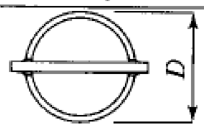
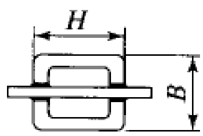
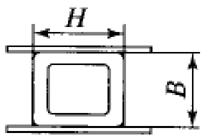
❖ U is the reduction coefficient which have the following values: -

⊔ = 0.9 W, M, S, shapes with $b_f \geq \frac{2}{3}d$; the connection is to the flanges with not less than three per line in the direction of stress.

⊔ = 0.85 W, M, S, shapes with $b_f < \frac{2}{3}d$; the connection is to the flanges with not less than three per line in the direction of stress.

and use AISC table:

TABLE 3.2 Shear Lag Factors for Connections to Tension Members

Case	Description of Element		Shear Lag Factor, U	Example
1	All tension members where the tension load is transmitted directly to each of the cross-sectional elements by fasteners or welds (except as in Cases 4, 5 and 6).		$U = 1.0$	—
2	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 or by longitudinal welds in combination with transverse welds. (Alternatively, for W, M, S and HP, Case 7 may be used. For angles, Case 8 may be used.)		$U = 1 - \bar{x}/l$	
3	All tension members where the tension load is transmitted only by transverse welds to some but not all of the cross-sectional elements.		$U = 1.0$ and A_n = area of the directly connected elements	—
4	Plates where the tension load is transmitted by longitudinal welds only.		$l \geq 2w \ldots U = 1.0$ $2w > l \geq 1.5w \ldots U = 0.87$ $1.5w > l \geq w \ldots U = 0.75$	
5	Round HSS with a single concentric gusset plate		$l \geq 1.3D \ldots U = 1.0$ $D \leq l < 1.3D \ldots U = 1 - \bar{x}/l$ $\bar{x} = D/\pi$	
6	Rectangular HSS	with a single concentric gusset plate	$l \geq H \ldots U = 1 - \bar{x}/l$ $\bar{x} = \frac{B^2 + 2BH}{4(B+H)}$	
		with two side gusset plates	$l \geq H \ldots U = 1 - \bar{x}/l$ $\bar{x} = \frac{B^2}{4(B+H)}$	
7	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 the 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 per line in the direction of loading	$U = 0.70$	—
8	Single and double angles (If U is calculated per Case 2, the larger value is permitted to be used.)	with 4 or more fasteners per line in the direction of loading	$U = 0.80$	—
		with 3 fasteners per line in the direction of loading (With fewer than 3 fasteners per line in the direction of loading, use Case 2.)	$U = 0.60$	—

l = length of connection, in. (mm); w = plate width, in. (mm); $\bar{x}$ = eccentricity of connection, in. (mm); B = overall width of rectangular HSS member, measured 90° to the plane of the connection, in. (mm); H = overall height of rectangular HSS member, measured in the plane of the connection, in. (mm)

Source: AISC Specification, Table D3.1, p. 16.1-28, June 22, 2010. Copyright © American Institute of Steel Construction. Reprinted with permission. All rights reserved.

b-When the load is transmitted by *welds* through some but not all of the cross sectional elements then: -

1- If the load is transmitted by transverse welds, for W, M, and S shapes then.

A_e = is the area of the directly connected elements.

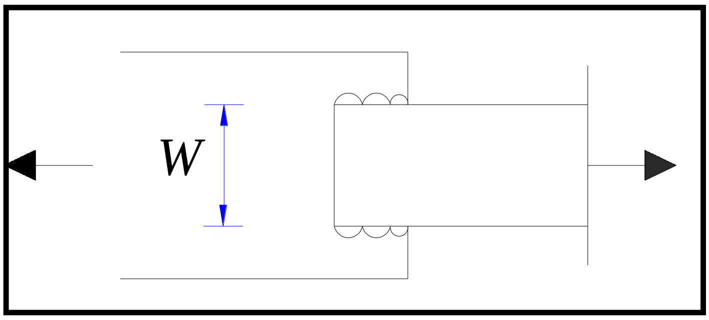
2- If the load is transmitted to a plate by longitudinal welds along both edges, then: -

Ae = ⊔ Ag (B3 - 2)

⊔ = 1 $l \geq 2w$

⊔ = 0.87 $2w > l \geq 1.5\,w$

⊔ = 0.75 $1.5\,w > l \geq w$



$$\text{w} = \text{plate width (distance between welds), in}$$

For welded connections, l is the length of the weld parallel to the line of force as shown in Figure C-D3.3 for longitudinal and longitudinal plus transverse welds.

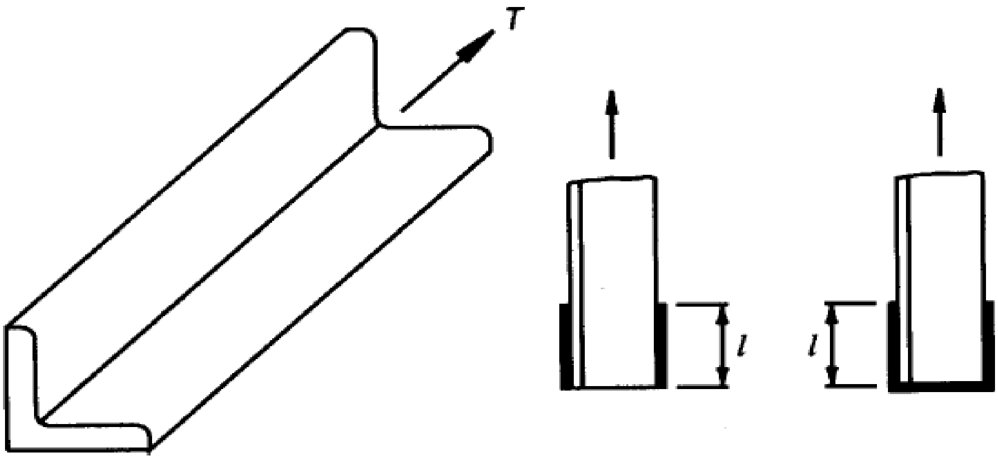


Fig. C-D3.3. Determination of l for U for connections with longitudinal and transverse weld

3-

Should the load be transmitted only by longitudinal welds to other than a plate member, or by longitudinal welds in combination with transverse welds, A is to equal the gross area of the member A_g (Table 3.2, Case 2).

4-

Should a tension load be transmitted only by transverse welds, A is to equal the area of the directly connected elements and U is to equal 1.0 (Table 3.2, Case 3).

*__slenderness ratio__ :

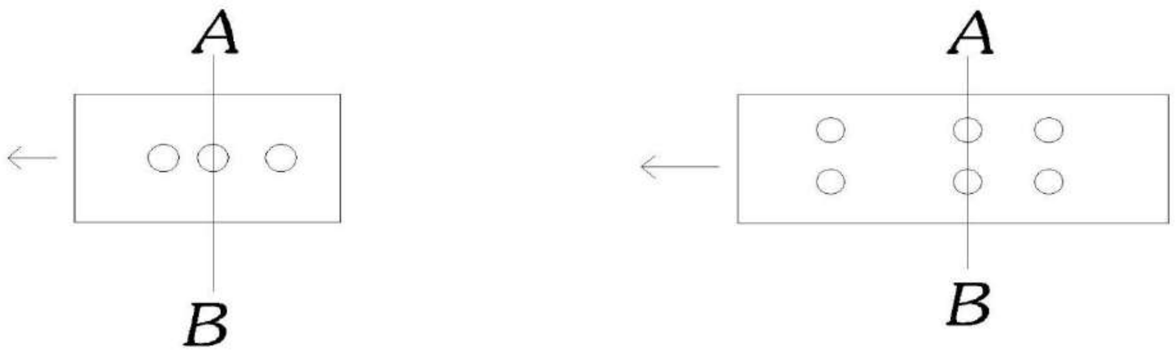
For tension member whose design is based on tensile force:

$\frac{L}{r} \leq 300$ where "r" is radius of gyration= $\sqrt{\frac{I}{A}}$

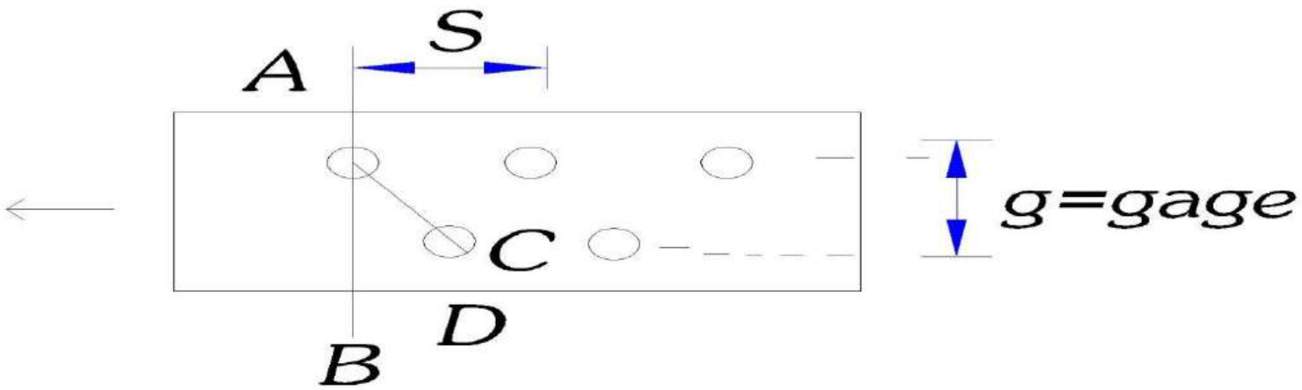
Effect of staggered holes: -

- When there are more than one row of bolts in a member it is often desirable to stagger them to provide a larger net area at any section.
- Several failure planes should be considered.

❖ In the fig. below, the failure line is along the section AB



❖ But for a staggered holes, as shown below the failure may be along path ABE or ABCD.

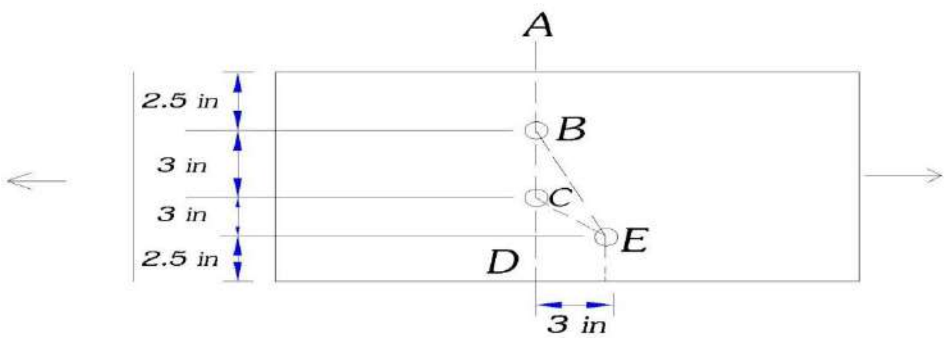


❖ AISC offers very simple method for computing the net width of a tensile member a long a zigzag section.

❖ The dia. Of the hole= bolt dia. $+\ \frac{1}{8}$

- The method is: the gross width of the member, subtract the dia. Of the holes along the zigzag path + $\frac{s^2}{4g}$
- Then, the one which gives the least value should be considered.

Ex.2: Determine the critical net area of the $\frac{1}{2}$ **in**.Thick plate shown in fig. below, using the AISC specification: the holes punched (standard holes) for $\frac{3}{4}$ **in**. bolts.



Critical path could be: -

- ABCD, ABCEF, ABEF
- Hole dia. to be subtracted = $\frac{3}{4}+\frac{1}{8}=\frac{7}{8}$ **in**
- Net widths

ABCD = $\mathbf{11-2\times\frac{7}{8}=9.25\ in}$

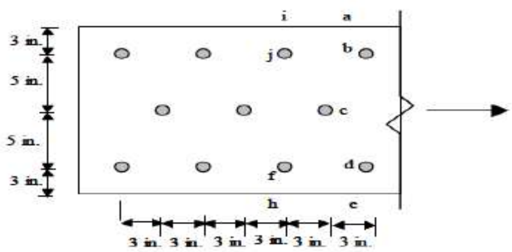
ABCEF= $\mathbf{11-3\times\frac{7}{8}+\frac{3^2}{4\times3}\ =9.125\ in}$ control

ABEF = $\mathbf{11-2\times\frac{7}{8}+\frac{3^2}{4\times6}\ =9.625\ in}$

$\therefore$ $\mathbf{An=9.125\times\frac{1}{2}in\ =4.56\ in^2}$ answer

---

***EX*. 3:** Compute the smallest net area for the plate shown below: The holes are for 1 in. diameter bolts.



- The effective hole diameter is 1 + 1/8 = 1.125 in.
- For line *a-b-d-e*

  w_n = 16.0 – 2 (1.125) = 13.75 in.
- For line *a-b-c-d-e*

  $w_n = 16.0 - 3\,(1.125) + 2 \times 3^2 / (4 \times 5) = 13.52$ in.
- The line *a-b-c-d-e* governs:
- $A_n = t\,w_n = 0.75\,(13.52) = 10.14\ \text{in}^2$

Ex.4: Determine the strength of a W10*45 with two lines of $\frac{3}{4}$ **in** diameter bolts in each flange using **A572 Gr. 50 steel**, with $\mathbf{F_y}$ **= 50 ksi** and $\mathbf{F_u}$ **= 65ksi** and **AISC specification.**

There are assumed to be at least three bolts in each line 4-in on center, and the bolts are not staggered.

Solution: - W 10*45 **(Ag = 13.3 in² , d = 10.10 in , $\mathbf{b_f}$ = 8.02 , $\mathbf{t_f}$ = 0.62 in)**

a-Nominal tensile strength: $P_n = F_y A_g = 50 \times 13.3 = 665\ k$

LRFD	ASD
$\phi\, P_n = 0.9 \times 665 = 598.5\ k$	$\dfrac{P_n}{F.S} = \dfrac{665}{1.67} = 398.2\ k$

b-Tensile rupture strength: -

$$A_n = 13.3 - 4\left(\frac{3}{4}+\frac{1}{8}\right)(0.620) = 11.13\ \text{in}^2$$

U = 0.9

Because $b_f = 8.02 > \frac{2}{3}\ d = \left(\frac{2}{3}\right)(10.1) = 6.73$ in

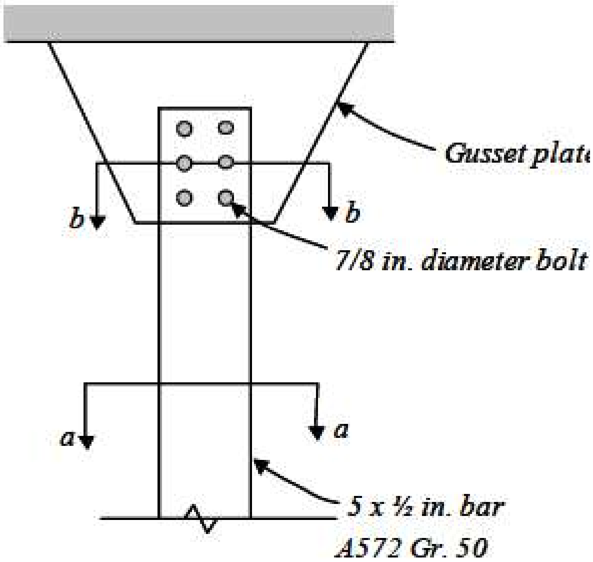
$$\therefore\ A_e = UA_n = (0.9)(11.13) = 10.02\ \text{in}^2$$

$$P_n = F_u A_e = 65 \times 10.02 = 651.3\ \text{k}$$

LRFD $\phi t = 0.75$	ASD F.S = 2
$\phi\, P_n = (0.75)(651.3) = 488.5$	$\frac{P_n}{F.S} = \frac{651.3}{2} = 325.6\ k$

Ans. LRFD = 488.5 ASD = 325.6 k

Ex.5 A $\mathbf{5 \times \frac{1}{2}}$ **in** bar of **A 572 Gr 50 steel** is used as a tension member. It is connected to a gusset plate with **six** $\mathbf{\frac{7}{8}}$ **in** diameter bolts as shown in the figure. Assume that the effective net area $\mathbf{A_e = A_n}$ and compute the tensile design strength of the member.Fu=65ksi



Solution: -

Gross area = $A_g = 5 \times \frac{1}{2} = 2.5\ in^2$

Net area (A_n):

Net hole diameter = $\frac{7}{8} + \frac{1}{8} = 1$

Net section area = $A_n = \left(5 - 2(1)\right)\frac{1}{2} = 1.5\ in^2$

a) Gross yielding design strength

Nominal tensile strength = $F_y A_g = 50 \times 2.5 = 125$ kips

LRFD $\phi t = 0.9$	ASD F.S = 1.67
$\phi\, P_n = 0.9 \times 125 = 112.5\ k$	$\frac{125}{1.67} = 74.85$

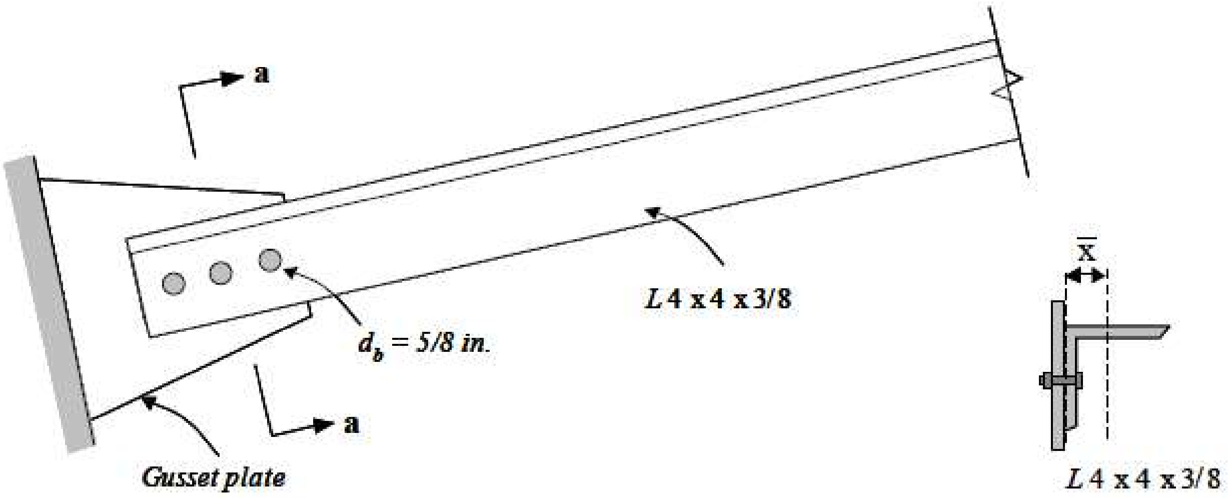
b) Tensile strength rupture ($A_n = A_e$)

LRFD $\phi t = 0.75$	ASD F.S = 2
$\phi\, P_n = \phi\, F_u A_e$ $= 0.75 \times 65 \times 1.5$ $= 73.125$	$\frac{P_n}{F.S} = \frac{F_u A_e}{2.0}$ $= \frac{65 \times 1.5}{2} = 48.75$

So, net section failure control.

Ex.6: A single angle tension member, L4*4*3/8 in. made from A36 steel is connected to a gusset plate with 5/8 in, diameter bolts, as shown in the figure below. The service loads are 35k dead load and 15k live load. Determine the adequacy of this member using AISC(LRFD) specification.

Assume A_e= 0.85 A_n .F_u= 58 ksi



solution: A_g=2.86 in^2.

Hole diameter for calculating net area=5/8 in+1/8= $\frac{3}{4}$ in

A_n=A_g- ($\frac{3}{4}$*$\frac{3}{8}$) = 2.86- ($\frac{3}{4}$*$\frac{3}{8}$) =2.579 in^2

A_e= 0.85* 2.579 =2.192 in^2

gross yielding design strength = ϕ_t*A_g*F_y = 0.9*2.86*36 =92.664k

net section fracture = ϕ_t*A_e*F_u= 0.75*2.192*58= 95.352 k

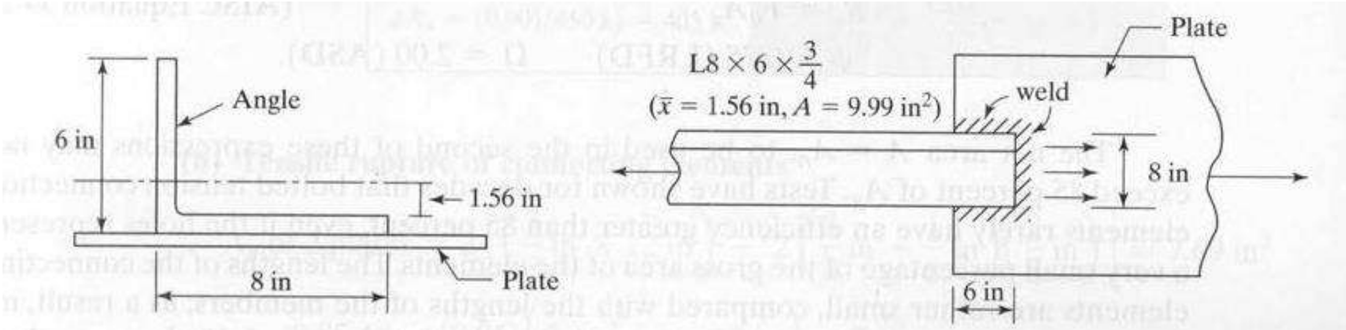
Design strength= 92.664 k

P_{u1}=1.4D= 1.4*35= 49k-----------1

P_{u1}=1.2D+1.6L= 66k---------------2 controls but designed strength 92.664 > ultimate designed load 66k

So, the given angle is adequate.

Ex.7: Compute the LRFD and the ASD allowable strength of the angle shown in the fig. below. It is welded on the ends and sides of the 8-in leg only. F_y=50ksi F_u=70ksi



solution:

nominal tensile strength of (L) Pn=F_yA_g=50*9.94=497k

a- gross section yielding:

LRFD $\phi t = 0.9$	ASD F.S = 1.67
$\phi P_n = 0.9 \times 497 = 447.3k$	$\frac{497}{1.67} = 297.6k$

b- Tensile rupture strength: one leg is connected-----A_e should be computed

U=1-$\frac{x}{L}$ = 1- $\frac{1.56}{6}$ = 0.74

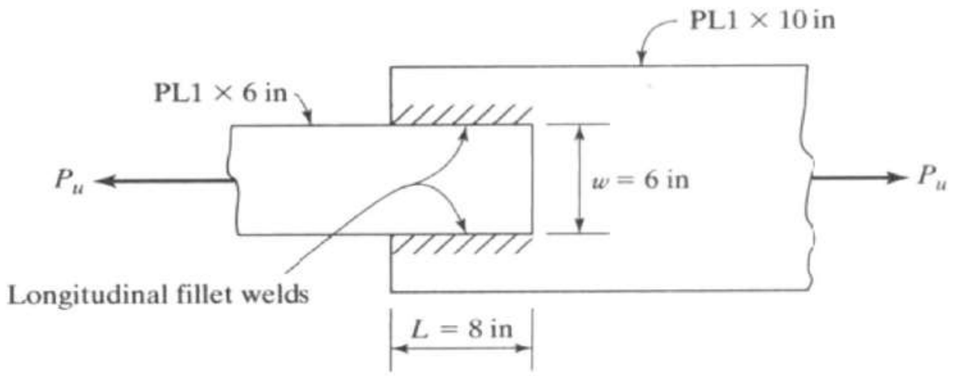
A_e= A_g*U= 9.94*0.74 = 7.36in^2

P_n= F_uA_e= 70*7.36 = 515.2k

LRFD $\phi t = 0.75$	ASD F.S = 2
$\phi P_n = \phi F_uA_e = 0.75 \times 515.2 = 386.4k$	$\frac{P_n}{F.S} = \frac{F_uA_e}{2.0} = \frac{515.2}{2} = 257.6k$

Ans, LRFD = 386.4k ASD= 257.6k

Ex.8 **:**The 1*6 in plate shown below is connected to a 1*10 in plate longitudinal fillet welds to transfer a tensile load. Determine the tensile strength of the member . F_y=50 ksi, F_u=65 ksi



solution: consider the tensile strength of the smaller plate 1*6 in

P_n= F_y*A_g=50*(1*6) = 300k

a-   Gross section yielding

LRFD $\boldsymbol{\phi t = 0.9}$	ASD F.S = 1.67
$\boldsymbol{\phi\, P_n = 0.9 \times 300 = 270k}$	$\dfrac{300}{1.67} = 179.5k$

b- tensile rupture strength:

1.5w=1.5*6 in=9in > L= 8 in > w=6in

$$\therefore U = 0.75$$

A_e= (6 in^2)(0.75)= 4.5in^2

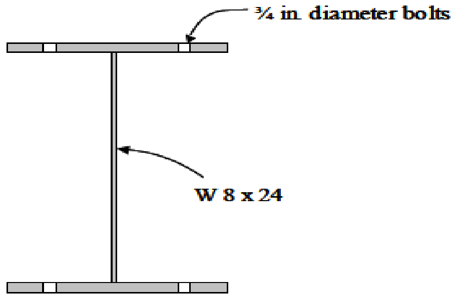
P_u=F_uA_e=65ksi * 4.5 in^2 = 292.5k

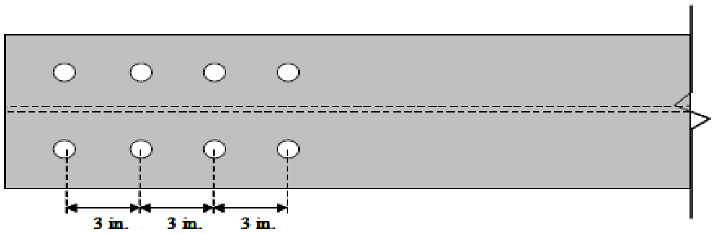
•

LRFD $\boldsymbol{\phi t = 0.75}$	ASD F.S = 2
$\boldsymbol{\phi\, P_n = \phi\, F_u A_e}$ $\boldsymbol{= 0.75 \times 292.5 = 219.4k}$	$\dfrac{P_n}{\mathrm{F.S}} = \dfrac{F_u A_e}{2.0}$ $= \dfrac{292.5}{2} = 146.2k$

Ans.   LRFD=219.4k   ASD=146.2k

Ex. 9:   Determine the design strength of an ASTM A992 W8 x 24 with four lines if ¾ in. diameter bolts in standard holes, two per flange, as shown in the Figure below. Assume the holes are located at the member end and the connection length is 9.0 in. Also calculate at what length this tension member would cease to satisfy the slenderness limitation in LRFD specification B7





Holes in beam flange

Solution:

• For ASTM A992 material: F_y = 50 ksi; and F_u = 65 ksi

• For the W8 x 24 section:

- A_g = 7.08 in^2 d = 7.93 in.
- t_w = 0.245 in. b_f = 6.5 in.
- t_f = 0.4 in. r_y = 1.61 in.

• Gross yielding design strength = $\varphi_t P_n = \varphi_t A_g F_y$ = 0.90 x 7.08 in^2 x 50 ksi = 319 kips

• Net section fracture strength = $\varphi_t P_n = \varphi_t A_e F_u$ = 0.75 x A_e x 65 ksi

- $A_e = U A_n$ - for bolted connection
- $A_n = A_g$ – (no. of holes) x (diameter of hole) x (thickness of flange)

  A_n = 7.08 – 4 x (diameter of bolt + 1/8 in.) x 0.4 in.

  A_n = 5.68 in^2
- $U = 1 - \frac{x}{L} \le 0.90$

- x can be obtained from the dimension tables for Tee section *WT 4 x 12*. See page ***1-50*** and ***1-51*** of the AISC manual:

  x = 0.695 in.

- $U = 1 - \frac{x}{L} = 1 - \frac{0.695}{9} = 0.923$

- But, $U \le 0.90$. Therefore, assume $U = 0.90$

• Net section fracture strength = $\varphi_t A_e F_u$ = 0.75 x 0.9 x 5.68 x 65 = 249.2 kips

• The design strength of the member is controlled by net section fracture = 249.2 kips

• According to LRFD specification B7, the maximum unsupported length of the member is limited to 300 r_y = 300 x 1.61 in. = 483 in. = 40.25 ft.

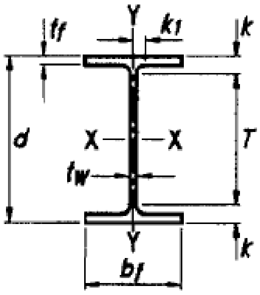


Table 1–1 (continued)
W Shapes
Dimensions

Shape	Area, A	Depth, d		Web			Flange				Distance				
				Thickness, t_w		$\frac{t_w}{2}$	Width, b_f		Thickness, t_f		k		k_1	T	Workable Gage
											k_{des}	k_{det}			
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W8×67	19.7	9.00	9	0.570	9/16	5/16	8.28	8 1/4	0.935	15/16	1.33	1 5/8	15/16	5 3/4	5 1/2
×58	17.1	8.75	8 3/4	0.510	1/2	1/4	8.22	8 1/4	0.810	13/16	1.20	1 1/2	7/8		
×48	14.1	8.50	8 1/2	0.400	3/8	3/16	8.11	8 1/8	0.685	11/16	1.08	1 3/8	13/16		
×40	11.7	8.25	8 1/4	0.360	3/8	3/16	8.07	8 1/8	0.560	9/16	0.954	1 1/4	13/16		
×35	10.3	8.12	8 1/8	0.310	5/16	3/16	8.02	8	0.495	1/2	0.889	1 3/16	13/16		
×31[f]	9.12	8.00	8	0.285	5/16	3/16	8.00	8	0.435	7/16	0.829	1 1/8	3/4		
W8×28	8.24	8.06	8	0.285	5/16	3/16	6.54	6 1/2	0.465	7/16	0.859	15/16	5/8	6 1/8	4
×24	7.08	7.93	7 7/8	0.245	1/4	1/8	6.50	6 1/2	0.400	3/8	0.794	7/8	9/16	6 1/8	4

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
67	4.43	11.1	272	60.4	3.72	70.1	88.6	21.4	2.12	32.7	2.43	8.07	5.05	1440
58	5.07	12.4	228	52.0	3.65	59.8	75.1	18.3	2.10	27.9	2.39	7.94	3.33	1180
48	5.92	15.9	184	43.2	3.61	49.0	60.9	15.0	2.08	22.9	2.35	7.82	1.96	931
40	7.21	17.6	146	35.5	3.53	39.8	49.1	12.2	2.04	18.5	2.31	7.69	1.12	726
35	8.10	20.5	127	31.2	3.51	34.7	42.6	10.6	2.03	16.1	2.28	7.63	0.769	619
31	9.19	22.3	110	27.5	3.47	30.4	37.1	9.27	2.02	14.1	2.26	7.57	0.536	530
28	7.03	22.3	98.0	24.3	3.45	27.2	21.7	6.63	1.62	10.1	1.84	7.60	0.537	312
24	8.12	25.9	82.7	20.9	3.42	23.1	18.3	5.63	1.61	8.57	1.82	7.53	0.346	259

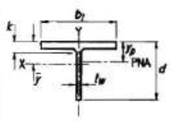


Table 1–8 (continued)
WT Shapes
Dimensions

Shape	Area, A	Depth, d		Stem Thickness, t_w		Stem $\frac{t_w}{2}$	Stem Area	Flange Width, b_f		Flange Thickness, t_f		Distance k		Distance Workable Gage
	in.^{2}	in.		in.		in.	in.^{2}	in.		in.		in.	in.	in.
WT6×11[c]	3.24	6.16	6 1/8	0.260	1/4	1/8	1.60	4.03	4	0.425	7/16	0.725	15/16	2 1/4[g]
×9.5[c]	2.79	6.08	6 1/8	0.235	1/4	1/8	1.43	4.01	4	0.350	3/8	0.650	7/8	↓
×8[c]	2.36	6.00	6	0.220	1/4	1/8	1.32	3.99	4	0.265	1/4	0.565	13/16	↓
×7[c,v]	2.08	5.96	6	0.200	3/16	1/8	1.19	3.97	4	0.225	1/4	0.525	3/4	↓
WT5×56	16.5	5.68	5 5/8	0.755	3/4	3/8	4.29	10.4	10 3/8	1.25	1 1/4	1.75	1 15/16	5 1/2
×50	14.7	5.55	5 1/2	0.680	11/16	3/8	3.77	10.3	10 3/8	1.12	1 1/8	1.62	1 13/16	↓
×44	12.9	5.42	5 3/8	0.605	5/8	5/16	3.28	10.3	10 1/4	0.990	1	1.49	1 11/16	↓
×38.5	11.3	5.30	5 1/4	0.530	1/2	1/4	2.81	10.2	10 1/4	0.870	7/8	1.37	1 9/16	↓
×34	9.99	5.20	5 1/4	0.470	1/2	1/4	2.44	10.1	10 1/8	0.770	3/4	1.27	1 7/16	↓
×30	8.82	5.11	5 1/8	0.420	7/16	1/4	2.15	10.1	10 1/8	0.680	11/16	1.18	1 3/8	↓
×27	7.91	5.05	5	0.370	3/8	3/16	1.87	10.0	10	0.615	5/8	1.12	1 5/16	↓
×24.5	7.21	4.99	5	0.340	5/16	3/16	1.70	10.0	10	0.560	9/16	1.06	1 1/4	↓
WT5×22.5	6.63	5.05	5	0.350	3/8	3/16	1.77	8.02	8	0.620	5/8	1.12	1 5/16	↓
×19.5	5.73	4.96	5	0.315	5/16	3/16	1.56	7.99	8	0.530	1/2	1.03	1 3/16	↓
×16.5	4.85	4.87	4 7/8	0.290	5/16	3/16	1.41	7.96	8	0.435	7/16	0.935	1 1/8	↓
WT5×15	4.42	5.24	5 1/4	0.300	5/16	3/16	1.57	5.81	5 3/4	0.510	1/2	0.810	1 1/8	2 3/4[g]
×13[c]	3.81	5.17	5 1/8	0.260	1/4	1/8	1.34	5.77	5 3/4	0.440	7/16	0.740	1 1/16	↓
×11[c]	3.24	5.09	5 1/8	0.240	1/4	1/8	1.22	5.75	5 3/4	0.360	3/8	0.660	15/16	↓
WT5×9.5[c]	2.81	5.12	5 1/8	0.250	1/4	1/8	1.28	4.02	4	0.395	3/8	0.695	15/16	2 1/4[g]
×8.5[c]	2.50	5.06	5	0.240	1/4	1/8	1.21	4.01	4	0.330	5/16	0.630	7/8	↓
×7.5[c]	2.21	5.00	5	0.230	1/4	1/8	1.15	4.00	4	0.270	1/4	0.570	13/16	↓
×6[c,f]	1.77	4.94	4 7/8	0.190	3/16	1/8	0.938	3.96	4	0.210	3/16	0.510	3/4	↓
WT4×33.5	9.84	4.50	4 1/2	0.570	9/16	5/16	2.57	8.28	8 1/4	0.935	15/16	1.33	1 5/8	5 1/2
×29	8.54	4.38	4 3/8	0.510	1/2	1/4	2.23	8.22	8 1/4	0.810	13/16	1.20	1 1/2	↓
×24	7.05	4.25	4 1/4	0.400	3/8	3/16	1.70	8.11	8 1/8	0.685	11/16	1.08	1 3/8	↓
×20	5.87	4.13	4 1/8	0.360	3/8	3/16	1.49	8.07	8 1/8	0.560	9/16	0.954	1 1/4	↓
×17.5	5.14	4.06	4	0.310	5/16	3/16	1.26	8.02	8	0.495	1/2	0.889	1 3/16	↓
×15.5[f]	4.56	4.00	4	0.285	5/16	3/16	1.14	8.00	8	0.435	7/16	0.829	1 1/8	↓
WT4×14	4.12	4.03	4	0.285	5/16	3/16	1.15	6.54	6 1/2	0.465	7/16	0.859	15/16	3 1/2
×12	3.54	3.97	4	0.245	1/4	1/8	0.971	6.50	6 1/2	0.400	3/8	0.794	7/8	3 1/2

[c] Shape is slender for compression with $F_y = 50$ ksi.
[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.
[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.
[v] Shear strength controlled by buckling effects ($C_v < 1.0$) with $F_y = 50$ ksi.

Table 1–8 (continued)
WT Shapes
Properties

WT6-WT4

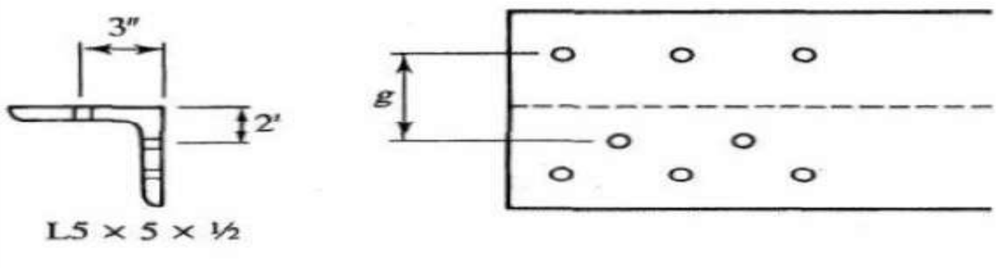
Nominal Wt.	Compact Section Criteria $\frac{b_f}{2t_f}$	Compact Section Criteria $\frac{h}{t_w}$	Axis X-X I	Axis X-X S	Axis X-X r	Axis X-X $\bar{y}$	Axis X-X Z	Axis X-X y_p	Axis Y-Y I	Axis Y-Y S	Axis Y-Y r	Axis Y-Y Z	Q_s $F_y = 50$ ksi	Torsional Properties J	Torsional Properties C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}		in.^{4}	in.^{6}
11	4.74	23.7	11.7	2.59	1.90	1.63	4.63	0.402	2.33	1.15	0.847	1.83	0.711	0.146	0.137
9.5	5.72	25.9	10.1	2.28	1.90	1.65	4.11	0.348	1.88	0.939	0.821	1.49	0.598	0.0899	0.0934
8	7.53	27.3	8.70	2.04	1.92	1.74	3.72	0.639	1.41	0.706	0.773	1.13	0.539	0.0511	0.0678
7	8.82	29.8	7.67	1.83	1.92	1.76	3.32	0.760	1.18	0.593	0.753	0.947	0.451	0.0350	0.0493
56	4.17	7.52	28.6	6.40	1.32	1.21	13.4	0.791	118	22.6	2.67	34.6	1.00	7.50	16.9
50	4.62	8.16	24.5	5.56	1.29	1.13	11.4	0.711	103	20.0	2.65	30.5	1.00	5.41	11.9
44	5.18	8.96	20.8	4.77	1.27	1.06	9.65	0.631	89.3	17.4	2.63	26.5	1.00	3.75	8.02
38.5	5.86	10.0	17.4	4.05	1.24	0.990	8.06	0.555	76.8	15.1	2.60	22.9	1.00	2.55	5.31
34	6.58	11.1	14.9	3.49	1.22	0.932	6.85	0.493	66.7	13.2	2.58	20.0	1.00	1.78	3.62
30	7.41	12.2	12.9	3.04	1.21	0.884	5.87	0.438	58.1	11.5	2.57	17.5	1.00	1.23	2.46
27	8.15	13.6	11.1	2.64	1.19	0.836	5.05	0.395	51.7	10.3	2.56	15.6	1.00	0.909	1.78
24.5	8.93	14.7	10.0	2.39	1.18	0.807	4.52	0.361	46.7	9.34	2.54	14.1	1.00	0.693	1.33
22.5	6.47	14.4	10.2	2.47	1.24	0.907	4.65	0.413	26.7	6.65	2.01	10.1	1.00	0.753	0.981
19.5	7.53	15.7	8.84	2.16	1.24	0.876	3.99	0.359	22.5	5.64	1.98	8.57	1.00	0.487	0.616
16.5	9.15	16.8	7.71	1.93	1.26	0.869	3.48	0.305	18.3	4.60	1.94	7.00	1.00	0.291	0.356
15	5.70	17.5	9.28	2.24	1.45	1.10	4.01	0.380	8.35	2.87	1.37	4.41	1.00	0.310	0.273
13	6.56	19.9	7.86	1.91	1.44	1.06	3.39	0.330	7.05	2.44	1.36	3.75	0.904	0.201	0.173
11	7.99	21.2	6.88	1.72	1.46	1.07	3.02	0.282	5.71	1.99	1.33	3.05	0.837	0.119	0.107
9.5	5.09	20.5	6.68	1.74	1.54	1.28	3.10	0.349	2.15	1.07	0.874	1.67	0.873	0.116	0.0796
8.5	6.08	21.1	6.06	1.62	1.56	1.32	2.90	0.311	1.78	0.887	0.844	1.40	0.843	0.0776	0.0610
7.5	7.41	21.7	5.45	1.50	1.57	1.37	2.71	0.305	1.45	0.723	0.810	1.15	0.810	0.0518	0.0475
6	9.43	26.0	4.35	1.22	1.57	1.36	2.20	0.322	1.09	0.551	0.785	0.869	0.593	0.0272	0.0255
33.5	4.43	7.89	10.9	3.05	1.05	0.936	6.29	0.594	44.3	10.7	2.12	16.3	1.00	2.51	3.56
29	5.07	8.58	9.12	2.61	1.03	0.874	5.25	0.520	37.5	9.13	2.10	13.9	1.00	1.66	2.28
24	5.92	10.6	6.85	1.97	0.986	0.777	3.94	0.435	30.5	7.51	2.08	11.4	1.00	0.977	1.30
20	7.21	11.5	5.73	1.69	0.988	0.735	3.25	0.364	24.5	6.08	2.04	9.24	1.00	0.558	0.715
17.5	8.10	13.1	4.82	1.43	0.968	0.688	2.71	0.321	21.3	5.31	2.03	8.05	1.00	0.384	0.480
15.5	9.19	14.0	4.28	1.28	0.969	0.668	2.39	0.285	18.5	4.64	2.02	7.03	1.00	0.267	0.327
14	7.03	14.1	4.23	1.28	1.01	0.734	2.38	0.315	10.8	3.31	1.62	5.04	1.00	0.268	0.230
12	8.12	16.2	3.53	1.08	0.999	0.695	1.98	0.272	9.14	2.81	1.61	4.28	1.00	0.173	0.144

***Staggered bolts in angles*:** If staggered lines of bolts are present in both legs of an angle, then the net area is found by first unfolding the angle to obtain an equivalent plate. This plate is then analyzed like shown below.

- The unfolding is done at the middle surface to obtain a plate with gross width equal to the sum of the leg lengths minus the angle thickness.

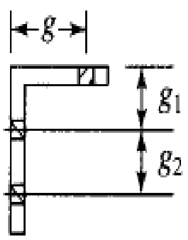
- AISC Specification B2 says that any gage line crossing the heel of the angle should be reduced by an amount equal to the angle thickness.

- See Figure below. For this situation, the distance g will be = 3 + 2 – ½ in.



Holes for bolts are drilled at certain standard locations depends on angle- leg widths and on the number of lines of holes so, for a designer AISC manual show gages to be used as in the below table:

TABLE 3.1 Workable Gages for Angles, in Inches



Leg	8	7	6	5	4	$3\frac{1}{2}$	3	$2\frac{1}{2}$	2	$1\frac{3}{4}$	$1\frac{1}{2}$	$1\frac{3}{8}$	$1\frac{1}{4}$	1
g	$4\frac{1}{2}$	4	$3\frac{1}{2}$	3	$2\frac{1}{2}$	2	$1\frac{3}{4}$	$1\frac{3}{8}$	$1\frac{1}{8}$	1	$\frac{7}{8}$	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{5}{8}$
g_1	3	$2\frac{1}{2}$	$2\frac{1}{4}$	2										
g_2	3	3	$2\frac{1}{2}$	$1\frac{3}{4}$										

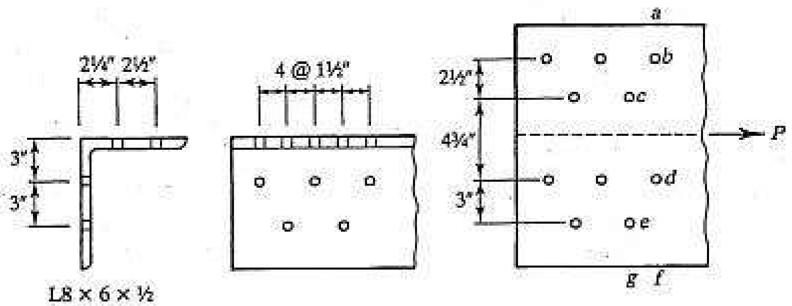
EX10

Find the design tensile strength of the angle shown in Figure 3.16. A36 steel is used, and holes are for ⅞-inch-diameter bolts.

SOLUTION Compute the net width:

$$w_g = 8 + 6 - \tfrac{1}{2} = 13.5 \text{ in.}$$

FIGURE 3.16



Effective hole diameter $= \tfrac{7}{8} + \tfrac{1}{8} = 1$ in.

For line *abdf,*

$$w_n = 13.5 - 2(1) = 11.5 \text{ in.}$$

For line *abceg,*

$$w_n = 13.5 - 3(1) + \frac{(1.5)^2}{4(2.5)} = 10.73 \text{ in.}$$

Because $\tfrac{1}{10}$ of the load has been transferred from the member by the fastener at *d,* this potential failure line must resist only $\tfrac{9}{10}$ of the load. Therefore the net width of 10.73 inch should be multiplied by $\tfrac{10}{9}$ to obtain a net width that can be compared with those lines that resist the full load. Use $w_n = 10.73(\tfrac{10}{9}) = 11.92$ inch. For line *abcdeg,*

$$g_{cd} = 3 + 2.25 - 0.5 = 4.75 \text{ in.}$$

$$w_n = 13.5 - 4(1) + \frac{(1.5)^2}{4(2.5)} + \frac{(1.5)^2}{4(4.75)} + \frac{(1.5)^2}{4(3)} = 10.03 \text{ in.}$$

The last case controls:

$$A_n = t(w_n) = 0.5(10.03) = 5.015 \text{ in.}^2$$

Both legs of the angle are connected, so

$$A_e = A_n = 5.015 \text{ in.}^2$$

The design strength based on fracture is

$$\phi_t P_n = 0.75 F_u A_e = 0.75(58)(5.015) = 218 \text{ kips}$$

The design strength based on yielding is

$$\phi_t P_n = 0.90 F_y A_g = 0.90(36)(6.75) = 219 \text{ kips}$$

ANSWER Fracture controls; design strength = 218 kips.

BLOCK SHEAR

- For some connection configurations, the tension member can fail due to ‘tear-out’ of material at the connected end. This is called *block shear*.
- For example, the single angle tension member connected as shown in the Figure below is susceptible to the phenomenon of *block shear*.

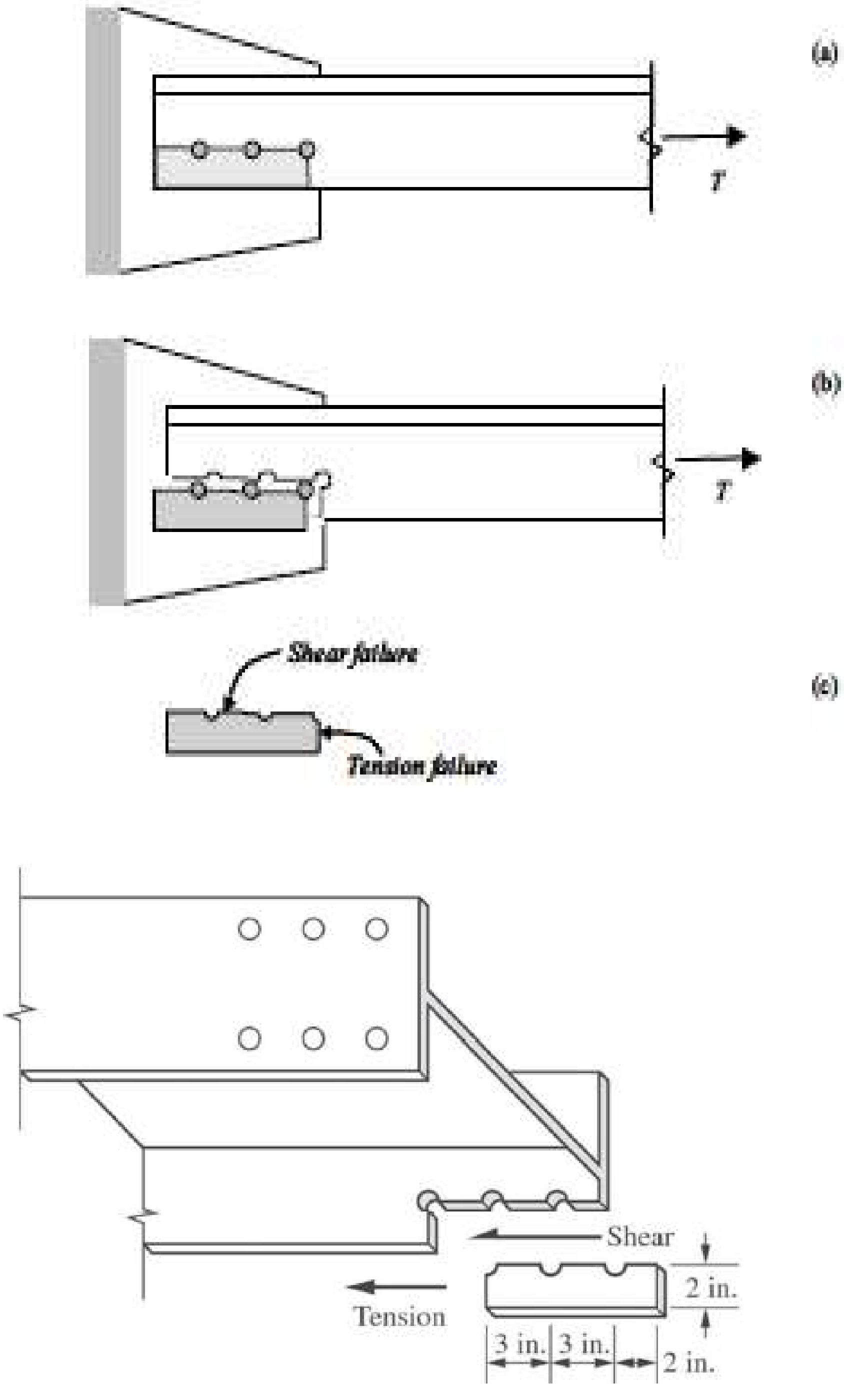


Figure 4.4 Block shear failure of single angle tension member and a W-shape

• For the case shown above, shear failure will occur along the longitudinal section a-b and tension failure will occur along the transverse section b-c

• AISC Specification (SPEC) Chapter D on tension members does not cover block shear failure explicitly. But, it directs the engineer to the Specification Section J4.

Block shear strength is determined as the sum of the shear strength on a failure path and the tensile strength on a perpendicular segment.

Strength of Elements in Shear:

The available shear yield strength of affected and connecting elements in shear the lower value obtained according to the *limit states* of *shear yielding* and *shear rupture*:

(a) For shear yielding of the element:

$R_n = 0.60 F_y A_g$ $\phi = 1.00$ (LRFD) AISC (J4-3)

(b) For shear rupture of the element:

$R_n = 0.6 F_u A_{nv}$ $\phi = 0.75$ (LRFD) (J4-4)

The model used in the AISC Specification assumes that failure occurs by rupture (fracture) on the shear area and rupture on the tension area. Both surfaces contribute to the total strength, and the resistance to block shear will be the sum of the strengths of the two surfaces. The shear rupture stress is taken as 60% of the tensile ultimate stress, so the nominal strength in shear is $0.6F_uA_{nv}$ and the nominal strength in tension is F_uA_{nt},

where

A_{nv} = net area along the shear surface or surfaces

A_{nt} = net area along the tension surface

This gives a nominal strength of

$$R_n = 0.6F_uA_{nv} + F_uA_{nt}$$

The AISC Specification limits the $0.6F_uA_{nv}$ term to $0.6F_yA_{gv}$, where

$0.6F_y$ = shear yield stress

A_{gv} = gross area along the shear surface or surfaces

and gives one equation to cover all cases as follows:

• The AISC specification (J4.3) states that the strength Rn for the block shear rupture design strength is as follows:

$$R_n = (0.6\,F_u A_{nv} + U_{bs} F_u A_{nt}) \le (0.6\,F_y A_{gv} + U_{bs} F_u A_{nt})$$

- Where, $\varphi = 0.75$ LRFD F.s= 2 ASD

A_{gt} = gross area subject to tension

A_{nv} = net area subject to shear

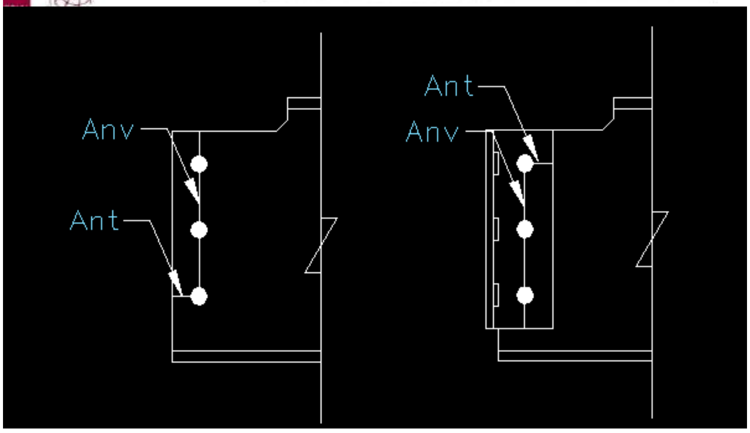
A_{nt} = net area subject to tension

U_{bs}=reduction factor=1 for angles, plates with one line of bolts.

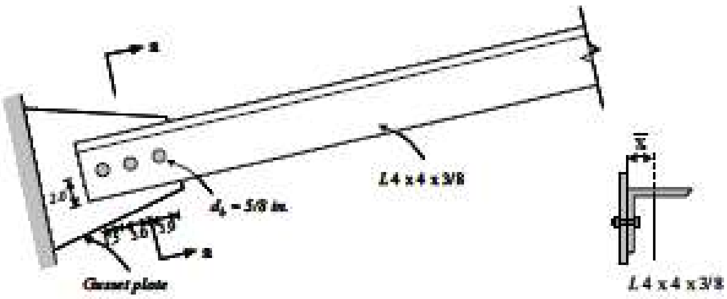
Where the tension *stress* is uniform, *Ubs* = 1 where the tension stress is non-uniform, *Ubs* = 0.5.

Block Shear Tear-out

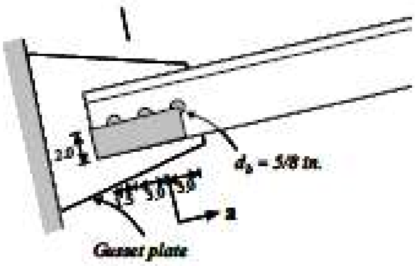
- Cl. 13.11
- Paths parallel & perpendicular to load



EX. 11. Calculate the block shear strength (using LRFD) method of the single angle tension member considered in Examples 6. The single angle *L 4 * 4 * 3/8* in. made from A36 steel(F_u=58k) is connected to the gusset plate with 5/8 in. diameter bolts as shown below. The bolt spacing is 3 in. center-to-center and the edge distances are 1.5 in and 2.0 in as shown in the Figure below.



- Step I. Assume a block shear path and calculate the required areas



• Step I. Assume a block shear path and calculate the required areas

- A_{gt} = gross tension area = 2.0 x 3/8 = 0.75 in^2
- A_{nt} = net tension area = 0.75 – **0.5** x (5/8 + 1/8) x 3/8 = 0.609 in^2
- A_{gv} = gross shear area = (3.0 + 3.0 +1.5) x 3/8 = 2.813 in^2
- A_{nv} = net shear area = 2.813 – **2.5** x (5/8 + 1/8) x 3/8 = 2.109 in^2

• $R_n = (0.6\ F_u A_{nv} + U_{bs} F_u A_{nt}) \leq (0.6\ F_y A_{gv} + U_{bs} F_u A_{nt})$

= 0.6 x 58 x 2.109 +(1) (58)(0.609) ≤ 0.6x 36x 2.813 + (1) (58)(0.609)

108.715 > 96.083

$\phi\ R_n$ = 0.75x 96.083 k= 72.06k

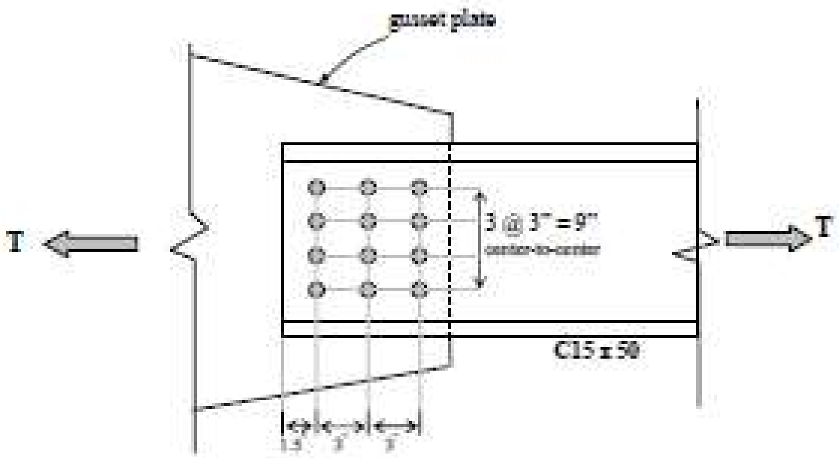
Member is still adequate to carry the factored load (P_u) = 66 kips

---

• *Important:*

- Any of the three limit states (gross yielding, net section fracture, or block shear failure) can govern.
- The design strength for all three limit states has to be calculated.
- The member design strength will be the smallest of the three calculated values
- The member design strength must be greater than the ultimate factored design load in tension.

<u>*Ex. 12*</u>**:** 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 (F_y) equal to 50 ksi and ultimate stress (F_u) equal to 65 ksi. (use LRFD procedure)



limit state of yielding due to tension :

$\emptyset Pn$ =0.9*50*14.7=662 Kips

limit state of fracture:

$A_n = A_g - n\, d_e = 14.7 - (4)(7/8)*0.716 = 12.19 in^2$

$U = 1 - x/L = \left(1 - \frac{0.798}{6}\right) = 0.867 < 0.9 \ Ok$

$A_e = UA_n = 0.867 * 12.19 = 10.57\ in^2$

$$\emptyset Pn = 0.75 * 65 * 10.57 = 515\ Kips$$

. Limit state of block shear rupture:

$R_n = (0.6\, F_u A_{nv} + U_{bs} F_u A_{nt}) \leq (0.6\, F_y A_{gv} + U_{bs} F_u A_{nt})$

$0.6F_uA_{nv} = 0.6*65*\{2*(7.5-2.5*7/8)\}*0.716 = 296.6925$

$F_uA_{nt} = 65* \{9-3*7/8\}*0.716 = 296.6925$

$R_n = 593.385k$

$Rn = \{0.6\, F_y A_{gv} + Ubs\, F_u\, A_{nt}\} = \{0.6*50*15*0.716 + 296.6925 = 618.89k$

$\phi Rn = 0.75 x 593.385 = 445\ k$ control

Block shear rupture is the critical limit state and the design tension strength is 445kips.

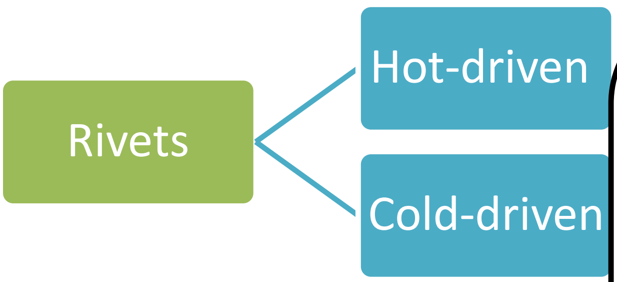
Chapter Two

Fasteners: -

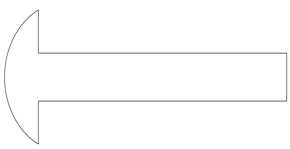
Every steel structure is an assemblage of many parts or members that must be fastened together using welding or fastener, such as rivets or bolts.

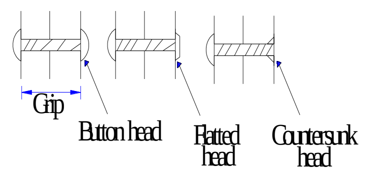
Rivets: -

Un-driven rivets are formed from bar steel (A502 Grade 1 or Grade 2 or Grade 3) with a head formed on one end.



The rivet is heated to a light cherry-red color, inserting into a hole. Then applying pressure to the preformed head & squeezing the other end at the same time to form the other end. After cooling the rivet shrinks and providing a clamping force, No clamping force since they do not shrink after driving.





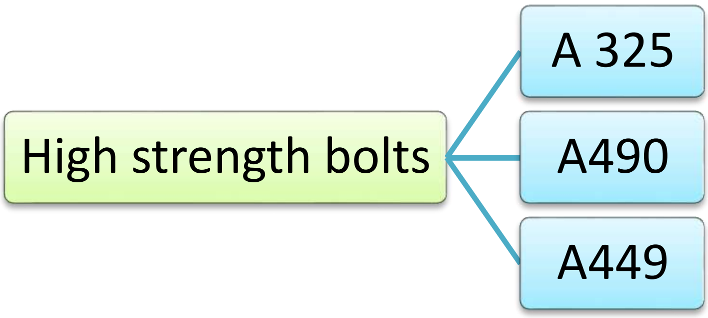
Bolts: -

1- *Unfinished bolts:* -called also ordinary bolts, made from carbon steels. These are classified by ASTM as A307 steel. They are used in light structures subjected to static load, secondary or bracing members, purlin. available in $\frac{1}{2}$ in to $1\frac{1}{2}$ in .

2- *High – strength bolts:* -(having tensile strength two or more times)

These bolts are tightened to develop a tensile stress in them which results in a clamping force on the joint. (made from medium carbon steel).(A-325, A490). *when bolts with diameters exceeding 1.5 in or length more than 8 in are required then use A449.*

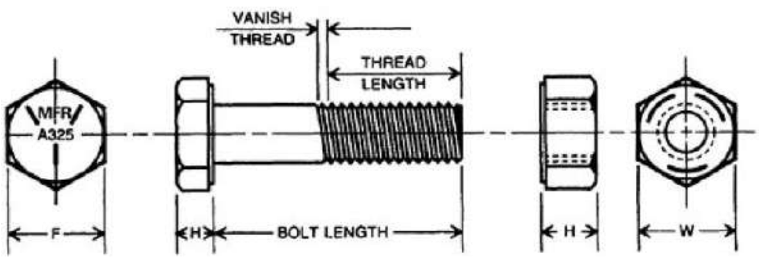
❖ See table J 3.2 AISC 2005. For min. pretension for fully tightened bolts.



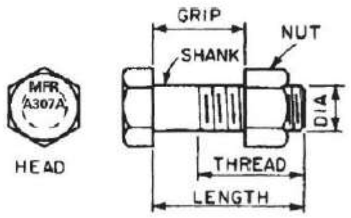
Anchor bolts & threaded rods.

❖ Common diameters in building $^{3}/_{4}{}''\&^{7}/_{8}{}''$. in bridge $^{7}/_{8}{}''\&1''$.

Structural bolts (A325 and A490) can be installed pretensioned or snug tight. Pretensioned means that the bolt is tightened until a tension force approximately equal to 70 percent of its minimum tensile strength is produced in the bolt.



High-strength structural-steel bolt and nut.



Unfinished (machine) or common bolts.

Allowable stress on the fasteners are given in table J3.2

Welding: -

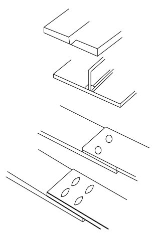
Shielded metal arc welding (SMAW) is one of the oldest welding processes.

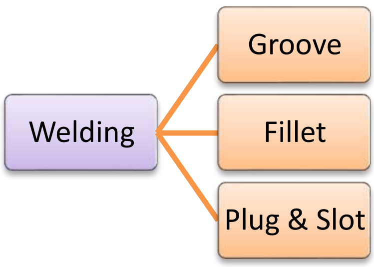
E 60 xx

E 70 xx

E 80 xx

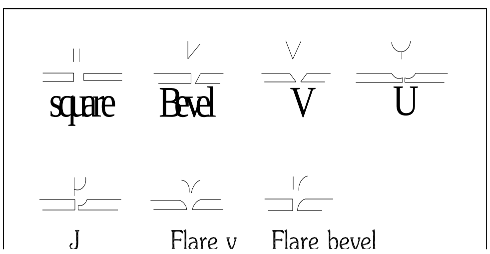
E 100 xx   . E 110 xx





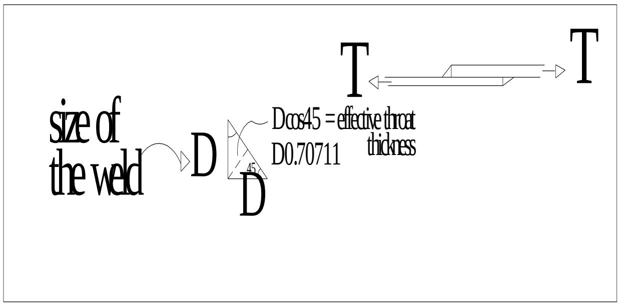
❖ Groove weld: -

Is used to connect structural members that are aligned in the same plane.



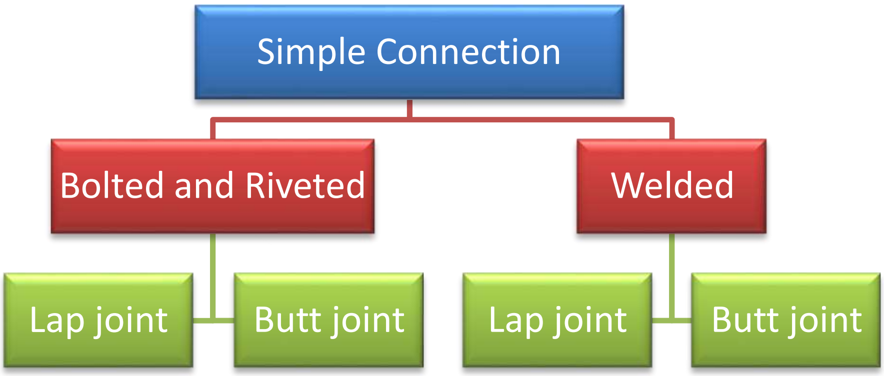
❖ Fillet weld: -

❖ Is the most widely used in of all basic welds.



2- Simple Connections

Connections are used where the various member ends must be attached to other members sufficiently to allow the load to continue an orderly flow to the foundation. In this type of connections, is assumed that no moment transfers between the connected parts.



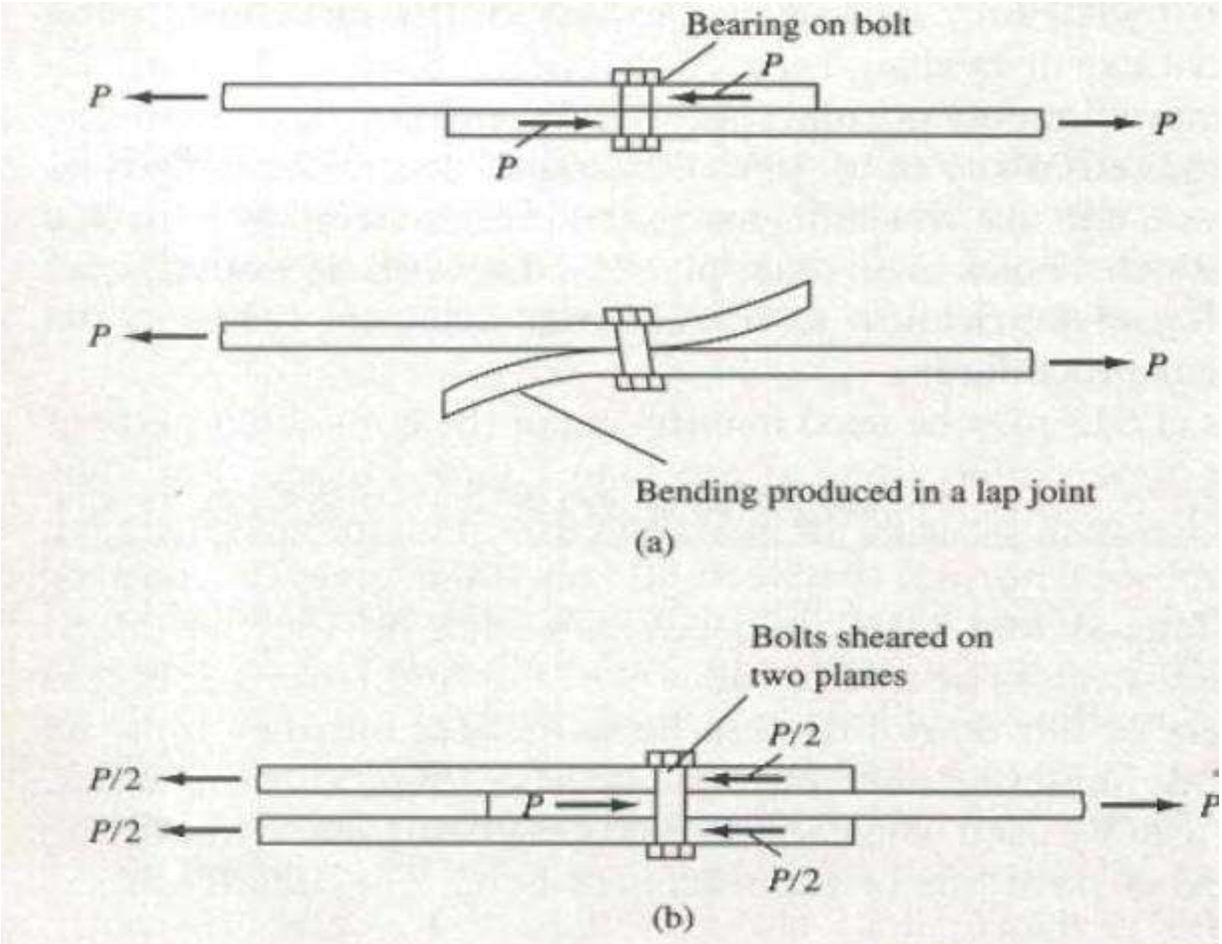


Figure 12-1 (a) Lap joint. (b) Butt joint.

As a result of loading ,the load in the plates will tend to shear the connectors off on the plane between the plates and press against the sides of the bolt ,these connectors are said to be in ***single shear.***

Bolted and riveted simple connections: -

If resistance of joints is considered, then there are two types: -

a) Bearing connection.
b) Slip-critical connection.

a) Bearing connection:
   In this connection a sufficient slip occurs at a design load which bring the shank of a rivet of bolt into contact with the side of the hole. So the resistance of the joint is taken as a combination of the fastener shear resistance and the bearing of the connected material against the fastener.

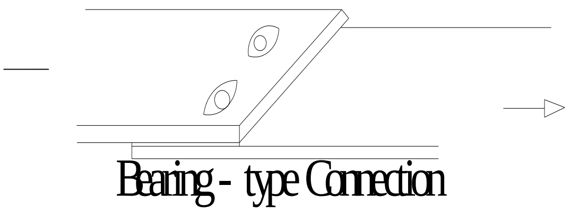
b) Slip-critical connection: -needed for joints subj. to fatigue, bolts with weld, end members.... (bridges)
In this connection there is no relative movement between the connected parts unless the design load is exceeded. So the resistance of this connection is represented by the friction ($F = \mu T$) which is produced by tightening of the high-strength bolts.

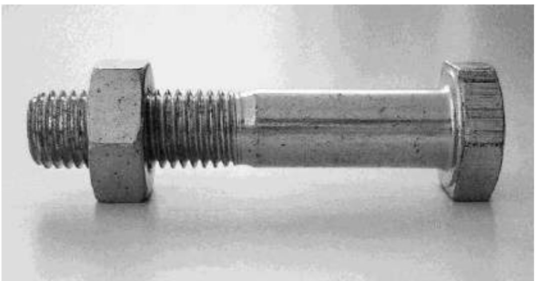
- ❖ After overload and once the frictional resistance is inadequate to transfer the load, bearing against the side of the hole will occur.
- ❖ T is also a clamping force.

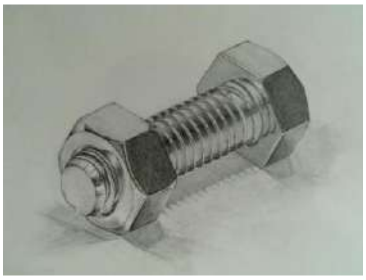
Size and use of holes:- J 3.2

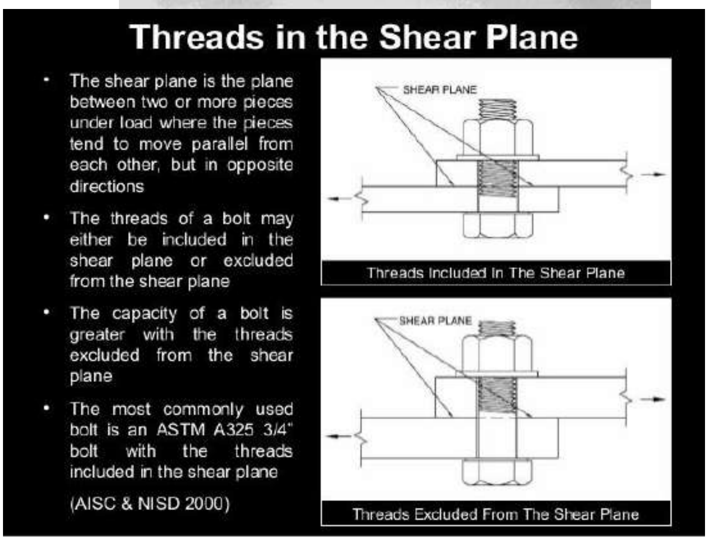
The nominal sizes of holes are given in table J3.1 (5-71)

- ❖ Standard holes, (S.H.), is used for member-to-member connections and in slip-critical connections. (Bolt dia. + $^{1}/_{16}{}''$) up to 7/8in dia. And Bolt dia. + $^{1}/_{8}{}''$ for 1in and more.
- ❖ Oversized holes, (O.S.H.), Are permitted in slip-critical connections but shall not be used in bearing-type connections.
- ❖ Short-slotted holes (S.S.H.) and Long-slotted holes (L.S.H), are permitted in slip-critical connections and in bearing connections but the length shall be normal to the direction of the load.





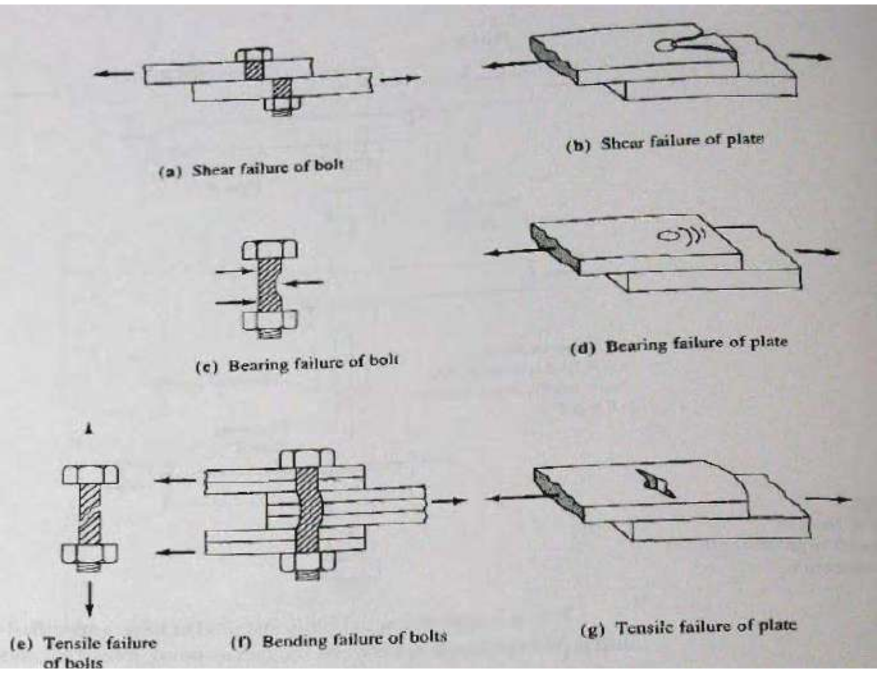




Possible modes of failure of bolted connections.

a) Failure of bolts.	b) Failure of plate.
Shear failure.	Shear failure.

Bearing failure.	Bearing failure.
Tensile failure.	Tensile failure.
Bending failure.	



Slip-critical joint, from structural engineering, is a type of bolted structural steel connection which relies on friction between the two connected elements rather than bolt shear or bolt bearing to join two structural elements.

Shear (and tension) loads can be transferred between two structural elements by either a bearing-type connection or a slip-critical connection.

In a slip-critical connection, loads are transferred from one element to another through friction forces developed between the faying surfaces of the connection. These friction forces are generated by the extreme tightness of the structural bolts holding the connection together. These bolts, usually tension control bolts or compressible washer tension indicating type bolts, are tensioned to a minimum required amount to generate large enough friction forces between the faying surfaces such that the shear (or tension) load is transferred by the structural members and not by the bolts(in shear) and the connection plates(in bearing). The "turn of the nut" method is also widely used to achieve that state of friction.

If slip-critical connections fail (by slipping), they revert to bearing-type connections, with structural forces now transferred through bolt shear and connection plate bearing. Thus a slippage failure of a slip-critical connection is not necessarily a catastrophic failure. However, slippage of a slip-critical connection in columns may lead to column instability. Slippage of a slip critical joint in a roof truss could result in unintended ponding effects.

The faying surfaces of slip-critical connections must be properly prepared in order to maximize friction forces between the surfaces joined. Usually this requires cleaning, descaling, roughening, and/or blasting of the faying surfaces. Painting the faying surfaces with a class B primer also allows to be in accordance with most of the design that ask for *Slip-critical joint*.

TABLE 12-2 NOMINAL HOLE DIMENSIONS

	Hole dimensions			
Bolt dia.	Standard (dia.)	Oversize (dia.)	Short-slot (width × length)	Long-slot (width × length)
$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{9}{16} \times \frac{11}{16}$	$\frac{9}{16} \times 1\frac{1}{4}$
$\frac{5}{8}$	$\frac{11}{16}$	$\frac{13}{16}$	$\frac{11}{16} \times \frac{7}{8}$	$\frac{11}{16} \times 1\frac{9}{16}$
$\frac{3}{4}$	$\frac{13}{16}$	$\frac{15}{16}$	$\frac{13}{16} \times 1$	$\frac{13}{16} \times 1\frac{7}{8}$
$\frac{7}{8}$	$\frac{15}{16}$	$1\frac{1}{16}$	$\frac{15}{16} \times 1\frac{1}{8}$	$\frac{15}{16} \times 2\frac{3}{16}$
1	$1\frac{1}{16}$	$1\frac{1}{4}$	$1\frac{1}{16} \times 1\frac{5}{16}$	$1\frac{1}{16} \times 2\frac{1}{2}$
$\geq 1\frac{1}{8}$	$d + \frac{1}{16}$	$d + \frac{5}{16}$	$(d + \frac{1}{16}) \times (d + \frac{3}{8})$	$(d + \frac{1}{16}) \times (2.5 \times d)$

Specifications:

Min Edge Distance: - AISC specification (J 3.4) for standard holes as follows:

TABLE J3.4
Minimum Edge Distance[a] from Center of Standard Hole[b] to Edge of Connected Part, in.

Bolt Diameter, in.	Minimum Edge Distance
1/2	3/4
5/8	7/8
3/4	1
7/8	1 1/8
1	1 1/4
1 1/8	1 1/2
1 1/4	1 5/8
Over 1 1/4	1 1/4d

[a] If necessary, lesser edge distances are permitted provided the applicable provisions from Sections J3.10 and J4 are satisfied, but edge distances less than one bolt diameter are not permitted without approval from the engineer of record.
[b] For oversized or slotted holes, see Table J3.5.

then for other types of holes, the edge distance is equal to the minimum for standard holes (above table) plus an increment C_2 :

TABLE J3.5
Values of Edge Distance Increment C_2, in.

Nominal Diameter of Fastener (in.)	Oversized Holes	Slotted Holes		
		Long Axis Perpendicular to Edge		Long Axis Parallel to Edge
		Short Slots	Long Slots[a]	
$\leq 7/8$	$1/16$	$1/8$	$3/4 d$	0
1	$1/8$	$1/8$		
$\geq 1\,1/8$	$1/8$	$3/16$		

[a]When length of slot is less than maximum allowable (see Table J3.3), C_2 is permitted to be reduced by one-half the difference between the maximum and actual slot lengths.

Min. spacing: minimum center to center distance : - (J 3.3)

$S \geq 3d \qquad (S.H., O.S.H, S.S.H., \& L.S.H.)$

<u>*Nominal bearing strength at bolt holes*</u>. (regarding the member) (J 3.6a)

$R_n = 1.2\, L_c\, t\, F_u \leq 2.4 d\, t\, F_u$ if deformation around the hole is a design consideration.

and

$R_n = 1.5\, L_c\, t\, F_u \leq 3 d\, t\, F_u$ if deformation around the hole is not a design consideration.

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)

For connections, the bearing resistance shall be taken as the sum of the bearing resistances of the individual bolts.

<u>Max. Edge distance</u>. (J 3.5)

$$l_{e_{max}} = 12\, t$$

$$l_{e_{max}} = 6''$$

Where (t) is the thickness of the connected part under consideration

Min. Connections. (J 1.6)

Connections shall be designed to support not less than 6 kips.

Design Calculations: - LRFD

1) Fasteners: -

❖ The shear resistance of the fastener: -The shear strength ϕR_n for the bolt according to shear rupture as follows:

❖ $R_n = F_v A_b$ nominal strength

$\phi R_n = \phi F_v A_b$ design strength of fastener where $\phi = 0.75$

$R_{S.S} = \frac{\pi d^2}{4} F_v$ ... ... for Single shear.

$R_{D.S} = 2 \times R_{S.S.}$ ... ...for Double shear.

2) Plate or the tension member around the hole: -

❖ The *design* bearing resistance of the plate at the back of the fastener: -

❖ $\phi R_n = \phi\, 1.2\, L_c\, t\, F_u \leq 2.4 d\, t\, F_u$ where $\phi = 0.75$

Table J3-2 nominal stress of Fasteners in ksi

3) The design strength of a tension member is the least of :

a- $\phi F_y A_g$ tensile yield away from the joint. $\phi=0.9$

b- $\phi F_u A_e$ tensile rupture at the joint. $\phi=0.75$

c- block shear.

d- the slenderness ratio. $L/r \leq 300$

e- bolts or weld strength.

a. To satisfy the first of these expressions, the minimum gross area must be at least equal to

$$\min A_g = \frac{P_u}{\phi_t F_y}.$$

b. To satisfy the second expression, the minimum value of A_e must be at least

$$\min A_e = \frac{P_u}{\phi_t F_u}.$$

And since $A_e = UA_n$ for a bolted member, the minimum value of A_n is

$$\min A_n = \frac{\min A_e}{U} = \frac{P_u}{\phi_t F_u U}.$$

Then the minimum A_g is

$$= \min A_n + \text{estimated area of holes}$$

$$= \frac{P_u}{\phi_t F_u U} + \text{estimated area of holes}$$

the net area may not be taken more than 0.85 A_g

TABLE J3.2
Nominal Strength of Fasteners and Threaded Parts, ksi (MPa)

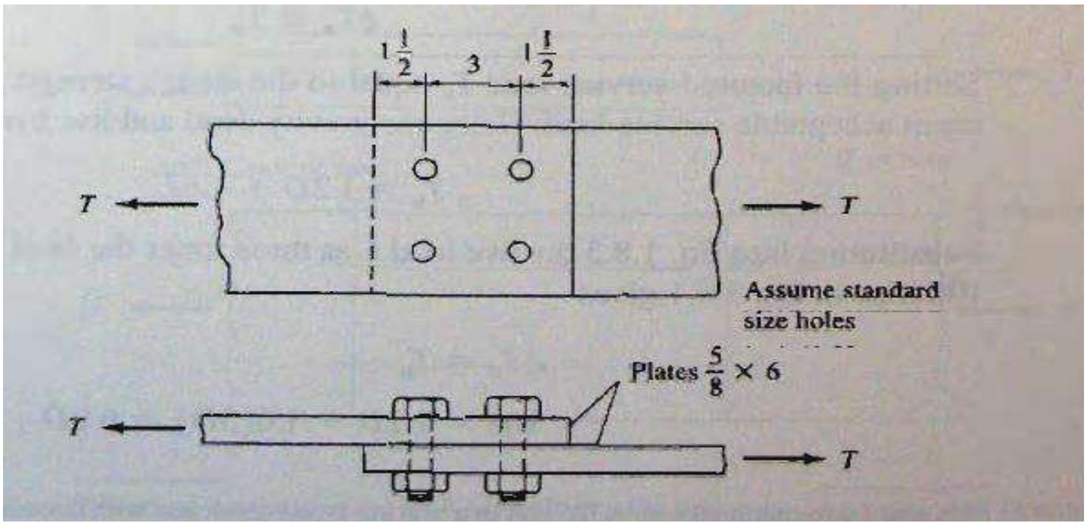
Description of Fasteners	Nominal Tensile Strength, F_{nt}, ksi (MPa)[a]	Nominal Shear Strength in Bearing-Type Connections, F_{nv}, ksi (MPa)[b]
A307 bolts	45 (310)[c]	27 (186)[c] [d]
Group A (e.g., A325) bolts, when threads are not excluded from shear planes	90 (620)	54 (372)
Group A (e.g., A325) bolts, when threads are excluded from shear planes	90 (620)	68 (469)
Group B (e.g., A490) bolts, when threads are not excluded from shear planes	113 (780)	68 (469)
Group B (e.g., A490) bolts, when threads are excluded from shear planes	113 (780)	84 (579)
Group C (e.g., F3043) bolt assemblies, when threads and transition area of shank are not excluded from the shear plane	150 (1040)	90 (620)
Group C (e.g., F3043) bolt assemblies, when threads and transition area of shank are excluded from the shear plane	150 (1040)	113 (779)
Threaded parts meeting the requirements of Section A3.4, when threads are not excluded from shear planes	$0.75F_u$	$0.450F_u$
Threaded parts meeting the requirements of Section A3.4, when threads are excluded from shear planes	$0.75F_u$	$0.563F_u$

[a] For high-strength bolts subject to tensile fatigue loading, see Appendix 3.
[b] For end loaded connections with a fastener pattern length greater than 38 in. (950 mm), F_{nv} shall be reduced to 83.3% of the tabulated values. Fastener pattern length is the maximum distance parallel to the line of force between the centerline of the bolts connecting two parts with one faying surface.
[c] For A307 bolts, the tabulated values shall be reduced by 1% for each $^{1}/_{16}$ in. (2 mm) over five diameters of length in the grip.
[d] Threads permitted in shear planes.

Ex.1 : Determine the tensile capacity of bearing type connection shown below if;

the bolt threads are excluded from the shear plan and

use bolts 7/8 in dia A325 and A572 grade 50 plate material with standard holes. Use LRFD procedure.(deformation is considered). Fu=65ksi



Solution:

Threads are excluded from shear plan:

A- *strength of plates:*

1-Yielding: ϕ Tn = $\phi F_y A_g$ = 0.9 x 50 x 3.75 =168.75 k

2-rupture ; ϕ Tn = $\phi F_u A_e$ = 0.75 F_u A_e

A_n= $[6 - 2\left(\frac{7}{8} + \frac{1}{16}\right)] 0.625 = 2.578\ in^2$

A_e= A_n=2.578 in ^{2}

ϕ T_n = 0.75 F_u A_e= 0.75* 65*2.578= 125.7 k control

B- *Strength of bolts:*

ϕR_n=ϕ F_{nv}*A_b=0.75* 68*0.6013=30.67 k/ bolt (Rss single shear) (F_v from table)

* bearing strength must also be checked.

ϕR_n= ϕ (1.2 L_c*t*Fu) $\leq$ ϕ(2.4dtFu)

L_c = 1.5-$(\frac{1}{2})(\frac{7}{8} + \frac{1}{16})$ = 1.03 or 3-(7/8+1/16) =2 in

ϕR_n= 0.75 (1.2*1.03*0.625*65) = 36.56 k/ bolt < 0.75*2.4*7/8*0.625*65=64 k
and for farthest bolt

ϕR_n= 0.75 (1.2*2*0.625*65) = 73.125 k/ bolt

$R_{ss} < R_b$ R_{ss} = 30.67 k control

Total capacity based on bolts= 30.67* 4 bolts= 122.68 k

So, strength of member = 122.68 k

End distance L_c should be checked

The end distance provided 1.5 in satisfies the 1.5in for sheared edge given in table.

Ex.14: Design the simple single lap joint carrying ultimate tensile load=160 k .(bearing type)shown knowing that deformation is considered , use:

a- 7/8 in dia. A325 bolts with standard holes.(threads are excluded from shear plan).

b- A-36 steel. F_u = 58 ksi c- 5/8 in plate is available

Solution:

$A_g = \frac{T}{0.9Fy} = \frac{160}{0.9*36} = 4.95$ in^2

$A_n = \frac{T}{0.75Fu} = \frac{160}{0.75*58} = 3.67$ in^2

$A_n \leq 0.85\ A_g$ $A_g = \frac{An}{0.85} = \frac{3.67}{0.85} = 4.33$ in^2

$A_e = A_n$

w*t =4.95 in^2

$w = \frac{4.95}{5/8} = 8''$

A_g= 8* 5/8 = 5 in^2

$R_{ss}=0.75*(\pi/4*\left((\frac{7}{8})^2\right)60 = 27$

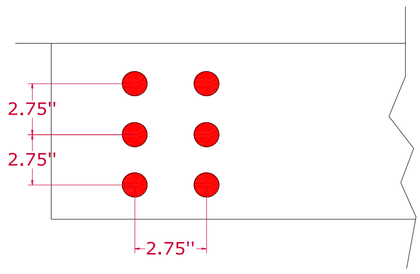
L_e= 1.5 in from table

$R_{\text{bearing}} = \phi(1.2*L_c*t*F_u) \leq \phi(2.4*d*t*F_u)$

$= 0.75*1.2*(1.5-0.5)*\frac{5}{8}*58 = 32.625$ k/bolt $<$ $0.75*2.4*\frac{7}{8}*\frac{5}{8}*58 = 57.1$ k/bolt no. of bolts required = 160/27 =5.9 use 6 bolts

Spacing: $S \geq 3d = 3*7/8 = 2.625$ in

S = 2.75 in



now width of plate =(2*2.75)+(2*1.5) =8.5 in check:

$T_{1\text{-}1} = [\, 8.5 - 3(\frac{7}{8} + \frac{1}{8})] \; \frac{5}{8} *0.75*58 = 149.53$ k

$T_{2\text{-}2} = [8.5 - 3(\frac{7}{8} + \frac{1}{8}) + 2\,\frac{(2.75)^2}{4*2.75}]\,\frac{5}{8} * 0.75 * 58 + 27 = 223\ k$

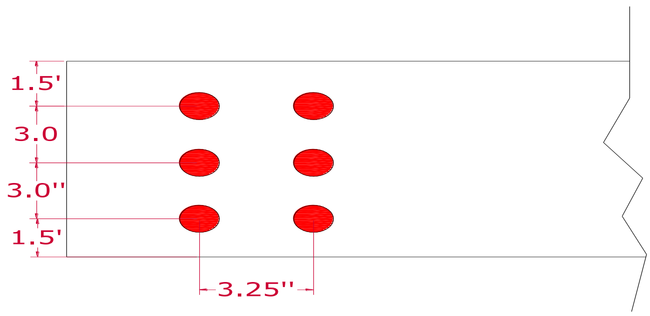
$T_{3\text{-}3} = [8.5-(3*1) + \frac{(2.75)^2}{4*2.75}]\,\frac{5}{8} *43.5 + 27 = 204$ k

$T_{Ag} = 8.5 *5/8 *0.9 *36 = 172$ k

$T_{1\text{-}1}$ control 149.53 k < 160 k not O.K

increase W = 9.0 in

$T_{1\text{-}1} = [9.0-3] *5/8 *43.5 = 163$ k > 160 k O.K



Ex.15: design a member carries factored tensile force =400k *in two lines in each flange* if:

a- W $_{10*w}$ section is available.

b- F_y=36 ksi F_u=60 ksi L= 23 ft

c- $\frac{7}{8}$ in dia. A490 bearing type bolt is used (threads are not excluded) and assume edge distance 2 in ,and center to center of holes 3 in.

d- the connection through gusset plates is through the flanges only.

Solution:

1- min. $A_g = P_u/0.9\ F_y = 400/0.9*36 = 12.34\ in^2$

2- min. $A_g = P_u/0.75\ F_uU$ + estimated hole areas

3- $r_{min.} \geq \frac{L}{300} = \frac{23*12}{300} = 0.92$ in

From AISC- 2005 LRFD tables pp(1-25 ,1-26):

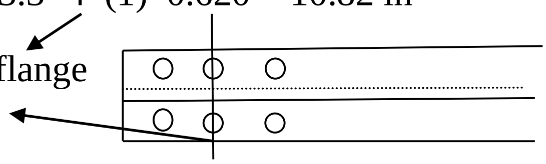
Try W $_{10*45}$ A=13.3 in ^{2} b_f= 8.02 in t_f= 0.620 in d= 10.1 in

t_w= 0.35 in r_x=4.32 in r_y= 2.01 in now from table to estimate U

$b_f \geq \frac{2}{3}$ d (8.02> 2/3 *10.1) then take U=0.9

$A_n = A_g - 4(7/8+1/8)\ t_f$ = 13.3- 4*(1)*0.620 = 10.82 in

failure section in flange



$\phi P_n = 0.75 F_u A_e = 0.75*60*(0.9*10.82) = 438$ k > 400 k O.K otherwise try heavier section

No of bolts:

$R_{s.s} = \pi/4\ d^2 * F_v = \pi/4*(\frac{7}{8})^2 * 60 = 36*0.75 = 27$ k (shearing strength of one bolt)

Lc lesser of $2 - \frac{\frac{7}{8}+\frac{1}{8}}{2} = 1.5$ in or 3-1= 2 in

$R_{bearing} = 0.75*(1.2 L_c*t*F_u) \leq (2.4d*t*Fu)$

$0.75(1.2*1.5*0.620*60) = 50.22 < 0.75*2.4*\frac{7}{8}*0.620*60 = 58.59$ k

∴ controlling strength of bolt = lesser of 27 or 50.22 which is 27 k

$\therefore No.of\ bolts = \frac{400}{27} = 15 = 16\ bolts$ *(8 in each flange)*

$\therefore No.of\ bolts > 3$ so U= 0.9 ok

Or to find the value of U we can use the equation $U = 1 - \frac{x}{L}$

for W sections *connected through flanges only* we will assume that the section is split into two structural tees, then the value of x used will be the distance from the outside edge of the flange to the c.g. of the structural tee.

So, referring to tables in manual for one half of a W10*45 (or that is a WT5x22.5) we find that x = 0.907 in

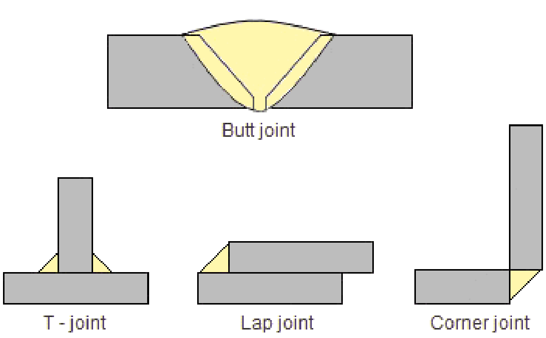
$U = 1 - \frac{0.907}{9} = 0.89$ <0.9 so, use 0.9 as permitted by AISC manual 2005

Welding

The process of welding denotes the jointing of metal pieces by heating to fluid state with or without pressure. (electric or gas welding).

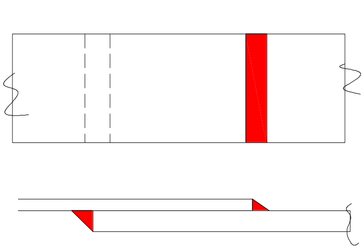
Types of joints: - but joint, lap (most common) , tee, corner..

.

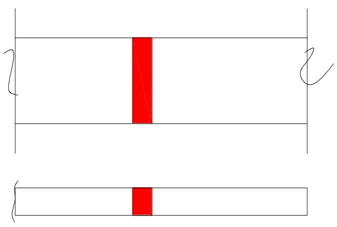


Butt joint is usually for end plates or members of nearly same thickness

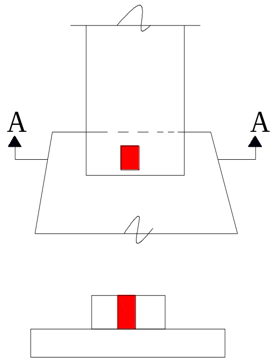
Types of weld:-



fillet weld



groove



slot

1. ***Groove weld: -***

Used to connect structural member that are aligned in the same plane.

❖ The weld must have the same strength as the pieces jointed.

2. ***Fillet weld:*** -

Is the most widely used in all of basic welds.

3. ***Slot & plug weld:*** -

Are used with or without fillet weld. The principal use is to transmit shear in the lap joint when the size of connections limits the length available for fillet or other edge weld.

Welding processes: there are many welding processes , the most common used are:

1- Shielded metal Arc Welding,(SMAW) usually done manually, typically used in field .

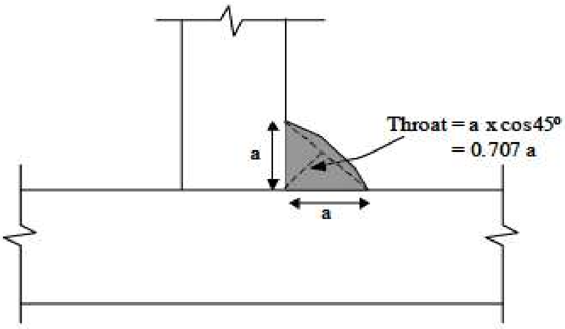
2- Submerged Arc Welding (SAW), usually done automatically, used for shop welding.

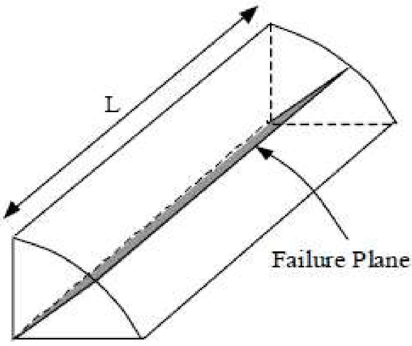
NOTE: we will consider here only the fillet weld with SMAW process.

Stress on welds: - (J.2.5 AISC 2005

- Fillet welds assumed for design purposes to transmit loads through "shear stress" on the effective area.
- A fillet weld can be loaded in any direction in shear, compression, or tension. However, it always **fails in *shear***.

- The shear failure of *the fillet weld* occurs along a plane through the throat of the weld, as shown in the Figure below.





designed weld strength= $\phi * F_w * 0.707a * L_w$ where

F_w is the weld strength and L_w weld length

- the tensile strength of the weld electrode can be 60,70,80,90,100,110,120 ksi.

-this strength is written in standard form E70XX ,means that the ultimate strength of the wire(electrode) is 70 ksi and so on.

Design strength of fillet weld is based on two limits:

1- For the base metal, the designed Shear *yield strength* is $\phi R_n = \phi F_{bm} A_{bm}$ *AISC Equation J2-2*

$= 0.9(0.6Fy) t_p L_w$

Shear Rupture $=0.75(0.6Fu)\, t_p L_w$

2- For the weld metal, $\phi R_n = \phi F_w A_w$ *AISC Equation J2-3* where:

F_w= nominal strength of weld metal

A_w = effective area of weld = $t_p * L_w$

$\phi = 0.75$

Table J2.5 in AISC specification provide the weld values to use in the above two equations.

for example:

F_w= nominal strength of fillet weld metal =0.6 F_{Exx}

and if E70XX is used then for the weld metal $\phi R_n = 0.75 * 0.6 * 70 = 31.5$ ksi

* always check weld metal and base metal strength. Smaller value govern.

❖ * In weld design problems it is advantageous to work with strength per unit length of the weld or base metal.

Size & limitation of fillet weld: - AISC –J2. 2 P.16.1-54

acos45 = effective size of weld.

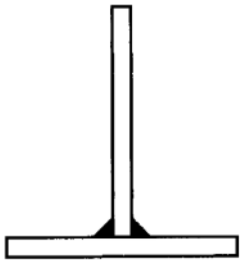
❖ Min. size of weld is based on the thinner of pieces being joined .

– "***a***" ***min. are given in AISC table J.2.4***

Material thickness of thinner part joined (in) mm	Min. size of fillet weld	
	(inch)	(mm)
To $\frac{1}{4}$" inclusive	1/8	**3**
Over $\frac{1}{4} - \frac{1}{2}$	3/16	**5**
Over $\frac{1}{2}$ to $\frac{3}{4}$	1/4	**6**
Over 3/4	5/16	**8**

– "***a***"$\ngtr$ t of the thinner plate.

For fillet welds other than along edges as in fig. below, the maximum size to be used in strength computation would be limited by the base metal strength.



Max. size of fillet weld: -

"***a***" max. is limited to prevent melting of base material.

$a_{max} = t \qquad iF \quad t < \frac{1}{4}''$

$a_{max} = t - \frac{1}{16} \qquad iF \quad t \geq \frac{1}{4}''$

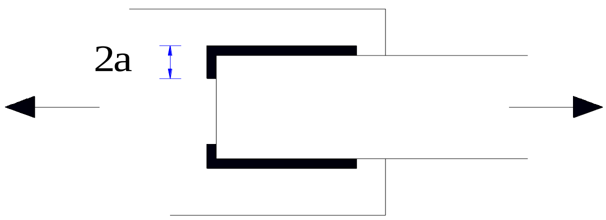
(along edge of material)

❖ ***Min. effective length of Fillet weld:*** - J – 2 .2a

- Min. effective length $\nless 4a$ otherwise a $_{eff.}$=L_w/4
- For end member AISC recommends end return around corners>$2a$

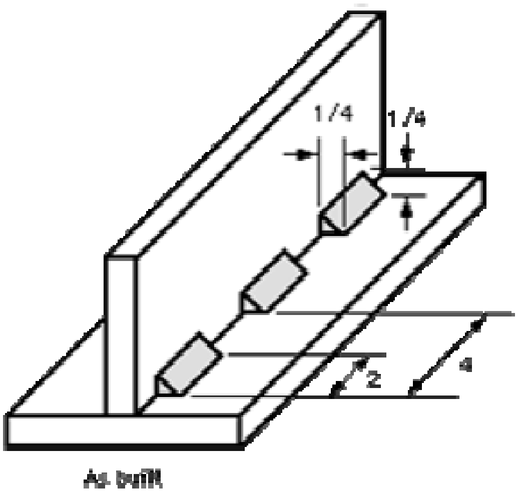
Effective area = effective length* effective throat thickness

(.707 a)



- ***Intermittent fillet weld***: (used when the strength required is less than that required by continuous weld).

$l \geq 4a \nless 1.5''$

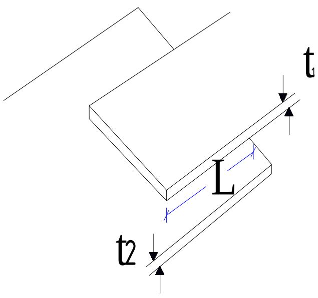


$l_{min} = 1\frac{1}{2}''$

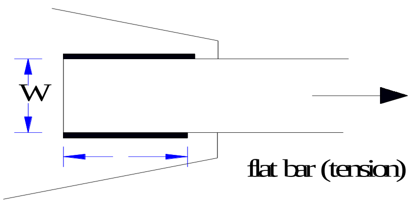
Lap joint: -

Min. lap shall be 5 times the thin part of joined materials, but not $< 1''$

$$l_{min} = 5 \times \text{smaller of } t_1 \text{ or } t_2 \not< 1''$$



IF ***longitudinal*** Fillet welds are used alone in end member (tension member)(plates or flat bar) then the length of each fillet weld shall not be taken less than the perpendicular distance between them

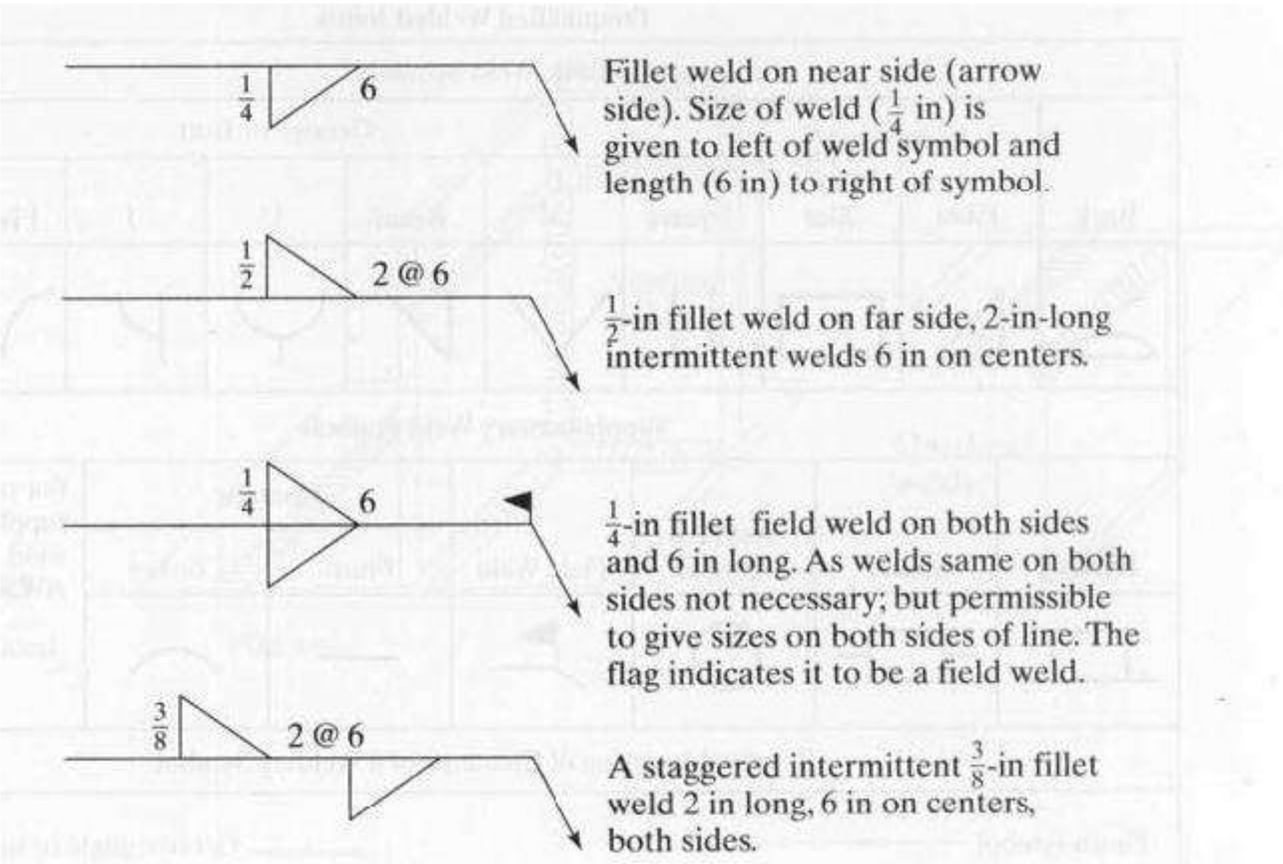


-maximum effective length in end locations-AISC J2.2b

- if weld length $L_w <$ 100a , then effective weld length=L_w
- if $L_w <$ 300a ,then effective weld length = $L_w(1.2\text{-}0.002L_w/a)$

-if $L_w > 300a$, then effective weld length =0.6 L_w

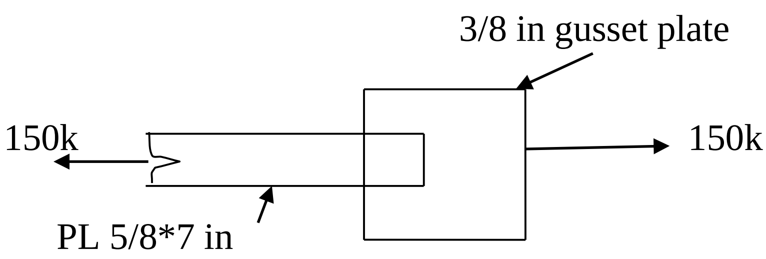
Standard location of elements of welding symbol: -



__Ex.16:__ - Determine the size & length of fillet weld for the lap joint. Use E70xx electrode and $F_y = 50$ ksi for the plate.Fu= 58 ksi

Solution: -

- $a_{max.} = \frac{5}{8} - \frac{1}{16} = \frac{9}{16}''$
- $a_{min.} = \frac{3}{16}''$



Use $a = \frac{3}{8}$ in

(shear force / in)$_{weld}$= ϕR_n=0.75*0.6 F_{EXX}*0.707*a

= 8.35 kips/in

(shear force / in)$_{plate}$= ϕR_n= 0.9*0.6F_y*t

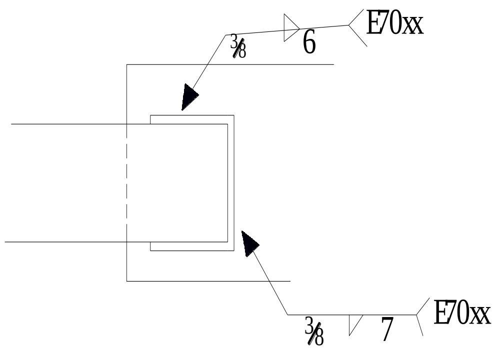
= 10.1 k/in

Shear rupture of the base plate= 0.75*0.6* 58*3/8= 9.78 k

shear strength of weld metal governs =8.35 k/in

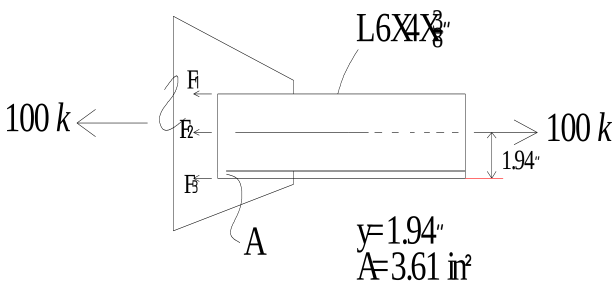
Length of weld (l_w)= $\frac{150}{8.35} = 17.96 \simeq 18.0''$

check limitation: 100a = 100*3/8=37.5 in > 9 in



Ex.17- Design the fillet weld to develop the full strength of angle shown in fig. minimizing the effect of eccentricity.

Use A – 36 steel, E70xx electrode. Also find "t" of gusset plate.



Solution

$a_{min.} = \frac{3}{16}$ in table

$$a_{max.} = \frac{3}{8} - \frac{1}{16} = \frac{5}{16}''$$

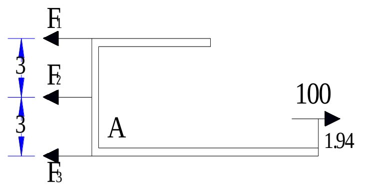
Use $a = \frac{3}{16}''$

$S.F/_{in}$ weld = 0.75*t*0.6F$_{EXX}$*1 in=0.75*$(\frac{3}{16} * 0.707) * (0.6 * 70) =$ 4.18 k/in

$F_2 = 6 \times 4.18 = 25.1\ k$

$\sum F_x = 0$

F_1+F_3=150-25.1= 125 kips



$\sum M_A = 0$

$6F_1 + 3F_2 = 150 \times 1.94$

$6F_1 + (3 \times 25.1) = 150 \times 1.94$

$F_1 = 36\ k$

$\therefore\ F_3 = 150 - 25.1 - 36 = 89\ k$

$l_{w1} = \frac{36}{4.18} = 8.6''$ $\quad say\ 9\ inch$

$l_{w3} = \frac{89}{4.18} = 21.3''$ say 22 inch

Thickness of gusset plate

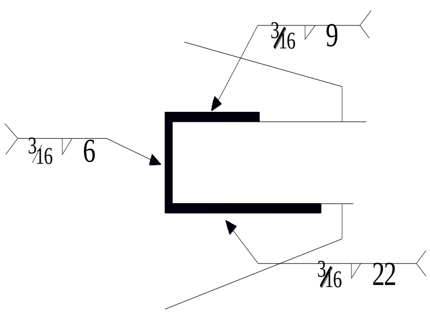
ϕR_n = 0.9*0.6F$_y$*area of base metal subjected to shear

=0.9*0.6*36**t*1in

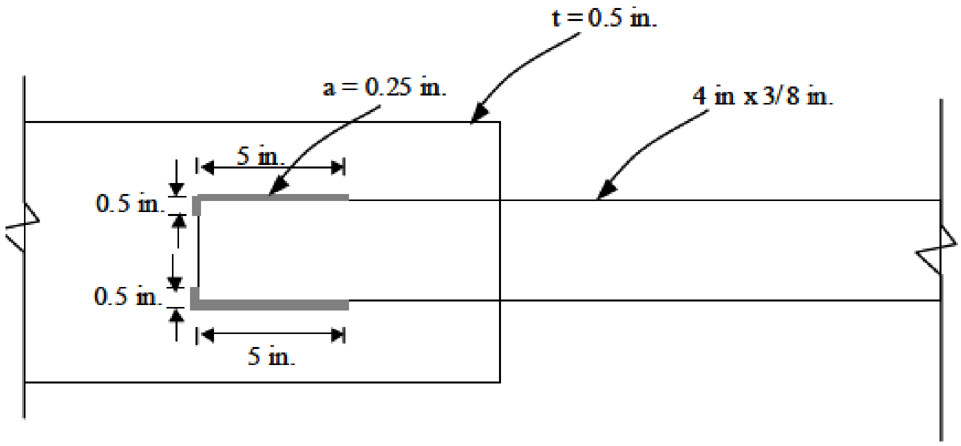
$4.18 = 0.9 * 0.6 * 36\ t \times 1$

$t = 0.215''$

use $\frac{1}{4}''$



Ex.18: - What is the bearing capacity of the connection shown in the fig. below, if A 50 steel used (F_u= 65 ksi), AISC specification, and E70 electrodes are used.



Step I. Check for the limitations on the weld geometry

t_{min} = 3/8 in. (member)

t_{max} = 0.5 in. (gusset)

Therefore, a_{min} = 3/16 in. - AISC **Table J2.4**

a_{max} = 3/8 - 1/16 = 5/16 in. - AISC **J2.2b**

Fillet weld size = a = 1/4 in. - *Therefore, OK!*

$L_{w\text{-}min}$ = 1.0 in. - OK.

- $L_{w\text{-}min}$ for each length of the weld = 4.0 in. (*transverse distance between welds*, see **J2.2b**)

- Given length = 5.0 in., which is > L_{min}. Therefore, *OK*!

Length/weld size = 5/0.25 = 20 < 100 - *Therefore, maximum effective length* **J2.2 b** *satisfied.*

End returns at the edge corner size - minimum = 2 a = 0.5 in. -*Therefore, OK!*

step 2: ***design strength of the weld***

Weld strength =ϕ* 0.707 * a * 0.60 * F_{EXX} * L_w

=0.75* 0.707 * 0.25 * 0.6* 70 * 10 = 55.67 kips

Base Metal strength =ϕ* 0.6 * F_y * L_w * t

$=0.9 * 0.6 * 50 * 10 * 3/8 = 101.25$ kips

Step 3. Tension strength of the member

$\phi R_n = 0.9 * 50 * 4 * 3/8 = 67.5$ kips - tension yield

$\phi R_n = 0.75 * A_e * 65$ - tension fracture

$A_e = U A$

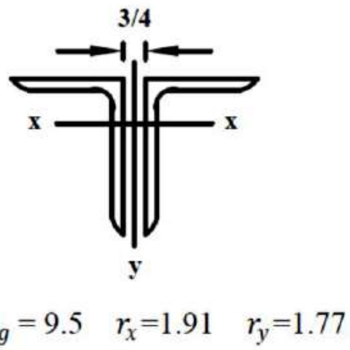
$A = A_g = 4 * 3/8 = 1.5 \text{ in}^2$ - See Spec. B3

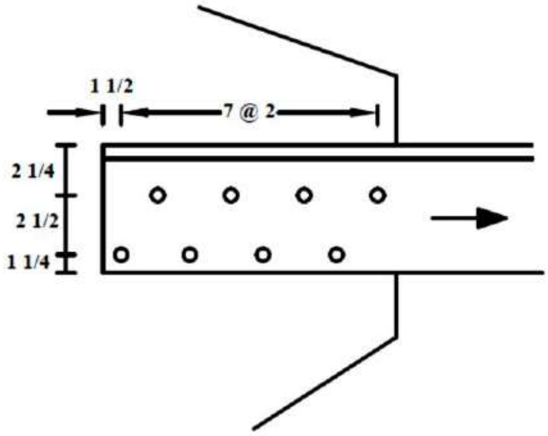
$U = 0.75$, since connection length (L_{conn}) < 1.5 w - See Spec. B3

Therefore, $\phi R_n = 54.8$ kips

The design strength of the member-connection system = 54.8 kips. Tension fracture of the member governs. The end returns at the corners were not included in the calculation.

Ex: Determine the load capacity in tension for a 2L6*4*1/2 in LLBB angle separated by 3/4 in back to back. Use A242 Grade50 steel, Fu=65 ksi &7/8 in bolts with S.H. L=20ft. Assume A 325 bolts is used S.H. Bearing types Threads are excluded from shear plan.





Solution:

$l/r_{min} \leq 300$

(20x12)/1.77 = 135.59 < 300 o.k.

$(L_e)_{min.}$ =1 1/2 sheared edge =1 1/2 o.k.

= 1 1/8 rolled edge < 1 1/4 o.k.

S= 3d =3(7/8) =2.625 < 4 o.k.

LRFD ,table 1-15 p,1-102 for a pair of L 6*4*1/2 in A_g= 9.5 in^2

LRFD ,table1-7 p,1-42 for L6*4*1/2 in x=0.981 in

1- yielding on A_g .... ϕT_n=0.9*50*Ag= 45*9.5=427.5 k............1

2- Fracture on A_e..............An_1= 2[9.5-(7/8+1/8)]*0.5= 8.5 in^2

$$An_2 = 2[9.5-2(7/8+1/8) +\frac{2^2}{4*2.5}]*0.5 = 7.9\ in^2 \text{ control}$$

U= 1-$\frac{x}{L}$ = 1-$\frac{0.981}{14}$ = 0.93> 0.9 then use 0.9

$\phi T_n = 0.75 F_u A_e$ = 0.75*65*(0.9*7.9) = 346.61 k 2

LRFD ,table 1-15 p,1-102 for <u>2 L 6*4*1/2 in</u> separated by 3/8 in plate $r_{min.}$=1. 64 in

now bolt strength:

$R_{D.S.} = 2[\pi(7/16)^2]60 = 72$ k control

Total 72 *8 = 432 k

$R_{bearing} = 1.2 L_c t F_u \le 2.4\, d t F_u$

$L_c = 1.5 - 0.5 = 1$

$R_{bearing}$ (gusset plate) = 1.2*1*0.75*65 = 58.5 k < 2.4*7/8*0.75*65 =102.14 k

$R_{bearing}$ (gusset plate) = 1.2*(3.5-1) *0.75*65 = 146.25 k > 2.4*7/8*0.75*65 =102.14

$R_{bearing}$ (gusset plate) = 1.2*(4-1) *0.75*65 = 175.5 k > 2.4*7/8*0.75*65 =102.14

Design bearing strength for 8 bolts = 0.75(58.5 + 7 x 102.375)= 581 k--------------3

$$\therefore\ Capacity\ of\ section\ is\ the\ \ least\ of\ 1,2,3\ = 346.1\ k$$

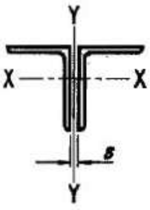
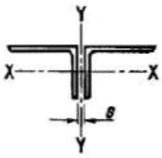


Table 1–15 (continued)
Double Angles
Properties



Shape	Area	Axis Y-Y Radius of Gyration LLBB Separation, s, in.			Axis Y-Y Radius of Gyration SLBB Separation, s, in.			LLBB Q_s Angles in Contact	LLBB Q_s Angles Separated	LLBB r_x	SLBB Q_s Angles in Contact	SLBB Q_s Angles Separated	SLBB r_x
	in.²	0	3/8	3/4	0	3/8	3/4			in.			in.
2L6×4×7/8	16.0	1.57	1.71	1.86	2.82	2.96	3.11	1.00	1.00	1.86	1.00	1.00	1.10
×3/4	13.9	1.55	1.68	1.83	2.80	2.94	3.08	1.00	1.00	1.88	1.00	1.00	1.12
×5/8	11.7	1.53	1.66	1.80	2.77	2.91	3.06	1.00	1.00	1.89	1.00	1.00	1.13
×9/16	10.6	1.52	1.65	1.79	2.76	2.90	3.04	1.00	1.00	1.90	1.00	1.00	1.14
×1/2	9.50	1.51	1.64	1.77	2.75	2.89	3.03	1.00	1.00	1.91	1.00	1.00	1.14
×7/16	8.36	1.50	1.62	1.76	2.74	2.88	3.02	1.00	0.973	1.92	1.00	0.973	1.15
×3/8	7.22	1.49	1.61	1.75	2.73	2.86	3.00	1.00	0.912	1.93	0.998	0.912	1.16
×5/16	6.05	1.48	1.60	1.74	2.72	2.85	2.99	1.00	0.826	1.94	0.914	0.826	1.17

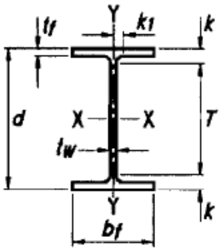
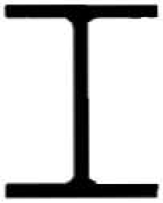


Table 1-1 (continued)
W Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k k_{des}	Distance k k_{det}	k_1	T	Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W12×58	17.0	12.2	12 1/4	0.360	3/8	3/16	10.0	10	0.640	5/8	1.24	1 1/2	15/16	9 1/4	5 1/2
×53	15.6	12.1	12	0.345	3/8	3/16	10.0	10	0.575	9/16	1.18	1 3/8	15/16	9 1/4	5 1/2
W12×50	14.6	12.2	12 1/4	0.370	3/8	3/16	8.08	8 1/8	0.640	5/8	1.14	1 1/2	15/16	9 1/4	5 1/2
×45	13.1	12.1	12	0.335	5/16	3/16	8.05	8	0.575	9/16	1.08	1 3/8	15/16	↓	↓
×40	11.7	11.9	12	0.295	5/16	3/16	8.01	8	0.515	1/2	1.02	1 3/8	7/8	↓	↓
W12×35[c]	10.3	12.5	12 1/2	0.300	5/16	3/16	6.56	6 1/2	0.520	1/2	0.820	1 3/16	3/4	10 1/8	3 1/2
×30[c]	8.79	12.3	12 3/8	0.260	1/4	1/8	6.52	6 1/2	0.440	7/16	0.740	1 1/8	3/4	↓	↓
×26[c]	7.65	12.2	12 1/4	0.230	1/4	1/8	6.49	6 1/2	0.380	3/8	0.680	1 1/16	3/4	↓	↓
W12×22[c]	6.48	12.3	12 1/4	0.260	1/4	1/8	4.03	4	0.425	7/16	0.725	15/16	5/8	10 3/8	2 1/4[g]
×19[c]	5.57	12.2	12 1/8	0.235	1/4	1/8	4.01	4	0.350	3/8	0.650	7/8	9/16	↓	↓
×16[c]	4.71	12.0	12	0.220	1/4	1/8	3.99	4	0.265	1/4	0.565	13/16	9/16	↓	↓
×14[c,v]	4.16	11.9	11 7/8	0.200	3/16	1/8	3.97	4	0.225	1/4	0.525	3/4	9/16	↓	↓
W10×112	32.9	11.4	11 3/8	0.755	3/4	3/8	10.4	10 3/8	1.25	1 1/4	1.75	1 15/16	1	7 1/2	5 1/2
×100	29.4	11.1	11 1/8	0.680	11/16	3/8	10.3	10 3/8	1.12	1 1/8	1.62	1 13/16	1	↓	↓
×88	25.9	10.8	10 7/8	0.605	5/8	5/16	10.3	10 1/4	0.990	1	1.49	1 11/16	15/16	↓	↓
×77	22.6	10.6	10 5/8	0.530	1/2	1/4	10.2	10 1/4	0.870	7/8	1.37	1 9/16	7/8	↓	↓
×68	20.0	10.4	10 3/8	0.470	1/2	1/4	10.1	10 1/8	0.770	3/4	1.27	1 7/16	7/8	↓	↓
×60	17.6	10.2	10 1/4	0.420	7/16	1/4	10.1	10 1/8	0.680	11/16	1.18	1 3/8	13/16	↓	↓
×54	15.8	10.1	10 1/8	0.370	3/8	3/16	10.0	10	0.615	5/8	1.12	1 5/16	13/16	↓	↓
×49	14.4	10.0	10	0.340	5/16	3/16	10.0	10	0.560	9/16	1.06	1 1/4	13/16	↓	↓
W10×45	13.3	10.1	10 1/8	0.350	3/8	3/16	8.02	8	0.620	5/8	1.12	1 5/16	13/16	7 1/2	5 1/2
×39	11.5	9.92	9 7/8	0.315	5/16	3/16	7.99	8	0.530	1/2	1.03	1 3/16	13/16	↓	↓
×33	9.71	9.73	9 3/4	0.290	5/16	3/16	7.96	8	0.435	7/16	0.935	1 1/8	3/4	↓	↓
W10×30	8.84	10.5	10 1/2	0.300	5/16	3/16	5.81	5 3/4	0.510	1/2	0.810	1 1/8	11/16	8 1/4	2 3/4[g]
×26	7.61	10.3	10 3/8	0.260	1/4	1/8	5.77	5 3/4	0.440	7/16	0.740	1 1/16	11/16	↓	↓
×22[c]	6.49	10.2	10 1/8	0.240	1/4	1/8	5.75	5 3/4	0.360	3/8	0.660	15/16	5/8	↓	↓
W10×19	5.62	10.2	10 1/4	0.250	1/4	1/8	4.02	4	0.395	3/8	0.695	15/16	5/8	8 3/8	2 1/4[g]
×17[c]	4.99	10.1	10 1/8	0.240	1/4	1/8	4.01	4	0.330	5/16	0.630	7/8	9/16	↓	↓
×15[c]	4.41	10.0	10	0.230	1/4	1/8	4.00	4	0.270	1/4	0.570	13/16	9/16	↓	↓
×12[c,f]	3.54	9.87	9 7/8	0.190	3/16	1/8	3.96	4	0.210	3/16	0.510	3/4	9/16	↓	↓

[c] Shape is slender for compression with $F_y = 50$ ksi.
[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.
[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.
[v] Shape does not meet the h/t_w limit for shear in Specification Section G2.1a with $F_y = 50$ ksi.

Table 1-1 (continued)
W Shapes
Properties



W12 – W10

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
58	7.82	27.0	475	78.0	5.28	86.4	107	21.4	2.51	32.5	2.82	11.6	2.10	3570
53	8.69	28.1	425	70.6	5.23	77.9	95.8	19.2	2.48	29.1	2.79	11.5	1.58	3160
50	6.31	26.8	391	64.2	5.18	71.9	56.3	13.9	1.96	21.3	2.25	11.6	1.71	1880
45	7.00	29.6	348	57.7	5.15	64.2	50.0	12.4	1.95	19.0	2.23	11.5	1.26	1650
40	7.77	33.6	307	51.5	5.13	57.0	44.1	11.0	1.94	16.8	2.21	11.4	0.906	1440
35	6.31	36.2	285	45.6	5.25	51.2	24.5	7.47	1.54	11.5	1.79	12.0	0.741	879
30	7.41	41.8	238	38.6	5.21	43.1	20.3	6.24	1.52	9.56	1.77	11.9	0.457	720
26	8.54	47.2	204	33.4	5.17	37.2	17.3	5.34	1.51	8.17	1.75	11.8	0.300	607
22	4.74	41.8	156	25.4	4.91	29.3	4.66	2.31	0.848	3.66	1.04	11.9	0.293	164
19	5.72	46.2	130	21.3	4.82	24.7	3.76	1.88	0.822	2.98	1.02	11.8	0.180	131
16	7.53	49.4	103	17.1	4.67	20.1	2.82	1.41	0.773	2.26	0.982	11.7	0.103	96.9
14	8.82	54.3	88.6	14.9	4.62	17.4	2.36	1.19	0.753	1.90	0.962	11.7	0.0704	80.4
112	4.17	10.4	716	126	4.66	147	236	45.3	2.68	69.2	3.07	10.1	15.1	6020
100	4.62	11.6	623	112	4.60	130	207	40.0	2.65	61.0	3.03	10.0	10.9	5150
88	5.18	13.0	534	98.5	4.54	113	179	34.8	2.63	53.1	2.99	9.85	7.53	4330
77	5.86	14.8	455	85.9	4.49	97.6	154	30.1	2.60	45.9	2.95	9.73	5.11	3630
68	6.58	16.7	394	75.7	4.44	85.3	134	26.4	2.59	40.1	2.91	9.63	3.56	3100
60	7.41	18.7	341	66.7	4.39	74.6	116	23.0	2.57	35.0	2.88	9.54	2.48	2640
54	8.15	21.2	303	60.0	4.37	66.6	103	20.6	2.56	31.3	2.86	9.48	1.82	2320
49	8.93	23.1	272	54.6	4.35	60.4	93.4	18.7	2.54	28.3	2.84	9.42	1.39	2070
45	6.47	22.5	248	49.1	4.32	54.9	53.4	13.3	2.01	20.3	2.27	9.48	1.51	1200
39	7.53	25.0	209	42.1	4.27	46.8	45.0	11.3	1.98	17.2	2.24	9.39	0.976	992
33	9.15	27.1	171	35.0	4.19	38.8	36.6	9.20	1.94	14.0	2.20	9.30	0.583	791
30	5.70	29.5	170	32.4	4.38	36.6	16.7	5.75	1.37	8.84	1.60	10.0	0.622	414
26	6.56	34.0	144	27.9	4.35	31.3	14.1	4.89	1.36	7.50	1.58	9.89	0.402	345
22	7.99	36.9	118	23.2	4.27	26.0	11.4	3.97	1.33	6.10	1.55	9.81	0.239	275
19	5.09	35.4	96.3	18.8	4.14	21.6	4.29	2.14	0.874	3.35	1.06	9.85	0.233	104
17	6.08	36.9	81.9	16.2	4.05	18.7	3.56	1.78	0.845	2.80	1.04	9.78	0.156	85.1
15	7.41	38.5	68.9	13.8	3.95	16.0	2.89	1.45	0.810	2.30	1.01	9.72	0.104	68.3
12	9.43	46.6	53.8	10.9	3.90	12.6	2.18	1.10	0.785	1.74	0.983	9.66	0.0547	50.9

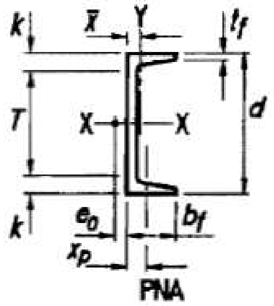


Table 1–5
C Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k	Distance T	Distance Workable Gage	r_{ts}	h_o
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
C15×50	14.7	15.0	15	0.716	11/16	3/8	3.72	3 3/4	0.650	5/8	1 7/16	12 1/8	2 1/4	1.17	14.4
×40	11.8	15.0	15	0.520	1/2	1/4	3.52	3 1/2	0.650	5/8	1 7/16	12 1/8	2	1.15	14.4
×33.9	10.0	15.0	15	0.400	3/8	3/16	3.40	3 3/8	0.650	5/8	1 7/16	12 1/8	2	1.13	14.4
C12×30	8.81	12.0	12	0.510	1/2	1/4	3.17	3 1/8	0.501	1/2	1 1/8	9 3/4	1 3/4 g	1.01	11.5
×25	7.34	12.0	12	0.387	3/8	3/16	3.05	3	0.501	1/2	1 1/8	9 3/4	1 3/4 g	1.00	11.5
×20.7	6.08	12.0	12	0.282	5/16	3/16	2.94	3	0.501	1/2	1 1/8	9 3/4	1 3/4 g	0.983	11.5

Table 1–5 (continued)
C Shapes
Properties

C SHAPES

Nominal Wt.	Shear Ctr, e_o	Axis X-X I	Axis X-X S	Axis X-X r	Axis X-X Z	Axis Y-Y I	Axis Y-Y S	Axis Y-Y r	Axis Y-Y $\bar{x}$	Axis Y-Y Z	Axis Y-Y x_p	Torsional Properties J	Torsional Properties C_w	Torsional Properties $\bar{r}_o$	Torsional Properties H
lb/ft	in.	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{6}	in.	
50	0.583	404	53.8	5.24	68.5	11.0	3.77	0.865	0.799	8.14	0.490	2.65	492	5.49	0.937
40	0.767	348	46.5	5.45	57.5	9.17	3.34	0.883	0.778	6.84	0.392	1.45	410	5.73	0.927
33.9	0.896	315	42.0	5.62	50.8	8.07	3.09	0.901	0.788	6.19	0.332	1.01	358	5.94	0.920
30	0.618	162	27.0	4.29	33.8	5.12	2.05	0.762	0.674	4.32	0.367	0.861	151	4.54	0.919
25	0.746	144	24.0	4.43	29.4	4.45	1.87	0.779	0.674	3.82	0.306	0.538	130	4.72	0.909
20.7	0.870	129	21.5	4.61	25.6	3.86	1.72	0.797	0.698	3.47	0.253	0.369	112	4.93	0.899
30	0.368	103	20.7	3.42	26.7	3.93	1.65	0.668	0.649	3.78	0.441	1.22	79.5	3.63	0.922
25	0.494	91.1	18.2	3.52	23.1	3.34	1.47	0.675	0.617	3.18	0.367	0.687	68.3	3.75	0.912
20	0.636	78.9	15.8	3.66	19.4	2.80	1.31	0.690	0.606	2.70	0.294	0.368	56.9	3.93	0.900
15.3	0.796	67.3	13.5	3.87	15.9	2.27	1.15	0.711	0.634	2.34	0.224	0.209	45.5	4.19	0.884

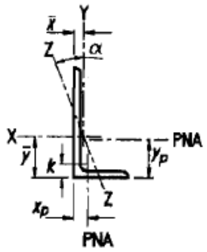
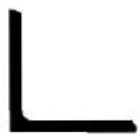


Table 1–7
Angles
Properties

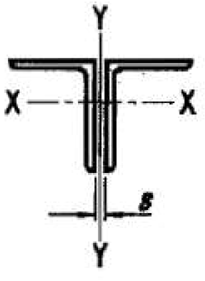
Shape	k	Wt.	Area, A	Axis X-X						Flexural-Torsional Properties		
				I	S	r	$\bar{y}$	Z	y_p	J	C_w	$\bar{r}_o$
	in.	lb/ft	in.^{2}	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{6}	in.
L8×8×$1\frac{1}{8}$	$1\frac{3}{4}$	56.9	16.7	98.1	17.5	2.41	2.40	31.6	1.05	7.13	32.5	4.29
×1	$1\frac{5}{8}$	51.0	15.0	89.1	15.8	2.43	2.36	28.5	0.943	5.08	23.4	4.32
×$\frac{7}{8}$	$1\frac{1}{2}$	45.0	13.2	79.7	14.0	2.45	2.31	25.3	0.832	3.46	16.1	4.36
×$\frac{3}{4}$	$1\frac{3}{8}$	38.9	11.4	69.9	12.2	2.46	2.26	22.0	0.720	2.21	10.4	4.39
×$\frac{5}{8}$	$1\frac{1}{4}$	32.7	9.61	59.6	10.3	2.48	2.21	18.6	0.606	1.30	6.16	4.42
×$\frac{9}{16}$	$1\frac{3}{16}$	29.6	8.68	54.2	9.33	2.49	2.19	16.8	0.548	0.961	4.55	4.43
×$\frac{1}{2}$	$1\frac{1}{8}$	26.4	7.75	48.8	8.36	2.49	2.17	15.1	0.490	0.683	3.23	4.45
L8×6×1	$1\frac{1}{2}$	44.2	13.0	80.9	15.1	2.49	2.65	27.3	1.47	4.34	16.3	3.88
×$\frac{7}{8}$	$1\frac{3}{8}$	39.1	11.5	72.4	13.4	2.50	2.60	24.3	1.41	2.96	11.3	3.92
×$\frac{3}{4}$	$1\frac{1}{4}$	33.8	9.94	63.5	11.7	2.52	2.55	21.1	1.34	1.90	7.28	3.95
×$\frac{5}{8}$	$1\frac{1}{8}$	28.5	8.36	54.2	9.86	2.54	2.50	17.9	1.27	1.12	4.33	3.98
×$\frac{9}{16}$	$1\frac{1}{16}$	25.7	7.56	49.4	8.94	2.55	2.48	16.2	1.23	0.823	3.20	3.99
×$\frac{1}{2}$	1	23.0	6.75	44.4	8.01	2.55	2.46	14.6	1.20	0.584	2.28	4.01
×$\frac{7}{16}$	$\frac{15}{16}$	20.2	5.93	39.3	7.06	2.56	2.43	12.9	1.16	0.396	1.55	4.02

Table 1–7 (continued)
Angles
Properties



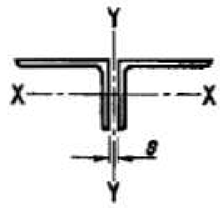
L8-L6

Shape	Axis Y-Y						Axis Z-Z				Q_s
	I	S	r	$\bar{x}$	Z	x_p	I	S	r	Tan α	$F_y = 36$ ksi
	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.		
L8×8×$1\frac{1}{8}$	98.1	17.5	2.41	2.40	31.6	1.05	40.9	7.23	1.56	1.00	1.00
×1	89.1	15.8	2.43	2.36	28.5	0.943	36.8	6.51	1.56	1.00	1.00
×$\frac{7}{8}$	79.7	14.0	2.45	2.31	25.3	0.832	32.7	5.78	1.57	1.00	1.00
×$\frac{3}{4}$	69.9	12.2	2.46	2.26	22.0	0.720	28.5	5.04	1.57	1.00	1.00
×$\frac{5}{8}$	59.6	10.3	2.48	2.21	18.6	0.606	24.2	4.27	1.58	1.00	0.997
×$\frac{9}{16}$	54.2	9.33	2.49	2.19	16.8	0.548	22.0	3.88	1.58	1.00	0.959
×$\frac{1}{2}$	48.8	8.36	2.49	2.17	15.1	0.490	19.7	3.49	1.59	1.00	0.912
L8×6×1	38.8	8.92	1.72	1.65	16.2	0.816	21.3	4.84	1.28	0.542	1.00
×$\frac{7}{8}$	34.9	7.94	1.74	1.60	14.4	0.721	18.9	4.31	1.28	0.546	1.00
×$\frac{3}{4}$	30.8	6.92	1.75	1.56	12.5	0.624	16.5	3.78	1.29	0.550	1.00
×$\frac{5}{8}$	26.4	5.88	1.77	1.51	10.5	0.526	14.1	3.22	1.29	0.554	0.997
×$\frac{9}{16}$	24.1	5.34	1.78	1.49	9.52	0.476	12.8	2.94	1.30	0.556	0.959
×$\frac{1}{2}$	21.7	4.79	1.79	1.46	8.52	0.425	11.5	2.64	1.30	0.557	0.912
×$\frac{7}{16}$	19.3	4.23	1.80	1.44	7.50	0.374	10.2	2.35	1.31	0.559	0.850



LLBB

Table 1–15 (continued)
Double Angles
Properties



SLBB

Shape	Area	Axis Y-Y						LLBB			SLBB		
		Radius of Gyration						Q_s		r_x	Q_s		r_x
		LLBB			SLBB			Angles in Contact	Angles Separated		Angles in Contact	Angles Separated	
		Separation, s, in.			Separation, s, in.								
	in.^{2}	0	$^3/_8$	$^3/_4$	0	$^3/_8$	$^3/_4$			in.			in.
2L6×4×$^7/_8$	16.0	1.57	1.71	1.86	2.82	2.96	3.11	1.00	1.00	1.86	1.00	1.00	1.10
×$^3/_4$	13.9	1.55	1.68	1.83	2.80	2.94	3.08	1.00	1.00	1.88	1.00	1.00	1.12
×$^5/_8$	11.7	1.53	1.66	1.80	2.77	2.91	3.06	1.00	1.00	1.89	1.00	1.00	1.13
×$^9/_{16}$	10.6	1.52	1.65	1.79	2.76	2.90	3.04	1.00	1.00	1.90	1.00	1.00	1.14
×$^1/_2$	9.50	1.51	1.64	1.77	2.75	2.89	3.03	1.00	1.00	1.91	1.00	1.00	1.14
×$^7/_{16}$	8.36	1.50	1.62	1.76	2.74	2.88	3.02	1.00	0.973	1.92	1.00	0.973	1.15
×$^3/_8$	7.22	1.49	1.61	1.75	2.73	2.86	3.00	1.00	0.912	1.93	0.998	0.912	1.16
×$^5/_{16}$	6.05	1.48	1.60	1.74	2.72	2.85	2.99	1.00	0.826	1.94	0.914	0.826	1.17
2L6×3$^1/_2$×$^1/_2$	9.04	1.27	1.40	1.54	2.82	2.96	3.11	1.00	1.00	1.92	1.00	1.00	0.968
×$^3/_8$	6.88	1.26	1.38	1.52	2.80	2.94	3.08	1.00	0.912	1.93	0.998	0.912	0.984
×$^5/_{16}$	5.78	1.25	1.37	1.50	2.78	2.92	3.06	1.00	0.826	1.94	0.914	0.826	0.991
2L5×5×$^7/_8$	16.0	2.16	2.30	2.44	2.16	2.30	2.44	1.00	1.00	1.49	1.00	1.00	1.49
×$^3/_4$	14.0	2.13	2.27	2.41	2.13	2.27	2.41	1.00	1.00	1.50	1.00	1.00	1.50
×$^5/_8$	11.8	2.11	2.25	2.39	2.11	2.25	2.39	1.00	1.00	1.52	1.00	1.00	1.52
×$^1/_2$	9.58	2.09	2.22	2.36	2.09	2.22	2.36	1.00	1.00	1.53	1.00	1.00	1.53
×$^7/_{16}$	8.44	2.08	2.21	2.35	2.08	2.21	2.35	1.00	1.00	1.54	1.00	1.00	1.54
×$^3/_8$	7.30	2.07	2.20	2.34	2.07	2.20	2.34	1.00	0.983	1.55	1.00	0.983	1.55
×$^5/_{16}$	6.13	2.06	2.19	2.32	2.06	2.19	2.32	0.998	0.912	1.56	0.998	0.912	1.56
2L5×3$^1/_2$×$^3/_4$	11.6	1.39	1.53	1.68	2.33	2.47	2.62	1.00	1.00	1.55	1.00	1.00	0.974
×$^5/_8$	9.85	1.37	1.50	1.65	2.30	2.45	2.59	1.00	1.00	1.56	1.00	1.00	0.987
×$^1/_2$	8.01	1.35	1.48	1.62	2.28	2.42	2.57	1.00	1.00	1.58	1.00	1.00	1.00
×$^3/_8$	6.10	1.33	1.46	1.59	2.26	2.39	2.54	1.00	0.983	1.59	1.00	0.983	1.02
×$^5/_{16}$	5.12	1.32	1.44	1.58	2.25	2.38	2.52	1.00	0.912	1.60	0.998	0.912	1.02
×$^1/_4$	4.13	1.31	1.43	1.57	2.23	2.37	2.51	1.00	0.804	1.61	0.894	0.804	1.03
2L5×3×$^1/_2$	7.51	1.11	1.24	1.39	2.35	2.50	2.64	1.00	1.00	1.58	1.00	1.00	0.824
×$^7/_{16}$	6.62	1.10	1.23	1.38	2.34	2.48	2.63	1.00	1.00	1.59	1.00	1.00	0.831
×$^3/_8$	5.73	1.09	1.22	1.36	2.33	2.47	2.62	1.00	0.983	1.60	1.00	0.983	0.838
×$^5/_{16}$	4.81	1.08	1.21	1.35	2.32	2.46	2.60	1.00	0.912	1.61	0.998	0.912	0.846
×$^1/_4$	3.88	1.07	1.19	1.33	2.30	2.44	2.58	1.00	0.804	1.62	0.894	0.804	0.853