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Basic Steel Design
Session L1: Tension Members
February 25, 2021

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Course Description

Tension Members

This lecture will focus on the design of structural steel tension members and the associated provisions in the AISC *Specification*. The session will begin with a discussion of strength, calculating area and available strength of tension members. Emphasis will then be placed on designing tension members before discussing the concept of block shear. The session concludes with an overview of eyebars, pin connected members, truss members and braces. Several design examples will be presented.

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Learning Objectives

- List the limit state that must be checked for the design of tension members.
- Compare the use of gross area, net area and effective net area in the design of structural steel tension members.
- List the unique design requirements for eyebars and pin connected members.
- Demonstrate the design of tension members through a design example.

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Basic Steel Design: A review of the principles of steel design according to ANSI/AISC 360-16

Winter Webinar 2021

Lesson L1

Tension Members

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L1.7

Lesson L1 – Tension Members

- Tension Members
  - Strength
  - Area calculations
  - Available strength
  - Design
  - Block shear
  - Eyebars and pin connected members

L1.8

Tension Members

B3.1. For LRFD, design shall be performed in accordance with:

Required Strength $\leq$ Available Strength

$$R_u \leq \phi R_n \qquad \text{(B3-1)}$$

where

R_u = required strength (LRFD) defined in Chapter C
R_n = nominal strength specified in Chapter D
ϕ = resistance factor specified in Chapter D
ϕR_n= design strength = resistance factor (nominal strength)

L1.9

Tension Members

B3.2. For ASD, design shall be performed in accordance with:

Required Strength $\leq$ Available Strength

$$R_a \leq R_n / \Omega \qquad \text{(B3-2)}$$

where

R_a = required strength (ASD) defined in Chapter C
R_n = nominal strength specified in Chapter D
Ω = safety factor specified in Chapter D

R_n/Ω = allowable strength = $\frac{\text{nominal strength}}{\text{safety factor}}$

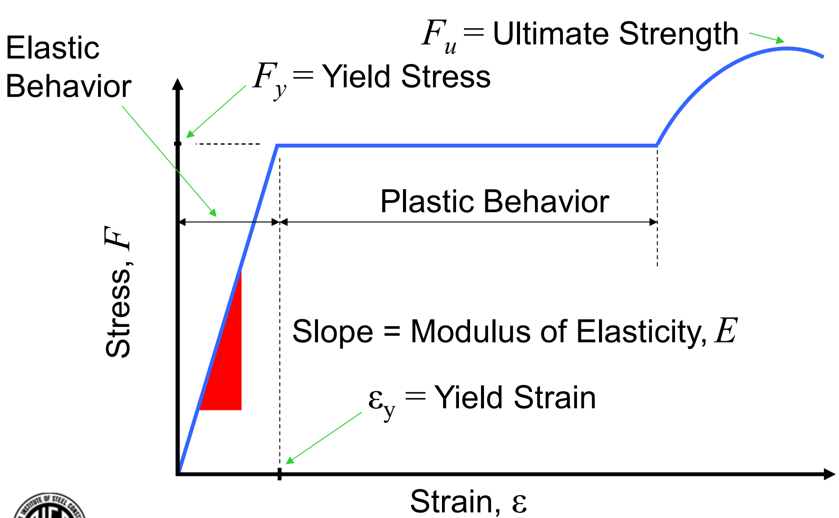
L1.10

Tension Members

D2. "The design tensile strength, $\phi_t P_n$, and the allowable tensile strength, P_n/Ω_t, of tension members, shall be the lower value obtained according to the limit states of tensile yielding in the gross section and tensile rupture in the net section."

L1.11

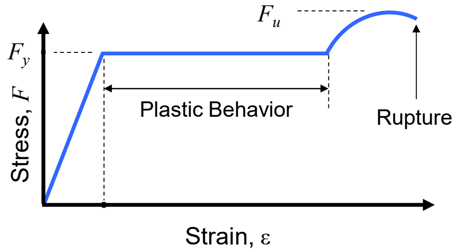
Steel as a Material



L1.12

Tension Members

- Tensile Test Specimen



L1.13

Tension Members

Two limit states to be considered for tension

D2.(a) Nominal Strength for Yielding;

$$P_n = F_y A_g \qquad \text{(D2-1)}$$

$\phi_t = 0.90$ (LRFD) $\quad \Omega_t = 1.67$ (ASD)

L1.14

Tension Members

Two limit states to be considered for tension

D2.(b) Nominal Strength for Rupture

$$P_n = F_u A_e \qquad \text{(D2-2)}$$

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

L1.15

Design for Tension

- Allowable Strength (ASD), $P_a \le \frac{P_n}{\Omega}$;

$$P_a \le 0.6 F_y A_g$$

$$\le 0.5 F_u A_e$$

L1.16

Tension Members

- Design Strength (LRFD), $P_u \leq \phi P_n$

$$P_u \leq 0.9 F_y A_g$$
$$\leq 0.75 F_u A_e$$

L1.17

Area Calculations

- Three different areas are defined for use in design of tension members.
  - Gross Area, A_g
  - Net Area, A_n
  - Effective Net Area, A_e

L1.18

B4.3a. Gross Area, A_g

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

$A_g = A$

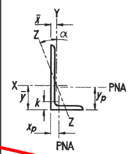


Table 1-7
Angles
Properties

Shape	k	Wt.	Area, A	Axis X-X						Flexural Prop	
				I	S	r	$\bar{y}$	Z	y_p	J	
	in.	lb/ft	in.^{2}	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	
L8×8×1 1/8	1 3/4	56.9	16.8	98.1	17.5	2.41	2.40	31.6	1.05	7.13	32
×1	1 5/8	51.0	15.1	89.1	15.8	2.43	2.36	28.5	0.944	5.08	23
×7/8	1 1/2	45.0	13.3	79.7	14.0	2.45	2.31	25.3	0.831	3.46	16
×3/4	1 3/8	38.9	11.5	69.9	12.2	2.46	2.26	22.0	0.719	2.21	10

L1.19

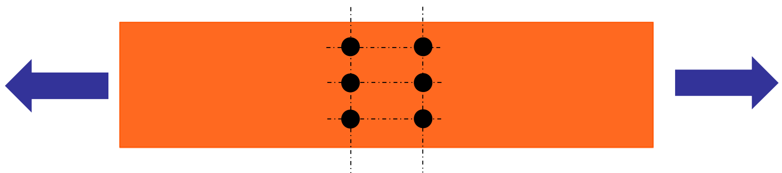
B4.3b. Net Area, A_n

- Net area accounts for holes in the member
- In the simplest case it is the gross area less the area of holes, (d_b + 1/16" + 1/16")t
  - d_b = bolt diameter
  - t = element thickness
  - (d_b + 1/16 in.) = the hole size for standard holes and bolts less than 1.0 in. in diameter
  - The other 1/16 in. is to account for damage to the element due to punching of the hole

L1.20

B4.3b. Net Area, A_n

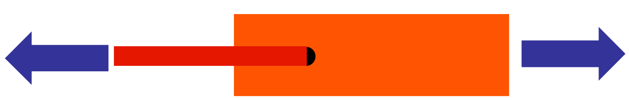
- Net area accounts for holes in the member
- In the simplest case it is the gross area less the area of holes, $(d_b + 1/16" + 1/16")t$



L1.21

B4.3b. Net Area, A_n

- For HSS with slots, deduct full width of the slots times the thickness



- For members without holes, $A_n = A_g$

L1.22

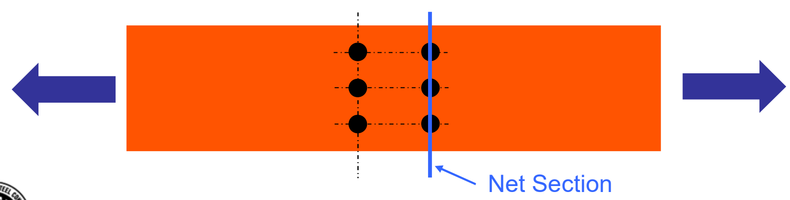
B4.3b. Net Area, A_n

- Look at the transverse section and determine the amount to be deducted

6 – ¾ in. bolts
8 in. plate width
½ in. plate thickness

$$A_g = tb = 0.5(8.0) = 4.0 \text{ in.}^2$$

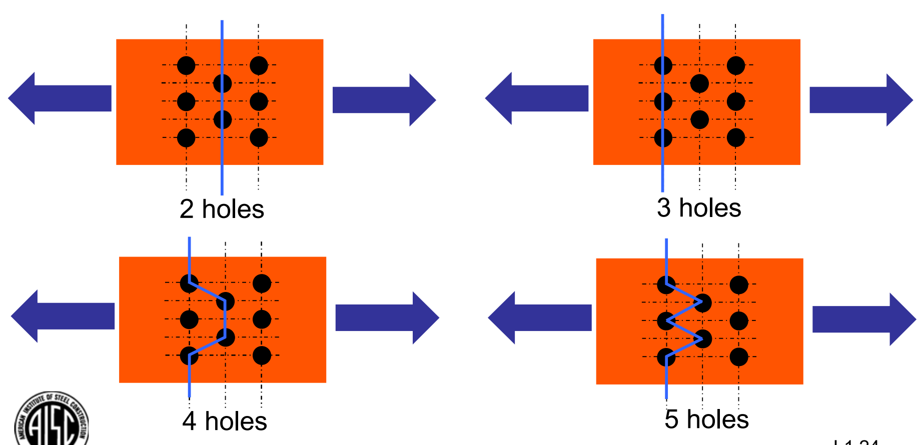
$$A_n = 4.0 - 3\left(\tfrac{3}{4} + \tfrac{1}{16} + \tfrac{1}{16}\right)(0.5) = 2.69 \text{ in.}^2$$



L1.23

B4.3b. Net Area, A_n

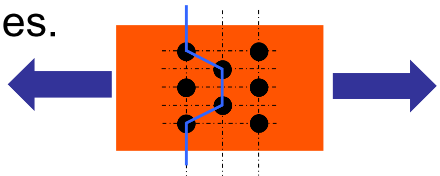
- What happens if the bolts are staggered?



L1.24

B4.3b. Net Area, A_n

- Look at the chain (path) if we are to deduct for 4 holes.



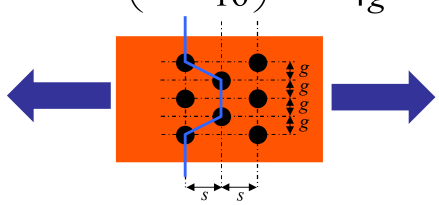
- There is more area to resist the force along the diagonals than if the holes were all in a straight line. So, after we deduct the 4 holes we must add something back.

L1.25

B4.3b. Net Area, A_n

Net width = gross plate width less the holes plus the term $s^2/4g$ for each diagonal path where s is the hole spacing and g is the gage

$$b_n = b - n\left(d_h + \frac{1}{16}\right) + \sum \frac{s^2}{4g}$$

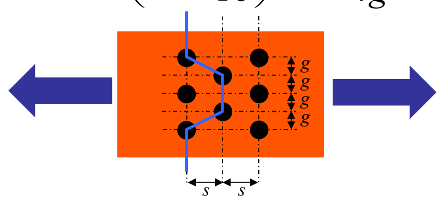


L1.26

B4.3b. Net Area, A_n

Consider a plate width of 11.0 in.
Holes for 5/8 in. bolts
Spacing of 4.0 in.
Gage of 2.0 in.

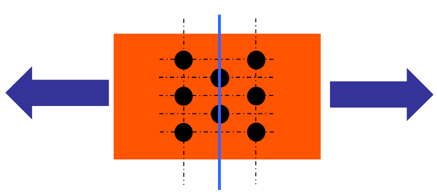
$$b_n = b - n\left(d_h + \frac{1}{16}\right) + \sum \frac{s^2}{4g}$$



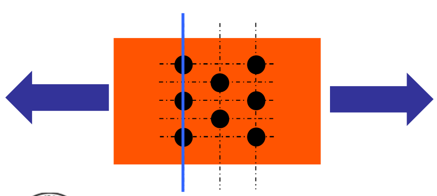
L1.27

B4.3b. Net Area, A_n

Look at the 4 possible chains



$$b_{n2} = 11.0 - 2\left(5/8 + 1/16 + 1/16\right)$$
$$= 11.0 - 1.5 = 9.5 \text{ in.}$$

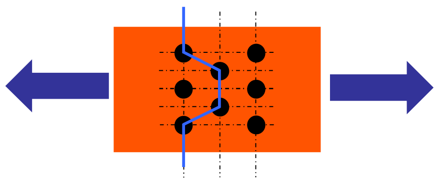


$$b_{n3} = 11.0 - 3\left(5/8 + 1/16 + 1/16\right)$$
$$= 11.0 - 2.25 = 8.75 \text{ in.}$$

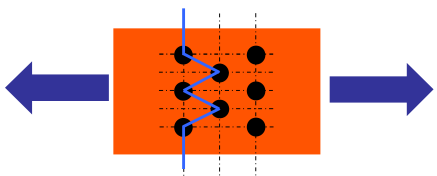
L1.28

B4.3b. Net Area, A_n

Look at the 4 possible chains



$$b_{n4} = 11.0 - 4\left(\tfrac{5}{8} + \tfrac{1}{16} + \tfrac{1}{16}\right) + 2\left(\frac{4^2}{4(2)}\right)$$
$$= 11.0 - 3.0 + 4 = 12.0 \leq 11.0$$



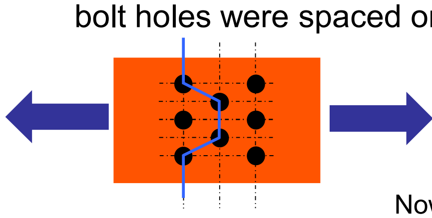
$$b_{n5} = 11.0 - 5\left(\tfrac{5}{8} + \tfrac{1}{16} + \tfrac{1}{16}\right) + 4\left(\frac{4^2}{4(2)}\right)$$
$$= 11.0 - 3.75 + 8.0 = 15.3 \leq 11.0$$

Therefore, $b_{n3} = 8.75$ in. controls

L1.29

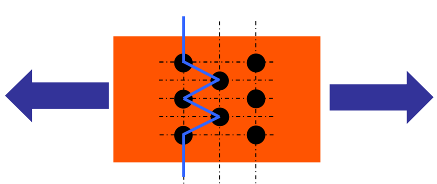
B4.3b. Net Area, A_n

Consider what would happen if the vertical lines of bolt holes were spaced only 2.0 in. apart



$$b_{n4} = 11.0 - 4\left(\tfrac{5}{8} + \tfrac{1}{16} + \tfrac{1}{16}\right) + 2\left(\frac{2^2}{4(2)}\right)$$
$$= 11.0 - 3.0 + 1 = 9.0$$

Now both chains show an area reduction



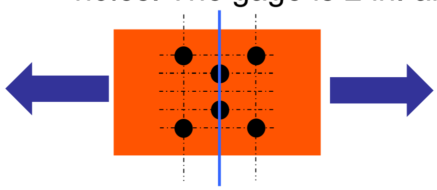
$$b_{n5} = 11.0 - 5\left(\tfrac{5}{8} + \tfrac{1}{16} + \tfrac{1}{16}\right) + 4\left(\frac{2^2}{4(2)}\right)$$
$$= 11.0 - 3.75 + 2.0 = 9.25$$

But, $b_{n3} = 8.75$ in. still controls

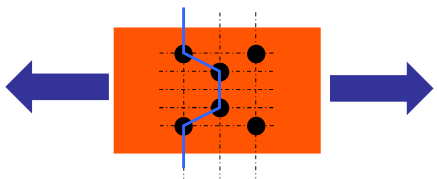
L1.30

B4.3b. Net Area, A_n

But what if each vertical line only contained 2 holes. The gage is 2 in. and the spacing is 2.0 in.



$$b_{n2} = 11.0 - 2\left(5/8 + 1/16 + 1/16\right) = 11.0 - 1.5 = 9.5 \text{ in.}$$



$$b_{n4} = 11.0 - 4\left(5/8 + 1/16 + 1/16\right) + 2\left(\frac{2^2}{4(2)}\right) = 11.0 - 3.0 + 1 = 9.0$$

Therefore, $b_{n4} = 9.0$ in. now controls

L1.31

D3. Effective Net Area, A_e

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

- *U* is the shear lag factor
- It accounts for unattached elements of a tension member at a connection

L1.32

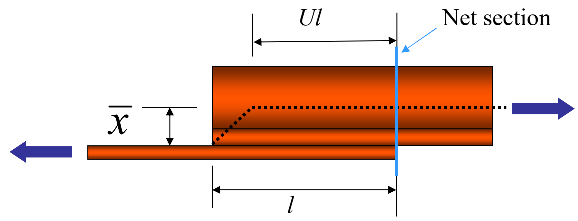
D3. Effective Net Area, A_e

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

- Account for unattached element
  - Examples: stem of T or outstanding leg of angle

l is defined here for a welded connection

$$U = 1 - \frac{\bar{x}}{l}$$



L1.33

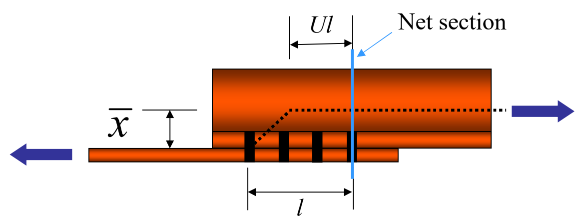
D3. Effective Net Area, A_e

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

- Account for unattached element
  - Examples: stem of T or outstanding leg of angle

l is defined here for a bolted connection

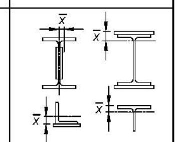
$$U = 1 - \frac{\bar{x}}{l}$$



L1.34

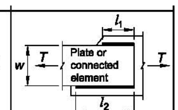
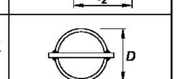
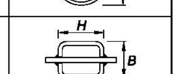
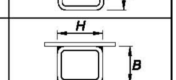
Table D3.1

TABLE D3.1
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 HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or by longitudinal welds in combination with transverse welds. Alternatively, Case 7 is permitted for W, M, S and HP shapes. (For angles, Case 8 is permitted to be used.)	$U = 1 - \frac{\bar{x}}{l}$	
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	–

L1.35

Table D3.1 continued

Case	Description of Element		Shear Lag Factor, U	Example
4[a]	Plates, angles, channels with welds at heels, tees, and W-shapes with connected elements, where the tension load is transmitted by longitudinal welds only. See Case 2 for definition of $\bar{x}$.		$U = \frac{3l^2}{3l^2 + w^2}\left(1 - \frac{\bar{x}}{l}\right)$	
5	Round HSS with a single concentric gusset plate through slots in the HSS.		$l \geq 1.3D$, $U = 1.0$ $D \leq l < 1.3D$, $U = 1 - \frac{\bar{x}}{l}$ $\bar{x} = \frac{D}{\pi}$	
6	Rectangular HSS.	with a single concentric gusset plate	$l \geq H$, $U = 1 - \frac{\bar{x}}{l}$ $\bar{x} = \frac{B^2 + 2BH}{4(B+H)}$	
		with two side gusset plates	$l \geq H$, $U = 1 - \frac{\bar{x}}{l}$ $\bar{x} = \frac{B^2}{4(B+H)}$	

L1.36

Table D3.1 continued

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 three or more fasteners per line in the direction of loading	$b_f \geq \frac{2}{3}d$, $U = 0.90$ $b_f < \frac{2}{3}d$, $U = 0.85$	–
		with web connected with four 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 four or more fasteners per line in the direction of loading	$U = 0.80$	–
		with three fasteners per line in the direction of loading (with fewer than three fasteners per line in the direction of loading, use Case 2)	$U = 0.60$	–

B = overall width of rectangular HSS member, measured 90° to the plane of the connection, in. (mm); D = outside diameter of round HSS, in. (mm); H = overall height of rectangular HSS member, measured in the plane of the connection, in. (mm); d = depth of section, in. (mm); for tees, d = depth of the section from which the tee was cut, in. (mm); l = length of connection, in. (mm); w = width of plate, in. (mm); $\bar{x}$ = eccentricity of connection, in. (mm).

[a] $l = \frac{l_1 + l_2}{2}$, where l_1 and l_2 shall not be less than 4 times the weld size.

L1.37

D3. Effective Net Area, A_e

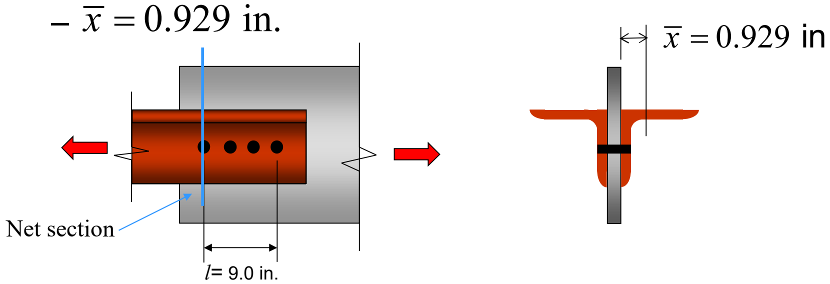
- Lower bound, for a single bolt or when sufficient information is not available to more accurately calculate *U*.

"For open cross-sections such as W, M, S, C, or HP shapes, WT's, ST's and single and double angles; the shear lag factor, *U*, need not be less than the ratio of the gross area of the connected element(s) to the member gross area."

L1.38

Example 1

- Determine the available tensile strength
  - 2 - L3 x 3 x 1/2 x 10'-0", A_g = 5.52 in.^{2}
  - A36 (F_y = 36 ksi)
  - 4 -7/8 in. A325-N bolts spaced at 3.0 in.
  - $\bar{x} = 0.929$ in.



L1.39

Example 1

- Net area
  - 2 angles, one hole in each angle

$$A_n = A_g - 2 \text{ holes}$$
$$= 5.52 - 2\left[(7/8 + 1/8)(1/2)\right] = 4.52 \text{ in.}^2$$

L1.40

Example 1

- Shear lag factor
  - Case 2 $$U = 1 - \frac{\bar{x}}{l} = 1 - \frac{0.929}{9} = 0.90$$ ☆
  - Case 8 $$U = 0.8$$
  - Minimum *U* $$U = \frac{3(0.5)}{2.76} = 0.543$$

L1.41

Example 1

- Effective net area

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

$$= 4.52(0.90) = 4.07 \text{ in.}^2$$

L1.42

Example 1

- Yielding

$$P_n = (F_y A_g) \quad \text{(D2-1)}$$
$$= (36)(5.52) = 199 \text{ kips}$$

- Rupture

$$P_n = (F_u A_e) \quad \text{(D2-2)}$$
$$= (58)(4.07) = 236 \text{ kips}$$

L1.43

Example 1

- ASD
  - Yielding

$$\frac{P_n}{\Omega} = \frac{199}{1.67} = 119 \text{ kips}$$

  - Rupture

$$\frac{P_n}{\Omega} = \frac{236}{2.00} = 118 \text{ kips}$$

Bolt limit states: 130 kips

L1.44

Example 1

- LRFD
  - Yielding

$$\phi_t P_n = 0.90(199) = 179 \text{ kips}$$

  - Rupture

$$\phi_t P_n = 0.75(236) = 177 \text{ kips}$$

Bolt limit states: 195 kips

The same limit state controls for ASD and LRFD

L1.45

Example 1

Yielding

For ASD

$2(59.5) = 119$ kips

For LRFD

$2(89.4) = 179$ kips

Just as we have determined

Table 5-2 (continued)
Available Strength in Axial Tension
Angles

F_y = 36 ksi
F_u = 58 ksi

L3½–L2½

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding, kips		Rupture, kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
L3½×3×½	3.02	2.27	65.1	97.8	65.8	98.7
×7/16	2.67	2.00	57.6	86.5	58.0	87.0
×3/8	2.32	1.74	50.0	75.2	50.5	75.7
×5/16	1.95	1.46	42.0	63.2	42.3	63.5
×1/4	1.58	1.19	34.1	51.2	34.5	51.8
L3½×2½×½	2.77	2.08	59.7	89.7	60.3	90.5
×3/8	2.12	1.59	45.7	68.7	46.1	69.2
×5/16	1.79	1.34	38.6	58.0	38.9	58.3
×1/4	1.45	1.09	31.3	47.0	31.6	47.4
L3×3×½	2.76	2.07	59.5	89.4	60.0	90.0
×7/16	2.43	1.82	52.4	78.7	52.8	79.2
×3/8	2.11	1.58	45.5	68.4	45.8	68.7
×5/16	1.78	1.34	38.4	57.7	38.9	58.3
×1/4	1.44	1.08	31.0	46.7	31.3	47.0
×3/16	1.09	0.818	23.5	35.3	23.7	35.6

L1.46

Example 1

Note that
$A_e = 0.75A_g$

Rupture

$$\frac{A_e}{A_g} = \frac{4.07}{5.52}$$
$$= 0.737$$

For ASD

$$2\left(\frac{0.737}{0.75}\right)(60.0) = 118 \text{ kips}$$

For LRFD

$$2\left(\frac{0.737}{0.75}\right)(90.0) = 177 \text{ kips}$$

Table 5-2 (continued)
Available Strength in Axial Tension
Angles

F_y = 36 ksi
F_u = 58 ksi

L3½–L2½

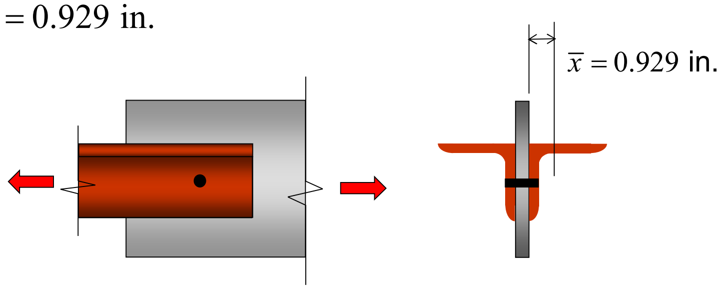
Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding, kips		Rupture, kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
L3½×3×½	3.02	2.27	65.1	97.8	65.8	98.7
×7/16	2.67	2.00	57.6	86.5	58.0	87.0
×3/8	2.32	1.74	50.0	75.2	50.5	75.7
×5/16	1.95	1.46	42.0	63.2	42.3	63.5
×¼	1.58	1.19	34.1	51.2	34.5	51.8
L3½×2½×½	2.77	2.08	59.7	89.7	60.3	90.5
×3/8	2.12	1.59	45.7	68.7	46.1	69.2
×5/16	1.79	1.34	38.6	58.0	38.9	58.3
×¼	1.45	1.09	31.3	47.0	31.6	47.4
L3×3×½	2.76	2.07	59.5	89.4	60.0	90.0
×7/16	2.43	1.82	52.4	78.7	52.8	79.2
×3/8	2.11	1.58	45.5	68.4	45.8	68.7
×5/16	1.78	1.34	38.4	57.7	38.9	58.3
×¼	1.44	1.08	31.0	46.7	31.3	47.0
×3/16	1.09	0.818	23.5	35.3	23.7	35.6

L1.47

Example 2

Example 1 with only one bolt as in a kicker

- 2 - L3 x 3 x 1/2 x 10'-0", A_g = 5.52 in.²
- A36 (F_y = 36 ksi)
- 1 -7/8 in. A325-N bolt
- $\bar{x} = 0.929$ in.



L1.48

Example 2

- Net area (Same as Example 1)
  - 2 angles, one hole in each angle

$$A_n = A_g - 2 \text{ holes}$$
$$= 5.52 - 2\left[(7/8 + 1/8)(1/2)\right] = 4.52 \text{ in.}^2$$

L1.49

Example 2

- Shear lag factor
  - Minimum *U* is the only applicable criteria

$$U = \frac{3(0.5)}{2.76} = 0.543$$

L1.50

Example 2

- Effective net area

$$A_e = A_n U$$
$$= 4.52(0.543) = 2.45 \text{ in.}^2$$
$$\frac{A_e}{A_g} = \frac{2.45}{5.52} = 0.444$$

L1.51

Example 2

- Yielding (Same as Example 1)

$$P_n = F_y A_g \qquad \text{(D2-1)}$$
$$= (36)(5.52) = 199 \text{ kips}$$

- Rupture

$$P_n = F_u A_e \qquad \text{(D2-2)}$$
$$= (58)(2.45) = 142 \text{ kips}$$

L1.52

Example 2

- ASD
  - Yielding

$$\frac{P_n}{\Omega} = \frac{199}{1.67} = 119 \text{ kips}$$

  - Rupture

$$\frac{P_n}{\Omega} = \frac{142}{2.00} = 71.0 \text{ kips}$$ ☆

Was 118 kips in Example 1.
Single bolt shear = 32.5 kips

L1.53

Example 2

- LRFD
  - Yielding

$$\phi_t P_n = 0.90(199) = 179 \text{ kips}$$

  - Rupture

$$\phi_t P_n = 0.75(142) = 107 \text{ kips}$$ ☆

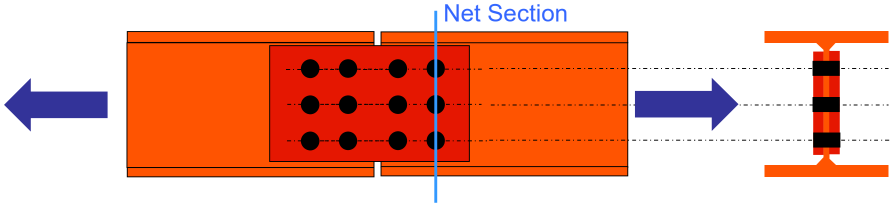
The same limit state controls for ASD and LRFD

Was 177 kips in Example 1.
Single bolt shear = 48.7 kips

L1.54

Example 3

- Determine the available strength of the W10x19, A992 tension member spliced at the web as shown with ¾ in. A325N bolts.

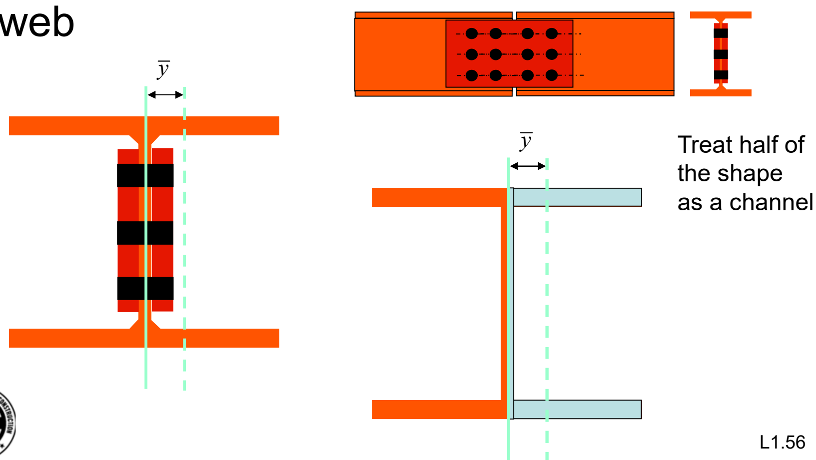


At the net section we are going to deduct for three bolt holes to get the net area. To get the effective net area we must account for the fact that the flanges are not connected

L1.55

Example 3

- Shear Lag Factor for a W attached only at web

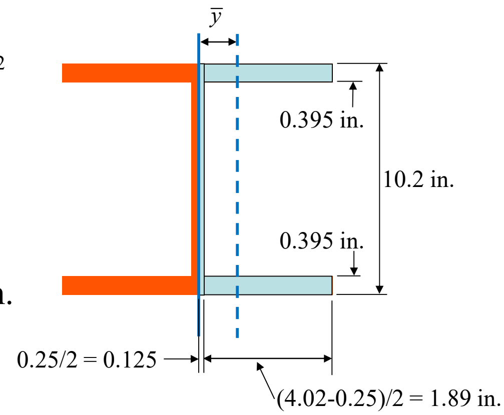


L1.56

Example 3

- W10x19

$$A_g = 5.62 \text{ in.}^2$$
$$d = 10.2 \text{ in.}$$
$$t_w = 0.25 \text{ in.}$$
$$b_f = 4.02 \text{ in.}$$
$$t_f = 0.395 \text{ in.}$$

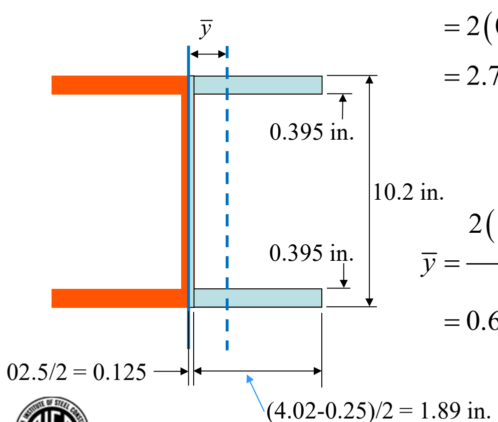


L1.57

Example 3

- W10x19

$$A_{\text{Channel}} = 2\left(1.89(0.395)\right) + 10.2(0.125)$$
$$= 2(0.747) + 1.28$$
$$= 2.77 \text{ in.}^2$$



$$\bar{y} = \frac{2(0.747)\left(0.125 + \frac{1.89}{2}\right) + 1.28\left(\frac{0.125}{2}\right)}{2.77}$$
$$= 0.606 \text{ in.}$$

L1.58

Example 3

- W10x19
  - Gross area given $A_g = 5.62 \text{ in.}^2$
  - Net area

$$A_n = 5.62 - 3\left(\tfrac{3}{4} + \tfrac{1}{16} + \tfrac{1}{16}\right)(0.25) = 4.96 \text{ in.}^2$$

  - Effective net area $U = 1 - \frac{\bar{x}}{l} = 1 - \frac{0.606}{3.0} = 0.80$

$$A_e = UA_n = 0.8(4.96) = 3.97 \text{ in.}^2$$

L1.59

Example 3

- Yielding

$$P_n = F_y A_g \quad \text{(D2-1)}$$

$$= (50)(5.62) = 281 \text{ kips}$$

- Rupture

$$P_n = F_u A_e \quad \text{(D2-2)}$$

$$= (65)(3.97) = 258 \text{ kips}$$

L1.60

Example 3

- ASD
  - Yielding

$$\frac{P_n}{\Omega} = \frac{281}{1.67} = 168 \text{ kips}$$

  - Rupture

$$\frac{P_n}{\Omega} = \frac{258}{2.00} = 129 \text{ kips}$$

Six bolts = 143 kips

L1.61

Example 3

- LRFD
  - Yielding

$$\phi_t P_n = 0.90(281) = 253 \text{ kips}$$

  - Rupture

$$\phi_t P_n = 0.75(258) = 194 \text{ kips}$$

The same limit state controls for ASD and LRFD

Six bolts = 215 kips

L1.62

Example 3

Note that
$A_e = 0.75A_g$

$$\frac{A_e}{A_g} = \frac{3.97}{5.62} = 0.706$$

Rupture

For ASD

$$\left(\frac{0.706}{0.75}\right)(137) = 129 \text{ kips}$$

For LRFD

$$\left(\frac{0.706}{0.75}\right)(206) = 194 \text{ kips}$$

F_y = 50 ksi
F_u = 65 ksi

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

W10–W8

Shape	Gross Area, A_g (in.²)	$A_e = 0.75A_g$ (in.²)	Yielding (kips) P_n/Ω_t ASD	Yielding (kips) $\phi_t P_n$ LRFD	Rupture (kips) P_n/Ω_t ASD	Rupture (kips) $\phi_t P_n$ LRFD
W10×112	32.9	24.7	985	1480	803	1200
×100	29.3	22.0	877	1320	715	1070
×88	26.0	19.5	778	1170	634	951
×77	22.7	17.0	680	1020	553	829
×68	19.9	14.9	596	896	484	726
×60	17.7	13.3	530	797	432	648
×54	15.8	11.9	473	711	387	580
×49	14.4	10.8	431	648	351	527
W10×45	13.3	9.98	398	599	324	487
×39	11.5	8.63	344	518	280	421
×33	9.71	7.28	291	437	237	355
W10×30	8.84	6.63	265	398	215	323
×26	7.61	5.71	228	342	186	278
×22	6.49	4.87	194	292	158	237
W10×19	5.62	4.22	168	253	137	206
×17	4.99	3.74	149	225	122	182
×15	4.41	3.31	132	198	108	161
×12	3.54	2.66	106	159	86.5	130

L1.63

Manual Tables

- Note that when $A_e = 0.75A_g$, rupture controls for A992 and yield for A36.

F_y = 50 ksi
F_u = 65 ksi

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

W10–W8

Shape	Gross Area, A_g (in.²)	$A_e = 0.75A_g$ (in.²)	Yielding (kips) P_n/Ω_t ASD	Yielding (kips) $\phi_t P_n$ LRFD	Rupture (kips) P_n/Ω_t ASD	Rupture (kips) $\phi_t P_n$ LRFD
W10×112	32.9	24.7	985	1480	803	1200
×100	29.3	22.0	877	1320	715	1070
×88	26.0	19.5	778	1170	634	951
×77	22.7	17.0	680	1020	553	829
×68	19.9	14.9	596	896	484	726
×60	17.7	13.3	530	797	432	648
×54	15.8	11.9	473	711	387	580
×49	14.4	10.8	431	648	351	527
W10×45	13.3	9.98	398	599	324	487
×39	11.5	8.63	344	518	280	421

F_y = 36 ksi
F_u = 58 ksi

Table 5-2 (continued)
Available Strength in Axial Tension
Angles

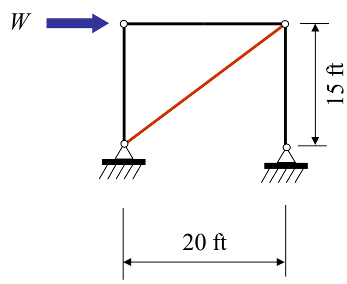
L3½–L2½

Shape	Gross Area, A_g (in.²)	$A_e = 0.75A_g$ (in.²)	Yielding (kips) P_n/Ω_t ASD	Yielding (kips) $\phi_t P_n$ LRFD	Rupture (kips) P_n/Ω_t ASD	Rupture (kips) $\phi_t P_n$ LRFD
L3½×3×½	3.02	2.27	65.1	97.8	65.8	98.7
×7/16	2.67	2.00	57.6	86.5	58.0	87.0
×3/8	2.32	1.74	50.0	75.2	50.5	75.7
×5/16	1.95	1.46	42.0	63.2	42.3	63.5
×1/4	1.58	1.19	34.1	51.2	34.5	51.8
L3½×2½×½	2.77	2.08	59.7	89.7	60.3	90.5
×3/8	2.12	1.59	45.7	68.7	46.1	69.2
×5/16	1.79	1.34	38.6	58.0	38.9	58.3
×1/4	1.45	1.09	31.3	47.0	31.6	47.4
L3×3×½	2.76	2.07	59.5	89.4	60.0	90.0
×7/16	2.43	1.82	52.4	78.7	52.8	79.2

L1.64

Example 4

- Design a diagonal tension brace as shown.



Use a single angle A36 member
Assume ¾ in. A325N bolts

From a first-order determinant analysis, brace force $P = W(25/20)$

Code specified lateral load = 50 kips

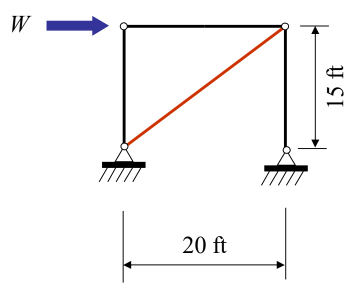
For wind load only,

LRFD: W =1.0(50) = 50 kips
ASD: W = 0.6(50) = 30 kips

L1.65

Example 4 (LRFD)

- Design a diagonal tension brace as shown.



For LRFD, W = 50 kips
From a first-order analysis

$$P_r = 50\left(\frac{25}{20}\right) = 62.5 \text{ kips}$$

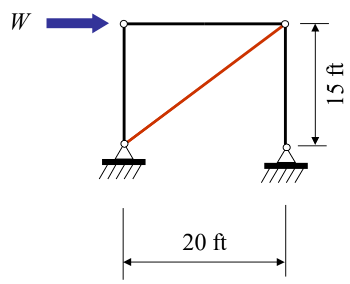
Start with the assumption that yielding controls and determine the required area

$$A_g = \frac{P_r}{\phi F_y} = \frac{62.5}{0.9(36)} = 1.93 \text{ in.}^2$$

L1.66

Example 4 (LRFD)

- Design a diagonal tension brace as shown.

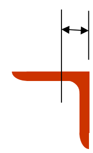


Try a 3x3x3/8 angle, A_g = 2.11 in.^{2}

Since this area is greater than required, we know the limit state of yielding has sufficient strength.

Now consider the limit state of rupture

$\bar{x} = 0.884$ in.



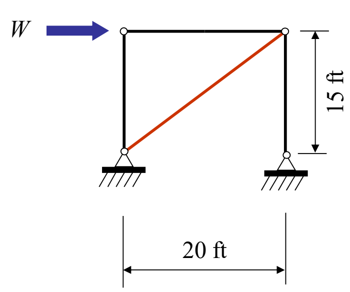
4 – ¾ in. A325N bolts will carry 71.6 kips. For a typical spacing of 3 in.,

$$U = 1 - \frac{\bar{x}}{l} = 1 - \frac{0.884}{9.0} = 0.902$$

L1.67

Example 4 (LRFD)

- Design a diagonal tension brace as shown.



Net area

$$A_n = 2.11 - 1\left(\tfrac{3}{4} + \tfrac{1}{16} + \tfrac{1}{16}\right)\left(\tfrac{3}{8}\right) = 1.78 \text{ in.}^2$$

Effective net area

$$A_e = UA_n = 0.902(1.78) = 1.61 \text{ in.}^2$$

Nominal rupture strength

$$P_n = F_u A_e = 58(1.61) = 93.4 \text{ kip}$$

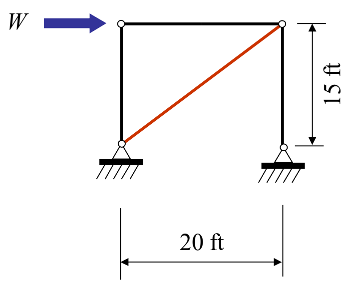
Available rupture strength

$$\phi P_n = 0.75(93.4) = 70.1 \text{ kip}$$

L1.68

Example 4 (LRFD)

- Design a diagonal tension brace as shown.



Available yield strength for LRFD

$$\phi P_n = 0.9(36)(2.11) = 68.4 > 62.5 \text{ kips}$$

Available rupture strength

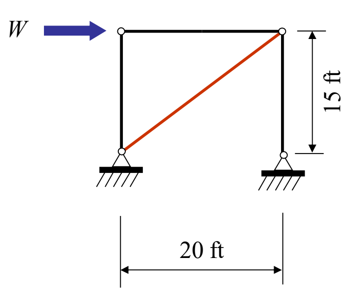
$$\phi P_n = 0.75(93.4) = 70.1 > 62.5 \text{ kips}$$

A 3x3x3/8 A36 angle will work. If more than 4 bolts are required, the shear lag factor will increase so this solution is conservative. If fewer than 4 bolts are needed, the rupture limit state must be rechecked.

L1.69

Example 4 (ASD)

- Design a diagonal tension brace as shown.



$$P_r = 30\left(\frac{25}{20}\right) = 37.5 \text{ kips}$$

Available yield strength for ASD

$$\frac{P_n}{\Omega} = \frac{(36)(2.11)}{1.67} = 45.5 > 37.5 \text{ kips}$$

Available rupture strength

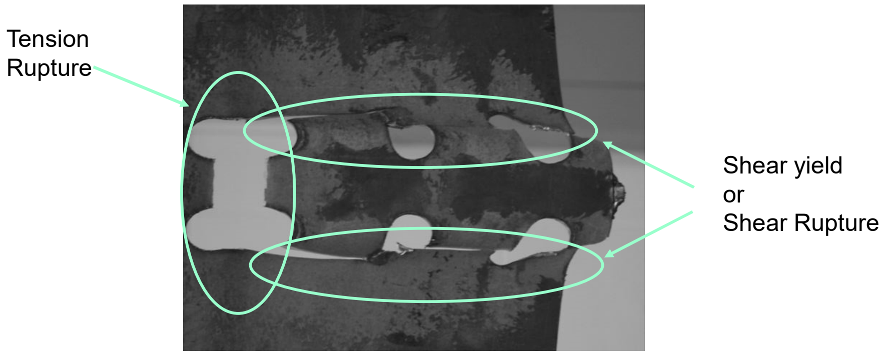
$$\frac{P_n}{\Omega} = \frac{93.4}{2.00} = 46.7 > 37.5 \text{ kips}$$

A 3x3x3/8 A36 angle will work. If more than 4 bolts are required, the shear lag factor will increase so this solution is conservative. If fewer than 4 bolts are needed, the rupture limit state must be rechecked.

L1.70

Block Shear

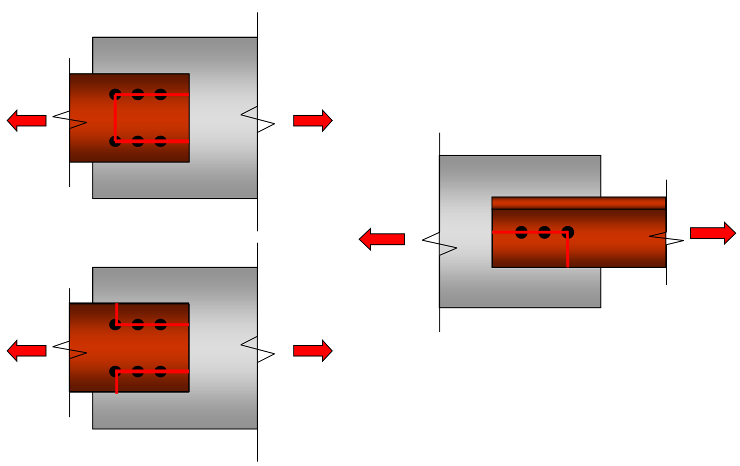
J4.3. Block Shear Strength (Another connection issue)



This is actually a connection issue.

L1.71

Block Shear



L1.72

Block Shear

- J4.3. Tear out strength based on a combination of limit states

Shear Yield +Tension Rupture

or

Shear Rupture +Tension Rupture

L1.73

Block Shear

- Compute Strength for Tensile Rupture Limit State

$$F_u A_{nt}$$

- Compute Strength for Shear Rupture and Shear Yield Limit States

$$0.6F_u A_{nv} \qquad 0.6F_y A_{gv}$$

L1.74

Block Shear

- Always use tension rupture term

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

- Use shear rupture unless shear yield is less, then

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

$\phi = 0.75$ (LRFD) $\Omega = 2.00$ (ASD)

L1.75

Block Shear Reduction Factor

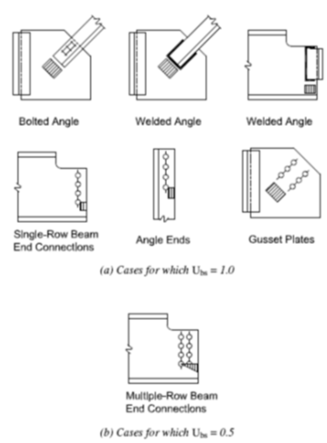
$$R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt} \quad \text{(J4-5)}$$

- For uniform tensile stress

$$U_{bs} = 1.0$$

- For nonuniform tensile stress

$$U_{bs} = 0.5$$



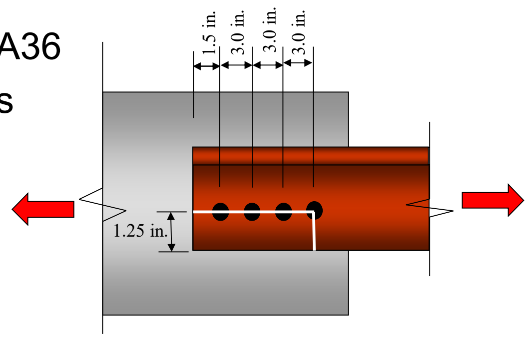
L1.76

Example 5

Determine the available block shear strength

From Example 1

- 2 - L3 *x* 3 *x* 1/2 A36
- 4 - 7/8 inch bolts



L1.77

Example 5

For one angle, shear areas

$$A_{gv} = 10.5\left(\frac{1}{2}\right) = 5.25 \text{ in.}^2$$

$$A_{nv} = \left(10.5 - 3.5\left(\frac{7}{8} + \frac{1}{16} + \frac{1}{16}\right)\right)\left(\frac{1}{2}\right) = 3.50 \text{ in.}^2$$

For one angle, tension area

$$A_{nt} = \left(1.25 - \frac{1}{2}\left(\frac{7}{8} + \frac{1}{16} + \frac{1}{16}\right)\right)\left(\frac{1}{2}\right) = 0.375 \text{ in.}^2$$

L1.78

Example 5

Shear rupture

$$0.6F_uA_{nv} = 0.6(58)(3.50) = 122 \text{ kips}$$

Shear Yield

$$0.6F_yA_{gv} = 0.6(36)(5.25) = 113 \text{ kips}$$

Tension rupture

$$F_uA_{nt} = 58(0.375) = 21.8 \text{ kips}$$

L1.79

Example 5

Nominal Block Shear Strength for one angle with U_{bs} = 1.0

$$R_n = 0.6F_yA_{gv} + U_{bs}F_uA_{nt} \qquad \text{(J4-5)}$$

$$R_n = (113 + 1.0(21.8)) = 135 \text{ kips}$$

L1.80

Example 5

Allowable block shear strength for both angles (ASD)

$$\frac{R_n}{\Omega} = 2\left(\frac{135}{2.0}\right) = 135 \text{ kips}$$

L1.81

Example 5

- Summary ASD
  - Tension on angles from Example 1

$$\frac{P_n}{\Omega} = 118 \text{ kips}$$

  - Block shear

$$\frac{R_n}{\Omega} = 135 \text{ kips}$$

Thus, block shear does not control

Bolt limit states = 130 kips

L1.82

Example 5

Design block shear strength for both angles (LRFD)

$$\phi R_n = 2\left(0.75\left(135\right)\right) = 203 \text{ kips}$$

L1.83

Example 5

- Summary LRFD
  - Tension on angles from Example 1

$$\phi P_n = 177 \text{ kips}$$ ☆

  - Block shear

$$\phi R_n = 203 \text{ kips}$$

Again, the same limit state controls

Bolt limit states = 195 kips

L1.84

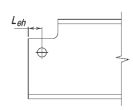
Example 5

- The Manual includes tables, in Part 9, that may be used to check block shear.
- These tables appear to be for coped beams but may also be used for other block shear situations

Example 5

Table 9-3a
Block Shear Tension Rupture Component
per inch of thickness, kips/in.

$U_{bs} = 1.0$

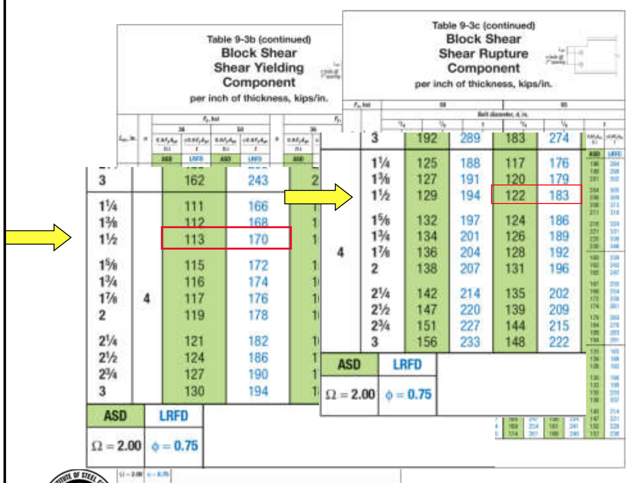


F_u	58 ksi					
	Bolt diameter, *d*, in.					
	3/4		7/8		1	
L_{eh}, in.	$\frac{F_u A_{nt}}{t\Omega}$	$\frac{\phi F_u A_{nt}}{t}$	$\frac{F_u A_{nt}}{t\Omega}$	$\frac{\phi F_u A_{nt}}{t}$	$\frac{F_u A_{nt}}{t\Omega}$	$\frac{\phi F_u A_{nt}}{t}$
	ASD	LRFD	ASD	LRFD	ASD	LRFD
1	16.3	24.5	14.5	21.8	12.7	19.0
1⅛	19.9	29.9	18.1	27.2	16.3	24.5
1¼	23.6	35.3	21.8	32.6	19.9	29.9
1⅜	27.2	40.8	25.4	38.1	23.6	35.3
1½	30.8	46.2	29.0	43.5	27.2	40.8
1⅝	34.4	51.7	32.6	48.9	30.8	46.2
1¾	38.1	57.1	36.3	54.4	34.4	51.7
1⅞	41.7	62.5	39.9	59.8	38.1	57.1
2	45.3	68.0	43.5	65.3	41.7	62.5

- Using Tables
  Tension rupture

  ASD 21.8 kips/in.
  LRFD 32.6 kips/in

Example 5



		ASD	LRFD
3		162	243
1¼		111	166
1⅜		112	168
1½		113	170
1⅝		115	172
1¾		116	174
1⅞	4	117	176
2		119	178
2¼		121	182
2½		124	186
2¾		127	190
3		130	194
		ASD: $\Omega = 2.00$	LRFD: $\phi = 0.75$

	3	192	289	183	274
	1¼	125	188	117	176
	1⅜	127	191	120	179
	1½	129	194	122	183
	1⅝	132	197	124	186
	1¾	134	201	126	189
4	1⅞	136	204	128	192
	2	138	207	131	196
	2¼	142	214	135	202
	2½	147	220	139	209
	2¾	151	227	144	215
	3	156	233	148	222
	ASD: $\Omega = 2.00$	LRFD: $\phi = 0.75$			

- Using Tables

Select the smaller value from shear yielding or rupture.

ASD 113 kips/in.

LRFD 170 kips/in.

L1.87

Example 5

- Block shear strength (ASD)

$$\frac{R_n}{\Omega} = (21.8 + 113)(0.5) = 67.4 \text{ kips for one angle.}$$

for both angles,

$$\frac{R_n}{\Omega} = 2(67.4) = 135 \text{ kips}$$

This is the same as what was previously determined

L1.88

Example 5

- Block shear strength (LRFD)

$$\phi R_n = (32.6 + 170)(0.5) = 101 \text{ kips for one angle}$$

for both angles,

$$\phi R_n = 2(101) = 202 \text{ kips}$$

This is essentially the same as what was previously determined

L1.89

Eyebars

D6 Eyebars

D6.1. Tensile Strength

- Yielding on the body of eyebar

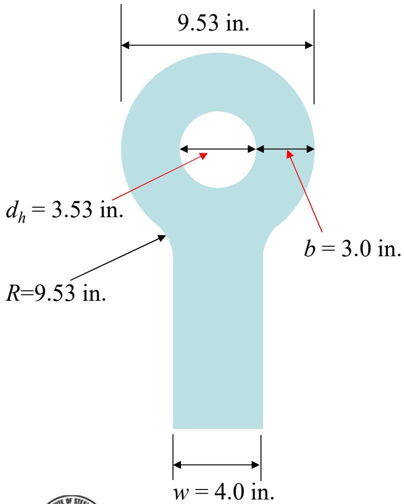
$$P_n = F_y A_g \qquad \text{(D2-1)}$$

D6.2. Dimensional Requirements

- Establish proportions so that yielding of the body is the only limit state to be checked.
- Requirements (a) through (f).

L1.90

Eyebars



Determine the available strength of a ½ in. by 4.0 in. eyebar with a 3.5 in. pin. First check the dimensional requirements of Section D6.2.

$$t \geq \tfrac{1}{2} \text{ in.} = 0.5 \text{ in.}$$

$$w \leq 8t = 8(0.5) = 4.0 \text{ in.}$$

$$d \geq \tfrac{7}{8} w = \tfrac{7}{8}(4.0) = 3.5 \text{ in.}$$

$$d_h \leq d + \tfrac{1}{32} \text{ in.} = 3.5 + \tfrac{1}{32} = 3.53 \text{ in.}$$

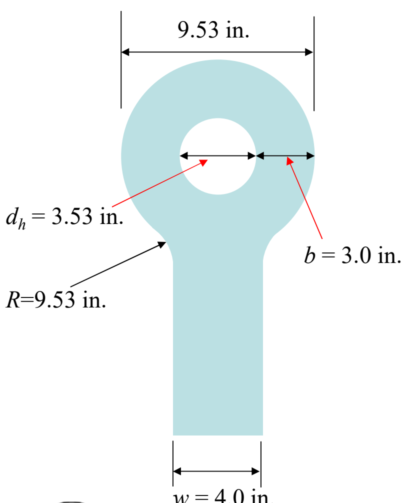
$$R \geq 2b + d_h = 2(3.0) + 3.53 = 9.53 \text{ in.}$$

$$\tfrac{2}{3} w \leq b \leq \tfrac{3}{4} w$$

$$\tfrac{2}{3}(4.0) \leq 3.0 \leq \tfrac{3}{4}(4.0)$$

L1.91

Eyebars



Determine the available strength of a ½ in. by 4.0 in. eyebar with a 3.5 in. pin. First check the dimensional requirements of Section D6.2.

The dimensional criteria of Section D6.2 are satisfied.

$$P_n = 36\left(\tfrac{1}{2}\right)(4.0) = 72.0 \text{ kips}$$

For LRFD

$$\phi P_n = 0.9(72.0) = 64.8 \text{ kips}$$

For ASD

$$\frac{P_n}{\Omega} = \frac{72.0}{1.67} = 43.1 \text{ kips}$$

L1.92

Pin Connected Members

D5 Pin Connected Members

D5.1. Tensile Strength

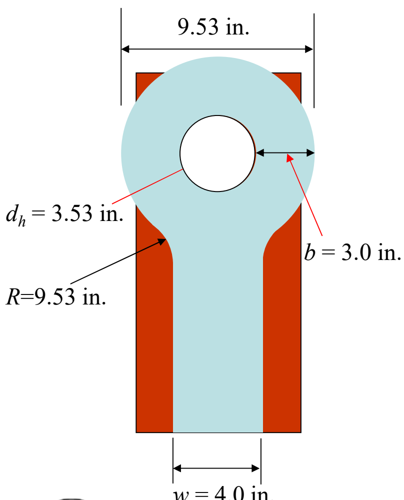
- Tensile rupture on effective net area
- Shear rupture on effective area
- Yielding on the gross section
- Bearing on the projected area of pin

D5.2. Dimensional Requirements

- Requirements (a) through (d).

L1.93

Pin Connected Members



Compare a pin connected member that uses the same size pin as the eyebar just considered. Assume a ½ in. thick plate.

Check the dimensional requirements in Section D5.

$$d = 3.5 \text{ in.}$$

$$t = 0.5 \text{ in.}$$

$$d_h \leq d + \tfrac{1}{32} = 3.53 \text{ in.}$$

$$b_e = 2t + 0.63 \text{ in.} = 2(0.5) + 0.63$$

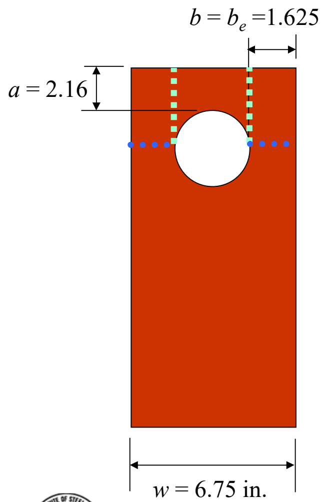
$$= 1.63 \text{ in.} \leq b$$

$$w \geq 2b_e + d = 2(1.625) + 3.5 = 6.75 \text{ in.}$$

$$a \geq 1.33b_e = 1.33(1.625) = 2.16 \text{ in.}$$

L1.94

Pin Connected Members



Compare a pin connected member that uses the same size pin as the eyebar just considered. Assume a ½ in. thick plate.

The dimensional requirements are satisfied.

Tensile rupture limit state (Eq. D5-1)

$$P_n = F_u(2tb_e) = 58(2(0.5)(1.63)) = 94.5 \text{ kips}$$

$$\phi P_n = 0.75(94.5) = 70.9 \text{ kips}$$

Shear rupture limit state (Eq. D5-2)

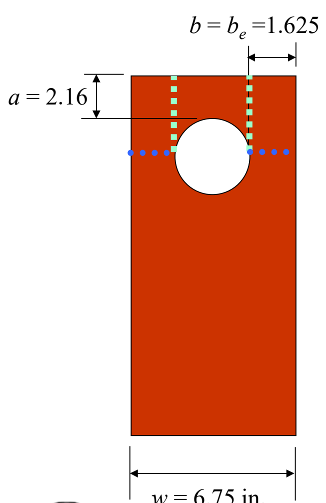
$$P_n = 0.6F_u\left(2t\left(a + d/2\right)\right)$$

$$= 0.6(58)\left(2(0.5)\left(2.16 + \frac{3.5}{2}\right)\right) = 136 \text{ kips}$$

$$\phi P_n = 0.75(136) = 102 \text{ kips}$$

L1.95

Pin Connected Members



Compare a pin connected member that uses the same size pin as the eyebar just considered. Assume a ½ in. thick plate.

The dimensional requirements are satisfied.

Bearing on projected area of pin (Eq. J7-1)

$$R_n = 1.8F_y A_{pb} = 1.8(36)(0.5)(3.5) = 113 \text{ kips}$$

$$\phi R_n = 0.75(113) = 85.1 \text{ kips}$$

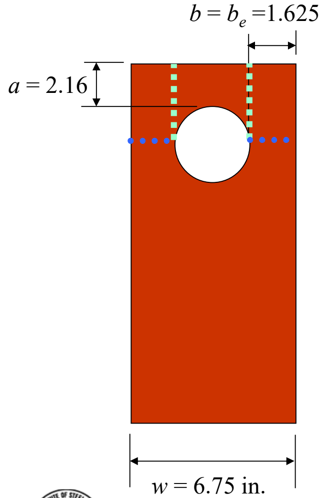
Yield on the gross section (Eq. D2-1)

$$R_n = F_y A_g = (36)(0.5)(6.75) = 122 \text{ kips}$$

$$\phi R_n = 0.9(122) = 110 \text{ kips}$$

L1.96

Pin Connected Members



Compare a pin connected member that uses the same size pin as the eyebar just considered. Assume a ½ in. thick plate.

The dimensional requirements are satisfied.

Tensile rupture is the controlling limit state.

For LRFD

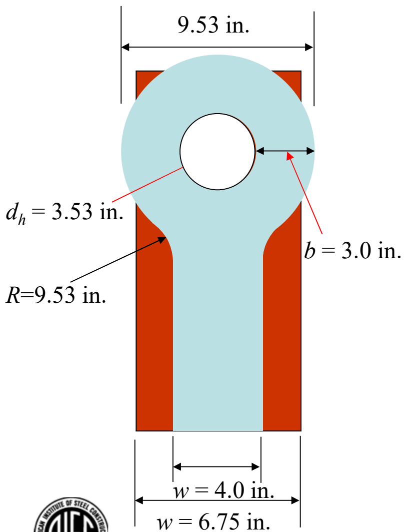
$$\phi R_n = 0.75(94.5) = 70.9 \text{ kips}$$

For ASD

$$\frac{R_n}{\Omega} = \frac{94.5}{2.00} = 47.3 \text{ kips}$$

L1.97

Pin Connected Members



Compare the strengths of the designed eyebar and pin connected member

Eyebar (Yield)

$$\phi P_n = 0.9(72.0) = 64.8 \text{ kips}$$

$$\frac{P_n}{\Omega} = \frac{72.0}{1.67} = 43.1 \text{ kips}$$

Pin Connected Member (Tensile rupture)

$$\phi R_n = 0.75(94.5) = 70.9 \text{ kips}$$

$$\frac{R_n}{\Omega} = \frac{94.5}{2.00} = 47.3 \text{ kips}$$

L1.98

Summary

- Looked at the limit states for tension members
- Addressed the required area calculations
- Compared hand calculations to the use of Manual tables
- Considered block shear limit state
- Designed an eyebar and a pin connected member

L1.99

Lesson L2

- The next lesson will look at the principles of design for compression members.
- We will look primarily at the material in Chapter E of the Specification
- We will also look at Part 4 of the Manual

L1.100

Thank You

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L1.101

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  - Option 2: Watch the recording and pass the associated quiz.

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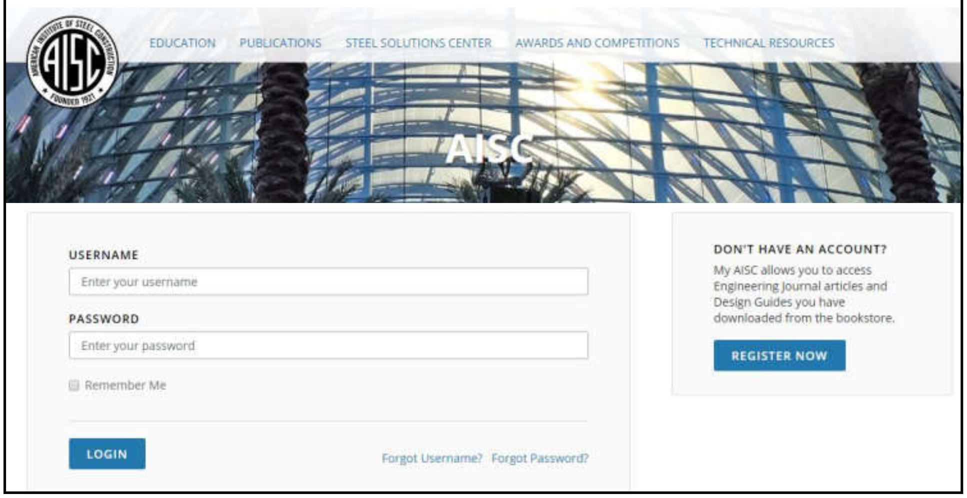

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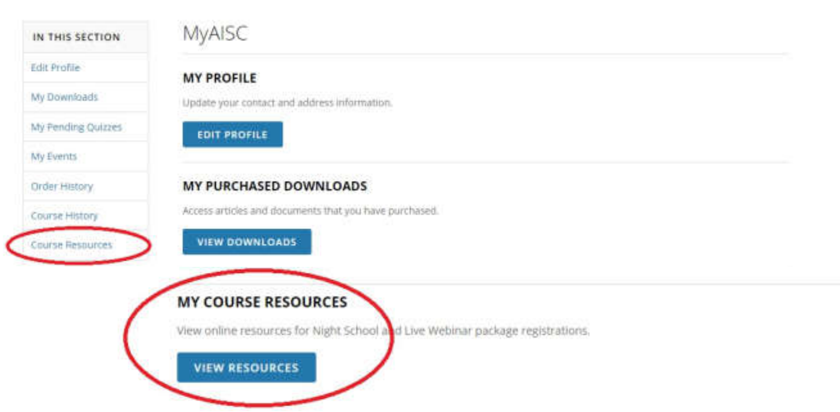

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Course Resources

Event	Start Date
Seismic Design in Steel	1/1/1900 12:00:00 AM
4-Session Package-Design of Facade Attachments	5/9/2019 1:30:00 PM
NS 15 8-Session Package-Night School 15 - Fundamentals of Connection Design	10/3/2017 7:00:00 PM
NS 16 8-Session Package-Night School 16 - Seismic Design in Steel	2/5/2018 7:00:00 PM
NS 17 4-Session Package-Night School 17: Design of Façade Attachments	7/16/2018 7:00:00 PM
NS 18 8-Session Package-Night School 18: Steel Construction: Mill To Topping Out	10/15/2018 7:00:00 PM
NS 19 8-Session Package-Night School 19: Connection Design	2/4/2019 7:00:00 PM
NS 20 8-Session Package-Night School 20: Classical Methods of Structural Analysis	6/3/2019 7:00:00 PM
8-Session Package-Seismic Design in Steel - Concepts & Examples	7/16/2018 1:30:00 PM

Course Package Registrants

Course Resources

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Design of Facade Attachments

4-SESSION PACKAGE RESOURCES

Event	Date	Handouts	Video	Quiz	Attendance
R1: Facade Fundamentals	N/A	Handouts	View Passcode: AZNS175	Pass Score: 100	N/A
L1: Façade Attachments Part 1	May 9 2019 1:30PM EDT	Handouts	Available 05/11/2019 5:00PM EDT	Available 05/11/2019 5:00 PM EDT	Pending
L2: Façade Attachments Part 2	May 16 2019 1:30PM EDT	Handouts	Available 05/18/2019 5:00PM EDT	Available 05/18/2019 5:00 PM EDT	Pending
L3: Façade Attachments - Building Lateral Drifts	May 23 2019 1:30PM EDT	Handouts	Available 05/25/2019 5:00PM EDT	Available 05/25/2019 5:00 PM EDT	Pending
Final Exam	N/A			Available 5/27/2019 5:00 PM EDT	


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