Basic Steel Design

Louis F. Geschwindner

Smarter.
Stronger.
Steel.

AISC Night School

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Session Description

22.2 Tension Members
February 4, 2020

This lecture will focus on the design of structural steel tension members and the associated provisions in the AISC Specifications. The session will begin with a discussion of strength, calcuating 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.

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

Night School 22
Lesson 2
Tension Members

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Lesson 2 – Tension Members

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

2.9

Tension Members

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

Required Strength ≤ 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)

2.10

Tension Members

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

Required Strength ≤ 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 = $\dfrac{\text{nominal strength}}{\text{safety factor}}$

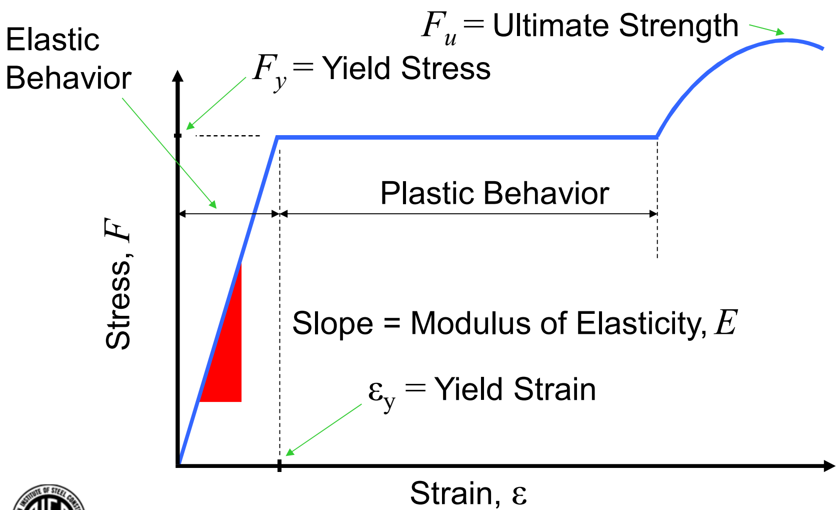
2.11

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."

2.12

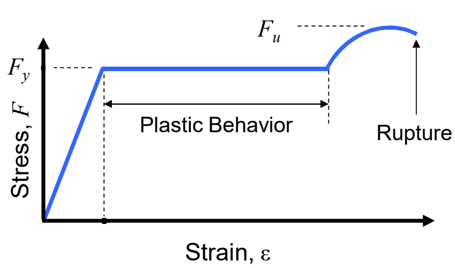
Steel as a Material



2.13

Tension Members

- Tensile Test Specimen



2.14

Tension Members

Two limit states to be considered for tension

D2.(a) Nominal Strength for Yielding;

$$P_n = F_y A_g$$ (D2-1)

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

2.15

Tension Members

Two limit states to be considered for tension

D2.(b) Nominal Strength for Rupture

$$P_n = F_u A_e$$ (D2-2)

$\phi_t = 0.75$ (LRFD) $\Omega_t = 2.00$ (ASD)

2.16

Design for Tension

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

$$P_a \le 0.6 F_y A_g$$
$$\le 0.5 F_u A_e$$

2.17

Tension Members

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

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

2.18

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

2.19

B4.3a. Gross Area, A_g

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

$A_g = A$

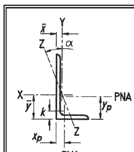


Table 1-7
Angles
Properties

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

2.20

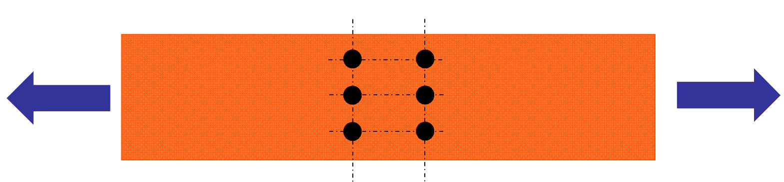
B4.3b. Net Area, A_n

- 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
  - 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

2.21

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



2.22

B4.3b. Net Area, A_n

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

- For members without holes, $A_n = A_g$

2.23

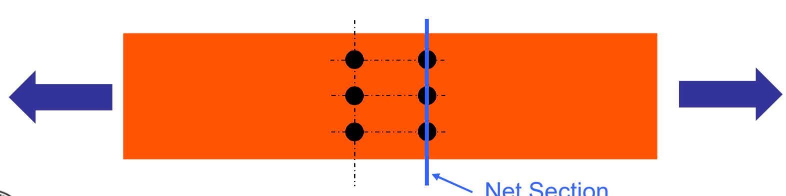
B4.3b. Net Area, A_n

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

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

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

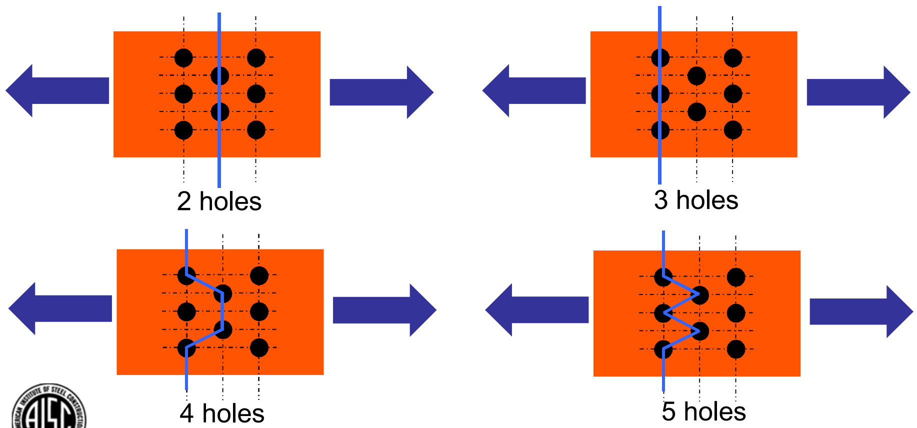
$$A_n = 4.0 - 3\left(3/4 + 1/16 + 1/16\right)(0.5) = 2.69 \text{ in.}^2$$



2.24

B4.3b. Net Area, A_n

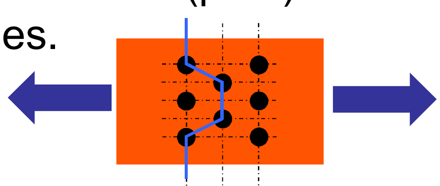
- What happens if the bolts are staggered?



2.25

B4.3b. Net Area, A_n

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



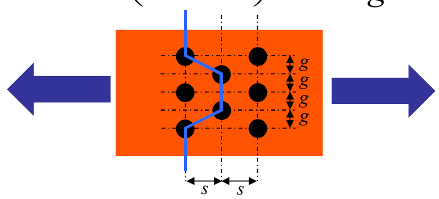
- There is more area to resist the force along the diagonals than if the holes were all in a straight line. So, after we deduct the 4 holes we must add something back.

2.26

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

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

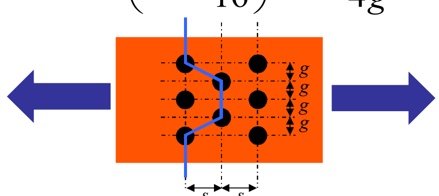


2.27

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.

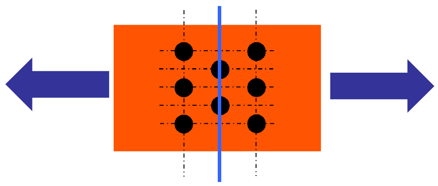
$$b_n = b - n\left(d_h + \frac{1}{16}\right) + \sum \frac{s^2}{4g}$$



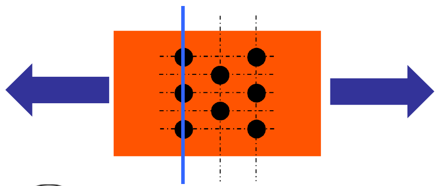
2.28

B4.3b. Net Area, A_n

Look at the 4 possible chains



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

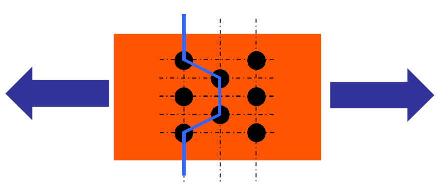


$$b_{n3} = 11.0 - 3\left(\tfrac{5}{8} + \tfrac{1}{16} + \tfrac{1}{16}\right) = 11.0 - 2.25 = 8.75 \text{ in.}$$

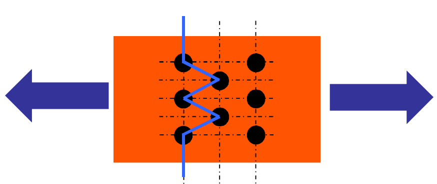
2.29

B4.3b. Net Area, A_n

Look at the 4 possible chains



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



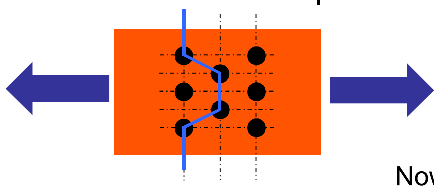
$$b_{n5} = 11.0 - 5\left(\tfrac{5}{8} + \tfrac{1}{16} + \tfrac{1}{16}\right) + 4\left(\frac{4^2}{4(2)}\right) = 11.0 - 3.75 + 8.0 = 15.3 \le 11.0$$

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

2.30

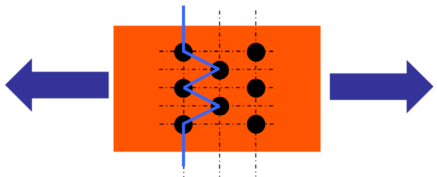
B4.3b. Net Area, A_n

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



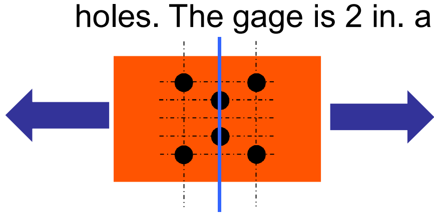
$$b_{n5} = 11.0 - 5\left(\tfrac{5}{8} + \tfrac{1}{16} + \tfrac{1}{16}\right) + 4\left(\frac{2^2}{4(2)}\right)$$
$$= 11.0 - 3.75 + 2.0 = 9.25$$

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

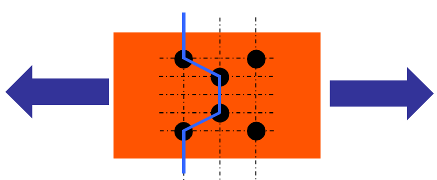
2.31

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.



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



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

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

2.32

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

2.33

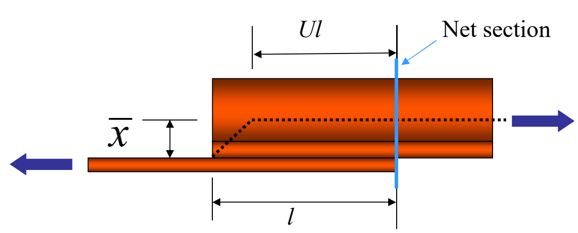
D3. Effective Net Area, A_e

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

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

l is defined here for a welded connection

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



2.34

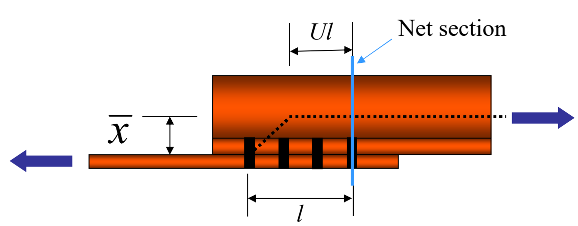
D3. Effective Net Area, A_e

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

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

l is defined here for a bolted connection

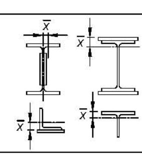
$$U = 1 - \frac{\bar{x}}{l}$$



2.35

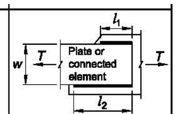
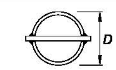
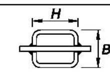
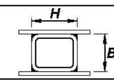
Table D3.1

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	–

2.36

Table D3.1 continued

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 $\bar{x}$.		$U = \dfrac{3l^2}{3l^2 + w^2}\left(1 - \dfrac{\bar{x}}{l}\right)$	
5	Round HSS with a single concentric gusset plate through slots in the HSS.		$l \geq 1.3D$, $U = 1.0$ $D \leq l < 1.3D$, $U = 1 - \dfrac{\bar{x}}{l}$ $\bar{x} = \dfrac{D}{\pi}$	
6	Rectangular HSS.	with a single concentric gusset plate	$l \geq H$, $U = 1 - \dfrac{\bar{x}}{l}$ $\bar{x} = \dfrac{B^2 + 2BH}{4(B+H)}$	
		with two side gusset plates	$l \geq H$, $U = 1 - \dfrac{\bar{x}}{l}$ $\bar{x} = \dfrac{B^2}{4(B+H)}$	

2.37

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

2.38

D3. Effective Net Area, A_e

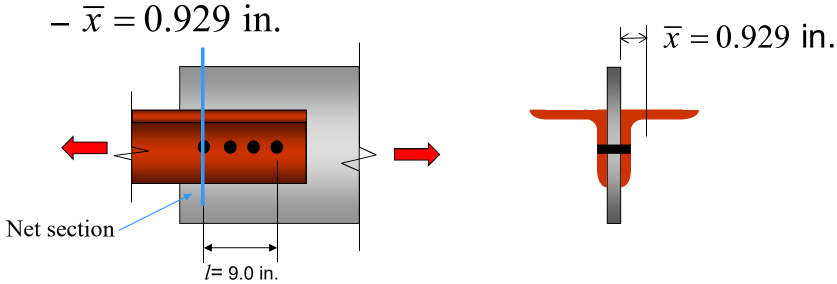
- Lower bound, for a single bolt or when sufficient information is not available to more accurately calculate *U.*

"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."

2.39

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.



2.40

Example 1

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

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

2.41

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

2.42

Example 1

- Effective net area

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

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

2.43

Example 1

- Yielding

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

$$= (36)(5.52) = 199 \text{ kips}$$

- Rupture

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

$$= (58)(4.07) = 236 \text{ kips}$$

2.44

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

2.45

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

2.46

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 P_n/Ω_t	Yielding kips $\phi_t P_n$	Rupture kips P_n/Ω_t	Rupture kips $\phi_t P_n$
	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

2.47

Example 1

Note that
$A_e = 0.75A_g$

Rupture

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

For ASD

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

For LRFD

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

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

F_y = 36 ksi
F_u = 58 ksi

L3½–L2½

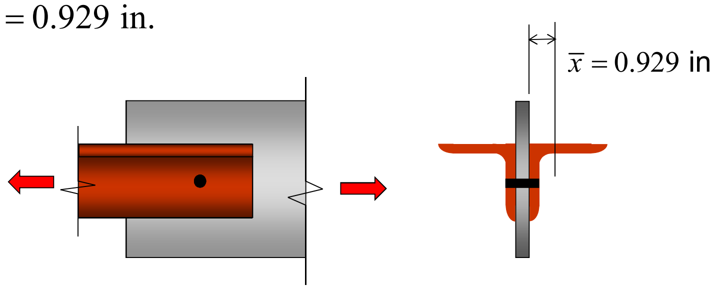
Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips P_n/Ω_t	Yielding kips $\phi_t P_n$	Rupture kips P_n/Ω_t	Rupture kips $\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
L3½×3×½	3.02	2.27	65.1	97.8	65.8	98.7
×7/16	2.67	2.00	57.6	86.5	58.0	87.0
×3/8	2.32	1.74	50.0	75.2	50.5	75.7
×5/16	1.95	1.46	42.0	63.2	42.3	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

2.48

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.



2.49

Example 2

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

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

2.50

Example 2

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

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

2.51

Example 2

- Effective net area

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

2.52

Example 2

- Yielding (Same as Example 1)

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

$$= (36)(5.52) = 199 \text{ kips}$$

- Rupture

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

$$= (58)(2.45) = 142 \text{ kips}$$

2.53

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

2.54

Example 2

- LRFD
  - Yielding

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

  - Rupture

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

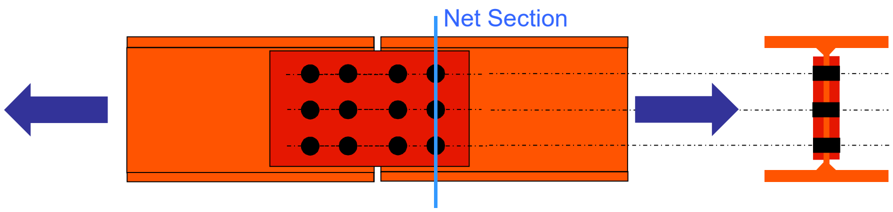
The same limit state controls for ASD and LRFD

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

2.55

Example 3

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

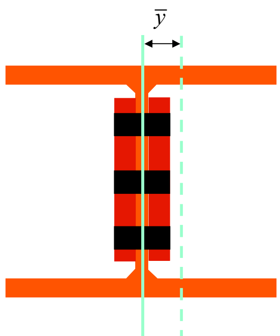


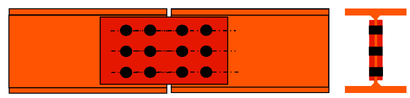
At the net section we are going to deduct for three bolt holes to get the net area. To get the effective net area we must account for the fact that the flanges are not connected

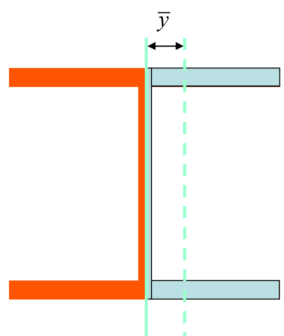
2.56

Example 3

- Shear Lag Factor for a W attached only at web







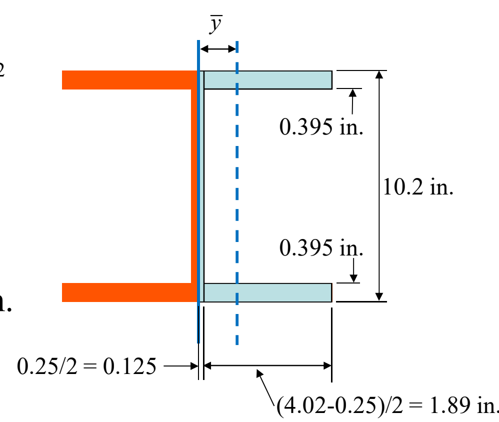
Treat half of the shape as a channel

2.57

Example 3

- W10x19

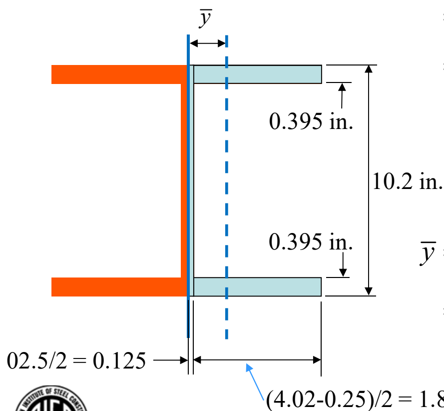
$$A_g = 5.62 \text{ in.}^2$$
$$d = 10.2 \text{ in.}$$
$$t_w = 0.25 \text{ in.}$$
$$b_f = 4.02 \text{ in.}$$
$$t_f = 0.395 \text{ in.}$$



2.58

Example 3

- W10x19



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

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

2.59

Example 3

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

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

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

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

2.60

Example 3

- Yielding

$$P_n = F_y A_g \quad \text{(D2-1)}$$
$$= (50)(5.62) = 281 \text{ kips}$$

- Rupture

$$P_n = F_u A_e \quad \text{(D2-2)}$$
$$= (65)(3.97) = 258 \text{ kips}$$

2.61

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

2.62

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

2.63

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	$A_e = 0.75A_g$	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
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

2.64

Manual Tables

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

Table 5-1 (continued)
Available Strength in Axial Tension
W-Shapes
F_y = 50 ksi
F_u = 65 ksi
W10–W8

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
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

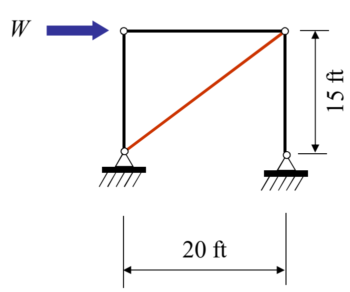
Table 5-2 (continued)
Available Strength in Axial Tension
Angles
F_y = 36 ksi
F_u = 58 ksi
L3½–L2½

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

2.65

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

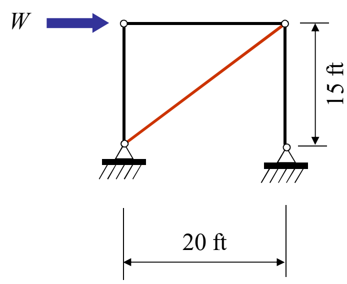
For wind load only,

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

2.66

Example 4 (LRFD)

- Design a diagonal tension brace as shown.



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

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

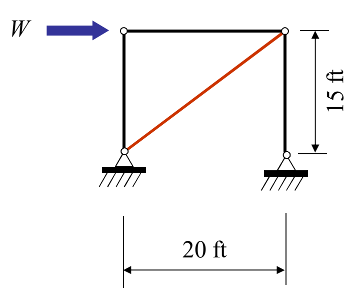
Start with the assumption that yielding controls and determine the required area

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

2.67

Example 4 (LRFD)

- Design a diagonal tension brace as shown.

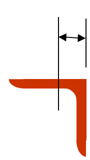


Try a 3x3x3/8 angle, A_g = 2.11 in.^{2}

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.



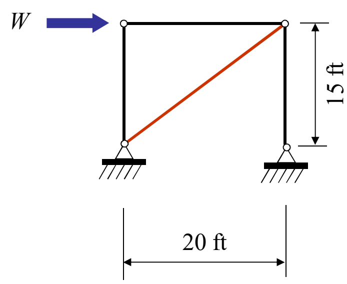
4 – ¾ in. A325N bolts will carry 71.6 kips. For a typical spacing of 3 in.,

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

2.68

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

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

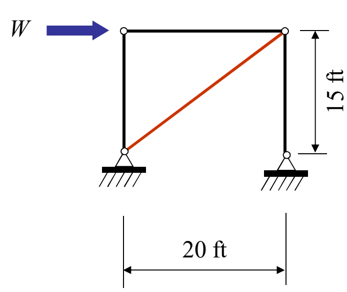
Available rupture strength

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

2.69

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

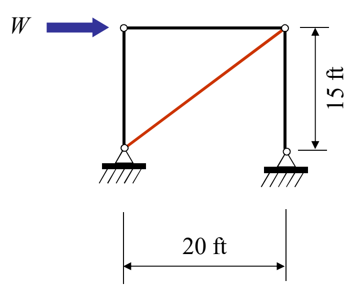
$$\phi P_n = 0.75(93.4) = 70.1 > 62.5 \text{ kips}$$

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.

2.70

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

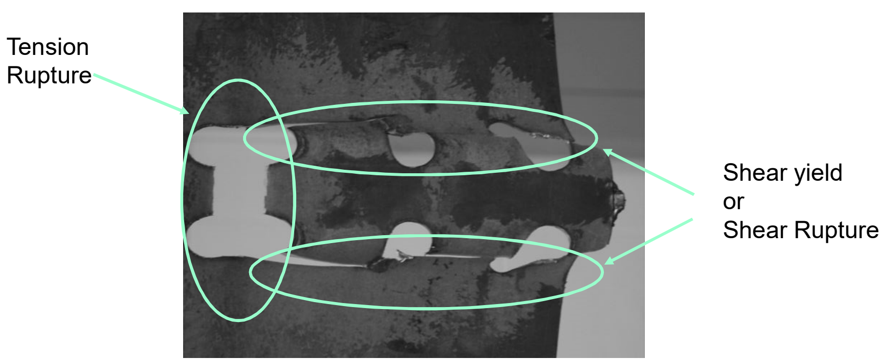
$$\frac{P_n}{\Omega} = \frac{93.4}{2.00} = 46.7 > 37.5 \text{ kips}$$

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.

2.71

Block Shear

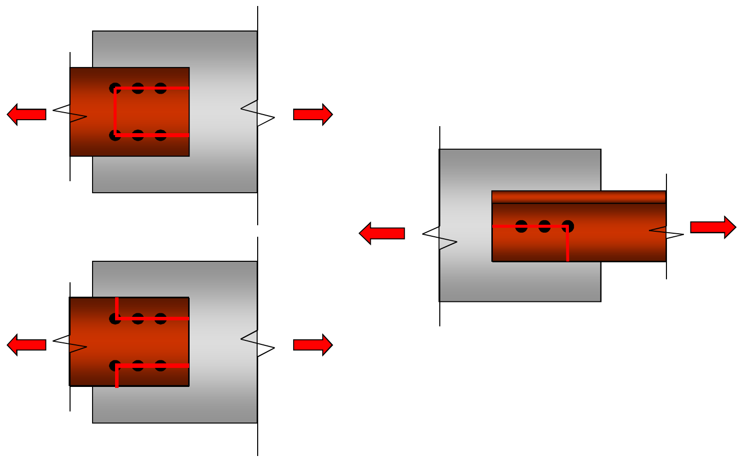
J4.3. Block Shear Strength (Another connection issue)



This is actually a connection issue.

2.72

Block Shear



2.73

Block Shear

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

Shear Yield +Tension Rupture

or

Shear Rupture +Tension Rupture

2.74

Block Shear

- Compute Strength for Tensile Rupture Limit State

$$F_u A_{nt}$$

- Compute Strength for Shear Rupture and Shear Yield Limit States

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

2.75

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)

2.76

Block Shear Reduction Factor

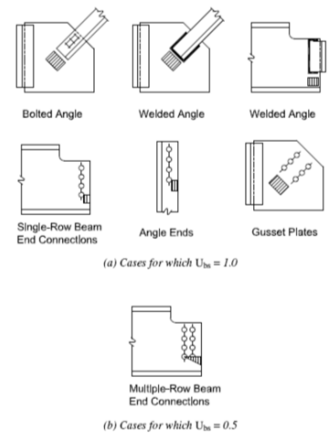
$$R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt} \quad \text{(J4-5)}$$

- For uniform tensile stress

$$U_{bs} = 1.0$$

- For nonuniform tensile stress

$$U_{bs} = 0.5$$



2.77

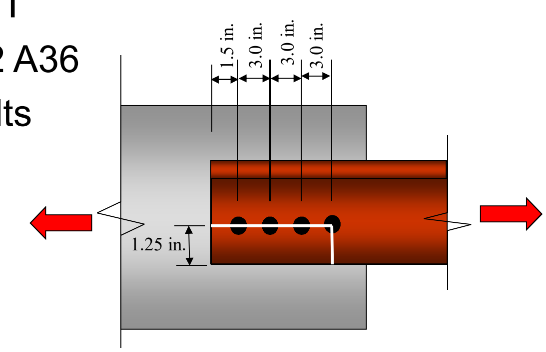
Example 5

Determine the available block shear strength

From Example 1

2 - L3 *x* 3 *x* 1/2 A36

4 - 7/8 inch bolts



2.78

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

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

2.79

Example 5

Shear rupture

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

Shear Yield

$$0.6F_y A_{gv} = 0.6(36)(5.25) = 113 \text{ kips}$$ ☆

Tension rupture

$$F_u A_{nt} = 58(0.375) = 21.8 \text{ kips}$$ ☆

2.80

Example 5

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

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

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

2.81

Example 5

Allowable block shear strength for both angles (ASD)

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

2.82

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

2.83

Example 5

Design block shear strength for both angles (LRFD)

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

2.84

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

2.85

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

2.86

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

- Using Tables
  Tension rupture

  ASD 21.8 kips/in.
  LRFD 32.6 kips/in

2.87

Example 5

Table 9-3b (continued)
Block Shear
Shear Yielding
Component
per inch of thickness, kips/in.

L_{ev}, in.	n	ASD	LRFD
3		162	243
1¼	4	111	166
1⅜		112	168
1½		113	170
1⅝		115	172
1¾		116	174
1⅞		117	176
2		119	178
2¼		121	182
2½		124	186
2¾		127	190
3		130	194

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

Table 9-3c (continued)
Block Shear
Shear Rupture
Component
per inch of thickness, kips/in.

L_{ev}, in.	n	ASD	LRFD	ASD	LRFD
3		192	289	183	274
1¼	4	125	188	117	176
1⅜		127	191	120	179
1½		129	194	122	183
1⅝		132	197	124	186
1¾		134	201	126	189
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	LRFD
$\Omega = 2.00$	$\phi = 0.75$

- Using Tables
  Select the smaller value from shear yielding or rupture.

  ASD 113 kips/in.
  LRFD 170 kips/in.

2.88

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

2.89

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

2.90

Eyebars

D6 Eyebars

D6.1. Tensile Strength

– Yielding on the body of eyebar

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

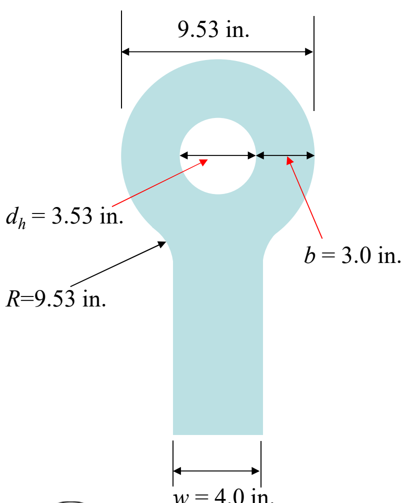
D6.2. Dimensional Requirements

– Establish proportions so that yielding of the body is the only limit state to be checked.

– Requirements (a) through (f).

2.91

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.

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

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

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

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

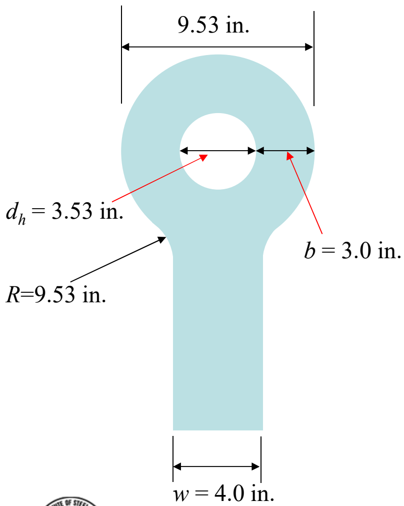
$$R \geq 2b + d_h = 2(3.0) + 3.53 = 9.53\text{ in.}$$

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

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

2.92

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.

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

For LRFD

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

For ASD

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

2.93

Pin Connected Members

D5 Pin Connected Members

D5.1. Tensile Strength

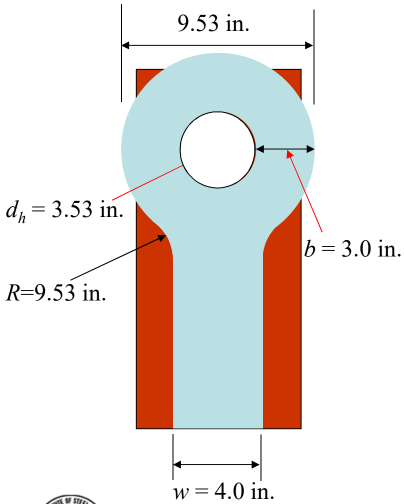
- Tensile rupture on effective net area
- Shear rupture on effective area
- Yielding on the gross section
- Bearing on the projected area of pin

D5.2. Dimensional Requirements

- Requirements (a) through (d).

2.94

Pin Connected Members



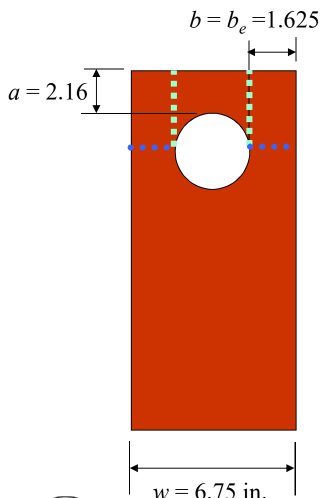
Compare a pin connected member that uses the same size pin as the eyebar just considered. Assume a ½ in. thick plate.

Check the dimensional requirements in Section D5.

$$d = 3.5 \text{ in.}$$
$$t = 0.5 \text{ in.}$$
$$d_h \le d + \tfrac{1}{32} = 3.53 \text{ in.}$$
$$b_e = 2t + 0.63 \text{ in.} = 2(0.5) + 0.63$$
$$= 1.63 \text{ in.} \le b$$
$$w \ge 2b_e + d = 2(1.625) + 3.5 = 6.75 \text{ in.}$$
$$a \ge 1.33b_e = 1.33(1.625) = 2.16 \text{ in.}$$

2.95

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.

Tensile rupture limit state (Eq. D5-1)

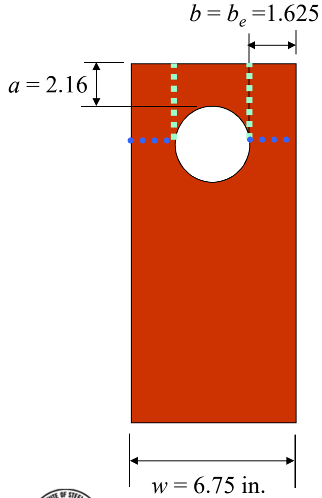
$$P_n = F_u(2tb_e) = 58(2(0.5)(1.63)) = 94.5 \text{ kips}$$
$$\phi P_n = 0.75(94.5) = 70.9 \text{ kips}$$

Shear rupture limit state (Eq. D5-2)

$$P_n = 0.6F_u\left(2t\left(a + \tfrac{d}{2}\right)\right)$$
$$= 0.6(58)\left(2(0.5)\left(2.16 + \frac{3.5}{2}\right)\right) = 136 \text{ kips}$$
$$\phi P_n = 0.75(136) = 102 \text{ kips}$$

2.96

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.

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

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

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

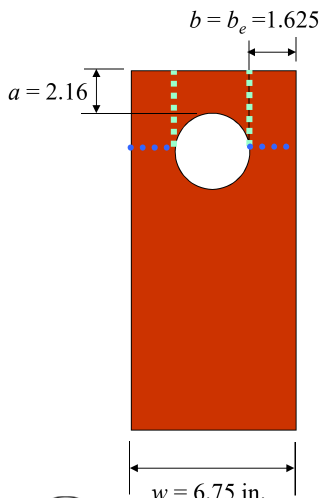
Yield on the gross section (Eq. D2-1)

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

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

2.97

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

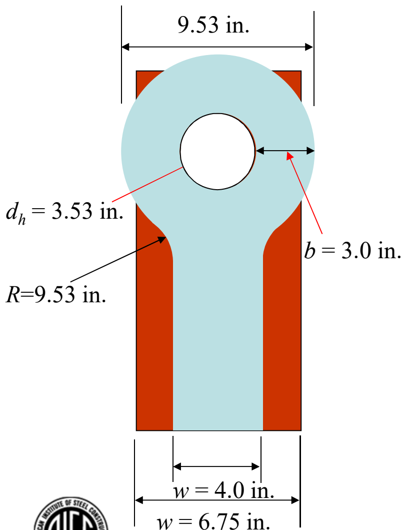
$$\phi R_n = 0.75(94.5) = 70.9 \text{ kips}$$

For ASD

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

2.98

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)

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

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

2.99

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

Lesson 3

- 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

1.101

Thank You

American Institute of Steel Construction
130 East Randolph St., Suite 2000
Chicago, IL 60601

2.102

Individual Session Registrants

PDH Certificates

- You will receive an email on how to report attendance from: registration@aisc.org.
- Be on the lookout: Check your spam filter! Check your junk folder!
- Completely fill out online form. Don't forget to check the boxes next to each attendee's name!

Individual Session Registrants

PDH Certificates

- Reporting site (URL will be provided in the forthcoming email).
- Username: Same as AISC website username.
- Password: Same as AISC website password.

8-Session Registrants

PDH Certificates

One certificate will be issued at the conclusion of all 8 sessions.

8-Session Registrants

Access to the quiz

Information for accessing the quiz will be emailed to you by Thursday. It will contain a link to access the quiz. EMAIL COMES FROM NIGHTSCHOOL@AISC.ORG.

Quiz and attendance records

Posted Thursday mornings. www.aisc.org/nightschool -- Click on Current Course Details.

Reasons for quiz

- EEU – You must take all quizzes and the final exam to receive EEU.
- PDHs – If you watch a recorded session, you must pass quiz for PDHs.
- REINFORCEMENT – Reinforce what you learn tonight. Get more out of the course.

Note: If you attend the live presentation, you do not have to take the quizzes to receive PDHs

8-Session Registrants

Access to the recording

Information for accessing the recording will be emailed to you by Thursday. The recording will be available for four weeks. (For 8-session registrants only.) EMAIL COMES FROM NIGHTSCHOOL@AISC.ORG.

PDHs via recording

If you watch a recorded session, you must take *and pass* the quiz for PDHs.

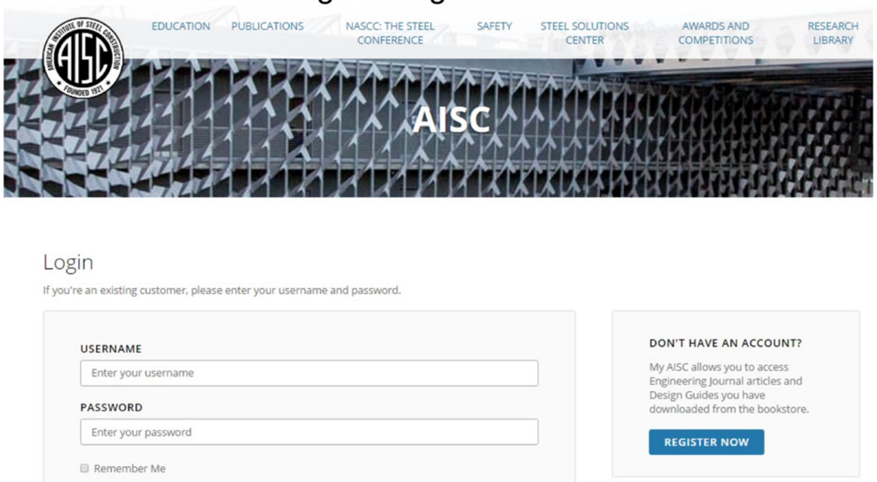
8-Session Registrants

Night School Resources

Find all your handouts, quizzes and quiz scores, recording access, and attendance information all in one place!

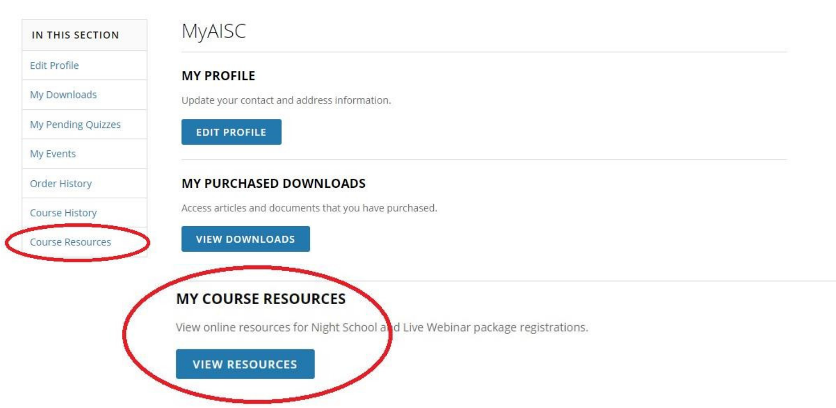
Night School Resources for 8-session package Registrants

Go to www.aisc.org and sign in.



Night School Resources for 8-session package Registrants

Go to www.aisc.org and sign in.



Please fill out our brief survey at the conclusion of the webinar. We greatly appreciate your feedback.

AISC | Thank you

Smarter.
Stronger.
Steel.