




**Basic Steel Design**  
Louis F. Geschwindner





Smarter.  
Stronger.  
Steel.



Welcome to today's webinar.



Today's audio will be broadcast through the internet.  
Please be sure and turn up the volume on your speakers.



Today's live webinar will begin shortly.  
Please standby.

Today's audio will be broadcast through the internet.

Please type any questions or comments through the Chat feature on the left portion of your screen.




**AIA Credit**

AISC is a Registered Provider with The American Institute of Architects Continuing Education Systems (AIA/CES). Credit(s) earned on completion of this program will be reported to AIA/CES for AIA members. Certificates of Completion for both AIA members and non-AIA members are available upon request.

This program is registered with AIA/CES for continuing professional education. As such, it does not include content that may be deemed or construed to be an approval or endorsement by the AIA of any material of construction or any method or manner of handling, using, distributing, or dealing in any material or product.

Questions related to specific materials, methods, and services will be addressed at the conclusion of this presentation.





### Copyright Materials

This presentation is protected by US and International Copyright laws. Reproduction, distribution, display and use of the presentation without written permission of AISC is prohibited.

© The American Institute of Steel Construction 2020



### 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, 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.



### 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



Smarter.  
Stronger.  
Steel.



## 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  $\leq$  Available Strength

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

where

$R_u$  = required strength (LRFD) defined in Chapter C

$R_n$  = nominal strength specified in Chapter D

$\phi$  = resistance factor specified in Chapter D

$\phi R_n$  = design strength = resistance factor (nominal strength)



2.10

## Tension Members

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

Required Strength  $\leq$  Available Strength

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

where

$R_a$  = required strength (ASD) defined in Chapter C

$R_n$  = nominal strength specified in Chapter D

$\Omega$  = safety factor specified in Chapter D

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



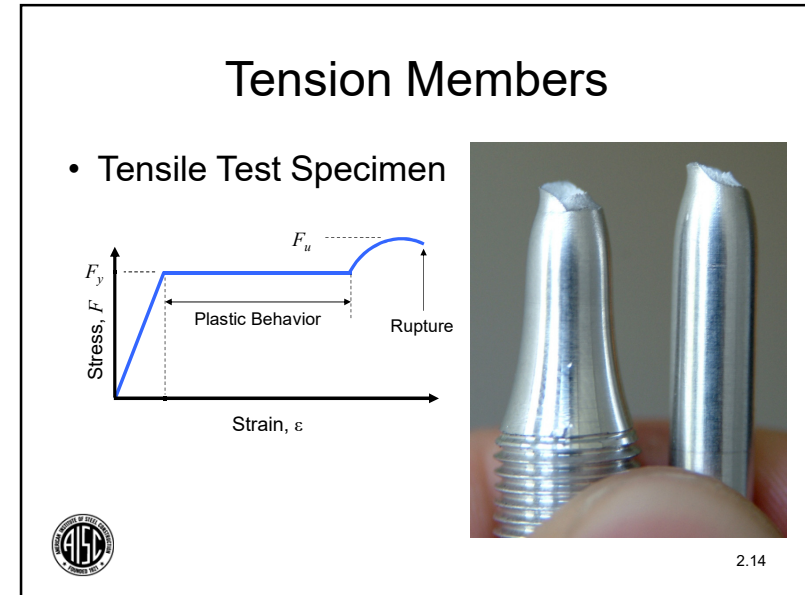
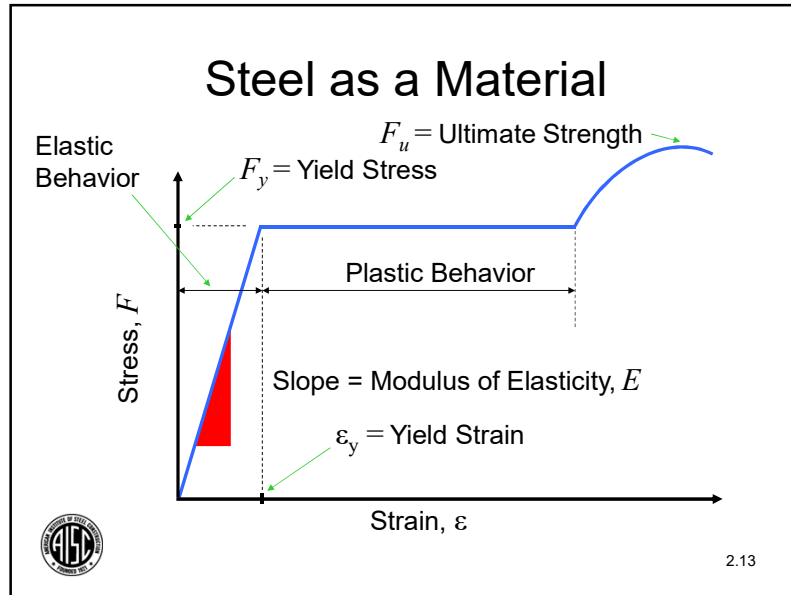
2.11

## Tension Members

D2. “The design tensile strength,  $\phi_t P_n$ , and the allowable tensile strength,  $P_n / \Omega_t$ , 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



### Tension Members

Two limit states to be considered for tension

D2.(a) Nominal Strength for Yielding;

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

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

2.15

### Tension Members

Two limit states to be considered for tension

D2.(b) Nominal Strength for Rupture

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

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

2.16

### Design for Tension

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

$$P_a \leq 0.6F_y A_g$$

$$\leq 0.5F_u A_e$$



2.17

### Tension Members

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

$$P_u \leq 0.9F_y A_g$$

$$\leq 0.75F_u A_e$$



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.”

Table 1-7  
**Angles**  
 Properties

Shape	k	WT		Area, $A$	$I$	Axis X-X					Flexura Prod $J$
		in.	lb/ft			in. <sup>2</sup>	in. <sup>4</sup>	$r$	$\bar{y}$	$Z$	
L8x8x1/8	1 1/4	56.9	16.8	98.1	17.5	2.41	2.40	31.6	1.05	7.13	32
x1	1 1/8	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	11



2.20

### B4.3b. Net Area, $A_n$

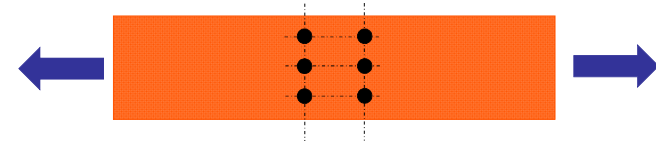
- Net area accounts for holes in the member
- In the simplest case it is the gross area less the area of holes,  $(d_b + 1/16" + 1/16")t$ 
  - $d_b$  = bolt diameter
  - $t$  = element thickness
  - $(d_b + 1/16 \text{ in.})$  = the hole size for standard holes and bolts less than 1.0 in. in diameter
  - 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,  $A_n = A_g$

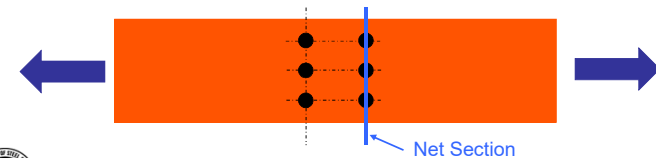


2.23

### B4.3b. Net Area, $A_n$

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

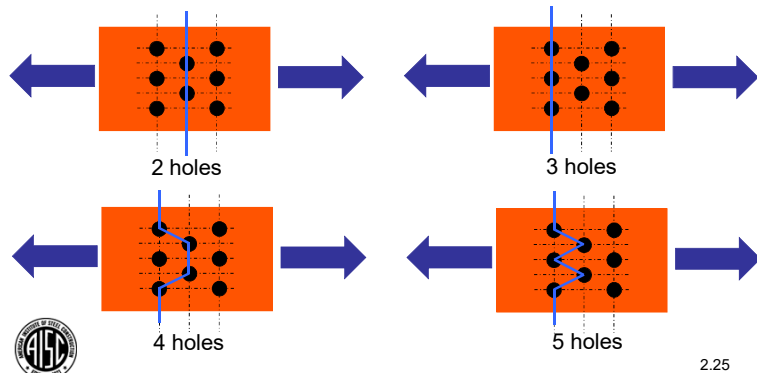
6 – 3/4 in. bolts       $A_g = tb = 0.5(8.0) = 4.0 \text{ in.}^2$   
 8 in. plate width  
 1/2 in. plate thickness       $A_n = 4.0 - 3\left(\frac{3}{4} + \frac{1}{16} + \frac{1}{16}\right)(0.5) = 2.69 \text{ in.}^2$



2.24

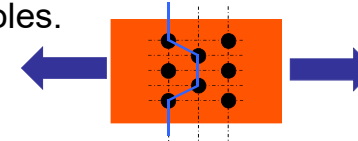
### B4.3b. Net Area, $A_n$

- What happens if the bolts are staggered?



### B4.3b. Net Area, $A_n$

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



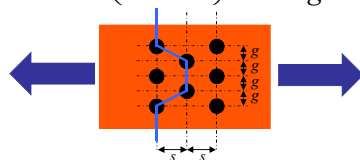
- There is more area to resist the force along the diagonals than if the holes were all in a straight line. So, after we deduct the 4 holes we must add something back.



### 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

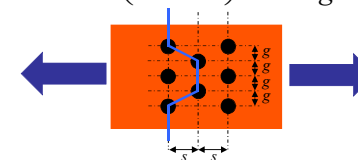
$$b_n = b - n \left( d_h + \frac{1}{16} \right) + \sum \frac{s^2}{4g}$$



### 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.

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



### B4.3b. Net Area, $A_n$

Look at the 4 possible chains

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

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

2.29

### B4.3b. Net Area, $A_n$

Look at the 4 possible chains

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

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

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

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

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

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

Now both chains show an area reduction

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

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.

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

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

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

2.32

### D3. Effective Net Area, $A_e$

$$A_e = A_n U \quad (D3-1)$$

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



2.33

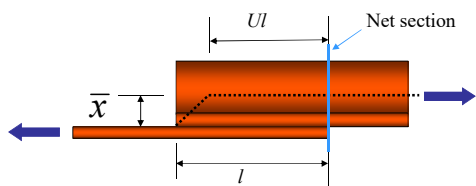
### D3. Effective Net Area, $A_e$

$$A_e = A_n U \quad (D3-1)$$

- 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

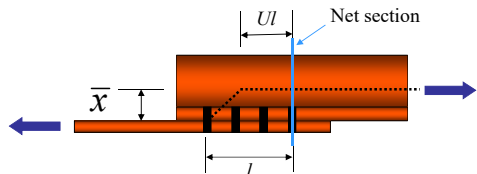
### D3. Effective Net Area, $A_e$

$$A_e = A_n U \quad (D3-1)$$

- 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

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 2 is permitted for W, M, S and HP shapes. (For angles, Case 6 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 <sup>(1)</sup>	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	$l \geq H, U = 1 - \frac{\bar{x}}{l}$ $\bar{x} = \frac{B^2}{4(B+H)}$	



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 $b_f \geq \frac{2}{3}d, U = 0.90$ $b_f < \frac{2}{3}d, U = 0.85$	
8	Single and double angles. (If U is calculated per Case 2, the larger value is permitted to be used.)	with web connected with four or more fasteners per line in the direction of loading	$U = 0.70$
		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);  
 $\bar{x}_1 = \frac{l_1 + l_2}{2}$ , 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  $U$ .

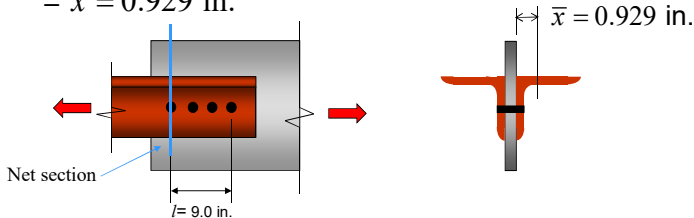
“For open cross-sections such as W, M, S, C, or HP shapes, WT’s, ST’s and single and double angles; the shear lag factor,  $U$ , need not be less than the ratio of the gross area of the connected element(s) to the member gross area.”



2.39

### Example 1

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



2.40

## Example 1

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

$$A_n = A_g - 2 \text{ holes}$$

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



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

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

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



2.43

## Example 1

- Yielding

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

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

- Rupture

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

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



2.44

## Example 1


- **ASD**
  - Yielding

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

- Rupture

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

Bolt limit states: 130 kips



2.45

## Example 1

- **LRFD**
  - Yielding


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

- Rupture

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

Bolt limit states: 195 kips

The same limit state controls for ASD and LRFD




2.46

## Example 1

Yielding  
For ASD  
 $2(59.5) = 119 \text{ kips}$   
For LRFD  
 $2(89.4) = 179 \text{ kips}$   
Just as we have determined

Shape	Gross Area, $A_g$	$A_e = 0.75A_g$	Yielding		Rupture	
			kips		kips	
			$P_t/\Omega_t$	$\phi_t P_t$	$P_t/\Omega_t$	$\phi_t P_t$
in. <sup>2</sup>	in. <sup>2</sup>	ASD	LRFD	ASD	LRFD	
L3½×3×½	×7/16	2.27	65.1	97.8	65.8	98.7
	×5/8	2.00	57.6	86.5	58.0	87.0
	×3/8	1.74	50.0	75.2	50.5	75.7
	×1/8	1.46	42.0	63.2	42.3	63.5
	×1/4	1.19	34.1	51.2	34.5	51.8
L3½×2½×½	×7/16	2.08	59.7	89.7	60.3	90.5
	×5/8	1.59	45.7	68.7	46.1	69.2
	×3/8	1.34	38.6	58.0	38.9	58.3
	×1/8	1.09	31.3	47.0	31.6	47.4
	×1/4	0.818	23.5	35.3	23.7	35.6
L3×3×½	×7/16	2.43	72.4	108.6	73.2	109.8
	×5/8	2.11	63.3	94.9	64.4	96.6
	×3/8	1.78	52.4	78.7	53.8	80.7
	×1/8	1.44	42.3	63.5	42.7	64.1
	×1/4	1.09	31.6	47.4	31.8	47.8



2.47

## Example 1


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

Note that  $A_e = 0.75A_g$

Shape	Gross Area, $A_g$	$A_e = 0.75A_g$	Yielding		Rupture	
			kips		kips	
			$P_t/\Omega_t$	$\phi_t P_t$	$P_t/\Omega_t$	$\phi_t P_t$
in. <sup>2</sup>	in. <sup>2</sup>	ASD	LRFD	ASD	LRFD	
L3½×3×½	×7/16	2.27	65.1	97.8	65.8	98.7
	×5/8	2.00	57.6	86.5	58.0	87.0
	×3/8	1.74	50.0	75.2	50.5	75.7
	×1/8	1.46	42.0	63.2	42.3	63.5
	×1/4	1.19	34.1	51.2	34.5	51.8
L3½×2½×½	×7/16	2.08	59.7	89.7	60.3	90.5
	×5/8	1.59	45.7	68.7	46.1	69.2
	×3/8	1.34	38.6	58.0	38.9	58.3
	×1/8	1.09	31.3	47.0	31.6	47.4
	×1/4	0.818	23.5	35.3	23.7	35.6
L3×3×½	×7/16	2.43	72.4	108.6	73.2	109.8
	×5/8	2.11	63.3	94.9	64.4	96.6
	×3/8	1.78	52.4	78.7	53.8	80.7
	×1/8	1.44	42.3	63.5	42.7	64.1
	×1/4	1.09	31.6	47.4	31.8	47.8

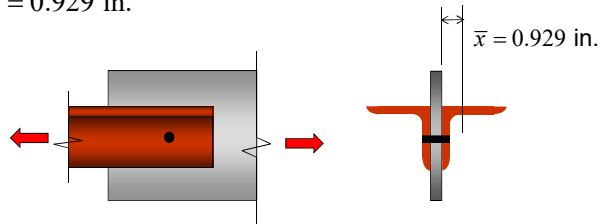


2.48

## Example 2

Example 1 with only one bolt as in a kicker

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



2.49

## Example 2

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

$$A_n = A_g - 2 \text{ holes}$$

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



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

$$= (36)(5.52) = 199 \text{ kips}$$
- Rupture
 
$$P_n = F_u A_e \quad (D2-2)$$

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




2.53

### Example 2

- ASD
  - Yielding
 
$$\frac{P_n}{\Omega} = \frac{199}{1.67} = 119 \text{ kips}$$
  - Rupture
 
$$\frac{P_n}{\Omega} = \frac{142}{2.00} = 71.0 \text{ kips} \star$$

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

The same limit state controls for ASD and LRFD

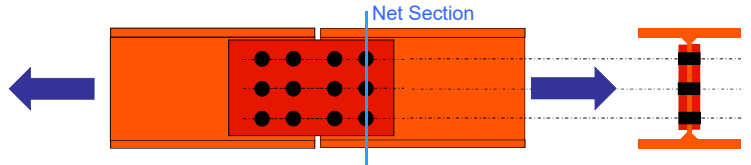
Was 177 kips in Example 1.  
Single bolt shear = 48.7 kips




2.55

### Example 3

- Determine the available strength of the W10x19, A992 tension member spliced at the web as shown with 3/4 in. A325N bolts.



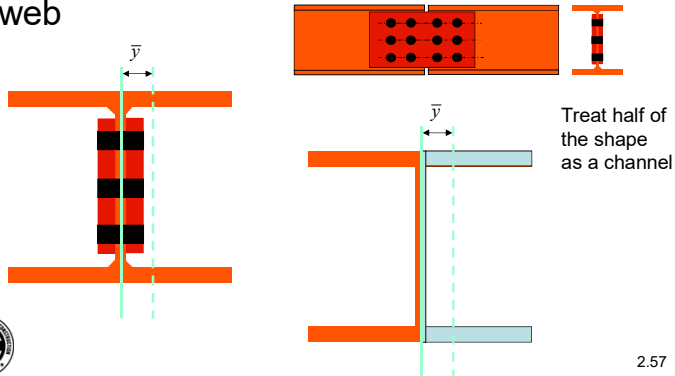
At the net section we are going to deduct for three bolt holes to get the net area. To get the effective net area we must account for the fact that the flanges are not connected



2.56

### Example 3

- Shear Lag Factor for a W attached only at web



2.57

### Example 3

- W10x19

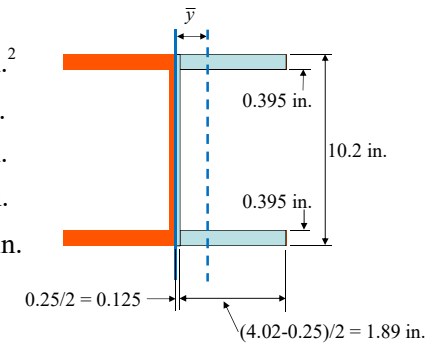
$$A_g = 5.62 \text{ in.}^2$$

$$d = 10.2 \text{ in.}$$

$$t_w = 0.25 \text{ in.}$$

$$b_f = 4.02 \text{ in.}$$

$$t_f = 0.395 \text{ in.}$$



2.58

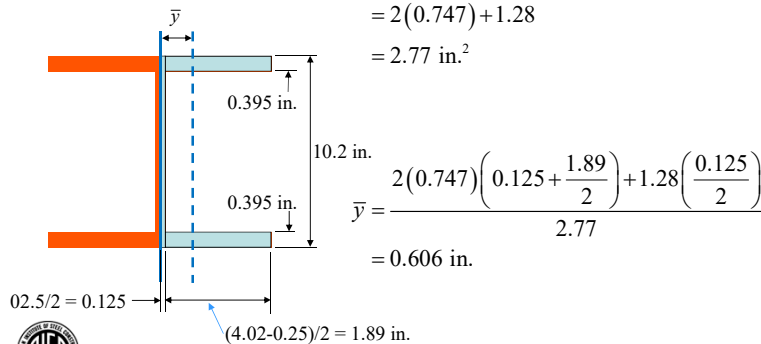
### Example 3

- W10x19

$$A_{\text{channel}} = 2(1.89(0.395)) + 10.2(0.125)$$

$$= 2(0.747) + 1.28$$

$$= 2.77 \text{ in.}^2$$



2.59

### Example 3

- W10x19

– Gross area given  $A_g = 5.62 \text{ in.}^2$

– Net area

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

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

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



2.60

### Example 3

- Yielding

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

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

- Rupture

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

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



2.61

### Example 3

- ASD

– Yielding

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

– Rupture

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

Six bolts = 143 kips



2.62

### Example 3

- LRFD

– Yielding

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

– Rupture

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

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

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

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



Shape	Gross Area, $A_g$		Yielding				Rupture	
	in. <sup>2</sup>	in. <sup>2</sup>	kips		kips		kips	
			$P_n/\Omega_t$	$\phi_t P_n$	$P_n/\Omega_t$	$\phi_t P_n$		
W10-112	32.9	24.7	885	1480	803	1200		
>100	29.3	22.0	877	1320	715	1070		
>80	26.0	19.5	779	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	388	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	6.40	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



Shape	Gross Area, $A_g$		Yielding				Rupture			
	$A_e = 0.75A_g$		kips		kips		kips		kips	
	in. <sup>2</sup>	in. <sup>2</sup>	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W10x112	32.9	24.7	985	1400	959	1200				
⋮										
W10x45	13.3	9.88	398	599	324	487				

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

$F_y = 36$  ksi  
 $F_u = 58$  ksi



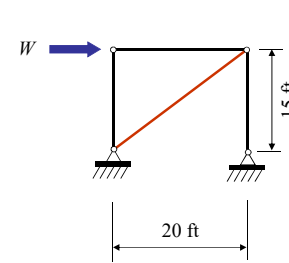
Shape	Gross Area, $A_g$		Yielding				Rupture			
	$A_e = 0.75A_g$		kips		kips		kips		kips	
	in. <sup>2</sup>	in. <sup>2</sup>	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
L3 1/2 x 3 1/2	3.02	2.27	65.1	95.8	65.0	88.7				
⋮										
L3 1/2 x 3 1/2	2.77	2.08	59.7	88.7	65.0	88.5				



2.65

## Example 4

- Design a diagonal tension brace as shown.



Use a single angle A36 member  
Assume 3/4 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,

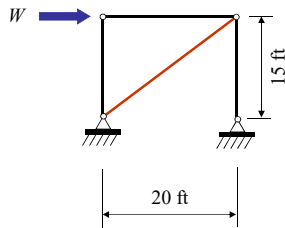
LRFD:  $W = 1.0(50) = 50$  kips  
ASD:  $W = 0.6(50) = 30$  kips



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

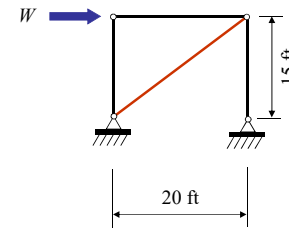
$$A_g = \frac{P_r}{\phi F_y} = \frac{62.5}{0.9(36)} = 1.93 \text{ in.}^2$$



2.67

## Example 4 (LRFD)

- Design a diagonal tension brace as shown.



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

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

Now consider the limit state of rupture

$$\bar{x} = 0.884 \text{ in.}$$

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

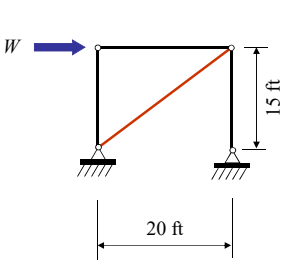
$$U = 1 - \frac{\bar{x}}{l} = 1 - \frac{0.884}{9.0} = 0.902$$



2.68

### Example 4 (LRFD)

- Design a diagonal tension brace as shown.



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

Effective net area  
 $A_e = U A_n = 0.902(1.78) = 1.61 \text{ in.}^2$

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

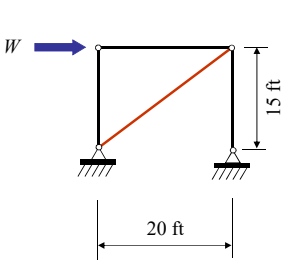
Available rupture strength  
 $\phi P_n = 0.75(93.4) = 70.1 \text{ kip}$



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

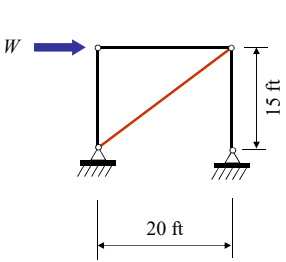
A 3x3x3/8 A36 angle will work. If more than 4 bolts are required, the shear lag factor will increase so this solution is conservative. If fewer than 4 bolts are needed, the rupture limit state must be rechecked.



2.70

### Example 4 (ASD)

- Design a diagonal tension brace as shown.



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.

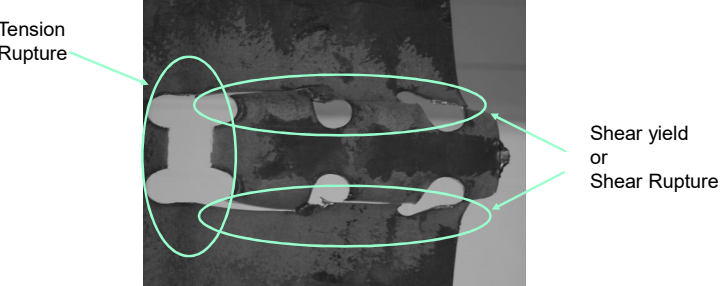
$P_r = 30\left(\frac{25}{20}\right) = 37.5 \text{ kips}$



2.71

### Block Shear

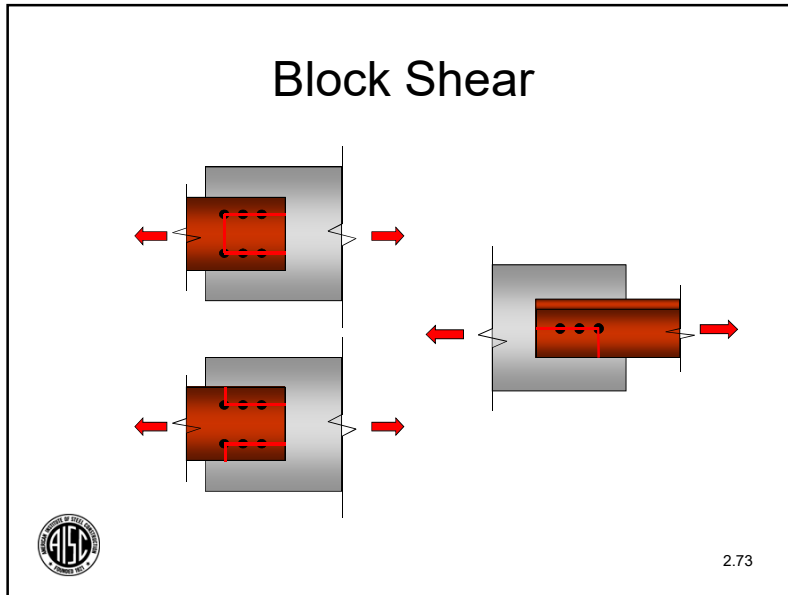
#### J4.3. Block Shear Strength (Another connection issue)



This is actually a connection issue.




2.72



### 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

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


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

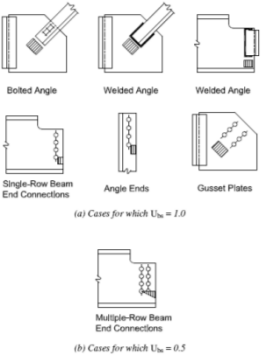
### Block Shear Reduction Factor

- For uniform tensile stress  $R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt}$  (J4-5)

$$U_{bs} = 1.0$$

- For nonuniform tensile stress

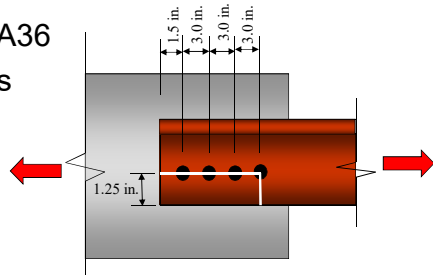
$$U_{bs} = 0.5$$



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

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

Tension rupture

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



2.80

## Example 5

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

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

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



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

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

- 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**)

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



2.84

### Example 5

- Summary **LRFD**
  - Tension on angles from Example 1

$$\phi P_n = 177 \text{ kips} \star$$

– Block shear

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

Again, the same limit state controls  
Bolt limit states = 195 kips



### 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

$U_{BS} = 1.0$

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

$L_{eh}$ , in.	58 ksi Bolt diameter, $d$ , in.					
	$3/4$		$7/8$		1	
	$F_u A_{nt}$ k/2	$\phi F_u A_{nt}$ t	$F_u A_{nt}$ k/2	$\phi F_u A_{nt}$ t	$F_u A_{nt}$ k/2	$\phi F_u A_{nt}$ t
1	16.3	24.5	14.5	21.8	12.7	19.0
1 1/4	19.9	29.9	18.1	27.2	16.3	24.5
1 1/2	23.6	35.3	21.8	32.6	19.9	29.9
1 3/4	27.2	40.8	25.4	38.1	23.6	35.3
1 3/8	30.8	46.2	29.0	43.5	27.2	40.8
1 5/8	34.4	51.7	32.6	48.9	30.8	46.2
1 7/8	38.1	57.1	36.3	54.4	34.4	51.7
2	41.7	62.5	39.9	59.8	38.1	57.1
2 1/4	45.3	68.0	43.5	65.2	41.7	62.5

- Using Tables  
Tension rupture  
ASD 21.8 kips/in.  
LRFD 32.6 kips/in.



### Example 5

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

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

$L_{eh}$ , in.	3		4		5		6	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3	162	243	211	317	259	389	274	411
1 1/4	111	166	142	214	183	274	176	264
1 1/2	112	168	144	216	186	276	178	266
1 3/4	113	170	145	217	187	277	179	267
1 5/8	115	172	147	219	189	279	181	269
1 7/8	116	174	148	220	190	280	182	270
2	117	176	149	221	191	281	183	271
2 1/4	119	178	151	223	193	283	185	273
2 1/2	121	182	153	225	195	285	187	275
2 3/4	124	186	156	228	198	288	190	278
3	127	190	159	231	201	291	193	281
3 1/4	130	194	162	234	204	294	196	284

- Using Tables  
Select the smaller value from shear yielding or rupture.  
ASD 113 kips/in.  
LRFD 170 kips/in.



### 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

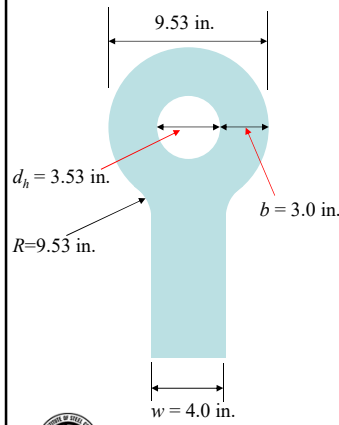


- D6 Eyebars
- D6.1. Tensile Strength
- Yielding on the body of eyebar
- $$P_n = F_y A_g \quad (D2-1)$$
- D6.2. Dimensional Requirements
- Establish proportions so that yielding of the body is the only limit state to be checked.
  - Requirements (a) through (f).



2.91

### Eyebars



Determine the available strength of a 1/2 in. by 4.0 in. eyebar with a 3.5 in. pin. First check the dimensional requirements of Section D6.2.

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

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

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

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

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

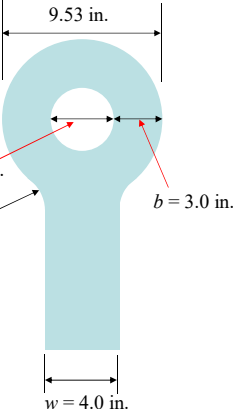
$$\frac{2}{3}w \leq b \leq \frac{3}{4}w$$

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



2.92

## Eyebars



Determine the available strength of a 1/2 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

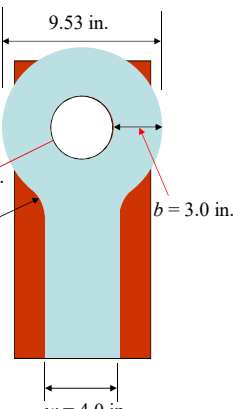
#### D5.2. Dimensional Requirements

- 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 1/2 in. thick plate.

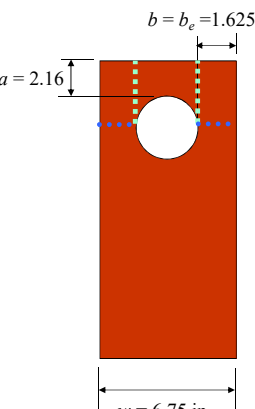
Check the dimensional requirements in Section D5.

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



2.95


## Pin Connected Members



Compare a pin connected member that uses the same size pin as the eyebar just considered. Assume a 1/2 in. thick plate.

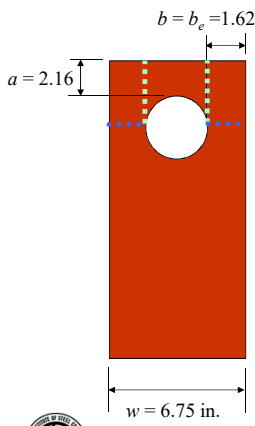
The dimensional requirements are satisfied.

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



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.

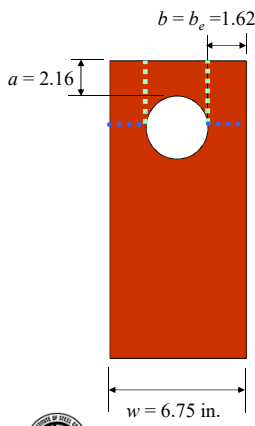
Bearing on projected area of pin (Eq. J7-1)  
 $R_n = 1.8F_y A_{pb} = 1.8(36)(0.5)(3.5) = 113$  kips  
 $\phi R_n = 0.75(113) = 85.1$  kips

Yield on the gross section (Eq. D2-1)  
 $R_n = F_y A_g = (36)(0.5)(6.75) = 122$  kips  
 $\phi R_n = 0.9(122) = 110$  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.

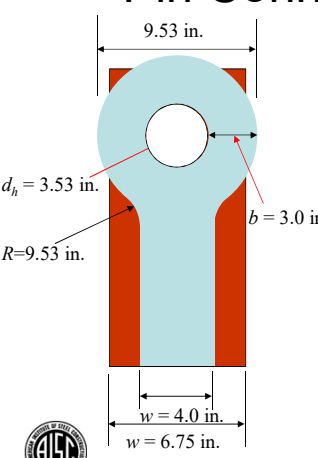
For LRFD  
 $\phi R_n = 0.75(94.5) = 70.9$  kips

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



2.98


### Pin Connected Members



Compare the strengths of the designed eyebar and pin connected member

Eyebar (Yield)  
 $\phi P_n = 0.9(72.0) = 64.8$  kips  
 $\frac{P_n}{\Omega} = \frac{72.0}{1.67} = 43.1$  kips


Pin Connected Member (Tensile rupture)  
 $\phi R_n = 0.75(94.5) = 70.9$  kips  
 $\frac{R_n}{\Omega} = \frac{94.5}{2.00} = 47.3$  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](mailto: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](mailto:NIGHTSCHOOL@AISC.ORG).

### Quiz and attendance records

Posted Thursday mornings. [www.aisc.org/nightsschool](http://www.aisc.org/nightsschool) -- 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](mailto:NIGHTSCHOOL@AISC.ORG).

### PDHs via recording

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



## 8-Session Registrants

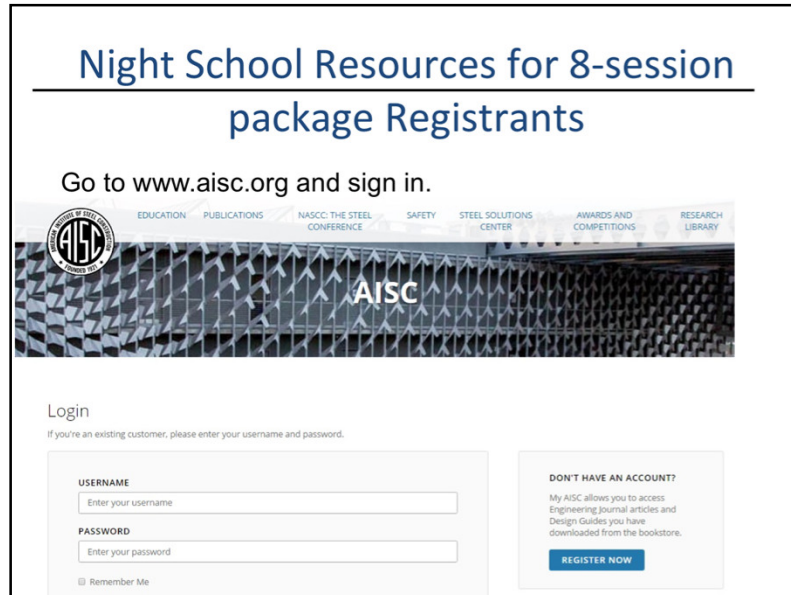
### Night School Resources

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](http://www.aisc.org) and sign in.



Education Publications NASCC: THE STEEL CONFERENCE SAFETY STEEL SOLUTIONS CENTER AWARDS AND COMPETITIONS RESEARCH LIBRARY

AINC

Login

If you're an existing customer, please enter your username and password.

USERNAME  
Enter your username

PASSWORD  
Enter your password

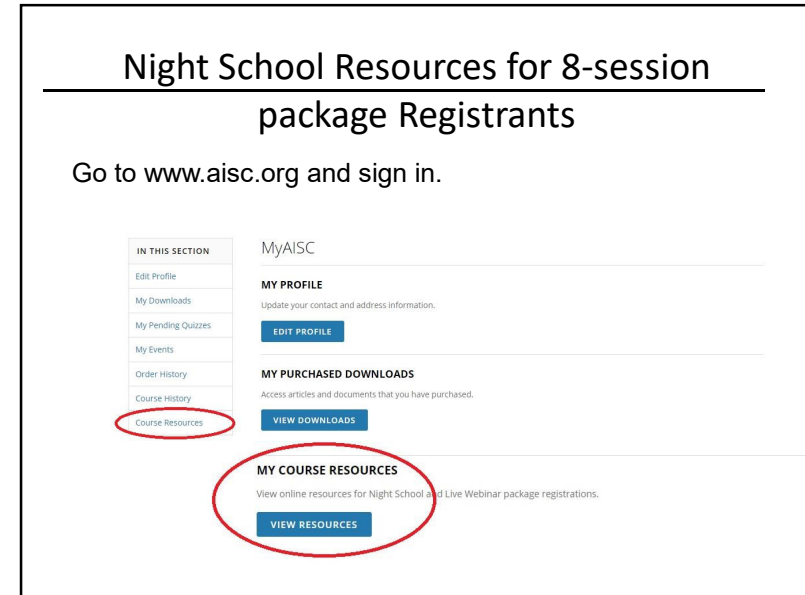
Remember Me

DON'T HAVE AN ACCOUNT?  
My AISC allows you to access Engineering Journal articles and Design Guides you have downloaded from the bookstore.

REGISTER NOW

## Night School Resources for 8-session package Registrants

Go to [www.aisc.org](http://www.aisc.org) and sign in.



MyAISC

IN THIS SECTION

- Edit Profile
- My Downloads
- My Pending Quizzes
- My Events
- Order History
- Course History
- Course Resources

MY PROFILE

Update your contact and address information.

EDIT PROFILE

MY PURCHASED DOWNLOADS

Access articles and documents that you have purchased.

VIEW DOWNLOADS


MY COURSE RESOURCES

View online resources for Night School and Live Webinar package registrations.

VIEW RESOURCES

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

AISC | Thank you



Smarter. Stronger. Steel.