INSTRUCTOR'S SOLUTION MANUAL

Salmon/Johnson/Malhas

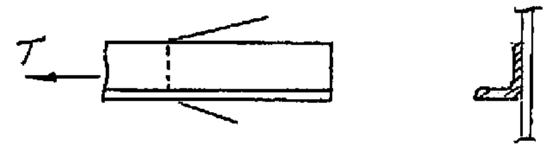
STEEL STRUCTURES
Design and Behavior
Fifth Edition

3.1 Compute the maximum acceptable tensile service bad that may be placed on a single angle L×4×3/4 that is welded along its long leg to a gusset plate. The live load is three times the dead load. Solve for (a) A36 steel and (b) A572 Grade 50 steel.

Section: Angle 6 × 4 × 3/4; A_g = 6.94 sq in.; x = 1.07 in.

Compute the effective net area A_e. When the length L of weld used is known the reduction coefficient U relating to shear lag can be computed using Eq. 3.5.2. In this case, use the practical U = 0.85 for an angle attached on one leg.

$$A_e = UA_g = 0.85(6.94) = 5.90 \text{ sq in.}$$



(a) <u>A36 steel. AISC</u>

Yielding in gross section:

$\phi_t T_n = \phi_t F_y A_g$ = 0.90(36)6.94 = 225 kips <u>Controls!</u>

Fracture in effective net section:

$\phi_t T_n = \phi_t F_u A_e$ =0.75(58)5.90 = 257 kips

Factored load: $T_u = 1.2D + 1.6L = 1.2D + 1.6(3D) = 6.0D$

or $T_u = 1.4D$

Service dead load $= \phi_t T_n/6$ = 225/6 = 37.5 kips

Service live load $=3D$ = 3(37.5) = 112 kips

Total service load $T = D + L$ = 37.5 + 112 = 150 kips

(b) <u>A572 Grade 50 steel. AISC</u>

Yielding in gross section:

$\phi_t T_n = \phi_t F_y A_g$ = 0.90(50)6.94 = 312 kips

Fracture in effective net section:

$\phi_t T_n = \phi_t F_u A_e$ = 0.75(65)5.90 = 288 kips <u>Controls!</u>

Factored load: $T_u = 1.2D + 1.6L = 1.2D + 1.6(3D) = 6.0D$

or $T_u = 1.4D$

Service dead load $= \phi_t T_n/6$ = 288/6 = 47.9 kips

Service live load = 3D = 3(47.9) = 144 kips

Total service load $T = D + L$ = 47.9 + 144 = 192 kips

3.2 Compute the maximum acceptable tensile service load that may be placed on a single angle L6×4×3/4 that is connected along both legs. The 4-in. leg contains a single gage line of 7/8-in.-diam bolts and the 6-in. leg contains a double gage line of 7/8-in.-diam bolts. Assume no stagger, and that all bolts participate in carrying the load. The live load is three times the dead load. Solve for (a) A36 steel and (b) A572 Grade 50 steel

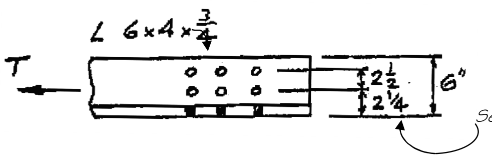
Section: Angle 6 × 4 × 3/4; A_s = 6.94 sq in.; x = 1.07 in.

Compute the effective net area A_e. When the attachment of load carrying bolts is to both legs of an angle, the reduction coefficient U relating to shear lag can be taken as U = 1.0. The net area A_n is

$A_n = A_g$ – 3holes

$= A_g$ – 3(hole diam $+\frac{1}{16}$)t = 6.94 - 3(0.9375 + $\frac{1}{16}$)0.75 = 4.69 sq in.

$A_e = UA_U$ = 1.00(4.69) = 4.69 sq in.



See AISC Manual, 'usual gages', p1-46

(a) <u>A36 steel, AISC</u>

Yielding in gross section:

$\phi_t T_n = \phi_t F_y A_a$ = 0.90(36)6.94 = 225 kips

Fracture in effective net section

$\phi_t T_n = \phi_t F_u A_a$ = 0.75(58)4.69 = 204 kips <u>Controls!</u>

Factored load: T_u = 1.2D + 1.6L = 1.2D + 1.6(3D) = 6.0D

or T_u = 1.4D

Service dead load = $\phi_t T_n$/6 = 204/6 = 34 kips

Service live load = 3D = 3(34) = 102 kips

Total service load $T = D + L$ = 34 + 102 = 136 kips

(b) <u>A572 Grade 50 steel, AISC</u>

Yielding in gross section: $\phi_t T_n = \phi_t F_y A_g$ = 0.90(50)6.94 = 312 kips

Fracture in effective net section:

$\phi_t T_n = \phi_t F_u A_a$ = 0.75(65)4.69 = 229 kips <u>Controls!</u>

Factored load: T_u = 1.2D + 1.6L = 1.2D + 1.6(3D) = 6.0D

or T_u = 1.4D

Service dead load = $\phi_t T_n$/6 = 229/6 = 38.1 kips

Service live load = 3D = 3(38.1) = 114 kips

Total service load $T = D + L$ = 38.1 + 114 = 152 kips

3.3 Compute the maximum acceptable service load on an A36 steel plate tension member 1/4-in.x 12 in. having a single bline of holes parallel to the direction of loading. The load is 25% dead load and 75% live load, and 7/8-in. diam bolts are used.

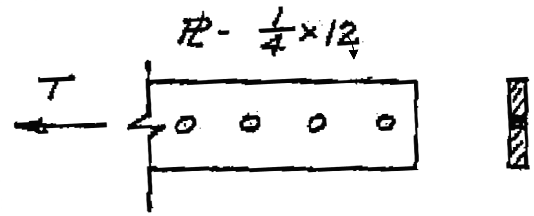
AISC-J4 for connecting elements. See AISC –D3.3 for effective net area.

Compute the effective net area A_e. The reduction coefficient U relating to shear lag = 1.0. The net area A_n is

$A_n = A_g$ – holes

$= A_g$ – (hole diam $+\frac{1}{16}$)t = 0.25(12) - 1(0.9375 + $\frac{1}{16}$)0.25 = 2.75 sq in.

$A_e = UA_g$ = 1.00(2.75) = 2.75 sq in.



A36 steel, AISC

$\phi_t T_n = \phi_t F_y A_a$ = 0.90(36)(3.00) = 97.2 kips <u>Controls!</u>

Fracture in effective net section

$\phi_t T_n = \phi_t F_u A_a$ = 0.75(58)2.75 = 120 kips

Factored load: T_u = 1.2D + 1.6L = 1.2D + 1.6(3D) = 6.0D

or T_u = 1.4D

Service dead load = $\phi_t T_n$/6 = 97.2/6 = 16.2 kips

Service live load = 3D = 3(16.2) = 48.6 kips

Total service load $T = D + L$ = 16.2 + 48.6 = 64.8 kips

If this were a splice plate under AISC-J4.1,

Max A_n = 0.85A_g = 0.85(3.00) = 2.55 sq in, Controls for A_e!

Actual A_n = 2.75 sq in.

Fracture in effective net section:

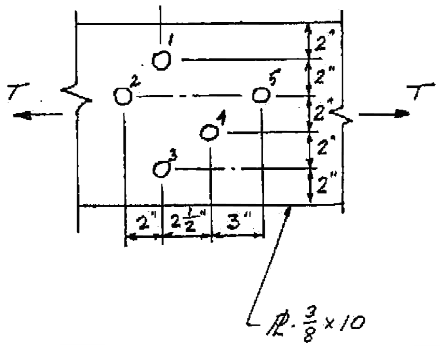
$\phi_t T_n = \phi_t F_u A_a$ = 0.75(58)2.55 = 111 kips

Strength is still controlled by yielding in gross section.

Total service load $T = D + L$ = 16.2 + 48.6 = 64.8 kips

3.4 Compute the net area A_n for the plate (a connecting element according to AISC-J4 using both the LRFD and ASD methods) shown in the accompanying figure. Then compute the maximum value for service load T when A36 steel is used, the live load is four times the deitd loud, and the holes are 13/16-in. diameter.

Compute the effective net area A_e. The reduction coefficient U relating to shear lag = 1.0. The net area A_n is



A_g= (0.375)10 = 3.75 sq in.

Max A_n = 0.85A_g
= 0.85(3.75)
= 3.19sqin.

Section 1-3 (no stagger):
A_n = A_g-2holes
= 3.75-2(0.8125 +$\frac{1}{16}$)0.375
= 3.09 sq in.

Section 1-2-3 with two stagger
A_n=A_g- 3holes + 2staggers
Stagger 1 - 2 = 2^2 / [4(2)] = 0.50 in.
Stagger 2 - 3 = 2^2 / [4(4)] = 0.25 in.
A_n=3.75-[3(0.875)-0.50-0.25]0.375
= 3.05 sq in. Minimum A_n Thus, Controls!

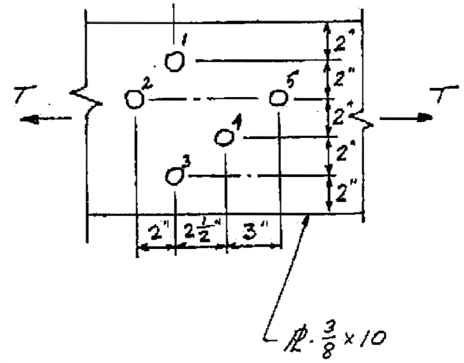
Section 1-4-3 with two stagger paths:
A_n= A_g - 3holes + 2staggers
Stagger 1-4 = 2.5^2/[4(2)] = 0.781in.
Stagger 4-3 = 2.5^2/[4(4)] = 0.391 in.
A_n=3.75-[3(0.875)-0.781-0.391]0.375
= 3.94 sq in.

<u>A36 steel, AISC</u>
Yielding in gross section: $\phi_t T_n = \phi_t F_y A_g$ = 0.90(36)(3.75) =122 kips
Fracture in effective net section: $\phi_t T_n = \phi_t F_u A_e$ = 0.75(58)3.05 = 135 kips
Controlling $\phi_t T_n$ = 122 kips
Factored load: $T_u = 1.2D + 1.6L = 1.2D + 1.6(4D) = 7.6D$ or $T_u = 1.4D$
Service dead load = $\phi_t T_n$/6 = 122/7.6 = 16.0 kips
Service live load = $4D$ = 4(16.0) = 64.0 kips
Total service load $T = D + L$ = 16.0 + 64.0 = 80.0 kips

3.5 Repeat Prob. 3.4. Compute the net area A_n for the plate (a connecting element according to AISC – J4 using both the LRFE and ASD methods) shown in the accompanying figure. Then compute the maximum value for service load T when A572 Grade 60 steel is used, the live load is four times the dead load, and the holes are 15/16-in. diameter.

Compute the effective net area A_e, The reduction coefficient U relating to shear lag = 1.0. The net area A_n is

A_g =(0.375)10 = 3.75 sq in.
Max A_n = 0.85A_g
= 0.85(3.75)
= 3.19 sq in.



Section 1-3 (no stagger):
A_n = A_g - 2holes
= 3.75-2(0.9375 +$\frac{1}{16}$) 0.375
= 3.00 sq in.

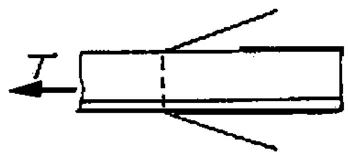
Section 1-2-3 with two stagger paths:
A_n = A_g -3holes + 2staggers
Stagger 1 - 2 = 2^2 / [4(2)] = 0.50 in.
Stagger 2 - 3 = 2^2 / [4(4)] = 0.25 in.
A_n=3.75-[3(1.00)-0.50-0.25]0.375
= 2.91sqin. Minimum A_n Thus, Controls!

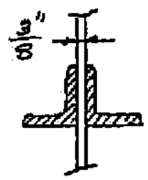
Section 1-4-3 with two stagger paths:
A_n = A_g - 3holes + 2staggers
Stagger 1-4 = 2.5^2/[4(2)] = 0.781 in.
Stagger 4-3 = 2.5^2/[4(4)] = 0.391 in.
A_n=3.75-[3(1.00)-0.781-0.391]0.375
= 3.06 sqin.

<u>A36 steel, AISC</u>
Yielding in gross section: $\phi_t T_n = \phi_t F_y A_g$ = 0.90(60)(3.75) =203 kips
Fracture in effective net section: $\phi_t T_n = \phi_t F_u A_e$ = 0.75(75)2.91 = 164 kips
Controlling $\phi_t T_n$ = 164 kips
Factored load: $T_u = 1.2D + 1.6L = 1.2D + 1.6(4D) = 7.6D$ or $T_u = 1.4D$
Service dead load = $\phi_t T_n$/6 = 164/7.6 = 21.5 kips
Service live load = $4D$ = 4(21.5) = 86.0 kips
Total service load $T = D + L$ = 21.5 + 86.0 = 108 kips

3.6 Select a pair of angles to support a tensile live load (LL) and dead load (DL) for the case assigned by the instructor. Assume the angles are separated back-to-back 3/8-in. by a connected gussel plate, and that the connection is welded. Assume the slenderncss ratio is desired to not exceed 300.

Case 1; DL = 70 kips; LL = 20 kips; A36 steel; Length L = 20 ft.





Since no holes to be deducted, $A_n = A_g$. Angles are connected along only one leg; U = 0.85 according to AISC- Comm.D3.3. Shear lag applies for both bolted and welded connections.

Factored load: T_u = 1.2D + 1.6L = 1.2(70) + 1.6(20) = 116 kips
or T_u = 1.4(70) = 98 kips

Design strengths: $\phi_t T_n = \phi_t F_y A_g$ = 0.90(36)A_g = 32.4A_g (yielding)

$\phi_t T_n = \phi_t F_u A_g$ = 0.75(58)(0.85) A_g = 37.0A_g (fracture)

Yielding of the gross section controls the design!

Required gross area $A_g = \frac{T_u}{32.4} = \frac{116}{32.4}$ = 3.58 sq in.

Desired radius of gyration $r = \frac{L}{300} = \frac{20(12)}{300}$ = 0.8 in.

From *AISC Manual* tables of double angle sections

2L-3×3×$\frac{5}{16}$; A_g = 3.55 sq in.; r_x = 0.92 in.; r_y = 1.40 in. Close!

2L-4×3$\frac{1}{2}$×$\frac{1}{4}$; A_g = 3.63 sq in.; r_x = 1.27 in.; r_y = 1.54 in. OK

(long legs back – to – back)

2L-4×3$\frac{1}{2}$×$\frac{1}{4}$; A_g = 3.63 sq in.; r_x = 1.07 in.; r_y = 1.85 in. OK

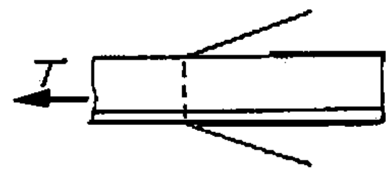
(short legs back – to – back)

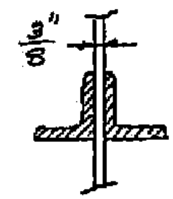
USE - 2L-4×3$\frac{1}{2}$×$\frac{1}{4}$; A_g = 3.63 sq in.;

2L-3×3×$\frac{5}{16}$ about 1% understrength

3.6 Select a pair of angles to support a tensile live load (LL) and dead load (DL) for the case assigned by the instructor. Assume the angles are separated back-to-back 3/8-in. by a connected gusset plate, and that the connection is welded. Assume the slendemess ratio is desired to not exceed 300.

Case 2: DL = 65 kips; LL = 22 kips; A36 steel; Length L = 30 ft.





Since no holes to be deducted, $A_n = A_g$. Angles are connected along only one leg; U = 0.85 according to AISC- Comm.D3.3. Shear lag applies for both bolted and welded connections.

Factored load: T_u = 1.2D + 1.6L = 1.2(65) + 1.6(22) = 113 kips
or T_u = 1.4(65) = 91 kips

Design strengths: $\phi_t T_n = \phi_t F_y A_g$ = 0.90(36)A_g = 32.4A_g (yielding)

$\phi_t T_n = \phi_t F_u A_g$ = 0.75(58)(0.85) A_g = 37.0A_g (fracture)

Yielding of the gross section controls the design!

Required gross area $A_g = \frac{T_u}{32.4} = \frac{113}{32.4}$ = 3.49 sq in.

Desired radius of gyration $r = \frac{L}{300} = \frac{30(12)}{300}$ = 1.2 in.

From *AISC Manual* tables of double angle sections

2L-4×3$\frac{1}{2}$×$\frac{1}{4}$; A_g = 3.63 sq in.; r_x = 1.27 in.; r_y = 1.54 in.

(long legs back-to-back)

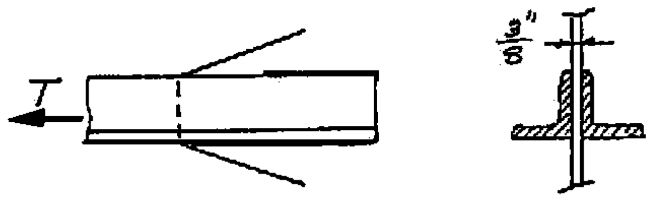
2L-5×3$\frac{1}{2}$×$\frac{1}{4}$; A_g = 3.88 sq in.; r_x = 1.62 in.; r_y = 1.21 in. OK

(long legs back – to – back)

USE - 2L-4×3$\frac{1}{2}$×$\frac{1}{4}$; A_g = 3.63 sq in.; long legs back-to-back

3.6 Select a pair of angles to support a tensile live load (LL) and dead load (DL) for the case assigned by the instructor. Assume the angles are separated back-to-back 3/8-in. by a connected gusset plate, and that the connection is welded. Assume the slenderness ratio is desired to not exceed 300.

Case 3: DL = 70 kips; LL = 20 kips; A572 Gr 60 steel; Length L = 20 ft.



Since no holes to be deducted, $A_n = A_g$. Angles are connected along only one leg; U = 0.85 according to AISC- Comm.D3.3. Shear lag applies for both bolted and welded connections.

Factored load: T_u = 1.2D + 1.6L = 1.2(70) + 1.6(20) = 116 kips
or T_u = 1.4(70) = 98 kips

Design strengths: $\phi_t T_n = \phi_t F_y A_g = 0.90(60)A_g = 54.0A_g$ (yielding)

$\phi_t T_n = \phi_t F_u A_g = 0.75(75)(0.85)\ A_g = 47.8A_g$ (fracture)

Fracture on effective net section controls the design!

Required gross area $A_g = \frac{T_u}{47.8} = \frac{116}{47.8} = 2.43$ sq in.

Desired radius of gyration $r = \frac{L}{300} = \frac{20(12)}{300} = 0.8$ in.

From *AISC Manual* tables of double angle sections

2L-$3\times2\frac{1}{2}\times\frac{1}{4}$;; A_g = 2.63 sq in.; r_x = 0.945 in.; r_y = 1.13 in. OK

(long legs back – to – back)

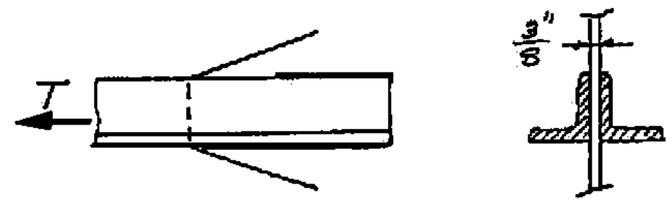
2L-$2\frac{1}{2}\times2\times\frac{5}{16}$; A_g = 2.62 sq in.; r_x = 0.776 in.; r_y = 0.948 in. Close!

(long legs back – to – back)

USE - 2L-$3\times2\frac{1}{2}\times\frac{1}{4}$; A_g = 2.63 sq in.; long legs back-to-back

3.6 Select a pair of angles to support a tensile live load (LL) and dead load (DL) for the case assigned by the instructor. Assume the angles are separated back-to-back 3/8-in. by a connected gusset plate, and that the connection is welded Assume the slenderness ratio is desired to not exceed 300.

Case 4: DL = 48 kips; LL = 30 kips; A36; Length L = 22 ft.



Since no holes to be deducted, $A_n = A_g$. Angles are connected along only one leg; U = 0.85 according to AISC- Comm.D3.3. Shear lag applies for both bolted and welded connections.

Factored load: T_u = 1.2D + 1.6L = 1.2(48) + 1.6(30) = 106 kips
or T_u = 1.4(48) = 67 kips

Design strengths: $\phi_t T_n = \phi_t F_y A_g = 0.90(36)A_g = 32.4A_g$ (yielding)

$\phi_t T_n = \phi_t F_u A_g = 0.75(58)(0.85)\ A_g = 37.0A_g$ (fracture)

Yielding of the gross section controls the design!

Required gross area $A_g = \frac{T_u}{32.4} = \frac{106}{32.4} = 3.27$ sq in.

Desired radius of gyration $r = \frac{L}{300} = \frac{22(12)}{300} = 0.88$ in.

From *AISC Manual* tables of double angle sections

2L-$4\times3\times\frac{1}{4}$; A_g = 3.38 sq in.; r_x = 1.28 in.; r_y = 1.29 in. CHOICE 1

(long legs back – to – back)

2L-$3\frac{1}{2}\times3\frac{1}{2}\times\frac{1}{4}$; A_g = 3.38 sq in.; r_x = 1.09 in.; r_y = 1.59 in. CHOICE 2

(long legs back – to – back)

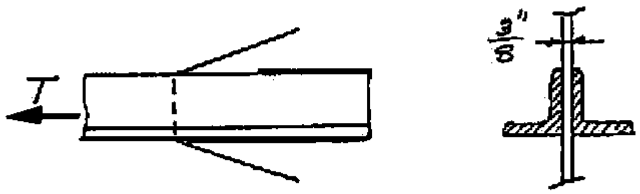
2L-$3\times2\times\frac{3}{8}$; A_g = 3.47 sq in.; r_x = 0.940 in.; r_y = 0.917 in. CHOICE 3

(long legs back – to – back)

USE - 2L-$4\times3\times\frac{1}{4}$; A_g = 3.38 sq in.; long legs back – to – back

3.6 Select a pair of angles to support a tensile live load (LL) and dead load (DL) for the case assigned by the instructor. Assume the angles are separated baclc-to-baek 3/8-in. by a connected gusset plate, and that the connection is welded. Assume the slendcmess ratio is desired to not exceed 300.

<u>Case 5:</u> DL = 50 kips; LL = 30 kips; F_y = 50 ksi; Length L = 20 ft.



Since no holes to be deducted, $A_n = A_g$. Angles are connected along only one leg; U = 0.85 according to AISC- Comm.D3.3. Shear lag applies for both bolted and welded connections.

Factored load: T_u = 1.2D + 1.6L = 1.2(50) + 1.6(30) = 108 kips

or T_u = 1.4(50) = 70 kips

Design strengths: $\phi_t T_n = \phi_t F_y A_g = 0.90(50)A_g = 45.0A_g$ (yielding)

$\phi_t T_n = \phi_t F_u A_g = 0.75(65)(0.85)\ A_g = 41.4A_g$ (fracture)

Fracture on effective net section controls the design!

Required gross area $A_g = \frac{T_u}{41.4} = \frac{108}{41.4} = 2.61$ sq in.

Desired radius of gyration $r = \frac{L}{300} = \frac{20(12)}{300} = 0.80$ in.

From *AISC Manual* tables of double angle sections

2L-$2\frac{1}{2}\times 2\times\frac{15}{16}$; A_g = 2.62 sq in.; r_x = 0.776 in.; r_y = 0.948 in. Close!

(long legs back – to – back) KL/r =309>300 limit N.G.

2L-$3\times 2\frac{1}{2}\times\frac{1}{4}$; A_g =2.63 sq in.; r_x = 0.945 in.; r_y = 1.13 in. CHOICE 1

(long legs back – to – back)

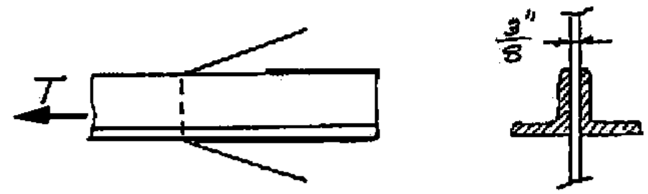
2L-$3\times 2\frac{1}{2}\times\frac{1}{4}$; A_g = 2.63 sq in.; r_x = 0.753 in.; r_y = 1.45 in. Close!

(short legs back – to – back) KL/r =309>300 limit N.G.

USE - 2L-$3\times 2\frac{1}{2}\times\frac{1}{4}$; A_g = 2.63 sq in.; long legs back – to – back

3.6 Select a pair of angles to support a tensile live load (LL) and dead load (DL) for the case assigned by the instructor. Assume the angles are separated back-to-back 3/8-in. by a connected gusset plate, and that the connection is welded. Assume the slendemess ratio is desired to not exceed 300.

<u>Case 6</u>: DL = 80 kips; LL = 30 kips; A36 steel; Length L = 20 ft.



Since no holes to be deducted, $A_n = A_g$. Angles are connected along only one leg; U = 0.85 according to AISC- Comm.D3.3. Shear lag applies for both bolted and welded connections.

Factored load: T_u = 1.2D + 1.6L = 1.2(80) + 1.6(30) = 144 kips

or T_u = 1.4(80) = 112 kips

Design strengths: $\phi_t T_n = \phi_t F_y A_g = 0.90(36)A_g = 32.4A_g$ (yielding)

$\phi_t T_n = \phi_t F_u A_g = 0.75(58)(0.85)\ A_g = 37.0A_g$ (fracture)

Yielding of the gross section controls the design!

Required gross area $A_g = \frac{T_u}{32.4} = \frac{144}{32.4} = 4.44$ sq in.

Desired radius of gyration $r = \frac{L}{300} = \frac{20(12)}{300} = 0.80$ in.

From *AISC Manual* tables of double angle sections

2L-$4\times 4\times\frac{5}{16}$;; A_g = 4.80 sq in.; r_x = 1.24 in.; r_y = 1.80 in. OK

(long legs back – to – back)

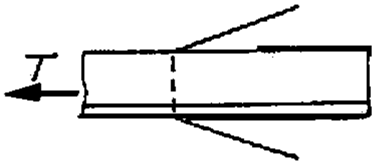
2L-$4\times 3\frac{1}{2}\times\frac{5}{16}$; A_g = 4.49 sq in.; r_x = 1.26 in.; r_y = 1.55 in. CHOICE 1

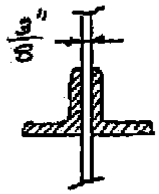
(long legs back – to – back)

USE - 2L-$4\times 3\frac{1}{2}\times\frac{5}{16}$; A_g = 4.49 sq in.; long legs back-to-back

3.6 Select a pair of angles to support a tensile live load (LL) and dead load (DL) for the case assigned by the instructor. Assume the angles are separated back-to-back 3/8-in. by a connected gusset plate, and that the connection is welded. Assume the slenderness ratio is desired to not exceed 300.

Case 7: DL = 80 kips; LL = 100 kips; F_y = 50 ksi; Length L = 28 ft.





Since no holes to be deducted, $A_n = A_g$. Angles are connected along only one leg; U = 0.85 according to AISC- Comm.D3.3. Shear lag applies for both bolted and welded connections.

Factored load: T_u = 1.2D + 1.6L = 1.2(80) + 1.6(100) = 256 kips
or T_u = 1.4(80) = 112 kips

Design strengths: $\phi_t T_n = \phi_t F_y A_g = 0.90(50)A_g = 45.0A_g$ (yielding)

$\phi_t T_n = \phi_t F_u A_g = 0.75(65)(0.8)\ A_g = 39.0A_g$ (fracture)

Fracture on effective net section controls the design!

Required gross area $A_g = \frac{T_u}{41.4} = \frac{256}{41.4} = 6.18$ sq in.

Desired radius of gyration $r = \frac{L}{300} = \frac{28(12)}{300} = 1.12$ in.

From *AISC Manual* tables of double angle sections

2L-4×4×$\frac{1}{2}$; A_g =7.50 sq in.; r_x = 1.22 in.; r_y = 1.83 in. OK

2L-4×3$\frac{1}{2}$; A_g =6.50 sq in.; r_x = 1.25 in.; r_y = 1.33 in. CHOICE 1

(long legs back – to – back)

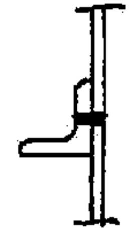
2L-6×3$\frac{1}{2}$×$\frac{8}{8}$; A_g = 6.84 sq in.; r_x = 1.94 in.; r_y = 1.39 in. CHOICE 2

(long legs back – to – back)

USE - 2L-4×3$\frac{1}{2}$; A_g = 6.50 sq in.; long legs back-to-back

3.7 Select a single angle (for the case assigned by the instructor) to support a tensile live load (LL) and dead load (DL). A single gage line of at least three bolts is to be used. Assume shear rupture (i.e., block shear) strength does not control. Assume the slenderness ratio is desired to not exceed 300.

Case 1: DL = 15 kips; LL = 40 kips; A36 steel; Length h = 15 ft; 3/4-in. diam bolts.



This is a long member; thus, AISC-J4 does not apply. Connection is to one leg; thus, take U = 0.85 according to AISC- Comm.D3.3. (at least for preliminary selection).

Factored load: T_u = 1.2D + 1.6L = 1.2(15) + 1.6(40) = 82 kips

or T_u = 1.4(15) = 21 kips

Design strengths: $\phi_t T_n = \phi_t F_y A_g = 0.90(36)A_g = 32.4A_g$ (yielding)

$\phi_t T_n = \phi_t F_u A_g = 0.75(58)(0.85)\ A_n = 37.0A_n$ (fracture)

Required gross area $A_g = \frac{T_u}{32.4} = \frac{82}{32.4} = 2.53$ sq in.

Required net area $A_n = \frac{T_u}{37.0} = \frac{82}{37.0} = 2.22$ sq in.

Desired radius of gyration $r_z = \frac{L}{300} = \frac{15(12)}{300} = 0.60$ in.

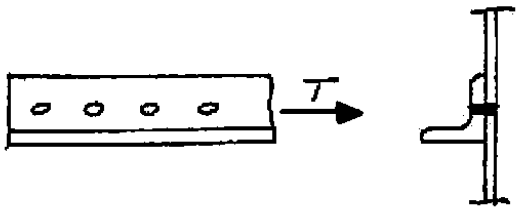
From the *AISC Manual* tables of single angle sections

L - 5×3$\frac{1}{2}$×$\frac{5}{16}$; A_g =2.56 sq in.; r_z =0.766 in OK
A_n =2.56 -0.875(5/16) = 2.29 sq in.
U = 1 - $\bar{x}$ /L = 1-(std gage –y)/L = 1 - (3-1.59)/(est 10) = 0.86
L - 4×3$\frac{1}{2}$×$\frac{3}{8}$; A_g = 2.67 sq in.; r_z = 0.727 in OK
A_n =2.67 -0.875(3/8) = 2.34 sq in.
U = 1 - $\bar{x}$ /L = 1-(std gage –y)/L = 1 - (2.5-1.21)/(est 10) = 0.87

USE - L - 5×3$\frac{1}{2}$×$\frac{5}{16}$; A_g =2.56 sq in.; long leg connected

3.7 Select a single angle (for the case assigned by the instructor) to support a tensile live load (LL) and dead load (DL). A single gage line of at least three bolts is to be used. Assume shear rupture (i.e., block shear) strength does not control. Assume the slenderness ratio is desired to not exceed 300.

Case 2: DL = 15 kips; LL = 40 kips; A572 Grade 50 steel;
Length L = 15 ft: 3/4-in. diam bolts.



This is a long member; thus, AISC-J4 does not apply. Connection is to one leg; thus, take $U = 0.85$ according to AISC- Comm.D3.3. (at least for preliminary selection).

Factored load: $T_u = 1.2D + 1.6L = 1.2(15) + 1.6(40) = 82$ kips
or $T_u = 1.4(15) = 21$ kips

Design strengths: $\phi_t T_n = \phi_t F_y A_g = 0.90(50)A_g = 45.0A_g$ (yielding)

$\phi_t T_n = \phi_t F_u A_g = 0.75(65)(0.8)\ A_n = 39.0A_n$ (fracture)

Required gross area $A_g = \frac{T_u}{45} = \frac{82}{45} = 1.82$ sq in.

Deduct = (Bolt Diam + 0.125)t for standard holes = 0.875t

Required net area $A_n = \frac{T_u}{41.4} = \frac{82}{41.4} = 1.98$ sq in. CONTROLS!

Desired radius of gyration $r_z = \frac{L}{300} = \frac{15(12)}{300} = 0.60$ in.

From the *AISC Manual* tables of single angle sections

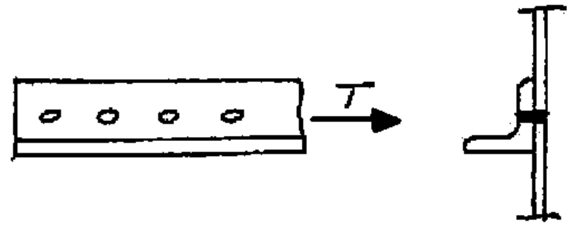
L - $4\times3\frac{1}{2}\times\frac{5}{16}$; $A_g = 2.25$ sq in.; $r_z = 0.730$ in OK
$A_n = 2.25 - 0.875(5/16) = 1.98$ sq in.
$U = 1 - \bar{x}/L = 1 -$(std gage $-y$)$/L = 1 - (2.5 - 1.18)/(\text{est } 10) = 0.87$
L - $5\times3\times\frac{5}{16}$; $A_g = 2.40$ sq in.; $r_z = 0.658$ in OK
$A_n = 2.40 - 0.875(5/16) = 2.13$ sq in.
$U = 1 - \bar{x}/L = 1 -$(std gage $-y$)$/L = 1 - (3 - 1.68)/(\text{est } 10) = 0.87$
L - $4\times4\times\frac{5}{16}$; $A_g = 2.40$ sq in.; $r_z = 0.791$ in OK

USE - L - $4\times3\frac{1}{2}\times\frac{5}{16}$; $A_g = 2.25$ sq in.; long leg connected

3.7 Select a single angle (for the case assigned by the instructor) to support a tensile live load (LL) and dead load (DL). A single gage line of at least three bolts is to be used. Assume shear rupture (i.e., block shear) strength does not control. Assume the slenderness ratio is desired to not exceed 300.

Case 3: DL = 15 kips; LL = 40 kips; A572 Grade 60 steel;

Length $L = 25$ ft; 7/8-in. diam bolts.



This is a long member; thus, AISC-J4 does not apply. Connection is to one leg; thus, take $U = 0.85$ according to AISC- Comm.D3.3. (at least for preliminary selection).

Factored load: $T_u = 1.2D + 1.6L = 1.2(15) + 1.6(40) = 82$ kips
or $T_u = 1.4(15) = 21$ kips

Design strengths: $\phi_t T_n = \phi_t F_y A_g = 0.90(60)A_g = 54.0A_g$ (yielding)

$\phi_t T_n = \phi_t F_u A_g = 0.75(75)(0.85)\ A_n = 47.8A_n$ (fracture)

Required gross area $A_g = \frac{T_u}{54} = \frac{82}{54} = 1.52$ sq in.

Deduct = (Bolt Diam + 0.125)t for standard holes = 1.0t

Required net area $A_n = \frac{T_u}{47.8} = \frac{82}{47.8} = 1.72$ sq in. CONTROLS!

Desired radius of gyration $r_z = \frac{L}{300} = \frac{25(12)}{300} = 1.0$ in.

From the *AISC Manual* tables of single angle sections

L - $5\times5\times\frac{5}{16}$; $A_g = 3.03$ sq in.; $r_z = 0.994$ in CLOSE!
$A_n = 3.03 - 1.0(5/16) = 2.72$ sq in.
$U = 1 - \bar{x}/L = 1 -$(std gage $-y$)$/L = 1 - (3 - 1.37)/(\text{est } 10) = 0.84$

SLENDERNESS CONTROLS!

USE - L - $5\times5\times\frac{5}{16}$; $A_g = 3.03$ sq in.

3.8 Select a standard threaded rod to carry a tensile force T of 4 kips dead load and 6 kips live load. Use A572 Grade 50 steel.

DL = 4 kips; LL = 6 kips; A572 Grade 50 steel;

The design strength of a threaded rod is given by AISC – J3.6 (Table J3.2)

Factored load: T_u = 1.2D + 1.6L = 1.2(4) + 1.6(6) = 14.4 kips
or T_u = 1.4(4) = 5.6 kips

Design strengths: $\phi_t T_n = 0.75 A_b F_n = 0.75 A_b (0.75 F_u)$

Using the minimum tensile strength F_u for A572 Grade 50 steel as 65 ksi from AISC Table 2-5, and equating the factored load T_u to the design strength $\phi_t T_m$, gives the required gross A_b area from Eq. As

$$\text{Required } A_b = \frac{Required\ \phi_t T_n}{0.75(0.75F_u)} = \frac{14.4}{0.75(0.75)(65)} = 0.39 \text{ sq in}$$

Select a standard threaded rod based on the required area A_b. The area computed is the gross area A_b based on the diameter of the unthreaded body of the rod (AISC – Table 7-18).

USE - $\frac{3}{4}$- in. – diam rod (10 threads per inch) (A_b = 0.442 sq in.)

3.9 Select a standard threaded rod to carry a tensile force T of 2 kips dead load and 4 kips live load. Use A36 steel.

DL = 2 kips; LL = 4 kips; A36 steel;

The design strength of a threaded rod is given by AISC – J3.6 (Table J3.2)

Factored load: T_u = 1.2D + 1.6L = 1.2(2) + 1.6(4) = 8.8 kips or
T_u = 1.4(2) = 2.8 kips

Design strengths: $\phi_t T_n = 0.75 A_b F_n = 0.75 A_b (0.75 F_u)$

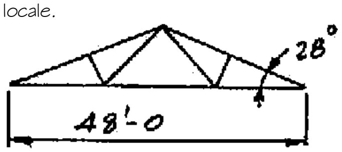
Using the minimum tensile strength F_u for A36 steel as 58 ksi from AISC Table 2-5, and equating the factored load T_u to the design strength $\phi_t T_m$, gives the required gross A_b area from Eq. As

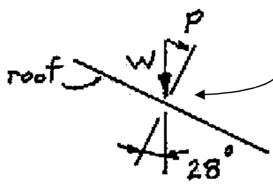
$$\text{Required } A_b = \frac{Required\ \phi_t T_n}{0.75(0.75F_u)} = \frac{8.8}{0.75(0.75)(58)} = 0.27 \text{ sq in}$$

Select a standard threaded rod based on the required area A_b. The area computed is the gross area A_b based on the diameter of the unthreaded body of the rod (AISC – Table 7-18).

USE - $\frac{5}{8}$- in. – diam rod (11 threads per inch) (A_b = 0.307 sq in.)

3.10 Design sag rods to support the purlins of an industrial building roof whose span and slope are shown in the accompanying figure. Sag rods are placed at 1/3 points between roof misses, which are spaced 30 ft apart. Assume roofing and purlin weight is 9 psf of roof surface. Use standard threaded rods and A36 steel. The snow load to be carried is 20, 30, or 40 psf of horizontal projection, whichever is appropriate for your locale.





Purlins support load component transverse to roof

Sag rods support component P parallel to roof

Load combination = 1.2D + 1.6 SNOW

Factored load:

Snow intensity =	2 0	3 0	4 0 psf
DL per sq ft of roof =	10.8	10.8	10.8
SNOW = $w\cos$ (angle) =	<u>28.25</u>	<u>42.38</u>	<u>56.51</u>
	39.05	53.18	67.31 psf

Factored force on sag rods:

Roof area = [24/(cos 28°)](spacing) = (24/0.8830)30 = 815.5 sq ft

Force on rods is component parallel to roof.

$T_u = w_u$ (sin28°)(roofarea)=	15.0	20.4	25.8 kips

With sag rods at 1/3 points, assume each carries P/3.

T_u =	4.98	6.79	8.59 kips

The strength requirement of AISC-J3.6 is

$$\phi_t T_n \geq T_u$$

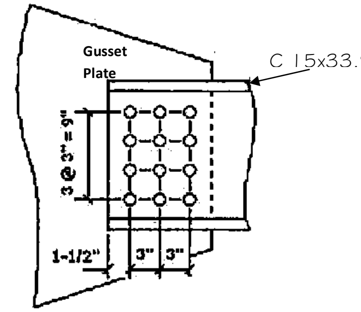
$$\phi_t T_n = \phi_t(0.75F_u)A_b = 0.75(0.75)(58)A_b = 32.6A_b$$

Required area $A_b = \frac{T_u}{32.6}$	0.153	0.208	0.263 sq in.

<u>SELECT RODS</u>: (from *AISC Manual* Table 7-18, p. 7-83)

Diameter	1/2	5 /8	5/8 in.
Actual Area A_b =	0.196	0.307	0.307 sq in.

3.11 Determine the maximum allowable tensile load (20% dead load, 80% live load) for a single C15x33.9 fastened to a 1/2-in. gusset plate as in the accompanying figure. Use A36 steel and assume holes are-for 3/4-in. diam bolts.



Properties: Ag = 9.96 sq in.

t_w =0-40 in.

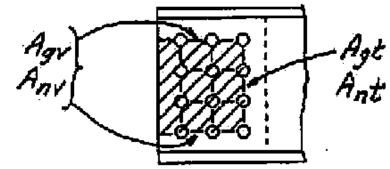
Since there are three bolts

per line, take U = 0.85 according to AISC- Comm.D3.3, par (b).

Alternatively, compute

$U = 1 - \bar{x}/L$

= 1 -0.787/ 6 = 0.87< 0.90 max as per AISC-D3.3.



Design strengths:

$\phi_t T_n = \phi_t F_y A_g$ = 0.90(36)9.96 =323 kips (yielding)

$A_n = A_g$ - 4holes = 9.96 - 4(d_b + 1 / 8)t_w

= 9.96-4(0.875)0.40 = 8.56 sq in.

$\phi_t T_n = \phi_t F_u A_g$ = 0.75(58)(0.87)8.56 = 324 kips (fracture)

Block shear strength:

$A_{gt} = 9t_w$ = 9(0.40) = 3.60 sq in.

A_{nt} = (9-3d_h)t_w =[9-3(0.875)]0.40 = 2.55 sq in.

A_{gv} = 2(7.5)t_w = 2(7.5)0.40 = 6.00 sq in.

A_{nv} = (15-5d_h)t_w = [15-5(0.875)]0.40 = 4.25 sq in.

Check:

[$F_u A_{nt}$ = 58(2.55) = 148 kips] ≈ [0.6$F_u A_{nv}$ = 0-6(58)4.25 = 148 kips]

Thus, shear fracture—tension yielding controls!

$T_n = 0.6F_u A_{nv} + F_y A_{gt}$ = 0.6(58)4.25 + 36(3.60) = 278 kips

$\phi_t T_n$ = 0.75(278) = 208 kips **CONTROLS!**

T_u = 1.2DL + 1.6LL = $\phi_t T_n$ = 208 kips

T_u = 1.2(0.2r) + 1.6(0.8T) = 208 kips

T = 137 kips; T_{DL} = 27.4 kips; T_{LL} = 109 kips

Allowable service load, T = 137 kips!

3.12 Determine the maximum allowable tensile load (20% dead load, 80% live load) for a single C10x25 fastened to a 5/8-in. gusset plate as in die accompanying figure. Use A36 steel and assume holes are for 3/4-in. diam bolts. Base answer on tension strength of the channel and include shear rupture (block shear) strength.

Properties: A_g = 7.35 sq in.

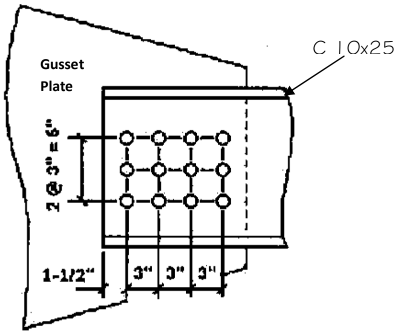
t_w =0.526 in.

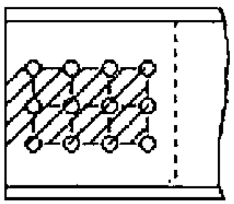
Since there are three bolts per line, U = 0.85 as per AISC - Comm.D3.3, par (b).

Alternatively, compute

$U = 1 - \bar{x}/L$

= 1 -0.617/9 = 0.93 > 0.90 max as per AISC – D3.3.





Design strengths:

$\phi_t T_n = \phi_t F_y A_g$ = 0.90(36)7.35 = 238 kips (yielding)

$A_n = A_g$ - 3holes = 9.96 - $3(d_b + 1/8)t_w$

= 9.96-3(0.875)0.526 = 5.97 sq in.

$\phi_t T_n = \phi_t F_u A_g$ = 0.75(58)(0.90)5.97 = 234 kips (fracture)

Block shear strength: Channel web is thinner than gusset plate

$A_{gt} = 6t_w$ = 6(0.526) = 3.16 sq in.

$A_{nt} = (6-2d_h)t_w$ =[6-2(0.875)]0.526 = 2.24 sq in.

$A_{gv} = 2(10.5)t_w$ = 2(10.5)0.526 = 11.05 sq in.

$A_{nv} = (21-7d_h)t_w$ = [21-7(0.875)]0.526 = 7.82 sq in.

Check:

[$F_u A_{nt}$ = 58(2.24) = 130 kips] ≈ [$0.6F_u A_{nv}$ = 0-6(58)7.82 = 272 kips]

Thus, shear fracture—tension yielding controls!

$T_n = 0.6F_u A_{nv} + F_y A_{gt}$ = 0.6(58)7.82 + 36(3.16) = 386 kips

$\phi_t T_n$ = 0.75(386) = 289 kips; Thus, member fracture controls!

T_u = 1.2DL + 1.6LL = $\phi_t T_n$ = 234 kips

T_u = 1.2(0.2r) + 1.6(0.8T) = 234 kips

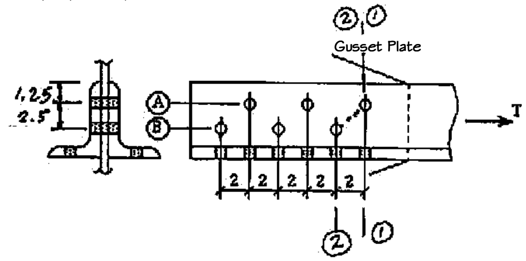
T = 154 kips; T_{DL} = 31 kips; T_{LL} = 123 kips

Allowable service load, T = 154 kips!

3.13 Determine the maximum allowable tensile load (15% dead load, 85% live load) for a pair of angles L6x4x3/8 attached to a gusset plate as shown. Use A36 steel and 3/4-in. diam bolts on standard gage lines. The force is transmitted to the gusset plate by the fasteners on lines A and B. The holes in the outstanding legs are open.

Properties: A_g = 7.22 sq in.

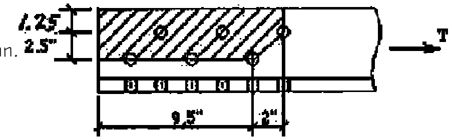
Since there are three bolts per line, U = 0.85 as per AISC - Comm.D3.3, par (b).



Alternatively, compute

$U = 1 - \bar{x}/L$

= 1-0.941/10 = 0.91 > 0.90 max as per AISC – D3.3.



Compute net area: t = 2(0.375) = 0.75 in.

Section 1-1 (two holes out):

$A_n = A_g - 2holes$

= 7.22 - 2(0.75 + 0.125)0.75 = 5.91 sq in.

Section 2-2 (3 holes staggered):

$A_n = A_e - 3holes + (s^2/4g)t$

= 7.22-3(0.75 + 0.125)0.75-r[2^2/4(2.5)]0.75 = 5.55 sq in.

Design strengths:

$\phi_t T_n = \phi_t F_y A_g$ = 0.90(36)7.22 =234 kips (yielding)

$\phi_t T_n = \phi_t F_u A_g$ = 0.75(58)(0.90)5.55 = 217 kips (fracture)

Block shear strength: Include diagonal in shear path.

A_{gt} = 1.25 t_w =1.25(0.75) = 0.94 sq in.

$A_{nt} = (3.15-1d_h)t_w$ = [3.75-1(0.875)]0.75 = 2.16 sq in.

A_{gv} = [9.5 +$\sqrt{2^2 + 2.5^2}$] t_w = [9.5 + 3.20]0.75 = 9.53 sq in.

A_{nv} = Ag, -3.5$d_h t_w$ =9.53-3.5(0.875)0.75 = 7.23 sqin.

Check: [$F_u A_{nt}$ = 58(2.16) = 125 kips]< [$0.6F_u A_{nv}$ = 0.6(58)7.23 = 252 kips]

Thus, shear fracture—tension yielding controls!

$T_n = 0.6F_u A_{nv} + F_y A_{gt}$ = 0.6(58)7.23 + 36(0.94) = 285 kips

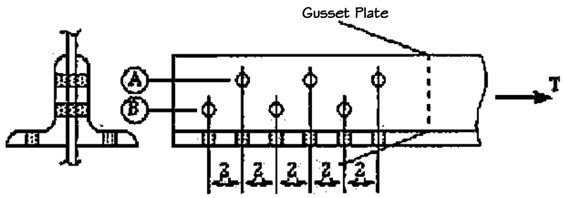
$\phi_t T_n$ = 0.75(285) = 214 kips; Thus, block shear controls!

T_u = 1.2(0.15^ + 1.6(0.852") = 214 kips

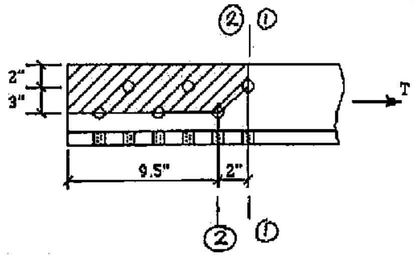
T = 139 kips; T_{DL} = 21 kips; T_{LL} = 118 kips

Allowable service load, T = 139 kips!

3.14 Determine the maximum allowable tensile load (15% dead load, 85% live load) for a pair of angles L8x6x3/4 attached to a gusset plate as shown. Use A36 steel and 7/8-in. diam bolts on standard gage lines. The force is transmitted to the gusset plate by the fasteners on lines A and B. The holes in the outstanding legs are open.



Properties: A_g = 19.90 sq in.
Since there are three bolts per line, U = 0.85 as per AISC - Comm.D3.3, par (b).
Alternatively, compute
$U = 1 - \bar{x}/L$
= 1-1.56/10 = 0.84< 0.90 max
as per AISC – D3.3.



Compute net area: t = 2(0.75) = 1.50 in.
Section 1-1 (two holes out):
$A_n = A_g$ - 2*holes*
= 19.90-2(0.875 + 0.125)1.50
= 16.90 sq in.
Section 2-2 (3 holes staggered):
$A_n = A_g$ - 3*holes* + $(s^2/4g)t$
= 19.90-3(0.875 + 0.125)1.50 + [2^2/4(3)]1.50 = 15.9 sq in.

Design strengths:
$\phi_t T_n = \phi_t F_y A_g$ = 0.90(36)19.9 = 645 kips (yielding)
$\phi_t T_n = \phi_t F_u A_g$ = 0.75(58)(0.84)15.9 = 581 kips (fracture)
Block shear strength: Include diagonal in shear path.

A_{gt} = 2.0t_w =2.0(1.50) = 3.00 sq in.
A_{nt} = (5.00-1 d_h)t_w =[5.00-l(1.00)]1.50 = 6.00 sq in
A_{gv} = [9.5 +$\sqrt{2^2+3^2}$] t_w = [9.5 + 3.61]1.50 = 19.7 sq in.
$A_{nv} = A_{gv}$ -3.5$d_h t_w$ =19.7-3.5(1.00)1.50 = 14.5 sq in.

Check: [$F_u A_{nt}$ =58(6.00) = 348 kips]<[$0.6F_u A_{nv}$ =0.6(58)14.5 = 505 kips]

Thus, shear fracture—tension yielding controls!

$T_n = 0.6F_u A_{nv} + F_y A_{gt}$ = 0.6(58)14.5 + 36(3.00) = 613 kips

$\phi_t T_n$ = 0.75(613) = 459 kips; Thus, block shear controls!

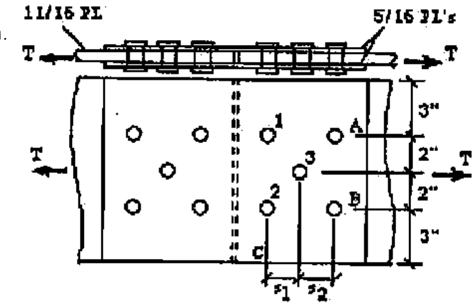
T_u = 1.2(0.157*0* + 1.6(0.857-) = 459 kips

T = 298 kips; T_{DL} = 45 kips; T_{LL} = 253 kips

Allowable service load, T = 298 kips!

3.15. Given the splice shown in the accompanying figure: (a) Determine the maximum capacity T (25% dead load, 75% live load) based on the A346 steel plates having holes arranged as shown, (b) What value of s should be specified to provide the maximum capacity T as computed in pan (a), if the final design is to have $s_1 = s_2 = s$?

The 5/16-in. plates are governed by AISC-J4; $A_n < 0.85A_g$.



A_g = 2(10)0.3125 = 6.25 sq in.;
Max A_n = 0.85(6.25) = 5.31 sq in.
Compute net area:
t = 2(0.3125) = 0.625 in.
Section 1-1 (two holes out):
$A_n = A_g$ - 2*holes*
= 6.25-2(0.875 + 0.125)0.625
= 5.00 sq in.

Design strengths:
$\phi_t T_n = \phi_t F_y A_g$ = 0.90(36)6.25 =203 kips (yielding) CONTROLS!
$\phi_t T_n = \phi_t F_u A_g$ = 0.75(58)(1.0)5.00 = 218 kips (fracture)
Section 1-1 (3 holes staggered):
$A_n = A_g$- 3*holes* + $\sum(s^2 / 4g)t$
= 6.25-3(0.875 +0.125)0.625+2[s^2 /4(2)]0.625
=[203/(0.75)58=4.66 sq in.
Solve for s = 1.34 in.

Compute block shear through A-1-2-B. The tensile area is known, but the shear area is a function of s.

A_{gt} = 4.0t = 4.0(0.625) = 2.50 sq in.
$A_{nt} = A_{gt}$ –1 $d_h t$ = 2.50 -1.0(0.625) = 1.88 sq in.
A_{gv} = 2(2s +1.5)t = 2(2s +1.5)0.625 = 2.5s+ 1.88
$A_{nv} = A_{gv}$ -3$d_h t_w$ =2.5s + 1.88-3(1.00)0.625 = 2.5s

Check: [F_u A_{nt} = 58(1.88) = 145 kips]<[$0.6F_u A_{nv}$ = 0.6(58)2.5s =113 kips]

Thus, shear yield - tension fracture controls!

$\phi_t T_n = \phi_t T_n$ [$0.6F_y$ A_{gv} +$F_u A_{nt}$] = 0.75[0.6(36)(2.5s + 1.88) + 58(1.88)]
= 0.75[54s +149.6] = 203 kips
Solve for s = 2.24 in. Check strength when s = 2.25 in.
$\phi_t T_n$ = ϕ_t [$0.6F_y$ A_{nv} +$F_y A_{gt}$] = 0.75[0.6(58)2.5s +36(2.50)]
= 0.75[87(2.25) + 90] = 214 kips

Member strength, $\phi_t T_n$ = 203 kips; *USE s* = 2.25 in.

3.16. An L5x3-1/2x1/2 angle, as shown in the accompanying figure, is to carry 20 kips dead load and 70 kips live load using the shortest length of connection using two gage lines of bolts in the 5-hr. leg. What is the minimum acceptable stagger, theoretical and specified (1/2-in. multiples), using A572 Grade 50 steel? Neglect shear rupture strength.

A_g = 4.00 sq in.

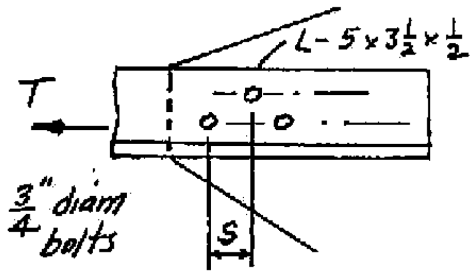
Since there are three bolts per line, U = 0.85 as per AISC - Comm.D3.3, par (b).

Alternatively, compute as per AISC – D3.3.

$U = 1 - \bar{x}/L$

= 1-0.906/45

For s = 1.5, U = 0.85



Section with one hole out:

$A_n = A_g$ -1 *holes* = 4.00-1(0.75 + 0.125)0.50 = 3.56 sq in.

Factored load: T_u = 1.2D + 1.6L = 1.2(20) + L6(70) = 136 kips

or T_u = 1.4(20) = 28 kips

Maximum design strengths:

$\phi_t T_n = \phi_t F_y A_g$ = 0.90(50)4.00 = 180 kips (yielding)

$\phi_t T_n = \phi_t F_u A_e = \phi_t F_u U A_n$ = 0.75(65)(0.85)3.56 = 148 kips (fracture)

Section staggered through two holes:

$A_n = A_g$ - 2*holes* + $(s^2/4g)t$

= 4.00-2(0.75 +0.125)0.50+ [s^2 /4(1.75)/0.50

= 3.13 +0.0714s^2

Required net area A_n =136/[0.75(0.85)65] = 3.28 sq in.

A_n = 3.13 +0.0714s^2 = 3.28 sq in.

s =1.48; say s = 1.5 in.

Check efficiency factor $U = 1 - \bar{x}/L$ = 1-0.906/[4(1.5)] = 0.85

Check stagger for minimum spacing of 2.67 diameters.

[2.67(0.75)]^2 = s^2+1.75^2

s = 0.97 in.

Member strength, $\phi_t T_n$ = 136 kips; *USE* s = 1.5 in.

3.17. An L5x3-1/2x1/2 angle, as shown in the accompanying figure, is to carry 20 kips dead load and 60 kips live load. Using one gage line of holes for 7/8-in. diam bolts in each leg, what is the minimum stagger s, to accomplish this? Consider the load to be transferred by bolts in the 5 in, leg while the holes in the 3-1/2 in. leg are to be considered open. Use A36 steel. Neglect shear rupture strength.

A_s = 4.00 sq in.

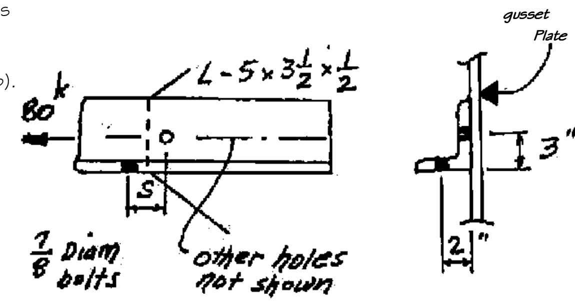
Since there are three bolts per line, U = 0.85 as per AISC - Comm.D3.3, par (b).

Alternatively, compute as per AISC – D3.3.

$U = 1 - \bar{x}/L$

= 1-0.906/4s

For s = 1.5, U = 0.85



Section with one hole out:

A_n=A_g -1 *holes* = 4.00-1(0.875 + 0.125)0.50 = 3.50 sq in.

Factored load: T_u=1.2D + 1.6L = 1.2(20) + 1.6(60) = 120 kips

or T_u = 1.4(20) = 28 kips

Maximum design strengths:

$\phi_t T_n = \phi_t F_y A_g$ = 0.90(36)4.00 = 130 kips (yielding)

$\phi_t T_n = \phi_t F_u A_e = \phi_t F_u U A_n$ =0.75(58)(0.85)3.50 = 129 kips (fracture)

Section staggered through two holes:

Gage g =3 + 2 -t =3 + 2 -0.5 = 4.5 in.

$A_n = A_g$ - 2*holes* + $(s^2/A_g)t$

= 4.00 - 2(0.875+ 0.125)0.50 + [s^2 /4(4.5)]0.50

= 3.00 + 0.0278s^2

Required net area A_n =120/[0.75(0.85)58] = 3.25 sq in.

A_n = 3.00 +0.0278s^2 =3.25 sqin.

s= 3.00 in.

Check efficiency factor

$U = 1 - \bar{x}/L$ = 1-0.906/[4(3)] = 0.92 > 0.90 max

Check stagger for minimum spacing of 2.67 diameters.

[2.67(0.875)]^2 = s^2+4.5^2

Min s = *none*

Member strength, $\phi_t T_n$ = 120 kips; *USE* s = 3 in.

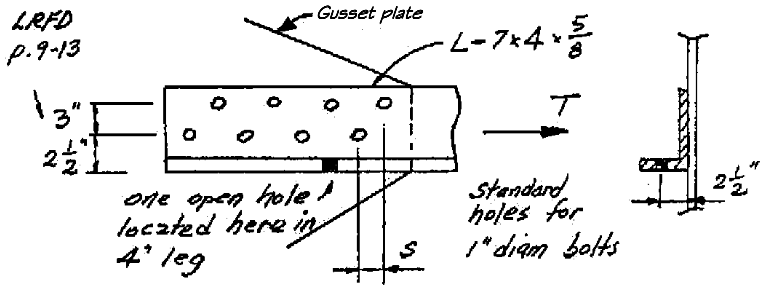
3.18. Compute the minimum value of s that could be used on the angle shown in (he figure, such that the maximum factored tensile force T_u may be carried. Assume m is large enough so that a failure pattern through the open hole will not govern. Include consideration of shear rupture strength. Use A36 steel with 1-in. diam bolts.

A_g = 6.48 sq in.

Since there are at least three bolts per line, U = 0.85 as per AISC Commentary - D3.3, par (b).

Alternatively, compute as per AISC - D3.3 $U = 1 - \bar{x}/L = 1 - 0.963/L$

For L = 6.4 in., U = 0.85



Section with one hole out:

$A_n = A_g - 1\,holes$ = 6.48-1(1.00 + 0.125)0.625 = 5.78 sq in.

Maximum design strengths:

$\phi_t T_n = \phi_t F_y A_g$ = 0.90(36)6.48 = 210 kips (yielding)

$\phi_t T_n = \phi_t F_u A_e = \phi_t F_u U A_n$ = 0.75(58)(0.85)5.78 = 214 kips (fracture)

Section staggered through two holes:

Gage g_1 = 2.5 in.; g_2=3 in.; g = 2.5 in., *AISC Manual*, p. 10-10.

$A_n = A_g - 2holes + (s^2/4g)t$

$= 6.48 - 2(1.00 + 0.125)0.625 + [s^2/4(3.0)]0.625 = 5.07 + 0.0521s^2$

Required net area A_n = 210/[0.75(0.85)58] = 5.68 sq in.

$A_n = 5.07 + 0.0521s^2 = 5.68$ sq in.; s = 3.42 in.

Check $U = 1 - \bar{x}/L = 1 - 0.963/[>4(3.42)]$ =>0.90 max; use 0.90

Revised Reqd A_n =210/[0.75(58)0.90] = 5.36 sq in.; Max s = 2.36 in.

Check rupture shear strength: Include diagonal in shear path.

$A_{gt} = (7 - g_2 - g_1)t$ = (7-3-2.5)0.625 = 0.94 sq in.

$A_{nt} = A_{gt} - 0.5d_h t$ = 0.94-0.5(1.00 + 0.125)0.625 = 0.59 sq in.

$A_{gv} = [1.5 + 6s + \sqrt{s^2 + 3^2}]t = [1.5 + 6(2.36) + \sqrt{2.36^2 + 3^2}]0.625$ = 12.17 sq in

$A_{nv} = A_{gv} - 3.5d_h t$ = 12.17-3.5(1.125)0.625 = 9.71 sq in.

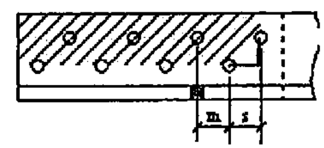
Check: $[F_u A_{nt}$ = 58(0.59) = 34 kips] < $[0.6F_u A_{nv}$ = 0.6(58)9.71 = 338 kips]

Thus, shear fracture - tension yielding controls!

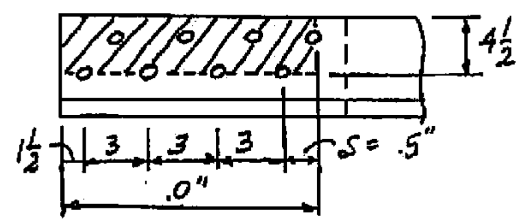
$T_n = 0.6F_u A_{nv} + F_y A_{gt}$ = 0.6(58)9.71 + 36(0.94) = 372 kips

$\phi_t T_n$ = 0.75(372) = 279 kips; Shear rupture strength does NOT control!

Member strength, $\phi_t T_n$ = 210 kips; *Minimum s* = 2.36 in.



Practical solution:



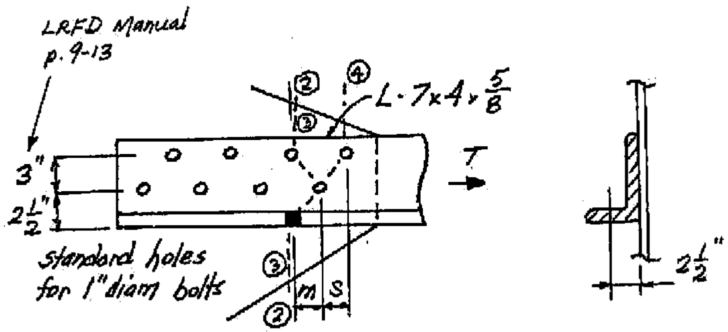
3.19. Assuming that s for the angle of Prob. 3.18 is made as large as required in Prob. 3.18, compute the minimum distance m required so that the open hole in the 4-in. leg will not reduce the strength below its maximum possible value. If Prob. 3.18 is not solved, assume s is 3.75 in. Use A36 steel with 1-in. diam bolts.

A_g = 6.48 sq in.

Since there are at least three holts per line, U = 0.85 as per AISC

Commentary - D3.3, par (b). U = 0.90 according to Prob. 3.18.

Max $\phi_t T_n$ = 210 kips (Prob. 3.18); Minimum s = 2.36 in.; A_n = 5.36 sq in.



Failure through the open hole is computed using section 2-2 with 6/8 of the load acting (the open hole is not involved in load transmission). If m is small, failure may be through three holes with 100% of load.

Section 2-2 with 6/8 of load acting:

$A_n = A_g - 2holes$ = 6.48 - 2(1.00 + 0.125)0.625 = 5.07 sq in.

6 / 8 of required A_n = (6 / 8)5.36 = 4.02 sq in. (does not control)

Section 3-3 with 7/8 of load acting:

Gagec g (short leg) = 2.5 in.; g (around corner) = 2.5 + 2.5-0.625 = 4.375 in.

$A_n = A_g\text{-}3holes + \sum(s^2 / 4g)t$

$= 6.48 - 3(1.00 + 0.125)0.625 + [m^2/ 4(3) + m^2 / 4(4.375)]0.625$

$= 4.37 + m^2/11.39$

7/8 of required A_n = (7/8)5.36 = 4.69 sq in.

$4.69 = 4.37 + m^2 /11.39$; Required m = 1.91 in.

Section 4-3 with 100% of load acting:

Assume s = 2.36 in. (Minimum indicated from Prob. 3.18)

$A_n = A_g - 3holes + \sum(s^2 / 4g)t$

$= 6.48\text{-}3(1.00 + 0.125)0.625 + [s^2/4(3) + m^2 /4(4.375)]0.625$

$= 4.37 + [2.36^2/4(3) + m^2/4(4.375)]0.625 = 4.66 + m^2/28$

$5.36 = 4.66 + m^2 / 28$; Required m = 4.43 in,

USE $s \geq$ 2.36 in. and $m \geq$ 4.43 in. for full strength of angle.

For practicality,

<u>*USE* s = 2.5 in. and m = 4.5 in. for full strength of angle.</u>

If s = 3.75 in. as given if Prob. 3.18 was not solved,

$A_n = A_g - 3holes + \sum(s^2 / 4g)t$

$= 6.48\text{-}3(1.00 + 0.125)0.625 + [s^2/4(3) + m^2/4(4.375)]0.625$

$= 4.37 + [3.75^2 / 4(3) + m^2 /4(4.375)]0.625 = 5.10 + m^2 / 28$

$5.36 = 5.10 + m^2 / 28$; Required m = 2.70 in.

USE s = 3.75 in. and m = 2.70 in. for full strength of angle.

3.20. Design an eyebar to carry 24 kips dead load and 76 kips live load, using flame – cut A572 Grade 50 steel plate. (Refer to AISC – D.D6)

DL = 24 kips; LL = 76 kips; A572 Grade 50 steel, AISC;

Yielding in gross section:

$\phi_t T_n = 0.9 A_g F_y = 0.9 A_g (50) = 45 A_g$ CONTROLS!

Fracture in effective net section:

$\phi_t T_n = 0.75 A_e F_u = 0.75 A_e (65) = 48.75 A_e$

Factored load: $T_u = 1.2D + 1.6L = 1.2(24) + 1.6(76) = 150.4$ kips

or $T_u = 1.4(24) = 33.6$ kips

Yielding on gross section controls the design! AISC – D6

Required gross area $A_g = \frac{T_u}{45} = \frac{150.4}{45} = 3.34$ sq in

Select a standard threaded rod based on the required area A_b. The area computed is the gross area A_b based on the diameter of the unthreaded body of the rod (AISC – Table 7-18).

USE - $2\frac{1}{4}$- in. – diam rod (4.5 threads per inch) (A_b = 3.98 sq in.)