Chapter D
Design of Members for Tension

D1. SLENDERNESS LIMITATIONS

Section D1 does not establish a slenderness limit for tension members, but recommends limiting L/r to a maximum of 300. This is not an absolute requirement. Rods and hangers are specifically excluded from this recommendation.

D2. TENSILE STRENGTH

Both tensile yielding strength and tensile rupture strength must be considered for the design of tension members. It is not unusual for tensile rupture strength to govern the design of a tension member, particularly for small members with holes or heavier sections with multiple rows of holes.

For preliminary design, tables are provided in Part 5 of the AISC *Manual* for W-shapes, L-shapes, WT-shapes, rectangular HSS, square HSS, round HSS, Pipe and 2L-shapes. The calculations in these tables for available tensile rupture strength assume an effective area, A_e, of $0.75A_g$. If the actual effective area is greater than $0.75A_g$, the tabulated values will be conservative and calculations can be performed to obtain higher available strengths. If the actual effective area is less than $0.75A_g$, the tabulated values will be unconservative and calculations are necessary to determine the available strength.

D3. EFFECTIVE NET AREA

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

In computing net area, A_n, AISC *Specification* Section B4.3 requires that an extra $\frac{1}{16}$ in. be added to the bolt hole diameter.

A computation of the effective area for a chain of holes is presented in Example D.9.

Unless all elements of the cross section are connected, $A_e = A_nU$, where U is a reduction factor to account for shear lag. The appropriate values of U can be obtained from Table D3.1 of the AISC *Specification*.

D4. BUILT-UP MEMBERS

The limitations for connections of built-up members are discussed in Section D4 of the AISC *Specification*.

D5. PIN-CONNECTED MEMBERS

An example of a pin-connected member is given in Example D.7.

D6. EYEBARS

An example of an eyebar is given in Example D.8. The strength of an eyebar meeting the dimensional requirements of AISC *Specification* Section D6 is governed by tensile yielding of the body.

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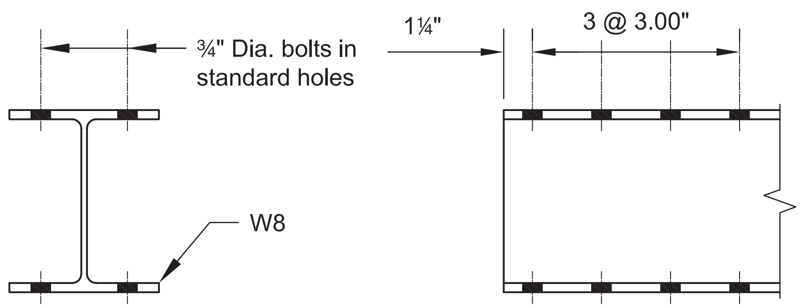
EXAMPLE D.1 W-SHAPE TENSION MEMBER

Given:

Select an 8-in. W-shape, ASTM A992, to carry a dead load of 30 kips and a live load of 90 kips in tension. The member is 25 ft long. Verify the member strength by both LRFD and ASD with the bolted end connection shown. Verify that the member satisfies the recommended slenderness limit. Assume that connection limit states do not govern.



Solution:

From Chapter 2 of ASCE/SEI 7, the required tensile strength is:

LRFD	ASD
P_u = 1.2(30 kips) + 1.6(90 kips) = 180 kips	P_a = 30 kips + 90 kips = 120 kips

From AISC *Manual* Table 5-1, try a W8×21.

From AISC *Manual* Table 2-4, the material properties are as follows:

W8×21
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From AISC *Manual* Tables 1-1 and 1-8, the geometric properties are as follows:

W8×21
A_g = 6.16 in.^{2}
b_f = 5.27 in.
t_f = 0.400 in.
d = 8.28 in.
r_y = 1.26 in.

WT4×10.5
$\bar{y}$ = 0.831 in.

Tensile Yielding

From AISC *Manual* Table 5-1, the tensile yielding strength is:

LRFD	ASD
277 kips > 180 kips **o.k.**	184 kips > 120 kips **o.k.**

Tensile Rupture

Verify the table assumption that $A_e/A_g \geq 0.75$ for this connection.

Calculate the shear lag factor, U, as the larger of the values from AISC *Specification* Section D3, Table D3.1 case 2 and case 7.

From AISC *Specification* Section D3, for open cross sections, U need not be less than the ratio of the gross area of the connected element(s) to the member gross area.

$$
\begin{aligned}
U &= \frac{2b_f t_f}{A_g} \\
&= \frac{2(5.27\text{ in.})(0.400\text{ in.})}{6.16\text{ in.}^2} \\
&= 0.684
\end{aligned}
$$

Case 2: Check as two WT-shapes per AISC *Specification* Commentary Figure C-D3.1, with $\bar{x} = \bar{y} = 0.831$ in.

$$
\begin{aligned}
U &= 1 - \frac{\bar{x}}{l} \\
&= 1 - \frac{0.831\text{ in.}}{9.00\text{ in.}} \\
&= 0.908
\end{aligned}
$$

Case 7:

$b_f = 5.27$ in.
$d = 8.28$ in.
$b_f < \tfrac{2}{3}d$
$U = 0.85$

Use $U = 0.908$.

Calculate A_n using AISC *Specification* Section B4.3.

$$
\begin{aligned}
A_n &= A_g - 4(d_h + \tfrac{1}{16}\text{ in.})t_f \\
&= 6.16\text{ in.}^2 - 4(\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})(0.400\text{ in.}) \\
&= 4.76\text{ in.}^2
\end{aligned}
$$

Calculate A_e using AISC *Specification* Section D3.

$$
\begin{aligned}
A_e &= A_n U \\
&= 4.76\text{ in.}^2(0.908) \\
&= 4.32\text{ in.}^2
\end{aligned}
$$
(*Spec*. Eq. D3-1)

$$
\begin{aligned}
\frac{A_e}{A_g} &= \frac{4.32\text{ in.}^2}{6.16\text{ in.}^2} \\
&= 0.701 < 0.75\text{; therefore, table values for rupture are not valid.}
\end{aligned}
$$

The available tensile rupture strength is,

$$P_n = F_u A_e$$ (*Spec*. Eq. D2-2)
$$= 65 \text{ ksi}(4.32 \text{ in.}^2)$$
$$= 281 \text{ kips}$$

From AISC *Specification* Section D2, the available tensile rupture strength is:

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(281 \text{ kips})$ $= 211 \text{ kips}$ 211 kips > 180 kips **o.k.**	$\Omega_t = 2.00$ $\dfrac{P_n}{\Omega_t} = \dfrac{281 \text{ kips}}{2.00}$ $= 141 \text{ kips}$ 141 kips > 120 kips **o.k.**

Check Recommended Slenderness Limit

$$\frac{L}{r} = \left(\frac{25.0 \text{ ft}}{1.26 \text{ in.}}\right)\left(\frac{12.0 \text{ in.}}{\text{ft}}\right)$$
$= 238 < 300$ from AISC *Specification* Section D1 **o.k.**

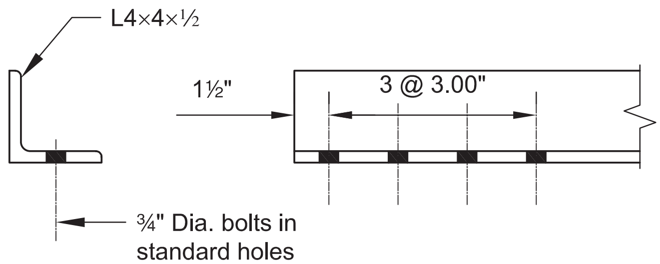
The W8×21 available tensile strength is governed by the tensile rupture limit state at the end connection.

See Chapter J for illustrations of connection limit state checks.

EXAMPLE D.2 SINGLE ANGLE TENSION MEMBER

Given:

Verify, by both ASD and LRFD, the tensile strength of an L4×4×½, ASTM A36, with one line of (4) ¾-in.-diameter bolts in standard holes. The member carries a dead load of 20 kips and a live load of 60 kips in tension. Calculate at what length this tension member would cease to satisfy the recommended slenderness limit. Assume that connection limit states do not govern.



Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

L4×4×½
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-7, the geometric properties are as follows:

L4×4×½
A_g = 3.75 in.^{2}
r_z = 0.776 in.
$\bar{y}$ = 1.18 in. = $\bar{x}$

From Chapter 2 of ASCE/SEI 7, the required tensile strength is:

LRFD	ASD
P_u = 1.2(20 kips) + 1.6(60 kips) = 120 kips	P_a = 20 kips + 60 kips = 80.0 kips

Tensile Yielding

$P_n = F_yA_g$ (*Spec*. Eq. D2-1)
= 36 ksi(3.75 in.^{2})
= 135 kips

From AISC *Specification* Section D2, the available tensile yielding strength is:

LRFD	ASD
ϕ_t = 0.90 ϕ_tP_n = 0.90(135 kips) = 122 kips	Ω_t = 1.67 $\dfrac{P_n}{\Omega_t} = \dfrac{135\text{ kips}}{1.67}$ = 80.8 kips

Tensile Rupture

Calculate U as the larger of the values from AISC *Specification* Section D3, Table D3.1 Case 2 and Case 8.

From AISC *Specification* Section D3, for open cross sections, U need not be less than the ratio of the gross area of the connected element(s) to the member gross area, therefore,

$$U = 0.500$$

Case 2:

$$\begin{aligned} U &= 1 - \frac{\bar{x}}{l} \\ &= 1 - \frac{1.18 \text{ in.}}{9.00 \text{ in.}} \\ &= 0.869 \end{aligned}$$

Case 8, with 4 or more fasteners per line in the direction of loading:

$$U = 0.80$$

Use $U = 0.869$.

Calculate A_n using AISC *Specification* Section B4.3.

$$\begin{aligned} A_n &= A_g - (d_h + 1/16)t \\ &= 3.75 \text{ in.}^2 - (13/16 \text{ in.} + 1/16 \text{ in.})(1/2 \text{ in.}) \\ &= 3.31 \text{ in.}^2 \end{aligned}$$

Calculate A_e using AISC *Specification* Section D3.

$$\begin{aligned} A_e &= A_n U \\ &= 3.31 \text{ in.}^2(0.869) \\ &= 2.88 \text{ in.}^2 \end{aligned}$$ (*Spec*. Eq. D3-1)

$$\begin{aligned} P_n &= F_u A_e \\ &= 58 \text{ ksi}(2.88 \text{ in.}^2) \\ &= 167 \text{ kips} \end{aligned}$$ (*Spec*. Eq. D2-2)

From AISC *Specification* Section D2, the available tensile rupture strength is:

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(167 \text{ kips})$ $= 125 \text{ kips}$	$\Omega_t = 2.00$ $\dfrac{P_n}{\Omega_t} = \dfrac{167 \text{ kips}}{2.00}$ $= 83.5 \text{ kips}$

The L4×4×½ available tensile strength is governed by the tensile yielding limit state.

LRFD	ASD
$\phi_t P_n = 122 \text{ kips}$ 122 kips > 120 kips **o.k.**	$\dfrac{P_n}{\Omega_t} = 80.8 \text{ kips}$ 80.8 kips > 80.0 kips **o.k.**

Recommended L_{max}

Using AISC *Specification* Section D1:

$$
\begin{aligned}
L_{max} &= 300r_z \\
&= (300)(0.776\text{ in.})\left(\frac{\text{ft}}{12.0\text{ in.}}\right) \\
&= 19.4\text{ ft}
\end{aligned}
$$

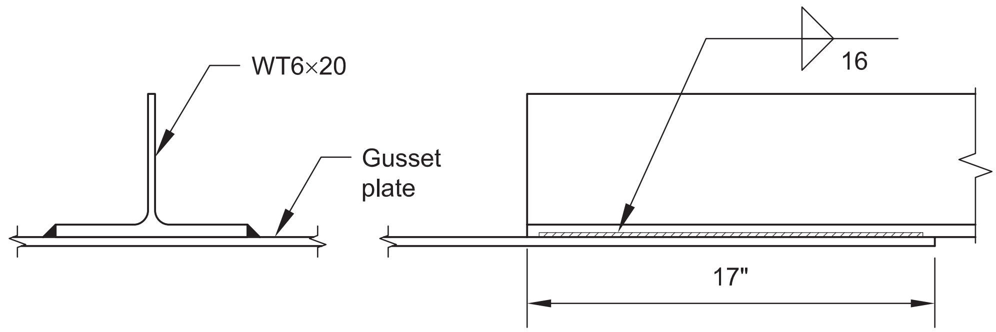
Note: The *L/r* limit is a recommendation, not a requirement.

See Chapter J for illustrations of connection limit state checks.

EXAMPLE D.3 WT-SHAPE TENSION MEMBER

Given:

A WT6×20, ASTM A992 member has a length of 30 ft and carries a dead load of 40 kips and a live load of 120 kips in tension. The end connection is fillet welded on each side for 16 in. Verify the member tensile strength by both LRFD and ASD. Assume that the gusset plate and the weld are satisfactory.



Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

WT6×20
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From AISC *Manual* Table 1-8, the geometric properties are as follows:

WT6×20
A_g = 5.84 in.^{2}
b_f = 8.01 in.
t_f = 0.515 in.
r_x = 1.57 in.
$\bar{y}$ = 1.09 in. = $\bar{x}$ (in equation for U)

From Chapter 2 of ASCE/SEI 7, the required tensile strength is:

LRFD	ASD
P_u = 1.2(40 kips) + 1.6(120 kips) = 240 kips	P_a = 40 kips + 120 kips = 160 kips

Tensile Yielding

Check tensile yielding limit state using AISC *Manual* Table 5-3.

LRFD	ASD
$\phi_t P_n$ = 263 kips > 240 kips **o.k.**	$\dfrac{P_n}{\Omega_t}$ = 175 kips > 160 kips **o.k.**

Tensile Rupture

Check tensile rupture limit state using AISC *Manual* Table 5-3.

LRFD	ASD
$\phi_t P_n = 214 \text{ kips} < 240 \text{ kips}$ **n.g.**	$\dfrac{P_n}{\Omega_t} = 142 \text{ kips} < 160 \text{ kips}$ **n.g.**

The tabulated available rupture strengths may be conservative for this case; therefore, calculate the exact solution.

Calculate U as the larger of the values from AISC *Specification* Section D3 and Table D3.1 case 2.

From AISC *Specification* Section D3, for open cross-sections, U need not be less than the ratio of the gross area of the connected element(s) to the member gross area.

$$\begin{aligned} U &= \frac{b_f t_f}{A_g} \\ &= \frac{8.01 \text{ in.}(0.515 \text{ in.})}{5.84 \text{ in.}^2} \\ &= 0.706 \end{aligned}$$

Case 2:

$$\begin{aligned} U &= 1 - \frac{\bar{x}}{l} \\ &= 1 - \frac{1.09 \text{ in.}}{16.0 \text{ in.}} \\ &= 0.932 \end{aligned}$$

Use $U = 0.932$.

Calculate A_n using AISC *Specification* Section B4.3.

$$\begin{aligned} A_n &= A_g \text{ (because there are no reductions due to holes or notches)} \\ &= 5.84 \text{ in.}^2 \end{aligned}$$

Calculate A_e using AISC *Specification* Section D3.

$$\begin{aligned} A_e &= A_n U \\ &= 5.84 \text{ in.}^2(0.932) \\ &= 5.44 \text{ in.}^2 \end{aligned}$$ (*Spec*. Eq. D3-1)

Calculate P_n.

$$\begin{aligned} P_n &= F_u A_e \\ &= 65 \text{ ksi}(5.44 \text{ in.}^2) \\ &= 354 \text{ kips} \end{aligned}$$ (*Spec*. Eq. D2-2)

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From AISC *Specification* Section D2, the available tensile rupture strength is:

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(354\text{ kips})$ $= 266\text{ kips}$ 266 kips > 240 kips **o.k.**	$\Omega_t = 2.00$ $\dfrac{P_n}{\Omega_t} = \dfrac{354\text{ kips}}{2.00}$ $= 177\text{ kips}$ 177 kips > 160 kips **o.k.**

Alternately, the available tensile rupture strengths can be determined by modifying the tabulated values. The available tensile rupture strengths published in the tension member selection tables are based on the assumption that $A_e = 0.75A_g$. The actual available strengths can be determined by adjusting the values from AISC *Manual* Table 5-3 as follows:

LRFD	ASD
$\phi_t P_n = 214\text{ kips}\left(\dfrac{A_e}{0.75A_g}\right)$ $= 214\text{ kips}\left(\dfrac{5.44\text{ in.}^2}{0.75\left(5.84\text{ in.}^2\right)}\right)$ $= 266\text{ kips}$	$\dfrac{P_n}{\Omega_t} = 142\text{ kips}\left(\dfrac{A_e}{0.75A_g}\right)$ $= 142\text{ kips}\left(\dfrac{5.44\text{ in.}^2}{0.75\left(5.84\text{ in.}^2\right)}\right)$ $= 176\text{ kips}$

The WT6×20 available tensile strength is governed by the tensile yielding limit state.

LRFD	ASD
$\phi_t P_n = 263\text{ kips}$ 263 kips > 240 kips **o.k.**	$\dfrac{P_n}{\Omega_t} = 175\text{ kips}$ 175 kips > 160 kips **o.k.**

Recommended Slenderness Limit

$$\frac{L}{r} = \left(\frac{30.0\text{ ft}}{1.57\text{ in.}}\right)\left(\frac{12.0\text{ in.}}{\text{ft}}\right)$$

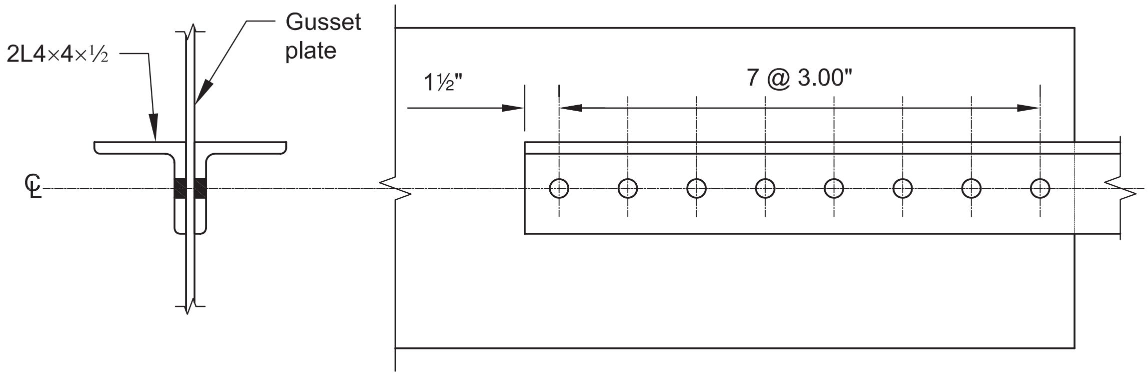
$= 229 < 300$ from AISC *Specification* Section D1 **o.k.**

See Chapter J for illustrations of connection limit state checks.

EXAMPLE D.6 DOUBLE ANGLE TENSION MEMBER

Given:

A 2L4×4×½ (⅜-in. separation), ASTM A36, has one line of (8) ¾-in.-diameter bolts in standard holes and is 25 ft in length. The double angle is carrying a dead load of 40 kips and a live load of 120 kips in tension. Verify the member tensile strength. Assume that the gusset plate and bolts are satisfactory.



Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Tables 1-7 and 1-15, the geometric properties are as follows:

L4×4×½
A_g = 3.75 in.^{2}
$\bar{x}$ = 1.18 in.

2L4×4×½ (s = ⅜ in.)
r_y = 1.83 in.
r_x = 1.21 in.

From Chapter 2 of ASCE/SEI 7, the required tensile strength is:

LRFD	ASD
P_n = 1.2(40 kips) + 1.6(120 kips) = 240 kips	P_n = 40 kips + 120 kips = 160 kips

Tensile Yielding

$$P_n = F_y A_g \qquad (Spec.\ \text{Eq. D2-1})$$
$$= 36 \text{ ksi}(2)(3.75 \text{ in.}^2)$$
$$= 270 \text{ kips}$$

From AISC *Specification* Section D2, the available tensile yielding strength is:

LRFD	ASD
$\phi_t = 0.90$ $\phi_t P_n = 0.90(270\text{ kips})$ $= 243\text{ kips}$	$\Omega_t = 1.67$ $\dfrac{P_n}{\Omega_t} = \dfrac{270\text{ kips}}{1.67}$ $= 162\text{ kips}$

Tensile Rupture

Calculate U as the larger of the values from AISC *Specification* Section D3, Table D3.1 case 2 and case 8.

From AISC *Specification* Section D3, for open cross-sections, U need not be less than the ratio of the gross area of the connected element(s) to the member gross area.

$$U = 0.500$$

Case 2:

$$\begin{aligned} U &= 1 - \frac{\bar{x}}{l} \\ &= 1 - \frac{1.18\text{ in.}}{21.0\text{ in.}} \\ &= 0.944 \end{aligned}$$

Case 8, with 4 or more fasteners per line in the direction of loading:

$$U = 0.80$$

Use $U = 0.944$.

Calculate A_n using AISC *Specification* Section B4.3.

$$\begin{aligned} A_n &= A_g - 2(d_h + \tfrac{1}{16}\text{ in.})t \\ &= 2(3.75\text{ in.}^2) - 2(\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})(\tfrac{1}{2}\text{ in.}) \\ &= 6.63\text{ in.}^2 \end{aligned}$$

Calculate A_e using AISC *Specification* Section D3.

$$\begin{aligned} A_e &= A_n U \\ &= 6.63\text{ in.}^2(0.944) \\ &= 6.26\text{ in.}^2 \end{aligned}$$ (*Spec*. Eq. D3-1)

Calculate P_n.

$$\begin{aligned} P_n &= F_u A_e \\ &= 58\text{ ksi}(6.26\text{ in.}^2) \\ &= 363\text{ kips} \end{aligned}$$ (*Spec*. Eq. D2-2)

From AISC *Specification* Section D2, the available tensile rupture strength is:

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(363\text{ kips})$ $= 272\text{ kips}$	$\Omega_t = 2.00$ $\dfrac{P_n}{\Omega_t} = \dfrac{363\text{ kips}}{2.00}$ $= 182\text{ kips}$

The double angle available tensile strength is governed by the tensile yielding limit state.

LRFD	ASD
243 kips > 240 kips **o.k.**	162 kips > 160 kips **o.k.**

Recommended Slenderness Limit

$$\frac{L}{r_x} = \left(\frac{25.0\text{ ft}}{1.21\text{ in.}}\right)\left(\frac{12.0\text{ in.}}{\text{ft}}\right)$$

= 248 < 300 from AISC *Specification* Section D1 **o.k.**

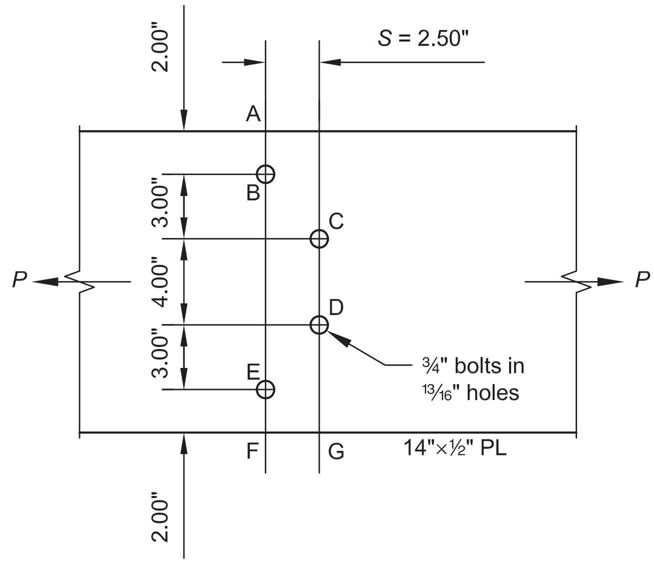
Note: From AISC *Specification* Section D4, the longitudinal spacing of connectors between components of built-up members should preferably limit the slenderness ratio in any component between the connectors to a maximum of 300.

See Chapter J for illustrations of connection limit state checks.

EXAMPLE D.9 PLATE WITH STAGGERED BOLTS

Given:

Compute A_n and A_e for a 14-in.-wide and ½-in.-thick plate subject to tensile loading with staggered holes as shown.



Solution:

Calculate net hole diameter using AISC *Specification* Section B4.3.

$d_{net} = d_h + 1/16$ in.
$= 13/16$ in. $+ 1/16$ in.
$= 0.875$ in.

Compute the net width for all possible paths across the plate. Because of symmetry, many of the net widths are identical and need not be calculated.

$$w = 14.0 - \Sigma d_{net} + \Sigma \frac{s^2}{4g}$$ from AISC *Specification* Section B4.3.

Line A-B-E-F: $w = 14.0 \text{ in.} - 2(0.875 \text{ in.})$
$= 12.3$ in.

Line A-B-C-D-E-F: $$w = 14.0 \text{ in.} - 4(0.875 \text{ in.}) + \frac{(2.50 \text{ in.})^2}{4(3.00 \text{ in.})} + \frac{(2.50 \text{ in.})^2}{4(3.00 \text{ in.})}$$
$= 11.5$ in. **controls**

Line A-B-C-D-G: $$w = 14.0 \text{ in.} - 3(0.875 \text{ in.}) + \frac{(2.50 \text{ in.})^2}{4(3.00 \text{ in.})}$$
$= 11.9$ in.

Line A-B-D-E-F: $$w = 14.0 \text{ in.} - 3(0.875 \text{ in.}) + \frac{(2.50 \text{ in.})^2}{4(7.00 \text{ in.})} + \frac{(2.50 \text{ in.})^2}{4(3.00 \text{ in.})}$$
$= 12.1$ in.

Therefore, $A_n = 11.5 \text{ in.}(0.500 \text{ in.})$

$= 5.75 \text{ in.}^2$

Calculate *U*.

From AISC *Specification* Table D3.1 case 1, because tension load is transmitted to all elements by the fasteners,

$U = 1.0$

$$\begin{aligned} A_e &= A_n U \qquad \text{(Spec. Eq. D3-1)} \\ &= 5.75 \text{ in.}^2(1.0) \\ &= 5.75 \text{ in.}^2 \end{aligned}$$