3.5 **BLOCK SHEAR**

For certain connection configurations, a segment or "block" of material at the end of the member can tear out. For example, the connection of the single-angle tension member shown in Figure 3.21 is susceptible to this phenomenon, called *block shear.*

\[
F_{yw} = 0.6 F_{yt} = 0.6 F_y
\]
For the case illustrated, the shaded block would tend to fail by shear along the longitudinal section \( ab \) and by tension on the transverse section \( bc \).

The model used in the AISC Specification assumes that failure occurs by rupture (fracture) on the shear area and rupture on the tension area. Both surfaces contribute to the total strength, and the resistance to block shear will be the sum of the strengths of the two surfaces. The shear rupture stress is taken as 60% of the tensile ultimate.
C/6x13 7/8" bolts

Structural 1" holes

Channel

Flange

FLANGE

WEB

FLANGE

GROSS

So \( A_{Gross} = (8")(0.437") (2\ \text{sides}) \)

= 6.99\text{in}^2

\[ A_{Gross} = \frac{8" - (2 \times \frac{1}{2} \text{holes})(1")}{2} (0.437") (2\ \text{sides}) \]

\[ A_{net} = \left[ 5" - 2 \times \left( \frac{3}{4}" \right) \right] (0.437") \]
Correct Block Shear Pattern

Ant = small

N.A.
Incorrect Block Shear Pattern

\[ \text{Amount} = \text{large} \]

\[ \frac{\text{Amount}}{4} \]

(same)

\[ \frac{\text{Amount}}{4} \]

(massive)

N.A.
Correct - Ant = small
Failure plane never goes across outstanding elements

Incorrect - Ant = massive
OK to check but wastes valuable time on quizzes

$U_{BS} = 1.0$
$U_{BS} = 0.5$
stress, so the nominal strength in shear is \(0.6F_u A_{nv}\) and the nominal strength in tension is \(F_u A_{nt}\).

where

\[
A_{nv} = \text{net area along the shear surface or surfaces}
\]

\[
A_{nt} = \text{net area along the tension surface}
\]

This gives a nominal strength of

\[
R_n = 0.6F_u A_{nv} + F_u A_{nt}
\]  

(3.3)

The AISC Specification uses Equation 3.3 for angles and gusset plates, but for certain types of coped beam connections (to be covered in Chapter 5), the second term is reduced to account for nonuniform tensile stress. The tensile stress is nonuniform when some rotation of the block is required for failure to occur. For these cases,

\[
R_n = 0.6F_u A_{nv} + 0.5F_u A_{nt}
\]  

(3.4)

The AISC Specification limits the \(0.6F_u A_{nv}\) term to \(0.6F_y A_{gv}\), where

\[
0.6F_y = \text{shear yield stress}
\]

\[
A_{gv} = \text{gross area along the shear surface or surfaces}
\]

and gives one equation to cover all cases as follows:

\[
R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.6F_y A_{gv} + U_{bs} F_u A_{nt}
\]  

(AISC Equation J4-5)

where \(U_{bs} = 1.0\) when the tension stress is uniform (angles, gusset plates, and most coped beams) and \(U_{bs} = 0.5\) when the tension stress is nonuniform. A nonuniform case is illustrated in the Commentary to the Specification.

For LRFD, the resistance factor \(\phi\) is 0.75, and for ASD, the safety factor \(\Omega\) is 2.00. Recall that these are the factors used for the fracture—or rupture—limit state, and block shear is a rupture limit state.

Although AISC Equation J4-5 is expressed in terms of bolted connections, block shear can also occur in welded connections, especially in gusset plates.

---

**Example 3.10**

Compute the block shear strength of the tension member shown in Figure 3.23. The holes are for \(\frac{3}{8}\)-inch-diameter bolts, and A36 steel is used.

- **a.** Use LRFD.
- **b.** Use ASD.

**FIGURE 3.23**

L3\(\frac{1}{2}\) \(\times\) 3\(\frac{1}{2}\) \(\times\) \(\frac{3}{8}\), A36

\[
1\frac{1}{2}'' \quad 3'' \quad 3''
\]

\[
\frac{3}{8}\text{-in. bolts}
\]
\[ \phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)} \]

where
\[ A_e = \text{effective net area} \text{ as defined in Section D3, in.}^2 \text{ (mm}^2) \text{; for bolted splice plates, } A_e = A_n \leq 0.85A_r. \]

User Note: The effective net area of the connection plate may be limited due to stress distribution as calculated by methods such as the Whitmore section.

2. **Strength of Elements in Shear**

The available shear strength of affected and connecting elements in shear shall be the lower value obtained according to the *limit states* of shear yielding and shear rupture:

(a) For shear yielding of the element:
\[ R_n = 0.60F_vA_{gv} \quad (J4-3) \]
\[ \phi = 1.00 \text{ (LRFD)} \quad \Omega = 1.50 \text{ (ASD)} \]

where
\[ A_{gv} = \text{gross area subject to shear, in.}^2 \text{ (mm}^2) \]

(b) For shear rupture of the element:
\[ R_n = 0.60F_uA_{nv} \quad (J4-4) \]
\[ \phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)} \]

where
\[ A_{nv} = \text{net area} \text{ subject to shear, in.}^2 \text{ (mm}^2) \]

3. **Block Shear Strength**

The *available strength* for the *limit state* of block shear rupture along a shear failure path or paths and a perpendicular tension failure path shall be taken as
\[ R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.60F_vA_{gv} + U_{bs}F_uA_{xt} \quad (J4-5) \]

Resistance factor \[ \phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)} \]

where
\[ A_{nt} = \text{net area} \text{ subject to tension, in.}^2 \text{ (mm}^2) \]

Where the tension stress is uniform, \( U_{bs} = 1 \); where the tension stress is nonuniform, \( U_{bs} = 0.5 \).

User Note: Typical cases where \( U_{bs} \) should be taken equal to 0.5 are illustrated in the Commentary.

4. **Strength of Elements in Compression**

The available strength of connecting elements in compression for the *limit states* of yielding and buckling shall be determined as follows:
The shear areas are:

\[
A_{wv} = \frac{3}{8}(7.5) = 2.813 \text{ in.}^2.
\]

and, since there are 2.5 hole diameters,

\[
A_{nv} = \frac{3}{8}\left[7.5 - 2.5\left(\frac{7}{8} + \frac{1}{8}\right)\right] = 1.875 \text{ in.}^2.
\]

The tension area is

\[
A_{nt} = \frac{3}{8}\left[1.5 - 0.5\left(\frac{7}{8} + \frac{1}{8}\right)\right] = 0.3750 \text{ in.}^2.
\]

(The factor of 0.5 is used because there is one-half of a hole diameter in the tension section.)

Since the block shear will occur in an angle, \( U_{bs} = 1.0 \), and from AISC Equation J4-5,

\[
R_{u} = 0.6F_{u}A_{uv} + U_{bs}F_{u}A_{nt}
\]

\[
= 0.6(58)(1.875) + 1.0(58)(0.3750) = 87.00 \text{ kips}
\]

with an upper limit of

\[
0.6F_{y}A_{gv} + U_{bs}F_{u}A_{nt} = 0.6(35)(2.813) + 1.0(58)(0.3750) = 82.51 \text{ kips}
\]

The nominal block shear strength is therefore 82.51 kips.

**ANSWER**

a. The design strength for LRFD is \( \phi R_{u} = 0.75(82.51) = 61.9 \text{ kips}. \)

b. The allowable strength for ASD is \( \frac{R_{u}}{\Omega} = \frac{82.51}{2.00} = 41.3 \text{ kips}. \)

**3.6 DESIGN OF TENSION MEMBERS**
Fig. C-J4.1. Failure surface for block shear rupture limit state.

(a) Cases for which \( U_{bs} = 1.0 \)

(b) Cases for which \( U_{bs} = 0.5 \)

Fig. C-J4.2. Block shear tensile stress distributions.
of gyration is the one corresponding to the minor principal axis of the cross section. This value is tabulated for all rolled shapes in the properties tables in Part 1 of the Manual.

rod subjected to wind loads. For this reason, the user note in AISC D1 suggests a maximum slenderness ratio of 300. It is only a recommended value because slenderness has no structural significance for tension members, and the limit may be exceeded when special circumstances warrant it. This limit does not apply to cables, and the user note explicitly excludes rods.

The central problem of all member design, including tension member design, is to find a cross section for which the required strength does not exceed the available strength. For tension members designed by LRFD, the requirement is

\[ P_u \leq \phi_t P_n \quad \text{or} \quad \phi_t P_n \geq P_u \]

where \( P_u \) is the sum of the factored loads. To prevent yielding,

\[ 0.90F_y A_y \geq P_u \quad \text{or} \quad A_y \geq \frac{P_u}{0.90F_y} \quad \text{Eq 68a} \]

To avoid fracture,

\[ 0.75F_u A_e \geq P_u \quad \text{or} \quad A_e \geq \frac{P_u}{0.75F_u} \quad \text{Eq 68b} \]
Example 3.11

A tension member with a length of 5 feet 9 inches must resist a service dead load of 18 kips and a service live load of 52 kips. Select a member with a rectangular cross section. Use A36 steel and assume a connection with one line of \( \frac{7}{8} \)-inch-diameter bolts.

\[
P_u = 1.2D + 1.6L = 1.2(18) + 1.6(52) = 104.8 \text{ kips}
\]

**LRFD Solution**

Try \( t = 1 \) in.

\[
\text{Required } w' = \text{required } A_{x} = \frac{3.235}{1} = 3.235 \text{ in.}
\]

Try a \( 1 \times 3\frac{1}{2} \) cross section.

\[
A_x = A_n = A_g - A_{\text{hole}}
\]

\[
= (1 \times 3.5) - \left( \frac{7}{8} + \frac{1}{8} \right) (1) = 2.5 \text{ in.}^2 > 2.409 \text{ in.}^2 \quad \text{(OK)}
\]

Check the slenderness ratio:

\[
I_{\text{min}} = \frac{3.5(1)^3}{12} = 0.2917 \text{ in.}^4
\]

\[
A = l(3.5) = 3.5 \text{ in.}^2
\]

From \( I = Ar^2 \), we obtain

\[
r_{\text{min}} = \sqrt{\frac{I_{\text{min}}}{A}} = \sqrt{\frac{0.2917}{3.5}} = 0.2887 \text{ in.}
\]

Maximum \( \frac{L}{r} = \frac{5.75(12)}{0.2887} = 239 < 300 \quad \text{(OK)}
\]

**Answer**

Use a PL \( 1 \times 3\frac{1}{2} \).
are two lines of bolts in a leg. The usual fabrication practice is to punch or drill holes in standard locations in angle legs. These hole locations are given in Table 1-7A in Part 1 of the *Manual*. This table is located at the end of the dimensions and properties table for angles. Figure 3.24 presents this same information. Gage distance \( e \) applies when

**EXAMPLE 3.12**

Select an unequal-leg angle tension member 15 feet long to resist a service dead load of 35 kips and a service live load of 70 kips. Use A36 steel. The connection is shown in Figure 3.25.
The factored load is

\[ P_u = 1.2D + 1.6L = 1.2(35) + 1.6(70) = 154 \text{ kips} \]

Required \( A_g = \)  

Required \( A_e = \)  

The radius of gyration should be at least

\[ \frac{L}{300} = \frac{15(12)}{300} = 0.6 \text{ in.} \geq \frac{3}{3.2} \]

To find the lightest shape that satisfies these criteria, we search the dimensions and properties table for the unequal-leg angle that has the smallest acceptable gross area and then check the effective net area. The radius of gyration can be checked by inspection. There are two lines of bolts, so the connected leg must be at least 5 inches long (see the usual gages for angles in Figure 3.24). Starting at either end of the table, we find that the shape with the smallest area that is at least equal to 4.75 in.\(^2\) is an L6 \(\times\) 4 \(\times\) \(\frac{1}{2}\) with an area of 4.75 in.\(^2\) and a minimum radius of gyration of 0.864 in.

Try L6 \(\times\) 4 \(\times\) \(\frac{1}{2}\).  

\[ A_e = A_g - A_{holes} = 4.75 - 2 \left( \frac{3}{4} + \frac{1}{8} \right) \left( \frac{1}{2} \right) = 3.875 \text{ in.}^2 \]

Because the length of the connection is not known, Equation 3.1 cannot be used to compute the shear lag factor \( U \). Since there are four bolts in the direction of the load, we will use the alternative value of \( U = 0.80 \).

\[ A_e = A_nU = 3.875(0.80) = 3.10 \text{ in.}^2 < 3.54 \text{ in.}^2 \quad \text{(N.G.)}^* \]

Try the next larger shape from the dimensions and properties tables.  

Try L5 \(\times\) 3\(\frac{1}{2}\) \(\times\) \(\frac{5}{8}\) (\(A_g = 4.93 \text{ in.}^2\) and \(r_{min} = 0.746 \text{ in.}\))

\[ A_e = A_g - A_{holes} = 4.93 - 2 \left( \frac{3}{4} + \frac{1}{8} \right) \left( \frac{5}{8} \right) = 3.836 \text{ in.}^2 \]

\[ A_e = A_nU = 3.836(0.80) = 3.07 \text{ in.}^2 < 3.54 \text{ in.}^2 \quad \text{(N.G.)} \]

(Note that this shape has slightly more gross area than that produced by the previous trial shape, but because of the greater leg thickness, slightly more area is deducted for the holes.) Passing over the next few heavier shapes,

Try L8 \(\times\) 4 \(\times\) \(\frac{1}{2}\) (\(A_g = 5.80 \text{ in.}^2\) and \(r_{min} = 0.863 \text{ in.}\))

\[ A_e = A_g - A_{holes} = 5.80 - 2 \left( \frac{3}{4} + \frac{1}{8} \right) \left( \frac{1}{2} \right) = 4.925 \text{ in.}^2 \]

\[ A_e = A_nU = 4.925(0.80) = 3.94 \text{ in.}^2 > 3.54 \text{ in.}^2 \quad \text{(OK)} \]

*The notation N.G. means "No Good."
### Table 1-7 (continued)

#### Angles Properties

<table>
<thead>
<tr>
<th>Shape</th>
<th>k</th>
<th>Wt.</th>
<th>Area, A</th>
<th>Axis X-X</th>
<th>Flexural-Torsional Properties</th>
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</thead>
<tbody>
<tr>
<td></td>
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<td>l</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>in.</td>
</tr>
<tr>
<td>L6\times 3/8</td>
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<td>8.00</td>
<td>27.7</td>
<td>7.13</td>
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<tr>
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<td>6.94</td>
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<td>6.23</td>
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Note: Not unequal legs

---

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### Table 1-7 (continued)
#### Angles

**Properties**

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<tr>
<th>Shape</th>
<th>Axis Y-Y</th>
<th>Axis Z-Z</th>
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<td>( S )</td>
<td>( r )</td>
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<td>in.(^3)</td>
<td>in.</td>
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# Table 1-7

## Angles

### Properties

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<th>( I )</th>
<th>( S )</th>
<th>( r )</th>
<th>( \bar{y} )</th>
<th>( Z )</th>
<th>( y_p )</th>
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<th>Flexural-Torsional Properties</th>
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<td></td>
<td></td>
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<td>lb/ft</td>
<td>in.²</td>
<td>in.⁴</td>
<td>in.³</td>
<td>in.</td>
<td>in.</td>
<td>in.⁴</td>
<td>J</td>
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**Note:** Table entries are approximations and subject to rounding errors.
### Table 1-7 (continued)

#### Angles

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<th>Shape</th>
<th>Axis Y-Y</th>
<th>Axis Z-Z</th>
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<td>r</td>
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<td>in.</td>
<td>in.</td>
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<table>
<thead>
<tr>
<th>Case</th>
<th>Description of Element</th>
<th>Shear Lag Factor, $U$</th>
<th>Example</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>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).</td>
<td>$U = 1.0$</td>
<td>——</td>
</tr>
<tr>
<td>2</td>
<td>All tension members, except plates and HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or longitudinal welds or by longitudinal welds in combination with transverse welds. (Alternatively, for W, M, S and HP, Case 7 may be used. For angles, Case 8 may be used.)</td>
<td>$U = 1 - \frac{\bar{x}}{l}$</td>
<td>——</td>
</tr>
<tr>
<td>3</td>
<td>All tension members where the tension load is transmitted only by transverse welds to some but not all of the cross-sectional elements.</td>
<td>$U = 1.0$ and $A_n = \text{area of the directly connected elements}$</td>
<td>——</td>
</tr>
<tr>
<td>4</td>
<td>Plates where the tension load is transmitted by longitudinal welds only.</td>
<td>$I \geq 2w...U = 1.0$  \hspace{1cm} $2w/I \geq 1.5w...U = 0.87$  \hspace{1cm} $1.5w/I \geq w...U = 0.75$</td>
<td>——</td>
</tr>
<tr>
<td>5</td>
<td>Round HSS with a single concentric gusset plate</td>
<td>$I \geq 1.3D...U = 1.0$  \hspace{1cm} $D \leq I &lt; 1.3D...U = 1 - \frac{\bar{x}}{l}$ \hspace{1cm} $\bar{x} = \frac{D}{\pi}$</td>
<td>——</td>
</tr>
<tr>
<td>6</td>
<td>Rectangular HSS with a single concentric gusset plate</td>
<td>$I \geq H...U = 1 - \frac{\bar{x}}{l}$ \hspace{1cm} $\bar{x} = \frac{B^2 + 2BH}{4(B + H)}$</td>
<td>——</td>
</tr>
<tr>
<td></td>
<td>with two side gusset plates</td>
<td>$I \geq H...U = 1 - \frac{\bar{x}}{l}$ \hspace{1cm} $\bar{x} = \frac{B^2}{4(B + H)}$</td>
<td>——</td>
</tr>
<tr>
<td>7</td>
<td>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.)</td>
<td>$b_t \geq 2/3d...U = 0.90$  \hspace{1cm} $b_t &lt; 2/3d...U = 0.85$</td>
<td>——</td>
</tr>
<tr>
<td></td>
<td>with flange connected with 3 or more fasteners per line in the direction of loading</td>
<td>$U = 0.70$</td>
<td>——</td>
</tr>
<tr>
<td></td>
<td>with web connected with 4 or more fasteners per line in the direction of loading</td>
<td>$U = 0.80$</td>
<td>——</td>
</tr>
<tr>
<td>8</td>
<td>Single and double angles (If $U$ is calculated per Case 2, the larger value is permitted to be used.)</td>
<td>with 4 or more fasteners per line in the direction of loading</td>
<td>$U = 0.80$</td>
</tr>
<tr>
<td></td>
<td>with 3 fasteners per line in the direction of loading (With fewer than 3 fasteners per line in the direction of loading, use Case 2.)</td>
<td>$U = 0.60$</td>
<td>——</td>
</tr>
</tbody>
</table>

$I =$ length of connection, in. (mm); $w =$ plate width, in. (mm); $\bar{x} =$ eccentricity of connection, in. (mm); $B =$ overall width of rectangular HSS member, measured 90° to the plane of the connection, in. (mm); $H =$ overall height of rectangular HSS member, measured in the plane of the connection, in. (mm)
Tables for the Design of Tension Members

Part 5 of the Manual contains tables to assist in the design of tension members of various cross-sectional shapes, including Table 5-2 for angles. The use of these tables will be illustrated in the following example.

EXAMPLE 3.13

Design the tension member of Example 3.12 with the aid of the tables in Part 5 of the Manual.

From Example 3.12,

\[ P_u = 154 \text{ kips} \]
\[ r_{\text{min}} \geq 0.600 \text{ in.} \]
The tables for design of tension members give values of $A_e$ and $A_e$ for various shapes based on the assumption that $A_e = 0.75A_g$. In addition, the corresponding available strengths based on yielding and rupture (fracture) are given. All values available for angles are for A36 steel. Starting with the lighter shapes (the ones with the smaller gross area), we find that an $6 \times 4 \times 1 \frac{1}{2}$, with $\phi P_n = 154$ kips based on the gross section and $\phi P_n = 155$ kips based on the net section, is a possibility. From the dimensions and properties tables in Part 1 of the Manual, $r_{\min} = 0.864 \text{ in}$. To check this selection, we must compute the actual net area. If we assume that $U = 0.80$, 

$$
A_n = A_g - A_{holes} = 4.75 - 2\left(\frac{3}{4} + \frac{1}{8}\right)\left(\frac{1}{2}\right) = 3.875 \text{ in}^2
$$

$$
A_e = A_nU = 3.875(0.80) = 3.10 \text{ in}^2
$$

$$
\phi P_n = \phi F_u A_e = 0.75(58)(3.10) = 135 \text{ kips} < 154 \text{ kips} \quad \text{(N.G.)}
$$

This shape did not work because the ratio of actual effective net area $A_e$ to gross area $A_g$ is not equal to 0.75. The ratio is closer to

$$
\frac{3.10}{4.75} = 0.6526
$$

This corresponds to a required $\phi P_n$ (based on rupture) of

$$
\frac{0.75 \times P_{\text{new}}}{\text{actual ratio}} = 0.75 \times 0.6526 = 177 \text{ kips}
$$

Try an $8 \times 4 \times 1 \frac{1}{2}$, with $\phi P_n = 188$ kips (based on yielding) and $\phi P_n = 189$ Kips (based on rupture strength, with $A_e = 0.75A_g = 4.31 \text{ in}^2$). From the dimensions and properties tables in Part 1 of the Manual, $r_{\min} = 0.863 \text{ in}$. The actual effective net area and rupture strength are computed as follows:

$$
A_n = A_g - A_{holes} = 5.80 - 2\left(\frac{3}{4} + \frac{1}{8}\right)\left(\frac{1}{2}\right) = 4.925 \text{ in}^2
$$

$$
A_e = A_nU = 4.925(0.80) = 3.94 \text{ in}^2
$$

$$
\phi P_n = \phi F_u A_e = 0.75(58)(3.94) = 171 > 154 \text{ kips} \quad \text{(OK)}
$$

**ANSWER**

Use an $8 \times 4 \times 1 \frac{1}{2}$, connected through the 8-inch leg.
Table 5-2 (continued)
Available Strength in Axial Tension
Angles

<table>
<thead>
<tr>
<th>Shape</th>
<th>Gross Area, $A_g$</th>
<th>$A_g = 0.75 A_g$</th>
<th>Yielding</th>
<th>Rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>in.$^2$</td>
<td>in.$^2$</td>
<td>ASD</td>
<td>LRFD</td>
</tr>
<tr>
<td>L6\times4\times7/8</td>
<td>8.00</td>
<td>6.00</td>
<td>172</td>
<td>259</td>
</tr>
<tr>
<td>$\times\frac{3}{4}$</td>
<td>6.94</td>
<td>5.21</td>
<td>150</td>
<td>225</td>
</tr>
<tr>
<td>$\times\frac{3}{8}$</td>
<td>5.86</td>
<td>4.40</td>
<td>126</td>
<td>190</td>
</tr>
<tr>
<td>$\times\frac{1}{16}$</td>
<td>5.31</td>
<td>3.98</td>
<td>114</td>
<td>172</td>
</tr>
<tr>
<td>$\times\frac{1}{2}$</td>
<td>4.75</td>
<td>3.56</td>
<td>102</td>
<td>154</td>
</tr>
<tr>
<td>$\times\frac{7}{16}$</td>
<td>4.18</td>
<td>3.14</td>
<td>90.1</td>
<td>135</td>
</tr>
<tr>
<td>$\times\frac{5}{8}$</td>
<td>3.61</td>
<td>2.71</td>
<td>77.8</td>
<td>111</td>
</tr>
<tr>
<td>$\times\frac{1}{8}$</td>
<td>3.03</td>
<td>2.27</td>
<td>65.3</td>
<td>98.2</td>
</tr>
<tr>
<td>L6\times3\frac{1}{2}\times1/2</td>
<td>4.50</td>
<td>3.38</td>
<td>97.0</td>
<td>146</td>
</tr>
<tr>
<td>$\times\frac{3}{4}$</td>
<td>3.44</td>
<td>2.58</td>
<td>74.2</td>
<td>111</td>
</tr>
<tr>
<td>$\times\frac{1}{16}$</td>
<td>2.89</td>
<td>2.17</td>
<td>62.3</td>
<td>93.6</td>
</tr>
<tr>
<td>L5\times5\times7/8</td>
<td>8.00</td>
<td>6.00</td>
<td>172</td>
<td>259</td>
</tr>
<tr>
<td>$\times\frac{3}{4}$</td>
<td>6.98</td>
<td>5.24</td>
<td>150</td>
<td>226</td>
</tr>
<tr>
<td>$\times\frac{3}{8}$</td>
<td>5.90</td>
<td>4.43</td>
<td>127</td>
<td>191</td>
</tr>
<tr>
<td>$\times\frac{1}{2}$</td>
<td>4.79</td>
<td>3.59</td>
<td>103</td>
<td>155</td>
</tr>
<tr>
<td>$\times\frac{7}{16}$</td>
<td>4.22</td>
<td>3.17</td>
<td>91.0</td>
<td>137</td>
</tr>
<tr>
<td>$\times\frac{5}{8}$</td>
<td>3.65</td>
<td>2.74</td>
<td>78.7</td>
<td>119</td>
</tr>
<tr>
<td>$\times\frac{1}{8}$</td>
<td>3.07</td>
<td>2.30</td>
<td>66.2</td>
<td>99.5</td>
</tr>
<tr>
<td>L5\times3\frac{1}{2}\times3/4</td>
<td>5.85</td>
<td>4.39</td>
<td>126</td>
<td>150</td>
</tr>
<tr>
<td>$\times\frac{3}{4}$</td>
<td>4.93</td>
<td>3.70</td>
<td>106</td>
<td>150</td>
</tr>
<tr>
<td>$\times\frac{1}{2}$</td>
<td>4.00</td>
<td>3.00</td>
<td>86.2</td>
<td>130</td>
</tr>
<tr>
<td>$\times\frac{3}{8}$</td>
<td>3.05</td>
<td>2.29</td>
<td>65.7</td>
<td>98.8</td>
</tr>
<tr>
<td>$\times\frac{1}{8}$</td>
<td>2 56</td>
<td>1.09</td>
<td>55.9</td>
<td>83.6</td>
</tr>
</tbody>
</table>

Limit State | ASD | LRFD | Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_g \geq 0.745 A_g$.

Yielding | $\Omega_t = 1.67$ | $\Phi_t = 0.90$ |
Rupture | $\Omega_t = 2.00$ | $\Phi_t = 0.75$ |

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