(Borello, D. J., Denavit, M. D., and Hajjar, J. F., 2009). The Specification gives the following values for the filler factor:

- Where bolts have been added to distribute loads in the filler, \( h_f = 1.0 \).
- Where bolts have not been added to distribute loads in the filler and one filler is used, \( h_f = 1.0 \).
- Where bolts have not been added to distribute loads in the filler and two or more fillers are used, \( h_f = 0.85 \).

\[ [16.1 - 406] \] Filler plates are covered in more detail in the Commentary to AISC Section J3.8.

In this book, we do not use fillers in any connections, so we will always use \( h_f = 1.0 \).

The resistance factor for LRFD and the safety factor for ASD are different for different types of bolt holes. For standard holes (the only type considered in this book), these factors are

\[ \phi = 1.00 \quad \text{and} \quad \Omega = 1.50 \]

Although slip-critical connections are designed to not slip, if slip does occur because of an overload, the bolts must be capable of resisting shear and bearing. AISC J3.8 requires that shear and bearing be checked in slip-critical connections. \[ [16.1 - 126] \]

**EXAMPLE 7.4**

The connection shown in Figure 7.13a uses \( \frac{3}{4} \)-inch-diameter Group A bolts with the threads in the shear plane. No slip is permitted. Both the tension member and the gusset plate are of A36 steel. Determine the strength of the connection.

**SOLUTION**

Both the design strength (LRFD) and the allowable strength (ASD) will be computed. For efficiency, the nominal strength for each limit state will be computed before specializing the solution for LRFD and ASD.
Chapter 7 Simple Connections

Shear strength: For one bolt,

\[ A_b = \frac{\pi(3/4)^2}{4} = 0.4418 \text{ in.}^2 \]

\[ R_n = F_{m1} A_b = 54(0.4418) = 23.86 \text{ kips/bolt} \]

**Slip-critical strength:** Because no slippage is permitted, this connection is classified as slip-critical. From AISC Table J3-1, the minimum bolt tension is \( T_b = 28 \text{ kips.} \) From AISC Equation J3-4,

\[ R_n = \mu D_u h_f T_b n_s = 0.30(1.13)(1)(28)(1) = 9.492 \text{ kips/bolt} \]

For four bolts,

\[ R_n = 4(9.492) = 37.97 \text{ kips} \]

**Bearing strength:** Since both edge distances are the same, and the gusset plate is thinner than the tension member, the gusset plate thickness of \( \frac{3}{8} \) inch will be used. For bearing strength computation, use a hole diameter of

\[ h = d + \frac{1}{16} = \frac{3}{4} + \frac{1}{16} = \frac{13}{16} \text{ in.} \]

For the holes nearest the edge of the gusset plate,

\[ l_c = \frac{h}{2} = 1.5 - \frac{13/16}{2} = 1.094 \text{ in.} \]

\[ R_n = 1.2 l_c F_u = 1.2(1.094)(\frac{3}{8})(58) = 28.55 \text{ kips} \]

**Plug shear limit**

\[ l_c = 2.4 d t F_a = 2.4 \left( \frac{3}{4} \right) \left( \frac{3}{8} \right)(58) = 39.15 \text{ kips} \]

Upper limit = 39.15 kips > 28.55 kips \( \therefore \) Use \( R_n = 28.55 \text{ kips for this bolt.} \)

For the other holes,

\[ l_c = s - h = 3 - \frac{13}{16} = 2.188 \text{ in.} \]

\[ R_n = 1.2 l_c F_u = 1.2(2.188)(\frac{3}{8})(58) = 57.11 \text{ kips plug shear} \]

**Crush plate limit**

Upper limit = 2.4 d t F_a = 2.4 \left( \frac{3}{4} \right) \left( \frac{3}{8} \right)(58) = 39.15 \text{ kips} \( \therefore \) Use \( R_n = 39.15 \text{ kips for this bolt.} \)

The shearing strength is less than the bearing strength at each hole, so the nominal strength based on shear and bearing is

\[ R_n = 4(23.86) = 95.44 \text{ kips} \]
### TABLE J3.2
Nominal Strength of Fasteners and Threaded Parts, ksi (MPa)

<table>
<thead>
<tr>
<th>Description of Fasteners</th>
<th>Nominal Tensile Strength, $F_{nt}$, ksi (MPa)$^{[a]}$</th>
<th>Nominal Shear Strength in Bearing-Type Connections, $F_{nv}$, ksi (MPa)$^{[b]}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A307 bolts</td>
<td>45 (310)</td>
<td>27 (188)$^{[c][d]}$</td>
</tr>
<tr>
<td>Group A (e.g., A325) bolts, when threads are not excluded from shear planes</td>
<td>90 (620)</td>
<td>54 (372)$^{[d/4]}$</td>
</tr>
<tr>
<td>Group A (e.g., A325) bolts, when threads are excluded from shear planes</td>
<td>90 (620)</td>
<td>68 (457)$^{[d/4]}$</td>
</tr>
<tr>
<td>Group B (e.g., A490) bolts, when threads are not excluded from shear planes</td>
<td>113 (780)</td>
<td>68 (457)</td>
</tr>
<tr>
<td>Group B (e.g., A490) bolts, when threads are excluded from shear planes</td>
<td>113 (780)</td>
<td>84 (579)</td>
</tr>
<tr>
<td>Threaded parts meeting the requirements of Section A3.4, when threads are not excluded from shear planes</td>
<td>$0.75F_u$</td>
<td>$0.450F_u$</td>
</tr>
<tr>
<td>Threaded parts meeting the requirements of Section A3.4, when threads are excluded from shear planes</td>
<td>$0.75F_u$</td>
<td>$0.563F_u$</td>
</tr>
</tbody>
</table>

$^{[a]}$ For high-strength bolts subject to tensile fatigue loading, see Appendix 3.

$^{[b]}$ For end loaded connections with a fastener pattern length greater than 38 in. (965 mm), $F_{nv}$ shall be reduced to 83.3% of the tabulated values. Fastener pattern length is the maximum distance parallel to the line of force between the centerline of the bolts connecting two parts with one faying surface.

$^{[c]}$ For A307 bolts the tabulated values shall be reduced by 1% for each $1/16$ in. (2 mm) over 5 diameters of length in the grip.

$^{[d]}$ Threads permitted in shear planes.

### 2. Size and Use of Holes

The maximum sizes of holes for bolts are given in Table J3.3 or Table J3.3M, except that larger holes, required for tolerance on location of anchor rods in concrete foundations, are permitted in column base details.

Standard holes or short-slotted holes transverse to the direction of the load shall be provided in accordance with the provisions of this specification, unless oversized holes, short-slotted holes parallel to the load, or long-slotted holes are approved.
Check the strength of the tension member.

**Tension on the gross area:**

\[ P_n = F_y A_g = 36 \left( 6 \times \frac{1}{2} \right) = 108.0 \text{ kips} \]

**Tension on the net area:** All elements of the cross section are connected, so shear lag is not a factor and \( A_e = A_n \). For the hole diameter, use

\[ h = d + \frac{1}{8} = \frac{3}{4} + \frac{1}{8} = \frac{7}{8} \text{ in.} \]

The nominal strength is

\[ P_n = F_u A_e = F_u t (w_g - \Sigma h) = 58 \left( \frac{1}{2} \right) \left[ 6 - 2 \left( \frac{7}{8} \right) \right] = 123.3 \text{ kips} \]

**Block shear strength:** The failure block for the gusset plate has the same dimensions as the block for the tension member except for the thickness (Figure 7.13b). The gusset plate, which is the thinner element, will control. There are two shear-failure planes:

\[ A_{gv} = 2 \times \frac{3}{8} (3 + 1.5) = 3.375 \text{ in.}^2 \]

Since there are 1.5 hole diameters per horizontal line of bolts,

\[ A_{n} = 2 \times \frac{3}{8} \left[ 3 + 1.5 - 1.5 \left( \frac{7}{8} \right) \right] = 2.391 \text{ in.}^2 \]

For the tension area,

\[ A_{nt} = \frac{3}{8} \left( 3 - \frac{7}{8} \right) = 0.7969 \text{ in.}^2 \]

Since the block shear will occur in a gusset plate, \( U_{bs} = 1.0 \). From AISC Equation J4-5,

\[ R_e = 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \]

\[ = 0.6(58)(2.391) + 1.0(58)(0.7969) = 129.4 \text{ kips} \]

with an upper limit of

\[ 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} = 0.6(36)(3.375) + 1.0(58)(0.7969) = 119.1 \text{ kips} \]

The nominal block shear strength is therefore 119.1 kips.
Design Strength for LRFD

Bolt shear/bearing strength:

\[ \phi R_n = 0.75(95.44) = 71.6 \text{ kips} \]

Slip-critical strength:

\[ \phi R_n = 1.0(41.96) = 41.96 \text{ kips} \]

Tension on the gross area:

\[ \phi P_n = 0.90(108.0) = 97.2 \text{ kips} \]

Tension on the net area:

\[ \phi P_n = 0.75(123.3) = 92.5 \text{ kips} \]

Block shear strength:

\[ \phi R_n = 0.75(119.1) = 89.3 \text{ kips} \]

Of all the limit states investigated, the strength corresponding to slip is the smallest.

**ANSWER** Design strength = 42.0 kips.

---

Allowable Strength for ASD

Bolt shear/bearing strength:

\[ \frac{R_n}{\Omega} = \frac{95.44}{2.00} = 47.7 \text{ kips} \]

Slip-critical strength:

\[ \frac{R_n}{\Omega} = \frac{41.96}{1.50} = 28.0 \text{ kips} \]

Tension on the gross area:

\[ \frac{P_n}{\Omega_t} = \frac{108.0}{1.67} = 64.7 \text{ kips} \]

Tension on the net area:

\[ \frac{P_n}{\Omega_t} = \frac{123.3}{2.00} = 61.7 \text{ kips} \]

Block shear strength:

\[ \frac{R_n}{\Omega} = \frac{119.1}{2.00} = 59.6 \text{ kips} \]

Of all the limit states investigated, the strength corresponding to slip is the smallest.

**ANSWER** Allowable strength = 28.0 kips.
Tables for Bolt Strength

*Manual* Tables 7-1 through 7-5 give values for bolt shear, tensile, and slip-critical strengths and bearing strength at bolt holes. Their use will be illustrated in Example 7.5.

Summary: 3/4" Group A - N Single Shear  \( a = 3'' \)  \( L_e = 1.5'' \)  \( \phi_{r_n} = 17.9 \text{ kips/bolt} \)  \( \phi_{r_n} = 9.49 \text{ kips/bolt} \)  
\( \phi_{r_n} = 78.3 \text{ kips/bolt} \)  \( \phi_{r_n} = 44.0 \text{ kips/bolt} \)  

**Example 7.5**

Determine the strength of the connection of Example 7.4 based on the limit states of shear, slip-critical, and bearing strengths. Use LRFD.

**Solution**

1. **Bolt shear strength:** From *Manual* Table 7-1, for 3/4-inch Group A bolts, type N (threads included in shear plane), and S (single shear),

   \[ \phi_{r_n} = 17.9 \text{ kips/bolt} \]

   (These tables use a lowercase \( r \) to denote the strength of an individual bolt.)

2. **Slip-critical strength:** From *Manual* Table 7-3 (for Group A bolts and \( \mu = 0.30 \)), use STD (standard hole) and S (single shear). The slip-critical strength is

   \[ \phi_{r_n} = 9.49 \text{ kips/bolt} \]

3. **Bearing strength:** There are two tables in the *Manual* available for bearing strength: Table 7-4 for strength based on bolt spacing and Table 7-5 for strength based on bolt edge distance.

   **Inner bolts:** From Table 7-4, for STD (standard hole), \( F_u = 58 \text{ ksi} \), the bearing strength is \( \phi_{r_n} = 78.3 \text{ kips/bolt per inch of connected part thickness} \). Therefore, for the gusset plate (the thinner part), the strength is

   \[ \phi_{r_n} = 78.3 \text{ kips/bolt} \]

   **Edge bolts:** From Table 7-5, only two edge distances are given: 1¼ inches and 2 inches. Our edge distance is 1.5 inches. We can conservatively use 1¼ inches, and if the strength is not adequate, we can manually compute the bearing strength. The gusset plate bearing strength is therefore

   \[ \phi_{r_n} = 44.0 \text{ kips/bolt} \]

Even with a conservative estimate of the bearing strength, the slip-critical strength controls.

**Answer**

The strength of the connection based on the limit states investigated is

\[ \phi R_n = 4(9.49) = 38.0 \text{ kips}. \]
### Table 7-1
Available Shear Strength of Bolts, kips

<table>
<thead>
<tr>
<th>Nominal Bolt Diameter, ( d ), in.</th>
<th>( \frac{3}{8} )</th>
<th>( \frac{1}{2} )</th>
<th>( \frac{3}{8} )</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal Bolt Area, in.(^2)</td>
<td>0.307</td>
<td>0.442</td>
<td>0.601</td>
<td>0.785</td>
</tr>
<tr>
<td><strong>ASTM Desig.</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Thread Cond.</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>ASD</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>LRFD</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Loading</strong></td>
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<td></td>
</tr>
<tr>
<td><strong>ASD</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>LRFD</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Group A</strong></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>N</td>
<td>27.0</td>
<td>40.5</td>
<td>S</td>
<td></td>
</tr>
<tr>
<td>X</td>
<td>34.0</td>
<td>51.0</td>
<td>D</td>
<td></td>
</tr>
<tr>
<td>N</td>
<td>34.0</td>
<td>51.0</td>
<td>S</td>
<td></td>
</tr>
<tr>
<td>X</td>
<td>42.0</td>
<td>63.0</td>
<td>D</td>
<td></td>
</tr>
<tr>
<td>A307</td>
<td>13.5</td>
<td>20.3</td>
<td>S</td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>8.29</td>
<td>12.5</td>
<td>D</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Nominal Bolt Diameter, ( d ), in.</th>
<th>( 1\frac{1}{8} )</th>
<th>( 1\frac{1}{4} )</th>
<th>( 1\frac{3}{8} )</th>
<th>( 1\frac{1}{2} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal Bolt Area, in.(^2)</td>
<td>0.994</td>
<td>1.23</td>
<td>1.48</td>
<td>1.77</td>
</tr>
<tr>
<td><strong>ASTM Desig.</strong></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Thread Cond.</strong></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td><strong>ASD</strong></td>
<td></td>
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</tr>
<tr>
<td><strong>LRFD</strong></td>
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</tr>
<tr>
<td><strong>Loading</strong></td>
<td></td>
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</tr>
<tr>
<td><strong>ASD</strong></td>
<td></td>
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<td></td>
</tr>
<tr>
<td><strong>LRFD</strong></td>
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</tr>
<tr>
<td><strong>Group A</strong></td>
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</tr>
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<td>27.0</td>
<td>40.5</td>
<td>S</td>
<td></td>
</tr>
<tr>
<td>X</td>
<td>34.0</td>
<td>51.0</td>
<td>D</td>
<td></td>
</tr>
<tr>
<td>N</td>
<td>34.0</td>
<td>51.0</td>
<td>S</td>
<td></td>
</tr>
<tr>
<td>X</td>
<td>42.0</td>
<td>63.0</td>
<td>D</td>
<td></td>
</tr>
<tr>
<td>A307</td>
<td>13.5</td>
<td>20.3</td>
<td>S</td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>8.29</td>
<td>12.5</td>
<td>D</td>
<td></td>
</tr>
</tbody>
</table>

\( \Omega = 2.00 \)  \( \phi = 0.75 \)

**American Institute of Steel Construction**
### Table 7-3

**Slip-Critical Connections**

*Available Shear Strength, kips (Class A Faying Surface, $\mu = 0.30$)*

<table>
<thead>
<tr>
<th>Hole Type</th>
<th>Loading</th>
<th>5/8</th>
<th>3/4</th>
<th>7/8</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Nominal Bolt Diameter, $d$, in.</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>28</td>
<td>39</td>
<td>51</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$r_n/\Omega$</td>
<td>$\phi r_n$</td>
<td>$r_n/\Omega$</td>
<td>$\phi r_n$</td>
<td>$r_n/\Omega$</td>
<td>$\phi r_n$</td>
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<td>ASD</td>
<td>LRFD</td>
<td>ASD</td>
<td>LRFD</td>
<td>ASD</td>
<td>LRFD</td>
</tr>
<tr>
<td>STD/SSLT</td>
<td>S</td>
<td>4.29</td>
<td>6.44</td>
<td>6.33</td>
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</tr>
<tr>
<td></td>
<td>D</td>
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<td>12.9</td>
<td>12.7</td>
<td>18.0</td>
</tr>
<tr>
<td>OVS/SSLP</td>
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<td>5.47</td>
<td>5.39</td>
<td>8.07</td>
</tr>
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<td></td>
<td>D</td>
<td>7.32</td>
<td>10.9</td>
<td>10.8</td>
<td>16.1</td>
</tr>
<tr>
<td>LSL</td>
<td>S</td>
<td>3.01</td>
<td>4.51</td>
<td>4.44</td>
<td>6.64</td>
</tr>
<tr>
<td></td>
<td>D</td>
<td>6.02</td>
<td>9.02</td>
<td>8.87</td>
<td>13.3</td>
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</table>

<table>
<thead>
<tr>
<th>Hole Type</th>
<th>Loading</th>
<th>1 1/8</th>
<th>1 1/4</th>
<th>1 3/8</th>
<th>1 1/2</th>
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</thead>
<tbody>
<tr>
<td><strong>Nominal Bolt Diameter, $d$, in.</strong></td>
<td></td>
<td></td>
<td></td>
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<td>56</td>
<td>71</td>
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<td>$\phi r_n$</td>
<td>$r_n/\Omega$</td>
<td>$\phi r_n$</td>
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<td>ASD</td>
<td>LRFD</td>
<td>ASD</td>
<td>LRFD</td>
</tr>
<tr>
<td>STD/SSLT</td>
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<td>16.0</td>
<td>24.1</td>
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<td>38.0</td>
<td>32.1</td>
<td>48.1</td>
</tr>
<tr>
<td>OVS/SSLP</td>
<td>S</td>
<td>10.8</td>
<td>16.1</td>
<td>13.7</td>
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<td></td>
<td>D</td>
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<td>32.3</td>
<td>27.4</td>
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</tr>
<tr>
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<td>S</td>
<td>8.87</td>
<td>13.3</td>
<td>11.2</td>
<td>16.8</td>
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<tr>
<td></td>
<td>D</td>
<td>17.7</td>
<td>26.6</td>
<td>22.5</td>
<td>33.7</td>
</tr>
</tbody>
</table>

STD = standard hole  
OVS = oversized hole  
SSLT = short-slotted hole transverse to the line of force  
SSLP = short-slotted hole parallel to the line of force  
LSL = long-slotted hole transverse or parallel to the line of force

*S = single shear  
D = double shear

Note: Slip-critical bolt values assume no more than one filler has been provided for bolts have been added to distribute loads in the fillers. See AISC Specification Sections J3.8 and J5 for provisions when fillers are present.

For Class B faying surfaces, multiply the tabulated available strength by 1.67.
### Table 7-4
Available Bearing Strength at Bolt Holes
Based on Bolt Spacing

#### kips/in. thickness

<table>
<thead>
<tr>
<th>Bolt Type</th>
<th>Hole Spacing, s, in.</th>
<th>$F_w$, ksi</th>
<th>$F_u$, ksi</th>
<th>$F_p$, ksi</th>
<th>$F_s$, ksi</th>
<th>$F_o$, ksi</th>
<th>$F_i$, ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>STD</td>
<td>2.5</td>
<td>35.2, 51.1</td>
<td>42.3, 60.0</td>
<td>49.9, 72.9</td>
<td>57.6, 83.7</td>
<td>65.3, 93.8</td>
<td></td>
</tr>
<tr>
<td>SSLT</td>
<td>3 in.</td>
<td>35.2, 51.1</td>
<td>42.3, 60.0</td>
<td>49.9, 72.9</td>
<td>57.6, 83.7</td>
<td>65.3, 93.8</td>
<td></td>
</tr>
<tr>
<td>SSLP</td>
<td>3 in.</td>
<td>33.3, 46.3</td>
<td>40.4, 52.1</td>
<td>47.1, 59.7</td>
<td>55.3, 70.7</td>
<td>63.5, 81.5</td>
<td></td>
</tr>
<tr>
<td>OVS</td>
<td>3 in.</td>
<td>33.3, 46.3</td>
<td>40.4, 52.1</td>
<td>47.1, 59.7</td>
<td>55.3, 70.7</td>
<td>63.5, 81.5</td>
<td></td>
</tr>
<tr>
<td>LSLP</td>
<td>3 in.</td>
<td>33.3, 46.3</td>
<td>40.4, 52.1</td>
<td>47.1, 59.7</td>
<td>55.3, 70.7</td>
<td>63.5, 81.5</td>
<td></td>
</tr>
</tbody>
</table>

#### Notes:
- WHEN PLATE CRUSHING STARTS TO CONTROL, $s > s_{full}$.
- $s_{full}$ is the minimum spacing for full bearing strength.

#### Spacing for full bearing strength

<table>
<thead>
<tr>
<th>STD, SSLT, LSLT</th>
<th>3/16</th>
<th>3/8</th>
<th>1/4</th>
</tr>
</thead>
<tbody>
<tr>
<td>OVS</td>
<td>3/8</td>
<td>7/16</td>
<td>1/4</td>
</tr>
<tr>
<td>SSLP</td>
<td>3/8</td>
<td>7/16</td>
<td>3/16</td>
</tr>
<tr>
<td>LSLP</td>
<td>3/16</td>
<td>7/16</td>
<td>3/16</td>
</tr>
</tbody>
</table>

#### Minimum Spacing

| STD = standard hole |
| SSLT = short-slotted hole oriented transverse to the line of force |
| SSLP = short-slotted hole oriented parallel to the line of force |
| OVS = oversized hole |
| LSLP = long-slotted hole oriented parallel to the line of force |
| LSLT = long-slotted hole oriented transverse to the line of force |

<table>
<thead>
<tr>
<th>ASD</th>
<th>LRFD</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi = 0.75$</td>
<td>$\phi = 0.75$</td>
</tr>
</tbody>
</table>

Note: Spacing indicated for the center of the hole or slot to the center of the adjacent hole or slot in the line of force. Hole deformation is not considered. When hole deformation is not considered, see AISC Specification Section J.3.10.

---

**AMERICAN INSTITUTE OF STEEL CONSTRUCTION**
## Table 7-5

Available Bearing Strength at Bolt Holes

Based on Edge Distance:

kips/in. thickness of plate

<table>
<thead>
<tr>
<th>Hole Type</th>
<th>Edge Distance</th>
<th>End Bolts</th>
</tr>
</thead>
<tbody>
<tr>
<td>STD</td>
<td>1(\frac{1}{4})</td>
<td></td>
</tr>
<tr>
<td>SSLT</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>SSLP</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>OVS</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>LSLP</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>LSLT</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>STD, SSLT, OVS, LSLP</td>
<td>(L_o \geq L_o_{full})</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Hole Diameter, (d), in.</th>
<th>Nominal Bolt Diameter, (d), in.</th>
<th>(\phi_d) asd</th>
<th>LRFD</th>
<th>(\phi_{d/4}) asd</th>
<th>LRFD</th>
<th>(\phi_{d/2}) asd</th>
<th>LRFD</th>
<th>(\phi_{d/3}) asd</th>
<th>LRFD</th>
<th>(\phi_{d/4}) asd</th>
<th>LRFD</th>
</tr>
</thead>
<tbody>
<tr>
<td>5/8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2(\frac{1}{4})</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3/4</td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
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<td></td>
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<td></td>
</tr>
<tr>
<td>7/8</td>
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</tr>
<tr>
<td>1</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Notes:**
- STD = standard hole
- SSLT = short-slotted hole oriented transverse to the line of force
- SSLP = short-slotted hole oriented parallel to the line of force
- OVS = oversized hole
- LSLP = long-slotted hole oriented parallel to the line of force
- LSLT = long-slotted hole oriented transverse to the line of force

---

### ASD = Standard Design

<table>
<thead>
<tr>
<th>(\Omega = 2.00)</th>
<th>LRFD</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.75</td>
<td></td>
</tr>
</tbody>
</table>
In Example 7.5, the tensile strength was not required. The use of Manual Table 7-2 for this value is intuitive and is similar to Table 7-1 for shear strength. The bearing-strength tables, however, contain some values that require explanation. In Table 7-4, a value \( s_{full} \) is defined as the spacing \( s \) at which the full bearing strength is available. This full bearing strength is the upper limit in AISC Equation J3-6a; that is, \( \phi r_n = \phi (2.4 dt F_u) \). Similarly, Table 7-5 defines a value \( L_e \) which is the edge distance at which the full bearing strength, \( \phi r_n = \phi (2.4 dt F_u) \), can be used.

7.7 DESIGN EXAMPLES

Although an elementary bolt design was illustrated in Example 7.3, most examples so far have been review or analysis. Examples 7.5–7.7 demonstrate more realistic design situations. Manual Tables 7-1 through 7-5 will be used in these examples.

**EXAMPLE 7.6**

A 5/8-inch-thick tension member is connected to two 1/4-inch splice plates, as shown in Figure 7.14. The loads shown are service loads. A36 steel and 3/4-inch-diameter, Group A bolts will be used. If slip is permissible, how many bolts are required? Each bolt centerline shown represents a row of bolts in the direction of the width of the plates.

**SOLUTION**

**All Bolts Shear:**

\[ \phi r_n = 24.9 \text{ kips/bolt} \]

**LRFD**

**Shear:** These bolts are in double shear. From Manual Table 7-1,

\[ \phi r_n = 24.9 \text{ kips/bolt} \]

\[ \frac{f}{m} = \frac{l}{d} \]

**FIGURE 7.14**

- 2(1/4") = 1/2" thick
- 1(5/8") = 5/8" thick

The 1/4" splice plates are thinner at 1/2"
Assume that shear controls and then check bearing. The required strength of the connection is

\[ P_u = 1.2D + 1.6L = 1.2(30) + 1.6(25) = 76 \text{ kips} \]

Number of bolts required = \[
\frac{\text{required strength}}{\text{strength per bolt}} = \frac{76}{24.9} = 3.05 \text{ bolts}
\]

Try four bolts, two per line, on each side of the splice.

**Bearing:** The bearing force on the \( \frac{3}{8} \)-inch-thick tension member will be twice as large as the bearing force on each of the \( \frac{1}{4} \)-inch splice plates. Because the total load on the splice plates is the same as the load on the tension member, for the splice plates to be critical the total splice plate thickness must be less than the thickness of the tension member—and it is. For the inner holes and a spacing of 3 inches, *Manual* Table 7-4 gives

\[ \phi_n = 65.3t = 65.3\left(\frac{1}{4} + \frac{1}{4}\right) = 32.7 \text{ kips/bolt} \]

For the holes nearest the edge of the plate, we will conservatively use an edge distance of \( 1\frac{3}{4} \) inches to find the bearing strength from *Manual* Table 7-5.

\[ \phi_n = 47.3t = 47.3\left(\frac{1}{4} + \frac{1}{4}\right) = 23.7 \text{ kips/bolt} \]

To obtain the strength of the connection based on shear and bearing, we consider the total strength of the connection to be the sum of the minimum strengths at each bolt location (AISC J3.6 and J3.10 User Notes). Thus,

\[ \phi R_n = 2(\text{edge bolt strength}) + 2(\text{inner bolt strength}) = 2(23.7) + 2(24.9) = 97.2 \text{ kips} > 76 \text{ kips} \quad (\text{OK}) \]

**ANSWER**

Use four bolts, two per line, on each side of the splice. A total of eight bolts will be required for the connection.

**ASD SOLUTION**

**Shear:** These bolts are in double shear. From *Manual* Table 7-1,

\[ \frac{r_n}{\Omega} = 16.6 \text{ kips/bolt} \]

Assume that shear controls and then check bearing. The required strength of the connection is

\[ P_a = D + L = 30 + 25 = 55 \text{ kips} \]

Number of bolts required = \[
\frac{\text{required strength}}{\text{strength per bolt}} = \frac{55}{16.6} = 3.31 \text{ bolts}
\]

(OK)
Try four bolts, two per line, on each side of the splice.

Bearing: The bearing force on the ¾-inch-thick tension member will be twice as large as the bearing force on each of the ¼-inch splice plates. Because the total load on the splice plates is the same as the load on the tension member, and the total splice plate thickness is less than that of the tension member, bearing on the splice plates controls. For the inner holes and a spacing of 3 inches, Manual Table 7-4 gives

\[
\frac{r_n}{\Omega} = 43.5t = 43.5\left(\frac{1}{4} + \frac{1}{4}\right) = 21.8 \text{kips/bolt}
\]

For the holes nearest the edge of the plate, we will conservatively use an edge distance of 1¼ inches to find the bearing strength from Manual Table 7-5.

\[
\frac{r_n}{\Omega} = 31.5t = 31.5\left(\frac{1}{4} + \frac{1}{4}\right) = 15.8 \text{kips/bolt}
\]

The total strength of the connection is the sum of the minimum strengths at each bolt location (AISC J3.6 and J3.10 User Notes). Thus,

\[
\frac{R_n}{\Omega} = 2(\text{edge bolt strength}) + 2(\text{inner bolt strength})
\]

\[
= 2(15.8) + 2(21.8) = 75.2 > 55 \text{kips} \quad \text{(OK)}
\]

**ANSWER** Use four bolts, two per line, on each side of the splice. A total of eight bolts will be required for the connection.

---

**EXAMPLE 7.7**

The C8 × 18.75 shown in Figure 7.15 has been selected to resist a service dead load of 18 kips and a service live load of 54 kips. It is to be attached to a ¾-inch gusset plate with ¾-inch-diameter, Group A bolts. Assume that the threads are in the plane of shear and that slip of the connection is permissible. Determine the number and required layout of bolts such that the length of connection L is reasonably small. A36 steel is used.

**LRFD SOLUTION**

The factored load is

\[
1.2D + 1.6L = 1.2(18) + 1.6(54) = 108.0 \text{kips}
\]
We will select the number of bolts based on shear and verify that the bearing strength is adequate once a final bolt layout has been determined. From *Manual* Table 7-1, the shear strength is

\[ \phi r_n = 24.4 \text{ kips/bolt} \]

Number of bolts required = \( \frac{\text{required strength}}{\text{strength per bolt}} = \frac{108}{24.4} = 4.43 \) bolts

Although five bolts will furnish enough capacity, try six bolts so that a symmetrical layout with two gage lines of three bolts each can be used, as shown in Figure 7.16. (Two gage lines are used to minimize the length of the connection.) We do not know whether the design of this tension member was based on the assumption of one line or two lines of fasteners; the tensile capacity of the channel with two lines of bolts must be checked before proceeding. For the gross area,

\[ P_n = F_n A_g = 36(5.51) = 198.4 \text{ kips} \]

The design strength is

\[ \phi P_n = 0.90(198.4) = 179 \text{ kips} \]

Tension on the effective net area:

\[ A_n = 5.51 - 2\left(\frac{7}{8} + \frac{1}{8}\right)(0.487) = 4.536 \text{ in.}^2 \]

The exact length of the connection is not yet known, so Equation 3.1 for \( U \) cannot be used. Assume a conservative value of \( U = 0.60 \).

\[ A_e = A_n U = 4.536(0.60) = 2.722 \text{ in.}^2 \]

\[ P_n = F_n A_e = 58(2.722) = 157.9 \text{ kips} \]

\[ \phi_P P_n = 0.75(157.9) = 118 \text{ kips} \quad \text{(controls)} \]
The member capacity is therefore adequate with two gage lines of bolts. Check the spacing and edge distance transverse to the load. From AISC J3.3,

\[
\text{Minimum spacing} = 2.667 \left( \frac{7}{8} \right) = 2.33 \text{ in.}
\]

From AISC Table J3.4,

\[
\text{Minimum edge distance} = 1\frac{1}{8} \text{ in.}
\]

A spacing of 3 inches and edge distances of 2\(\frac{1}{2}\) inches will be used transverse to the load.

The minimum length of the connection can be established by using the minimum permissible spacing and edge distances in the longitudinal direction. The minimum spacing in any direction is \(2\frac{1}{2}d = 2.33\) in. Try \(2\frac{1}{2}\) in. The minimum edge distance in any direction is \(1\frac{1}{8}\) in. So that we can use the values in Manual Table 7-5, try \(1\frac{1}{4}\) in. These distances will now be used to check the bearing strength of the connection.

The gusset plate is the thinner of the two parts in bearing and will control. For the inner holes, conservatively use the spacing of \(2\frac{1}{2}d_b\) found in Manual Table 7-4. The bearing strength based on this spacing is

\[
\phi_r = 72.9t = 72.9 \left( \frac{3}{8} \right) = 27.3 \text{ kips/bolt}
\]

For the holes nearest the edge of the gusset plate, use Manual Table 7-5 and an edge distance of \(1\frac{1}{4}\) inches.

\[
\phi_r = 40.8t = 40.8 \left( \frac{3}{8} \right) = 15.3 \text{ kips/bolt}
\]

Using the minimum of shear and bearing strengths for each bolt location, the total connection strength is

\[
\phi R_r = 2(\text{edge bolt strength}) + 4(\text{inner bolt strength})
\]

\[
= 2(15.3) + 4(24.4) = 128 \text{ kips} > 108 \text{ kips} \quad \text{(OK)}
\]

The tentative connection design is shown in Figure 7.17 and will now be checked for block shear in the gusset plate (the geometry of the failure block in the channel is identical, but the gusset plate is thinner).

**Shear areas:**

\[
A_{gw} = 2 \times \frac{3}{8} (2.5 + 2.5 + 1.25) = 4.688 \text{ in.}^2
\]

\[
A_{ww} = 2 \times \frac{3}{8} (6.25 - 2.5(1.0)) = 2.813 \text{ in.}^2
\]
Tension area:

\[ A_{nt} = \frac{3}{8} (3 - 1.0) = 0.7500 \text{ in.}^2 \]

For this type of block shear, \( U_{bs} = 1.0 \). From AISC Equation J4-5,

\[ R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt} \]
\[ = 0.6(58)(2.813) + 1.0(58)(0.7500) = 141.4 \text{ kips} \]

with an upper limit of

\[ 0.6F_y A_{gv} + U_{bs} F_u A_{nt} = 0.6(36)(4.688) + 1.0(58)(0.7500) = 144.8 \text{ kips} \]

The nominal block shear strength is therefore 141.4 kips, and the design strength is

\[ \phi R_n = 0.75(141.4) = 106 \text{ kips} \quad < 108 \text{ kips} \quad \text{(N.G.)} \]

The simplest way to increase the block shear strength for this connection is to increase the shear areas by increasing the bolt spacing or the edge distance, we will increase the spacing. Although the required spacing can be determined by trial and error, it can be solved for directly, which we do here. If we assume that the upper limit in AISC Equation J4-5 does not control, the required design strength is

\[ \phi R_n = 0.75(0.6F_u A_{nv} + U_{bs} F_u A_{nt}) \]
\[ = 0.75[0.6(58)A_{nv} + 1.0(58)(0.7500)] = 108 \text{ kips} \]

Required \( A_{nv} = 2.888 \text{ in.}^2 \)

\[ A_{nv} = \frac{3}{8}[s + s + 1.25 - 2.5(1.0)](2) = 2.888 \text{ in.}^2 \]

Required \( s = 2.55 \text{ in.} \)
\[ \therefore \text{ Use } s = 2\frac{3}{4} \text{ in.} \]
Compute the actual block shear strength.

\[ A_{gv} = 2 \times \frac{3}{8} (2.75 + 2.75 + 1.25) = 5.063 \text{ in.}^2 \]

\[ A_{nv} = 5.063 - \frac{3}{8} (2.5 \times 1.0)(2) = 3.188 \text{ in.}^2 \]

\[ \phi R_n = 0.75(0.6 F_y A_{nv} + U_{bs} F_y A_{nt}) \]

\[ = 0.75[0.6(36)(3.188) + 1.0(58)(0.750)] = 116 \text{ kips} > 108 \text{ kips} \quad \text{(OK)} \]

Check the upper limit:

\[ \phi[0.6 F_y A_{gv} + U_{bs} F_y A_{nt}] = 0.75[0.6(36)(5.063) + 1.0(58)(0.7500)] \]

\[ = 115 \text{ kips} < 116 \text{ kips} \]

Therefore, the upper limit controls, but the strength is still adequate.

Using the spacing and edge distances selected, the minimum length is

\[ L = 1\frac{1}{4} \text{ in. at the end of the channel} \]

\[ + 2 \text{ spaces at } 2\frac{3}{4} \text{ in.} \]

\[ + 1\frac{3}{4} \text{ in. at the end of the gusset plate} \]

\[ = 8 \text{ in. total} \]

**ANSWER**

Use the connection detail as shown in Figure 7.18.

**ASD SOLUTION**

The total load is

\[ P_a = D + L = 18 + 54 = 72 \text{ kips} \]

We will select the number of bolts based on shear and verify that the bearing strength is adequate once a final bolt layout has been determined. From *Manual* Table 7-1, the shear strength is

\[ \frac{r_n}{\Omega} = 16.2 \text{ kips/bolt} \]
The number of bolts required is

\[
\frac{72}{16.2} = 4.44 \text{ bolts}
\]

Although five bolts will furnish enough capacity, try six bolts so that a symmetrical layout with two gage lines of three bolts each can be used, as shown in Figure 7.16. (Two gage lines are used to minimize the length of the connection.) We do not know whether the design of this tension member was based on the assumption of one line or two lines of fasteners; the tensile capacity of the channel with two lines of bolts must be checked before proceeding. For the gross area,

\[
P_n = F_yA_g = 36(5.51) = 198.4 \text{ kips}
\]

The allowable strength is

\[
\frac{P_n}{\Omega_t} = \frac{198.4}{1.67} = 119 \text{ kips}
\]

Tension on the effective net area:

\[
A_n = 5.51 - 2\left(\frac{7}{8} + \frac{1}{8}\right)(0.487) = 4.536 \text{ in.}^2
\]

The exact length of the connection is not yet known, so Equation 3.1 for \( U \) cannot be used. Assume a conservative value of \( U = 0.60 \).

\[
A_e = A_nU = 4.536(0.60) = 2.722 \text{ in.}^2
\]

\[
P_n = F_uA_e = 58(2.722) = 157.9 \text{ kips}
\]

\[
\frac{P_n}{\Omega_t} = \frac{157.9}{2.00} = 79.0 \text{ kips} \quad \text{(controls)}
\]

The member capacity is therefore adequate with two gage lines of bolts.

Check the spacing and edge distance requirements transverse to the load. From AISC J3.3,

Minimum spacing = \( 2.667 \left( \frac{7}{8} \right) = 2.33 \text{ in.} \)

From AISC Table J3.4,

Minimum edge distance = 1¼ in.

A spacing of 3 inches and edge distances of 2½ inches will be used transverse to the load.

The minimum length of the connection can be established by using the minimum permissible spacing and edge distances in the longitudinal direction. The minimum spacing is \( 2\frac{1}{2}d = 2.33 \) inches. Try 2½ inches. The minimum edge distance in any direction is 1¼ inches. So that we can use the values in Manual Table 7-5, try 1¼ inches. These distances now will be used to check the bearing strength of the
connection. The gusset plate is the thinner of the two parts in bearing and will control. For the inner holes, conservatively use the spacing of $2\frac{3}{4}d_e$ in Manual Table 7-4. The bearing strength based on this spacing is

$$\frac{r_n}{\Omega} = 48.6t = 48.6 \left( \frac{3}{8} \right) = 18.2 \text{ kips/bolt}$$

For the holes nearest the edge of the plate, the bearing strength from Table 7-5 is

$$\frac{r_n}{\Omega} = 27.2t = 27.2 \left( \frac{3}{8} \right) = 10.2 \text{ kips/bolt}$$

Using the minimum of shear and bearing strengths for each bolt location, the total connection strength is

$$\frac{R_n}{\Omega} = 2(\text{edge bolt strength}) + 4(\text{inner bolt strength})$$

$$= 2(10.2) + 4(16.2) = 85.2 \text{ kips} > 72 \text{ kips} \quad \text{(OK)}$$

The tentative connection design is shown in Figure 7.17 and will now be checked for block shear in the gusset plate (the geometry of the failure block in the channel is identical, but the gusset plate is thinner).

**Shear areas:**

$$A_{sv} = 2 \times \frac{3}{8}(2.5 + 2.5 + 1.25) = 4.688 \text{ in}^2$$

$$A_{nv} = 2 \times \frac{3}{8}[6.25 - 2.5(1.0)] = 2.813 \text{ in}^2$$

**Tension area:**

$$A_{nt} = \frac{3}{8}(3 - 1.0) = 0.7500 \text{ in}^2$$

For this type of block shear, $U_{bs} = 1.0$. From AISC Equation J4-5,

$$R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt}$$

$$= 0.6(58)(2.813) + 1.0(58)(0.75) = 141.4 \text{ kips}$$

with an upper limit of

$$0.6F_y A_{sv} + U_{bs} F_u A_{nt} = 0.6(36)(4.688) + 1.0(58)(0.75) = 144.8 \text{ kips}$$

The nominal block shear strength is therefore 141.4 kips, and the allowable strength is

$$\frac{R_n}{\Omega} = \frac{141.4}{2.00} = 70.7 \text{ kips} < 72 \text{ kips} \quad \text{(N.G.)}$$

The simplest way to increase the block shear strength for this connection is to increase the shear areas by increasing the bolt spacing or the edge distance; we will
increase the spacing. Although the required spacing can be determined by trial and error, it can be solved for directly, which we do here. If we assume that the upper limit in AISC Equation J4-5 does not control, the required allowable strength is

\[
\frac{R_u}{\Omega} = \frac{0.6F_u A_{nv} + U_{bs} F_u A_{nt}}{\Omega} = \frac{0.6(58)A_{nv} + 1.0(58)(0.7500)}{2.00} = 72 \text{ kips}
\]

Required \( A_{nv} = 2.888 \text{ in.}^2 \)

\[
A_{nv} = \frac{3}{8} [s + s + 1.25 - 2.5(1.0)](2) = 2.888 \text{ in.}^2
\]

Required \( s = 2.55 \text{ in.} \) \quad \therefore \text{ Use } s = 2\frac{3}{4} \text{ in.}

Compute the actual block shear strength.

\[
A_{gv} = 2 \times \frac{3}{8} (2.75 + 2.75 + 1.25) = 5.063 \text{ in.}^2
\]

\[
A_{nv} = 5.063 - \frac{3}{8} (2.5 \times 1.0)(2) = 3.188 \text{ in.}^2
\]

\[
\frac{R_u}{\Omega} = \frac{0.6F_u A_{nv} + U_{bs} F_u A_{nt}}{\Omega} = \frac{0.6(58)(3.188) + 1.0(58)(0.750)}{2.00} = 77.2 \text{ kips} > 72 \text{ kips} \quad \text{(OK)}
\]

Check the upper limit:

\[
\frac{0.6F_y A_{gv} + U_{bs} F_u A_{nt}}{\Omega} = \frac{0.6(36)(5.063) + 1.0(58)(0.7500)}{2.00} = 76.4 \text{ kips} < 77.2 \text{ kips}
\]

Therefore, the upper limit controls, but the strength is still adequate.

Using the spacing and edge distances selected, the minimum length is

\[
L = 1\frac{1}{4} \text{ in. at the end of the channel}
+ 2 \text{ spaces at } 2\frac{3}{4} \text{ in.}
+ 1\frac{1}{4} \text{ in. at the end of the gusset plate}
= 8 \text{ in. total}
\]

**Answer** Use the connection detail as shown in Figure 7.18.
The bolt layout in Example 7.7 is symmetrical with respect to the longitudinal centroidal axis of the member. Consequently, the resultant resisting force provided by the fasteners also acts along this line, and the geometry is consistent with the definition of a simple connection. If an odd number of bolts had been required and two rows had been used, the symmetry would not exist and the connection would be eccentric. In such cases, the designer has several choices: (1) ignore the eccentricity, assuming that the effects are negligible; (2) account for the eccentricity; (3) use a staggered pattern of fasteners that would preserve the symmetry; or (4) add an extra bolt and remove the eccentricity. Most engineers would probably choose the last alternative.

**EXAMPLE 7.8**

Use LRFD and design a 13-foot-long tension member and its connection for a service dead load of 8 kips and a service live load of 24 kips. No slip of the connection is permitted. The connection will be to a $\frac{3}{8}$-inch-thick gusset plate, as shown in Figure 7.19. Use a single angle for the tension member. Use Group A bolts and A572 Grade 50 steel for both the tension member and the gusset plate.

**SOLUTION**

The factored load to be resisted is

$$P_u = 1.2D + 1.6L = 1.2(8) + 1.6(24) = 48.0 \text{ kips}$$

Because the bolt size and layout will affect the net area of the tension member, we will begin with selection of the bolts. The strategy will be to select a bolt size for trial, determine the number required, and then try a different size if the number is too large or too small. Bolt diameters typically range from $\frac{1}{2}$ inch to $1\frac{1}{2}$ inches in $\frac{1}{8}$-inch increments.

**Try $\frac{5}{8}$-inch bolts:** From Manual Table 7-1, assuming that the threads are in the shear plane, the shear strength is

$$\phi r_n = 12.4 \text{ kips/bolt}$$

No slip is permitted, so this connection is slip-critical. We will assume class A surfaces, and for a $\frac{5}{8}$-inch-diameter Group A bolt, the minimum tension is $T_b = 19 \text{ kips}$.
(from AISC Table J3.1). From Manual Table 7-3, the slip-critical strength for one bolt is

\[ \phi r_n = 6.44 \text{ kips/bolt} \]

The slip-critical strength controls. We will determine the number of bolts based on this strength and check bearing after selecting the member (because the bearing strength cannot be computed until the member thickness is known). Hence

\[ \text{Number of bolts} = \frac{\text{total load}}{\text{load per bolt}} = \frac{48.0}{6.44} = 7.5 \text{ bolts} \]

Eight bolts will be required. Figure 7.20 shows two potential bolt layouts. Although either of these arrangements could be used, the connection length can be decreased by using a larger bolt size and fewer bolts.

**Try \( \frac{7}{8} \)-inch bolts:** From Manual Table 7-1, assuming that the threads are in the shear plane, the shear strength is

\[ \phi r_n = 24.3 \text{ kips/bolt} \]

From Manual Table 7-3, the slip-critical strength is

\[ \phi r_n = 13.2 \text{ kips/bolt} \quad \text{(controls)} \]

The number of \( \frac{7}{8} \)-inch bolts required is

\[ \frac{48.0}{13.2} = 3.6 \text{ bolts} \]

Four \( \frac{7}{8} \)-inch-diameter Group A bolts will be used. From AISC J3.3, the minimum spacing is

\[ s = 2.667d = 2.667 \left( \frac{7}{8} \right) = 2.33 \text{ in.} \quad \text{(or, preferably, } 3d = 3 \left( \frac{7}{8} \right) = 2.63 \text{ in.}) \]

From AISC Table J3.4, the minimum edge distance is

\[ L_e = 1\frac{1}{8} \text{ in.} \]
Try the layout shown in Figure 7.21 and select a tension member. The required gross area is

$$A_g \geq \frac{P_u}{0.90F_y} = \frac{48.0}{0.90(50)} = 1.07 \text{ in.}^2$$

and the required effective net area is

$$A_e \geq \frac{P_u}{0.75F_u} = \frac{48.0}{0.75(65)} = 0.985 \text{ in.}^2$$

The required minimum radius of gyration is

$$r_{min} = \frac{L}{300} = \frac{13(12)}{300} = 0.52 \text{ in.}$$

**Try an L3\(\frac{1}{2}\) × 2\(\frac{1}{2}\) × \(\frac{1}{4}\):**

$$A_g = 1.45 \text{ in.}^2 > 1.07 \text{ in.}^2 \quad \text{(OK)}$$

$$r_{min} = r_z = 0.541 \text{ in.} > 0.52 \text{ in.} \quad \text{(OK)}$$

For net area computation, use a hole diameter of \(\frac{7}{8} + \frac{1}{8} = 1.0 \text{ in.}\)

$$A_n = A_g - A_{\text{hole}} = 1.45 - \left(\frac{7}{8} + \frac{1}{8}\right)\left(\frac{1}{4}\right) = 1.2 \text{ in.}^2$$

Compute \(U\) with Equation 3.1:

$$U = 1 - \frac{\overline{x}}{L}$$

$$= 1 - \frac{0.607}{9} = 0.9326$$

where \(\overline{x} = 0.607\) inch for the long leg vertical. The effective net area is

$$A_e = A_nU = 1.2(0.9326) = 1.12 \text{ in.}^2 > 0.985 \text{ in.}^2 \quad \text{(OK)}$$
Now check the bearing strength. The edge distance for the angle is the same as the edge distance for the gusset plate and the angle is thinner than the gusset plate, so the angle thickness of $\frac{1}{4}$ inch will be used. For the inner holes and a spacing of 3 inches, the bearing strength from *Manual* Table 7-4 is

$$\phi r_n = 102t = 102\left(\frac{1}{4}\right) = 25.5 \text{ kips/bolt}$$

For the holes nearest the edge of the tension member, use *Manual* Table 7-5 and an edge distance of $\frac{1}{4}$ inches.

$$\phi r_n = 45.7t = 45.7\left(\frac{1}{4}\right) = 11.4 \text{ kips/bolt}$$

Using the minimum of shear and bearing strengths for each bolt location, the connection strength based on shear and bearing is

$$\phi R_n = \text{edge bolt strength} + 3(\text{inner bolt strength})$$

$$= 11.4 + 3(24.4) = 84.6 \text{ kips} > 48.0 \text{ kips} \quad \text{(OK)}$$

Now check block shear. With the bolts placed in the long leg at the usual gage distance (see Chapter 3, Figure 3.24), the failure block is as shown in Figure 7.22. The shear areas are

$$A_{g\nu} = \frac{1}{4}(1.25 + 9) = 2.563 \text{ in.}^2$$

$$A_{n\nu} = \frac{1}{4}[1.25 + 9 - 3.5(1.0)] = 1.688 \text{ in.}^2 \quad (3.5 \text{ hole diameters})$$

The tension area is

$$A_{n\text{t}} = \frac{1}{4}[1.5 - 0.5(1.0)] = 0.2500 \text{ in.}^2 \quad (0.5 \text{ hole diameter})$$

**FIGURE 7.22**

![Diagram showing the angles and bolt placement](image-url)

1¼”  3”  3”  3”  1¼”

2”

1¼”