When the resisting moment is equated to the applied moment, the resulting equation can be solved for the unknown bolt tensile force \( r_t \). (This method is the same as Case II in Part 7 of the Manual.)

\[
\begin{bmatrix}
  7 & 12
\end{bmatrix}
\]

**EXAMPLE 8.4**

A beam-to-column connection is made with a structural tee as shown in Figure 8.14. Eight \( \frac{3}{4} \)-inch-diameter, Group A, fully tightened bearing-type bolts are used to attach the flange of the tee to the column flange. Investigate the adequacy of this connection (the tee-to-column connection) if it is subjected to a service dead load of 20 kips and a service live load of 40 kips at an eccentricity of 2.75 inches. Assume that the bolt threads are in the plane of shear. All structural steel is A992.

**FIGURE 8.14**

\[
\begin{align*}
D &= 20^k \quad I = 40^k \\
\phi r_t &= 0.615 \left[ 1 - 0.58 \right] \quad 65 \text{ kips}
\end{align*}
\]

**LRFD SOLUTION**

Determine the shear and bearing strengths. The tables in Part 7 of the Manual will be used. From Table 7-1, the shear strength is \( (493c) \left[ 7-22 \right] \). For the inner bolts with a spacing of 3 inches, the bearing strength from Table 7-4 is \( (7-26) (493d) \).

- **Bolt Shear**: \( \phi r_t = 17.9 \) kips/bolt
- **Bolt Bearing**: \( \phi r_t = 87.8 t = 87.8(0.560) = 49.2 \) kips/bolt
- **Shearing plug & crushing interior bolts**
Eccentricity Normal to the Plane of the Faying Surface

<table>
<thead>
<tr>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>$r_{my} = \frac{P_u}{n}$ (7-9a)</td>
<td>$r_{av} = \frac{P_a}{n}$ (7-9b)</td>
</tr>
</tbody>
</table>

where $n$ is the number of bolts in the connection.

A trial position for the neutral axis can be selected at one-sixth of the total bracket depth, measured upward from the bottom (line X-X in Figure 7-6(a)). To provide for reasonable proportions and to account for the bending stiffness of the connection elements, the effective width of the compression block, $b_{eff}$, should be taken as

$$b_{eff} = 8t_f \leq b_f$$ (7-10)
where
\[ t_f = \text{lesser connection element thickness, in.} \]
\[ b_f = \text{connection element width, in.} \]

This effective width is valid for bracket flanges made from W-shapes, S-shapes, welded plates and angles. Where the bracket flange thickness is not constant, the average flange thickness should be used.

The assumed location of the neutral axis can be evaluated by checking static equilibrium assuming an elastic stress distribution. Equating the moment of the bolt area above the neutral axis with the moment of the compression block area below the neutral axis,

\[ (\Sigma A_b) y = b_{eff} \frac{d}{2} \]  
(7-11)

where:
\[ \Sigma A_b = \text{sum of the areas of all bolts above the neutral axis, in.}^2 \]
\[ y = \text{distance from line X-X to the CG of the bolt group above the neutral axis, in.} \]
\[ d = \text{depth of compression block, in.} \]

The value of \( d \) may then be adjusted until a reasonable equality exists.

Once the neutral axis has been located, the tensile force per bolt, \( r_{at} \) or \( r_{af} \), as illustrated in Figure 7-6(b), may be determined as

<table>
<thead>
<tr>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>( r_{at} = \left( \frac{P_{att} \cdot ec}{I_x} \right) A_b )</td>
<td>( r_{af} = \left( \frac{P_{att} \cdot ec}{I_s} \right) A_b )</td>
</tr>
</tbody>
</table>

where
\[ c = \text{distance from neutral axis to the most remote bolt in the group, in.} \]
\[ I_x = \text{combined moment of inertia of the bolt group and compression block about the neutral axis, in.}^4 \]

\[ \frac{(b_{eff})(d)^3}{3} + 2A_b y_1^2 + 2A_b y_2^2 + \ldots \]

Fig. 7-6. Location of neutral axis (NA) for out-of-plane eccentric loading using Case I.
Bolts above the neutral axis are subjected to the shear force, the tensile force, and the effect of prying action (see Part 9); bolts below the neutral axis are subjected to the shear force, \( r_{av} \) or \( r_{av} \), only.

**Case II—Neutral Axis at Center of Gravity**

This method provides a more direct, but also a more conservative result. As for Case I, the shear force per bolt, \( r_{av} \) or \( r_{av} \), due to the concentric force, \( P_u \) or \( P_u \), is determined as

<table>
<thead>
<tr>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>( r_{av} = \frac{P_u}{n} ) \hspace{1cm} (7-13a)</td>
<td>( r_{av} = \frac{P_u}{n} ) \hspace{1cm} (7-13b)</td>
</tr>
</tbody>
</table>

where \( n \) is the number of bolts in the connection.

The neutral axis is assumed to be located at the CG of the bolt group as illustrated in Figure 7-7. The bolts above the neutral axis are in tension and the bolts below the neutral axis are said to be in "compression." To obtain a more accurate result, a plastic stress distribution is assumed; this assumption is justified because this method is still more conservative than Case I. Accordingly, the tensile force in each bolt above the neutral axis, \( r_{at} \) or \( r_{at} \), due to the moment, \( P_a e \) or \( P_a e \), is determined as

<table>
<thead>
<tr>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>( r_{at} = \frac{P_a e}{n'd_m} ) \hspace{1cm} (7-14a)</td>
<td>( r_{at} = \frac{P_a e}{n'd_m} ) \hspace{1cm} (7-14b)</td>
</tr>
</tbody>
</table>

**Fig. 7-7. Location of neutral axis (NA) for out-of-plane eccentric loading using Case II.**
where

\[ n' = \text{number of bolts above the neutral axis} \]
\[ d_m = \text{moment arm between resultant tensile force and resultant compressive force, in.} \]

Bolts above the neutral axis are subjected to the shear force, the tensile force, and the effect of prying action (see Part 9); bolts below the neutral axis are subjected to the shear force, \( r_{au} \) or \( r_{av} \), only.

### SPECIAL CONSIDERATIONS FOR HOLLOW STRUCTURAL SECTIONS

#### Through-Bolting to HSS

Long bolts that extend through the entire HSS are satisfactory for shear connections that do not require a pretensioned installation. The flexibility of the walls of the HSS precludes installation of pretensioned bolts. Standard structural bolts may be used, although ASTM A449 bolts may be required for longer lengths. The bolts are designed for static shear and the only limit-state involving the HSS is bolt bearing. The available bearing strength is determined as \( \phi R_n \) or \( R_n/\Omega \), where

\[ R_n = 1.8nF_y t_{design} \]

\[ \phi = 0.75 \quad \Omega = 2.00 \]  

(7-15)

where

\[ n = \text{number of fasteners} \]
\[ d = \text{fastener diameter, in.} \]
\[ F_y = \text{specified minimum yield strength of HSS, ksi} \]
\[ t_{design} = \text{design wall thickness of HSS, in.} \]

#### Blind Bolts

Special fasteners are available that eliminate the need for access to install a nut (Korol et al., 1993; Henderson, 1996). The shank of the fastener is inserted through holes in the parts to be connected until the head bears on the outer ply (see Figure 7-8). In some cases, a special wrench is used on the open side to keep the outer part of the shank from rotating and simultaneously turn the threaded part of the shank. A wedge or other mechanism on the blind side causes the fixed part of the shank to expand and form a contact with the inside of the HSS. Some fasteners contain a break-off mechanism when the fastener is pretensioned. Recent versions of these fasteners meet the requirements for a pretensioned ASTM A325 bolt (Henderson, 1996) and could be used in slip-critical or tension conditions. HSS limit states are bolt bearing in shear, tear-out of the bolt in tension, and wall distortion. Manufacturers’ literature must be consulted to determine the available strength of blind bolts.

#### Flow-Drilling

Flow-drilling is a process that can be used to produce a threaded hole in an HSS to permit blind bolting when the inside of the HSS is inaccessible (Sherman, 1955; Henderson, 1996). The process is to force a hole through the HSS with a carbide conical tool rotating at sufficient speed to produce high rapid heating, which softens the material in a local area. The material
### Table 7-1

**Available Shear Strength of Bolts, kips**

<table>
<thead>
<tr>
<th>Nominal Bolt Diameter, d, in.</th>
<th>1/8</th>
<th>1/4</th>
<th>3/8</th>
<th>1/2</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Nominal Bolt Area, in.²</strong></td>
<td>0.994</td>
<td>1.23</td>
<td>1.48</td>
<td>1.77</td>
</tr>
<tr>
<td><strong>ASTM Design.</strong></td>
<td><strong>Thread Cond.</strong></td>
<td><strong>F_m/\Omega (ksi)</strong></td>
<td><strong>ϕF_m (ksi)</strong></td>
<td><strong>Loading</strong></td>
</tr>
<tr>
<td><strong>ASD LRFD</strong></td>
<td><strong>ASD LRFD</strong></td>
<td><strong>ASD LRFD</strong></td>
<td><strong>ASD LRFD</strong></td>
<td><strong>ASD LRFD</strong></td>
</tr>
<tr>
<td>Group A</td>
<td>N</td>
<td>27.0</td>
<td>40.5</td>
<td>S</td>
</tr>
<tr>
<td></td>
<td>X</td>
<td>34.0</td>
<td>51.0</td>
<td>S</td>
</tr>
</tbody>
</table>

For the loaded connections greater than 20 ft, see ASD specification.
<table>
<thead>
<tr>
<th>Hole Type</th>
<th>Bolt Spacing, s, in.</th>
<th>F₀ ksi</th>
<th>( \frac{r_d}{\Omega} )</th>
<th>( \phi_d )</th>
<th>( \frac{r_d}{\Omega} )</th>
<th>( \phi_d )</th>
<th>( \frac{r_d}{\Omega} )</th>
<th>( \phi_d )</th>
<th>( \frac{r_d}{\Omega} )</th>
<th>( \phi_d )</th>
</tr>
</thead>
<tbody>
<tr>
<td>STD SSLT</td>
<td>5/8 3 in.</td>
<td>34.1</td>
<td>51.1</td>
<td>41.3</td>
<td>62.0</td>
<td>48.6</td>
<td>72.9</td>
<td>55.8</td>
<td>83.7</td>
<td>55.8</td>
</tr>
<tr>
<td>SSLP</td>
<td>5/8 3/16</td>
<td>43.5</td>
<td>65.3</td>
<td>52.2</td>
<td>82.2</td>
<td>60.9</td>
<td>91.4</td>
<td>67.4</td>
<td>101</td>
<td>67.4</td>
</tr>
<tr>
<td>LSLP</td>
<td>5/8 3/16</td>
<td>48.8</td>
<td>76.3</td>
<td>58.5</td>
<td>87.8</td>
<td>68.3</td>
<td>91.4</td>
<td>67.4</td>
<td>101</td>
<td>67.4</td>
</tr>
<tr>
<td>LSLT</td>
<td>5/8 3/16</td>
<td>47.1</td>
<td>59.6</td>
<td>52.8</td>
<td>73.2</td>
<td>58.3</td>
<td>73.2</td>
<td>52.8</td>
<td>73.2</td>
<td>52.8</td>
</tr>
<tr>
<td>STD SSLT, SSLP, OVS, LSLP</td>
<td>5/8 3/16</td>
<td>50.8</td>
<td>78.1</td>
<td>60.2</td>
<td>84.3</td>
<td>60.2</td>
<td>84.3</td>
<td>60.2</td>
<td>84.3</td>
<td>60.2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Spacing for full bearing strength ( s_{full} ), in.</th>
<th>STD, SSLT, LSLT</th>
<th>1( \frac{1}{16} )</th>
<th>2( \frac{1}{16} )</th>
<th>2( \frac{1}{16} )</th>
<th>3( \frac{1}{16} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>OVS</td>
<td>2( \frac{1}{16} )</td>
<td>2( \frac{1}{16} )</td>
<td>2( \frac{3}{16} )</td>
<td>3( \frac{1}{4} )</td>
<td></td>
</tr>
<tr>
<td>SSLP</td>
<td>2( \frac{1}{8} )</td>
<td>2( \frac{1}{8} )</td>
<td>2( \frac{1}{8} )</td>
<td>3( \frac{1}{8} )</td>
<td></td>
</tr>
<tr>
<td>LSLP</td>
<td>2( \frac{1}{8} )</td>
<td>3( \frac{1}{8} )</td>
<td>3( \frac{1}{8} )</td>
<td>4( \frac{1}{8} )</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Minimum Spacing ( s_{min} ), in.</th>
<th>1( \frac{1}{16} )</th>
<th>2</th>
<th>2( \frac{1}{16} )</th>
<th>2( \frac{1}{16} )</th>
</tr>
</thead>
</table>

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

Note: Spacing indicated is from the center of the hole or slot to the center of the adjacent hole or slot in the line of force. Hole deformation is considered. When hole deformation is not considered, see AISC Specification Section J3.10.  
* Decimal value has been rounded to the nearest sixteenth of an inch.
For the edge bolts, use Table 7-5 and a conservative edge distance of 1¼ inches. The bearing strength for these bolts is

\[
\phi r_n = 49.4t = 49.4(0.560) = 27.7 \text{ kips/bolt}
\]

Since the shear strength is less than the bearing strength of any bolt, the shear strength controls.

The factored load is

\[
P_u = 1.2D + 1.6L = 1.2(20) + 1.6(40) = 88 \text{ kips}
\]

and the shear/bearing load per bolt is 88/8 = 11 kips. The shear design strength per bolt is

\[
\phi r_n = 17.9 \text{ kips} > 11 \text{ kips (OK)}
\]

Compute the tensile force per bolt and then check the tension–shear interaction.

Because of symmetry, the centroid of the connection is at middepth. Figure 8.15 shows the bolt areas and the distribution of bolt tensile forces.

From Equation 8.6, the resisting moment is

\[
M = nr_d = 4r(3 + 1.5 + 3 + 7.5) = 24r
\]

The applied moment is

\[
M_u = P_u e = 88(2.75) = 242 \text{ in.-kips}
\]

Equating the resisting and applied moments, we get

\[
24r = 242, \text{ or } r = 10.08 \text{ kips}
\]

The factored load shear stress is

\[
f_{rv} = \frac{11}{0.4418} = 24.90 \text{ ksi}
\]

and from AISC Equation J3-3a, the nominal tensile stress is

\[
F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv} \leq F_{nt}
\]

\[
= 1.3(90) - \frac{90}{24.90} \leq (24.90) = 61.67 \text{ ksi} < 90 \text{ ksi}
\]

**FIGURE 8.15**

Stress left for you in tension after accounting for shear

Limit, still need to check the tension load or stress caused by the moment, **SIGN**
### Available Bearing Strength at Bolt Holes Based on Edge Distance

#### kips/in. thickness

<table>
<thead>
<tr>
<th>Hole Type</th>
<th>Edge Distance ( L_e ), In.</th>
<th>( F_{te} ), ksi</th>
<th>Nominal Bolt Diameter, ( d ), in.</th>
<th>( \frac{\sigma_{te}}{F_{te}} )</th>
<th>( \frac{2}{\sqrt{d}} )</th>
<th>( \frac{2}{\sqrt{2\frac{d}{8}}} )</th>
<th>( \frac{7}{8} )</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASD</td>
<td>LRFD</td>
<td>ASD</td>
<td>LRFD</td>
<td>ASD</td>
<td>LRFD</td>
<td>ASD</td>
<td>LRFD</td>
<td>ASD</td>
</tr>
<tr>
<td>STD</td>
<td>1( \frac{1}{4} )</td>
<td>56</td>
<td>31.1</td>
<td>47.3</td>
<td>29.4</td>
<td>44.1</td>
<td>27.2</td>
<td>40.2</td>
</tr>
<tr>
<td>SSSLT</td>
<td>2</td>
<td>56</td>
<td>43.5</td>
<td>65.3</td>
<td>52.2</td>
<td>78.3</td>
<td>53.3</td>
<td>79.9</td>
</tr>
<tr>
<td>SSLP</td>
<td>1( \frac{1}{4} )</td>
<td>56</td>
<td>28.3</td>
<td>42.4</td>
<td>26.1</td>
<td>39.2</td>
<td>23.9</td>
<td>35.9</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>56</td>
<td>48.8</td>
<td>73.1</td>
<td>58.5</td>
<td>87.8</td>
<td>59.7</td>
<td>89.6</td>
</tr>
<tr>
<td>OVS</td>
<td>1( \frac{1}{4} )</td>
<td>56</td>
<td>29.4</td>
<td>44.0</td>
<td>27.2</td>
<td>40.8</td>
<td>25.0</td>
<td>37.5</td>
</tr>
<tr>
<td>LSLP</td>
<td>2</td>
<td>56</td>
<td>43.5</td>
<td>65.3</td>
<td>52.2</td>
<td>78.3</td>
<td>51.1</td>
<td>76.7</td>
</tr>
<tr>
<td>LSLT</td>
<td>1( \frac{1}{4} )</td>
<td>56</td>
<td>16.3</td>
<td>24.5</td>
<td>10.9</td>
<td>16.3</td>
<td>5.44</td>
<td>8.16</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>56</td>
<td>18.3</td>
<td>27.4</td>
<td>12.2</td>
<td>18.3</td>
<td>6.09</td>
<td>9.14</td>
</tr>
<tr>
<td>STD, SSSLT, SSLP, OVS, LSLP</td>
<td>( L_e \geq L_e \text{ full} )</td>
<td>56</td>
<td>43.5</td>
<td>65.3</td>
<td>52.2</td>
<td>78.3</td>
<td>60.9</td>
<td>81.4</td>
</tr>
<tr>
<td>SSLT</td>
<td>( L_e \geq L_e \text{ full} )</td>
<td>56</td>
<td>36.3</td>
<td>54.4</td>
<td>43.5</td>
<td>65.3</td>
<td>44.4</td>
<td>66.6</td>
</tr>
<tr>
<td></td>
<td>( L_e \geq L_e \text{ full} )</td>
<td>65</td>
<td>40.8</td>
<td>60.9</td>
<td>48.8</td>
<td>73.1</td>
<td>49.8</td>
<td>74.6</td>
</tr>
</tbody>
</table>

#### Edge distance for full bearing strength

<table>
<thead>
<tr>
<th>( L_e \geq L_e \text{ full} ), In.</th>
<th>STD, SSSLT, LSLT</th>
<th>1( \frac{1}{4} )</th>
<th>1( \frac{1}{16} )</th>
<th>2( \frac{1}{4} )</th>
<th>2( \frac{9}{16} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>OVS</td>
<td>1( \frac{1}{16} )</td>
<td>2</td>
<td>2( \frac{1}{16} )</td>
<td>2( \frac{9}{16} )</td>
<td></td>
</tr>
<tr>
<td>SSLP</td>
<td>1( \frac{1}{16} )</td>
<td>2</td>
<td>2( \frac{1}{16} )</td>
<td>2( \frac{9}{16} )</td>
<td></td>
</tr>
<tr>
<td>LSLP</td>
<td>2( \frac{1}{16} )</td>
<td>2( \frac{1}{16} )</td>
<td>2( \frac{1}{16} )</td>
<td>3( \frac{1}{4} )</td>
<td></td>
</tr>
</tbody>
</table>

STD = standard hole
SSSLT = 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

\( \sigma_{te} \) = indicates spacing less than minimum spacing required per AISC Specification Section J3.3.

Note: Spacing indicated is from the center of the hole or slot to the center of the adjacent hole or slot in the line of force. Hole deformation is considered. When hole deformation is not considered, see AISC Specification Section J3.10.

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AMERICAN INSTITUTE OF STEEL CONSTRUCTION
### 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)$^b$</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)</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)</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 $\frac{1}{16}$ 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.
The design tensile strength is

\[ \phi R_n = 0.75F'_{n_t} A_b = 0.75(61.67)(0.4418) = 20.4 \text{ kips} > 10.08 \text{ kips} \quad \text{(OK)} \]

**ANSWER**

The connection is satisfactory.

**ASD SOLUTION**

Determine the shear and bearing strengths. The tables in Part 7 of the *Manual* will be used. From Table 7-1, the shear strength is

\[ \frac{r_n}{\Omega} = 11.9 \text{ kips/bolt} \]

For the inner bolts with a spacing of 3 inches, the bearing strength from Table 7-4 is

\[ \frac{r_n}{\Omega} = 58.5t = 58.5(0.560) = 32.8 \text{ kips/bolt} \]

For the edge bolts, use Table 7-5 and a conservative edge distance of 1\( \frac{1}{4} \) inches. The bearing strength for these bolts is

\[ \frac{r_n}{\Omega} = 32.9t = 32.9(0.560) = 18.4 \text{ kips/bolt} \]

The shear strength controls.

The total applied load is

\[ P_a = D + L = 20 + 40 = 60 \text{ kips} \]

and the shear/bearing load per bolt is \( 60/8 = 7.5 \) kips. The allowable shear strength per bolt is

\[ \frac{r_n}{\Omega} = 11.9 \text{ kips} > 7.5 \text{ kips} \quad \text{(OK)} \]

Compute the tensile force per bolt, then check the tension–shear interactions. The applied moment is

\[ M_a = P_a e = 60(2.75) = 165 \text{ in.-kips} \]

From Equation 8.6, the resisting moment is

\[ M = nr_r d = 4r_r(3 - 1.5 + 3 + 3 - 1.5) = 24r_r \]

Equating the resisting and applied moments, we get

\[ 24r_r = 165 \quad \text{or} \quad r_r = 6.875 \text{ kips} \]

The shearing stress is

\[ f_r = \frac{7.5}{0.4418} = 16.98 \text{ ksi} \]
and from AISC Equation J3-3b, the nominal tensile stress is

\[ F'_{nt} = 1.3F_{nt} - \frac{\Omega F_{nt}'}{F_{nv}} f'_{re} \leq F_{nt} \]

\[ = 1.3(90) - \frac{2.00(90)}{54} (16.98) = 60.40 \text{ ksi} < 90 \text{ ksi} \]

The allowable tensile strength is

\[ \frac{R_n}{\Omega} = \frac{F'_{nt}A_b}{\Omega} = \frac{60.40(0.4418)}{2.00} = 13.3 \text{ kips} > 6.875 \text{ kips} \quad \text{(OK)} \]

**ANSWER** The connection is satisfactory.

When bolts in slip-critical connections are subjected to tension, the slip-critical strength is ordinarily reduced by the factor given in AISC J3.9 (see Section 7.9). The reason is that the clamping effect, and hence the friction force, is reduced. In a connection of the type just considered, however, there is additional compression on the lower part of the connection that increases the friction, thereby compensating for the reduction in the upper part of the connection. For this reason, the slip-critical strength should not be reduced in this type of connection.

### 8.4 ECCENTRIC WELDED CONNECTIONS: SHEAR ONLY

Eccentric welded connections are analyzed in much the same way as bolted connections, except that unit lengths of weld replace individual fasteners in the computations. As in the case of eccentric bolted connections loaded in shear, welded shear connections can be investigated by either elastic or ultimate strength methods.

**Elastic Analysis**

The load on the bracket shown in Figure 8.16a may be considered to act in the plane of the weld—that is, the plane of the throat. If this slight approximation is made, the load will be resisted by the area of weld shown in Figure 8.16b. Computations are simplified, however, if a unit throat dimension is used. The calculated load can then be multiplied by 0.707 times the weld size to obtain the actual load.
An eccentric load in the plane of the weld subjects the weld to both direct shear and torsional shear. Since all elements of the weld receive an equal portion of the direct shear, the direct shear stress is

$$f_c = f_i = \frac{P}{L}(\ell)$$

where $L$ is the total length of the weld and numerically equals the shear area, because a unit throat size has been assumed. If rectangular components are used,

$$\begin{align*}
&f_{c,\ell} = \frac{P_x}{L}(\ell) \quad \text{and} \quad f_{c,\ell} = \frac{P_y}{L}(\ell) \\
&J = I_{x,x} + I_{y,y}
\end{align*}$$

where $P_x$ and $P_y$ are the $x$ and $y$ components of the applied load. The shearing stress caused by the couple is found with the torsion formula

$$f_m = \frac{Md}{J}$$

where

- $d =$ distance from the centroid of the shear area to the point where the stress is being computed
- $J =$ polar moment of inertia of that area

Figure 8.17 shows this stress at the upper right-hand corner of the given weld. In terms of rectangular components,

$$\begin{align*}
&f_{m,x} = \frac{M_y}{J} \\
&f_{m,y} = \frac{M_x}{J}
\end{align*}$$
Also,

\[ J = \int_A r^2 dA = \int_A (x^2 + y^2) dA = \int_A x^2 dA + \int_A y^2 dA = I_x + I_y \]

where \( I_x \) and \( I_y \) are the rectangular moments of inertia of the shear area. Once all rectangular components have been found, they can be added vectorially to obtain the resultant shearing stress at the point of interest, or

\[ f_v = \sqrt{\left(\sum f_x\right)^2 + \left(\sum f_y\right)^2} \]

As with bolted connections, the critical location for this resultant stress can usually be determined from an inspection of the relative magnitudes and directions of the direct and torsional shearing stress components.

Because a unit width of weld is used, the computations for centroid and moment of inertia are the same as for a line. In this book, we treat all weld segments as line segments, which we assume to be the same length as the edge of the connected part that they are adjacent to. Furthermore, we neglect the moment of inertia of a line segment about the axis coinciding with the line.

**Example 8.5**

Determine the size of weld required for the bracket connection in Figure 8.18. The service dead load is 10 kips, and the service live load is 30 kips. A36 steel is used for the bracket, and A992 steel is used for the column.
FORCE
A force is a vector quantity. It is defined when its (1) magnitude, (2) point of application, and (3) direction are known.

RESULTANT (TWO DIMENSIONS)
The resultant, \( F \), of \( n \) forces with components \( F_{x,i} \) and \( F_{y,i} \) has the magnitude
\[
F = \left( \sum_{i=1}^{n} F_{x,i}^2 + \sum_{i=1}^{n} F_{y,i}^2 \right)^{1/2}
\]
The resultant direction with respect to the x-axis using fourquadrant angle functions is
\[
\theta = \arctan\left( \frac{\sum_{i=1}^{n} F_{y,i}}{\sum_{i=1}^{n} F_{x,i}} \right)
\]
The vector form of a force is
\[
F = F_x \hat{i} + F_y \hat{j}
\]

RESOLUTION OF A FORCE
\[
F_x = F \cos \theta; \quad F_y = F \sin \theta; \quad F_z = F \cos \theta
\]
\[
\cos \theta_x = F_x / F; \quad \cos \theta_y = F_y / F; \quad \cos \theta_z = F_z / F
\]
Separating a force into components when the geometry of force is known and \( R = \sqrt{x^2 + y^2 + z^2} \)
\[
F_x = (x/R)F; \quad F_y = (y/R)F; \quad F_z = (z/R)F
\]

MOMENTS (COUPLES)
A system of two forces that are equal in magnitude, opposite in direction, and parallel to each other is called a couple. A moment \( M \) is defined as the cross product of the radius vector \( r \) and the force \( F \) from a point to the line of action of the force.
\[
M = r \times F; \quad M_x = yF_y - zF_y, \quad M_y = zF_x - xF_x, \quad M_z = xF_y - yF_x.
\]

SYSTEMS OF FORCES
\[
F = \Sigma F_n, \quad M = \Sigma (r_x \times F_n)
\]

Equilibrium Requirements
\[
\Sigma F_n = 0 \quad \Sigma M_n = 0
\]

CENTROIDS OF MASSES, AREAS, LENGTHS, AND VOLUMES
Formulas for centroids, moments of inertia, and first moment of areas are presented in the MATHEMATICS section for continuous functions. The following discrete formulas are for defined regular masses, areas, lengths, and volumes:
\[
r_c = \frac{\Sigma m_i r_i}{\Sigma m_i}\text{, where}
\]
\[
m_i = \text{the mass of each particle making up the system,}
\]
\[
r_i = \text{the radius vector to each particle from a selected reference point, and}
\]
\[
r_c = \text{the radius vector to the center of the total mass from the selected reference point.}
\]
The moment of area \( (M_y) \) is defined as
\[
M_y = \Sigma x_i a_i
\]
\[
M_x = \Sigma y_i a_i
\]
\[
M_z = \Sigma z_i a_i
\]
The centroid of area is defined as
\[
x_{ac} = \frac{M_y}{A}; \quad y_{ac} = \frac{M_x}{A}; \quad z_{ac} = \frac{M_z}{A}
\]
where \( A = \Sigma a_i \)

The centroid of a line is defined as
\[
x_{lc} = \frac{(\Sigma x_i a_i)}{L}, \quad \text{where} \quad L = \Sigma l_i
\]
\[
y_{lc} = \frac{(\Sigma y_i a_i)}{L}
\]
\[
z_{lc} = \frac{(\Sigma z_i a_i)}{L}
\]
The centroid of volume is defined as
\[
x_{vc} = \frac{(\Sigma x_i v_i)}{V}, \quad \text{where} \quad V = \Sigma v_i
\]
\[
y_{vc} = \frac{(\Sigma y_i v_i)}{V}
\]
\[
z_{vc} = \frac{(\Sigma z_i v_i)}{V}
\]

MOMENT OF INERTIA
The moment of inertia, or the second moment of area, is defined as
\[
I_x = \int x^2 \, dA \quad I_y = \int y^2 \, dA
\]
The polar moment of inertia \( J \) of an area about a point is equal to the sum of the moments of inertia of the area about any two perpendicular axes in the area and passing through the same point.
\[
J_z = I_x + I_y = \int (x^2 + y^2) \, dA = r_p^2 A, \quad \text{where}
\]
\[
r_p = \text{the radius of gyration} \quad \text{(see the DYNAMICS section and the next page of this section).}
\]
\[ \frac{P_x}{L} = f_{cx} \]
\[ \frac{P_y}{L} = f_{cy} \]

**Forces due to direct force at centroid**

\[ f_{cx} = \frac{P_x}{L} \]
\[ f_{cy} = \frac{P_y}{L} \]

\[ f_{mx} = \frac{M_y}{J} \]
\[ f_{my} = \frac{M_x}{J} \]

\[ M = P_e = P(d_N) \]

**Forces due to moment about centroid**
\[ P_u = 1.2D + 1.6L = 1.2(10) + 1.6(30) = 60 \text{ kips} \]

The eccentric load may be replaced by a concentric load and a couple, as shown in Figure 8.18. The direct shearing stress is the same for all segments of the weld and is equal to

\[ f_{cy} = f_{cxy} = \frac{60}{8 + 12 + 8} = \frac{60}{28} = 2.143 \text{ ksi} \]

Before computing the torsional component of shearing stress, the location of the centroid of the weld shear area must be determined. From the principle of moments with summation of moments about the \( y \)-axis,

\[ x(28) = 8(4)(2) \quad \text{or} \quad x = 2.286 \text{ in.} \]

The eccentricity \( e \) is \( 10 + 8 - 2.286 = 15.71 \text{ in.} \), and the torsional moment is

\[ M = Pe = 60(15.71) = 942.6 \text{ in.-kips} \]

If the moment of inertia of each horizontal weld about its own centroidal axis is neglected, the moment of inertia of the total weld area about its horizontal centroidal axis is

\[ I_x = \frac{1}{12} (1)(12)^3 + 2(8)(6)^2 = 720.0 \text{ in.}^4 \]

Similarly,

\[ I_y = 2\left[ \frac{1}{12} (1)(8)^3 + 8(4 - 2.286)^2 \right] + 12(2.286)^2 = 195.0 \text{ in.}^4 \]

and

\[ J = I_x + I_y = 720.0 + 195.0 = 915.0 \text{ in.}^4 \]

Figure 8.18 shows the directions of both components of stress at each corner of the connection. By inspection, either the upper right-hand corner or the lower right-hand corner may be taken as the critical location. If the lower right-hand corner is selected,

\[ f_{mx} = f_{cxy} = \frac{M_y}{J} = \frac{942.6(6)}{915.0} = 6.181 \text{ ksi} \]

\[ f_{my} = f_{cxy} = \frac{M_x}{J} = \frac{942.6(8 - 2.286)}{915.0} = 5.886 \text{ ksi} \]

\[ f_v = \sqrt{(6.181)^2 + (2.143 + 5.886)^2} = 10.13 \text{ ksi} = 10.13 \text{ kips/in.} \]

Check the strength of the base metal. The bracket is the thinner of the connected parts and controls. From Equation 7.35, the base-metal shear yield strength per unit length is

\[ \phi R_u = 0.6F_c t = 0.6(36) \left( \frac{9}{16} \right) = 12.2 \text{ kips/in.} \]
From Equation 7.36, the base-metal shear rupture strength per unit length is

\[ \phi R_u = 0.45 F_{u,i} = 0.45(58)\left(\frac{9}{16}\right) = 14.7 \text{ kips/in.} \quad P_{g450} \text{Eq. 7.36} \]

The base metal shear strength is therefore 12.2 kips/in. > 10.13 kips/in. \(\text{OK}\)

From Equation 7.29, the weld strength per inch is

\[ R_u = 10.13 \frac{k}{\text{in}} \leq \phi R_n = \phi(0.707wF_{nw}) = \phi(0.707w(0.6F_{exx})) \quad [16.1-115] \]

The matching electrode for A36 steel is E70. Because the load direction varies on each weld segment, the weld shear strength varies, but for simplicity, we will conservatively use \(F_{nw} = 0.6F_{exx}\) for the entire weld. The required weld size is therefore

\[ \text{Request} = R_u = \frac{\phi R_n}{\phi(0.707wF_{nw})} = 10.13 \frac{k}{\text{in}} \leq 0.75(0.707)(0.6 \times 70) = 0.455 \text{ in.} \quad [16.1-115] \]

Alternatively, for E70 electrodes, \(\phi R_n = 1.392 \text{ kips/in.}\) per sixteenth of an inch in size. The required size in sixteenths is therefore

\[ \frac{10.13}{1.392} = 7.3 \text{ sixteenths} \quad \text{use} \quad \frac{8}{16} \text{ in.} = \frac{1}{2} \text{ in.} \]

ANSWER

Use a \(\frac{1}{2}\)-inch fillet weld, E70 electrode.

ASD SOLUTION

The total load is \(P_a = D + L = 10 + 30 = 40 \text{ kips}\).

The eccentric load may be replaced by a concentric load and a couple, as shown in Figure 8.18. The direct shearing stress is the same for all segments of the weld and is equal to

\[ f_{iy} = \frac{40}{8 + 12 + 8} = \frac{40}{28} = 1.429 \text{ ksi} \]

To locate the centroid of the weld shearing area, use the principle of moments with summation of moments about the \(y\) axis.

\[ \bar{x}(28) = 8(4)(2) \quad \text{or} \quad \bar{x} = 2.286 \text{ in.} \]

The eccentricity \(e\) is \(10 + 8 = 2.286 = 15.71 \text{ in.}\), and the torsional moment is

\[ M = Pe = 40(15.71) = 628.4 \text{ in.-kips} \]

If the moment of inertia of each horizontal weld about its own centroidal axis is neglected, the moment of inertia of the total weld area about its horizontal centroidal axis is

\[ I_x = \frac{1}{12}[(1)(12)^3 + 2(8)(6)^2] = 720.0 \text{ in.}^4 \]
\[ J = \frac{1}{12} \left( 1(8)^3 + 8(4 - 2.286)^3 \right) \]
\[ + 12(2.286)^2 = 1950 \text{ in.}^4 \]

Similarly,
\[ I_r = \frac{1}{12} \left( \frac{1}{3}(8)^3 + 8(4 - 2.286)^3 \right) \]
\[ + 12(2.286)^2 = 1950 \text{ in.}^4 \]

Figure 8.18 shows the directions of both components of stress at each corner of the connection. By inspection, either the upper right-hand corner or the lower right-hand corner may be taken as the critical location. If the lower right-hand corner is selected,

Check the strength of the base metal. The bracket is the thinner of the connected parts and controls. From Equation 7.37, the base metal shear yield strength per unit length is

\[ f_y = 6756 \text{ ksi} = 6756 \text{ kips/in.} \]

From Equation 7.38, the base metal shear rupture strength per unit length is

\[ f_t = 9150 \text{ ksi} = 9150 \text{ kips/in.} \]

The base metal shear strength is therefore 8100 kips/in. > 6756 kips/in. (OK)

The matching electrode for A36 steel is E71T. The weld shear strength varies with the load direction, so use a conservative weld size.

\[ w = \frac{Q_k R_{k}}{F_{sw}} = \frac{0.707 F_{sw}}{0.06750} = 0.455 \text{ in.} \]

Use \( \frac{1}{2} \) in.

The weld size is therefore

\[ R_{sw} = 0.707 w F_{sw} \]

\[ \Omega = 0.36 \] (9/16)

\[ 0.707(0.06750) \]

\[ 0.455 \text{ in.} \]

\[ 0.707 \text{ in.} \]

\[ 0.455 \text{ in.} \]
Alternatively, for E70 electrodes, $R_w/\Omega = 0.9279$ kips/in. per sixteenth of an inch in size. The required size in sixteenths is, therefore,

$$\frac{6.756}{0.9279} = 7.3 \text{ sixteenths} \quad \text{use} \quad \frac{8}{16} \text{ in.} = \frac{1}{2} \text{ in.}$$

**ANSWER**

Use a $\frac{1}{2}$-inch fillet weld, E70 electrode.

---

**Ultimate Strength Analysis**

Eccentric welded shear connections may be safely designed by elastic methods, but the factor of safety will be larger than necessary and will vary from connection to connection (Butler, Pal, and Kulak, 1972). This type of analysis suffers from some of the same shortcomings as the elastic method for eccentric bolted connections, including the assumption of a linear load-deformation relationship for the weld. Another source of error is the assumption that the strength of the weld is independent of the direction of the applied load. An ultimate strength approach, based on the relationships in AISC J2.4b, is presented in Part 8 of the *Manual* and is summarized here. It is based on research by Butler et al. (1972) and Kulak and Timler (1984) and closely parallels the method developed for eccentric bolted connections by Crawford and Kulak (1971).

Instead of considering individual fasteners, we treat the continuous weld as an assembly of discrete segments. At failure, the applied connection load is resisted by forces in each element, with each force acting perpendicular to the radius constructed from an instantaneous center of rotation to the centroid of the segment, as shown in Figure 8.19. This concept is essentially the same as that used for the fasteners. However, determining which element has the maximum deformation and computing the force in each element at failure is more difficult, because unlike the bolted case, the weld strength is a function of the direction of the load on the element. To determine the critical element, first compute the deformation of each element at maximum stress:

$$\Delta_m = 0.209(\theta + 2)^{-0.32}w$$

**Figure 8.19**

[Diagram showing segment with forces and deformation]
where

\[ \Delta = \text{deformation of the element at maximum stress} \]
\[ \theta = \text{angle that the resisting force makes with the axis of the weld segment (see Figure 8.19)} \]
\[ w = \text{weld leg size} \]

Next, compute \( \Delta / r \) for each element, where \( r \) is the radius from the IC to the centroid of the element. The element with the smallest \( \Delta / r \) is the critical element, that is, the one that reaches its ultimate capacity first. For this element, the ultimate (fracture) deformation is

\[ \Delta_u = 1.087(\theta + 6)^{-0.65} w \leq 0.17w \]

and the radius is \( r_{\text{crit}} \). The deformation of each of the other elements is

\[ \Delta = r \frac{\Delta_h}{r_{\text{crit}}} \]

The stress in each element is then

\[ F_{nw} = 0.60 F_{EYX} \left(1 + 0.5 \sin^{1.5} \theta\right) \left[p(1.9 - 0.9p)\right]^{0.3} \]

where

\[ F_{EYX} = \text{weld electrode strength} \]
\[ p = \frac{\Delta}{\Delta_m} \text{ for the element} \]

The force in each element is \( F_{nw} A_w \), where \( A_w \) is the weld throat area.

The preceding computations are based on an assumed location of the instantaneous center of rotation. If it is the actual location, the equations of equilibrium will be satisfied. The remaining details are the same as for a bolted connection.

1. Solve for the load capacity from the equation

\[ \sum M_{IC} = 0 \]

where IC is the instantaneous center.

2. If the two force equilibrium equations are satisfied, the assumed location of the instantaneous center and the load found in Step 1 are correct; otherwise, assume a new location and repeat the entire process.

The absolute necessity for the use of a computer is obvious. Computer solutions for various common configurations of eccentric welded shear connections are given in tabular form in Part 8 of the Manual. Tables 8-4 through 8-11 give available strength coefficients for various common combinations of horizontal and vertical weld segments based on an ultimate strength analysis. These tables may be used for either design or analysis and will cover almost any situation you are likely to encounter. For those connections not covered by the tables, the more conservative elastic method may be used.
\[ \bar{R} = \frac{8''(4'') + 8''(4'') + 12''(0'')}{8 + 8 + 12} = 2.286'' \]

8.4 Eccentric Welded Connections: Shear Only

and the radius is \( r_{\text{crit}} \). The deformation of each of the other elements is

\[ \Delta = r \frac{\Delta L}{r_{\text{crit}}} \]

The stress in each element is then

\[ F_w = 0.60F_{\text{EXX}} \left(1 + 0.5 \sin^{1.5} \theta \right)[p(1.9 - 0.9p)]^{3/2} \]

where

\[ F_{\text{EXX}} = \text{weld electrode strength} \]

\[ p = \frac{\Delta}{\Delta_m} \text{ for the element} \]

The force in each element is \( F_wA_w \) where \( A_w \) is the weld throat area.

The preceding computations are based on an assumed location of the instantaneous center of rotation. If it is the actual location, the equations of equilibrium will be satisfied. The remaining details are the same as for a bolted connection.

1. Solve for the load capacity from the equation

\[ \sum M_{\text{IC}} = 0 \]

where IC is the instantaneous center.

2. If the two force equilibrium equations are satisfied, the assumed location of the instantaneous center and the load found in Step 1 are correct; otherwise, assume a new location and repeat the entire process.

The absolute necessity for the use of a computer is obvious. Computer solutions for various common configurations of eccentric welded shear connections are given in tabular form in Part 8 of the Manual. Tables 8-4 through 8-11 give available strength coefficients for various common combinations of horizontal and vertical weld segments based on an ultimate strength analysis. These tables may be used for either design or analysis and will cover almost any situation you are likely to encounter. For those connections not covered by the tables, the more conservative elastic method may be used.

**Example 8.6** Determine the weld size required for the connection in Example 8.5, based on ultimate strength considerations. Use the tables for eccentrically loaded weld groups given in Part 8 of the Manual.

**Solution**

The weld of Example 8.5 is the same type as the one shown in Table 8-8 (angle = 0°), and the loading is similar. The following geometric constants are required for entry into the table:

\[ a = \frac{aL}{\ell} = \frac{e_x}{\ell} = \frac{15.7}{12} = 1.3 \]

\[ k = \frac{kL}{\ell} = \frac{8}{12} = 0.67 \]
Can use tables to find 
\[ \overline{\varepsilon} \] where \( \varepsilon \) is listed at bottom of page.

Since \( r = \frac{8}{l} = \frac{8}{12} = 0.666 \), so \( r = 0.666 \)

\[ \begin{bmatrix} 0.6 & 0.666 & 0.7 \\ 0.164 & 0.1904 & 0.204 \end{bmatrix} \] Value of \( \overline{\varepsilon} \)

\[ \overline{\varepsilon} = r \cdot l = 0.1904 \cdot 12" = 2.285" \] easy way!

\[ e' = e - \overline{\varepsilon} \]

So \( e = 8" + 10" - 2.285" = 15.715" \)

Since \( e'_n = a \cdot l \)

\[ a = \frac{e'_n}{l} = \frac{15.715"}{12"} = 1.31 \]

Now knowing \( k \) & \( a \), you can get \( c \) from Table 8-8.
**Table 8-8**

**Coefficients C for Eccentrically Loaded Weld Groups**

**Angle = 0°**

Available Strength of a weld group, $C_{min}$ or $D_{min}$, is determined with $R_n = CC_D$ or $R_n = \frac{P_u}{CC_I}$ ($\phi = 0.75, \Omega = 2.0$)

<table>
<thead>
<tr>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_{min} = \frac{P}{\phi CC_D}$</td>
<td>$D_{min} = \frac{\Omega P}{CC_I}$</td>
</tr>
<tr>
<td>$D_{min} = \frac{P}{CC_I}$</td>
<td>$D_{min} = \frac{\Omega P}{CC_I}$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>$\alpha$</th>
<th>0.00</th>
<th>0.10</th>
<th>0.20</th>
<th>0.30</th>
<th>0.40</th>
<th>0.50</th>
<th>0.60</th>
<th>0.70</th>
<th>0.80</th>
<th>0.90</th>
<th>1.00</th>
<th>1.20</th>
<th>1.40</th>
<th>1.60</th>
<th>1.80</th>
<th>2.00</th>
</tr>
</thead>
<tbody>
<tr>
<td>$k$</td>
<td>0.00</td>
<td>0.67</td>
<td>0.67</td>
<td>0.67</td>
<td>0.67</td>
<td>0.67</td>
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**Design Considerations for Welds**

*American Institute of Steel Construction, Inc.*
# Table 8-3

**Electrode Strength Coefficient, $C_1$**

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<th>Electrode</th>
<th>$F_{xxx}$ (ksi)</th>
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*Most often used*
Chapter 8  Eccentric Connections

By interpolation in Table 8-8 for \(a = 1.3\),

\[
C = 1.52 \quad \text{for} \quad k = 0.6 \quad \text{and} \quad C = 1.73 \quad \text{for} \quad k = 0.7
\]

Interpolating between these two values for \(k = 0.67\) gives \(C = 1.67\).

For E70XX electrodes, \(C = 1.0\). \[8-65\]

**LRFD Solution**  From Table 8-8, the nominal strength of the connection is given by

\[
R_n = CC_1D\ell
\]

For LRFD,

\[
\phi R_n \geq P_u
\]

so

\[
\frac{P_u}{\phi CC_1\ell} \geq \frac{P_u}{\phi}
\]

and the required value of \(D\) is

\[
D \geq \frac{P_u}{\phi CC_1\ell} = \frac{60}{0.75(1.67)(1.0)(12)} = 3.99 \text{ sixteenths}
\]

The required weld size is therefore

\[
\frac{3.99}{16} = 0.249 \text{ in.} \quad (\text{versus} \ 0.455 \text{ inch required in Example 8.5})
\]

**Answer**  Use a \(\frac{1}{4}\)-inch fillet weld, E70 electrode.

**ASD Solution**  From Table 8-8, the nominal strength of the connection is given by

\[
R_n = CC_1D\ell
\]

For ASD,

\[
\frac{R_n}{\Omega} = P_a
\]

so

\[
\Omega P_a = CC_1D\ell
\]

and the required value of \(D\) is

\[
D = \frac{\Omega P_a}{CC_1\ell} = \frac{2.00(40)}{1.67(1.0)(12)} = 3.99 \text{ sixteenths}
\]
## Table 8-4
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 0°

Available Strength of a weld group, \( \Phi R_n \) or \( R_n/\Omega \), is determined with
\[
R_n = CC_L Dl \ (\phi = 0.75, \ \Omega = 2.00)
\]

### LRFD

\[
C_{\min} = \frac{P_U}{\phi C_D L} \quad D_{\min} = \frac{P_U}{\phi C_C l} \quad l_{\min} = \frac{\Omega P_U}{\phi C_D}
\]

### ASD

\[
C_{\min} = \frac{\Omega P_a}{C_D} \quad D_{\min} = \frac{\Omega P_a}{CC_C l} \quad l_{\min} = \frac{\Omega P_a}{CC_D}
\]

where
- \( P = \) required force, \( P_U \) or \( P_a \), kips
- \( D = \) number of sixteenths-of-an-inch in the fillet weld size
- \( l = \) characteristic length of weld group, in.
- \( a = e_y/l \)
- \( e_y = \) horizontal component of eccentricity of \( P \)
- \( C = \) coefficient tabulated below
- \( C_a = \) electrode strength coefficient from Table 8-4
  (1.0 for E70XX electrodes)

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AMERICAN INSTITUTE OF STEEL CONSTRUCTION, INC.
EXAMPLE 8.6

Determine the weld size required for the connection in Example 8.5, based on ultimate strength considerations. Use the tables for eccentrically loaded weld groups given in Part 8 of the Manual.

SOLUTION

The weld of Example 8.5 is the same type as the one shown in Manual Table 8-8 (angle = 0°), and the loading is similar. The following geometric constants are required for entry into the table:

\[ a = \frac{a\ell}{\ell} = \frac{e}{12} = 1.3 \]
\[ k = \frac{k\ell}{\ell} = \frac{8}{12} = 0.67 \]

By interpolation in Table 8-8 for \( a = 1.3 \),

\[ C = 1.52 \quad \text{for} \quad k = 0.6 \quad \text{and} \quad C = 1.73 \quad \text{for} \quad k = 0.7 \]

Interpolating between these two values for \( k = 0.67 \) gives

\[ C = 1.67 \]

For E70XX electrodes, \( C_1 = 1.0 \).

LRFD SOLUTION

From Table 8-8, the nominal strength of the connection is given by

\[ R_n = CC_1D\ell \]

For LRFD,

\[ \phi R_n = P_u \]

so

\[ \frac{P_u}{\phi} = CC_1D\ell \]

and the required value of \( D \) is

\[ D = \frac{P_u}{\phi CC_1\ell} = \frac{60}{0.75(1.67)(1.0)(12)} = 3.99 \text{ sixteenths} \]

The required weld size is therefore

\[ \frac{3.99}{16} = 0.249 \text{ in.} \quad (\text{versus} \ 0.455 \text{ inch required in Example 8.5}) \]
8.4 Eccentric Welded Connections: Shear Only

ANSWER

Use a ¼-inch fillet weld, E70 electrode.

From Table 8-8, the nominal strength of the connection is given by

\[ R_n = CC_i D \ell \]

For ASD,

\[ \frac{R_n}{\Omega} = P_a \]

so

\[ \Omega P_a = CC_i D \ell \]

and the required value of \( D \) is

\[ D = \frac{\Omega P_a}{CC_i \ell} = \frac{2.00(40)}{1.67(1.0)(12)} = 3.99 \text{ sixteenths} \]

The required weld size is, therefore,

\[ \frac{3.99}{16} = 0.249 \text{ in.} \quad \text{(versus 0.455 inch required in Example 8.5)} \]

ANSWER

Use a ¼-inch fillet weld, E70 electrode.

---

Special Provision for Axially Loaded Members

When a structural member is axially loaded, the stress is uniform over the cross section and the resultant force may be considered to act along the gravity axis, which is a longitudinal axis through the centroid. For the member to be concentrically loaded at its ends, the resultant resisting force furnished by the connection must also act along this axis. If the member has a symmetrical cross section, this result can be accomplished by placing the welds or bolts symmetrically. If the member is one with an unsymmetrical cross section, such as the double-angle section in Figure 8.20, a symmetrical placement of welds or bolts will result in an eccentrically loaded connection, with a couple of \( T_e \), as shown in Figure 8.20b.

AISC J1.7 permits this eccentricity to be neglected in statically loaded members.

When the member is subjected to fatigue caused by repeated loading or reversal of stress, the eccentricity must be eliminated by an appropriate placement of the welds or...
bolts. *(Of course, this solution may be used even if the member is subjected to static loads only.) The correct placement can be determined by applying the force and moment equilibrium equations. For the welded connection shown in Figure 8.21, the first equation can be obtained by summing moments about the lower longitudinal weld:

\[
\sum M_{L_2} = Tc - P_3 \frac{L_3}{2} - R_1 L_3 = 0 \quad \text{Since } L_3 \text{ is known, } P_3 \text{ is known}
\]

This equation can be solved for \( P_1 \), the required resisting force in the upper longitudinal weld. This value can then be substituted into the force equilibrium equation:

\[
\sum F = T - P_1 - P_2 - P_3 = 0
\]

This equation can be solved for \( P_2 \), the required resisting force in the lower longitudinal weld. For any size weld, the lengths \( L_1 \) and \( L_2 \) can then be determined. We illustrate this procedure, known as balancing the welds, in Example 8.7.