Note that the value of $C_m$ is nearly the same regardless of whether factored or unfactored moments are used.

**Example 6.4**  
**W8 × 35 - Effective Length**

The horizontal beam-column shown in Figure 6.13 is subject to the service live loads shown. This member is laterally braced at its ends, and bending is about the x-axis. Check for compliance with the AISC Specification. $K_s = K_r = 1.0$.

**Solution**

The factored axial load is

$$P_u = 1.6(28) = 44.8 \text{ kips}$$

The factored transverse loads and bending moment are

$$Q_u = 1.6(28) = 44.8 \text{ kips}$$
$$w_u = 1.2(0.035) = 0.042 \text{ kips/ft}$$

$$M_u = \frac{44.8(10)}{4} + \frac{0.042(10)^2}{8} = 112.5 \text{ ft-kips}$$

This member is braced against sidesway, so $M_{l1} = 0$. (Always for us)

Compute the moment amplification factor. For a member braced against sidesway and transversely loaded, $C_m$ can be taken as 1.0. A more accurate value can be found in the Commentary to AISC Appendix 8:

$$C_m = 1 + \Psi \left( \frac{\alpha P}{P_{el}} \right)$$

(AISC Equation C-A-8-2)

From Commentary Table C-A-8.1, $\Psi = -0.2$ for the support and loading conditions of this beam-column. For the axis of bending,

$$P_{el} = \frac{\pi^2 EI_x}{(K_s L)^2} = \frac{\pi^2 EI_x}{(K_r L)^2} = \frac{\pi^2 (29,000)(127)}{(10 \times 12)^2} = 2524 \text{ kips}$$

$$C_m = 1 + \frac{\alpha P}{P_{el}} = 1 - 0.2 \left( \frac{100P}{P_{el}} \right) = 1 - 0.2 \left( \frac{44,800}{2524} \right) = 0.9965$$

Not using Case 1 - Direct Analysis Method since $EI$ not reduced.

Not using $0.6 - 0.4(M_1/M_2)$ because transversely loaded.
The amplification factor is

\[
B = \frac{C_m}{1 - (\alpha P_a / P_{et})} = \frac{C_m}{1 - (0.00 P_a / P_{et})} = \frac{0.9965}{1 - (44.8/2524)} = 1.015
\]

\[
(16.1 - 238) \text{ ft-kips}
\]

\[
(306b)
\]

The amplified bending moment is

\[
M_n = B_1 M_n + B_2 M_e = 1.015(112.5) + 0 = 114.2 \text{ ft-kips}
\]

From the beam design charts, for \(L_b = 10 \text{ ft and } C_b = 1\),

\[
\phi_b M_n = 123 \text{ ft-kips} \quad \phi_b C_b = 1.0
\]

Because the beam weight is very small in relation to the concentrated live load, \(C_b\) may be taken from Figure 5.15c as 1.32. This value results in a design moment of

\[
\phi_b M_n = 1.32(123) = 162.4 \text{ ft-kips} \quad \phi_b C_b = 1.32
\]

This moment is greater than \(\phi_b M_p = 130 \text{ ft-kips}\), so the design strength must be limited to this value. Therefore,

\[
\phi_b M_n = 130 \text{ ft-kips}
\]

Check the interaction formula. From the column load tables, for \(KL = 10 \text{ ft}\),

\[
\phi_c P_n = 358 \text{ kips} \quad 359 \text{ kips}
\]

\[
\frac{P_n}{\phi_c P_n} = \frac{44.8}{359} = 0.1251 < 0.2 \quad \therefore \text{ Use Equation 6.4 (AISC Equation H1-1b).}
\]

\[
\frac{P_n}{2 \phi_c P_n} + \left( \frac{M_{ux}}{\phi_b M_{ux}} + \frac{M_{uy}}{\phi_b M_{uy}} \right) = \frac{0.1251}{2} + \left( \frac{114.2}{130} + 0 \right)
\]

\[
= 0.941 < 1.0 \text{ (OK)}
\]

**ANSWER**

A W8 × 35 is adequate.

**ASD SOLUTION**

The applied axial-load is

\[
P_a = 28 \text{ kips}
\]

The applied transverse loads are

\[
Q_a = 28 \text{ kips and } w_a = 0.035 \text{ kips/ft}
\]
<table>
<thead>
<tr>
<th>$F_y = 50$ ksi</th>
<th>$f_p = 1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{pl}/\Omega_3$</td>
<td>$\phi_p M_o$</td>
</tr>
<tr>
<td>kip-ft</td>
<td>kip-ft</td>
</tr>
<tr>
<td>ASD</td>
<td>LRFD</td>
</tr>
</tbody>
</table>

### Table 3-10 (continued)

#### W-Shapes

Available Moment vs. Unbraced Length

<table>
<thead>
<tr>
<th>Unbraced Length (0.5-ft increments)</th>
<th>ASD</th>
<th>LRFD</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>120</td>
<td>180</td>
</tr>
<tr>
<td>4</td>
<td>116</td>
<td>174</td>
</tr>
<tr>
<td>6</td>
<td>112</td>
<td>168</td>
</tr>
<tr>
<td>8</td>
<td>104</td>
<td>156</td>
</tr>
<tr>
<td>10</td>
<td>100</td>
<td>150</td>
</tr>
<tr>
<td>12</td>
<td>96</td>
<td>144</td>
</tr>
<tr>
<td>14</td>
<td>92</td>
<td>138</td>
</tr>
<tr>
<td>16</td>
<td>88</td>
<td>132</td>
</tr>
<tr>
<td>18</td>
<td>84</td>
<td>126</td>
</tr>
<tr>
<td>20</td>
<td>80</td>
<td>120</td>
</tr>
<tr>
<td>22</td>
<td>76</td>
<td>114</td>
</tr>
<tr>
<td>24</td>
<td>72</td>
<td>108</td>
</tr>
<tr>
<td>26</td>
<td>68</td>
<td>102</td>
</tr>
<tr>
<td>28</td>
<td>64</td>
<td>96</td>
</tr>
<tr>
<td>30</td>
<td>60</td>
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<td>32</td>
<td>56</td>
<td>84</td>
</tr>
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<td>34</td>
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<tr>
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<td>52</td>
<td>16</td>
<td>36</td>
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<td>54</td>
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<td>32</td>
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<td>56</td>
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<td>4</td>
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<tr>
<td>60</td>
<td>0</td>
<td>20</td>
</tr>
</tbody>
</table>

American Institute of Steel Construction
## Table 3-1
### Values of $C_b$ for Simply Supported Beams

<table>
<thead>
<tr>
<th>Load</th>
<th>Lateral Bracing Along Span</th>
<th>$C_b$</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1" alt="Single Load at Midpoint" /></td>
<td>None, Load at midpoint</td>
<td>1.32</td>
</tr>
<tr>
<td><img src="image2" alt="Double Loads at Midpoints" /></td>
<td>None, Loads at third points</td>
<td>1.14</td>
</tr>
<tr>
<td><img src="image3" alt="Triple Loads at Quarter Points" /></td>
<td>None, Loads at quarter points</td>
<td>1.14</td>
</tr>
<tr>
<td><img src="image4" alt="Single Load at Fifth Points" /></td>
<td>None, Loads at quarter points</td>
<td>1.30</td>
</tr>
<tr>
<td><img src="image5" alt="Single Load at Third Points" /></td>
<td>None, Loads at third points</td>
<td>1.45</td>
</tr>
<tr>
<td><img src="image6" alt="Single Load at Quarter Points" /></td>
<td>None, Loads at quarter points</td>
<td>1.52</td>
</tr>
<tr>
<td><img src="image7" alt="Single Load at Fifth Points" /></td>
<td>None, Loads at fifth points</td>
<td>1.56</td>
</tr>
</tbody>
</table>

Note: Lateral bracing must always be provided at points of support per AISC Specification Chapter F.
Table 4-1 (continued)  
Available Strength in Axial Compression, kips

<table>
<thead>
<tr>
<th>Shape</th>
<th><em>P_0_0</em>C_0_0</th>
<th><em>P_0_0</em>C_0_0</th>
<th><em>P_0_0</em>C_0_0</th>
<th><em>P_0_0</em>C_0_0</th>
<th><em>P_0_0</em>C_0_0</th>
<th><em>P_0_0</em>C_0_0</th>
<th><em>P_0_0</em>C_0_0</th>
<th><em>P_0_0</em>C_0_0</th>
<th><em>P_0_0</em>C_0_0</th>
<th><em>P_0_0</em>C_0_0</th>
<th><em>P_0_0</em>C_0_0</th>
<th><em>P_0_0</em>C_0_0</th>
<th><em>P_0_0</em>C_0_0</th>
<th><em>P_0_0</em>C_0_0</th>
<th><em>P_0_0</em>C_0_0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design</td>
<td>AS</td>
<td>LRFD</td>
<td>AS</td>
<td>LRFD</td>
<td>AS</td>
<td>LRFD</td>
<td>AS</td>
<td>LRFD</td>
<td>AS</td>
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<td>AS</td>
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<td>AS</td>
<td>LRFD</td>
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</tr>
<tr>
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<td>526</td>
<td>388</td>
<td>350</td>
<td>273</td>
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<td>409</td>
<td>399</td>
<td>329</td>
<td>467</td>
<td>409</td>
<td>399</td>
</tr>
<tr>
<td>10</td>
<td>467</td>
<td>409</td>
<td>399</td>
<td>329</td>
<td>467</td>
<td>409</td>
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<td>467</td>
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<td>399</td>
<td>329</td>
<td>467</td>
<td>409</td>
<td>399</td>
</tr>
</tbody>
</table>

Effective length, KL (ft), with respect to least radius of gyration, *r_y*

<table>
<thead>
<tr>
<th>Design</th>
<th>AS</th>
<th>LRFD</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>350</td>
<td>273</td>
</tr>
<tr>
<td>10</td>
<td>467</td>
<td>409</td>
</tr>
<tr>
<td>20</td>
<td>231</td>
<td>197</td>
</tr>
</tbody>
</table>

Properties

<table>
<thead>
<tr>
<th>Properties</th>
<th>ASD</th>
<th>LRFD</th>
</tr>
</thead>
<tbody>
<tr>
<td>*P_0_0_, kips</td>
<td>128</td>
<td>190</td>
</tr>
<tr>
<td>*P_0_0_, kips/ln.</td>
<td>28.5</td>
<td>17.6</td>
</tr>
<tr>
<td>*P_0_0_, kips</td>
<td>507</td>
<td>761</td>
</tr>
<tr>
<td>*P_0_0_, kips</td>
<td>164</td>
<td>246</td>
</tr>
<tr>
<td>*L_0_0_, ft</td>
<td>7.49</td>
<td>7.42</td>
</tr>
<tr>
<td>*L_0_0_, ft</td>
<td>47.6</td>
<td>41.6</td>
</tr>
<tr>
<td>*A_y, in.²</td>
<td>19.7</td>
<td>17.1</td>
</tr>
<tr>
<td>*I_y, in.⁴</td>
<td>272</td>
<td>228</td>
</tr>
<tr>
<td>*I_y, in.⁴</td>
<td>88.6</td>
<td>75.1</td>
</tr>
<tr>
<td>*r_y, in.</td>
<td>2.12</td>
<td>2.10</td>
</tr>
<tr>
<td>*r_y, in.</td>
<td>1.75</td>
<td>1.74</td>
</tr>
<tr>
<td>*P_0_0_(KL)²/10⁴, k-in.²</td>
<td>7790</td>
<td>6530</td>
</tr>
<tr>
<td>*P_0_0_(KL)²/10⁴, k-in.²</td>
<td>2540</td>
<td>2150</td>
</tr>
</tbody>
</table>

ASD | LRFD |
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Ω_0_0_ = 1.67</td>
<td>0.90</td>
</tr>
</tbody>
</table>

Note: Heavy line indicates KL/*r_y* equal to or greater than 200.

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
and the maximum bending moment is

\[ M_{ul} = \frac{28(10)}{4} + \frac{0.035(10)^2}{8} = 70.44 \text{ ft-kips} \]

The member is braced against end translation, so \( M_{ul} = 0 \).

Compute the moment amplification factor. For a member braced against sideways and transversely loaded, \( C_m \) can be taken as 1.0. A more accurate value can be found in the Commentary to AISC Appendix 8:

\[ C_m = 1 + \Psi \left( \frac{\alpha P_e}{P_{el}} \right) \quad \text{(AISC Equation C-A-8-2)} \]

From Commentary Table C-A-8.1, \( \Psi = -0.2 \) for the support and loading conditions of this beam—column. For the axis of bending,

\[ P_{el} = \frac{\pi^2 EI}{(K_1 L)^2} = \frac{\pi^2 E I_x}{(K_2 L)^2} = \frac{\pi^2 (29,000)(127)}{(10 \times 12)^2} = 2524 \text{ kips} \]

\[ C_m = 1 + \Psi \left( \frac{\alpha P_e}{P_{el}} \right) = 1 - 0.2 \left( \frac{1.60 P_e}{P_{el}} \right) = 1 - 0.2 \left( \frac{1.60 \times 28}{2524} \right) = 0.9965 \]

\[ B_1 = \frac{C_m}{1 - (\alpha P_e/P_{el})} = \frac{C_m}{1 - (1.60 P_e/P_{el})} = \frac{0.9965}{1 - (1.60 \times 28/2524)} = 1.015 \]

\[ M_a = B_1 M_{ul} = 1.015(70.44) = 71.50 \text{ ft-kips} \]

From the Beam Design Charts with \( C_b = 1.0 \) and \( L_b = 10 \text{ feet} \), the moment strength is

\[ \frac{M_n}{\Omega_b} = 82.0 \text{ ft-kips} \]

Because the beam weight is very small in relation to the concentrated live load, \( C_b \) may be taken from Figure 5.15c as 1.32. This results in an allowable moment of

\[ \frac{M_n}{\Omega_b} = 1.32(82.0) = 108.2 \text{ ft-kips} \]

This result is larger than \( \frac{M_p}{\Omega_b} = 86.6 \); therefore, use \( \frac{M_n}{\Omega_b} = \frac{M_p}{\Omega_b} = 86.6 \text{ ft-kips} \).

Compute the axial compressive strength. From the column load tables, for \( K L = 10 \text{ ft} \),

\[ \frac{P_n}{\Omega_c} = 238 \text{ kips} \]
Determine which interaction formula to use:

\[
\frac{P_a}{P_n/\Omega_c} = \frac{28}{238} = 0.1176 < 0.2 \quad \therefore \text{Use Equation 6.6 (AISC Equation H1-1b).}
\]

\[
\frac{P_a}{2P_n/\Omega_c} + \left( \frac{M_{ax}}{M_{ax}/\Omega_b} + \frac{M_{ay}}{M_{ay}/\Omega_b} \right) = 0.1176 + \left( \frac{71.50}{86.6} + 0 \right) = 0.884 < 1.0 \text{ (OK)}
\]

**ANSWER** The W8 × 35 is adequate.

---

**EXAMPLE 6.5** W12 × 65 - Case I - Small-Moment-Load-Added

The member shown in Figure 6.14 is a W12 × 65 of A242 steel. First-order analyses were performed with reduced member stiffnesses. The approximate second-order analysis method of AISC Appendix 8 can be used, making this a direct analysis method. For LRFD, the analysis results for the controlling factored load combination are \( P_n = 300 \text{ kips}, \, M_{nx} = 135 \text{ kips}, \text{ and } M_{ny} = 30 \text{ ft-kips}. \) For ASD, the analysis results for the controlling load combination are \( P_n = 200 \text{ kips}, \, M_{nx} = 90 \text{ ft-kips}, \text{ and } M_{ny} = 20 \text{ ft-kips}. \) Use \( K_y = 1.0, \) and investigate this member for compliance with the AISC Specification.

**FIGURE 6.14**

\[
P_n = 300 \text{ kips}
\]

\[
M_{NTx} = 135 \text{ k} \cdot \text{ft}
\]

\[
M_{NTy} = 30 \text{ k} \cdot \text{ft}
\]

W12 × 65
A242 steel
First, determine the yield stress $F_y$. From Table 2-4 in Part 2 of the Manual, we see that A242 steel is available in three different versions. From the dimensions and properties table in Part 1 of the Manual, a W12 × 65 has a flange thickness of $t_f = 0.605$ in. This matches the thickness range corresponding to footnote l in Table 2-4; therefore, $F_y = 50$ ksi.

**Strong Axis Bending Solution**

Compute the strong axis bending moments required.

$$\frac{M_1}{M_2} = \frac{0.6 - 0.4}{135} = 0$$

$$M_2 = 135$$

Since a modified flexural rigidity, $EI^*$, was used in the frame analysis, it must also be used in the computation of $P_{el}$. From Equation 6.8, $EI^* = 0.8\tau_b EI = 0.8(1.0)EI = 0.8EI$

$$P_{el} = \frac{\pi^2 EI^*}{(K_1L)^2} = \frac{0.8(29,000)(533)}{(1.0 \times 15 \times 12)^2} = 3767 \text{kips}$$

$$B_{lx} = \frac{C_{mx}}{1 - (\alpha P_{c}/P_{el})} = \frac{0.6}{1 - (1.00P_a/P_{el})} = \frac{0.6}{1 - (300/3767)} = 0.652 < 1.0 \therefore \text{Use } B_{lx} = 1.0.$$  

The required moment strength is

$$M_r = M_{ux} = B_{lx}M_{ux} + B_{2x}M_{tx} = 1.0(135) + 0 = 135.0 \text{ ft-kips}$$

From the Beam Design Charts with $C_b = 1.0$ and $L_b = 15$ feet, the moment strength is

$$\phi_b M_{nx} = 340 \text{ ft-kips and } \phi_b M_{px} = 356 \text{ ft-kips}$$

From Figure 5.15g, $C_b = 1.67$ and

$$C_b \times (\phi_b M_{nx} \text{ for } C_b = 1.0) = 1.67(340) = 567.8 \text{ ft-kips}$$

This result is larger than $\phi_b M_{px}$; therefore, use $\phi_b M_{nx} = \phi_b M_{px} = 356 \text{ ft-kips}$.

**Weak Axis Bending**

Compute the weak axis bending moments requested.

$$C_{my} = 0.6 - 0.4\left(\frac{M_1}{M_2}\right) = 0.6 - 0.4(0) = 0.6$$

$$P_{dy} = \frac{\pi^2 EI^*}{(K_1L)^2} = \frac{\pi^2 (0.8 EI_y)}{(K_1L)^2} = \frac{\pi^2 (0.8)(29,000)(174)}{(1.0 \times 15 \times 12)^2} = 1230 \text{kips}$$
factor given by Expression 6.7 was derived for the worst case, so $C_m$ will never be greater than 1.0. The final form of the amplification factor is

$$B_1 = \frac{C_m}{1 - (\alpha P_r / P_{el})} \geq 1$$

where

$$P_r = \text{required unamplified axial compressive strength (} P_{mt} + P_{el}) \quad = P_y \text{ for LRFD}$$

$$= P_y \text{ for ASD}$$

$$\alpha = 1.00 \text{ for LRFD} \quad = 1.60 \text{ for ASD}$$

$$P_{el} = \frac{\pi^2 \kappa y^*}{(K_1 L)^2}$$

$$\kappa y^* = \text{flexural rigidity}$$

In the direct analysis method, $\kappa y^*$ is a reduced stiffness obtained as

$$\kappa y^* = 0.8 \tau_\kappa \kappa y$$

(AISC Equation A-8-5)

where

$$\tau_\kappa = \tau_{\text{stiffness reduction factor}} = 1.0 \text{ when } \frac{\alpha P_r}{P_y} \leq 0.5$$

(AISC Equation C2-2a)

$$= 4 \left( \frac{\alpha P_r}{P_y} \right)^2 \left( \frac{P_r}{P_y} \right) \text{ when } \frac{\alpha P_r}{P_y} > 0.5$$

(AISC Equation C2-2b)

This stiffness reduction factor is the same one used in Chapter 4 in connection with the alignment charts for inelastic columns. Under certain conditions, $\tau_\kappa$ can be taken as 1.0 even when $\frac{\alpha P_r}{P_y} > 0.5$. As mentioned in Section 6.3, acceptable frame analysis methods, including the direct analysis method, require the application of notional loads to account for initial out-of-plumbness of columns. AISC C2.3(3) permits the use of $\tau_\kappa = 1.0$ if a small additional notional load is included. In this book, we assume that that is the case, and we will use $\tau_\kappa = 1.0$. Keep in mind that in this book, we do not perform any structural analyses; we only use the results of the analyses.

In the effective length and first-order methods, the flexural rigidity is unreduced, and $\kappa y^* = \kappa y$. The moment of inertia $I$ and the effective length factor $K_1$ are for the axis of bending, and $K_1 = 1.0$ unless a more accurate value is computed (AISC C3). Note that the subscript 1 corresponds to the braced condition and the subscript 2 corresponds to the unbraced condition.
The required moment strength is

\[ M_{n} = M_{p} = 0.0(178.7) = 178.7 \text{ ft-kips} \]

The value of \( \phi_{M_{n}} \) is also given in the Z table, listed as \( \phi_{M_{n}} \). For a W12 x 65, it

\[ \phi_{M_{n}} = 0.90(178.7) = 160.8 \text{ ft-kips} \]

Since \( A_{p} < A_{c} \), the flange of this shape is noncompact (see footnote in the dimensions and properties table), the weak axis bending strength is limited by F.I.R. (see Section 5.15 of this book and Chapter F of the AISC Specification).

The flange moment is

\[ M_{f} = M_{n} = 178.7 \text{ ft-kips} \]

The required moment strength is

\[ M_{n} = M_{p} = 0.0(178.7) = 178.7 \text{ ft-kips} \]

Because the flange of this shape is noncompact (see footnote in the dimensions and properties table), the weak axis bending strength is limited by F.I.R. (see Section 5.15 of this book and Chapter F of the AISC Specification).

The required moment strength is

\[ M_{n} = M_{p} = 0.0(178.7) = 178.7 \text{ ft-kips} \]

Because the flange of this shape is noncompact (see footnote in the dimensions and properties table), the weak axis bending strength is limited by F.I.R. (see Section 5.15 of this book and Chapter F of the AISC Specification).
Determine which interaction formula to use:

\[
\frac{P_a}{\phi_e P_n} = \frac{300}{662} = 0.4532 > 0.2 \quad \therefore \text{Use Equation 6.3 (AISC Equation H1-1a)}.
\]

\[
\frac{P_a}{\phi_e P_n} + \frac{8}{9} \left( \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) = 0.4532 + \frac{8}{9} \left( \frac{135}{356} + \frac{30}{160.8} \right) = 0.956 < 1.0 \quad (\text{OK})
\]

**ANSWER**

The W12 x 65 is satisfactory.

**ASD SOLUTION**

Compute the strong axis bending moments:

\[
C_{nx} = 0.6 - 0.4 \left( \frac{M_1}{M_2} \right) = 0.6 - 0.4(0) = 0.6
\]

Since a modified flexural rigidity, \(EI^*\), was used in the frame analysis, it must also be used in the computation of \(P_{el}\). From Equation 6.8,

\[
EI^* = 0.8 \tau_b EI = 0.8(1.0)EI = 0.8EI
\]

\[
P_{elx} = \frac{\pi^2 EI^*}{(K_1 L)^2} = \frac{\pi^2 (0.8EI_x)}{(K_x L)^2} = \frac{\pi^2 (0.8)(29,000)(533)}{(1.0 \times 15 \times 12)^2} = 3767 \text{kips}
\]

\[
B_{lx} = \frac{C_{mx}}{1 - (\alpha P_a / P_{elx})} = \frac{C_{mx}}{1 - (1.60 P_a / P_{elx})} = \frac{0.6}{1 - (1.60 \times 200 / 3767)} = 0.656 < 1.0 \quad \therefore \text{Use } B_{lx} = 1.0.
\]

\[
M_r = M_{nx} = B_{lx} M_{ntx} + B_{zx} M_{ttx} = 1.0(90) + 0 = 90 \text{ ft-kips}
\]

From the Beam Design Charts with \(C_b = 1.0\) and \(L_b = 15\) feet, the moment strength is

\[
\frac{M_{nx}}{\Omega_b} = 226 \text{ ft-kips and } \frac{M_{px}}{\Omega_b} = 237 \text{ ft-kips}
\]

From Figure 5.15g, \(C_b = 1.67\) and

\[
C_b \times \left( \frac{M_{nx}}{\Omega_b} \right. \text{ for } C_b = 1.0) = 1.67(226) = 377.4 \text{ ft-kips}
\]

This result is larger than \(\frac{M_{px}}{\Omega_b}\); therefore, use \(\frac{M_{nx}}{\Omega_b} = \frac{M_{px}}{\Omega_b} = 237 \text{ ft-kips.} \)
Connections of structural steel members are of critical importance. An inadequate connection, which can be the "weak link" in a structure, has been the cause of numerous failures. Failure of structural members is rare; most structural failures are the result of poorly designed or detailed connections. The problem is compounded by the confusion that sometimes exists regarding responsibility for the design of connections. In many cases, the connections are not designed by the same engineer who designs the rest of the structure, but by someone associated with the steel fabricator who furnishes the material for the project. The structural engineer responsible for the production of the design drawings, however, is responsible for the complete design, including the connections. It is therefore incumbent upon the engineer to be proficient in connection design, if only for the purpose of validating a connection designed by someone else.

Modern steel structures are connected by welding or bolting (either high-strength or "common" bolts) or by a combination of both. Until fairly recently, connections were either welded or riveted. In 1947, the Research Council of Riveted and Bolted Structural Joints was formed, and its first specification was issued in 1951. This document authorized the substitution of high-strength bolts for rivets on a one-for-one basis. Since that time, high-strength bolting has rapidly gained in popularity, and today the widespread use of high-strength bolts has rendered the rivet obsolete in civil engineering structures. There are several reasons for this change. Two relatively unskilled workers can install high-strength bolts, whereas four skilled workers were required for riveting. In addition, the riveting operation was noisy and somewhat dangerous because of the practice of tossing the heated rivet from the point of heating to the point of installation. Riveted connection design is no longer covered by the AISC Specification, but many existing structures contain riveted joints, and the analysis of these connections is required for the strength evaluation and rehabilitation of older structures. Section 5.2.6 of AISC Appendix 5, "Evaluation of Existing Structures," specifies that ASTM A502 Grade 1 rivets should be assumed unless there is evidence
to the contrary. Properties of rivets can be found in the ASTM Specification (ASTM, 2010c). The analysis of riveted connections is essentially the same as for connections with common bolts; only the material properties are different.

Welding has several advantages over bolting. A welded connection is often simpler in concept and requires few, if any, holes (sometimes erection bolts may be required to hold the members in position for the welding operation). Connections that are extremely complex with fasteners can become very simple when welds are used. A case in point is the plate girder shown in Figure 7.1. Before welding became widely used, this type of built-up shape was fabricated by riveting. To attach the flange plates to the web plate, angle shapes were used to transfer load between the two elements. If cover plates were added, the finished product became even more complicated. The welded version, however, is elegant in its simplicity. On the negative side, skilled workers are required for welding, and inspection can be difficult and costly. This last disadvantage can be partially overcome by using shop welding instead of field welding whenever possible. Quality welding can be more easily ensured under the controlled conditions of a fabricating shop. When a connection is made with a combination of welds and bolts, welding can be done in the shop and bolting in the field. In the single-plate beam-to-column connection shown in Figure 7.2, the plate is shop-welded to the column flange and field-bolted to the beam web.
In considering the behavior of different types of connections, it is convenient to categorize them according to the type of loading. The tension member splices shown in Figure 7.3a and b subject the fasteners to forces that tend to shear the shank of the fastener. Similarly, the weld shown in Figure 7.3c must resist shearing forces. The connection of a bracket to a column flange, as in Figure 7.3d, whether by fasteners or welds, subjects the connection to shear when loaded as shown. The hanger connection shown in Figure 7.3e puts the fasteners in tension. The connection shown in Figure 7.3f produces both shear and tension in the upper row of fasteners. The strength of a fastener depends on whether it is subjected to shear or tension, or both. Welds are weak in shear and are usually assumed to fail in shear, regardless of the direction of loading.

Once the force per fastener or force per unit length of weld has been determined, it is a simple matter to evaluate the adequacy of the connection. This determination is the basis for the two major categories of connections. If the line of action of the resultant force to be resisted passes through the center of gravity of the connection, each part of the connection is assumed to resist an equal share of the load, and the connection is called a simple connection. In such connections, illustrated in Figure 7.3a, b, and c, each fastener or each unit length of weld will resist an equal amount of...
force.* The load capacity of the connection can then be found by multiplying the capacity of each fastener or inch of weld by the total number of fasteners or the total length of weld. This chapter is devoted to simple connections. Eccentrically loaded connections, covered in Chapter 8, are those in which the line of action of the load does not act through the center of gravity of the connection. The connections shown in Figure 7.3d and f are of this type. In these cases, the load is not resisted equally by each fastener or each segment of weld, and the determination of the distribution of the load is the complicating factor in the design of this type of connection.

The AISC Specification deals with connections in Chapter J, “Design of Connections,” where bolts and welds are covered.

### 7.2 BOLTED SHEAR CONNECTIONS: FAILURE MODES

Before considering the strength of specific grades of bolts, we need to examine the various modes of failure that are possible in connections with fasteners subjected to shear. There are two broad categories of failure: failure of the fastener and failure of the parts being connected. Consider the lap joint shown in Figure 7.4a. Failure of the fastener can be assumed to occur as shown. The average shearing stress in this case will be

\[ f_v = \frac{P}{A} = \frac{P}{\pi d^2/4} \]

where \( P \) is the load acting on an individual fastener, \( A \) is the cross-sectional area of the fastener, and \( d \) is its diameter. The load can then be written as

\[ P = f_v A \]

Although the loading in this case is not perfectly concentric, the eccentricity is small and can be neglected. The connection in Figure 7.4b is similar, but an analysis

---

*There is actually a small eccentricity in the connections of Figure 7.3b and c, but it is usually neglected.
plane, is called single shear. The addition of more thicknesses of material to the connection will increase the number of shear planes and further reduce the load on each plane. However, that will also increase the length of the fastener and could subject it to bending.

Other modes of failure in shear connections involve failure of the parts being connected and fall into two general categories.

1. **Failure resulting from excessive tension, shear, or bending in the parts being connected.** If a tension member is being connected, tension on both the gross area and effective net area must be investigated. Depending on the configuration of the connection, block shear might also need to be considered. Block shear must also be examined in beam-to-column connections in which the top flange of the beam is coped. (We covered block shear in Chapters 3 and 5, and it is also described in AISC J4.3.) Depending on the type of connection and loading, connection fittings such as gusset plates and framing angles may require an analysis for shear, tension, bending, or block shear. The design of a tension member connection will usually be done in parallel with the design of the member itself because the two processes are interdependent.

2. **Failure of the connected part because of bearing exerted by the fasteners.** If the hole is slightly larger than the fastener and the fastener is assumed to be placed loosely in the hole, contact between the fastener and the connected part will exist over approximately half the circumference of the fastener when a load is applied. This condition is illustrated in Figure 7.5. The stress will vary from a maximum at A to zero at B; for simplicity, an average stress, computed as the applied force divided by the projected area of contact, is used.

Thus the bearing stress would be computed as \( f_b = P / (dt) \), where \( P \) is the force applied to the fastener, \( d \) is the fastener diameter, and \( t \) is the thickness of the part subjected to the bearing. The bearing load is therefore \( P = f_b dt \).
of free-body diagrams of portions of the fastener shank shows that each cross-sectional area is subjected to half the total load, or, equivalently, two cross sections are effective in resisting the total load. In either case, the load is $P = 2f_c A$, and this loading is called double shear. The bolt loading in the connection in Figure 7.4a, with only one shear plane, is called single shear. The addition of more thicknesses of material to the connection will increase the number of shear planes and further reduce the load on each plane. However, that will also increase the length of the fastener and could subject it to bending.

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**FIGURE 7.5**
Thus the bearing stress would be computed as \( f_p = P/(dt) \), where \( P \) is the force applied to the fastener, \( d \) is the fastener diameter, and \( t \) is the thickness of the part subjected to the bearing. The bearing load is therefore \( P = f_p dt \).

The bearing problem can be complicated by the presence of a nearby bolt or the proximity of an edge in the direction of the load, as shown in Figure 7.6. The bolt spacing and edge distance will have an effect on the bearing strength.

### 7.3 BEARING STRENGTH, SPACING, AND EDGE-DISTANCE REQUIREMENTS

Bearing strength is independent of the type of fastener because the stress under consideration is on the part being connected rather than on the fastener. For this reason, bearing strength, as well as spacing and edge-distance requirements, which also are independent of the type of fastener, will be considered before bolt shear and tensile strength.

The AISC Specification provisions for bearing strength, as well as all the requirements for high-strength bolts, are based on the provisions of the specification of the Research Council on Structural Connections (RCSC, 2009). The following discussion, which is based on the commentary that accompanies the RCSC specification, explains the basis of the AISC specification equations for bearing strength.

A possible failure mode resulting from excessive bearing is shear tear-out at the end of a connected element, as shown in Figure 7.7a. If the failure surface is idealized as shown in Figure 7.7b, the failure load on one of the two surfaces is equal to the shear fracture stress times the shear area, or

\[
\frac{R_u}{2} = 0.6F_u l_c t
\]

where

- \( 0.6F_u \) = shear fracture stress of the connected part
- \( l_c \) = distance from edge of hole to edge of connected part
- \( t \) = thickness of connected part
Welcome

The purpose of the RCSC is:

To stimulate and support such investigation as may be deemed necessary and valuable to determine the suitability, strength and behavior of various types of structural connections.

To promote the knowledge of economical and efficient practices relating to such structural connections.

To prepare and publish related standards and such other documents as necessary to achieving its purpose.

About Us

The RCSC is a non-profit, volunteer organization, comprised of over 65 leading experts in the field of structural steel connection design, engineering, fabrication, erection and bolting. Previous, current, and future research projects funded by the RCSC serve to provide safety, reliability, and standard practice for the steel construction industry throughout the world.

The RCSC is actively soliciting research contributions to further our efforts to provide meaningful research, clear specifications, and practical application advice for our industry. Membership in the RCSC is open to any qualified individual, corporation, or organization in accordance with our bylaws.

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Estes Park, Colorado
7.3 Bearing Strength, Spacing, and Edge-Distance Requirements

The bearing problem can be complicated by the presence of a nearby bolt or the proximity of an edge in the direction of the load, as shown in Figure 7.6. The bolt spacing and edge-distance will have an effect on the bearing strength.

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\[
\frac{R_n}{2} = 0.6F_uL_c
\]

where

\[
F_u = 0.6F_{ut} = 0.6F_u
\]

\[
0.6F_u = \text{shear fracture stress of the connected part}
\]

\[
L_c = \text{distance from edge of hole to edge of connected part}
\]

\[
t = \text{thickness of connected part}
\]

The total strength is

\[
R_n = 2(0.6F_uL_c t) = 1.2F_uL_c t
\]

\[
\left(\begin{array}{c}
\frac{J3-6a}{10.1-11}
\end{array}\right)
\]

(7.1)
The total strength is

\[ R_u = 2(0.6F_u \ell_c t_f) = 1.2F_u \ell_c t_f \] (7.1)

This tear-out can take place at the edge of a connected part, as shown, or between two holes in the direction of the bearing load. To prevent excessive elongation of the hole, an upper limit is placed on the bearing load given by Equation 7.1. This upper limit is proportional to the projected bearing area times the fracture stress, or

\[ R_u = C \times \text{bearing area} \times F_u = CdtF_u \] (7.2)

where
- \( C \) = a constant
- \( d \) = bolt diameter
- \( t \) = thickness of the connected part

The AISC Specification uses Equation 7.1 for bearing strength, subject to an upper limit given by Equation 7.2. If excessive deformation at service load is a concern, and it usually is, \( C \) is taken as 2.4. This value corresponds to a hole elongation of about \( 1/4 \) inch (RCSC, 2009). In this book, we consider deformation to be a design consideration. The nominal bearing strength of a single bolt therefore can be expressed as

\[ R_u = 1.2\ell_c t_F_u \leq 2.4dtF_u \] (AISC Equation J3-6a)

where
- \( \ell_c \) = clear distance, in the direction parallel to the applied load, from the edge of the bolt hole to the edge of the adjacent hole or to the edge of the material
- \( t \) = thickness of the connected part
- \( F_u \) = ultimate tensile stress of the connected part (not the bolt)
Max seems like it should be:
\[ d_t F_u \]

But you get to have:
\[ 2.4 d_t F_u \]

Why?

Steel is trapped under the head of the bolt & also under nut on other side.

\[ d + \frac{1}{16} + \frac{1}{16} \]

\[ h = d + \frac{1}{16} \]

\[ h \text{ for bearing} \]
This tear-out can take place at the edge of a connected part, as shown, or between two holes in the direction of the bearing load. To prevent excessive elongation of the hole, an upper limit is placed on the bearing load given by Equation 7.1. This upper limit is proportional to the projected bearing area times the fracture stress, or

\[ R_n = C \times \text{bearing area} \times F_u = CdtF_u \]  

(7.2)

where

\begin{align*}
C & = \text{a constant} \\
d & = \text{bolt diameter} \\
t & = \text{thickness of the connected part}
\end{align*}

The AISC Specification uses Equation 7.1 for bearing strength, subject to an upper limit given by Equation 7.2. If excessive deformation at service load is a concern, and it usually is, \( C \) is taken as 2.4. This value corresponds to a hole elongation of about \( \frac{1}{8} \) inch (RCSC, 2004). In this book, we consider deformation to be a design consideration. The nominal bearing strength of a single bolt therefore can be expressed as

\[ R_n = 1.2L_c t F_u \leq 2.4dtF_u \]  

(AISC Equation J3-6a)

where

\begin{align*}
L_c & = \text{clear distance, in the direction parallel to the applied load, from the edge of the bolt hole to the edge of the adjacent hole or to the edge of the material} \\
t & = \text{thickness of the connected part} \\
F_u & = \text{ultimate tensile stress of the connected part (not the bolt)}
\end{align*}

For load and resistance factor design, the resistance factor is \( \phi = 0.75 \), and the design strength is

\[ \phi R_n = 0.75R_n \]

\[ \frac{\phi}{R_n} \]
For load and resistance factor design, the resistance factor is $\phi = 0.75$, and the design strength is

$$\phi R_n = 0.75 R_n$$

For allowable strength design, the safety factor is $\Omega = 2.00$, and the allowable strength is

$$\frac{R_m}{\Omega} = \frac{R_n}{2.00}$$

Figure 7.8 further illustrates the distance $l_c$. When computing the bearing strength for a bolt, use the distance from that bolt to the adjacent bolt or edge in the direction of the bearing load on the connected part. For the case shown, the bearing load would be on the left side of each hole. Thus the strength for bolt 1 is calculated with $l_c$ measured to the edge of bolt 2, and the strength for bolt 2 is calculated with $l_c$ measured to the edge of the connected part.

For the edge bolts, use $l_c = l_e - h/2$. For other bolts, use $l_c = s - h$,

where

- $l_e$ = edge-distance to center of the hole
- $s$ = center-to-center spacing of holes
- $h$ = hole diameter

AISC Equation J3-6a is valid for standard, oversized, short-slotted and long-slotted holes with the slot parallel to the load. We use only standard holes in this book (holes $\frac{1}{16}$-inch larger than the bolt diameter). For those cases where deformation is not a design consideration, and for long-slotted holes with the slot perpendicular to the direction of the load, AISC gives other strength expressions.

When computing the distance $l_{ct}$, use the actual hole diameter (which is $\frac{1}{16}$-inch larger than the bolt diameter), and do not add the $\frac{1}{16}$ inch as required in AISC B4.3b for computing the net area for tension and shear. In other words, use a hole diameter of

$$h = d + \frac{1}{16} \text{ in.}$$

not $d + \frac{1}{8}$ inch (although if $d + \frac{1}{8}$ were used, the slight error would be on the conservative side).
7.3 Bearing Strength, Spacing, and Edge-Distance Requirements

For allowable strength design, the safety factor is $\Omega = 2.00$, and the allowable strength is:

$$\frac{R}{\Omega} = \frac{R_a}{2.00}$$

Figure 7.8 further illustrates the distance $L_c$. When computing the bearing strength for a bolt, use the distance from that bolt to the adjacent bolt or edge in the direction of the bearing load on the connected part. For the case shown, the bearing load would be on the left side of each hole. Thus the strength for bolt 1 is calculated with $L_c$ measured to the edge of bolt 2, and the strength for bolt 2 is calculated with $L_c$ measured to the edge of the connected part.

For the edge bolts, use $L_c = L_e - h/2$. For other bolts, use $L_c = s - h$.

where

$L_e$ = edge-distance to center of the hole
$s$ = center-to-center spacing of holes
$h$ = hole diameter

AISC Equation J3-6a is valid for standard, oversized, short-slotted and long-slotted holes with the slot parallel to the load. We use only standard holes in this book (holes $\frac{1}{16}$-inch larger than the bolt diameter). For those cases where deformation is not a design consideration, and for long-slotted holes with the slot perpendicular to the direction of the load, AISC gives other strength expressions.

When computing the distance $L_c$, use the actual hole diameter (which is $\frac{1}{16}$-inch larger than the bolt diameter), and do not add the $\frac{1}{16}$ inch as required in AISC D3.2 for computing the net area for tension and shear. In other words, use a hole diameter of $h = d + \frac{1}{16}$ in. not $d + \frac{1}{8}$ inch (although if $d + \frac{1}{8}$ were used, the slight error would be on the conservative side).

Spacing and Edge-Distance Requirements

To maintain clearances between bolt nuts and to provide room for wrench sockets, AISC J3.3 requires that center-to-center spacing of fasteners (in any direction) be no less than $2\frac{3}{4}d$ and preferably no less than $3d$, where $d$ is the fastener diameter.
Spacing and Edge-Distance Requirements

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Summary of Bearing Strength, Spacing, and Edge-Distance Requirements (Standard Holes)

a. Bearing strength:
\[ R_n = 1.24dF_u \leq 2.4dtF_u \] (AISC Equation J3-6a)

b. Minimum spacing and edge distance: In any direction, both in the line of force and transverse to the line of force,
\[ s \geq 2\frac{3}{4}d \quad \text{(preferably } 3d) \] \[ \ell_e \geq \text{value from AISC Table J3.4} \]

For single- and double-angle shapes, the usual gage distances given in Table 1-7A in Part I of the Manual (see Section 3.6) may be used in lieu of these minimums.

**EXAMPLE 7.1**

Check bolt spacing, edge distances, and bearing for the connection shown in Figure 7.10.

**SOLUTION**

From AISC J3.3, the minimum spacing in any direction is

\[ 2\frac{3}{4}d = 2.667 \left( \frac{3}{4} \right) = 2.00 \text{ in.} \]

Actual spacing = 2.50 in. > 2.00 in. (OK)
When Group B bolts over 1 in. (25 mm) in diameter are used in slotted or oversized holes in external plies, a single hardened washer conforming to ASTM F436, except with $\frac{5}{16}$-in. (8 mm) minimum thickness, shall be used in lieu of the standard washer.

User Note: Washer requirements are provided in the RCSC Specification, Section 6.

Long-slotted holes are permitted in only one of the connected parts of either a slip-critical or bearing-type connection at an individual faying surface. Long-slotted holes are permitted without regard to direction of loading in slip-critical connections, but shall be normal to the direction of load in bearing-type connections. Where long-slotted holes are used in an outer ply, plate washers, or a continuous bar with standard holes, having a size sufficient to completely cover the slot after installation, shall be provided. In high-strength bolted connections, such plate washers or continuous bars shall be not less than $\frac{5}{16}$-in. (8 mm) thick and shall be of structural grade material, but need not be hardened. If hardened washers are required for use of high-strength bolts, the hardened washers shall be placed over the outer surface of the plate washer or bar.

3. **Minimum Spacing**

The distance between centers of standard, oversized or slotted holes shall not be less than $2^{2/3}$ times the nominal diameter, $d$, of the fastener; a distance of $3d$ is preferred.

User Note: ASTM F1554 anchor rods may be furnished in accordance to product specifications with a body diameter less than the nominal diameter. Load effects such as bending and elongation should be calculated based on minimum diameters permitted by the product specification. See ASTM F1554 and the table, “Applicable ASTM Specifications for Various Types of Structural Fasteners,” in Part 2 of the AISC Steel Construction Manual.

4. **Minimum Edge Distance**

The distance from the center of a standard hole to an edge of a connected part in any direction shall not be less than either the applicable value from Table J3.4 or Table J3.4M, or as required in Section J3.10. The distance from the center of an oversized or slotted hole to an edge of a connected part shall be not less than that required for a standard hole to an edge of a connected part plus the applicable increment, $C_2$, from Table J3.5 or Table J3.5M.

User Note: The edge distances in Tables J3.4 and J3.4M are minimum edge distances based on standard fabrication practices and workmanship tolerances. The appropriate provisions of Sections J3.10 and J4 must be satisfied.

5. **Maximum Spacing and Edge Distance**

The maximum distance from the center of any bolt to the nearest edge of parts in contact shall be 12 times the thickness of the connected part under consideration,
### TABLE J3.4
Minimum Edge Distance\textsuperscript{[a]} from Center of Standard Hole\textsuperscript{[b]} to Edge of Connected Part, in.

<table>
<thead>
<tr>
<th>Bolt Diameter, in.</th>
<th>Minimum Edge Distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \frac{1}{2} )</td>
<td>( \frac{3}{4} )</td>
</tr>
<tr>
<td>( \frac{5}{8} )</td>
<td>( 7/8 )</td>
</tr>
<tr>
<td>( \frac{3}{4} )</td>
<td>( 1 )</td>
</tr>
<tr>
<td>( \frac{7}{8} )</td>
<td>( 1 \frac{1}{8} )</td>
</tr>
<tr>
<td>1</td>
<td>1( \frac{1}{4} )</td>
</tr>
<tr>
<td>1( \frac{1}{8} )</td>
<td>1( \frac{1}{2} )</td>
</tr>
<tr>
<td>1( \frac{1}{4} )</td>
<td>1( \frac{5}{8} )</td>
</tr>
<tr>
<td>Over 1( \frac{1}{4} )</td>
<td>1( \frac{1}{4} \times d )</td>
</tr>
</tbody>
</table>

\textsuperscript{[a]} If necessary, lesser edge distances are permitted provided the appropriate provisions from Sections J3.10 and J4 are satisfied, but edge distances less than one bolt diameter are not permitted without approval from the engineer of record.

\textsuperscript{[b]} For oversized or slotted holes, see Table J3.5.

### TABLE J3.4M
Minimum Edge Distance\textsuperscript{[a]} from Center of Standard Hole\textsuperscript{[b]} to Edge of Connected Part, mm

<table>
<thead>
<tr>
<th>Bolt Diameter, mm</th>
<th>Minimum Edge Distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>16</td>
<td>22</td>
</tr>
<tr>
<td>20</td>
<td>26</td>
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<tr>
<td>22</td>
<td>28</td>
</tr>
<tr>
<td>24</td>
<td>30</td>
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<tr>
<td>27</td>
<td>34</td>
</tr>
<tr>
<td>30</td>
<td>38</td>
</tr>
<tr>
<td>36</td>
<td>46</td>
</tr>
<tr>
<td>Over 36</td>
<td>1.25( d )</td>
</tr>
</tbody>
</table>

\textsuperscript{[a]} If necessary, lesser edge distances are permitted provided the appropriate provisions from Sections J3.10 and J4 are satisfied, but edge distances less than one bolt diameter are not permitted without approval from the engineer of record.

\textsuperscript{[b]} For oversized or slotted holes, see Table J3.5M.