CHAPTER G
DESIGN OF MEMBERS FOR SHEAR

This chapter addresses webs of singly or doubly symmetric members subject to shear in the plane of the web, single angles and HSS sections, and shear in the weak direction of singly or doubly symmetric shapes.

The chapter is organized as follows:

G2. Members with Unstiffened or Stiffened Webs
G3. Tension Field Action
G4. Single Angles
G5. Rectangular HSS and Box-Shaped Members
G6. Round HSS
G7. Weak Axis Shear in Doubly Symmetric and Singly Symmetric Shapes
G8. Beams and Girders with Web Openings

User Note: For cases not included in this chapter, the following sections apply:
- H3.3 Unsymmetric sections
- J4.2 Shear strength of connecting elements
- J10.6 Web panel zone shear

G1. GENERAL PROVISIONS

Two methods of calculating shear strength are presented below. The method presented in Section G2 does not utilize the post buckling strength of the member (tension field action). The method presented in Section G3 utilizes tension field action.

The design shear strength, $\phi_v V_n$, and the allowable shear strength, $V_n/\Omega_v$, shall be determined as follows:

For all provisions in this chapter except Section G2.1(a):

$$\phi_v = 0.90 \text{ (LRFD)} \quad \Omega_v = 1.67 \text{ (ASD)}$$

G2. MEMBERS WITH UNSTIFFENED OR STIFFENED WEBs

1. Shear Strength

This section applies to webs of singly or doubly symmetric members and channels subject to shear in the plane of the web.

The nominal shear strength, $V_n$, of unstiffened or stiffened webs according to the limit states of shear yielding and shear buckling, is

$$V_n = 0.6 F_y A_w C_v \quad \text{(G2-1)}$$
MEMBERS WITH UNSTIFFENED OR STIFFENED WEB

For webs of rolled I-shaped members with \( h/t_w \leq 2.24\sqrt{E/F_y} \):

\[ \phi_v = 1.00 \text{ (LRFD)} \quad \Omega_x \leq 0.85 \frac{F_{y}}{F} \]

and

\[ C_v = 1.0 \]  

\[(G2-2)\]

User Note: All current ASTM A6 W, S and HP shapes except W44x230, W40x149, W36x135, W33x118, W30x90, W24x55, W16x26 and W12x14 meet the criteria stated in Section G2.1(a) for \( F_y = 50 \text{ ksi} \) (345 MPa).

(b) For webs of all other doubly symmetric shapes and singly symmetric shapes and channels, except round HSS, the web shear coefficient, \( C_v \), is determined as follows:

(i) When \( h/t_w \leq 1.10\sqrt{k_v E/F_y} \)

\[ C_v = 1.0 \]  

\[(G2-3)\]

(ii) When \( 1.10\sqrt{k_v E/F_y} < h/t_w \leq 1.37\sqrt{k_v E/F_y} \)

\[ C_v = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \]  

\[(G2-4)\]

(iii) When \( h/t_w > 1.37\sqrt{k_v E/F_y} \)

\[ C_v = \frac{1.51k_v E}{(h/t_w)^2 F_y} \]  

\[(G2-5)\]

where
- \( A_w \) = area of web, the overall depth times the web thickness, \( dt_w \), in.\(^2\) (mm\(^2\))
- \( h \) = for rolled shapes, the clear distance between flanges less the fillet or corner radii, in. (mm)
- \( h/t_w \) = for built-up welded sections, the clear distance between flanges, in. (mm)
- \( h/t_w \) = for built-up bolted sections, the distance between fastener lines, in. (mm)
- \( t_w \) = for tees, the overall depth, in. (mm)

The web plate shear buckling coefficient, \( k_v \), is determined as follows:

(i) For webs without transverse stiffeners and with \( h/t_w < 260 \):

\[ k_v = 5 \]

except for the stem of tee shapes where \( k_v = 1.2 \),
(ii) For webs with transverse stiffeners:

\[
 k_v = 5 + \frac{5}{(a/h)^2}
\]

= 5 when \(a/h > 3.0\) or \(a/h \geq \left[ \frac{260}{(h/t_w)} \right]^2\)

where \(a\) = clear distance between transverse stiffeners, in. (mm)

**User Note:** For all ASTM A6 W, S, M and HP shapes except M12.5×12.4, M12.5×11.6, M12×11.8, M12×10.8, M12×10, M10×8, M10×7.5, when \(F_y = 50\) ksi (345 MPa), \(C_v = 1.0\).

2. **Transverse Stiffeners**

Transverse stiffeners are not required where \(h/t_w \leq 2.46\sqrt{E/F_y}\), or where the available shear strength provided in accordance with Section G2.1 for \(k_v = 5\) is greater than the *required shear strength*.

The moment of inertia, \(I_{st}\), of transverse stiffeners used to develop the available web shear strength, as provided in Section G2.1, about an axis in the web center for stifferener pairs or about the face in contact with the web plate for single stiffeners, shall meet the following requirement

\[
 I_{st} \geq b t_w^3 j
\]

where

\[
 j = \frac{2.5}{(a/h)^2} - 2 \geq 0.5
\]

and \(b\) is the smaller of the dimensions \(a\) and \(h\)

Transverse stiffeners are permitted to be stopped short of the tension flange, provided *bearing* is not needed to transmit a concentrated load or reaction. The weld by which transverse stiffeners are attached to the web shall be terminated not less than four times nor more than six times the web thickness from the near toe to the web-to-flange weld. When single stiffeners are used, they shall be attached to the compression flange, if it consists of a rectangular plate, to resist any uplift tendency due to torsion in the flange.

Bolts connecting stiffeners to the girder web shall be spaced not more than 12 in. (305 mm) on center. If intermittent *fillet welds* are used, the clear distance between welds shall not be more than 16 times the web thickness nor more than 10 in. (250 mm).
and the web area is $A_w = d t_w = 14.0(0.440) = 6.160$ in.$^2$

$$2.24 \frac{E}{F_y} = 2.24 \sqrt{\frac{29,000}{50}} = 54.0 \quad [1-24] (221a)$$

Since

$$25.9 = \frac{h}{t_w} < 2.24 \sqrt{\frac{E}{F_y}} = 54$$

the strength is governed by shear yielding of the web and $C_v = 1.0$. (As pointed out in the Specification User Note, this will be the case for most W shapes with $F_y \leq 50$ ksi.) The nominal shear strength is

$$V_n = 0.6F_y A_w C_v = 0.6(50)(6.160)(1.0) = 184.8 \text{ kips}$$

Determine the resistance factor $\phi_v$.

$$\phi_v = 1.00$$

and the design shear strength is

$$\phi_v V_n = 1.00(184.8) = 185 \text{ kips}$$

From Example 5.6, $w_u = 2.080$ kips/ft and $L = 45$ ft. For a simply supported, uniformly loaded beam, the maximum shear occurs at the support and is equal to the reaction.

$$V_u = \frac{w_u L}{2} = \frac{2.080(45)}{2} = 46.8 \text{ kips} \leq 185 \text{ kips} \quad \text{(OK)}$$

Also see [3-24] (221c)
### Table 1-1 (continued)  
**W-Shapes Dimensions**

<table>
<thead>
<tr>
<th>Shape</th>
<th>Area, $A$</th>
<th>Depth, $d$</th>
<th>$t_w$</th>
<th>$t_f$</th>
<th>Width, $b_f$</th>
<th>Thickness, $t_f$</th>
<th>$k$</th>
<th>$T$</th>
<th>Workable Gage</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\times 132$</td>
<td>38.8</td>
<td>14.7</td>
<td>$14^{1/2}$</td>
<td>0.645</td>
<td>$9^{1/2}$</td>
<td>$14^{1/2}$</td>
<td>1.03</td>
<td>1</td>
<td>$2^{1/2}$</td>
</tr>
<tr>
<td>$\times 120$</td>
<td>35.3</td>
<td>14.5</td>
<td>$14^{1/2}$</td>
<td>0.590</td>
<td>$9^{1/2}$</td>
<td>$14^{1/2}$</td>
<td>0.940</td>
<td>$15^{1/2}$</td>
<td>$2^{1/2}$</td>
</tr>
<tr>
<td>$\times 109$</td>
<td>32.0</td>
<td>14.3</td>
<td>$14^{1/2}$</td>
<td>0.525</td>
<td>$1^{1/2}$</td>
<td>$14^{1/2}$</td>
<td>0.860</td>
<td>$7^{1/2}$</td>
<td>$2^{1/2}$</td>
</tr>
<tr>
<td>$\times 99$</td>
<td>28.1</td>
<td>14.2</td>
<td>$14^{1/2}$</td>
<td>0.485</td>
<td>$1^{1/2}$</td>
<td>$14^{1/2}$</td>
<td>0.780</td>
<td>$3^{1/2}$</td>
<td>$2^{1/2}$</td>
</tr>
<tr>
<td>$\times 90$</td>
<td>26.5</td>
<td>14.0</td>
<td>0.440</td>
<td>$1^{1/2}$</td>
<td>$14^{1/2}$</td>
<td>0.710</td>
<td>$1^{3/4}$</td>
<td>$2^{1/2}$</td>
<td></td>
</tr>
<tr>
<td>$\times 82$</td>
<td>24.0</td>
<td>14.3</td>
<td>$14^{1/2}$</td>
<td>0.510</td>
<td>$1^{1/2}$</td>
<td>$10^{1/2}$</td>
<td>0.855</td>
<td>$7^{1/2}$</td>
<td>$1^{1/2}$</td>
</tr>
<tr>
<td>$\times 74$</td>
<td>21.8</td>
<td>14.2</td>
<td>$14^{1/2}$</td>
<td>0.450</td>
<td>$1^{1/2}$</td>
<td>$10^{1/2}$</td>
<td>0.785</td>
<td>$3^{1/4}$</td>
<td>$1^{1/2}$</td>
</tr>
<tr>
<td>$\times 68$</td>
<td>20.0</td>
<td>14.0</td>
<td>0.415</td>
<td>$1^{1/2}$</td>
<td>$10^{1/2}$</td>
<td>0.720</td>
<td>$1^{1/2}$</td>
<td>$1^{1/2}$</td>
<td></td>
</tr>
<tr>
<td>$\times 61$</td>
<td>17.9</td>
<td>13.9</td>
<td>$13^{1/2}$</td>
<td>0.375</td>
<td>$3^{1/8}$</td>
<td>$10^{1/2}$</td>
<td>0.645</td>
<td>$5^{1/2}$</td>
<td>$1^{1/2}$</td>
</tr>
</tbody>
</table>

$^8$ Shape is slender for compression with $F_p = 50$ ksf.

$^9$ Shape exceeds compact limit for flexure with $F_p = 50$ ksf.

$^*6$ The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.

$^*6$ Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1a.
### Table 1-1 (continued)

#### W-Shapes

Properties

<table>
<thead>
<tr>
<th>Nominal Wt</th>
<th>Compact Section Criteria</th>
<th>Axis X-X</th>
<th>Axis Y-Y</th>
<th>Torsional Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$b_h$ $h$ $t_w$</td>
<td>$l$ $S$ $r$ $Z$</td>
<td>$l$ $S$ $r$ $Z$</td>
<td>$J$ $h_b$ $C_w$</td>
</tr>
<tr>
<td>lb/ft</td>
<td>in.</td>
<td>in. $^4$</td>
<td>in. $^3$</td>
<td>in. $^3$</td>
</tr>
<tr>
<td>132</td>
<td>7.15</td>
<td>17.7</td>
<td>1530</td>
<td>209</td>
</tr>
<tr>
<td>120</td>
<td>7.60</td>
<td>19.3</td>
<td>1380</td>
<td>190</td>
</tr>
<tr>
<td>109</td>
<td>8.49</td>
<td>21.7</td>
<td>1240</td>
<td>173</td>
</tr>
<tr>
<td>99</td>
<td>9.34</td>
<td>23.5</td>
<td>1110</td>
<td>157</td>
</tr>
<tr>
<td>90</td>
<td>10.2</td>
<td>25.5</td>
<td>999</td>
<td>143</td>
</tr>
<tr>
<td>82</td>
<td>5.92</td>
<td>22.4</td>
<td>881</td>
<td>123</td>
</tr>
<tr>
<td>74</td>
<td>5.41</td>
<td>25.4</td>
<td>795</td>
<td>112</td>
</tr>
<tr>
<td>68</td>
<td>6.97</td>
<td>27.5</td>
<td>722</td>
<td>103</td>
</tr>
</tbody>
</table>

8/  7.48 18.9  740 |  118 |  5.39 |  132 |  241 |  39.7 |  3.07 |  60.4 |  3.46 |  11.7 |  5.10 |  8270 |
79  6.22 20.7  662 |  107 |  5.34 |  119 |  216 |  35.8 |  3.05 |  54.3 |  3.43 |  11.7 |  3.84 |  7330 |
72  6.99 22.6  597 |  97.4 |  5.31 |  108 |  195 |  32.4 |  3.04 |  49.2 |  3.41 |  11.6 |  2.93 |  6540 |
65  6.92 24.9  533 |  87.9 |  5.28 |  96.8 |  174 |  29.1 |  3.02 |  44.1 |  3.38 |  11.5 |  2.18 |  5780 |

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### Table 3-2 (continued)

#### W Shapes

Selection by $Z_x$

<table>
<thead>
<tr>
<th>Shape</th>
<th>$Z_x$</th>
<th>$M_{px}/\Omega_x$</th>
<th>$\phi_x M_{px}$</th>
<th>$M_{tx}/\Omega_x$</th>
<th>$\phi_x M_{tx}$</th>
<th>BF</th>
<th>$L_p$</th>
<th>$L_f$</th>
<th>$I_x$</th>
<th>$V_{px}/\Omega_x$</th>
<th>$\phi_x V_{px}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>W12×120</td>
<td>186</td>
<td>594</td>
<td>636</td>
<td>269</td>
<td>404</td>
<td>14.1</td>
<td>21.2</td>
<td>6.61</td>
<td>18.8</td>
<td>1830</td>
<td>197</td>
</tr>
<tr>
<td>W24×68</td>
<td>177</td>
<td>442</td>
<td>644</td>
<td>269</td>
<td>404</td>
<td>14.1</td>
<td>21.2</td>
<td>6.61</td>
<td>18.8</td>
<td>1830</td>
<td>197</td>
</tr>
<tr>
<td>W16×89</td>
<td>155</td>
<td>337</td>
<td>656</td>
<td>271</td>
<td>407</td>
<td>7.74</td>
<td>11.6</td>
<td>8.90</td>
<td>30.2</td>
<td>1300</td>
<td>176</td>
</tr>
<tr>
<td>W14×99fl</td>
<td>173</td>
<td>430</td>
<td>646</td>
<td>274</td>
<td>412</td>
<td>4.99</td>
<td>7.35</td>
<td>13.5</td>
<td>45.3</td>
<td>1110</td>
<td>110</td>
</tr>
<tr>
<td>W21×73</td>
<td>172</td>
<td>429</td>
<td>645</td>
<td>264</td>
<td>396</td>
<td>12.9</td>
<td>19.4</td>
<td>6.39</td>
<td>19.2</td>
<td>1600</td>
<td>193</td>
</tr>
<tr>
<td>W21×106</td>
<td>164</td>
<td>399</td>
<td>615</td>
<td>253</td>
<td>381</td>
<td>3.93</td>
<td>5.90</td>
<td>11.0</td>
<td>50.7</td>
<td>933</td>
<td>167</td>
</tr>
<tr>
<td>W18×76</td>
<td>163</td>
<td>407</td>
<td>611</td>
<td>255</td>
<td>383</td>
<td>8.49</td>
<td>12.8</td>
<td>9.22</td>
<td>27.1</td>
<td>1330</td>
<td>155</td>
</tr>
<tr>
<td>W21×68</td>
<td>160</td>
<td>395</td>
<td>600</td>
<td>245</td>
<td>368</td>
<td>12.5</td>
<td>18.8</td>
<td>6.36</td>
<td>18.7</td>
<td>1480</td>
<td>182</td>
</tr>
<tr>
<td>W2×90fl</td>
<td>157</td>
<td>392</td>
<td>573</td>
<td>250</td>
<td>375</td>
<td>4.80</td>
<td>7.22</td>
<td>15.2</td>
<td>42.6</td>
<td>999</td>
<td>123</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>ASTM</th>
<th>LRFD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ωₐ = 1.67</td>
<td>0.93</td>
</tr>
<tr>
<td>Ωₜ = 1.50</td>
<td>1.00</td>
</tr>
</tbody>
</table>

* Shape exceeds compact limit for flexure with $F_y = 50$ ksi.
* Shape does not meet the $M_{tx}$ limit for shear in Specification Section G2.1a with $F_y = 50$ ksi.

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Values of $\phi, V_n, \text{ and } A_T$ are given in several tables in Part 3 of the Manual, including the $Z_x$ table, so computation of shear strength is unnecessary for hot-rolled shapes.

**Block Shear**

FIGURE 5.20
Values of $\phi$, $V_n$, and $V_n/\Omega_n$ are given in several tables in Part 3 of the *Manual*, including the $Z_x$ table, so computation of shear strength is unnecessary for hot-rolled shapes.

Block Shear

(Figure with annotations)

**See pg 3-24 (22.1 C)**
5.9 DEFLECTION

In addition to being safe, a structure must be serviceable. A serviceable structure is one that performs satisfactorily, not causing any discomfort or perceptions of unsafety, the structure. For a beam, being serviceable usually primarily

Excessive deflection is usually an indication of problems with vibrations. The deflection attached to the beam can be damaged structure may view large deflections negatively and wrongly assume that the structure is unsafe.

For the common case of a simply supported, uniformly loaded beam such as that in Figure 5.22, the maximum vertical deflection is

\[ \Delta = \frac{5}{384} \frac{wL^4}{EI} \]
Table 1–1 (continued)

**W Shapes**

**Dimensions**

<table>
<thead>
<tr>
<th>Shape</th>
<th>Area, ( A )</th>
<th>Depth, ( d )</th>
<th>Web Thickness, ( t_w )</th>
<th>Flange Width, ( b_f )</th>
<th>Flange Thickness, ( t_f )</th>
<th>Distance ( k )</th>
<th>( k_{dez} )</th>
<th>( k_{def} )</th>
<th>Workable Gage</th>
</tr>
</thead>
<tbody>
<tr>
<td>W21x93</td>
<td>27.3</td>
<td>21.6</td>
<td>21/32 0.580 9/32 1/32</td>
<td>9/32 8.42 8/32</td>
<td>8.36 8/32 0.930 15/32</td>
<td>1.43 1/4</td>
<td>3/16 1/4</td>
<td>3/16 1/4</td>
<td>18 7/8 5/16</td>
</tr>
<tr>
<td>W3x83c</td>
<td>24.3</td>
<td>21.4</td>
<td>21/32 0.515 5/32 1/32</td>
<td>4/32 8.42 8/32</td>
<td>8.36 8/32 0.835 1/32</td>
<td>1.34 1/2</td>
<td>3/16 1/2</td>
<td>3/16 1/2</td>
<td>18 7/8 5/16</td>
</tr>
</tbody>
</table>

---

6. Shape is slender for compression with \( F_c = 50 \) ksi.
7. Shape exceeds compact limit for flexure with \( F_c = 50 \) ksi.
8. The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.
9. Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

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standard analytical methods such as the method of virtual work may be used. Deflection is a serviceability limit state, not one of strength, so deflections should always be computed with service loads. The appropriate limit for the maximum deflection depends on the function of the beam and the likelihood of damage resulting from the deflection. The AISC Specification furnishes little guidance other than a statement in Chapter L, "Design for Serviceability," that deflections should not be excessive. There is, however, a more detailed discussion in the Commentary to Chapter L. Appropriate limits for deflection can usually be found from the governing building code, expressed as a fraction of the span length L, such as L/360. Sometimes a numer-

The limits shown in Table 5.4 for deflection due to dead load plus live load do not apply to steel beams, because the dead load deflection is usually compensated for by some means, such as cambering. Camber is a curvature in the opposite direction of the dead load deflection curve and can be accomplished by bending the beam, with or without heat. When the dead load is applied to the cambered beam, the curvature is removed, and the beam becomes level. Therefore, only the live load deflection is of concern in the completed structure. Dead load deflection can also be accounted for by pouring a variable depth slab with a level top surface, the variable depth being a consequence of the deflection of the beam (this is referred to as ponding of the concrete). Detailed coverage of control of dead load deflection is given in an AISC seminar series (AISC, 1997a) and several papers (Ruddy, 1986; Ricker, 1989; and Larson and Huzzard, 1990).

TABLE 5.4
Deflection Limits
### Table 3-23
Shears, Moments and Deflections

1. SIMPLE BEAM — UNIFORMLY DISTRIBUTED LOAD

<table>
<thead>
<tr>
<th>Equation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Equip. Uniform Load</td>
<td>$= \frac{wI}{2}$</td>
</tr>
<tr>
<td>$R = V$</td>
<td></td>
</tr>
<tr>
<td>$V_x$</td>
<td>$= w \left(\frac{I}{2} - x\right)$</td>
</tr>
<tr>
<td>$M_{max}$ (at center)</td>
<td>$= \frac{wI^2}{12}$</td>
</tr>
<tr>
<td>$M_x$</td>
<td>$= \frac{wx}{2} \left(\frac{I}{2} - x\right)$</td>
</tr>
<tr>
<td>$\Delta_{max}$ (at center)</td>
<td>$= \frac{5wI^4}{384EI}$</td>
</tr>
<tr>
<td>$\Delta_x$</td>
<td>$= \frac{wx}{24EI} \left(\frac{I^3}{2} - 2I^2 + x^3\right)$</td>
</tr>
</tbody>
</table>

2. SIMPLE BEAM — LOAD INCREASING UNIFORMLY TO ONE END

---

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Compute $\phi_b M_p$ for the beam shown:

$F_y = 50 \text{kpsi}$
$F_u = 65 \text{kpsi}$

$Z_{\text{top}} = 8'(1')(0.5'') = 4 \text{in}^3$
$Z_{\text{bottom}} = (1')(8')(4'') = 32 \text{in}^3$
$Z = 36 \text{in}^3$

$\phi_b M_p = 0.9 (50 \text{kpsi})(36 \text{in}^3)$
$= 1620 \text{ kips}$

$= 135 \text{ kips}$
Compute $\phi M_p$ for the beam shown:

$F_y = 50$ ksi

$F_u = 65$ ksi

$Z_x = (8\text{''})(1\text{''})(1.5\text{''}) + (1)(1)(0.5) + (9)(1)(4.5) = \ldots$

$M_p = F_y Z_x$

$M_u \leq \phi M_p$
1. Compute the required moment strength (i.e., the factored load moment $M_r$ for LRFD or the unfactored moment $M_u$ for ASD). The weight of the beam is part of the dead load but is unknown at this point. A value may be assumed and verified after a shape is selected, or the weight may be ignored initially and checked afterwards.

2. Select a shape that satisfies this strength requirement. This can be done in one of two ways.
   a. Assume a shape, compute the available strength, and compare it with the required strength. Revise if necessary. The trial shape can be easily selected in only a limited number of situations (as in Example 5.10).
   b. Use the beam design charts in Part 3 of the *Manual*. This method is preferred, and we explain it following Example 5.10.

3. Check the shear strength.
4. Check the deflection.

---

**Example 5.10**

Select a standard hot-rolled shape of A992 steel for the beam shown in Figure 5.24. The beam has continuous lateral support and must support a uniform service live load of 4.5 kips/ft. The maximum permissible live load deflection is $L/240$.

**Figure 5.24**

Ignore the beam weight initially then check for its effect after a selection is made.

$$w_u = 1.2w_D + 1.6w_L = 1.2(0) + 1.6(4.5) = 7.2 \text{ kips/ft}$$
From $\phi_M M_n \geq M_u$,
\[ \frac{\phi_M F_y Z_x}{\phi_M} \geq M_u \]
\[ Z_x \geq \frac{M_u}{\phi_M F_y} = \frac{810.0(12)}{0.90(50)} = 216 \text{ in.}^3 \]

Try a W24 x 84. This shape is compact, as assumed (noncompact shapes are marked as such in the table); therefore $M_n = M_p$, as assumed.

Account for the beam weight.
\[ w_u = 1.2w_D + 1.6w_L = 1.2(0.084) + 1.6(4.5) = 7.301 \text{ kips/ft} \]

Required moment strength \[ M_u = \frac{1}{8} w_u L^2 = \frac{1}{8} (7.301)(30)^2 = 821.4 \text{ ft-kips} \]

The required section modulus is
\[ Z_x = \frac{M_u}{\phi_M F_y} = \frac{821.4(12)}{0.90(50)} = 219 \text{ in.}^3 < 224 \text{ in.}^3 \text{ (OK)} \]

In lieu of basing the search on the required section modulus, the design strength $\phi_M M_p$ could be used, because it is directly proportional to $Z_x$ and is also tabulated. \[ \phi_M M_p \]

Next, check the shear:
\[ V_u = \frac{w_u L}{2} = \frac{7.301(30)}{2} = 110 \text{ kips} \]

From the $Z_x$ table,
\[ \phi_M V_n = 340 \text{ kips} > 110 \text{ kips} \text{ (OK)} \]

Finally, check the deflection. The maximum permissible live load deflection is $L/240 = (30 \times 12)/240 = 1.5 \text{ inch}$.
\[ \Delta_L = \frac{5}{384} \frac{w_L L^4}{E I_x} = \frac{5}{384} \frac{(4.5/12)(30 \times 12)^4}{29,000(2370)} = 1.19 \text{ in.} < 1.5 \text{ in.} \text{ (OK)} \]
### Table 3-2 (continued)

#### W Shapes

Selection by $Z_x$

<table>
<thead>
<tr>
<th>Shape</th>
<th>$Z_x$</th>
<th>$M_{pl}/\Omega_x$</th>
<th>$\phi_BM_{pl}$</th>
<th>$M_{pl}/\Omega_i$</th>
<th>$\phi_iM_{pl}$</th>
<th>BF</th>
<th>$L_p$</th>
<th>$L_T$</th>
<th>$L_x$</th>
<th>$V_y/\Omega_y$</th>
<th>$\phi_{V_y}$</th>
<th>kips</th>
<th>kips</th>
<th>kips</th>
<th>kips</th>
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<tr>
<td>W24×64</td>
<td>177</td>
<td>404</td>
<td>464</td>
<td>299</td>
<td>404</td>
<td>14.1</td>
<td>21.2</td>
<td>6.61</td>
<td>18.8</td>
<td>1830</td>
<td>197</td>
<td>153</td>
<td>279</td>
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<tr>
<td>W21×93</td>
<td>176</td>
<td>437</td>
<td>656</td>
<td>271</td>
<td>407</td>
<td>7.74</td>
<td>11.5</td>
<td>8.30</td>
<td>30.2</td>
<td>1300</td>
<td>176</td>
<td>153</td>
<td>264</td>
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<tr>
<td>W14×99</td>
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<td>430</td>
<td>646</td>
<td>274</td>
<td>412</td>
<td>4.89</td>
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<td>10.5</td>
<td>35.5</td>
<td>1110</td>
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<td>W18×76</td>
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<td>7.22</td>
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<td>574</td>
<td>229</td>
<td>344</td>
<td>16.8</td>
<td>24.1</td>
<td>4.97</td>
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<td>856</td>
<td>110</td>
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<td>244</td>
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<td>1510</td>
<td>192</td>
<td>151</td>
<td>225</td>
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<tr>
<td>W12×96</td>
<td>147</td>
<td>311</td>
<td>551</td>
<td>223</td>
<td>344</td>
<td>3.07</td>
<td>5.81</td>
<td>10.9</td>
<td>46.6</td>
<td>833</td>
<td>140</td>
<td>150</td>
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<td></td>
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<tr>
<td>W10×112</td>
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<td>282</td>
<td>551</td>
<td>223</td>
<td>331</td>
<td>3.03</td>
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<tr>
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<td>244</td>
<td>548</td>
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<td>333</td>
<td>10.3</td>
<td>15.7</td>
<td>6.00</td>
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<td>183</td>
<td>148</td>
<td>274</td>
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<td></td>
</tr>
</tbody>
</table>

---

*Shape exceeds compact limit for flexure with $F_y = 50$ ksi.

*Shape does not meet the $A_{pl}$ limit for shear in Specification Section G2.1a with $F_y = 50$ ksi.

$\Omega_x = 1.67$, $\phi_B = 0.80$

---

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is compact or noncompact. This is because for noncompact shapes, the tabulated values of $M_p, M_{pl}, \sigma_{pl}$ are based on flange local buckling and not the plastic moment (see Section 5.6). This means that for laterally supported beams, the $Z_s$ table can be used for design without regard to whether the shape is compact or noncompact.

**Beam Design Charts**

Many graphs, charts, and tables are available for the practicing engineer, and these aids can greatly simplify the design process. For the sake of efficiency, they are widely used in design offices, but you should approach their use with caution and not allow basic principles to become obscured. It is not our purpose to describe in this book all available design aids in detail, but some are worthy of note, particularly the curves of moment strength versus unbraced length given in Part 3 of the Manual.
shown in Figure 5.25. Two sets of curves are available, one for W shapes with $F_y = 50$ ksi and one for C and MC shapes with $F_y = 36$ ksi. Each graph gives the flexural strength of a standard hot-rolled shape. Instead of giving the nominal strength $M_n$, however, both the allowable moment strength $M_{n}/\Omega_{b}$ and the design moment strength $\phi_b M_n$ are given. Two scales are shown on the vertical axis—one for $M_{n}/\Omega_{b}$ and one for $\phi_b M_n$. All curves were generated with $C_b = 1.0$. For other values of $C_b$, simply multiply the moment from the chart by $C_b$. However, the strength can never exceed the value represented by the horizontal line at the left side of the graph. For a compact shape, this represents the strength corresponding to yielding (reaching the plastic moment $M_p$). If the curve is for a noncompact shape, the horizontal line represents the flange local buckling strength.

Use of the charts is illustrated in Figure 5.26, where two such curves are shown. Any point on this graph, such as the intersection of the two dashed lines, represents
if the charts are entered with a given unbraced length and a required strength, curves above and to the right of the point correspond to acceptable beams. If a dashed portion of a curve is encountered, then a curve for a lighter shape lies above or to the right of the dashed curve. Points on the curves corresponding to \( L_p \) are indicated by a solid circle; \( L_r \) is represented by an open circle.

In the LRFD solution of Example 5.10, the required design strength was 810.0 ft-kips, and there was continuous lateral support. For continuous lateral support, \( L_g \) can be taken as zero. From the charts, the first solid curve above the 810.0 ft-kip mark is for a W24 x 84, the same as selected in Example 5.10. Although \( L_g = 0 \) is not on this particular chart, the smallest value of \( L_b \) shown is less than \( L_p \) for all shapes on that page.

---

**FIGURE 5.27**

\[
P_y \frac{Z}{M} = M_p'
\]

\( M_p' \) = nominal strength based on FLB

Aids in the *Manual.* Whether a shape is compact or noncompact is irrelevant to the use of the charts.

\[
\frac{F_4-13}{F_4-14} \left[16.1 - 52\right]
\]

(Not covered)
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<td>510</td>
<td>765</td>
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<tr>
<td>500</td>
<td>750</td>
</tr>
</tbody>
</table>

Available Moment, $M_a$, (2 kip-ft Increments) $\phi M_a$, (3 kip-ft Increments)

**Table 3-10 (continued)**

**W Shapes**

Available Moment vs. Unbraced Length

- W14x132
- W19x10
- W24x76
- W20x84
- W22x11
EXAMPLE 5.11

The beam shown in Figure 5.28 must support two concentrated live loads of 20 kips each at the quarter points. The maximum live load deflection must not exceed $L/240$. Lateral support is provided at the ends of the beam. Use A992 steel and select a W shape.

If the weight of the beam is neglected, the central half of the beam is subjected to a uniform moment, and

$$M_A = M_B = M_C = M_{max} \quad \therefore \quad C_b = 1.0$$

Even if the weight is included, it will be negligible compared to the concentrated loads, and $C_b$ can still be taken as 1.0, permitting the charts to be used without modification.

Temporarily ignoring the beam weight, the factored-load moment is

$$M_u = 6(1.6 \times 20) = 192 \text{ ft-kips}$$

Required ultimate
From the charts, with $L_b = 24$ ft, try $W12 \times 53$:

\[
\phi_b M_n = 208.5 \text{ ft-kips} > 192 \text{ ft-kips} \quad \text{(OK)}
\]

Now, we account for the beam weight:

\[
M_u = 192 + \frac{1}{8} (1.2 \times 0.053)(24)^2 = 197 \text{ ft-kips} < 208.5 \text{ ft-kips} \quad \text{(OK)}
\]

The shear is

\[
V_u = 1.6(20) + \frac{1.2(0.053)(24)}{2} = 32.8 \text{ kips}
\]

From the $Z_x$ table (or the uniform load table),

\[
\phi_v V_n = 125 \text{ kips} > 32.8 \text{ kips} \quad \text{(OK)}
\]

The maximum permissible live load deflection is

\[
\frac{L}{240} = \frac{24(12)}{240} = 1.20 \text{ in.}
\]

From Table 3-23, "Shears, Moments, and Deflections," in Part 3 of the Manual, the maximum deflection (at midspan) for two equal and symmetrically placed loads is

\[
\Delta = \frac{Pa}{24EI} (3L^2 - 4a^2) \quad \text{Case 9}
\]

**Answer**: Use a $W12 \times 53$. 

\[
\boxed{23\text{-25}}
\]
Table 3–10 (continued)

W Shapes

Available Moment vs. Unbraced Length

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