Structural Steel Design & Timber Design

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Structural Steel Design

American Institute of Steel Construction (AISC)

3 Design Methods

- Allowable Stress Design (ASD)
- Plastic Design (Load Factor Design)
  Chapter N of ASD Manual
- Load and Resistance Factor Design (LRFD)
  Second Edition, Volume I & II
LRFD DESIGN METHOD

Design Equation:

\[ \sum \gamma_i Q_i \leq \phi R_n \]

where

- \( Q_i \) = nominal load effect (service loads)
- \( \gamma_i \) = load factor corresponding to \( Q_i \) (usually > 1.0)
- \( \sum \gamma_i Q_i \) = total factored load effect (or required strength)
- \( R_n \) = nominal resistance
- \( \phi \) = resistance factor corresponding to \( R_n \) (< 1.0)
- \( \phi R_n \) = design strength

LOAD COMBINATIONS  
(Note: differ from ACI Code!)

1.4\( D \)  
1.2\( D \) + 1.6\( L \) + 0.5\( L_r \) or \( S \) or \( R \)  
1.2\( D \) + 1.6\( L_r \) or \( S \) or \( R \) + (0.5\( L \) or 0.8\( W \))  
1.2\( D \) + 1.3\( W \) + 0.5\( L \) + 0.5\( L_r \) or \( S \) or \( R \)  
1.2\( D \) \( \pm \) 1.0\( E \) + 0.5\( L \) + 0.2\( S \)  
0.9\( D \) \( \pm \) (1.3\( W \) or 1.0\( E \))  

For members subject to dead and live load: 1.2\( D \) + 1.6\( L \)

See Examples A-1 and A-2, p. 2-9
MATERIAL PROPERTIES OF STEEL

\[ \sigma \]

\[ F_y \]

\[ E = 29,000 \text{ ksi} \]

\[ \varepsilon_r = 0.002 \]

\[ 0.1 < \varepsilon_r < 0.3 \]

DUCTILITY

STRUCTURAL STEELS

<table>
<thead>
<tr>
<th>ASTM Designation</th>
<th>Grade</th>
<th>Yield Stress ( F_y ) (ksi)</th>
<th>Tensile Stress ( F_u ) (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A36</td>
<td>36</td>
<td>58-80</td>
<td></td>
</tr>
<tr>
<td>A572</td>
<td>42</td>
<td>60</td>
<td></td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>65</td>
<td></td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>75</td>
<td></td>
</tr>
<tr>
<td></td>
<td>65</td>
<td>80</td>
<td></td>
</tr>
<tr>
<td>A588</td>
<td>50</td>
<td>70</td>
<td></td>
</tr>
<tr>
<td>A852</td>
<td>(plate only)</td>
<td>70</td>
<td>90-110</td>
</tr>
<tr>
<td>A514</td>
<td>(plate only)</td>
<td>90</td>
<td>90-130</td>
</tr>
<tr>
<td>A514</td>
<td>(plate only)</td>
<td>100</td>
<td>100-130</td>
</tr>
</tbody>
</table>

See Table 1-1, p. 1-15 for complete listing and availability
STRUCTURAL STEEL ROLLED SHAPES

<table>
<thead>
<tr>
<th>Shape</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>Channel</td>
</tr>
<tr>
<td>W</td>
<td>Wide Flange</td>
</tr>
<tr>
<td>L</td>
<td>Angle</td>
</tr>
<tr>
<td>S</td>
<td>Standard</td>
</tr>
<tr>
<td>WT</td>
<td>Tee (cut from W)</td>
</tr>
</tbody>
</table>

Section Properties in Part 1 of the LRFD Manual (Vol. I)

WELDED BUILT-UP MEMBERS
(Symmetrical Cross Sections)

\[
I_x = \frac{1}{12} t_w h^3 + 2 \left[ b_f t_f \left( \frac{h + t_f}{2} \right)^2 \right]
\]

\[
S_x = \frac{I_x}{c} = \frac{I_x}{(h + t_r)/2}
\]

\[
Z_x = t_w \frac{h^2}{4} + b_f t_f (h + t_f)
\]

See Tables 4-183 to 4-185 for section properties of selected built-up wide-flange sections
TENSION MEMBERS

- Gross Section Yielding (GSY)
- Net Section Fracture (NSF)
- Block Shear Rupture (BSR)

- Serviceability Check

\[ \frac{L}{r_{\text{min}}} \leq 300 \quad r_{\text{min}} = \text{Least Radius of Gyration} \]

\[(L \text{ in inches}) = \sqrt{\frac{I_{\text{min}}}{A}}\]

GROSS SECTION YIELDING (GSY)

\[ f_t = \frac{P}{A_y} \]

Prevents excessive elongation of the member

Design based on the Gross Area, \( A_g \)

\[ \phi P_n = 0.9F_y A_g \]
NET SECTION FRACTURE (NSF)

\[ f_t = \frac{P}{A_g} \]

Prevents fracture at a connection

Design based on the Effective Net Area, \( A_e \), and Net Area, \( A_n \)

\[ \phi P_n = 0.75 F_y A_e = 0.75 F_y U A_n \]

NET AREA, \( A_n \)

Net area is the reduced gross area owing to the holes

\[ A_n = A_g - A_{holes} = A_y - d_{hole} \times t \]

\[ d_{hole} = d_{bolt} + 1/16" + 1/16" \]
Staggered Holes

If the holes are staggered, a correction factor is used to account for the increased tensile strength on an inclined fracture path:

Correction Factor $= \frac{S^2}{4g}$

$A_n = A_g - \sum A_{holes} + t \sum \frac{S^2}{4g}$

Example: Determine the net area for the plate shown. Assume the holes are for 7/8" dia. bolts.

Path 1-1

$A_n = \frac{(1/2)(8) - 2(1/2)(7/8 + 1/16 + 1/16) + 1/2 \frac{(2)^2}{4(4)}}{4(4)}$

$= 3.13 \text{ in}^2$ - controls

Path 2-2

$A_n = 4 - 2(1/2)(1) + 1/2 \frac{(5)^2}{4(4)}$

$= 3.78 \text{ in}^2$

Path 3-3

$A_n = 4 - 1(1/2)(1)$

$= 3.50 \text{ in}^2$
Net Effective Area, $A_e$

Defined as the reduced gross area or net area to account for non-uniform stress distributions.

**REDUCTION COEFFICIENT**

When the load is transmitted by bolts or welds through some but not all of the cross sectional elements, the effective area is given as:

$$A_e = U A_n$$

where

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

$\bar{x}$ – connection eccentricity, in.

$L$ = length of connection, in.
Example  Determine the reduction coefficient and tensile capacity for the angle shown bolted to the gusset plate. Assume A36 steel and a single row of 7/8" dia. bolts.

From Section Property Tables: \( A_g = 4.75 \text{ in.}^2, \bar{x} = 0.987 \text{ in.} \)

\[
U = 1 - \frac{\bar{x}}{L} = 1 - \frac{0.987}{6} = 0.84
\]

NSF:

\[
\phi P_n = \phi F_u U A_n = 0.75 (58)(0.84)(4.75 - 0.5 (7/8 + 1/8)) = 155 \text{ kips}
\]

GSY:

\[
\phi P_n = \phi F_y A_g = 0.90 (36)(4.75) = 154 \text{ kips} \quad \text{controls}
\]

<table>
<thead>
<tr>
<th>Reduction Coefficient, ( U )</th>
<th>Bolted Shapes</th>
<th>Welded Shapes</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.90 W and WT shapes with ( b_f &gt; 2/3d ) w/ at least 3 bolts</td>
<td>W and WT shapes with ( b_f &gt; 2/3d )</td>
<td>W and WT shapes with ( b_f &gt; 2/3d )</td>
</tr>
<tr>
<td>0.85 W and WT shapes with ( b_f &lt; 2/3d ), C, and L, shapes with at least 3 bolts</td>
<td>All other cases</td>
<td>All other cases</td>
</tr>
<tr>
<td>0.75 All shapes with 2 bolts in line</td>
<td>N.A.</td>
<td>N.A.</td>
</tr>
</tbody>
</table>
Bolted Flat Plates: \( U = 1.0 \)

Welded Flat Plates:

- with end weld: \( U = 1.0 \)

- no end weld: \( U = 1.0 \) \( L > 2w \)
  \( U = 0.87 \) \( 2w > L > 1.5w \)
  \( U = 0.75 \) \( 1.5w > L > w \)
**BLOCK SHEAR RUPTURE:**

Note: Fracture path must surround entire bolt pattern(s)

---

**BLOCK SHEAR RUPTURE:**

\[
\phi R_p = \phi \left( 0.6 F_y A_{gv} + F_u A_{nt} \right) \quad \text{(AISC-LRFD J4-3a)}
\]

where

- \( \phi = 0.75 \)
- \( A_{gv} = \text{gross shear area} \)
- \( A_{nt} = \text{net area in tension} \)

\[
\psi R_p = \phi \left( 0.6 F_u A_{nv} + F_y A_{gt} \right) \quad \text{(AISC-LRFD J4-3b)}
\]

where

- \( \phi = 0.75 \)
- \( A_{nv} = \text{net area in shear} \)
- \( A_{gt} = \text{gross tension area} \)

Equation with larger fracture term controls
Example

Determine the design strength of the single angle bolted connection shown. Assume A36 steel and 13/16 in. holes.

\[
A_g = 3.75 \text{ in.}^2
\]

\[
\bar{x} = 1.18 \text{ in.}
\]

Block shear rupture pattern:

\[
A_{tg} = 1.5(0.5) = 0.745 \text{ in.}^2
\]

\[
A_{gv} = (6 + 1)(0.5) = 3.75 \text{ in.}^2
\]

\[
A_{nt} = (1.5)(0.5) - 0.5[(0.5)(13/16 + 1/16)] = 0.53 \text{ in.}^2
\]

\[
A_{nv} = (7.5)(0.5) - 2.5[90.5(13/16 + 1/16)] = 2.66 \text{ in.}^2
\]

Example (cont.)

Controlling fracture term:

\[
F_u A_{nt} = 58(0.53) = 30.7 \text{ kips}
\]

\[
0.6 F_u A_{nv} = 0.6(58)(2.66) = 92.6 \text{ kips}
\]

\[
\therefore \text{shear fracture controls}
\]

BSR: \[
\phi P_n = \phi(0.6 F_u A_{nv} + F_y A_{gt})
\]

\[
= 0.75(92.6 + 30(0.75)) = 89.7 \text{ kips}
\]

NSF: \[
U = 1 - \frac{\bar{x}}{L} = 1 - \frac{1.18}{6} = 0.80
\]

\[
\phi P_n = \phi F_u U A_n
\]

\[
= 0.75(58)(0.80)[(3.75 - 0.5(16/16)] = 115 \text{ kips}
\]

GSY: \[
\phi P_n = \phi F_y A_g
\]

\[
= 0.90(36)(3.75) = 122 \text{ kips}
\]
COLUMNS

The AISC column design provisions based on Euler column buckling behavior

Euler column has pinned ends

End conditions for building columns differ

Determine the effective column length, $KL$

Can relate a building column back to the Euler column using $K$

BRACED FRAMES  (Sideways Inhibited)

Note: $K \leq 1.0$ for braced frames
UNBRACED FRAMES  (Sidesway uninhibited)

Note:  \( K > 1.0 \) for unbraced frames

RELATIVE STIFFNESS

\[
G = \frac{\sum \frac{l_c}{L_c}}{\sum \frac{l_g}{L_g}} \quad \text{lying in the plane of buckling}
\]

\( c = \text{columns} \)

\( g = \text{beams or girders} \)
CONTROLLING AXIS

\[
\left( \frac{K_x L_x}{r_x} \right)_x
\]

Larger value controls

\[
\left( \frac{K_y L_y}{r_y} \right)_y
\]

Calculate: \( \lambda_c = \frac{KL}{r \pi \sqrt{\frac{F_y}{E}}} \)
CRITICAL BUCKLING STRESS

when $\lambda_c \leq 1.5$  \[ F_{cr} = (0.658\lambda_c^2)F_y \]  \hspace{1cm} (E2.2)

when $\lambda_c > 1.5$  \[ F_{cr} = \left[ \frac{0.877}{\lambda_c^2} \right]F_y \]  \hspace{1cm} (E2.3)

Note: can use Tables on pp. 6-147 to 6-151

DESIGN LOAD

$\phi P_n = 0.85 F_{cr} A_g$  \hspace{1cm} (E2-1)
LOCAL BUCKLING

\[
\frac{b_f}{2t_f} \leq \frac{95}{\sqrt{F_y}}
\]

\[
\frac{h}{t_w} \leq \frac{253}{\sqrt{F_y}}
\]

Note: Buckling of singly symmetric and unsymmetric columns require consideration of flexural-torsional buckling (see LRFD Appendix E)

COLUMN DESIGN TABLES  Part 3 - AISC-LRFD Manual

<table>
<thead>
<tr>
<th>Designation</th>
<th>W12</th>
</tr>
</thead>
<tbody>
<tr>
<td>W12.0T</td>
<td>210</td>
</tr>
<tr>
<td>Fy (kips)</td>
<td>36</td>
</tr>
<tr>
<td>0</td>
<td>1900</td>
</tr>
<tr>
<td>1</td>
<td>1800</td>
</tr>
<tr>
<td>2</td>
<td>1700</td>
</tr>
<tr>
<td>3</td>
<td>1600</td>
</tr>
<tr>
<td>4</td>
<td>1500</td>
</tr>
<tr>
<td>5</td>
<td>1400</td>
</tr>
<tr>
<td>6</td>
<td>1300</td>
</tr>
<tr>
<td>7</td>
<td>1200</td>
</tr>
<tr>
<td>8</td>
<td>1100</td>
</tr>
<tr>
<td>9</td>
<td>1000</td>
</tr>
<tr>
<td>10</td>
<td>900</td>
</tr>
<tr>
<td>11</td>
<td>800</td>
</tr>
<tr>
<td>12</td>
<td>700</td>
</tr>
<tr>
<td>13</td>
<td>600</td>
</tr>
</tbody>
</table>

4 - 17
COLUMN DESIGN TABLES

For W-shapes, design loads are for weak-axis (Y-Y) buckling.

Can use for strong-axis buckling by using an equivalent slenderness ratio:

\[
\left( \frac{KL}{r} \right)_{\text{equiv}} = \frac{\left( \frac{KL}{r} \right)_x}{\frac{r_x}{r_y}}
\]

When designing, do not know \( r_x/r_y \). Estimate using the range of values given at the bottom of the design tables.

Example: For the column shown, select the lightest W12x for a design load of 750 kips. Use A36 steel.

\[
\begin{align*}
(KL)_x &= 2.0(20) = 40 \text{ ft.} \\
(KL)_y &= 0.80(20) = 16 \text{ ft.} \\
\frac{(KL)_x}{(KL)_y} &= \frac{40}{16} = 2.50
\end{align*}
\]

From AISC-LRFD Column Design Tables: \( r_x/r_y \) ranges from 1.75 to 1.80 for W12x columns with a design capacity of approximately 750 kips at \((KL)_y = 16 \text{ ft.}\).

Since \( \frac{(KL)_x}{(KL)_y} = 2.50 > \frac{r_x}{r_y} = 1.75 \text{ to } 1.80 \),

axis buckling controls. Since Column Design Tables are for \((KL)_y\), must find equivalent \( KL \) for the x-x axis:

\[
\left( \frac{KL}{r} \right)_{\text{equiv}} = \frac{(KL)_x}{r_x/r_y} = \frac{40}{1.75} = 22.9 \text{ ft.}
\]

Use W12x136 since \( \phi P_n = 788 \) kips at \((KL)_y = 24 \text{ ft.}\).
DESIGN OF STEEL BEAMS

Flexure
  - Lateral Torsional Buckling

Shear

Deflections

Bearing

Strong Axis Bending

Weak Axis Bending
STRONG AXIS BENDING OF I-SHAPED MEMBERS

Maximum design moment capacity:

$$\phi M_p = \phi M_p = 0.9F_c Z_x$$

where $Z_x$ is the plastic section modulus about the x-x axis.

The beam may fail prior to reaching $\phi M_p$ due to local buckling or lateral torsional buckling.

LOCAL BUCKLING

Flange

$$\frac{b_f}{2t_f} \leq \frac{65}{\sqrt{F_y}}$$

(Web

$$\frac{h}{t_w} < \frac{640}{\sqrt{F_y}}$$

(FLB)

(WLB)
LATERALLY UNBRACED BEAMS (Compact Sections)

If $L_b > L_p$ must reduced $\phi M_n$ due to lateral torsional buckling

$L_b$ is the distance between compression flange bracing points
$L_p$ is the maximum unbraced length that allows $\phi M_p$

Lateral Buckling

In-Plane Deflection

Function of the slenderness ratio $\frac{L_b}{f_y}$

Lateral Bracing Provided By:

Continuous Support

Supported at intervals
Nominal Moment Capacity - $M_n$

If $L_b \leq L_p$

$$M_n = M_p$$

If $L_p < L_b \leq L_r$

$$M_n = C_b \left[ M_p - (M_p - M) \left( \frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_p$$

If $L_b > L_r$

$$M_n = M_{cr} = \frac{C_b S_x X_1 \sqrt{2}}{L_b/r_y} \sqrt{1 + \frac{X_1^2 X_2}{2 (L_b/r_y)^2}} \leq M_p$$

Moment Correction Factor

The design equations based on a constant moment along the unbraced length, $L_b$

$$M_1 \left( \begin{array}{c}
\uparrow \\
\downarrow \\
\downarrow \\
\uparrow
\end{array} \right) M_2 \quad M_1 = M_2$$

Accounts for moment gradients between bracing points

$$C_b = 1.75 + 1.05 \left( \frac{M_1}{M_2} \right) + 0.3 \left( \frac{M_1}{M_2} \right)^2 \leq 2.3$$

$M_1$ is the smaller and $M_2$ is the larger bending moment at the end of the unbraced length.

New Method in $2^{nd}$, see p. 6-53 and Table 4-1, p. 4-9
$M_1/M_2$ Negative

$M_1/M_2$ Positive

$C_b = 1.0$

Note: conservative to use $C_b = 1.0$
**Beam Chart Observations**

- Beam Charts for $C_b = 1.0$
- The horizontal portion of each curve defines $\phi M_p$
- Limit of $\phi M_n = \phi M_p$ at $L_p$ is given by ●
- Limit of inelastic bending strength (Eq. F1-2) at $L_p$ is given by ○
- A solid curve or line represents the lightest and, hence, most economical section
- A dashed curve or line indicates that a lighter section with equivalent or larger moment capacity exists

**Designing with the Beam Charts:**

Enter with $M_u = \phi M_p$ and $L_b$

Select first solid curve or line above intersection point for most economical section

When $C_b > 1.0$

Conservatively assume $C_b = 1.0$ and use the beam charts as is, or calculate:

$$M_{equiv} = M_u / C_b$$

Make selection and check $\phi M_p > M_u$

Note: $M_{equiv}$ is only used to enter the beam charts.
## Load Factor Design Selection Tables

**LOAD FACTOR DESIGN SELECTION TABLE**
For shapes used as beams
\( \phi_a = 0.90 \)

| BF | 12.7 | 8.0 | 6.0 | 6.0 | 7.0 | 2.4 | 11.3 | 4.1 | 3.7 | 2.6 | 1.9 | 1.75 | 1.05 | 2.88 | 4.09 | 1.97 | 7.65 | 10.5 | 2.87 | 6.43 | 3.91 | 7.31 | 1.90 | 2.80 |
|----|------|-----|-----|-----|-----|-----|------|-----|-----|-----|-----|------|------|------|-----|-----|-----|-----|-----|-----|-----|-----|-----|
| Lf | 16.6 | 13.2 | 11.0 | 10.3 | 10.6 | 12.4 | 17.3 | 20.0 | 16.4 | 20.0 | 11.0 | 11.0 | 17.7 | 12.7 | 22.8 | 34.7 | 30.5 | 20.5 | 30.1 | 22.6 | 6.9 | 21.6 |
| Lp | 5.5 | 7.0 | 12.8 | 11.0 | 10.3 | 10.6 | 5.6 | 10.3 | 11.0 | 10.3 | 11.0 | 12.4 | 17.7 | 12.7 | 22.8 | 34.7 | 30.5 | 20.5 | 30.1 | 22.6 | 9.3 | 17.6 |
| \( e^\phi M \) | 222 | 228 | 230 | 218 | 228 | 228 | 216 | 218 | 218 | 218 | 218 | 218 | 209 | 209 | 211 | 275 | 180 | 173 | 166 | 168 | 264 | 264 | 264 |
| \( \phi_a \) | 1.34 | 1.33 | 1.32 | 1.30 | 1.30 | 1.30 | 1.29 | 1.26 | 1.26 | 1.26 | 1.26 | 1.26 | 1.19 | 1.19 | 1.19 | 1.02 | 1.02 | 1.01 | 1.01 | 1.01 | 1.01 | 1.01 |
| \( Z_x \) | W24×45 | W21×50 | W18×65 | W18×65 | W18×65 | W18×65 | W18×75 | W18×75 | W18×75 | W18×75 | W18×75 | W18×75 | W18×75 | W18×75 | W18×75 | W18×75 | W18×75 | W18×75 | W18×75 | W18×75 | W18×75 | W18×75 |
| \( F_y = 36 \text{ ksl} \) | | | | | | | | | | | | | | | | | | | | | | | | | |
| \( F_y = 50 \text{ ksl} \) | | | | | | | | | | | | | | | | | | | | | | | | | |

**Example**

Select the lightest W18x for a simply supported beam shown. Assume \( w_D = 2 \text{ kip/ft} \) (including the beam weight) and \( w_L = 3 \text{ kip/ft} \). Use A36 steel and assume lateral bracing the end supports only.

\[
w_u = 1.2(2) + 1.6(3) = 7.2 \text{ kips/ft}
\]

\[
M_u = \frac{wL^2}{8} = \frac{1.2(20)^2}{8} = 360 \text{ k-ft}
\]

\[
Z_{req} = \frac{360(12)}{0.9(36)} = 133 \text{ in}^3
\]

\( L_b = 20 \text{ ft} \) and \( C_b = 1.0 \) since \( M_{max} \) occurs inside \( L_b \)

(LRFD 2\textsuperscript{nd} Ed. allows \( C_b = 1.14 \))

From Beam Charts (p. 4-128) use W18x76
Example  Repeat last example assuming that the compression flange is laterally braced at midspan as well as at the end supports.

\[ L_b = 10 \text{ ft, conservatively assuming } C_b = 1.0: \text{ Use W18x71} \]

Using the correct value for \( C_b \) (2nd Ed.)

\[ C_b = 1.30 \text{ from Table 4-1 (p. 4-9)} \]

\[ M_{\text{equiv}} = 360/1.30 = 277 \text{ kip-ft} \]

Try W18x55, check \( Z_x = 112 \text{ in.}^3 < Z_{\text{req}} = 133 \text{ in.}^3 \text{ N.G.} \)

The beam can also be checked for adequacy by following the W18x55 curve back up to its plateau (\( \phi M_p \)) which in this case is 302 k-ft.

---

**FLEXURAL SHEAR**

Typically, the beam selection is based on satisfying the bending moment requirements and then checked for shear

\[ F_v = 0.6F_y \]

The web of a W-shape section resists the shear force, therefore the shear capacity of the cross section is given as:

\[ \phi V_n = 0.9(0.6)F_yd t_w \]

Shear capacities for W-shapes are tabulated in the Maximum Factored Uniform Load Tables (pp. 4-35 to 4-108)

Shear controls the design for short spans and when concentrated loads are applied close to a support.
Example Verify the shear capacity of the W18x71 selection in the previous example.

\[ V_u = \frac{7.2(20)}{2} = 72 \text{ kips} \]

For W18x71: \( d = 18.47 \text{ in.} \), \( t_w = 0.495 \text{ in.} \)

\[ \phi V_n = 0.9(0.6)(36)(18.47)(0.495) = 178 \text{ kips} > 72 \text{ kips} \checkmark \]

Using beam table (p. 4-50) \( \phi V_n = 178 \text{ kips} \checkmark \)

DEFLECTIONS: (AISC-LRFD Chapter L)

The 2\textsuperscript{nd} Edition only states "Deformations in structural members and structural elements due to service loads shall not impair the serviceability of the structure."

Often, the design of floor beams is controlled by deflections.

Dead load deflection is negated by reverse camber at fabrication

From AISC-ASD, the deflection due to live load is limited by:

\[ \Delta_l < \frac{L}{360} \]

where \( L \) is the span length in inches, \( \Delta_l \) calculated based on unfactored live loads.
Example  Check deflection criteria for the W18x71 section selected in the previous example.

For W18x71: \( I_x = 1170 \text{ in.}^4 \)

\[
\Delta_L = \frac{5 \omega L^4}{384 EI} = \frac{5(3/12)(20\times12)^4}{384(29,000)(1170)} = 0.32 \text{ in.} \quad \text{(watch units!)}
\]

\[
\Delta_{max} = \frac{20(12)}{360} = 0.67 \text{ in.} \quad \checkmark
\]

- W18x71 O.K.

CONCENTRATED LOADS:

- Local Web Yielding \ (Sec. K1.3, p. 6-92)
- Web Crippling \ (Sec.K1.4, p. 6-93)
LOCAL WEB YIELDING:

Greater than a distance \(d\) from the end of the member:

\[
\phi R_n = 1.0(N + 5k)F_y t_w
\]

At or near the end of the member:

\[
\phi R_n = 1.0(N + 2.5k)F_y t_w
\]

Can use Maximum Factored Uniform Load Tables (at bottom)

\[\phi R_1 + \phi N R_2\text{ and } \phi R\text{ for } N = 3\frac{3}{4}''\text{ (end bearing)}\]

WEB CRIPLING:

At a distance not less than \(d/2\) for the end of the member:

\[
\phi R_n = 0.75(135)t_w^2 \left[ 1 + 3 \left( \frac{N}{d} \right) \left( \frac{t_w}{t_f} \right)^{1.5} \right] \sqrt[1.5]{\frac{F_{yw} t_f}{t_w}}
\]

At or near the end of the member:

\[
\text{for } N/d \leq 0.2: \quad \phi R = 0.75(68)t_w^2 \left[ 1 + 3 \left( \frac{N}{d} \right) \left( \frac{t_w}{t_f} \right)^{1.5} \right] \sqrt[1.5]{\frac{F_{yw} t_f}{t_w}}
\]

\[
\text{for } N/d > 0.2: \quad \phi R = 0.75(68)t_w^2 \left[ 1 + \left( \frac{4N}{d} \cdot 0.2 \right) \left( \frac{t_w}{t_f} \right)^{1.5} \right] \sqrt[1.5]{\frac{F_{yw} t_f}{t_w}}
\]

\[\phi R_{max} = \phi R_3 + N \phi R_4\]
Example

Determine the adequacy of 3 in. bearing plates for the end supports of the W18x71 of the previous problem.

\[ R_u = V_u = 72 \text{ kips} < \phi R_n \]

Using beam table, p. 4-50:

Local Web Yielding:

\[ \phi R_1 + N\phi R_2 = 66.8 + 17.8(3) = 120 \text{ kips} \checkmark \]

Web Crippling:

\[ \phi R_3 + N\phi R_4 = 95.9 + 7.44(3) = 118 \text{ kips} \checkmark \]

COMBINED LOADING

- BIAXIAL BENDING

- COMBINED AXIAL TENSION AND BENDING

- COMBINED AXIAL COMPRESSION AND BENDING

BEAM-COLUMN

Use Interaction Equations:

\[ \sum \frac{\text{Load Effect}}{\text{Design Strength}} \leq 1.0 \]
BIAXIAL BENDING:

\[
\frac{M_{ux}}{\phi M_{nx}} + \frac{M_{uy}}{\phi M_{uv}} \leq 1.0
\]

where

\[\phi M_{nx} \leq \phi M_{px}\] (may be reduced due to LTB)
\[\phi M_{ny} = \phi M_{py}\] (no LTB for bending about y-y axis)

Example  Verify that a W18x50 of A36 is adequate for the following loading: \(M_{ux} = 100\) k-ft and \(M_{uv} = 20\) k-ft.

W18x50: \(Z_x = 101\) in.\(^3\) and \(Z_y = 16.6\) in.\(^3\)

\[
\frac{M_{ux}}{\phi M_{nx}} + \frac{M_{uy}}{\phi M_{uv}} = \frac{100(12)}{0.9(101)(36)} + \frac{20(12)}{0.9(16.6)(36)}
\]

\[= 0.37 + 0.45 = 0.81 < 1.0 \checkmark\]

\[\therefore \text{W18x71 of A36 steel is adequate}\]
AXIAL TENSION AND BENDING:

For $\frac{P_u}{\phi_t P_n} \geq 0.2$

$$\frac{P_u}{\phi_t P_n} + \frac{8}{9} \left( \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \leq 1.0$$

For $\frac{P_u}{\phi_t P_n} < 0.2$

$$\frac{P_u}{2\phi_t P_n} + \left( \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \leq 1.0$$

Consider all possible failure states for a tension member (GSY, NSR, BSR)

AXIAL COMPRESSION AND BENDING:

$$M_{max} = \frac{NL}{4} + P\Delta$$

AISC-LRFD estimates the P delta effect using a moment magnification factor, $B_1$
MOMENT MAGNIFICATION FACTOR (Braced Frames)

\[ B_1 = \frac{C_m}{1 - \frac{P_u}{P_{e1}}} \geq 1.0 \]

where

\[ P_{e1} = \frac{A_g F_y}{\lambda_c^2} \] (Euler Buckling)

\[ C_m = \text{Moment Correction Factor (similar to } C_b) \]
(see Chapter C, p. 641)

\[ = 1.0 \text{ for conservative design} \]

BEAM-COLUMN INTERACTION EQUATIONS

For \( \frac{P_u}{\phi_c P_n} \geq 0.2 \)

\[ \frac{P_u}{\phi_c P_n} + \frac{8}{9} \left( \frac{B_{1x} M_{ux}}{\phi_b M_{nx}} + \frac{B_{1y} M_{vy}}{\phi_b M_{ny}} \right) \leq 1.0 \]

For \( \frac{P_u}{\phi_c P_n} < 0.2 \)

\[ \frac{P_u}{2\phi_c P_n} + \left( \frac{B_{1x} M_{ux}}{\phi_b M_{nx}} + \frac{B_{1y} M_{vy}}{\phi_b M_{ny}} \right) \leq 1.0 \]

Note: P-delta effect occurs in both axes for biaxial bending
Example  Determine the adequacy of a W10x30 of A588, Gr. 50 steel for the loading condition shown. Assume that the member is laterally braced only at the end supports.

\[ P_u = 80 \text{ kips} \]

\[ A_y = 8.84 \text{ in}^2 \]
\[ Z_y = 8.84 \text{ in}^3 \]
\[ r_x = 4.38 \text{ in.} \]
\[ r_y = 1.37 \text{ in.} \]

Check column buckling: since \((KL)_x = (KL)_y = 12 \text{ ft}\), \(y-y\) axis controls

\[
\left( \frac{KL}{l} \right)_y = 1.0(12)(12) \div 1.37 = 105, \quad \lambda_{cy} = \frac{105}{\pi} \sqrt{\frac{50}{29,000}} = 1.39 < 1.5
\]

Example (cont.)

\[ \Phi_{p_n} = 0.85F_{cr}, \quad A_g = 0.85(0.685^{1.392})(50)(8.84) = 167.5 \text{ kips} \]

\(y-y\) bending: \(M_{uy} = 0.5(12)^2/8 = 9.0 \text{ k-ft}\), No \(x-x\) bending: \(M_{ux} = 0\).

\[ \Phi_{bM_{ny}} = \Phi_{bF_yZ_y} - 0.9(50)(8.84) = 398 \text{ k-in.} = 33.2 \text{ k-ft} \]

\[ C_{ny} = 1.0 \text{ (transverse loading and unrestrained ends (pinned))} \]

\[ P_{el} = \Phi_{p_n} = \frac{\pi^2(29,000)(8.84)}{(105)^2} = 229 \text{ kips}, \quad B_{1y} = \frac{1.0}{1.0 - 80/229} = 1.54 \]

\[ P_u/\Phi_{p_n} = 80/167.5 = 0.478 > 0.20 \quad \text{Use Eq. (H1-1a)} \]

\[
\frac{P_u}{\Phi_{p_n}} + 8 \left( \frac{B_{1y}M_{uy}}{\Phi_{bM_{ny}}} \right) = 0.478 + 8 \left( \frac{1.54(9.0)}{0.9(50)(8.84)} \right) = 0.85 < 1.0 \checkmark
\]

4 - 34
STRUCTURAL FASTENERS

Unfinished Bolts: A307

High-Strength Bolts: A325
A490

LOAD CONDITION:

Tension

\[ \phi r_n = 0.75 A_b F_t \]

where

\[ F_t = \text{Nominal Strength} \]
\[ = 90 \text{ ksi, A325} \]
\[ = 113 \text{ ksi, A490} \]

\[ A_b = \text{nominal bolt diameter} \]

See Table 8-15, p. 8-27, Vol II  [Table 1-A, p. 5-3, 1st Ed.]
HIGH-STRENGTH BOLTS SUBJECT TO SHEAR:

Bearing: A325-X  A325-N
A490-X  A490-N

N = threads in shear plane(s)
X = threads excluded

Slip Critical:
A235-SC  A490-SC

BOLT HOLE TYPES

Bearing-Type Connections:
Hole cannot be oversized by more than 1/16" in the direction of loading

Slip-Critical Connections:
Oversized holes reduce the friction surface, reducing the resisting force against slip
BEARING TYPE BOLT STRENGTH

\[ \phi_{r_n} = 0.75F_vA_bN_s \]

where

\[ F_v = \text{nominal bolt shear strength} \]
\[ = 24 \text{ ksi for A307} \]
\[ = 48 \text{ ksi for A325-N} \]
\[ = 60 \text{ ksi for A325-X} \]
\[ = 60 \text{ ksi for A490-N} \]
\[ = 75 \text{ ksi for A490-X} \]

\[ A_b = \text{nominal area of bolt} \]

\[ N_s = \text{number of shear planes (1 or 2)} \]

See Table 8-11, p. 8-24, Vol. II [Table I-D, p. 5-5, 1st Ed.]

SLIP-CRITICAL TYPE BOLT STRENGTH

New to LRFD 2nd Edition, can base strength on factored loads
(see Appendix J, p. 6-130)

\[ \phi_{r_n} = \phi \cdot 1.13\mu T_mN_s \]

where

\[ \phi = 1.0 \text{ for standard (STD) holes (see p. 6-130 for other hole types)} \]

\[ \mu = \text{slip coefficient (function of plate surface finish)} \]
\[ = 0.33 \text{ for unpainted clean mill scallop (Class A)} \]

\[ N_s = \text{number of slip planes (1 or 2)} \]
<table>
<thead>
<tr>
<th>Bolt Designation</th>
<th>Connection Type</th>
<th>Loading</th>
<th>Nominal Diameter d, in.</th>
<th>Hole Types</th>
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<td></td>
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<td>3/4</td>
<td>7/8</td>
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<tr>
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<td></td>
<td>Single Shear</td>
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<td>Double Shear</td>
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<td>in shear plane</td>
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<td>and short slotted</td>
</tr>
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<td></td>
<td>Single Shear</td>
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<td>15.5</td>
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<td>in shear plane</td>
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<td>49.7</td>
<td>67.7</td>
</tr>
</tbody>
</table>

Table: Design loads (kips) for structural bolts subject to shear (from AISC-LRFD Vol. II, Tables, 8-11 and 8-17).
SPACING BETWEEN BOLTS:

\[ S_{\text{max}} = 3d_{\text{bolt}} \]

Use 3” for 1.0” dia. or less

EDGE DISTANCE:

\[ L_e = 1.5d \]

Use 1.5” for 1.0” dia. or less

BOLT BEARING:

\[ \phi r_n = 0.75(2.4)d \ t \ F_u \]

where

\begin{align*}
    d & = \text{nominal diameter of the bolt} \\
    t & = \text{thickness of the connected part (plate)} \\
    F_u & = \text{specified tensile strength of connected part}
\end{align*}

Reduced bearing strengths when spacing criteria not satisfied (see Sec. J3.10)
Example  Determine the maximum design load (axial tension) for the single shear lap splice shown. Assume four 7/8" dia A325-N bolts, STD holes, and A36 steel.

Bolt Bearing:

From Table 8-11 (p. 8-24)

\[ \phi R_n = 4(21.6) = 86.4^k \]

GSY:

\[ \phi P_n = 0.9(36)(0.5)(6) \]
\[ = 97.2 \text{ kips} \]

NSF:

\[ \phi P_n = 0.75(58)[(0.5)(6) - 2(7/8 + 1/8)0.5)] = 87.0 \text{ kips} \]

Example (cont.)

Bolt Bearing:

\[ \phi r_n = 0.75(2.4)(7/8)(0.5)(58) = 46.7 \text{ kips/bolt} \]

\[ \phi R_n = 4(46.7) = 183 \text{ kips} \]

Check Bolt Spacing

\[ s_{min} = 3d = 3(7/8) = 2.63 \text{ in.} < 3.0 \text{ in.} \quad \text{O.K.} \]

\[ L_v = 1.5d = 1.5(7/8) = 1.31 \text{ in.} < 1.5 \text{ in.} \quad \text{O.K.} \]

\[ \phi P_n = 87.0 \text{ kips} \quad \text{(Net Section Fracture of the plate)} \]
NET SECTION SHEAR STRENGTH

Similar to net section fracture of tension members

\[ \phi R_n = 0.75(0.6)F_u A_n \]

\[ A_n = A_{gv} - A_{holes} \]

\[ = Lt - n_{holes}(d_{bolt} + 1/8")t \]

\[ \phi R_n = 0.75(0.6)F_u t (L - n_{holes}(d_{bolt} + 1/8")] \]

ECCENTRIC LOADS ON BOLT GROUPS

Vector or Elastic Method (Conservative)

\[ J = \sum_i r_i^2 A_i \]

\[ f_{v,x} = \frac{f_{v,y}}{r} \]

\[ f_v = \frac{Mr_{critical}}{J} \]

\[ f_{v,d} = \frac{P}{\sum A_i} \]

\[ f_{v,v} = \frac{f_{v,x}}{r} \sqrt{(f_{v,d} + f_{v,y})^2 + f_{v,x}^2} \]

Use Eccentrically Loaded Bolt Group Tables
(pp. 8-40 to 8-87, Vol II)
(pp. 5-63 to 5-86, 1st Ed.)

Based on Instantaneous Center of Rotation Method
**ECCENTRIC LOADS ON BOLT GROUPS**

Table 8-19: Coefficients $C$ for Eccentrically Loaded Bolt Groups

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<thead>
<tr>
<th>$s$, in.</th>
<th>$C_s$</th>
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<th>6</th>
<th>7</th>
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<th>9</th>
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<td>1.58</td>
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<td>5.72</td>
<td>6.85</td>
<td>8.06</td>
<td>10.0</td>
</tr>
</tbody>
</table>

**Example**

Determine the maximum design load that the bolt group for the crane rail support can carry. Assume A325-X, 7/8" dia. bolts in single shear.

\[ e_x = 10" \]

From Table 8-24

\[ \phi r_n = 27.1 \text{ kips} \]

From Table 8-19 (p. 8-46)

\[ C = 1.46 \]

\[ \therefore P_u = C \phi r_n = 1.46(27.1) = 39.6 \text{ kips} \]
WELDS

Weld Types:
- fillet weld
- complete penetration groove weld
- partial penetration groove weld
- plug and slot weld

Weld Processes:
- Shielded Metal Arc Welding (SMAW)
- Submerged Arc Welding (SAW)

ELECTRODE - FILLER METAL

Match Weld Metal:

Base Metal: \( F_y = 36 \text{ ksi}, \quad F_u = 60 \text{ ksi} \)

Use E60XX electrodes
(Can use E70XX)

Base Metal: \( F_y = 50 \text{ ksi}, \quad F_u = 70 \text{ ksi} \)

Use E70XX electrodes
DESIGN STRENGTH OF FILLET WELDS

\[ \phi k_w = \phi 0.60 F_{Ex} t_e \]
\[ = 0.75(0.6)F_{Ex} 0.707a \]
\[ = \text{strength per unit length} \]

For E60XX, \( \phi k_w = 1.19 \text{ kips} / 1/16" \) of weld

For E70XX, \( \phi k_w = 1.39 \text{ kips} / 1/16" \) of weld

A fillet weld size, \( a \), of 5/16 in. is common since it is the maximum size that can be placed down in one pass.

Fillet weld along plate edge cannot exceed the plate thickness minus 1/16 in. when the plate thickness is 1/4 in. or greater.
Example  Two plates of A36 steel are to be spliced using a 5/16 in. weld. One plate is 1''x8'' and the other is ½'' x 6''. Determine the required weld length based on the smaller plate capacity.

GSY: \[ P = 0.9(36)(0.5)(6.0) = 97.2 \text{ kips} \]

Use E60XX electrodes

\[ L_{req} = \frac{97.2}{0.75(0.6)(60)(0.707)(5/16)} \]

\[ = 16.3 \text{ in.} \]

---

**ECCENTRIC LOADS ON WELD GROUPS**

\[ C = \frac{P}{C_1DL} \quad D = \frac{P}{CC_1L} \quad L = \frac{P}{CC_1D} \]

- \( D \) = number of 1/16" fillet weld size
- \( C_1 \) = Electrode Coefficient (\( 1.0 \) for E70XX)
  (see Table 8-37, p. 8-158)
ECCENTRIC LOADS ON WELD GROUPS

Table 8-38.
Coefficients C for Eccentrically Loaded Weld Groups
Anglo = 0°

\[ \phi R_n = C C_1 D L \]

\[ C_{\text{max}} = \frac{R_u}{C C_1 D} \]

\[ D_{\text{max}} = \frac{R_u}{C C_1} \]

\[ L_{\text{max}} = \frac{R_u}{C C_0} \]

where

- \( P_u \) = factored force, kips
- \( D \) = number of sustinements-in-an-inch
  in the fillet weld size
- \( i \) = characteristic length of weld group, in
- \( a \) = \( e_x / i \), in
- \( e_x \) = horizontal component of eccentricity of
  \( P_u \) with respect to centroid of weld group, in
- \( C \) = coefficient tabulated below which includes \( a = 0.75 \)
- \( C_1 \) = electrode strength coefficient from Table 8-37
  (1.0 for E70XX electrodes)

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<th>( k )</th>
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<th>0.8</th>
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</table>

Example

Determine the maximum design load that the weld group shown can support. Assume 5/16 in. fillet welds made with E60XX electrodes.

Use Table 8-42 (p. 8-187)

\[ L = 10" \text{, } kL = 6" \]
\[ \therefore k = 0.6 \]
\[ x = 0.164 \text{ (from bottom row)} \]
\[ xL = 0.164(10) = 1.64" \]
\[ e_x = aL = 11 - 1.64 = 9.4" \]
\[ \therefore a = 9.4/10 = 0.94 \]
For \( a = 0.94 \) and \( k = 0.6 \), \( C = 1.51 \)
\( C_1 = 0.857 \) (from p. 8-158)

\[ \phi R_n = C C_1 D L = 1.51(0.857)(5)(10) = 64.7 \text{ kips} \]
COMPLETE PENETRATION GROOVE WELDS

Use "Matching" weld metal

No explicit design

PARTIAL PENETRATION GROOVE WELDS

Weld Metal: \( \phi F_w A_w = 0.80(0.60)F_{EX} A_w \)

Base Metal: \( \phi F_w A_w = 0.9F_y A_w \)
STEEL CONNECTIONS

a)  

SIMPLE CONNECTIONS  (Shear)

- Double Angle (Framed Beam Connection)
- Single Angle
- Seated Beam

MOMENT CONNECTIONS  (Shear and Moment)

- Flange Plate
- Split Tee
- End Plate

DOUBLE ANGLE CONNECTION
DOUBLE ANGLE CONNECTION

1) Shear Capacity of the beam web
2) Bolt hole bearing strength of the beam web
3) Shear Capacity of the bolts (double shear)
4) Bolt hole bearing strength of the angles
5) Net section shear capacity of angles (usually controls)
6) Gross section yielding of angles
7) Shear capacity of bolts at column (single shear but 2x)
8) Bolt hole bearing capacity of the column flange

BOLTED DOUBLE-ANGLE CONNECTIONS

Table 9-2 (pp. 9-22 to 9-87)
WELDED FRAMED BEAM CONNECTIONS

As an alternative to bolting, the angles can be fillet welded to the beam web.

Design Tables are given on pp. 9-88, 9-89, and 9-90 (E70XX)

Bolt bearing in the beam web is no longer a concern if welded

UNSTIFFENED BEAM SEATS

Column
Angle Stabilizes Top Flange
Beam
Flexible Angle

4 - 50
UNSTIFFENED BEAM SEATS

1) Shear capacity of beam web
2) Local web yielding and web crippling of beam web
3) Bending capacity of angle
4) Bolt hole bearing capacity of angle
5) Bolt shear strength at angle-column connection
6) Bolt bearing capacity of column flange (web)

Design tables are provided:

p. 9-136 for bolted connections
p. 9-137 for welded connections
MOMENT CONNECTIONS

A moment connection is made by connecting the flanges

The design of the connection is then based on the magnitude of the force carried through the beam flanges:

\[ P_{uf} = \frac{M_u}{d_m} \]

where

\[ d_m = \text{moment arm and is either the full depth of the beam, } d, \text{ or } d - t, \text{ depending on the type of connection} \]

The shear is still carried by a web connection

MOMENT CONNECTION - WELDED FLANGES

The flanges are welded to the column using complete penetration groove welds.

Only need to match the weld metal with the base metal.

The shear is carried by an end-plate shear connection

\[ d_m = d - t_f \]
FLANGE-PLATED MOMENT CONNECTIONS

Flange-to-Beam Flange Connection

- fillet welded
  (top plate narrower than beam flange)
  (bottom plate wider than beam flange)
- bolted (single shear)

Plate-to-Column Connection

- fillet welded
- complete penetration groove weld

\[ d_m = d \]

MOMENT CONNECTIONS

The beam flange force, \( P_{uf} \), may cause failure of the column flange and or web

The tension force causes:

- flange bending
- local web yielding

The compression force causes:

- web crippling
- compression buckling

The column resistance to these failures is reduced at the end or top of the column (free end)
TIMBER DESIGN

National Design Specifications for Wood Construction (NDS®)

Revised 1991 Edition (Green Cover)

American Forest & Products Associate
1111 19th Street, N.W., Seventh Floor
Washington, D.C. 20037
Attn: Publications Dept.

National Design Specifications Supplement

Contains sawn lumber section properties, allowable stresses for lumber and glued laminated timber

ASD DESIGN METHOD

Design Equation:

\[ f \leq F \]

where

\[ f = \] design stress level due to the load effect
\[ = \] load effect divided by the member section property

\[ F = \] allowable stress level
\[ = \] material resistance divided by a factor of safety
### Structural Sawn Lumber

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Beams &amp; Stringers</th>
<th>Post &amp; Timbers</th>
<th>Decking</th>
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<tbody>
<tr>
<td>b = 2&quot; to 4&quot;</td>
<td>b ≥ 5&quot;</td>
<td>b ≥ 5&quot;</td>
<td>b = 2&quot; to 4&quot;</td>
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<tr>
<td>d ≥ 2&quot;</td>
<td>d ≥ b + 2&quot;</td>
<td>d ≤ b + 2&quot;</td>
<td>used primarily as bending members</td>
</tr>
<tr>
<td>used primarily as columns</td>
<td>weak-axis bending</td>
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</table>

### Section Properties of Standard Dressed Sawn Lumber

(NDS-Sup Table 1B)

<table>
<thead>
<tr>
<th>Nominal Size</th>
<th>Standard Treated Size (S4S) b x d</th>
<th>Area of Section</th>
<th>X-X Axis</th>
<th>Y-Y Axis</th>
<th>Y-Y Axis</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>inches x inches</td>
<td>in²</td>
<td>Section Modulus Sₓₓ</td>
<td>Moment of Inertia Iₓₓ</td>
<td>Section Modulus Sᵧᵧ</td>
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<td>1 x 3</td>
<td>3/4 x 2-1/2</td>
<td>1.675</td>
<td>0.071</td>
<td>0.977</td>
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<td>1 x 4</td>
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<td>2.625</td>
<td>1.531</td>
<td>2.680</td>
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<td>1 x 6</td>
<td>3/4 x 5-1/2</td>
<td>4.125</td>
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<td>1 x 8</td>
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</table>

4 - 55
ALLOWABLE STRESS

Allowable stress values for wood differ for different stress orientations and member types due to its anisotropic nature.

The following designations are used for the tabulated allowable design stresses:

- $F_b$ = extreme fiber in bending
- $F_t$ = tension parallel to grain
- $F_c$ = compression parallel to grain
- $F_{c\perp}$ = compression perpendicular to grain
- $F_v$ = horizontal shear
- $F_g$ = end grain in bearing
### NDS Sup. Base Design Values for Visually Graded Lumber

<table>
<thead>
<tr>
<th>Species and commercial grade</th>
<th>Size classification</th>
<th>Design values in pounds per square inch (psi)</th>
<th>Modulus of Elasticity, E (ksi)</th>
<th>Grading Rules Agency</th>
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<td>625</td>
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<td>Standard</td>
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<td>1,000 x 700</td>
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<td>100</td>
<td>660</td>
</tr>
<tr>
<td>Utility</td>
<td>2&quot; x 8 x 12</td>
<td>850 x 560</td>
<td>100</td>
<td>660</td>
</tr>
<tr>
<td>Construction</td>
<td>2&quot; x 8 x 12</td>
<td>1,100 x 675</td>
<td>90</td>
<td>660</td>
</tr>
<tr>
<td>Utility</td>
<td>2&quot; x 8 x 12</td>
<td>725 x 475</td>
<td>90</td>
<td>660</td>
</tr>
<tr>
<td><strong>SOUTHERN PINE</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dense Select Structural</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Select Structural</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No 1</td>
<td>2&quot; x 10 x 12</td>
<td>2,700 x 1,500</td>
<td>90</td>
<td>660</td>
</tr>
<tr>
<td>No 2</td>
<td>2&quot; x 10 x 12</td>
<td>2,350 x 1,200</td>
<td>90</td>
<td>660</td>
</tr>
<tr>
<td>No 3</td>
<td>2&quot; x 8 x 12</td>
<td>1,740 x 900</td>
<td>90</td>
<td>660</td>
</tr>
<tr>
<td>Standard</td>
<td>2&quot; x 8 x 12</td>
<td>1,500 x 800</td>
<td>90</td>
<td>660</td>
</tr>
<tr>
<td>Utility</td>
<td>2&quot; x 8 x 12</td>
<td>1,240 x 700</td>
<td>90</td>
<td>660</td>
</tr>
</tbody>
</table>

---

4 - 57
ADJUSTMENT FACTORS

Tabulated design values of allowable stress for various species and grades of structural lumber may require modification for certain load conditions and applications:

\[ F' = \sum C \times F \]

<table>
<thead>
<tr>
<th>Adjustment Factor</th>
<th>( F_d )</th>
<th>( F_t )</th>
<th>( F_v )</th>
<th>( F_{c_1} )</th>
<th>( F_c )</th>
<th>( F_g )</th>
<th>( E )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Duration of load, ( C_D )</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Size, ( C_F )</td>
<td>X</td>
<td>X</td>
<td></td>
<td>X</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lateral stability of beams, ( C_L )</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lateral stability of columns, ( C_P )</td>
<td></td>
<td></td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wet use, ( C_M )</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
</tbody>
</table>

See NDS Table 2.3.1 for complete listing

\( C_D \) - DURATION OF LOAD

Accounts for the fact that wood can support higher stresses for shorter periods of time

Tabulated allowable stresses are for normal duration of load

<table>
<thead>
<tr>
<th>Type of Load</th>
<th>Duration of Load</th>
<th>Modification Factor, ( C_D )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead load</td>
<td>Over 10 years, permanent</td>
<td>0.9</td>
</tr>
<tr>
<td>Floor live load</td>
<td>10 years</td>
<td>1.0</td>
</tr>
<tr>
<td>Snow load</td>
<td>2 months</td>
<td>1.15</td>
</tr>
<tr>
<td>Roof Live load</td>
<td>7 days</td>
<td>1.25</td>
</tr>
<tr>
<td>Wind or earthquake</td>
<td>1 day</td>
<td>1.33</td>
</tr>
<tr>
<td>Impact</td>
<td>2 seconds</td>
<td>2.0</td>
</tr>
</tbody>
</table>

Conservative to assume \( C_D = 1.0 \)
BEAMS AND BENDING MEMBERS

Allowable bending stress:

\[ f_b \leq F_{b'} \]

where

\[ f_b = \frac{M}{S} \]

\[ F_{b'} = \sum C \times F_b \]

\[ = C_D \times C_M \times C_L \times C_F \times C_V \times C_{fu} \times C_D \times C_r \times F_b \]

Some factors may already be incorporated into the tabulated values.

For sawn lumber 2" to 4" thick loaded on the wide face, apply Flat Use Factor, \( C_{fu} \).

REPETITIVE MEMBER FACTOR

For dimension lumber 2" to 4" thick used as joists, truss chords, rafters, studs, planks, decking, or similar members:

\[ F_{b'} = C_r \times F_b \]

where

\[ C_r = \text{repetitive member factor} \]

\[ = 1.15 \]

Requires:

- 3 or more members
- spacing \( \leq 24" \) c/c
- floor, roof or other load distributing elements

maximum
## Stability of Bending Members

<table>
<thead>
<tr>
<th>Nominal Depth to Width Ratio</th>
<th>Rule</th>
</tr>
</thead>
<tbody>
<tr>
<td>2:1 or less</td>
<td>No lateral Support is required</td>
</tr>
<tr>
<td>3:1 to 4:1</td>
<td>The ends shall be held in position, as by full-depth solid blocking, bridging, hangers, nailing or bolting to other framing members, or other acceptable means</td>
</tr>
<tr>
<td>5:1</td>
<td>One edge shall be held in line for its entire length</td>
</tr>
<tr>
<td>6:1</td>
<td>Bridging, full-depth at intervals not exceeding 0 ft unless both edges are held in line or unless the compression edge of the member is supported throughout its length to prevent lateral displacement, as by adequate sheathing or subflooring, and the ends at points</td>
</tr>
<tr>
<td>7:1</td>
<td>Both edges shall be held in line for their entire length</td>
</tr>
</tbody>
</table>

## Allowable Shear Stress

For rectangular cross-sections, the shear formula simplifies to the following:

\[ f_v = \frac{3V}{2bd} \]

where:

- \( d \) = depth of the cross-section (in.)
- \( b \) = width of the cross-section (in.)
- \( V \) = design shear force or reaction

When calculating the shear force, \( V \), all loads within a distance \( d \) from either support may be neglected (loads applied to one surface and beam fully supported by the opposite face)
SHEAR AT NOTCHED BENDING MEMBERS

Notched in tension face:

\[ f_v = \frac{3V}{2bd_n} \left[ \begin{array}{c} d \\ d_n \end{array} \right] \]

where

\( d \) = depth of unnotched member

\( d_n \) = depth of member remaining at a notch

\( V \) = shear force determined by conventional means

SOLID RECTANGULAR COLUMNS

\[ r = \frac{d}{\sqrt{12}} \]

Determine effective column length: \( l_e = KL \)

\( \frac{l_e}{r} < 50 \) (75 during construction)
COLUMN STABILITY FACTOR - $C_p$

\[ C_p = \frac{1 + (F_{cE}/F_c^{'})}{2c} - \sqrt{1 + \left(\frac{F_{cE}/F_c^{'}}{2c}\right)^2} - \frac{F_{cE}/F_c^{'}}{c} \]

where

$F_c^{'}$ = tabulated compression design value multiplied by all applicable adjustment factors except $C_p$

$F_{cE} = \frac{K_{cE}E'}{(L_c/d)^2}$

$K_{cE} = 0.3$ for visually graded lumber

BEARING PERPENDICULAR TO GRAIN

Based on net bearing area:

\[ f_{c1} \leq F_{c1}^{'}. \]

For bearing less than 6" in length and not nearing than 3" to the end of the member:

\[ C_b = \text{Bearing area factor} \]

\[ = \frac{L_b - 0.375}{L_b} \]

where $L_b$ is the bearing length measured parallel to grain (in.)
BEARING PARALLEL TO GRAIN

\[ f_g \leq F_g' \]

BEARING PERPENDICULAR TO GRAIN

\[ F_g = \frac{F_g' F_c'}{F_g' \sin^2 \theta + F_c' \cos^2 \theta} \]

\[ \text{Direction of Load} \]
\[ \text{90°} \]
\[ \text{Direction of Grain} \]

CONNECTIONS

Design tables given in NDS

- General Provisions - Part VII
- Bolts - Part VIII
- Lag Screws - Part IX
- Split Ring and Shear Plate Connectors - Part X
- Wood Screws - Part XI
- Nails and Spikes - Part XII

Strength values a function of specific gravity of the wood (see NDS Table 8A)
STRUCTURAL GLUED LAMINATED TIMBER

Fabricated from relatively thin laminations of wood. Stronger, higher grades of wood placed at the outer laminations

Require application of adjustment factors as per NDS Sup. Table 5A, and NDS Sec. 2.3 and Part 5

NDS Sup. Table 5A provides design values (allowable stresses, etc.) for different lamination combinations with various wood species. Section Properties for glued laminated timber are given in NDS Sup. Table 1C

Lamination designation given as XXF-VY where XXF is the tension zone bending stress ($x10^3$ psi), V is for visually graded wood, and Y is for the lamination combination (see UBC Standard No. 25-11).

---

**NDS Sup. Design Values for Glued Laminated Timber**

<table>
<thead>
<tr>
<th>Combination Symbol</th>
<th>Species</th>
<th>Outer Laminations/ Core Laminations</th>
<th>Design values in pounds per square inch (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>BENDING ABOUT X-X AXIS (Loaded Perpendicular to Wide Faces of Laminations)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Design</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>16F-V1</td>
<td>SPISP</td>
<td>1600 800 560$^{14}$ 560$^{13}$ 560$^{10}$ 200 1,400,000 1450 560 175 90 1,300,000</td>
<td></td>
</tr>
<tr>
<td>16F-V2</td>
<td>SPISP</td>
<td>1600 800 560$^{14}$ 560$^{13}$ 560$^{10}$ 200 1,400,000 1450 560 175 90 1,300,000</td>
<td></td>
</tr>
<tr>
<td>16F-V3</td>
<td>SPISP</td>
<td>1600 800 560$^{14}$ 560$^{13}$ 560$^{10}$ 200 1,400,000 1450 560 175 90 1,300,000</td>
<td></td>
</tr>
<tr>
<td>16F-V4$^{+}$</td>
<td>SPISP</td>
<td>1600 800 560$^{14}$ 560$^{13}$ 560$^{10}$ 200 1,400,000 1450 560 175 90 1,300,000</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1600 1650</td>
</tr>
<tr>
<td>20F-V1</td>
<td>SPISP</td>
<td>2000 1000 650 560$^{14}$ 650$^{13}$ 650$^{10}$ 200 1,600,000 1450 560 175 90 1,400,000</td>
<td></td>
</tr>
<tr>
<td>20F-V2</td>
<td>SPISP</td>
<td>2000 1000 650 560$^{14}$ 650$^{13}$ 650$^{10}$ 200 1,600,000 1450 560 175 90 1,400,000</td>
<td></td>
</tr>
<tr>
<td>20F-V3</td>
<td>SPISP</td>
<td>2000 1000 650 560$^{14}$ 650$^{13}$ 650$^{10}$ 200 1,600,000 1450 560 175 90 1,400,000</td>
<td></td>
</tr>
<tr>
<td>20F-V4$^{+}$</td>
<td>SPISP</td>
<td>2000 2000 650 650 650$^{14}$ 650$^{13}$ 650$^{10}$ 200 1,600,000 1450 560 175 90 1,400,000</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2000 2050</td>
</tr>
<tr>
<td>22F-V1</td>
<td>SPISP</td>
<td>2200 1100 650 560$^{14}$ 650$^{13}$ 650$^{10}$ 200 1,600,000 1600 550 175 90 1,500,000</td>
<td></td>
</tr>
<tr>
<td>22F-V2</td>
<td>SPISP</td>
<td>2200 1100 650$^{14}$ 650$^{13}$ 650$^{10}$ 200 1,400,000 1600 550 175 90 1,400,000</td>
<td></td>
</tr>
<tr>
<td>22F-V3</td>
<td>SPISP</td>
<td>2200 1100 650$^{14}$ 650$^{13}$ 650$^{10}$ 200 1,600,000 1600 550 175 90 1,500,000</td>
<td></td>
</tr>
<tr>
<td>22F-V4$^{+}$</td>
<td>SPISP</td>
<td>2200 2200 650 650 650$^{14}$ 650$^{13}$ 650$^{10}$ 200 1,600,000 1600 550 175 90 1,500,000</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2200 2250</td>
</tr>
<tr>
<td>24F-V1</td>
<td>SPISP</td>
<td>2400 1200 650 560$^{14}$ 650$^{13}$ 650$^{10}$ 200 1,700,000 1800 560 175 90 1,500,000</td>
<td></td>
</tr>
<tr>
<td>24F-V2</td>
<td>SPISP</td>
<td>2400 1200 650 560$^{14}$ 650$^{13}$ 650$^{10}$ 200 1,700,000 1800 560 175 90 1,500,000</td>
<td></td>
</tr>
<tr>
<td>24F-V3</td>
<td>SPISP</td>
<td>2400 1200 650 560$^{14}$ 650$^{13}$ 650$^{10}$ 200 1,700,000 1800 560 175 90 1,500,000</td>
<td></td>
</tr>
<tr>
<td>24F-V4$^{+}$</td>
<td>SPISP</td>
<td>2400 1200 650 560 560$^{14}$ 560$^{13}$ 560$^{10}$ 200 1,700,000 1800 560 175 90 1,500,000</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2400 1250</td>
</tr>
</tbody>
</table>

**VISUALLY GRADED SOUTHERN PINE**

---

4 64
### NDS Sup. Section Properties of Glued Laminated Timber

#### SOUTHERN PINE (based on 1-3/8" thick laminations)

<table>
<thead>
<tr>
<th>Net Finished Dimensions b x d</th>
<th>Number of Laminations</th>
<th>Area of Section A</th>
<th>X-X AXIS</th>
<th>Y-Y AXIS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>5' x 5'</td>
<td>5' x 5'</td>
</tr>
<tr>
<td>6-3/4 x 19-1/4</td>
<td>14</td>
<td>179.9</td>
<td>416.9</td>
<td>4012</td>
</tr>
<tr>
<td>6-3/4 x 20-5/8</td>
<td>15</td>
<td>139.2</td>
<td>478.6</td>
<td>4935</td>
</tr>
<tr>
<td>6-3/4 x 22</td>
<td>16</td>
<td>148.5</td>
<td>544.5</td>
<td>5990</td>
</tr>
<tr>
<td>6-3/4 x 23-3/8</td>
<td>17</td>
<td>137.8</td>
<td>614.7</td>
<td>7184</td>
</tr>
<tr>
<td>6-3/4 x 24-3/4</td>
<td>18</td>
<td>167.1</td>
<td>689.1</td>
<td>8529</td>
</tr>
<tr>
<td>6-3/4 x 26-1/8</td>
<td>19</td>
<td>176.3</td>
<td>767.8</td>
<td>10010</td>
</tr>
<tr>
<td>6-3/4 x 27-1/2</td>
<td>20</td>
<td>185.6</td>
<td>850.8</td>
<td>11700</td>
</tr>
<tr>
<td>6-3/4 x 28-7/8</td>
<td>21</td>
<td>194.9</td>
<td>938.0</td>
<td>13540</td>
</tr>
<tr>
<td>6-3/4 x 30-1/4</td>
<td>22</td>
<td>204.2</td>
<td>1029</td>
<td>15570</td>
</tr>
<tr>
<td>6-3/4 x 31-5/8</td>
<td>23</td>
<td>213.5</td>
<td>1125</td>
<td>17970</td>
</tr>
<tr>
<td>6-3/4 x 33</td>
<td>24</td>
<td>222.8</td>
<td>1220</td>
<td>20010</td>
</tr>
<tr>
<td>6-3/4 x 34-1/4</td>
<td>25</td>
<td>232.0</td>
<td>1325</td>
<td>22050</td>
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<tr>
<td>6-3/4 x 35-3/4</td>
<td>26</td>
<td>241.3</td>
<td>1438</td>
<td>25700</td>
</tr>
<tr>
<td>6-3/4 x 37-1/8</td>
<td>27</td>
<td>250.6</td>
<td>1551</td>
<td>28780</td>
</tr>
<tr>
<td>6-3/4 x 38-1/2</td>
<td>28</td>
<td>259.9</td>
<td>1668</td>
<td>32100</td>
</tr>
<tr>
<td>6-3/4 x 39-7/8</td>
<td>29</td>
<td>269.2</td>
<td>1789</td>
<td>36690</td>
</tr>
<tr>
<td>6-3/4 x 41-1/4</td>
<td>30</td>
<td>278.4</td>
<td>1914</td>
<td>39463</td>
</tr>
<tr>
<td>6-3/4 x 42-5/8</td>
<td>31</td>
<td>287.7</td>
<td>2044</td>
<td>43563</td>
</tr>
<tr>
<td>6-3/4 x 44</td>
<td>32</td>
<td>297.0</td>
<td>2178</td>
<td>47920</td>
</tr>
<tr>
<td>6-3/4 x 45-9/16</td>
<td>33</td>
<td>306.3</td>
<td>2316</td>
<td>52550</td>
</tr>
<tr>
<td>6-3/4 x 46-15/16</td>
<td>34</td>
<td>315.6</td>
<td>2459</td>
<td>57470</td>
</tr>
<tr>
<td>6-3/4 x 48-1/8</td>
<td>35</td>
<td>324.8</td>
<td>2606</td>
<td>62700</td>
</tr>
</tbody>
</table>

#### WESTERN SPECIES (based on 1-1/2" thick laminations)

<table>
<thead>
<tr>
<th>Net Finished Dimensions b x d</th>
<th>Number of Laminations</th>
<th>Area of Section A</th>
<th>X-X AXIS</th>
<th>Y-Y AXIS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>5-1/8 x 31-1/2</td>
<td>5-1/8 x 31-1/2</td>
</tr>
<tr>
<td>5-1/8 x 31-1/2</td>
<td>21</td>
<td>161.4</td>
<td>847.5</td>
<td>13350</td>
</tr>
<tr>
<td>5-1/8 x 33</td>
<td>22</td>
<td>165.1</td>
<td>920.2</td>
<td>15260</td>
</tr>
<tr>
<td>5-1/8 x 34-1/2</td>
<td>23</td>
<td>176.8</td>
<td>1017</td>
<td>17540</td>
</tr>
<tr>
<td>5-1/8 x 36</td>
<td>24</td>
<td>184.5</td>
<td>1107</td>
<td>19930</td>
</tr>
<tr>
<td>6-3/4 x 31-7/8</td>
<td>5</td>
<td>50.63</td>
<td>62.28</td>
<td>237.7</td>
</tr>
<tr>
<td>6-3/4 x 33</td>
<td>6</td>
<td>56.75</td>
<td>91.31</td>
<td>410.1</td>
</tr>
<tr>
<td>6-3/4 x 34-1/2</td>
<td>7</td>
<td>70.86</td>
<td>112.4</td>
<td>651.2</td>
</tr>
<tr>
<td>6-3/4 x 35-3/4</td>
<td>8</td>
<td>81.90</td>
<td>126.0</td>
<td>927.9</td>
</tr>
<tr>
<td>6-3/4 x 37-1/8</td>
<td>9</td>
<td>91.13</td>
<td>136.5</td>
<td>1184.1</td>
</tr>
<tr>
<td>6-3/4 x 38-1/2</td>
<td>10</td>
<td>101.3</td>
<td>152.1</td>
<td>1696.2</td>
</tr>
<tr>
<td>6-3/4 x 39-7/8</td>
<td>11</td>
<td>111.4</td>
<td>306.2</td>
<td>2287.6</td>
</tr>
<tr>
<td>6-3/4 x 41-1/4</td>
<td>12</td>
<td>121.5</td>
<td>364.5</td>
<td>3281.7</td>
</tr>
<tr>
<td>6-3/4 x 42-5/8</td>
<td>13</td>
<td>131.6</td>
<td>476.8</td>
<td>4174.8</td>
</tr>
<tr>
<td>6-3/4 x 44</td>
<td>14</td>
<td>141.8</td>
<td>496.1</td>
<td>5209.4</td>
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<tr>
<td>6-3/4 x 45-9/16</td>
<td>15</td>
<td>151.5</td>
<td>569.5</td>
<td>6407.1</td>
</tr>
<tr>
<td>6-3/4 x 46-15/16</td>
<td>16</td>
<td>162.0</td>
<td>648.0</td>
<td>7776.9</td>
</tr>
<tr>
<td>6-3/4 x 48-1/8</td>
<td>17</td>
<td>172.1</td>
<td>731.5</td>
<td>9327.3</td>
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Note: The table provides the area of section, moment of inertia in X-X and Y-Y axes, and finished dimensions for various laminated timber sections.