Chapter 5  Beams

Note that in Example 5.13, the design was controlled by serviceability rather than strength. This is not unusual, but the recommended sequence in beam design is still to select a shape for moment and then check shear and deflection. Although there is no limit on the dead load deflection in this example, this deflection may be needed if the beam is to be cambered.

\[ \Delta_D = \frac{5}{384} \frac{w_{\text{glcr-beam}} L^4}{EI} = \frac{5}{384} \frac{(0.350 + 0.035)/12(30 \times 12)^4}{29,000(510)} = 0.474 \text{ in. for camber} \]

5.12 HOLES IN BEAMS

If beam connections are made with bolts, holes will be punched or drilled in the beam web or flanges. In addition, relatively large holes are sometimes cut in beam webs to provide space for utilities such as electrical conduits and ventilation ducts. Ideally, holes should be placed in the web only at sections of low shear, and holes should be made in the flanges at points of low bending moment. That will not always be possible, so the effect of the holes must be accounted for.

For relatively small holes such as those for bolts, the effect will be small, particularly for flexure, for two reasons. First, the reduction in the cross section is usually small. Second, adjacent cross sections are not reduced, and the change in cross section is actually more of a minor discontinuity than a "weak link." 50/65 = 77%

Holes in a beam flange are of concern for the tension flange only, since bolts in the compression flange will transmit the load through the bolts. This is the same rationale that is used for compression members, where the net area is not considered. The AISC Specification requires that bolt holes in beam flanges be accounted for when the nominal tensile rupture strength (fracture strength) of the flange is less than the nominal tensile yield strength—that is, when

\[ F_u A_{\text{fn}} < Y_f F_y A_{\text{fg}} \]

where

- \( A_{\text{fn}} = \) net tension flange area
- \( A_{\text{fg}} = \) gross tension flange area

If \( F_u / F_y > 0.8 \), the Specification requires that the right hand side of Equation 5.9 be increased by 10%. Equation 5.9 can be written more generally as follows:

\[ F_u A_{\text{fn}} < Y_f F_y A_{\text{fg}} \]

where

- \( Y_f = 1.0 \) for \( F_u / F_y \leq 0.8 \)
- \( Y_f = 1.1 \) for \( F_u / F_y > 0.8 \)

Note that, for A992 steel, the preferred steel for W shapes, the maximum value of \( F_y / F_u \) is 0.85. This means that unless more information is available, use \( Y_f = 1.1 \) for A992. If the condition of Equation 5.10 exists—that is, if

\[ F_u A_{\text{fn}} < Y_f F_y A_{\text{fg}} \]

then...
Table 2-3
Applicable ASTM Specifications for Various Structural Shapes

<table>
<thead>
<tr>
<th>Steel Type</th>
<th>ASTM Designation</th>
<th>$F_Y$ Min. (ksi)</th>
<th>$F_u$ Tensile Stress (ksi)</th>
<th>Applicable Shape Series</th>
<th>HSS</th>
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<th>Pipe</th>
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</table>

- = Preferred material specification.

$^a$ Minimum unless a range is shown.
$^b$ For shapes over 426 lb/ft, only the minimum of 58 ksi applies.
$^c$ For shapes with a flange thickness less than or equal to 1 1/2 in. only.
$^d$ To improve weldability a maximum carbon equivalent can be specified (per ASTM Supplementary Requirement S78). If desired, maximum tensile stress of 90 ksi can be specified (per ASTM Supplementary Requirement S79).
$^e$ If desired, maximum tensile stress of 70 ksi can be specified (per ASTM Supplementary Requirement S91).
$^f$ For shapes with a flange thickness less than or equal to 2 in. only.
$^g$ ASTM A618 can also be specified as corrosion-resistant; see ASTM A618.
$^h$ Minimum applies for walls nominally 3/8 in. thick and under. For wall thicknesses over 3/8 in., $F_y = 46$ ksi and $F_u = 67$ ksi.
$^i$ If desired, maximum yield stress of 65 ksi and maximum yield-to-tensile strength ratio of 0.85 can be specified (per ASTM Supplementary Requirement S79).
$^j$ A maximum yield-to-tensile strength ratio of 0.45 and carbon equivalent formula are included as mandatory in ASTM A992.
$^k$ For shapes with a flange thickness greater than 2 in. only.
$^l$ For shapes with a flange thickness greater than 1 1/2 in. and less than or equal to 2 in. only.
$^m$ For shapes with a flange thickness less than or equal to 1 1/2 in. only.
Why the 1.1? \[ F = 13 - 1 \times 16.1 - 64 \times 244d \]
Bad News: You have to cut your section's modulus down to account for the holes.

Good News: You get to run $F_b$ up to $F_u$ around bolts.

This is similar to what you did in tension members.

5.12 Holes in Beams

(244c)

then AISC F13.1 requires that the nominal flexural strength be limited by the condition of flexural rupture. This limit state corresponds to a flexural stress of

$$f_b = \frac{M_n}{S_x(A_{ph}/A_{fb})} = F_u$$

Solve for $M_n$ including holes

(5.11)

where $S_x(A_{ph}/A_{fb})$ can be considered to be a "net" elastic section modulus. The relationship of Equation 5.11 corresponds to a nominal flexural strength of

$$M_n = F_y Z_x$$

The AISC requirement for holes in beam flanges can be summarized as follows:

If

$$F_u A_{ph} < \frac{Y_t F_y A_{fb}}{Y_t}$$

The nominal flexural strength cannot exceed

$$M_n = \frac{F_u A_{ph}}{A_{fb}} S_x$$

(AISC Equation F13-1)

Examples:

A36 (ductile)

A992 (nonductile)

Whereas, if $F_u A_{ph} \geq \frac{Y_t F_y A_{ph}}{Y_t}$, $M_n = M_p$ (or lift and FLB or...)

Example 5.14

The shape shown in Figure 5.33 is a W18 × 71 with holes in each flange for 1-inch-diameter bolts. The steel is A992. Compute the nominal flexural strength for an unbraced length of 10 feet. Use $C_b = 1.0$.

FIGURE 5.33

$b_f = 7.64''$

t_f = 0.810''

W18 × 71
F13. PROPORTIONS OF BEAMS AND GIRDERS

1. Strength Reductions for Members With Holes in the Tension Flange

This section applies to rolled or built-up shapes and cover-plated beams with holes, proportioned on the basis of flexural strength of the gross section.

In addition to the limit states specified in other sections of this Chapter, the nominal flexural strength, $M_n$, shall be limited according to the limit state of tensile rupture of the tension flange.

(a) When $F_u A_{fn} \geq Y_f F_y A_{fr}$, the limit state of tensile rupture does not apply. Ignore Holes

(b) When $F_u A_{fn} < Y_f F_y A_{fr}$, the nominal flexural strength, $M_n$, at the location of the holes in the tension flange shall not be taken greater than Account for Holes

$$M_n = \frac{F_u A_{fn}}{A_{fr}} S_x$$  \hspace{1cm} (F13-1)

where

$A_{fr}$ = gross area of tension flange, calculated in accordance with the provisions of Section B4.3a, in.$^2$ (mm$^2$)

$A_{fn}$ = net area of tension flange, calculated in accordance with the provisions of Section B4.3b, in.$^2$ (mm$^2$)

$Y_f = 1.0$ for $F_y/F_u \leq 0.8$

$= 1.1$ otherwise
Solution

To determine the nominal flexural strength $M_n$, all applicable limit states must be checked. From the $Z_x$ table, a W18 x 71 is seen to be a compact shape (no footnote to indicate otherwise). Also from the $Z_x$ table, $L_p = 6.00$ ft and $L_r = 19.6$ ft. Therefore for an unbraced length $L_b = 10$ ft,

$$L_p < L_b < L_r$$

and the beam is subject to inelastic lateral-torsional buckling. The nominal strength for this limit state is given by

$$M_n = C_b \left[ M_p - (M_p - 0.7F_y S_x \left( \frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_p$$

(AISC Equation F2-2)

where

$$M_p = F_y Z_x = 50(146) = 7300 \text{ in.-kips}$$

$$M_p = 1.0 \frac{7300 - (7300 - 0.7 \times 50 \times 127)}{19.6 - 6} = 6460 \text{ in.-kips}$$

Check to see if the flange holes need to be accounted for. The gross area of one flange is

$$A_{fb} = t_f b_f = 0.810(7.64) = 6.188 \text{ in.}^2$$

The effective hole diameter is

$$d_h = 1 + \frac{1}{8} = 1.125 \text{ in.}$$

and the net flange area is

$$A_{fn} = A_{fb} - t_f \sum d_h = 6.188 - 0.810(2 \times 1.125) = 4.366 \text{ in.}^2$$

$$F_y A_{fn} = 65(4.366) = 283.8 \text{ kips}$$

Available flange force $F_u$ with holes

Determine $Y_r$. For A992 steel, the maximum $F_y/F_u$ ratio is 0.85. Since this is greater than 0.8, use $Y_r = 1.1$.

$$Y_r F_y A_{fb} = 1.1(50)(6.188) = 340.3 \text{ kips}$$

$$2.8 \approx 3.4$$

Since $F_y A_{fn} < Y_r F_y A_{fb}$, the holes must be accounted for. From AISC Equation F13-1,

$$M_n = \frac{F_y A_{fn}}{A_{fb}} S_x = \frac{283.8}{6.188}(127) = 5825 \text{ in.-kips}$$

This value is less than the LTB value of 6460 in.-kips, so it controls.

Answer

$$M_n = 5825 \text{ in.-kips} = 485 \text{ ft-kips}$$

Then $\Delta M_n = 0.9(485 \text{ ft}) = 436 \text{ ft-kf}$
Table 3-2 (continued)  

**W Shapes**  
Selection by $Z_x$  

<table>
<thead>
<tr>
<th>Shape</th>
<th>$Z_x$</th>
<th>$M_{xx} / \Omega_x$</th>
<th>$\phi_x M_{xx}$</th>
<th>$M_{xx} / \Omega_x$</th>
<th>$\phi_x M_{xx}$</th>
<th>$BF$</th>
<th>$L_p$</th>
<th>$L_f$</th>
<th>$I_x$</th>
<th>$V_{nax} / \Omega_x$</th>
<th>$\phi_x V_{nax}$</th>
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</thead>
<tbody>
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<td>ASD</td>
<td>LRFD</td>
<td>ASD</td>
<td>LRFD</td>
<td>ASD</td>
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<td>ASD</td>
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---

**Notes:**
- ASD and LRFD refer to American Standard and Load and Resistance Factor Design, respectively.
- Shapes exceed compact limit for flexure with $F_y = 50$ ksi.
- Shape does not meet the $N_{nax}$ limit for shear in Specification Section G2.1a with $F_y = 50$ ksi.

**Design of Flexural Members**

---

**American Institute of Steel Construction, Inc.**
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<th>Shape</th>
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<th>Flange Width, (b_f)</th>
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5 Shape is slender for compression with \(F_c = 50\) ksl.
6 Shape exceeds compact limit for flexure with \(F_c = 50\) ksl.
7 The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.
8 Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.
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**Note:** All values are in inches and pounds per linear foot.
Table 3-10 (continued)

**W Shapes**

**Available Moment vs. Unbraced Length**

1) Look on pg B-16 for W18x71's $\phi b'M_y = 548\text{ kip-ft}$

2) Find the W18x71 curve & follow it down until you get to the Lb you want.
Table 3-10 (continued)

**W-Shapes**

Available Moment vs. Unbraced Length

<table>
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<th>$M_0$ (kip-ft)</th>
<th>$\phi M_n$ (kip-ft)</th>
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<td>480</td>
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<tr>
<td>310</td>
<td>465</td>
</tr>
<tr>
<td>300</td>
<td>450</td>
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Unbraced Length (0.5-ft increments)

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American Institute of Steel Construction
5.12 HOLES IN BEAMS

If beam connections are made with bolts, holes will be punched or drilled in the beam web or flanges. In addition, relatively large holes are sometimes cut in beam webs to provide space for utilities such as electrical conduits and ventilation ducts. Ideally, holes should be placed in the web only at sections of low shear, and holes should be made in the flanges at points of low bending moment. That will not always be possible, so the effect of the holes must be accounted for.

For relatively small holes such as those for bolts, the effect will be small, particularly for flexure, for two reasons. First, the reduction in the cross section is usually small. Second, adjacent cross sections are not reduced, and the change in cross section is actually more of a minor discontinuity than a "weak link."

Holes in a beam flange are of concern for the tension flange only, since bolts in the compression flange will transmit the load through the bolts. This is the same rationale that is used for compression members, where the net area is not considered. The AISC Specification requires that bolt holes in beam flanges be accounted for when the nominal tensile rupture strength (fracture strength) of the flange is less than the nominal tensile yield strength—that is, when

\[ F_u A_{fn} < F_y A_{fg} \]  \hspace{1cm} (5.9)

where

- \( A_{fn} \) = net tension flange area
- \( A_{fg} \) = gross tension flange area

If \( F_y / F_u > 0.8 \), the Specification requires that the right hand side of Equation 5.9 be increased by 10%. Equation 5.9 can be written more generally as follows:

\[ F_u A_{fn} < Y_t F_y A_{fg} \]  \hspace{1cm} (5.10)

where

- \( Y_t = 1.0 \) for \( F_y / F_u \leq 0.8 \)
- \( Y_t = 1.1 \) for \( F_y / F_u > 0.8 \)

Note that, for A992 steel, the preferred steel for W shapes, the maximum value of \( F_y / F_u \) is 0.85. This means that unless more information is available, use \( Y_t = 1.1 \).

If the condition of Equation 5.10 exists—that is, if

\[ F_u A_{fn} < Y_t F_y A_{fg} \]

then AISC FI3.1 requires that the nominal flexural strength be limited by the condition of flexural rupture. This limit state corresponds to a flexural stress of

\[ f_b = \frac{M_n}{S_x(A_{fn}/A_{fg})} = F_u \]  \hspace{1cm} (5.11)

where \( S_x(A_{fn}/A_{fg}) \) can be considered to be a "net" elastic section modulus. The relationship of Equation 5.11 corresponds to a nominal flexural strength of

\[ M_n = \frac{F_u A_{fn}}{A_{fg}} S_x \]
The AISC requirement for holes in beam flanges can be summarized as follows:
If
\[ F_s A_{fn} < Y_i F_y A_{fg} \]
The nominal flexural strength cannot exceed
\[ M_n = \frac{F_u A_{fn}}{A_{fg}} S_x \]  \hfill (AISC Equation F13-1)

where
\[ Y_i = \begin{cases} 1.0 & \text{for } F_y/F_u \leq 0.8 \\ 1.1 & \text{for } F_y/F_u > 0.8 \end{cases} \]

The constant \( Y_i \) should be taken as 1.1 for A992 steel or if the maximum value of \( F_y/F_u \) is not known.

**EXAMPLE 5.14**

The shape shown in Figure 5.33 is a W18 × 71 with holes in each flange for 1-inch-diameter bolts. The steel is A992. Compute the nominal flexural strength for an unbraced length of 10 feet. Use \( C_b = 1.0 \).

**FIGURE 5.33**

![Figure 5.33](image)

**SOLUTION**

To determine the nominal flexural strength \( M_n \), all applicable limit states must be checked. From the \( Z_x \) table, a W18 × 71 is seen to be a compact shape (no footnote to indicate otherwise). Also from the \( Z_x \) table, \( L_p = 6.00 \) ft and \( L_r = 19.6 \) ft. Therefore, for an unbraced length \( L_b = 10 \) ft,

\[ L_p < L_b < L_r \]
and the beam is subject to inelastic lateral-torsional buckling. The nominal strength for this limit state is given by

\[ M_n = C_b \left[ M_p - (M_p - 0.7F_yS_x \left( \frac{L_b - L_p}{L_r - L_p} \right) ) \right] \leq M_p \]  

(AISC Equation F2-2)

where

\[ M_p = F_yZ_x = 50(146) = 7300 \text{ in.-kips} \]

\[ M_n = 1.0 \left[ 7300 - (7300 - 0.7 \times 50 \times 127) \left( \frac{10 - 6}{19.6 - 6} \right) \right] = 6460 \text{ in.-kips} \]

Check to see if the flange holes need to be accounted for. The gross area of one flange is

\[ A_{fg} = t_f b_f = 0.810(7.64) = 6.188 \text{ in.}^2 \]

The effective hole diameter is

\[ d_h = 1 + \frac{1}{8} = 1 \frac{1}{8} \text{ in.} \]

and the net flange area is

\[ A_{fn} = A_{fg} - t_f \sum d_h = 6.188 - 0.810(2 \times 1.125) = 4.366 \text{ in.}^2 \]

\[ F_yA_{fn} = 65(4.366) = 283.8 \text{ kips} \]

Determine \( Y_r \). For A992 steel, the maximum \( F_y/F_u \) ratio is 0.85. Since this is greater than 0.8, use \( Y_r = 1.1 \).

\[ Y_rF_yA_{fg} = 1.1(50)(6.188) = 340.3 \text{ kips} \]

Since \( F_yA_{fn} < Y_rF_yA_{fg} \), the holes must be accounted for. From AISC Equation F13-1,

\[ M_n = \frac{F_yA_{fn}}{A_{fg}} S_x = \frac{283.8}{6.188}(127) = 5825 \text{ in.-kips} \]

This value is less than the LTB value of 6460 in.-kips, so it controls.

**Answer** \( M_n = 5825 \text{ in.-kips} = 485 \text{ ft-kips} \).
Beams with large holes in their webs will require special treatment and are beyond the scope of this book. *Design of Steel and Composite Beams with Web Openings* is a useful guide to this topic (Darwin, 1990).

## 5.13 OPEN-WEB STEEL JOISTS

Open-web steel joists are prefabricated trusses of the type shown in Figure 5.34. Many of the smaller ones use a continuous circular bar to form the web members and are commonly called *bar* joists. They are used in floor and roof systems in a wide variety of structures. For a given span, an open-web joist will be lighter in weight than a rolled shape, and the absence of a solid web allows for the easy passage of duct work and electrical conduits. Depending on the span length, open-web joists may be more economical than rolled shapes, although there are no general guidelines for making this determination.

Open-web joists are available in standard depths and load capacities from various manufacturers. Some open-web joists are designed to function as floor or roof joists, and others are designed to function as girders, supporting the concentrated reactions from joists. The AISC Specification does not cover open-web steel joists; a separate organization, the Steel Joist Institute (SJI), exists for this purpose. All aspects of steel joist usage, including their design and manufacture, are addressed in the publication *Standard Specifications, Load Tables, and Weight Tables for Steel Joists and Joist Girders* (SJI, 2005).

An open-web steel joist can be selected with the aid of the standard load tables (SJI, 2005). These tables give load capacities in pounds per foot of length for various standard joists. Tables are available for both LRFD and ASD, in either U.S. Customary units or metric units. One of the LRFD tables is reproduced in Figure 5.35. For each combination of span and joist, a pair of load values is given. The top number is the total load capacity in pounds per foot. The bottom number is the live load per foot that will produce a deflection of 1/360 of the span length. For span lengths in the shaded areas, special bridging (interconnection of joists) is required. The ASD tables use the same format, but the loads are unfactored. The first number in the designation is the nominal depth in inches. The table also gives the approximate weight in pounds per foot of length. Steel fabricators who furnish open-web steel joists must certify that a particular joist of a given designation, such as a 10K1 of span length 20 feet, will have a safe load capacity of at least the value given in the table. Different manufacturers’ 10K1 joists may have
FIGURE 5.35

LRFD

STANDARD LOAD TABLE FOR OPEN WEB STEEL JOISTS, K-SERIES
Based on a 50 ksf Maximum Yield Strength - Loads Shown in Pounds per Linear Foot (plf)

<table>
<thead>
<tr>
<th>Joist Designation</th>
<th>8K1*</th>
<th>10K1</th>
<th>12K1</th>
<th>12K3</th>
<th>12K5</th>
<th>14K1</th>
<th>14K3</th>
<th>14K5</th>
<th>16K2</th>
<th>16K3</th>
<th>16K4</th>
<th>16K5</th>
<th>16K6</th>
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<td>Approx. Wt. (lbs/ft)</td>
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<td>5.2</td>
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<td>Span (ft)</td>
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</table>


*BK1 is no longer made as of 2010.

different member cross sections, but they all must have a nominal depth of 10 inches and, for a span length of 20 feet, a factored load capacity of at least 361 pounds per foot.

The open-web steel joists that are designed to function as floor or roof joists (in contrast to girders) are available as open-web steel joists (K-series, both standard and KCS), longspan steel joists (LH-series), and deep longspan steel joists (DLH-series). Standard load tables are given for each of these categories. The higher you move up the series, the greater the available span lengths and load-carrying capacities become. At the lower
end, an 8K1 is available with a span length of 8 feet and a factored load capacity of 825 pounds per foot, whereas a 72DLH19 can support a load of 745 pounds per foot on a span of 144 feet.

With the exception of the KCS joists, all open-web steel joists are designed as simply supported trusses with uniformly distributed loads on the top chord. This loading subjects the top chord to bending as well as axial compression, so the top chord is designed as a beam-column (see Chapter 6). To ensure stability of the top chord, the floor or roof deck must be attached in such a way that continuous lateral support is provided.

KCS joists are designed to support both concentrated loads and distributed loads (including nonuniform distributions). To select a KCS joist, the engineer must compute a maximum moment and shear in the joist and enter the KCS tables with these values. (The KCS joists are designed to resist a uniform moment and a constant shear.) If concentrated loads must be supported by an LH or a DLH joist, a special analysis should be requested from the manufacturer.

Both top and bottom chord members of K-series joists must be made of steel with a yield stress of 50 ksi, and the web members may have a yield stress of either 36 ksi or 50 ksi. All members of LH- and DLH-series joists can be made with steel of any yield stress between 36 ksi and 50 ksi inclusive. The load capacity of K-series joists must be verified by the manufacturer by testing. No testing program is required for LH- or DLH-series joists.

Joist girders are designed to support open-web steel joists. For a given span, the engineer determines the number of joist spaces, then from the joist girder weight tables selects a depth of girder. The joist girder is designated by specifying its depth, the number of joist spaces, the load at each loaded top-chord panel point of the joist girder, and a letter to indicate whether the load is factored ("F") or unfactored ("K"). For example, using LRFD and U.S. Customary units, a 52G9N10.5F is 52 inches deep, provides for 9 equal joist spaces on the top chord, and will support 10.5 kips of factored load at each joist location. The joist girder weight tables give the weight in pounds per linear foot for the specified joist girder for a specific span length.

**EXAMPLE 5.15**

Use the load table given in Figure 5.35 to select an open-web steel joist for the following floor system and loads:

- Joist spacing = 3 ft 0 in.
- Span length = 20 ft 0 in.

The loads are

- 3-in. floor slab
- Other dead load: 20 psf
- Live load: 50 psf

The live load deflection must not exceed $L/360$. 
For the dead loads of

Slab: \[ 150 \left( \frac{3}{12} \right) = 37.5 \text{ psf} \]

Other dead load: = 20 psf
Joist weight: = 3 psf (est.)
Total: = 60.5 psf
\[ w_D = 60.5(3) = 181.5 \text{ lb/ft} \]

For the live load of 50 psf,
\[ w_L = 50(3) = 150 \text{ lb/ft} \]

The factored load is
\[ w_u = 1.2w_D + 1.6w_L = 1.2(181.5) + 1.6(150) = 458 \text{ lb/ft} \]

Figure 5.35 indicates that the following joists satisfy the load requirement: a 12K5, weighing approximately 7.1 lb/ft; a 14K3, weighing approximately 6.0 lb/ft; and a 16K2, weighing approximately 5.5 lb/ft. No restriction was placed on the depth, so we choose the lightest joist, a 16K2.

To limit the live load deflection to \( L/360 \), the live load must not exceed
\[ 297 \text{ lb/ft} > 150 \text{ lb/ft} \quad \text{(OK)} \]

ANSWER Use a 16K2.

The standard load tables also include a K-series economy table, which facilitates the selection of the lightest joist for a given load.

5.14 BEAM BEARING PLATES AND COLUMN BASE PLATES

The design procedure for column base plates is similar to that for beam bearing plates, and for that reason we consider them together. In addition, the determination of the thickness of a column base plate requires consideration of flexure, so it logically belongs in this chapter rather than in Chapter 4. In both cases, the function of the plate is to distribute a concentrated load to the supporting material.

Two types of beam bearing plates are considered: one that transmits the beam reaction to a support such as a concrete wall and one that transmits a load to the top flange of a beam. Consider first the beam support shown in Figure 5.36. Although many beams are
Chapter 5  Beams

Solution

For the dead loads of:

- Slab: \(150 \times \left(\frac{3}{12}\right) = 37.5 \text{ psf}\)
- Other dead load: = 20 \text{ psf}
- Joist weight: = 3 \text{ psf (est.)}
- Total: = 60.5 \text{ psf}

\(w_D = 60.5(3) = 181.5 \text{ lb/ft}\)

For the live load of 50 psf,

\(w_L = 50(3) = 150 \text{ lb/ft}\)

The factored load is

\(w_u = 1.2w_D + 1.6w_L = 1.2 \times (181.5) + 1.6(150) = 458 \text{ lb/ft}\)

Figure 5.35 indicates that the following joists satisfy the load requirement: a 12K5, weighing approximately 7.1 lb/ft; a 14K3, weighing approximately 6.0 lb/ft; and a 16K2, weighing approximately 5.5 lb/ft. No restriction was placed on the depth, so we choose the lightest joist, a 16K2.

To limit the live load deflection to \(L/360\), the live load must not exceed

\(297 \text{ lb/ft} > 150 \text{ lb/ft}\)

Use a 16K2.

Answer

The standard load tables also include a K-series economy table, which facilitates the selection of the lightest joist for a given load.

5.14 BEAM BEARING PLATES AND COLUMN BASE PLATES

The design procedure for column base plates is similar to that for beam bearing plates, and for that reason we consider them together. In addition, the determination of the thickness of a column base plate requires consideration of flexure, so it logically belongs in this chapter rather than in Chapter 4. In both cases, the function of the plate is to distribute a concentrated load to the supporting material.

Two types of beam bearing plates are considered: one that transmits the beam reaction to a support such as a concrete wall and one that transmits a load to the top flange of a beam. Consider first the beam support shown in Figure 5.36. Although many beams are connected to columns or other beams, the type of support shown here is occasionally used, particularly at bridge abutments. The design of the bearing plate consists of three steps.

1. Determine dimension \(N\) so that web yielding and web crippling are prevented.
2. Determine dimension \(B\) so that the area \(B \times N\) is sufficient to prevent the supporting material (usually concrete) from being crushed in bearing.
3. Determine the thickness \(t\) so that the plate has sufficient bending strength.
Web yielding, web crippling, and concrete bearing strength are addressed by AISC in Chapter 1, "Design of Connections." [16.1-105]

**Web Yielding (ON BOTTOM OF WEB) [16.1-134]**

*Web yielding* is the compressive crushing of a beam web caused by the application of a compressive force to the flange directly above or below the web. This force could be an end reaction from a support of the type shown in Figure 5.36, or it could be a load delivered to the top flange by a column or another beam. Yielding occurs when the compressive stress on a horizontal section through the web reaches the yield point. When the load is transmitted through a plate, web yielding is assumed to take place on the nearest section of width \( t_w \). In a rolled shape, this section will be at the toe of the fillet, a distance \( k \) from the outside face of the flange (this dimension is tabulated in the dimensions and properties tables in the Manual). If the load is assumed to distribute itself at a slope of 1 : 2.5, as shown in Figure 5.37, the area at the support subject to yielding is \((2.5k + N)t_w\). Multiplying this area by the yield stress gives the nominal strength for web yielding at the support:

\[
R_y = (2.5k + N)t_w
\]

(AISC Equation 110-3) [16.1-134]

The bearing length \( N \) at the support should not be less than \( k \).

At the interior load, the length of the section subject to yielding is:

\[
2(2.5k + N) = 5k + N
\]

(INTERIOR PLATE)
2. **Web Local Yielding**

This section applies to *single-concentrated forces* and both components of *double-concentrated forces*.

The *available strength* for the *limit state* of web local yielding shall be determined as follows:

\[ \phi = 1.00 \text{ (LRFD)} \quad \Omega \]

The *nominal strength*, \( R_n \), shall be determined as follows:

(a) When the concentrated *force* to be resisted is applied at a distance from the member end that is greater than the depth of the member, \( d \),

\[ R_n = F_{yw} t_w (5k + l_b) \quad \text{(Interior)} \]  
(J10-2)

(b) When the concentrated force to be resisted is applied at a distance from the member end that is less than or equal to the depth of the member, \( d \),

\[ R_n = F_{yw} t_w (2.5k + l_b) \quad \text{(End)} \]  
(J10-3)

where

- \( F_{yw} \) = *specified minimum yield stress* of the web material, ksi (MPa)
- \( k \) = distance from outer face of the flange to the web toe of the fillet, in. (mm)
- \( l_b \) = length of bearing (not less than \( k \) for end beam reactions), in. (mm)
- \( t_w \) = thickness of web, in. (mm)

When required, a pair of *transverse stiffeners* or a *doubler plate* shall be provided.

3. **Web Local Crippling**

This section applies to compressive *single-concentrated forces* or the compressive component of *double-concentrated forces*.

The *available strength* for the *limit state* of web local crippling shall be determined as follows:

\[ \phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)} \]

The *nominal strength*, \( R_n \), shall be determined as follows:

(a) When the concentrated compressive *force* to be resisted is applied at a distance from the member end that is greater than or equal to \( d/2 \):

\[ R_n = 0.80 t_w^2 \left[ 1 + 3 \left( \frac{l_b}{d} \right) \left( \frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{E F_{yw} t_f} \]  
(J10-4)

(b) When the concentrated compressive force to be resisted is applied at a distance from the member end that is less than \( d/2 \): (*Close to end*)

See (251C)
(i) For \( l_b/d \leq 0.2 \) (short plate close to end of beam)

\[
R_n = 0.40 t_w^3 \left[ 1 + 3 \left( \frac{l_b}{d} \right) \left( \frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{E F_w t_f}{t_w}}
\]  
(J10-5a)

(ii) For \( l_b/d > 0.2 \) (long plate close to end of beam)

\[
R_n = 0.40 t_w^3 \left[ 1 + \left( \frac{4 l_b}{d} - 0.2 \right) \left( \frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{E F_w t_f}{t_w}}
\]  
(J10-5b)

where

\( d = \) full nominal depth of the section, in. (mm)

When required, a transverse stiffener, a pair of transverse stiffeners, or a doubler plate extending at least one-half the depth of the web shall be provided.

4. Web Sidesway Buckling

This section applies only to compressive single-concentrated forces applied to members where relative lateral movement between the loaded compression flange and the tension flange is not restrained at the point of application of the concentrated force.

The available strength of the web for the limit state of sidesway buckling shall be determined as follows:

\[ \phi = 0.85 \text{ (LRFD)} \quad \Omega = 1.76 \text{ (ASD)} \]

The nominal strength, \( R_n \), shall be determined as follows:

(a) If the compression flange is restrained against rotation

(i) When \( (h/t_w)/(L_b/b_f) \leq 2.3 \)

\[
R_n = \frac{C_r t_w^3 f}{h^2} \left[ 1 + 0.4 \left( \frac{h}{t_w} \right) \left( \frac{l_b}{L_b} \frac{t_f}{b_f} \right) \right]^{3}
\]  
(J10-6)

(ii) When \( (h/t_w)/(L_b/b_f) > 2.3 \), the limit state of web sidesway buckling does not apply.

When the required strength of the web exceeds the available strength, local lateral bracing shall be provided at the tension flange or either a pair of transverse stiffeners or a doubler plate shall be provided.

(b) If the compression flange is not restrained against rotation

(i) When \( (h/t_w)/(L_b/b_f) \leq 1.7 \)

\[
R_n = \frac{C_r t_w^3 f}{h^2} \left[ 0.4 \left( \frac{h}{t_w} \right) \left( \frac{l_b}{L_b} \frac{t_f}{b_f} \right) \right]^{3}
\]  
(J10-7)
WEB Crippling
INTERIOR LOAD
Web yielding on bottom of web:

\[ l_b + 2.5 k_{des} \]

Web gets thin after sides

Web yielding on top of web:

\[ l_b + 2(2.5 + k) \]

\[ = l_b + 5k_{des} \]
and the nominal strength is

\[ R_n = (5k + N)F_y t_w \]  \hspace{1cm} \text{(AISC Equation J10-2)}

For LRFD, the design strength is \( \phi R_n \), where \( \phi = 1.0 \).

For ASD, the allowable strength is \( R_n / \Omega_2 \), where \( \Omega_2 = 1.50 \).

### Web Crippling

Web crippling is buckling of the web caused by the compressive force delivered through the flange. For an interior load, the nominal strength for web crippling is

\[ R_n = 0.80 t_w^2 \left[ 1 + 3 \left( \frac{N}{d} \right)(t_w/t_f)^{1.5} \right] \sqrt{\frac{EF_t f}{t_w}} \]  \hspace{1cm} \text{(AISC Equation J10-4)}

For a load at or near the support (no greater than half the beam depth from the end), the nominal strength is

\[ R_n = 0.40 t_w^2 \left[ 1 + 3 \left( \frac{N}{d} \right)(t_w/t_f)^{1.5} \right] \sqrt{\frac{EF_t f}{t_w}} \]  \hspace{1cm} \text{(AISC Equation J10-5a)}

or

\[ R_n = 0.40 t_w^2 \left[ 1 + \left( \frac{4N}{d} - 0.2 \right)(t_w/t_f)^{1.5} \right] \sqrt{\frac{EF_t f}{t_w}} \]  \hspace{1cm} \text{(AISC Equation J10-5b)}

The resistance factor for this limit state is \( \phi = 0.75 \). The safety factor is \( k = 2.00 \).

### Concrete Bearing Strength

The material used for a beam support can be concrete, brick, or some other material, but it usually will be concrete. This material must resist the bearing load applied by the steel plate. The nominal bearing strength specified in AISC J8 is the same as that given in the American Concrete Institute’s Building Code (ACI, 2005) and may be used if no other building code requirements are in effect. If the plate covers the full area of the support, the nominal strength is

\[ P_n = \text{Concrete term} \cdot P_p = 0.85 f' c A_{conc} = A_{plate} \]  \hspace{1cm} \text{(AISC Equation J8-1)}

If the plate does not cover the full area of the support,

\[ P_p = 0.85 f' c \sqrt{\frac{A_2}{A_1}} \leq 1.7 f' c A_{plate} \]  \hspace{1cm} \text{(AISC Equation J8-2)}

\( f' c \) = 28 day compressive strength
J8. COLUMN BASES AND BEARING ON CONCRETE

Proper provision shall be made to transfer the column loads and moments to the footings and foundations.

In the absence of code regulations, the design bearing strength, $\Phi_c P_p$, and the allowable bearing strength, $P_p/\Omega_c$, for the limit state of concrete crushing are permitted to be taken as follows:

$$\Phi_c = 0.65 \text{ (LRFD)} \hspace{1cm} \Omega_c = 2.31 \text{ (ASD)}$$

The nominal bearing strength, $P_p$, is determined as follows:

(a) On the full area of a concrete support:

$$P_p = 0.85 f'_c A_1 \tag{J8-1}$$

(b) On less than the full area of a concrete support:

$$P_m = P_p = 0.85 f'_c A_1 \sqrt{A_2 / A_1} \leq 1.7 f'_c A_1 \tag{J8-2}$$

where

$A_1$ = area of steel concentrically bearing on a concrete support, in.$^2$ (mm$^2$)

$A_2$ = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, in.$^2$ (mm$^2$)

$f'_c$ = specified compressive strength of concrete, ksi (MPa)

J9. ANCHOR RODS AND EMBEDMENTS

Anchor rods shall be designed to provide the required resistance to loads on the completed structure at the base of columns including the net tensile components of any bending moment that may result from load combinations stipulated in Section B2. The anchor rods shall be designed in accordance with the requirements for threaded parts in Table J3.2.

Design of column bases and anchor rods for the transfer of forces to the concrete foundation including bearing against the concrete elements shall satisfy the requirements of ACI 318 or ACI 349.

User Note: When columns are required to resist a horizontal force at the base plate, bearing against the concrete elements should be considered.

When anchor rods are used to resist horizontal forces, hole size, anchor rod setting tolerance, and the horizontal movement of the column shall be considered in the design.

Larger oversized holes and slotted holes are permitted in base plates when adequate bearing is provided for the nut by using ASTM F844 washers or plate washers to bridge the hole.
a. Axial Compression

In members that sustain chiefly or exclusively axial compression loads, such as building columns, it is economical to make the concrete carry most of the load. Still, some steel reinforcement is always provided for various reasons. For one, very few members are truly axially loaded; steel is essential for resisting any bending that may exist. For another, if part of the total load is carried by steel with its much greater strength, the cross-sectional dimensions of the member can be reduced—the more so, the larger the amount of reinforcement.

The two chief forms of reinforced concrete columns are shown in Fig. 1.15. In the square column, the four longitudinal bars serve as main reinforcement. They are held in place by transverse small-diameter steel ties that prevent displacement of the main bars during construction operations and counteract any tendency of the compression-loaded bars to buckle out of the concrete by bursting the thin outer cover. On the left is shown a round column with eight main reinforcing bars. These are surrounded by a closely spaced spiral that serves the same purpose as the more widely spaced ties but also acts to confine the concrete within it, thereby increasing its resistance to axial compression. The discussion that follows applies to tied columns.

When axial load is applied, the compression strain is the same over the entire cross section, and in view of the bonding between concrete and steel, is the same in the two materials (see propositions 2 and 3 in Section 1.8). To illustrate the action of such a member as load is applied, Fig. 1.16 shows two typical stress-strain curves, one for a concrete with compressive strength $f'_c = 4000$ psi and the other for a steel with yield stress $f_y = 60,000$ psi. The curves for the two materials are drawn on the same graph using different vertical stress scales. Curve b has the shape which would be
obtained in a concrete cylinder test. The rate of loading in most structures is considerably slower than that in a cylinder test, and this affects the shape of the curve. Curve c, therefore, is drawn as being characteristic of the performance of concrete under slow loading. Under these conditions, tests have shown that the maximum reliable compressive strength of reinforced concrete is about 0.85 $f'_c$, as shown.

**Elastic Behavior** At low stresses, up to about $f'_c/2$, the concrete is seen to behave nearly elastically, i.e., stresses and strains are quite closely proportional; the straight line d represents this range of behavior with little error for both rates of loading. For the given concrete the range extends to a strain of about 0.0005. The steel, on the other hand, is seen to be elastic nearly to its yield point of 60 ksi, or to the much greater strain of about 0.002.

Because the compression strain in the concrete, at any given load, is equal to the compression strain in the steel,

$$\epsilon_c = \frac{f_c}{E_c} = \varepsilon_s = \frac{f_s}{E_s}$$

from which the relation between the steel stress $f_s$ and the concrete stress $f_c$ is obtained as

$$f_s = \frac{E_s}{E_c} f_c = \eta f_c$$  \hspace{1cm} (1.6)

where $n = E_s/E_c$ is known as the **modular ratio**.
\[ P_p = P_m \]

\[
\frac{(251 - da)}{[6.1 - 132]} \]

Eq. J8-1

\[ P_p = 0.85 f'_c A_1 = P_m \]

\[
\frac{A_2}{A_1} \leq 4
\]

When Limit \( A_2 \) reaches
\[ A_2 \leq 4A_1 \]

\[
P_p = 0.85 f'_c A_1 \sqrt{\frac{4A_1}{A_1}}
\]

\[
= 1.7 f'_c A_1 = \text{limit} \quad \text{(J8-2)}
\]

\[
\frac{[16.1 - 132]}{[251 - 0]} 
\]