
EXAMPLE 4.5

Compute the available strength of the compression member of Example 4.2 with the aid of (a) Table 4-22 from Part 4 of the *Manual* and (b) the column load tables.

**LRFD
SOLUTION**

- a. From Example 4.2, $KL/r = 96.77$ and $F_y = 50$ ksi. Values of $\phi_c F_{cr}$ in Table 4-22 are given only for integer values of KL/r ; for decimal values, KL/r may be rounded *up* or linear interpolation may be used. For uniformity, we use interpolation in this book for all tables unless otherwise indicated. For $KL/r = 96.77$ and $F_y = 50$ ksi,

$$\phi_c F_{cr} = 22.67 \text{ ksi}$$

$$\phi_c P_n = \phi_c F_{cr} A_g = 22.67(21.8) = 494 \text{ kips}$$

- b. The column load tables in Part 4 of the *Manual* give the available strength for selected W-, HP-, single-angle, WT-, HSS, pipe, double-angle, and composite shapes. (We cover composite construction in Chapter 9.) The tabular values for the symmetrical shapes (W, HP, HSS and pipe) were calculated by using the minimum radius of gyration for each shape. From Example 4.2, $K = 1.0$, so

$$KL = 1.0(20) = 20 \text{ ft}$$

For a $W14 \times 74$, $F_y = 50$ ksi and $KL = 20$ ft,

$$\phi_c P_n = 495 \text{ kips}$$

**ASD
SOLUTION**

- a. From Example 4.2, $KL/r = 96.77$ and $F_y = 50$ ksi. By interpolation, for $KL/r = 96.77$ and $F_y = 50$ ksi,

$$F_{cr}/\Omega_c = 15.07 \text{ ksi}$$

Note that this is the allowable stress, $F_a = 0.6F_{cr}$. Therefore, the allowable strength is

$$\frac{P_n}{\Omega_c} = F_a A_g = 15.07(21.8) = 329 \text{ kips}$$

- b. From Example 4.2, $K = 1.0$, so

$$KL = 1.0(20) = 20 \text{ ft}$$

From the column load tables, for a $W14 \times 74$ with $F_y = 50$ ksi and $KL = 20$ ft,

$$\frac{P_n}{\Omega_c} = 329 \text{ kips}$$

The values from Table 4-22 (*Manual*) are based on flexural buckling and AISC Equations E3-2 and E3-3. Thus, local stability is assumed, and width-thickness ratio limits must not be exceeded. Although some shapes in the column load tables exceed those limits (and they are identified with a “c” footnote), the tabulated strength has been computed according to the requirements of AISC Section E7, “Members with Slender Elements,” and no further reduction is needed.

From a practical standpoint, if a compression member to be analyzed can be found in the column load tables, then these tables should be used. Otherwise, Table 4-22 can be used for the flexural buckling strength. If the member has slender elements, the local buckling strength must be computed using the provisions of AISC E7.

4.6 DESIGN

The selection of an economical rolled shape to resist a given compressive load is simple with the aid of the column load tables. Enter the table with the effective length and move horizontally until you find the desired available strength (or something slightly larger). In some cases, you must continue the search to be certain that you have found the lightest shape. Usually the category of shape (W, WT, etc.) will have been decided upon in advance. Often the overall nominal dimensions will also be known because of architectural or other requirements. As pointed out earlier, all tabulated values correspond to a slenderness ratio of 200 or less. The tabulated unsymmetrical shapes—the structural tees and the single and double angles—require special consideration and are covered in Section 4.8.

EXAMPLE 4.6

A compression member is subjected to service loads of 165 kips dead load and 535 kips live load. The member is 26 feet long and pinned at each end. Use A992 steel and select a W14 shape.

LRFD SOLUTION

Calculate the factored load:

$$P_u = 1.2D + 1.6L = 1.2(165) + 1.6(535) = 1054 \text{ kips}$$

$$\therefore \text{Required design strength } \phi_c P_n = 1054 \text{ kips.}$$

From the column load tables for $KL = 1.0(26) = 26$ ft, a W14 \times 145 has a design strength of 1230 kips.

ANSWER

Use a W14 \times 145.

ASD SOLUTION

Calculate the total applied load:

$$P_a = D + L = 165 + 535 = 700 \text{ kips}$$

$$\therefore \text{Required allowable strength } \frac{P_n}{\Omega_c} = 700 \text{ kips}$$