Chapter 22

Beam Columns, (Unedited)

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Examples taken from *Structural Steel Design; LRFD*, by T. Burns, Delmar Publishers, 1995

22.1 Potential Modes of Failures

22.2 AISC Specifications

$$\boxed{\begin{array}{rcl} \dfrac{P_u}{\phi_c P_n} + \dfrac{8}{9}\left(\dfrac{M_{ux}}{\phi_b M_{nx}} + \dfrac{M_{uy}}{\phi_b M_{ny}}\right) & \leq & 1.0 \text{ if } \frac{P_u}{\phi_c P_n} \geq .20 \\ \dfrac{P_u}{2\phi_c P_n} + \dfrac{M_{ux}}{\phi_b M_{nx}} & \leq & 1.0 \text{ if } \frac{P_u}{\phi_c P_n} \leq .20 \end{array}} \tag{22.1}$$

22.3 Examples

22.3.1 Verification

■ **Example 22-1: Verification, (?)**

A W 12 × 120 is used to support the loads and moments as shown below, and is not subjected to sidesway. Determine if the member is adequate and if the factored bending moment occurs about the weak axis. The column is assumed to be perfectly pinned ($K = 1.0$) in both the strong and weak directions and no bracing is supplied. Steel is A36 and assumed $C_b = 1.0$.

be developed or whether buckling behavior controls the bending behavior. The values for L_p and L_r can be calculated using Eq. 6-1 and 6-2 respectively or they can be found in the beam section of the LRFD manual. In either case these values for a W 12 × 120 are:

$$\begin{aligned} L_p &= 13 \text{ ft} \\ L_r &= 75.5 \text{ ft} \end{aligned}$$

5. Since our unbraced length falls between these two values, the beam will be controlled by inelastic buckling, and the nominal moment capacity M_n can be calculated from Eq. 6-5. Using this equation we must first calculate the plastic and elastic moment capacity values, M_p and M_r.

$$\begin{aligned} M_p &= F_y Z_x = (36) \text{ ksi}(186) \text{ in}^3 \frac{\text{ft}}{(12) \text{ in}} = 558 \text{ k.ft} \\ M_r &= (F_{yw} - 10) \text{ ksi} S_x \\ & \quad (36 - 10) \text{ ksi}(163) \text{ in}^3 = 353.2 \text{ k.ft} \end{aligned}$$

6. Using Eq. 6-5 (assuming C_b is equal to 1.0) we find:

$$\begin{aligned} M_n &= C_b[M_p - (M_p - M_r)] \frac{l_b - L_p}{L_r - L_p} \\ M_n &= 1.0[558 - (558 - 453.2)] \text{ k.ft} \frac{(15-13) \text{ ft}}{(75.5-15) \text{ ft}} \\ &= 551.4 \text{ k.ft} \end{aligned}$$

7. Therefore the design moment capacity is as follows:

$$\phi_b M_n = 0.90(551.4) \text{ k.ft} = 496.3 \text{ k.ft}$$

8. Now consider the effects of moment magnification on this section. Based on the alternative method and since the member is not subjected to sidesway ($M_{lt} = 0$)

$$\begin{aligned} M_u &= B_1 M_{nt} \\ B_1 &= \frac{C_m}{1 - \frac{P_u}{P_e}} \\ C_m &= .60 - .4 \frac{M_1}{M_2} \\ C_m &= .60 - .4 \frac{-50}{50} = 1.0 \\ P_u &= 400 \text{ k} \\ P_e &= \frac{\pi^2 E A_g}{\frac{Kl}{r}^2} \quad \text{(Euler's Buckling Load Equation)} \\ P_e &= \frac{\pi^2 (29{,}000 \text{ ksi})(35.3 \text{ in}^2)}{32.67^2} = 9,456 \text{ k} \end{aligned}$$

9. Therefore, calculating the B_1 magnifier we find:

$$B_1 = \frac{1}{1 - \frac{(400) \text{ k}}{(9{,}456) \text{ k}}} = 1.044$$

Calculating the amplified moment as follows:

$$\begin{aligned} M_u &= B_1 M_{nt} \\ M_u &= 1.044(50) \text{ k.ft} = 52.2 \text{ k.ft} \end{aligned}$$

Therefore the adequacy of the section is calculated from Eq. as follows:

$$\begin{aligned} \frac{P_u}{\phi_c P_n} + \frac{8}{9} \frac{M_u x}{\phi_b M_n x} &\leq 1.0 \\ \frac{94000 \text{ k}}{(907.6) \text{ k}} + \frac{8}{9} \frac{(52.2) \text{ k.ft}}{(496.3) \text{ k.ft}} &= \boxed{.53} < 1.0 \surd \end{aligned}$$

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