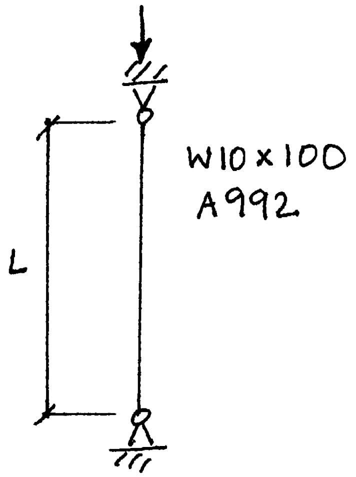
CE470 SPRING 2009 HW6 DUE 3/12/09

Segui Problems 4.3-1 (a & b), 4.3-5 (a, LRFD only)
4.7-3(a), 4.7-5(a), 4.7-12 (a, b, c), 4.7-14 (a)

For all, assume global buckling only; for 4.7-12 (a), do **NOT** use stiffness reduction factor.

## Problem 4.3-1 (a & b)

Use AISC Equation E3-2 or E3-3 and determine the nominal axial compressive strength for the following cases:




a) L = 10 ft.
$A = 29.4 \text{ in}^2$ K = 1.0 pinned-pinned
$r_y = 2.65 \text{ in}$

$$\frac{KL}{r} = \frac{(1.0)(120 \text{ in})}{2.65 \text{ in}} = 45.3$$

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000 \text{ ksi}}{50 \text{ ksi}}} = 113.4$$

$$\frac{KL}{r} < 4.71\sqrt{\frac{E}{F_y}}$$ ∴ Inelastic buckling use (E3-2)

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2(29000 \text{ ksi})}{(45.3)^2} = 140 \text{ ksi}$$

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right]F_y = \left(0.658^{\frac{50}{140}}\right)50 \text{ ksi} = 43.1 \text{ ksi}$$

$$P_n = F_{cr}A_g = (43.1 \text{ ksi})(29.4 \text{ in}^2) = \boxed{1270 \text{ kips}}$$

b) L = 30 ft

$$\frac{KL}{r} = \frac{(1.0)(360 \text{ in})}{2.65 \text{ in}} = 136 > 4.71\sqrt{\frac{E}{F_y}}$$ ∴ elastic buckling use (E3-3)

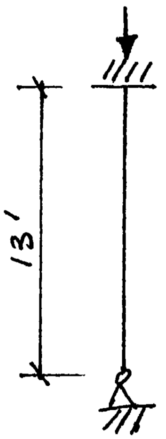
$$F_e = \frac{\pi^2(29000 \text{ ksi})}{(136)^2} = 15.5 \text{ ksi}$$

$$F_{cr} = 0.877 F_e = 0.877(15.5 \text{ ksi}) = 13.6 \text{ ksi}$$

$$P_n = F_{cr}A_g = (13.6 \text{ ksi})(29.4 \text{ in}^2) = \boxed{400 \text{ kips}}$$

Problem 4.3-5 (a, LRFD only)

Determine the available compressive strength. Use AISC Equation E3-2 or E3-3. Use LRFD.




W14x82
A992

Table C-C2.2 – case (b)
Recommended design value – 0.80

W14x82 $A = 24.0 \text{ in}^2$
$r_y = 2.48 \text{ in}$

$$\frac{KL}{r} = \frac{(0.80)(13 \text{ ft} \times 12 \frac{\text{in}}{\text{ft}})}{2.48 \text{ in}} = 50.32$$

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000 \text{ ksi}}{50 \text{ ksi}}} = 113.4 > \frac{KL}{r}$$

$\therefore$ inelastic buckling use E3-2

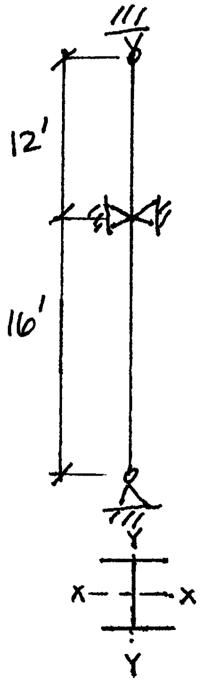
$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 (29000 \text{ ksi})}{(50.32)^2} = 113.0 \text{ ksi}$$

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) F_y = \left(0.658^{\frac{50}{113}}\right) 50 \text{ ksi} = 41.6 \text{ ksi}$$

$$\phi P_n = \phi F_{cr} A_g = 0.9 (41.6 \text{ ksi})(24.0 \text{ in}^2) = \boxed{899 \text{ kips}}$$

## Problem 4.7-3(a)

A W12×79 of A572 Grade 60 steel is used as a compression member. It is 28 feet long, pinned at each end, and has additional support in the weak direction at a point 12 ft. from the top. Can this member resist a service dead load of $180^k$ and a service live load of $320^k$? Use LRFD.




$L_x = 28$ ft $\quad K_x = K_y = 1.0$

$L_y = 16$ ft

$P_u = 1.2 P_D + 1.6 P_L = 1.2(180^k) + 1.6(320^k)$
$= 728$ kips.

W12×79 $\quad A = 23.2\ in^2$
$\quad r_x = 5.34$ in
$\quad r_y = 3.05$ in

$$\frac{K_y L_y}{r_y} = \frac{(1.0)(16\,ft \times 12\,\frac{in}{ft})}{3.05\,in} = 62.95 \leftarrow \text{controls (barely)}$$

$$\frac{K_x L_x}{r_x} = \frac{(1.0)(28\,ft \times 12\,\frac{in}{ft})}{5.34\,in} = 62.92$$

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000\,ksi}{60\,ksi}} = 103.5 > \frac{K_y L_y}{r_y} \quad \therefore \text{inelastic (E3-2)}$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 (29000\,ksi)}{(62.95)^2} = 72.2\ ksi$$

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) F_y = \left(0.658^{\frac{60}{72.2}}\right) 60\,ksi = 42.4\ ksi$$

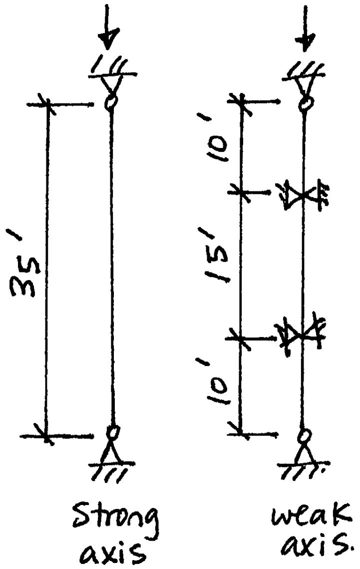
$$\phi P_n = \phi F_{cr} A_g = 0.9(42.4\,ksi)(23.2\,in^2) = 885\ kips$$

$> P_u = 728$ kips ✓ OK

$\boxed{\text{Yes, compression member can resist service dead and live loads.}}$

Problem 4.7-5(a).

Use A992 Steel and select a W-shape. Use LRFD.




D = 380 K

L = 1140 K

$K_x = K_y = 1.0$

$L_x = 35$ ft

$L_y = 15$ ft

$P_u = 1.2D + 1.6L$

$= 1.2(380^k) + 1.6(1140^k)$

$= 2280^k$

try weak axis first (AISC Table 4-1)

$KL = 15$ ft.

W14 x 211 $\phi P_n = 2420^k$

W12 x 230 $\phi P_n = 2450^k$

no W8 or W10 with enough capacity

$r_x/r_y = 1.61$ strong axis.

$$\frac{K_x L_x}{r_x/r_y} = \frac{(1.0)(35\text{ft})}{1.61} = 21.74 \text{ ft.}$$

$\phi P_n \approx 2050^k < 2280^k$ N.G.

TRY W14 x 233

$$\frac{K_x L_x}{r_x/r_y} = \frac{(1.0)(35\text{ft})}{1.62} = 21.6 \text{ ft.}$$

KL = 20ft $\phi P_n = 2400^k$

KL = 22 ft $\phi P_n = 2270^k$

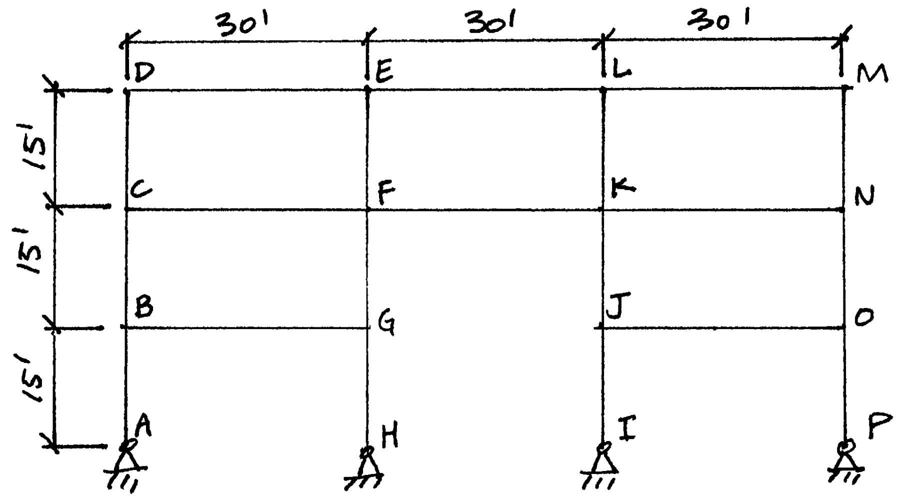
> interpolate $\phi P_n = 2296$ kips $> 2280^k$ ✓ ok.

$\boxed{\text{USE W14x233.}}$

Problem 4.7-12 (a, b, c)

The frame shown is unbraced against sidesway. Assume all columns are W14x61 and all girders are W18x76. ASTM A992 is used for all members. The members are oriented so that bending is about the x-axis. Assume that $K_y = 1.0$.

a) Use the alignment chart to determine $K_x$ for member GF. (Do not use stiffness reduction factor.)




W14x61
$I_x = 640 \text{ in}^4$

W18x76
$I_x = 1330 \text{ in}^4$

$$G_G = \frac{2(640\text{in}^4/15')}{1330\text{in}^4/30'} = 1.92$$

$$G_F = \frac{2(640\text{ in}^4/15')}{2(1330\text{in}^4/30')} = 0.96$$

$$\boxed{K_x \approx 1.45}$$

b) Compute the nominal compressive strength of GF.

W14x61 $\quad A = 17.9 \text{ in}^2$

$r_x = 5.98$ in

$r_y = 2.45$ in

$$\frac{K_x L}{r_x} = \frac{1.45(15' \times 12\frac{\text{in}}{\text{ft}})}{5.98\text{ in}} = 43.64$$

$$\frac{K_y L}{r_y} = \frac{(1.0)(15' \times 12\frac{\text{in}}{\text{ft}})}{2.45\text{in}} = 73.47 \leftarrow \text{controls}$$

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000}{50}} = 113.4 > 73.47 \therefore \text{ inelastic buckling}$$

$$F_e = \frac{\pi^2 E}{(\frac{KL}{r})^2} = \frac{\pi^2(29000\text{ksi})}{(73.47)^2} = 53.0$$

$\longrightarrow$

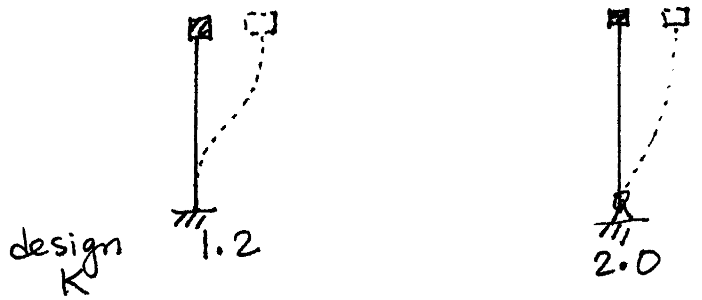
Problem 4.7-12 (a,b,c), cont'd.

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right)F_y = \left(0.658^{\frac{50}{53}}\right)50\,\text{ksi} = 33.69\,\text{ksi}$$

$$P_n = F_{cr}A_g = (33.69\,\text{ksi})(17.9\,\text{in}^2) = \boxed{603\ \text{kips.}}$$

c) Estimate $K_x$ from Table C-C2.2 and compare your estimate with part a).

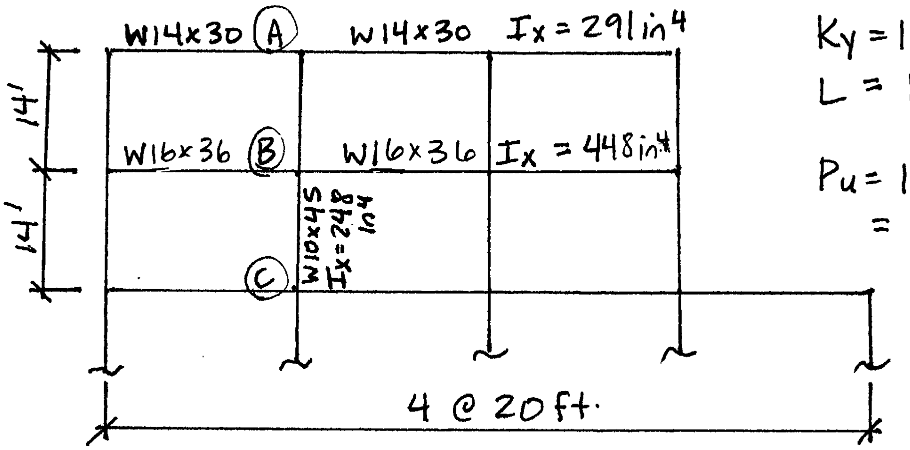
between case (c) and case (f)




→ average of cases ⇒ K = 1.6
a bit more conservative than $K_x$ from (a) ≈ 1.45

Problem 4.7-14 (a)

The rigid frame shown is unbraced in the plane of the frame. In the direction perpendicular to the frame, the frame is braced at the joints (simple (moment-free) connections). Roof girders are W14x30, floor girders are W16x36. Member BC is a W10x45. Use A992 Steel and select a W-shape for AB. Assume no moment in AB for controlling load case. The service dead load is $25^k$ and service live load is $75^k$. Use LRFD.




$K_y = 1.0$

$L = 14$ ft.

$P_u = 1.2(25^k) + 1.6(75^k)$
$= 150^k$

Choose W10 or W8 to splice to W10x45

AISC Table 4-1 $\quad KL = 14$ ft weak axis

W10x33 $\quad \phi P_n = 253^k > 150^k$ ✓ ok.

Smallest sizes in Table. $\rightarrow$ W8x31 $\quad \phi P_n = 248^k > 150^k$ ✓ ok

$\hookrightarrow r_x/r_y = 1.72$

$I_x = 110 in^4$

$$G_A = \frac{(110 in^4/14')}{2(291 in^4/20')} = 0.27$$

$$G_B = \frac{110 in^4/14' + 248 in^4/14'}{2(448 in^4/20')} = 0.57$$

alignment chart $K_x \simeq 1.15$

$$\frac{K_x L}{r_x/r_y} = \frac{1.15(14')}{1.72} = 9.36 \text{ ft} < KL = 14 \text{ ft}$$

$\therefore$ weak axis controls

$\phi P_n = 248^k$.

$\boxed{\text{USE W8x31.}}$