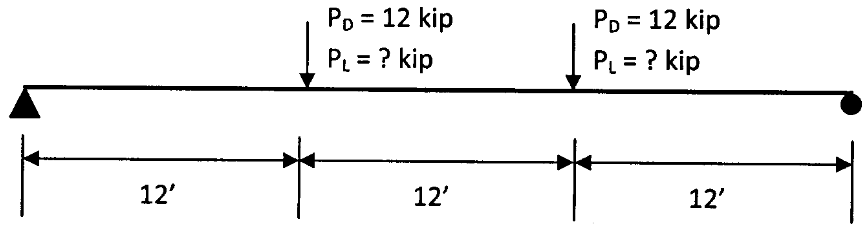
CEE08.381 - Analysis and Design of Steel Frames

Exam 2

April 10, 2008

1. Determine the service live load (P_L) that can be applied at each load point of the beam shown below. The compression flange of the beam is laterally supported at the load points. You may neglect the self-weight of the beam and deflections do not need to be **considered.**

20 PTS



W 24 x 76

A992 Steel

$L_b = 12'$

$C_b = 1.0$

$\phi M_n = 750 - 22.5(12 - 6.78) = 632.6 \text{ ft-K}$

$P_u = 1.2(12) + 1.6 P_L$

$M_u = (1.2(12) + 1.6 P_L)(12') = 172.8 + 19.2 P_L$

$= 632.6$

$\therefore P_L = 23.9^K$

$\phi V_n = 316^K > 1.2(12) + 1.6(23.9) = 52.6$ ✓

2. Select the lightest W shape that can span 25' (simply-supported) and carry a service dead load of 1.5 kip/ft and a service live load of 2.0 kip/ft. The beam must be designed for strength and live load deflection may not exceed span/360 and dead load deflection may not exceed span/240. Assume A992 Gr50 steel and neglect self-weight. The beam is laterally supported only at the ends. 30 PT

$w_u = 1.2(1.5) + 1.6(2) = 5\ ^K/ft$

$M_u = \dfrac{5\ ^K/ft\ (25')^2}{8} = 390.6\ ft\text{-}K$

$C_b = 1.14$

USING TABLE 3-10 SELECT BASED ON $\dfrac{390.6}{1.14} = 342.6\ ft\text{-}K$

W14x74 OR W18x76

$\dfrac{240}{360} = .67$ $\quad \dfrac{2.0}{3.5} = 0.57$ ∴ TOTAL LOAD DEFLECTION CONTROLS OVER LIVE

$\dfrac{25'(12''/ft)}{240} = 1.25'' = \dfrac{5(3.5\ ^K/ft)(1'/12'')(300'')^4}{384(29000\ ^K/in^2)(795\ in^4)} = 1.33''$ N.G.

↑ W14x74

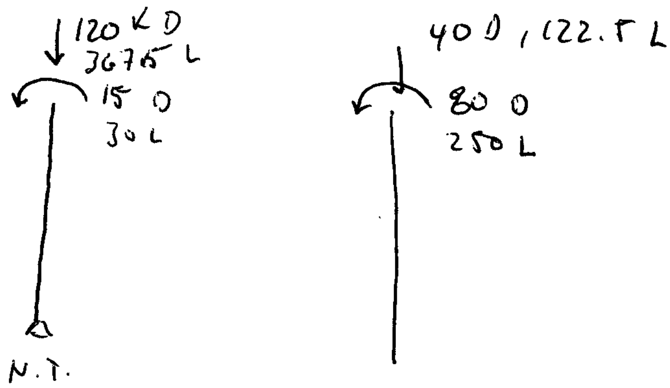
$\Delta\ W18x76 = \dfrac{1.33''(795)}{1330} = 0.78'' < 1.25''$ O.K.

VERIFY STRENGTH

$\phi M_n = 1.14(607 - 12.8(25 - 9.22)) = 466\ ft\text{-}K > 391\ ft\text{-}K$

O.K.

3. A W 14 x 176, A992 member is proposed for use as a 12.5-ft column in an unbraced frame. Will this member be adequate to carry a compressive dead load of 160 kips and live load of 490 kips (assume 75% of the compression load comes from the no-translation case)? The top of the column is loaded with a no-translation dead load moment of 15 ft-kips and a no-translation live load moment of 30 ft-kips. The translation moments applied to that column end are a dead load moment of 80 ft-kips and a live load moment of 250 ft-kips. The lower end of the column is considered pinned and the effective length factor is taken as 1.5. For strong & 1.0 for weak 30 PT



$P_{N.T.} = 1.2(120) + 1.6(367.5)$
$= 732\ K$

$P_{L.T.} = 1.2(40) + 1.6(122.5) = 244^K$

$M_{N.T.} = 1.2(15) + 1.6(30) = 66\ ft\text{-}K$

$M_{LT} = 1.2(80) + 1.6(250) = 496\ ft\text{-}K$

COLUMN STRENGTH

$\frac{(KL)_x}{r_x/r_y} = \frac{1.5(12.5)}{1.6} = \frac{18.75'}{1.6} = 11.72'$

$(KL)_y = 12.5'$

$\phi P_n = 2105^K$ (TABLE 4-1)

BEAM STRENGTH

$L_b = 12.5' < L_p = 14.2'$

$\phi M_n = \phi M_p = 1200\ ft\text{-}K$

MAGNIFIERS

$$B_2 = \frac{1}{1 - \alpha \frac{\Sigma P_{nt}}{\Sigma P_{e2}}}$$

$$P_{e2} = \frac{\pi^2 E I_x}{(KL)_x^2} = \frac{\pi^2 (29000)(2140 \; \text{~~5680~~})}{(18.75 \times 12)^2} = 12100^K \; \text{~~34845~~}^K$$

$$B_2 = \frac{1}{1 - (1)\frac{732}{12100^K \; \text{~~34845~~}}} = 1.06$$

$$B_1 = \frac{C_m}{1 - \alpha \frac{P_r}{P_{e1}}}$$

$$C_m = 0.6 - .4 \frac{M_1}{M_2} = 0.6 \quad (M_1 = 0)$$

$$P_{e1} = \frac{\pi^2 (29000)(2140)}{(12.5 \times 12)^2} = 27223^K$$

$$= \frac{.6}{1 - 1\left(\frac{732 + 1.06(244)}{27223}\right)} = \text{~~1.04~~} \; .58 \nearrow 1.0$$

$P_r = 732 + 1.06(244) = 990.6$

$P_c = 2105^K$

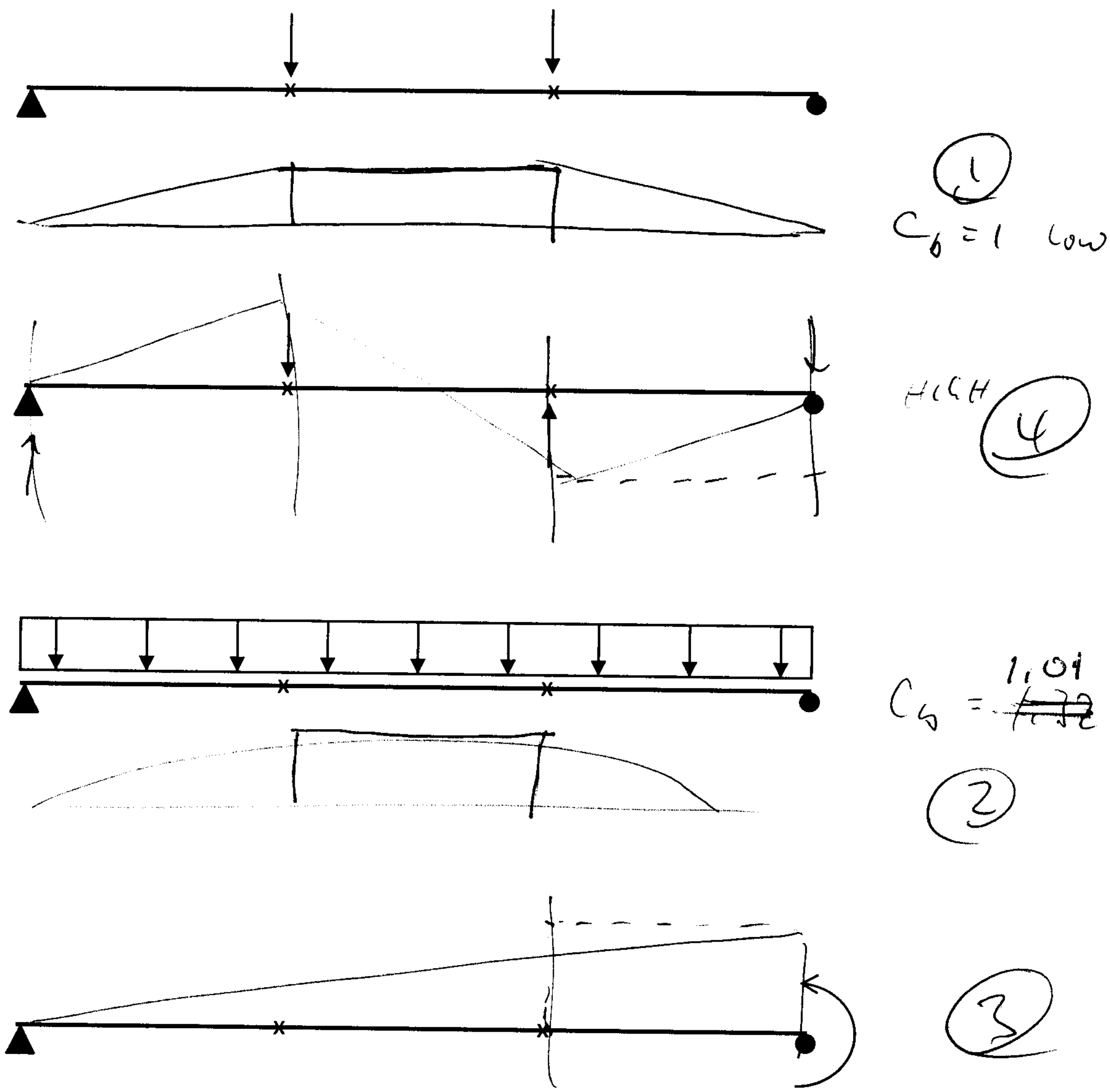
$$\frac{P_r}{P_c} = \frac{990.6}{2105} = 0.47$$

~~.47 + 8/9 [...]~~

$$.47 + \frac{8}{9}\left[\frac{66 + 1.06(496)}{1200}\right] = 0.91 < 1.0 \quad \text{O.K.}$$

.44

4. Four loading diagrams are shown below. The beams are laterally braced at the ends and at the locations marked by an "x". Sketch approximate moment diagrams for each loading case and rank them based on C_b from lowest (1) to highest (4). 10 PTS



OR 2ND & 4TH COULD BOTH TIE AS HIGHEST