5.2-3

For the centroid of half of a W18 × 50, use the centroid of a WT9 × 25:

$$a = d - 2(2.12) = 17.99 - 2(2.12) = 13.75$$

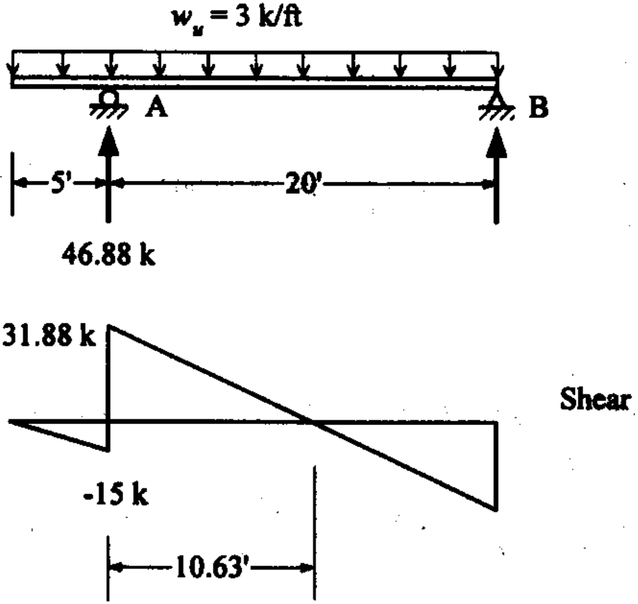
$$Z_x = \frac{A}{2}a = \left(\frac{14.7}{2}\right)(13.75) = 101 \text{ in.}^3$$

$Z_x = 101 \text{ in.}^3$

5.5-2

Factored load and moment:

$$w_u = 1.2w_D + 1.6w_L = 1.2(0.5) + 1.6(1.5) = 3 \text{ kips / ft}$$



Maximum negative moment:

$$M_u = -3(5)(5/2) = -37.5 \text{ ft - kips}$$

Maximum positive moment occurs where the shear is zero:

$$M_u = 46.88(10.63) - 3(5 + 10.63)^2 / 2 = 132 \text{ ft - kips (controls)}$$

Design strength:

$$\frac{b_f}{2t_f} = 6.3 < \frac{65}{\sqrt{36}}, \qquad \frac{h}{t_w} < \frac{640}{\sqrt{36}} \quad \therefore \text{ shape is compact.}$$

$$M_n = M_p = F_y Z = 36(54.0) = 1944 \text{ in.- kips}$$

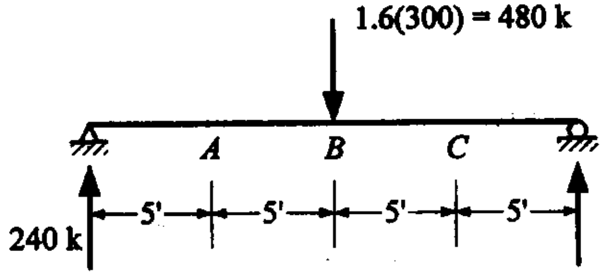
(The check for $F_y Z \le 1.5 M_y = 1.5 F_y S$ is not necessary for I or H shapes bent about the major axis.)

$$\phi_b M_n = 0.9(1944) = 1750 \text{ in.- kips} = 146 \text{ ft - kips}$$

Since $M_u < \phi_b M_n$ (132 ft-kips < 146 ft-kips), <u>Beam is adequate</u>

5.5-6

(a)



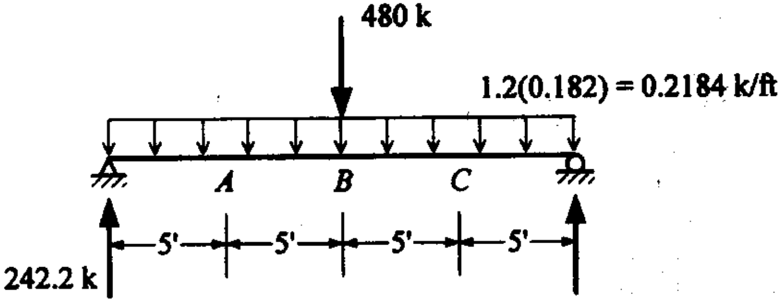
$$M_A = 240(5) = 1200 \text{ ft-kips} = M_C \ , \qquad M_B = M_{max} = 240(10) = 2400 \text{ ft-kips}$$

$$C_b = \frac{12.5 M_{max}}{2.5 M_{max} + 3M_A + 4M_B + 3M_C}$$

$$= \frac{12.5(2400)}{2.5(2400) + 3(1200) + 4(2400) + 3(1200)} = 1.32$$

$\underline{C_b = 1.32}$

(b)



$$M_A = 242.5(5) - 0.2182(5)^2 / 2 = 1208 \text{ ft-kips} = M_C$$

$$M_B = M_{max} = 242.2(10) - 0.2182(10)^2 / 2 = 2411 \text{ ft-kips}$$

$$C_b = \frac{12.5 M_{max}}{2.5 M_{max} + 3M_A + 4M_B + 3M_C}$$

$$= \frac{12.5(2411)}{2.5(2411) + 3(1208) + 4(2411) + 3(1208)} = 1.31$$

$\underline{C_b = 1.31}$

5.10-12

(a) Dead loads:

$$\text{Slab weight} = \frac{6}{12}(150) = 75 \text{ psf}, \quad 75(12) = 900 \text{ lb/ft (tributary width = 12 ft)}$$

Beam: assume beam weight = 100 lb/ft

$w_D = 900 + 100 = 1000$ lb/ft

Live load:

$w_L = 250(12) = 3000$ lb/ft

Factored load and moment:

$$w_u = 1.2w_D + 1.6w_L = 1.2(1.0) + 1.6(3.0) = 6 \text{ kips / ft}$$

$$M_u = \frac{1}{8}w_u L^2 = \frac{1}{8}(6.0)(30)^2 = 675 \text{ ft - kips}$$

From the beam charts with $L_b < L_p$,

Try a W30 × 90:

$\phi_b M_n = 764$ ft-kips $> M_u = 675$ ft-kips

Weight is OK (90 lb/ft < 100 lb/ft)

Shear: From the Factored Uniform Load tables, $\phi_v V_n = 270$ kips

$$V_u \approx \frac{6(30)}{2} = 90 \text{ kips} < 270 \text{ kips (OK)}$$

<u>Use a W30 × 90</u>

(b) Dead loads:

Slab weight = 75 psf, 75(6) = 450 lb/ft (tributary width = 6 ft)

Beam: assume beam weight = 50 lb/ft

$w_D = 450 + 50 = 500$ lb/ft

Live load:

$$w_L = 250(6) = 1500 \text{ lb/ft}$$

Factored load and moment:

$$w_u = 1.2w_D + 1.6w_L = 1.2(0.5) + 1.6(1.5) = 3 \text{ kips / ft}$$

$$M_u = \frac{1}{8}w_u L^2 = \frac{1}{8}(3)(30)^2 = 338 \text{ ft - kips}$$

From the beam charts with $L_b < L_p$,

Try a W24 × 55:

$$\phi_b M_n = 361.5 \text{ ft-kips} > M_u = 338 \text{ ft-kips}$$

Check weight:

$$w_u = 1.2(0.5 + 0.055) + 1.6(1.5) = 3.066 \text{ kips/ft}$$

$$M_u = \frac{1}{8}w_u L^2 = \frac{1}{8}(3.066)(30)^2 = 345 \text{ ft - kips} > 338 \text{ ft - kips} \quad \text{(OK)}$$

Shear: From the Factored Uniform Load tables, $\phi_v V_n = 181$ kips

$$V_u = \frac{3.066(30)}{2} = 46.0 \text{ kips} < 181 \text{ kips} \quad \text{(OK)}$$

<u>Use a W24 × 55</u>

5.13-1

(a) Assume beam weight = 120 lb/ft (based on preliminary calculations)

$$w_u = 1.2w_D = 1.2(2.5 + 0.120) = 3.144 \text{ kips/ft}$$
$$P_u = 1.6P_L = 1.6(30) = 48 \text{ kips}$$
$$M_u = \frac{1}{8}w_u L^2 + \frac{P_u L}{4} = \frac{1}{8}(3.144)(40)^2 + \frac{48(40)}{4} = 1109 \text{ ft-kips}$$

From the Load Factor Design Selection table, try a W33 × 118.

$$\phi_b M_n = \phi_b M_p = 1120 \text{ ft-kips} > 1109 \text{ ft-kips (OK)}$$

Beam weight is OK (2 lb/ft less than assumed):

Shear:

$$V_u = \frac{48}{2} + \frac{1.2(2.5 + 0.118)(40)}{2} = 86.83 \text{ kips}$$

From the Factored Uniform Load tables,

$$\phi_v V_n = 351 \text{ kips} > 86.83 \text{ kips (OK)}$$

Deflection (use service loads):

$$\text{Maximum permissible deflection} = \frac{L}{240} = \frac{40(12)}{240} = 2 \text{ in.}$$

$$\Delta = \frac{5wL^4}{384EI} + \frac{PL^3}{48EI} = \frac{5[(2.5 + 0.118)/12](40 \times 12)^4}{384(29{,}000)(5900)} + \frac{30(40 \times 12)^3}{48(29{,}000)(5900)}$$
$$= 0.8813 + 0.4040 = 1.29 \text{ in.} < 2 \text{ in. (OK)}$$

Use a W33×118

(b) Bearing plate at support:

$$R_u = V_u = 86.83 \text{ kips}$$

Web yielding:

$$\phi R_n \ge R_u:$$
$$\phi(2.5k + N)F_y t_w \ge R_u$$
$$1.0[2.5(1\tfrac{9}{16}) + N](36)(0.550) \ge 86.83 \quad \Rightarrow \quad N \ge 0.479 \text{ in.}$$

Web crippling: assume $\frac{N}{d} < 0.2$

$$\phi R_n \ge R_u:$$
$$\phi\left\{68t_w^2\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{F_y t_f}{t_w}}\right\} \ge R_u$$
$$0.75\left\{68(0.550)^2\left[1 + 3\left(\frac{N}{32.86}\right)\left(\frac{0.550}{0.740}\right)^{1.5}\right]\sqrt{\frac{36(0.740)}{0.550}}\right\} \ge 86.83, \quad N \ge -3.27 \text{ in.}$$
$$\frac{N}{d} = \frac{-3.27}{32.86} = -0.10 < 0.2 \quad \therefore \text{ correct equation was used.}$$

Web yielding controls; required $N = 0.479$ in.

Try $N = 6$ in.

Bearing on concrete support:

Since the support area will be larger than the bearing plate,

$$P_p = 0.85 f_c' A_1 \sqrt{A_2 / A_1} \quad \text{(AISC Eq. J9-2)}$$

As an initial trial, assume conservatively that the full area of the support is in contact, and use

$$P_p = 0.85 f_c' A_1 \quad \text{(AISC Eq. J9-1)}$$

Let $\phi_c P_p > R_u$

$$0.60(0.85)(3.5)(A_1) \geq 86.83, \quad A_1 \geq 48.6 \text{ in.}^2$$

Beam flange width is 11½ in., so try $B = 12$ in.

Try a 6-in. × 12-in. plate.

$$A_1 = 6(12) = 72 \text{ in.}^2 > 48.6 \text{ in.}^2 \quad \text{(OK)}$$

Since the conservative value of P_p is more than adequate, there is no need to use the value from AISC Eq. J9-2.

Plate thickness:

$$n = \frac{B - 2k}{2} = \frac{12 - 2\left(1\frac{9}{16}\right)}{2} = 4.438 \text{ in.}$$

$$t \geq \sqrt{\frac{2.222 R_u n^2}{BNF_y}} = \sqrt{\frac{2.222(86.83)(4.438)^2}{12(6)(36)}} = 1.47 \text{ in.; use } 1\tfrac{1}{2} \text{ in}$$

<u>Use a PL $1\frac{1}{2} \times 6 \times 1'\!-\!0$ at the supports</u>

Bearing plate at concentrated load:

$$R_u = 48 \text{ kips}$$

Web yielding:

$$\phi R_n \geq R_u:$$

$$\phi(5k + N) F_y t_w \geq R_u$$

$$1.0\left[5\left(1\tfrac{9}{16}\right) + N\right](36)(0.550) \geq 48 \quad \Rightarrow \quad N \geq -5.4 \text{ in.}$$

Web crippling:

$$\phi R_n \geq R_u:$$

$$\phi\left\{135 t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{F_y t_f}{t_w}}\right\} \geq R_u$$

$$0.75\left\{135(0.550)^2\left[1 + 3\left(\frac{N}{32.86}\right)\left(\frac{0.550}{0.740}\right)^{1.5}\right]\sqrt{\frac{36(0.740)}{0.550}}\right\} \geq 48, \quad N \geq -13.2 \text{ in.}$$

Required $N < 0$, so bearing plate not needed.

<u>Bearing plate not required at concentrated load</u>