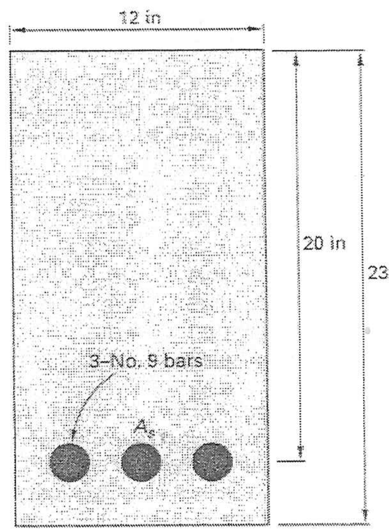
STRUCTURAL DESIGN

Problems 1 and 2 are based on the following information and illustration.

The cross section of a reinforced concrete beam with tension reinforcement is shown. Assume that the beam is underreinforced.

$$f'_c = 3000 \text{ lbf/in}^2$$
$$f_y = 40{,}000 \text{ lbf/in}^2$$
$$A_s = 3 \text{ in}^2 \quad \text{[three no. 9 bars]}$$



Problem 1

In accordance with American Concrete Institute (ACI) strength design, the allowable moment capacity of the beam is most nearly

(A) 160 ft-kips
(B) 180 ft-kips
(C) 200 ft-kips
(D) 210 ft-kips

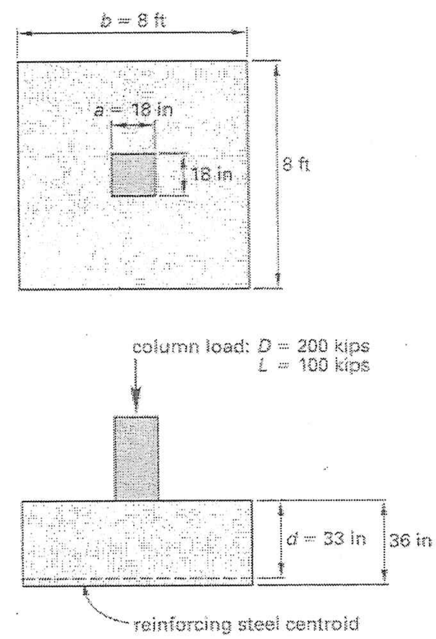
Problem 2

If the dead load shear force in the beam is 5 kips and the live load shear force in the beam is 15 kips, then the minimum amount of shear reinforcement needed for a center-to-center stirrup spacing of 12 in based on ACI strength design is most nearly

(A) 0.0010 in^2
(B) 0.0012 in^2
(C) 0.135 in^2
(D) 0.18 in^2

Problem 3

A square column is supported by a square reinforced concrete footing with depth to reinforcement of d = 33 in as shown. The column supports a dead load of 200 kips and a live load of 100 kips. The ACI code requires that the loaded area of footing for beam shear starts at distance d away from the column face and that the loaded area of footing for punching shear starts at distance $d/2$ away from the column face.

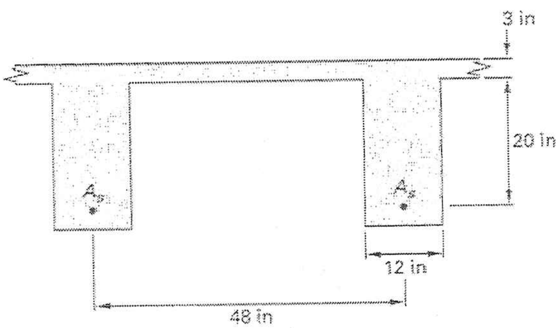


In accordance with ACI strength design, the controlling (maximum) factored shear stress is most nearly

(A) 25 lbf/in^2
(B) 30 lbf/in^2
(C) 35 lbf/in^2
(D) 43 lbf/in^2

Problems 4-6 are based on the following information and illustration.

A floor system consists of 20 reinforced concrete beams and a continuous 3 in deck slab. (A typical section is shown for the deck and two of the beams.) Assume the beams are underreinforced.



$f'_c = 3000$ lbf/in^2

$f_y = 60{,}000$ lbf/in^2

$L = 30$ ft [simple span length]

Problem 4

For each beam in the floor system, the ACI-specified effective top flange width is most nearly

(A) 36 in
(B) 48 in
(C) 60 in
(D) 90 in

Problem 5

Assume the effective flange width for this beam is 48 in. If the area of reinforcing steel per beam is 7.25 in^2, the nominal moment capacity of each beam based on ACI strength design is most nearly

(A) 680 ft-kips
(B) 770 ft-kips
(C) 800 ft-kips
(D) 880 ft-kips

Problem 6

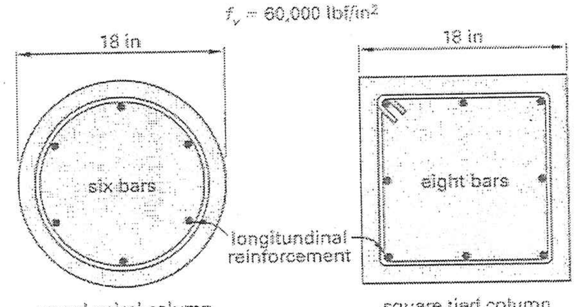
Assume the effective flange width for this beam is 48 in. If the area of reinforcing steel per beam is 6.00 in^2, the nominal moment capacity of each beam based on ACI strength design is most nearly

(A) 150 ft-kips
(B) 160 ft-kips
(C) 590 ft-kips
(D) 650 ft-kips

Problems 7 and 8 are based on the following information and illustration.

The cross sections of two short, concentrically loaded reinforced concrete columns are shown.

$f'_c = 4000$ lbf/in^2
$f_y = 60{,}000$ lbf/in^2



round spiral column (cross section) (Prob. 7)

square tied column (cross section) (Prob. 8)

7. For the short round spiral column, the applied axial dead load is 150 kips, and the applied axial live load is 350 kips. Assuming that the longitudinal reinforcing bars are all the same size, the minimum required size of each longitudinal reinforcing bar is

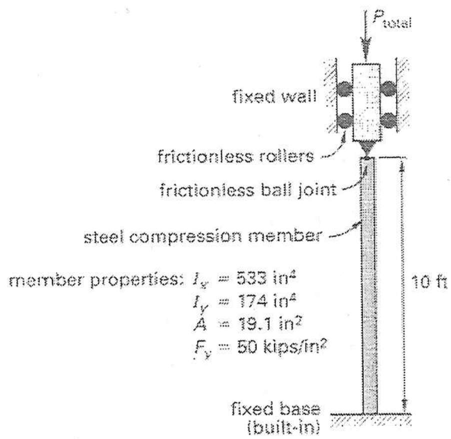
(A) no. 7
(B) no. 8
(C) no. 9
(D) no. 10

8. For the short square tied column, the applied axial dead load is 150 kips, and the applied axial live load is 250 kips. Assuming that the longitudinal reinforcing bars are all the same size, the minimum required size of each longitudinal reinforcing bar is

(A) no. 3
(B) no. 4
(C) no. 5
(D) no. 6

Problems 9 and 10 are based on the following information and illustration.

A steel compression member has a fixed support at one end and a frictionless ball joint support at the other as shown. The total applied design load consists of a dead load of 7 kips (which includes the weight of the member) and an unspecified live load. Recommended effective lengths are to be used.



9. In accordance with American Institute of Steel Construction (AISC) load and resistance factor design (LRFD) specifications, this compression member is a

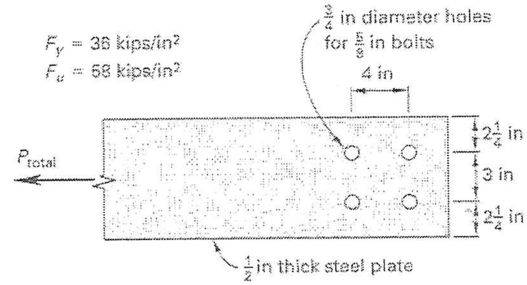
(A) pier
(B) short column
(C) intermediate column
(D) long column

10. In accordance with AISC LRFD specifications, the maximum allowable design live load is most nearly

(A) 340 kips
(B) 490 kips
(C) 550 kips
(D) 650 kips

Problems 11 and 12 are based on the following information and illustration.

A bolted steel tension member is shown. The total applied design load consists of a dead load of 15 kips and an unspecified live load.



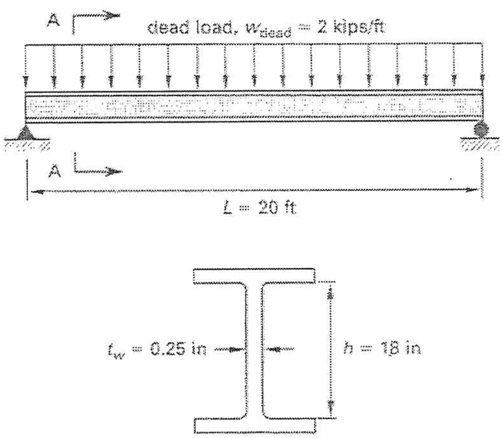
11. What is the effective net area in tension for this plate?

(A) 2.25 in^2
(B) 2.5 in^2
(C) 3.0 in^2
(D) 3.2 in^2

12. In accordance with AISC LRFD specifications, the maximum allowable design live load is most nearly

(A) 50 kips
(B) 56 kips
(C) 65 kips
(D) 70 kips

13. A steel beam is shown.



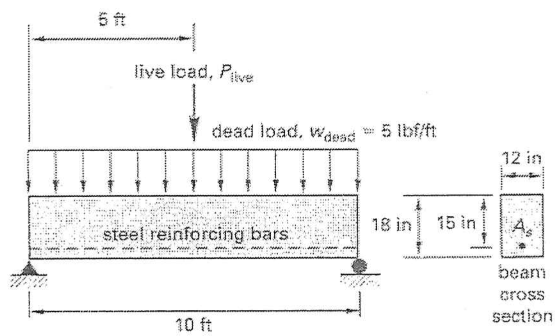
section A-A

The yield strength is 50 kips/in^2. Neglect beam weight. In accordance with AISC LRFD specifications, the maximum allowable live load is most nearly

(A) 2 kips/ft
(B) 5 kips/ft
(C) 6 kips/ft
(D) 8 kips/ft

Problems 14–16 are based on the following information and illustration.

The span length and cross section of a reinforced concrete beam are shown. The beam is underreinforced. The concrete and reinforcing steel properties are f'_c = 3000 lbf/in², f_y = 40,000 lbf/in², and A_s = 3 in².



14. Neglecting beam self-weight and based only on the allowable moment capacity of the beam as determined using American Concrete Institute (ACI) strength design specifications, the maximum allowable live load is most nearly

(A) 23,000 lbf
(B) 29,000 lbf
(C) 35,000 lbf
(D) 50,000 lbf

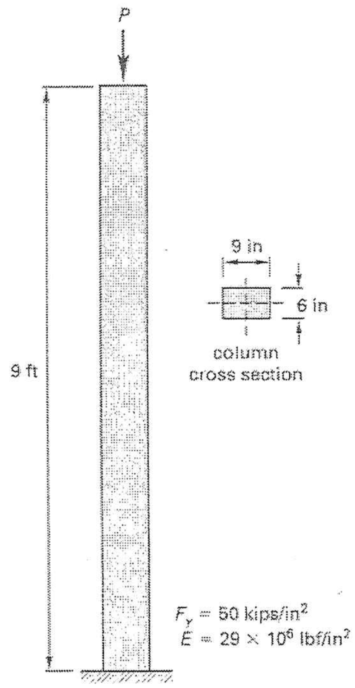
15. The beam supports a concentrated live load of 50,000 lbf. Neglect beam self-weight. The minimum amount of shear reinforcement required for a center-to-center stirrup spacing of 12 in under ACI strength design specifications is most nearly

(A) 0.18 in²
(B) 0.36 in²
(C) 0.67 in²
(D) 0.78 in²

16. The balanced reinforcing steel ratio for this beam in accordance with ACI specifications is most nearly

(A) 0.037
(B) 0.043
(C) 0.051
(D) 0.058

Problems 17 and 18 are based on the following information and illustration. A solid steel column with a fixed support and material and geometric properties as shown is concentrically loaded.



17. In accordance with American Institute of Steel Construction (AISC) load and resistance factor design (LRFD) specifications, the available axial compressive stress for design purposes is most nearly

(A) 18 kips/in²
(B) 26 kips/in²
(C) 29 kips/in²
(D) 39 kips/in²

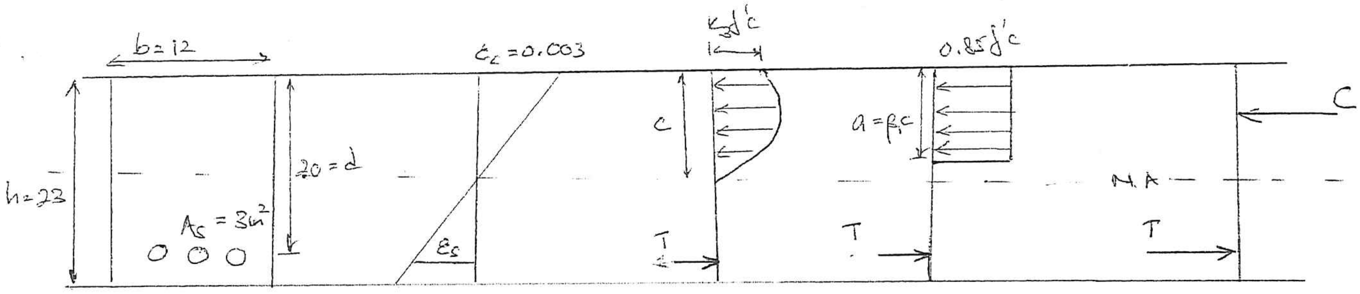
18. If the column is braced against buckling in the weak direction at midheight, the available capacity is

(A) 470 kips
(B) 780 kips
(C) 940 kips
(D) 1400 kips

FE REVIEW

STRUCTURAL DESIGN

1



$\beta_1 = 0.85$ since $f'_c < 4000$ psi

$a = \dfrac{A_s f_y}{0.85 f'_c \cdot b} = \dfrac{3 \times 40}{0.85 \times 3 \times 12} = 3.92$ in → refer BEAMS - FLEXURE in reference.

$M_n = 0.85 f'_c \, a . b . (d - \frac{a}{2}) = A_s . f_y (d - \frac{a}{2})$

$= 0.85 \times 3 \times 3.92 \times 12 \times (20 - \frac{3.92}{2})$

$= 2163.9$ k-in $= 180.3$ k-ft

$M_{all} = \phi M_n$ → refer RESISTANCE FACTORS

$= 0.9 \times 180.3$

$= 162.3$ k-ft.

Answer is A.

---

reinforcement ratio $\rho = \dfrac{A_s}{bd} = \dfrac{3}{12 \times 20} = 0.0125$

$A_{s\,min} = \begin{cases} \dfrac{3 b_w d \sqrt{f'c}}{f_y} = \dfrac{3 \times 12 \times 20 \sqrt{3000}}{40{,}000} = 0.986 \text{ in}^2 \\ \dfrac{200\, b_w d}{f_y} = \dfrac{200 \times 12 \times 20}{40{,}000} = 1.2 \text{ in}^2 \end{cases}$ → controls

$A_{s\,max} = \dfrac{0.85 f'_c \beta_1 b}{f_y} \left(\dfrac{3 d_t}{7}\right) = \dfrac{0.85 \times 3 \times 0.85 \times 12}{40} \times \left(\dfrac{3 \times (20 + \frac{1.128}{2})}{7}\right)$ → refer ASTM Rebar size

$= 5.73$ in^2

$\therefore$ 1.2 in^2 < 3 in^2 < 5.73 in^2 → O.K.

(2)

$V_u = 1.2\, V_{DEAD} + 1.6\, V_{LIVE}$ → refer load factors.

$= (1.2 \times 5) + (1.6 \times 15)$

$= 30$ kips.

Nominal shear strength → refer BEAMS - SHEAR

$V_n = V_c + V_s$

$V_c = 2\sqrt{f'_c}\, b.d = 2 \times \sqrt{3000} \times 12 \times 20$

$= 26.29$ kips

$\phi = 0.75$ for shear → refer RESISATNCE FACTORS

$\therefore \dfrac{\phi V_c}{2} = \dfrac{0.75 \times 26.29}{2} = 9.86$ kips $< V_u$

$\therefore$ shear reinforcement is required

$\phi V_c = 0.75 \times 26.29 = 19.72$ kips $< V_u$

$\therefore V_s = \dfrac{V_u}{\phi} - V_c = \dfrac{30}{0.75} - 26.29 = 13.71$ kips.

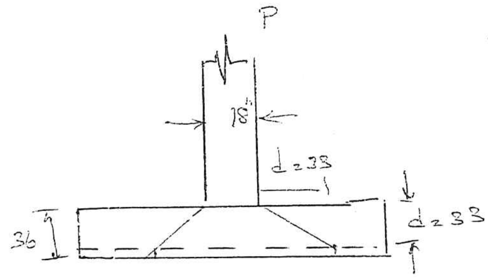
$\therefore A_v = \dfrac{s.\, V_s.}{f_y.\, d} = \dfrac{12 \times 13.71}{40 \times 20} = 0.20\ in^2$

$A_{v\,min} = \begin{cases} \dfrac{s \times 50 \times b_w}{f_y} = 0.18 \rightarrow \text{controls} \\ \dfrac{s \times 0.75 \times b_w \times \sqrt{f'_c}}{f_y} = 0.148 \end{cases}$

$\therefore A_v > A_{vmin}$ use $A_v = 0.2\ in^2$ //

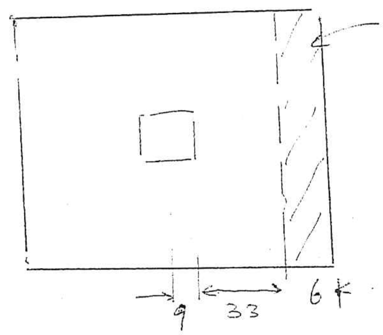
Answer is D ?

(3)



$$P_u = 1.2D + 1.6L$$
$$= (1.2 \times 200) + (1.6 \times 100)$$
$$= 400 \text{ kips}.$$

Soil pressure $q_u = \frac{P_u}{A_g} = \frac{400}{8 \times 8} = 6.25 \text{ kip/ft}^2$

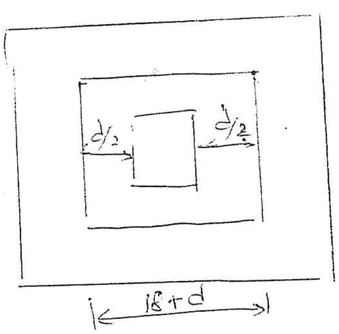


loaded area for beam shear

$$v_{bs} = \frac{V_u}{bd} = \frac{\left(6.25 \times 8 \times \frac{6}{12}\right)}{\left(8 \times \frac{33}{12}\right)} = 1.14 \text{ kip/ft}^2$$

$$v_{bs} = 7.89 \text{ psi}$$

//



$$v_{ps} = \frac{V_u}{bd} = \frac{6.25 \times \left(8 \times 8 - \frac{(18+33)^2}{144}\right)}{4 \times \frac{(18+33)}{12} \times \frac{33}{12}}$$

$$= 6.14 \text{ kip/ft}^2$$

$$= 42.65 \text{ psi}$$

//

∴ punching shear governs.

Answer is D

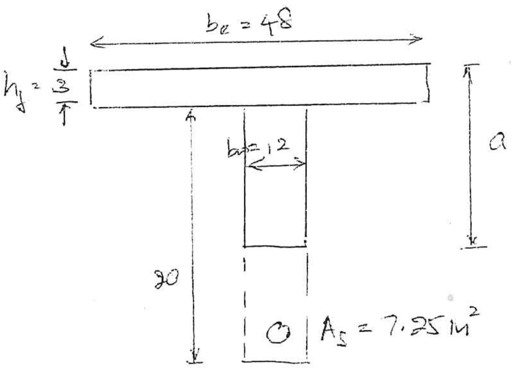
(4) Refer – BEAM FLEXURE – T-beams.

$$b_e = \text{smaller} \begin{cases} 1/4 \times \text{span} & = \frac{30 \times 12}{4} = 90 \text{ in} \\ b_w + (16 \times h_f) & = 12 + (3 \times 16) = 60 \text{ in} \\ \text{center to center spacing} & = 48 \text{ in} \end{cases}$$

$\therefore b_e = 48$ in //

Answer is B

(5)



Assume beam is rectangular.

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{7.25 \times 60}{0.85 \times 3 \times 48}$$

$= 3.55$ in $> h_f$.

$\therefore$ N.A is within web

$\therefore$ Again from force equilibrium

$$0.85 f'_c A_c = A_s f_y$$

$$0.85 f'_c \left[(b_e - b_w) \times h_f + b_w \cdot a \right] = A_s \cdot f_y$$

$$0.85 \times 3 \left[(48-12) \times 3 + 12a \right] = 7.25 \times 60$$

$$30.6\, a = 159.6$$

$$a = 5.22 \text{ in} //$$

$$\therefore M_n = 0.85 f'_c \left[h_f (b_e - b_w)\left(d - \frac{h_f}{2}\right) + a b_w \left(d - \frac{a}{2}\right) \right]$$

$$= 0.85 \times 3 \left[3 (48-12)\left(23 - \frac{3}{2}\right) + 5.22 \times 12 \times \left(23 - \frac{5.22}{2}\right) \right]$$

$= 9178$ kip-in

$= 765$ kip-ft // Answer is B

(6)

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{6 \times 60}{0.85 \times 3 \times 48}$$

$= 2.94$ in $< h_f = 3$ in

$$\therefore M_n = 0.85 f'_c \times a \times b_e \left(d - \frac{a}{2}\right)$$

$$= 0.85 \times 3 \times 2.94 \times 48 \times \left(23 - \frac{2.94}{2}\right)$$

$= 7747.5$ kip-in

$= 645.6$ kip-ft //

Answer is D

7

Refer SHORT COLUMNS

$0.01 \leq \rho_g \leq 0.08$

$A_s = 0.01\, A_g = 0.01 \times \left(\frac{\pi \times 18^2}{4}\right)$

$= 2.54 \text{ in}^2$

$P_u = 1.2\, P_{Dead} + 1.6\, P_{LIVE}$

$= (1.2 \times 150) + (1.6 \times 350)$

$= 740 \text{ kips}$

$\phi P_n \geq P_u \quad \phi = 0.7 \longrightarrow$ refer RESISTANCE FACTORS

$\phi P_n = \phi \cdot 0.85 \left[0.85 f'_c (A_g - A_s) + A_s f_y\right] \geq P_u$

$\Rightarrow 0.7 \times 0.85 \left[0.85 \times 4 \times \left(\frac{\pi \times 18^2}{4} - A_s\right) + A_s \times 60\right] \geq P_u$

$\Rightarrow 0.7 \times 0.85 \left[56.6 A_s + 865.2\right] \geq 740 \text{ kips}$

$\Rightarrow \quad A_s \geq 6.69 \text{ in}^2$

$\therefore A = \frac{6.69}{6} = 1.11 \text{ in}^2/\text{bar} \rightarrow$ Use # 10 bar.

6 ← # of bars

Answer is D

8

$A_s = 0.01\, A_g = 0.01 \times 18^2$

$= 3.24 \text{ in}^2$

$P_u = (1.2 \times 150) + (1.6 \times 250) = 580 \text{ kips}$

$\phi P_n = 0.8\, \phi \left[0.85 f'_c (A_g - A_s) + A_s f_y\right] \geq P_u$

$\Rightarrow 0.8 \times 0.65 \left[0.85 \times 4 \times (18^2 - A_s) + 60 A_s\right] \geq 580 \text{ kips}$

$\Rightarrow 56.6\, A_s \geq 13.78$

$A_s \geq 0.24$

$\therefore$ Use minimum $A_s = 3.24$

$A = \frac{3.24}{8} = 0.405 \text{ in}^2 \rightarrow$ use # 6 bar

Answer is D

(9) Refer COLUMNS (AISC table C-C2.1).

$k_x = k_y = 0.8$

$L_x = L_y = 120$ in

$r_x = \sqrt{\frac{I_x}{A}}$

$= 5.28$ in

$r_y = \sqrt{\frac{I_y}{A}}$

$= 3.02$ in

$S.R_y > S.R_x$

$\therefore$ $S.R_y$ governs.

$S.R_x = \frac{k_x L_x}{r_x}$

$= \frac{0.8 \times 120}{5.28}$

$= 18.2$

$SR_y = \frac{k_y L_y}{r_y}$

$= \frac{0.8 \times 120}{3.02}$

$= 31.8$

$\frac{kL}{r} \leq 4.71\sqrt{\frac{E}{F_y}}$ — inelastic

$\frac{kL}{r} > 4.71\sqrt{\frac{E}{F_y}}$ — elastic

S.R < 50 — short

50 < S.R. < 200 — intermediate

Answer is B

(10)

$\phi F_{cr} = \phi\left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y$.

$F_E = \frac{\pi^2 E}{\left(\frac{kL}{r}\right)^2} = \frac{\pi^2 \times 29000}{31.8^2}$

$= 283.04$

$\phi = 0.9 \longrightarrow$ refer AISC table.

$\phi F_{cr} = 0.9 \times \left(0.658^{\frac{50}{283.04}}\right) \times 50$

$= 41.79$ kip/in^2 $\longrightarrow$ can be read directly from table.

$\therefore$ Column capacity $= P = \phi F_{cr} \cdot A$

$= 41.79 \times 19.1$

$= 798.2$ kips

$\therefore$ $P_{tot} = P = 1.2 P_{Dead} + 1.6 P_{Live}$

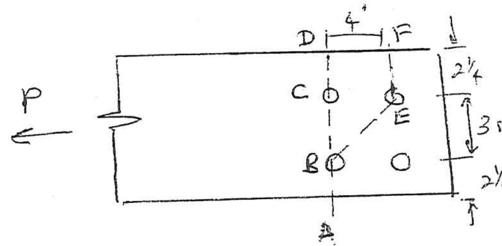
$798.2 = (1.2 \times 7) + 1.6 P_{Live}$.

$\therefore P_{Live} = 493.6$ kips.

$\therefore$ Answer is B.

(11)

Refer STEEL STRUCTURES TENSION MEMBERS



$A_g = (2.25 + 3 + 2.25) \times 0.5 = 3.75\ in^2$

$b_{n\,ABEF} = b - \Sigma d + \Sigma \frac{s^2}{4g}$

$= 7.5 - (2 \times 0.75) + \frac{4^2}{4 \times 3}$

$= 7.33$ in

$b_{n\,ABCD} = b - \Sigma d$

$= (2.25 + 3 + 2.25) - (2 \times 0.75)$

$= 6$ in ← controls.

$\therefore A_n = b_{n\,ABCD} \times t = 6 \times 0.5 = 3\ in^2$ //

$\therefore A_e = U.A_n$ $U = 1.0$ for plate/flat bar

$\therefore A_e = 3\ in^2$ //

Answer is C

(12)

$\phi T_n = \phi_y \times A_g \times F_y = 0.9 \times 3.75 \times 36 = 121.5$ kips → for Yielding

$\phi T_n = \phi_f \times A_e \times F_u = 0.75 \times 3 \times 58 = 130.5$ kips → for Fracture.

$\therefore$ Yielding governs.

$\therefore \phi T_n = P_{all} = 1.2\, P_{Dead} + 1.6\, P_{Live}$

$121.5 = (1.2 \times 15) + 1.6\, P_{Live}$

$P_{LIVE} = 64.69$ kips

$\therefore$ Answer is C

(13) Refer — Shear - unstiffened beams

$$\frac{h}{t_w} = \frac{18}{0.25} = 72$$

$$\frac{417}{\sqrt{F_y}} = \frac{417}{\sqrt{50}} = 58.97$$

$$\frac{523}{\sqrt{F_y}} = \frac{523}{\sqrt{50}} = 73.96$$

$$\therefore \frac{417}{\sqrt{F_y}} \leq \frac{h}{t_w} \leq \frac{523}{\sqrt{F_y}}$$

$$\therefore \phi V_n = \phi \times (0.6 F_y) A_w \left[\frac{417}{(h/t_w)\sqrt{F_y}}\right]$$

$$= 0.9 \times 0.6 \times 50 \times (18 \times 0.25) \times \frac{417}{72 \times \sqrt{50}}$$

$$\phi V_n = 99.52 \text{ kips.}$$

$$\therefore \phi V_n = V_{all} = \frac{(1.2 W_{Dead} + 1.6 W_{Live}) . L}{2}$$

$$\therefore 99.52 = \frac{1.2 \times (2 \times 20) + 1.6 \times 20 \times W_{LIVE}}{2}$$

$$W_{LIVE} = 4.71 \text{ kips/ft}$$

Answer is B

(14) Refer - Beams - flexure - singly reinforced

$$a = \frac{A_s f_y}{0.85 f'_c \cdot b} = \frac{3 \times 40}{0.85 \times 3 \times 12} = 3.92 \text{ in.}$$

$$M_u = A_s f_y . (d - \frac{a}{2}) = 3 \times 40 \times (15 - \frac{3.92}{2})$$

$$= 1564.8 \text{ k-in} = 130.4 \text{ k-ft.}$$

$$\phi M_n \geq M_u = \left(1.2 W_{Dead} \frac{L^2}{8}\right) + \left(1.6 P_{Live} \frac{L}{4}\right)$$

$$0.9 \times 130.4 \times 10^3 \geq \left(1.2 \times \frac{5 \times 10^2}{8}\right) + 1.6 \times \frac{P_{Live} \times 10}{4}$$

$$P_{Live} \leq 29.32 \text{ kips} //$$

Answer is B

(15)

@ support

$V_u = 1.2\, V_{Dead} + 1.6\, V_{LIVE}$

$= \left(1.2 \times \frac{5 \times 10}{2}\right) + \left(1.6 \times \frac{50000}{2}\right)$

$= 40.03$ kips //

Refer – Beams → shear.

$V_c = 2 b_w . d \sqrt{f'_c} = 2 \times 12 \times 15 \times \sqrt{3000}$

$= 19.718$ kips.

$\phi V_c = 0.75 \times 19.718 = 14.788$ kips

$\frac{\phi V_c}{2} = \frac{0.75 \times 19.718}{2} = 7.394$ kips.

$V_u > \frac{\phi V_c}{2}$

∴ shear r/f is required

Minimum $A_v = \frac{s . 50 . b_w}{f_y} = \frac{12 \times 50 \times 12}{40,000} = 0.18\ in^2$

$\phi\,(V_s + V_c) \geq V_u = 40.03$.

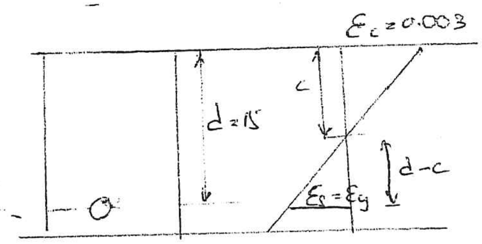
∴ $V_s + V_c \geq \frac{40.03}{0.75}$

$V_s = \frac{A_v f_y . d}{s} \geq \frac{40.03}{0.75} - 19.718$

$A_v = \frac{12}{40000 \times 15} \times 33.655 \times 10^3 = 0.67\ in^2$ // $>$ Av min.

Answer is C

(16)



$\varepsilon_y = \frac{40}{29,000} = 0.00138$

from similar triangles $\frac{\varepsilon_c}{c} = \frac{\varepsilon_y}{d-c}$

∴ $0.003(d-c) = 0.00138\,c$

$0.045 = 0.00438c$

$c = 10.27$ in //

Force Balance.

$A_s . f_y = 0.85 f'_c . \beta_1 c . b$

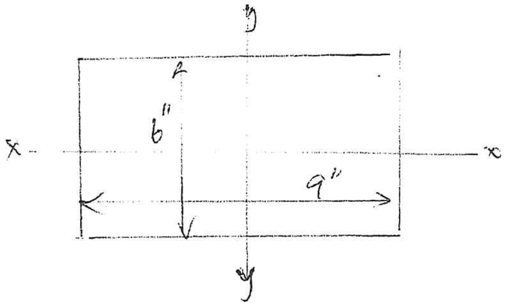
$\rho_b = \frac{A_s}{bd} = \frac{0.85 f'_c . \beta_1 . c}{f_y . d} = \frac{0.85 \times 3 \times 0.85 \times 10.27}{40 \times 15}$

$= 0.0371$ //

β_1 will be $\neq 0.85$ if $f'_c > 4000$ psi

(17)

$L_x = L_y = 9 \times 12 = 108$ in



$I_{xx} = \frac{9 \times 6^3}{12} = 162 \text{ in}^4$ — weak axis

$I_{yy} = \frac{6 \times 9^3}{12} = 364.5 \text{ in}^4$

$\therefore r_{weak} = r_{xx} = \sqrt{\frac{I_{xx}}{A}} = \sqrt{\frac{162}{9 \times 6}} = 1.73$ in

Refer table C-C2.1 for k value.

$k_{xx} = 2.10$

$\therefore \frac{kL}{r} = \frac{2.1 \times 108}{1.73} = 130.94$ (for $131 = \frac{kL}{r}$)

from AISC table 4-22 $\phi F_{cr} = \underline{13.2 \text{ ksi}}$

$\therefore$ Answer is A ?

(18)

$\left(\frac{kL}{r}\right)_{xx} = \frac{130.94}{2} = 65.47$

$r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{364.5}{9 \times 6}} = 2.6$

$\left(\frac{kL}{r}\right)_{yy} = \frac{2.1 \times 108}{2.6} = 87.2 > \left(\frac{kL}{r}\right)_{xx}$

$\therefore \left(\frac{kL}{r}\right)_{yy} = \left(\frac{kL}{r}\right)_{weak}$

$\therefore$ from table 4-22 $\phi F_c = \underline{25.9 \text{ ksi}}$

$\therefore$ Capacity $P = (\phi F_{cr}) \cdot A$

$= 25.9 \times 9 \times 6$

$= 1398.6$ kips.

$\therefore$ Answer is D