**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 in²
(B) 0.0012 in²
(C) 0.135 in²
(D) 0.18 in²

**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/in²
(B) 30 lbf/in²
(C) 35 lbf/in²
(D) 43 lbf/in²
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.

\[ f' = 3000 \text{ lb/ft}^2 \]
\[ f_y = 60,000 \text{ lb/ft}^2 \]
\[ L = 30 \text{ ft} \] (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) 90 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 in², the nominal moment capacity of each beam based on ACI strength design is most nearly

(A) 650 ft-kips  
(B) 770 ft-kips  
(C) 880 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 in², 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.

\[ f_c' = 4000 \text{ lb/ft}^2 \]
\[ f_y = 60,000 \text{ lb/ft}^2 \]

**Problem 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

**Problem 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.

\[ P_{total} \]

fixed wall
frictionless rollers
frictionless ball joint
steel compression member

member properties:
\[ I_x = 533 \text{ in}^4 \]
\[ I_y = 174 \text{ in}^4 \]
\[ A = 19.1 \text{ in}^2 \]
\[ F_y = 50 \text{ kips/in}^2 \]

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

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

(A) 2.25 in²
(B) 2.5 in²
(C) 3.0 in²
(D) 3.2 in²

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.

The yield strength is 50 kips/in². 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 $f'_c = 3000$ lb/in$^2$, $f_y = 40,000$ lb/in$^2$, and $A_s = 3$ in$^2$.

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 lb. 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 in$^2$
(B) 0.36 in$^2$
(C) 0.67 in$^2$
(D) 0.78 in$^2$

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/in$^2$
(B) 26 kips/in$^2$
(C) 29 kips/in$^2$
(D) 39 kips/in$^2$

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
1. \( \beta_1 = 0.85 \), \( \sin \theta \leq 4000 \text{ psi} \)

\[
\alpha = \frac{A_e f_y}{0.85 f'_c b} = \frac{3 \times 30}{0.85 \times 3 \times 12} = 3.92 \text{ in} \quad \rightarrow \text{refer BEAMS - PLEXURES in reference.}
\]

2. \( M_n = 0.85 \frac{1}{2} a. b. (d - \frac{a}{2}) = A_e f_y (d - \frac{a}{2}) \)

\[
= 0.85 \times 2 \times 3.9 \times 12 \times (20 - 3.92) = 3168.9 \text{ k-in} = 180.8 \text{ k-ft}
\]

3. \( M_{all} = \phi M_n \)

\[
-0.9 \times 180.8 = 162.8 \text{ k-ft}
\]

Answer is A.

Reinforcement ratio \( \rho = \frac{A_s}{bd} = \frac{3}{12 \times 20} = 0.0125 \)

\[
A_{e,min} = \left\{ \begin{array}{l}
\frac{\rho b d f'_c}{f_y} = \frac{2 \times 12 \times 20 \times 6000}{40000} = 0.986 \text{ m}^2 \\
200 \frac{b d}{f_y} = \frac{900 \times 12 \times 20}{40000} = 1.2 \text{ m}^2 \rightarrow \text{controls}
\end{array} \right.
\]

\[
A_{e,min} = 0.85 \frac{h}{f_c} b \left( \frac{h}{3} \right) = 0.85 \times \frac{3}{40} \times \frac{85 \times 12 \times (20 + 1.12 k)}{7} \rightarrow \text{check ASTM and ASCE}
\]

\[
= 5.78 \text{ m}^2
\]

1.2 m² \( < 3 \text{ m}^2 \) \( < 5.78 \text{ m}^2 \) \( \rightarrow \text{OK.} \)
\( V_u = 1.2 \, V_{dead} + 1.6 \, V_{live} \quad \rightarrow \text{refr load factors} \)
\[ = (1.2 \times 5) + (1.6 \times 15) \]
\[ = 30 \, \text{kip} \]

Nominal shear strength

\[ V_n = V_c + V_s \quad \rightarrow \text{refr REMUS - SHEAR} \]
\[ V_c = \frac{3\sqrt{f'_c}}{1} \, b \, d = 3 \times \sqrt{3000} \times 12 \times 20 \]
\[ = 26.29 \, \text{kip} \]

\[ \phi = 0.75 \, \text{for shear} \quad \rightarrow \text{refr RESISTANCE FACTORS} \]
\[ \frac{\phi V_c}{2} = 0.75 \times 26.29 = 9.86 \, \text{kip} < V_u \]

Shear reinforcement is required

\[ \phi V_c = 0.75 \times 26.29 = 19.72 \, \text{kip} < V_u \]

\[ V_s = \frac{V_u - V_c}{\phi} = \frac{30}{0.75} - 26.29 = 13.71 \, \text{kip} \]

\[ A_v = \frac{s \cdot V_s}{f_y \cdot d} = \frac{12 \times 12.71}{40 \times 20} \]

controls

\[ A_v_{min} = \left\{ \begin{array}{l}
\frac{s \times 50 \times b_c}{f_y} = 0.18 \\
\frac{s \times 0.75 \times b_c \sqrt{f'_c}}{f_y} = 0.148
\end{array} \right. \]

\[ A_v > A_{min} \quad \text{use} \quad A_v = 0.2 \, \text{in}^2 \]

Answer is D?
\[ P_u = 1.2D + 1.6L = (1.2 \times 900) + (1.6 \times 100) = 400 \text{ kips} \]

Soil pressure \( q_u = \frac{P_u}{A_g} = \frac{400}{8 \times 8} = 6.25 \text{ kips/ft}^2 \)

Loaded area for beam shear
\[ V_{bs} = \frac{V_u}{bd} = \frac{(6.25 \times 8 \times 8)}{12} = 1.14 \text{ kips/ft}^2 \]

\[ f_{bs} = 7.89 \text{ psi} \]

\[ V_{pc} = \frac{V_u}{bd} = \frac{6.25 \times (\frac{8 \times 8}{4 \times (18+33)} - \frac{(18+33)^2}{104})}{12} = 6.14 \text{ kips/ft}^2 = 42.65 \text{ psi} \]

Punching shear criterion.
Answer is D.
\( a = \frac{A_y}{0.85 \frac{f_c}{f_y}} = \frac{6 \times 60}{0.85 \times 2 \times 48} \)

\[ a = 2.94 \text{ in} < h_y = 3 \text{ in} \]

Answer is D.
Answer is (D)

\[ \frac{8}{3.24} = 0.405 \text{ in} \leq \text{use #6 hex} \]

The minimum \( A_5 = 0.24 \)

\[ A_5 \leq 0.24 \]

\[ 0.19 \leq 0.24 \]

\[ 56.64 \text{ in} \leq 13.78 \]

\[ \phi = 0.8 \phi \left( 0.85 \right)^{ \left( 4 - A_5 \right) + A_5} \]

\[ \phi = 3.24 \text{ in}^2 \]

\[ A_5 = 0.0149 = 0.0149 \]

Answer is (D)

\[ \text{No. of legs} \rightarrow 6 \]

\[ A = 6.69 \text{ in}^2 = \frac{1}{1.11} \text{ in.}^2/	ext{leg} \]

\[ A_5 \leq 8.69 \text{ in}^2 \]

\[ 7.40 \text{ kips} \leq \left[ 0.77 \times 0.85 \right] 
\[ 56.64 \text{ in}^2 + 86.52 \]

\[ \phi = 0.77 \times 0.85 \text{ in}^2 \leq 0.085 \times 4 \times (A_5 - A_5^2) + A_5 \times 60 \]

\[ \phi = 0.77 \times 0.85 \text{ in}^2 \leq 0.085 \times 4 \times (A_5 - A_5^2) + A_5 \times 60 \]

\[ \phi = 0.77 \rightarrow \phi = 0.7 \]

\[ R_n = 0.7 R_n \text{ in.}^2 \text{ kips} \]

\[ R_n = 740 \text{ kips} \]

\[ (1.0 \times 1.52) + (1.6 \times 0.52) \]

\[ R_n = 1.0 \text{ in.}^2 \text{ kips} \text{ in.} \]

\[ R_n = 4.54 \text{ in}^2 \]

\[ \frac{4}{x} 0.10 \times 0.10 \text{ in}^2 = 0.0 \text{ in} \leq 0.0 \text{ in} \]

\[ h_p = 0.008 \text{ in} \leq 0.0 \text{ in} \]

Refer: Section Corrume
9. Ref to columns (AISC Table C-2.1).

\[ k_x = k_y = 0.8 \]
\[ L_x = L_y = 120 \text{ in} \]

\[ \sqrt{\frac{f_x}{A}} = \sqrt{5.28} \text{ in} \]
\[ = 2.3 m/\text{in} \]
\[ \sqrt{\frac{f_y}{A}} = \sqrt{3.02} \text{ in} \]

\[ S_R_y > S_R_x \]
\[ \therefore S.R. \text{ is generous.} \]

\[ \frac{k_L}{r} \leq 4.71 \sqrt{\frac{E}{f_y}} \quad \text{elastic} \]
\[ = 18.2 \text{ in} \]

\[ \frac{k_L}{r} > 4.71 \sqrt{\frac{E}{f_y}} \quad \text{elastic} \]

\[ r = 30 \text{ in} \]

Answer is B.

10.

\[ \phi_{Fy} = \phi (0.658 \frac{k_y}{E}) \cdot f_y \]

\[ \phi = 0.9 \quad \rightarrow \text{refer AISC table.} \]

\[ \phi_{Fy} = 0.9 \times (0.658 \frac{30}{283.04}) \times 50 \]
\[ = 41.79 \text{ kip/m}^2 \quad \rightarrow \text{can be read directly from table.} \]

\[ \therefore \text{Column capacity} = P = \phi_{Fy} \cdot A \]
\[ = 41.79 \times 19.1 \]
\[ = 798.2 \text{ kips} \]

\[ P_{\text{tot}} = P = 1.2 P_{\text{dead}} + 1.6 P_{\text{live}} \]
\[ 798.2 = (1.2 \times 7) + 1.6 P_{\text{live}} \]
\[ \therefore P_{\text{live}} = 493.6 \text{ kips} \]

Answer is B.
Ref: STEEL STRUCTURES
TENSION MEMBERS

\[ A_y = (2.05 + 3 + 2.25) \times 0.5 = 3.75 \text{ in}^2 \]

\[ b_n^{ABCD} = b - \sum \frac{d_i}{4} \]
\[ b_n^{ABEF} = b - \sum d + \frac{\sum d_i^2}{4} \]
\[ = 7.5 - (2\times0.75) + \frac{4^2}{4\times3} \]
\[ = 7.33 \text{ in} \]

\[ A_n = b_n \times t = 6 \times 0.5 = 3 \text{ in}^2 / \]
\[ A_e = 0.8 A_n \text{ for plate/dowel} \]
\[ A_e = 3 \text{ in}^2 / \]
Answer is C

(12)

\[ \phi T_n^2 \phi \times A_y \times F_y = 0.9 \times 3.75 \times 36 = 121.5 \text{ kips} \rightarrow \text{Yielding}\]
\[ \phi T_n = \phi \times A_e \times F_n = 0.75 \times 3 \times 58 = 130.5 \text{ kips} \rightarrow \text{Fracture}\]

Yielding governs.

\[ \phi T_n = P_{all} = 1.2 P_{dead} + 1.6 P_{live} \]
\[ 121.5 = (1.2 \times 15) + 1.6 P_{live} \]
\[ P_{live} = 64.69 \text{ kips} \]

Answer is C
Ref: Shear - unsharpened beams

\[ \frac{h}{l_w} = \frac{1.8}{0.25} = 7.2 \]

\[ \frac{417}{f_y} = \frac{417}{580} = 58.97 \]

\[ \frac{523}{f_y} = \frac{523}{580} = 73.96 \]

\[ \phi V_u = \phi_f (0.6 f_y) A_w \left[ \frac{417}{(h/w_o)^{1/3} f_y} \right] \]

\[ = 0.9 \times 0.6 \times 50 \times (18 \times 0.25) \times \frac{417}{72 \times 580} \]

\[ \phi V_u = 99.52 \text{ kips.} \]

\[ \phi V_u = \frac{V_{\text{all}}}{L} \left( 1.2 W_{\text{dead}} + 1.6 W_{\text{live}} \right) \]

\[ 99.52 = \frac{1.2 \times (2 \times 20) + 1.6 \times 20 \times W_{\text{live}}}{2} \]

\[ W_{\text{live}} = 4.71 \text{ kips/ft} \]

Answer is B

Ref: - Beams - Flexure - singly reinforced

\[ a = \frac{A_{f_y}}{0.85 f_y} = \frac{3 \times 40}{0.85 \times 0.25 \times 12} = 3.92 \text{ m.} \]

\[ M_u = A_{f_y} (d - a) = 3 \times 40 \times (15 - 3.92) \]

\[ = 1564.8 \text{ k-in.} \]

\[ \phi M_o \geq M_u = \left( 1.2 W_{\text{dead}} \frac{L^2}{8} + 1.6 W_{\text{live}} \frac{L}{4} \right) \]

\[ 0.9 \times 150.4 \times 10^2 \times \left( 1.2 \times 5 \times 10^2 \right) + 1.6 \times W_{\text{live}} \times 10 \frac{L}{4} \]

\[ \rho_{\text{live}} \leq 29.32 \text{ kips/ft} \]

Answer is B
\( V_u = 1.2V_{end} + 1.6V_{civi} \)
\( = \left(1.2 \times \frac{5 \times 10}{2}\right) + \left(1.6 \times \frac{50000}{2}\right) \)
\( = 40.02 \text{ kips} \)

Refer - Beams - Shear

\( V_c = 2b_w.d\sqrt{f_c'} = 2 \times 12 \times 15 \times \sqrt{3000} \)
\( = 19.718 \text{ kips} \)

\( \phi V_c = 0.75 \times 19.718 = 14.788 \text{ kips} \)

\( \phi V_c = 0.75 \times 19.718 = 7.394 \text{ kips} \)

\[ \begin{align*}
V_u & > \frac{\phi V_c}{2} \\
\Rightarrow \quad V_u & > 40.02 \text{ kips}
\end{align*} \]

Minimum \( A_v = \frac{8.50 \cdot b_w}{d_h} = \frac{12 \times 50 \times 12}{40,000} \)

\( \phi (V_e + V_c) \geq V_u = 40.02 \)

\( \Rightarrow V_e + V_c \geq \frac{40.02}{0.75} \)

\( V_e = A_v \frac{f_y}{d} = 40.02 \times \frac{0.75}{15} - 19.718 \)

\( A_v = \frac{12 \times 33.655 \times 10^3}{40,000 \times 15} = 0.67 \text{ m}^2 \)

Answer \( 18 \text{ C} \)

[Diagram]

\( E_y = \frac{40}{29,000} = 0.00138 \text{ kips/in}^2 \)

From similar triangles, \( \frac{E_c}{C} = \frac{E_y}{d-c} \)

\( 0.003(d-c) = 0.00138c \)

\( 0.045 = 0.00438c \)

\( c = 10.27 \text{ in} \)

\( \beta_i \) will be \( \neq 0.85 \)

If \( f_c' > 4000 \text{ psi} \)
(17)

\[ I_x = I_y = 9 \times 12 = 108 \text{ in}^4 \]

\[ I_{xx} = \frac{9 \times 6^3}{12} = 162 \text{ in}^4 \quad \text{--- weak axis} \]

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

\[ t_{weak} = \sqrt{\frac{5A}{I_{xx}}} = \sqrt{\frac{162}{9 \times 6}} = 1.73 \text{ in} \]

Refer to Table C-2.1 for \( k \) value.

\[ k_{xx} = 2.10 \]

\[ \frac{k_L}{r} = \frac{2.1 \times 10^8}{1.73} = 120.94 \]

From AISC Table 4-22, \( \phi_{Fe} = 13.2 \text{ ksi} \) (\( \psi = 131 = \frac{k_L}{r} \))

(18)

\[ (\frac{k_L}{r})_{xx} = \frac{120.94}{2} = 60.47 \]

\[ x_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{364.5}{9 \times 6}} = 9.6 \]

\[ (\frac{k_L}{r})_{yy} = \frac{9.1 \times 10^8}{26} = 351.2 \quad > (\frac{k_L}{r})_{xx} \]

\[ \therefore (\frac{k_L}{r})_{yy} = (\frac{k_L}{r})_{weak} \]

\[ \phi_{Fe} = 25.9 \text{ ksi} \]

From table 4-22, \( \phi_{Fe} = 25.9 \text{ ksi} \)

\[ \therefore \quad \text{Capacity} \quad P = (\phi_{Fe}) \cdot A = 25.9 \times 9 \times 6 \]

\[ = 1398.6 \text{ kips} \]

\[ \therefore \quad \text{Answer is D} \]