A 10-foot-long beam with the cross-section shown is subjected to a uniformly-distributed service dead load of 5 lb/ft and a concentrated service live load at midspan. Taking \( f_c' = 3000 \) psi, \( f_y = 40000 \) psi, what is most nearly the maximum allowable live load using LRFD?

A. 23000 lb  
B. 29000 lb  
C. 35000 lb  
D. 50000 lb

\[
\begin{align*}
M_D &= \frac{W_D L^2}{8} = \frac{(5 \text{ lb/ft})(10 \text{ ft})^2}{8} = 62.5 \text{ ft-lb} \\
M_L &= \frac{P_L L}{4} = \frac{P_L (10\text{ ft})}{4} = 2.5 P_L
\end{align*}
\]

Find \( M_V \)  
\[
M_V = 1.2 M_D + 1.6 M_L = 1.2 (62.5 \text{ ft-lb}) + 1.6 (2.5 P_L)
\]

Find \( M_n \)  
\[
M_n = A_s f_y (d - \frac{d'}{2}) = (3 \text{ in}^2) (40000 \frac{\text{lb}}{\text{in}^2}) (15\text{ in} - \frac{3.92}{2}) = 156480 \text{ lb-in}
\]

\[ M_n = 156480 \text{ lb-in} (\frac{6}{12\text{ in}}) = 130400 \text{ ft-lb} = M_n \]

Find \( \beta_i \)  
\[
\beta_i = 0.85 \geq 0.85 - 0.05 (\frac{f_c' - 40000}{1000}) \geq 0.65
\]

Note: If greater than 0.85 use 0.85  
\[
\beta_i = 0.25 \geq 0.9 \geq 0.65 = 0.85
\]

Find \( C \)  
\[
C = \frac{f_c'}{\beta_i} = \frac{3.92 \text{ in}}{0.85} = 4.6118 \text{ in}
\]

Find \( P_L \)  
\[
\phi M_n \geq M_V  
\]
\[
0.9 (130400 \text{ ft-lb}) = 75 \text{ ft-lb} + (4 \text{ ft}) P_L
\]

\[
P_L = 29350 \text{ lb}
\]
For the singly-reinforced concrete beam shown below (\(f'_c = 4000 \text{ psi, } f_y = 60000 \text{ psi}\)), what is most nearly the ultimate shear strength, \(\phi V_n\)?

A. 83.6 kip  
B. 94.8 kip  
C. 145.6 kip  
D. 300.7 kip

\[
\begin{align*}
\text{Page 155:} & \quad V_c = 2bd\sqrt{f'_c} = 2(18 \text{ in})(33 \text{ in})\sqrt{4000 \text{ psi}} \\
& = 75135.7 \text{ lb} \\
\text{Page 155:} & \quad V_s = \frac{A_v f_y d}{S} = \frac{0.22 \text{ in}(60000 \text{ lb/ft}^2)(33 \text{ in})}{12 \text{ in}} \\
& = 36300.1 \text{ lb} \\
\text{Page 155:} & \quad V_n = V_c + V_s = 75136 + 36300 = 111436 \text{ lb} \\
\text{Page 155:} & \quad \phi = 0.75 \text{ FOR SHEAR MEMBERS} \\
\text{Page 155:} & \quad V_u = \phi V_n = 0.75(111436 \text{ lb}) = 83577.1 \text{ lb}
\end{align*}
\]
For the short column shown below (f'_c = 4000 psi, f_y = 60000 psi), what is most nearly the ultimate axial strength, \( \phi P_n \)? Tie reinforcements are utilized.

A. 68 kip  \( f'_c = 4000 \) psi  
B. 254 kip  \( f_y = 60000 \) psi  
C. 501 kip  \( A_g = 16 \text{ in} (16 \text{ in}) = 256 \text{ in}^2 \)  
D. 688 kip  

\[ \phi = 0.65 \]

\[ \phi P_n = 0.80 \cdot \phi \left[ 0.85 f'_c (A_g - A_{st}) + A_{st} f_y \right] \]

\[ \phi P_n = 0.80 \cdot 0.65 \left[ 0.85 (4000 \text{ psi}) (256 \text{ in}^2 - 8 \text{ in}^2) + 8 \text{ in} (60000 \text{ psi}) \right] \]

\[ \phi P_n = 6880.64 \text{ kip} = 688 \text{ kip} \]
From the choices below, what is the lightest wide-flange shape ($F_y = 50$ ksi) available that can resist a service dead load of 125 kips and a service live load of 200 kips (using LRFD)? For this column, take $K_x = K_y = 1.0$ and $L = 15$ feet.

A. W12×53
B. W12×50
C. W12×45
D. W12×40

\[ P_0 = 1.2 P_d + 1.6 P_l = 1.2(125 \text{ kip}) + 1.6(200 \text{ kip}) = 470 \text{ kip} = \frac{\phi P_a}{\phi P_a} \]

\[ P_g \text{ AISC TABLE} \]
\[ \phi = 0.9 \]
\[ \text{EFFECTIVE LENGTH (Y-AXIS)} \]
\[ L = KL = (1)(15 \text{ ft}) = 15 \text{ ft} \]
\[ \text{PLOT } P_0 = 470 \text{ (EXCEEDS } P_0) \]

\[ \therefore \text{ W12×53} \]

Note:
Weak axis governs
Since column is square
There is no weak axis
A W24 x 55 (F_y = 50 ksi) is to be used as a simply-supported beam (span length = 12 feet) to support a uniformly distributed load. Taking C_b = 1.0, what is most nearly the maximum design flexural capacity (LRFD) of the beam if only the ends of the beam are laterally supported?

A. 503 ft-kip  
B. 344 ft-kip  
C. 299 ft-kip  
D. 150 ft-kip

\[ M_a = C_b \left[ M_p - \left( M_p - 0.7 F_y S_x \right) \frac{(L_b - L_p)}{(L_r - L_p)} \right] \leq M_p \]

**Page 157 Lateral Torsion Buckling \( \phi M_a \)**

**Page 160 AISC Table 3-2**

\[ M_r = 0.7 F_y S_x, \quad BF = \frac{M_p - M_r}{L_r - L_p} \]

\[ M_a = \left[ M_p - \left( M_p - M_r \right) \frac{(L_b - L_p)}{(L_r - L_p)} \right] = M_p - \left( M_p - M_r \right) \frac{(L_b - L_p)}{(L_r - L_p)} \]

\[ \phi M_a = \phi M_p - \phi BF \left( L_b - L_p \right) = 503 - 22.2 \left( 12 - 9.3 \right) \]

\[ M_a = 542 \text{ kip-ft} \]
The bolt layout for an L6×4×3/8 of A36 (F_y = 36 ksi, F_u = 58 ksi) steel is shown below. 5 bolts that are 3/4 inches in diameter are used. What is most nearly the design axial capacity, P_n, of the member? For the purposes of this problem, you may neglect block shear rupture and bolt capacity.

\[ A_g = 3.61 \text{ in}^2 \]
\[ \bar{x} = 0.933 \text{ in} \]

- **A.** 14 kip
- **B.** 83 kip
- **C.** 117 kip
- **D.** 169 kip

**PS 15k Design Strength: Smaller of the Yielding and Fracture Limits.**

**Yielding**
\[ \phi P_n = \phi_y F_y A_g = 0.9 \left( \frac{14}{\text{kip}} \right) \left( 3.61 \text{ in}^2 \right) = 116.96 \text{ kip} \]

**Fracture**
\[ \phi P_n = \phi_f F_u A_e = A_e = N A_n \]
A floor system for use in an office building utilizes 45-foot-long floor beams spaced at 9 feet. The floor will be subjected to a service dead load of 50 psf and a unreduced service live load of 80 psf. Using LRFD load combinations, what is most nearly the maximum factored bending moment on a given floor beam?

- A. 126 ft-kip
- B. 363 ft-kip
- C. 428 ft-kip
- D. 800 ft-kip

**BEAMS SPACED AT 9 FEET:**

\[ L = 45 \text{ ft} \]

**LIVE LOAD REDUCTION**

\[ K_{LL} = 2 \text{ (BEAMS)} \]

\[ A_T = \frac{\omega_T \cdot L}{9 \text{ ft}} = 9 \text{ ft}(15 \text{ ft}) = 135 \text{ ft}^2 \]

\[ L_{red} = L_n \left( 0.25 + \frac{15}{\sqrt{K_{LL} A_T}} \right) \geq 0.4 L_n \]

\[ L = 80 \text{ psf} \left( 0.25 + \frac{15}{\sqrt{2}(135)} \right) = 61.2 \text{ psf} \]

\[ U = 1.2 D + 1.6 L = 1.2(50 \text{ psf}) + 1.6(61.2 \text{ psf}) = 158 \text{ psf} \]

\[ M_{max} = \frac{U L^2}{4} = \frac{158 \frac{16}{45} (45 \text{ ft})^2}{4} = 720 \text{ ft-kip} \]
According to ACI 318-02, find the value of $\phi$ that should be used in computing the design moment strength ($\phi M_n$) for the beam section shown.

- $f_c' = 4$ ksi
- $f_y = 60$ ksi
- $A_s = 5.08$ in$^2$
- 4-No. 10 bars (tension)

\[ \alpha = \frac{A_s f_y}{0.85 f_c' b} = \frac{5.08 \text{ in}^2 (40 \text{ ksi})}{0.85 (4 \text{ in})(12 \text{ in})} = 7.47 \text{ in} \]

\[ \alpha = \frac{\beta_1 c}{\beta_3} \]

\[ \beta_1 = 0.85 \geq 0.85 - 0.05 \left( \frac{f_c' - 4000}{1000} \right) = 0.65 \]

\[ \beta_3 = 0.85 - 0.05 \left( \frac{4000 - 43000}{1000} \right) = 0.25 \]

\[ c = \frac{a}{\beta_3} = \frac{7.47 \text{ in}}{0.85} = 8.79 \text{ in} \]

\[ e_t = 0.003 \left( c - c \right) = 0.003 \left( 21.5 \text{ in} - 8.79 \text{ in} \right) = 0.0043 \]

\[ \therefore \phi = 0.48 + 83 e_t = 0.84 \]
Find the minimum adequate width (b) for the beam.

\[ M_u = 648 \text{ kips-ft} \]
\[ f'_c = 4000 \text{ psi} \]
\[ f_y = 60,000 \text{ psi} \]
\[ \phi = 0.9 \]
8-No. 8 Bars (tension)

\[ d = 30 \text{ in} - 2.5 \text{ in} = \frac{3}{2} \text{ in} = 1.5 \text{ in} \]

\[ A_t = 8 (0.79) = 6.32 \text{ in}^2 \]

\[ \phi M_u \geq M_u \]

\[ M_u = \phi M_n = \phi A_t f_y \left( d - \frac{a}{2} \right) \]

\[ 648 \text{ kip-ft} \left( \frac{12 \text{ in}^2}{1 \text{ ft}^2} \right) = (0.4) (6.32 \text{ in}^2) (60 \text{ kip/ft}^2) \left( 2.5 \text{ in} - \frac{9}{2} \right) \]

\[ a = 17.43 \text{ in} \]

\[ a = \frac{A_s f_y}{0.85 f'_c b} \]

\[ = \frac{6.32 \text{ in}^2 (60 \text{ kip/ft}^2)}{0.85 (4 \text{ kip/ft}^2) (b)} \]

\[ b = 15 \text{ in} \]
According to American Concrete Institute, what is the allowable moment capacity of the beam.

\[ f_c' = 3,000 \text{ psi} \]
\[ f_y = 40,000 \text{ psi} \]
\[ A_s = 3 \text{ in}^2 \]
3-No. 9 bars (tension)

\[
\alpha = \frac{A_s f_y}{0.85 f_c' b} = \frac{3 \text{ in}^2 (40 \text{ ksi})}{0.85 (3 \text{ ksi})(12 \text{ in})}
\]
\[
\alpha = 3.92 \text{ in}
\]

\[
\phi M_n \geq M_u
\]

\[
\phi M_n = \phi A_s f_y (d - \frac{a}{2}) = M_u
\]

\[
\beta_i = 0.85 - 0.05 \left( \frac{f_c' - 4000}{1000} \right) = 0.85 + 0.05 = 0.9
\]

\[
C = \frac{\alpha}{\beta_i} = \frac{3.92 \text{ in}}{0.9} = 4.36 \text{ in}
\]

\[
\epsilon_f = 0.003 (d - C) = 0.003 (20 \text{ in} - 4.3 \text{ in}) = 0.011 \quad \therefore \phi = 0.9
\]

It is an ideal condition to have beams underreinforced. Underreinforced beams allow the steel to fail before the concrete, which provides more time for evaluation in the event of a failure.

\[
M_u = \phi M_n = \phi A_s f_y (d - \frac{a}{2}) = 0.91 (3 \text{ in}^2) (90 \text{ ksi})(20 - \frac{3.92}{2})
\]

\[
M_u = 1418 \text{ kip in} \left( \frac{34}{12 \text{ in}} \right) = 162 \text{ kip ft}
\]
STIRUP SPACING

\[ f' = 20 \text{,000 psi} \]
\[ f_y = 40 \text{,000 psi} \]
\[ A_s = 3.\text{in}^2 \]

\[ V_b = 5 \text{ kips} \]
\[ V_L = 15 \text{ kips} \]
\[ \phi = 0.75 \]

MN SHEAR REINFORCE FOR STIRUP SPACING OF 12"?

**pg 152**

\[ V_v = 1.2V_b + 1.6V_L = 1.2(5 \text{ kips}) + 1.6(15 \text{ kips}) = 30 \text{ kips} \]

**pg 155**

\[ V_L = 2b_wc\sqrt{f_c} = 2(12\text{ in})(20\text{ in})\sqrt{5000} = 262.9 \text{ lb} = 2.62 \text{ kips} \]

\[ \frac{\phi V_v}{2} = 0.75(2.62 \text{ kips}) = 1.966 \text{ kips} < V_v \quad \text{STIRREPS REQUIRED} \]

\[ \phi V_v = 0.75(26.3 \text{ kips}) = 19.73 \text{ kips} < V_v \quad \text{COLUMN 2} \]

\[ \frac{V_{n}}{\phi} = \frac{V_{w}}{\phi} = V_c + V_s \]

\[ \frac{30 \text{ kips}}{0.75} = 26.3 \text{ kip} + V_s \quad V_s = 13.7 \text{ kip} \]

\[ s = \frac{A_v f_y d}{V_s} = \frac{2A_s (40 \text{ kips}) (20\text{ in})}{13.7 \text{ kip}} \]

\[ A_s = 0.21 \text{ in}^2 \]
Determine the design moment capacity of the beam shown:
\( f_c' = 4000 \text{ psi} \)
\( f_y = 60,000 \text{ psi} \)

**Doubly Reinforced**

**NOT ON EXAM**

Pg 37-5 LINDEBURG REVIEW MANUAL

**IF COMPRESSION STEEL YIELDS**

\[
A_s - A_s' \geq \frac{0.85 f_c' d' b}{f_y} \left( \frac{87000}{87000 - f_y} \right)
\]

\[
A_s = \frac{0.85 f_c' B_1 b}{f_y} \left( \frac{3 d_2}{7} \right) + A_s'
\]

\[
a = \frac{(A_s - A_s') f_y}{0.85 f_c' b}
\]

\[
M_n = f_y \left[ (A_s - A_s')(d - \frac{d_2}{2}) + A_s'(d - d') \right]
\]

**IF COMPRESSION STEEL DOES NOT YIELD**

\[
M_n = 0.85 B_1 f_c' \frac{f_y}{f_c} \left( d - \frac{B_1 c}{2} \right) + A_s' \left( \frac{c - d'}{c} \right)(d - d') 87000
\]
EFFECTIVE TO FLANGE WIDTH \( (b_e) \)

\[ b_e = \frac{f'_c}{f_y} \]

- \( f'_c = 3000 \text{ psi} \)
- \( f_y = 60000 \text{ psi} \)
- \( L = 30' \)
- \( A_s = 7.25 \text{ in}^2 \)

\[ 48'' \]
Find the available design strength for a W12x50 column.
Pinned-Pinned - FIXED-FIXED

P = 200 kips
L = 22 feet

\[ P_{\text{允许}} = 182 \text{ kips} \]

\[ KL = (1)(22 \text{ ft}) = 22 \text{ ft} \]

\[ \phi P_n = 182 \text{ kips} \]
SPIRAL COLUMN, FIND % STEEL PER GROSS AREA OF COLUMN.

\[ P_u = 600 \text{ kips} \]
\[ e = 6'' \]
\[ f'_c = 4000 \text{ psi} \]
\[ f_y = 60000 \text{ psi} \]
\[ \phi = 0.7 \text{ FOR SPIRAL COLUMNS} \]

Plot \( k_n \) and \( R_n \), use next biggest ring to obtain reinforcement ratio \( (S_y) \)

\[ S_y = 0.05 = 5\% \text{ STEEL} \]

\[ A_{st} = S_y \cdot A_{st} = \frac{A_{st}}{A_s} \]

\[ A_{st} = 0.05 \cdot (314 \text{ in}^2) = 15.7 \text{ in}^2 \]
1. The span length and cross section of a reinforced concrete beam are shown. The beam is underreinforced. The concrete and reinforcing steel properties are $f'_c = 3000 \text{ lbf/in}^2$, $f_y = 40,000 \text{ lbf/in}^2$, and $A_s = 3 \text{ in}^2$.

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  \hspace{1cm} M_u = 1.2 M_d + 1.6 M_L
(B) 29,000 lbf
(C) 35,000 lbf
(D) 50,000 lbf

\[ \alpha = \frac{A_s}{0.85 \frac{f_y}{f'_c}} = \frac{(3 \text{ in})(11,000 \text{ psi})}{0.85(3000)(12 \text{ in})} = 3.92 \text{ in} \]

\[ \beta = 0.65 \geq 0.85 - 0.05 \left( \frac{f'_c}{1000} \right) = 0.85 \]

\[ \Phi M_n = M_u \]

\[ M_n = A_s f_y (d - \frac{a}{2}) = 3.92^2 (40,000 \text{ psi})(15 \text{ in} - \frac{3.92}{2}) = 1.564 \text{ ft-16} \]

\[ M_n = 130400 \text{ ft-16} \times \frac{12}{12} = 130400 \text{ ft-16} \]

\[ (0.4)(130400 \text{ ft-16}) = 15A_{16} + 4P_L \]

\[ P_L = 29336.25 \text{ ft-16} \]
2. A floor system consists of ten 30 ft long reinforced concrete beams and a continuous 5 in deck slab. (A typical section is shown for the deck and two of the beams.) Assume the beams are underreinforced.

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

(A) 36 in
(B) 50 in
(C) 60 in
(D) 90 in

\[
\begin{align*}
\text{be(smallest)} &= \begin{cases} 
\frac{1}{4} \times \text{(SPAN LENGTH)} = \frac{30 \times 12}{4} = 7.5 \times 12 = 90 \text{ in} \\
bw + 16h &= 90 \text{ in} \\
\text{BEAM CENTER LINE} &= 50 \text{ in SPACING}
\end{cases}
\end{align*}
\]
3. The cross section of a reinforced concrete beam with tension reinforcement is shown. Assume that the beam is underreinforced.

\[ V_d = 5 \text{ kips} \]
\[ V_l = 15 \text{ kips} \]
\[ s = 8 \text{ in} \]

\[ f'_c = 3000 \text{ lbf/in}^2 \]
\[ f_y = 40,000 \text{ lbf/in}^2 \]
\[ A_s = 3 \text{ in}^2 \quad \text{[three no. 9 bars]} \]
\[ A'_s = 1 \text{ in}^2 \]

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 8 in based on ACI strength design is most nearly:

(A) 0.10 in\(^2\)
(B) 0.12 in\(^2\)
(C) 0.14 in\(^2\)
(D) 0.18 in\(^2\)

\[ V_s = \frac{A_v f_y d}{s} \]

\[ 13.7 \text{ kips} = \frac{A_v (40 \frac{\text{kips}}{\text{in}^2})(20 \text{ in})}{8 \text{ in}} \]

\[ A_v = 0.137 \text{ in}^2 \]
The span length and cross section of a reinforced concrete beam are shown. The beam is underreinforced. The concrete and reinforcing steel properties are $f'_c = 2500$ lb/in$^2$, $f_y = 50,000$ lb/in$^2$, and $A_s = 3.8$ in$^2$.

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) 0.17 in$^2$

(B) 0.23 in$^2$

(C) 0.38 in$^2$

(D) 0.78 in$^2$

$V_u = 1.2V_0 + 1.1V'_s = 1.2\left(1.5\frac{P}{2}\right) + 1.6\frac{P}{2} = 1.2(12)(150) + 1.6(50000)$

$V_u = 40022$ lbf

$V_n = \frac{V_u}{\phi} = V_s + V'_s = 40022 lbf = 25200 lbf + 3829 lbf = 28229 lbf$

$V_s = \frac{A_V f_Y d}{s} = \frac{28229 lbf}{12 in}$

$A_V = 0.336$ in$^2$
5. A floor system consists of 30 reinforced concrete beams and a continuous 4 in deck slab. (A typical section is shown for the deck and two of the beams.) Assume the beams are underreinforced.

\[ A_s = 6 \text{ in}^2 \]

\[ f'_c = 2800 \text{ lbf/in}^2 \]
\[ f_y = 42,000 \text{ lbf/in}^2 \]
\[ L = 30 \text{ ft} \quad \text{[simple span length]} \]

Assume the effective flange width for this beam is 40 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) 520 ft-kips
- (D) 650 ft-kips

\[ \alpha = \frac{A_s f_y}{0.85 \sqrt{f'_c b_e}} = \frac{6 \times (42,000 \text{ lbf/in}^2)}{0.85 \sqrt{2800 \text{ lbf/in}^2 \times 40 \text{ in}}} \approx 3.6 \text{ in} \]

\[ M_a = A_s f_y \left( d - \frac{d^2}{2} \right) = 6 \times (42,000 \text{ lbf/in}^2) \left( 26 \text{ in} - \frac{2 \times 2800 \text{ lbf/in}^2}{2} \right) = 542,140 \text{ in-lbf} \]

\[ M_b = 542,140 \text{ lbf-in} \left( \frac{b_e}{12 \text{ in}} \right) = 493,531 \text{ ft-lbf} \]

\[ \alpha = 0.85 \sqrt{\frac{f'_c}{f_y}} \approx 0.85 \sqrt{42,000 \text{ lbf/in}^2} = 578 \text{ ft-lbf} \]
6. The span length and cross section of a reinforced concrete beam are shown. The beam is underreinforced. The concrete and reinforcing steel properties are $f'_c = 3100 \text{ lbf/in}^2$, $f_y = 35,000 \text{ lbf/in}^2$, and $A_s = 2.5 \text{ in}^2$. 

![Beam Diagram]

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

(A) 0.037
(B) 0.046
(C) 0.051
(D) 0.058
7. 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.

\[
\frac{bc}{d} = 4 \text{ in}
\]

\[
A_s = 7.25 \text{ in}^2
\]

\[
f'_c = 3000 \text{ lbf/in}^2
\]

\[
f_y = 60,000 \text{ lbf/in}^2
\]

\[
L = 30 \text{ ft} \quad \text{[simple span length]}
\]

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

\[
M_n = A_s f_y \left( d - \frac{d}{2} \right)
\]

\[
M_n = 7.25 \text{ in}^2 \left( 60,000 \text{ lbf/in}^2 \right) \left( 23 - \frac{3.6}{2} \right) = 9222 \text{ in-kips}
\]

\[
M_n = 9222 \text{ in-kips} \left( \frac{68}{12 \text{ in}} \right) = 768.5 \text{ ft-kips}
\]
8. The cross section of a reinforced concrete beam with compression reinforcement is shown.

\[ f'_c = 3000 \text{ lbf/in}^2 \]
\[ f_y = 40,000 \text{ lbf/in}^2 \]
\[ A_s = 3 \text{ in}^2 \]
\[ A'_s = 1 \text{ in}^2 \]

The nominal moment capacity of the beam is most nearly

(A) 130 ft-kips
(B) 150 ft-kips
(C) 170 ft-kips
(D) 190 ft-kips
A monolithic slab-beam floor system is supported on a column grid of 18 ft on centers as shown. The dimensions of the cross section for the beams running in the north-south direction have been determined.

What is most nearly the effective flange width?

(A) 45 in

(B) 54 in

(C) 63 in

(D) 72 in
1. For the short, concentrically loaded round spiral column shown, the applied axial dead load is 150 kips, and the applied axial live load is 350 kips.

\[ f'_e = 4000 \text{ lbf/in}^2 \]
\[ f_y = 60,000 \text{ lbf/in}^2 \]

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
2. For the short, concentrically loaded square tied column shown, the applied axial dead load is 150 kips, and the applied axial live load is 250 kips.

\[
P_d = 150 \text{ kips} \\
P_l = 250 \text{ kips} \\
\phi = 0.65 \\

A_g = 18 \text{ in}(18 \text{ in}) = 324 \text{ in}^2
\]

\[
P_0 = 1.2P_d + 1.6P_l = 1.2(150) + 1.6(250) = 580 \text{ kips}
\]

\[
\phi P_n = P_0 
\]

\[
\phi = 0.8 \phi [0.85 \frac{f'_c}{f_y} (A_g - A_{st}) + A_{st} f_y]
\]

\[
580 \text{ kips} = 0.8 \times (0.65) [0.85(4 \text{ ksi})(324 \text{ in}^2 - A_{st}) + A_{st}(60 \text{ ksi})]
\]

\[
A_{st} = 3.24 \text{ in}^2 \quad \text{(LESS THAN MIN VALUE OF } \phi \text{)}
\]

Use \( A_{st} = 3.24 \text{ in}^2 \)

Area of single bar = \( \frac{3.24 \text{ in}^2}{8 \text{ bars}} = 0.405 \text{ in}^2 \)

Use ASTM reinforcement chart (B15Y) to determine

\# 6 bars
3. A reinforced concrete tied column is subjected to a design axial compression force of 1090 kips that is concentrically applied. Slenderness effects are negligible, and the column is to be designed using ACI 318. Given a specified compressive strength of 5000 psi, grade 60 rebars, and a specified longitudinal steel ratio of 0.02, what is most nearly the width of the sides of the smallest square column that will support the load?

(A) 12 in  
(B) 16 in  
(C) 20 in  
(D) 24 in

**FE INTERACTION CHART**

**ASSUME** $f_y = 60000$ psi 
**BECAUSE** GRADE 60 REBAR IS USED

\[
P = 1090 \text{ kips} \\
f'_c = 5000 \text{ psi} \\
\phi = 0.02 \\
\phi = 0.65 \\
f_y = 60 \text{ psi} \\
A_{st} = 0.02 \frac{A_g}{h^2}
\]

\[
\phi P_n = 0.8 \phi \left[ 0.65 f'_c (A_g - A_{st}) + A_{st} f_y \right] \\
1090 \text{ kip} = 0.8(0.65) \left[ 0.65 \left( \frac{5000 \text{ psi}}{f'_c} \right) \left( h^2 - 0.02 h^2 \right) + 0.02 h^2 \left( \frac{60 \text{ psi}}{f'_c} \right) \right]
\]

\[
h = 19.77 \text{ in}
\]
4. A 16 in (gross dimension) square, tied column must carry 220 kip dead and 250 kip live loads. The dead load includes the column self-weight. The column is not exposed to any moments. Sidesway is prevented at the top, and slenderness effects are to be disregarded. The concrete compressive strength is 4000 lbf/in$^2$, and the steel tensile yield strength is 60,000 lbf/in$^2$. The longitudinal reinforcement of this column is most nearly

\[ \text{(A) } 0.028 \]
\[ \text{(B) } 0.061 \]
\[ \text{(C) } 0.092 \]
\[ \text{(D) } 0.11 \]

\[ A_g = 16\text{in}(16\text{in}) = 256 \text{ in}^2 \]

\[ P_u = 1.2 P_d + 1.6 P_L = 1.2(220 \text{ kip}) + 1.6(250 \text{ kip}) = 664 \text{ kip} \]

\[ \phi P_n = 0.85 \phi \left[ 0.85 f'_c (A_g - A_{st}) + A_{st} f_y \right] \]

\[ 664 = 0.8 \cdot 0.65 \left[ 0.85 \left( \frac{4 \text{ kip}}{\text{in}^2} \right)(256 \text{ in}^2 - A_{st}) + A_{st} \left( \frac{60 \text{ kip}}{\text{in}^2} \right) \right] \]

\[ A_{st} = 7.182 \text{ in}^2 \]

\[ f = \frac{A_{st}}{A_g} = \frac{7.182 \text{ in}^2}{256 \text{ in}^2} \approx 0.028 \]
5. An 18 in square tied column is reinforced with 12 no. 9 grade 60 bars and has a concrete compressive strength of 4000 lbf/in\(^2\). The column, which is braced against sidesway, has an unsupported height of 9 ft and supports axial load only without end moments. What is most nearly the design axial load capacity?

- (A) 930 kips
- (B) 970 kips
- (C) 1800 kips
- (D) 1900 kips

\[ \phi P_n = 0.8 \phi \left[ 0.85 \frac{f_y}{f_c} (A_g - A_{st}) + A_{st} f_y \right] \]

\[ = 0.8 (0.65) \left[ 0.85 \left(4 \frac{1000}{\text{kip}}\right)(324 \text{ in}^2 - 12 \text{ in}^2) + 12 \text{ in}^2 \left(60 \frac{\text{kip}}{\text{in}^2}\right) \right] \]

\[ \phi P_n = 1925 \text{ kips} \]
6. The short spiral column shown uses eight no. 8 bars. Assume the loading has a low eccentricity. $f'_c = 3500 \text{ lbf/in}^2$, and $f_y = 40,000 \text{ lbf/in}^2$.

The design strength is most nearly

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

$\phi P_n = 0.8 \phi \left[ 0.85 (3500)(360 - 6.32) + 6 \times 2 (40) \right] = 764 \text{ kips}$
1. A 25 ft long steel beam is loaded uniformly (live) at 4 kips/ft. Loading due to self-weight is negligible, and there is adequate lateral support provided to the beam. The required plastic section modulus for a W12 shape using grade-50 steel is most nearly

(A) 60 in$^3$
(B) 95 in$^3$
(C) 110 in$^3$
(D) 120 in$^3$

\[ M_0 = 1.6 \left( \frac{w \cdot L^2}{6} \right) = 1.6 \left( \frac{4}{6} \right) \left( \frac{25}{8} \right)^2 = 500 \text{ ft-kip} \]

\[ M_n = M_0 \cdot \frac{1}{1.25} = 666.7 \text{ in-kip} \]

\[ \phi = \frac{M_n}{M_0} = 55.5 \text{ kip (12 in)} = 666.7 \text{ in-kip} \]

\[ M_n = F_y \cdot Z_x \]

\[ 500 \text{ ft-kip (12 in)} = 50 \frac{\text{kip}}{\text{in}^2} \cdot Z_x \]

\[ Z_x = 120 \text{ in}^3 \]
2. A W-shaped beam has a warping constant of 3450 in\(^6\). The moment of inertia about the strong axis is 45.1 in\(^4\), and its elastic section modulus about the weak axis is 99.1 in\(^3\). The effective radius of gyration of the compression flange is most nearly

(A) 2.0 in  
(B) 3.0 in  
(C) 4.0 in  
(D) 5.0 in
3. An I-shaped steel beam is built up with 0.300 in thick plates. The material's yield strength is 50 ksi. If the height of the web is 18 in, the available shear strength is most nearly

(A) 140 kips
(B) 150 kips
(C) 180 kips
(D) 190 kips
2. A long column member has one end built-in and the other end pinned. The column is loaded in compression evenly until buckling occurs. Which statement about the column after buckling is true?

(A) The column experiences maximum deflection on its midpoint.

(B) The maximum deflection point is closer to the pinned end than the built-in end.

(C) The deflection curve is S-shaped.

(D) The column experiences no deflection under buckling.
1. 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. Design (not theoretical) effective lengths are to be used.

\[ I = I_x + I_y = 533 \text{ in}^4 + 174 \text{ in}^4 \]
\[ I = 707 \text{ in}^4 \]

\[ \gamma = \sqrt{\frac{I_x}{A}} = \sqrt{\frac{533 \text{ in}^4}{19.1 \text{ in}^2}} = 5.283 \text{ in} \]

**EFFECTIVE SLENDERNESS RATIO**

This compression member is controlled by which type of buckling?

(A) local
(B) torsional
(C) inelastic
(D) elastic
3. A solid steel column with a fixed bottom support and free upper end is concentrically loaded. Material and geometric properties are shown.

![Diagram of a solid steel column]

The available axial compressive design stress is most nearly

(A) 13 kips/in²
(B) 18 kips/in²
(C) 29 kips/in²
(D) 39 kips/in²

**Alternate**

\[ \frac{F_y}{50,000 \text{ psi}} = 0.877 \text{ ksi} \]

\[ F_c = 0.877 \left( \frac{29,000 \text{ lb/ft}^2}{131.1} \right) = 14.6 \]

\[ \Phi F_c = 0.9(14.6) = 13.1 \]
4. A steel column is built-in at one end and free to translate and rotate at the other end. The column uses a 12 ft long W12 x 45 beam. If the yield strength of the steel is 50 kips/in², the available design stress in the column is most nearly:

(A) 7.3 kips/in²
(B) 9.0 kips/in²
(C) 9.7 kips/in²
(D) 10 kips/in²

---

TABLE 1-1: W-SHAPE DIMENSIONS

<table>
<thead>
<tr>
<th>W12 x 45</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>( R_x = 57.7 \text{ in} )</td>
<td>( R_y = 1.95 \text{ in} )</td>
<td>SMALLER ( R )-VALUE GOVERNS</td>
</tr>
<tr>
<td>( A = 13.1 \text{ in}^2 )</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

SLENDERNESS RATIO: THE LARGER VALUE GOVERNS

\[
\frac{KL}{R_x} = \frac{(2.1)(12 \text{ ft})(12 \text{ in})}{57.7 \text{ in}} = \frac{302.4 \text{ in}}{57.7 \text{ in}} = 5.241
\]

\[
\frac{KL}{R_y} = \frac{1302.4 \text{ in}}{1.95 \text{ in}} = 665
\]

---

AISC TABLE 4-22: AVAILABLE CRITICAL STRESS

\[
\frac{KL}{R_y} = 155 \quad \therefore \quad \phi F_y = 9.4 \text{ ksf}
\]

DESIGN STRESS \( F_d \) = \[\frac{\phi F_y}{0.9} = 10.4 \text{ ksf}\]
1. A bolted steel tension member is shown.

\[ F_y = 36 \text{ kips/in}^2 \]
\[ F_u = 58 \text{ kips/in}^2 \]

What is most nearly the effective net area in tension for this plate?

(A) 2.3 in\(^2\) \[ b_o = 2 \frac{3}{4} + 3 + 2 \frac{1}{4} = 7. \frac{1}{2} \]

(B) 2.9 in\(^2\)

(C) 3.4 in\(^2\)

(D) 3.8 in\(^2\)

\[ A_n = \left[ b_o - \sum (d_h + \frac{1}{16}) \right] t \]
\[ = \left[ 7 \frac{1}{2} - 2 \left( \frac{3}{4} + \frac{1}{16} \right) \right] \frac{1}{2} = 2.938 \text{ in} \]

**Effective Area**

\[ A_c = UA_n \quad U = 1 \quad \text{for flat bars} \]

\[ A_c = (1)(2.9) = 2.9 \text{ in} \]
2. A steel tension member is 5 in long and \( \frac{1}{2} \) in thick. There are two holes in the bar. The holes are in parallel and have a diameter of \( \frac{1}{4} \) in each. The net area is most nearly

\[ A_n = \left[ b_2 - 2 \sum (d_k + \frac{1}{16}) \right] t \]

\[ = \left[ 5\text{ in} - 2(\frac{1}{4} + \frac{1}{16}) \right] \frac{1}{2} = 2.19 \text{ in} \]

→ (A) 2.0 in\(^2\)  \( \text{ch} = \frac{1}{4} \text{ in} \)
(B) 2.2 in\(^2\)  \( t = \frac{1}{2} \text{ in} \)
(C) 2.5 in\(^2\)
(D) 2.7 in\(^2\)
3. A W-shape member (yield strength of 36 ksi; ultimate strength of 58 ksi) carries an axial live tensile load of 420 kips. The member’s flanges are bolted to a connection bracket. The shear lag factor for the connection is 0.90. The required net area based on the fracture (rupture) criterion is most nearly

\[\text{QUESTION}\]

(A) 12 in\(^2\)

(B) 17 in\(^2\) \[F_y = 36 \text{ ksi}\]

(C) 22 in\(^2\) \[F_u = 58 \text{ ksi}\]

(D) 27 in\(^2\) \[T_L = 420 \text{ kips}\] \[U = 0.9\]

\[T_u = 1.6 T_L = 1.6 (420 \text{ kips}) = 672 \text{ kips}\]

Fracture \(\phi_f = 0.75\)

\[\frac{T_u}{T_L} = \phi_f \frac{F_u A_e}{F_v A_n}\]

\[672 \text{ kips} = 0.75 \left( \frac{58 \text{ kips}}{\text{in}^2} \right) (0.9) A_n\]

\[A_n = 17.16 \text{ in}\]
4. An L4 × 4 × 3/8 angle made from A36 steel is used as a tension member as shown. The angle is connected to a gusset plate with 5/8 in diameter bolts. The center-to-center bolt spacing is 3 in. The distance from the centroid of the area connected to the plane of connection (the edge), $\bar{x}$, is 1.13 in.

What is most nearly the shear lag factor?

(A) 0.72
(B) 0.78
(C) 0.81
(D) 0.86

\[ U = 1 - \frac{\bar{x}}{L} = 1 - \frac{1.13 \text{ in}}{6 \text{ in}} = 0.812 \]
1. A W14 × 120, A992 steel beam has been chosen to carry an axial live compressive load of 140 kips and a factored 480 ft-kips live moment about the strong axis. The unsupported length is 20 ft. Sidesway is permitted in the direction of bending. $K = 1.0$. The compressive strength is 780 kips, and the bending strength is 495 ft-kips. The beam-column is subjected to

(A) small compression and is adequate
(B) large compression and is adequate
(C) small compression and is inadequate
(D) large compression and is inadequate

$W14 \times 120$

$P_L = 140$ kips
$M_L = 480$ ft·kips
$L = 20$

A992 steel $F_y = 50$ kips
$K = 1$

$P_c = 780$ kips
$M_o = 495$ ft·kips

NOT ON EXAM
2. A W14 × 132 beam has been chosen to carry an axial live load of 160 kips and a maximum live moment of 320 ft-kips about the strong axis. The unsupported length is 32 ft. \( K = 1 \), \( C_m = 1 \), and \( I_x = 1530 \text{ in}^4 \). Taking into account the second-order effects, what is most nearly the required flexural strength?

(A) 340 ft-kips
(B) 380 ft-kips
(C) 420 ft-kips
(D) 460 ft-kips
1. The connection shown consists of 11 grade A307 \( \frac{7}{8} \) in diameter bolts. Bolt hole sizes are standard. The ultimate strength of the connected member is 58 ksi. The connected member is 0.5 in thick. The edge clear distance is 2.5 in, and the center-to-center spacing of the holes is 3 in.

The available bearing strength per bolt per inch of thickness in the connection is most nearly

(A) 76 kips/in
(B) 83 kips/in
(C) 91 kips/in
(D) 110 kips/in
2. A connection is made from two $\frac{3}{4}$ in bolts in parallel, placed on a 8.5 in wide steel bar. The bar is 1 in thick that has an ultimate strength of 65 ksi. The holes are 3 in from their centers to the side of the bar and 2.25 in from center to center. Bolt hole sizes are standard. The bearing resistance of the entire connection is most nearly

(A) 84 kips
(B) 95 kips
(C) 170 kips
(D) 210 kips
A beam of Grade 50 steel is adequately braced and has a lateral-torsional buckling modification factor of $C_b = 1.0$. The maximum factored moment applied to the component is $M_u = 300 \text{ kip-ft}$. The lightest suitable W-shape required is most nearly:

a). W14 x 43
b). W14 x 48
c). W14 x 53
d). W14 x 61

\[ \phi M_a \geq M_u = 300 \text{ ft-kip} \]

PS 160 AISC TABLE 3-2

For a design strength ($\phi M_a$) of 300 ft-kips, the lightest beam required is W14 x 53 which has a $\phi M_a$ of 327 ft-kip.
Example 14.5

A W18 \times 65 \text{ Grade 50 beam has an unbraced segment length of } L_b = 14 \text{ ft} \text{ with } C_b = 1.0. \text{ The design flexural strength of the segment is most nearly:}

a). 297 kip-ft
b). 394 kip-ft

\text{ c). 432 kip-ft}
d). 499 kip-ft

\text{ \textit{FY} = 50 \text{ksi - Grade 50 Steel}}

PS 161 \textbf{TABLE 3-10}

\text{ Find W18 x 65 Line and } x = 14 \text{ ft}

\[ \phi M_n = 380 \text{ ft-kips} \]

\text{ ALTERNATE}

M_n = C_b \left[ M_p - (M_p - 0.7 F_y S_x) \left( \frac{L_b - L_r}{L_r - L_p} \right) \right], \quad M_n = M_p = F_y \cdot Z_x

\[ = M_p - (M_p - M_r) \left( \frac{L_b - L_p}{L_r - L_p} \right) = M_p - \left( \frac{M_p - M_r}{L_r - L_p} \right) (L_b - L_p) \]

\[ M_n = M_p - B F (L_b - L_p) \]

\[ \therefore \phi M_n = \phi M_p - \phi B F (L_b - L_p) = 1.99 - (14.9 \text{ kips})(13 \text{ ft} - 5.5 \text{ ft}) \]

\[ \phi M_n = 394 \text{ ft-kips} \]
A W12 × 50 Grade 50 column has a length of 9 feet and is pinned at each end and braced at third points about the minor axis. The maximum available design strength in compression of the column is most nearly:

a). 520 kips
b). 560 kips
c). 600 kips
d). 640 kips

\[ F_y = 50 \text{ ksf} \]
\[ L = 9 \text{ ft} \]

\[ \text{BRACED } \frac{L}{3} \text{ POINTS ABOUT Y AXIS} \]

\[ \text{UNBRACED LENGTH} = \frac{9}{3} = 3 \text{ ft} \]

**X-AXIS**

\[ \frac{KL}{r_x} = \frac{(1)(9 \text{ ft})(\frac{12 \text{ in}}{1 \text{ ft}})}{5.18 \text{ in}} = 20.85 \]

**Y-AXIS**

\[ \frac{KL}{r_y} = \frac{(1)(3 \text{ ft})(\frac{12 \text{ in}}{1 \text{ ft}})}{1.96 \text{ in}} = 18.37 \]

LARGER \( \frac{KL}{r} \) GOVERNS

**PS 163 ASDC TABLE 4-22**

IF \( \frac{KL}{r} = 20.9 \) THEN \( \phi F_c = 0.936 \text{ ksf} \)

**PS 157**

\[ P_n = F_{cr} A_g \]

\[ \phi P_n = \phi F_{cr} A_g = 0.936 \text{ ksf} (14.6 \text{ in}^2) = 636.6 \text{ kips} \]
Figure 14.12 shows a bolted connection with 3x3x3/8 inch angles bolted to a 3/4-inch gusset plate with four 3/4-inch diameter bolts in standard holes. All components are Grade 36 steel and the gross area of the double angles is $A_g = 4.22 \text{ in}^2$. Assuming that bolt strength and block shear do not govern, the design tensile strength of the double angles is most nearly:

a) 105 kips  \[ P_T = \frac{7}{8} \text{ in} \]
b) 115 kips  \[ P_T = 3.49 \text{ kips} \]
c) 125 kips  \[ P_T = 4.12 \text{ in} \]
d) 135 kips

\[
\begin{align*}
L &= 2 \text{ in} \\
L_e &= 2 \text{ in} \\
L &= 3 \times 3 \text{ in} = 9 \text{ in}
\end{align*}
\]

Figure 14.12 Calculation of shear lag factor

\[
\begin{align*}
A_d &= A_g - A_{	ext{holes}} = 4.22 \text{ in}^2 - 2(d_h)(t) = 4.22 \text{ in}^2 - 2(0.14 \text{ in})(0.3) = 3.56 \text{ in}^2 \\
U &= 0.6 \text{ (3 bolts)} \\
A_e &= UA_h = 0.6 (3.56) = 2.14 \text{ in}^2 \\
U &= 0.8 \text{ (4 bolts or more)}
\end{align*}
\]

**Fracture**  \[ \phi = 0.75 \text{ shear, pg 154} \]

\[
\phi P_n = \phi A_e F_U = 0.75 (2.14 \text{ in}^2) (58 \text{ kips}) = 124 \text{ kips}
\]

**Yield**  \[ \phi = 0.9 \text{ tension, pg 154} \]

\[
\phi P_n = \phi A_g F_y = 0.9 (4.22 \text{ in}^2) (36 \text{ kips}) = 136 > 124 \text{ kips}
\]

Take smaller of the two
A reinforced concrete beam, with an overall depth of 16 inches, an effective depth of 14 inches, and a width of 12 inches, is reinforced with Grade 60 bars and has a concrete cylinder strength of 3000 pounds per square inch. Determine the area of tension reinforcement required for the beam to support a superimposed live load of one kip per foot run over an effective span of 20 feet.

\[ \text{Grade} = 60 \]
\[ f_y = 60 \text{ ksf} \]
\[ f_c = 3 \text{ ksi} \]
\[ L = 20 \text{ ft} \]
\[ w_c = 1 \frac{1}{2} \text{ kip/ft} \]

**Example 14.24**

\[ M_v = 1.2M_D + 1.6M_c = 1.2 \frac{w_0l^2}{8} + 1.6 \frac{w_cl^2}{8} \]
\[ M_v = 1.2 \left( \frac{0.52}{20} \right) + 1.6 \left( \frac{20}{5} \right) = 92 \text{ ft-kip} \]

\[ P = 0.85 \leq 0.55 - 0.05 \left( \frac{0.12 - 4000}{10000} \right) \geq 0.45 \quad 0.85 = 0.85 = P \]

\[ \frac{M_a}{P} = 0.85 \frac{f_y}{f_c} A_b \left( d - \frac{a}{2} \right) \quad \phi M_a = M_v \]

\[ 92 \text{ ft-kip} \left( \frac{12 \text{ in}}{4} \right) = 0.85 \left( 3 \frac{\text{ kip}}{\text{ in}} \right) \left( a \right) \left( 12 \text{ in} \right) \left( 14 \text{ in} - \frac{a}{2} \right) \]

\[ a = 2.87 \text{ in} \]

\[ a = \frac{A_s f_y}{0.85 f_c b} \]

\[ 2.87 \text{ in}^2 = \frac{A_s (60)}{0.85(3)(12)} \]

\[ A_s = 1.46 \text{ in}^2 \]