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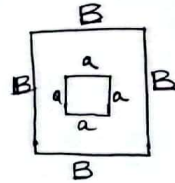
RCC

Footing

A 12" square column reinforced with a 10' square footing. The footing thickness is 20" and clear cover is 3.5". Calculate the ultimate punching shear capacity if $f'_c = 3000 \text{ psi}$, $f_y = 60,000 \text{ psi}$.

Soln

Column width, $a = 12''$



Footing size, $B = 10'$

Footing thickness, $t = 20''$

Clear cover, c.c. = 3.5''

Ultimate punching shear capacity, $V_u = ?$

Now, Effective depth, $d = t - \text{c.c.}$

$$= 20'' - 3.5''$$

$$= 16.5''$$

Punching perimeter, $b_o = 4(a + d)$

$$= 4(12 + 16.5)$$

$$= 114''$$

Length, $D_f = t = \text{thickness}$

①, ②, ③ → 5 marks serial

∴ Punching shear capacity,

$$V_u = \phi \cdot V_c$$

$$V_u = \phi \cdot [4\lambda \sqrt{f'_c} b_o d]$$

$$= 0.75 [4 \times 1 \times \sqrt{3000} \times 114 \times 16.5]$$

$$= 309079 \text{ lb}$$

$$= 309.08 \text{ kip}$$

(Ans)

$$\left. \begin{array}{l} \phi = 0.75 \\ \lambda = 1 \end{array} \right\}$$

$\left. \begin{array}{l} \text{Strength } f'_c \rightarrow \frac{\text{lb}}{\text{in}^2} \\ \text{Capacity } V_u \rightarrow \text{lb} \end{array} \right\}$

Hints:

For Footing in R.C.C. maths:

$$\checkmark \phi = 0.75$$

$$\checkmark \lambda = 1$$

$$\sqrt{f'_c} \rightarrow \text{psi}$$

$q_u = \text{Bearing Capacity}$

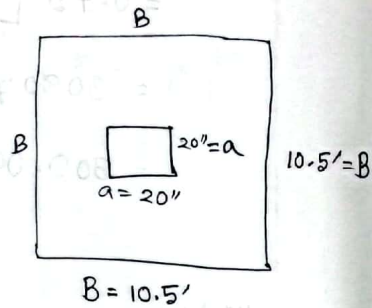
② *
BCE-2017

A footing size $10.5' \times 10.5'$. Factored load on footing is 330 Kip and column size is $20'' \times 20''$. Calculate the depth of footing considering punching shear check. $f'_c = 4 \text{ Ksi}$.

Soln: Factored Load, $P_u = 330 \text{ Kip}$

Column size, $a \times a = 20'' \times 20''$

Depth of Footing, $t = ?$
(= Thickness)



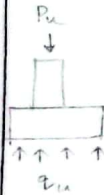
Now,

Upward pressure, $q_u = \frac{\text{Total Load on Column}}{\text{Column Area}}$

$$\Rightarrow q_u = \frac{P_u}{B \times B}$$

$$= \frac{330}{10.5 \times 10.5} \text{ Kip/ft}^2$$

$$= 3 \text{ Ksf}$$



$d = 14''$ trial
 \leftarrow q_u \rightarrow compare
 \leftarrow $d = 12''$ \rightarrow

$e.c. = 3''$ (Footing)
 $\phi = 0.75$
 $\lambda = 1$

Let, $d = 14''$ and clear cover for footing = $3''$.

\therefore Punching perimeter, $b_o = 4(a+d)$

$$= 4(20+14)$$

$$= 136 \text{ in}$$

Shear strength, $V_u = P_u - q_u (a+d)^2$

$$= 330 - 3 \times \left(\frac{20}{12} + \frac{14}{12}\right)^2$$

$$= 306 \text{ Kip}$$

And, $V_u = \phi \cdot V_c$

$$\Rightarrow V_u = \phi \cdot [4\lambda \sqrt{f'_c} b_o d]$$

$$\Rightarrow \frac{306}{1000} = 0.75 \times 4 \times 1 \times \sqrt{4000} \times 136 \times d$$

$$\therefore d = 11.85 \text{ in} \approx 12''$$

\therefore Thickness = Depth of footing, $t = d + e.c.$

$$\Rightarrow t = 12'' + 3''$$

$$= 15'' \text{ (Ans.)}$$

Q. A 16" square column reinforced with 8 Nos. 20 mm bar up to footing and the column carries total 550 kip load. If the footing size is 9' x 9' and effective depth is 24", find the punching shear stress developed in the footing.

Soln: Column load, $P_u = 550$ kip

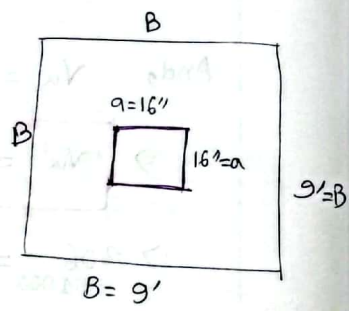
Footing size, $B \times B = 9' \times 9'$

Column size, $a \times a = 16" \times 16"$

$d = 24" = 2'$

Punching shear stress,

$$\frac{V_u}{b_o d} = ?$$



8 Nos. 20 mm Bar
 ↪ No use
 $K_s f = K/ft^2$
 $K_c f = K/ft^3$

$$V_u = \text{Punching shear}$$

$$\frac{V_u}{b_o d} = \text{Punching shear stress}$$

Punching perimeter, $b_o = 4(a+d)$

$$= 4(16 + 24)$$

$$= 160 \text{ in}$$

$$= 13.33 \text{ ft}$$

Upward pressure, $q_u = \frac{P_u}{B \times B}$

$$= \frac{550}{9 \times 9}$$

$$= 6.79 \text{ Ksf}$$

Shear strength, $V_u = P_u - q_u(a+d)^2$

$$= 550 - 6.79 \left(\frac{16}{12} + \frac{24}{12} \right)^2$$

$$= 474.55 \text{ kip}$$

Punching shear stress = $\frac{V_u}{b_o d}$

$$= \frac{474.55}{13.33 \times 2}$$

$$= 17.80 \text{ K/ft}^2 \text{ (Ans)}$$

⑧ * If DL is 225 kips and LL is 185 kips on column, determine the footing size when bearing capacity of soil is 2 tsf.

Soln: Footing size, $B \times B = ?$

Bearing capacity of soil = soil pressure
 $\Rightarrow q_u = 2 \text{ tsf}$
 $= 2 \times 2.204 \text{ kip/ft}^2$
 $= 4.408 \text{ k/ft}^2$

$tsf = \text{Ton/ft}^2$
 $1 \text{ tsf} = 2.204 \text{ Ksf}$
 $1000 \text{ kg/ft}^2 = 2.204 \text{ K/ft}^2$

Now, Total Service Load, $P_u = DL + LL$
 $= 225 + 185$
 $= 410 \text{ kip}$

And, $q_u = \frac{P_u}{B \times B}$
 Footing Area = $\frac{\text{Service Load}}{\text{Soil pressure}}$

$\Rightarrow B \times B = \frac{P_u}{q_u}$ [Considering, Square footing.]

$$\Rightarrow B \times B = \frac{410}{4.408}$$

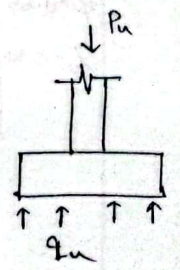
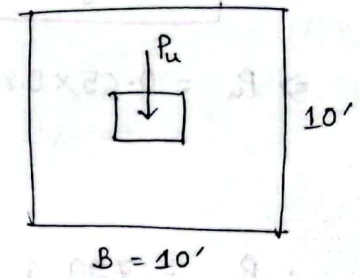
$$\Rightarrow B \times B = 93 \text{ ft}^2$$

$$\Rightarrow B = \sqrt{93}$$

$$= 9.644 \text{ ft}$$

$$\approx 10 \text{ ft}$$

\therefore Footing size, $B \times B = 10' \times 10'$ (Am)



Column or, Tied Column (Rectangular)

Find the axial load of column $30'' \times 12''$ which is reinforced with 12 Nos. 20 mm diameter bars, consider $f'_c = 3.5 \text{ Ksi}$ and $f_y = 60 \text{ Ksi}$.

$$A_g = B \times L$$

$$A_g = 30 \times 12 = 360 \text{ in}^2$$

$$A_{st} = 12 \text{ Nos. } 20 \text{ mm} \\ = 12 \times \frac{\pi}{4} \left(\frac{20}{25.4} \right)^2 \text{ in}^2$$

$$= 5.84 \text{ in}^2$$

$$A_{st} = n \times \frac{\pi}{4} d^2$$

$$A_{st} = 12 \times \frac{\pi}{4} \left(\frac{20}{25.4} \right)^2 \\ = 5.84 \text{ in}^2$$

Now, Axial Load, $P_u = \phi \cdot P_n$

$$\Rightarrow P_n = \phi \cdot \alpha [0.85 f'_c (A_g - A_{st}) + A_{st} f_y]$$

$$\Rightarrow P_n = 0.65 \times 0.80 [0.85 \times 60 \times (360 - 5.84) + 5.84 \times 60]$$

$$\therefore P_n = 730.1 \text{ Kip}$$

(Am)

$$\phi = 0.65$$

$$\alpha = 0.80$$

Determine the maximum nominal axial load for a $15'' \times 20''$ tied column, having 2.5% steel reinforcement. $f'_c = 4 \text{ Ksi}$ and $f_y = 60 \text{ Ksi}$.

Solⁿ: Maximum nominal axial load, $P_n = ?$

$$A_g = 15 \times 20 = 300 \text{ in}^2$$

$$A_g = B \times L$$

$$f = 2.5\% = 0.025$$

$$f = \frac{2.5}{100}$$

Maximum nominal axial load,

$$P_n = \alpha [0.85 f'_c (A_g - A_{st}) + A_{st} \cdot f_y]$$

$$P_n = \alpha \cdot A_g [0.85 f'_c (1 - f) + f \cdot f_y]$$

$$= 0.80 \times 300 [0.85 \times 4 \times (1 - 0.025) + 0.025 \times 60]$$

$$= 1155.6 \text{ Kip}$$

(Am)

Hints: For Column \rightarrow

$$\checkmark \phi = 0.65$$

$$\checkmark \alpha = 0.80$$

$$\text{No. 9 bar} = \#9\varnothing = \frac{9}{8} \text{ " dia}$$

TGTDEL-2018
BEPZA-2019
NHA-2020

④
A rectangular column of 15" x 15" section has reinforcement of 8 Nos. 9 bars. If the value of f'_c is 3000 psi and f_y is 60000 psi, find the ultimate design axial load of the column with zero eccentricity.

Solⁿ: For zero eccentricity, $\alpha = 1$

$$A_g = 15 \times 15 = 225 \text{ in}^2 \quad [A_g = B \times h], \quad [A_{st} = n \times \frac{\pi}{4} d^2]$$

$$A_{st} = 8 \times \frac{\pi}{4} \left(\frac{9}{8}\right)^2 = 8 \times 1 = 8 \text{ in}^2$$

Ultimate design axial load,

$$P_u = \phi \cdot \alpha [0.85 f'_c (A_g - A_{st}) + A_{st} \cdot f_y]$$

$$= 0.65 \times 0.80 [0.85 \times 3 \times (225 - 8) + 8 \times 60]$$

$$= 671.7 \text{ kip} \quad (\text{Ans})$$

$\phi = 0.65$

⑤
*
GTCL-2016

The value of f'_c is 4000 psi, f_y is 60000 psi and steel area is 2% and the column section is 12" x 12". Find the design ultimate axial stress of the column with zero eccentricity.

Solⁿ: $f = 2\% = 0.02 = \frac{2}{100} = \text{Reinforcement Ratio}$

$$A_g = 12 \times 12 = 144 \text{ in}^2$$

zero eccentricity, $\alpha = 1$

Design ultimate axial load,

$$P_u = \phi \cdot \alpha [0.85 f'_c (A_g - A_{st}) + A_{st} \cdot f_y]$$

$$P_u = \phi \cdot \alpha \cdot A_g [0.85 f'_c (1 - f) + f \cdot f_y]$$

$$= 0.65 \times 0.80 \times 144 [0.85 \times 4 (1 - 0.02) + 0.02 \times 60]$$

$$= 424.19 \text{ kip}$$

∴ Design ultimate axial stress, $\sigma_u = \frac{P_u}{A_g} = \frac{\text{Load}}{\text{Area}}$

$$= \frac{424.19}{144}$$

$$= 2.94 \text{ kip/in}^2$$

(Ans)

 PRCCL-2014
 PNCC-2016
 ERL-2017
 BGFCL-2017

Determine the nominal axial compression force for RCC column of 12" x 22" section having 2.5% mild steel reinforcement. Given, $f'_c = 4 \text{ Ksi}$, $f_y = 60 \text{ Ksi}$.

Soln: Nominal axial compression force, $P_n = ?$

$$A_g = 12 \times 22 = 264 \text{ in}^2$$

$$\rho = 2.5\% = 0.025 = \frac{2.5}{100}$$

Nominal axial compression force,

$$P_n = \alpha [0.85 f'_c (A_g - A_{st}) + A_{st} f_y]$$

$$P_n = \alpha \cdot A_g [0.85 f'_c (1 - \rho) + \rho \cdot f_y]$$

$$= 0.80 \times 264 \times [0.85 \times 4 \times (1 - 0.025) + 0.025 \times 60]$$

$$= 1016.93 \text{ kips}$$

(Ans)

#9 bar = $\frac{3}{8}$ " dia
 #10 ϕ = $\frac{10}{8}$ " dia

Size and spacing of Tie bar:

30

 BGFCL-2017

Design tie size and spacing of a 24" x 16" rectangular column which is reinforced with 12 #9 bar.

Soln: Longitudinal bar = 12 #9 ϕ bar

As longitudinal bar is less than #10 ϕ ,

Use #3 ϕ bar as Tie bar.

Spacing:

(i) $48 \times d(\text{tie}) = 48 \times \frac{3}{8}'' = 18 \text{ in}$

(ii) $16 \times D(\text{main}) = 16 \times \frac{9}{8}'' = 18 \text{ in}$

(iii) Least dimension = 16 in
 (B or L)

So, Use #3 ϕ bar @ 16" c/c for tie bar.

(Ans)

Column Design

Design a square tied column to support dead load of 130 kips and Live load of 180 kips. Reinforcement ratio must be within 1 to 2%. Assume, $f'_c = 4 \text{ ksi}$, $f_y = 60 \text{ ksi}$.

Soln: DL = 130 kips

LL = 180 kips

$\rho = 1\% \text{ to } 2\% = 2\%$

$\rho = 2\% = 0.02$

Design a square column. $A_g = ?$ $A_{st} = ?$

Now, $P_u = \phi \cdot \alpha \cdot A_g [0.85 f'_c (1 - \rho) + \rho \cdot f_y]$

$\Rightarrow 444 = 0.65 \times 0.80 \times A_g [0.85 \times 4 \times (1 - 0.02) + 0.02 \times 60]$

$\therefore A_g = 188.4 \text{ in}^2$

$P_u = 1.2 \text{ DL} + 1.6 \text{ LL}$
 $= 1.2 \times 130 + 1.6 \times 180$
 $= 444 \text{ kip}$

$\phi = 0.65$
 $\alpha = 0.80$

For square column, Area $A_g = b \times h = b \times b$

$\therefore b = \sqrt{A_g}$

$= \sqrt{188.4}$

$= 13.7''$

$\approx 14''$ (say)

$\therefore 14'' \times 14''$ square tied column.

So, $A_g = 14 \times 14 = 196 \text{ in}^2$

Again, $P_u = \phi \cdot \alpha [0.85 f'_c (A_g - A_{st}) + A_{st} \cdot f_y]$

$\Rightarrow 444 = 0.65 \times 0.80 [0.85 \times 4 \times (196 - A_{st}) + A_{st} \times 60]$

$\therefore A_{st} = 3.31 \text{ in}^2$

Use #6 bars, $A_b = \frac{\pi}{4} \times \left(\frac{6}{8}\right)^2 = 0.44 \text{ in}^2$

Nos. of bars = $\frac{3.31}{0.44} = 7.5 \approx 8 \text{ Nos.}$

So, Use 8 Nos. #6 bars. (Ans)

22*
NCEB-2017

Design a square tied column to support dead load of 500 kips and live load of 400 kips. Reinforcement ratio must be within 1 to 2%. Assume, $f'_c = 4 \text{ ksi}$, $f_y = 60 \text{ ksi}$.

Soln:

$$P_u = 1.2 \text{ DL} + 1.6 \text{ LL}$$

$$= 1.2 \times 500 + 1.6 \times 400$$

$$= 1240 \text{ kips}$$

$$\left. \begin{aligned} A_g &=? \\ A_{st} &=? \\ \phi &= 0.65 \\ \alpha &= 0.80 \end{aligned} \right\}$$

Steel ratio, $\rho = 1\% \text{ to } 2\%$ $\rho = 2\%$

$$\therefore \rho = 2\% = 0.02 \text{ (say)}$$

$$P_u = \phi \cdot \alpha \cdot A_g [0.85 f'_c (1-\rho) + \rho \cdot f_y]$$

$$\Rightarrow 1240 = 0.65 \times 0.80 \times A_g \times [0.85 \times 4 \times (1-0.02) + 0.02 \times 60]$$

$$\Rightarrow A_g = 526.17 \text{ in}^2$$

$$\therefore b = \sqrt{A_g} = 22.94'' \approx 23'' \text{ (say)}$$

$$\therefore A_g = b \times b = 23 \times 23 = 529 \text{ in}^2$$

Again,

$$P_u = \phi \cdot \alpha [0.85 f'_c (A_g - A_{st}) + A_{st} \times f_y]$$

$$\Rightarrow 1240 = 0.65 \times 0.80 \times [0.85 \times 4 \times (529 - A_{st}) + A_{st} \times 60]$$

$$\Rightarrow A_{st} = 10.35 \text{ in}^2$$

$$\begin{aligned} A_{st} &= \rho \times A_g \\ &= 0.02 \times 529 \\ &= 10.58 \text{ in}^2 \end{aligned}$$

Use, # 11 ϕ bars,

$$A_b = \frac{\pi}{4} \times \left(\frac{11}{8}\right)^2 = 1.49 \text{ in}^2$$

$$\text{Nos of bars, } n = \frac{A_{st}}{A_b} = \frac{10.35}{1.49}$$

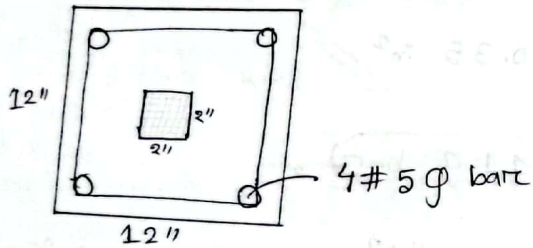
$$= 6.97 \approx 7$$

So, Use 7 # 11 ϕ bars. (Ans)

Hollow Column

A Hollow rectangular column has shown in figure, the column is reinforced with 4#5 bars. Determine the capacity of the column and also design the tie reinforcement.

Soln:



$$A_g = (12 \times 12) - (2 \times 2) = 144 \text{ in}^2 - 4 \text{ in}^2 = 140 \text{ in}^2$$

$$A_{st} = 4 \times \frac{\pi}{4} \left(\frac{5}{8}\right)^2 = 1.23 \text{ in}^2$$

Capacity of Column, $P_u = ?$

Design the tie bar. size & spacing = ?

$$f'_c = 4 \text{ ksi} \quad \phi = 0.65$$
$$f_y = 60 \text{ ksi} \quad \alpha = 0.80$$

Now, Capacity of Column,

$$P_u = \phi \cdot \alpha [0.85 f'_c (A_g - A_{st}) + A_{st} \cdot f_y]$$
$$= 0.65 \times 0.80 [0.85 \times 4 \times (140 - 1.23) + 1.23 \times 60]$$
$$= 283.72 \text{ KIP}$$

As, the main bar is less than #10 bar,

Use #3 bar as Tie.

Tie Spacing:

- smallest
- (i) $48 \times d(\text{tie}) = 48 \times \frac{3}{8}'' = 18''$
 - (ii) $16 \times D(\text{main}) = 16 \times \frac{5}{8}'' = 10''$
 - (iii) Least dimension (b or L) = 12'' (Hollow शिवाव शयना)

So, Use #3 bar @ 10" c/c for

Tie bar. (Am)

BIET M.T. 2018
DDDC-2013

WSD: Rectangular Column

A reinforced concrete column of effective length 4.8 m and overall dimension $250\text{ mm} \times 400\text{ mm}$. Design axial load on column is 60 tones. Concrete mix used is M20 and mild steel used. Using working stress design (WSD) method to determine the reinforcement required for column, also design tie spacing.

Soln. Effective length, $L_e = 4.8\text{ m} = 4800\text{ mm}$

$$A_g = 250 \times 400 = 10^5 \text{ mm}^2$$

$$b = 250\text{ mm}$$

$$\begin{aligned} \sqrt{P_u} &= 60\text{ ton} \\ &= 60 \times 1000 \times 9.81\text{ N} \\ &= 5.886 \times 10^5\text{ N} \end{aligned}$$

$A_s = ?$ Spacing of tie = ?

For M20 concrete, $f'_c = 20\text{ MPa}$
 $= 20\text{ N/mm}^2$

For mild steel, $f_y = 415\text{ MPa}$
 $= 415\text{ N/mm}^2$

$$\therefore f_s = 0.4 \times f_y = 166\text{ N/mm}^2$$

Now, $\frac{L_e}{b} = \frac{4800\text{ (mm)}}{250\text{ (mm)}} = 19.2 > 12$

So, Long column. Reduction factor will be used.

Reduction factor,

$$C_{rc} = 1.25 - \frac{L_e}{48b}$$

$$= 1.25 - \frac{4800}{48 \times 250}$$

$$= 0.85$$

Axial Load, $P = \frac{P_u}{C_{rc}} = \frac{5.886 \times 10^5}{0.85} = 6.92 \times 10^5\text{ N}$

And, $P = C_{rc} (0.25 f'_c A_g + f_s A_s)$

$$\Rightarrow 6.92 \times 10^5 = 0.85 (0.25 \times 20 \times 10^5 + 166 \times A_s)$$

$$\therefore A_s = 1895 \text{ mm}^2$$

For, 16mm bar, $A_b = \frac{\pi}{4} \times \left(\frac{16}{25.4}\right)^2 \text{ mm}^2$

$$= 201.06 \text{ mm}^2$$

$$\therefore \text{No. of bar, } n = \frac{A_s}{A_b} = 9.44 \approx 10 \text{ (say)}$$

As Main bar 16mm, Use Tie bar 10mm

Spacing: (i) $48 \times d(\text{tie}) = 48 \times 10 = 480 \text{ mm}$

(ii) $16 \times D(\text{main}) = 16 \times 16 = 256 \text{ mm}$

(iii) Least Dimension = 250 mm

Use 10mm bar @ 250mm c/c for Tie bars (Am.)

WSD: Circular Column

A reinforced concrete column of effective length 7m and column diameter is 300 mm with 8-16 mm rebar. Concrete mix used is M15 and $f_y = 415 \text{ MPa}$. Using working stress design (WSD) method to determine the column strength.

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*
AB-2017

Soln: Effective length, $L_e = 7\text{m} = 7000 \text{ mm}$

b=d=dia
circular

Dia of column, $b = 300 \text{ mm} = d$

$$A_s = 8 \times \frac{\pi}{4} \times 16^2 = 1608.5 \text{ mm}^2$$

M15, $f'_c = 15 \text{ MPa} = 15 \text{ N/mm}^2$

$$f_y = 415 \text{ MPa} = 415 \text{ N/mm}^2$$

$$\therefore f_s = 0.4 \times f_y = 0.4 \times 415 = 166 \text{ N/mm}^2$$

$$A_g = \frac{\pi}{4} d^2 = \frac{\pi}{4} \times 300^2 = 70686 \text{ mm}^2$$

Column strength, $P = ?$ (\because circular)

$$\text{Now, } \frac{L_e}{b} = \frac{7000}{300} = 23.33 > 12$$

$\frac{L_e}{b} > 12$ So, it is long column. Reduction factor will be used.

$$C_{rc} = 1.25 - \frac{L_e}{48b}$$

$$= 1.25 - \frac{7000}{48 \times 300}$$

$$= 0.76$$

$$P = C_{rc} (0.25 f_c A_g + f_s A_s)$$

$$= 0.76 (0.25 \times 15 \times 70686 + 166 \times 1608.5)$$

$$= 404383.46 \text{ N}$$

$$= 404.38 \text{ KN}$$

(Am)

Hints for Column

P_n = Nominal Axial compression Load

= Allowable compression Load

= Allowable Axial Load

= Axial Load

P_u = Design Capacity

= Design Axial Load

= Allowable axial design load

= Axial service Load

= Compressive Load

= Column Strength

Tie bar :

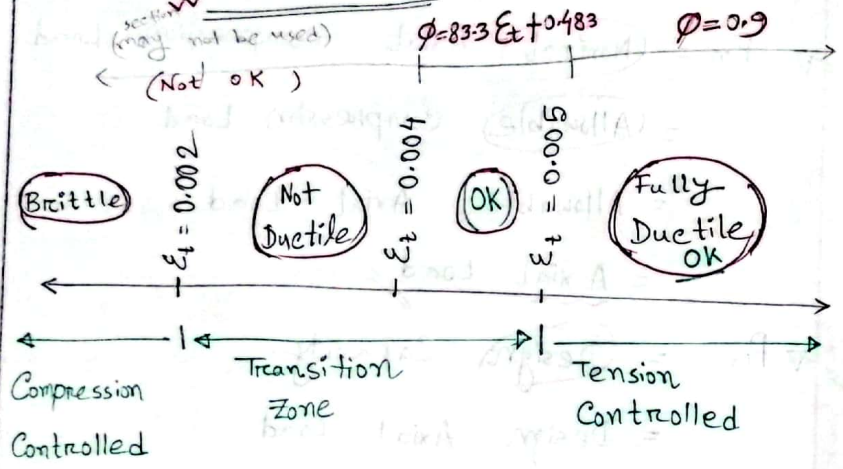
Main bar #10 \Rightarrow Tie = #3 bar

Main bar #10 \Rightarrow Tie = #4 bar

• यदि main bar 16 mm \Rightarrow Tie = 5 mm.

Beam :

Concepts



- ✓ $\epsilon_t > 0.005$ = Tension Controlled
 $\Rightarrow \phi = 0.9$
- ✓ $0.004 < \epsilon_t < 0.005$ = Transition zone
 $\Rightarrow \phi = 83.3 \times \epsilon_t + 0.483$
- ✓ $0.004 > \epsilon_t$ = Not Ductile
 \Rightarrow May not be used as per ACI Code.

Hints :

- ✓ $\rho = \frac{A_s}{bd}$ | $\epsilon_u = 0.003$
- ✓ $\rho_b = 0.85 \beta_1 \frac{f'_c}{f_y} \frac{0.003}{0.003 + \epsilon_y}$
- ✓ $\rho_{max} = 0.85 \beta_1 \frac{f'_c}{f_y} \frac{0.003}{0.003 + 0.005}$
- ✓ $\rho_{min} = 0.0033$
- ✓ $\rho > \rho_{min}$ (OK)
 Singly reinforced Beam.
- ✓ $\rho_{max} > \rho$ = Failure of steel by yielding.
- ✓ $\rho_{max} < \rho$ = Beam will fail by cracking of concrete.

Beam: USD → Normal maths

BB-2021

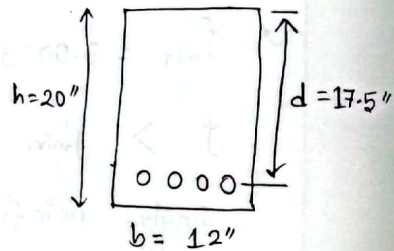
34

* A rectangular beam has width $b=12''$, depth $h=20''$ and effective depth $d=17.5''$. The beam is reinforced with 4#9 bars, $f_y = 60 \text{ ksi}$ and $f'_c = 4 \text{ ksi}$. Take the beam as under-reinforced and calculate the nominal strength of the beam.

Sol'n

Nominal strength, $M_n = ?$

Take "Under-reinforced Beam."



[4#9 bars]

$$A_s = n \times A_b$$

$$A_s = n \times \frac{\pi}{4} (\text{dia})^2$$

$$= 4 \times \frac{\pi}{4} \left(\frac{9}{8}\right)^2$$

$$= 4 \times 1 = 4 \text{ in}^2$$

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

$$= \frac{4 \times 60}{0.85 \times 4 \times 12}$$

$$= 5.88''$$

$$\text{Nominal Strength, } M_n = A_s f_y \left(d - \frac{a}{2}\right)$$

$$= 4 \times 60 \times \left(17.5 - \frac{5.88}{2}\right)$$

$$= 3949 \text{ kip-in}$$

(Am.)

Hints: For Beam:

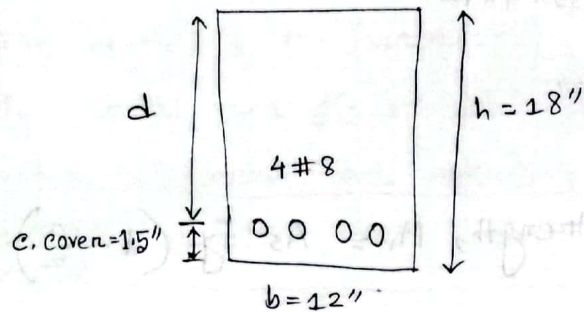
$$\text{When } \rightarrow f'_c = \frac{3}{4} \text{ ksi} \text{ \& } \beta_1 = 0.85$$

$$\rightarrow f'_c = 5 \text{ ksi} \text{ , } \beta_1 = 0.80$$

$$\left(f'_c = 3 \text{ ksi} \text{ , } \beta_1 = 0.85 \right)$$

BWDB-2018

Q9) * Determine the ultimate moment for the beam shown below where $f'_c = 4 \text{ Ksi}$, $f_y = 60 \text{ Ksi}$, clear cover = $1.5''$.



Solⁿ: Ultimate moment, $M_u = ?$

$$d = h - \text{c. cover}$$

$$= 18'' - 1.5''$$

$$= 16.5''$$

$$A_s = 4 \times \left(\frac{\pi}{4} \times 1^2 \right) = 3.16 \text{ in}^2$$

Check-01

$$\rho = \frac{A_s}{bd} = \frac{3.16}{12 \times 16.5} = 0.016$$

$\rho > \rho_{min} (0.0033)$, So Singly Reinforced beam
 $\rho_{min} = 0.0033$

check-02

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

$$= \frac{3.16 \times 60}{0.85 \times 4 \times 12}$$

$$= 4.65''$$

$\beta_1 = 0.85$,
 as $f'_c = 4 \text{ Ksi}$

$$c = \frac{a}{\beta_1} = \frac{4.65}{0.85} = 5.47''$$

$$\xi_t = 0.003 \times \left(\frac{d-c}{c} \right)$$

$$= 0.003 \times \left(\frac{16.5 - 5.47}{5.47} \right)$$

$$= 0.00605 > 0.005$$

So, Tension controlled section, $\phi = 0.9$

$$M_u = \phi \cdot M_n$$

$$M_u = \phi \cdot A_s f_y \left(d - \frac{a}{2} \right)$$

$$= 0.9 \times 3.16 \times 60 \times \left(16.5 - \frac{4.65}{2} \right)$$

$$= 2418.82 \text{ Kip-in (Ans.)}$$

25*
DE 500-2015

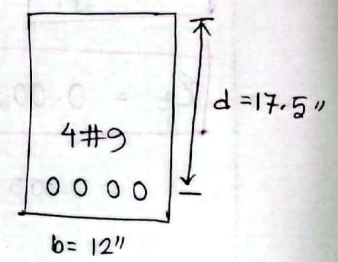
$$f = \frac{A_s}{A_g}$$
$$f = \frac{A_s}{bd}$$

A rectangular beam has a width of 12", an effective depth to the centroid of the reinforcing steel of 17.5". It is reinforced with four Nos 9 bar in one row. If $f_y = 60$ ksi and $f'_c = 4$ ksi, what is the ultimate moment capacity of the beam? Also check what would be the ϕ value.

Soln: Ultimate moment capacity, $M_u = ?$

ϕ value = ?

$$A_s = n \times A_b$$
$$= 4 \times 1 \text{ in}^2$$
$$= 4 \text{ in}^2$$



C-01%

$$f = \frac{A_s}{bd} = \frac{4}{12 \times 17.5} = 0.02 > f_{min} (0.0033)$$

so, singly reinforced beam. (OK)

C-02%

$$a = \frac{A_s f_y}{0.85 f'_c b}$$
$$= \frac{4 \times 60}{0.85 \times 4 \times 12} = 5.88"$$

$$c = \frac{a}{\beta_1} = \frac{5.88}{0.85} = 6.92"$$

$$\beta_1 = 0.85$$

as $f'_c = 4$ ksi

$$\epsilon_t = 0.003 \left(\frac{d-c}{c} \right)$$
$$= 0.003 \left(\frac{17.5 - 6.92}{6.92} \right)$$
$$= 0.0046$$

As, $0.004 < \epsilon_t < 0.005$

So, Beam is in transition zone.

$$\phi = 83.3 \times \epsilon_t + 0.483$$
$$= 83.3 \times 0.0046 + 0.483$$
$$= 0.866$$

13

Imp. check

Now, Ultimate moment capacity,

$$M_u = \phi \cdot M_n$$
$$M_u = \phi \cdot A_s f_y \left(d - \frac{a}{2} \right)$$
$$= 0.866 \times 4 \times 60 \left(17.5 - \frac{5.88}{2} \right)$$
$$= 3026 \text{ kip-in} / (\text{ft}) \quad (Am)$$

22
 **
 BWDB-2016
 BEPZA-2021

A rectangular beam of width 12" and effective depth 16" has four Nos. 9 bars in a row on tension side. Check whether the beam is tension controlled or not if $f'_c = 4 \text{ Ksi}$ and $f_y = 60 \text{ Ksi}$. Determine design capacity of the beam.

Soln: $A_s = 4 \times 1 \text{ in}^2$

Check-1: $\rho = \frac{A_s}{bd} = \frac{4}{12 \times 16} = 0.0208 > \rho_{min} (0.0033)$

So, single reinforced beam.

Check-2: $a = \frac{A_s f_y}{0.85 f'_c b} = \frac{4 \times 60}{0.85 \times 4 \times 12} = 5.88"$

$\beta = 0.85$
 $c = \frac{a}{\beta_1} = \frac{5.88}{0.85} = 6.92"$

$\epsilon_t = 0.003 \times \left(\frac{d-c}{c} \right) = 0.0039 < 0.004$

The beam section is in transition region and section is not ductile.

So, this section may not be used as per ACI.
 (Am.)

Beam Design

20
 NESCO-2021

The cross section of a beam reinforced with 3 in² of steel. Assume the tension steel yield at maximum moment. What is the area required to balance the steel force when the steel yields? $f'_c = 4 \text{ Ksi}$, $f_y = 60 \text{ Ksi}$.

Soln: Beam area to balance the steel force, $b d = ?$

$\epsilon_y = \frac{f_y}{E} = \frac{60 \text{ (Ksi)}}{29 \times 10^3 \text{ (Ksi)}} = 0.00207 = 0.0021 \text{ (say)}$

Balance steel ratio,

$\rho_b = 0.85 \beta_1 \frac{f'_c}{f_y} \left(\frac{\epsilon_u}{\epsilon_u + \epsilon_y} \right)$ | $\epsilon_u = 0.003$

$= 0.85 \times 0.85 \times \frac{4}{60} \times \left(\frac{0.003}{0.003 + 0.0021} \right) = 0.0285$

And, $\rho_b = \frac{A_s}{bd}$ | Given, $A_s = 3 \text{ in}^2$

$\Rightarrow 0.0285 = \frac{3}{bd}$

$\therefore bd = 105.24 \text{ in}^2$ (Ans.)

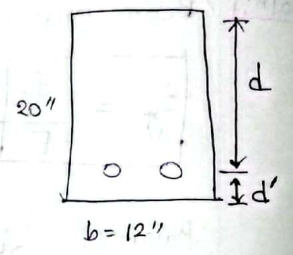
For Design,
 $\phi = 0.9$

WASA-2017
 NGA Forum-2023

A rectangular beam of section 12" x 20" carries a moment 180 K-ft. Find out the reinforcement required to resist the moment and either failure occurs by cracking of concrete or yield of steel. Consider $f'_c = 3 \text{ Ksi}$, $f_y = 60 \text{ Ksi}$ and $d' = 3 \text{ in}$.

Soln: $A_s = ?$ $M_u = 180 \text{ K-ft}$

Failure occurs by concrete cracking or steel yielding?



$$d = h - d' = 20 - 3 = 17 \text{ inch}$$

$d' = e_{\text{cover}}$

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

$$= \frac{A_s \times 60}{0.85 \times 3 \times 12}$$

$$\therefore a = 1.96 A_s \quad \text{--- (i)}$$

Moment = Ultimate Moment
 $= M_u$

$$M_u = \phi \cdot M_n$$

$$\Rightarrow M_u = \phi \cdot A_s f_y \left(d - \frac{a}{2} \right)$$

$$\Rightarrow 180 \times 12 = 0.9 \times A_s \times 60 \left(17 - \frac{1.96 A_s}{2} \right)$$

$$\Rightarrow A_s = 2.80 \text{ in}^2$$

$$\text{Now, } \rho = \frac{A_s}{bd} = \frac{2.80}{12 \times 17} = 0.0137$$

$$\rho_{\text{max}} = 0.85 \beta_1 \frac{f'_c}{f_y} \frac{0.003}{0.003 + 0.005}$$

$$= 0.85 \times 0.85 \times \frac{3}{60} \times \left(\frac{0.003}{0.003 + 0.005} \right)$$

$$= 0.0135$$

$\rho > \rho_{\text{max}}$ (Not OK)

So, Beam will fail by cracking of concrete
 (Am)

22

USD

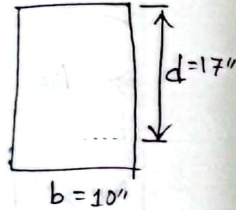
For Design
 $\phi = 0.9$

*** Find out the amount of steel required when ultimate moment capacity is 1600 Kip-in, Effective depth, $d = 17''$, width = 10 inch. $f'_c = 4 \text{ ksi}$, $f_y = 60 \text{ ksi}$.

Soln:

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

$$= \frac{A_s \times 60}{0.85 \times 4 \times 10}$$



$$\therefore a = 1.765 A_s \quad \text{--- (i)}$$

$$M_u = \phi \cdot M_n$$

$$\Rightarrow M_u = \phi \cdot A_s f_y \left(d - \frac{a}{2} \right)$$

$$\Rightarrow 1600 = 0.9 \times A_s \times 60 \times \left(17 - \frac{1.765 A_s}{2} \right)$$

$$\therefore A_s = 1.938 \text{ in}^2$$

Use # 8 ϕ bar. ($A_b = 0.79 \text{ in}^2$)

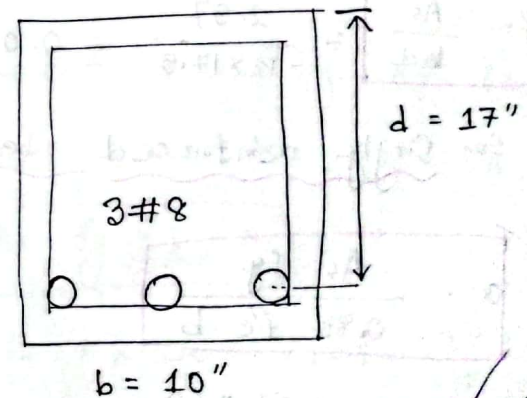
$$\text{Nos. of bar, } n = \frac{A_s}{A_b}$$

$$= \frac{1.938}{0.79}$$

$$= 2.47$$

$$\approx 3 \text{ Nos.}$$

So, Use 3 Nos. 8 ϕ bar.



(Am.)

→ # Figure Doubly reinforced but Singly beam

29

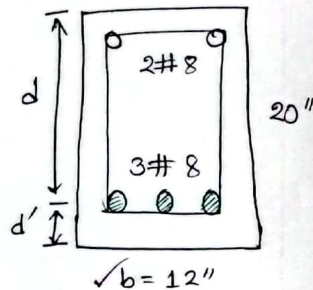
* Calculate the ultimate moment capacity of the following beam. given $h = 20''$, $b = 12''$, $d' = 2.5''$, $f'_c = 4 \text{ ksi}$, $f_y = 60 \text{ ksi}$. Assuming singly reinforced beam.

Soln:

$$d = h - d'$$

$$= 20 \text{ in} - 2.5 \text{ in}$$

$$= 17.5 \text{ in}$$



$$A_s = 3 \times 0.79 = 2.37 \text{ in}^2$$

check-01:

$$\rho = \frac{A_s}{bd} = \frac{2.37}{12 \times 17.5} = 0.011 > \rho_{\min} (0.0033)$$

So, singly reinforced beam (OK).

check-02:

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

$$= \frac{2.37 \times 60}{0.85 \times 4 \times 12}$$

$$= 3.84 \text{ in}$$

$$c = \frac{a}{\beta_1} = \frac{3.84}{0.85} = 4.51 \text{ in}$$

$f'_c = 4 \text{ ksi}$, $\beta_1 = 0.85$

Now,

$$\epsilon_t = 0.003 \times \left(\frac{d-c}{c} \right)$$

$$= 0.003 \times \left(\frac{17.5 - 4.51}{4.51} \right)$$

$$= 0.008 > 0.005$$

∴ Tension controlled. $\phi = 0.9$

∴ Ultimate moment capacity of beam,

$$M_u = \phi \cdot M_n$$

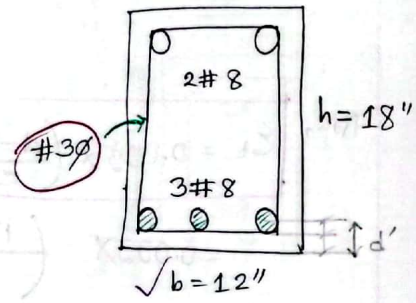
$$M_u = \phi \cdot A_s f_y \left(d - \frac{a}{2} \right)$$

$$= 0.9 \times 2.37 \times 60 \times \left(17.5 - \frac{3.84}{2} \right)$$

$$= 1994 \text{ kip-inch (Ans)}$$

23
B/WTA-2019

Calculate ultimate load carrying capacity of beam.
 $f'_c = 4 \text{ ksi}$, $f_y = 60 \text{ ksi}$, clear cover = 1.5".



Hints: d' ना बल clear cover देना आवश्यक, $d = h - d'$
 $d = h - (c.\text{cover} + 1 \text{ inch})$

Soln: $d = h - (\text{clear cover} + 1 \text{ inch})$
 $= 18" - (1.5 + 1)$
 $= 15.5"$

Single reinforced beam
 $A_s = 3 \times 0.79 = 2.37 \text{ in}^2$ (Let, singly rein.)

check-01%

$$\rho = \frac{A_s}{bd} = \frac{2.37}{12 \times 15.5} = 0.0127 > \rho_{\min} (0.0033)$$

So, singly reinforced beam.

check-02%

Now, $a = \frac{A_s f_y}{0.85 f'_c b}$
 $= \frac{2.37 \times 60}{0.85 \times 4 \times 12} = 3.485"$

$$c = \frac{a}{\beta_1} = \frac{3.485}{0.85} = 4.1 \quad \beta_1 = 0.85$$

$$\epsilon_t = 0.003 \times \left(\frac{d-c}{c} \right)$$

$$= 0.003 \times \left(\frac{15.5 - 4.1}{4.1} \right) = 0.0083 > 0.005$$

$\therefore \phi = 0.9$ \therefore Tension controlled.

ultimate load carrying capacity,

$$M_u = \phi \cdot A_s f_y \left(d - \frac{a}{2} \right)$$

$$= 0.9 \times 2.37 \times 60 \times \left(15.5 - \frac{3.485}{2} \right)$$

$$= 1760.68 \text{ kip-inch} \quad (Am)$$

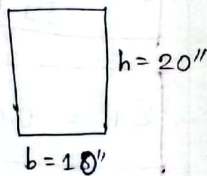
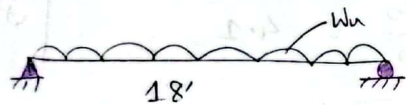
Span Length and Support type Given

20

26th Dec

A rectangular beam that must carry a service live load of 2.47 kip/ft and a calculated dead load of 1.05 kip/ft on an 18 ft simple span which is limited in cross section to 10" width and 20" total depth. If $f_y = 60000$ psi and $f'_c = 4000$ psi, Estimate the steel requirement.

Soln:



$$LL = 2.47 \text{ kip/ft}$$

$$DL = 1.05 \text{ kip/ft}$$

$$A_s = ?$$

$$d = h - c.e.$$

$$= 20 - 1.5 = 18.5 \text{ inch}$$

Assume,

Clear cover for beam = 1.5 inch

Factored load, $W_u = 1.2 DL + 1.6 LL$

$$= 1.2 \times 1.05 + 1.6 \times 2.47$$

$$= 5.21 \text{ K/ft}$$

Factored moment, $M_u = \frac{W_u L^2}{8}$ (Simple with UDL)

$$= \frac{5.21 \times 18^2}{8}$$

$$= 211.08 \text{ K-ft}$$

$$\therefore M_u = 2533 \text{ K-in}$$

Now, $a = \frac{A_s f_y}{0.85 f'_c b} = \frac{A_s \times 60}{0.85 \times 4 \times 10} = 1.765 A_s$

$M_u = \phi \cdot A_s f_y \left(d - \frac{a}{2} \right)$

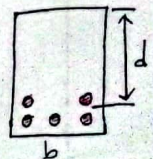
$$\Rightarrow 211.08 \times 12 = 0.9 \times A_s \times 60 \times \left(18.5 - \frac{1.765 A_s}{2} \right)$$

$$\therefore A_s = 2.95 \text{ in}^2$$

Use #7 ϕ as main bar. ($A_b = 0.60 \text{ in}^2$)

$$\therefore \text{no of bar} = \frac{2.95}{0.60} = 4.9 \approx 5 \text{ Nos.}$$

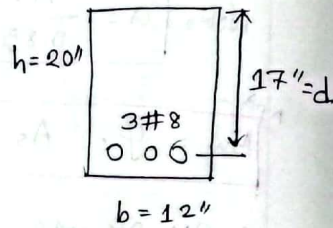
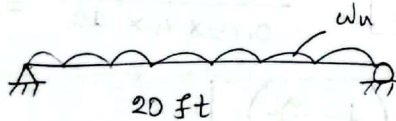
Use 5 #7 ϕ bar (Am.)



Live load = Service load

24 * A simply supported beam has a span of 20 ft. If the cross section of the beam is as shown figure, $f'_c = 3 \text{ ksi}$ and $f_y = 60 \text{ ksi}$. Determine the allowable uniformly distributed service load in the beam assuming the dead load is that due to beam weight. Given $b = 12 \text{ in}$, $d = 17 \text{ in}$, total depth $h = 20 \text{ in}$ and reinforced with 3#8 bars. ($A_s = 2.37 \text{ in}^2$).

Soln:



DL = Self weight

LL = Service load = ?
(Live load)

concrete
Dead load, $DL = \gamma_{conc} \times (bh)$

$$= 150 \times \left(\frac{12}{12} \times \frac{20}{12} \right)$$

$$\therefore DL = 250 \text{ lb/ft} = 0.25 \text{ k/ft}$$

$$\gamma_{conc} = 150 \text{ lb/ft}^3 = 150 \text{ pcf}$$

$$DL_{conc} = \gamma_{conc} \times bh \quad | \quad b, h \rightarrow \text{ft}$$

$$= 150 \times bh$$

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

$$= \frac{2.37 \times 60}{0.85 \times 3 \times 12} = 4.64 \text{ inch}$$

$$M_u = \phi A_s f_y \left(d - \frac{a}{2} \right)$$

$$= 0.9 \times 2.37 \times 60 \times \left(17'' - \frac{4.64}{2} \right)$$

$$= 1875 \text{ K-in}$$

$$= 156.5 \text{ K-ft}$$

And,

$$M_u = \frac{W_u l^2}{8} \quad (\text{simple \& UDL})$$

$$\Rightarrow 156.5 = \frac{W_u \times 20^2}{8}$$

$$\therefore W_u = 3.13 \text{ k/ft}$$

Again,

$$W_u = 1.2 DL + 1.6 LL$$

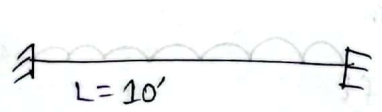
$$\Rightarrow 3.13 = 1.2 \times 0.25 + 1.6 \times LL$$

$$\therefore LL = 1.769 \text{ k/ft} \quad (\text{Ans})$$

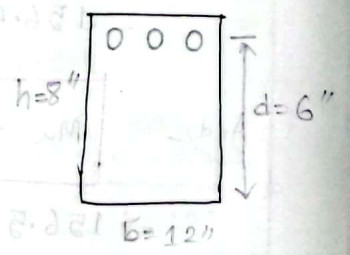
BB AD-2018

Q. The span of a beam 10' long and ends are fixed. The maximum load controlled by the capacity in the negative moment region. Based on the flexural requirements, what is the maximum uniform live load the beam can carry? ($f_y = 40 \text{ ksi}$, $f'_c = 3.5 \text{ ksi}$, $M_n = 140.2 \text{ K-in}$, $A_s = 0.62 \text{ in}^2$.)

Soln:



Negative Moment.



Max^m Live load, $LL = ?$

DL = Self weight

$$DL_{(conc)} = \gamma_{conc} \times (b h)$$

$$= 150 \times \left(\frac{12}{12} \times \frac{8}{12} \right)$$

$$= 100 \text{ lb/ft}$$

$$= 0.1 \text{ K/ft}$$

$\gamma_{conc} = 150 \text{ lb/ft}^3$

Hints:
 Min. tension ratio,
 A 733 204 20174 20,

$$M_u = \phi \cdot M_n$$

$$= 0.9 \times 140.2$$

$$= 126.18 \text{ Kip-in} = 10.515 \text{ Kip-ft}$$

Forc Both End fixed, (UDL) $M_u = \frac{W_u L^2}{12}$

$$\Rightarrow 126.18 = \Rightarrow 10.515 = \frac{W_u \times 10^2}{12}$$

$$\therefore W_u = 1.262 \text{ K/ft}$$

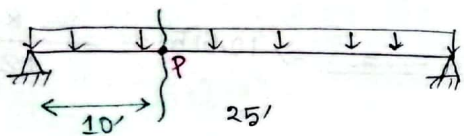
$$\text{Now, } W_u = 1.2 \text{ DL} + 1.6 \text{ LL}$$

$$\Rightarrow 1.262 = 1.2 \times 0.1 + 1.6 \times \text{LL}$$

$$\Rightarrow \text{LL} = 0.71375 \text{ K/ft}$$

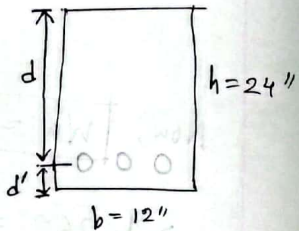
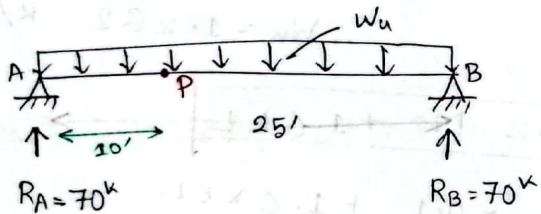
$$\therefore \text{LL} = 713.75 \text{ lb/ft} \checkmark \text{ (Ans.)}$$

A rectangular beam of section 12" x 24" carries DL = 2 K/ft and LL = 2 K/ft. If the value of $f'_c = 4 \text{ Ksi}$, $f_y = 60 \text{ Ksi}$, $d' = 1.5"$, Design the reinforcement at 10' from the support for the following beam.



$A_s = ?$

Soln:



$$W_u = 1.2 \text{ DL} + 1.6 \text{ LL}$$

$$= 1.2 \times 2 + 1.6 \times 2$$

$$= 5.6 \text{ K/ft}$$

$$\therefore \text{Total Load} = 5.6 \text{ (K/ft)} \times 25' = 140 \text{ K}$$

$$d = h - d'$$

$$= 24 - 1.5$$

$$= 22.5"$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{A_s \times 60}{0.85 \times 4 \times 12} = \frac{1.47 A_s}{1} \quad (i)$$

Moment at point "P" on Left Side,

$$M_p = 70 \times 10 - 5.6 \times 10 \times \frac{10}{2}$$

$$= 420 \text{ K-ft}$$

$$= 420 \times 12 \text{ Kip-inch}$$

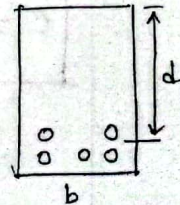
$$= M_u$$

$$\text{Now, } M_u = \phi A_s f_y \left(d - \frac{a}{2} \right)$$

$$\Rightarrow 420 \times 12 = 0.9 \times A_s \times 60 \times \left(22.5 - \frac{1.47 A_s}{2} \right)$$

$$\therefore A_s = 4.95 \text{ in}^2$$

Use 5#9 ϕ bars as main bars.



Cantilever Beam

22
PACB-2019*

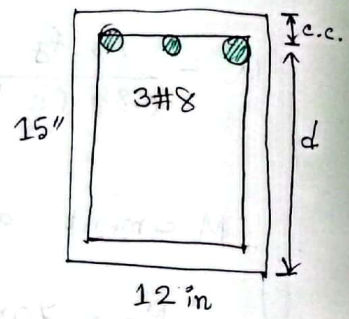
Determine the flexural moment capacity of the cantilever beam having 15" depth and concrete clear cover 2.5 inch. Given $f'_c = 4 \text{ ksi}$, $f_y = 60 \text{ ksi}$.

Soln:

$$d = h - \text{c.c.}$$

$$= 15 - 2.5$$

$$= 12.5''$$



$$\sqrt{A_s} = 3 \times 0.79$$

$$= 2.37 \text{ in}^2$$

check-01:

$$\rho = \frac{A_s}{bd} = \frac{2.37}{12 \times 12.5} = 0.0158 > \rho_{\min} (0.0033)$$

So, singly reinforced beam.

check-02:

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{2.37 \times 60}{0.85 \times 4 \times 12} = 3.49''$$

$$c = \frac{a}{\beta_1} = \frac{3.49}{0.85} = 4.11 \quad \left| \beta = 0.85 \right.$$

$$\epsilon_t = 0.003 \times \left(\frac{d-c}{c} \right)$$

$$= 0.003 \times \left(\frac{12.5 - 4.11}{4.11} \right)$$

$$= 0.0061 > 0.005 \quad \text{[Tension Controlled]}$$

$$\therefore \phi = 0.9$$

Now, Flexural moment capacity of beam,

$$M_u = \phi \cdot M_n$$

$$M_u = \phi \cdot A_s f_y \left(d - \frac{a}{2} \right)$$

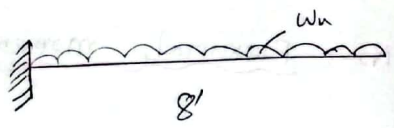
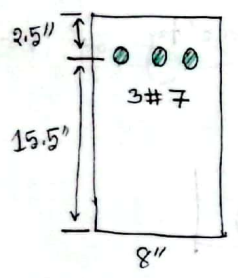
$$= 0.9 \times 2.37 \times 60 \times \left(12.5 - \frac{3.49}{2} \right)$$

$$= 1376.42 \text{ kip-in} \quad (\text{Ans.})$$

BUET BIC - 2017
EGCB - 2020

22)

A 8' span cantilever beam has a rectangular section and reinforcement as shown in figure. The beam carries a dead load including its own weight of 1.5 k/ft and live load of 0.9 k/ft. Using $f'_c = 4 \text{ ksi}$, $f_y = 60 \text{ ksi}$, check if the beam is safe to carry above loads.



DL = 1.5 k/ft
LL = 0.9 k/ft

Soln:

Factored load,
$$W_u = 1.2 DL + 1.6 LL$$

$$= 1.2 \times 1.5 + 1.6 \times 0.9$$

$$= 3.24 \text{ k/ft}$$

Factored moment, (Cantilever beam),
$$M_{uF} = \frac{W_u L^2}{2}$$

$$= \frac{3.24 \times 8^2}{2}$$

$$= 103.68 \text{ K-ft}$$

27) (3/1/20)

$$A_s = 3 \times 0.6 = 1.8 \text{ in}^2$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{1.8 \times 60}{0.85 \times 4 \times 8} = 3.97 \text{ in}$$

$$c = \frac{a}{\beta_1} = \frac{3.97}{0.85} = 4.67$$

$$\epsilon_t = 0.003 \times \left(\frac{d-c}{c} \right)$$

$$= 0.003 \times \left(\frac{15.5 - 4.67}{4.67} \right) = 0.007 > 0.005$$

$$\therefore \phi = 0.9$$

$$M_u = \phi \cdot A_s f_y \left(d - \frac{a}{2} \right)$$

$$= 0.9 \times 1.8 \times 60 \times \left(15.5 - \frac{3.97}{2} \right)$$

$$= 1313.66 \text{ Kip-in}$$

$$= 109.47 \text{ Kip-ft}$$

$$\therefore M_u > M_{uF} \text{ (OK)}$$

So, The beam is safe to carry the above loads. Safe in flexure. (Ami)

Concepts : "USD"

① $\gamma_D = 1.2$, $\gamma_L = 1.6$ प्रयोग आयात,

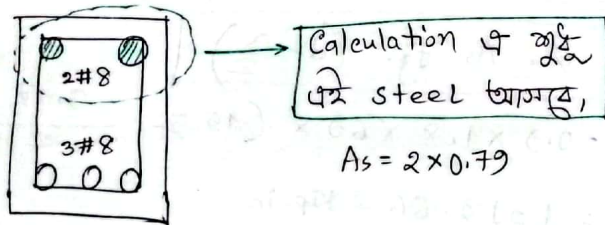
$$W_u = \gamma_D \times DL + \gamma_L \times LL$$

$$= 1.2 DL + 1.6 LL$$

② यदि $\epsilon_t < 0.005$ तब,

$$\phi = 83.3 \epsilon_t + 0.483$$

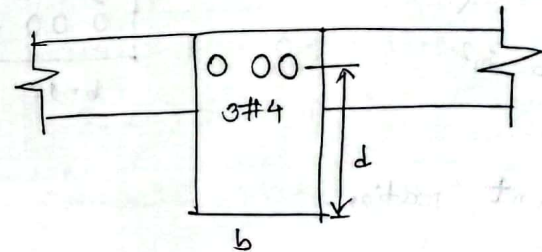
③ Cantilever Beam का आकार,



As the beam is cantilever beam,
Only top reinforcement will used
in determining flexural moment capacity.

④ $M_u =$ Flexural strength
 $=$ Flexural moment capacity
 $=$ Ultimate moment capacity
 $=$ Ultimate Load carrying capacity

⑤ दिए गए का आकार,
 एक बिकल्प section द्वारा Cantilever beam
 का आकार निम्न शब्द,

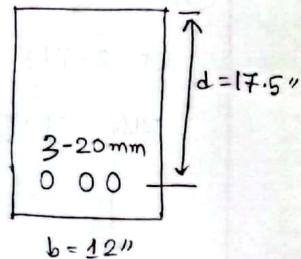


Beam : WSD \rightarrow Analysis

A rectangular beam has a width of 12" and effective depth to centroid of the reinforcing steel of 17.5". It is reinforced with 3-20 mm bars in one row. Find out the allowable working moment that can be resisted by the section if $f_s = 24000$ psi, $f_c = 12000$ psi.

soln:

$$A_s = 3 \times \frac{\pi}{4} \times \left(\frac{20}{25.4}\right)^2 = 1.46 \text{ in}^2$$



Reinforcement ratio,

$$\rho = \frac{A_s}{bd} = \frac{1.46}{12 \times 17.5} = 0.00695$$

Let, Modular Ratio, $n = 9$.

$$k = \sqrt{2\rho n + (\rho n)^2} - \rho n$$
$$= \sqrt{2 \times 0.00695 \times 9 + (0.00695 \times 9)^2} - (0.00695 \times 9)$$
$$= 0.297$$

$$j = 1 - \frac{k}{3} = 0.90$$

If steel stress controls,

$$M_s = A_s f_s j d$$

$$= 1.46 \times 24 \times 0.9 \times 17.5 = 552.75 \text{ kip-in}$$

If concrete stress controls,

$$M_c = \frac{1}{2} f_c k j b d^2$$

$$= \frac{1}{2} \times 1.2 \times 0.297 \times 0.90 \times 12 \times 17.5^2 = 589.4 \text{ kip-in}$$

\therefore Allowable working moment, $M_{\text{Allow}} = 552.75 \text{ k-in}$

Minimum ρ

(Am)

WASA-2017
PGCL-2017
MPA-2019

57

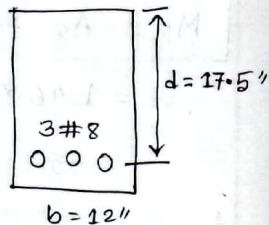
* A rectangular beam has a width of 12" and an effective depth to the centroid of the reinforcing steel of 17.5". It is reinforced with Three No. 8 bars in one row. Find out the Maximum working moment that can be resisted by the section.

$f'_c = 4000 \text{ psi}$, $f_y = 60000 \text{ psi}$.

Solⁿ: $M_{max} = M_{allow} = ?$

$f_c = 0.45 f'_c$
 $= 0.45 \times 4000$
 $= 1800 \text{ psi}$

$f_s = 0.40 f_y$
 $= 0.40 \times 60000$
 $= 24000 \text{ psi}$



$A_s = 3 \times 0.79 = 2.37 \text{ in}^2$

$\rho = \frac{A_s}{bd} = \frac{2.37}{12 \times 17.5} = 0.0112$

$K = \sqrt{(\rho n)^2 + 2 \rho n} - \rho n$

$= \sqrt{(0.0112 \times 9)^2 + 2 \times (0.0112 \times 9)} - (0.0112 \times 9)$

$= 0.359$

OKet, $n = 9$

$j = 1 - \frac{K}{3} = 0.88$

If steel stress controls,

$M_s = A_s f_s j d = 2.37 \times 24 \times 0.88 \times 17.5$
 $= 875.95 \text{ kip-inch}$

If concrete stress controls,

$M_c = \frac{1}{2} f_c K j b d^2 = \frac{1}{2} \times 1.8 \times 0.359 \times 0.88 \times 12 \times 17.5^2$
 $= 1044.9 \text{ kip-inch}$

$\therefore M_{max} = M_{allow} = 875.95 \text{ kip-in (Am)}$
 (minimum τ)

Beam: WSD → Design

62

BKB-2018

A rectangular beam of 12" x 24" section must carry a uniformly distributed live load 680 p/f and support the dead load of a wall weighting 380 p/f in addition to its own self weight, on a simple span of 24 ft. Find out the area of steel required, using intermediate grade steel at a working stress of 20000 psi and 3000 psi concrete at a working stress of 1350 psi. $d' = 2.5"$.

Soln: $LL = 680 \text{ lb/ft}$

$DL = 380 \text{ lb/ft} + \text{Self wt.}$
(Wall weight)

$\text{Self wt.} = \gamma_{\text{conc}} \times (bh)$

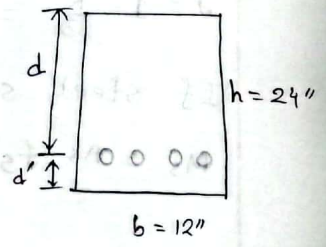
$= 150 \times \left(\frac{12}{12} \times \frac{24}{12}\right)$

$= 300 \text{ lb/ft}$

$\therefore DL = 380 + 300 = 680 \text{ lb/ft}$

WSD method, $W = DL + LL$

$= 680 + 680 = 1360 \text{ lb/ft}$



$d = h - d' = 21.5"$

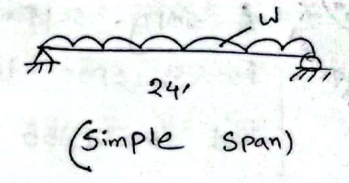
Given,
 $f_s = 20000 \text{ psi}$ $f_c = 0.45 \times f_c'$
 $f_c' = 3000 \text{ psi}$ $f_c = 1350 \text{ psi}$

Moment, $M = \frac{wL^2}{8}$

$\Rightarrow M = \frac{1360 \times 24^2}{8}$

$= 97920 \text{ lb-ft}$

$= 97.92 \text{ Kip-ft}$



Let, $n = 9$ (Modular Ratio)

$r = \frac{f_s}{f_c} = \frac{20000}{1350} = 14.81$

$K = \frac{n}{n+r} = \frac{9}{9+14.81} = 0.378$

$j = 1 - \frac{K}{3} = 0.874$

Moment, $M = A_s f_s j d$

$\Rightarrow 97.92 \times 12 = A_s \times 20 \times 0.874 \times 21.5$
(KIP-in)

$\therefore A_s = 3.13 \text{ in}^2$

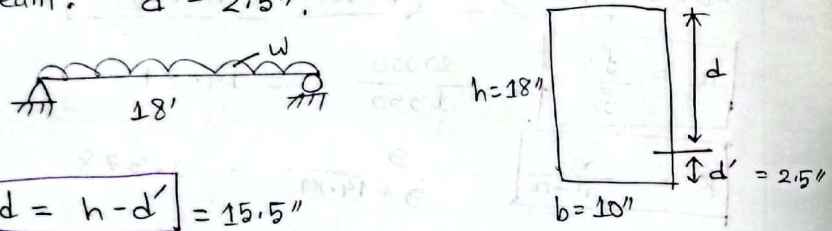
Use, $4\#8 \phi$ bars

$A_s(\text{provided}) = 3.16 \text{ in}^2$ (OK) (Am.)

Q.50 *
BHP-2017

A simply supported beam of 10" x 18" section is a span length of 18' and distributed load is 1355 p/f in addition of its self wt. If $f'_c = 3000$ psi and $f_s = 20000$ psi, then calculate the required steel and draw the long and cross section of beam. $d' = 2.5"$.

Soln.



$$d = h - d' = 15.5"$$

Given, $W = \text{Distributed load} + \text{Self wt.}$

$$\begin{aligned} \text{Self wt.} &= \gamma_{\text{conc}} \times (bh) \\ &= 150 \times \left(\frac{10}{12} \times \frac{18}{12}\right) \\ &= 187.5 \text{ lb/ft} \end{aligned}$$

$\gamma_{\text{conc}} = 150 \frac{\text{lb}}{\text{ft}^3}$

$$\begin{aligned} \therefore W &= 1355 + 18.7 \\ &= 1542.5 \text{ lb/ft} \\ &= 1.54 \text{ K/ft} \end{aligned}$$

$$\begin{aligned} f_c &= 0.45 f'_c \\ &= 0.45 \times 3000 = 1350 \text{ psi} \end{aligned}$$

$$\begin{aligned} \text{Moment, } M &= \frac{wL^2}{8} \\ &= \frac{1.54 \times 18^2}{8} \\ &= 62.47 \text{ K-ft} \end{aligned}$$

Let, $n = 9$

$$r = \frac{f_s}{f_c} = \frac{20000}{1350} = 14.81$$

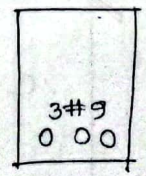
$$K = \frac{n}{n+r} = \frac{9}{9+14.81} = 0.37$$

$$j = 1 - \frac{K}{3} = 0.87$$

$$\begin{aligned} \text{Moment, } M &= A_s f_s j d \\ \Rightarrow 62.47 \times 12 &= A_s \times 20 \times 0.87 \times 15.5 \\ \Rightarrow A_s &= 2.78 \text{ in}^2 \end{aligned}$$

Use, $3\#9$ bars all in a row

$$A_s (\text{provide}) = 3 \times 1 = 3 \text{ in}^2 \quad (\text{OK})$$



(Am)

T-Beam

APRIL-2017
BPPB-2018

Determine the ultimate moment capacity of the

T-beam, $f'_c = 3 \text{ ksi}$, $f_y = 40 \text{ ksi}$.

Given,
Soln: $b_w = 10''$

$b_e = 30''$

$h_f = 3''$

$$d = d_t - \frac{2.5''}{2}$$

$$d_t = 20'' + 3'' - 2.5'' = 20.5''$$

$$d = 19.25''$$

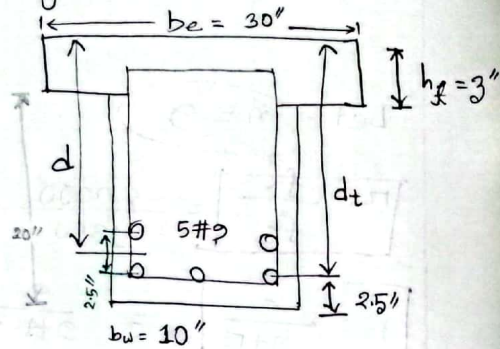
$$A_s = 5 \times 1 \text{ in}^2 = 5 \text{ in}^2$$

$$a = \frac{A_s f_y}{0.85 f'_c b_e} = \frac{5 \times 40}{0.85 \times 3 \times 30} = 2.61''$$

$\therefore a < h_f$, so rectangular section analysis

$$c = \frac{a}{\beta_1} = \frac{2.61}{0.85} = 3.07$$

$$\left[\beta_1 = 0.85 \text{ when } f'_c = 3 \text{ ksi} \right]$$



$$\epsilon_t = 0.003 \times \left(\frac{d_t - c}{c} \right)$$

$$= 0.003 \times \left(\frac{20.5 - 3.07}{3.07} \right)$$

$$= 0.017 > 0.005$$

$\therefore \phi = 0.9$, Tension controlled section.

Now, Ultimate moment capacity,

$$M_u = \phi \cdot M_n$$

$$= \phi \cdot A_s f_y \left(d - \frac{a}{2} \right)$$

$$= 0.9 \times 5 \times 40 \times \left(19.25 - \frac{2.61}{2} \right)$$

$$= 3230.1 \text{ Kip-inch}$$

(Am.)

Hints: T-Beam এর ক্ষেত্রে:

ϵ_t হিসাব করতে d_t ধরা হবে.

M_u বা M_n হিসাব করতে d ধরা হবে.

PGCB-2018

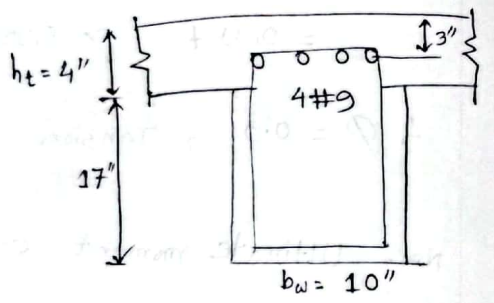
Calculate the negative nominal moment of the beam. Clear distance between the webs is 36 in and span of the beam is 20'. $f_y = 60 \text{ Ksi}$, $f'_c = 5 \text{ Ksi}$,

$\beta_1 = 0.8$,

$A_s = 4 \times 1 \text{ in}^2$
 $= 4 \text{ in}^2$

$M_n = ?$

$d = 17 + 4 - 3 = 18"$



Solⁿ: "When T beams are resisting negative moments, their flanges will be in tension and the bottom will be in compression. For this situation,

The rectangular beam design formula will be used."

$$a = \frac{A_s f_y}{0.85 f'_c b_w} = \frac{4 \times 60}{0.85 \times 5 \times 10} = 5.64"$$

$M_n = A_s f_y \left(d - \frac{a}{2} \right)$

$= 4 \times 60 \times \left(18" - \frac{5.64}{2} \right)$

$= 3643.2 \text{ kip-in}$

(Ans)

Hints : T-beam

① Positive moment = reinforcement तल्ले
 Negative moment = reinforcement उल्ले

- ② Span length (L) ३ clear distance of web लुअर शक्य.
 (i) $b_e = b_w + 16 h_t$
 (ii) $b_e = \text{clear distance of web} + b_w$
 (iii) $b_e = \frac{L}{4} \text{ (inch)}$

५ उल्ले शक्य minimum ले शक्य,
 ५ लुअर लुअर नो शक्य,
 $b_e = b_w + 16 h_t$ शक्य,

(P.T.O)

66
GTCL-2018

* A reinforced concrete T-beam has a cross section in the positive region of span as shown below. The beam has a 30' span and is cast integrally with a floor slab that is 4" thick. The clear distance between webs is 18". Determine the design positive moment capacity. $f'_c = 3000$ psi, $f_y = 60000$ psi.

Soln:

$h_t = 4'' = \text{slab thickness}$

$b_w = 12''$

$b_e = 9'' + 12'' + 9''$

$b_e = 30''$ (flange calculation)

$L = 30'$

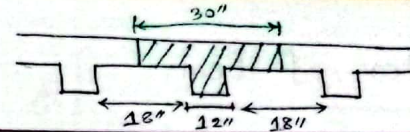
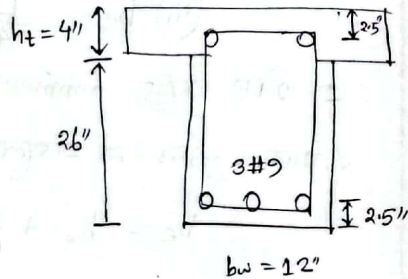
Now,

minimum b_e

(i) $b_e = b_w + 16 \times h_t = 27.6$ inch

(ii) $b_e = \text{clear distance of web} + b_w = 30$ inch

(iii) $b_e = \frac{L}{4} = \frac{30 \times 12}{4} = 90$ inch



$A_s = 3 \times 1 \text{ in}^2 = 3 \text{ in}^2$

$a = \frac{A_s f_y}{0.85 f'_c b_e} = \frac{3 \times 60}{0.85 \times 3 \times 30} = 2.35''$

$a < h_t$, so rectangular section analysis.

$c = \frac{a}{\beta_1} = \frac{2.35}{0.85} = 2.76''$

$d = 26 + 4 - 2.5 = 27.5''$

$\epsilon_t = 0.003 \times \left(\frac{d-c}{c} \right) = 0.003 \times \left(\frac{27.5-2.76}{2.76} \right) = 0.026 > 0.005$

$\therefore \phi = 0.9$

$M_u = \phi \cdot A_s f_y \left(d - \frac{a}{2} \right) = 0.9 \times 3 \times 60 \times \left(27.5 - \frac{2.35}{2} \right) = 4264.65 \text{ K-in (Ans)}$

Shear of Beam

Concepts: Forc Beam →

$$V_n = V_s + V_c$$

$$V_u = \phi \cdot V_n$$

$$\Rightarrow V_u = \phi \cdot (V_s + V_c)$$

$$\Rightarrow V_u = \phi \cdot V_s + \phi \cdot V_c$$

$$\phi \cdot V_s = V_u - \phi \cdot V_c$$

$$V_s = \frac{A_v \cdot f_y \cdot d}{S}$$

$$\phi \cdot \frac{A_v \cdot f_y \cdot d}{S} = V_u - \phi \cdot V_c$$

$$\Rightarrow S = \frac{\phi \cdot A_v \cdot f_y \cdot d}{V_u - \phi \cdot V_c}$$

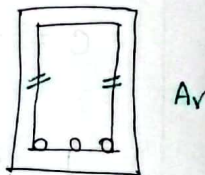
A beam having $b=12''$ and $d=13.5''$ is reinforced with 4 Nos. 8 bars in a row as main reinforcement and #3 bars 6 inch e/c spacing as shear reinforcement. Considering $f_c = 3 \text{ Ksi}$, $f_y = 60 \text{ Ksi}$. Check if the section is tension controlled or not and determine the allowable shear strength.

$\lambda = 1$ for shear

$\phi = 0.75$ for shear

$A_v =$ Shear Bar Area

$S =$ Spacing of Tie bar



Sol^{no}:

Spacing $S = 6''$

$$A_s = 4 \times 0.79 = 3.16 \text{ in}^2$$

$$A_v = 2 \times \frac{\pi}{4} \left(\frac{3}{8}\right)^2 = 0.22 \text{ in}^2$$

$$a = \frac{A_s \cdot f_y}{0.85 f_c \cdot b}$$

$$= \frac{3.16 \times 60}{0.85 \times 3 \times 12} = 6.19''$$

$$c = \frac{a}{\beta_1} = \frac{6.19}{0.85} = 7.28''$$

$$\epsilon_t = 0.003 \times \left(\frac{d-c}{c}\right) = 0.003 \left(\frac{13.5-7.28}{7.28}\right) = 0.0026 < 0.004$$

So, The section is not Tension controlled.

Now, Allowable shear strength,

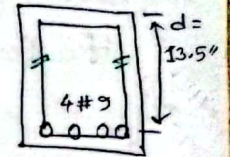
$$V_u = \phi \cdot V_s + \phi \cdot V_c$$

$$V_u = \phi \cdot \frac{A_v \cdot f_y \cdot d}{S} + \phi \cdot \frac{2 \lambda \sqrt{f_c} b d}{1000}$$

$$= 0.75 \times \frac{0.22 \times 60 \times 13.5}{6} + \frac{0.75 \times 2 \times 1 \times \sqrt{3000} \times 12 \times 13.5}{1000}$$

(kip) (kip)

$$\therefore V_u = 35.57 \text{ Kips} \quad (\text{Ans})$$



$b = 12''$

(#3 bars)

80

* EGCB-2015

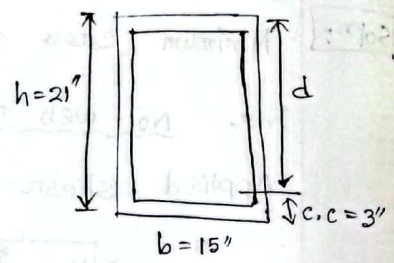
Design stirrup of a beam which size is 21" x 15" and 10 # 10 bar used in this beam. Given, $V_u = 40000$ lb, $f_y = 60$ ksi, $f'_c = 4$ ksi and clear cover is 3 inch.

Soln:

$A_s = 10 \# 10$ bar

$d = 21 - 3 = 18''$

$V_u = 40000$ lb



$$\phi \cdot V_c = \phi \cdot 2 \lambda \sqrt{f'_c} b d$$

$$= 0.75 \times 2 \times 1 \times \sqrt{4000} \times 15 \times 18$$

$$= 25615 \text{ lb}$$

$\therefore V_u > \phi \cdot V_c$, so stirrup needed.

Provide, 10 mm bar as stirrup.

Spacing, $S = \frac{d}{2} = \frac{18}{2} = 9''$

Use 10 mm bar @ 9" c/c.

(Am)

Hints: यह बार 2/4 रखा.

X

concept :

	Area (A_s)
✓ # 3 ϕ = 10 mm bar	0.11 in ²
✓ # 4 ϕ → 12 mm bar	0.20 in ²
✓ # 5 ϕ → 16 mm bar	0.31 in ²
# 6 ϕ → 20 mm bar	0.44 in ²
✓ # 7 ϕ → 22 mm bar	0.60 in ²
✓ # 8 ϕ → 25 mm bar	0.79 in ²
# 9 ϕ = 28 mm bar	1.00 in ²
# 10 ϕ = 32 mm bar	1.27 in ²
# 11 ϕ = 36 mm bar	1.56 in ²

Slab:

Two Designers

*** PGCB-2021
 A designer designed 6" one way slab with 60 grade f_{y1} reinforcement. He used #4 bar as main reinforcement and provided spacing at 5 in c/c. A second designer designs the slab with 500W reinforcement instead of 60 grade reinforcement and used 12mm bar as main rebar instead of #4 bar. What should be the spacing of slab by the second designer?

Soln: 1st Designer:

$f_{y1} = 60 \text{ Grade} = 60 \text{ Ksi} = f_{y1}$

Slab thickness, $t = 6"$

$A_{b1} = \frac{\pi}{4} \times \left(\frac{4}{8}\right)^2 = 0.20 \text{ in}^2$ [#4 bar] $A_{b1} = \frac{\pi}{4} \times d_1^2$

Spacing, $S_1 = 5"$ c/c $A_s = 2$

Now, Spacing, $S_1 = \frac{A_{b1}}{A_{s1}} \times 12$

$\Rightarrow A_{s1} = \frac{A_{b1}}{S_1} \times 12$

$\Rightarrow A_{s1} = \frac{0.20}{5} \times 12 = 0.48 \text{ in}^2 = A_{s1}$

#4 = $\frac{4}{8}"$

#6 = $\frac{6}{8}" = \frac{3}{4}"$

1 MPa = 145 psi
1 MPa = 0.145 Ksi

1 in = 25.4mm = 2.54 cm

2nd Designer:

$f_{y2} = 500 \text{ W} = 500 \text{ MPa} = 500 \times 0.145 \text{ Ksi}$
 $= 72.5 \text{ Ksi} = f_{y2}$

$A_{b2} = \frac{\pi}{4} \times \left(\frac{12}{25.4}\right)^2 = 0.175 \text{ in}^2$ [12mm bar]

Now, $A_{s1} \times f_{y1} = A_{s2} \times f_{y2}$

$\Rightarrow 0.48 \times 60 = A_{s2} \times 72.5$

$\therefore A_{s2} = 0.397 \text{ in}^2 = A_{s2}$

Spacing, $S_2 = \frac{A_{b2}}{A_{s2}} \times 12$

$= \frac{0.175}{0.397} \times 12$

$= 5.3 \text{ inch}$

$= 5 \text{ inch (say)}$

\therefore Spacing, $S = 5"$ c/c

Round or Half Round
 इन्च याद, कि
 कमाए किड शड
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 (Am)

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BEIC-2019
BAPC-2020

A designer designed a 6" one way slab with 60 grade reinforcement. He used #4 bar as main reinforcement and provided spacing at 4.5 inch c/c. A second designer designs the slab with 500W reinforcement instead of 60 grade reinforcement and used 12 mm bar as main rebar instead of #4 bar. What would be the spacing of slab by the second designer?

Soln: For First Designer:

Slab thickness, $t = 6"$

$$f_{y1} = 60 \text{ grade} = 60 \text{ Ksi} = f_{y1}$$

$$A_{b1} = \#4 \text{ bar} = \frac{\pi}{4} \times \left(\frac{4}{8}\right)^2 = 0.196 \text{ in}^2$$

$$\text{Spacing, } S_1 = 4.5" \text{ c/c}$$

$$\text{Spacing, } S_1 = \frac{A_{b1}}{A_{s1}} \times 12$$

$$\Rightarrow 4.5 = \frac{0.196}{A_s} \times 12$$

$$\therefore A_{s1} = 0.523 \text{ in}^2 = A_{s1}$$

$$A_{b1} = \frac{\pi}{4} d_1^2$$

For Second Designer:

$$f_{y2} = 500 \text{ W} = 500 \text{ MPa} = 500 \times 0.145 \text{ Ksi} = 72.5 \text{ Ksi} = f_{y2}$$

$$A_{b2} = 12 \text{ mm bar} = \frac{\pi}{4} \times \left(\frac{12}{25.4}\right)^2 = 0.175 \text{ in}^2$$

Spacing, $S = ?$

$$\text{Now, } A_{s1} \times f_{y1} = A_{s2} \times f_{y2}$$

$$\Rightarrow 0.523 \times 60 = A_{s2} \times 72.5$$

$$\therefore A_{s2} = 0.433 \text{ in}^2 = A_{s2}$$

$$\text{And, } \text{Spacing, } S_2 = \frac{A_{b2}}{A_{s2}} \times 12$$

$$= \frac{0.175}{0.433} \times 12$$

$$= 4.85 \text{ inch}$$

$$= 4.50 \text{ inch c/c (say)} \quad (\text{Am.})$$

Hints: For Spacing \rightarrow

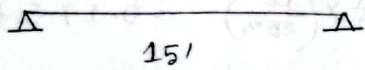
- * Spacing = 5.3" আসলে, $s = 5"$ ঠিক c/c
- = 4.85" আসলে, $s = 4.5"$ " c/c
- = 6.47" আসলে, $s = 6"$ ঠিক c/c,

Temperature & shrinkage reinforcement

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LGD-2018
SI BMA

Determine the slab thickness and spacing required for temperature and shrinkage reinforcement of one way slab, considering $f_y = 60 \text{ Ksi}$.



Soln. Span length, $L = 15' = 15 \times 12 \text{ inch}$ [Note: $f_y \rightarrow \frac{\text{Kip}}{\text{in}^2}$]

Slab thickness, $t = ?$

Spacing, $S = ?$

Assume, $b = 12''$

For simply supported,

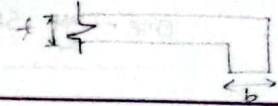
(i)
$$\boxed{\text{Slab thickness, } t = \frac{L}{20}}$$

$$= \frac{15 \times 12}{20}$$

$$= 9 \text{ inch}$$



$$\begin{cases} b = 12'' \\ d = 10 \text{ mm} \end{cases}$$



(ii) Shrinkage and Temperature steel on Transverse direction.

Minimum steel area for slab,

$$\boxed{A_s = 0.0018 \times b \times t}$$

$$= 0.0018 \times 12 \times 9$$

$$= 0.1944 \text{ in}^2$$

$$t = \frac{L}{20}$$

Using 10 mm bar , $A_b = \frac{\pi}{4} \times \left(\frac{10}{25.4}\right)^2$
(for temp. & shrinkage reinf.)

$$= 0.11 \text{ in}^2$$

$$\therefore \boxed{\text{Spacing, } S = \frac{A_b}{A_s} \times 12}$$

$$= \frac{0.11}{0.1944} \times 12$$

$$= 6.79 \text{ in c/c}$$

$$= 6.50 \text{ inch c/c (say)}$$

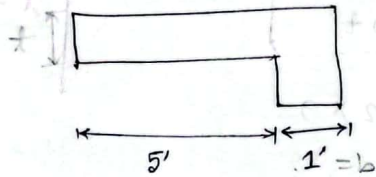
So, Provide 10 mm bar @ 6.5" c/c.

(Am)

One-way Slab Design

* BB AD-2018

A one-way slab is supported on RC beam as shown. The working live load is 100 psf. Design the slab. Given, $f_y = 60$ Ksi, $f'_c = 4$ Ksi.



Soln: Span length, $L = 5' = 5 \times 12$ inch

Width, $b = 1' = 12$ inch

Let, clear cover = 1" (for slab)

For cantilever slab,

$$\text{Thickness, } t = \frac{L}{10} = \frac{5 \times 12}{10} = 6''$$

$$\therefore d = t - \text{clear cover}$$

$$= 6'' - 1''$$

$$= 5''$$

DL = Self wt.

$$= \gamma_{\text{conc}} \times t$$

$$= 150 \text{ (lb/ft}^3) \times \frac{6}{12} \text{ (ft)}$$

$$= 75 \text{ lb/ft}^2$$

$$W_u = 1.2 \text{ DL} + 1.6 \text{ LL}$$

$$= 1.2 \times 75 + 1.6 \times 100$$

$$= 250 \text{ psf}$$

$$w_u = 0.25 \text{ Ksf}$$

$$\text{Moment, } M_u = \frac{w_u L^2}{2} = \frac{0.25 \times 5^2}{2} = 3.125 \text{ K-ft}$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{A_s \times 60}{0.85 \times 4 \times 12} = 1.47 A_s$$

$$M_u = \phi A_s f_y \left(d - \frac{a}{2} \right)$$

$$\Rightarrow 3.125 \times 12 = 0.9 \times A_s \times 60 \times \left(5 - \frac{1.47 A_s}{2} \right)$$

$$\therefore A_s = 0.142 \text{ in}^2$$

(P.T.O.)

Use, #3 ϕ bar.

$$A_b = 0.11 \text{ in}^2$$

$$\therefore \text{Spacing, } s = \frac{A_b}{A_s} \times 12$$

$$= \frac{0.11}{0.142} \times 12$$

$$= 9.29 \text{ in c/c}$$

$$\approx 9'' \text{ c/c (say)}$$

Provide #3 bar @ 9" c/c (Ann)

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