Tazhar Kabir
CUET (Civil)
Passing Year : 2018

Assistant Engineer (Civil)
Payra Port Authority

Contact : 01406316084.
email : tazhar.ppa@gmail.com

RCC

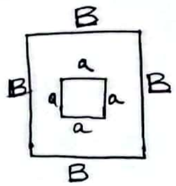
Footing

② # 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 (V_u) if f'_c = 3000 psi, f_y = 60,000 psi.

BWDB-2014

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

$= 20'' - 3.5''$

$= 16.5''$

$\boxed{\text{Punching perimeter, } b_o = 4(a+d)}$

$= 4(12 + 16.5)$

$= 114''$

এখানে, $D_f = t =$ thickness

①,②,③ → প্রশ্নের serial

∴ Punching Shear capacity,

$V_u = \phi . V_c$

$\boxed{V_u = \phi . [4\lambda \sqrt{f'_{c\,(psi)}} \; b_o d]}$

$\phi = 0.75$

$\lambda = 1$

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

$= 309079$ lb

$= 309.08$ kip (Ans)

যেহেতু $f'_c \rightarrow \frac{lb}{in^2}$

যেহেতু $V_u \rightarrow lb.$

Hints:

For Footing in R.C.C. maths:

✓ $\phi = 0.75$

✓ $\lambda = 1$

$\sqrt{f'_c} \longrightarrow psi$

q_u = Bearing capacity

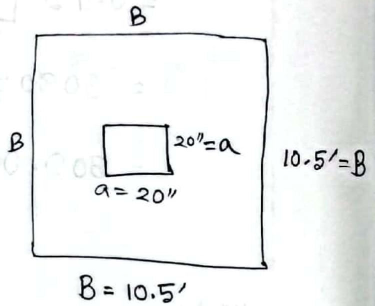
② * BCIC-2017

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 ($D_f = t$) considering punching shear check. $f'_c = 4$ Ksi.

Soln: Factored Load, $P_u = 330$ Kip

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

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

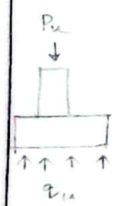


Now,

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

$q_u = \dfrac{Load}{Area}$

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



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

$$= 3 \; Ksf$$

C.cover = 3'' (Footing)
$\phi = 0.75$
$\lambda = 1$

d= 14'' ধরে trial করলাম, Vu এর সাথে compare করলে d=12'' আসবে

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

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

$$= 4(20+14)$$
$$= 136 \text{ in}$$

Shear Strength, $\boxed{V_u = P_u - q_u (a+d)^2}$

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

$$= 306 \text{ Kip}$$

And, $V_u = \phi . V_c$

$$\Rightarrow \boxed{V_u = \phi . [4\lambda \sqrt{f'_{c(psi)}} \, b_o \, d]}$$

খেয়াল $f'c \rightarrow psi$

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

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

$\therefore$ Thickness = $\boxed{\text{Depth of footing}, t = d + c.c}$

$$\Rightarrow t = 12'' + 3''$$
$$= 15''$$ (Ans.)

⑨ * BWDB-2019

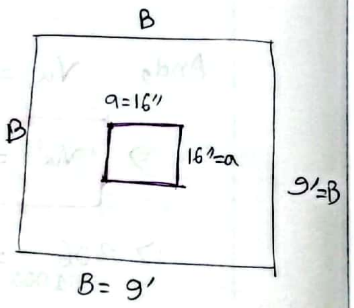
A 16" square column reinforced with 8 Nos. 20 mm bar up to footing and the column carries total 550 Kip load. If the footing size is 9' × 9' and effective depth is 24", find the punching shear stress developed in the footing.

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

Ksf = K/ft^2

Kcf = K/ft^3

V_u = Punching shear

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

Punching perimeter, $\boxed{b_o = 4(a+d)}$

$= 4(16+24)$

$= 160$ in

$= 13.33$ ft

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

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

$= 6.79$ Ksf

Shear strength → $\boxed{V_u = P_u - q_u(a+d)^2}$

ফে মুখস্থ নাই

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

$= 474.55$ Kip

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

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

$= 17.80$ K/ft^2 (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.

BSEC-2014

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

$tsf = Ton/ft^2$

$\boxed{1\ tsf = 2.204\ Ksf}$

$1000\ kg/ft^2 = 2.204\ K/ft^2$

Bearing capacity of soil = soil pressure

$\Rightarrow q_u = 2\ tsf$

$= 2 \times 2.204\ kip/ft^2$

$= 4.408\ K/ft^2$

Now, Total Service load, $\boxed{P_u = DL + LL}$

$= 225 + 185$

$= 410\ kip$

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

And, Footing Area $= \frac{\text{Service Load}}{\text{Soil pressure}}$

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

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

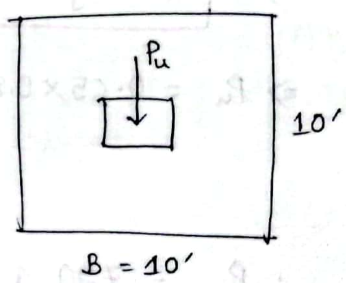
$\Rightarrow B \times B = 93\ ft^2$

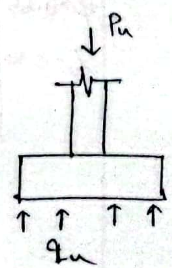
$\Rightarrow B = \sqrt{93}$

$= 9.644\ ft$

$\approx 10\ ft$

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





Column or, Tied Column (Rectangular)

⑤ *** DPDC-2014, PGCB-2015, CPGCBL-2018, Cuet MSC-2020

Find the axial load of column 30" x 12" which is reinforced with 12 Nos. 20 mm diameter bar, consider f'_c = 3.5 Ksi and f_y = 60 Ksi.

Soln.

$\boxed{A_g = B \times L}$

$A_g = 30 \times 12 = 360$ in²

A_{st} = 12 Nos. 20 mm

$= 12 \times \frac{\pi}{4}\left(\frac{20}{25.4}\right)^2$ in²

$= 5.84$ in²

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

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

$= 5.84$ in²

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

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

$\phi = 0.65$

$\alpha = 0.80$

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

$\therefore P_u = 730.1$ Kip (Ans)

⑥ * WRGCL-2014

Determine the maximum nominal axial load for a 15" x 20" tied column having 2.5% steel reinforcement. f'_c = 4 Ksi and f_y = 60 Ksi.

Soln: Maximum nominal axial load, P_n = ?

$A_g = 15 \times 20 = 300$ in²

$\boxed{A_g = B \times L}$

$\rho = 2.5\% = 0.025$

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

Maximum nominal axial load,

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

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

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

$= 1155.6$ Kip (Ans)

Hints: For Column →

✓ $\phi = 0.65$

✓ $\alpha = 0.80$

No. 9 bar = #9ϕ = $\frac{9}{8}$ " dia

(a) *** TGTDCL-2018, BEPZA-2019, NHA-2020

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 $\boxed{\text{zero eccentricity}, \ \alpha = 1}$

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

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

Ultimate design axial load,

$\phi = 0.65$

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

$= 0.65 \times \cancel{0.80}^{1} \left[0.85 \times 3 \times (225 - 8) + 8 \times 60\right]$

$= 671.7$ kip (Ans)

P_u = Ultimate Load
σ_u = Ultimate stress

(b) * GTCL-2016

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

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

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

$\boxed{\text{zero eccentricity}, \ \alpha = 1}$

Design ultimate axial load,

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

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

$= 0.65 \times \cancel{0.80}^{1} \times 144 \left[0.85 \times 4 (1 - 0.02) + 0.02 \times 60\right]$

$= 424.19$ kip

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

$= \frac{424.19}{144}$

$= 2.94$ kip/in² (Ans)

***(29)

PGCL-2014
DNCC-2016
ERL-2017
BGFCL-2017

Determine the nominal axial compression force for RCC column of 12″ x 22″ section having 2.5% (ρ) mild steel reinforcement. Given, $f'_c = 4$ ksi, $f_y = 60$ 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,

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

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

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

$$= 1016.93 \text{ kips}$$

(Ans)

#9 bar $= \frac{9}{8}$″ dia

#10 ϕ $= \frac{10}{8}$″ dia

Size and spacing of Tie bar :

(30) ** BGFCL-2017

Design tie size and spacing of a $\boxed{24'' \times 16''}$ rectangular column which is reinforced with 12#9 bar.

Soln: Longitudinal bar $= 12\ \#9\ \phi$ bar

As, longitudinal bar is less than #10 ϕ,

Use $\boxed{\#3\ \phi \text{ bar as Tie bar.}}$

Spacing:

Smallest

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

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

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

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

(Ans)

Column Design

(50) ** JB-2017, MES-2022

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, $f'_c = 4$ si, $f_y = 60$ ksi.

Soln: $DL = 130$ Kips

$LL = 180$ Kips

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

$= 2\% = 0.02$

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

$= 1.2 \times 130 + 1.6 \times 180$

$= 444$ kip

$\varphi = 0.65$

$\alpha = 0.80$

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

Now, $\boxed{P_u = \varphi . \alpha . A_g [0.85 f'_c (1-\rho) + \rho . f_y]}$

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

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

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

$\therefore b = \sqrt{A_g}$

$= \sqrt{188.4}$

$= 13.7''$

$\approx 14''$ (say)

$\therefore$ 14" × 14" square Tied Column.

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

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

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

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

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

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

So, Use 8 Nos. #6 φ bar. (Ans)

32 * NPCBL-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, $f'c = 4$ ksi, $f_y = 60$ ksi.

Soln.:

$\boxed{P_u = 1.2\,DL + 1.6\,LL}$

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

$= 1240$ Kips

$A_g = ?$
$A_{st} = ?$

$\varphi = 0.65$
$\alpha = 0.80$

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

$\therefore \rho = 2\% = 0.02$ (say)

$\boxed{P_u = \varphi \cdot \alpha \cdot A_g [0.85\, f'c\,(1-\rho) + \rho \cdot f_y]}$

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

$\Rightarrow A_g = 526.17$ in^2

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

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

Again,

$\boxed{P_u = \varphi \alpha [0.85\, f'c\, (A_g - A_{st}) + A_{st} \times f_y]}$

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

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

$A_{st} = \rho \times A_g$
$= 0.02 \times 529$
$= 10.58$ in^2

Use, # 11 φ bar,

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

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

$= 6.97$

≈ 7

So, Use 7 # 11 φ bar. (Ans)

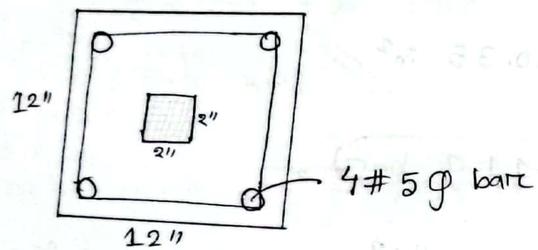
(29)

Hollow Column

** BUET MSc-2018, DPDC-2013

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

Soln:



$\boxed{A_g = (12\times12) - (2\times2)} = 144\ in^2 - 4\ in^2 = 140\ in^2$

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

Capacity of Column, $P_u = ?$

Design the tie bar. size & spacing = ?

$f'c = 4ksi$ | $\varphi = 0.65$
$f_y = 60ksi$ | $\alpha = 0.80$

Now, Capacity of Column,

$$\boxed{P_u = \varphi.\alpha\,[0.85\, f'c\,(A_g - A_{st}) + A_{st}\cdot f_y]}$$

$$= 0.65 \times 0.80\,[0.85 \times 4 \times (140 - 1.23) + 1.23 \times 60]$$

$$= 283.72\ \text{Kip}$$

As, the main bar is Less than # 10 bar, Use # 3 bar as Tie.

Tie Spacing:

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

(ii) $16 \times D_{(main)} = 16 \times \frac{5}{8}'' = 10''$ smallest

(iii) Least dimension (b or L) = 12" (Hollow বিয়োগ হবেনা)

So, Use # 3 bar @ 10" c/c for Tie bar. (Ans)

(38) *** HBFC-2018, Combined Bank-2019

(WSD) Rectangular Column

A reinforced concrete column of effective length 4.8 m (L_e) and overall dimension 250 mm × 400 mm ($B \times H$). 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.

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

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

$b = 250$ mm

$P_u = 60$ Ton

$= 60 \times 1000 \times 9.81$ N

$= 5.886 \times 10^5$ N

$A_s = ?$ Spacing of tie $= ?$

For M20 concrete, $f'_c = 20$ MPa

$= 20$ N/mm^2

For $\boxed{\text{mild steel, } f_y = 415 \text{ MPa}}$

$= 415$ N/mm^2

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

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

$\boxed{\frac{L_e}{b} > 12}$

So, Long column. Reduction factor will be used.

Reduction Factor,

$$\boxed{C_r = 1.25 - \frac{L_e}{48\,b}}$$

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

$= 0.85$

Axial Load, $\boxed{P = \frac{P_u}{C_r}} = \frac{5.886 \times 10^5}{0.85} = 6.92 \times 10^5$ N

And, $\boxed{P = C_r \left(0.25\, f'_c\, A_g + f_s\, A_s\right)}$

$\Rightarrow 6.92\times10^5 = 0.85\left(0.25 \times 20 \times 10^5 + 166 \times A_s\right)$

$\therefore A_s = 1895\ mm^2$

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

$= 201.06\ mm^2$

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

As Main bar 16mm, $\boxed{\text{Use Tie bar 10 mm}}$

Spacing:
(i) $48 \times d_{(Tie)} = 48\times10 = 480\ mm$
smallest
(ii) $16 \times D_{(main)} = 16\times16 = 256\ mm$
(iii) Least Dimension $= 250\ mm$

Use 10 mm bar @ 250mm c/c for Tie bar (Ans)

WSD: Circular Column

32 * AB-2017

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 $f_y = 415$ MPa. Using working stress design (WSD) method to determine the column strength.

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

b=d=dia circular

Dia of column, $b = 300\ mm = d$

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

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

$f_y = 415$ MPa $= 415\ N/mm^2$

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

$A_g = \frac{\pi}{4} d^2 = \frac{\pi}{4} \times 300^2 = 70686\ mm^2$ ($\because$ circular)

Column strength, $P = ?$

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

$\frac{L_e}{b} > 12$

So, it is long column. Reduction factor (C_R) will be used.

$$\boxed{C_R = 1.25 - \frac{L_e}{48b}}$$

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

$$= 0.76$$

$$\boxed{P = C_R \left(0.25 f'_c A_g + f_s A_s\right)}$$

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

$$= 404383.46 \text{ N}$$

$$= 404.38 \text{ KN}$$

(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:

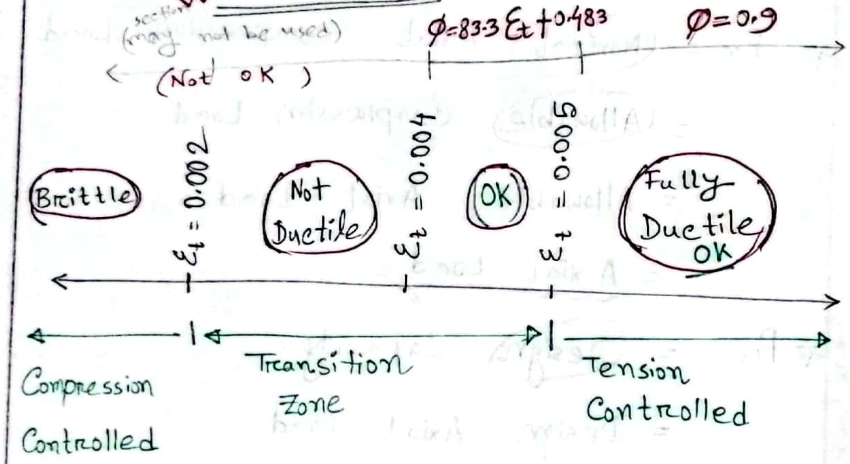
Main bar #10 থেকে কম, Tie = #3 bar

Main bar #10 থেকে বেশি, Tie = #4 bar

- যদি main bar 16 mm, Tie = 5 mm.

Beam :

Concepts



- $\varepsilon_t > 0.005$ = Tension Controlled

  $\Rightarrow \phi = 0.9$

- $0.004 < \varepsilon_t < 0.005$ = Transition zone

  $\Rightarrow \phi = 83.3 \times \varepsilon_t + 0.483$

- $0.004 > \varepsilon_t$ = Not Ductile

  $\Rightarrow$ section May not be used as per ACI Code.

Hints :

$\varepsilon_u = 0.003$

- $\rho = \frac{A_s}{bd}$

- $\rho_b = 0.85\, \beta_1 \frac{f'_c}{f_y} \frac{0.003}{0.003 + \varepsilon_y}$

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

- $\rho_{min} = 0.0033$

  $\rho > \rho_{min}$ (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

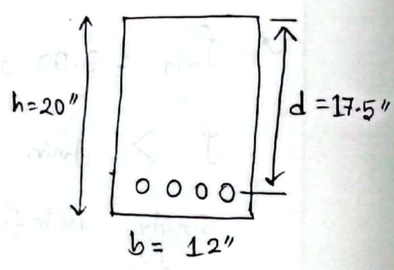
(24)

BB-2021

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

Soln. Nominal Strength, $M_n = ?$

Take "Under-reinforced Beam."



[4 #9 bars]

$A_s = n \times A_b$

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

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

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

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

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

$$= 5.88''$$

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

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

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

(Ans.)

Hints : For Beam :

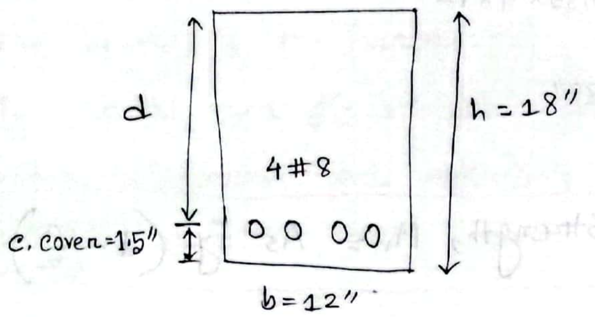
When → $\boxed{f'_c = 3 \text{ Ksi } \& \ 4 \text{ Ksi}, \quad \beta_1 = 0.85}$

→ $\boxed{f'_c = 5 \text{ Ksi}, \quad \beta_1 = 0.80}$

($f'_c = 3$ Ksi, $\beta_1 = 0.85$)

59 * BWDB-2018

Determine the ultimate moment for the beam shown below where $f'_c = 4$ Ksi, $f_y = 60$ Ksi, Clear cover $= 1.5''$.



Sol^n: Ultimate moment, $M_u = ?$

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

$= 18'' - 1.5''$

$= 16.5''$

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

Check-01

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

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

$\rho_{min} = 0.0033$

Check-02:

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

$= \frac{3.16 \times 60}{0.85 \times 4 \times 12}$ | $\boxed{\beta_1 = 0.85,}$ as $f'_c = 4$ksi

$= 4.65''$

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

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

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

$= 0.00605 > 0.005$

So, Tension controlled section, $\boxed{\phi = 0.9,}$

$M_u = \phi \cdot M_n$

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

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

$= 2418.82$ Kip-in (Ans.)

$$\rho = \frac{A_s}{A_g}$$

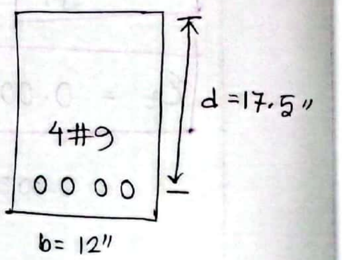
$$\rho = \frac{A_s}{bd}$$

28 * DESCO-2015

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 whould be the ϕ value.

Soln: Ultimate moment capacity, $M_u = ?$

ϕ value $= ?$



$$\boxed{A_s = n \times A_b}$$

$= 4 \times 1$ in^2

$= 4$ in^2

C-01:

$$\boxed{\rho = \frac{A_s}{bd}} = \frac{4}{12 \times 17.5} = 0.02 > \rho_{min}\ (0.0033)$$

so, singly reinforced beam. (ok)

C-02:

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

$$= \frac{4 \times 60}{0.85 \times 4 \times 12} = 5.88''$$

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

$\beta_1 = 0.85$ as $f'_c = 4$ ksi

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

$$= 0.003 \times \left(\frac{17.5 - 6.92}{6.92}\right)$$

$$= 0.0046$$

As, $0.004 < \varepsilon_t < 0.005$

So, Beam is in transition zone.

$$\therefore \boxed{\phi = 83.3 \times \varepsilon_t + 0.483}$$

$$= 83.3 \times 0.0046 + 0.483$$

$$= 0.866$$

Imp. check

Now, Ultimate moment capacity,

$M_u = \phi . M_n$

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

$$= 0.866 \times 4 \times 60 \left(17.5 - \frac{5.88}{2}\right)$$

$$= 3026 \text{ Kip-in}$$

(Ans)

(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 weather the beam is tension controlled or not if $f'_c = 4$ ksi and $f_y = 60$ ksi. Determine design capacity of the beam.

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

$= 4\ in^2$

Check-01:

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

So, singly reinforced beam,

Check-02:

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

$\beta = 0.85$

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

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

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

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

(Ans.)

Beam Design

(20) * NESCO-2021

The cross section of a beam reinforced with $3\ in^2$ of steel. Assume the tension steel yield at maximum moment. What is the area required to balance the steel force when the steel yeilds? $f'_c = 4$ ksi, $f_y = 60$ ksi.

Soln: Beam area to balance the steal force, bd = ?

$$\boxed{\varepsilon_y = \frac{f_y}{E}} = \frac{60\ (ksi)}{29 \times 10^3\ (ksi)} = 0.00207$$

$= 0.0021$ (say)

Balance steel ratio,

$$\boxed{\rho_b = 0.85\ \beta_1\ \frac{f'_c}{f_y}\left(\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_y}\right)} \quad \Big|\ \varepsilon_u = 0.003$$

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

$= 0.0285$

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

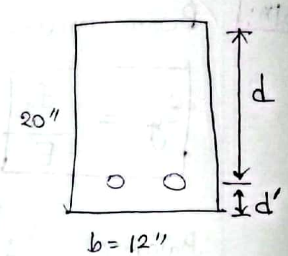
$$\Rightarrow 0.0285 = \frac{3}{bd}$$

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

For Design, $\phi = 0.9$

(22) ** 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 $f'_c = 3$ Ksi, $f_y = 60$ Ksi and $d' = 3$ in.

WASA-2017
NGO Forum-2022



Soln: $A_s = ?$ $M_u = 180$ K-ft

Failure occurs by concrete cracking or steel yeilding?

$\boxed{d = h - d'}$

$= 20 - 3 = 17$ inch

$d' = $ c.cover

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

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

$\therefore a = 1.96 A_s$ ——— (i)

Moment = Ultimate Moment $= M_u$

$M_u = \phi . M_n$

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

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

$\Rightarrow A_s = 2.80$ in^2 (Ans)

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

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

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

$= 0.0135$

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

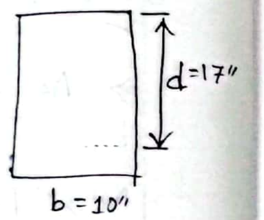
So, Beam will fail by cracking of concrete (Ans)

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. $f'_c = 4$ Ksi, $f_y = 60$ Ksi.



Soln:

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

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

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

$$M_u = \phi . M_n$$

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

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

(K-in)

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

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

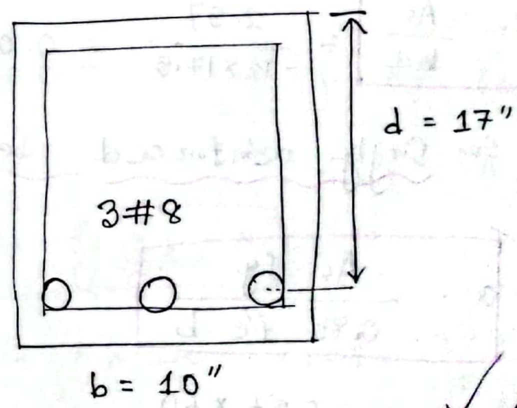
Nos. of bar, $\boxed{n = \frac{A_s}{A_b}}$

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

$$= 2.47$$

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

So, Use 3 Nos. 8 Ø bar.



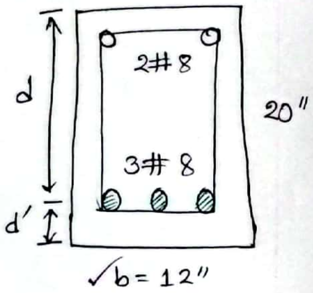
(Ans.)

⇒ # Figure Doubly reinforced but Singly beam

(29)

DMTCL-2019

* Calculate the Ultimate moment capacity of the following beam given $h = 20''$, $b = 12''$, $d' = 2.5''$, $f'c = 4$ ksi, $f_y = 60$ ksi. Assuming singly reinforced beam.



Soln:

$$\boxed{d = h - d'}$$

$= 20$ in $- 2.5$ in

$= 17.5$ in

$A_s = 3 \times 0.79 = 2.37$ in^2

C-01:

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

So, Singly reinforced beam (OK).

Check-02:

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

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

$= 3.84$ in

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

$f'c = 4$ ksi, $\beta_1 = 0.85$

Now, $$\boxed{\varepsilon_t = 0.003 \times \left(\frac{d-c}{c}\right)}$$

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

$= 0.008 > 0.005$

∴ Tension controlled. $\boxed{\phi = 0.9}$

∴ Ultimate moment capacity of beam,

$M_u = \phi \cdot M_n$

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

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

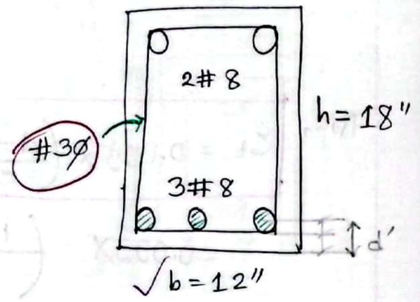
$= 1994$ kip-inch

(Ans)

28 ** BIWTA-2019

Calculate Ultimate load carrying capacity of Beam.

$f'_c = 4$ Ksi , $f_y = 60$ Ksi . Clear cover = 1.5".



Hints: যখন Tie dia দেওয়া থাকবে,
d' না বলে clear cover উল্লেখ থাকবে,
$d = h - d'$
$\Rightarrow d = h - (\text{c.cover} + 1 \text{ inch})$

Soln: $\boxed{d = h - (\text{clear cover} + 1 \text{ inch})}$

$= 18" - (1.5 + 1)$

$= 15.5"$

singly reinforced হলে বিবেচনা করব

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

Check-01:

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

So, Singly reinforced beam,

Check-02:

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

$= \frac{2.37 \times 60}{0.85 \times 4 \times 12} = 3.485"$

$\boxed{C = \frac{a}{\beta_1}} = \frac{3.485}{0.85} = 4.1$ | $\beta_1 = 0.85$

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

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

∴ Tension controlled.

$\therefore \phi = 0.9$

Ultimate load carrying capacity,

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

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

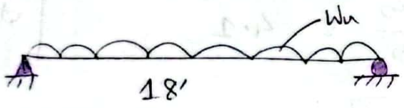
$= 1760.68$ kip-inch (Ans)

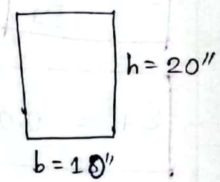
Span Length and Support type Given

(23) * 36th BCS

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 Kip/ft

DL = 1.05 Kip/ft

A_s = ?

Assume,

$\boxed{\text{Clear cover for beam} = 1.5 \text{ inch}}$

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

$= 20 - 1.5 = 18.5$ inch

Factored load, $\boxed{W_u = 1.2\,DL + 1.6\,LL}$

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

$= 5.21$ K/ft

Factored moment, *** $\boxed{M_{uF} = \dfrac{W_u L^2}{8}}$ (Simple with UDL)

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

$= 211.08$ K-ft

$\therefore M_u = 2533$ K-in

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

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

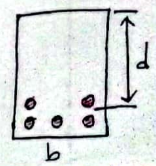
$\Rightarrow 211.08 \times 12 = 0.9 \times A_s \times 60 \times \left(18.5 - \dfrac{1.765\,A_s}{2}\right)$

$\therefore A_s = 2.95$ in^2

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

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

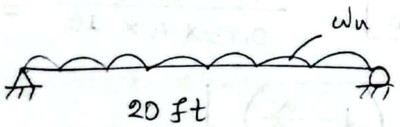
Use 5 #7 ϕ bar (Ans.)

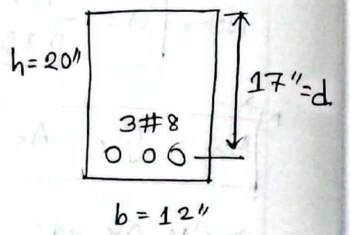


Live Load = Service Load

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

Soln:





DL = Self weight

LL = Service Load = ?
(Live Load)

$\boxed{\text{Concrete Dead Load, } DL = \gamma_{conc} \times (bh)}$

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

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

$\therefore DL = 250$ lb/ft $= 0.25$ K/ft

$DL_{conc} = \gamma_{conc} \times bh$
$= 150 \times bh$

| b, h → ft

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

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

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

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

$= 1875$ K-in

$= 156.5$ K-ft

And, $\boxed{M_u = \frac{W_u l^2}{8}}$ (simple & UDL)

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

$\therefore W_u = 3.13$ K/ft

Again, $\boxed{W_u = 1.2 DL + 1.6 LL}$

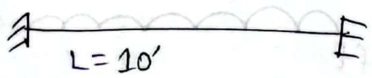
$\Rightarrow 3.13 = 1.2 \times 0.25 + 1.6 \times LL$

$\therefore LL = 1.769$ K/ft (Ans)

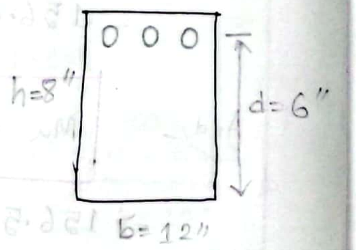
29 * The span of a beam 10′ long and ends are fixed. The maximum load controlled by the capacity in the negative moment region. Based on the flexural requirements, what is the maximum uniform live load the beam can carry? ($f_y = 40$ ksi, $f'_c = 3.5$ ksi, $M_n = 140.2$ k-in, $A_s = 0.62$ in^2.)

BB AD-2018

Soln:



Negative Moment.



Max^{m} Live Load, LL = ?

DL = Self weight

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

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

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

$= 100$ lb/ft

$= 0.1$ K/ft

Hints:
Mn দেওয়া আছে,
a বের করার দরকার নাই,

$\boxed{M_u = \phi \cdot M_n}$

$= 0.9 \times 140.2$

$= 126.18$ Kip-in $= 10.515$ Kip-ft

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

$\cancel{\Rightarrow 126.18 =}$ $\Rightarrow \underset{(K\text{-}ft)}{10.515} = \frac{W_u \times 10^2}{12}$

$\therefore W_u = 1.262$ K/ft

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

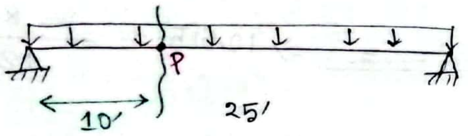
$\Rightarrow 1.262 = 1.2 \times 0.1 + 1.6 \times LL$

$\Rightarrow LL = 0.71375$ K/ft

$\therefore LL = 713.75$ lb/ft (Ans.)

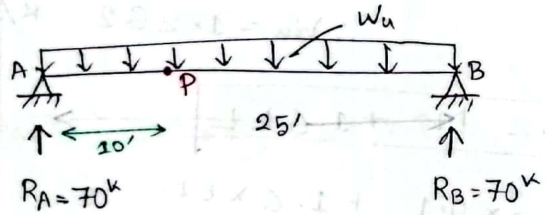
Combined Bank-2020

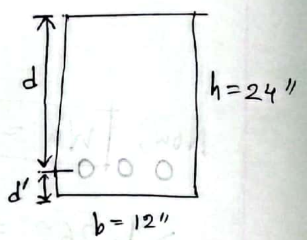
* A rectangular beam of section 12"x24" carries DL = 2 K/ft and LL = 2 K/ft. If the value of $f'_c = 4$ Ksi, $f_y = 60$ Ksi, $d' = 1.5"$, Design the reinforcement at 10' from the support for the following beam.



$A_s = ?$

Soln:





$W_u = 1.2\,DL + 1.6\,LL$

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

$= 5.6$ K/ft

$\therefore$ Total Load $= 5.6$ (K/ft) $\times 25' = 140^k$

$d = h - d'$

$= 24 - 1.5$

$= 22.5"$

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

Moment at point "P" on Left Side,

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

$= 420$ K-ft

$= 420 \times 12$ Kip-inch

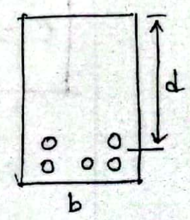
$= M_u$

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

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

$\therefore A_s = 4.95\ in^2$

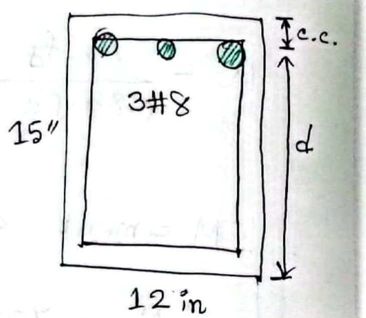
Use 5#9 ϕ bar as main bar.



Cantilever Beam

(28) * PGCB-2019

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



Soln:

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

$= 15 - 2.5$

$= 12.5''$

$A_s = 3 \times 0.79$

$= 2.37 \text{ in}^2$

check-01:

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

So, Singly reinforced beam.

check-02:

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

$$\boxed{c = \frac{a}{\beta_1}} = \frac{3.49}{0.85} = 4.11 \qquad \beta = 0.85$$

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

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

$= 0.0061 > 0.005$ [Tension Controlled]

$\therefore \phi = 0.9$

Now, Flexural moment capacity of beam,

$M_u = \phi . M_n$

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

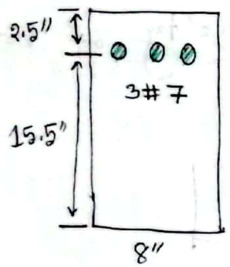
$= 0.9 \times 2.37 \times 60 \times \left(12.5 - \frac{3.49}{2}\right)$

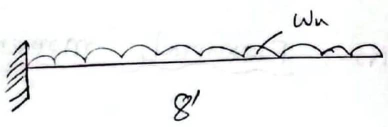
$= 1376.43$ kip-in (Ans.)

২৯

*** BCIC – 2017
BUET MSc – 2019
EGCB – 2020

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 $f'_c = 4$ ksi, $f_y = 60$ ksi, check if the beam is safe to carry above loads.





DL = 1.5 K/ft

LL = 0.9 K/ft

Soln:

১ম ধাপঃ

Factored load, $\boxed{W_u = 1.2\ DL + 1.6\ LL}$

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

$= 3.24$ K/ft

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

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

$= 103.68$ K-ft

২য় ধাপঃ

$A_s = 3 \times 0.6 = 1.8$ in^2

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

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

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

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

$\therefore \phi = 0.9$

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

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

$= 1313.66$ Kip-in

$= 109.47$ kip-ft

$\therefore M_u > M_{uF}$ (ok)

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

Concepts : "USD"

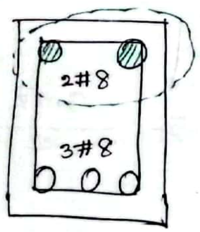
① $\gamma_D = 1.2$, $\gamma_L = 1.6$ দেওয়া থাকলে,

$$W_u = \gamma_D \times DL + \gamma_L \times LL$$
$$= 1.2\ DL + 1.6\ LL$$

② যদি $\varepsilon_t < 0.005$ হয়,

$$\phi = 83.3\ \varepsilon_t + 0.483$$

③ Cantilever Beam বলা থাকলে,



$\boxed{\text{Calculation এ শুধু এই steel আসবে,}}$

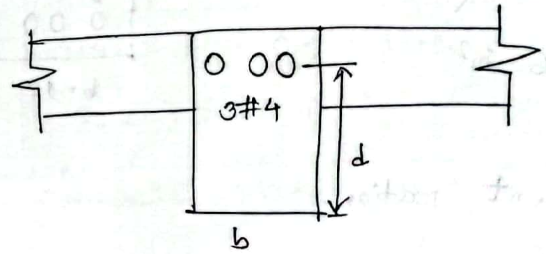
$A_s = 2 \times 0.79$

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

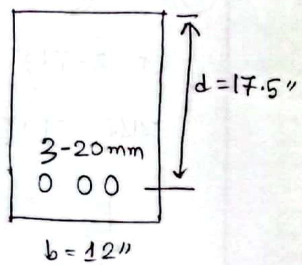
⑤ উল্লেখ না থাকলেও ,
এ ধরনের section দেখে Cantilever beam বুঝে নিতে হবে,



Beam : WSD → Analysis

60 *** WASA-2017, PGCL-2017, MPA-2019

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, $f_c = 1200$ 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,

$$\boxed{M_s = A_s f_s j d}$$

$$= 1.46 \times 24 \times 0.9 \times 17.5$$

$$= 552.75 \text{ kip-in}$$

If concrete stress controls,

$$\boxed{M_c = \frac{1}{2} f_c K j bd^2}$$

$$= \frac{1}{2} \times 1.2 \times 0.297 \times 0.90 \times 12 \times 17.5^2$$

$$= 589.4 \text{ kip-in}$$

∴ Allowable working moment, $M_{Allow} = 552.75$ k-in. (Ans)

Minimum ET

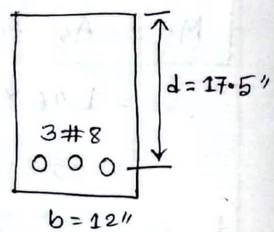
(59)

* 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.

$f'_c = 4000$ psi , $f_y = 60000$ psi.

Soln:

$M_{max} = M_{allow} = ?$



$f_c = 0.45\, f'_c$

$= 0.45 \times 4000$

$= 1800$ psi

$f_s = 0.40\, f_y$

$= 0.40 \times 60000$

$= 24000$ psi

(60)

$A_s = 3 \times 0.79 = 2.37\ in^2$

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

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

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

$= 0.359$

Let, $n = 9$

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

If steel stress controls,

$M_s = A_s f_s j d = 2.37 \times 24 \times 0.88 \times 17.5$

$= 875.95$ kip-inch

If concrete stress controls,

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

$= 1044.9$ kip-inch

$\therefore M_{max} = M_{allow} = 875.95$ kip-in (Ans)

(minimum টা)

02

Beam: WSD → Design

** BKB-2018

A rectangular beam of 12"X24" 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 (WSD) of 20000 psi and 3000 psi concrete at a working stress of 1350 psi. $d' = 2.5"$.

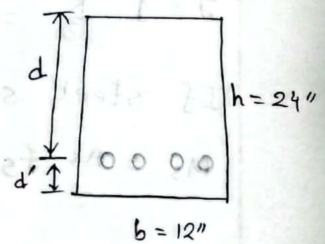
Soln: $LL = 680$ lb/ft

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

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

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

$= 300$ lb/ft



$d = h - d' = 21.5"$

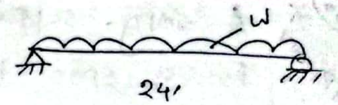
$\therefore DL = 380 + 300 = 680$ lb/ft

WSD method, $\boxed{W = DL + LL}$

$= 680 + 680 = 1360$ lb/ft

Given,
$f_s = 20000$ psi $f_c = 0.45 \times f'_c$
$f'_c = 3000$ psi, $f_c = 1350$ psi

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



(Simple Span)

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

$= 97920$ ~~lb/~~ lb-ft

$= 97.92$ Kip-ft

Let, $\boxed{n = 9 \quad \text{(Modular Ratio)}}$

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

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

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

Moment, $\boxed{M = A_s f_s j d}$

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

$\therefore A_s = 3.13$ in^2

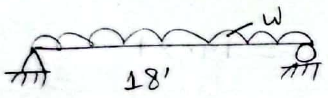
Use, $\boxed{4\#8\ \phi \text{ bar}}$ (Ans)

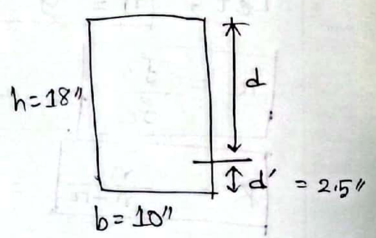
A_s (provided) $= 3.16$ in^2 (OK)

(60) * BHP-2017

A simply supported beam of 10" × 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. $d' = 2.5"$.

$\boxed{\text{Soln.}}$





$\boxed{d = h - d'} = 15.5"$

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

$\boxed{\text{Self wt.} = \gamma_{conc} \times (bh)}$ | $\gamma_{conc} = 150\ \frac{lb}{ft^3}$

$= 150 \times \left(\frac{10}{12} \times \frac{18}{12}\right)$

$= 187.5$ lb/ft

$\therefore W = 1355 + 18.7$

$= 1542.5$ lb/ft

$= 1.54$ K/ft

$\boxed{f_c = 0.45 f'_c}$

$= 0.45 \times 3000 = 1350$ psi

Moment, $\boxed{M = \frac{wL^2}{8}} = \frac{1.54 \times 18^2}{8}$

$= 62.47$ K-ft

Let, $n = 9$

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

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

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

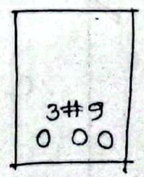
Moment, $\boxed{M = A_s f_s j d}$

$\Rightarrow \underset{(K\text{-in})}{62.47 \times 12} = A_s \times 20 \times 0.87 \times 15.5$

$\Rightarrow A_s = 2.78\ in^2$

Use, $\boxed{3\#9\ \varphi\ \text{bar}}$ all in a row.

$A_{s\,(provide)} = 3 \times 1 = 3\ in^2$ (ok)

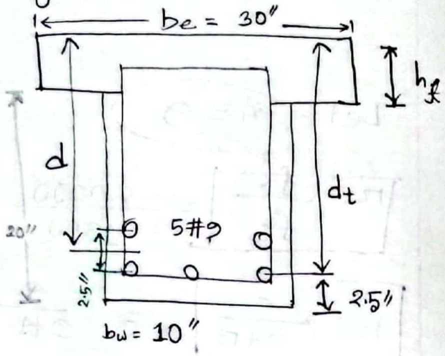


(Ans)

T-Beam

(98) ** RPGCL-2017, BPDB-2018

Determine the ultimate moment capacity of the T-beam, $f'_c = 3$ ksi, $f_y = 40$ ksi.



Soln:

Given,

$b_w = 10''$

$b_e = 30''$

$h_f = 3''$

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

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

$d = 19.25''$

$d_t \rightarrow$ For Tension bar

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

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

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

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

$[\beta_1 = 0.85$ when $f'_c = 3$ Ksi$]$

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

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

$$= 0.017 > 0.005$$

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

Now, Ultimate moment capacity,

$$M_u = \phi . M_n$$

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

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

$$= 32\ 30.1 \text{ Kip-inch}$$

(Ans.)

Hints: T-Beam এর ক্ষেত্রে :

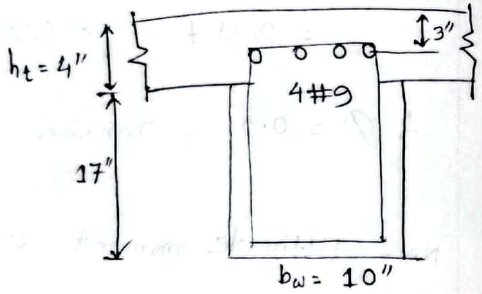
ε_t হিসাব করতে d_t নিতে হবে,

M_u বা M_n হিসাব করতে d নিতে হবে।

(৬৫)

* 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'. $f_y = 60$ Ksi, $f'_c = 5$ Ksi, $\beta_1 = 0.8$,



$A_s = 4 \times 1\ in^2$

$= 4\ in^2$

$M_n = ?$

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

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 নিচে

Negative moment = reinforcement উপরে।

②// Span length (L) ও clear distance of web দেওয়া থাকলে, (i) $b_e = b_w + 16 h_t$

(ii) b_e = clear distance of web + b_w

(iii) $b_e = \frac{L}{4}$ (inch)

এ তিনটির মধ্যে minimum টা হবে।

যদি/আর দেওয়া না থাকলে,

$b_e = b_w + 16 h_t$ হবে।

(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:

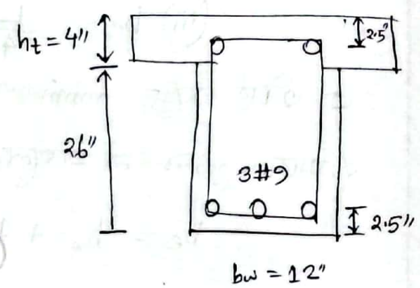
$h_t = 4''$ = Slab Thickness

$b_w = 12''$

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

$b_e = 30''$ (नीचे Calculation)

$L = 30'$



Now,

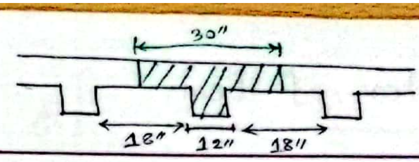
minimum नेंवा

(i) $b_e = b_w + 16 \times h_t = 76$ inch

(ii) b_e = Clear distance of web + b_w

$= 30$ inch

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



$A_s = 3 \times 1\ in^2 = 3\ in^2$

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

$a < h_t$, so rectangular section analysis.

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

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

$$\varepsilon_t = 0.003 \times \left(\frac{d-c}{c}\right) = 0.003 \times \left(\frac{27.5-2.76}{2.76}\right)$$

$$= 0.026 > 0.005$$

$\therefore \phi = 0.9$.

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

$$= 0.9 \times 3 \times 60 \times \left(27.5 - \frac{2.35}{2}\right)$$

$$= 4264.65 \text{ k-in} \quad \text{(Ans)}$$

Shear of Beam

Concept: For Beam →

$\lambda = 1$ for Shear

$\phi = 0.75$ for Shear

A_v = Shear Bar Area

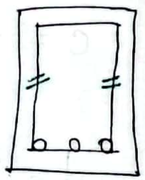
S = Spacing of Tie bar

$$V_n = V_s + V_c$$

$$V_u = \phi \cdot V_n$$

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

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



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

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

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

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

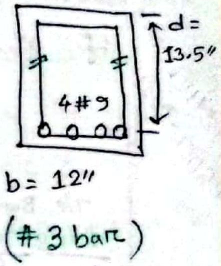
(69) * SGFCL-2017

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

$\phi = 0.75$
$\lambda = 1$

Spacing, S = 6"

Sol^n:



$A_s = 4 \times 0.79 = 3.16$ in^2

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

Check

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

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

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

$$\boxed{\varepsilon_t = 0.003 \times \left(\frac{d-c}{c}\right)} = .003\left(\frac{13.5-7.28}{7.28}\right)$$

$$= 0.0026 < 0.004$$

So, The section is not Tension controlled.

Now, Allowable shear strength,

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

$$\boxed{V_u = \phi \cdot \frac{A_v f_y d}{S} + \phi \cdot \frac{2\lambda \sqrt{f'_c}\ bd}{1000}}$$

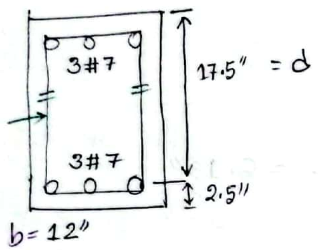
$$= \underbrace{0.75 \times \frac{0.22 \times 60 \times 13.5}{6}}_{(\text{Kip})} + \underbrace{\frac{0.75 \times 2 \times 1 \times \sqrt{3000} \times 12 \times 13.5}{1000}}_{(\text{Kip})}$$

$\therefore V_u = 35.57$ Kips (Ans)

$\phi = 0.75$
$\lambda = 1$

(৬৮) ** Determine ultimate shear capacity of the beam if $f'_c = 3$ Ksi and $f_y = 60$ Ksi.

BCMCL-2020
BGDCL-2021



Tie Bar $\underbrace{\#4}_{A_v}$ @ $\underbrace{8'' c/c}_{S}$

Spacing = S = 8" c/c

Soln: ∴ $A_v = 2 \times \frac{\pi}{4}\left(\frac{4}{8}\right)^2 = 0.4 \ in^2$

Now, Ultimate shear capacity,

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

$$\therefore \boxed{V_u = \phi . \frac{A_v . f_y . d}{S} + \frac{\phi . 2\lambda \sqrt{f'_c} \ bd}{1000}}$$

$$= 0.75 \times \frac{0.4 \times 60 \times 17.5}{8} + \frac{0.75 \times 2 \times 1 \times \sqrt{3000} \times 12 \times 17.5}{1000}$$

(kip) (kip) এ [illegible]

$= 56.628$ Kip

(Ans)

For shear, Clear cover = 2.5"

(৬৯) *** A RCC beam is to be designed to carry a shear force V_u of 30K, No web reinforcement is to be used and $f'_c = 4$ Ksi. What is the minimum cross section if controlled by shear?

BBAD-2018

$\phi = 0.75$
$\lambda = 1$

Soln: Minimum cross section, $b \times h = ?$

For No web reinforcement,

Applied shear, $\boxed{V_u = \frac{1}{2} \phi . V_c}$

$$\Rightarrow \boxed{V_u = \frac{1}{2} \phi . [2\lambda \sqrt{f'_{c(psi)}} \ b \ d]}$$

$$\Rightarrow \underset{(psi)}{30 \times 1000} = \frac{1}{2} \times 0.75 \times 2 \times 1 \times \sqrt{4000} \times bd$$

$$\Rightarrow bd = 632.46 \ in^2$$

Consider, $\boxed{b = b_w = 20 \ in}$

$\therefore d = \frac{632.46}{20} = 31.62$ in

≈ 32 in (say)

$\boxed{h = d + \text{clear cover}}$

$= 32'' + 2.5''$

$= 34.5''$

$\approx 35''$ (say)

$\therefore b \times h = 20'' \times 35''$

(Ans)

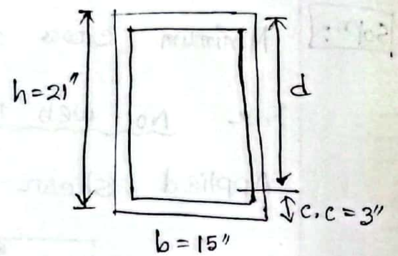
* Design stirrup of a beam which size is 21″ × 15″ and 10 # 10 bar used in this beam. Given, $V_u = 40000$ lb, $f_y = 60$ ksi, $f'_c = 4$ ksi and clear cover is 3 inch. (EGCB-2015)

Soln:

$A_s = 10$ # 10 bar

$d = 21 - 3 = 18''$

$V_u = 40000$ lb



$\phi V_c = \phi \cdot 2\lambda \sqrt{f'_c}\, bd$

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

$= 25615$ lb

$\because V_u > \phi V_c$, so stirrup needed.

Provide, 10 mm bar as stirrup.

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

Use 10 mm bar @ 9″ c/c. (Ans)

Hints : বড় Bar হলে $S = \frac{d}{4}$ হবে।

X

# Concept :		Area (A_s)
# 3 ϕ	= 10 mm bar	0.11 in²
# 4 ϕ	→ 12 mm bar	0.20 in²
# 5 ϕ	→ 16 mm bar	0.31 in²
# 6 ϕ	→ 20 mm bar	0.44 in²
# 7 ϕ	→ 22 mm bar	0.60 in²
# 8 ϕ	→ 25 mm bar	0.79 in²
# 9 ϕ	= 28 mm bar	1.00 in²
# 10 ϕ	= 32 mm bar	1.27 in²
# 11 ϕ	= 36 mm bar	1.56 in²

Slab :

Two Designer

(83)

*** PGCB-2021

A designer designed 6" one way slab with 60 grade (f_{y_1}) reinforcement. He used #4 bar (A_{b_1}, d_1) as main reinforcement and provided spacing at 5 in c/c (S_1). A second designer designs the slab with 500W (f_{y_2}) reinforcement instead of 60 grade reinforcement and used 12 mm bar (A_{b_2}, d_2) as main rebar instead of #4 bar. What should be the spacing (S_2) of slab by the second designer?

$\boxed{\text{Soln:}}$ 1st Designer:

$f_{y_1} = 60$ Grade $= 60$ Ksi $= f_{y1}$

Slab thickness, $t = 6''$

$A_{b_1} = \frac{\pi}{4} \times \left(\frac{4}{8}\right)^2 = 0.20$ in^2 [#4 bar] | $A_{b1} = \frac{\pi}{4} \times d_1^2$

Spacing, $S_1 = 5''$ c/c

$A_s = ?$

Now, $\boxed{\text{Spacing, } S_1 = \frac{A_{b_1}}{A_{s_1}} \times 12}$

$\Rightarrow A_{s_1} = \frac{A_{b_1}}{S_1} \times 12$

$\Rightarrow A_{s_1} = \frac{0.20}{5} \times 12 = 0.48$ in^2 $= A_{s1}$

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

$\#6 = \frac{6}{8}'' = \frac{6}{8}$ in

$\boxed{1 \text{ MPa} = 145 \text{ psi}}$

$\boxed{1 \text{ MPa} = 0.145 \text{ Ksi}}$

1 in = 25.4 mm = 2.54 cm

2nd Designer:

$f_{y_2} = 500$ W $= 500$ MPa $= 500 \times 0.145$ Ksi

$= 72.5$ Ksi $= f_{y2}$

$A_{b_2} = \frac{\pi}{4} \times \left(\frac{12}{25.4}\right)^2 = 0.175$ in^2 [12 mm bar]

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

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

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

$\boxed{\text{Spacing, } S_2 = \frac{A_{b_2}}{A_{s_2}} \times 12}$

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

$= 5.3$ inch

$= 5$ inch (say)

Round বা Half Round করতে হবে, কিন্তু কমায়ে নিতে হবে যেহেতু Spacing।

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

(Ans)

82

BCIC-2019
BADC-2020

*** A designer designed a 6" one way slab with 60 grade (f_{y_1}) reinforcement. He used #4 (d_1) bar as main reinforcement and provided spacing at 4.5 inch c/c (s_1). A second designer designs the slab with 500W (f_{y_2}) reinforcement instead of 60 grade reinforcement and used 12 mm (d_2) 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, $t = 6''$

$f_{y_1} = 60$ grade $= 60$ Ksi $= f_{y_1}$

$A_{b_1} = \#4$ bar $= \frac{\pi}{4} \times \left(\frac{4}{8}\right)^2 = 0.196$ in^2

$A_{b_1} = \frac{\pi}{4} d_1^2$

Spacing, $S_1 = 4.5''$ c/c

$$\boxed{\text{Spacing, } S_1 = \frac{A_{b_1}}{A_{s_1}} \times 12}$$

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

$\therefore A_{s_1} = 0.523$ in^2 $= A_{s1}$

For Second Designer:

$f_{y_2} = 500\,W = 500$ MPa $= 500 \times 0.145$ Ksi

$= 72.5$ Ksi $= f_{y_2}$

$A_{b_2} = 12$ mm bar $= \frac{\pi}{4} \times \left(\frac{12}{25.4}\right)^2 = 0.175$ in^2

Spacing, $s = ?$

Now, $\boxed{A_{s_1} \times f_{y_1} = A_{s2} \times f_{y_2}}$

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

$\therefore A_{s2} = 0.433$ in^2 $= A_{s_2}$

And, $\boxed{\text{Spacing, } S_2 = \frac{A_{b_2}}{A_{s_2}} \times 12}$

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

$= 4.85$ inch

$= 4.50$ inch c/c (say) (Ans.)

Hints: For Spacing →

Spacing $= 5.3''$ আসলে, $S = 5''$ ধরবো c/c

$= 4.85''$ আসলে, $S = 4.5''$,, c/c

$= 6.47''$ আসলে, $S = 6''$ ধরবো c/c,

Temperature & shrinkage reinforcement

8\) *** Determine the slab thickness and spacing required for temperature and shrinkage reinforcement of one way slab, considering $f_y = 60$ Ksi.

LGD-2018
51 BMA

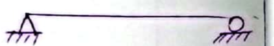
Soln: Span Length, $L = 15' = 15 \times 12$ inch [যেহেতু $f_y \rightarrow \frac{Kip}{in^2}$]

Slab thickness, $t = ?$

Spacing, $S = ?$

Assume, $b = 12''$

For simply supported,

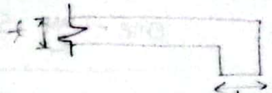


i) $\boxed{\text{Slab thickness, } t = \frac{L}{20}}$

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

$= 9$ inch

$b = 12''$
$d = 10$ mm



ii) Shrinkage and Temprature steel on Transverse direction.

Minimum steel area for Slab, $t = \frac{L}{20}$

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

$= 0.0018 \times 12 \times 9$

$= 0.1944$ in^2

Using $\boxed{10 \text{ mm bar}}$, (for Temp. & Shrinkage Reinforc.) $A_b = \frac{\pi}{4} \times \left(\frac{10}{25.4}\right)^2$

$= 0.11$ in^2

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

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

$= 6.79$ in c/c

$= 6.50$ inch c/c (say)

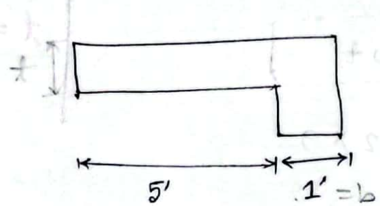
So, Provide 10 mm bar @ 6.5" c/c.

(Ans)

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,

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

$\therefore d = t -$ clear cover

$= 6'' - 1''$

$= 5''$

DL = Self wt.

$= \gamma_{conc} \times t$

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

$= 75 \ lb/ft^2$

$W_u = 1.2 \ DL + 1.6 \ LL$

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

$= 250$ psf

$= 0.25$ Ksf

$$\boxed{\text{Moment, } M_u \text{ (cantilever)} = \frac{W_u L^2}{2}} = \frac{0.25 \times 5^2}{2} = 3.125 \text{ K-ft}$$

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

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

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

$\therefore A_s = 0.142 \ in^2$

(P.T.O.)

Use, #3 ϕ bar.

$A_b = 0.11$ in^2

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

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

$= 9.29$ in c/c

$\approx 9''$ c/c (say)

Provide #3 bar @ 9" c/c

(Ans)

Tazhar Kabir

CUET (Civil)

Passing Year : 2018

Assistant Engineer (Civil)

Payra Port Authority

Contact : 01406316084.

email : tazhar.ppa@gmail.com