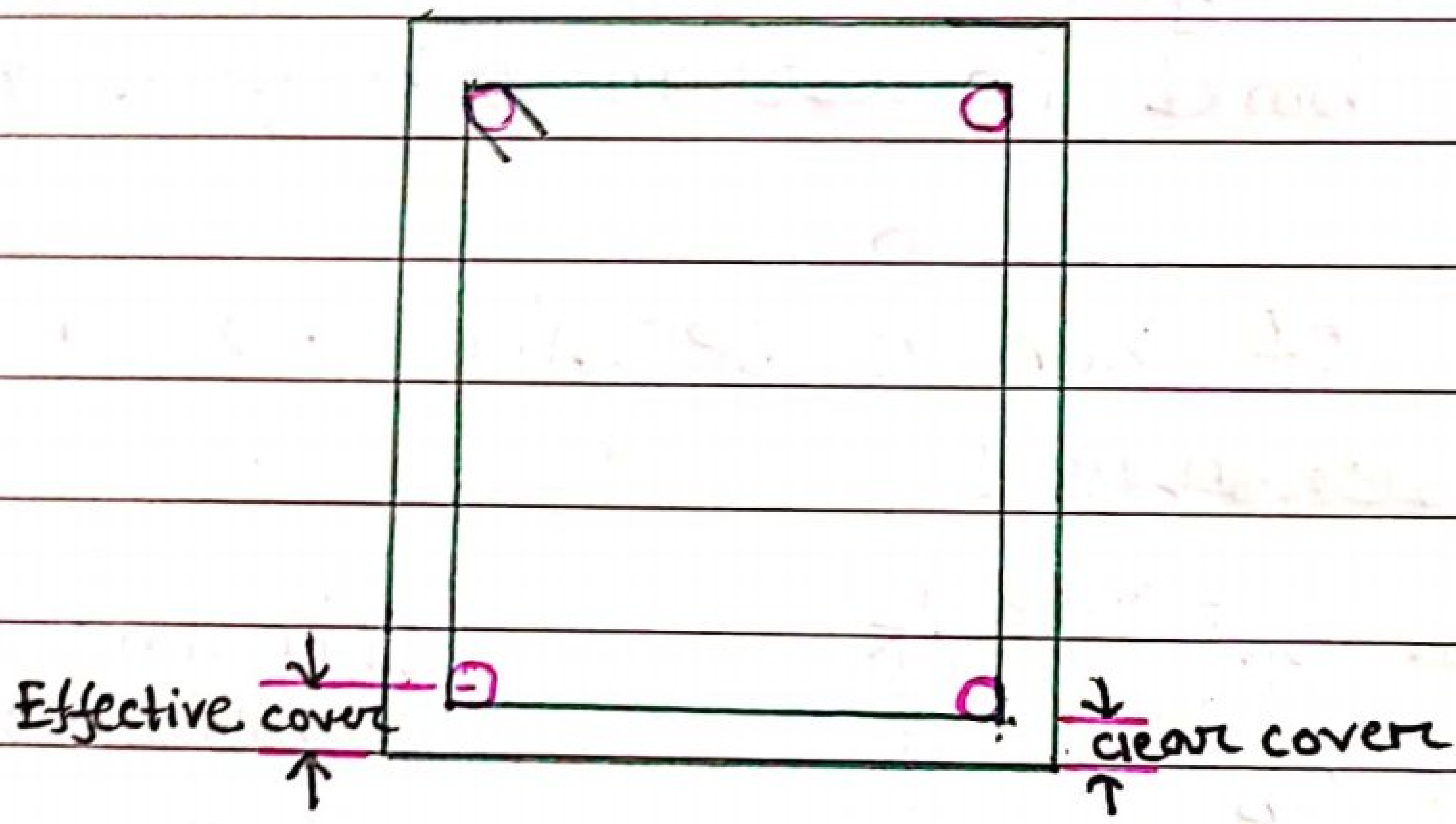
Q7 What is clear cover? Why is it used?

[CPGCBL'15] [RRI'15]

Ans: Clear cover is the distance between the outer surface of steel bar to the concrete face.

Clear cover is necessary for protecting the reinforcement from fire, corrosion & other effects.



Effective Cover: Clear cover + $\frac{1}{2}$ bar diameter

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Q9 What do you mean by (i) Under reinforced (ii) over reinforced (iii) balanced reinforced section? [DMLC'19]

Ans: Balanced Beam Section:

Reinforced concrete beam sections in which the tension steel also reaches yield strain (0.002) simultaneously as the concrete reaches the failure strain (0.003) in bending are called balanced sections.

Under-Reinforced Beam Section:

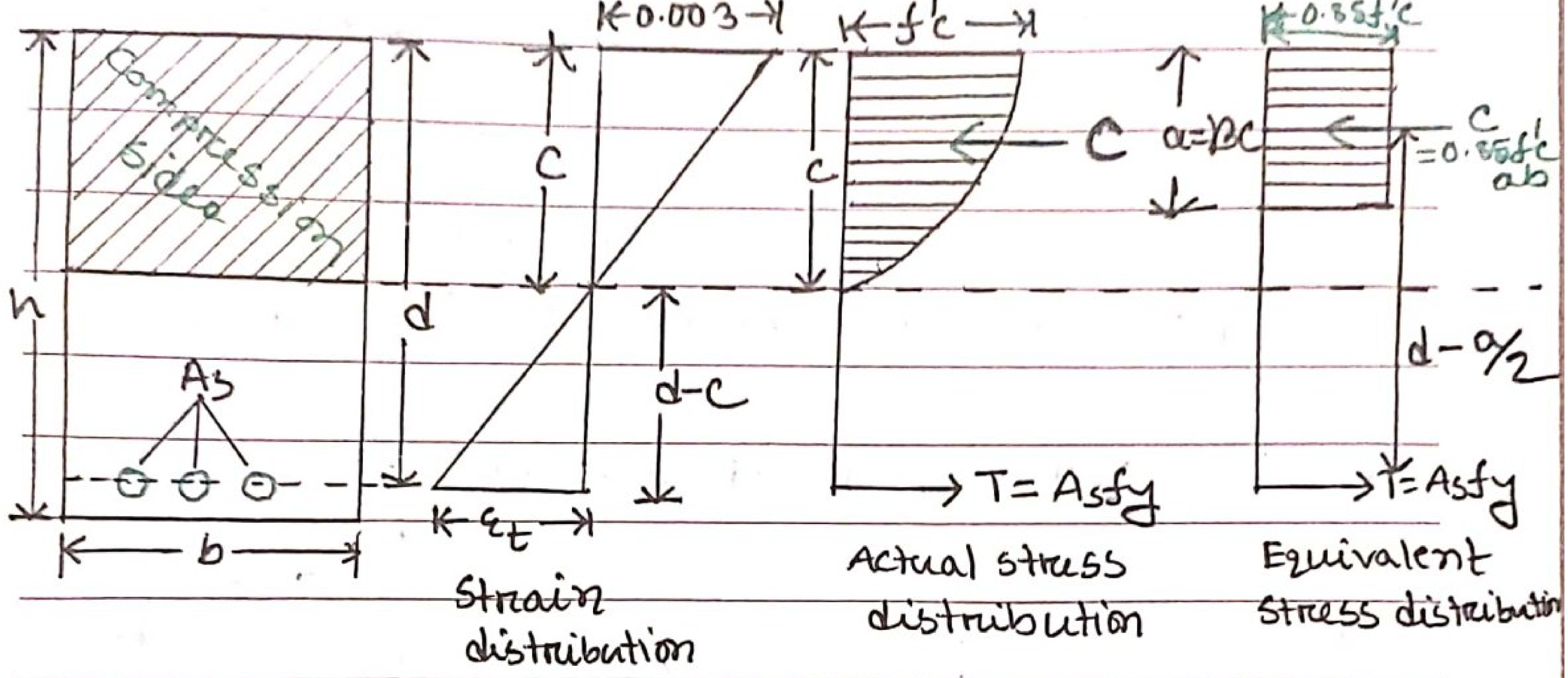
Reinforced concrete beam sections in which the steel reaches yield strain (0.002) at loads lower than the load at which the concrete reaches failure strain (0.003) are called under reinforcement.

(concrete এর strain 0.003 হওয়ার আগেই steel strain 0.002 হয়ে যায়। যখন 0.003 হইতে হইতে steel strain 0.005 এর উপরে চলে যায়।)

Over-reinforced Beam Section:

Reinforced concrete beam sections in which the ~~failure~~ failure strain in concrete (0.003) is reached earlier than the yield strain of steel (0.002) is reached are called over-reinforced beam sections.

Equivalent stress block (Whitney) for Singly Reinforced Beam:



* equivalent rectangular block এর জন্য f'_c হয় যেমন $0.85f'_c$। উচ্চতা c থেকে কম a হইবে।

$\beta = 0.85$ for $f'_c \leq 4000$ psi

$= 0.85 - \dfrac{0.05(f'_c - 4000)}{1000} \geq 0.65$ for $f'_c > 4000$ psi

$= 0.8$ for 5000 psi

$= 0.75$ for 6000 psi

Formula derivation:

Compression (C) = Tension (T)

$\therefore 0.85 f'_c ab = A_s f_y$

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

% of steel $\rho = \dfrac{A_s}{bd}$

$\Sigma M = 0$ (Moment = Force × distance)

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

Ultimate moment, $M_u = \phi M_n$

$$= \phi A_s f_y (d - a/2)$$

ϕ = strength redeuction factor.
depends on beam section design type

From strain distribution figure,

$$\frac{\varepsilon_t}{d-c} = \frac{0.003}{c} \qquad c = \frac{a}{\beta}$$

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

If, $\varepsilon_t \geq 0.005$; tension controlled $\Rightarrow \phi = 0.9$

$0.004 \leq \varepsilon_t < 0.005$; transition section $\Rightarrow$

$$\left.\begin{aligned} \phi &= 0.65 + (\varepsilon_t - 0.002) \times \frac{250}{3} \\ &= 0.483 + 83.3\,\varepsilon_t \end{aligned}\right\}$$ এটার মুখস্ত করতে

If $\varepsilon_t < 0.004$; compression controlled → reject section

$\boxed{\text{Design moment } M_d \geq M_u}$

$$^*A_{s(min)} = \text{maximum} \begin{cases} \dfrac{3\sqrt{f'_c}}{f_y} bd & \text{for } f'_c \geq 4500\,psi \text{ (তখন এটার max}^m \text{ হবে)} \\ \dfrac{200}{f_y} bd & \text{for } f'_c < 4500\,psi \\ = 0.0033\,bd & \text{(এই ক্ষেত্রে এটার max}^m \text{ হবে)} \end{cases}$$

* f'_c এবং f_y সবসময়ই psi তে আসতে হবে। ksi তে আসলেই মিলবেনা।

Q10 Find out the ultimate strength of the beam. Given, the beam size 12"X18". 4 Nos of #8 bar is used as reinforcement. Given, $f'_c = 4$ ksi, $f_y = 60$ ksi, clear cover $= 1\frac{1}{2}$ inch [BWDB'18] [APSCL'20] [DMTCL'19] [BWMRI'22]

<u>Soln:</u> $A_s = 4 \times \frac{\pi}{4} \times \left(\frac{8}{8}\right)^2 = 3.14 \text{ in}^2$

$A_{s(min)} = 0.0033 \times 12 \times 16.5 = 0.65 \text{ in}^2$

$\therefore A_s > A_{s(min)}$ (OK)

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{3.14 \times 60}{0.85 \times 4 \times 12} = 4.62$ inch

$c = \frac{a}{\beta_1} = \frac{4.62}{0.85} = 5.43$ inch

$\varepsilon_t = \left(\frac{d-c}{c}\right) 0.003 = \frac{16.5 - \cancel{4.62}\ 5.43}{5.43} \times 0.003$

~~= 0.0061 > 0.005 (tension controlled)~~

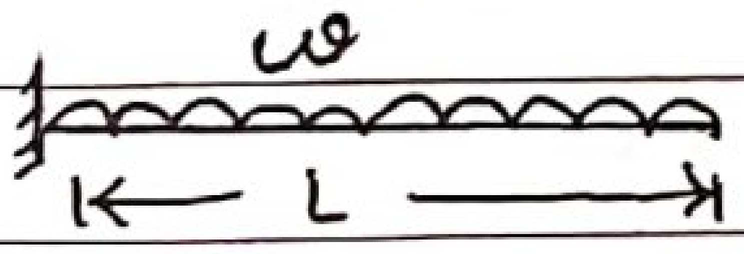
$= 0.0061 > 0.005$ (tension controlled)

$\therefore \phi = 0.9$

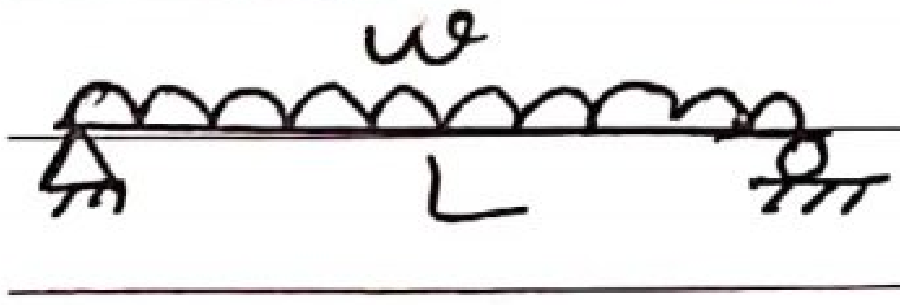
$\therefore M_u = \phi A_s f_y \left(d - \frac{a}{2}\right)$

$= 0.9 \times 3.14 \times 60 \left(16.5 - \frac{4.62}{2}\right)$

$= 2406$ k-in Ans



$M_{max} = \frac{wL^2}{2}$ Cantilever beam with UDL



$M_{max} = \frac{wL^2}{8}$ simply supported beam with UDL

Important formula for moment calculation in beam design

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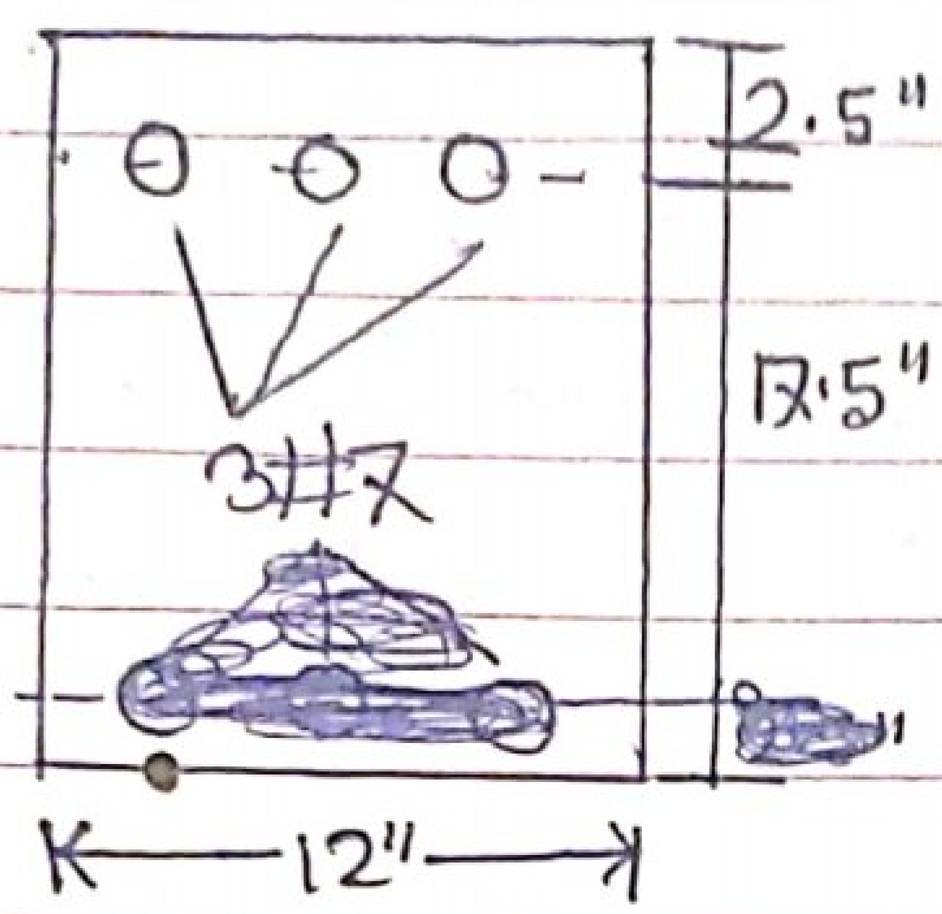
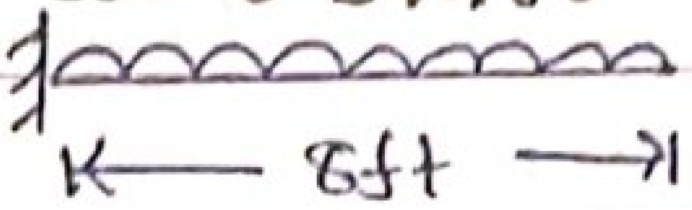
Q11



DL = 1.5 k/ft (including self weight)

LL = 0.9 k/ft



$f'_c = 4\,ksi$

$f_y = 60\,ksi$

[PGCB'19]
[EGCB'20]
[BCIC'16]

Check wheather the beam is safe in flexure?

Soln: $W_u = [1.2 \times 1.5 + 1.6 \times 0.9]\ k/ft$

$= 3.24\ k/ft$

including self weight বলা না দিলে আলাদাভাবে self weight calculation করতে হবে dead load এ যাবে। formula for self weight $= \frac{b \times h}{144} \times 150$

$\because$ concrete unit weight = 150 pcf

$\therefore$ Moment, $M = \frac{wl^2}{2} = \frac{3.24 \times 8^2}{2} = 103.68\ k\text{-}ft$

$= 1244.16\ k\text{-}in$

$A_s = 3 \times \frac{\pi}{4} \times \left(\frac{7}{8}\right)^2 = 1.8\ inch^2$

$A_{s(min)} = 0.0033 \times 12 \times 17.5 = 0.693\ inch^2$

$\therefore A_s > A_{s(min)}$ (ok)

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{1.8 \times 60}{0.85 \times 4 \times 12} = 2.65\ inch$

$c = \frac{a}{\beta} = \frac{2.65}{0.85} = 3.12\ inch$

$\varepsilon_t = \left(\frac{d-c}{c}\right) \times 0.003 = \frac{17.5 - 3.12}{3.12} \times 0.003$

$= \cancel{0.002} \cancel{\geq 0.005}$

$= 0.014 > 0.005$

P.T.O

~~Therefore the beam is not safe, as it's~~ ~~compression controlled section~~.

~~However for~~ tension ~~compression~~ controlled section

$\therefore \cancel{\phi = 0.65}$ $\therefore \phi = 0.9$

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

$$= 0.9 \times 1.8 \times 60 \left(17.5 - \frac{2.65}{2}\right)$$

$$= \cancel{1135.5 < 1244.16 \text{ (not safe)}}$$

$$= 1572 > 1244.16 \text{ (beam is safe)}$$

Reinforcement Bar Size

Bar No#	Dia (in)	Dia (mm)
3	3/8	10
4	4/8	12
5	5/8	16
6	6/8	19
7	7/8	22
8	8/8	25
9	9/8	29

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$\boxed{Q12}$ A rectangular beam has -
$b = 12''$, $d = 21''$, $h = 24''$, 3 Nos 16mm bar.
$f_y = 60ksi$, $f'_c = 3ksi$. calculate the max^m moment capacity of the beam.

[Padma bridge'15]

Soln: $A_s = 3 \times \frac{\pi}{4} \times \left(\frac{5}{8}\right)^2$ [16mm = #5] [16÷3≈5]

$= 0.92\ in^2$

$A_{s(min)} = 0.0033 \times 12 \times 21 = 0.83\ in^2$

$\therefore A_s > A_{s(min)}$ [ok]

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.92 \times 60}{0.85 \times 3 \times 12} = 1.804\ in$

$c = \frac{a}{\beta} = \frac{1.804}{0.85} = 2.122\ in$

$\varepsilon_t = \left(\frac{d-c}{c}\right) 0.003 = \left(\frac{21 - 2.122}{2.122}\right) \times 0.003 = 0.027$
> 0.005

$\therefore$ tension controlled section $\Rightarrow \phi = 0.9$

$\therefore M_u = 0.9\, A_s f_y \left(d - a/2\right)$
$= 0.9 \times 0.92 \times 60 \left(21 - \frac{1.804}{2}\right)$
$= 998\ k\text{-}in$ Ans

or, $A_s = 3 \times \frac{\pi}{4} \times \left(\frac{16}{25.4}\right)^2$ $\boxed{1\ inch = 25.4mm}$
$= 0.935\ in^2$

Q 13

[66] A rectangular beam has width of 12inch, effective depth 17.5 inch. It is reinforced with 4#9bar in one row. what is the ultimate moment capacity of the beam? Also check what would be the φ value? Given, $f_y = 60ksi$, $f'_c = 4ksi$

[BWDB'16][DESCO'15]

Solⁿ: $A_s = 4 \times 1 = 4 in^2$ [Area of #9 bar = $1 in^2$]

$A_{s(min)} = 0.0033 \times 12 \times 17.5 = 0.693 \, inch^2$

$\therefore A_s > A_{s(min)}$ [OK]

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

$$c = \frac{5.88}{0.85} = 6.92 \, in$$

$$\varepsilon_t = \left(\frac{d-c}{c}\right) \times 0.003 = \left(\frac{17.5 - 6.92}{6.92}\right) \times 0.003 = 0.0046 < 0.005$$

∴ Transitional section

$$\therefore \varphi = 0.65 + (\varepsilon_t - 0.002) \times \frac{250}{3}$$
$$= 0.65 + (0.0046 - 0.002) \times \frac{250}{3}$$
$$= 0.87$$

∴ Ultimate moment capacity, $M_u = \phi A_s f_y (d - a/2)$

$$= 0.87 \times 4 \times 60 (17.5 - \frac{5.88}{2})$$
$$= 3040 \, k\text{-}in$$

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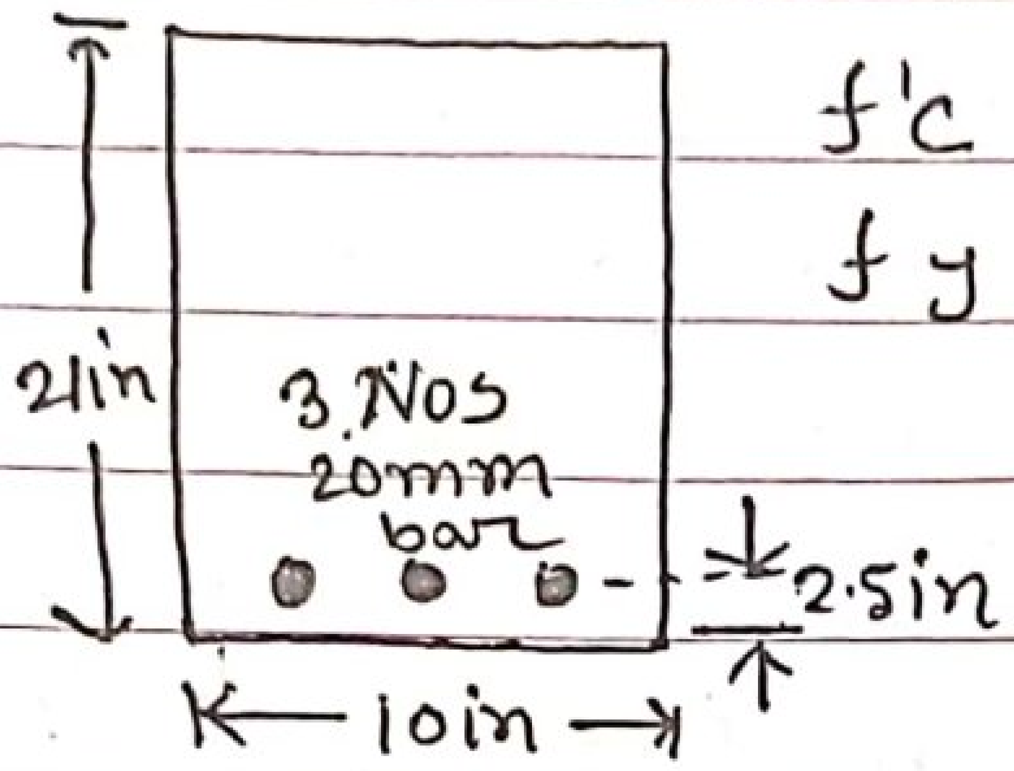
Q14



$f'c = 3000\,psi$

$fy = 60{,}000\,psi$

check whether yielding or concrete cracking failure [DWASA'17]

Solⁿ: $A_s = 3 \times \frac{\pi}{4} \times \left(\frac{6}{8}\right)^2$ [$20 \div 3 \approx \#6$]

$= 1.325\ in^2$

$A_{s(min)} = 0.0033 \times 10 \times (21 - 2.5)$

$= 0.61\ in^2$

$\therefore A_s > A_{s(min)}$ [okay]

এই mathএ এটা না করলেও চলবে

$a = \frac{A_s f_y}{0.85 f'c\, b} = \frac{1.325 \times 60}{0.85 \times 3 \times 10} = 3.12\ in$

$\therefore c = \frac{3.12}{0.85} = 3.67\ in$

$\varepsilon_t = \frac{d-c}{c} \times 0.003 = \frac{18.5 - 3.67}{3.67} \times 0.003$

$= 0.012 > 0.005$

$\therefore$ This beam will fail in steel yielding

Ans

* A beam 18 inch width and 24 inch depth is used 5#8 reinforcement. The beam is under reinforcement with $f'_c = 4000psi$, $f_y = 72500psi$. calculate the nominal flexural strength

[SPCBL'21]

Soln:

$A_s = 5 \times \frac{\pi}{4} \times \left(\frac{8}{8}\right)^2 = 3.927 in^2$

$\boxed{d = 24 - 2.5 = 21.5}$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{3.927 \times 72.5}{0.85 \times 4 \times 18} = 4.65$ inch

$\therefore c = \frac{a}{\beta} = \frac{4.65}{0.85} = 5.47$ inch

$M_n = A_s f_y (d - a/2) = 3.927 \times 72.5 \times \left(21.5 - \frac{4.65}{2}\right)$

$= 5459.26$ k-in

$= 454.94$ k-ft

Nominal flexural strength, $\delta_n = \frac{M_n C}{I}$

$= \frac{5459.26 \times 5.47}{\frac{18 \times 24^3}{12}}$

$= 1.44$ ksi

$= 207.376$ ksf

flexural capacity বললে moment কেই বোঝানো হবে কিন্তু flexural strength বললে অবশ্যই δ_n কে বোঝাতে হবে।

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Left out Maths

* The cross section of a beam reinforced with $3 in^2$ of steel. Assume the tension steel yields at maximum moment. What are the area of concrete required to balance the steel force when the steel yields? $f'_c = 4ksi$ & $f_y = 60ksi$.

Soln: [NESCO'21]

Tensile force = Compression force

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

$\Rightarrow 3 \times 60 = 0.85 \times 4 \times A_c$

$\Rightarrow A_c = 52.94 \ in^2$ $\boxed{\text{Ans.}}$

T Beam

Analysis procedure:

(i) calculate b_e

(ii) check $A_s > A_{s(min)}$

(iii) Assume section as rectangular beam
($a \le h_f$)

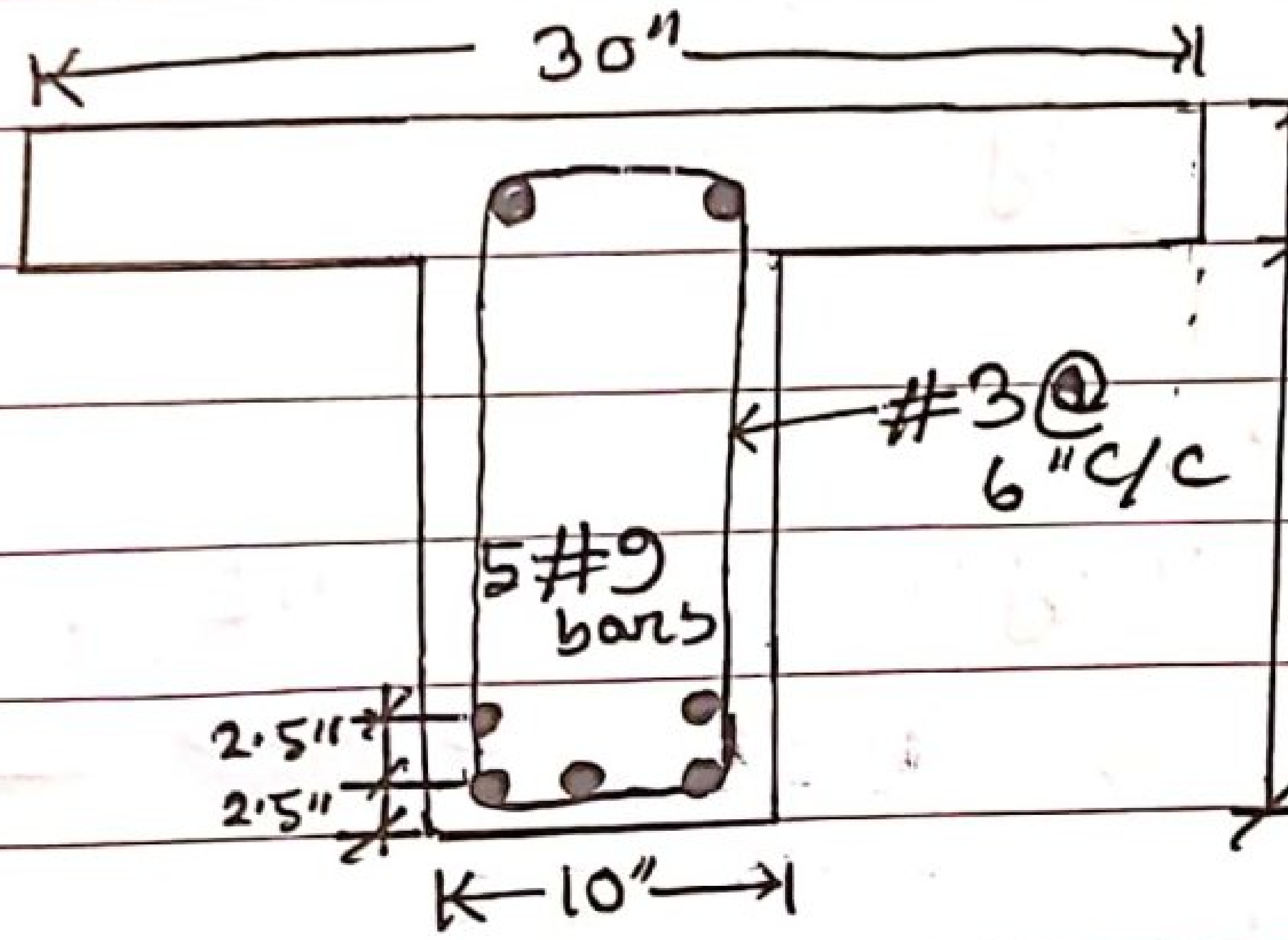
$$\therefore a = \frac{A_s f_y}{0.85 f'_c b_e}$$

[কোনোভাবে ভুলেও হবে b_e not b_w]

If $a \le h_f$ = Assumption is correct.
Analyze as rectangular beam

Q18 Check if it is T or rectangular beam. Determine the ultimate moment capacity of the beam. $f_y = 40ksi$, $f'_c = 3ksi$

[BPDB'18] [GTCL'18] [RPGCL'17]



Soln: $d = (20+3) - (2.5 + \frac{2.5}{2}) = 19.25"$

$A_s = 5 \times 1 = 5 in^2$ (area of #9 bar = $1 in^2$)

$A_{s(min)} = 0.0033 \times 10 \times 19.25 = 0.64 in^2$

$\therefore A_s > A_{s(min)}$ [Okay]

$a = \frac{A_s f_y}{0.85 f'_c b_e} = \frac{40 \times 5}{0.85 \times 3 \times 30} = 2.614 in$

$\therefore a < h_f = 3"$

$\therefore$ This is rectangular beam!

$c = \frac{2.614}{0.85} = 3.075$

$\varepsilon_t = \frac{d-c}{c} \times 0.003 = \frac{19.25 - 3.075}{3.075} \times 0.003$

$= \cancel{0.016} = 0.016 > 0.005$

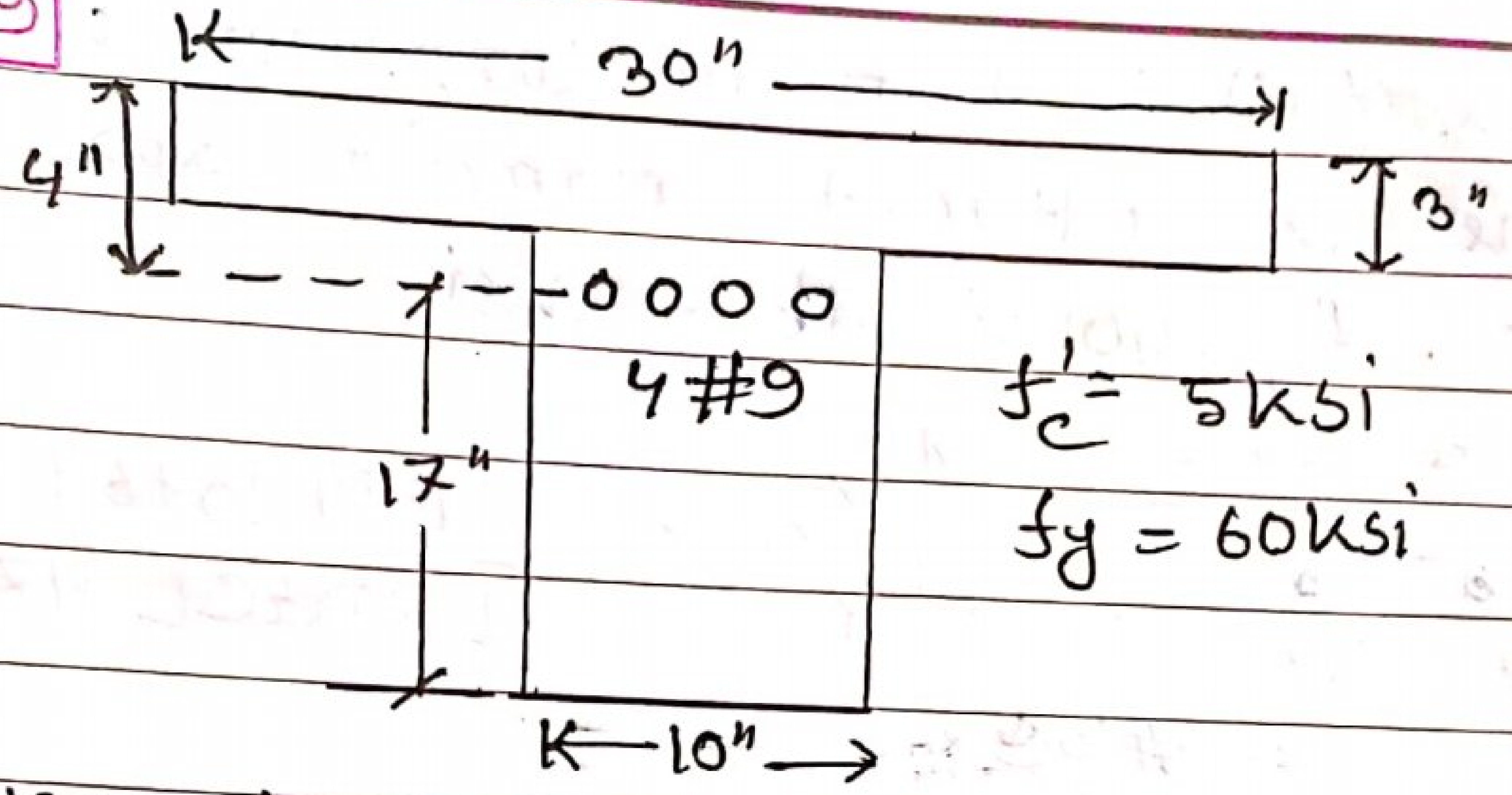
$\therefore$ Tension controlled $\Rightarrow \phi = 0.9$

$\therefore$ Ultimate moment, $M_u = \phi A_s f_y (d - a/2)$

$= 0.9 \times 5 \times 40 \times (19.25 - \frac{2.614}{2})$

$= 3229.74$ k-in

Q.19.



$f'_c = 5\text{ksi}$

$f_y = 60\text{ksi}$

Determine nominal negative moment.

[PGCB'18]

Solⁿ:

When T-beams are subjected to negative moments, the flange is located in the tension zone. Since concrete strength in tension zone is usually neglected in USD, the sections are treated as rectangular sections of width b_w.

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

$A_{s(min)} = \dfrac{3\sqrt{f'_c}}{f_y} bd \quad [\because f'_c = 5\text{ksi}]$

$= \dfrac{3 \times \sqrt{5000}}{60{,}000} * 10 * 17$

$= 0.601 \text{ in}^2 < A_s$ (okay)

$a = \dfrac{A_s f_y}{0.85 f'_c b_w} = \dfrac{4 \times 60}{0.85 \times 5 \times 10} = 5.65 \text{ in}^2$

$c = \dfrac{a}{\beta} = \dfrac{5.65}{0.80} = 7.0625 \text{ in}$

$[\beta = 0.8 \text{ for } 5\text{ksi}]$

$$\varepsilon_t = \left(\frac{d-c}{c}\right) \times 0.003 = \frac{17 - 7.0625}{7.0625} \times 0.003$$

$$= 0.0042 \quad (\text{Transition section}) \; < 0.005$$

$\therefore$ ~~ϕ~~

$$\phi = 0.65 + (0.0042 - 0.002) \times \frac{250}{3}$$

$$= 0.83$$

$\therefore$ ~~$M_u = \phi$~~

$$M_n = A_s f_y \left(d - a/2\right)$$

$$= 4 \times 60 \times \left(17 - \frac{5.65}{2}\right)$$

$$= 3402 \text{ k-in Ans}$$

$$M_u = \phi M_n = 0.83 \times 3402 = 2823.66 \text{ k-in}$$

যেহেতু ultimate moment চায় নাই, সেহেতু c, ε_t, ϕ এদের কোন দরকার নাই।

$$\rho_{max} = \frac{0.003}{0.003+0.004} \times \left(\frac{0.85\beta f'_c}{f_y}\right)$$

$$= \frac{3}{7} \times \left(\frac{0.85\beta f'_c}{f_y}\right)$$

If $\rho \leq \rho_{max} \Rightarrow$ singly beam

If $\rho \geq \rho_{max} \Rightarrow$ doubly beam

$$\rho_{0.005} = \frac{0.003}{0.003+0.005} \times \left(\frac{0.85\beta f'_c}{f_y}\right)$$

$$= \frac{3}{8}\left(\frac{0.85\beta f'_c}{f_y}\right)$$ ধরে হিসাব করলে

$\rho \leq \rho_{max}$ হলে

singly reinforcement beam এর জন্য

$\phi = 0.9$ হিসাব না করেই ধরে নেয়া যায়।

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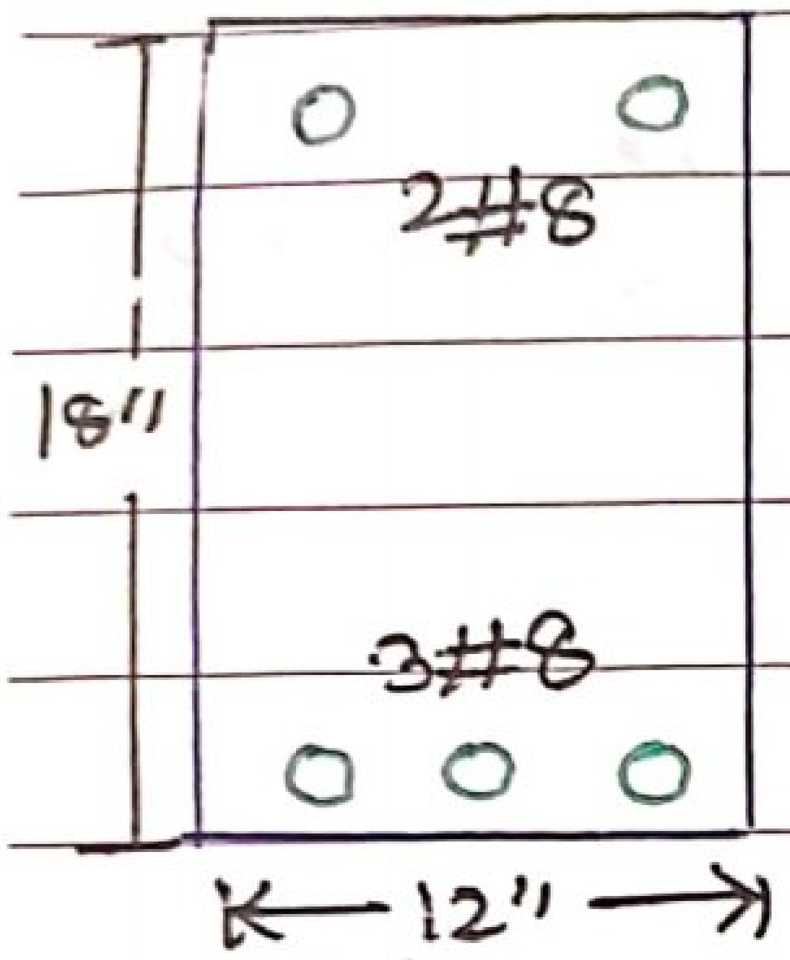
Q20 Determine ultimate capacity of a beam if clear cover 1.5 inch, $f'_c = 4$ ksi, $f_y = 60$ ksi

[NPDBL '17] [BIWTA '19]



Sol^n: Tensile steel ratio, $\rho = \frac{2.36}{12 \times 16.5} = 0.012$

$$\rho_{0.005} = \frac{3}{8} \times \frac{0.85\beta f'_c}{f_y} = \frac{3}{8} \times \frac{0.85 \times 0.85 \times 4}{60}$$

$$= 0.018 > \rho = 0.012$$

$\therefore$ singly RC beam analysis required!

Also $\phi = 0.9$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{2.36 \times 60}{0.85 \times 4 \times 12} = 3.47 \text{ inch}$$

$$\therefore M_u = \phi A_s f_y (d - a/2)$$

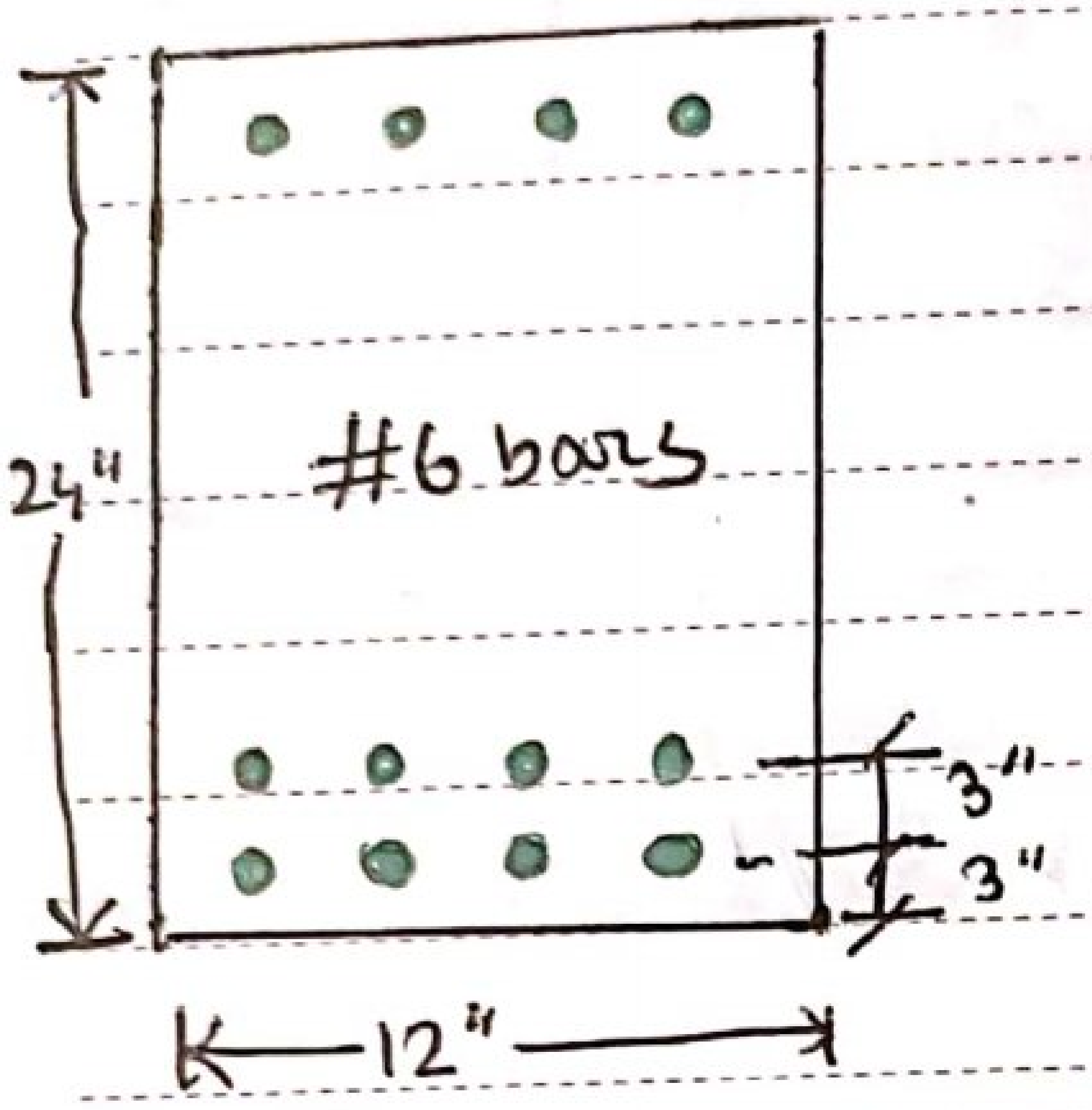
$$= 0.9 \times 2.36 \times 60 \left(16.5 - \frac{3.47}{2}\right)$$

$$= 1881.65 \text{ k-in}$$ Ans

$$\rho_{max} = \frac{3\beta}{7} * \frac{0.85 f'_c}{f_y}$$

(6) Check if the beam is either singly reinforced or doubly reinforced beam. If materials strengths are $f'_c = 4ksi$ & $f_y = 60ksi$, then find the ultimate moment capacity of the beam.

[BAPEX 23]



effective depth, $d = 24 - 3 - \frac{3}{2}$

$= 19.5$ in

Soln: Tensile steel ratio, $\rho = \dfrac{\frac{\pi}{4} \times \left(\frac{6}{8}\right)^2 \times 8}{12 \times 19.5} = 0.015$

$$\rho_{0.005} = 0.85\beta \frac{f'_c}{f_y} \times \frac{0.003}{0.003+0.005}$$

$$= 0.85 \times 0.85 \times \frac{4}{60} \times \frac{3}{8} = 0.018$$

$\because \rho_{0.005} > \rho$ $\therefore$ singly reinforced beam.

again, since, $\rho_{0.005} > \rho$ therefore, $\phi = 0.9$

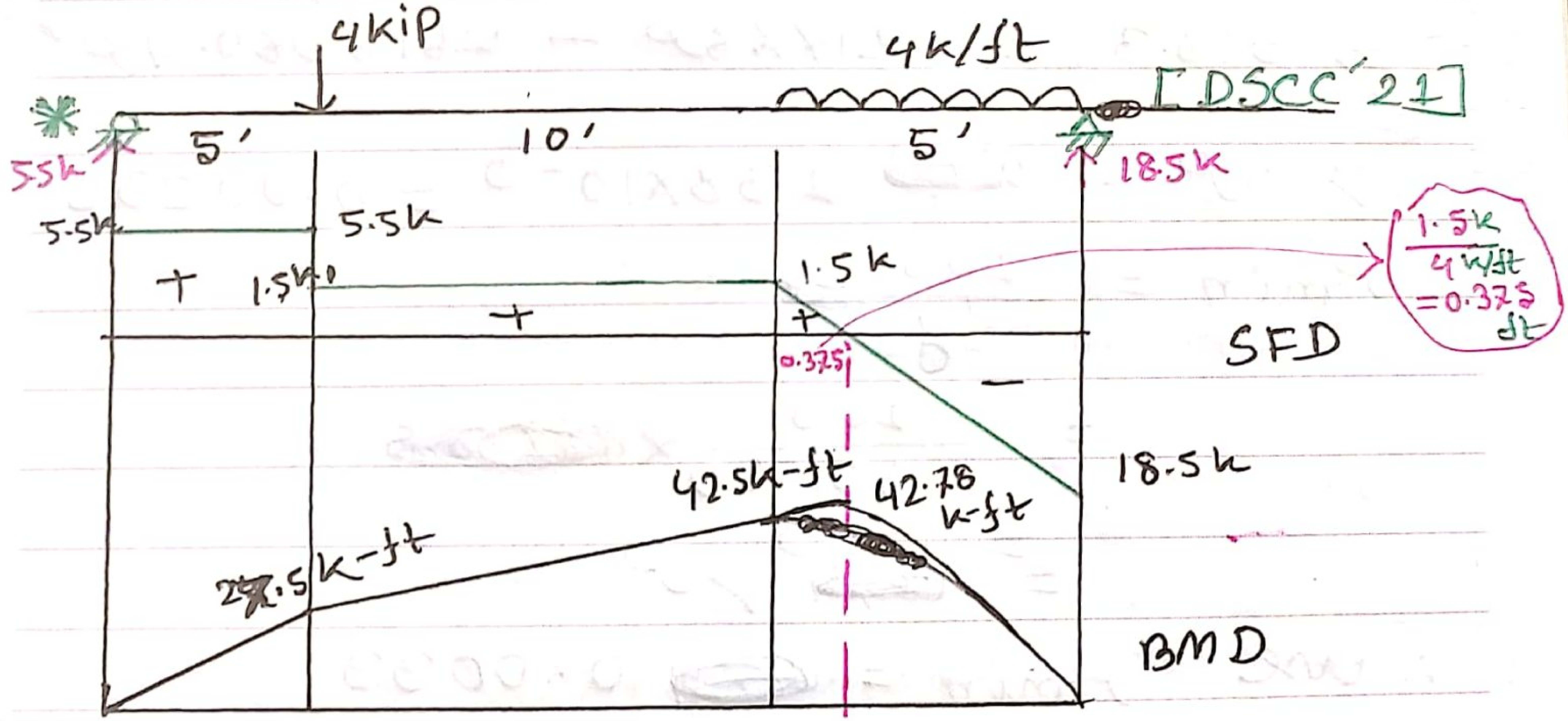
$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{\frac{\pi}{4} \times (6/8)^2 \times 8 \times 60}{0.85 \times 4 \times 12} = 5.197 \text{ inch}$$

$$\therefore M_u = \phi A_s f_y (d - a/2)$$

$$= 0.9 \times \frac{\pi}{4} \times (6/8)^2 \times 8 \times 60 \times \left(19.5 - \frac{5.197}{2}\right)$$

$$= 3225.68 \text{ k-in}$$

Ans



* Design flexural reinforcement of the beam given in previous question. Assume, $f'c = 3k$, $fy = 60k$. Take, $b = 12''$, $h = 21''$ [DSCC'21]

Soln: Maximum moment, $M_{max} = 42.78$ k-ft

$= 42.78 \times 12$ k-in

$= 513.36$ k-in

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* Depth [illegible] =

$$\boxed{\text{Beam length (in)} = \text{Span length (ft)}}$$

Take, $d = 21 - 2.5 = 18.5''$

$$M_u = \phi A_s f_y (d - 0.59 \rho n d)$$

$$\Rightarrow M_u = \phi A_s f_y \left(d - 0.59 \rho \frac{f_y}{f'_c} d\right)$$

$$= \phi \rho b d \cdot f_y \left(d - 0.59 \rho \frac{f_y}{f'_c} d\right)$$

$$= \phi \rho b d^2 f_y \cdot \left(1 - 0.59 \rho \frac{f_y}{f'_c}\right)$$

$$513.36 = 0.9 \rho \times 12 \times (18.5)^2 \times 60 \left(1 - 0.59 \rho \frac{60}{3}\right)$$

$$\Rightarrow 513.36 = 221778 \rho - 2616980.4 \rho^2$$

$$\Rightarrow \therefore \rho = 2.38 \times 10^{-3} = 0.00238$$

[illegible] $\rho = 0.08$ [illegible]

$$\rho_{min} = \frac{200}{f_y}$$

$$= \frac{200}{60{,}000}$$

$$= 0.0033 > \rho$$

$\therefore$ use $\rho_{min} = 0.0033$

$\therefore A_s = 0.0033 \times 12 \times 18.5 = 0.74 \text{ in}^2$

$\therefore$ use 2 #6 bars (area 0.88 in^2)

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* A singly reinforced beam is to be designed to carry a service live load of 1300 lb/ft on a 26 ft simple span. Take $f'_c = 3$ ksi & $f_y = 60$ ksi

[CoxDA '22]

Sol^n: Take,

beam depth = 26 inch [beam depth (in) = span length (ft)]

$\therefore$ Effective depth, $d = (26 - 2.5)$ inch
$= 23.5$ inch

beam width, $b = 12$ inch

Factored load $= 1.2 \times$ self weight $+ 1.6 \times$ Live Load

$= 1.2 \times 150 \times \frac{26 \times 12}{12 \times 12} + 1.6 \times 1300$

$= 2470$ lb/ft $= 2.47$ k/ft

concrete unit weight = 150 lb/ft^3

$\therefore$ Moment, $M = \frac{wl^2}{8} = \frac{2.47 \times 26^2}{8}$

$= 208.715$ k-ft

$= 2505$ k-in

again,

$M_u = \phi \rho f_y b d^2 \left(1 - 0.59 \rho \frac{f_y}{f'_c}\right)$

$\Rightarrow 2505 = 0.9 \times \rho \times 60 \times 12 \times (23.5)^2 \left[1 - 0.59 \times \rho \times \frac{60}{3}\right]$

$\Rightarrow 2505 = 357858 \rho - 4222724.4 \rho^2$

$\therefore \rho = 0.077$ or $\rho = 0.0077$

$\therefore \rho = 0.0077$ [সবচেয়ে ছোটটির use করা হয়।]

$\therefore \rho_{min} = \frac{200}{f_y} = \frac{200}{60,000} = 0.0033$

$\rho_{max} = 0.85 \beta \times \frac{3}{8} \times \frac{f'_c}{f_y}$

$= 0.85 \times 0.85 \times \frac{3}{8} \times \frac{3}{60} = 0.0135$

$\beta = 0.85$ for ≤ 4000 psi
$= 0.80$ for 5000 psi
$= 0.75$ for 6000 psi

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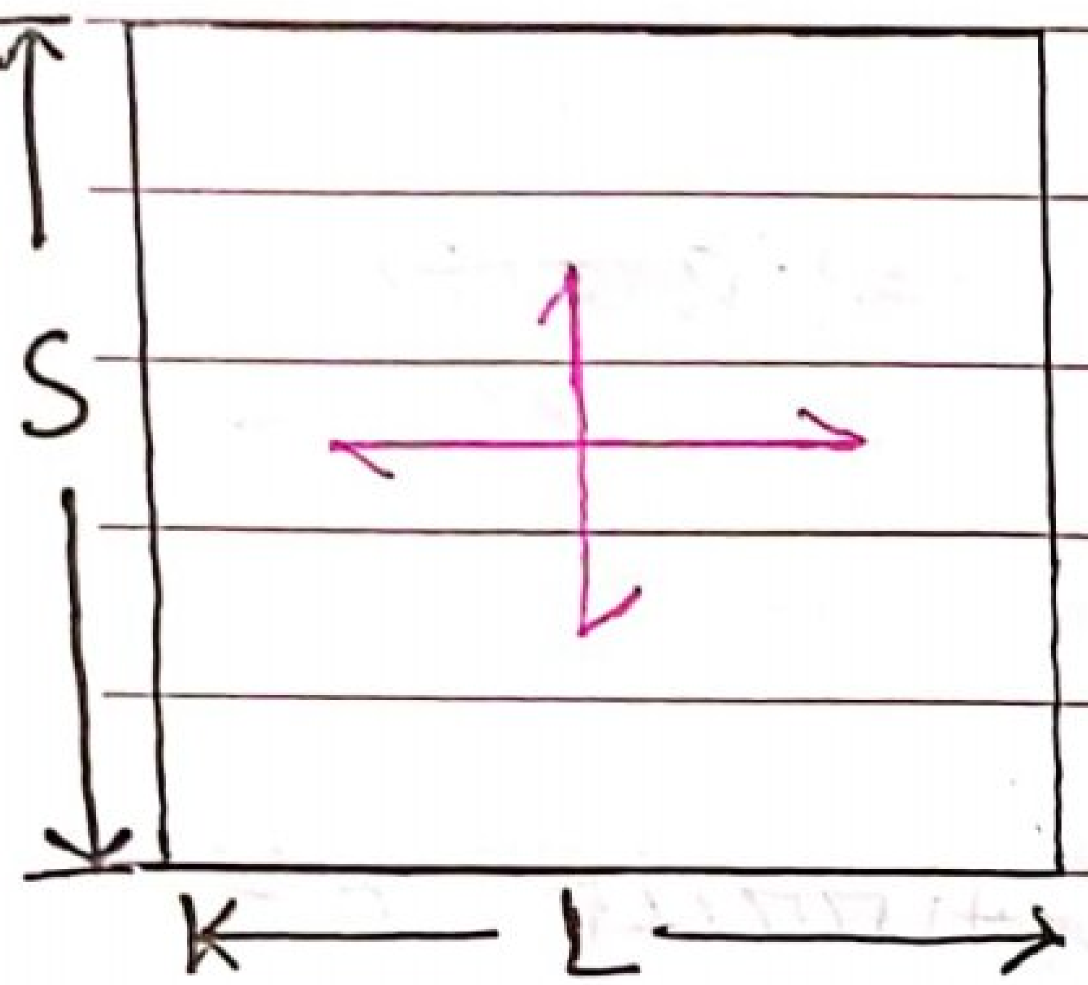
$\therefore \rho_{min} < \rho < \rho_{max}$ (ok)

$\therefore A_s = \rho bd = 0.0077 \times 12 \times 23.5$

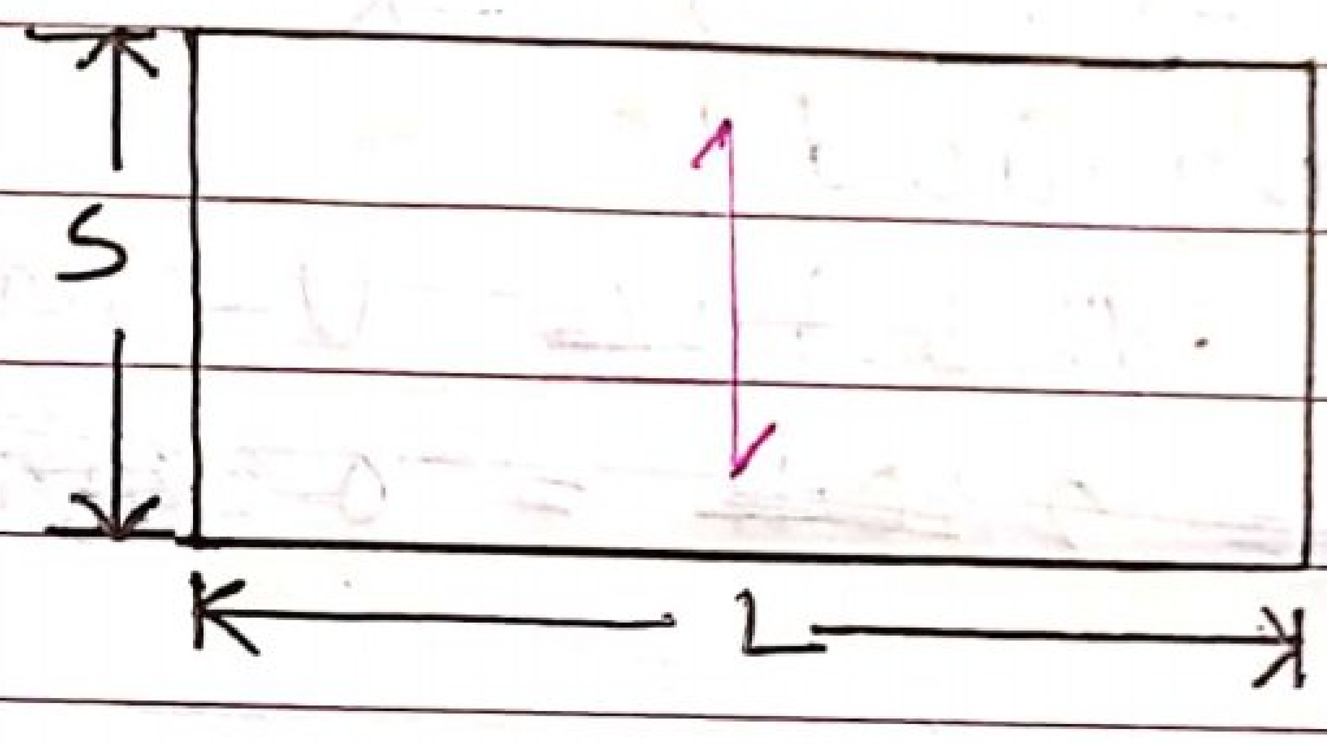
$= 2.17 in^2$

$\therefore$ use 3nos No.8 bars ($A_s = 2.356 in^2$)

Slab Design & Analysis



Two way slab $L/s < 2$



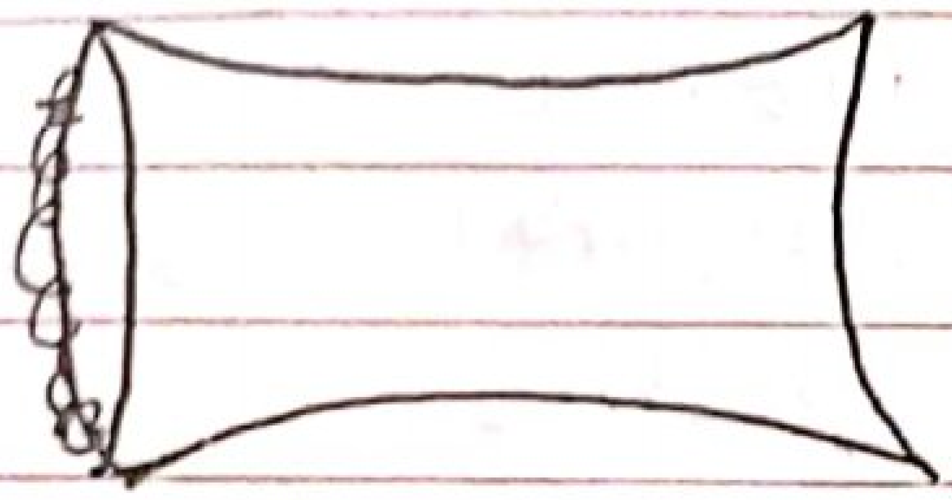
One-way slab $L/s \geq 2$

A one way solid slab curves under loads in one direction only. এরজন্য longer side এ ২ টি beam দেওয়া হয়।

এই bending টা কন্ট্রোল করার জন্য one way slab এ shorter direction এ main reinforcement provide করা হয়। আর longer direction এ দেওয়া হয় shrinkge/temperature/distribution steel.

In slab, short direction bar is placed below the long direction bar [GTCL'16]

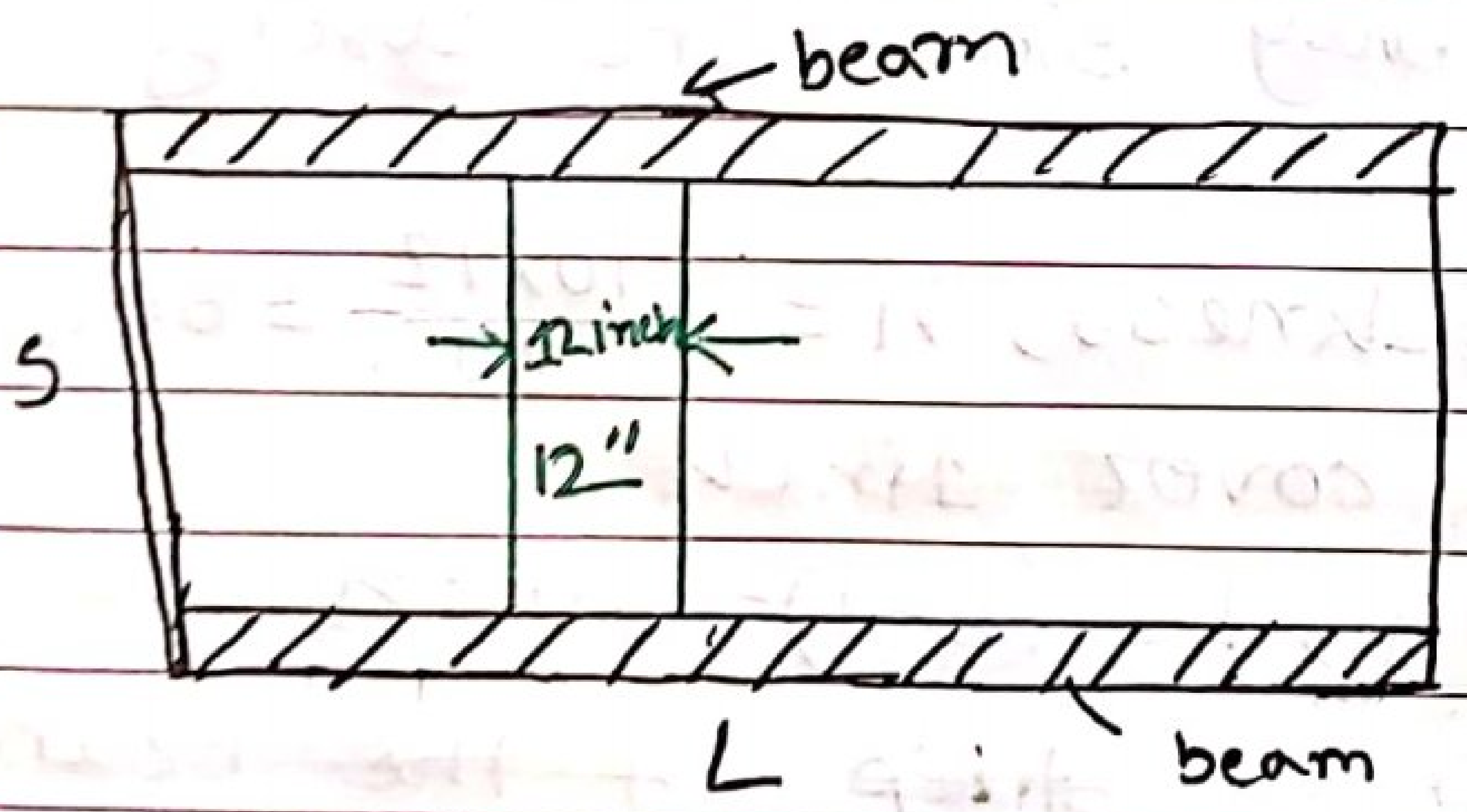
temperature change এর জন্য shrinkage crack প্রতিরোধী এই steel use করা হয়।



two way slab এ দুই direction এই bending হয়। সেজন্য চারপাশেই beam দেয়া হয়। এবং short, long দুই direction এই main reinforcement থাকে।

minimum thickness of oneway slab : [BCPCL'16] [DNLC'15]

Simply supported = $L/20$
one end continuous = $L/24$
both end continuous = $L/28$
cantilever = $L/10$



Long direction এ 12 inch এর একটি strip নিয়ে beam design এর principle অনুযায়ী slab design করা হয়।
here, $b = 12$ inch

Minimum area of shrinkage/temperature/ distribution bar :

For $f_y = 60{,}000\,psi$ (420MPa)

$$A_{min} = 0.0018bh$$

For $f_y < 60{,}000\,psi$ (420 MPa)

$$A_{min} = 0.002bh$$

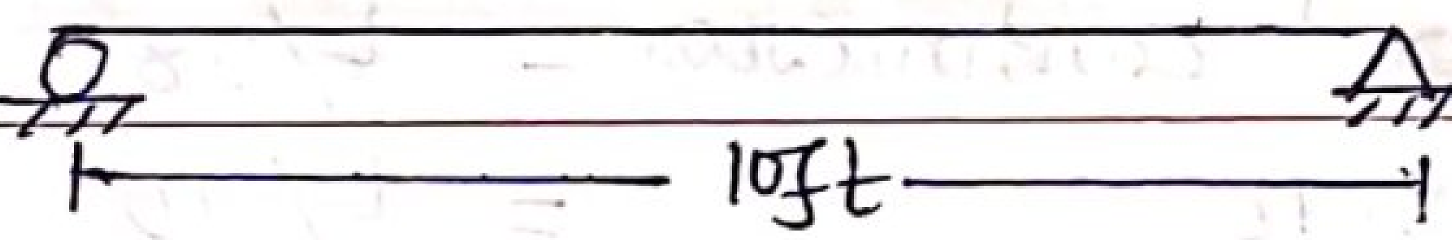
for $f_y > 60{,}000\,psi$ (420 MPa)

$$A_{min} = \frac{0.0018 \times 60000\,bh}{f_y}$$

* 500 MPa = 72 ksi

420 MPa = 60 ksi [1MPa = 145psi]

Example :



LL = 200psf

$f'_c = 4000\,psi$

$f_y = 60{,}000\,psi$

Design a one way slab for using this data.

Sol^{n}: Slab thickness, $h = \frac{10 \times 12}{20} = 6\,in$

Assume effective cover 1inch

$\therefore$ effective depth, $d = (6-1) = 5\,inch$

~~Take a b=12inch strip of the beam.~~

~~∴ DL = slab weight = 1... $\frac{12}{12} \times \frac{6}{12}$~~

Design a 12-in strip of the slab. Thus take b=12inch

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$DL = \text{slab weight} = \frac{12}{12}\times\frac{6\,in}{12}ft \times 150\,lb/ft^3$

$= \cancel{75\,psf} = 75\,lb/ft$

$LL = 200psf/1ft = 200\,lb/ft$ [b=12in=1ft]

$w_u = 1.2\times75 + 1.6\times200 = 410\,lb/ft = 0.41\,k/ft$

$M_u = \frac{0.41\times10^2}{8} = 5.125\,k\text{-}ft$

$\therefore$ steel ratio, $\rho = \frac{0.85f'_c}{f_y}\left[1-\sqrt{1-\frac{2M_u}{0.85\phi f'_c bd^2}}\right]$

Assume, $\phi = 0.9$

$\therefore \rho = \frac{0.85\times4}{60}\left[1-\sqrt{1-\frac{2\times5.125\times12}{0.85\times0.9\times4\times12\times5^2}}\right]$

$= 0.00393$

$\rho_{max} = \frac{3}{8}\left(\frac{0.85\beta f'_c}{f_y}\right) = \frac{3}{8}\times\left(\frac{0.85\times0.85\times4}{60}\right)$

$= 0.018 > \rho$

$\therefore \phi = 0.9$ (okay)

$A_s = \rho bd = 0.00393\times12\times5 = 0.236\,in^2/ft$

use #4 bar

spacing, $s = \frac{12\times\frac{\pi}{4}\times\left(\frac{4}{8}\right)^2}{0.236} \approx 10\,inch$

$\boxed{\text{spacing } s = 12\times\frac{a_s}{A_s}}$

$\left[s = \frac{b\,a_s}{A_s}\right]$

< max^m spacing = 18inch

or $3h = 3\times6 = 18$ inch

(ok)

Shrinkage reinforcement Area

$A_s = 0.0018\,bh$

$= 0.0018\times12\times6$

$\simeq 0.13\,in^2/ft$

~~$A_s = 0.0018bh$~~

$= \cancel{0.0018\times12\times6}$

$=$

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use #3 bar

$$\text{spacing} = \frac{12 \times \pi/4 \times (3/8)^2}{0.13} = 10 \text{ inch} < 18 \text{ inch (ok)}$$

WI

* 15'×15' আকারের RCC slab এ #4 steel @8"c/c spacing এ temperature steel হিসাবে ব্যবহার করা হয়েছে। উক্ত slab এ #3 bar ব্যবহার করলে spacing কত হবে?

Soln:

$$\text{spacing} = \frac{bA_b}{A_s}$$

$$8 = \frac{12 \times \pi/4 \times (4/8)^2}{A_s}$$

$$\Rightarrow A_s = 0.295 \text{ in}^2$$

Second case,

$$\text{spacing} = \frac{12 \times \frac{\pi}{4} \times \left(\frac{3}{8}\right)^2}{0.295}$$

$$= 4.5 \text{ inch c/c}$$

Alternative

Spacing

$$\frac{S_2}{S_1} = \frac{d_2^2}{d_1^2}$$

$$\Rightarrow S_2 = \frac{(3/8)^2}{(4/8)^2} \times 8$$

$$= 4.5 \text{ in c/c}$$

* Find the spacing of #3 steel bars to be used as temperature and shrinkage reinforcement in one-way slab having thickness of 6".
Consider 0.18% reinforcement for temperature and shrinkage. [BWMRI'2022]

Solⁿ:

$$A_{s\,(req)} = 0.0018bt$$

$$= 0.0018 \times 12 \times 6$$

$$= 0.13\ in^2/ft$$

$$\text{Spacing} = \frac{A_s\,(provided)}{A_s\,(req)} \times 12$$

$$= \frac{0.11}{0.13} \times 12$$

$$\approx 10\ \text{inch c/c}$$

area of #3 bar $= \frac{\pi}{4} \times \left(\frac{3}{8}\right)^2$

$= 0.11\ in^2$

⊞ (29) A designer used 60 grade deformed bar for the design of 6" RCC one way slab. He used #4 bar with spacing of 5" c/c distance for main bar reinforcement. A second designer wanted to use 500W bar instead of 60 grade bar. Determine the spacing of the main reinforcement if the 2nd designer use 12mm 500W bar instead of 60 grade #4 bars. [BWDB'19] [BCIC'19] [BADC'20] [PGCB'21] [BIFPCL'21] [CPGCBL'22]

Sol^n: For first case,

$$\text{Spacing} = \frac{\pi/4 * (\frac{4}{8})^2 \times 12}{A_{s_1}} = 5$$

or, $A_{s_1} = 0.47 \text{ in}^2$

Again, $A_{s_1} f_1 = A_{s_2} f_2$

$$\Rightarrow A_{s_2} = \frac{A_{s_1} f_1}{f_2}$$

$$= \frac{0.47 * 60}{72.5}$$

$$= 0.39 \text{ in}^2$$

$f_2 = 500W$

$= 500 \times 145$

$= 72500 \text{ psi}$

$= 72.5 \text{ ksi}$

for second case,

$$\text{spacing} = \frac{A_b \times 12}{A_{s_2}}$$

$A_b = \frac{\pi}{4} * \frac{12^2}{(25.4)^2}$

$= 0.175 \text{ in}^2$

$$= \frac{0.175 \times 12}{0.39} = 5.385 \text{ inch c/c}$$

Ans

Shortcut

$$S_2 = \frac{f_{y_2}}{f_{y_1}} \times \frac{A_{b_2}}{A_{b_1}} \times S_1 = \frac{72.5}{60} \times \frac{0.175}{0.196} \times 5 = 5.39'' \text{ c/c}$$

Shortcut: $S_2 = \frac{f_{y_2}}{f_{y_1}} \times \frac{d_2^2}{d_1^2} \times S_1 = \frac{72.5}{60} \times \frac{(\frac{12}{25.4})^2}{(4/8)^2} \times 5 = 5.39$ inch Ans

~~∴ Ultimate shear strength, V_u = 0.75 (23.36 + 51.54) = 56.625 kips~~

Ans

Shear Design & Analysis of Beams

Q21 Why strength reduction factor is 0.75 for shear & 0.9 for flexure? [MES'15] [Rajuk'14]

Ans:

Shear failure is far more dangerous than flexural failure due to its brittle nature. It occurs without prior warning. Therefore beams are designed to rather fail in bending. This is done providing larger safety factor (0.75) against shear failure than those provided for bending (0.9)

Q22 Describe different shear cracks of beams. (Draw shear & flexure cracks in RC beam CPGCBL'18)

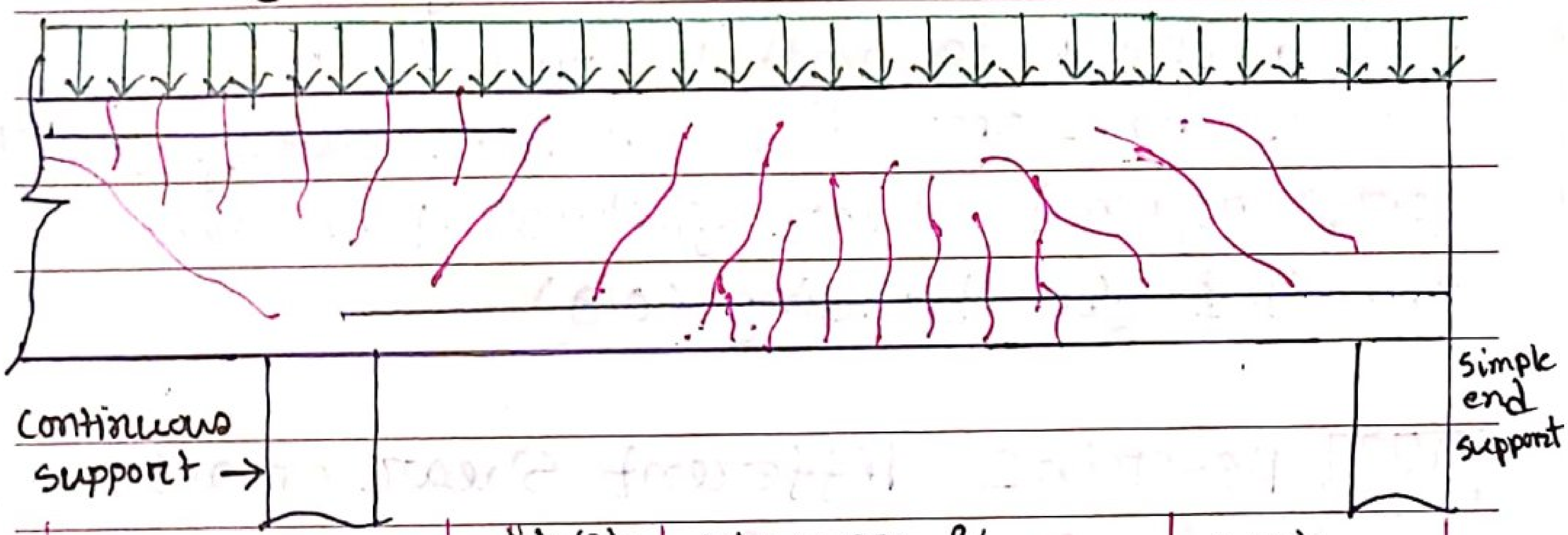
Ans: Two types of shear cracking occur in beams:

1. Web Shear Cracks:

Web shear cracking begins from an interior point in a member at the level of the centroid of the uncracked section and moves on a diagonal path to the tension face when the diagonal tensile stresses produced by shear exceed the tensile strength of concrete.

2. Flexure-Shear Cracks:

The most common type, develops from the tip of a flexural crack at the tension side of the beam and propagates towards mid depth until it reaches the compression side of the beam.



Flexure & Flexure-Shear	Web Shear	Flexure & Flexure Shear	Web Shear

—— tension bar

* এই তিনটি cracks প্রতিরোধ করার জন্য shear reinforcement use করা হয়।

Shear reinforcement = web reinforcement
= Stirrup (same)

Shear Design/Analysis Formulas:

① Shear force resisted by concrete or, concrete shear strength, $V_c = 2\sqrt{f'_c}\, b_w d$

② Shear strength resisted by steel or, steel shear strength, $V_s = \frac{A_v f_y d}{s} \le 4V_c$

(when $V_s \ge 4V_c$, the section is too small to ensure ductile failure & thus it will need to be enlarged)

③ Design shear, $\phi V_n = \phi (V_c + V_s)$

Always, design shear ≥ critical shear

$$\phi V_n \ge V_{cr}$$

(4) For $V_{cr} < \frac{\phi V_c}{2} \Rightarrow$ no shear reinforcement required

$V_{cr} \ge \frac{\phi V_c}{2} \Rightarrow$ ~~minimum~~ shear reinforcement is required

(5) Max^m spacing $= d/2$, when $V_s < 2V_c$

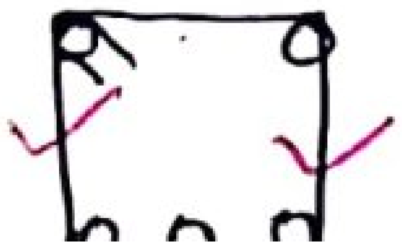
$= d/4$ when $V_s \ge 2V_c$

Here,

$\phi = 0.75$ (for shear)

s = stirrup/web reinforcement spacing

A_v = stirrup area $= 2A_s$ [for two lagged stirrup]



Two lagged stirrup

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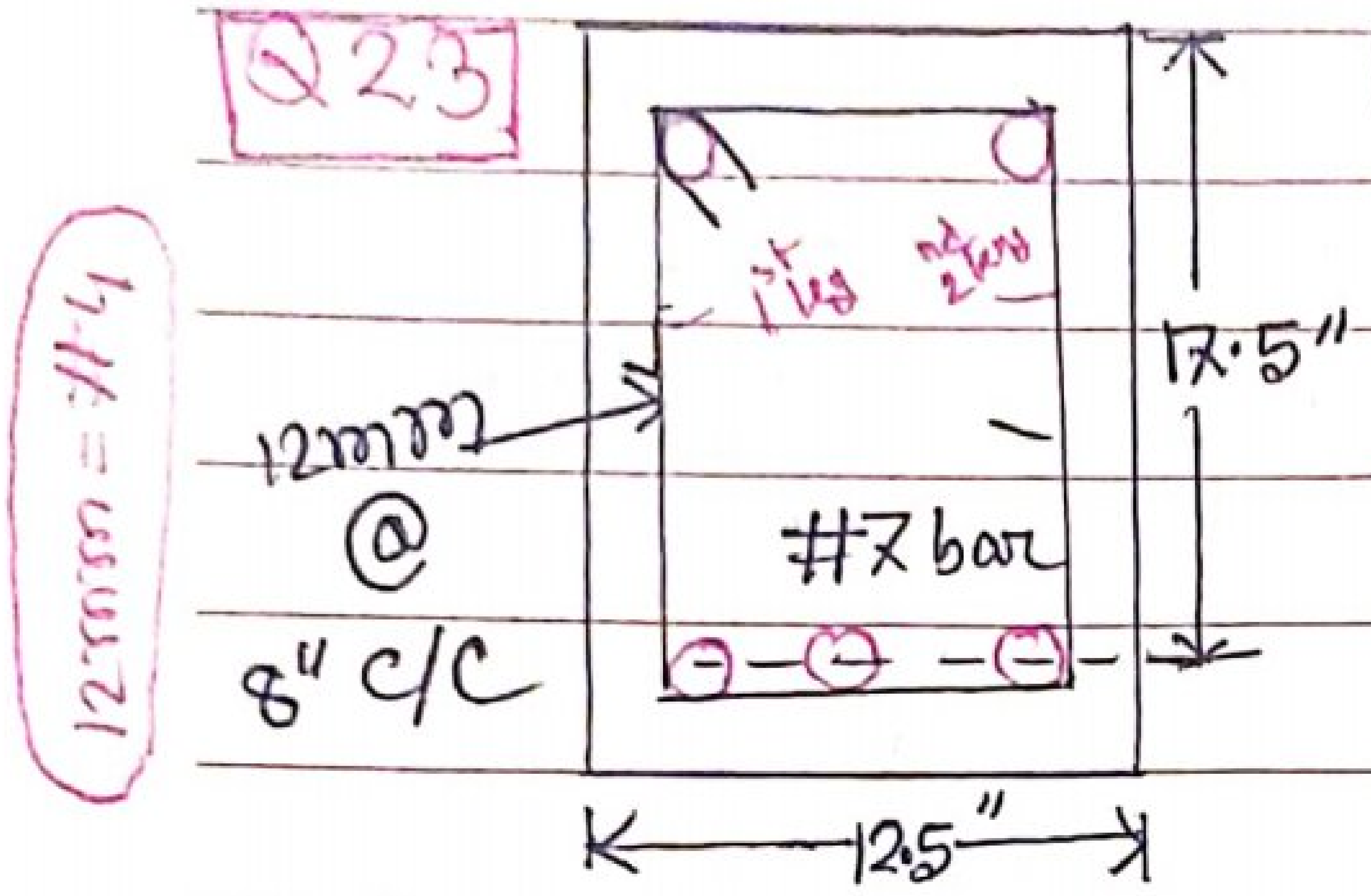
* সাধারণত stirrup হিসেবে #3 (10mm) বা #4 (12mm) এর bar use করা হয়।

Q23



Determine the ultimate shear capacity of the beam. $f'_c = 3ksi$, $f_y = 60ksi$

[APSCL'20]

[SGCL'17]

[BGDCL'21]

Soln:

Concrete shear strength, $V_c = 2\sqrt{f'_c}\,b_w d$

($\sqrt{f'_c}$ থাকলে কখনোই ksi তে বসানো যাবেনা। সবসময়ই psi তে বসাতে হবে)

$= 2 \times \sqrt{3000} \times 12.5 \times 17.5$

$= 23962.87 \text{ lbs}$

$= 23.96 \text{ kips}$

Steel shear strength $= \frac{A_v f_y d}{s}$

2 legged stirrup

$2 \times \frac{\pi}{4} \times \left(\frac{4}{8}\right)^2 = 0.39 \text{ in}^2$

$= \frac{0.39 \times 60 \times 17.5}{8}$

$= 51.54 \text{ kips}$

$\therefore$ Ultimate shear strength, $V_u = \phi (V_c + V_s)$

$= 0.75 \times (23.96 + 51.54)$

$= 56.625 \text{ kips}$ Ans

$V_s = A_{st} \cdot f_y \cdot \frac{d}{s}$

* Determine the minimum cross-section required for a rectangular beam from a shear standpoint so that no web reinforcement is required by the ACI Code if $V_u = 38k$ & $f'_c = 4000psi$.

Soln:

For no web reinforcement

$V_u = \frac{\phi}{2} V_c$ [38×10^3 lbs]

$\Rightarrow 38 \times 10^3 = \frac{0.75}{2} \times (2\sqrt{f'_c}\, b_w d)$

$\Rightarrow 38 \times 10^3 = 0.375 \times 2 \times \sqrt{4000} \times b_w d$

$\Rightarrow b_w d = 801.1 \text{ in}^2$

$\therefore$ Here, $b_w = 24$ (take)

$d = 33.5$

$\therefore h = 35$

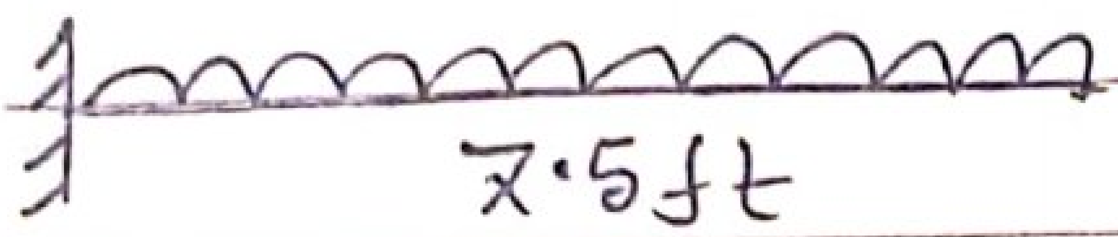
$\therefore$ use 24in × 35inch beam

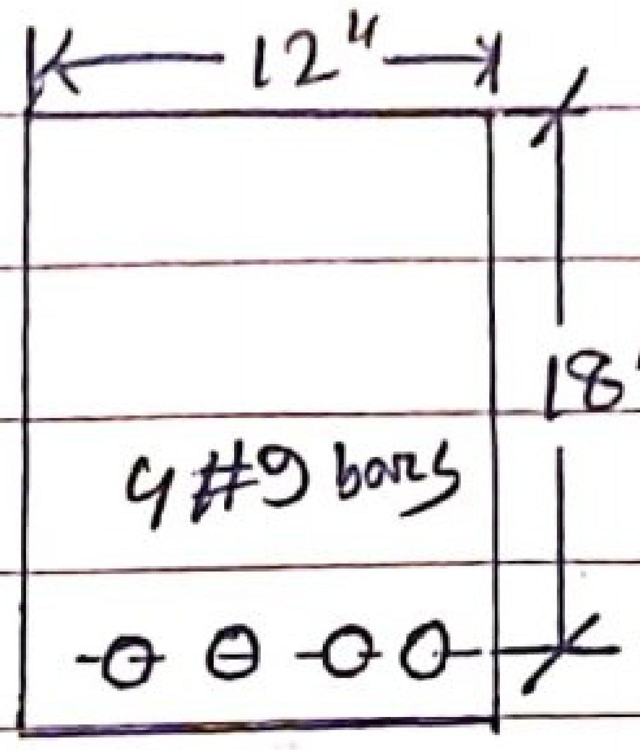
$\phi V_c = 2V_u$

or, $A = \frac{V_u}{0.75\sqrt{f'_c}} = \frac{38 \times 10^3}{0.75 \times \sqrt{4000}} = 801.1 \text{ in}^2$

Ans

Q24 Design the shear/transverse reinforcement for the following beam.





[MSc'16]
[MSc'14]

DL = 4k/ft
LL = 2k/ft
$f'_c = 4000$ psi, $f_y = 60{,}000$ psi

Soln: Factored load $= 1.2\times4 + 1.6\times2 = 8$ k/ft

$V_u = 7.5\times8$ kips [cantilever so max^m shear]
$= 60$ kips

Cracking shear, $V_{cr} = 60 - 8\times1.5$ [$18" = 1.5$ ft]
$= 48$ kips

$\therefore$ Required design shear strength, $V_d = \frac{48}{0.75}$
$= 64$ kips

Concrete shear strength, $V_c = \frac{2\sqrt{4000}\times12\times18}{1000}$
$= 27.32$ kips

Since, $V_{cr} > \frac{\phi V_c}{2} = \frac{0.75\times27.32}{2} = 10.24$
therefore, shear reinforcement is required!

$\therefore V_s = V_d - V_c = 64 - 27.32 = 36.68$ kips
$< 2V_c = 54.64$ kips

$\therefore$ max^m spacing for stirrup $= d/2 = 18/2 = 9$ inch

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Assume #3 bar U-shape (2 legged) stirrup.

$\therefore$ Spacing, $s = \frac{A_v f_y d}{V_s} = \frac{2 \times 0.11 \times 60 \times 18}{36.68}$

$= 6.47$ inch

≈ 6 inch < 9 inch (ok)

Minimum steel area $= \frac{50 b_w s}{f_y} =$

$= \frac{50 \times 12 \times 6}{60,000} = 0.060 \text{ in}^2$

$< 0.22 \text{ in}^2$

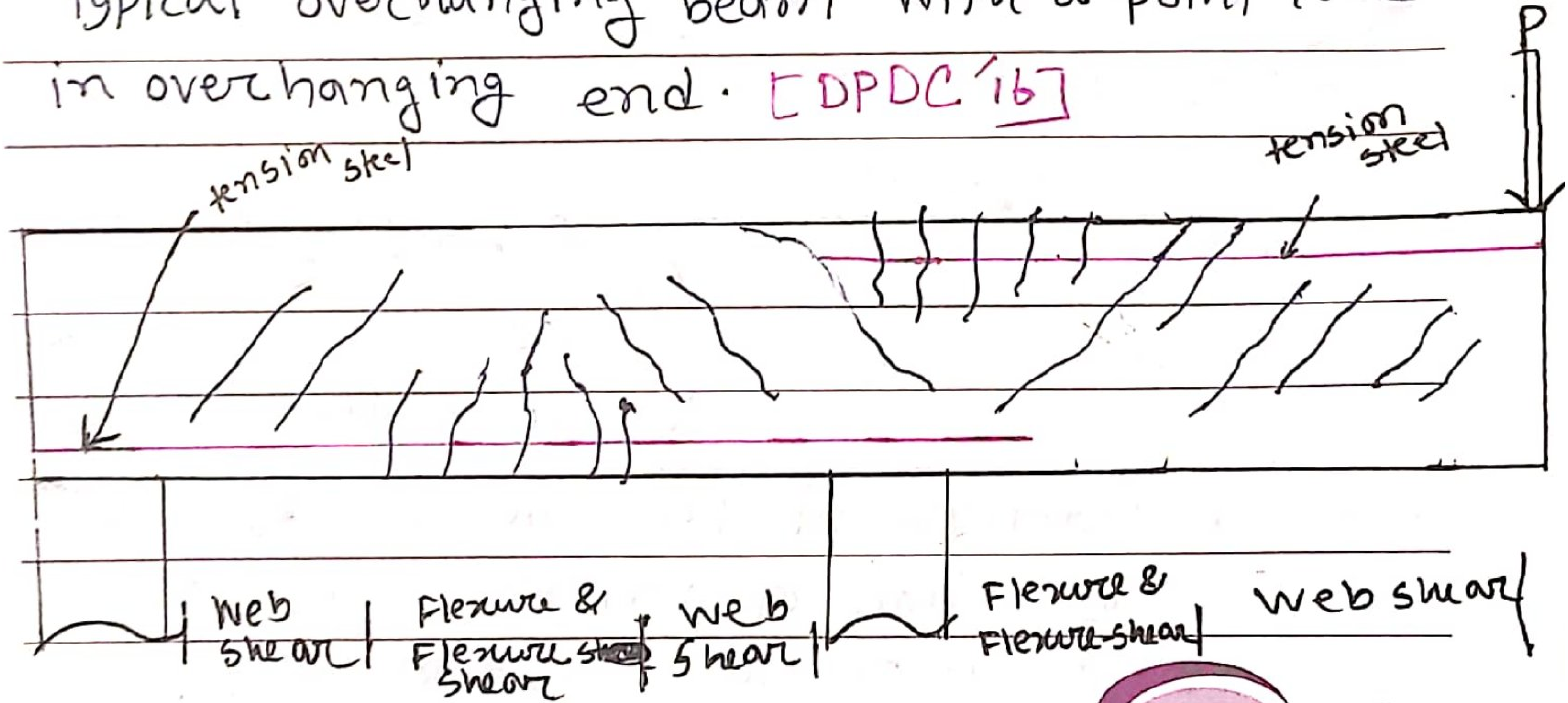
ok

Minimum steel area check

$\therefore$ use #3 stirrup at 6 inch c/c spacing.

Ans

*Q26 Draw the shear & bending crack on a typical overhanging beam with a point load P in overhanging end. [DPDC'16]



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Analysis & Design of Footings

Square Isolated Footings:

Square isolated footings are used to support single columns. This is ~~one~~ the most economical types of footings and is used when columns are spaced relatively long distances.

জায়গার বাধ্যবাধকতার কারণে অন্য কোন spacial case ছাড়া isolated footing সাধারণত rectangle হওয়ার square ই হয়।

Depth calculation of square Isolated footings:

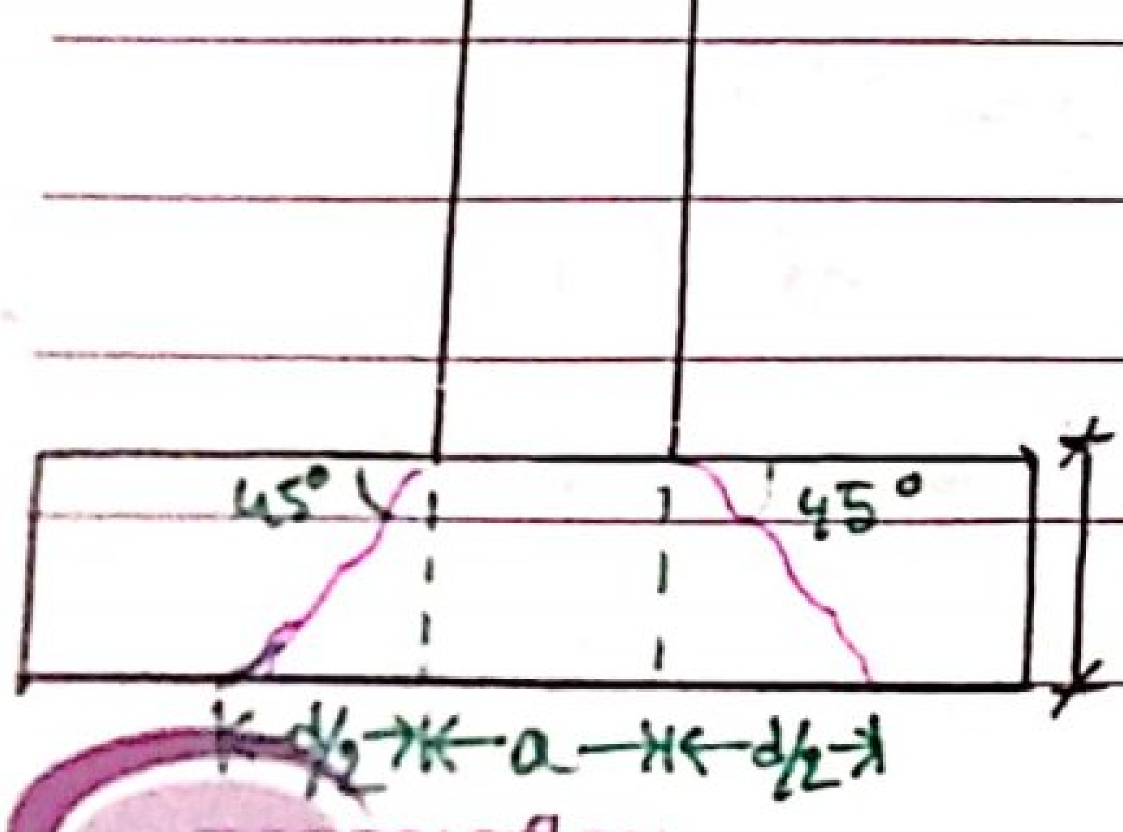
Two shear conditions must be considered in column footings to determine the depth of footing.

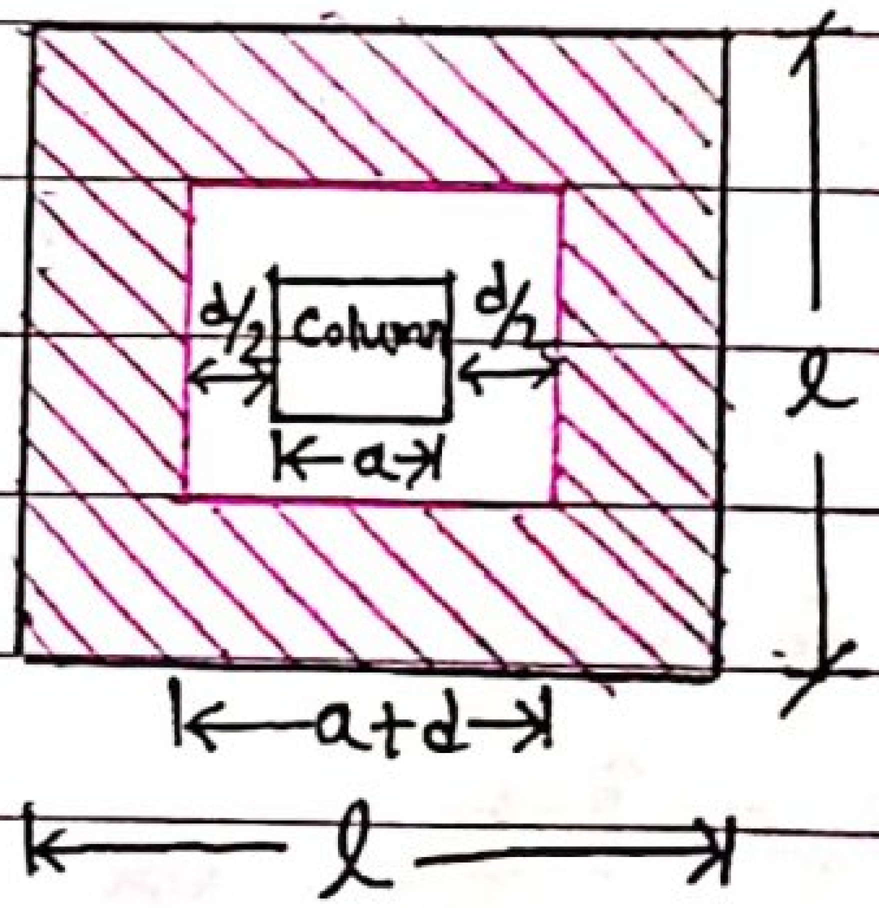
(i) one way or beam shear

(ii) two-way or punching shear

But for square footings, two way or punching share always governs!

Punching Shear:





critical punching shear, $V_u = q_u[l^2 - (a+d)^2]$

q_u = net ultimate soil pressure = $\frac{\text{factored load}}{\text{footing area}}$

Shear capacity of the concrete must be greater or equal the critical punching shear. $\therefore$ $V_u \leq \phi V_c$

Now, $\phi V_c = \phi 4 \sqrt{f'_c}\, b_o d$

$\therefore$ $\boxed{V_u = 4\phi\sqrt{f'_c}\, b_o d}$ → from this eqn we can calculate the depth of footing.

where,

b_o = perimeter of punching area

$= 4(a+d)$

* column $(a \times b)$ rectangular ও; footing $L \times B$ rectangular হলে,

$V_u = q_u[(L \times B) - (a+d)(b+d)]$

$b_o = 2(a+d+b+d)$

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$\boxed{39}$ A footing size is 20"×20". Factored load on footing 330kip. Column size is 10.5"×10.5". Calculate the depth of footing considering punching shear check only. [BCIC'17] [BPDB'11]

Solⁿ: Take $f'_c = 3000psi$

Soil ultimate bearing pressure,

$$q_u = \frac{330\times10^3}{20\times20} = 825psi$$

critical punching shear, $V_u = q_u[l^2 - (a+d)^2]$

$$= 825[20^2 - (10.5+d)^2]$$

Again,

$$V_u = 4\phi\sqrt{f'_c}\,b_o d$$

$$\Rightarrow 825[400 - (10.5+d)^2] = 4\times0.75\times\sqrt{3000} \times 4(10.5+d)d$$

$$\Rightarrow 330000 - 825(110.25 + 21d + d^2) = 657.27(10.5d + d^2)$$

$$\Rightarrow 330000 - 90956.25 - 17325d - 825d^2 = 6901.335d + 657.27d^2$$

$$-1482.27d^2$$

$$\Rightarrow \cancel{1482.27d^2} - 24226.335d + 239043.75 = 0$$

$\therefore d = 7$ inch

$\therefore$ Footing depth = d + minimum clear cover + $\frac{1}{2}$ bar dia

$$= 7 + 3 + 0.5$$

$$= 10.5$$

$$\approx 11 \text{ inch}$$

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Q40 Determine the two-way shear capacity of footing if the footing size is 12ft × 12ft & effective depth 30 inch. Column size 20"×20". DL = 230 kips, LL = 270 kips. f'_c = 3.5ksi, f_y = 60ksi

[BUET]

Sol^n: Factored load = $1.2 \times 230 + 1.6 \times 270 = 708^k$

∴ ~~Ultimate shear~~ bearing ~~capacity~~

Ultimate bearing pressure of soil $= \frac{708 \times 10^3}{144 \times 144}$

$= 34.14$ psi

No need

two-way shear capacity, $v_u = 4\phi\sqrt{f'_c}\, b_o d$

$= 4 \times 0.75 \sqrt{3500} \times 4(30+20) \times 30$

$= 985900$ lb

$= 985.9^k$

Q.41 Calculate ultimate punching shear capacity of a 10ft × 10ft footing. Column size is 12"×12". footing thickness is 20 inch. f'_c = 3ksi, f_y = 60ksi [BWDB'14] [DSCC'16] [BWDB'19]

Sol^n:

effective depth, $d = 20 - 3 - 0.5 = 16.5$ inch

Punching area perimeter, $b_o = 4(12 + 16.5)$

$= 114$ inch

Ultimate punching shear capacity $= 4\phi\sqrt{f'_c}\, b_o d$

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

$b_o = 4$(column size + footing thickness)

$= 309079.84$ lb

$= 309.08$ kips

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* A RCC column 16-inch square, $f'_c = 3$ksi, reinforced with eight #7 bars of $f_y = 60$ksi, supported a total design load of 550kips on 9ft square footing with a depth of 24 inches. Calculate the punching shear stress that developed on the footing. [ELBL22]

$F = \frac{q_u[B^2-(c+d)^2]}{4(c+d)\times d}$

Punching shear stress $= \frac{q_u[B^2-(c+d)^2]}{4(c+d)\times d}$

Solⁿ: effective depth, $d = 24 -$ minimum clear cover $- \frac{1}{2}$ bar dia

$= (24 - 3 - \frac{1}{2})$ in $= 20.5$ in

Ultimate Soil pressure, $q_u = \frac{550}{9\times 9} = 6.79$ ksf

Ultimate punching shear force, $V_u = q_u[B^2 - (c+d)^2]$

$= 6.79[9^2 - (\frac{16+20.5}{12})^2]$

$= 487.18$ kips

∴ Punching Shear stress, $V_c =$

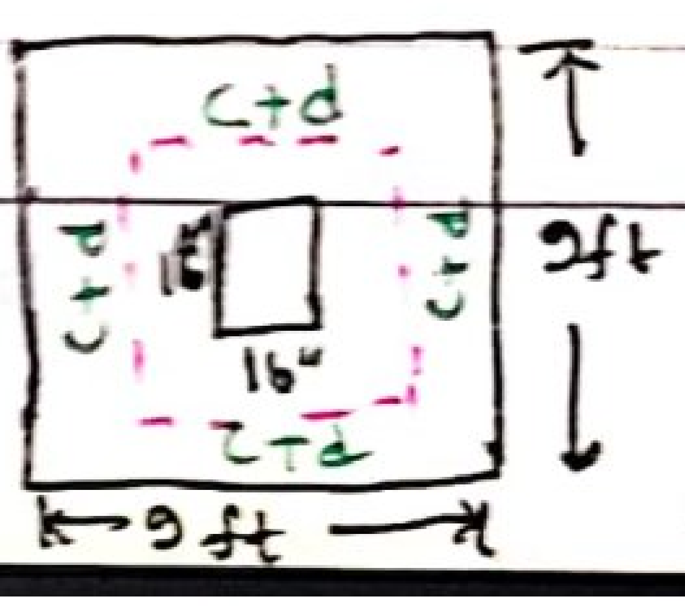
$= \frac{\text{Force}}{\text{Punching Area}} = \frac{V_u}{4(c+d)d} = \frac{487.18}{4\times (16+20.5)\times 20.5}$

$= \frac{V_u}{B_o . d}$

$= 0.16275$ ksi

$B_o = 4(c+d)$

Perimeter



Column Design & Analysis Formulae:

(1) Maximum nominal capacity of column under concentric Loads,

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

γ = reduction factor due to accidenal eccentricity

= 0.80 (for tied column)

= 0.85 (for spiral column)

(2) Ultimate capacity, $P_u = \phi P_n$

$\phi = 0.65$ (tied column)

$\phi = 0.75$ (spiral column)

Design capacity must be equal or greater than ultimate capacity.

* Minimum Column size as per BNBC & ACI Code = 12" X 12"

(3)* According to ACI code, steel percentage for column must be between 1% to 8%

তবে practically 1.5% থেকে 3% এর মধ্যে steel area রাখার চেষ্টা করা হয়। 4% এর বেশি হলেই সাধারণত steel গুলো কলামের x-section এ fit করতে হিমসিম খেতে হয়।

* Column design করার সময় initially steel percentage 2% ধরতে হয় অর্থাৎ $A_{st} = 0.02A_g$

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(5) Tie spacing (for square / rectangular columns)

$$\text{max}^m \text{ spacing}, \; s = \min \begin{cases} 16 d_b \\ 48 d_{stirrup/tie} \\ \text{least dimension of column} \end{cases}$$

* d_b = dia of longitudinal (main) bar

(6) Minimum number of longitudinal (main) bars =

(i) min^m 6 bars for spiral column

(ii) min^m 4 bars for rectangular or circular ties O O O [circular ties]

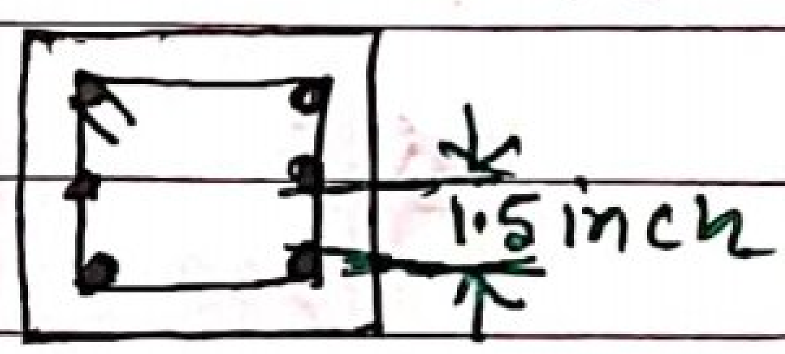
(7) minimum bar size for spirals ⇒ #3

minimum bar size for ties ⇒ #3
for main bar ≤ #10

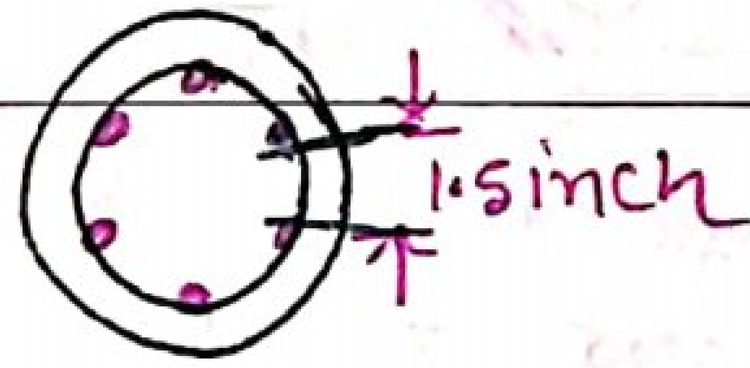
minimum bar size for ties ⇒ #4
for main bar > #10

(8) Minimum clear distance between main reinforcing bars:

$$S_b = \max \begin{cases} 1.5 d_b \\ 1.5 \text{ inch} \end{cases}$$



*clear distance — [illegible]
*c/c spacing [illegible]



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Q.32 Design the ties (size & spacing) of a 24"x 16" column which is reinforced with 12 #9 bars. [BGFCL'17] [DPDC'14]

Solⁿ:

Given, Column size = 24"x 16"

reinforcement = 12 #9 bar

lets use #3 bar for tie

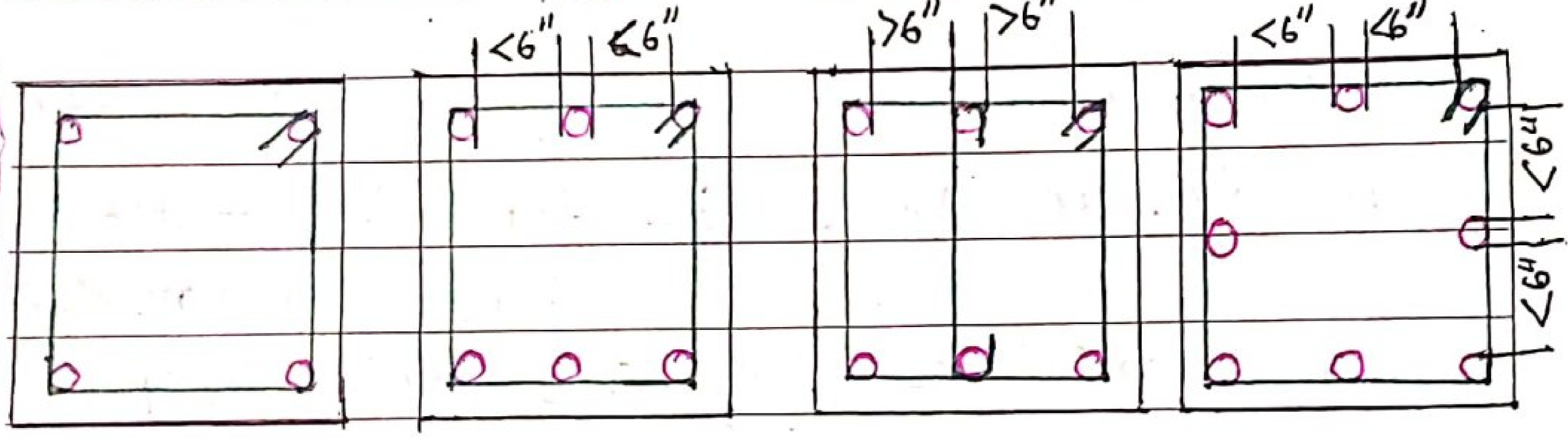
$$\therefore \text{Tie spacing} = \min \begin{cases} 16 \times d_b = 16 \times \frac{9}{8} = 18 \text{ inch} \\ 48 \times d_s = 48 \times \frac{3}{8} = 18 \text{ inch} \\ \text{lest dimension} = 16 \text{ inch} \end{cases}$$

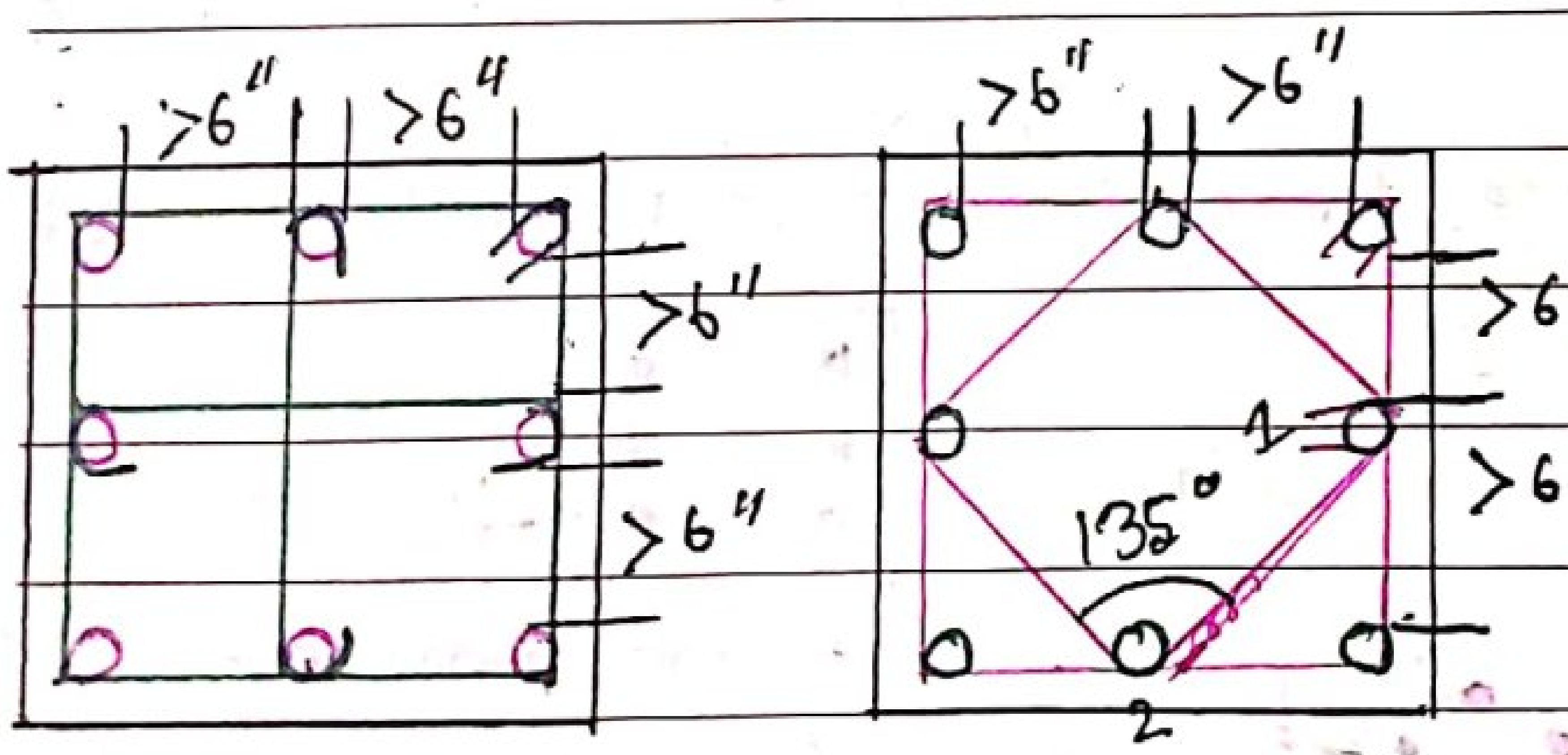
∴ use #3 tie bar at 16 inch c/c spacing.

Ans

Tie bar Arrangements:

clear distance not c/c distance





Clear distance 6 inch এর বেশি হইলেই extra tie দিতে হবে।

1,2 এর angle যদি 135° এর চেয়ে কম হলে এই extra tie লাগবেনা

Q32 Find the nominal axial compression & design capacity of a rectangular column size is 16"X24" which is reinforced with 12 nos of 20mm bars. Given, $f'_c = 4ksi$ & $f_y = 60ksi$

[MSc'16] [NHA'19] [CPGCBL'19] [BEPZA'19] [BPDB'16] [PBC B'15'] [ERL'17] [Titas'18]

Soln: Here, $A_g = 384 in^2$, $A_{st} = 12 \times \frac{\pi}{4} \times (20)^2 = 3769.91 mm^2$

$= \frac{3769.91}{645}$

$= 5.84 in^2$

$25.4^2 = 645$

$\frac{mm^2}{645} = inch^2$

Nominal compression capacity,

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

$= 0.8 \times [0.85 \times 4 \times (384 - 5.84) + 5.84 \times 60]$

$= 1309$ kip

r = 0.8 for tied columns

Design capacity $= \phi P_n = 0.65 \times 1309$

or ultimate capacity $= 850.85$ kips Ans

Q33 Determine the maximum allowable axial compression force for a 15"X20" tied column having 2.5% steel reinforcement. Given, $f'_c = 4ksi$ & $f_y = 60ksi$ [NRGCL'14]

Soln: Here, $A_g = 15 \times 20 = 300 in^2$

$A_s = 0.025 \times 300 = 7.5 in^2$

$P_n = 0.8 [0.85f'_c(A_g - A_{st}) + A_{st}f_y]$

$= 0.8[0.85 \times 4 \times (300 - 7.5) + 7.5 \times 60]$

$= 1155.6$ kip

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Q34 A column with x-section of 12"×12" & reinforced with 4 #7 vertical bars. $f_y = 50ksi$, $f'_c = 3ksi$. compute allowable axial load. The column will support without modification for length or eccentricity [DMLC'19]

Ans:

$A_g = 12 \times 12 = 144\ in^2$ $A_s = 4 \times \frac{\pi}{4} \times (\frac{7}{8})^2 = 2.41\ in^2$

Allowable axial load,

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

no modification factor $n = 0.8$ for tied column

$= 0.85 \times 3 \times (144 - 2.41) + 2.41 \times 50$

$= 481.55$ kips

Q35 Determine the axial load of a column having cross-section of 12"×12", 4 #5 main bar. A hole is at the middle of the column which area is 2"×2". Given $f'_c = 4ksi$, $f_y = 60ksi$ [DPDC'19] [MSc'18]

Soln: $A_g = 144 - 4 = 140\ in^2$

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

$P_n = 0.8[0.85f'_c(A_g - A_{st}) + A_{st}f_y]$

$= 0.8[0.85 \times 4 \times (140 - 1.23) + 1.23 \times 60]$

$= 436.5$ kip Ans

Also design the tie reinforcement. [MSc'18]

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lets use #3 bar as tie bar.

Tie spacing = minimum of
$$\begin{cases} 16 \times d_b = 16 \times \frac{5}{8} = 10\,in \\ 48 \times d_s = 48 \times \frac{3}{8} = 18\,in \\ \text{least dimension} = 12\,in \end{cases}$$

$\therefore$ Tie spacing = 10 inch c/c **Ans**

Q36 The value of f'_c is 4000psi, f_y is 60000psi and steel area is 2% and the column section is 12"×12". Find the design ultimate axial stress of the column with zero eccentricity. [GTCL'16]

Soln: $A_g = 12 \times 12 = 144\,in^2$, $A_{st} = 0.02 \times A_g = 2.88\,in^2$

Nominal axial load, $P_n = 0.85 f'_c (A_g - A_{st}) + A_{st} f_y$

zero eccentricity so, $r = 1$ (no modification)

$$= 0.85 \times 4 \times (144 - 2.88) + 2.88 \times 60$$

$$= 652.61\ kip$$

$\therefore$ Design axial load = 0.65×652.61

$$= 424.2\ kip$$

Design stress = $\frac{424.2}{144} = 2.95\,ksi$ **Ans**

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Q37 Design a square tied column to support an axial dead load of 130k & an axial live load of 180k. Steel ratio must be within 1% to 2%. Assume $f'_c = 4000$ psi & $f_y = 60,000$ psi. [Janata Bank 2017] [NPCBL'17] [BCPCL'15]

Soln: $P_u = 1.2 \times 130 + 1.6 \times 180 = 444k$.

Assuming 2% steel ratio, selecting column dimensions,

$\phi P_n = \phi\, 0.80\, [0.85 f'_c (A_g - A_{st}) + A_{st} f_y]$ — (i)

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

$\Rightarrow A_g = 188.40\ in^2$

use 14 in × 14 in column ($A_g = 196\ in^2$)

Putting $A_g = 196\ in^2$ in eq^n (i) we get,

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

$\Rightarrow A_{st} = 3.31\ in^2$

use 6#7 bars ($3.61\ in^2$)

Design of Ties

Assuming #3 bars.

Spacing (i) $48\ in \times \frac{3}{8}\ in = 18\ in$

(ii) $16\ in \times \frac{7}{8}\ in = 14\ in$

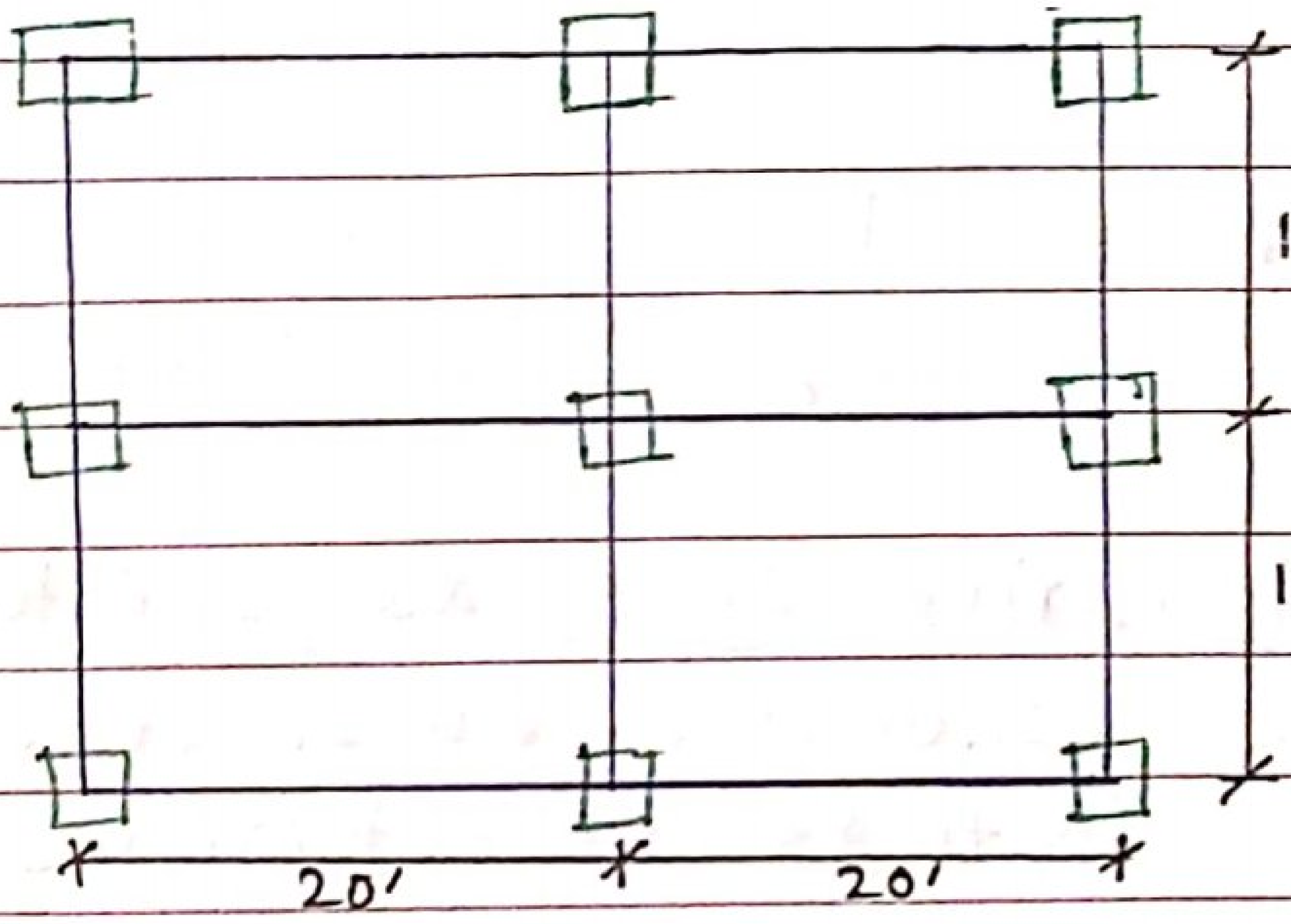
(c) Least dim = 14 in

∴ use #3 ties @ 14 c/c spacing.

37 Preliminary

* ~~Primary~~ design a column. A load of 250 psf per floor. The column (middle one) is at the ground floor of a five storied building. Given, $f'_c = 3000$ psi, $f_y = 60000$ psi

[SGFL'21]



Soln:

Load at ground floor, ~~Pu = Σ~~

$$P_u = 250 \times (7.5 + 7.5)(10 + 10) \times 5$$

$$= 375000\ lb = 375\ k$$

Assume 2% steel, selecting column dimensions

$$\phi P_n = \phi\, 0.80 [0.85 f'_c (A_g - A_{st}) + A_{st} f_y] \quad \text{--- (i)}$$

$$\Rightarrow 375 = 0.65 \times 0.80 [0.85 \times 3 \times (A_g - 0.02 A_g) + 0.02 A_g \times 60]$$

$$\Rightarrow A_g = 194.96\ in^2$$

use 14in × 14in column ($A_g = 196\ in^2$)

Again, putting $A_g = 196$ in eqn (i),

$$375 = 0.65 \times 0.80 [0.85 \times 3 \times (196 - A_{st}) + A_{st} \times 60]$$

$$\Rightarrow A_{st} = 3.85\ in^2$$

$\therefore$ use 4 #9 bars ($4\ in^2$)

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* Column size 15"X20". It has 6 nos 20mm longitudinal bar. Determine the axial load that can produce a 1400psi stress on column surface. Given, $f'_c = 5ksi$, $f_y = 60ksi$
(concrete)

[RPGCL'22]

Soln: modular ratio, $n = \frac{E_s}{E_c}$

for, $f'_c = 3ksi$ $n=9$
$f'_c = 4ksi$ $n=8$
$f'_c = 5ksi$ $n=7$

$$= \frac{29 \times 10^6}{57000 \times \sqrt{5000}}$$

$$= 7.19$$

$$\approx 7$$

gross Area, $A_g = 15 \times 20 = 300 in^2$

Steel area, $A_{st} = 6 \times \frac{\pi}{4} \times \left(\frac{20}{25.4}\right)^2 = 2.92$

Load on column, $P = f_c A_c + f_s A_{st}$

$$= \cancel{f_c A_c + n f_c A_{st}}$$

$$= f_c (A_g - A_{st}) + n f_c A_{st}$$

$$= f_c [A_g + (n-1) A_{st}]$$

$$= 1400 [300 + (7-1) * 2.92]$$

$$= 444528 \text{ lb}$$

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* A RCC column of size 20"*25" has 12, 32mm longitudinal bar is used. Determine the axial load of column that will cause a 1500psi stress to concrete. Take, $f'_c = 4.5$ksi, $f_y = 60$ksi and $n = 8$ [DWASA'21]

Sol^{n}:

Here, $A_{st} = \frac{\pi}{4} * \left(\frac{32}{25.4}\right)^2 * 12$

$= 14.96 \text{ in}^2$

$A_g = 20 \times 25$

$= 500 \text{ in}^2$

Now, Axial load, $P = f_c A_c + f_s A_{st}$ $[f_s = n f_c]$

$= f_c (A_g - A_{st}) + n f_c A_{st}$

$= f_c [A_g + (n-1) A_{st}]$

যেহেতু concrete stress, f_c 1500 psi বলে দিছে
সেহেতু তাই extra
লোড factor লাগবেনা

$\therefore P = 1500 \times [500 + (8-1) * 14.96]$

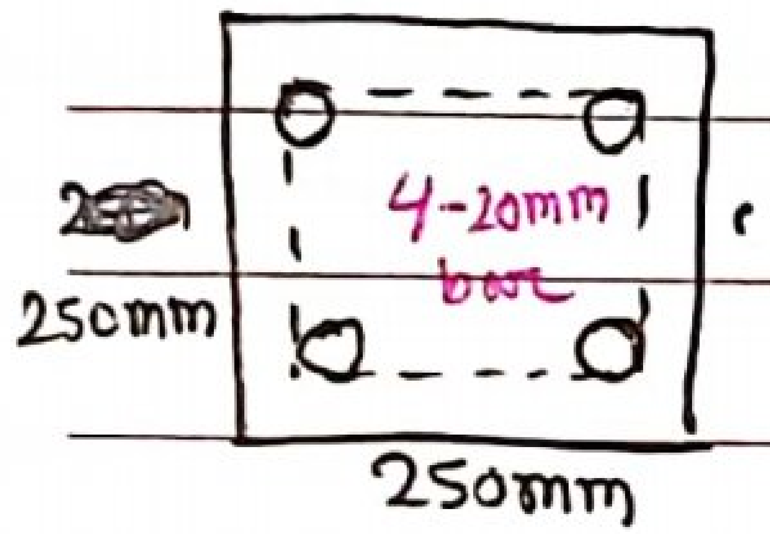
$= 907080$ lb

$= 907.08$ kips

Column Design (WSD)

* Determine the load capacity of the column. Assume, $f_c = 3.5\,N/mm^2$ & $f_s = 130\,N/mm^2$

[DSCC'21]



* যেহেতু f_y এর বদলে f_s দেয়া আছে সেহেতু ধরে নিতে হবে যে WSD

Sol^n:

$f_c = 0.45 f'_c$

$\therefore f'_c = \dfrac{3.5}{0.45} = 7.78\,N/mm^2$

$A_g = 250^2 = 62500\,mm^2$

$A_{st} = 4 \times \dfrac{\pi}{4} \times 20^2 = 1256.64\,mm^2$

~~P = 0.85 [0.25~~

$P = 0.85\,[0.25 f'_c A_g + f_s A_s]$ [WSD formula. reduction factor, n, ø এইক্ষেত্রে নাই]

$= 0.85\,[0.25 \times 7.78 \times 62500 + 130 \times 1256.64]$

$= 242186.84\,N$

$= 242.186\,kN$ Ans.