BCS Written, Note

RCC.

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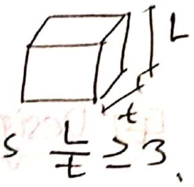
▭ Column এর Axial load capacity:

- Column: It is avertical member that can carry loads chiefly in compression and additionally can carry moment about one or both axes of cross-section.



→ compression force dominates its behaviour.

→ It's ratio of height to least lateral dimension is $\frac{L}{t} \geq 3$.

▭ Relation between axial load and gross area of Column.

For tied column: $A_g \geq \dfrac{P_u}{0.45(f'_c + f_y \rho_g)}$

For spiral column: $A_g \geq \dfrac{P_u}{0.55(f'_c + f_y \rho_g)}$

▭ Function of tie/spiral/transverse R.F:

→ To hold the longitudinal bars in position while the concrete is being placed.

→ To prevent highly stressed longitudinal bars from bucking outward by bursting the thin concrete cover.

→ To increase the ductileness of column.

▭ Guide line for tie/spiral R.F:

i) Use #3 bar as tie upto longitudinal of size #10

ii) Use #4 bar as tie more than longitudinal bar size #10.

▭ Spacing of tie.

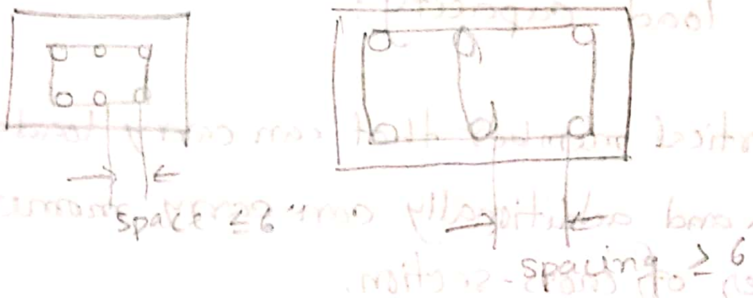
$$\text{Spacing} = \begin{cases} \leq 16D \\ \leq 48d \\ \leq t_{min} \end{cases}$$

D = dia of main bar

d = dia of tie bar

t = least dimension.

⇒ No bar shall be further than 6" clear on either side from a such laterally supported bar.



⊞ Design of Axially Loaded Column:

Basic equation, $P = A_c f_c' + A_s f_y$.

⊞ W.S.D method: for concentrically loaded tied Column

$$P = A_g \{0.85 f_c' (1 - \rho_g) + \rho_g f_y\}$$

→ For spiral Column

$$P = 0.85 A_g (0.25 f_c' + f_s \rho_g)$$

for tied column: $P = 0.85 A_g (0.25 f_c' + f_s \rho_g)$

For spiral Column: $P = A_g (0.25 f_c' + f_s \rho_g)$

⊞ U.S.D Method:

$$P = \alpha \varphi A_g \{0.85 f_c' (1 - \rho_g) + f_y \rho_g\}$$

[এ সূত্র use করব
Ag বের করতে ρ_g Assume
হবে]

for spiral column $\begin{cases} \alpha = 0.85 \\ \varphi = 0.75 \end{cases}$

for tied column $\begin{cases} \alpha = 0.8 \\ \varphi = 0.65 \end{cases}$

Problem: Design a tied column for a load of 500 kips. Use W.S.D method.

Solution: Let us consider $\rho_g = 0.02$ $f'_c = 3\text{ksi}$ $f_y = 60\text{ ksi}$

Now. $P = \cancel{\alpha \phi A_g \{0.85 f'_c}$ $0.85 A_g (0.25 f'_c + f_s \rho_g)$.

$500 = 0.85 \times A_g (0.25 \times 3 + \overset{0.4}{60} \times 0.02)$ | $f_s = 0.4 f_y$

$\Rightarrow A_g = 478.24 \text{ in}^2$

∴ Provide a square column of $22'' \times 22''$

∴ $A_s = A_g \rho_g = 0.02 \times 22 \times 22 = 9.68 \text{ in}^2$

Provide 10 # 9 bar.

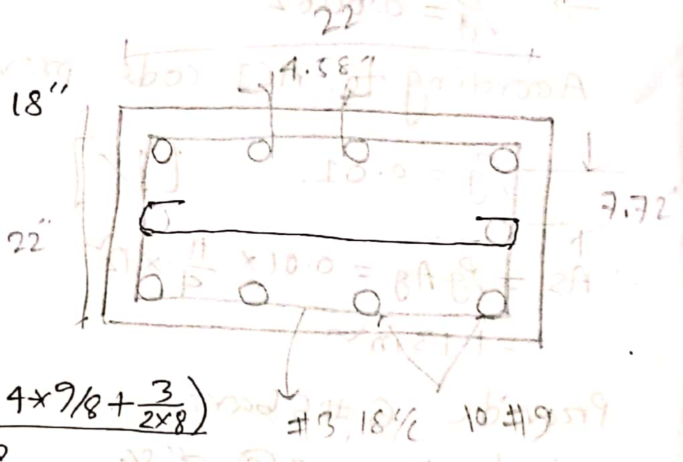
Tie spacing: $16D = 16 \times \frac{9}{8} = 18''$

$48d = 48 \times \frac{3}{8} = 18''$

$t_{min} = 22''$

Taking lower/minimum spacing 18''

Use #3 @ 18'' c/c.



Clear spacing check:

$$\text{spacing at 3 gaps} = \frac{22 - (2 \times 1.5 + 4 \times 9/8 + \frac{3}{2 \times 8})}{3}$$

$= 4.58'' < 6''$

$$\text{spacing at 2 gaps} = \frac{22 - (2 \times 1.5 + 3 \times 9/8 + 3/2 \times 8)}{2}$$

$= 7.72'' > 6''$

So extra tie is needed.

Column.

⊘ A reinforcement column of effective length 4.8 m. overall dimension 250 × 250 mm². Design Axial load on column is 60 tonnes. concrete mix is used M20 and $F_y = 415$ MPa. Design steel requirements also ties. Use WSD.

Sol^n:

$P = 60$ ton $= 120$ kip.

$f'_c = 20$ MPa $= 20 \times 145 = 2900$ psi

$f_s = 60000$ psi.

$P = \phi A_g (0.25 f'_c + \rho f_s)$

$\Rightarrow 120 = 0.85 \times 100 \times (0.25\ 2900 + \frac{A_{st}}{100} \times 60000)$

$\Rightarrow A_{st} = 0.75\ in^2$

4 #4 bar

tie bar. #3

$$\begin{cases} 48 \times 3/8 = 18'' \\ 16\ D = 16 \times 4/8 = 8'' \\ 10'' \end{cases}$$

$A_c = \frac{250 \times 250}{25 \times 25}$

$= 100$ ~~mm~~ in^2

Adopt lower spacing $= 18''$ c/c.

Problem: Design a tied column for a load of 500 kips. Use W.S.D method.

Solution: Let us consider $\rho_g = 0.02$ $f'_c = 3\,ksi$ $f_y = 60\,ksi$

Now. $P =$ ~~$\alpha\phi\, A_g\{0.85 f'_c$~~ $0.85\, A_g\,(0.25 f'_c + f_s \rho_g)$

$500 = 0.85 \times A_g\,(0.25 \times 3 + \overset{0.4}{60} \times 0.02)$

$f_s = 0.4 f_y$

$\Rightarrow \quad A_g = 478.24\ in^2$

$\therefore$ Provide a square column of $22'' \times 22''$

$\therefore A_s = A_g\,\rho_g = 0.02 \times 22 \times 22 = 9.68\,in^2$

Provide 10 # 9 bar.

Tie spacing: $16D = 16 \times \frac{9}{8} = 18''$

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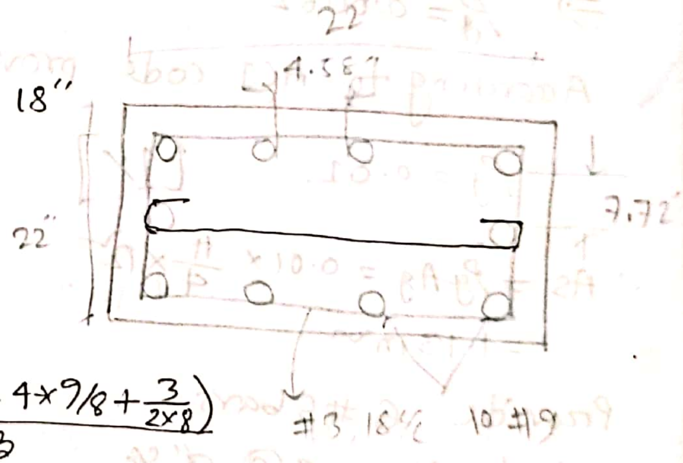

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So extra tie is needed.

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Example-2: Given D.L = 90 kips L.L = 70 kips $f'_c = 4$ ksi

$f_y = 60$ ksi.

Design a spiral column in USD method.

Solution:

Load, $P = 1.2 \times D.L + 1.6 \times L.L$

$= 1.2 \times 90 + 1.6 \times 70$

$= 220$ kips.

Assume the diameter of column $= 12''$

$\therefore$ $P = \alpha\phi A_g [0.85 f'_c (1-\rho_g) + f_y \rho_g]$

$\Rightarrow$ $220 = 0.75 * .85 * \frac{\pi}{4} \times 12^2 [0.85 \times 4 (1-\rho_g) + 60 \times \rho_g)$

$\Rightarrow$ $\rho_g = 0.0062$

According to ACI code provide minimum

$\rho_g = 0.01$. [1%]

$\therefore$ $A_s = \rho_g A_g = 0.01 \times \frac{\pi}{4} \times 12^2$

$= 1.13$ in^2

Provide 6 #5 bars

spiral tie #3 @ 3'' c/c.

Example-3: Determine the allowable design axial load on a 12-in square, short tied column reinforced with four no. 9 bars. Ties are 3 spaced at 12 in. Use $f'_c = 4$ ksi and $f_y = 60$ ksi (USD).

Solution: We know.

$$P_u = \alpha \varphi A_g \{0.85 f'_c (1 - \rho_g) + f_y \rho_g\}$$

$$= 0.8 \times 0.65 \times 12 \times 12 \{0.85 \times 4 ((1 - 0.02778) + 60 \times 0.02778\}$$

$$= 581.76 \text{ K.}$$

for tied column

$\alpha = 0.80$

$\varphi = 0.65$

$f_c = 4$ ksi

$f_y = 60$ ksi

$\rho_g = \frac{A_s}{12 \times 12} = \frac{4}{12 \times 12}$

$= 0.02778$

[illegible] Axial load এর জন্য equation $\boxed{10\text{-}8}$

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

$$= 0.65 \times 0.8 [0.85 \times 4 \times 12 \times 12 + 4 (60 - 0.85 \times 4)]$$

$$= 372.32 \text{ kips.}$$

Example 4: Design a square tied column to support an axial dead

Example - 5: Design a circular spiral column to support an axial load DL = 475 k L.L = 250 k, $f'_c = 4$ ksi, $f_y = 60$ ksi and steel ratio of about 3%. Also necessary spirals.

Solution:

$$P_u = 1.2 \times 475 + 1.6 \times 250 = 970 \text{ K}$$

Now. $P_u = \alpha \phi A_g \{ 0.85 f'_c (1 - \rho_g) + f_y \rho_g \}$

$$\Rightarrow 970 = 0.85 \times 0.75 \times A_g \{ 0.85 \times 4 (1 - 0.03) + 60 \times 0.03 \}$$

$\therefore A_g = 298.46 \text{ in}^2$ and column dia $d = 19.5$ in

Provide $A_g = 314.2 \text{ in}^2$

$d = 20$ in.

Calculate Ast needed from eq^n 10.8.

$$\boxed{P_u = \alpha\phi\left[0.85 f_c' A_g + A_{st}(f_y - 0.85 f_c')\right]} \rightarrow$$ (শুধু steel হ. জন্য

$$\Rightarrow 970 = 0.75 \times 0.85\left[0.85 \times 4 \times 314.2 + A_{st}(60 - 0.85 \times 4)\right\}$$

$$\Rightarrow A_{st} = 8 \text{ in}^2$$

$[A_{st} = \rho_g A_g]$

Use 8 #9 bar.

Spiral spacing: use #3 bar.

$$\boxed{\rho_s = \frac{4 a_s (D_{ch} - d_b)}{s D_{ch}^2}}$$

$$\Rightarrow \overset{0.01152}{\cancel{0.0110.11}} = \frac{4.0.11 \times (17 - 0.375)}{s \cdot 17^2}$$

$$\Rightarrow s = 2.2 \text{ in}$$

Dia of core $= 20 - 2\times1.5$

$= 17$ in

d_b = dia spirals.

a_s = As of spirals.

ρ_s = (As)

$= 0.11$

$$\rho_s = 0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c}{f_y}$$

$$= 0.45\left(\frac{20^2}{17^2} - 1\right)\frac{4}{60}$$

$$= 0.01152$$

Use #3 bar 2" c/c.

Example-6: Design a rectangular tied short column. factored axial load of 1765 kN. $f_c' = 30$ MPa $f_y = 400$ MPa. Column width $b = 300$ mm. steel ratio 2%.

Sol^n:

$$P_u = \phi\alpha A_g\{0.85 f_c (1-\rho_g) + f_y \rho_g\}$$

$$\Rightarrow 10^3 \times 1765 = 0.65 \times 0.8 \times A_g\{0.85 \times 30 * (1-0.02) + 400 \times 0.02\}$$

$$\Rightarrow A_g = 102,887.6 \text{ mm}^2 \cancel{\times 10^3} = 102887 \text{ mm}^2$$

$b = 300$ mm. $\therefore h = \frac{102887}{300} = 343 \approx 350$ mm

$\therefore A_g = 300 \times 350. = 105000\ mm^2$

$\therefore A_s = 0.02 \times 105000\ mm^2 = 2657\ mm^2$

Use 6 # bars of 22 mm dia. ($A_s = 2280\ mm^2$)

Check Axial load strength.

$$\phi P_n = \alpha \phi \left[0.85 f_c (A - A_{st}) + A_s f_y\right)$$

$$= 0.8 \times 0.65 \left[0.85 \times 30 \times (105000 - 2280) + 2280 \times 400\right] \times 10^{-3}$$

$= 1836\ kN. > 1768\ kN$ OK.

Tie spacing: use 10 mm dia tie @ of

$16 \times D = 16 \times 22 = 352\ mm$

$48 \times d = 48 \times 10 = 480\ mm$

$t_{min} = 300\ mm.$

Adopt 300 mm spacing.

** Minimum ratio of spirals $\rho_s = 0.45 \left(\frac{A_g}{A_{ch}} - 1\right) \times \frac{f_c}{f_y}$

spacing, s, $\rho_s = \frac{4 a_s (D_{ch} - d_s)}{D_{ch}^2 * S}$

for Ag. $P_U = \alpha \phi A_g \{0.85 f_c (1 - \rho_g) + f_y \rho_g\}$

for As/P. $P_U = \alpha \phi \left[0.85 f_c A_g + A_{st} (f_y - 0.85 f_c)\right\}$

A straight pre-tensimed member 40 ft long with a cross-section of 15"x15" is concentrically pre-stressed with 1.2 in² of steel wires which are anchored to bulk headed with the stress of 150 ksi. if $E_c = 4.8x1$ $E_s = 29 \times 10^6$ psi, Compute loss of press due to the elastic shortening of concrete at the transfer of concrete.

Solⁿ. Given $E_s = 29 \times 10^6$ psi

$E_c = 4.8 \times 10^6$ psi

$\therefore$ Modular ratio $= n = \dfrac{29 \times 10^6}{4.8 \times 10^6} \approx 6.$

$A_s = 1.2$ in²

$A_c = 15 \times 15$ in²

$P = f_s \times A_s = 1.2 \times 150 = 180$ kip.

$\therefore$ Stress in concrete $f_c = \dfrac{P}{A} = \dfrac{1.80 \times 10^3}{15 \times 15} = 0.8$ ks

$\therefore$ loss of stress in steel $= n f^c$

$= 6 \times 0.8$

$= 4.8$ ksi

Stress in steel after loss $= 150 - 4.8 = 145.2$ ksi

2(a) What are the maximum and minimum values of percentage of steel in Column? Why does the codes suggest those values?

Ans: Maximum and minimum values of steel in column are 8% and 1% respectively.

~~Min. 1% steel~~ is used ~~to resist compressive~~ stress.

→ To resist ~~compressive~~ tensile stress, minimum 1% steel shall be provided.

→ When eccentricity is increasing, the steel area/ratio also have to be increase upto a certain limit. So that the structure will fail by ductile manner. and also it be an economical.

This is why 8% of steel is used as max steel ratio in column.

2(b) Why the φ values for column are lower than those for flexure or shear

There are various reasons why φ values for columns are lower than those for flexure (0.9) or shear (φ=0.75). They are as follow:

(P.T.O)

1. Strength of flexural member (beam) is not much affected by variation in concrete strengt Since it depends primarily on the yield streng of the steel.

2. while the axially loaded member (column) depend strongly on the concrete compressive strength,

2. In column, concrete is being placed from the top down in a long narrow form. which is subjected to segregation than horizontally cast beam.

3. Electrical and other conduits or doors are frequently located in building columns. This Reduces their effective cross-sections.

4. Finally the consequence of column failure would be more catastrophic than that of a single beam in a floor system.

7(b) Design a spiral column to support an axial load of 500 kips. and a live load 25 kip. $f_c' = 4.5$ ksi, $f_y = 60$ ksi steel ratio is 3%. Also design necessary spirals.

Sl^n:

~~$P_u = 1.4\times 250 + 1.7\times 25\ 70 =$~~

$P_u = 1.2\times 500 + 1.6\times 25 = 640$ k

we know.

$$P = \alpha \phi A_g \{0.85 f_c'(1-\rho_g) + f_y \rho_g\}$$

$$\Rightarrow 640 = 0.85\times 0.70\times A_g\{0.85\times 4.8\times(1-0.03) + 60\times 0.03\}$$

$$\Rightarrow A_g = \frac{640}{3.425} = 166.82 \text{ in}^2$$

$$\Rightarrow \pi/4\, D^2 = 166.82$$

$$\therefore D = 15.42 \text{ in}$$

$$\approx 16''$$

$$\therefore A_s = \pi/4\times 16^2\times 0.03 = 5.99 \text{ in}^2$$

Given,

$\alpha = \cancel{0.85}\ 0.70$

$\phi = 0.85$

$\rho = 0.03$

$f_y = 60$ ksi

$f_c' = 4.5$ ksi

provide 6 #9 bars.

spirals #3 spacing.

$$s = \frac{4a_2(D_c - d_b)}{\rho_s D_c^2}$$

$$= \frac{4\times 0.11\times(13-3/8)}{0.0185\times 13^2}$$

$$= 1.77''$$

$a_2 = 0.11$ in^2

$D_c = 16 - 1.5\times 2 = 13$ in

$d_b = 3/8'' = 0.375$ in.

$$\rho_s = 0.45\frac{f_c'}{f_y}\left(\frac{A_g}{A_c}-1\right)$$

$$= 0.95\times\frac{4.8}{60}\times\left(\frac{16^2}{\cancel{13^2}_{15}}-1\right)$$

$$= 0.0185$$

∴ spirals spacing is #3 @ 1.75" c/c.

8(a) A rectangular beam that must carry a service LL = 2.47 k/ft. and DL = 1.05 k/ft. on 18 ft. simple span. Limited cross-section 10" wide and 20 in depth. $f_y = 60$ ksi and $f'_c = 4000$ psi.

As = ? Estimate the steel required.

Soln: Given. LL = 2.47 k/ft

DL = 1.05 k/ft

$$\therefore w_u = 1.2 \times 1.05 + 1.6 \times 2.47 = 5.212 \text{ k/ft}.$$

$$\therefore M_u = \frac{wL^2}{8} = \frac{5.212 \times 18^2}{8} \times 12 = 2533 \text{ kip.in.}$$

To use $\phi = 0.90$ a maximum steel ratio at tensi have to use.

$$\therefore \rho_{0.005} = 0.85 \beta_1 \frac{f'_c}{f_y} * \frac{.003}{.003 + 0.005} = 0.0181$$

$$\therefore A_s = \rho b d = 0.0181 \times 10 \times 16$$

$$= 2.89 \text{ in}^2$$

Assume $d' = 2.5"$ & clearance $= 4"$.

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

$$= 233223 \text{ kip-in.} < M_u.$$

$$a = \frac{2.89 \times 60}{0.85 \times 10 \times 4} = 5.1"$$

$M_u = 0.9 \times 2332.2$ kip-in

$= 2099.$ kip-in < 2533 kip-in

So the beam must be analyse as doubly beam.

∴ Remaining moment $\phi M_{n_2} = (2533 - 2099)$ kip-in

$= 433.99$ kip-in

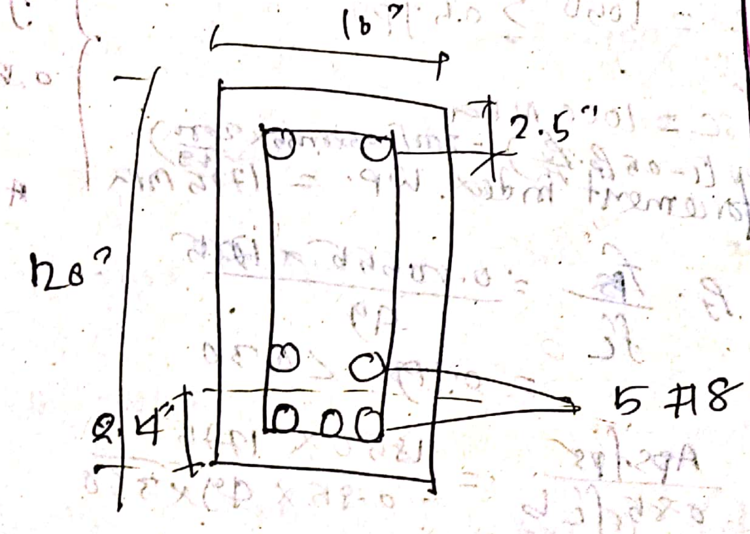
$$\therefore As_1 = \frac{433.99}{0.9\,(16-2.5)\times 60} = 0.595 \text{ in}^2$$

∴ Total tensile reinforcement · $As = (0.599 + 2.89)$ in²

$= 3.485$ in².

provide. tensile steel 5 #8 bar.

and compressive bar 2 #5 bar.



As,min for flexure

$$= \frac{3\sqrt{f'c}}{fy}\,bd$$

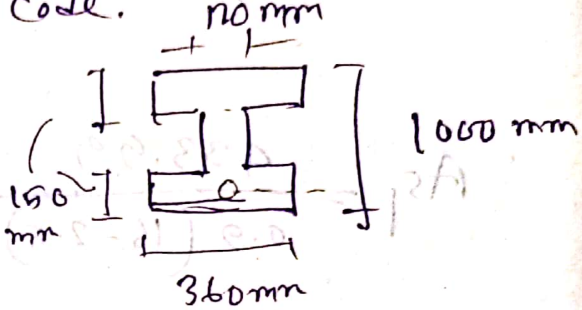
$$= \frac{\cancel{fy}}{\cancel{200}}\,bd \;\; \frac{200}{fy}$$

Column

1% ~ 8%

③ An I-shape beam is prestressed with Aps=1850 mm² as pre-stressing steel with an effective stress. fse = 1050 MPa. Material properties are fpu = 2000 MP

Fc = 49 MPa, Find the ultimate resisting moment of the section. As per ACI code.



Sol^n: $\rho_p = \frac{A_s}{bd} = \frac{1850}{360 \times 925}$

$= 0.00555$

Assume
d = 1000-75
= 925mm

effective steel stress at ultimate by ACI code

As. $f_{se} = 1050 \geq 0.5 f_{pu}$

$\therefore f_{se} = 1000$ MPa

and $f_{ps} = f_{pu}\left[1 - 0.5\, \rho_p \cdot \frac{f_{pu}}{f_c'}\right) = 2000\left[1 - 0.5 \times 0.00555 \times \frac{2000}{49}\right)$

check reinforcement index. wp. = 1775 MPa

$f_{se} = 1050$ MPa
$0.5 f_{pu} = 0.5 \times 2000$
$= 1000$ MP
Adopt lower value

$w_p = \rho_p \cdot \frac{f_{ps}}{f_c'} = \frac{0.00555 \times 1775}{49}$

$= 0.199 < 0.30$.

$a = \frac{A_{ps} f_{ps}}{0.85 f_c' b} = \frac{1850 \times 1775}{0.85 \times 49 \times 360}$

$= 219$ mm > 150 mm

T- beam Now determine the eatent of compressin zone.

$T = f_{ps} A_{ps} = \cancel{1480}\ 1775 \times 1850 = 2928.75\ kN.$

$$\text{Area of compression zone} = \frac{2928.75 \times 10^3}{0.85 f'_c} = \frac{2928.75 \times 10^3}{0.85 \times 49}$$

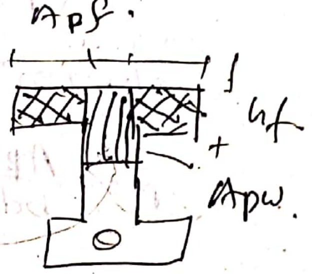
$$= 70.318 \times 10^3\ mm^2$$

$\text{Flange Area} = 360 \times 150 = 54 \times 10^3\ mm^2$

web area below flange = compression zone - flange ar

$= 16.31 \times 10^3\ mm^2$

$$\therefore a = 150 + \frac{16.31 \times 10^3}{b_w = 120} = 285\ mm$$



$$\therefore A_{pf} = 0.85 \times \frac{49}{f_{ps}} (b_f - b_w) \times h_f.$$

$$= 0.85 f'_c (b - b_w) h_f / f_{ps}$$

$$= 0.85 \times 49 (360 - 120) \times 150 / 1775$$

$$= 845\ mm^2$$

$A_{pw} = A_{ps} - A_{pf} = 1850 - 845 = 1005\ mm^2$

$M_{web} = A_{pw} f_{ps} (d - a/2) = 1005 \times 1775 (925 - 285/2)$

$= 1395.8\ kN.m$

$M_{flange} = 0.85 f'_c (b - b_w) h_f (d - h_f/2)$

$= 0.85 \times 49 (360 - 120) \times 150 (925 - 150/2) = 1275\ k.m$

$M_n = A_{pf} f_{ps} (d - h_f/2) + A_{pw} f_{ps} (d - a/2)$

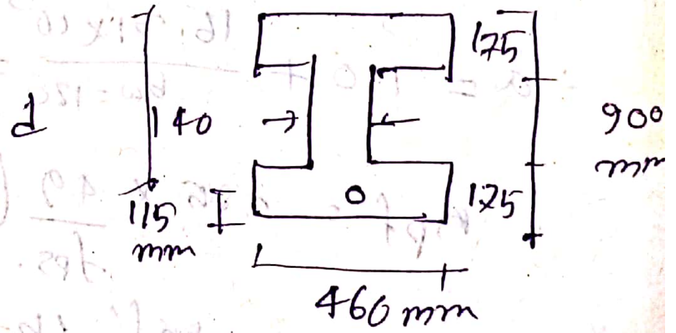
✓

pre-stressed

Design for flexure

problem-1: An I-shape beam is pre-stressed with $A_{ps} = 1750\ mm^2$ as prestressing steel. with an effecti stress $f_{se} = 1100$ MPa. c.g.s of strands is 115 mm ab the bottom of the beam. Material properties are as $f_{pu} = 1860$ MPa, $f'_c = 48$ MPa. find the ultimate Resisting moment. following the ACI code

Solution:



$$\rho_p = \frac{A_{ps}}{bd} = \frac{1750}{460 \times (900-115)}$$

$$= 0.00485$$

effective stress in steel at Ultimate

$f_{se} = 1100$ MPa $> 0.5 f_{pu}$

$\therefore f_{se} = 925$ MPa

$0.5 f_{pu} = 1860 \times 0.$
$= 925$ MP

$f_{se} = 1100$ MP

([illegible]

$$\therefore f_{ps} = f_{pu}\left(1 - 0.5\, \rho_p \frac{f_{pu}}{f'_c}\right)$$

$$= 1860\left(1 - 0.5 \times \frac{0.00485 \times 1860}{48}\right)$$

$$= 1686 \text{ MPa}.$$

R

check reinforcement index. w_p.

$$w_p = \rho_p \times \frac{f_{ps}}{f'_c} = 0.00485 \times \frac{1685}{48} = 0.17 < 0.30$$

$$\therefore T' = A_{ps} f_{ps} = 1685 \times 1750 \text{ N} = 2948.75 \text{ kN}.$$

$$\therefore a = \frac{A_{ps} f_{ps}}{0.85 f'_c b} = \frac{2948.75 \times 10^3}{0.85 \times 48 \times 460} = 157 \text{ mm} < 175 \text{ mm}$$

∴ Rectangolar beam section.

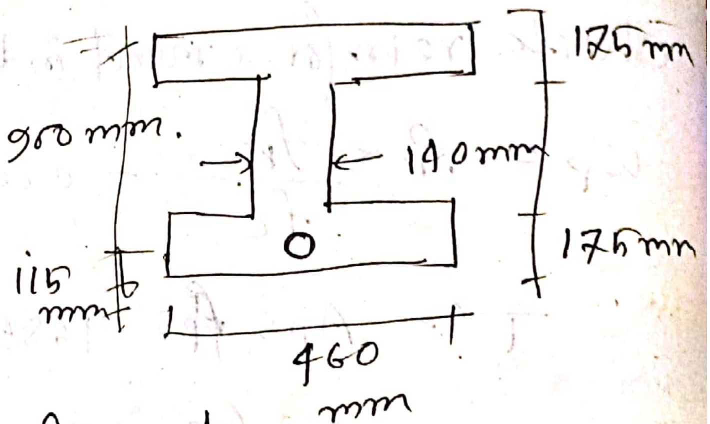
$$\therefore M_n = A_{ps} f_{ps} (d - a/2) = 2948 (785 - 157/2)$$

$$= 2082.762 \text{ kN m}.$$

$$\phi M_n = 0.9 \times 2082.762 = 1875 \text{ kN.m} \text{ (Ans)}$$

Problem-02: An I-shape beam is pre-stressed with $A_{ps} = 2350 \text{ mm}^2$, effective stress $f_{se} = 1100$ MPa. The c.g.s of the stands is 115 mm above the bottom of the beam. Material properties are $f_{pu} = 1860$ MPa $f'_c = 48$ MPa. Find the ultimate restisting Moment of the section. following ACI Code.

Solution:



effective stress at ultimate

$f_{se} = 1100$ MPa $> 0.5 f_{pu}$

$\therefore f_{se} = 930$ MPa.

$f_{pu} = 1860$ MPa

$\Rightarrow f_{ps} = f_{pu}\left[1 - 0.5\, \rho_p \cdot \frac{f_{pu}}{f'_c}\right)$

$= 1625$ MPa.

$\rho_p = \frac{A_{ps}}{bd}$

$= \frac{2350}{460 \times 785}$

$= 0.00651$

Check reinforcement index.

$w_p = \rho_p \cdot \frac{f_{ps}}{f'_c} = 0.00651 \times \frac{1625}{48}$

$= 0.22 < 0.30.$ (tension controls)

$a = \frac{A_{ps} f_{ps}}{0.85 f'_c b} = \frac{1625 \times 2850}{0.85 \times 48 \times 460} = 203.47 \text{ mm} > 125 \text{ mm}.$

So the beam must be analyse as T beam, secti

$\therefore$ Determine the extent of the compression zone.

$T = A_{ps} f_{ps} = 1625 \times 2350 = 3819$ kN.

$\therefore$ Area of compression zone $= \frac{3819}{0.85 f'_c} = \frac{3819 \times 10^3}{0.85 \times 48}$

$= 93 \times 10^3 \text{ mm}^2$

Flange Area = $460 \times 175 = 80.5 \times 10^3$ mm²

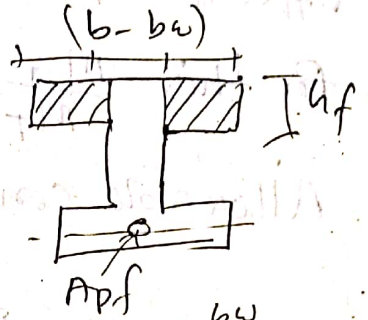
∴ Web area below flange = compression zone − flange

$= 93 \times 10^3 - 80.5 \times 10^3$

$= 12.7 \times 10^3$ mm²

$$a = 175 + \frac{12.7 \times 10^3}{140} = 265.7 \approx 266 \text{ mm}$$

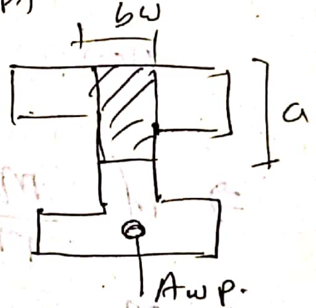
Using ACI commentary equations.



$A_{pf} = 0.85\, f'_c\, (b - b_w)\, h_f / f_{ps}$.

$= 0.85 \times 48\, (460 - 140)\; 125 / 1625$

$= 1406$ mm²



$A_{pw} = A_{ps} - A_{pf}$

$= 2350 - 1406 = 944$ mm².

∴ Moment $M_n = M_{n_1} + M_{n_2}$

$= M_{web} + M_{flang} = A_{pfw}\, f_{ps}\, (d - a/2)$

$+ \underline{0.85 f'_c (b - b_w) . h_f} . (d - h_f/2)$

$= 944 \times 1625\, (785 - 266/2) + 0.85 \times 48\, (460 - 140)$

$\times 175\, (785 - 175/2)$

$= 2593.81.6$ kN m

$\phi M_n = 0.9\, M_n = 2334.4$ kN m.

33th BCS

(g) How $\frac{M_T}{M_{CT}}$ ratio influence the flexural desig of a prestressed concrete structural element?

Ans: If $\frac{M_G}{M_T} < 0.2$, the design is controlled by M_L (live load moments). ∴ $M_L = M_T - M_G$.

Effective pre-stressed force, $F = \frac{M_L}{0.5h}$

for this $\frac{M_G}{M_T}$ is small, so I section is selected.

Allowable concrete stress = 50% of the max. allow. f
$= 0.5 f_c$.

Now. $F = \boxed{A_{ps} \times f_{se} = 0.5 f_c \cdot A_c}$

⇒ if $\frac{M_G}{M_T} > 0.2$, The design is controlled by M_T;

In this case. $F = \frac{M_T}{0.65h}$

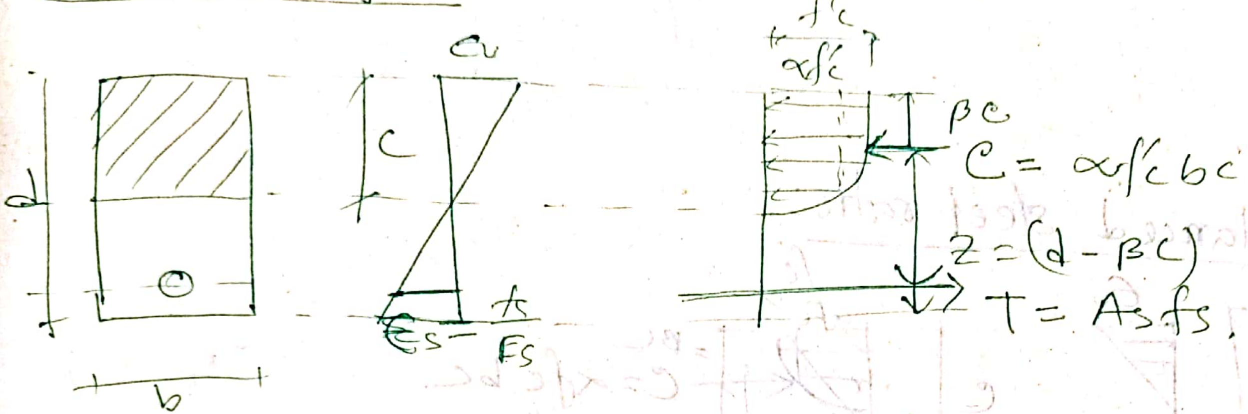
for large M_G/M_T ratio. T section is selected

M_G = Girder moment
M_T = total moment
h = overall height

8

⑥ Derive the equation. $M_n = \rho b d^2 (1 - 0.59\, \rho \frac{f_y}{f'_c})$.

Flexural strength:



at balanced failure $f_s = f_y$. or. failure by yielding of steel. and failure by crusing of concrete

from figure.

$$C = T$$

$$\alpha f'_c b c = A_s f_s$$

$$\Rightarrow c = \frac{A_s f_s}{\alpha f'_c b}$$

$$= \frac{A_s f_y}{\alpha f'_c b}$$ [for tension failure $f_s = f_y$)

$$\left\{ \begin{array}{l} \beta = 0.425 \\ \alpha = 0.72 \end{array} \right\}$$

and moment, $M = Tz$

$$= A_s f_s (d - \beta c)$$

$$= \rho b d\, f_y \left(d - \beta \cdot \frac{A_s f_y}{\alpha f'_c b}\right)$$ [$\because f_s = f_y$]

$$= \rho b d^2 f_y \left(1 - \frac{\beta}{\alpha} \cdot \frac{A_s}{bd} \cdot \frac{f_y}{f'_c}\right)$$

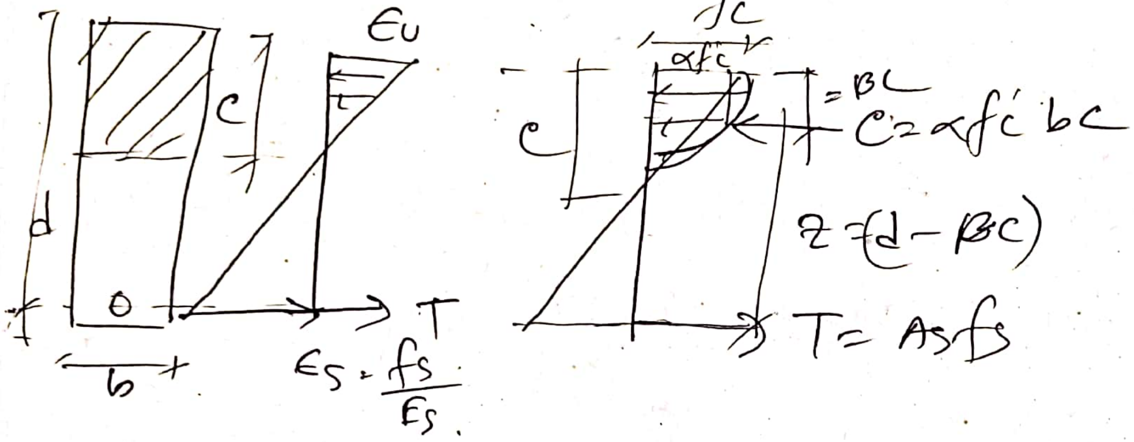
$$= \rho b d^2 f_y \left(1 - \frac{0.425}{0.425} \rho \frac{f_y}{f'_c}\right)$$

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

Derive $\rho_b = \alpha \frac{f_c'}{f_y} \cdot \frac{0.003}{0.003 + f_y/E_s}$

$= 0.85\,\beta_1 \frac{f_c'}{f_y} \cdot \frac{87}{87 + f_y}$.

Balanced steel ratio:



from figure: $f_s = E_s \epsilon_s$

$\Rightarrow \epsilon_s = \frac{f_s}{E_s}$

and. $\frac{\epsilon_u}{c} = \frac{\epsilon_s}{d-c} \Rightarrow \frac{d-c}{c} = \frac{\epsilon_s}{\epsilon_u}$

$\Rightarrow \frac{d}{c} = \frac{\epsilon_s}{\epsilon_u} + 1 = \frac{\epsilon_s + \epsilon_u}{\epsilon_u}$.

$\Rightarrow \epsilon_s = \frac{d-c}{c} \times \epsilon_u$ $\Rightarrow c = d \times \frac{\epsilon_u}{\epsilon_s + \epsilon_u}$.

$$\frac{d}{c} = \frac{\epsilon_u + \epsilon_y}{\epsilon_u}$$

$$\Rightarrow c = d \cdot \frac{\epsilon_u}{\epsilon_u + \epsilon_y}$$

$$\therefore C = T$$

$$\Rightarrow A_s f_s = \alpha f'_c b c$$

$$\Rightarrow \cancel{A}\rho bd \cdot f_y = \alpha f'_c b \times \frac{\epsilon_u}{\epsilon_u + \epsilon_y} \cdot d$$

$$\Rightarrow \rho f_y = \alpha f'_c \frac{\epsilon_u}{\epsilon_u + \epsilon_y} .$$

$$\therefore \boxed{\rho = \alpha \frac{f'_c}{f_y} \cdot \frac{\epsilon_u}{\epsilon_u + \epsilon_y}}$$

$\beta_1 = 2\beta$.
$\alpha = 0.85 \beta_1$
$= \gamma \beta_1$

1) T- beam.

Effective flange width

① $b_e = L/4$ · (span = L)

ⅱ $b_e = 16 h_f + b_w$

ⅲ b_e = c/c . beam distance.

adopt smollest one.

for overhanging beam.

i) $b - b_w \leq L/12$

ii) $b - b_w \leq 6 h_f$

iii) $b - b_w \leq \frac{1}{2}$ of c/c beam distance.

⇒ should be $b \leq 4 b_w$) total flange width = b

⇒ flange thickness. $h_f \geq b_w/2$.

⊞ Doubly reinforced beam.

procedure.

1. if $\rho > \rho_{max}$ beam must be analyse as doubly reinforced beam.

2. if $\rho > \bar{\rho}_{cy}$ ~~tension~~ compression steel yields, when beam fails.

$$\therefore M_n = (A_s - A_s') f_y (d - a/2) + A_s' f_y (d - d')$$

$$\left[\varphi = 0.483 + 83.3 \times \epsilon_t \longrightarrow \epsilon_t = \left(\epsilon_u \times \frac{d-c}{c}\right).\right.$$

3. if $\rho < \bar{\rho}_{cy}$ then calculate c

from this equation. $A_s f_y = 0.85 \beta_1 f_c' b c + A_s' f_s'$

$$= 0.85 \beta_1 f_c' \cdot bc + A_s' \epsilon_u \cdot E_s \frac{c-d}{c}$$

And calculate. $f_s' = \epsilon_u . E_s . \frac{c-d}{c} \leq f_y$.

$$\left[a = \cancel{\frac{A_s f_y - A_s' f_s'}{0.85 f_c' b}}\right] \quad (a = \beta_1 c)$$

$\therefore$ 4. calculate $M_n = A_s' f_s' (d - d') + 0.85 f_c' ab (d - a/2)$

$$= A_s' f_s' (d - d') + \underline{0.85 f_c' ab} (d - a/2)$$

$$A_s f_y = 0.85 f_c' bc + A_s \epsilon_u E_s \frac{d-c}{c}$$

$$= 0.85 f_c' ab (d - a/2)$$

② Define T-beam.

When.

→ RC beam and slab are monolithically cast, beam stirrups and bent bars extended into the slab.

→ A part of slab act along with beam to take longitudinal compression.

→ Slab form the beam flange.

→ Part of beam below slab is called web/steb.

This type of beam is known as T-beam.

Design procedure

⇒ it may be Rectangular or T-beam

⇒ if $a > h_f$. then T-beam analyse must be carried out.

① calculate $a = \frac{A_s f_y}{0.85 f'_c b}$.

| $a > h_f$: T-beam
| $a \ngtr h_f$: Rectangular beam.

⑪ $A_{sf} = \frac{0.85 f'_c (b-b_w) \cdot h_f \cdot}{f_y} \cancel{fd} -$

$\therefore M_{n_1} = A_{sf} \cdot f_y (d - h_f/2)$

Remaining steel Area $= A_s - A_{sf}$.

| $a = \frac{(A_s - A_{sf}) f_y}{0.85 f'_c b_w}$

$M_{n_2} = (A_s - A_{sf}) f_y (d - a/2)$.

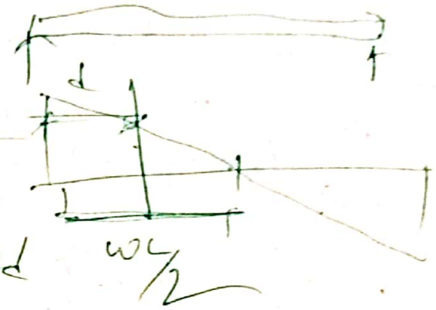
$\therefore$ Total moment $= M_{n_1} + M_2$

$$= A_{sf}\, f_y \left(d - h_f/2\right) + \left(A_s - A_{sf}\right)\left(d - a/2\right)$$

Shear reinforcement design

1. Shear reinforcement লাগবে যদি $V_{cr} > \phi V_c$

$$V_{cr} = \boxed{\frac{wL}{2} - wd}$$



at a distance $= \frac{L}{2}\frac{(V_{cr} - \phi V_c)}{V_{cr}}$

2. Minimum shear must be provided

at a distance $= \frac{L}{2}\left(\frac{V_{cr} - \frac{\phi V_c}{2}}{V_{cr}}\right)$.

3. Spacing. column থেকে d. distance পর্যন্ত

i) $S = \frac{\phi \cancel{A_3} A_v f_y d}{V_s}$

$= \frac{\phi A_v f_y d}{V_s}$.

$V_s = V_{cr} - \phi V_c$
= excess shear.
$A = \#3 = 2 \times 0.11 = 0.22$
d = depth

4. d থেকে required distance পর্যন্ত -

i) $S_{max} = \frac{A_v f_y}{0.75\sqrt{f'_c}\ b_w} \le \frac{A_v f_y}{50 b_w}$

ii) $S_{max} = d/2$

iii) $S_{max} = 24''$

Adopt lower

(a) From support to d distance provide

#3 @ 5" c/c

(b) from distance to $\boxed{\text{requr}}$ # 11" c/c.

(c) from $\boxed{\text{req}}$ to minim s = minimum spacing by ACI code.

W.S.D method

1. Maximum shear at support $= \frac{wL}{2}$

$$V_{cr} = \boxed{\frac{wL}{2} - w \times d}$$

shear check $V_{cr} > V_c$

$V_c = 1.1\sqrt{f_c'} \cdot bd$

provide at a distance

$$= \frac{L}{2}\left(\frac{V_{cr} - V_c}{V_{cr}}\right)$$

2. minimum shear @ 250 upto $\boxed{\text{required (d)}}$

∴ total distance = required (d)

= spacing = ① support to d distance

$$S = \frac{A_v f_y d}{(V_u - V_c)}$$

$V_U = V_{cd} - V_e$

if $v_u < \frac{6\sqrt{f'_c}}{bd}$

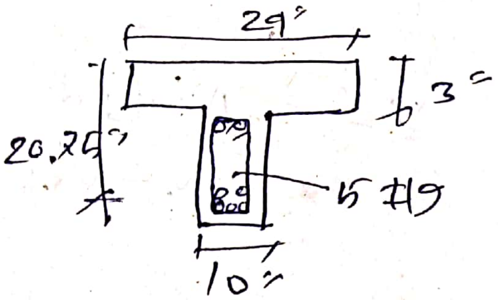
② d to required distance

$S_{max} = d/2$

$S_{max} = \frac{A_v}{0.0015 b}$

4. Determine ultimate moment of T-beam.

$f'_c = 3\,ksi$, $f_y = 40\,ksi$.



Sol^n: check $a < h_f$

$$a = \frac{\cancel{0.85} A_s f_y}{0.85 f'_c b} = \frac{5 \times 40}{0.85 \times 3 \times 24} = 4.90'' > h_f$$

The beam must be analyse as T-beam

Reinforcement in flange. $A_{sf} = \frac{0.85 f'_c \times (b_e - b_w) h_f}{f_y}$

$= 2.67\ in^2$

web Reinforcement $A_{s1} = 5 - 2.67 = 2.33\ in^2$

$$a = \frac{\cancel{0.85}\ 2.33 \times 40}{0.85 \times 3 \times 10} = 3.65''$$

$$c = a/\beta_1 = \frac{3.65}{0.85} = 4.29.$$

$$\therefore c/d_t = \frac{4.29}{20.75} = 0.207 < 0.375 \therefore \varphi = 0.90$$

$$\varphi M_n = 0.9 \left[A_{s1} f_y (d - a/2) + A_{sf} f_y \left(d - \frac{h_f}{2}\right) \right]$$

$$= 0.9 \times \left[2.33 \times 40 (20.75 - 3.65/2) + 2.67 \times 40 (20.75 - 3) \right]$$

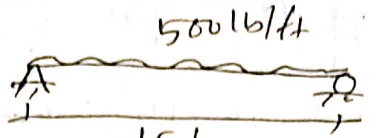
$= \cancel{3437.749}$ k.in.

$= \cancel{277.47}$ ~~kip-ft~~

$= 3437.74$ kip-in $= 286.47$ k-ft.

BCS written — sheet.

① #1. Design the following beam . Also show x-section with reinforcement. given $f_c' = 3\,ksi$, $f_y = 40\,ksi$. use WSD method.



Soln: ∴ Moment, $M = \frac{wL^2}{8} = \frac{500 \times 15^2}{8}$

$= \overset{42187.5\ lb\text{-}ft}{\cancel{14062.5}}\ lb\text{-}ft$

$= \cancel{168750}\ lb\text{-}in.$

$= 506250.\ lb\text{-}in$

Given. $f_c' = 3\,ksi$
$f_y = 40\,ksi$
$L = 15'$
$w = 1500\,lb/ft$

Modular ratio, $n = \frac{E_s}{E_c} = \frac{29 \times 10^6}{57.000\sqrt{fc}}$

$= \frac{29 \times 10^6}{57000 \times \sqrt{3000}} = 9.2 \approx 9.$

$r = \frac{f_s}{f_c} = \frac{0.4 \times 40}{0.45 \times 3} = 11.\cancel{1}85 = 11.85.$

$k = \frac{n}{n+r} = \frac{9}{9+11.85} = 0.432$

$\therefore j = 1 - k/3 = 1 - 0.432/3 = 0.856.$

Now, $M_c = \frac{1}{2} f_c k j bd^2 = \frac{1}{2} \times 0.432 \times b \times d^2 \times \underline{fc}$

$\Rightarrow 506250 = \frac{1}{2} \times 0.432 \times bd^2 \times fc$

Assume $b = 10''$

$\therefore d = 13.176'' \approx 14''$

Again, $As = \frac{Mc}{fy\, j\, d} = \frac{506250}{40000 \times 0.856 \times 14}$

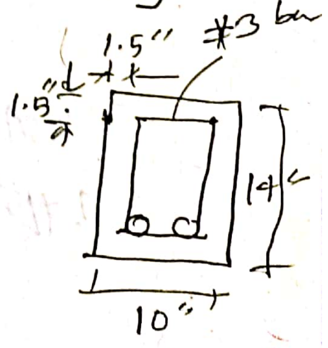
$= 1.056\ in^2$

$Ms = As\, fy\, jd.$

but $As.min = \frac{3\sqrt{fc}\ bd}{fy} \geq \frac{200\, bd}{fy}$

$= 0.57\,in^2 \geq 0.7\,in^2 < As.$

So. $As = 1.056\ in^2$
use 2 #7 as main bar.



Desing the 'previous beam by USD method.

Ans: Given, $L = 15'$, $w = 1500$ lb/ft. $f_c' = 3$ ksi $f_y = 40$ ksi

$\therefore$ Factored load $= 1.4 \times 1500$ lb/ft $= 2250$ lb/ft

$\therefore$ Moment, $M_U = \frac{w_u l^2}{8} = \frac{2250 \times 15^2}{8} = 63281.25$ lb-ft

$= 759.375$ kip-in.

$\rho_{0.005} = 0.85 \beta_1 \frac{f_c'}{f_y} \times \frac{0.003}{0.003+0.005} = 0.85 \times 0.85 \times \frac{3}{40} \times \frac{0.003}{0.003+0.0}$

$= 0.0203.$

$\therefore M_U = \phi \rho b d^2 f_y (1 - 0.59 \rho f_y / f_c')$

$\Rightarrow 759.375 = 0.9 \times 0.0203 \times 40 \times b d^2 (1 - 0.59 \times 0.0203 \times \frac{40}{3}$

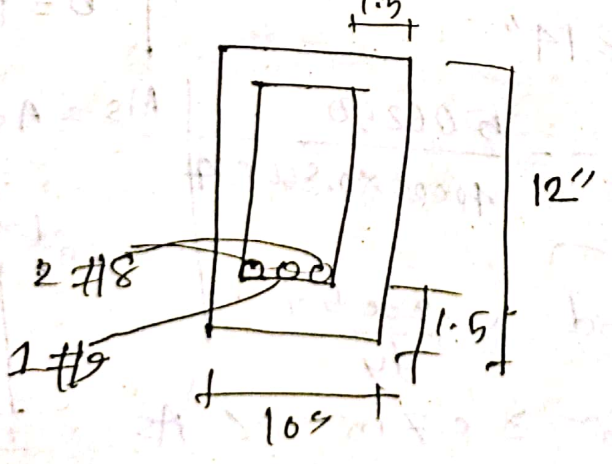
$\Rightarrow b d^2 = 1236.57$

$\therefore d = 11.12'' \approx 12''$

Assume, $b = 10''$

$\therefore A_s = \rho b d = 0.0203 \times 10 \times 12 = 2.436$ in^2

provide 2#8 and 1#9 bar.



$\boxed{1}$ Columns stirrups requirements.

Use #3 bars as stirrups upto longitudinal bar #10.
use #4 bars as stirrups utpo " more than #10.

Maximum tie spacing in Column. (minimum of the following)

i) 48d
ii) 16D
iii) least dimension of column.

Here,
d = dia of stirrup.
D = dia of main bar.

$\boxed{2}$ An interior column of a six storied building at ground floor carries 300 kip. Design column as tied column. Also design a square footing for this load. Given. $f_c' = 3$ ksi $f_y = 45$ ksi. and safe bearing capacity is 2 k/ft²

Ans: Now.

$$P = \alpha \phi A_g \left(0.85 f_c' (1 - \rho_g) + f_y \rho_g\right\}$$

$$\Rightarrow 300 = 0.80 \times 0.65 \times \{0.85 \times 3 (1 - 0.02) + 45 \times 0.02\} A_g$$

$$\Rightarrow A_g = 169.733 \text{ in}^2$$

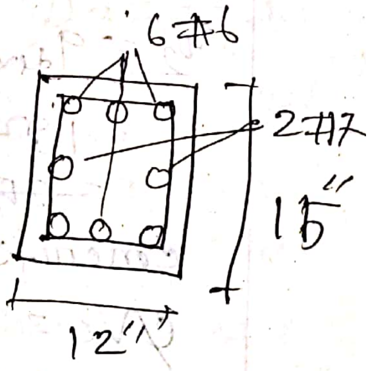
$$\Rightarrow b \times d = 169.733 \text{ in}^2$$

$$\therefore d = 14.14 \approx 15 \text{ in.}$$

Given. $f_c' = 3$ ksi
$f_y = 45$ ksi,
$P = 300$ kips.
Assume $\rho_g = 0.02$.
Assume $b = 12''$

$$\therefore A_{st} = \rho bd = 0.02 \times 12 \times 15 = 3.6 \text{ in}^2$$

provide 6 #6 and 2 #7 bar.



tie spacing

$16D = 16 \times 6/8 = 12''$
$48D = 48 \times 3/8 = 18''$
$t_{min} = 12''$

Adopt lower spacing
provide #3 as tie
@ 12" c/c.

Footing design:

Given load = 300 kip.

net bearing capacity = 2 k/ft^2

∴ Footing Area: $A = \frac{P}{q_c} = \frac{300}{2} = 150$ ft^2

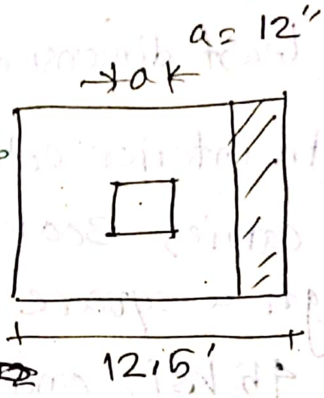
∴ $A = 12.24 \times 12.24 \simeq 12.5 \times 12.5$ ft^2

Developed bearing pressure,

$q_{dve} = \frac{\cancel{1.2} \times 300}{12.5 \times 12.5} = 1.92$ k/ft.

Beam shear check.

$V_u = \left(\frac{b-a}{2} - d\right) \times b \times q_{dev}$.



$V_u = q_{dev} \times \left(\frac{b-a}{2} - d\right) \times 12.5 \times \cancel{12}$

$= 1.92 \times \left(\frac{12.5 \times 12 - 12}{2 \times 12} - \frac{21}{12}\right) \times 12.5 \times \cancel{12}$

~~= 19368~~ k

~~= 1614 kip.~~ = 96 kip.

Assume
d = 21"
h = 24"

$\phi V_c = 2\phi\sqrt{f_c}\, bd$

$= 2 \times 0.75 \times \sqrt{3000} \times 12.5 \times 12 \times 21$

= 258.798.9 kip.

Punching Shear. check:

$V_{u2} = q_{dev} \times$ $(A_g - A_{pun})$

$V_{u2} = q_{dev} \times \left\{12.5^2 - \left(\frac{a+d}{12}\right)^2\right\}$ ~~× 12.5~~

$= 1.92 \times \left\{12.5^2 - \left(\frac{12+21}{12}\right)^2\right\}$ ~~× 12.5~~

~~= 3568.5 kip~~. = 297.375 kip

concrete shear strength

$\phi V_c = 4\phi\sqrt{f_c}\, bd$

b = punching perimeter

$= 4 \times 0.75 \sqrt{4000} \times 4 \times (12+21) \times 21$

525.95 kip > V_{u2}.

1/9

Ex. Design a spiral column to support an axial load of 500 kip and live load 25 kip. $f'_c = 4.5$ ksi $f_y = 60$ ksi. and steel ratio 3%. Also design necessary spirals.

Ans: $P_u = 1.2\times500 + 1.6\times25 = 640$ kip.

Given
$\rho_g \leq 0.05$

We know.

$$P_u = \alpha\phi A_g\{(1-\rho_g)0.85f'_c + f_y\rho_g\}$$

$$\Rightarrow 640 = 0.75\times0.85\times A_g\{0.85\times4.5\times(1-0.03) + 0.03\times60\}$$

$$\Rightarrow 640 = 0.6375A_g(3.710 + 1.8)$$

$$\Rightarrow A_g = \frac{640}{3.5126} = 182 \text{ in}^2$$

$$\Rightarrow \frac{\pi}{4}d^2 = 182 \text{ in}^2$$

$\therefore d = 15.20$ in.

Assume $d = 16$ in

$\therefore A_{st} = 0.03\times\frac{\pi}{4}\times16^2$
$= 6.03$ in^2

Adopt 7 #9 bars.

use #3 as spirals.

spiral spacing.

$$\rho_s = \frac{4a_2(D_{ch}-d_b)}{sD_{ch}^2}$$

$$\Rightarrow \overset{0.0173}{\cancel{0.0447}} = \frac{4\times0.11(\overset{13}{\cancel{10}}-0.375)}{s\times10^2}$$

$$\Rightarrow s = \frac{4\times\cancel{0.11}\times0.626}{\overset{}{\cancel{0.0447}}\times100} = 2.1069683$$
0.01736

Here.

$d_b = 3/8 = 0.375$ in

$D_{ch} = 16 - 2\times\overset{1.3}{\cancel{3}} = 13$ in

$\therefore \rho_s = 0.45\times\left(\frac{D^2}{D_{ch}^2}-1\right)\times\frac{\cancel{4.5}}{\cancel{60}}\frac{f'_c}{f_y}$

$= 0.01478 = 0.01476$

$\rho_s = 0.45\times\left(\frac{A_g}{A_{ch}}-1\right)\frac{f'_c}{f_y}$

$= 0.01736$

$$\rho_s = \frac{4a_s(D_{ch} - d_b)}{s D_{ch}^2}$$

$$\Rightarrow 0.0173 = \frac{4 \times 0.11 \times (13 - 0.375)}{s \times 13^2}$$

$$\Rightarrow s = \frac{6.55}{0.0173 \times 13^2}$$

$$= 1.89 \text{ in}.$$

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

$D_{ch} = 16 - 3 = 13 \text{ in}$

$$\rho_s = 0.45 \times \left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_y}$$

$$= 0.45 \times \left(\frac{15^2}{13^2} - 1\right) \cdot \frac{4.5}{60}$$

$$= 0.0173$$

∴ Adopt spiral spacin #3 bar @ of 1.8" c/c.
Main reinforcement 7 #8 bar.

$\boxed{4}$ Define passenger load factor. (PLF). Calculate PLF from the following data. 5 flights per day. seat capacity = 100 passengers. Average distance travelled per flight = 250 km. Average passenger per flight = 68

Passengers load Factors (PLF): It is a measure of the capacity utilization of public transport service like airlines, trains, buses, etc. It is generally used to asses how efficiency a transport provider fills seats and generates for revenue.

Plm: PLF = $\frac{\text{RPK (Revenue passenger kilometer)}}{\text{ASK (Available passenger seats per flight} \times \text{travelled dist.)}}$

$$= \frac{5 \times 68 \times 250}{5 \times 100 \times 250}$$

$$= 0.68.$$

Define EIA:

Environmental Impact Assessment is the process by which the anticipated effects on the environ: of a proposed development project are measured If the likely effects are unacceptable design measures or other relevant mitigation measures can be taken to reduce or avoid those effects.

Define T-beam

When R.C beams and slab are monolithically cast, beam stirrups and bent bars are extended into the slab. A part of slab acts along with beam to take longitudinal compression. Slab forms the beam flange. A part of beam below slab is called web/steb.

This type of beam is known as T-beam.

Advantages of T-beam

i) It offers greater resistance than Rectangular beam.

ii) It saves 9~20 % reinforcement compared to R.C beam.

iii) Required depth of beam will also reduce as the flange or web contribute against sagging moments

(iii) It reduces the depth of beam as the flange or web contribute against the sagging moments.

[E] Define a simpledy supported rectangular beam for a bending moment of 1440 k-in. Given $f_c' = 3.5$ ksi, $f_y = 60$ ksi, Modular ratio, $n = 9$. Use WSD method.

Ans: $M = 1440$ k-in, $n = 9$, $r = \frac{f_s}{f_c} = \frac{0.4 \times 60}{0.45 \times 3.5}$

$= 15.24$.

$\therefore k = \frac{n}{n+r} = \frac{9}{9+15.24} = 0.38$

$\therefore j = 1 - k/3 = 1 - 0.38/3 = 0.87$.

We know, $M_c = \frac{1}{2} f_c' kjbd^2$

$\Rightarrow 1440 = \frac{1}{2} \times 0.45 \times 3.5 \times 0.38 \times 0.87 \times bd^2$

$\Rightarrow bd^2 = 5538.46$

$\therefore d = \sqrt{\frac{5538.46}{12}} = 21.48$

$\approx 22''$

Assume $b = 12''$

$\therefore bd = 12 \times 22$ in^2

$\varphi = 0.65 + 0.25\left(\frac{1}{c/d_t} - 5/3\right)$

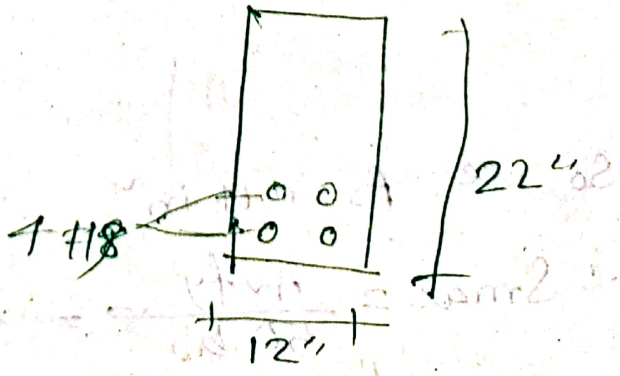
$= 0.483 + 83.3\,\epsilon_t$

$\epsilon_t = \epsilon_s \times 0.003 \times \frac{c-d}{c}$

Again, $M_s = A_s f_s jd$

$1440 = A_s \times 0.4 \times 60 \times 0.87 \times 22$

$\therefore A_s = 3.13$ in^2



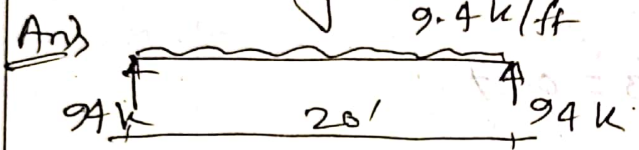
*** ⓪ Shear design:

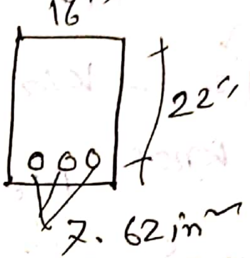
A rectangular beam of 16×22 in^2 effective depth = 22" carries a total factored load of 9.4 k/ft span = 20' $f_y = 60$ ksi $f'_c = 4$ ksi

What point of the beam is web reinforcement req'd?

Find spacing of web reinforcement for the beam.

Ans





$V_u = 94$ k

$V_{cr} = 94 - \frac{22}{12} \times 9.4$

$= 76.8$ kips.

Shear resisted by concrete

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

$= 2 \times 0.75 \times \sqrt{4000} \times 16 \times 22 = 33.4$ kips.

$\frac{\phi V_c}{2} = 16.7$ kips.

∴ stirrup should be provided upto $= 10 \times \frac{94 - 16.7}{94}$

$= 8.22'$ from column face.

So, use $A_s = 0.11$ in^2 ∴ $A_v = 2 \times 0.11 = 0.22$ in^2

∴ $S_{max} = \frac{A_v f_y}{50\ b_w} = \frac{0.22 \times 60000}{50 \times 16} = 16.5"$

or $t_{min} = 16"$

$= d/2$ } Adop lower spacing of 11" c/c
$= 11$ in

Now. shear carried by steel

$\phi V_s = V_u - \phi V_c = 94 - 33.4 = 61.6$ kips

$$S_{req} = \frac{\phi A_v f_y d}{\phi V_s} = \frac{0.75 \times 0.22 \times 60 \times 22}{61.6} = 4.75''$$

provide #3 · bar as stirrup @ 4.75 c/c upto ϕV_c from column face and @ 11" c/c from ϕV_c to $\frac{\phi V_c}{2}$

☒ Use of disturbed sample

1. Grain size analysis.
2. Determination of LL and PL.
3. Specific gravity of soil solids.
4. classification of soil.

☒ Use of undisturbed sample.

1. consolidation
2. Hydraulic conductivity.
3. Shear strength test.

Area Ratio $A_R(\%) = \frac{D_o^2 - D_i^2}{D_i^2}$

D_o = out side dia. of sampler
D_i = insite dia of -

10% < disturbed sample [<

~~10%~~ → Undisturbed sample. < 10%

[illegible] A rectangular beam that must carry a service live load 2.47 k/ft and dead load of 1.05 k/ft on 18' span is limited to cross-section for architectural reason 10". and 20" depth. $f_y = 60$ ksi $f_c' = 4$ ksi, find steel requir^n.

Ans: factored load $w_u = 1.6 \times 2.47 + 1.2 \times 1.05 =$

$= 3.952 + 1.26 = $ ~~4.644 k/ft~~

$= 5.212$ k/ft

$\therefore$ Moment $= \dfrac{w_u l^2}{8} = \dfrac{5.212 \times 18^2}{8} = 211$ k-ft

$\therefore$ $M_u = 2533$ k-in.

$$\rho_{0.005} = 0.85 \times \beta_1 \times \frac{f_c'}{f_y} \times \frac{0.003}{0.003 + 0.005} = 0.0181$$

$\therefore$ $A_s = \rho b d = 0.0181 \times 10 \times 17.5 = 3.16$ in^2

$\therefore$ $M_{n_1} = A_s f_y (d - a/2)$

$= 3.16 \times 60 (17.5 - 5.925/2)$

$= 2756.78$

$\phi M_{n_1} = 2481.10$ k-in. $< M_u$.

$a = \dfrac{A_s f_y}{0.85 f_c' b}$

$= \dfrac{3.16 \times 60}{0.85 \times 4 \times 10}$

$= 5.925$

$\therefore$ Remaining moment $= (2533 - 2481.1) = 51.89$ k-in

$f_s' = E_s \times 0.003 \times \dfrac{(c - d')}{c}$

$= \dfrac{29000 \times 0.003 \times (6.97 - 2.5)}{6.97}$

$= 55.79$ ksi.

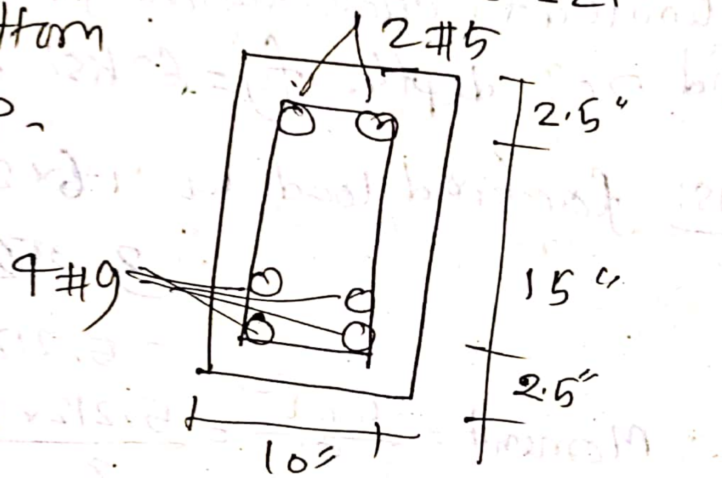
$c = a/\beta_1$

$= 6.97"$

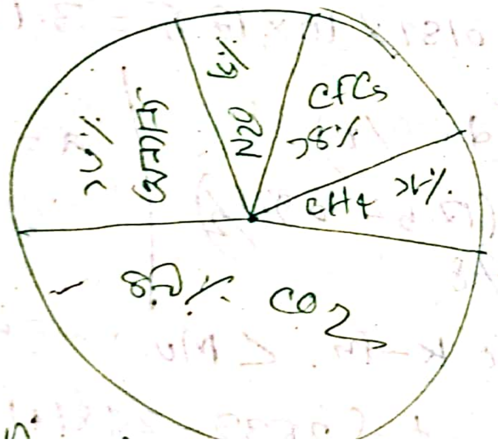
$\therefore$ $A_s' = \dfrac{M_{n2}}{\phi f_y (d - d')} = \dfrac{51.89}{0.9 \times 55.79 (17.5 - 2.5)} = 0.068$ in^2

provide total tensile reinforcement = 3.16 + 0.06
= 3.22 in^2

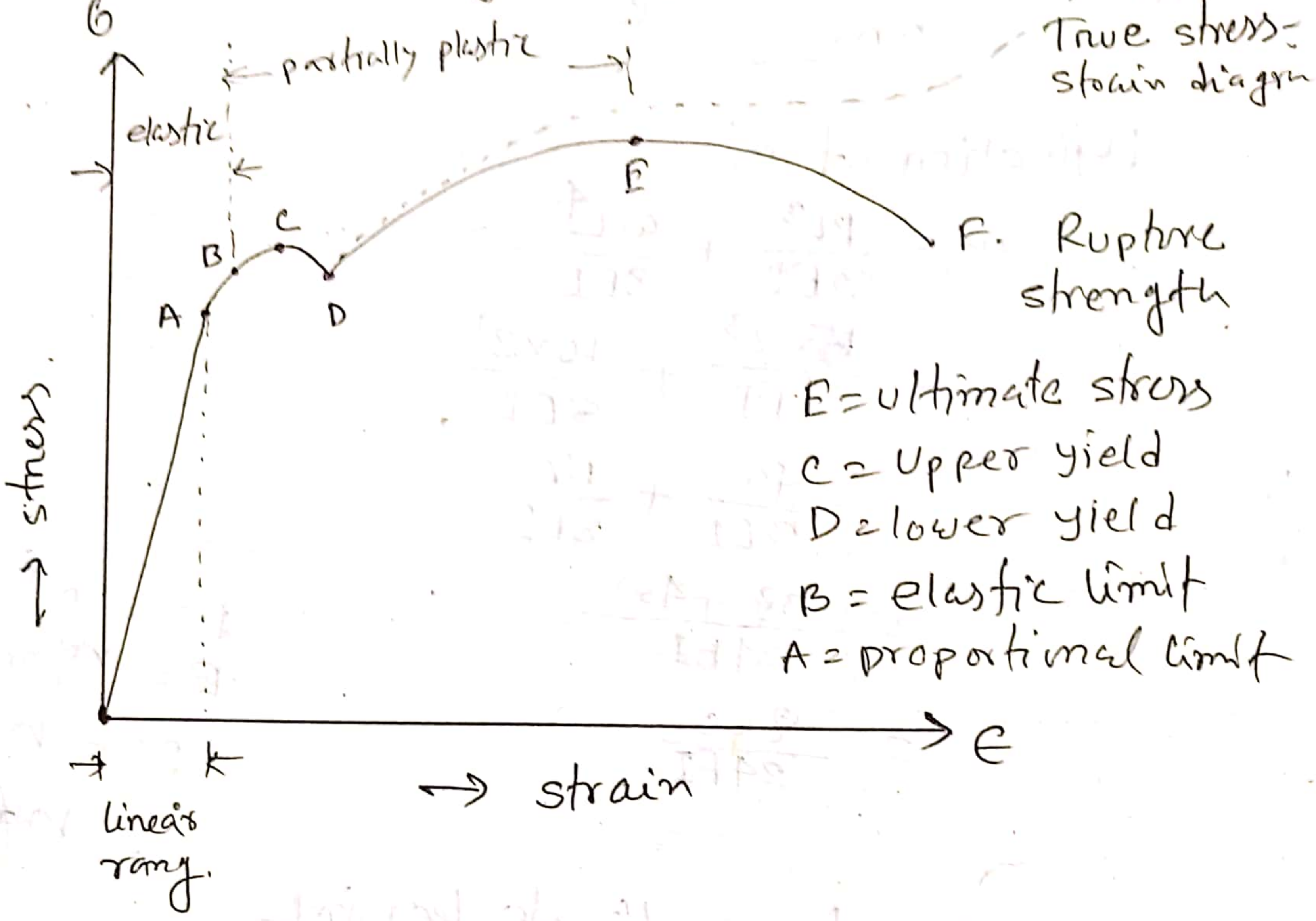
4 #9 at bottom
2 #5 at top.



প্র বায়ুমন্ডলে গ্রীন-হাউজ গ্যাস



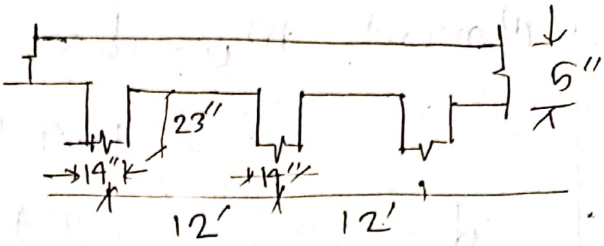
4 Draw a stress-strain digram of MS steel

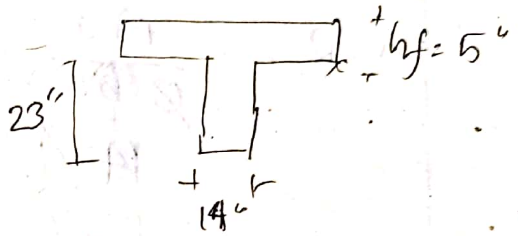


E = ultimate stress
C = Upper yield
D = lower yield
B = elastic limit
A = proportional limit

Design a T- beam.

span of Beam = 30'
$f'_c = 4$ ksi $f_y = 60$ ksi.
FF = 30 Psf. P.W = 40 psf
L.L = 90 psf: Design the beam.





Sol^n:

Effective flange width = 16 hf + bw

① be = 16×5+14 = 94"

(ii) be = L/4 = $\frac{30\times12}{4}$ = 90"

(iii) = c/c beam distance
= 12×12 = 144"

Adopt lower

∴ be = 90".

load calculation:

From slab	$\frac{5\times150}{12}$ = 62.5 psf
F.F	= 30 psf
P.W	= 40 psf
	132.5 psf.

self weight of stem = $\frac{23\times14}{144}\times150$ = 335.42 lb/ft

Total UDL from slab = 132.5×12 = 1590 lb/ft

∴ Total DL = 1590+335.42 = 1925.42 lb/ft

L.L = 90×12 = 1080 lb/ft

$\therefore W_u = 1.2 \times 1925.42 + 1.6 \times 1080 = 4.04$ k/ft

Moment . $M_u = \frac{1}{8} \times w_u \times L^2 = \frac{1}{8} \times 4.04 \times 30^2$

$= 454.5$ k-ft.

$d = 28'' - 4'' = 24''$

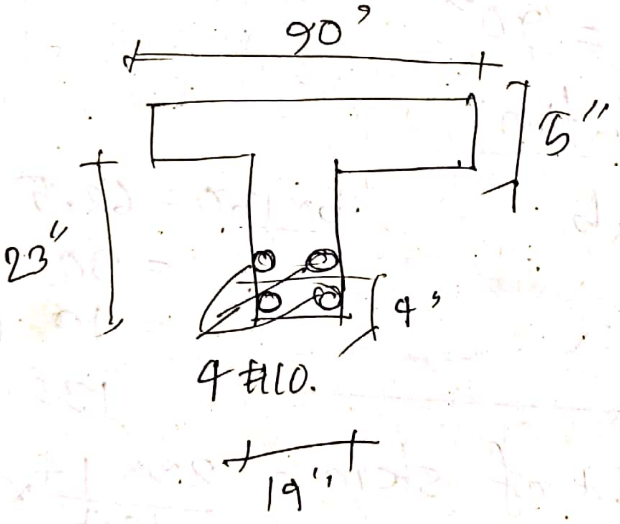
$$\therefore R = \frac{M_u}{\phi \, b \, d^2} = \frac{454.5 \times 12}{0.90 \times \cancel{90} \times 24^2} = \cancel{0.116898} = 0.877$$

~~14~~??/12" b=12 (14)

$$\rho = \frac{0.85 f_c}{f_y}\left(1 - \sqrt{1 - \frac{2R}{0.85 f_c}}\right) = 0.0172$$

$\therefore A_s = \rho b d = 0.0172 \times 14 \times 24 = 4.95$ in²

$(A_s)_{provided}$. 4 #10 $A_s = 5.08$ in²



$\boxed{4}$ Write down the atterberg's limit test. Define the following terms.
i) Liquid limit
ii) Plastic limit.
iii) shrinkage limit.
iv) Plasticity index.
v) STP.

What are ther relationship with these properties to bearing capacity of soil.

Ans: There are three tests. for Atterberg's limits. These are

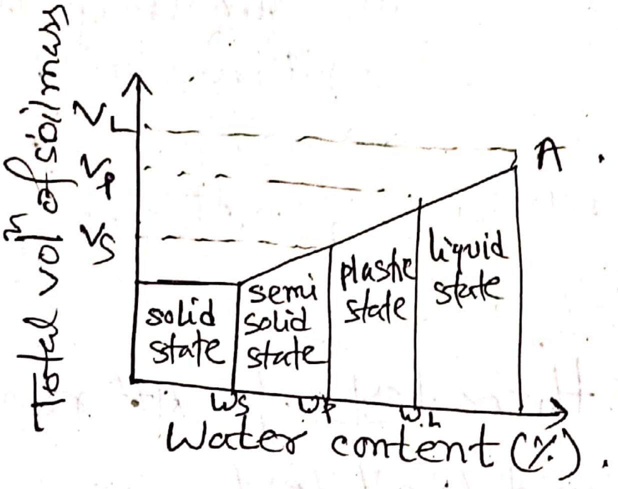
i) Liquid limit test.
ii) Plastic limit test
iii) Shrinkage limit test.

Liquid limid: It is defined as the minimum water content at which the soil is sufficiently fluid to flow a specified amount when jarred with a standard apparatus by 25 times

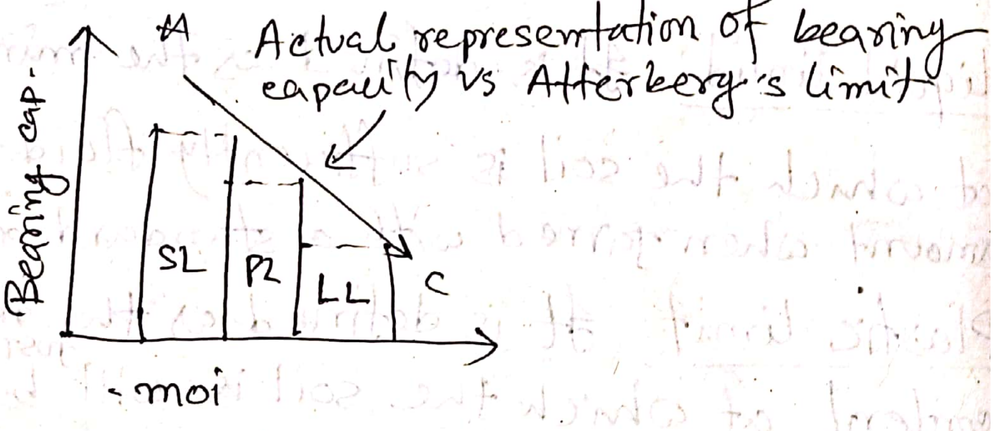
Plastic limit: It is defined as the minimum water content at which the soil ~~is~~ will just begin to crumble when rolled into a thread approximately 3mm dia.

Shrinkage limit (w_s). Shrinkage limit is defined as the max water content at which a reduction in water content doesnot cause a reduction in the volume of soil. It is the lowest water content at which a soil can still be completely saturated.

Plasticity Index (I_P): The plasticity index is defined as the numerical difference between the liquid limit and plastic limit of a soil. $I_P = \omega_L - \omega_P$.



Relationship bet^n Atterberg's limit and bearing capacity of soil.



Actual representation of bearing capacity vs Atterberg's limit

⧉ SPT: It stands for standard penetration Test. It is a test conducted to determine the bearing capacity of soil on field. It is aretually an indirect test for determining soil bearing capacity based on N- value.

⊞ Write down shorts on
- i) Spread footing
- ii) Combined footing
- iii) pile foundation.
- iv) Raft foundation
- v) Grilage foundation
- vi) Cofferdam
- vii) Caisson
- viii) friction pile.

Spread footing: A spread footing is a type of shallow foundation used to transmit the loads of ~~the~~ an isolated column or that of a wall to the soil. The base of the column/wall is spread over the soil to support the loads.

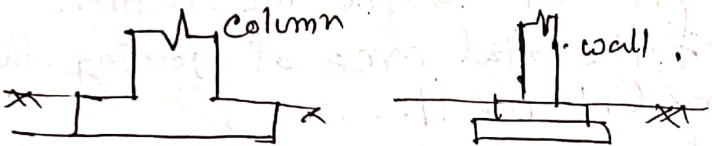


fig. Typical spread footing

Grillage foundation: Grillage foundation is used when heavy loads from columns, piers, are required to be transmitted to a soil of low bearing capacity. It provides necessary area at the base ~~of~~ to reduce the intensity of pressure within safe bearing capacity of soil. Based on materials. Grillage foundations are two types
- i) . Steel grillage foundation.
- ii) Timber grillage foundation.

Combined footing: A spread footing which suppo two or more columns is termed as a combined footi They are used when two columns are so close that single footings can't be used.

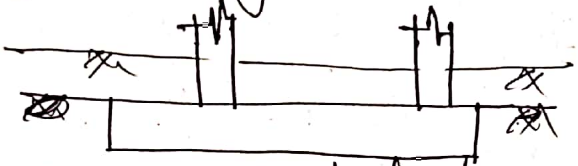
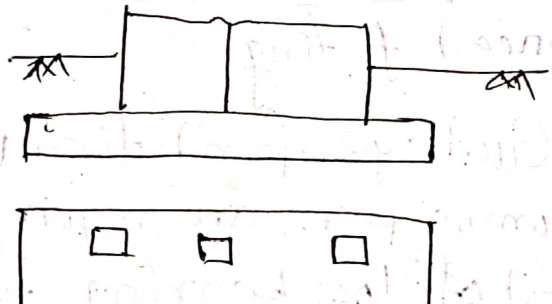


Fig: Combined footing

Raft footing: It consists of thick reinforced concr slab covering the entire area of the foundation They are used when soil bearing capacity is low, columns loads are high, single footing can't be used. When the area of the footing is more than the half of the total area of footing, then it is more economical to use it.



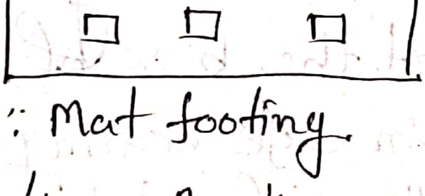


Fig: Mat footing

Pile foundation: A pile is a long vertical load transfer member composed of either steel, timber or concrete. In pile foundation a number piles are driven in the base of the structure. Pile foundation is generally used when soil is compressible, water-logged and made-up type. Types of piles are.

i) Compaction piles, ii) Tension iii) Anchor pile
iii) fender pile/ dolphines v) batter pile vi) Sheet pile.

4. Caissons foundation: A caisson foundation is a watertight retaining structure used as a bridge pier. In the construction of concrete dam, or for the repair of ships.

- It is a prefabricated hollow box, or cylinder sunk into the ground to some desired depth and then filled with concrete. thus formin a findation.
- It is ~~most often~~ widely used in construction of bridge piers beneath rivers and other water bodies.
- This is because caissons can be floated to the job site and sink into place.

Types of caissons are:

i) Box caissons, ii) Open caissons, iii) Floating caissons,
iv) excavated caissons v) Pneumatic caissons,
vi) sheeted caissons.

5. Cofferdams: A cofferdam is a temporary structure designed to keep water and or soil out of the excavation in which a bridge pier or other structure is built. When construction must take place below the water level a cofferdam is built to give workers a dry work environment.

→ Sheet piles are driven around the work site and seal concrete is placed into the bottom to prevent wat from segage underneath the sheet piles and water is pumped out.

Types of cofferdam:

i) Earth type cofferdam ii) Timber crib cofferdam
iii) Double-walled sheet pile.
iv) Cellular cofferdam.
v) Braced cofferdam.

Difference between Cofferdam and Caisson:

→ A cofferdam is removed after the structure is completed. while a caisson remains in place and forms an integral part of the structure.

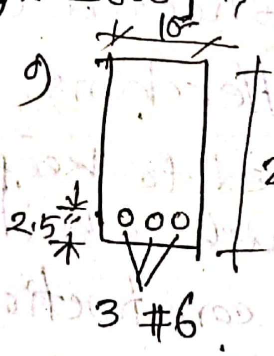
→ During the construction period the caisson functions as a cofferdam.

3. Calculate cracking moment and ultimate moment of the beam. $f'_c = 3$ ksi, $f_s = 20000$ psi, $n = 9$

Ans: Given, $f_y = 50$ ksi $f'_c = 3$ ksi, $n = 9$

$$r = \frac{f_s}{f_c} = \frac{20000}{0.45 \times 3000} = 14.81$$

$$\therefore k = \frac{n}{n+r} = \frac{9}{9+14.81} = 0.38$$



$\therefore j = 1 - k/3 = 0.87$

$As = 1.32 in^2$

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

$= \frac{1}{2} \times 0.45 \times 3 \times 0.38 \times 0.87 \times 10 \times 17.5^2$

$= 56.95$ k-ft.

Moment resisted by steel, $M_s = A_s f_s j d$

$= 1.32 \times 20 \times 0.87 \times 17.5$

$= 33.49$ k-ft. < 56.95 k-ft

$\therefore$ cracking moment, $= 33.49$ k-ft.

Ultimate/Allowable moment, $M_U = \phi M_s$

$= 0.9 \times 33.49$

$= 30.15$ k-ft. (Ans)

$\boxed{4}$ Design a square footing for moment and shear to support a RCC column of 20"x20" Given DL = 350 kips LL = 250 kips Allowable soil pressure $q_{all} = 3 ksf$. $f'_c = 3 ksi$ $f_y = 60 ksi$, $f_s = 24 ksi$. use USD.

Ans: $W_U = 1.2 \times 350 + 1.6 \times 250 = 820$ kips.

Footing Area $= \frac{load}{q_{net}} = \frac{350 + 250}{3} = 200 ft^2$.

$= 14.14^2 \simeq 14.5 \times 14.5 ft^2$

4-1 Define Bearing capacity of soil:
It is defined as the ability of sub-soil to resist the loads of the structures without yielding or displacement. It is als defined as the max. load per unit area that a soil can resist safely.

Techniques of developing Bearing capacity.

- i) Vibrofloatation.
- ii) Dynamic compaction.
- iii) Stone column.
- iv) Increasing the depth of foundation.
- v) Driving sand pilling.
- vi) Drawing sub-soil water
- vii) Ramming granular materials.
- viii) Injecting groutings.

4-1 Define the following terms:

- i) Prismatic member.
- ii) Stiffness factor.
- iii) Modified stiffness factor.

Prismatic member: A member whose geometric properties (I) and material properties are constant through/across its total span is called prismatic member. i.e. EI is constant

Stiffness factor: It is a measure of the stiffness of a structural member. For a prismatic member the stiffness is equal to $\frac{EI}{L}$. | E = Modulus of elasticity
I = Moment of inertia

Modified stiffness factor:

Eeulers proves that stiffness factor will be changed by the following way.

→ for end hinged / pinned / Roller support:

$K' = \frac{3}{4}K$

→ For symmetrical beam loading also symmetrical

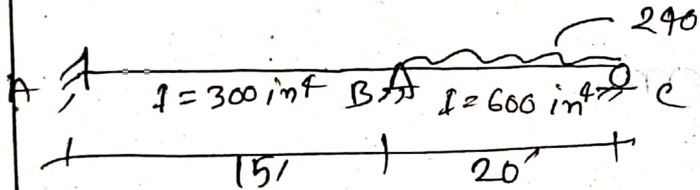
$K' = \frac{1}{2}K.$

→ for Symmetric beam and anti-symmetric loading

$K' = \frac{3}{2}K.$

These stiffness factors are modified stiffness factor.

☐ Draw SFD and BMD of the following Beam use. moment distribution method.



	0	0.47	0.53	1
FEM			+8000	+8000
D.M.				-8000
		+3760	+4240	
			-4000	
C.OM	1880			
D.m		+1880	+2120	
C.O.M	+940			
	+2820	+5640	-5640	0

$K_{AB} = \frac{EI}{L} = \frac{\overset{20}{\cancel{300}} \times E}{\cancel{15}} = 20E$

$K_{BC} = \frac{\overset{30}{\cancel{600}} E}{\cancel{20}} = 30E$

$\therefore K'_{AB} = 20$

$K'_{BC} = \frac{3}{4} \times 30 = 22.5$

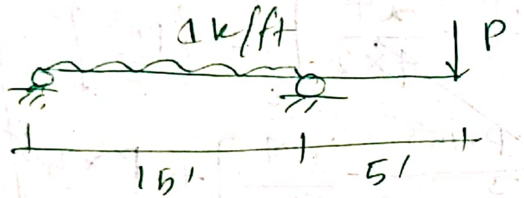
$DF_{BA} = \frac{20}{20 + 22.5} = 0.47$

$DF_{BC} = 0.53.$

***

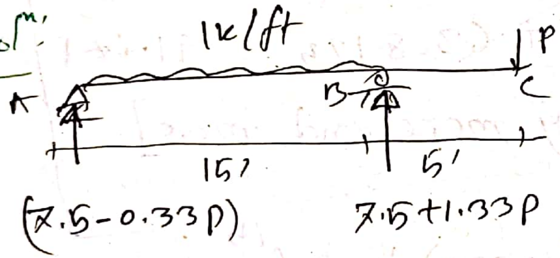
☐ Desirable properties of aggregates:
i) strength ii) Hardness iii) toughness.
iv) Durability v) shape of aggregate vi) Adhesion with bitumen.

☐ Determine the value of p. for which the deflection under p will be zero.



$\overset{+ve}{\curvearrowright} M_A = 0;$

Soln:

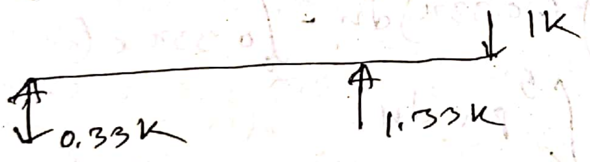


$15 \times 7.5 \times 1 + P \times 20 = R_B \times 15$

$\Rightarrow R_B = (7.5 + 1.33p)$ kip.

$R_A = -7.5 + 1.33p + P + 15$

$= (7.5 - 0.33P)$ kips



member	AB	BC
origin	A	B
limit	0–15	0–5
M	$(7.5-0.33p)x - x^2/2$	px
m	$-0.33x$	x

Now.

$$EI\,\Delta = \int_0^{15} \left\{(7.5 - 0.33P)x - x^2/2\right\} \times (-0.33x)\, dx + \int_0^5 Px \times x\, dx$$

$$= \int_0^{15} \left(7.5x - 0.33\,px - x^2/2\right) dx \times (-0.33x) + \int_0^5 px^2\, dx$$

$$= \int_0^{15} -0.33\left(7.5x^2 - 0.33\,px^2 - x^3/2\right) dx + \int_0^5 px^2\, dx$$

~~$= -0.33\left[\frac{7.5x^3}{3} - \frac{0.33\,px^3}{3} - \frac{x^4}{4\times 2}\right]_0^{15} + \left[\frac{px^3}{3}\right]_0^5$~~

~~$= -0.33\left[\frac{7.5\times 15^3}{3} - \frac{0.33\,P\times 15^3}{3} - \frac{15^4}{8}\right] + \frac{P\times 5^3}{3}$~~

~~$= -2812.5 + 375P + 6328.125 + 41.67P$~~

~~$\therefore P = 4.24k$~~ (try more and more]

$$= \int_0^{15} \left(7.5x - x^2/2\right)(-0.33x)\, dx - \int_0^{15} 0.33x \times (-0.33x)\, dx + \int_0^5 px\, dx$$

$\Rightarrow 0 = -696.1 + 122.51P + 41.67P.$

$\therefore P = 4.24\,k$ Ans

$\boxed{4}$ Design a drainage channel for non-Alluvium soil. Given $Q = 10\ m^3/s$. $V = 1\ m/s$, side slope 0.5:1 bed slope 1:5000 and $n = 0.0225$.

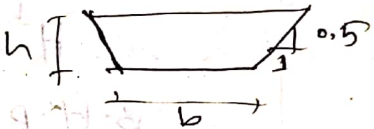
Given,

$Q = 10\ m^3/s$

$V = 1\ m/s$.

$s : 1 = 0.5 : 1$

$S = \frac{1}{5000}$



∴ Area, $A = \frac{Q}{V} = \frac{10}{1} = 10\ m^2$

Again, $A = (b+sh)h = (b+0.5h)h$

$= bh + 0.5h^2$.

∴ $bh + 0.5h^2 = 10$. -- (i)

$P = b + 2h\sqrt{1+s^2}$

$= b + 2h \times \sqrt{1+0.5^2} = b + 2.24h$.

Now, $V = \frac{1}{n} R^{2/3} S^{1/2} = \frac{1}{0.0225} \times \left(\frac{A}{P}\right)^{2/3} S^{1/2}$

$\Rightarrow 1 = \frac{1}{0.0225} \times \left(\frac{10}{b+2.24h}\right)^{2/3} \times \left(\frac{1}{5000}\right)^{1/2}$

$\Rightarrow (b+2.24h)^{2/3} = \cancel{202.95}\ 2.92$

$\Rightarrow b + 2.24h = 4.98$

$\Rightarrow b = 4.98 - 2.24h$ -- ⓘⓘ

from ⓘ and ⓘⓘ

$(4.98 - 2.24h)h + 0.5h^2 = 10$

$\Rightarrow 4.98h - 2.24h^2 + 0.5h^2 - 10 = 0$;

$\Rightarrow 1.74h^2 - 4.98h + 10 = 0$;

∴ $h = 1.431\ m$

∴ $b = 4.98 - 2.24 \times 1.431 = 1.77\ m$

⊞ Design a pump on a well which is set up to irrigate rice crop over 3.5 hac. If duty is 850h/cm pump efficiency = 50%. The lowest water level at well is 9m. Minimum required B.H.P = ?

Ans: $Q = \frac{A}{D} = \frac{3.5}{850}$ m3/s.

$= ~~3.5 \times 1000~~$

$= 4.117 \times 10^{-3}$ m/s.

$\eta = 50\%$

$H = 9m$

$\therefore B.H.P = \frac{wQH}{75\eta} = \frac{1000 \times 9 \times 4.117 \times 10^{-3}}{75 \times 0.50}$

$= 0.988 \approx 1$ BHP. (Ans)

Given

A = 3.5 ha.

D = 850 h/cm

⊞ Define equilibrium can't.

Find amount of equilibrium cant. Given design Speed 50kmh for BG. of 5°. track.

Ans: for Broad gauge.

$S.E = 1.315 \frac{V^2}{R}$

$= 1.315 \times \frac{50^2 \times 5}{1719}$

$= 9.56$ cm. A

$R = \frac{1719}{D^0}$

$= \frac{1719}{5} =$

For. M.G

$S.E = 0.8 \frac{V^2}{R}$

For. N.G

$S.E = 0.6 \frac{V^2}{R}$

V = kmh

R = m

S.E = cm

Find out the number of sleepers. for rail of 12.80m length with sleepers. as (n+3).

Ans! Sleepers = n+3

= 12.80+3

= 15.80

≈16. sleepers. (Ans)

A 6 degree branches of 3 degree main track in an opposite direction in the layout of broad gauge. If the speed of branch line is restricted by 35 kmh. Determine the speed restriction on the main line. Assume cant diffiiciency 75mm.

Ans: S.E. for branch line, $S.E = 1.315 \frac{V^2}{R} = 1.315 \times \frac{35^2}{\frac{1719}{5}}$

$= 45.62$ cm

Negative super elevation = (5.62 − 7.5) = 1.88 cm

Max S.E. can be given on main line = 1.88 cm

∴ Therefore S.E. on main line = 1.88 + 7.5 = 9.38 cm

Now. $S.E = 1.315 \frac{V^2}{R} \Rightarrow 9.38 = 1.315 \times \frac{V^2}{\frac{\cancel{1912}\ 1719}{3}}$

$\Rightarrow V = 63.93$ kmh. Ans

$\boxed{4}$ What are the environmental significance of BOD and COD.

BOD ⎫ বেশি হলে, বেশি contaminated, unsuitable to irrigat
BOD ⎭ harmful for aquatic life, non-palatable.

Mention with reasons the type of treatment syste you would prefer for the following industry:

waste water parameter	Industry-I	Industry-II
pH	8.0	12.0
BOD (mg/l)	2500	250
COD (mg/l)	3000	5000

Ans: For industry-I

$\frac{BOD}{BOD} = \frac{3000}{2500} = 1.2 < 2$; So the materials in Ind. is biodegradable type. So management of bio deg organic is usually done by the BOD test.

For Industry-II

$\frac{COD}{BOD} = \frac{5000}{250} = 200 \gg BOD$.

So management non-bio-degradable is usually done by COD test.

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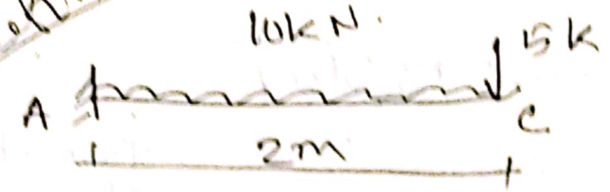
Deflection



Deflection at c

$$\Delta_c = \frac{PL^3}{3EI} + \frac{\omega L^4}{8EI}$$

$$= \frac{\cancel{15}\times 2^3}{3EI} + \frac{10\times 2^4}{8EI}$$

$$= \frac{40}{3EI} + \frac{160}{8EI}$$

$$= \frac{320+480}{24EI}$$

$$= \frac{800}{24EI}$$

$I = m^4$

$E = kN/m^2$

$\therefore EI = kN/m^2 \times m^4$

$= kNm^2$

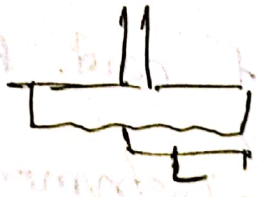
Mat foundation, ultimate bearing capacity

$$q_{ult} = 5.14\, c_u \times \left(1 + 0.195\frac{B}{L}\right)\left(1 + 0.4\frac{D}{B}\right)$$

$$M_u = \frac{q_u * L^2}{2}$$

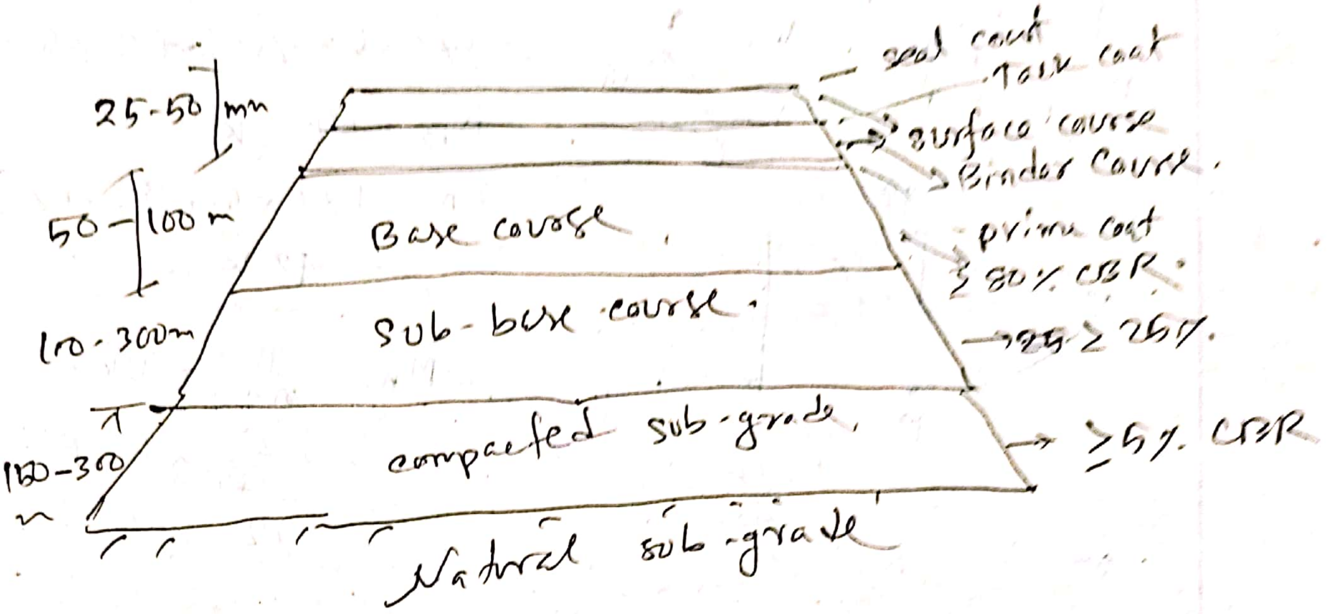
$$R = \frac{M_u}{\phi b d^2}$$

$$\rho = \frac{0.85 f'c}{f_y}\left(1 - \sqrt{1 - \frac{2R}{0.85 f'c}}\right)$$



$L = \left(\frac{b-a}{2}\right)\times 1$

13 Draw a x-section of flexible pavement



14 CBR test.

$$CBR(\%) = \frac{0.1'' \text{ penetration load}}{100} = \frac{2.5\text{mm required load}}{1370}$$

$$CBR\% = \frac{0.2'' \text{ penetration load}}{1500} = \frac{5\text{mm penetration load}}{2055}$$

15 Shear strength of soil

$$S_u = \frac{T}{\pi D^2 H/2 + \pi D^3/6}$$

***

⑩ Define pavement: Write down the function of base, sub-base, and surface course.

Pavement: It is the surface which seperates the tires of vehicles from the underlying foundation materials.

→ It should be strong enough to resist the stresses imposed on it and it should be thick enough to distribute external loads on the earthen sub-grade.

Function of surface course:

1. Perform as a structural portion of the pavement
2. Resist the abrasive forces of traffic.
3. Reduce the penetration of surface water throug pavement
4. provide skid resistance.
5. Provide smooth and uniform riding surface
6. Transmit loads from the vehicles to the base course.

Function of Bas course:

1. Transmits loads from the surface to sub-bas
2. Prevent intrusion of fine grained materials into the pavement.

Function of sub-base:

2. Distributes the loads from base to the natural sub-grade
2. Prevent intrusion of fine-grained materials.
3. Minimize the damaging effects of frost actio
4. Facilitate drainage of free water that might get accumulated below the pavement.

Factors affecting pavement design.

1. Design life
2. Reliability
3. Traffic factors
   - Types of traffics,
   - Number of
   - Wheel loads
4. Climate factors
   - Rainfull,
   - Temperature.
5. Road geometry
   - horizontal curve
   - vertical curve
   - super elevation
6. Sub-grade strength & dra
7. Material properties for structural design.

$\boxed{4}$ Losses of pre-stress.

The initial prestress in concrete undergoes a gradual reduction with time from the stage of transfer due to various causes. This is known as losses of prestress. A reasonably good estimate of the magnitude of loss of pre-stress is necessary from the point of view of design.

Types of losses of pre-stresses:

Pre-tensioning	Post-tensioning
1. Elastic deformation of concrete.	1. No loss due to elastic deforma if all the wires are simultaneous tensioned. If the wires are successively tensioned, there will be loss of prestress due to elastic deformation of [illegible]
2. Shrinkage of concrete.	2. Relaxation of stress in steel.
3. Creep of concrete	3. Creep of concrete
4. Relaxation of stress in steel.	4. Shrinkage of concrete
	5. Friction
	6 Anchorage slip,

Beam design:

Problem:1: Calculate the nominal strength of the beam.

$b = 10\text{ in}$ $d = 23\text{ in}$ $A_s = 2.37\text{ in}^2$. $f'_c = 4\text{ ksi}$ $f_y = 60\text{ ksi}$ and $\beta_1 = 0.85$

Solution: $\rho = \frac{A_s}{bd} = \frac{2.37}{10\times23} = 0.0103\text{ in}^{-1}$

$$\rho_b = 0.85\beta_1\frac{f'_c}{f_y} * \frac{\epsilon_u}{\epsilon_u+\epsilon_y}$$

$$= 0.85\times0.85\times\frac{4}{60}\times\frac{0.03}{0.03+60/29\times10^3}$$

$$= 0.028 > \rho$$

$\therefore$ failure by yielding of steel.

$$\therefore M_n = \rho f_y bd^2\left(1-0.59\,\rho\frac{f_y}{f'_c}\right)$$

$$= 0.0103\times60\times10\times23^2\left(1-0.59\times\frac{0.0103\times60}{4}\right)$$

$= 2972.21$ kip-in

$= 247.60$ kip-ft.

≈ 248 kip-ft

অথবা

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

$$= 2.37\times60\left(23-\frac{\overset{4.18}{\cancel{3.486}}}{2}\right)$$

$= 2973.23$ kip-in.

$= 248$ kip-ft.

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

$$= \frac{2.37 * 60}{0.85\times4\times10}$$

$= \cancel{3.486}$.

$= 4.18$

Design moment:

$\emptyset M_n = 0.90\times248$ kip-ft

$= 223$ k-ft.

$\emptyset = 0.90$ হবে কিনা তা চেক করতে হবে,

$$\frac{c}{d_t} = \frac{4.81/0.85}{23}$$

$= 0.215 < 0.375$

i.e $\epsilon_t > 0.005$

$\therefore \emptyset = 0.90$

Problem: 2: Calculate the nominal flexural strength and what is the maximum design moment According to ACI code. of a beam b = 12 in d = 17.5 in 4 #9 bars f_y = 60 ksi f'_c = 4 ksi.

$$M_n = A_s f_y \left(d - a/2\right)$$
$$= 4 \times 60 \left(17.5 - \frac{5.88}{2}\right)$$
$$= 3494.5 \text{ kip-in}$$
$$= 291.2 \text{ k-ft.}$$

$$a = \frac{A_s f_y}{0.85 f'_c b}$$
$$= \frac{4 \times 1 \times 60}{0.85 \times 4 \times 12}$$
$$= 5.88''$$

$$c = \frac{a}{\beta_1} = \frac{5.58}{0.85}$$
$$= 6.92''$$

$$\frac{c}{d_t} = \frac{6.92}{17.5} =$$
$$= 0.395 > 0.375$$

Transition zone.

$$\therefore \boxed{\phi = 0.65 + 0.25\left(\frac{1}{c/d_t} - 5/3\right)}$$
$$= 0.65 + 0.25\left(\frac{1}{0.395} - 5/3\right)$$
$$= 0.87$$

∴ Design moment $\phi M_n = 0.87 \times 291.2$ k-ft

$= 253.344$ kip-ft.

Problem-3: ~~Concrete~~ Compute the concrete cross-section and steel area required for a simply supported beam of 15′ span.

$DL = 1.27$ kips/ft $L.L = 2.15$ kip/ft $f_c' = 4$ ksi $f_y = 60$ ksi.

Solution: $W_U = 1.2 \times 1.27 + 1.6 \times 2.15 = 4.96$ kip/ft

$$M_U = \frac{W_U L^2}{8} = \frac{4.96 \times 15^2}{8} = 139.5 \text{ kip-ft} = 1674 \text{ k-in.}$$

To maintain $\phi = 0.90$, The maximum R.F ratio corresponding to a net tensile strain of 0.005. will be selected.

$$\therefore \rho_{0.005} = 0.85 \beta_1 \times \frac{f_c}{f_y} * \frac{\epsilon_U}{\epsilon_U + 0.005} = 0.85 \times 0.85 \times \frac{4}{60} \times \frac{0.003}{0.003 + 0.005}$$

$$= 0.0181$$

$$\rho_{max} = 0.85 \times 0.85 \times \frac{4}{60} \times \frac{0.003}{0.003 + 0.004} = 0.0206 > \rho_{0.005} \quad \text{(ok}$$

Now. $M_U = \phi M_n$

Assume $b = 10''$

$$\Rightarrow 1674 = 0.9 \times \rho b d^2 f_y \left(1 - 0.59 \frac{\rho f_y}{f_c}\right)$$

$$\Rightarrow 1674 = 0.9 \times 0.0181 \times 10 \times d^2 \times 60 \left(1 - \frac{0.59 \times 0.0181 \times 60}{4}\right)$$

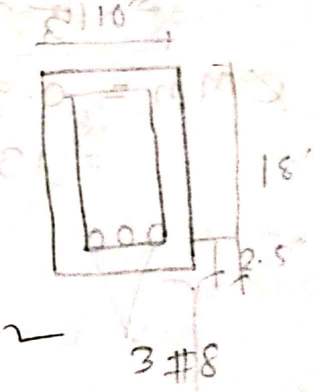
$$\Rightarrow d = 14.28''$$

Assume clear cover $= 2.5''$

$$\therefore h = 14.28 + 2.5 = 16.78$$

Provide $h = 18''$ and $b = 10''$

$$\therefore A_s = \rho b d = 0.0181 \times 15.5 \times 10 = 2.8055 \text{ in}^2$$



Use 3 #8 bar $\therefore A_s = 2.37$ in^2.

Example-4: b=10" d=17.5" h=20" f'_c=4 ksi f_y= 60 ksi. compute steel area. to resist a moment M_U = 1300 k-in.

Solution:

$$A_s = \frac{M_U}{\phi f_y (d - a/2)}$$

Assume a = 2.5"

$$= \frac{1300}{0.9 \times 60\,(17.5 - 1.25)}$$

$$= 1.482 \text{ in}^2$$

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

$$= 2.614"$$

Again
$$A_s = \frac{1300}{0.9 \times 60\left(17.5 - \frac{2.614}{2}\right)}$$

$$= 1.486 \text{ in}^2$$

a = 2.62"

$\therefore$ A_s = 1.486 in^2.

Use 2 #8 A_s = 1.58 in^2.

Check:

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

$$= 1.58 \times 60\left(17.5 - \frac{2.62}{2}\right)$$

$$= 1534.812 \text{ k.in.}$$

$$c = \frac{a}{\beta_1} = 3.08$$

$$\frac{c}{d_t} = \frac{3.08}{17.5} = 0.176 < 0.375.$$

$\therefore \phi = 0.90.$

$$\phi M_n = 0.9 \times 1534.812$$

$$= 1381.33 \text{ k.in} > 1300 \text{ k.in.} \quad \text{(OK)}$$

Example-5: Architectural considerations limits is h= 16 in b = 12 in span L=20 ft. Wd = 0.79 k/f wL = 1.65 k/ft f'_c = 5 ksi fy = 60 ksi. Determine the reinforcement for the beam.

Solution: $W_u = 1.2 \times 0.79 + 1.6 \times 1.65 = 3.59$ k/ft

$$M_u = \frac{W_u L^2}{8} = \frac{3.59 \times 20^2}{8} = 179.5 \text{ k-ft} = 2154 \text{ k-in.}$$

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

$$= \frac{2154}{0.90 \times 60 (13.5 - 1.5)}$$

$$= 3.324 \text{ in}^2$$

$$A_s = \frac{2154}{0.90 \times 60 (13.5 - \frac{3.91}{2})}$$

$$= 3.448 \text{ in}^2$$

$$A_s = \frac{2154}{0.90 \times 60 \times (13.5 - \frac{4.05}{2})}$$

$$A_s = 3.477 \text{ in}^2$$

$$\therefore A_s = 3.477 \text{ in}^2$$

Use 3 #10 $A_s = 3.81$ in²

Check Moment:

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

$$= 0.89 \times 3.81 \times 60 (13.5 - \frac{4.09}{2})$$

$$= 2330.56 \text{ kip-in} > 2154.$$

(Ok)

Assume $a = 3''$

$\phi = 0.90$

$d = 16 - 2.5 = 13.5''$

$$a = \frac{3.324 \times 60}{0.85 \times 5 \times 12}$$

$a = 3.91''$

$$a = \frac{3.448 \times 60}{0.85 \times 5 \times 12}$$

$a = 4.05''$

$a = 4.09''$

$f'_c = 5$ ksi

$\therefore \beta_1 = 0.8$

check $\phi = 0.90$ or not

$$c = \frac{a}{\beta_1} = \frac{4.09}{0.8}$$

$$= 5.11$$

$$\frac{c}{d_t} = \frac{5.11}{13.5} = 0.378 > 0.375$$

$\therefore \phi = 0.9$ not ok

$$\phi = 0.65 + 0.25 \times \left(\frac{1}{c/d_1} - 5/3\right)$$

$$= 6.65 + 0.25 \times \left(\frac{1}{0.378} - 5/3\right)$$

$$= 0.89.$$

Foundation:

Example-16.1: A 16 in concrete wall support a load DL= 14 k/ LL= 10 k/ft. Allowable bearing pressure is $q_a = 4.5$ k/ft^2. footing Level 4 ft from grade. Design a footing for this wall. Also given $f_y = 60$ ksi, $f_c' = 4$ ksi.

Solution:

$$A_{req} = \frac{DL + LL}{q_e}$$

q_e = 4.5 − (surcharge load)

$= 4.5 - (0.150 \times 1 + 0.100 \times 3)$

$= 4.05$ k/ft^2

$\therefore$ Area $= \frac{10 + 14}{4.05} = 5.925$ ft^2

$\therefore$ A = 6 ft^2

Consider 1′ strip.

So width of the footing will be 6 ft.

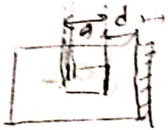
⇒ Bearing capacity due to factored load:

$$q_u = \frac{1.2 \times DL + 1.6 \times LL}{A} = \frac{1.2 \times 14 + 1.6 \times 10}{6} = 5.467 \text{ k/ft}^2$$

$= 5467$ psf.

⇒ Shear strength caused by factored load.

$$V_u = q_u * \left\{ \frac{b-a}{2} - d \right\}$$



$\left(\left(\frac{b-a}{2} \right) - d \right)$

Assume $d = 9''$

$$V_u = 5467 * \left\{\frac{6-1.33}{2} - \frac{9}{12}\right\}$$

$$= 8665 \text{ lb/ft}$$

⇒ Check Assumed d is ok or not

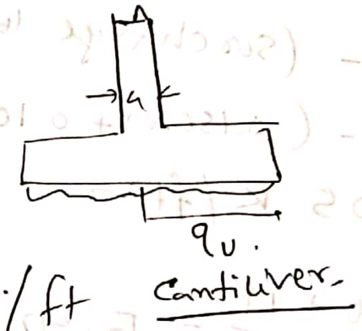
$$\emptyset V_c = 2\emptyset\sqrt{f'_c}\, bd$$

$$\Rightarrow 8665 = 2\times0.75\times\sqrt{4000}\times12\times d$$

$$\Rightarrow d = 7.6'' < 9''$$

∴ Assumed d is ok.

⇒ Moment calculation.



$$M_u = \frac{q_u L^2}{2}$$

$$= \frac{5467 * \left(\frac{6-1.33}{2}\right)^2}{2} \text{ ft-lb/ft}$$

$$= 14903.68 \text{ ft-lb}$$

$$= 178843.88 \text{ in-lb/ft}$$

$$= 178.843 \text{ in-k/ft.}$$

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

$$\Rightarrow 178.843 = 0.9\times\rho * 12\times9^2\times60\left(1 - \frac{0.5\times\rho\times60}{4}\right)$$

$$\Rightarrow 178.843 = 52488\rho(1-8.85\rho)$$

$$\rho - 8.85\rho^2 - 0.003407 = 0$$

$$\therefore \rho = 0.00351$$

$\therefore A_s = \rho bd$
$= 0.00352 \times 12 \times 9$
$= 0.38 \text{ in}^2$

Provide #5 @ $\frac{12 \times 0.3}{0.38} = 9'' \text{ c/c}$.

longitudinal bar $= 0.002bt$
$= 0.002 \times 12 \times 12 = 0.288 \text{ in}^2$

Provide #5 @ $\frac{12 \times 0.3}{0.28} \simeq 12'' \text{ c/c}$.

Example-16.2: Design a square footing. Column dimension $18 \times 18 \text{ in}^2$. $f'_c = 4$ ksi, $f_y = 60$ ksi. 8 #8 DL = 225 kips. LL = 175 kips. $q_a = 5 \text{ kips/ft}^2$. Base 5 ft below grade.

Solution:

$q_e = q_a - (\text{surcharge})$
$= 5 - (0.125 \times 5)$
$= 5 - 0.625$
$= 4.375 \text{ k/ft}^2$.

$\therefore$ Average pressure
$= \frac{150 \times 1 + 100 \times 1}{2 \times 100}$
$= 0.125$ ksf.

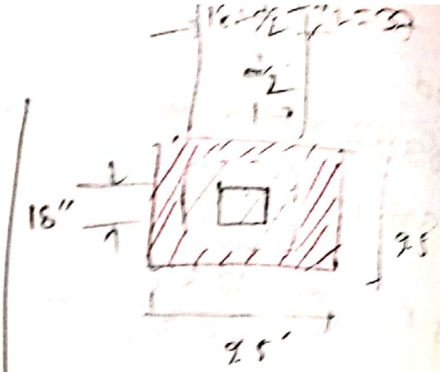
$\therefore$ Area of footing $= \frac{175 + 225}{4.375} = 91.43 \text{ ft}^2$

So, $A_{req} = 9' 6'' \times 9' 6''$

$\therefore$ Bearing pressure due to factored load.

$q_u = \frac{1.2 \times 225 + 1.6 \times 175}{9.5 \times 9.5} = 6.1 \text{ k/ft}^2$.

Punching shear check:



Assumed d=19"

$V_{u_1} = q_u * (9.5^2 - \text{Punching area})$

$= 6.1 \times \{ 9.5^2 - (\frac{37}{12})^2 \}$

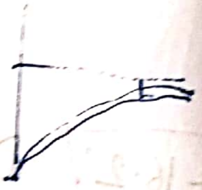
$= 493$ kips.

Concrete shear strength

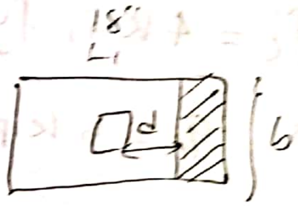
$\phi V_c = 4 \phi \sqrt{f_c'}\ b d$ → (Punching perimeter)

$= 4 \times 0.75 \times \sqrt{4000} \times (4 \times 37) * 19$ (in [illegible])

$= 533.54 > V_{u_1}$



Beam shear check:



$V_{u_2} = q_u * \{ \frac{b-a}{2} - d \} * b$

$= 6.1 * \{ \frac{9.5-1.5}{2} - \frac{19}{12} \} * 9.5$

$= 140.$ kips.

Concrete shear strength

$\phi V_c = 2 \phi \sqrt{f_c'}\ b\ d$

$= 2 \times 0.75 \times \sqrt{4000} \times 9.5 \times \frac{19}{12} \times 12$

$= 205.484$ kips. $> V_{u_2}$ (OK)

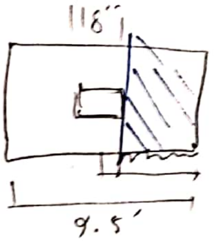
Moment calculation:

$$M_u = \frac{q_u * L^2}{2}$$

$$= \frac{6.1 * 4^2 \times 9.5}{2} \text{ ft-k}$$

$$= \frac{6.1 \times 9.5 \times 4^2}{2} \times 12 \text{ in-k}$$

$$= 5563.2 \text{ in-k.}$$



$$= \frac{b-a}{2} = 4'$$

$$q_u = 6.1 \text{ kip/ft}^2.$$

$$= 6.1 \times 9.5 \text{ kip/ft}$$

Now.

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

$$\Rightarrow 5563.2 = 0.9 * \rho * 12 * 19^2 * 60 \left(1 - 0.59 * \frac{\rho * 60}{4}\right)$$

$$\Rightarrow 5563.2 = 233928 \rho (1 - \cancel{0.89}\ 8.85\rho)$$

$$\Rightarrow \rho^2 - 8.85\rho - 0.02378 = 0;$$

$$\therefore \rho = \boxed{\cancel{0.0789}} \overset{Not}{\cancel{}} / \cancel{0.0034}$$

$$\therefore A_s = \cancel{\rho b d} = \cancel{0.0789 \times 12}$$

$$R = \frac{M_u}{\phi b d^2} = \frac{5563.2}{0.9 \times 12 \times 16^2}$$

$$= 1.926$$

$$\rho = \frac{0.85 f_c'}{f_y}\left(1 - \sqrt{1 - \frac{2R}{0.85 f_c'}}\right)$$

$$= 0.0339.$$

$$\rho = 0.0034 \qquad \therefore A_s = 0.0034 \times 12 \times 19 = 7.75 \text{ in}^2.$$

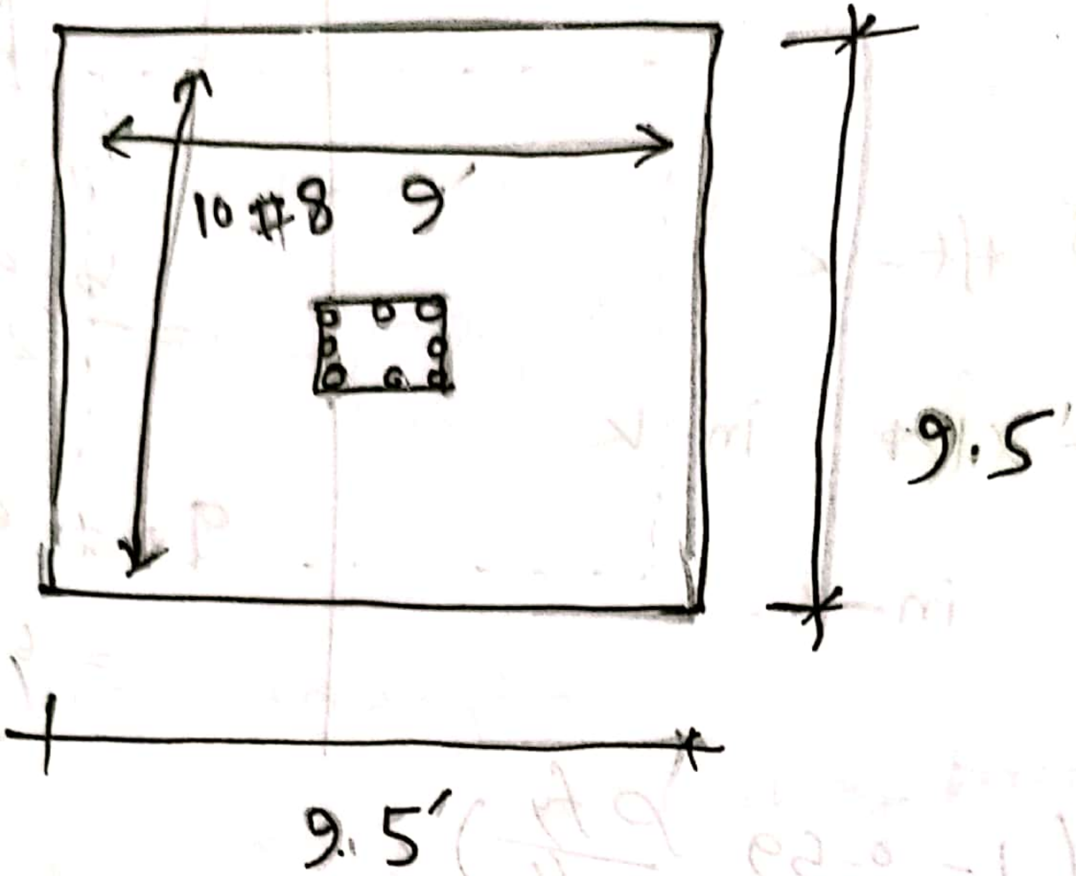
Check the minimum reinforcement ratio.

$$A_{s.min} = \frac{3\sqrt{f_c'}}{f_y} = \frac{3\sqrt{4000}}{60000} = 6.65 \text{ in}^2$$

but not less than $A_{s\,min} = \frac{200}{f_y} bd = \frac{200}{60000} \times 114 \times 19$

$$= 7.22 \text{ in}^2$$

So provide 10 # 8 bar $A_s = 7.9 \text{ in}^2$.


10 #8 9'
9.5'
9.5'

Mat foundation.

① Determine the net ultimate bearing capacity of a mat foundation measuring 15m × 10m on saturated clay with clay with $C = 95$ kN/m² $\varphi = 0$, $D_f = 2$m.

Solⁿ:

$$q_{net} = 5.14\, c_u \left[1 + \frac{0.195B}{L}\right]\left[1 + 0.4\frac{D_f}{B}\right]$$

$$= 5.14 \times 95 \left[1 + \frac{0.195 \times 10}{15}\right]\left[1 + 0.4 \times \frac{2}{10}\right]$$

= ~~69084.6~~8

= 597.67 kN/m²

② The mat has dimension of 30m × 40m. The live load and dead load on the mat are 20 MN. The mat is placed over a lair of soft clay. The unit wt of clay 18.7 kN/m3. Find the fully compensated foundation.

Solⁿ:

$$D_f = \frac{Q}{A\gamma} = \frac{20 \times 10^6}{30 \times 40 \times 18.7 \times 1000} = 0.891 \text{ m}.$$