A combined footing supports the loads of two or more adjacent column, such footing is provided under the following circumstances.

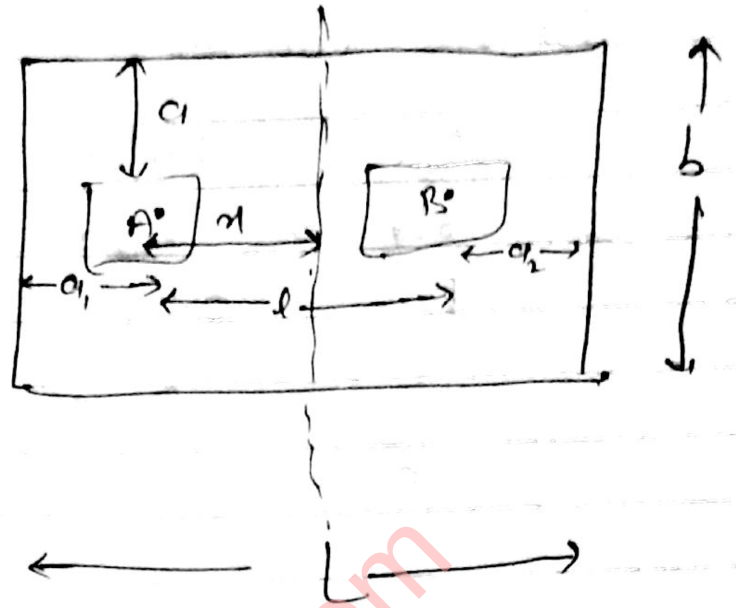
* The bearing capacity of soil is less, require more area under individual footing.

Sec-B

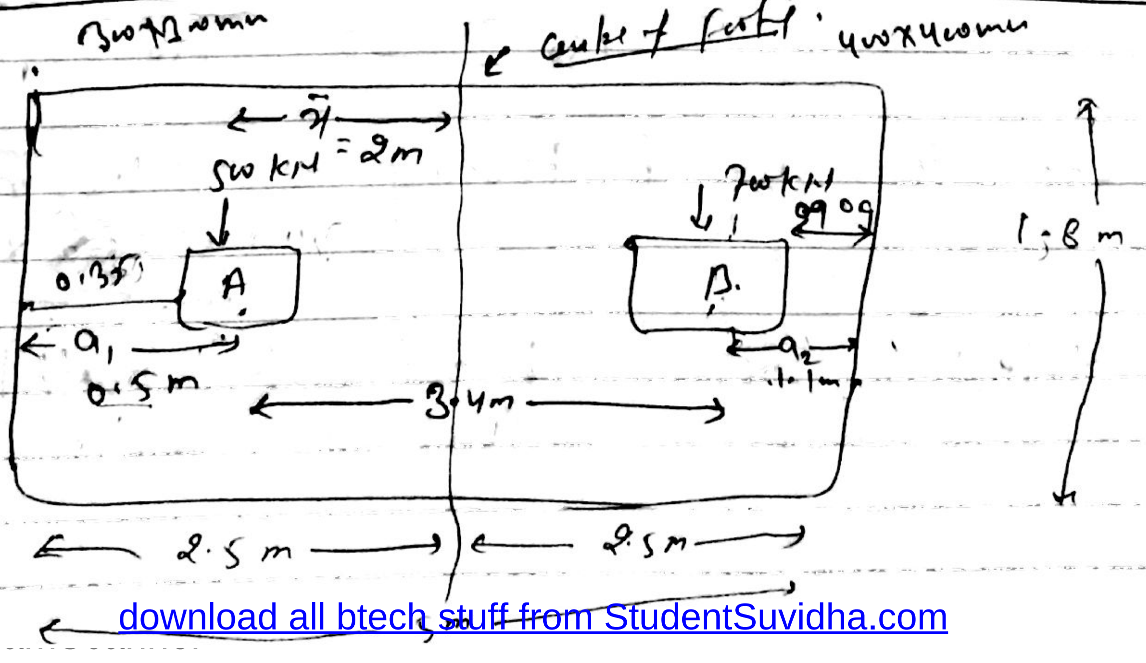
Foundations

* when the column are very near to each other so that their footing overlapes.
* when the end column is near a property line

Design of Combined footing ↓



Design Combined Rectangular footing for two columns A and B carrying loads of 500 and 700 kN. resp. Column A is 300 mm x 300 mm in size and B is 400 mm x 400 mm in size. The centre to centre spacing of the columns is 3.4 m. The safe bearing capacity of soil may be taken as 150 kN/m², use M20 and Fe415.



(restricted width means column is given)

Step – I

Design constants –

$\sigma_{cbc} = 7 \, N/mm^2$

$\sigma_{st} = 230 \, N/mm^2$

$k = 0.289$ or 0.290

$j = 0.904$

$Q = 0.914$

Safe bearing capacity $(q_o) = 150 \, N/m^2$

Step – II – Design of Footing

Area of Footing $= (A) = \frac{W_1 + W_2 + W'}{q_o}$

$W' = 10\% (W_1 + W_2)$

$= 1200 \times \frac{10}{100} = 120 \, kN$

$\therefore A = \frac{500 + 700 + 120}{150} = \frac{1320}{150} = 8.8 \, m^2$

Let the size of Footing $= 5 \times 1.8 \, m$

i.e. $5 m \times 1.8 m$

{Note :- Their (length and width) multiplication should be equal to area i.e. 8.8 m

[The distance of $\bar{x}$ of the C.G of column load from centre of column A.]

$$\bar{x} = \frac{w_2 \times l}{w_1 + w_2} = \frac{500 \times 3.4}{1200} = 2\,m$$

$$a_1 = 2.5 - \bar{x} = 2.5 - 2 = 0.5\,m.$$

(half of length)

$$a_2 = L - (l + a_1) = 5 - (3.4 + 0.5)$$

$$= 1.1\,m$$

Step - III

Net upward pressure $(P_0) = \frac{w_1 + w_2}{B \times L}$

$$= \frac{1200}{\not{5\times4}\ 1.8 \times 5} = 139\ldots$$ say 140 kN/m²

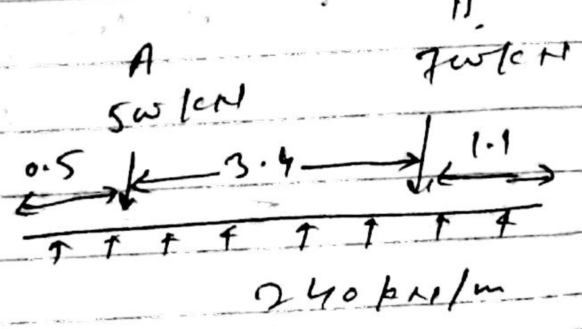
pressure (w) per meter length

$(P_0)B =$

$= P_0 \times B = 140 \times 1.8$

$= 240$ kN/m

Step - IV Design of S.F and B.M.



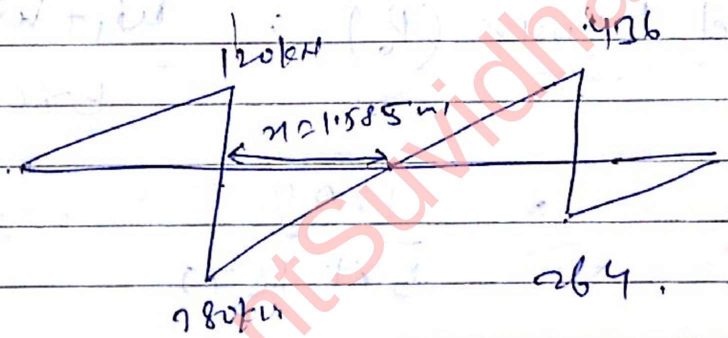
$240 \times \frac{(1.585 - 0.5)^2}{2}$

S.F. just to the left of A = ~~500~~ 240 × 0.5
= 120 kN

S.F. just to the right of A = 500 − 120
= 380 kN

S.F. just to the right of B = ~~700 ×~~ 1.1
= 240 × 1.1
= 264 kN

S.F. just to the left of B
= 700 − 264
= 436 kN.



S.F will zero at distance (x) = $\frac{380}{240}$

= 1.585 m

(m) Max. B.M = $500 \times 1.585 - 240 \times (1.585 - 0.5) \times \frac{(1.585 - 0.5)}{2}$

$= 271 \times 10^6$ Nmm

[500×500 mm]
[0.5 − 0.15 = 0.35 m]

[80 t = 800 kN]
80 t = 800 kN

Sagging Bending moment at A = $\frac{240 \times (0.5)^2}{2}$

$= 30 \times 10^6$ N mm

Sagging B.M at B = $\frac{240 \times (1.1)^2}{2}$

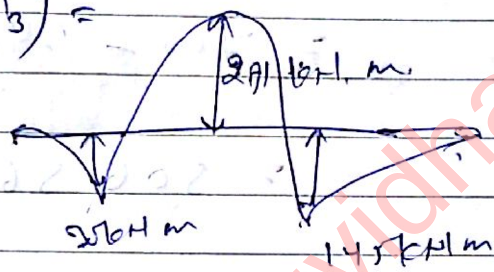
$= 145 \times 10^6$ Nm.

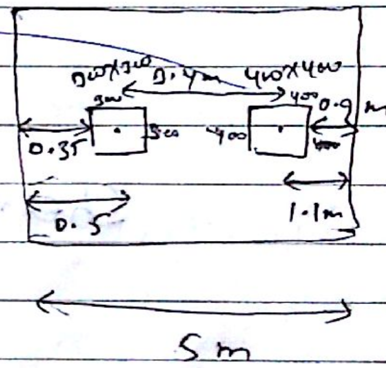
Sagging B.m at the outer face of column B is (M_2) $= \frac{240}{2} \times (0.9)^2$

$= 97 \times 10^6$ N-mm.

Sagging Bm at the outer face of column A is (M_3) = $\frac{240}{2} \times (0.35)^2$

$= 15 \times 10^6$ N-mm





Step – VI

Effective depth

$= \sqrt{\frac{M}{Q \cdot b}}$

$= \sqrt{\frac{271 \times 10^6}{0.914 \times 1800}} = 406$ mm

Step – VII Check for punching shear :-

Column B

(i) Punching shear force at column B =

$700 - \frac{400}{3}(0.806)^2$ [d 406 + b 400 = 806 mm] (with of column)

$= 613.38$ kN

{b = 400 mm = width of column}

$d = 406$ mm
$b = 400$ mm.

Perimeter $(b_0) = 4(b+d)$
$= 4(406+400)$
$= 3224$ mm.

Shear force $(V) = w/_2 - p(b+d)$
$= 700 - 240(0.400 + 0.406)$
$= 506.56$ kN.

Shear stress $(\tau_v) = \frac{V}{b_0 d}$
$= \frac{506.56 \times 10^3}{3224 \times 406}$
$= \frac{506.56 \times 10^3}{1308944}$
$= 0.3869$ N/mm²

Permissible shear stress $= 0.16\sqrt{f_{ck}}$
$= 0.16\sqrt{20}$
$= 0.715$ N/mm²

τ_v < Permissible shear stress

Hence design section is safe.

VII :- Design of bending tension in longitudinal beam —

1) Reinforcement for Bending moment (M_1)

Effective depth (d) = 406 mm

overall depth (D) = 406 + 60 [60 = size of bars and cover]

effective depth = 466 − 60

= 406 say 410 mm

$A_t = \frac{M_1}{\sigma_t \cdot j \cdot d} = \frac{271 \times 10^6}{230 \times 0.904 \times 410}$

= 3179 mm²

Use 16mm φ bar.

$A_\phi = \frac{\pi}{4} \times (16)^2 = 201\ mm^2$

No. of bar = $\frac{3179}{201}$ = 15.8

= 16 bars.

ii) Reinforcement for bending moment (M_2) ⇒

$$A_{st} = \frac{M_2}{\sigma_{st} \cdot j d} = \frac{97 \times 10^6}{230 \times 0.904 \times 410}$$

$$= 1138 \text{ mm}^2$$

Use 12 mm ϕ bar.

Area of one bar (A_ϕ) $= \frac{\pi}{4} \times (12)^2$

$= 113 \text{ mm}^2$

No. of bars $= \frac{1138}{113} = 10.0707$

say 11 bars

iii) Reinforcement for Bending moment (M_3)

$$A_{st} = \frac{M_3}{\sigma_{st} \cdot j \cdot d} = \frac{15 \times 10^6}{230 \times 0.904 \times 410}$$

$$= \frac{15 \times 10^6}{85247.2} = 175.95$$

$= 176 \text{ mm}^2$

Use 12 mm ϕ bars

$A_\phi = \frac{\pi}{4} (12)^2 = 113 \text{ mm}^2$

No. of bars $= \frac{176}{113} = 1.$

Minimum reinforcement @ ~~0.12~~ 0.12% of overall cross-sectional area.

$$= \frac{0.12}{100} \times 470 \times 1800$$

$$= 1016 \text{ mm}^2$$

[~~400~~ 470 = overall depth

1800 = 1.8 m = width of the footing]

Use 12 mm ϕ bars

$$A_\phi = \frac{\pi}{4}(12)^2 = 113$$

$$\text{No. of bars} = \frac{1016}{113} = 8.99$$

= 9 bars