the length $db = da = \frac{B/2}{\cos\phi}$ $\left[\because \cos\phi = \frac{B/2}{db \text{ or } da}\right]$

and hence vertical component of cohesion on each of the surface db and da $= c \times \frac{B/2}{\cos\phi} \sin\phi \times B/2 \cos\phi$

$= \frac{B}{2} c \tan\phi$

Hence $q_f \times B + \frac{1}{4}\gamma B^2 \tan\phi = 2P_p + 2 \cdot \frac{B}{2} c \tan\phi$

or, $q_f B = 2P_p + B.c\tan\phi - \frac{1}{4}\gamma B^2 \tan\phi$ — (1)

the resultant passive earth pressure P_p can be divided into three component i.e (i) $P_{p\gamma}$ (ii) P_{pc} (iii) P_{pq}

these components of passive pressure are computed separately and then added to obtain the value of P_p.

Substituting these components in equation — (1)

$$q_f B = 2(P_{p\gamma} + P_{pc} + P_{pq}) + B.c\tan\phi - \frac{1}{4}\gamma B^2 \tan\phi \quad \text{— (2)}$$

$$q_f B = \left(2P_{p\gamma} - \frac{1}{4}\gamma B^2 \tan\phi\right) + (2P_{pc} + B.c\tan\phi) + 2P_{pq} \quad \text{— (3)}$$

let $2P_{p\gamma} - \frac{1}{4}\gamma B^2 \tan\phi = B \times \frac{1}{2}\gamma B N_\gamma$

$2P_{pc} + B.c\tan\phi = B\,c\,N_c$ and $2P_{pq} = B \times \bar{\sigma} N_q$

where $\bar{\sigma}$ = effective over burden pressure above the base of footing.

Substituting in equation (3) $q_f \times B = B c N_c + B \times \bar{\sigma} N_q \times B \times \frac{1}{2}\gamma B N_\gamma$

$\Rightarrow q_f = cN_c + \bar{\sigma} N_q + \frac{1}{2}\gamma B N_\gamma$ — (4)

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q_{us}

N_c, N_q, N_γ = B.C factor

Terzaghi's gave the following expression for the bearing capacity factor

$$N_q = \frac{a^2}{2\cos^2(45^\circ + \phi/2)} \quad \text{where } a = e^{(0.75\pi - \phi/2)}$$

$$N_c = (N_q - 1)\cot\phi$$

$$N_\gamma = \frac{\tan\phi}{2}\left(\frac{k_{p\gamma}}{\cos^2\phi} - 1\right)$$

$k_{p\gamma}$ = passive earth pressure coefficient, depend on ϕ.

$$q_f = cN_c + qN_q + 0.5\gamma B N_\gamma$$

~~$q_{nf} = cN_c + q(N_q - 1) + 0.5\gamma B N_\gamma$~~

$$q_{nf} = cN_c + qN_q + 0.5\gamma B N_\gamma - q$$

$$= cN_c + q(N_q - 1) + 0.5\gamma B N_\gamma$$

and $q_s = \frac{1}{F}[q_{nf}] + q$

for purely cohesive soil $\phi = 0$ $N_\gamma = 0$

$$q_f = cN_c + qN_q$$

$$= 5.7c + q$$

Brinch Hansen's Analysis

→ In recent years many new equations have been proposed by various workers, giving the ultimate bearing capacity of foundations.

→ these equations are based on the assumption of various values of angle ϕ and on the shape of failure surface.

→ Notable amongst the workers are Hansen (1970), Hu (1964), Chen and Davidson (1973) and Balla (1962).

out of these the one proposed by Hansen's gives better results.

According to Hansen, the ultimate bearing capacity is given by

$$q_f = cN_c s_c d_c i_c g_c b_c + q N_q s_q d_q i_q g_q b_q + \frac{1}{2}\gamma B N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$$

where, q = effective overburden pressure at foundation level

s = shape factor, to account for the effect of the shape of the foundation in developing a failure surface.

d = depth factor

i = inclination factor

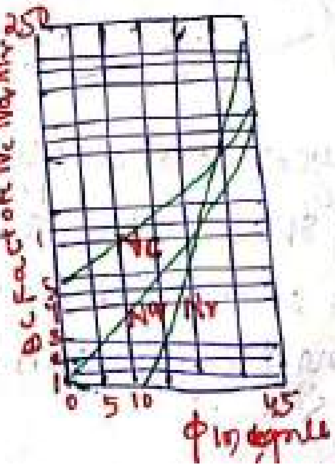
g = ground factor; b = base factor

γ = density of the soil below the foundation level.

$N_q = \tan^2(45° + \frac{\phi}{2})(e^{\pi\tan\phi})$

$N_c = (N_q - 1)\cot\phi$

$N_\gamma = 1.5(N_q - 1)\tan\phi$



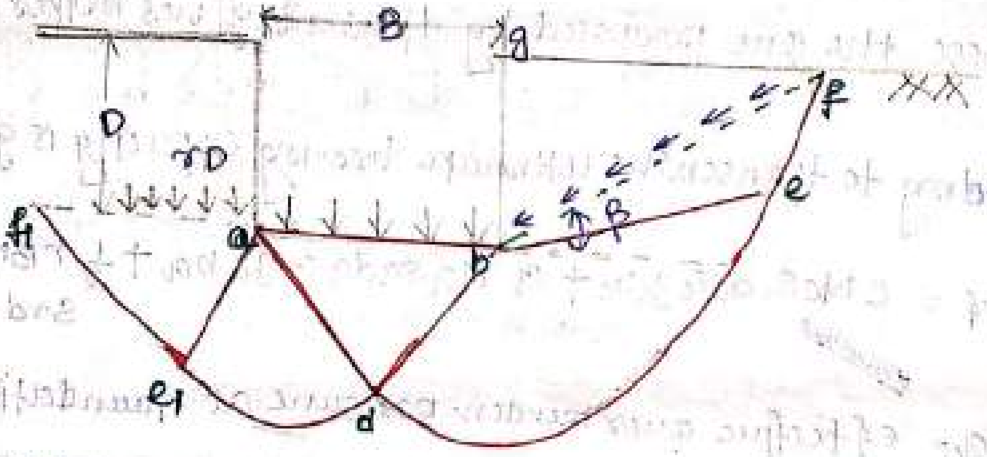
$\phi = 0$ $N_q = 1$

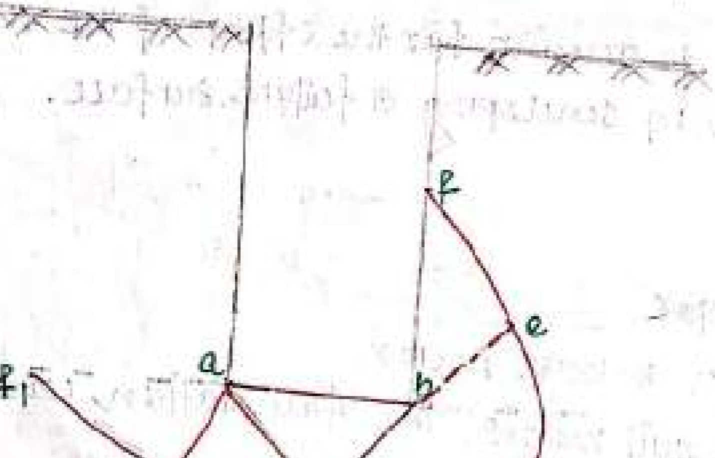
$N_c = 5.14$

Meyerhof's Analysis

Meyerhof's expended the analysis of plastic equili of a surface footing to shallow and deep foundations.

Fig ⓐ and ⓑ shows the failure mechanism for shallow and deep foundations according to both Terzaghi and Meyerhof analysis.





In the Meyerhof's analysis abd is the elastic zone, bde is the radial shear zone and befg is the mixed shear zone in which shear varies between radial and plane shear, depending largely upon the depth and roughness of the foundation.

To simplify the analysis, Meyerhof introduced a parameter β, the angle to define the line of joining point b to f where the assumed boundary failure slip line intersects the soil surface.

The resultant effect of the wedge of soil bfg is represent by the normal and tangential stresses P_0 and S_0 on bf.

The plane bf is termed as the equivalent free surface. and P_0 and S_0 are termed as the equivalent free surface stresses.

The angle β increases with depth, and become 90° for deep foundations.

Meyerhof (1963) gave the equation for ultimate B.C taking into account the shape, depth and inclination factors.

(a) vertical load: $q_f = cN_c s_c d_c + qN_q s_q d_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma$

(b) inclined load: $q_f = cN_c d_c i_c + qN_q d_q i_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma$

where $N_q = e^{\pi \tan\phi} \tan^2(45 + \phi/2)$

$N_c = (N_q - 1)\cot\phi$

$N_\gamma = (N_q - 1)\tan(1.4\phi)$

N_c, N_q & N_γ are given in table. It will be seen that the factors N_c and N_q are the same as given by Hansen while the factor N_γ is different.

Vesic Bearing Capacity Equation

vesic bearing capacity equation is similar to Hansen's analysis equation.

the essential difference in vesic's and Hansen's procedures are ① use of slightly different value of N_γ. and ② variation on some of Hansen's inclination, base and ground factors.

vesic's bearing capacity factors are given by the following equations:

$$N_q = \tan^2(45^\circ + \phi/2)\, e^{\pi \tan\phi}$$

$$N_c = (N_q - 1)\cot\phi$$

$$N_\gamma = 2(N_q + 1)\tan\phi$$

the values of N_q and N_c are the same as in Hansen's analysis while the value of N_γ are different.

Effect of water table on Bearing Capacity

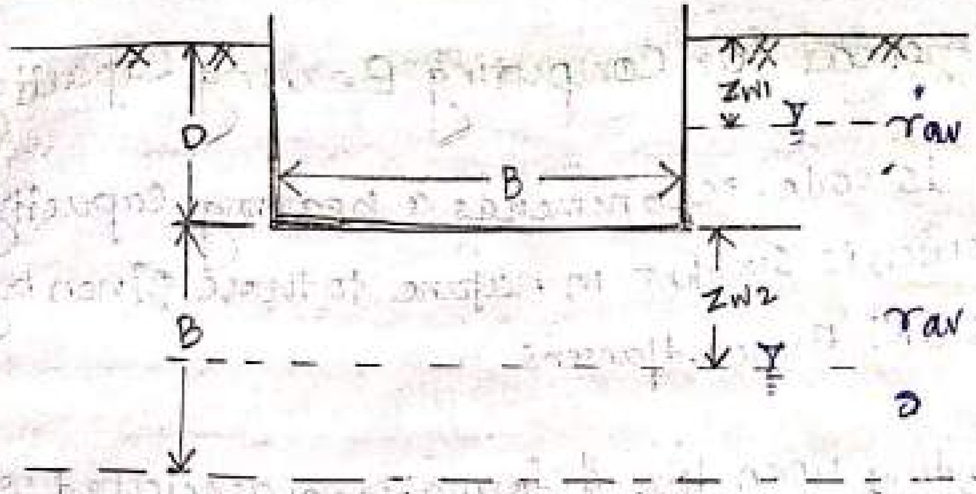
i. First method

for any position of the water table the equation ①

$$q_f = cN_c + qN_q + \frac{1}{2}\gamma B N_\gamma \quad \text{—①}$$

modified to $q_f = c.N_c + qN_q R_{w1} + \frac{1}{2}\gamma B N_\gamma R_{w2}$

where, R_{w1} and R_{w2} are the reduction factors for water table.



γ_{av} for q =

γ_{av} for q =

$$R_{w1} = 0.5\left(1+\frac{Z_{w1}}{D}\right)$$

At $Z_{w1}=0$, $R_{w1}=\frac{1}{2}$; At $Z_{w1}=D$, $R_{w1}=1$

And $$R_{w2} = 0.5\left(1+\frac{Z_{w2}}{B}\right)$$

At $Z_{w2}=0$, $R_{w2}=0.5$; At $Z_{w2}=B$, $R_{w2}=1$

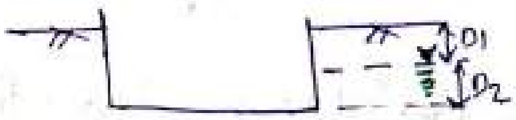
ii) Second method : IS Code method

$$q_f = cN_c + qN_q + \frac{1}{2}\gamma B N_\gamma R_w$$

$$R_w = R_{w2} = 0.5\left(1+\frac{Z_{w2}}{B}\right) \quad \text{— (1)}$$

equation-(1) has been recommended by Indian Standard.

when the water table situated at a depth D_1 below the ground level $(D_1<D)$ or D_2 above the base of the footing



$$q = (\gamma D_1 + \gamma_{sat} D_2) - \gamma_w D_2$$
$$= \gamma D_1 + \gamma' D_2$$

knowing q and $R_w (=R_{w2})$ q_f can be computed.

I.S code method for Computing Bearing Capacity

1. General: IS code, recommends a bearing capacity equation which is similar in nature to those given by meyerhof's and Brinch Hansen's.

the code depending upon the deformations associated with the load and the extent of development of failure surface.

three type of failures of soil support beneath the foun...

they are
(a) general shear failure
(b) Local shear failure
(c) Punching shear failure

2. Bearing capacity equation for strip footing for C-ϕ soils:

the ultimate B.C of soil for strip footing

$$q_f = cN_c + qN_q + \frac{1}{2}\gamma B N_\gamma$$

• for general shear failure.

i. net ultimate B.C = $q_{nf} = cN_c + q(N_q - 1) + 0.5\gamma B N_\gamma$

for the case of general shear failure

for Local shear failure.

ii. $q_{nf} = \frac{2}{3}cN_c' + q(N_q' - 1) + \frac{1}{2}\gamma B N_\gamma'$

for the case of Local shear failure

q = surcharge load N_c, N_q, N_γ = B.C factor.

$$N_c = (N_q - 1)\cot\phi$$

$$N_q = \tan^2(45 + \phi/2)\, e^{\pi\tan\phi} \quad N_\gamma = 2(N_q + 1)\tan\phi$$

vesics

N_c', N_q', N_γ' from $\phi' = \tan^{-1}(0.67\phi)$

$c' = 2/3\, c$ reduced parameter

$\tan\phi' = 2/3 \tan\phi$

5. Shape factor, depth factor and inclination factor:

The above B.C equations applicable for strip footing, shall be modified to take into account, the shape of the footing, inclination of loading, depth of embedment and effect of water table.

The modified B.C formulae

(i) for general shear failure

$$q_f = c N_c s_c d_c i_c + q N_q s_q d_q i_q + 0.5 \gamma B N_\gamma s_\gamma d_\gamma i_\gamma W'$$

$$q_{nf} = q_f - q$$

(ii) for local shear failure

$$q_f = 2/3\, c N_c' s_c d_c i_c + q N_q' s_q d_q i_q + 0.5 \gamma B N_\gamma' s_\gamma d_\gamma i_\gamma W'$$

4. Effect of water table

5. B.C equation for general shear failure

$q_u = cN_c + qN_q + 1/2 \gamma B N_\gamma$ (continuous footing)
$q_u = 1.3\, cN_c + qN_q + 0.4 \gamma B N_\gamma$ (square ")
$q_u = 1.3\, cN_c + qN_q + 0.3 \gamma B N_\gamma$ (circular ")

General

B.C equation for local shear failure mode in soil above equation modified to

Local

$q_u = 2/3\, cN_c' + qN_q' + 1/2 \gamma B N_\gamma'$ (strip foundation)
$q_u = 0.867\, cN_c' + qN_q' + 0.4 \gamma B N_\gamma'$ (square foundation)
$q_u = 0.867\, cN_c' + qN_q' + 0.3 \gamma B N_\gamma'$ (circular foundation)

Example 2.45

A strip footing 2m wide carries a load intensity of 400 kN/m² at a depth of 1.2m in sand. the saturated unit weight of sand is 19.5 kN/m³ and unit weight above water table is 16.8 kN/m³. the shear strength parameters $c = 0$ and $\phi = 35°$. Determine the factor of safety with respect to shear failure for the following case of location of water table

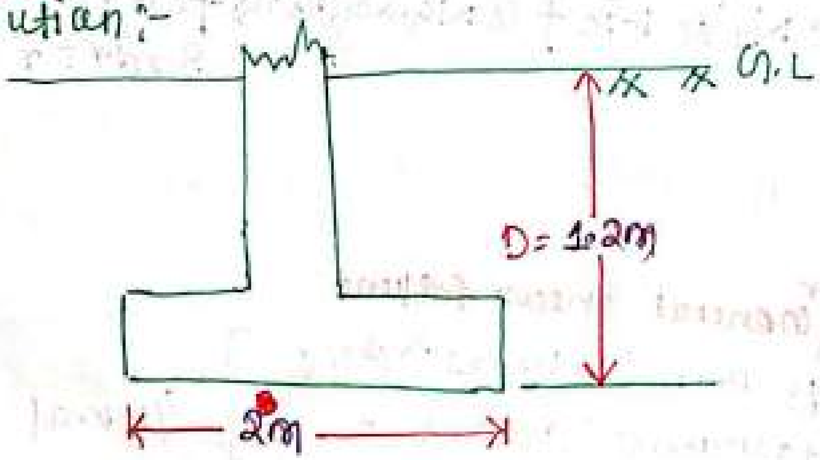
for $\phi = 35°$ $N_q = 41.4$
$N_\gamma = 42.4$

(a) W.T is 4 m below G.L
(b) " " 1.2m " G.L
(c) " 2.5m " G.L
(d) " 0.5m " G.L
(e) " at G.L itself

use Terzaghi's equation

(f)

Solution:-



for strip footing, the bearing capacity equation is given

$$q_f = cN_c + qN_q + 0.5\gamma BN_\gamma$$

taking into account the reduction factor

$$q_f = cN_c + qN_q R_{w1} + 0.5\gamma B N_\gamma R_{w2}$$

for the present case $c = 0$,

$$q_f = \gamma D N_q R_{w1} + 0.5 \gamma B N_\gamma R_{w2}$$

for $\phi = 35^\circ$, assuming general shear failure $N_q = 41.4$ and $N_\gamma = 42.4$

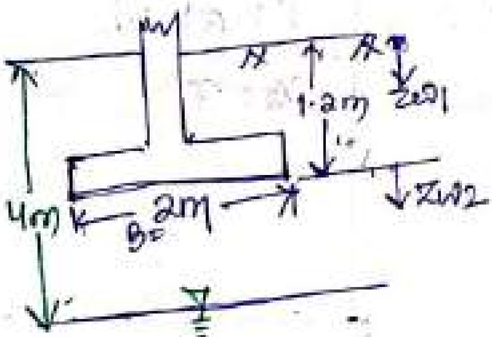
$$\therefore q_f = 41.4 \times 1.2 \gamma R_{w1} + 0.5 \gamma \times 2 \times 42.4 R_{w2}$$

Imp proof

$$= 49.68 \gamma R_{w1} + 42.4 \gamma R_{w2}$$

Case-a

water table is 4m below G.L.



$$R_{w1} = 0.5\left(1 + \frac{z_{w1}}{D}\right) = 0.5\left(1 + \frac{1.2}{1.2}\right) = 1$$

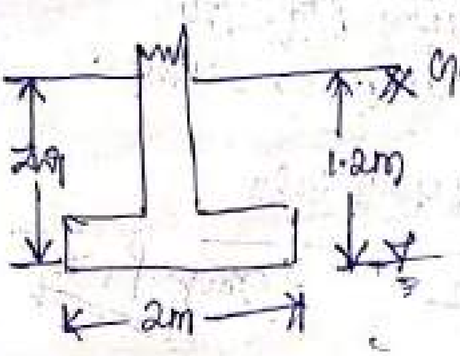
$$R_{w2} = 0.5\left(1 + \frac{2.8}{2.8}\right) = 1 \quad \text{as } z_{w2} > B$$

$$q_f = 49.68 \times 16.8 \times 1 + 42.4 \times 16.8 = 1549.9 \text{ kN/m}^2$$

the actual footing load $= q_a = 400 \text{ kN/m}^2$

$$F.S = \frac{q_f}{q_a} = \frac{1549.9}{400} \approx 3.87$$

Case-b water table is just at the base of footing



$R_{w1} = 0.5\left(1+\frac{Z_{w1}}{D}\right)$

$= 0.5(1+1)$

$= 1$

$R_{w2} = 0.5(1+0) = 0.5$

$q_f = 49.68\,\gamma\,R_{w1} + 42.4\,\gamma_{sat}\cdot R_{w2}$

$= 49.68 \times 16.8 \times 1 + 42.4 \times 19.5 \times 0.5$

$= 1248\ kN/m^2$

$f.s = \frac{q_f}{q_a} = \frac{1248}{400} = 3.12$

∵ As per depth of w.T $\gamma_2 = \gamma_{sat}$

& method

$\gamma_2 = \gamma'$

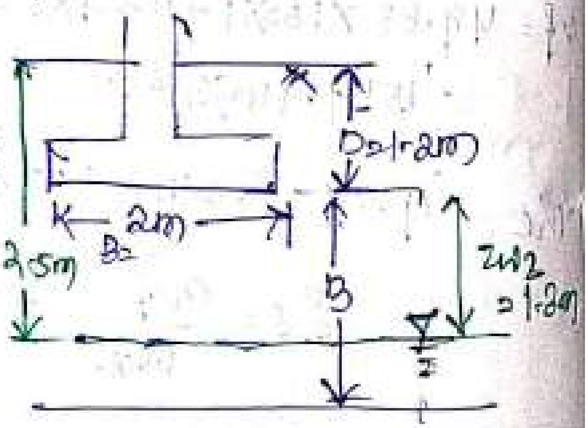
Case (c) water table 2.5m below the GL

$Z_{w1} > D \quad R_{w1} = 1$

$Z_{w2} = 2.5 - 1.2 = 1.3 < B$

$R_{w2} = 0.5\left(1+\frac{1.3}{2}\right)$

$= 0.825$



$\gamma_{av} = \frac{16.8 \times 1.3 + 19.57 \times 0.7}{1.3 + 0.7}$

$= 17.075\ kN/m^3$

$q_f = 49.68\,\gamma\,R_{w1} + 42.4\,\gamma_{av}\,R_{w2} = 1455.5\ kN/m^2$

$$fs = \frac{Q_{nf}}{Q_{na}} = \frac{1455.5}{400} = 3.64$$

Case-d : when water table is 0.5m below G.L

$$Z_{w1} = 0.5$$

$$R_{w1} = 0.5\left(1+\frac{0.5}{1.2}\right) = 0.708$$

$$R_{w2} = 0.5$$

$$\gamma_{av} = \frac{(16.8\times0.5)+(19.5\times0.7)}{0.5+0.7}$$

$$= 18.38$$

$$q_{nf} = 49.68\ \gamma_{av} R_{w1} + 42.4\ \gamma_{sat} R_{w2}$$

$$= 1060\ kN/m^2$$

$$f.s = q_{nf}/q_{na} = \frac{1060}{400} = 2.65.$$

Case- e : water table at the ground level.

$R_{w1} = 0.5$, $R_{w2} = 0.5$

$$\gamma = \gamma_{sat} = 19.5$$

$$q_{nf} = 49.08\ \gamma_{sat} R_{w1} + 42.4\ \gamma_{sat} R_{w2}$$

$$= 897.78\ kN/m^2$$

$$f.s = q_{nf}/q_{na} = \frac{897.78}{400} = 2.24.$$

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Example 24.10. A square footing located at a depth of 1.3m below the ground has to carry a safe load of 800kN. Find the size of the footing if the desired factor of safety is 3. The soil has the following properties

$e = 0.55$; $S_r = 50\%$; $G = 2.67$

$C = 8\ kN/m^2$; $\phi = 30°$. Use Terzaghi's analysis assume. $N_c = 37.2$, $N_q = 22.5$ & $N_\gamma = 19.7$

Solution:- $\gamma = \frac{\gamma_w(G + e.S_r)}{1+e}$

$= \frac{9.81(2.67 + 0.55 \times 0.5)}{1 + 0.55}$

$= 18.64\ kN/m^3$

for. $\phi = 30°$, $N_c = 37.2$, $N_q = 22.5$ and $N_\gamma = 19.7$

$q_f = 1.3\ cN_c + qN_q + 0.4\ \gamma B N_\gamma$.

$q = \gamma D = 18.64 \times 1.3$.

$q_f = 1.3 \times 8 \times 37.2 + 18.64 \times 1.3 \times 22.5 + 0.4 \times 18.64 \times B \times 19.7$

$q_f = 932.1 + 146.88\ B\ kN/m^2$

$q_{nf} = q_f - q$ or γD

$= 907.87 + 146.88B$

$q_s = \frac{q_{nf}}{F} + \gamma D = 326.86 + 48.96\ B$ —— (1)

Actual load intensity $q_a = 800/B^2$ —— (2)

equating ① and ②

$$\frac{800}{B^2} = 326.86 + 48.96B$$

$$B^3 + 6.676B^2 = 16.34$$

$B \simeq 1.42m$ solved by try and error.

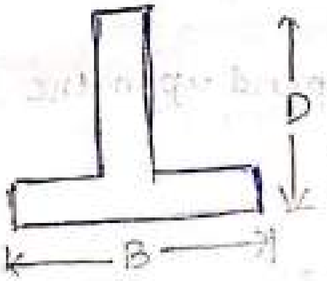
Shall foundation

foundation may be broadly classifned under two heads

- shall foundation
- Deep foundation

Shallow foundation

According to terzaghi. a faundation is shallow foundation if the width of foundation is more then equal to the depth of faundation:



$B \geq D$

the example of shallow foundation?

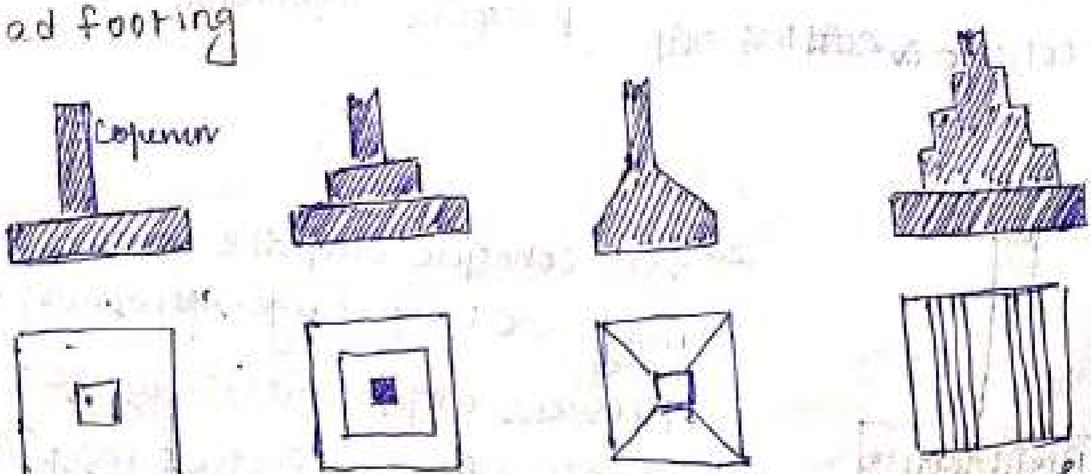
i. spread footing

ii. strap footing 

iii. combined footing  equal/unequal load

iv. mat or raft footing 

Spread footing



a. single footing b. stepped footing. c. slopped footing d. wall footing

→ A spread footing or simply footing is a type of shallow foundation used to transmit the load of an isolated column, or that of a wall to the subsoil.

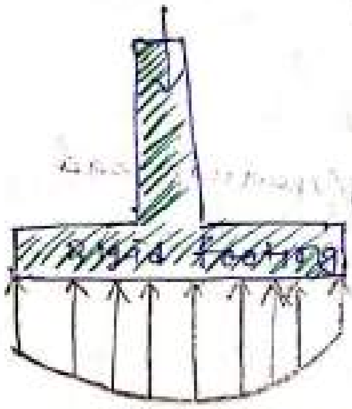
→ this is the most common type of foundation.

→ the base of the column is enlarged or wall is enlarged or spread to provide individual support for the load.

pressure distribution

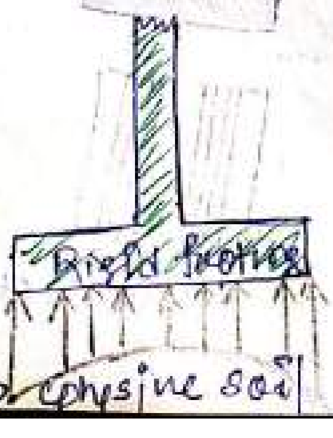
→ the pressure distribution at beneath footing, symmetrically loaded is not uniform.

→ the pressure distribution intensity depend upon the
(i) rigidity of footing
(ii) the soil type
(iii) the condition of soil.



a. cohesionless soil

→ the soil grains at the corner edge have no lateral restraint whereas in centre the soil is relatively confined resulting pressure distribution.



b. cohesive soil

→ for cohesive soil, the edge stresses may be very large. However, the pressure distribution may be considered to be stresses may be very large.

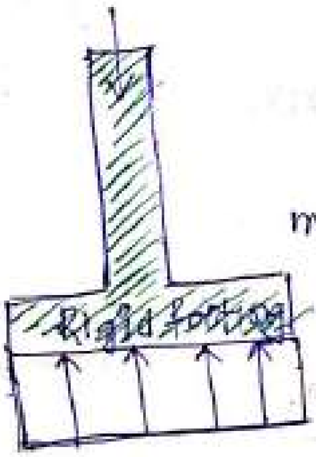
* In both case as point load acting at centre so max^m load will developed at the centre of footing

→ for the case of sand or cohesionless soil simply the soil get settle or deform and max^m resisting force will develop at the centre and decreasing linearly towards edge of footing.

→ for cohesive soil pore water pressure will develope at the centre max^m due to max^m load at centre so the effective stress will decreases. so resisting force also decreases

$$\sigma' = \sigma - u$$

but ~~at the edge~~ then towards the edge the ~~excess~~ pore water pressure decreases linearly so effective stress increases linearly so resisting force also increases linearly.



the pressure distribution diagram may consider to be linear as shown in fig for the purpose of the design of reinforced concrete footing.

c) uniform pressure distribution.

Once the pressure distribution known, the bending moment and shear force in footing can be calculated and thickness of the structural member of the footing along with the reinforcement etc, can be calculated using principles of reinforced concrete.

Safe bearing pressure :→

In conventional design the allowable bearing capacity should be taken as the smaller of the following two values.

✓ (i) the safe bearing capacity based on ultimate capacity.

✓ (ii) the allowable bearing pressure on tolerable settlement

As per Terzaghi

for strip footing $q_s = \frac{1}{F}[cN_c + \gamma D(N_q - 1) R_{w1} + 0.5\gamma B N_\gamma R_{w2}] + \gamma D$

for square footing $q_s = \frac{1}{F}[1.3cN_c + \gamma D(N_q - 1)R_{w1} + 0.4\gamma B N_\gamma R_{w2}] + \gamma D$

for circular footing $q_s = \frac{1}{F}[1.3cN_c + \gamma D(N_q - 1)R_{w1} + 0.3\gamma B N_\gamma R_{w2}] + \gamma D$

for rectangular footing

$$q_s = \frac{1}{F}\left[cN_c\left(1 + 0.3\frac{B}{L}\right) + \gamma D(N_q - 1)R_{w1} + 0.5\gamma B N_\gamma\left(1 - 0.2\frac{B}{L}\right)R_{w2}\right] + \gamma D$$

where, D = depth of footing

B = width of footing (strip or square) or diameter of circular footing.

L = length of the footing

N_c, N_q, N_γ = Bearing capacity factors for general shear failure.

for local shear failure $c', N_c', N_q', N_\gamma'$ should be used. $c' = \frac{2}{3}c$, $\tan\phi' = \frac{2}{3}\tan\phi$, $\phi' = \tan^{-1}(\frac{2}{3}\tan\phi)$

R_{w1}, R_{w2} = Water Reduction factor. F = F.O.S. 2 to 3 the net allowable bearing pressure q_s based on limiting the maximum settlement of individual footing to 25mm can be computed from the following empirical relation.

$$q_s = 34.3(N-3)\left(\frac{B+0.3}{2B}\right)^2 R_{w2} R_d$$

q_s = allowable net increase in soil pressure over existing soil pressure for settlement of 25mm, in kN/m².
N = Standard penetration number with applicable corrections

B = width of footing in meters.

R_{w2} = water reduction factor $= 0.5\left[1+\frac{Z_{w2}}{B}\right]$

R_d = depth factor $= \left(1+\frac{0.2D}{B}\right) \le 1.20$

q_a = permissible net increase in the B.pressure.

the permissible gross increase in the bearing pressure will be

$$q_{sg} = q_s + \gamma D$$

the allowable bearing capacity (q_a) will be then be greater of the values of q_s and q_{sg}.

Suppose max^m permissible settlement is S' instead of S (25mm) the net allowable b.p

$$q'_s = q_s \times \frac{S'}{25}$$

It should be noted that S_p (or q'_s) is the permissible net increase in the bearing pressure

Settlement of footing

If the safe bearing capacity is taken to be smaller of two value (i) allowable b. pressure on tolerable settlem
in comparison less err is safe b.c [(ii) ~~net~~ the safe bearing capacity based on ultimate capacity

as discuss above.

the footing on granular soil will not suffer detrimental settlement.

the total settlement of a footing in clay may be considered to consist of three components (Skempton & Bjerrum, 1957)

$$S = S_i + S_c + S_s$$

where S = total settlement

S_i = immediate elastic settlement

S_c = consolidation settlement

S_s = settlement due to secondary consolidation

S_i, It can be expressed based on theory of elasticity

$$S_i = qB\left[\frac{1-\mu^2}{E_s}\right]I_w$$

where q = intensity of contact pressure

B = least lateral dimension of footing

E_s = modulus of elasticity of soil.

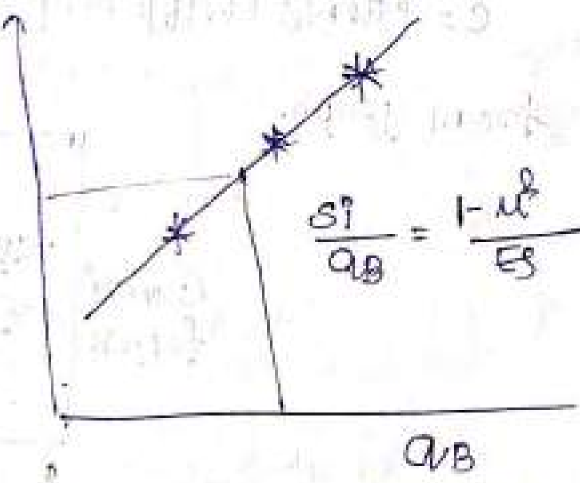
I_w = influence factor = 0.88 for rigid ○ler footing
= 0.82 □re
= 1.06 rectangular, with L/B = 1.5
= 1.70 " L/B = 5

Table also given L/B | I_w

given BIS code IS.

It is difficult to determine μ and E_s for soils, however the entire term $\frac{1-\mu^2}{E_s} I_w$

It can be determine from the plate load test by using different size of the plate.



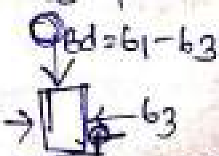
$$\frac{S_i}{q_B} = \frac{1-\mu^2}{E_s} \cdot I_w$$

Janbu, Bjerrum and Kjaernsli (1966) have proposed the following equation for Si (Immediate Settlement)

$$S_i = \mu_0 \mu_1 q_B \frac{1-\mu^2}{E_s}$$

where E_s can be calculate from the triaxial test

$$E_s = \frac{\sigma_1 - \sigma_3}{\Delta L / L}$$



μ_0 & μ_1 can be taken from fig.

the consolidation settlement S_c is computed from the following equation

$$S_c = C \cdot \frac{C_c}{1+e_0} H \log_{10} \frac{\sigma_0 + \Delta\sigma}{\sigma_0}$$

σ_0 = effective initial overburden pressure due to soil overburden, measured at the centre of the layer

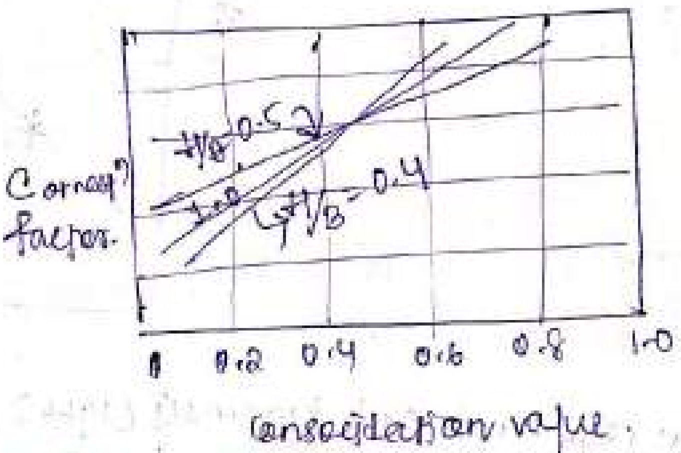
$\Delta\sigma$ = vertical stress increment due to footing load, at the centre of layer.

C_c = compression index $= 0.009\,(w_L - 10)$

e = void ratio, H = thickness of the compressible layer,

C = ~~corres~~ coefficient of correction factor.

from graph.



$\phi = 0$, $N_c = 5.7$, $N_q = 1$ & $N_\gamma = 0$

Ex:- A square footing 1.2m x 1.2m rests at a depth of 1m in a saturated clay layer 4m deep. The clay is normally consolidated, having an unconfined compressive strength of 40 kN/m².

The soil has a liquid limit of 30%.

$\gamma_{sat} = 17.8\ kN/m^3$, $w = 28\%$ and $G = 2.68$

Determine the load which the footing can carry safely with a factor of safety of 3 against shear. Also, determine the settlement if the footing is loaded with this safe load. Use Terzaghi's analysis for bearing capacity.

Solution -:

Since $\phi = 0$, $N_c = 5.7$; $N_q = 1$ and $N_\gamma = 0$.

Also $\bar{\sigma} = \gamma_{sat} D = 17.8 \times 1 = 17.8$

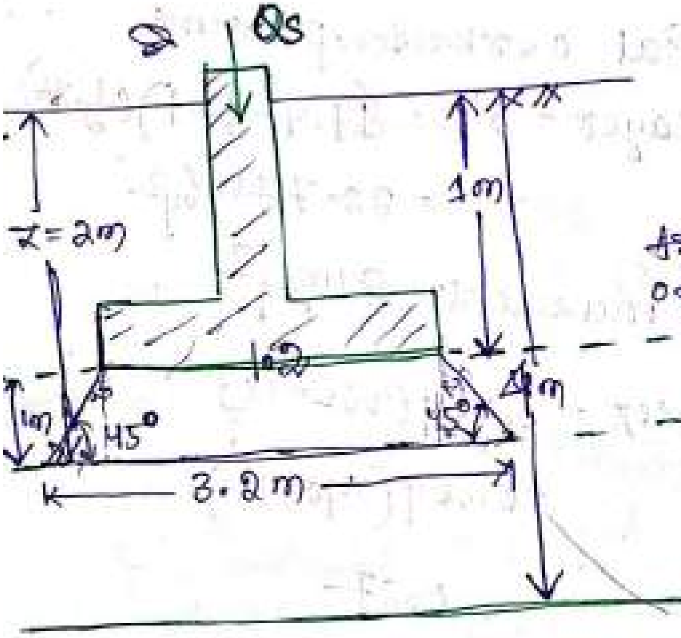
$c = \frac{q_u}{2} = \frac{40}{2} = 20 \, kN/m^2$

$$q_s = \frac{q_{nf}}{F} + \gamma_{sat} D = \frac{1}{F}\left[1.3 c N_c + \bar{\sigma}(N_q - 1) + 0.4 B \cdot \gamma N_\gamma\right] + \gamma_{sat} D$$

or, $$q_s = \frac{1}{3}\left[1.3 \times 20 \times 5.7 + 17.8(1-1) + 0\right] + 17.8 = 49.4 + 17.8$$

$$= 67.2 \, kN/m^2$$

$$q_s = \frac{Q_s}{B^2} \Rightarrow Q_s = 67.2 \frac{kN}{m^2} \times (1.2 \times 1.2) m^2 = 96.768 \, kN$$



Thickness of the clay layer = 4m.

Depth of centre of clay layer below footing level $= \frac{4}{2} - 1 = 1m$.

Assuming load dispersion at 45° width of load spread $= 1.2 + 2(1) = 3.2m$

$\therefore$ vertical stress increment due to foundation load

$$= \Delta\sigma = \frac{96.77}{3.2 \times 3.2} = 9.45 \, kN/m^2$$

Next, consolidation settlement of footing is given by

$$S_c = C \frac{c_c}{1+e_0} H \log_{10} \frac{\sigma_0 + \Delta\sigma_0}{\sigma_0}$$

Assume $C = 1$. $e_0 = wG = 0.28 \times 2.61 = 0.73$

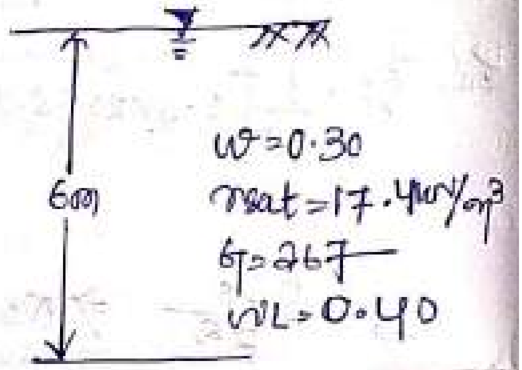
$c_c = 0.009(w_L - 10) = 0.009(30-10) = 0.18$

Initial over burden pressure at the centre of clay layer

$= \sigma_0 = \gamma_{sat} z = 17.8 \times 2 = 35.6 \, kN/m^2$

$$S_c = \frac{0.18}{1 + 0.73} \times 4 \log_{10} \frac{35.6 + 9.45}{35.6} = 0.042m = 42mm$$

Example 25.1. A soft, normally consolidated clay layer is 6m thick with a natural water content of 30%. The clay has $\gamma_{sat} = 17.4\ kN/m^3$ and $G = 2.67$ and $W_L = 40\%$. The ground water level is at the surface of the clay. Determine the settlement of the foundation if the foundation load will subject the centre of the clay layer to a vertical stress increase of $8\ kN/m^3$.



σ_0 = effective initial overburden pressure measured at the centre of layer $= \gamma' z = (17.4 - 9.81)(\frac{1}{2} \times 6)$
$= 22.77\ kN/m^2$

$\Delta\sigma$ = vertical stress increment $= 8\ kN/m^2$.

C_c = compression index $= 0.009(W_L - 10)$
$= 0.009(40 - 10)$
$= 0.27$

$e_0 = \frac{wG}{S_r}$, $wG = 0.30 \times 2.67 = 0.801$

$$S_c = C \frac{C_c}{1+e_0} H \log_{10} \frac{\sigma_0 + \Delta\sigma}{\sigma_0}$$

Taking $C = 1$. [a coefficient of correction factor depending upon the geometry of the footing and system of loading on the clay]

$$S_c = 1 \times \frac{0.27}{170.801} \times 6 \log_{10}\left[\frac{22.77+8}{22.7}\right]$$

$$= 0.087\ m = 87\ mm.$$

Example 2.5.3 A rectangular footing 2m×3m carries a column load of 600kN at a depth of 1m. the footing. the footing rest on a c-φ soil strata 6m thick, having poisson's ratio of 0.25 and young's modulus of elasticity as $2 \times 10^4\ kN/m^2$. calculate the immediate elastic settlement of the footing.

$$S_i = qB\left(\frac{1-\mu^2}{E_s}\right) I_w$$

q = intensity of pressure $= \frac{600}{2 \times 3} = 100\ kN/m^2$

μ = poisson's ratio $= 0.25$; $E_s = 2 \times 10^4\ kN/m^2$

I_w = influence factor $= 1.06$ for rigid rectangular footing having $L/B = 1.5$

$$S_i = 100 \times 2\left[\frac{1-(0.25)^2}{2 \times 10^4}\right] \times 1.06$$

$$= 9.94 \times 10^{-3}\ m = 9.94\ mm.$$

Combined footing

- 2/3
- □
- assume

→ A spread footing which supports two or more column is termed as a combined footing.

→ The combined footing may be rectangular in shape if both the column carry equal loads.

→ the combined footing may be trapezoidal if they carry unequal loads.

→ Generally combined footing are constructed of reinforced concrete.

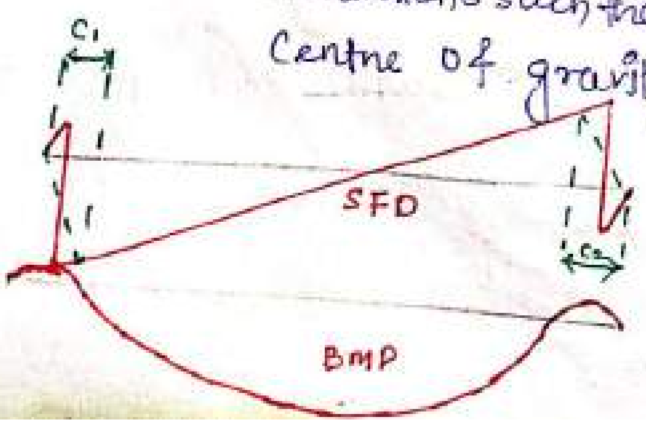
→ It is assumed that the footing is rigid and rests on a homogeneous soil, so as to give rise to a linear stress distribution on the bottom of the footing.

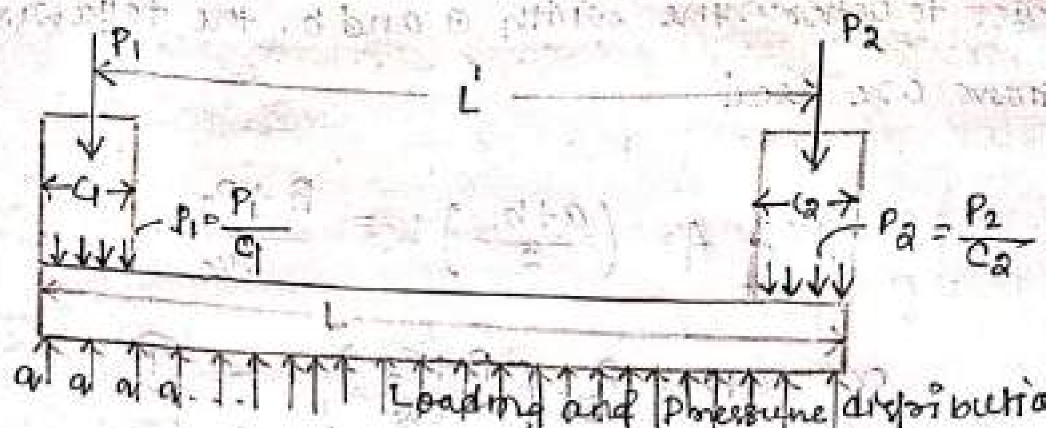
→ If the resultant of the soil pressure distribution coincides with the resultant of the loads, the soil pressure is assumed to be uniformly distributed.

→ Rectangular Combined footing

→ the design of rigid rectangular combined footing consists of

- determining the location of centre of gravity of the column loads and using length and width dimensions such that centroid of footing and Centre of gravity of column loads Coincide.





Loading and Pressure distribution diagram

The resultant pressure distribution diagram will be rectangular with the pressure intensity $q = \frac{P_1 + P_2}{B}$ (per unit area)

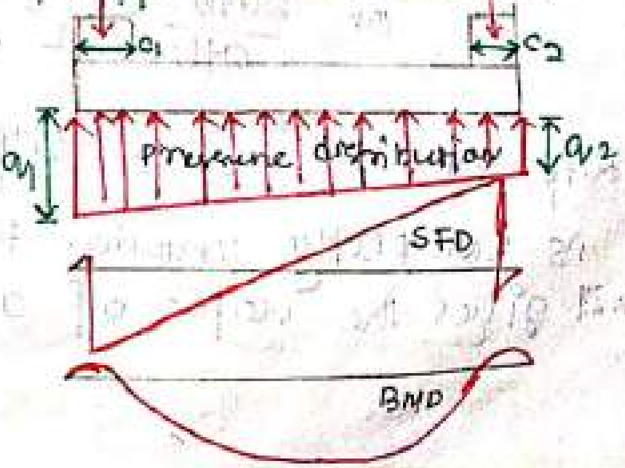
→ the column loads may be considered concentrated loads. and the resulting shear force and bending moment diagram can be plotted.

→ the maximum B.M should be adopted as the design value for the reinforced concrete design footing. which should also be checked for maximum shear and bond etc.

→ Trapezoidal combined footing

→ when two column loads are unequal, with the outer column carrying heavier load, and when there is space limitation beyond the outer column, a trapezoidal combined footing is provided.

→ the resulting pressure distribution is linear (but not rectangular).



In order to determine width a and b, the following relations are used

$$\text{Safe Area } A = \left(\frac{a+b}{2}\right)L = \frac{P_1+P_2}{q_s} = \frac{P_1+P_2}{P_1+P_2/\text{safe area}} = \frac{P_1+P_2 \times \text{Safe}}{P_1+P_2}$$

that is our design

$$\text{or,} \quad a+b = \frac{2}{L}\left(\frac{P_1+P_2}{q_s}\right) \quad \text{---} (4)$$

where q_s = allowable soil pressure.

Also, taking moments about the centre of the column (1),

$P_2 \times L_1 + P_1 \times 0$

we have $(P_1+P_2)x' = P_2 L_1$

$$\text{or} \quad x' = \frac{P_2 L_1}{(P_1+P_2)}$$

$$\therefore \quad \bar{x} = \frac{C_1}{2} + \frac{P_2 L_1}{P_1+P_2} \quad \text{---} (1)$$

But for a trapezoidal section $\bar{x}$ is given by

$$\bar{x} = \frac{L}{3}\left[\frac{2a+b}{a+b}\right] \quad \text{---} (2)$$

Equating equation (1) and (2) we get

$$\frac{L}{3}\left[\frac{2a+b}{a+b}\right] = \frac{C_1}{2} + \frac{P_2 L_1}{(P_1+P_2)}$$

$$\text{or} \quad \frac{2a+b}{a+b} = \frac{3}{L}\left[\frac{C_1}{2} + \frac{P_2 L_1}{P_1+P_2}\right] \quad \text{---} (3)$$

R.H.S of equat^n-(3) and ~~L.H.S of~~ R.H.S of eq^n (4) is completely known. From both equation will gives the value of a and b.

knowing a and b, the pressure intensities q_1 and q_2 are calculated as

$$q_1 = b \times q_s$$

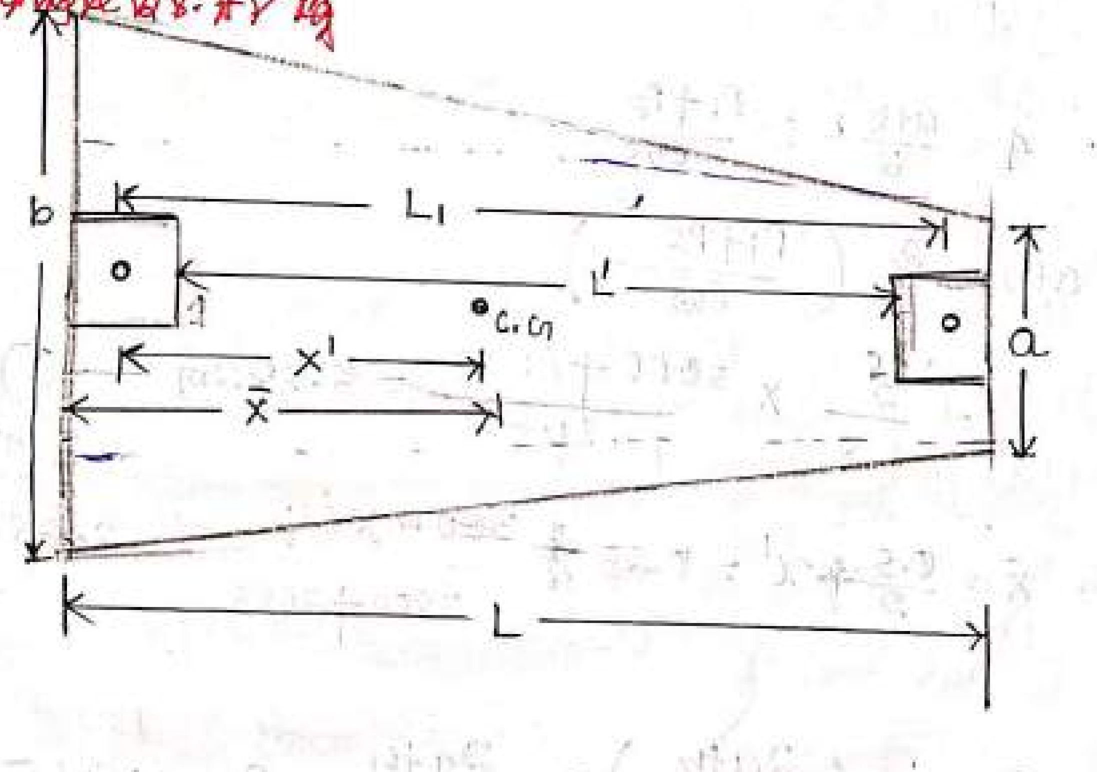
area. and $q_2 = a \times q_s$

the B.M and S.F.D can now be drawn

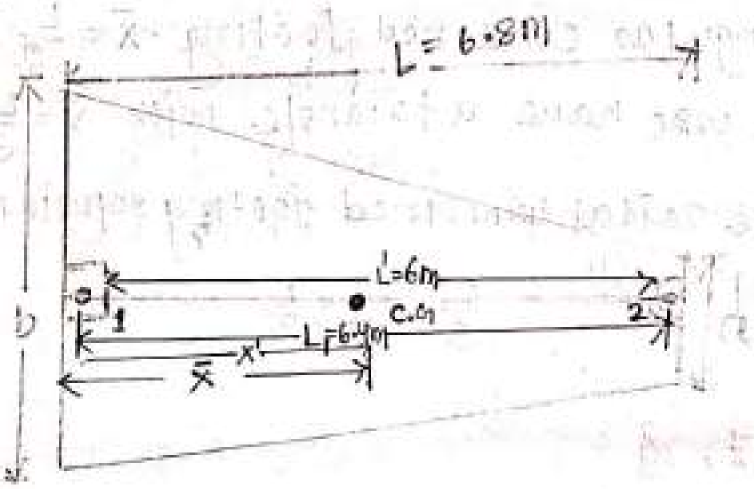
for rectangular combined footing $\bar{x} = \frac{L}{2}$

if $a = 0$ we have a triangle with $\bar{x} = \frac{L}{3}$;

Hence a trapezoidal combined footing solution exists between the following limits $\frac{L}{3} < \bar{x} < \frac{L}{2}$.



Example 23.7. A trapezoidal footing is to be produced to support two square column of 30x50 cm columns are 6 m apart and the safe bearing capacity of the soil is 400 kN/m^2. The bigger column carries 5000 kN and the smaller 3000 kN. Design a suitable size of the footing so that it doesnot extend beyond the faces of the column.



Area. $A = \frac{a+b}{2} L = \frac{P_1+P_2}{q_s}$

$\therefore a+b = \frac{2}{L}\left(\frac{P_1+P_2}{q_s}\right)$

$= \frac{2}{6.8} \times \frac{5000+3000}{400} = 5.882\,m$ — (1)

Also, $\bar{x} = \frac{0.5}{2} + x' = 0.25 + \frac{3000 \times 6.4}{5000+3000} = 2.65\,m$

But $\bar{x} = \frac{L}{3}\left(\frac{2a+b}{a+b}\right)$. $\frac{2a+b}{a+b} = \frac{3}{6.8} \times 2.65$

$= 1.169$

or, $0.831a - 0.169b = 0$

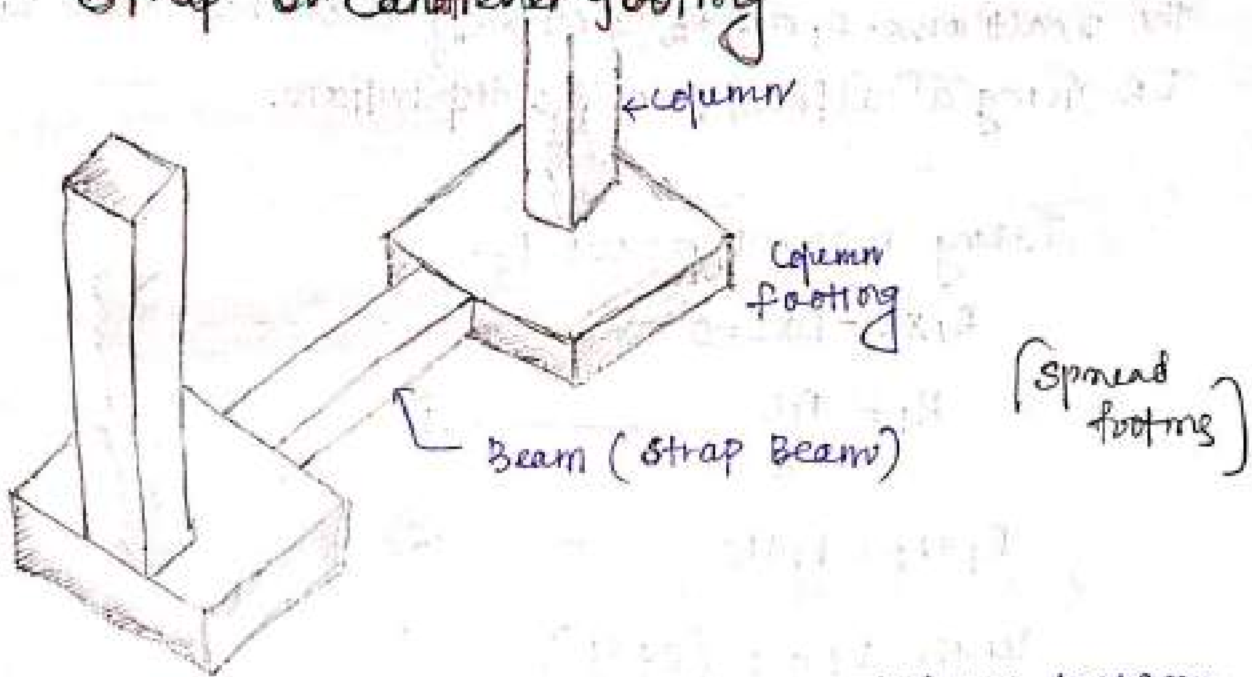
or $b = 4.917a$.

Substituting this values in ①

we get $a = \frac{5.882}{5.917} = 0.994m$ and $b = 4.889m$.

Hence trapezoidal footing of size $a = 1m$; $b = 4.9m$
and $L = 6.4m$.

Strap or cantilever footing



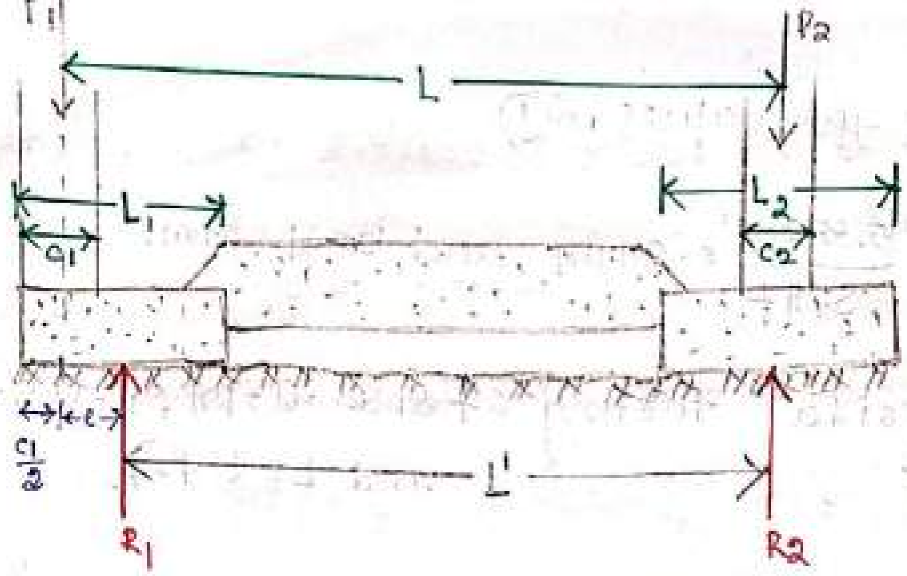
→ A strap footing may be used where the distance between the column is so great

→ ~~A combined trapezoidal footing becomes quite narrow with high bending moments. or where $\bar{x} < L/3$.~~

→ A strap footing consists of spread footing of two column connected by a strap beam.

→ the strap beam does not remain in contact with soil, and thus does not transfer any pressure to the soil,

→ the strap, assumed to be infinitely stiff, serve to transfer the column loads onto the soil with equal and uniform soil pressure under both footings.



the reactions R_1 and R_2 act centrally over the length L_1 and L_2 giving a uniform pressure distribution.

Taking moment about P_2.

$R_1 \times L' - P_1 \times L = 0$

$R_1 = \frac{P_1 L}{L'}$ ——— (1)

$R_1 + R_2 = P_1 + P_2$ ——— (2)

length $L_1 = 2\left(e + \frac{c_1}{2}\right)$

width of footing $B = \frac{R_1}{L_1 q_s}$

upward = down
R_1 = Area × q_s
[∴ R_1 = Area × q_s
= $B_1 \times L_1$

Knowing L, L_1 and L_2, L' is known.

Hence R_2 and R_1 are calculated from equation (1) and

But R_1 is also ~~eq~~ equal to $q_s \times L_1 \times B$ and $R_2 = q_s \times L_2 \times$

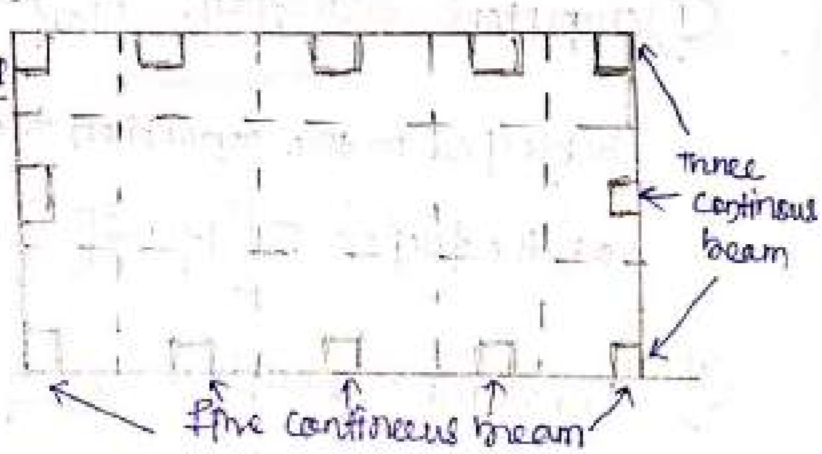
Hence the calculation are repeated with another value of e till values R_1 and R_2 calculated by both the procedures coincide.

After knowing the final values of the various component, the shear and moment in the strap are determined, and the strap is designed to withstand the shear and moments.

The footing are assumed to be subjected to uniform soil pressure and designed as simple spread footings.

Mat or raft footing

→ A raft or mat is a combined footing that covers the entire area beneath a structure and supports all the walls and columns.



→ when the soil allowable soil pressure is low, or the building loads are heavy, the use of spread footing would cover more than one-half (1/2) of the area and it may prove more economical to use mat or raft foundation.

→ they are also used where the soil mass contains compressible lenses or the soil sufficiently erratic so that the differential settlement would be difficult to control.

→ the mat or raft tends to bridge over the erratic deposits and eliminates the differential settlement.

→ Raft foundation is also used to reduce settlement above highly compressible soils, by making the weight of structure and raft approximately equal to the weight of the soil excavated.

→ the weight of raft is not considered in the foundation design because it is assumed to be carried directly by the subsoil.

→ Since this method doesnot take into account moments and shear caused by differential settlements,

the net ultimate bearing capacity may be determined from the following expression

(i) Skempton's equation: $q_{nf} = 5\left[1+0.2\frac{D}{B}\right]\left[1+0.2\frac{B}{L}\right]c$

subjected to the condition that $D/B \leq 2.5$

or the factor $5\left[1+0.2\frac{D}{B}\right]$ Does not exceed (or) (<) 7.5

(ii) Terzaghi's equation: $q_{nf} = 5.7\left[1+\frac{0.3B}{L}\right]c$

q_{nf} c = cohesion = $q_u/2$

the net pressure intensity can be calculated from the following equation for raft having width (B) greater than 6m.

$$q_p = 20\,(N-3)\,R_{w2}\ \text{kN/m}^2$$

N = Penetration resistance, should be taken at 75cm intervals for depth equal to width of raft, below the base of the raft.