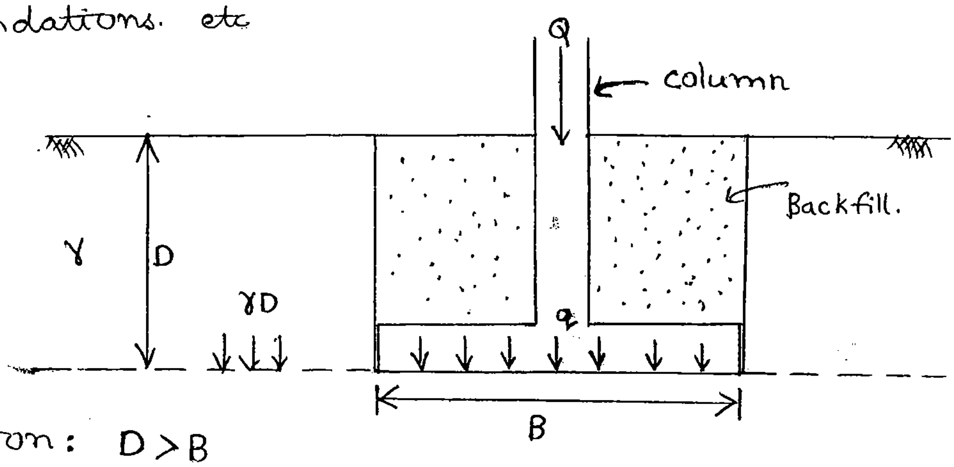
25th sept,
THURSDAY

16. BEARING CAPACITY

* Shallow Foundation : $D \leq B$

  Eg: Spread footings
  Raft foundations. etc



* Deep Foundation : $D > B$

  Eg: Pile foundation
  Well foundation.

* Original overburden pressure due to self weight of soil $= \gamma D$

Gross Pressure $= q$

Net Pressure, $q_n = q - \gamma D$

* Gross Ultimate BC of soil or Ultimate BC of soil $\}$, q_u

  Min. gross pressure required to cause shear failure of soils

* Net Ultimate BC of soil, $q_{nu} = q_u - \gamma D$.

  Min. net pressure required to cause shear failure of soils.

* Net safe BC of soil, $q_{ns} = \frac{q_{nu}}{F}$ $(F = 3)$

* Gross safe BC of soil or Safe BC of soil } $q_s = q_{ns} + \gamma D$

* Net safe settlement pressure, q_{np}.

It is the max. net pressure which the ~~the~~ soil can carry without exceeding allowable settlement.

* Net Allowable BC of soil, q_{na} = Smaller of q_{ns} or q_{np}

$q_{ns} \rightarrow$ based on shear failure criteria

$q_{np} \rightarrow$ based on settlement criteria.

It is the net pressure at which soil neither fails in shear nor undergoes excessive settlement.

→ Condition to be satisfied for Design of Foundation

The external pressure on soil ≤ net allowable BC of so

$$\boxed{q_n \leq q_{na}}$$

* If footing is backfilled,

$$q_n \approx \frac{Q}{A}$$

where Q = column load.

A = area of footing

* If footing is not backfilled, (raft)

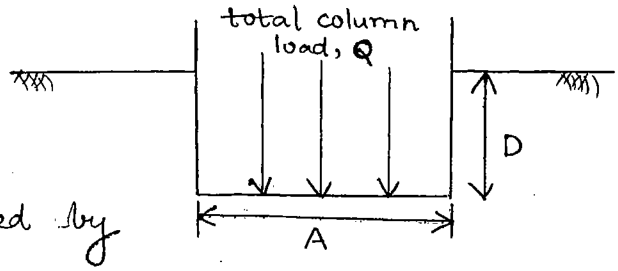
$$q_n \approx \frac{Q}{A} - \gamma D$$

It is based on the assumption that self weight of concrete is equal to unit weight of soil ($\gamma_c = 25$; $\gamma_s = 20$)

→ Compensated Raft Foundation. (Floating Raft)

$$q_n = \frac{Q}{A} - \gamma D.$$

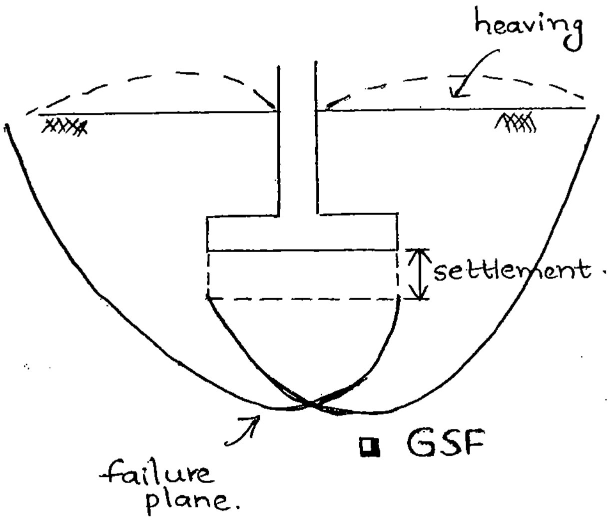
If $\gamma D = \frac{Q}{A}$; $q_n = 0$



Pressure applied is just balanced by pressure released.

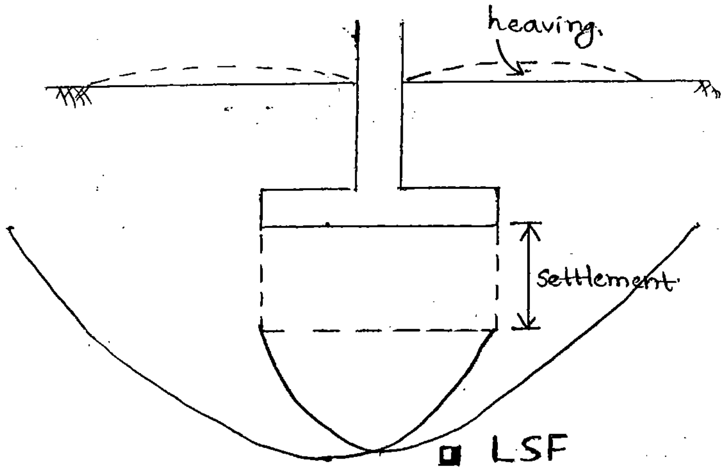
→ Types of Shear Failure:

1. General Shear Failure (GSF)
2. Local Shear Failure (LSF).
3. Punching Shear Failure (PSF).



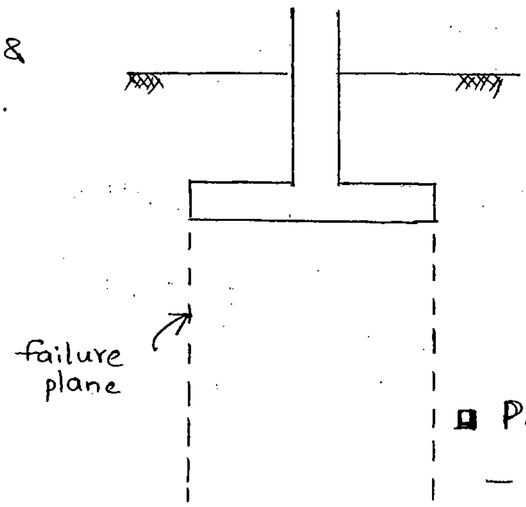
▪ GSF

– dense sand & stiff clays.



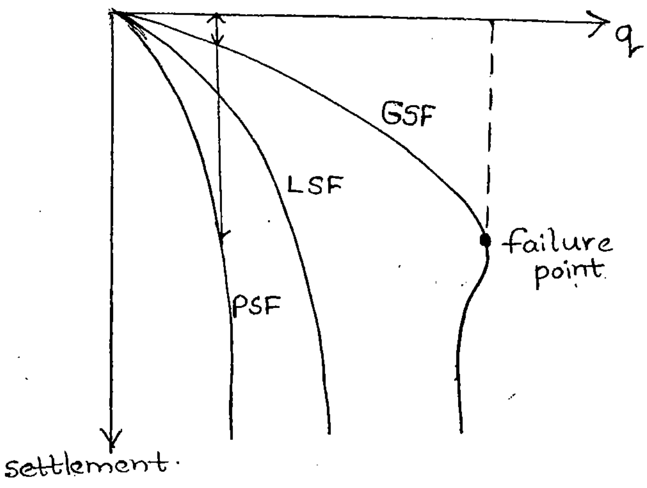
▪ LSF

– medium dense sand & medium clays.



▪ PSF

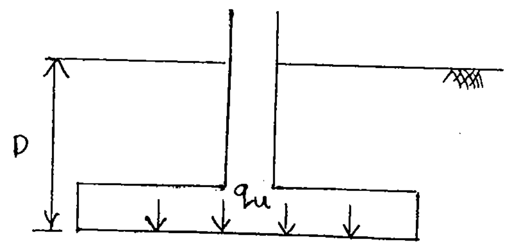
– loose sand & soft clays.



- For GSF, there will be a definite failure point.
- For the same load intensity, $(\text{Settlement})_{PSF} > (\text{settlement})_{GSF}$

→ Rankine's Theory

- Soil is cohesionless.
- Footing base is smooth.
- Plastic equilibrium.



$$q_u = \gamma D \left(\frac{1+\sin\phi}{1-\sin\phi}\right)^2$$

To avoid shear failure of soil, the min. depth of foundation required,

$$D_{min} = \frac{q}{\gamma}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2$$

$$\boxed{D_{min} = \frac{q}{\gamma} k_a^2}$$

However, as per this equation, as $D=0$, $q_u=0$; which is not possible. So this equation is not used to calculate bearing capacity.

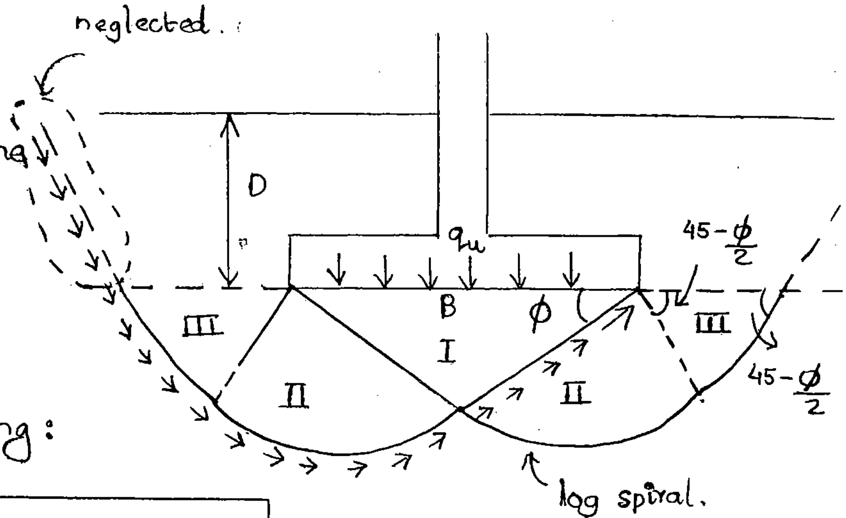
→ Terzaghi's Theory

- Footing base is rough.
- shallow foundation
- Continuous footing (Strip footing, $L >> B$)
- General Shear Failure.

Zone I : Elastic Zone

Zone II : Radial Shear Zone

Zone III : Rankine Passive Zone



* for Continuous footing:

$$\boxed{q_u = CN_c + \gamma D N_q + 0.5\gamma B N_\gamma}$$

$N_c, N_q, N_\gamma \rightarrow$ Bearing Capacity Factors of Soil. (depends on ϕ-value only)

ϕ	N_c	N_q	N_γ

If $\phi = 0$ (Pure Clay),

$N_c = 5.7$

$N_q = 1$

$N_\gamma = 0$

$\therefore$ For pure clay,

$$\boxed{q_u = 5.7C + \gamma D}$$

$$q_{nu} = q_u - \gamma D.$$

$$\Rightarrow \boxed{q_{nu} = 5.7C}$$

q_{nu} is independent of B & D of foundation for pure clay.

$$q_u = CN_c + \gamma D N_q + 0.5\gamma B N_\gamma$$

$$q_{nu} = q_u - \gamma D.$$

$$= CN_c + \gamma D(N_q - 1) + 0.5\gamma B N_\gamma$$

$$\Rightarrow q_{ns} = \frac{q_{nu}}{F}$$

$$\therefore \quad q_s = q_{ns} + \gamma D$$

* for circular footing

$$\boxed{q_u = 1.3\,CN_c + \gamma D N_q + 0.3\,\gamma B N_\gamma}$$

where, B → diameter of footing

* for square footing

$$\boxed{q_u = 1.3\,CN_c + \gamma D N_q + 0.4\,\gamma B N_q}$$

0.3, 0.5, 1.3, 0.4 are called 'shape factors'

* for rectangular footing

$$\boxed{q_u = \left(1 + 0.3\frac{B}{L}\right) CN_c + \gamma D N_q + \left(1 - 0.2\frac{B}{L}\right) 0.5\,\gamma B N_\gamma}$$

All the above equations are for GSF.

– For LSF, use C_m & ϕ_m to find BC of soil.

$$C_m = \frac{2}{3}C \quad \& \quad \tan\phi_m = \frac{2}{3}\tan\phi$$

$$\therefore\ q_u = C_m N_c' + \gamma D N_q' + 0.5\,\gamma B N_\gamma'$$

N_c', N_m', N_γ' are based on ϕ_m value.

– If $\phi > 36^\circ \Rightarrow$ GSF
  $\phi < 28^\circ \Rightarrow$ LSF

– If failure strain < 5% ⇒ GSF
  failure strain 10 to 20% ⇒ LSF

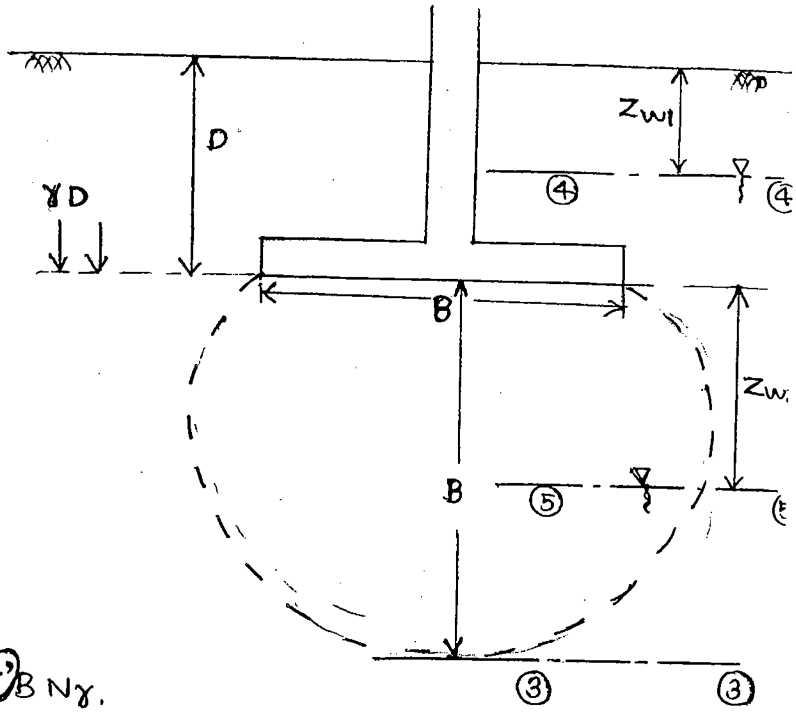
20th sept, SATURDAY

→ Effect of Water Table on Bearing Capacity of Soil:

$$q_u = \underset{\text{(I)}}{CN_c} + \underset{\text{(II)}}{\gamma D N_q} + \underset{\text{(III)}}{0.5\,\gamma B N_\gamma}$$

cohesion effect — Depth effect — width effect.

1. When water table is at or above GL



$$q_u = c'N_c + \gamma' D N_q + 0.5 \gamma' B N_\gamma$$

2. When water table is at footing level.

$$q_u = c'N_c + \gamma D N_q + 0.5 \gamma' B N_\gamma.$$

3. When water table is at a level ③-③.

⊙ No effect of water table..

4. When water table is at level ④-④

$$q_u = c'N_c + \gamma_a D N_q + 0.5 \gamma' B N_\gamma$$

$$\gamma_a = \frac{Z_{w1}\gamma + (D - Z_{w1})\gamma'}{D.}$$

5. When water table is at level ⑤-⑤

$$\gamma_a = \frac{Z_{w2} \times \gamma + (B - Z_{w2})\gamma'}{B.}$$

$$q_u = CN_c + \gamma D N_q + 0.5 \gamma_a B N_\gamma$$

* Approximate Method:

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

R_{w1} & R_{w2} → water table correction factors

$$R_{w1} = 0.5\left(1 + \frac{Z_{w1}}{D}\right) \qquad R_{w2} = 0.5\left(1 + \frac{Z_{w2}}{B}\right)$$

$$\boxed{0.5 \leq R_w \leq 1}$$

If WT is at GL:

$Z_{w1} = Z_{w2} = 0$

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

If WT at footing level:

$Z_{w1} = D$; $R_{w1} = 1$

$Z_{w2} = 0$; $R_{w2} = 0.5$

If WT is below the footing:

$R_{w1} = 1$ (no correction reqd.)

If WT is above footing:

$Z_{w2} = 0$

$R_{w2} = 0.5$

If WT is at a depth B below footing:

$R_{w1} = R_{w2} = 1$ (no correction reqd)

For cohesionless soils,

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

If WT is at or above GL, $R_{w1} = R_{w2} = 0.5$

$$q_u = \gamma D N_q \underbrace{(0.5)}_{R_{w1}} + (0.5 \gamma B N_\gamma)\underbrace{(0.5)}_{R_{w2}}.$$

∴ for cohesionless soils, bearing capacity reduces by 50% when WT raises to GL.

In case of cohesive soils, the effect of WT on the bearing capacity is negligible.

$$q_{nu} = 5.7\, C \quad \text{(no '}\gamma\text{' included)}$$

→ Skempton's Theory :

- For cohesive soils only ($\phi = 0$)

$$\boxed{q_{nu} = C N_c}$$

- For strip footing :

$$N_c = 5\left(1 + 0.2 \frac{D}{B}\right) \qquad \boxed{5.14 \leq N_c \leq 7.50}$$

- For rectangular footing :

$$N_c = 5\left(1 + 0.2 \frac{D}{B}\right)\left(1 + 0.2 \frac{B}{L}\right) \qquad \boxed{6.2 \leq N_c \leq 9}$$

→ Plate Load Test.

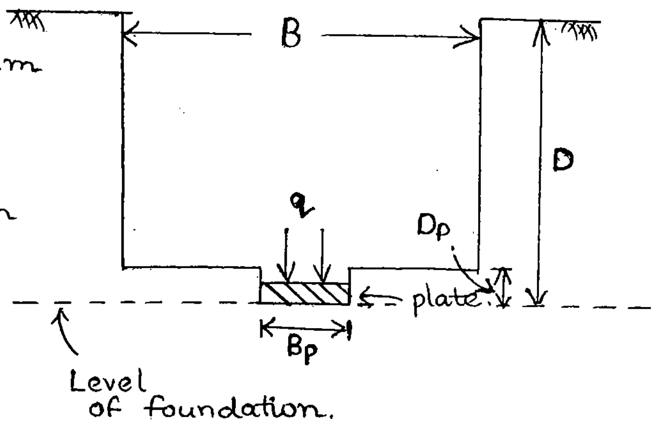
– to find BC and settlements.

* specifications:

Min. size of plate
= 30 cm x 30 cm

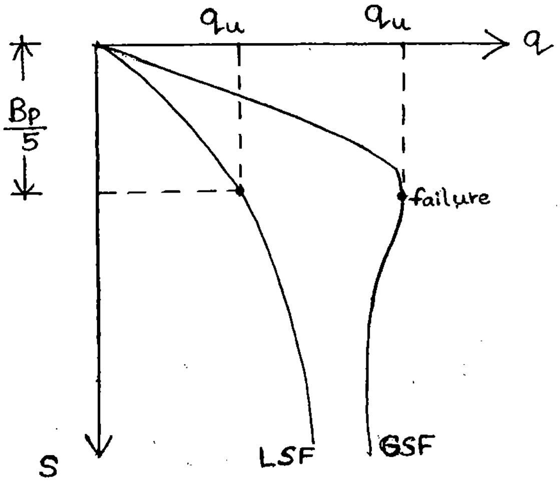
Max. size of plate
= 75 cm x 75 cm

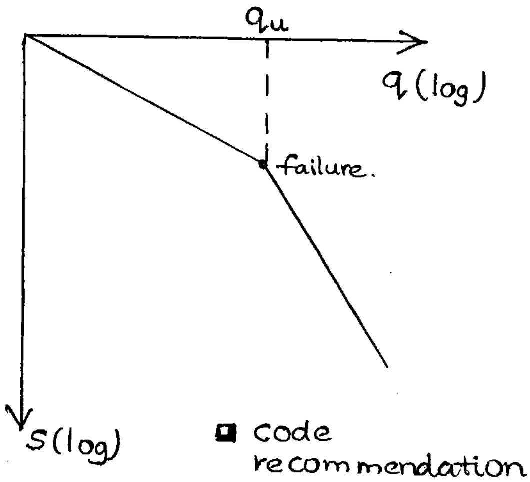
Min. thickness = 25 mm (1")



$$\frac{B}{B_p} = 5 \quad \& \quad \frac{D}{D_p} = 5$$

– Initially, a seating pressure of 7.5 kPa applied





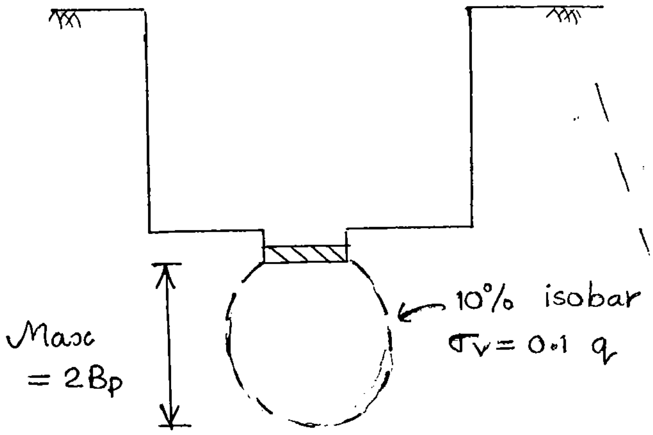
Safe bearing capacity, $q_s = \frac{q_u}{F}$

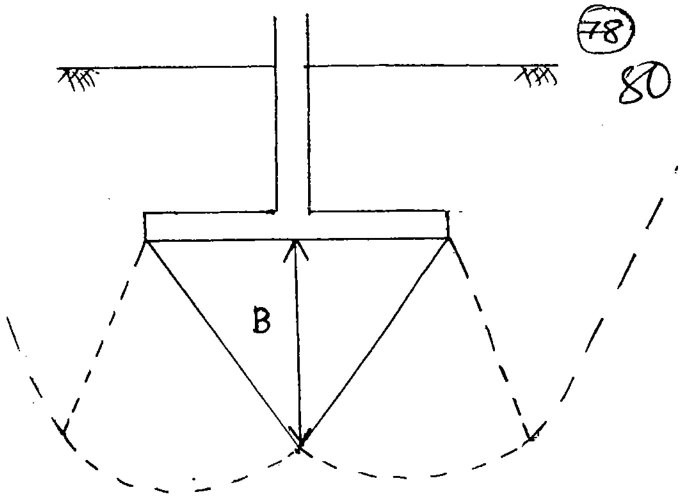
* Limitations of Plate Load test:

(i) It is a short duration test. Hence not reliable for pure clays. (consolidation settlement occurs for pure clays)

(ii) There is a width effect. (q_u depends on B in Terzaghi's theory & Skemptons theory)

(iii) Depth effect. (max. depth of pressure bulb in plate load test = $2B_p$)





→ Corrections for Plate Load test Results:

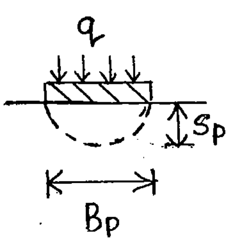
* Correction for Settlements

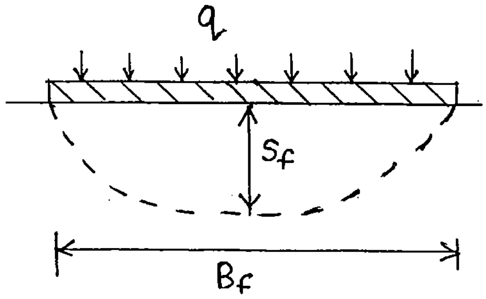
(i) For Clays

$$\frac{S_F}{S_P} = \frac{B_F}{B_P} \Rightarrow S \propto B$$

(ii) For sands.

$$\frac{S_F}{S_P} = \left[\frac{B_F\,(B_P + 0.3)}{B_P\,(B_F + 0.3)}\right]^2$$ B_F & B_P in <u>metres</u>





- q is same in both plate & footing

* Correction for Bearing Capacity

(i) For clays

$$q_f = q_p \quad (q_u \text{ independent of } B)$$

(ii) For sands

$$\frac{q_f}{q_p} = \frac{B_F}{B_P}$$

→ Meyerhof's Theory :

- For both shallow and deep foundations.

$$q_u = CN_c \underline{S_c d_c i_c} + \gamma D N_q . \underline{S_q . d_q . i_q} + 0.5 \gamma B N_\gamma . \underline{S_\gamma . d_\gamma . i_\gamma}$$

S → shape factor

D → depth factor

i → load inclination factor

* For (DL + LL) → BC obtained by formulae can be used directly.

For DL + LL + WL, DL + LL + EL → above BC is increased by 25%

→ Loads for Settlement Calculation :

* For sands : DL + LL + WL or EL

For clays : Permanent loads (DL + 50% LL)

→ Settlements

* Uniform Settlement : equal settlement everywhere.

* Differential Settlement : more detrimental to struct (additional moments are created)

Differential settlement ≈ 75% of total uniform settleme

* Permissible limits:

	Isolated Foundations	Raft Foundations. (differential settlements.)
Sand & Hard clay	50 mm	75 mm
Plastic clay (settlement occurs slowly for plastic clay)	75 mm.	100 mm.

01. $q_{ns} = \frac{1}{F}(CN_c)$; for clay.

Usually, $F \approx 3$. & $N_c = 5.7$ for clays.

$q_{ns} = \frac{1}{3}(c \times 5.7) \approx 2c \Rightarrow$ unconfined compressive strength.

02. $B = 3\,m$, $\gamma_{sat} = 20\ kN/m^3$, $\phi = 35^0$, $c = 0$, $N_q = 33$, $N_\gamma = 34.0$

$D = 2m$, $\gamma = 18\ kN/m^3$

(i). WT at GL

$q_u = 1.3\, c' N_c + \gamma' D N_q + 0.4\, \gamma' B N_\gamma$.

$= 0 + 10 \times 2 \times 33 + 0.4 \times 34 \times 3 \times 10 = \underline{\underline{1068}}$ kPa

(ii) WT at footing level

$q_u = \gamma D N_q + 0.4\, \gamma' B N_\gamma$.

$= 18 \times 2 \times 33 + 0.4 \times 10 \times 3 \times 34 = \underline{\underline{1596}}$ kPa

(iii) at 1 m below footing

$R_{w2} = 0.5\left(1 + \frac{1}{3}\right) = 0.666$

$q_u = 18 \times 2 \times 33 + 0.4 \times \overset{20}{\cancel{10}} \times 3 \times 34 \times 0.66$ {use accurate method}

$= \underline{\underline{\overset{1672}{\cancel{1460}}}}$ kPa

(iv) at 1 m below GL.

$\gamma_a = \dfrac{\gamma Z_{w1} + \gamma'(D - Z_{w1})}{D} = \dfrac{18 \times 1 + 10 \times (2-1)}{2} = 14$

$q_u = 14 \times 2 \times 33 + 0.4 \times 10 \times 3 \times 34 = \underline{\underline{1332}}$ kPa

03. Strip footing $\rightarrow$ WT at footing.

$q_{ns} = \frac{1}{F}\left(cN_c + \gamma D(N_q - 1) + 0.5\, \gamma' B N_\gamma\right)$.

For short term condition: use C_u & ϕ_u

For long term condition : use c' & ϕ'

a) Short term:

$$q_{ns} = \frac{1}{2}\left(80\times 6 + 16\times 1(1-1) + 0\right)$$

$$= \underline{\underline{240 \text{ kPa}}}$$

b) Long term:

$$q_{ns} = \frac{1}{2}\left(0\times 37.2 + \overset{16}{\cancel{20}}\times 1(22.5-1) + 0.5\times 10\times 2\times 19.7\right)$$

$$= \underline{\underline{270.5}}$$

4. For clays:

$$q_s = \frac{1}{F}(CN_c) + \gamma D$$

$N_c = 5.7$ for rough base (Terzaghi)

$= 5.14$ for smooth base (Prandtl)

$$C = \frac{1}{2}\times q_u = 10 \text{ t/m}^2.$$

$$q_s = \frac{1}{2}(10\times 5.14) + 2\times 1 = \underline{\underline{27.7}} \text{ t/m}^2$$

5. For design purpose, the condition to be satisfied:

$$q_n \leq q_{na}.$$

$q_{na} \rightarrow$ smaller of q_{ns} & q_{np}

Since q_{np} is not given, $q_{na} = q_{ns}$.

$$\therefore \quad q_n \leq q_{ns}$$

$$q_n = \frac{Q}{A} = \frac{1000}{B^2} \text{ kN/m}^2$$

$$q_{ns} = \frac{1}{F}\left(1.3\, CN_c + \gamma D(N_q-1) + 0.4\,\gamma B N_\gamma\right).$$

$$= \frac{1}{2.5}(0.4\times 19\times B\times 42).$$

$$\Rightarrow \quad \frac{1000}{B^2} = (0.4\times 19\times B\times 42)\times\frac{1}{2.5}$$

$$\Rightarrow B = 1.98\text{m} \approx \underline{\underline{2\text{m}}}$$

06. Since q_{np} not given,

$$q_n \leq q_{ns}.$$

$$\text{or} \quad q \leq q_s$$

$$q = \frac{300}{B \times 1} \ kN/m^2$$

$$q_n = q - \gamma D = \frac{300}{B} - 18 \times 1$$

$$q_{ns} = \frac{1}{F}(CN_c) = \frac{1}{3}(60 \times 5.7) = 114$$

$$\frac{300}{B} - 18 = 114.$$

$$\therefore \ B = \underline{\underline{2.27}} \ m$$

* For cohesionless soil → use ϕ to decide GSF & LSF

For cohesive soil. → use C to decide GSF & LSF

For $c-\phi$ soil → use <u>strain</u> to decide GSF & LSF

07 Elastic settlement, $S_i = \frac{q_n}{E_s} B(1-\mu^2) I.$

$$\boxed{S_i \propto q_n}$$

$$\therefore \ \frac{S_2}{S_1} = \frac{q_2}{q_1}$$

$$\frac{10}{25} = \frac{q_2}{7.2/0.3^2}$$

$$\Rightarrow q_2 = \underline{\underline{32}} \ t/m^2$$

08. $q_n = q - \gamma D.$

$$0 = 150 - 20D$$

$$\Rightarrow D = \underline{\underline{7.5 \, m}}$$

9. $B_P = 0.3\,m, \quad B_F = 1.5\,m$

$q_P = 6\ t/m^2, \quad q_F = ?$

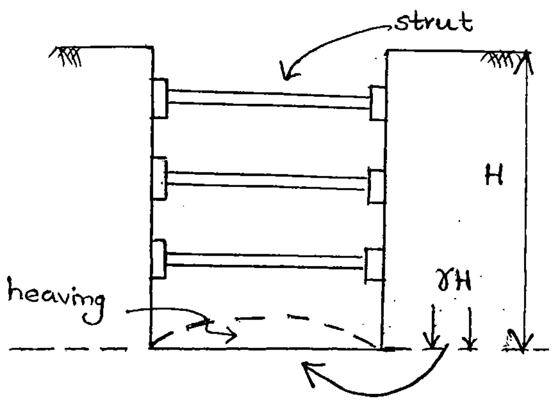
For sands,

$$\frac{q_F}{q_P} = \frac{B_F}{B_P}.$$

$$\frac{q_F}{6} = \frac{1.5}{0.3} \Rightarrow q_F = 30\ t/m^2.$$

Load = area x q_F = $1.5^2 \times 30$ = $\underline{\underline{67.5 \text{ tons}}}$

→ Braced Excavations – Heave Failure of Bottom



Factor of safety against heave failure, $F = \dfrac{CN_c}{\gamma H}$

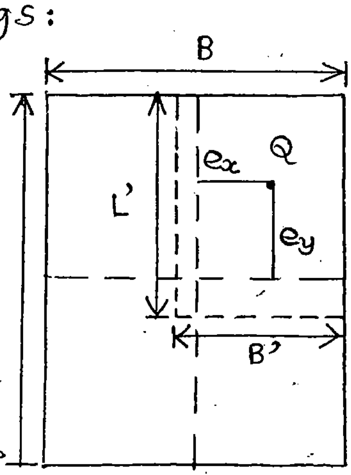
$N_c = 5.7$ (Terzaghi).

$= 5\left(1 + 0.2\dfrac{D}{B}\right)$; Skempton's theory

10. $N_c = 5\left(1 + 0.2 \times \dfrac{5}{2.5}\right) = \underline{\underline{7}}$ $\quad (D = H)$

$$FOS = \frac{CN_c}{\gamma H} = \frac{20 \times 7}{20 \times 5} = \underline{\underline{1.4}}$$

Bearing Capacity of Eccentric Footings:



For eccentric footings, modified dimensions (reduced) are to be taken to calculate the bearing capacity. The reduced dimensions are taken in such a manner that the load acting point should become the CG of modified dimensions

$$B' = 2\left(\frac{B}{2} - e_x\right) \quad \& \quad L' = 2\left(\frac{L}{2} - e_y\right).$$

Accordingly the modified dimensions B', L', A' are shown below:

$$B' = B - 2e_x$$

$$L' = L - 2e_y$$

$$A' = B'L'$$

$$\therefore \quad q_u = \left(1 + 0.3\frac{B'}{L'}\right) CN_c + \gamma D N_q + 0.5\gamma B' N_\gamma \left(1 - 0.2\frac{B'}{L'}\right)$$

Safe load capacity, $Q_{safe} = A' q_s$.