NAME- TANUJ GUPTA

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FOUNDATION ENGINEERING.

Classification of foundation

- Shallow foundation
  - $D_f \leq B$..
  - Raft/Mat
  - Footing
    - Isolated
      - Square
      - Circular
      - Rectangular
    - Continous
      - Ex Strip or Wall footing
    - Combined
      - Rectangular
      - Trapezoidal
      - Strap footing
- Deep foundation
  - $D_f > B$
  - Deep footing
  - Pile
  - Well/Caission
  - Pier foundation

Guidelines for the selection of foundation:-

(i) If Structural load is less and soil is medium to dense then shallow footing may be provided.

(ii) If column spacing is less and footing area is more than 50% of plinth area then either combined footing or Raft foundation may be provided.

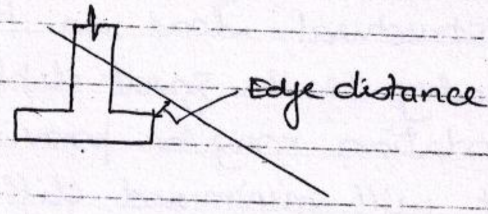
(iii) If structural load is heavy and soil is loose,- extanding to the great depth then either raft or pile foundation may be provided.

(iv). Raft will minimized differential settlement and provide Uniform distribution of pressure over the weak pockets of soil

(iv) If soil is expansing such as black soil then either Under reamed piles or floating foundation/Balancing foundation may be provided. (Balancing Foundation is a raft foundation in which weight of soil excavated is equal to weight of foundation and superstructure

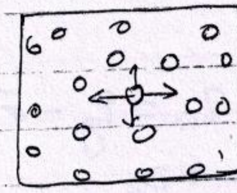
(v) In running water such as rivers and streams to support heavy load, well foundation should be provided.
(vi) to prevent failure due to uplift pressure/swelling pressure Under reambed piles should be provided.
(vii) In loose soils compaction piles should be provided.
(viii) If top 2-3 m soil is loose and then followed by dense soil then deep footing may be provided.

Guidelines for minimum depth of foundation ('As per IS codes)
(i) The depth should be always greater than depth of frost action.
(ii) To prevent erosion due to running water, a minimum depth of 30 cm for single and double story buildings and min. of 60 cm for multistory buildings is recommended.
(iii) The foundation depth should be always greater than depth of organic fill.
(iv) In slopping ground and hilly areas the minimum edge distance should be 60 cm in rocks and 90 cm in soils is recommended.



Note :- (i) If piles are driven in soft clays then due to driving (dynamic action) clay may get remoulded and shearing strength may be lost. therefore the structure should not be loaded immediately after driving the piles.
(ii) If piles are to be driven in dense soils then a pile driving process should start from centre and it should proceed radially outward Hence

in such case the difficulty in pile driving will be less.



(I) Shallow foundation:-

There are following two criteria of design.

(i) Shear Criteria:- The safe pressure which is permitted at the base of foundation without danger of shear failure is called safe bearing capacity.

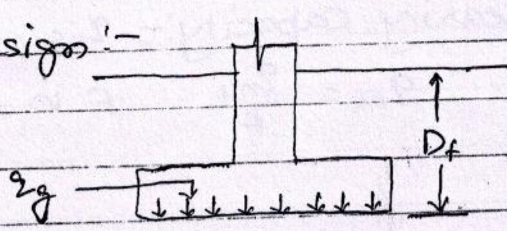
(ii) Settlement criteria:- The safe pressure which is permitted at the base of foundation without exceeding permissible value of settlement is called safe bearing pressure or allowable bearing pressure.

The total permissible pressure at the base of foundation should be smaller of the

(i) Safe bearing capacity of soil

(ii) Safe bearing pressure on the soil.

Shear Criteria of Design:-



(i) Gross Pressure (q_g):-

It is the total pressure at the base of foundation due to self weight of soil, self weight of footing and applied load on the footing.

(ii) Net pressure (q_n):- It is the pressure at the base of foundation in excess to the initial effective over burden pressure

$$q_n = q_g - \bar{\sigma}$$

$$\bar{\sigma} = \gamma D_f$$
$$= \gamma' D_f \text{ --- Under Water table effect.}$$

3) Ultimate Bearing capacity (q_u):- It is the maximum pressure at the base of foundation which can be applied without shear failure. If pressure exceeds then Ultimate bearing capacity, the soil will fail in shear.

4) Net Ultimate Bearing Capacity (q_{nu}):-

$$q_{nu} = q_u - \bar{\sigma}$$

$\bar{\sigma}$ initial effective overburden pressure

It is the net pressure at the base of foundation in excess to initial effective overburden pressure at which soil may just fail in shear.

5) Net safe Bearing capacity:- q_{ns}

$$q_{ns} = \frac{q_{nu}}{F} \quad F \text{ is } 2.5 \text{ to } 3.0$$

6) Safe Bearing capacity (q_{safe}):-

$$\boxed{q_{safe} = q_{ns} + \bar{\sigma}}$$

$$q_{safe} = \frac{q_{nu}}{F} + \bar{\sigma}$$
$$= \frac{q_u - \bar{\sigma}}{F} + \bar{\sigma}$$

Note that (i) The factor of safety is applied to the Net increment of pressure at the base of footing overburden pressure acts since historical ~~prio~~ period.

2) The ~~pre~~ permissible pressure at the base of foundation should be less than or equal to safe bearing capacity of soil.

Safe load on the foundation

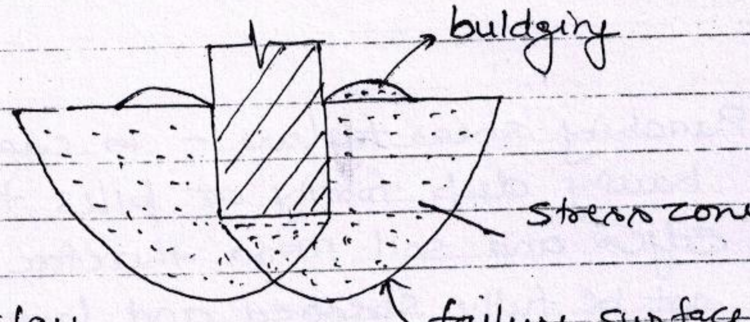
$$\boxed{Q_{safe} = q_{safe} \times Area}$$

Factors affecting Bearing Capacity:-

(i) position of Ground Water table with respect to width and depth of footing.
(ii) Type of soil and its physical and Engineering properties
(iii) Type of foundation and its dimensions (Shape, size and depth)
(iv) Initial stresses on the soil if any.

Types of shear failure:-

(i) General shear failure



If soil is medium to dense sand or stiff clay then soil fails in General shear. In such case there is no excessive settlement. At the time of failure soil reaches into plastic state. The foundation at failure get tilted and buldging of soil may occur at the surface.

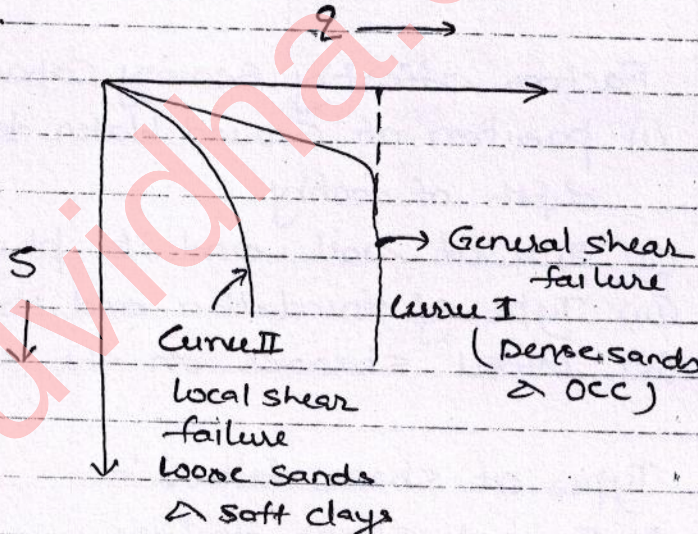
(ii) local shear failure :- If soil is loose sand or Soft clay then large settlement may occur below the foundation before soil reaches in plastic equilibrium

There may be little or no bulging at the sides. If soil fails in local shear then modified values of cohesion and friction may be used. (c' & ϕ')

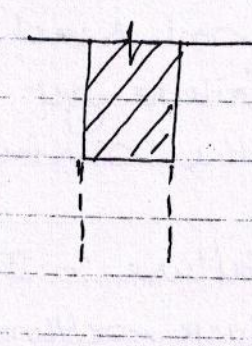
$$c' = \frac{2}{3}c$$

$$\phi' = \tan^{-1}\left(\frac{2}{3}\tan\phi\right)$$

$$\tan\phi' = \frac{2}{3}\tan\phi$$



Punching shear failure :- In case of very loose soils having deep footing or piles the soil gets sheared from edges and soil mass therefore adjacent soil mass will not be fully sheared and large settlements will be recorded without change in shear characteristics of adjacent soils.



Guidelines to differentiate General and Local shear failure.

Parameter	General shear failure	Local shear failure
(a) **Sands.**		
(i) friction angle	More than 36° (>36°)	less than <28°
(ii) Strain at failure	< 5%	> 15%
(iii) SPT (standard penetration test) (N)	> 30	< 5
(iv) Relative density	> 70%	< 20%
(v) Void ratio	< 0.55	> 0.75
Clays.		
(i) Unconfined Comp. strength q_u	> 100 kN/m²	< 80 kN/m²
Load settlement curve	Curve I	Curve II

Method to find Bearing Capacity :-

(i) Analytical Methods:-

(ii) Codal Provisions :- The various building codes such as National Building codes, BIS, IRC, CPWD have published bearing capacity of zonal soils. This can be used for rough computation

(iii) Field Methods:-

(i) Plate load test

(ii) Standard penetration test

(iii) Cone penetration test.

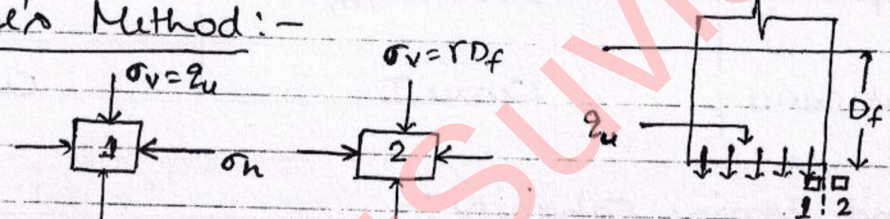
1) Analytical Methods:-

- Elastic theory
  - Schleicher's Method
- Classical Earth pressure theory
  - → Rankine's Method [φ soils]
  - → Pauker's Method [φ soils]
- Plastic theory
  - → Fellinius
  - → ~~Prandtl~~
  - → Terzaghi
  - → Skempton

Plastic theory

→ Fellinius Method [C- soils]
→ Prandtl Method [C & ϕ]
→ Terzaghi [C & ϕ]
→ Skempton [C soil]
→ Muyorhoff [C & ϕ]
→ Brinch-Hansen [C & ϕ]
→ Vesic's Method [C & ϕ]
→ Is code [C & ϕ]
→ Balla's Method [C & ϕ].

Rankine's Method:-



only for ϕ soils.

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \tan^2(45 - \phi/2)$$

$$K_p = \frac{1+\sin\phi}{1-\sin\phi} = \tan^2(45 + \phi/2)$$

This theory is application for frictional soils only.
Rankine considered equilibrium of two elements ~~aj~~adjascment to each other. one just below the footing at the corner (Element 1) and the other adjaceent to it but outside the footing (Element 2). and he applied the concept of passive earth pressure for element 2.
For element 2

$$\sigma_h = K_p \sigma_v$$
$$= \tan^2(45 + \frac{\phi}{2}) \gamma D_f$$

element 1

$$\sigma_h = k_a \sigma_v$$
$$= \tan^2(45 - \phi/2)\, q_u$$

By equating

$$\tan^2(45+\phi/2)\,\gamma D_f = \tan^2(45-\phi/2)\, q_u$$
$$q_u = \left[\tan^4\left(45+\frac{\phi}{2}\right)\right]\gamma D_f$$
$$q_u = \left(\frac{1+\sin\phi}{1-\sin\phi}\right)^2 \gamma D_f$$
$$= N_\phi^2\, \gamma D_f$$

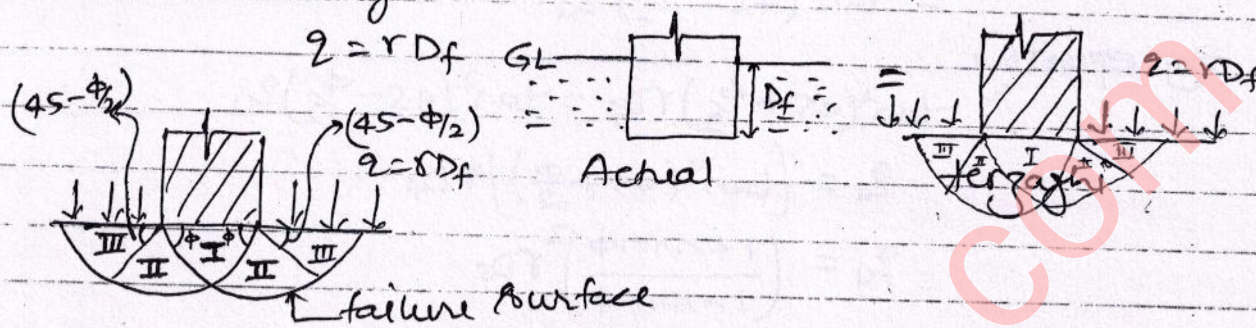
N_ϕ Influence Value

Limitations:- (i) Not applicable for clays.
(ii) If $D_f = 0$ (ie If footing is placed at ~~ga~~ ground level) then $q_u = 0$ which is not realistic
(iii) There is no effect of shape and size of footing on bearing capacity.

Fellenius Method:- It is applicable for purely cohesive soils only. The failure surface is assumed to be an arc of a circle and failure is considered general shear failure.

Terzaghi Theory:- It is an improvement over prandtl method Prandtl consider the base of footing to be smooth whereas terzaghi considered the base of footing to be rough.
Assumptions:- The foundation is shallow ($D_f \le B$)
(ii) Base of foundation is rough
(iii) Footing is continous (strip footing) it makes analysis two Dimensional. (2D)
(iv) Failure is general shear failure
(v) At the time of failure soil reaches into plastic stage
(vi) He replaced the soil above the base of foundation by an

Equivalent Surcharge



→ Terzaghi considered only base resistance and he ignored side ~~tog~~ resistance.

Terzaghi devided the soil below the footing in 3 stress Zones.

Zone I → It is called linear shear Zone or Central Zone. It remains in elastic Equilibrium as long as footing is considered rough whereas if footing is considered smooth then it will be in plastic Equilibrium.

Zone II :- (Radial shear Zone) :- It remains in plastic Equilibrium for pure clays the surface of this Zone is circular whereas for c, φ soils it is spiral.

Zone III :- Rankine's passive Zone :- It makes an angle of $(45-\phi/2)$ with the horizontal

Note :- Terzaghi considered the shear zone and failure surfaces extended upto foundation level only whereas in meyerhoff brinch hansen and vesic theory the stress Zone and failure surface is assumed to be extended upto Ground level therefore Terzaghi theory is applicable only for Shallow foundation whereas other theories can be applied for deep foundation also.

The Ultimate bearing capacity of strip footing (continous footing) is given by

$$q_u = CN_c + \gamma D_f N_q + \frac{1}{2}\gamma B N_\gamma$$

where C is Unit cohesion at the base of footing.
γ Unit weight of soil
N_c, N_q, N_γ are bearing capacity factors which depend upon friction angle of soil.
B is the width of footing.

$$N_\phi = \tan^2(45 + \phi/2)$$

$$N_q = N_\phi e^{\pi \tan\phi}$$

$$N_\gamma = 1.8 \tan\phi (N_q - 1)$$

$$N_c = \cot\phi [N_q - 1]$$

special case :- For pure clays

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

Note :- Prandtl eqn and terzaghi eqn is same but bearing capacity factor are little different
For pure clay according to prandtl

$$N_c = 5.14, \ N_q = 1, \ N_\gamma = 0$$

Modified Bearing capacity eqn for the other shape of footing
(i) for square footing.

$$q_u = 1.3 C N_c + \gamma D_f N_q + 0.4 \gamma B N_\gamma$$

(ii) for circular footing

$$q_u = 1.3 C N_c + \gamma D_f N_q + 0.3 \gamma D N_\gamma$$

D is dia of footing

3) For rectangular footing and raft foundation.

$$q_u = \left(1 + 0.3\frac{B}{L}\right)CN_c + \gamma D_f N_q + \frac{1}{2}\left(1 - 0.2\frac{B}{L}\right)\gamma B N_\gamma$$

$\boxed{\text{where } B < L}$

Note:- Ultimate bearing capacity of strip footing on purely cohesive soils.

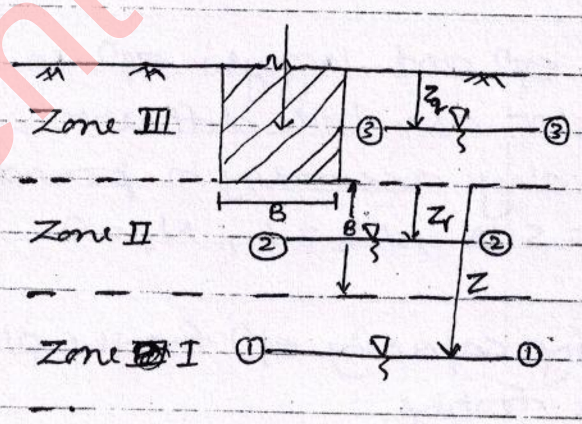
$$q_u = 5.7C + \gamma D_f \qquad N_c = 5.7,\ N_q = 1,\ N_\gamma = 0$$

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

it means the bearing capacity in clays is independent of the size of footing and net ultimate bearing capacity is independent of size and depth both.

Effect of Water table on the bearing capacity:-

$$q_u = \underbrace{CN_c}_{\text{I}} + \underbrace{\gamma D_f N_q}_{\text{II}} + \underbrace{\frac{1}{2}\gamma B N_\gamma}_{\text{III}}$$



Case 1 If water table is in Zone I, the depth of water table below the foundation level is greater then B. In this case there will be no effect on bearing capacity due to water table. because the effective stress zone usually does not extend in this zone. (Zone I)

Case 2 :- If water table is in zone II, let the depth of

$$0 \leq Z_r \leq B$$

In this case only IIIrd term of bearing capacity of eq^n will be affected where as the 2nd and 1st term will remain unaffected.

Either use effective unit weight in 3rd term or use water table correction factor (R_γ^*)

R_γ^* is given as

$$R_\gamma^* = 0.5\left[1 + \frac{Z_r}{B}\right]$$

$$0.5 \leq R_\gamma^* \leq 1.$$

Case 3:- If water table is in Zone III, in this case Zone II is submerged, hence R_γ^* is 0.5 and in 2nd and 1st terms also effect of water table will occure.

In 2nd term either use effective unit weight or use water table correction factor R_q^* which is given as

$$R_q^* = 0.5\left[1 + \frac{Z_q}{D_f}\right] \qquad 0.5 \leq R_q^* \leq 1$$

In Ist term use effective cohesion and effective values of bearing capacity factor.

Usually effect on c, & ϕ is negligible, therefore Ist term is negligible affected. If water table rises to Ground level (Neglecting effect on c, ϕ)

$$q_u = CN_c + \gamma' D_f N_q + \frac{1}{2}\gamma' B N_\gamma$$

$$\boxed{q_u = CN_c + \gamma D_f N_q R_q^* + \frac{1}{2}\gamma B N_\gamma R_\gamma^*}$$

at Ground level (water table)

$$R_q^* = 0.5 \qquad R_\gamma^* = 0.5$$

and γ_{sat} is used.

Special case 1:- In ~~f~~ cohesionless soils (sands) if watertable rises to Ground level then Ultimate bearing capacity is nearly reduced by 50% because γ' is nearly $\frac{1}{2}$ of γ_{sat}.

$$q_u = \gamma' D_f N_q + \frac{1}{2}\gamma' B N_\gamma$$

Special case 2:- For purely cohesive soils (clays) if water table rises to Ground level then

$$q_u = 5.7C + \gamma D_f$$

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

It means the effect of WT on clays is negligible and net ultimate bearing capacity is unaffected by water table in clays if c and ϕ are unchanged.

Note:- In terzaghi eqn general shear failure is considered but if soil fails in local shear then modified values of c and ϕ should be used

$$c' = \frac{2}{3}c \qquad \phi' = \tan^{-1}\left(\frac{2}{3}\tan\phi\right)$$

Skempton's Method:- It is applicable for purely cohesive soils only in which bearing capacity is independent of size. This method accounts for end bearing resistance and side shear resistance both, therefore it can be used for deep footings also

The net Ultimate bearing capacity is given by

$$q_{nu} = CN_c$$

N_c is bearing capacity factor which depends upon $\frac{D_f}{B}$ ratio

<u>Case 1</u>:- when $D_f/B = 0$ ie footing is placed at Ground surface

$N_c = 5.0$ for strip footing

$N_c = 6.0$ for square / circular / rectangular / Raft.

<u>Case 2</u>:- when ~~Df/B~~ $0 < D_f/B < 2.5$.

$$N_c = 5.0\left[1 + 0.2\frac{D_f}{B}\right] \text{ --- for strip footing}$$

$$= 6.0\left[1 + 0.2\frac{D_f}{B}\right] \text{ --- for square \& Circular}$$

$B = D$ for circle

$$N_c = 5.0\left[1 + 0.2\frac{B}{L}\right]\left[1 + 0.2\frac{D_f}{B}\right] \text{ — for Rectangular footing \& Raft Foundation}$$

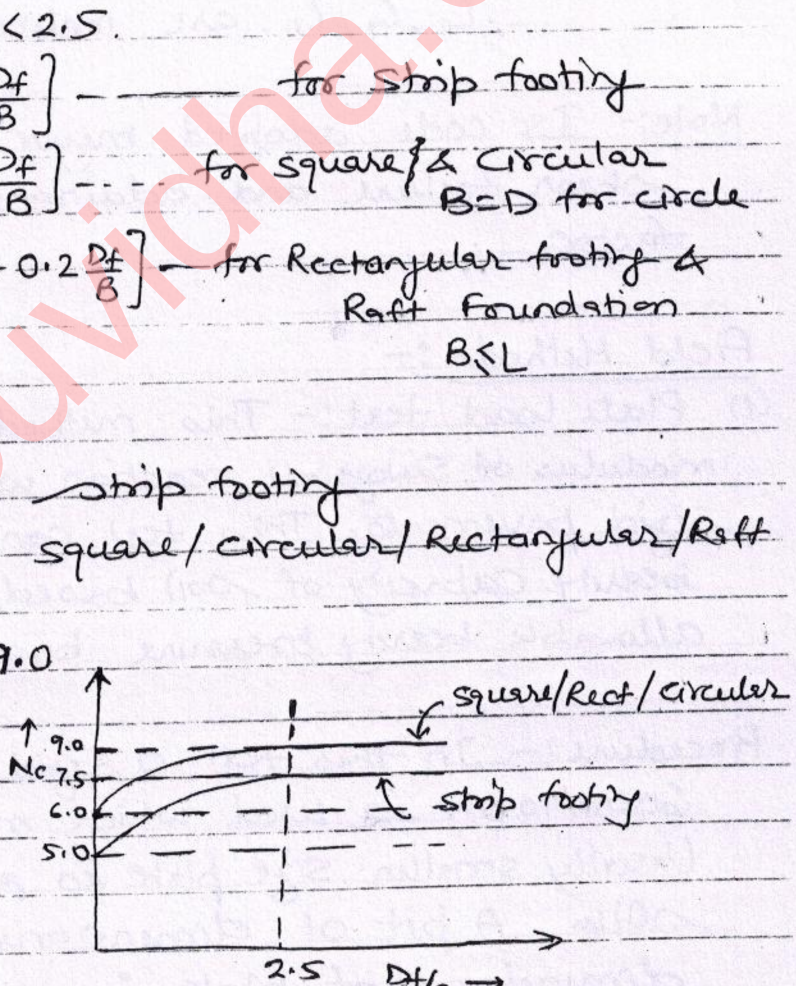
$B \leq L$

<u>Case 3</u>:- when $D_f/B \geq 2.5$

$N_c = 7.5$ for strip footing

$N_c = 9.0$ for square / circular / Rectangular / Raft

The max. value of N_c is 9.0



<u>Meyerhoff Method</u>:- This theory can be applied for shallow and deep footing both. Meyerhoff considered the failure surface and stress zone to be extended upto Ground level. He ~~[illegible]~~ accounted side shear resistance and end bearing resistance both.

He applied correction factor for shape, depth and inclination of the Ground surface.

$$q_u = CN_c S_c d_c i_c + \gamma D_f N_q S_q d_q i_q + \frac{1}{2} \gamma B N_\gamma S_\gamma d_\gamma i_\gamma$$

S_c, S_q, S_γ are shape correction factor
d_c, d_q, d_γ are depth correction factor
i_c, i_q, i_γ are inclination correction factor.

Note:- IS code adopted meyorhoff eqn with general shear failure and additional water table correction factor.

Field Methods:-

(1) Plate load test:- This method was designed to determine modulus of Subgrade reaction which is used in design of rigid pavements. This test can also be used to find bearing capacity of soil based on shear criteria and allowable bearing pressure based on settlement criteria.

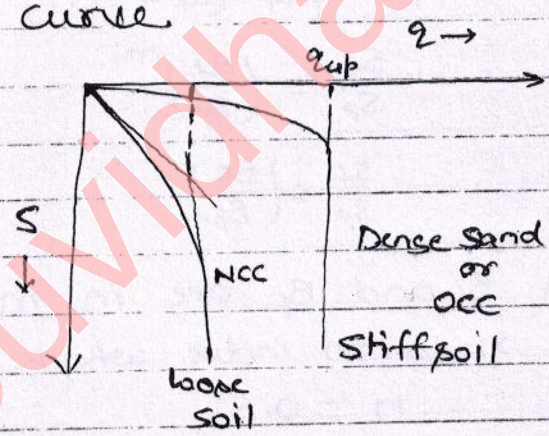
Procedure:- In this test a rigid plate of size 30cm/45cm/60cm/90cm is used which may be circular or square. Usually smaller size plate is suitable for dense and ~~stiff~~ stiff soils. A pit of dimensions not less 5 times dimensions of plate is excavated at same depth equal to the depth of foundation and plate is seated at the centre of pit.

If water table is present above the foundation level then it must be lowered by pumping below the foundation level.

The load on the plate is applied through a Jacking mechanism and settlement of plate is recorded by 3 dial gauges attached at 120° to each other and avg. settlement is taken.

→ The settlement Vs load curve is plotted by taking Settlement in Y axis and pressure (Load / Area) in X axis.

→ The shear failure occurs at which plate starts settling at faster rate without much increase in pressure. The bearing capacity based on shear criteria is determine Using load settlement curve.



If soil is cohesive then there is no effect of size on bearing capacity. hence bearing capacity for footing equal to bearing capacity for plate.

$$q_{uf} = q_{up} \quad \text{for clay}$$

If soil is sand then bearing capacity is proportional to size of footing

$$q_{uf} = q_{up} \times \frac{B_f}{B_p} \quad \text{—— for sands}$$

B_f = size of footing

B_p = size of plate.

Procedure to determine allowable bearing pressure based on settlement criteria.

The permissible settlement of footing is given by IS code say (S_f). Using Empirical relations the corresponding permissible settlement of plate can be computed (say S_p)

$$\frac{S_f}{S_p} = \left[\frac{B_f}{B_p}\frac{(B_p + 0.3)}{(B_f + 0.3)}\right]^2 \text{ --- for Dense sand}$$

$$\frac{S_f}{S_p} = \left(\frac{B_f}{B_p}\right)^{n+1} \text{ --- for silts } [c-\phi]$$

$$\frac{S_f}{S_p} = \left(\frac{B_f}{B_p}\right) \text{ --- for clays } [c\text{-soils}]$$

where B_f and B_p are in meter.

n is an index which depends on type of c, ϕ.

$n = 0.5$.

S_p is computed from above relations and from load settlement curve allowable bearing pressure for plate is Computed. If soil is clay then same will be allowable bearing pressure for footing. but if soil is sand, then according to size allowable bearing pressure will be modified

Housel's Approach :- According to housel the Ultimate load carrying capacity of a plate or foundation at failure is function of its area and perimeter

$$Q = A \cdot m + Pn$$

where A is base area

and P is perimeter

and m, n are Constants which depend on type

If Q_1 is Ultimate load supported by a plate or footing having Area A_1 and Perimeter P_1 then

$$Q_1 = A_1 m + P_1 n \quad \text{—— (1)}$$

let Q_2 is load supported by another plate having area A_2 and perimeter P_2 placed on same soil.

$$Q_2 = A_2 m + P_2 n \quad \text{—— (2)}$$

then find the Load which can be supported by footing ~~are~~ having area A_3 and Perimeter P_3.

$$\boxed{Q_3 = A_3 m + P_3 n}$$

find m and n by solving (1) and (2).

Note:- when allowable ~~B~~ bearing pressure is determined using settlement criteria then additional factor of safety is not required because the permissible settlement given by IS codes are already after applying the factor of safety.

Standard Penetration test :-

This test can be used to determine

(i) Angle of shearing resistance
(ii) Relative density / density Index
(iii) Allowable bearing pressure on the basis of settlement criteria
(iv) Point resistance of pile.
(v) Confined Compressive Strength of cohesive soils.

Note: This test is suitable for Granular soils because in clays remoulding may occur.

Procedure:- (i) A split spoon sampler is used in a bore-hole in which dynamic approach is followed. The standard weight of hammer is 65kg and height of free fall is 75cm.

(ii) The test is conducted either at every 2m depth or at foundation level or at change of stratum.

(iii) The SPT No. is defined as no. of blows of hammer required for 300mm penetration on sampler. The penetration per blow of hammer is Called ~~set~~ SET.

(iv) Using test is performed in three stages 150 mm penetration each. and SPT No. is taken as No. of blows required for last 300mm penetration.

The observed value of SPT No is subjected to following two corrections in sequence.

(i) Overburden pressure Correction.

(ii) dilatancy Correction/water table correction.

(i) Overburden pressure Correction:- Let N_0 is observed value of SPT No. then let N_1 is corrected value for overburden pressure then

$$N_1 = N_0 \times \frac{350}{\bar{\sigma} + 70}$$

Where $\bar{\sigma}$ is effective overburden pressure at the test level in kN/m^2. $\bar{\sigma} \leq 280\, kN/m^2$

If $\bar{\sigma}$ is greater than 280 then this correction is not required.

2) dilatancy Correction :- If Water table is present at or above the test level then water table Correction

Let N_2 is corrected value for dilatancy. Let N_1 is correcte value for over burden pressure then

$$N_2 = 15 + \frac{1}{2}(N_1 - 15)$$

~~This~~ if test is conducted at more than 1 level then final SPT No. is avg. of corrected values.

Procedure to determine allowable bearing pressure / safe bearing pressure using SPT test Data and permissible settlement.

(I) Pack Hansen Eqn :- The net allowable bearing pressure in kN/m^2 is given as

$$q_{anet} = 0.41\, N S C_w \text{ -- } kN/m^2$$

where N final SPT No. and S permissible settlement in mm, and C_w water table correction factor which is given as

$$C_w = 0.5\left[1 + \frac{D_w}{B + D_f}\right]$$

where D_w depth of water table from Ground level B width of footing and D_f depth of footing.

(II) Teng's Eqn :- The net allowable bearing pressure or net safe bearing pressure in kN/m^2 is given as

$$q_{netsafe} = 1.4(N-3)\left[\frac{B+0.3}{2B}\right]^2 S \cdot C_w C_D$$

where C_w is water table correction factor defined as $C_w = 0.5\left[1 + \frac{D_w}{B}\right]$

where D_w is depth of water table below foundation

C_D depth correction factor and given as

$$C_D = 1 + \frac{D_f}{B} \quad (\leq 2) \text{ for shallow footing}$$

iii) **IS Code Method:**– IS code has adopted the terzaghi's Eqn with small modification and without depth correction factor. The Net safe bearing pressure in kN/m² is given as

$$q_{net_{safe}} = 1.38(N-3)\left[\frac{B+0.3}{2B}\right]^2 S\, C_w$$

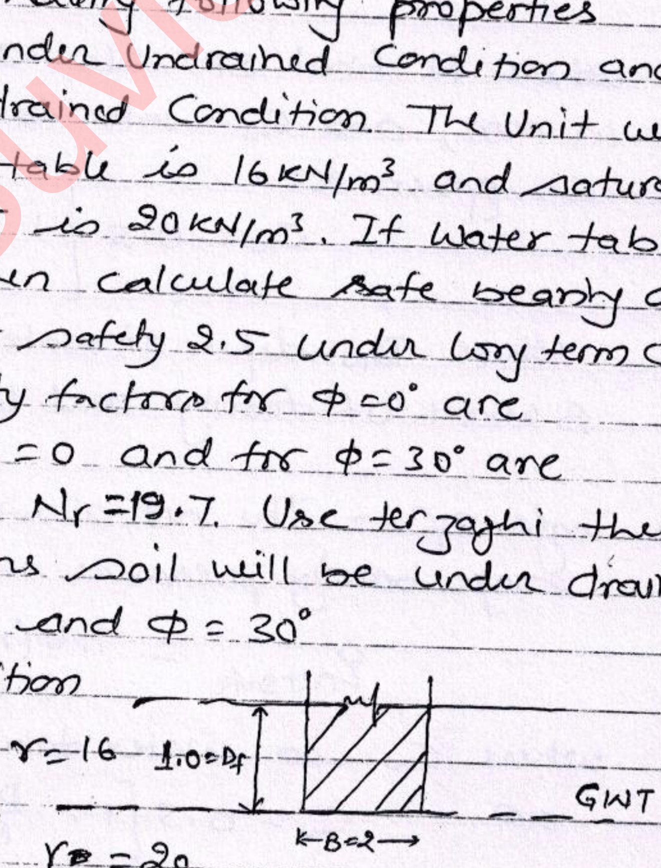
$$= 0.88\, N S\, C_w \quad \text{for Raft}$$

$$\simeq 21N \quad \text{for raft in sand.}$$

Ques 1) 2m wide strip footing is placed 1m below the ground level of a clay having following properties $C = 80\,kN/m^2$ and $\phi = 0°$ under Undrained Condition and $C = 0$ and $\phi = 30°$ under drained Condition. The Unit weight of soil above the water table is $16\,kN/m^3$ and saturated Unit weight below the WT is $20\,kN/m^3$. If water table is at foundation level then calculate safe bearing capacity of footing using factor of safety 2.5 under long term Condition. Given that bearing capacity factors for $\phi = 0°$ are
$N_c = 5.7$, $N_q = 1.0$, $N_\gamma = 0$ and for $\phi = 30°$ are
$N_c = 37.2$, $N_q = 22.5$, $N_\gamma = 19.7$. Use terzaghi theory.

Ans In long term conditions soil will be under drained Condition. Hence Use $c = 0$ and $\phi = 30°$

Under Long term Condition
$C = 0$, $\phi = 30°$



$N_c = 37.2$, $N_q = 22.5$
$N_\gamma = 19.7$

$$q_u = \cancelto{0}{cN_c} + \gamma D_f N_q + \frac{1}{2}\gamma' B N_\gamma$$

$$\gamma' = \gamma_{sat} - \gamma_w = 20 - 9.81 = 10.19$$

$$q_u = 16\times1\times22.5 + \frac{1}{2}[10.19]\times2\times19.7$$

$$q_u = 560.7$$

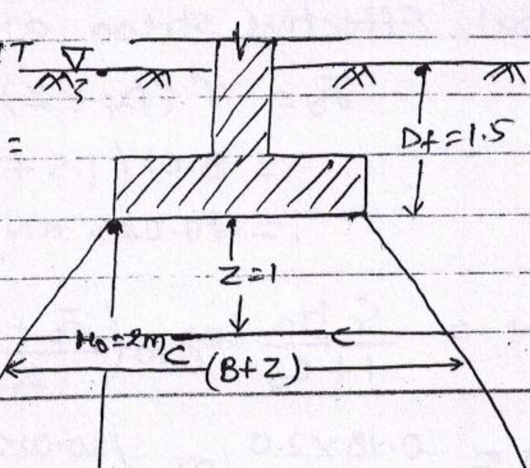
$$q_{nu} = q_u - \gamma D_f$$
$$= 560.7 - 16\times1$$
$$q_{nu} = 544.7.$$

$$q_{ns} = \frac{q_{nu}}{F} = \frac{544.7}{2.5} = 217.8$$

$$q_{safe} = q_{ns} + \gamma D_f$$
$$= 217.8 + 16\times1 = 233.8\ kN/m^2$$

Ques 2m square footing is resting on a clay having depth of 1.5m from Ground level. The total thickness of clay deposit is 3.5m which is underlaid by a firm sand. The clay has following properties Liquid limit = 30% specific gravity of solid = 2.67, S = 1, Natural water content = 40%, cohesion c = 0.45 kg/cm², $\phi = 0$.
find the safe bearing capacity of the soil at the base of footing using Skempton's approach. Adopt FOS = 3.0.
If a load is applied on the footing having pressure at the base of footing equal to the safe bearing capacity of soil then find the settlement of the footing. Assume load to be distributed below the footing at 2V:1H. The water table is located quite close to Ground level.

Ans $Se = wG$



$$\gamma' = \left(\frac{G-1}{1+e}\right)\gamma_w$$

$$\gamma' = \left(\frac{G-1}{1+\frac{wG}{S}}\right)\gamma_w$$

$$\gamma' =$$

$$= \left(\frac{2.7-1}{1+0.4\times2.7}\right)\times9.81$$

$\gamma' = 8.01\ \text{KN/m}^3$

$C = 0.45\ \text{kg/cm}^2$

$= \dfrac{0.45 \times 9.81 \times 10^{-3}}{10^{-4}}\ \text{KN/m}^2 = 44.1\ \text{KN/m}^2$

$q_{nu} = C N_c$

$N_c = 6.0\left[1 + 0.2\dfrac{D_f}{B}\right]$

$= 6.0\left[1 + 0.2 \times \dfrac{1.5}{2}\right]$

$= 6.9$

$q_{nu} = 44.1 \times 6.9$

$q_{ns} = \dfrac{q_{nu}}{F} = \dfrac{44.1 \times 6.9}{3.0} = 101.43\ \text{KN/m}^2$

$q_{safe} = q_{ns} + \gamma' D_f$

$= 101.43 + 8.01 \times 1.5$

$\boxed{q_{safe} = 113.44\ \text{KN/m}^2}$

$\Delta\bar{\sigma} = \dfrac{q_{safe} \times B \times B}{(B+Z)(B+Z)} = \dfrac{113.44 \times 2 \times 2}{(2+1)(2+1)} = 50.41\ \text{KN/m}^2$

Initial Effective stress at C-C

$\bar{\sigma}_0 = \gamma'(D_f + Z)$

$= 8.01(1.5 + 1)$

$= 20.025\ \text{KN/m}^2$

$\Delta H = \dfrac{C_c H_0}{1+e_0} \log_{10}\left(\dfrac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$

$C_c = 0.009(w_L - 10)$

$= 0.009(30 - 10)$

$\boxed{C_c = 0.18}$

$\Delta H = \dfrac{0.18 \times 2.0}{\ } \log \left(\dfrac{20.025 + 50.41}{\ }\right)$

Ques A square footing located at a depth of 1.3m below the Ground surface has to carry a safe load of 800 kN. Find the size of the footing if the desire factor of safety is 3.0 The soil has following properties
$e = 0.55$, $S = 0.5$, $G_s = 2.67$, $C = 8\,kN/m^2$, $\phi = 30°$
Bearing capacity factor for $\phi = 30°$ are $N_c = 37.2$, $N_q = 22.5$ and $N_\gamma = 19.7$.

Ans let us solve by Terzaghi theory
let the size of footing is $B \times B$ m.

$$Q_{safe} = \text{Safe Bearing capacity} \times \text{Area}$$

$$800 = q_{safe} \times \text{Area} \quad \text{—— (1)}$$

$$\gamma = \left(\frac{G + Se}{1+e}\right)\gamma_w = 18.64\,kN/m^3$$

for square footing

$$q_u = 1.3\,C N_c + \gamma D_f N_q + 0.4\,\gamma B N_\gamma$$

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

$$q_u = 932.1 + 146.8\,B$$

$$q_{nu} = q_u - \gamma D_f$$

$$= 932.1 + 146.8B - 18.64 \times 1.3$$

$$q_{nu} = 907.8 + 146.8B$$

$$q_{ns} = \frac{q_{nu}}{F} = 302.6 + 48.93\,B$$

$$q_{safe} = q_{ns} + \gamma D_f$$

$$= 302.6 + 48.93B + 18.64 \times 1.3$$

$$q_{safe} = 326.8 + 48.93B \quad \text{—— (2)}$$

from eqn (1)

$$800 = (326.8 + 48.93B)\,B^2$$

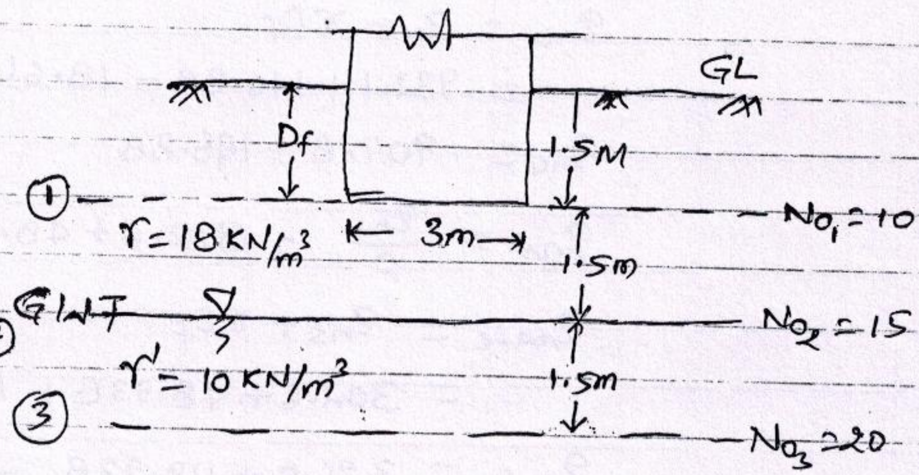
$$\boxed{B = 1.42\,m}$$

Ques. A building has to be supported over a Raft foundation of dimensions 14m x 21m. The soil is a clay having unconfined compressive strength of 15 KN/m². The ~~per~~ safe pressure at the base of Raft on the soil is 140 KN/m². The building has provision of Basement floors. At what depth raft should be placed to ensure factor of safety equal to 3 against shear failure. Use $\gamma_{clay} = 19\ KN/m^3$ adopt Skempton approach. Recommend No. of Basement floor.

Ans $D = 6.57$ meter   2 Basement floor.

Ques. Using ~~Peck~~ henson Method find Net safe bearing pressure. The Data are shown in fig. the permissible settlement of footing is 40mm. The SPT test is conducted at 1.5m interval as shown in fig and observed values of SPT No. are 10, 15, 20, 25 at a depth of 1.5m, 3.0m, 4.5m respectively. The unit weight of soil is shown in fig.

Given $D_f = 1.5m$



Corrected values of SPT for over burden pressure

$$N_o' = N_o \left(\frac{350}{\bar{\sigma} + 70} \right) \qquad \bar{\sigma} \text{ at}_1 = \gamma z$$

$$N'_{o_1} = 10 \times \left(\frac{350}{27+70}\right) = 36.08.$$

$$N'_{o_2} = N_{o_2} \times \left(\frac{350}{\bar{\sigma}+70}\right)$$

$$\bar{\sigma}_{at\,2} = \gamma z_2 = 18 \times 3 = 54$$

$$N'_{o_2} = 15 \times \left(\frac{350}{54+70}\right) = 42.3.$$

$$N'_{o_3} = 20 \times \left(\frac{350}{\bar{\sigma}+70}\right).$$

$$\bar{\sigma}_{at\,3} = \gamma z_1 + \gamma z_2 + \gamma' z_3$$

$$= 1.5 \times 18 + 18 \times 1.5 + 10 \times 1.5 = 69$$

$$N'_{o_3} = 20 \times \left(\frac{350}{69+70}\right) = 50.36.$$

Corrected values for Water table/Dilatancy

$N''_{o_1} = N'_{o_1}$ [No correction is required. Water table is below this level].

$$N''_{o_1} = 36.08.$$

$$N''_{o_2} = 15 + \frac{1}{2}(N'_{o_2} - 15)$$

$$= 15 + \frac{1}{2}(42.3 - 15)$$

$$= 28.665$$

$$N''_{o_3} = 15 + \frac{1}{2}(N'_{o_3} - 15)$$

$$= 15 + \frac{1}{2}(50.36 - 15)$$

$$= 32.68.$$

Final SPT No. is average of Corrected.

$$= \frac{36.08 + 28.665 + 32.68}{3} = 32.475 \simeq \underline{\underline{32}}$$

Peck Hm Eq^n

$$q_{anet} = 0.41 N S C_w$$

$$C_w = 0.5\left[1 + \frac{D_w}{B + D_f}\right]$$

$$= 0.5\left[1 + \frac{3}{3 + 1.5}\right] = 0.83$$

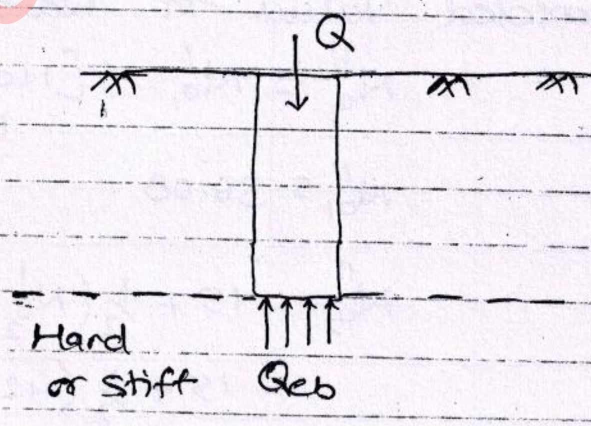
$$q_{anet} = 0.41 \times 32 \times 40 \times 0.83$$

$$= 435.05 \ kN/m^2$$

Pile foundation :-

(1) Types of piles on the basis of their action.

① End bearing pile

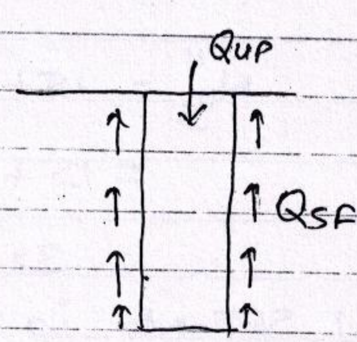


The load bearing capacity is due to base resistance

(ii) these piles rest over hard or stiff strata length of the pile depend upon depth of stiff strata.

$$Q = Q_{eb}$$

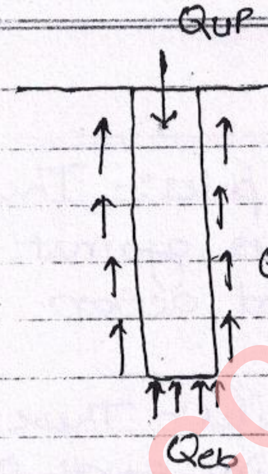
② Friction piles.



The load supporting power is mainly due to ~~frich~~ skin friction resistance. such piles are driven piles through soft stratum.

$$Q_{up} = Q_{sf}$$

③ bearing and friction piles



The load supporting power is jointly due to end bearing action and skin friction action

$$Q_{up} = Q_{sf} + Q_{eb}$$

Types of piles on the basis of Method of Installation

① Driven piles :- These are essentially precast piles which are either made of metal or good/wood. These are driven through dynamic action using drop hammer or steam hammer. In driven piles end bearing resistance and skin friction resistance both are developed.

② Bored piles :- These may be precast or cast insitu. These may be made of sand/Concrete/Wood/Metal.

Note :- Usually efficiency and load carrying capacity of driven pile is greater than Bored piles.

Types of piles on the basis of their function:
① Compaction piles :- These are used to compact loose sand in order to increase bearing capacity.

② Uplift piles/tension piles :- These are used to anchor the structures subjected to uplift pressure.

3) Fender piles:- These are used to protect water front structures against impact from waves created by earthquake or wind action

4) Sheet piles:- These are used for
(i) Retaining Earthfill at vertical cuts.
(ii) to prevent piping failure below hydraulic structures to minimise seepage force.

Ultimate load carrying capacity of a pile (Q_{up})
It is that max. load which can be applied on the pile without shear failure.

$$Q_{up} = Q_{eb} + Q_{sf}$$

The allowable load on the pile is

$$Q_{ap} = \frac{Q_{up}}{F.O.S} = \frac{Q_{up}}{F} = \frac{Q_{eb}}{F_1} + \frac{Q_{sf}}{F_2} \quad (\underset{3}{F_1} > \underset{2}{F_2})$$

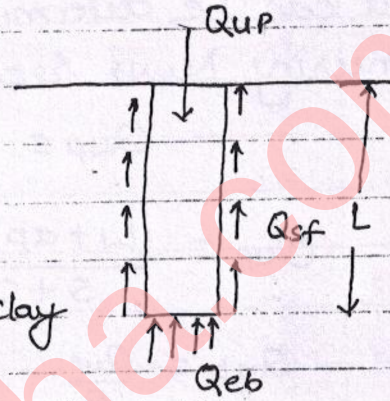
overall F.O.S $F = 2.5$ to 3.0 but F_1 is factor of safety in bearing action and F_2 is factor of safety in skin friction.

Methods to determine pile load capacity
(i) Analytical Method:- Suitable for clays.
(ii) Dynamic Method:- Suitable for sands (Dense sands)
(iii) Field Methods:- Suitable for clays and sands both
a) Pile load test:- It is the best method to find pile load capacity but test pile becomes waste Hence it is destructive test.
b) Cyclic pile load test:- It is the only method which gives end bearing resistance and skin friction

Standard Penetration test:- Suitable for sands.

Cone penetration test:-

Analytical Method:- (Static Method)



$$Q_{up} = Q_{eb} + Q_{sf}$$

$$= (q_b \cdot A_b) + (q_s \cdot A_s)$$

$$= CN_c A_b + C_a \cdot A_s \text{ ---- for clay}$$

$$N_c \approx 9$$

C_a = Unit adhesion over depth of pile.

$$= \alpha \bar{C}$$

$$\boxed{Q_{up} = 9C A_b + \alpha \bar{C} A_s}$$

$C \rightarrow$ Unit cohesion at the base of pile

$\bar{C} \rightarrow$ Average Unit cohesion over depth of pile.

$\alpha \rightarrow$ Adhesion factor b/w pile & soil.

$\alpha = 0.6$ to 0.9 for loose clay (sticky behavior" More in loose)

$\alpha = 0.4$ to 0.6 for Medium clays

$\alpha = 0.3$ to 0.4 for Dense/stiff clays

A_b = Bearing Area of pile / Base area.

$$= \frac{\pi}{4} D^2 \text{ for circular pile}$$

$$= B^2 \text{ for square pile.}$$

A_s = Surface area of pile in contact of soil

$$= \pi DL \text{ for circular pile}$$

$$= 4BL \text{ for Square Pile.}$$

Dynamic Method :- Using hammer test load carrying capacity of pile can be determined.

(i) Engineering News Record formula (ENR formula).

$$Q_{up} = \frac{WH}{S+C} \quad \text{for Drop hammer \& single acting steam Hammer}$$

$$Q_{up} = \frac{(W+ap)H}{S+C} \quad \text{for Double Acting Steam Hammer}$$

$$Q_{ap} = \frac{Q_{up}}{F} \qquad \boxed{F \simeq 6} \quad \text{More Uncertainity}$$

W is weight of hammer, H height of fall of hammer in cm. S is final SET (Penetration of pile per blow of hammer in cm). S is taken avg. penetration of last five blows for drop hammer and S is taken avg. penetration of last 20 blows for steam hammer in cm., C is a constant which accounts for Energy loss against friction, Elastic compression of pile and soil. C = 2.5 cm for drop hammer.
C = 0.25 cm for single and double acting steam hammer.

a is area of hammer or piston, p is steam pressure applied on the hammer or piston.

(ii) Hiley Formula :- (IS code recommends Hiley formula)

$$Q_{up} = \frac{\eta_h \eta_b W H}{S + C/2}$$

$$Q_{ap} = \frac{Q_{up}}{F} \quad (F \simeq 3)$$

where η_h is efficiency of hammer

$\eta_h = 1$ for Drop hammer

$\eta_h = 0.7 - 0.8$ for Double acting steam hammer

η_b it is efficiency of hammer blow, it is defined as ratio of energy of hammer after the impact divide by energy of hammer before the impact. It depends upon Coefficient of restitution b/w hammer and pile.

$\eta_b =$

$$\eta_b = \left(\frac{W + e^2 P}{W + P}\right) \text{ ...... } W > eP$$

$$= \frac{W + e^2 P}{W + P} - \left(\frac{W - eP}{W + P}\right)^2 \text{ -- when } W \leq eP$$

where W is weight of hammer

P weight of pile + pile cap

e Coefficient of restitution b/w hammer and pile

S final SET per blow of hammer

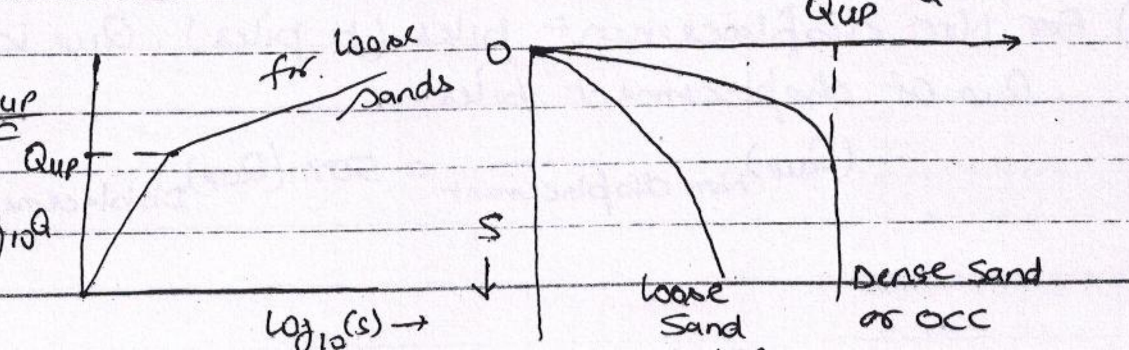
C is a Constant which includes total Elastic Compression of pile, pile cap and soil

H is height of free fall of hammer.

3) Field Method :-

(i) Pile Load test :- The loaded pile becomes waste. This test is performed in insitu conditions and it is more suitable for dense sands because clays are likely to get remoulded due to dynamic action of pile driving. The test pile is loaded and load settlement curve is plotted. At failure the pile settles at faster rate without much increase in load.

$$Q_{ap} = \frac{Q_{up}}{F}$$



IS code recommendation to find allowable load on pile using settlement criteria.

(i) The allowable load is taken as 50% of Ultimate load at which total settlement of pile is $1/10^{th}$ of its dia.

or (ii) the allowable load is taken as $2/3^{rd}$ of Ultimate load at which total settlement of pile is 12mm.

or (iii) the allowable load is taken as $2/3^{rd}$ of Ultimate load at which the Net plastic settlement (permanent settlement) of pile is 6mm.

(ii) Standard penetration test:— Determination of Qup using SPT Value:-

let N is Corrected SPT Number at the base of pile

let $\bar{N}$ is Corrected average SPT Number over the depth of pile.

(1) Ultimate load capacity of Driven/Displacement piles is

$$Q_{up} = 400 N \cdot A_b + 2\bar{N} A_s$$

Where A_b is bearing area in m^2. A_s Surface area in m^2 and Q_{up} is in kN.

(2) for Bored piles Qup is taken as $1/3^{rd}$ of Qup of driven piles

$$(Q_{up})_{Bored} = \frac{1}{3}(Q_{up})_{driven\ piles}$$

(3) For Non displacement piles (H piles), Qup is 50% of Qup of displacement piles

$$(Q_{up})_{Non\ displacement} = 50\%\,(Q_{up})_{Displacement\ piles}$$

Cone penetration test:-

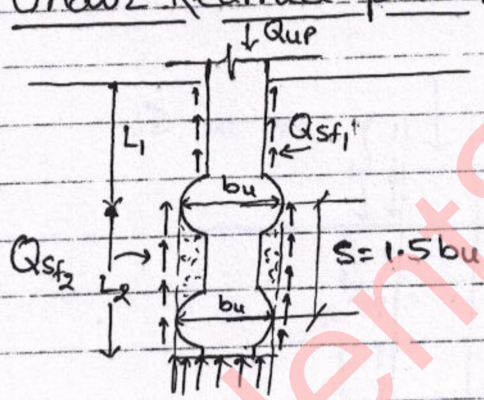
Let q_c is static cone resistance in kg/cm^2 at the base of pile. Let $\bar{q}_c$ is average cone resistance over the depth of pile in kg/cm^2.

The Ultimate load carrying capacity of displacement pile is $Q_{up} = q_c A_b + \frac{\bar{q}_c}{2} A_s$

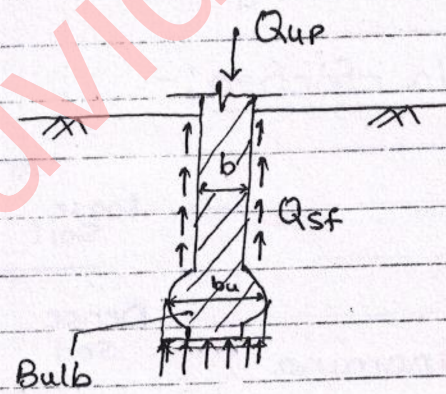
where A_b and A_s are in m^2 and Q_{up} is in KN.

Q_{up} for Bored piles is $\frac{1}{3}$rd of Q_{up} for displacement piles and Q_{up} for Non displacement piles (H piles) is $\frac{1}{2}$ of Q_{up} for displacement/driven piles.

Under Reamed piles:-



Multi bulb under Reamed pile



Single bulp U/R pile

Under reamed piles are used in expansive soils to prevent uplift pressure due to swelling of soils Ex Black soil. The depth at which bulb should be placed should be in stable condition that is no variation of water content. Due to increase in bulb area bearing area of pile increases. In multi bulb under reambed pile surface area also increases. The bulb diameter should be kept 2-3 times the shaft diameter

$$\frac{b_u}{b} = 2 \text{ to } 3$$

and spacing b/w the bulbs (c/c) should be ~~[struck]~~ $1.5b_u$

The load carrying capacity of single bulb under reamed pile

$$Q_{up} = Q_{eb} + Q_{sf}$$

$$= (9c)\frac{\pi}{4} b_u^2 + (\alpha \bar{c})\pi b L.$$

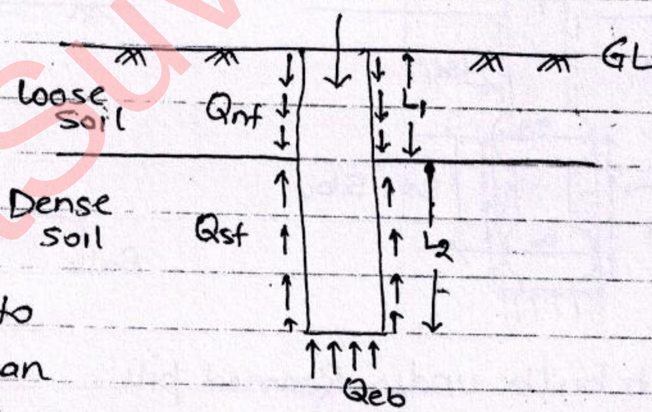
for multibulb under reamed pile Q_{up} is

$$Q_{up} = Q_{ue} + Q_{sf_1} + Q_{sf_2}$$

$$= (9c)\frac{\pi}{4} b_u^2 + (\alpha_1 \bar{c}_1)\pi b L_1 + (\alpha_2 \bar{c}_2)\pi b_u L_2$$

1 (soil to soil shear)

Negative skin friction:-



It is the phenomena in which surrounding soil to the pile settles more than settlement of pile. This Condition occurs when the soil surrounding to the pile is very loose or soft. Following condition may cause negative skin friction.

(i) Increase in surcharge over surrounding soil

(ii) Lowering of Ground Water table.

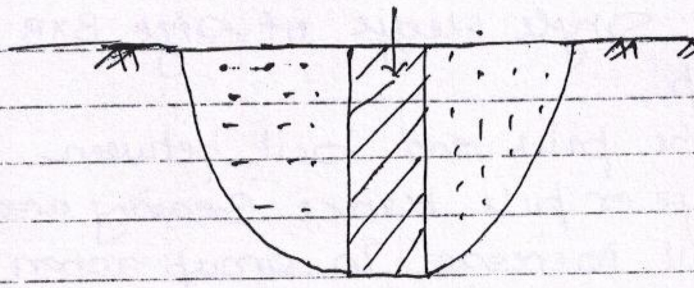
(iii) Disturbance due to earthquake or other dynamic effects.

The negative skin friction reduces load carrying capacity of pile

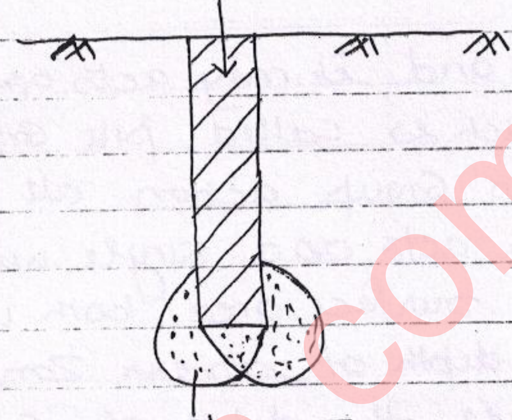
$$Q_{up} = Q_{eb} + Q_{sf} - Q_{nf}$$

$$= 9c A_b + (\alpha \bar{c}) A_s - (\alpha \bar{c}) A_n$$

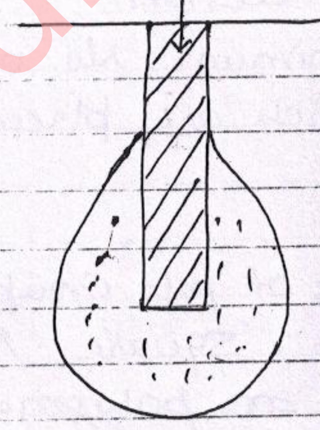
Group Action of piles:-



Stress Zone of soil due to friction action [low stress intensity]

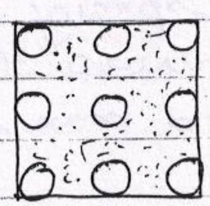


Stress Zone Due to End bearing action [Highly stressed Zone]

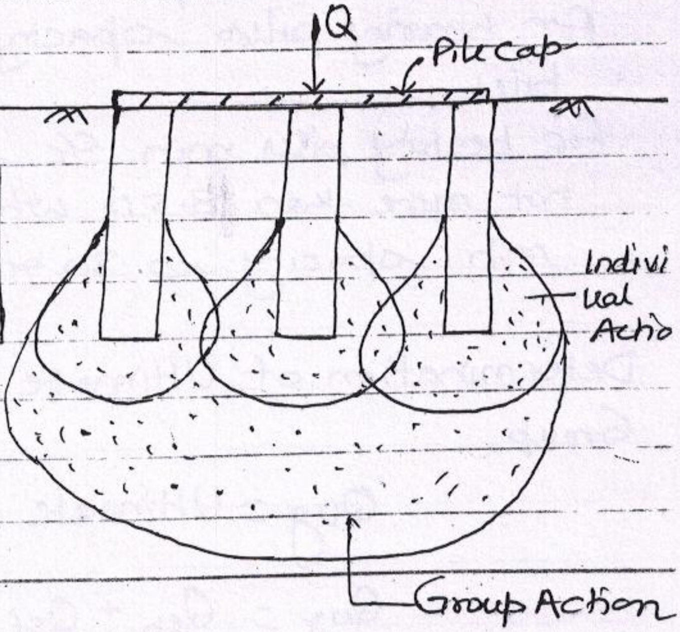


Effective Stress Zone [B and F Action]

If load applied is large and more No. of piles are used then piles may act individually or in Group depends upon the spacing between piles.
If spacing is (c/c) 2.5D to 4D then soil may get Compacted b/w



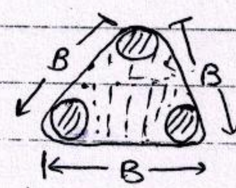
Pile Group



piles and it may acts as a single wedge of size B×B. which is called pile Group.

In Group action all the piles and soil between piles acts as a single wedge or pile. Hence Bearing area and surface area both will increase. In Group action the depth of stress Zone extends to Greater depth Hence the depth of Compressible layer will be more therefore Consolidation settlement of pile Group will be always Greater than settlement of pile under individual action.

The minimum No. of piles required for the Group action is 3 which are placed in the form of a Equilateral triangle



The shape of pile Group may be triangular, square, rectangular, circular or polygonal however square pile Group is preffered. The c/c spacing between the piles depends upon action of piles and method of installation.

For bearing piles spacing is kept less than ~~fric~~ friction piles.

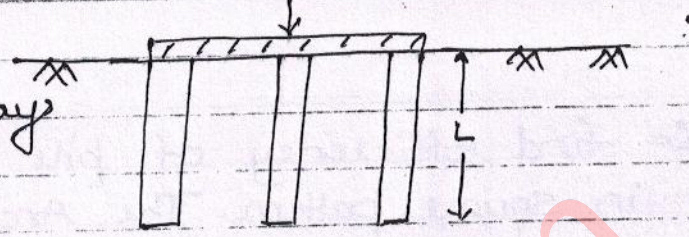
For bearing piles min c/c spacing should be 2.5D and not more than 3.5D, whereas for friction pile Group min. spacing is 3D and max is 4D.

Determination of Ultimate load carrying capacity of pile Group

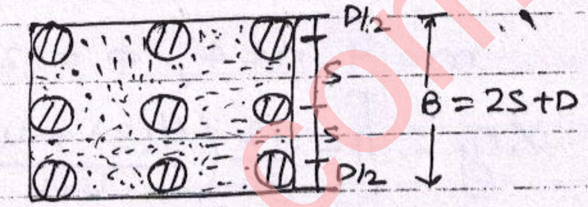
Q_{ug} = Ultimate load of pile Group

$Q_{ug} = Q_{eb} + Q_{sf}$

$$\boxed{Q_{ug} = 9cB^2 + \bar{c}(4BL)} \text{ for clays}$$



The safe load of the pile Group will be smaller of the following 2 values.



(1) $\frac{Q_{ug}}{F}$ and $\frac{nQ_{up}}{F}$

where n is total No. of piles in the Group

Q_{up} Ultimate load of single pile.

F is factor of safety 2.5 - 3.0.

Group ~~Settlement~~ Efficiency ~~Ratio~~ :- It is defined as ratio of load carrying capacity of pile Group to the sum of load carrying capacity of all the piles.

$$\text{Group Efficiency} = \eta_g = \frac{Q_{ug}}{n \cdot Q_{up}}$$

$$\boxed{\eta_g \geq 1}$$

Note:- In case of sands, usually Group Efficiency is >1 as long as spacing is 2.5D to 4D whereas in case of clays it may be $>, =, < 1$ depending upon soil properties and spacing.

Converse - labarre formula to find Efficiency of pile Group

$$\% \eta_g = \left[1 - \frac{\phi}{90}\left[\frac{m(n-1)+n(m-1)}{m \cdot n}\right]\right] \times 100$$

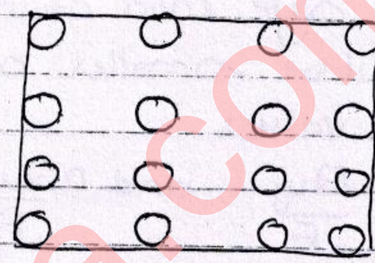
where ϕ is arc tan value

m No. of rows in the pile Group

n No. of columns in the pile Group

Eg: find Efficiency of pile Group having 16 piles arranged in square pattern. The Arc tan Value is 12

$m = 4,\ n = 4,\ \phi = 12$



$$\% \eta_g = \left[1 - \frac{12}{90}\left(\frac{4(3) + 4\times3}{4\times4}\right)\right]\times 100$$

$$= 80\%$$

Group Settlement Ratio:- It is defined as Ratio of settlement of pile Group to the settlement of individual pile

$$S_R = \frac{S_g}{S_i} = \frac{\text{Settlement of pile Group}}{\text{Settlement of individual pile}}$$

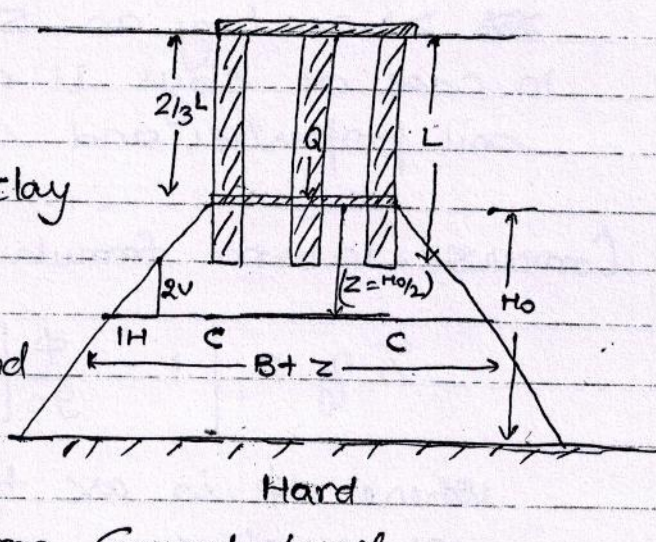
It is always > 1 and be as high as 16.

Determination of settlement of pile Group:-

Case 1:- when piles are embedded on a uniform clay deposit and pile Group acts as a friction pile Group

$$\Delta\bar{\sigma} = \frac{Q}{(B+Z)^2} \quad \text{—— (I)}$$

$$\Delta H = \frac{C_c H_0}{1+e_0}\log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right) \quad \text{—— (II) clay}$$



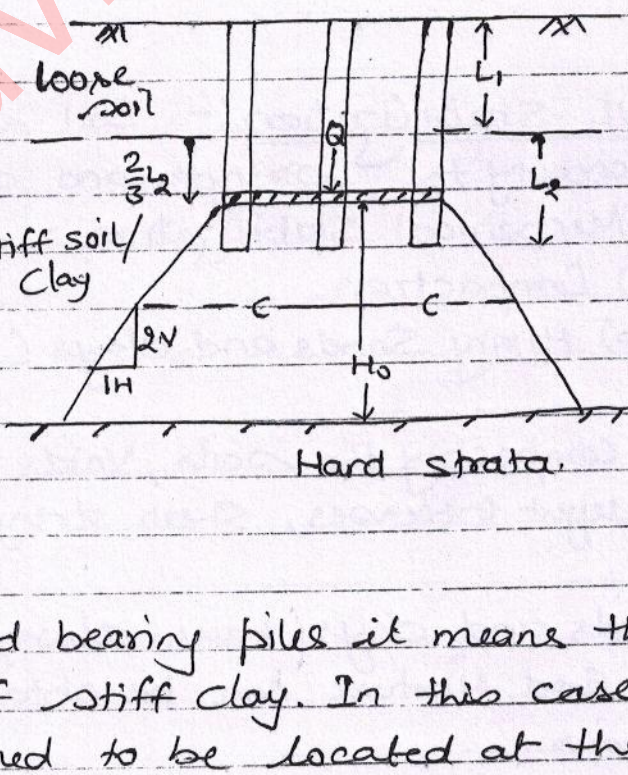
It is assumed that applied load acts on a imaginary equivalent raft of size (B×B) which is placed at a depth of $\frac{2}{3}L$ from Ground level

Below the assumed raft load is assumed to be

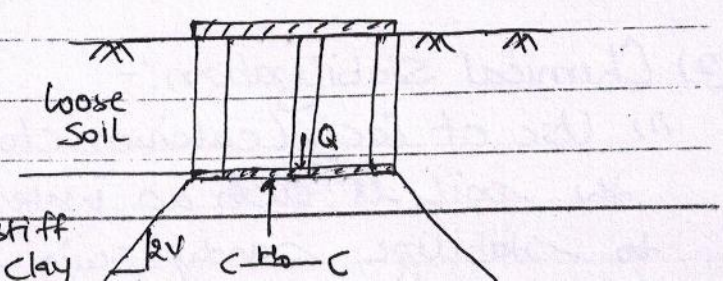
The depth of compressible layer will be below the raft = H_0, let C-C is centre of compressible layer at a depth $Z = H_0/2$ below the raft.
Let $\bar{\sigma}_0$ is initial effective overburden pressure at C-C. and let $\Delta\bar{\sigma}$ is increased in effective stress at C-C due to applied load which is given by eqn (1)
then settlement of pile group ΔH is given by eqn (2).

Case 2:- If piles are driven through a soil/clay having two layers. Top layer of depth L_1 is loose soil and bottom layer is stiff soil/clay which is resting over hard strata as shown in fig.

The equivalent raft is assumed to be present at $\frac{2}{3}L_2$ below soft stratum where L_2 is depth of pile embedded in stiff soil/clay. Rest of the procedure is similar to previous case.



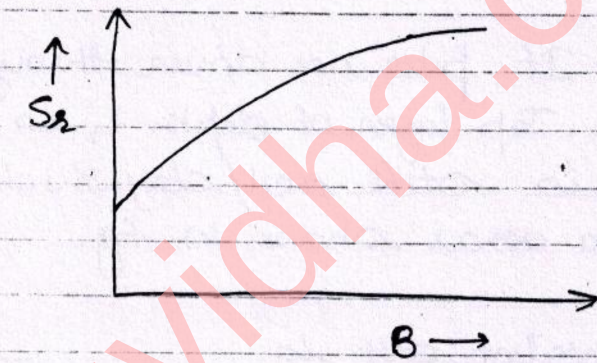
Case 3:- When piles are end bearing piles it means the piles rest over surface of stiff clay. In this case Equivalent raft is assumed to be located at the base of piles.
Remaining procedure is similar to previous case



Case 4 :- In case of sands the Group settlement Ratio can be determined directly using size of pile Group

$$\boxed{S_R = \frac{S_g}{S_i} = \left[\frac{4B + 2.7}{B + 3.6}\right]^2}$$

$S_R \uparrow$ with $\uparrow$ in the size of pile Group (B)



Soil Stabilization :- Soil stabilization is the process of increasing the strength and durability of soil.

(1) Mechanical Stabilization :-

a) Compaction

b) Mixing Sands and clays (ie Well Grading)

By compacting the soils, Voids are reduced therefore Unit Weight increases, Shear strength and bearing capacity increases.

Sands and clays have always opposite behaviour hence a Good Mixture can be obtained by mixing these two soils.

2) Chemical Stabilization :-

(i) Use of $CaCl_2$ (calcium cloride) :- When it is added to the soil, it acts as water repulsive agent and is used to stabilize sandy soils such as base and subbase

Sodium silicate Na_2SiO_4 ($NaSiO_2$) :- Sodium silicate with water and calcium chloride is known as water Glass. This mixture is injected for stabilization of deep soil deposits. It forms soluble precipitate of silica gel within the pores of soil. and makes the soil impervious. It increases shear strength and it is used for sands.

Lime stabilization:- the hydrated lime $Ca(OH)_2$ is used to stabilize highly plastic clays such as Black soils. 4-6% volume of soil is the volume of lime required.

Bitumen stabilization:- Bitumen, tar and Emulsion are used to stabilize Granular soils. It acts as water ~~binding~~ repulsing agent as well as binding agent.

Cement stabilization:- For increasing the strength and durability of all the soils except highly organic clays Cement Grouting can be done. Cement requirement is nearly 5% of volume of soils in sands and nearly 15% of volume of soils in clays.

Soil EXPLORATION :- Exploration is the process of soil sampling to collect soil sample for laboratory testing.

Method of exploration:-

(i) Augar Boring :- Hand operative Auger is used upto 6m depth in soft soils. whereas power driven auger may be used for greater depth and harder strata.

(ii) Wash boring :- It consists of driving a pipe through drop hammer and water is forced under pressure inside the pipe. It can be used below WT also for all type of soils except Hard rocks.

3) Percussion Boring:- Percussion drilling is carried out by repeated blows inside a cast Iron pipe. The bore hole is kept dry and to reduce friction, pulverised slurry is used. It is suitable ~~for~~ for Gravely and bouldurly strata.

4) Rotary drilling:- It is usefull for those soils which are highly resistant to Auger and Wash boring. It can be used for Sands and clays both.

Type of soil Samples:-

(i) Undisturbed samples:- If original soil structure, Mineral Content and Moisture Content of the soil remain unchanged while sampling then sample is called Undisturbed. such samples are practically difficult. but small degree of disturbence may be ignored.

(ii) Disturbed samples:-

a) Representative sample:- If only soil structure is modified and mineral content and water content remains unchanged then it is called representative sample.

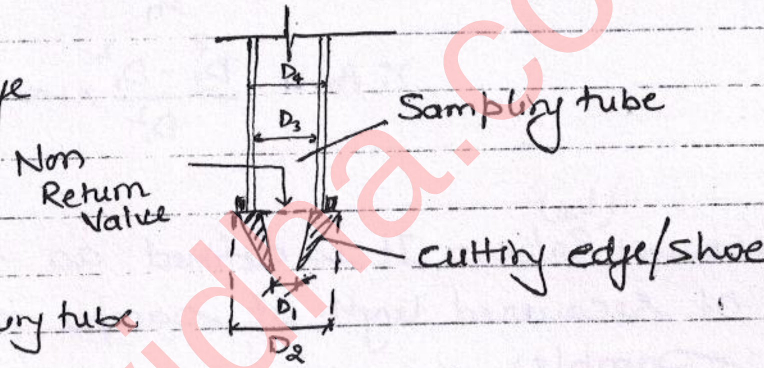
b) Non representative sample:- If Soil structure, Mineral Content and Water Content, all get modified during sampling then the sample is called non-representative.

Note:- For Consistency limits, Specific Gravity, Grain size distribution, either representative samples or Undisturbed samples may be used.

for coefficient of permeability, Consolidation parameter and shear strength parameter, Undisturbed samples should be used.

The degree of disturbance of a sample depends upon design of sampler.

D_1 = Inside dia of cutting edge
D_2 = Outside dia of cutting edge
D_3 = Inside dia of Sampling tube
D_4 = Outside dia of sampling tube



1) Inside clearance :- It is governed to reduce friction b/w soil sample and sampler. when Soil enters into the sampling tube it allows for elastic Expansion of soil. It is kept b/w 1% - 3%.

$$\% C_i = \frac{D_3 - D_1}{D_1} \times 100$$

$$= 1\% \text{ to } 3\%$$

2) Outside clearance:- It helps to reduce friction while sampler is driven and withdrawn. It is kept b/w 1% to 2%

$$\% C_o = \frac{D_2 - D_4}{D_4} \times 100$$

$$= 1\% \text{ to } 2\%$$

3) Area Ratio :- It is kept as less as possible. for sensitive clays it should be less than or equal to 10% but in no case it should exceed > 20%.

A_1 = Inside area of cutting edge $= \frac{\pi}{4}D_1^2$

A_2 = Outside area of cutting edge $= \frac{\pi}{4}D_2^2$

$$\% A_r = \frac{A_2 - A_1}{A_1} \times 100$$

$$\% A_r = \frac{D_2^2 - D_1^2}{D_1^2} \times 100 \quad \leq 10\%, \quad \not> 20\%.$$

Recovery Ratio (L_r):- It is defined as the ~~recovered length~~ ratio of recovered length of sample to the penetration length of sampler.

If $L_r = 1 \Rightarrow$ Good recovery

$L_r > 1 \Rightarrow$ sample has swelled.

$L_r < 1 \Rightarrow$ sample has shrunk.

~~FM~~ Field Method of Subsurface Investigation:-

a) Standard penetration test

b) Cone penetration test

c) Plate load test

d) Vane shear test

e) Pressure meter test

f) Geophysical Methods.

a) Seismic refraction method

b) Electrical resistivity method.

Foundation on Expansive Soils:- Expensive soils are those which represents large volume change due to change in water content. for Ex Black cotton soils, Bentonite soils. Such soil contains montorellite minerals.

Expensive soils have large liquid limit, large plasticity

For Indian Black cotton soil
$W_L = 50-100\%$, $PI = 20-65\%$, clay size particles = 50-60
Shrinkage limit = 9-12%

The swelling behaviour of a soil largely depends upon type of clay mineral and its fraction present in the soil. The Commonly used methods to determine expansion behaviour are (i) Differential thermal Analysis (DTA)
(ii) X-Ray Differaction method.
(iii) Electron microscopic Method.

Free Swelling test:- (Given by Holtz & Gibbs)
10 cc of dry soil which is finer than 425 μm size is put in 100cc water filled in a Graduated Glass. the Volume of swelled soil is ~~red~~ read after 24 hr. The free swell Value in % is given as

$$\%\text{ Free Swell Value} = \frac{\text{Final Volume} - \text{Initial Vol.}}{\text{Initial Vol.}} \times 100$$

The Bentonite which contain montmorellite mineral has free swell value of 1200-2000% where as kaolinite has around 80%. If free swell value is less than 50% then it is not of much consequence.

Differential free swell test:-
Two samples of dry soil passing 425 μm seive and each 10 gm are used. One is put in 50cc water and other is put in 50cc Karosine (Non polar liquid) both samples are left undisturbed for 24 hr. The differential free swell Value is given as

$$\%\text{ DFS} = \frac{\text{Soil Volume in Water} - \text{Soil Vol. in karosine}}{\text{Soil Vol. in Kerosine}} \times 100$$

If DFS is more than 35% then ~~the~~ Conventional Shallow footing will not be adequate.

Differential free swell Value %DFS	Degree of Expensiveness.
0 - 20%	Low
20 - 35%	Moderate
35 - 50%	high
>50%	Very high.

Relation b/w Swelling potential and plasticity Index.

plasticity Index %	Swelling potential
0-15%	Low
15-25%	Medium
25-35%	High
35-above	Very high.

Relation b/w Shrinkage limit, degree of Expension and Linear Shrinkage

Shrinkage limit	Degree of Expansion	Linear shrinkage
<10%	Critical	>8%
10-12%	Marginal	5-8%
>12%	Non critical	0-5%

Note:- If Swelling pressure is less than 20 kN/m² then it may be regarded that there is no much consequence. The swelling pressure is a ~~a~~ unique parameter of a soil but it is influence by (i) Initial water content, (ii) Initial dry density (iii) Method of compaction (iv) surcharge over the

It a foundation is to ~~de~~ be design ~~of~~ in expensive soil then it should be design for as high as possible bearing pressure which should be consistent with bearing capacity (shear criteria) and settlement requirement both.

It is important to note that heavily loaded structures are less influenced by swelling pressure whereas lightly loaded single and double story building experience max. damage

Swelling can be controlled by providing an impervious appron around the structure. By providing appron the moisture gradient b/w the centre of structure and its edges is minimised and hence differential swelling is controlled.

The swelling can also be controlled by providing ~~a~~ Goods surface drainage nearby surface which removes rain water to the drain. Subsurface system may also help to remove water.

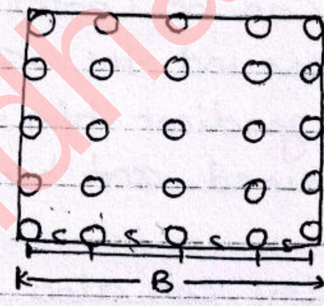
The paving the surface around the structure may prevent entry of moisture into the ~~structure~~ foundation and may help in reducing the swelling. Note that if murrum ~~and~~ or cohesionless soil is provided as backfill then it is not necessary that it ~~is~~ will take care of swelling if proper drainage is not available
Without proper drainage it may aggravate problems.

Ex Find the Group Efficiency and safe load which can be applied on the pile Group as shown in the fig. The dia of piles is 0.3m, c/c spacing is 0.9m, length of piles is 10m. Shear strength of the soil at the base of the pile is 180 KN/m², Average shear strength over the depth of pile is 110 KN/m². Soil is clay with adhesion factor = 0.45 and F.O.S = 2.5

Ans size of pile Group = 4S+D

$= 4 \times 0.9 + 0.3$

$= 3.9m.$



$B = 4s + d.$

Q_{up} = Ultimate Load on single pile.

$= Q_{eb} + Q_{sf}$

$= 9 C A_b + (\alpha \bar{c}) A_s$

$= 9 \times 180 \times \frac{\pi}{4}(0.3)^2 + 0.45 \times 110 \times \pi \times 0.3 \times 10$

$Q_{up} = 581.03 KN$

$n Q_{up} = 25 \times 581.03$

$= 14525.93 KN.$

for clay $\phi = 0$

$S = C + \sigma \tan\phi$

$S = C$

$C = 180$

$\tau = 110$

Ultimate Load of pile Group

$Q_{ug} = 9 C B^2 + \alpha \bar{c}\, 4BL$ (α = 1)

$= 9 \times 180 \times 3.9^2 + 1 \times 110 \times 4 \times 3.9 \times 10$

$= 41800.2 KN$

$\eta_g = \frac{Q_{ug}}{n Q_{up}} = \frac{41800.2}{14523.93} = 2.87 > 1$ ok.

safe load on pile Group is smaller of $\frac{Q_{ug}}{F}$ & $\frac{n Q_{up}}{F}$

$Q_{safe} = \frac{n Q_{up}}{F} = \frac{14525.93}{2.5} = 5810.37 KN.$

Guidelines for design of a pile Group:-

(i) length for friction piles, length should be 10 - 20m for bearing piles, length will depend on stiff strata.

(ii) Diameter:- 0.3 - 0.9m but more commonly 0.4 - 0.5m

(iii) Spacing:- c/c spacing 2.5 - 3.5D but commonly S = 3D.

(iii) Group Efficiency $\eta_g \geq 1$ $\eta \nless 1$

(iv) No. of piles:- Commonly used Groups 4x4, 5x5, 6x6.

(v) Adhesion factor:- for NCC or loose clays 0.6 - 0.8.

for Medium dense 0.4 - 0.5

for OCC/stiff clay 0.3 - 0.4.

Ques Design a friction pile Group to carry safe load of 300ton over a uniform clay deposit upto a depth of 20m. which is resting over a rock. the average Unconfined Comp. strength of clay is 7 ton/m² adopt FOS = 3, adhesion factor = 0.6

Ans. let us design square pile group with dia 0.4m, spacing = 3D and let no. of piles are 5x5 = 25.

$$C = q_u/2 = 7/2 = 3.5 \text{ tonne/m}^2$$

$$Q_{safe} = 300 \text{ tonne} = n \frac{Q_{up}}{F}$$

$$Q_{up} = \frac{300 \times 3}{25} = 36 \text{ tonne}.$$

for friction piles $Q_{eb} = 0$

$$Q_{up} = Q_{eb} + Q_{sf}$$

$$36 = \alpha \bar{C} \pi D L$$

$$36 = 0.6 \times 3.5 \times 3.14 \times 0.4 L$$

$$L = 13.64 \quad \text{OK} \quad L < 20$$

$$Q_{ug} = \cancel{Q_{eb}}^{0} + Q_{sf}$$

$$= \alpha \bar{C}\, 4BL \quad (\alpha = 1)$$

$$= 3.5 \times (4S + D) L = 3.5 (4 \times 3D + D) L$$

$$= 3.5 [13 \times 0.4] \times 13.64$$

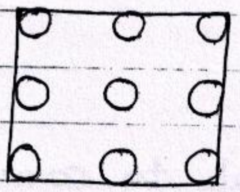
$$= 992.9 \text{ tonne}$$

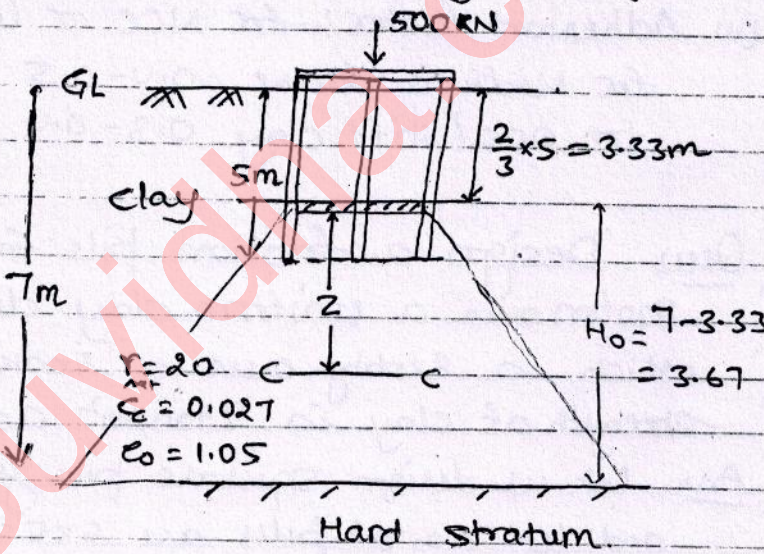
$$\eta_g = \frac{Q_{ug}}{n\,Q_{up}} = \frac{992.9}{25 \times 36} = 1.1 > 1 \quad \text{OK}$$

Ques For a 3x3 pile Group shown in fig. find the settlement of pile Group in a Normally Consolidated clay having properties shown in Fig.

Dia of pile = 0.2m

Spacing c/c = 0.5m





$$Z = \frac{H_0}{2} = 1.835m$$

$$\Delta\bar{\sigma} \text{ at C-C} = \frac{Q}{(B+Z)^2}$$

$$B = 2S + D = 2 \times 0.5 + 0.2 = 1.2$$

$$\Delta\bar{\sigma} = \frac{500}{(1.2 + 1.835)^2} = 54.28 \text{ kN/m}^2$$

$$\bar{\sigma}_0 = \gamma_{sat}(3.33 + 1.835) = 20 \times (3.33 + 1.835)$$
$$= 103.3 \text{ kN/m}^2$$

$$\Delta H = \frac{C_c H_0}{1+e_0} \log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

$$= \frac{0.027 \times 3.67}{1 + 1.05} \log_{10}\left(\frac{103.3 + 54.28}{103.3}\right) m$$

$$\Delta H = 8.86 mm$$