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GEOTECHNICAL ENGG. FOUNDATION ENGINEERING



BY JASPAL SIR

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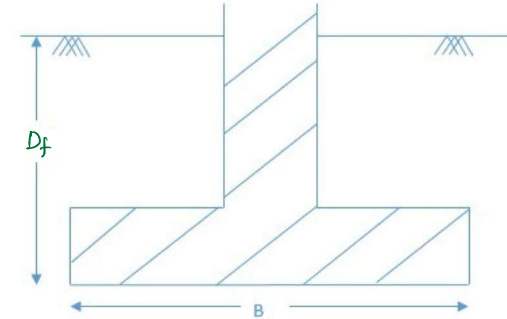
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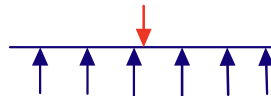
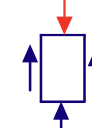
FOUNDATION ENGINEERING



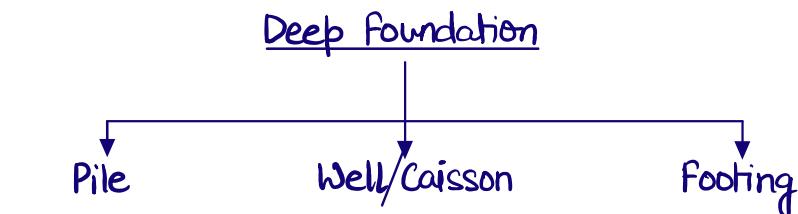
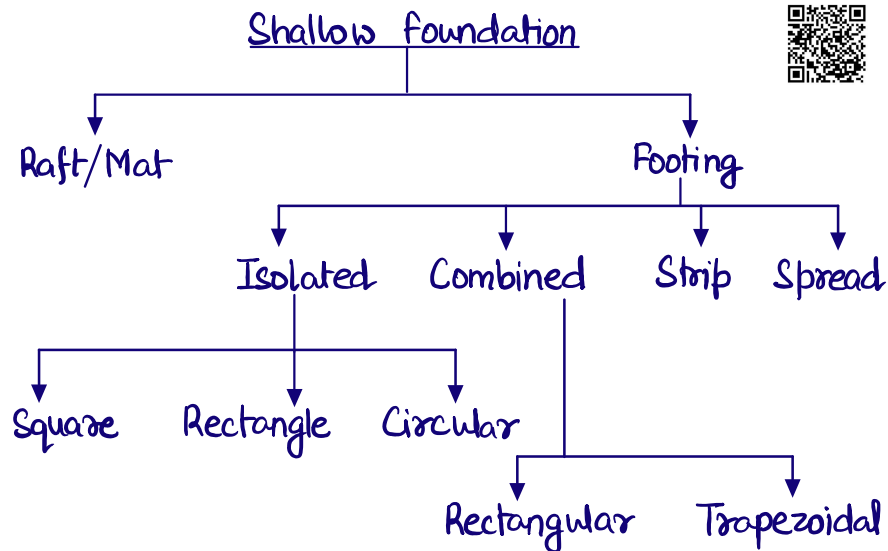
- Structure foundations are the Substructure elements which transmit the structural load to the earth in such a way that the supporting soil is not overstressed & not undergo deformation that would cause excessive settlement of the structure.
- Here, the properties of the supporting soil must be expected to affect the choice of type of structural foundation suitable for the structure.
- The failure of foundation may be due to
 - (i) Settlement of soil/foundation (It is called as Settlement failure).
 - (ii) Sliding/Slipping of soil foundation (It is called as Shear failure).
- The foundation should be safe in Shear Criteria as well as Settlement Criteria.

• Types of foundations



Shallow Foundation	Deep Foundation
(a) Terzaghi, $\frac{D_f}{B} \leq 1$ (b) Carries the load due to Base Resistance. 	(a) Terzaghi, $\frac{D_f}{B} > 1$ (b) Carries the load due to Base & Side Resistance. 





Note: As per Skempton,
for Shallow foundation, $\frac{D_f}{B} \leq 2.5$.

(a) Shallow foundation

→ A Shallow foundation Transmits Structural load to the Soil Strata at a relatively Small depth by the action of end Bearing.

→ As per Terzaghi if $\frac{D_f}{B} \leq 1$, it is termed as

Shallow foundation.



Note: $\frac{D_f}{B} = 1 - 15$, it is termed as Moderate deep Foundation.

→ It is further classified into Raft/Mat & footing.

(b) Deep foundation

→ If $\frac{D_f}{B} > 1$, it is termed as Deep foundation.

→ If $\frac{D_f}{B} > 15$, it is termed as Very deep foundation.

→ In deep foundation load is Supported partly by frictional resistance around the Surface and rest by bearing at the base of foundation.

→ While Constructing Shallow foundation in open excavation, the disturbance in Soil is minimal.

However in case of deep foundation disturbance of soil extends to a larger zone along the length of deep foundation.

Note: For reasons of economy, shallow foundation are always preferred over deep foundation.

• Guidelines for Selection of Foundation



Type of Soil & loading Condition	Suitable Foundation
→ If Structural load is less & Soil is medium to Dense.	Shallow Foundation
→ If Structural load is heavy & Soil is loose/weak.	Raft / Deep Foundation
→ If Swelling pressure is high & differential free swell value is more than 35%.	Raft / Deep Foundation
→ If footing area is more than 40% of plinth area.	Raft foundation / Combined footing.
→ If Structural load is heavy and foundation is to be placed in running water. (river, sea, Canal).	Compaction pile.

- If Soil is loose Saturated Sand & is prone to liquifaction.
- If Structural load is less (1-3 Storey Building).
- If Soil is expansive & high Swelling & Shrinkage Characteristics is found.

Compaction pile.



Isolated footing.

Floating or balancing foundation / Under-reamed pile.

• General Requirements of foundations

- For a Satisfactory performance, a foundation must satisfy the following three Conditions –
 - (i) Location & depth Criterion.
 - (ii) Shear failure criterion (Bearing Capacity Criterion)
 - (iii) Settlement Criterion.
- A foundation must be properly located and founded at such a depth that its performance is not affected by factors such as lateral expulsion of soil beneath the foundation, Seasonal Volume Changes caused by freezing, thawing & presence of moisture & adjoining structure.
- A foundation must be safe against shear failure or soil rupture.

- The Settlement of the foundation especially differential Settlement, must be within the permissible limit.
- Excessive Settlement may effect the utility of the Structure & may also cause its damage & decreases its aesthetic value.

Note: The three requirement are independent of each other and must be Satisfied Separately.

(i) Location & Depth Criterion of foundation

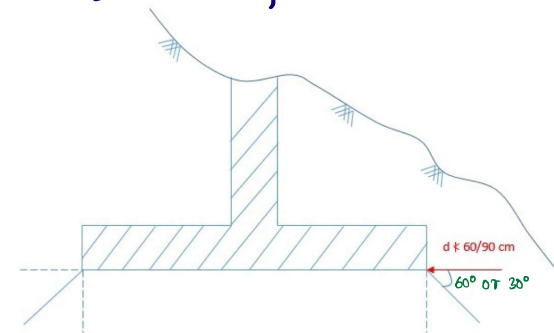
- As a general rule, any foundation should be placed at a depth where Soil Strata is adequate from the point of view of bearing Capacity or Settlement Criteria.
- However, it must be placed at a minimum depth of 0.5 m below natural Ground Surface.
- Further Foundation must be placed below the Zone of Volume Changes, where Volume Change is expected.
- In expansive Soil depth of foundation must be placed below the depth of frost Zone.
- The Zone of Seasonal variation in water Content varies in thickness from 1.5m - 3.5 m (for Black Cotton Soil).

- Foundations for Structure in river have to be protected from the Scouring action of Water. The depth of foundation in such case must be below the Scour level.

- When footing are adjacent to Sloping ground or Where the base of footing are near by at different levels or at level different from the of footing of adjacent Structure.

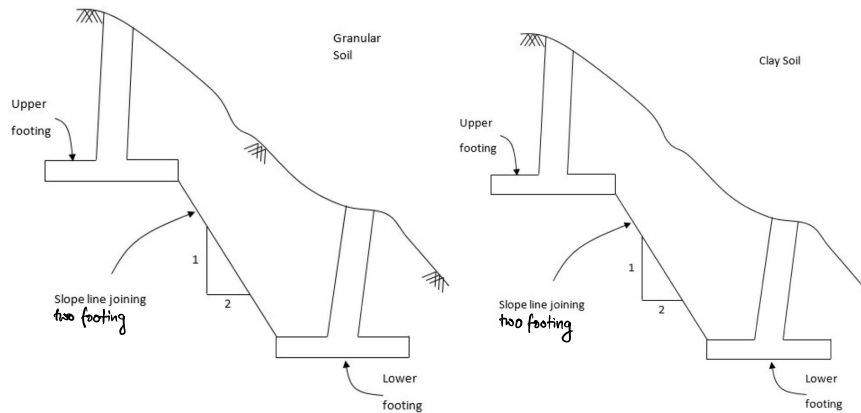
Following recommendations are adopted :

- (a) When the Ground Surface Slopes downward adjacent to a footing, the Sloping Surface should not encroach upon a frustum of Bearing material Under the footing having Sides which make an angle of 60° with horizontal for rock and 30° for Soil & the horizontal distance from the lower edge of the footing to the Sloping Surface shall be of atleast 60cm for rock & 90 cm for Soil.



(b) for footing in Granular Soils, the line joining the lower adjacent edges of the adjacent footing should not have a slope steeper than 2H:1V.

(c) In Clayey Soil, slope of line joining the lower adjacent edge of the upper footing & the upper adjacent edge of lower footing should not be steeper than 2H:1V.



(d) To avoid damage to an existing structure, the foundation for a new structure at an adjacent site should be located at suitable distance such that

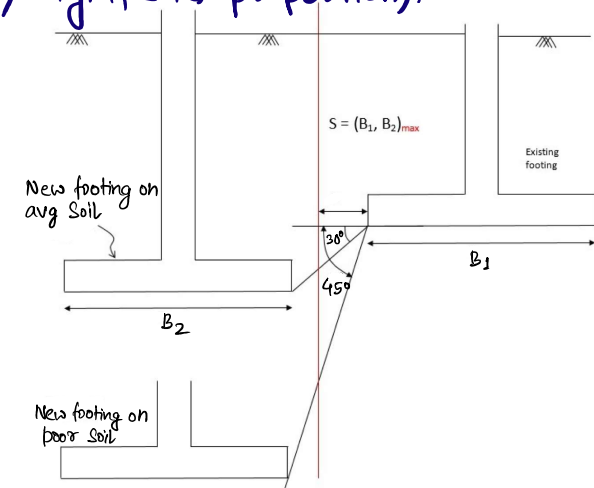
→ The adjacent edge of the new footing must be at least at a distance 's' from the edge of the existing footing where 's' is the width of

larger footing.

(e) The line from the edge of new footing to the edge of 45° or less with the horizontal plane, that is the distance 's' should be greater than the difference in elevation between the adjacent footing.

→ If even position of new & existing footing were to be interchanged, the recommendation would remain same.

(This provision ensure that the stress overlap due to the adjacent footing does not assume any significant proportion).



(f) When a new footing is placed lower than an old footing the existing structure may be endangered because of lateral flow of soil

- from beneath the existing footing.
- The excavation must not, therefore be too close to the existing footing & if it is done proper provision of bracing must be provided.

• Bearing Capacity of Shallow Foundation

- Terminology Used for analysis of bearing Capacity.



(i) Gross Pressure/Gross Loading Intensity (q_g)

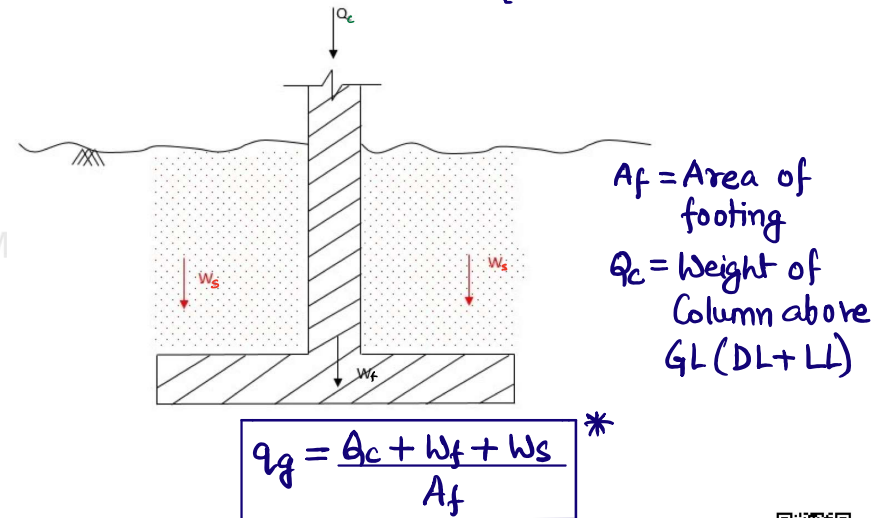
- For a footing constructed with its base at the depth D_f below the ground surface, the total pressure at the base of footing due to weight of the Superstructure, weight of footing & weight of Soil fill over the footing, it is termed as Gross pressure/loading intensity.

(ii) Net pressure/Net Loading Intensity (q_n)

- The deformation of Soil below the base of the footing is Caused only by the pressure over & above that which existed before the Construction of the footing of the Superstructure.

- The difference between the gross pressure & the Over burden pressure at the base of footing is termed as net pressure or net loading intensity.

$$\begin{aligned} q_n &= q_g - \gamma \cdot D_f \\ q_n &= q_g - \gamma' D_f \end{aligned} \quad \left\{ \begin{array}{l} \text{If water table effect} \\ \text{is also Considered} \end{array} \right\} *$$



(iii) Ultimate Bearing Capacity (q_u)

- If the load at Base of footing is Gradually increased, a Stage will be reached when the load will Cause the Shear failure of the Supporting Soil.

- The maximum Gross pressure intensity of the loading that the soil can support before it fails in shear or minimum gross pressure intensity of the loading at which soil fails in shear is termed as Ultimate Bearing Capacity.
- Ultimate Bearing Capacity of the soil is not only related to the properties of soil but also to the characteristics of footing such as its size, shape, depth, mode of loading & whether loading is applied axially or eccentrically.

(iv) Net Ultimate Bearing Capacity (q_{nu})

- The maximum net intensity of loading at the base of the foundation that the soil can support before failure in shear.
- It is difference of Ultimate Bearing Capacity & Overburden pressure.

$$\text{or, } \boxed{q_{nu} = q_u - \gamma D_f}$$

(If water table effect is also considered).

(v) Net Safe Bearing Capacity (q_{ns})

- It is the maximum net intensity of loading that the soil can safely support without the risk of shear failure.

- It is obtained by dividing q_{nu} by desired factor of Safety F .
- Usually, as factor of Safety of 2.5-3 is used -

$$\boxed{q_{ns} = \frac{q_{nu}}{F}}^*$$



(vi) Gross Safe Bearing Capacity (q_s)

- It is the maximum gross intensity of loading that the soil can safely support without failing in shear.

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

$$q_{ns} = \frac{q_u}{F} - \gamma D_f$$

$$\boxed{q_s = \frac{q_u - \gamma D_f}{F} + \gamma D_f}^*$$

Note: Since the additional term due to self weight of soil (γD_f) is available in full depth & is present from past history, it seems logical & rational not to use factor of Safety for this term.

- It is preferred to use only net Safe Bearing Capacity in calculation instead of Gross safe Bearing Capacity.

(vii) Net Safe Bearing Pressure (q_{ns})

- It is maximum net intensity of loading that can be allowed on the soil without the settlement exceeding the permissible value.
- No factor of safety is to be applied in this as it is already included in permissible value of settlement.

(viii) Allowable Bearing pressure (q_{anet})

- It is the maximum net intensity of loading that can be imposed on soil with no possibility of shear failure or settlement failure.
- Hence, it is the smaller of net safe bearing capacity (shear failure criterion) & net safe bearing pressure (settlement criterion).

$$q_{anet} = (q_{nsc}, q_{nsp})_{min} *$$

Note: It is often confusing to come across varied terminology in different literature & even in code of practice.

- But the terminology used here is logical & rational.
- In IS 6403 allowable bearing pressure is

is named as allowable bearing capacity but bearing capacity is associated with only shear failure criterion.

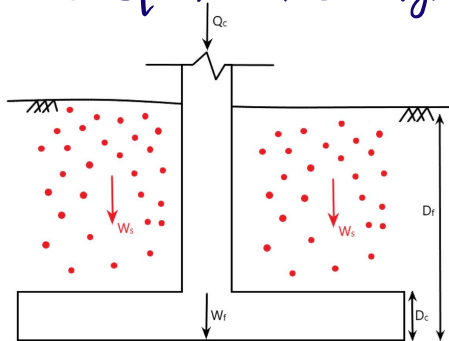
• Choice of Net allowable Bearing pressure

- For a safe design of a footing, the loading intensity at the base of the foundation should be less than allowable bearing pressure.
- It can be net or gross allowable pressure.
- It is conventional & also convenient to use net allowable bearing pressure in the proportioning of footing, due to following reason-
(i) It is logical to compare the net allowable bearing pressure with net loading intensity (i.e. the loading intensity at the base of footing in excess of loading intensity to which it was originally subjected).
- Where the excavated soil in the trench is back-filled after construction of footing & column, the net loading intensity is the gross loading intensity minus loading intensity due to back-fill soil.
- If there is no backfilling of soil after foundation of structure is incorporated in basement floor

Like in case of raft/Mat, the soil is excavated upto the level of Base of raft, Here the gross loading intensity is only due to Structural load & Self weight of raft & does not include the weight of Soil above the foundation.

→ In Such Case, net loading intensity would get reduced & even become Zero.

(ii) Another reason for Using net allowable Bearing pressure in design of footing Subjected to Superimposed load (Q_c) upto Ground level. { It consists Column load in soil below the Ground level equal to the weight of Soil. }



$$Q_g = Q_c + W_f + W_s$$

$$\text{Area of footing (A)} = \frac{Q_g}{q_{a, \text{gross}}}$$

Assuming the footing to be Square of Size 'B'.



$$B^2 = \frac{Q_g}{q_{a, \text{gross}}}$$

$$B = \sqrt{\frac{Q_c + W_s + W_f}{q_{a, \text{gross}}}}$$

Here, $W_f = B^2 D_c \gamma_c$, $W_s = (D_f - D_c) \cdot B^2 \cdot \gamma$

$$B = \sqrt{\frac{Q_c + B^2 D_c \gamma_c + B^2 (D_f - D_c) \gamma}{q_{a, \text{gross}}}} \quad *$$

- The above equation Can be Used provided W_f & W_s are known.
- W_f & W_s Cannot be Calculated Unless depth of foundation ' D_f ' & Size ' B ' is Known.
- Depth of foundation can be Selected by Code provision but Still Calculation of Size ' B ' of footing would require Trial & error approach.
- However, this problem can be eliminated by using net loading intensity/net pressure.

$$Q_g = Q_c + W_f + W_s$$

$$\frac{Q_g}{B^2} = \frac{Q_c}{B^2} + \frac{W_f}{B^2} + \frac{W_s}{B^2}$$

$$q_g = \frac{Q_c}{B^2} + D_c \cdot \gamma_c + (D_f - D_c) \cdot \gamma$$



$$q_g = \frac{Q_c}{B^2} + D_c(\gamma_c - \gamma) + D_f \cdot \gamma$$



$$q_g - D_f \cdot \gamma = \frac{Q_c}{B^2} + D_c(\gamma_c - \gamma)$$

Here, $q_g - D_f \cdot \gamma = q_n$

$$q_n = \frac{Q_c}{B^2} + D_c(\gamma_c - \gamma) *$$

Maximum value of $\gamma_c = 24-26 \text{ kN/m}^3$ & $\gamma = 19-21 \text{ kN/m}^3$.
If γ_c is assumed to be equal to γ , then -

$$q_n = \frac{Q_c}{B^2}$$

$\therefore$ For safe design of footing, net loading intensity must be less than net allowed bearing pressure.

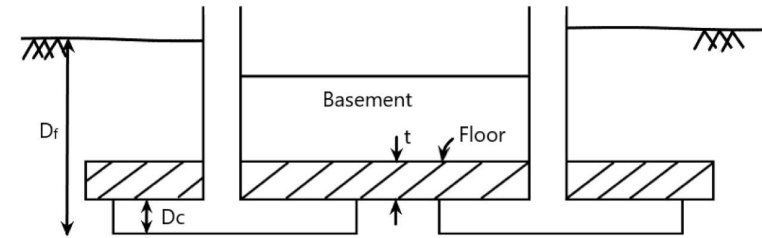
$$q_n \leq q_{a.net}$$

$$\frac{Q_c}{B^2} \leq q_{a.net}$$

$$B = \sqrt{\frac{Q_c}{q_{a.net}}}$$

→ Since there is no Unknown, there in RHS, this approach is more preferable for proportioning of footing.

→ If the foundation is not backfilled as in case of Raft/Mat then,



$$q_g = \frac{Q_c}{B^2} + D_c \cdot \gamma_c + t \cdot \gamma_c$$

$$q_n = q_g - D_f \cdot \gamma$$

$$q_n = \frac{Q_c}{B^2} + (D_c + t) \cdot \gamma_c - D_f \cdot \gamma$$

$\therefore$ Since $D_c, t \ll D_f \rightarrow D_c \& t$ can be neglected.

$$q_n = \frac{Q_c}{B^2} - D_f \cdot \gamma *$$



→ It can be analysed that if Backfilling is not done net loading intensity is reduced by Overburdened pressure due to ' $D_f \cdot \gamma$ ' & it may also be reduced to Zero when $\frac{Q_c}{B^2} = D_f \cdot \gamma$.

(i.e. when weight of Soil excavated is equal to Safe Structural Load).

- This principle is Used Behind designing of floating/Balancing/Compensated raft/footing. Further, for safe design of footing.
- ∴ $q_n \leq q_{a.net} \Rightarrow \frac{Q_c}{B^2} - D_f \cdot \gamma \leq q_{a.net}$

$$\boxed{\frac{Q_c}{B^2} \leq q_{a.net} + D_f \cdot \gamma}^*$$



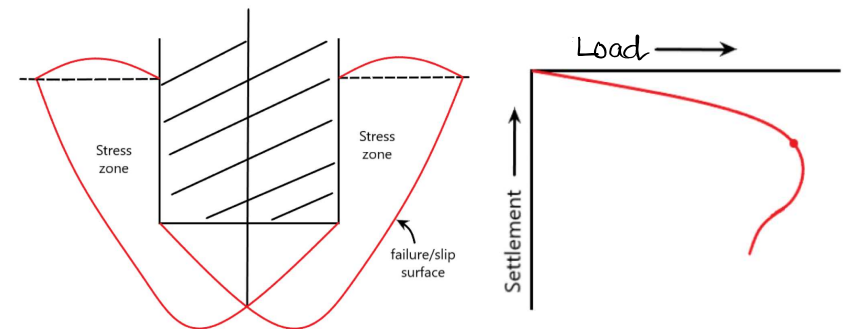
- It can be referred that load carrying Capacity of foundation is Considerably enhanced when foundation trench is not backfilled.

• Modes of Shear failure

- When a horizontal Strip footing is Subjected to the Gradually increasing load, it Undergoes characteristic Settlement.
- This Settlement behaviour is found to be related to the Soil properties.
- The Soil Zones involved in Generating resistance to foundation load, that is responsible for Bearing Capacity, have been identified Qualitatively.
- Three different types of failure mechanism based on pattern of Shearing Zones have been identified as follows-

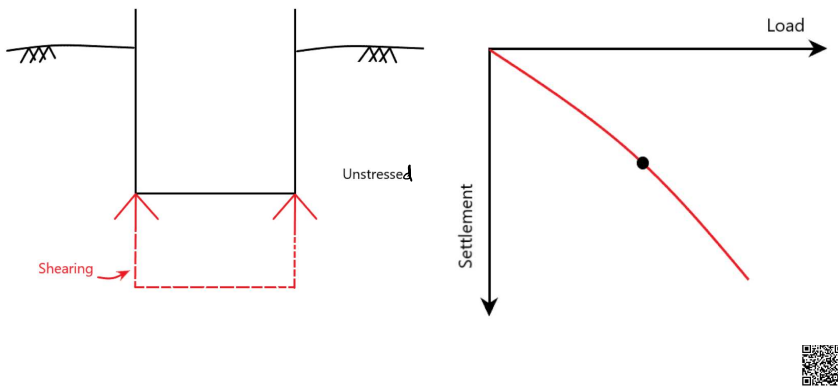
(I) General Shear failure

- It is typical of Soils possessing brittle type Stress-strain behaviour (medium-dense Sand).
- This mode of failure is found in Shallow foundation in very dense-medium dense Sand.
- It has well defined failure pattern.
- Stress Zone in this Case extend Upto Ground Surface.
- A Sudden failure accompanied by tilting of foundation is observed in this Case.
- A bulging of ground adjacent to the foundation takes place.
- Before failure Settlement would be Small.
- Load Settlement Curve indicates that failure is abrupt & Ultimate load can be easily located.



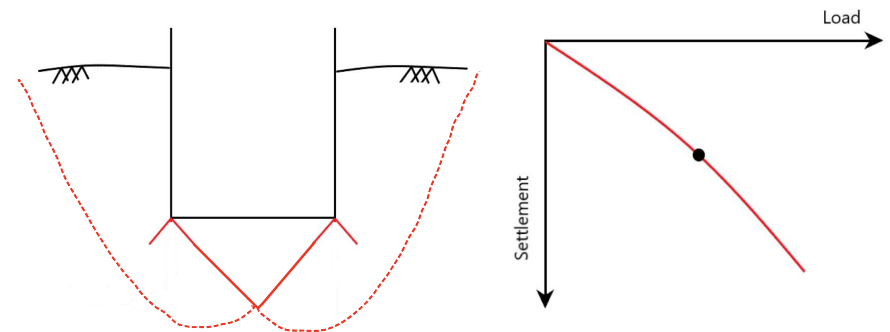
(II) Punching Shear Failure.

- It Occurs in Soils possessing the Stress-Strain Characteristics of Very plastic Soil.
- It is found in Shallow foundation in loose Sand/ NCC & in deep foundation.
 - It has poorly defined Shear plane.
 - Soil Zones beyond Loaded area is not/little affected.
 - Significant penetration/Settlement of wedge Shaped Soil Zone beneath the foundation is observed accompanied by Vertical Shear beneath edges of foundation.
- The load Settlement Curve indicates a Continuous increase in Settlement with increasing load.
- Ultimate load Cannot be clearly identified.



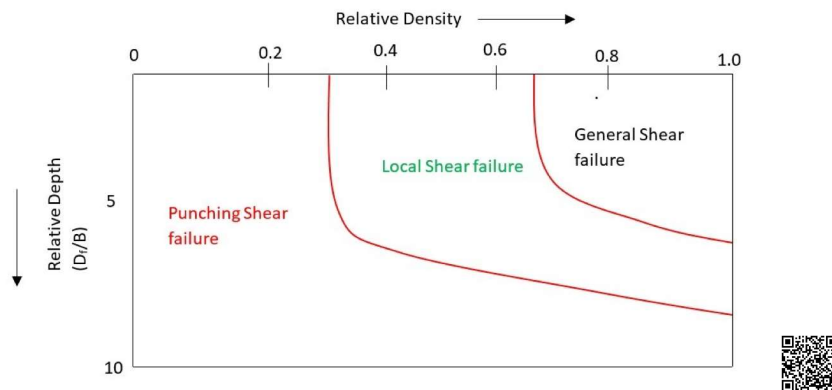
(III) Local Shear Failure.

- Its pattern has some of the Characteristics of Both general and Punching Shear failure.
- Main failure of Local Shear failure is its well defined wedge & Slip Surface only beneath the foundation.
- Slip Surface not Visible beyond the edges of the foundation.
- Slight bulging of the Ground Surface adjacent to the foundation may be Observed.
- Significant Settlement of Soil directly beneath the foundation takes place.
- The local-Settlement Curve does not indicate Ultimate load clearly.
- Soil possessing plastic Stress-Strain properties fail in this mode. eg. loose Sand.



Note: Guidelines to identify mode of failure.

	General	Local
<u>Sand</u>		
→ Frictional Angle (ϕ)	$> 36^\circ$	$< 28^\circ$
→ SPT No.	> 30	< 5
→ Relative Density	$> 70\%$	$< 30\%$
→ Void ratio	< 0.55	> 0.75
<u>Clay</u>		
→ UCS	$> 100 \text{ kN/m}^2$	$< 80 \text{ kN/m}^2$



• Factors Affecting Bearing Capacity

- Position of Ground Water table.
- Type of Soil & its physical & average properties (γ , c & ϕ).

- Type of foundation (Strip, Square, Circular).
- Size of footing (depth & width)
- Nature of Ground Surface (horizontal or inclined).
- Type of loading (Concentric/eccentric).
- Initial Stress on Soil (Normal or Overconsolidated).
- Type of Shear failure (General, Punching, local)

• Methods to Determine Bearing Capacity

- It Can be Analysed by any of the following –
 - Analytical/Static method.
 - Codal Provision: Various building codes are Published by Codal Agencies like BIS, NBC, IRC, CPWD etc in which bearing Capacity of Zonal Soil is Published which can be Used directly for rough Analysis.
 - Field method.
 - Plate load test (PLT)
 - Standard Penetration test (SPT)
 - Cone Penetration test (CPT).



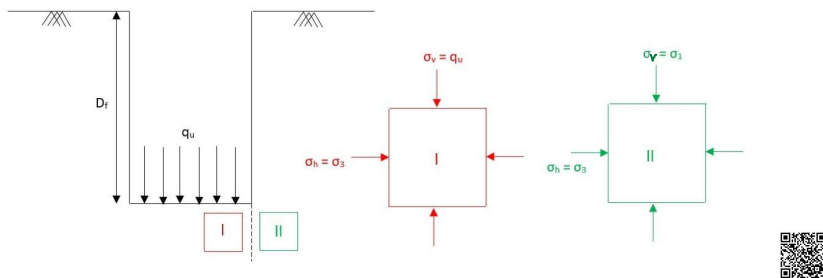
(I) Rankine's Method

- It considered the equilibrium of two soil elements, one immediately below the foundation (element I) & the other just beyond the edge of footing (element II), but adjacent to element I.
- When the load on footing increases & approaches Ultimate Bearing Capacity (q_u), a state of plastic equilibrium is reached under the footing.
- For the shear failure of element I, element II must also fail by lateral thrust from element I.
- During the state of shear failure (Plastic equilibrium), the analysis can be done as follows—

$$\sigma_1 = \sigma_3 \tan^2 \alpha + 2c \tan \alpha \quad *$$

For Cohesionless Soil, $c = 0$

$$\sigma_1 = \sigma_3 \tan^2 \alpha \quad *$$



- For element I, which is in active state of equilibrium.

$$\sigma_1 = q_u = \sigma_u$$

$$\sigma_3 = \sigma_h$$

$$q_u = \sigma_h \tan^2 \alpha \quad \text{--- (i)}$$

- for element II, which is in passive state of equilibrium.

$$\sigma_1 = \sigma_h$$

$$\sigma_3 = \sigma_u = \gamma D_f$$

$$\sigma_h = \gamma D_f \tan^2 \alpha \quad \text{--- (ii)}$$

From (i) & (ii),

$$q_u = \gamma D_f \tan^2 \alpha$$

or

$$q_u = \gamma D_f \left\{ \frac{1 + \sin \phi}{1 - \sin \phi} \right\}^2 \quad *$$

Note: In the effective stress analysis, q_u is given by,

$$q_u = \gamma' D_f \left\{ \frac{1 + \sin \phi'}{1 - \sin \phi'} \right\}^2$$

- This Analysis can also be used to find the minimum depth of foundation as follows,

$$D_{fmin} = \frac{q_u}{\gamma} \left\{ \frac{1 - \sin \phi}{1 + \sin \phi} \right\}^2$$



- This theory is Only applicable to Cohesionless soil.
- Effect of width & Size of footing is not Considered.
- If $D_f = 0$ then $q = 0$, Which is not practical.

(II) Fellenius Theory

- It is Based on plastic theory & is applicable for Only pure Cohesive Soil (Clays).
- In this theory, the rupture plane/failure plane is assumed to be an arc of Circle & general Shear failure is Considered.

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(III) Terzaghi's Theory

- It is an improvement Over Prandtl's theory.
- Prandtl Considered the base of the footing to be Smooth whereas, Terzaghi Considered the Base to be Rough.

• Assumptions in Terzaghi Theory

- (i) Foundation is Shallow ($D_f \leq B$).
- (ii) Base of foundation is rough.

(iii) footing is Continuous (Strip) ($L \gg B$). It means analysis in 2D along width & depth.

(iv) At the time of failure Soil reaches into plastic Stage.

(v) Failure is General Shear failure.

(vi) The Stress Zone of Soil extends upto the foundation level only but not Upto the Ground level.

(vii) The Shear resistance of Soil above the foundation is ignored, it means only base resistance is Considered while side resistance is Ignored.



Note: It is the main reason due to which this theory is not applicable for deep foundation.

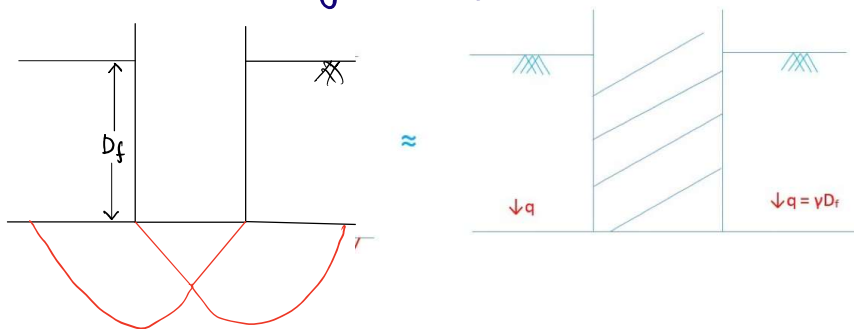
(viii) The Soil above the Base of foundation is removed & replaced by an equivalent Surcharge. ($q = \gamma D_f$).

(ix) The load is Vertical & Concentric (Note: if load is eccentric Meyerhoff's theory is Used.)

(x) Ground Surface is horizontal & foundation is also horizontal.

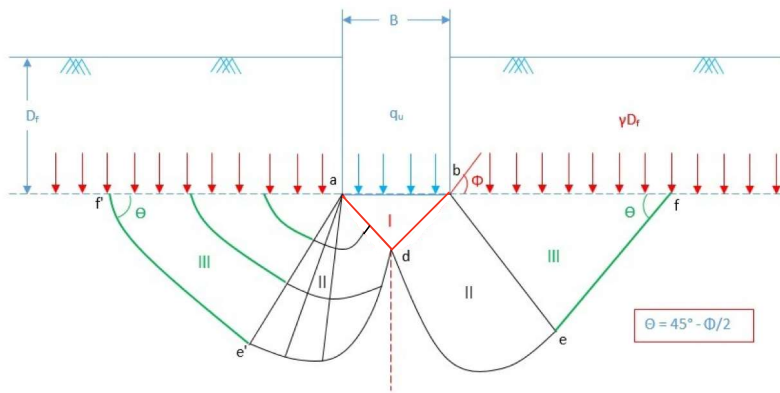
(xi) Water table is beyond the Zone of influence of stress hence, it does not effect the Bearing Capacity.

→ This theory is based on the limiting equilibrium approach, wherein the forces acting on soil wedge immediately beneath the foundation are examined for static equilibrium condition & the Ultimate bearing Capacity is determined.



→ The base of footing being rough, when the footing sinks into the soil, a certain portion of soil, i.e. soil wedge 'adb' immediately beneath the footing is prevented from undergoing any lateral movement by friction & adhesion between the base of footing and the soil.

- The wedge of soil 'adb' is termed as ZONE-I and remain in a state of 'ELASTIC EQUILIBRIUM' & is effectively a part of the footing itself.
- At failure, the Vertical downward movement of the footing & the intact soil wedge 'adb' pushes the soil on either side of elastic wedge & transforms it into a state of plastic equilibrium.
- In Zone II, where soil is in plastic equilibrium, it is termed as 'Zone of Radial Shear'.
- One set of shear plane in these zone radiate from the edge of footing & the curved lower boundary of these zone has shape of a circle (for cohesive soil) & logarithmic spiral (for c-φ soil).
- Zone III is termed as Rankine passive zone of linear shear in which the two sets of shear plane are inclined at angle of $(\theta = 45^\circ - \phi/2)$ to the horizontal.
- The lower boundary 'ad' & 'bd' of the elastic wedge are failure plane rising at angle of ϕ to the horizontal.
- The failure surface 'def' & 'def' are taken to be vertical at 'φ'.



- The footing cannot break into the soil, until the passive resistance of soil masses is Overcome.
- The inclined face 'ad' or 'bd' can be considered as a rough back of rigid wall with shear parameters c & ϕ .
- At failure, $\Sigma F_y = 0$

$$q_u(B/2) + \gamma \left(\frac{1}{2} \times B \times \frac{B}{2} \tan \phi \right) = 2P_p + 2c \cdot \sin \phi$$

$$c = c \times (ad \times 1) = c \left(\frac{B/2}{\cos \phi} \right)$$

$$q_u B = 2P_p + BC \tan \phi - \frac{1}{4} \gamma B^2 \tan \phi$$



Note: The total passive resistance P_p on the Surface ad & bd is made Upto 3 Components –

- P_{py} produced by weight of Soil in Shear Zone assuming the soil to be Cohesionless ($c=0$) & neglecting the Surcharge.
- P_{pc} produced due to Soil Cohesion, assuming soil to be weight less ($\gamma=0$) and neglecting the Surcharge.
- P_{pq} produced by the Surcharge assuming Soil to be Cohesionless & Weightless ($c=0, \gamma=0$).

$$\Rightarrow q_u \cdot B = 2(P_{py} + P_{pc} + P_{pq}) + BC \tan \phi - \frac{1}{4} \gamma B^2 \tan \phi$$

$$\text{Assuming } 2P_{py} - \frac{1}{4} \gamma B^2 \tan \phi = B \times \frac{1}{2} \times B N_\gamma$$

$$2P_{pc} + BC \tan \phi = B \times C \cdot N_c$$

$$2P_{pq} = B \times N_q$$

$$q_u B = BC N_c + B \cdot q \cdot N_q + \frac{1}{2} \gamma B N_\gamma$$

$$q_u = C N_c + q N_q + \frac{1}{2} \gamma N_\gamma$$

I II III

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



- γ in II term is for Soil above the base of footing.
 - γ in III term is for Soil below the base of footing.
 - c is unit Cohesion for Soil below the footing.
 - N_c, N_q, N_γ are bearing Capacity factors which depends upon friction angle (ϕ) of Soil.
- $$N_\phi = \tan^2(45^\circ + \phi/2) \Rightarrow N_q = N_\phi e^{\pi \tan \phi}$$
- $$N_\gamma = 1.8 \cdot \tan \phi (N_q - 1)$$
- $$N_c = \cot \phi (N_q - 1)$$

Note: For Pure Clay ($\phi = 0$)

$$N_\phi = 1$$

$$N_q = 1$$

$$N_\gamma = 0$$

$$N_c = 5.7 \text{ (As per L'Hospital rule.)}$$

$$q_u = 5.7c + \gamma D_f \times 1 + 0.5B \cdot \gamma(0)$$

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

$$q_{nu} = q_u - \gamma D_f = 5.7c + \gamma D_f - \gamma D_f$$

$$q_{nu} = 5.7c$$

- The net Ultimate Bearing Capacity of pure Cohesive Soil is independent of its Size & depth.

- The Bearing Capacity q in Terzaghi & Prandtl theory is same with difference in only Bearing Capacity factor.

eg: for Pure Clay as per Prandtl theory,

$$N_c = 5.14$$

$$N_q = 1$$

$$N_\gamma = 0$$



(i) Modification for Different Shape of the footing

(a) for Strip footing: $q_u = CN_c + qN_q + 0.5B \cdot \gamma N_\gamma$

(b) for Square footing: $q_u = 1.3CN_c + qN_q + 0.4B \gamma N_\gamma$

(c) for Circular footing: $q_u = 1.3CN_c + qN_q + 0.3B \gamma N_\gamma$

(d) for rectangular/Raft footing:

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

	Strip	Square	Circular	Rectangular/Raft
S_c	1	1.3	1.3	$(1 + 0.3 B/L)$
S_q	1	1	1	1
S_γ	1	0.8	0.6	$(1 - 0.2 B/L)$

(ii) Modification for Shear failure

- Terzaghi's theory is applicable for general Shear Failure.

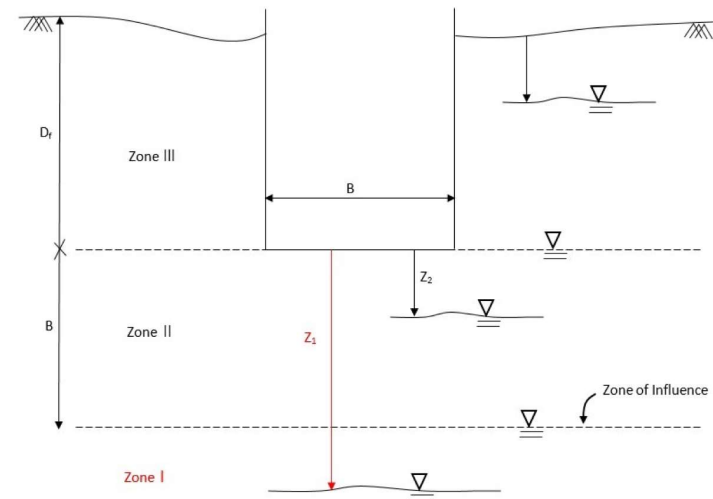
- No theoretical solution is available for local & Punching Shear failure.
 - Punching Shear failure is very uncommon, since footing are rarely placed on very loose sand.
 - Local Shear failure is however common.
 - In case of footing on loose sand or soft clays where local shear failure can take place, shearing resistance is not mobilised along the entire length of failure surface as observed in general shear failure.
 - Terzaghi proposed empirical adjustments to shear strength parameters (c, ϕ) to cover the case of local shear failure.
 - Shear strength parameters c_m & ϕ_m must be used in bearing capacity equation & bearing capacity factors are determined on basis of ϕ_m .
- $$c_m = \frac{2}{3}c \quad \& \quad \tan \phi_m = \frac{2}{3}\tan \phi$$

Hence,

$$q_u = \frac{2}{3}CN_c' - qNq' + 0.3B\gamma N\gamma'$$



(iii) Modification due to Water table level



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

(i) (ii) (iii)



- Case (i): When water table is in Zone-I i.e. ($Z_1 > B$)
- Water table level below the Zone of influence of stress.
- Water table in this Zone has no influence on the bearing capacity. hence, no change in bearing capacity is introduced i.e. terms (i), (ii) & (iii) remains unaffected.

→ Case (ii): When Water table is in Zone-II, at depth of Z_2 below the foundation. ($0 \leq Z_2 \leq B$).

→ In this Case terms (i) & (iii) of bearing Capacity equation will be affected whereas term (ii) will remain Unaffected.

→ To account for Water table effect either use effective parameters c' , ϕ' & effective Unit Weight of the Soil or Use water table Correction factor.

→ Generally, effect on c & ϕ is neglected in absence of data & doesnot given, but effect on Unit weight of Soil is accounted, i.e. effect on first term is negative.

∴ In absence of data, $c' \simeq c$
 $\phi' \simeq \phi$



Effective Parameters	Water table Correction factor
$q_u = c'N_c' + \gamma D_f N_q' + 0.5B \gamma_c N_{\gamma'}$ $\gamma_c = \frac{Z_2 \gamma + (B - Z_2) \gamma'}{B}$	$q_u = c'N_c' + \gamma D_f N_q' + 0.5B R \gamma + \gamma_a N_{\gamma'}$ $\gamma_a = \frac{Z_2 \gamma + (B - Z_2) \gamma_{sat}}{B}$

$N_q' = N_q$ $\text{If } Z_2 = B, \gamma_c = \gamma$ $q_u = c'N_c' + \gamma D_f N_q + 0.5B \gamma N_{\gamma'}$ $\text{If } Z_2 = 0, \gamma_c = \gamma'$ $q_u = c'N_c' + \gamma D_f N_q$	$R \gamma = \frac{1}{2} \left\{ 1 + \frac{Z_2}{B} \right\}$ $\text{If } Z_2 = 0, \gamma_a = \gamma_{sat}, R \gamma = \frac{1}{2}$ $q_u = c'N_c' + \gamma D_f N_q + 0.5B \left\{ \frac{1}{2} \gamma_{sat} \right\} N_{\gamma'}$
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→ Case (iii): When Water table is in Zone-III, i.e. at the depth Z_3 from Ground level. ($0 \leq Z_3 \leq D_f$).

→ In this Case, all three terms of Bearing Capacity equation (i), (ii), (iii) will be affected, but effect on Ist term (i) is negligible.

→ Here also either use effective parameters or water table Correction factor.



Effective Parameter	Water table Correction Factor
$q_u = c'N_c' + \gamma_e D_f N_q' + 0.5B \gamma' N_{\gamma'}$ $\gamma_e = \frac{Z_3 \gamma + (D_f - Z_3) \gamma'}{D_f}$	$q_u = c'N_c' + (R_q^* \gamma_a) D_f N_q' + 0.5B \gamma' N_{\gamma'}$ $\gamma_a = \frac{Z_3 \gamma + (D_f - Z_3) \gamma_{sat}}{D_f}$

→ If $Z_3 = 0, \gamma_e = \gamma'$
 $q_u = C'N_c' + \gamma' D_f N_q' + 0.5 B \gamma' N_{\gamma}'$

→ If $Z_3 = D_f, \gamma_e = \gamma$
 $q_u = C'N_c' + \gamma D_f N_q + 0.5 B \gamma' N_{\gamma}'$

$$R_q^* = \frac{1}{2} \left(1 + \frac{Z_3}{D_f} \right)$$

→ If $Z_3 = 0, \gamma_a = \gamma_{sat}, R_q^* = \frac{1}{2}$
 $q_u = C'N_c' + \left(\frac{1}{2} \gamma_{sat} \right) D_f N_q' + 0.5 B \gamma' N_{\gamma}'$

→ If $Z_3 = D_f, \gamma_a = \gamma, R_q^* = 1$
 $q_u = C'N_c' + \gamma D_f N_q + 0.5 B \gamma' N_{\gamma}'$

• Special Case

(i) for Cohesionless Soil ($c = 0$) } i.e. Sand {

$$q_u = C N_c + \gamma D_f N_q + 0.5 B \gamma N_{\gamma}$$

$$q_u = \gamma D_f N_q + 0.5 B \gamma N_{\gamma}$$

If Water table rises to the Ground level

$$q_u = \gamma' D_f N_q' + 0.5 B \gamma' N_{\gamma}'$$

or

$$q_u = \left(\frac{1}{2} \gamma_{sat} \right) D_f N_q' + 0.5 \left(\frac{1}{2} \gamma_{sat} \right) N_{\gamma}'$$

→ In Cohesionless Soil, Ultimate Bearing Capacity is proportional to depth (D_f) & width of foundation (B).

→ Due to rise in water table level upto Ground level

Ultimate Bearing Capacity approximately reduces to half.

(ii) For Pure Cohesive Soil ($\phi = 0$) } i.e. Clay {

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

$$q_u = C N_c + \gamma D_f N_q + 0.5 B \gamma N_{\gamma}$$

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

→ If Water table rises to the Ground level.

$$q_u = 5.7 C' + \gamma' D_f \Rightarrow q_{nu} = 5.7 C' + \gamma' D_f - \gamma' D_f = 5.7 C'$$

$$q_{nu} = 5.7 C'$$

- The Ultimate Bearing Capacity of foundation placed on Clay is independent of width/Size of foundation, but is dependent on its depth (D_f).
- Net Ultimate bearing Capacity is nearly Unaffected by rise of Water table to the Ground level.

(IV) SKEMPTON THEORY

- This theory is Partly Based on field Observation partly on laboratory test & Partly on theoretical Analysis.
- It is Suitable for pure Cohesive Soil (Clay).

- It can be applied for both Shallow footing as well as deep footing.
- In this theory, base resistance and side resistance both are considered.

The net Ultimate bearing Capacity is given as—

$$q_{nu} = C N_c^*$$

Where,

C = Unit Cohesion

N_c = Skempton bearing Capacity factor
Which depends Upon $\frac{D_f}{B}$ ratio.

- Case(i) If $\frac{D_f}{B} = 0$,

<u>Footing</u>	<u>Strip</u>	<u>Square/Circular/Raft/ Rectangular</u>
N_c	5	6

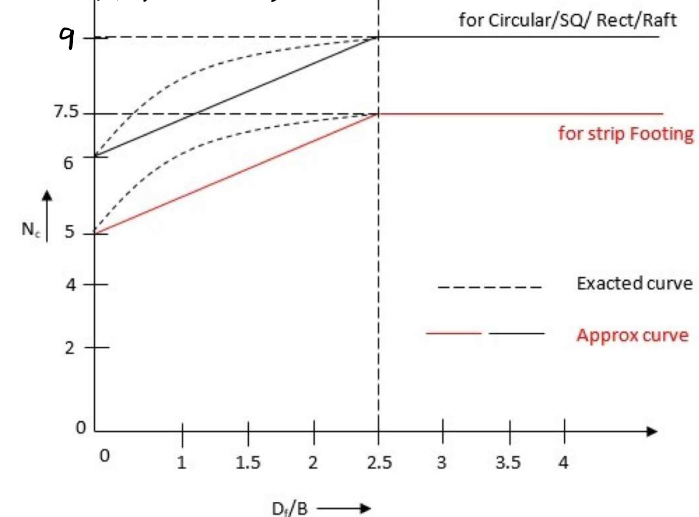
- Case(ii) If $\frac{D_f}{B} \geq 2.5$

<u>Footing</u>	<u>Strip</u>	<u>Square/Circular/Raft/ Rectangular</u>
N_c	9.5	9

- Case(iii) if $0 < \frac{D_f}{B} < 2.5$

<u>Footing</u>	<u>Strip</u>	<u>Square</u>	<u>Rectangular/Raft</u>
N_c	$5(1 + 0.2 \frac{D_f}{B})$	$6(1 + 0.2 \frac{D_f}{B})$	$5(1 + 0.2 \frac{D_f}{B})(1 + 0.2 \frac{B}{L})$

Note: Unlike Terzaghi's theory Skempton's can apply at any depth.



(V) MEYERHOF'S THEORY

- It is the most Generalised theory available to find the bearing Capacity as it gives Comprehensive analysis for bearing parameters.
- The failure mechanism in this is Similar to that

of Terzaghi but it considered failure surface that extended above the foundation level, hence Shearing Resistance of soil above the base of foundation is also taken into account.

→ Ultimate bearing Capacity for Shallow foundation in this Case is given by –

$$q_u = \underbrace{C N_c}_{(i)} S_c d_c i_c + \underbrace{q N_q}_{(ii)} S_q d_q i_q + \underbrace{0.5 B \gamma N_\gamma}_{(iii)} S_\gamma d_\gamma i_\gamma$$

→ Here, N_c, N_q, N_γ are Meyerhof bearing Capacity factor Same as proposed by Prandtl.

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

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

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

→ Here S, d, i are Shape factor, depth factor & inclination factor for each of Bearing Capacity term respectively, given as –

Factors	Expressions
S_c, S_q, S_γ S_c	1 for Strip footing $1 + 0.2 \frac{B}{L} \tan^2(45^\circ + \phi/2)$

• S_q, S_γ	$1 + 0.1 \frac{B}{L} \tan^2(45^\circ + \phi/2)$ for $\phi > 10^\circ$ and 1 for $\phi = 0$.
• d_c	$1 + 0.2 \frac{D}{B} \tan(45^\circ + \frac{\phi}{2})$
• d_q, d_γ	$1 + 0.1 \frac{D}{B} \tan(45^\circ + \frac{\phi}{2})$
• i_c, i_q	$(1 - \frac{\alpha}{90})^2$, α is in degree
• i_γ	$(1 - \frac{\alpha}{90})^2$

- Here, 'B' refers to the width or diameter of foundation.
- The inclination factor 'i' takes into account the effect of inclination of load on Bearing Capacity.
- 'α' is the inclination of the resultant force from the Vertical. { i.e. $\alpha = \tan^{-1}(H/V)$ }
- Meyerhof also recommends that in case of Strip & rectangular footing involving plain Strain value of q_{ps} is determined from the triaxial test should be converted to plain Strain as follows –

$$\phi_{ps} = \left\{ 1.1 - 0.1 \frac{B}{L} \right\} \cdot \phi_{tr}$$

- Meyerhof introduced the concept of Useful width to compute the bearing capacity when the resultant load on footing eccentrically with respect to centre of the footing.
- To account for eccentricity of loading the footing dimensions are modified in such a way that becomes concentric to the reduced dimensions of the footing.
- For a strip footing, if the load has an eccentricity e_x in direction of width a modified width ' B' ' is used in place of B in bearing capacity equation & in the determination of shape and depth factors.

$$B' = B - 2e_x$$

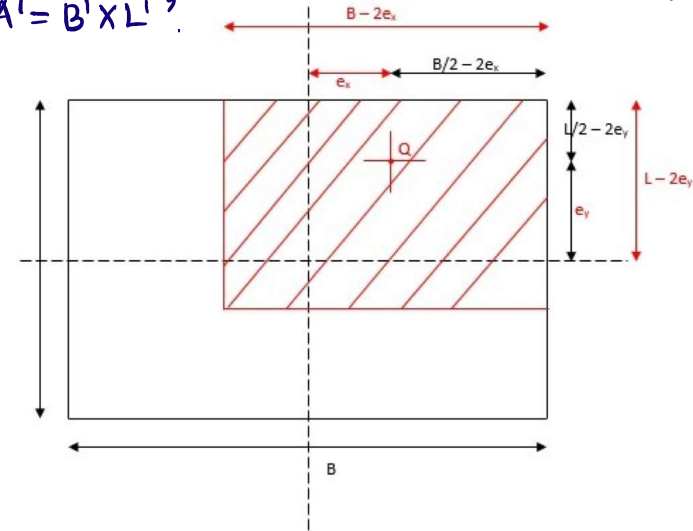
- In a rectangular footing, there can be eccentricity of loading in direction of both width & length equal to e_x & e_y .
- In such case, reduced dimension B' & L' are used in place of actual dimension.

$$B' = B - 2e_x$$

$$L' = L - 2e_y$$



- The effective area ' A ' for the purpose of the calculation of total vertical load Q is given by ' $A' = B' \times L'$ '.



(VI) IS CODE RECOMMENDATION FOR BEARING CAPACITY (IS 6403)

- IS code recommends that for computation of ultimate bearing capacity c/a shallow foundation in general shear failure, following equation is used.

$$q_{nu} = C N_c S_c d_c i_c + q (N_q - 1) S_q d_q i_q + 0.5 B \gamma N_\gamma S_\gamma d_\gamma i_\gamma w'$$

- Here, N_c, N_q, N_γ are bearing capacity factors recommended by Vesic.

- $N_q = c \pi \tan \phi \cdot \tan^2(45^\circ + \phi/2)$
- $N_\gamma = 2(N_q + 1) \cdot \tan \phi$
- $N_c = (N_q - 1) \cdot \cot \phi$



→ Here, w' is water table correction factor & it is equivalent to R_γ^* .

$$w' = \frac{1}{2} \left(1 + \frac{Z_2}{B} \right)^*$$

- Z_2 is measured from base of foundation.
- γ in third term is γ_{avg} .
- The influence of Water table is taken care in second term of Bearing Capacity equation by taking 'q' as effective surcharge at the level of base of the footing.
- The Slope, depth & inclination factors are as follows-

Factor	Expressions
S_c	$1 + 0.2 \frac{B}{L}$ for rectangle
	1.3 for Square & Circle
S_q	$1 + 0.2 \frac{B}{L}$ for rectangle.
	1.2 for Square & Circle.

S_γ	$1 - 0.4 \frac{B}{L}$ for rectangle.
	0.8 for Square & 0.6 for Circle.
d_c	$1 + 0.2 \frac{D_f}{B} \tan(45^\circ + \frac{\phi}{2})$
$d_q = d_\gamma$	$1 + 0.1 \frac{D_f}{B} \tan(45^\circ + \frac{\phi}{2})$ for $\phi > 10^\circ$ & 1 for $\phi < 10^\circ$.
$i_c = i_q$	$(1 - \frac{\alpha}{90})^2$, α in degrees
i_γ	$(1 - \frac{\alpha}{\phi})^2$



→ In local failure, the recommendations are same as of Terzaghi theory.

$$\therefore C_m = \frac{2}{3} C, \therefore \tan \phi_m = \frac{2}{3} \tan \phi.$$

→ For Cohesive Soil, net Ultimate Bearing Capacity of a footing immediately upon construction ($\phi_u = 0$) is given by -

$$q_{nu} = C_u N_c S_c d_c i_c$$

→ Here, $N_c = 5.14$ & the Undrained Shear Strength C_u is obtained either from Unconfined Compres-

since Strength test or from co-relation with point resistance value obtained from Static Cone penetration test.

→ C_u Varied between $\frac{q_c}{18} - \frac{q_c}{15}$ for NCC & between $\frac{q_c}{26} - \frac{q_c}{22}$ for OCC } Where q_u is point resistance.



• Comparison of Bearing Capacity Factors.

→ (i) $N_c = (N_q - 1) \cot \phi$, $N_q = e^{\pi \tan \phi} \cdot \tan^2(45^\circ + \phi/2)$

If $\phi = 0$, $N_c = 5.14$ (instead of 5.7 given by Terzaghi).

→ The above two equations have been adopted by-

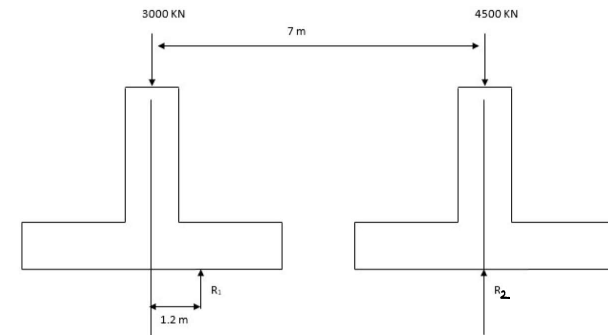
- Terzaghi & Peck
- Meyerhof
- Hansen
- Vesic
- BIS

→ (ii) $N_\gamma = (N_q - 1) \cdot \tan(1.4 \phi)$ Meyerhof
 $N_\gamma = 1.5(N_q - 1) \cdot \tan \phi$ Hansen
 $N_\gamma = 2(N_q + 1) \cdot \tan \phi$ Vesic
 $N_\gamma = 1.1(N_q - 1) \cdot \tan 1.3 \phi$ Spangler & Hardy

Note: N_γ factor has the widest range of values for any N factor.

→ In General, $38 \leq N_\gamma \leq 192$, for $\phi = 40^\circ$.

Q. A Strap footing is to be provided for two Columns as Shown



Calculate the length of footing considering width of both footing as 3.5 m & Safe Bearing Capacity of Soil as 350 kN/m².

- (a) 2 m (b) 2.5 m (c) 3.5 m (d) 2.95 m

Note: The Strap footing is considered to be Shift, transferring the Column loads with equal & Uniform Soil pressure Under Both footing.

$$\sum M_{R2} = 0$$

$$\sum F_y = 0$$

$$3000 \times 7 - R_1(7 - 1.2) = 0 \quad 3000 + 4500 - R_1 - R_2 = 0$$

$$R_1 = 3620.7 \text{ kN}$$

$$R_2 = 3000 + 4500$$

$$- 3620.7$$

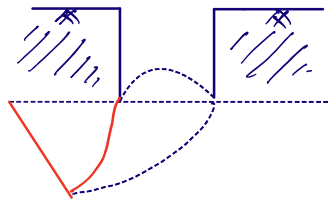
$$R_2 = 3879.3 \text{ KN.}$$

$$q_u = \frac{R_1}{L_1 B} \Rightarrow L_1 = \frac{3620.7}{3.5 \times 350} = 2.95 \text{ m.}$$

$$q_u = \frac{R_2}{L_2 B} \Rightarrow L_2 = \frac{3879.3}{3.5 \times 350} = 3.167 \text{ m.}$$

Q. In a Braced Vertical excavation of 5m height & 2.5 m width in a Cohesive Soil having Undrained Cohesion = 20 kN/m² & $\gamma_s = 20 \text{ kN/m}^3$. What is the FOS Against heave failure at Base.

Sol:



$$\frac{D_f}{B} = \frac{5}{2.5} = 2$$

$$\frac{D_f}{B} < 2.5$$

For No failure, $q_n \leq q_{ns}$
 $\Rightarrow \gamma H = \frac{q_{nu}}{F} \Rightarrow F = \frac{CN_c}{\gamma H}$

$$\Rightarrow F = \frac{20 \times 5}{20.5} \left\{ 1 + 0.2 \frac{D_f}{B} \right\} = 1.4$$

Q. An embankment is to be constructed with a Granular Soil ($\gamma = 20 \text{ kN/m}^3$) on a Saturated Clayey Silt deposit (Undrained Shear Strength = 25 kPa). Assuming undrained general shear failure & Bearing Capacity factor of 5.7. Compute the max. height of the embankment at point of failure.

Sol: $q_n = \gamma H = 204 \text{ kN/m}^2$

$$q_{nu} = CN_c = 25 \times 5.7 = 142.5 \text{ kN/m}^2$$

At point of failure, FOS (F) = 1

For Safe designing $q_n \leq q_{ns} \Rightarrow 20H = 142.5$

$$H = 7.125 \text{ m}$$

Q. A building has to be Supported on a RCC raft foundation of dimensions 14m X 21m. The Subsoil is Clay which has an average Unconfined Compressive Strength of 0.15 kg/cm². The pressure on the Soil due to the weight of building & load that it will carry will be 14 t/m² at the Base of raft. If the γ of excavated Soil is 1.9 t/m³ at What depth should the bottom of raft be placed to provide a FOS of 3 against Shear failure.



Sol: $q_g = 14 \text{ t/m}^2$, $q_n = 14 - \gamma D_f$
 $q_{nu} = CN_c = \left\{ \frac{0.15 \times 10^{-3}}{2 \times 10^{-4}} \right\} \times 5 \times (1 + 0.2 \frac{D_f}{B}) \times \left\{ 1 + 0.2 \frac{B}{L} \right\}$
 $= 0.75 \times 5 \times \left\{ 1 + 0.2 \frac{D_f}{14} \right\} \left(1 + 0.2 \times \frac{14}{21} \right)$
 $= 4.25 + 0.06 D_f$

$$q_{ns} = \frac{q_{nu}}{F} = \frac{4.25 + 0.06 D_f}{3} = 1.4165 + 0.02 D_f$$

for Safe design of footing, $q_n \leq q_{ns}$
 $14 - 1.9 D_f = 1.4165 + 0.02 D_f \Rightarrow \underline{D_f = 6.56 \text{ m}}$

Q. A Square footing of 2 m Sides rests on the Surface of a homogeneous Soil bed having the properties $C = 24 \text{ kPa}$, $\phi = 25^\circ$, $\gamma = 18 \text{ kN/m}^3$. Terzaghi Bearing Capacity factors for $\phi = 25^\circ$ are $N_c = 25.1$, $N_q = 27.1$, $N_\gamma = 9.7$, $N_c' = 14.8$, $N_q' = 5.6$, $N_\gamma' = 3.2$.

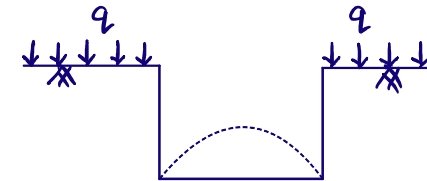
Compute the Ultimate Bearing Capacity of the foundation.

Sol: $q_u = 1.30 C' N_c' + q N_q' + 0.4 B \gamma N_\gamma'$
 $\Rightarrow C' = \frac{2}{3} \cdot C$

$$q_u = 1.3 \times \left\{ \frac{2}{3} \times 24 \right\} \times 14.8 + 0 + 0.4 \times 2 \times 18 \times 3.2$$

$$q_u = 353.9 \text{ kPa.}$$

Q. A raft foundation 10 m wide & 12 m long is to be Constructed in a Clayey Soil having Shear Strength of 12 kN/m^2 , γ of Soil is 16 kN/m^3 . The Ground Surface Carries a Surcharge of 20 kN/m^2 , $FOS = 1.2$ & $N_c = 5.7$. Find the Safe depth of foundation.



Sol: $q_u = \left\{ 1 + 0.3 \frac{B}{L} \right\} C N_c + (\gamma D_f + q) N_q + \left\{ 1 - 0.2 \frac{B}{L} \right\} \times 0.5 B \cdot \gamma N_\gamma$

$$q_u = \left\{ 1 + 0.3 \left(\frac{10}{12} \right) \right\} 12 \times 5.7 + 16 \times D_f + 20$$

$$\Rightarrow q_u = 105.5 + 16 D_f \text{ kN/m}^2$$

$$q_{nu} = q_u - \gamma D_f = 105.5 \text{ kN/m}^2$$

$$\Rightarrow q_{ns} = \frac{q_{nu}}{F} = \frac{105.5}{1.2} = 87.91$$

For Safe design, $q_n \leq q_{ns}$

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

$$\Rightarrow D_f = \frac{87.91}{16} = \underline{\underline{5.49 \text{ m}}}$$

(III) FIELD METHOD



(i) Plate load test (IS 1888)

- The test was designed to determine modulus of Subgrade reaction which is Used in designing of rigid Pavement.
- It is field test to determine the Ultimate Bearing Capacity of Soil, and probable Settlement Under a given loading.
- The test essentially Consists of loading a rigid plate at foundation level & determining the Settlement Corresponding to load increment.
- The Ultimate Bearing is then taken at the load at Which the plate Starts Sinking at a rapid rate.
- This method assumes that down to the depth of influence of Stresses the Soil Strata is reasonably Uniform.

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(a) Bearing Plate: The bearing is either Square or Circular made of mild Steel of not less than 25 mm thickness & Size ranging from 300 – 750 mm.

- Small Size plate is Used for dense & Stiff Soil Whereas Large Size plate is Used for loose & Soft Soils.
- The Size of plate Shall be atleast 4 times the maximum Size of the Soil particles present at test location.

(b) Test pit: The test pit, usually at foundation level, must have normally width equal to five times the Size of the test plate.

- The test pit should preferably have Steps to go into the pit for Setting & marking reading.

(c) Loading Arrangement: The loading to the test plate may be applied with the help of a hydraulic jack. The reaction of the hydraulic Jack can be taken by any of two methods

- Gravity loading method.
- Reaction Truss method.



- When load is applied to the plate, it Sinks to Settle where Settlement of plate is measured with the help of atleast two dial gauges Which are mounted on independent datum bar.
- No Support of loading platform should be locked within a distance of test plate from its Centre.



- (d) Setting of plates: The test plate shall be placed over a fine sand layer of max. thickness 5mm so that the Centre of plate coincides with the Centre of the reaction girders/beam.
- A minimum Seating pressure of 0.7 t/m^2 shall be applied & removed before starting the load test.

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- (e) Load Increments: Apply the load to Soil in Cumulative equal increment upto 10 t/m^2 or $1/5$ of the estimated Ultimate Bearing Capacity, Whichever is less.



- (f) Settlement & Observation: Settlement should be observed for each increment of load at interval of 1, 2.25, 4, 6.25, 9, 16, 25 mins &

there after at every one hour interval to the nearest of 0.02mm.

- In Case of Clayey Soils, the time-Settlement Curve shall be plotted at each load Stage & load shall be increased to next Stage either When the Curve indicates that the Settlement has exceeded 70-80% of Probable Ultimate Settlement or at the end of 24 hour period.
- For Soils other than Clay, rate of Settlement get appreciably reduced to a Value 0.02 mm/min.
- The next increment of load shall then be applied & Observation is repeated.
- The test shall be Continued till a Settlement of 25 mm or (50 mm extreme/Special Case) Such as in dense Gravel, Gravel & Sand mix is Obtained or till failure, Whichever is earlier.
- When Settlement doesnot reach 25 mm, the test should be Continued to atleast 'two times' the estimated design pressure.



- (g) load Settlement Curve & Ultimate Bearing Capacity:

- A load Settlement Curve is plotted out to

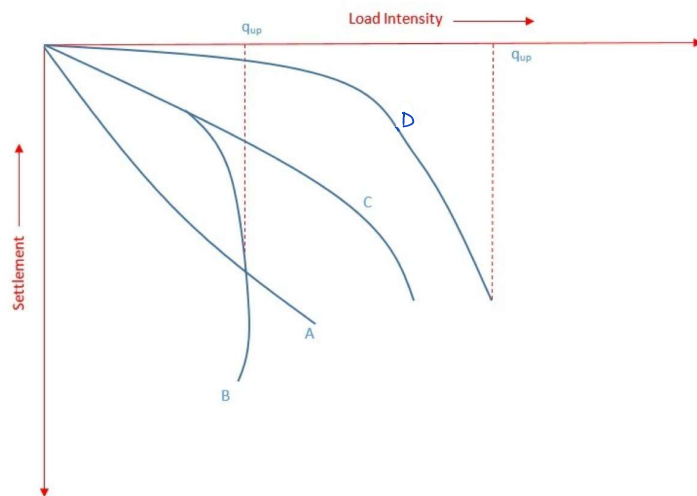
arithmetic Scale, Standard nature of which for different soils is as follows :

A: loose to medium Cohesionless Soil.

B: Cohesive Soil.

C: Partially Cohesive Soil.

D: Dense Cohesive Soil.



→ Curve A: Is typical for loose to medium Cohesionless Soil which is Straight line in early Stages but flattens out in later Stage and there is no clear point of failure.

→ Curve B: Is for Cohesive Soil which is not Quite

Straight in early Stages & leans towards Settlement axis as the Settlement increases.

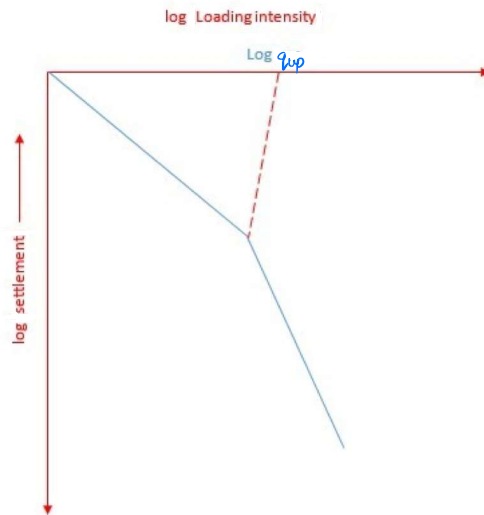
→ Curve C: Possess Characteristics of Both Curve A & B.

→ Curve D: Is for Pure dense Cohesionless Soil.

Note: for Curve B & D, no difficulty is experienced in Observing Ultimate Bearing Capacity as the failure is well defined.

→ However, in Case of Curve A & C, Where yield point is not well defined, Settlement Curve is plotted on Semi-log Scale.

→ IS Code does not Specify any FOS, hence in Order to determine the Safe Bearing Capacity it is taken as 2-2.5.



• Determination of Ultimate Bearing Capacity as per Shear Criteria

→ Case A) for Clays: In Clays, bearing Capacity is approximately independent of width of footing hence -

$$q_{uf} = q_{up}$$

∴ q_{uf} = Ultimate Bearing Capacity for footing.
 q_{up} = Ultimate Bearing Capacity for plate which is determined from load intensity v/s Settlement Curve.



→ Case B) for Sand: In Sandy Soil, bearing Capacity is proportional to the width of the footing.

$$q_{uf} = q_{up} \cdot \frac{B_f}{B_p} \quad \{q_{uf} \propto B_f\}$$



B_f = Width of footing.
 B_p = Width of Plate.

Note: To Compute Safe Bearing Capacity, Fos of 2-2.5 is applied over Bearing Capacity Computed above.

• Determination of Safe Settlement pressure using PLT as per Settlement Criteria.

→ Let S_f be the Permissible Settlement of footing prescribed by Codal agencies then by Using following empirical relation.

$$\frac{S_f}{S_p} = \left\{ \frac{B_f(B_p + 0.3)}{B_p(B_f + 0.3)} \right\}^2 \quad (\text{for dense Sand})$$

B_p & B_f is in m.

$$\frac{S_f}{S_p} = \frac{B_f}{B_p} \quad \text{for Clays.}$$

$$\frac{S_f}{S_p} = \left\{ \frac{B_f}{B_p} \right\}^{n+1} \quad \text{for Silt } (n=0.5).$$

→ Using above relationship for a particular Soil, find the permissible Settlement of the plate (S_p) and then Using load intensity Settlement Curve, read the Safe Bearing pressure for the plate (q_{up}) which is further Used to find Safe Bearing pressure for footing (q_{uf}) by use of following relationship -

$$q_{af} = q_{ap} \quad \text{for Clays.}$$

$$q_{af} = q_{ap} \cdot \frac{B_f}{B_p} \quad \text{for Sands.}$$

q_{af} = allowable bearing pressure for footing.
 q_{ap} = allowable bearing pressure for plate.

Note: The above Computed pressure is allowable pressure or Safe pressure in which further FOS is not required because permissible Settlement for footing, given by IS Code is Safe Settlement.

Q. The load Settlement Curve data from a plate load test for Sandy Soil are as follows -

load (t/m^2)	10	20	30	40	50	60	70	80
Settlement (mm)	4.5	10	15.5	22	29	38.5	50	64

The Size of plate Used was 0.8 m X 0.3 m. Determine the Size of Square Column footing to Carry a load of 250 t with a maximum Settlement of 25 mm.

Sol: Since B_f is not known,

$$\text{let } B_f = 1.5 \text{ m.}$$

$$q_{uf} = \frac{250}{1.5^2} = 111.11 \text{ t/m}^2$$

$$q_{uf} = q_{up} \cdot \frac{B_f}{B_p} \Rightarrow q_{up} = 22.22 \text{ t/m}^2$$

Settlement from table → for 22.22 t/m^2 load, by interpolation,

$$S_p = 11.11 \text{ mm.}$$

$$\frac{S_f}{S_p} = \left\{ \frac{B_f(B_p + 0.3)}{B_p(B_f + 0.3)} \right\}^2 \quad S_f = 30.83 \text{ mm} > 25 \text{ mm}$$

Redesign,

Increase B_p in ratio of $\sqrt{\frac{30.83}{25}}$

$$B_f = 1.5 \times \sqrt{\frac{30.83}{25}} = 1.67 \text{ m} \Rightarrow q_{uf} = \frac{250}{1.67^2} = 89.64 \text{ t/m}^2$$

$$q_{up} = 16.1 \text{ t/m}^2, \text{ from table } S_p = 7.855 \text{ mm}$$

$$S_f = 22.58 \text{ mm} < 25 \text{ mm.}$$

• Housel Approach to find Ultimate Bearing Capacity.

- According to Housel, the failure load is function of area and perimeter of plate/foundation & it is also influenced by Soil properties.
- In this method, PLT is Conducted on two rigid plates of different Size at Same 'D_f'.

- Let A₁ & P₁ be area & Perimeter of the 1st plate & Q₁ is failure load on 1st plate then -

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

Where, m and n are Constants which depends on type of Soil.

- Let A₂ & P₂ be area & Perimeter of 2nd plate and Q₂ is failure/Ultimate load on 2nd plate Corresponding to the given Settlement.

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

From (i) & (ii), m & n are Computed.

- If area of footing is A_f & Perimeter of footing P_f then Ultimate load at failure of foundation.

$$Q_f = A_f m + P_f n$$

$$\text{Ultimate Bearing Capacity, } q_{uf} = \frac{Q_f}{A_f}$$



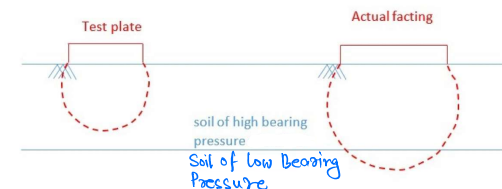
- Some of the most important Considerations which need to be taken Case of are as follows -

- In no Case Shall a test plate Smaller than 30 cm can be Used because load-Settlement Behaviour of Soil is Qualitatively different for Smaller width of test plate.

- It Can be Shown for Settlement equation 'x' Settlement of a foundation Cannot exceed about four times the Settlement of plate of 30 cm width, howsoever large is the width may be, hence this equation is Valid Only for medium to dense Sand.

- If the Soil at Site is not homogeneous upto a large depth relative to the Size of footing, this test gives misleading result (It would give much Smaller Settlement than actual).

- Hence, plate load test must be Supplemented by adequate Soil exploration through Borehole, Which may reveal any non-homogeneity of Strata, upto a depth of 1.5-2 times the width of foundation.



- The effect of Capillarity in Sand bed is to increase effective Stress, that would give less Settlement than actual, hence this test should be performed at Water table level if it is within 1m below the foundation.
- This test is Short duration test, hence is suitable for Granular Soil in which immediate Settlement can be taken as total Settlement, but not for Cohesive Soil.

Note: If the load test is Carried out above Water table, the Settlement Computed from curve will have to be Corrected if there is likelihood of rise in water table in near future.

Actual Settlement = $\frac{\text{Settlement from plate load test}}{\text{Correction factor}}$

• PENETRATION TEST

- These test involves the measurement & resistance to penetration of Sampling Spoon, cone or other Shaped tools Under dynamic or Static loadings.
- The resistance is then empirically Correlated

with some of the engineering properties of Soil such as density index, Consistency, bearing Capacity etc.

- The two Commonly Used test are –
- (a) Standard Penetration test.
- (b) Cone penetration test.

(a) Standard Penetration test (IS 2131)

- This test is Used to determine,
 - Relative density/density index.
 - Angle of Shearing resistance.
 - Unconfined Compressive Strength.
 - Pile load Capacity.
 - Ultimate Bearing Capacity on the basis of Shear Centre.
 - Allowable bearing pressure on basis of Settlement Criteria.



Note: This test is Suitable for medium to dense Sand. In clays due to dynamic loading remoulding may Occur & excess pore water pressure may Set up whereas in loose saturated Sand liquifaction may Occur.



- This test is performed in a clean hole, 55 to 150 mm in diameter.
- A casing or drilling mud is used to support the side of the hole.
- A thick wall split spoon sampler, 50.8 mm outer diameter & 35 mm inner diameter is driven into the undisturbed soil at the bottom of hole under the blow of 63.5 kg drive weight with 75 cm of free fall. The minimum open length of the sample should be 60 cm.
- The split tube sampler commonly known as spoon sampler resting on bottom of the bore hole is allowed to sink under its own weight.
- It is then seated 15 cm with the blow of hammer falling through a height of 75 cm.
- Thereafter the spoon sampler is further driven by 30 cm.
- The no. of blows required to affect each 15 cm penetration is recorded.
- The first 15 cm of drive may be considered to be seating drive.
- The total no. of blows required for second & third 15 cm of penetration is termed as Penetration Resistance Number.

- If the split spoon sampler is driven less than 45 cm (total), then the penetration resistance of last 30 cm is considered.
- The entire sample may sometimes sink under its own weight when very soft sub-soil stratum is considered/encountered, under such conditions it may not be necessary to give any blow to the sample & SPT value would be indicated to zero.
- SPT value is not considered in fraction.
- The test is repeated at every 2 m to 5 m interval or at the change of stratum.
- The observed SPT number may be required to be corrected for the following -
 - (i) Overburden pressure correction.
 - (ii) Water table/dilatancy/fine's correction.

(i) Overburden pressure correction

- In granular soil, overburden pressure affects the penetration resistance.
- If two soils, having same relative density, but different confining pressure are tested, the one with a higher confining pressure gives a higher penetration number as the confining



pressure in Cohesionless Soil increases with depth.

- The Penetration number for Soil at Shallow depth is Underestimated & that at a greater depth is Overestimated.
- For Uniformity, the Numbers obtained from field test Under different effective overburden pressure are Corrected to a Standard effective Overburden pressure.

$$N_1 = N_0 \left\{ \frac{350}{\sigma'_0 + 70} \right\} \quad \text{or} \quad \left\{ \frac{500}{1.42\sigma'_0 + 100} \right\}$$

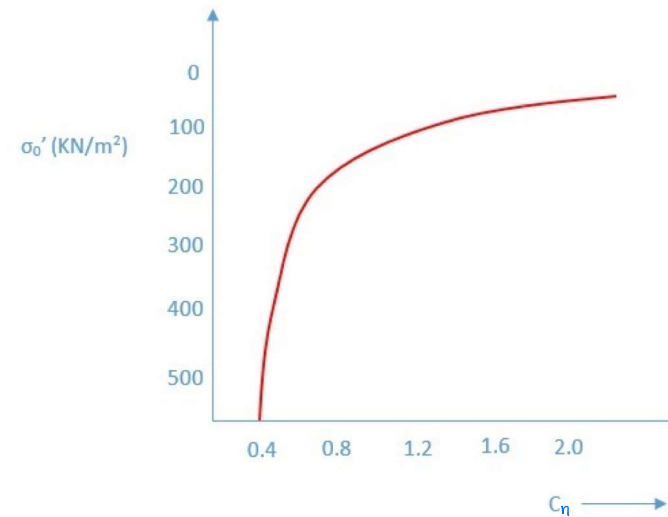
σ'_0 = Effective Overburden pressure (KN/m²).

The above Correction is applicable for,
 $\sigma'_0 \leq 280 \text{ KN/m}^2$ & for dry or moist Sand.

Note: The N Value for Cohesionless Soil corrected for Overburden pressure as per PECIC is given by –

$$N_1 = N_0 \cdot C_n$$

C_n = Normalising Correction factor.



- Ratio of N_1/N_0 should be between 0.45 – 2.
- If this ratio is more than 2, then N_1 should be divided by 2 to obtain the design value used in finding the bearing Capacity of Soil.
- Overburden pressure correction is applied first & then dilatency Correction is applied.

(ii) Dilatency Correction

- Silty fine Sand & fine Sands below the water table develops pore pressure which is not easily dissipated.



- Pore pressure increases the resistance of Soil, then penetration number (N) also increases.
- This Correction is applied When the Observed Value of Number after first Correction exceeds 15.

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

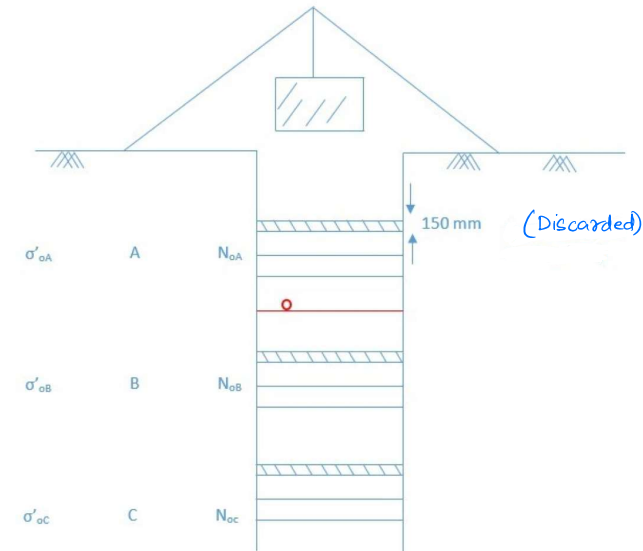


- This Correction is required only When weight is at or above the test level, i.e. if weight is below the test level, this Correction is not required.
- Final Corrected Value of penetration number is average of Corrected value of Penetration Number at different levels.

$$N_{final} = \frac{N_A + N_B + N_C + \dots + N_n}{n}$$

- If it is Conducted at different levels say A, B, C, D... N, the above analysis is done.
- At last Check is applied that all the Corrected SPT Number Should be in the limits as follows-
 $N_{final} \pm 0.5N_{final}$

- If any of the Value $N_A, N_B, \dots, N_n$ has more than 50% Variation on either Side of final N_{avg} , that Value is discarded & average is found from remaining values of SPT Number.



Q. During the Subsurface investigation for design of foundation a SPT is conducted at 4.5 m below ground Surface. The record of number of blows is given below :

Penetration depth (cm)	No. of blows
0 - 7.5	3
7.5 - 15	3
15 - 22.5	6
22.5 - 30	6

} discarded
 } 12

30-37.5
37.5- 45

8 } 15
7 }



Assuming the water table at ground level, Soil is fine Sand and Correction factor for Overburden is 1, the Corrected N Value for Soil is,

Sol: $N_0 = 27$, $N_1 = N_0 C_n \Rightarrow N_1 = 27 \times 1 = 27$.

Considering Water table Correction,

$$N_2 = 15 + \frac{1}{2} \{27 - 15\} \Rightarrow N_2 = 21.$$

• Ultimate Bearing Capacity as per Shear Criteria

→ The final Corrected average SPT number is related to the friction angle & relative density, Which is further expressed either graphically or in tabular form.

→ Using friction angle, bearing Capacity factor (N_c , N_q , N_γ) can be determined hence Ultimate Capacity can be found using any of Analytical theory.

→ Relationship between SPT Value and Relative Density is as follows –

SPT(N)-Value

Relative Density

0-4

Very loose

4-10

loose

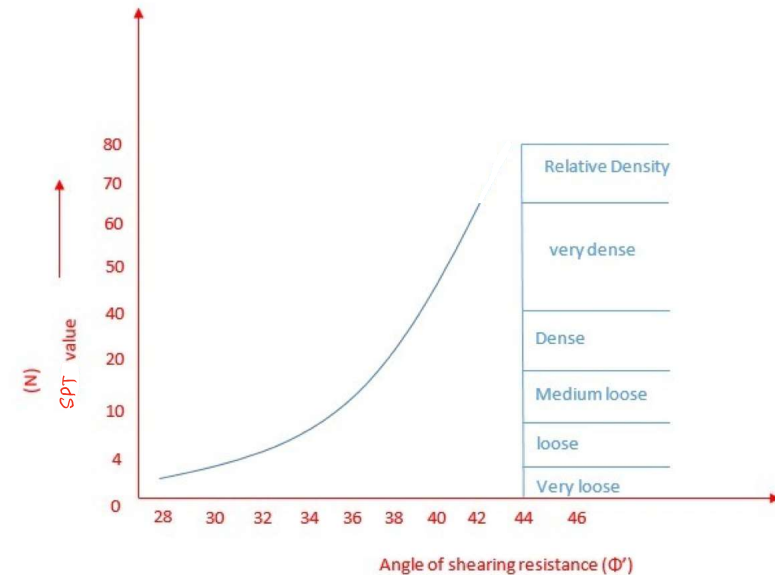
10-30

Medium



30-50
>50

Dense
Very Dense.



• Determination of Allowable bearing pressure/ Safe Bearing pressure as per Settlement Criterion
Empirical relationship Can be Used, to find allowable bearing pressure on the basis of SPT Number.

(i) Peck Hanson Equation

→ The net allowable bearing pressure or net Safe Settlement pressure is given by –



$$q_{\text{net}} = 0.41 \cdot C_w \cdot S \cdot N \quad (\text{KN/m}^2)$$

Where,

S = Permissible Settlement of foundation in mm as given by IS Code.

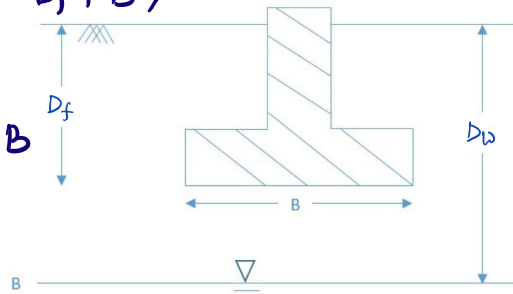
N = Final Corrected average SPT Number.

C_w = Water table Correction factor.

$$C_w = \frac{1}{2} \left\{ 1 + \frac{D_w}{D_f + B} \right\}$$

$$0.5 \leq C_w \leq 1$$

$$0 \leq D_w \leq D_f + B$$



Note: Here depth of Water table is measured from Ground Surface.

(ii) Teng's Equation

→ The net allowable bearing pressure is given by —

$$q_{\text{net}} = 1.4 \left\{ \frac{B + 0.3}{2B} \right\}^2 \cdot S \cdot (N - 3) \cdot C_w \cdot C_d \quad \text{KN/m}^2$$

Where,

S = Permissible Settlement (mm).

B = Width of the footing.

N = Final Corrected average SPT Number

C_d = Depth Correction factor.

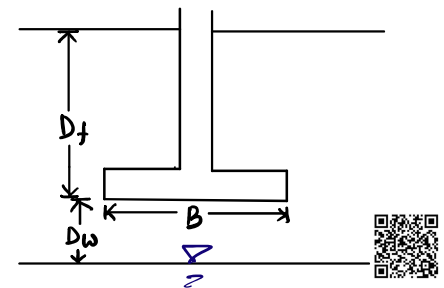
$$C_d = \left\{ 1 + \frac{D_f}{B} \right\} \leq 2$$

C_w = Water table Correction factor.

$$C_w = \frac{1}{2} \left(1 + \frac{D_w}{B} \right)$$

$$0.5 \leq C_w \leq 1$$

$$0 \leq D_w \leq B$$



Note: Here, Depth of Water table is measured from foundation level.

→ Teng's equation also includes the effect of depth in Combination of Safe Bearing pressure.

• IS Code Method

→ IS Code recommends use of Teng's equation with some modification to find Safe Bearing pressure for footing.

$$q_{\text{net}} = 1.38 \left\{ \frac{B + 0.3}{2B} \right\}^2 \cdot S \cdot (N - 3) \cdot C_w \quad (\text{KN/m}^2)$$

→ IS Code recommends use of Peck, Hanson equation to find Safe Bearing pressure for raft.

$$q_{\text{net}} = 0.88 C_u N_s \quad (\text{KN/m}^2)$$



→ For Permissible Settlement of 25 mm.

$$q_{\text{net}} = 22 C_u N \quad (\text{KN/m}^2)$$

→ If Water is much below the foundation level.

$$q_{\text{net}} = 22 N \quad (\text{KN/m}^2)$$

Note: A raft or mat foundation covers the entire plan area of the building.

→ It is Used when individual Spread footing Cannot be Used because of heavier load or due to poor Soil Condition or Both.

→ Because of rigidity a raft tends to bridge over local soft pockets or any other heterogeneity of Soil & irregularities tends to get even distributed below a raft.

→ Due to these reasons, a raft foundation suffers much less differential Settlement than Isolated footing.

→ Hence, the Safe Bearing pressure for a raft should be much higher than isolated footing (for same limit of differential Settlement) or total possible Settlement for raft is almost two times that of an isolated footing but still differential Settlement would be same.

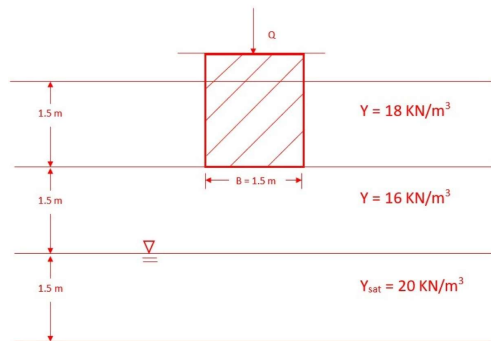
→ The allowable bearing pressure of raft is always governed by Settlement Criteria as due to its large size raft assumes a very high Safe Bearing Capacity.



• Unconfined Compressive Strength of Cohesive Soil

Consistency	$q_u (\text{KN/m}^2)$					
	very Soft	Soft	Medium	Stiff	Very Stiff	Solid
SPT(N) Value	<2	2-4	4-8	8-15	15-30	>30
q_u	<25	25-50	50-100	100-200	200-400	>400

Q. Using Peck-Hanson equation, find net Safe Bearing pressure below the foundation for the given Situation. Test is Conducted at 1.5 m interval and the observed value of SPT number are 10, 15, 20 at depth of 1.5 m, 3 m & 4.5 m respectively $\{\gamma_s = 10 \text{ KN/m}^3\}$.



Sol: $q_{anet} = 0.41 C_w N_s$

$$C_w = \frac{1}{2} \left\{ 1 + \frac{D_w}{D_f + B} \right\} = \frac{1}{2} \left\{ 1 + \frac{3}{1.5 + 1.5} \right\} = 1$$

$s = 40 \text{ mm}$

$N_{oA} = 10$

$$N_{oA} = N_oA \left\{ \frac{350}{\sigma'_A + 70} \right\} \quad \sigma'_A = \gamma H_A = 18 \times 1.5 = 27 \text{ kN/m}^2$$

$$N_1A = 10 \left\{ \frac{350}{27 + 70} \right\} = 36.08$$

$N_2A = N_1A = 36.08$

$$\frac{N_1A}{N_{oA}} = \frac{36.08}{10} = 3.608 > 2$$

$$N_1A = \frac{36.08}{2} = 18.04 \checkmark$$

$$\Rightarrow N_{oB} = 15 \Rightarrow \sigma'_B = \gamma_1 H_1 + \gamma_2 H_2 = 18 \times 1.5 + 16 \times 1.5 = 51 \text{ kN/m}^2$$

$$N_1B = N_{oB} \left\{ \frac{350}{\sigma'_B + 70} \right\} = 43.38$$



$$\frac{N_1B}{N_{oB}} > 2 \Rightarrow N_1B = \frac{43.38}{2} = 21.69 > 15$$

$$N_2B = 15 + \frac{1}{2}(N_1B - 15) = 18.345$$

$$\Rightarrow N_{oC} = 20, \sigma'_C = \gamma_1 H_A + \gamma_2 H_B + \gamma_3 H_C = 66 \text{ kN/m}^2$$

$$N_1C = N_{oC} \left\{ \frac{350}{\sigma'_C + 70} \right\} = 51.47$$

$$\frac{N_1C}{N_{oC}} > 2 \Rightarrow N_1C = \frac{51.47}{2} = 25.75 > 15$$

$$N_2C = 15 + \frac{1}{2}(N_1C - 15) = 20.365$$

$$N_{final} = \frac{N_A + N_B + N_C}{3} = \frac{18.04 + 18.345 + 20.365}{3} = 18.91 \approx 19$$

Check,

$$N_{final} \pm 0.5 N_{final} = 19 \pm 0.5 \times 19 = 28.5 \text{ \& 9.5}$$

9.5 to 28.5

$$\therefore q_{anet} = 0.41 \times 1 \times 19 \times 40 = 311.61 \text{ kN/m}^2$$

(iii) Static Cone Penetration test (CPT)

- The Static Cone penetration test, simply termed as CPT is a simple test that is used in place of SPT particularly for soft clays & fine to medium sand deposits.
- This test is also termed as 'DUTCH CONE TEST'.
- The test assembly consists of a cone with an apex angle of 60° & base area of 10 cm^2 .
- The sequence of operations of penetrometer are as follows –
 - (I) The cone of the friction jacket is in stationary position.
 - (II) The cone is pushed into the soil by the inner rod to a depth of 'a' at a steady rate of 20 mm/sec .
- The tip resistance q_c called cone or point resistance is computed as $(q_c = Q/A)$ where Q is force read on pressure gauge.
- (III) The sounding rod is pushed further to a depth 'b'. This has effect of pushing both friction jacket & cone assembly together. The total force (Q_t) required for this is again noted.
- The force required to push friction jacket (Q_f) is obtained as $Q_f = Q_t - Q$.

→ The side or skin friction $f_s = Q_f/A_f$, where A_f is surface area of friction jacket. 

Note: a & b are normally taken to be 40 mm however their minimum value is 35 mm .

(IV) The outside mantle tube is pushed down to distance (a+b) bringing cone & friction jacket to original position.

- The above procedure is continued till the proposed depth of sounding is reached.
- CPT gives a continuous record of variation of both cone resistance & friction resistance of soil, however it does not give yield value of any sample.

Note: It is unsuitable in gravel & very dense sand owing to difficulty in pushing the cone.

- Data from CPT is used to calculate the point bearing resistance & skin friction resistance of pile foundation.
- In granular soil relationship between q_c & N is further established.

Type of Soil	$q_c/N, q_c (\text{Kg/cm}^2)$
Sandy gravel or gravel	8-10
Coarse Sand	5-10
Fine to medium Sand	3-4
Silty Sand.	2



→ Further q_c & N for fine Silty medium loose to medium dense Sand is given by –

$$q_c = 4N$$

→ q_c can also be related with Undrained Shear Strength of Clay (C_u).

$$q_c = N_k C_u + \sigma_0$$

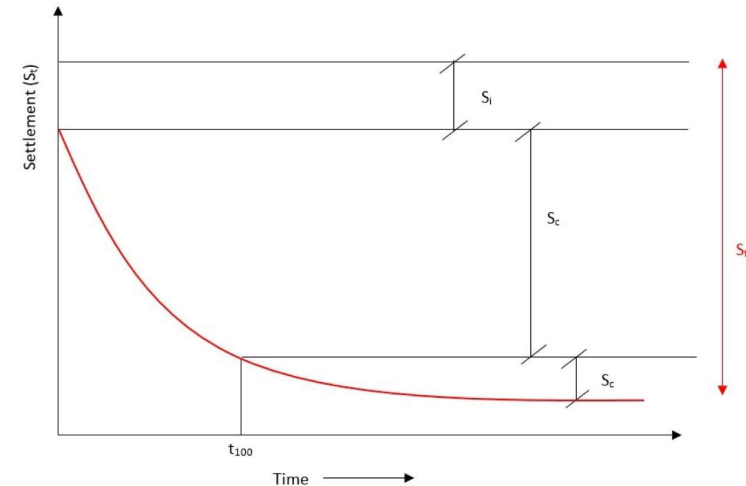
Where, N_k = Cone factor

σ_0 = total Overburden pressure.

• Settlement of Shallow Foundation

→ Total Settlement (S_t) in general case for any type of Soil is given by –

$$S_t = S_i + S_c + S_s \quad *$$



→ Immediate or elastic Settlement ' S_i ' which takes place immediately or Over a short time (approx about 7 days) after the load is placed is termed as 'Initial Compression'.

→ In Clay, it is also known as 'Distortion Settlement' and is due to change in the Shape of Soil Skeleton with any change in water content.

→ In Saturated Clays immediate Settlement is Computed Using elastic theory.

→ It is Sometimes Considered small compared to the longer term Consolidation Settlement hence neglected.

→ In granular Soil immediate Settlement accounts

- approves the entire Settlement.
- In inorganic clays primary Consolidation accounts for most of Settlement.
- In Organic Soil, Secondary Consolidation assumes Greater Significance.
- The theory of elasticity can be Used to determine Settlement Caused by load acting over flexible or rigid area of different geometrical Shape and is given by -

$$S_i = q \cdot B \left(1 - \frac{\mu^2}{E}\right) \cdot I_f *$$



Where, S_i = Vertical Settlement
 B = Width of foundation
 μ = Poisson's ratio.
 E = Young's Modulus of elasticity.
 I_f = Influence factor for Settlement
 Which depends upon Shape & rigidity of foundation.

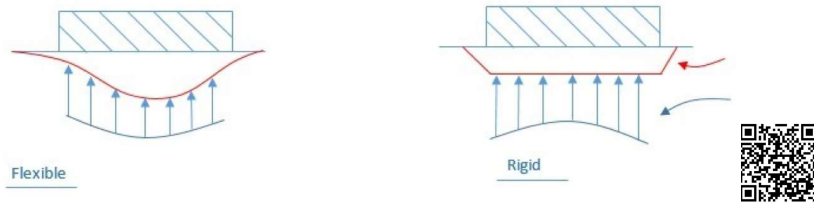
Shape	(If)			If
	Flexible Foundation			Rigid Foundation
	Centre	Corner	Average	
Circle	1	0.64	0.85	0.86
Square	1.2	0.56	0.95	0.82
Rectangle $L/B = 1.5$	1.36	0.68	1.2	1.06



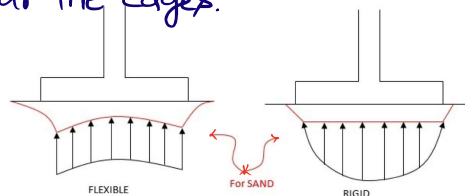


- Settlement of rigid foundation Such as beam & Slab raft is approx equal to mean Settlement for Corresponding flexible foundation.
- In a flexible footing, the Contact pressure i.e. pressure at the interface between the footing & Soil is Uniformly distributed.
- A Uniform pressure produces a dish Shaped Pattern of displacement in Clay Soil.
- For a rigid footing the Settlement is more or less Uniform Over the area of Contact.
- Since a Uniform Contact pressure produces dish Shaped Settlement pattern the Contact pressure

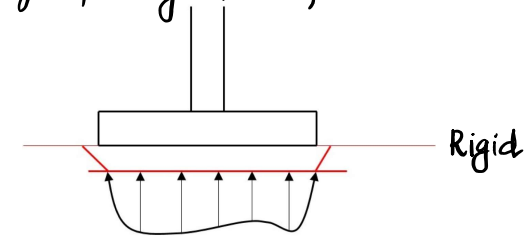
must be more near the edges of loaded area & less near the centre in order to produce Uniform Settlement.



- Because of Variation in modulus of elasticity of granular Soil, the immediate Settlement cannot be accurately found in Such Soils.
- In these Soils, E increases with Confining pressure and thus it increases with depth.
- Further E Varies across the width of the loaded area, it is more near Centre, than near edges.
- As a Consequence the distribution of pressure will be Uniform below flexible footing, but deformation will be inverted dish pattern.
- For a rigid footing where Settlement is Uniform, the Contact pressure is more near the Centre than near the edges.



Note: For rigid footing in Silt,

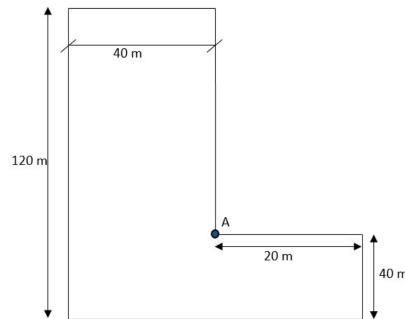


Note: The elastic Constants of the Soil are determined in the Lab Under Condition of Stress range & mode of deformation which are identical with field observation.

- E & n for different types of Soil can be related with SPT Value and Cone resistance value empirically.
- According to IS 8009, the ratio of total Settlement of a rigid foundation to the total Settlement at the Centre of flexible foundation is Called 'Rigidity Factor'.
- A rigidity factor of 0.8 is recommended by Code for Computing the Settlement of rigid footing.

Q. The plan of a proposed heap is Shown. The heap will Stand on a thick deposit of Soft Clay with

$E = 15 \text{ MN/m}^2$. The Uniform pressure on Soil mass is assumed as 150 kN/m^2 . The immediate Settlement Under the point A at Surface of Soil is _____ cm. Take $N = 0.5$.



Value of Influence factor

Shape of loaded area Rectangle (L/B)	Flexible footing at corner (I_f)
1	0.56
1.5	0.68
2.0	0.77
5	1.05
10	1.26
100	1.69

Sol:

$$S_{iA} = q \left\{ \frac{1-N^2}{E} \right\} \sum_{i=1}^3 B_i \cdot I_{fi}$$

$$= 150 \left\{ \frac{1-0.5^2}{15 \times 10^3} \right\} [40 \times 0.77 + 40 \times 0.56 + 20 \times 0.77]$$

$$= 67 \text{ cm.}$$

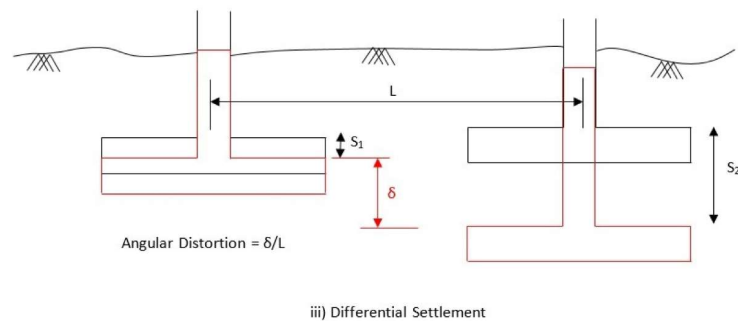
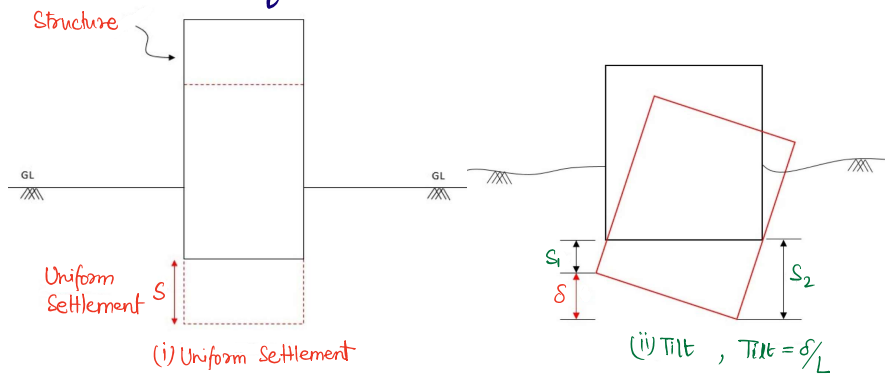
• ALLOWABLE SETTLEMENT

- Settlement can cause Cracks in masonry walls and interior plaster walls of building.
- It can cause a Structure to tilt, which may become noticable in high building or can avoid the function of Structure to fulfilled in in many ways.
- Settlement can be of different patterns Under different conditions & the effect Caused on the Structure will depend on the type of Settlement.
- If the Structure Settles Uniformly it is not likely to Suffer damage.
- A Structure with a very rigid raft or mat foundation will experience Uniform Settlement.
- A Structure is said to Undergo differential Settlement, if One of its part Settles more than the others.
- The difference in total Settlement between any two points is Called 'DIFFERENTIAL SETTLEMENT'.
- Angular distobation is the ratio of differential

Settlement between two Columns ' δ ' to the Spacing between them ' l '.

$$\therefore \text{Angular distortion} = \frac{\delta}{l}$$

→ The differential Settlement may also lead to the Uniform tilt, When the entire Structure rotates as its Consequences.



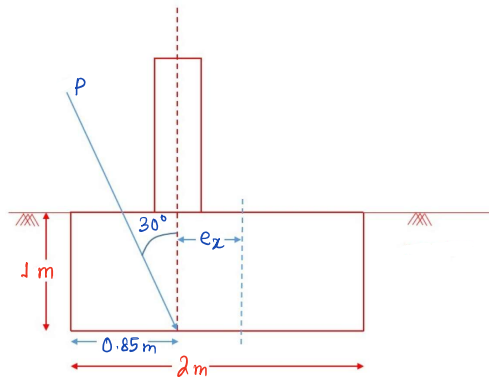
- It is relatively easy to estimate the total Settlement & is very difficult to estimate differential Settlement.
- In granular soils, the minimum differential Settlement can in some cases, be close to maximum total Settlement whereas in clay the differential Settlement is much less than total Settlement.
- Maximum differential Settlement, generally does not exceed 75% of the maximum total Settlement in granular soil, while in clay it nearly exceeds 50% of maximum total Settlement.
- Higher total Settlement is permissible in Clay than in Sand.
- It is due to difference in rate of Settlement in Sand & Clay.
- In Sand Settlement occurs almost immediately on placement of load while in Clay it takes time thus there is time for Structure resting on Clay to adjust to differential Settlements.
- In Sand, the differential Settlement occurs as soon as total Settlement, thus leaving the Structure no time for gradual adjustment.

→ In any Settlement, analysis of all temporary & transient load have to be considered in determining the Settlement in Sand, Whereas in Clay Permanent load can Cause Consolidation Settlement is to be Considered.

Q. A Square footing (2m X 2m) is Subjected to an inclined point Load 'P' as Shown. The Water table is located well below the base of footing Consider Only One-way eccentricity and Compute the net Safe load Capacity of the foundation for $FOS = 3$.

Assume: Capacity Factors

	q	γ
Bearing	33.3	37.16
Shape	1.314	1.314
Depth	1.143	1.113
Inclination	0.444	0.02



$$\text{Sol: } q_u = C N_c S_c i_c d_c + q N_q S_q d_q i_q + 0.5 B' \gamma N_{\gamma} d_{\gamma} S_{\gamma} i_{\gamma}$$

$$q_u = 0 + (1 \times 18) \times 35 \times 1.314 \times 1.113 \times 0.444 + 0.5 \times (2.2 \times 0.15) \times 18 \times 37.16 \times 1.314 \times 1.113 \times 0.02$$

$$q_u = 405.84 \text{ kN/m}^2$$

$$q_{nu} = q_u - \gamma D_f = 405.84 - 1 \times 18 = 387.84 \text{ kN/m}^2$$

$$q_{nf} = \frac{387.84}{3} = 129.3 \text{ kN/m}^2$$

$$Q_{ns} = q_{nf} \times A = 129.3 \times (2 \times 2) = 517.2 \text{ kN}$$

ESE

Q. A 30 cm Square Bearing plate Settles by 8 mm in the plate load test on Cohesionless Soil when the intensity of loading is 180 kN/m^2 . The Settlement of a Shallow foundation of 1.5m Square Under the Same intensity of loading will be nearly

- (a) 30 mm.
- (b) 26 mm.
- (c) 22 mm.
- (d) 18 mm.

IES-2020

ESENOTES.COM

Ans: (c) 22 mm.

Sol: $B_p = 30 \text{ cm} = 0.3 \text{ m}$.

$S_p = 8 \text{ mm}$.

$B_f = 1.5 \text{ m}$

Settlement of foundation

$$\frac{S_f}{S_p} = \left\{ \frac{B_f (B_p + 0.3)}{B_p (B_f + 0.3)} \right\}^2$$

$$\frac{S_f}{S_p} = \left\{ \frac{1.5 \times (0.3 + 0.3)}{0.3 \times (1.5 + 0.3)} \right\}^2$$

$$S_f = 22.22 \text{ mm}.$$

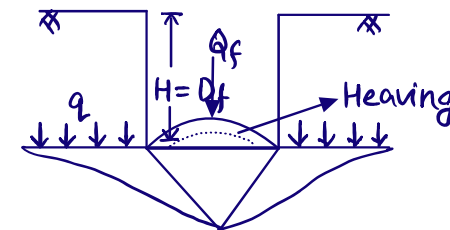
Q. A Raft foundation 10 m wide and 12 m long is to be Constructed in a Clayey Soil having Shear Strength of 12 kN/m^2 . Unit weight of Soil is 16 kN/m^3 , the ground Surface Carries a Surcharge of 20 kN/m^2 ; the factor of Safety is 1.2 and the value of $N_c = 5.7$. The Safe depth of foundation will be nearly

- (a) 8.2 m.
- (b) 7.3 m.
- (c) 6.4 m.
- (d) 5.5 m.

IES-2019

Ans: (d) 5.5 m.

Sol:



Given : $q = 20 \text{ kN/m}^2$, $\tau_u = c_u = 12 \text{ kN/m}^2$,
 $\gamma = 16 \text{ kN/m}^3$
 $B = 10 \text{ m}$, $L = 12 \text{ m}$

Bearing Capacity of Soil for rectangular raft footing in cohesive soil is given by.

$$Q_f = \left\{ 1 + 0.3 \frac{B}{L} \right\} C \cdot N_c + \gamma D_f + q$$

$$= \left\{ 1 + 0.3 \times \frac{10}{12} \right\} 12 \times 5.7 + 16 D_f + 20$$

$$= 105.5 + 16 D_f$$

Base failure will occur when $Q_f = 0$.

$$\text{Hence, } D_f = -\frac{105.5}{16} = -6.59$$

(minus sign indicates that it is excavation).

$$\text{Safe depth} = \frac{D}{FOS} = \frac{6.59}{1.2} = 5.49 \text{ m.}$$

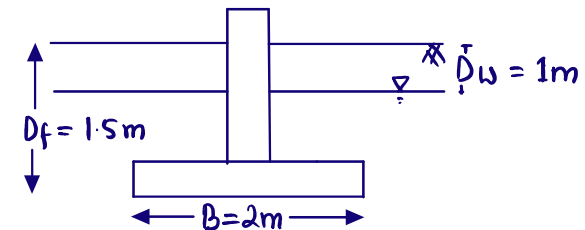
Q. A Strip footing 2m in width, with its base at a depth of 1.5m below Ground Surface, rests on a Saturated Clay Soil with $\gamma_{sat} = 20 \text{ kN/m}^3$, $C_u = 40 \text{ kN/m}^2$, $\phi_u = 0$; $c' = 10 \text{ kN/m}^2$; and $\phi'_u = 20^\circ$. The natural Water table is at 1m depth below Ground level. As per IS:6403-1981, the Ultimate Bearing Capacity of this footing will be (taking the relevant N_c as 5.14).

- (a) 327 kN/m^2
- (b) 285 kN/m^2
- (c) 253 kN/m^2
- (d) 231 kN/m^2

IES-2019

Ans: (d) 231 kN/m^2

Sol:



As per IS6403: 1981

$$q_u = S_c \cdot d_c \cdot i_c C N_c + S_q \cdot d_q \cdot i_q \gamma N_q + \frac{1}{2} B \gamma N_\gamma (i_\gamma S_\gamma d_\gamma) \omega$$

For footing we consider end of Construction Stability.

$$\therefore C = C_u = 40 \text{ kN/m}^2, \phi = \phi_u = 0^\circ.$$

$$\frac{D_f}{B} < 1, d_c = d_q = 1$$

For Strip footing,
 $S_c = S_q = 1$

For $\phi = 0, N_c = 5.14, N_q = 1, N_\gamma = 0$
 $\therefore q_u = CN_c + (\gamma_t \times 1 + \gamma_{sub} \times 0.5) \times N_q$

$$q_u = 40 \times 5.14 + (20 + (20 - 9.81) \times 0.5) \times 1$$

$$= 230.695 \text{ kN/m}^2 \approx 231 \text{ kN/m}^2$$

(Assuming $\gamma_t = \gamma_{sat}$ for Clay).

GATE

Q. The Percent Reduction in the Bearing Capacity of a Strip footing resting on Sand under flooding Condition (water level at the Base of the footing) When Compared to the Situation where the water level is at a depth much greater than the width of footing, is approximately

- (a) 0
- (b) 25
- (c) 50
- (d) 100

GATE-2018, Shift-I

Ans: (c) 50

Sol: Terzaghi's Bearing Capacity equation

$$q_u = CN_c + qN_q + \frac{1}{2} B \cdot \gamma \cdot N_\gamma$$

For Cohesionless Soil, $C = 0$, footing resting on Sand Surface so, $D_f = 0$

When water level is at Base of footing.

$$q_{u1} = \frac{1}{2} B \cdot \gamma_{sub} \cdot N_\gamma$$

When water level is at much greater depth

$$q_{u2} = \frac{1}{2} B \gamma_t \cdot N_\gamma$$

Percent reduction in Bearing Capacity,

$$= \frac{q_{u2} - q_{u1}}{q_{u1}} \times 100$$

$$= \frac{\gamma_t - \gamma_{sub}}{\gamma_t} \times 100$$

As, $\gamma_{sub} = 0.5 \cdot \gamma_t$.

So, percentage reduction = 50%.

Q. The Plate load test was Conducted on a Clayey Strata by Using a plate of 0.3 m X 0.3 m dimensions, and the Ultimate load per Unit area for the plate was found to be 180 KPa. The Ultimate Bearing Capacity (in KPa) of a 2 m wide Square footing would be

- (a) 27.
- (b) 180.
- (c) 1200.
- (d) 2000

GATE-2017, Set-II

Ans: (b) 180 KPa.

Sol :

Bearing Capacity of footing on purely Cohesive Soil does not depend upon Size of footing.

$$q_{up} = q_{uf} \text{ (for Clay)}$$

∴ Ultimate Bearing Capacity of a 2 m wide Square footing = 180 KPa.

Q. A Circular raft foundation of 20 m diameter and 1.6 m thick is provided for a tank that applies a bearing pressure of 110 KPa on Sandy Soil with Young's Modulus $E_s = 30 \text{ MPa}$ and $\mu = 0.3$. The raft is made of Concrete $E_c = 30 \text{ GPa}$ and $\mu = 0.15$. Considering the raft as rigid, the elastic Settlement is -

- (a) 50.96
- (b) 53.36
- (c) 63.72
- (d) 66.71

GATE-2014, Set-II

Ans: (b) 53.36.

Sol : Elastic Settlement of rigid footing

$$= 0.8 \times \left[\frac{q B (1 - \mu^2)}{E_s} \right]$$

$$= 0.8 \times \left[\frac{110 \times 20 \times (1 - 0.09)}{30 \times \frac{10^{-3}}{10^{-6}}} \right]$$

$$= 66.733 \times 0.8$$

$$= 53.36 \text{ mm.}$$

SSC-JE

Q. Match the type of test (List-I) with the Objective of the test (List-II)

List-I

- A. Standard Penetration test.
- B. Hydrometer test.
- C. Proctor test.
- D. Vane test.

List-II

- 1. Grain Size Analysis
- 2. Shear Strength.
- 3. Bearing Capacity.
- 4. Compaction.

(a) A-3, B-1, C-4, D-2

(b) A-3, B-1, C-2, D-4

(c) A-4, B-2, C-3, D-1.

(d) A-2, B-1, C-4, D-3.

2018-Morning Session.

Ans : (a) A-3, B-1, C-4, D-2.

Q. A 300 mm Square bearing plate settles by 15 mm in plate load test on a cohesive soil when the intensity of loading is 0.2 N/mm^2 . The settlement of a prototype shallow footing '1m' square under the same intensity of loading is:

- (a) 15 mm.
- (b) 30 mm.
- (c) 50 mm.
- (d) 167 mm.

2013.

Ans: (c) 50 mm.

Sol: Square Bearing plate : $B_p = 300 \text{ mm} = 0.3 \text{ m}$.
 Size of the Square footing, $B_f = 1 \text{ m}$.
 Settlement in Bearing plate, $S_p = 15 \text{ mm}$.

For Clayey Soil,

$$\frac{S_f}{S_p} = \frac{B_f}{B_p}$$

$$S_f = \frac{1}{0.3} \times 15$$

$$S_f = 50 \text{ mm.}$$

Q. Which of the following has least Bearing Capacity?

- a. loose Gravel.
- b. Hard Rocks.
- c. Soft rocks.
- d. Compact gravel.

2014-Morning Session

Ans: (a) loose Gravel.

Sol: Hard rocks are usually refers to igneous or Metamorphic rocks, whereas soft rocks are usually refers to sedimentary rocks.

Bearing Capacity of these are as follows:
 Igneous rocks > metamorphic rocks >
 Sedimentary rocks > Compact gravel > loose gravel.

DEEP FOUNDATION

- In Situation where Soil is poor at shallow depth, in order to transmit the load safely, the depth of foundation has to be increased till a suitable soil stratum is met.
- In View of increased depth, Such foundations is called DEEP FOUNDATION.
- Piles, Pier, well are examples of Deep Foundation.

• Pile Foundation

- A pile is a relatively small diameter shaft which is driven or installed into the ground by suitable means.
- The piles are usually driven in groups to provide foundation for structure.
- The Pile group may be subjected to vertical loads, horizontal loads or a combination of both.
- Piles have been classified on the basis of the following –

(i) Material Used for Construction

- Timber pile.
- Steel pile.
- Concrete Pile.



- Sand pile.



(ii) Cross Section

- Circular Pile.
- Square pile.
- Hexagonal pile.
- I-Section pile.
- H-Section pile.

(iii) Shape

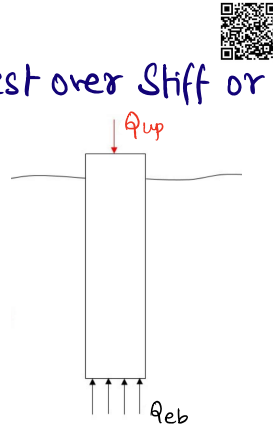
- Cylindrical pile.
- tapered pile.
- Under-reamed pile.

- These piles are considered useful in expansive soil where the use of shallow spread footings is not feasible due to excessive shrinkage or swelling behaviour of such soils.
- A further development of these piles is multi-Under reamed piles, in which when the number of bulbs is increased from one to two, the carrying capacity of pile increases by about 50%.

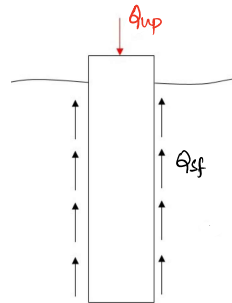
(iv) Mode of load transfer

→ End Bearing pile: These piles rest over stiff or hard strata & load carrying capacity is due to end bearing action or point resistance.

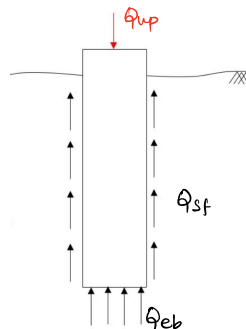
→ The length of these piles depends on position of hard strata.



→ Friction/hanging piles: Such piles are driven in soft clay or loose sand extending to greater depth. The load carrying capacity of these piles is due to skin friction action. The length of friction piles may be 10-20m.



→ Bearing & Friction Piles: If piles are driven in medium to dense/stiff soil, then load carrying capacity is due to combined action of end bearing action & skin friction action.

(v) Method of forming

- Pre Cast pile.
- Pre Stressed pile.
- Cast in Situ

(vi) On the Basis of Functions(a) Laterally loaded piles

→ Piles are also used to resist horizontal load, as in case of foundation for retaining wall, bridge abutments etc.

→ As the horizontal load acts perpendicular to the pile axis, these piles are known as laterally loaded piles.

(b) Batter Piles

→ In case of large lateral loads piles are driven at an angle and are termed as Batter Piles, which serve better than vertical piles.

(c) Compaction Piles

→ Short piles are sometimes used for compacting loose sand deposits, which get densified by vibration set up on driving are termed as compaction piles.

(d) Tension Piles

- Piles are Sometimes Used to resist Uplift Loads and are thus in tension, hence known as Tension/Uplift piles.
- They are Suitable to be provided in Swelling Soils. For eg: Black Cotton Soil.

(e) Anchor Piles

- Piles can also be Used to provide anchorage against horizontal pull as in case of anchored bulk heads termed as Anchor Piles.

(f) Sheet Pile

- It is Used to retain earthfills and provided below hydraulic Structures.

(g) Fender Pile

- These are Used to anchor the Structure against tidal waves or floating objects in water.

(vii) On the basis of method of installation

- (a) Driven Piles.
- (b) Bored Piles
- (c) Vibrated Piles.
- (d) Jetted Piles.

(a) Driven/Displacement Piles

- These Piles are driven through hammer action, Such piles are essentially pre Cast, made of metal/wood.
- In driven piles, end bearing resistance & skin friction resistance both are efficiently developed.

(b) Bored Piles

- Bored Piles may be precast or Cast in situ. These Piles are less efficient than driven piles.

Note: The best way to Classify the pile is on the basis of the effect of installation of Pile on the Soil.

- Based on this Criterion, piles can be Considered to fall into two Classes –

- (a) Displacement Piles
- (b) Non- Displacement Piles.

- If during the installation of the pile, a large Volume of Soil is displaced laterally & Upwards Such a pile is termed as 'DISPLACEMENT PILE'.
- In loose Sand, Such Pile densifies the Sand upto

a distance of about 3.5 times the diameter of pile from centre.

- Compaction of Sand leads to increase in its angle of Shearing resistance within the Zone of influence, impact of which can be Computed as follows -

$$\phi_2 = \frac{\phi_1 + 40^\circ}{2}$$

Where, ϕ_1 = Initial Value of angle of Shearing resistance.

ϕ_2 = final Value of angle of Shearing after Compaction.

- If $\phi_1 > 40^\circ$, no Benefit will be derived from Piles driving.
- For $\phi_1 > 40^\circ$, piles driving shall in fact, have the effect of reducing the angle of Shearing resistance due to dilatancy effects.
- In Clays, large displacement pile remould the Soil to a distance of about twice the diameter of pile.
- During driving Very high pore water pressure are set up around the pile & Soil regains its initial Structure Only after a period of time,

When the excess pore water pressure dissipates.

- This is why displacement piles are preferred for Use in 'loose to medium dense Sand' & not in 'dense Sand or Clay'.
- Driven & Cast in situ or precast piles, made up of Concrete or timber are examples of displacement piles.
- A Void/Opening is formed in the Soil by boring or excavation & is then filled with Concrete.
- These piles offers a advantage of no ground heaving noise or vibration, their length can be easily varied at the Site & it is possible to install very long piles with large diameter.

• Selection of Pile type

- The Selection of the type of pile to be used depends upon Several factors.
 - (a) Type of Structure & load it Carries.
 - (b) Location of the site.
 - (c) Soil Condition & position of Water table.
 - (d) Required pile length & Structural Capability of pile.
 - (e) Economy.

- The Selection of material of the Pile would depend upon the magnitude of Structural load.
- For light load timber pile can be used & for heavy loads only Steel or RCC Piles are considered.

• Pile Load Capacity

- For satisfactory behaviour of Pile foundation, it must be safe in Shear failure & Settlement failure criterion (same as that of Shallow foundation).
- The load Capacity of the pile can be estimated by several methods as follows -
 - (a) Static/Analytical Pile load method.
 - (b) Dynamic/Pile Driving method.
 - (c) Pile Load test.
 - (d) Correlation with Penetration test data.

(a) Static Pile Load Method

- When a Compressive load is applied at the top of pile, it tends to move Vertical downward relative to the surrounding Soil.
- As a result, the applied load is distributed as friction load along a certain length of pile from the top.

- As the load at top is increased, the friction load distribution will extend more & more towards the tip of pile till a certain extent the entire length of pile is involved in generating friction resistance.
- This is termed as Ultimate Skin friction resistance.
- Load in excess of this, begins to be transferred to the Soil at the base of pile & is termed as Point resistance/base/end bearing resistance at the point where pile fails by Punching Shear failure.
- The maximum load which pile can support through combined resistance is Skin friction & end bearing is termed as 'ULTIMATE LOAD CAPACITY (Q_u)' of the Pile.

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

- If $Q_{eb} \gg Q_{sf}$ the pile is termed as end/point bearing pile & if $Q_{sf} \gg Q_{eb}$ the pile is termed as friction pile.
- The relative proportion of load carried by end bearing & skin friction depends on the Shear strength & elasticity of Soil.

Note: It is being observed that -

- (i) When the ultimate skin friction resistance is mobilised only a fraction of Ultimate point Load is mobilised.
- (ii) When the Ultimate point load resistance is mobilised the skin friction resistance is decreased to a lower value than its peak.

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

$$Q_{eb} = q_{eb} \cdot A_b$$

$$Q_{sf} = q_{sf} \cdot A_s$$



$$Q_u = q_{eb} \cdot A_b + q_{sf} \cdot A_s *$$

A_s = Surface area of pile upon which skin friction acts.

A_b = Area of Cross-Section of pile on which bearing resistance acts.

Note: For tapered pile ' A_b ' may be taken as the Cross sectional area at the lower one-third of the embedded length.

q_{eb} = Unit load/toe/end bearing resistance.

q_{sf} = average skin friction resistance.

Case(i): Cohesive Soil

- For the Pile in Cohesive Soil, point bearing is generally neglected for individual pile action, since it is negligible as compared to frictional resistance.
- The Unit Skin friction may be taken equal to the Shear Strength multiplied by a reduction factor α or m .

$$q_{sf} = \text{average skin friction along the length of Pile.}$$

$$= \alpha \cdot \bar{c} \text{ or } m \cdot \bar{c}.$$

$$q_{eb} = CN_c$$

$$\text{Here, } N_c = 9 \Rightarrow q_{eb} = 9c$$

$$Q_u = 2\bar{c} \cdot A_s + 9cA_b *$$

α or m = Adhesion Coefficient or reduction factor, Value of which depends upon type of Clay & its value can be determined by Pile load test.

Consistency	N-Value	Value of α	
		Bored Piles	Driven Piles
• Soft to Very Soft	< 4	0.7	1
• Medium	4-8	0.5	0.7
• Stiff	8-15	0.4	0.4
• Stiff to hard	> 15	0.3	0.3

$\bar{c}$ = Average Undrained Cohesion along the length of Pile.

c = Average Undrained Cohesion of Soil at tip of pile.

Note: In absence of any depth both $\bar{c}$ & c may be taken equal to ' $q_u/2$ '.

→ The allowable load for pile is given by $Q_a = \frac{Q_u}{F}$

→ $F = \text{FOS} (2.5-3)$

→ If different FOS Value are adopted for Skin friction resistance & end Bearing resistance, then,

$$Q_a = \frac{\alpha \cdot \bar{c} \cdot A_s}{F_1} + \frac{q \cdot c \cdot A_b}{F_2} *$$

Case (ii): For Cohesionless Soil

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

$$Q_{sb} = q_{sf} \cdot A_s$$

$$q_{sf} = \mu \times \text{Earth Pressure.}$$

$$\therefore \mu = \tan \delta$$

δ = Frictional Angle between Piles & Soil.

→ Earth Pressure = $K \cdot \bar{\sigma}$

K = earth Pressure Coefficient.

$$\bar{\sigma} = \left\{ \frac{0 + K \cdot \gamma L}{2} \right\} = \frac{K \cdot \gamma L}{2}$$

$$\rightarrow q_{sf} = \frac{1}{2} K \gamma H \cdot \tan \delta$$

$$\rightarrow Q_{eb} = q_{eb} \cdot A_b$$

$$q_{eb} = q_u = c \cdot \gamma_c + q N_q + 0.5 B \gamma N_\gamma$$

$$\Rightarrow q_{eb} = q N_q + 0.5 B \gamma N_\gamma$$

$$\text{Since, } q N_q \gg 0.5 B \gamma N_\gamma \Rightarrow q_{eb} = q N_q = \gamma N_q L$$

$$Q_u = \frac{1}{2} K \gamma L \tan \delta A_s + \gamma N_q L A_b *$$

Area

Circular

Square

A_s

$\pi D L$

$4 B L$

A_b

$\pi D^2/4$

B^2

Pile Material	δ	Value of k	
		Loose Sand	Dense Sand
Steel	20	0.5	1
Concrete	0.75ϕ	1	2
Timber	0.37ϕ	1.5	4

(b) Dynamic Load

- When a Pile hammer hits Pile, the total driving energy is equal to the weight of hammer times the height of drop or Stroke.
- In addition to this, in case of double acting hammer, some energy is consumed by work done in penetrating the Pile & by certain losses, moreover in this case energy is also imparted by Steam Pressure.
- Following methods are available on this Approach -

(I) Engineering News Formula

- As per this method, Allowable load on pile is given by -

$$Q_a = \frac{WH}{F(S+C)} *$$

Where, Q_a = Allowable load.

H = Height of fall (cm).

F = Factor of Safety = 6

S = Final Set (Penetration) per blow, Usually taken as an average Penetration (cm) per blow for last 5 blows of a drop hammer & 20 Blows of Steam Hammer.

C = Empirical Formula.

= 2.5 cm for Drop Hammer.

= 0.25 cm for Single & Double Acting Hammer.

→ Hence,

(i) For Drop Hammer: $Q_a = \frac{WH}{6(S+2.5)}$

(Drop hammer is lifted normally & is allowed to fall freely).

(ii) For Single Acting Steam Hammer: $Q_a = \frac{WH}{6(S+0.75)}$

(Single acting Steam Hammer is lifted by Steam Pressure & is allowed to fall freely).

(iii) For Double Acting Steam Hammer: $Q_a = \frac{(W+ap)H}{6(S+0.25)}$

a = effective area of Piston (cm²).

p = mean effective Steam pressure (kg/cm^2).
(double acting Steam hammer is both lifted & dropped Under Steam Pressure).



(II) Hilley's Formula

- This method is recommended by IS Code also.
- In this method, allowable load Carrying Capacity of Pile is given by -

$$Q_u = \frac{WH \cdot \eta_H \cdot \eta_b}{S + \frac{C}{2}}$$

Where, Q_u = Ultimate load on Pile.

W = Weight of Hammer (kg).

H = Height of drop on Hammer (cm).

S = Penetration on Set (cm) per blow.

C = total elastic Compression (Constant)

$$\therefore C = C_1 + C_2 + C_3$$

C = Temporary elastic Compression of dolly & Packing, pile, Soil respectively.

η_H = efficiency of Hammer.

= 65% - 100% (65% for some double acting Hammer & 100% for drop Hammer.

η_b = efficiency of Hammer blow.

(i.e. ratio of energy after impact to the Striking energy of ram).

'It Signifies the loss of energy during impact'.

$$\eta_b = \frac{W + e^2 P}{W + P}, \text{ If } W > eP$$



or,

$$\eta_b = \frac{W + e^2 P}{W + P} - \left\{ \frac{W - eP}{W + P} \right\}^2, \text{ If } W < eP$$

P = Weight of Pile, helmet & follower.

e = Coefficient of restitution.

= It is Variable from Zero for a timber pile with Poor Condition of head to 0.5 for double action Steam hammer for Steel pile or RCC Piles.

→ The allowable load is given by

$$Q_a = \frac{Q_u}{F}$$

F = Factor of Safety (2-3).

Note: Dynamic formula are best Suited to Coarse grained Soil for Which the Shear Strength is independent of rate of loading as they allow no development of excess Pore Pressure

around the Pile driving if Saturated or dry.

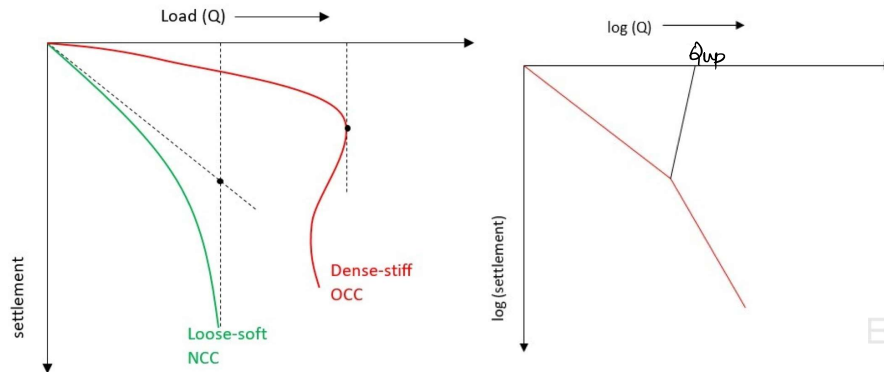
- The Great Objection of any of the Pile driving formula is Uncertainty about the relationship the dynamic & Static resistance of Soil.
- In Case of Submerged loose Uniform fine Sand impact of driving may Cause liquifaction of Soil, thus Showing much less resistance than that Which will Occur Under a Static load.
- On the Same Lines, a very dense Saturated fine Sand may Show an increased driving resistance.
- For Clay, dynamic formula are Useless because the Skin friction developed is very much less & remoulding also takes place.
- Dynamic formula gives no indication about Probable future Settlement.
- It does not take into account the reduced bearing Capacity of Pile in group.
- Weight of Pile in inertia effect is neglected.
- In Hilley's formula, a number of Constant are involved which are difficult to Calculate.

(C) Pile Load Test



- This test Can be performed either on working Pile which forms the foundation of the Structure or on a test Pile.
(Since, after the test Pile Cannot be put into Service/Use, it is a type of destructive test.)
- The test load is applied with the help of Calibrated jack placed Over a rigid Circular or Square Plate which in turn is placed over the head of Pile Projecting above the ground level.
- The reaction of Jack is borne by a Truss which may have gravity loading or Truss Can be anchored to the ground with the help of anchor piles.
- The load is applied in equal increments of about $\frac{1}{5}^{th}$ of estimated allowable load.
- The Settlement is recorded by the help of three dial gauge (placed at an angle of 120°).
- Each load increment is kept/applied upto an extent the rate of Settlement reduces 0.02 mm/hr .
- The test pile is loaded Until Ultimate load is reached, Ordinarily the test load is increased upto 2.5 times the estimated allowable value or a load which causes the Settlement equal

- to $\frac{1}{10}$ th of Pile diameter, which occur earlier.
- The results are reported in terms of load – Settlement Curve either on Arithmetic or log Scale.



- In Case Ultimate load Cannot be Obtained from load Settlement Curve, the allowable load is taken as follows –
- $\frac{1}{2}$ to $\frac{1}{3}$ of final load which cause the Settlement equal to 10% of Pile diameter.
 - $\frac{2}{3}$ of final load which Causes a total Settlement of 12 mm.

- $\frac{2}{3}$ of final load which Causes a net Settlement of 6 mm (residual Settlement after the removal of load.)

- This method is more accurate & recommended by IS Code.
- This method can also be Used to find the allowable load on pile Using Settlement Criteria.
- In Order to separately find both end Bearing resistance & Skin friction resistance Cyclic plate load test is performed.

(IV) Correlation with SPT(N) Value & CPT resistance Value :

(a) Relation with SPT(N) Value

- According to Meyerhoff, for driven piles (Displacement type).

$$Q_{up} = q_{eb} \cdot A_b + q_{sf} \cdot A_s$$

$$Q_{up} = 900 N A_b + 2 \bar{N} \cdot A_s$$

N = SPT no. at the base of Pile.

$\bar{N}$ = Avg. SPT no. over the length of pile.

- For Driven Pile (Non-displacement type) (H-Pile).

$$Q_{up} = Q_{eb} + \frac{1}{2} Q_{sf}$$

$$Q_{up} = 400 N A_b + \frac{1}{2} (2\bar{N}) \cdot A_s$$



(iii) for Bored Piles,

$$Q_{up} = \frac{1}{3} (Q_{up})_{\text{displacement}}$$

$$Q_{up} = \frac{400}{3} \cdot N A_b + \frac{2}{3} \cdot \bar{N} \cdot A_s$$

(b) Relation with CPT resistance

(i) for driven Pile (displacement type)

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

$$Q_{up} = q_c \cdot A_b + \frac{\bar{q}_c}{2} \cdot A_s$$

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(ii) for driven pile (Non displacement type).

$$Q_{up} = Q_{eb} + \frac{1}{2} Q_{sf}$$

$$Q_{up} = q_c \cdot A_b + \frac{\bar{q}_c}{4} \cdot A_s$$

(iii) for Bored Piles.

$$Q_{up} = \frac{1}{3} (Q_{up})_{\text{displacement}}$$

$$Q_{up} = \frac{q_c}{3} \cdot A_b + \frac{\bar{q}_c}{6} \cdot A_s$$

Where, q_c = Static Cone resistance of Soil at base of Pile (Kg/cm^2).

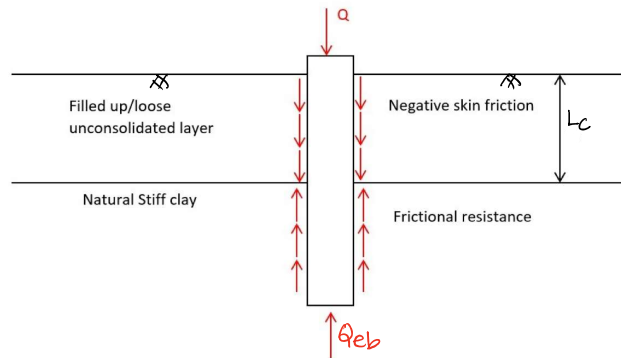
$\bar{q}_c$ = Average Static Cone Resistance of Soil over the length of Pile (Kg/cm^2).



• Negative Skin Friction

- Piles installed in freshly placed fills of soft compressive deposits are subjected to a downward drag, a consequence of consolidation of soil after pile are installed.
- This downward drag on pile surface when the soil moves down relative to the pile is termed as 'NEGATIVE SKIN FRICTION'.
- This negative skin friction may also develop if the fill material is loose, sand deposit or due to lowering of the ground water table which increases the effective stress, thus causing consolidation of the soil.
- It may also be observed due to placement of surcharge over the ground surface due to dynamic load on soil.
- A small relative movement between the soil & the pile of order of about 10 mm, may be sufficient for the full negative skin friction to materialise.

→ In Bearing Piles, where Settlement of Pile is negligible negative Skin friction becomes a pile Capacity problem, but for Piles in Compressive Soil whose pile Capacity is Contributed by both point resistance & Skin, the problem of negative Skin friction Should be Considered as Settlement Problem.



→ This magnitude of negative Skin friction for a Single pile in filled up Soil deposit may be given as -

(a) for Cohesive Soil,

$$F_n = \alpha \cdot \bar{c} A_s$$

$$F_n = \alpha \cdot \bar{c} \cdot P \cdot L_c$$

Where, α = adhesion factor

$\bar{c}$ = Average Cohesion of Compressible layer

L_c = Length of pile in Compressible layer.

P = Perimeter Of Pile.

(b) for Cohesionless Soil,

$$F_n = \frac{1}{2} \cdot K \gamma \cdot L_c \tan \delta \cdot A_s = \frac{1}{2} K \gamma \cdot L_c \tan \delta \cdot P \cdot L_c$$

$$F_n = \frac{1}{2} K_p \cdot \gamma \cdot L_c^2 \cdot \tan \delta$$

Where, K = Lateral earth pressure Coefficient.

δ = Angle of friction between the Pile & Soil.
 $(\frac{1}{2} \phi - \frac{2}{3} \phi)$.

• Group Action Of Pile

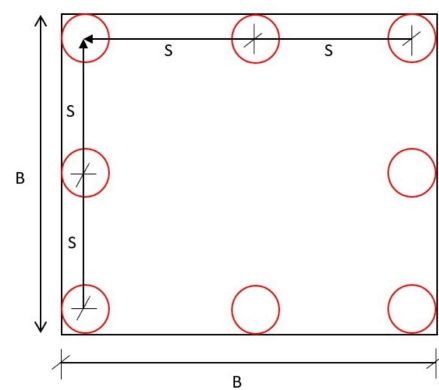
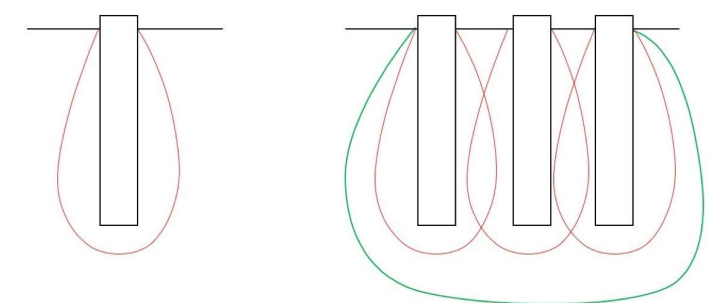
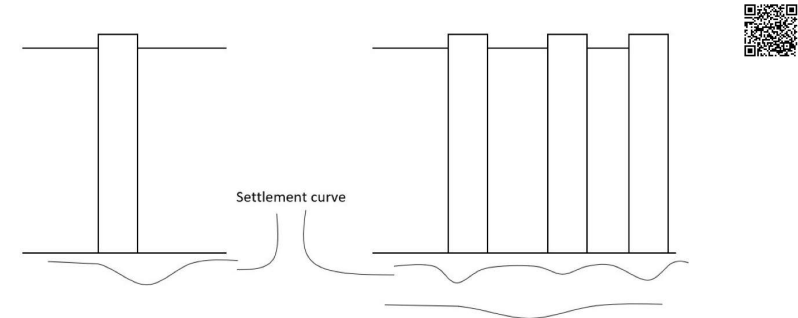
→ If Applied load is Large & more no. of piles are Used, then either Piles will act individually or in the group depending upon the Spacing in between the piles.

→ If c/c Spacing is 2.5D to 4D, then Soil may get Compacted between Piles & entire wedge of Size (B x B) may act as a Single pile Such a action is Called 'GROUP ACTION OF PILE'.

→ In group action, base Area & Surface area both will increase.

→ In group action, the depth of Stress Zone extends to greater depth than in Individual action of Pile therefore, Settlement due to Consolidation in group action is more than Settlement in Individual action of Pile.

- The minimum no. of Pile required for Group action is '3'.
- The Pile group may be triangular, Circular, rectangular or Polygon (however Square Pile group is Preferred).
- Group action depends upon c/c Spacing between Piles.
 - (i) for end bearing Pile Spacing is $(2.5 - 3.5)D$.
 - (ii) for Friction pile Spacing is $(3 - 4)D$.
 - (iii) for Pile in loose Sand Spacing is kept $2D$.
- If the Spacing is governed as above then load carrying Capacity of Pile group in Sand comes to be greater than Sum of load Carrying of all the piles, whereas in case of Clay, it depends upon properties of Soil & Spacing.
- If Piles are to be driven, then Pile driving mechanism should Start from centre & proceed outward i.e. It means Central pile is to driven first & processing piles are driven in radially outward direction as in this process resistance in pile driving will be less & hence it would be more economical.



$$\begin{aligned}
 3 \times 3 \quad B &= 2S + d \\
 4 \times 4 \quad B &= 3S + d \\
 5 \times 5 \quad B &= 4S + d \\
 \text{for } m \times n \\
 B &= (m-1)S + d
 \end{aligned}$$

• Determination of Load Carrying Capacity of Pile Group.

$$Q_{ug} = Q_{eb} + Q_{sf} \Rightarrow Q_{ug} = q_{eb} \cdot A_b + q_{sf} \cdot A_s$$



(i) For Cohesive Soil

$$Q_{ug} = q_c B^2 + \alpha \bar{c} (4BL)$$

Here, $\alpha = 1$ (adhesion between Soil & Soil).

$$Q_{ug} = q_c B^2 + \bar{c} (4BL) *$$

C = average cohesion at Base of Pile group.

$\bar{c}$ = Average cohesion at Base of Pile group.

(ii) For Cohesionless Soil

$$Q_{ug} = q N_q \cdot A_b + \frac{1}{2} K \gamma L \tan \delta \cdot A_s$$

Here, $\delta = \phi$ (friction angle between Soil & Soil).

$$Q_{ug} = \gamma L N_q B^2 + \frac{1}{2} K \gamma L \tan \phi (4BL)$$

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• Allowable Load/Safe Load On the Pile Group

→ Allowable Load/Safe load on the Pile group is given as minimum of following –

$$Q_{safe} = \left. \begin{array}{l} Q_{ug}/f \\ n Q_{up}/f \end{array} \right\} \text{minimum}$$

Where, n = no. of Piles in Group.

Q_{up} = Individual Pile Capacity.

Q_{ug} = Pile Capacity of Group.

f = FOS.

• Group Efficiency



→ It is defined as ratio of Ultimate load Carrying Capacity of Pile group to the Sum of Ultimate load Carrying Capacity of all the piles Under individual action.

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

Where, n = no. of Piles in the Group.

Note: If $\eta_g > 1$, then $Q_{safe} = n \cdot \frac{Q_{up}}{f}$

Therefore, In design, η_g should be ≥ 1 .

- Disturbance of Soil during installation of pile & Overlap of Stresses between adjacent piles may Cause the group Capacity to become less than the Sum of individual Capacity i.e. $\eta_g < 1$.
- Generally, for smaller Spacings between the piles

$\eta < 1$, for larger spacing, the effect of pile interaction reduces & η_g approaches Unity.

- In driven piles, where the soil around the piles get densified as in loose to medium sand, η_g may be more than 1.
- In this case, group tends to behave like a block or like equivalent single pile.
- The η_g depends mainly on the spacing between piles, type of soil in which pile is installed & manner of pile installation (i.e. driven, bored).
- η_g can also be computed empirically as using:-

(a) Converse Labarre Formula

$$\eta_g = 1 - \frac{\theta}{90} \left\{ \frac{(n-1)m + (m-1)n}{mn} \right\}$$

Here, $\theta = \tan^{-1}(d/s)$ {degrees}

Where, m = no. of rows

n = no. of piles in a row.

d = diameter of piles.

s = Spacing of piles.

(b) Feld's Rule

- According to this rule, the value of each pile is reduced by $\frac{1}{16^{th}}$ on account of effect of nearest

pile in each diagonal or straight row of which the pile in question is a member.

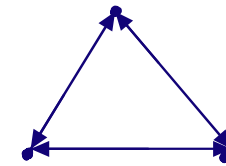
eg:



2 piles @ $15/16$

$$\eta_g = \frac{2 \times (15/16)}{2} \times 100$$

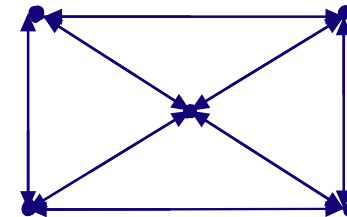
$$= 93.75 \%$$



3 piles @ $14/16$

$$\eta_g = \frac{3 \times (14/16)}{3} \times 100$$

$$= 87.5 \%$$



4 pile @ $13/16$ + 1 pile @ $12/16$

$$\eta_g = \frac{4 \times (13/16) + 1 \times (12/16)}{5} \times 100$$

$$\eta_g = 80 \%$$

• Group Settlement Ratio

- For eliminating the settlement of pile group in sand, pile load test is used {interpolation of load settlement curve is done}.
- Settlement of no. of pile groups, consisting of driven piles, with the settlement of individual

Pile in Sand, for the same load per pile after being compared can be expressed in terms of settlement of an individual pile & is termed as 'GROUP SETTLEMENT RATIO' given by -

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

Where, B = Width of Pile group (m).

S_g = Settlement of Pile group.

S_i = Settlement of individual pile.

Note: The above ratio is for same load Q per pile.

→ S_g in Sand varies in the range of 1-16, irrespective of width of pile.

• Determination of Settlement of pile group in Clay

→ The settlement of a pile group in Clay cannot be estimated from the data of load test on a single pile, because of time effect, the effect of remoulding of soil due to pile driving & the scale effect, which are different for the single test pile & group of pile.

→ In this case, approach used for settlement of

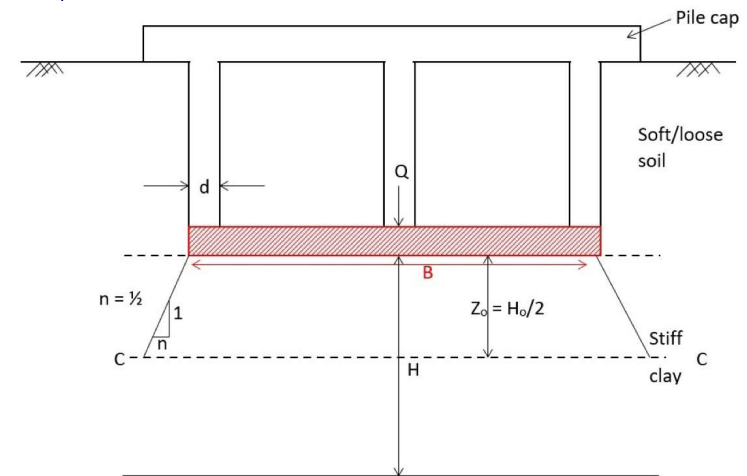
Pile group is 'Equivalent Raft Approach'.

→ Depending on the sub soil conditions, several assumptions are being made to identify the location of equivalent raft over which the pile are assumed to transfer the vertical load acting on them.

→ Case (i): When Pile are end Bearing

→ It means the soil below the base of pile is stiff/dense whereas above the base of pile is loose/soft.

→ In this case, skin friction is negligible & equivalent raft may be assumed at the base of pile group.



→ In this Settlement is Computed as –

(a) Assume equivalent raft at Base of Pile Group.

(b) Identify thickness of Compressible Soil (H_0).

(c) σ'_0 at CC.

(d) Increase in effective Stress $\Delta\sigma'$.

$$\Delta\sigma' = \frac{Q}{(B + 2nz)^2}$$

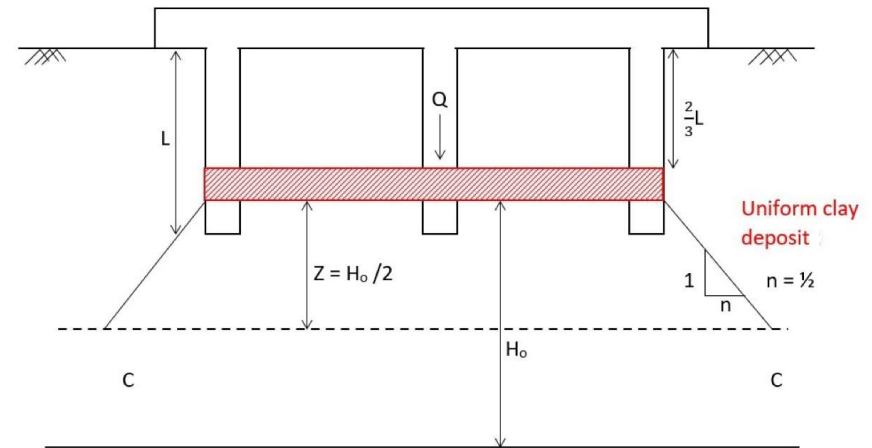
$$(e) \Delta H = \frac{H_0 C_c}{1 + e_0} \cdot \log\left(\frac{\sigma'_0 + \Delta\sigma'}{\sigma'_0}\right)$$

→ Case (ii): When Piles are driven through Uniform Clay deposit & pile group act as a friction pile group.

→ Let the top layer is Soft, of length L_1 & Bottom layer is Stiff and is embedded upto the length of L_2 .

→ In this Case, end Bearing resistance & Skin friction resistance both will be developed.

→ The equivalent raft in this case assumed to be present at $\frac{2}{3} L_2$ from base of top layer & remaining procedure is Same.



• Guidelines to Design a Pile Group



(i) Length: For friction pile group length may be 10-20 m. Whereas for end Bearing pile group, it is equal to depth of hard strata.

(ii) Diameter: It is kept in range of 0.3 – 0.9 m.

(iii) Spacing: It is kept in range of 2.5D – 4D (generally)

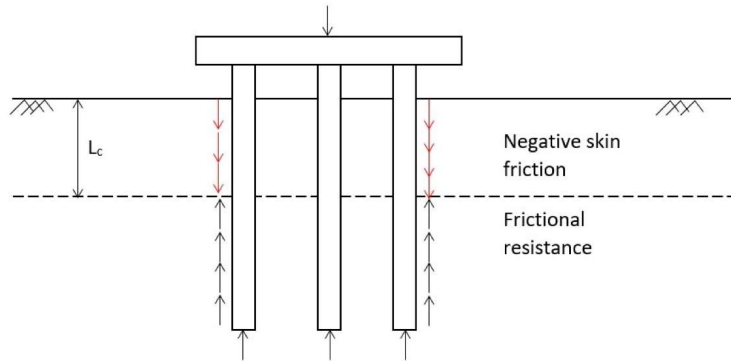
(iv) No. of Piles in Group: Generally Square pile group $[3 \times 3, 4 \times 4, 5 \times 5]$ is preferred.

(v) Group Efficiency: It should be ≥ 1 .

Note: In end Bearing pile, $Q_{up} = Q_{eb}$

In friction pile, $Q_{up} = Q_{sf}$

• Negative Skin Friction in Pile Group



→ When pile group passes through a Soft, Unconsolidated Stratum, the magnitude of negative skin friction on the group F_{ng} is given as higher of the value obtained through following relationship.

$$F_{ng} = n \cdot F_n \text{ ————— ①}$$

$$F_{ng} = \alpha \cdot \bar{C}_u \cdot L_c \cdot P_g + \gamma L_c A_g$$

($\because \alpha = 1 \rightarrow$ adhesion between Soil & Soil)

$$F_{ng} = \bar{C}_u \cdot L_c \cdot P_g + \gamma \cdot L_c \cdot A_g \text{ ————— ②}$$

Where,

n = no. of Piles in group.

P_g = Perimeter of group.

γ = Unit weight of Soil with pile group upto depth L_c .

A_g = Area of Pile group within the perimeter P_g .



Note: The Second relationship is Computed on the basis of block Shear failure along the Perimeter of Pile group which includes the Volume of Soil enclosed in the group ($A L_c$).

→ The effect of negative skin friction on the FOS w.r.t estimate Load Capacity of pile or a Pile group Can be Considered as –

$$FOS = \frac{\text{Ultimate load Capacity of Single pile/pile group}}{\text{Working load \& negative Skin friction Load.}}$$

Note: $Q_u = Q_{eb} + Q_{sf}$ (for Cohesionless Soil)

$$Q_{eb} = q_{eb} \cdot A_b \Rightarrow q_{eb} = q \cdot N_q \Rightarrow q = \gamma \cdot L$$

$$q_{eb} = \gamma \cdot L \cdot N_q$$

$$q_{eb} \propto L$$

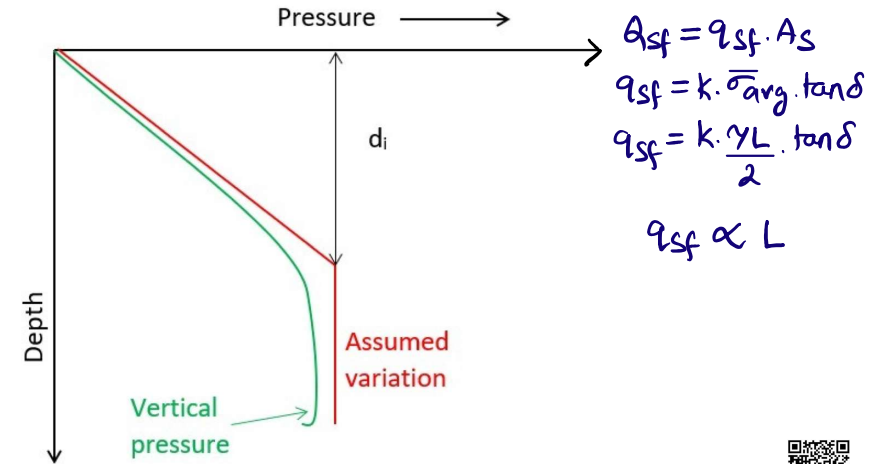
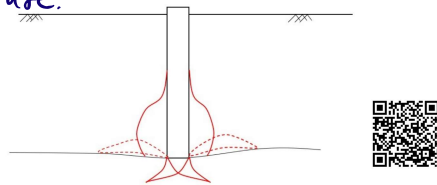


→ From the above relationship it Can be Analysed that the Unit point resistance increases in direct proportion to the embedded length of pile.

→ However, field Observation indicate that these

values increases only upto a limited depth; beyond which it remains constant.

- This depth is called the 'Critical depth of Pile'.
- This Phenomenon is due to 'ARCH EFFECT' in the granular Soil.
- The critical depth depends on the angle of Shearing resistance of Soil & Size of pile.
- Its value may vary from about 15D in loose to medium Sand to 20D in dense Sand Where D is pile diameter or width.
- It is also recommended that maximum value of Unit point resistance q_{eb} be 11,000 kN/m² in normal Silica Sand or 5000 kN/m² for Calcareous Sand.
- 'Arching' Occurs when there is a difference of the Stiffness between the installed Structure & the Surrounding Soil.
- If the Structure is Stiffer than the Soil, then load arches onto the Structure otherwise, if the Structure is less Stiffer than Soil, then load arches away from the Structure.



- From the above relationship it can be analysed that q_{sf} increases with depth, but in actual it does so only upto Critical depth which varies from 15-20 times the pile diameter.
- Below the Critical depth, the value of $\bar{\sigma}$ & the value of q_{sf} remains constant.
- The maximum value of q_{sf} should be limited to 100 kN/m² for Straight piles in normal Silica Sand & upto 20 kN/m² for Calcareous Sand.

Q. A Single Vertical friction pile of diameter 500 mm & length 20 m is Subjected to a Vertical Compressive load. The pile is embedded in a

homogeneous Sandy Strata where angle of internal friction $\phi = 30^\circ$, $\gamma = 20 \text{ kN/m}^3$ & angle of wall friction $\delta = \frac{2}{3}\phi$. Considering the Coefficient of lateral earth pressure $K = 2.7$, $N_q = 25$, find the Ultimate bearing Capacity of Pile. Check for Arching Effect.

Sol: $\frac{L}{D} = \frac{20}{0.5} = 40 > 15$.

$$\begin{aligned} Q_{eb} &= q_{eb} \cdot A_b \\ \Rightarrow q_{eb} &= q N_q \\ \Rightarrow q_{eb} &= \gamma (15D) N_q \\ &= 150 \times 25 \\ &= 3750 < 11000 \text{ kN/m}^2. \end{aligned}$$

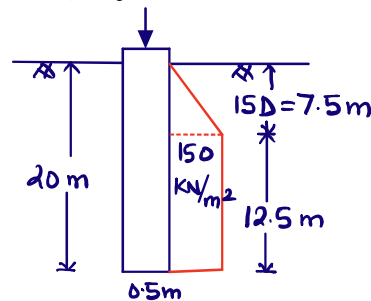
$$\begin{aligned} Q_{eb} &= q_{eb} \cdot A_b \\ &= 3750 \times \frac{\pi}{4} (0.5)^2 \end{aligned}$$

$$= 736 \text{ kN}.$$

$$\begin{aligned} \Rightarrow q_{sf} &= q_{sf1} + q_{sf2} \quad \Rightarrow q_{sf1} = K \cdot \bar{\sigma} \cdot \tan \delta \\ &= 2.7 \times \frac{150}{2} \cdot \tan\left(\frac{2}{3} \times 30^\circ\right) \end{aligned}$$

$$\Rightarrow q_{sf1} = 73.70 \text{ kN/m}^2.$$

$$\Rightarrow q_{sf2} = K \cdot \bar{\sigma} \cdot \tan \delta = 2.7 \cdot \left\{ \frac{150 + 150}{2} \right\} \cdot \tan\left(\frac{2}{3} \times 30^\circ\right)$$



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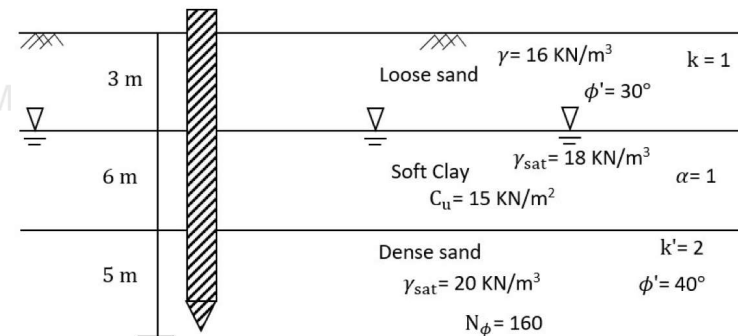
$$\Rightarrow q_{sf2} = 147.4 \text{ kN/m}^2 > 100 \text{ kN/m}^2.$$

$$\Rightarrow q_{sf} = 73.70 \times \pi (0.5) 15 \times 0.5 + 100 \times \pi \times (0.5) \times 7.5$$

$$\Rightarrow \boxed{q_{sf} = 2831.5 \text{ kN}}$$

$$\Rightarrow Q_u = Q_{eb} + Q_{sf} = 736 + 2831.5 = 3567.7 \text{ kN}$$

Q. Determine the allowable pile load Capacity of the 40 cm diameter, driven Concrete pile, $F = 2.5$



Sol: (i) Loose Sand

$$\frac{L}{D} = \frac{3}{0.4} = 7.5 < 15, \text{ here no arching effect}$$

$$\Rightarrow Q_{sf} = q_{sf} \cdot A_s$$

$$q_{sf1} = K \cdot \bar{\sigma} \cdot \tan \delta = 1 \times \frac{16 \times 3}{2} \times \tan\left(\frac{3}{4} \times 30^\circ\right)$$

$$= 9.9 \text{ kN/m}^2 < 100 \text{ kN/m}^2.$$

$$Q_{sf1} = 9.9 \times \pi \times 0.4 \times 3 = 37.47 \text{ kN}$$



(ii) Soft Clay

$$Q_{sf2} = \alpha \cdot C_u \cdot A_{s2} = 1 \times 15 \times \pi (0.4) 6 \Rightarrow Q_{sf2} = 113.09 \text{ kN}$$

(iii) Dense Sand

$$\frac{L}{D} = \frac{5}{0.4} = 12.5 < 20, \text{ No Arching Effect.}$$

$$Q_{sf3} = q_{sf3} \cdot A_{s3}$$

$$q_{sf3} = k \cdot \bar{\sigma}' \cdot \tan \delta$$

$$\Rightarrow \bar{\sigma}'_t = 16 \times 3 + 6(18-10) = 96 \text{ kN/m}^2$$

$$\Rightarrow \bar{\sigma}'_b = 16 \times 3 + 6(18-10) + 5(10) = 146 \text{ kN/m}^2 \quad \left\{ \bar{\sigma}'_b = \bar{\sigma}'_{\text{bottom}} \right\}$$

$$\Rightarrow \bar{\sigma}' = \frac{96 + 146}{2} = 121 \text{ kN/m}^2$$

$$\Rightarrow q_{sf1} = 2 \times 121 \times \tan\left(\frac{3}{4} \times 40^\circ\right) = 139.71 \text{ kN/m}^2 > 100 \text{ kN/m}^2$$

$$\Rightarrow Q_{sf3} = 100 \times \pi \times (0.4) \times 5 = 628.31 \text{ kN}$$

$$\Rightarrow Q_{eb} = q_{eb} \cdot A_b$$

$$\therefore q_{eb} = q_{Nq} = \bar{\sigma}'_b \cdot N_q \Rightarrow 146 \times 160 = 23360 > 11000 \text{ kN/m}^2$$

$$\therefore Q_u = (Q_{sf1} + Q_{sf2} + Q_{sf3}) + Q_{eb}$$

$$Q_u = 2161 \text{ kN} \quad \therefore Q_a = \frac{Q_u}{F} = \frac{2161}{2.5} \Rightarrow Q_a = 864.5 \text{ kN}$$

• Piles Subjected to Uplift Loads



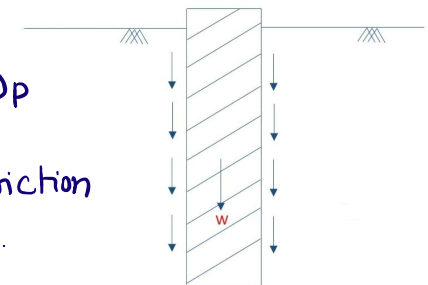
- Structure Such as tall towers, Silos, Chimneys, Off Shore platforms and dry docks are Usually provided with pile foundation to resist the large Uplift pressure (due to Water) & Overturning moments.
- Piles Used for this purpose are Called as Uplift or tension Piles.
- The Uplift Capacity of Pile is Calculated in a manner Similar to pile Subjected to Compressive load.
- Uplift Piles are invariably provided with an enlarged area at the base in form of bulb or ball.
- Pile develop resistance to pull-out Only from the Skin friction develop along the embedded length & point bearing is not included but the weight of pile is included in Uplift Resistance.

→ Case (i) Pile in Clay

$$Q_{ut} = q_{sft} \cdot A_s + W_p$$

Where,

q_{sft} = Unit Skin friction in tension.



A_s = embedded area of Pile Shaft.

W_p = Weight of Pile.

q_{stt} is taken equal to q_{sf} in Compression for Undrained Conditions.



→ When the Base of Pile is enlarged in form of bulb or a ball, the Smaller of two Values is Considered given by –

$$Q_{ut} = C_u \cdot \bar{A}_s \cdot K + W_s + W_p \text{ ——— (I)}$$

or,

$$Q_{ut} = 2.25 \cdot \pi (D_b^2 - D^2) \cdot C_u + W_p \text{ ——— (II)}$$

Where,

$\bar{A}_s$ = Surface area of Vertical Cylinder above the base.

D_b = Diameter of Base

D = Diameter of Pile Shaft.

K = Coefficient.

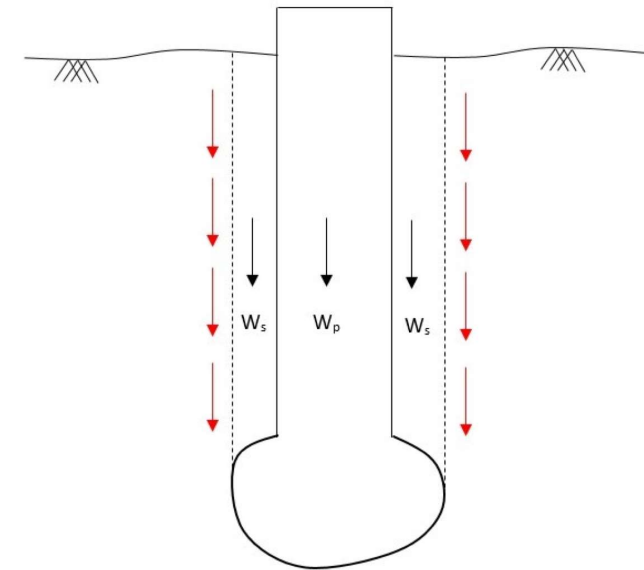
W_s = weight of Soil included in the region between pile Shaft and Cylinder above the base.

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Type of Soil	K
Soft Clay	1–1.25
Medium Clay	0.7
Stiff Clay	0.5
Fissured Clay	0.25



Note : The (i) equation is on basis of failure assumed through full mobilisation of friction resistance along the cylindrical surface above the base diameter and eq (ii) is based on bearing Capacity failure of Base.



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FOUNDATION ON EXPANSIVE SOIL

- Expansive Soils or Swelling Soils are those which have the tendency to increase in volume when water is available & to decrease in volume if water is removed.
- Eg: Black Cotton Soil, Bentonite Soil.
- Black Cotton Soil of India have liquid limit (W_L) ranging from 50–100 %, plasticity Index (I_p) ranging from 20–65 % & Shrinkage limit (W_s) from 9–14 %.
- The minus 2 μ fraction in soil varies from 40–75 %, The soil is very hard in dry state and possess high shearing strength which get reduced appreciably with addition of water.
- In general, expansive soil have $W_L \uparrow$, $W_s \downarrow$, $I_p \uparrow$, $I_s \uparrow$.
- The swelling behaviour of soil would depend largely on the type of clay mineral present in the soils and proportions of these minerals.
- In order to determine this behaviour following techniques are available –
 - (i) Differential Thermal Analysis (DTA).
 - (ii) X-Ray diffraction method.

(iii) Electron Microscopy.

- The DTA method is based on the fact that certain characteristic reaction takes place at specific temperature for different minerals. When these are heated to high temperature, resulting in loss or gain in heat.
- Different minerals will have different regular patterns of crystalline structure will different X-ray to yield different X-ray diffraction pattern.
- In electron-microscopy the soil is observed under polarised light in a electron microscope to identify the characteristic strain of a mineral.
- From engineering point of view, following test are performed to find the swelling potential of soil.

(i) Free Swell test

- This is performed by pouring slowly 10 cm³ of dry soil passing through 425 micron sieve, into a 100 cc graduated cylinder filled with water.
- The volume of swelled soil is read after 24 hrs from graduation of cylinder to give free swell value.

$$\text{Free Swell (\%)} = \frac{\text{Final Volume} - \text{Initial Volume}}{\text{Initial Volume}} \times 100$$

- Bentonite Soil, containing montmorillonite, have free Swell Value of 1200 – 2000 %.
- Kaolinite has free Swell Value of 80 % & illite have Swell value of 30 – 80 %.

(ii) Differential free Swell test

- In this test, two Sample of dried Soil weighing 10 gm each, passing through 425 μ Sieve are taken.
- One is put in a 50 cc graduated glass Cylinder Containing Kerosene oil (a non polar liquid).
- The other Sample is put in a Similar Cylinder Containing distilled water.
- Both the sample are left Undisturbed for 24 hours and Volume is noted.
- The differential free Swell (DFS) is expressed.

$$\text{DFS} = \frac{\text{Soil Volume in Water} - \text{Soil Volume in Kerosene}}{\text{Soil Volume in Kerosene}} \times 100$$

- Degree of expansiveness & Possible damage on Lightly Loaded Structure may be referred as follows –

Degree of Expansiveness	DFS(%)
Low	< 20
Moderate	20 – 35
High	35 – 50
Very High	> 50

Note : Expansive/Swelling potential of Soil can also be related as follows –

Colloid Content(%)	Ip(%)	Ws(%)	Probable Expansion (%)	Degree of Expansion
< 15	< 18	> 15	< 10	Low
13 – 23	15 – 28	10 – 16	10 – 20	Medium
20 – 31	25 – 41	7 – 12	20 – 30	High
> 28	> 35	< 11	> 30	Very High

Swelling Potential	Ip(%)
Low	0 – 15
Medium	10 – 35
High	20 – 35
Very High	> 35

Ws (%)	Linear Shrinkage	Degree of Expansion
<10	>8	Critical
10-12	5-8	Marginal
>12	0-5	Non-Critical

(iii) Swelling Pressure Test

- When an expansive Soil imbibes water from outside, pressure builds up inside the Soil.
- If free Swelling of Soil is restrained by placement of Structure over the Soil, this pressure is called 'SWELLING PRESSURE' and is exerted by Soil on the Overlying Structure.
- The force required to prevent expansion in the Soil is function of time.
- A Swelling pressure less than 20 kN/m² may not be regarded as of much Consequence.

• Field Condition that favours Swelling

- A Soil may have high Swelling potential but still may or may not swell in actual practice.
- This would depend on many factors, the most significant of them is being the difference between the field moisture content & equilibrium moisture content that will be materialised after construction.

If $FMC < EMC \rightarrow$ Expansion of Soil results.
 $FMC > EMC \rightarrow$ Shrinkage of Soil results.

- A fill which has been compacted to a great degree or an over consolidated deposit will have more tendency to swell, given access to water.
- If load that is placed on Swelling Soil is more, the Swelling of Soil is inhibited & vice-versa.

• Design of Foundation On Expansive Soil

- The rational method of foundation design which are being used to reduce or prevent the effect of Swelling can be grouped into 3 Categories -
 - (i) Isolating the Structure from Swelling Soil.
 - (ii) Designing a Structure to withstand the effect of Swelling.
 - (iii) Preventing the Swelling.
- (i) In first Approach either belled piers or Under-reamed piles are provided.
- The principle of Under-reamed piles is based on fact that to involve the load transfer to pile at a depth beyond the Zone of Seasonal Variation.

- The depth of black Cotton Soil is approx 3.5 - 4 m in India.
- For other Expansive Soil, the pile is taken down to atleast 0.6 m into the non-expansive layer Under lying the Swelling Soil.
- The Under-seamed piles are boxed Cast-insitu piles with their lower portion enlarged or seamed in the form of Bulb.
- The Spacing of pile is in the range of 1.5 - 3 m.
- Diameter of these Piles Varies between 200-500 mm & ratio of diameter of enlarged Base or bulb to that of Shaft (D_u/D) is in range of 2-3.
- In Case of multi-seamed pile having more than One bulb, the top most bulb Should be at a minimum depth of 2 times the bulb diameter
- The C/C distance between the bulb is $(1.25 - 1.5) D_u$.
- A FOS of 2.5-3 is Used to Calculate Safe load.
- For Clay Soil, the Ultimate load Carrying Capacity of Under-seamed pile is given by -

$$Q_u = A_p \cdot N_c \cdot C_p + A_a N_c \cdot C_a' + C_a' \cdot A_s' + \alpha \cdot C_a \cdot A_s$$

Where, Q_u = Ultimate Bearing Capacity of pile.

A_p = Cross-section area of pile Stem at toe level. $(\pi D^2/4)$.

N_c = Bearing Capacity factor (9).

A_a = Area of Soil Surrounding the Stem below the bulb. $\left\{ \frac{\pi}{4} (D_u^2 - D^2) \right\}$.

C_a' = Average Cohesion of Soil around Under-seamed bulb.

α = reduction factor.

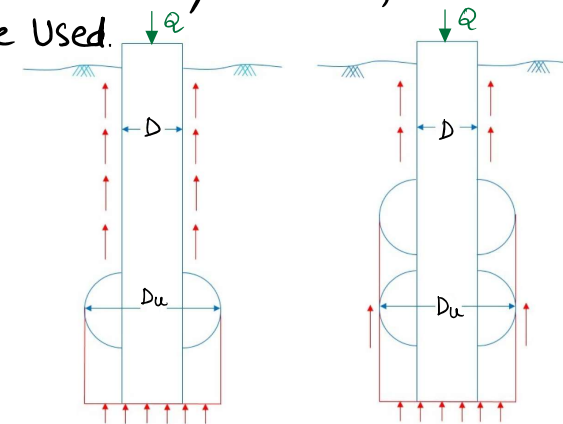
C_p = Average Cohesion of Soil around the toe.

C_a = Average Cohesion of Soil along pile stem.

A_s = Surface Area of Stem.

A_s' = Surface Area of Cylinder Circumscribing the bulb.

Note: If Pile has Only One bulb, third term will not be Used.



(ii) Designing a Structure which is Strong & rigid enough to withstand the effect of Swelling may prove to be highly Uneconomical, except in case of Very Small Structure Where even if the loads are supported by the Central area or Peripheric-al area much Smaller than the plan area, the bearing pressure are within limits.

(iii) Swelling can often be Controlled but Cannot be eliminated by providing an impervious apron around the Structure.

→ By providing the apron, the moisture gradient between the Centre of the Structure & its edges is minimised hence the differential Swelling is Controlled.

→ Elimination of possible Swelling can be achieved by -

(a) Pre-wetting the ground to a moisture Content equal to the equilibrium moisture Content.

(b) Making downward load large to exceed Swelling pressure.

(c) By Chemical Stabilisation.

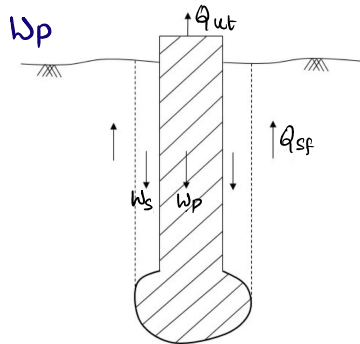
Q. A pile 45 cm in diameter & 20 m long is driven into a Clay Soil with $C_u = 40 \text{ kN/m}^2$, $\phi_u = 0$, $\gamma_{\text{sat}} = 19 \text{ kN/m}^3$ ground water is almost at ground level.

If pile has an enlarged base of diameter 1.25 m Determine the Uplift Capacity of pile Under Undrained Condition. Submerged Unit weight of pile is 40 kN.

Also Consider Skin friction resistance while analysing the Capacity by bearing failure.

Sol: $Q_{ut} = C_u \cdot \bar{A}_2 \cdot K + W_s + W_p$

Assume, $K = 0.75$.



$$Q_{ut} = 40 \times \pi(1.25) \times 20 \times 0.75 + (19 - 10) \times 20 \times \frac{\pi(1.25^2 - 0.45^2)}{4} + 40$$

$$Q_{ut} = 2588.45 \text{ kN}$$

Or,

$$Q_{uf} = 2.25 \cdot \pi (D_b^2 - D^2) C_u + W_p + \alpha \cdot C_u \cdot A_s$$

$$= 2.25 \times 3.14 \times (1.25^2 - 0.45^2) \times 40 + 40$$

$$+ 0.5 \times 40 \times \pi \times (0.45) \times 20$$



$$Q_{uf} = 990 \text{ KN.}$$

$$\therefore Q_{uf} = (2588.45, 990) \text{ KN minimum}$$

$$Q_{uf} = 990 \text{ KN}$$

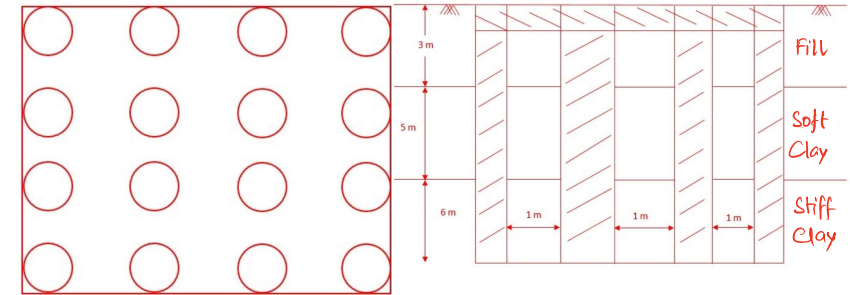
Q. A group of 16 piles of diameter (500 cm), length= 14 m, C/C Spacing of 1m arranged in Square pattern passes through a recent fill (thickness 3m) Overlying a Soft Clay deposit (thickness 5m) Which is Consolidating Under the fill load & rest is Stiff Clay Strata.

All the Strata are Saturated & Soil properties are given below.

Estimate Ultimate Load Carrying Capacity of Pile group.

Type of Soil	Unit Wt. (KN/m ³)	Strength Parameter C _u (KPa)	φ _u	Adhesion Parameter
Fill	16	50	0	0.6
Soft Clay	17	20	0	0.4
Stiff Clay	21	70	0	0.45

Sol:



(i) Pile acting Individually

$$Q_{ug} = n \cdot Q_{up}$$

$$= n [Q_{eb} + Q_{sf}]$$

$$= n [q_{eb} \cdot A_b + q_{sf} \cdot A_s]$$

$$= 16 [C_u \cdot N_c \cdot A_b + \sum \alpha_i \cdot C_{ui} (\pi \cdot D) L_i]$$

$$= 16 \left[70 \times 9 \times \frac{\pi}{4} \times (0.5)^2 + (\pi \times 0.5) \times \{ 0.6 \times 50 \times 3 + 0.4 \times 20 \times 5 + 0.45 \times 70 \times 6 \} \right]$$

$$Q_{ug} = 9996.54 \text{ KN.}$$

(ii) Pile Acting in Group

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

$$Q_{ug} = q_{eb} \cdot A_b + q_{sf} \cdot A_s$$

$$= C_u \cdot N_c \cdot A_b + \sum \alpha \cdot C_{ui} \cdot (4B) \cdot L_i$$

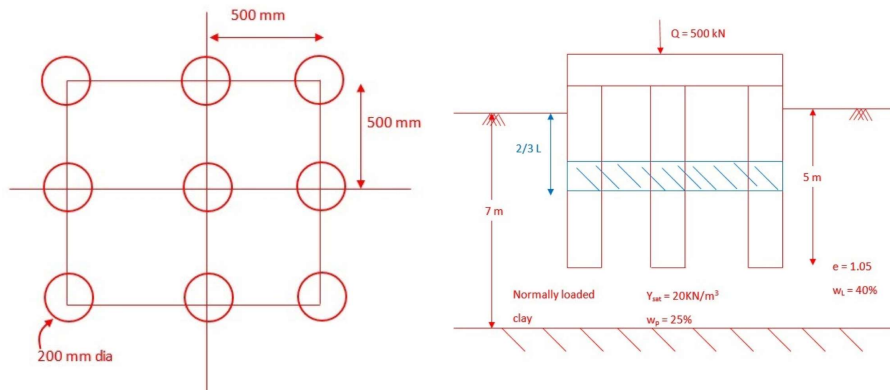
$$= 70 \times 9 \times (3.5)^2 + 1 \times 4 \times 3.5 [50 \times 3 + 20 \times 5 + 70 \times 6]$$

$$Q_{ug} = 17097.5 \text{ kN}$$

$$Q_{ug} = (9996.54, 17097.5) \text{ kN minimum}$$

$$Q_{ug} = 9996.54 \text{ kN}$$

Q. Calculate the Settlement of 9 pile group for Condition indicated as follows.



Sol: Location of equivalent raft from Ground Surface = $\frac{2}{3} \times L = \frac{2}{3} \times 5 = 3.34 \text{ m}$.

Thickness of Centre of compressible layer from Ground Surface, $= 3.34 + \frac{8.67}{2} = 5.17 \text{ m}$.

$$\Rightarrow \sigma'_0 = \gamma' D = (20 - 10) \cdot 5.17 = 51.7 \text{ kN/m}^2$$

$$B = 2s + d = 2 \times 0.5 + 0.2 = 1.2 \text{ m}$$

$$B' = B + 2nz = 1.2 + 2 \times \frac{1}{2} \times \frac{3.67}{2} = 3 \text{ m}$$

$$\Delta \sigma' = \frac{500}{3^2} = 55.5 \text{ kN/m}^2$$

$$\Delta H = \frac{C_c \cdot H_0}{1 + e} \log_{10} \left\{ \frac{\sigma'_0 + \Delta \sigma'}{\sigma'_0} \right\}$$

$$C_c = 0.009 (w_L - 10) = 0.009 (40 - 10) = 0.27$$

$$\Delta H = \frac{0.27 \times 3.67}{1 + 1.05} \log_{10} \left\{ \frac{51.7 + 55.5}{51.7} \right\}$$

$$\Delta H = 153 \text{ mm}$$

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STABILISATION OF SOILS



- Stabilisation of the Soil includes Various methods for modifying the properties of the Soil to improve its engineering performance.
- Stabilisation is being Used for a Variety of engineering Works like roadways, airfields, Foundation work etc.
- The main objective in this Case is to increase the Strength or Stability of the Soil to make it more Cost effective.
- Methods of Stabilisation is grouped into two –
 - (a) Modification or Improvement of a Soil property of the existing Soil (Compaction, drainage).
 - (b) Modification of property with the help of admixtures (Mechanical Stabilization, Bitumen, Lime, Cement Chemical, Electrical, Thermal Stabilisation).

(i) Mechanical Stabilisation



- It involves two Operations:
 - (a) Changing the Composition of Soil by addition or removal of Certain Constituents.
 - (b) Densification or Compaction.

- This is been Carried out largely in Construction of Cheap roads Using locally available material.



(ii) Cement Stabilisation

- The Soil Stabilised with Cement is Known as Soil Cement.
- The Cementing action is due to result of the Chemical reaction of Cement with Siliceous Soil & by binding action of the individual particles through Cement (In Coarse Grained Soil).
- In Case of fine Grained Soil binding action is due to Cohesion.
- Well Graded Soil with less than 50 % fraction finer than 75 μ Sieve & $I_p < 20$ are found to be most responsive to Cement Stabilisation.
- The amount of Cement required is in range of 5-15 % by weight of dry Soil.
- For Cement, it is 5-10 %.
- For Sand, it is 7-15 %.
- For Silt, it is 12-15 %.
- For Clay, it is 12-20 %.
- The Amount of Water to be added is decided from Consideration of Good Compaction and this amount must be adequate for Complete

Hydration of Cement.

Q. Calculate the number of Cement bag required for preparing a layer of Settlement having Surface Area of $5 \times 5 \text{ m}^2$ & thickness of 50 cm with dry density of 18 kN/m^3 . If 10% of Cement (assume) by weight of dry Soil is required.

Sol: Let weight of Soil be 'x' Kg.

$$\text{Weight of Cement} = 10\% \text{ of } x = 0.1x \text{ kg}$$

$$\text{Weight of Soil Cement} = x + 0.1x = 1.1x \text{ kg} \quad \text{--- (i)}$$

$$\begin{aligned} \text{Weight of Soil Cement Stabilised} &= 25 \times 0.50 \times 18 \times 10^2 \\ &= 225 \times 10^2 \text{ kg} \\ &= 22500 \text{ kg} \quad \text{--- (ii)} \end{aligned}$$

$$1.1x = 22500$$

$$\Rightarrow x = 20454.54 \text{ kg.}$$

$$\begin{aligned} \text{Weight of Cement} &= 0.1 \times x = 0.1 \times 20454.54 \\ &= 2045.5 \text{ kg} \end{aligned}$$

$$\text{Number of Cement bags} = \frac{2045.5}{50} = 40.91$$

$$\therefore \text{Number of Cement bags} = 41 \text{ Bags.}$$

(iii) Lime Stabilisation

- Hydrated Lime is very effective in treating heavy Plastic Clay Soil.
- Lime may be Used alone or in Combination with Cement, Bitumen, flyash.
- Sandy Soil Can also be Stabilised using this Combination.
- It is mainly Used for Stabilising the roads, Sub-grades etc.
- On addition of lime to Soil, two types of Chemical reaction occurs
 - Alteration in the nature of absorbed layer through base exchange phenomenon.
 - Cementing or Pozzolonic action.
- Lime reduces the Ip of highly plastic Soil. It increases the Optimum moisture Content and decreases the Compacted density but Strength durability increases.
- Normally, 2-8 % of lime may be required for Coarse grained Soil and 5-10 % for plastic Soil.

(iv) Bitumen Stabilisation

- Asphalt and tar are the bituminous material which are used for stabilisation of soil for pavement construction.
- Since the viscosity of these material is comparatively very high, it is 1st reduced before adding it into the soil.
- The bituminous material when added imparts binding action or water proofing action or both in soil.
- Depending upon these action & nature of soil bitumen stabilisation is further classified as –
 - a. Soil bitumen
 - b. Sand Bitumen
 - c. Water proof Mechanical Stabilisation.
 - d. Oiled Earth.
- a. It refers for Cohesive Soil in which main function is to preserve the Cohesive Strength of Soil by Water proofing the Soil.
- b. This term refers to the Bitumen Stabilised Cohesionless Soil such as loose, dunes, river Sand.
- Here, Primary function of bitumen as to bind the Soil Particles.

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c. In this, Small amount of bitumen 1–3% are Sometimes added to mechanically Stabilised Soil to make them water proof.

d. Slow & medium curing road oils are Spread on ground Surface to make it water and abrasion resistant.

The oil Penetrates into the Soil to the Short depth & also imparts Cementing action.

(v) Chemical Stabilisationa. Calcium Chloride

- It is Used as Water retentive additive in mechanical Stabilised bases & Surfacing.
- Being hygroscopic & deliquescent, this absorbs moisture from the atmosphere and retain it.
- It acts as a type of flock forming reagent and helps in Compaction of Soil.

b. Sodium Chloride

- The action of Sodium Chloride is Same as that of $CaCl_2$ but it is not widely Used.
- It attracts & retains moisture and reduces rate of evaporation.

C. Sodium Silicate

- It is Used in Combination of other Chemicals Such as CaCl_2 , NaCl_2 etc for Stabilisation of Soil.



(VI) Stabilisation by Heat

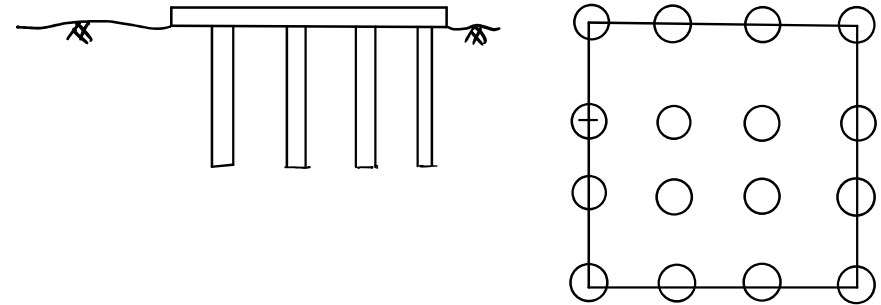
- Heating a fine grained Soil to a temperature of $400-600^\circ\text{C}$ Causes irreversible Changes in Clay mineral.
- The Soil becomes non plastic, less water Sensitive & non-expansive.
- This method is deployed in furnaces.

(VII) Electrical Stabilisation

- The Stability or Shear Strength of fine grained Soil Can be increased by draining them with passage of direct Current through them.
- This process also termed as Electroosmosis.

Q. A Square pile group of 16 piles penetrates through a filled up Soil 3 m depth. The pile $\phi = 250 \text{ mm}$ & Spacing = 0.75 m . The Unit Cohesion of the material is 18 kN/m^2 & the Unit Weight of Soil is 15 kN/m^3 . Compute the negative

Skin friction on the Group. Assume $\alpha = 0.4$.



$$\text{Sol: (i) } F_{ng} = n \cdot F_n$$

$$(ii) F_{ng} = \bar{c}_u \cdot P_g \cdot L_c + \gamma \cdot L_c \cdot A_g \quad \left. \vphantom{F_{ng}} \right\} \text{maximum}$$

$$F_{ng} = n \cdot F_n = 16 \times \alpha \cdot \bar{c}_u \cdot P_g \cdot L_c$$

$$= 16 \times 0.4 \times 18 \times \pi \times 0.25 \times 3$$

$$= 271.43 \text{ kN.}$$

$$F_{ng} = \bar{c}_u \cdot P_g \cdot L_c + \gamma \cdot L_c \cdot A_g$$

$$= 16 \times 2.5 \times 3 \times 4 + 15 \times 3 \times (3 \times 0.75 + 0.25)^2$$

$$= 821.25 \text{ kN.}$$

$$\therefore \text{ Adopt } F_{ng} = 821.25 \text{ kN.}$$

Q. A Single Under-reamed pile is installed in a Soft Clay deposit, the Centre of Under-ream is loca-

ted at a depth of 15m from the Ground Surface. The diameter of pile shaft and bulb is 1 m & 2.5 m respectively. Determine the allowable load with $FOS = 2.5$. The Undrained Shear Strength of the Soil obtaining from Vane Shear test is given by $C_u = 65 + 7D$.

$C_u = 10 \text{ kN/m}^2$ & D is Depth (m). Assume $\alpha = 1$ and average Cohesion of Soil around the bulb is $0.9C_u$.

Sol: $C_u = 65 + 7D = 65 + 7 \times 15 = 170 \text{ kN/m}^2$

$C' = 0.9 \cdot C_u = 0.9 \times 170 = 153 \text{ kN/m}^2$

$$Q_u = \frac{\pi}{4} \times 1^2 \times 9 \times 170 + \frac{\pi}{4} (2.5^2 - 1^2) \times 9 \times 153 + \frac{1 \times 65 + 170}{2} \times \pi \times 1 \times 15$$

$Q_u = 12416.55 \text{ kN}$

$Q_a = \frac{12416.55}{2.5} = 4966.62 \text{ kN}$

Note: If there is another bulb of same ϕ below the 1st bulb & Spacing between the Centre of bulb is 4 m. Compute the allowable load

$$Q_u = \frac{\pi}{4} \times 1^2 \times 9 \times 170 + \frac{\pi}{4}$$

$C_u = 65 + 7 \times 19 = 198 \text{ kN/m}^2$

$C_u' = 0.9 \times 198 = 178.2 \text{ kN/m}^2$

$C_a = \frac{65 + 65 + (7 \times 15)}{2} = 117.5 \text{ kN/m}^2$

$C_a' = \frac{(65 + 7 \times 15) + (65 + 7 \times 19)}{2} = 184 \text{ kN/m}^2$

or

$C_a' = 65 + 7 \times 17$
 $= 184 \text{ kN/m}^2$

$$Q_u = \frac{\pi}{4} \times 1^2 \times 9 \times 198 + \frac{\pi}{4} (2.5^2 - 1^2) \times 9 \times 178.2 + 1 \times 184 \times \pi \times 2.5 \times 4 + 1 \times 117.5 \times \pi \times 1 \times 15$$

$Q_u = 19330.18 \text{ kN}$

$Q_a = \frac{19330.18}{2.5} = 7732.07 \text{ kN}$

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SUB-SOIL EXPLORATION

- The Objective of Site exploration is to provide reliable, Specific & detailed information about the Soil & ground water Condition of the Site which may be required for a Safe and economic design of engineering Works.
- The exploration must be preceded by the Site reconnaissance.
- The information required is
 - The Order of Occurrence & extent of Soil & rock Strata.
 - The nature & engineering properties of Soil and rock formation.
 - Location of GWT and its Variation.
- Depth of Exploration
- Exploration in general should be carried out upto a depth at which increase in pressure due to Structural loading is likely to Cause Settlement or Shear failure.
- Such a depth is termed as Significant depth.
- It depends upon type of Structure weight, Size, Shape and deposition of loaded Area.

- It is Generally Safe to assume Significant depth as depth of 10% or 20% Overburden pressure Residual.
- The depth of exploration for different works are as follows -
 - i. Isolated Spread footing/raft : $1.5B$
 - ii. Adjacent footing with clear Spacing less than $2B : 1.5L$.
 - iii. Pile foundation : 10-30 m or more
or atleast 1.5 width of Structure
 - iv. Base of retaining wall : 1.5 the base width or 1.5 times exposed height of face, whichever is Greater.
 - v. Floating basement : Depth of Construction.
 - vi. Dams : 1.5 Bottom width of earth dam, 2 times height of bed to Crest from Concrete dams
 - vii. Road, Cut, fills : One meter where little Cut or fills is required, can be extended upto 2m for deep Cut.
 - viii. Borrow Area : Convenience of excavation or thickness available.
 - ix. From the Consideration : 1.5m in General.
of Weathering



• Method of Site exploration.

→ Site exploration can be done by any of the following methods:



- (i) Open excavation.
- (ii) Boring.
- (iii) Subsurface Sounding.
- (iv) Geophysical method.

i. Open Excavation

- Test pit and Trenches can be Used for all type of Soils.
- Soil Can be inspected in their natural Condition & Samples disturbed or Undisturbed Can be Conveniently taken.
- The Cost of Open excavation however increases with depth.
- They are generally Considered Suitable for Shallow depth upto 3 m.

ii. Boring

→ The method of Boring & drilling are as follows:

a. Auger Boring

- These are Used in Cohesive & other Soft Soils above watertable.

- Hand Augers are Used upto depth of 6 m & mechanical augers are Used for greater depth & they can also be Used in Gravelly Soil.
- Sample recovered from these Soil brought up by augers are badly disturbed & are useful for identification purpose Only.



b. Auger & Shell Boring

- Cylindrical Auger & Shell with cutting edge on teeth at the lower end can be Used for making the borings.
- Augers are Suitable for Soft to Stiff Clay, Shell for Very Stiff & hard clays & Shell pumps for Sandy Soils.

c. Wash Boring

- It is a fast & Simple method for advancing holes in all types of Soils.
- Boulders & rocks cannot be penetrated by this method.
- This method consists of first driving a Casing through a hollow drill, rod with a Sharp Chisel & water is forced Under pressure through the drilled rod.

- The Cuttings are forced up on the ground and are further tested.



d. Percussion Boring

- In this method, soil & rock formation are broken by repeated blows of heavy Chisels by Cable or drilled rod.
- Water is added to the hole during the boring if not already present to form the Slurry.
- It is Suitable for all type of Soil boulders and rocks.

e. Rotary Boring

- It is a Very fast method of advancing a hole in both rocks & Soils.
- A drill bit fixed to the lower end of drill rods is rotated by a Suitable mechanism.

• Soil Sample

- Soil Sample is generally Classified into 2 Categories.

I. Disturbed Sample.

II. Undisturbed Sample.

I. Disturbed Samples



- These are those where the Soil Structure gets modified or destroyed during the Sampling Operation.

- These are further Classified into following :

a. Representative Sample

- With Suitable precautions, the natural moisture Content & proportion of mineral Constituent can be preserved in the Sample termed as Representative Sample.

b. Non-Representative Sample

- If during the Sampling in the addition to the alteration in the Original Soil Structure, Soil from the other layer gets mixed up or mineral Constituents gets altered, the Sample is termed as Non-Representative Sample.

- Representative Samples are Useful for identification test but non-Representative Samples are of No Use.

II. Undisturbed Sample

- These are those where Original Soil Structure is preserved & the material properties have not been Altered.

→ These Samples are practically not possible to Obtain but Samples with minor alteration are Still Suitable for lab test.



→ Type of Soil Samples required for Lab test :

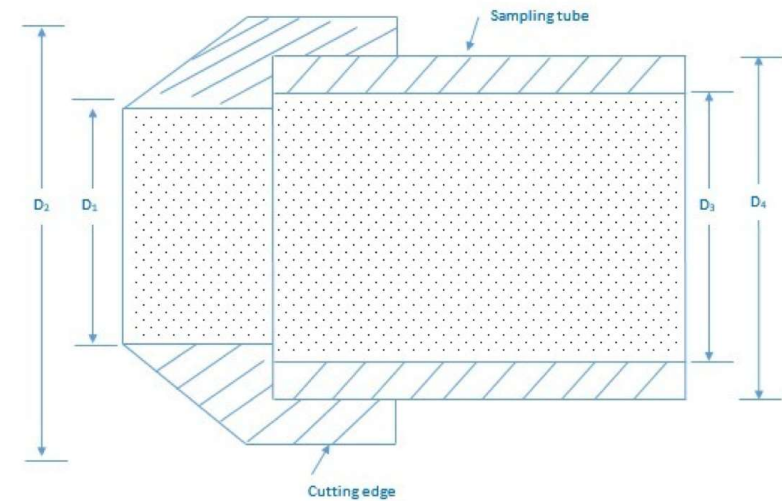
Type of Test	Type of Sample required
Natural Water Content	Undisturbed
Density	Undisturbed
Specific Gravity	Representative or Undisturbed
Grain Size distribution	Representative or Undisturbed
Atterberg Limit	Representative or Undisturbed
Coefficient of permeability	Undisturbed
Consolidation Parameters	Undisturbed
Shear Strength Parameter	Undisturbed

→ The extent of Disturbance of the Sample, due to Sampler depends Upon three features.

- Cutting Edge.
- Inside Wall friction.
- Non-Return Valve.



→ The following ratios related to the dimensions of the Cutting edge & the Sampler are Useful.



→ Inside Clearance (C_i)

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

$$C_i = 1-3\%$$

→ Outside Clearance (C_o)

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

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

$$C_o = 0-2\%$$



→ Area Ratio (A_r),

$$A_r = \frac{D_2^2 - D_1^2}{D_1^2} \times 100$$



Where,

D_1 = Inside diameter of Cutting edge.

D_2 = Outside diameter of Cutting edge.

D_3 = Inside diameter of Sampling tube.

D_4 = Outside diameter of Sampling tube.

→ The inside Clearance is meant to reduce the friction between the Soil Sample & the Sampler when the Soil enters into the tube by allowing for elastic expansion.

→ If the inside Clearance is too high, there will be too much of lateral expansion.

→ Outside Clearance will help reduce the friction while the Sampler is being driven & being withdrawn after the Sample is Collected.

→ $C_i = 1-3 \%$.

→ $C_o = 0-2 \%$. (It should not be much greater than C_i).

→ Area ratio should be kept as low as possible, consistent with the strength requirements of

The drive Shoe & the Sampling tube so as to reduce Sample disturbance.



→ $A_r \geq 20\%$, for Stiff formation.

→ $A_r \geq 10\%$, for Soft Sensitive Soil.

→ Another Parameter which is an index of Sample disturbance is recovery Ratio (L_r).

$$\text{Recovery Ratio} = \frac{\text{recovered length of Sample}}{(\text{L}_r) \quad \text{Penetration length of Sample}}$$

→ If $L_r = 1 \Rightarrow$ Good Recovery.

→ If $L_r < 1 \Rightarrow$ Compression while Sampling.

→ If $L_r > 1 \Rightarrow$ Expansion/Swelling while Sampling.

→ To reduce wall friction, the Sampling tube should have smooth finish & should be properly oiled before Use.

• Type of Sampler

→ The Commonly Used Sampler are as follows:

I. Open drive Sampler

→ This Sampler essentially consist of a Seamless Open end Steel tube with a Cutting edge.

→ The tube is Connected through a head to the Drill rod.

- The sampler head is provided with Vents to permit water & air to escape during the sampling & also a Check Valve to retain the sample.
- Thin wall Samplers are Used in this Case to Obtain Undisturbed Samples.

II. Piston Sampler

- A piston sampler consists of 2 separate parts
 - a. Sampler Cylinder.
 - b. Piston Cylinder.
- The Piston which is actuated separately and fits tightly in the sampler cylinder.
- It is Useful in sampling Saturated Sand & other Soft and wet Soil which cannot be sampled by Open drive sampler.

III. Rotatory Sampler

- It is a double walled tube sampler with an inner Removable Liner.
- The Outer tube or the rotating barrel is provided with a Cutting bit.
- The bit cuts an annular ring when the barrel is rotated.

- The inner tube which is stationary slides over the cylindrical sample cut by outer rotating barrel.

IV. Block/Chunk Sampler

- Block or Chunk can be obtained from open excavation like test pit, shaft etc.
- For chunk sampling, it is necessary that the soil has some cohesion.

iii. Sounding & Penetration test

- Subsoil exploration can be done by following Penetration test.
 - a. Standard Penetration test (SPT).
 - b. Static Cone Penetration test (CPT).
 - c. Dynamic Cone Penetration test.
 - d. Pressure meter test.
 - e. Plate load test.



C. Dynamic Cone Penetration test

- In this test, a cone which has an apex angle of 60° & attached to the drill rod is driven into the soil by blows of the hammer of 65 kg falling freely from the height of 750 mm.

- The blow Count for every 100 mm penetration is noted Continuously.
- The Cone is driven Until refusal or upto the required depth.
- The number of blows required for 30 mm penetration is noted as dynamic Cone resistance.
- This test gives a Continuous record of N_{cd} with depth.
- No Samples however can be obtained in this test.
- Correlation between dynamic Cone resistance & SPT Values can be referred as follows when 50 mm diameter Cone is Used,

$$\begin{aligned} N_{cd} &= 1.5 N && \text{for depth upto 3 m.} \\ N_{cd} &= 1.76 N && \text{for depth upto 3-6 m.} \\ N_{cd} &= 2 N && \text{for depth } > 6 \text{ m.} \end{aligned}$$

d. Pressuremeter test

- It is a form of load test & the load in this Case is being applied by a Uniform radial pressure to the Sides of the borehole in which a pressuremeter is placed.
- There are two basic types of pressuremeter.

- I. The Menard Pressuremeter (MPM), which is Lowered into a preformed Borehole.
 - II. The Self Boring pressuremeter (SBP), which forms its Own Borehole & thus causes much less disturbance to the Soil prior to the testing.
- In this test, elastic Constants of the Soil like Menard's modulus of elasticity & Menard's Shear Modulus is also found.

Note: Relationship between Menard's modulus of elasticity (E_m) & Young's Modulus of elasticity (E) is given by,

$$E = \frac{E_m}{\alpha}$$

Where, α = Rheological factor.

Note: In Clayey Soil, Undrained Shear Strength can also be determined as follows,

$$C_u = \frac{P_i}{9}$$

Where, P_i = Limit Pressure.

iv. Geophysical Method

- These methods were developed in connection with prospecting for Useful minerals & oils.
- The major methods of these Category are -
 - a. Gravitational method.
 - b. Magnetic method.
 - c. Seismic Refraction method.
 - d. Electrical Resistivity Method.



c. Seismic Refraction method

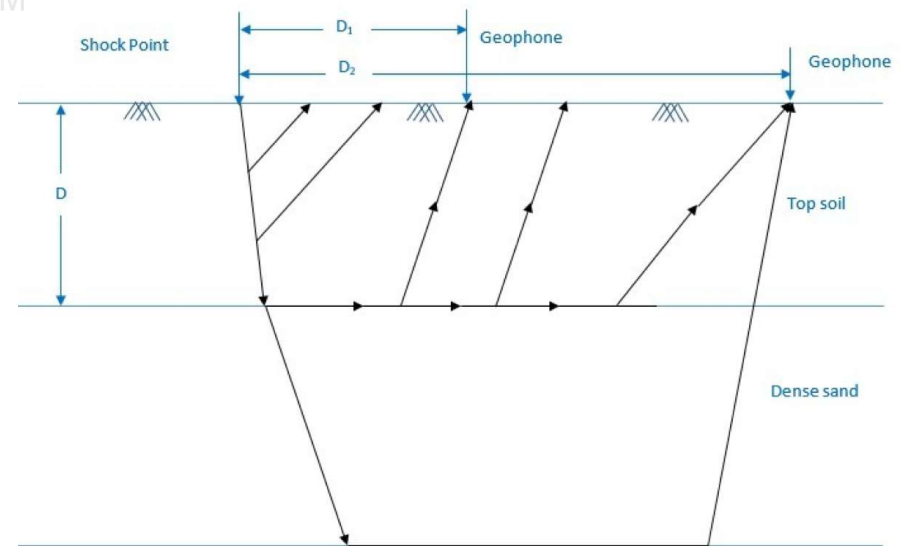
- In this method, Shock waves are Created into the Soil at their Ground level or certain depth below it by exploding Small Charges.
- These waves are then detected by Geophones Where the time of travel gets recorded.
- Either number of Geophones along a line are Used or Shock producing device is moved away from the Geophone to produce Shock waves in given interval.
- Some of the waves are directly reached to the geophones along the Ground Surface Whereas Some are first penetrated downwards into Underlying denser medium & then Reflected to the Geophones.

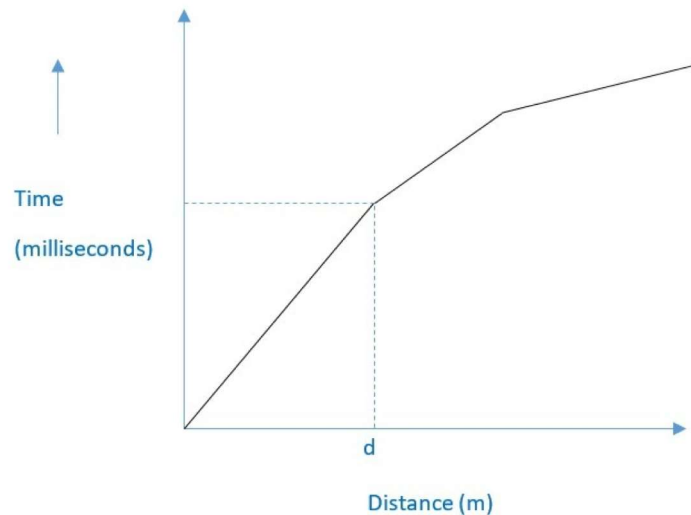
- Waves travel much faster in denser medium.
- As the distance between the Shock point & Geophones increases the refracted waves are able to reach earlier than direct wave to the Geophone.
- The depth of boundary between two Strata can be estimated as,

$$D = \frac{d}{2} \sqrt{\frac{V_2 - V_1}{V_2 + V_1}}$$



Where, V_2 = Velocity of Wave in Denser Medium.
 V_1 = Velocity of Wave in loose medium.





d. Electrical Resistivity method

- This method is based upon the measurement of changes in the mean resistivity or specific resistance of various soil.
- The resistivity is defined as the resistance between opposite faces of a unit cube of the material.
- Each soil has its own resistivity depending upon water content, compaction, and mineral structure.
- This test is conducted by driving 4 metal electrodes into the ground along a straight line at equal distance.
- A direct voltage is imposed between the two

outer electrodes & potential drop is measured between the inner electrodes.

→ The mean resistivity is given by,

$$\rho = 2\pi \cdot D \cdot \frac{E}{I} = 2\pi \cdot DR$$



Where, ρ = Mean resistivity

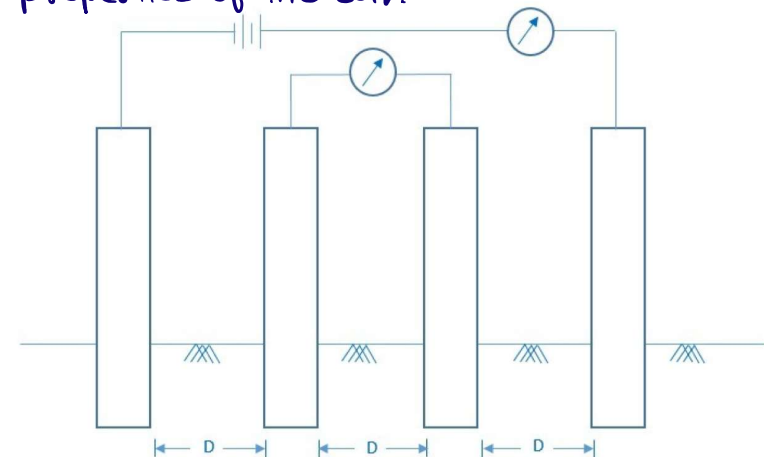
D = Distance between the electrode.

E = Potential drop between outer electrode

I = Current flowing between outer electrode

R = Resistance.

→ This Resistivity is further used to gauge the properties of the soil.



ESE

Q. The Skin friction resistance of a pile driven in Sand does not depend on

- (a) Lateral earth pressure Coefficient.
- (b) Angle of friction between pile and Soil.
- (c) Pile material.
- (d) Total Stress Analysis.

IES-2019

Ans: (d) Total Stress Analysis.

Sol:

Skin friction resistance for a driven pile in Sand.

$$Q_f = (K \cdot \bar{\sigma}_{av} \cdot \tan \delta) \times A_{\text{surface}}$$

Where,

K = lateral earth pressure Coefficient.

δ = angle of friction between pile and Soil.

K, δ depends upon the pile material.

Q. The Engineering News Record formula,

$$Q_a = \frac{W \cdot h}{6(S + 0.25)}$$

is used for the case of

- 1. Drop hammer.
- 2. Single acting hammer.
- 3. Double acting hammer.

Which of the above are correct?

- (a) 1 Only
- (b) 2 Only.
- (c) 3 Only.
- (d) 1, 2 and 3.

IES-2016

Ans: (b) 2 Only.

Sol: Empirical formula 'c' taken as 0.25 cm for Single-acting hammer.

Q. A drop hammer is Used to drive a wooden pile. The hammer weight is 25 kN and its free falling height is 0.8 m. The Penetration in the last blow is 12 mm. What is the nearest approximation to the load Carrying Capacity of the pile according to the Engineering News formula?

- (a) 125 kN
- (b) 110 kN
- (c) 3000 kN

(d) 90 kN

IES-2014

Ans: (d) 90 kN.

Sol: $W = 2 \text{ kN}$.

$h = 0.8 \text{ m} = 80 \text{ cm}$.

$S = 12 \text{ mm} = 1.2 \text{ cm}$.

As per engineering news formula,

$$Q_{\text{allowable}} = \frac{W \cdot h}{FOS(S+C)}$$

$C = 2.5 \text{ cm}$ for drop hammer.

$FOS = 6$

$$\therefore Q = \frac{25 \times 60}{6(1.2 + 2.5)} = 90.09 \text{ kN}.$$

Q. When the observed Value of N exceeds 15, the Corrected penetration number N_c as per Terzaghi and Peck recommendation in the Silty fine Sands will be

(a) $15 - \frac{1}{2}(N_R - 15)$

(b) $15 - \frac{1}{2}(N_R + 15)$

(c) $15 + \frac{1}{2}(N_R - 15)$

(d) $15 + \frac{1}{2}(N_R + 15)$

Where N = Penetration number, and N_R = Recorded Value.

IES-2020

Ans: (c) $15 + \frac{1}{2}(N_R - 15)$

Sol:

As per Terzaghi and Peck, 1948.

Corrected penetration number.

$$N_c = 15 + \frac{1}{2}(N_R - 15).$$

Q. The observed N Value from a Standard penetration test Conducted in Saturated Sandy Strata is 30; the N -Value Corrected for dilatancy may be taken as,

(a) 15

(b) 20

(c) 23

(d) 39

IES-2011

Ans: (C) 23

Sol: N Value corrected for dilatancy is N' ,

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

$$= 15 + \frac{(30-15)}{2} = 22.5$$

$$= 23$$

Q. Which technique of Stabilization for the Sub-base is preferred for a heavy plastic Soil?

- (a) Cement Stabilization
- (b) Mechanical Stabilization.
- (c) Lime Stabilization.
- (d) Bitumen Stabilization.

IES-2016

Ans: (C) Lime Stabilization.

Sol: Lime changes the nature of the adsorbed layer and provides pozzolanic action. Plasticity index of highly plastic soils are reduced by addition of lime with soil. There is an increase in optimum water content and a decrease in the maximum compacted density and the strength and durability of soil increases.

GATE

Q. A 0.5 X 0.5 m Square Concrete pile is to be driven in a homogeneous Clayey Soil having Undrained Shear Strength $C_u = 50$ kPa and Unit weight, $\gamma = 18.0$ kN/m³. The design Capacity of the pile is 500 kN. The adhesion factor α is given as 0.75. The length of the pile required for the above design load with a factor of Safety of 2.0 is

- (a) 5.2 m
- (b) 5.8 m
- (c) 11.8 m
- (d) 12.5 m.

GATE-2018, Shift-I

Ans: (C) 11.8 m.

Sol: Given, Size of pile = 0.5 m X 0.5 m.

Undrained Shear Strength, $C_u = 50$ kPa.

Unit weight, $\gamma = 18$ kN/m³.

adhesion factor, $\alpha = 0.75$.

Factor of Safety = 2.

Design capacity of pile = 500 kN.

$\therefore$ Ultimate Capacity of pile, $Q_u = 500 \times 2 = 1000 \text{ KN}$.

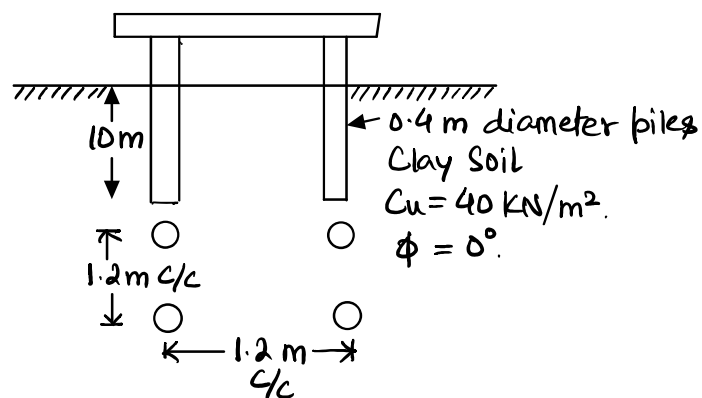
$$Q_u = C N_c A_p + \alpha \cdot C_u \cdot PL$$

$$1000 = 9 \times 50 \times 0.5 \times 0.5 + 0.75 \times 50 \times 4 \times 0.5 \times L$$

$$L = 11.83 \text{ m.}$$

$$= 11.8 \text{ m.}$$

Q. What is the Ultimate Capacity in KN of the pile group shown in the figure assuming the group to fail as a Single Block?

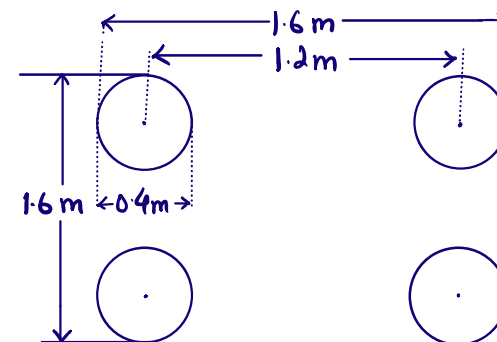


- (a) 921.6
- (b) 1177.6
- (c) 2438.6
- (d) 3481.6

GATE-2007.

Ans: (d) 3481.6

Sol: Pile group is failing as a Single Block.



Size of pile group 1.6 m x 1.6 m.

Ultimate Capacity of pile group = Bearing Capacity at Base $\times$ Base area + frictional resistance $\times$ Surface area.

$$= Q_u \cdot A_b + f_s \cdot A_s$$

$$= C N_c \cdot A_b + \alpha \cdot C_u \cdot A_s$$

$$= 40 \times 9 \times 1.6 \times 1.6 + 1 \times 40 \times 4 \times 1.6 \times 10$$

$$= 3481.6 \text{ KN.}$$

Q. A Core Cutter of 130 mm height has inner and outer diameters of 100 mm and 106 mm, respectively. The area ratio of the core cutter (in % upto two decimal places) is _____.

GATE-2018, Shift-I

Ans: 12.36

Sol: Given,

Inner diameter $\Rightarrow D_1 = 100 \text{ mm}$.

Outer diameter $\Rightarrow D_2 = 106 \text{ mm}$.

$\therefore$ Area ratio,

$$\begin{aligned} A_r &= \frac{D_2^2 - D_1^2}{D_1^2} \times 100 \\ &= \frac{106^2 - 100^2}{100^2} \times 100 \\ &= 12.36. \end{aligned}$$

SSC-JE

Q. A friction pile of diameter 50 cm is embedded 15 m into the homogeneous Consolidated Clay deposit. If the adhesion factor is 0.7 and adhesion develops between the Clay and pile Shaft is 4 t/m^2 . What is the Safe load (t) for factor of safety 3.0?

- (a) 15
- (b) 22
- (c) 26
- (d) 30

2017 - Evening Session.

Ans: (b) 22.

Sol: Given:

$$D = 50 \text{ cm} = 0.5 \text{ m}$$

$$L = 15 \text{ m.}$$

$$\alpha = 0.7.$$

$$C_u = 4 \text{ t/m}^2.$$

$$\text{FOS} = 3.$$

Ultimate Bearing Capacity for friction pile,

$$q_u = \alpha \cdot C_u \cdot \pi D \cdot L$$

$$q_u = 0.7 \times 4 \times \pi \times 0.5 \times 15 \\ = 65.97 \text{ t.}$$

Safe load,

$$q_{\text{safe}} = \frac{q_u}{\text{FOS}} \\ = \frac{65.97}{3}$$

$$= 21.99 \approx 22 \text{ t.}$$

Q. If the damping ratio is given by 0.10 and damping coefficient is 225 kN-s/m. The value of Critical damping ratio (kN-s/m)?

- (a) 22.5.
- (b) 225.
- (c) 2250.
- (d) 2500.

2017 - Morning Session

Ans: (c) 2250.

Sol: Damping ratio = 0.1
Damping coefficient,
 $C = 225 \text{ kN-s/m.}$

$$\text{Damping ratio} = \frac{C}{C_c} \rightarrow \text{Critical damping Coefficient}$$

$$\Rightarrow 0.1 = \frac{225}{C_c}$$

$$\Rightarrow C_c = \frac{225}{0.1}$$

$$= 2250.$$

Q. When Compared with Open Caissons, floating Caissons generally _____.

- (a) Have a higher load carrying Capacity.
- (b) Have a slow and inconvenient installations.
- (c) Are less expensive.
- (d) Have a greater depth below the ground surface.

2018 - Morning Session

Ans: (c) Are less expensive.

Q. The Lime Stabilization is very effective in treating

- (a) Sandy Soils.
- (b) Silty Soils.
- (c) Non-plastic Soils.
- (d) Plastic Clayey Soils.

2013.

Ans: (d) Plastic Clayey Soils.

Sol: Lime is used for the stabilisation of highly Plastic Soil like Black Cotton Soil.

- The fine clay particles react with lime and get flocculated or aggregated into larger particle groups which are fairly stable even under subsequent soaking.
- Due to this flocculation, the lime-treated clays indicate different grain size distribution.

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