Lecture (13)
17/11/19

Foundation

* Foundation is that part of structure load is finally trans-
- ferred to the soil / earth.

* The failure of foundation may take's place due to

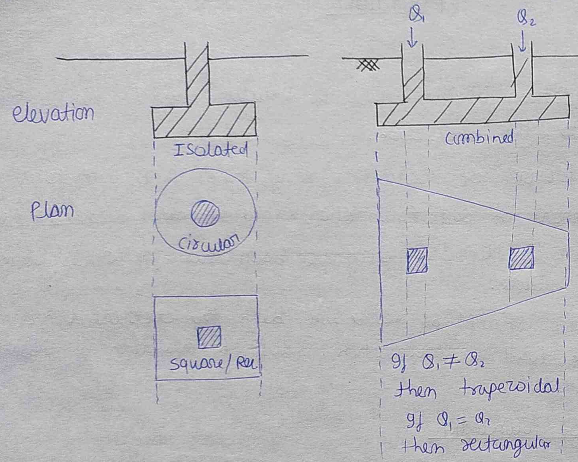
1) Settlement of soil / foundation which is called settlement failure
2) Slipping / sliding / of foundation / soil this failure is termed as shear failure.

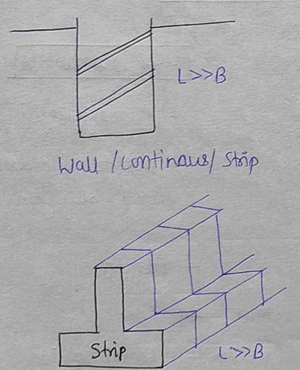
Foundation should be same in both the criteria Hence allowable load in foundation should be minimum of above two criteria -

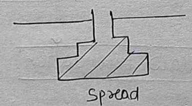
- Type of foundation

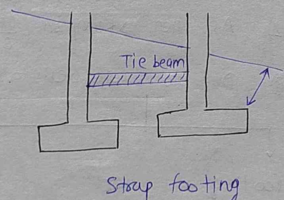
Foundation

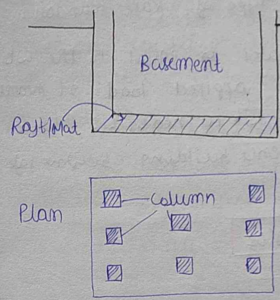
- Shallow foundation
  1) As per terzaghi $\left(\frac{D_f}{B} \leq 1\right)$
  2) As per Skempton $\left(\frac{D_f}{B} \leq 2.5\right)$
  3) Due to base resistance
     - footing
       - Isolated
         - Square
         - rectangular
         - circular
       - combined
       - Strip
       - Strap
       - Spread
     - Raft / Mat
- deep foundation
  1) As per terzaghi $\left(\frac{D_f}{B} > 1\right)$
  2) Due to side resistance and base resistance
     - Footing
     - Pile
     - Well / cassion

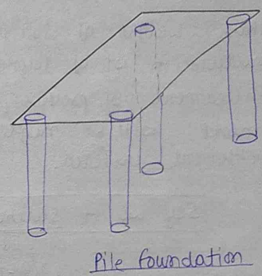












- Guidelines for Selection of foundation

Type of soil and loading $cond^d$	Suitable foundation
① Structural load is less soil is Medium to dense	• Shallow footing
② Structural load is heavy and soil is Medium to loose	• either Raft (or) deep footing
③ Structural load is very heavy and soil is Medium to loose	• Raft + Pile
④ Structural load is heavy and foundation is to be placed in Running water	• Well foundation
⑤ If free swell value is more than 50% and differential free swell value is more than 35% then shallow footing is not adequate.	• either Raft and deep footing
⑥ Footing area is more than 40% of Plinth area	• Generally raft (or) combine.
⑦ Soil is loose saturated sand which is Prone to liquifaction	• compaction Pile
⑧ If soil is expensive (high swelling and Shrinkage) eg. black cotton soil	• either floating / balancing Raft (or) underreamed Pile

(question आते हैं)

Note — Floating foundation is a type of Raft foundation in which weight of soil is excavated is equal to the wt. of structure + wt of foundation + applied load. It means net increament of pressure on the soil is negligible. Hence settlement will be negligible. Hence building become safe in settlement criteria.

$$\boxed{\text{Self wt} + \underbrace{\text{surcharge}}_{(D.L + L.L)} = \gamma D_f}$$

Shallow foundation

* Foundation should be safe as per shear criteria and settlement criteria both

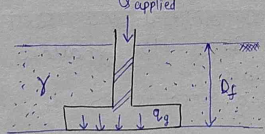
① Safe load as per shear criteria = Safe bearing capacity × Area

② Safe load as per settlement criteria = Safe bearing Pressure × Area

* For design min^m of the above two criteria should be adopted.

* Important definition as per shear criteria

① Gross pressure (q_g)



it is the total Pressure at the base of the foundation due to self wt of soil / Footing and applied load on the footing (or) the gross pressure may be greater, equal and smaller than bearing capacity.

For design gross pressure should be less than safe bearing capacity. In this analysis difference b/w unit wt of concrete and unit wt of soil is neglected.

② Net Pressure (q_n) → It is that Part of gross Pressure at the base of footing which is in axis to the initial effective overburden pressure.

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

$q_n = q_g - \gamma D_f$ (If G.W.T is not Present)

$q_n = q_g - \gamma' D_f$ (If G.W.T is present at G.L)

$q_n = \frac{Q\ \text{applied}}{\text{Area}}$

③ Ultimate bearing capacity (q_u)

It is the max^m gross pressure at the base of foundation which can be applied without shear failure (or) It is the min^m gross pressure at which soil is just fail in shear.

④ Net ultimate Bearing capacity (q_{nu})

It is the max^m net Pressure which can be applied at the base of foundation without shear failure (or) It is the min^m net pressure at which soil just fail in shear

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

If GWT is not Present ($q_{nu} = q_u - \gamma D_f$)

If GWT is present at G.L ($q_{nu} = q_u - \gamma' D_f$)

⑤ Net safe Bearing Capacity (q_{ns})

If is that net pressure which can be applied safely at the base of footing without risk of shear failure

$$\boxed{q_{ns} = \frac{q_{nu}}{F.O.S} = \frac{q_u - \bar{\sigma}}{F.O.S}}$$

Generally, F.O.S is 2.5 to 3

$\bar{\sigma} = \gamma D_f$ (or) Use $\gamma' D_f$ if W.T. is present

⑥ Safe bearing capacity q_s / q_{safe}

1) It is that gross pressure at the base of footing which can be applied safely without risk of shear failure

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

Generally, F.O.S is 2.5 to 3

$\bar{\sigma} = \gamma D_f$ (or) if G.W.T is present then $\bar{\sigma} = \gamma' D_f$

Note ① F.O.S is applied to (q_{nu}) which is increment in the pressure on the soil but not to the $\bar{\sigma}$ because it is acting from historical time of period.

Note ② Net safe load = $q_{ns} \times$ Area

Safe load = $q_s *$ Area

Settlement criteria

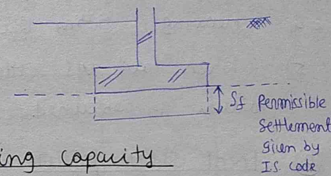
→ If footing should be design such that max^m settlement is restricted below the permissible values permitted by code agencies (BIS) it is already the safe settlement Hence further F.O.S is not required.

Net allowable bearing pressure / Net safe settlement pressure ($q_{u\,net}$)

① It is the net increase in pressure at the base of footing without risk of settlement pressure.

Allowable bearing Pressure

$= q_{net} + \bar{\sigma}$



Factor's Affecting the bearing capacity

① Positions of Water table (GWT rise, bearing capacity decreases)

② Type of soil and it's Physical and engineering Properties (γ, c and ϕ)

③ Type of footing (strip, square, circular, raft)

④ Size of footing (depth and bridth)

⑤ Type of loading (concentric and essentric)

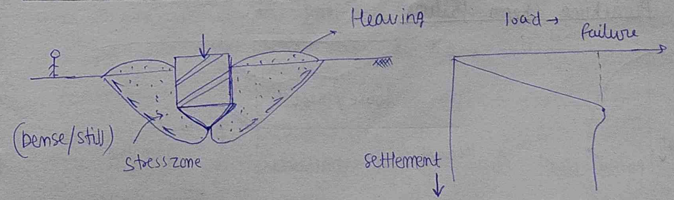
⑥ Initial stresses on the soil (N.C and O.C.)

⑦ Nature of ground surface (Horizontal (or) inclined)

⑧ Type of shear failure (General, local, (or) Punching

Type of shear failure

General shear failure



1) It occurs in shallow foundation which is placed on dense / stiff soil.

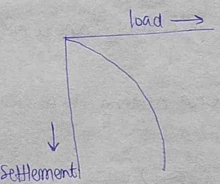
2) Before failure, settlement is small

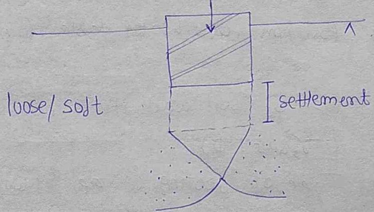
3) Stress zone extend upto G.L.

4) Heaving / tilting occurs in the sides after failure

5) When load v/s settlement curve is plotted, clear cut failure is obtained.

(B) Local shear failure





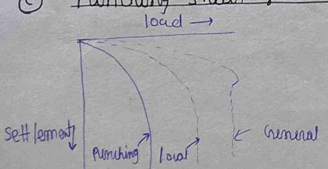
1) It occurs in shallow foundation which is placed on loose or soft soil.

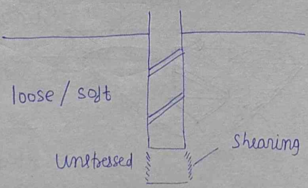
2) ~~Before failure~~ settlement is excessive

3) Stress zone does not occurs at the G.L hence little or no heaving occurs in the sides

4) When load v/s settlement curve is plotted progressive failure is obtained

(C) Punching shear failure





(1) It occurs in the deep foundation which is placed on loose / soft soil.

(2) Excessive settlement found in small time

(3) Adjacent soil mass remain unstressed

(4) When load vs settlement is plotted, progressive settlement failure is obtained.

Imp

SAND	General	Local
* Friction angle (ϕ)	$>36°$	$<28°$
* SPT No (N) (Standard Penetration test)	>30	<5
Density index (I_D)	$>70\%$	$<30\%$
Void ratio (e)	<0.55	>0.75

Method to determine bearing capacity

(1) Static / Analytical Method

Elastic theory	Earth Pressure theory	Plastic theory
Schleicher's theory	• Rankin's theory (ϕ)	• Fellinius th (c)
	• Bell's theory (c, ϕ)	• Prandtl's th (c, ϕ)
	• Packer's th (c, ϕ)	• Terzaghi th (c, ϕ) ★★★
		• Skempton's th (c) I.S.C
		• Meyerhof th (c, ϕ)
		• Vesic's th (c, ϕ)
		• I.S Code (c, ϕ)

WFF (Water → Footing → Failure)

* Terzaghi theory ⇒

→ It is an improvement over Prandtl's theory Prandtl's considered the base of footing to be smooth, whereas terzaghi considered the base to be rough.

Assumption

(1) Foundation is shallow ($\frac{D_f}{B} \leq 1$)

(2) Base of footing is rough

(3) Footing is continuous / strip footing ($L >> B$)

it make the analyses 2 Dimensionsional (along the width and depth only

(4) At the time of failure soil reaches into Plastic equilibrium

(5) load is vertical and concentric

(6) Ground surface is Horizontal

(7) W.T. is beyond the zone of influence of stressing it means effect of Water table is not considered

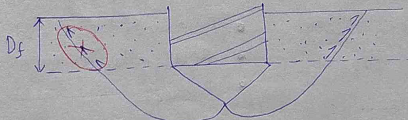
(8) Failure is under (General shear failure)

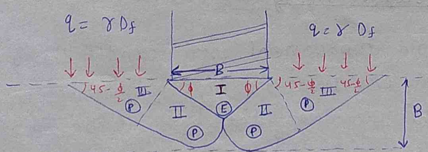
(9) The stress zone of the soil ~~stressed~~ extend upto foundation level only but not upto the ground level.

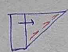
(10) The shear resistance of the soil above the ~~cut~~ foundation level is ignored, it mean only base resistance is considered Side resistance is ignored

Note It is the main reason due to which theory is not applicable for deep footing

(11) The soil above the footing level is removed and replaced by an equivalent surcharge of $q = \gamma d_f$







General Shear failure →

Zone 1 (center zone)

Soil get compacted below the footing and become the part of footing Hence it is an elastic equilibrium. it makes the angle (φ) with the footing.

Zone (2) Radial shear zone

It is circular in clayey soil and log spiral in sand and silt. It is in plastic equilibrium

Zone (3) linear shear zone (or) Rankin's Passive Zone

It makes an angle $\left(45 - \frac{\phi}{2}\right)$

Note (1) the stress zone on soil extend upto a Max^m depth of B below the foundation where 'B' is Bridth of footing

Terzaghi's eqn of ultimate bearing capacity for strip footing

family eqn

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

for above the soil → for below the soil

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

Where γ in 2nd term is for the soil above the base of footing and γ in 3rd term is for the soil below the base footing.

c → unit cohesion for the soil below the footing

N_c, N_q & N_γ are terzaghi's Bearing capacity which depends upon friction angle φ only

$$\left(K_p = \frac{1+\sin\phi}{1-\sin\phi}\right)$$

* $$a = e^{\left(\frac{3\pi}{4} - \frac{\phi}{2}\right)\tan\phi}$$

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

$$N_q = \frac{a^2}{2\cos^2\left(45 + \frac{\phi}{2}\right)}$$

$$N_c = \cot\phi\left[\frac{a^2}{2\cos^2\left(45+\frac{\phi}{2}\right)} - 1\right]$$

Special case — for pure cohesive soil / clay $\phi = 0$

$$\boxed{N_c = 5.7 \qquad N_q = 1 \qquad N_\gamma = 0} \ *$$

$$q_u = 5.7C + \gamma D_f + 0$$

$$q_{nu} = q_u - \bar{\sigma} = q_u - \gamma D_f = 5.7C$$

(1) Modifications ~~of~~ for different shape of footing (Shee safe factor)

For strip footing $q_u = CN_c + qN_q + 0.5B\gamma N_\gamma$ (dia)

For square footing $q_u = 1.3CN_c + qN_q + 0.4B\gamma N_\gamma$

For circular footing $q_u = 1.3CN_c + qN_q + 0.3B\gamma N_\gamma$ (B → dia)

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

Check (B=L) → square

(2) Modifications for the shear failure

1) In armonge eqn General shear failure is considered but soil may fail in local shear failure also Hence modified shear parameter should be used. Modified friction angle (ϕ_m) is used to determined (N_c, N_q and N_γ)

$$\boxed{C_m = \frac{2C}{3}} ** \qquad \tan\phi_m = \frac{2}{3}\tan\phi$$

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

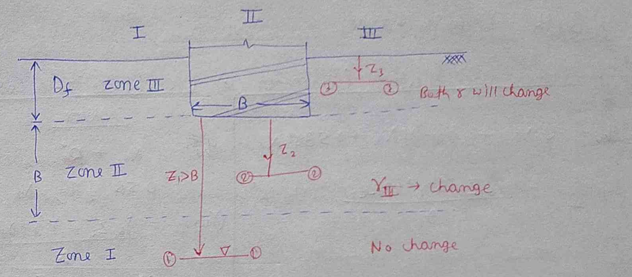
1st → local given

2nd → table and $\phi < 28°$ (question में A/C करें)

	N_c	N_q	N_γ
ϕ_1			
ϕ_2	✓	✓	✓
ϕ_3			

↓ local

(3) Effect of water table on bearing capacity



case I — When GWT is in Zone-I at depth Z_1 below the footing such that $Z_1 > B$

- Water table is beyond the stress zone hence no change on bearing capacity.

case II — When GWT is in Zon II, at depth Z_2 below footing level such that $0 \le Z_2 < B$

third γ will change, IInd γ will remain unaffected either

Use effective parameter	or Use water table correction factor
$q_u = C'N_c + \gamma D_f N_q + 0.5B\gamma_{eff} \cdot N_\gamma$	$q_u = C'N_c + \gamma D_f N_q + 0.5B(R_\gamma^* \cdot \gamma_{avg}) \cdot N_\gamma$
$(\gamma_{eff})_{III} = \frac{\gamma_b Z_2 + \gamma'(B - Z_2)}{B}$ $(\gamma)_{II} = \gamma$	$(\gamma_{avg})_{III} = \frac{\gamma_b Z_2 + \gamma_{sat}(B - Z_2)}{B}$ $R_\gamma^* = \frac{1}{2}\left(1 + \frac{Z_2}{B}\right)$
⇒ If $Z_2 = 0$ GWT is at footing level $(\gamma_{eff})_{III} = \gamma'$ $(\gamma)_{II} = \gamma$ $q_u = C'N_c + \gamma D_f N_q + 0.5B\gamma' N_\gamma$	⇒ If $Z_2 = 0$ $R_\gamma^* = \frac{1}{2}$ $(\gamma_{avg})_{III} = \gamma_{sat}$ $q_u = C'N_c + \gamma D_f N_q + 0.5B\left(\frac{1}{2}\gamma_{sat}\right)N_\gamma$ $\gamma' \approx \frac{1}{2}\gamma_{sat}$
⇒ If $Z_2 = B$, $(\gamma)_{III} = \gamma$ $(\gamma_{II}) = \gamma$ (No change)	⇒ $Z_2 = B$, $R_\gamma^* = 1$ $(\gamma_{avg}) = \gamma$ (No change)

case III — When GWT is in Zone III at depth Z_3 below ~~footing~~ ground level such that $0 \le Z_3 \le D_f$

Both γ will change

either use effective parameter (or) Use Water table correction factor

$q_u = c'N_c + \gamma_{eff} D_f N_q + 0.5B\gamma' N_\gamma$

$(\gamma_{eff})_{II} = \dfrac{\gamma_b Z_3 + \gamma'(D_f - Z_3)}{D_f}$ $(\gamma)_{III} = \gamma'$

$q_u = c'N_c + R_q^* \gamma_{avg} D_f N_q + 0.5B\left(\frac{1}{2}\gamma_{sat}\right) N_\gamma$

$(\gamma_{avg})_{II} = \dfrac{\gamma_b Z_3 + \gamma_{sat}(D_f - Z_3)}{D_f}$

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

If Z = 0 GWT is at ground level

$(\gamma_{eff})_{II} = \gamma'$ $(\gamma_{III}) = \gamma'$

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

if $Z_3 = 0$ $R_q^* = \frac{1}{2}$ $(\gamma_{avg})_{II} = \gamma_{sat}$

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

$\left(\gamma' \simeq \frac{1}{2}\gamma_{sat}\right)$

- **Special case - I for cohesionless soil**

$q_u = \cancel{cN_c}^{\,0} + \gamma D_f N_q + 0.5B\gamma N_\gamma$

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

If GWT rises to G.L

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

(or)

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

Note① In sandy soil, ultimate bearing capacity depend upon width of footing

Note② In sandy soil, ultimate bearing capacity approx. reduce to half when water table rises to ground level

- **Special case - II for pure cohesive soil / clay ($\phi = 0$)**

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

$q_u = 5.7c$, $N_q = 1$, $N_\gamma = 0$

$q_u = 5.7c + \gamma D_f = 0$

$q_{nu} = q_u - \bar{\sigma} = 5.7c$

If G.W.T rises to G.L

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

$q_{nu} = q_u - \bar{\sigma} = q_u - \gamma' D_f = 5.7c'$ $(c' \simeq c)$

Note③ In clayey soil ultimate bearing capacity is independent of width of footing.

Note-④ Net ultimate bearing capacity is nearly unaffected by the rise of water table

② **Skempton theory**

1) It is applicable for pure cohesive soil / clay only

2) In this theory side resistance and base resistance both are consider. Hence it is applicable for shallow and deep footing both.

3) Net ultimate bearing capacity for clay is

$\boxed{q_{nu} = cN_c}$

Where $N_c \rightarrow$ Skempton bearing capacity factor which depend on $\left(\frac{D_f}{B}\right)$ ratio

for ESE

for strip footing	$N_c = 5\left(1 + 0.2\frac{D_f}{B}\right)$	$5 \le N_c \le 7$
for square / circular	$N_c = 6\left(1 + 0.2\frac{D_f}{B}\right)$	$6 \le N_c \le 9$
for Rect / Raft	$N_c = 5\left(1 + 0.2\frac{D_f}{B}\right)\left(1 + 0.2\frac{B}{L}\right)$	$6 \le N_c \le 9$

③ **Meyerhof theory**

① It is most generalized theory in which safe factor, depth factor and inclination factor are used to account for shape of footing (strip, square, circular, rectangular, Raft)

Depth of footing (shallow or deep) and for inclination of load (vertical or inclined)

Note: In this theory the stress zone is considered to be extended upto ground level therefore this theory is applicable for both shallow and deep footing because side resistance is considered.

② Ultimate bearing capacity is given as-

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

$S_c, S_q, S_\gamma \rightarrow$ Shape factor

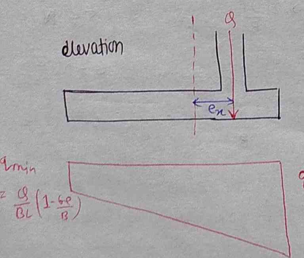
$d_c, d_q, d_\gamma \rightarrow$ depth factor

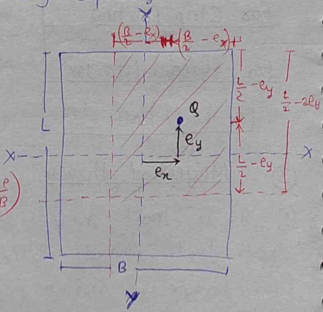
$i_c, i_q, i_\gamma \rightarrow$ inclination factor

$N_c, N_q, N_\gamma \rightarrow$ Meyerhof bearing capacity depends upon ϕ

for clay ($\phi = 0$), $N_c = 5.14$, $N_q = 1$, $N_\gamma = 0$

Note: If total load Q on the footing act eccentrically it means line of action of Q is not passing through the C.G. of footing area then width and length should be reduced such that load will act at the C.G. of reduced area to determine the safe bearing capacity.





$B' = B - 2e_x$

$L' = L - 2e_y$

$A' = B' * L'$

$$q_u = CN_c S_c d_c i_c + q N_q S_q d_q i_q + 0.5 B' \gamma N_\gamma S_\gamma d_\gamma i_\gamma$$

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

Net safe load $= q_{ns} * A'$

Safe load $= q_s * A'$

(Question नहीं आता)

④ **I.S. Code Method (IS: 6403 : 1981)**

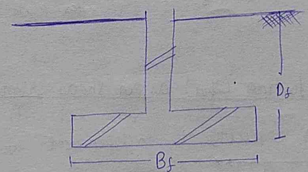
Net ultimate bearing capacity

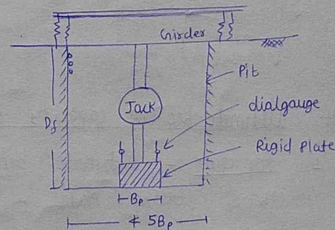
$$\boxed{q_{nu} = CN_c S_c d_c i_c + q(N_q - 1) S_q d_q i_q + 0.5 B \gamma N_\gamma S_\gamma d_\gamma i_\gamma R^*}$$

$R_\gamma^* = \frac{1}{2}\left(1 + \frac{Z_2}{B}\right)$ $Z_2 \rightarrow$ G.W.T from footing level

$0 \le Z_2 \le B$, $\frac{1}{2} \le R_\gamma^* \le 1$

Plate load test IS: 1888 : 1982





* A Pit of soil not less than 5 times the size of Rigid plate is excavated having depth equal to the depth of foundation.
* If water table is present at or above the test level then it must be lowered below the test level by Pumping

before conducting the test.

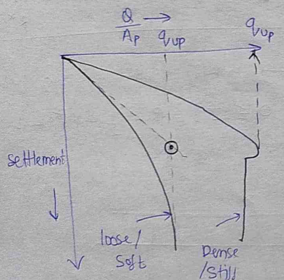
→ Rigid plate is placed at the center of pit which may have size of 30cm, 45cm, 60cm or 75cm

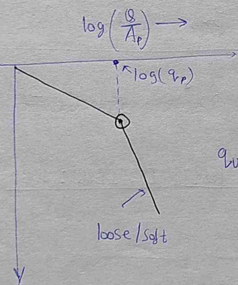
→ Initial a load of 7 KN/m² is applied and removed then after 3 dialgauges r attached to the rigid plate to measure the avg settlement

→ the load on the plate is applied through Jacking mechanism which can be recorded and load settlement curve is plotted by taking pressure $\frac{Q}{A_p}$ on X-axis and settlement on y-axis

→ the test is conducted upto the failure or atleast untill the settlement of about 25mm has occur

Load v/s settlement curve





q_{up} = ultimate bearing capacity for plate

Ultimate bearing capacity for footing (q_{uf}) as per shear criteria

1) For clay - ultimate bearing capacity is independent of width of footing / plate $\boxed{q_{uf} = q_{up}}$ **

2) For sand - ultimate bearing capacity is proportional to width of footing / plate

$$\boxed{q_{uf} = q_{up} \times \frac{B_f}{B_p}} \; **$$

B_f = width of footing

B_p = width of plate

→ using F.O.S for of 2.5 to 3 safe bearing capacity can be determined

Determination of allowable bearing pressure / safe settlement pressure as per settlement criteria

Let S_f = Permissible settlement of footing given by IS: code that using following empirical relation permissible settlement of plate (S_p) can be determined as

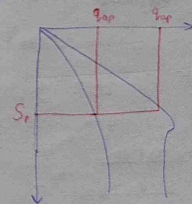
$$\boxed{\frac{S_f}{S_p} = \frac{B_f}{B_p}} \; ** \rightarrow \text{for clay}$$

$$\boxed{\frac{S_f}{S_p} = \left[\frac{B_f\,(B_p+0.3)}{B_p\,(B_f+0.3)}\right]^2} \; ** \rightarrow \text{for dense ~~soil~~ sand}$$

B_f and B_p is in meter

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

→ using above empirical relation permissible settlement of plate (S_p) can be determine and using load vs settlement curve allowable bearing pressure for plate (q_{ap}) can be determine.



Allowable Bearing pressure for footing (q_{af}) as per settlement criteria

① For clay $\boxed{q_{af} = q_{ap}}$ **

② For sand $\boxed{q_{af} = q_{ap} \times \frac{B_f}{B_p}}$ **

Note: above computed pressure is safe pressure in which further F.O.S is not required bcoz permissible settlement given by IS: code is already safe settlement using F.O.S

Housel approach to find ultimate bearing capacity of foundation

$$\boxed{Q = Am + Pn}$$

→ A/c to Housel the failure load / ultimate load is func of area and perimeter of footing / Plate and it is also influence by soil properties. In this Method (PLT) (Plate load test) is conducted on two rigid plates having area (A_1, A_2) and Perimeter (P_1, P_2). Corresponding ultimate load is (Q_1, Q_2) then

$$Q_1 = A_1 m + P_1 n \quad \text{--- (1)}$$
$$Q_2 = A_2 m + P_2 n \quad \text{--- (2)}$$

from (2) and (1) determine m and n

→ m and n are constant which depends upon type of soil

m = bearing pressure below the footing
n = Perimeter shear

* Ultimate load / failure load for footing

$$q_{uf} = \frac{Q_f}{A_f}$$ A_f and P_f → Area and Perimeter of footing

Note (1) 20% isobar occurs at 1.5B (where B → width of footing / Plate)

Limitations

(1) Size effect → Results are not realiable bcoz it represent the behaviour of soil upto lesser depth. While in actual stress isobar extend upto greater depth below the footing

(2) Shape effect → Result are not applicable for strip footing bcoz (L >> B)

(3) Time effect → It is a short duration test while in actual settlement takes time.

• Standard penetration test (IS: 2131: 1981)

(1) A split spoon sampler is placed over the soil in a bore hole of 55 mm to 150 mm dia

(2) Sampler is given by dynamic mechanism of hammer the wt of hammer is 65 kg and height of fall of hammer is 75 cm

Note the SPT No. is defined as no. of blows of hammer required for 300 mm penetration of sampler.

(3) Usually test is perform in three stage, 150 mm penetration each and SPT no. is taken as no. of blows required for last 300 mm penetration it means no. of blows required for 1st 150 mm penetration is ignored

(4) This test is repeated at every 2m - 5m interval (or) at the change of strata

(5) The observe value of SPT no. are subjected to following two correction applied in sequence

Overburden Pressure correction :-

The SPT No. value for the foundation should be corresponding to the avg SPT. Due to lesser overburden at shallow depth the SPT No. value gets underestimated and that at greater depth gets overestimated. Hence normalisation is required to the overburden

Let, N_0 = observed SPT No.
N_1 = corrected SPT No. for overburden

$$\boxed{N_1 = N_0 C_1 = N_0 \left(\frac{350}{\bar{\sigma}_0 + 70}\right)} \;\; ^{**}$$

$\bar{\sigma}_0$ = Eff. stress at test level (kN/m²)

If $\bar{\sigma}_0 > 250 \frac{kN}{m^2}$ then this correction is not required

(2)

② Water table / dilatancy / Fine correction →

① If W.T. is present at or above the test level then water table correction is required b/c due to sudden impact load excess pore pressure developed which ↑ the penetration resistance

Note: If WT is ~~above~~ below the test level then this correction is not required

① Let N_2 is the corrected value for the water table

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

if $N_1 < 15$ then the correction is not required

② If test is conducted at different levels say A, B, C and D then the final SPT no. is taken as the avg of corrected SPT No.

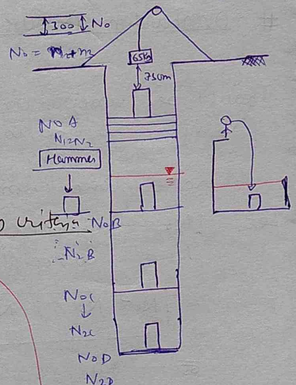
if ~~N_1~~ $N_{final} = \dfrac{N_{2A} + N_{2B} + N_{2C} + N_{2D}}{4}$ (standard penetration test)

③ Represented as a integers

$N_{final} \rightarrow I_D \rightarrow \phi$

$N_{final} \rightarrow I_C \rightarrow UCS \rightarrow c$

$\phi \rightarrow N_c, N_q, N_\gamma$



Bearing capacity as per shear criteria

नहीं आएगा

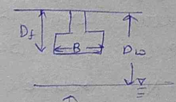
1) The final SPT No value is related to friction angle and relative density which is given in the table using friction angle bearing capacity factor (N_c, N_q, N_γ) can be determined Hence, ultimate bearing capacity can be determined by any static / analytical method

(यहाँ हैं ये)

Net Allowable bearing pressure as per settlement criteria

नहीं आएगा Gate में (conventional के लिए है)

A) Peck-Hansen Eqn



$$\boxed{q_{anet} = 0.41\, S N C_w} \quad \frac{kN}{m^2}$$

C_w = water table correction factor

S = Permissible settlement of footing given by IS code in mm

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

$D_w \rightarrow$ Depth of W.T. from G.L.

$(0 \le D_w \le D_f + B)$

$\frac{1}{2} \le C_w \le 1$

B) Teng's eqn (for conventional)

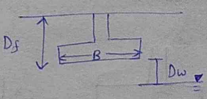
GATE (X)

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

$B \rightarrow$ width of footing in meter

$C_D \rightarrow$ Depth correction factor

$C_D = \left(1 + \frac{D_f}{B}\right) \; (\le 2)$



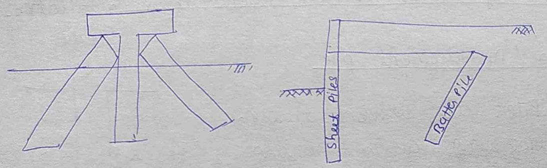
C_w = water table correction factor $= \frac{1}{2}\left(1 + \frac{D_w}{B}\right)$

D_w = Depth of W.T. from footing

$(0 \le D_w \le B)$, $\frac{1}{2} \le C_w \le 1$

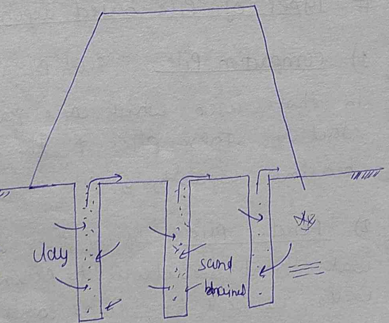
C) IS Code Method

$$q_{anet} = 1.38\left(\frac{B+0.3}{2B}\right)^2 (N-3)\, S\, C_w \quad \frac{kN}{m^2}$$



Types of piles based upon their Material

1) Wooden / timber pile
2) Steel Piple
3) cast- iron pile
4) Concrete pile
   a) simplex pile
   b) franki pile
   c) Vibro Floatation Pile
   d) Raymond Pile
   e) Padestal Pile
5) SAND piles



SAND DRAINS are Provided to ↑ Rate of consolidation Hence all the consideration settlement completed during the construction period only

Smear effect A smear zone is formed around the sand drain due to remoulding Permeability in Radial dirn decreases

∴ Flocculated structure convert dispersed structure due to remoulded

Ultimate load carrying capacity of piles (Q_{up})

It is the Maximum load which can be applied on the pile without shear failure. It is the sum of end bearing action and friction action

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

Safe load carrying capacity of Pile (Q_{safe})

It is the maximum safe load which can be applied on Pile without risk of shear failure

$$\boxed{Q_{safe} = \frac{Q_{up}}{F.O.S} = \frac{Q_{eb} + Q_{sf}}{F.O.S}}$$

generally F.O.S is 2.5 to 3

Methods to determine load carrying ~~bearing~~ capacity

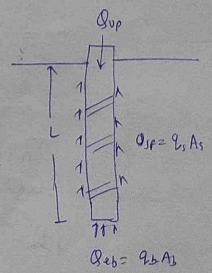
1) Static / Analytical method
2) Dynamic Method
3) Field Method
   A) Pile load test
   B) Cyclic Pile load test
   - It Gives end bearing and skin friction resistance separately
   C) SPT
   D) CPT

① Static / Analytical Mtd

	circular	square
A_b	$\frac{\pi}{4}D^2$	B^2
A_s	πDL	$4BL$

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

$$\boxed{Q_{up} = q_b A_b + q_s A_s}$$



Limitations

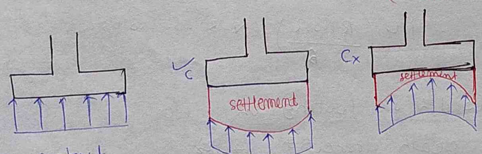
* this method is suitable for medium to dense sand Only bcoz in loose saturated sand liquifaction may occur and in clays remoulding may occur and excess pore water pressure may setup.
* Not reliable method for determination of C and ϕ due to empirical Relations

contact Pressure v/s elastic settlement

A) for Flexible footing

contact Pressure is uniform and settlement depends upon type of soil.

$(E_s)_{center} > (E_s)_{corner}$

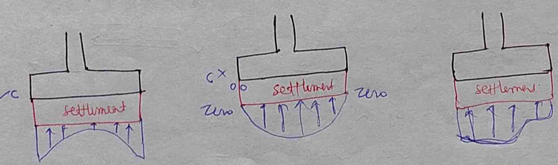


Flexible footing

contact Pressure is uniform

clay

sand



clay and sand at great depth

sand

silt

Rigid footing — settlement will be uniform and contact pressure depends upon type of soils

---

W.B. 13

(64)

$$C = \frac{UCS}{2} = 25$$

$$q_u = 1.3\,C\cancel{N_c}^{5.7} + \gamma D_f \cancel{N_q}^{1} + 0.4\gamma B\cancel{N_\gamma}^{0}$$

$$q_u = 1.3 \times 5.7 \times 25$$

$$q_u = 185.25 \text{ KN/m}^2$$

(14) $q_{nu} = C N_c$

At surface

$D_f = 0$

$$N_c = 6\left(1 + 0.2 \times \frac{\cancel{D_f}^{0}}{B}\right) = 6$$

$$q_{nu} = 25 \times 6 = 150$$

$$q_u = q_{nu} + \cancel{\gamma D_f}^{0} = 150 \text{ Ans}$$

(15) $q_u = 450$

$$q_{nu} = q_u - \bar{\sigma} = q_u - \gamma D_f = 450 - 20 \times (1) = 430 \text{ Ans}$$

(16) Raft 6x9

$$q_u = \left(1 + 0.3\frac{B}{L}\right) C\cancel{N_c}^{5.7} + \gamma D_f \cancel{N_q}^{1} + 0.5\left(1 - 0.2\frac{B}{L}\right) B\gamma \cancel{N_\gamma}^{0}$$

$$q_{nu} = q_u - \gamma D_f = \left(1 + 0.3 \times \frac{6}{9}\right) \times 120 \times 5.7 = 820 \text{ KN/m}^2$$

(a)

(17) PLT $\quad P = 0.6m$

$f = 3m$

SAND

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

$$S_f = \left(\frac{3(0.6 + 0.3)}{0.6(3 + 0.3)}\right)^2$$

$$S_f = 92mm$$

Q-36

~~0-75~~	6	→ Not considered
~~75-150~~	6	
150-225	12	
225-300	12	
300-375	16	
375-450	20	

$N_0 = 60$

$N_1 = N_0 C_1 = 60 \times 1.1 = 66$

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

$= 15 + \frac{1}{2}(66 - 15) = 40.5$

$= 40$ Ans

(47) square footing 2.5m × 2.5m

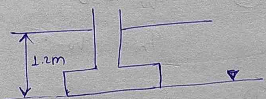
$D_f = 1.2m$

$\gamma_b = 18 KN/m^3$

$UCS = 55 KN/m^2$

safe bearing capacity F.O.S = 2.5

(i)



$\cancel{q_u = c'N_c + \gamma_{eff} D_f + 0.5 B\gamma' N_\gamma}$

$\cancel{\gamma_{eff} = 18 \times (1.2) +}$

$C = \frac{UCS}{2} = \frac{55}{2} =$

$q_u =$

$q_u = 1.3\, c\overset{5.7}{N_c} + \gamma D_f \overset{1}{N_q} + 0.4 B\gamma N_\gamma = 0$

$q_u = 1.3 \times (5.7) \times \frac{55}{2} + 18 \times 1.2 = 225.37 KN/m^2$

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

$q_s = \frac{q_u - \gamma D_f}{F} + \gamma D_f = \frac{225.37 - 18 \times 1.2}{2.5} + 18 \times 1.2$

$= 103.11 KN/m^2$

S Kempton

$q_{nu} = 6\left(1 + 0.2\frac{D_f}{B}\right) = 6\left(1 + 0.2 \times \frac{1.2}{2.5}\right) = 8.57$

$q_{nu} = CN_c = 27.5 \times 6.57 = 180.84 KN/m^2$

$q_s = q_{ns} + \bar{\sigma} = \frac{q_{nu}}{F} + \bar{\sigma} = \frac{180.84}{2.5} + 18 \times 1.2$

$= 93.93 KN/m^2$

Q (40)

$N_q = 33.3$

$N_\gamma = 37.16$

$F_{qs} = F_{\gamma s} = 1.314$

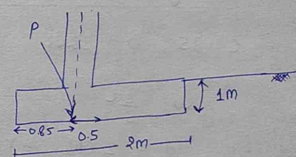
$F_{qd} = F_{\gamma d} = 1.113$

$\cancel{F_{qp}}$ 0.444

$F_{\gamma i}$ = 0.02

$F_{qi} = 0.444$

$\gamma = 18$ F.O.S = 3



Net safe load

$B' = B - 2e_x = 2 - 2 \times 0.15 = 2 \times 0.85 = 1.7m$

$L' = L = 2m$

$q_u = CN_c(s_c d_c i_c) + \gamma D_f N_q(s_q d_q i_q) + 0.5\gamma B' N_\gamma(s_\gamma d_\gamma i_\gamma)$

$q_u = 18 \times 1 (33.33)[1.314 \times 0.444 + 1.113] + 0.5 \times 1.7 \times 18 \times 37.16$

$= 16(1.314 \times 1.13 \times 0.02)$

$q_u = 405.84 KN/m^2$

$q_{no} = \frac{q_{nu}}{F} = \frac{q_u - \bar{\sigma}}{F} = \frac{405.84 - 18 \times 1}{3} = 129.3 KN/m^2$

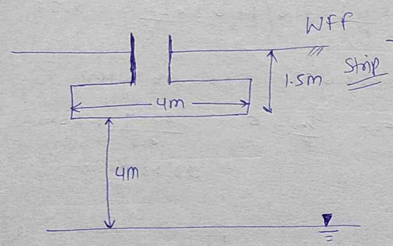
Net safe load $= q_{ns} \times B' \times L = 129.3 \times 1.7 \times 2$

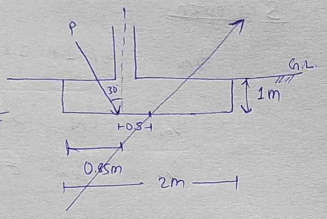
$= 439.5 KN$

(42) $C = 35, \quad \phi \; 28.63$

WFF

Strip





$C = 35 \;, \; \phi = 28.63$

$C_u = \frac{2}{3} C$

$q_u = \frac{2}{3} C N_c + \gamma D_f N_q + 0.5 B \gamma N_\gamma$

GWT is at depth B below footing then $\rightarrow$ No effect of GWT

$q_u = \frac{2}{3} * 35 * 17.7 + 17 \times 1.5 \times 74 + 0.5 \times 4 \times 17 \times 5$

$= 77.1 \; KN/m^2$

$q_{ns} = \frac{q_{nu}}{F} = \frac{q_u - \bar{\sigma}}{F} = \cancel{\frac{q_u}{F} - \bar{\sigma}} \; \cancel{77.1} \; \frac{771.61 - 17 \times 1.5}{2.5}$

$= 298.44 \, KN/m^2$

$q_{nu} = C N_c$

$N_c = 6\left(1 + 0.2 \frac{D_f}{B}\right) = 6\left(1 + 0.2 \times \frac{1.2}{2.5}\right) = 6.57$

$q_{nu} = C N_c = 275 \times 6.57 = 180.84 \; KN/m^2$

$q_s = q_{ns} + \bar{\sigma} = \frac{q_{nu}}{F} + \bar{\sigma} = \frac{180.84}{2.5} + 18 \times 1.2$

$= 93.93 \; KN/m^2$

(45) circular footing

$D_f = 1.3m$

Safe load = 800 KN at the base of footing

Area = ? $\quad$ F.O.S = 3 , $e = 0.55$

degree of saturation = 0.5

$\phi = 30°$ $\quad$ $G = 2.65$, $C = 8 \; KN/m^2$

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

$\cancel{q_u = 1.3 N_c \times C + q N_c + 0.3 B \gamma N_\gamma}$

$\cancel{= 1.3 \times 37.2 \times 8 + q \times (37.2) + 0.3 \times 18.49 \times (19.7)}$

$q_s = \frac{\text{Safe load}}{\text{Area}} = \frac{800KN}{\frac{\pi}{4} \times d^2}$

$q_s = \left(\frac{q_u - \bar{\sigma}}{f} + \bar{\sigma}\right)$

$q_s = \left\{\frac{1.3 C N_c + \gamma D_f N_q + 0.3 B \gamma N_\gamma - \gamma D_f}{f} + \gamma D_f\right\}$

$\frac{800}{\frac{\pi}{4} d^2} = \frac{(1.3 \times 8 \times 37.2 + 18.51 \times 1.3 \times 22.5 + 0.3 \times D \times 18.51 \times 19.7) - (18.51 \times 1.3)}{3}$

$\boxed{D = 1.62 m}$ Ans

$\gamma_d = \frac{\gamma_b}{1+w}$

$\gamma_d = \frac{G \gamma_w}{1+e}$

$\gamma_d = \frac{2.65 \times 9.81}{1 + 0.55} = 16.77$

$Se = wG$

$0.55 \times 0.5 = w \times 2.65$

$w = \frac{0.55 \times 0.5}{2.65}$

$w = 0.103$

$16.77 = \frac{\gamma_b}{1 + 0.103}$

16.77×1.103

$(18.49 \; KN/m^2 = \gamma_b)$

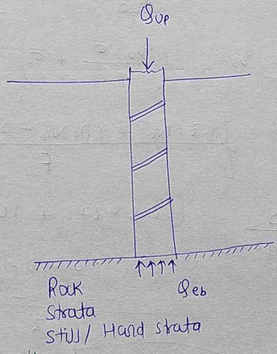
CHAPTE- 12

PILE'S FOUNDATION

Types of pile's on the basis of their Action

① End bearing pipe

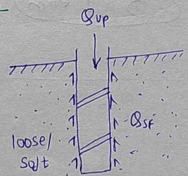
$\boxed{Q_{up} = Q_{eb}}$ **



- Resistance is due to end bearing (or) Point resistance
- Such piles are rest over Still / Hand strata
- Depth of such Piles will be same as the depth of hand strata

② Friction / Floating / Hanging piles

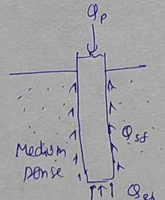
$\boxed{Q_{up} = Q_{SF}}$ **



1) such Piles are Provided in Soft / loose soil Mass which is extended upto great depth
2) Resistance is due to skin friction action
3) length may be 10cm to 20cm.

③ End bearing and friction Pile

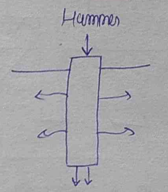
$\boxed{Q_{up} = Q_{eb} + Q_{sf}}$ ***



1) Resistance is due to end bearing and Skin friction both.
2) Such Pile are Provided in medium /dense soil

Types of piles based upon their Installation

① Driven / Displacement pile



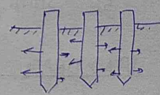
1) Such Piles are driven through the hammering action. During installation of pipe soil get displaced Hence also called displacement Pile
2) End bearing and skin friction properly develped in these Piles
3) Such piles are essentially precast pile's made of wood/ Metel

② Bored Pile / Non-displacement Pile

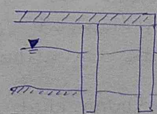
1) Such Piles may be Pre-cast (or) cast in-situ efficiency of such pile will be less than driven Pile

Types of Pile based upon their function

1) Compaction Pile Such Piles are Provided in those area which are proven to lique-faction these piles ↑ the density and bearing capacity of pile



2) Fender Pile Such Piles r Provided in water front structure to Prevent tidal waves caused by ships and seismic activities



3) Batter Pile Such Pile are Provider in inclined dirn to Prevent Horizontal thrust or inclined forces
4) Sheet Pile Such Piles r Provided to retain earth mass (OR) to Prevent piping below the dams

⇒ **for clay**

q_b = unit end bearing Resistance ≈ ultimate bearing capacity at base

q_s = unit skin friction Resistance ≃ unit adhesion

As per Meyerhoff

$\boxed{q_b = cN_c = 9c}$

$c \rightarrow$ unit cohesion at the base of pile

$q_s = \alpha \bar{c}$

$\bar{c}$ = avg cohesion

α = Adhesion factor / shear mobilization factor

α = 0.6 to 0.9 for soft clay

α = 0.4 to 0.6 for medium clay

α = 0.2 to 0.4 for stiff clay

Pile load capacity in clay

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

$$= q_b A_b + q_s A_s$$

$$\boxed{Q_{up} = 9cA_b + \alpha \bar{c} A_s}$$ ★★★

(α - eqn)

Note: For very long pile ($L \geq 25m$) the above method for estimating the skin friction is very conservative. For such pile's unit skin friction also depends upon the effective overburden pressure. The avg unit friction can be expressed at $q_s = \lambda(\bar{\sigma}_v + 2\bar{c})$

$$\boxed{Q_{up} = 9cA_b + \lambda(\bar{\sigma}_v + 2\bar{c})A_s} \longrightarrow \lambda\text{-method}$$

λ = friction capacity factor

$\bar{\sigma}_v$ = Avg. eff. stress along the embedded length of pile.

For Sand

q_b = unit end bearing Resistance = ultimate bearing capacity of base

$\boxed{q_b = \gamma L N_q}$

$\gamma \rightarrow$ unit wt along the length of pile

As per meyerhoff

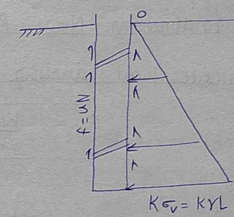
$$q_u = \cancel{cN_c}^{\,0} + \gamma D_f N_q + 0.5B\gamma N_\gamma$$

$$\gamma D_f N_q \ggg 0.5B\gamma N_\gamma$$

$$q_u = \gamma D_f N_q = \gamma L N_q$$

q_s = skin unit friction Resistance

$q_s = \mu N = \tan\delta$ (avg earth pressure)



$$q_s = \tan\delta \left(\frac{0 + K\bar{\sigma}_v}{2}\right)$$

$$= \tan\delta \left(\frac{0 + K\gamma L}{2}\right)$$

$$\boxed{q_s = \frac{1}{2} K\gamma L \tan\delta}$$

$K \rightarrow$ earth pressure coefficient

$\delta \rightarrow$ friction angle b/w pile and soil

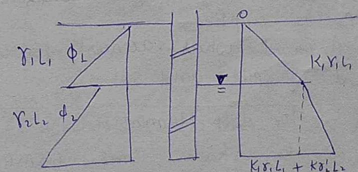
Pile load capacity in SAND

$$Q_{up} = q_b A_b + q_s A_s$$

★★★★

$$\boxed{Q_{up} = \gamma L N_q (A_b) + \frac{1}{2} K\gamma L \tan\delta (A_s)} \quad \delta\text{-eqn}$$

Note ①



$\triangle_{avg} \tan\delta\, A_{s_1}$

⏢$_{avg} \tan\delta\, A_{s_2}$

Note ② As per IS 2911 $\delta = \phi$

(पानी का effect neglect करेंगे क्योंकि पानी की वजह से friction neglected है।

② Dynamic method

This analysis is based on the Assumption that the dynamic resistance to dry the pile is equal to the ultimate load capacity using static method

1) In this Method K.E / Potential energy of hammer is equated to the Work done by the pile when it penetrates through the soil + loss

A) Engineering News Record formula (ENR formula)

Ultimate load carrying capacity

$$\boxed{Q_{up} = \frac{WH}{S+C}}$$ *** → { For drop hammer and { for single acting steam Hammer (SASH)

$$Q_{up} = \frac{(W+aP)H}{(S+C)}$$ → { for double acting steam Hammer (DASH)

Safe load carrying capacity

$$\boxed{Q_{safe} = \frac{Q_{up}}{F} = \frac{Q_{up}}{6}}$$ $\boxed{FOS = 6}$ *

W → weight of Hammer (KN)

H → Height of free fall of Hammer (cm)

S → settlement of pile per blow of Hammer (cm)

a → Area of Hammer / Piston over which steam Pressure 'P' is applied

C → Elastic constant which accounts for elastic settlement of pile and soil.

*
C = 2.5 cm	for drop Hammer
C = 0.25 cm	for Steam Hammer (SASH and DASH)

✓ $(Q_{safe})_{drop} = \frac{1}{6}\left(\frac{WH}{S + 2.5\,cm}\right)$ (H → cm, S → cm) ✓ $(Q_{safe})_{SASH} = \frac{1}{6}\frac{(WH)}{(S + 0.25\,cm)}$

B) Hiley's formula

Ultimate load carrying capacity

$$\boxed{Q_{up} = \eta_h \eta_b \frac{WH}{S + \frac{C}{2}}}$$

Safe load carrying capacity

$$\boxed{Q_{safe} = \frac{Q_{up}}{F} = \frac{Q_{up}}{3}}$$

$\boxed{F.O.S = 3}$

η_h = Efficiency of Hammer

η_h = 1 for drop Hammer (एक ही बार आया है)

η_h = 0.75 − 0.85 for SASH

η_h = 0.7 − 0.8 for DASH

⇒ η_b = Efficiency of Hammer blow

$$\cancel{\eta_b = \frac{W + e^2 P}{W + P}}$$

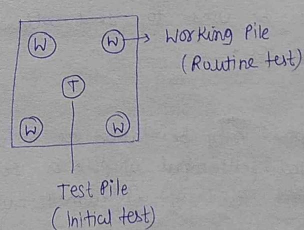
$$\cancel{\eta_b = \frac{W + e^2 P}{W + P} - \left(\frac{W - eP}{W + P}\right)^2, \quad W \le eP}$$

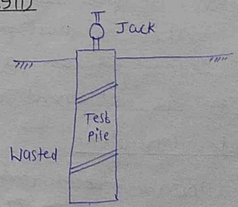
W → wt of hammer

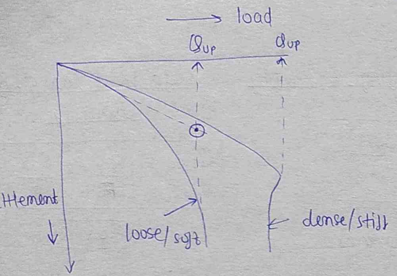
P ⇒ wt of pile

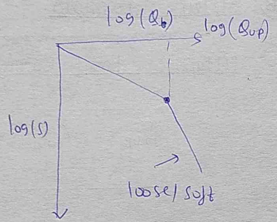
e ⇒ coeff of restitution

A) PILE LOAD TEST (IS 2911)









→ The load on the test pile is applied through Jacking mechanism and settlement of pile is recorded

→ Load settlement curve is plotted in which shear failure is represented by either progressive failure or by sudden settlement at faster rate and corresponding ultimate load carrying capacity is determined.

→ In this method loaded pile becomes waste it is a destructive test

Note - Routine test is conducted in working pile in which 1.5 times of allowable load is applied and settlement is recorded

→ This method is most accurate and recommended by IS code. It can also be used to find allowable load as per settlement criteria

IS code Guideline

The allowable load on pile may be taken as 50% of

① The allowable load at which total settlement of pile is 10% of it's dia in normal pile and 7.5% of under-reamed bulb dia in under-reamed pile

② Allowable load on pile may be taken as $\frac{2}{3}$ of ultimate load at which total settlement is 12mm

~~③ Allowable load in pile may be taken as $\frac{2}{3}$ of the ultimate load at which net plastic settlement is 6mm~~

(Remove from I.S. code)

(c) SPT (Standard Penetration test)

let $N \rightarrow$ SPT No. at the base of pile

$\bar{N} \rightarrow$ avg. SPT No. along the length of pile

(नहीं आता question civil service में कम आता है)

As per Meyerhof

For driven / displacement pile

$Q_{up} = q_b A_b + q_s A_s$

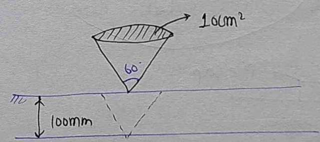
$Q_{up} = 400 N A_b + 2\bar{N} \cdot A_s$ (KN)

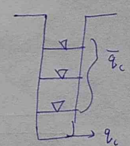
For Bored pile

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

$(Q_{up})_{Bored} = \frac{1}{3}(400 N A_b + 2\bar{N} A_s)$

Dutch/ static cone Penetration test





$q_c =$ Kg/cm²

→ the vertex angle of cone is 60°

The cone is penetrated into the soil for 100mm and when penetration is 100 mm the cone penetration of the soil is determine in kg/cm² which is called static cone resistance q_c

let q_c is cone penetration resistance at the base of pile

$\bar{q}_c$ is avg penetration resistance along the length of pile

(नहीं आता है)

for driven / displacement Pile

$Q_{up} = q_b A_b + q_s A_s$

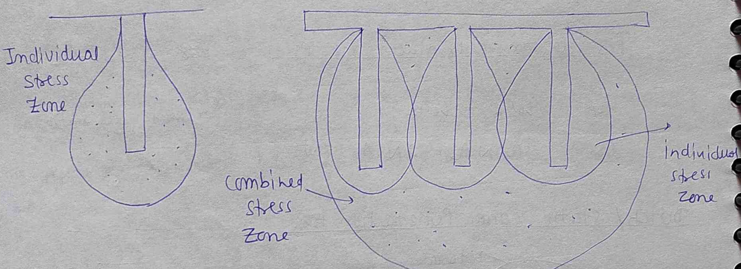
$Q_{up} = q_c A_b + \frac{\bar{q}_c}{2} A_s$ (KN)

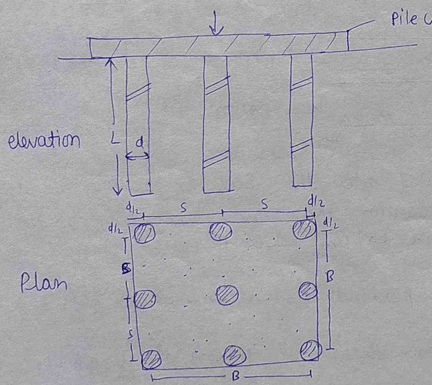
For Bored pile

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

$(Q_{up})_{Bored} = \frac{1}{3}\left(q_c A_b + \frac{\bar{q}_c}{2} A_s\right)$

GROUP ACTION OF PILE :-





2x2 $B = S + d$

3x3 $B = 2S + d$

4x4 $B = 3S + d$

mxm $B = (m-1)S + d$

$A_b = B^2$

$A_s = 4BL$

* If applied load is large and more more no. of Piles are used then either Piles will acts individually (or) in the group depending upon the spacing in b/w the Pile

* If center to center spacing is 2.5d – 4d then soil get compacted b/w piles and entire wedges of size (B×B) may acts as a single Pile then such action is called Group action of piles

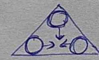
* In group action end bearing resistance and skin friction resistance both will ↑ses

* In group action depth of stress zone extend upto greater depth then in individual action therefore settlement due to consolidation in group action will always be greater than settlement in individual action.

* Minimum no. of Piles require for group action is three

* For group action center to center spacing should be

for end bearing action 2.5d-3.5d



for friction action 3d to 4d

* If Pile are to be driven then Pile driven mechanism should start from center and Proceed radially outward in this Process resistance in pile driven is less Hence cheaper.

1) **Load carrying capacity of Pile group (Q_{ug})**

① For clay $Q_{ug} = q_b A_b + q_s A_s$

$\boxed{Q_{ug} = 9 \cdot C \cdot B^2 + \bar{c}\,(4B \cdot L)}$

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

$\alpha = 1$ होगा ($\alpha = 1$) ∵ cohesion in b/w soil and soil

$\delta = \phi$ ∵ friction in b/w soil and soil

(2) Safe load carrying capacity of pile group

$$Q_{safe} \rightarrow \left[\begin{matrix} \frac{Q_{ug}}{F} \\ \text{or} \\ \frac{n Q_{up}}{F} \end{matrix} \right\} \text{min}^m$$

n = No. of Piles in group

Q_{up} = Individual capacity of Pile

F.O.S = 2.5 to 3

3) Group efficiency (η_g)

It is the ratio of load carrying capacity of pile group to the sum of individual Pile load capacity

$$\boxed{\eta_g = \frac{Q_{ug}}{n\, Q_{up}}}$$

If $\eta_g \geq 1$ then

$$Q_{safe} = \frac{n\, Q_{up}}{F}$$

4) Converse - labarre formula of Group efficiency

For ESE के लिए or conventional

radian मे होगा

$$\boxed{\eta_g = \left[1 - \frac{\phi^\circ}{90} \left\{ \frac{m(n-1) + n(m-1)}{m \cdot n} \right\} \right]}$$

m → No. of Rows

n → No. of columns

ϕ° → Arc tan value $= \tan^{-1}\left(\frac{d}{s}\right)$

(5) Guidelines for design of Pile group (conventional मे आया है)

① Length → for friction Pile group length should be 10 m - 20 m whereas for end bearing pile length will be equal to the depth of hard strata.

② Diameter → 0.45 m to 1 m

③ Spacing → 2.5d to 4d because generally 3d is Provided

④ No. of piles → Prefer square ~~by~~ group such as 3/3 4/4 5/5 etc.

(5) Group efficiency

It should be greater than equal to 1

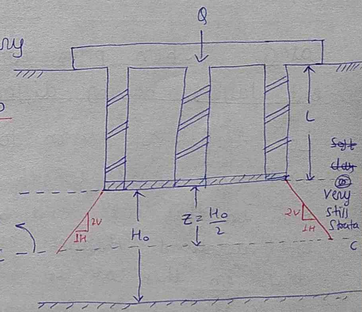
(6) Group Settlement Ratio

It is the ratio of settlement of pile group to the settlement of individual pile it is always greater than 1 and May be as high as 16.

$$\boxed{G.S.R = \frac{S_g}{S_i}}$$

* determination of settlement of pile group in clays

case ① When pile group is end bearing pile group



Step① Assume an imaginary equivalent Raft of B * B at the base of Pile group

Step② H_0

Step③ $\bar{\sigma}_0$ at c/c of compressible layer

Step④ load distribution in 2V:1H ($n = 0.5$)

$$\Delta\bar{\sigma} = \frac{\text{force}}{\text{Area}} = \frac{Q}{(B + 2nZ)^2} = \frac{Q}{(B+Z)^2}$$

Step⑤ ultimate settlement

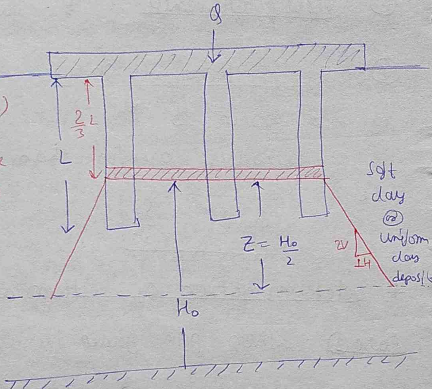
$$S_g = \Delta H = \frac{H_0 C_c}{1+e_0} \log\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

$$\left[G.S.R = \frac{S_g}{S_i} \right]$$

Imp ****

case② When Piles are driven through uniform clay deposit and Pile group acts as a friction Pile group o Assume an imaginary equivalent raft at $\frac{2}{3}L$ from the GL)

Remaining steps are same as that of previous case)

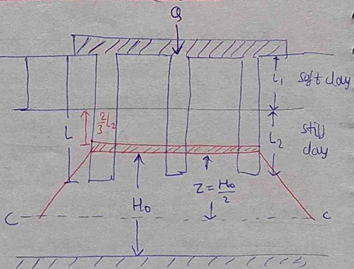


Case ③ When piles are driven through two different layer's having different density

(कुछ खास important नहीं है)

Let length of pile in top soft clay layer is L_1 and embedded length is bottom stiff clay layer is L_2 then in this case end bearing and skin friction both will developed through stiff clay. Hence the equivalent raft in this case is assumes at $\frac{2}{3}L_2$ below the top layer rest of the procedure is similar to (case) ①

G.S.R in sand



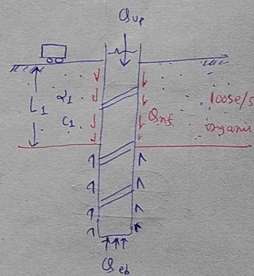
G.S.R in sand

$$\boxed{G.S.R. = \left(\frac{4B + 2.7}{B + 3.6}\right)^2}$$

$B \uparrow \rightarrow G.S.R \uparrow$

Negative skin friction

It is the phenomen in which soil surrounding to the pile settle's more than the settlement of pile's under such con[dition] friction on the pile act's downward which reduces the load carrying capacity of the pile. This cond[ition] occur's when soil surrounding to the pile is loose / soft / organic / fill



→ Following cond[itions] may cause Negative skin friction

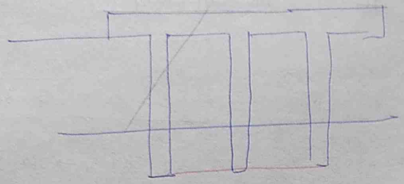
1) Increase in surcharge over the surrounding soil
2) Lowering of ground water table
3) Disturbance due to seismic activity and other dynamic loading.

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

Q_{nf} = Negative skin friction

- For clay $Q_{nf} = \alpha_1 \bar{C}_1 (\pi d L_1)$
- For sand $Q_{nf} = \frac{1}{2} K \gamma L_1 \tan\delta (\pi d L_1)$
- For group: Group Action → $\bar{C}_1 \cdot (4BL_1)$ + (Wt of soil in negative zone) → $B^2 \cdot L_1 \gamma$; or Individual Action → $n \cdot (\alpha_1 \bar{C}_1 \pi d L_1)$

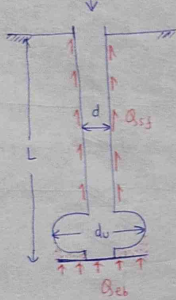
} Greater



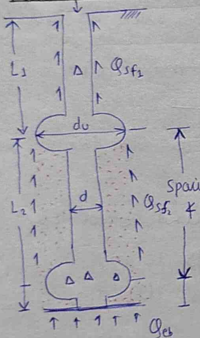
Under - Reamed Pipe

It is used in expansive soils which shows high swelling and shrinkage to prevent the uplift pressure cause by swelling of soil such as in Black Cotton soil

(Single Bulb U-R Pile)



(Multi-bulb U-R Pile)



Guidelines in U-R Pile

① U-R bulb dia should be 2-3 times of shaft dia generally 2.5 times is Provided

$\boxed{\frac{d_u}{d} = 2.5}$

② Spacing b/w bulbs ≮ 1.5 dia

③ Min^m length of U-R Pile 3.5 m

④ First bulb should be Provided at 1.5 m @ 2du whichever is max^m because swelling Potential decreases with depth

⑤ These Pile will be essentially cast-in-situ bored pile

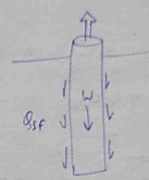
- Ultimate load carrying capacity of single bulb (U-R) Pile

$$\boxed{Q_{up} = q_b A_b + q_s A_s = 9.c\left(\frac{\pi}{4} d_u^2\right) + \alpha \bar{\tau}(\pi d L)}$$

- Ultimate load carrying capacity of multi bulb U-R Pile

$$\boxed{Q_{up} = Q_{eb} + Q_{sf_1} + Q_{sf_2} = 9c\left(\frac{\pi}{4} \cdot d_u^2\right) + \alpha_1 \bar{\tau}_1 (\pi d L_1) + \alpha_2 \bar{\tau}_2 (\pi d_u L_2)}$$

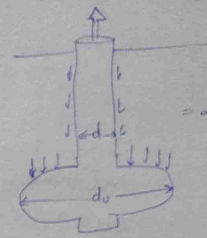
- Pile tensile capacity (Pile pulling test)



Pile tensile capacity

$= Q_{sf} + \text{wt of Pile}$

$= \alpha \bar{\tau}(\pi d L) + W$



Pile tensile capacity

$= \alpha \bar{\tau}(\pi d L) + W + 9c\left[\frac{\pi}{4} d_u^2 - \frac{\pi}{4} d^2\right]$

10) 0.5 m dia

L = 16m

UCS = 100

C = 50

$Q_{up} = Q_{sf} = \alpha \bar{\tau}(\pi d L)$

$= 0.75 \times 50(\pi \times 0.5 \times 16) = 940 KN$

11) $Q_{up} = \frac{WH}{(S+C)} = \frac{WH}{(S+2.5 cm)} = \frac{25 KN (800m)}{1.2 cm + 2.5 cm} = 540 KN$

$Q_{safe} = \frac{Q_{up}}{6} = \boxed{90 KN}$

37) Single vertical friction dia = 500 mm

length = 20m

$\phi = 30$

$\gamma_d = 20 KN/m^3$

$\delta = \frac{2\phi}{3}$

K = 2.7

$N_q = 25$

Ultimate bearing capacity (KN)

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

$= q_b A_b + q_s A_s$

$= q_b \times \frac{\pi}{4}(0.5)^2 + q_s \times ($

$q_s = \alpha \tau$

=

$Q_{up} = Q_{sf} = \cancel{\gamma L N_q^{\circ} A_b} + \frac{1}{2} K \gamma L \tan\delta (\pi d L)$

$Q_{up} = \frac{1}{2} \times 2.7 \times 20 \times 20 \tan\left\{\frac{2}{3}(20)\right\} \times (\pi \times 0.5 \times 20)$ (kN/m^3)

$= 617.4\ kN$

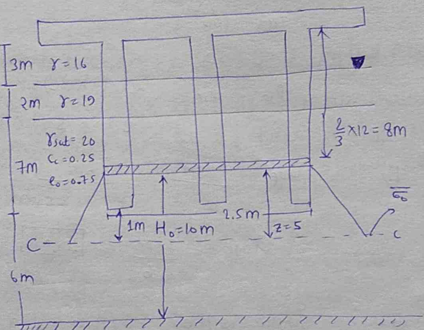
Solu (39)

Dia = 30cm

Net load = 2500 kN

$\Delta H = ?$ $n = 0.5$

Step ⓪



① $H_0 = 10\ m$

② $\overline{\sigma_0} = (3\gamma_1 + 2\gamma_2 + 8\gamma_3) - 10\gamma_w$

$= 3\times16 + 2\times19 + 8\times20 - 10\times10 = 146$

③ $\Delta\overline{\sigma} = \frac{force}{area}$

$= \frac{Q}{(B+Z)^2}$

$= \frac{2500}{(2.5+5)^2} = 44.44\ kN/m^2$

④ $\Delta H = \frac{H_0 C_c}{1+e_0} \log\left(\frac{\overline{\sigma_0} + \Delta\overline{\sigma}}{\overline{\sigma_0}}\right) = \frac{10\times0.25}{1+0.75} \log\left(\frac{146 + 44.44}{146}\right)$

$= .16486\ m$

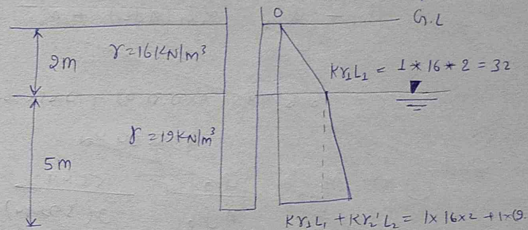
$= 164.8\ mm$

Ques (43)

$3m^2$ per m length

(43)

$3m^2$ per m length



$= 77.95$

Homogeneous sand

$\phi' = 32°$

$K = 1.0$

$\delta = 23°$

(total axial frictional) (kN)

1) $H_0 = 10m$

2) $\overline{\sigma_0} = \cancel{[3\gamma_1 + 2\gamma_2 + 8\gamma_3]}$

$= 3\times16 + 2$

$\Delta_{avg} \tan\delta\ A_{s_1}$

$\left[\frac{0+32}{2}\right] \tan 23°\ (3\times2)$

+

$\Delta_{avg} \tan\delta A_2$

$\left(\frac{32+77.95}{2}\right) \tan(23)(3*5)$

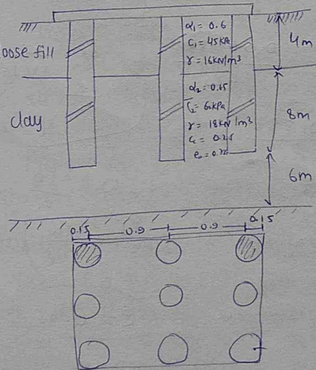
⇓

$390.18\ kN$

(45) solution

dia = 0.3m

F.O.S = 2.5



① $\eta_g = \frac{Q_{ug}}{n Q_{up}}$

$\cancel{Q_{ug} = Q \times 60 \times}$

$B = 2.1\ m$

$Q_{ug} = Q_{sf_1} + Q_{sf_2}$

$= \overline{\tau_1}(4BL_1) + \overline{\tau_2}(4BL_2)$

$= 45(4\times2.1\times4) + 60(4\times2.1\times8)$

$= 55.41\ kN$

$Q_{up} = Q_{ss_1} + Q_{ss_2} = \alpha_1 \bar{c}_1 (\pi d L_1) + \alpha_2 \bar{c}_2 (\pi d L_2)$

$= 0.6 \times 45 (\pi \times 0.3 \times 4) + 0.65 \times (60)(\pi \times 0.3 \times 8)$

$= 395.84$ KN

(1) $\Rightarrow \eta_g = \dfrac{Q_{ug}}{n Q_{up}} = \dfrac{55.44}{9(395.84)} = 1.55$

Q_{nf} — Group: $\bar{c}(4BL_1)$ + wt of soil in negative zone
$45(4 \times 2.1 \times 4) + (2.1)^2 \times 4 \times 16 = 1794.2$ KN

individual $= n[\alpha_1 \bar{c}_1 (\pi d L_1)]$
$= 9(0.6 \times 45 (\pi \times 0.3 \times 4)) = 916.08$ KN

maxm

Chapter (6)

Ans - (6)

$w = \dfrac{23-20}{20} \times 100 = 15 = 0.15$

$\gamma_d = \dfrac{\gamma_b}{1+w} = \dfrac{1.9 \times 10^3}{1+0.15}$

$\gamma_b = \dfrac{W}{V} = \dfrac{15}{944}$

$\gamma_b = 0.015$

$\gamma_d =$

$\gamma_b = \dfrac{15}{944} = \dfrac{0.01588}{1+w}$

$\gamma_d = \underline{0.01588}$

$Se = wG$

$\dfrac{Se}{G}$

$\gamma_d = 18.85$ KN/m³

$w = \dfrac{Se}{G}$

$\gamma_d = \left(\dfrac{G\gamma_w}{1+e}\right)$

$e = 0.405$

$\gamma_d = \dfrac{2.7 \times 9.81}{1.40}$

(18) $e = 0.5$

$e_{max} = 0.75$

$e_{min} = 0.35$

$I_D = \dfrac{e_{max} - e}{e_{max} - e_{min}} = \dfrac{0.5 - 0.35}{0.75 - 0.35} \times 100$

$= 62.5\%$

$R.C = \dfrac{1 + e_{min}}{1+e} \times 100$

$= \dfrac{1+0.35}{1+0.5} \times 100 = 90\%$ Ans