2.2 Foundation Engineering:

2.2.1 Introduction

Foundation:

↳ The lower part of structure which receives the load from super-structure and transmit it to any stable layer of soil is known as foundation.

Functions:

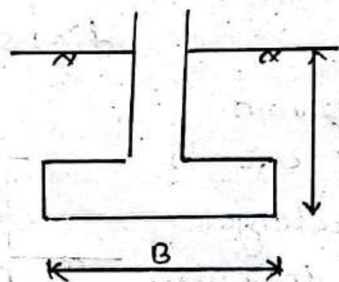
(i) to provide vertical and lateral stability to super structure.

(ii) to provide horizontal surface to superstructure.

(iii) to distribute the superimposed load to ground surface.

Types of foundation and their suitability:

↳ Foundation is broadly classified as:



Shallow foundation

- $\frac{D_f}{B} \leq 1$

Suitability:

(i) soil layer:
- dense to very dense sand
- stiff to stiff clay

(ii) Structure
- low to medium loaded structure

eg: residential building
short span bridge

Deep foundation

- $\frac{D_f}{B} > 1$

Suitability:

(i) soil layer:
- strong soil layer situated at large depth.

(ii) structure:
- Heavy structure

eg: high rise building
long span bridge.

Types:

(i) strip footing
(ii) Isolated footing
(iii) combined footing
(iv) strap footing
(v) Mat/Raft footing

Types:

(i) pile foundation
(ii) Pier foundation
(iii) cassion foundation/well foundation.

Explanation: Shallow Foundation

(i) strip footing:
- Run along wall
- Also known as wall footing or continuous footing.
- $\frac{l}{B} = \infty$

Use: for masonary structure (load bearing wall)

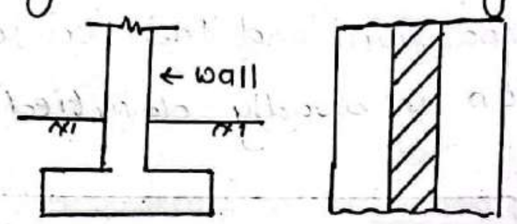


fig: strip footing

(ii) Isolated footing:
- Provided just below column
- to support individual column
- circular, square, rectangular slab of uniform thickness.

Use: for RCC framed residential building.

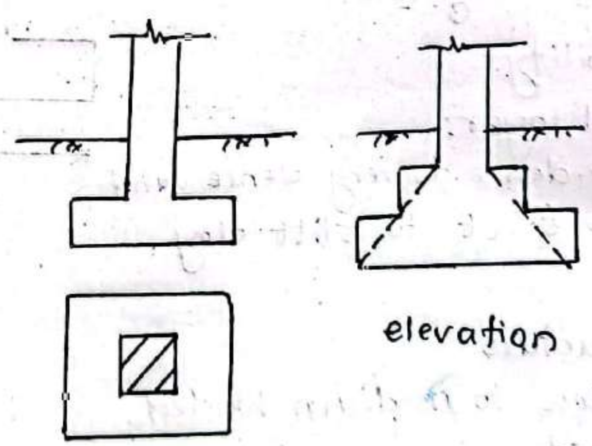
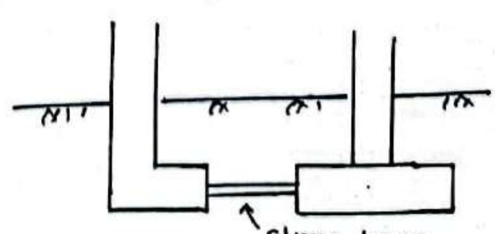


fig: isolated footing.

(iii) strap footing:
- A strap beam is provided at foundation level.
- It is also known as cantilever footing.

Use: to support column at property line.



(iv) Combined footing:

↳ single footing supporting two or more column in a line.

↳ Preferred in situation of limited space on one side owing to existence of boundary

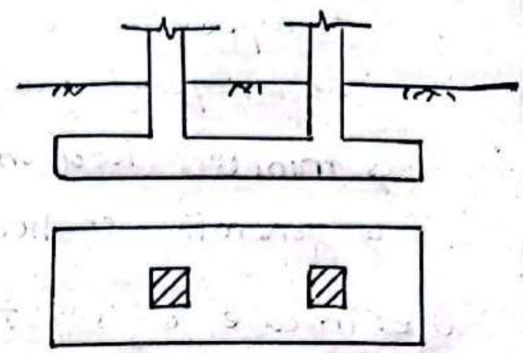


fig: combined footing.

Use:

↳ to support column at property line

↳ for closely spaced column.

(v) Mat footing /Raft footing:

↳ single slab supporting large number of column in many rows.

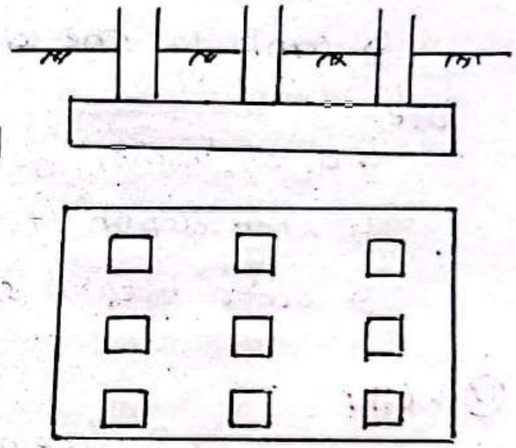


fig: mat foundation.

Use:

structure: Heavy structure like commercial complex.

condition: if isolated footing covers >50% of plinth area.

soil: compressive layer where the other shallow foundation doesn't perform well.

Explanation: Deep foundation

(i) Pile:

↳ A type of deep foundation having small x-section and slender body.

↳ possible shapes: circular, square, H-section, Helical etc.

Use: compressible clay layer,

(ii) Pier:

- A type of deep foundation having large x-section to support the super structure and transfer large superimposed load to firm strata below.
- transfers load only through bearing.
- generally shallow depth than pile.

Use: in-case of stiff clay.

(iii) Cassion:

- Pre-fabricated hollow sub-structures constructed on or near the surface ground.
- sunk to the desired depth and filled with concrete.

Use:

- as an 'anchor' for foundation.
- when depth of water is high in river.
- water breaks structure for shore protection.

(iv) Well:

- A type of deep foundation having large x-section and hollow body.

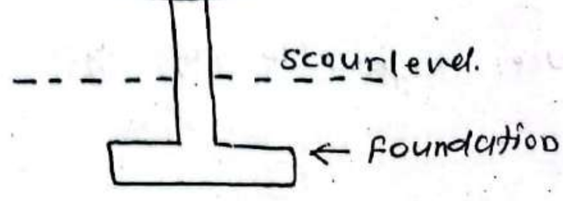
✗: Requirements or criteria for ideal foundation:

- Any ideal foundation for any structure should satisfy three requirements:

A. Minimum depth requirement:

criteria:

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



b. Scour depth:

- Foundation should be below scour level.

$D_s = 0.473 \left(\frac{Q}{f}\right)^{1/3}$

; Q = discharge (m^3/s)

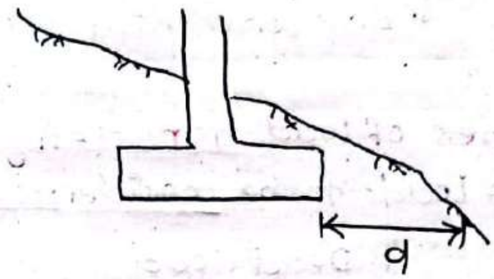
f = silt factor given by f = -

c. below frost penetration depth

d. below heat penetration depth.

e. below root penetration depth.

f. On slope ;



d ≥ 0.6m for rock slope

d ≥ 0.9m for normal soil.

B. Shear failure requirement:

↳ Foundation should not overstress the soil below foundation.

criteria:

$\frac{Q}{A} \leq q_{ns}$ → to prevent from shear failure.

; q_{ns} = net safe bearing capacity

C. settlement requirement:

↳ The settlement of the soil below foundation should be within permissible limit.

criteria:

$\frac{Q}{A} \leq q_{np}$

; q_{np} = net permissible settlement pressure.

x: Factors affecting the selection of type of foundation:

Main factors:

(a) load from superstructure

(b) sub-soil conditions and properties of soil.

(c) function of superstructure.

(d) cost of foundation w.r.to superstructure.

others factors:

- Risk level : location of site.
- Factor of safety required.
- Interest of client
- Environmental consideration.

#. Types of load for design of foundation:

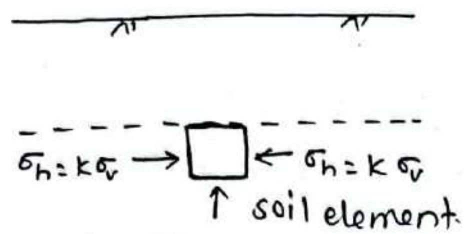
↳ Loads to be considered for design of foundation are:

1. Dead load
2. Live load
3. Impact load
4. Snow load
5. Wind load
6. Earthquake force
7. Earth pressure
8. Wave pressure
9. Water pressure and buoyancy
10. Stream flow pressure
11. Ice thrust
12. Thermal forces
13. Shrinkage
14. Other lateral forces;
    - (a) Traction or braking
    - (b) centrifugal force
    - (c) Rib shortening
    - (d) mooring pull
    - (e) ship impact
    - (f) swelling pressure

2.2.2 Earth Pressure and Retaining Structures:

Concept:

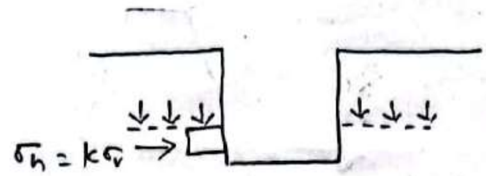
• H_z- surface.



↳ equal and opposite force.

↳ No radial movement of soil element.

• [illegible]



↳ At this condition;

• The radial stress deform the soil radially.

✱: Earth pressure:

↳ The horizontal stress which has a tendency to deform the soil radially is termed as earth pressure.

↳ To prevent the plastic failure in vertical cut or slope, the retaining structure should be designed against earth pressure

✱: Types of earth pressure:

↳ Depending on tendency of movement of wall, the earth pressure can be classified as:

(i) Earth pressure at rest.

(ii) Active earth pressure.

(iii) Passive earth pressure.

(i) Earth pressure at rest:

↳ No tendency of movement of wall.

↳ No radial deformation on soil.

↳ No plastic failure condition.

↳ The horizontal stress on wall is earth pressure at rest.

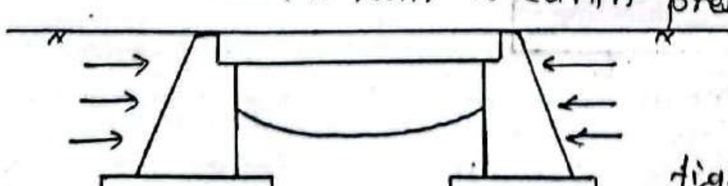
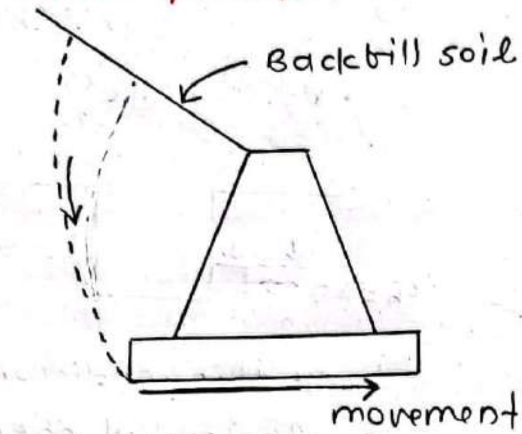


fig: Earth pressure on abutment or pier of bridge.

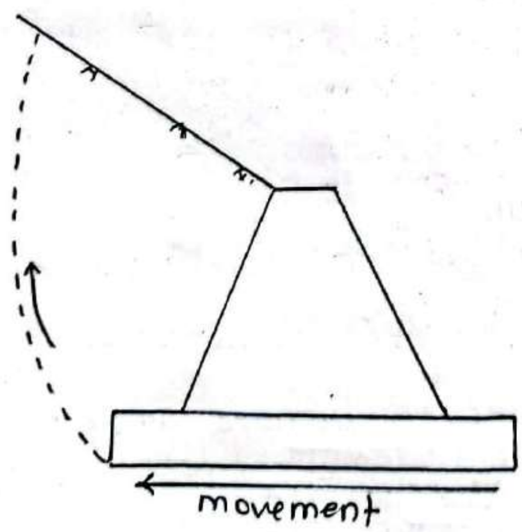
(II) Active Earth pressure:



- If the wall has a tendency to move outward than backfill soil, the backfill soil start to be relaxed, the horizontal stress decreases with movement.
- If sufficient movement is allowed, the Hz. stress reaches to minimum condition.
- Backfill soil reaches to plastic failure condition making failure wedge moves downward.
- The minimum horizontal stress between the wall and the failure wedge is termed as active earth pressure.

(III) Passive Earth pressure:

-



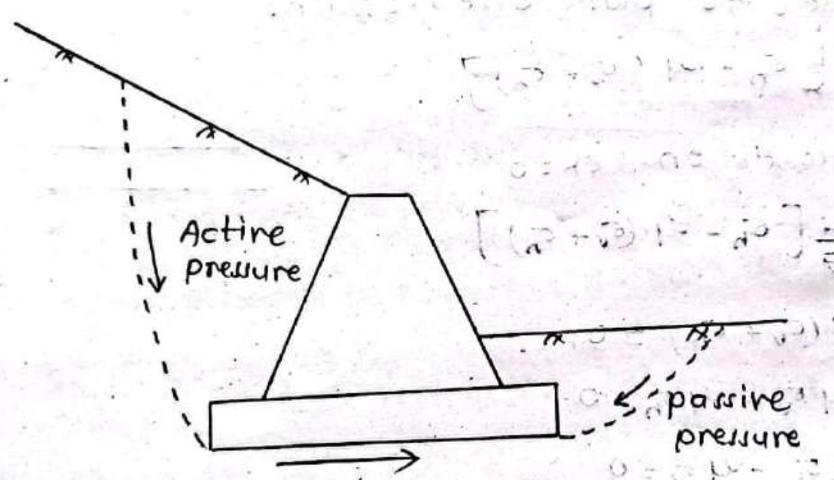
↳ If the wall has tendency to move inward to back to backfill soil, the backfill soil start to be compressed, the horizontal stress increases with movement.

↳ If sufficient movement is allowed the Hz. stress reaches to maximum condition.

↳ Backfill soil reaches to plastic condition making failure envelope moves upward.

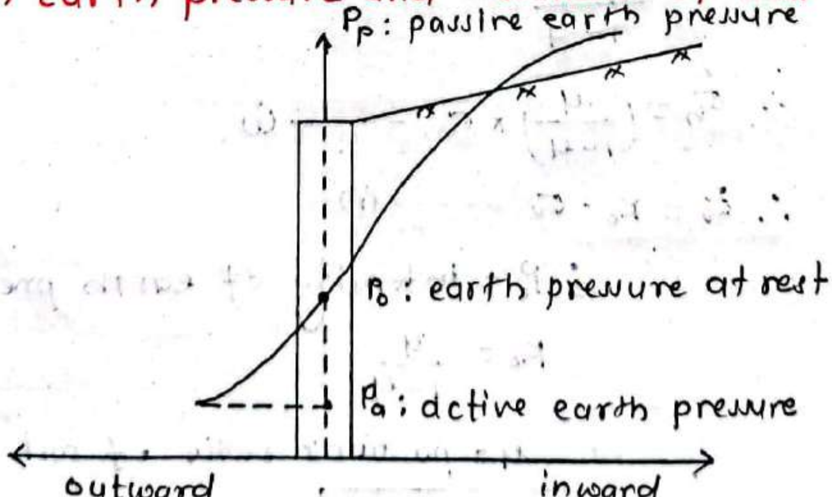
↳ The maximum Hz. stress between wall and failure wedge is termed as passive earth pressure.

In real field,



↳ Active pressure is resisted with the help of passive pressure.

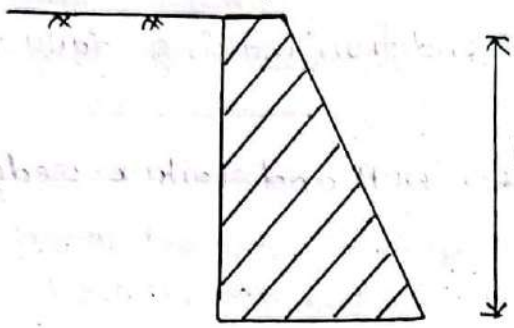
*: Relation between Earth pressure and movement of wall



* Calculation of Earth Pressure at rest:

Assumption:

- Soil is cohesion less.
- Backfill is horizontal.



From, Hooke's law for plane stress-strain:

$$\varepsilon_h = \frac{1}{E}\left[\sigma_h - \mu(\sigma_v + \sigma_h)\right]$$

for rest condition, $e_h = 0$

$$0 = \frac{1}{E}\left[\sigma_h - \mu(\sigma_v + \sigma_h)\right]$$

$$\sigma_h - \mu(\sigma_v + \sigma_h) = 0$$

$$\sigma_h - \mu\sigma_v - \mu\sigma_h = 0$$

$$\sigma_h - \mu\sigma_h - \mu\sigma_v = 0$$

$$\sigma_h(1-\mu) - \mu\sigma_v = 0$$

$$\sigma_h = \frac{\mu\sigma_v}{1-\mu}$$

$$\therefore \sigma_h = \left(\frac{\mu}{1-\mu}\right) \times \sigma_v \text{ ----- (i)}$$

$$\therefore P_o = K_o \cdot \sigma_v \text{ ----- (ii)}$$

; P_o = intensity of earth pressure at rest.

$$K_o = \frac{\mu}{1-\mu}$$

μ = poisson's ratio of soil. 0.3 to 0.5.

$$\boxed{K_o = 1 - \sin\phi.}$$

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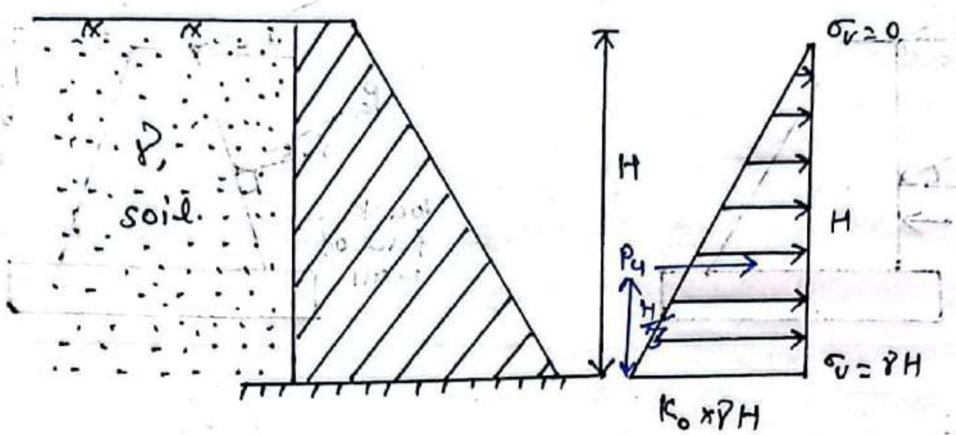
*: Calculation of total thrust on wall:



↳ Total rest thrust $(P_0) = \frac{1}{2} K_0 \gamma H^2$

$\therefore P_0 \propto H^2$ i.e. thrust $\propto H^2$

and lateral pressure $\propto$ depth(H) of soil.

*: Earth Pressure Theories:

↳ Basically we use two earth pressure theory for calculation:

a. Rankine's Earth Pressure theory.

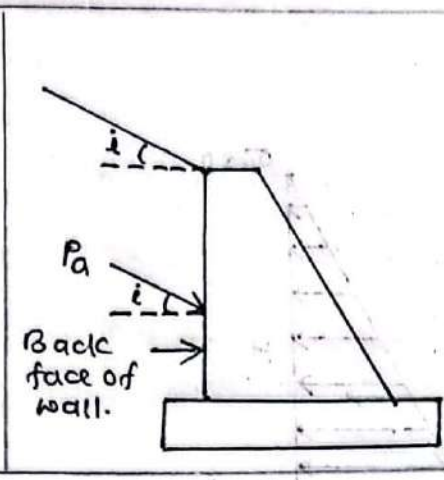
b. Coulomb's Earth Pressure theory.

Comparision of Rankine's and coulomb earth pressure theory:

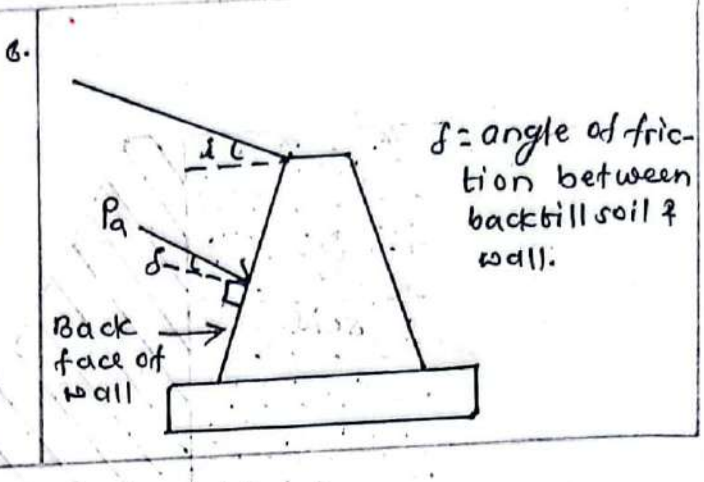
S.No.	Rankine's earth pressure theory		Coulomb earth pressure theory
1.	Backface of the wall is smooth ($\delta = 0$)	1.	Backface of the wall may not be smooth. (δ)
2.	Back face of the wall is vertical.	2.	Back face of the wall may be inclined.
3.	In this theory, elemental failure is considered.	3.	Here, wedge failure is considered.
4.	Both active and passive earth pressure can be found out.	4.	Not suitable for passive stage.
5.	It is releated to plastic equilibrium.	5.	It is releated to elastic equilibrium.

CS CamScanner

6.



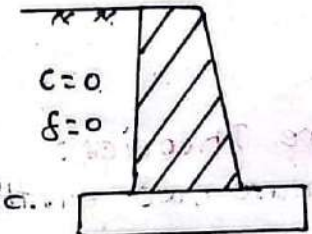
6.



a. Rankine Earth Pressure Theory:

↳ classical earth pressure theory : for cohesionless soil.

Assumption:



(I) soil mass is cohesionless.

(II) Horizontal backfill soil.

(III) soil mass is homogenous and isotropic.

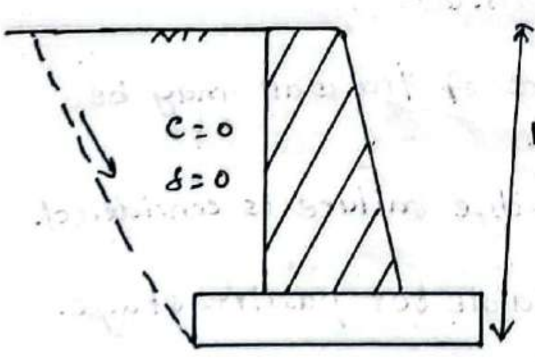
(IV) Coulomb theory of earth pressure valid ($\tau = c + \sigma \tan\phi$)

(V) Backface of the wall is vertical and smooth.

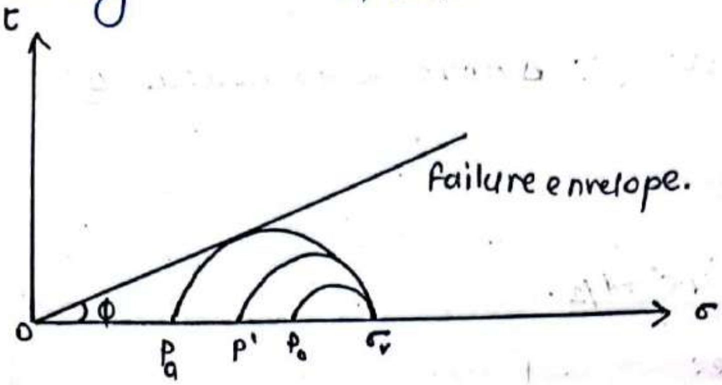
(VI) sufficient movement of the wall i.e plastic failure condition.

For cohesionless soil.

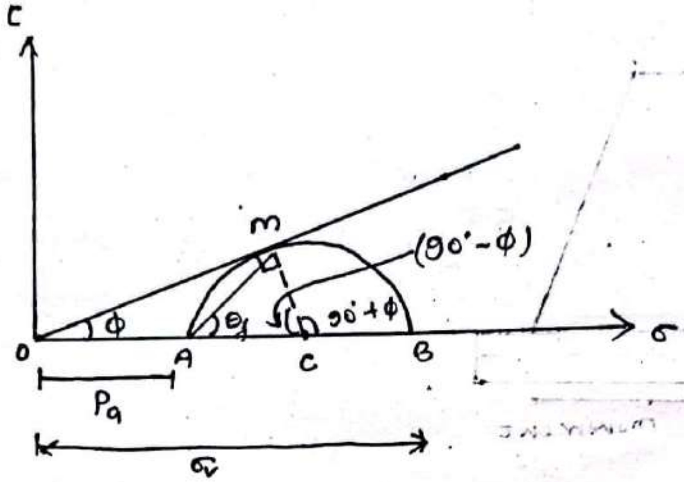
a. Active Earth pressure:



• Mohr's circle at any instant of wall:



• At failure:



from figure;

$P_a = OA = OC - AC$

$\sigma_v = OB = OC + CB$

from ΔOMC

$CM = OC\sin\phi$

$\therefore AC = BC = CM$

Now,

$P_a = OC - OC\sin\phi = OC(1-\sin\phi)$

$\sigma_v = OC + OC\sin\phi = OC(1+\sin\phi)$

we know,

$P_a = K_a \sigma_v$

$K_a = \frac{P_a}{\sigma_v}$

$K_a = \frac{OC(1-\sin\phi)}{OC(1+\sin\phi)}$

$K_a = \frac{1-\sin\phi}{1+\sin\phi}$

Note: $K_a \leq 1$

$K_a = \tan^2(45° + \phi/2)$

For failure angle:

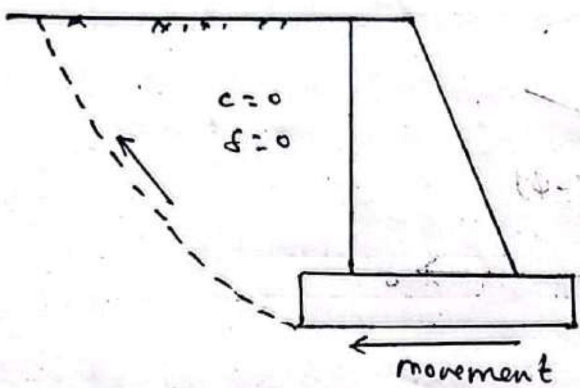
$$\theta_f + \theta_f + 90° - \phi = 180° \quad (\because \triangle AMC \text{ is an isosceles } \triangle)$$

$$2\theta_f - \phi = 90°$$

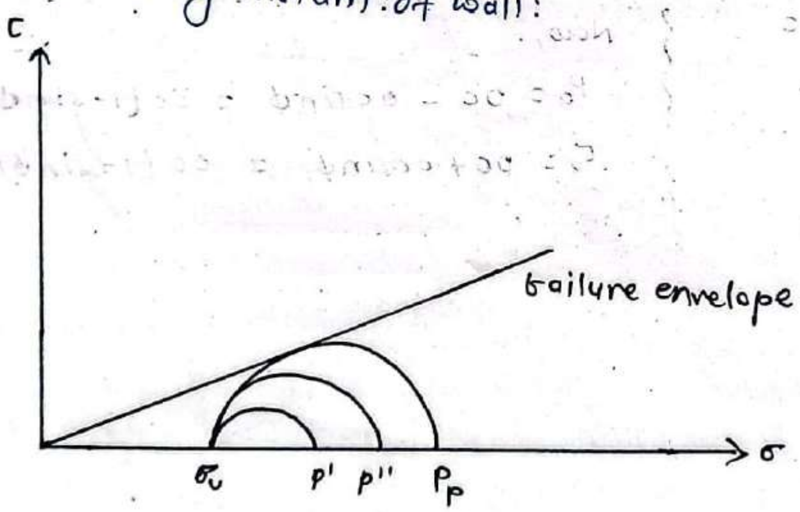
$$\theta_f = 45° + \phi/2$$

$\therefore$ Failure angle, $\theta_f = 45° + \phi/2$.

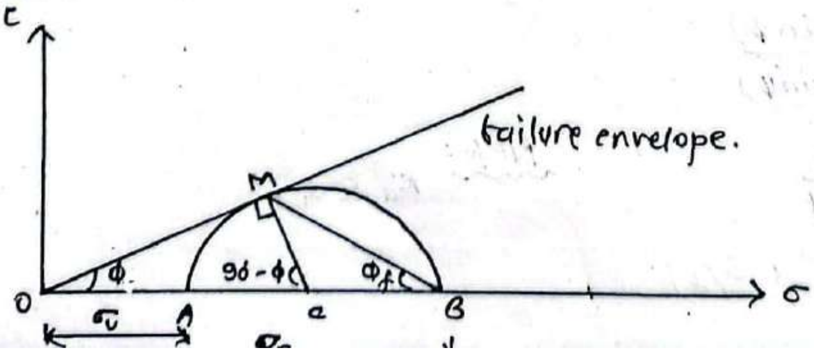
b. Passive Earth Pressure:



• Mohr's circle at any instant: At wall:



• At failure:



From figure;

$\sigma_v = OA = OC - AC$

$P_p = OB = OC + CB$

from Δ O C

$CM = OC \sin\phi$

$\therefore AC = CB = CM$

$\sigma_v = OC - OC\sin\phi = OC(1-\sin\phi)$

$P_p = OC + OC\sin\phi = OC(1+\sin\phi)$

we know,

$P_p = K_p \sigma_v$

$OC(1+\sin\phi) = K_p \times OC(1-\sin\phi)$

$$K_p = \frac{1+\sin\phi}{1-\sin\phi}$$

$K_p = \tan^2(45^\circ - \phi/2)$

For failure angle,

$\theta_f + \theta_f + 90^\circ + \phi = 180^\circ$

$2\theta_f = 90^\circ - \phi$

$\theta_f = 45^\circ - \phi/2$

$\therefore$ Failure angle, $\theta_f = 45^\circ - \phi/2$.

Rankine Earth pressure theory for cohesive soil:

Assumptions:

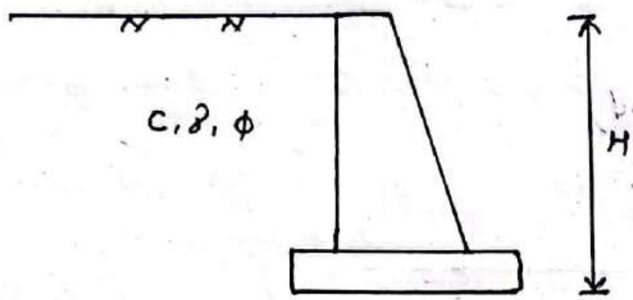
(i) soil mass is cohesive.

(ii) Horizontal backfill soil.

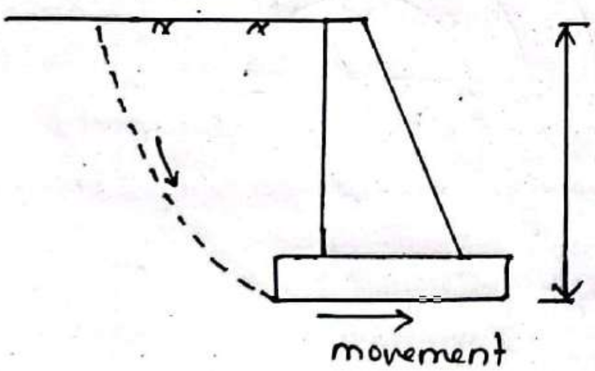
(iii) Homogenous and isotropic soil.

(iv) coulomb theory of shear strength valids. ($\tau = c + \sigma \tan\phi$)

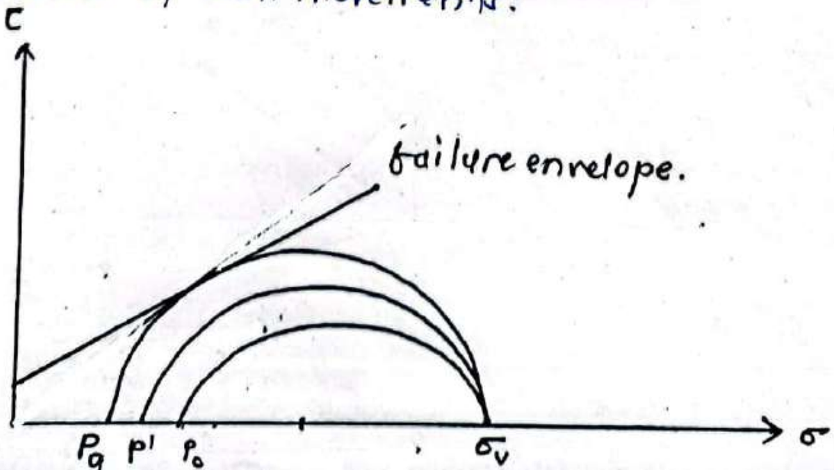
(v) sufficient movement of wall i.e plastic failure condition.



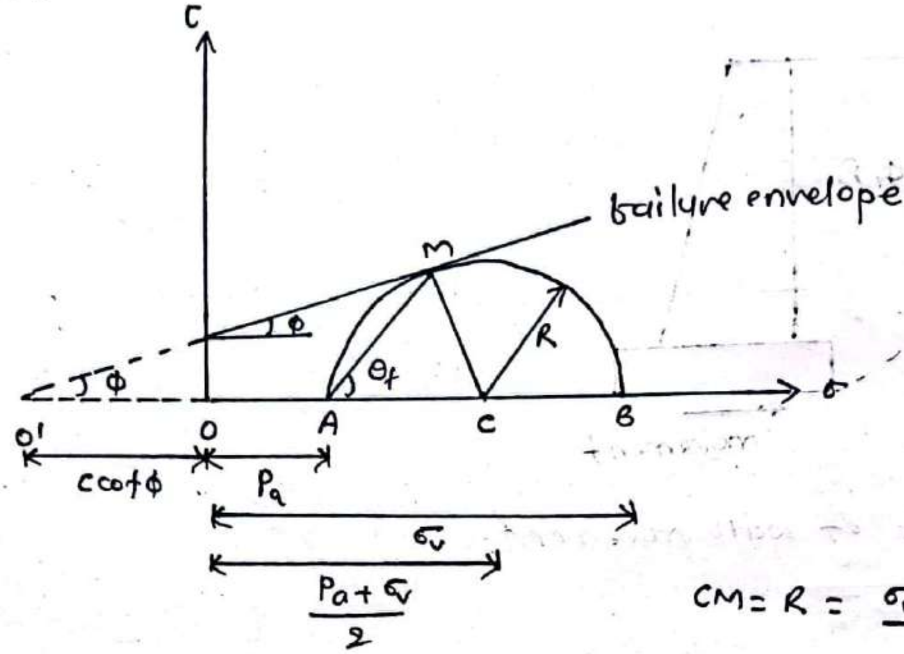
(a) Active Earth pressure theory:



• At many instants of wall movements:



- At failure condition:



$$CM = R = \frac{\sigma_v - P_a}{2}$$

From $\Delta O'MC$

$$\sin\phi = \frac{CM}{O'C}$$

$$\sin\phi = \frac{CM}{c\cot\phi + \frac{P_a+\sigma_v}{2}}$$

$$\frac{\sigma_v}{2} - \frac{P_a}{2} = c\cos\phi + \frac{P_a}{2}\sin\phi + \frac{\sigma_v}{2}\sin\phi$$

$$P_a = \sigma_v\left(\frac{1-\sin\phi}{1+\sin\phi}\right) - 2c\left(\frac{\cos\phi}{1+\sin\phi}\right)$$

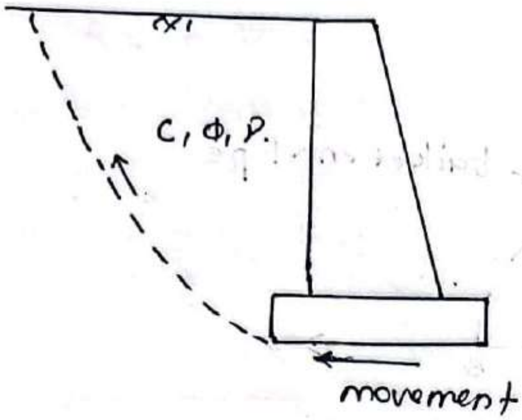
$$P_a = \sigma_v \times K_a - 2c\sqrt{K_a}$$

$$\therefore P_a = K_a\sigma_v - 2c\sqrt{K_a} \quad ; \quad K_a = \frac{1-\sin\phi}{1+\sin\phi}$$

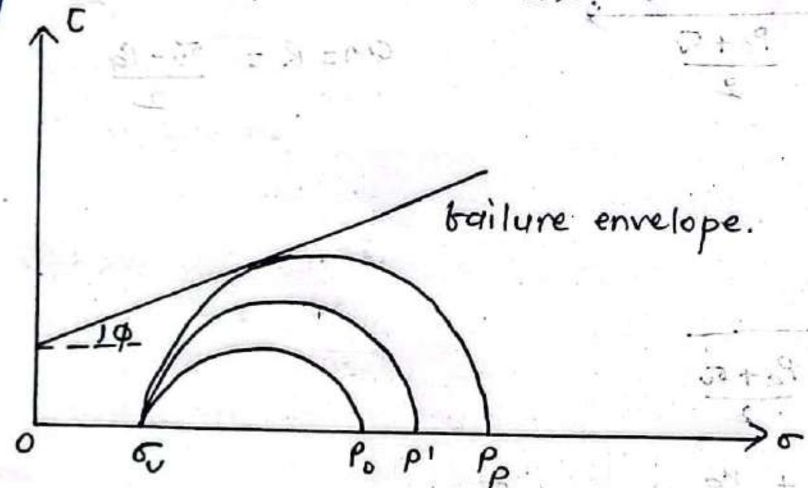
- Failure angle

$$\theta_f = 45^\circ + \phi/2$$

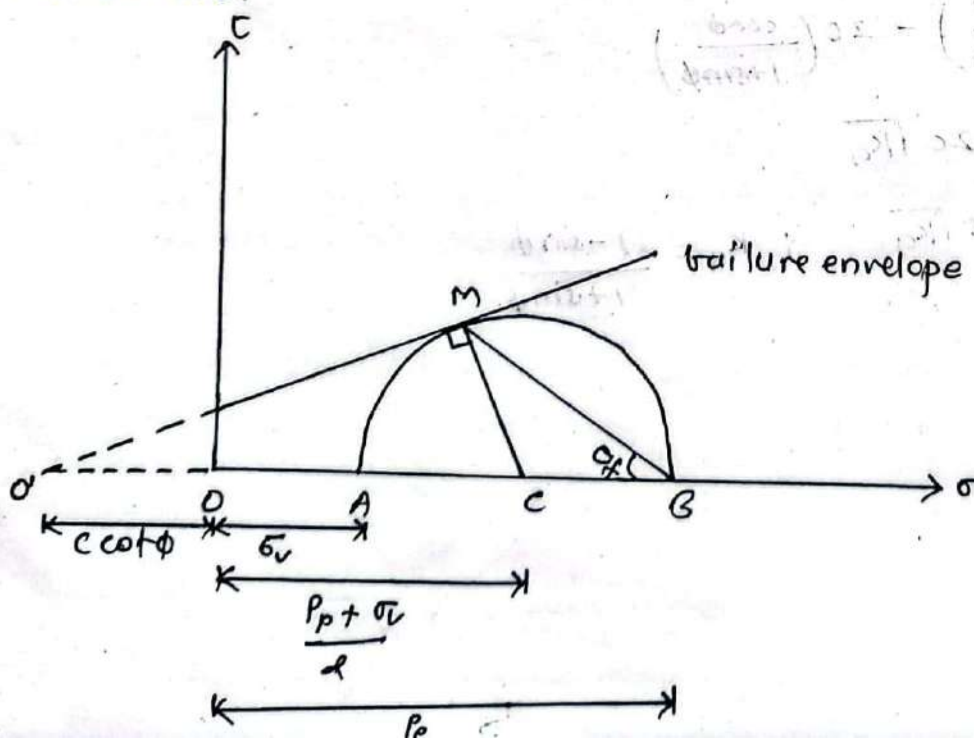
b. Passive Earth Pressure:



• At many instants of wall movement:



• At failure:



$CM = \frac{P_p - \sigma_v}{2}$

from OO'MC

$$\sin\phi = \frac{CM}{O'C}$$

$$\sin\phi = \frac{CM}{O'O + OC}$$

$$\sin\phi = \frac{\frac{P_p - \sigma_v}{2}}{c\cot\phi + \frac{P_p + \sigma_v}{2}}$$

or, $-\frac{\sigma_v}{2} + \frac{P_p}{2} = c\cos\phi + \frac{P_p}{2}\sin\phi + \frac{\sigma_v}{2}\sin\phi$

or, $P_p = \sigma_v\left(\frac{1+\sin\phi}{1-\sin\phi}\right) + 2c\left(\frac{\cos\phi}{1-\sin\phi}\right)$

or, $P_p = K_p \times \sigma_v + 2c\sqrt{K_p}$

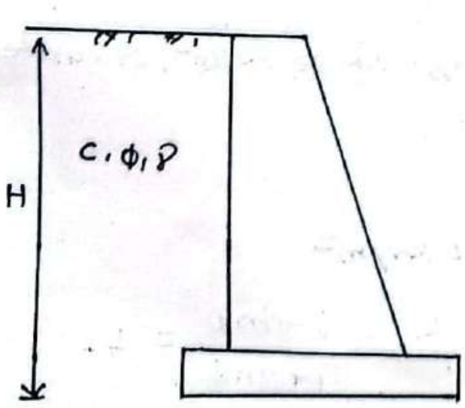
$\therefore P_p = K_p\sigma_v + 2c\sqrt{K_p}$; $K_p = \frac{1+\sin\phi}{1-\sin\phi}$

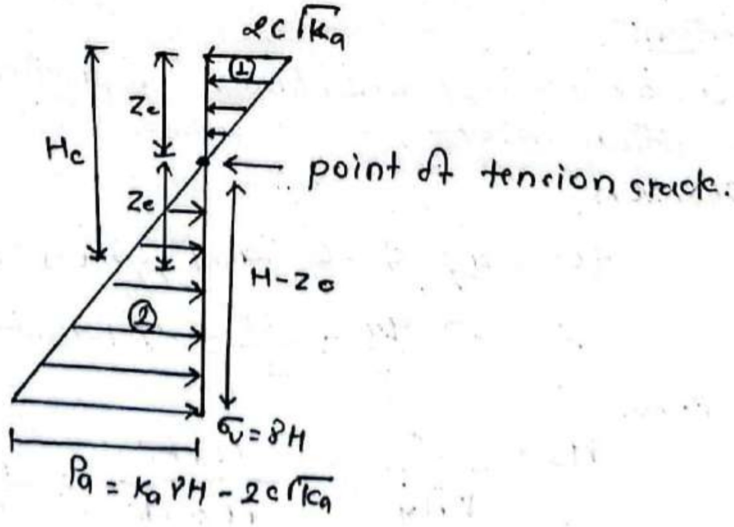
failure angle;

$\theta_f = 45° - \phi/2$

calculation of total thrust on wall:

↳ For active case :-





for depth of tension crack;

$P_a = 0$

$K_a \gamma \; - 2c\sqrt{K_a} = 0$

$K_a \gamma H = 2c\sqrt{K_a}$

$$H = \frac{2c\cancel{\sqrt{K_a}}}{(\sqrt{K_a})^{\cancel{2}} \times \gamma}$$

$$H = \frac{2c}{\gamma\sqrt{K_a}} = z_c$$

$$\therefore z_c = \frac{2c}{\gamma\sqrt{K_a}}$$

and $H_c = 2 z_c$

$$H_c = 2 \times \frac{2c}{\gamma\sqrt{K_a}}$$

$$\therefore H_c = \frac{4c}{\gamma\sqrt{K_a}}$$

Critical height (H_c):

↳ The maximum height of vertical cut that can be made without any support.

$$H_c = 2z_c = \frac{4c}{\gamma\sqrt{K_a}}$$

Numerical:

For a clay having unconfined compressive strength of 50 kN/m², calculate the critical height, $\gamma = 18$ kN/m³.

solution

for clay $\phi = 0$ and given UCS = 50 kN/m²

$\therefore c = \frac{q_u}{2} = \frac{50}{2} = 25$ kN/m². and $K_a = \frac{1-\sin\phi}{1+\sin\phi} = 1$.

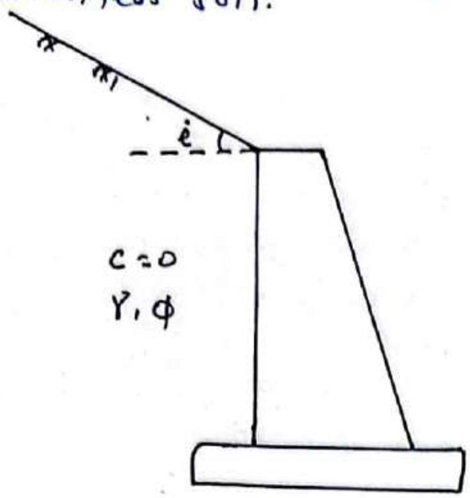
Now,

$$H_c = \frac{4c}{\gamma\sqrt{K_a}} = \frac{4\times 25}{18\times\sqrt{1}} = \frac{100}{18} = 5.56\text{ m}$$

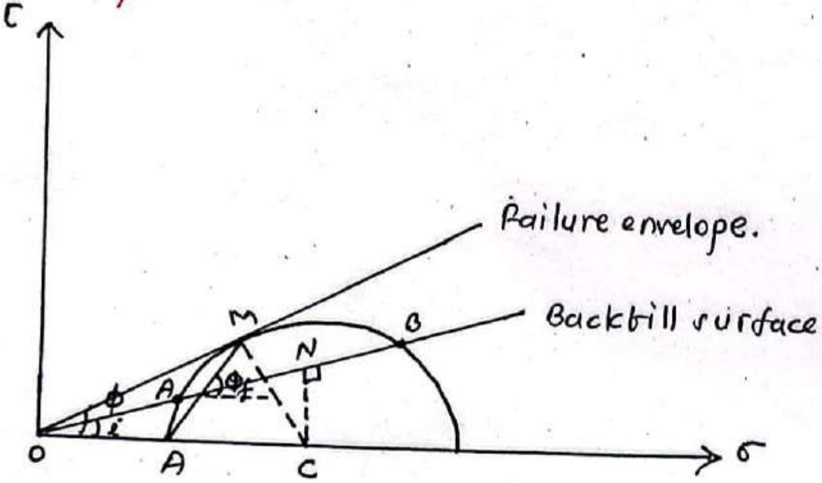
&

Rankine Earth Pressure theory for inclined backfill:

- for cohesionless soil:



a. Active Earth pressure:



$P_a = OA = ON - AN$

$\sigma_v = OB = ON + NB$

$ON = OC \cos i$ (from ΔONC)

$AN = NB$ ($\because \perp^{er}$ from center divide chord).

$AN = \sqrt{AC^2 - NC^2}$

$NC = OC \sin i$ (ΔONC)

$AC = MC$ (both are radius) $AN = OC\sqrt{[illegible] - \sin^2 i}$

$MC = OC \sin\phi$ (ΔOMC)

Now,

$$P_a = \sigma c\cos i - \sigma c\sqrt{\sin^2\phi - \sin^2 i}$$

$$\sigma_v = \sigma c\cos i + \sigma c\sqrt{\sin^2\phi - \sin^2 i}$$

we know,

$$P_a = K_a \times \sigma_v$$

$$K_a = \frac{P_a}{\sigma_v}$$

$$K_a = \frac{\cos i - \sqrt{\sin^2\phi - \sin^2 i}}{\cos i + \sqrt{\sin^2\phi - \sin^2 i}}$$

$$K_a = \frac{\cos i - \sqrt{\cos^2 i - \cos^2\phi}}{\cos i + \sqrt{\cos^2 i - \cos^2\phi}}$$

Total thrust on wall:

$$P_a = \frac{1}{2}\gamma H^2 \times K_a \cos i$$

$$P_a = \frac{1}{2}K_A \gamma H^2$$

$$\because K_A = K_a \times \cos i$$

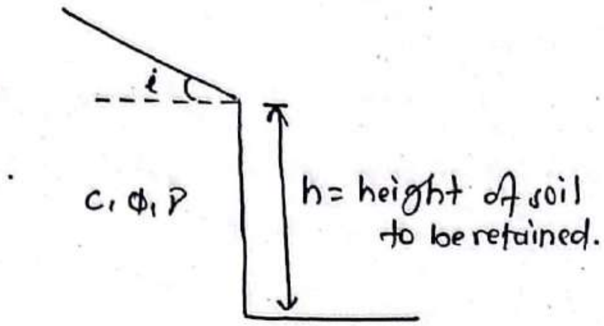
$$K_A = \cos i \times \frac{\cos i - \sqrt{\cos^2 i - \cos^2\phi}}{\cos i + \sqrt{\cos^2 i - \cos^2\phi}}$$

for $i = 0$ i.e. horizontal backfill.

$$K_A = K_a = \frac{1 - \sin\phi}{1 + \sin\phi}$$

Design of Retaining wall:

field condition:



Here, c, ϕ and P can be obtained from lab-test:

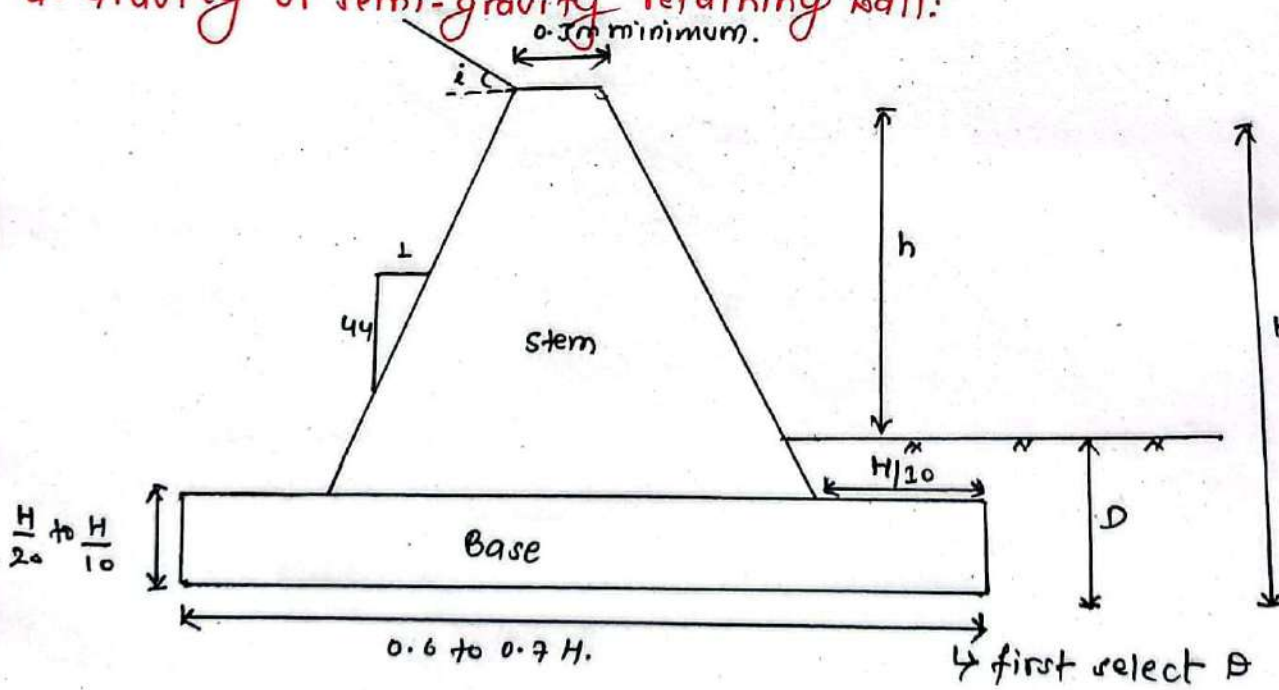
∴ For known value of i and h, retaining wall can be designed.

Q! Types of retaining wall:

↳ Retaining wall is a structure designed and constructed to withstand lateral pressure of soil.

↳ lateral pressure could also be due to earth fillings, liquid pressure, sand and other granular materials behind the retaining wall structure.

a. Gravity or semi-gravity retaining wall:



↳ first select D

↳ ∴ H = h + D

fig: Gravity/semi-gravity retaining wall.

↳ large body
↳ self weight is responsible for stability
↳ masonary structure: stone masonary.
↳ Best use: < 6m height.

b. cantilever retaining wall:

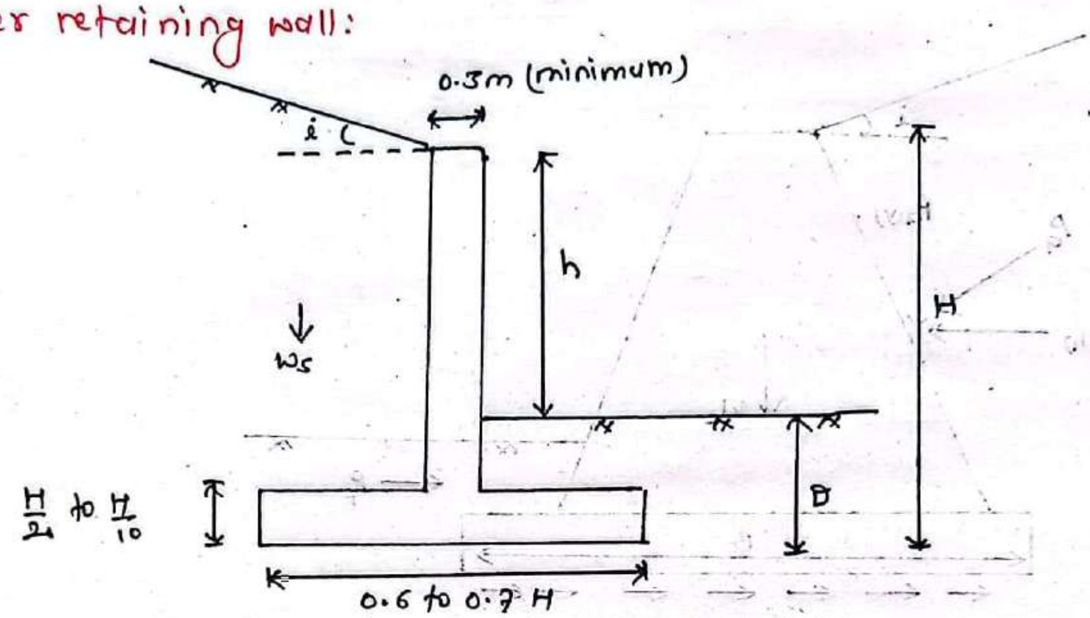


fig: cantilever retaining wall.

↳ small body
↳ weight of soil is also mobilized for stability
↳ RCC structure
↳ Use ≥ 6m

c. counterfort or buttressed retaining wall:

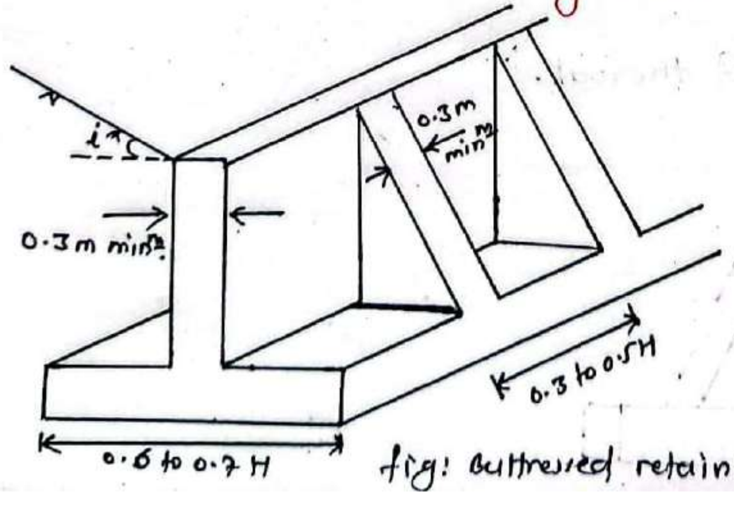
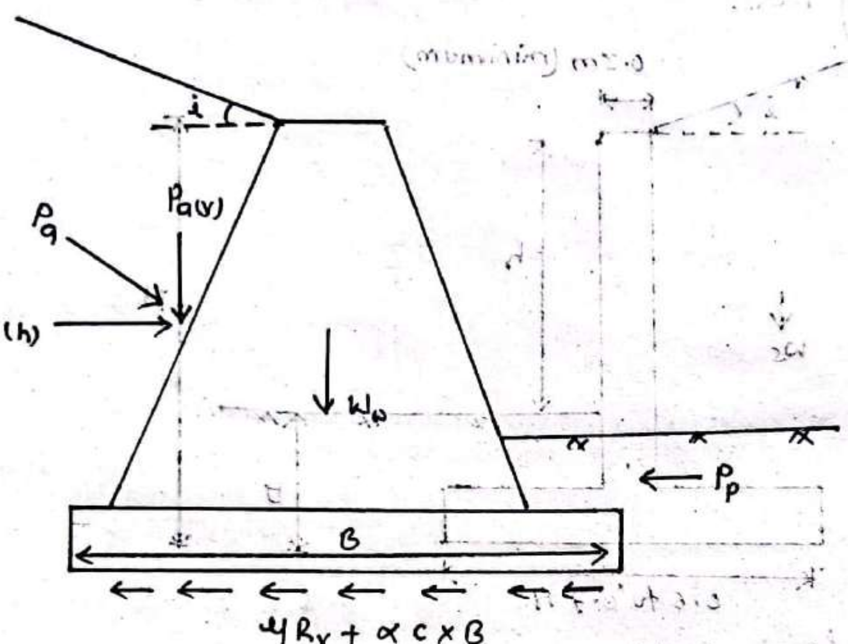


fig: buttressed retaining wall.

↳ If the retained height is very high, stem of wall is supported by RCC slab known as counterfort / buttressed retaining wall.

↳ economical at height > 6 m.

Stability check of retaining wall:



Here, $\mu = \tan\delta$

R_v = total vertical weight.

$= W_\omega + P_{a(v)} + W_s$ ← in case of cantilever retaining wall.

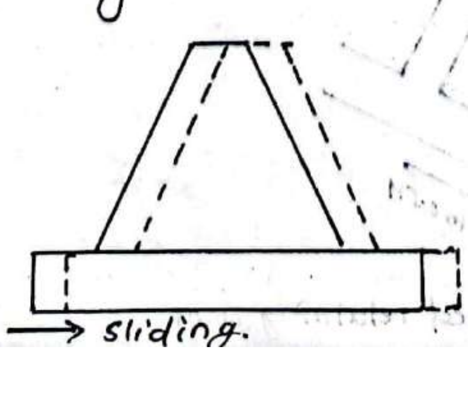
α = adhesion factor.

*: Possible failures of retaining wall:

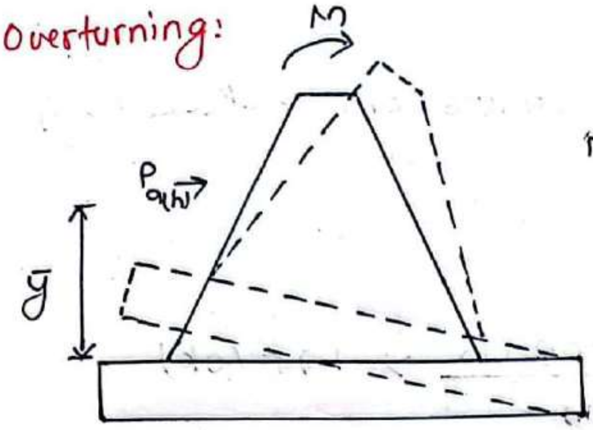
a. Sliding:

↳ Horizontal movement of the wall.

↳ $P_{a(h)}$: sliding force.



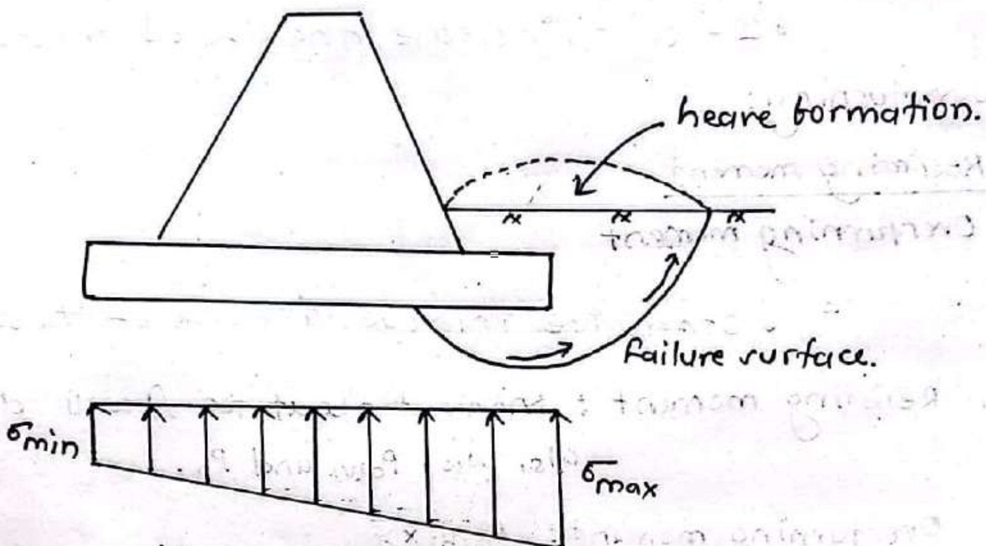
b. Overturning:



M = overturning moment

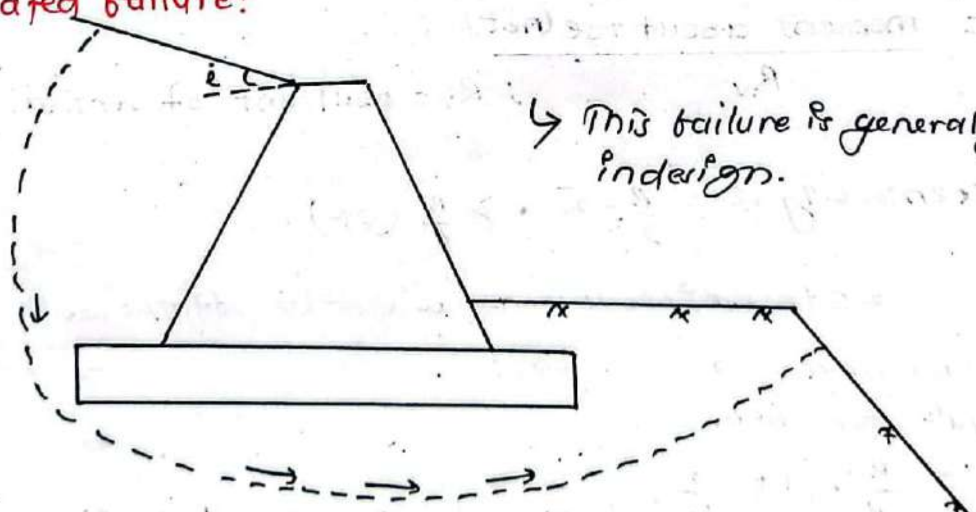
$= P_{a(h)} * \bar{y}$

c. Bearing failure:



if σ_{max} > Bearing capacity

d. Deep seated failure:



↳ This failure is generally not considered in design.

deep seated failure

Stability check:

↳ stability of retaining wall against failure can be checked by, calculating fos.

① Against sliding:

$$FOS = \frac{\text{Resisting force}}{\text{Sliding force}} = \frac{\mu R_v + \alpha c B + P_p}{P_{a(h)}} > 1.75 \text{ (OK)}$$

otherwise

$$FOS = \frac{\mu R_v}{P_{a(h)}} \geq 1.5 \text{ (ok)}$$

- If fails, increase the size of the wall.

(ii) Against overturning:

$$FOS = \frac{\text{Resisting moment}}{\text{Overturning moment}}$$

- otherwise increase the size of the wall.

Here, Resisting moment : moment about toe of wall due to W_s, W_w, $P_{a(v)}$ and P_p.

Overturning moment : $P_{a(h)} \times \bar{y}$

(iii) Against bearing:

$$\bar{x} = \frac{\text{moment about toe (net)}}{R_v}$$; R_v = resultant of vertical forces.

$$\text{eccentricity } (e) = \frac{B}{2} - \bar{x} \not> \frac{B}{6} \text{ (ok)}$$

- otherwise increase width (B) of the wall.

∝: Maximum and minimum stress on wall:

general equ^n for stress,

$$\sigma = \frac{R_v}{B}\left(1 \pm \frac{6e}{B}\right)$$; +ve for max^m value (σ_{max})

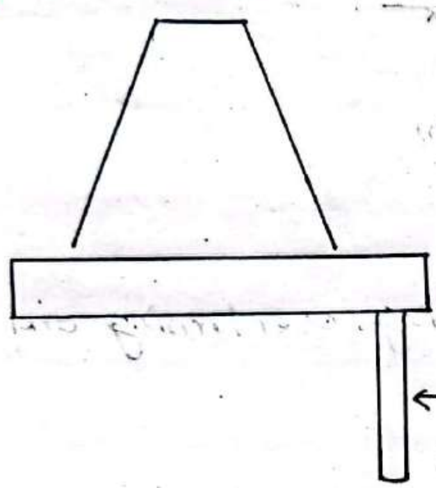
−ve for min^m value (σ_{min}).

$$FOS = \frac{\text{Allowable bearing capacity}}{\sigma_{max}} \geq 2.0 \text{ (OK)}$$

- otherwise increase 'B'.

Some special technique to increase the stability of retaining wall:

a. Shear key or Pedestal at toe:

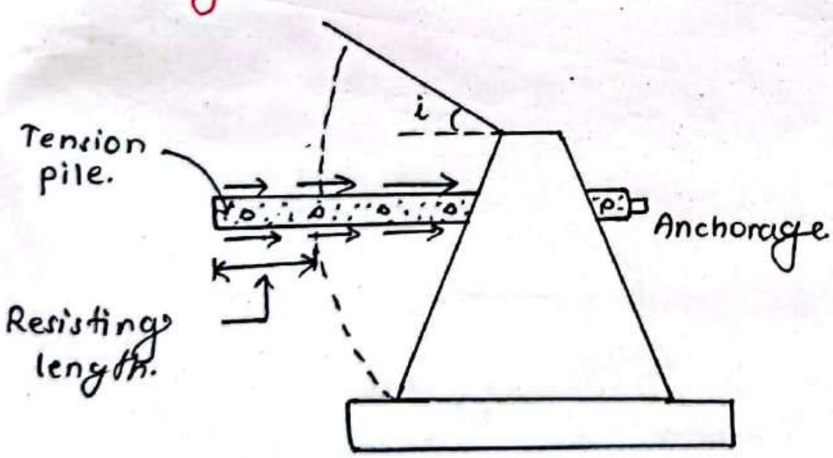


← Pedestal

(सानो size को लाई shear key भनिन्छ!)

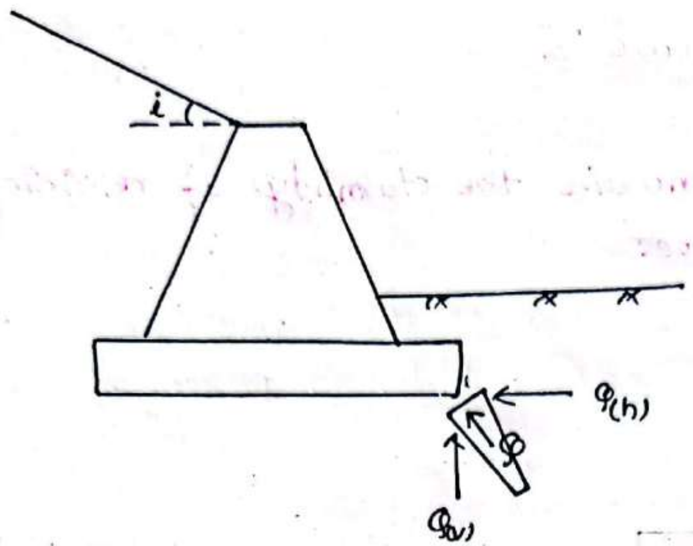
↳ Increase the stability against sliding.

b. Anchorage:



↳ Increases stability against overturning.

c. Batter pile:



↳ Increases stability against sliding, overturning and bearing. failure.

2.2.3 Bearing capacity and settlements:

Introduction:

Bearing capacity:

- The maximum pressure that can be applied through foundation at which the soil below the foundation neither fails in shear nor gives excessive settlements is known as bearing capacity.
- Mutual response of foundation and the soil on which foundation lies.
- Foundation converts the load from column to stress on soil.

*: Effects of applied stress:

a. shear failure.

b. settlement.

Stages of shear failure:

Stage 1: Distortion of soil.

stage 2: Formation of rigid cone.

stage 3: local cracking

stage 4: Failure surface development.

stage 5: Heave formation ground surface around footing.

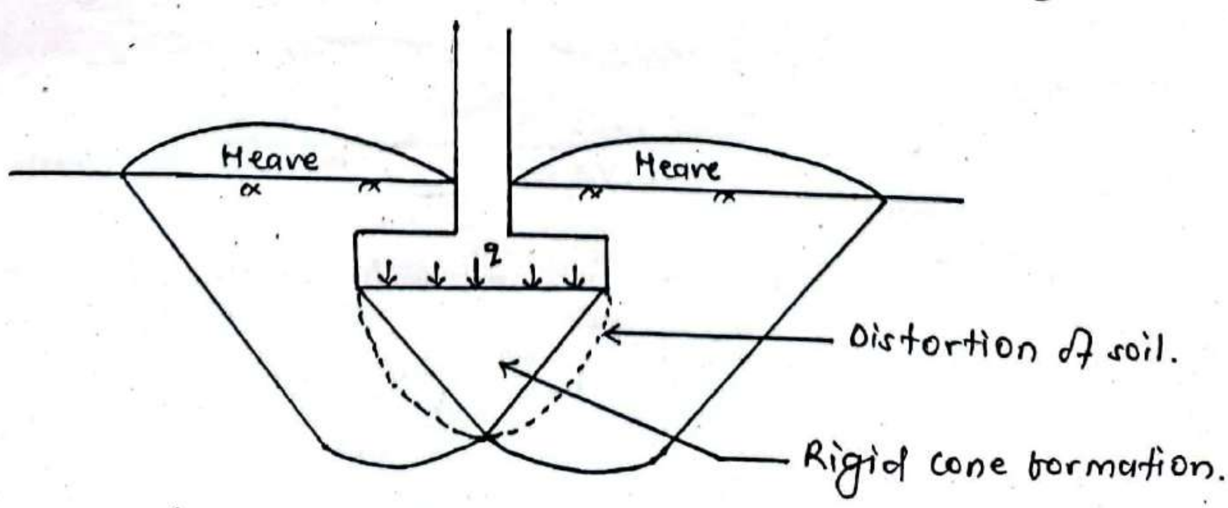


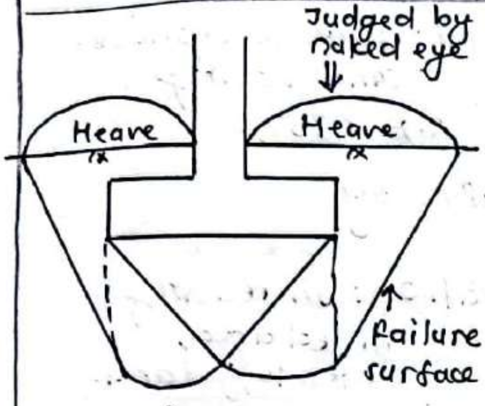
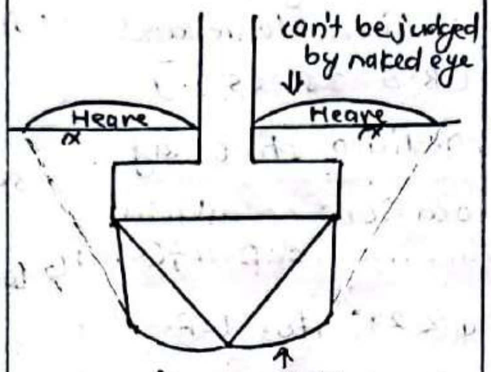
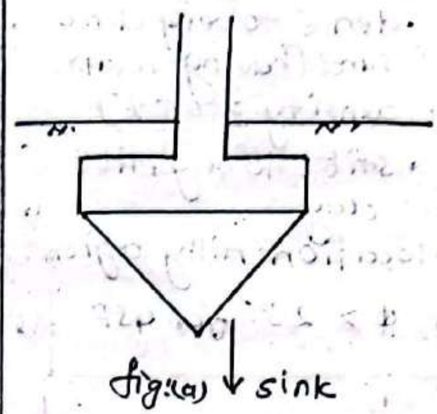
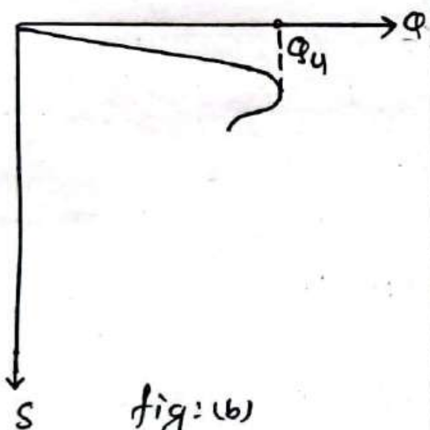
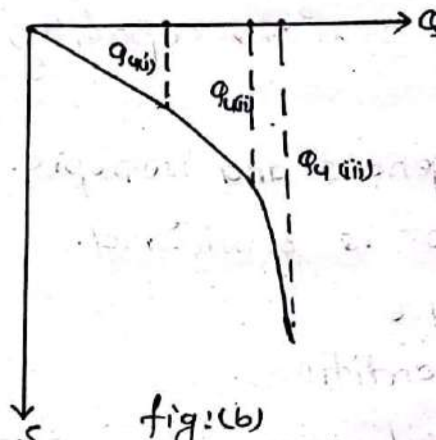
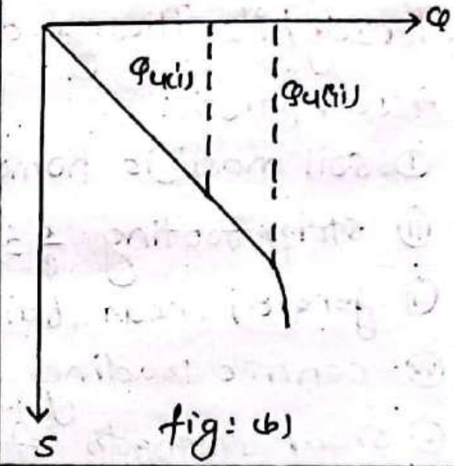
fig: stages of shear failure.

*: Principal mode and Type of shear failure:

↳ Depending upon the strength of soil in site, possible types of shear failure are:

a. General shear failure. (GSF)
b. local shear failure. (LSF)
c. Punching shear failure. (PSF)

Terai Region	Hilly Region	Himalayan Region
Top soil: Alluvial deposit i.e. B.C low. so, L.S.F occurs.	Top soil: Lacustrine deposit ↳ compressible & medium strength soil. ∴ LSF occurs.	Top soil: Residual deposit. ↳ strength maximum i.e. B.C. is good ↳ Bottom rock found ∴ GSF occurs.

General shear failure (GSF)	Local shear failure (LSF)	Punching shear failure (PSF)
 fig:(a)	 fig:(a)	 fig:(a)
 fig:(b)	 fig:(b)	 fig:(b)

Characteristics of failure:-

↳ Failure surface develops as shown in fig.(a) above. ↳ large heave formation on ground surface around footing as shown in fig(a). ↳ Defined ultimate load of failure (q_u) as shown in fig(b).	↳ Failure surface develops below the foundation only as shown in fig(a). ↳ small heave formation on ground surface around footing only after more than 8% strain developed. ↳ No defined ultimate load of failure as shown in fig.(b).	↳ No failure surface develops, only rigid cone formed and sinks continuously. ↳ No heave formation on ground surface. ↳ No defined ultimate load of failure as shown in fig.(b).

↳ occurs in high strength soil layer. i.e. • dense to very dense sand [having relative density ≥ 65%] • stiff to very stiff clay ↳ location: Hilly region ↳ $\phi > 28°$ for GSF	↳ Occurs in medium strength soil layer. i.e • medium dense sand [R.D 35-65%] • Medium stiff clay ↳ location: on alluvial deposit (Ktm) ↳ $\phi < 28°$ for LSF	↳ occurs in weak soil layer i.e. • sand: loose to very loose sand having R.D < 35%. • soft clay ↳ location: on recently filled area, water logged area.

Terzaghi's theory of bearing capacity :-

Assumptions:

(i) soil mass is homogenous and isotropic.

(ii) strip footing $\frac{L}{B} = \infty$ is considered.

(iii) general shear failure

(iv) centric loading condition.

(v) shear strength of soil above base of footing has no role on bearing capacity, so he has replaced it by surcharge.

$q = \gamma D_f$.

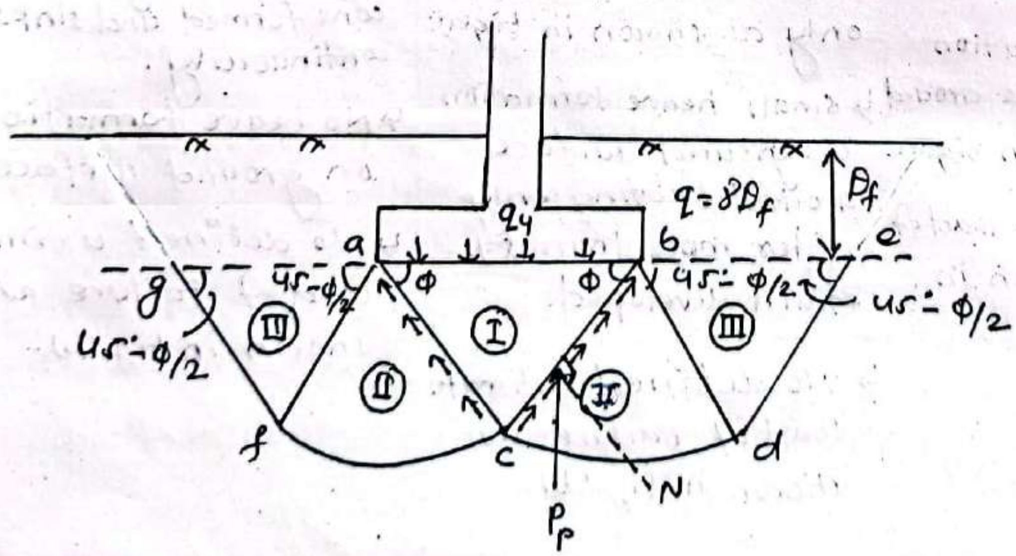


fig:

Failure surface:

fc, cd : logarithmic spiral having radius $r = r_0 e^{n\tan\phi}$ & centre at a and b respectively.

de, fg : linear

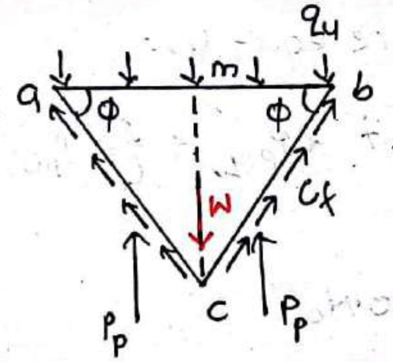
Failure wedges:

wedge I : Active zone.

wedge II : Shear zone.

wedge III : Passive zone.

Derivation:



Applying equilibrium condition;

total downward force = total upward force

$$q_u \times (B\times 1) + W = 2\, C_f \times \sin\phi + 2P_p$$

$$q_u \times B + (A\times 1)\times \gamma = 2\left[c \times (bc\times 1)\right]\times \sin\phi + 2P_p \quad \left[\because C_f = c\times(bc\times 1)\right]$$

$$q_u \times B + \frac{1}{2}\times B\times \frac{B}{2}\tan\phi \times\gamma = 2c \times \frac{B/2}{\cos\phi}\times\sin\phi + 2P_p \quad \left[\because bc = \frac{B/2}{\cos\phi} \text{ \& area of } \Delta abc = \frac{1}{2}\times B\times\frac{B}{2}\tan\phi\right]$$

$$q_u \times B + \frac{1}{4}B^2\gamma\tan\phi = B\times c\tan\phi + 2P_p. \text{ ----- (i)}$$

Passive pressure P_p depends on cohesion (c), unit weight (γ) and surcharge (q).

$\therefore$ P_p can be divided into three components.

a. $P_{p(c)}$: Neglecting weight & surcharge.

b. $P_{p(q)}$: Neglecting cohesion & unit weight.

c. $P_{p(\gamma)}$: Neglecting cohesion & surcharge.

Now, applying law of superposition:

$$P_p = P_{p(c)} + P_{p(\gamma)} + P_{p(q)}$$

Substituting this value in above equ^n (i):

$$q_u \times B = \{B \times c\tan\phi + 2P_{p(c)}\} + \alpha P_{p(q)} + \{\alpha P_{p(\gamma)} - \tfrac{1}{4}B^2\gamma\tan\phi\}$$

Replacing

$$B \times c\tan\phi + \alpha P_{p(c)} = B \times c \times N_c$$

$$\alpha P_{p(q)} = B \times q \times N_q$$

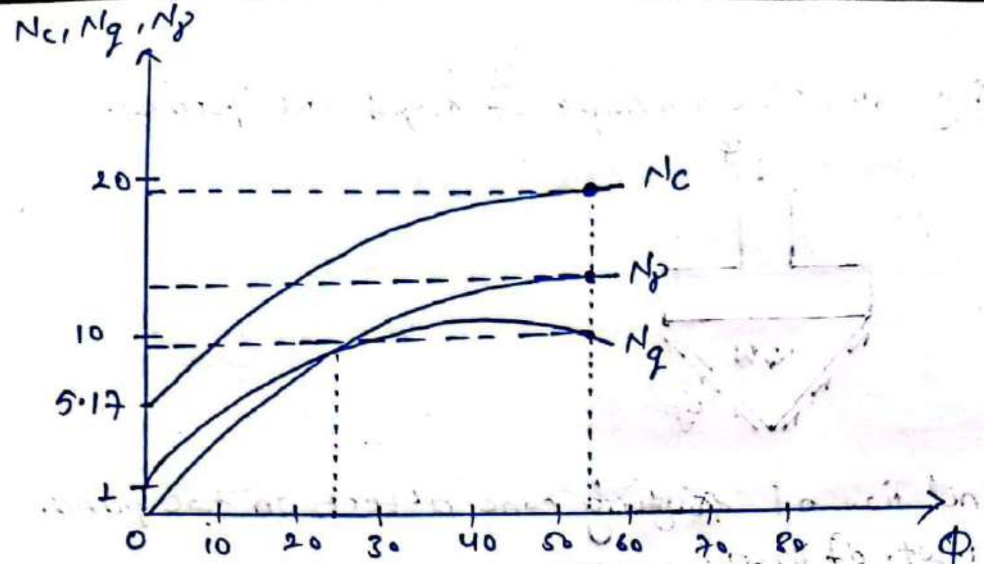
$$\alpha P_{p(\gamma)} - \tfrac{1}{4}B^2\gamma\tan\phi = \tfrac{1}{2}B^2\gamma N_\gamma$$

Finally, we get;

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

where, $q = \gamma D_f$.

Here, N_c, N_q and N_γ are the terzaghi's bearing capacity factors which depends on 'ϕ'.



Mathematically;

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

$$N_q = \left[\frac{a^2}{2\cos^2(45^\circ + \phi/2)}\right] \; ; \; a = e^{\left(\frac{3\pi}{4} - \phi/2\right)\cdot\tan\phi}$$

$$N_\gamma = \frac{1}{2}\left(\frac{K_p}{\cos^2\phi} - 1\right)\cdot\tan\phi$$; K_p = coefficient of passive pressure.

*: Factors affecting bearing capacity:

a. size of footing
b. shape of footing
c. position of GWT.
d. Type of soil.
e. Eccentricity of load.
f. Mode of shear failure.

a. size of footing:

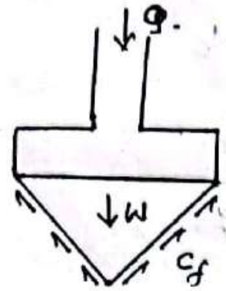
↳ Depending on shear failure criteria.

$q_u \propto B$.

$q_u \propto D_f$.

b. Shape of footing:

↳ Shape of footing affects on shape of rigid cone formed.



↳ The shape and size of rigid cone affects on two parts.

I. wt. of rigid cone.

II. cohesive force mobilized.

So, these can be corrected according to shape of footing.

- for rectangular footing:

$$q_u = c\left[1+0.3\frac{B}{L}\right]N_c + \gamma D_f N_q + \frac{1}{2}B\gamma N_\gamma\left[1-0.2\frac{B}{L}\right]$$

- for square footing:

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

- for circular footing:

$$q_u = 1.3cN_c + \gamma D_f N_q + 0.3B\gamma N_\gamma$$

c. Type of soil:

↳ for pure cohesive soil: clay.

$\phi = 0^\circ$, $N_c = 5.17$

$N_q = 1$

$N_\gamma = 0$

↳ for cohesionless soil: sand

$c = 0$

↳ for mixed soil: c-ϕ soil.

- superior property
- high bearing capacity.

d. Position of ground water table:

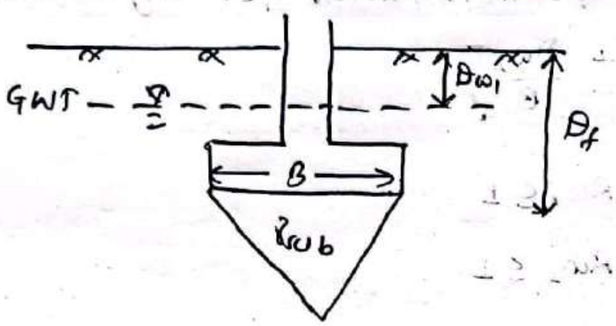
↳ GWT makes soil mass submerged.

$$\gamma_{sub} \simeq \frac{\gamma}{2}$$

So, rise of GWT, makes reduction in bearing capacity.

*: correction of bearing capacity according to position of GWT.

case(i): GWT is above base of footing.

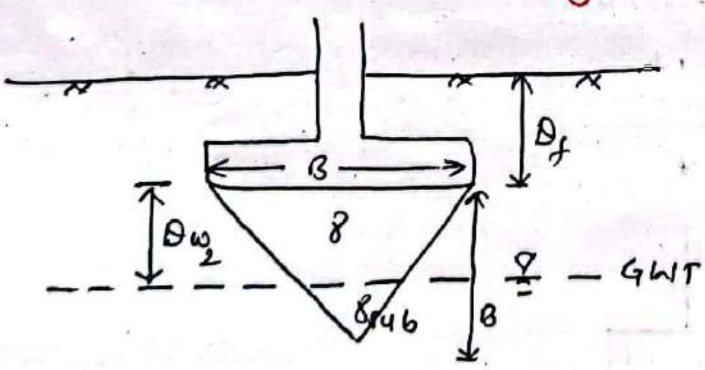


$\gamma_{avg} = R_{w_1} \times \gamma$; $R_{w_1} = 0.5\left[1 + \frac{D_{w1}}{D_f}\right]$

corrected eqn:

$$q_u = cN_c + \gamma D_f N_q \times R_{w_1} + \frac{1}{2} B \gamma_{sub} N_\gamma$$

case(ii): GWT is below base of footing.



$\gamma_{avg} = R_{w_2} \times \gamma$

; $R_{w_2} = 0.5\left[1 + \frac{D_{w2}}{B}\right]$

corrected equation:

$$q_u = cN_c + \gamma D_f N_q + \frac{1}{2} B \gamma N_\gamma \times R_{w_2}$$

↳

General expression for correction of GWT in bearing capacity:

$$q_u = cN_c + \gamma D_f N_q R_{w_1} + \frac{1}{2} B\gamma N_\gamma R_{w_2}$$

where,

R_{w_1} = GWT reduction factor for surcharge

$$= 0.5\left[1 + \frac{D_{w_1}}{D_f}\right]$$

R_{w_2} = GWT reduction factor for rigid cone

$$= \frac{1}{2}\left(1 + \frac{D_{w_2}}{B}\right)$$

Note:

$0 \le D_{w_1} \le D_f \qquad\qquad 0.5 \le R_{w_1} \le 1$

$\qquad\qquad\qquad\qquad \Rightarrow$

$0 \le D_{w_2} \le B \qquad\qquad 0.5 \le R_{w_2} \le 1$

Q. If GWT is on G.S. then, reduction factor are:

ⓐ 0.5, 0.5   ⓑ 0.5, 1

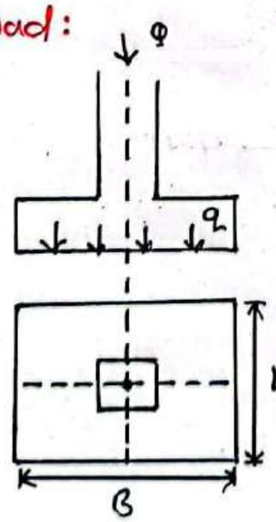
ⓒ 1, 0.5   ⓓ 1, 1

Q. If GWT is on footing, then reduction factor are:

ⓐ 1, 0.5   ⓑ 0.5, 0.5

ⓒ 0.5, 1   ⓓ 1, 1

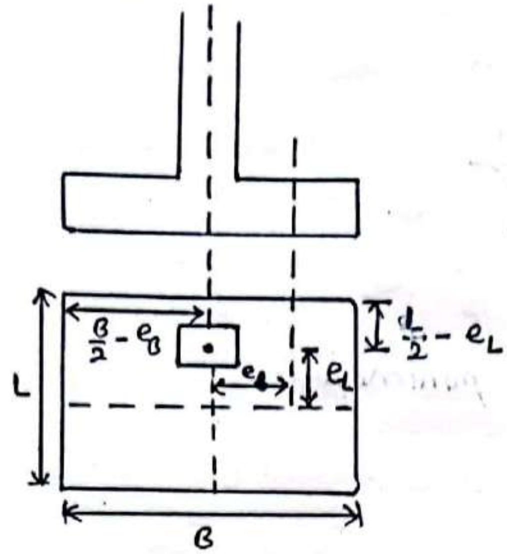
e. Eccentricity of load:

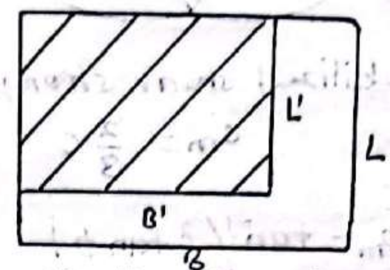


$$q_u = cN_c\left(1 + 0.3\frac{B}{L}\right) + \gamma D_f N_q + \frac{1}{2} B\gamma N_\gamma\left(1 - 0.2\frac{B}{L}\right)$$

CamScanner

for eccentric loading;





effective size of footing.

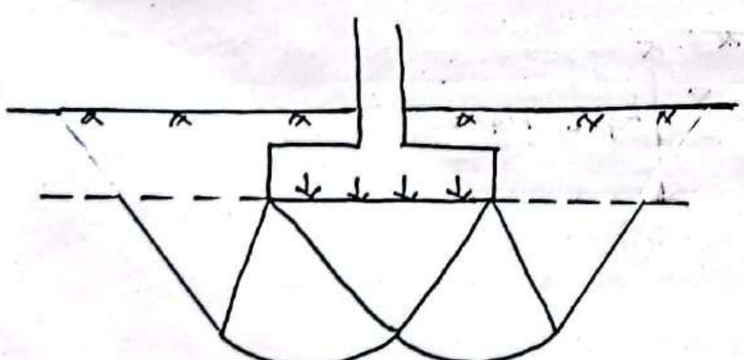
$B' = B - 2e_B$

$L' = L - 2e_L$

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

f. Mode of shear failure:

- General shear failure

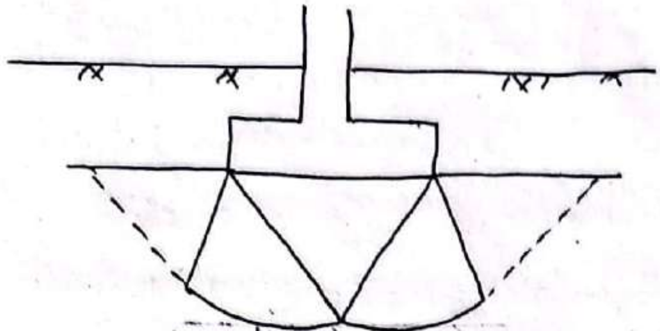


for strip;

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

; N_c, N_q & $N_\gamma \rightarrow$ from ϕ

• for local shear failure.



↳ Mobilized shear strength parameter,

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

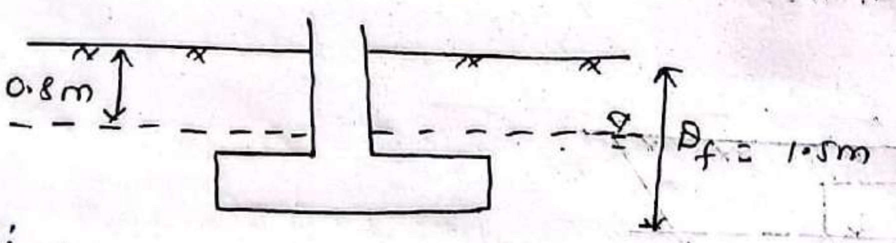
↳ $\phi_m = \tan^{-1}\left(\frac{2}{3}\tan\phi\right)$

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

; N_c', N_q' & N_γ' → from ϕ'.

Numerical:

Q. Design the size of footing: [square]:



Given:

$c = 20\ kN/m^2$

$\gamma = 16\ kN/m^3$

$\gamma_{sat} = 18\ kN/m^3$

$Q = 6000\ kN$.

$N_c = 14.8$, $N_q = 5.6$, $N_\gamma = 3.2$, $\phi = 25°$

solution:

$D_{w_1} = 0.8m$

Q. A strip footing is to be designed to carry a load of 1000 kN/m at a depth 1m in gravely sand. The angle of internal friction of soil is 40°. Determine the width of footing taking FOS = 3. Assume water table at foundation level:

Given, $\gamma = 17\ kN/m^3$, $\gamma_{sat} = 20\ kN/m^3$

$\phi = 40°$, $N_\gamma = 95$, $N_q = 64$.

solution:

$D_f = 1m$

$Q_s = 1000\ kN/m$

$FOS = 3$

$\gamma = 17\ kN/m^3$

$\gamma_{sat} = 20\ kN/m^3$

$N_\gamma = 95$

$N_q = 64$

$c = 0$ (for sand)

without considering GWT,

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

↳ When GWT is at foundation level, γ in terms of D_f remain at site i.e. above WT & γ in term N_γ becomes submerged.

considering GWT,

$$q_u = cN_c + \gamma D_f N_q R_{w_1} + \frac{1}{2} B \gamma_{sat} N_\gamma \times R_{w_2}$$

$$= 0 + 17 \times 1 \times 64 \times R_{w_1} + \frac{1}{2} \times B \times 20 \times 95 \times R_{w_2}$$

Here, $R_{w_1} = 0.5\left(1 + \frac{D_{w_1}}{D_f}\right) = 0.5\left(1 + \frac{D_f}{D_f}\right) = 1$

$R_{w_2} = 0.5\left(1 + \frac{D_{w_2}}{B}\right) = 0.5\left(1 + \frac{0}{B}\right) = 0.5$

$$q_u = 17 \times 1 \times 64 \times 1 + \frac{1}{2} \times B \times 20 \times 95 \times 0.5$$

$$q_u = 1088 + 475B.$$

Again,

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

$= 1088 + 475B - 17\times 1$

$= 1071 + 475B$

$q_{ns} = \frac{q_{nu}}{FOS} =$

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

$= \frac{1071+475B}{3} + 17\times 1$

$= 357 + 158.33B + 17\times 1$

$= 374 + 158.33B$

Now, for strip footing,

$q_s \times B = Q_s$

$(374 + 158.33B)\times B = 1000$

on solving, we get;

$B = 1.595\ m \simeq 1.60\ m$ &

PSC 2079-10-14 [Civil 7th central]

Q. A square footing is to be designed/constructed at a depth of 3.6m below the ground level on a sandy clay for which cohesion is 0.575 kg/cm² and density is 1.73 g/cm³. The total load applied on the soil is 375 tonnes uniformly distributed over the area of contact. Find the size of footing using a load factor of 3. Take relevant value of factors as $N_c = 10$, $N_q = 4$ and $N_\gamma = 2$

Solution:

Shape: square

depth $(D_f) = 3.6\ m$

cohesion $(c) = 0.575\ kg/cm^2 = \frac{0.575\times 10\times 10^{-3}}{(\frac{1}{100})\times(\frac{1}{100})} = 57.5\ kN/m^2$

density (ρ) = $1.73\ g/cm^3$ = $17.3\ kN/m^3$

total load (Q) = 375 tonnes.

$$= \frac{375 \times 10^3 \times 10}{10^3} = 3750\ kN.$$

$N_c = 10,\ N_q = 4,\ N_\gamma = 2$

Now,

for square footing

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

$q_u = 1.3 \times 57.5 \times 10 + 17.3 \times 3.6 \times 4 + 0.4 \times B \times 17.3 \times 2$

$q_u = 747.3 + 249.12 + 13.84 B$

$q_u = 996.42 + 13.84 B$

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

$= 996.42 + 13.84 B - 17.3 \times 3.6$

$= 934.14 + 13.84 B$

$$q_{ns} = \frac{q_{nu}}{FOS}$$

$$= \frac{934.14 + 13.84 B}{3} = 311.38 + 4.613 B$$

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

$= 311.38 + 4.613 B + 17.3 \times 3.6$

$= 373.66 + 4.61 B$

Now

$Q_s = q_s \times (B \times B)$

$3750 = (373.66 + 4.61 B)(B \times B)$

on solving, we get;

$B = 3.108\ m \simeq 3.11\ m$

$\therefore$ Adopt square footing of size $3.11\ m \times 3.11\ m$

Q: Types of Bearing capacity:

a. Ultimate bearing capacity (q_u):

- Gross pressure on soil below foundation at which soil fail in shear.
- calculated from terzaghi's expression of bearing capacity.

b. Net ultimate bearing capacity (q_{nu}):

- Net pressure on soil through foundation upto shear failure.

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

c. Net safe bearing capacity (q_{ns}):

- Net pressure on soil through foundation upto shear failure considering FOS.
- Net pressure on soil through foundation without shear failure

$$q_{ns} = \frac{q_{nu}}{FOS} \; ; \; FOS \geq 2.5$$

d. Safe bearing capacity: (q_s):

- Gross pressure on soil without shear failure.

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

e. Net permissible settlement pressure: (q_{np})

- Net pressure on soil through foundation at which settlement of soil is within permissible limit.
- calculated from settlement analysis.

f. Allowable bearing capacity: (q_{na}):

- Net pressure on soil through foundation under which soil neither fail in shear nor gives larger settlement than permissibl limit.

$$q_{na} = q_{ns} \text{ if } q_{ns} < q_{np}$$

$$q_{na} = q_{np} \text{ if } q_{np} < q_{ns}$$

Settlement:

- Downward movement of footing due to compression of supporting soil under the effect of applied stress (load) is known as settlement.
- settlement should be within permissible limit, otherwise it will give adverse effect on structure.

Effects of settlements:

a. Loss of aesthetic value:

- crack in floor, plaster
- distortion of door and window of masonary structure.
-

b. Crack of utility connection pipe:

- damage of water supply pipe.
- damage of sewerage pipe.
- de-alignment of machine connection.

c. Structural collapse:

- crack in beam
- crack in column
- crack in foundation
- crack in slab etc.

Types of settlements:

1. Depending on stage:

a. Immediate settlement:

- Due to elastic compression of soil.
- occurs within a week of loading.
- small settlements.

b. Primary settlements:

↳ compression of water void due to expulsion of water.

↳ large settlement on clay.

↳ takes long time.

c. Secondary settlement:

↳ Due to plastic rearrangements of soil particles.

↳ small settlements.

↳ occurs for very large amount of time.

2. Depending on nature of settlement / mode of settlement:

a. Uniform settlement:

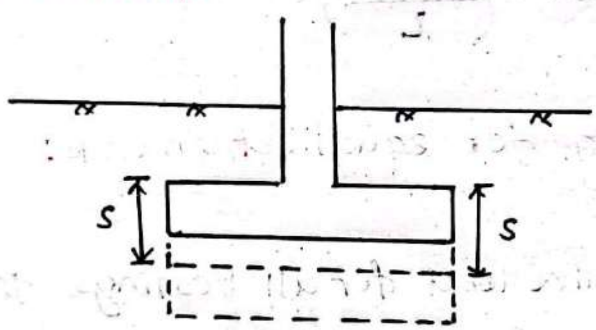


fig: uniform settlement.

↳ Equal settlement on all parts of footing.

↳ If excessive, it creates cracks on utility connection pipes.

b. Differential settlement:

↳ Unequal settlements at different parts of a foundation structure.

↳ Effects: structural collapse.

• Types of differential settlements:

a. Tilt:

↳ settlement is higher at one side of structure.

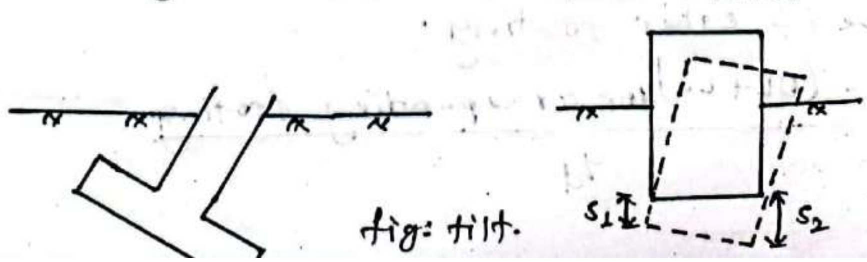


fig: tilt.

$S_2 > S_1$

6. Angular distortion:

↳ occurs when two foundation supporting column/wall settles unequally.

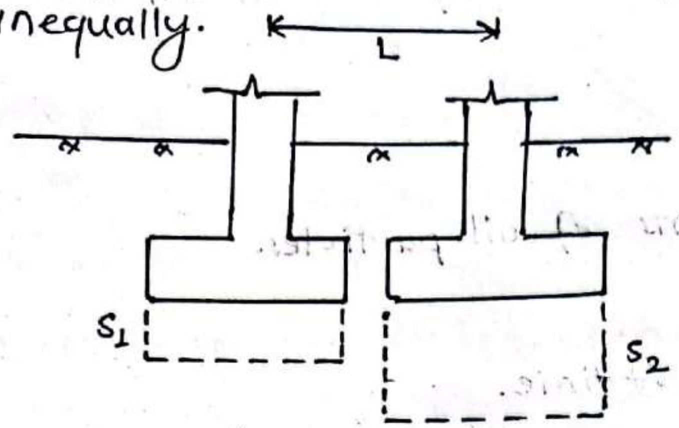


fig: angular distortion.

$$\text{Angular distortion } (\delta) = \left(\frac{S_2 - S_1}{L}\right)$$

Proportioning of footing for equal settlements:

Steps:

1. Calculate dead load and live load for all footings from super structure.
2. Identify the governing footing.
   ↳ footing having maximum live load to dead load ratio.
3. Find the area of governing footing.
   $$A_g = \frac{(LL + DL)_{\text{governing footing}}}{\text{Allowable bearing capacity } (q_a)} = \text{----} \quad (\text{adopt, } A_g)$$
4. calculate the equivalent bearing pressure for other footing.
   $$q_d = \frac{\left(\frac{DL + LL}{2}\right)_{\text{governing footing}}}{A_{g\,(\text{adopted})}}$$
5. Design size of other footing.
   $$A = \frac{(DL + LL)_{\text{for corresponding footing}}}{q_d}$$

Measurement of settlement of foundation:

↳ settlement of foundation / footing can be measured by:

a. Change in void ratio (Δe)

b. Change in a_v or m_v.

c. Change in C_c or C_r.

Mat

2.2.5 Design of foundation:

*: Design of spread footing:

Q: Design of combined footing:
↳ structural part बाट हेर्नै!!!

Q: Design of strap footing:

*: Design of mat foundation:

Introduction:

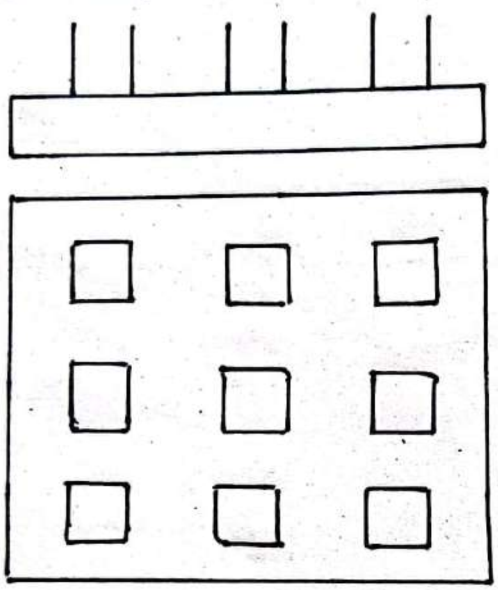
↳ single slab supporting large number of column in many rows is known as mat foundation.

Use:

structure: Heavy structure like commercial complex.

condition: if isolated footing covers > 50% of plinth area.

soil: compressive soil layer where other foundation doesnot perform well.



*: Sorting of mat foundation:

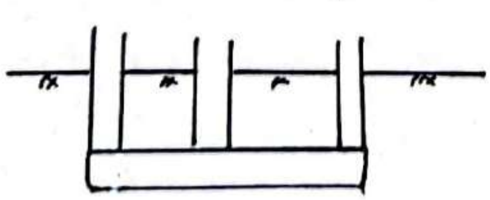
a. Based on approach of construction:

1. Backfilled mat:

↳ Excavated soil is backfilled after construction of mat.

↳ Increase in stress on soil below mat,

$$\Delta\bar{\sigma} = \frac{Q}{A} \; ; \; Q = \text{total load}, \; A = \text{area of mat.}$$

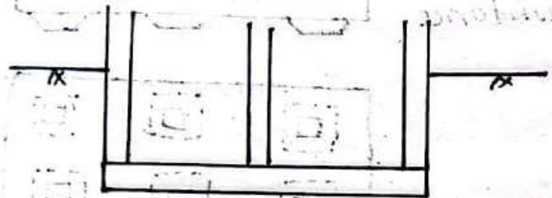


2. compensated mat:

- excavated soil is not backfilled after construction.
- basement use approach.
- the certain portion of stress imposed by structure (Q/A) is distress due to excavation ($= \gamma D_f$).
- increase in stress on soil.

$$\Delta \bar{\sigma} = \frac{Q}{A} - \gamma D_f .$$

- mostly mat is constructed in this approach.



c. floating mat:

- stress imposed by structure ($\frac{Q}{A}$) is fully balanced by distress due to excavation. (γD_f).

$$\frac{Q}{A} - \gamma D_f = 0$$

$$\Rightarrow \frac{Q}{A} = \gamma D_f .$$

- stability of mat doesnot depend on shear strength of soil below mat. so the mat constructed in this approach is floating mat.
- This is theoretical achievement, but practically not reliable.

*: Types of mat foundation:

a. uniform slab mat.

b. slab thickened under column mat.

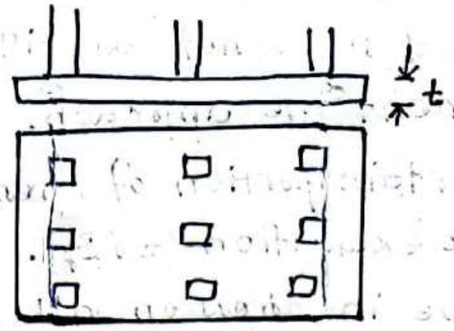
c. two way beam and slab mat.

d. rigid framed slab mat.

e. mat on group pile.

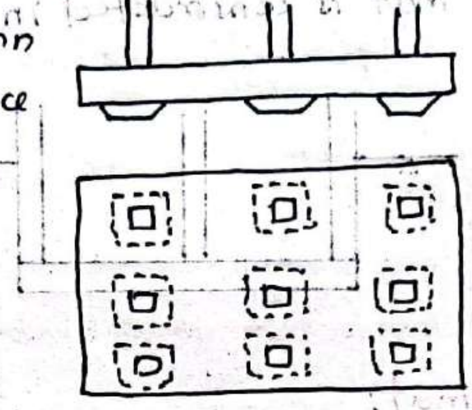
a. Uniform slab mat:

- slab of uniform thickness is provided to support all column.
- easy for design and construction.



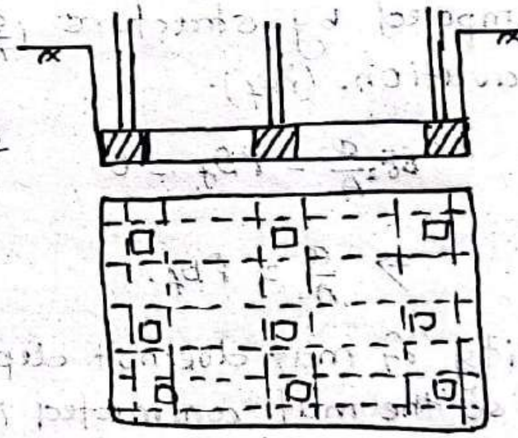
b. slab thickened under column mat:

- slab is thickened below column to increase shear force resistance capacity.



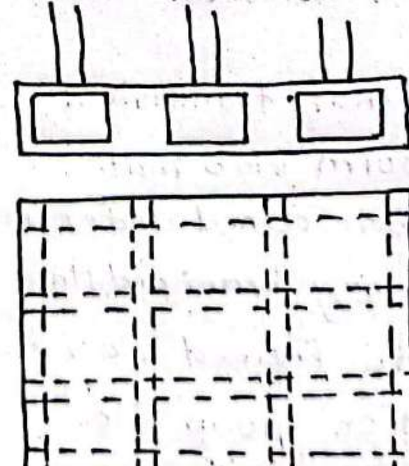
c. Two way beam and slab mat:

- first two way beam are constructed.
- slab is constructed on two way beams.
- columns are erected from the junction of crossings of two ways beams.



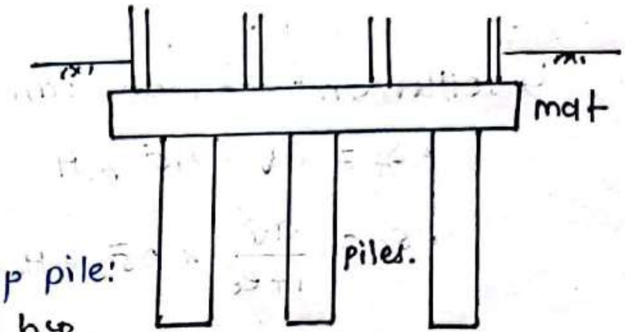
d. Rigid Framed slab mat:

- large thickness hollow slab is created by rigid connection of two horizontal slab at top and bottom and series of two way vertical slab in between two horizontal slab.
- Hollow boxes in between slab is filled with brick pieces.



e. Mat on group pile:

↳ Mat is constructed on group pile of m×n layout pattern.



Note:

* Pile foundation
  ↳ all load is taken by piles.

* Mat on group pile:
  ↳ partial load by mat and piles.

*: Bearing capacity of mat:-

• For cohesionless soil: sand, gravel

↳ Insitu test is more reliable: standard penetration test.

*from shear failure criteria:

$$q_{ns} = 0.22 N^2 B R_{w_1} + 0.54(100+N^2) D_f R_{w_2}$$

where;

N = SPT value. (corrected) R_{w_1} & R_{w_2} are water table correction factor

B = width of mat.

*. from settlement criteria

$$q_{np} = 17.5 (N-3) R_w \rightarrow \text{for 25 mm settlement.}$$

$$q_{np} = 22 (N-3) R_w \rightarrow \text{for 50 mm settlement.}$$

where; q_{ns} & q_{np} are in kN/m^2.

N = SPT value (corrected)

R_w = water table correction factor.

• For cohesive soil: clay

↳ lab test is more reliable.

*: from shear failure criteria:

$$q_{nu} = c N_c \left(1 + 0.2 \frac{B}{L}\right)\left(1 + 0.2 \frac{D_f}{B}\right)$$

where; $N_c = 5.17$

$$FOS = \frac{q_{nu}}{\Delta \bar{\sigma}}$$

where; $\Delta \bar{\sigma} = q/A - \gamma D_f$ (for compensating mat)

$\Delta \bar{\sigma} = q/A$ (for backfilled mat)

*! Settlement analysis of mat:

↳ settlement of mat can be found out using:

- $S_f = m_v \times \Delta\bar{\sigma} \times H$
- $S_f = \frac{a_v}{1+e_0} \times \Delta\bar{\sigma} \times H$
- $S_f = \frac{C_c}{1+e_0} \times H \times \log\left(\frac{\Delta\bar{\sigma} + \bar{\sigma}_0}{\bar{\sigma}_0}\right)$

; $\Delta\bar{\sigma} = Q/A - \gamma D_f$ (for compensating mat)

$\Delta\bar{\sigma} = \frac{Q}{A}$ (for backfilled mat)

Design of mat foundation:

↳ Mat foundation is designed considering two aspects:

a. Geotechnical design.

b. Structural design.

a. Geotechnical design:

↳ selection of
- size: $B \times L$ → size floor area ले govern गर्छ।
- depth: D_f → floor height ले govern गर्छ।

(depth generally 3 to 4 m राखिन्छ)

Steps:

↳ Assume depth (D_f) and size ($B \times L$).

↳ shear failure criteria: check

$$\Delta\bar{\sigma} \leq q_{ns} \text{ (ok)}$$

$$FOS = \frac{q_{nu}}{\Delta\bar{\sigma}} \geq 2.5 \text{ (ok)}$$

otherwise increase D_f or $B \times L$ or both.

↳ settlement criteria:

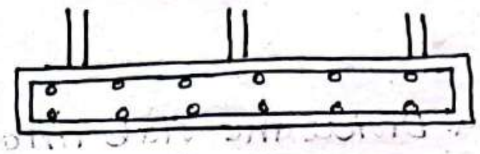
$$\Delta\bar{\sigma} \le q_{np} \text{ (OK)}$$

where; $\Delta\bar{\sigma} = \frac{q}{A} - \gamma D_f$

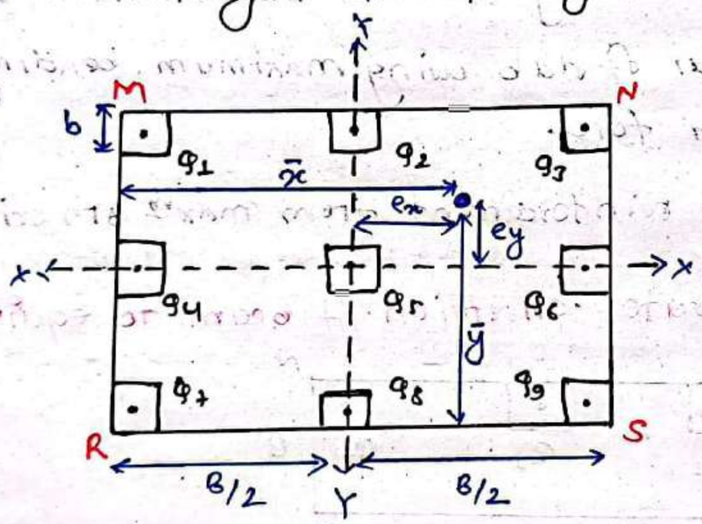
otherwise increase size $(B \times L')$.

6. Structural design:

↳ selection of
- thickness of slab.
- reinforcement area.



↳ Methods:
- convectional method (Rigid beam method)
- FEM : flexible beam method
  - ↳ using modulus of sub-grade reaction.



Steps:

1. Calculate CG of load from column taking moment about MR face.

$$\bar{x} = \frac{\Sigma \text{moment}}{\Sigma q_i} \quad ; \; q_i = q_1 + q_2 + \dots + q_9$$

taking moment about RS face.

$$\bar{y} = \frac{\Sigma \text{moment}}{\Sigma q_i}$$

2. Calculate the eccentricity:

$$e_x = \bar{x} - B/2$$

$$e_y = \bar{y} - L/2$$

3. calculate stress developed below mat at any point.

$$q = \frac{Q}{A} \pm \frac{Q \times e_x \times x}{I_{yy}} \pm \frac{Q \times e_y \times y}{I_{xx}}$$

where; $I_{yy} = \frac{LB^3}{12}$ & $I_{xx} = \frac{BL^3}{12}$

4. Divide the slab into no. of beam in both x and y-direction and convert each beam into equivalent beam.

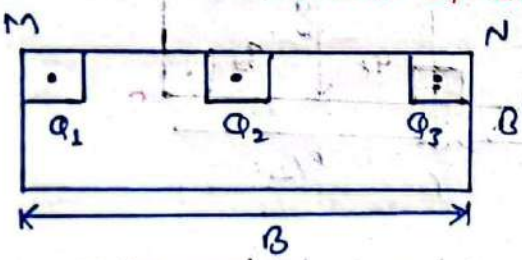
5. Draw SFD and BMD for each equivalent beam.

6. Find maximum bending moment and maximum shear force.

7. Design thickness of slab using maximum bending moment and maximum shear force.

8. Determine area of reinforcement from max^m BM criteria.

Example to illustrate conversion of beam to equivalent beam.



↳ calculate the stress below M and N point.

- For point M

$x = B/2$ and $y = +L/2$

$\therefore q_m = \text{----}$

- similarly for point N.

$x = B/2$ & $y = L/2$

$\therefore q_n = \text{---}$

$$\therefore q_{avg} = \frac{q_m + q_s}{2}$$

↳ Average load on beam

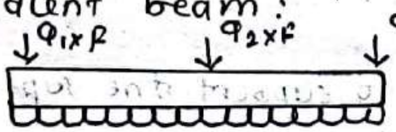
$$Q_{avg} = \frac{1}{2} \times (\text{downward} + \text{upward})$$

$$= \frac{1}{2} (Q_1 + Q_2 + Q_3 + q_{avg} \times B \times B')$$

↳ Average stress on beam

$$\bar{q} = \frac{Q_{avg}}{B \times B'}$$

$\therefore$ Equivalent beam:



; $F = \frac{Q_{avg}}{Q_1 + Q_2 + Q_3}$ is equivalent factor.

Numerical:

Q. A mat 12m × 16m is constructed at 3.5m depth on clay to support 50000 kN load. The site consists of clay of 70 kN/m² unconfined compressive strength. Comment about the foundation. $\gamma = 17$ kN/m³.

Hints:-
↳ comment about foundation खालको questions को लागि!!
- calculate FOS
- if FOS > 2.5 (ok)
- if not foundation should be redesigned.

Solution:

soil type: clay

so, $\phi = 0°$ & $c = \frac{q_u}{2}$

$B \times L = 12m \times 16m$

$D_f = 3.5m$

$q_u = 70$ kN/m² $\therefore c = \frac{70}{2} = 35$ kN/m²

Total load (Q) = 50,000 kN.

$\gamma = 17$ kN/m³

Now,

$$q_{nu} = cN_c\left(1+0.2\frac{D_f}{B}\right)\left(1+0.2\frac{B}{L}\right)$$

Here, $N_c = 5.17$ for clay

$$q_{nu} = 35 \times 5.17\left(1+0.2\times\frac{8.5}{12}\right)\left(1+0.2\times\frac{12}{16}\right)$$

$$= 220.231\ kN/m^2$$

$$\frac{q}{A} - \gamma D_f = \frac{50000}{12\times 16} - 17\times 3.5 = 200.91\ kN/m^2$$

$$FOS = \frac{q_{nu}}{\frac{q}{A}-\gamma D_f} = \frac{220.231}{200.91} = 1.096 < 2.5 \text{ (Not ok)}$$

∴ Foundation base is not stable to support the superimposed load.

Rather size should be increased to bear superimposed load.

Q. A mat 12m x 16m size is constructed at 4m depth to support 20,000 kN load. calculate FOS if site consists of clay with unconfined compressive strength 60 kN/m².
$\gamma = 17\ kN/m^3$
comment about the foundation:

Hints:-
- if FOS > 2.5 foundation is safe enough to bear superimposed load.

solution:

$B\times L = 12m \times 16m$

$D_f = 4m$

$q = 20000\ kN$

soil type: clay

$\phi = 0°$

$N_c = 5.17$

$q_u = 60\ kN/m^2$

$\gamma = 17\ kN/m^3$

$c = \frac{q_u}{2} = 30\ kN/m^2$

Now, $q_{nu} = cN_c\left(1+0.2\frac{D_f}{B}\right)\left(1+0.2\frac{B}{L}\right)$

$$= 30\times 5.17\left(1+0.2\times\frac{4}{12}\right)\left(1+0.2\times\frac{12}{16}\right)$$

$$= 190.256 \text{ kN/m}^2$$

Also,

$$\frac{Q}{A} - \gamma D_f = \frac{20000}{12\times16} - 17\times4 = 36.167 \text{ kN/m}^2$$

So, $$FOS = \frac{q_{nu}}{\frac{Q}{A}-\gamma D_f} = \frac{190.256}{36.167} = 5.26 > 2.5 \text{ (ok)}$$

∴ Foundation is strong enough to bear superimposed load.

Additional information:

↳ Permissible settlement of isolated and mat footing:

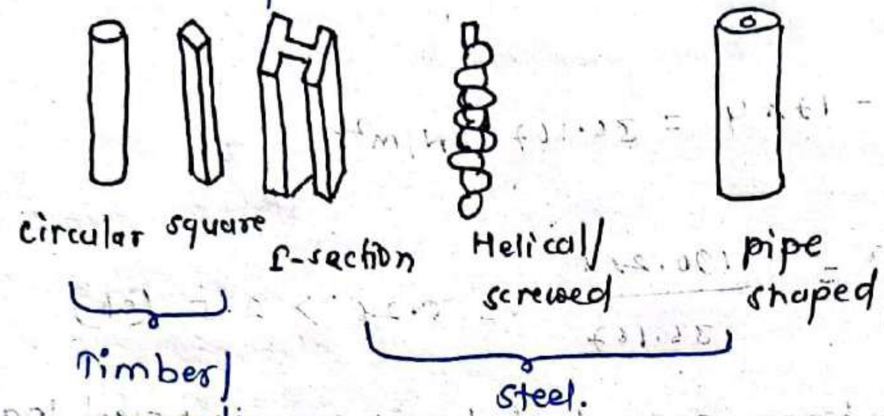
Isolated footing	Mat foundation.
clay: 40mm	clay: 40-65 mm
sand: 65mm	sand: 65-200mm

*: Design of Pile Foundation:

Introduction:

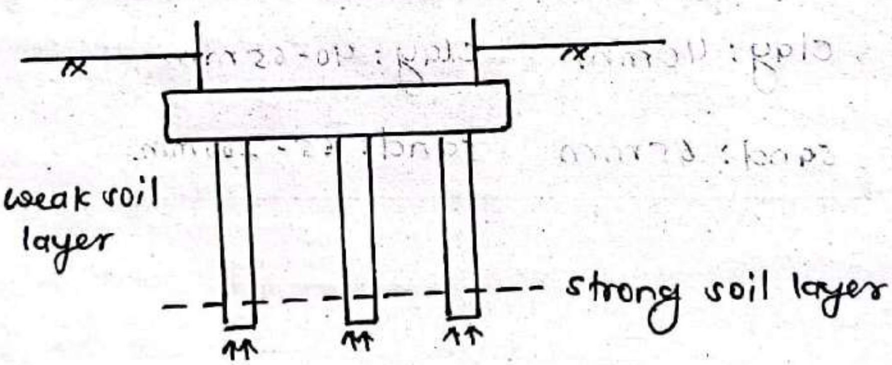
↳ A type of deep foundation having small cross section and slender body.

Possible shapes:

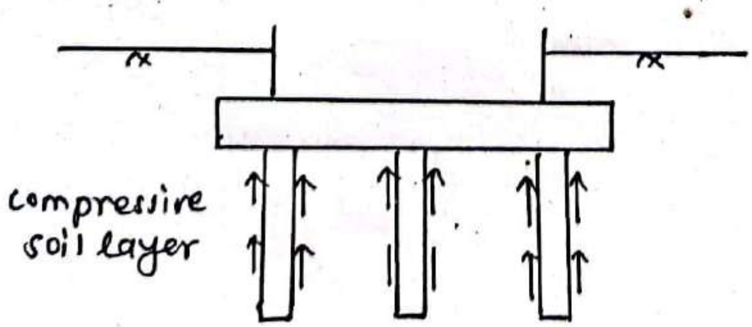


*: functions/circumstances for use of pile: Purpose.

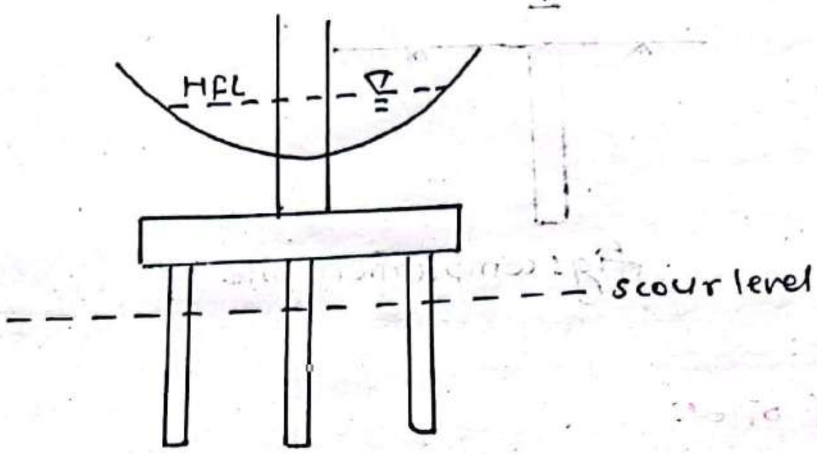
a. If high strength soil layer is found in large depth, to transfer load to that layer: end-bearing pile.



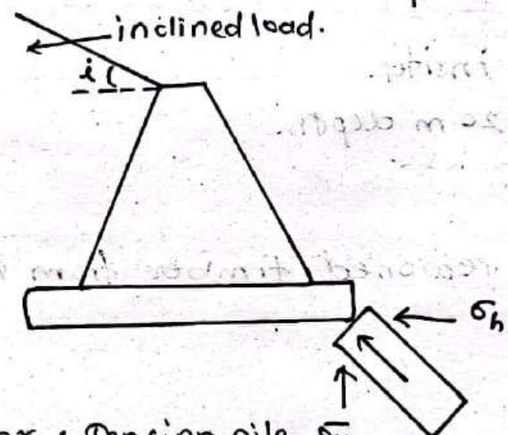
b. If compressible soil layer is found to be extended to very large depth, to transfer load through skin resistance: friction pile.



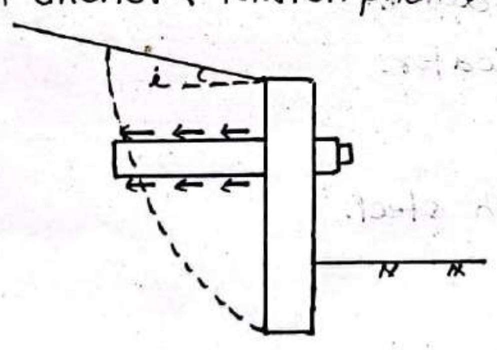
c. if scour depth is very high, to transfer the load below scour level.



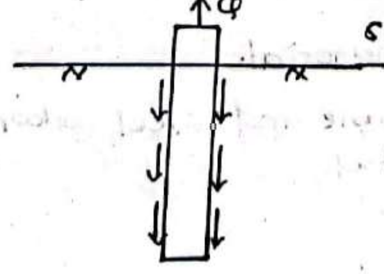
d. To resist the inclined load: Batter pile.



e. To support anchor : Tension pile.



f. To transfer the uplift load : chimney, transmission tower, hydraulic structure etc.



g. For deep compaction of sand: compaction pile:

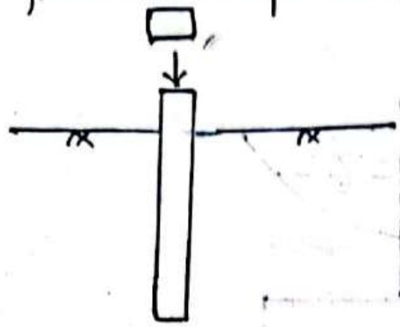


fig: compaction pile

*: Types of piles:

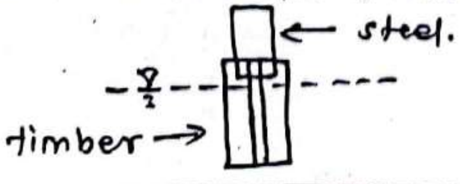
a. Depending on materials:

1. concrete pile
   - ↳ RCC element
   - ↳ Pre-cast or cast insitu.
   - ↳ Best use: upto 20 m depth.

2. Timber pile
   - ↳ Made of well seasoned timber from matured tree or trunk of tree.
   - ↳ Driven pile.
   - ↳ Best use: upto 16m depth.
   - ↳ Most durable below water.

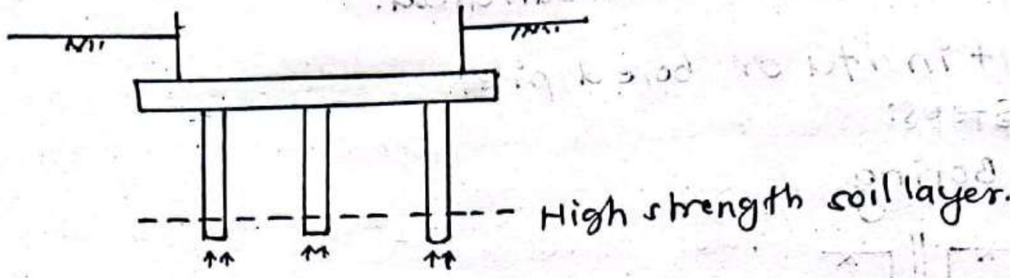
3. steel pile.
   - ↳ made of high strength steel.
   - ↳ driven pile.
   - ↳ Best use: upto 40 m.

4. Composite pile:
   - ↳ made of two or more material.
   - ↳ timber below water table and steel above water table.

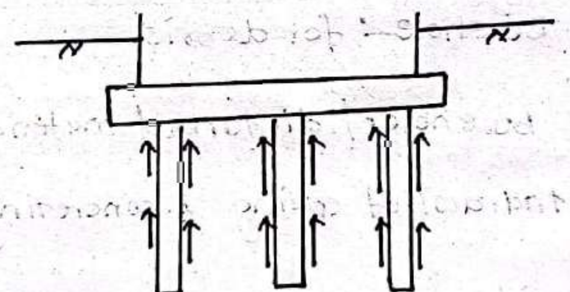


b. Depending on load transfer mechanism:

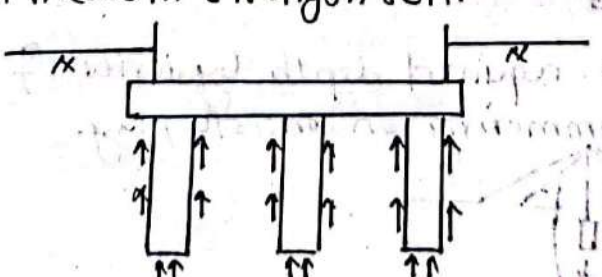
1. End / tip / base bearing pile.
   - transfers load through base resistance between soil and pile.
   - small size pile
   - Use: in strong soil layer.



2. Friction Pile:
   - transfers load through ~~both base resistance and~~ skin resistance. between soil and pile.
   - ~~large size~~ pile
   - Use: in compressive soil layer.



3. Both end bearing and friction pile:
   - transfer load through both base resistance and skin resistance
   - medium size pile
   - Use: in medium strength soil.



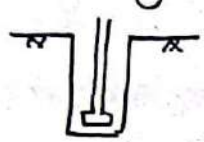
C. Depending on construction methods:

1. Pre-cast or driven pile:
   - ↳ pile is casted in workshop.
   - ↳ drived in filled by hammering.
   - ↳ best load carrying capacity.
   - ↳ not usuable in urban areas.

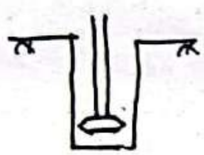
2. Cast insitu or bored pile:

   Steps:

   i. Boring



   ii. Under reaming



   iii. Inspection of borehole : for debris.

   iv. cleaning of boreholes / disturbed materials.

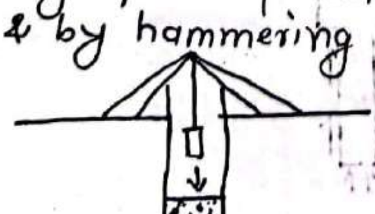
   v. Casting : withdrawal of casing & concreting in many steps.

3. Driven and cast insitu pile :

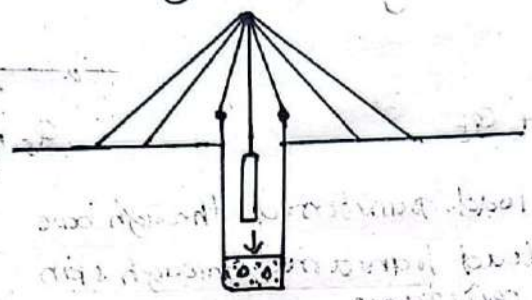
   Steps:

   i. casting of bottom plug in one steel casing

   ii. Drive casing upto required depth by series of connection of casing & by hammering on concrete plug.



iii. Removal of bottom casing plug from casing by hammering with hold of casing.



[ ] ← Bottom plug after removal.

iv. Reinforcement.

v. casting: withdrawal of casing and concreting in many stages.

d. Depending on soil displacement during construction:

1. large soil displacement pile:
   ↳ driven pile.

2. Small soil displacement pile:
   ↳ steel: helical pile

3. No soil displacement pile:
   ↳ Bored or cast insitu.

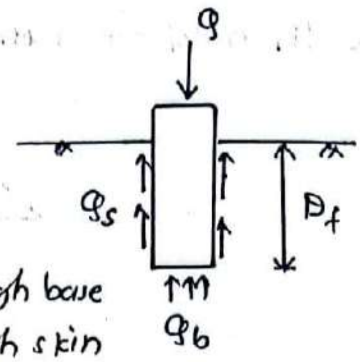
*! Load carrying capacity of piles:

↳ load carrying capacity of piles can be determined by following methods:

- ✓ a. static formula method
- b. Dynamic formula method (x)
- c. using insitu test (x)
  - (i) using N-value (SPT)
  - (ii) using q_c (cone resistance) (CPT).
- ✓ d. pile load test.

a. Static formula method:

↳ considering both end bearing and friction pile.



Ultimate load $(Q_u) = Q_b + Q_s$

where; Q_b = load transferred through base

Q_s = load transferred through skin resistance

Safe load or allowable load (Q)

$$Q = \frac{Q_u}{FOS} \; ; \; FOS \geq 2.5$$

- Calculation of Q_b:

$$Q_b = q_b \times A_b$$

where; A_b = x-sectional area of pile

q_b = base resistance

from terzaghi's theory;

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

since size of pile (B) is very small i.e. $\frac{1}{2} B \gamma N_\gamma$ can be neglected.

$$\therefore q_u = cN_c + \gamma D_f N_q$$

$$q_b = q_u - \gamma D_f$$

$$q_b = cN_c + \gamma D_f N_q - \gamma D_f$$

$$q_b = cN_c + \gamma D_f (N_q - 1) \text{ ----} \circledast$$ General form.

- for pure clay, $\phi = 0$

$\therefore N_q = 1$

$\therefore q_b = cN_c$

where; c = cohesion of soil at base level.

N_c = 9 (always)

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- for coherionless soil : sand

$$c = 0$$

$$q_b = \gamma D_f (N_q - 1)$$

; N_q depends on ϕ.

- Calculation of Q_s:

$$Q_s = q_s \times A_s$$

where; A_s = surface area of pile.

q_s = skin resistance.

$$q_s = \alpha \bar{c} + k \bar{\sigma_v} \tan\delta \text{ ------* General form.}$$

- For pure clay : $\phi = 0$ & $\delta = 0$

$$\therefore q_s = \alpha \bar{c}$$

- For coherionless soil : sand

$$c = 0$$

$$q_s = k \bar{\sigma_v} \tan\delta$$

where; $k = 1$ for very loose to medium dense

$k = 2$ for dense to very dense

$\delta = 0.75\phi$ for concrete

$\delta = 0.67\phi$ for timber

$\delta = 21°$ for steel

$\bar{\sigma_v}$ = avg. vertical stress across depth.

b. Pile load test:

Objective:

↳ to determine the allowable load for pile.

↳ to determine settlement under design load.

Types of pile load test:

a. Initial test:

↳ done on model pile before design of pile.

Objective: to assist the design.

Load: upto 2.5 x design load.

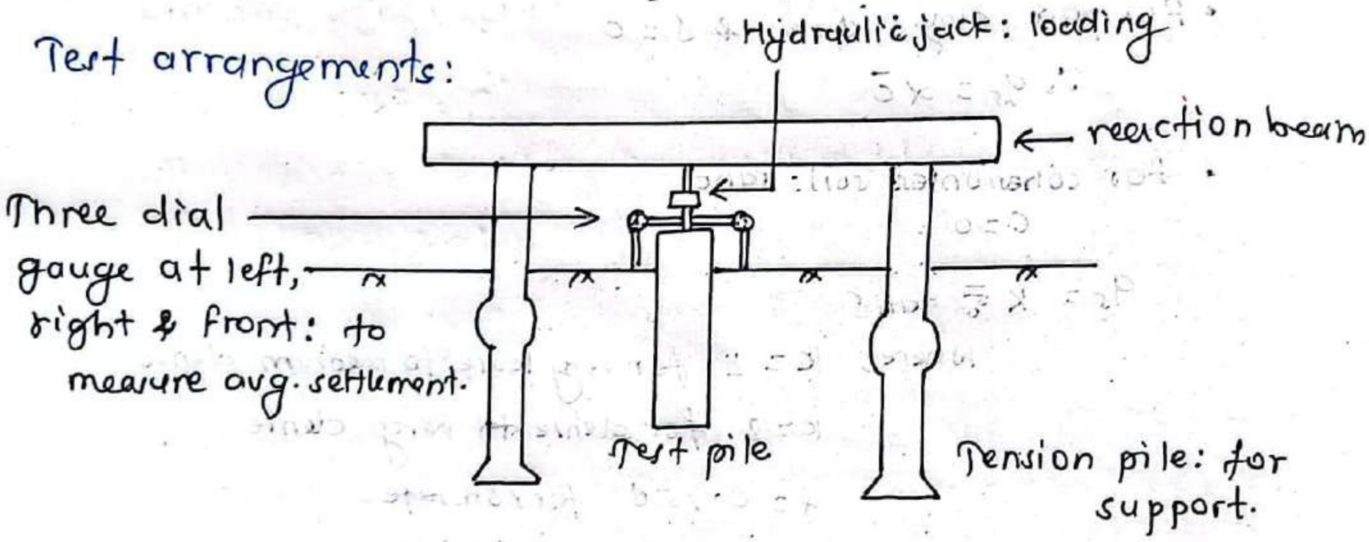
b. Routine test:

↳ done on working pile after construction.

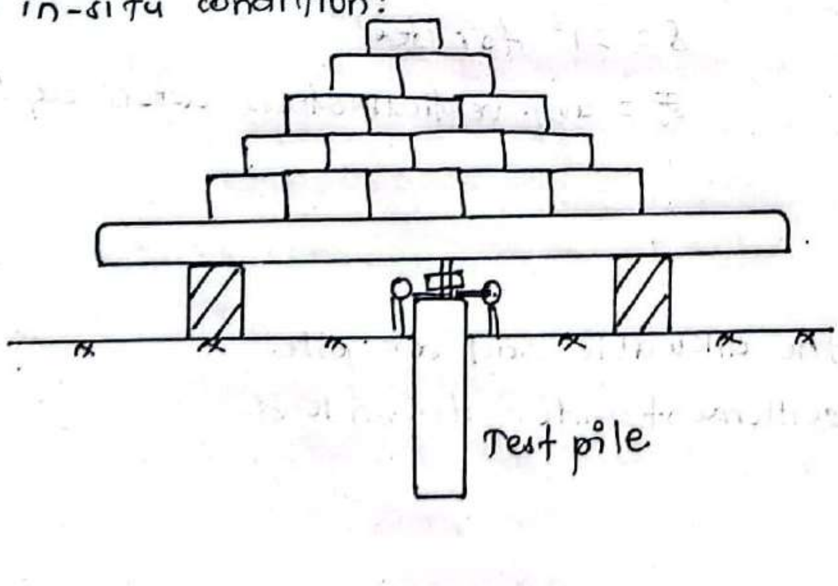
Objective: to verify / check the design & construction.

load : upto 1.5 x design load.

Test arrangements:



But in-situ condition:



Procedure:

• loading

1st test load : 25% of design load

In each stage of loading, increase 25% of design load.

Last test load :

• initial test: 2.5 x design load.

• routine test: 1.5 x design load.

In each stage of loading, dial gauge reading at time pattern:

0 sec, 15 sec, 30 sec, 1 min, 2 min, 4 min, 8 min, 16 min, 30 min, and each hour interval until settlement rate reduces to

→ 0.1 mm/hr for sand.

→ 0.2 mm/hr for clay.

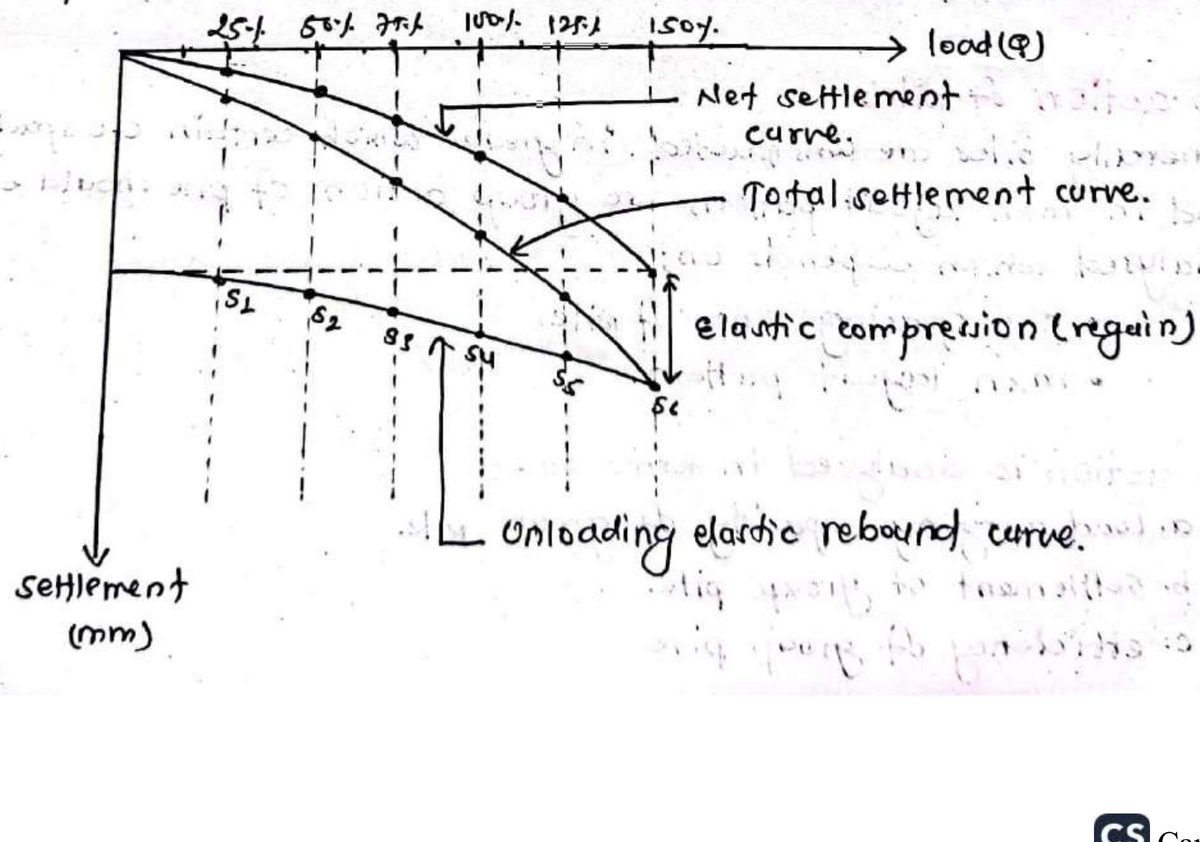
• Unloading:

↳ stage by stage with 25% decrement in each stage

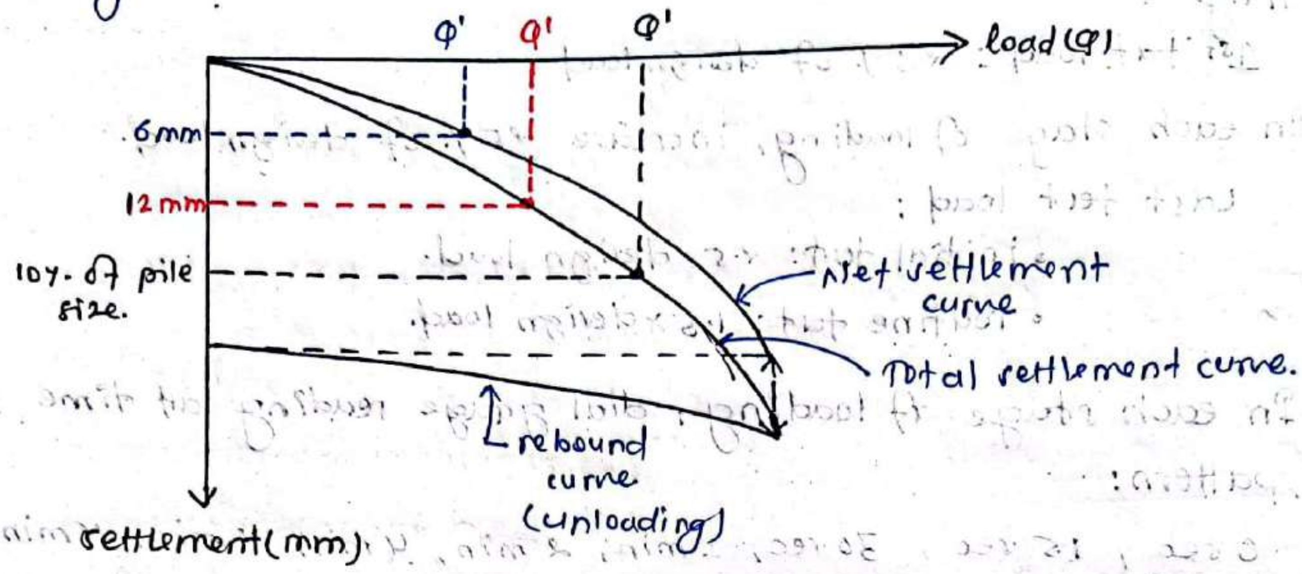
↳ dial gauge observation: to determine elastic rebound in same time pattern as in loading.

Result:

↳ plot of load vs settlement.



Analysis:



- (i) Allowable load (for 6mm settlement)

$$Q_a = \frac{2}{3} Q'$$

(ii) Allowable load (for 12mm settlement)

$$Q_a = \frac{2}{3} Q'$$

(iii) Allowable load (for 10% of pile size)

$$Q_a = \frac{1}{2} Q'$$

↳ Smallest of (i), (ii) and (iii) is allowable load for pile.

*! Group action of pile:

↳ Generally piles are constructed in groups which certain c-c spacing and in m×n layout pattern. so group action of pile should be analysed which depends on;

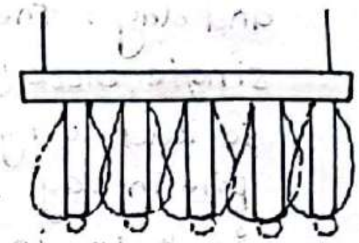
- c-c spacing & size of pile.
- m×n layout pattern.

Group action is analysed in terms of:

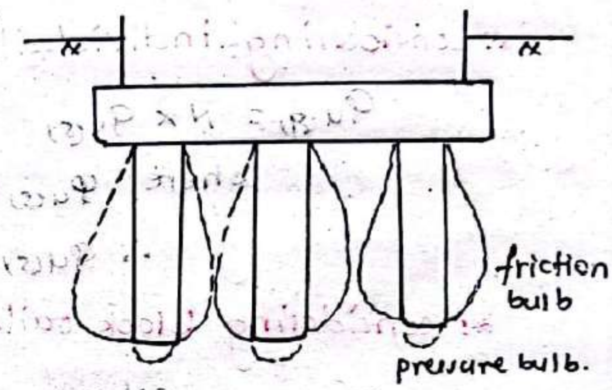
a. Load carrying capacity of group pile.
b. Settlement of group pile.
c. efficiency of group pile.

Effect of c-c spacing on group action of pile:

- Small c-c spacing:
  - ↳ Friction bulb overlaps
  - ↳ reduction in load carrying capacity of group piles.



- Large c-c spacing
  - ↳ Pile act as individual.
  - ↳ thickness of pile cap increases with increase of c-c spacing.
  - ↳ More efficient but costly.



*: Minimum c-c spacing:

- Friction pile: perimeter of pile.
- End bearing pile: 2 x size of pile.
- Helical pile: 1.5 x diameter.

q. Load carrying capacity of group pile:

- In cohesionless soil: sand.
  - ↳ Pile act as individually, so load carrying capacity is governed by individual failure criteria.

$$Q_{u(g)} = N \times Q_{u(s)}$$

where;

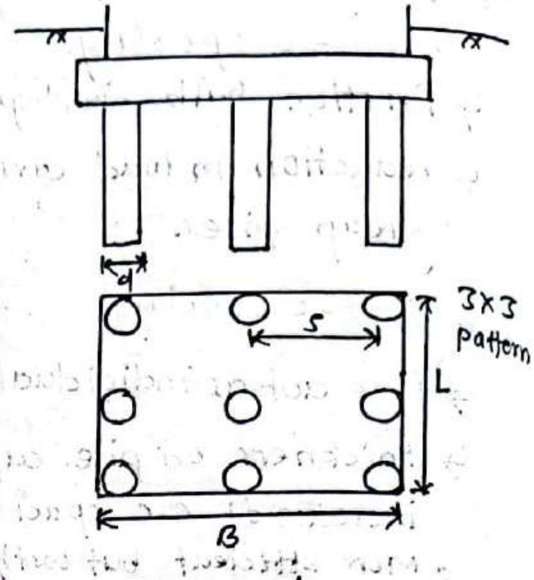
$Q_{u(s)}$ = ultimate load for single pile.

N = total no. of piles.

↳ safe load for group pile

$$Q_{(g)} = \frac{Q_{u(g)}}{FOS}$$

- In cohesive soil : clay:



↳ Due to adhesive force between pile and clay, there is possibility of single block (B×L×D_f) size formation. So load carrying capacity of group pile in clay should be calculated in both individual failure & group failure.

*: considering individual failure:

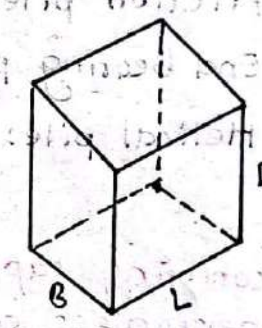
$$q_{u(g)} = N \times q_{u(s)}$$

where; $q_{u(s)} = q_b + q_s$

$\therefore q_{u(s)} = cN_c \alpha A_b + \alpha \bar{c} A_s$

$B = 2s + d$
$L = 2s + d$
} for 3x3 pattern.

*: considering block failure:



$$q_{u(g)} = cN_c \alpha A_{b(B)} + \alpha \bar{c} A_{s(B)}$$

where; $A_{b(B)} = B \times L$

$A_{s(B)} = 2(B+L) \times D_f$

Note:

↳ smaller of above two value is taken as ultimate load of group pile.

↳ safe load for group pile:

$$q_s = \frac{q_{u(g)}}{FOS}; \; FOS \geq 2.5$$

Numerical!:

Q. Twenty numbers of bored pile of dia. 0.5m & 16m depth are constructed in 1.75 c-c spacing in site having clay of unconfined compressive strength of 60 kN/m². calculate the safe load for group pile. take $\alpha = 0.45$.

solution:

dia. of pile (d) = 0.5m

spacing of pile (s) = 1.75m

depth of pile (D_f) = 16m

unconfined compressive strength (q_u) = 60 kN/m²

cohesion (c) = $\frac{q_u}{2}$ = 30 kN/m²

adhesion factor (α) = 0.45

considering unit weight of soil = 17 kN/m³

soil type: clay ($\therefore N_c = 9$)

(i) considering individual failure.

$$Q_{u(g)} = N \times q_{u(s)}$$

$$\text{where } q_{u(s)} = Q_b + Q_s$$

$$= cN_cA_b + \alpha\bar{c}A_s$$

$$= 30 \times 9 \times \frac{\pi \times d^2}{4} + 0.45 \times 30 \times \pi d \times D_f$$

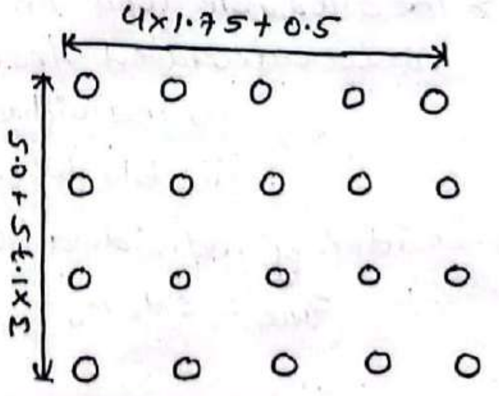
$$= 30 \times 9 \times \pi \times \frac{0.5^2}{4} + 0.45 \times 30 \times \pi \times 0.5 \times 16$$

$$= 392.30 \text{ kN.}$$

$$Q_{u(g)} = 20 \times 392.30$$

$$= 7846.12 \text{ kN.}$$

(ii) considering block failure:



$$Q_{u(g)} = cN_c \times A_{b(B)} + \alpha\bar{c}A_{s(B)}$$

Here,

$$A_{b(B)} = B \times L$$

$$= 7.5 \times 5.75$$

$$= 43.125 \text{ m}^2$$

$A_{s(B)} = 2(B+L) \times D_f$

$= 2(7.5+5.75) \times 16$

$= 424 \ m^2$

$Q_{u(g)} = 30 \times 9 \times 43.125 + 0.45 \times 30 \times 424$

$= 17367.5 \ kN$

∴ safe load for group pile.

= smallest of above two value

= 7846.12 kN.

9. A group of 9 piles with 3 piles in a row was driven into soft clay extending from ground level to great depth. The diameter and the length of the pile were 30 cm and 10 m respectively. The unconfined compressive strength of clay is 70 kPa. If the piles were placed 90 cm c/c, compute allowable load on the group pile on the basis of shear failure criteria for a factor of safety 2.5. (Assume $\alpha = 0.60$)

solution:

diameter of pile (d) = 30 cm = 0.3 m

spacing of pile = 90 cm = 0.9 m

length of pile (L) = 10 m

soil type: clay

$\therefore c = \frac{q_u}{2} = \frac{70}{2} = 35 \ kN/m^2$

↳ The allowable load on the group pile is taken as the least of the value calculated from two conditions:

(i) individual failure

(ii) block failure.

(i) considering individual failure.

$Q_{u(s)} = cN_c A_b + \alpha \bar{c} A_s$

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Here, $A_b = \frac{\pi}{4} \times (0.3)^2 = 0.07\, m^2$

$A_s = \pi d L = \pi \times 0.3 \times 10 = 9.42\, m^2$

$Q_{u(s)} = 9 \times 35 \times 0.07 + 0.60 \times 35 \times 9.42$

$= 219.87$ KN.

$\therefore Q_{u(g)} = 9 \times 219.87 = 1978.83$ KN.

$Q_a = \frac{Q_{u(g)}}{FOS} = \frac{1978.83}{2.5} = 791.53$ KN.

(ii) considering ~~group~~ group failure

$Q_{u(g)} = C N_c A_{b(B)} + \alpha \bar{C} A_{s(B)}$

$A_b = B \times L$

$= (2 \times 0.9 + 0.3)(2 \times 0.9 \times 3)$

$= 4.41\, m^2$

$A_s = 2(B+L) D_f$

$= 2(2.1 + 2.1) \times 10$

$= 84\, m^2$

$Q_{u(g)} = C N_c A_{b(B)} + \alpha \bar{C} A_{s(B)}$

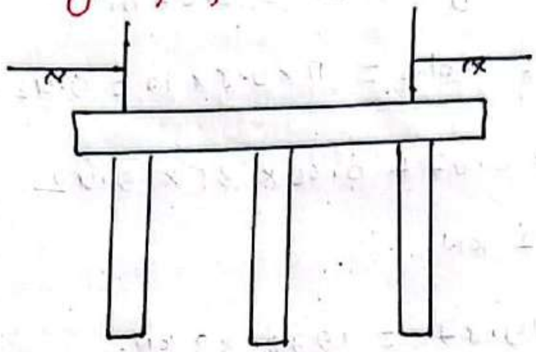
$= 35 \times 9 \times 4.41 + 0.60 \times 35 \times 84$

$= 3153.15$ KN

$q_a = \frac{Q_{u(g)}}{FOS} = \frac{3153.15}{2.5} = 1261.2$ KN

Hence the allowable load on the pile is 791.5 KN.

b. Settlement analysis of group pile:

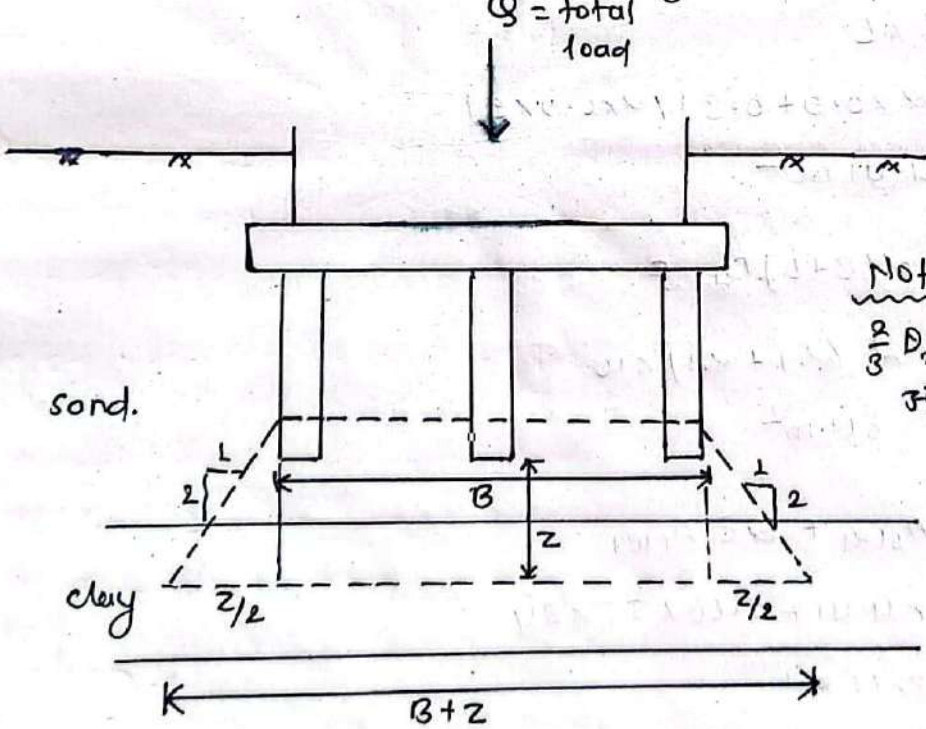


↳ If piles are constructed on clay, settlement analysis of group pile should be done.

consideration:

↳ B×L size equivalent pile in $\frac{2}{3}D_f$ level.

↳ 2:1 shear stress distribution by mat.



Note:
$\frac{2}{3}D_f$ तल जुनजुकै soil आएनी केहि लाग्दैन!

↳ Increase in stress at mid-height of clay

$$\Delta\bar{\sigma} = \frac{Q}{(B+z)(L+z)}$$

↳ Then, $S_f = m_v \times H \times \Delta\bar{\sigma}$

$$S_f = \frac{a_v}{1+e_0} \times H \times \Delta\bar{\sigma}$$

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

c. Efficiency of group pile:

Efficiency, ϵ

$$\epsilon = \frac{q_{u(g)}}{N \times q_{u(s)}}$$

where; $q_{u(s)}$ = ultimate load for single pile.

$q_{u(g)}$ = ultimate load for group pile

N = no. of piles

Emperical relation:

$$\epsilon = 1 - \left[\frac{m(n-1)+n(m-1)}{90\, m \times n}\right] \times \theta$$

where;

$\theta = \tan^{-1}(d/s)$ in degree

s = c-c spacing

d = size of pile.

$m \times n$: layout pattern

Numerical:

Q. 20 nos. of piles are constructed in 4×5 pattern with 2× size of pile c-c spacing. If single pile carry 100 kN load, calculate load carrying capacity of group pile.

Hint:-

↳ combination of above two efficiency formula बाट solve गर्ने!

Solution:

$m \times n = 4 \times 5$

$\therefore B = 3 \times s + d = 3 \times 2d + d = 7d$

$\therefore L = 4s + d = 4 \times 2d + d = 9d$

we have,

$$\epsilon = \frac{q_{u(g)}}{N \times q_{u(s)}} = \frac{q_{u(g)}}{20 \times 100} \quad \text{---(i)}$$

Also,

$$\epsilon = 1 - \left[\frac{4(5-1)+5(4-1)}{90 \times 4 \times 5}\right] \times \tan^{-1}\left(\frac{d}{2d}\right) \quad \text{---(ii)}$$

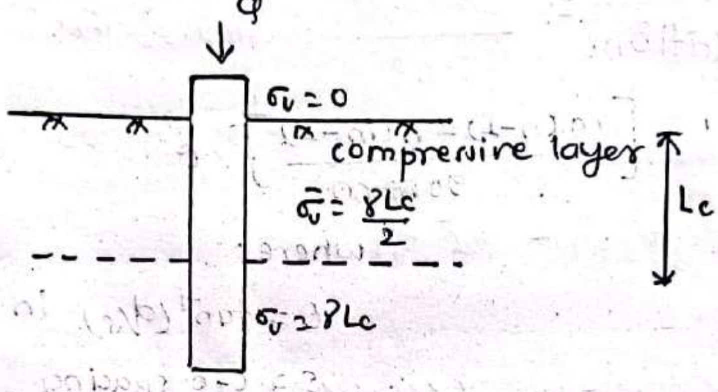
Equating equ^n (I) and (II)

$$\frac{Q_{u(g)}}{20 \times 100} = 1 - \left(\frac{4(5-1)+5(4-1)}{90 \times 4 \times 5}\right) \times \tan^{-1}\left(\frac{1}{2}\right)$$

on solving, we get

$Q_{u(g)} = 1084.98$ KN.

Negative skin friction:-(NSF)



↳ If the surrounding soil near piles settles more than pile relatively, surrounding soil drags the pile downward, this drag force is known as Negative skin friction (NSF).

*: Causes of Negative skin friction:

↳ settlement of surrounding soil due to;

a. load of adjacent structure

b. fall of GWT

c. dynamic loading

*: Potential on:

↳ loose or very loose sand layer.

↳ soft clay.

*: Effects of NSF:

↳ Increase in load of pile.

*: Preventive measures of NSF:

↳ design of end bearing pile instead of friction pile.

↳ coating surface of pile with bitumen before driving in case of driven pile.

↳ creating space between pile and soil using double casing and filling space with viscous material.

Calculation of NSF:

• For cohesionless soil: sand

*. for single pile:

$$NSF_{(s)} = K \times \bar{\sigma_v} \tan\delta \times A_s$$

where; $K = 1$

$$\bar{\sigma_v} = \frac{\gamma L_c}{2}$$

*. for group pile:

$$NSF_{(g)} = N \times NSF_{(s)}$$

• For cohesive soil: clay

*. for single pile:

$$NSF_{(s)} = \alpha \bar{c} \times A_s$$

where; A_s = perimeter $\times L_c$ = surface area.

*. for group pile:

(I) considering individual failure

$$NSF_{(g)} = N \times NSF_{(s)}$$

(II) considering block failure

$$NSF_{(g)} = \gamma \times L_c \times A_{b(Block)} + \alpha \bar{c} \times A_{s(Block)}$$

where;

$$A_{b(Block)} = B \times L$$

$$A_{s(Block)} = 2(B+L) \times L_c$$

Note:

↳ Larger value of (I) and (II) is taken as NSF for group pile in case of cohesive soil.

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*: Design of caisson / well:

Introduction:

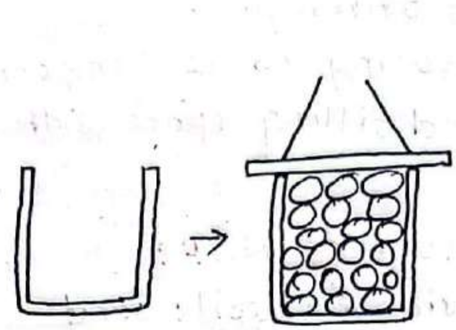
↳ A type of deep foundation having large x-section and hollo body is termed as caisson.

Types of caisson:

a. Box caisson

↳ closed at bottom

↳ casted outside and sinked at excavated hole.



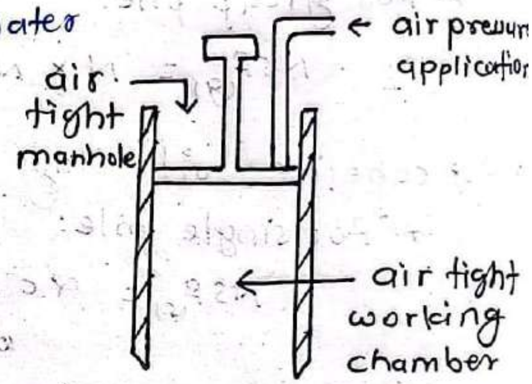
Best use:

↳ upto 8m depth.

b. Pneumatic caisson:

↳ air tight working chamber is created to apply air pressure such that water can be displaced for combortable dredging.

↳ costlier pneumatic system



Best use:

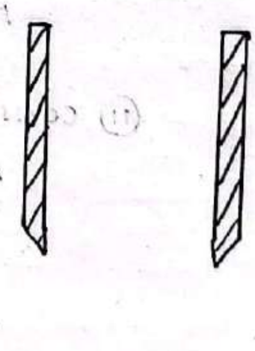
↳ upto 16 m depth.

c. open caisson : well

↳ open at both end

↳ casting, dredging and sinking at many stages.

↳ economical than pneumatic caisson.



Components of well and their function:

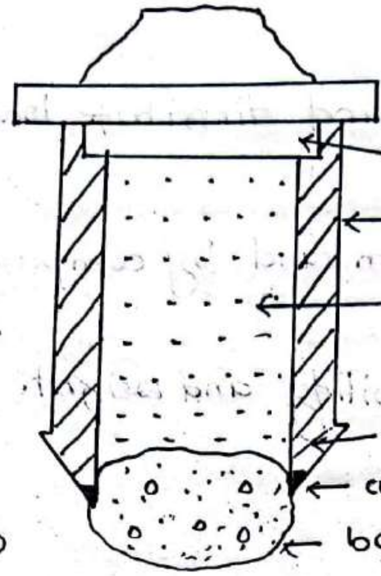
- components:

a. Well cap:

↳ RCC slab

Function:

↳ receive load from abutment / pier and transmit to steining.



b. Top plug:

↳ RCC slab

Function:

↳ provide support to well cap in inner portion through sand filling.

c. Steining:

↳ RCC element.

↳ Major component having both Hz. and Vt. reinforcement as main reinforcement (≥ 2.5 % to resist all vertical and lateral loads.

↳ Thickness ≥ 0.6 m

↳ Responsible to resist all loads.

d. Well curb:

↳ RCC component

↳ Act as base of steining

↳ Tapered in shape

e. Cutting edge:

↳ made of high strength steel.

↳ sharp and strong.

Function:

↳ provide easy sinking.

f. Bottom plug:

↳ PCC component.

Function:

↳ support steining and distribute load to larger area.

g. Sand filling:

↳ Well is filled with sand by compaction in many layers.

Function:

↳ increases the stability and weight.

Types of load for well:

a. Live load:

↳ load of vehicle.

b. Dead load.

↳ load of slab, girder, railing, black top, approach slab, abutment/pier, self weight of well.

c. Impact load.

↳ 10% of live load.

d. Braking load.

↳ Tractive force.

e. Earth pressure.

↳ active and passive.

f. Water pressure.

↳ $P = K \times V^2$

; V = velocity of water

K = constant depends on discharge.

g. Air pressure.

↳ $P = I \cdot A$; I = intensity of air pressure depends on altitude.

A = exposed area of bridge

h. centrifugal force:

↳ considered if curve bridge is to be provided.

i. Buoyancy force:

↳ uplift pressure of water.

j. Temperature stress

↳ due to moving vehicle and increase in temperature.

k. Seismic load.

↳ 25% of LL and D.L.

7: Design of well:

↳ Design of well consists of two consideration

a. selection of shape and size.

b. selection of depth of well.

a. Selection of shape and size:

Criteria:

i. sufficient inside area for comfortable dredging.

ii. sufficient thickness of steining to resist all loads.

iii. sufficient size to tolerate permissible tilt.

↳ 1:60 permissible tilt.

iv. Shear failure criteria

↳ well should not overstress soil below.

$\sigma_{max} \leq q_{ns}$

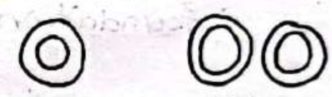
v. Settlement criteria

↳ settlement should be within permissible limit.

$\sigma_{max} \leq q_{np}$.

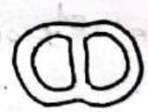
possible shapes:

① circular:

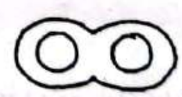


single twin

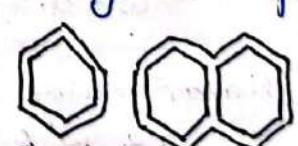
② Double D-shaped.



③ Dumbbell shape

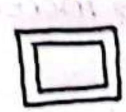


④ Hexagonal shape.

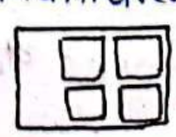


single twin

⑤ Rectangular well.



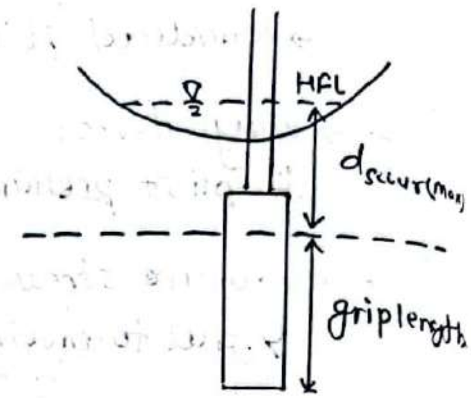
⑥ Multi dredge hole well



6. Selection of depth:

criteria:

① minimum grip length



*: Grip length:

↳ Depth of the well provided below scour level is known as grip length.

↳ stability of well depends on grip length.

↳ grip length $\geq \frac{1}{3} d_{scour(max)}$

*: Scour depth:

↳ Maximum depth of erosion that can occour around a bridge foundation due to water flow is known as scour depth.

*: calculation of scour depth:

From lacey's formula,

$$d_{scour} = 0.473 \left(\frac{Q}{f}\right)^{1/8}$$

where; Q = design discharge (m^3/s)

f = lacey's silt factor $= 1.76\sqrt{d_{mm}}$

d_{mm} = mean size of particles in mm.

Natural waterway (w)

$w = 4.75\sqrt{Q}$

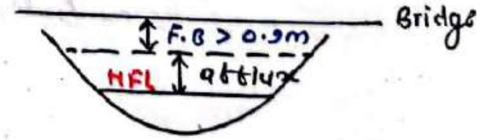
↳ If linear water way provided (w') is less than natural water way (w)

i.e. $w' < w$

• It has two effects:

① Afflux:

↳ increase in height of bridge



⑪ Increase in scouring:

$$d'_{scour} = d_{scour} \times \left(\frac{w'}{w}\right)^{0.61}$$

iv. Effects of meandering:

$$d_{scour(max)} = C \times d'_{scour}$$

where;

$C = 1.27$ to 2.5

(II) shear failure criteria:

↳ well should not overstress soil below.

↳ $\sigma_{max} \leq q_{ns}$

(III) settlement criteria:

↳ settlement should be within permissible limit.

↳ $\sigma_{max} \leq q_{np}$.

Probable questions:

Q. Write down the effects when linear waterway provided is less than natural water way (W)? [PSC, 5 marks]

Q. What do you mean by grip length? Write about consideration to be made about grip length for well foundation design. [PSC, 5 marks]

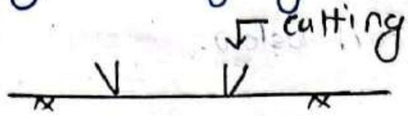
CamScanner

Construction of well foundation:

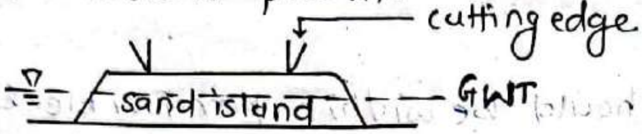
↳ construction of well foundation needs continuous observation by civil engineers.

Steps for construction:

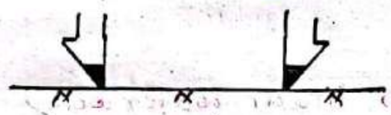
step 1: centering and laying of cutting edge.



• if water table is present.



step 2: Reinforcement and casting of well curb:



step 3: Casting of steining:

↳ initially 1.5m casting in each stage.

↳ After sinking 6m depth: 3m casting in each stage.

step 4: Sinking:

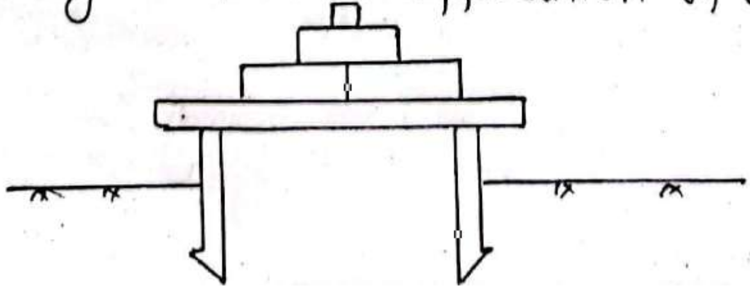
↳ casting and sinking are done simultaneously in many stages.

• Initially:

↳ sinking is done by dredging under the effect of self weight of steining.

• After 6 m depth:

↳ sinking is done with application of centric loading.



step 5: construction of bottom plug after sinked to design depth.

step 6: Well is filled with sand by compaction in many layers.

step 7: Construction of top plug.

step 8: Construction of well cap.

Tilt and shift of well foundation:

↳ When the well is sloped against vertical alignment, it is called tilting of well.

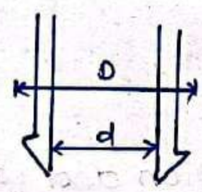
↳ When the wall is moved away horizontally from the desired position, it is called shifting of wall.

↳ Well should be sinked within permissible tilt (1:60)

Preventive measures for tilt:

I. Cutting edge should be sharp and strong.

II. Outer diameter of well should be larger than outer diameter of steining by 20 to 40 cm.



• D > d by 20 to 40 cm.

III. Great care should be given to make the outer surface smooth.

iv. During application of load for sinking, great care should be given to make load centric and uniformly distributed.

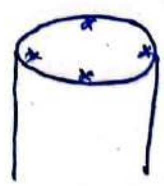
v. Dredging should be uniform.

vi. Verticality of well should be observed continuously during entire process of sinking.

• It can be done by;

a. Using plum bob.

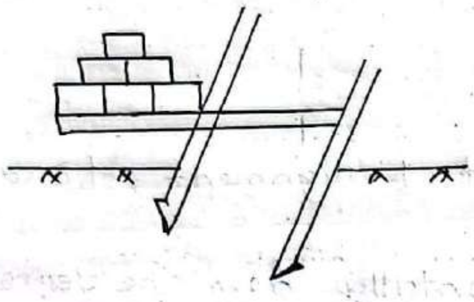
b. Using level machine.



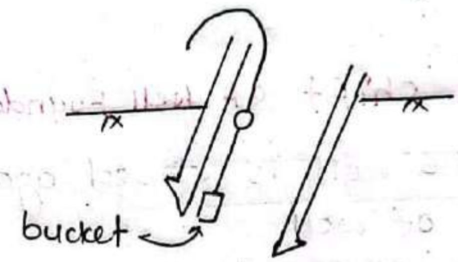
*: Retrification of tilt and shift:-

↳ Tilt and shift can be rectified using anyone or combination of following technique:

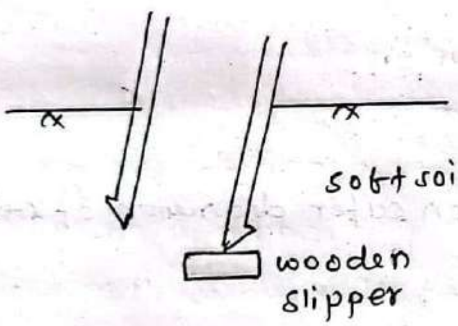
1. Ecentric loading:



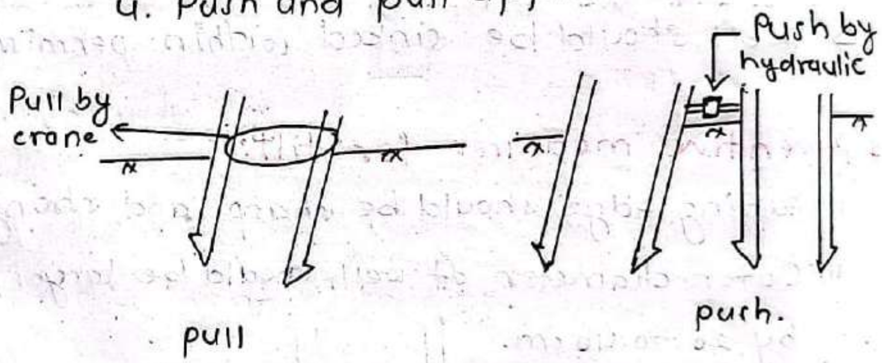
2. control in dredging



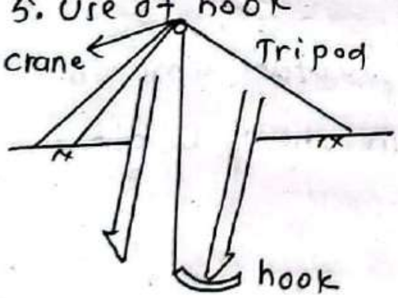
3. Application of stopper



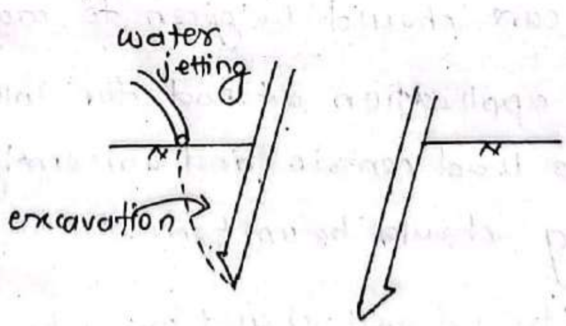
4. Push and pull application.



5. Use of hook



6. Excavation and water jetting.



*: Comparision between pile, pier and well :-

Parameter	Pile	Pier	Well.
Size	≤ 2m — ≤ 1 m practically	2-3.5m	> 3.5m — > 5m practically
Body	solid	Solid	Hollow
Arrangement	In m×n pattern	In a line	single or twin
Depth (best use)	Upto 20m	Upto 40m	Upto 30m
suitability soil	clay	clay extended to very large depth.	sand, silty sand, clayey sand.
structure	High rise building, bridge	Bridge	Bridge.
Location	Terai, kathmandu	kathmandu	Terai
Construction	Pre-cast or cast-insitu	cast insitu	casting → dredging → sinking

Types of foundation for bridge:

↳ Possible foundation for bridge are:

(I) Open foundation } shallow foundation

(II) Pile foundation

(III) Pier foundation } deep foundation

(IV) Well foundation

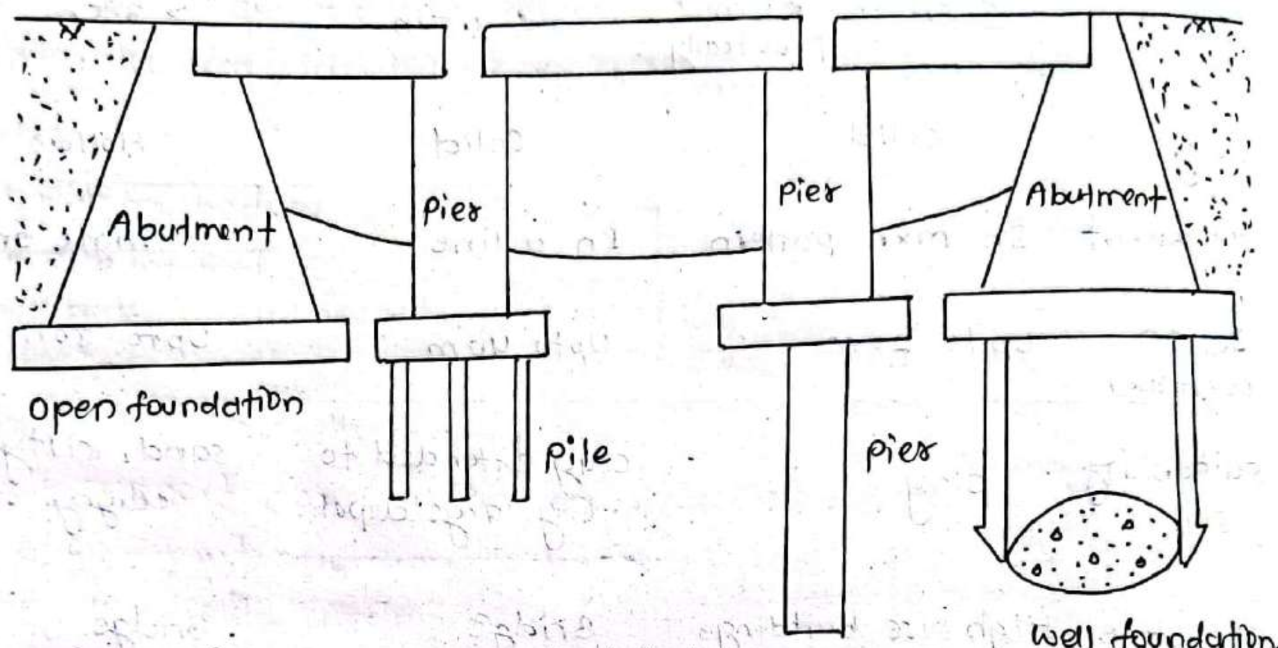


fig: foundations used in bridge

① Open foundation:

↳ RCC slab supporting pier or abutment of bridge.

↳ shallow foundation

suitability:

- soil: strong soil layer
  dense to very dense sand
  rock layer
  boulder layer etc.
- structure:
  short span bridge.
- location:
  hilly area.

(ii) Pile foundation:

↳ A type of deep foundation having small c/s area and slender body.

suitability:

- soil: in clay
- structure: long span bridge
- Location: kathmandu, terai

(iii) Pier foundation:

↳ Large pile with large depth which can replace group of piles.

suitability:

- soil: clay extended to very large depth.
- structure: long span bridge
- location: kathmandu, hilly area.

(iv) Well foundation:

↳ A type of deep foundation having large c/s and hollow body.

suitability:

- soil: in sand (sandy soil)
- structure: long span bridge
- location: terai

2.2.6 Foundation stabilization, underpinning and geotechnical process:

*: Underpining:

↳ Replacing the foundation for existing structure with larger or deeper type of foundation is termed as underpinning.

Methods:

① Pit method.
⑪ Pile method.

① Pit method:

Steps:

i. Provide support for wall/column.
ii. Excavation of pit according to deign.
iii. Dismantle of old foundation.
iv. Reinforcement and casting of new one.

⑪ Pile method:

Steps:

i. construction of pile.
ii. insert beam through wall rested on pile.

Modification of soil and Ground improvement for foundation soil:-

↳ Enhancing the property of less ideal soil such that it meets the requirements for proposed structure is termed as modification of soil and the technique applied are ground improvement technique.

*: Advantages:

A. To increase the shear strength of soil:

↳ increase the bearing capacity for foundation.
↳ increase lateral stability on slope.
↳ reduce pavement failure on road.

B. Reduce compressibility of soil:
- reduce settlement for foundation.
- reduce lateral deformation on slope.
- reduce undulation, rut formation on pavements.

C. Increase volume stability of soil:
- swelling and shrinkage potential ⇒ nearly null.
- reduce pavement failure.

D. Reduce Permeability of soil:
- reduce seepage in dam, reservoir, canal etc.
- increase durability of project.
- reduce liquefaction potential of soil.

*: Ground improvement methods:-

A. Hydraulic modification:
   (i) De-watering
   (ii) Preloading

B. Deep compaction.

C. physical modification: stabilization
   (i) cement stabilization
   (ii) lime stabilization
   (iii) bitumen stabilization.

D. Inclusion in soil:
   (i) stone column
   (ii) reinforced earth
   (iii) soil nailing

E. Grouting

F. Ground freezing.

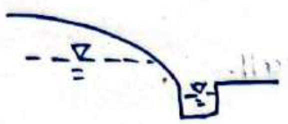
A. Hydraulic modification:

(i) Dewatering:

↳ lowering the position of ground water table.

*: Methods:

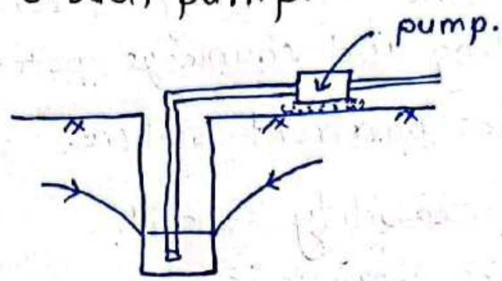
a. Open sump:



↳ Preferred in sand, sandy soil.

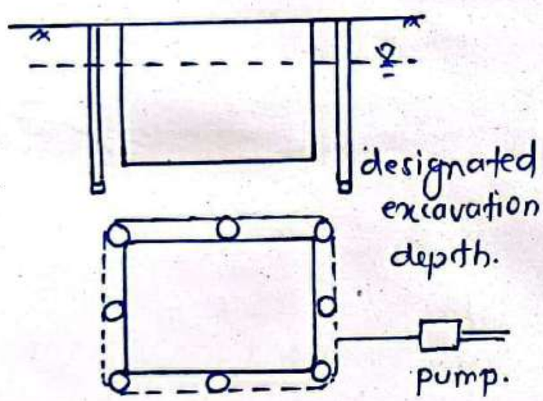
↳ If GWT is near to ground surface

b. Well pump.

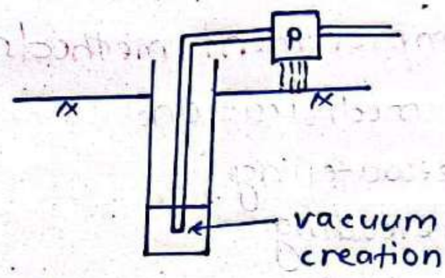


↳ To lower the GWT to large depth.

c. Series of well pump

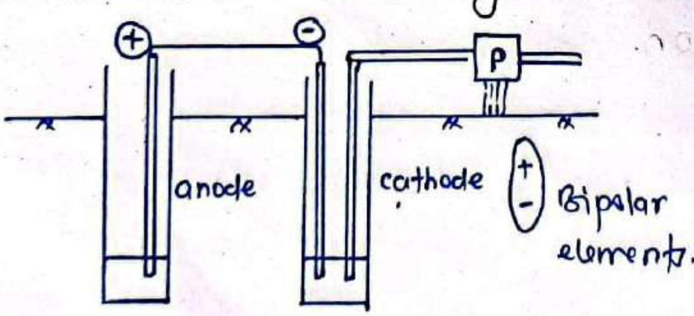


d. Vacuum dewatering:



↳ if low permeability soil is available.

e. Elechomoissis Dewatering



↳ used in rich clay

↳ electrically forced dewatering.

(ii) Pre-loading:

↳ Application of load equivalent to proposed structure on site prior to construction is termed as preloading.

Application:

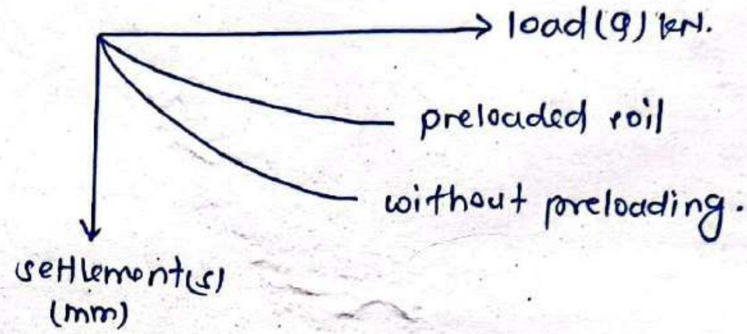
↳ on compressive soil layer.

Objective:

↳ certain % of degree of consolidation is completed before construction such that settlement after construction can be reduced.

Advantages:

(i) settlement reduces



(ii) costlier foundation can be replaced by economical type.

Two approach of pre-loading:

a. Pre-loading only:

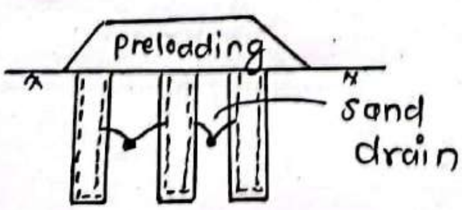
↳ if sufficient time is available before construction.

↳ Two ways:

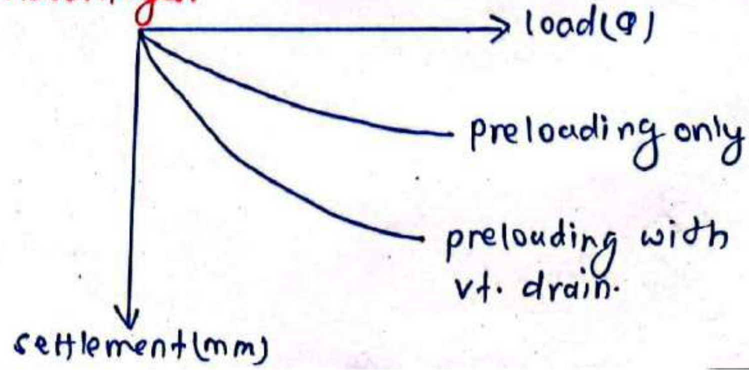
- using soil layer.
- using steel sheets.

b. Pre-loading with vertical drain:

↳ accelerate consolidation.



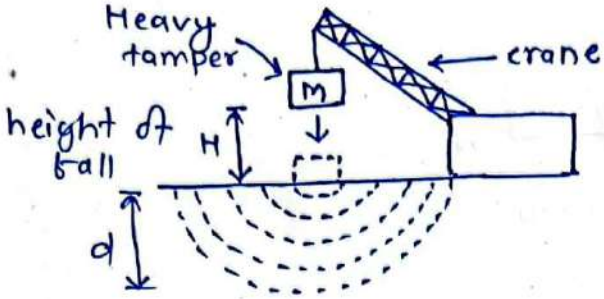
Advantages:



B. Deep compaction:

a. Dynamic loading:

↳ preferred on sandy soil.



↳ sandy soil भएको ठाउँमा
eg: Narayani को किनारा.

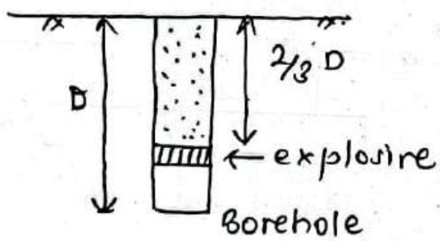
Here, d = depth of soil.

$$d = \alpha\sqrt{H \times M}$$; M = mass of tamper
α = 0.3 to 0.5 (constant)

Use: sand or sandy soil.

↳ Increase density of soil to large depth.

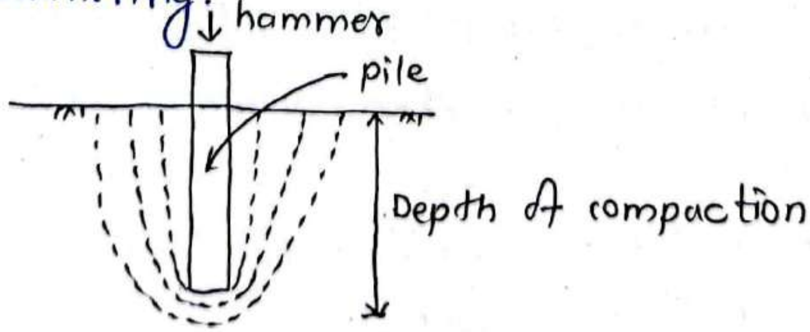
b. Explosion method:



Use: generally for sand.

c. Compaction pile method:

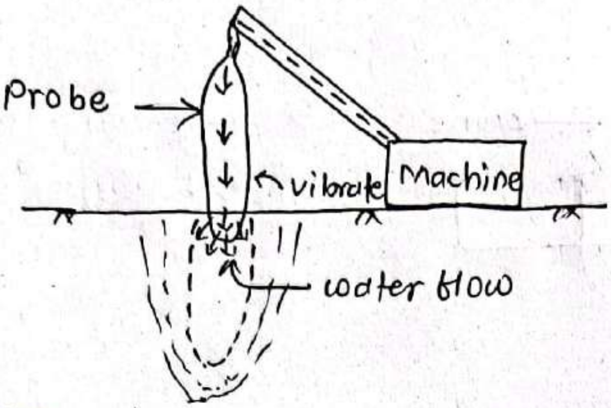
↳ Timber or concrete pile should be drived randomly in field by hammering.



Use: for sand or sandy soil.

d. Vibroflotation:

↳ use of vibrating probe to penetrate granular soil.



Best use: upto 75m

↳ on sandy or granular soil.

c. Inclusion in soil:

a. Stone column:

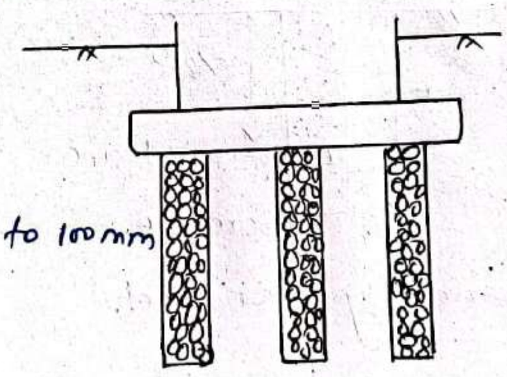
↳ Bore hole filled with aggregate by compaction.

↳ size: 0.3 m to 2m.

↳ c-c spacing: 2 to 3 m

↳ size of aggregate: 10mm to 100mm

↳ layout pattern
- m×n pattern
- triangular pattern.



Characteristics:-

↳ Flexible in nature: so perform well in seismic load.

↳ Damping: prevent transfer of seismic wave to superstructure.

Function:

↳ increase in load carrying capacity.

↳ Drainage: draw down water table.

Application:

- Below mat in clay
- Below water tank in clay
- Below embankment/dam.

D. Grouting:

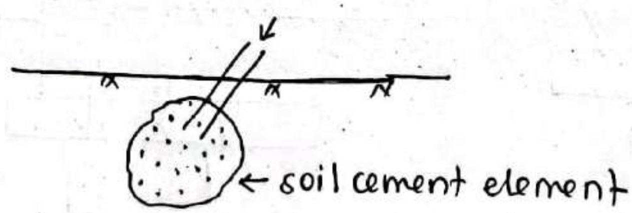
- Injection of slurry in soil, soil fissures or crack in rock is grouting.
- Mostly cement slurry is used as grout.

Types of grouting are:

1. Permeation grouting.
2. Displacement grouting.
3. Replacement grouting.

(i) Permeation grouting:

- Injection of very fine slurry into void of soil.
- Bind the soil particles and create soil-cement element.

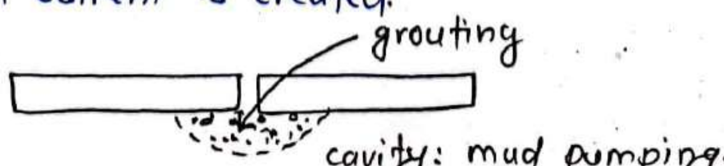


Use:

(i) Below dam to reduce permeability of soil.
(ii) Below foundation to increase shear strength of soil.

(ii) Displacement grouting: compaction grouting

- Injection of the thick cement slurry into the ground.
- Displace the soil particles and bind soil particles, so compacted soil cement element is created.



Use:

(i) To seal void below rigid footing.
(ii) To seal the crack in rock.
(iii) To seal cavity in soil mass.

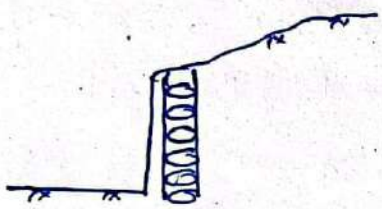
(iii) Replacement Grouting: Jet grouting:

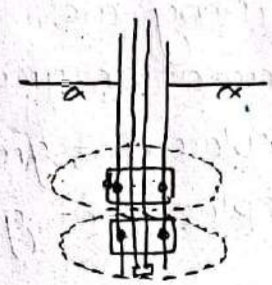
↳ Injection of very thick cement slurry into soil in high pressure.

↳ Replace the soil and create cement element within the soil.

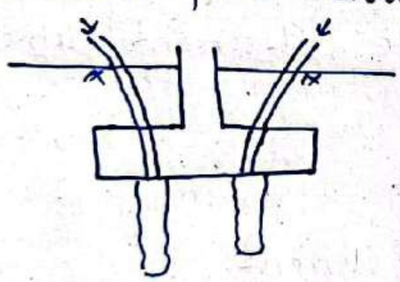
Use:

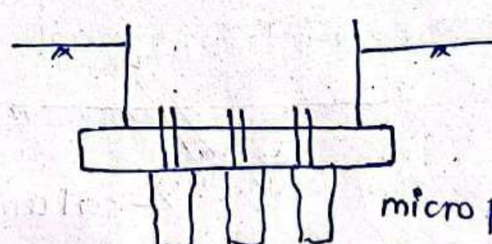
(i) To protect vertical cut.





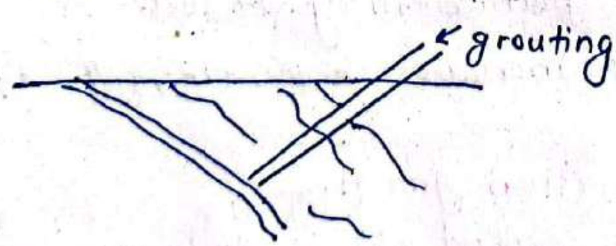
(ii) To create pile below isolated footing.





micro pile of 30 cm radius having rod.

(iv) To seal the crack in rock.



E. Physical modification: stabilization.

↳ Mixing admixture like cement, bitumen, lime in soil before compaction and compacting in many layers.

Use:

↳ In roads, runways, dam, embankment etc.

Q. Types of stabilization:

a. cement stabilization

b. Lime stabilization

c. Bitumen stabilization.

a. cement stabilization:

↳ 5-12 % cement by volume is mixed in soil. (can be used >12;

Best application: In sand or sandy soil.

b. Lime stabilization:

↳ 3-9% lime by volume is mixed in soil. (9% maxm)

Best application: In clay (plastic clay)

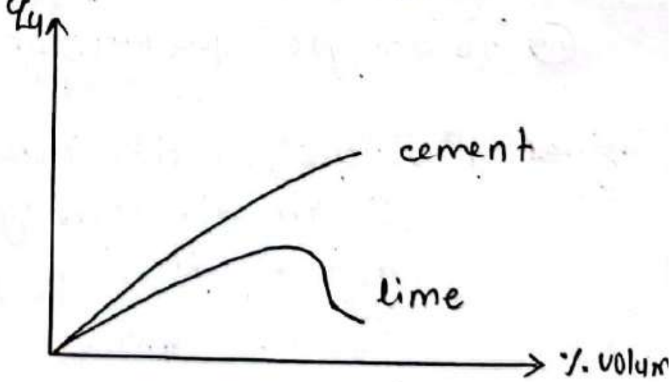
c. Bitumen stabilization:

↳ 3 to 12 l/m² is sprayed in soil before compaction.

Best application:

(i) in organic soil

(ii) in dam, reservoir etc.



2.3 site Investigation and soil exploration:

***: Site investigation:**

↳ Process of acquiring geological, geotechnical and other relevant information which might effect on design, construction and environment of any civil engineering project is termed as site investigation.

Purpose / objectives:

1. to find lateral and vertical extension of each soil layer in site
2. to determine engineering properties of each soil layer through series of conduction of lab test and insitu test.
3. to determine index properties of soil.
4. to classify soil and make soil profile in site.
5. to determine the position of GWT.
6. to analyze the bearing capacity and select the size and type of foundation.
7. to analyze stability of slope and design of stabilization measures
8. to analyze liquefaction potential of each soil layer.

Types or stages of site investigation:

1. Primary investigation
2. Detailed investigation
3. Investigation during construction: conformation test.

① Primary investigation:

objective: site selection

↳ Depending on geotechnical merit and demerit of site.

- It consists of:

a. office work:

↳ Map study: google earth

↳ seismicity study

↳ morphology study

↳ Hydrology study

↳ photographs study: aerial photographs study

b. Reconaissance (Recy)

↳ To verify the complexity observed in deskstudy.

Trial pit: 3m x 3m x 3m.

(ii) Detailed study:

↳ To acquire information for design and construction.

↳ It consists of:

- Boring
- Insitu test for each layer.
- sampling from each layer.
- Obrervation of GWT.
- lab test of each soil layer.

Analysis:

↳ Bearing capacity, stability, liquefaction potential.

(iii) conformation test:

↳ If altered soil layer and situation is found in site during construction, a borehole with insitu test and soil sampling is done for conformation.

Methods of soil exploiration:

↳ The field and laboratory investigation needed to obtain the required soil data for proper design and successful construction of structure at the site collectively known as the methods of soil exploiration.

Basically there are 3 methods of soil exploiration:

① Direct methods
- Test pits, trial pits or trenches.

⑪ Semi-direct methods
- Boring

⑪ Soil-exploiration : indirect methods
- soundings or penetration tests and geophysical methods.

① Direct methods:

↳ test pits or trial pits are excavated.

↳ avg. depth 3-15 ft.

↳ soil is examined in natural condition.

↳ suitable only for small depth upto 3m.

⑪ Semi-direct method:

↳ Boring:

- Making or drilling boreholes into the ground with an outlook to obtaining soil or rock samples from known depth is boring.

*: Depth of boring according to (IS-1892: 1979) different types of foundation are:

↳ For isolated spread or raft foundation, boring depth should be one and half times width of foundation.

↳ Boring depth for adjacent footing with precise spacing less than 2× width should be one and half times length of footing.

↳ For pile and wall foundation, to a depth of one and half times the width of structure from foundation level.

↳ In case of road cut, the boring depth should be equal to the base width of cut.

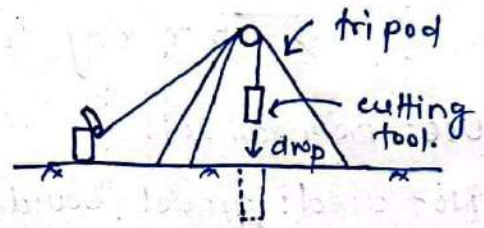
↳ For road fill it will be two meters below ground level or equal to the height of fill, whichever.

Methods of boring:

a. Percussion boring:

↳ Bore hole is advanced by repeated blows of cutting tool.

↳ very time consuming method, mostly not used.

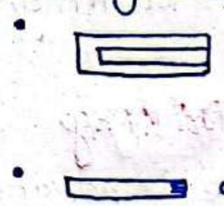


Use: for normal soil.

↳ easy, fast and economical.

Location: ktm, terai

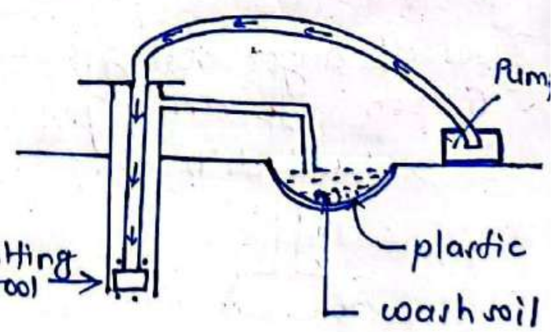
cutting tool:



b. Wash boring

↳ Borehole is advanced by rotating cutting tool with pressure flow of water.

↳ wash up the cut soil by same water.



Use:

↳ sand, silt, silty sand.

Not used:

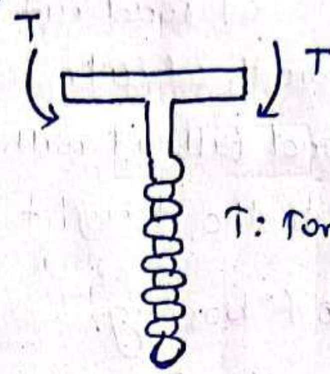
↳ Rock, gravel, boulder, stiff clay.

Location:

↳ Terai

c. Auger boring:

↳ Borehole is advanced by advancing helical auger into soil by push and rotation.



Types of auger boring:

① Hand auger

↳ upto 5m hole depth.

② Machine auger

(i) Hand operated machine

↳ upto 12 to 16m depth.

(ii) Vehicle mounted machine

↳ up to very large depth (40m).

Use: normal soil

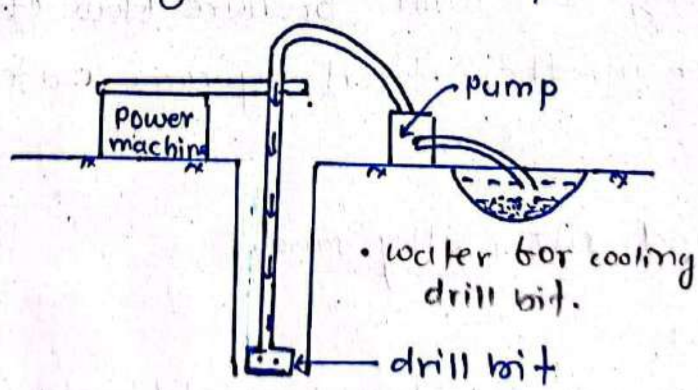
Not used: gravel, boulder, rock, pure sand below GWT.

Location: kathmandu, terai

d. Rotary Drilling

↳ Bore hole is advanced by rotating drill bit with power machine.

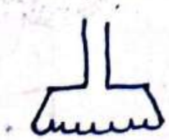
↳ Most costlier method.



Types of auger boring:

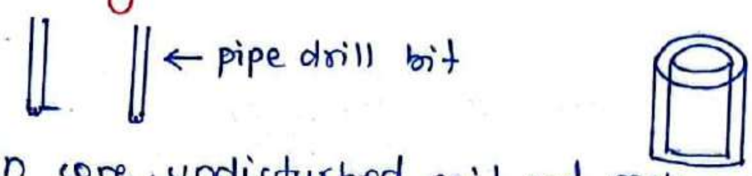
① open hole drilling:

↳ closed drill bit.



↳ destroy the soil sample.

(ii) core drilling:



↳ obtain core, undisturbed soil and rock sample.
↳ mostly used.

Use: rock layer

Location: hilly area.

★: Insitu tests:

↳ Insitu test refers to the test that are carried out directly on ground, rock or soil at a particular site.

↳ allows engineers and geologists to evaluate the subsurface conditi and suitability of site for various types of construction project

Objectives/purposes/ helps to understand:

(i) whether shallow or deep foundation is required for land.
(ii) order of occurrence of soil and rock strata.
(iii) bearing capacity of soil.
(iv) settlement rate.
(v) Position of water table.
(vi) Compressibility of soil.
(vii) To foresee problems releated to foundations.
(viii) Details on soil composition.

Types of insitu test:

① standard penetration test: (SPT)
② Dynamic cone penetration test (DCPT) } → Penetration resistance test.
③ static cone penetration test. (SCPT)

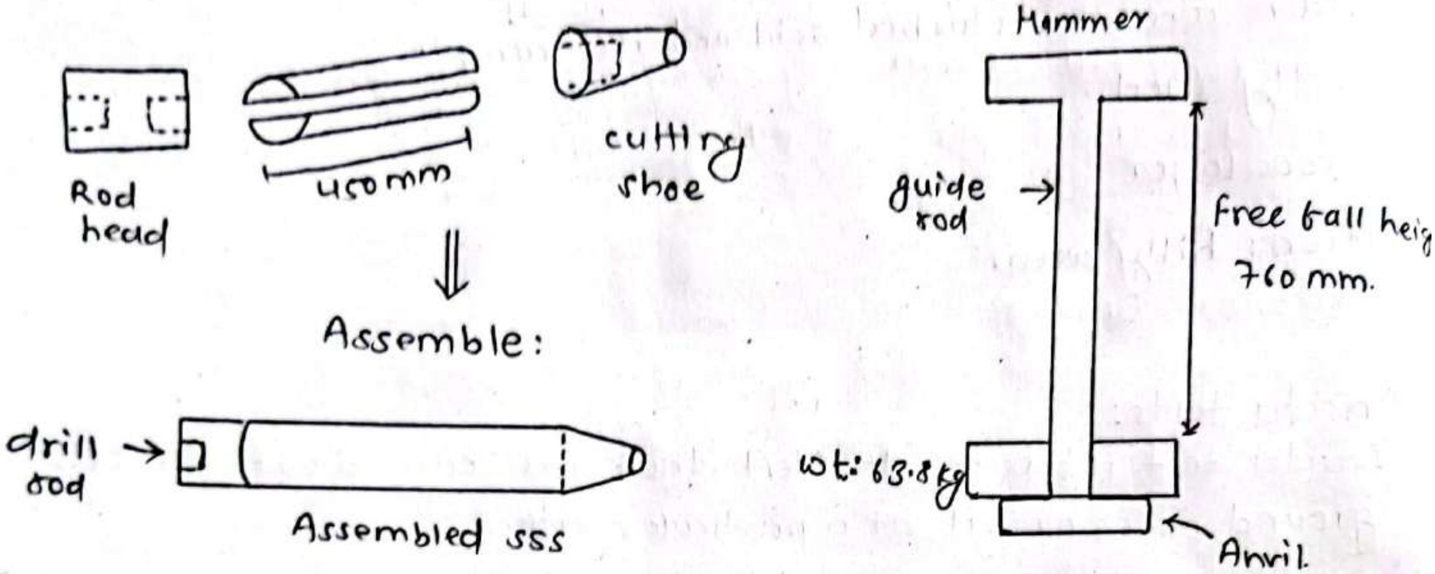
mostly used { ④ Flato dialatometer test (FDT) } → deformation resistance test.
⑤ Pressuremeter test. (PMT)

⑥ Vane shear test (VST) ⇒ shear resistance test.

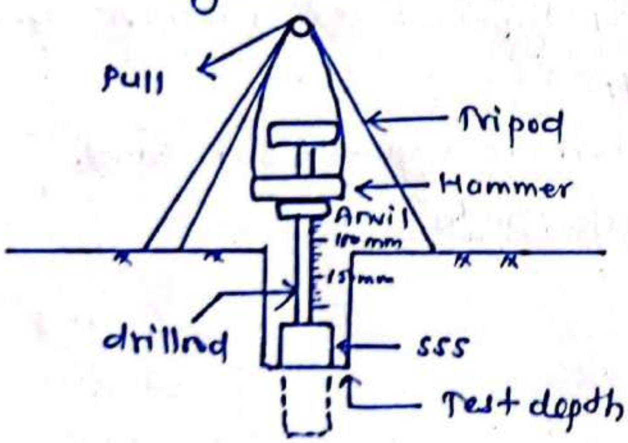
1. Standard Penetration test:

Apparatus required:

Penetration probe: split spoon sampler (SSS)



Test arrangement:



Note:

SPT-value	Type of soil.
0-4	loose soil
5-10	loose soil
11-30	Medium soil
30-50	Dense soil
>50	very dense soil.

Procedure:

- ↳ Drive the SSS into soil by dropping hammer (63.8 kg) from 760 mm height.
- ↳ count the number of blows.

measurement:

Bore hole log:

BH NO:

Project:

Location:

Date:

Boring machine:

Recorded by:

Depth (m)	Soil type	No. of drop			N-value
		150 mm	150 mm	150 mm	
1.5m	sandy clay	N_1	N_2	N_3	N_2+N_3
3m	---	-	-	-	
4.5m	--	-	-	-	

N-value: No. of drop of hammer required to drive 300 mm length of SSS.

Use: in normal soil.

Not used:

- Gravel, boulder & rock layer.

Note:
No. of drops required for first 150 mm of penetration is called seating drive and is ignored.

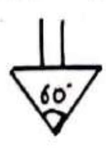
N-value: 50 (max^m) ≤ 50.

2. Dynamic cone penetration test (DCPT):

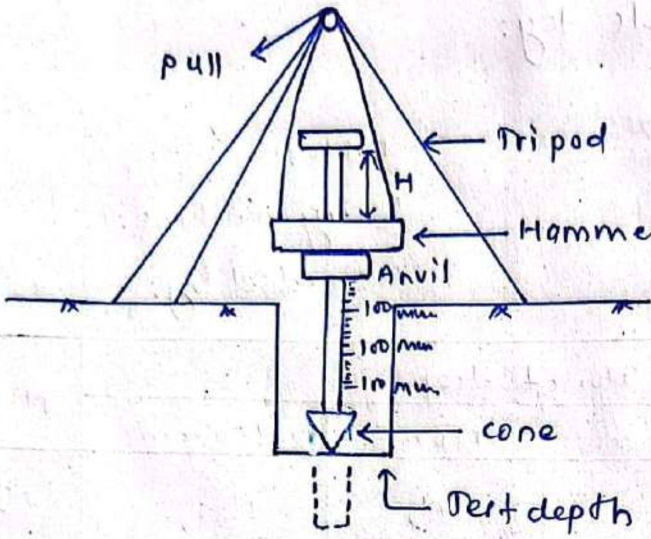
- Preferred in the hard soil layer.
- cone having apex angle 60° is used.
- quick test and cover large area economically.
- No sample is obtained.

Apparatus required:

Penetration probe: cone.



Test arrangement:



Procedure:

↳ Drive the penetration probe: cone (apex angle 60°) by dropping hammer (68.8 kg) from 750 mm height.

↳ count the number of blows.

Measurement:

Bore hole log:

B H NO: Date:

Project: Boring machine

Location: Recorded by:

Depth	soil type	No. of drop			Nc-value
		100 mm	100 mm	100 mm	
1.5m	sandy clay	N_1	N_2	N_3	$\frac{N_1+N_2+N_3}{100}$
3 m	Rock	100/90	–	–	100
4.5m	Rock	80	100/70	–	100

N-value: No. of blows required for 300 mm penetration of probe.

Use: for hard soil like gravel, boulder etc.

Not used: clay, very loose soil etc.

Conversion of Nc-value to N-value:-

- $N_c = 1.5\,N$ for depth upto 3m.
- $N_c = 1.75\,N$ for depth upto 6m i.e. (3 to 6m)
- $N_c = \alpha\,N$ for depth greater than 6m.

correction of N-value:

The N-value observed during testing is not utilized directly in assessing the soil properties. These values are corrected to account for;

a. overburden pressure

b. dilatancy in fine sands and silts.

a. Overburden correction:

↳ should be done in all types of soil.
↳ correction against overburden stress which confined the soil and gives larger value of N in large depth.

$$N_{corrected} = N_m \times 0.77 \log\left(\frac{1950}{\bar{\sigma}}\right), \text{ for } \bar{\sigma} \geq 25\ kN/m^2$$

$$= N_m \times C_f$$

; where $C_f = 0.77 \log\left(\frac{1950}{\bar{\sigma}}\right)$

$\bar{\sigma}$ = effective stress at test depth (in kN/m^2)

↳ for $\bar{\sigma} < 25\ kN/m^2$,

C_f is obtained from graph.

6. Dilatancy correction:

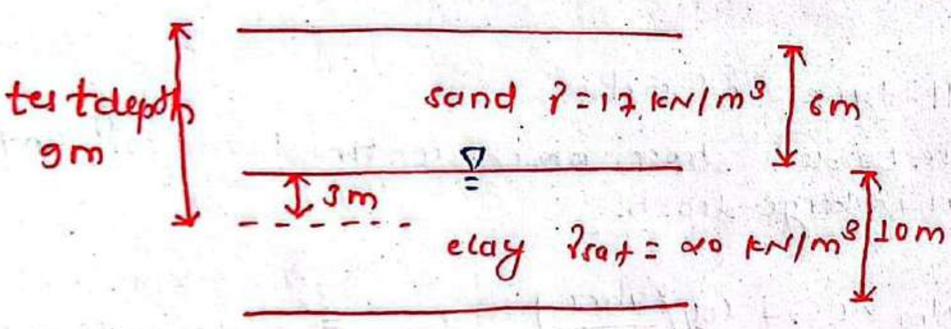
- should be done for clay, fine sands and silts only.
- correction against porewater pressure, N value is over estimated due to porewater pressure.
- Applied only when $N_{corrected}$ obtained after overburden correction exceeds 15.

$$N'_{corrected} = 15 + \left(\frac{N_{corrected} - 15}{2}\right) \text{ if } N_{corrected} > 15$$

$$N'_{corrected} = N_{corrected}, \text{ if } N_{corrected} < 15$$

Numerical:

Q. Calculate the corrected N-value for a site condition as shown below:



Given, $N_{measured} = 19$.

solution:

Test depth = 9m i.e at clay so both the correction is needed.

Note: If test depth is at sand, only overburden correction is required.
But in case of clay, both correction is required.

a. correction for over burden:

effective stress at 9m depth

$\bar{\sigma} = \gamma H_{sand} + \gamma_{sub} \times H_{clay}$

$= 17 \times 6 + (\gamma_{sat} - \gamma_w) H_{clay}$

$= 17 \times 6 + (20-10) \times 3$

$= 132 \ kN/m^2$

$N_{corrected} = N_m \times 0.77 \log\left(\frac{1950}{132}\right)$

$= 19 \times 0.77 \log\left(\frac{1950}{132}\right)$

$= 17.10 > 15$

Hence, dilatancy correction is required,

b. Dilatancy correction:

$N_{corrected} = \left(\frac{N_{corrected} - 15}{2}\right) + 15$

$= 15 + \left(\frac{17.10 - 15}{2}\right)$

$= 16.05$ Ans.

*! Soil sampling:

↳ Process of obtaining samples of soil from desired depth is known as soil sampling.

Purpose/objective:

- access the engineering properties of soil.
- determination of ground water table.

*: Types of soil samples:

Hvorclore's soil classification:

a. Undisturbed soil sample.
b. Represen tive sample
c. Non- represen tive soil sample.

a. Undisturbed soil sample: (UD samples

Characteristics:

- Insitu structure of soil mass is preserved.
- No change of moisture.
- Not mixed from different layer.
- Not missed any portion of soil.

Obtained from:

- Box sampling from pit.
- tube sampling from borehole.
- coredrilling.

Use:

- In lab test of engineering properties.

b. Representive soil sample:

Characteristics:

- Insitu structure of soil mass is disturbed.
- Natural moisture content may altered.
- Not mixed from different layer.
- Not missed any portion.

Obtained from:

- pit excavation
- percussion drilling.

Use:

↳ Index properties of soil.

↳ sieve analysis.

↳ Liquid limit, plastic limit test.

↳ sp. gravity

↳ sensitivity, thixotropy

c. Non-representive soil sample. (D-sample)

Characteristics:

↳ original structure destroyed.

↳ alter in moisture content.

↳ mixed from different layer.

↳ some portion may be missing.

↳ chemically altered sample.

Obtained from:

↳ wash boring

↳ augering

↳ closed drilling (open hole drilling).

Use:

↳ field identification: visual inspection.

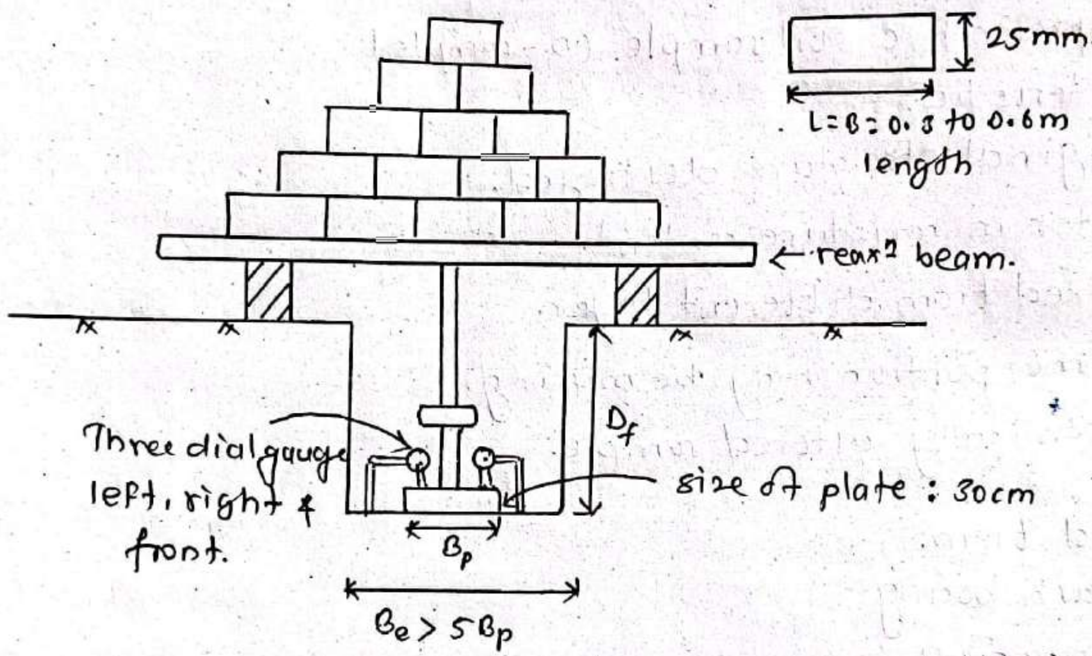
↳ not used in lab test.

8: Plate load test:

Objective:

↳ to determine ultimate bearing capacity and settlement for shallow foundation.

Test arrangement:



Procedure:

1. Apply the setting load of 5 kN/m² for some time.
ii. 1st test load : 25% of design load.
   ↳ increase load stage by stage with increment at 25% of design load.
   ↳ last test load : 2.5 x design load.

Observation:

↳ dial gauge reading at each stage of loading at time pattern 0 sec, 15 sec, 30 sec, 1 min, 2 min, 4 min, 8 min, 16 min, 0.5 hrs, and each hrs interval until settlement rate reduced to 0.5 mm/hr.

Results:

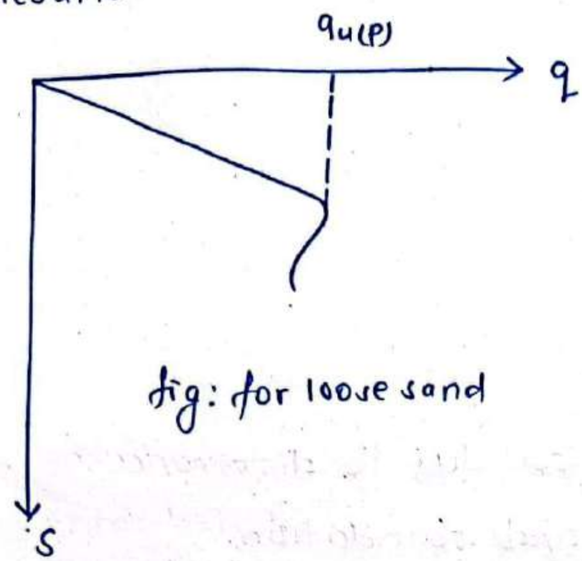


fig: for loose sand

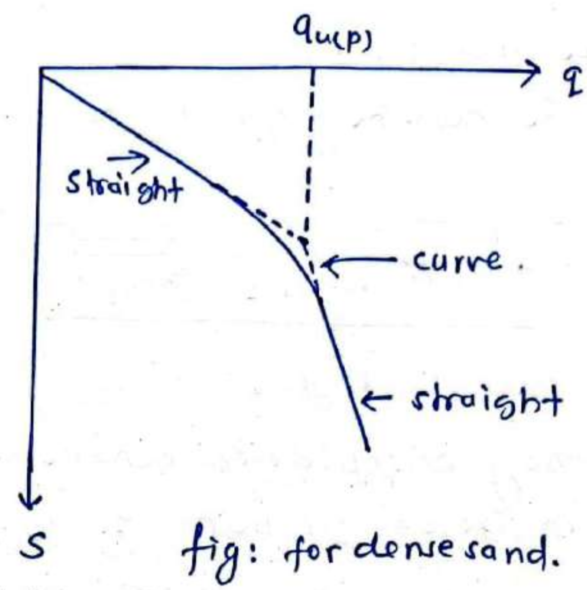


fig: for dense sand.

Analysis and calculation:

- for clay

$$q_{u(f)} = q_{u(p)}$$

$$S_f = S_p \times \frac{B_f}{B_p}$$; where

$q_{u(p)}$ = settlement of plate

$q_{u(f)}$ = settlement of foundation

S_p = settlement of plate

S_f = settlement of foundation.

B_p = size of plate

B_f = size of foundation.

- for sand:

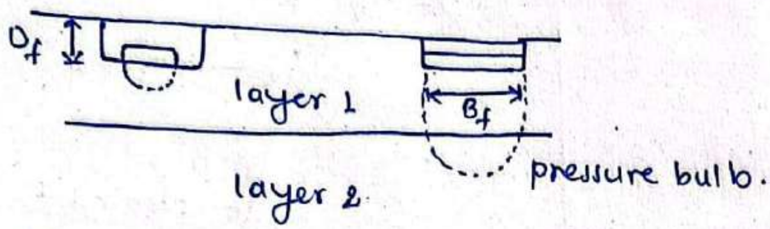
$$q_{u(f)} = q_{u(p)} \times \frac{B_f}{B_p}$$

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

*: Limitations of plate load test:

1. Size limitation

For non-homogenous soil:



↳ may misguide in non-homogenous soil due to difference in size of pressure bulb for plate and foundation.

2. Shape limitation:

↳ shape of plate may effect on result for other shape of foundation.

3. Scale limitation:

↳ conversion of results from plate to foundation may not represent truely for all conditions of soil.

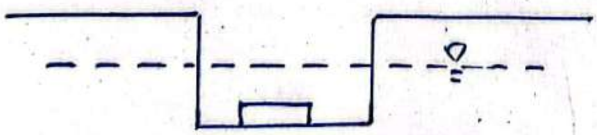
4. Time limitation:

↳ plate test: short term loading

↳ foundation: long term loading

Result from short term loading test may not represent long term effect in foundation truely.

5. GWT condition:



↳ During test below GWT, dewatering is done for test. The result doesnot include the true effect of GWT.

*: Site Investigation Report:

↳ site investigation report consists of:

Chapter 1: Introduction

- 1.1 Background of project.
- 1.2 Objectives
- 1.3 Scope
- 1.4 Limitation.

Chapter 2: Methodology

- 2.1 Boring.
- 2.2 Sampling
- 2.3 Ground water table observation
- 2.4 Insitu test methodology
- 2.5 lab tests
- 2.6 Bearing capacity analysis. (methods, formulas etc)
- 2.7 settlement analysis.
- 2.8 slope stability analysis (theory)
- 2.9 liquefaction analysis. (theory)

Chapter 3: Findings

- 3.1 soil layers description
- 3.2 Position of GWT.
- 3.3 shear strength parameter.

Chapter 4: Analysis:

- 4.1 Bearing capacity analysis.
- 4.2 settlement analysis
- 4.3 slope stability analysis.
- 4.4 liquefaction analysis.

chapter 5: Recommendation:

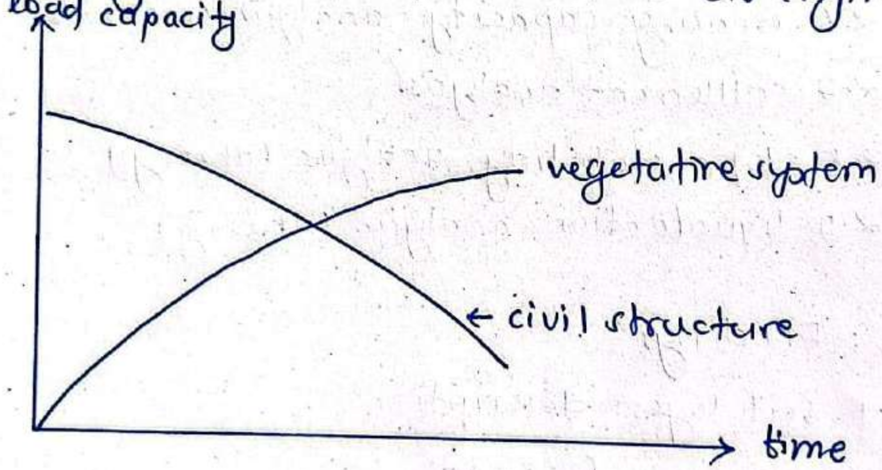
5.1 Type of foundation & bearing capacity.
5.2 Preventire meaure slope stability
5.3 Prerentire meaure for liquetaction.

Annex:

↳ photographs: field test, lab test, soil sample
↳ Bore hole log:
↳ summary sheet of lab test results.
↳ sheet of each lab test.

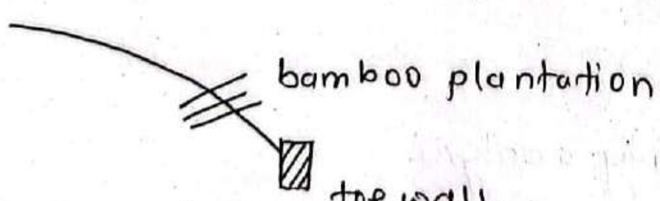
*: Bio-Engineering:

↳ slope stabilization using vegetative system with small civil engineering structure. is known as bio-engineering.

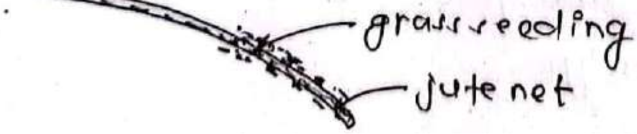


example:

① toe wall + bamboo plantation



⑪ jute net + grass seeding

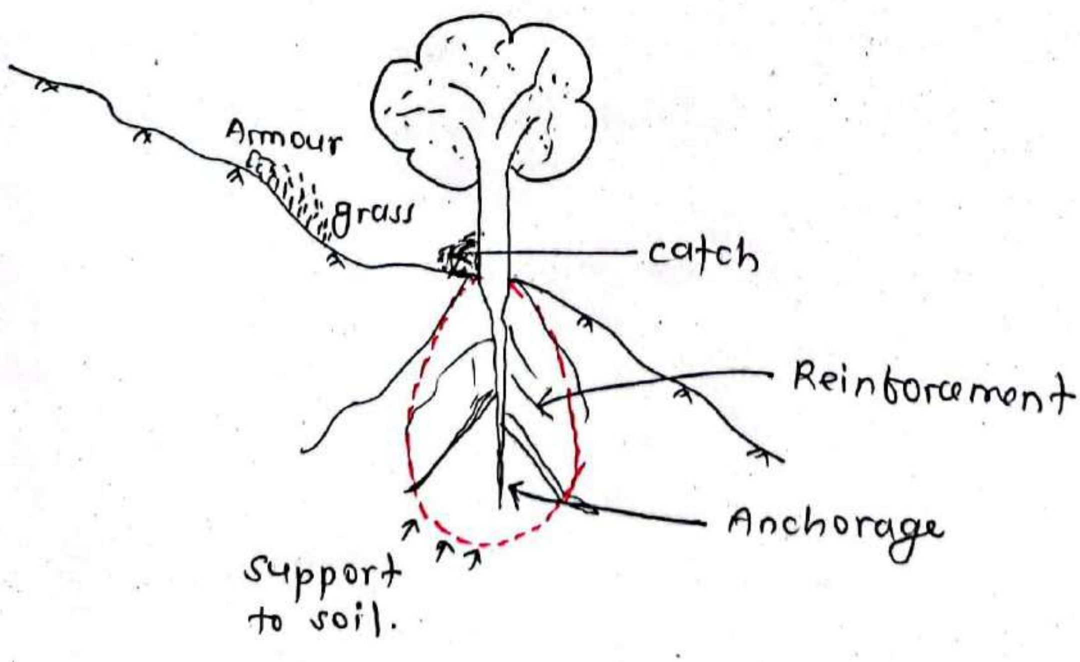


civil engineering structures
- toe wall
- brest wall
- detention wall
- bolster
- checkdam
- jutenet
- wirenet
- battle fence

vegetative system.
- grass plantation
- shurbs / herbs
- tree
- bamboo
- brush layering
- palisade
- fascine
- Live checkdam.

Drainage:
- cascade
- catch drain
- stone rip-rap
- french drain

*: Functions of vegetative system:



Advantages:

(i) environment friendly
(ii) donot require special design
(iii) no need of trained manpower (local people, local technology)
(iv) only one solution (may be ultimate solution)
(v) long last solution
(vi) economical way of stability (अलैंची, bambo etc grow गर्ने!)

Disadvantages:

(i) donot function immediately
(ii) flexible in application among different people.