Foundation Engineering

2.2.1 Introduction. Factors Affecting selection, Types of foundation, requirements and criteria of foundation, types of load for of ideal foundation, criteria for depth foundation design, selection.

Methods of exploration of soil strata. [5]
Type of foundation : See real book Pg 2, 3, 4

(i) Foundation Engineering:
→ application of soil mechanics and rock mechanics in the design of foundation of various structures.

* **Factors Affecting selection of foundation:**

(i). Structure and its load:
→ heavy structures may need superior foundation types such as mat & pile. Bridge needs pier foundation, caissons, etc.
→ Load variation in column greater ⇒ mat foundation.

(ii). Bearing capacity and other engineering property of soil:
→ For strong soils isolated footing may be preferred.

→ For weak soils mat may be preferred.
→ when hard, strong stratum at certain depth below the economical shallow footing depth ⇒ deep foundation

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(iii) Economy:
Generally, interms of economy,
Isolated footing > strap footing > combined > mat > deep foundations
For normal residential buildings → spread/isolated footing
→ strip footing for masonry
For larger commercial buildings → mat foundation.

(iv). Need of client
(v) service life of structure
(vi). Environmental considerations, etc.

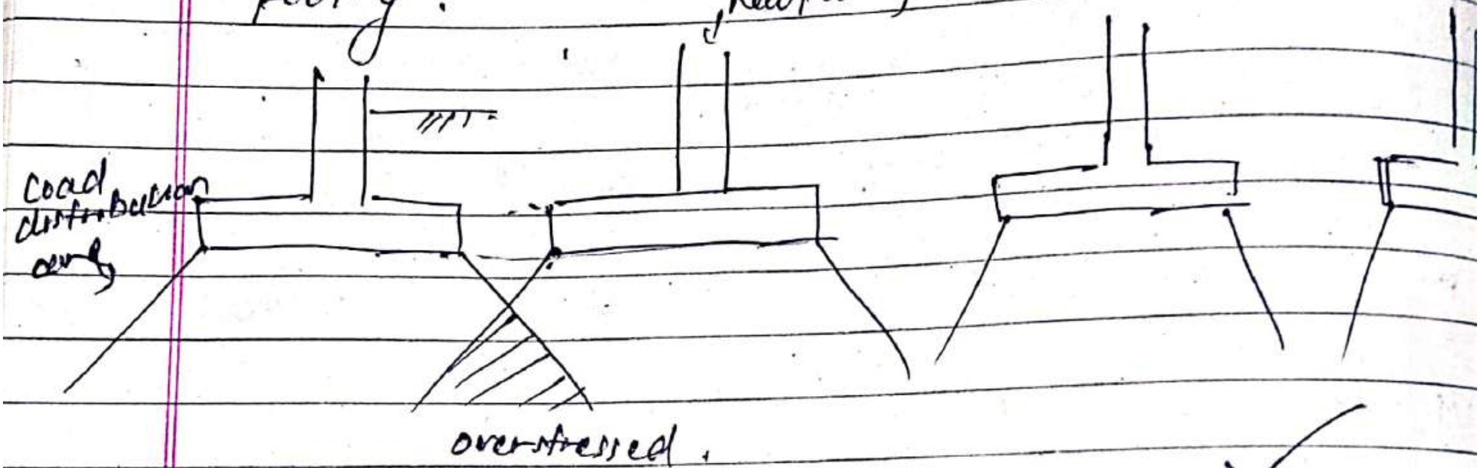
* <u>Requirements and criteria of ideal foundation:</u>

(i). Depth of top soil:
→ top soil consist of organic matters. So, must be removed. foundation should always be below the top soil layer
→ A foundation should never be exposed to the surface.

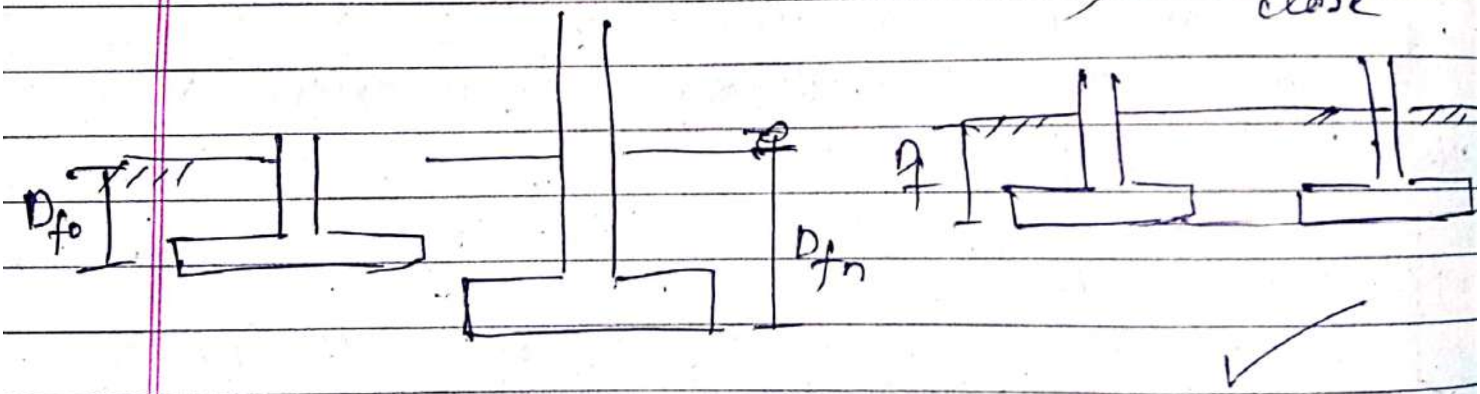
(ii) Frost Depth:
→ footing should be placed below the frost penetration depth to prevent damage from <u>frost heave</u> (ice lenses). uplift and on melting settles down

(iii) Zone of soil volume change — ज्यादा तल हुनु पर्छ
Eg: black cotton soil expands highly on wetting and shrinks on drying that may lead to differential settlement.

(iv) Adjacent footings and property lines:
नयाँ footing near and existing structure adds up stress to the soil beneath. So, should be certain distance far from existing footing.

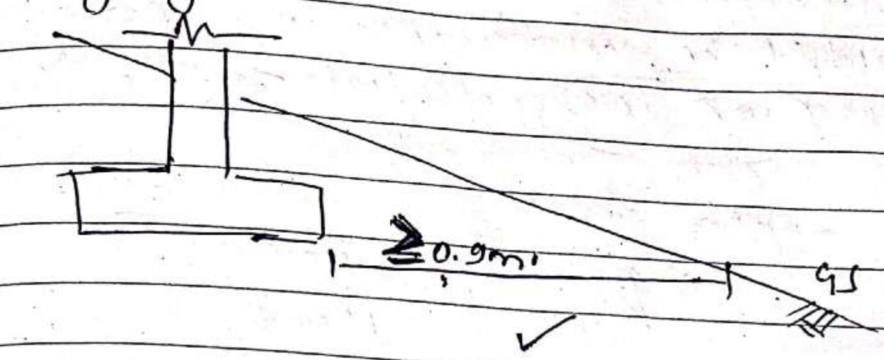


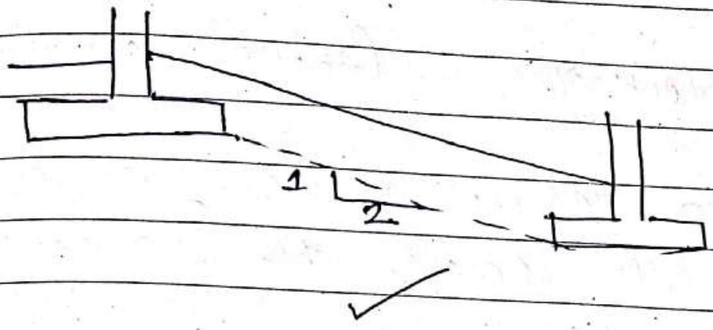
shouldn't be too close

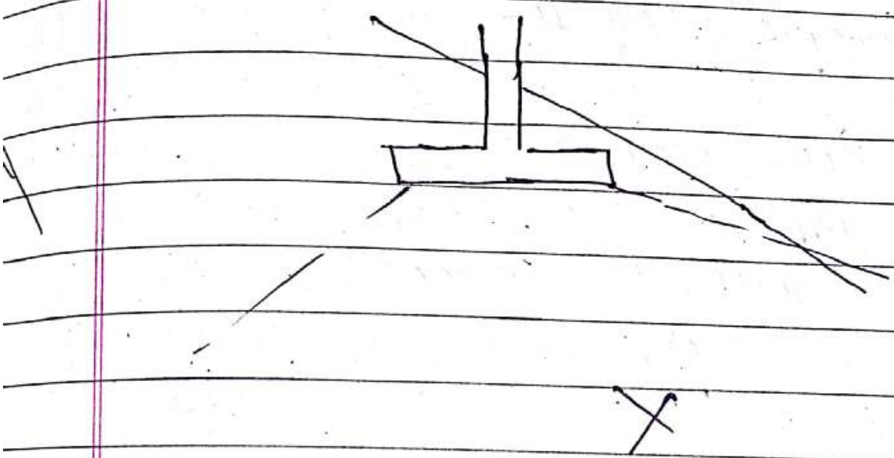


should be at same depth as far as possible

(v) Sloping ground :







(vi) water table :

→ should be above GWT as far as possible as reduces bearing capacity

→ difficult during construction.

(vii) Scour depth :

→ in case of bridge piers. depth should be > than the potential scour d

$R = 1.35\, q^2/f$

(viii). Underground defects:
→ shouldn't be constructed over ~~poter~~ caves, sink hole, potential zones, etc.

(ix). Minimum depth

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

q ⇒ loading intensity ⇒ Rankine's formulae.

IS at least 0.5 m below ground for soil and for rocks just remove the top soil.

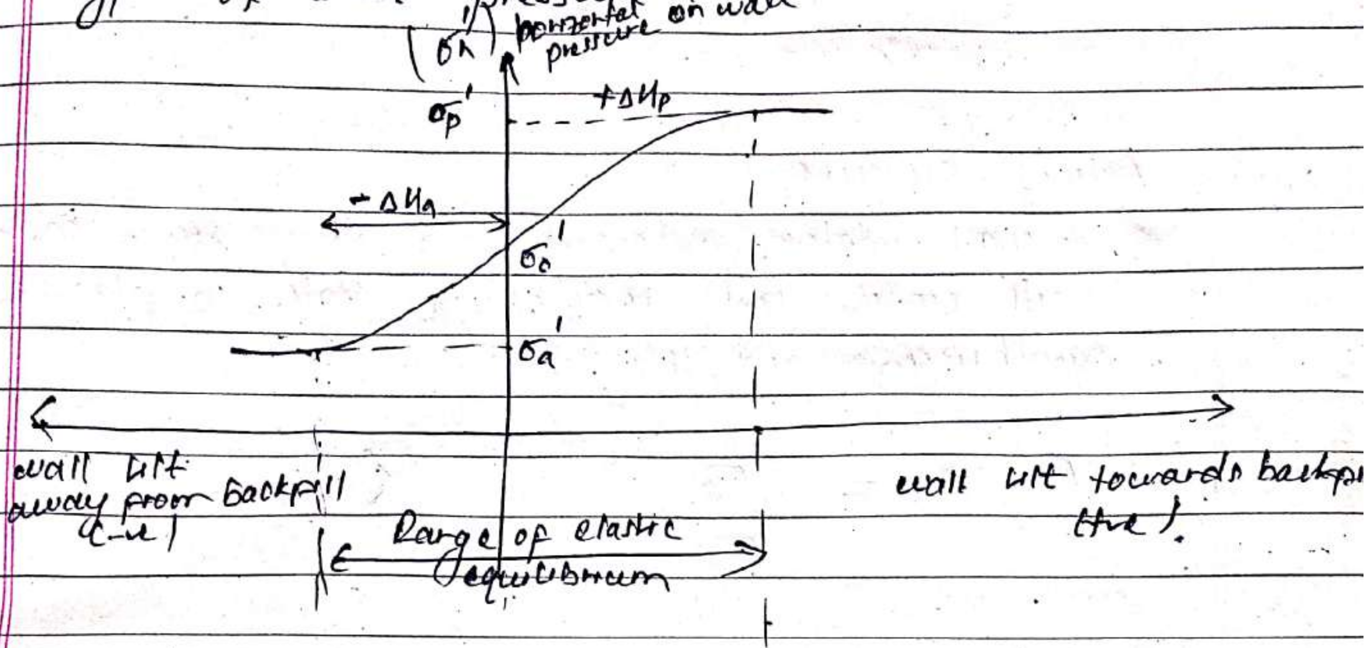
Loads (see Hemraj.oli Pg 117)

Criteria of an ideal foundation:
→ should safely transmit loads of superstructure to the ground without exceeding its bearing capacity along with a safety margin.

→ differential settlement shouldn't occur or should be on within permissible limits.

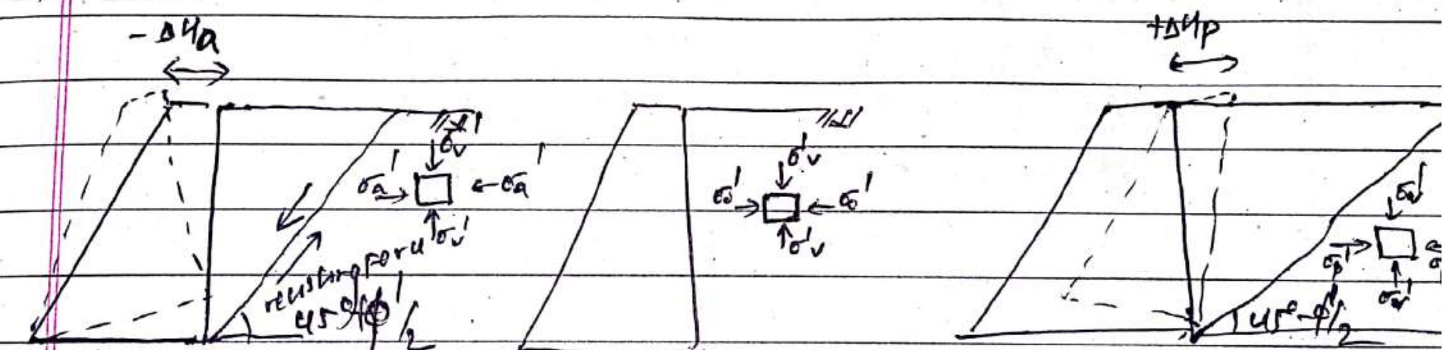
→ Economically justificable.

Types of earth pressure



$P_p > P_o > P_a \Rightarrow K_p > K_o > K_a$

~~(1) At rest~~



Active Earth Pressure

At rest earth pressure

Passive Earth Pressure

(1) At rest earth pressure (σ_o')

→ wall is static

→ i.e. no movement of wall and soil

→ lateral pressure exerted on the walls by soil in this condition is ~~at~~ σ_o'

$$K = \frac{\sigma_h}{\sigma_v'}$$

$$\therefore K_0 = \frac{\sigma_0'}{\sigma_v'}$$ ⇒ coff. of ~~active~~ at rest earth pressure

(ii). Active pressure :

→ wall rotates outward and away from back-fill until soil reaches a state of plastic equilibrium and thus fails.

$$K_a = \frac{\sigma_a'}{\sigma_v'} < 1$$

(iii). Passive pressure

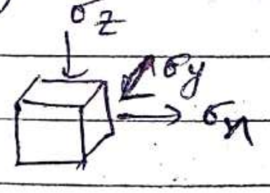
→ wall rotates inwards towards back fill until a triangular wedge reaches a state of plastic equilibrium.

$$\therefore K_p = \frac{\sigma_p'}{\sigma_v'}$$

Determination of at rest earth pressure

→ ∵ in elastic zone using theory of elasticity

$$\epsilon_x^{\,0} = \frac{\sigma_x \cdot 0}{E} - \frac{\mu}{E}(\sigma_y + \sigma_z)$$



$$\frac{\sigma_x}{E} = \frac{\mu \sigma_z}{E} - \cancel{\mu\sigma}$$

$$\frac{\sigma_x}{\sigma_z} = \frac{\mu}{1-\mu}$$

But v.H, E varies with depth. So,
empirical, $K_o = (1-\sin\phi')$ (Jaky)
⇒ Loose sand

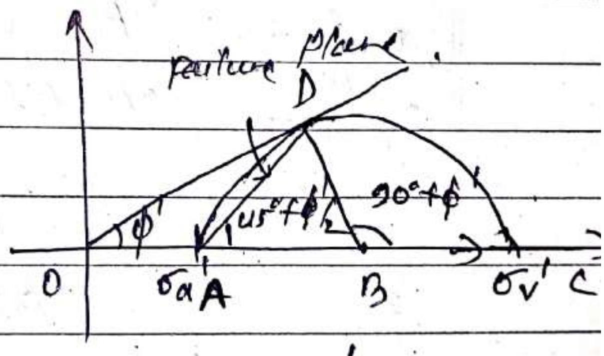
$K_o = (1-\sin\phi')(OCR)^{\sin\phi'}$ — Mayne, Kulhawy
⇒ clay to gravel

* Rankine's Theory:
Assumption:
→ soil is dry and cohesionless
→ wall surface of the retaining structure is smooth & vertical.
→ The backfill is plane: horizontal or inclined.
→ The failure surface is a plane passing through the heel of the wall.
→ The soil element is in a state of plastic equilibrium i.e. verge of failure.
→ soil is homogeneous, isotropic and semi-infinite

Derivation
a) Active Earth Pressure

$\sigma_a' = K_a \gamma H$

$= \left(\frac{1-\sin\phi'}{1+\sin\phi'}\right)\gamma H$



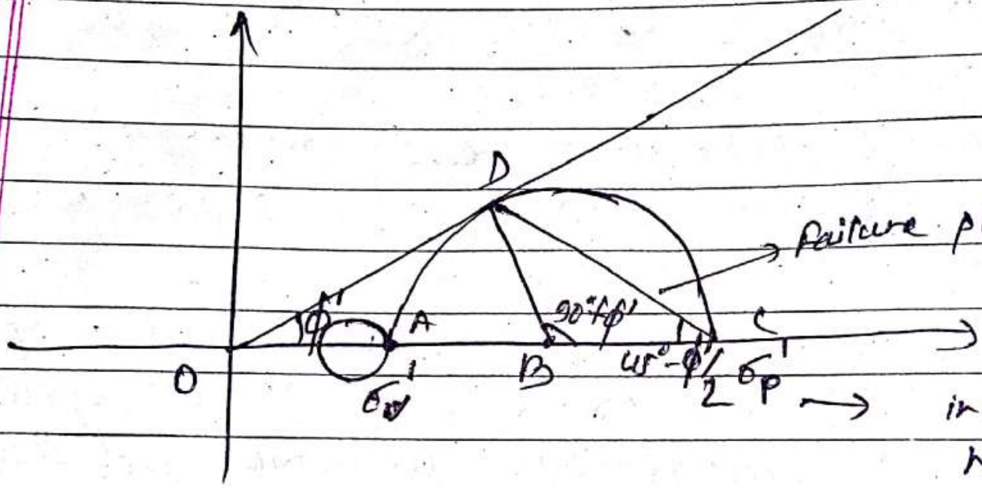
$\sigma_a' = OA = OB - BA$
$= OB - BD = OB - OB\sin\phi$
$= OB(1-\sin\phi')$ — ①

$$\sigma_v' = OB + BC = OB + OB\sin\phi'$$
$$= OB(1+\sin\phi')$$

$$\frac{\sigma_a}{\sigma_v'} = \frac{OB(1-\sin\phi')}{OB(1+\sin\phi')} = \frac{1-\sin\phi'}{1+\sin\phi'}$$

$$\sigma_a = \frac{1-\sin\phi'}{1+\sin\phi'}\,\sigma_v' = \phi$$

b) Passive.



→ inclined with horizontal পৰিছে
Horizontal

$\sigma_v' = OA = OB - AB = OB(1-\sin\phi')$ | σ_p' হৈ সৰ্বাধিক। আৰু σ_a নিম্ন

$\sigma_p' = OB + BC = OB(1+\sin\phi')$

$$\frac{\sigma_p'}{\sigma_v'} = \frac{\cancel{OB}(1+\sin\phi')}{\cancel{OB}(1-\sin\phi')}$$

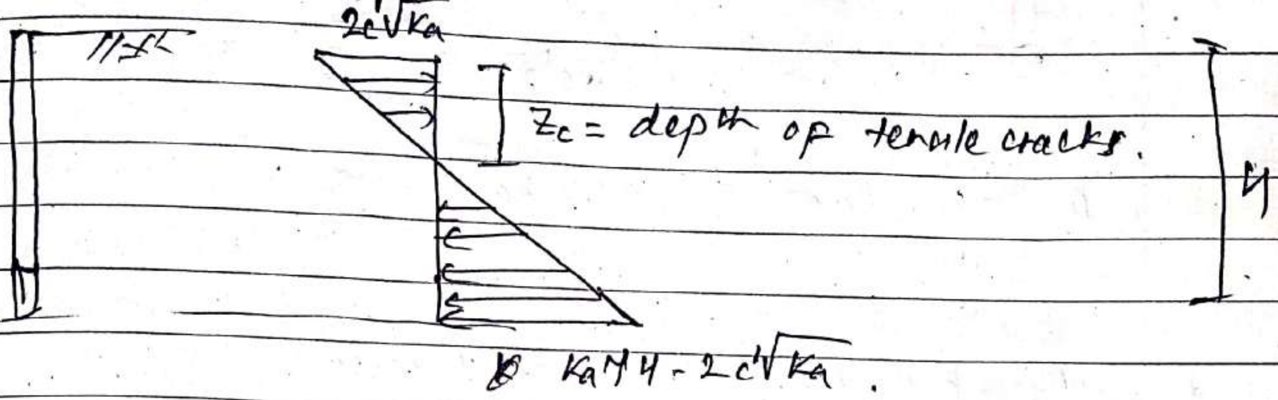
or, $\sigma_p' = K_p\,\sigma_v'$

$$= \frac{1+\sin\phi'}{1-\sin\phi'}\,\text{MH}$$

For inclined consider text book.

Similarly for cohesive soil.

$K_a \Rightarrow$ $\sigma_a' = K_a \sigma_v' - 2c'\sqrt{K_a}$



$Z_c \Rightarrow$ At $H = Z_c$ $\sigma_a' = 0$

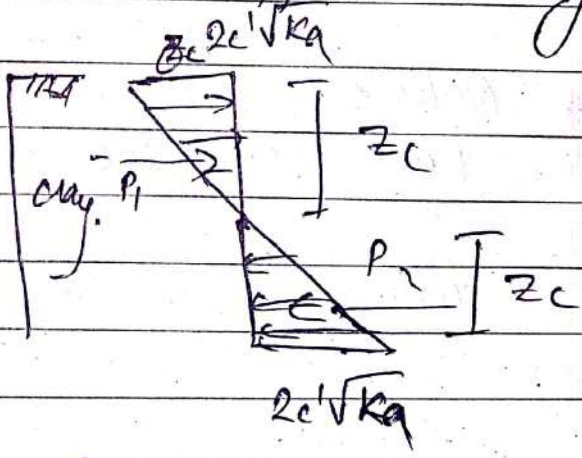
$0 = K_a \gamma Z_c - 2c\sqrt{K_a}$

or, $Z_c = \dfrac{2c'}{\gamma\sqrt{K_a}}$

critical height of unsupported clay

$H_c = 2Z_c$

$= 2 \times \dfrac{2c'}{\gamma\sqrt{K_a}}$

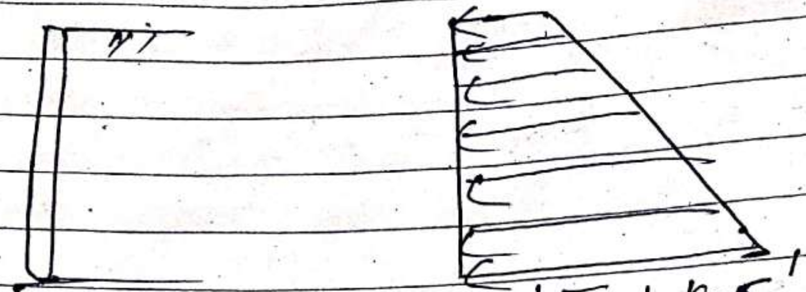


For $\phi' = 0$

$H_c = \dfrac{4c'}{\gamma}$

$P_1 = P_2$

b) Passive.



$$\sigma_p' = K_a\sigma_v' + 2c\sqrt{K_a} \quad \#$$

* Coloumb's Earth Pressure Theory:

Assumptions:

(i) Soil is dry, cohesionless, homogeneous, isotropic and ideally plastic material.

(ii) The back of the retaining wall is ~~smooth~~ rough. So, wall friction is taken into account

(iii) Failure surface is a plane through the heel of the wall.

(iv) Sliding wedge acts as a rigid body.

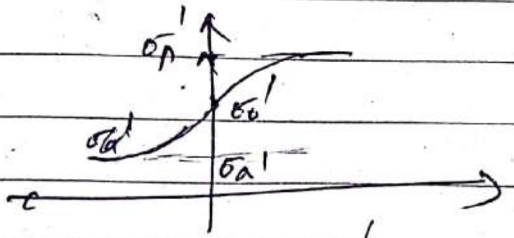
→ more general than Rankine's theory as it directly deals with forces involved.

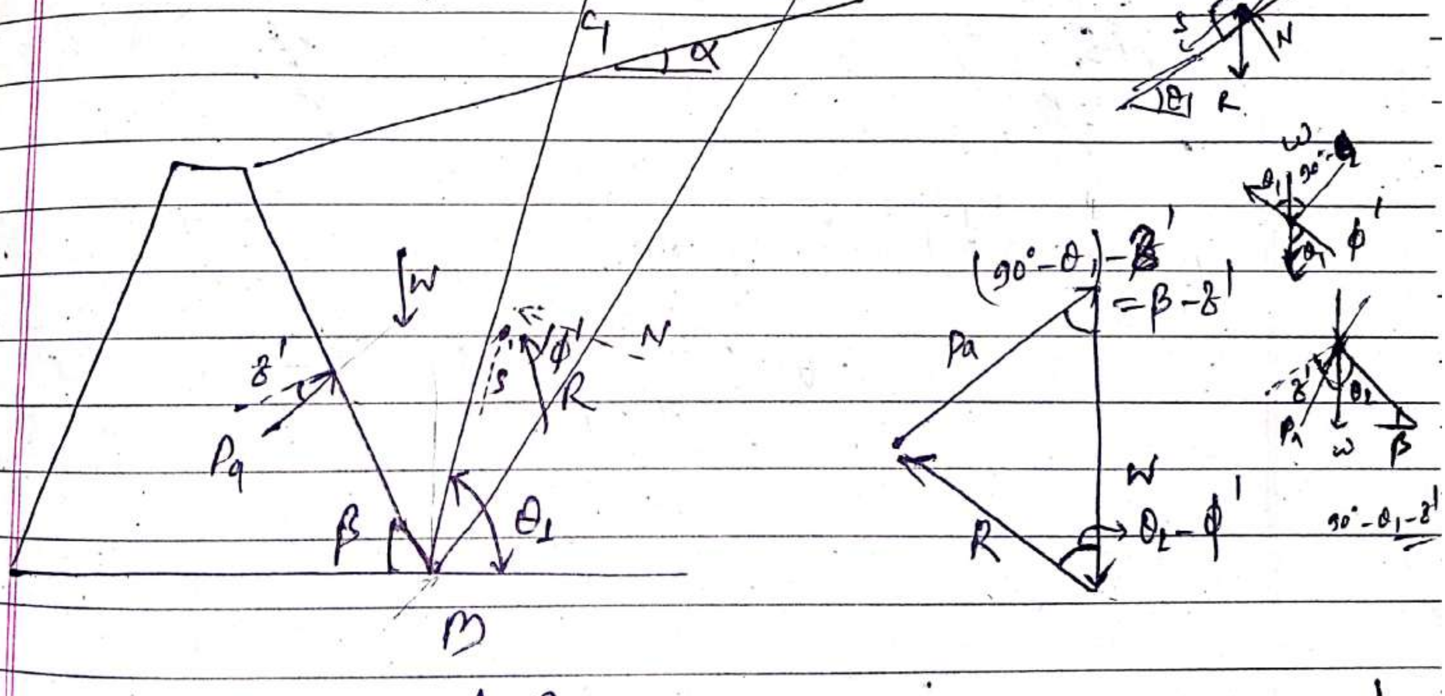
a failure plane assumed and the lateral force required to keep the wedge in equilibrium is found.

procedure repeated for a number of failure planes

For active case: the surface giving the maximum force is the failure plane

For passive: " " minm force is the failure plane. why!





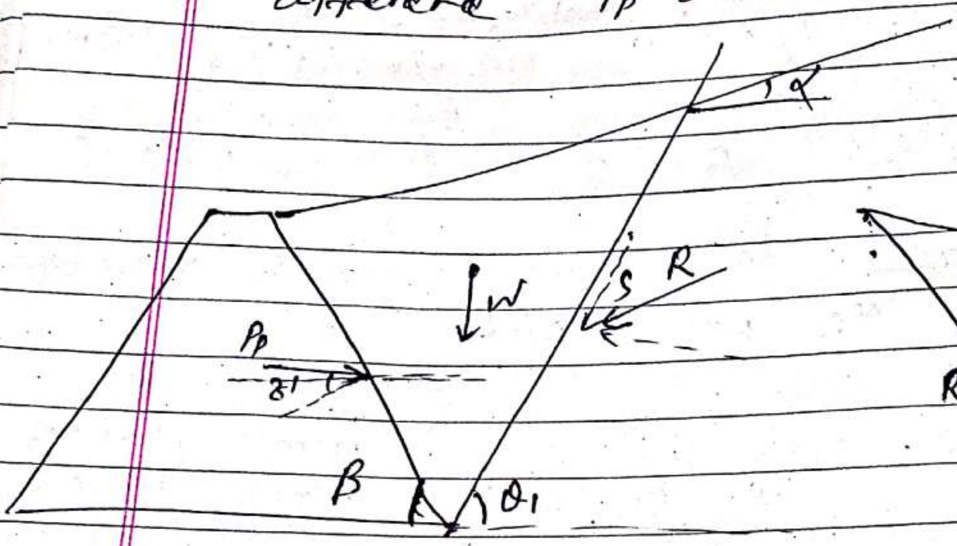
using sine law

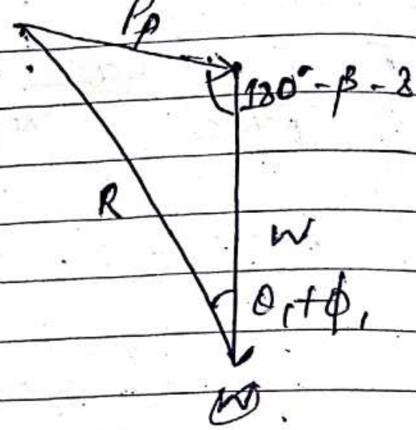
$$K_a = \frac{\sin^2(\beta+\phi')}{\sin^2\beta \sin(\beta-\delta')\left[1+\sqrt{\frac{\sin(\phi'+\delta')\sin(\phi'-\alpha)}{\sin(\beta-\delta')\sin(\alpha+\beta)}}\right]^2}$$

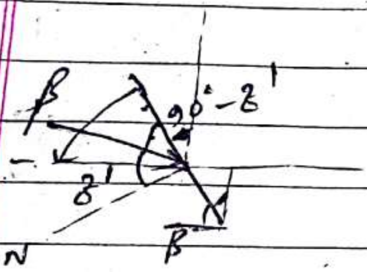
$90°-\theta_1$

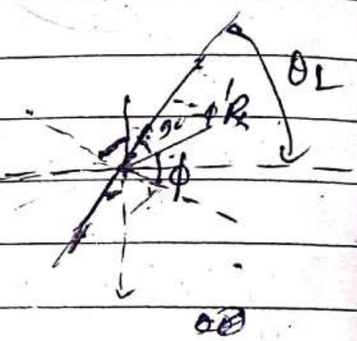
Passive:

difference P_p directed, relative downwards

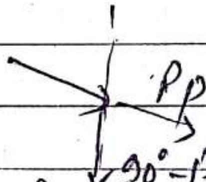






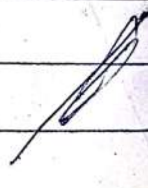


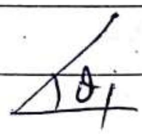
$\beta - (90° - \delta')$

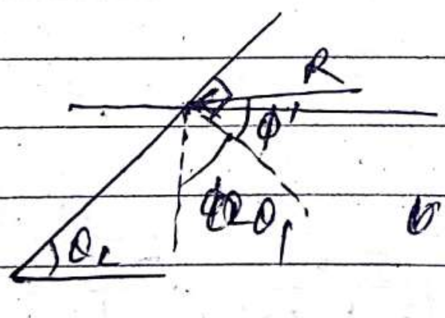


$90° - \beta + 90° - \delta'$

$180° - \beta - \delta'$







circle.

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$$K_p = \frac{\sin^2(\beta - \phi')}{\sin^2\beta \, \sin(\beta + \delta') \left[1 - \sqrt{\frac{\sin(\phi' + \delta')\sin(\phi' + \alpha)}{\sin(\beta + \delta')\sin(\beta + \alpha)}}\right]^2}$$

* Trial wedge Theory:

→ for cohesionless soil

→ ∵ analytic method lengthy and monotonous. instead we can use graphical method.

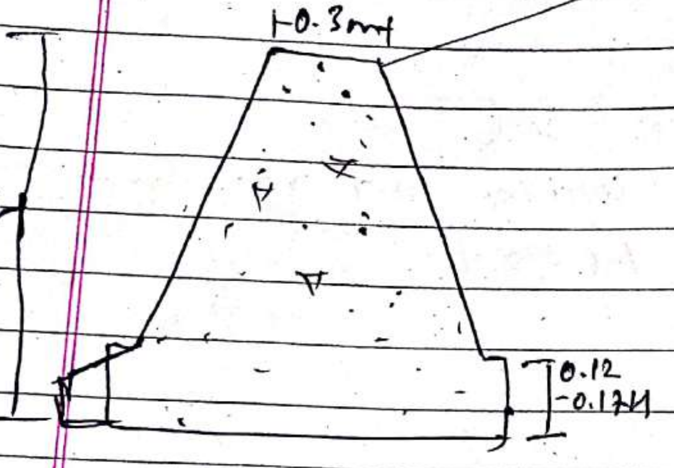
* Types of retaining walls:

(i) Gravity : stability fully by weight. uneconomical for higher

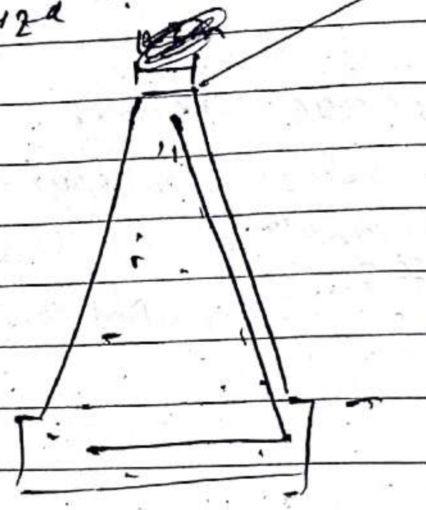
(ii) Cantilever : 8m

(iii) Counterfort : counterfort: tie two wall and slab. similar to cantilever

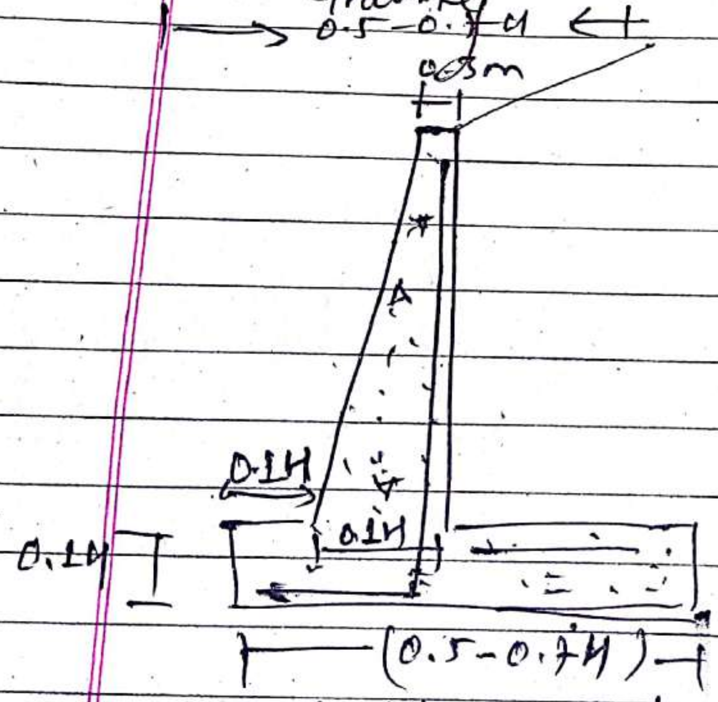
(iv) Semi-gravity : steel used to minimize section size.



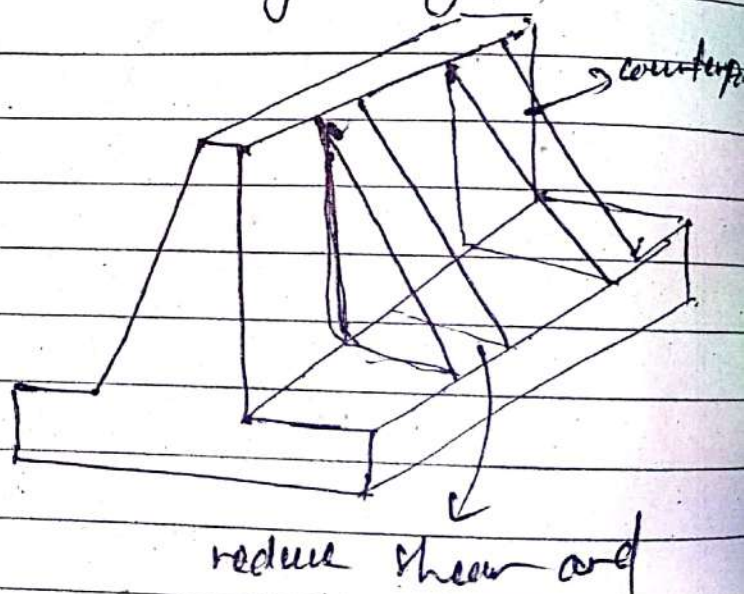
Gravity



Semi-gravity.



Cantilever wall



reduce shear and BM.

ϕ min^m

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* Stability analysis:
  → similar to hydro.

FOS overturning ≥ 2
" " sliding ≥ 1.5
" " bearing ≥ 3

@ middle third rule.
$e \leq B/6$ for
. no tension.

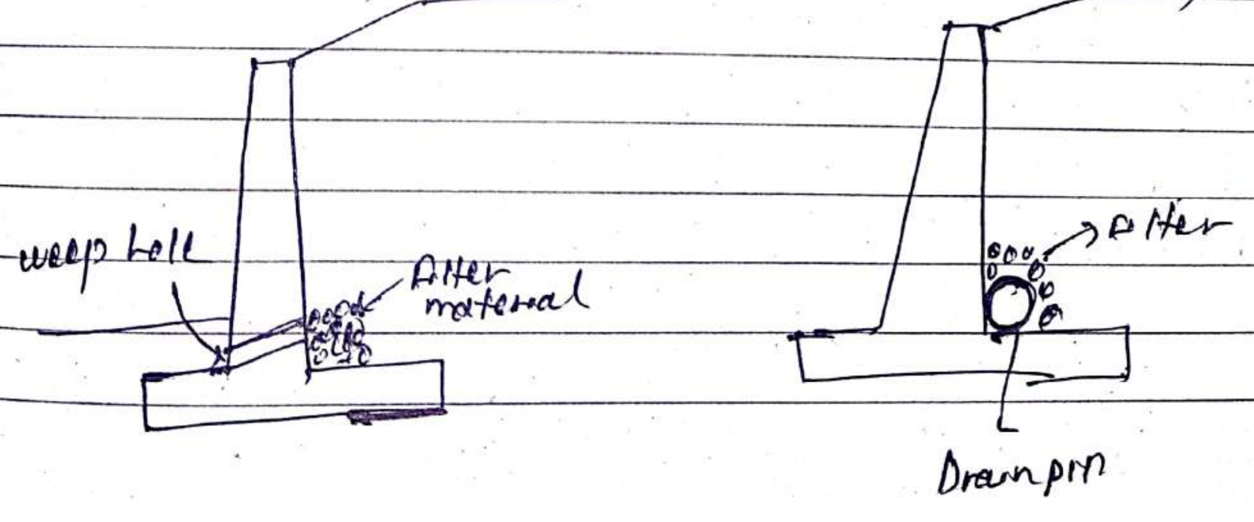
* Techniques to increase the stability of retaining walls.

(1). Proper drainage from backfill:
→ water makes saturated ↑ pressure lead to unstability. So drainage required.

→ weep holes are provided (0.1m min dia) and adequately spaced.

→ to prevent clogging of weep holes and drain pipes filter provided behind weep holes.

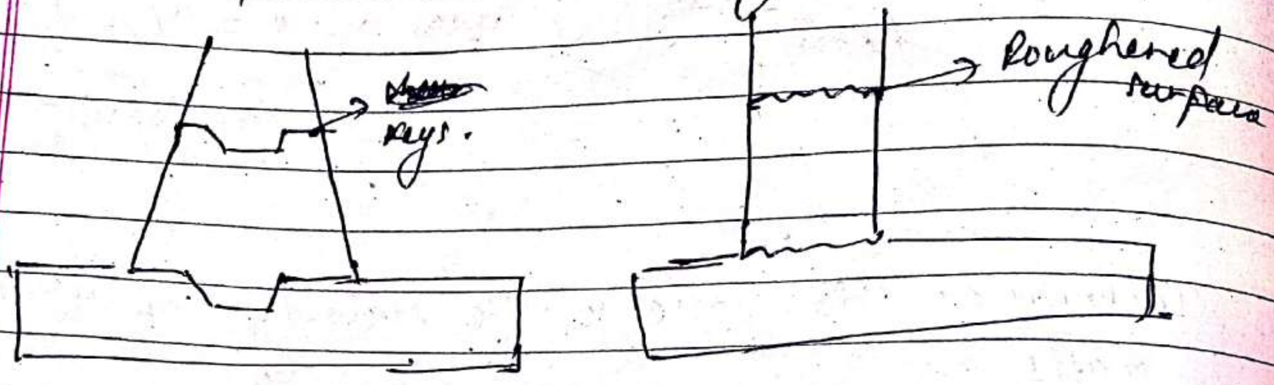
$$\frac{D_{15}(F)}{D_{85}(B)} \leq 4 \text{ to } 5 \leq \frac{D_{15}(F)}{D_{15}(B)}$$

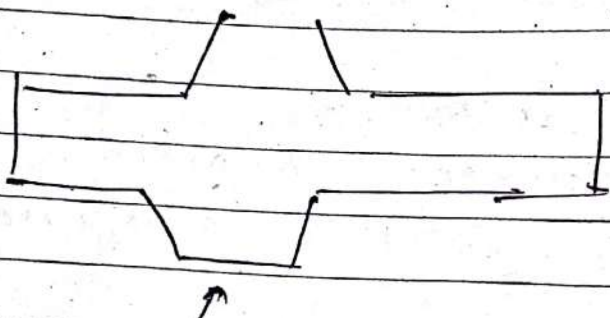


(11). Providing proper contraction joints, expansion and cont

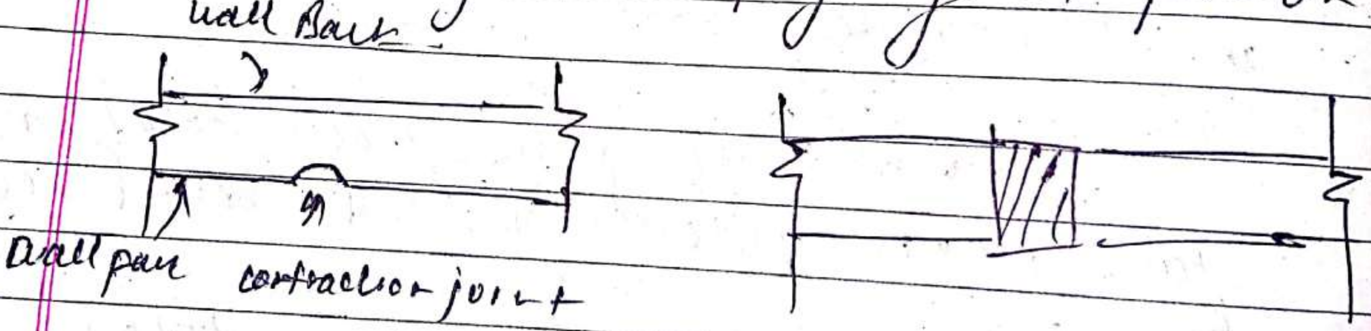
→ shear keys at construction joints

→ or, ~~the~~ surface should be roughered.





Shear keys for safety against friction.



Plan. Expansion joint.

(iii). Mechanically stabilized retaining walls:

→ just like rebar in concrete we can make an incredibly strong composite material with soil by adding soil reinforcing elements.

→ retaining wall created using such soil reinforcements is known as mechanically stabilized earth walls (MSE walls).

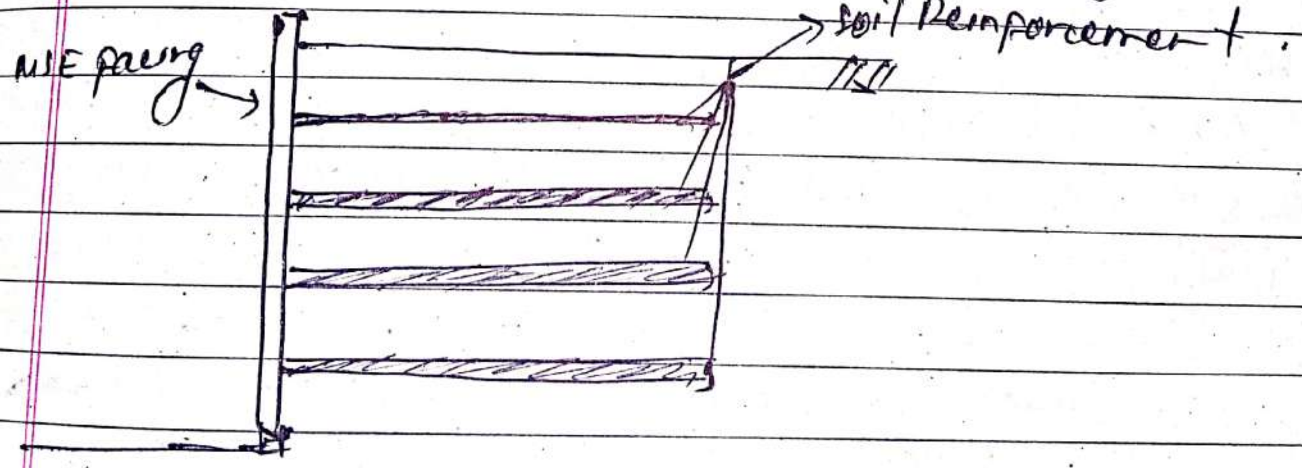
(Road side को fill को wall हो).

<u>mechanism</u>

→ tension in reinforcement generates confining pressure in the soil. This pressure acts perpendicularly to the failure plane, increasing the shear strength of soils (sand)

$$\tau_f = \cancel{c'}^{\,0} + \sigma' \tan\phi'$$

σ' ↑ ↑ strength ↑



compacted soil added in layers with reinforcement in betⁿ each layers.

→ relatively new concept - | vidal, France 196

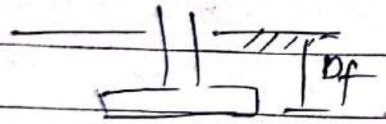
Reinforcement element ⇒ metallic strips, geotextiles, geogrids.

2.2.3. Bearing Capacity and settlements :

(i) Types of bearing capacity and factors influencing bearing capacity, modes of foundation failure, Terzaghi's general bearing capacity theory, ultimate bearing capacity of cohesive and cohesionless soils, settlements: type nature and effects) .

* Types of bearing capacity (see ~~pdf~~ word doc. in comp).

(i) $q_u = \dfrac{\text{Failure}}{\text{Area}}$



(ii) $q_n = q_u - \gamma D_f$

no FOS for soil pressure Imp.

(iii) $q_{ns} = q_n / FOS$ (iv) $q_s = q_{ns} + \gamma D_f$

(v) q_{np} ⇒ Load without settlement failure beyond permissible limits (25mm – 40mm)   FOS ≥ 3

(vi) q_{na} (allowable) $= q_{ns}$ if $q_{ns} < q_{np}$

$= q_{np}$ if $q_{np} < q_{ns}$

* Factors Influencing Bearing Capacity :

(i)

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

(i). Foundation dimension :

$B \uparrow \quad q_u \uparrow$

$q = \gamma D_f \quad D_f \uparrow \quad q_u \uparrow$

(b) Soil type

$\phi' = 0$, $N_q = 1$, $N_\gamma = 0$

$q_u = c'N_c + q$ $= c'N_c + \gamma D_f$

$N_c = 5.7$ (5.14)

$c' = 0$, $q_u = qN_q + 0.5\gamma B N_\gamma$

(c). water table:

(d) Soil failure mode:

Local shear failure. $c'_l = \frac{2}{3} c'$

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

c', ϕ for general failure.

#. What are the effects of a) breadth b) depth and c) GWT on soil bearing capacity. (PSC)

How do you determine the bearing capacity of the soil? Factors influencing bearing capacity (PSC)

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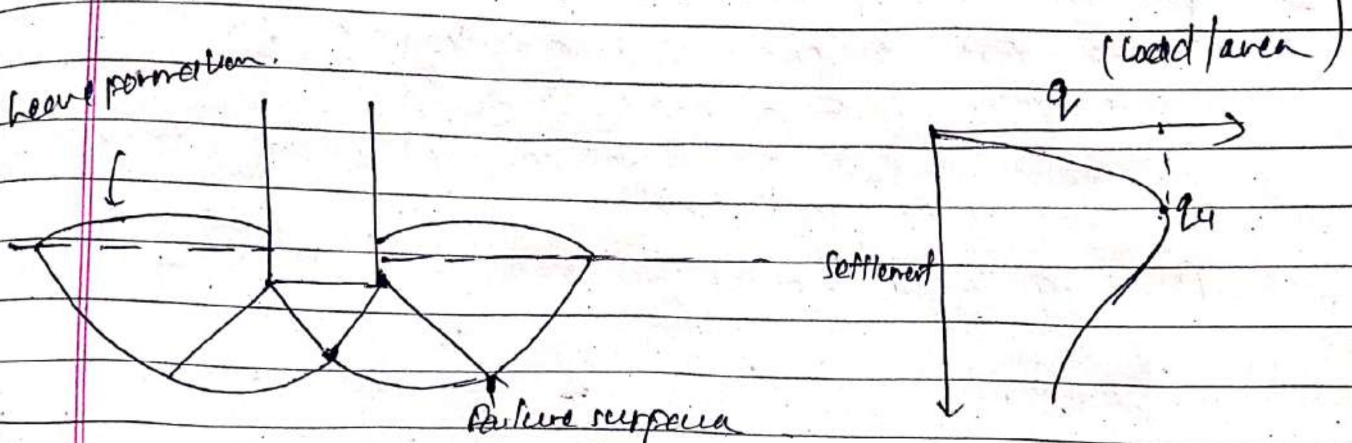
*. Modes of foundation failure :

(i) General shear failure :



→ occurs in dense sand and stiff clay

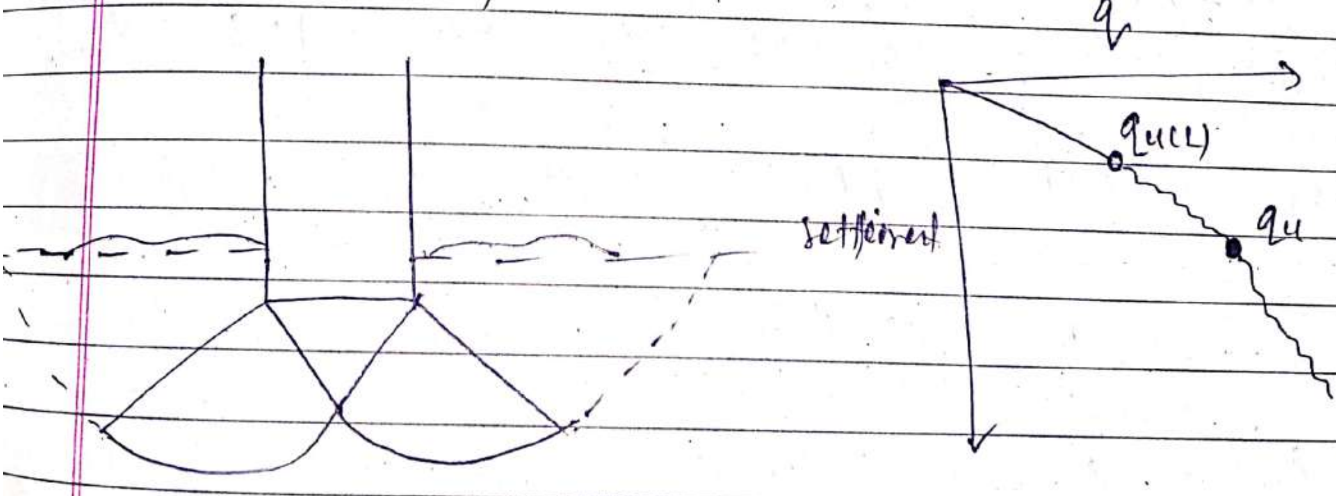
→ sudden in nature

→ failure surface extend to the ground.

→ large heave formation around the foundation

→ failure pattern is well-defined.

(ii) Local shear failure



→ occurs in medium dense sand and medium stiff clay.

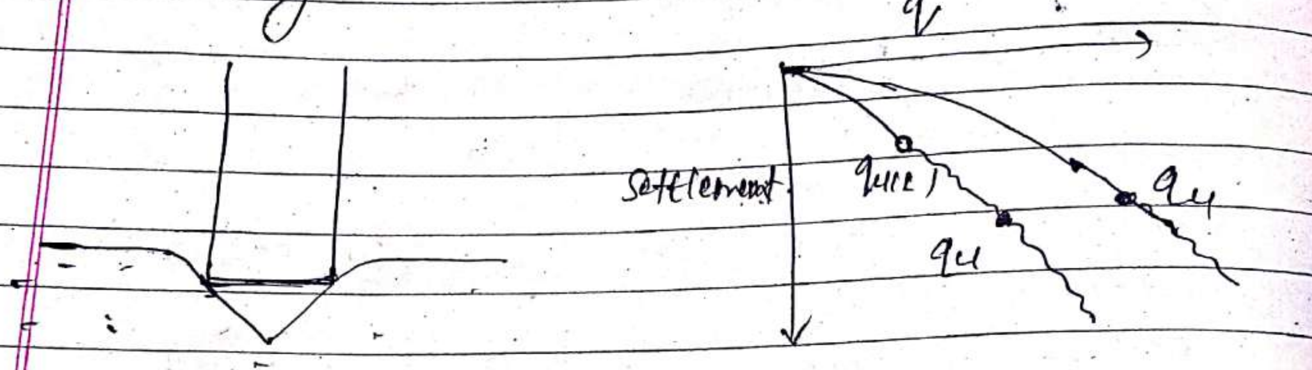
→ first failure load .

sudden

→ when $q = q_{u(L)}$ movement accompanied by sudden jerks.
→ failure surface पनि solid line जस्तो (हुँदैन)
ground मा entered हुँदैन considerable settlement हुनै पर्छ . अनि heave form हुँदैन
substantial settlement needed.

→ q_u value is not realized.

(iii) Punching Shear Failure :



→ fairly loose soil मा
→ failure surface doesn't extend to the ground.

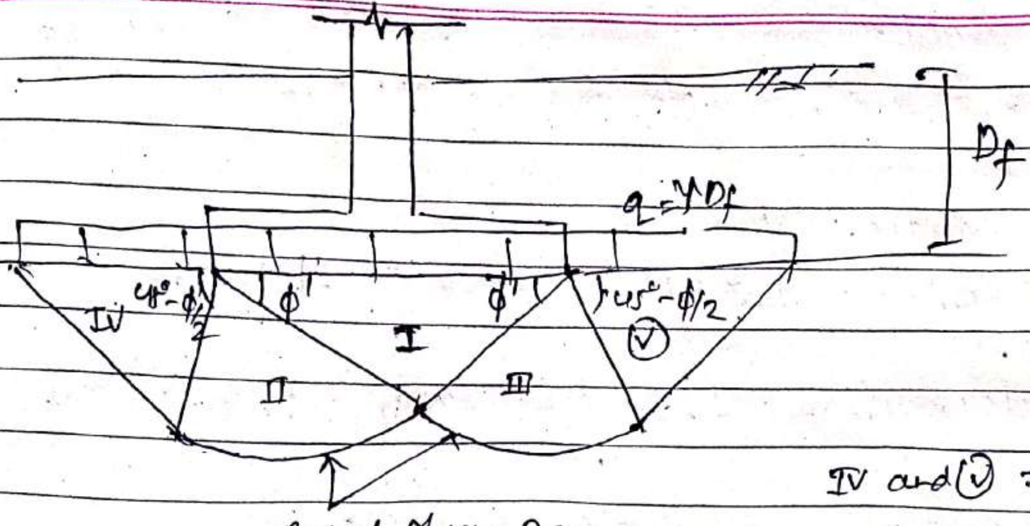
* Terzaghi's General Bearing Capacity Theory.
Assumptions : see नोट (Book) Pg. 148.

$$q_u = c'N_c + qN_q + \frac{1}{2}\gamma B N_\gamma$$

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

$$\boxed{q_u = c'N_c + qN_q + 0.5\gamma B N_\gamma}$$

$\Rightarrow$ Terzaghi's bearing capacity equation.



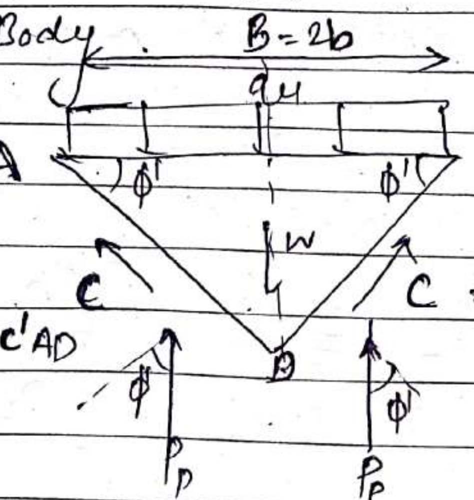
Radial shear zone.

IV and (V) => Passive Earth pressure zone.

I => Rigid Body.

q_u ?

main
Δ wedge equilibrium

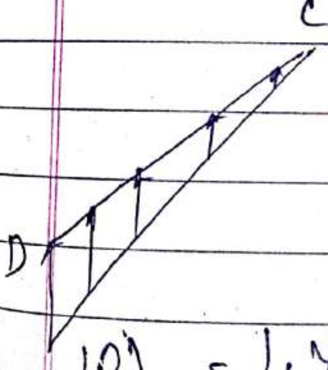


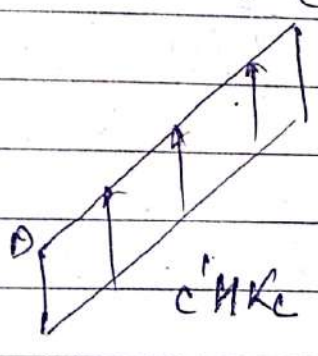
$C = c'DC = \frac{c'b}{\cos\phi'}$.

$$q_u B(1) + W = 2C\sin\phi' + 2P_p \quad \text{— (I)}$$

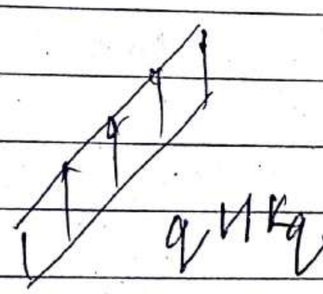
$$P_p = (P_p)_c + (P_p)_q + (P_p)_\gamma \quad \text{— (II)}$$

face DC — cohesion — surcharge — selfweight





$c'HK_c$



$q\gamma K_q$

$(P_p)_{\gamma'} = \frac{1}{2}\gamma H^2 K_p$

γ'

$q_u = c'N_c + qN_q + \ldots$

algebra

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

Rect square में

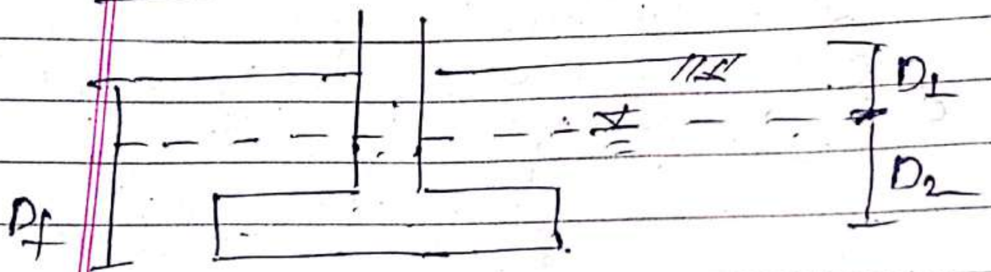
$q_u = 1.3c'N_c + qN_q + 0.4\gamma B N_\gamma$

Circular $q_u = 1.3c'N_c + qN_q + 0.3\gamma B N_\gamma$

* Modification for water table:

→ WT लोकेशन के लिए modification चाहिए।

Case I: $0 \le D_1 \le D_f$

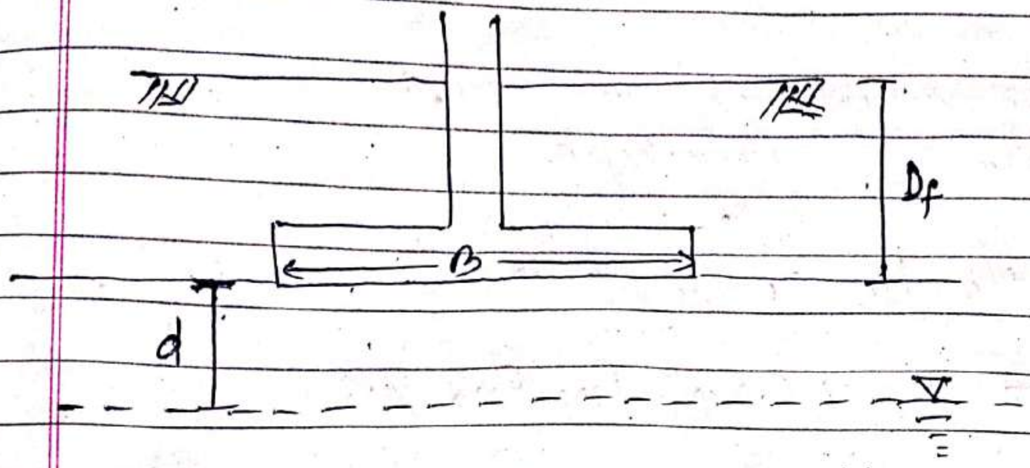


$\bar{q} = \gamma D_1 + (\gamma_{sat} - \gamma_w) D_2$ ⇒ effective surcharge

Also, replace γ of last term by γ'

$q_u = c'N_c + \bar{q}N_q + 0.5\gamma' B N_\gamma$

Case II : $0 \le d \le B$



$q = \gamma D_f$ (same as normal)

But $\bar{\gamma} = \gamma' + \frac{d}{B}(\gamma - \gamma')$

यहाँ ∵ γ आएको weight of soil below the footing सँग Related (Δ wedge)

यदि $d = 0$ सबै डुबेको हुन्छ

$\bar{\gamma} = \gamma' + 0 = \gamma'$

यदि $d = B$ हुँदा dry अवस्था हुन्छ (wedge माथि छैन)

$\bar{\gamma} = \gamma' + 1(\gamma - \gamma')$

$= \gamma$

Practice : $\bar{\gamma} = \gamma' + \frac{d}{B}(\gamma - \gamma')$

$\bar{\gamma} = \gamma' + \frac{d}{B}(\gamma - \gamma')$

Case III : $d \geq B$ $\quad \bar{\gamma} = \gamma$ normal

no effect on bearing capacity.

Terzaghi assumption ठिक छैन! non-realistic.

correction $\Rightarrow$ Meyerhoff

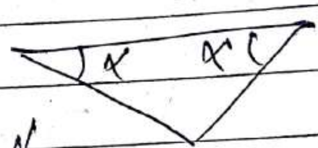
$$q_u = c' N_c F_{sc} F_{dc} F_{ic} + q N_q F_{sq} F_{dq} F_{di}$$
$$+ 0.5 \gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}$$

$F_{\gamma s} \Rightarrow$ shape factor $\Rightarrow$ ~~कुनै पनि shape लाई work~~

$F_{\gamma d} \Rightarrow$ depth factor $\Rightarrow$ bottom of found. [illegible] soil को shear acc

$F_{\gamma i} \Rightarrow$ inclination factor $\Rightarrow$ load inclined [illegible]

some correction



$\alpha \neq \phi'$ $\quad \alpha = 45° + \phi'/2$

if so,

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

$$N_q = \tan^2\left(45° + \frac{\phi'}{2}\right) e^{\pi \tan\phi'}$$

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

$$N_\gamma = 2(N_q + 1)\tan\phi'$$

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

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

$N_\gamma = 2(N_q + 1)\tan\phi'$

$\phi' = 0 \quad N_q = 1 \quad N_\gamma = 0$ //

$\phi' = 0 \quad N_q = 1, \; N_\gamma = 0$

For cohesive soil :

$$q_u = c' N_c + q = c' N_c + \gamma D_f$$

$N_c = 5.7$

for cohesionless soil : $c' = 0$,

$$q_u = q N_q + 0.5 \gamma B N_\gamma$$

* ~~Settlement~~ for local shear failure

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

* ⇒ Because of progressive settlement in clays better strain adjustment occurs (settl. S.)

* Settlement (Type, nature and effects):

Type — settlm. predicted upto 25-30% accuracy

$S_e + S_p + S_s$

$S_e = \frac{\Delta\sigma (1-\mu^2) \times B'}{E_s} I_s I_f$

$S_p = \frac{CH}{1+e_0} \log\left(\frac{\sigma_0 + \Delta\sigma}{\sigma_0}\right)$

$S_s = \frac{C_\alpha H}{1+e_p} \log\left(\frac{t_2}{t_1}\right)$

C_α = coff. of secondary consolidation

Nature:

(i) Uniform settlement:
→ not so harmful for structure.
→ limit ⇒ 20mm – 300mm
But when > 150mm affects utilities water pipes, sewage lines,

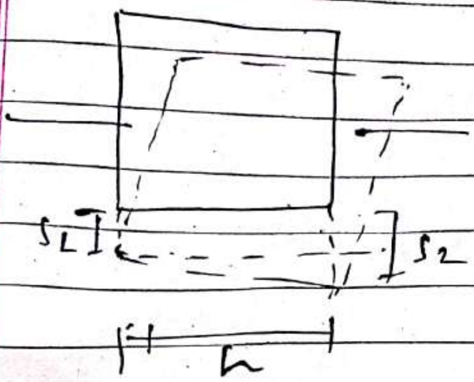
IS: Isolated footing ⇒ 40mm (max) ⇒ sand
Raft/mat ⇒ 40-65mm (sand) 65mm (max) ⇒ clays
65-100mm (clay) ? higher in clay?

(ii) Differential settlement:
→ different foundation settle by different amount

→ more difficult to predict.
→ ds < 50% of max^m settlement.

So if maxm. settlement is controlled. ds automatically controlled.

Tilt



Tilt / angular distortion (β) = $\frac{S_2 - S_1}{L}$

RCC ⇒ $\beta \le \frac{1}{150}$ ⇒ without damage

but for architectural safety

$\beta \le \frac{1}{300}$

⇓

20 mm in ~~corresponding~~ adjacent columns (6m btw c-c)

Generally ⇒ 25 mm to 40 mm differential settlement.

sandy / clayey.

Effects :

i) Appearance

⇒ crack in walls, paints

⇒ distortion of door windows

ii) Utility

⇒ ~~water displacement~~ of water pipes sewer lines

⇒ dealignment of machine connections.

2.2.04 Types of foundation and their suitability in Nepal (condition to use spread or combined footing ; mat : types, bearing capacity , construction approach , floating mat , compensating mat ; piles : types , load carrying capacity , negative skin friction (NSF) and calculation , comparison between pile, pier and caisson ; caisson : types, bearing capacity , construction of well , tilt and shift of well and its rectification and prevention)

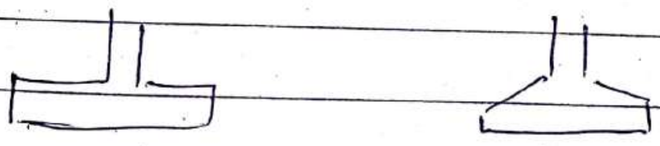
#. when do we prefer a mat foundation ?
#. Pile group efficiency and how is the bearing capacity of a pile group determined ?
#. Differentiate betn pile and well foundation.
mat design using conventional method .
#. well sinking operation . How to avoid tilt and shifts . and measures to correct them .

* conditions to use combined or spread footing:

a) Spread / Isolated footing :

→ Loads are small

→ Bearing capacity is fair good / sufficient



Uniform thickness tapered thickness

b) combined footing : two or more columns when provided under two are close to each other

→ two columns are close to each other such that their separate footings overlap.

→ bearing capacity is low enough to cause overlapping.

→ when a building is too close to a property line, building or sewer line.

* Mat foundation : when more than one line of column is supported by concrete slab.

condition of using mat:

→ Load to in the columns are too high.

→ Allowable soil pressure is very small such that → individual footings overlap

→ a lot or cover more than 50% of the built area.

→ they are also used to reduce differential settlement of structures on highly compressible soils.

Types of mat foundation:
(मेरो Book) Pg 199

(1) Flat plate type:

Bearing capacity:

(i) On cohesionless soil:

Bearing capacity in c=0 soil depends on width of foundation. For raft B is large so generally bearing capacity governed by settlement in c' c=0 soil.

$$q_n = \frac{N_{60}}{0.08}\left[\underbrace{1+0.35\frac{D_f}{B}}_{F_d}\right]\left[\frac{S_e\,(mm)}{25}\right]$$

$$\leq 16.6\,B\,\frac{S_e\,(mm)}{25} \quad \text{—— } (*)$$

(modified Meyerhof's equation).

यदि shallow foundation मा total Δ = 25mm and d_s = 19mm Design गरिन्छ
mat मा smaller d_s हुने हुँदा Δ = 50mm
मा d_s = 19mm गरिन्छ Design मा

So that (*) becomes

$$\boxed{q_{nf} \approx 25\,N_{60}}$$

$$\cancel{q_{ns} = q_{nu} - \gamma D_f}$$
$$\cancel{= \frac{Q}{A} - \gamma D_f}$$

For clay.
q_u doesn't depend on width
$$q_{nu} = c N_c$$
$$N_c = 5c_u\left(1+0.2\frac{B}{L}\right)\left(1+0.2\right.$$
$$q_{nu} = 5c_u\left(1+0.2\frac{B}{L}\right)\left(1+0.2\frac{D_f}{B}\right)$$

raft मा Backfill गरिदैन

net pressure applied on foundation

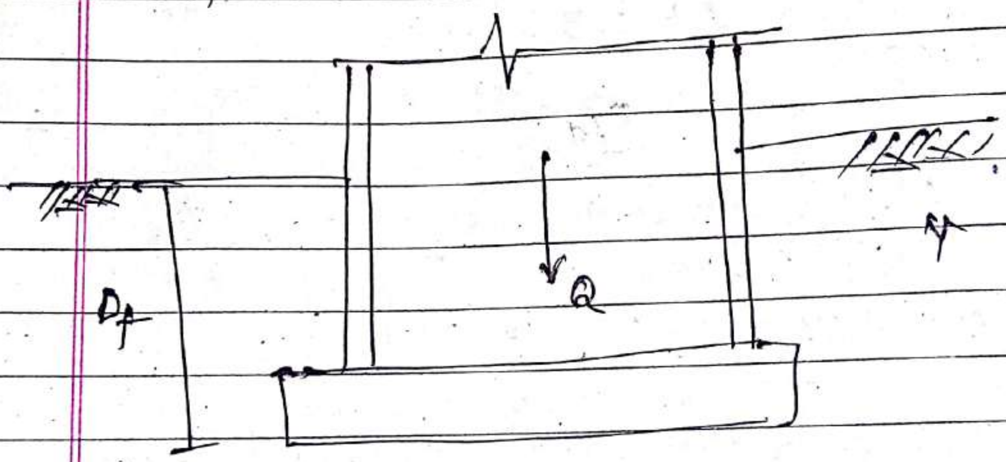
$$q = \frac{Q}{A} - \gamma D_f$$

पहिला normal condⁿ मा soil को भार load हुन्थ्यो (γD_f) हटछ। कति घटे ?

$$S_e(mm) \approx \frac{2\, q_{net(all)}}{N_{60}}$$

raft मा कम settlement.

* compensated foundation.



raft मा Backfill हुँदैन।

Net pressure on soil caused by raft foundation

$$q_{net} = \frac{Q}{A} - \gamma D_f$$

raft * As D_f ↑ q_{net} ↓ :

Y net pressure under foundation in soil can be decreased by increasing the depth. ⇒ compensated foundation.

For no net increase in pressure

$$q_{net} = 0$$

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

$\Rightarrow$ Fully compensated / floating mat.

i.e. structure को ~~weight~~ load बराबर माटो को weight झिकेर फालियो । त्यो depth मा पहिला को soil pressure जति नै load छ ।

For partially compensated mat

$$FOS = \frac{q_{nf(u)}}{q_{net}}$$

$$= \frac{q_{nf(u)}}{\frac{Q}{A} - \gamma D_f}$$

For clay

$$FOS = \frac{5c_u\left(1+0.2\frac{B}{L}\right)\left(1+0.2\frac{D_f}{B}\right)}{\frac{Q}{A} - \gamma D_f}$$

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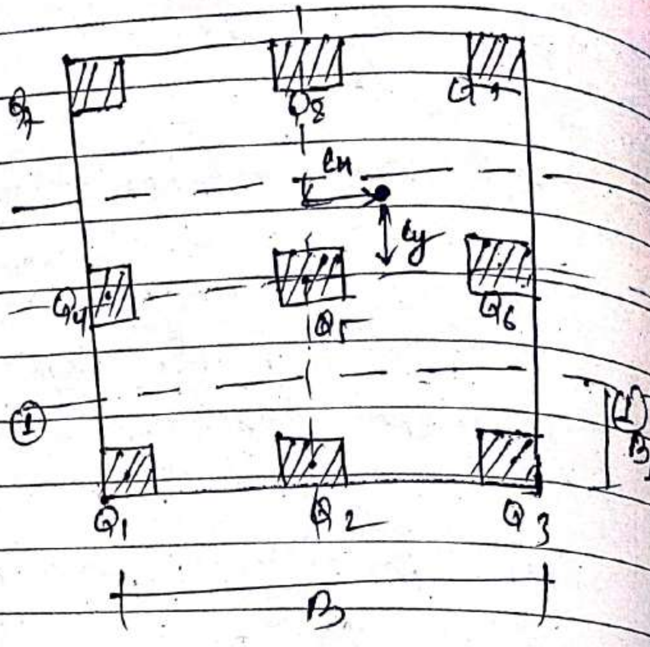
* Conventional Method of Mat Foundation Design (Rigid)

(I). Compute the line of action of the resultant



(II). Compute the base pressure distribution in the mat

a). If no eccentricity

$$q = \frac{Q}{A} \quad \text{—(I)}$$

b) If eccentricity present

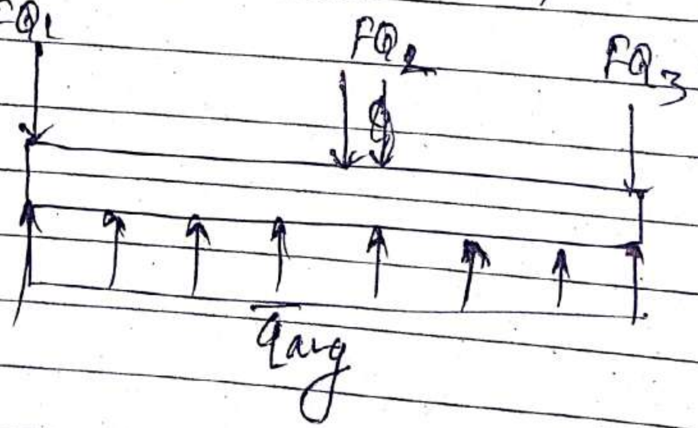
$$q = \frac{Q}{A} \pm \frac{Q e_x}{I_{yy}} x \pm \frac{Q e_y}{I_{xx}} y \quad \text{—(II)}$$

The maximum pressure from (II) should be less than the allowable bearing pressure.

$q_{max} < q_{all}$

III). Divide the mat into a number of strips in x and considering it to be rigid (no transfer of shear between adjacent strips.

IV).



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(iv). The strip is not in equilibrium considering the column forces and avg. base pressure. To correct it let us consider strip (1)-(2) of width B_1

$$Q_{avg} = \frac{1}{2}\left(Q_1 + Q_2 + Q_3 + q_{avg} B_1 B\right)$$

$$\bar{q}_{avg} = \frac{Q_{avg}}{B_1 B}$$ => corrected avg. base pressure.

correction factor for column load,

$$F = \frac{Q_{avg}}{Q_1 + Q_2 + Q_3}$$

Multiply each load by F as shown in figure

(v). Calculate SF and BM considering the strip as a beam and calculate reinforcement, and depth required.

Note: as the analysis is approximate provide twice the amount of reinforcement than computed.

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* Pile (Types, load carrying capacity, NSF and it's calculation, comparison Betn pile, pier and caisson)

Pile ⇒ slender structural member made of steel, concrete or wood.

Foundation made using piles ⇒ pile foundation.

→ deep foundation $D_f > B$

→ costly

* Conditions that require pile (VVIC)

1. when one or more upper soil layers are too weak and highly compressible piles used to transmit load to the underlying stronger layer.

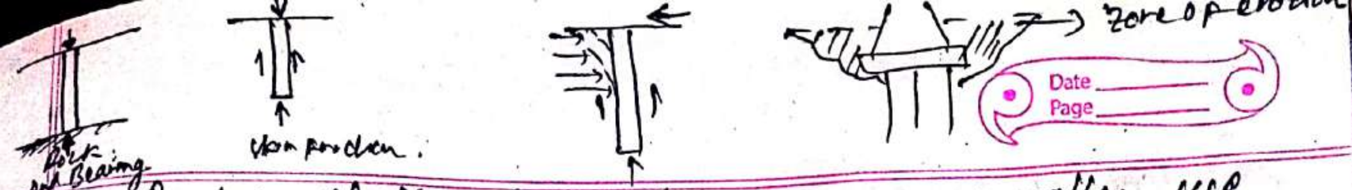
most load is transferred from the frictional resistance developed at soil-pile interface.

1). when structure is subjected to significant horizontal loads. Eg: retaining ~~walls~~ structures, tall buildings.

when the soil is active piles may be extended beyond this depth to transfer the load safely. Eg: in loess.

shrink and expand easily

) to resist uplifting force when GWT is high.



(v). Bridge abutments and piers are usually use piles to avoid loss of bearing capacity that a shallow foundation might at deeper because of ~~a~~ scouring.

pile ~~load transfer~~ mechanism not fully understood yet → more of an art due to uncertainties involved.

Types of pile :

(1) Steel piles .

→ pipe-piles or rolled steel H-section more preferred .

$Q_{all} = A_s f_s$

A_s ⇒ Area of cross-section of steel

$f_s = 0.33 - 0.5 f_y$

Usual length ⇒ 15 m to 60 m

Load ⇒ 300 kN to 1200 kN .

Advantage	Disadvantage .
→ Easy to handle w.r.t. cutoff, extension	→ costly
→ high strength	→ High level of noise during driving
→ can penetrate hard layers .	- corrosion.
- withstand hard driving force	

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(ii) Concrete piles:

Two basic categories

a) Pre-cast piles

usual length: 10m to 15m

Load: 300kN to 3000kN

Advantage

→ withstand hard driving force

→ corrosion resistant.

→ can be easily combined with concrete superstructure.

Disadvantage.

→ difficult to handle achieve proper cut-off.

→ difficult to transport.

b) Cast in-situ piles

usual length: 5m - 15m

max (30-40m)

Load (max): 200-500 kN

Advantage

→ relatively cheap

→ allows for inspection

→ easy to extend.

Disadvantage

→ casing may be damaged

→ difficult to splice.

$$Q_{all} = A_s f_s + A_c f_c$$

steel concrete.

(iii) Timber Piles:

→ 10m - 20m

→ straight, sound timber without any defect.

Disadvantage:

(i). Pest attack

(ii). moisture

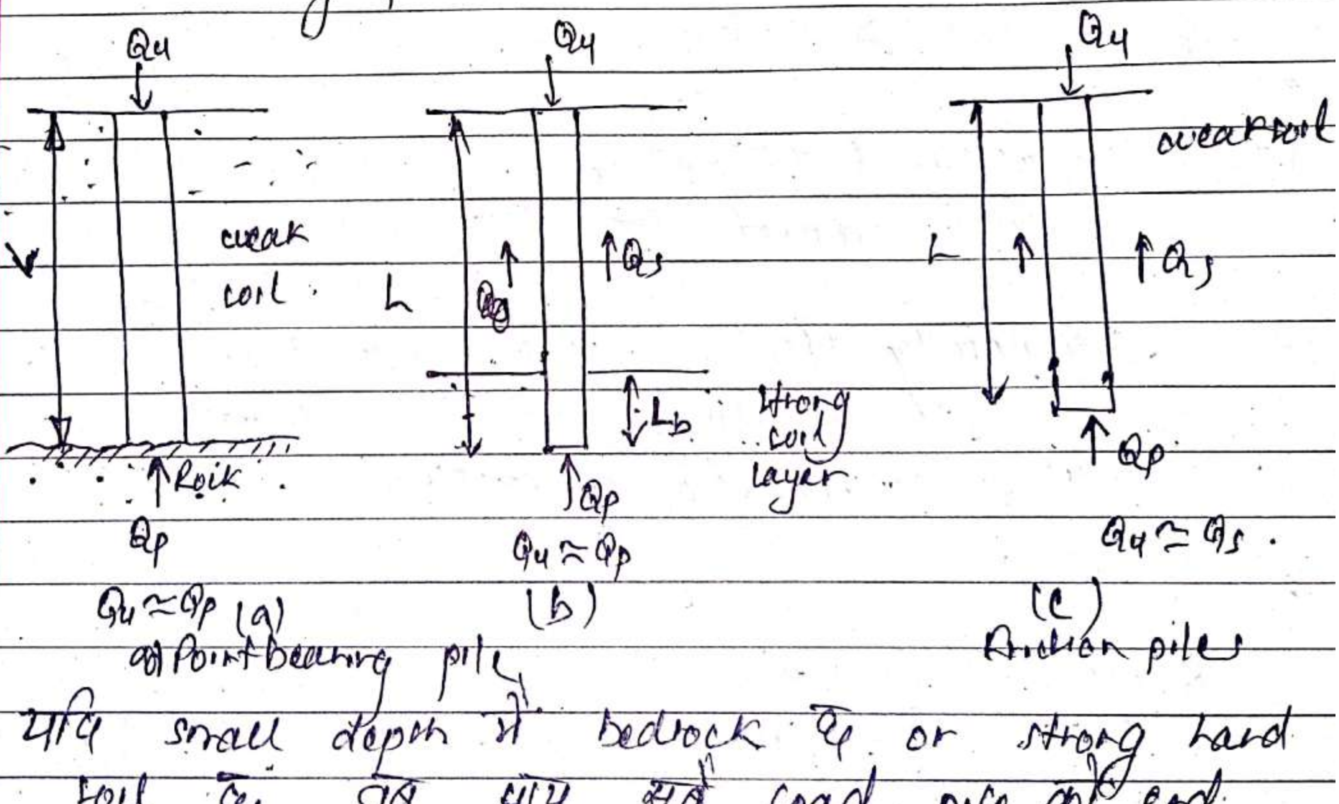
(iii). brooming while driving (use metal caps to avoid).

(iv) Composite :
concrete - timber → below WT
above बाकि concrete.

(v) Continuous Flight Auger (CFA) piles :

On the basis of load transfer mechanism and length:
(i). Point bearing piles (ii) Friction piles
(iii) compaction piles.

(I) Point Bearing piles :



→ यदि small depth में bedrock है or strong hard soil है, तब प्राय सारे load pile की end द्वारा Bear करके pass करते हैं / underlying material को

(ii) Friction piles :
जब no rock or hard stratum at reasonable depth point bearing piles become very long and uneconomical.

So in such case soft layer मा ही certain depth सम्म राख्ने load transferred by friction resistance (skin friction).

clay मा adhesion को लि हुन्छ ।

कति length सम्म राख्ने ?
→ depends on soil shear strength, load applied and pile size.

(iii) Compaction piles :
→ soil compact गर्न राखिन्छ ।

→ generally short length depend on
⇒ relative density desired
कति ⇒ depth सम्म गर्ने compaction
⇒

* Pile Installation
(i) ~~vibratory~~ / Hammer (ii) Jetting (iii) Partial Augering
→ Drop hammer
→ Single - acting hammer air or steam
→ double acting differential air or steam hammer.
→ diesel
(iv) Vibratory

Load carrying capacity :
$$Q_u = Q_p + Q_s.$$

$B = D$ for pile

$$q_u = c'N_c^* + q'N_q^* + \gamma D N_\gamma^*$$

⇒ shallow से modify गर्दा by N_c^* N_q^*, N_γ^* shallow भन्दा अलि फरक हुन्छ ।

$D \approx 0$ So,
$$q_p = c'N_c^* + q'N_q^*$$

$$Q_p = A_p q_p = A_p(c'N_c^* + q'N_q^*)$$

A_p → pile tip area.

c' = cohesion of soil supporting pile tip
q_p = unit point resistance
q' = effective vertical stress at pile tip level

$$Q_s = \Sigma p \Delta L f$$

ΔL → length over which p and f are constant
f → unit friction at depth.
p → perimeter of pile section.

Note: for full mobilization of Q_p pile tip should displace by 10 - 25% of its width (D).

$$Q_{all} = \frac{Q_u}{FOS} \quad (2.5 \text{ to } 4$$

* Meyerhoff's method to determine Q_p

a) Sand

For sand $c' = 0$

$\therefore \; q_p = q' N_q^*$

$q_p \uparrow$ with depth but only upto a certain limit. $\left(\frac{L}{D}\right)_{critical}$ called critical embedment ration. So, यो critical q_p को value अर्थात q_l हो given as

$$q_l = 0.5 p_a N_q^* \tan\phi'$$

* (यो भन्दा बढि हुँदैन q_p)

So, $Q_p = A_p q_p \leq A_p q_l$

$= A_p q' N_q^* \leq A_p \, 0.5 \, p_a N_q^* \tan\phi'$

i.e. $\boxed{\begin{aligned} Q_p &= A_p q_p \\ &= A_p q_l \end{aligned}}$ for $q_p \leq q_l$ / for $q_p > q_l$ *

$$\left(\frac{L}{D}\right)_{critical} = 16 - 18$$

(b) Clay (saturated, undrained cond^n)

$\phi' = 0 \qquad q_{p_{net}} = c' N_c^*$

$Q_{p_{net}} = A_p q_p = A_p c' N_c^* \qquad (N_c^* \simeq 9)$.

$= 9 c_u A_p$

c_u = undrained cohesion of soil below tip of pile.

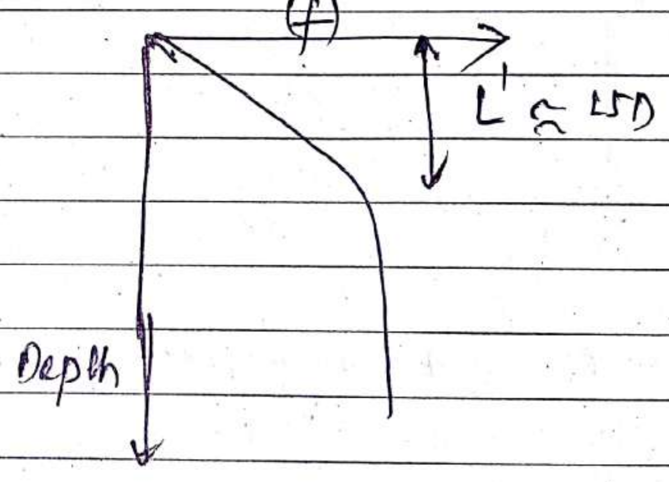
* Frictional Resistance (Q_s) :

a) Sand : $Q_s = A_s f = \Sigma\, p \Delta L f$

f => unit frictional resistance => hard to estimate
some point regarding f.

→ zone of sand densification (drive pile)
= 2.5 D

→ f ∝ linearly increase upto L' then constant.
$L' \simeq 15D$



→ 'f' of driven piles > bored or jetted piles for same depth.

$f \simeq K \sigma_0' \tan \delta'$ Take $K = 1 - \sin\phi'$
$z = 0$ to L'

depends on pile driving type.

K = earth pressure coff $\approx 1 - \sin\phi'$

σ_0' = effective vertical pressure at considered depth.

δ' = friction angle.

(b) Friction resistance in clay:

α-method

$$\boxed{f = \alpha c_u}$$

α = empirical adhesion factor.

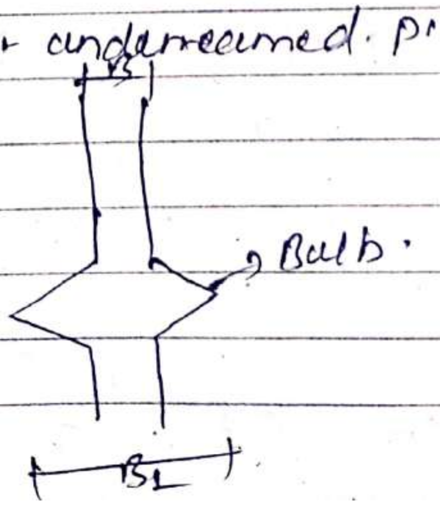
$$\alpha = C\left(\frac{\bar{\sigma}_0'}{c_u}\right)^{0.45}$$ no need to memorize.

$\bar{\sigma}_0$ Avg. vertical stress

$C \approx 0.5$

$$Q_s = f A_s = \cancel{c_u \alpha}\ c \alpha A_s$$

Note: For underreamed pile in clay.



$$Q_u = \frac{\pi}{4} B^2 (9c) + \frac{\pi}{4}(B_1^2 - B^2) \times 9c' + \alpha c' A_s$$ #.

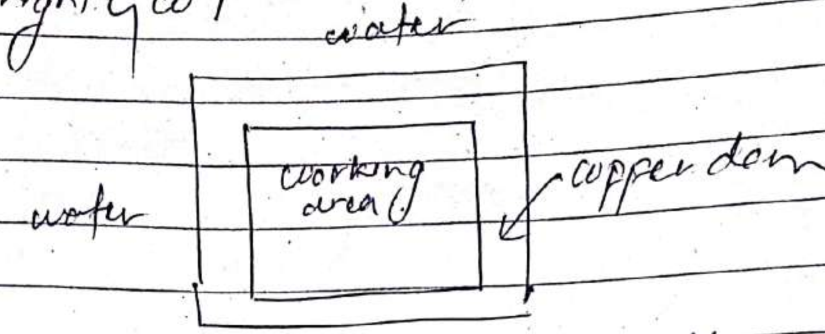
$q_{all} = q_u / FOS$ $FS = 2.5$ to 4

* Negative ~~screen~~ skin friction (NSF)

See भाग दूसरा of Pg 227.

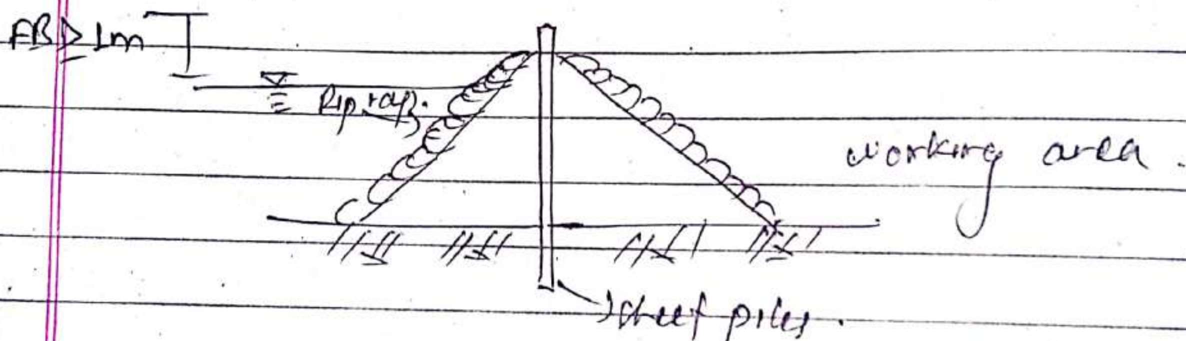
* copper dam.
→ foundation to be built in rivers / water bod
such as bridge piers, dams, etc.

→ lightly cut dewatering



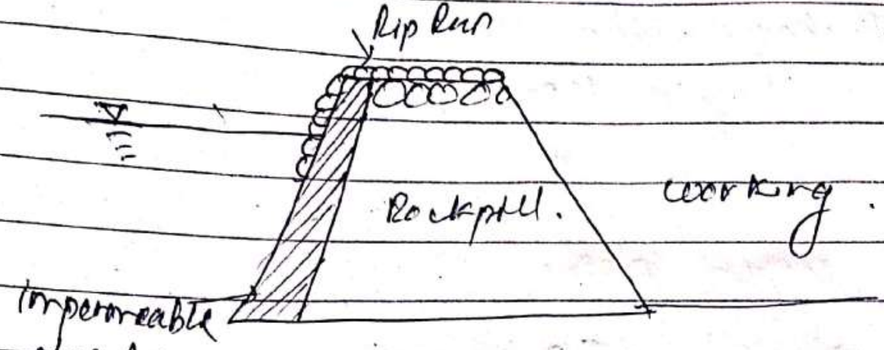
→ enclosed, impermeable wall ~~with~~ o

Types:
a) Earth copper dams:
~~→ simplest~~



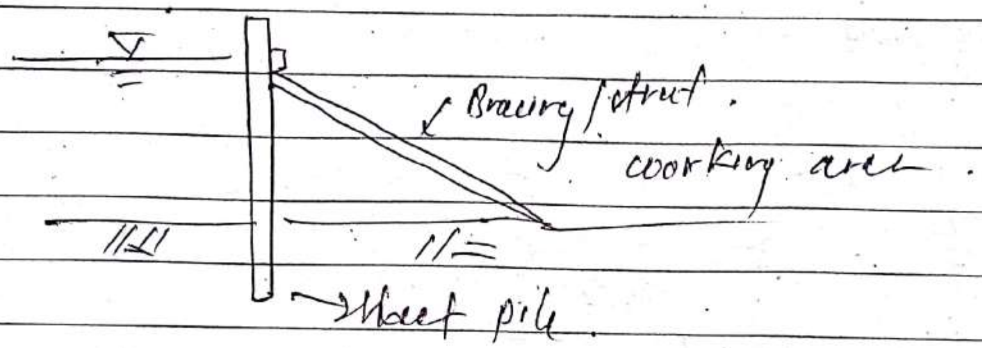
→ simplest type.
→ local materials.
→ sheet-pile or clay core
→ water seeping pumped out.

b) Rock-fill coffer dam



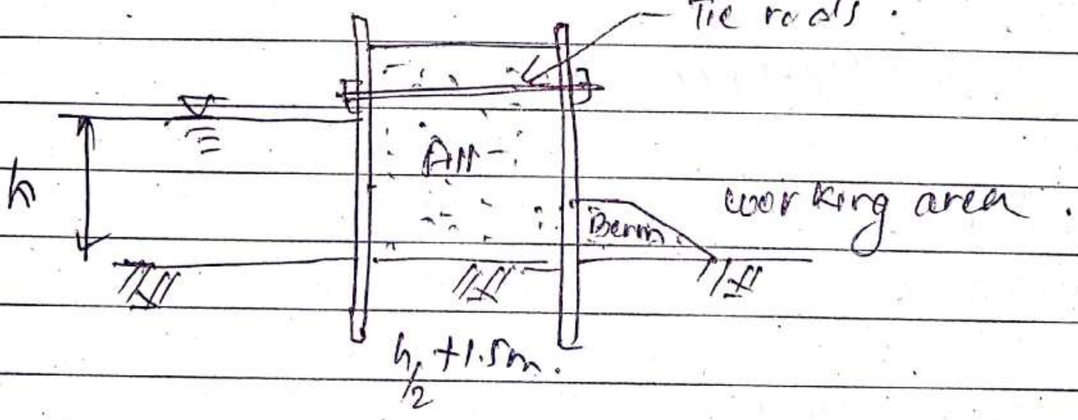
→

c). ~~Single pile~~ single-sheet pile coffer dam:



→ small excavation area.

d) Double sheet-pile coffer dam:

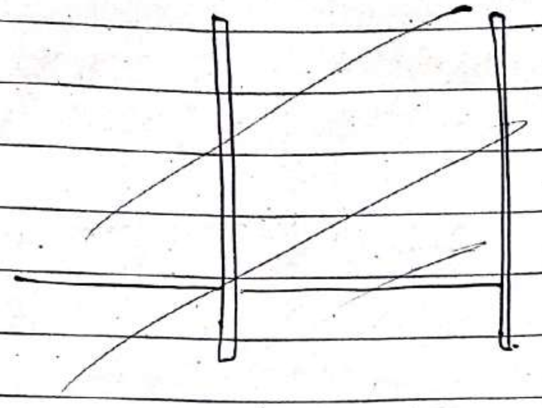


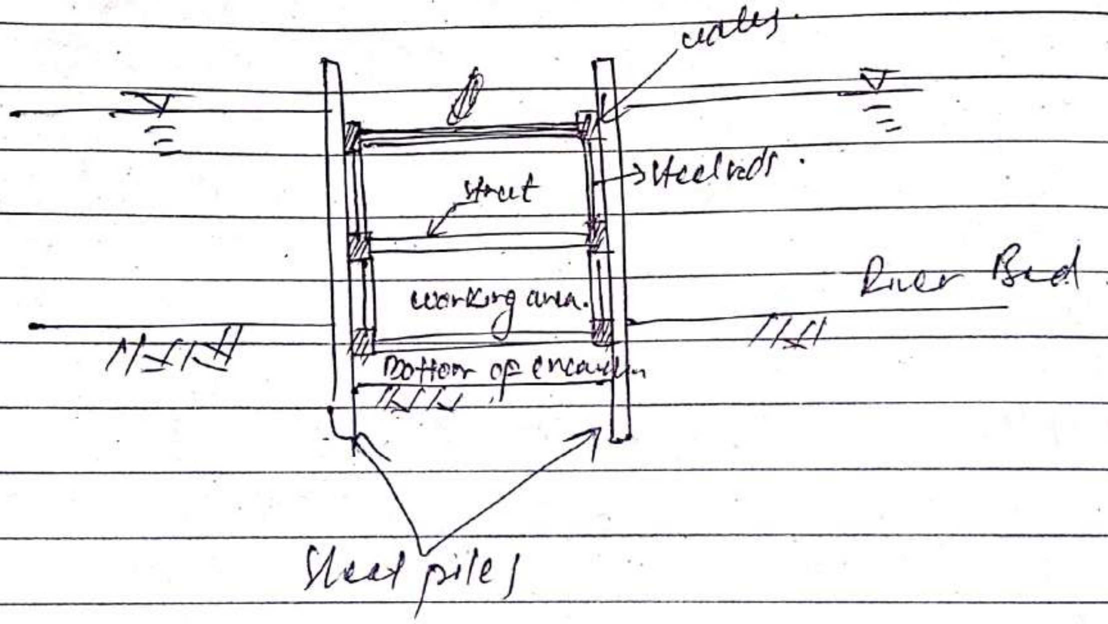
→ h > 2.5m ⇒ strutting should be done

⇒ Fill ⇒ stability.

→ Steel pile as deep as possible against uplift.
→ Less leakage than single.
→ 6 upto 20m

(v). Braced coffer dams





→ isolate area surrounded by water.
→ susceptible to flood damage.

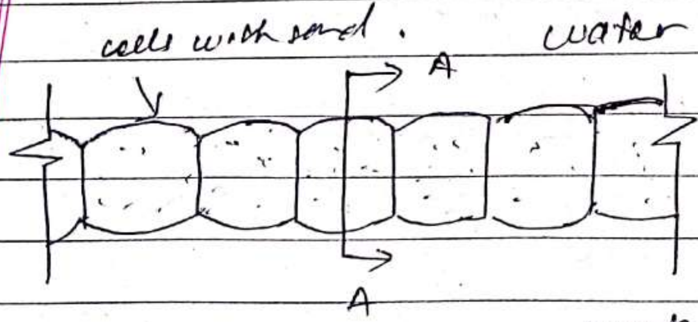
→ can be used in land as well where high GWT.

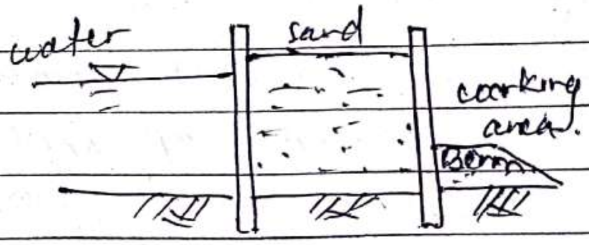
(v). ~~Diaphgram Type:~~
(vi) cellular type cofferdam:

→ series of cell formed using sheet piles of special shape.
→ cells filled with soil for stability.

2 types

a). Diaphgram type.

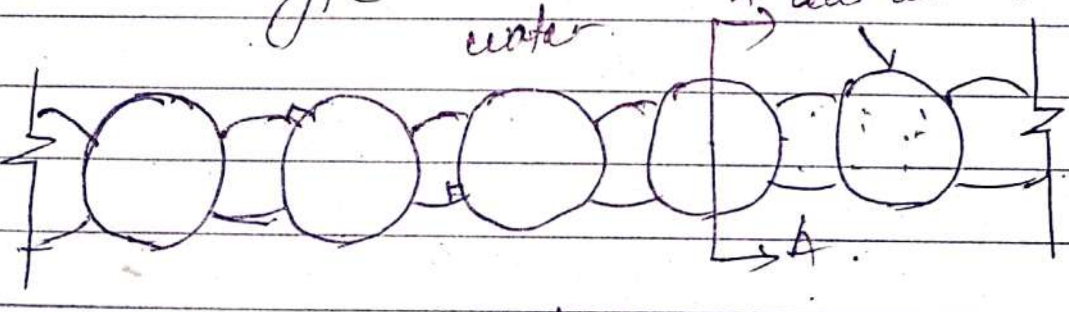




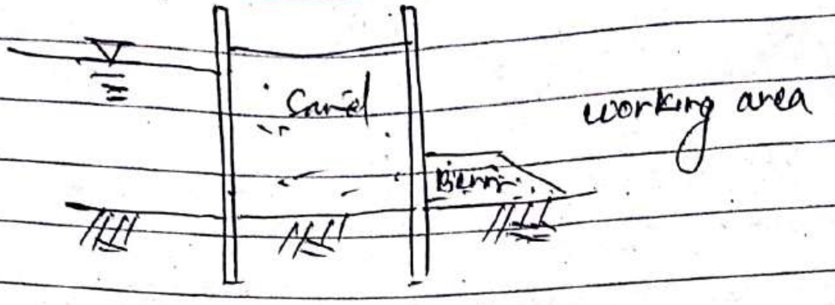
working area. A-A section.

→ circular are sheet pile at inner and outer edge connected by straight diaphgram walls.
→ soil (sand) filled to increase stability.
→ easy to extend
→ one cell dependent on other so should filled at same rate

c) circular type



working area.



Section A-A

⇒ Large of main circular cells connected by smaller arc cells.

⇒ arc ⊥ar to main cell walls.

⇒ self-sustaining → independent of adjacent circular cells. So, each cell can be filled independently.

⇒ stability > diaphragm type.

⇒ more expensive as more sheet piles.
greater skill to set up.