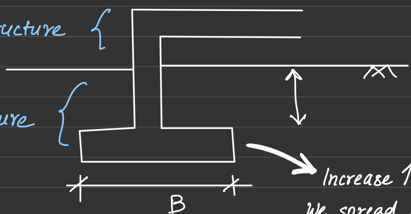
CE352A
FOUNDATION DESIGN
CHAPTER 1

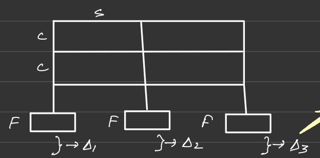
CE352A: Foundation Design

Live loads
Dead loads



→ Increase ↑ surface area,
We spread it over large area
Otherwise the soil will break.

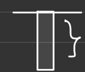
* In case of heavy loads, we use deep foundation.
* Why crack comes in buildings?
  ↳ A good geotechnical engineer won't be able to immediately tell.
  You've to think and analyse.
  ↳ Due to several reasons, cracks could be developed.



* In actual situation, soil is non-homogenous.
  ↳ Now, bending moment → buckling of columns.

* If you want to design foundation you should know,
  → Type of soil, its strength → These all tests you've done in CE351.

* Where to stop to collect soil sample?

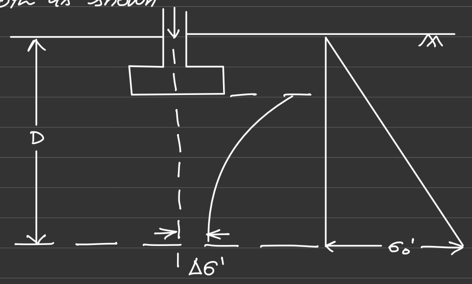


→ Depth of soil.

↳ Basic Introduction

Determination of minimum depth of boring [As per ASCE (1972)]

- Determine the net increase in the effective stress, $\Delta\sigma'$ under a foundation with depth as shown



- Estimate the variation of the vertical effective stress, σ_0' with depth.
- Determine the depth, $D = D_1$ at which the $\Delta\sigma' = \left(\frac{1}{10}\right) q$, q = estimated net stress on the foundation.
- Determine the depth, $D = D_2$ at which the $\frac{\Delta\sigma'}{\sigma_0'} = 0.05$.
- Approximate minimum depth of boring required = smaller (D_1, D_2), unless bedrock is encountered.

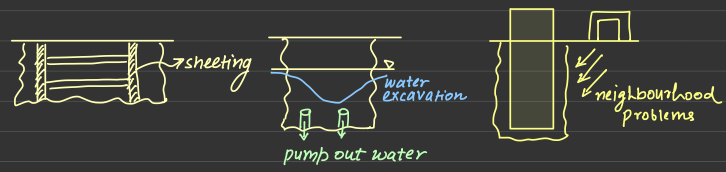
Soil exploration, sampling and in-situ testing

- Site exploration usually ranges from about 0.5 - 1.0 % of the total construction cost.

<u>Elements of site investigation</u>

- Information to determine the type of foundation required (shallow or deep).
- Information to allow the geotechnical consultant to make a recommendation on the allowable load capacity of the foundation.
- Sufficient data/lab tests to make settlement predictions.
- Location of ground water table.
- Information so that the identification and solution of construction problems (sheeting & dewatering or rock excavation) can be made.
- Identification of potential problems (settlements, existing damage etc.) concerning adjacent property.
- Identification of environmental problems and their solution.

In-situ testing → Testing directly in the field itself.



Methods of exploration

Auger Boring	Depths up to about 35m	All soils; Difficult in gravelly soil
Rotary drilling	Depths up to about 70m or more	All soils
Wash Boring	" " " " 70m or more	All soils
Percussion drilling	" " " " 70m or more	All soils
Test pits and open cuts	" usually less than 6 m	All soils

Soil sampling

(i) Two types of samples can be obtained during subsurface exploration :-
Disturbed sample
Undisturbed sample

* "Soil sensitivity" → some soils if you disturb, it will not get upset. But some other soil may get more upset with the same degree of perturbation.
Soil sensitivity can be quantified.
For very sensitive soil → Disturbed soil may not give accurate results.

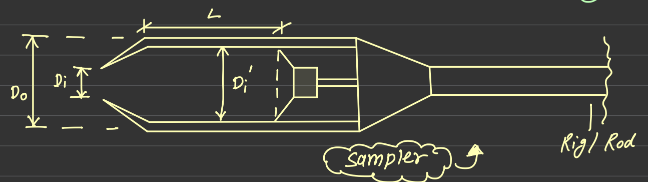
(ii) It is nearly impossible to obtain undisturbed samples of cohesionless (sandy soil) material for strength testing.

(iii) Soil disturbance depends on factors such as rate of penetration, whether the cutting force is required by pushing or driving, and presence of gravel.

(iv) Sample Disturbance also depends on the ratio of the volume of soil displaced to the volume of collected sample, expressed as an Area Ratio (A_r)

$$A_r = \frac{D_o^2 - D_i^2}{D_i^2} \times 100 \quad \text{--- (1)}$$

D_o = outside diameter of tube
D_i = inside diameter of cutting edge of box.



people also use liner inside the sampler so that the wall of the soil sampler do not get damaged from rocks, etc. {Walls are costly.}

(v) Well designed sample tubes should have an Area Ratio of less than 10%.

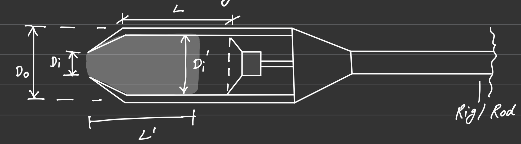
For sand, $L \geq 5-10\ D_o$
For clay, $L \geq 10-15\ D_o$
} → {In case of sand, we take lesser length, b'coz sand gets less disturbed vis-a-vis other.}

Common D_o : 51, 64, 76 & 89 mm. ⟶ Standard Values

(v) Another term used in estimating the degree of disturbance of a cohesive or rock core sample is the Recovery Ratio (L_r).

$$L_r = \frac{\text{Actual length of recovered sample}}{\text{Theoretical length of recovered sample}} \quad \text{--- (2)}$$

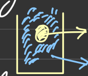
where recovered length of sample = the length of sampler was forced into the stratum.



(vi) $L_r = 1 \Rightarrow$ Theoretically, the sample did not become compressed from friction on the tube. ⟶ no compression in the soil

soil is undisturbed by friction. (undisturbed soil sample)

(vii) $L_r > 1 \Rightarrow$ A loosening of the sample from rearrangement of stones, roots, removal of preload, or other factors.



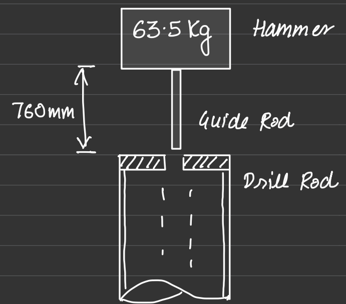
(viii) $L_r < 1 \Rightarrow$ Compression is happening ⟶ disturbing the soil.

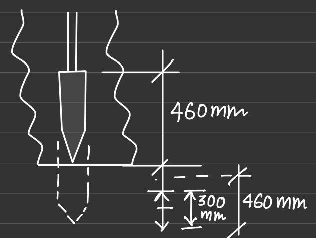
Standard Penetration Test (SPT) [As per ASTM D 1586]

(i) Currently the most popular and economical means to obtain sub-surface information.

The test consists of

(ii) Driving the standard split-barrel sampler a distance of 460 mm into the soil at the bottom of the boring.

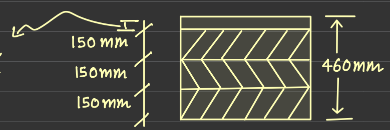




split-spoon sampler

(iii) Counting the number of blows to drive the sampler the last two 150 mm distances (total = 300 mm) to obtain the N number.

N_1 = ~ } $N = N_2 + N_3$
N_2 = ~
N_3 = ~



(iv) Using a 63.5 kg driving mass (or hammer) falling free from a height of 760 mm.

(v) The exposed drill rod is referenced with three chalk marks 150 mm apart, and the guide rod is marked at 760 mm (for manual hammers).

(vi) The assemblage is then seated on the soil in the borehole.

(vii) Next the sampler is driven a distance of 150 mm to seat it on undisturbed soil, with this blow count being recorded.

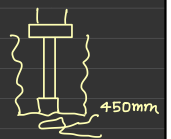
(viii) The sum of the blow counts for the next two 150 mm increments is used as the penetration count N unless the last increment cannot be completed.

(ix) In this case, sum of the first two 150 mm penetrations is recorded as N.

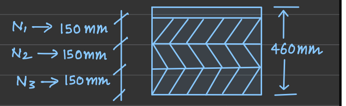
(x) The boring log shows refusal and the test is halted if

- 50 blows are required for any 150 mm increments.
- 100 blows are obtained to drive the required 300 mm.
- 10 successive blows produce no advance.

→ What could be the reason? White color hops in the soil due to cementation of soil and several other reasons as well.



* If all N_1, N_2, N_3 completed $\Rightarrow N = N_2 + N_3$
* If unable to complete $N_3 \Rightarrow N = N_1 + N_2$ (that's why we recorded N_1)



Input Driving Energy :-

(i) Blow count would be directly related to driving energy.

$$E_{in} = \frac{1}{2} m v^2 = \frac{1}{2} \frac{W}{g} v^2 \quad \text{---- (3a)}$$

and $v = \sqrt{2gh}$ ---- (3b)

(ii) Putting v value in 3a.

$$E_{in} = \frac{1}{2} \frac{W}{g} (2gh) = Wh \quad \text{--- (3c)}$$

where W = Weight of the hammer, h = height of fall

(iii) Kovacs & Salomone (1982) reported actual input driving energy E_a to the sampler ranged from 30–80%.

* More Energy Applied ⇒ Less Resistance

(liner) → some protecting surface made of some composite material. It is a kind of protective layer to protect the sampler walls from pebbles and other obstruction that may affect the walls.

(iv) N value is increased if we use a liner inside the sampler.

(Why?) → If we add liner, the resistance of soil increases and hence the no. of blows required will also increase. (N value ↑) Wall of liner has lesser resistance vis-a-vis liner.

(v) N value should be larger for soils with OCR > 1 and larger relative density.

(vi) N values are smaller if effective overburden pressure p_0' is smaller (near the ground surface) for soils of same density.

(vii) N values increases (↑) with increase in degree of cementation.

Why? → (When the cement hardens, it will require more no. of blows.)

Standardization of SPT

(i) SPT should be standardized to some energy ratio E_r,
E_r = SPT hammer energy efficiency

$$E_r (\%) = \frac{\text{Actual hammer energy to sampler, } E_a}{\text{Input energy, } E_{in}} \times 100 = \frac{E_a}{E_{in}} \times 100 \quad \text{-- (4)}$$

↳ To count to a common platform.

Theoretical Input energy $= Wh$

where W = weight of hammer ≈ 0.623 kN, h = height of drop ≈ 0.76 mm } so, Wh = 0.474 kN-m

(ii) Since there is a wide scatter in E_r and the resulting blow count N when it is reasonable to expect there should be a unique N for the soil at some depth, it is suggested the drill system dependent E_r be referred to a standard energy ratio value E_{rb}.

(iii) In this way, a drill rig with, say $E_r = 45$ would, on adjustment to the standard E_{rb}, compute approximately the same N count from a drill rig with $E_r = 70$.

(iv) The standard blow count N_{70}' is given by

$$N_{70}' = C_N \times N \times \eta_1 \times \eta_2 \times \eta_3 \times \eta_4 \quad \text{--- --- --- (4)}$$

where, η_i = adjustment factors such as η_1 is for hammer
η_2 is rod length correction
η_3 is sampler correction
η_4 is borehole diameter correction

$$\eta_1 = \frac{E_r}{E_{rb}} = \frac{E_r}{70}$$

other factors ↳ available in textbooks

N_{70}' = adjusted N using the subscript for the $E_{rb} = 70$.
C_N = adjustment for the effective overburden pressure p_o' (kPa)
[Liao & Whitman Relationship, 1986]

95.76 kN/m² = atmospheric pressure

$$C_N = \left(\frac{95.76}{p_o'}\right)^{1/2}$$

→ Corelations will be provided in exams. No need to remember it. Just rem. the basic corelations only.

(v) Note the larger values of E_r, decrease the N value nearly linearly.

$$E_{r1} \times N_1 = E_{r2} \times N_2 \quad \text{--- --- --- (6)}$$

SPT Corelations

The SPT is used in corelations for
- Unit Weight, γ
- Relative Density, D_r
- Angle of internal friction, ϕ
- Unconfined compressive strength, q_u
- Bearing capacity of foundations, q_{ult}
- Stress - Strain Modulus, E_s

ϕ vs N

$$\phi = 4.5 N_{70}' + 20 \quad \text{--- --- --- (7)}$$

D_r vs N

$$\frac{N_{70}'}{D_r} = 32 + 0.288 p_o' \quad \text{--- --- --- (8)}$$

where p_o' is in kPa, D_r is in %
Also, $\phi = 28 + 0.15 D_r$, D_r is in % [Meyerhof, 1959]

q_u vs N

$$q_u = k N_{70}' \quad \text{--- --- --- (9)}$$

where, k tends to be site dependent and generally equals to 12.

SPT Corelations

(i) With the current practice of recovering samples and routinely inspecting them, performing on-site q_u tests with a pocket penetrometer or using an UCS test device, it is not neccesary to use strength corelation.

Design N value

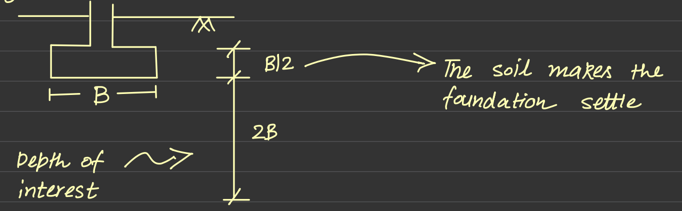
(i) Early recommendations were to use the smallest N value in the boring or an average of all of the values for the particular stratum.

(ii) Current Practice is to use an average N but in the zone of major stressing.

(iii) For example, for a spread footing the zone of interest is from about one-half the footing width B above the estimated base location to a depth of about 2B below.

$$N_{av} = \frac{\Sigma N z_i}{\Sigma z_i} \quad \text{(weighted average)} \quad \text{--- --- --- (10)}$$

* As per IS : 2131 – 1981 (SPT)
— Driven into the soil for a distance of 450mm.
— A drop of hammer of 65kg falling vertically and freely from a height of 750mm.

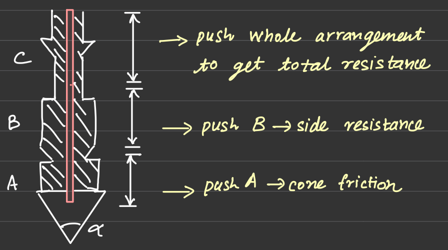


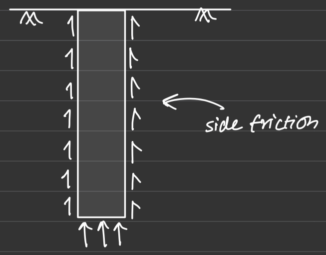
* Minimum N value ⟹ Soil is weakest.

Cone Penetration Test (CPT) [As per ASTM D 3441]

- It is used particularly for soft clays, soft silts and in fine to medium sand deposits.
- It is not well accepted to gravel deposits or to stiff/hard cohesive deposits.

Procedure





Resistance from bottom

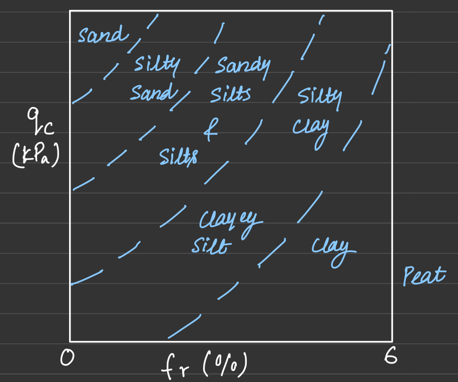
{ $\alpha = 30$ for $30°$ cone; $\alpha = 60$ for $60°$ cone }

- Pore pressure, vertical alignment and temperature can be recorded if allowed by the equipment configuration.
- The cone system is stationary initi .
- The cone is advanced by pushing an inner rod to extrude the cone tip and a short length of cone shaft. This action measures the tip resistance q_c.
- The other shaft is now advanced to the cone base, and skin resistance is measured as the force neccesary to advance the shaft q_s.
- Now, the cone and sleeve are advanced in combination to obtain a $q_{total} \approx q_c + q_s$

Friction Ratio (fr)

$$f_r = \frac{q_s}{q_c} \times 100\ \%$$

- Friction Ratio (fr) is used for soil classification.



* Peat → low q_c → high fr
* Sand → high q_c → low fr

Soil sensitivity (S_t)

$$S_t \approx \frac{10}{f_r} \quad \text{(in \%)}$$

$$S_t = \frac{q_u\ \text{(undisturbed)}}{q_u\ \text{(remoulded)}} \quad \text{→ From UU test.}$$

- $S_t = 1 \Rightarrow$ insensitive soil
- $S_t > 1 \Rightarrow$ as $S_t \uparrow$ sensitivity $\uparrow$
- $S_t < 1 \Rightarrow$ cannot happen normally bcoz $q_{remoulded} < q_{undisturbed}$ usually.

SPT Correlations

- q_c vs undrained shear strength (S_u)

$$q_c = N_k S_u + P_0$$

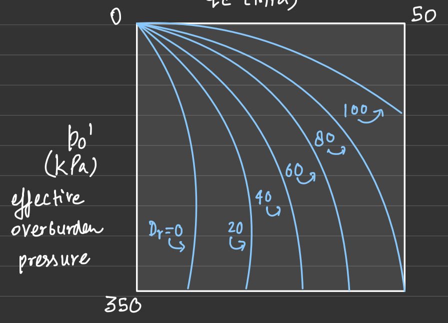
where $P_0 = \gamma z$ = overburden pressure where q_c is measured.
N_k = cone factor → range from 5–75 based on I_p (plasticity index)

- q_c vs unconfined compression strength (q_u)

$$q_c = 525.1 + 1.076 q_u$$

q_c & q_u are in kPa.

- q_c vs relative density (D_r) → The curves are for normally consolidated cohesionless material.



q_c vs D_r

- $D_r = 0$ ⇒ soil is in loosen state → You'll get minimum resistance
- As D_r ↑es, your resistance ↑es.
- overburden pressure has a crucial role in foundation.

- q_c vs SPT N value

$$q_c = K N'_{60}$$

(q_c in MPa)

K ranges from 0.1 to 1.0,

Dynamic Cone Penetration Test (DCPT) [As per IS:4968-1976]

★ No direct correlation b/w DCPT and soil properties.
★ One DCPT to SPT value correlation is there.
★ Everything here is mapped to SPT.

- A cone is driven into the ground in the same way as SPT, but unlike in SPT, there is no probing involved.
- IS Code recommends 50 mm and 60 mm diameter cones with apex angle of 60°.
- However, 65 mm cone is preferable as it yields more consistent relationship with SPT values. [Mohan et. al 1970]
- When depth of investigation > 6 m, bentonite or mud slurry is recommended as otherwise friction on the rods would be tremendous.
  (bentonite → a kind of sticky and greasy material.)
- Curve is plotted for No. of blows (N_c) per 300 mm of penetration vs. depth.
- N_c need to be corrected for overburden pressure in cohesionless soil like N values of SPT.

DCPT Correlations

- N_c vs SPT N-value

→ $$N_c = \frac{N}{C}$$

N_c — blow count value from DCPT
N — " " " " SPT
for corresponding depth in the same soil.

$C = 0.8 - 1.2$ when bentonite is used with 65 mm cone.

→ when Bentonite is not used then

$$N_c = 1.5N$$ → For depths up to 3m.

And $$N_c = 1.75N$$ → For depths b/w 3m & 6m.

Limitations

- No samples or only wash samples are obtained.
- Presence of gravels / boulders within the soil strata can give misleading results.

Plate Load Test (PLT) [As per ASTM D 1194]

★ V.V.Imp test in foundation engineering.

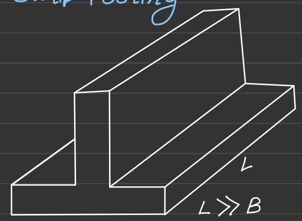
CE352A
FOUNDATION DESIGN
CHAPTER 2

Shallow Foundations

- Transmits structural loads to the soil strata at a relatively small depth. $\frac{D_f}{B} \leq 1$
- Moderately deep: $1 \leq \frac{D_f}{B} < 15$
- Deep: $\frac{D_f}{B} \geq 15$
- Footing: A footing is a portion of the foundation of a structure that transmits loads directly to the soil.
- Foundation: A foundation is that part of the structure which is in direct contact with & transmits loads to the ground.
- Strip Footing: $L \gg B$

Types

- Strip Footing

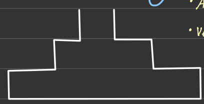


Strip Footing

Eventually a 2D problem

- Spread Footing

- Step Footing



- A kind of spread footing.
- Very uncommon these days
- used in old houses, etc.

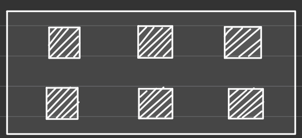
- Strap Footing

→ Tremendous moment

Differential settlement

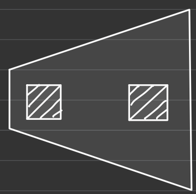
- Raft or Mat Foundation

1.2 to 2m



Bearing Capacity Low
↓
More Columns you need
If it settles then whole raft will settle down.

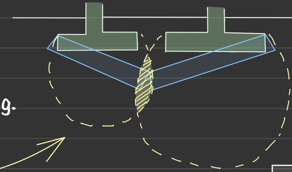
- Combined Footing



- If this column is carrying more load then this area would be more
- Not always trapezoidal

General Requirements of Foundation

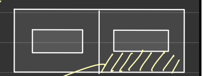
Presence of adjoining structure may lead to tilting of structure.



It lead to tilting of foundation and consequently tilting of structure

- **Location and depth criteria**

  Performance is not affected by
  - Lateral expulsion of soil beneath the foundation.
  - Seasonal volume changes caused by freezing and thawing.
  - Presence of adjoining structure.



Earlier by laws – you need to keep some space extra.

- **Bearing capacity criterion**
  - A foundation must be safe against shear strength failure or soil rupture.
- **Settlement criterion**
  - The settlement of a foundation, especially the differential settlement, must be within the permissible limit.

All three requirements must be satisfied separately.

Terminology

- **Gross Pressure (q_g)**
  Weight of superstructure + self weight of footing + weight of soil fill over the footing.

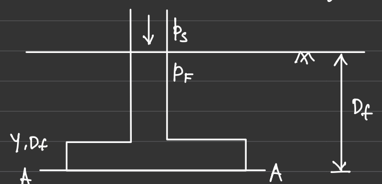
- **Net Pressure (q_n)**
  Difference b/w gross pressure and the overburden pressure $= q_g - \gamma_w D$.

- **Bearing capacity**
  The supporting power of a soil /rock is referred to as its bearing capacity.

- **Ultimate Bearing Capacity (q_u)**
  The maximum gross intensity of loading that the soil can support before it fails in shear.

- **Net Ultimate Bearing Capacity (q_{nu})**
  $q_{nu} = q_u - \gamma D_f$

- **Net safe bearing capacity (q_{ns})**
  $q_{ns} = \frac{q_{nu}}{F}$ — F → Factor of safety



γ, Df — can be predicted with good degre of confidence

- **Gross safe Bearing Capacity (q_s)**
  $q_s = q_{ns} + \gamma D_f = \frac{q_{nu}}{F} + \gamma D_f$

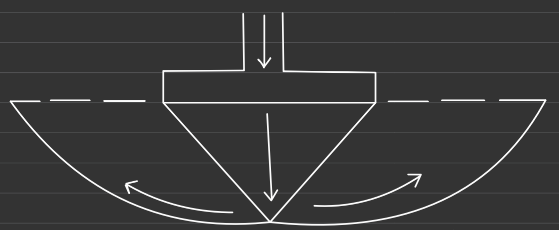
- **Allowable Bearing capacity ($q_{a\text{-}net}$)**
  Maximum net loading intensity at which neither the soil fails in shear nor there is excessive settlement.

Imp Topic ☆

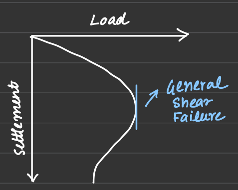
Principle Modes of soil failure

General shear failure

- Well defined failure pattern.
- A sudden, catastrophic failure accompanied by tilting of foundation.
- A bulging of ground surface adjacent to the foundation.



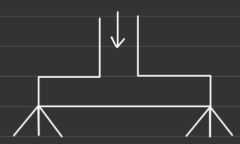
- This is failure surface
- Along that surface soil fails
- Well defined failure pattern

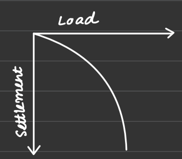


Principle Modes of soil failure

Punching shear failure

- Poorly defined shear planes.
- Soil zones beyond the loaded area being little affected.
- Significant penetration of a wedge shaped soil zone beneath the foundation.
- Ultimate load cannot be clearly recognised.

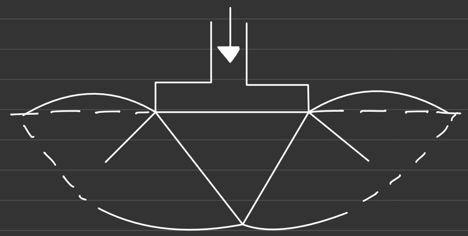


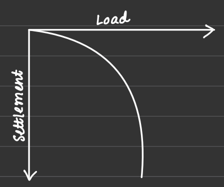


Principle Modes of soil failure

Local shear failure

- Well defined wedge and slip surfaces only beneath the foundation.
- Slip surfaces are not visible beyond the edges of the foundation.
- Slight bulging of the ground surface adjacent to the foundation.
- Significant compression of the soil directly below the foundation.
- Load settlement curve does not indicate the ultimate load clearly.





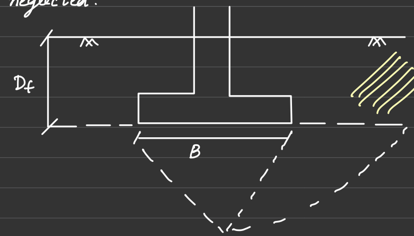
- These dotted surface are not fully developed.

Terzaghi's Bearing Capacity Theory

This theory is for strip footing, general shear failure, vertical load, horizontal ground surface

Assumptions

- 2D problem i.e. plane strain → go back to MoS notes! Bcoz of strip footing — one length very long
- Shallow footing i.e. $D_f \leq B$
- The shearing resistance of the soil between the surface and the depth D_f is neglected.

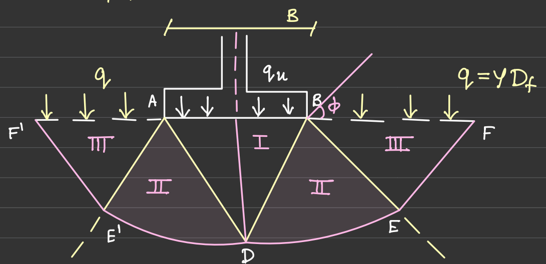


→ This part of soil is underutilized i.e. underestimating the soil.

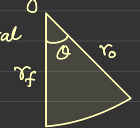
- General shear failure and soil volume is unchanged prior to failure.
- Mohr-Coulomb failure criterion $\tau = c + \sigma \tan(\phi)$ (τ → shear stress, c → cohesion, σ → normal stress, ϕ → angle of internal friction)

Different Zones

- Zone I triangular wedge, elastic, acts as a part of the footing, active zone. → Not gone to plastic region. → No plastic theory applicable.
- Zone II radial shear zone, curved boundary is logarithmic spiral. → under plastic region
- Zone III Rankine (a person) passive zones of linear shear.



→ log spiral

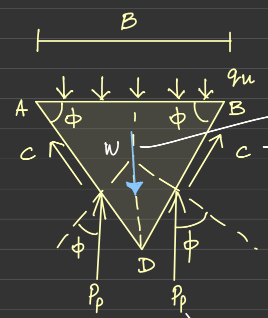


Eq^n: $r_f = r_0 \exp(\theta \tan\phi)$ — ϕ log spiral → Angle of internal friction.

Passive — towards the soil

Active — against the soil — Zone I is going away from the footing.

- Weight of the wedge $= \frac{1}{2} \gamma \times B \times \frac{B}{2} \tan\phi = \frac{1}{4} \gamma B^2 \tan\phi$ — Assumed base angle ϕ.



Triangular Wedge

→ weight of wedge (W)

→ cohesive force (cohesion)

(Total Passive Resistance) It comes from mathematics. Not a emperical relation of P_p.

- Consider the force equil^m of ΔABD

$$q_u B = 2P_p + 2C\sin\phi - \frac{1}{4}\gamma B^2 \tan\phi \quad \text{where } C = c \times \overline{AD} \text{ or } c \times \overline{BD} = c \times \frac{B}{2\cos\phi}$$

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

- The total passive resistance P_p is made up of three components
  - $P_{p\gamma} \rightarrow$ produced by weight of soil in shear zone $(c = q = 0)$
  - $P_{pc} \rightarrow$ produced by the soil cohesion $(\gamma = q = 0)$
  - $P_{pq} \rightarrow$ produced by the surcharge $(\gamma = c = 0)$

- Therefore,

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

$$q_u = \left(2P_{p\gamma} - \frac{1}{4}\gamma B^2 \tan\phi\right) + \left(2P_{pc} + BC\tan\phi\right) + 2P_{pq}$$

let $2P_{p\gamma} - \frac{1}{4}\gamma B^2 \tan\phi = B \times \frac{1}{2}\gamma B N_\gamma$ | let $2P_{pc} + BC\tan\phi = B \times cN_c$ | let $2P_{pq} = B \times q N_q$

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

Bible in Foundation Engineering

$N_c, N_q, N_\gamma \rightarrow$ Bearing Capacity Factors $\longrightarrow$ dependent on ϕ only

$\phi = 0$ (purely cohesive soil) $\Rightarrow N_\gamma = 0$, etc...

Terzaghi Bearing Capacity Factor Table.

- Local shear failure

$c_m = \frac{2}{3}c$; $\tan\phi_m = \frac{2}{3}\tan\phi$

(c_m ↓ mobilised cohesion; ϕ_m ↓ mobilised friction)

Use c_m and ϕ_m instead of c and ϕ in the bearing capacity eqⁿ.

$$q_u = \frac{2}{3}cN_c + qN_q + \frac{1}{2}\gamma B N_\gamma'$$

For $\phi > 36°$ — General shear Failure
For $\phi < 28°$ — Local shear Failure
For $28° < \phi < 36°$ — Interpolation b/w general and local shear failure.

For $D_r > 70\%$ — General shear failure
For $D_r < 20\%$ — Local shear failure

Modification for square and circular footing

- For square footing

$$q_u = 1.3cN_c + qN_q + 0.4\gamma B N_\gamma$$

- For Circular Footing

$$q_u = 1.3cN_c + qN_q + 0.3\gamma B N_\gamma$$

Terzaghi Theory was purely for strip footing.

Effect of Water Table

- If $D_w' \geq B$, $\gamma = \gamma_t$ (Bulk unit weight) in both q and $0.5\gamma B N_\gamma$
- If $D_w' = 0$, $\gamma = \gamma'$ (submerged unit weight) in $0.5\gamma B N_\gamma$

When $0 < D_w' \leq B$, $\gamma = \gamma' + \left(\frac{D_w'}{B}\right)(\gamma_t - \gamma')$ (γ_t: Bulk unit wt.; γ': submerged)

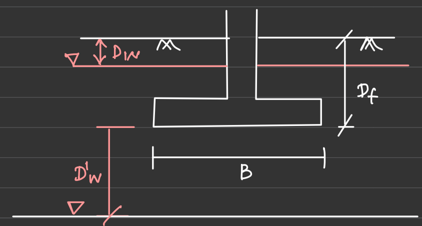
Effect of Water Table

- If $D_w = 0$, $\gamma = \gamma'$ in both q and $0.5\gamma B N_\gamma$
- If $0 < D_w < D_f$, $\gamma = \gamma' + (D_w/D_f)(\gamma_t - \gamma')$ in q term and γ' in $0.5\gamma B N_\gamma$ term.

- Another way

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

where $R_w, R_w' \equiv$ correction factors for water table.



* For Sandy soil $(c = 0)$

$$q_u = qN_q + 0.5\gamma B N_\gamma$$

It will be effected by γ' (water table)
$\gamma \rightarrow \gamma_s$ (submerged)

* Water Table will lower the bearing capacity of the soil.

Meyerhof's Analysis

Assumptions

- Bearing capacity of a strip footing at any depth.
- Failure surfaces extend above the foundation level.
- Shear resistances of the soil above the base of the foundation was considered.

$$q_u = cN_c s_c d_c i_c + qN_q s_q d_q i_q + \frac{1}{2}\gamma B N_\gamma s_\gamma d_\gamma i_\gamma$$

s – shape factor
d – depth factor
i – inclination factor

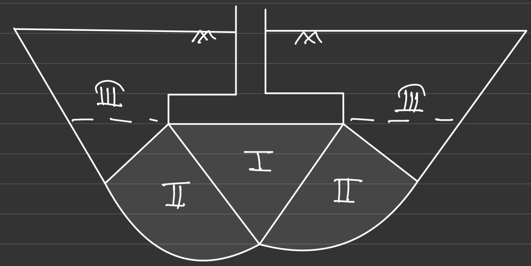
$N_c = (N_q - 1)\cot\phi$, $N_q = e^{\pi\tan\phi}\tan^2(45+\frac{\phi}{2})$
$N_\gamma = (N_q - 1)\tan(1.4\phi)$

Hansen's Analysis

- Same as Meyerhof, but the Eq^n is valid for $\phi > 0$.
- For $\phi = 0$ (purely cohesive soil), $q_u = cN_c(1 + s_c + d_c - i_c) + q$

$N_c = (N_q - 1)\cot\phi$
$N_q = e^{\pi\tan\phi}\tan^2(45 + \frac{\phi}{2})$ and

$N_\gamma = 1.5(N_q - 1)\tan\phi$



★ Vesic's Analysis

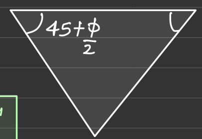
- same as Meyerhof, but the Eq^n is valid for $\phi > 0$

$N_c = (N_q - 1)\cot\phi$ $\quad N_q = e^{\pi\tan\phi}\tan^2(45 + \frac{\phi}{2})$

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

→ Most researchers find this because this is very sensitive.

IS Code Recommendations [IS 6403-1981]



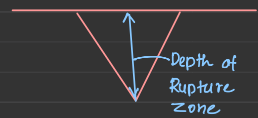
- Net ultimate bearing capacity

$$q_{nu} = cN_c s_c d_c i_c + q(N_q - 1)s_q d_q i_q + \frac{1}{2}\gamma B N_\gamma s_\gamma d_\gamma i_\gamma W'$$

N_c, N_q, N_γ are B.C. factors recommended by Vesic. W' is a factor that takes into account the effect of water table.

- At $D_w' = B$, $W' = 1$ — no effect of water table, far away from water table.
- At $D_w' = 0$, $W' = 0.5$ — half capacity
- Linear Interpolation between 0.5 and 1 for $0 < D_w' < B$

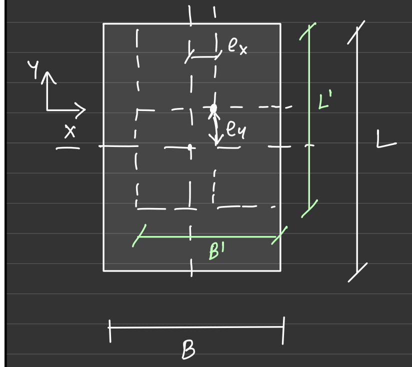
Bearing Capacity of Footing in layered soils



- The Depth of Rupture zone $= 0.5B\tan(45° + \frac{\phi}{2})$
- For c

Eccentric Loading

Actually load is eccentric, we'll make it concentric by modifying the dimensions of footing.

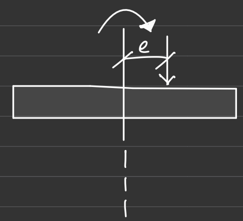


The dimensions of footing have to be modified :-

$B' = B - 2e_x$

$L' = L - 2e_y$

$A' = B' \times L'$



Numerical Problems

In a prob.
What will be the ultimate B.C for strip footing ?

$c' = 0 \quad \gamma_d = 17\ kN/m^3 \quad \phi' = 38^\circ$

$B = 1.5m \quad D_f = 1m \quad$ (As per IS 6403-1981)

For $\phi' = 38^\circ$ I need N_q, N_γ ? → $N_q = 60,\ N_\gamma = 75$

$$q_u = cN_c s_c d_c i_c + q(N_q - 1) s_q d_q i_q + \frac{1}{2}\gamma B N_\gamma s_\gamma d_\gamma i_\gamma W' =$$

$cN_c s_c d_c i_c$ → 0 ($\because c' = 0$); q = overburden pressure

$i_\gamma = i_q = 1$ ($\because$ No inclined surface)
Inclination factor

$d_q = d_\gamma = \boxed{1 + 0.1 \frac{D}{B} \tan\left(45 + \frac{\phi}{2}\right)}$ → As per IS Code
depth factor

$d_q = d_\gamma = 1.137$

strip — 1
square, circular } → Formula in IS Code

$s_q = s_\gamma = 1$
safe factor

$W' = 1$ → $W' = 0.5$

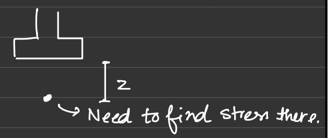
CE352A
FOUNDATION DESIGN
CHAPTER 3

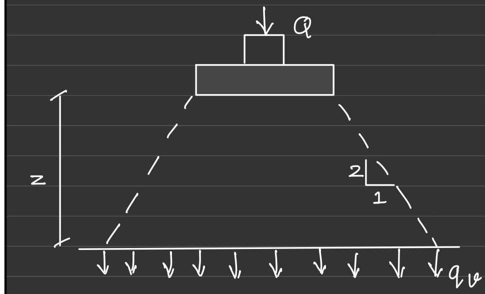
Stresses in Soil Mass due to Footing Pressure

Early Method

- An Early Method is to use 2:1 slope.
- The pressure increase q_v at depth z beneath the loaded area due to base load Q.

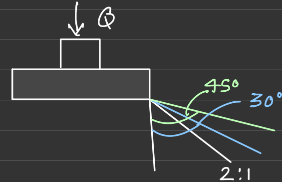
$$q_v = \frac{Q}{(B+z)(L+z)}$$





· Not right/scientifically correct
→ Assumption.

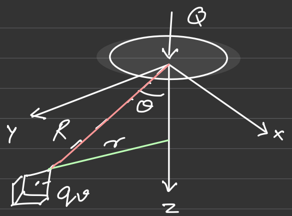
· Others have proposed



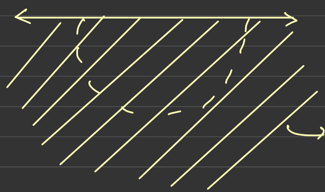
Boussinesq Method (1885)

stress developed due to point load. Truly, No point load below found foundation..

$$q_v = \frac{3Q}{2\pi z^2} \cdot \frac{1}{\left[1 + \left(\frac{r}{z}\right)^2\right]^{5/2}}$$



elastic half space



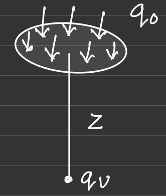
↳ only this half considered. → MOS

→ No single location where there is max. load.

Modification

$$q_v = q_0\left(1 - \frac{1}{\left[1+\left(\frac{r}{z}\right)^2\right]^{3/2}}\right)$$

↳ can be directly applied at depth z for a round footing of radius r.



Application to line loads

$$q_v = \frac{2z^3 q}{\pi(x^2+z^2)^2}$$

$q \rightarrow$ stress at the top
$q_v \rightarrow q$ at some depth

square and rectangular loaded area

B×L area

$$q_v = q_0 \frac{1}{4\pi}\left[\frac{2MN\sqrt{V}}{V+V_1} \cdot \frac{V+1}{V} + \tan^{-1}\left(\frac{2MN\sqrt{V}}{V-V_1}\right)\right]$$

↳ Derivable not emperical

$M = \frac{B}{z}$, $N = \frac{L}{z}$ $(q_v = q_0 \text{ for } z = 0)$

$V = M^2 + N^2 + 1$, $V_1 = (MN)^2$

When $V_1 > V$ then $\tan^{-1}$ term = -ve and its necessary to add π

$$q_v = q_0 I_\sigma$$

↳ stress influence value → table for a given B×L

Westergaard's Method

— some advancement in boussinesq method — for some type of soil
— When soil is layered

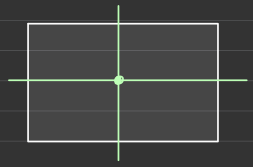
} still elastic half space but non-homogenous

Poisson's effect :— Apply load → lateral deflection is considered

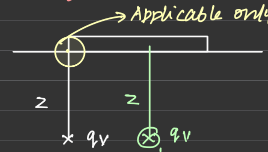
$a = \frac{1-2\mu}{2-2\mu} = f(\mu) \Rightarrow$ poisson's effect taken into consideration

↳ not empirical — derivable

* Applicable for corner of rectangular area



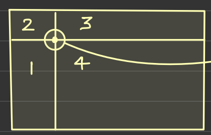
↳ Divide the area
use method of superposition



→ Applicable only for corner (Not any part)

↳ How to find q_v here?

- can find for any area by dividing the area and using the method of superposition.



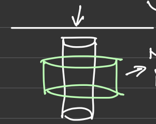
→ $q_v = \dots ?$

Make that point a corner point of the other area.

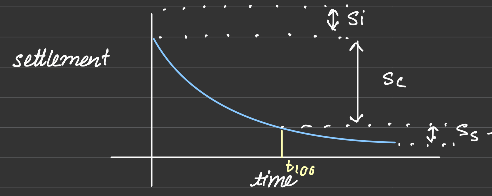
SETTLEMENT OF SHALLOW FOUNDATION

$S_t = S_i + S_c + S_s$

- S_c → primary consolidation settlement ↓ CE351
- S_s → secondary settlement ↓ only rearrangement can happen ↓ often neglected (very negligible)

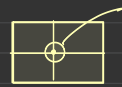


→ No change in volume

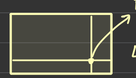


S_s → negligible — typical soils
↓
very significant soil may have a little high, otherwise always low.

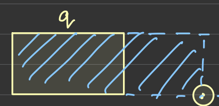
E_s – CE351A – from lab
I_1, I_2, I_3 – influence factors
$B' = B/2$ for center
B for corner
$L' = L/2$ for center
L for corner



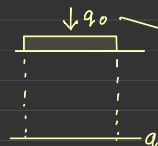
Always we need corner



$B' = \frac{3}{4} B$ → B will be always shortest.
$L' = \frac{3}{4} L$



→ To find q_v here, what to do?
Extend the foundation and make an imaginary extension.



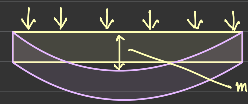
q_0 → Cause → Put cause in eqn

→ Effect — ultimately calc q_v

q_v → This stress is used for settlement analysis and not q_0

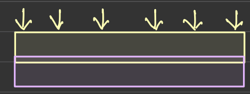
I_1, I_2, I_3 .... → These factors are implicitly making use of q_v.
q_v is not directly calculated, but implicitly taking q_v there.

Flexible Foundation?



max. settlement

Flexible Foundation

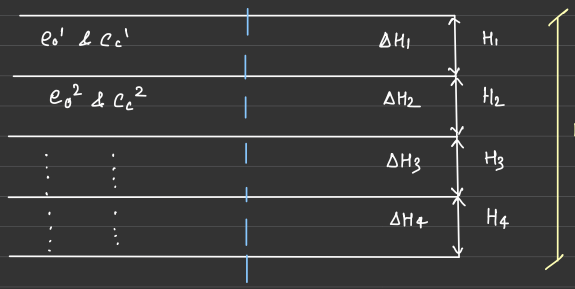


less settlement around 7%

Rigid Foundation

C_c – compression index $= \frac{\Delta e}{\log(P_2/P_1)}$

$S_c = \Delta H = \frac{\Delta e}{1+e_0} H$

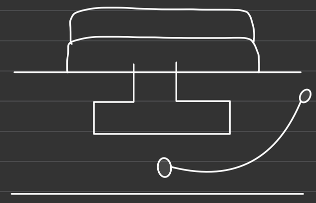


$$\Delta H = \sum \Delta H_i$$

P_0 at upper interface → Lower

P_0 at lower interface → Higher

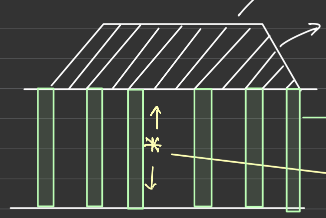
that's why P_0 is calculated at the mid point of soil layer.



Problem – Consolidation Theory

soft clay + water

sand / gravel

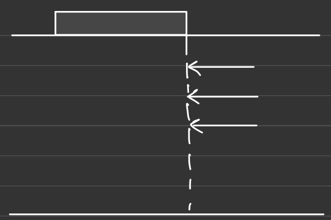


16 years → As a geotech engineer, people will kill you!!

Leave this load for a lot amount of time before constructing a building

→ Drainage Pipes → Beyond this course's scope, but it happens actually. (made of PVCs, etc)

→ solution :- Put vertical drains made of gravel, etc.

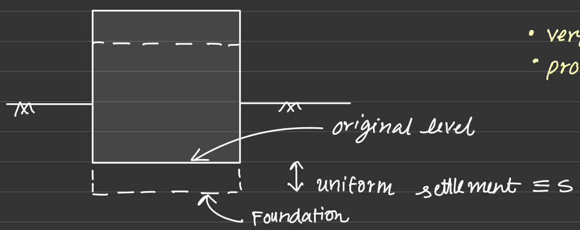


· sand need confinement

· you have to provide it confinement.

Settlement of Shallow Foundation

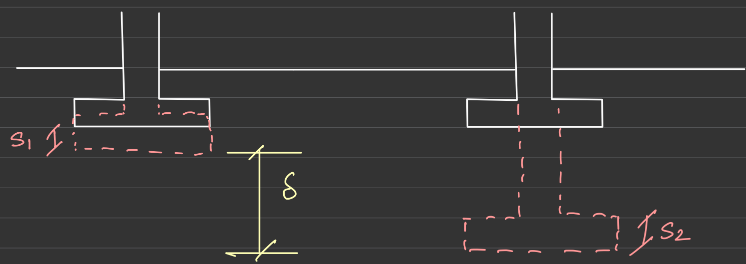
Uniform Settlement



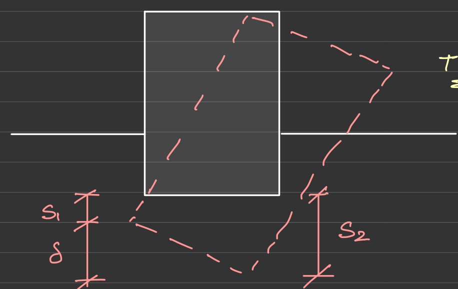
• very unlikely to happen

• problem → serviceability.

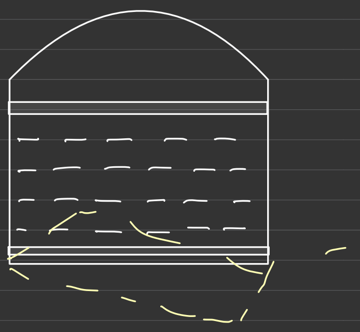
Non-uniform Settlement

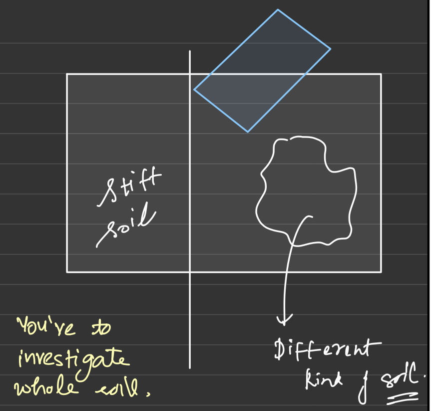


Angular Distortion δ

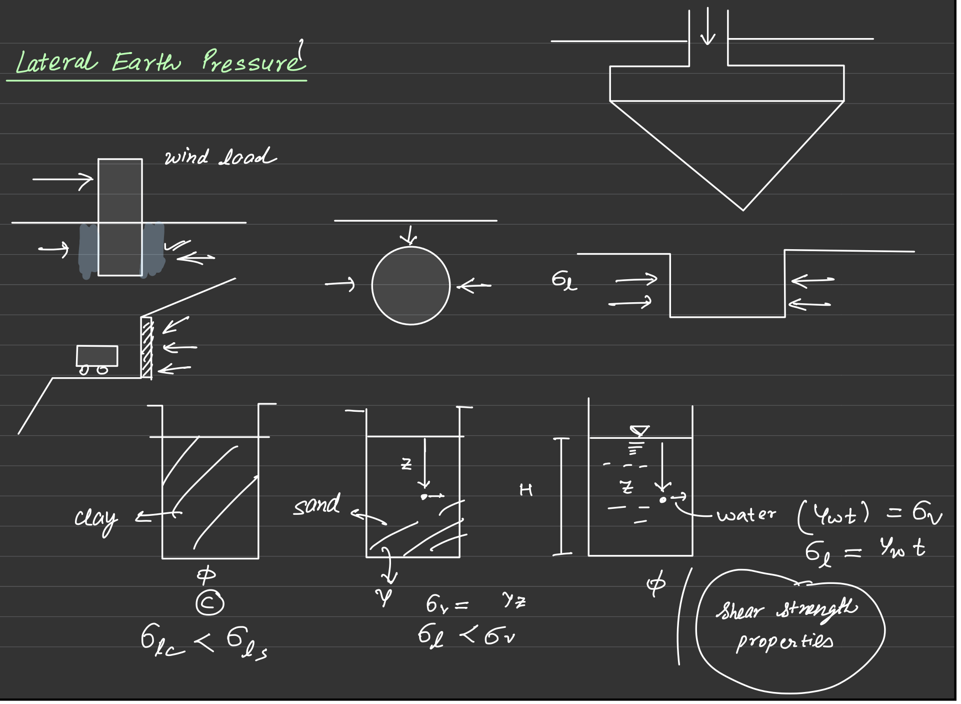


Tilting

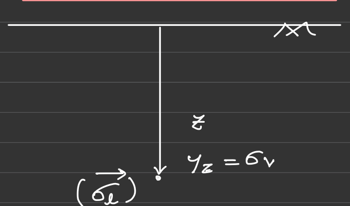




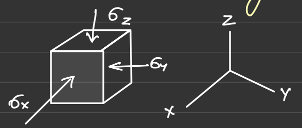
Lateral Earth Pressure



Lateral Stress 'at-rest'



Elasticity



(E, ν)

$$\varepsilon_z = \frac{\sigma_z}{E} - \frac{\nu\sigma_x}{E} - \frac{\nu\sigma_y}{E}$$

$$= \frac{\sigma_z}{E} - \frac{2\nu}{E}(\sigma_x)$$

$$\sigma_x = \sigma_y$$

$$0 = \varepsilon_x = \frac{\sigma_x}{E} - \frac{\nu\sigma_z}{E} - \frac{\nu\sigma_x}{E}$$

$$\sigma_x - \nu\sigma_z - \nu\sigma_x = 0$$

$$\nu\sigma_z = \sigma_x(1-\nu)$$

$$\frac{\sigma_x}{\sigma_z} = \left(\frac{\nu}{1-\nu}\right)$$

$$\sigma_x = \sigma_z\left(\frac{\nu}{1-\nu}\right)$$

$$= \gamma_z\left(\frac{\nu}{1-\nu}\right)$$

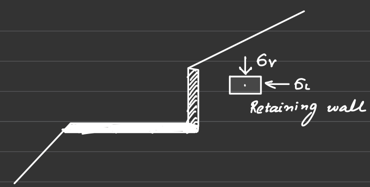
Factor K

Soil: → $\sigma_x = \sigma_z \times K$ — Empirical Relⁿ

→ $K = [1-\sin\phi]$

$= [1-\sin\phi]\, OCR^{0.5}$

$K = [1-\sin\phi](OCR)^{0.5}$ where OCR = Over Consolidation Ratio



$$\sigma_v = \gamma z$$

$$\sigma_l = \sigma_v K_o$$

$$= \sigma_v K\downarrow$$

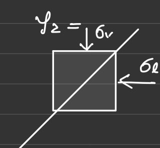
$$= \sigma_v\left(\frac{1-\sin\phi}{1+\sin\phi}\right)$$

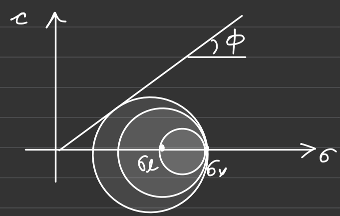
Active Earth Pressure condition [soil is pushing the soil]

*Other case:- Passive Earth Pressure [wall is pushing the soil]

Soil Failure → Shear Failure

$$\tau_f = c + \sigma\tan\phi \quad \text{(Mohr Coulomb Criteria)}$$





$$\sigma_1 = \sigma_3\left(\frac{1+\sin\phi}{1-\sin\phi}\right) + 2c\left(\frac{\cos\phi}{1-\sin\phi}\right)$$

$$= \sigma_3\tan^2\left(45+\frac{\phi}{2}\right) + 2c\tan\left(45+\frac{\phi}{2}\right)$$

$$\sigma_3 = \sigma_1\,\frac{1-\sin\phi}{1+\sin\phi}$$

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

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

$= \sigma_v \, K_p$

$\theta = 45 - \frac{\phi}{2}$

$\phi = 30^\circ$

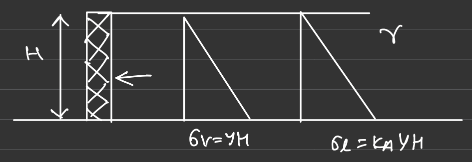
0.3	0.5	3
K_A	K_o	K_p

Active — lowest condition

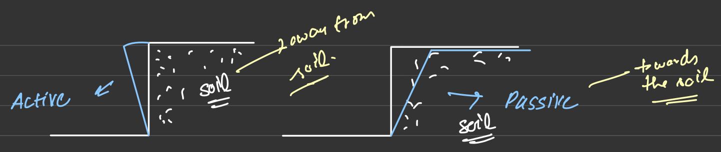
'at rest' — somewhere in b/w

Passive — highest condition

	θ
$K_A = \tan^2\left(45 - \frac{\phi}{2}\right) = \frac{1-\sin\phi}{1+\sin\phi}$	$45 + \frac{\phi}{2}$ (with horizontal)
$K_p = \tan^2\left(45 + \frac{\phi}{2}\right) = \frac{1+\sin\phi}{1-\sin\phi}$	$45 - \frac{\phi}{2}$
$K_o = 1 - \sin\phi$	× → No failure so no angle



$P_A = \frac{1}{2} \times (K_A \gamma H) \times H$



Lateral Earth Pressure coefficient $K = \frac{\sigma_h}{\sigma_v}$ — K_o, K_A, K_f

$K_o = 1 - \sin\phi$ empirical relⁿ or $\left(\frac{\nu}{1-\nu}\right)$

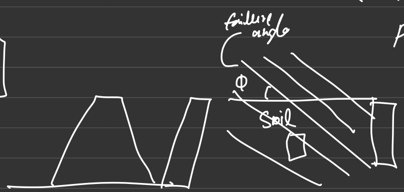
→ $(1-\sin\phi')\, OCR^{\sin\phi'}$

$OCR = \left(\frac{P_o}{P}\right) \geq 1$

$P_o > P$

$K_A = \tan^2\left(45 - \frac{\phi}{2}\right)$

↓ But the angle will be $\left(45 + \frac{\phi}{2}\right)$



$K_p = \tan^2\left(45 + \frac{\phi}{2}\right)$ → Angle of failure $(\theta) = \left(45 - \frac{\phi}{2}\right)$

$\sigma_x = \gamma z$

$\sigma_1 = \sigma_3 \tan^2\left(45 + \frac{\phi}{2}\right) + 2c\tan\left(45 + \frac{\phi}{2}\right)$

($\sigma_1 \to \sigma_v$, $\sigma_3 \to \sigma_l$)

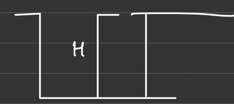
$\sigma_3 = \sigma_1 \tan^2\left(45 - \frac{\phi}{2}\right) - 2c\tan\left(45 - \frac{\phi}{2}\right)$

$0 = \sigma_1 K_A - 2c\sqrt{K_A}$

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

$0 = \gamma z K_A - 2c\sqrt{K_A}$

$z_c = \frac{2c}{\gamma\sqrt{K_A}}$ → tension crack.



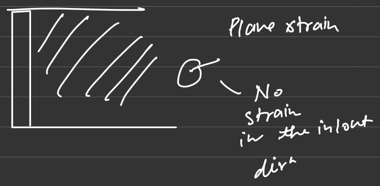
Maximum vertical cut

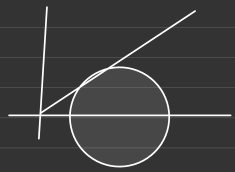
$H = 2 z_c$

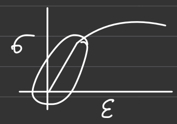
Rankine's Earth Pressure

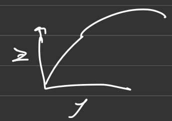
K_A
K_P

Coulomb Earth Pressure Theory

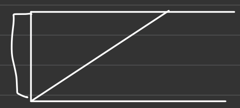


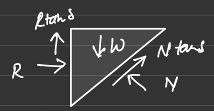






Rigid Plastic scenario.





upper bound theory
coulomb's Theory

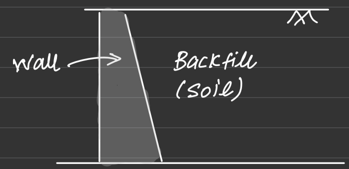
DAY-3

Lateral Earth Pressure

K_0 – at rest coefficient
K_A – Active "
K_P – Passive "

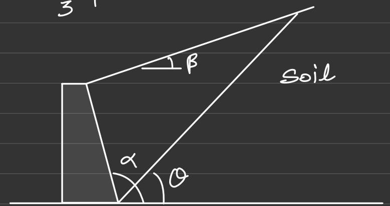
① Rankine → Stress Equilibrium (Lower Bound)

② Coulomb's Earth Pressure → Force Equilibrium (Upper Bound)

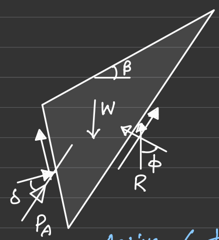


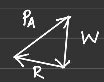
ACTIVE CASE

$\delta = \frac{1}{3}\phi$

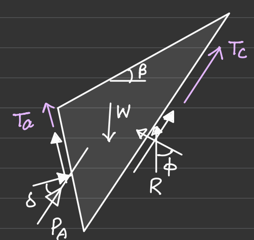


Active. (φ) — No cohesion → consider a failure wedge.

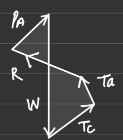




Active (φ, c)

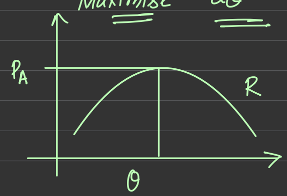


$T_a = c \times \text{length}$
$T_c = c \times \text{length}$



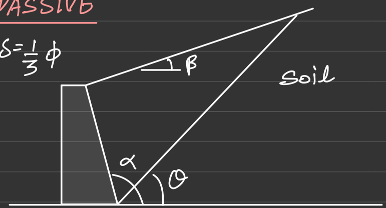
$P_A(\theta, \alpha, \beta, \phi, \delta, T_c, T_a)$

Maximise $\frac{dP_A}{d\theta} = 0$

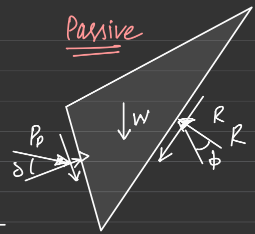


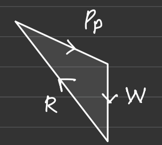
PASSIVE

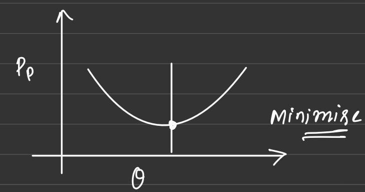
$\delta = \frac{1}{3}\phi$



Passive

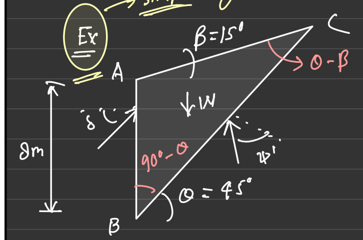






→ Simple Trigonometric Problems

Ex



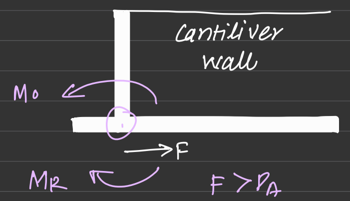
$$\frac{AB}{\sin(\theta - \beta)} = \frac{AC}{\sin(90 - \theta)}$$

$$AC = \frac{AB \sin(90-45)}{\sin(45-15)} = AB\sqrt{2}$$

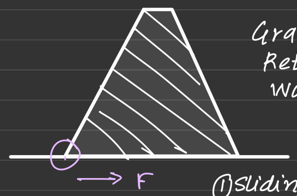
$AB = 8m$ (given)

$\therefore AC = 8\sqrt{2}$ m

Retaining Wall Design



$F > P_A$

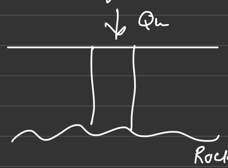


Gravity Retaining wall

① sliding
② overturning

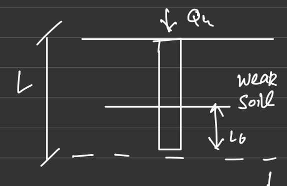
Estimating Pile length

Point bearing piles



$Q_u \approx Q_p$ → some amount of resistance will be there which will be negligible.

Rock → No settlement will be there.



strong soil layer.

$Q_u \simeq Q_p + Q_s$ → due to settlement

→ No rock here → some settlement will be there

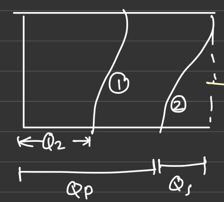
Q_p —
Q_s — } major component of pile resistance

Q_s — small then $Q_u \approx Q_p$

Friction piles — major component Q_s — dominant

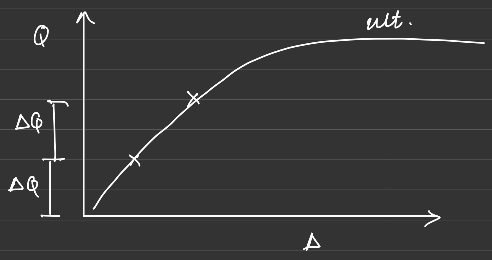
↳ also known as protein pile — do support at bottom.

Q_2 – at the base
Q_1



non-uniform

→ frictional resistance not uniform → continuously varying.



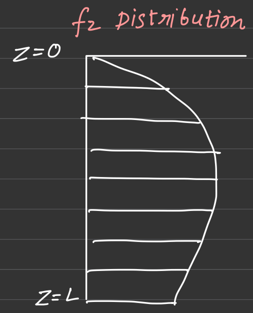
skin friction
tip resistance } → not taking → happening at ult.

• Frictional resistance not ↑sing with time.

frictional res. per unit at depth.

$f_z = \frac{\Delta Q_z}{p\,(\Delta z)}$

f_z Distribution



$f_z = \frac{\Delta Q_z}{p.(\Delta z)}$

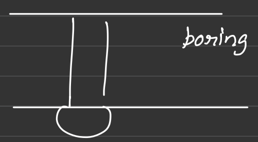
· People have done lot of research in it.
· Not a straight forward distribution.

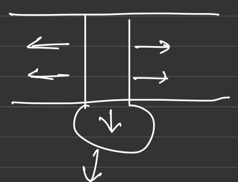
• 5-10 mm → relative displacement
↓
required to mobilise fully the pile foundation.
↓
max. frictional resistance

Ex: Kanpur Metro — 500 mm
↳ 10% = 50mm
↳ This much settlement cannot afford.

Why 10% for driven piles?

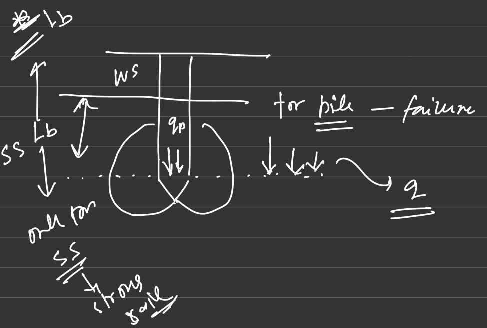
Why 25% for bored piles?





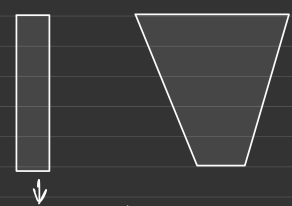
driving
↓
making soil more dense
↓
compacting
↓
little bit settlement will give more resistance.

Q_p, Q_s → these terms
↓
got what they are!



$Q_p = A_q Q_p = A_p\,(c' N_c^* + q N_q^*)$
↓
Imp. ☆ Eq^n — Eq^n(5)

* These N_c and N_q are the same B.C. factors for shallow foundation.
* No seperate N_c, N_q for pile.



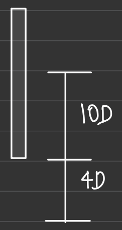
Tapered pile
↳ 'p' not constant

'p' — constant for this

$\bar{f}$ and $\bar{p}$
↓
from load distribution
↓
different and not constant.

(p – perimeter)

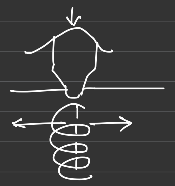
(f – unit friction) — quite difficult to obtain



Pile Installation
- Driven Piles — densification happens
- Bore Piles — no densification happens

$L' = 15D$
└ Diameter of pile

K varies with depth.
Not a constant.



→ At top you can expect some kind of passive movement.

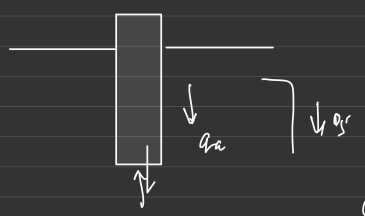
Why $K \neq K_p$ at top and $K = K_o$?
Bcoz there is some passive movement and no passive movement.
ϕ' — effective angle of internal friction.

Rem. $f_{av} = 0.02 p_a (\bar{N}_1)_{60}$ → use only for high displacement driven pile.

$f = \alpha' q_s$ ↓ varies

Main target → f find → skin res.

If any foundation is manufactured on rock base — very difficult to get.



S_1 & ξ

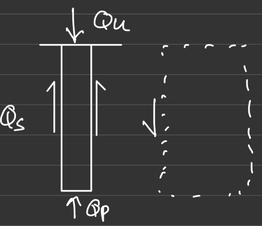
S_1, S_2, S_3

Q_{ws} —

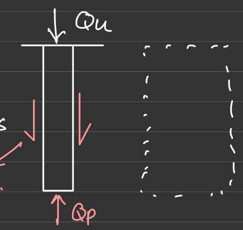
ξ — symbol — ξ

• Q_u — total capacity of pile foundation.
$= \frac{W_R h}{S + C}$

V.Imp → ★ Negative skin friction
↓
Very problematic term in pile foundation!!



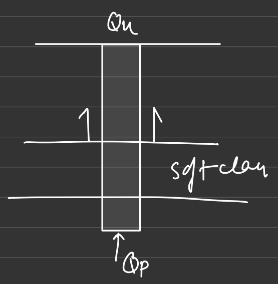
P = perimeter



Acting along Qu
↓
Reduce the capacity.

• Why sand → working as a drainage pipe — facilitates drainage.

$f_n = K'\sigma_0' \tan\delta$ → coming from Mechanics



$Q_u \approx Q_p + Q_s \rightarrow 0$

Negative Skin Friction (Granular soil fill over clay)

★ Rem 'z' is starting from sand

↳ DON'T DO MISTAKE IN EXAM

sand is overlying soft clay.

Wind Turbine – constructed on mono single bulky pile

Water construction → Retaining wall → Buffer wall
→ Drain out water
→ then construct.

Group Efficiency