• Classification of Soils

1. Residual Soils - Formed by weathering of rocks and still occupy the position of the rock from which it has been formed.
   a) Cohesionless soils are formed due to physical disintegration of rocks.
   b) Cohesive (Clay) soils are formed by Chemical weathering.

2. Transported Soil - Soil has been transported from its place of origin by wind, water, ice or some other agency, and has been re-deposited.
   a) Aeolian soil - Transported by wind (Eg: Loess, is a silt or silty clay)
   b) Alluvial soil - Transported by water (Eg. River Sand)
   c) Colluvial soil - Transported by Gravity (Eg. Talus)

3. Cumulose Soil - The accumulation (collection) of Decaying and Chemically deposited Vegetable matter under the Condition of Excessive moisture.
   Eg: Peat & Muck.

• Properties of Fine Grained Soil

1. Plasticity
   Property of a soil which allows it to be deformed rapidly without rupture, without elastic rebound (i.e to original shape) and without volume change.

2. Cohesion
   The capacity of soil to resist shearing stress.

   Thixotropy
   The phenomenon of 'Strength lose and strength gain' when allowed to rest.
   (i.e The formation of loose sand into Dense sand).

3. Consolidation
   Is the process involving a decrease in the water content of saturated soil without replacement of the water by air.
   (i.e The process of Expulsion of water from pores due to Compaction) under steady pressure/load

   Swelling
   opposite to Consolidation.
   i.e Is the process involving increase in the water content due to an increase in the volume of voids.

ISI particle size Classification

Clay	Silt	Sand			Gravel		Cobble	Boulder
		Fine	Medium	Coarse	Fine	Coarse		
2μ	75μ	425μ	2mm	4.75mm	20mm	80mm	300mm	>300

∴ Loam = It is a mixture of Sand, silt & clay.

Unified Soil Classification System (USCS)

Soil Type	Prefix
(1) Gravel	G
(2) Sand	S
(3) Silt	M
(4) Clay	C
(5) Organic	O
(6) Peat	Pt

Sub Group	Suffix
(i) Well graded	W
(ii) Poorly graded	P
(iii) Silty	M
(iv) clayey	C
(v) Low plastic ($W_L < 50\%$)	L
(vi) High plastic ($W_L > 50\%$)	H

Particle Size Distribution Curve

a) Well graded Soil

The soil has good representation of particles of all sizes.

b) Poorly (or) uniformly graded soil

The soil has an Excess of Certain particles and deficiency of other,

(or) If it has most of particles of about the same size.

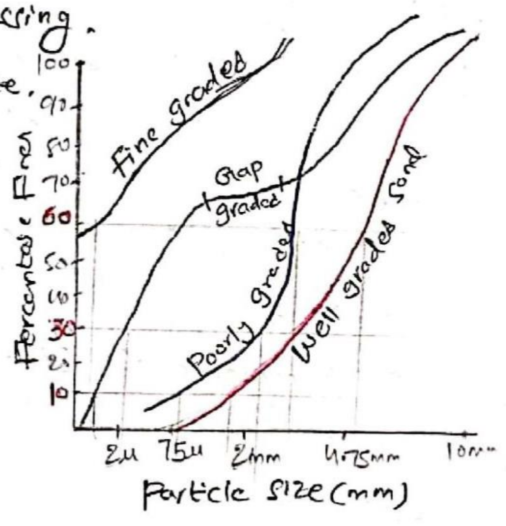
c) Gap (or) Skip graded soil

Some intermediate size particles are missing.

D_n = n% of the particle are finer than that size.

(eg: For well graded Curve

D_{30} = 30% of the particles are finer than 3mm (from graph)



Note

(D_{10} = Effective size (or) Effective Diameter)

1. Measure of particle size range by

$$\text{Uniformity Coefficient} (C_u) = \frac{D_{60}}{D_{10}}$$

2. The Shape of particle size curve by

$$\text{Coefficient of Curvature} (C_c) = \frac{(D_{30})^2}{D_{10} \times D_{60}}$$

*) Classify by Cu & Cc

a) For poorly (or) uniformly graded soil $C_u \simeq 1$

b) For Well Graded Soil

$C_c = 1 - 3$

$C_u > 4$ (For gravel)

> 6 (For Sand)

Sedimentation Analysis (By stoke's law)

∵ Silt & Clay particles (i.e < 75μ) Cannot be seperated by Sieving. Hence The soil particles less than 75μ Size can be analysed by Sedimentation.

* Stoke's law - The velocity at which grains Settle for Suspension - all factors (i.e shape, size & weight) being equal.

* Sedimentation tests a) pipette method b) Hydrometer method @ 27°c.

* Limitations:
→ Stokes law is valid for particle Dia. 200μ - 0.2μ
→ Considering all soil particles are Spherical (But actual they are Flaky)
→ All have same specific gravity (G).
→ The walls of jar & adjacent particles do not effect the particle settlement
* The terminal velocity of spherical particle (By stokes law)

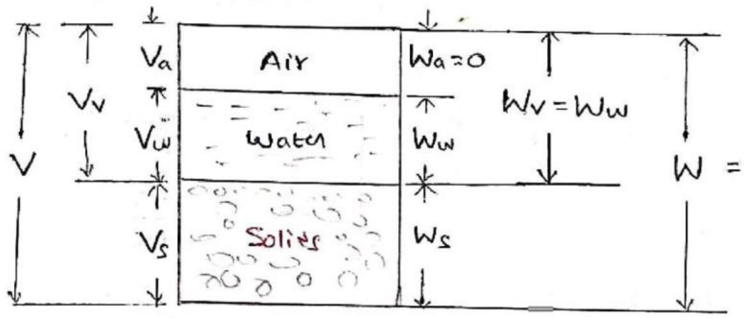
$$V = \frac{D^2}{18}\frac{(G-1)\gamma_w}{\mu} \quad (m/s) \qquad (\because \mu = \text{viscosity } (kNs/m^2))$$

Three-phase System of Soil

$\therefore V_v = V_w + V_a$

$V = V_v + V_s$

$= V_a + V_w + V_s$



Technical Terms used in Soil Mechanics.

1. Water Content $(w) = \frac{W_w}{W_s} \times 100 = \left[\frac{W}{W_s} - 1\right]100$

$= \frac{M_w}{M_s} \times 100 = \left[\frac{M}{M_s} - 1\right]100$

	Density of Soil (kg/m³)	Unit wt of Soil (kN/m³)
a	Bulk (or) Moist Density $\rho = \frac{M}{V}$	Bulk (or) Moist unit wt $\gamma = \frac{W}{V}$
b	Dry Density $\rho_d = \frac{M_s}{V}$	Dry unit wt $\gamma_d = \frac{W_s}{V}$
c	Density of Solids $\rho_s = \frac{M_s}{V_s}$	Unit wt of Solids $\gamma_s = \frac{W_s}{V_s}$
d	Saturated Density $\rho_{sat} = \frac{M_{sat}}{V}$	Saturated unit wt $\gamma_{sat} = \frac{W_{sat}}{V}$
e	Submerged (or) Buoyant Density $\rho_{sub} = \frac{(M_s)_{sub}}{V}$	Submerged (or) Buoyant unit wt $\gamma_{sub} = \frac{(W_s)_{sub}}{V}$
	$\rho_{sub} = \rho_{sat} - \rho_w$	$\gamma_{sub} = \gamma_{sat} - \gamma_w$

∵ Soil is fully Saturated ($V_a = 0 \Rightarrow V_v = V_w$ & $W_{sat} = W_s + W_w$)

$\rho_w = 1000 kg/m^3$ $\qquad \gamma_w = 9810 N/m^3 = 9.81 kN/m^3$.

4. Specific gravity of Soil Solids (G_{rm}) $= \frac{\gamma_s}{\gamma_w}$

a) Apperent (or) Mass (or) Bulk Sp. gr (G_m) $= \frac{\gamma}{\gamma_w}$

5. Void Ratio (e) $= \frac{V_v}{V_s}$

6. porosity (n) $= \frac{V_v}{V}$

7. Degree of Saturation of Soil (S) $= \frac{V_w}{V_v}$

→ The Submerged Soils are fully Saturated (i.e $V_v = V_w$) ⇒ $S = 1$

→ For a perfectly dry soil mass ($V_w = 0$) ⇒ $S = 0$

8. percentage air voids (n_a) $= \frac{V_a}{V} \times 100$

9. Air Content (a_c) $= \frac{V_a}{V_v}$

10. Relative Density (or) Degree of Density (or) Density Index of soil mass

$$I_D = \frac{e_{max} - e}{e_{max} - e_{min}} \quad \begin{bmatrix} \text{Natural State } (e) \\ \text{Densest state } (e_{max}) \\ \text{Loosest State } (e_{min}) \end{bmatrix}$$

Important Relations

1. $e = \frac{n}{1-n}$ (or) $n = \frac{e}{1+e}$

2. $Se = wG$

3. $a_c = 1 - S$ (or) $n_a = n a_c$

$$\gamma = \frac{G+Se}{1+e}\gamma_w = \gamma_d(1+w)$$

$$\gamma_d = \frac{G\gamma_w}{1+e} = \frac{\gamma}{1+w} \quad (\because S=0)$$

$$\gamma_{sat} = \frac{G+e}{1+e}\gamma_w \quad \left[\frac{G+Se}{1+e} = \frac{G+wG}{1+e} = G\left(\frac{1+w}{1+e}\right)\right]$$

$$\gamma_{sub} = \gamma_{sat} - \gamma_w = \frac{G-1}{1+e}\gamma_w$$

Cosistency of Soils (Atterberg limits)

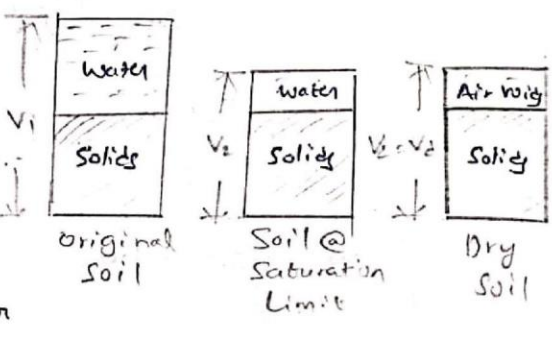
Consistency :- Degree of firmness (Cohesiveness) of Soil. which may be termed; Soft, Firm, Stiff & Hard

Liquid Limit (w_L) - The minimum water content at which the soil is still in liquid state. But has a small Shearing strength against flowing. (A Device measured under the impact of 25 blows)

plastic Limit (w_p) - The minimum water content, which makes the Soil to be rolled in to 3mm Dia thread

Shrinkage Limit (w_s) - The minimum water content, at which the Soil is fully saturated, (or)

The maximum w.c of Saturated Soil at which a reduction in it's moisture does not cause a decrease in volume of the Soil.



∴ At Shrinkage limit $\boxed{S=1}$ ⇒ $e = wG$

Indexes

a) Plasticity Index (I_P) = $W_L - W_P$

The range of Consistency within which a Soil exibits plastic properties ($\because$ $I_P = 0$ @ $W_L < W_P$) (i.e I_P Not -ve)

b) Consistency Index (I_C) = $\dfrac{W_L - W}{I_P}$

Is useful in the Study of the field behaviour of Saturated fine grained Soils. $\because$ $I_C = 0$, i.e Soil is at Liquid Limit

$= 1$, i.e soil is at plastic limit

> 1 i.e Soil has a Semi solid state & will be stiff

< 0 (i.e $W_L < W$) Soil behaves like a liquid.

c) Liquidity Index (I_L) = $\dfrac{W - W_P}{I_P}$

$\therefore$ $\boxed{I_C + I_L = 1}$

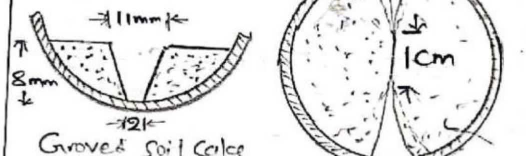
d) Flow Index (I_f) = $\dfrac{W_1 - W_2}{\log_{10}\frac{n_2}{n_1}}$

Is also Called as 'Slope of the flow curve"

Used to determine "Liquid limit (W_L) of soil.

e) Toughness Index (I_T) = $\dfrac{I_P}{I_f}$

Finding W_L by Casagrade apparatus for Soil < 425µ



Groved Soil Cake before test

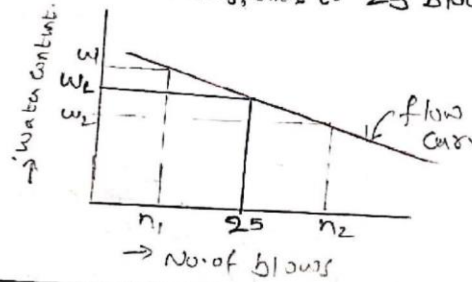
Soil Cake after test

$\because$ n = No. of blows Counted, When bottom groove Contact about 1cm.

$\because$ W_1 = W.C Corresponding to n_1 blows

W_2 = W.C Corresponding to n_2 blows.

W_L = W.C Corresponds to 25 blows.



Activity of Soil

It is a measure of Water holding capacity of Clayey Soils.

→ Activity also depends upon Nature & Shape of Clay mineral.

$\therefore$ Activity (A_C) = $\dfrac{I_P}{\%\text{ of Clay}}$

Activity	Soil Type
$A_c < 0.75$	Inactive
$0.75 < A_c < 1.40$	Normal
$A_c > 1.40$	Active

Activity	Clay Mineral
(Highly Active) $A > 4$	Monotomorillinite
$1 < A < 4$	Illite
(Low Active) $A < 1$	Kaolinite

Sensitivity of Soils

The degree of disturbance of Undisturbed Clay Sample, due to Remoulding (i.e Reworking)

$$\text{Sensitivity} = \frac{q_u\,(\text{Undisturbed})}{q_u\,(\text{Remoulded})}$$

q_u = Unconfined Compressive Strength

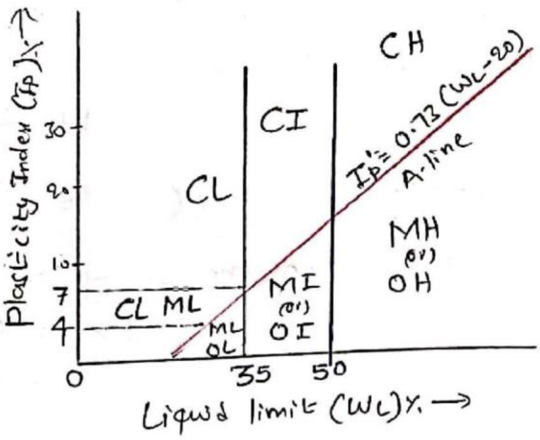
= 10–25 kN/m² (Very Soft) Clay)

= 400 kN/m² (Hard Clay)

Sensitivity	Clay Classification
1	Insensitive (Highly Consolidation)
2 - 4	Normal
4 - 8	Sensitive
8 - 16	Extra Sensitive
>16	Quick (Unstable for Construction)

Casagrande's plasticity chart (USCS) of fine grained Soils.

Soils is termed as fine grained soil, if more than 50% of soil sample passes Sieve No. 200 US (75μ)



Plasticity	W_L
Low plasticity (L)	< 35%
Intermediate plast (I)	35 - 50%
High plasticity (H)	> 50%

Original $(I_p) = W_L - W_p$.

A-line $(I_p') = 0.73(W_L - 20)$

a) $I_p > I_p'$ (Clay) b) $I_p < I_p'$ (Silt (or) Organic)

c) $(I_p = 4-7) > I_p'$ (CL & ML)

Group Index of a Soil (G.I)

[Used for pavement design in Highway construction]

$$GI = 0.2a + 0.005ac + 0.01bd$$

a = (% Soil passing 75μ sieve) − 35%. $\in [0-40]$

b = (% Soil passing 75μ sieve) −15%. $\in [0-40]$

c = W_L − 40%. $\in [0-20]$

d = I_p − 20%. $\in [0-20]$

G.I = 0 (Soil is Best for pavement)

GI = 20 (Soil is Poor for pavement)

[Soils are Devided into 7 Groups (A-1, A-2, A-3 . . . A-7)
A-1 ⇒ 2 sub groups
A-2 ⇒ 4 " "
A-7 ⇒ 2 " "

A-1, 2, 3 (Granular materials ≤ 35% passing 75μ)

A-4, 5, 6, 7 (Silty Clay materials > 35% passing 75μ)]

Determination of Water Content (W)

a) Oven drying method
Most accurate Method. Takes about 24 hours
→ Temperature should be maintain $105 \pm 5°C$

[Max Temperature
Clay ≯ 110°C
Soil + gypsum ≯ 80°C
organic (or) peat ≯ 60°C]

b) Sand - Bath method
Dryed or Heated over a kerosene Stove (about 1 hour)
→ Should not be used for containing gypsum or organic or peat.

c) Alcohol method
Dryed by Methylated Spirit &

$$W = \frac{M_2 - M_3}{M_3 - M_1} \times 100$$

M_1 = Mass of lid
M_2 = lid + wet Soil
M_3 = lid + Dry Soil

d) Calcium Carbide method
Very quickest method (about 5-10 minutes)
→ Mostly adopted in the Field work.

e) Pycnometer method
Quick method. Adopted when Sp.gr (G) is accurately known.
→ Capacity of bottle − 900ml & Top hole dia − 6mm.

f.) Torsional Balance method
Main parts (i) Infra-red lamp (ii) Torsion balance.
Size of Soil ≯ 2mm.

Determination of Specific gravity (G)

a) 50ml Density bottle Method

The most accurate method.

→ The standard method used in the laboratory.

→ Suitable for all type of soils

For Cohesionless soils (Distilled water is enough)

For Cohesive soils (kerosene is prefered)

$$G = \frac{\text{Dry mass of Soil}}{\text{Mass of water of equal volume}}$$

$$G = \frac{\text{Dry mass of soil} \times G_k}{\text{Mass of kerosine of equal volume}}$$

$G = Sp.Gr @ T_1°C$

$G_k = Sp.Gr$ of kerosene @ $T_1°C$.

G_k (at 27°c) = 0.773

G_w (at 27°c) = 0.9965

G_w (at 4°c) = 1.0

$$\therefore G(\text{at } T_2°C) = G(\text{at } T_1°C) \times \frac{G_w(\text{at } T_1°C)}{G_w(\text{at } T_2°C)}$$

i.e $\boxed{G \cdot G_w (\text{at } T_1°C) = G\, G_w (\text{at } T_2°C)}$

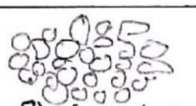
b) 500ml Flask (or) pycnometer Method

Suitable for Coarse grained Soils only.

→ Distilled water is preferred

Soil Structure

→ For Coarse grained Soils (> 0.02mm)



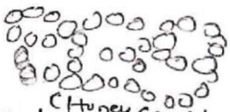
Single grained.

a) Single Grained Structure

Surface forces are too small. Hence settle by Gravitational forces only

So, having high void ratio @ loose state & low void ratio @ Dense state

→ For Fine grained soils (< 0.02mm)



(Honey Comb)

b) HoneyComb Structure

Surface forces & Gravitational forces are play an equally important role

→ The volume will not decrease due to Compaction, due to Surface forces.

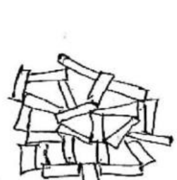
Hence most Suitable for Heavy loads having large areas.

→ The Structure will broken, it results decrease in volume due to vibrations.

Hence it is not suitable for Structures like pile driven foundations.

→ For Clay minerals (< 2μ)

Generally Clays are Flaky in shape. Hence the shape of the structure depend upon the Net electrical forces between adjacent clay particles.

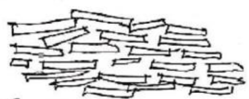


(Flocculent structure)

c) Flocculent structure

The Net Electrical forces between ajacent soil particles at the time of deposition are attraction forces. Hence they are oriented Edge - Edge (or) Edge - Face.

→ Hence these are having high void ratio. Having good drainage property.



(Dispersed Structure)

d) Dispersed (or) Oriented Structure

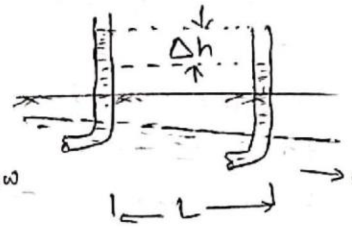
The Net Electrical forces between adjacent soil particles at the time of deposition are Repulsion. Hence they are oriented Face - Face only.

→ Hence these are having low void ratio.

Purely Impervious & becomes dense when compacted.

Permeability of Soils (By Darcy's law)

Is the property of soil mass which permits the flow of water through it's inter connected voids.



Hydraulic gradient $(i) = \frac{\Delta h}{L}$

Darcy's law

The rate of flow or Discharge (q) per unit time is proportional to hydraulic gradient for laminar flow

i.e $q \propto i \Rightarrow q = kAi$ (Actually $q = VA$).

$\therefore$ Velocity of flow or Average Discharge velocity

$$\boxed{v = ki}$$

where

k = permeability of soil (cm/sec) (or) Coefficient of permeability

Permeability (k) will depend on Void ratio (e)

$\Rightarrow$ For Gravel & Sand is highly permeable i.e $k > 10^{-1}$ cm/sec

$\Rightarrow$ For Stiff Clay is impermeable i.e $k < 10^{-7}$ cm/sec

Seepage (or) True (or) actual velocity

Since, Actually the flow takes through voids. Although from continuity equation Discharge through soil medium (VA) = Discharge through voids ($V_s A_v$)

$$\therefore \boxed{V_s = \frac{V}{n} = \frac{1+e}{e} V}$$

$\because V_s = \frac{A}{A_v} V = \left(\frac{1}{\left(\frac{A_v}{A}\right)} V\right) = \frac{1}{n} V$

i.e As void ratio (e) will increase, seepage velocity (V_s) will decrease $\because V_s = \left(1+\frac{1}{e}\right) V$

$\therefore$ Seepage velocity > Discharge velocity.

Coefficient of percolation

$$\boxed{k_p = \frac{k}{n}} \Rightarrow V_s = k_p i$$

$\because V_s = \frac{V}{n} = \frac{ki}{n} = k_p i$

Renold flow

Here characteristic length L = Average particle Diameter (D).

$$\therefore R_e = \frac{\rho V D}{\mu} = \frac{\rho_w V D}{\eta g}$$

$\left(\because\right.$ Here flow is water $\therefore \rho = \rho_w$

μ = viscosity (N-s/m²) $= \eta g$

η = viscosity (kg-s/m²) $\left.\right)$

As flow in soil, is said to be

a) Laminar flow ($Re \leq 1$) & b) Turbulent flow ($Re > 1$)

According to Darcy $\boxed{V = ki}$

According to Hough $\boxed{V = k(i)^{0.65}}$

Validity of Darcy's law

1. For laminar flow conditions only (i.e $Re \leq 1$)
   ($\because$ Laminar flow generally seen in clays, silts & fine sands)
2. For very low velocities ($< 10^{-6}$ cm/sec) also not valid.
3. As compared to other V & V_s are very low, Hence velocity head ignored.
   $\therefore$ Total head becomes = piezometric head (i.e. Pressure head + Datum head)

Poiseuille's law (Applied by comparing with Darcy's law)

According to him, permeability depends not only soil but also property of water,

$$\text{Permeability } (k) = \frac{\gamma_w}{\mu} \cdot C \frac{e^3}{1+e} D^2 \Rightarrow \boxed{k = \frac{\gamma_w}{\mu} k_o}$$

$\therefore \boxed{k \propto \frac{e^2 D^2}{\mu}}$

$\because$ Coefficient of absolute permeability $\boxed{k_o = C \frac{e^3}{1+e} D^2}$

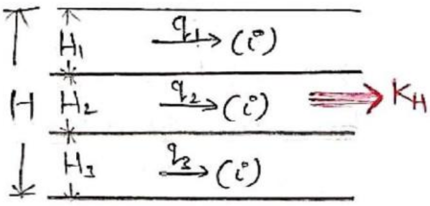
C = Constant $\simeq 100$ when D in (cm)

Units for k_o is mm², cm², m² (or) Darcy $\boxed{1 \text{ Darcy} = 0.987 \times 10^{-8} \text{ cm}^2}$

$k \propto$ Tempe

Permeability of Stratified Soil Deposits/Layers. (i.e Anisotropic)

When a Soil deposit Consists of a number Horizontal layers having different permeabilities. Hence The Average value of permeability can be obtained seperately



a) For Horizontal flow

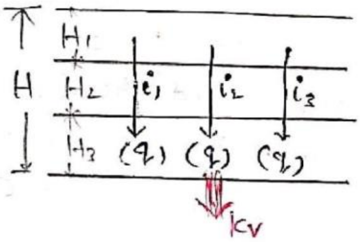
Total flow $q = q_1 + q_2 + q_3 + \cdots \Rightarrow AV_H = A_1V_1 + A_2V_2 + A_3V_3 \cdots$

Consider Under width $\Rightarrow (A = 1 \times H)$

$H k_H i = H_1 k_1 i + H_2 k_2 i + H_3 k_3 i + \cdots$

$\therefore$ The average permeability for horizontal flow

$$k_H = \left[\frac{k_1H_1 + k_2H_2 + \cdots}{H}\right] = \frac{\Sigma kH}{H}$$



b) For Vertical flow

each layer has same flow (i.e $q = q_1 = q_2 = q_3 \cdots$ & $A_1 = A_2 = A_3 \cdots$)

$\therefore k_V i = k_1 i_1 = k_2 i_2 = k_3 i_3 = \cdots$ & $V = V_1 = V_2 = \cdots \Rightarrow k_V \frac{h}{H} = k_1 \frac{h_1}{H_1} = k_2 \frac{h_2}{H_2} \cdots$

The total head loss accross the layers $(h = h_1 + h_2 + h_3 + \cdots$

$\therefore$

$$h = \left(k_V \frac{h}{H}\right)\frac{H_1}{k_1} + \left(k_V \frac{h}{H}\right)\frac{H_2}{k_2} + \cdots$$

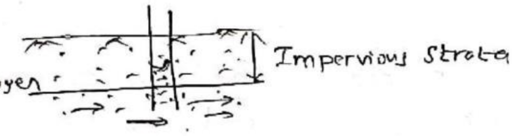
The average permeability for vertical flow

$$k_V = \left[\frac{H}{\frac{H_1}{k_1} + \frac{H_2}{k_2} + \cdots}\right] = \frac{H}{\Sigma \frac{H}{k}}$$

Measurement of permeability

1. Laboratory Tests
   a) Constant head flow
   → Is Recommended for Coarse grained Soils only.
   b) Falling Head flow
   → Is Recommended for Fine grained Soils.
2. Field Tests
   a) Unconfined Flow pumping test
   Test Conducted Just below the G.L

   b) Confined Flow pumping test
   Test Conducted below the impervious layer



Factors effecting permeability

As we know $k = V \times \frac{L}{\Delta h}$ (or) $k \propto \frac{e^2 D^2}{\mu}$

Permeability increases when void ratio (e) increases

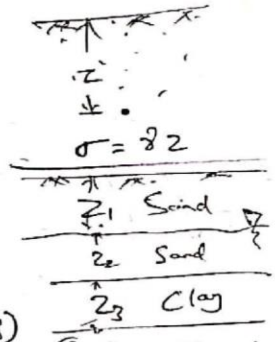
→ Angular Shape Soils are less void ratio than Spherical shape

→ Diameter (i.e size of soil) increases it leads to increase void ratio.

→ Honey Comb & Floculated Structure void ratio is more, while Single grained & Dispersed Structure void ratio is less.

Permeability decreases when Viscosity (μ) increases

Stresses in the Ground



$\sigma = (\gamma z_1)_{Sand} + (\gamma_{sat} z_2)_{sand} + (\gamma_{sat} z_3)_{clay}$

Total Stress : (σ)

The total downward verticle stress acting at a point below the ground surface, is due to weight of everything lies above that point.

$$\sigma = \gamma z \quad (\text{Single layer})$$

$$= \gamma_1 z_1 + \gamma_2 z_2 + \gamma_3 z_3 + \cdots \quad (\text{Having differ layers})$$

Porewater pressure (u) (or) ~~Seepage pressure~~

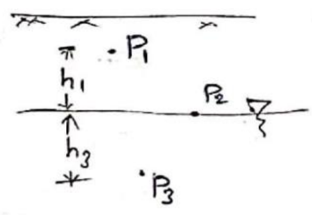
The upward buoyant force due to water only.

Neutral Stress $u = \gamma_w h$ (point is below WT)

$= 0$ (point is on the WT)

$= -\gamma_w h$ (point lies above the WT)

Hence h = vertical distance b/n point - water table



$u(@P_1) = -h_1 \gamma_w$

$u(@P_2) = 0$

$u(@P_3) = +h_3 \gamma_w$

Effective Stress ($\bar{\sigma}$)

Is a force that keep/balance a Collection of particles rigid/stable.

$$\bar{\sigma} = \sigma - u$$

Thus

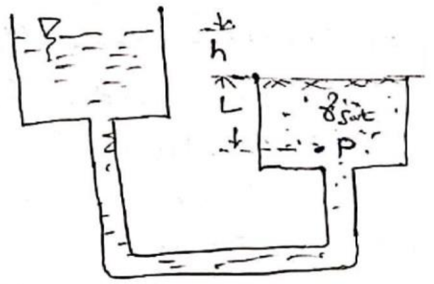
→ Flow occurs in downward direction, $\bar{\sigma}$ will increse ($\because$ WT decreases)

Flow occurs in upword direction, $\bar{\sigma}$ will decrese ($\because$ WT increases) & becomes zero @ quick Sand Condition

* Quick Sand Condition

The point at where Effective stress is zero (i.e $\bar{\sigma} = 0 \Rightarrow \sigma = u$)

i.e At where Seepage pressure (u) = Total pressure (σ)



$$(L+h)\gamma_w = L\gamma_{sat}$$

$$h\gamma_w = L(\gamma_{sat} - \gamma_w) = L(\gamma_{sub})$$

$$h\gamma_w = L\frac{G-1}{1+e}\gamma_w \Rightarrow \frac{h}{L} = \frac{G-1}{1+e}$$

$\therefore$ Critical hydraulic gradient $i = \boxed{i_c = \frac{G-1}{1+e}}$

Consider "Coarse silt or Fine sand" having $G = 2.67$ & $e = 0.67$

$\Rightarrow i_c = 1$

Seepage in Soils

Seepage means the slow escape of a fluid through porous material.

The energy developed b/n the water & soil is called "Seepage pressure".

Hence, The flow equation for anisotropic material (i.e having different layers)

$$k_x \frac{\partial^2 h}{\partial x^2} + k_z \frac{\partial^2 h}{\partial z^2} = 0 \qquad \left(\because \frac{\partial V_x}{\partial x} = \frac{\partial}{\partial x}\left(k_x \frac{\partial h}{\partial x}\right)\right)$$

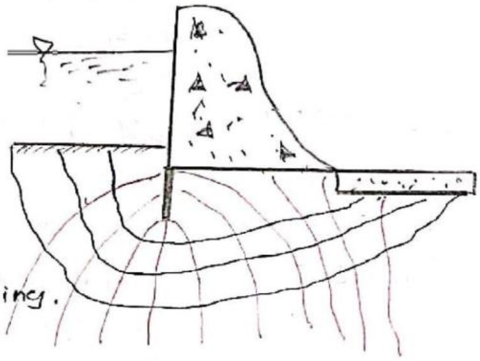
The flow equation for Isotropic (i.e $k_x = k_z$) material (having same layer)

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial z^2} = 0 \quad (\text{The Laplace Equation})$$

The general solution of Laplace equation yields two sets of curves orthogonal to each other. i.e one set – Flow lines

other set – Equipotential lines.

Laplance lines



Flow (or) Stream (or) Seepage lines

The lines indicate the direction of Seepage.

→ Flow lines always Start from U/S Side and ends to D/S Side.

→ Considering the flow is Laminar ⇒ Discharge is Same for all flow lines.

Equipotential lines

The line joining the points of equal hydraulic potentials (i-e Equal piezometric readings).

— Flow lines
— Equipotential lines

→ The total head (i-e $\frac{P}{w}+z$) is equal for all points in a potential line

→ The head loss/different b/n two adjacent potential lines is Called "potential drop", & lines spaced by Equal drops.

Notes:

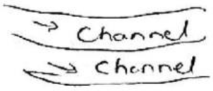
→ The flow lines intersect Equipotential lines at Right angles (i-e 90°)

→ The lines drawn in Such a way that, the flow elements ≃ Square.

→ The lines are drawn by Trail and Error.

→ Two flow lines Can never meet, Simillarly, two equipotential lines Can never meet.

a) Flow Channel - The portion b/n two adjacent flow lines



b) Field - The portion enclores b/n two Successive flow & equipotential lines

c) Flow net - The System of all flow & equipotential lines

Seepage loss (m^3/day/m)

The total quantity of water escapes from U/S Side to D/S Side by Seepage,

a) Quantity of Seepage loss for Isotropic Soil

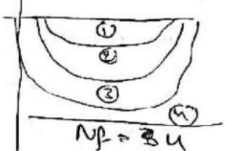
$$q = Kh\frac{N_f}{N_d}$$

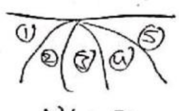
[h = Total Head loss = Total head in U/S − Total head in D/S

b) Quantity of Seepage loss for Anisotropic Soil

$$q = \sqrt{K_x K_z}\, h\frac{N_f}{N_d}$$

N_f = No. of Flow Channels
N_d = No. of potential drops



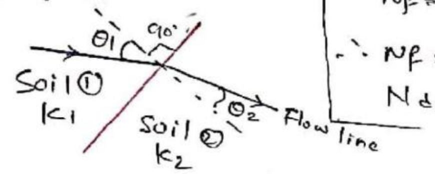


∴ N_f = Flow lines + 1
N_d = potential lines + 1

Deflection of flow lines at Interface of Dissimilar Soils

The two Soils may be Non-Homogeneous (or) Anisotropic Soils

The fourth unknown Can be Calculated by (i-e k_1, k_2, θ_1 or θ_2)



$$\frac{K_1}{K_2} = \frac{\tan\theta_1}{\tan\theta_2}$$

Factor of Safety $= \frac{i_{cr}}{i_{max}}$ or $\frac{\text{Critical gradient}}{\text{Exit gradient}}$ (∵ Fos > 1, i-e $i_{cr} > i_{max}$)

∵ i_{cr} = Critical hydraulic gradient $= \frac{G-1}{1+e}$

i_{max} = Maximum hydraulic Gradient $= \frac{\Delta h}{L_{min}}$

Generally i_{max} will occur @ Exit (i-e D/S End)
Exit Gradient $i_{max} = \boxed{i_e = \frac{i_{cr}}{Fos}}$
also Called Down Stream (or) Tail water gradient.

Consolidation of Soils

Compaction - The process by which the soil particles are artificially rearranged and packed together by mechanical means. To increase the unit wt of soil.

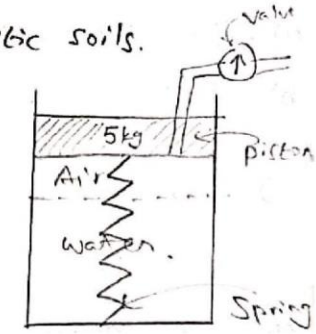
Compression - Decrease in volume (Due to expulsion of air from voids) of soil under stress (i.e by compaction)

Compressibility - The property of soil, having the chance of compression

Consolidation - The compression of soil under steady pressure (i.e weight imposed by structure or earth filling).
→ which is due to Expulsion of water from the pores.

→ The volume change caused by compression and expulsion of air is called Initial Compression. (The total air cannot be removed, due to skelton structure of soil)
→ And then, the volume change caused by expulsion of pore water is called primary consolidation (The expelled water may be transfer to the upper/adjacent soil may have voids)
→ And then, if further volume change takes place due to readjustment of soil molecules then it is called Secondary Consolidation it may be 10-20% of total settlement of highly plastic soils.

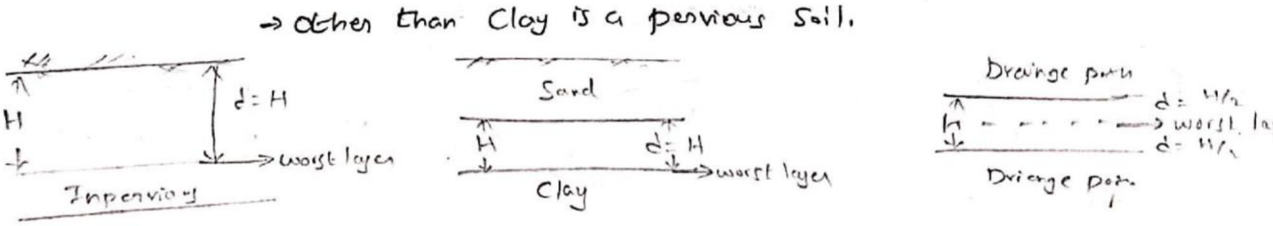
Comparision of Consolidation with Spring Analogy



Consider the soil composed of = Air + water + soil
1st addition of load, 1st escaping agent is air. After air will escape through valve, the water will start escaping.
Comparision
Spring → The skelton of soil solids
Piston → The soil surface (upper) or External load
Valve → Voids in the soil
Water → pore water in the soil
Q_{outlet} → k (i.e Coefficient of permeability)

Drainage path

During consolidation water escapes from the soil to the surface or to a permeable sub-surface layer above or below.
Drainage path = ~~Distance~~ Shortest distance from worst pore water layer to the near pervious strata/open surface.
→ other than Clay is a pervious soil.



∴ Drainage path (d) = H (when, Drainage path is on one side only)
$= \frac{H}{2}$ (when, Drainage path is on Both Sides)

$\bar{\sigma}_0$ = Effective stress, before consolidation $= \sigma - u$

e_0 = Initial void ratio before consolidation (or) At the start of consolidation.

$\Delta\bar{\sigma}$ = Increase in stress (Although, it is an increase in Effective stress)

$\bar{\sigma}_0 + \Delta\bar{\sigma}$ = Total Effective stress after $\Delta\bar{\sigma}$ $= \sigma - u + \Delta\bar{\sigma}$

e_f = Final void ratio after Final settlement

ΔH = Final settlement due to $\Delta\bar{\sigma}$

H = Thickness of the layer effecting consolidation & it is fully saturated.

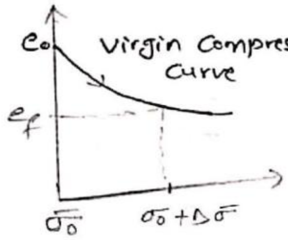
(The total consolidation settlement depends upon increase in Eff. stress ($\Delta\bar{\sigma}$))

Normally Consolidated Soil

If the soil is loaded by $\Delta\bar{\sigma}$ first time in the history at present, then the soil will be in Normally consolidated.

Co-efficient of Compressibility (a_v)

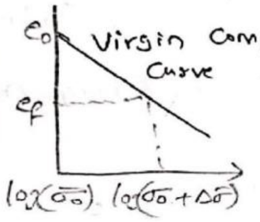
The slope of Effective stress Vs void ratio on Arthmatic scale



$$a_v = \frac{dy}{dx} = \frac{-\Delta e}{\Delta\bar{\sigma}}$$

Co-efficient of Compression (C_c)

The slope of Eff. stress Vs void ratio is taken on log scale. $C_c = 0.1 - 0.8$



$$C_c = \frac{dy}{dx} = \frac{-\Delta e}{\log(\bar{\sigma}_0 + \Delta\bar{\sigma}) - \log\bar{\sigma}_0}$$

$$C_c = \frac{-\Delta e}{\log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)}$$

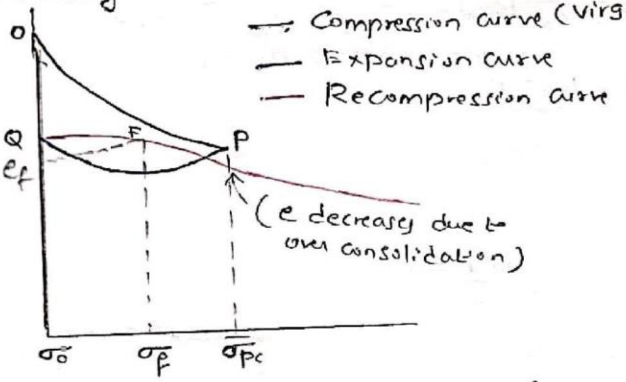
-ve sign indicates void ratio will be decreased due to consolidation.

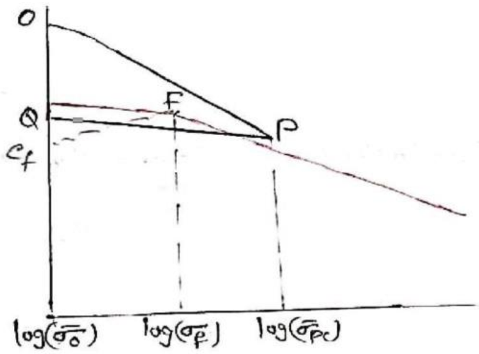
Over Consolidated (or) Pre-consolidated Soil.

If the soil is loaded by ($\Delta\bar{\sigma}$) at present, which is (or may be more than $\Delta\bar{\sigma}$) already experienced by the soil in past, Then the soil is said to be over consolidated (or) pre consolidated soil. (i.e The soil is already consolidated in past upto e_{pc})

Pre-consolidated Stress ($\bar{\sigma}_{pc}$)

The maximum effective stress experienced by the soil up to present. (Generally it is equal to (i.e observed) The final effective stress in Normally consolidated soil).





When the stress ($\bar{\sigma}_{pc}$) is removed from a consolidated clay, the soil will Expand (Results expansion curve) due to surface forces to a small volume. Again will applying stress ($\bar{\sigma} < \bar{\sigma}_{pc}$), then the soil will recompresses (Results Recompression curve). It is observed that "The consolidation is more in over consolidation process".

Expansion (or) Swelling Index

$$C_e = (+)\ \frac{e_{pc} - e_0}{\log\left(\frac{\bar{\sigma} + \Delta\bar{\sigma}}{\bar{\sigma}}\right)}$$

Recompression Index

$$C_r = (-)\ \frac{e - e_f}{\log\left(\frac{\bar{\sigma} + \Delta\bar{\sigma}}{\bar{\sigma}}\right)}$$

$\because \Delta\bar{\sigma}$ in all process

$$\boxed{C_c > C_r > C_e}$$

Notes:

→ The current stress ($\bar{\sigma}$) = $\bar{\sigma}_{pc}$ it is Normally Consolidated

→ The current stress ($\bar{\sigma}$) < $\bar{\sigma}_{pc}$ it is Over Consolidated

∵ $\bar{\sigma}$ will never > $\bar{\sigma}_{pc}$
∵ $\bar{\sigma}_{pc}$ is the Maxxx Eff stress

Over Consolidation ratio (OCR)

$$OCR = \frac{\text{Pre consolidated stress}}{\text{Current effective stress}} = \frac{\bar{\sigma}_{pc}}{\bar{\sigma}}$$

$= 1$ (For Normally Consolidated Soil)

> 1 (for over consolidated soil)

Formula

$$\text{Degree of Consolidation } (U) = \frac{\text{Settlement at any stage}}{\text{Final Settlement}} = \frac{\Delta h}{\Delta H}$$

$$= \frac{\bar{\sigma}_t - \bar{\sigma}_0}{\bar{\sigma}_f - \bar{\sigma}_0} = \frac{e_0 - e_t}{e_0 - e_f}$$

$$\text{Coefficient of Compressibility } (a_v) = \frac{\Delta e}{\Delta \bar{\sigma}} \quad (m^2/N)$$

$$\text{Coefficient of Volume Change } (m_v) = \frac{a_v}{1+e_0} = \frac{\Delta e}{1+e_0} \cdot \frac{1}{\Delta \bar{\sigma}} \quad (m^2/N)$$

$$\text{Coefficient of Permeability } (k) = C_v m_v \gamma_w \quad (cm/sec)$$

$$\text{Coefficient of Consolidation } (C_v) = \frac{k}{m_v \gamma_w} = \frac{T_v d^2}{t} \quad (m^2/year)$$

$$\text{Time Factor } (T_v) = \frac{C_v t}{d^2}$$

or) $T_v = \frac{\pi}{4} U^2$; $U < 60\%$

$= 1.781 - 0.933 \log_{10}(100 - U\%)$; $U > 60\%$

$= 0.28$; $U = 60\%$

$$\text{Final settlement } (\Delta H) = \frac{C_c H}{1+e_0} \log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right) = \frac{\Delta e}{1+e_0} H$$

$$= m_v \Delta\bar{\sigma} H.$$

Coefficient of Compression or Compression Index

$$C_c = \frac{\Delta e}{\log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)}$$

$= 0.009(W_L - 10)$ (Undisturbed Clay of Medium Sensitivity)

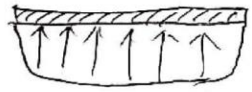
$= 0.007(W_L - 10)$ (Remoulded Soils of Med-low Sensitivity)

$= 0.40(e_0 - 0.25)$

Constant pressure on the base of foundation

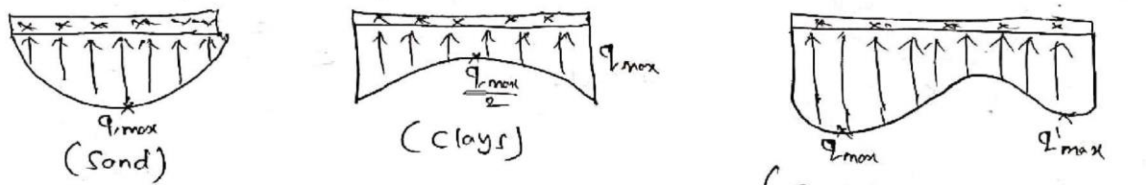
1. Flexible foundation

Constant pressure for all type of soils is nearly uniform.
(The settlement may be unequal for diff soils)



2) Rigid Foundation

The settlement is uniform (but pressure distribution depends on type of soil)



(Sand) (Clays) (Sand/Cohesive less Soils)

Settlement analysis (As per BIS)

Total settlement (S) = Immediate settlement (S_i) + primary consolidation settlement (S_c) + Secondary consolidation settlement (S_s)

Type of Foundation	Permissible Total Settlement
1. Isolated foundation on Clay	65 mm
2. Isolated foundation on Sand	40 mm
3. Raft foundation on Clay	65 - 100 mm
4. Raft foundation on Sand	40 - 65 mm

Compaction of Soils

Compaction is the application of mechanical Energy to a soil so as to rearrange it's particles and reduce the void ratio.

→ Compaction increases the dry density of the soil, in turn increases it's shear strength & bearing capacity.

→ Compaction reduces the permeability of the soil

→ Compaction decreases the settlement of the soil (in future)

Tamping - By continuous impact loads, Kneading - By continuous pressing by Hydraulics

Laboratory Compaction Tests

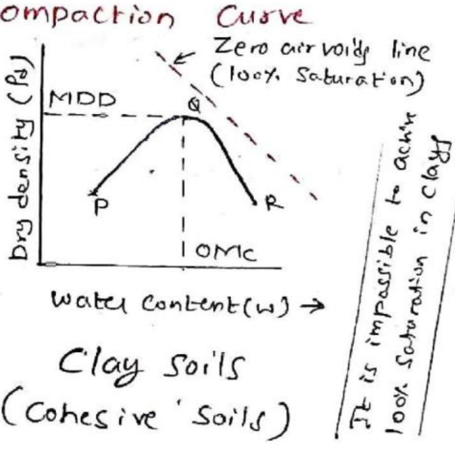
a) Indian Standard Light Compaction Test (Standard proctor test)

1. Mould Capacity - 1000 cm^3, 2. Hammer wt - 2.6 kg, Height of fall - 310 mm
3. No. of layers - 3, No. of blows/layer = 25

b) Indian Standard Heavy Compaction Test (Modified proctor test)

1. Mould Capacity - 1000 cm^3, 2. Hammer wt - 4.9 kg, Height of fall - 450 mm
3. No. of layers - 5, No. of blows/layer - 25

Compaction Curve



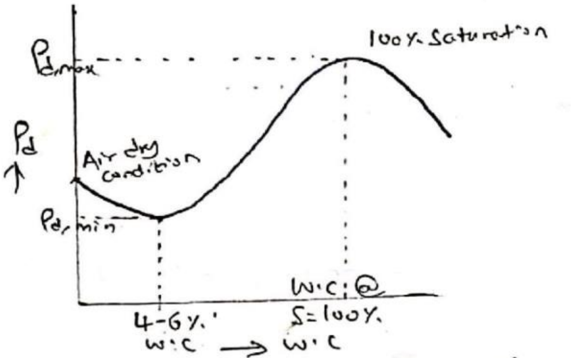
Clay Soils
(Cohesive Soils)

→ Compaction starts from point (P)

→ PQ curve - water facilitate easier movement of particles and their closure packing.

→ At Q. It is the Maximum dry Density (MDD) $\rho_{d,max}$ at optimum moisture content (OMC) (favorable)

→ QR curve - Further increase in W.C, the moisture occupies voids. (These voids may be filled by adjacent soil particles). Hence it results decrease in Dry density.



(Sandy soils) Cohesionless

→ Campoction Stordy from Dry Condition (i.e. S=0)
→ Due to increase in w:c upto (4-6%) the Sand become bulking.
(it results increase volume ⇒ P_d decreases)
→ In Sandy 100% Saturation may be possible.

Relative Density

$$(I_D) = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100$$

Relative Density (I_D)	Classification
<10%	Very loose
15-35%	Loose
35-65%	Medium
65-85%	Dense
>85%	Very dense

SHEAR STRENGTH

is the Resistence to Shearing stress just before failure.

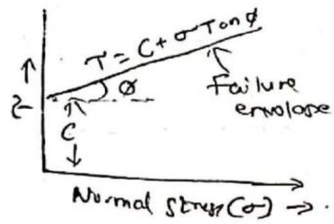
A Soil may derive it's Shear Strength from
1. Interlocking of particle
2. Friction Resistance
3. Co-hesion & Adhesion between molecules.

→ For granular soils derive its shear strength from interlocking & friction. Whereas Clays derive it's shear strength from Cohesion.

→ In granular soil, shear strength increases with the depth, whereas In pure Clays, shear strength approximately constant with the depth

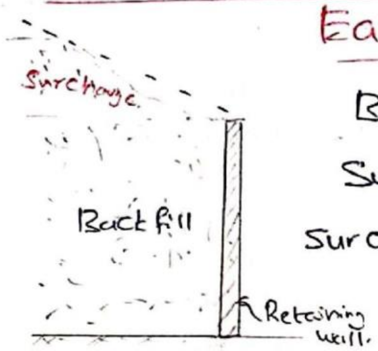
According to Colomb

Shearing strength $(\tau) = C + \sigma \tan\phi$



where
- C = Cohesion (For granular soils $C = 0$)
- σ = Normal stress
- ϕ = Angle of internal friction or Angle of Shearing Resistance.

Earth pressure (Theory of Elasticity)



Backfill - Material retained or supported by structure

Surcharge - Excess backfill on the top of the structure

Surcharge angle - Inclination of surcharge to the Horizontal.

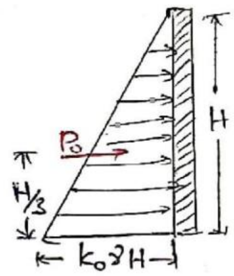
Earth pressure: The pressure exerted by the Backfill on the Retaining wall. Lateral earth pressure is a function of
→ Type and amount of wall movement
→ Shear strength parameters of Soil (i.e. C, ϕ)
→ Unit weight of Soil
→ Drainage Condition of the backfill.

i) Earth pressure at rest (P_o)

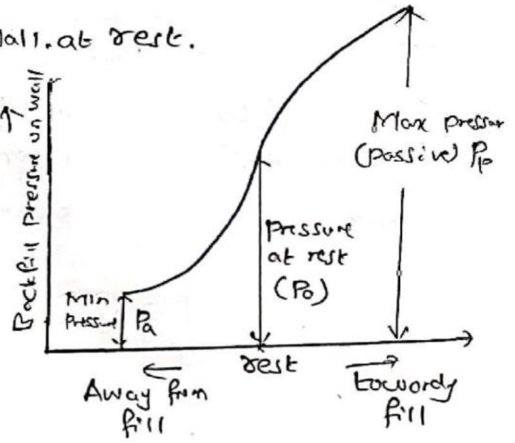
The pressure exerted by the backfill on the wall at rest.

Since, wall Should be designed to resist the Backfill pressure

Type of Soil	k_o
Dense Sand	0.4 - 0.5
Loose Sand	0.45 - 0.55
Normally Consolidated Clay	0.5 - 0.6
over Consolidated Clay	1 - 4



$P_o = \frac{1}{2}(k_o\gamma H)H$



Earth pressure at Rest (P_o) $= \frac{1}{2} k_o \gamma H^2$ (kN/m)

where

k_o = Coefficient of Earth pressure at rest

$= \frac{\mu}{1-\mu}$ (Any type of Soil) $\left(\because \text{Lateral Strain } \epsilon_x = 0 \Rightarrow -\frac{\sigma_x}{E} + \mu\frac{\sigma_y}{E} + \mu\frac{\sigma_z}{E} = 0 \right.$ $\left. \Rightarrow (1-\mu)\sigma_h = \mu\sigma_v \;\; (\because \sigma_x = \sigma_y = \sigma_h \;\&\; \sigma_z = \sigma_v)\right)$

$= 1 - \sin\phi$ (For Normally Consolidated Clay) $\left(\because \text{OCR} = \text{Over Consolidated Ratio} = \bar{\sigma}_{pc}/\bar{\sigma}\right)$

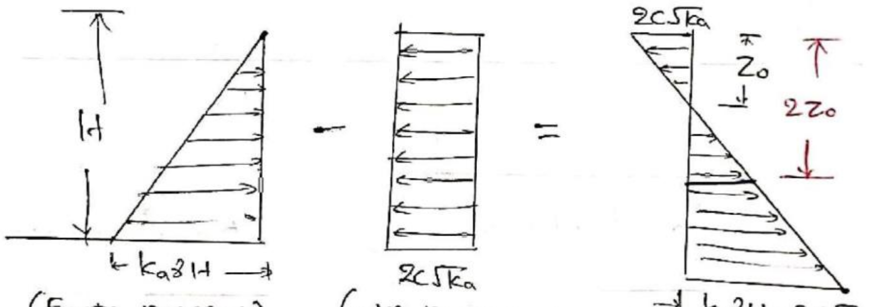
$= (1-\sin\phi)\times\sqrt{OCR}$ (For over Consolidated Clay)

*) Active Earth Pressure (P_a)

The minimum pressure exerted by the back fill on the wall.

→ Since, Retaining wall tends to move away from the backfill. (Due to Hinge action (or) Other reasons)

→ Hence, the total earth pressure may not fully applied on the wall.



(Earth pressure) (-ve pressure due to cohesion) (Resultant pressure on the wall)

Resultant Rankine's

Active Earth pressure (P_a) $= \frac{1}{2}k_a\gamma H^2 - 2ck_aH$ (kN/m)

where

k_a = Co-efficient of active earth pressure

$= \frac{1-\sin\phi}{1+\sin\phi} = \cot^2(45 + \frac{\phi}{2})$

Note:

→ For Cohesiveless soils Consider ($c = 0$)

→ -ve earth pressure will be developed at the top of the wall $= -2c\sqrt{k_a}$

→ The pressure will zero at $(Z_o) = \frac{2c}{\gamma\sqrt{k_a}}$ $\left(\because k_a\gamma Z_o - 2c\sqrt{k_a} = 0\right)$

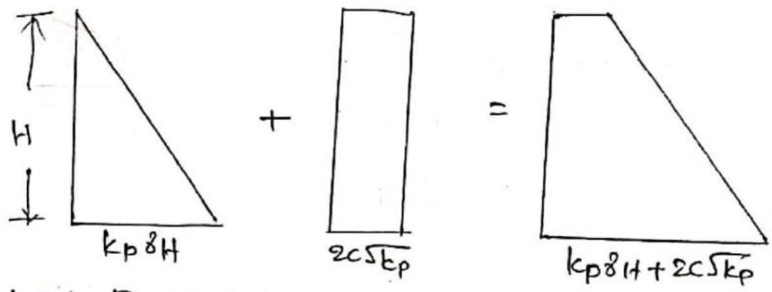
→ Cohesive Clay Soil Can withstand $(2Z_o) = \frac{4c}{\gamma\sqrt{k_a}}$ depth without any lateral support.

c) Passive Earth Pressure (P_p)

The maximum pressure exerted by the backfill on the wall.

→ Since, Retaining wall tends to move towards the backfill.
(Due to lifting action of wind force or any other reason)

→ Hence, the earth pressure full applied on the wall.



The Resultant total Rankine's

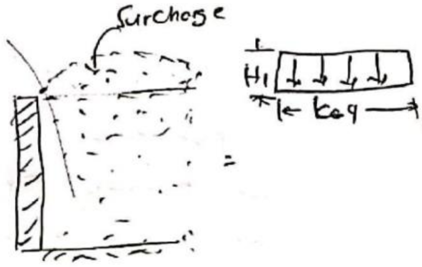
Passive earth pressure $(P_p) = \frac{1}{2} k_p \gamma H^2 + 2c\sqrt{k_p}\, H$ (kN/m)

where

k_p = Coefficient of passive earth pressure

$$= \frac{1+\sin\phi}{1-\sin\phi} = \tan^2\left(45 + \frac{\phi}{2}\right)$$

Surcharge weight (q)



The Resultant earth pressure due to Surcharge

$$p' = p + k q H_1 \quad (kN/m)$$

$\left[\begin{array}{l} \because p = P_o \text{ or } P_a \text{ or } P_p \\ k = k_o \text{ or } k_a \text{ or } k_p \end{array}\right.$ & H' = Surcharge height, q = Uniform surcharge load (kN/m²)

Note: $\boxed{k_a \times k_p = 1}$ & $\boxed{k_a < k_o < k_p}$ & $\boxed{P_a < P_o < P_p}$

Foundations

- Shallow foundations
  - Footings
    - Spread footing
      - Isolated or pad footing
      - Strip footing
    - Strap footing
    - Combined footing
  - Raft foundations
- Deep foundations
  - Pile foundation
  - Pier foundation
  - Well foundations

1. Shallow foundation

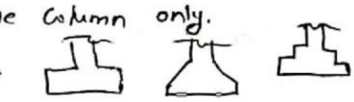
Depth of the foundation ≤ width of foundation ($D_f \le B$)

a) Spread footings

which spread the superimposed load of wall or column over a large area.

→ Is the one which supports either one wall (or) one column only.

→ These may be Rectangular, Circular, Isolated (or) Stepped



(i) Strip footing - Is the spread footing for continuous wall

(ii) Isolated (or) pad footing - " " for column

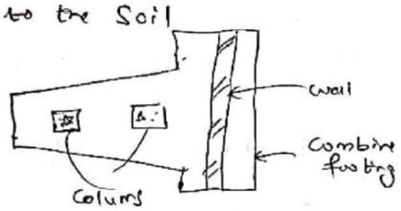
b) Strap footing

If the independent footings of two columns are connected by a Beam, it is called Strap footing.

→ May be used where distance b/n columns is so great.

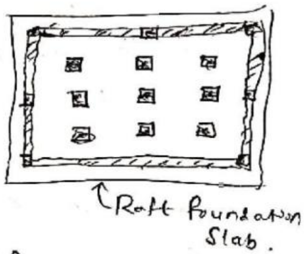
→ They transfer loads from column to column but not to the soil

c) Combined footing.



A spread footing which supports two or more columns/walls is termed as Combined footing.

d) Mat (or) Raft foundation



Is a Combined footing that covers the entire area beneath a structure and supports all the walls & columns.

→ To reduce settlements for highly compressible soils.

2. Deep foundations

Depth of foundation > width of foundation ($D_f > B$)

a) pile foundation

The deep vertical members, may be timber, concrete or steel.

(i) End bearing pile - Driven upto Natural hard strata (Soft soils)

(ii) Friction piles - Supported by skin friction developed by soil. (granular soil)

(iii) Compaction piles - Used to dense the sand. (Due to volume of soil occupied by compaction piles)
→ They do not carry any loads.

b) pier foundation

It consists of a cylindrical column of large diameter to support and transfer large super-imposed loads to the firm strata.

(i) Masonry (or) Concrete pier

(ii) Drilled Caissons (Steel caissons filled with concrete (or) concrete + Rolled steel)

→ Adopted at where driving of piles is difficult (like decomposed rocky soil)

c) Well caissons (or) foundations

These are box like structure - Circular (or) Rectangular, filled with sand.

→ They much large in diameter than piers.

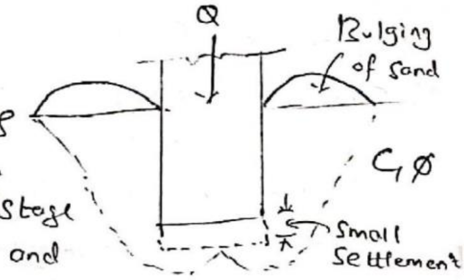
→ Adopted in heavy structures, like
Bridge piers and abutments in Rivers, lakes ..etc.

Types of Shear failure

a) General Shear failure

In Case of medium to dense Sandy and Stiff Clays Shear failure occurs without excessive Settlement.

At the time of failure Soil reaches into plastic stage and due to Shear, foundation may get tilted and adjacent to the foundation. Hence bulging of soil will take place.

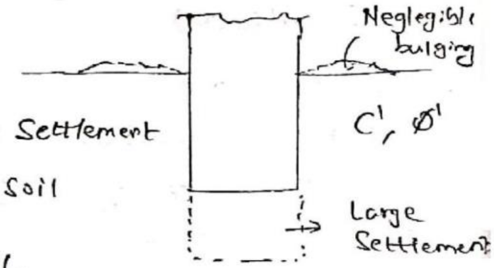


b) Local Shear failure

In case of Loose Sandy & Soft Clays; Large Settlement may occur below the foundation before the soil reaches into plastic stage.

→ There may be little or No-bulging of the Soil.

→ When Soil fails in Local Shear, Modified values of Cohesion (c) & Angle of friction (ϕ) should be used to Calculate bearing Capacity.



Modified values are

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

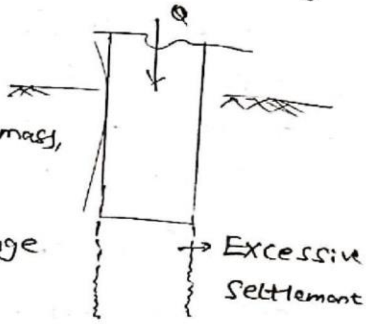
$\tan\phi' = \frac{2}{3}\tan\phi$

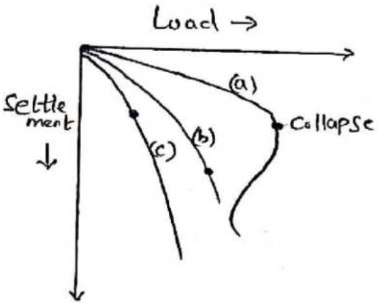
c) Punching Shear failure

In case of Very loose Soil in deep footing and pile the soil get Sheared from adjacent Soil mass,

→ hence adjacent soil is not stressed.

→ Excessive Settlements are reaching without Change in Characteristics of adjacent Soil.





(Result of plate Load Test)

No.	Parameter	General Shear failure	Local Shear failure
1	Friction angle (ϕ)	$>36°$	$<28°$
2.	Void Ratio (e)	<0.55	>0.75
3.	Relative Density (I_D)	$>70\%$	$<20\%$
4.	Unconfined Compressive Strength	$>100\ kN/m^2$	$<80\ kN/m^2$
5.	Shear Strength at Failure	$<5\%$	$>15\%$

Bearing Capacity (kN/m^2)

1. Ultimate or Gross bearing Capacity (q_u)

q_u = weight of (Super Structure + footing + Soil fill over the footing)

Net ultimate bearing capacity (q_{nu})

q_{nu} = weight of super structure only = q_u − initial effective over burden pressure.

$= q_u - \gamma D$ (∵ D = Depth of foundation below G.L, γ = Bulk unit wt of Soil & foundation) (∵ γ(foundation) is almost equal to γ(Soil))

2. Net Safe Bearing Capacity of Soil (q_{ns})

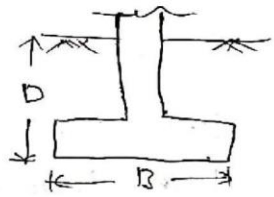
$$q_{ns} = \frac{q_{nu}}{F} = \frac{q_u - \gamma D}{F}$$ (∵ F = factor of safety (FoS))

Safe bearing capacity or safe load

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

[Maximum pressure which the soil can carry safely without risk of failure ∴ $q_s = q_{ns}$ + Effective over burden pressure]

1. General Shear failure ($\phi > 36°$)



a) Strip footings

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

b) Square foundations

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

c) Circular foundations

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

N_ϕ = Influence value $= \tan^2(45 + \phi/2)$

$N_q = N_\phi e^{\pi \tan\phi}$

$N_c = \cot\phi (N_q - 1)$ for ($\phi > 0$)

$= 5.7$ for ($\phi = 0$)

$N_\gamma = 1.8 \tan\phi (N_q - 1)$

d) Rectangular foundations

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

2. Local Shear failure ($\phi < 28°$)

Repalace $c' = \frac{2}{3}c$

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

N_c', N_q', N_γ' = Bearing capacity factory (function of ϕ')

(eg: $q_u = c'N_c' + \gamma D N_q' + \frac{1}{2}\gamma B N_\gamma'$)

3. Special Cases:

a) For Cohesionless soils ($c = 0$ & $N_c = 0$)

b) For Cohesive soils ($\phi = 0$ & $N_c = 5.7$, $N_q = 1$, $N_\gamma = 0$)

(∵ $q_{nu} = 5.7c$ (strip) $= 7.4c$ (square & circular))

4. Effect of water table

eg: $q_u = cN_c + (\gamma D N_q)R_{w1} + \left(\frac{1}{2}\gamma_{sat} B N_\gamma\right)R_{w2}$ (Use $\gamma = \gamma_{sat}$ in 3rd term (when WT ≤ D) $= \gamma$ (when WT > D))

where

R_{w1} = Reduction factor for water table above Base level of foundation

$= 0.5\left(1 + \frac{Z_{w1}}{D}\right)$ → Co-efficient for N_q

R_{w2} = Reduction factor for water table below Base level of foundation

where $= 0.5\left(1 + \frac{Z_{w2}}{B}\right)$ → Coefficient for N_γ

Z_{w1} = Distance from G.L − W.T (when W.T is above foundation)

$= D$ (when W.T is below foundation or at)

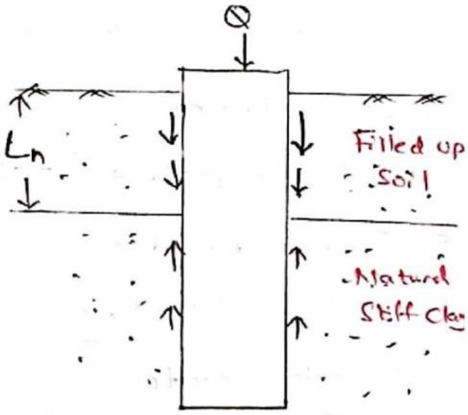
Z_{w2} = Distance from F.L − W.T (when W.T is below foundation)

$= 0$ (when W.T is above foundation (or) at)

(∵ G.L = Ground level, F.L = Foundation base level & W.T = water table level)

Negative Skin friction

When the unconsolidated filled up soil layer surrounding a portion of the pile shaft settles. A downward drag occur in the pile. This drag is called Negative Skin friction.



↓ - Negative skin friction

↑ - Friction resistance

→ -ve skin friction developed when the surrounding fill material is loose cohesionless soil (or) Soft Clay.

Negative skin friction for single pile.

$$F_n = (P L_n)\ \tau \quad \text{(Cohesive soil)}$$

$$= \frac{1}{2} P L_n^2 k \tan\delta \quad \text{(Cohesionless soil)}$$

P = Perimeter of pile

L_n = Length of pile in fill soil

τ = Shear strength of fill

k = Earth pressure Coefficient

δ = Angle of wall friction