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SOIL

Soil Mechanics

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CHAPTER - 1 (INTRODUCTION)

Properties of soil

→ Karl Terzghi is father of soil mechanics.

→ The process of formation of soil is termed as pedogenesis. The soil is formed due to weathering of rocks which may be carried out with physically or chemically. Physical agencies involved in weatheringof rocks include running water, wind, ice, gravity etc. and chemical agencies involved in weathering of rocks include oxidation, reduction, carbonation etc

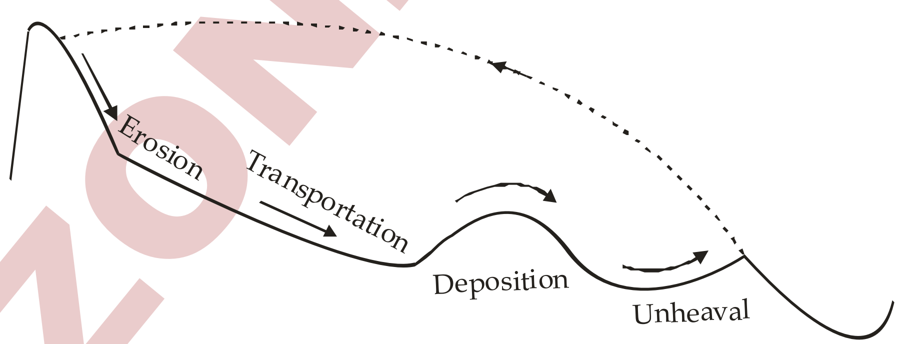
→ If the weathered rock material is retained over parent rock, soil is termed as residual soil & if it is transported then it is called transported soil.

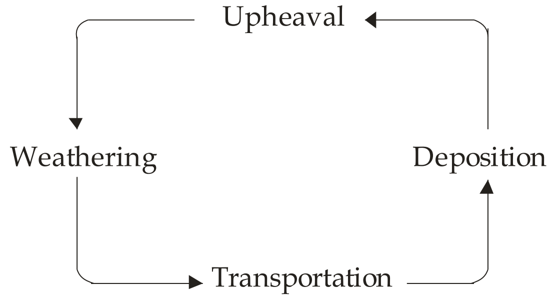
→ Thickness of transported soil > Residual soil

Geological Cycle of Formation of Soil are

Soils are formed by weathering of rocks due to mechanical disintegration or chemical decomposition. When a rock surface gets exposed to atmosphere for an appreciable time, it disintegrtates or decomposes into small particles and thus the soils are formed.

Soil may be considered as an incidental material obtained from the geologic cycle which goes on continuously in nature. The geologic cycle consists of erosion, transportation, deposition and upheaval of soil. Exposed rocks are eroded and degraded by various physical and chemical processes. The products of erosion are picked up by agencies of transportation such as water and wind, and are carried to new locations where they are deposited .This shifting of the material disturbs the equilibrium of forces on the earth and causes large scale earth movements and upheavals. This process results in further exposure of rocks and the geologic cycle gets repeated.





Type Of Soil

1. **Alluvial soil**

→ Deposited from suspension in running water (formed due to physical weathering).

→ It is transported soil and the transporting agency involved is water.

→ This type of soil is found along the banks of river.

2. **Lacustrine soil :-**

→ Formed due to deposition from suspension in fresh still water of lakes.

→ It is layered soil.

3. **Marine soil :-**

→ deposited from suspension in sea water

4. **Aeoline soil (sand dunes):-**

→ Soil which is transported by wind

5. **Loess soil :-**

→ It is uniformly graded wind blown silt.

→ Slightly cemented due to calcium compounds, when it is wet it becomes soft compressible and loses its cementing action & collapses.

6. **Colluvial (Talus)**

→ It is formed due to transportation by gravitational force.

→ It is found in mountain valley.

→ Generally consists of irregular coarse particles of rocks.

7. **Glacial soil**

→ It is the soil which is transported by ice

→ Drift is a general term used for the deposits made by glaciers directly & indirectly.

→ Till - deposits made by melting of glaciers.

8. **Marl**

→ Fine graded calcium carbonated soil of marine origin which is formed due to decomposition of animal bones & aquatic plants [marine calcareous clay of greenish colour].

9. **Bentonite soil**

→ It is chemically weathered volcanic ash, generally used as lubricant in drilling operation.

→ It is also a clay containing high amount of montmorillonite.

→ It is highly plastic & has high swelling & shrinkage.

10. **Tuff**

→ Slightly cemented volcanic ash that has been transported by wind or water.

11. **Black cotton soil**

→ It is residual soil formed from basalt, containing a high % of clay mineral montmorillonite.

→ It is dark in colour & suitable for growing cotton. It shows high swelling & shrinkage, high compressibility & low shear strength.

12. **Laterite soil / Lateritic**

→ It is a type of soil formed due to leaching (washing out of silicious compound & accumulation of aluminium & iron oxide)

→ Found in hilly areas having humid climate (western ghats, eastern ghats & north - east region)

13. **Muck :-**

→ It is a mixture of inorganic soil & black decomposed organic matters.

→ It is dark in colour.

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14. Peat

→ Highly organic soil which almost entirely consists of vegetative matters in different stages of decomposition.

→ Its colour varies from black to dark brown & it possesses the organic matter. It is highly compressible soil.

Note :-

Muck & Peat soil are also called cumulose soil

15. Loam :-

→ It is a mixture of sand, silt & clay and in soil Enginnering. It may be noted that the material which is called mantle (regolith) in geology is known as soil in soil engineering.

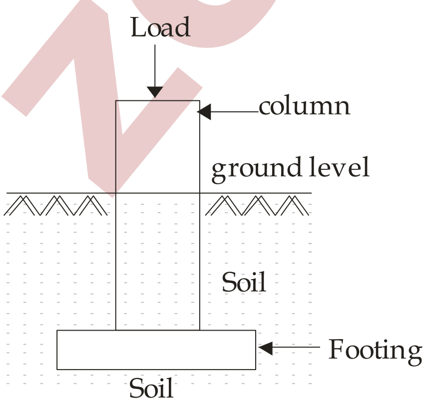
1.2 DEFINITION OF SOIL MECHANICS

The term 'soil mechanics' was coined by Dr. Karl Terzaghi in 1925 when his book Erdbaumechanic on the subject was published in German. According to Terzaghi, soil mechanics is the application the laws of mechanics and hydraulics to engineering problems dealing with sediments and other unconsolidated accumulations of solid particles produced by the mechanical and chemical disintegration of rock, regardless of whether or not they contain an admixture of organic consituents. Soil mechanics is therefore a branch of mechanics which deals with the action of forces on soil ad with the flow of water in soil.

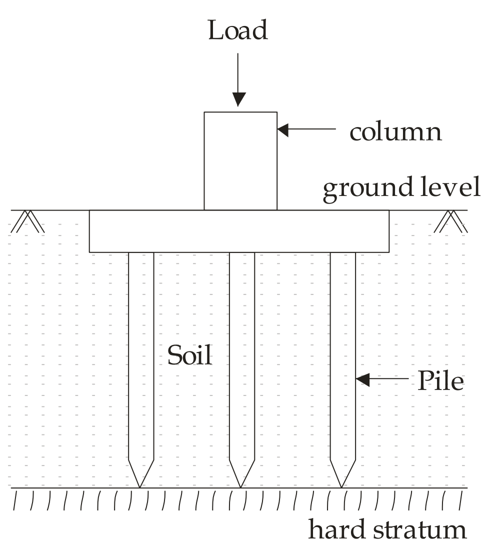
The soil consists of discrete solid particles which are neither strongly bonded as in solids nor they are as free as particles of fluids. Consequently, the behaiour of soil is somewhat intermeadiate between that of a solid and a fluid. It is not therefore surprising that soil mechanics draws heavily from solid mechanics and fluid mechanics. As the soil is inherently a particulate system, soil mechanics is also called particulate mechanics. Rock mechanics is the science dealing with the mechanics of rocks

1.4 SCOPE OF SOIL ENGINEERING

Soil engineering has vast application in the construction of various civil engineering works. Some of the important applications are as under



(a) Shllow foundation



(b)Pilefoundation

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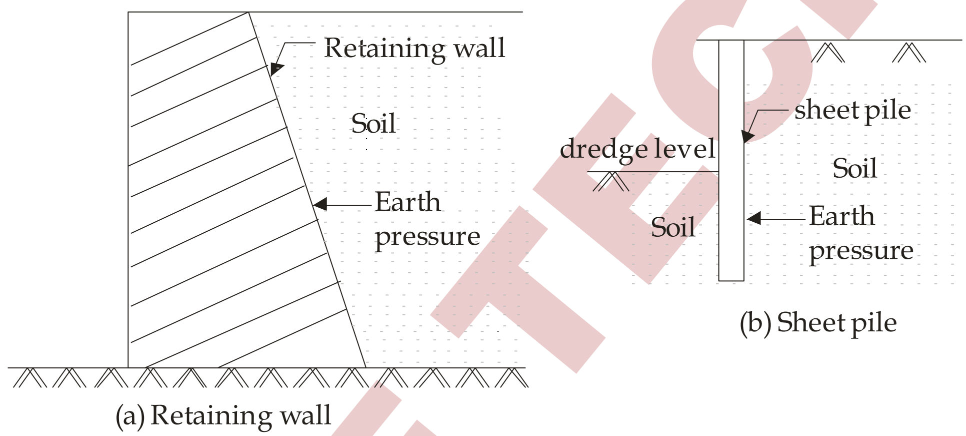
(1) Foundations :- Every civil engineering structure , whether it is a building a bridge, or a dam, is founded on or below the surface of the earth. Foundations are required to transmit the load of the structure to soil safely and efficiency.

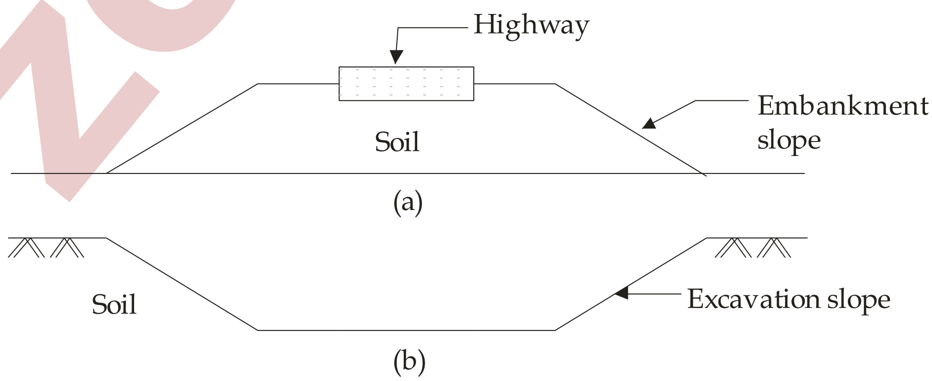
A foundation is termed shallow foundation when it transmits the load to upper strata of earth. A foundation is called deep foundation when the load is transmitted to strata at considerable depth below the ground surface. Pile foundation is a type of deep foundation. Foundation engineering is an important branch of soil engineering.

(2) Retaining Structures :- When sufficient space is not available for a mass of soil to spread and form a safe slope, a structure is required to retain the soil. An earth retaining structure is also required to keep the



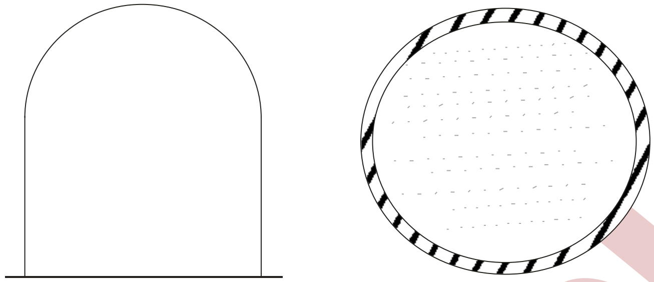
Soil at different levels on its either side. The retaining structure may be a rigid retaining wall or a sheet pile bulkhead which is relatively flexible. Soil engineering gives the theories of earth pressure on retaining structures.

(3) Stability of Slopes :- If soil surface is not horizontal, there is a component of weight of the soil which



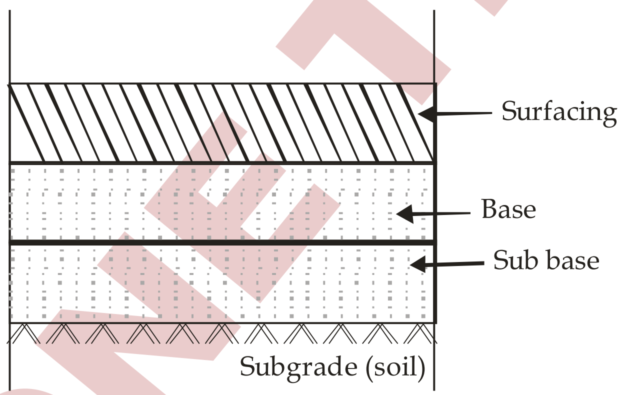
tends to move it downward and thus causes instability of slope. The slopes may be natural or man-made shows slopes in filling and cutting. Soil engineering provides the methods for checking the stability of slopes.

(4) Underground Structures :- The design and construction of underground structures, such as tunnels shafts, and conduits, require evaluation of forces exerted by the soil on these structures. These forces are discussed in soil engineering shows a tunnel constructed below the ground surface and a conduit laid below the ground surface.

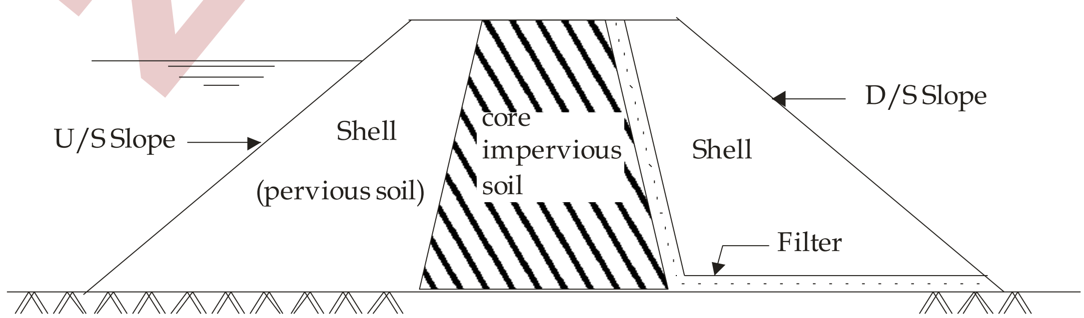


(a) Tunnel (b)Conduit

(5) Pavement Design :- A pavement is a hard crust placed on soil (subgrade) for the purpose of providing a smooth and strong surface on which vehicles can move. The pavement consists of surfacing, such as a bitumen layer base and subbase. The behaviour of subgrade under various conditions of loading and environmental changes is studied in soil engineering.



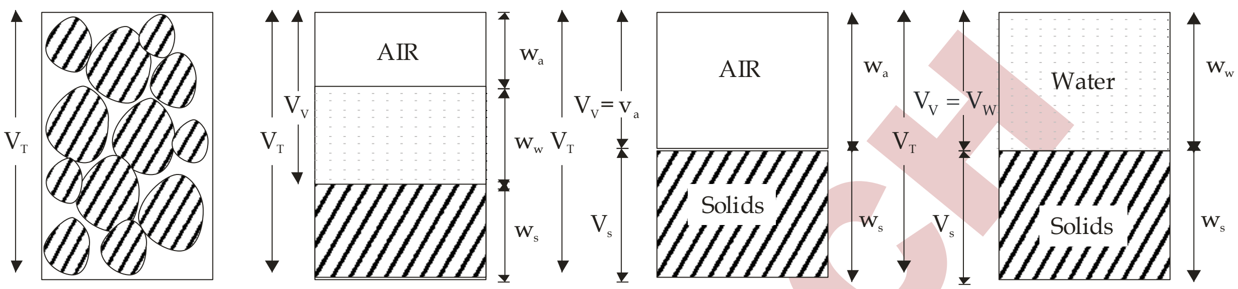
(6) Earth Dam :- Earth dams are huge structures in which soil is used as a construction material. The earth dams are built for creating water reservoirs. Since the failure of an earth dam may cause widespread catastrophe, extreme care is taken in its design and construction. It requires a thorough knowledge of soil engineering.



(7) Miscellaneous Soil Problems :- The geotechnical engineer has sometimes to tackle miscellaneous problems related with soil, such as soil heave, soil subsidence, frost heave, shrinkage and swelling of soils. Soil engineering provides an in depth study of such problems.

Chapter - 2 (Index Properties of Soil)

Properties of Soil (Phase diagram)



3 phase diagram

Dry stage   Saturated stage

2 phase diagram

→ A soil mass is a 3-phase system that may consist of solids, water and air. Which do not occupy seperate spaces but are handed (mixed) with each other in definite proportion which in turn determines the properties of soil mass.

1. Water Content / Moisture Content (w)

→ Defined as ratio of wt. of water to the wt of solids present in the soil mass.

$$w = \frac{\text{wt. of water}}{\text{wt. of solids}} \times 100 = \frac{W_w}{W_s} \times 100 = \frac{M_w}{M_s} \times 100$$

Note - 1 :- Range of water content $w \geq 0\%$ [can be more than 100%]

Note - 2 :- Relation between w, W_T and W_S

W_w - wt. of water

W_s = wt of soil solids

$$w = \frac{W_w}{W_s}$$

$$1 + w = 1 + \frac{W_w}{W_s} = \frac{W_s + W_w}{W_s} = \frac{W_T}{W_S}$$

$$W_S = \frac{W_T}{1+w}, \; W_T = \text{weight of soil}$$

→ Weight of solids is stable quantity in comparison to total weight of soil because it does not changes with change in weight of water hence engineering significance of w is more than w'

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2. Void Ratio

→ Ratio of volume of voids to the volume of solids present in given soil mass

$$e = \frac{\text{volume of voids}}{\text{volume of solids}} = \frac{V_v}{V_s}$$

Note 1

Range of e $\Rightarrow$ e > 0 {can be more than 1 also}

Volume of voids for any other medium could be zero but not for the soil thus void ratio of soil will never be equal to zero. It is generally expressed in a decimal fraction eq 0.6, 0.9, 1.3 etc.

Note-2

Relation between V_S, V_T, & e

$$e = \frac{V_v}{V_s}$$

$$1 + e = 1 + \frac{V_v}{V_s}$$

$$1 + e = \frac{V_s + V_v}{V_s} = \frac{V_T}{V_S}$$

$$V_S = \frac{V_T}{1+e}$$

Note - 3 : Though the size of individual void is more in coarse grained soil but the total volume of voids in fine grained soil is more than that of coarse grained soil due to more number of voids

3. Porosity (n)

$$n = \frac{\text{Vol. of voids}}{\text{Total vol. of soil mass}} \times 100 = \frac{V_v}{V_T} \times 100$$

Note : 1 - Range of n $\Rightarrow$ $0 < n < 100\%$

$$n \neq 0 \qquad n \neq 100\%$$

Note : 2 - Relation between e and n

$$n = \frac{V_v}{V_T} = \frac{V_v}{V_v + V_s} = \frac{1}{1 + \left(\frac{V_s}{V_v}\right)} = \frac{1}{1 + \left(\frac{1}{e}\right)} = \frac{e}{(1+e)}$$

$$n = \frac{e}{1+e} \qquad\qquad e = \frac{n}{1-n}$$

Note - 3 :

Both void ratio & porosity represents volume of voids in given soil mass but void ratio is prefered more significant because it represents volume of voids to volume of solids ratio and volume of solids is more stable than total volume of soil

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4. **Degree of Saturation (S) -** water in voids

$$S = \frac{\text{vol. of water}}{\text{vol. of voilds}} \times 100 = \frac{V_w}{V_v} \times 100$$

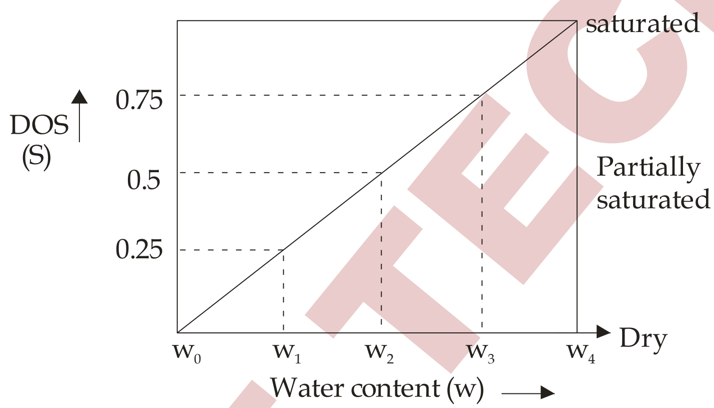
Note : 1 Range of S

$$0 \le S \le 100$$

Note - 2 :

Depending upon degree of saturation soil may be classified as dry, humid, damp, moist, wet and saturated

If S = 0 $\rightarrow$ then soil is dry



5. **Air content :** air in voids

$$a_c = \frac{\text{vol. of air}}{\text{vol of voids}} = \frac{V_a}{V_v} \times 100$$

Note 1

$$0 \le a_c \le 100$$

Note 2 Relation between S & a_c

$$a_c = \frac{V_a}{V_v} \quad (\because V_a = V_v - V_w)$$

$$a_c = \frac{[V_v - V_w]}{V_v} = 1 - \frac{V_w}{V_v} = 1 - S$$

$$a_c + S = 1$$

(6) % air voids (η_a)

$$n_a = \frac{\text{vol. of air}}{\text{total vol. of soil}} \times 100 = \frac{V_a}{V_T} \times 100$$

Note - 1 $0 \le n_a < 100\%$

Note - 2 Relation between $\eta_a, a_c \& n$

$$\eta_a = \frac{V_a}{V_T} \times \frac{V_v}{V_v}$$

$$\eta_a = \frac{V_v}{V_T} \times \frac{V_a}{V_v}$$

$$\eta_a = n \times a_c$$

(7) Unit weight/Density

$$\text{Density} = \frac{M}{\text{Volume}} = \left(\frac{g}{cc}, \frac{kg}{m^3}\right)$$

eq. $\rho_w = 1 gm/cc. = 1000 kg/m^3$

$$\text{unit weight} = \frac{\text{weight}}{\text{volume}} = \frac{N}{m^3}$$

$$\gamma_w = 9810 N/m^3 = 9.81 kN/m^3$$

(4) Bulk Unit weight/Bulk Density :-

Ratio of total weight of soil mass in existing condition expressed in terms of total volume of soil

$$\gamma_b = \frac{\text{Total wt}}{\text{Total vol.}} = \frac{W_T}{V_T} = \frac{N}{m^3} \text{ or } \frac{kN}{m^3}$$

Bulk density $\rho_b = \frac{\text{Total mass}}{\text{Total volume}} = \frac{M_T}{V_T} = \frac{gm}{cc}\left(kg/m^3\right)$

Total weight : $W_T = W_S + W_W + W_a$

Total volume : $V_T = V_s + V_v = V_s + V_w + V_a$

(b) Saturated unit wt / saturated density (γ_{sat}/ρ_{sat})

Ratio of weight of soil mass in saturated stage to total volume of soil

$$\gamma_{sat} = \frac{\text{saturated weight}}{\text{Total volume}} = \frac{W_{sat}}{V_T}$$

$$\rho_{sat} = \frac{M_{sat}}{V_T} = \left(gm/cc, kg/m^3\right)$$

(C) Dry Unit weight/Dry density (γ_d / ρ_d)

$$\frac{\text{weight of soil mass in dry stage}}{\text{Total volume of soil mass}}$$

$$\gamma_d = \frac{\text{dry weight}}{\text{total volume}} = \frac{W_d}{V_T}$$

$$\gamma_d = \frac{W_S}{V_T}$$

$$\rho_d = \frac{M_d}{V_T} = \frac{M_S}{V_T}$$

Total weight = Dry weight = W_s

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Total volume $V_d = V_s + V_v = V_s + V_a$

If bulk is asked then it means existing condtion

Note :-

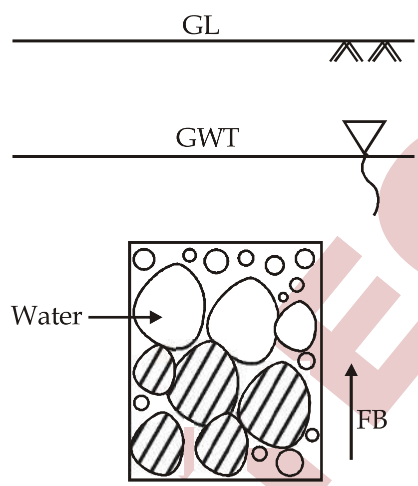
If soil is dry, its bulk unit wt (γ_{bulk}) is same as of its dry unit wt(γ_d)

if s = 0 then $\gamma_{bulk} = \gamma_d$

If soil is saturated its bulk unit wt will be same as that of its saturated unit wt (γ_{sat})

i.e S = 1 then $\gamma_{bulk} = \gamma_{sat}$

(a) Submerged unit weight / submerged density (γ'/γ_{sub})



→ When the soil mass is submerged below the GWT, it is being acted upon by force of buoyancy in vertically upward direction magnitude of which is equal to the weight of water displaced by soil solids hence, it results in reduced weight of soil solids.

$$\gamma_{sub} \text{ or } \gamma' = \frac{(W_s)_{sub}}{V_T}$$

$$\gamma' = \gamma_{sat} - \gamma_w \text{ or } \rho_{sat} - \rho_w$$

Note :- Soil in submerged condition will be in saturated stage but soil in saturated condition need not be in submerged condtion.

→ Soil mass below the water table is in submerged as well as in saturated condition whereas soil mass in capillary zone is in saturated condition only.

(C) Unit weight of solids / Density of solids (γ_s/ρ_s)

$$\gamma_s = \frac{\text{wt. of solids}}{\text{vol. of solids}} = \frac{W_s}{V_s}$$

$$\rho_s = \frac{M_s}{V_s}$$

Note :- Unit wt of solids $\left((\gamma_s) = \frac{W_s}{V_s}\right)$ is more significant than dry unit $\left(\gamma_d = \frac{W_s}{V_T}\right)$ because volume of solids is more stable than total volume

Note-2

$$\gamma_s > \gamma_{sat} > \gamma_b > \gamma_d > \gamma'$$

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(8) Specific gravity :-

(a) **True specific gravity / Absolute specific gravity :-**

Define as the ratio of wt of solids of given volume to the weight of standard fluid (water) of same volume (V_s) or

⇒ It is defined as ratio of unit wt of solids to the unit wt of standard fluid (water)

$$G \,\&\, G_S = \frac{\text{wt. of solids of given volume}}{\text{wt. of water having same volume}\,(V_s)}$$

$$G \,\&\, G_S = \frac{\gamma_s}{\gamma_w} = \frac{\rho_s}{\rho_w}$$

Note :- Range of G

for inorganic soil – 2.6 to 2.9

for organic soil - 1 – 2

(B) Mass/ Bulk / Apparent specific gravity

→ It is defined as the ratio of unit wt of soil to the unit wt of standard fluid (water) or

→ It is defined as the ratio of total wt of soil of given volume to the wt of water having same volume V_T

$$G_m = \frac{\text{wt. of soil of given volume}}{\text{wt. of water having same volume}\,(V_T)}$$

$$G_m = \frac{\rho_b}{\rho_w} = \frac{\gamma_b}{\gamma_w}$$

Important Relationships between various Index Properties:-

1. $Se = wG$

2. $\gamma_b = \dfrac{(G+Se)\gamma_w}{1+e}$

3. $\gamma_{sat} = \dfrac{(G+e)\gamma_w}{1+e}$

4. $\gamma' = \dfrac{(G-1)\,\gamma_w}{(1+e)}$

5. $\gamma_d = \dfrac{G\gamma_w}{(1+e)}$

6. $\gamma_d = \dfrac{\gamma_b}{(1+w)}$

7. $\gamma_d = \dfrac{(1-\eta_a)G.\gamma_w}{(1+wG)}$

8. $e = \dfrac{n}{1-n}$

9. $n = \dfrac{e}{1+e}$

10. $W_s = \dfrac{W_T}{(1+w)}$

11. $V_S = \dfrac{V_T}{(1+e)}$

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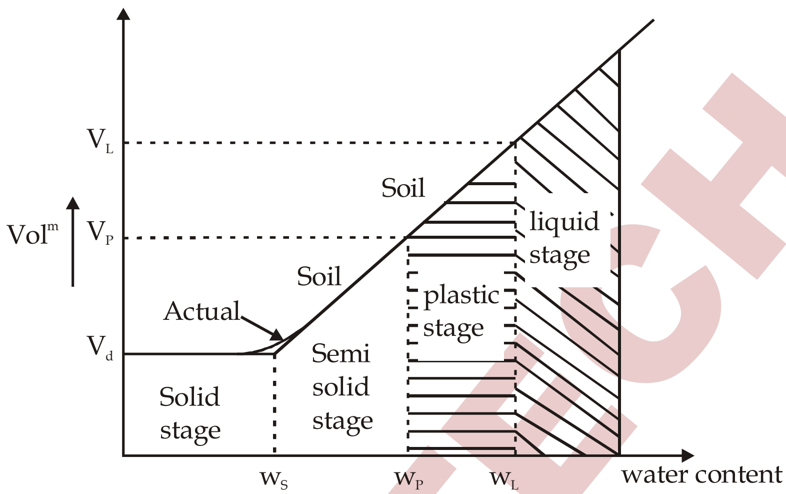
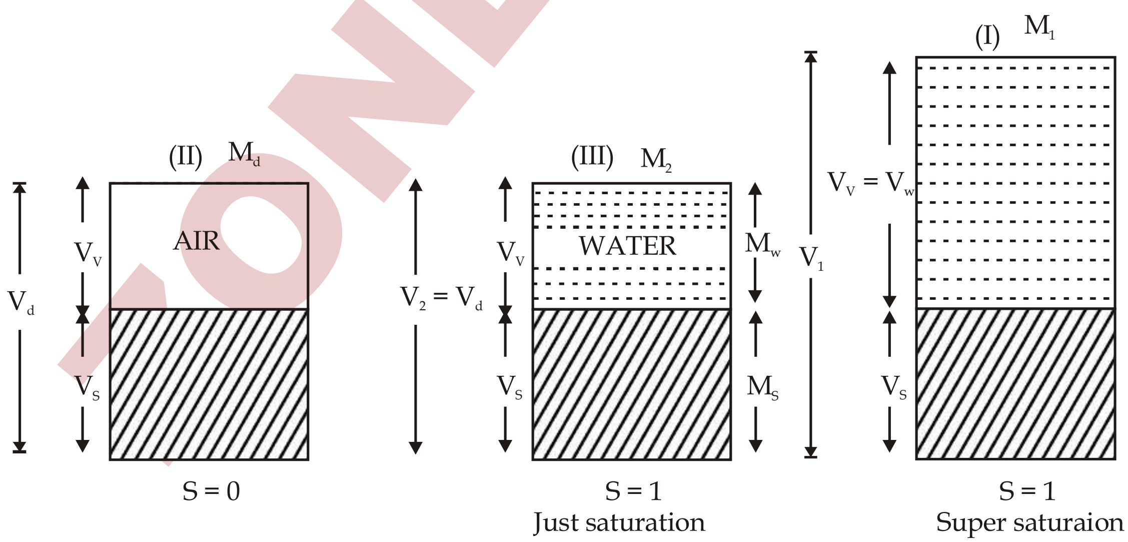
Consistency of soil :-

→ Consistency means relative ease with which the soil can deformed. It also denotes the degree of firmness of soil which may be termed as soft, stiff and hard (indirectly it also represents the strength of soil)

→ This term is used only for fine grained soil and is related with water content



Note :- If soil is partially saturated then upto just saturation any change in water content does not result in change in volome of voids, volume of soil, void ratio and porosity. But if soil is super saturated (S = 1) any change in water content leads to corresponding change in volume of voids and volume of soil



Note :- $\frac{dy}{dx} = \frac{V_L - V_P}{w_L - w_P} = \frac{V_P - V_d}{w_P - w_S} = \frac{V_L - V_d}{w_L - w_S}$

→ Atterberg analyzed consistency of soil in 4 stages as liquid, plastic, semi-solid, and solid stage. The water content at which soil passes from 1 stage of consistency to another stage of consistency is known as consistency limits or atterberg limits.

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Liquid limit (w_L)

→ It is defined as min^m water content at which soil has tendency to flow.

→ At liquid limit, soil passes from liquid stage to plastic stage of consistency and vice versa.

→ All soils at liquid limit, possesses, same negligible shear strength of 2.7 kN/m^2 smallest value that can be measured in lab.

Type of soil	Liquid limit (W_L)
Gravel	Non plastic
Sand	Non plastic
Silt	30 - 40
Clay (Alluvial soil)	30 - 150
Clay (Black soil)	400 - 1500
Clay Bentonite	400 - 800

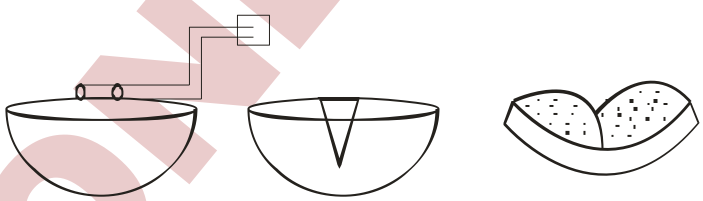
Note :- Soils having high value of w_L possesses high compressibility (volume change in these voids are more)

Determination of Liquid Limit:-

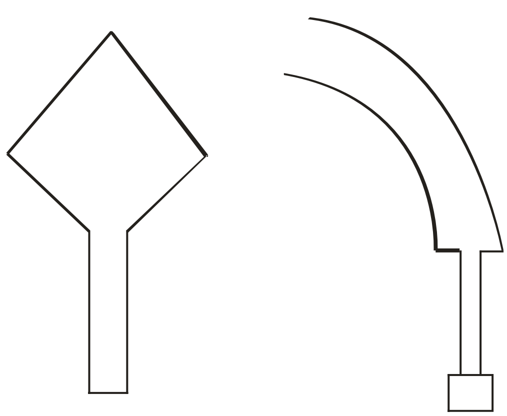
(i) Cassagrande's Apparatus :- Liquid limit may also be defined as minimum water content at which a part of soil cut by a tool of standard dimensions flows together by a distance of approximately 12 mm under the impact of 25 number of blows where height of fall in each blow is adjusted to 1 cm.

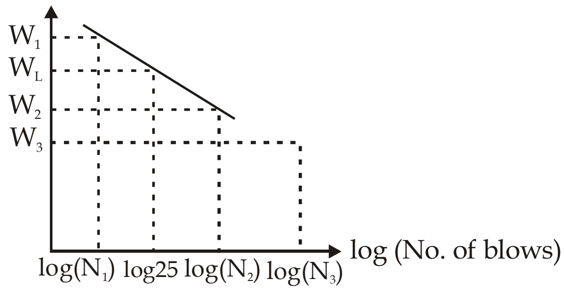
Cassagrande's tool or ASTM tool can be used for creating the groove.

In order to find liquid limit, number of blows required by the soil to flow a distance of 0.5 inch corresponding to a particular value of water content is noted. This test is repeated for same soil at different water contents and corresponding number of blows is noted. The result of test are represented in the form of a curve termed as flow curve.



Casagrande tool :- cuts a grove of width 2mm at bottom $\frac{11}{6}$mm at top and 8mm up





Flow curve

Flow index

Note :- The slope of flow curved is called flow index which represents the rate of loss of shear strength of soil with water content

$$\text{slope of flow curve} = \frac{w_1 - w_2}{\log N_1 - \log N_2}$$

$$\text{flow index} = \left|\frac{w_1 - w_2}{\log N_1 - \log N_2}\right| = \frac{w_1 - w_2}{\log N_2 - \log N_1}$$

$$I_f = \frac{w_1 - w_2}{\log\left(\frac{N_2}{N_1}\right)}$$

One point method

$$w_L = w_y \left(\frac{N}{25}\right)^x$$

→ LL can also be computed using the single observation of cassagrande test emperically.
where
N = no of blows required to close the groove corresponding to water content w_y
x = 0.068 to 1.21 which depends upon number of blows

Plastic limit

→ It is defined as min^m water content at which soil is in plastic stage of consistency or behaves as plastic material at plastic limit soil passes from plastic stage to semi solid stage of consistency and vice versa.

Type of soil	Plastic limit (W_P)
Gravel	Non plastic
Sand	Non plastic
Silt	20 - 25
Clay (alluvial soil)	25 - 50
Clay (Black soil)	200 - 250

→ For determination, plastic limit is defined as min^m water content at which 3mm dia thread can be formed without any crack.

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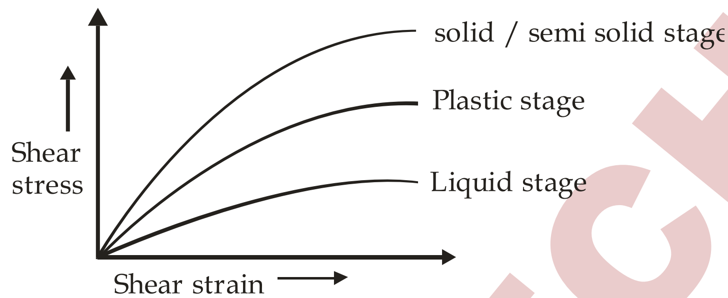
Shrinkage Limit (w_s)

→ It is defined as max^m water content beyond which further reduction in the water content of soil does not lead to the reduction in the volume of soil because below the shrinkage limit replacement of water by air in equal volume takes place on reducing the water content.

→ It is min^m water content at which soil is just completely saturated.

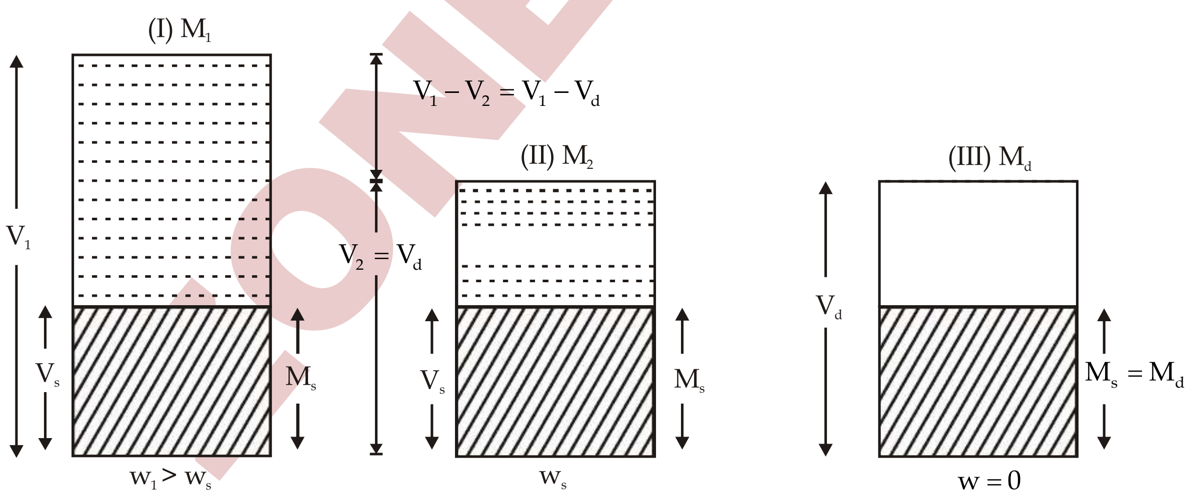
→ At shrinkage limit soil passes from semi solid to solid stage of consistency and vice versa.



Note :- Compressible soil possesses lower value of shrinkage limit while L.L is very high for such soils.

Determination of w_s :-

Consider a sample of soil having water content > w_s. Let M_1 and V_1 be the mass and volume of the soil at this stage. The soil is subjected to drying as a result of which its volume decreases. At a particular point when its water content $= w_s$. Let M_2 and V_2 be the mass and volume of the soil. On complete drying Let M_d and V_d be the mass and volume of the soil.



$$w_1 = \frac{W_w}{W_s} \qquad \Rightarrow w_1 = \frac{M_1 - M_s}{M_s} = \frac{M_1 - M_d}{M_d}$$

1st approach :-

$$w_s = w_1 - \Delta w = w_1 - \frac{\Delta V.\rho_w}{M_s} = w_1 - \frac{(V_1 - V_d)\rho_w}{M_d}$$

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→ Mass of water in I $= M_1 - M_s$

Change in mass of water from I to II $= (V_1 - V_2)\rho_w = (V_1 - V_d)\rho_w$

Mass of water in (II) $= (M_1 - M_s) - (V_1 - V_d)\rho_w$

Water content in (II) $= \dfrac{\text{mass of water in II}}{\text{mass of solids in II}}$

$$= \frac{(M_1 - M_s) - (V_1 - V_d)\rho_w}{M_s}$$

$$= \frac{M_1 - M_d}{M_d} - \frac{(V_1 - V_d)\rho_w}{M_d}$$

$$w_s = w_1 - \frac{(V_1 - V_d)\rho_w}{M_d}$$

2nd approach :-

Volume of water in (II) $= V_2 - V_s = (V_d - V_s)$

Mass of water in (II) $= (V_d - V_s)\,\rho_w$

Water content in (II) $(w_s) = \dfrac{\text{mass of water in(II)}}{\text{mass of solids in(II)}} = \dfrac{(V_d - V_s)\rho_w}{M_s}$

Shrinkage limit $(w_s) = \dfrac{V_d}{M_s}\cdot\rho_w - \dfrac{V_s}{M_s}\rho_w$

$$w_s \Rightarrow \frac{\rho_w}{\rho_d} - \frac{\rho_w}{\rho_s}$$

$$w_s = \frac{1}{R} - \frac{1}{G}$$

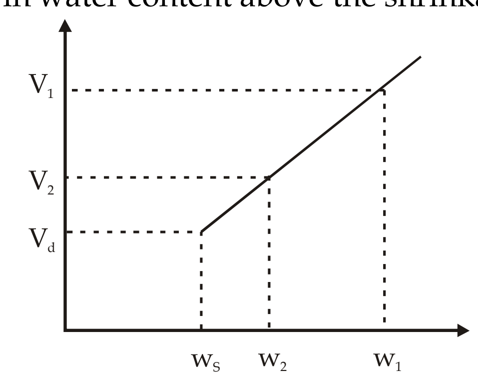
$$w_s = \frac{1}{R} - \frac{1}{G}$$

$R \rightarrow$ shrinkage ratio $= \dfrac{\gamma_d}{\gamma_w} = \dfrac{\rho_d}{\rho_w}$

$G \rightarrow$ specific gravity $= \dfrac{\gamma_s}{\gamma_w} = \dfrac{\rho_s}{\rho_w}$

Shrinkage ratio (R)

→ It is defined as the ratio of decrease in volume of soil expressed as the % of its dry volume to the corresponding change in water content above the shrinkage limit.



$$R = \frac{\frac{(V_1 - V_2)}{V_d}}{(w_1 - w_2)} \times 100$$

If $w_2 = w_s$ then $v_2 = v_d$

$$R = \frac{\left(\frac{V_1 - V_d}{V_d}\right)}{(w_1 - w_s)} \times 100$$

→ **Relation between R, γ_d and γ_w :-**

Change in water content $(w_1 - w_2) = \frac{(V_1 - V_2)\rho_w}{M_s}$ ... (i)

We know that $R = \frac{\frac{(V_1 - V_2)}{V_d}}{(w_1 - w_2)}$

$$R(w_1 - w_2) = \frac{V_1 - V_2}{V_d} \quad \text{... (ii)}$$

$$R = \frac{M_s}{V_d} \times \frac{1}{\rho_w} = \frac{M_d}{V_d} \times \frac{1}{\rho_w} = \frac{\rho_d}{\rho_w}$$

$$R = \frac{\gamma_d}{\gamma_w} = \frac{\rho_d}{\rho_w}$$

→ **Volumetric Shrinkage :-**

→ Volumetric shrinkage is defined as the decrease in volume of soil expressed as % of its dry volume when water content is reduced from its given value upto the shrinkage limit.

volumetric shrinkage $= \frac{V_1 - V_d}{V_d} \times 100$

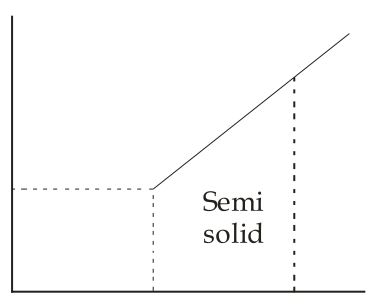
We know that $R = \frac{\left(\frac{V_1 - V_d}{V_d}\right)}{(w_1 - w_2)} \times 100$

$R\,(w_1 - w_2) =$ volumetric shrinkage $= \frac{V_1 - V_d}{V_d} \times 100$

Shrinkage index (I_s) :-

It represents the range of semi-solid stage of consistency

$$I_s = w_p - w_s$$



Plasticity index (I_p) :-

It is the range of consistency in which soil shows plastic properties and behaves as plastic material. It is defined as difference between L.L and P.L of soil

$$I_p = w_L - w_P$$

	(I_P)
Gravel	0
Sand	0
Soft	10 - 15
Clay (alluvial soft)	15 - 100
Clay (Black soil)	100 - 250

Plasticity Index (I_P)	Description
0	Non plastic
< 7	Low plastic
7 - 17	Medium plastic
> 17	High plastic

Note :- Plasticity is defined as prop. of soil by virtue of which it under goes deformation without cracking, fracturing and rupturing.

→ Plasticity of soil is due to the presence of minerals (kaolinite, illite, montmorrilonite) and adsorbed water on it.

→ When non plastic soil like sand is mixed with plastic soil like clay. Its plasticity index will reduce and can be calculated as

$X_1 \rightarrow I_{P1}$

$X_2 \rightarrow I_{P2}$

$$(I_P)_{mix.} = \frac{(I_{P1})X_1 + (I_P)_2 X_2}{(X_1 + X_2)}$$

Consistency Index and Liquidity Index:-

→ These indices represent in situ behaviour degree of firmness of soil depending upon natural water content.

$$I_C = \frac{W_L - W_n}{W_L - w_P} = \frac{W_L - W_n}{I_P}$$

$I_C < 0$	$W_n > W_L$	liquid stage
$I_f > 1$	$W_n < W_P$	solid/semi-solid stage
$0 < I_C < 1$	$W_P < W_n < W_L$	Plastic

Liquidity Index (I_L)

$$I_L = \frac{W_n - W_p}{W_L - W_p} = \frac{W_n - W_p}{I_p}$$

$I_L > 1$	$W_n > W_L$	liquid stage
$I_: < 0$	$W_n < W_P$	solid/semi-solid
$0 < I_L < 1$	$W_P < W_n < W_L$	plastic stage

Note :- $I_C + I_L = 1$

I_C	I_L	Consistency	Description
< 0	> 1	Liquid	Liquid
0 - 0.25	1 - 0.75		Very soft
0.25 - 0.5	0.75 - 0.5	Plastic	Soft
0.5 - 0.75	0.5 - 0.25		Medium stiff
0.75 - 1	0.25 - 0		Stiff
> 1	< 0	Semi-solid	Very stiff to hard
> 1	< 0	Solid	Hard to very hard

Toughness Index (I_T) :-

→ It represents strength of soil at its plastic limit.

→ It is defined as the ratio of plasticity index to the flow index of soil.

$$I_T = \frac{I_P}{T_f}$$

→ Toughness index generally varies bewteen 0-3. If for any soil, $I_T < 1$ then it is considered to be easily breakable at plastic limit.

Density Index/ Relative Density/Degree Of Density (I_D) :-

→ Density index is used to represent the relative compactness of coarse grained soil.

→ It is defined as

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

e_{max} = maximum void ratio in the loosest conditon of soil

e_{min} = minimum void ratio of soil in the density stage

e = void ratio of soil in its natural stage.

I_D in terms of γ_d :-

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

$$I_D = \frac{\left(\frac{G\gamma_w}{(\gamma_d)_{min}} - 1\right) - \left(\frac{G\gamma_w}{\gamma_d} - 1\right)}{\left(\frac{G\gamma_w}{(\gamma_d)_{min}} - 1\right) - \left[\frac{G\gamma_w}{\gamma_{d\,max}} - 1\right]} \times 100 \Rightarrow I_D = \frac{\frac{1}{(\gamma_d)_{min}} - \frac{1}{(\gamma_d)}}{\frac{1}{(\gamma_d)_{min}} - \frac{1}{(\gamma_d)_{max}}} \times 100$$

Density Index	Description
0 - 15	Very loose
15 - 35	loose
35 - 65	Medium
65 - 85	Dense
85 - 100	Very dense

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Sensitivity and Thixotropy :-

→ Consistency of undisturbed sample of clay is changed upon remoulding even at same water content.

→ This change in consistency or decrease in degree of firmness or decrease in strength takes place due to following factors.

1. Permanent destruction of soil solid upon remoulding (permanent loss)
2. Reorientation of water molecules in the adsorbed layer of soil solids (recoverable)

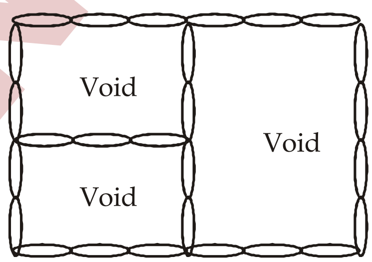
→ This loss in strength of soil is represented in terms of its sensitivity which denotes the degree of disturbance of sample upon remoulding.

→ Sensitivity is defined as the ratio of unconfined compressive strength of soil (UCS) in its undisturbed state to unconfined compressive strength of soil (UCS) in its remoulded state.

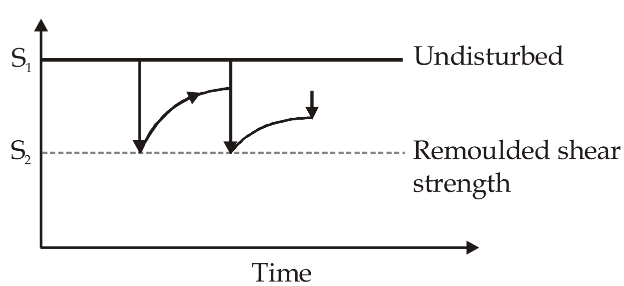
$$st = \frac{\text{Undisturbed shear strength}}{\text{Remoulded shear strength}}$$

$$st = \frac{(\text{UCS})\text{undisturbed}}{(\text{UCS})\text{remoulded}}$$

Sensitivity	Description
< 1	In sensitive
2 - 4	Normal/low sensitive (honey comb)
4 - 8	medium sensitive (honey comb)
8 - 16	Extra sensitive (flocculant)
> 16	unstable/quick



→ Over a period of time, soil regains a part of its lost strength, this property of soil by virtue of which its regains a part of its lost strength at constant water content is termed as thixotropy.



Activity

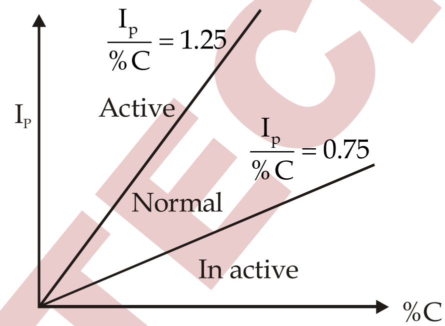
→ The behaviour or plasticity of soil depends on type of mineral (kaolinite, lllite, montmorillonite) and the amount of adsorbed water present in the soil

→ Skempton defined a parameter termed as activity which represents the compressibility or volume change in soil with change in water content (swelling and shrinkage)

→ Activity is defined as the ratio of plasticity index of the soil to the % of particles present in it having size < 2μm (clay size)

At $= \frac{I_P}{\%C}$ where = percentage of clay size particle

Activity	Description
< 0.75	In active
0.75 - 1.25	Normal
> 1.25	Active



Slope of I_p Vs % clay curve denotes activity. More the slope more active will be the soil.

	Activity
Kaolinite	0.4 - 0.5
Illite	0.5 - 7
Montmorillonite	1 - 7
Na - Montmorillonite	4 - 7
Ca - Montmorillonite	1.5

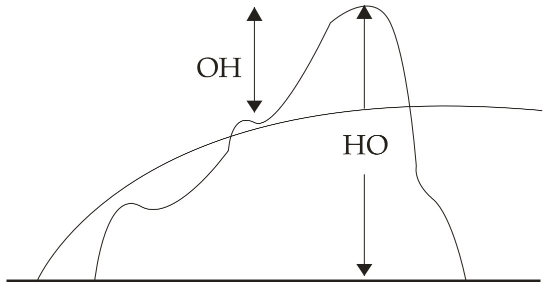
Note :- Highly active soils are not considered suitable for construction. Black cotton soil consists of high amount of montmorillonite. This shows large volume change with change in water content.

Collapsibillty :-

→ Soil which shows large ↓e in volume with ↑ in water content without any ↑in pressure and termed as collaprible soil Eg. loess.

→ Collapribility of soil is measured in the form of a parameter urmed as collaprifle potential which is defined as the ratio of decrease in volume of soil to its original volume with change in its water content

$$C.P. = \frac{b_v}{v_0} = \frac{\Delta H}{H_0}$$



1. **Water content :- Water content can be determined using any of the following methods.**

A. **Oven Drying Method :-**

→ This is most accurate laboratory method

→ in this method moist sample of soil is placed is empty container of known mass (W_1) and after placing, it is weighed again (W_2).

→ Container with moist, soil sample is placed in temperature controlled oven for drying.

→ For Inorganic soil, temperature is maintained between 105°C to 110°C for 24 hours.

→ For organic soil, temperature is not increased beyond 60°C in order to avoid oxidation of organic matters.

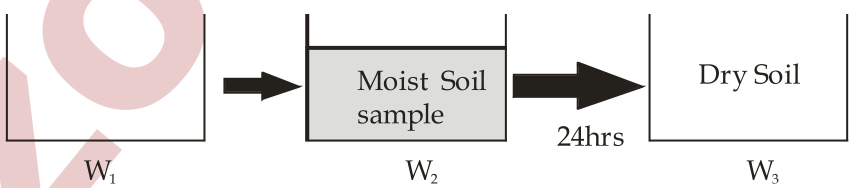
→ For soil having gypsum, temperature is limited to 80°C in order to avoid, loss of water of crystallization (stucutral water).

→ In no case, temperature is increased beyond 110°C because it results in destruction of soil solids causing loss of structural water.

Note :- In oven drying, all types of soil water (free water, capillary water absorbed water) are removed, except structural water or water of crystallization if present.

→ After drying container with dry soil is again weighed (W_3) and water content is measured follows

$$\omega = \frac{\text{Weight of water}}{\text{Weight of solids}} \times 100 = \frac{W_2 - W_3}{W_3 - W_1} \times 100$$



105 to 100°C
< 60°C
< 80°C

B. **Sand Bath Method :-**

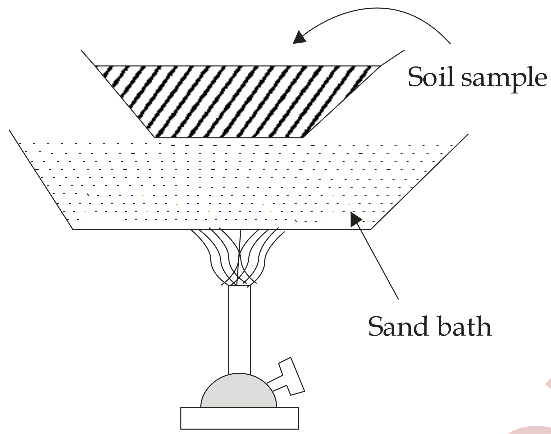
→ It is quick field method used when oven is not available

→ In this method, container with moist soil is placed in sand bath and is heated over stove

→ As there is no control over temperature it is not generally suitable for organic soil and soil with gypsum content.

→ Results obtained from this test are inaccurate because it may include structural water also.

→ Steps of observation in this method are same as that of oven drying method



C. Alcohol method :-

→ In this method, methylated spirit is added to sample in order to increase, rate of evaporation during drying.

→ It is also a quick field method

→ Because there is no control over temperature, it is not suitable for organic soil and soils with high gypsum content.

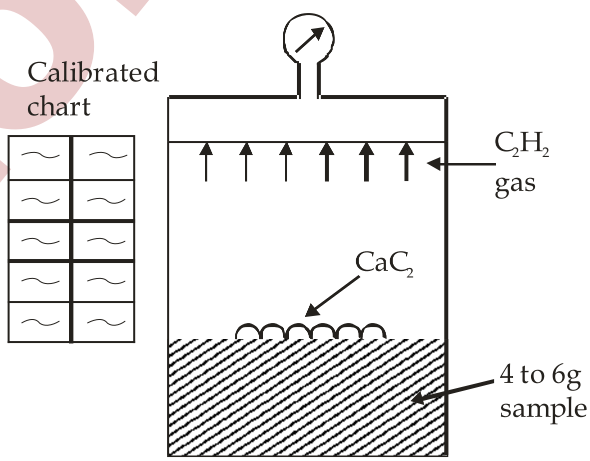
→ Steps of obeservation are same as that of oven drying method

D. Calcium Carbide Method

→ It is one of the quickest methods to find water content of soil which gives the result within 5 to 7 minutes.

→ It is field method.

→ In this method 4 to 5 g (fixed weight is taken due to calibration) of sample is placed in moisture meter and calcium oxide is added in it which reacts with the water present in the sample resulting in the formation of acetylene gas that exsits pressure over the gauge which is further calibrated to given water content of soil is terms of total mass of soil ()



$$CaC_2 + H_2O \longrightarrow C_2H_2\uparrow + Ca(OH)_2$$

Acetylene

G. Pyconometer method

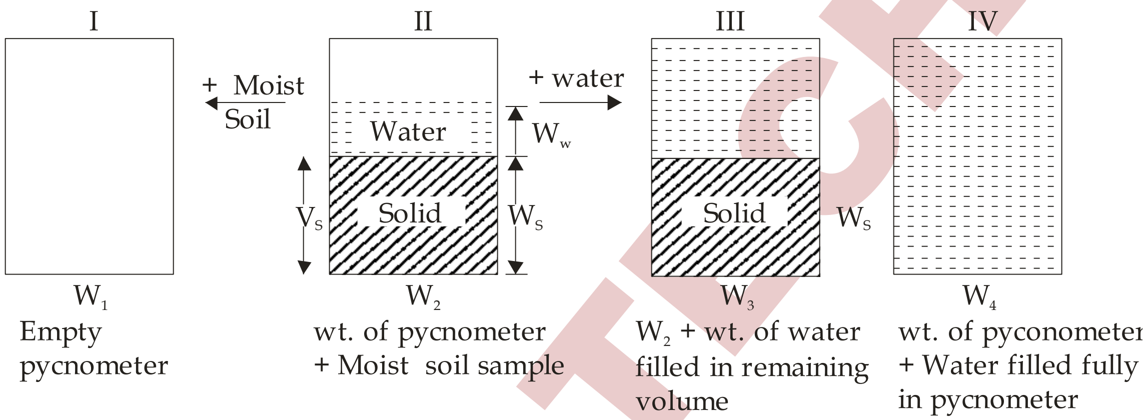
→ It is quick method gives results within 10 to 20 mins.

→ This method is used for those soil for which (sp. gravity) is known.

→ Pycnometer is 900 ml flask having conical top with 6mm diameter circular hole at its centre for the removal of air.

→ This method is suitable for cohesionless soil (because removal of entrapped gases in difficult from cohesive soils).

Steps of observation



It can be seen that, if from W_3, weight of solids is removed and replaced by the weight of an equivalent volume of water, the weight W_4 is obtained.

$$W_4 = W_3 - W_s + V_s \gamma_w$$

Weight of water having equivalent volume

$$V_s = V_s U_w$$

$$W_4 = W_3 - W_s + \frac{W_s}{G_{\gamma w}} \cdot \gamma_w \qquad \because \gamma_s = \frac{W_s}{V_s} = G\gamma_w$$

$$W_3 - W_4 = W_s\left(1 - \frac{1}{G}\right)$$

$$W_S = W_3 - W_4\left(\frac{G}{G-1}\right) \rightarrow (A)$$

$$\text{Water content} = \frac{\text{Weight of water}}{\text{Weight of solids}} \times 100$$

$$\omega = \frac{W_2 - W_1 - W_S}{W_S} \times 100 = \left(\frac{W_2 - W_1}{W_S}\right) \times 100$$

$$\omega = \left(\frac{W_2 - W_1}{W_3 - W_4}\left(\frac{G-1}{G}\right) - 1\right) \times 100$$

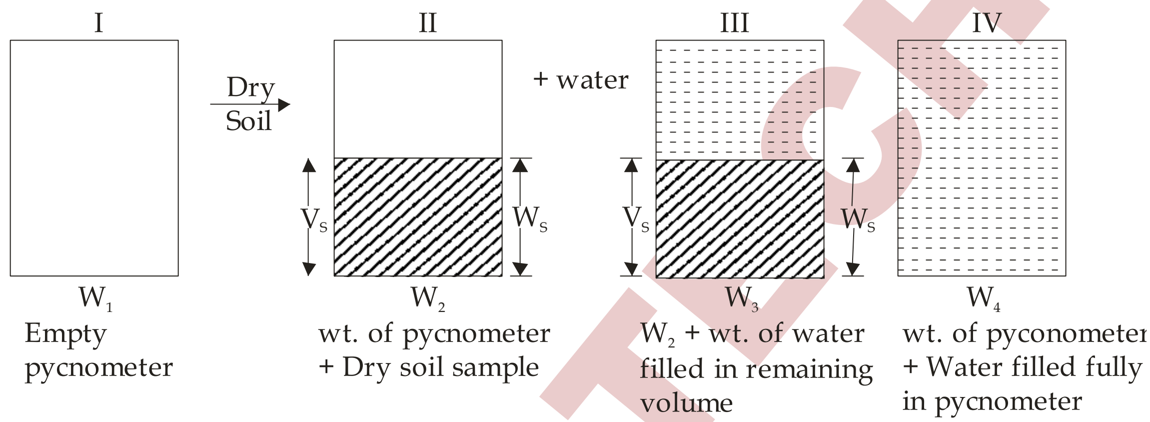
2. Specific Gravity

→ Specific gravity of soil can be computed using 50 ml density bottle, 500 ml flask or by pynometer.

→ Specific gravity of solids is given as

$$G = \frac{\text{Weight of solids of given volume}}{\text{Weight of water having same volume}}$$

Step of observation ; In this method dry soil sample is used in pycnometer



Weight of solids = Dry wt of soil mass = $W_2 - W_1 = W_d$

Weight of water in III = $W_3 - W_2$

Weight of water IV = $W_4 - W_1$

$$\left.\begin{array}{r}\text{Weight of water having same volume}\\ \text{as that of soil solids}\left(V_s\right)\end{array}\right\} = \text{Weight of water in IV} - \text{Weight of water is III}$$

$$= W_4 - W_1 - (W_3 - W_2) = W_4 - W_3 + W_2 - W_1$$

$$G = \frac{\text{Weight of solids of given volume}}{\text{Weight of water having same volume}}$$

$$= \frac{W_2 - W_1}{W_4 - W_3 + W_2 - W_1} = \frac{W_d}{W_4 - W_3 + W_d}$$

Note :- Kerosene is used instead of water for fine grained soil. Because it is better wetting agent that water. If kerosene is used instead of water, then SG is calculated as

$$G = \frac{W_2 - W_1}{W_4 - W_3 + W_2 - W_1} \times G_k = \frac{\gamma_s}{\gamma_{kerosene}} \times G_k$$

$$= \frac{\gamma_\sigma}{\gamma_\kappa} \times \frac{\gamma_k}{\gamma_\omega} = \frac{\gamma_s}{\gamma_\omega}$$

(G_k = specific gravity of kerosene)

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3. **Unit Weight of Soil**

Unit weight of soil can be determined using following methods.

A. **Core-Cutter Method :-**

→ It is field method

→ Core cutter of known volume 1000 CC is pushed into the ground and then core containing soil is taken out from the ground.

→ Weight of core with soil sample (W_2) and empty core (W_1) is measured Hence bulk unit weight is given as

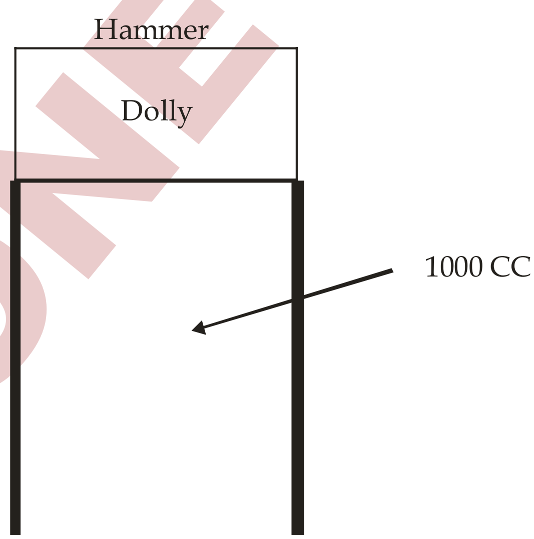
$$\gamma_b = \frac{\text{Weight of soil}}{\text{Volume}} = \frac{W_2 - W_1}{V}$$

→ By measuring water content of given sample, dry unit weight can be calculated as

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

→ This method is generally not suitable for gravels, sand and dry soil

→ It is generally used for soft silt and clay.



B. **Water Displacement Method :-**

→ This method is generally used for cohesive soils that are highly sticky in nature

→ In this method, sample of soil is trimmed into more or less uniform shape and weighed (W_1)

→ Sample is then coated with parafin wax and is again weighed (W_2), coating prevent entry of water into sample.

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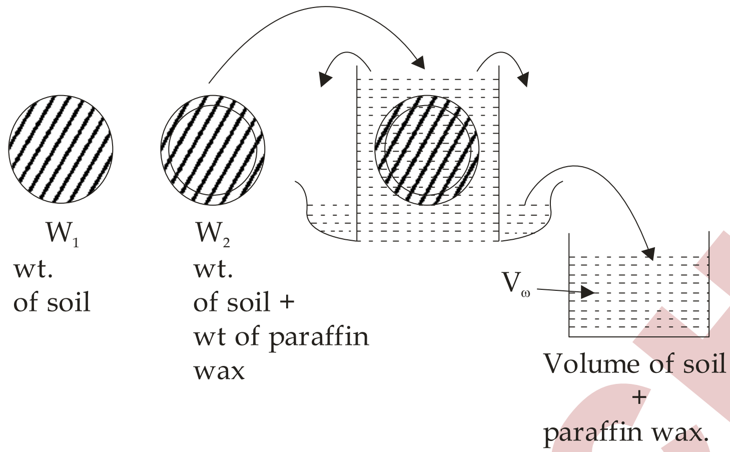
Water Displacement Method



→ The coated specimen of soil is immersed slowly in a container, that is completely filled with water and volume of water displaced by coated specimen is V.

$$\text{Volume of paraffin wax} = \frac{W_2 - W_1}{\gamma_{PW}} = \frac{W_2 - W_1}{G_{PW}\gamma_\omega}$$

$$V_\omega = \text{Volume of soil} + \text{Volume of paraffin wax.}$$

$$\text{Volume of soil} = V_\omega - \text{Volume of paraffin wax}$$

$$= V_\omega - \frac{W_2 - W_1}{\gamma_{PW}}$$

$$\text{Unit weight of soil} = \frac{\text{Weight of soil}}{\text{Volume of soil}} = \frac{w_1}{V_w - \dfrac{(w_2 - w_1)}{\gamma_{pw}}}$$

D. Sand Replacement Method :-

→ It is field method

→ A small pit is excavated and the excavated soil sample is a weighted.

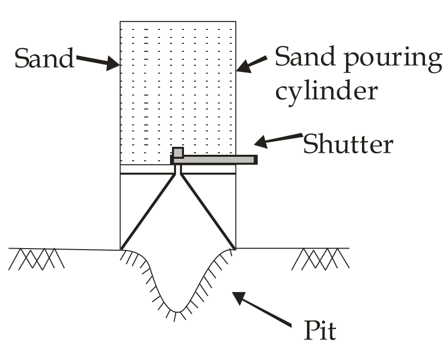
→ A calibrated cylinder containig sand is placed over the excavated pit and is filled with sand

→ Volume of the pit is obtained from the calibrated cylinder

→ The bulk unit weight is calculated as

$$\gamma_b = \frac{\text{Weight}}{\text{Volume}}$$

→ It is suitable for gravel, sand and dry soil.



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CHAPTER - 3 (CLASSIFICATION OF SOIL)

Particle Size Distribution Analysis

Coarse grained soil	Gravel	4.75 mm	to	80 mm
	Sand	75 μm	to	4.75 mm
		0.075 mm	to	4.75 mm
Fine grained soil	Silt	2 μm	to	75 mm
		0.002 mm	to	0.075 mm
	Clay	< 2 μm		
		< 0.002 mm		

Sieve Analysis

→ Sieve analysis is done for the soil fraction size greater than 0.075mm. It means particles which are retained over 75μ sieve.

→ As per IS460 : 1962 sieves are designed by the size of the square opening in mm (or) μm (1 micron = 10^{-6}m = 10^{-3} mm = 1 μm)

→ In this method different sieves are arranged one over each other in descending order of their size.

→ An oven dried sample of soil is placed over top most sieve and sieving is done atleast for 10 min either manually or in sieve shaker.

→ Weight of soil fractions retained over each sieve is noted to compute the percentage finer corresponding to given size of sieve.

[% finer (%N) = 100 – cumulative % weight retained]

(a) Coarse Sieve / Gravel Sieving :-

→ It is done for soil fraction having size greater than 4.75 mm. It means retained over 4.75 mm sieve.

→ Standard sieves used are 80 mm, 40 mm, 20 mm, 10 mm and 4.75 mm.

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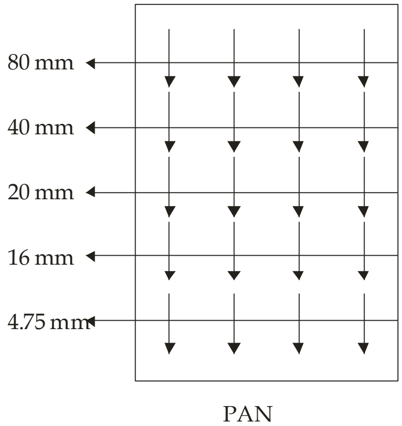

Sieve size (mm)	Wt. Retained	%wt. Retained	Cumulative %wt. Retained	%N = 100 - Cumulative %wt. Retained
80	10 g	10%	10%	90%
40	30 g	30%	40%	60%
20	30 g	30%	70%	30%
10	20 g	20%	90%	10%
4.75	10 g	10 %	100 %	0 %
	100 g			

(b) Fine Sieving / Sand Sieving :-

→ It is done for soil fraction that passes through 4.75 mm sieve but are retained over 0.075mm sieve.

→ Standard sieves used are 2mm, 1mm, 600μ, 425μ, 212μ, 150μ, 75μ

→ The result of the sieve analysis is represented in terms of size of the particle and corresponding % finer.

→ It is generally preferred to wash the soil fraction that passes through 4.75 mm sieve before carrying out sieving in order to separated clay and silt particles present over sand this process is termed as wet sieving.

(c) Sedimentation Analysis :-

→ Sedimentation analysis is carried out for the soil fraction having size less than 0.075 mm or which passes through the 0.075 mm sieve.

→ Particles having size less than 0.2μ (0.0002 mm) can not be analyses even by sedimentation. For them special techniques like electron microscope and X-ray diffraction techniques are used.

→ Sedimentation analysis is based on stoke's law, according to which velocity of particle undergoing settlement within infinite medium is depend upon size, shape and mass of the particle.

→ According to stokes law, terminal velocity is given as

$$V_s = \frac{(\gamma_s - \gamma_\omega)d^2}{18\mu} = \frac{(G-1)\gamma_w d^2}{18\mu}$$

(d) Limitation of Stoke's law :-

1. Particles undergoing settlement is assumed to be spherical but in actual, fine grained particles are flaky in nature.
2. The medium in which settlement takes place is assumed to be infinite but in actual, sedimentation jar has finite dimension.
3. The particles are assumed to be undergoing discrete settling. But in actual fine grained particles aggregates during the settlement.

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4. Stoke's law is valid for the particles having size between 0.2 mm to 0.2 μm.

→ If the size of particle is greater than 0.2 mm gravity acceleration sets up during settllement due to turbulent motion as a result of which constant velocity is not achieved throughout the settlement.

→ If the size of particle is less than 0.2 μm, then Brownian motion setup which does not allow settlement of particle.

General Procedure for Sedimentation Analysis :-

→ Sedimentation analysis can be carried out either by pipette method or by hydrometer method. Method of preparation of soil suspension is same in both method as follows :

Pretreatment

→ Before preparing soil suspension, organic matter and calcium compounds present in soil should be removed in order to avoid the aggregation of solids during settlement.

→ Organic matters are removed by oxidizing agents like hydrogen peroxide (H_2O_2) which carries out oxidation of organic matters and calcium compounds are removed by 0.2N, HCl.

→ Soil passing through 75μm sieve is mixed in water to prepare standard volume of soil suspension.

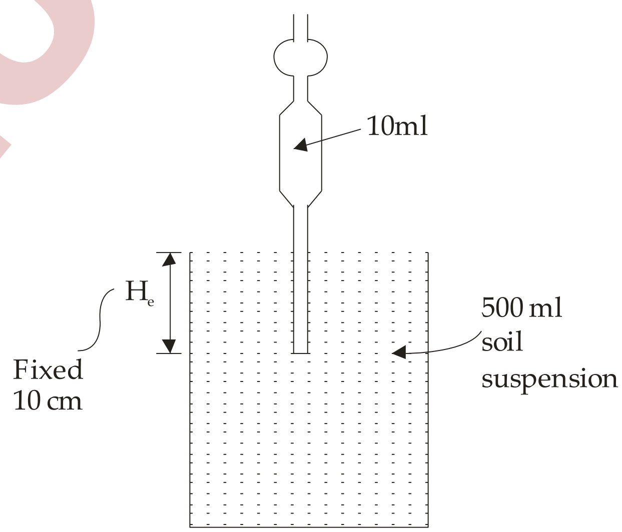
→ In pipette method, 12 to 30 gram of soil is added in water for preparation of 500 ml soil suspension total volume and In hydrometer method 24 to 60 g of soil is added in water for preparation of 1000 ml of soil suspension.

Post treatment

→ After preparing suspension, dispersing agents or deflocculating agent is added in it to prevent the flocculation of solids during the settlement.

→ Commonly used deflocculating agents are sodium hexa metaphosphate, sodium carbonate, sodium oxilate, sodium phosphate.

Pipette Method :-



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→ In this method 10 ml of sample is collected from soil suspension at fixed sampling depth at 10 cm at different time intervals.

→ The collected sample is further being tested for mass of solids (m_d) present in it by oven drying method.

→ The size of particles that settles in the soil suspension by distance of sampling depth (H_e = 10cm) is computed using Stoke's law.

$$V_s = \frac{(G-1)\gamma_\omega d^2}{18\mu} = \frac{H_e}{t}$$

$$d^2 = \frac{18\mu}{(G-1)\gamma_w} \cdot \frac{H_e}{t}$$

$$d = \sqrt{\frac{18}{(G-1)\gamma_w}} \cdot \sqrt{\frac{H_e}{t}} = k\sqrt{\frac{H_e}{t}}$$

Where, $$k = \sqrt{\frac{18}{(G-1)\gamma_w}}$$

Where t = Time is second after which the pipette is immersed in soil suspension.

% finer (%N) corresponding to computed size of particle of which settles in suspension by H_e in given time t is analyzed as

$$\%N = \frac{\text{Mass of solids / unit volume at depth } H_e \text{ at time 't'}}{\text{Mass of solids per unit volume initially}}$$

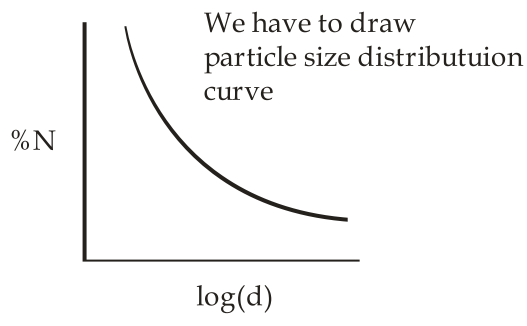
$$= \frac{(m_d / V_P)}{M / V} \times 100$$

m_d : mass of solids in collected sample from sampling depth H_e at time 't'

V_P : volume of pipette (10 ml)

M : mass of solids added initially of the preparation of soil suspension (12 to 30 g)

V : total volume of suspension (500 ml)



Hydrometer Method

→ Hydrometer is device which is used for the measurement of density of the suspension or specific gravity of soil suspension at depth H_e at time t.

$$\rho_{ss} = 1 + \frac{R_h}{1000}$$

$$S_{ss} = \frac{\rho_{ss}}{\delta_\omega} = \frac{\rho_{ss}}{1g/cc} = 1 + \frac{R_h}{1000}$$

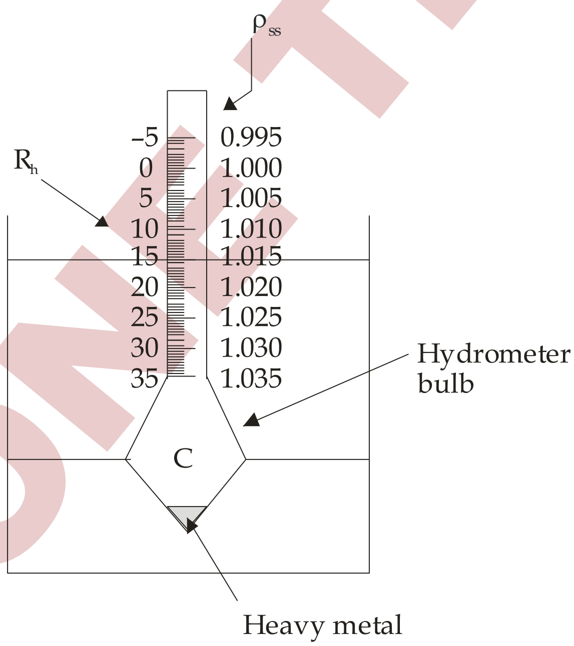
Where R_h = Corrected hydrometer reading

→ Volume of hydrometer below the centre of its bulb is approximate 50% of its total volume.

→ Reading on the hydrometer increase downward.

→ For calibration, reading on the hydrometer is marked as

$$R_h = (\rho_{ss} - 1) \times 1000$$



→ In hydrometer method %N (m_d) is computed indirectly by noting density of suspension at sampling depth H_e at different time intervals.

→ In pipette method, sampling depth was constant (H_e = 10cm) whereas in hydrometer method sampling depth goes on increasing with increase in time and settlement of particles, due to which calibration of hydrometer and sedimentation jar is done before each test.

→ The size of particles that settle in suspension by a distance of sampling depth H_e in any time t is given as

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$$d = k\sqrt{\frac{H_e}{t}}$$

Where $$k = \sqrt{\frac{18\mu}{(G-1)\gamma_w}}$$

→ %N corresponding to noted size of particle d is computed indirectly by noting density of the suspension at sampling depth H_e at any time t as

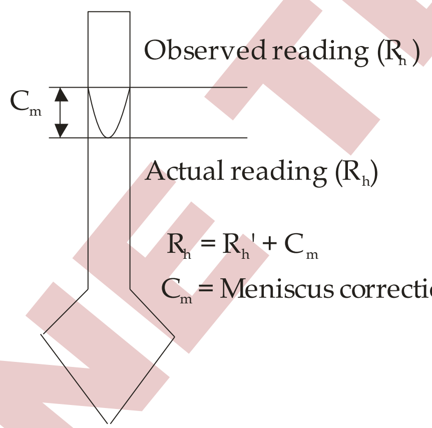
$$\%N = \frac{\frac{R_h}{1000} \times \frac{G}{G-1}}{\frac{M}{V}} \times 100$$

M : Mass of the solids added initially (in grams)

V : Total volume of soil suspension (1000 ml)

The observed reading of hydrometer are further being corrected for the following.

1. Meniscus Correction (C_m)



$R_h = R_h' + C_m$

C_m = Meniscus correction

→ Due to presence of turbidity in soil suspension upper level of meniscus is noted instead of lower, which leads to the reduced value of the observation being noted. Hence meniscus correction applied is positive.

2. Temperature Correction (C_T)

→ The calibration of hydrometer is done at 27°C.

→ If the temperature during the test is greater than 27°C, it results in reduced value of observation being noted. Hence correction applied is positive and vice versa.

Temp > 27°C C_T = +ve

Temp < 27°C C_T = –ve

$$R_h = R_h' \pm C_T$$

3. Dispersing Agent Correction (C_d) :-

→ Addition of dispersing agent in suspension increase its density, which results in increased value of observation being noted. Hence dispersing agent correction applied is negative.

$$R_h = R_h' - C_D$$

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Particle Size Distribution Curve:-

→ In this curve, %N (finer) is expressed on y-axis and corresponding size of particle is expressed on x-axis is log-scale.

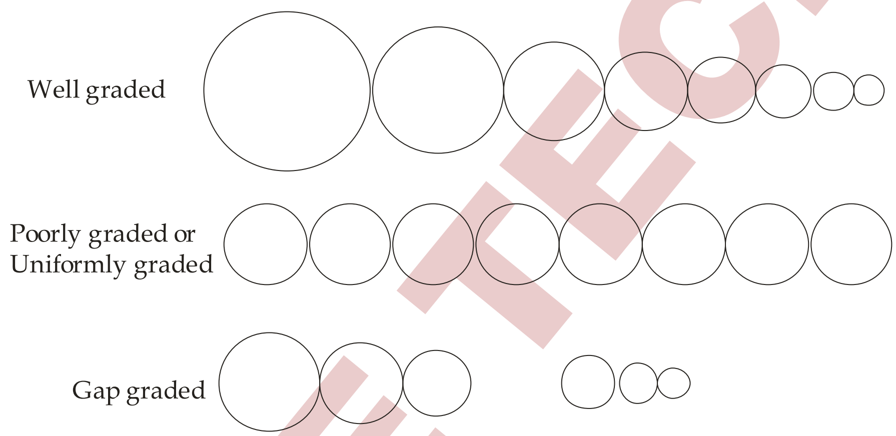
→ The curve helps in analyzing the type and gradation of soil.

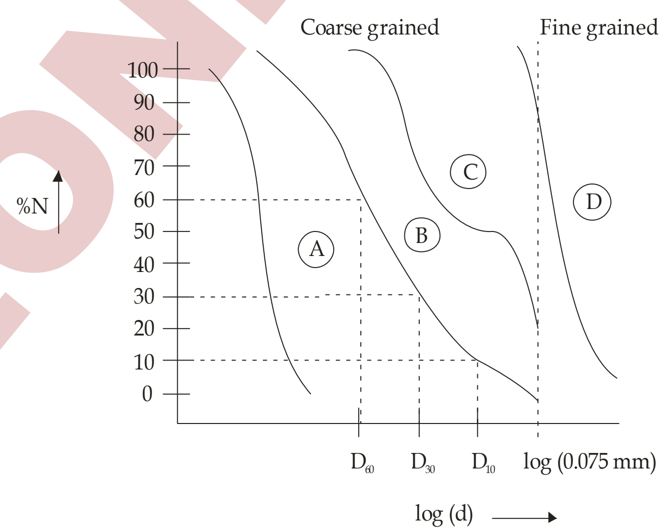
→ The soil may be termed as well graded soil, Poorly/uniformly graded soil & gap graded soil.

→ If the soil has good representation of all the sizes of the particle present in it, it is termed as well graded soil.

→ If the soil consists of excess of one size of particles or is in deficient of another size of particles (or it consist of same size of particles) it is termed as poorly graded soil [or uniformly graded soil in later case]

→ If certain size of particles is missing from soil, it is know as gap graded soil.





Curve A : Uniformly / poorly graded coarse grained soil

Curve B : Well graded coarse grained soil

Curve C : Gap graded coarse grained soil

Curve D : Well graded fine grained soil

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From particle size distribution curve, D_{10}, D_{30}, D_{60}, can be found out.

D_{10} - Effective size, defined by Allen Hazen, size in mm such that 10% of particles are finer than this size.

D_{30} - Size in mm such that 30% of particles are finer than this size.

D_{60} - Size in mm such that 60% of particles are finer than this size.

Coefficient of Uniformly (C_u)

→ It represents particle size range of distribution curve.

→ It is defined as ratio of D_{60} to the D_{10}

$$C_u = \frac{D_{60}}{D_{10}}$$

→ For uniformly graded soil, $C_u = 1$ (or less than 2)

→ For well graded sand $C_u > 6$

→ For well graded gravel, $C_u > 4$

Coefficient of Curvature (C_c)

→ It represents shape of particle size distribution curve

→ It is defined as

$$C_C = \frac{D_{30}^2}{D_{60}D_{10}}$$

→ For well graded soil, C_C should be in the range of 1 to 3

→ If C_C is less than 1 or greater than 3, then soil is gap graded soil.

Methods of Classification:-

Classification of soil is done to arrange it into different groups on the basis of its engineering properties. Classification of soil can be done by any of the following method

1. Particle Size Classification

In this method, classification of soil is done on the basis of particle size composition according to which soil may be classified as clay, slit, sand, gravel, cobbles and boulders.

1.	Clay	$d < 2\mu$
2.	Silt	$2\mu < d < 75\ \mu$
→	Fine silt	$2\mu < d < 10\ \mu$
→	Medium silt	$10\mu < d < 20\ \mu$
→	Coarse silt	$20\mu < d < 75\ \mu$
3.	Sand	$75\mu < d < 4.75$ mm
→	Fine sand	$75\mu < d < 0.425$ mm
→	Medium sand	$0.425m < d < 2$ mm
→	Coarse sand	$2mm < d < 4.75$ mm
4.	Gravel	$4.75mm < d < 80$ mm
→	Fine gravel	$4.75mm < d < 20$ mm
→	Coarse gravel	$20mm < d < 80$ mm
5.	Cobbles	$80mm < d < 300$ mm
6.	Boulders	$d > 300$ mm

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3. Highway Research Board Classification System

→ This method is generally used for pavement construction.

→ In this method soil is divided into 7 groups which are further divided into several subgroups, on the basis of group index.

→ Higher is the value of group index, lower is the quality of material when used for pavement construction

→ Group index is dependent upon percentage passing through 75 μ, liquid limit and plasticity index of soil.

$$G.I. = 0.2a + 0.01\,bd + 0.005\,Ca$$

Where,

a = Portion of percentage passing through 75μ sieve greater than 35 but not exceeding 75, expressed as a whole number between 0 to 40.

b = Portion of percentage passing through 75μ sieve greater than 15 not exceeding 55, expressed between 0 to 40.

c = Portion of numerical liquid limit greater than 40 but not exceeding 60, expressed between 0 to 20.

d = Portion of numerical plasticity index grater than 10 but not exceeding 30, expressed between 0 to 20.

a	35	75	75μ (passing)
b	15	55	75μ (passing)
c	40	60	Liquid limit
d	10	30	Plasticity index

4. Indian Standard Classification System

→ In this system classification of soil is done on the basis of particle size distribution, plasticity characteristics and its compressibility.

→ In this system, soil is broadly classified into coarse grained soil and fine grained soil.

A. Coarse Grained Soil :

1. A soil is termed as coarse grained if 50% or more ($\geq 50\%$) soil fraction are greater than 0.075 mm or retained over 0.075 mm sieve (75μ)

2. Coarse grained soil is further classified into gravel and sand

(a) if gravel fraction (4.75 mm < d < 80mm) $\geq$ sand fraction (75μ < d < 4.75mm)
⇒ soil is gravel (G)

→ if sand fraction (75μ < d < 4.75 mm) > Gravel fraction (4.75 mm < d < 80 mm)
⇒ soil is sand (S)

3. In this system, coarse grained soil is further classified on the basis of % fineness (percentage of particles finer than 0.075 mm are termed as % fineness)

Case (a) : When % fineness is less than 5%

G → GW : Well Graded gravel : $C_u > 4$ & $1 \leq C_c \leq 3$; G > S

G → GP : Poorly Graded gravel : $C_u \not> 4$ & $C_c < 1$ or $C_c > 3$; G > S

S →
- SW : Well Graded Sand : $C_u > 6$ & $1 \le C_C \le 3$; S > G
- SP : Poorly Graded Sand : $C_u \not> 6$ & $C_C < 1$ or $C_C > 3$; S > G

Case (b) : When % fineness is greater than 12%

G Gravel
- GC : Clayey Gravel : $I_P > 7\%$ or above A-line; G > S; C > M
- GM : Silty Gravel : $I_P < 4\%$ or Below A-line; G > S; M > C
- GM : GC : Silt ~ clay $4\% \le I_P \le 7\%$

S(sand)
- SC : Clayey Sand : $I_P > 7\%$ or above A-line; S ≥ G; C > M
- SM : Silty Sand : $I_P < 4\%$ or Below A-line; S > G; M > C
- SM : SC : Silt ~ Clay $4\% \le I_P \le 7\%$; S > G; M ~ C

Case (c) : When % fineness is in between 5% to 12%

S
- SW-SC : Well graded Sand with clay
  $C_u > 6$, & $1 \le C_C \le 3$; $I_P > 7$ (or) Above A-line
- SW-SM : Well graded Sand with silt
  $C_u > 6$, & $1 \le C_C \le 3$; $I_P < 4$ (or) Below A-line
- SP-SC : Poorly graded Sand with clay
  $C_u \not> 6$, & $C_C < 1$ or $C_C > 3$; $I_P > 7$ (or) Above A-line
- SP-SM : Poorly graded Sand with silt
  $C_u \not> 6$, & $C_C < 1$ or $C_C > 3$; $I_P < 4$ (or) Below A-line

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B. Fine grained soil :-

1. A soil is termed as fine grained soil, if more than 50% of the soil fraction passes through the 75μ sieve or have size smaller than 75μ
2. Fine grained soil is further classified into clay (C), silt (M) and organic soil (O) [Organic clay or Organic silt]
3. In this classification system, fine grained soil is further classified on the basis of compressibility as

Low compressible (L) : $W_L < 35\%$

Intermediate compressible (I) : $35 \le W_L < 50\%$

High compressible (H) : $W_L > 50\%$

CL : Low compressible clay

CI : Intermediate compressible clay

CH : High compresible clay

ML : Low compressible silt

MI : Intermediate compressible silt

MH : High compressible silt

OL : Low compressible organic soil

OI : Intermediate compressible organic soil

OH : High compressible organic soil

→ In order to separate inorganic clay from silt and organic soil, A. Casagrande defined a term A-line which represents the relationship between plasticity index and liquid limit of soil.

→ Soil above A - line is clay and soil below A - line is silt (or) organic soil.

→ Equation of A - line

$I_P = 0.73 (w_L - 20)$

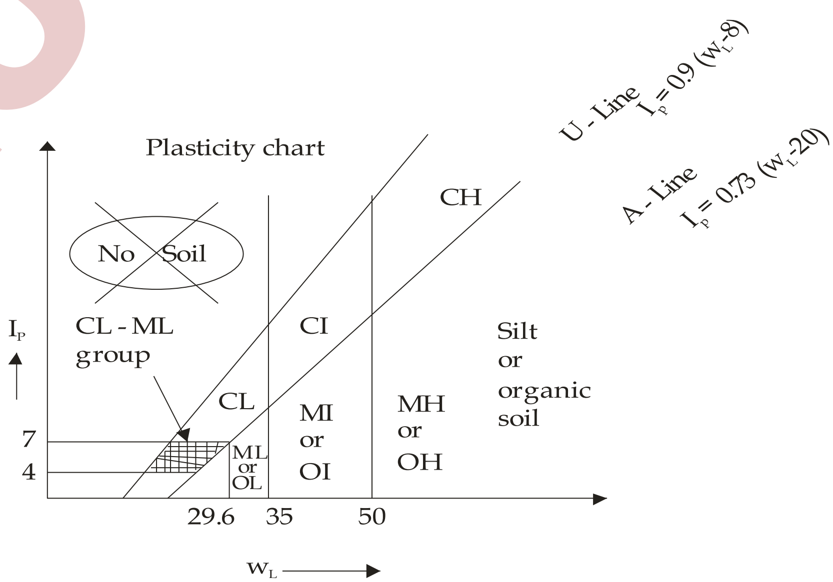
w_L – Liquid limit in %

→ In order to mark the presence of upper of soil, a line termed as U- line is defined.

→ No soil is found to exist above U-line

→ Equation of U - line,

$I_P = 0.9 (w_L - 8)$



Chapter - 4 (Soil structure and Clay Mineralogy)

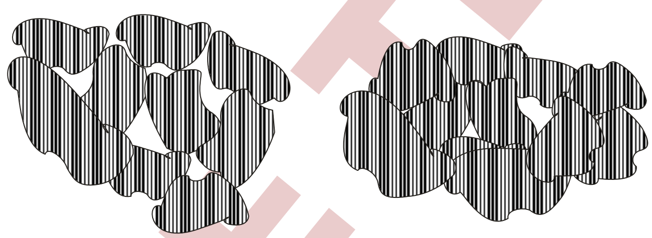
Soil Structure and Fibres

The arrangement of soil particles in soil mass is termed as structure of soil. A given sample of soil may have following types of structure.

→ Single Grained structure
→ Honey comb structure
→ Flocculent & Dispersed Structure

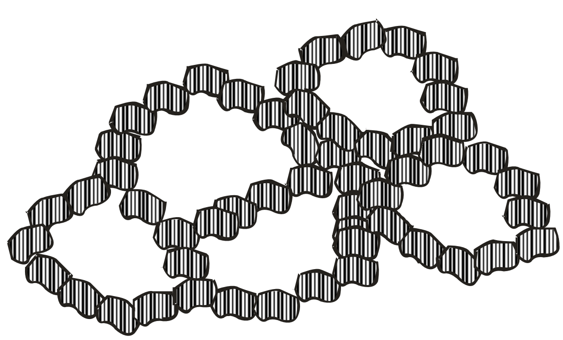
1. Single Grained Structure / Coarse Grained Structure

→ This type of structure is found for coarse grained soil having size greater than 0.02mm (Sand, gravel and coarse silt)
→ In the formation of this structure gravitational forces play the predominating role and surface electric force is insignificant as a result of which soil particles settles down under the effect of their own weight, leading to particle to particle contact. The arrangement may be dense or loose



2. **Honey Comb Structure**

→ This type of structure is found for the soil having particle size in the range of 0.0002mm (0.2 micron) to 0.02mm (silt and clay)
→ In the formation of this structure both gravitational force and surface electric force play equally important role, when solids rolled down under the effect of their own weight during deposition, surface electric force holds the solids in contact with each other in the form of cluster of honeycomb enclosing large volume of voids in between them.
→ If the structure is unbroken, load carrying capacity is comparatively high along with high permeability but once the structure is broken, load carrying capacity and permeability both reduces



3. Flocculated and Dispersed Structure

→ This type of structure is found, if size of particle is less than 0.0002mm (0.2 micron). e.g. fine clays and colloids

→ In the formation of these structure, surface electric force plays a predominating role and gravitational force is insignificant.

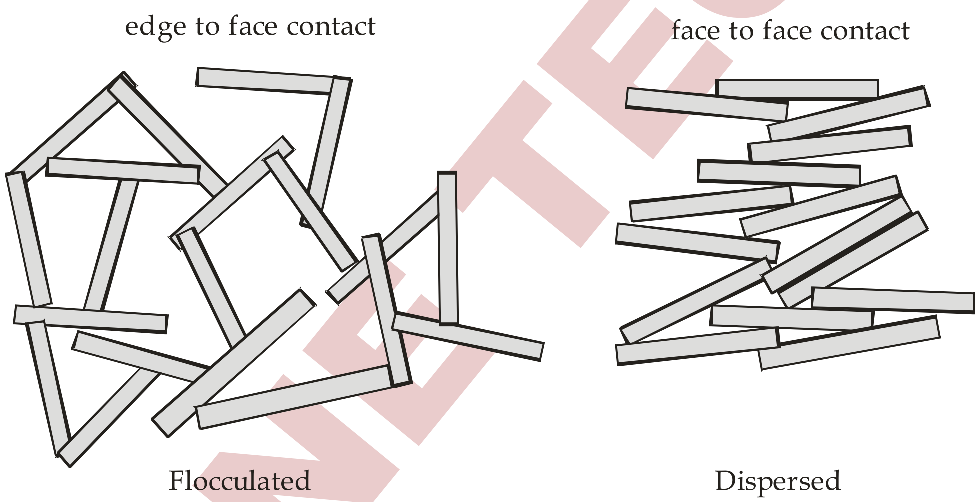
→ These fine grained particle in which these type of structure are observed are flaky in nature and carries surface electric charges which are generally positive at the edges and negative at the centre.

→ Depending upon the nature of net resultant force (attractive or repulsive) flocculated or dispersed structure may be formed.

Flocculated Structure

→ If the net force during the deposition are attractive, it leads to the formation of flocculated structure in which clay platelets are aligned in head to head (or) head to face orientation.

→ Soil having flocculated structure is found to have high strength, high permeability and low compressibility (when structure is unbroken)



Dispersed Structure

→ If the net force during deposition are repulsive, it results in the formation of dispersed structure.

→ In dispersed structure, clay platelets are aligned face to face orientation.

→ Soil having dispersed structure possesses lower strength, lower permeability and high compressibility in comparison to flocculated structure.

Note :- marine clay is found to have flocculated structure (in sea water ion concentration is high) whereas lacustrine clay (fresh water, no iron) is found to have dispersed structure.

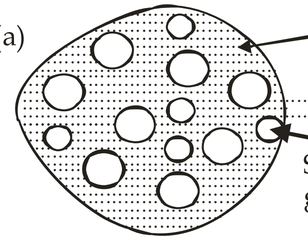
Structure of Composite Soils

→ Structure of composite soil depends upon relative proportion of coarse grained and fine grained particles present in it according to which it may be classified as ;

(a) Cohesive Matrix Structure

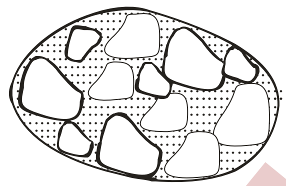
→ In this structure, the relative proportion of fine grained particles is more than coarse grained particles such that coarse grained particles are not in contact with each other.

→ In this type of structure, fine grained particle acts as load bearing members and results in the formation of highly compressible soil.



(a) Cohesive

Matrix Structure



(b) Coarse grained

Soil Skeleton

(b) Coarse grained Skeleton Structure

→ If the relative proportion of coarse grained particles is more than fine grained, such that fine grained particles occupy empty void spaces between coarse grained particles which are in contact with each other.

→ In this type of structure, coarse grained particles act as load bearing members and results in the formation of low compressible soil.

Clay Mineralogy

→ Clay mineral are formed form two basic structural units : tetrahedral and octaheral. Considering the valencies of the atoms forming the units, it is clear that the units are not electrically neutral and as such do not exist as single units.

→ The basic units combine to form sheets in which the oxygen or hydroxyl ions are shared among adjacent units. Three types of sheets are thus generally formed, namely silica sheet, gibbsite sheet and brucite sheet.

→ Isomorphous substitution is the replacement of the central atom of the tetrahedral or octahedral unit by another atom during the formation of the sheets.

→ These sheets then combine to form various two-layer or three-layer sheet minerals.

All the clay minerals are found to have following two fundamental building blocks

1. Silica Tetra-Hedral Unit :

→ A tetrahedral unit consists of a central silicon atom that is surrounded by four oxygen atoms located at the corners of a tetrahedron. All the oxygen at base of tetrahedron lies in same common plane and is being shared in between two tetrahedral units. A combination of tetrahedrons forms a silica sheet.

→ Net charge present over silica tetra hedron unit is - 1

→ Tetra hedral unit is symbolically represented as trapezium.

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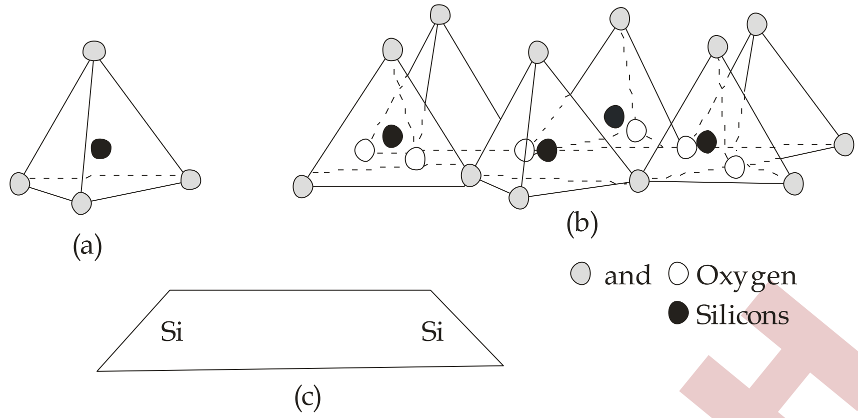


(a) Silica tetrahedron

(b) Silica tetrahedral sheet

(c) Schematic of silica sheet

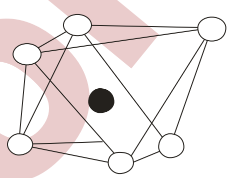
Octahedral Unit

→ An octahedral unit consists of a central ion, either aluminium or magnesium or iron, that is surrounded by six hydroxyl ions located at the corners of an octahedron. A combination of aluminium-hydroxyl octahedrons forms a gibbsite sheet, whereas a combination of megnesium-hydroxyl octahedrons forms a brucite sheet and combination of iron-hydroxyl forms a Ferrite sheet.

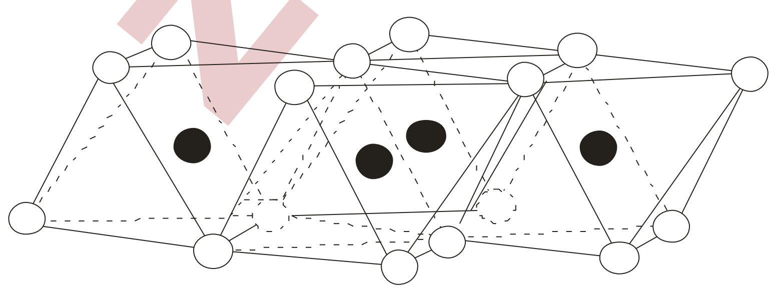
→ Each hydroxyl atom is being shared in between three octahedral unit

→ Net charge present over gibbsite unit is +1

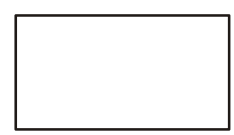
→ Octahedron unit is represented symbolically as rectangle



Octahedral unit



Octahedral Sheet



Representation of octahedral unit

These two fundamental building blocks join with each other in different geometry resulting in formation of different clay minerals

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1. **Kaolinite Mineral**

→ The structural unit is formed when octahedron unit combines with Silica-tetrahedron unit having thickness of 7Å.

→ These basic units are then stacked one on top of the other to form a lattice of the mineral. The units are held together by hydrogen bonds. The strong bonding does not permit water to enter the lattice.Thus, kaolinite minerals are stable and shows lesser swelling & shrinkage properties.

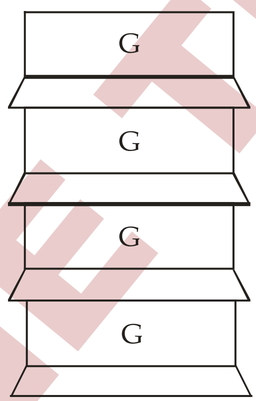
→ The total thickness of these layers may extend upto 50-2000 nanometer.

→ China clay is the example of kaolinite

→ Halloysite is the another mineral of the kaolinite group.

Note :

The kaolinite mineral is electrically neutral. However, in the presence of water, kaolinite may show negative charge.



2. **Montmorillonite (Smectite)**

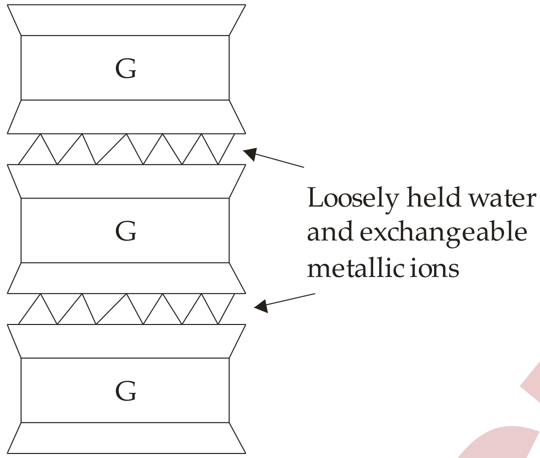
→ It is the most common mineral of clay whose structure is formed when gibbsite unit is sandwiched between two silica tetrahedron unit resulting in structure having thickness of 10Å

→ In this case, silica in tetrahedron unit, in some cases, is replaced by aluminium atom which results in development of net. negative charge that is balanced by any of the cation (Ca+, Na+) present in mineral.

→ Several such structural units of montmorillonite are joined with each other by weak water bond which can be easily displace off as the result of which, soils having montmorillonite mineral show high swelling and shrinkage characteristics.

→ the total thickness of these mineral layers is limited to 3 nanometer.

→ Black cotton soil is found to have excess of montmorillonite.



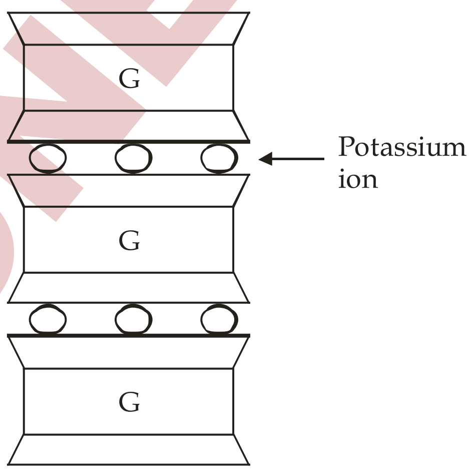
Illite mineral

→ The structure of illite is same as that of montmorillonite with the only difference that substantial (>20%) replacement of silica in tetrahedron unit by aluminium atom takes place and resultant negative charge is balanced by potassium ion (K^+)

→ Several structural unit of illite are joined by ionic bond which is stronger than water bond of montmorillonite but weaker than hydrogen bond of kaolinite, as the result of which it shows intermediate swelling and shrinkage characteristics.

→ The total thickness of the layers in this case extends upto 30 nanometers.

→ For example laterite.



Soil water

Water present in soil in any form is termed as soil water.

1. Ground water

→ It is the sub surface water which fills the voids of the soil continuously upto the GWT level.

→ This water is subjected to no other force than gravitational force. Hence it is also termed as gravity water or free water.

→ This water obeys all the laws of hydraulics

2. Adsorbed Water

A. Hygroscopic water **B. Film water**

A. Hygroscopic water

→ It is the water which is being adsorbed by the soil solids from the atmosphere by physical forces of attraction and is held over the surface due to adhesion.

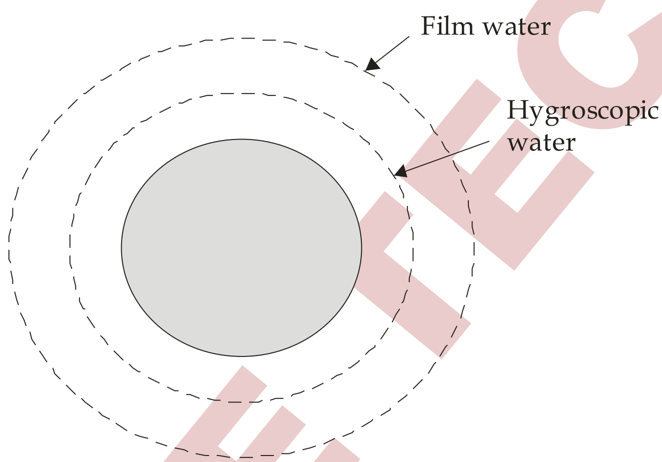
→ The water adsorption capacity depends on the size of the solids present in the soil. It is more for fine grained soil in comparison to coarse grained soil due to more specific surface area.

→ Water adsorption capacity for clay 16 to 17%

→ Water adsorption capacity for silt 6 to 7%

→ Water adsorption capacity for sand < 1%

→ **Film Water :** It is also adsorbed water which is formed due to condensation of aqueous vapour on the layer of the hygroscopic water.



3. Structural Water

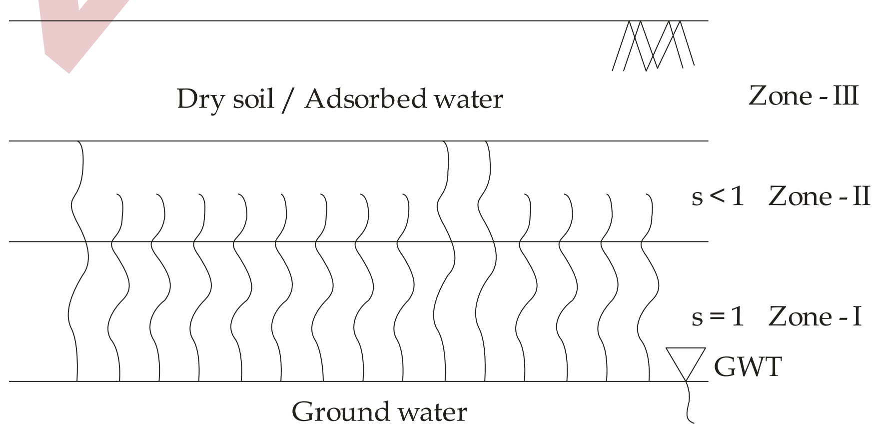
→ It is the water which is combined chemically to the crystal structure of the soil mineral.

→ Under normal engineering activities this water cannot be separated or removed. Hence it is of no significance.

4. Capillary Water

→ It is the water which is being lifted by surface tension forces above the free water surface level.

→ This water is present in the suspension in the voids of soil and fills it upto a certain distance above the GWT known as zone of capillary saturation.



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Zone - I : Zone of capillary saturation

Zone - II : Zone of partially capillary saturation

I. The size of voids is assumed to be 20% of effective size (D_{10}) to soil solids

$$d = 20\% \text{ of } D_{10} = 0.2\,D_{10} = \frac{D_{10}}{5}$$

$$h_{c\,max} = \frac{4T_s}{\gamma_\omega \dfrac{D_{10}}{5}}$$

$$h_{c\,max} = \frac{20T_s}{\gamma_\omega D_{10}}$$

II. Allen Hazon equation

$$H_{c\,max} = \frac{C}{e.D_{10}}$$

e - Void ratio ; D_{10} - effective size (cm);

C - Constant = 0.1 to 0.5 cm^2

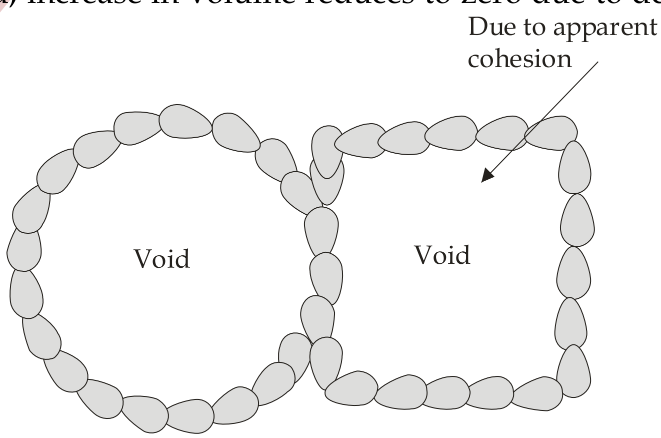
Type of soil	Capillary rise (cm)
Gravel	2 to 10 cm
Sand	10 to 100
Silt	100 to 1000
Clay	1000 to 3000

6. Bulking of Sand

→ When dry sample of sand is slightly moist and dumped loosely, its volume increase relative to its dry volume.

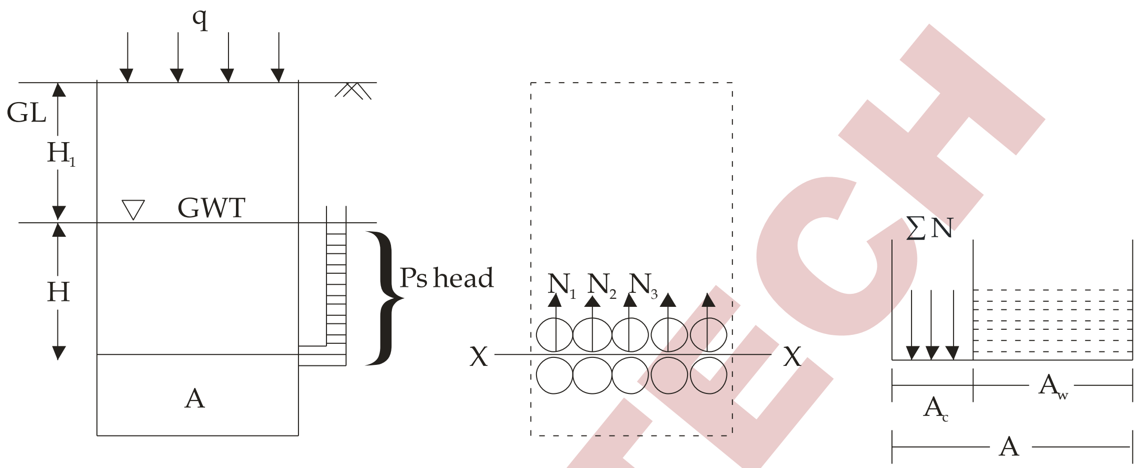
→ On the account of apparent cohesion which holds the soil solids in contact with each other in the form of cluster of honey comb enclosing large volume of voids in between them.

→ This increase in volume of sand is found upto particular water content of 4% to 5% beyond which if water content in increased, radius of meniscus starts increasing, leads into decrease in apparent cohesion, which in turn results in decrease of volume of sand. When sand becomes completely saturated, increase in volume reduces to zero due to destruction of mensicus.



CHAPTER - 5 (EFFECTIVE STRESS)

Effective Stress :--



Total stress (σ) $= \dfrac{\text{total wt}}{\text{Area}} = \dfrac{\text{wt of (solids + water)}}{\text{Area}}$

$$\sigma = \frac{\text{Vol} \times \text{unit wt}}{\text{Area}} = \frac{A \times h \times \gamma}{A} = h \times \gamma$$

$$\sigma = h\gamma + q \quad \text{(if uniform surcharge q)}$$

At plane X – X

$$\text{Total stress } (\sigma) = \gamma_b H_1 + \gamma_{sat} \times H$$

$$\sigma = \gamma_b H_1 + \gamma_{sat} \times H + q \rightarrow \text{if surcharge} = q$$

Total stress = Eff. stress + pore water pr. (pwp)

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

$\rightarrow$ Pore water pr $= \dfrac{\text{wt. of water}}{\text{Area}} = \dfrac{\text{Vol} \times \gamma_w}{\text{Area}}$

$$u = \frac{A_w \times H \times \gamma_w}{A_w} = H.\gamma_w$$

Eff. stress $\sigma = \sigma - u$

$$\bar{\sigma} = (\gamma_b H_1 + \gamma_{sat.} H) - \gamma_w H \quad [\gamma' = \gamma_{sat} - \gamma_w]$$

$$\bar{\sigma} = \gamma_b H_1 + \gamma' H$$

OR

$$\bar{\sigma} = q + \gamma_b H_1 + \gamma' H$$

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Total Stress :-

→ At any given plane section in the soil mass, total stress is due to the self wt. of the soil $\left[\text{wt. of solids} + \text{water} + \cancel{\text{air}}^{0}\right]$ or due to the applied over burden pressure [uniform surcharge]

→ The total stress further consists of 2 diff. components

(i) Eff. stress [intergranular pressure]

(ii) Pore water pressure [neutral pressure]

Total stress = eff stress + PWP

Effective stress :-

It is the stress which is being transfered in soil mass by grain-to-grain contact which tends to force the particles to come into closer state of contact resulting in its decreased void ratio, increased degree of denseness & mobilization of shear strength. These stress are also termed as inter-granular stress.

→ Inter granular stress is really a misnomer ∵ the effective stress is really not the stress at particle contacts. Actual contact stress can be very high since area of contact between particle is very small.

eff stress = sum of contact force divided by gross area

Pore water pressure (u)

→ It is the pressure which is being transmitted by pore fluid & is equal to the wt. of fluid (water column) above the concerned section in soil mass.

→ This water pr. acts all around the soil solid hence does not tend to force the soil solids into closer state of contact. It doesnot have any shear component.

→ It can also be determined by the help of piezometer as pressure head × γ_w

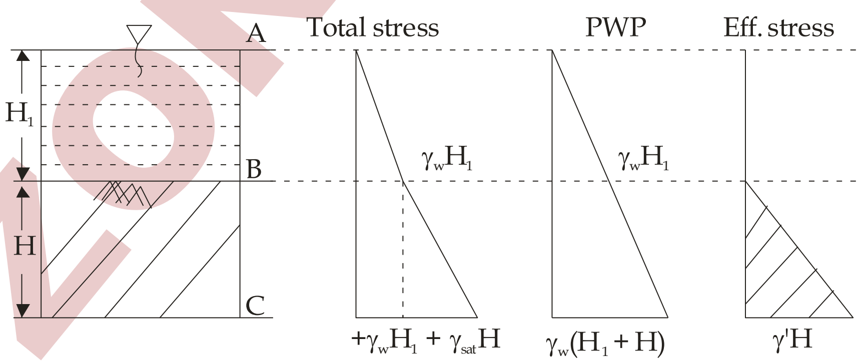
u = Pr. head × γ_w

→ It is also termed as neutral pressure

Note :- The shear strength, bearing capacity, consolidation all depend upon effective stress

Case -I Submerged soil mass

i.e water table is above GL



Pt A	**Pt B**	**PtC**
$\sigma = 0$	$\sigma = \gamma_w H_1$	$\sigma = \gamma_w H_1 + \gamma_{sat} H$
$u = 0$	$u = \gamma_w H_1$	$u = \gamma_w (H_1 + H)$
$\bar{\sigma} = \sigma - u = 0$	$\bar{\sigma} = \sigma - u = 0$	$\bar{\sigma} = \sigma - u$

$= \gamma_w H_1 + \gamma_{sat} H - \gamma_w H_1 - \gamma_w H$

$= \gamma_{sat} H - \gamma_w H$

$\bar{\sigma} = \gamma' H$

Note :-

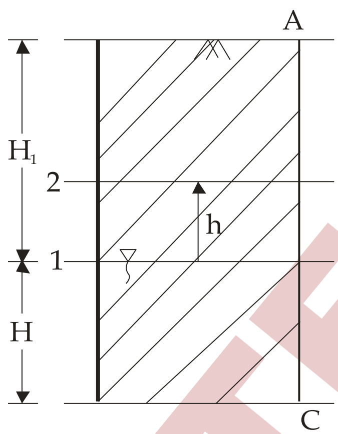
→ If the water table is above the ground level the variation in GWT level will not affect the effective stress

Eg : Lakes, Ponds, Rivers etc.

→ But if the water table is below the ground level its variation will affect the effective stress

Case - 2:

Water table is rising [GWT is below the GL]



– Effective stress at C when GWT is at (1) - (1)

$$(\bar{\sigma}_C)_{1-1} = \sigma - u$$

$$= (\gamma_b H_1 + \gamma_{sat} H) - \gamma_w H$$

$$(\bar{\sigma}_c)_H = \gamma_b H_1 + \gamma^1 H$$

– Effective stress at C when GWT is at 2 - 2

$$(\bar{\sigma}_c)_{2-2} = \sigma - u$$

$$= \gamma_b [H_1 - h] + \gamma_{sat} [H + h] - \gamma_w (H + h)$$

$$= \gamma_b [H_1 - h] + [\gamma_{sat} - \gamma_w][H + h]$$

$$= \gamma_b [H_1 - h] + \gamma^1 (H + h)$$

Change in effective stress

$$\Delta\bar{\sigma} = (\bar{\sigma}_c)_{2-2} - (\bar{\sigma}_c)_{1-1}$$

$$= \gamma_b (H_1 - h) + \gamma'[H + h] - [\gamma_b H_1 + \gamma' H]$$

$$\Delta\sigma \Rightarrow (\gamma' - \gamma_b) h$$

As $\gamma' < \gamma_b$ $\therefore \Delta\bar{\sigma} = -ive$

Conclusion :-

Eff. stress will reduce with ↑in GWT, if the GWT is below the ground level and vice versa.

If GWT is lowered due to pumping out of water then

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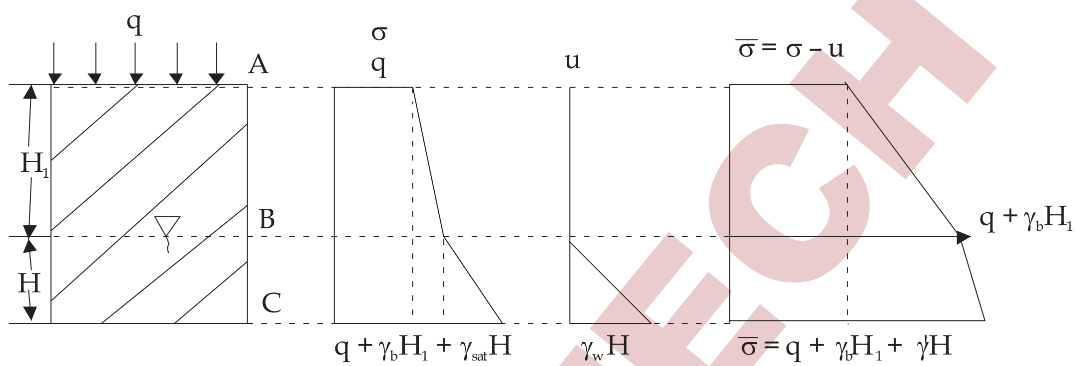
(a) Short Term :- Soil will be treated as saturated in lowered water table zone

$$\Delta\bar{\sigma} = \left(\gamma_{sat} - \gamma^1\right)h$$

(b) **Long Term :-** After a long time - soil will achieve bulk conditon in lowered water table zone

$$\Delta\sigma = \left(\gamma_b - \gamma^1\right)H$$

Case -3 - Soil mass with uniform surcharge and water table is below the depth H_1 below the ground level



Point A

$\sigma = q$

u = 0

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

Point B

$\sigma = q + \gamma_b.H_1$

u = 0

$\bar{\sigma} = \sigma - u = q + \gamma_b.H_1$

Point C

$\sigma = q + \gamma_b H_1 + \gamma_{sat}.H$

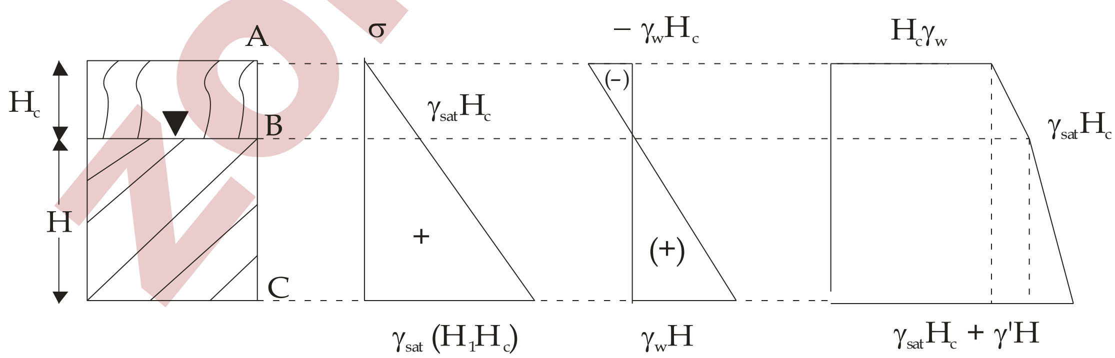
$u = \gamma_w H$

$\bar{\sigma} = \sigma - u = q + \gamma_b.H_1 + \gamma_{sat}H - \gamma_w H$

$\bar{\sigma} = \sigma + +\gamma_b H_1 + \gamma^1 H$

Case -4 - Soil mass with capillary fringes :

Soil is saturated above GWT due to capillary effect.



Point A

$\sigma = 0$

$\mu = -\gamma_w H_C$

$\bar{\sigma} = \sigma - u = 0 - \left(-\gamma_w H_C\right) = \gamma_w H_C$

Point B

$\sigma = \gamma_{sat}.H_C$

u = 0

$\bar{\sigma} = \gamma_{sat} H_C - 0 = \gamma_{sat} H_C$

Point C

$\sigma = \gamma_{sat}\left(H + H_C\right)$

$u = \gamma_w H$

$\bar{\sigma} = \gamma_{sat} H + \gamma_{sat} H_C - \gamma_w H$

$= \gamma_{sat} H + \gamma^1 H$

Notes :-

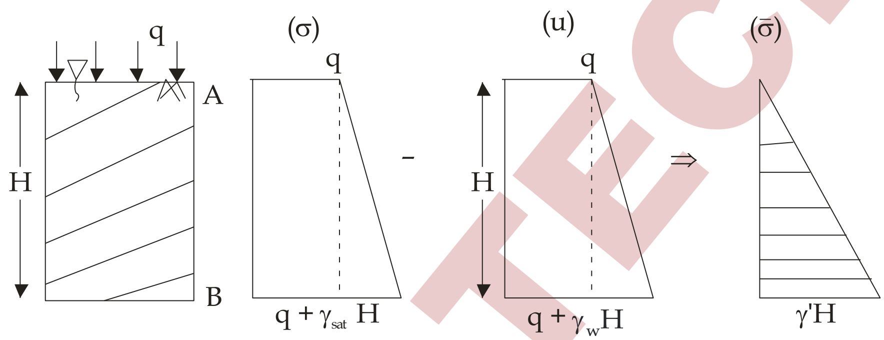
1. The effect of capillarity is same as that of surcharge $\left[q = \gamma_w H_C\right]$ it helps in increasing effective stress

→ If the soil would have been saturated above pt B due to ground water instead of capillary water, effective stress in soil would have decreased by $H_C\gamma_w$ due to increased pore water pressure.

Due to capillarity effective stress increases due to increases in effective stress shear strength also increases.

Case -5 : Surcharge is applied on soil mass & GWT @ ground level

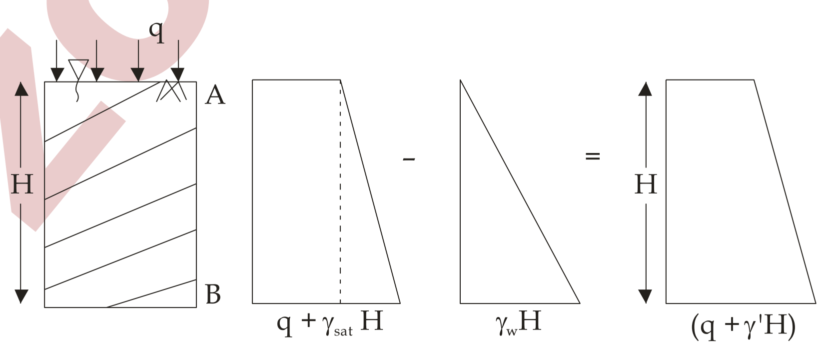
Case -A : Surcharge is applied suddenly



Point A	**Point B**
$\sigma = q$	$\sigma = q + \gamma_{sat} \times H$
$u = q$	$u = q + \gamma_w H$
$\bar{\sigma} = \sigma - u = 0$	$\bar{\sigma} = \sigma - u = \gamma^1 H$

Just after application of load all the extra stress is taken by water

Case -(b) When surcharge is applied gradually



Pt A	**Pt B**
$\sigma = q$	$\sigma = \gamma_{sat} H + q$
$u = 0$	$u = \gamma_w H$
$\bar{\sigma} = \sigma - u = q$	$\bar{\sigma} = q + \gamma^1 H$

Note :-

When water table is present at ground level and surcharge is applied gradually, it is being carried by soil solids.

But if it is applied suddenly, it is being carried by water in it which causes increase in PWP. That causes water to support from voids of soil during which water transfer a part of surcharge being carried by it on the soil solids and when increase PWP is being dissipated then toal surcharge is being carried by soil solids.

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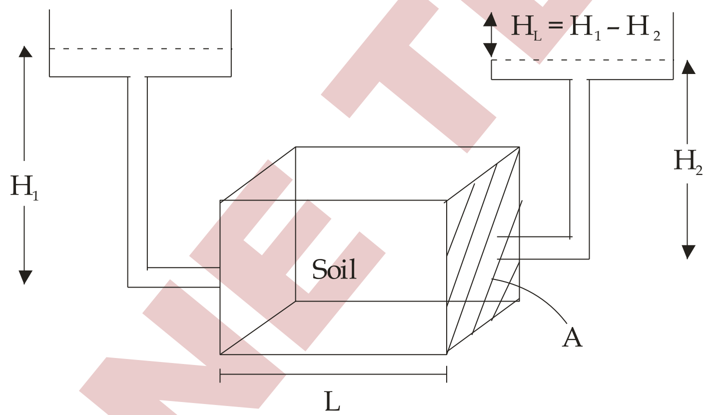
Chapter - 6 (Permeability of Soil)

Permeability :-

Permeability is the property of soil by virtue of which it allows the flow of fluid through it. Permeability is also termed as hydraulic conductivity.

Type of soil	Permeability (cm/sec)
Gravel	1 cm/s
Sand	$1 - 10^{-3}$ cm/sec
Silt	$10^{-3} - 10^{-7}$ cm/sec
Clay	$< 10^{-7}$ cm/sec

Darcy's Law



→ Permeability of coarse grained soil is more than that of fine grained soil

$$\text{Hydraulic gradient} = \frac{\text{seepage head}}{\text{Length}} = \frac{\text{Head loss}}{\text{Length}}$$

$$i = \frac{H_1 - H_2}{L} = \frac{H_L}{L}$$

As per darcy, for laminar flow in saturated soil mass, velocity of flow is directly proportional to the hydraulic gradient

$$V \propto i$$

$$V = ki$$

$$k = \text{coefficient of permeability} \left(\frac{cm}{sec}, \frac{m}{s}\right)$$

$$\text{Discharge } q = A \times V$$

$$q = kiA \left(\frac{m^3}{sec}\right)$$

Note :- The velocity of flow considered above is average or discharge velocity because total area of X-section is considered. But in actual, flow takes place through the interconnecting voids of medium, of voids which is much less than total area of X-section of flow, hence actual / true / seepage velocity of flow is much more than average/discharge velocity considered above.

$$\text{discharge } q = A \times V = A_V . V_S$$

$$\frac{V}{V_S} = \frac{A_V}{A} = \frac{A_V \times L}{A \times L} = n$$

Seepage velocity

$$V_S = \frac{V}{n} \qquad n \rightarrow \text{porosity}$$

$$0 < n < 1$$

$$V_S > V$$

Relation between K_p & K

$$V_S \; \alpha \; i$$

$$V_S = K_p \times i$$

K_p = coefficient of percolation (Actual coefficient of permeability)

$\because$ $$V = Ki,\ V_S = \frac{V}{n} \ \&\ V_S = K_p i$$

$$V_S = K_p i \Rightarrow \frac{V}{n} = \frac{Ki}{n}$$

$$K_p = \frac{K}{n}$$

Note :- Darcy's law is not valid in case of gravel because flow through them is turbulent flow. Reynolds no for laminar flow in soil ≤ 1

Methods to determine permeability

Lab Method

(A) Constant head permeability test

(B) Variable head permeability test

Indirect Methods

(a) Kozney carman equation

(b) Allen Hazen equation

(c) Consolidation equation

(d) Loudon's equation

(e) Terzaghi's equation

Field methods

(a) Pumping out test (suitable for large area of influence)

(A) Constant Head Permeability Test

→ This method is generally used for coarse grained soil for which substantial discharge can be obtained during the test in given small time.

→ In this method, water is allowed to flow through the medium under constant head & volume of water flowing through the soil mass is collected in a vessel in time 't'

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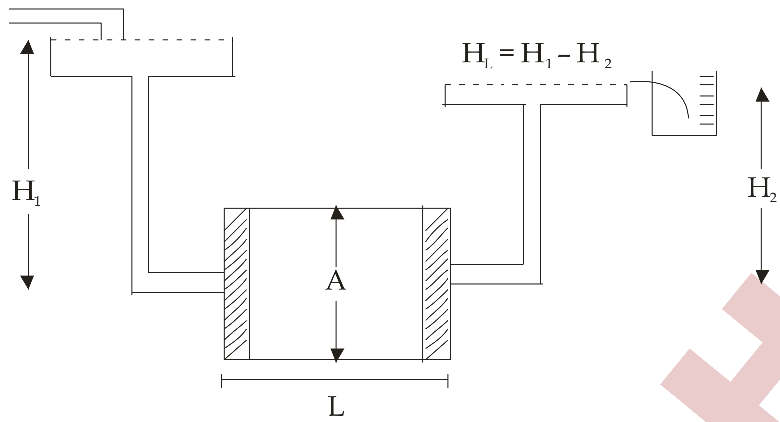


$$q = \frac{V}{t} = \frac{\text{Volume}}{\text{time}}$$

As per darey q = KiA

$$\frac{V}{t} = KiA$$

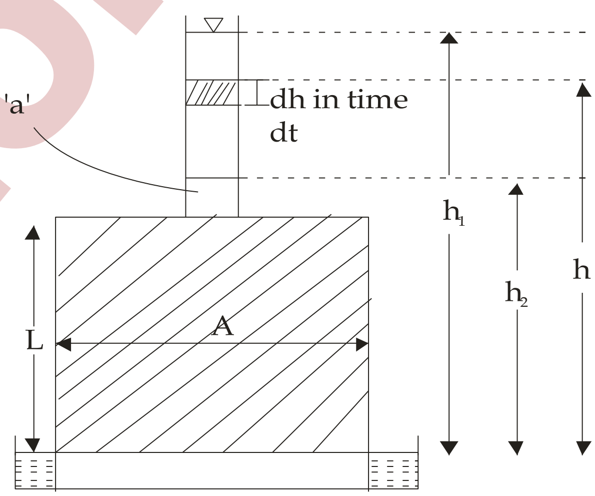
$$k = \frac{V}{t.i.A} = \frac{V}{t.\frac{H_L A}{L}} = \frac{V}{t.\frac{H_L A}{L}}$$

(B) Variable / Falling Head Permeability Test

→ This method is generally used for fine grained soil for which constant head method is not suitable.

→ In this method, a stand pipe of known area is inserted into the medium & water is allowed to flow through it.

→ In order to compute permeability of medium height of water in stand pipe is noted at different time interval.



→ Let at time 't' water is flowing under head 'h' & moves a distance 'dh' in time 'dt'

→ Volume of water flowing in time 'dt' (dV) = a.dh

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→ Discharge through soil mass in time 'dt' $dq = \frac{dv}{dt} = -\frac{adh}{dt}$

As per darcy dq = kiA

$$dq = k.\frac{h}{L}A = -\frac{adh}{dt}$$

$$\frac{kA}{L}\int_{t_1}^{t_2} dt = -a\int_{h_1}^{h_2}\frac{dh}{h}$$

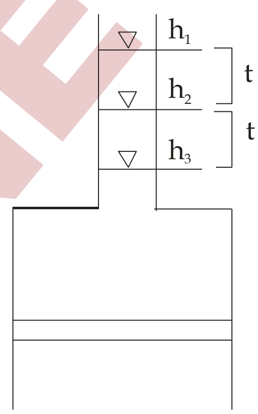
$$\frac{kA}{L}[t_2 - t_1] = \frac{kAt}{L} = -a\,ln\left[\frac{h_2}{h_1}\right] = a\,ln\left[\frac{h_1}{h_2}\right]$$

$[t_2 - t_1 = t]$ = time interval

$$k = \frac{a}{A}.\frac{L}{t}ln\frac{h_1}{h_2}$$

$$k = 2.303\ \frac{a}{A}.\frac{L}{t}.ln\left[\frac{h_1}{h_2}\right]$$

Note :- If in given time interval 't'. Height of water in stand pipe falls from h_1 to h_2 & in the same time 't' it falls from h_2 to h_3



$$K = 2.303\frac{a}{A}\frac{L}{t}\log_{10}\left(\frac{h_1}{h_2}\right) = 2.303.\frac{a}{A}\frac{L}{t}\log_{10}\left(\frac{h_2}{h_3}\right)$$

$$\frac{h_1}{h_2} = \frac{h_2}{h_3}$$

$$h_2^2 = h_1h_3$$

$$h_2 = \sqrt{h_1h_3}$$

→ This method is used to check the consistency of readings.

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Indirect Methods

(1) Kozney Carman Equation

$$K = \frac{1}{K_k} \cdot \left(\frac{\gamma}{\mu}\right)_{fluid} \left(\frac{e^3}{1+e}\right) . d^2$$

$$K = \frac{1}{K_k'} \cdot \left(\frac{\gamma}{\mu}\right)_{fluid} \frac{e^3}{1+e} \cdot \frac{1}{S_s^2}$$

Here K_k & K_k' are constant

γ, μ = unit weight and dynamic viscosity of fluid

e = void ratio of soil

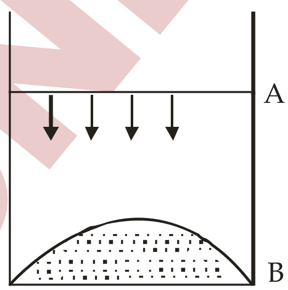
d = particle size of soil mas

S_S = specific surface area = surface area per unit volume

$$S_S = \frac{4\pi r^2}{\frac{4}{3}\pi r^3} = \frac{3}{r} = \frac{6}{d}$$

$$S_S \propto \frac{1}{d}$$

→ If particles are not spherical and passing through sieve of size A and retained on sieve of size B.



$$S_S = \frac{6}{\sqrt{A.B}}$$

S_S = specific surface area

(2) Allen - Hazen Equation :-

$$K = C \times D_{10}^2$$

$K = 100 \times D_{10}^2$ where K in cm/sec, D_{10} in cm

(3) Consolidation Equation :-

$$K = C_V . m_v . \gamma_w$$

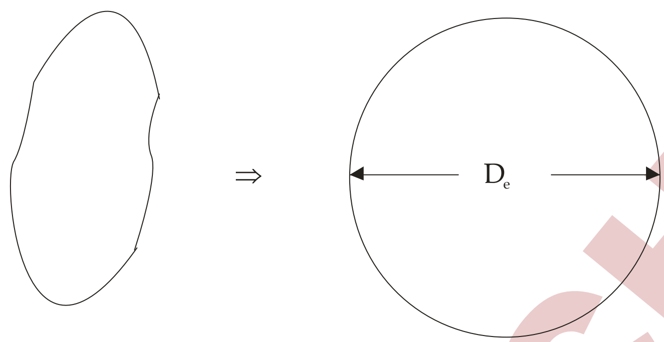
C_V = coefficient of consolidation

m_V = coefficient of volume compressibility

(4) Terzaghi equation

$K = 200\, e^2\, D_e^{\,2}$

D_e = size of sphere for which ratio of volume to surface area is same for given soil particle.



$$(S_s)_{particle} = (S_s)_{sphere} = \frac{6}{D_e}$$

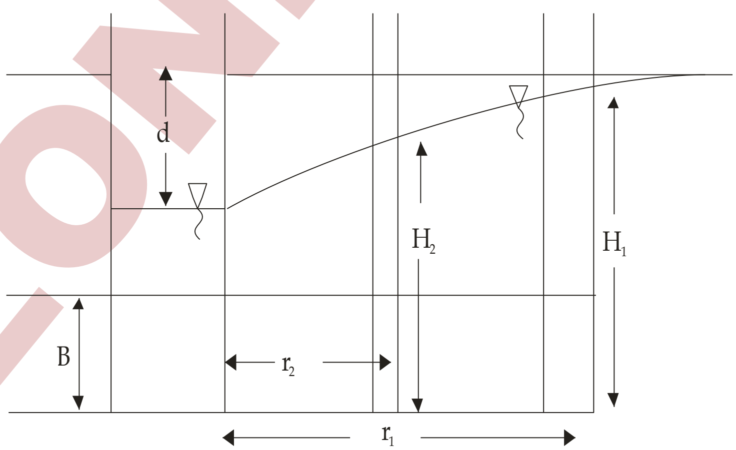
(5) Loudon's Equaion

$$\text{Log}_{10}\left(KS_C^2\right) = a + b_n$$

a = 1.3, b = 5.1 , n $\rightarrow$ porosity

Field Methods :-

(A) Pumping out test:- done only for large projects. This method is suitable generally for large area of influence. In this method water is pumped out of the medium in any of the form to analyse permeability of homogenous, coarse grained soil.



Well in unconfined aquifer

(1) Thiem's Theory

$$k = \frac{2.303q.\log\left(\frac{r_1}{r_2}\right)}{\pi\left[h_1^2 - h_2^2\right]}$$

(2) Dupit's Theory

$$k = \frac{2.303q.\log\left(\frac{R}{r}\right)}{\pi\left[H^2 - h^2\right]}$$

R = Radius of influence (m)

R = $3000d\sqrt{k}$

d = drawdown(m)

k = permeability (m/s)

Well in confined aquifer

(1) Thiem's theory :-

$$k = \frac{2.303q.\log\left(\frac{r_1}{r_2}\right)}{2\pi B\left(h_1 - h_2\right)}$$

(2) Dupits theory :-

$$k = \frac{2.303q\log\left(\frac{R}{r}\right)}{2\pi B\left[H - h\right]}$$

Factors Affecting Permeability

1. Particle size :-

$$K \propto d^2$$

$$K \propto D_{10}^2$$

Permeability of coarse grained soil is more than fine grained soil.

2. Specific Surface Area

$$K \propto \frac{1}{{S_s}^2}$$

3. Void ratio :-

$$k = \frac{1}{K_k}\left(\frac{\gamma}{\mu}\right)_{fluid} \cdot \left(\frac{e^3}{1+e}\right).d^2$$

$$k \propto \frac{e^3}{1+e}$$

→ Loose sand and flocculant structure have more ratio than dense sand and dispersed structure respectively have more permeability (particle size constant)

4. Fluid properties:-

$$K \propto \left(\frac{\gamma}{\mu}\right)_{fluid}$$

$$\mu \propto \frac{1}{T} \qquad \Rightarrow \quad K \propto T$$

Note :-

It is difficult to compare and analyse the two soil samples because permeability is depenent upon both medium properties and fluid properties thus coefficient of intrinsic permeability or absolute permeability is used which is dependent only upon medium properties

$$k = \frac{1}{k_k}\left(\frac{\gamma}{\mu}\right)_{fluid}\left(\frac{e^3}{1+e}d^2\right)$$

k = f(fluid properties) f(medium properties)

Intrinsic permeability (k_0)

K_0 = f(medium property)

$$K_0 = \frac{K}{\left(\frac{\gamma}{\mu}\right)_{fluid}}$$

(5) Degree of Saturation :-

$k \propto S$:- because air resistance is reducing

(6) Entrapped gases :-

$$k \propto \frac{1}{\text{Entrapped gases}}$$

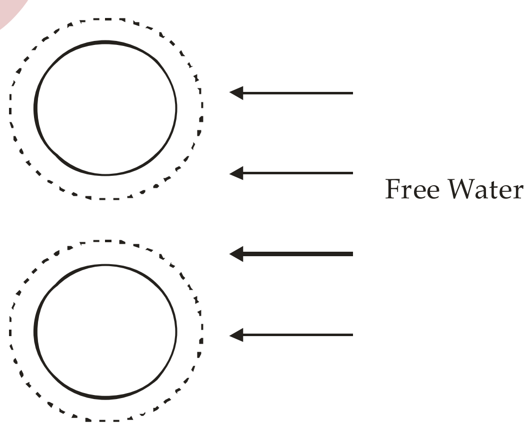
(7) Foreign impurities in water :-

$$k \propto \frac{1}{\text{foreign impurities}}$$

(8) Adsorbed water :-

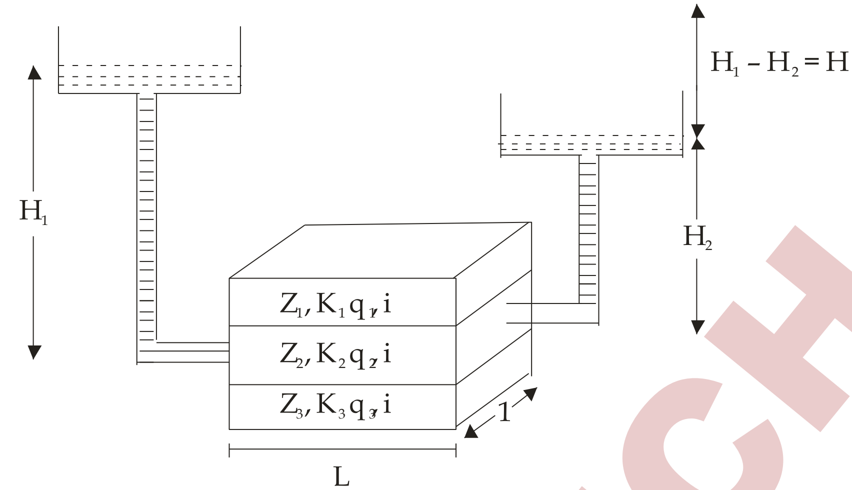
Higher is the presence of adsorbed water in the medium lower is its permeability

Since lower is the area available to the flow of fluid. It is assumed that in the presence of adsorbed water void ratio is approximately reduced by 0.1



Direction of flow through the soil:- Permeability along the bedding plane is more than the permeability normal to the bedding plane.

(A) Average permeability along the bedding plane



Total discharge $q = q_1 + q_2 + q_3$ ......

As per decay $q = K_{avg}\, i(z_1 + z_2 + z_3) \times 1$

$q_1 = K_1 i(Z_1 X1)$

$q_2 = K_2 i(Z_2)$

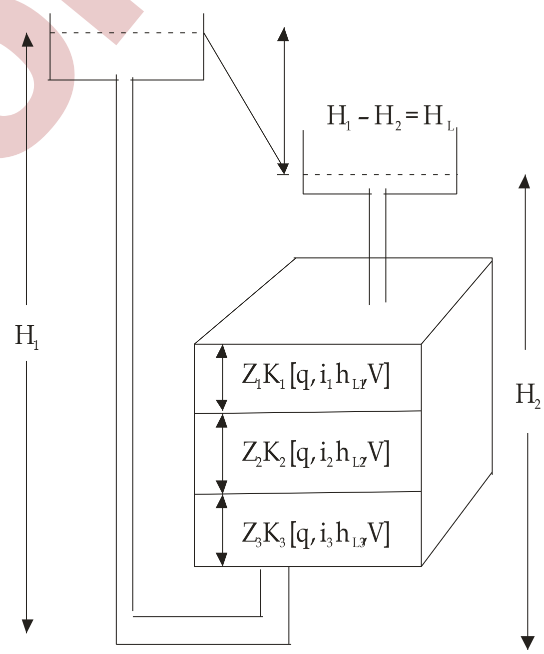
$q_3 = K_3 i(Z_3)$

$iK_{avg}\,[Z_1 + Z_2 + Z_3] = K_1 iZ_1 + K_2 iZ_2 + K_3 iZ_3$

$$K_{avg} = \frac{K_1 Z_1 + K_2 Z_2 + K_3 Z_3}{Z_1 + Z_2 + Z_3}$$

$$K_{avg} = \frac{\sum K_i Z_i}{\sum Z_i}$$

(B) Average Permeability Normal To Bedding Plane



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In normal direction to bedding plane

$$q_1 = K \times \left[\frac{h_{L_1}}{Z_1}\right] \times A$$

$$h_{L_1} = \frac{qZ_1}{AK_1}$$

$$h_{L_2} = \frac{qZ_2}{AK_2}$$

$$h_{L_3} = \frac{qZ_3}{AK_3}$$

Normal to bedding plane discharge will be same

Total head loss

$$H_L = h_{L_1} + h_{L_2} + h_{L_3}$$

$$q \text{ is same} = k_1 i_1 A = K_2 i_2 A = K_3 i_3 A$$

$$V = K_1 i_1 = K_2 i_2 = K_3 i_3$$

$$V = K_1 \frac{h_{L_1}}{Z_1} = K_2 \frac{h_{L_2}}{Z_2} = \frac{K_3 h_{L_3}}{Z_3}$$

$$h_{L_1} = \frac{VZ_1}{K_1}, h_{L_2} = \frac{VZ_2}{K_2}, h_{L_3} = \frac{VZ_3}{K_3}$$

From equation A & 1

$$H_L = h_{L_1} + h_{L_2} + h_{L_3}$$

$$\frac{\cancel{V} \Sigma Z_1}{K_{avg}} = \frac{\cancel{V} Z_1}{K_1} + \frac{\cancel{V} Z_2}{K_2} + \frac{\cancel{V} Z_3}{K_3}$$

$$K_{avg} = \frac{\Sigma Z_i}{\frac{Z_1}{K_1} + \frac{Z_2}{K_2} + \frac{Z_3}{K_3}}$$

$$K_{avg} = \frac{\Sigma Z_i}{\Sigma \frac{Zi}{Ki}}$$

CHAPTER - 7 (SEEPAGE ANALYSIS)

Seepage Analysis

→ When water flows through saturated soil mass, total head at any consists of pressure head, datum head (Elevation head) and velocity head.

→ Since flow through the soil mass is considerably less, velocity head is neglected in it. Hence total head at any point consists of elevation head and pressure head which is also termed as hydraulic head

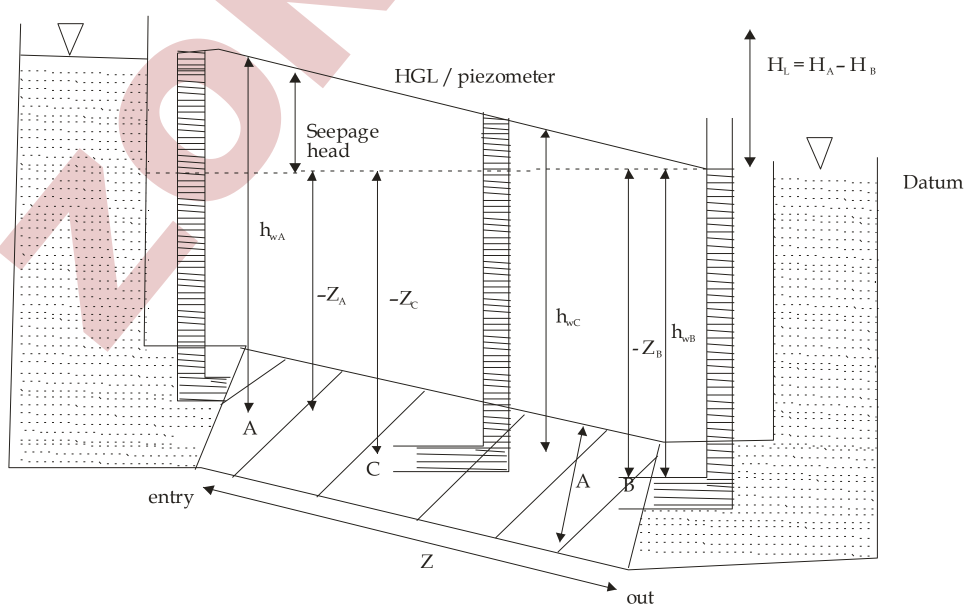
$$\because \qquad \text{Velocity head} = \frac{V^2}{2g} = \frac{(Ki)^2}{2g} \approx 0$$

$$\text{Total head} = \text{Pressure head} + \text{Datum head}$$

→ Elevation head at a point is the vertical distance of that point measured from an assumed datum plane which is normally taken at the tail water level elevation for convinience.

→ If a piezometer of an open stand pipe is inserted at a point of flow, water would stand upto a particular height inside the piezometers termed as pressure head.

→ The difference between total heads at any two points in a soil through which flow is occuring represents the head loss between these two points.



	A	B	C
D.H	$-Z_A$	$-Z_B$	$-Z_C$
P.H	h_{wA}	h_{wB}	h_{wC}
T.H or Hydraulic head	$h_{wA} - Z_A = H_A$	$H_B - h_{wB} = Z_B = 0$ $h_{wB} = H_B$	$h_{wC} - Z_C = H_C$

Hydraulic head under which seepage is taking place (H) = $h_{entry} - h_{exit} = H_A - H_B$

seepage head = $(h_{wA} - Z_A) - (h_{wB} - Z_B)$

seepage head (h) = $h_{wA} - Z_A$

Hydraulic gradient (i) = $\frac{\text{seepage head}}{\text{length of flow}} = \frac{\text{head loss}}{\text{length}}$

$$i = \frac{h}{Z}$$

Seepage pressure :-

→ When water flows through saturated soil mass, water pressure pushes soil skeleton by virtue of frictional drag. This pressure exerted by water is termed as seepage pressure.

→ If 'h' is the seepage head or hydraulic head under which seepage takes place then seepage pressure is given as

seepage pressure = seepage head × γ_w

$$P_S = h.\gamma_w$$

we know that $P_S = i.Z.\gamma_w$

$$i = \frac{h}{Z} \Rightarrow h = i.Z$$

Seepage force $P_S = P_S \times A$

$= iZ\gamma_w.\ A$

specific seepage force $(P_{SS}) = \frac{\text{seepage force}}{\text{volume}} \Rightarrow \frac{iZ.\gamma_w.A}{Z.A}$

$$(P_{SS}) = i\gamma_w$$

Note :- Seepage pressure always acts in the direction of flow hence vertical effective stress may be increased or decreased due to seepage depending upon the direction of flow.

→ If flow takes place in vertically downward direction, effective stress will increase because seepage pressure also acts in downward direction.

→ If flow takes place in vertically upward direction, effective stress is reduced because seepage pressure acts in upward direction.

Effective stress = $Z\gamma' \pm P_S = Z\gamma' \pm iZ\gamma_w$

If flow is downward $\bar{\sigma} = Z\gamma' + iZ\gamma_w$

If flow is upward $\bar{\sigma} = Z\gamma' - iZ\gamma_w$

Special case :- When flow takes place in upward direction seepage pressure also acts in upward direction and effective stress is reduced. If seepage pressure equals the submerged weight of soil then effective stress reduces to zero. In such case Cohesionless soil mass losses all its shear strength and have the tendency to flow along with the water. This phenomenon in which soil particles leave the soil mass and flow along with the water is termed as quick sand, piping, sand boiling or floating condition.

$$\bar{\sigma} = Z\gamma' - P_S = 0$$

$$Z\gamma' = P_S = \text{seepage head} \times \gamma_w = iZ\gamma_w$$

$$Z\gamma' = iZ\gamma_w$$

$$i_c \Rightarrow \frac{\gamma'}{\gamma_w} = \frac{(G-1)\gamma_w}{(1+e)\gamma_w} = \left(\frac{G-1}{1+e}\right) \Rightarrow \text{Critical Hydraulic Gradient}$$

$\Rightarrow$ Quick sand flow condition

→ To avoid quick sand condition, hydraulic gradient should always be smaller than critical hydraulic gradient

$$i < i_C \Rightarrow \text{FOS} = \frac{i_C}{i} > 1. \text{ for no quick sand}$$

→ For fine sand for which specific gravity is 2.65 & void ratio is approx 0.6 then approx critical hydraulic gradiet is unity.

$$L_c = \frac{2.65-1}{1+0.65} \approx 1$$

Note :- Quick sand is not a type of sand but is a flow condition in cohesionless soil mass when effective stress is reduced to zero due to upward flow condition.

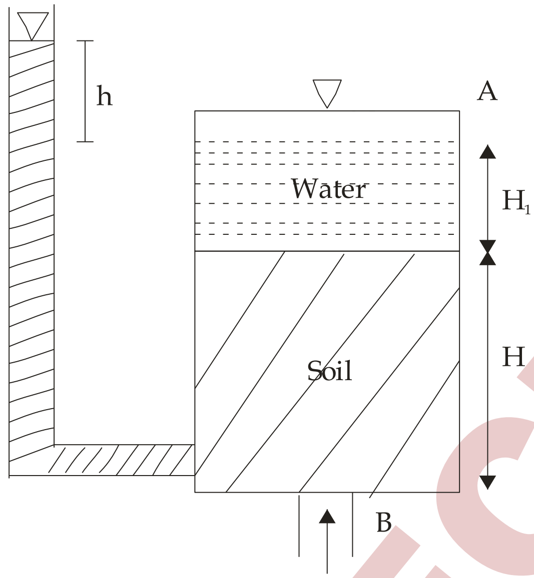
Quick sand generally occurs in sand and coarse silt & is not found in gravel, clay & fine silt

→ In cohesive soil like clay shear strength is not reduced to zero even if effective stress is reduced to zero & soil particles are held due to their in present cohesion

Shear strength S = $C + \bar{\sigma}_n \tan\phi$

For cohesive soil	**for cohesionless soil (C = 0)**
In quick sand $\bar{\sigma}_n = 0$	$S = \bar{\sigma}_n \tan\phi$
$S = C + \cancel{\bar{\sigma}}^{0} \tan\phi$	in quick sand $\bar{\sigma}_n = 0$
S = C $\Rightarrow S \neq 0$	$\Rightarrow$ S = 0

→ In gravels & coarse sand which & highly permeable soils high discharge is required to obtain critical hydraulic gradient which in natural condition is not practically possible.



For quick sand condition

$$\bar{\sigma} = \sigma - u = 0$$

$$\sigma = u$$

$$\gamma_w H_1 + \gamma_{sat} H = (\text{pr.head}) \times \gamma_w$$

$$= [H + H_1 + h].\, \gamma_w$$

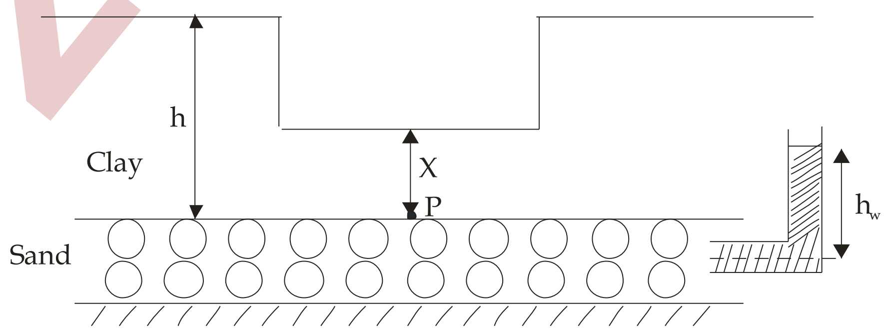
$\therefore$
$$\gamma_w H_1 + \gamma_{sat} H = H\gamma_w + H_1\gamma_w + h\gamma_w$$

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

$$h\gamma_w = (\gamma_{sat} - \gamma_w)H = \gamma' H$$

$$i = \frac{h}{H} = \frac{\gamma'}{\gamma_w} = \frac{G-1}{1+e}$$

→ Quick sand also occurs when sand under artesian pressure is overlain by impermeable layer like clay



Total stress approach

Total stress at P $\quad X\gamma_{sat} = h_w.\gamma_w$

Flow net & Seepage Discharge

Laplace Equation & Flow Net :-

If seepage takes place in 2-dimension. It can be analysed using laplace equation which represents the loss of energy head in any resistive medium

H = head loss

$$\frac{\partial^2 H}{\partial x^2} + \frac{\partial^2 H}{\partial y^2} = 0$$, Medium is isotropic

$$k_x \cdot \frac{\partial^2 H}{\partial x^2} + k_y \frac{\partial^2 H}{\partial y^2} = 0$$, Medium is non - isotropic

→ The graphical solution of laplace equation is flownet which represents the description of equipotential lines and stream lines

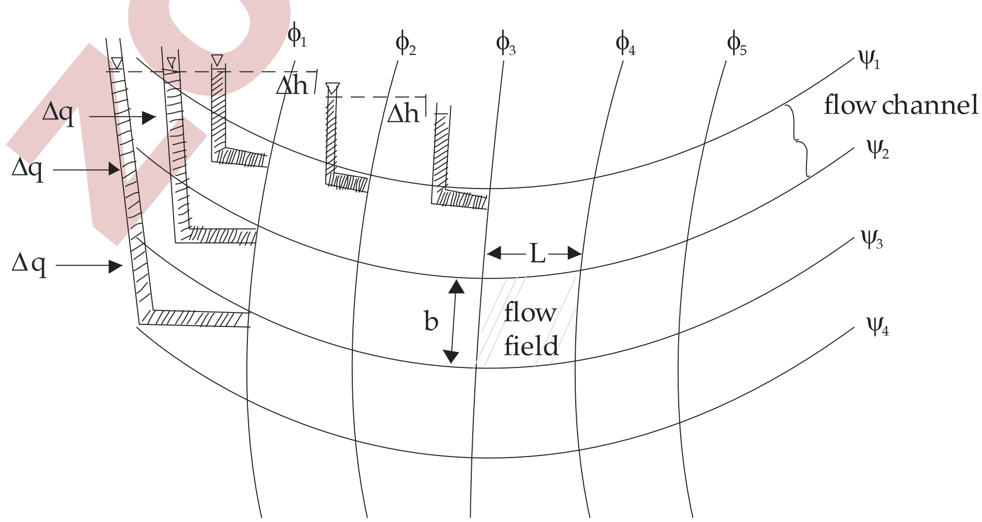
→ **Assumptions in flow net**

1. Soil is homogenous & isotropic
2. Darcy's law is valid
3. Pore fluid and soil solids are incompressible

→ **Properties of flow net :-**

1. Equipotential lines (ϕ lines) and stream lines (ψ lines) intersect each other perpendicularly.
2. Flow will take place along the streamlies and velocity of flow is always perpendicular to the equipotential lines.
3. Loss of head between 2 equipotential lines is always same and is termed as equipotential drop
4. The area between 2 flow lines is known as flow channel and discharge through each flow channel is same.
5. Area bounded between 2 equipotential lines and flow lines is known as flow field which are approximately square in isotropic medium that may be linear or curvilinear while in non isotropic medium they are approximately rectangular which may be linear or curvilinear.
6. If water levels are reversed U/S ad D/S side without changing boundary condition there will be no change is flow net it means flow net is unique for given set of boundary conditions.

→ The quantity of seepage in each flow channel is independent of size of field



$$q = \Delta q \times N_f \qquad \Delta h = \frac{H}{N_d}$$

N_f = No of flow channels N_d = No of equipotential drops

N_f = No of flow lines - 1 N_d = No of equipotential lines - 1

Applications of flow net:-

→ **Flow net can be used for determination of**

1. Seepage discharge
2. Seepage pressure
3. Pore water pressure (hydrostatic pressure)
4. Exit gradient

Determination of seepage discharge

Let Δq is the discharge through flow field under the head of H consider unit length of dam.

As per darcy

$$q = KiA$$

$$\Delta q = K.\frac{\Delta h}{L}(b \times 1)$$

$$\Delta q = K.\Delta h\left(\frac{b}{L}\right)$$

case (a) Medium is isotropic

Flow field will be square $b \approx L$

$$\Delta q = K.\Delta h\left[\frac{b}{L}\right] = K\Delta h$$

Total discharge (q) = $\Delta q \times N_f = K.\Delta h.N_f$

$$q = K.\frac{H}{N_d} \times N_f \quad \left[\because \Delta h = \frac{H}{N_d}\right]$$

N_f = no. of flow channels

N_f = no. of equipotential drops

$$\frac{N_f}{N_d} = \text{Shape factor}$$

Depends only on boundary condition

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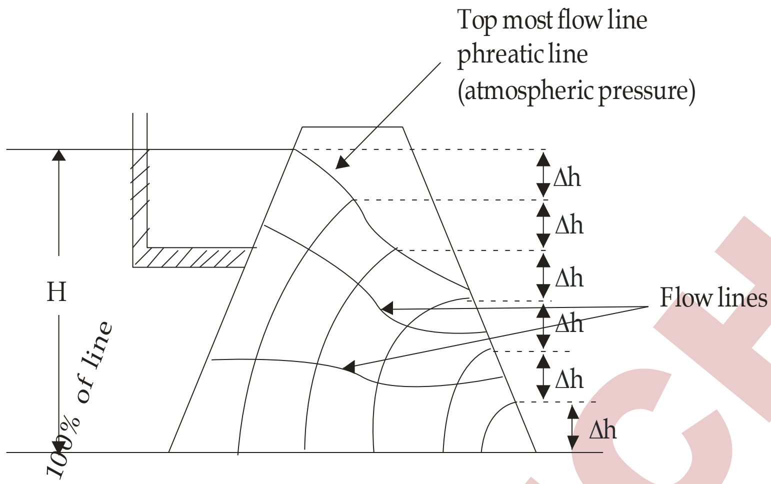
Phreatic Lines :-



→ Phreatic line is the topmost flow line below which seepage takes place. It is used to seperate saturated soil mass from unsaturated soil mass.

→ Pressure below the phreatic line is hydrostatic and pressure on and above phreatic line is atmospheric

→ Phreatic line follows the path of base parabola for which any point lying on it is equidistant from focus & directrix.

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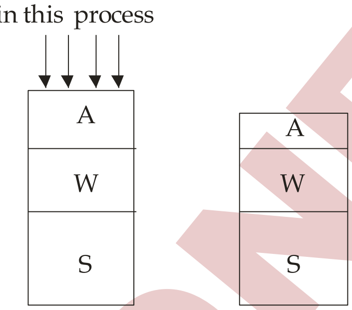
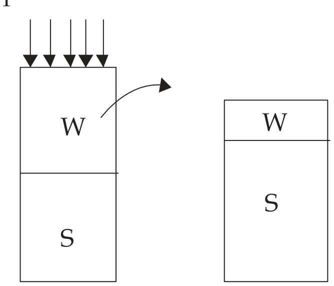
CHAPTER - 8 (COMPACTION OF SOIL)

Compaction of Soil

It is the process in which soil particles are artifically rearranged and packed together into closer state of contact by mechanical means (rollers, rammers, vibrators etc.), in order to reduce the void ratio, permeability, compressibility and in order to increase the degree of denseness, stability shear strength, bearing capacity (in order to modify engineering properties of soil)

Diference Between compaction and Consolidation :

Compaction	Consolidation
1. It is almost instantaneous process. 2. Soil is unsaturated. 3. Volume reduction is due to expulsion of pore air. 4 Specified mechanical techniques are used in this process	1. It is time dependent process 2. Soil is completely saturated 3. Volume reduction is due to expulsion of pore water. 4 Consolidation occurs an account of a load placed on the soil.
	

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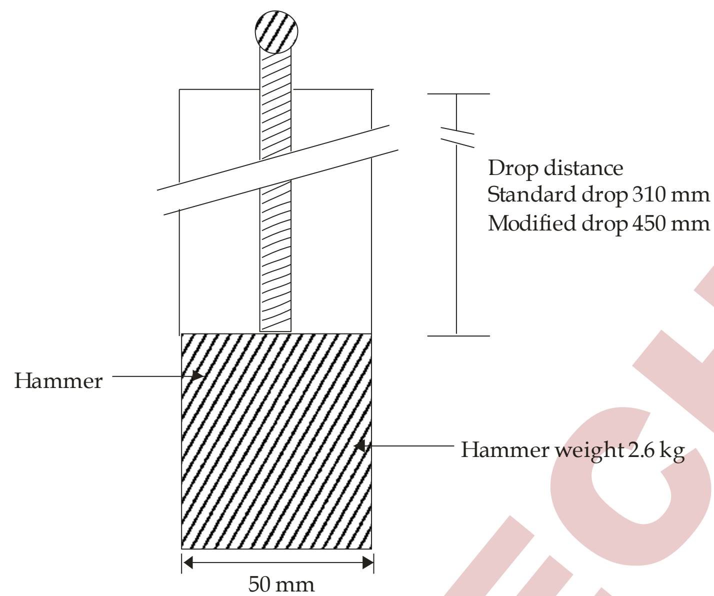
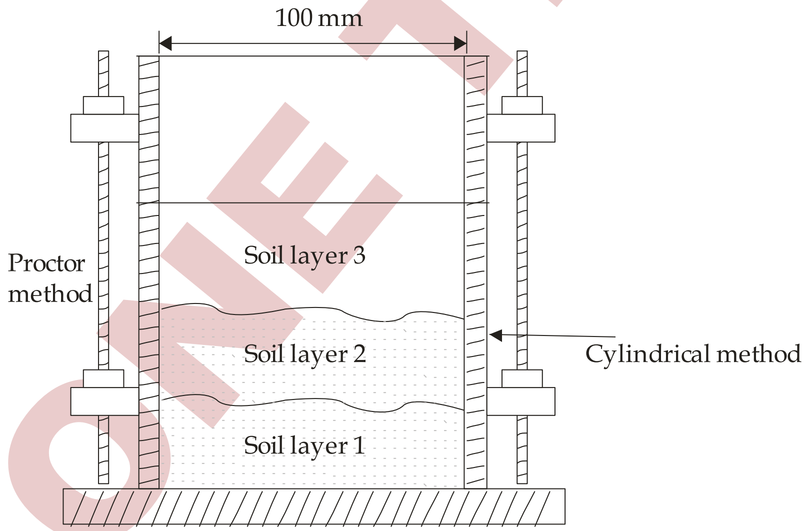
→ The compaction of soil is represented in terms of dry density.

→ Compaction characteristics of soil are first analyzed in the lab by performing any of the following tests.

Compaction Laboratory Test :

→ These tests consist of a standard cylindrical mould in which soil to be tested is filled in layers and each layer is compacted by subjecting it to 25 number of blows with the help of hammer.





	Standard Proctor Test	IS 2720 Light compaction test	Modified Proctor test	IS 2720 Heavy compaction test
1. Volume of mould	945 cc	1000 cc	945 cc	1000 cc
2. No of layers	3	3	5	5
3. No of blows on each layer	25	25	25	25
4. wt of hammer	5.5 lb (2.445 kg)	2.6 kg	10 lb (4.54 kg)	4.9 kg
5. Height of freefall of hammer	12 inches (304.8 mm)	310 mm	18 inches (457.8 mm)	450 mm

NOTE :-

$$\frac{(E)_{Heavy}}{(E)_{Light}} = \frac{(mgH)_H \times \text{No. of blows / volume}}{(mgH)_L \times \text{No.of blows / volume}}$$

$$\frac{(E)_H}{(E)_L} = \frac{4.9 \times g \times 0.45 \times 5 \times 25 / 1000 \times 10^{-6}}{2.6 \times g \times 0.3 \times 3 \times 25 / 1000 \times 10^{-6}} \frac{Nm/m^3}{Nm/m^3}$$

$$\frac{(E)_H}{(E)_L} = \frac{2704}{593} \frac{kNm/m^3}{kNm/m^3} = 4.55$$

NOTE :-

Compaction energy provided in standard proctor test is 595 kJ/m^3 and compaction energy provided in modified proctor test is 2674 kJ/m^3.

→ After performing above compaction test on soil bulk unit weight (γ_b) is determined as

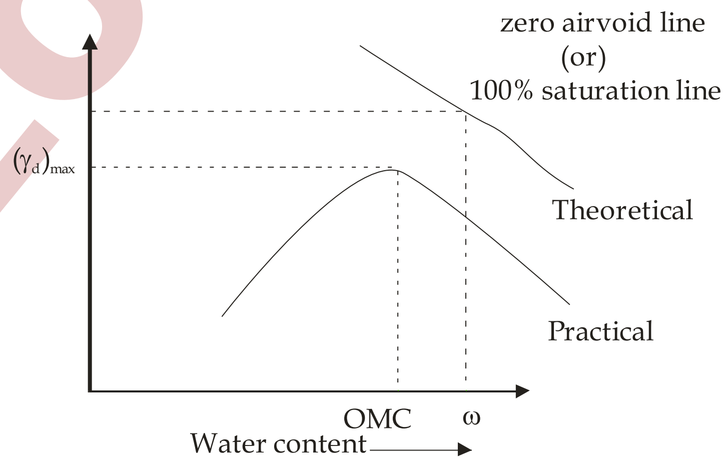
$$\gamma_b = \frac{\text{wt of soil in mould}}{\text{volume of mould}}$$

→ A sample is collected from mould and tested for water content using oven dry method which is used to find dry unit weight of soil.

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

→ The test is repeated at different water contents and corresponding dry unit weight achieved is noted. The results of these test are expressed in the form of a curve termed as compaction curve (water content on x-axis and γ_d on y-axis)

→ This curve is used to find the water content at which maximum dry unit weight of soil is achieved which is termed as optimum moisture content (OMC).



→ A line showing the relationship between water content and dry unit weight of compacted soil at constant percentage of air voids is termed as air void line.

Equation of air void line

$$\gamma_d = \frac{(1-\eta_a)G\gamma_w}{1+wG}$$

$\eta_a =$ constant

Equation of Saturation line

→ A line showing relationship between water content and dry unit weight of compacted soil at constant percent of degree of saturation is termed as saturation line.

$$\gamma_d = \frac{G\gamma_w}{1+\frac{wG}{s}}$$

→ A line showing relationship between water content and γ_d at zero air void (or) 100% degree of saturation is known as zero air void line or 100% saturation line.

Equation of zero air void line (or) 100% saturation line

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

NOTE :

Theoretical maximum dry unit weight at particular water content during compaction is achieved at zero% percent air voids or 100% degree of saturation.

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

NOTE :

In normal soils (silt, clay, well graded coarse grained soil) maximum dry unit weight is attained at 85% to 95% degree of saturation. Because it is not possible to remove the entire air from the sample of soil under heaviest possible compaction (as soon as hammer is lifted, after the impact some amount of air again enters into the soil).

NOTE :

Although zero air void line corresponds to 100% saturation line but for example 85% or 95% degree of saturation line are not same as 15% or 5% air void lines.

$s + a_c = 1$

If $s = 1, a_c = 0$

Then $\eta_a = \eta a_c = 0$

if $s = 85\%$

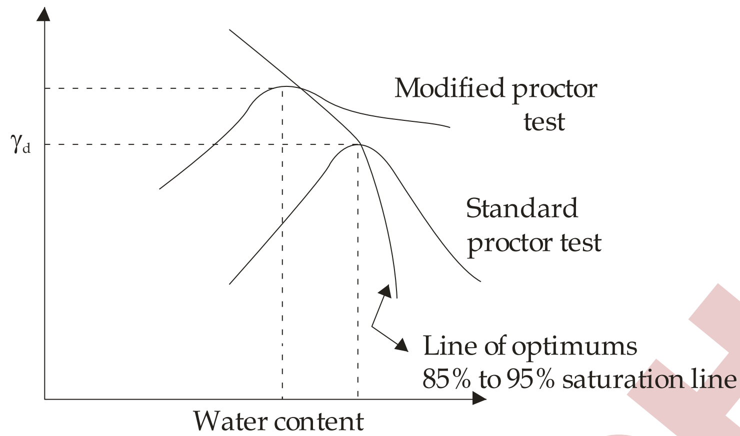
$a_c = 0.15$

But $\eta_a = n.\ a_c$ and $0 < n < 1$

$n_a \neq a_c$

Hence 85% saturation line is not same as that of 15% air void line.

→ With increase in compactive effort maximum dry unit weight increases and corresponding OMC decreases.



Quality Control at the Field

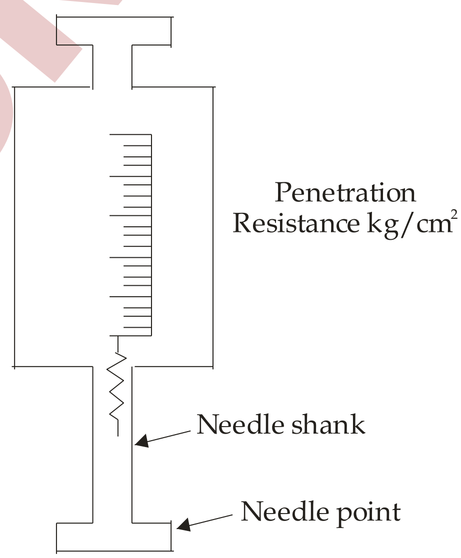
→ Compaction characteristics of the soil in the field requires the rapid computation of water content which can be done either by the use of calcium carbide method or proctor needle method.

Proctor Needle Method

→ Proctor needle is inserted in the sample to be tested for its water content upto the depth of 7.5 cm at the rate of 1.25 cm/s and resistance offered by soil to needle point against the penetration is noted.

→ The test is repeated at different water content and corresponding penetration resistance is noted. The result of this test are then expressed in form of a curve (water content on x-axis and corresponding penetration resistance on y-axis).

→ This curve is further used to find water content in the field corresponding to given penetration resistance.



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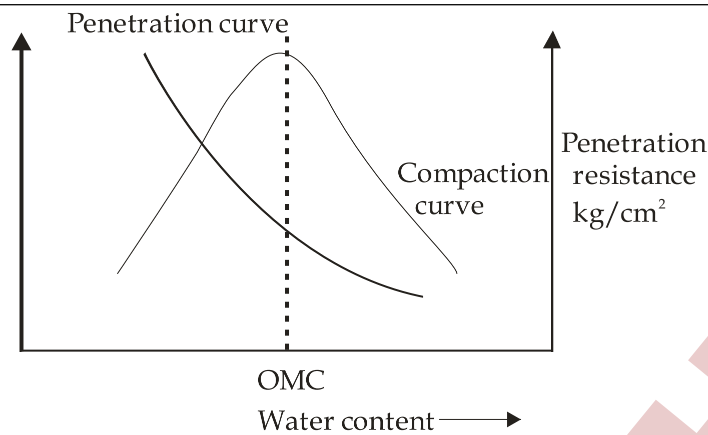


Relative Compaction

→ The dry density achieved in the field is compared with the maximum dry density obtained in the laboratory compaction test with the help of relative compaction.

$$\text{Relative compaction} = \frac{(\gamma_d)_{\text{in the field}}}{(\gamma_d)_{\text{max in the lab}}} \times 100$$

→ Relative compaction can also be related to density index empirically

$$R.C. = 80 + 0.2\, I_D$$

I_D = Density index in %

Factor Affecting Compaction

1. Water Content

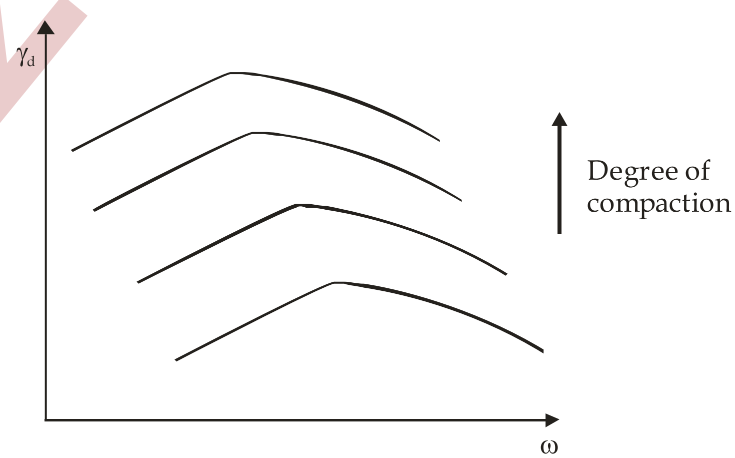
→ At lower water content soil is relatively stiff. Hence offers higher resistance against compaction and does not pack into denser state of contact resulting in lower γ_d.

→ With increase in water content more and more layers of water are formed surrounding the soil solids which induces the lubrication effect under which soil solids can be easily worked around in denser state of contact resulting in higher γ_d.

→ This increase in denseness of soil takes place upto a particular water content termed as OMC at which lubrication effect is maximum beyond which if water content is increased there is no change in lubrication effect but water starts replacing solids resulting in decreased γ_d. ($\gamma_w << \gamma_s$).

2. Degree of Compaction

→ With increase in degree of compaction, $(\gamma_d)_{max}$ increases and corresponding OMC decreases



3. Type of Equipment

Equipment	Effect/ Action
1. Roller	
(a) Smooth wheeled roller	Pressure
(b) Sheep foot roller	Kneading
(c) Pneumatic type roller	Kneading + Pressure
2. Rammers	Impact
3. Vibrators	Vibration

Suitable Soil	Suitable Construction
(1) Crushed stone Gravel, Rock	Pavement construction
(2) Cohesive soil (Clay)	Core of earthen dam
(3) All type of soil	Airfield, homogeneous dam, Confined area, behind retaining wall, basement wall, Embankment oil storage
(4) Sand	Embankment

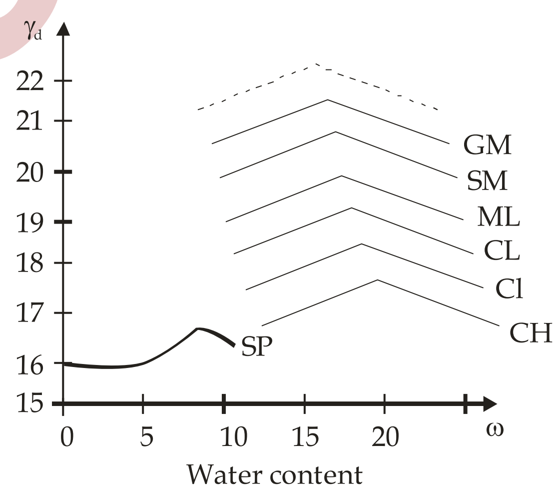
4. Type of Soil

→ Well graded coarse grained soil with fine particle are found to have maximum $(\gamma_d)_{max}$ and corresponding lower OMC.

→ With increase in % fineness in these soils, there $(\gamma_d)_{max}$ decreases

→ Poorly graded coarse grained soil is found to have minimum $(\gamma_d)_{max}$

→ Fine grained soil generally has lower $(\gamma_d)_{max}$ and higher OMC. With increase in plasticity of these fine grained soil, their $(\gamma_d)_{max}$ decreases and corresponding OMC increases.



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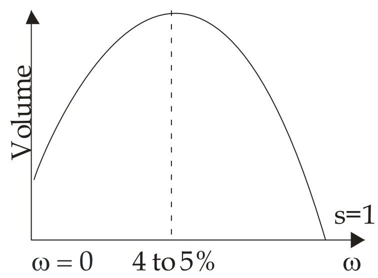
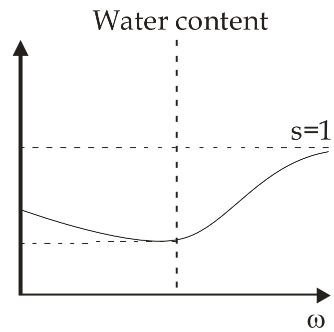




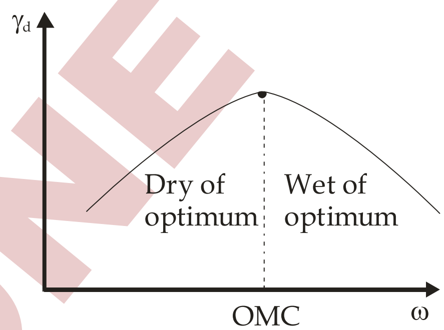
Effects of Compaction on Engineering Properties

Structure of Soil

→ At the same compactive effort on the dry of optimum, structure is found to be more random and on the wet of optimum side it is found more orientated. For Example

→ On the dry of optimum, structure is found to be flocculating and on the wet of optimum is found to be dispersed.

→ With increase in water content at same compactive effort orientation of particles in soil structure is improved.



2. Permeability

→ At the same compactive effort, permeability on dry of optimum is found to be more than that on wet of optimum side due to more random orientation of particles on dry side.

3. Pore Water Pressure

→ As deficiency of water is more on dry of optimum side pore water pressure is found to be less on this side.

4. Shrinkage

→ Due to random orientation of soil particles, deficiency of water and stronger interparticles bonds, shrinkage on dry of optimum is found to be less than shrinkage on wet of optimum side.

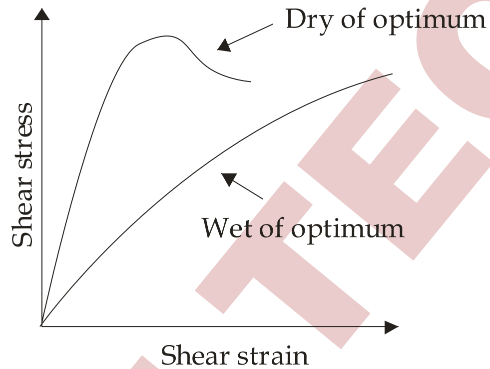
5. **Swelling**

→ Due to higher deficiency of water on dry of optimum side, higher swelling pressure is exerted when water is imbibed in it. Hence higher swelling is found on dry of optimum.

6. **Compressibility**

→ At lower stress level, when the structure is unbroken on dry of optimum side, stronger inter-particle bonds are available which results in lower compressibility in comparison to wet of optimum side. But at higher stress level, when the structure is broken, compressibility at dry of optimum increases in comparison to wet of optimum.

7. **Shear Strength :** More in dry of optimum side.



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Chapter - 9 (Consolidation of Soil)

Compressibility

→ It is defined as gradual decrease in the volume of soil due to application of load on soil mass under consideration

The volume change may take place due to

1. Compression & expulsion of pore air
2. Expulsion of pore water
3. Plastic readjustment of soil solids

- Compressibility
  - Initial compresion OR Compaction
    - Due to compression & expulsion of pore air
  - Consolidation
    - Primary consolidtion
      - Due to expulsion of pore water
    - Secondary consolidation
      - Due to Plastic readjustment of soil solids

Intitial compression / Compaction

→ It is an instataneous process which occus immediately after loading & is due to compression & expulsion of pore air

Primary consolidation

→ It begins when soil reaches to full saturation (after expulsion of pore air) & remains saturated during the entire process of primary consolidation.

→ Primary consolidation is due to expulsion of pore water & it completes when expulsion of pore water stops.

→ It is time taking phemomenon which depends upon permeability of soil, loading condition & length of drainage path.

Secondary Consolidation

→ It starts after completion of primary consolidation

→ If load is constant & volume change is recorded with passage of time without expulsion of pore water then volume change is due to plastic readjustment of soil molecules and termed as secondary consolidation.

→ In coarse grained soil (gravel/sand) secondary consdidation is insignificant but in highly plastic clays secondary consolidation can be 10% to 20% of total settlement.

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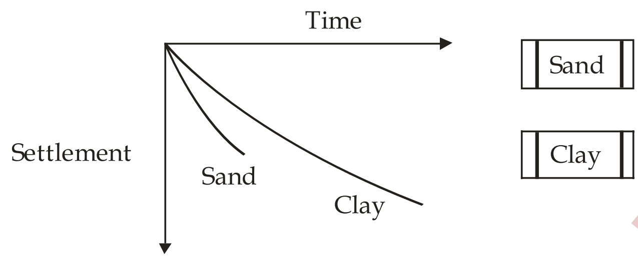
Primary Consolidation Characteristics.

1. Settlement V Time

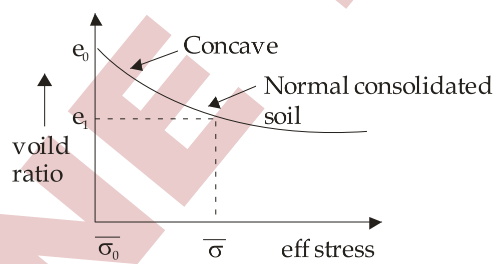


→ The rate of consolidation in sand is more due to its high permeability but total settlement of clay is much greater than that of sand at same loading rate & having same thickness of sample of both the soils.

→ The setlement in clay is higher because of its high void ratio or porosity. Thus for all practical purposes compressibility of granular soil is negleted on account of consolidation.

2. Void Ratio V Effective Stress (Normal Consolidation Curve or Normal Compression Curve)

→ Normal consolidated soils are those which are subjected to first time in the history to present applied effective stress $(\text{present} \geq \text{past})$



Note :- The slope of above curve is termed as coefficient of compressibility

Slope of $e \text{ Vs } \bar{\sigma} \text{ curve} = \dfrac{e_0 - e_1}{\bar{\sigma}_0 - \bar{\sigma}_1}$

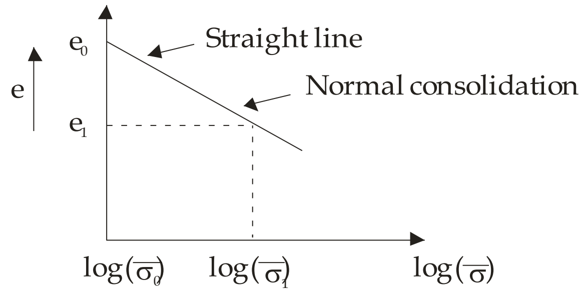
Coefficient of compressibility = $\left|\dfrac{e_0 - e_1}{\bar{\sigma}_0 - \bar{\sigma}_1}\right|$

Coefficient of compressibility = $a_v = \dfrac{\Delta e}{\Delta \bar{\sigma}}$

Since a_v is not constant for whole curve, hence it is not widely used.

3. Void Ratio Vs log effective stress

If graph of effective stress in log scale & void ratio in arithmetic scale is plotted then straight line is obtained.



Slope of e Vs log $\bar{\sigma}$ curve = $\left|\dfrac{e_0 - e_1}{\log\bar{\sigma}_0 - \log\bar{\sigma}_1}\right|$

The slope of above curve is termed as coefficient of compression or compression index.

coefficient of compression = $\left|\dfrac{e_0 - e_1}{\log\bar{\sigma}_0 - \log\bar{\sigma}_1}\right|$

Coefficient of compression $C_c = \dfrac{\Delta e}{\log\left(\dfrac{\bar{\sigma}_1}{\bar{\sigma}_0}\right)}$

→ For most of the clays C_c lies in between 0.1 - 0.8. Greater C_c means soil is not compressible.

Empirical relations to find C_C

(a) For undisturbed soil with medium sensitivity (sensitivity 4 to 8)

$C_C = 0.009\ (W_L - 10)$

W_L = L.L in %

(b) For remoulded clays with medium to low sensitivity (sensitivity 2 to 4)

$C_C = 0.007\ (W_L - 10)$

W_L = L.L in %

(c) For organic clays :

$C_C = 0.0115\ W_n$

$W_n \rightarrow$ natural water content in %

(d) When initial void ratio is known

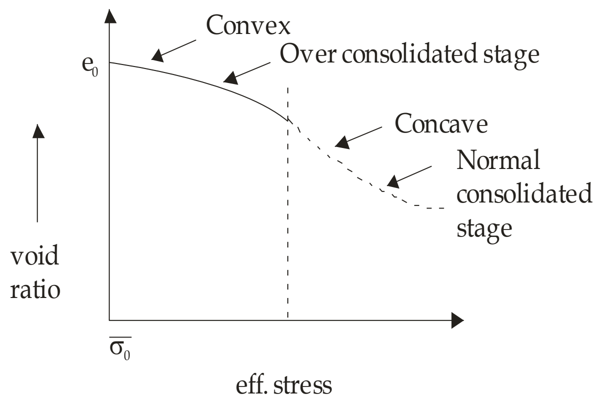
$C_C = 1.15\ (e_0 - 0.35)$

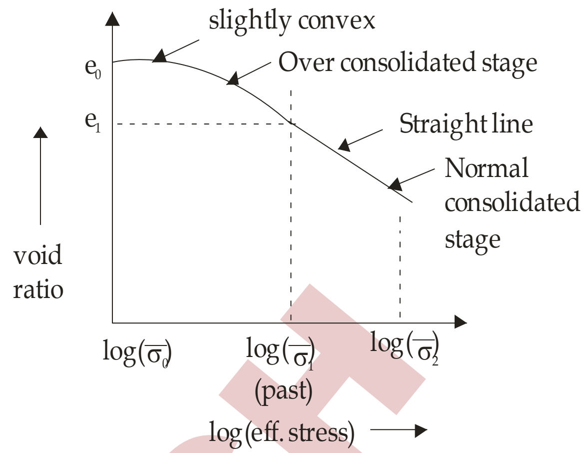
e_0 = initial void ratio

4. Over Consolidated soils / Pre-consolidated soil.

→ If presently applied effective stress is less than applied effective stress in past then soil is said to be in over consolidated stage.

→ Over consolidated soils show less volume change & highly over consolidated soils behave like dense sands.





{Slope of e Vs $\log(\bar{\sigma})$ in Over consolidated stage} = $\dfrac{e_0 - e_1}{\log \bar{\sigma}_1 - \log(\bar{\sigma}_0)}$

Coefficient of recompression = $\left|\dfrac{e_0 - e_1}{\log \bar{\sigma}_1 - \log(\bar{\sigma}_0)}\right|$

$$C_R = \frac{\Delta e}{\log\left(\frac{\bar{\sigma}_1}{\bar{\sigma}_0}\right)}$$

generally C_R well be $\frac{1}{5}$ to $\frac{1}{10}$ of C_C

Over Consolidation Ratio

$$\text{O.C.R} = \frac{\text{maximum applied effective stress in past}}{\text{maximum applied effective stress in present}}$$

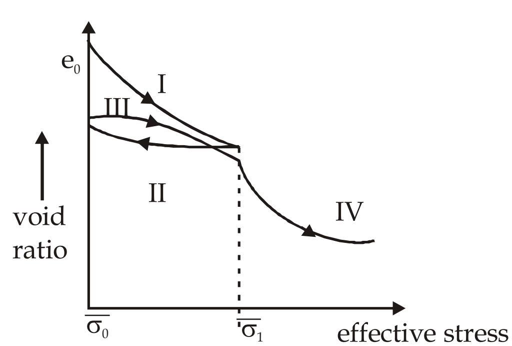
If OCR > 1 over consolidated soil

OCR = 1 normal consolidated soil

OCR < 1 under consolidated soil [when the building is under construction

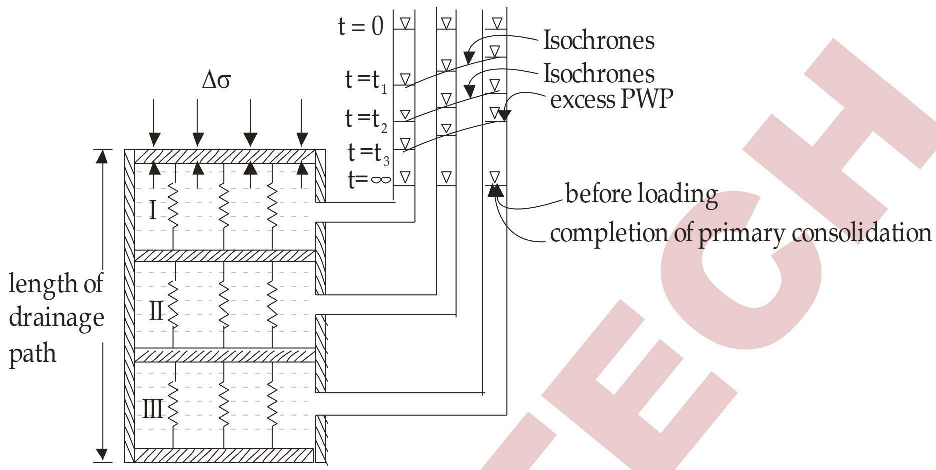
Note :- Desiccation of clay, removal of glacier, removal of construction load rising of water table are same example of over consolidation.

Multistage Cycle Loading.



(I) Normal consolidation curve/virgin compression curve (concave)
(II) Swelling / Rebound Compression curve (concave)
(III) Over consodidation curve (convex)
(IV) Normal consodidation curve (concave)

Terzaghi 1 - D Consolidation Theory



(1) At application of total stress
(2) at t = 0 [development of excess PWP (u_i)]

$\Delta\sigma = \Delta\bar{\sigma} + u_i$

(3) At any time 't' [Expulsion of pore water will take place, effective stress increases]

$\Delta\sigma = \Delta\bar{\sigma}\uparrow + u_i\downarrow$

(4) At time $t = \infty$ [Excess pore pressure reduces to zero; Primary consolidation completed]

$\Delta\sigma = \Delta\sigma + \cancel{u}^{0}i$

Assumptions

1. Soil mass is homogenous & isotropic
2. The consolidation is one-dimensional. It means flow of expelling water is unidirectional (in vertical) & there is no change in area (volume change is due to change in depth only)
3. Darcy's law is valid
4. Soil is fully saturated and remains saturated throughout the process of consolidation. It means initial compression is not considered.
5. Strain coming on soil solids are small & negligible.
6. The hydro dynamic lag is considered whereas plastic lag is ignored. However plastic lag is found to exist. It means time required for plastic readjustment is ignored. It means secondary consolidation is not considered.

→ In order to develop field condition in lab consolidation test is being performed over the soil sample as per Terzaghi theory of 1-D consolidation in consolidometer or oedometer in which settlement of given sample of soil is noted at different time interval. In consolidation test pressure on the soil in each stage is doubled.

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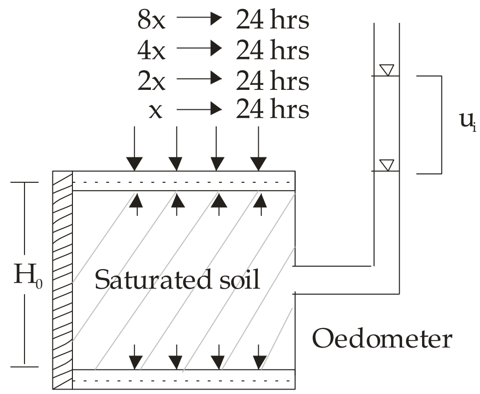
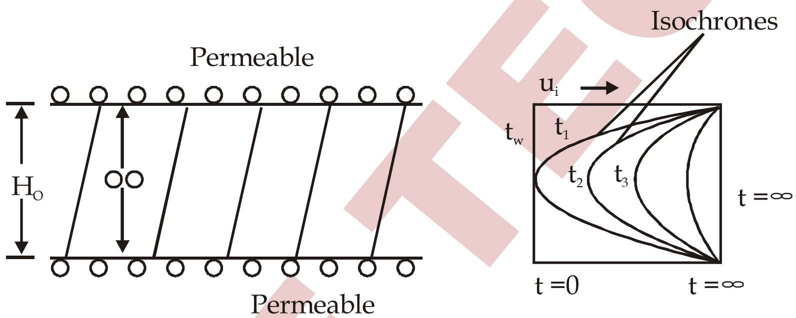
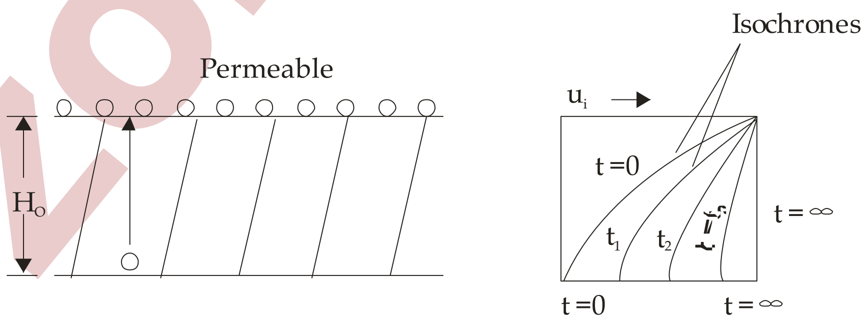


2 - Way drainage



$$\text{Length of drainage path} = d = \frac{H_0}{2}$$

1 - Way drainage



$$\text{Length of drainage path} = d = H_0$$

Terzaghi 1-D Consolidation equation

$$\frac{\partial U}{\partial t} = C_v \frac{\partial^2 U}{\partial z^2}$$

where $\frac{\partial U}{\partial t}$ = rate of change of PWP which represents the rate of consolidation.

C_v = coeff of consolidation

$$C_v = \frac{K}{m_v \gamma_w}$$

Where K = coeff of permeability

m_V = coeff of volume compressibility or modulus of volume change

$$m_V = \frac{\frac{-\Delta V}{V_0}}{\Delta\bar{\sigma}} = \frac{\frac{-\Delta e}{1+e_0}}{\Delta\bar{\sigma}}$$

$$m_v = \frac{-\Delta e}{\Delta\bar{\sigma}} \times \frac{1}{(1+e_0)}$$

$$m_v = \frac{a_v}{1+e_0}$$

a_v = coeff of compressibility $\left(\left|\frac{\Delta e}{\Delta\bar{\sigma}}\right|\right)$

e_0 = initial void ratio

Solution of Terzaghi Equation :-

1. Degree of Consolidation :-

→ It is fraction of ultimate consolidation which is completed at any stage of time during consolidation.

→ Defined for primary consolidation process

→ At t = 0 at the beginning %U = 0%

→ At $t = \infty$ at the end of primary consolidation %U = 100%

→ Degree of consolidation (%U) can be determined by 3 ways.

1. When Settlement at any stage is known.

→ Let Δh is the settlement at any time 't'

→ ΔH = ullimate consolidation settlement at the end

$$\%U = \frac{\Delta h}{\Delta H} \times 100$$

2. When excess pore pressure is known.

Let after time t excess pore pressure is u and at the beginning of consolidation excess pore water pressure = u_i

$$\%U = \left(\frac{u_i - u}{u_i - 0}\right) \times 100 = \frac{u_i - u}{u_i} \times 100$$

3. When void ratio is known

$$\%U = \frac{e_0 - e}{e_0 - e_{100}} \times 100$$

e_{100} = Final void at the end of primary consolidation

e_0 = Initial void ratio at time $t = 0$

e = Void ratio at any time t

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4. **Time Factor :-** It is the parameter which relates the degree of consolidation & time required for that consolidation

$$T_V = C_V \frac{t}{d^2}$$

C_v = coefficient of consolidation $\left(\frac{k}{m_v \gamma_w}\right)$

$$C_v = \frac{K}{\frac{a_v}{1+e_0}.\gamma_w} = \frac{K}{\frac{\Delta e}{\Delta\bar{\sigma}} \times \frac{1}{(1+e_0)}\gamma_w} = \frac{K\Delta\bar{\sigma}}{\left(\frac{\Delta e}{1+e_0}\right).\gamma_w}$$

→ d = length of drainage path :

for 2 way drainage $d = \frac{H_0}{2}$

for 1 way drainage $d = H_0$

→ t = time at which time factor needs to be calculated.

→ T_V = time factor which depends upon %U

$$T_V = \frac{\pi}{4}\left(\frac{U}{100}\right)^2, \ \%U \le 60\%$$

$T_V = 1.781 - 0.9332 \log_{10} [100 - \%U]$, %U > 60%

$(T_V)_{50\%} = 0.196$, $(T_V)_{90\%} = 0.848$ $(T_V)_{100\%} = \infty$

NOTE :- Theoretially for 100% primary conside dation infinite time is required but primary consolidation is assumed to be computed if degree of consolidation has reached $\geq 90\%$

3. **Method of determination of C_V :-** C_V depends on type of soil, permeability & change in effective stress

→ **Square root of time fitting method**

%U = 90% $\quad T_V = C_V \frac{t}{d^2} \quad (T_V)_{90\%} = 0.848 = C_V \frac{t_{90}}{d^2}$

t_{90} = time required for 90% consolidation & is observed from observation of lab

$$t_{90} = 0.848 \frac{d^2}{C_V}$$

(b) **Casagande's Logarithmic of Time fitting method (%U = 50%)**

$$T_V = C_V \frac{t}{d^2}, (T_V)_{50} = 0.196 = C_v \frac{t_{50}}{d^2} \Rightarrow C_V = \frac{0.196\,d^2}{t_{50}}$$

Note :- Square root of time fitting method is better for soils having high secondary consolidation settlement

Settlement Analysis

→ The total settlement is the sum of immediate / initial settlement, primary consolidation settlement & secondary consolidation settlement

→ I.S code specifications for permissible settlement

→ For isolated footing on sand = 40 mm

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→ For isolated footing on clay = 65 mm
→ For raft footing on sand = 40-65 mm
→ For raft footing on clay = 65 - 100 mm

Permissible differential settlement
- Isolated footing on sand = 25 mm
- Isolated footing on clay = 40 mm

Determination of Initial / Immediate Settlement

Small elastic settlement can occur due to deformation of clay particle and squeezing of water.

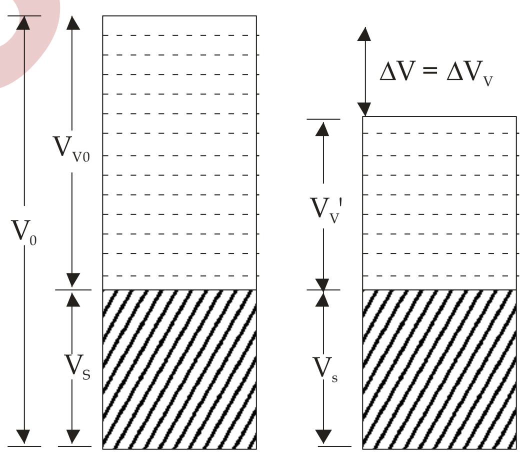
$$S_i = \frac{qB\left(1-\mu^2\right)}{E_s} \times I_t$$

→ q = Uniform pressure at the base of footing
→ B = Width of footing
→ μ = Poisson's ratio (0.35 to 0.5)
→ E = Young's modulus of soil
→ I_t = Influence factor which depends upon (L/B)

	Flexible Foundation			Rigid Foundation
	centre	corner	Avg	
Circle	1.00	0.64	0.85	0.80
Square	1.12	0.56	0.95	0.82
Rectangular				
L/B = 1.5	1.36	0.65	1.20	1.09
L/B = 2	1.52	0.76	1.31	1.22

Determination of primary consolidation settlement

(a) When void ratio is known



$$e_0 = \frac{V_{V0}}{V_S} \qquad e_{100} = \frac{V_V'}{V_S}$$

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$$\frac{\Delta V}{V_0} = \frac{\Delta H}{H_0} \times \frac{A}{A} = \frac{\Delta H}{H_0} \text{ ..... (A)}$$

$$\frac{\Delta V}{V_0} = \frac{\Delta V_V}{V_0} = \frac{\Delta V_V}{V_s + V_{v0}} = \frac{\Delta V_v}{V_s\left(1+\frac{V_{v0}}{V_s}\right)}$$

$$\frac{\Delta V}{V_0} = \frac{\frac{\Delta V_v}{V_s}}{\left(1+\frac{V_{v0}}{V_s}\right)} = \frac{\Delta e}{1+e_0} \text{ .... (B)}$$

from (A) & (B)

$$\frac{\Delta H}{H_0} = \frac{\Delta e}{1+e_0} \Rightarrow \Delta H = H_0 \times \frac{\Delta e}{1+e_0}$$

ΔH = ultimate consolidation settlement

H_0 = initial thickness of layer

$\Delta e = e_0 - e_{100} \rightarrow$ change in void ratio

(b) When coefficient of volume compressibility is known (m_v)

$$\frac{\Delta H}{H_0} = \frac{\Delta e}{1+e_0} = \frac{\Delta e}{\Delta\bar{\sigma}} \times \frac{\Delta\bar{\sigma}}{1+e_0} = a_V \times \frac{\Delta\bar{\sigma}}{1+e_0} = m_V \Delta\bar{\sigma}$$

$$\therefore \Delta H = H_0 \times m_V . \Delta\bar{\sigma}$$

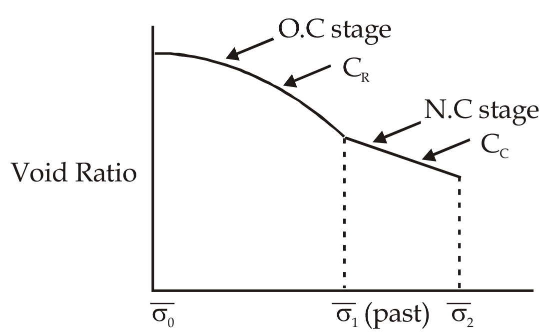
(c) When coefficient of compression is known

$$\frac{\Delta H}{H} = \frac{\Delta e}{1+e_0} = \frac{\Delta e}{1+e_0} \times \frac{\log\left(\bar{\sigma}_1/\bar{\sigma}_0\right)}{\log\left(\bar{\sigma}_1/\bar{\sigma}_0\right)} = \frac{C_C}{(1+e_0)}\log\left(\frac{\bar{\sigma}_1}{\bar{\sigma}_0}\right)$$

$$\frac{\Delta H}{H_0} = \frac{C_C}{(1+e_0)}\log\left(\frac{\bar{\sigma}_1}{\bar{\sigma}_0}\right) \qquad \Delta H = \frac{H_0 C_C}{1+e_0}\log\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

Note : If soil is normally consolidated then use coefficient of compression (C_C)

→ If soil is over consolidated then use C_R (coeff of recompression)



$$\Delta H = \frac{H_0}{(1+e_0)} \times C_R \log\left(\frac{\bar{\sigma}_1}{\bar{\sigma}_0}\right) + \frac{H_0}{(1+e_0)} \times C_c \log\left(\frac{\bar{\sigma}_2}{\bar{\sigma}_1}\right)$$

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CHAPTER - 10 (SHEAR STRENGTH OF SOIL)

→ Shear strength of a soil is the resistance offered by the soil grains against shear diformation.

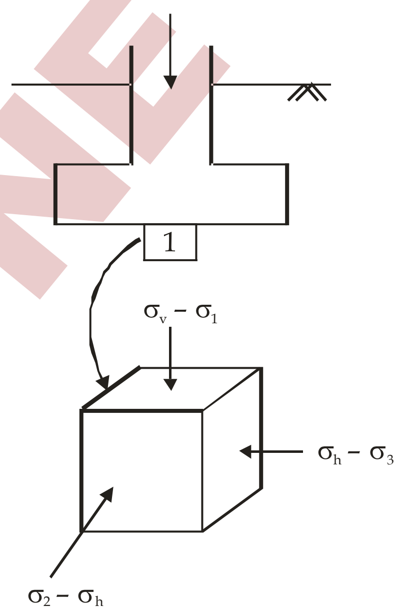
→ Soil may derive its shear strength from following parameters.

1. Friction between particles due to sliding & rolling (Eg - sand, gravel & silt)
2. Interlocking between particles (gravel & dense sand)
3. Inter molecular attraction due to cohesion or odhesion (silt & clay)

Examples of shear failure :-

1. Slidingof land mass in infinite slope
2. Failure of finite slope [face failure, base failure & toe failure]
3. Failure of soil below a building foundation (gravel, local, punching shear failure)

Stability Analysis of Sand [Mohr's Theory]



Consider an element just below the fooling

σ_1 = major principal stress

σ_2 = intermediate principal stress

σ_3 = minor principal stress

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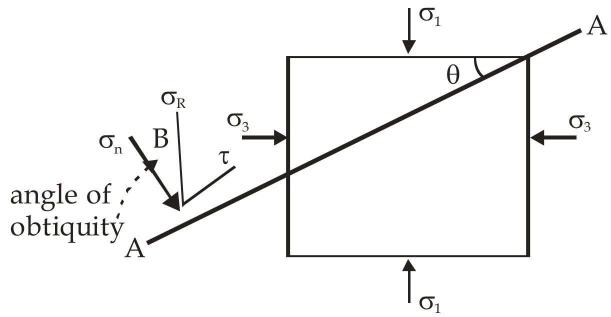
2 - D Analysis



Consider a plane at an angle θ with the major principal plane

Normal stress $\sigma_n = \left(\frac{\sigma_1 + \sigma_3}{2}\right) + \left(\frac{\sigma_1 - \sigma_3}{2}\right)\cos\theta$

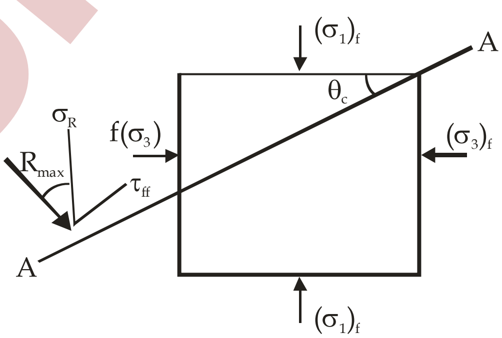
Shear stress $\tau = \left(\frac{\sigma_1 - \sigma_3}{2}\right)\sin 2\theta$

Resultant stress $\sigma_R = \sqrt{\sigma_n^2 + \tau^2}$

Angle of obliquity $\tan\beta = \frac{\tau}{\sigma_n}$

Mohr found that

→ The shear failure will occur on that plane in which resultant stress (σ_R) is most inclined with the normal of that plane such a plane is called critical plane or failure plane. If plane A - A becomes critical plane/failure plane or failure plane. If plane A - A becomes critical plane / failure plane than angle θ is called critical angle o failure plane (θ_C) plane / failure plane than angle θ is called critical angle of failure plan (θ_C)



$\tan\beta_{max} = \frac{\tau_{ff}}{\sigma_n}$

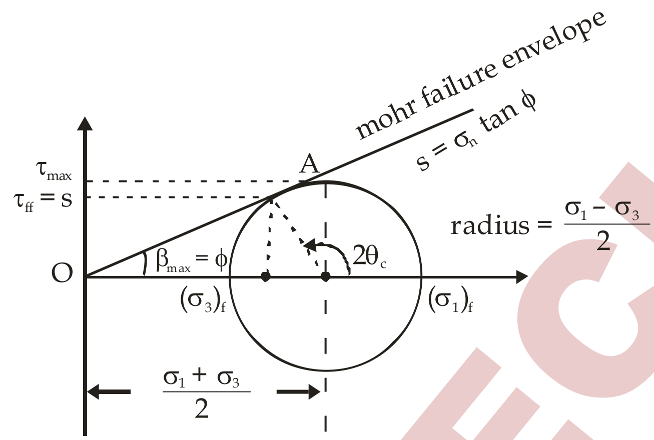
→ Shear strength (S) is equal to shear stress produced on critical plane at limiting condition (just before the failure) on the plane A - A σ_R is most inclined with normal of that plane hence β_{max} is called maximum angle of obliquity which is also termed as internal angle of friction.

$\beta_{max} = \phi =$ int. angle of friction

$$\tan \beta_{max} = \frac{\tau_{ff}}{\sigma_n} = \frac{S}{\sigma_n} = \tan \phi$$

$$S = s_n \tan \phi$$

Mohr Circle For Sand :-



$\Delta OAC\ \angle O + \angle A + \angle C = 180°$

$\beta_{max} + 90 + 180 - 2\phi 2\theta_C = 180$

$2\theta_C = \beta_{max} + 90$

$\theta_C = 45 + \frac{\beta_{max}}{2}$

$\theta_C = 45 + \frac{\phi}{2}$

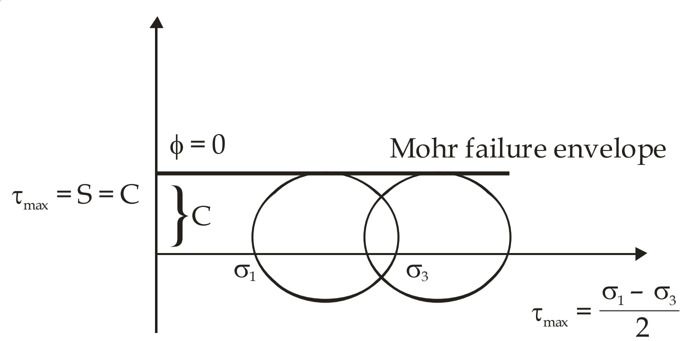
→ with maximum major principal plane

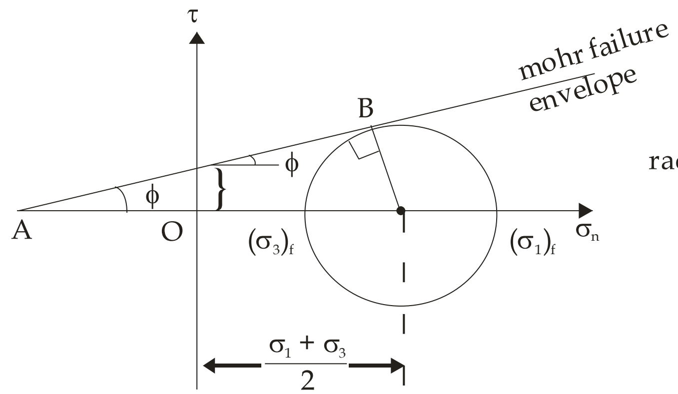
Note :-

Failure will occur on that plane in which shear stress lie on or above the mohr failure envelope whereas if the pt is located below the failure invelope than such condition represents stable condition

Note :-

In case of sand and silt $\tau_{ff} < \tau_{max}$ but in case of clay $\tau_{ff} = \tau_{max}$ in clays, failure plane will be plane of τ_{max} which forms horizontal mohr failure envelope





radius = $\frac{\sigma_1 - \sigma_3}{2}$

$$\sin\phi \Rightarrow \frac{BC}{AC} \Rightarrow \frac{BC}{AO+OC} \Rightarrow \frac{\sigma_1 - \frac{\sigma_3}{2}}{(c\cot\phi)+\left(\frac{\sigma_1+\sigma_3}{2}\right)}$$

$$\sin\phi.c.\cot\phi + \left(\frac{\sigma_1+\sigma_3}{2}\right)\sin a = \left(\frac{\sigma_1-\sigma_3}{2}\right)$$

$$2c\cos\phi + (\sigma_1+\sigma_3)\sin\phi = (\sigma_1 - \sigma_3)$$

$$\sigma_1[1-\sin\phi] = \sigma_3(1+\sin\phi) + 2C\cos\phi$$

$$\sigma_1 = \sigma_3\frac{[1+\sin\phi]}{(1-\sin\phi)} + 2C\frac{\cos\phi}{(1-\sin\phi)}$$

$$\sigma_1 = \sigma_3\frac{[1+\sin\phi]}{(1-\sin\phi)} + 2C\sqrt{\frac{\cos\phi}{(1-\sin\phi)}}$$

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

$$\cos^2\phi = 1-\sin^2\phi$$

$$\cos\phi = \sqrt{1-\sin^2\phi}$$

$$\frac{\cos\phi}{(1-\sin\phi)} = \frac{\sqrt{(1-\sin\phi)(1+\sin\phi)}}{(1-\sin\phi)} = \sqrt{\frac{1+\sin\phi}{1-\sin\phi}}$$

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

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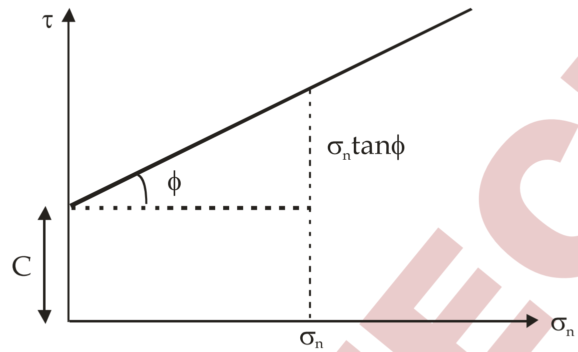
Mohr - Coulomb's Theory

Acc to this theory, shear strength of soil depends upon

1. Cohesion between particles on critical plane
2. Angle of internal friction between particles which accounts for interlocking resistance & friction resistance

   (dense sand has greater friction angle)
3. Normal stress σ_n on critical plane / failure plane which increases with depth in soil.



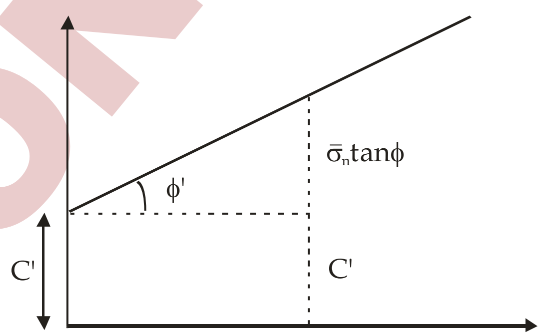
$S = C + \sigma_n \tan\phi$

C = Total cohesion

ϕ = Total friction angle

σ_n = Total normal stress at critical plane.

→ The above theory is found to be incorrect for the condition & when water table is present hence the above theory is modified & improved by Terzaghi which is called modified mohr coulomb's theory



$S = C' + \bar{\sigma}_n \tan\phi'$

C' = effective cohesion

ϕ' = effective friction angle

$\bar{\sigma}_n$ = effective normal stress at critical plane.

Note :- C & ϕ are not fundamental properties of soil. These depend on, type of test, water content & drainage condition under which the testing is done. Test must represent the field condition. The field condition could be rapid or slow construction, granular or clayey soil, drains or undrained condition

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Limiations :

1. Mohr failure envelope is approximated to a st line which is found invalid for over consolidated soils.
2. Practical results show that mohr failure envelope in O.C soil is found to be slightly curved
3. The analysis is 2 -D in which σ_1 and σ_3 are considered whereas actual stress condition in soil is 3-D in which $\sigma_2 = \sigma_3$

Type of Shear Parameters

Undrained Shear Strength

→ If saturated soil mass is subjected to shear loading & soil is undrained (pore pressure is developed) such as in unconsolidated undrained and consolidated undrained test in clay then undrained shear strenght parameters computed and total stress should be used

Trixial shear test = $\sigma_1 = \sigma_3 \tan^2\left(45+\frac{\phi}{2}\right) + 2C\tan\left(45+\frac{\phi}{2}\right)$

Direct shear test $S_u = C + \sigma_n \tan\phi$

C/Cu = Total / undrained cohesion

$\phi/\phi u$ =Total / undrained friction angle

σ_n = Total normal stress on failure plane / critical plane

Note :

If fast loading is done on saturated clay sample then in short term condition, undrained shear parameters should be determined

Drained Shear stress

→ If loading rate is slow & expulsion of porewater completes during loading then pore pressure will become zero. Eg- consolidated drained test then effective stress should be used & drained shear parameters are given as

Triaxial shear test $\bar{\sigma}_1 = \bar{\sigma}_3 \tan^2\left(45+\frac{\phi}{2}\right) + 2C\tan\left(45+\frac{\phi}{2}\right)$

Direct shear test $S_d = C' + \bar{\sigma}_n \tan\phi$

$s_d = c_d + \bar{\sigma}_n \tan\phi d$

C'/cd = effective/drained cohesion

$\phi'/\phi d$ = effective/drained friction angle

$\bar{\sigma}_n$ = effective normal stress on critical plane

Note :-

Loading on sand at slow rate for short term & long term both whereas in clay for long term condition under slow loading rate drained shear parameters should be determined.

Types of Triaxial Shear Test (On the basis of drainage conditions)

→ The selection of test will depend upon type of soil, purpose of test and drainage condition

→ There are two stage of soil loading

1. Confining pressure stage / cell pressure stage
2. Shear stage/ deviator stage

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(I) Unconsolidated - Undrained Test / Quick Test / UU Test

→ This test takes only 5 - 7 minutes. In this test expulsion of pore water is not permitted in both the stages.

→ Suitable for saturated clay for short term analysis under undrained condition at fast loading rate.

(II) Consolidated - Drained Test :-

→ It is maximum time taking test. The rate of consolidation is very slow hence it may take few weeks for fine clays.

→ In this test expulsion of pore water is permitted in both the stages

→ Suitable for saturated sand (short term & long terms both) & can also be used for long term stability analysis of clay.

(II) Consolidated - Undrained Test / CU test :-

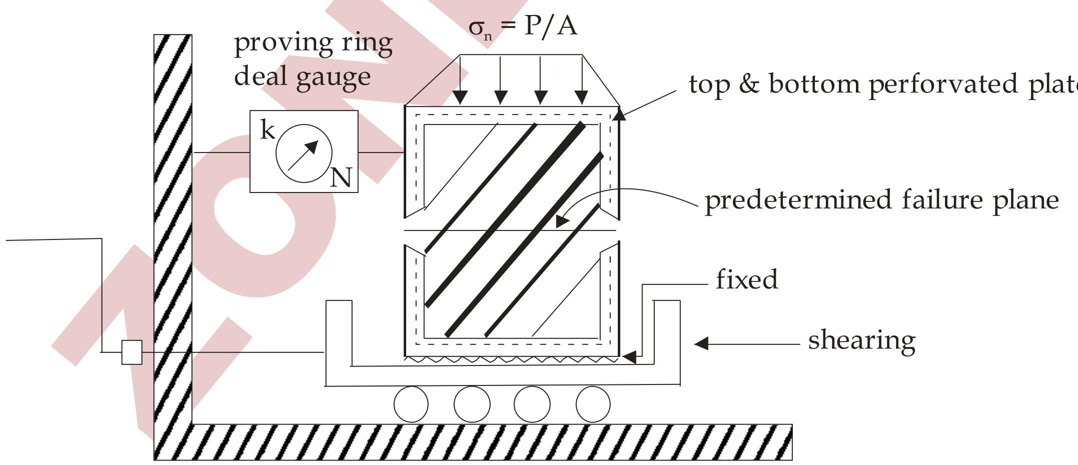
→ In this test expulsion of pore water is permitted in 1st stage but not in 2nd stage.

→ It is suitable for investigation of safety against failure of earthen dam which may occur down to sudden draw down of water to be

Note :

Unconsolidated drained test (UD) test is no performed parcticaly confining pressure acts from a very time & if soi is unconsolidated in long time then it connot be drained in smal pressure of shear loading

Direct Shear Test



→ Shear box is either square or circular having ring 60 - 90 mm .

→ Since there is no mechanism to measure pore pressure hence this test is preferred under drained condition. However it can be conducted under undrained condition also.

→ There are 2 mechanisms to conduct this test

(a) strain controlled - screw

(b) stress controlled - with the help of pulley and wt.

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→ Strain controlled is preferred in most of the cases.

→ Saturated sample of soil is placed in shear box. It has two parts, lower and upper which are seperated to each other.

→ A normal force (say P_1) is applied from top when expulsion of pore water stops shearing is introduced on a pre determined horizontal plane.

→ At constant normal stress shear $\left(M = \frac{P_1}{A}\right)$ shear displacement is given and shear resistance is recorded on proving ring dial gauge wt N_1 is proving ring dial gauge reading at shear failure under normed stress σ_{N1} then shear stress at failure is given as

shear stress = $\frac{SF}{A}$

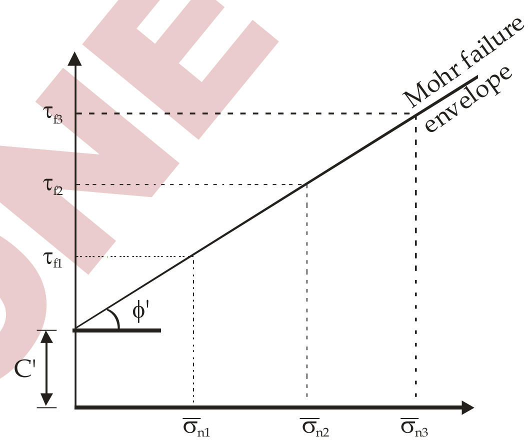
$$\tau_{f1} = \frac{kN_1}{A}$$

→ σ_{N1} and τ_{f1} are plotted on graph & test is repeated at different valus of normal stress and corresponding shear stress at failure (τ_f) is noted which are plotted on the graph

→ It test is drained then

$$\sigma = \bar{\sigma}_n + u$$

$$\sigma = \bar{\sigma}_n$$



$$\tau_f = S = C + \bar{\sigma}_n \tan\phi$$

Limitations :-

1. Failure plane is pre determined which way not be the weakest plane
2. No mechanism to measure pore pressure
3. No control over drain age condition
4. The stress distribution on failure plane is not uniform
5. Stress conditions are known only at failure as it is difficult to draw mohr circle from single observation

Triaxial Shear Test ;-

→ This test is conducted in 2 stages in which drainage valves may be opened or closed (drained or undrained)

→ A cylindrical soil specimen is prepared from a saturated soil mass which is inclosed inside impermeable rubber membrane. The length of sample is usually 2 times its dia

→ There is complete control over drainage condition & there is mechanism to and measure pore pressure

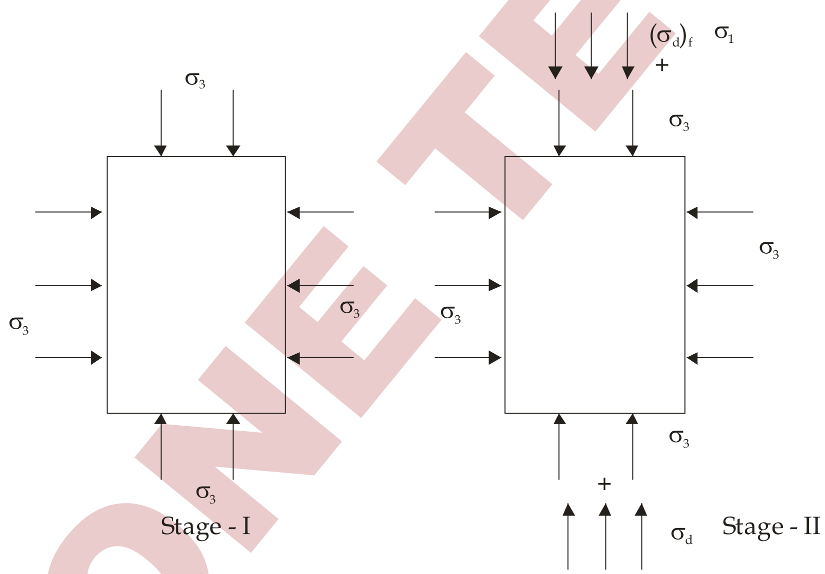
Stage - 1 Confining Pressure stage / cell pressure stage/ consolidation stage

→ All around confining pressure (σ_3/ σ_C) is applied using external water pressure.

→ If test is unconsolidated then drainage valves will be closed

→ If test is consolidated then drainage valves will be opened and expulsion of pore water is permitted.

→ When expulsion of porewater stops 1st stage will be completed



Stage - II :- Deviator stage / Shear stage / Back pressure stage

→ Confining pressure is kept constant and additional axial stress is applied called as deviator stress .

→ Deviator stress is increase gradually untill the soil fails in shear

→ If test is undrained then drainage valves will be closed and if test is drained then valves will be opened

Note :- Deviator stress at failure is termed as confined compressive strength (CCS)

$$CCS = (\sigma_1 - \sigma_3) = \frac{P}{A}$$

Area of C/S of specimen at filure

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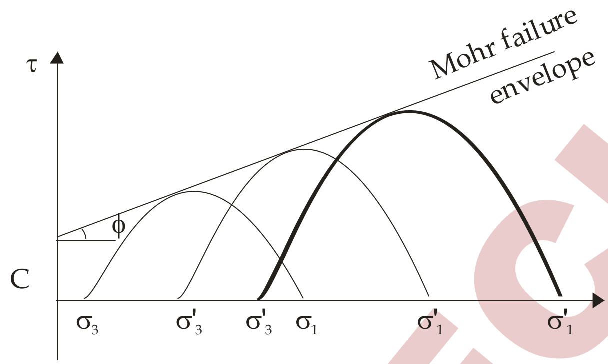
→ For σ_1 & σ_3 at failure a mohr circle is drawn and test is repeated another sample of same soil with changed value of minor principal stress (σ_3) and corresponding deviator stress and major principal stress (σ_1) is noted and mohr circle for each case is drawn.

→ A common tangent to all mohr circles will give Mohr failure envelope from which shear parameters C and ϕ can be determined



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

Note :- If mohr circles are plotted for total stress (σ_1 & σ_3) then total/ undrained shear parameters (C & ϕ) can be obtained if we use effective stress $\left[(\bar{\sigma}_1 = \sigma_1 - u) \& (\bar{\sigma}_3 = \sigma_3 - u)\right]$ be draw the mohr circles then effective drained shear parameters C' & ϕ' can be obtained

Area of C/S at the time of failure (A_f)

Let $A_0 = \frac{\pi}{4}D^2$ is initial C/S area

$$(\text{Area})_f = \frac{(\text{vol.})_f}{(\text{length})_f} = \frac{V_0 \pm \Delta_V}{L_0 - \Delta_L} = \frac{V_0\left[1 \pm \frac{\Delta_V}{V_0}\right]}{L_0\left[1 - \frac{\Delta L}{L_0}\right]} = A_0\left[\frac{1 \pm \Sigma_V}{1 - \Sigma_L}\right]$$

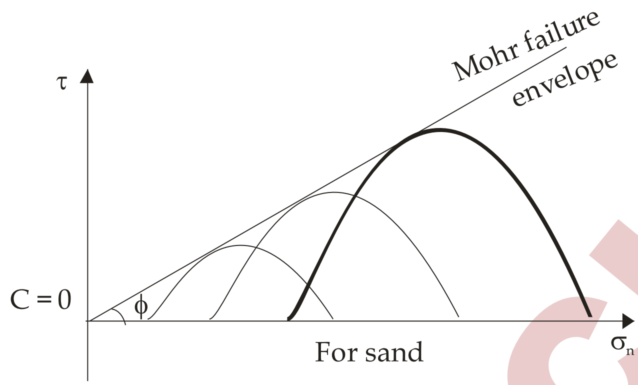
$$A_f = A_0\left[\frac{1 \pm \Sigma_V}{1 - \Sigma_L}\right]$$

In case of U - U test $\Delta V = 0$

$$\therefore A_f = A_0 + \frac{A_0}{1 - \Sigma_L}$$

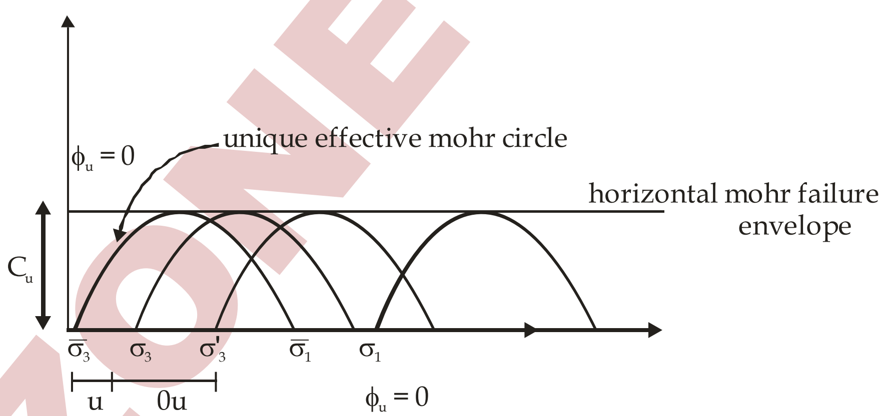
Type of Mohr Failure Envelope Obtained In Traxial Test

1. **For Sand**



2. **For Saturated Clays Under Undrained Condition**

If unconsolidated undrained test (UU test) is condcted on saturated clay then the behaviour of clay is cohesive and no friction is observed when test results are plotted then radius of all the mohr circles is found to be equal and if mohr circles are plotted in effective stress then unique effective mohr circle will be obtained

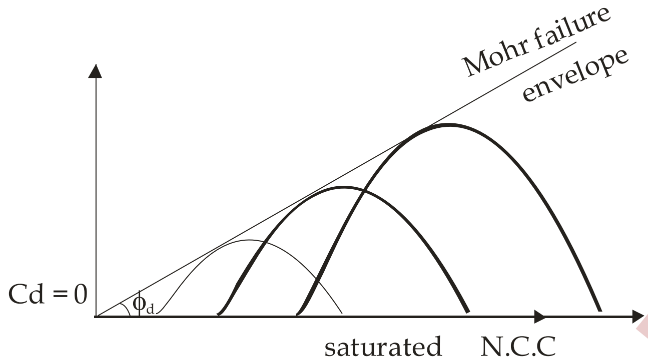


C_u = 25 - 30 kN/m^2 → for NCC

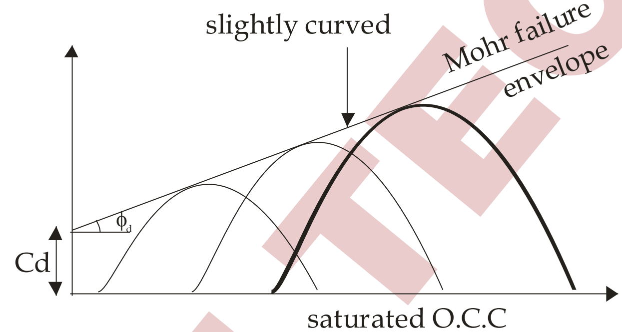
C_u = 100 - 200 kN/m^2 → for OCC (over consolidated clay)

3. **Saturated clay under drained condition**

If consolidated drained test (CD test) is conducted on saturated clays then behaviour of normally consolidated clay is similar to that of sand whereas behaviour of over consolidated clay is similarly to that of silt.

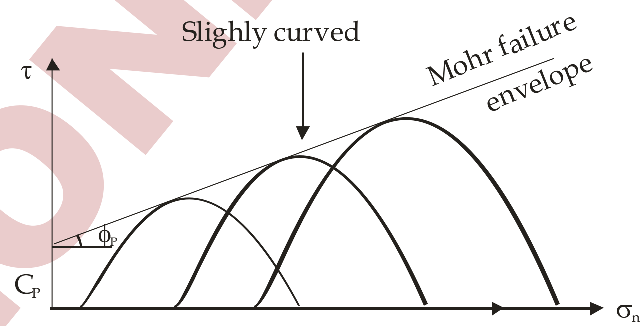


$Cd = 60 - 100\ kN/m^2$
$\phi a = 10 - 15$



3. Partially Saturated Clay Under Undrained Condition

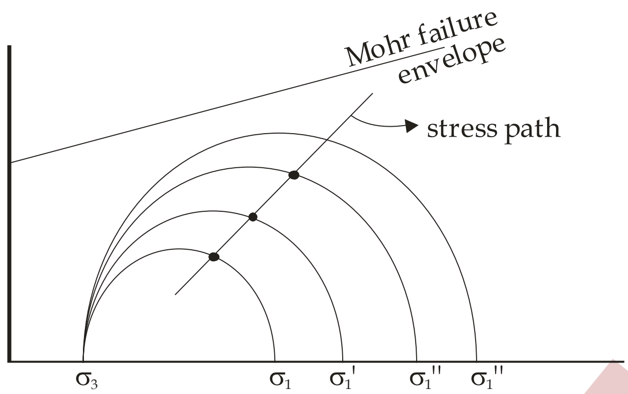
→ It behaves similar to that of silt

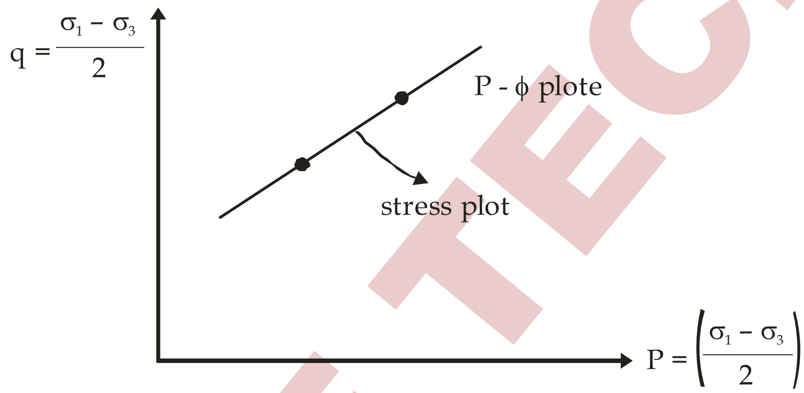


$C_P = 60 - 70\ kN/m^2$ (for drained condition)
$C_P = 80 - 150\ kN/m^2$
$\phi_P = 0 - 5\ kN/m^2$

Stress Path :-

→ Stress path is a curve which shows change in stresses as the load acting on the soil specimen changes. It is a curve drawn through the pts representing te maximum shear stress acting on the specimen as the load is changed in other words, it is the locus of pts of maximum shear stress experienced by a specimen as the load change take place





Where excavating σ_1 reduces and σ_3 remains same

Merits of Triaxial Test :-

1. Failure plane is not pre determined and it is the weakest plane
2. Mohr circle can be drawn at failure from single observation
3. Stress distribution on failure lane is much more uniform
4. There is complete control over drainage condition
5. There is mechanism to measure pore pressure
6. Suitable for all types of soil
7. Results are accurate and can be used in research work

Unconfined Compressive Strength Test (UCS)

→ It is special case of triaxial test in which confining pressure is zero ($\sigma_3 = 0$). It means there is no 1st stage no rubber membrane is required without rubber membrane dry soil & sand cannot be held in position hence this test can be conducted in saturated silt & clay. But it is more suitable for clay.

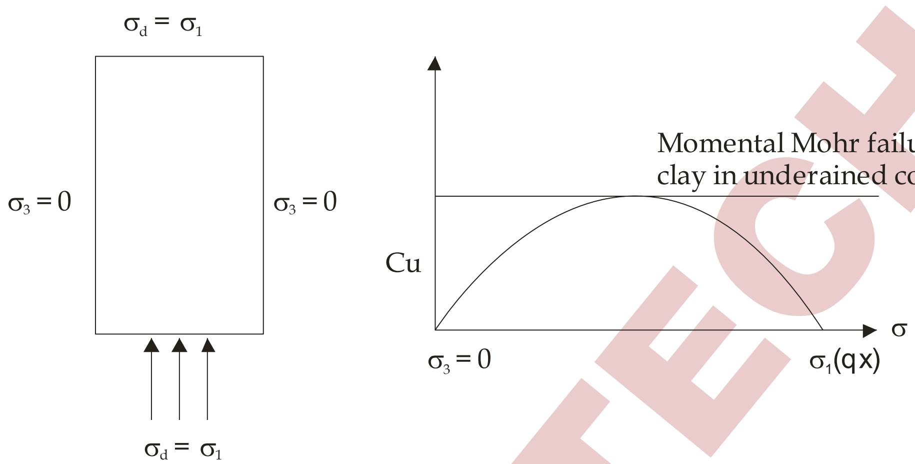
→ The saturated sample is subjected to axial loading & let deviator stress at failure is σ_d

Note :- Deviator stress at failure is called unconfined compressive strength (q_u)

$$UCS = (\sigma_d)_f = (\sigma_1)_f = \frac{P}{Af}$$

Af = area of C/S of specimen at the time of failure

because σ_3 = 0 hence σ_1 obtained at failure will be same for each sample of some soil hence a unique Mohr circle is obtained which passes through origin.



Note :- Axial load is applied rapidly to failure so that the speciman does not get dislodged (undrained conditions prevail)

→ The test is suitable for clay under undrained condition for which mohr failure envelope is a horizontal line UCS = unconfined compressive strength = (σ_d)d = σ1

$\sigma_1 = \sigma_3 \tan(45+\phi/2)+2c\tan(45+\phi/2)$

= 2c tan (45 + ϕ/2)

For clays in undrained condition [ϕ = 0]

UCS = σ_1 = 2C

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

VANE SHEAR TEST

→ A can also be used to find L.L as shear strength at LL $\approx 2.7 kN/m^2$

→ In this test there is no mechanism to measure porepressure and no drainage facility is provided. Hence is can be conducted only under undrained condition.

→ Suitable for soft saturated clay. (like marine clay) to find undrained shear strength & it can also be used to find sensitivity of clay

→ It can be conducted in lab as well as in the fluid. The mechanism is same the difference is only in the size of vane.

→ The vane is punched into the soil & torque is applied by rotating the vane at 6°/minute.

→ The vane is calibrated to a spring having orginal stiffness K. The torque & shear failure is determined at

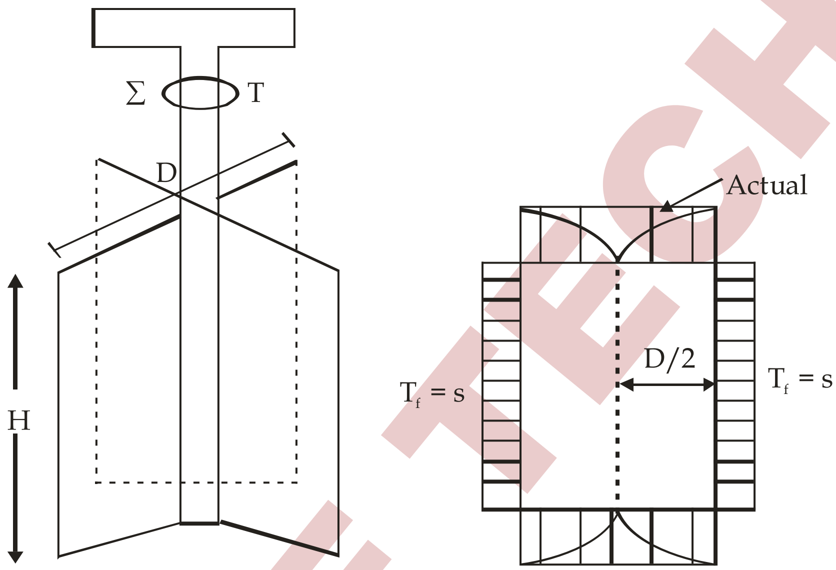
$T = k\theta$

θ = angular rotation of vane at which complete shear failure occurs.

→ While punching the ane in the hole following 2 condition may occur

2 way shearing :-

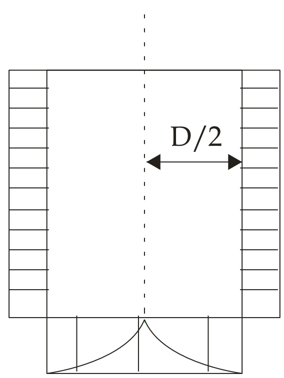
If vane is punched such that top of vane is at certain depth below the ground level then shearing occurs at sides & at top & bottom both.



$$T_f = \frac{T}{\pi D^2\left[\frac{H}{2}+\frac{D}{6}\right]} \quad \text{2 - way shearing}$$

1 - way shearing :-

If vane is punched into the soil such that top of vane is at ground level then shearing will occur at sides and bottom only



$$S = T_f = \frac{T}{\pi D^2\left[\frac{H}{2}+\frac{D}{12}\right]} \quad \text{1 - way shearing}$$

ZONE TECH

Determination of Sensitivity of Clay (S_t)

$$St = \frac{(UCS)_{undisturbed}}{(UCS)_{remoulded}} = \frac{(2C)_{undisturebed}}{(2C)_{remoulded}}$$

$$St = \frac{\text{Undisturbed Vane Shear Strength}}{\text{Remoulded Vane Shear Strength}}$$

$$St = \frac{\dfrac{T_{undisturbed}}{\pi D^2\left[\cancel{\dfrac{H}{2}}+\cancel{\dfrac{D}{6}}\right]}}{\dfrac{T_{remoulded}}{\pi D^2\left[\cancel{\dfrac{H}{2}}+\cancel{\dfrac{D}{6}}\right]}} = \frac{T_{undistrubed}}{T_{remoulded}}$$

Undrained shear strength of clay S = $C+\bar{\sigma}_n$

$$S(t) = \frac{(C)_{undisturbed}}{(C)_{remoulded}}$$

→ Initially vane shear test is performed on undisturbed soil. Hence shear strength of undisturbed clay is known.

→ Vane is further rotated quickly several time. Hence soil becomes remoulded. Vane shear Test is again performed on remoulded soil to find remoulded shear strength

PORE PRESSURE PARRAMETERS

→ If it is not possible to measure pore pressure by practical means then theoritical approach given by skempton can be adopted

→ Parameter B & A represent the response of change in pore pressure due to change in vertical pressure under uundrained condition.

→ The pore pressure change can be classified in two stages.

Stage - 1 :- Confining pressure stage.

The parameter B represents the ratio of change in pore pressure to the change in conerning pressure

$$B = \frac{\Delta U_c}{\Delta \sigma_{dc}} = \frac{\Delta U_c}{\Delta \sigma_3}$$

For dry soil : B = 0

For saturated soil : B = 1

For partially saturated soil : 0 < B < 1

Stage - 2 Deviator stage/Shear stage

→ The parameter A is defined in terms of anoter parameter $\vec{A}$ such that $\vec{A} = A.B$

→ The parameter $\vec{A}$ represents the ratio of change in pore pressure to the change in deviator stress in shear stage under undrained condition

$$\vec{A} = A.B$$

$$\vec{A} = \frac{\Delta U_d}{\Delta \sigma_d} = \frac{\Delta Ud}{\Delta \sigma_1 - \Delta \sigma_3}$$

→ The parameter A depends on the degree of saturation OCR, strain in soil and stratification of soil

→ It value may be as low as –0.5 (for highly soil sonsolidated soil & dense sand) & may be as high as +3 (for loose sand)

→ If confining pressure & deviator stress both change then total change in pore pressure is given a

$$\Delta U = \Delta Uc + \Delta Ud$$

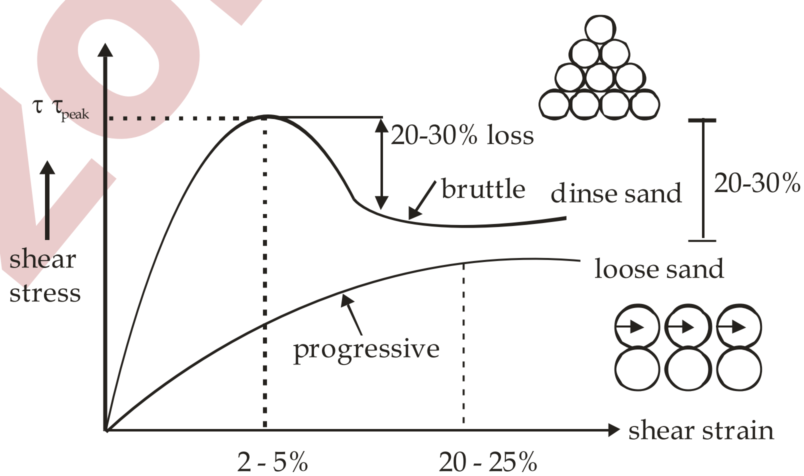
$$\Delta U = B.\Delta \sigma_3 + \vec{A}(\Delta \sigma_1 - \Delta \sigma_3)$$

$$\Rightarrow B.\Delta \sigma_3 + A.B(\Delta \sigma_1 - \Delta \sigma_3)$$

$$\Delta U = B[\Delta \sigma_3 + A(\Delta \sigma_1 - \Delta \sigma_3)]$$

SHEAR CHARACTERISTICS OF SAND :-

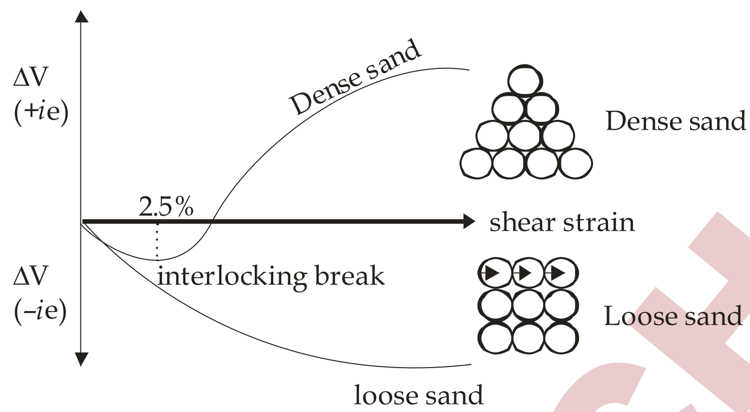
→ In dense sand the shear strength is due to interlocking & friction both. Interlocking resistance my be 20 - 30% of total strength .and at 2 - 5% of strain, interlocking breaks & suddenly decrease in shear strength is observed. Brittle failure will be obtained in dense sand, higly O.C clay & in sensitive clays.



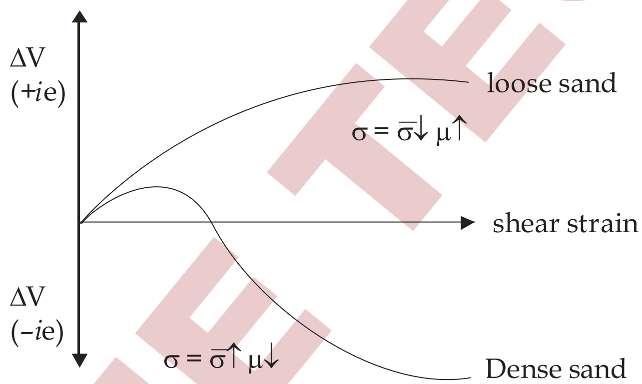
→ Loose sand has only frictional resistance .In loose sand volume continues to decrease & a clear failure does not occur hence failure is assumed when strain is reached 20 - 25% progressive failure occurs in loose sand, remoulded clay.

EFFECT OF DYNAMIC OR SEISMIC LOADING ON SATURATED SAND

(I) Volume change vs shear strain

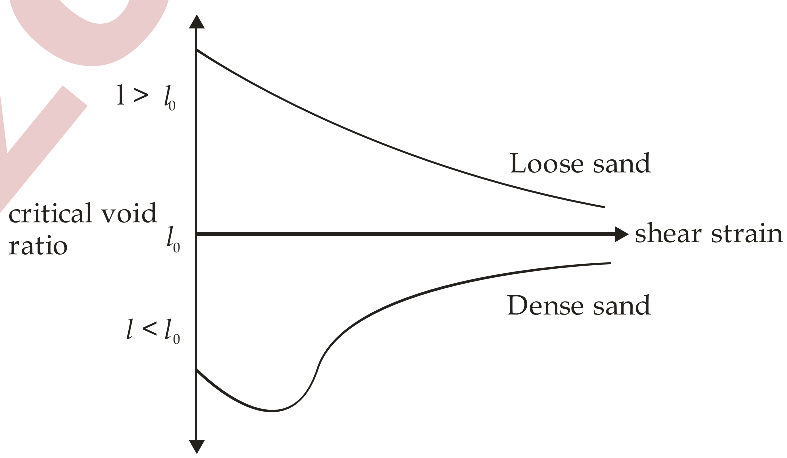


(II) Change in pore pressure vs shear strain



→ During EQ the damage is more in loose sand than in dense sand [more in sea or river soil in land mass]

(III) Void Ratio V shear strain



l_0 = critical void ratio it is that void ratio at which there will be no change in volume due to shear disterance

ZONE TECH

LIQUEFACTION :-

In loose saturated sand, due to seismic disturbance or dynamic loading, volume decreases hence pore pressure change is +ive. Due to built up of high pore pressure sudden ↓in eff. stress and ↓in shear strength is recorded hence large settlement of foundation suddenly occurs along with the vertical upward flow of muddy water such a phenomenon is called liquefaction of sand.

It is generally observed near rivers & sea during siesmic (Eθ)

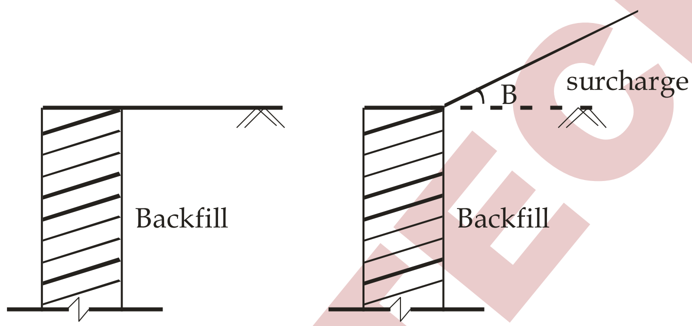
ZONE TECH

Chapter - 11 (Earth Pressure)

→ In the designing of retaining walls, sheet pile walls and other retaining structures. Pressure exerted by material retained by these structure is also considered.

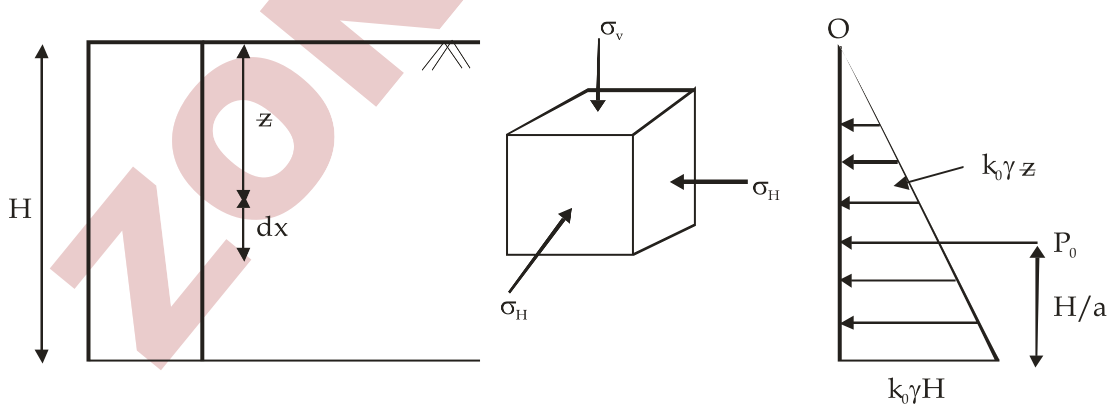
→ This pressure exerted by retained material is termed as earth pressure.

→ A retaining wall or structure is termed as backfill which may have its top surface horizontal (or) inclined.



Earth Pressure at rest Condition :-

If the wall is rigid and unyielding, soil retained by it is in the state of rest and there is no displacement or deformation the pressure exerted by the soil on the wall in this stage is termed as earth pressure at rest condition.



* **IN ABOVE FIGURE REPLACE H/a with H/3**

σ_v = vertical earth pressure

σ_h = horizontal earth pressure

$\frac{\sigma_h}{\sigma_v}$ = k = Earth pressure coefficient

At rest condition

$$\epsilon_H = 0$$

$$\frac{\sigma_H}{E} - \mu\frac{\sigma_h}{E} - \mu\frac{\sigma_v}{E} = 0$$

$\sigma_H (1 - \mu) = H\sigma_v$

$$\frac{\sigma_h}{\sigma_v} = \frac{\mu}{1-\mu} = k_0 \qquad \mu \text{ – poissons ratio}$$

Earth pressure coeff. at rest $k_0 = \frac{\mu}{1-\mu}$

$\sigma_h = k_0\sigma_v \qquad p_0 = k_0\sigma_v$

At depth z, $\qquad \sigma_v = \gamma z$

$p_0 = k_0\gamma z$

at $z = 0, p_0 = 0$

at $z = H, p_0 = k_0\gamma_H$

Note :

For pure sand, $k_0 = 1 - \sin\theta$

for O.C soil, $k_0 = (k_0)_{N.soil}\sqrt{0.C.R}$

Active and passive earth pressure

During the active stage, wall moves away from the backfill the portion of the backfill just behind the wall leaves the rest of the soil mass and moves along with the wall. This portion of the backfill which moves along with the wall is known as failure wedge. The resistance of the soil due to its shear strength is developed at upward and outward direction on the rupture surface which decreases the pressure on the wall.

This decrease in pressure takes place upto a particular extent where entire shear resistance of the soil is mobilised

The minimum pressure acting on the wall at this stage is termed as active pressure.

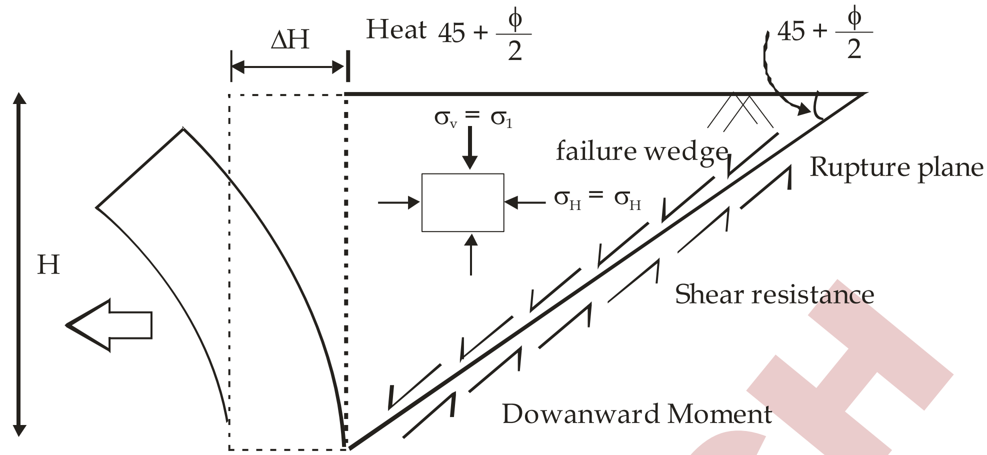
Passive Stage :-

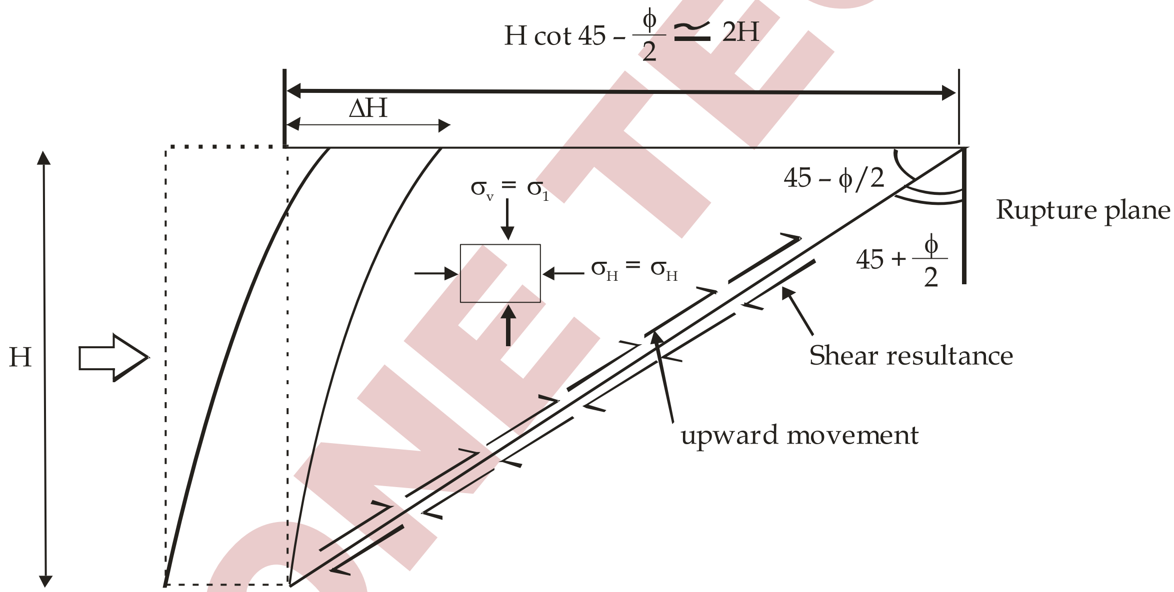
In this case, if the wall moves towards the backfill, the shearing resistance of the soil builds up against the wall, which results in increase of pressure of the wall. This increase in pressure takes place upto an extent when the entire shear resistance of the soil is mobilised, The maximum pressure acting on the wall is termed as passive earth pressure. For passive case, strain required is in the order of 2% to 15%

2% - dense sand

15% - loose sand

ZONE TECH





Rankine Earth Pressure Theory.

Assumptions :

1. Soil is homogeneous, isotropic, semi infinite, elastic, dry and cohesionless
2. The ground surface is planar which may be horizontal an inclined.
3. The face of wall in contact with backfill is vertical and smooth.
4. Soil is in the state of plastic equilibrium in passive and active earth pressure condition.
5. The rupture surface is planar

From stress relationship of soil

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

for cohesionless soil , c = 0

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

For active stage :-

$$\sigma_1 = \sigma_v$$

$$\sigma_3 = \sigma_H$$

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

$$\frac{\sigma_H}{\sigma_v} = \cot^2\left(45+\frac{\phi}{2}\right) = k_a$$

$$\frac{\sigma_h}{\sigma_v} = k_a$$

$$p_a = k_a \sigma_v$$

If $\sigma_v = \gamma z$

$$P_a = k_a \gamma z$$

For passive stage :-

$$\sigma_1 = \sigma_H$$

$$\sigma_3 = \sigma_v$$

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

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

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

Passive Earth Pressure Coefficient.

$$\frac{\sigma_H}{\sigma_V} = k_p$$

$$p_p = \sigma_H = k_p \sigma_v$$

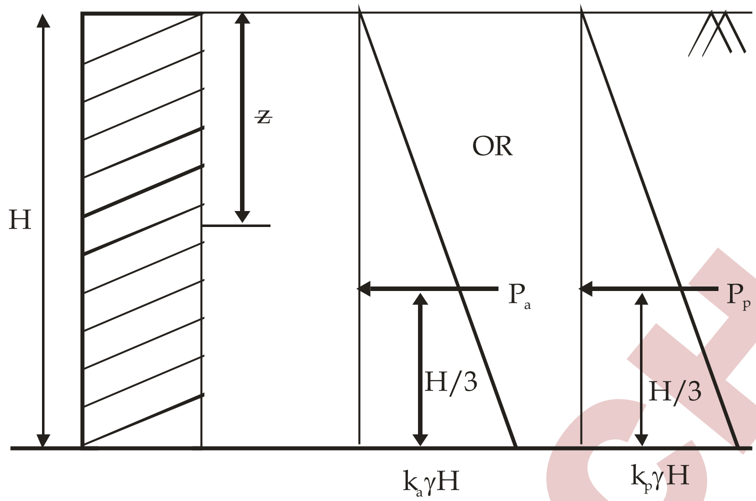
If $\sigma_v = \gamma z$

$$p_p = k_p \gamma z$$

ZONE TECH

Note :-

$k_a . k_p = 1$



$\sigma_v = \gamma z$

Active Earth Pressure	Passive Earth Pressure
$p_a = k_a \sigma_v$	$p_p = k_p \sigma_v$
$= k_a \gamma_z$	$= k_p \gamma_z$
$@z = 0$, $Pa = 0$	$@z = 0$, $Pa = 0$
$@z = H$, $p_a = k_a \gamma_H$	$@z = H, p_p = k_p \gamma_H$

Considering unit length of wall

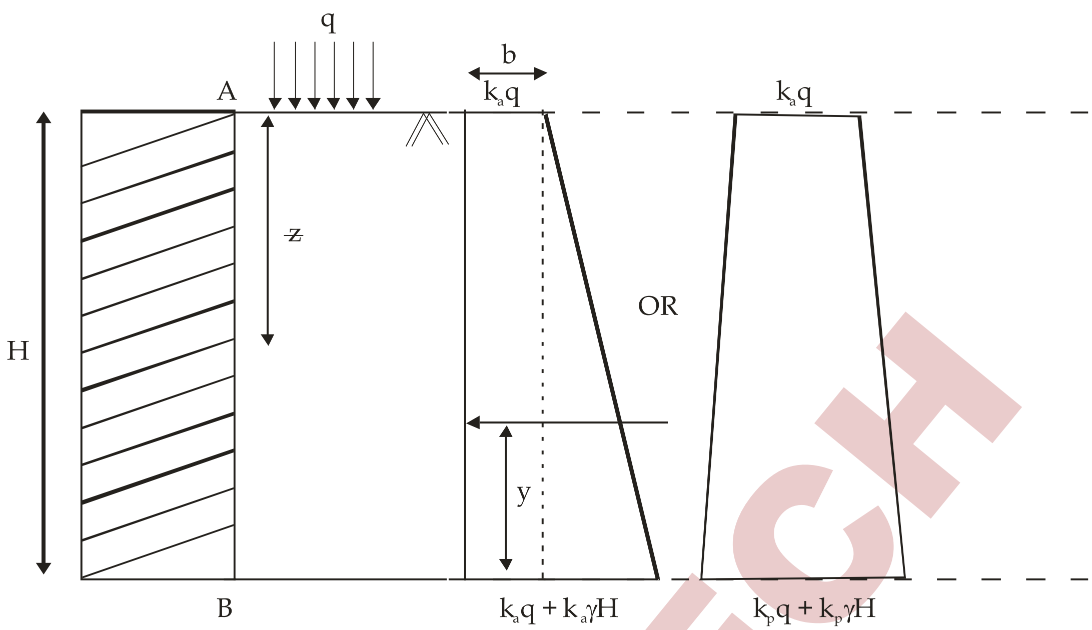
Total Earth pressure

$P_a = \frac{1}{2} k_a \gamma H^2$ will act at $\qquad$ $P_p = \frac{1}{2} k_p \gamma H^2$ will act at

$\bar{z} = \frac{H}{3}$ from base $\qquad$ $\bar{z} = \frac{H}{3}$ from base

Case-2 :

Cohesionless backfill with uniform surcharge :-



NOTE:- IN THE ABOVE FIGURE DELETE THE LAST PRESSURE DIAGRAM

$\sigma_v = q + \gamma z$

$\bar{y} = \left(\frac{a+2b}{a+b}\right) \cdot \frac{H}{3}$

Active Earth Pressure

$P_a = k_a \sigma_v$

$= k_a q + k_a \gamma_z$

$@z = 0,\ P_a = k_a q$

$@z = H,\ P_a = k_a q + k_a \gamma_H$

Passive Earth Pressure

$p_p = k_p \sigma_v$

$= k_a q + k_p \gamma_z$

$@z = 0,\ p_p = k_p q$

$@z = H,\ P_a = k_p q + k_p \gamma_H$

Considering unit length of wall

Total earth pressure

$P_a = P_{a1} + P_{a2}$

$= k_a \gamma H + \frac{1}{2} k \gamma g H^2$

P_a will act at

$$\bar{z} = \frac{p_{a1}\frac{H}{2} + p_{a2}\frac{H}{3}}{P_{a1} + P_{a2}}$$

from base

$p_p = p_{p1} + p_{p2}$

$k_p \gamma H + \frac{1}{2} k_p \gamma H^2$

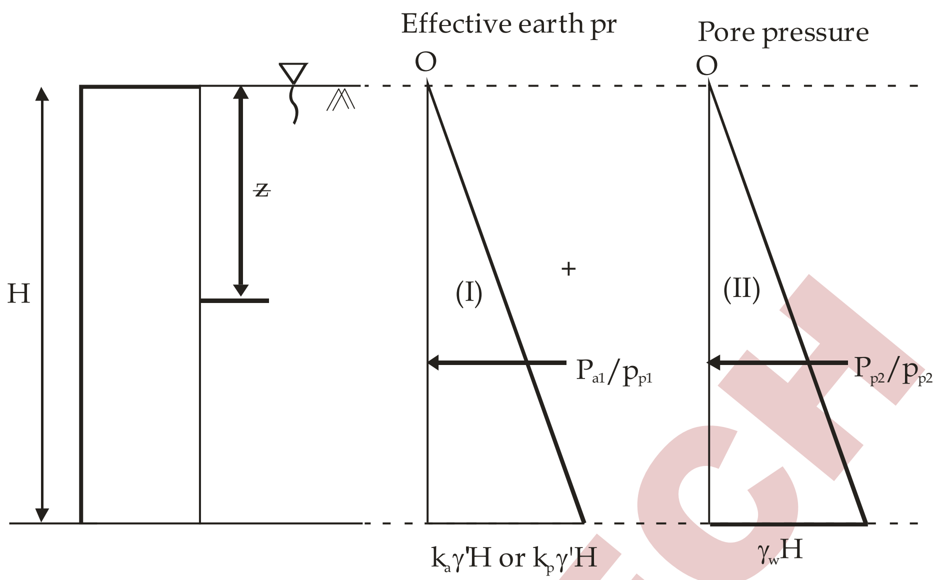
P_p will act at

$$\bar{z} = \frac{p_{p1}\frac{H}{2} + p_{p2}\frac{H}{3}}{P_{p1} + P_{p2}}$$

from base

Case - 3 Submergd Backfill

ZONE TECH



$\sigma_v = \gamma_{sat}\ z = \gamma'\ z + \gamma_w z$

Active Earth Pressure

$Pa = k_a s_v$

$= k_a \gamma' z + g_w z$

$\gamma' \because \gamma_w$ is

seperated

from γ_{sat}

$@ z = 0, p_a = 0$

$@ z = H, p_a \gamma' H + \gamma_w H$

$P_a = ka\gamma'H + \gamma_w H$

Total earth pressure

$P_a = P_{a1} + P_{a2}$

$= \frac{1}{2} k_a \gamma' H^2 + \frac{1}{2}\gamma_w H^2$

P_a will act at $z = \frac{H}{3}$

from base

Passive Earth Pressure

$p_p = k_p \sigma_v$

$= k_p \gamma' z + \gamma_\omega z$

water pressure

will not be

multiplied by ka/kp

$@ z = 0, p_p = 0$

$@ z = H, p_p = k_p \gamma' + \gamma_w H$

$p_p = k_p\gamma'H + \gamma_w H$

$P_p = P_{p1} + P_{p2}$

$= \frac{1}{2} k_p \gamma' H^2 + \frac{1}{2}\gamma_w H^2$

P_p will act at $z = \frac{H}{3}$

from base

depth $(\gamma_w z)$ in submerged condition which will not be multipled by k_a or k_p.

Water pressure is not considered for designing of retaining wall because it may cause uneconomy in construction thus weep holes are provided in the retaining wall

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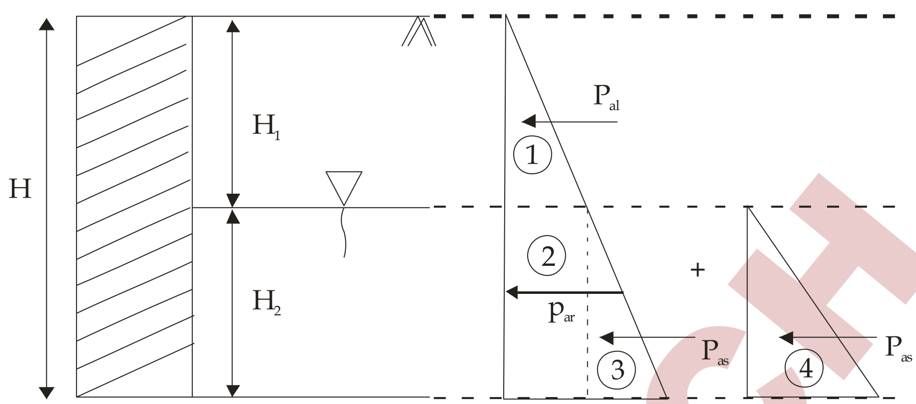
Case-IV :

Submerged backfill with W.T @ depth H_1 from G.L



Above the water table

$\sigma_v = \gamma_b z$

Active Earth Pressure

$P_a = k_a \sigma_v$

$= k_a \gamma_b z$

Passive Earth Pressure

$p_p = k_p \sigma_v$

$= k_p \gamma_b z$

Active & passive earth pressive for cohesive soil. Rankine theory was taken forward by bell in cohesive soil. As cohesive soil are partially self supporting soils, active pressure exerted by them on the wall is comparatively less than cohesionless soil. But passive pressure exerted by them is comparatively more than cohesionless soil.

From stress relationship of soil

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

For active stage

$\sigma_1 = \sigma_v$

$\sigma_3 = \sigma_H$

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

$$\sigma_H = \sigma_V \cot^2\left(45 + \frac{\phi}{2}\right) - 2c \cot\left(45 + \frac{\phi}{2}\right)$$

w.k.t

$$\cot^2 = \left(45 + \frac{\phi}{2}\right) = ka$$

$$\sigma_H = \sigma_v k_a - 2C\sqrt{k_a}$$

If $\sigma_v = \gamma z$

$$\sigma_H = k_a \gamma z - 2c\sqrt{k_a}$$

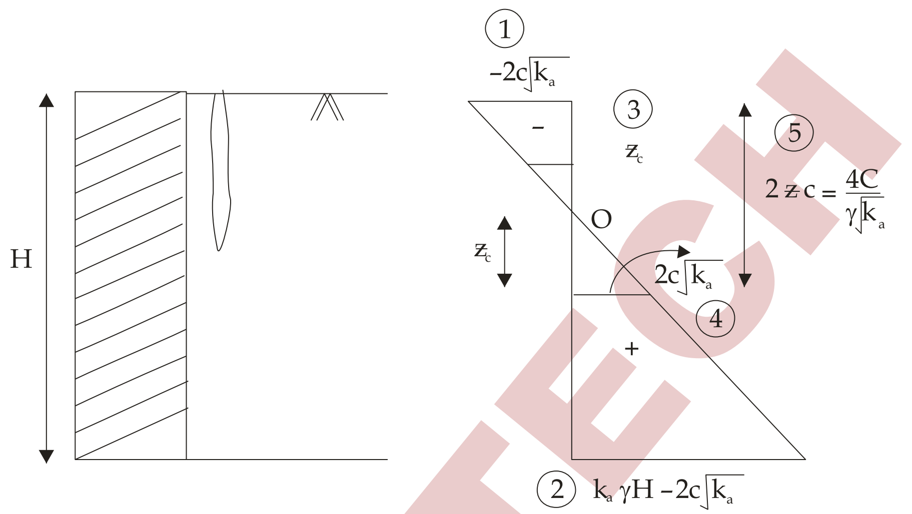
ZONE TECH

$@z = H, p_a = k_a \gamma_H - 2c\sqrt{k_a}$

If $@z = z_c, p_a = 0$

$k_a \gamma H_c - 2c\sqrt{k_a} = 0$

$$z_c = \frac{2c}{\sqrt{k_a}}$$



Note : 1

Cohesion in cohesive soils reduces the active pressure at any given depth by an amount of $2c\sqrt{ka}$ in comparison to cohesionless soil.

Note : 2

Total net pressure in cohesive soil in active stage upto the depth of $2z_c$ is zero hence these cohesive soils can sand with their face up, upto this depth without any lateral support therefore this depth is critical depth of unsupported cut

$H_c = 2z_c$

$$H_c = \frac{4c}{\gamma\sqrt{ka}}$$

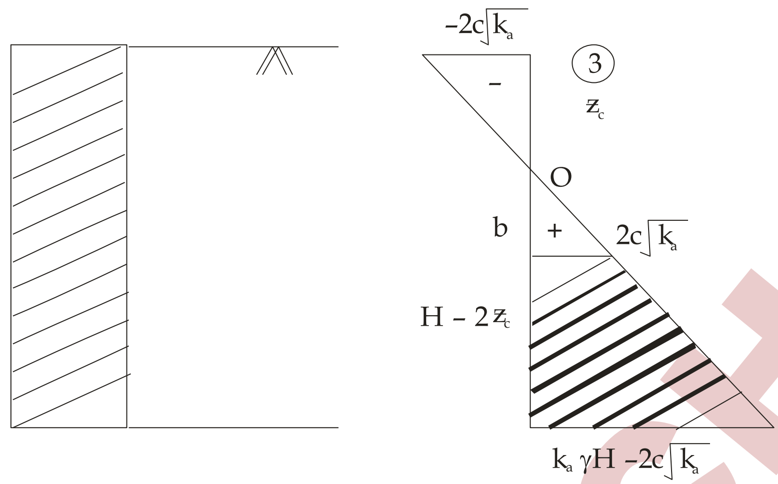
Note : 3

Because of presence of tension, tension cracks may develop in cohesive soil as a result of which it does not remain bounded to the face of the wall. Hence portion of the soil is considered to be exert so pressure on the wall

Total Active Earth Pressure On Unit Length of Wall

Case - A

When Tension Cracks are not developed

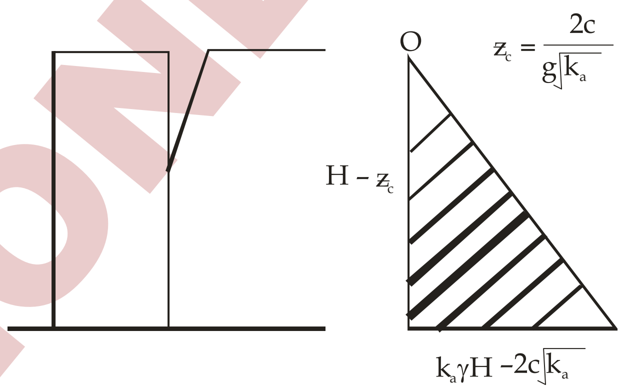


$$Pa = \frac{1}{2}\left[ka\gamma H - 2C\frac{1}{\sqrt{ka}} + 2c\sqrt{ka}\right]\left[H - 2zc\right]$$

$$\bar{z} = \left(\frac{k_a\gamma H2C\sqrt{ka} + 4c\sqrt{ka}}{ka\left(H - 2c\sqrt{ka} + 2c\sqrt{ka}\right)}\left(\frac{H - zc}{3}\right)\right)$$

Case - B

When Tension Cracks are Developed



Coulomb's Wedge Theory :-

1. Soil is homogeneous, isotropic, semi-infinite, elastic, dry and cohesionless for sand [both rankine & coulomb gave theory for cohesionless soil]
2. The face of wall in contact with backfill vertical can be inclined and is rough.
3. The failure wedge acts as a rigid body and stresses over it are uniformly distributed
4. The failure is essentially 2-D and rupture surface is planar and passes through the heel of the wall

ZONE TECH

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5. The location and direction of resultant thrust between wall and soil is known. The point of application is taken at the lower third point of the wall by assuming triangular distribution of earth pressure

6. Wedge failure is considered which is under equilibrium of a forces

   (1) Self weight of wedge ABC, acting vertically downward

   (2) Resultant reaction R, acts at downward angle of ϕ with normal to slip plane

   (3) Resultant thrust P_a bewteen wall and soil acts at downward angle δ with the normal of wall.

ZONE TECH

CHAPTER - 12 (SLOPE STABILITY)

Stability of Slopes

Slopes for embankments are provided in roadways, railways, earthern dams, river training works etc

Failure of slopes takes place due to the following forces

(1) Gravity force

(2) Seepage force

(3) Earthquake

(4) Sudden drawdown of water table

(5) Erosion due to water

(6) Excavation near the slopes

Slopes are of two types

(1) Infinite slope

(2) Finite slope

Infinite slope :

→ If the slope represents the boundary surface of semi infinite soil mass then the properties of soil at all the similar depth below the surface will be same. Then the slope is termed as infinite slope.

→ Failure of infinite slope takes place due to sliding and failure surface is parallel to ground slope.

If the slope is bounded by top and bottom surfaces, then it is termed as finite slope.

Failure of finite slope takes place due to rotation and failure surface is either circular or spiral.

Finite slope my have any of the following modes of failure

1. Slope failure

(a) Face failure

(b) Tae failure

2. Base failure

Face failure

→ failure surface passes through slope above the toe.

→ This type of failure takes place in case of steep slopes where soil mass near the toe is rigid and stronger equation compare to soil mass above the toe.

Toe failure :

→ It is the most common mode of failure of finite slope in which failure surface passes through toe.

→ This failure also occurs in case of steep slopes when the soil mass is homogeneous above and below the

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Base Failure :-

→ Failure surface passes below the toe this type of failure

If (H + D) is the total depth of failure surface and H is height of slope then depth factor is defined as

Depth factor Df = $\frac{H+D}{H}$

If $D_f = 1$, Toe failure

$D_f = 1$, Base failure

Note ;-

For slope angle B > 53°, the critical slip circle passes through the toe (toe failure). This is true for any angle of friction (ϕ)

→ The stability of finite slope is analysed in 2 stage

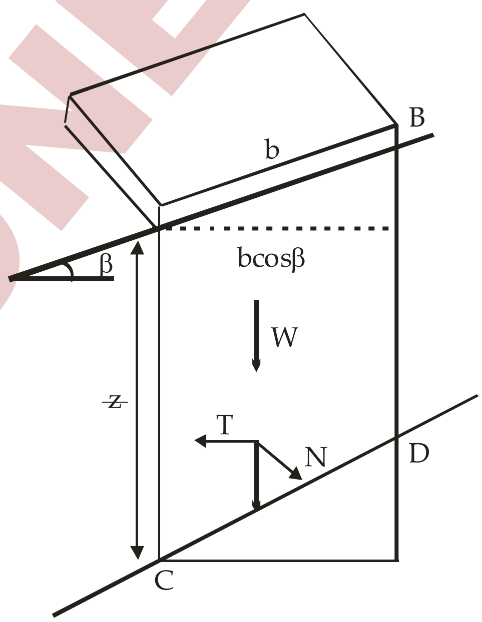
1. Immediately after construction

2. Long time after construction

→ For sand, effective stress analysis (drained) is preferred for both the stages.

→ For clays, total stress analysis (undraned) is preferred immediately after construction and effective stress analysis (drained) is prefered long term after construction.

Let AB represent infinite slope having slope angle of β with the horizontal, failure of which takes place along critical section CD, it means parallel to ground surface and is at the depth of z from the surface considered unit length of slope



Area of wedge ABCD = $(b \cos \beta)\, z$

Volue of wedge ABCD = $(b \cos \beta) \times z \times 1$

$= b\, z \cos \beta$

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Weight of wedge ABCD = γ × volume

$$= b\gamma\, b\, z \cos\beta$$

Normal component of wt = W cos β

$$N = b\gamma z \cos^2\beta$$

Tangential component after T = w sin β

$$T = b\gamma z \cos\beta\sin\beta$$

Normal stress,

$$\sigma_n = \frac{N}{Area}$$

$$= \frac{b\gamma z\cos^2\beta}{b\times 1} = \gamma z\cos^2\beta = \gamma z \cos^2\beta$$

Tangential stress $\tau = \dfrac{T}{Area}$

$$= \frac{b\gamma z\cos\beta\sin\beta}{b\times 1} = \gamma z\cos\beta\sin\beta$$

$$S_H = \gamma z\cos^2\beta$$

$$\tau = \gamma z\cos\beta\sin\beta$$

Note :-

Similarly, if water table is present, then normal pore water pressure at plane CD will be

$$\sigma_H = \gamma_w z\cos^2\beta$$

Note :-2

The tangential stress is also termed as shear stress that induces failure along critical plane CD by sliding which resisted is being by shear strength of soil

$$\text{F.O.S.} = \frac{S}{\tau} = \frac{\tau_f}{\tau} = \frac{\text{shear strength of soil}}{\text{shear stress on that plane}}$$

1. Cohesionless soil :- (C = 0)

Case - 1 ; Dry / Moist slope

$$S = \cancel{c}^{\,0} + \sigma_n \tan\phi$$

Shear strength, S = $\gamma z\cos^2\beta\tan\phi$

Shear stress, τ = $\gamma z\cos\beta\sin\beta$

$$\text{F.O.S} = \frac{s}{\tau}$$

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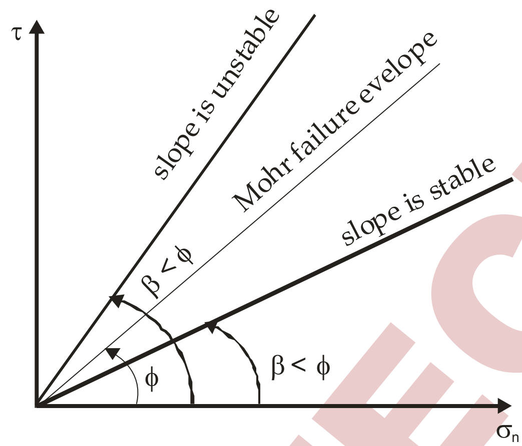
$$\frac{\gamma z \cos\beta \tan\phi}{\gamma z \cos\beta \sin\beta}$$

$$\text{F.O.S} = \frac{\tan\phi}{\tan\beta}$$



If

$\phi > \beta$, F.O.S > 1 slope is stable

$\phi = \beta$, F.O.S = 1 slope is critical

$\phi < \beta$, F.O.S < 1 slope is unstable

Case-2 :

Submerged Slope :-

If slope is submerged under water, then normal effective stress and shear stress are calculated using the submerged unit

$$\delta = \bar{\sigma}_n \tan\phi$$

Shear strength $\delta = \gamma' z \cos^2\beta \tan\phi$

Shear stress $= \tau = \gamma' z \cos\beta \sin\beta$

$$\text{F.O.S} = \frac{S}{\tau} = \frac{\tan\phi}{\tan\beta}$$

$$\text{F.O.S} = \frac{\tan\phi}{\tan\beta}$$

Case-3:

Slope subjected to steady seepage and W.T is at depth h above the critical section

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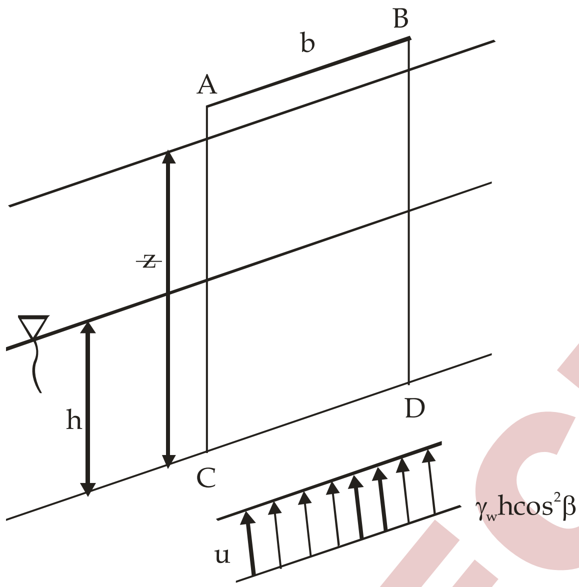


$$S = \bar{\sigma}_n \tan\phi$$

$$= (\sigma_n - u)\tan\phi$$

$$\sigma_n = \gamma_{avg}\, z \cos^2\beta$$

$$u = \gamma_w h \cos^2\beta$$

$$\bar{\sigma}_n = \gamma_{avg}\, z \cos^2\beta - \gamma_w h \cos^2\beta$$

$$\text{shear strength} = \left(\gamma_{avg} \cos^2\beta - \gamma_w\, h\cos^2\beta\right)\tan\phi$$

$$\text{shear stress} = \gamma_{avg}\, z \cos\beta \sin\beta$$

$$\text{F.O.S} = \frac{\text{shear strength}}{\text{Shear stress}}$$

$$= \frac{\left(\gamma_{avg}\, z - \gamma_w h\right)\cos^2\beta\tan\phi}{\left(\gamma_{avg}\, z\right)\cos\beta\sin\beta}$$

$$\text{F.O.S} = \left[1 - \frac{\gamma_w h}{\gamma_{avg}\, z}\right]\frac{\tan\phi}{\tan\beta}$$

Special case :

If $h = z$ water is flowing at ground slope

$$\bar{\sigma}_n = \left(\gamma_{sat} - \gamma_w\right) z \cos^2\beta$$

$$\tau = \gamma_{sat}\, z \cos\beta\sin\beta$$

$$\text{F.O.S} = \frac{1}{2}\frac{\tan\phi}{\tan\beta} \qquad \gamma' \simeq \frac{1}{2}\gamma_{sat}$$

Note : If slope is completely submerged and steady seepage takes place at ground slope surface then, FOS is reduced to half is comparison to FOS of dry/submerged slope. Hence in dry (or) submerged condition , FOS to be provided should always be greater than 2.

2. Cohesive Soil :

Case-1 : Dry/Moist Slope -

$$S = C + \sigma_n \tan\phi$$

$$= C + \gamma z \cos^2\beta \tan\phi$$

$$\tau = \gamma z \cos\beta \sin\beta$$

$$F.O.S = \frac{c + \gamma z \cos^2\beta \tan\phi}{\gamma z \cos\beta \sin\beta}$$

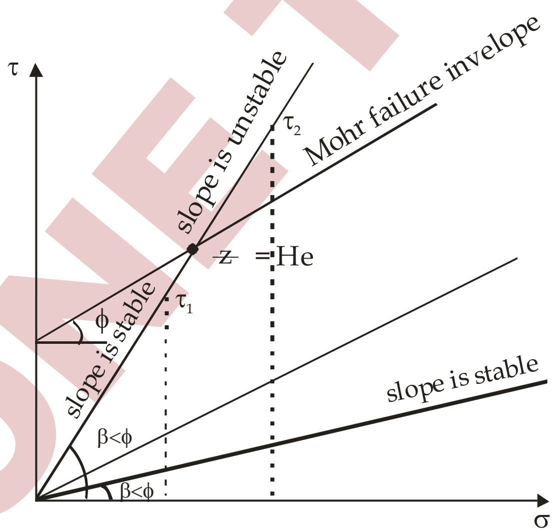
Note :-

If $B < \phi$ - slope is always stable

If $B > \phi$

(i) slope is stable upto depth $(z = H_c)$ $\tau_1 < S$

(ii) slope is unstable $(z = H_c)$ $\tau_1 > S$



To find cricital height H_c

$$F.O.S = 1$$

$$\frac{c + \gamma H_c \cos^2\beta \tan\phi}{\gamma H_c \cos\beta \sin\beta} = 1$$

$$C = \gamma H_c \cos\beta \sin\beta - \gamma H_c \cos^2\beta \tan\phi$$

$$H_C = \frac{c}{\gamma \cos^2\beta [\tan\beta - \tan\phi]}$$

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$$\frac{c}{\gamma H_c} = \cos^2\beta\left[\tan\beta - \tan\phi\right] = \text{stability number}$$

Taylors stability No $S_n = \frac{c}{\gamma H_c}$

Note ; F.O.S of cohesive soil can be represented in terms of height

F.O.S of cohesive soil in terms of height $(F.O.S.)_H = \frac{H_C}{H}$

FOS in terms of cohesion :

If H < H_c, soil will not utilise its all shear strength it means, mobilised cohesion is cohesion used in stability of slopes. Hence FOS can be represented as

$$(FOS)_C = \frac{C}{C_m}$$

where

C_m – mobilised (utilized) cohesion.

C – Total cohesion (a heavy strength of soil)

w.r.t

$$S_n = \frac{C}{\gamma H_c} (or) \frac{C_m}{\gamma H}$$

$$S_n = \frac{c}{\gamma H_c} = \frac{c_m (FOS)_c}{gH(FOS)_H}$$

→ $(FOS)_C = (FOS)_H$

3. FOS wrt friction :

If is the ratio of available frictional strength to the mobilised frictional strength.

$$(FOS)_\phi = \frac{\sigma_n \tan\phi}{\sigma_n \tan\phi_m}$$

$$(FOS) = \frac{\tan\phi}{\tan\phi_m}$$

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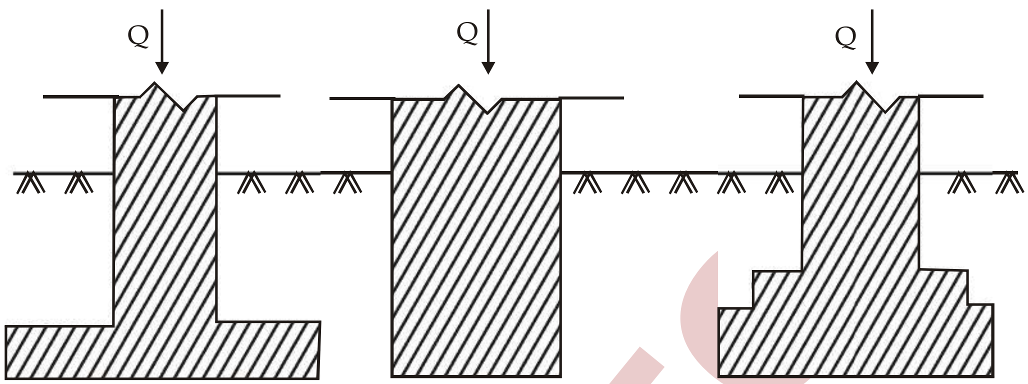
Chapter - 13 (Shallow Foundation)



B × B

B × L OR B × D

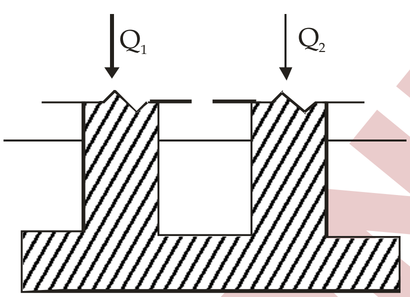
Isolated

B

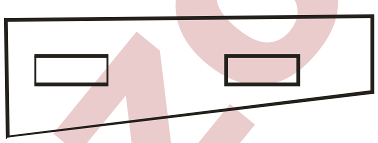
L >>B

Strip

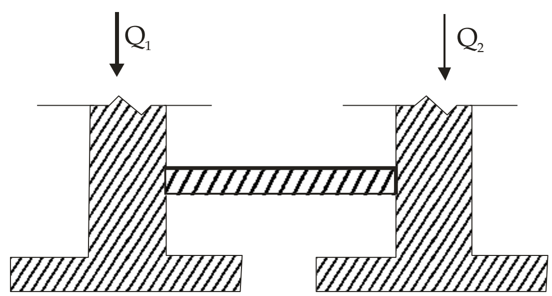
Spread footing



combined



$Q_1 > Q_2$



Strap

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3. Guide lines for selection of footing/ foundation

S. No.	Type of soil and loading cond.	Suitable Foundation
1.	Structural load is less and soil is medium to dense	Shallow footing (Rect, surface, circ.)
2.	Structural load is heavy and soil is medium to dense	Raft (OR) deep foundation
3.	Structural load very heavy and soil is medium to loose	Raft + pile
4.	Structural load is heavy and foundation is to be placed in running water	Well foundation
5.	Swelling pressure is high and differential swelling value is more than 35% than shallow footing is not adequate	Either raft (OR) deep footing
6.	If column spacing is less and footing area is more than 40% of plinth	Either raft OR combined footing use
7.	Soil is loose saturated sand and prone to liquefaction	Compaction pile
8.	For individual houses (OR) light building (2 to 4) story	Strip, isolated
9.	Black cotton soil (expansive soil) high swelling and shrinkage	either footing or balancing raft or under reamed pile

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Note :-

→ Floating foundation is a type of raft foundation in which weight of soil excavated is equal to the weight of structure + wt. of foundation + applied load. It means net increment of pressure on the soil will be negligible hence settlement will be negligible hence soil become safe in settlement criteria.

4. Shallow Foundation

→ Foundation should be safe in shear criteria as well as setlement criteria

→ The allowable load on foundation should be minimum of below 2 criteria.

safe load against stear criteria = safe bearing capacity × A_f

safe load against settelement criteria = safe bearing pr. XA_f

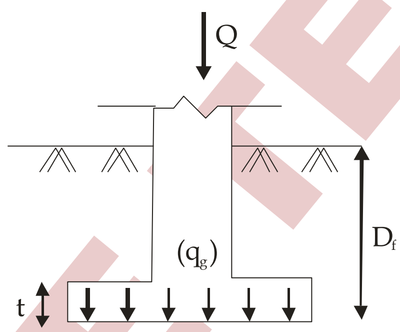
→ In sandy soils generally design of foundation is governed by settlement criteria because bearing capacity of sand is quite high. However design of foundation in clay soil can be governed either by shear or settlement criteria.

Type of soil	Bearing capacity
Alluvial soil (Indo-gangatic plane)	60 - 50 kN/m^2
Black cotton soil (deccan plateau)	30 - 60 kN/m^2
Red soil	120 -150 kN/m^2
Desert soil	100 -150 kN/m^2

Important Definations

(1) Gross pressrue (q_g)

→ It is the total pressure at the base of the footing due to the self wt of soil/footing and applied load on the footing and sub and super structure weight



→ The q_g may be greater equal or smaller than bearing capacity.

→ But for design purpose q_g should be less than safe bearing capacity

2. Net pressure (q_n)

→ It is gross pressure at the base of footing in excess to intial effective overburden pressure

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

$$q_n = q_g - \gamma D_f \rightarrow \text{GWT is not present}$$

$$q_n = q_g - \gamma' D_f \rightarrow \text{GWT is present}$$

3. Ultimate bearing capacity (q_u)

→ It is the max gross pressure which can be applied at the base of footing without risk of shear failure.

→ It is the min gross pressure at which soil just fail in shear

4. Net ultimate bearing capacity (q_{nu})

→ It is the max net presure at the base of footing which can be applied without shear failure

→ It is the min net pressure at which footing fails in shear

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

$$q_{nu} = q_u - \gamma D_f \rightarrow \text{GWT is not present}$$

$$q_{nu} = q_u - \gamma' D_f \rightarrow \text{GWT is present}$$

5. Net safe bearing capacity (q_{ns})

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

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

F.O.S – 2.5 to 3

6. Safe Bearing capacity (q_s / q_{safe})

→ It is the gross pressrue which can be applied safely at the base of footing without risk of shear failure.

$$q_s = q_{ns} + \bar{\sigma}$$

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

Safe Load against shear criteria

→ Safe load = P_s = q_s × area of footing (A_f)

Settlement Criteria

→ A footing is designed such that max settlement is restricted below the permissible value permitted by local agencies (BIS)

Factors affecting bearing capacity

→ Type of foundation and its dimension

→ Type of ground surface (horizontal or inclined)

→ Position of GWT wrt width and depth of foundation.

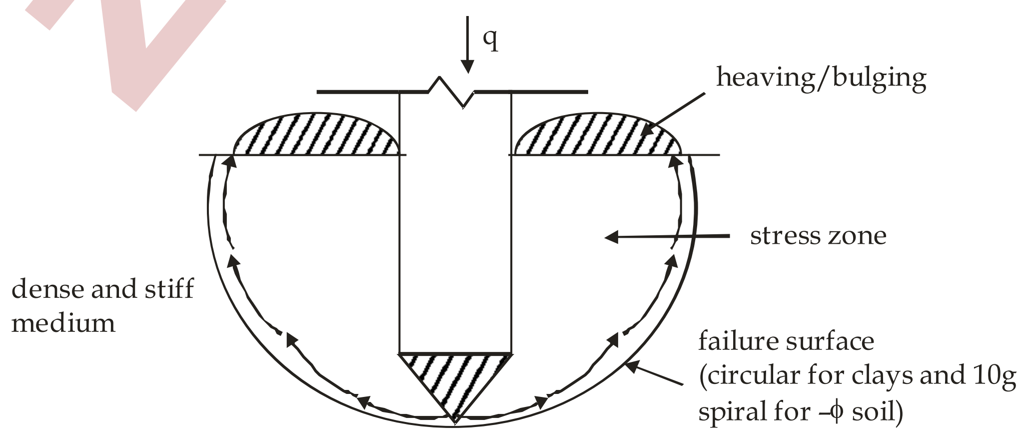
→ intial stress on the soil (whether soil is OC OR NC)

→ Type of loading (concentric or eccentric)

→ Type of shear failure

type of shear failure

(A) Genral shear failure



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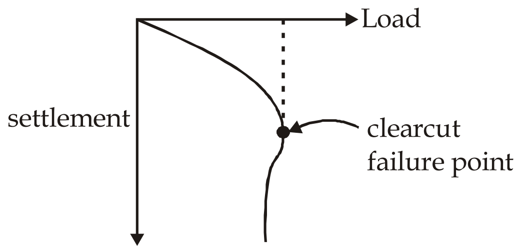


→ This failure occurs in the shallow foundation placed on medium to dense sand and stiff clays (O.C)

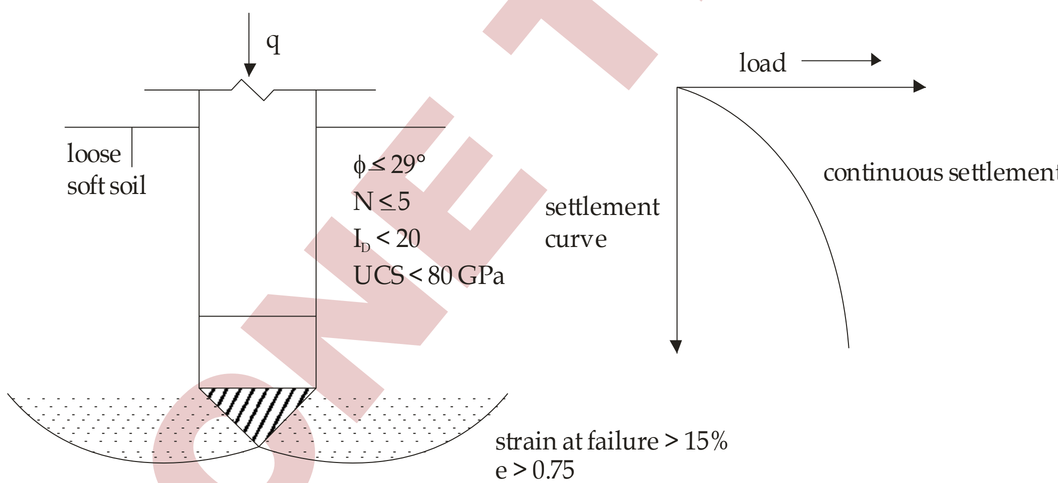
→ At the time of failure soil reaches into plastic state and stress zone extends upto G.L

→ At the time of failure foundation may get tilted and heaving of building above G.L will occur at the sides

→ In load (VS) settlement curve clear cut failure point is obtained

→ Large settlement below footing is not recorded it is very small and negligible.

(B) Local shear failure



→ It occurs in loose sand and soft clays under shallow foundation

→ Before failure large settlement is recorded

→ Stress zone doesnot extend upto the G.L

→ There may be little (OR) no heaving at the sides

→ If load (VS) settlement curve is plotted than progressive failure will occur.

(C) Punching Shear Failure

→ It occurs in deep and pile footings which are placed on loose sand (OR) soft clays (it is not the common case of shallow foundation)

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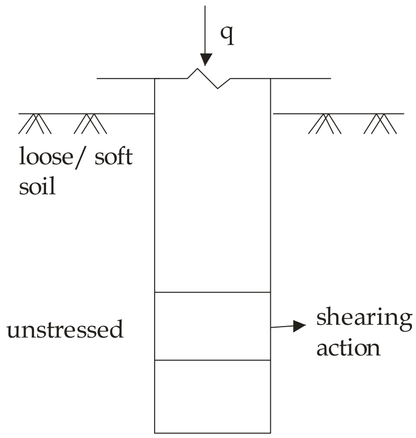
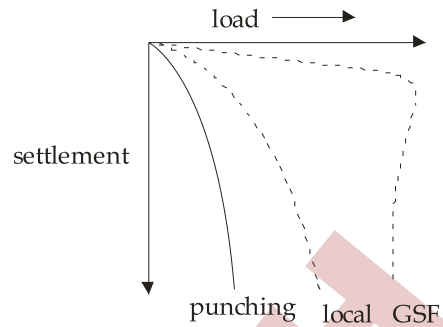


failure occure because area is small so str. is low



→ In this failure soil below the foundation gets cut off from adjacent soil by shearing and excessive large settlement is recorded in small time.

→ The adjacent soil mass remains unstressed so there will be no tilting and no heaving at the sides

Methods to determine bearing capacity of shallow foundation

7. Terzaghi's Method

→ it is an improvement over prandtl's theory

→ Prandte considered base of footing to be smooth whereas terzaghi considered base of footing to be rough.

Assumptions

(i) Foundation is shallow $(D_f \leq B_f)$

(ii) Base of footing is rough

(iii) Failure is GSF and footing is continious / strip (L>>B) because it makes the analysis 2D

(iv) At the time of failure soil reaches into plastic equilibrium

(v) The stress due of soil extends upto foundation level only but not upto the GL

(vi) Shear resistance of soil above the foundation level is ignored it means only base resistance is considered and side resistance is ignored and it is the main reason due to which Terzaghi theory is not applicable for deep foundation.

→ In Terzaghi theory soil above foundation level upto GL is removed and replaced by an equivalent surcharge

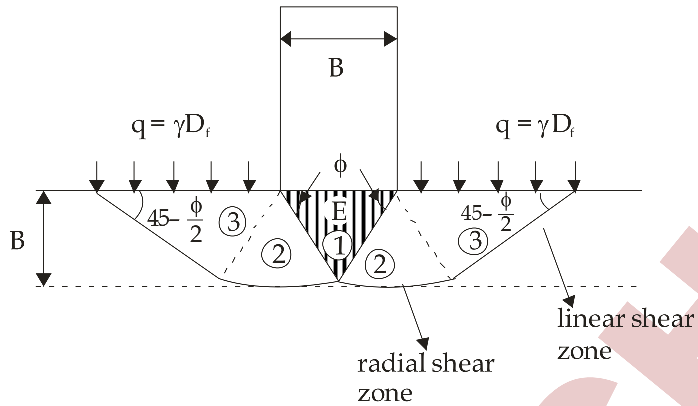
$q = \gamma D_f$

(vii) Load is vertical and concentric

(viii) Ground surface is horizontal and foundation level is also horizontal

(ix) Water level is beyond the zone of influence of stressing it means effect of water table is not considered

GSF



Zone (I) Central shear zone

→ It gets compacted and becomes the part of footing it remains in elastic equilibrium.

Zone (II) Radial shear zone

→ It remains in plastic equilibrium

→ Failure surface is circular for clay soil and log spiral in sand and silt.

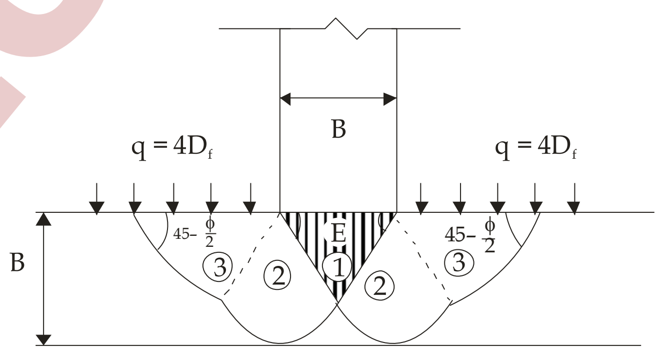
Zone (III) Rankine/ passive zone / linear shear zone

→ It is make an angle $45 - \frac{\phi}{2}$ with horizontal

Note :-

The stress zone of soil is extended upto max depth B below the foundation level where B is width of footing

Ultimate bearing capacity of strip footing (Terzaghi's equation)



$$q_u = CN_c + qN_q + \frac{1}{2}\gamma N_\gamma \, . \, B \text{ main equation}$$

Special case → For pure cohesive clay ($\phi = 0$)

$N_{\phi} = 1 \qquad N_4 = 0$

$N_q = 1 \qquad N_c = 5.7$

$q_u = 5.7c + q$

$q_u = 5.7c + \gamma D_f$

valid only for clay.

→ γ in 3rd term of main equation is for the soil below the foundation level

→ C is for the soil below the foundation level

→ γ in term γD_f of for the soil below GL to foundation level. (surcharge)

→ Generally footing place in same soil stratum so γ in 2nd and 3rd term is same but if soil is different than use rule mention above this point.

→ N_c and N_q and N_4 depends on soil below footing level.

Modified form of q_u

(a) For strip footing - $CN_c + qN_q + 0.5\,B\gamma N_\gamma$

(b) For square footing - $1.3\,CN_c + qN_q + 0.4\,B\gamma N_\gamma$

(c) For circular footing - $1.3\,CN_c + qN_q + 0.3\,B\gamma N_\gamma$

(d) For raft - $\left(1+\dfrac{0.3B}{L}\right)CN_c + qN_q + \left(1-\dfrac{0.2B}{L}\right)\dfrac{1}{2}B\gamma N_\gamma$

Modification for shear failure

→ If LSF occurs instead of GSF than use ϕ_m and C_m instead of C and ϕ

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

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

Note :- As perxis / 9.4 - 1986

MS
Max. settlement

DS
Diff settlement

AS
Angular settlement

S.No	Type of footing	MS (mm)	DS (mm)	AS (mm)
	(A) Isolated footing lasting (i) steel structure (ii) RCC structure	 50 mm 75 mm	 0.0033L 0.0015 L	 1/300 1/666
	(B) Raft foundation plastic clay (i) steel structure (ii) RCC structure	 100 mm 100 mm	 0.003L 0.002L	 1/300 1/500

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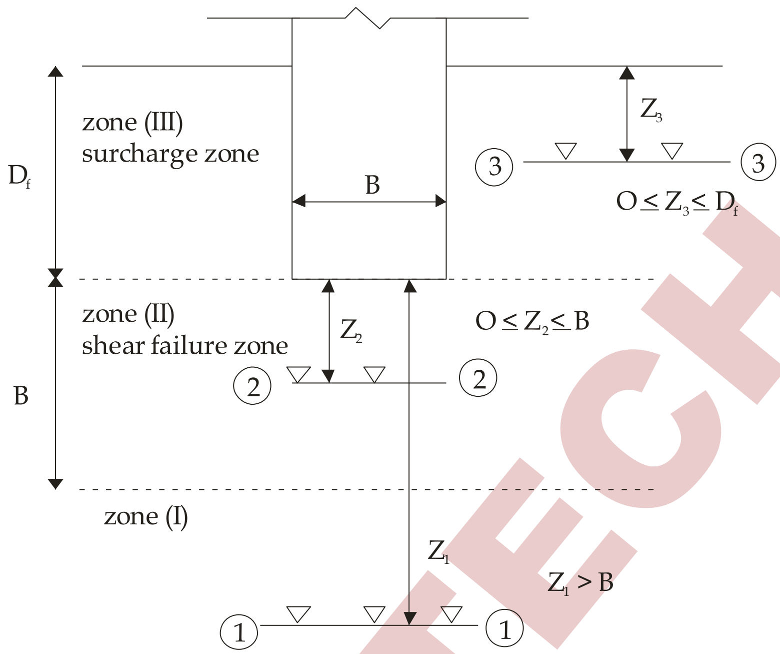
Modification For Water Table



Case(I) ;- When the water table is in zone (I) and at depth Z_1 below foundation level such that $Z_1 > B$

→ In this case zone (I) is below the stress zone therefore no effect of water table is recorded on surcharge zone and shear failure zone.

Case (II) :- When water table is in zone (II) at a depth Z_2 below foundation level such that $(0 \le Z_2 \le B)$

→ In this case I^{st} and 3^{rd} term of bearing capacity equation will be affected where as 2^{nd} term will remain uneffected.

→ Generally effect on C and ϕ is ignored in absence of data but effect on unit weight is accounted.

Use of effective parameters	Use of water table correction factors
$q_u = C'N_c + q.N_q + \frac{1}{2}B(\gamma_{eff})N_\gamma$ Here $(\gamma_{eff}) = \frac{\gamma_{buck}.Z_2 + \gamma'(B - Z_2)}{B}$	$q_u = C'N_c + q.N_q + \frac{1}{2}B(R_\gamma.\gamma_{avg})N_\gamma$ $R_\gamma = 0.5\left[1 + \frac{Z_2}{B}\right]$ $\gamma_{avg} = \left[\frac{\gamma_{bulk}.Z_2 + \gamma_{sat}(B - Z_2)}{B}\right]$

Case (III) When water table is in zone (III) at depth Z_3 below GL such that $(0 \le Z_3 \le D_f)$

→ All there terms of bearing capacity equation will be effected but effect on 1st terms is negligible.

Use effective parameters	Use water table correction factor
$q_u = C'N_c + y_{eff}D_fN_q + \frac{1}{2}B\gamma'N_\gamma$ $\gamma_{eff} = \frac{\gamma_{bulk}.Z_3 + \gamma'(D_f \quad Z_3)}{D_f}$	$q_u = C'N_C + R_q\gamma_{avg}D_fN_q + \frac{1}{2}B\left(\frac{1}{2}\gamma_{sat}\right)N_\gamma$ $\gamma_{eff} = \frac{\gamma_{bulk}.Z_3 + \gamma'_{sat}(D_f \quad Z_3)}{D_f}$

Note :- For cohesion less soil (c = 0)

$$q_u = CN_C + \gamma D_fN_q + \frac{1}{2}B\gamma N_\gamma$$

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

if GWT rises to G.L

$$q_u = \gamma'D_fN_q + \frac{1}{2}B\gamma'N_\gamma \qquad \rightarrow \gamma' = \frac{1}{2}\gamma_{sat}$$

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

→ This equation show that in sandy soil bearing capacity increase with increase in width of footing

→ And if GWT ↑ at G.L level than q_u in sandy soil reduces upto 50%

→ For pure cohesive soil/clay ($\phi = 0$)

$$N_C = 5.7 \; N_q = 1 \; N_y = 0$$

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

if GWT is rises to GL

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

$$q_{nu} = q_u - \bar{\sigma} \; q_u - \gamma'D_f$$

→ Inclay q_u is independent of width of footing

→ q_{nu} of clay is uneffected to rise of water table

8. Skempton theory

→ This theory is applicable only for pure cohesive soil or clay ($\phi = 0$)

→ Base resistance and side resistance both considered in this theory hence this theory is applicable for shallow as well as deep foundation.

→ This theory gives q_{nu} value directly

$$q_{nu} = CN_C$$

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where $N_C \rightarrow$ Skempton bearing capacity factor depends upon D_f/B ratio

Type of footing	Case (I) $D_f/B = 0$	Case (II) $D_f/B \geq 2.5$
For strip footing	$N_C = 5$	$N_C = 7.5$
For square/Circular/Rect and raft	$N_C = 6$	$N_C = 9$

Type of footing	N_C if $0 < D_f < 2.5$
Strip footing	$5\left(1+0.2\frac{D_f}{B}\right)$
Square/Circular	$6\left(1+0.2\frac{D_f}{B}\right)$
Rect./Raft	$5\left(1+0.2\frac{B}{L}\right)\left(1+0.2\frac{D_f}{B}\right)$
Sp. note if than for rect and raft	$7.5\left(1+0.2\frac{B}{L}\right)$

9. Meyerhof Theory

→ This theory is most generalized theory in which shape factor, depth factor and inclination factor are used to account for shape of footing (square/raft/circular) variation of depth (shallow or deep) and for inclination of load (inclined /vertical)

Note :-

In this theory the stress zone considered is extended upto GL so this theory suitable for both shallow and deep where as Terzaghi considered stress zone upto foundation level only hence side resistance was not considered there for suitable shallow

$$q_u = CN_C S_C d_f i_C + qN_q S_q d_q i_q + \frac{1}{2} B\gamma N_\gamma S_\gamma d_\gamma i_\gamma$$

$S_C S_q S_\gamma \rightarrow$ shape factor

$d_C d_q d_\gamma \rightarrow$ depth factor

$i_C i_q i_\gamma \rightarrow$ inclination factor

10. IS Code Method

(IS 6103 : 1981)

$$q_u = (C.N_C.S_C.D_C.\gamma_C + qN_q S_q d_q i_q + \frac{1}{2} B\gamma N_\gamma S_\gamma d_\gamma i_\gamma \times R)$$

$R \rightarrow \frac{1}{2}\left(1+\frac{Z_2}{B}\right)$ water table correction factor

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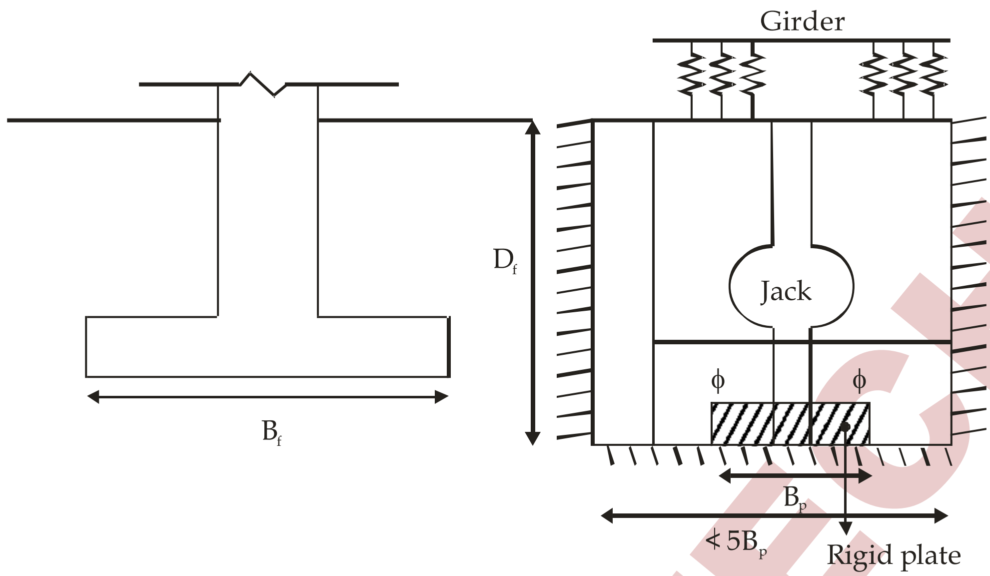
11. Plate Load Test (Field Method)



→ This method is design to determine modulus of subgrade reaction which is used in design of rigid pavement

→ This test can also be used to find bearing capacity based on shear criteria and allowable bearing pressure based on settlement criteria.

→ A pit of size not less than 5 times the size of rigid plate is excavated having depth equal to depth of foundation

→ If water table is present at or above the test level than it must be lowered below the test level by pumping before conducting the test.

→ The rigid plate may have size 30cm, 45cm, 60cm, 75cm or 90cm which may be circular or square

→ Rigid plate is placed at the centre of pit and intially a load of $7kN/m^2$ is first applied and removed there after 3 dial gauge are attacted to the rigid plate to measure the avg. settlement

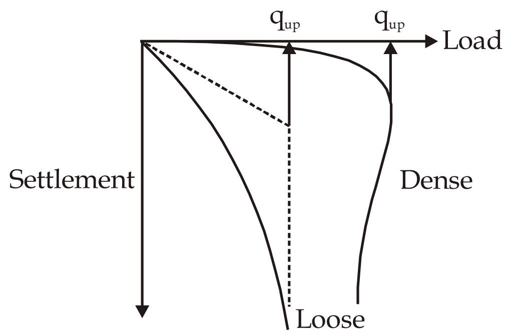
→ The load on plate is applied through jacking mechanism which may be recorded and load settlement curve is plotted by taking pressure (Q/Ap) on x-axis at settlement on y-axis

Note :- The test is conducted untill failure on at least untill 25mm settlement has occure.

→ The shear failure occurs when plates start settling suddenly at faster rate in case of dense sand and stiff soil

→ Where as in loosse soils failure is progressive

→ Load settlement curve is plotted arithmetic scale OR log – log scale



Ultimate bearing capacity of plate as per shear criteria

(1) For clay

$q_{uf} \rightarrow q_{up}$

in clays UBC is does not depends upon

width of footing

(2) For sand

$$\frac{q_{uf}}{q_{up}} = \frac{B_f}{B_p}$$

$$q_{uf} = q_{up}\frac{B_f}{B_p}$$

for $(q_{uf})_{safe}$ = use FOS 2.5 to 3

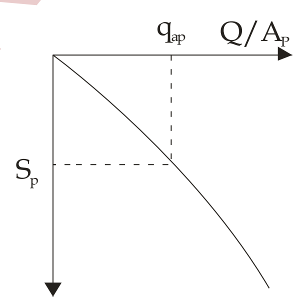
→ Safe settlement pr/allowable bearing pr as per settlement criteria

For dense sand

$$\frac{S_F}{S_P} = \left[\frac{B_F}{B_P} \times \frac{(B_P + 0.3)}{B_F + 0.3}\right]^2$$

For clay

$$\frac{S_F}{S_P} = \frac{B_F}{B_P}$$



→ Using above relation find permissible settlement of plate and using load settlement curve find allowable/bearing pressure of safe settlement pr. (q_{ap}) for plate.

Allowable bearing pr. for footing (q_{af})

(1) For clay $\rightarrow q_{af} = q_{ap}$

(2) For sand $\rightarrow q_{af} = q_{ap} \times \frac{B_F}{B_P}$

12. SPT (IS 2131)

→ The test can be conducted to determine

(a) Relative density (OR) density index

(b) Angle of internal friction (ϕ)

(c) unconfined comp. str. of clay and consistency of clay

(d) ultimate pile load carrying capacity

(e) Bearing capacity on the basis of shear criteria

(f) allowable bearing pressure on the basis of settlement criteria.

Note :-

This test is suitable for medium to dense sand because in clays due to dynamic action remoulding may occur and pore pr may set up where as in loose saturated sands liquefaction may occur.

Procedure

→ Sampler is placed over the soil in a bore hole of 55mm to 150mm dia split spoon sampler. It is driven by dynamic mechanism of hammer. The wt of hammer is 63.5kg and height of fall of hammer is 75 cm

Note :-

The spt no is define as no of blows of hammer red. for 300 mm penetration of sampler

→ Usually this test is perform in 3 stages 150mm penetration each and spt no is taken as no of blows required for last 300 mm penetration it means no of blow required for first 150 mm penetration is ignored.

→ This test conducted at every 2m to 5m interval or at the change of start.

(1) Overburden pressure Correction

→ The SPT no value for the foundations should be corresponding to the avg. SPT no value

→ Due to lesser overburden at shallow depth the SPT no value at shallow depth gets under estimated and that at greater depth gets over estimated hence correction is required for over burden.

$$N_1 = N_0\left[\frac{350}{\bar{\sigma}_0 + 70}\right]$$

N_0 = observed SPT no

N_1 = corrected SPT no

if $\bar{\sigma}_0 > 280$ GPa than this correction is not required

(2) Water table correction / dilatancy/ fine correction

→ If water table is present at OR above the test level than water table correction is required because due to sudden impact load excess pore pr. develops which increases the penetration resistance

Note :-

If water table is below the test level than this test is not required

Let N_2 = Corrected SPT no for water table

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

→ If $N_1 < 15$ then this correction not applied

→ If test is conducted at diff. levels say A,B,C,D than final SPT no is taken as the avg. of corrected values

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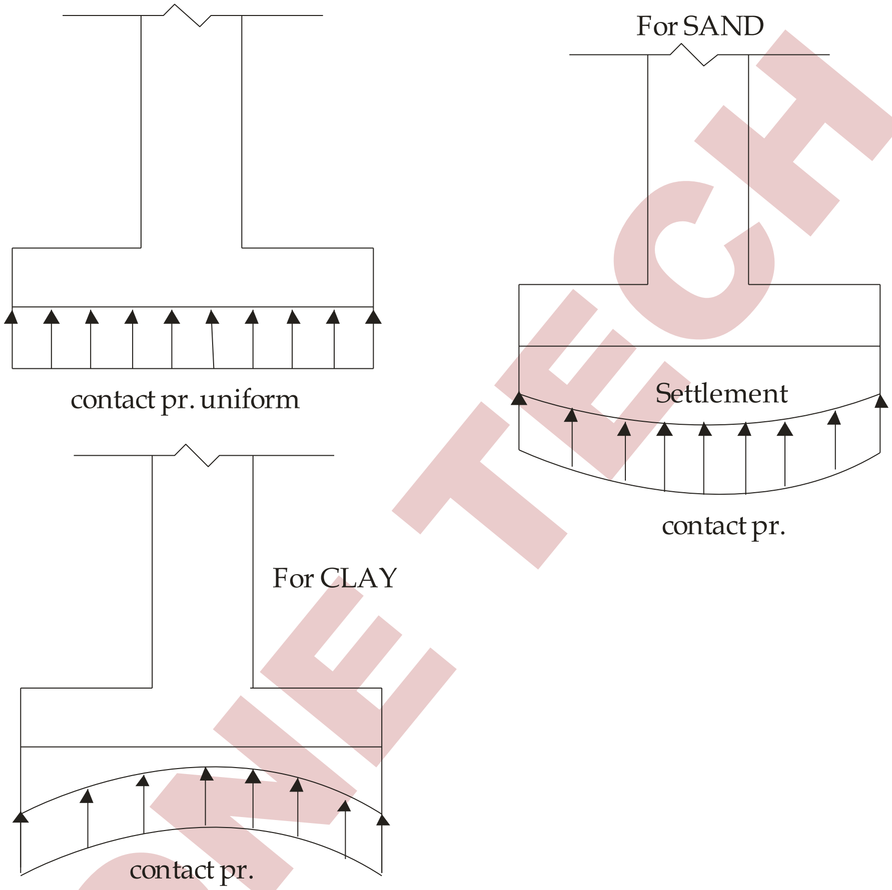
$$N_{final} = \frac{N_{2A} + N_{2B} + N_{2C} + N_{2D}}{4}$$

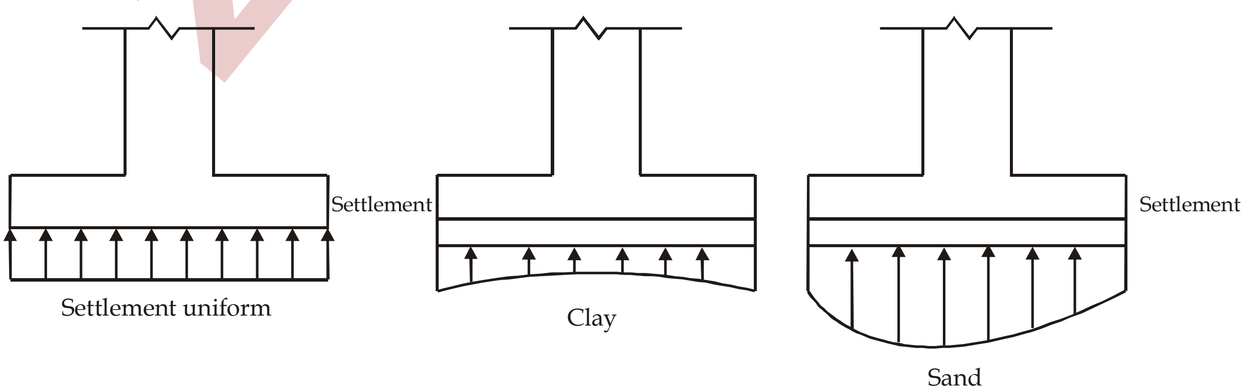
13. Contact Pressure and Elastic Settlement

(A) Flexible Footing



→ The flexible footing contact pr is uniform and settlement depends upon type of footing

(B) Rigid Footing



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CHAPTER - 14 (PILE FOUNDATION)

→ Pile foundation is a type of deep foundation.

→ Acc to Terzaghi if $D_f > B_f$ than foundation is called deep foundation.

→ But if $D_f > 15\ B_f$ than pile foundation is used as a deep foundation.

→ Pile foundations are used in situation when

(a) The starta of good bearing capacity is not available near the ground

(b) Structral load is too heavy and top soil is loose followed by dense soil.

2. Type of Piles

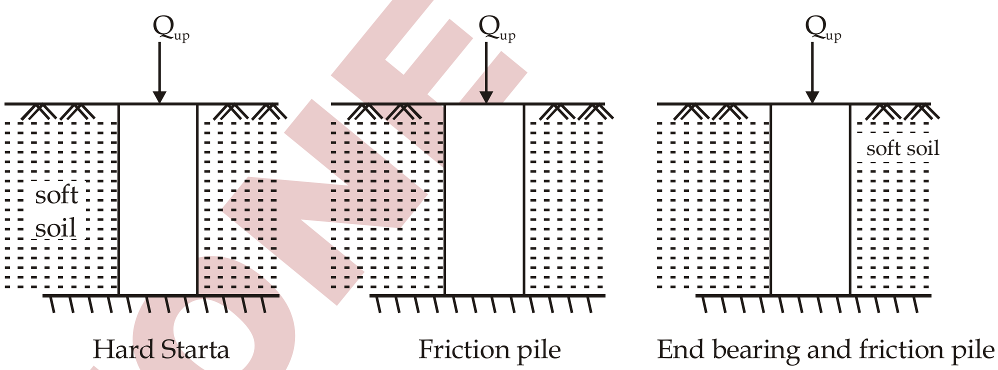
Acc. To Mechanism of Load Transfer

(a) **End Bearing Piles :-** These are restover stiff/hard starta and load supporting power is essentially only at base that is called end bearing action (OR) point resistance. The length of there piles are depends upon position of stiff starta.

(b) **Friction piles :-** These piles are driven through soft strata and than load supporting power is due to skin friction only along the surface area of pile.

There are called hanging pile also

(c) **Bearing and Friction piles :-** Conbination of above 2



End bearing pile

$Q_{up} = Q_{eb}$

$Q_{up} = Q_{SF}$

Frction pile

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

Type of pile based on Function

(a) **Anchor pile :-** These are used for providing anchorage against horizontal pull from sheet piles (OR) Retaining wall.

(b) **Sheet piles :-** Used for retaining backfill (OR) Soil in open excavation

→ Provided below hydraulic structures to reduce uplift force and prevent piping.

(c) **Tension (OR) uplift piles :-** Used to anchor the structure subjected to uplift pressure caused by swelling of soil such as in black cotton soil.

(d) **Compaction pile :-** There are driven in loose sand in order to increase density and bearing capacity.

→ Such piles are provided in these area which are prone to liquification such as near the sea (OR) rivers.

(e) Fender pile :- Used to anchore the structure subjected to tidal waves caused by ships or seismic activityes

(f) Batter piles :- There piles are driven in inclined direction to prevent horizontal thruet/inclined force in case of large lateral loads.

Acc to Method of installation

(a) Driven/Dis placement pile :- There piles are driven through hammering action such piles should be pre cast pile made of wood or metal.

→ Indriven piles end bearing action and friction action both are effeciently develop.

(b) Bored piles :- Bored piles may be precast and cast in situ (concrete pile)

Note :-

Piles with displace the soil and cause disturbance while driving are called displacement piles where as if it does not displace soil are called non displacement pile

3. Ultimate Load Carrying Capacity of pile

→ It is that max load which can be applied on the pile without shear failure.

→ it is sum of end bearing resistance (Q_{eb}) and skin resistance (Q_{SF})

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

Note :-

If piles are end bearing pile than Q_{SF} is ignored in calculation of Q_{up} and if piles are friction pile than Q_{eb} is ignored in calculation of Q_{up}

→ Safe load carrying capacity / Allowable load carrying capacity (Q_{safe}/Q_{up})

→ It is the max safe load which can be a allowed on the piles without risk of shear failure.

$$Q_{safe} = \frac{Q_{up}}{FOS} = \frac{Q_{eb} + Q_{SF}}{FOS}$$

F.O.S is taken 2.5 to 3

4. Methods to Determine Pile Load Capacity

(A) Static/ Analytical Method - Suitable for clays

(B) Dynamic method - suitable for dense sand

(C) Field method

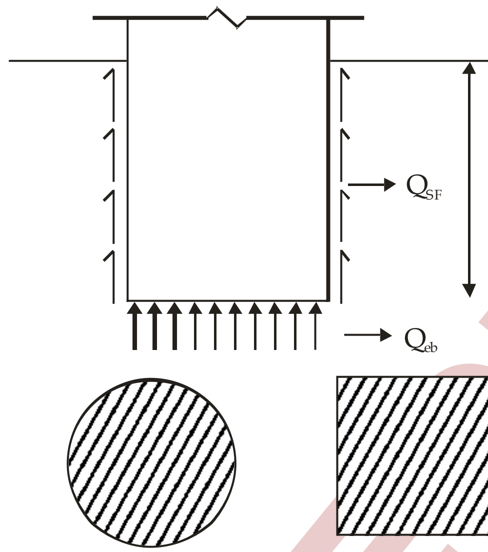
(i) Pile and test - Best method and more suitable for dense

(ii) Cyclic pile load test - this test gives end bearing resistance and skin resistance seperately

(D) SPT - For sands

(E) CPT

(A) Static/ Analytical Method :-



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

$Q_{ef} = A_b \times q_b$

$Q_{SF} = A_S + q_S$

$Q_{up} = A_b\, q_b + A_S q_S$

A_S = Perimeter × Length soil contact

Case (I) - Piles driven in clay As per mayurhoff

$q_b = CN_C = q.c. \rightarrow$ (where $D_F > 2.5\ B_F$ than N_C is 9)

$q_S = \alpha\bar{C}$

$\bar{C} \rightarrow$ avg. cohesion

$\alpha \rightarrow$ adhesion factor

$\alpha = 0.6$ to $0.9 \rightarrow$ soft clay

$\alpha = 0.4$ to $0.6 \rightarrow$ medium clay

$\alpha = 0.2$ to $0.4 \rightarrow$ stiff clay

$Q_{up} = q_b A_b + q_S A_S$

$Q_{up} = q_C A_b + \alpha\bar{C}A_S$

Note :-

For very long piles $(L \geq 25m)$ the above method for estimating the skin friction is very conservative. For such piles unit skin friction may also depends upon the effective overburden pressure The avg unit friction can be exp. as

$q_S = \lambda(\bar{\sigma}_V + 2\bar{C})$

$q_{up} = q_C A_b + \lambda(\bar{\sigma}_v + 2\bar{C})A_S$

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Where

λ = Friction capacity Factor

$\bar{\sigma}_V$ = Mean eff. stress for embedded length of pile

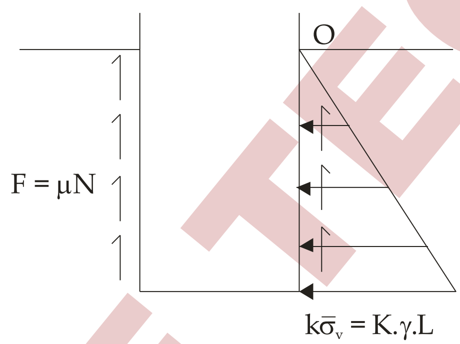
Case (II) For Sand :- As per mayerhof

$$q_b = \gamma L.Nq$$

$$q_S = \mu N$$

$$q_S = \tan\delta\left(\frac{0+k\sigma_v}{2}\right)$$

$$q_S = \left(\frac{1}{2}K\gamma L\right)\tan\delta$$



$K \rightarrow$ Earth pr. coeff

$L \rightarrow$ Length of pile

$\delta \rightarrow$ Friction angle between pile and soil

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

where

$$\delta = \frac{2}{3}\phi \rightarrow \text{timber pile}$$

$$\delta = \frac{3}{4}\phi \rightarrow \text{concrete pile}$$

$$q_{up} = \left(\gamma L N_q\right)A_b + \left(\frac{1}{2}K\gamma L\tan\delta\right)A_S$$

$\delta \rightarrow 20$ For steel pile

(B) Dynamic Method :-

→ This analysis is based on the assumption that the dynamic resistance to drive the pile is equal to the ultimate load carrying capacity of pile using static method.

→ In this method kinetic energy/ potential energy of hammer is equal to the work done by pile when it penetrates through the soil.

Eng. News Record Formula (ENR)

$$Q_{up} = \frac{WH}{S+C} kN \rightarrow \text{drop formmer}$$

$$Q_{up} = \frac{(W+\alpha p)H}{S+C} \rightarrow \text{double acting frommer}$$

$$Q_{safe} = \frac{Q_{up}}{FOS} = \frac{Q_{up}}{6}$$

F.O.S = 6

Where :-

W = wt. of hammer in kN

H = ht of fall hammer in cm

S = Set penetration of pile per blow of hammer in cm

avg of las 5 blow - For drop hammer

avg of last 25 blow - For steam hammer

C = constant which accounts for elastic compression of pile and soil in cm

C = 2.5 cm → drop hammer

a = area of piston / hammer on which steam pressure p is applied.

For drop hammer

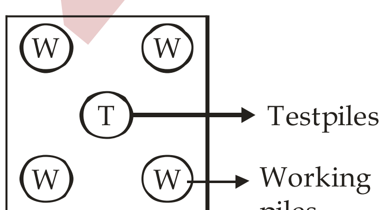
$$Q_{safe} = \frac{Q_{up}}{6} = \frac{\left(\frac{W+H}{S+C}\right)}{6} = \frac{1}{6}\left(\frac{WH}{S+2.5}\right)$$

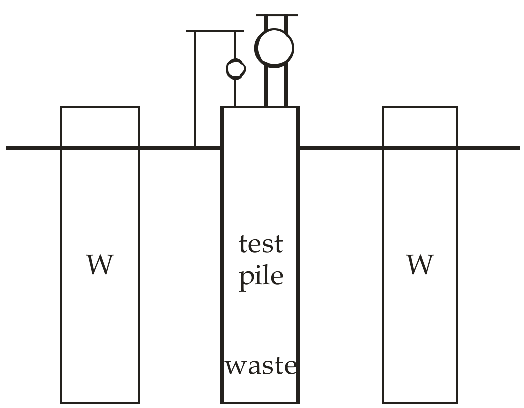
For single hammer

$$Q_{safe} = \frac{Q_{up}}{6} = \frac{1}{6}\left(\frac{WH}{S+0.25}\right)$$

(C) Field Methods

Pile Load Test





Destructive Test

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→ The load on test pile is transferred through jacking mechanism and settlement of pile is recorded

→ Load settlement curve is plotted in which shear failure is represented by sudden settlement at faster rate or by progressive settlement.

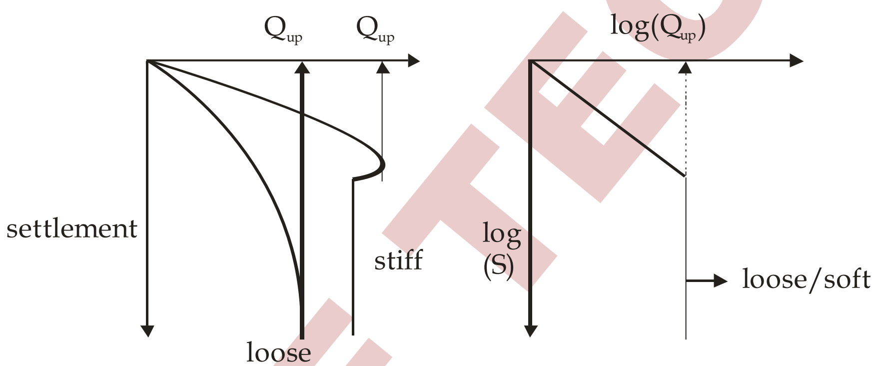
→ In progressive settlement curve is plotted in airthmatic scale OR log – log scale

→ The loaded pile in this test becomes waste hence it is a destructive test.

→ This method is most accurate and recomended by IS code. This method can also be used to find the allowable load on the pile using settlement criteria

IS Code Guideline :- The allow able load an pile may be taken as 50% of ultimate load at which total settlement of pile is 10% of its dia.

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



(D) SPT

→ Let N = SPT No. at the base of pile

→ $\bar{N}$ = avg SPT no along the side of pile

→ as per mayerhoff ultimate pile load capacity is given as

For driven / displacement pile

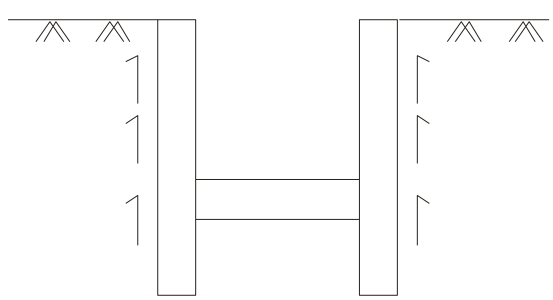
$$Q_{up} = 400\,N\,A_b + 2\bar{N}\,A_S \rightarrow \text{kN}$$

where A_b and A_s in m^2

For nondisplacement piles (H – pile)

$$Q_{up} = Q_{eb} + \frac{Q_{SF}}{2}$$

(Q_{up}) non displacement pile = 50% (Q_{up}) displacement pile



$$Q_{up} = 400NA_b + \bar{N}A_S$$

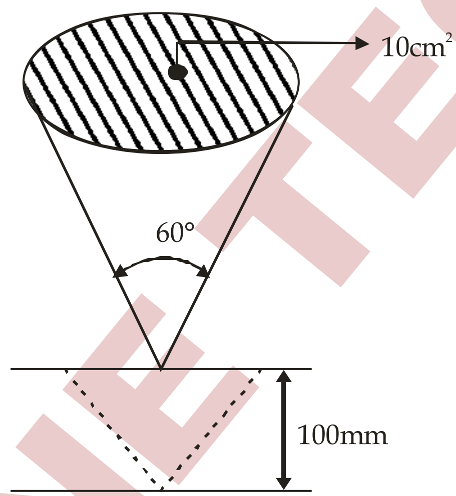
For bored piles

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

$$(Q_{up}) = \frac{1}{3}\left(400NA_b + 2\bar{N}A_S\right)$$

(e) SCPT :- Dutch Conepenetration Test

→ the cone resistance of soil is determined in kg/cm² when the cone is penetrated into the soil for 100mm@ 5mm /minute



→ Let $q_c \rightarrow$ Static cone penetration resistance of soil at the base of pile

$q_c \rightarrow$ avg. static cone penetration

$$Q_{up} = q_c A_b + \frac{q_c}{2} A_s \rightarrow kN \rightarrow \text{For displacement pile}$$

Where

A_b and A_s in m²

q_c and $\bar{q}_c$ in kg/cm²

$$Q_{up} = Q_{eb} + \frac{Q_{SF}}{2} = q_c A_b + \frac{\bar{q}_c}{4} A_s \rightarrow \text{For nondisplacement pile}$$

→ For bored pile $(Q_{up})_{bored} = \frac{1}{3}(Q_{up})_{driven}$

→ $(Q_{up})_{\text{non displacement pile}} = 50\% \ (Q_{up})_{\text{displacement pile}}$

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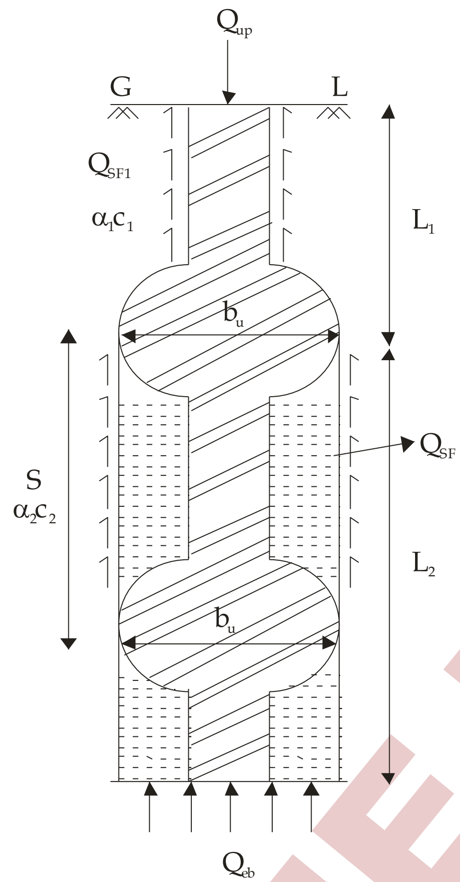
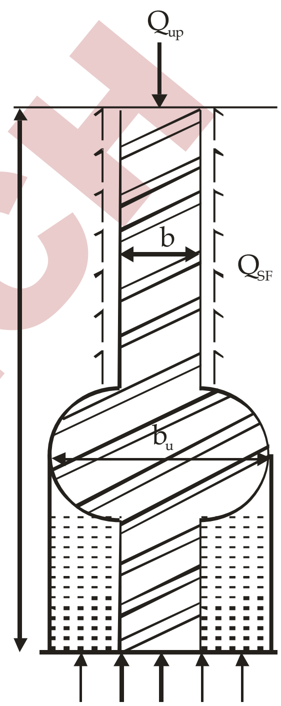
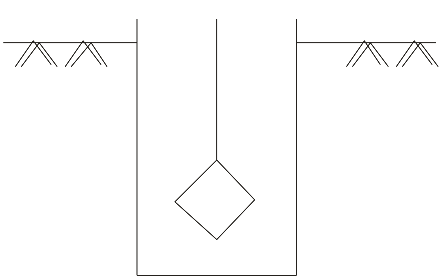
5. **Under Reammed Pile**



multi bulb under reammed piles





$\frac{b_u}{b} \rightarrow 2 \text{ to } 3$

- → Under reammed piles are used in expansive soils (high swelling and shrinkage) to prevent uplift pr caused by swelling such as in black soils.
- → Under reammed pile cannot be driven therefor these are cast in situ in bored piles
- → The depth at which bulb should be provided should be in stable zone it means there showed be no change of water content.

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→ Due to the bulb base resistance increase because the compacted soil below the bulb acts as part of pile

→ The bulb dia should be 2 to 3 times of shaft dia

→ In multibulb under reammed piles surface resistance also increase and min spacing (c/c) shuld be 1.5 times the bulb dia.

→ Ultimate load carrying capacity of single bulb under reammed pile is given as

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

$$Q_{up} = q.c.\frac{\pi}{4}(b_u)^2 + \alpha\bar{C}(\pi bL)$$ single bulb under reammed

→ For multibulb under reammed pile

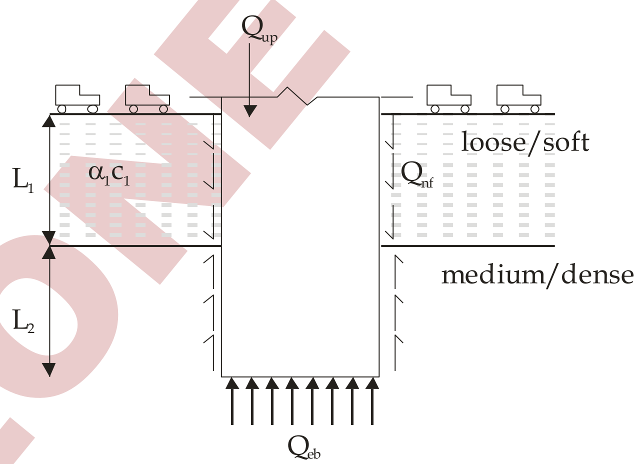
$$Q_{up} = Q_{eb} + Q_{SF1} + Q_{SF2}$$

$$Q_{up} = q.c.\left(\frac{\pi}{4}b_u^2\right) + \alpha_1\bar{c}_1(\pi bL_1) + \alpha_2\bar{c}_2(\pi b_u L_2)$$

$\alpha_2 = 1$ coheion between soil and soil

→ When the number of bulbs increased from one to two, the carrying capacities of the piles increase by about 50%

6. Negative Skin Friction ;-



→ It is a phenomenon in which soil surrounding to the pile settles more than the settlement of pile this condition occurs when soil surrounding to the pile is loose/soft under such condition friction on the pile acts down ward which reduces the load carrying capacity.

→ Following in cond. may cause negative skin friction

(i) increase the surcharge over the surrounding soil

(ii) Lowering of water table

(iii) due to seismic loading (OR) other dynamic loading

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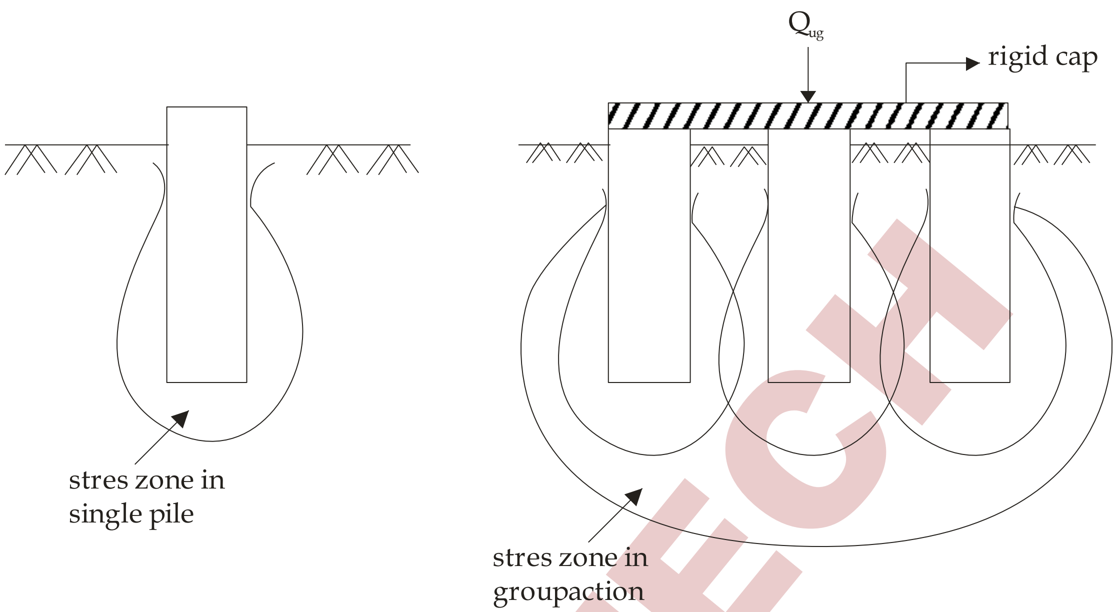
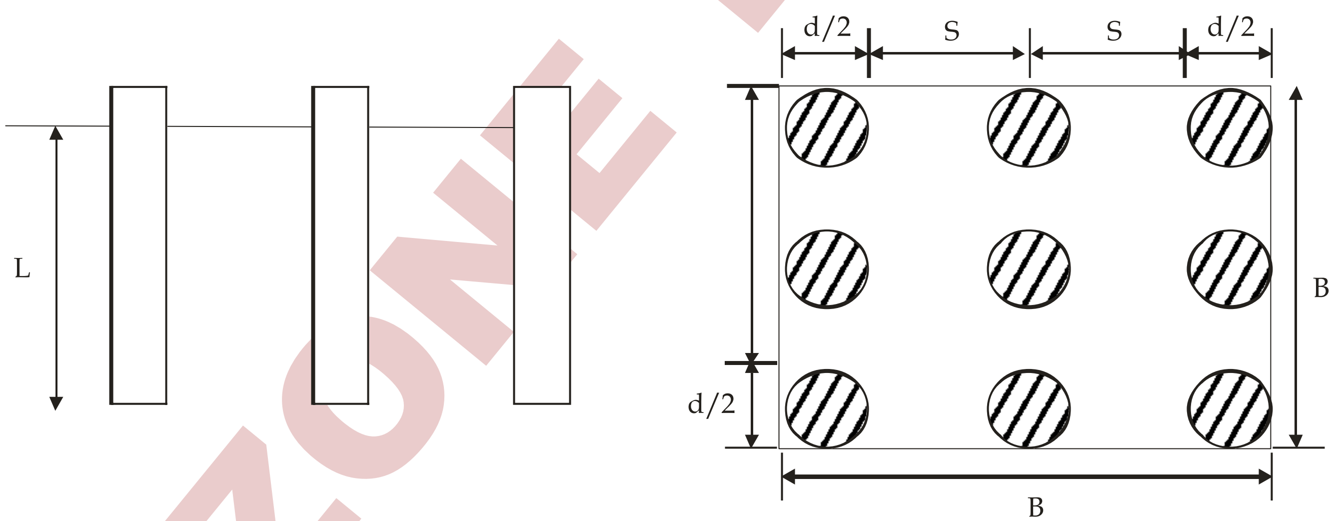
7. **Group Action of Piles**





$B = 2S + D = (x - 1)\,S + D$

x = No of pile in row/column

For 3 × 3 $\rightarrow$ $B = 2S + D$

For 4 × 4 $\rightarrow$ $B = 3S + d$

$A_b \rightarrow B^2$

$A_S \rightarrow 4B \times L$ (perimeter × Length of soil contact)

$\rightarrow$ If applied load is large and more no piles are used either piles will act indivisually or in the group depending upon spacing in between the piles

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- → If c/c spacing is 2.5d to 4d than soil may get compacted between piles and entire base of size B × B may act as a single pile such a action is called group action
- → in group action base area and surface area both will increase
- → in group action the depth of stress zone extends to greater depth then in indivisual action there for settlement due to consolidation in group action will always be greater than settlement in indivisual action
- → Min no of piles red for group action is 3
- → Pile group may be square/ triangle/ rectangular/ polygonal / circular however in design square pile group is preferred.
- → For group action c/c spacing should be

  (i) For end bearing action = 2.5d – 3.5d

  (ii) For friction action = 3d – 4d
- → The diameter of piles in group is 0.3m to 0.5m
- → Typical length and capacities of various piles

Determination of Ultimate Load Bearing Capacity of Pile Group (q_{ug})

$$q_{ug} = \gamma L N_q (B)^2 + \frac{1}{2} K\gamma L \tan\phi (4BL) \text{ or sand}$$

$$q_{ug} = 9cB^2 + \bar{C}(4BC)$$

$\alpha = 1$ = cohesion between soil and soil

$\delta = \phi$ = friction between soil and soil

Allowable load/ Safe load on pile group

$Q_{safe} \rightarrow$
- $\rightarrow \dfrac{Q_{ug}}{F.O.S}$ min. of these two
- $\rightarrow \dfrac{n.\ Q_{up}}{F.O.S}$ n = no. piles in group

Group Efficiencies (n_g)

→ If is defined as the ratio of ultimate load carying capacity of the pile group to the ultimate load carrying capacity of all the piles in individual.

action

$$\eta_g = \frac{Q_{ug}}{n.Q_{up}}$$

n = no ofpile in group

→ if $\eta_g \geq 1$ than $Q_{safe} = \dfrac{\eta.Q_{up}}{F.O.S}$

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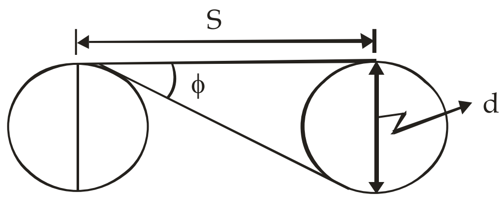
→ In design of pile group $\eta_g \geq 1$

Converse Labarre Formula

$$\eta_g = 1 - \frac{\phi}{90}\left[\frac{m(n-1)+n(m-1)}{m.n}\right]\times 100$$



$$\phi^\circ = \tan^{-1}\left(\frac{d}{s}\right)$$

m = No of columns

n = no of rows

m and n interchangable

Group Settlement ratio (GSR)

$$GSR = \frac{S_g}{S_i}$$

where

S_g = settlement of group

S_i = settlement of individual pile in group

ZONE TECH