AE/JE Foundation

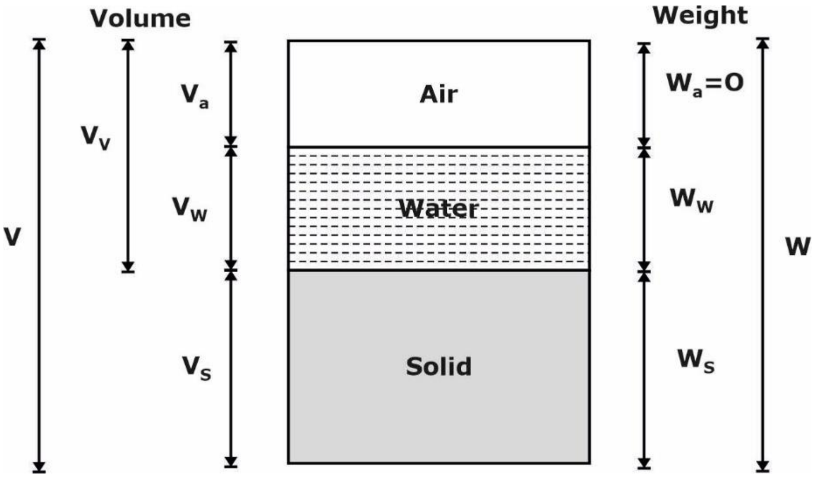
Civil Engineering

Soil Mechanics

Important Formula Notes

IMPORTANT FORMULAS ON SOIL MECHANICS

CHAPTER-1- ORIGIN & PROPERTIES OF SOIL



1. **Water content (w)**

$$w = \frac{\text{weight of water}}{\text{weight of soil solids}} = \frac{w_w}{w_s}$$

- The water content can be any value greater than or equal to zero, there is no upper limit.
- Dry weight of solids,

$$w_s = \frac{w}{1+w} = \frac{\text{total weight}}{1+\text{ water content}}$$

2. **Void Ratio (e)**

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

- void ratio can be any value greater than or equal to zero, there is no upper limit.
- Volume of soil solid,

$$V_s = \frac{V}{1+e} = \frac{\text{total volume}}{1+\text{ void ratio}}$$

3. **Porosity (n)**

$$n = \frac{\text{volume of void}}{\text{total volume}} = \frac{V_v}{V}$$

- Porosity can be any value between 0 to 1.
- It can be expressed in terms of void ratios.

$$\Rightarrow \eta = \frac{\text{e}}{1+\text{e}} \text{ or e} = \frac{\eta}{1-\eta}$$

4. **Degree of Saturation (S)**

$$S = \frac{\text{volume of water}}{\text{volume of void}} = \frac{V_w}{V_v} \times 100$$

- It can take any value from 0 to 100.
- If, S=0 then dry soil. If S=100 then fully saturated state, if S is in between 0 to 100 then partially saturated state.

5. **Air Content (a_c)**

$$a_c = \frac{\text{volume of air}}{\text{volume of void}} = \frac{V_a}{V_v} = 1 - S$$

- It can take any value between 0 to 100

6. **Percentage Air Voids (n_a)**

$$n_a = \frac{\text{volume of air}}{\text{total volume}} = \frac{V_a}{V} \times 100$$

7. **Density of Soil**

$$\text{bulk unit weight} = \frac{(G + eS)\gamma_w}{1 + e}$$

- For Dry unit weight, Put S=0.
- For saturated unit weight, Put S=1.

8. **Density Index or Relative density or Degree of Density (I_D)**

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

$$I_D = \frac{\gamma_{d_{max}}}{\gamma_d}\left[\frac{\gamma_d - \gamma_{d_{min}}}{\gamma_{d_{max}} - \gamma_{d_{min}}}\right]$$

I_D	Description of soil
0-15	Very loose soil
15-35	Loose Soil
36-65	Medium dense soil
66-85	Dense soil
86-100	Very dense soil

Soil having higher relative density is denser, possesses high shear strength and low compressibility.

9. **Relation between e, w, G and S**

$$e\,S = G\,w$$

10. **Relation between w, G, e, S, γ_w we know that**

$$\text{bulk unit weight} = \gamma = \frac{(G+es)\gamma_w}{1+e}$$

Case 1: Soil is dry

$\gamma = \gamma_d$; S = O

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

Case 2: Soil is saturated

$\gamma = \gamma_{sat}$; S = 1.

$$\Rightarrow \gamma_{sat} = \frac{(G+e)\gamma_w}{1+e}$$

Case 3: Soil is submerged

$\gamma = \gamma'$

$$\gamma' = \gamma_{sat} - \gamma_w \Rightarrow \gamma' = \frac{(G+e)\gamma_w}{1+e} - \gamma_w$$

$$\Rightarrow \gamma' = \frac{(G-1)\gamma_w}{1+e}$$

11. **Relationship between γ_d, γ and w**

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

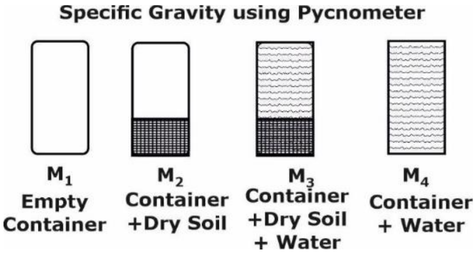
12. **Relationship between γ_d, η_a, G, γ_w, w**

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

13. **Classification of soil water:**

- **Gravitational water (free water):**water free to move through soil under influence of gravity
- **Held water: (held against gravity)**
  - **Structural water:** chemically combined in crystal structure of soil particle, it cannot be removed without breaking the structure of soil particles.This water is not removed by heating upto 105 to 110 degrees Celsius (used for drying the soil in laboratory condition).
  - **adsorbed water:**The water held by the fine-grained soil particles due to electrochemical force of adhesion it can be completely removed only above 200 degrees Celsius. At 105 to 110 degrees Celsius it cannot be completely removed but part of that can be removed that is called hygroscopic water.
  - **capillary water:** it is the water held in soil Mass due to capillary action. Capillary water can exist on a macroscopics scale as compared to other type of held water which can exist on microscopic scale.

14. **Pycnometer**



Calculation of specific gravity (G),

$$G=\frac{(m_2-m_1)}{(m_4-m_1)-(m_3-m_2)}=\frac{(m_2-m_1)}{(m_4-m_3)+(m_2-m_1)}$$

- For calculation ofG, dry soil is taken.
- The value of G is reported at 27℃ if the temperature during testing is different, than we should convert the G value at 27℃ to report its value

$$(G\gamma_w)_{T^\circ\mathrm{C}}=(G\gamma_w)_{27^\circ\mathrm{C}}$$

Calculation of water content (w),

$$w=\left[\frac{(m_2-m_1)}{(m_3-m_4)}\times\left(\frac{G-1}{G}\right)-1\right]$$

- For calculation of water content (w), moist soil is taken.

15. **Calculation of water content**

$$\text{moisture content }\%=w=\frac{w_2-w_1}{w_1}\times 100$$

where W_2 is weight of moist soil and W_1 is weight of dry soil.

16. **Measurement of unit weight**

- **By core cutter method**: it is a field method and applicable for soil which are soft and fine grained. It cannot be used for Stoney, gravelly and dry soil. Methodis applicable when surface of soil is exposed, and core cutter can be easily pushed**.** Volume of core cutter is 1000cc.
- **By sand replacement method:** it is a field method and suitable for gravelly,sandy and dry soil. Where core cutter method cannot be used.W= weight of soil excavated from the pit.

$$\gamma_t=\frac{W}{V}=\frac{W\times\gamma_{\text{sand}}}{\text{weight of sand in pit}}$$

$$\text{and }\gamma_{\text{sand}}=\frac{\text{weight of sand in cylinder}}{1000cc}$$

- **Water displacement method:**

$$\text{volume of sample}+\text{ volume of wax}=\frac{\text{weight of water overflowed}(W_2)}{\gamma_w}$$

$$\text{volume of } wax = \frac{\text{weight of sample after applying } wax(w_1) - \text{ original weight of sample}(w)}{\gamma_{\text{wax}}}$$

$$so, \gamma_t = \frac{W}{\frac{w_2}{\gamma_w} - \frac{w_1 - w}{\gamma_{wax}}}$$

17. **Index properties of soil**

- Index properties are those properties which helps in assessing the engineering behavior of soil and in classification of the soil into different groups so that a particular group is representative of a particular behavior.
- For coarsegrain soil, property of particle and the relative state of compactness are the most significant properties and for fine grained soil consistency and plasticity are important.
- **The soil grain** properties can be determined from remolded or disturbedsample, and they depend on the shape and size of grains and their mineralogical composition. It is independent of mode of soil formation.
- **soil aggregate properties** depend on mode of soil formation, soil history and soil structure. and these properties are determined from undisturbed sample or preferably from insitu tests.

18. **Grain size analysis:**

- Grain size analysis is important mainly for coarse grain soil because in fine grained soil interparticle forces and water content is more important.
- For coarse grain soilgrain size analysis is carried out by sieve analysis and for fine grain soil it is analyzed by sedimentation analysis.
- For wet analysis soil passing 4.75mm sieve is washed over 75μ sieve and then oven dried and fine sieve analysis is performed with this oven dried sample.
- **Fineness modulus:** it is the sieve number at which average size of particles are said to lie when counted from the lower order sieve size to the higher order sieve size.

s. no.	Sieve size	Wt. retained (gm)	Cumulative weight retained (gm)	% Cumulative weight retained	% Finer
↓	↓	↓	↓	↓	↓

$$\text{Fineness modulus} = \frac{\sum \%\text{ cummulative weight retained}}{100}$$

$$C_u = \frac{D_{60}}{D_{10}} = \text{ uniformity coefficient}$$

$$c_c = \frac{{D_{30}}^2}{D_{10} \times D_{60}} = \text{ coefficient of curvature}$$

- C_u basically represent the slope of the curve between D_{60} and D_{10} and hence it is related to degree of uniformity of the sample. If C_u is 1, then soil is perfectly uniform graded, and the curve will be vertical. If C_u is lessthen 2 or 3 it is assumed that soil is uniformly graded.
- If C_c is not between 1-3 then it is said that the soil is gap graded, it means certain size of soil are missing.

19. **Sedimentation analysis:** stokes law is valid for particle od diameter 0.2mm to 0.0002mm.

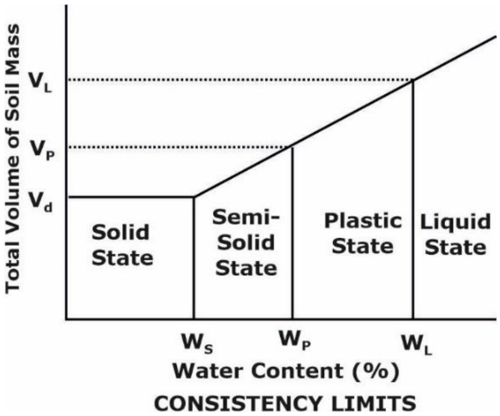
$V_S = \dfrac{(\gamma_s - \gamma_w) \times d^2}{18\mu}$ and $\dfrac{H_e}{t} = \dfrac{(\gamma_s - \gamma_w) \times d^2}{18\mu}$ $\%\text{ finer} = \dfrac{\text{concentration of sample at time 't' from height } H_e}{\text{original concentration of sample}}$	here, γ_s = unit weight of soil particles γ_w = unit weight of water d = size of particle μ = dynamic viscosity of water

- Deflocculating agent or dispersing agent like sodium hexametaphosphate and sodium oxalate is used.

20. **Consistency of soil:**

- Consistency represents the relative ease with which soil can be deformed.It is related to clay and the water content. Change in water content leads to the change in consistency of the soil.
- Atterberg analysed the consistency of soil in four stages.

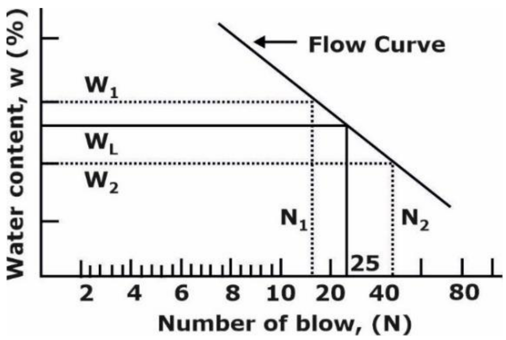
(a) Solid,
(b) Semi-solid,
(c) Plastic,
(d) Liquid



21. **The liquid limit** is determined by plotting a graph between number of blows as abscissa on a logarithmic scale and the corresponding water content as ordinate. Liquid limit is corresponding to 25 number of blows.

   - slope of flows curve is known as **'Flow Index'** and it represents the rate of loss of shear strength of soil with increase in water content. Mathematically, flow index (I_f) is given as

$$\tan\theta = I_f = \frac{W_1 - W_2}{\log_{10}\dfrac{N_2}{N_1}}$$

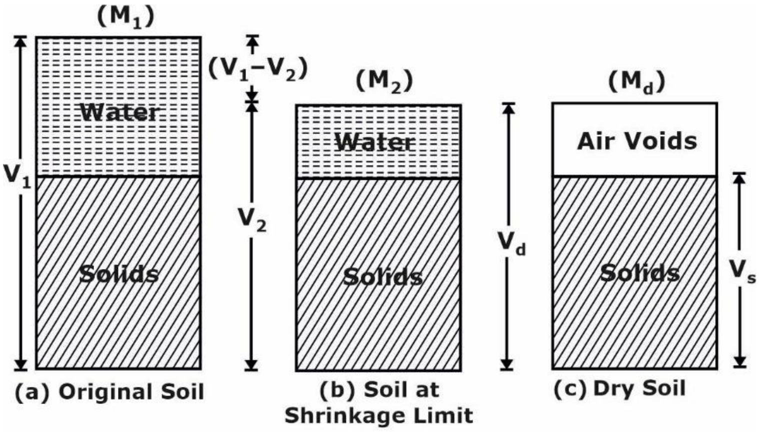


Note: At liquid limit all soils will have same shear strength equal to $2.7 KN/m^2$.

Note: if the rubber pad used during Casagrande method is harder than the standard rubber than the liquid limit reported will be smaller.

22. **Plastic limit:** Plastic limit is defined as the water moisture content at which a thread of soil with 3.0 mm diameter begins to crumble.

23. **Shrinkage Limit (W_S):** maximum water content, below which change in water content does not change volume of soil.



SHRINKAGE LIMIT DETERMINATION.

- **Shrinkage Ratio (SR):**

Shrinkage ratio,

$$SR = \frac{\frac{V_1 - V_2}{V_d}}{W_1 - W_2} \times 100 = \frac{\frac{V_2 - V_d}{V_d}}{W_2 - W_d} \times 100 = \frac{\text{slope}}{V_d} \times 100$$

Here,

V_1 = volume of soil mass at water content W_1

V_2 = volume of soil mass of water content W_2.

V_d = volume of dry soil mass.

- **Volumetric Shrinkage (V_S):**

$$VS = \left(\frac{V_1 - V_d}{V_d}\right) \times 100$$

24. **Consistency Indices**

(i) Plasticity Index (I_P):

$$I_P = W_L - W_P$$

Also, the plasticity can be classified based on I_P as

I_P	Plasticity
0	Non plastic
< 7	Low plastic
7-17	Medium plastic
> 17	High plastic

(ii) Relative Consistency / Consistency Index:

$$I_C = \frac{W_L - W}{I_P} = \frac{W_L - W}{W_L - W_P}$$

(iii) Liquidity Index (I_L)/Water plasticity ratio:

$$I_C = \frac{W - W_P}{I_P} = \frac{W - W_P}{W_L - W_P}$$

Note: The sum of liquidity index and consistency index will always be equal to 1.

$I_L = 1 - I_C$

Consistency	Description	I_C	I_L	UCS (Kg/cm^2)
Liquid	Liquid	<0	>1	-1
Plastic	Very soft	0-0.25	1-0.75	0-0.25
	Soft	0.25-0.5	0.75-0.50	0.25-0.5
	Medium stiff	0.5-0.75	0.5-0.25	0.5-1.0
	Stiff	0.75-1.0	0.25-0	1-2
Semi-solid	Very stiff to hard	>1	<0	2-4
Solid	Hard to very hard	>1	<0	>4

(iv) Toughness Index (I_T):

$$I_t = \frac{I_p}{I_f} = \frac{\textit{plasticity}\text{ index}}{\text{flow index}}$$

It represents the shear strength at plastic limit.

For most of the soil it is between 0-3, if $I_t<1$ then soil is friable. For most of the clay it is 1-3.

25. **Sensitivity:** degree of disturbance on remoulding is called sensitivity.

$$S_t = \frac{(UCS)_{\text{undisturbed state}}}{(UCS)_{\text{remoulded state}}}$$

S_t	Description
1	Intensive Soil
2-4	Normal/Less sensitive soil
4-8	Sensitive soil
8-16	Extra sensitive soil
>16	Unstable/Quick soil

26. **Thixotropy:** it is the property of soil due to which loss of strength(shear) on remolding can be regained if left undisturbed for some time.

- Increase in strength in due passes of time is due to tendency of clay soil to gain their chemical equilibrium with reorientation of water molecules in the absorbed layer.
- Clay soil will have large thixotropy then single grain soils.

27. **Activity:**

$$A_t = \frac{I_P}{\%C}$$

Here,

A_t	Description
<0.75	Inactive soil

I_P = plasticity index

%C = percent of clay particles

0.75–1.25	Normal active soil
>1.25	Active

Note 1:AsI_p↑ then, plasticity↑, shrinkage limit↓, swelling shrinkage↑, permeability↓,clay fraction↑, creep rate↑, organic content↑, dry strength↑,compressibility↑, cohesion↑ and smaller compacted density and larger OMC.

Note 2: As W_L↑ then, compress ability↑, organic matter↑, rate of volume change↓

Note 3: As A_c↑ then, compressibility↑,swelling shrinkage↑.

Note 4: As organic content↑ then,moisture content↑, density of soil↓, plasticity index↑, UCS↓,compressibility↑, secondary compression↑, shrinkage↑, permeability↓.

Note 5:

characteristics	W_L same, I_P↑	I_p same, W_L↑
dry strength	↑	↓
toughness near plastic limit	↑	↓
compressibility	-------	↑
permeability	↓	↑
rate of volume change	↓	↑

- If W_p is less, it means that at smaller moisture content also soil is plastic this means that there must be greater clay fraction.
- W_P is more it means soil is plastic at higher moisture content only.

CHAPTER-2 CLASSIFICATION OF SOIL

PARTICLE SIZE CLASSIFICATION

According to the IS classification:

Type of soil		Size of particles (mm)
Clay	-	<0.002
silt		0.002-0.075
Sand	Fine	0.075-0.425
	Medium	0.425-2.0
	Coarse	2.0-4.75
Gravel	Fine	4.75-20
	Coarse	20-80
Cobble	-	80-300
Boulder	-	>300

INDIAN SOIL CLASSIFICATION (ISC) SYSTEM

- Classification of soil is done the basic of IS1498. Soil isclassified as coarse grained/fine grained/peat.
- **Prefix and suffix used in soil classification.**

Soil type	Prefix	subgroup	suffix
Gravel	G	Well graded	W
Sand	S	Poor graded	P
Silt	M	Silty	M
Clay	C	Clayey	C
organic	O	$W_L<35$	L
peat	P	$35<W_L<50$	I
		$W_L>50$	H

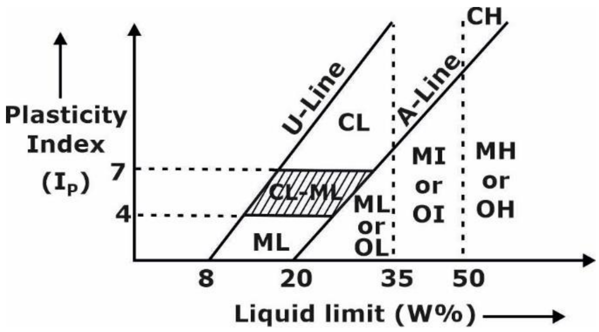
- **Plasticity chart:**
- Equation of A-line:

$$I_P = 0.73\ (W_L - 20)$$

- Equation of U-line:

$$I_P = 0.9\ (W_L - 8)$$

- points which represent different sample from the same soil stratum will define a line that is roughly parallel to A-line.



- The soil sample from common geological origin will have relationship between I_p and W_L which will be parallel to A-line.

- soil is divided into 3 broad divisions:
    - **coarse grain soil**: when 50% or more of the total material by weight is retained on 75 microns IS sieve.
    - **Fine grain soil**: open 50% or more of the total material by weightpasses the 75 MicronsIS sieve
    - If the soil is highly organic and contains a large percentage of organic matter and particle decomposedvegetables,then it is kept in a separate category called peat.
- Organics soil is distinguished from inorganic one by color and odor, organic soil is dark in color, which have distinctive odor and odor is made more noticeable by heating the wet sample.
- If organic content is still doubtful, material can be oven dried and remixed with water and tested for liquid limit.If reduction in liquid limit is more than 25% of the liquid limit before oven drying, then it is an organic sample.

COARSE GRAINED SOIL:

- It is done on the basic of (i) grain size, (ii) gradation characteristic, (iii) percentage of fines (<75-micron particles) present in the soil weight. when 50% or more of the total material by weight is retained on 75 microns IS sieve.

%ofFines is less than 5%

Gravel		Sand	
Well graded gravel (GW)	**Poorly graded gravel (GP)**	**Well graded sand (SW)**	**Poorly graded sand (SP)**
More than half of the coarse fraction is retained on 4.75mm sieve. (gravel>sand)		**More than half of the coarse fraction passes over 4.75mm sieve. (gravel<sand)**	
$C_u>4$ and C_c is between (1,3)	**Otherwise poorly graded**	**$C_u>6$ and C_c is between (1,3)**	**Otherwise poorly graded**

% Fines is greater than 12%

- Gravel and sand are classified as specified above and type of fine is calculated based on the plasticity chart, name can be GC,GM, SC, SM.
- If fines get plotted in hazed zone it means fines >12% and I_p is between (4-7) then dual symbol is used, GM-GC, SM-SC.

% Fines is between 5 to 12%

- It is a border line case and dual symbol are used, first part indicate gradation and second part indicates nature of fines.
- Symbols can be: GW-GC or GW-GM, GP-GC or GP-GM,GW-GC or SW-GM,SW-GC or SW-SM
- if on plasticity chart soil lies near A line or hatched zone then we prefer non-plastic classification. Names will be like: GW-GM, not GM-GC.

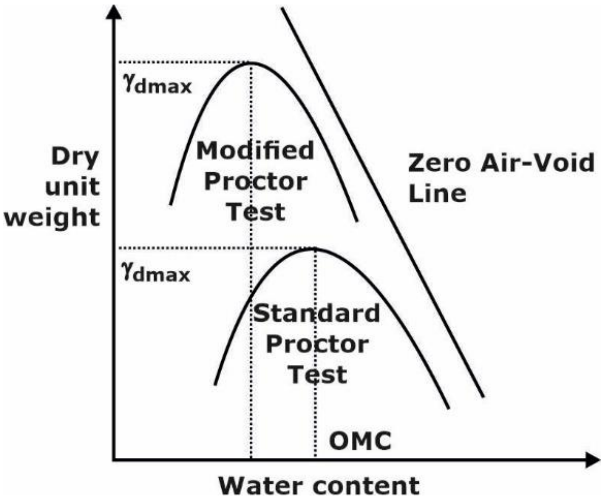
FINE GRAINED SOIL

- Fine grained soil is classified based on the plasticity chart.
- For soil less than 425-micron size, W_L and W_p are determined and plotted on the plasticity chart which is used for naming the soil accordingly where it is plotted.

Note: if sand and gravels are present in equal amount, then dual symbols are used, for ex: GW-SW, GP-SP or GM-SM, GC-SC. If fines classification is also at border, then non-plastic classification is classified as already dual symbols are present.

CHAPTER-3 SOIL COMPACTION

PROCTOR TEST



OMC = optimum moisture content

- The moisture content at maximum dry density is known as optimum moisture content.
- Maximum dry density is a function of type of soil, compaction effort and method of compaction.
- **Zero air voids line:** maximum dry density achieved at a given m/c by removing all air voids is called zero air void dry density.

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

here,

w = water content of compacted soil

γ_d = dry unit weight

G = specific gravity

γ_w = unit weight of water

- Note: line of optimum is almost parallel to zero air void dry density.
- **Field control of compaction:** In field we need to calculate γ_d and water content. Proctor needle can be used to calculate the field density and then using water content we can calculate the dry density.

$$\text{relative density} = \frac{\gamma_{\text{dry}}\ (\text{field})}{\gamma_{d_{\max}}\ (\text{lab})}$$

Note: 10% air content density and 90% saturation density are same, and 0% air content and 100% saturation density are same. But, 10% air void and 90% saturation density are not same, although 0% air void and 100% saturation density are same.

Comparison and data for proctor tests:

	Standard proctor test	Modified proctor test	IS light compaction	Is heavy compaction
Weight of hammer	2.495 kg	4.54 kg	2.6 kg	4.9 kg
Height of fall	304.8 mm	457.2 mm	310 mm	450 mm
Volume of mould	944 cc	944 cc	1000 cc	1000 cc
No of layers	3	5	3	5
No of blows in each layer	25	25	25	25
Compaction energy per unit volume.	592.7 KJ/m^3	2696.3 KJ/m^3	593.01 KJ/m^3	2703.88 KJ/m^3

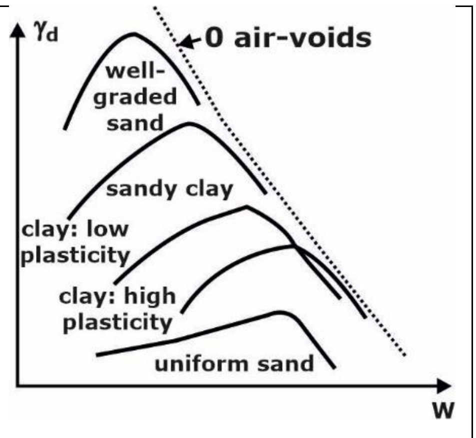
Note: energy in heavy is 4.54 times energy used in light compaction, the ratio is same for modified and standard proctor test as well.

Effect of compactive effort:

- for a given type of soil higher is the compaction effort higher will be the dry density and lesser will be the OMC.
- On wet side of optimum increase in efforts will have little effect on dry density. If large compactive effort is applies on wet side of optimum, then density will not increase much instead pore pressure may built up which will create slope stability problem and induce time dependent settlement.
- Degree of saturation at OMC is almost same in all cases.

Effect of type of soil:

- Coarse grained soil, well graded can be compacted to high dry density, especially when some fines are present, but as quantity of fines increases maximum dry density decreases.
- Poorly graded sand will have least maximum dry density.
- In case of clays, maximum dry density decreases, and OMC increases as plasticity of the soil increases.
- Increases in organic content will increase OMC but decreases the maximum dry density.



Effect of compaction on property of soil:

Property	Dry of optimum	Wet on optimum
Structure	Flocculated (random)	Dispersed (oriented)
Water deficiency	More	Less

Swell ability		More	Less
Construction pore water pressure		Less	More
Shrinkage		Less	More
Permeability It is minimum at w/c slightly more than OMC.		More (isotropic)	Less (anisotropic) ($K_h > K_v$)
Sensitivity		More	Less
Shear strength		High	Low
Stress strain behaviour		Brittle (High modulus of elasticity) high peak	Ductile (Low modulus of elasticity) no peak
Compressibility	At low stress	Less	More
	At higher stress	More	Less

Suitability of compaction equipment:

Type of equipment	Suitability of soil type	Nature of projects
Rammers, tampers or frog hammer	All type of soil	In confined area such as fills behind the retaining wall, basement wall trench fill etc.
Smooth wheel roller	Crushed rock, gravel, sand, ballast	Road construction. Not suitable for embankment Not suitable for soft subgrade and uniform sand
Pneumatic tyred roller	Sand, gravel, silts, clay of all compressibility	Not suitable for uniformly graded soil, base-subbase, embankments, earthen dam, air field.
Sheep foot roller	Clayey soil, silty and clayey sand, silty clay of all compressibility	Core of earthen dam
Vibratory roller	Sand, granular base coarse, asphalt	Embankment for oil storage tank and base course
Grid roller	Well graded coarse soil, weathered rock, crushed over size material.	Not suitable for clayey soil, silty clay or uniform subgrade

CHAPTER-4 EFFECTIVE STRESS &CAPILLARITY

STRESS CONDITIONS IN SOIL

Mathematically total stress in the soil mass can be expressed as follows

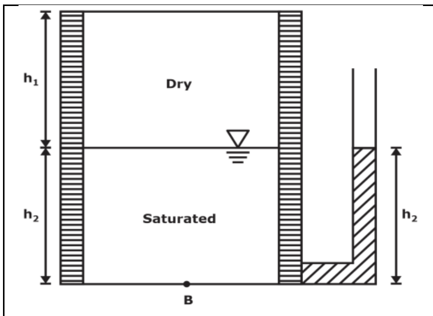
Total stress (σ)= Effective stress ($\bar{\sigma}$) + Pore water pressure (u)

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

- Total stress on a plane within a soil mass is the force per unit area transmitted in normal direction on a plane across the soil mass.
- Pore pressure is the pressure applied by the fluid in the pores of the soil mass. It is also called neutral stress because it acts on all sides of the particles and does not cause particles to stress against each other.
- Effective stress does not represent the contact stress because we are not dividing summation of normal force with the area of contact between grains rather, we divide it by total area. Hence it does not have any physical significance. It is just a term taken as (σ-u).

EFFECT OF EXTERNAL LOAD ON PORE PRESSURE:

- If soil is fully saturated and soil particles and water are incompressible. Also, soil is laterally restrained. At time t=0, all the external loads will be taken by water. Then slowly water leaks out and load will be transferred to soil particles. At time t=0, as soil particles and water both are incompressible so soil particles will try to occupy a position closer together, soil is laterally restrained hence it is not possible to rearrange without expulsion of pore water. Excess pore water pressure will be generated which is over and above the hydraulic water pressure.



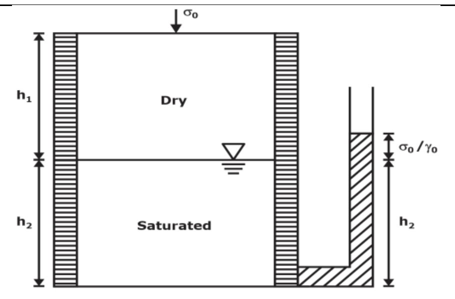
Before load application

$$u_2 = \gamma_w h_2$$

$$\bar{\sigma}_B = \sigma_B - u_B$$

$$\bar{\sigma}_B = (\gamma_d h_1 + \gamma_{sat} h_2) - (\gamma_w h_2)$$

$$\bar{\sigma}_B = \gamma_d h_1 + \gamma_{sub} h_2$$

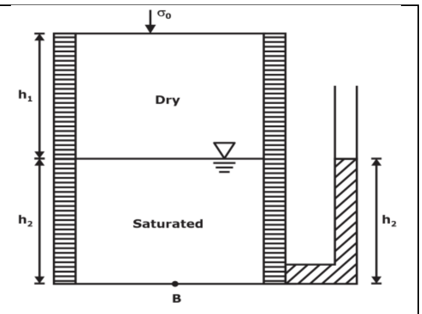


At time t = 0, σ_0 is applied

$$\sigma_B = \sigma_0 + \gamma_d h_1 + \gamma_{sat} h_2$$

$$u_B = h_2\gamma_w + \frac{\sigma_0}{\gamma_w} + \gamma_w$$

$$\bar{\sigma}_B = \sigma_B - \sigma_B = \gamma_d h_1 + \gamma_{sub} h_2$$

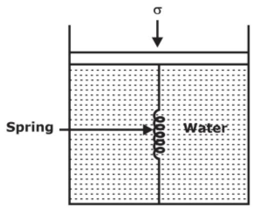


After a long time of application of σ_0

$$\bar{\sigma}_B = \sigma_B - u_B$$

$$\bar{\sigma}_B = \sigma_0 + \gamma_d h_1 + \gamma_{sub} h_2$$

❖ Excess pose water pressure will be zero.



- At t = 0 all the load will be taken by water. After water leaves then spring will take load
- If at t=0, soil is subjected to reduction in total stress then the excess pore water pressure will become negative and in long term it will be zero and effective stress will also decrease.

EFFECT OF CAPILLARY RISE IN THE SOIL

- Due to capillary effect, pore water pressure become negative and hence effective stress increases.
- Certain depth above the water table is completely saturated with capillary water and it is called capillary saturation zone and in this zone the pore water pressure in taken to vary linearly.
- The depth of capillary saturation zone in sand and silt in givenby,

$$\frac{0.03}{0.2D_{10}} = \alpha$$

- D_{10} in mm and $\alpha \to$ meter.

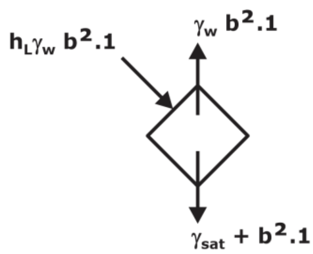
$$\alpha = \frac{C}{eD_{10}}$$

- $\alpha \to$ cm, D10 $\to$ cm
- C $\to$ empirical constant (value 0.1 – 0.5 cm2)

- Soil having same D_{10} value can have different capillary rise depending on soil structure and zoological history.
- Due to increase in effective stress on account of capillary action shear strength of soil increase.
- Bulking of sand is due to capillary effect.

EFFECT OF SEEPAGE ON EFFECTIVE STRESS

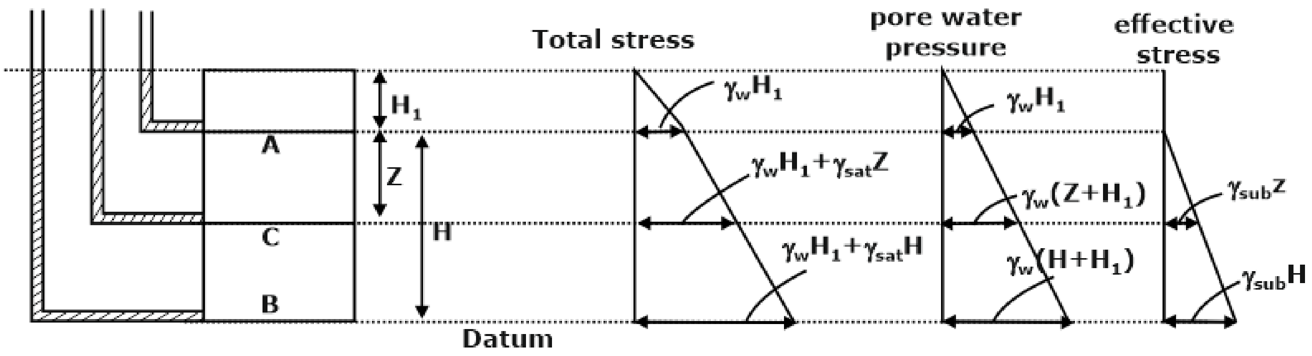
- When water seeping through the soil total head is dissipated as viscous friction producing a frictional drag in the direction of flows on the soil particles. This drag force results in seepage force on the soil mass which acts in the direction of flow.
- Component of seepage force acting vertically upward will reduce the effective vertical stress and component of this seepage force acting vertically downward will increase the effective vertical stress.
- Under no flow condition surface force acting is only equal to buoyant force acting in vertically upward direction and under flowing condition, apart from the buoyant force there will also be a seepage force acting in the flow direction and seepage force is equal to [Seepage pressure × Area]
- Seepage pressure = $h_2\gamma_w$
- Seepage force per unit volume = $\frac{h_2\gamma_w g \times 1}{b^2 \times 1} = \frac{h_2\gamma_w}{b} = i\gamma_w$
- Resultant force on the soil mass in the vector summation of buoyant weight and seepage force.



- The above resultant force will be balanced by the contact forces and vertical component of the contact force will be equal to

[Buoyant weight ± vertical component of seepage force].

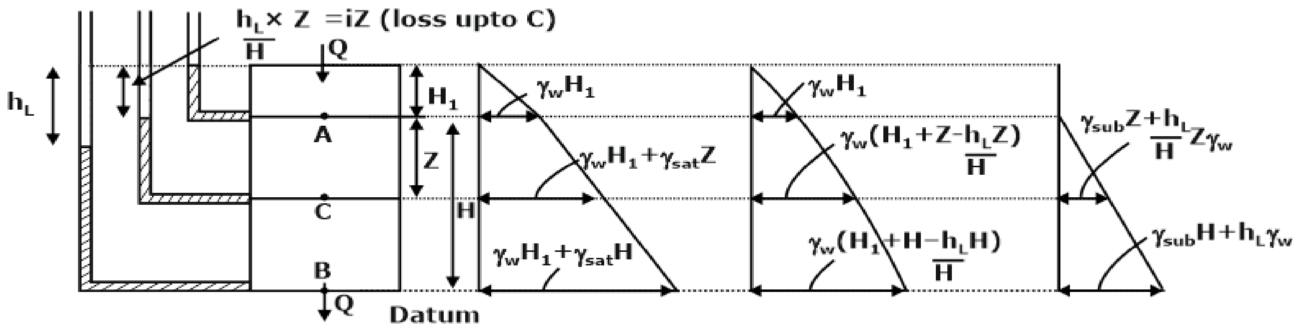
No flow condition:



	Datum head	Pressure head	Total head
A	H	H_1	$H + H_1$
B	0	H_1+H	$H + H_1$
C	H – Z	$H_1 +Z$	$H + H_1$

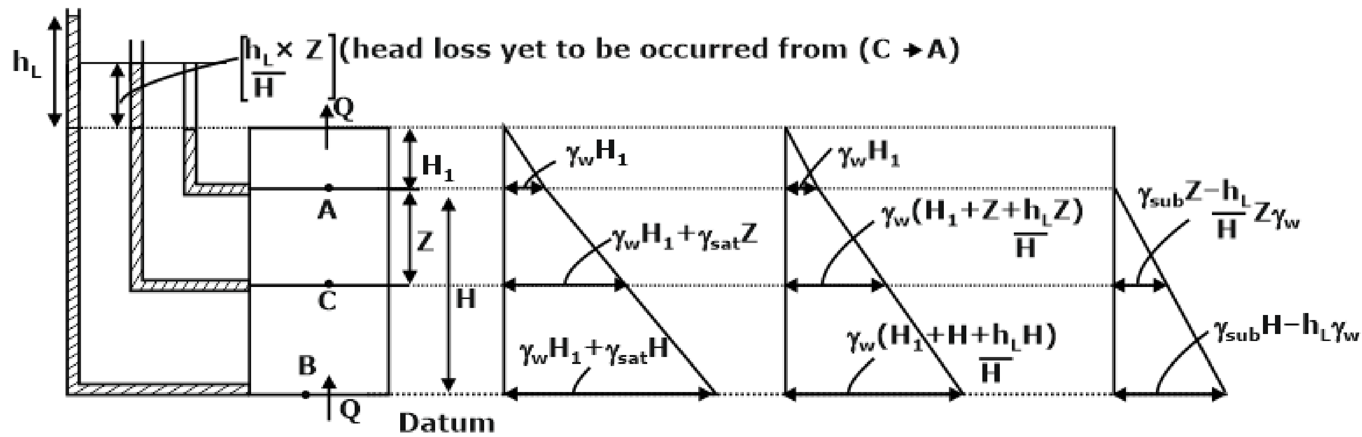
Note: Pressure is not affected by datum assumed.

Downward flow situation



	Datum head	Pressure head	Total head
A	H	H_1	$H + H_1$
B	0	$H_1 + H - h_L$	$H + H_1 - H_L$
C	H – Z	$H_1 + H - \frac{h_L}{H} \cdot Z$	$H + H_1 - \frac{h_L}{H} \cdot Z$

Upward flow situation



	Datum head	Pressure head	Total head
A	H	H_1	$H + H_1$
B	0	$H_1 + H + h_L$	$H + H_1 + H_L$
C	H – Z	$H_1 + \frac{h_L}{H} \cdot Z + Z$	$H + H_1 + \frac{h_L}{H} \cdot Z$

If a force F acting at top, then

$$\bar{\sigma} = \frac{F}{A} \pm \frac{\text{Seepage force}}{A} + \frac{\text{Buoyant force}}{A}$$

$$\begin{bmatrix} + \downarrow \text{Flow} \\ - \uparrow \text{Flow} \end{bmatrix}$$

$\bar{\sigma}$ = effective stress under no flow ± Seepage pressure ($h_L.\gamma_w$)

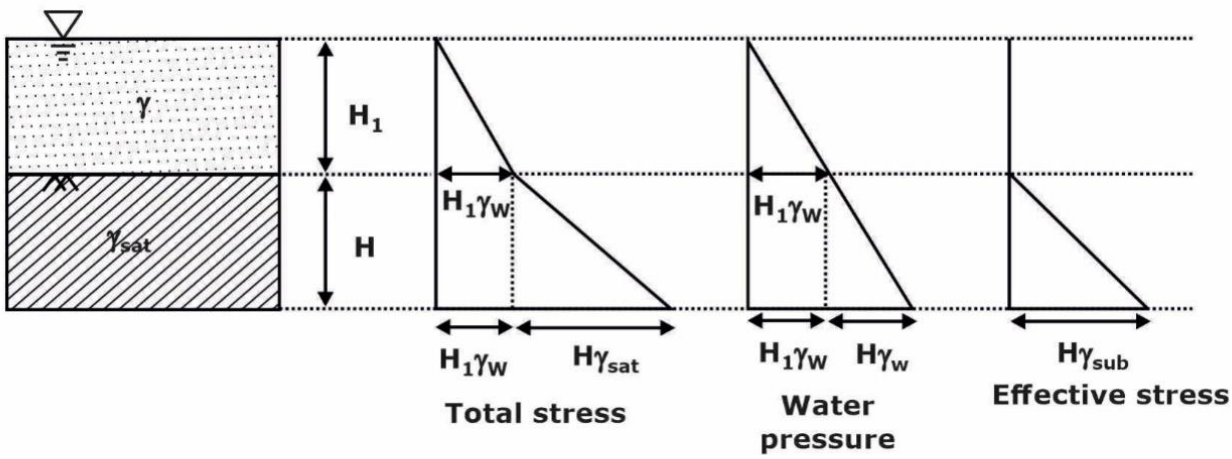
$h_L \rightarrow$ Loss already occurred up to the point under consideration in downward flow.

$h_L \rightarrow$ Loss yet to occur beyond the point under consideration in upward flow.

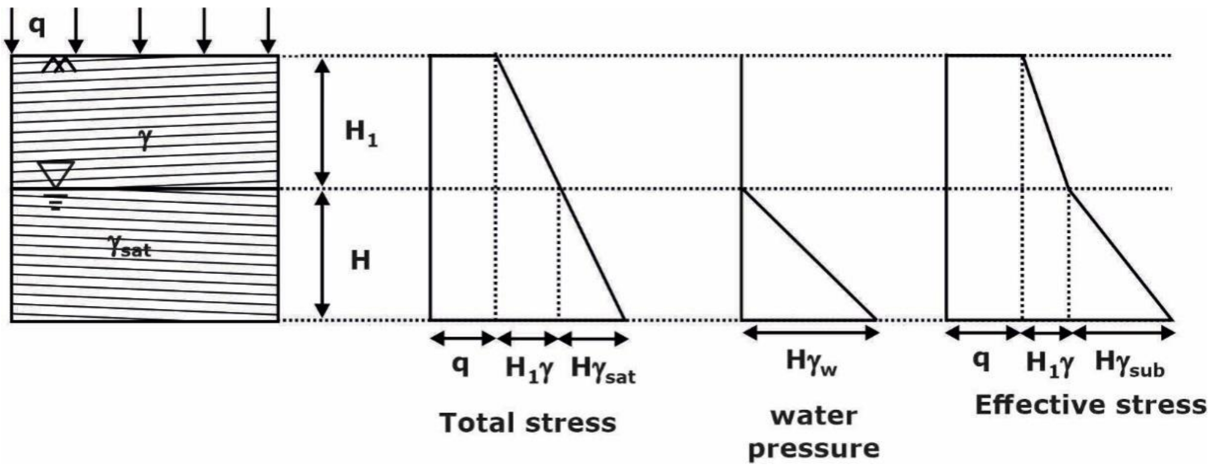
Effective vertical stress at any point for any direction of flow can be easily calculated as = [Total vertical stress – Pore water pressure]

Note: Buoyant force is equal to effective stress under no flow condition.

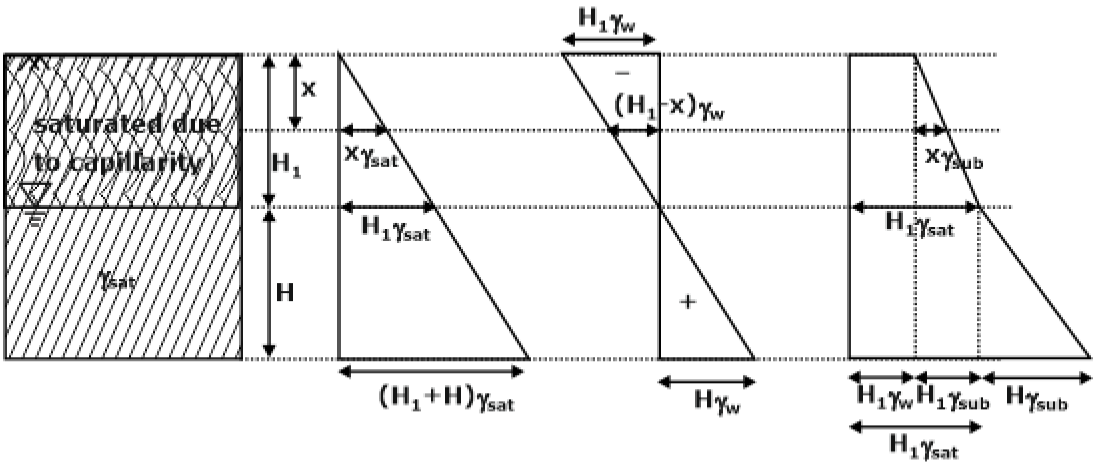
Variation of stressin case of water is above the soil mass



Variation of stress for Soil mass with surcharge



Variation of Stress with Soil mass with capillary fringes



Note: If the soil above H is saturated by ground water table instead of capillary water, effective stresses in the soil will be reduced and it will behave as submerged soil.

QUICKSAND CONDITION

- In case of upward seepage flow if the upward seepage force becomes equal to buoyant weight of soil effective stress at every point in the soil will be equal to zero.

  $(\bar{\sigma})_C = \gamma_{sub}Z - \frac{h_L}{H}\cdot Z\gamma_w = 0 \Rightarrow i = \frac{\gamma_{sub}}{\gamma_w}$ and $(\bar{\sigma})_B = \gamma_{sub}H - \frac{h_L H r_w}{H} = 0 \Rightarrow i = \frac{\gamma_{sub}}{\gamma_w}$
- critical hydraulic gradient at which effective stress is zero at every point in the soil mass.

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

- In case of sand $\left[t = C + \bar{\sigma}\tan\phi\right]$, $c = 0$ hence if $\bar{\sigma} = 0$ then τ = 0, sand will loose its shear strength completely. This condition is called quicksand condition.
- To safeguard against the quicksand condition during excavation the option are:
  ⇒ Lower the water table by pumping.
  ⇒ Increase the depth of embedment of sheet pile.
  ⇒ by helping certain depth of water in pit.
  ⇒Sometimes are also add graded filter on down stress side of masonry dam because at that point there could be chance of quicksand condition taking place.
  - **Factor of safety against quicksand**condition is equal to,

$$\text{FoS} = \frac{\text{Buoyant force}}{\text{Seepage force}}$$

$$\text{FoS} = \frac{\text{Effective stress under no flow condition}}{\text{Seepage pressure}}$$

Note: In case of clay if the pore water pressure exceeds the total stress, we will have a heaving problem and that may lead to cracking of clay. To safeguard against heave,wehave to ensure that stress is greater than pore water pressure and

$$\textbf{FoS against heavying} = \frac{\text{Total stress}}{\text{Pore water pressure}}$$

CHAPTER-5 PERMEABILITY

DARCY'S LAW

As per Darcy's law, for 1-D in saturated soils, velocity of flow is directly proportional to hydraulic gradient. Darcy law is valid when the flow through the soil is laminar, soil is saturated, flow is steady, and flow is irrotational.

Note: flow through the soil finer than coarse sand is generally laminar in the field, however flow through the gravel and rock fills generally becomes turbulent and Darcy's law is invalid. If the volume and shape of water passage varies with time due to impurity in flowing fluid, then also Darcy's law is invalid.

As per Darcy's law, $v \propto i \Rightarrow v = ki$

Here, **k = permeability or coefficient of permeability** and v is discharge velocity.

Discharge:

$$Q = kiA$$

Here, k= coefficient of permeability
A= total area of flow
i= hydraulic gradient

Seepage velocity:

$$\frac{V_{discharge}}{n} = V_{seepage}$$

Note: seepage velocity is generally small and hence in seepage condition, for calculation the total head taken is piezometric head only. And difference of piezometric head between two points is the head loss between two points. i.e. difference in level of liquids in the two piezometers inserted at two points in the flow is the head loss between these two points.

Note: actual rise in piezometer is pressure head.

Factors Affecting Permeability

$$k = C_s\left(\frac{\gamma}{\mu}\right)\left(\frac{1}{S_0^2}\right)\left(\frac{e^3}{1+e}\right)S^3$$

Here, C_s= shape factor
μ =dynamic viscosity of fluid
S= degree of saturation
γ =unit weight of fluid

S_0= wetted surface area per unit volume of surface and it is dependent on particle size and soil fabric. It is assumed to be a measure of pore size.

(i) Grain size:

- Permeability varies approximately as the square of the grain size.
- According to Allen Hazen

$$\boxed{k = CD_{10}^2}$$

k = coefficient of permeability (cm/sec)
c = constant (value = 100)
D_{10} = effective diameter (cm)

(ii) Effect of void ratio:

- Void ratio and permeability can be related by the following expression.

$$\boxed{k = \frac{e^3}{1+e}}$$

Where, e = void ratio of soil

(iii) Density and Viscosity of Fluid:

- This relation between the fluid property and permeability can be given as

$$k \propto \left(\frac{\gamma}{\mu}\right)$$

Where, γ = unit weight of fluid

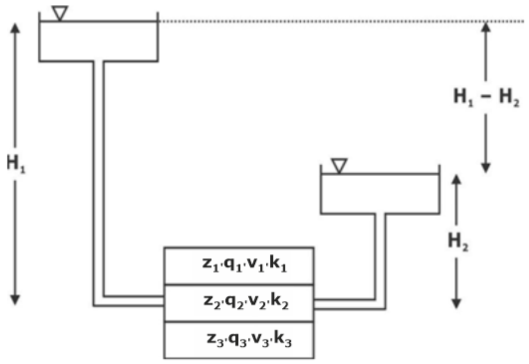
μ = dynamic viscosity of fluid.

Note: intrinsic permeability (k_i), unit is M^2 or Darcy. It is independent of fluid property.

$$k = k_i \frac{\gamma}{\mu}$$

DETERMINATION OF AVERAGE PERMEABILITY

- **parallel to Bedding plane**

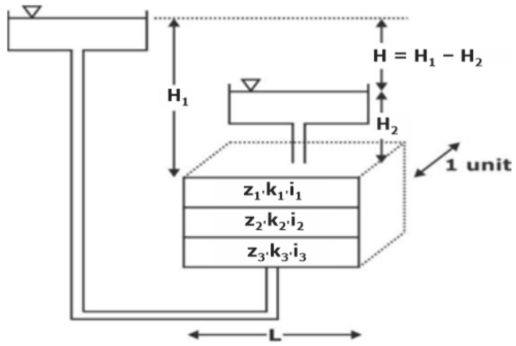


$$\boxed{k_x = \frac{\Sigma k_i z_i}{\Sigma z_i}}$$

Here,

- Z_1, Z_2, and Z_3 = thickness of layer
- v_1, v_2 and v_3 = velocity through layers.
- k_1, k_2 and k_3 = coefficient of permeability
- hydraulic gradient (i) will be same in all
- q_1, q_2 and q_3 = discharge through the layer

- **perpendicular to bedding plane**



$$\boxed{K_z = \frac{\Sigma z_i}{\Sigma \frac{z_i}{K_i}}}$$

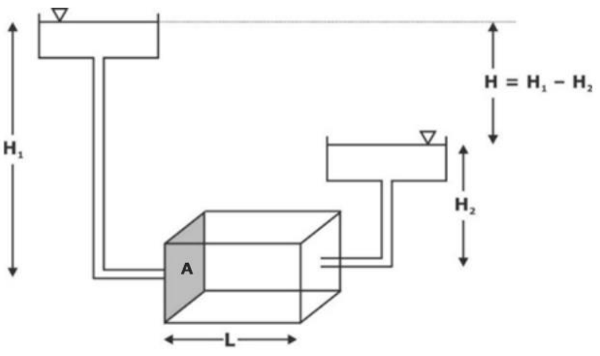
- Z_1, Z_2, and Z_3 = thickness of layer
- i_1, i_2, and i_3 = hydraulic gradient for layers
- velocity(v) and discharge(q) will be same in all
- k_1, k_2 and k_3 = coefficient of permeability

DETERMINATION OF PERMEABILITY

Permeability can be determined using the following methods:

Laboratory Method

(a) Constant Head Method:



$$\boxed{k = \frac{VL}{t\ HA}}$$

Here,

V = velocity of fluid flowing through soil

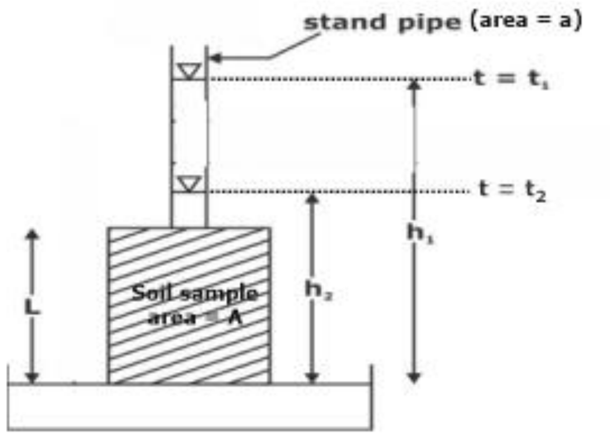
L = length of soil medium

t = time

H = head

A = Area of cross section

(b) Falling Head Method:



$$\boxed{k = 2.303\frac{aL}{At}\log_{10}\frac{h_1}{h_2}}$$

Here, a = area of standpipe

h_1 is height at time t_1

h_2 is height at time $t_2=t_1+t$.

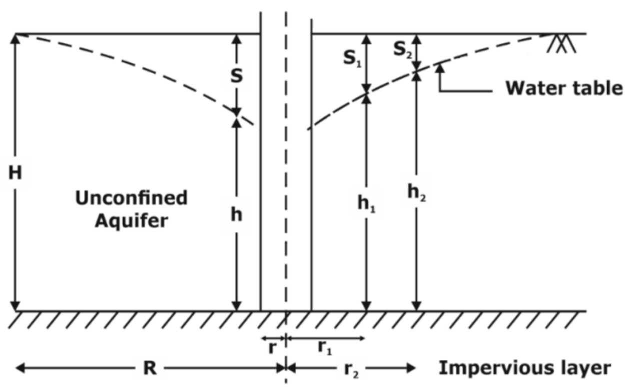
Note: This test is used for relatively less permeable soils where discharge is small. Basically, be suited for fine grained soil.

Note: If water travel from h_1 to h_2 in time t and it travels from h_2 to h_3 in another time t, then

$$\boxed{h_2 = \sqrt{h_1 h_3}}$$

Field Method

Case I: Well in unconfined aquifer



here, $R = 3000s\sqrt{k}$

S = drawdown at the well in (m).

K = coefficient of permeability in 'm/sec'

r = radius of well

(i) As per Dupit's Theory:

$$\boxed{k = \frac{2.303q\log_{10}(R/r)}{\pi\left(H^2 - h^2\right)}}$$

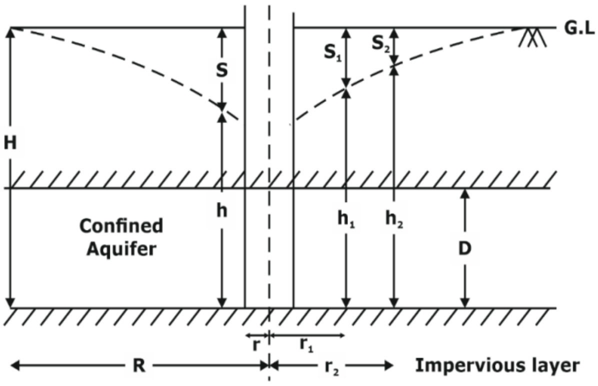
(ii) As per Theim's theory:

$$\boxed{K = \frac{2.3036\,q\log_{10}(r_1/r_2)}{\pi\left(h_1^2 - h_2^2\right)}}$$

q = discharge pumping out

h = depth of water in the well measured above the impermeable layer.H = thickness of aquifer, measured from the impermeable layer to the initial level of water table.

Case II: Well is in confined aquifer



(i) As per Dupit's theory:

$$\boxed{K = \frac{2.303\, q \log_{10}(R/r)}{2\pi D(H-h)}}$$

Here, D = width of confined aquifer.

(ii) As per Theim's theory:

$$\boxed{K = \frac{2.303\, q \log_{10}(r_1/r_2)}{2\pi D(h_1-h_2)}}$$

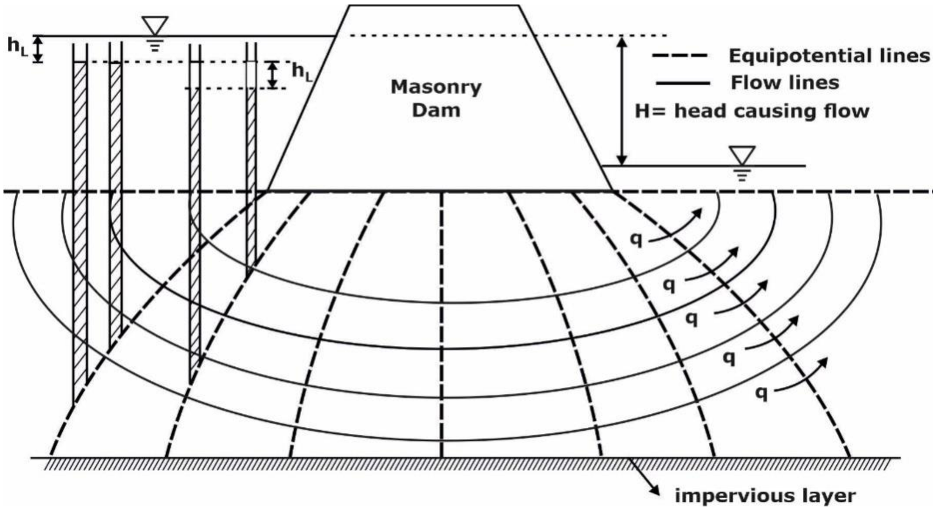
r_1 and r_2 = radial distance of two wells

h_1 and h_2 = depth of water in both wells

CHAPTER-6 SEEPAGE THROUGH SOIL

- Seepage in the process in which liquid leaks through a porous medium from high head towards low head. Seepage problems could be loss of water from reservoir, uplift force below the dam and piping problem etc.
- Hence the need to calculate amount of seepage, uplift pressure and exit gradient. All of there can be calculate using flow net.
- Flow net is graphical representation of path taken by water particles and the head variation along the path.
- The concept of flow net is based on Laplace equation.
- **Laplace equation of 2-D flowAssumptions**
    - Soil is fullysaturated and Darcy's law is valid.
    - Soil mass in homogeneous and isotropic ($K_x = K_y$)
    - Both soil grains and pore fluid are incompressible.
    - Steady state condition exists.

Properties and use of flow net



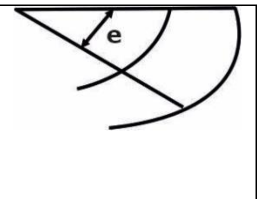
- Flow lines are drawn in such a way that between two successive flows lines discharge in constant.
- Equipotential lines are locus of points having same total head i.e. piezometer inserted at any two point on the same equipotential line will rise to the same level.
- Equipotential lines are drawn in such a way that they have same potential drop between them.
- The loss between two equipotential lines = $\frac{H}{N_d}$. H →(different between upstream and downstream water level.)
- Actual rise in piezometer in the pressure head.
- Each flow field is and elementary square (average distance between two flow line and average distance between two equipotential line is same).
- Flow line and equipotential lines are orthogonal to each other for isotropic soil.
- Space between two adjacent flow line is called flow channel and area between two adjacent flow line and two equipotential line is called flow field.
- N_f = No. of flow channel = No. of flow lines – 1
- N_d = No. of potential drop = No. of equipotential lines – 1

$$q = K\frac{h_L}{b_1} \times a_1 \times 1 = K\frac{h_L}{b_2} \times a_2 \times 1 = K\frac{h_L}{b_3} \times a_3 \times 1$$

$$q = h_L \times K \times 1 \Rightarrow Q = qN_f$$

$$Q = K\frac{N_f}{N_d} \cdot H \qquad \text{Where } \frac{N_f}{N_a} \to \text{Shape factor}$$

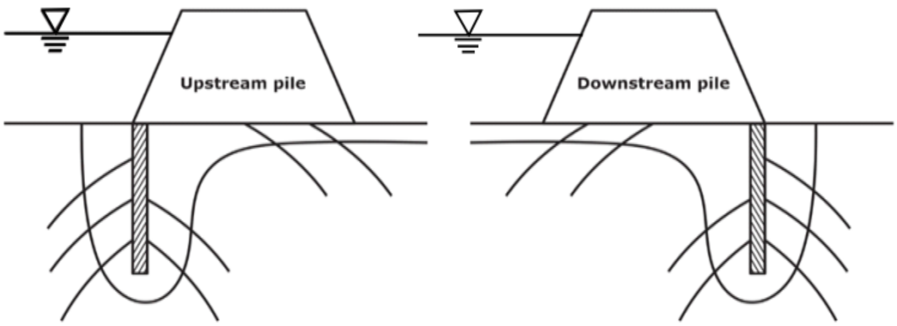
Note: The shape factor is a function of boundary conditions only. It will not change with permeability of soil, head causing flow, direction of flow or number of N_f and N_d changes.It will only change if boundary condition or extent of flow is changed.

• Exit gradient is maximum near the toe because flow field is smaller there • $\frac{h_L}{\ell}$ = Exist gradient and $h_L = \frac{H}{N_D}$	

- If the exit gradient is more piping begins near the downstream toe and lengthens progressively towards the upstream side as the seeping water gradually washes away more and more of soil particle leaving a void or pipe in the soil.Piping can proceed backwards along the base of the dam or along the bedding plane in the soil strata where the resistance is minimum. Dam may collapse into the pipes created and it could be a catastrophic failure (sudden).

• FOS against piping = $\frac{i_c}{i_e}$	• i_c is critical hydraulic gradient • i_e is the exit gradient. • FOS is taken conservatively minimum as 6.

- D/S vertical cut-off leads to larger flow field at exit hence ie falls.
- U/S cut-off piles helps in reducing the uplift pressure force below the dam as more no of equipotential drops could have already occurred by the time water reaches the base of dam thus uplifting pressure reduces.
- Provision of downstream cut-off will reduce exit gradient but will increase the uplift pressure.
- Provision of upstream pile will reduce the uplift pressure, but it will increase the exit gradient.

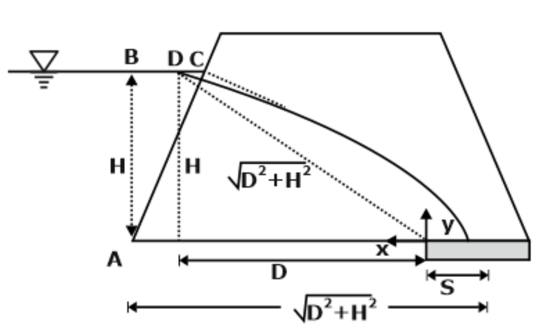


Note: Provision ofcut-off upstream or downstream will change the (n_f/n_d) value and hence the discharge.

Flow net for confined flow: For solution of Laplace equation in 2-D we required four boundary conditions, if four boundary conditionone known then the flow net drawn is called flow net for confined flow. Example: flow under sheet pile, concrete or masonry dam.

Flow net for unconfined flow: In case of earthen dam top flow line is not known in advance thus we have only three boundary conditionsknown hence the flow net drawn for this is called flow net for unconfined flow.

Locating the top flow line for seepage through earthen dam



According to casegrande the flow lines and equipotential lines are 2 sets of parabola with point 'P' as the common focus and top parabola is called base parabola.

DC = 0.3 BC

$$\sqrt{x^2+y^2} = x + s$$

$$\Rightarrow y^2 = (x+s)^2 - x^2$$

$$y^2 = s^2 + 2sx$$

also, $\sqrt{D^2+H^2} = D + S$

then find (S)

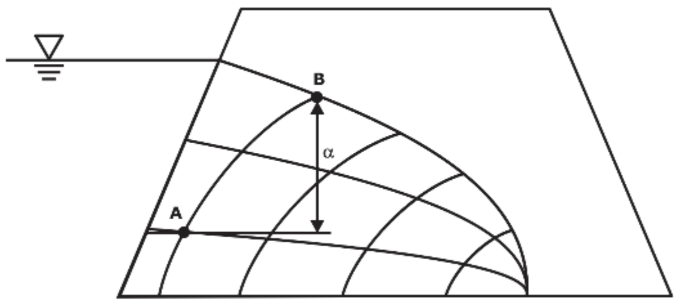
- Once the top flow line is located flow net cam be determined and seepage calculation line $\left(K \cdot H\frac{N_t}{N_D} = q\right)$ can be obtained. Q = ki.A, q = k × 1 × S × 1

  Andq = ks (per unit width)

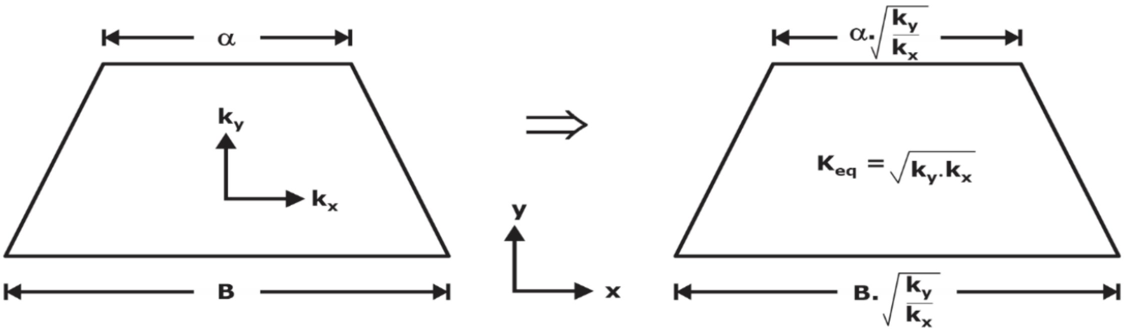
- Note: $\frac{PA}{\gamma} = \alpha$

  because A and B both have equal potential

  (applying Bernoulli) $\frac{PA}{\gamma} + 0 = 0 + \alpha$



Flow through anisotropic soil



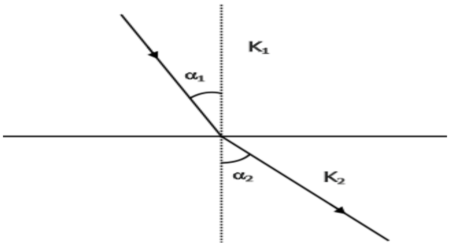
Anisotropic **Isotropic**

- In case of anisotropic conditionwe transform the section into an equivalent isotropic system by reducing the horizontal dimension by a factor of $\left[\sqrt{\frac{k_y}{k_x}} \text{ or } \sqrt{\frac{k_v}{k_H}}\right]$. While keeping the vertical dimension intact. On the transformed section flow net can be drawn and flow lines and equipotential lines can be drawn.
- Transformed section can be used to determine the pore pressure head at any point but to determine the hydraulic gradient one should use the length of original section.

$$i_{Actual} = \frac{h_L \text{ (Between two points)}}{\text{Original distance between those points}}$$

$$L_{AB} = \sqrt{y^2 + \left(x\sqrt{\frac{k_x}{k_y}}\right)^2} \quad \text{in actual case}$$

FLOW THROUGH NON-HOMOGENEOUS SECTION



- $\frac{k_1}{k_2} = \frac{\tan\alpha_1}{\tan\alpha_2}$

PREVENTION OF EROSION

- To prevent the possibility of erosion and piping two approaches are used –
  - Control seepage and seepage force.
  - Used of protective filter.
- Seepage force and erosion can be prevented by provision of cut-off wall or increasing the length of flow path.
- Use of protective filter prevents erosion and helps in safeguarding against piping.
- Filter will prevent migration of particles without significant loss of head in the filter, if the void in the filter are large, particles will be washed out and if the void are small then seepage force may develop in the filter, to achieve this filter must have grain size that satisfies certain requirement.

$\frac{D_{15}\text{ filter}}{D_{85}\text{ protective material}} < 5$	I. This will ensure no significant invasion of particle from the protected material to the filter occurs and thus governs the upper limit of grain size of filter.
$4 < \frac{D_{15}\ (\text{filter})}{D_{15}\ (\text{protected material})} < 20$	II. This will ensure that seepage pressure does not build up in the filter and governs the lower limit of material size.
$\frac{D_{50}\ (\text{filter})}{D_{50}\ (\text{protected layer})} < 25$	III. Additional criteria

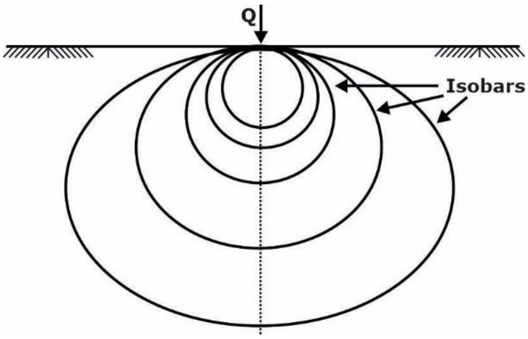
CHAPTER-7 VERTICAL STRESS DISTRIBUTION IN SOIL

BOUSSINESQ'S THEORY:

Assumptions:

- Soil mass is elastic.
- Soil is homogeneous and isotropic.
- Soil is Semi Infinite.
- Soil is weightless and unstressed before the application ofload.
- Distribution of the stress is symmetrical about the vertical axis.

ISOBAR: It is curve or a contour connecting points of equal vertical pressure below the ground surface.



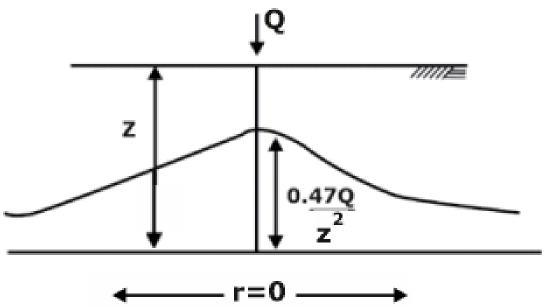
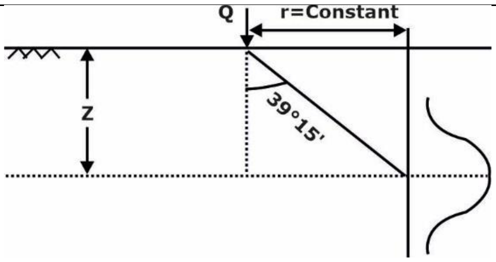
PRESSURE BULB: it is zone of the soil in which there is significant stress distribution and beyond that stress is less.

VERTICAL STRESS DUE TO POINT LOAD

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

- if the point is directly under the load, i.e. r=0, then

$$\sigma_z = 0.4775\left(\frac{Q}{z^2}\right)$$

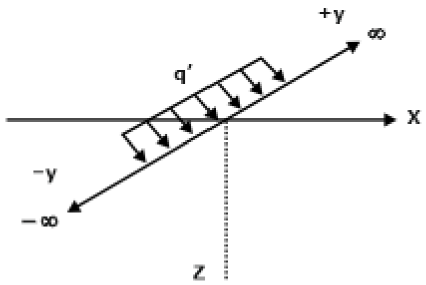
if r is varying and z=constant	if r=constant and z varies
	
maximum value will be at r=0, $\sigma_z = 0.4775\left(\frac{Q}{z^2}\right)$	Maximum value will be at angle 39°15'. $\sigma_z = 0.1332\left(\frac{Q}{z^2}\right)$ *and* $\sigma_z = 0.0888\left(\frac{Q}{r^2}\right)$

EQUIVALENT POINT LOAD METHOD

- In this method the pressure loading over an area is converted to a single point load and applied at centre of the area and then Boussinesue's equation for point load is used.

VERTICAL PRESSURE DUE TO LINE LOAD

- Here type of loading can be boundary wall, railway line etc.

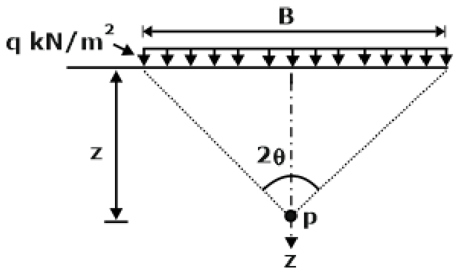


$$\sigma_z = \frac{2q'}{\pi z}\left[\frac{1}{1+\left(\frac{r}{z}\right)^2}\right]^2$$

VERTICAL STRESS DUE TO STRIP LOAD

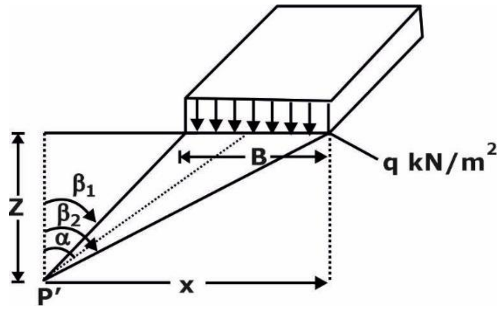
- The expression for vertical stress at any point P under a strip load can be developed from the equation of line load.

Case I. Point P below the centre of the strip.



$$\sigma_z = \frac{q}{\pi}(2\theta + \sin 2\theta)$$

Case II. Point P not below the centre of strip



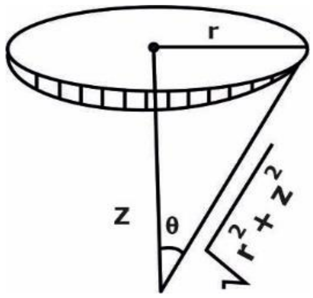
$$\sigma_z = \frac{q}{\pi}\left[\left(\beta_2-\beta_1\right)+\sin\left(\beta_2-\beta_1\right)\cos\left(\beta_2+\beta_1\right)\right]$$

VERTICAL STRESS UNDER A CIRCULAR AREA

- Consider a uniform load of intensity q acting over a circular area of radius R on the surface of a semi-infinite soil mass.
- Point P is directly below the centre of the circle,

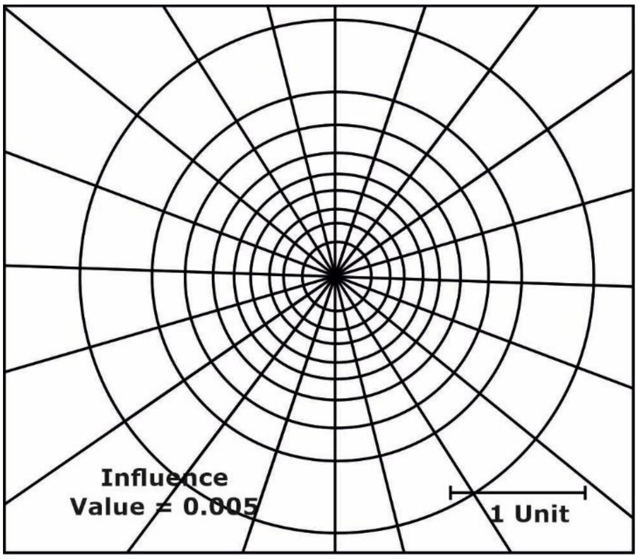
$$\sigma_z = q\left(1-\cos^3\theta\right)$$

$$\cos\theta = \frac{Z}{\sqrt{z^2+R^2}}$$



NEWMARK'S INFLUENCE CHART METHOD

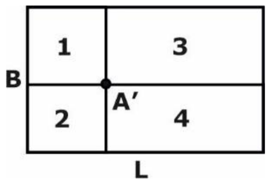
- A chart consisting of the number of circles and radiating lines, is so prepared that the influence of each area unit is the same at the centre of the circles i.e. each area unit causes the equal vertical stress at the centre of the diagram. Let it consist of 'm' number of concentric circles and 'n' number of radial lines.
- Therefore, the number of area units = m × n
- Vertical stress is $\boldsymbol{\sigma_z = qnI}$ and Influence value, $\boldsymbol{I = \frac{1}{mn}}$.
- The plan of the loaded area is traced on a tracing paper and placed over the influence chart such that the point at which the stress is required should coincide with the centre of the chart
- Number of area units occupied by the plane of the loaded area on the influence chart including the fractional areas are counted.
- NOTE: we draw a line of 1 cm on the graph and scale of our loaded area will be 1cm=Hm, here H is depth of the point below loading area.



EQUIVALENT STRESS AT POINT LOCATED BELOW THE CORNER OF RECTANGULAR AREA:

$$\boldsymbol{\sigma_z = qI}$$

- here I is the influence factor based on the parameter m and n, $m = \frac{L}{z}, n = \frac{B}{z}$
- Any point can be placed at corners of various rectangles and then net vertical stress is calculated based on superposition.



Vertical stress at a point within rectangular area

$$= q\left[I_{2(1)} + I_{2(2)} + I_{2(3)} + I_{2(4)}\right]$$

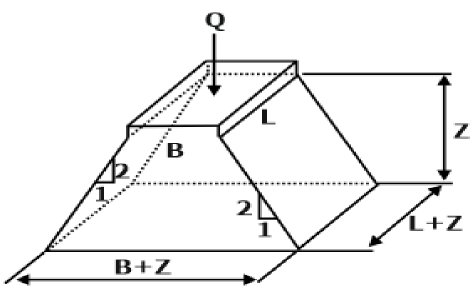
LOAD DISPERSION THEORY (2V:1H)

- if L>B, rectangular loading,

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

- if L>>B, strip loading then,

$$\sigma_z = \frac{qB}{(B+z)}$$



WESTERGAARD'S THEORY

Assumptions:

- soil as a thin sheet layered one over other.
- Poisson's effect is not considered, and lateral strain development is taken zero.
- Hisanalysis is best suited for stratified soil deposits.
- Vertical stress at any given point below the loading is given by

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

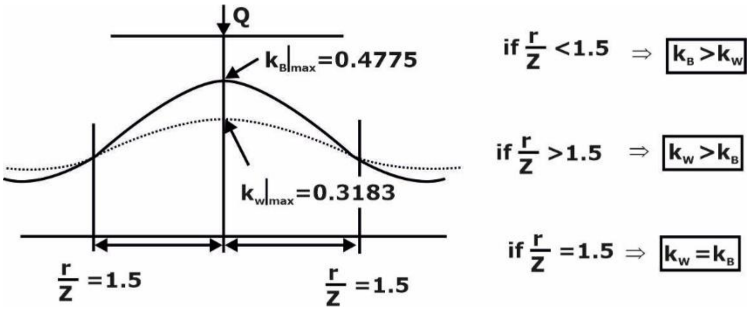
- if r=0, then

$$\sigma_z = 0.318\frac{Q}{z^2}$$

Note: Boussinesq's theory is preferred over westergaard's theory as the results obtained from it are more conservative i.e. safe for the design.

Note: the limiting point at which the stress due to bousnisque and wasterguard theory gives equal result is approximately,

$$\frac{r}{z} = \mathbf{1.5}.$$



CHAPTER-8 COMPRESSIBILITY & CONSOLIDATION

Compaction Vs Consolidations

Compaction	consolidation
It is an instantaneous process.	It is time dependent.
Soil is always partially saturated.	Soil is completely saturated.
Densification is due to reduction in void at a given moisture content.	Volume reduction is due to expulsion of pore water from voids.
Roller, tempers are used.	Static load is applied.

Types Of Settlement

Primary settlement: If the soil is initially partially saturated, expulsion of air as well as compression of pore air may take place with the application of external loads which is called Initial Compression. Itis animmediate phenomenon.After the initial compression, soil reaches into fully saturated state, further reduction in volume occurs due to expulsion of pore water i.e. water present in the soils. Immediate settlement can also occur if significant lateral strain takes place. This is due to deformation of soil under undrained condition. This immediate settlement can be calculated from elastic theory.

Primaryconsolidation: occurs due to expulsion of excess pore water pressure generated due to increase in total stress. It is a time dependent phenomenon.

For Example:When a structure is built over a layer of saturated clay.

When water table is lowered permanently in a structure overlaying a clay layer.

Magnitude of settlement due to 1°-consolidation depends on:

1. Compressibility of soil, 2. Magnitude of stress increase, 3. Thickness of soil layer, 4. Permeability of soil

Secondary consolidation:

Experimentally, it has been shown that compression of soil layer does not cease when excess pore water pressure has been completely dissipated to zero. It continues at a gradually decreasing rate under constant effective stress, 2°-Consolidation is thought to be due to gradual readjustment of clay particles into a more stableconfiguration following the structural disturbance caused by the decrease in void ratio.

Rate of 2-consolidation is thought to be controlled by highly viscous film of absorbed water, surrounding the clay mineral particles in soil. A very slow viscous flow of adsorbed water takes place from the zones of film contact, allowingthe particles to move closer together. Viscosity of film increases as particles move closer, resulting in a decrease in the rate of compressionof the soil.

It is more important for peat, for organic soil. It is unimportant for pre-consolidated clay and stiff clay.

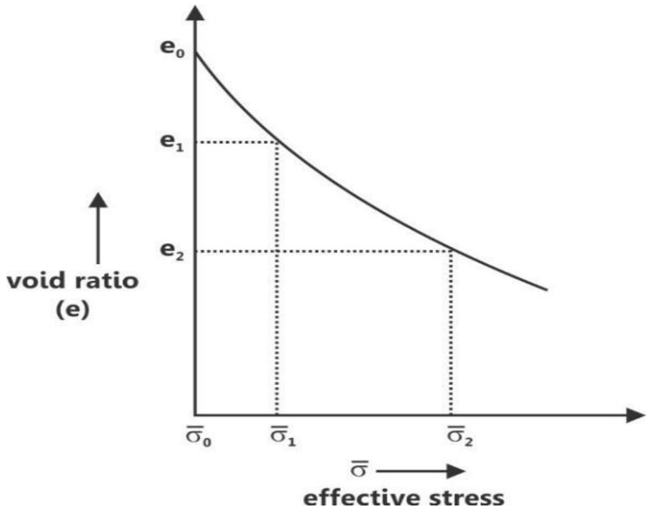
Note: In coarse grained soil, any volume change resulting from change in loading occurs immediately after the loading increases and water pressure dissipates rapidly due to high permeability. This is called drained loading. In fine soils with low permeability, the soil is undrained as load is applied, slow seepage occurs, and excess pore water dissipates leading to consolidation. Hence in coarse grained soil 1° consolidation is = 0.

Normal And Over Consolidated Soil

$$\text{Over consolidation ratio (OCR)} = \frac{\text{maximum applied effective stress in the past}}{\text{present applied effective stress}}$$

- For over consolidated soil,OCR>1
- For normal consolidated soil, OCR=1
- For Under consolidated soil, OCR<1

Effective stress v/s void ratio (NC soil):



$$\boxed{a_v = \frac{-(\Delta e)}{\Delta\bar{\sigma}}}$$

a_v =coefficient of compressibility (m^2/KN)

Δe = change in void ratio

$\Delta\bar{\sigma}$ = change in effective stress.

Coefficient of volume compressibility:

$$\boxed{m_v = \frac{a_v}{1+e}} \qquad \boxed{m_v = -\frac{\left(\frac{\Delta v}{v}\right)}{\Delta\bar{\sigma}}}$$

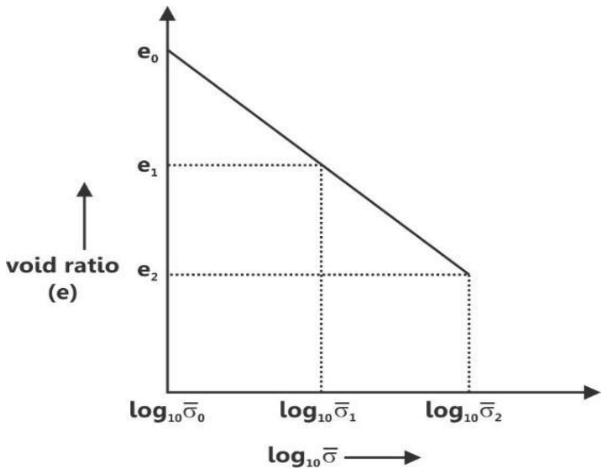
$$\boxed{\Delta H = m_v \times \Delta\bar{\sigma} \times H}$$

In term of void ratio, $\boxed{\Delta H = H_0 \frac{\Delta e}{1+e_0}}$

also, $\boxed{\Delta H = \left(\frac{a_v}{1+e}\right) \times \Delta\bar{\sigma} \times H}$

Note: coefficient of compressibility decreases with increase in effective stress, because with increase in loading soil become more densified hence resistance to further compression increases.

Effective stress v/s void ratio on logarithmic scale:



$$\boxed{C_c = \frac{-(\Delta e)}{\log_{10}\left(\frac{\bar{\sigma}_1}{\bar{\sigma}_0}\right)}}$$

C_c = compression index

Δe = change in void ratio

$\bar{\sigma}_1 = \bar{\sigma}_0 + \Delta\bar{\sigma}$, $\Delta\bar{\sigma}$ = change in effective stress

for undisturbed soil, $C_c = 0.009\,(w_L - 10)$

for remoulded soil, $C_c = 0.007\,(w_L - 10)$

When soil is in normal consolidation region: $\bar{\sigma}_0$ and $\bar{\sigma}_0 + \Delta\bar{\sigma}$ both lies in NCR.

When soil is in over consolidation region:

$\bar{\sigma}_0$ and $\bar{\sigma}_0 + \Delta\bar{\sigma}$ both lies in OCR.

$$S_c = \Delta H = \frac{C_c H_0}{1+e_0}\log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

$$S_c = \Delta H = \frac{C_r H_0}{1+e_0}\log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

When $\bar{\sigma}_0$ **lies in NCR and** $\bar{\sigma}_0 + \Delta\bar{\sigma}$ **lies in OCR,**

then **total settlement,** $\Delta H = \Delta H_1 + \Delta H_2$

$$\Delta H_1 = \frac{C_r H_0}{1+e_0}\log_{10}\left(\frac{\bar{\sigma}_P}{\bar{\sigma}_0}\right)$$

here, e' and H' are void ratio and height after entering the NC region.

$$\Delta H_2 = \frac{C_C H'}{1+e'}\log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_P}\right)$$

$\bar{\sigma}_P$ **= pre-consolidation stress**

Similarly swelling can be: $\Delta H = \frac{C_s H}{1+e_1}\log_{10}\left(\frac{\bar{\sigma}_1}{\bar{\sigma}_2}\right)$ here, e_1 is void ration corresponding to $\bar{\sigma}_1$.

Note: insitu e-Log$\bar{\sigma}$curve is steeper than laboratory e-Log$\bar{\sigma}$ curve. This is because lab curve is obtained for sample collected from field in which unloading has taken place during sampling hence the laboratory curve will be recompression curve leading to flatter slope.

Immediate Settlement

(a) Immediate settlement in sand:	
$\bar{\sigma}_0$ = Initial effective overburden pressure at centre of soil $\Delta\bar{\sigma}$ = decrease in effective stress at the centre of compressibility.	$S_i = \frac{H_0}{C_s}\log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$ And $C_s = 1.5\frac{C_r}{\bar{\sigma}_0}$ C_r = static cone resistance constant (kN/m^2) H_0 = initial thickness of soil
(b) Immediate settlement in clays:	
$S_i = \frac{qB(1-\mu^2)I_t}{E_s}$ E_s = young's modulus of soil I_t = shape factor or influence factor.	B = width of foundation q = uniform pressure at the base of foundation μ = Poisson's ratio of soil (0.3 to 0.5)

2° Consolidation Settlement (S_S)

$$S_S = \frac{C_\alpha H_1}{1+e_1}\log_{10}\left(\frac{t}{t_1}\right)$$

$$C_\alpha = \frac{\Delta e}{\log_{10}\left(\frac{t_2}{t_1}\right)}$$

S_S = 2° settlement after time 't' from completion of 1° consolidation.

t_1 = time required for completion of 1° consolidation.

e_1 = void ratio after 1° consolidation

H_1 = thickness of soil after 1° consolidation

$C\alpha$ = 2° compression index. It is 4-6% of C_c.

One dimensional consolidation equation of Terzaghi: the rate of change of settlement is directly related to the rate of dissipation of pore water pressure. Terzaghi predicted the time rate of consolidation for 1-D consolidation using a mathematical theory, if stress is constant:

$$\boxed{\frac{\partial u}{\partial t} = C_v \cdot \frac{\partial^2 u}{\partial z^2}}$$

C_v = coefficient of consolidation

u = excess pore pressure.

- The coefficient of consolidation can be calculated as

$$C_v = \frac{K}{\gamma_w \cdot m_v}$$

K = coefficient of permeability

γ_w = unit weight of water

m_v = coefficient of volume compressibility

- As per Terzaghi, M_v and C_v both are constant, but in reality C_v decreases with stress increment.

Degree Of Consolidation

The following methods are used in computation of degree of consolidation.

(i) If settlement is given:

$$\boxed{\%U = \frac{\Delta h}{\Delta H} \times 100}$$

Here, Δ h is the settlement of soil after time 't'

And Δ H is the total settlement.

(ii) If void ratio is given:

$$\boxed{\%U = \frac{e_0 - e}{e_0 - e_{100}} \times 100}$$

here, e_0 = void ratio at the centre of soil at the beginning of consolidation.

e_{100} = void ratio at the centre of soil after the completion of consolidation.

e = void ratio after time 't' at the centre of the soil.

(iii) If excess pore pressure is given:

$$\%U = \frac{u_i - u}{u_i} \times 100$$

Let 'u' be the excess pore pressure after the time 't',

Where, u_i = initial excess pore pressure at the beginning of test.

Time Factor

Mathematically, $\boxed{T_v = \frac{C_v t}{H^2}}$

$T_v = \frac{\pi}{4}U^2$, when $U \le 0.6$

$T_v = 0.9332(1 - U) - 0.0851$, when U > 0.6

U = degree of consolidation

Here, T_v = time factor

H = length of drainage path, and C_v = coefficient of consolidation

- for two-way drainage: $H = \frac{H}{2}$
- One way drainage: $H = H$

Note: for a particular degree of consolidation, in a soil T_v and C_v both are fixed. So time required will be directly proportional to H^2. Time required for single drainage is 4 times the time required for double drainage.

Determination of Coefficient of Consolidation

There are two methods to find C_v which are based on time fitting of the curve approach.

(i) Taylor's square root of time fitting method	(ii) Casagrande logarithm of time fitting Method
(i) Taylor's square root of time fitting method: we will find t_{90} from the graph of dial gauge reading and square root t. as we know $T_{(90)}$ is 0.848, we find expression for C_V. $C_v = \frac{T_{90} \cdot H^2}{t_{90}} = \frac{0.848 \cdot H^2}{t_{90}}$ $\boxed{C_v = \frac{0.848H^2}{t_{90}}}$	**(ii) Casagrande logarithm of time fitting Method:** we will find t_{50} from the graph of dial gauge reading and Log t. as we know T_{50} is 0.197, we find expression for C_V. $\boxed{c_v = \frac{0.197H^2}{t_{50}}}$

Note:

- Cv is inversely related to W_L and I_P.
- C_V is inversely related to compressibility of soil.
- In over consolidated soil faster dissipation occurs and in NC soil slow dissipation will occur.
- Organic soils will dissipate slowly. As coefficient of permeability increases then faster will be rate of dissipation.

CHAPTER-9 SHEAR STRENGTH OF SOIL

Shear strength is the capacity to resist shear stress. Generally, soil loaded in compression fails due to shear, not due to crushing of particles. Various strength property of soil like bearing capacity, earth pressure, slope stability depends on shear strength of soil.

Mechanism of shear resistance:

frictional strength:it accounts forgrain to grain contact friction and interlocking between particles.

Cohesive strength:it account for true cohesion between particle and apparent cohesion between particles. True cohesion is due to cementation, electrostatics and electromagnetic attraction, primary valance bonding and adhesion. Apparent cohesion is generated due to capillary stresses.In saturated soil apparent cohesion will be zero.

Mohr Hypothesis: Shear strength on the failure plane at time of failure is a unique function of normal stress on that plane.

COULOMB HYPOTHESIS

$$\tau_f = C + \sigma_f \tan\varphi$$

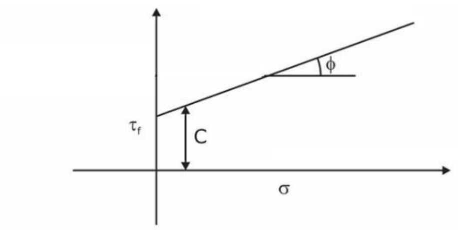
Where,

τ_f = shear strength of soil

C = apparent cohesion of soil

σ = Normal stress on the plane of rupture

φ= Angle of internal friction



Graphical representation of Coulomb equation

$$\tau_f = C' + \bar{\sigma}_f \tan\varphi'$$

Where,$\bar{\sigma}_f = (\sigma - \mu)$ = effective stress

σ_f = total stress

μ= pore water pressure

C′ &φ' are effective stress shear strength parameters.

- Hence,

$$\tau_f = C + \sigma_f \tan\varphi$$

C &φ are total stress parameter

$$\tau_f = C' + \bar{\sigma}_f \tan\varphi'$$

C′ &φ' are effective stress parameter

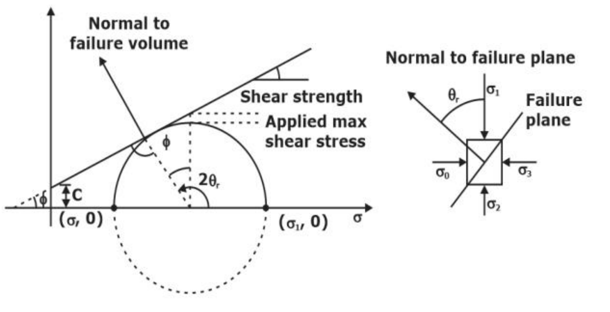
Relation between angle of failure plane (θ_f) and angle of shearing resistance (φ)

σ_1= major principal stress

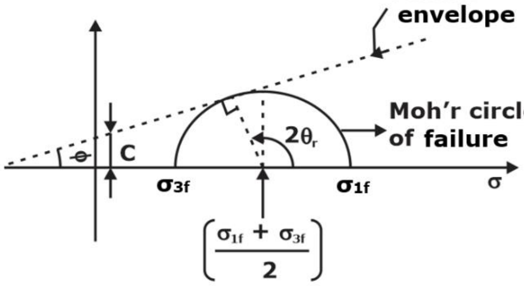
σ_2 = minor principal stress

From the diagram it is clear that, $2\theta_f = 90 + \varphi$.

i.e. the failure plane makes an angle of $\left(45^\circ + \frac{\phi}{2}\right)$ degree with the major principal plane.



RELATION BETWEEN MAJOR & MINOR PRINCIPAL STRESS AT FAILURE IN A SOIL MASS ON THE BASIS OF MOHR COULOMB CRITERIA OF FAILURE

$\sigma_{1f} = \sigma_{3f} \tan^2\left(45+\frac{\phi}{2}\right) + 2C\tan\left(45+\frac{\phi}{2}\right)$ $\sigma_{1f} = \sigma_{3f}\frac{1-\sin\phi}{1+\sin\phi} + 2c\sqrt{\frac{1-\sin\phi}{1+\sin\phi}}$	

Note: From above equation,a straight line can be plotted by taking $(\frac{\sigma_{1f} - \sigma_{3f}}{2})$ on y-axis and $(\frac{\sigma_{1f} + \sigma_{3f}}{2})$ on x-axis, Known as p-q plot.

Maximum Obliquity Relationship:

angle of obliquity in the angle between the resultant stress on a plane and normal stress on the plane.

Failure Plane is the one on which angle of obiquity is maximum for cohesion less soil.

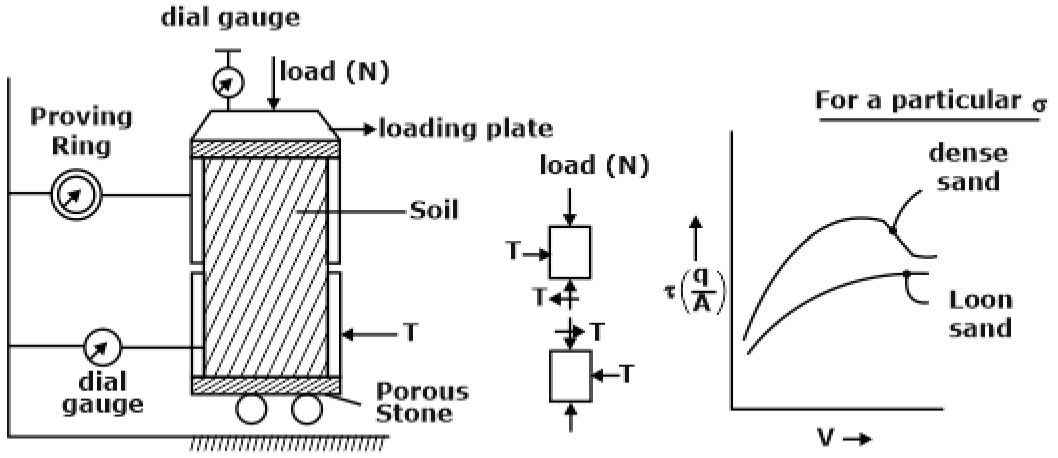
DETERMINATION OF C AND Φ:

- C and φ are not the inherent properties of soil they are related to the type of test and the condition under which the test are performed. The test performed to determine c and φ simulates the field condition:

(i) rapid or slow construction.
(ii) drained or contained condition
(iii) Type of soil.

- The drain and undrained condition depend on the type of the loading and the soil type, and these are relative terms, if the rate of loading is greater the rate of dissipation of excess pore water pressure the condition is undrained, and excess pore pressure will exist. However, if the rate of loading if lessees then the maximum possible rate of excess pore pressure dissipation, we will have drained condition and excess pore pressure will not exist. Only hydrostatic pressure will exist.
- It we can measure the effective stress in the field then to calculate the shear strength parameter we use the effective stress approach and c′ and φ′ are calculated in the laboratory and shear strength in the field is calculated as $\tau = c' + \bar{\sigma}\tan\phi'$. Here, $\bar{\sigma}$ is effective stress in the field.
- If we cannot measure the effective stress in the field then we used total stress approach and find C and φ in the laboratory and shear strength in the field is taken as $\tau = C + \sigma\tan\phi$. Here, σ is total vertical stress in the field.
- It the application of loading leads to positive pore pressure development then critical condition is the immediately after construction condition and undrained analysis is performed. However, if negative pore pressure develops due to load application, critical condition is the long term condition and drained analysis is performed corresponding to long term loading.
- NC clay and loose sand compresses on shearing and hence on shear load application under undrained condition +ve excess pore pressure develops. And in heavily over consolidate clay (OCR>2) and in dense sand under undrained condition there is dilation tendency and hence, negative excess pore pressure develops.

DIRECT SHEAR TEST:(shear box test)

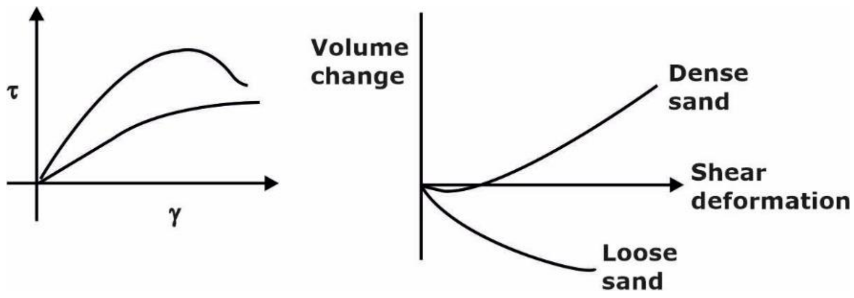


- Normal load in the shear box test simulates the effective vertical stress in the field.
- Shear is normally applied at constant rate of strain. Strain controlled test is better because stress-Strain characteristic can be easily obtained in these tests as the shape of stress strain curve beyond peak point can be observed only in the strain-controlled test and the strain-controlled test is also easy to perform.
- Normal stress $\sigma = \frac{N}{A}$, shear stress $\tau = \frac{T}{A}$, where A is the Normal area of cross section of sample, N → Normal load and T-shear load.
- As drainage cannot be controlled in this test hence rate of loading should be such that the pore water pressure does not develop i.e. we will be performing only drain controlled test. This test is thus good for free draining soil like sand and gravel.
- Direct shear test can be used for performing drain test for clay also. This is because sample is smaller in direct shear test hence drained testing will take lesser time as compared to triaxial test, further it is more convenient to subject a sample of clay to large strains in the direct shear test.
- As specimen fails along a predetermined plane, this soil is useful when soil has a predetermined fault joint.

Disadvantage: Drainage condition cannot be controlled, and pore water pressure cannot be measured.

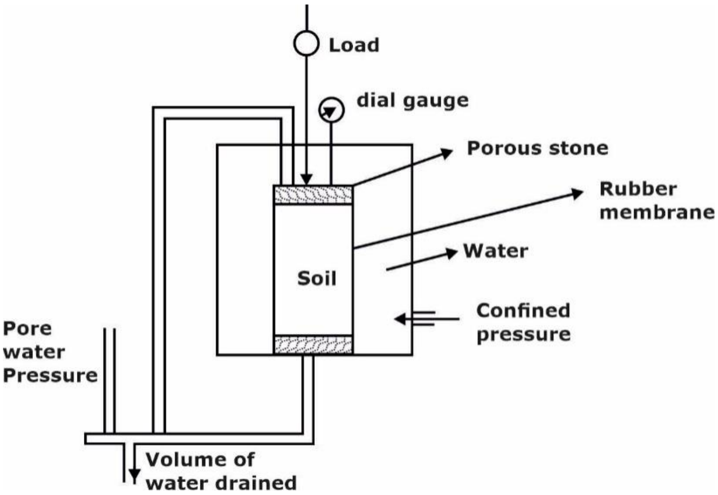
- failure plain is always horizontal which may not be the weakest plane.
- Non-uniform stress distribution on shear plane.
- direction of principal planesare not known at every stage of test it is only after we have determined the φ value that the principal plane will be known.
- Area under normal load does not remain constant during test.

Other Results that can be derived from direct shear test:



TRIAXIAL TEST

- Triaxial test is most widely used shear strength test and is suitable for all type of soil.
- Drainage can be controlled whatever be the type of soil i.e. sand can be tested under undrained condition and clay can be test under drained condition.
- Pore water pressure can be measured and Volume changes can also be measured.
- In triaxial test at any time either one of these can be measured. Both of those are not done together.



- Failure plane is not predetermined.
- At every stage of testing principal plane and principal strains are known.
- Stress distribution on failure plane is fairly uniform.
- Triaxial cell in filled with water and specimen is sealed inside a rubber membrane and cell pressure is applied. It is called confining pressure (σ_c). Confirming pressure is applied to simulate effective vertical stress in the plane.
- With cell pressure held constant additional axial stress is gradually applied. This additional axial stress (σ_d) is called deviator stress and it produces shear in the soil on all plane except the horizonal and vertical plane.
- At the time of failure major principle stress ($\sigma_{1f} = \sigma_d + \sigma_c$) and minor principle stress ($\sigma_{3f} = \sigma_c$) are noted and by plotting deviator stress vs axial strain curve.
- During the application of loading cross sectional area of the specimen will change hence to calculate the deviator stress at the time of failure (σ_{af}) us need to know the cross-sectional area of the specimen of the time of failure.

$$\sigma_{df} = \frac{P}{A}$$

P = deviator load at the time of failure.
A = cross section area at the time of failure.

$$\boxed{A = \frac{A_0(1-\varepsilon_V)}{(1-\varepsilon_a)}}$$

for vol. reduction and height reduction.

$$\boxed{A = \frac{A_0}{1-\varepsilon_a}}$$

For undrained condition $\varepsilon_V = 0$

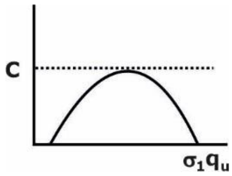
Note:- C &φ values obtained from drained test will be taken as effective stress shear strength parameter. Hence, In the field $\Rightarrow \tau = c + \overline{\sigma}\tan\phi$. and $\overline{\sigma} = \sigma - u_h$

Note: Calculation of shear strength by the formula "$c = c + \overline{\sigma}\tan\phi$" is an approximation because shear strength of the soil should be calculated as shear stress on failure plane at failure, but we are calculating effective vertical stress.

UNCONFINED COMPRESSION STRENGTH TEST

- No confining pressure is applied, it is used to test cohesive soil, load is rapidly applied hence it is undrained test and hence angle of internal friction will not get mobilised. As there is only one confining pressure σ_3 = 0. Only one mohr's circle is obtained.

$$\boxed{c = \frac{q_u}{2}}$$



$$q_u = \frac{p}{A} = \frac{p}{\frac{A_0}{1-C_d}}$$

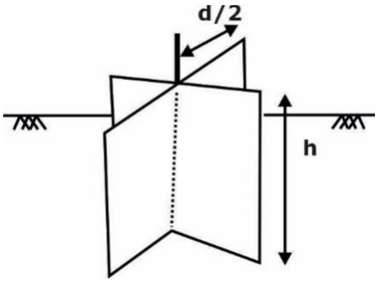
$$\varepsilon_a = \frac{\Delta h}{h}$$

$$\Rightarrow A_0 = \frac{A_0}{1-\varepsilon_a}\left[\varepsilon_v = 0, \begin{array}{c}\text{since it is}\\ \text{undrained conditon}\end{array}\right]$$

- As no lateral pressure i.e. confining pressure exist, sand/cause grained soil cannot stand in the equipment and hence us test is not done for sand and cause grained soil.

VANE SHEAR STRENGTH

- In a highly sensitive cohesive soil obtaining undisturbed specimen is difficult, because shear strength of such soils can be significantly effected during sampling and handling. For such soils vane shear test can be done in the field.



Top end not shearing the soil

$$\boxed{\tau_f = \frac{T}{\frac{\pi d^2}{2}\left[h + \frac{d}{6}\right]}}$$

Both top and bottom shearing the soil.

$$\boxed{\tau_f = \frac{T}{\frac{\pi d^2}{2}\left[h + \frac{d}{3}\right]}}$$

- The above test is also sometimes used to find out the liquid limit in the lab. The moisture at which shear strength is negligible it is called liquid limit moisture content.
- This test can also be used to find the sensitivity of soil, in which after initial failure if the vane is rotated several times the soil becomes simulated and the shear strength under remoulded conditions can be calculated.

$$\text{Sensitivitg} = \frac{q_u \text{ undisturbed}}{q_u \text{ remoulded}} = \frac{2c_u \text{ undisturbed}}{2c_u \text{ remoulded}} = \frac{\tau_f \text{ undisturbed}}{\tau_f \text{ remoulded}}.$$

Types of Triaxial Test:

1. Consolidated drained test (CD Test)→ Takes long time

2. Consolidated undrained test (Cu Test)→ Takes 24 hr. in 1st stage & 2 hr. in 2nd stage.

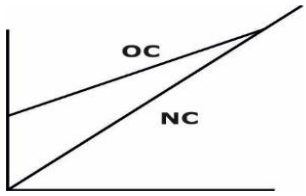
3. Unconsolidated undrained (UU test) →Takes only 15 min**.**

4. Unconsolidated drained test→ (Not a realistic one as it does not occur in field).

CD test

- drainage is allowed in both stages. And loading is slow, to allow water to expel out and therefore pore pressure does not build up.
- Progressively bigger mohr's circles are formed because initial void ratio will progressively decrease as more and more higher confining pressure is applied and Hence greater confining Load has become effective progressively. Thus deviator stress required to cause failure will go on increasing as the confining pressure is being increased.

- for N-C clays: C=0, failure envelope passing through origin.
- for O−C clays: C≠0
- we get effective stress parameter.



Use of CD test: for all cases involving coarse sand and gravel. Except when very rapid load like Bomb blast or earth quack vibration are applied.long term loading of any soil for ex:earthen dam with steady seepage, foundation on clay long time after excavation, cut slopes serval years after excavation and embankment constructed very slowly.

CU test

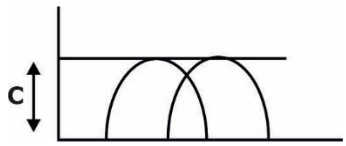
- In the first stage, drainage is permitted & in the second stage drainage is not permitted. I.e. volume change not allowed. We can measure total stress parameter and effective stress parameter. Because pore pressure can be measured at failure.
- Effective stress failure envelope will generally pass through the origin i.e. c' = 0. [not necessarily c = o]

Use of CU: Most of the cases involving short term strength. For example:- Building, Embankment, Earth dam during rapid draw down, Sudden unloading condition.

UU test:

- Drainage is not permitted in the 1st and 2nd stage. After application of confining pressure excess pore pressure is developed which is not allowed to dissipate and during deviator stress application also pore pressure develop which is not allowed to dissipate. Since load does not become effective in any stage, hence function is not mobilised, I.e.$\varphi = 0$

Circles of same dia.
⇓
Failure occurat same deviator stress.

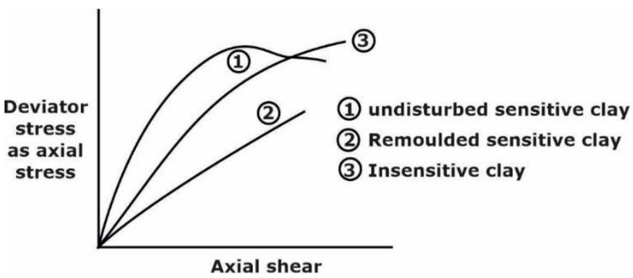


Uses of UU: Sudden loading condition such as rapid construction. rapid construction of earthen dam, rapid construction of building, Strength of soil in excavation, immediately after cut is made.

Note: Some time it is preformed **UCS** instead of **UU** test.

STRESS STRAIN AND VOLUME CHANGE RELATIONSHIP FOR CLAYEY SOIL

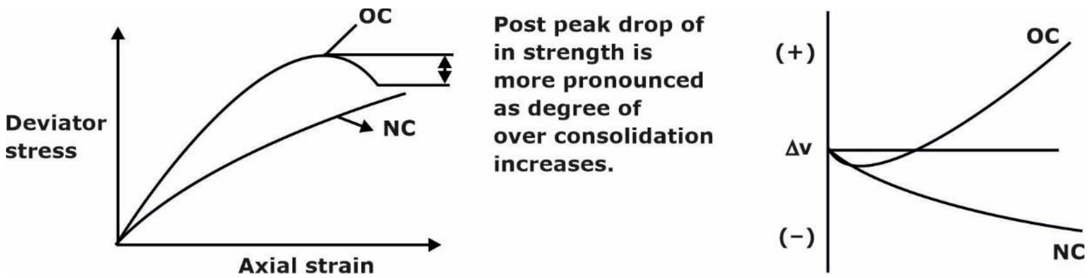
Curve 1 [undisturbed sensitive clay]
- Sharp peak at low strain.
- Specimen shear along a well-defined plane
- Failure is called brittle failure.
- Failure is considered corresponding to peak condition.



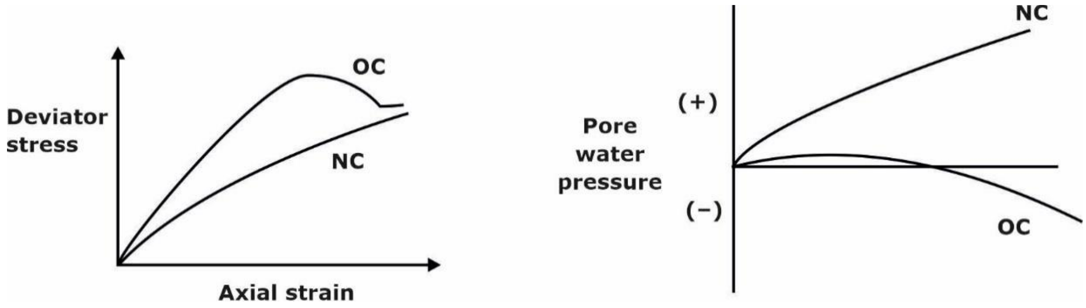
Curve 2 [remolded sensitive clay]
- Plastic failure
- Result in building of specimen
- Failure is decided on the basic of some pre determined strain.

Curve 3 [Insensitive clay]
- Failure can be taken corresponding to horizontal portion of curve

Result of C-D test



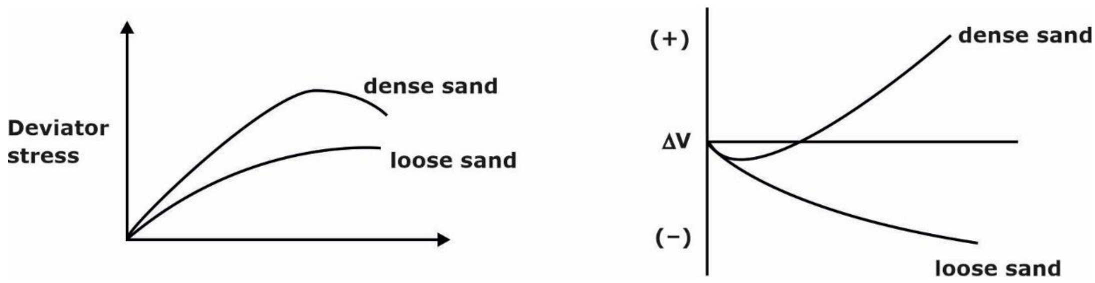
Result of C-U test



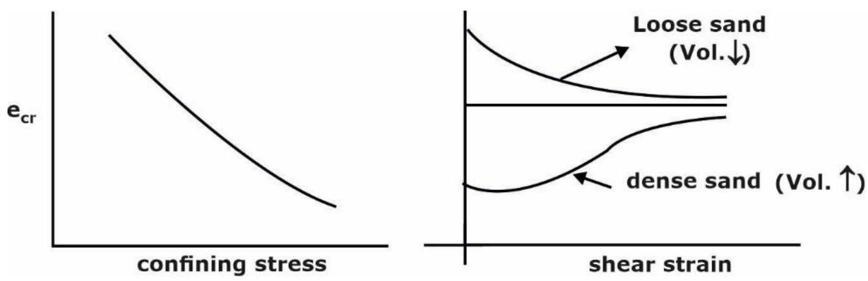
- N-C soil has volume decrease tendency on shearing but O-C soil has volume in case tendency on shearing. Hence, under undrained condition of volume changes are not allowed.
- +ve pore pressure develops in case of N-C soil.
- -ve pore pressure develops in case of O-C soil.

STRESS STAIN & VOLUME CHANGE BEHAVIORS OF SAND

Result of C-D test.



Note: Peak value of deviator stress is generally called compressive strength of soil.



- It e = e_{cr}. There is no tendency of volume change and hence drained and undrained strength will be some.

- e >e_{cr}, there is volume decrease tendency and hence +ve pore pressure develops under undrained condition and hence effective stress gets reduced hence undrained strength will be smaller than the drained strength.
- if e > er, there is volume increase tendency on shearing and hence negative pore pressure develops under undrained condition hence effective stress increases and undrained strength will be greater than the drained strength.

SOIL LIQUEFACTION

- Loose sand will have +ve pore pressure developed under undrained condition and hence $\bar{\sigma}$ decreases under undrained condition. If $\bar{\sigma}$ reduces to '0', sand will lose all of its shearing strength this phenomenon is called liquefaction.
- It occurs in case of saturated loose sand during pile driving, machine vibration, EQ shock & explosive blasting.
- It occurs under high frequency of vibration (pore pressure increases progressively). There is a cumulative increase in pore water pressure under successive cycle of loading.

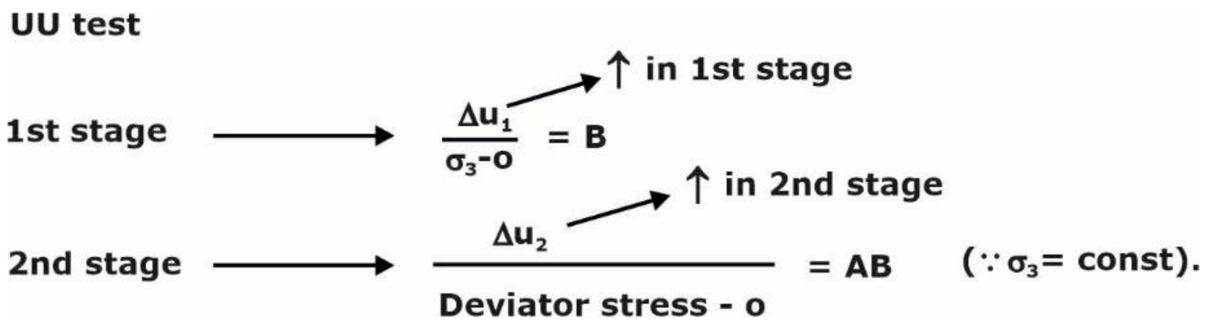
Note: Strength degradation of clay due to cyclic loading flows similar pattern to that of sand but degradation of strength is less than for cohesionless or slightly cohesive soil.

Note: strength decreases in clay with increase in plasticity index (I_P). So under consolidated soil is prone to greater loss of strength as compared to N-C or O-C clay due to cyclic loading.[as under consolidated clay has pore pressure already present]

PORE PRESSURE COEFFICIENT

$\Delta u = B\left[\Delta\sigma_3 + A\left(p\sigma_1 - \sigma r_3\right)\right]$ $\boxed{B = \frac{\Delta u_1}{\Delta\sigma_3}}$ $\boxed{AB = \frac{\Delta U_2}{\Delta\sigma_L - \Delta\sigma_3}}$	• A & B and skemptan's pore pressure coefficients • B = 1 for completely submerged soil • B = 0 for completely dry soil. • For pointily submerged soil, (B is very small) • Even at soil situation, B is approximately = 0. • The value of B varies with the stress range.

value of A & B can be calculated by UU test and if B is known we can also use CU test to find out the value of A.



CHAPTER-10 EARTH PRESSURE AND RETAINING WALLS

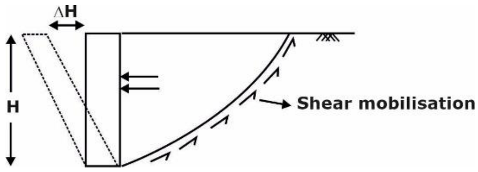
- In fluids the hydrostatic pressure acts equally in all direction at any point given. i.e.$\sigma_x = \sigma_y = \sigma_z$. The horizontal and vertical pressure are same because water does not have any shear strength.

- In case of soil the vertical and the Horizontal stresses are not equal it is because the soil has shear strength due to either friction, cohesion, or both.Hence, $\sigma_x \neq \sigma_y$(exceptions can be there based on different theory)
- Earth Pressure can be defined on the basic of three types. Terzaghi has performed various test based on which he defined these above earth pressure

(i) **Rest Pressure:** If the walls of a rigid structure are unyielding.i.e. there is no movement of the wall then the soil is said to be in state of rest.

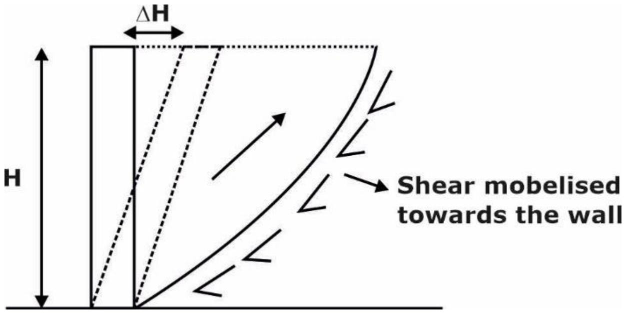
(ii) **Active condition**: If the soil exerts the force on the wall (wall moves away from the soil) due to which it is called in the state of active pressure.

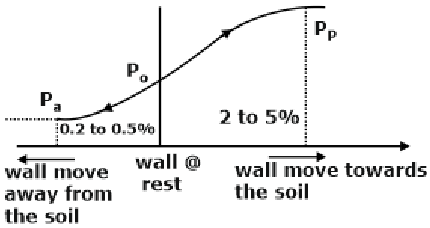
- Due to the wall movement the soil tries to expend and shear planes are developed with the mobilisation of shear away from the wall.
- The soil mass which is falling tries to move down wards and outward



(iii) **Passive earth pressure**: When the wall is pushed towards the back fill the soil is getting compacted and the shear resistance mobilised is towards the wall.

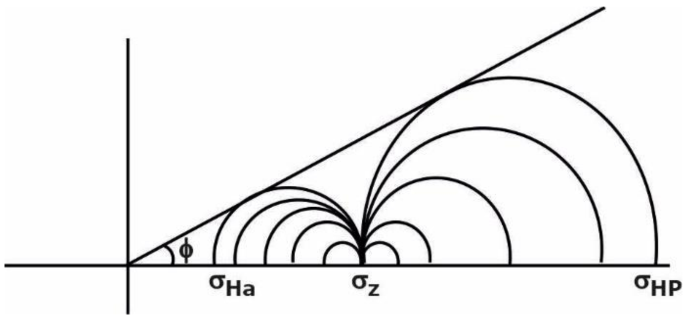
- The soil tries to move toward and inward and the pressure exceeds the rest pressure.





- The movement of top of retaining wall to generally active earth pressure condition, for dense soil is 0.2% and for loose condition is 0.5%
- The yielding of the wall is given by $\frac{\Delta H}{H}$ ratio.
- To generate a passive condition for dense soil the yield shall be 2% and loose soil it should be by 5%.

Mohr circle:



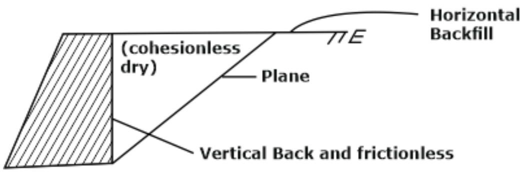
Rest Pressure: $\epsilon_x = \frac{\sigma_x}{E} - \mu\left(\frac{\sigma_x}{E} + \frac{\sigma_y}{E}\right)$ and $\epsilon_x = 0, \sigma_x = \sigma_y$

$$\sigma_H = \sigma_V\left(\frac{\mu}{1-\mu}\right) \Rightarrow \boxed{K_0 = \frac{\mu}{1-\mu}}$$

- It the soil is considered elastic, homogeneous, isotopic semi-infinite-then the value of earth pressure coefficient based on elastic theory is given by $\boxed{K_0 = \frac{\mu}{1-\mu}}$
- μ is considered as a constant but its actual value changes depending on drainage condition.
- Based on field measurement, earth pressure given by different scientist are as follows.
    - (i) ALPAN ⇒ NCC → $K_o = 0.19 + 0.233 \log(I_p)$

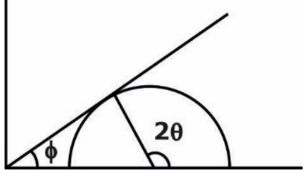
      OCC → $K_o = K_{o(NCC)} \times \sqrt{OCR}$
    - (ii) Jaky ⇒ $K_o = 1 - \sin\varphi$ [for sandy type of soil.]

Rankine theory



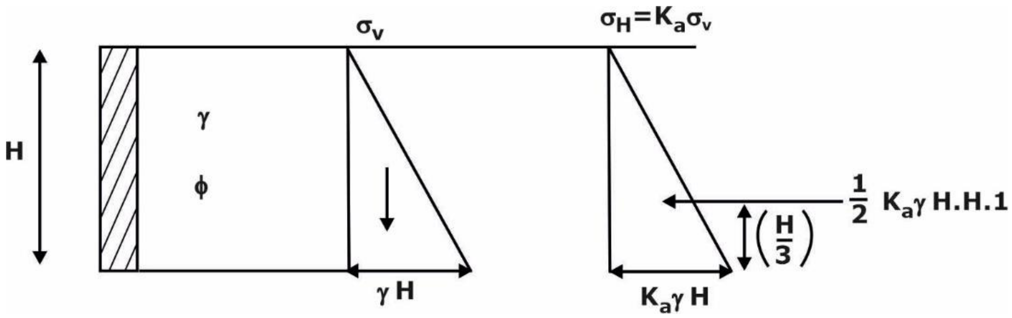
- It is considered that back of retaining wall is vertical and smooth and backfill is horizontal.
- It considers the stress in a state of plastic equilibrium.
- The soil is homogeneous, dry, cohesion (later this theory was modified to consider effect of water table and cohesion).
- Rupture surface is a plane surface.

Active Earth Pressure

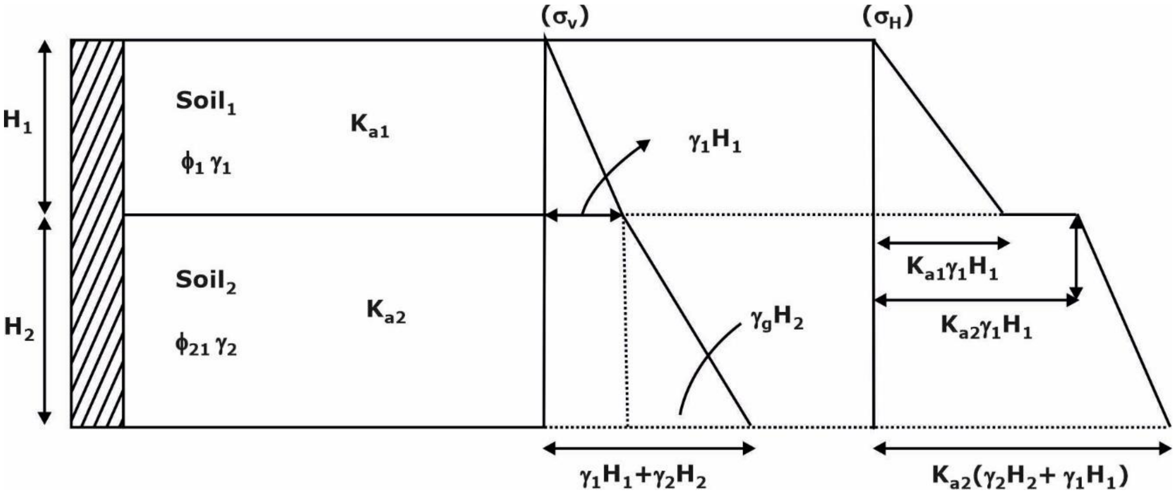
$\sigma_1 = \sigma_3 \tan^2\left(45 + \frac{\phi}{2}\right) + 2c\tan\left(45 + \frac{\phi}{2}\right)$ $\Rightarrow c = 0 \Rightarrow \sigma_V = \sigma_H\left(\frac{1+\sin\phi}{1-\sin\phi}\right)$ $\frac{\sigma_H}{\sigma_V} = ka = \frac{1-\sin\phi}{1+\sin\phi}$		• $2\theta = \varphi + 90$ angle between normal to failure plane and normal to major principal plane. • $\theta = 45 + \frac{\phi}{2}$

Analysis of various conditions:

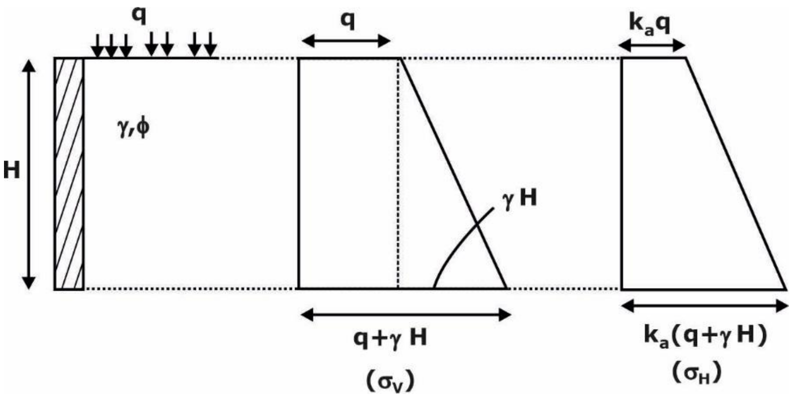
1. single soil mass



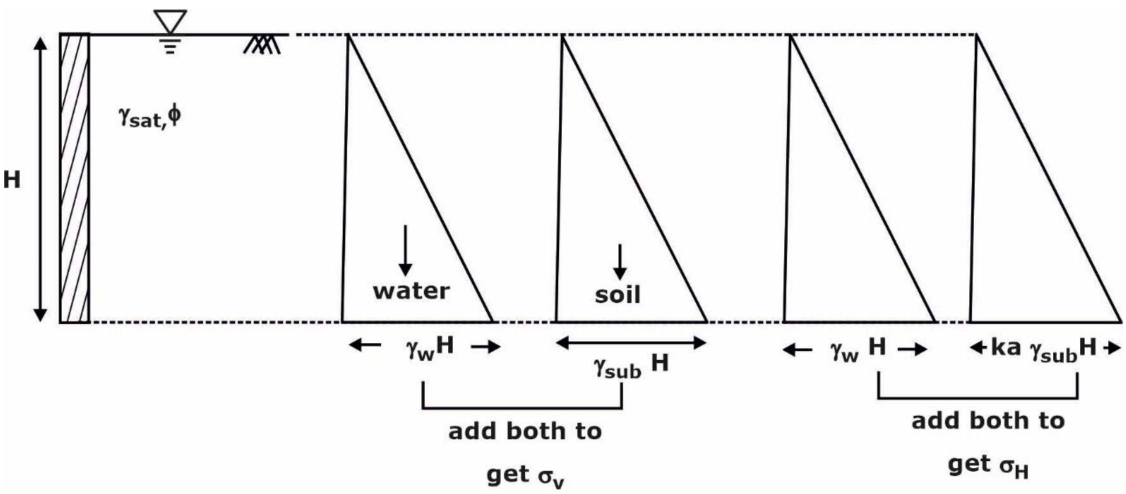
2. stratified soil mass



3. external loading is acting



4. Water table rise upto ground



In short, $\boxed{\text{add [water pressure} + K_a \text{ (effective vertical stress)]}}$

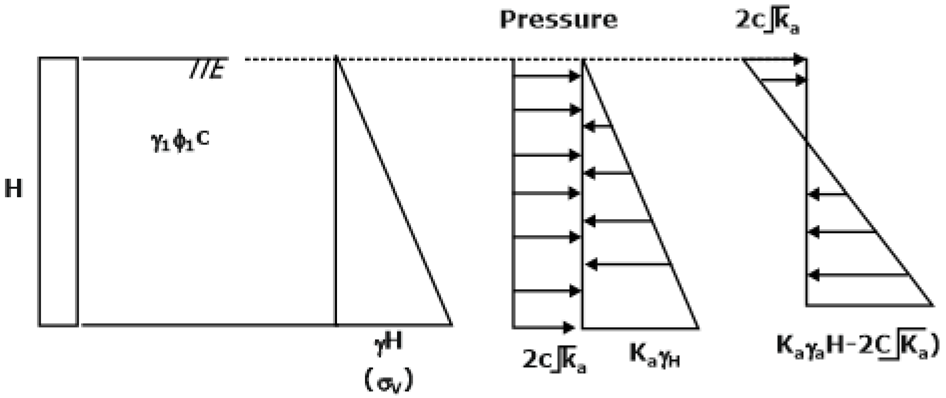
5. C - φ soil(No water table)

Plastic equilibrium eq^n ⇒ $\sigma_1 = \sigma_3 \tan^2\left(45^\circ + \frac{\phi}{2}\right) + 2C.\tan\left(45^\circ + \frac{\phi}{2}\right)$

$$\sigma_V = \sigma_H\left[\frac{1+\sin\phi}{1-\sin\phi}\right] + 2c\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}$$

$$\boxed{\sigma_H = \sigma_V \times k_a - 2c\sqrt{k_a}}$$

Note: Effect of cohesion is to reduces active earth pressure

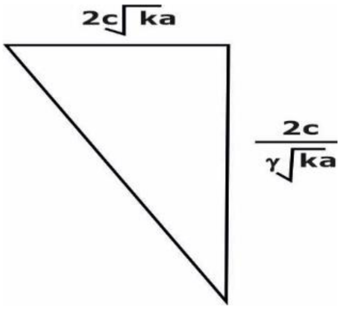


Total active thrust $\Rightarrow \boxed{\left(-2c\sqrt{k_a}H\right) + \frac{K_a\gamma H^2}{2} = F_R}$

For $\sigma_H = 0 \Rightarrow K_a(\gamma z_0 + q) - 2c\sqrt{k_a} = 0$

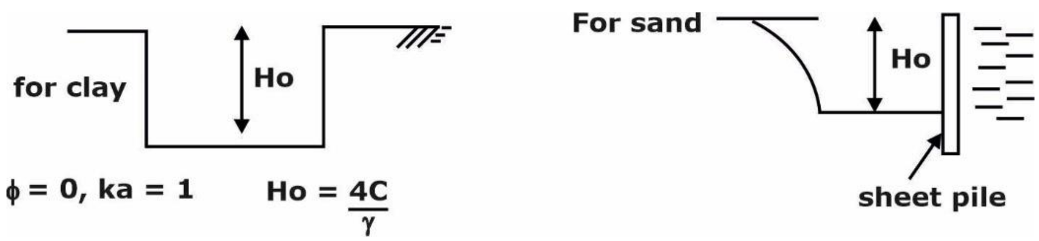
$$\Rightarrow \boxed{Z_0 = \frac{2c}{\gamma\sqrt{k_a}}}$$

It tension crack has not occurred, then force will decrease by



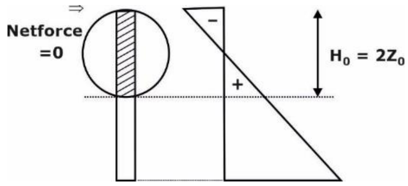
$$F = \frac{1}{2}2c\sqrt{ka} \times \frac{2c}{\gamma\sqrt{ka}} = \frac{2c^2}{\gamma}$$

Box cutting:

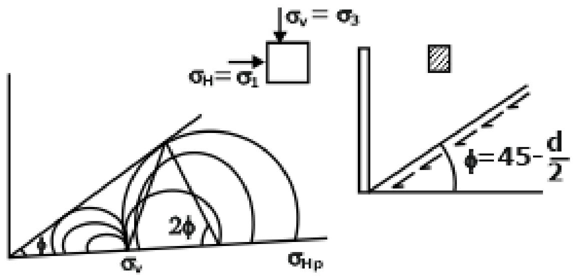


- For design purpose cracks are assumed to be occurred.Because In practice tension cannot be taken on the wall, since tension cracks tends to develop within the soil and it may not remain adhered to the wall.

- So, in calculation of total thrust usually tension zone is ignored which results in a larges value of the active earth pressure thrust. As a theoretical concept no 'tension crack condition will result in a net active thrust value for a region upto a depth Ho equal to zero.
- $\boxed{H_0 = \frac{4c}{\gamma\sqrt{Ra}}}$ Critical depth



Passive Pressure:



$$\text{If } c = 0 \Rightarrow \sigma_H = \sigma_V \cdot \tan^2\left(45^\circ + \frac{\phi}{2}\right)$$

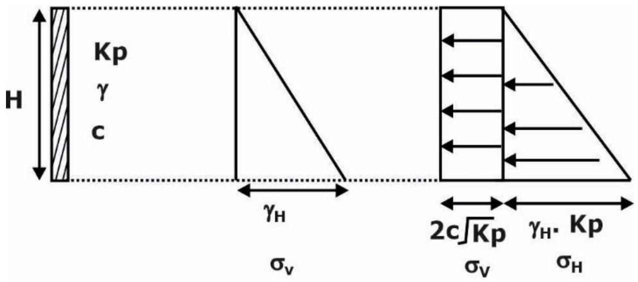
$$\Rightarrow K_p = \tan^2\left(45^\circ + \frac{\phi}{2}\right) = \frac{1+\sin\phi}{1-\sin\phi}$$

$$\boxed{Kp = \frac{1+\sin\phi}{1-\sin\phi}} \quad \boxed{KP = \frac{1}{k_a}}$$

c - φ soil:

$$\sigma_H = \sigma_{V.}K_p + 2c\sqrt{k_p}$$

Note: The effect of cohesion is to increase the passive earth pressure.



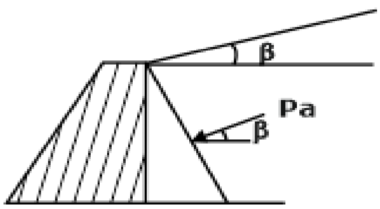
$$\boxed{F_p = \frac{1}{2}k_p\gamma H^2 + 2c\sqrt{k_p}H}$$

Inclined Backfill

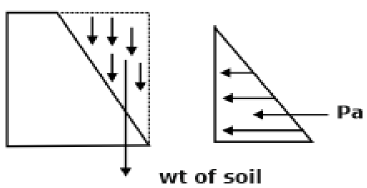
$$P_a = \frac{1}{2}K_a\gamma H^2 \cos\beta$$

$$\boxed{ka = \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\text{con}\beta + \sqrt{\cos^2\beta - \cos^2\phi}}.\cos\beta}$$

Resultant force will act at a distance $\frac{H}{3}$ from bottom and will act parallel to the backfill inclination.



Inclined Back of wall

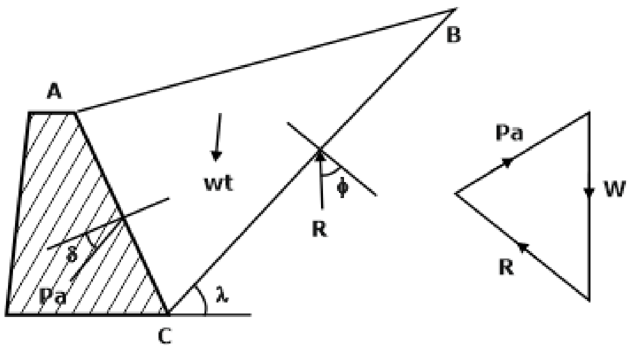


Columb's theory: assumptions: -

1. The back fill is assumed dry, cohesionless, homogenesis and isotropic.
2. The back of the wall and the backfill can be inclined.
3. The friction between the wall as the soil is considered.
4. Actual failure plane is curved in bottom region but assumed plane for analysis.
5. The failure plane is assumed to pass through heel
6. The sliding wedge is considered as a rigid body and λ the angle of failure plane with the horizontal.
7. At the failure condition the trial wedge is in equilibrium under the action of three forces that are (W,R,P_a).

Pa – path pressure forace which is inclined at an angle s with the normal of the back of the wall	w → self-weight.	R- Reaction force at failure plane which is inclined at an angle & from the normal to the failure plane.

8. Value of δ is assumed as follows.
   - For smooth wall, $\delta = \frac{\phi}{3}$
   - For ordinary retaining wall made of concrete, $\delta = \frac{2\phi}{3}$
   - For retaining wall where provision of drainage of soil is provided through deep holes. $\delta = \frac{3}{4}\phi$
   - If soil is subjected to vibration δ = 0.



9. The direction of all the forces is known and value of w is also known hence using this data a force triangle can be drawn and the value of P_a can be found out.
10. For active earth pressure force the highest among all the trial planes is considered as the critical value and for the passive earth pressure force the minimum of all the trial planes is considered as the critical.

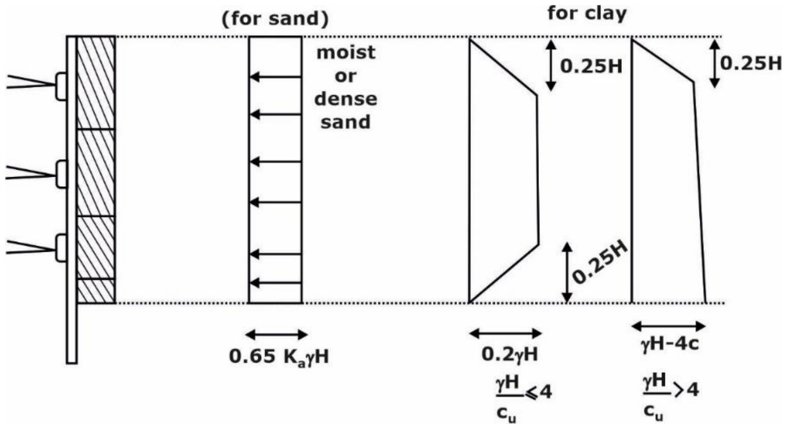
Note: Rankine earth Pressure theoryOverestimates the active earth pressure and underestimates the passive earth pressure. Retaining walls are designed for active earth pressure.

Note: Movement of water table upwards will increase the active earth pressure and will decrease the pressive earth pressure.

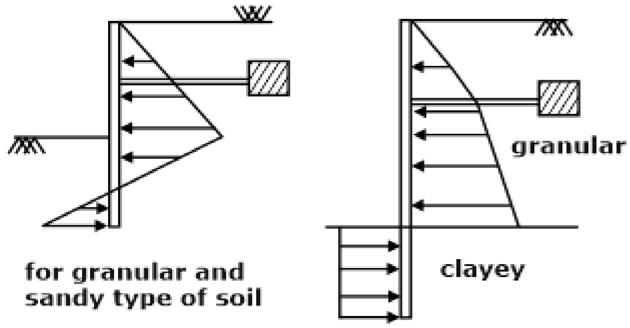
Note: On compacting the soil.Active earth pressure decreasesand Passive earth pressure increases.

Braced System:

- In a Braced extraction the theory proposed by peck Hanson and Thorburn suggests that the earth pressure distribution on the backfill side of the sheet pile is represented by an apparent earth pressure diagram.
- An imaginary strut is assumed at the dredge line for the analysis
- Load carried by each is based on the tributary area which town up to half of the distance between the strut above and below the street level.

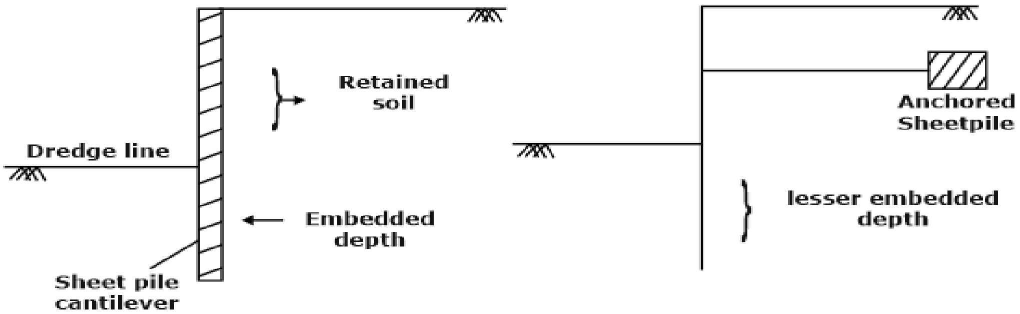


Anchored Bulb head ⇒

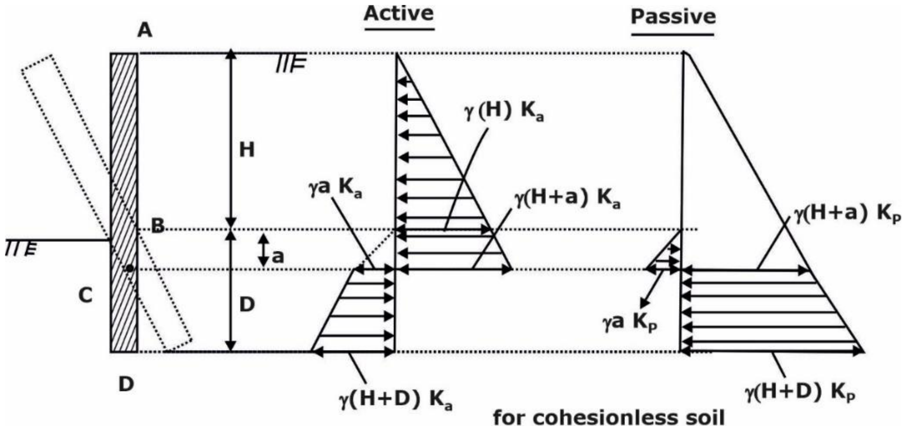


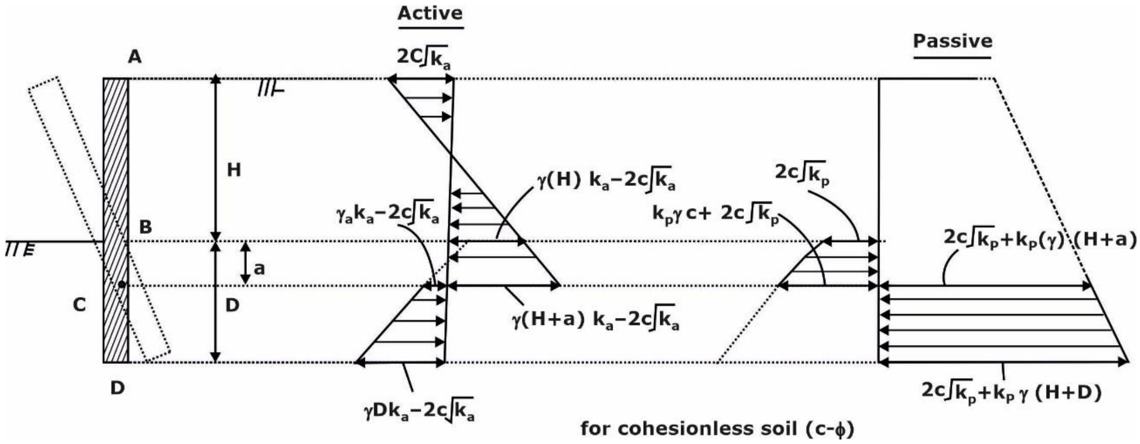
Note: In anchored bulb head the moment equilibrium is considered about the anchor level and resultant earth pressure diagram is as follows.

Sheet Piles: Sheet piles are also known as flexible retaining wall that are used to support the soil temporary after the excavation. They are driven into the soil and act as continuous wall which is retaining earth mass. These are generally made of steel which has a high strength, it is having very light weight and is thin.

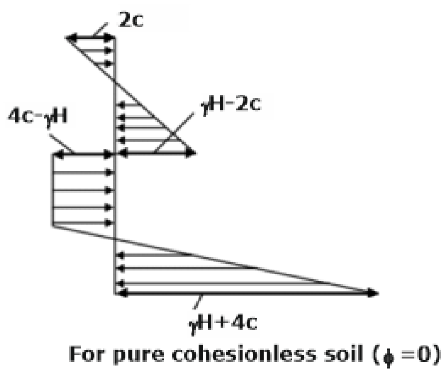


Cantilever pile

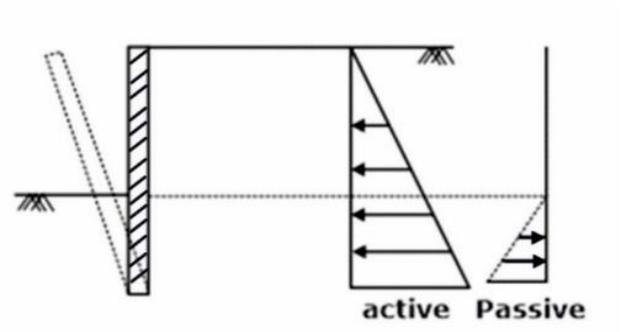




- The resultant diagram is based on assuming that there will be some consolidation over a period of time which will take place, also pile is of flexible nature, Hence it will get deformed.
- Due to overturning there will be shrinkage, swelling loss of contact with the pimple or development of tuition cracks.



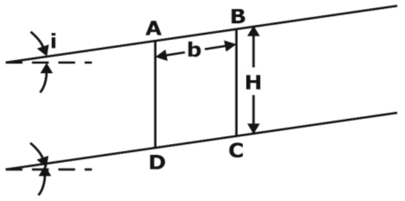
Approximate diagram for analysis:(most simplified diagram)



CHAPTER-11 STABILITY OF SLOPES

STABILITY OF INFINITE SLOPES

- Let's consider an infinite slope AB having a slope angle of 'i' with horizontal, failure of which takes place along the plane 'CD' that is at a depth of H below the surface.

	• Area of soil Wedge = $H.b.cosi$ • Volume of soil Wedge = $(H.b.cosi) \times 1$ • Weight of soil Wedge = $\gamma.H.b.cosi$

Vertical stress on plane $CD = \sigma_z = \dfrac{\gamma \cdot H \cdot b \cdot \cos i}{b \times 1}$	$\Rightarrow \sigma_z = \gamma.H.cosi$
Normal stress on plane CD, $\sigma_n = \sigma_z.cosi$	$\Rightarrow \sigma_n = \gamma.H.\cos^2 i$
Tangential stress on plane CD, $\tau = \sigma_z.Sini$	$\Rightarrow \tau = \gamma.H.cosi.sini$

$$F.O.S. = \frac{s}{\tau} = \frac{c + \overline{\sigma_n} \tan\phi}{\tau}$$

Case 1: Cohesionless soil (c = 0)

$$\Rightarrow FOS = \frac{0 + \sigma_n \tan\phi}{\tau} = \frac{\sigma_n = \gamma \cdot H \cdot \cos^2 i \cdot \tan\phi}{\gamma \cdot H \cdot \cos i \cdot \sin i}$$

$$\Rightarrow FOS = \frac{\tan\phi}{\tan i}$$

Case 2: Cohesionless submerged soil mass

$$FOS = \frac{\gamma'.H.\cos^2 i\ .tan\ \phi}{\gamma'.H.cosi.sini} = \frac{tan\ \phi}{tani}$$

Case 3: Slope subjected to steady seepage and water table is up to depth of 'h' above the failure plane**.**

$$FOS = \left(1 - \frac{\gamma_w h}{\gamma_{eff} H}\right)\frac{tan\ \phi}{tani}, \text{ here } \gamma_{eff} = \frac{\gamma.(H-h)+\gamma_{sub}.h}{H}$$

$$If\ \ h = H, FOS = \frac{\gamma'}{\gamma_{eff}} \times \frac{tan\ \phi}{tani} = \frac{1}{2}\frac{tan\ \phi}{tani}$$

Case 4: Cohesive soil having dry or moist slope

$$FOS = \frac{C + \gamma.H.Cos^2 i.tan\ \phi}{\gamma.H.cosi.sini}$$

- If slope angle 'i' is less than frictional angle 'ϕ' no failure takes place as for any value of normal stress shear stress is less than shear strength of the soil.

For $H = H_c, FOS = 1$

$$1 = \frac{C + \gamma . H . Cos^2 i . \tan\phi}{\gamma . H . cos i . sin i}$$

$$\gamma . H . \cos i . \sin i = C + \gamma . H . Cos^2 i . \tan\phi$$

Here, S_n= Taylor's stability number

$$S_n = \frac{c}{\gamma . H_c} = cos i . sin i - Cos^2 i . \tan\phi$$

FOS with respect to height:

Here,$H_c = critical\ height$

$$(F.O.S.)_h = \frac{H_c}{H}$$

FOS with respect to cohesion:

C = effective cohesion

C_m =mobilized cohesion

$$(F.O.S.)_c = \frac{c_m}{c}$$

In terms of cohesion,

$$\boldsymbol{S_n = \frac{c}{\gamma H_c} = \frac{c}{\gamma \times FOS \times H}}$$

Case 5: Cohesive soil in submerged condition

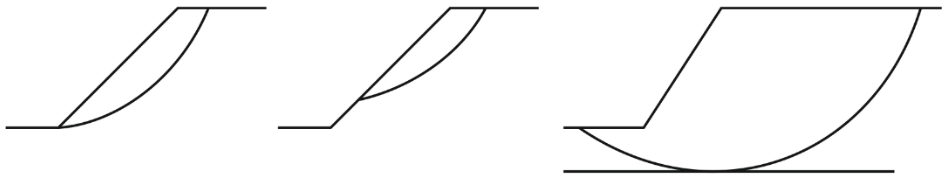
$$FOS = \frac{C + (\gamma' H) . Cos^2 i . \tan\phi}{\gamma' . H . cos i . sin i}$$

Case 6: Slope subjected to steady seepage and water table is up to depth of 'h' above the failure plane**.**

$$FOS = \frac{C + (\gamma_{eff} H - \gamma_w h) . Cos^2 i . \tan\phi}{\gamma_{eff} . H . cos i . sin i}$$

STABILITY OF FINITE SLOPES: Failure of finite slopes has following three modes

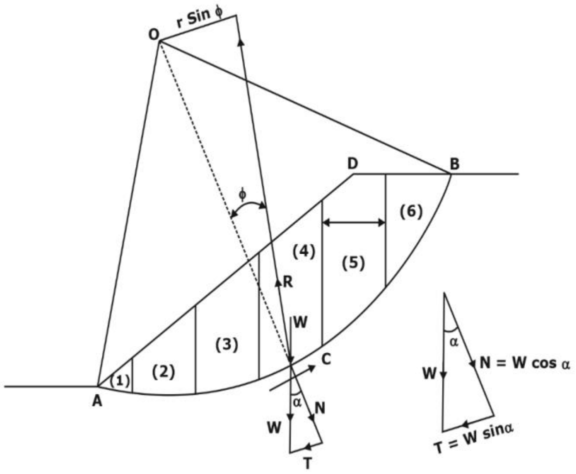
(i) Slope failure: In this case the failure surface passes through the sloping face. Observed in steep slopes when soil below the toe is hard and rigid comparison to soil above the toe.
(ii) Base failure: The failure surface passes below the toe. It is generally seen in case of flatter slopes where soil below the toe is comparatively softer then the soil above the toe.
(iii) Toe failure: In this case failure occurs along a surface that passes through the toe. In this failure soil above and below have same property.



(a) Toe failure **(b) Slope failure** **(c) Base failure**

Swedish Slip Circle Method

This method is suitable for both cohesive and frictional soil. The soil is cut into a number of slices of equal with the forces between different slices is neglected and each slice acts as an independent column of unit thickness.



- The factor of safety is given by,

$$FOS = \frac{resisting\ moment}{overturning\ moment}$$

$$\Rightarrow FOS = \frac{\tau_f r}{\sum T.r}$$

$$\Rightarrow FOS = \frac{[C_m(L \times 1) + \sum N tan\,\phi]r}{\sum T.r}$$

$$\Rightarrow FOS = \frac{C_m r\theta + \sum N tan\,\phi}{\sum T}$$

For seepage condition:

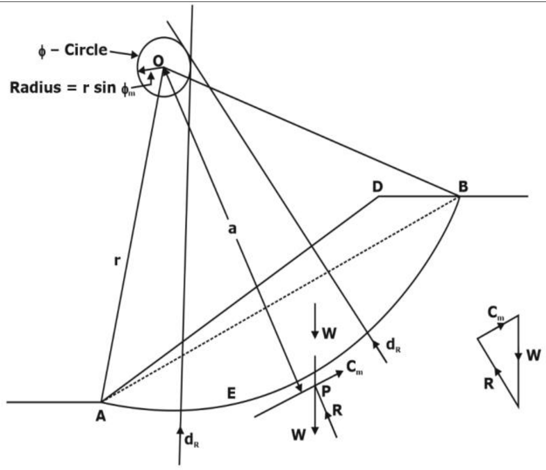
$$FOS = \frac{C_m r\theta + (\sum N - \sum V)\,tan\,\phi}{\sum T}$$

Friction Circle Method

This method is based upon total stress analysis and uses the shearing angle of the soil in order toanalyse the stability of finite slopes.

In this method, it is assumed that the resultant reactions between the two portions of the soil mass on either side of the slip plane passes tangentially to a smaller circle with the slip plane where radius of the circle is $(r.sin\phi)$ and is termed as a friction circle.

$$F_c = (FOS)_c = \frac{c}{c_m}$$



CHAPTER-12 SHALLOW FOUNDATION

BASIC DEFINITION

- **Gross Pressure or Gross loading intensity(q_g):**

It is the total pressure intensity at the base of footing.

$$q_g = \frac{P}{B^2} + \gamma * D_f$$

- **Net pressure Intensity (q_n):**

It is generally the loading intensity at the base of footing more than the load intensity that the soil was originally subjected to that causes deformation in soil. Hence, net pressure intensity

$$q_n = \frac{P}{B^2} = q_g - \gamma * D_f$$

If after placing of load, foundation is not filled with soil again. Then net pressure intensity will be

$$q_n = \frac{P}{B^2} - \gamma * D_f$$

- **Ultimate bearing capacity (q_u):**

The ultimate bearing capacity is the gross pressure at the base of the foundation at which the soil fails in shear.

- **Net Ultimate Bearing Capacity (q_{nu}):**

The net ultimate bearing capacity is the net pressure at the base of the foundation at which the soil fails in shear.

$$q_{nu} = q_g - \gamma * D_f$$

- **Net safe bearing capacity (q_{ns}):**

$$q_{ns} = \frac{q_{nu}}{F.O.S.}$$

Where F = Factor of safety, which is usually taken as 3.0.

- **Gross Safe Bearing Capacity (q_s):**

$$q_s = q_{ns} + \gamma * D_f \text{ or } q_{ns} = \frac{q_{nu}}{F.O.S.} + \gamma * D_f$$

- **Safe bearing pressure(q_{ps}):**

Maximum net intensity of loading that can be allowed on soil without the settlement exceeding the permissible value. No factor of safety is used when dealing with settlement.

- **Allowable bearing pressure** ($q_{a,\ net}$)

Maximum net intensity of loading that can be imposed on the soil with no possibility of shear failure or the possibility of excessive settlement. It is the smaller of net safe bearing capacity (**q_{ns}**) and safe bearing pressure (**q_{ps}**).

Note: According to IS code Allowable bearing pressure ($q_{a,\ net}$)is Allowable bearing capacity.

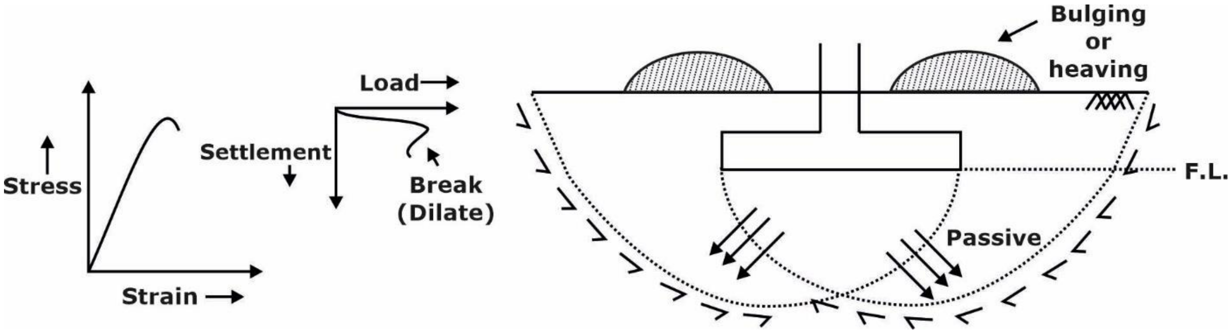
SHEAR FAILURE: The shear failure is in three modes.

(i) General shear failure

(ii) Local shear failure

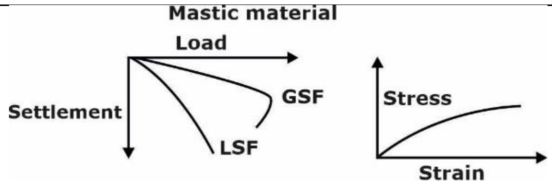
(iii) Punching shear failure

General shear failure: It occurs in shallow footing on the soils having brittle type of shear stress curve.



- The soils that are of medium dense sand, silt and over consolidated clays represent general shear failure, these shown less compressibility.
- At the time of failure, the foundation gets tilted on one side
- Before the failure the settlement will be negligible, and the stress zone willbe extended upto the ground level. A state of plastic equilibrium is reached initially around the edges, and it spread outward and downward. The portion of the soil below the foundation extents passive pressure on the surrounding soil.
- The shear planes developed are well defined.
- Bulging and heaving can be noticed in the surrounding of formation
- Density Index > 70% ; SPT value (N) > 30
- The angle of frictional resistance φ> 30°
- Void ratio < 0.55

Local Shear failure: The load settlement curve for a soil subjected to local shear failure represents the stress curve for a plastic material.

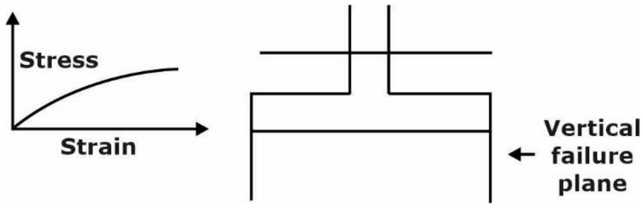
• Construable compression of the soil occurs the failure surface does not reach the ground surface and only a slight heaving observed due to excessive load. • This failure is observed in loose sand and soft clay. • No sudden failure is observed • A well-defined slip surface is only observed below the foundation.	 • The density index (30,70%); φ≤ 28°;SPT value < 5 • void ratio > 0.75;UCS<80 kN/m^2

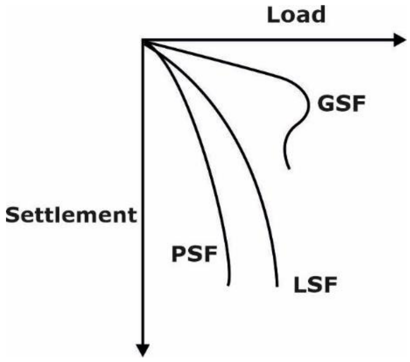
- for local shear failure, $c_m = \frac{2}{3}c$, $\tan\phi_m = \frac{2}{3}\tan\phi$

Punching shear failure

- The failure results in high compression of the soil accompanied by the shearing in vertical direction which is around the edge of footing.
- No heaving or budging is observed in the soil.
- The load settlement curve resembles the stress curve of a highly plastic material.
- This failure occurs in very loose sand and very soft clays.

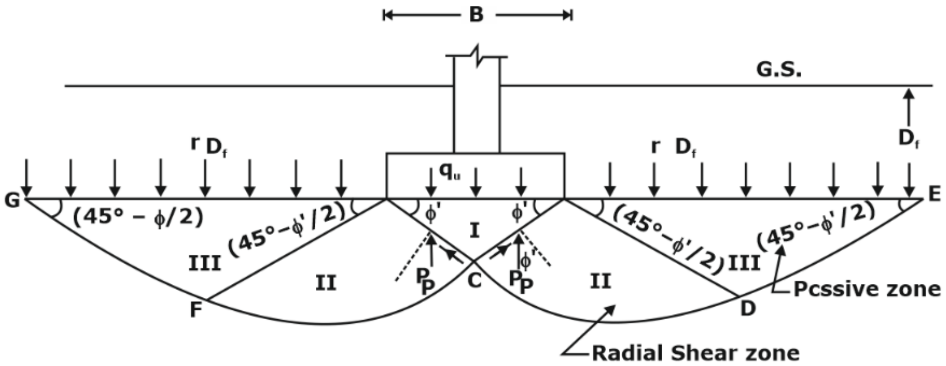
- The zone & beyond the vertical shear plane are not stressed.
- The density index < 30%.





TERZAGHI'S BEARING CAPACITY THEORY

- **The failure surface as given by Terzaghi is as follows:**



- **Assumptions:**

1. Footing is a strip footing (L >> B)
2. Soil is Homogenous
3 2-D plane strain condition prevails.
4. Base of footing is rough.
5. The base of footing is laid down at shallow depth.
6. Loading is vertical and symmetric i.e. (moment = 0)
7.General shear failure occurs.
8. Ground is Horizontal.
9. Shearing resistance of soil between the ground surface and base of footing is neglected. Thus, footing considered as a surface footing with uniform surcharge = $\boldsymbol{\gamma * D_f}$at the footing.
10. Shear strength of soil is goverend by Mohr's coulmb criteria.

- **Failure zone in the soil mass is divided into three zones:**

1.Zone (I)→ zone of elastic equilibrium. Assumed to be a part of the footing.
2. Zone (II)→→ Radial shear zone. Its lower curved boundary has shape of logarithmic spiral.
3. Zone (III)→→ Rankine's passive zone. Boundary makes an angle of $(45 - \frac{\varphi}{2})$ with horizontal.

- **The ultimate bearing capacity for strip footing:**

$$q_u = c.N_c + q.N_q + 0.5B_f.\gamma.N_\gamma$$

c = Effective cohesion, B_f = width of strip footing.
q=$\gamma * D_f$ and D_f = Depth of footing.

Nc, Nq and N_γ are the dimensionless numbers known as bearing capacity factors depending upon angle of friction of soil.

- **Local shear failure:** use c′ and φ′in case of local shear failure.

Mobilised cohesion, $c_m = \frac{2}{3}c$ and Mobilised angle of shearing resistance, $\phi_m = \tan^{-1}\left(\frac{2}{3}\tan\phi\right)$.

- **Net ultimate bearing capacity for strip footing:**

$$q_{nu} = c.N_c + q.(N_q - 1) + 0.5B_f.\gamma.N_\gamma$$

- **For clayey soil:**

For clayey soil, $\varphi = 0$ *and* $N_c = 5.7, N_q = 1, N_\gamma = 0.$

$$q_u = 5.7c + q$$

- **Bearing Capacity of Square and Circular Footing**

(a) Square footing:

$$q_u = 1.3c.N_c + q.N_q + 0.4B_f.\gamma.N_\gamma$$

Where, B is the dimension of each side of the footing.

(b) Circular footing

$$q_u = 1.3c.N_c + q.N_q + 0.3D.\gamma.N_\gamma$$

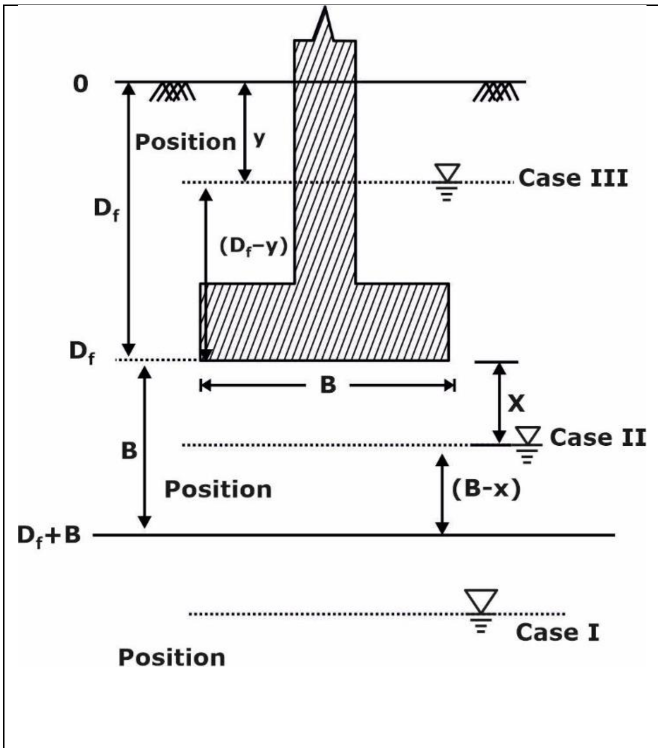
Where, D is the diameter of the footing.

(c) Rectangular footing:

$$q_u = \left(1 + 0.3\frac{B}{L}\right)cN_c + \gamma D_f N_q + 0.5\left(1 - 0.2\frac{B}{L}\right)\gamma B N_\gamma$$

Where, B and L are the width and length of the footing respectively.

- **Effect of Water Table on Bearing Capacity**



Case I: when water table is at depth morethan $D_f + B.$

Then $\boldsymbol{q_{nu} = c.N_c + \gamma_1.D_f(N_q - 1) + 0.5B.\gamma_2.N_\gamma}$.

Here, $\boldsymbol{\gamma_1}$ ***and*** $\boldsymbol{\gamma_2}$ are bulk unit weight $(\boldsymbol{\gamma_t})$ of soil.

Case II: when water table is at depth less than $D_f + B$, but more than D_f.

Then $\boldsymbol{q_{nu} = c.N_c + \gamma_1.D_f(N_q - 1) + 0.5B.\gamma_2.N_\gamma}$.

Here, $\boldsymbol{\gamma_1}$ is bulk unit weight of soil

and $\boldsymbol{\gamma_2.B = x * \gamma_t + (B - x) * \gamma_{sub}}$

Case III: when water table is at depth less than $D_f + B$, but more than D_f.

Then $\boldsymbol{q_{nu} = c.N_c + \gamma_1.D_f(N_q - 1) + 0.5B.\gamma_2.N_\gamma}$.

Here, $\boldsymbol{\gamma_1.D_f = y * \gamma_t + (D_f - y) * \gamma_{sub}}$

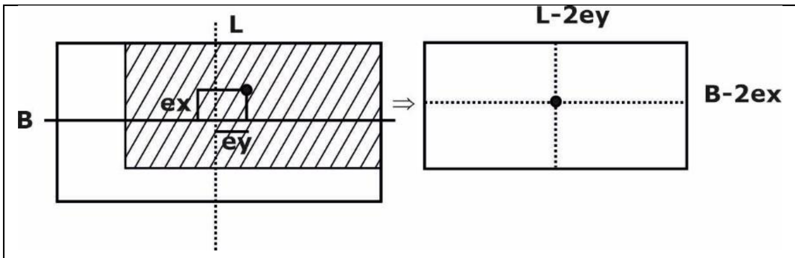
and $\boldsymbol{\gamma_2.B = \gamma_{sub}}$

MEYERHOFF'S BEARING CAPACITY THEORY

- It is most generalized theory in which the shape factor, depth factor and inclination factor are used to account for the shape of footing, variation in depth of footing, inclination of load and ground surface. the stress zone is considered to extend up tothe ground level hence this theory can be applied to deep foundation also.

$$q_u = c.N_c.(s_c.d_c.i_c) + \gamma.D_f.N_q(s_q.d_q.i_q) + 0.5\gamma.B_f.(s_\gamma.d_\gamma.i_\gamma)$$

- Meyerhoff has also given the concept of reduced area for the eccentric loading condition.It means on the footingthe line of action of the force is not passing through the centroid.Then the width and length are reduced such that loadactson the centerof the remaining area.



(Reduced area) is taken,

Now pressure is uniform.

$$Pr = \frac{Q}{(L-2e_y)(B-2e_y)}$$

SKEMPTON'S ANALYSIS FOR COHESIVE SOILS

- Skempton analysis is suitable for soils having an angle of internal friction equal to zero. As per Skempton,$q_{nu} = c_u N_c$.
- Here N_c is a function of shape of footing and $\frac{D_f}{B}$. This theory does not neglect the shear strength of soil above foundation of soil foundation level.
- The bearing capacity factors can be calculated using the following relations:
- For Strip Footing:

$$N_c = 5\left(1 + 0.2\frac{D_f}{B}\right) \leq 7.5$$

- For square and circular footings:

$$N_c = 6\left(1 + 0.2\frac{D_f}{B}\right) \leq 9$$

- For rectangular footing:

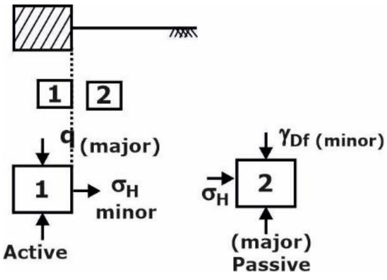
$$N_c = 5\left(1 + 0.2\frac{D_f}{B}\right)\left(1 + 0.2\frac{B}{L}\right);\ for\ \frac{D_f}{B} \leq\ 2.5$$

$$N_c = 7.5\left(1 + 0.2\frac{B}{L}\right); for\ \frac{D_f}{B} >\ 2.5$$

RANKINE THEORY

- This Theory is applicable for cohesionless soil, it considers the equilibrium of the elements adjacent to each other at the corner of footing. Element 1 is assumed in active state and element 2 is in passive state.

$$q = \gamma_{Df}\left[\frac{1+\sin\phi}{1-\sin\phi}\right]^2$$



- If the footing is at zero depth, then bearing capacity from this formula is also zero which is not practical.
- This theory does not consider effective of shape, as well as the size of the Foundation and hence it is absolute nowadays. Also does not fill it for clay soil.

S.P.T.

Test Procedure:

- It is Suitable for Granular soil.
- Split spoon samples is used in the bore hole.
- Bore hole in advanced to a depth at which N-value is to be calculated.
- The split-spoon sampler is allowed to penetrate the soil by applying impact load of 65 Kg having a free fall of 75 cm.
- The sample (is allowed to penetrate for 150 mm depth, but reading is not noted. (No. of blows required for 150 mm penetration is not Noted).
- Then the sampler is allowed to penetrate, further for 300 mm and No. of blows required to penetrate the sampler to this 300 mm is the SPT N-value.
- The next test is carried out at level 750 mm below the previous test reference level.
- If bore hole depth is large, then interval of next test is taken at a depth of (2.5 to 2) m or at the change of strata.
- The following values are refused (i) if for successive 10 number of blows there is no advancement in penetration of sampler. (ii) if for any 150mm penetration the N value is more than 50, then refused. (iii) if for any 300mm penetration N value is more than 100 it is refused.

The following corrections are required for N-values obtained above.

- **Over burden correction**
- It is necessary because the N-value will have effect on it due to confinement of soil at various depth.
- Two granular soils possessing the same relative density but having different confining pressures are tested, the one with a higher confining pressure will give higher N value.
- Since the confining pressure increases with depth, the N values at shallow depths are underestimated and the N values at larger depths are overestimated
- Therefore, if no correction is applied to recorded N values, the relative densities at shallow depths will be underestimated and at higher depths, it will be overestimated.
  - If N_0 = observed S.P.T. value Then,

$$N_1 = N_0 \times \frac{350}{\overline{\sigma} + 70}$$

Where, $\overline{\sigma}$ = Effective stress at level of test (KN/m²),

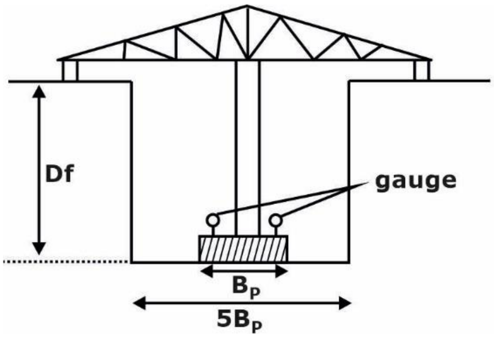
N_1 = Corrected N-value of overburden,

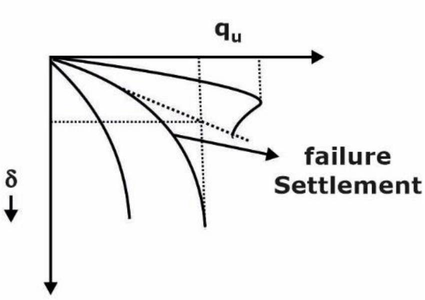
Overburden correction will not be applied if, $\overline{\sigma}$>280 KN/m².

- **Dilatancy Correction.**
- It is applied to the already corrected N-values for overburden pressure, Dilatancy correction is required only if [N1>15] in saturated fine sand and silt (when water table is above test level).
- (N> 15) basically represents the Dense sand which will have the tendency to dilate under rapid loading (undrained condition) and (-ve) pore water pressure will develop. Hence, observed (N) value will be more because shear resistance will increase.
  - Corrected N value after Dilatancy Correction is $N_2=15+0.5*(N_1-15)$
- This correction becomes more significant for fine dense sand.

Late load test:

- It is performed to calculate the ultimate bearing capacity of the soil, and pressure vs ultimate settlement graph is made.
- The size of the plate is kept between 30cm and 75 cm, the smaller size plate is used for granular soil and for clayey soil large size are used.The width of the test pit should be greater than 5 times the plate width.
- Initially a pressure of 70 kg/cm^2 which is also known as the this is to stablish a perfect contact between plate and soil.





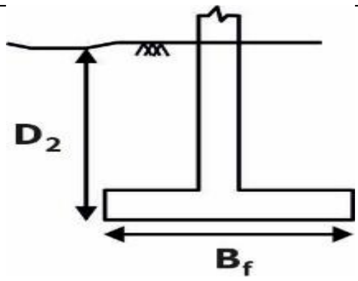
- The water table is kept below the test level so as to avoid any pore pressure development.
- The test is performed in the increment of of 1/5th of assumed ultimate load upto the breaking point or upto a total settlement of 25mm whichever occurs first.
- If a marked breakpoint is not visible then approximate value of ultimate bearing capacity is determined by double tangent method.

Correction:

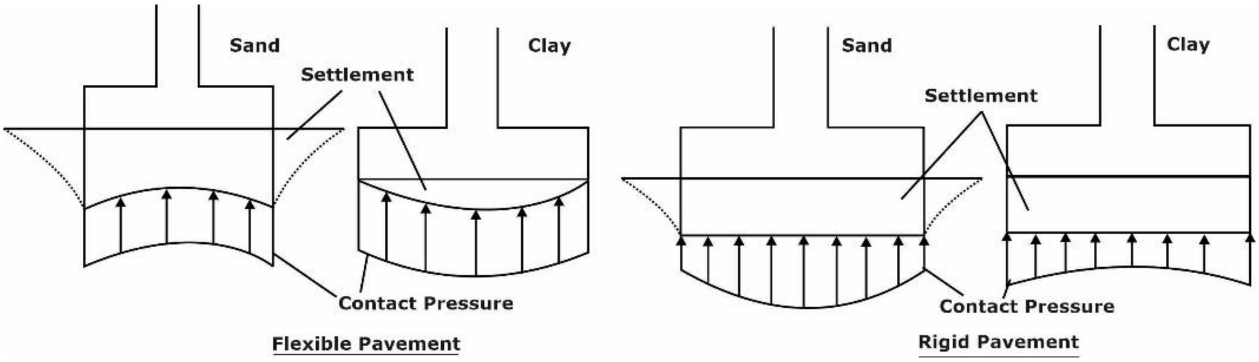
Clay	Sand
$q_{uf} \equiv q_{up}$	$\frac{q_{uf}}{q_{aP}} = \frac{B_f}{B_P}$
$\frac{S_f}{S_p} = \frac{B_f}{B_p}$.	$\frac{S_f}{S_p} = \left[\frac{B_f}{B_P} \times \frac{(B_p+03)}{(B_f+0.3)}\right]^2$

All values are in m.

- It the actual foundation level is below the plate load test level then the correction need to be performed for settlement.

$(S_\delta)_{\text{corrected}} = S_f \left[\frac{1}{1+\frac{D_2}{B_f}}\right]^{1/2}$	

Settlement and pressure distribution:



Immediate settlement or elastic settlement:

$\boxed{S_i = \frac{q \cdot BI\left(1-\mu^2\right)}{E}}$	B – width I – Influence factor E - modulus of elasticity of soil μ - poisons ratio q - not pressure intensity

- In flexible footing the contact pressure is uniform over thefoundation width whereas in rigid footing the settlement is uniform along the foundation width. To increases to settlement at any point the contact pressure at that particular point head s to be increased.
- In case of granular soil E varies along the loaded area with lesser value at the corner compare to the centralpoint.

CHAPTER-13 DEEP FOUNDATION

CLASSIFICATION OF PILE:

- **Based on material:** steel, concrete, timber.
- **Based on load transfer:** end bearing, friction, combined.
- **Based on installation method:** bored, driven.
- **Based on function and location:** compression, tension, anchor, batten, compaction, fender/dolphin pile are provided in case of hydraulic structure.
- **Based on soil displacement:** displaced, non-displaced.

PILE LOAD CAPACITY BY STATIC FORMULA:

$$Q_U = Q_B + Q_S \qquad Q_B = \text{end bearing}$$

$$Q_U = \text{ultimate bearing capacity} \qquad Q_S = \text{skin friction}$$

End bearing:		
c − φ soil	**sand**	**clay**
$q_u = cN_c + \bar{\sigma}N_q$	$q_u = \bar{\sigma}N_q$	$q_u = c_uN_c$
$Q_B = q_u \times A_b$ A_b = area of the end	$\bar{\sigma}$ = Effective overburden pressure at the tip of the pile L = Length of embedment of the pile	N_c=9 for deep foundation. C_u = undrained cohesion

Skin friction:	
sand	**clay**
$Q_s = f_s * A_s$ and $A_s = \pi DL$	$Q_s = \alpha * C_u$ and $A_s = \pi DL$
frictional reisistance = $f_s = \mu.N$	C_u = *undrained cohesion*
N = lateral earth pressure = $K * \bar{\sigma}_{av}$ μ = coefficient of friction = $\tan\delta$	α = adhesion factor $\alpha = 1$ for loose soil and $\alpha = 0.3$ for very stiff soil

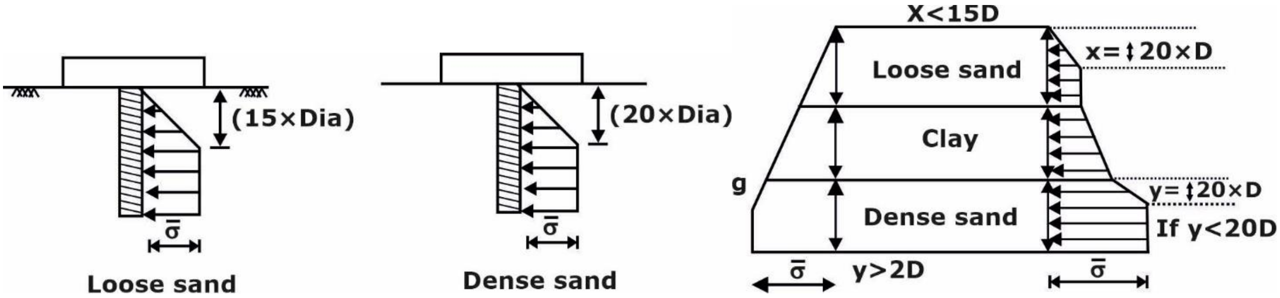
Table: Value of K and δ

Pile material	**Δ**	**Values of K** **Loose sand**	 **Dense sand**
Steel	20°	0.5	1.0
Concrete	0.75 φ	1.0	2.0
Timber	0.67 φ	1.5	4.0

Arching effect in sand: it is observed that value of effective stress increases up to a certain depth of sand, after which it become constant, this is due to arching effect.

For loose sand, critical depth is 15d.

For dense sand, critical depth is 20d.



Note: this critical depth concept is not applicable for clay strata, in clay arching effect is negligible.

Note: All these calculations are for driven piles, in case of bored cast in-situ pile: due to boring condition, the value of K is very small. K is usually taken as 0.5 and $\delta = \varphi$. Rest of the calculations are same. Point bearing resistance of Board cast insitu piles= 0.5 (point bearing resistance of driven pile).

PILE CAPACITY USING DYNAMIC PILE FORMULA

- **Engineering News Formula**

$$Q_u = \frac{WH}{F(S+C)}$$

F = factor of safety is taken as 6.

- W = load (in KN)
- Height of fall H (in cm)
- S = set (penetration per blow of hammer) in cm (taken for last 5 blows of drop hammer).
- C is taken as 2.5 cm for drop hammer and 0.25 cm for Single Acting Steam Hammer (SASH).

- **Modified hilly formula**

$$Q_u = \frac{Wh\eta}{(S+\frac{C}{2})}$$

F = factor of safety is taken as 6.

- W = load (in KN)
- Height of fall H (in cm)
- S = set (penetration per blow of hammer) in cm (last blows).
- C is total elastic compression per blow (elastic compression of pile + soil + dolly)
- η is efficiency of blow, depends on coeff. Of restitution.

PILE CAPACITY BASED ON SPT VALUES

$$Q_u = 40N.\frac{D_f}{B}.A_B + 2\bar{N}A_s$$

$$and\ 40N.\frac{D_f}{B} < 400N\ [\frac{KN}{m^2}]$$

- Q_u =ultimate load capacity for driven pile
- Same formula is used for displacement piles also.

For H-pile

$$Q_u = 40N.\frac{D_f}{B}.A_B + \bar{N}A_s\ and\ 40N.\frac{D_f}{B} < 400N\ [\frac{KN}{m^2}]$$

- For bored piles value is $1/3^{rd}$ of driven pile.

GROUP ACTION OF PILES

- When single pile is used as a driven pile, there is uncertainty regarding vertical installation of piles. Hence, in case of driven piles, minimum three piles used. Whereas in case of bored piles verticality can be ensured. Hence, single piles can also be used. However, we always provide group of piles.

- If pile cap is above the ground surface then it is called free standing pile group (used in case of expanding soil at top).
- **Minimum spacing between pile:**

  (1) 2.5 × Diameter → for point bearing piles [Centre to centre]

  (2) 3 x Diameter → for friction piles.

  (3) 2 × Diameter → for loose sand or fill deposits.

  In case of non-circular piles, diameter of circumscribed circle will be taken as the diameter
- **Ultimate Bearing capacity of pile group for Clay:**
  - The group of pile may fail as: (a) Block failure (b) Individual pile failure
  - In block failure, soil is bound by perimeter of pile group and embedded length of pile as one unit or block.
  - Ultimate load capacity of the pile group for block failure is given by.

$$\boldsymbol{Q_{ug} = [C_{ub} * N_C * area\ of\ base\ of\ block + \alpha * C_u * L * perimeter\ of\ the\ block]}$$

  - C_{ub} = Undrained strength of clay at the base of the pile group.
  - N_c = 9, for deep foundation.
  - C_u = average undrained strength of clay over the length of block.
  - Value of $\alpha = 1$, in case of pile group (soil-soil interaction)
  - Whereas safe load capacity of pile group is given by $\boldsymbol{min(Q_{ug}\ and\ n * Q_u)}$.
- **Ultimate Bearing capacity of pile group for in sands:**
  - It has been observed that group efficiency of driven piles in loose or medium dense sand is >1 . This is because soil around and between the piles get compacted due to the vibration caused during the driving operations. Whereas in dense sand above phenomenon is not true.
  - For design purpose we never take group efficiency greater than 1. Hence an efficiency factor of 1 is commonly assumed in design.
- **Group efficiency:**

$$group\ efficiency = \eta = \frac{Q_{ug}}{n * Q_u}$$

- n = No. of piles
- Q_{ug} = ultimate load capacity of pile group.
- Q_u= ultimate load capacity of single pile.

NEGATIVE SKIN FRICTION

- Negative skin friction or 'down drag' is a phenomenon, which occurs when a portion of soil layer surrounding a pile settles more than the pile. This condition can develop when a soft soil stratum located above the pile tip is subjected to a compressive loading, the soil may settle more than the pile, also by lowering of ground water table which includes consolidation of the soft soil.
- Normally friction between pile and soil helps in carrying the axial load. Where negative skin friction (or downward drag) developed, it increases the load acting on the pile because the weight of consolidating layer is transferred to pile by friction, thus imposing extra load on the pile.
- Negative skin friction can be reduced either by providing a casing around the pile or by providing a bitumen coating around the precast pile.
- **Negative Skin Friction in Single Pile**

Cohesive soil: $$F_n = P * L_s * \alpha * C$$ Cohesionless soil: $$F_n = 0.5 * P * L_s^2 * \gamma * K * Tan\delta$$	F_n = negative skin friction on single pile P = perimeter of pile L_s = length of pile in settling zone C_u = undrained cohesion of compressible soil. K = lateral earth pressure coefficient δ = angleof friction between pile and soil $(0.5 - 0.66)\phi$ α = adhesion factor

- **Negative skin friction in pile group**
  the magnitude of negative skin friction

F_{ng}=	(Frictional force on the block + weight of soil enclosed in the block) Or (Negative skin friction due to single pile x Number of files in the pile group)	$N * F_n$ Or $C_u * L_c * P_g + \gamma * L_c * A_g$

Effect of Negative skin friction on Factor of safety:

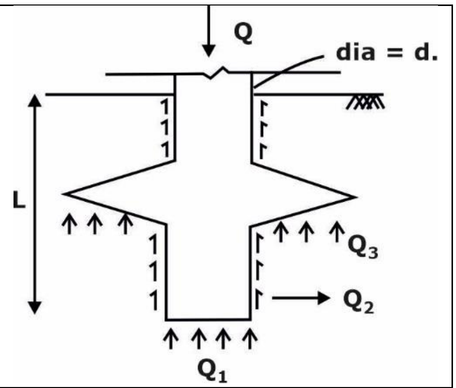
As it is necessary to subtract negative skin friction force from the total load that the pile can support. In such a case factor of safety will be as below:

$$FOS = \frac{ultimate\ load\ capacity\ of\ single\ pile\ or\ a\ group}{working\ load + negative\ skin\ friction}$$

UNDER REAMED PILES

- These are provided on expansive soil which are bored into the soil and are cast insitu with a bubble. As per IS2911, maximum 2 bulbs can be provided.

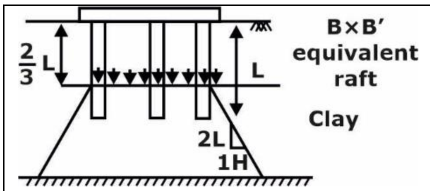
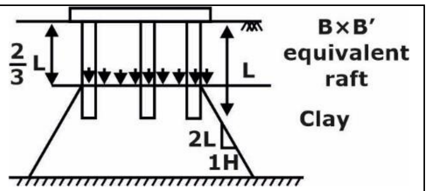
- Q+W=resistance=Q_1 +Q_2 +Q_3
- $Q_1 = f_B * A_B$
$$f_B = \overline{\sigma_V} * N_q (for\ sand)$$
$$f_B = C * N_c\ (for\ clay)$$
- $Q_2 = f_s * A_s$
$$A_s = \pi DL$$
$$f_s = \overline{\sigma_V}_{av.} * k * tan\delta\ \ (for\ sand)$$
$$f_s = C * \alpha\ (for\ clay)$$
$$Q_3 = (f_B)_{(bulb\ level)} * [\frac{\pi}{4} D_B^2 - \frac{\pi}{4} D^2]$$



- Bulb helps in bearing and anchorage.
- The diameter of bulb should be between 2-3 times diameter of shaft.
- Vertical distance between bulbs shall be more than 1.5 times the bulb diameter.
- Horizontal pile spacing shall be more than 2 times the bulb diameter.

SETTLEMENT OF PILE GROUP

- The settlement of the pile group is greater than that of individual pile, this is because a large volume of soil gets influenced by the group behaviour.
- In clay soil since the major part of settlement is due to primary consolidation hence the pile load test which is performed for short duration, do not give accurate results. Hence an equivalent raft theory is assumed for calculation of group settlement.
- 1. Friction pile (NCC): an equivalent raft a a depth of 2/3rd of embedded length of the pile is covered to transform the load on the soil in case of friction pile and also displacement pile.

 $S_f = C_C \cdot \frac{H}{1+e_0} \log_{10} \frac{\overline{\sigma_0} + \overline{\Delta\sigma}}{\overline{\sigma_0}}$ $\boxed{q = \frac{Q}{B \times B'}}$	

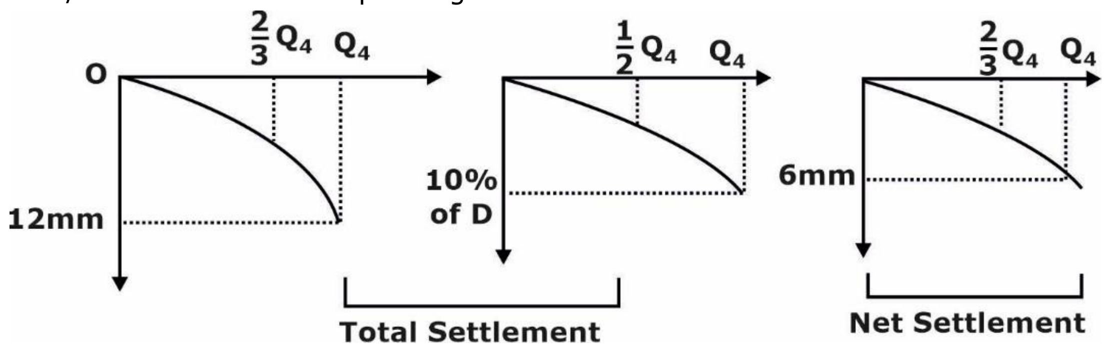
- 2. In case of end bearing pile the equivalent raft is considered at the tip of the pile.
- 3. For sandy soil the settlement is calculated empirically in reference to the settlement obtained from pile load test.

$\frac{S_g}{S_i} = \left[\frac{4B+2.7}{B+3.6}\right]^2$	B → width of the bock (m) (smaller) S_g→ group settlement S_i→ individual file statement

- **Note:** Load carrying capacity of a pile group can be calculated either from shear Strength criteria and Settlement criteria and minimum value will be adopted.

PILE LOAD TEST

- It is a direct method to evaluate the file capacity by doing institute test. IS911 states that if the number of files is more than 200 then at least two number of initial test needs to be performed. For routine test approximately 0.5-2% of total number of piles are considered for testing and initial test piles do not form the part of structural system and are abandoned after the test. Initial test is performed to obtain the allowable load and to check the settlement.
- Routine test is done on the piles which are part of the structure system, to assess the soil densification and the suitability against the working load.
- Safe load for the piles is taken as the minimum of the following three criteria:
  1. 2/3rd of ultimate load value corresponding to settlement of 12mm in total.
  2. 1/2nd of the load corresponding to the settlement of 10% of pile diameter in total.
  3. 2/3rd of the load corresponding to net settlement of 6mm.



CHAPTER-14 SOIL EXPLORATION

Soil Sample

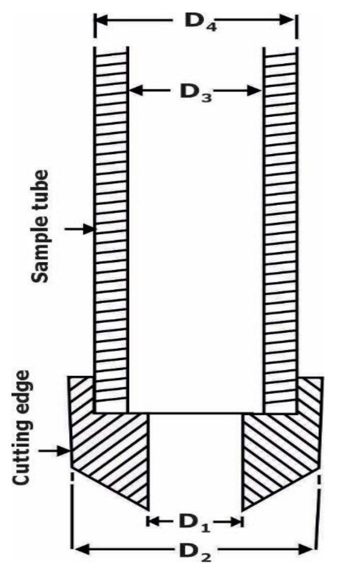
- Disturbed samples are those in which natural soil structure gets modified or destroyed during the sampling operation. But we can observe natural water content, minerals composition, consistency limits and specific gravity of solidsfrom those samples are called representative samples.
- Undisturbed samples are those in which original soil structure is preserved as well as mineral properties have not undergone any change. These are used for calculation of permeability, shear strength and consolidation characteristics.

Basic Terminology in a Sampler:

<table>
<tr><td>(a) Inside clearance
$\boxed{C_i = \frac{D_3 - D_1}{D_1} \times 100}$ it should be 1-3%.</td><td rowspan="3">D_1 =inner diameter of cutting edge

D_2 =outer diameter of cutting edge

D_3 =inner diameter of sampling tube

D_4 =outer diameter of sampling tube</td><td rowspan="3">
</td></tr>
<tr><td>(b) Outside Clearance
$\boxed{C_0 = \frac{D_2 - D_4}{D_4} \times 100}$ it should be 0-2%.</td></tr>
<tr><td>(c) Area ratio
$$A_r = \frac{D_2^2 - D_1^2}{D_1^2} \times 100$$
it should be < 20 For stiff clays and < 10 For sensitive clays</td></tr>
</table>

- In order to express the results, of the sampling operations and to have the satisfactory design requirements following design criteria shall be used for samplers.
  - $Recovery\ Ratio = \frac{L}{H}$
  - Where L is length of the sample before withdrawal,
  - H is penetration of the sample in the soil mass
  - If recovery Ratio is

= 1, Good recovery

<1, compressed

>1, Soil is swelled

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