Soil Mechanics & Foundation Engineering

Index

Properties of Soils & Classification and Structure of Soil

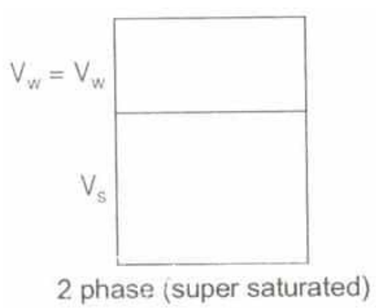
Properties of Soils

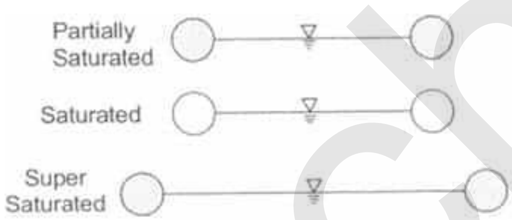
Phase Diagram

- Soil mass is in general a three-phase system composed of solid, liquid and gaseous matter in a blended form with each other.

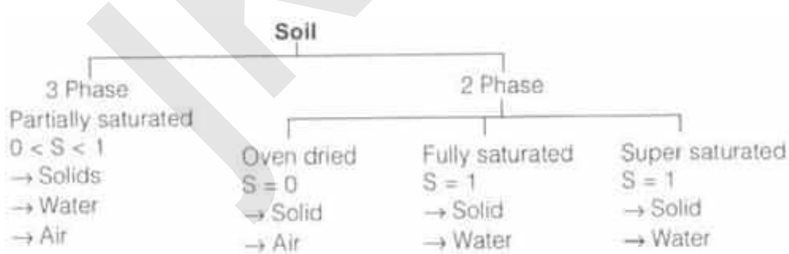
But in phase diagram – for understanding, these three matters are shown separately.

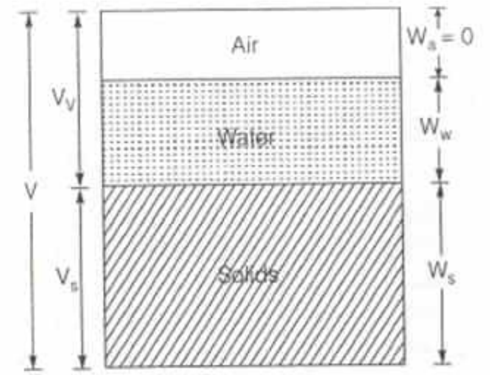
- In supersaturated, state with change in water content volume of voids changes hence volume of soil changes.



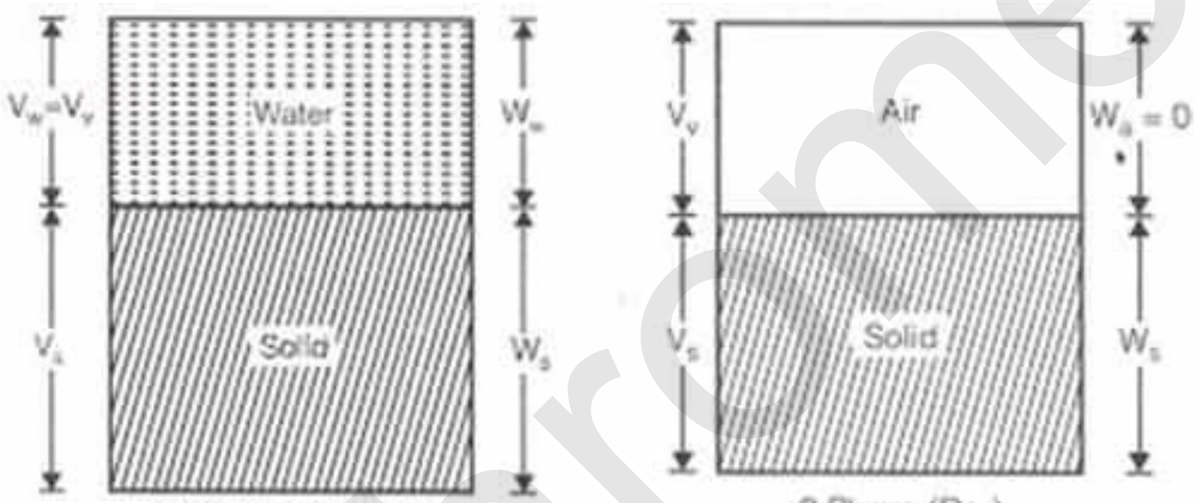


The diagrammatic representation of the different phases in a soil mass is called the "phase diagram".





3-Phase: (Partially Saturated)



2-Phase (Saturated)
2-Phase (Dry)

Water content

$$w = \frac{W_W}{W_S} \times 100$$

where, W_W = Weight of power

W_S = Weight of solids

There can be no upper limit to water content, i.e. w ≥ 0

Void ratio

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

where, V_v = Volume of voids

V = Total volume of soil

Porosity cannot exceed 100% i.e.,

0 < n < 100

Void ratio is more important engineering property.

Degree of Saturation

$$S = \frac{V_w}{V_v} \times 100$$

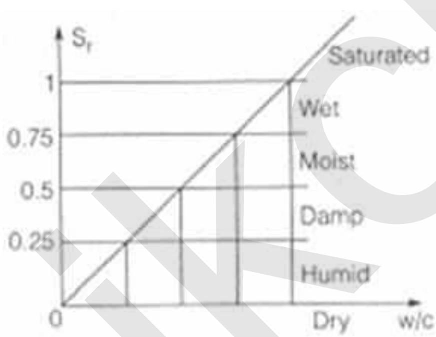
where, V_w = Volume of water

V_v = Volume of voids

$0 \leq S \leq 100$

for perfectly dry soil : S = O

for Fully saturated soil : S = 100%



Air Content

$$a_c = \frac{V_a}{V_v} = 1 - s$$

V_a = Volume of air

$S_r + a_c = 1$

% Air Void

$$\%n_a = \frac{\text{Volume of air}}{\text{Total volume}} \times 100 = \frac{V_a}{V} \times 100$$

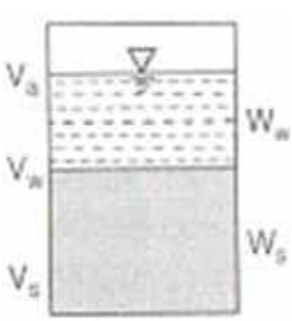
$n_a = n.a_c$

Unit Weight

A. Bulk unit weight

$$\gamma = \frac{W}{V} = \frac{W_s + W_w}{V_s + V_w + V_a}$$

Thus Bulk unit weight is total weight per unit volume.



B. Dry Unit Weight is the weight of soil solids per unit volume.

$$\gamma_d = \frac{W_s}{V}$$

- Dry unit weight is used as a **measure of denseness** of soil. More dry unit weight means more compacted soil.

C. Saturated unit weight: It is the ratio of total weight of fully saturated soil sample to its total volume.

$$\gamma_{sat} = \frac{W_{sat}}{V}$$

Submerged unit weight or Buoyant unit weight (γ): It is the submerged weight of soil solids per unit volume.

$$\gamma' = \gamma_{sat} - \gamma_w$$

γ_{sat} = unit wt. of saturated soil

γ = unit wt. of water

Unit wt. of solids:

$$\gamma_s = \frac{W_s}{V_s}$$

γ is roughly 1/2 of saturated unit weight.

Specific Gravity

True/Absolute Special Gravity, G

- Specific gravity of soil solids (G) is the ratio of the weight of a given volume of solids to the weight of an equivalent volume of water at 4℃.

$$G = \frac{W_s}{V_s . \gamma_w} = \frac{\gamma_s}{\gamma_w}$$

G = 2.6 to 2.75 for inorganic solids

= 1.2 to 1.4 for organic solids

- **Apparent or mass specific gravity (G_m):** Mass specific gravity is the specific gravity of the soil mass and is defined as the ratio of the total weight of a given mass of soil to the weight of an equivalent volume of water.

$$G_m = \frac{W}{V . \gamma_w} = \frac{\gamma \text{ or } \gamma_d \text{ or } \gamma_{sat}}{\gamma_w}$$

where, γ is bulk unit wt. of soil

$\gamma = \gamma_{sat}$ for saturated soil mass

$\gamma = \gamma_d$ for dry soil mass

$G_m < G$

In India, G is reported at 27℃,

$$G_{T°C} = G_{27°C}\left(\frac{\gamma_w \text{ at } 27°\text{C}}{\gamma_w \text{ at } \text{T}°\text{C}}\right)$$

Relative density (I_D)

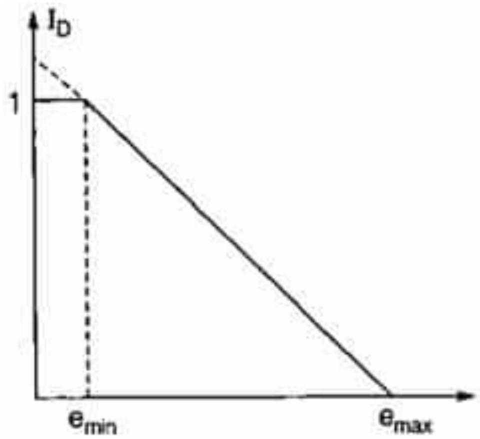
To compare the degree of denseness of two soils.

$$I_D \propto Shear\ strength \alpha \frac{1}{Compressibility}$$

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

$$\%I_D = \frac{\frac{1}{\gamma_{d\min}} - \frac{1}{\gamma_d}}{\frac{1}{\gamma_{d\min}} - \frac{1}{\gamma_{d\max}}} \times 100$$

% I_D	Description
0-15	Very loose soil
15-30	Loose soil
30-65	Medium soil
65-85	Dense soil
85-100	Very dence soil



A. when particles are arranged in cubical array

e_{max} = 91%, n_{max} = 47.6%

B. When particles are arranged in prismoidal array (Rhomohedral Array)

e_{min} = 35%, n_{min} = 25.9%

Relative Compaction

Indicate: Degree of denseness of cohesive + cohesionless soil

$$R_c = \frac{\gamma_D}{\gamma_{D_{max}}}$$

Relative Density

Indicate: Degree of denseness of natural cohesionless soil

Some Important Relationships

(i) Relation between γ_d, γ

$$\gamma_d = \frac{\gamma}{1+w}, \quad V_s = \frac{V}{1+e} \quad \& \quad W_s = \frac{W}{1+w}$$

(ii) Relation between e and n

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

(iii) Relation between e, w, G and S:

Se = w. G

(iv) Bulk unit weight (γ) in terms of G, e, w and γ_w γ, G, e, S_r, γ_w

$$\gamma = \frac{(G + eS_r)\gamma_w}{1+e}$$

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

(v) Saturated unit $(\gamma sat.)$ weight in terms of G, e & γ_w

S_r = 1 $$\gamma_{sat} = \left[\frac{G+e}{1+e}\right].\gamma_w$$

(vi) Dry unit weight γ_d in terms of G, e and γ_w

S_r = 0

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

(vii) Submerged unit weight (γ') in terms of G, e and γ_w

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

(ix) Relation between degree of saturation (s) w and G

$$S = \frac{W}{\frac{\gamma_w}{\gamma}(1+W) - \frac{1}{G}}$$

Methods for determination of water content

(i) Oven Drying Method

- Simplest and most accurate method
- Soil sample is dried in a controlled temperature (105-110℃)
- For organic soils, temperature is about 60℃. Soil having gyprum, temperature $\ngtr 80°C$
- Sample is dried for 24 hrs.
- For sandy soils, complete drying can be achieved in 4 to 6 hrs.
- Water content is calculated as:

$$w = \frac{W_2 - W_3}{W_3 - W_1} \times 100\%$$

where, W_1 = weight of container

W_2 = weight of container + moist sample

W_3 = weight of container + dried sample

Weight of water = $W_2 - W_3$

Weight of solids = $W_3 - W_1$

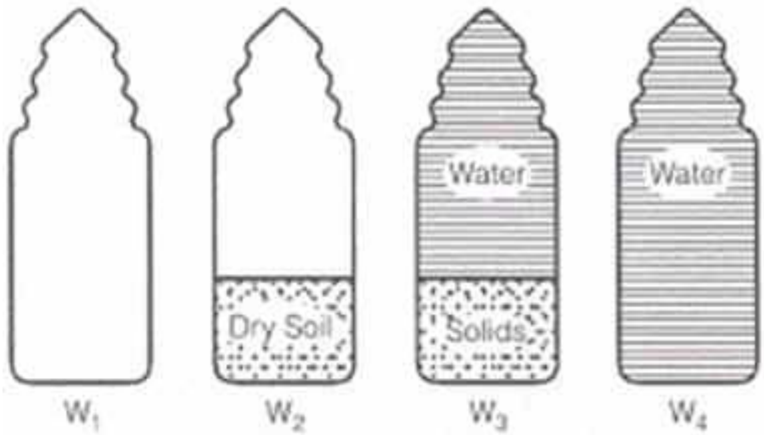
(ii) Pycnometer Method

- quick method
- capacity of pycnometer = 900 m/.
- this method is more suitable for cohesionless soils.
- **used when specific gravity of soil solids is known**
- Let W_1 = Wt. of empty dried pycnometer bottle

W_2 = Wt. of pycnometer + Soil

W_3 = Wt. of pycnometer + Soil + Water

W_4 = Wt. of pycnometer + Water.



$$w = \left[\frac{(W_2 - W_1)}{(W_3 - W_4)}\left(\frac{G-1}{G}\right) - 1\right] \times 100\%$$

$(W_1, W_2, W_3 and W_4$

are in anticlockwise order)

(iii) Calcium Carbide Method/Rapid moisture Meter Method Field Method

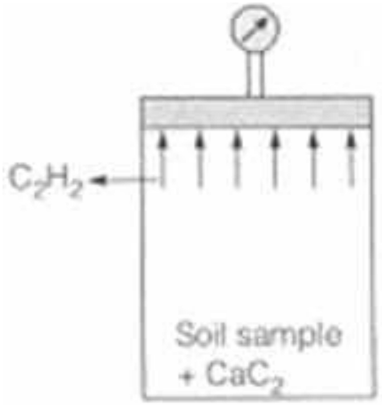
- Quick method (requires 5 to 7 minutes); but may not give accurate results.
- The reaction involved is

$$CaC_2 + 2H_2O \rightarrow C_2H_2 \uparrow + Ca(OH)_2$$

- Soil sample weights 4-6 gms.
- The gauge reads water content with respect to total mass of soil. i.e., \
  $$w_r = \frac{W_w}{(W_s)_{wet}}$$

(In this equipment pressure calibrated against water content with respect to total mass)

- Actual water content $w = \dfrac{w_r}{1 - w_r} \times 100\%$



w_r is moisture content recorded, expressed as fraction of moist wt. of solid.

w is actual water content.

(iv) Sand Bath Method (Field Method)

- quick, field method
- used when electric oven is not available.
- soil sample is put in a container & dried by placing it in a sand bath, which is heated on the kerosene store.
- water content is determined by using same formula as in oven drying method.

(v) Torsion Balance Moisture Meter Method

- quick method for use in laboratory.
- Infrared radiations are used for drying sample.
- **Principle:** The torsion wire is prestressed accurately to an extent equal to 100% of the scale reading. Then the sample is evenly distributed on the balance pan to counteract the prestressed torsion and the scale is brought back to zero. As the sample dries, the loss in weight is continuously balanced by the rotation of a drum calibrated directly to read moisture% on wet basis.

(vi) Alcohol Method

- It is a quick method adopted in field.
- Should not be used for organic soil and soils containing calcium compound.

Determination of specific gravity of soil solids

- Pycnometer method is used.
- Instead of pycnometer, Density bottle (50 ml) OR Flask (500 ml) can also be used.

Let, W_1 = Weight of empty pycnometer

W_2 = Weight of pycnometer + soil sample (oven dried)

W_3 = Weight of pycnometer + soil soilds + water

W_4 = Weight of pycnometer + water

$$G = \frac{W_2 - W_1}{(W_2 - W_1) - (W_3 - W_4)} \Rightarrow G = \frac{W_s}{W_s - W_3 + W_4}$$

Methods for the determination of insity unit weight

(A) Core-Cutter Method

- Used in case of cohesive soils.
- Cannot be used in case of hard and gravelly soils.

$$\gamma = \frac{W}{V}$$

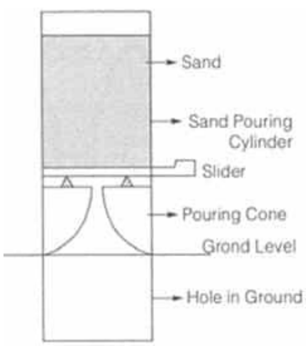
- The method consists of driving a core-cutter (Volume = 1000 cc) into the soil and removing it, the cutter filled with soil is weighted. Volume of cutter is known from its dimensions and in situ unit weight is obtained by dividing soil weight by volume of cutter.

- If water content is known in the laboratory, the dry unit weight can also be computed.

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

(b) Sand Replacement Method

- Used in case of hard and gravelly soils.
- A hole in ground is made. The excavated soil is weighted. The volume of hole is determined by replacing it with sand. Insitu unit weight is obtained by dividing weight of excavated soil with volume of hole.



(c) Water Displacement Method

- Suitable for cohesive soils only, where it is possible to have a lump sample.
- A regular shape, well trimmed sample is weighted. (W_1). It is coated with paraffin wax & again weighted (W_2). The sample is now placed in a metal container filled with water upto the brim. Let the volume of displaced water be V_m. Then volume of uncoated specimen is calculated as,

$$V = V - \left(\frac{W_2 - W_1}{\gamma_p} \right)$$

where, = unit wt. of paraffin wax

Thus, bulk unit wt. of soil

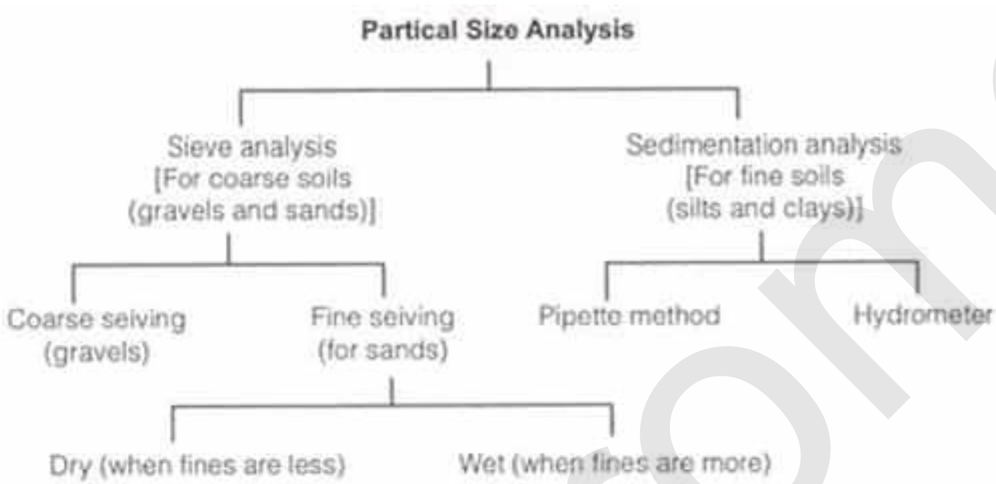
- **Sands + Gravels:** Bulky grains

Bulk grains classified as – angular, Subangular, Sub rounded, rounded, well rounded

Higher angularity ∝ Higher Shear Strength

- **Clay Minerals:** Flaky grains

Grain size distribution



- **Sieve Analysis: (For Coarse Grained Soils)**

The fraction retained on 4.75 mm sieve is called the gravel fraction which is subjected to coarse sieve analysis.

The material passing 4.75 mm sieve is further subjected to fine sieve analysis if it is sand or to a combined sieve and sedimentation analysis if silt and clay sizes are also present.

- **Coarse Sieves:** 4.75 mm, 10 mm, 20 mm, 80 mm.
- **Fine Sieves:** 75 μ, 150 μ, 212 μ, 425 μ, 600 μ, 1 mm, 2 mm.
- **Concept of "Percentage finer"**

% retained on a particular sieve

$$= \frac{\text{Weight of soil retained on that sieve}}{\text{Total weight of soil taken}} \times 100$$

Cumulative % retained = sum of % retained on all sieves of larger sizes and the % retained on that particular sieve.

"Percentage finer" than the sieve under reference = 100% - Cumulative % retained.

- **Sedimentation Analysis**

According to stokes law, the terminal velocity is given by,

$$V = \frac{g}{18} \cdot \frac{\rho_s - \rho_w}{\mu} \cdot D^2$$

= density of grains (g/cm^3)

ρ_s = density of water (g/cm^3)

μ = viscosity of water

g = acceleration due to gravity (cm/s^2)

D = Diameter of grain (cm)

If 'h' the height through which particle falls in time't', then

$$\frac{h}{t} = k.D^2 \quad \therefore \frac{D_1}{D_2} = \sqrt{\frac{h_1}{h_2} \cdot \frac{t_2}{t_1}}$$

- **Pipette Method**

In this method, the weight of solids per cc of suspension is determined directly by collecting 10 cc of soil suspension from a specified sampling depth.

If m_d = dry mass (obtained after drying the sample) then, mass present in unit vol. of pipette

$$\frac{m_d}{\text{Vol. of pipette } (v_p)} = \frac{m_d}{10 \text{ ml. } (v_p)}$$

If M_d = total mass of soil dissolved in total volume of water (V) then mass/unit volume

$$= \frac{M_d}{V}$$

Therefore, % finer is given by = %

$$N = \frac{m_d N_p}{m_d N}$$

In m is the mass of dispersing agent dissolved in the total volume V, then actual % finer,

$$\%N = \frac{\frac{\mathrm{m}_d}{V_p} \qquad \frac{m}{V}}{\frac{\mathrm{M}_d}{V}} \times 100$$

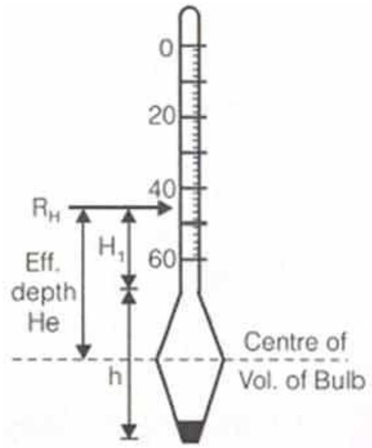
- **Hydrometer Method**

In this method the weight of solids present at any time is calculated indirectly by reading the density of soil suspension.

- **Calibration of Hydrometer**

Establishing a relation between the hydrometer reading R_H and effective depth (H_e).

The effective depth is the distance from the surface of the soil suspension to the level at which the density of soil suspension is being measured.



Effective depth is calculated as

$$H_e = H_1 + \frac{1}{2}\left(h - \frac{V_H}{A_j}\right)$$

where, H_1 = distance (cm) between any hydrometer reading and neck.

h = length of hydrometer bulb

V_H = volume of hydrometer bulb

A_J = area of the cross section of the jar.

Reading of Hydrometer is related to sp. gr. or density of soil suspension as:

$$G_{SS} = 1 + \frac{R_H}{1000}$$

Thus, a reading of R_H = 25 means, G_{ss} = 1.025 and a reading of R_H = -25 means, G_{ss} = 0.975% finer is given as:

$$N = \frac{G}{G-1} . \gamma_w . \frac{v}{W} . \frac{R_H}{10} \%$$

where, G = sp. gr. of soil solids

R_H = final corrected value of hydrometer

V = Total volume of soil suspension

W = weight of soil mass dissolved.

- **Corrections to Hydrometer Reading**

(i) Meniscus correction: (C_m)

Hydrometer reading is always corresponding to the upper level of meniscus.

Therefore, meniscus correction is always positive ($+C_m$).

(ii) Temperature correction: (C_t)

Hydrometers are generally calibrated at 27℃. If the test temperature is above the standard (27℃) the correction is added and, if below, it is subtracted.

(iii) Dispersing/Deflocculating agent correction: (C_d)

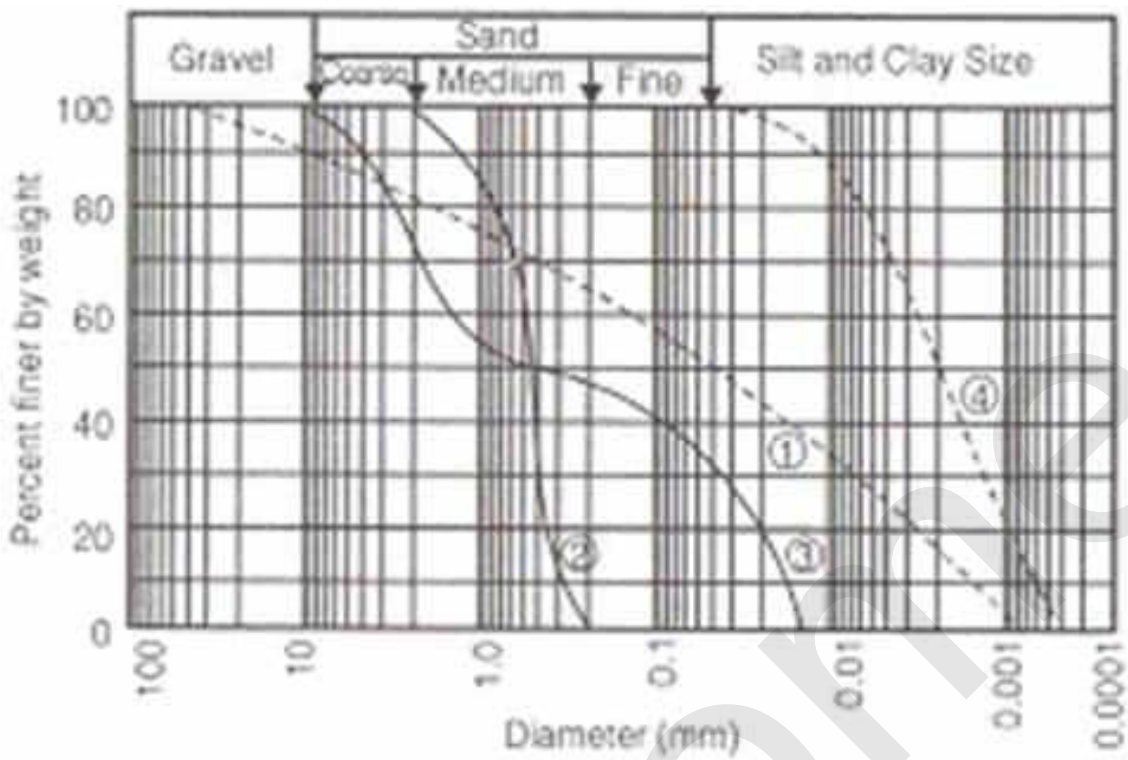
The correction due to rise in specific gravity of the suspension on account of the addition of the deflocculating agent is called Dispersing agent correction (C_d).

C_d is always negative.

The corrected hydrometer reading is given by

$$(R_H) = R_H + C_m \pm C_t - C_d$$

- **Grain Size Distribution Curves**



Curve-1: Well graded soil: good representation of grain sizes over a wide range and its gradation curve is smooth.

Curve-2: Poorly graded soil/ Uniform gradation:

It is either an excess or a deficiency of certain particle sizes or has most of the particles about the same size.

Curve-3: Gap graded soil: In this case some of the particle sizes are missing.

Curve-4: Predominantly coarse soil.

Curve-5: Predominantly fine soil.

The diameter D_{10} corresponds to 10% of the sample finer in weight on the Grain size distribution curve. This diameter D_{10} is called effective size.

Similarly, D_{30} and D_{60} are grain dia. (mm) corresponding to 30% fine and 60% finer.

The shape parameters related to these are:

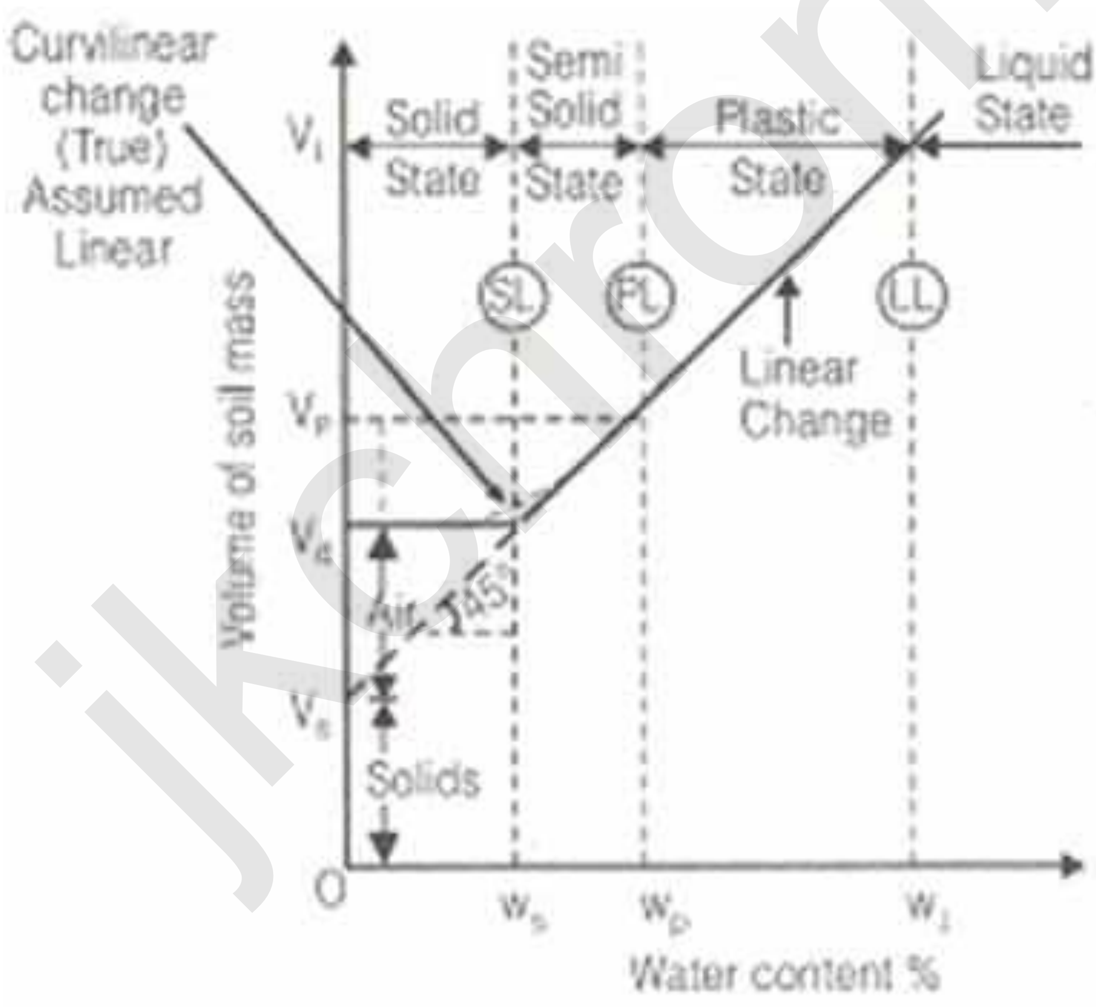
(A) Coefficient of Uniformity

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

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

- for a soil to be well graded:
  [1 < C_c < 3] and [Cu > 4] for gravels:
  [C_u > 6] for sands.
- Cu = 1 for uniform soils/poorly graded soils.

Consistency of clays: Atterberg limits



LL = W_l = liquid limit

PL = W_p = plastic limit

SL = W_s = Shrinkage limit

V_1 = Volume of soil mass at LL

V_p = Volume of soil mass at PL

V_d = Volume of soil mass at SL

V_s = Volume of solids

Plasticity Index (I_p): It is the range of moisture content over which a soil exhibits plasticity.

$I_p = W_L - W_p$

W_L = water content at LL

W_p = water content at PL

If PL ≥ LL, I_p is reported as zero.

Soil classification related to plasticity index:

I_p (%)	Soil Description
0	Non plastic
1 to 5	Slight plastic
5 to 10	Low plastic
10 to 20	Medium plastic
20 to 40	Highly plastic
> 40	Very highly plastic

Relative Consistency or Consistency – index (I_c): to study behaviour saturated fine grained soil at its natural water content

$$I_C = \frac{W_L - W_N}{I_p}$$

$$\left.\begin{aligned} \therefore For\ W_N = W_L \Rightarrow I_C = 0 \\ For\ W_N = W_P \Rightarrow I_C = 1 \end{aligned}\right\}$$

If $I_C < 0$, the natural water content of soil (w_N) is greater than w_L and the soil mass behaves like a liquid, but only upon disturbance.

If $I_C > 1$, soil is in semi solid state and will be very hard or stiff.

- **Liquidity Index (I_L)**

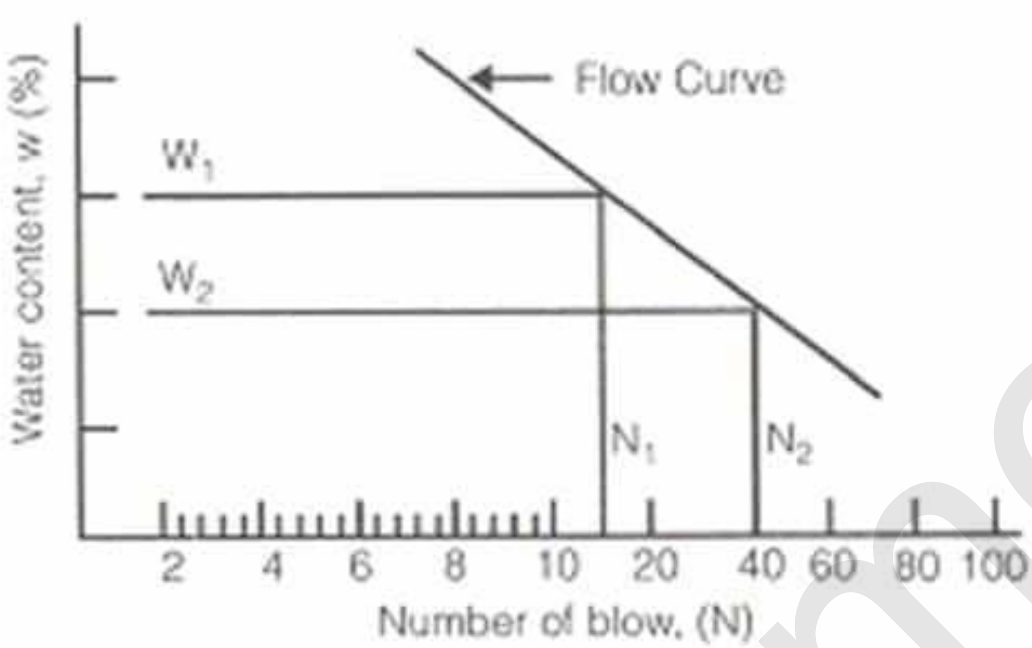
$$I_L = \frac{W_N - W_P}{I_P}$$

For a soil in plastic state I_L varies from 0 to 1.

Consist.	Description	I_C	I_L
Liquid	Liquid	<0	>1
Plastic	Very soft	0-0.25	0.75-1.00
	soft	0.25-0.5	0.50-0.75
	medium stiff	0.50-0.75	0.25-0.50
	stiff	0.75-1.00	0.0-0.25
Semi-solid	Very stiff or Hard	>1	< 0
Solid	Hard or very hard	>1	< 0

- **Flow Index (I_f)**

$$I_f = \frac{W_1 - W_2}{\log 10(N_2 / N_1)}$$



- **Toughness Index (I_t)**

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

For most of the soils: $0 < I_T < 3$

When $I_T < 1$, the soil is friable (easily crushed) at the plastic limit.

- **Shrinkage Ratio (SR)**

$$SR = \frac{\frac{V_1 - V_2}{V_d} \times 100}{w_1 - w_2}$$

where, V_1 = Volume of soil mass at water content w_1%.

V_2 = volume of soil mass at water content w_2%.

V_d = volume of dry soil mass

Now, at SL, $w_2 = w_s$ and $V_2 = V_d$

$$SR = \frac{\left(\frac{V_1 - V_d}{V_d} \times 100\right)}{(W_1 - W_s)}$$

∴

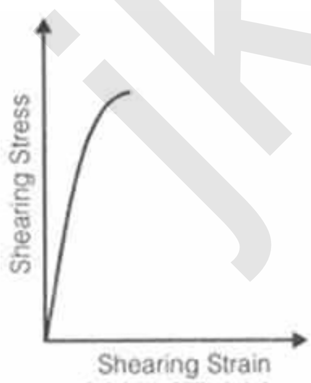
If w_1 & w_2 are expressed as ratio,

$$SR = \frac{(V_1 - V_2)/V_d}{W_1 - W_2} \; But, w_1 - w_2 = \frac{(V_1 - V_2)/\gamma_w}{W_s}$$

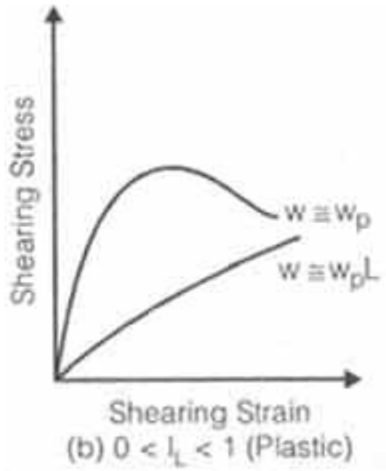
$$SR = \frac{W_s}{V_d} \cdot \frac{1}{\gamma_w} = \frac{\gamma_d}{\gamma_w}$$

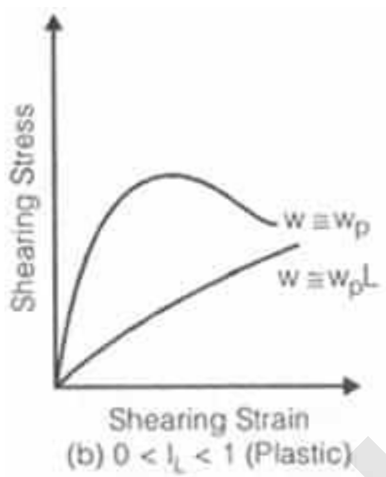
Properties	Relationship	Governing Parameters
Plasticity	∝	Plasticity Index
Better Foundation Material upon Remoulding	∝	Consistency Index
Compressibility	∝	Liquid Limit
Rate of loss in shear strength with increase in water content	∝	Flow Index
Strength of Plastic Limit	∝	Toughness Index

Stress-strain curve for different consistency states



(a) $I_L < 0$ (brittle)





- **Unconfined Compressive Strength (q_u)**

Defined as the load per unit area at which an unconfined prismatic or cylindrical specimen of standard dimensions of a soil fails in a simple compression test.

q_u = 2 x shear strength of a clay soil (under undrained condition).

q_u is related to consistency of clays as:

Consistency	Q_u (KN/m²)	(Kg/cm²)
Very soft	< 25	< 0.25
Soft	25-50	0.25-0.50
Medium	50-100	0.50-1.0
Stiff	100-200	1.0-2.0
Very stiff	200-400	2.0-4.0
Hard	> 400	> 4.0

- **Sensitivity (S_t):** It is defined as the ratio of the unconfined compressive strength of an undisturbed specimen of the soil to the unconfined compressive strength of a specimen of the same soil after remolding at unaltered water content.
- $$S_t = \frac{(q_u)undisturbed}{(q_u)remoulded}$$

$S_t \leq 1$: in case of stiff clay having cracks and fissures.

Soil classification based on sensitivity:

Sensitivity	Classification
1	No loss in strength on remoulding
2-4	Soil is normal sensitive
4-8	Sensitive
8-15	Extra-Sensitive
> 15	Quick

- **Thixotropy:** It is the property of certain clays by virtue of which they regain, if left alone for a time, a part of the strength lost due to remoulding, at unaltered moisture content.

Higher the sensitivity, larger the thixotropic hardening.

$$Activity\ of\ clay = \frac{\text{Plasticity index}}{\text{\% by weight finer } 2\ \mu}$$

Activity based classification of clays

Activity	Classification
< 0.75	Inactive
0.75-1.25	Normal
>1.25	Active

Volume change during swelling or shrinkage = (I_p and % clay) of Activity

Classification and Structure of Soil

Classification of Soils

USCS

It is adopted by IS code. It was given by A-Casagrande. It uses particle size distribution for coarse soils and plasticity for fine soils.

Classification of Soils

Object:

Sorting soils into groups showing similar behaviour based on index property, Generally used property are

Grand Size Distribution (ii) Plasticity

Depending upon intended use different classification systems have evolved:

1. Unified Soil Classification System (USCS)

Given by Casagrande

Intended for use in Airfield, Construction

Major Soils Groups	Soil Type	Prefix	Classification Parameters
Coarse Grained	Gravel	G	Grain size distribution
	Sand	S	
Fine Grained	Silt	M	Plasticity characteristics
	Clay	C	
Organic		O	Percentage of organic matter and particles of decomposed vegetation
Peat		Pt	

Note: ISCS is a modified USCS system.

2. AASHTO Classification System

For Highway Construction

- Soil Classified into 8 groups divided into subgroups based on group index. GI.

GI value ranges between

- O(Good Subgrade Material) to 20 (Poor Subgrade Material)

3. Indian Standard Soil Classification System (ISSCS) % Fineness:

In the **Indian Standard Soil Classification System (ISSCS),** soils are classified into groups according to size, and the groups are further divided into coarse, medium and fine sub-groups.

The grain-size range is used as the basis for grouping soil particles into boulder, cobble, gravel, sand, silt or clay.

Very coarse soils	Boulder size		> 300 mm
	Cobble size		80 - 300 mm
Coarse soils	Gravel size (G)	*Coarse*	20 - 80 mm
		Fine	4.75 - 20 mm
	Sand size (S)	*Coarse*	2 - 4.75 mm
		Medium	0.425 - 2 mm
		Fine	0.075 - 0.425 mm
Fine soils	Silt size (M)		0.002 - 0.075 mm
	Clay size (C)		< 0.002 mm

Gravel, sand, silt, and clay are represented by **group symbols G, S, M, and C** respectively.

Physical weathering produces very coarse and coarse soils. Chemical weathering produce generally fine soils.

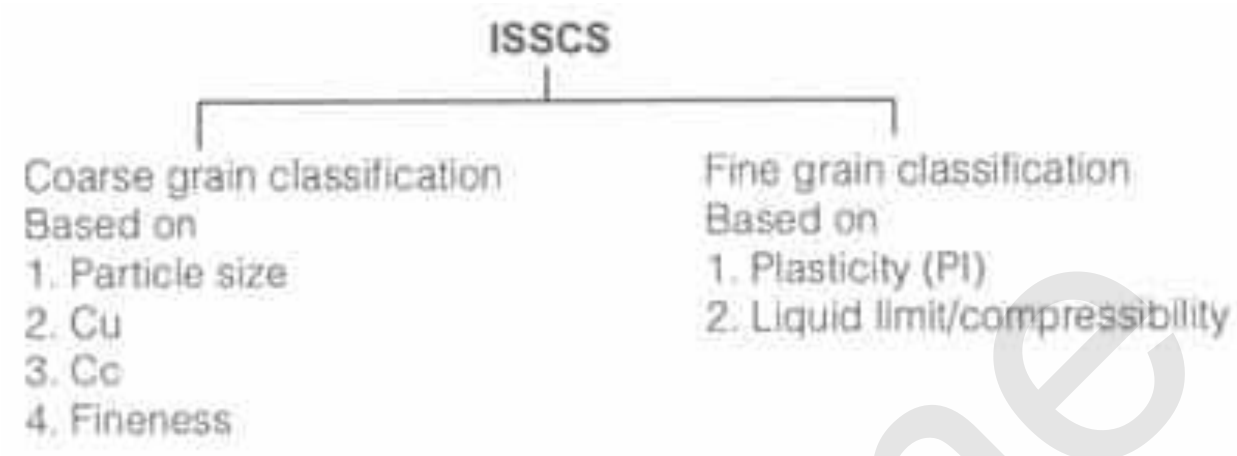
- % of soil passing through the 75μ sieve.

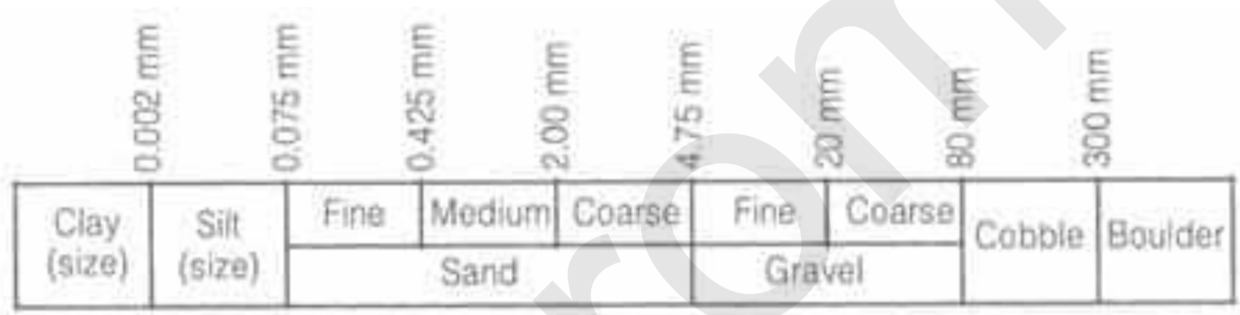
1. % Fineness < 50 % = Soil contain mainly
   Coarse Grained fraction otherwise Fine grained fraction
2. Fraction retained over the 75μ is undergone with plasticity studies, i.e. W_L + I_P identifies.

Classification chart for Coarse Grained Soil
i.e. when % fineness, Less than 50%

	Gravel (G)		Sand (S)		Remarks
	Well graded (W) (If) $C_u > 4$, $C_c = 1$ to 3	Poorly Graded (P) (else)	Well graded (W) (If) $C_u > 6$, $C_c = 1$ to 3	Poorly Graded (P) (else)	
(i) p < 5%	GW	GP	SW	SP	Gradation only govern the properties
(ii) p – 5 to 12%	GW – GC or GW – GM	GP – GC or GP – GM	SW – SC or SW – SM	GP – GC or GP – GH	Dual symbol check presence of either clay (C) or silt (M), using plasticity chart
(ii) p > 12 %	$I_p < 7\%$ GM	$I_p > 7\%$ GC	$I_p < 4\%$ SM	$I_p > 7\%$ SC	Presence of fine particles dominate the soil characteristics
	For I_p 4 – 7%, Dual Symbols used				

where P - % passing through 75μ or micron sieve.





Classification of soils based on grain size.

- **On the basis of fineness, coarse grain soils are further classified**

Case-I: Well fineness is **< 5%**

1. GW – Well graded gravel
   Cu . 4
   1 < Cc < 3
   Fineness < 5%
2. GP – Poorly graded gravel
   Above values of Cu and Cc are not satisfied.
3. SW – Well graded sand
   Cu > 6
   1 < Cc <3
4. SP – Poorly graded sand/uniformly graded sand Cu and Cc are not in range.

Case-II: If fineness is **5% to 12%** the dual symbol are used.

1. GW – GC well graded gravel containing clay.
   Fineness – 5 to 12%
   Clay > Silt
   Gravel > Sand
   Cu > 4; 1 ≤ Cc ≤ 3
2. GW – GM Well graded gravel containing silt
   Cu > 4
   1 ≤ Cu ≤ 3
   Silt > Clay
   Gravel > Sand
3. SW – SC Well graded sand containing clay
   Sand > Gravel
   Clay > Silt
   Cu > 6
   1 ≤ Cc ≤ 12%
4. SW – SM Well graded sand containing silt
   Sand > Gravel
   Silt > Silt
   Cu > Clay
   1 ≤ Cc ≤ 3
   Fineness 5 to 12%

For poorly graded soils like GP-GC, GP, GM, SP-SC SP-SM the values of Cu and Cc are not satisfied.

Case-III: When fineness is more than 12%

GC: Clayey gravel

Gravel > Sand

Clay > Silt I_p > 7%

GM: Silty gravel

Sand < Gravel

Clay < Silt I_p < 4%

SC: Clayey silt

Sand > Gravel

Silti < Clay $I_p > 7\%$

SM: Silty sand

Sand > Gravel

Silt > Clay $I_p < 4\%$

Note: For I_P between 4 and 7, Dual Symbols are used.

Classification of Fine Soils

1. Silts (0.002 mm to 0.075 mm)
    - Coarse 0.02 to 0.075 mm
    - Medium 0.01 to 0.02 mm
    - Fine 0.002 to 0.01 mm
2. Clay → <0.002 mm

(i) Low plastic soils (LL < 35%)

CL → Low plastic inorganic clay

ML → Low plastic silt

OL → Low plastic organic clay

(ii) Medium plastic soils (35% < 50%)

CI → Medium plastic inorganic clay

MI → Medium plastic silt

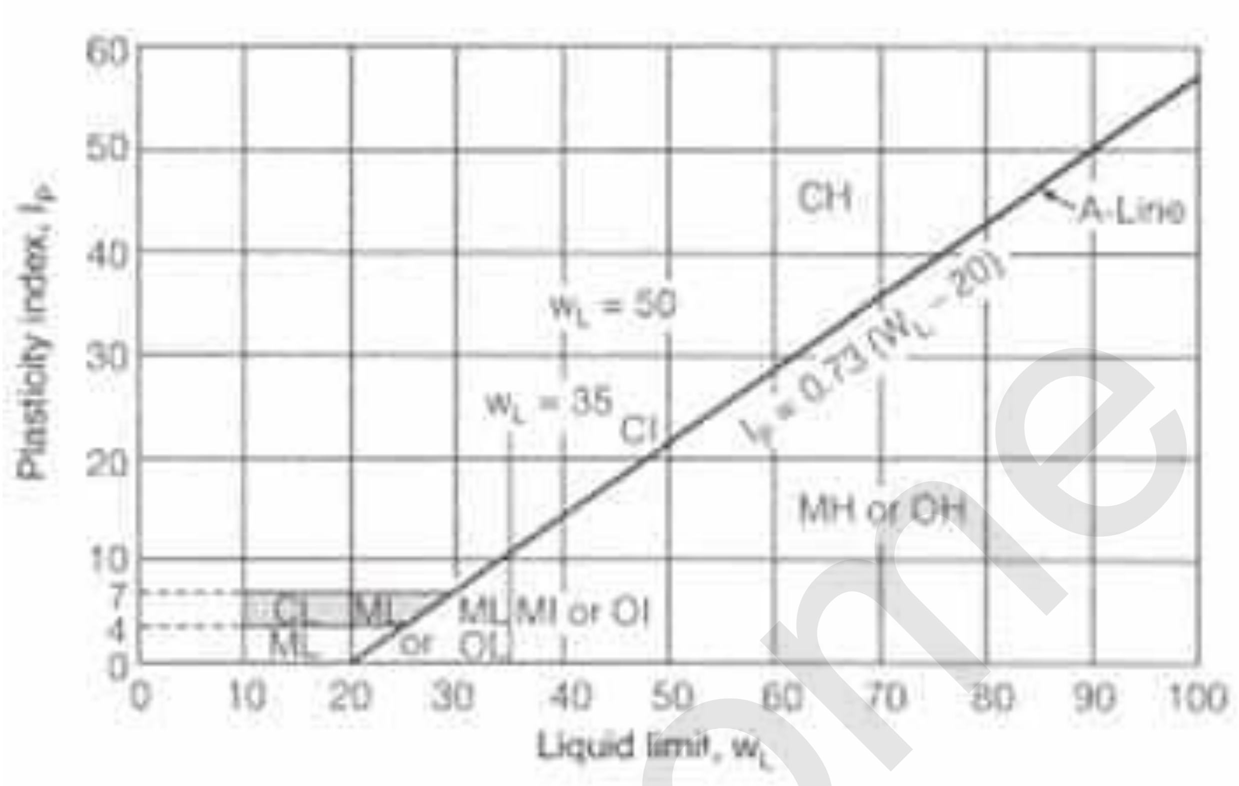
OI → Medium plastic organic clay

(iii) High plastic soils (LL > 50%)

CH → High plastic inorganic clay

MH → High plastic silt

OH → High plastic organic clay

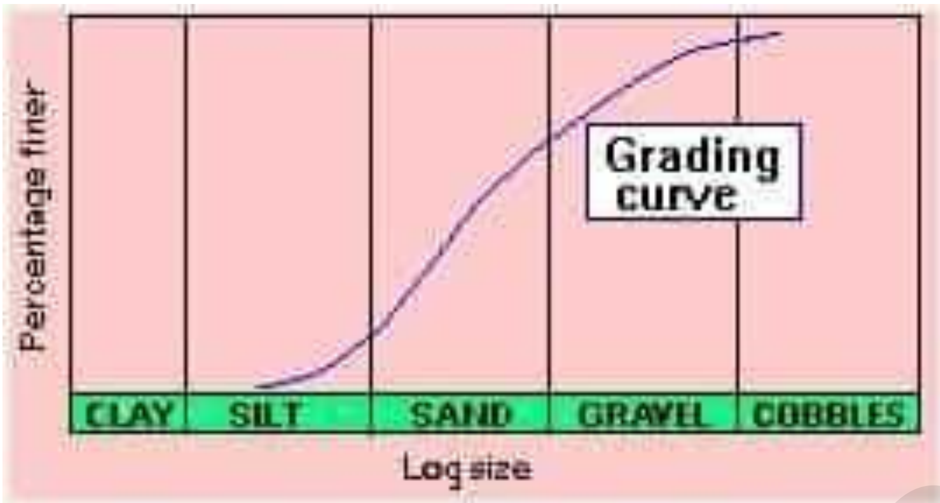


Equation of A-line $I_P = 0.73 (W_L - 20)$

Equation of U-line $I_P = 0.9 (W_L - 8)$

Grain-Size Distribution Curve

The size distribution curves, as obtained from coarse and fine grained portions, can be combined to form one complete **grain-size distribution curve** (also known as **grading curve**). A typical grading curve is shown.

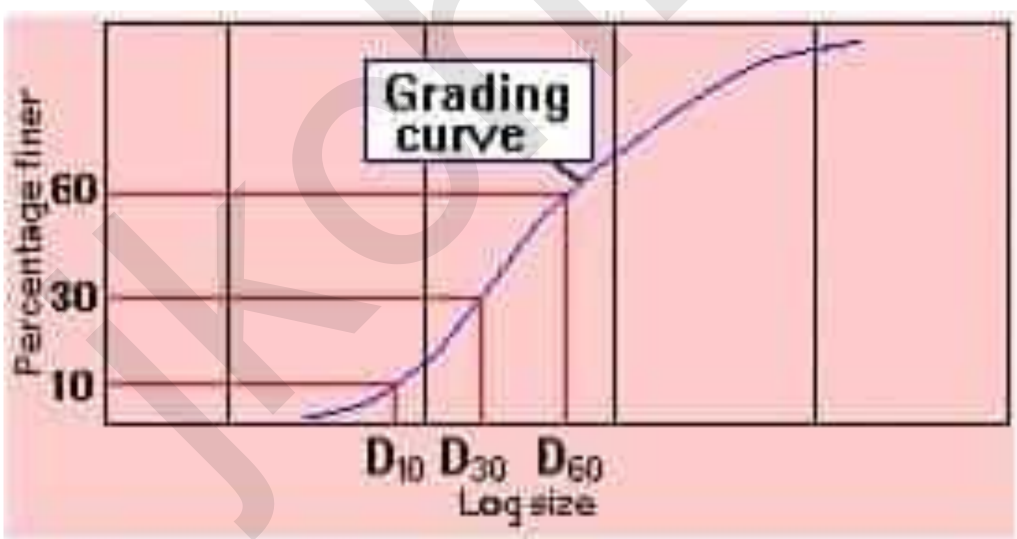


From the complete grain-size distribution curve, useful information can be obtained such as:

1. **Grading characteristics**, which indicate the uniformity and range in grain-size distribution.

2. **Percentages** (or **fractions)** of gravel, sand, silt and clay-size.

A grading curve is a useful aid to soil description. The geometric properties of a grading curve are called **grading characteristics**.



To obtain the grading characteristics, three points are located first on the grading curve.

D_{60} = size at 60% finer by weight
D_{30} = size at 30% finer by weight
D_{10} = size at 10% finer by weight

The grading characteristics are then determined as follows:

1. **Effective size** = D_{10}
2. **Uniformity coefficient**, $C_u = \frac{D_{60}}{D_{10}}$
3. **Curvature coefficient**, $C_c = \frac{(D_{30})^2}{D_{60} D_{10}}$

Both $C_u$and C_c will be 1 for a single-sized soil.

C_u **> 5** indicates a **well-graded soil**, i.e. a soil which has a distribution of particles over a wide size range.

C_c **between 1 and 3** also indicates a well-graded soil.

C_u **< 3** indicates a **uniform soil**, i.e. a soil which has a very narrow particle size range.

Structure of Soil

A soil particle may be a mineral or a rock fragment. A mineral is a chemical compound formed in nature during a geological process, whereas a rock fragment has a combination of one or more minerals. Based on the nature of atoms, minerals are classified as silicates, aluminates, oxides, carbonates and phosphates.

Out of these, silicate minerals are the most important as they influence the properties of clay soils. Different arrangements of atoms in the silicate minerals give rise to different silicate structures.

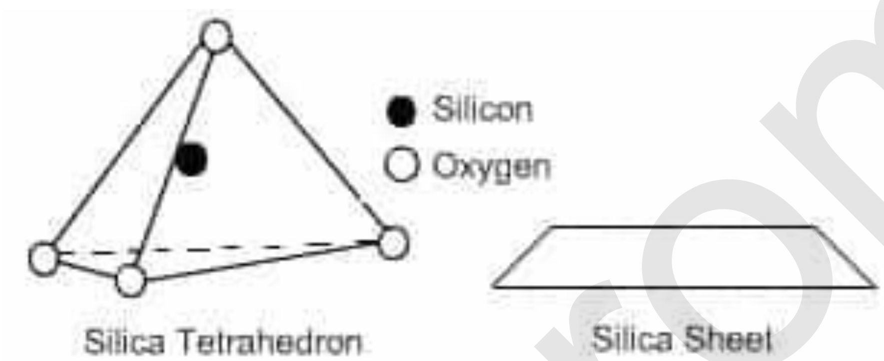
Basic Structural Units
Soil minerals are formed from two basic structural units: tetrahedral and octahedral. Considering the valencies of the atoms forming the units, it is clear that the units are not electrically neutral and as such do not exist as single units.

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

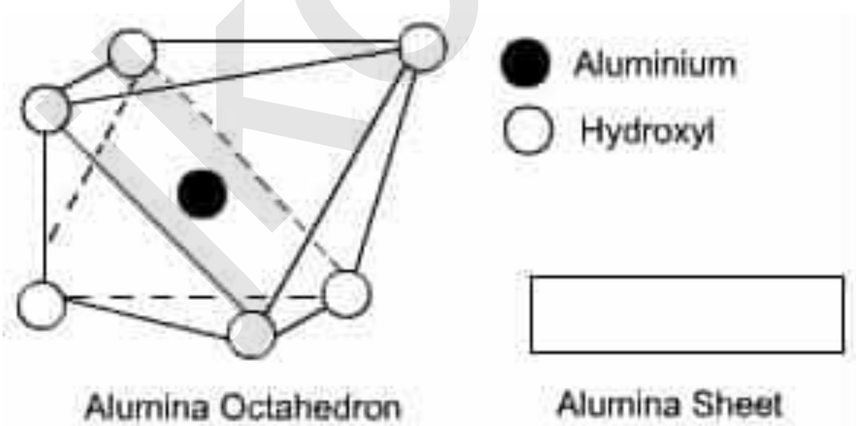
Isomorphous substitution is the replacement of the central atom of the tetrahedral or octahedral unit by another atom during the formation of the sheets.

The sheets then combine to form various two-layer or three-layer sheet minerals. As the basic units of clay minerals are sheet-like structures, the particle formed from stacking of the basic units is also plate-like. As a result, the surface area per unit mass becomes very large.

- A tetrahedral unit consists of a central silicon atom that is surrounded by four oxygen atoms located at the corners of a tetrahedron. A combination of tetrahedrons forms a ***silica sheet.***



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



- **Montmorillonite Mineral**
  The bonding between the three-layer units is by van der Waals forces. This

bonding is very weak and water can enter easily. Thus, this mineral can imbibe a large quantity of water causing swelling. During dry weather, there will be shrinkage.

- **Illite Mineral**
  Illite consists of the basic montmorillonite units but are bonded by **secondary valence forces** and **potassium ions**, as shown. There is about 20% replacement of aluminium with silicon in the gibbsite sheet due to ***isomorphous substitution***. This mineral is very stable and does not swell or shrink.
- **Kaolinite Mineral**
  A basic kaolinite unit is a two-layer unit that is formed by stacking a gibbsite sheet on a silica sheet. These basic units are then stacked one on top of the other to form a lattice of the mineral. The units are held together by hydrogen bonds. The strong bonding does not permit water to enter the lattice. Thus, kaolinite minerals are stable and do not expand under saturation.

  Kaolinite is the most abundant constituent of residual clay deposits.

	Clay mineral	**Properties**
1.	Kaolinite mineral	Hydrogen bond is there which is strongest bond. Ex Chine clay
2.	Illite mineral	Ionic bond. Medium change in volume due to moisture change.
3.	Mont morillonite	Water bond which is weakest bond. Max change in volume due to moisture change. Ex. Black soils & Bentonite soils

Compaction of Soil & Stress distribution in soils

Compaction of Soil

Compaction is the application of mechanical energy to soil so as to rearrange its particles and reduce the void ratio.

It is applied to improve the properties of existing soil or in the process of placing fill such as in the construction of embankments, road bases, runways, earth dams, and reinforced earth walls. Compaction is also used to prepare a level surface during construction of buildings. There is usually no change in the water content and in the size of the individual soil particles.

The objectives of compaction are:

- To increase soil shear strength and therefore its bearing capacity.
- To reduce subsequent settlement under working loads.
- To reduce soil permeability making it more difficult for water to flow through.

Laboratory Compaction

The variation in compaction with water content and compactive effort is first determined in the laboratory. There are several tests with standard procedures such as:

- Indian Standard Light Compaction Test (similar to Standard **Proctor** Test/Light Compaction Test)
- Indian Standard Heavy Compaction Test (similar to Modified **Proctor** Test/Heavy Compaction Test)

Standard proctor test (Light compaction test)	**Modified proctor test (Heavy compaction test)**
Volume of mould 942cc	Volume of mould 942 cc
No. of layers -3	No. of layers -5
No. of blows per layer - 25	No. of blows per layer -25
Height of free fall -304.8 mm (12 inches)	Height of free fall -457.2 mm (18 inches)
Wt. of hammer -2.495 kg (5.5 /b)	Wt. of hammer -4.54 kg (10 /b)

- **Indian Standard Light Compaction Test**

Soil is compacted into a 1000 cm^3 mould in 3 equal layers, each layer receiving **25 blows of a 2.6 kg** rammer dropped **from a height of 310 mm** above the soil. The compaction is repeated at various moisture contents.

- **Indian Standard Heavy Compaction Test**

It was found that the Light Compaction Test (Standard Test) could not reproduce the densities measured in the field under heavier loading conditions, and this led to the development of the Heavy Compaction Test (Modified Test). The equipment and procedure are essentially the same as that used for the Standard Test except that the soil is compacted in 5 layers, each layer also receiving **25 blows**. The same mould is also used. To provide the increased compactive effort, a heavier rammer of **4.9 kg** and a greater drop **height of 450 mm** are used.

Compactive energy applied per unit

$$Volume = \frac{wH_n N}{V}$$

Indian standard light compaction	Indian standard heavy compaction
V – Volume of mould 1000 cc	Volume of mould 1000 cc
H – Height of free fall 310 mm	Height of free fall 450 mm
W – Wt. of hammer 2.6 kg	Wt. of hammer 4.9 kg
N – No. of layers 3	No. of layers 5
N – Blows per layer 25	Blows per layer 25

- The ratio of total energy given in heavy compaction test to that given in light compaction test

$$= \frac{4.9 \times g \times (5 \times 25) \times 450}{2.6 \times g \times (3 \times 25) \times 310} = 4.5$$

Dry Density - Water Content Relationship

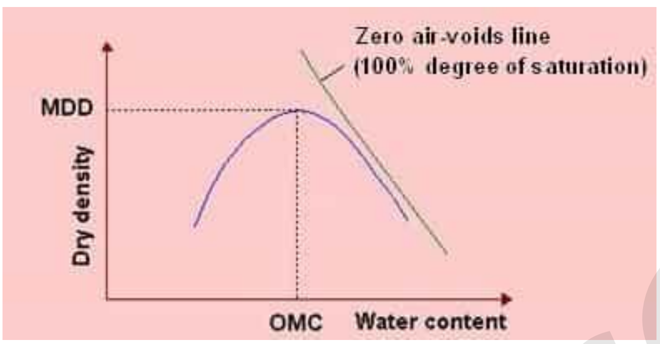
- To assess the degree of compaction, it is necessary to use the dry unit weight, which is an indicator of compactness of solid soil particles in a given volume.
- Laboratory testing is meant to establish the maximum dry density that can be attained for a given soil with a standard amount of compactive effort.

  In the test, the dry density cannot be determined directly, and as such the bulk density and the moisture content are obtained first to calculate the dry density as

$$\gamma_d = \frac{\gamma_t}{1+w}$$,

where γ_d = bulk density, and w = water content.

- A series of samples of the soil are compacted at different water contents, and a curve is drawn with axes of dry density and water content. The resulting plot usually has a distinct peak as shown. Such inverted “V” curves are obtained for **cohesive soils** (or soils with fines), and are known as compaction curves.



- Dry density can be related to water content and degree of saturation (S) as

$$\gamma_d = \frac{G_s \cdot \gamma_w}{1+e} = \frac{G_s \cdot \gamma_w}{1+\frac{wG_s}{S}}$$

Thus, it can be visualized that an increase of dry density means a decrease of voids ratio and a more compact soil.

Similarly, dry density can be related to percentage air voids (n_a) as

$$\gamma_d = \frac{(1-n_a)G_s \cdot \gamma_w}{1+wG_s}$$

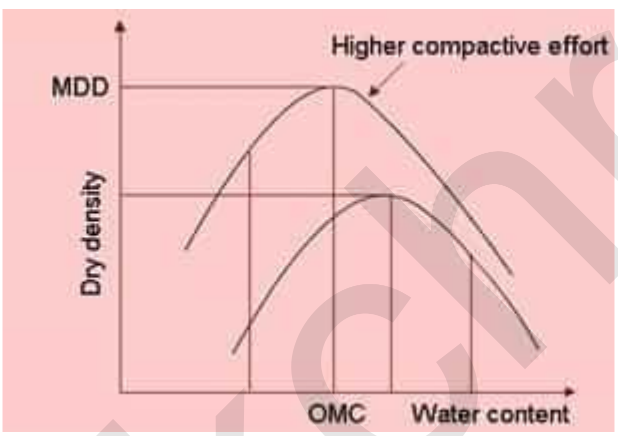
Relation between moisture content and dry unit weight for a saturated soil is the **zero air-voids line.** It is not feasible to expel air completely by compaction, no matter how much compactive effort is used and in whatever manner.

Effect of Increasing Water Content

- As water is added to a soil at low moisture contents, it becomes easier for the particles to move past one another during the application of compacting force. The particles come closer, the voids are reduced and this causes the dry density to increase. As the water content increases, the soil particles develop larger water films around them.
- This increase in dry density continues till a stage is reached where the water starts occupying the space that could have been occupied by the soil grains. Thus the water at this stage hinders the closer packing of grains and reduces the dry unit weight. The **maximum dry density (MDD)** occurs at an **optimum water content (OMC),** and their values can be obtained from the plot.

Effect of Increasing Compactive Effort

- The effect of increasing compactive effort is shown. Different curves are obtained for different compactive efforts. A greater compactive effort reduces the optimum moisture content and increases the maximum dry density.



- An increase in compactive effort produces a very large increase in dry density for soil when it is compacted at water contents drier than the optimum moisture content. It should be noted that for moisture contents greater than the optimum, the use of heavier compaction effort will have only a small effect on increasing dry unit weights.

It can be seen that the compaction curve is not a unique soil characteristic. It depends on the compaction effort. For this reason, it is important to specify the compaction procedure (light or heavy) when giving values of MDD and OMC.

Factors Affecting Compaction

The factors that influence the achieved degree of compaction in the laboratory are:

- Plasticity of the soil
- Water content
- Compactive effort

Compaction of Cohesionless Soils

For **cohesionless soils** (or soils without any fines), the standard compaction tests are difficult to perform. For compaction, application of vibrations is the most effective method. Watering is another method. To achieve maximum dry density, they can be compacted either in a dry state or in a saturated state.

- For these soil types, it is usual to specify a magnitude of **relative density (I_D)** that must be achieved. If **e** is the current void ratio or g_d is the current dry density, the relative density is usually defined in percentage as

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

or

$$I_D = \frac{\gamma_{dmax}(\gamma_d - \gamma_{dmin})}{\gamma_d(\gamma_{dmax} - \gamma_{dmin})} \times 100$$

where $\boldsymbol{e_{max}}$ and $\boldsymbol{e_{min}}$ are the maximum and minimum void ratios that can be determined from standard tests in the laboratory, and $\boldsymbol{g_{dmin}}$ and $\boldsymbol{g_{dmax}}$ are the respective minimum and maximum dry densities

On the basis of relative density, sands and gravels can be grouped into different categories:

Relative density (%)	Classification
< 15	Very loose
15-35	Loose
35-65	Medium
65-85	Dense

> 85	Very dense

It is not possible to determine the dry density from the value of the relative density. The reason is that the values of the maximum and minimum dry densities (or void ratios) depend on the gradation and angularity of the soil grains.

Engineering Behaviour of Compacted Soils

The water content of a compacted soil is expressed with reference to the OMC. Thus, soils are said to be compacted **dry of optimum** or **wet of optimum** (i.e. on **the dry side** or **wet side** of OMC). The structure of a compacted soil is not similar on both sides even when the dry density is the same, and this difference has a strong influence on the engineering characteristics.

1. **Soil Structure**
   For a given compactive effort, soils have a flocculated structure on the dry side (i.e. soil particles are oriented randomly), whereas they have a dispersed structure on the wet side (i.e. particles are more oriented in a parallel arrangement perpendicular to the direction of applied stress). This is due to the well-developed adsorbed water layer (water film) surrounding each particle on the wet side.
   

2. **Swelling**
   Due to a higher water deficiency and partially developed water films in the dry side, when given access to water, the soil will soak in much more water and then swell more.
3. **Shrinkage**
   During drying, soils compacted in the wet side tend to show more shrinkage than those compacted in the dry side. In the wet side, the more orderly orientation of particles allows them to pack more efficiently.
4. **Construction Pore Water Pressure**
   The compaction of man-made deposits proceeds layer by layer, and pore water pressures are induced in the previous layers. Soils compacted wet of optimum will have higher pore water pressures compared to soils

compacted dry of optimum, which have initially negative pore water pressure.

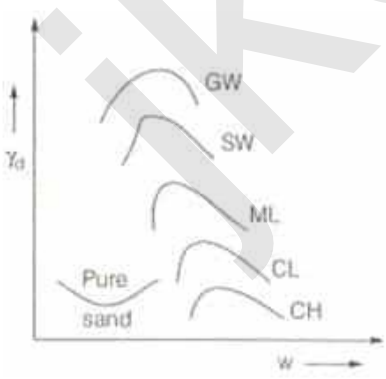
5. **Permeability**
   The randomly oriented soil in the dry side exhibits the same permeability in all directions, whereas the dispersed soil in the wet side is more permeable along particle orientation than across particle orientation.
6. **Compressibility**

At low applied stresses, the dry compacted soil is less compressible on account of its truss-like arrangement of particles whereas the wet compacted soil is more compressible.

The stress-strain curve of the dry compacted soil rises to a peak and drops down when the flocculated structure collapses. At high applied stresses, the initially flocculated and the initially dispersed soil samples will have similar structures, and they exhibit similar compressibility and strength.

Some extra details about compaction -

	Type of Equipment	Suitability for soil type	Nature of project
1	Rammers or Tampers	All soils	In confined areas such as fills behind retaining walls, basement walls etc. Trench fills.
2	Smooth wheeled rollers	Crushed rocks, gravels sands	Road construction
3	Pneumatic tyred rollers	Sand, gravels silts, clayey soils	Base, sub-base and embankment compaction for highways, air fields etc. Earth dams.
4	Sheep foot Rollers	Clayey soils	Core of earth dams.
5	Vibratory Rollers	Sands	Embankment for oil storage tanks etc.



1. Coarse grained well graded – Higher γ_d

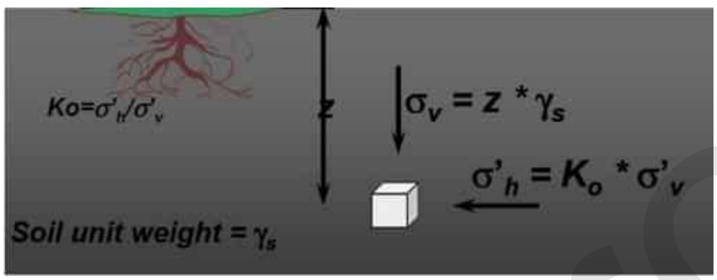
2. In clays with higher plasticity - γ_d decrease
3. V shape due to bulking of pure sand

Stress Distribution in The Soil

At a point within a soil mass, stresses will be developed as a result of the soil lying above the point and by any structural or other loading imposed onto that soil mass .

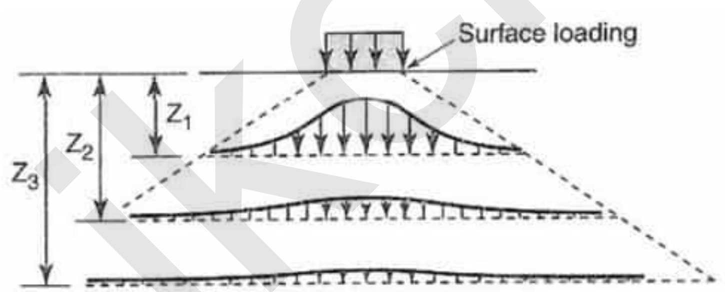
Stress in the soil may be caused by:

1. Self weight of soil
2. Applied load on soil



Finitely loaded area

If the surface loading area is finite (point, circular, strip, rectangular, square), the vertical stress increment in the subsoil decreases with increase in the depth and the distance from the surface loading area.



Methods have been developed to estimate the vertical stress increment in sub-soil considering the shape of the surface loading area.

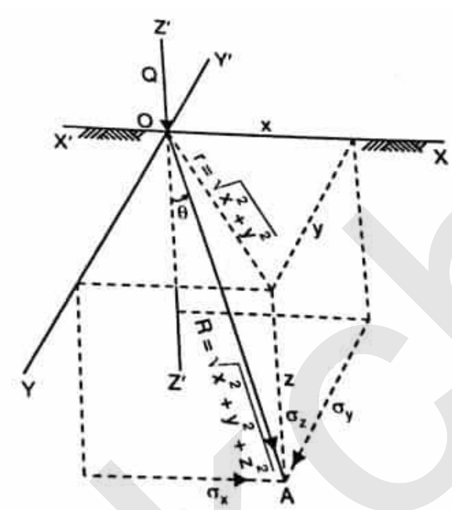
Boussinesq's Theory

Point Load

A point load or a Concentrated load is, strictly speaking, hypothetical in nature, consideration of it serves a useful purpose in arriving at the solutions for more complex loadings in practice.

Assumptions made by Boussinesq.

(i) The soil medium is an elastic, homogeneous, isotropic and semi-infinite medium, which infinitely in all directions from a level surface.
(ii) The medium obeys Hookes law.
(iii) The self-weight of the soil is ignored.
(iv) The soil is initially unstressed
(v) The change in volume of the soil upon application of the loads on to it is neglected.
(vi) The top surface of the medium is free of shear stress and is subjected to only the point load at a specified location.
(vii) Continuity of stress is considered to exist in the medium.
(viii) The stresses are distributed symmetrically with respect to z axis.



The Boussinesq equations are as follows :

$$\sigma_z = \frac{3Q}{2\pi}\frac{Z^3}{R^5}$$

$$= \frac{3Q}{2\pi}\frac{\cos^5\theta}{Z^2}$$

$$= \frac{3Q}{2\pi}\frac{Z^3}{(r^2+z^2)^{5/2}}$$

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

$$\sigma_x = \frac{Q}{2\pi}\left[\frac{3x^2Z}{R^5} - (1-2v)\left\{\frac{x^2-y^2}{Rr^2(R+Z)} + \frac{y^2Z}{R^3r^2}\right\}\right]$$

$$\sigma_y = \frac{Q}{2\pi}\left[\frac{3y^2Z}{R^5} - (1-2v)\left\{\frac{y^2-x^2}{Rr^2(R+Z)} + \frac{x^2Z}{R^3r^2}\right\}\right]$$

$$\sigma_\theta = \frac{3Q}{2\pi}\frac{\cos\theta}{R^2}$$

$$\sigma_r = \frac{Q}{2\pi}\left[\frac{3zr^2}{R^2} - \frac{(1-2v)}{R(R+Z)}\right]$$

$$\tau_{rz} = (3QrZ^2)/(2\pi R^5)$$

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

Equations (1) may be rewritten as

$$\sigma_z = K_B\frac{Q}{Z^2}$$

where K_B, Boussinesq's influence factor is given by :

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

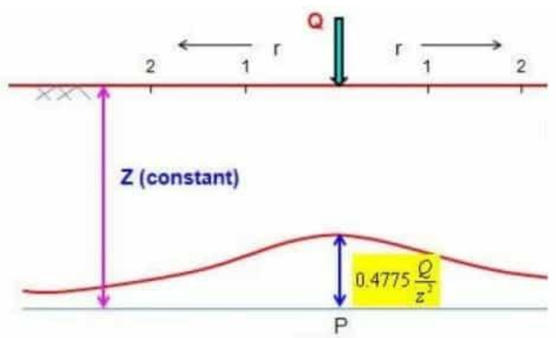
This intensity of vertical stress directly below the point load, on its axis of loading (r=0) is given by:

$$\sigma_z = \frac{0.4775Q}{Z^2}$$

The vertical stress on a horizontal plane at depth „Z" is given by

$$\sigma_z = K_B \frac{Q}{Z^2}$$

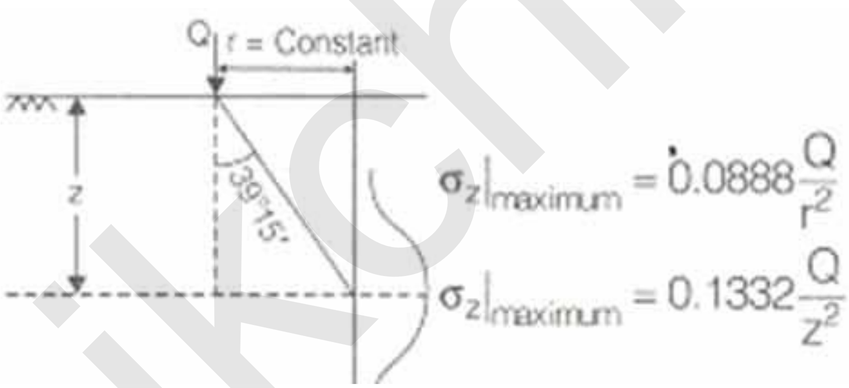
Z being a specified depth.



Boussinesq's Result

$$\sigma_z \big|_{\max} = 0.0888 \frac{Q}{r^2}$$

$$\sigma_z \big|_{\max} = 0.1332 \frac{Q^2}{2^2}$$



Westergaard's Theory

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

(ii) $\sigma_z = k_W . \frac{Q}{z^2}$

(iii) $k_w |_{max} = 0.3183$

Westergaard's Results

(i) Vertical Stresss due to Live Loads

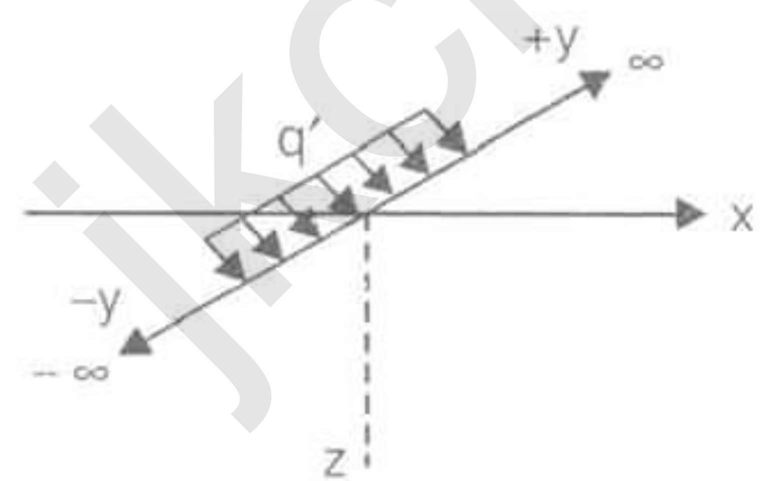
$$\sigma_z = \frac{2q'}{\pi z}\left[\frac{1}{1+\frac{X^2}{z^2}}\right]^2$$

where, σ_z = Vertical stress of any point having coordinate (x, z)

Load intensity = q'/m

at X = 0

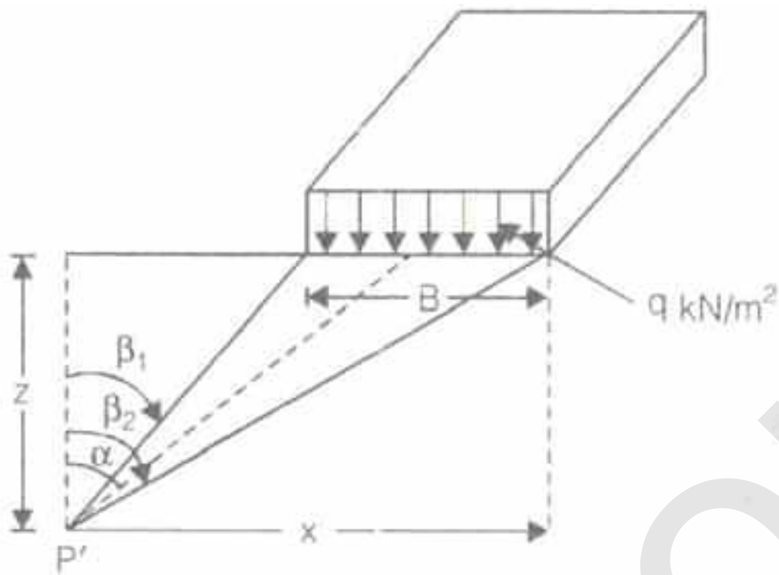
$$\sigma_z = \frac{2q'}{\pi z}$$



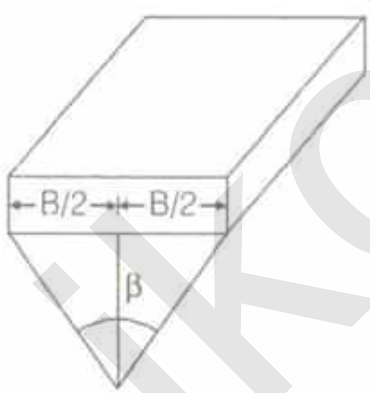
(ii) Vertical Stress due to Strip Loading

$$\sigma_z = \frac{2q}{\pi}\left(\frac{X}{B}\alpha - \frac{\sin 2\beta}{2}\right)$$

where, σ_z = Vertical stress at point 'p'



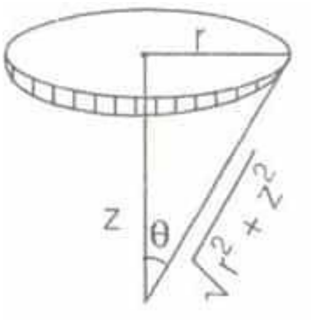
(iii) $\sigma_Z = \frac{q}{\pi}[\beta + \sin\beta]$



(iv) Vertical stress below uniform load acting on a circular area.

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

where, $\cos\theta = \frac{z}{\sqrt{r^2+z^2}}$



Newmark's Chart Method (Uniform Load on irregular Areas)

- Newmark (1942) constructed influence chart, based on the Boussinesq solution to determine the vertical stress increase at any point below an area of any shape carrying uniform pressure.
- This method is applicable to semi-infinite, homogeneous, isotropic and elastic soil mass. It is not applicable for layered structure.
- The greatest advantage of this method is that it can be applied for a uniformly distributed area of an irregular shape.
- Chart consists of influence areas which have an influence value of 0.005 per unit pressure.
- Position the loaded area on the chart such that the point at which the vertical stress required is at the centre of the chart.
- Newmark's chart is made of concentric circles and radial lines. Normally there are 10 concentric circles and 20 radial lines.

No. of concentric circle = 10

No. of radial lines = 20

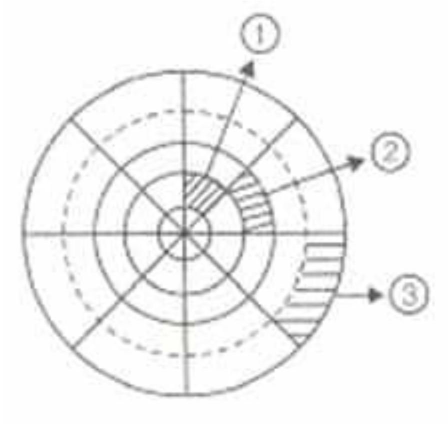
Influence of area (1) = Influence of area (2) = Influence of area (3)

Influence of each area

$$= \frac{1}{\text{Total no. of sectoral area}} = 0.005$$

$\sigma_z = 0.005 q N_A$

where, N_A = Total number of sectorial area of Newmark's chart.



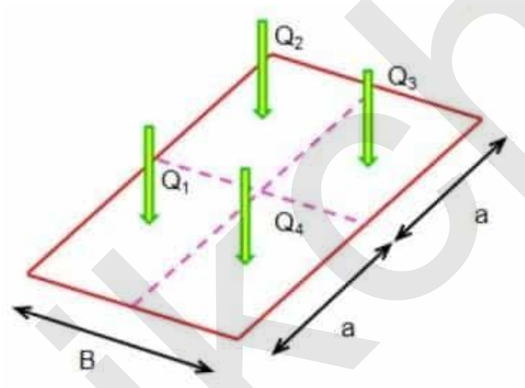
Approximate method

(i) Equivalent Load Method

$$\sigma_Z = \sigma_{Z_1} + \sigma_{Z_2} + \sigma_{Z_3} + \ldots$$

where,

$$\sigma_{z_1} = k_{B_1} \frac{Q_1}{Z^2} \; \sigma_{z_2} = k_{B_2} . \frac{Q_2^2}{Z^2} \ldots$$



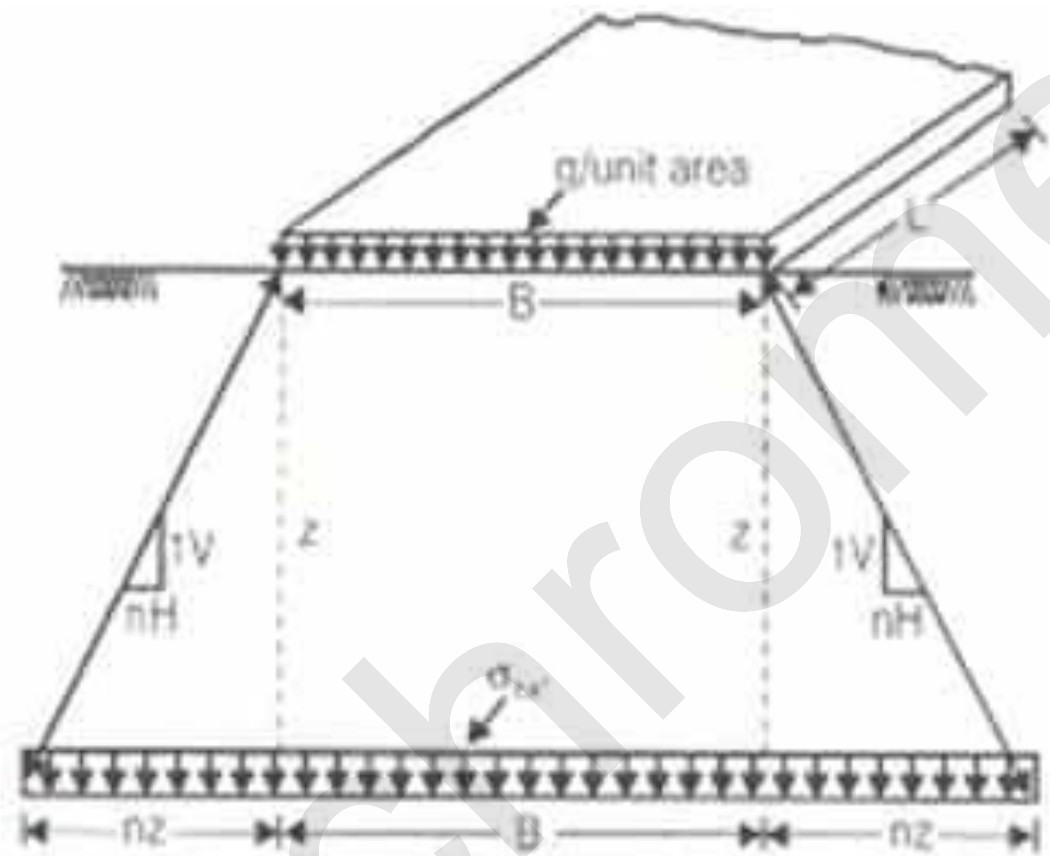
(ii) Trapezoidal Method

σ_z at depth $'z' = \dfrac{q(B \times L)}{(B + 2\eta z)(L + 2\eta z)}$

For 1H : 1 V

$$\sigma_z = \frac{q(B \times L)}{(B+2z)(L+2z)}$$

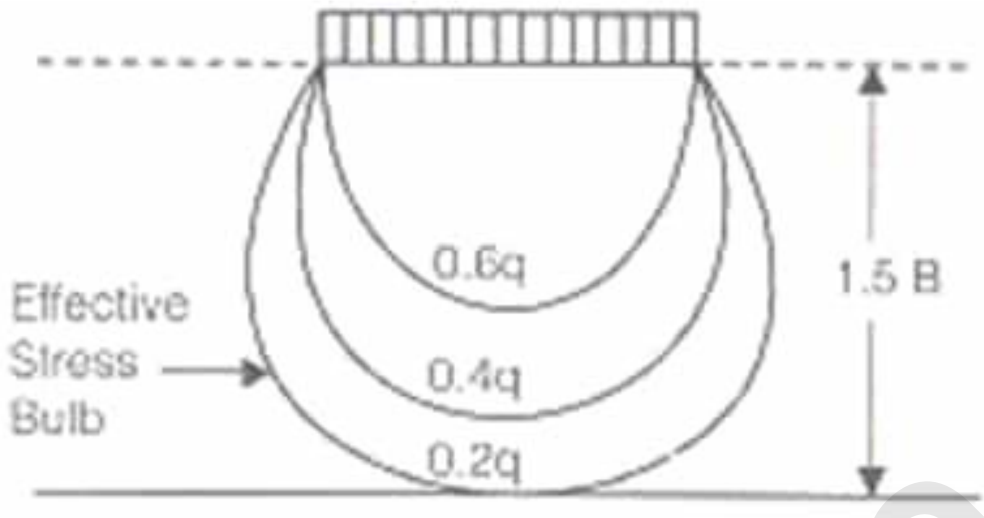
$$\sigma_z = \frac{q(B \times L)}{(B+4z)(L+4z)}$$



(iii) Stress Isobar Method:

Area bounded by 0.2q stress isobar is considered to be stressed by vertical stress on loading.

0.2q = 20% Stress Isobar



Q. A concentrated load of 22.5 KN acts on the surface of a homogeneous soil mass of large extent. Find the stress intensity at a depth of 15 metres and (i) directly under the load, and (ii) at a horizontal distance of 7.5 metres. Use Boussinesq's equations.

A : According to Boussinesq's theory,

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

(i) Directly under the load :

$r = 0;\ \therefore \frac{r}{z} = 0$

$z = 15\text{m},\ Q = 22.5\text{ KN}$

$$\therefore \sigma_z = \frac{22.5}{15\times 15}\cdot\frac{\left(\frac{3}{2\pi}\right)}{\left((1+0)^{5/2}\right)}$$

$= 47.75\text{ N/m}^2$

(ii) At a horizontal distance of 7.5 metres :-

$r = 7.5\text{ m},\ z = 15\text{ m}$

$r/z = 7.5/15 = 0.5$

$$\sigma_z = \frac{22.5}{15\times 15}\cdot\frac{\left(\frac{3}{2\pi}\right)}{\left[\left(1+(0.5)^2\right)\right]^{5/2}}$$

$= 27.33\text{ N/m}^2$

Principle of Effective Stress & Darcy's Law and 1D-Permeability

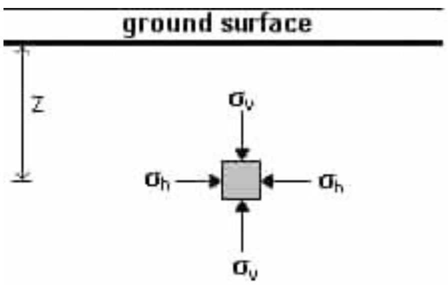
Effective Stress Principle, Capillarity & Seepage

Stresses in the Ground

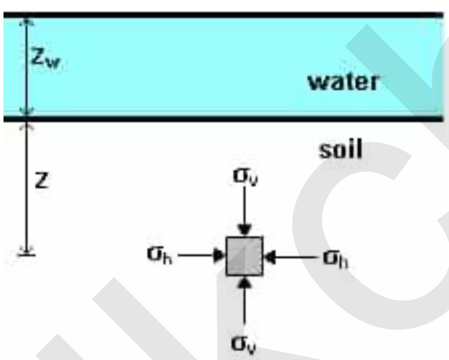
Total Stress

When a load is applied to soil, it is carried by the solid grains and the water in the pores. The **total vertical stress** acting at a point below the ground surface is due to the weight of everything that lies above, including soil, water, and surface loading. Total stress thus increases with depth and with unit weight.

Vertical total stress at depth z, $\mathbf{s_v = g.Z}$



Below a water body, the total stress is the sum of the weight of the soil up to the surface and the weight of water above this. $\mathbf{s_v = g.Z + g_w.Z_w}$

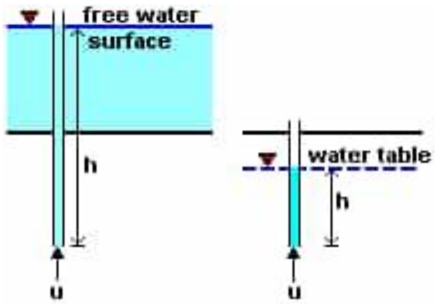


The total stress may also be denoted by $\mathbf{s_z}$ or just **s.** It varies with changes in water level and with excavation.

Pore, Water Pressure

The pressure of water in the pores of the soil is called **pore water pressure (u).** The magnitude of pore water pressure depends on:

- the depth below the water table.
- the conditions of seepage flow.



Under hydrostatic conditions, no water flow takes place, and the pore pressure at a given point is given by

$u = g_w.h$

where h = depth below water table or overlying water surface

It is convenient to think of pore water pressure as the pressure exerted by a column of water in an imaginary standpipe inserted at the given point.

The natural level of ground water is called the **water table** or the **phreatic surface.** Under conditions of no seepage flow, the water table is horizontal. The magnitude of the pore water pressure at the water table is zero. Below the water table, pore water pressures are positive.

Principle of Effective Stress

The **principle of effective stress** was enunciated by **Karl Terzaghi** in the year 1936. This principle is valid only for saturated soils, and consists of two parts:

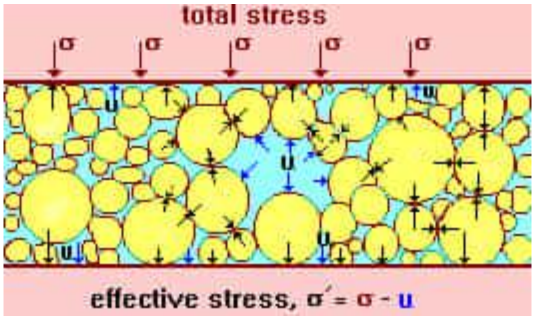
1. At any point in a soil mass, the effective stress (represented by σ' or s') is related to total stress (s) and pore water pressure (u) as

$\sigma' = s - u$

Both the total stress and pore water pressure can be measured at any point.

2. All measurable effects of a change of stress, such as compression and a change of shearing resistance, are exclusively due to changes in effective stress.

Compression = $f_1(\sigma')$
Shear Strength = $f_2(\sigma')$



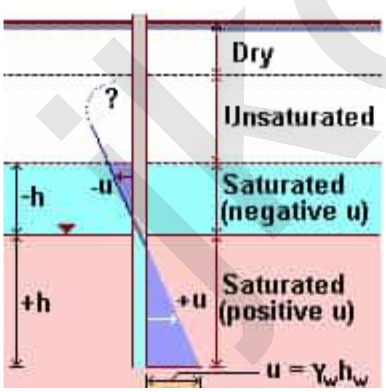
In a saturated soil system, as the voids are completely filled with water, the pore water pressure acts equally in all directions.

The effective stress is not the exact contact stress between particles but the distribution of load carried by the soil particles over the area considered. It cannot be measured and can only be computed.

If the total stress is increased due to additional load applied to the soil, the pore water pressure initially increases to counteract the additional stress. This increase in pressure within the pores might cause water to drain out of the soil mass, and the load is transferred to the solid grains. This will lead to the increase in effective stress.

Effective Stress in Unsaturated Zone

Above the water table, when the soil is saturated, pore pressure will be negative (less than atmospheric). The height above the water table to which the soil is saturated is called the **capillary rise**, and this depends on the grain size and the size of pores. In coarse soils, the capillary rise is very small.



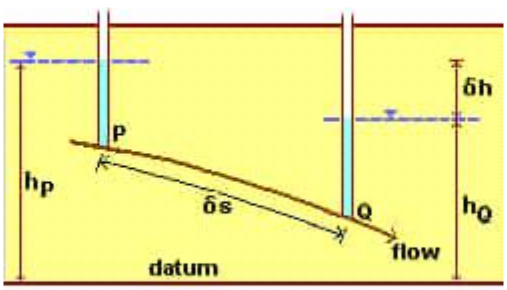
Between the top of the saturated zone and the ground surface, the soil is partially saturated, with a consequent reduction in unit weight . The pore pressure in a partially saturated soil consists of two components:

Pore water pressure = u_w
Pore air pressure = u_a

Water is incompressible, whereas air is compressible. The combined effect is a complex relationship involving partial pressures and the degree of saturation of the soil.

Effective Stress Under Hydrodynamic Conditions

There is a change in pore water pressure in conditions of **seepage flow** within the ground. Consider seepage occurring between two points **P** and **Q**. The potential driving the water flow is the hydraulic gradient between the two points, which is equal to the head drop per unit length. In steady state seepage, the gradient remains constant.

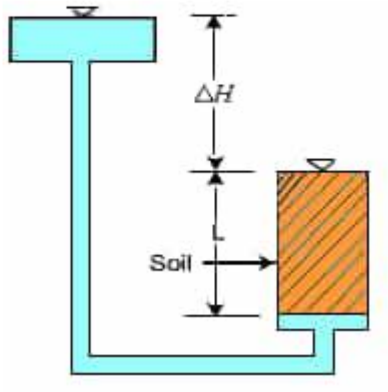


Hydraulic gradient from P to Q, **i = dh/ds**

As water percolates through soil, it exerts a drag on soil particles it comes in contact with. Depending on the flow direction, either downward of upward, the drag either increases or decreases inter-particle contact forces.

A downward flow increases effective stress.

In contrast, an upward flow opposes the force of gravity and can even cause to counteract completely the contact forces. In such a situation, effective stress is reduced to zero and the soil behaves like a very viscous liquid. Such a state is known as **quick sand condition.** In nature, this condition is usually observed in coarse silt or fine sand subject to artesian conditions.



At the bottom of the soil column,

$$\sigma = \gamma . L$$

$$u = \gamma_w (L + \Delta H)$$

During quick sand condition, the effective stress is reduced to zero.

$$L(\gamma - \gamma_w) = \gamma_w . \Delta H$$
$$L.\gamma_b = \gamma_w . \Delta H$$
$$\frac{\Delta H}{L} = \frac{\gamma_b}{\gamma_w} = i_{cr} \approx 1$$

where i_{cr} = **critical hydraulic gradient**

This shows that when water flows upward under a hydraulic gradient of about 1, it completely neutralizes the force on account of the weight of particles, and thus leaves the particles suspended in water.

The Importance of Effective Stress

- At any point within the soil mass, the magitudes of both total stress and pore water pressure are dependent on the ground water position. With a shift in the water table due to seasonal fluctuations, there is a resulting change in the distribution in pore water pressure with depth.
- Changes in water level ***below ground*** result in changes in effective stresses below the water table. A rise increases the pore water pressure at all elevations thus causing a decrease in effective stress. In contrast, a fall in the water table produces an increase in the effective stress.
- Changes in water level ***above ground*** do not cause changes in effective stresses in the ground below. A rise above ground surface increases both

the total stress and the pore water pressure by the same amount, and consequently effective stress is not altered.

- If both total stress and pore water pressure change by the same amount, the effective stress remains constant.
- Total and effective stresses must be distinguishable in all calculations.Ground movements and instabilities can be caused by changes in total stress, such as caused by loading by foundations and unloading due to excavations. They can also be caused by changes in pore water pressures, such as failure of slopes after rainfall.

Permeability

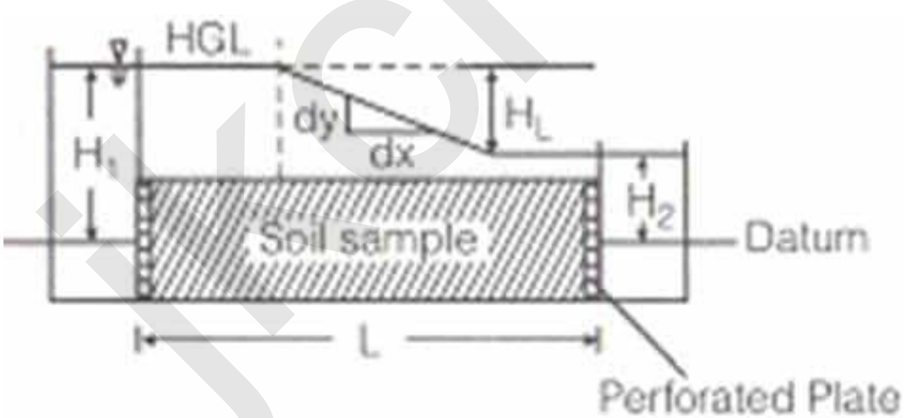
Permeability of Soil

The permeability of a soil is a property which describes quantitatively, the ease with which water flows through that soil.

Darcy's Law : Darcy established that the flow occurring per unit time is directly proportional to the head causing flow and the area of cross-section of the soil sample but is inversely proportional to the length of the sample.

(i) Rate of flow (q)

$$q \alpha \frac{\Delta h}{L} A \rightarrow q = KiA$$



Where, q = rate of flow in m^3/sec.

K = Coefficient of permeability in m/s

I = Hydraulic gradient

A = Area of cross-section of sample

$$i = \frac{H_L}{L}$$

where, H_L = Head loss = ($H_1 - H_2$)

$$i = \tan\theta \frac{dy}{dx}$$

(ii) Seepage velocity

$$V_s = \frac{V}{n}$$

where, V_s = Seepage velocity (m/sec)

n = Porosity & V = discharge velocity (m/s)

(iii) Coefficiency of percolation

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

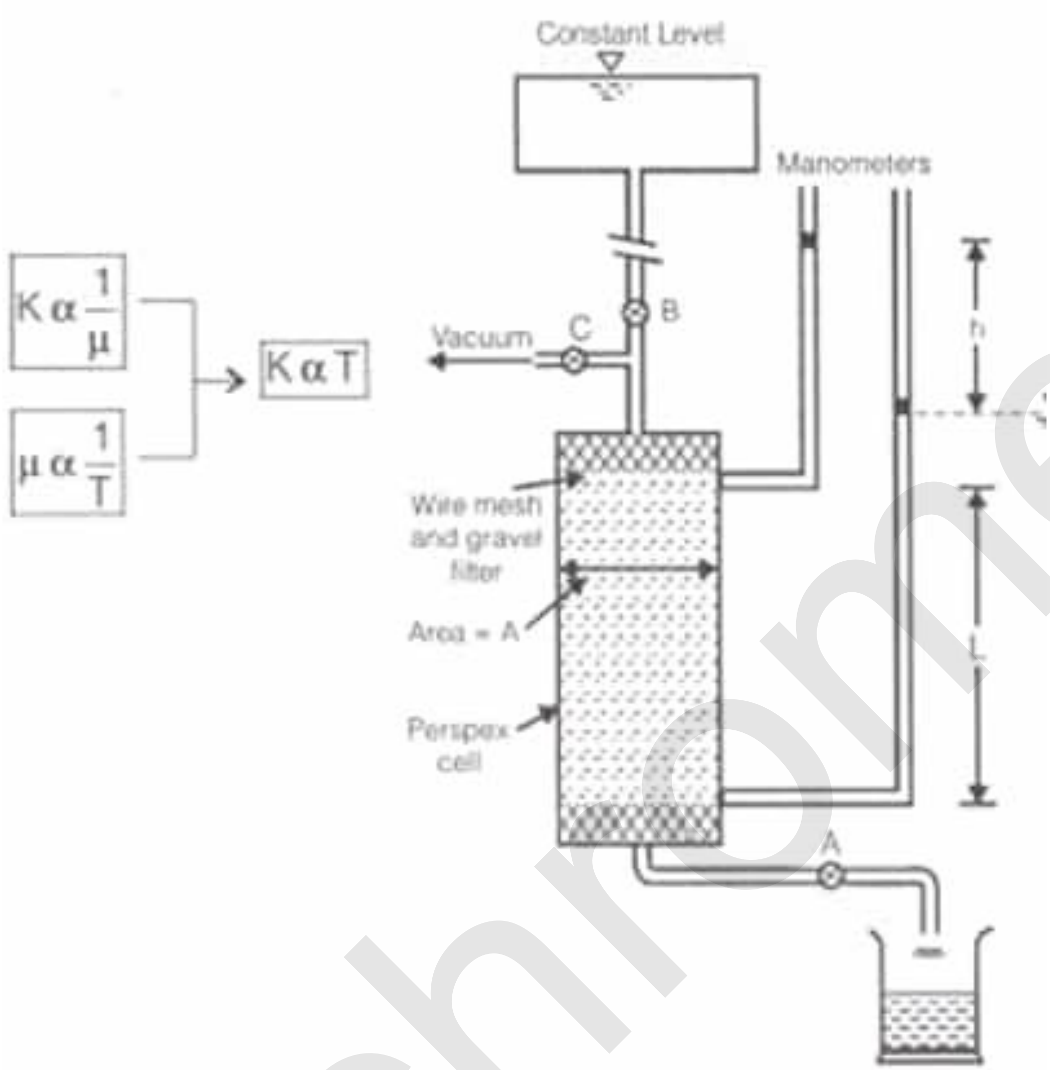
where, K_P = coefficiency of percolation and n = Porosity.

Constant Head Permeability Test

$$K = \frac{QL}{tH_L A}$$

where, Q = Volume of water collected in time t in m^3.

Constant Head Permeability test is useful for coarse grain soil and it is a laboratory method.



Falling Head Permeability Test or Variable Head Permeability Test

$$K = \frac{2.303aL}{At}\log_{10}\left(\frac{h_1}{h_2}\right)$$

a = Area of tube in m^2

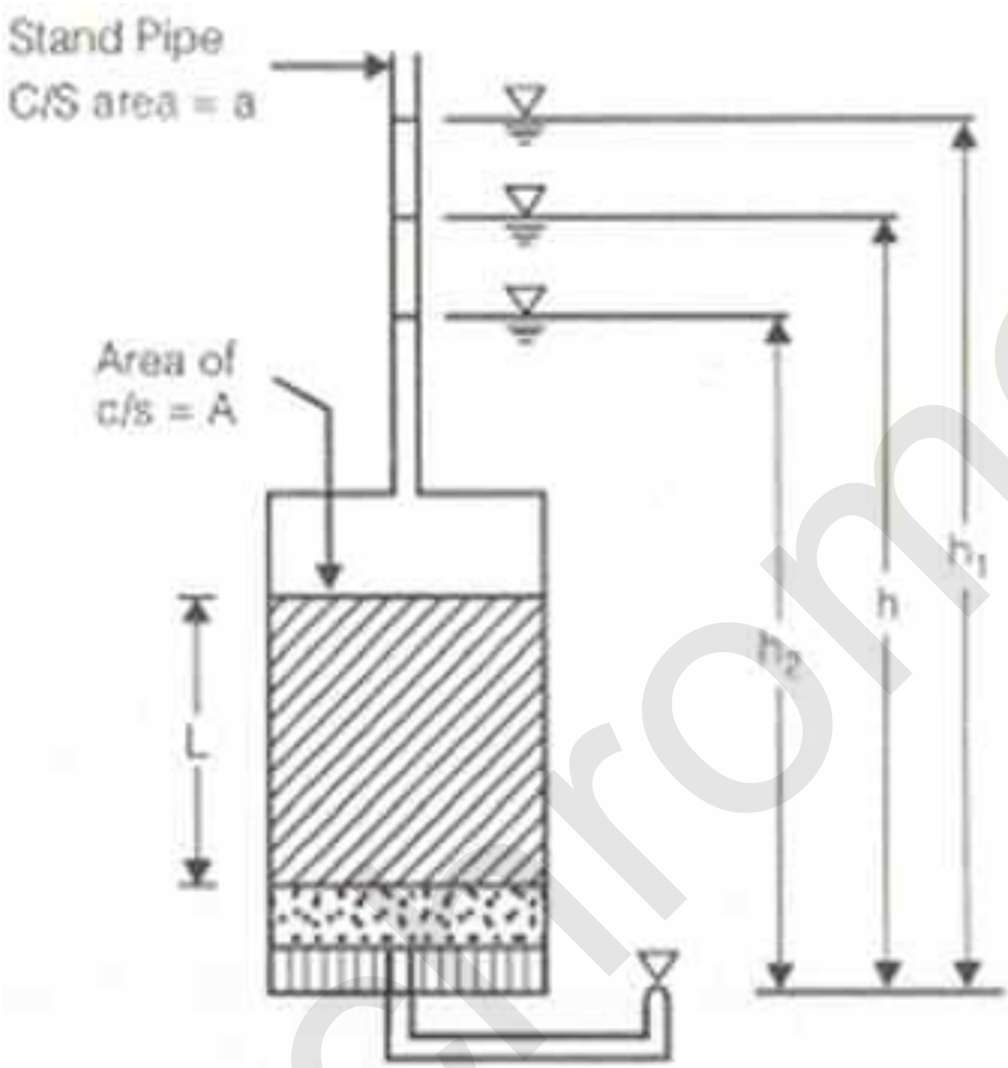
A = Area of sample in m^2

t = time in 'sec'

L = length in 'm'

h_1 = level of upstream edge at t = 0

h_2 = level of upstream edge after 't'.



Konzey-Karman Equation

$$K = \frac{1}{C} \cdot \frac{1}{S^2} \cdot \frac{\gamma_w}{\mu} \cdot \frac{e^3}{1+e}$$

Where, C = Shape coefficient, ~5mm for spherical particle

S = Specific surface area = $\frac{Area}{Volume}$

For spherical particle.

$$S = \frac{4\pi R^2}{\frac{4}{3}\pi R^3} = \frac{6}{Diameter}$$

R = Radius of spherical particle.

$$S = \frac{6}{\sqrt{ab}}$$

When particles are not spherical and of variable size. If these particles passes through sieve of size 'a' and retain on sieve of size 'n'.

e = void ratio

μ = dynamic viscosity, in (N-s/m^2)

γ_w = unit weight of water in kN/m^3

$$\frac{k_1}{k_2} = \frac{e_1^2}{e_2^2}$$

Allen Hazen Equation

$$K = C.D_{10}^2$$

Where, D_{10} = Effective size in cm. k is in cm/s C = 100 to 150

Lioudens, Equation

$$\log_{10} KS^2 = a + b.n$$

Where, S = Specific surface area

n = Porosity.

a and b are constant.

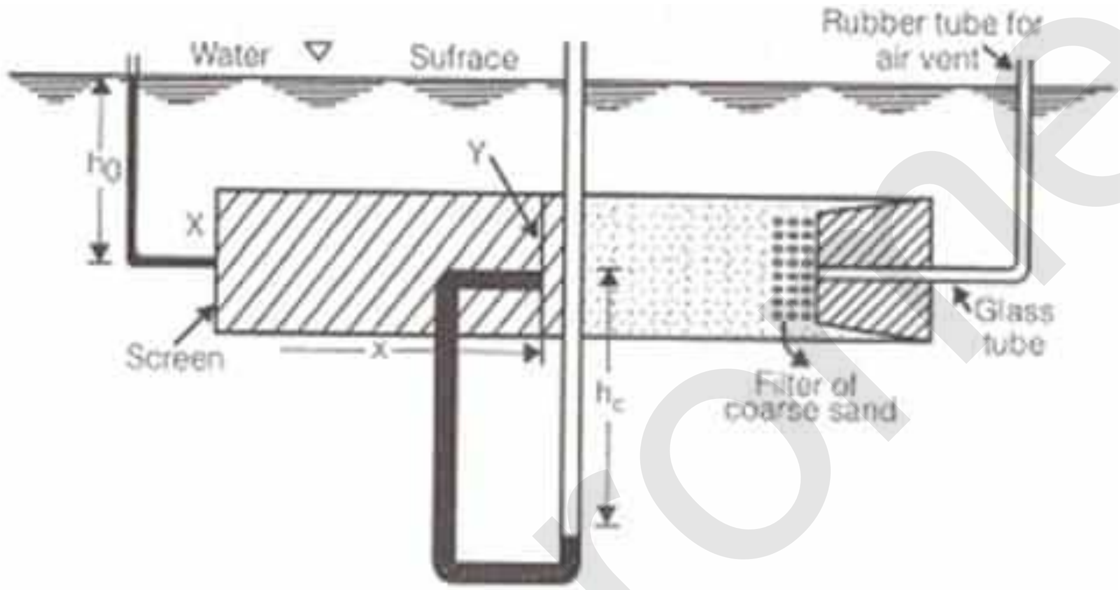
Consolidation equation

$$K = C_v m_v \gamma_w$$

Where, C_v = Coefficient of consolidation in cm^2/sec

m_v = Coefficient of volume Compressibility in cm^2/N

Capillary Permeability Test



$$i = \frac{h_o + h_c}{x}$$

where, S = Degree of saturation

K = Coefficient of permeability of partially saturated soil.

$$\frac{X'^2_2 - X'^2_1}{t'_2 - t'_1} = \frac{2K}{S.n}[h_{o_2} + h_c]$$

where h_c = remains constant (but not known as depends upon soil)

= head under first set of observation,

n = porosity, h_c = capillary height

Another set of data gives,

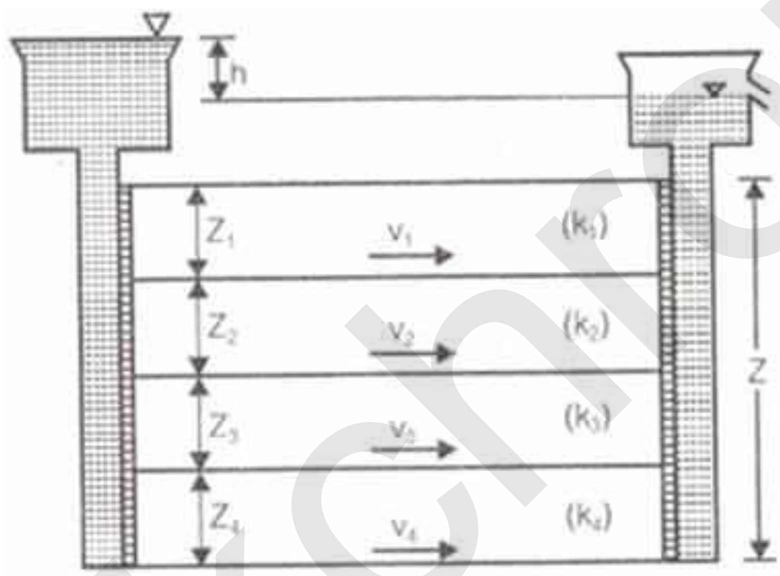
h_{o_2} = head under second set of observation

- For S = 100%, K = maximum. Also, $k_u \propto S$.

Permeability of a stratified soil

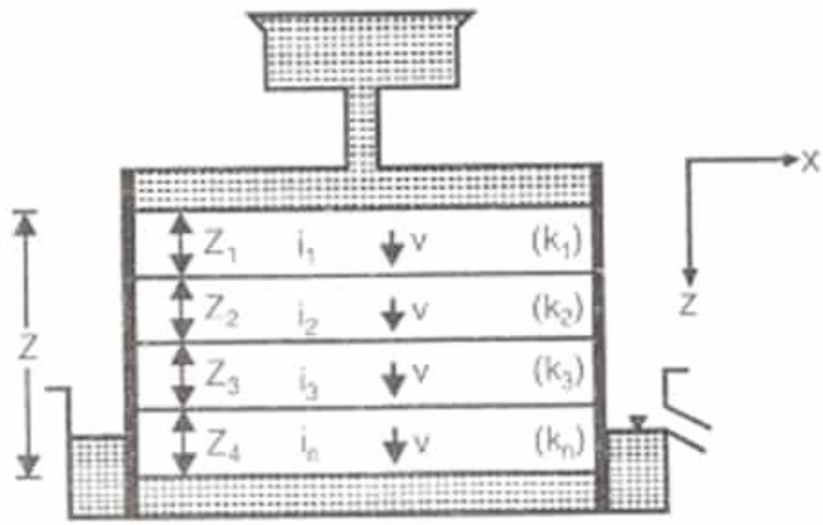
(i) Average permeability of the soil in which flow is parallel to bedding plane,

$$K_{eq} = \frac{k_1 z_1 + k_2 z_2 + \ldots k_n z_n}{z_1 + z_2 + \ldots + z_n} \quad k_{eq} \sim k_x$$



(ii) Average permeability of soil in which flow is perpendicular to bedding plane.

$$k_{eq} = \frac{z_1 + z_2 + \ldots + z_n}{\frac{z_1}{k_1} + \frac{z_2}{k_2} + \ldots + \frac{z_n}{k_n}} \quad k_{eq} \sim k_z$$



(iii) For 2-D flow in x and z direction

$$k_{eq} = \sqrt{k_x . k_z}$$

(iv) For 3-D flow in x, y and z direction $k_{eq} = (k_x . k_y . k_z)^{1/3}$

Coefficient of absolute permeability (k_0)

$$k_0 = k . \frac{\mu}{\gamma_w}$$

Consolidation and Compressibility

Compression and Consolidation of Soils

When a soil layer is subjected to vertical stress, volume change can take place through rearrangement of soil grains, and some amount of grain fracture may also take place. The volume of soil grains remains constant, so change in total volume is due to change in volume of water. In saturated soils, this can happen only if water is pushed out of the voids. The movement of water takes time and is controlled by the **permeability** of the soil and the locations of free draining boundary surfaces.

It is necessary to determine both the magnitude of volume change (or the settlement) and the time required for the volume change to occur. The magnitude of settlement is dependent on the magnitude of applied stress, thickness of the soil layer, and the compressibility of the soil.

When soil is loaded undrained, the pore pressure increases. As the excess pore pressure dissipates and water leaves the soil, settlement takes place. This process takes time, and the rate of settlement decreases over time. In coarse soils (sands and gravels), volume change occurs immediately as pore pressures are dissipated rapidly due to high permeability. In fine soils (silts and clays), slow seepage occurs due to low permeability.

Components of Total Settlement

The total settlement of a loaded soil has three components: Elastic settlement, primary consolidation, and secondary compression.

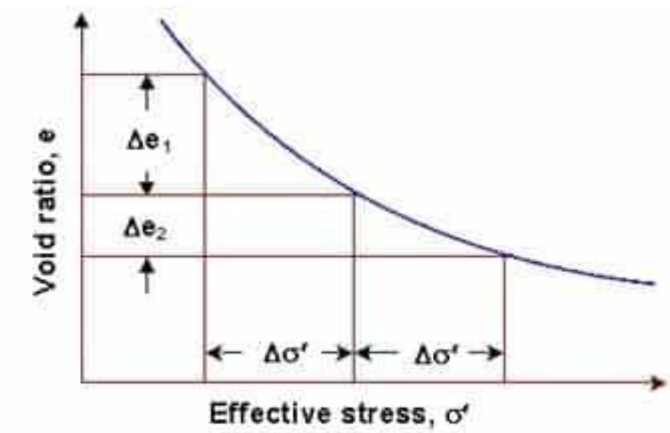
Elastic settlement is on account of change in shape at constant volume, i.e. due to vertical compression and lateral expansion. **Primary consolidation (**or simply **consolidation)** is on account of flow of water from the voids, and is a function of the permeability and compressibility of soil. **Secondary compression** is on account of creep-like behaviour.

Primary consolidation is the major component and it can be reasonably estimated. A general theory for consolidation, incorporating three-dimensional flow is complicated and only applicable to a very limited range of problems in geotechnical engineering. For the vast majority of practical settlement problems, it is sufficient to consider that both seepage and strain take place in one direction only, as **one-dimensional consolidation** in the vertical direction.

Compressibility Characteristics

Soils are often subjected to uniform loading over large areas, such as from wide foundations, fills or embankments. Under such conditions, the soil which is remote from the edges of the loaded area undergoes vertical strain, but no horizontal strain. Thus, the settlement occurs only in one-dimension.

The compressibility of soils under one-dimensional compression can be described from the decrease in the volume of voids with the increase of effective stress. This relation of void ratio and effective stress can be depicted either as an **arithmetic plot** or a **semi-log plot.**



In the arithmetic plot as shown, as the soil compresses, for the same increase of effective stress Ds', the void ratio reduces by a smaller magnitude, from **De_1** to **De_2**. This is on account of an increasingly denser packing of the soil particles as the pore water is forced out. In fine soils, a much longer time is required for the pore water to escape, as compared to coarse soils.

It can be said that the compressibility of a soil decreases as the effective stress increases. This can be represented by the slope of the void ratio – effective stress relation, which is called the coefficient of compressibility, a_v.

$$a_v = -\frac{de}{d\sigma'}$$

For a small range of effective stress, $a_v = -\frac{\Delta e}{\Delta\sigma'}$

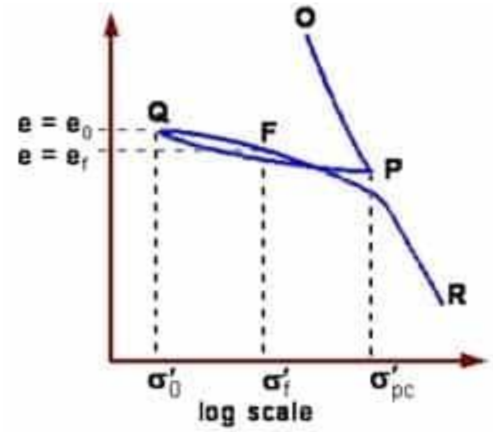
The -ve sign is introduced to make a_v a positive parameter.

If $\mathbf{e_0}$ is the initial void ratio of the consolidating layer, another useful parameter is the **coefficient of volume compressibility**, m_v, which is expressed as

$$m_v = \frac{a_v}{1+e_0}$$

It represents the compression of the soil, per unit original thickness, due to a unit increase of pressure.

NC & OC Clays



OP corresponds to initial loading of the soil. **PQ** corresponds to unloading of the soil. **QFR** corresponds to a reloading of the soil. Upon reloading beyond **P,** the soil continues along the path that it would have followed if loaded from **O to R** continuously.

The **preconsolidation stress,** s'_{pc}, is defined to be the maximum effective stress experienced by the soil. This stress is identified in comparison with the effective stress in its present state. For soil at state **Q or F,** this would correspond to the effective stress at point **P.**

If the current effective stress, s', is equal (note that it cannot be greater than) to the preconsolidation stress, then the deposit is said to be **normally consolidated (NC).** If the current effective stress is less than the preconsolidation stress, then the soil is said to be **over-consolidated (OC).**

It may be seen that for the same increase in effective stress, the change in void ratio is much less for an overconsolidated soil **(from e_0 to e_f)**, than it would have been for a normally consolidated soil as in path **OP.** In unloading, the soil swells but the increase in volume is much less than the initial decrease in volume for the same stress difference.

The distance from the normal consolidation line has an important influence on soil behaviour. This is described numerically by the **overconsolidation ratio (OCR)**, which is defined as the ratio of the preconsolidation stress to the current effective stress.

$$OCR = \frac{\sigma'_{pc}}{\sigma'}$$

Note that when the soil is normally consolidated, **OCR = 1**

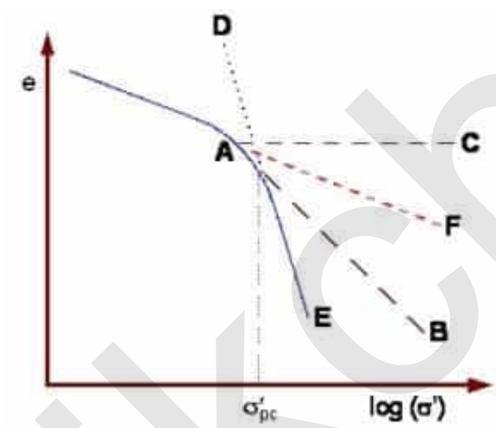
Settlements will generally be much smaller for structures built on overconsolidated soils. Most soils are overconsolidated to some degree. This can be due to shrinking and swelling of the soil on drying and rewetting, changes in ground water levels, and unloading due to erosion of overlying strata.

For **NC clays,** the plot of void ratio versus log of effective stress can be approximated to a straight line, and the slope of this line is indicated by a parameter termed as **compression index, C_c**.

$$C_c = \frac{\Delta e}{\log_{10}\left(\frac{\sigma_2'}{\sigma_1'}\right)}$$

Estimation of Preconsolidation Stress

It is possible to determine the preconsolidation stress that the soil had experienced. The soil sample is to be loaded in the laboratory so as to obtain the void ratio - effective stress relationship. Empirical procedures are used to estimate the preconsolidation stress, the most widely used being **Casagrande's construction** which is illustrated.



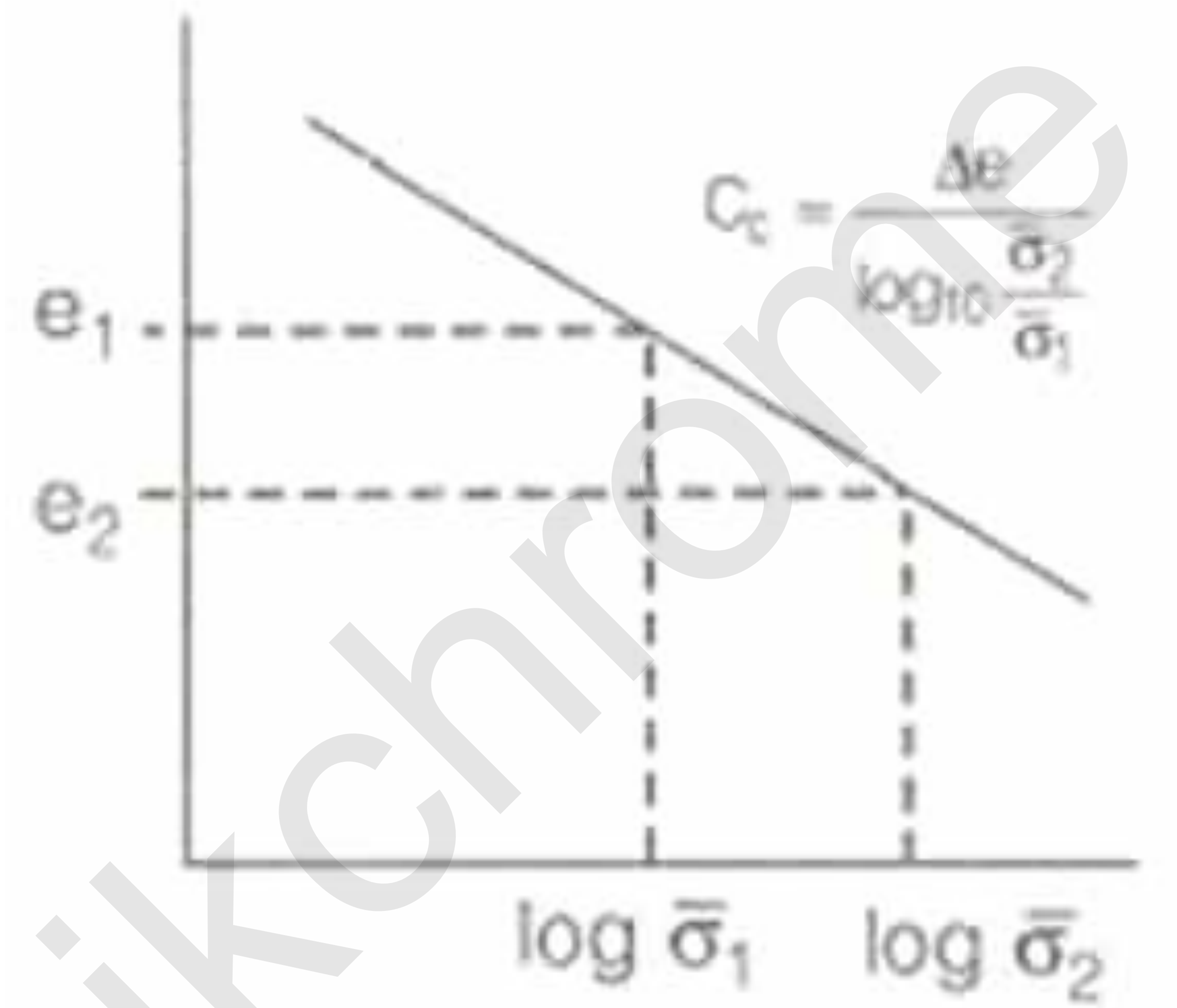
The steps in the construction are:

- Draw the graph using an appropriate scale.
- Determine the point of maximum curvature **A.**
- At **A,** draw a tangent **AB** to the curve.
- At **A,** draw a horizontal line **AC.**

• Draw the extension **ED** of the straight line portion of the curve.

• Where the line **ED** cuts the bisector **AF** of angle **CAB**, that point corresponds to the preconsolidation stress.

Coefficient of Compression (C_c)



A.

$$C_c = \frac{e_1 - e_2}{\log_{10} \overline{\sigma_2} \log_{10} \overline{\sigma_1}}$$

$$C_c = \frac{e_1 - e_2}{\log_{10}\left(\frac{\overline{\sigma_2}}{\overline{\sigma_1}}\right)}$$

B.

$$C_c = 0.009(W_L - 10)$$

For undisturbed soil of medium sensitivity.

W_L = % liquid limit.

C.

$$C_c = 0.009(W_L - 7)$$

For remolded soil of low sensitivity

D.

$$C_c = 0.40(e_0 - 0.25)$$

For undisturbed soil of medium sensitivity e_{o} = Initial void ratio

E.

For remoulded soil of low sensitivity.

C_c = 1.15(e_0-0.35)

F.

C_c = 0.115*w* where, w = Water content

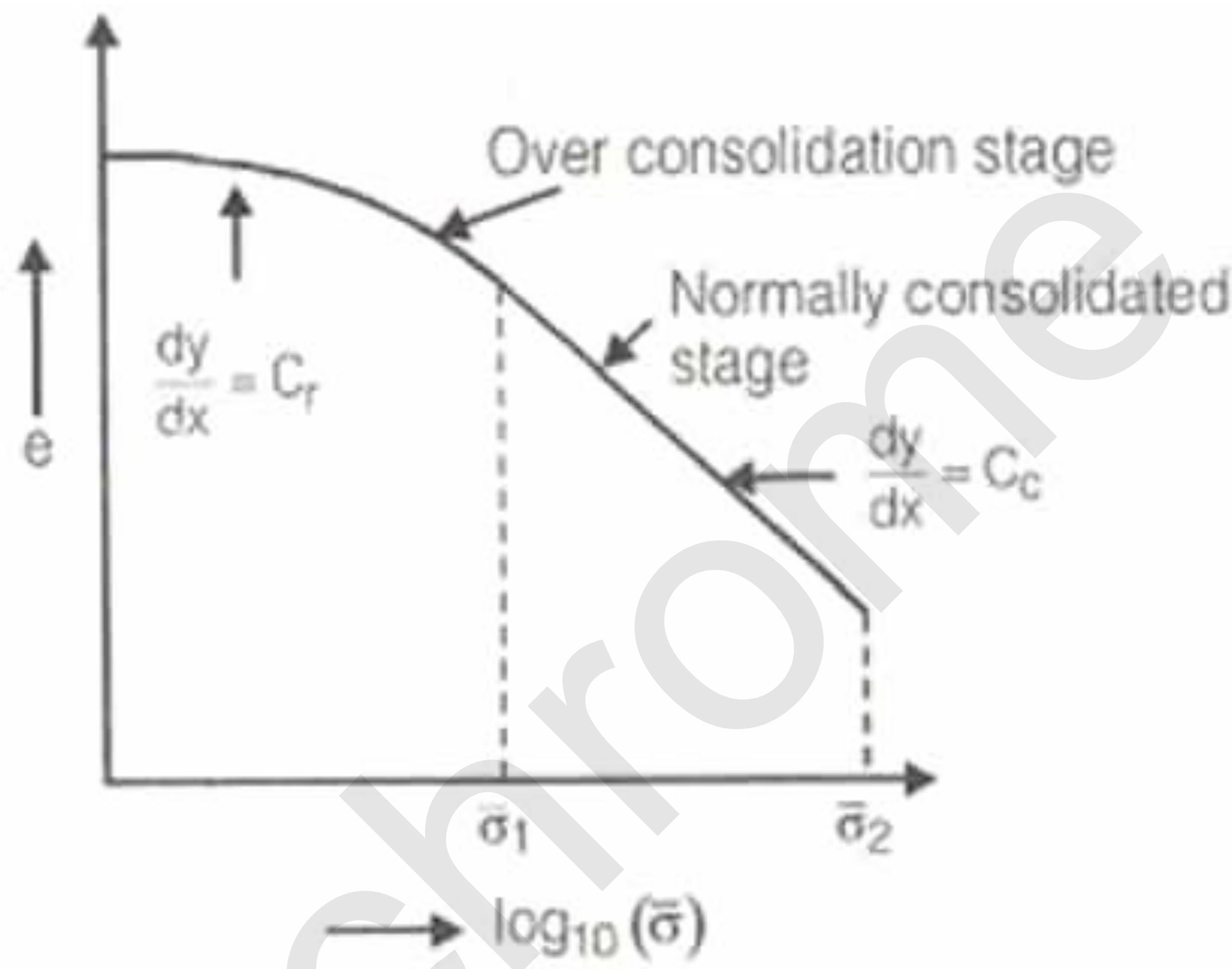
Over consolidation ratio

$$O.C.R. = \frac{\text{Maximum effective stress applied in the past}}{\text{Existing effective stress}}$$

O.C.R > 1 For over consolidated soil.

O.C.R = 1 For normally consolidated soil.

O.C.R < 1 For under consolidated soil.



Differential Equation of 1-D Consolidation

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

where, u = Excess pore pressure.

$\frac{\partial u}{\partial t}$ = Rate of change of pore pressure

C_v = Coefficient of consolidation

$\frac{\partial u}{\partial z}$ = Rate of change of pore pressure with depth.

Coefficient of volume compressibility $m_v = \frac{a_v}{1+e_0}$ where, e_0 = Initial void ratio

m_v = Coefficient of volume compressibility

Compression modulus

$E_c = \frac{1}{m_v}$ where, E_c =Compression modulus.

Degree of consolidation

(i)

$$\%U = \left(1 - \frac{U}{U_1}\right) \times 100$$

where,

%U = % degree of consolidation.

U = Excess pore pressure at any stage.

U_1 = $\overline{\Delta\sigma}$ = Initial excess pore pressure

at $t = 0, u = u_1 \Rightarrow \%u = 0\%$

at $t = \infty, u = 0 \Rightarrow \%u = 100\%$

(ii)

$$\%u = \frac{e_0 - e}{e_0 - e_f} \times 100$$

where,

e_f = Void ratio at 100% consolidation.

i.e. of t = ∞

e = Void ratio at time 't'

e_0 = Initial void ratio i.e., at t = 0

(iii) $$\%u = \frac{\Delta h}{\Delta H} \times 100$$

where,

ΔH = Final total settlement at the end of completion of primary consolidation i.e.,

at t = ∞

Δh = Settlement occurred at any time 't'.

Time factor

$$T_v = C_v \cdot \frac{t}{d^2}$$ where, T_V = Time factor

C_V = Coeff. of consolidation in cm^2/sec.

d = Length of drainage path

t = Time in 'sec'

$$d = \frac{H_0}{2}$$ For 2-way drainage

$d = H_0$ For one-way drainage.

where, H_0 = Depth of soil sample.

Some cases

(i) $T_v = \frac{\pi}{4}(u)^2 \ldots$ if $u \le 60\%$ $T_{50} = 0.196$

(ii) $T_v = -0.9332 \log_{10}(1-u) - 0.0851 \ldots$ if $u > 60\%$

Method to find 'C_v'

(i) Square Root of Time Fitting Method

$$C_v = \frac{T_{90}.d^2}{t_{90}}$$

where,

T_{90} = Time factor at 90% consolidation

t_{90} = Time at 90% consolidation

d = Length of drainage path.

(ii) Logarithm of Time Fitting Method

$$C_v = \frac{T_{50}.d^2}{t_{50}}$$

where, T_{50} = Time factor at 50% consolidation

t_{50} = Time of 50% consolidation.

Compression Ratio

(i) Initial Compression Ratio

$$r_i = \frac{R_i - R_0}{R_i - R_f}$$

where, R_i = Initial reading of dial gauge.

R_0 = Reading of dial gauge at 0% consolidation.

R_f = Final reading of dial gauge after secondary consolidation.

(ii) Primary Consolidation Ratio

$$r_s = \frac{R_0 - R_{100}}{R_i - R_f}$$

where, R_{100} = Reading of dial gauge at 100% primary consolidation.

(iii) Secondary Consolidation Ratio

$$r_s = \frac{R_{100} - R_f}{R_i - R_f} \quad r_i + r_p + r_s = 1$$

Total Settlement

$S = S_i + S_p + S_s$ where, S_i = Initial settlement

S_p = Primary settlement

S_s = Secondary settlement

(i) Initial Settlement

$$S_i = \frac{H_0}{C_s} . \log_{10} \frac{\overline{\sigma_0} + \overline{\Delta\sigma}}{\overline{\sigma_0}}$$

For cohesionless soil.

where, $$C_s = 1.5 \frac{C_r}{\sigma_0}$$

where, C_r = Static one resistance in kN/m^2

H_0 = Depth of soil sample $$S_i = \frac{q\sqrt{A}(1-\mu^2)}{E_s}(I_t)$$ For cohesive soil.

where, I_t = Shape factor or influence factor

A = Area.

(ii) Primary Settlement

- $$S_P = \Delta H = H_0 \frac{\Delta e}{1+e_0}$$
- $$\Delta H = H_0 m_v \overline{\Delta\sigma}$$
- $$\Delta H = \frac{C_c H_0}{1+e_0} \log_{10}\left(\frac{\overline{\sigma_0} + \overline{\Delta\sigma}}{\overline{\sigma_0}}\right)$$
- $$S_P = S_{C_1} + S_{C_2} \; S_{C_1} << S_{C_2} \rightarrow S_P \sim S_{C_2}$$

S_{C_1} = Settlement for over consolidated stage

S_{C_2} = Settlement for normally consolidation stage

$$S_P = \frac{C_r H_0}{1+e_0} \log_{10}\left(\frac{\overline{\sigma_1}}{\overline{\sigma_0}}\right) + \frac{C_c H_0'}{1+e_0'} \log_{10}\left(\frac{\overline{\sigma_2}}{\overline{\sigma_1}}\right)$$

(ii) Secondary Settlement

$$S_S = \frac{C_s H_0}{1+e_{100}} \log_{10}\left(\frac{t_2}{t_1}\right)$$

where, $H_0 \sim H_{100}$

H_{100} = Thickness of soil after 100% primary consolidation.

e_{100} = Void ratio after 100% primary consolidation.

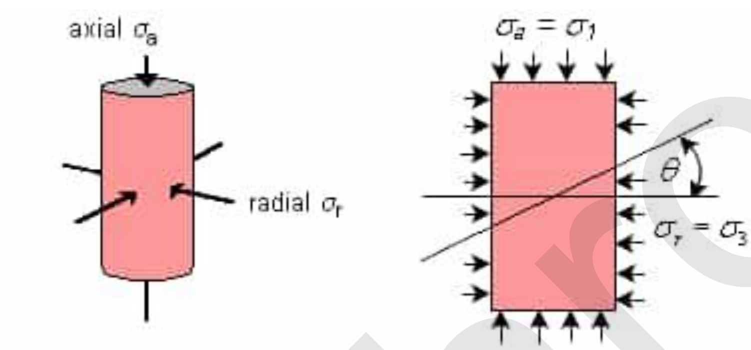
t_2 = Average time after t_1 in which secondary consolidation is calculated

Shear Strength of Soil

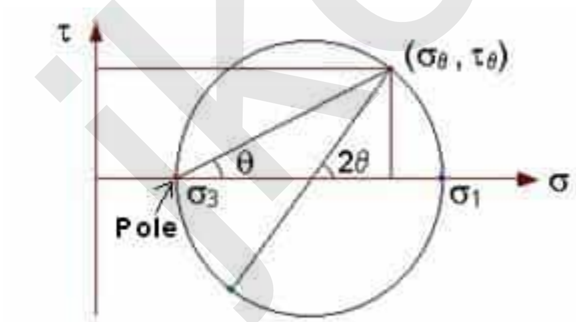
Shear strength of a soil is equal to the maximum value of shear stress that can be mobilized within a soil mass without failure taking place.

The shear strength of a soil is a function of the stresses applied to it as well as the manner in which these stresses are applied. A knowledge of shear strength of soils is necessary to determine the bearing capacity of foundations, the lateral pressure exerted on retaining walls, and the stability of slopes.

Mohr Circle of Stresses
In soil testing, cylindrical samples are commonly used in which radial and axial stresses act on principal planes. The vertical plane is usually the minor principal plane whereas the horizontal plane is the major principal plane. The radial stress (s_r) is the minor principal stress (s_3), and the axial stress (s_a) is the major principal stress (s_1).



A graphical representation of stresses called the Mohr circle is obtained by plotting the principal stresses. The sign convention in the construction is to consider compressive stresses as positive and angles measured counter-clockwise also positive.



Draw a line inclined at angle θ with the horizontal through the pole of the Mohr circle so as to intersect the circle. The coordinates of the point of intersection are the normal and shear stresses acting on the plane, which is inclined at angle θ within the soil sample.

- **Normal stress**

$$\sigma_\theta = \frac{(\sigma_1 + \sigma_3)}{2} + \frac{(\sigma_1 - \sigma_3)}{2}\cos 2\theta$$

- **Shear stress**

$$\tau_\theta = \frac{(\sigma_1 - \sigma_3)}{2}\sin 2\theta$$

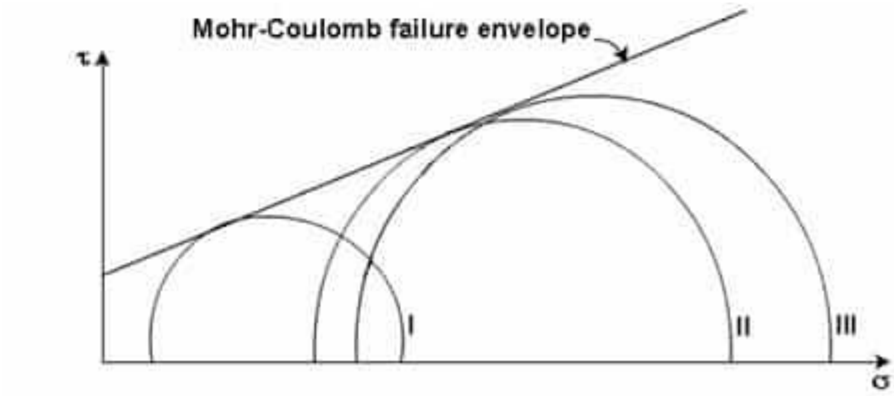
- **The plane inclined at an angle of 45^0 to the horizontal has acting on it the maximum shear stress equal to $\frac{\sigma_1 - \sigma_3}{2}$, and the normal stress on this plane is equal to $\frac{\sigma_1 + \sigma_3}{2}$.**
- **The plane with the maximum ratio of shear stress to normal stress is inclined at an angle of $45^0 + \frac{\alpha}{2}$ to the horizontal, where a is the slope of the line tangent to the Mohr circle and passing through the origin.**

Mohr-Coulomb Failure Criterion

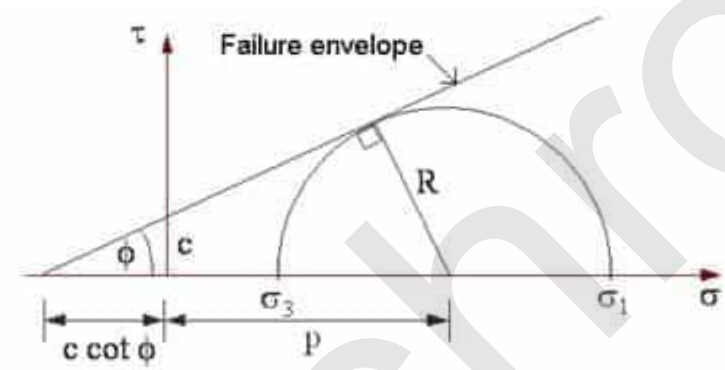
When the soil sample has failed, the shear stress on the failure plane defines the shear strength of the soil. Thus, it is necessary to identify the failure plane. Is it the plane on which the maximum shear stress acts, or is it the plane where the ratio of shear stress to normal stress is the maximum?

For the present, it can be assumed that a failure plane exists and it is possible to apply principal stresses and measure them in the laboratory by conducting a triaxial test. Then, the Mohr circle of stress at failure for the sample can be drawn using the known values of the principal stresses.

If data from several tests, carried out on different samples upto failure is available, a series of Mohr circles can be plotted. It is convenient to show only the upper half of the Mohr circle. A line tangential to the Mohr circles can be drawn, and is called the Mohr-Coulomb failure envelope.



If the stress condition for any other soil sample is represented by a Mohr circle that lies below the failure envelope, every plane within the sample experiences a shear stress which is smaller than the shear strength of the sample. Thus, the point of tangency of the envelope to the Mohr circle at failure gives a clue to the determination of the inclination of the failure plane. The orientation of the failure plane can be finally determined by the pole method.



Mohr-Coulomb failure criterion can be written as the equation for the line that represents the failure envelope. The general equation is

$$\tau_f = c + \sigma_f . \tan\phi$$

Where,

τ_f =

shear stress on the failure plane

c = apparent cohesion

σ_f

= normal stress on the failure plane
f = angle of internal friction

The failure criterion can be expressed in terms of the relationship between the principal stresses. From the geometry of the Mohr circle,

$$\sin\phi = \frac{R}{c.\cot\phi + p} = \frac{\frac{\sigma_1 - \sigma_3}{2}}{c.\cot\phi + \frac{\sigma_1 + \sigma_3}{2}}$$

Rearranging,

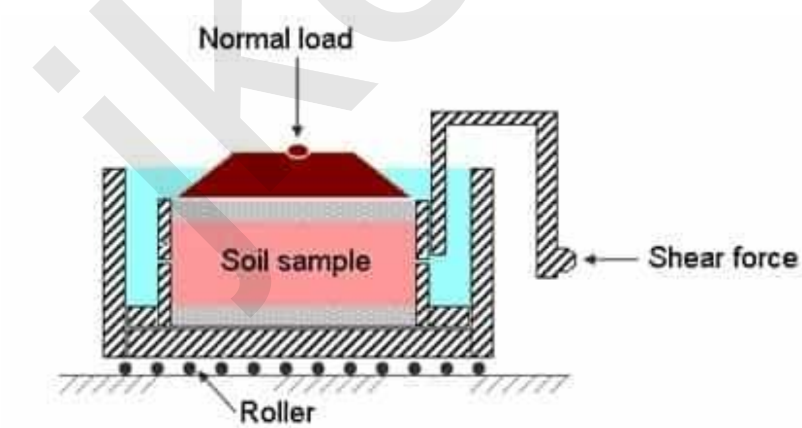
$$\sigma_1 = \sigma_3\left(\frac{1+\sin\phi}{1-\sin\phi}\right) + 2c\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}$$

where $\frac{1+\sin\phi}{1-\sin\phi} = \tan^2\left[\frac{\pi}{4} + \frac{\phi}{2}\right]$

Methods of Shear Strength Determination

1. **Direct Shear Test**

The test is carried out on a soil sample confined in a metal box of square cross-section which is split horizontally at mid-height. A small clearance is maintained between the two halves of the box.The soil is sheared along a predetermined plane by moving the top half of the box relative to the bottom half. The box is usually square in plan of size
60 mm x 60 mm. A typical shear box is shown.



If the soil sample is fully or partially saturated, perforated metal plates and porous stones are placed below and above the sample to allow free drainage. If the sample is dry, solid metal plates are used. A load normal to the plane of shearing can be applied to the soil sample through the lid of the box.

Tests on sands and gravels can be performed quickly, and are usually performed dry as it is found that water does not significantly affect the drained strength. For clays, the rate of shearing must be chosen to prevent excess pore pressures building up.

As a vertical normal load is applied to the sample, shear stress is gradually applied horizontally, by causing the two halves of the box to move relative to each other. The shear load is measured together with the corresponding shear displacement. The change of thickness of the sample is also measured.

A number of samples of the soil are tested each under different vertical loads and the value of shear stress at failure is plotted against the normal stress for each test. Provided there is no excess pore water pressure in the soil, the total and effective stresses will be identical. From the stresses at failure, the failure envelope can be obtained.

The test has several **advantages:**

- It is easy to test sands and gravels.
- Large samples can be tested in large shear boxes, as small samples can give misleading results due to imperfections such as fractures and fissures, or may not be truly representative.
- Samples can be sheared along predetermined planes, when the shear strength along fissures or other selected planes are needed.

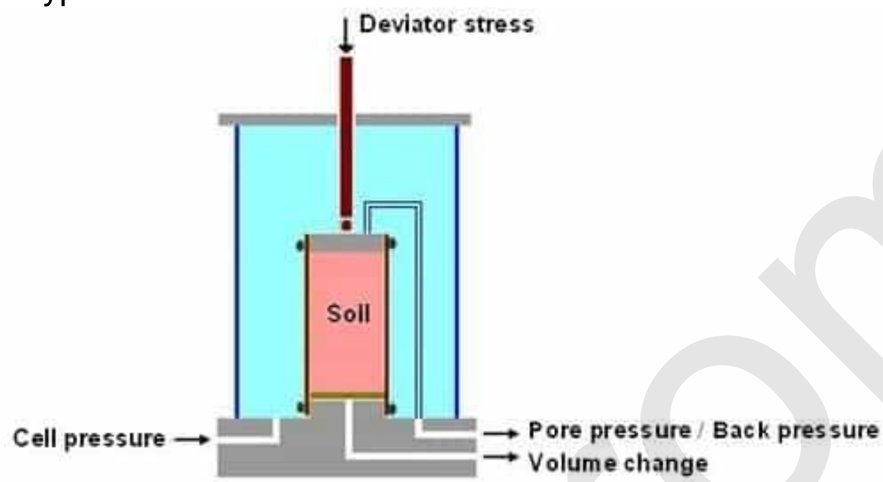
The **disadvantages** of the test include:

- The failure plane is always horizontal in the test, and this may not be the weakest plane in the sample. Failure of the soil occurs progressively from the edges towards the centre of the sample.
- There is no provision for measuring pore water pressure in the shear box and so it is not possible to determine effective stresses from undrained tests.
- The shear box apparatus cannot give reliable undrained strengths because it is impossible to prevent localised drainage away from the shear plane.

Triaxial test

- The triaxial test is carried out in a cell on a cylindrical soil sample having a length to diameter ratio of 2.
- The usual sizes are 76 mm x 38 mm and 100 mm x 50 mm. Three principal stresses are applied to the soil sample, out of which two are applied water pressure inside the confining cell and are equal.
- The third principal stress is applied by a loading ram through the top of the cell and is different to the other two principal stresses.

A typical triaxial cell in 2D is shown as



The soil sample is placed inside a rubber sheath which is sealed to a top cap and bottom pedestal by rubber O-rings. For tests with pore pressure measurement, porous discs are placed at the bottom, and sometimes at the top of the specimen. Filter paper drains may be provided around the outside of the specimen in order to speed up the consolidation process. Pore pressure generated inside the specimen during testing can be measured by means of pressure transducers.

The triaxial compression test consists of two stages:

- ***First stage:*** In this, a soil sample is set in the triaxial cell and confining pressure is then applied.
- ***Second stage:*** In this, additional axial stress (also called deviator stress) is applied which induces shear stresses in the sample. The axial stress is continuously increased until the sample fails.

During both the stages, the applied stresses, axial strain, and pore water pressure or change in sample volume can be measured.

Test Types

There are several test variations, and those used mostly in practice are:

- ***UU (unconsolidated undrained) test:*** In this, cell pressure is applied without allowing drainage. Then keeping cell pressure constant, deviator stress is increased to failure without drainage.
- ***CU (consolidated undrained) test:*** In this, drainage is allowed during cell pressure application. Then without allowing further drainage, deviator stress is increased keeping cell pressure constant.
- ***CD (consolidated drained) test:*** This is similar to **CU test** except that as deviator stress is increased, drainage is permitted. The rate of loading must be slow enough to ensure no excess pore water pressure develops.

In the UU test, if pore water pressure is measured, the test is designated by

$\overline{UU}$

In the CU test, if pore water pressure is measured in the second stage, the test is symbolized as $\overline{CU}$.

Significance of Triaxial Testing

The first stage simulates in the laboratory the in-situ condition that soil at different depths is subjected to different effective stresses. Consolidation will occur if the pore water pressure which develops upon application of confining pressure is allowed to dissipate. Otherwise the effective stress on the soil is the confining pressure (or total stress) minus the pore water pressure which exists in the soil.

During the shearing process, the soil sample experiences axial strain, and either volume change or development of pore water pressure occurs. The magnitude of shear stress acting on different planes in the soil sample is different. When at some strain the sample fails, this limiting shear stress on the failure plane is called the shear strength.

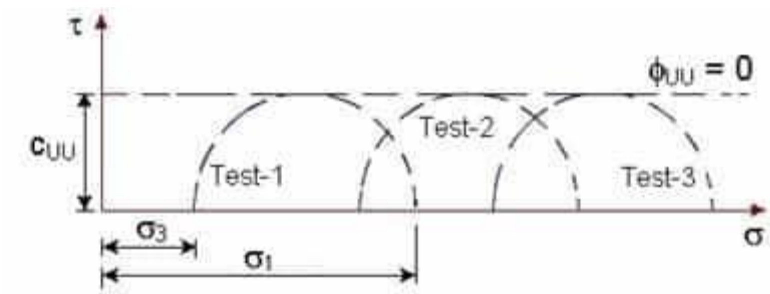
The triaxial test has many **advantages** over the direct shear test:

- The soil samples are subjected to uniform stresses and strains.
- Different combinations of confining and axial stresses can be applied.
- Drained and undrained tests can be carried out.
- Pore water pressures can be measured in undrained tests.

- The complete stress-strain behaviour can be determined.

Total Stress Parameters

UU Tests



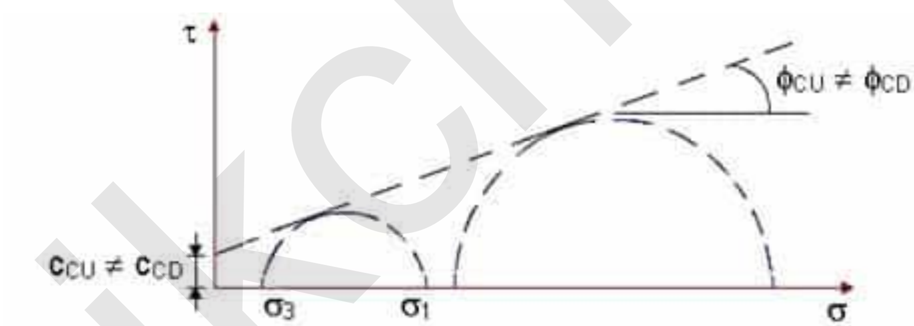
All Mohr circles for UU test plotted in terms of total stresses have the same diameter.

The failure envelope is a horizontal straight line and hence $\phi_{UU} = 0$

It can be represented by the equation:

$$\tau_f = c_{UU} = \frac{\sigma_1 - \sigma_3}{2}$$

CU & CD Tests:



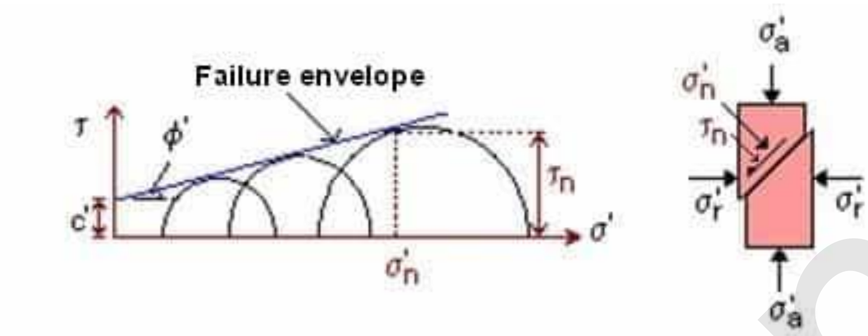
For tests involving drainage in the first stage, when Mohr circles are plotted in terms of total stresses, the diameter increases with the confining pressure. The resulting failure envelope is an inclined line with an intercept on the vertical axis.

It is also observed that c_{CU} ¹ c_{CD} and f_{CU} ¹ f_{CD}

It can be stated that for identical soil samples tested under different triaxial conditions of UU, CU and CD tests, the failure envelope is not unique.

Effective Stress Parameters

If the same triaxial test results of **UU, CU and CD tests** are plotted in terms of effective stresses taking into consideration the measured pore water pressures, it is observed that all the Mohr circles at failure are tangent to the same failure envelope, indicating that shear strength is a unique function of the effective stress on the failure plane.



This failure envelope is the shear strength envelope which may then be written as

$$\tau_f = c' + \sigma'.\tan\phi'$$

where *c'* = cohesion intercept in terms of effective stress
f' = angle of shearing resistance in terms of effective stress

If σ'_n is the effective stress acting on the rupture plane at failure, τ_n is the shear stress on the same plane and is therefore the shear strength.

The relationship between the effective stresses on the failure plane is

$$\sigma'_1 = \sigma'_3\left(\frac{1+\sin\phi'}{1-\sin\phi'}\right) + 2c'\sqrt{\frac{1+\sin\phi'}{1-\sin\phi'}}$$

Stress-Strain Behaviour of Sands

Sands are usually sheared under drained conditions as they have relatively higher permeability. This behaviour can be investigated in direct shear or triaxial tests. The two most important parameters governing their behaviour are the **relative density (I_D)** and the magnitude of the **effective stress (σ').** The relative density is usually defined in percentage as

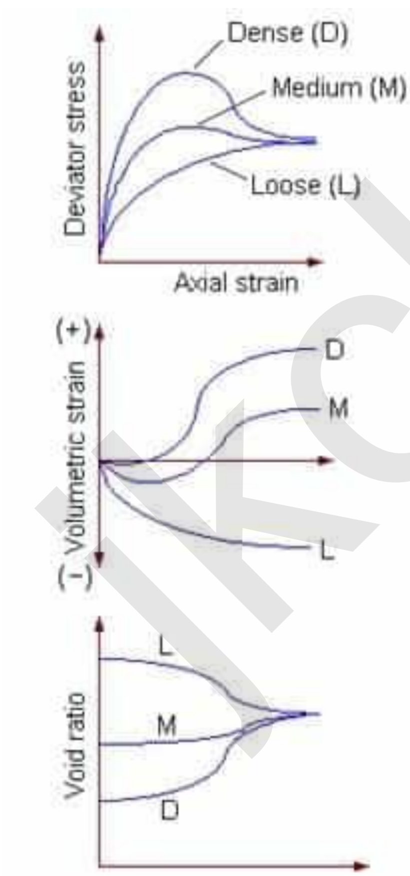
$$I_D = \frac{e_{max} - e}{e_{max} - e_{min}} x100$$

where e_{max} and e_{min} are the maximum and minimum void ratios that can be determined from standard tests in the laboratory, and **e** is the current void ratio. This expression can be re-written in terms of dry density as

$$I_D = \left(\frac{\gamma_d - \gamma_{d\,min}}{\gamma_{d\,max} - \gamma_{d\,min}} \right) x \frac{\gamma_{d\,max}}{\gamma_d} x100$$

where g_{dmax} and g_{dmin} are the maximum and minimum dry densities, and g_d is the current dry density. Sand is generally referred to as dense if I_D > 65% and loose if < 35%.

The influence of relative density on the behaviour of saturated sand can be seen from the plots of CD tests performed at the **same effective confining stress.** There would be no induced pore water pressures existing in the samples.



For the dense sand sample, the deviator stress reaches a peak at a low value of axial strain and then drops down, whereas for the loose sand sample, the deviator stress builds up gradually with axial strain. The behaviour of the medium sample is in between.

The following observations can be made:

• All samples approach the same ultimate conditions of shear stress and void ratio, irrespective of the initial density. The denser sample attains higher peak angle of shearing resistance in between.

• Initially dense samples expand or dilate when sheared, and initially loose samples compress.

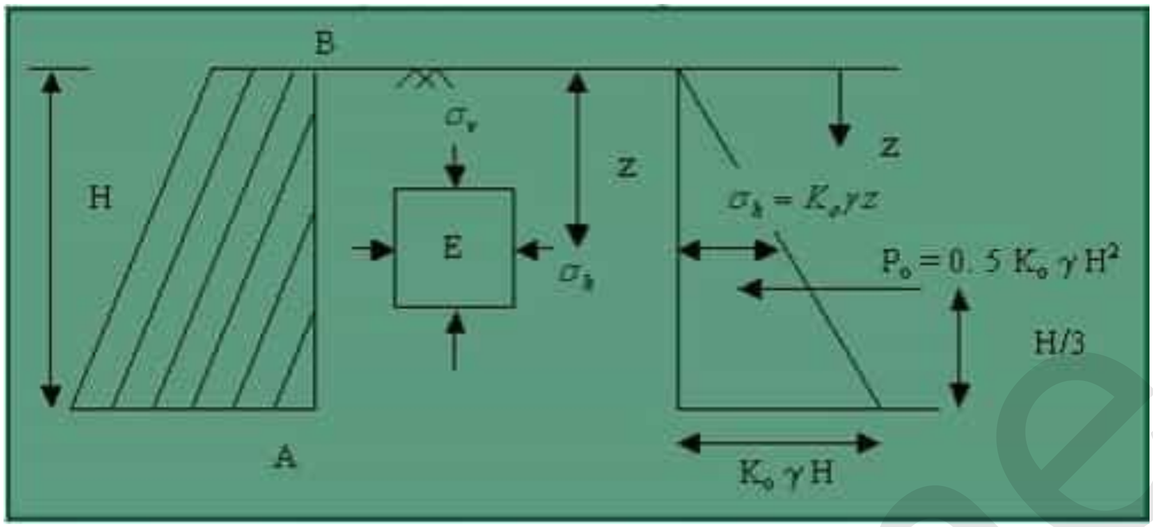
Earth Pressure Theories

Rankine's Earth Pressure Theory

The Rankine's theory assumes that there is no wall friction ($\delta = 0$), the ground and failure surfaces are straight planes, and that the resultant force acts parallel to the backfill slope.

In case of retaining structures, the earth retained may be filled up earth or natural soil. These backfill materials may exert certain lateral pressure on the wall. If the wall is rigid and does not move with the

pressure exerted on the wall, the soil behind the wall will be in a state of elastic equilibrium. Consider the prismatic element E in the backfill at depth, z, as shown in Fig.



The element E is subjected to the following pressures :
Vertical pressure = $\sigma_v = \gamma z$

Lateral pressure = σ_h, where g is the effective unit weight of the soil.

If we consider the backfill is homogenous then both σ_v and σ_h increases rapidly with depth z. In that case the ratio of vertical and lateral pressures remain constant with respect to depth, that is $\sigma_h / \sigma_v = \sigma_h / \gamma z$=constant $= K_o$, where K_o is the coefficient of earth pressure for at rest condition.

Earth Pressure at Rest

The at-rest earth pressure coefficient (K_o) is applicable for determining the active pressure in clays for strutted systems. Because of the cohesive property of clay there will be no lateral pressure exerted in the at-rest condition up to some height at the time the excavation is made. However, with time, creep and swelling of the clay will occur and a lateral pressure will develop. This coefficient takes the characteristics of clay into account and will always give a positive lateral pressure.

The lateral earth pressure acting on the wall of height H may be expressed as $\sigma_h = K_o \gamma H$.

The total pressure for the soil at rest condition, $P_o = 0.5 K_o \gamma H^2$.

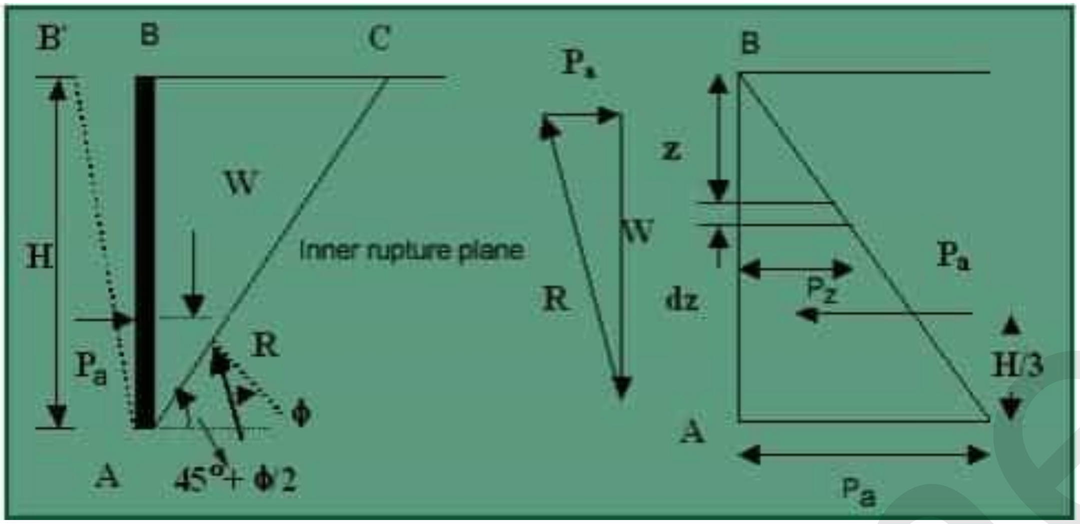
The value of K_o depends on the relative density of sand and the process by which the deposit was formed. If this process does not involve artificial tamping the value of K_o ranges from 0.4 for loose sand to 0.6 for dense sand. Tamping of the layers may increase it upto 0.8.

From elastic theory, $K_o = \mu/(1-\mu)$. ,where μ is the poisson's ratio.

According to Jaky (1944), a good approximation of K_o is given by , $K_o = 1 - \sin\phi$.

Rankine's Earth Pressure Against A Vertical Section With The Surface Horizontal With Cohesionless Backfill

Active earth pressure:



Rankine's active earth pressure in the cohesionless soil

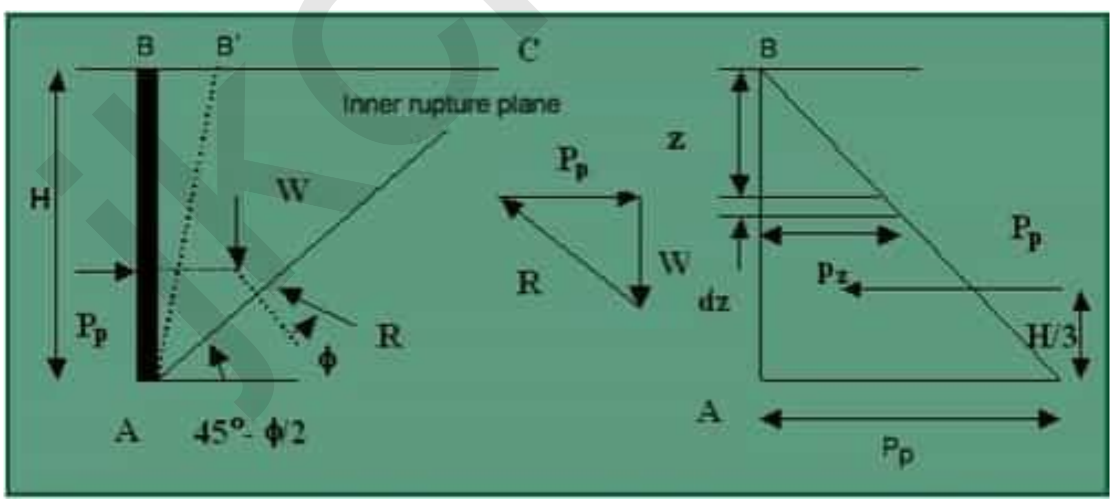
The lateral pressure acting against a smooth wall AB is due to mass of soil ABC above the rupture line AC which makes an angle of $(45^0 + \phi/2)$ with the horizontal. The lateral pressure distribution on the wall AB of height H increases in same proportion to depth.

The pressure acts normal to the wall AB.

The lateral active earth pressure at A is $P_a = K_A \gamma H$, which acts at a height H/3 above the base of the wall. The total pressure on AB is therefore calculated as follows:

$$P_a = \int_0^H p_z dz = \int_0^H K_A \gamma z dz = 0.5 K_A \gamma H^2, \text{where } K_A = \tan^2(45^0 + \phi/2)$$

Passive earth pressure:



Rankine's passive earth pressure in cohesionless soil

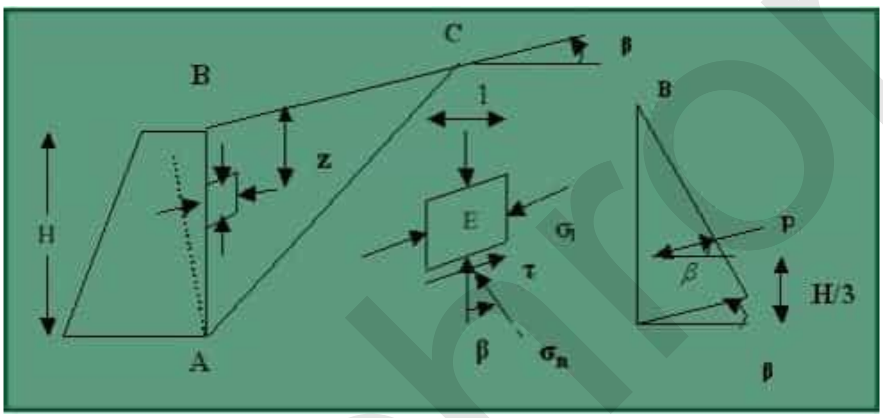
If the wall AB is pushed into the mass to such an extent as to impart uniform compression throughout the mass, the soil wedge ABC in fig. will be in Rankine's Passive State of plastic equilibrium. The inner rupture plane AC makes an angle $\left(45^0 + \phi/2\right)$ with the vertical AB. The pressure distribution on the wall is linear as shown.

The lateral passive earth pressure at A is $P_p = K_p \gamma H$. which acts at a height H/3 above the base of the wall. The total pressure on AB is therefore

$$P_p = \int_0^H p_z dz = \int_0^H K_p \gamma z dz = 0.5 K_p \gamma H^2, \text{where } K_p = \tan^2\left(45^0 + \phi/2\right)$$

Rankine's active earth pressure with a sloping cohesionless backfill surface

As in the case of horizontal backfill, active case of plastic equilibrium can be developed in the backfill by rotating the wall about A away from the backfill. Let AC be the plane of rupture and the soil in the wedge ABC is in the state of plastic equilibrium.



The pressure distribution on the wall is shown in fig. The active earth pressure at depth H is $P_a = K_A \gamma H$ which acts parallel to the surface. The total pressure per unit length of the wall is $P_a = 0.5 K_A \gamma H^2$ which acts at a height of H/3 from the base of the wall and parallel to the sloping surface of the backfill. In case of active pressure,

$$K_A = \cos\beta\left(\cos\beta - \sqrt{\left(\cos^2\beta - \cos^2\phi\right)}\right) / \left(\cos\beta + \sqrt{\left(\cos^2\beta - \cos^2\phi\right)}\right).$$

In case of passive pressure,

$$K_p = \cos\beta\left(\cos\beta + \sqrt{\left(\cos^2\beta - \cos^2\phi\right)}\right) / \left(\cos\beta - \sqrt{\left(\cos^2\beta - \cos^2\phi\right)}\right)$$

Rankine's active earth pressures of cohesive soils with horizontal backfill on smooth vertical walls

In case of cohesionless soils, the active earth pressure at any depth is given by

$P_a = K_A \gamma z$ In case of cohesive soils the cohesion component is included and the expression becomes $P_a = K_A \gamma z - 2c\sqrt{K_A}$.

When $P_a = 0, z = z_o = (2c\sqrt{K_A})/\gamma$.

This depth is known as the depth of tensile crack. Assuming that the compressive force balances the tensile force (-), the total depth where tensile and compressive force neutralizes each other is $2z_0$. This is the depth upto which a soil can stand without any support and is sometimes referred as the depth of vertical crack or critical depth $(H_c)(H_c = 4c\sqrt{K_A})/\gamma$.

However Terzaghi from field analysis obtained that $(H_c = 4c\sqrt{K_A})/\gamma - z_o$, where,

$z_o \approx H_c/2$ and is not more than that.

The Rankine formula for passive pressure can only be used correctly when the embankment slope angle equals zero or is negative. If a large wall friction value can develop, the Rankine Theory is not correct and will give less conservative results. Rankine's theory is not intended to be used for determining earth pressures directly against a wall (friction angle does not appear in equations above). The theory is intended to be used for determining earth pressures on a vertical plane within a mass of soil.

Coulomb's Wedge Theory

Coulomb (1776) developed a method for the determination of the earth pressure in which he considered the equilibrium of the sliding wedge which is formed when the movement of the retaining wall takes place. The sliding wedge is torn off from the rest of the backfill due to the movement of the wall. In the Active Earth Pressure case, the sliding wedge moves downwards & outwards on a slip surface relative to the intact backfill & in the case of Passive Earth pressure, the sliding wedge moves upward and inwards. The pressure on the wall is, in fact, a force of reaction which it has to exert to keep the sliding wedge in equilibrium. The lateral pressure on the wall is equal and opposite to the reactive force exerted by the wall in order to keep the sliding wedge in equilibrium. The analysis is a type of limiting equilibrium method.

The following assumptions are made

- The backfill is dry, cohesion less, homogeneous, isotropic and ideally plastic material, elastically undeformable but breakable.
- The slip surface is a plane surface which passes through the heel of the wall.
- The wall surface is rough. The resultant earth pressure on the wall is inclined at an angle δ to the normal to the wall, where δ is the angle of the friction between the wall and backfill.
- The sliding wedge itself acts as a rigid body & the value of the earth pressure is obtained by considering the limiting equilibrium of the sliding wedge as a whole.
- The position and direction of the resultant earth pressure are known. The resultant pressure acts on the back of the wall at one third height of the wall from the base and is inclined at an angle δ to the normal to the back. This angle is called the angle of wall friction.
- The back of the wall is rough & relative movement of the wall and the soil on the back takes place which develops frictional forces that influence the direction of the resultant pressure.

Some Graphical solutions for lateral Earth Pressure are

- **Culman's solution**
- **The trial wedge method**

- **The logarithmic spiral**

Shallow Foundations

Shallow Foundation & Bearing Capacity

Bearing Capacity

It is the load carrying capacity of the soil.

- **Ultimate bearing capacity or Gross bearing capacity (q_u)**

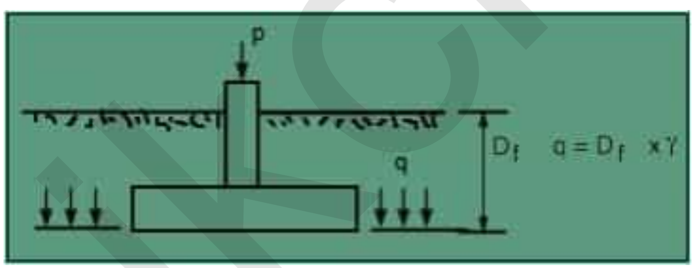
It is the least gross pressure which will cause shear failure of the supporting soil immediately below the footing.

- **Net ultimate bearing capacity (q_{un})**:

It is the net pressure that can be applied to the footing by external loads that will just initiate failure in the underlying soil. It is equal to ultimate bearing capacity minus the stress due to the weight of the footing and any soil or surcharge directly above it. Assuming the density of the footing (concrete) and soil (**γ**) are close enough to be considered equal, then

$$q_{nu} = q_u - \gamma D_f$$

Where, D_f is the depth of footing



- **Safe bearing capacity:**

It is the bearing capacity after applying the factor of safety (FS). *These are of two types*,

Safe net bearing capacity (q_{ns}):

It is the net soil pressure which can be safety applied to the soil considering only shear failure. It is given by,

$$q_{ns} = \frac{q_{nu}}{FS}$$

Safe gross bearing capacity (q_s):

It is the maximum gross pressure which the soil can carry safely without shear failure. It is given by,

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

Allowable Bearing Pressure:

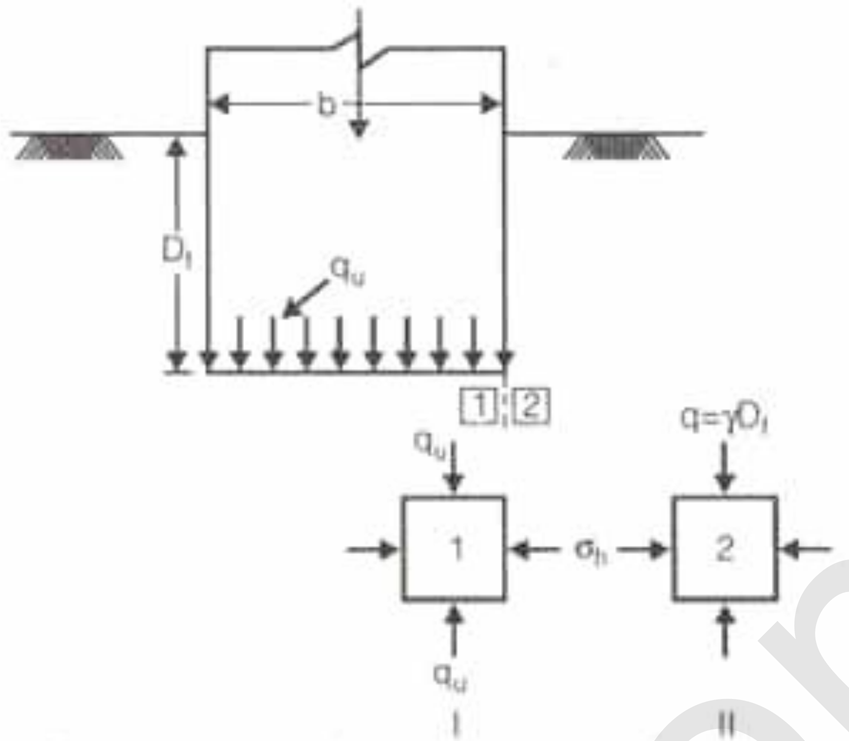
It is the maximum soil pressure without any shear failure or settlement failure

$$q_s = q_{ns} + \overline{\sigma}$$

where, q_s = Safe bearing capacity.

Method to determine bearing capacity

(i) Rankines Method (ø - soil)



Rankine's method for bearing capacity of a footing

$$q_u = \gamma D_f \tan^4\left(45° + \frac{\phi}{2}\right)$$ or

$$q_u = \gamma D_f \left(\frac{1+\sin\phi}{1-\sin\phi}\right)^2$$

(ii) Bells Theory (C - Ø)

$$q_u = CN_c + \gamma D_f N_q$$

where, N_c and N_q are bearing capacity factors.

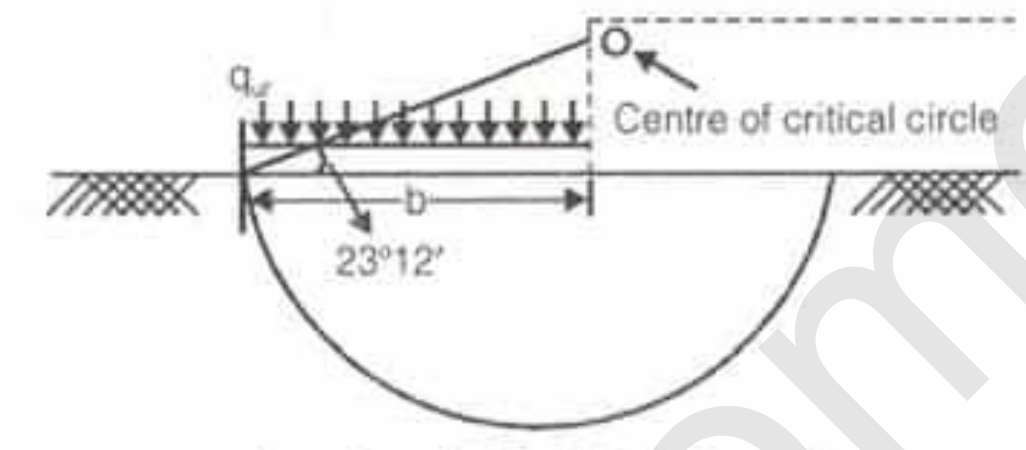
For pure clays → C = 4, q = 1

(iii) Fellinious Method: (C-soil)

- The failure is assumed to take place by slip and the consequent heaving of a mass of soil is on one side.

$$q_{ult} = \frac{W.I_r + CR}{b.I_0} \quad q_{ult} = 5.5C$$

- Location of Critical circle



Location of critical circle for surface footing in Fellenius' method

(iv) Prandtl Method: (C - Ø)

For strip footing

$$q_u = CN_c + \gamma D_f N_q + \frac{1}{2}\gamma B N_\gamma \rightarrow$$

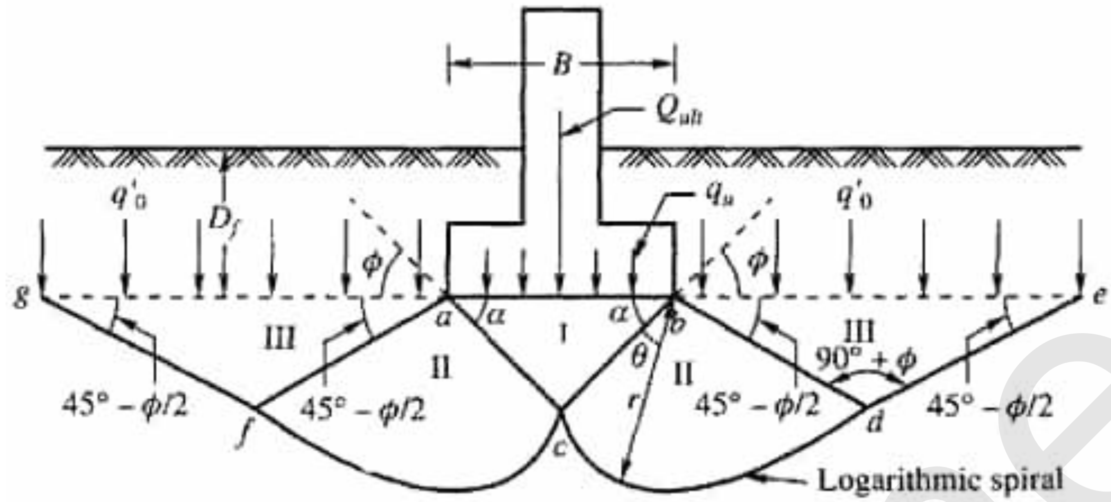
For C-soil

$$N_c = 5.14,\ N_q = 1,\ N_\gamma = 0$$

(v) Terzaghi Method (C - Ø)

Assumptions

S – Strip footing, S – Shallow foundation, G – General shear failure, H – Horizontal ground, R – Rough base



For strip footing

$$q_u = CN_c + \gamma D_f N_q + \frac{1}{2}\gamma B N_\gamma$$

For square footing

$$q_u = 1.3CN_c + \gamma D_f N_q + 0.4\gamma B N_\gamma$$

For rectangular footing

$$q_u = \left(1 + 0.3\frac{B}{L}\right)CN_c +$$

$$\gamma D_f N_q + \frac{1}{2}\left(1 - \frac{0.2B}{L}\right)\gamma B N_\gamma$$

For circular footing

$$q_u = 1.3CN_c + \gamma D_f N_q + 0.3\gamma D N_\gamma$$

where,

D = Dia of circular footing

CN_c → Contribution due to constant component of shear strength of soil.

$\gamma D_f N_q$ → Contribution due to surcharge above the footing

$\frac{1}{2}\gamma B N_\gamma$ → Contribution due to bearing capacity due to self weight of soil.

Bearing capacity factors

$$N_q = N\phi.\theta^{\pi \tan\phi}$$

where, N_ϕ = influence factor

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

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

$$N_C = \cot\phi(N_q - 1)$$

For C-soil:

$N_C = 5.7$, $N_q = 1$, $N_\gamma = 0$

(vi) Skemptons Method (c-soil)

This method gives net ultimate value of bearing capacity.

Applicable for purely cohesive soils only.

$$q_{nu} = CN_c$$

For strip footing.

$$N_C = 5 to 7.5$$

For circular and square footing.

$$N_C = 6 to 9.0$$

Values of N_C

$$\frac{D_f}{B} = 0 i.e.$$

- at the surface.
  Then N_C = 5 For strip footing
  N_C = 6.0 For square and circular footing.
  where D_f = Depth of foundation.
- If

$$0 \le \frac{D_f}{B} \le 2.5$$

$$N_C = 5\left[1 + 0.2\frac{D_f}{B}\right],$$

for strip footing

$$N_C = 6\left[1 + 0.2\frac{D_f}{B}\right],$$

For square and circular footing.
B =D in case of circular footing.

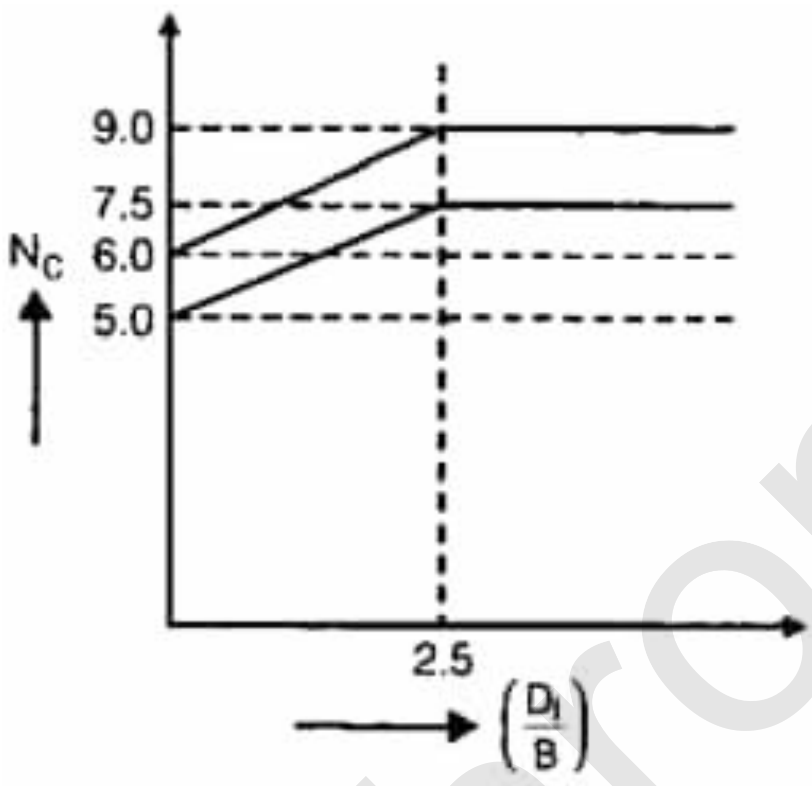
$$N_C = 5\left[1 + 0.2\frac{B}{L}\right]\left[1 + 0.2\frac{D_f}{B}\right]$$

for rectangular footing

- if

$$\frac{D_f}{B} \le 2.5$$

N_C =7.5
for strip footing
N_C = 9.0 for circular, square and rectangular footing.



(vii) Meyorhoff's Method → (C - ∅ soil)

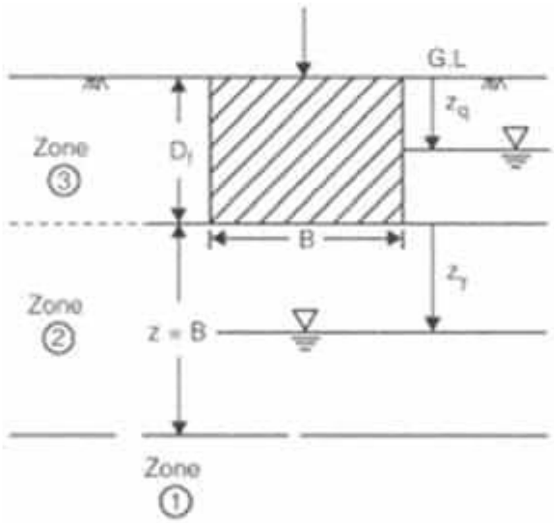
$$q_u = CN_c.s_c.d_c.i_c + \gamma D_f N_q.s_q.d_q.i_q + \frac{1}{2}\gamma BN_\gamma s_\gamma.d_\gamma.i_\gamma$$

(viii) IS code:

$$q_{nu} = CN_c.s_c.i_c.d_c + \gamma D_f(N_q - 1)s_q.i_a.d_c + \frac{1}{2}B\gamma N_\gamma s_\gamma d_\gamma$$

Effect of Water Table on Bearing Capacity of Soil

$$q_u = CN_c + \gamma D_f N_q R_q^* + \frac{1}{2}\gamma BN_\gamma R_q^*$$



where R_q^* and R_γ^* are water table correction factor.

$$R_q^* = \frac{1}{2}\left[1 + \frac{z_q}{D_f}\right] R_\gamma^* = \frac{1}{2}\left[1 + \frac{z_\gamma}{B}\right]$$

when $0 \le z_q \le D_f when 0 \le z_\gamma \le B.$

If $z_\gamma > B$ they $R_\gamma^* = 1$

If $z_\gamma \le 0$ they

If water table rise to G.L

$R_q^* = \frac{1}{2}$ and $R_\gamma^* = \frac{1}{2}$

Plate Load Test

1. Significant only for cohesionless.

2. Short duration test hence only results in immediate settlement.

(i) $\frac{q_{uf}}{q_{up}} = \frac{B_f}{B_p}$ (ii) $q_{uf} = q_{up}$

..for Ø=soil ... for C-soil

If plate load test carried at foundation level then

$$S_{f\,corrected} = S_f \times \left[\frac{1}{1+\frac{D_2}{B_f}}\right]^{0.5}$$

(iii)

$$\frac{S_f}{S_p} = \left[\frac{B_f(B_p+0.3)}{B_p(B_f+0.3)}\right]^2$$

(iv) $\frac{S_f}{S_p} = \frac{B_f}{B_p}$

... for dense sand. ... for clays

(v) $\frac{S_f}{S_p} = \left(\frac{B_f}{B_p}\right)^{n+1}$

... for silts.

where,

q_{uf} =Ultimate bearing capacity of foundation

q_{up} = Ultimate bearing capacity of plate

S_f = Settlement of foundations

S_p = Settlement of plate

B_f = Width of foundation in m

B_p = Width of plate in m

Housels Approach

$$Q_P = mA_p + nP_p$$
$$Q_f = mA_f + nP_f$$

where, Q_p = Allowable load on plate m and n are constant

P = Perimeter A_p = Area of plate

A_f = Area of foundation

Standard Penetration Test

Significant for Granular Soils

(i) $N_1 = N_0 \frac{350}{(\bar{\sigma}+70)}$ and $\bar{\sigma} \not> 280$

where, N_1 = Overburden pressure correction

N_0 = Observed value of S.P.T. number.

= Effective overburden pressure at the level of test in kM/m^2.

(ii) For Saturated $\bar{\sigma}$ fine sand and silt, when $N_1 > 15$

$$N_2 = \frac{1}{2}(N_1 - 15) + 15$$

where, N_2 = Dilatancy correction or water table correction.

$N_q + N_\gamma$ related to N value using peck Henson curve or (code method)

- Teng's formula relate N value with reading capacity of granular soil.

Pecks Equation

$$q_{a\,net} = 0.44NS = C_w kN/m^2$$

$$C_w = 0.5\left(1 + \frac{D_w}{D_f + B}\right)$$

D_w = depth of water table below G.L

D_f = Depth of foundation

B = Width of foundation

N = Avg. corrected S.P.T. no.

S = Permissible settlement of foundation

C_w = Water table correction factor

$q_{a\,net}$ = Net allowable bearing pressure.

Teng's Equations

$$q_{ns} = 1.4(N-3)\left(\frac{B+0.3}{2B}\right)^2 SC_w C_D kN/m^2$$

$$C_W = 0.5\left(1 + \frac{D_w}{B}\right)$$

$$C_D = \left(1 + \frac{D_f}{B}\right) \leq 2$$

C_w =Water table correction factor

D_w = Depth of water table below foundation level

B = Width of foundation

C_d =Depth correction factor

S = Permissible settlement in 'mm'.

I.S Code Method

$$q_{ns} = 1.38(N-3)\left(\frac{B+0.3}{2B}\right)^2 SC_w$$

q_{ns} =Net safe bearing pressure in kN/m^2

B = Width in meter.

S = Settlement in 'mm'.

I.S. Code Formula for Raft:

$$q_{ns} = 0.88NSC_w$$

C_w : Same as of peck Henson.

Meyer-Hoffs Equation

$$q_{ns} = 0.49NSC_wC_d$$

where, q_{ns} = Net safe bearing capacity in kN/m^2.

B < 1.2 m

$$C_d = \left(1+\frac{D_f}{B}\right) \le 2 \quad C_w = \frac{1}{2}\left(1+\frac{D_w}{B}\right)$$

$$q_{ns} = 0.32N\left(\frac{B+0.3}{2B}\right)^2 .S.C_d.C_w$$

B ≥ 1.2 m (where q_{ns} is in kN/m^2.

Cone Penetrations Test

(i) $$C = 1.5\left[\frac{q_c}{\sigma_0}\right]$$

where, = Static cone resistance in kg/cm^2

c = Compressibility coefficient

$\overline{\sigma_0}$ = Initial effective over burden pressure in kg/cm^2.

(ii) $S = 2.3\frac{H_0}{C}\log_{10}\left[\frac{\overline{\sigma_0} + \overline{\Delta\sigma}}{\overline{\sigma_0}}\right]$

where, 'S' = Settlement.

(iii) $q_{ns} = 3.6q_s R_w$ B > 1.2 m.

where, q_{ns} = Net safe bearing pressure in kN/m^2.

(iv) $q_{ns} = 2.7q_c.R_w$ B < 1.2 m.

where, R_w = Water table correction factor.

Deep Foundations

(i) $Q_{up} = Q_{eb} + Q_{sf}$

(ii) $Q_{up} = q_b A_b + q_s A_s$

where,

Q_{up} = Ultimate load on pile

Q_{eb} = End bearing capacity

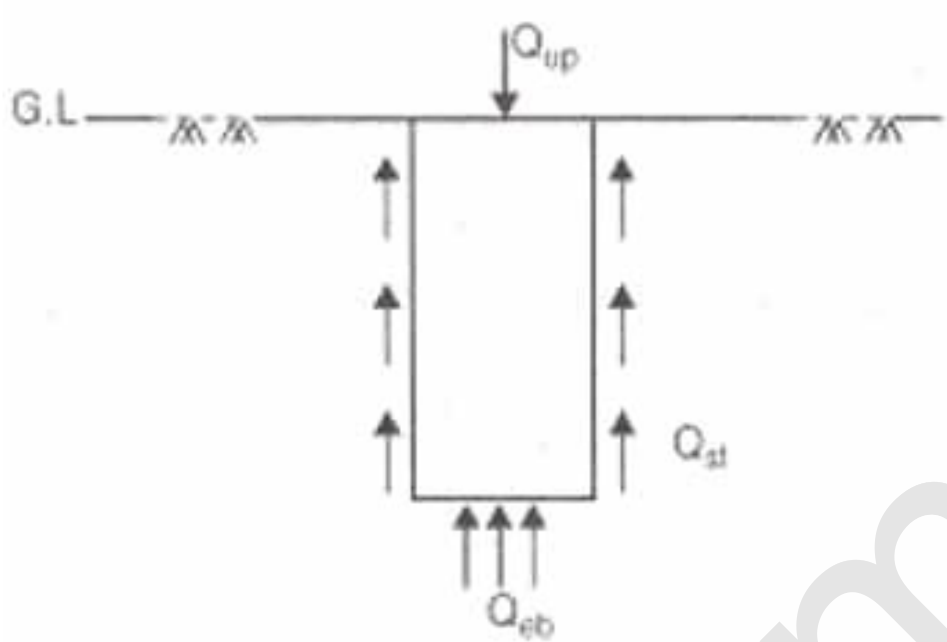
Q_{sf} = Skin friction

q_b = End bearing resistance of unit area.

q_s = Skin friction resistance of unit area.

A_b = Braking area

A_s = Surface area



(iii) $q_b \sim 9C$

where, C = Unit Cohesion at base of pile for clays

(iv) $q_s = \alpha\overline{C}$

where, α = Adhesion factor

$\alpha\overline{C} = C_a =$ Unit adhesion between pile and soil.

$\overline{C} =$ Average Cohesion over depth of pile.

(v)

$$Q_{safe} = \frac{Q_{up}}{F_s}$$

where, F_s = Factor of safety.

(vi)

$$Q_{safe} = \frac{Q_{eb}}{F_1} + \frac{Q_{sf}}{F_2}$$

$F_1 = 3$ and $F_2 = 2$

$$\simeq F_1 = F_2 = 2.5$$

(vii) For Pure Clays $Q_{up} = 9C.A_b + \alpha \bar{c} A_s$

B. Dynamic Approach

Dynamic methods are suitable for dense cohesionless soil only.

(i) Engineering News Records Formula

(a) $Q_{up} = \frac{WH}{S+C}$

(b) $Q_{ap} = \frac{Q_{up}}{6} = \frac{WH}{(S+C)}$

where,

Q_{up} = Ultimate load on pile

Q_{ap} = Allowable load on pile

W = Weight of hammer in kg.

H = Height of fall of hammer in cm.

S = Final set (Average penetration of pile per blow of hammer for last five blows in cm)

C = Constant

= 2.5 cm → for drop hammer

= 0.25 cm → for steam hammer (single acting or double acting)

(c) for drop hammer

$$Q_{ap} = \frac{WH}{6(S+2.5)}$$

(d) For single Acting Stream Hammer

$$Q_{ap} = \frac{WH}{6(S + 02.5)}$$

(e) For Double Acting Stream Hammer

$$Q_{ap} = \frac{W + ap)H}{6(S + 02.5)}$$

where P = Stream pressure

and a = Area of hammer on which pressure acts.

(ii) Hiley Formula (I.S. Formula)

$$Q_{ap} = \frac{\eta_h . \eta_b . WH}{S + \frac{C}{2}} \quad Q_{ap} = \frac{Q_{up}}{F_s}$$

where, F_s = Factor of safety = 3

η_h = Efficiency of hammer

η_b = Efficiency of blow.

η_h = 0.75 to 0.85 for single acting steam hammer

η_h = 0.75 to 0.80 for double acting steam hammer

η_h = *1 for drop hammer.*

$$\eta_b = \frac{Energy\ of\ hammer\ after\ impact}{Energy\ of\ hammer\ just\ before\ impact}$$

$$\eta_b = \frac{W + e^2 P}{W + P}\ when\ w > e.p$$

$$\eta_b = \left(\frac{W + e^2 P}{W + P}\right) - \left(\frac{W - e^2 P}{W + P}\right)^2 ..\ when\ w < e.p$$

where, w = Weight of hammer in kg.

p = Weight of pile + pile cap

e = Coefficient of restitutions

= 0.25 for wooden pile and cast iron hammer

= 0.4 for concrete pile and cast iron hammer

= 0.55 for steel piles and cast iron hammer

S = Final set or penetrations per blow

C = Total elastic compression of pile, pile cap and soil

H = Height of fall of hammer.

C. Field Method

(i) Use of Standard Penetrations Data

$$Q_{up} = 400NAb + 2\overline{N}A_s$$

where, N = Corrected S.P.T Number

$\overline{N}$ = Average corrected S.P.T number for entire pile length

$$Q_{ap} = \frac{Q_{up}}{F_s}$$

F_s = Factor of safety

= 4 → For driven pile

= 2.5 → for bored pile.

$$q_b = 400N \ and \ q_s = \overline{N}$$

(ii) Cone penetration test

$$Q_{up} = q_c A_b + \frac{\overline{q}_c}{2} A_s$$

where, q_c = static cone resistance of the base of pile in kg/cm^2

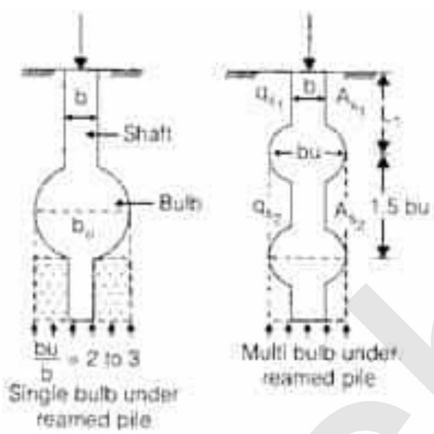
q_c = average cone resistance over depth of pile in kg/cm^2

$A_b = \frac{1}{4}(b_u)^2$ Area of bulb (m)^2

Under-Reamed Pile

An 'under-reamed' pile is one with an enlarged base or a bulb; the bulb is called 'under-ream'.

Under-reamed piles are cast-in-situ piles, which may be installed both in sandy and in clayey soils. The ratio of bulb size to the pile shaft size may be 2 to 3; usually a value of 2.5 is used.



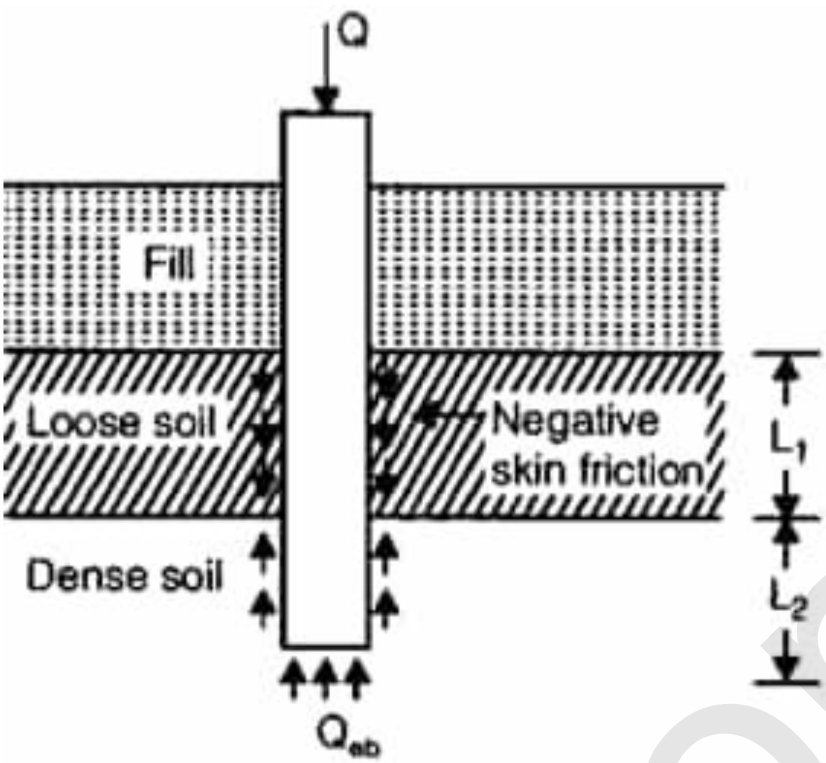
$A_{s_1} = \pi b L_1 \; q_{s_1} = \alpha C \; \alpha < 1.$

$A_{s_2} = \pi bu L_2 \; q_{s_2} = \alpha C \; \alpha = 1.$

where, b_u = dia of bulb, Spacing = 1.5 b_u.

$Q_{up} = q_b A_b + q_{s_1} A_{s_1} + q_{s_2} A_{s_2}$

Negative Skin Friction



(i) For Cohesive sol

Q_{nf} = *Perimeter.* $L_1 \alpha C$ for Cohesive soil.

where, Q_{nf} = Total negative skin frictions

$F_s = \frac{Q_{up} - Q_{nf}}{Applied\ load}$ where, F_s = Factor of safety.

(ii) For cohesionless soils

Q_{nf} = P x force per unit surface length of pile

$$= P \times \frac{1}{2} K \gamma D_n^2 . \tan \delta$$

$$Q_{nf} \frac{1}{2} P D_n^2 K . \tan \delta . \gamma$$

(friction force = μH)

Where γ = unit weight of soil.

K = Earth pressure coefficient ($K_a < K < K_p$)

δ = Angle of wall friction. ($\varphi/2<\delta<\varphi$)

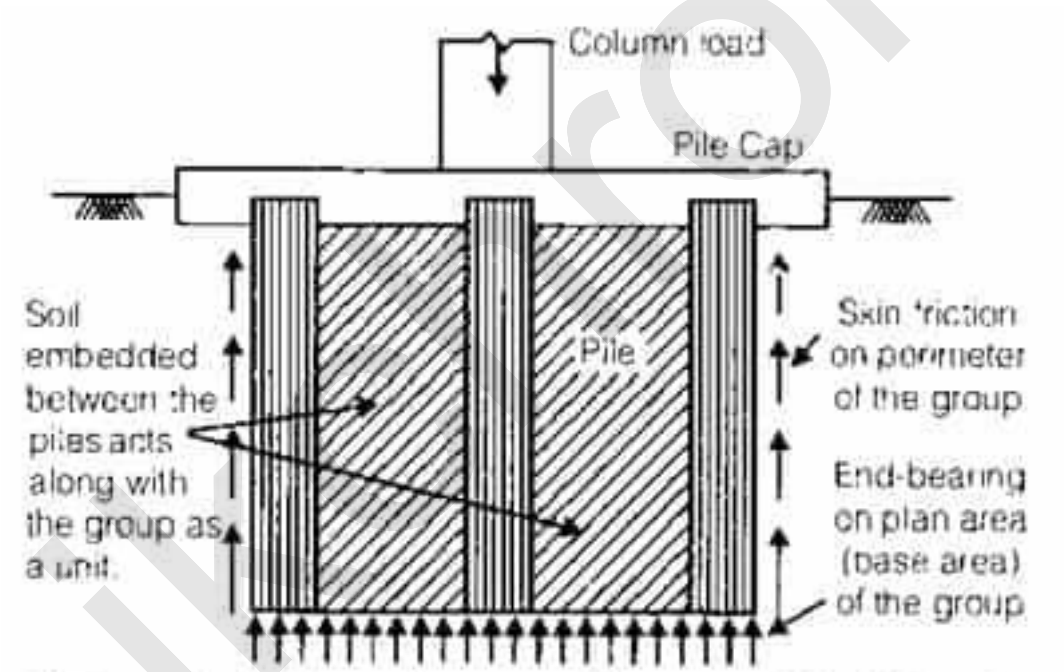
Group Action of Pile

The ultimate load carrying capacity of the pile group is finally chosen as the smaller of the

(i) Ultimate load carrying capacity of n pile ($n\ Q_{up}$)

and (ii) Ultimate load carrying capacity of the single large equivalent (block) pile (Q_{ug}).

To determine design load or allowable load, apply a suitable factor of safety.



Single equivalent large pile concept for a group (block failure)

(i) Group Efficiency (*η_g*)

$$n_g = \frac{Q_{ug}}{n.Q_{up}}$$

Q_{ug} = Ultimate load capacity of pile group

Q_{up} = Ultimate load on single pile

For sandy soil → $\eta_g > 1$

For clay soil → $\eta_g < 1$ and $\eta_g > 1$

Minimum number of pile for group = 3.

$Q_{ug} = q_b A_b + q_s A_s$

where q_b = 9C for clays

$A_b = B^2 \; q_s = \overline{C} \; A_s = 4B.L$

- **For Square Group**

Size of group, B = (n – 1) S + D

where, η = Total number of pile if size of group is x.x

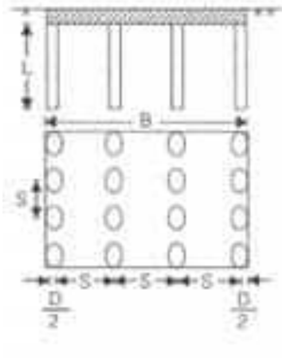
They $\eta = x^2$

- $Q_{ug} = \eta . Q_{up}$
- $Q_{ug} = \frac{Q_{ug}}{FOS}$ where, Q_{ug} = Allowable load on pile group.
- $S_r = \frac{S_g}{S_i}$

where, S_r = Group settlement ratio

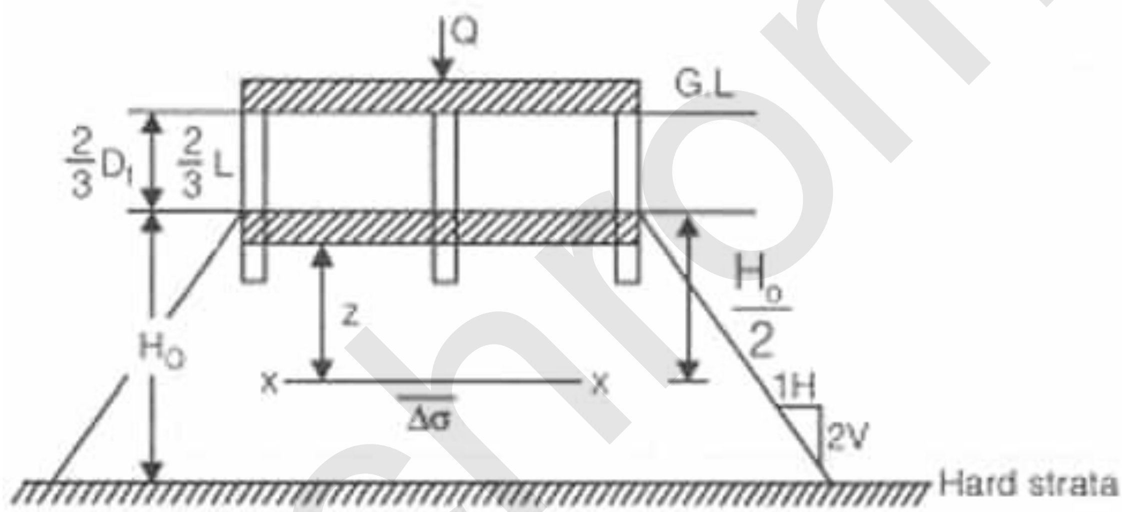
S_g = Settlement of pile group

S_i = Settlement of individual pile.



(ii) When Piles are Embended on a Uniform Clay

$$S_g = \Delta H = \frac{C_c H_0}{1+e_0}\log_{10}\frac{\overline{\sigma_0}+\overline{\Delta\sigma}}{\overline{\sigma_0}} \; and \; \overline{\sigma_0} = \frac{Q}{(B+z)^2}$$



(iii) In case of Sand

$$S_r = \frac{S_g}{S_i} = \left(\frac{4B+2.7}{B+3.6}\right)^2$$ where, B = Size of pile group in meter.