

BHUBANANANDA ODISHA SCHOOL OF ENGINEERING, CUTTACK

DEPARTMENT OF CIVIL ENGINEERING



LECTURE'S NOTE ON: GEO-TECHNICAL ENGINEERING (TH-2)

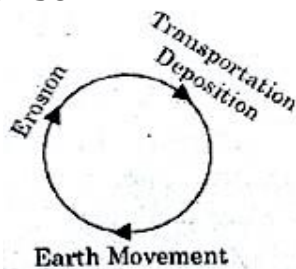
3RD SEMESTER

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CIVIL ENGINEERING DEPT BOSE, CUTTACK

DEFINITION OF SOIL

The term 'soil' in soil engineering is defined as an unconsolidated material, composed of solid particles, produced by the disintegration of rocks. The void space between the particles may contain air, water or both. The soil particles may organic matter.

ORIGIN OF SOIL

- In a broad sense, soil may be thought of as an incidental material in the geological cycle which has been going on continuously for millions of years of geological time.
- The geological cycle consists of 3 phases, Erosion; Transportation and deposition & Earth Movement.

IMPORTANT DEFINITIONS**WATER CONTENT (W)**

$$w = \frac{W_w}{W_s}; w \geq 0$$

- Water content or moisture content of a soil mass is defined as the ratio of weight of water to the weight of solids (dry weight) of the soil mass.
- $w = \frac{W_w}{W_s} \times 100$
- It is denoted by the letter symbol w and is commonly expressed as a percentage. e.g. 20%, 40% etc.
- The minimum value for water content is 0

- There is no upper limit for water content.
- Generally fine grained soils have higher water content as compared to coarse grained soil.

VOID RATIO (E)

$$e = \frac{V_v}{V_s}; e > 0$$

- Void ratio is defined as ratio of volume of voids to the volume of solids.
- It is denoted by letter symbol (e) and generally expressed as a decimal fraction eg 0.20, 0.45 etc.
- There is no upper limit of void ratio in soil suspension and in macro-porous soils like loess, V_v could be much greater than V_s .
- Void ratio of fine grained soil is generally higher than those of coarse grained soil.
- Size of void in coarse grained soil is generally larger than that in fine grained soil.

POROSITY (N)

$$n = \frac{V_v}{V} \times 100; 100 > n > 0$$

- Porosity is defined as ratio of volume of voids to the total volume expressed as a percentage. It is also known as percentage voids.
- Porosity is denoted by letter symbol (n) & is commonly expressed as a percentage.
- $V_v = V_a + V_w$
- $V = V_a + V_w + V_s$
- The porosity of soil cannot exceed 100% hence it has an upper limit of 100%.
- Both porosity & void ratio are measures of denseness (or looseness) of soil.

Note: Porosity is defined with respect to V while void ratio is defined with respect to V_s . The total volume V is a variable quantity. But, since solids are incompressible, V_s remains invariant in the total volume V of the soil. Thus in Engineering studies, void ratio serves as a useful parameter for representing change in volume under compression.

DEGREE OF SATURATION(S)

$$S = \frac{V_w}{V_v} \times 100; 0 \leq S \leq 100$$

- Degree of Saturation of a soil mass is defined as the ratio of the volume of water in the voids to the volume of voids.
- It is denoted by letter symbol S and is commonly expressed as a percentage.

$$V_v = V_a + V_w$$
- For a fully saturated soil mass $V_v = V_w$, hence for a saturated soil mass $S = 100$
- For fully dry soil mass $V_w = 0$, hence for a fully dry soil $S = 0\%$.
- For partially saturated soil mass degree of saturation varies between 0 – 100%, which is most common condition in nature.

PERCENTAGE AIR VOIDS(n_a)

$$n_a = \frac{V_a}{V} \times 100$$

- Percentage air voids of a soil mass are defined as the ratio of the volume of air voids to the total volume of the soil mass.
- It is denoted by letter symbol (n_a) and commonly expressed as a percentage.

AIR CONTENT(a_c)

$$a_c = \frac{V_a}{V_v}$$

- Air content of a soil mass is defined as a ratio of volume of air voids to the total

volume of voids. It is denoted by letter symbol a_c and commonly expressed as a percentage.

BULK UNIT WEIGHT(γ_t)

$$\gamma_t = \frac{W_t}{V} = \frac{W_s + W_w}{V_s + V_v + V_a}$$

- Bulk unit weight of soil mass is defined as the weight per unit volume of soil mass.
- It is denoted by letter symbol γ_t and is generally expressed as $\frac{\text{kN}}{\text{m}^3}$, $\frac{\text{N}}{\text{m}^3}$, $\frac{\text{kgf}}{\text{cm}^3}$

Note: Density(γ) is mass per unit volume

UNIT WEIGHT OF SOLIDS(γ_s)

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

- Unit weight of solids is the weight of soil solids per unit volume of solids alone. It is also called as “absolute unit weight” of a soil.
- It is denoted by letter γ_s .

UNIT WEIGHT OF WATER(γ_w)

$$\gamma_w = \frac{W_w}{V_w}$$

- Unit weight of water is the weight per unit volume of water.
- It is denoted by letter symbol γ_w .

Note: The value of γ_w changes with temperature but usually we take $\gamma_w = 9.81 \text{ kN/m}^3$ which is at 4°C

DRY UNIT WEIGHT(γ_d)

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

- The saturated unit weight of soil is defined as bulk unit weight of soil mass in saturated condition.
- It is denoted by symbol γ_{sat} and has unit kN/m^3 .

SUBMERGED (BUOYANT) UNIT WEIGHT (γ_{sub})

$$\gamma_{\text{sub}} = \frac{(W_s)_{\text{sub}}}{V}$$

- Submerged unit weight is defined as submerged unit weight of soil solids per unit of total volume.
- It is denoted by symbol γ_{sub} and has unit kN/m^3 .
- When the soil exists below ground water i.e. in submerged condition, a buoyant force acts on the soil solids.
- Hence it is obvious that net weight of saturated soil solids has been reduced and this reduced mass is known as Submerged Unit Weight or Buoyant Unit Wt.

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

Note: Soil in Submerged condition will be in saturated state but soil in saturated condition need not to be submerged. For example, soil mass below water table is in submerged as well as saturated condition whereas soil mass in capillary zone is in saturated condition only.

SPECIFIC GRAVITY OF SOLIDS (G)

$$G = \frac{\gamma_s}{\gamma_w}$$

- The specific gravity of solids is defined as the ratio of the unit weight of solids (absolute unit weight of soil) to the unit weight of water of water. It is denoted by letter G and is a Unit less quantity.
- This is also known as "Absolute specific gravity" or "Grain specific gravity".

MASS SPECIFIC GRAVITY OF SOIL (G_m)

$$G_m = \frac{t}{\gamma_w}$$

- Mass specific gravity is defined as the ratio of bulk unit weight of soil to unit weight of water.

- It is denoted by letter symbol G_m and is a unitless quantity.

1.3.14 RELATIVE DENSITY (D_r)

$$(D_r) = \frac{e_{\text{max}} - e}{e_{\text{max}} - e_{\text{min}}}$$

- The relative density is a parameter used in **sandy and gravelly soils**.
- The value of e_{max} & e_{min} represents the soil in very dense and loose conditions, respectively, and are determined by a standard laboratory test.
- Loose soil have values of D_r , while dense soils high values.
- The theoretically lowest possible value of D_r is 0% and highest theoretical possible value is 100%. Thus D_r is often more useful than void ratio (e) because we can easily compare the field value to the lowest and highest possible values. According to relative density, the soil is classified as:

Relative density	Classification
0-15	Very loose
15-35	Loose
35-65	Medium dense
65-85	Dense
85-100	Very dense

SOME IMPORTANT RELATIONSHIPS

Abbreviations

w = Water Content

W_s = Weight of solid

W = Total weight of soil

V_s = Volume of solid

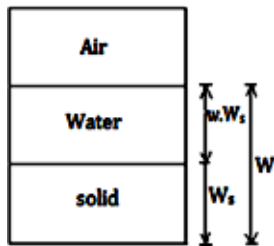
V = Total volume of soil

S = Degree of Saturation

W_w = Weight of water

V_v = Volume of voids

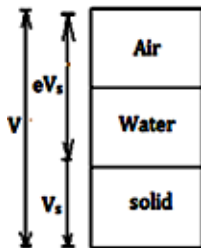
V_w = Volume of water



$$W_s = \frac{W}{1+w}$$

$$\Rightarrow W = W_s(1+w)$$

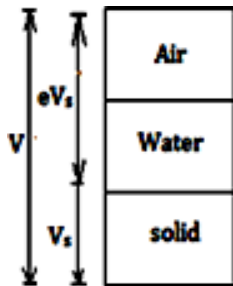
$$\Rightarrow W_s = \frac{W}{1+w}$$



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

$$\Rightarrow V_s(1+e) = V$$

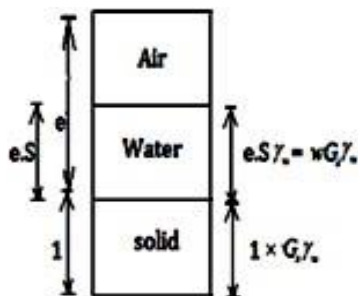
$$\Rightarrow V_s = \frac{V}{1+e}$$



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

$$\Rightarrow n = \frac{V_s}{V} = \frac{eV_s}{V_s + V_s} = \frac{eV_s}{V_s + eV_s} = \frac{e}{1+e}$$

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



$$eS = wG_s$$

$$e = \frac{V_s}{V_s} \times \frac{V_s}{V_s} \times \frac{1}{V_s} \times \frac{\gamma_w}{\gamma_s} \times \frac{W_w}{W_s}$$

$$e = \frac{1}{S} \times \frac{W_w}{W_s} \times \frac{\gamma_w}{\gamma_s} = \frac{1}{S} \times wG_s \times \frac{\gamma_w}{\gamma_s}$$

$$\Rightarrow eS = wG_s$$

$$\Rightarrow eS = wG$$

$$\gamma_t = \frac{G_s + Se}{1+e} \gamma_w$$

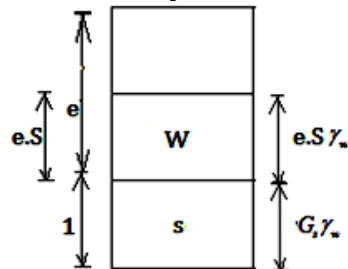
$$\gamma_t = \frac{W}{V} = \frac{W_s + W_w}{V_s + V_w} = \frac{W_s \left(1 + \frac{W_w}{W_s} \right)}{V_s \left(1 + \frac{V_w}{V_s} \right)}$$

$$= \frac{W_s(1+w)}{V_s(1+e)}$$

$$\gamma_t = \frac{G_s \gamma_w (1+w)}{1+e} = \frac{G_s + Se}{1+e} \gamma_w$$

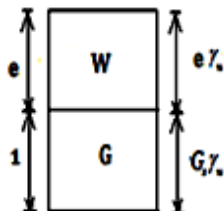
$$\Rightarrow \gamma_t = \frac{G_s + Se}{1+e} \gamma_w$$

Alternatively



$$\gamma_t = \frac{W_s + W_w}{V_s + V_w}$$

$$= \frac{G_s \gamma_w + Se \gamma_w}{1+e} \Rightarrow \gamma_t = \frac{G_s + Se}{1+e} \gamma_w$$



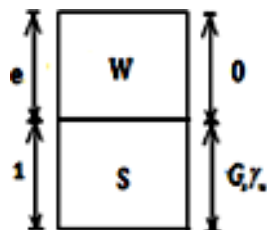
$$\gamma_{sat} = \frac{G_s + e}{1+e} \gamma_w$$

In the expression $\gamma_t = \frac{G_s + Se}{1+e} \gamma_w$

by putting $S=1$ for saturated condition

$$\gamma_{sat} = \frac{G_s + e}{1+e} \gamma_w$$

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



$$\gamma_{sat} = \frac{G_s \gamma_w}{1+e}$$

In the expression $\gamma_t = \frac{G_s + Se}{1+e} \gamma_w$

by putting $S=0$ for completely dry condition

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

$$\Rightarrow \gamma = \frac{G_s \gamma_w}{1+e}$$

$$\gamma' = \frac{G_s - 1}{1+e} \gamma_w$$

We know that $\gamma' =$ submerged unit wt =

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

$$= \frac{G_s + e}{1+e} \gamma_w - \gamma_w = \frac{G_s - 1}{1+e} \gamma_w$$

$$\Rightarrow \gamma' = \frac{G_s - 1}{1+e} \gamma_w$$

$$\gamma_d = \frac{1}{1+w} \frac{W}{W_s + W_w} = \frac{W_s(1+w)}{W_s + W_w} = \gamma_d(1+w)$$

$$\gamma_t = \frac{V}{\gamma_t} = \frac{V}{V} = \frac{V}{V} = \gamma_d(1+w)$$

$$\Rightarrow \gamma = \frac{\gamma_t}{1+w}$$

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

We know that

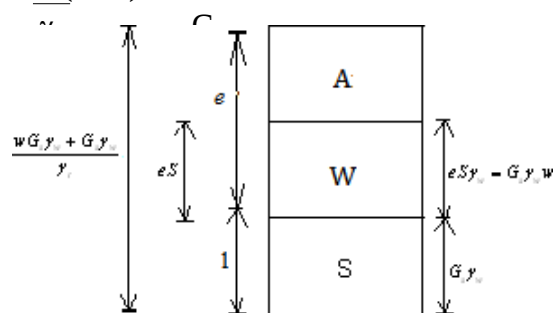
$$\gamma_d = \frac{G_s \gamma_w}{1+e} = \frac{G_s \gamma_w}{1 + \frac{w G_s}{S}}$$

$$1 - n_a = 1 - \frac{V_a}{V} = \frac{V_w + V_s}{V} = \frac{W_w}{\gamma_w V} + \frac{W_s}{G_s \gamma_w V}$$

$$= \frac{\gamma_w W}{G_s \gamma_w V} + \frac{\gamma_w (1)}{G_s \gamma_w V}$$

$$\Rightarrow \gamma_d = \frac{(1 - n_a) G_s \gamma_w}{1 + w G_s}$$

$$S = \frac{\gamma_w (1 + w) - 1}{\gamma_w (1 + w) - 1}$$



$$S = \frac{V_w}{V_v} = \frac{G_s \gamma_w \gamma_w}{G_s \gamma_w - 1}$$

$$= \frac{\gamma_d}{G_s \gamma_w (1 + w) - 1} = \frac{\gamma_w (1 + w) - 1}{\gamma_t - G}$$

Summary

1. $W = \frac{W}{S(1+w)}$
2. $V = \frac{V}{S(1+e)}$
3. $e = \frac{n}{1-n}$
4. $n = \frac{e}{1+e}$
5. $eS = w G_s$
6. $\gamma = \frac{G_s + Se}{1+e} \gamma_w$
7. $\gamma_{sat} = \frac{G_s + e}{1+e} \gamma_w$

$$8. \gamma = \frac{G_s \gamma_d}{1 + e}$$

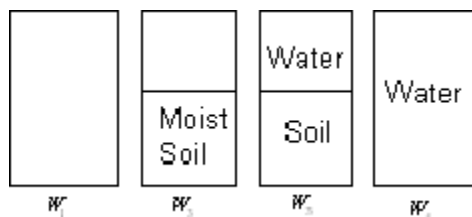
$$9. \gamma' = \frac{G_s - 1}{1 + e} \gamma_w$$

$$10. \gamma = \frac{\gamma_t}{1 + \frac{w}{G_s}}$$

$$11. \gamma = \frac{\frac{G_s \gamma_{sw}}{1 + \frac{w}{G_s}} - (1 - n_a) \gamma_{sw}}{1 + w G_s}$$

PYCNOETER METHOD

- A pycnometer is a glass jar of about 1 litre capacity and fitted with a brass conical cap by means of a screw-type cover. The cap has a small hole 6 mm diameter at its apex.
- The pycnometer method for the determination of water content can be used only if the specific gravity of solid (G) particles is known.
- First, the weight of the empty pycnometer is determined (W_1) in the dry condition. Then the sample of moist soil is placed in the pycnometer and its weight with the soil is determined (W_2). The remaining volume of the pycnometer is then gradually filled with distilled water or kerosene. The entrapped air should be removed either by gentle heating and vigorous shaking or by applying vacuum. The weight of the pycnometer, soil and water is obtained (W_3) carefully. Lastly, the bottle is emptied, thoroughly cleaned and filled with distilled water or kerosene, and its weight taken (W_4).



$$W_w = W_2 - W_1 - W_s$$

$$w = \frac{W_w}{W_s} = \frac{W_2 - W_1}{W_3 - W_1} \frac{W_3}{W_4 - W_1}$$

$$W_w = \frac{(W_3 - W_4) G_s}{G_s - 1}$$

$$w = \frac{W_2 - W_1}{(W_3 - W_4) G_s} \frac{G_s - 1}{1}$$

$$w = \left[\frac{(W_2 - W_1) (G_s - 1)}{(W_3 - W_4) G_s} \right] \frac{1}{1}$$

- Removal of entrapped air is difficult from cohesive soil. Hence this method is more suited for cohesionless soil.

INDEX PROPERTY OF SOIL

- Those properties which help to access the engineering behavior of a soil and which assist in determining its classification accurately are termed as index properties. Index properties include indices which help in determining the engineering behavior such as
 - strength
 - load-bearing capacity
 - swelling and shrinkage
 - settlement etc.
 These properties may be relating to
 - Individual soil grain
 - Aggregates soil mass
- The properties of individual particles can be determined from a remoulded, disturbed sample. These depend upon the individual grains their mineralogical composition, size and shape of grains and are independent of soil formation.

Type of soil	Index property
Coarse soil	Particle size, Relative density, Grain shape
Fine soil	Atterberg's Limit & Consistency

- The soil aggregate properties depend upon the mode of soil formation, soil history and soil structure. These properties should be determined from undisturbed samples or preferably from in situ tests.

PROCEDURE OF SIEVE ANALYSIS

- The soil sample to be tested is dried, lumps are broken if necessary, and the sample is passed through the series of sieves by shaking.
- The fractions retained on and passing 2 mm IS Sieve are tested separately.
- An automatic sieve-shaker, run by an electric motor, may be used; about 10 to 15 minutes of shaking is considered adequate.
- Larger particles are caught on the upper sieves, while the smaller ones filter through to be caught on one of the smaller underlying sieves.
- The material retained on any particular sieve should naturally include that retained on the sieves on top of it, since the sieves are arranged with the aperture size decreasing from top to bottom.
- The weight of material retained on each sieve is converted to a percentage of the total sample.
- The percentage material finer than a sieve is obtained by subtracting this from 100.
- The material passing the bottom-most sieve, which is usually the 75-μ sieve, is used for conducting sedimentation analysis for the fine fraction.
- If the soil is **clayey in nature** the fine fraction cannot be easily passed through the 75-μ sieve in the dry condition.
- In such a case, the material is to be washed through it with water (preferably mixed with 2 gm of sodium hexametaphosphate per liter), until the wash water is fairly clean

- The material which passes through the sieve is obtained by evaporation. This is called 'wet sieve analysis, and may be required in the case of cohesive granular soils'.
- The resulting data are conventionally presented as a "Grain-size distribution curve" plotted on semi-log co-ordinates, where the sieve size is on a horizontal 'logarithmic' scale, and the percentage by weight of the size smaller than a particular sieve-size is on a vertical 'arithmetic' scale.
- Logarithmic scales for the particle diameter gives a very convenient representation of the sizes because a wide range of particle diameter can be shown in a single plot.

SEDIMENTATION ANALYSIS

- The soil particles less than 75-μ size can be further analyzed for the distribution of the various grain-sizes of the order of silt and clay by 'sedimentation analysis' or 'wet analysis'.
- The soil fraction is kept in suspension in a liquid medium, usually water.
- The particles descend at velocities, related to their sizes, among other things.
- The analysis is based on 'Stokes Law'.
- As per this law, if a single sphere is allowed to fall in an infinite liquid medium without interference, its velocity first increases under the influence of gravity, but soon attains a constant value.
- This constant velocity, which is maintained indefinitely unless the boundary conditions change, is known as the 'terminal velocity'.
- The principle is obvious; coarser particles tend to settle faster than finer ones.
- By Stokes' law, the terminal velocity of the spherical particle is given by

$$v = \frac{(\gamma_s - \gamma_l) D^2}{18\mu}$$

- Stokes' Law is considered valid for particle diameters ranging from 0.2 to 0.0002 mm.
- For particle sizes greater than 0.2 mm, turbulent motion is set up and for particle sizes smaller than 0.0002 mm, Brownian motion is set up. In both these cases, Stokes' law is not valid.
- The limitations of sedimentation analysis, based on Stokes' law, are as follows:
 - i) The finer soil particles are never perfectly spherical. Their shape is plate-like or needle-like. However, the particles are assumed to be spheres, with equivalent diameters (the basis of equivalent being the attainment of the same terminal velocity as that in the case of a perfect sphere.)
 - ii) Stokes' law is applicable to a sphere falling freely without any interference, in an infinite liquid medium. This sedimentation analysis is conducted in a one-litre jar, the depth being finite; the walls of the jar could provide a source of interference to the free fall of particles near it. The fall of any particle may be affected by the presence of adjustment particles; thus, the fall may not be really free. However, it is assumed that the effect of these sources of interference is insignificant if suspension is prepared with about 50 g of soil per litre of water.
 - iii) All the soil grains may not have the same specific gravity. However, an average value is considered all right, since the variation may be insignificant in the case of particles constituting the fine fraction.
 - iv) Particles constituting to fine soil fraction may carry surface electric charges, which have a tendency to create 'flocs'. Unless these flocs are broken, the sizes calculated may be those of the flocs. Flocs can be

source of erroneous results. A deflocculating agent, such as sodium silicate, sodium oxalate, or sodium hexametaphosphate, is used to get over this difficulty.

The general procedure for sedimentation analysis, which may be performed either with the aid of a pipette or a hydrometer is as follows:

- An appropriate quantity of an oven-dried soil sample, finer than 75- μ size, is mixed with a known volume (V) of distilled water in jar.
- The sample is pretreated with an oxidizing agent and an acid to remove organic matter and calcium compounds.
- Addition of hydrogen peroxide on heating would remove organic matter. Treatment with 0.2 N hydrochloric acid would remove calcium compounds.
- Later, a deflocculating or a dispersing agent, such as sodium hexametaphosphate is added to the solution. The mixture is shaken thoroughly by means of a mechanical stirrer and the test is started, keeping the jar vertical.
- The soil particles are assumed to be uniformly distributed throughout the suspension, at the instant of commencement of the test.

PARTICLE SIZE DISTRIBUTION CURVE

- A soil sample may be either well graded or poorly (uniformly graded). A soil is said to be well graded when it has good representation of particles of all sizes. On the other hand, a soil is said to be poorly graded if it has an excess of certain size.
- For coarse grained soil, certain particle size such as D_{10} , D_{30} and d_{60} are important. The d_{10} represent a size, in mm such that 10% of the particles are finer than this size. Similarly, the soil particles finer than d_{60} size are 60 per

cent of the total mass of the sample. The size d_{10} is sometimes called the effective diameter.

Coefficient of uniformity

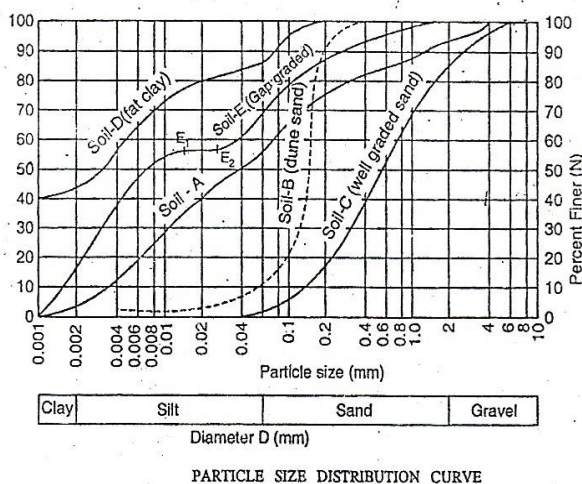
The uniformity coefficient C_u (or coefficient of uniformity) is a measure of particle size range and is given by the ratio of d_{60} and d_{10} size:

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

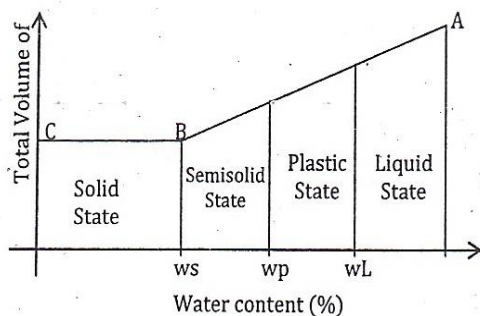
Coefficient of curvature

The shape of the particle size curve is represented by the coefficient of curvature C_c given by

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



Consistency of Soil



Liquid Limit

Water content corresponding to arbitrary limit between liquid and plastic state of soil.

Plastic Limit

Arbitrary limit between plastic and semi-solid state of consistency of soil. It is

minimum water content at which soil starts crumbling.

Shrinkage Limit

Maximum water content at which reduction in water content will not cause decrease in volume of soil mass.

Plasticity Index: $I_p = W_L - W_p$

Consistency Index: $I_c = \frac{W - w}{I_p}$

Liquidity Index: $I_L = \frac{w - w_p}{I_p}$

Flow Index: $I_f = \frac{w_1 - w_2}{\left(\log_{10} \frac{n_1}{n_2} \right)}$

Where,

W_L : water content at liquid limit

W_p : water content at plastic limit

w : natural water content

I_f : flow index

w_1 : water content corresponding to blow n_1
 w_2 : water content corresponding to blow n_2

Shrinkage Limit

$$w_s = w_1 - \left[\frac{(V_1 - V_d) \gamma_w}{w_d} \right] \times 100$$

Where,

w_1 : water content of original sample of volume V_1 .

Activity of Clay

Ratio of plasticity indexes to percentage by weight of 'clay size' the particle of size less than 2 microns.

Sensitivity of Clay

Ratio of unconfined compression strength to that of unconfined compression strength $q_u(\text{undisturbed})$

$$S_t = \frac{q_u(\text{undisturbed})}{q_u(\text{remoulded})}$$

INDIAN STANDARD SOIL CLASSIFICATION SYSTEM

- Indian Standard Soil Classification (ISSCS) system adopted by Bureau of Indian Standards is in many respects similar to the Unified Soil Classification (USC) system. But, there is one basic difference in the classification of fine-grained soils.
- The fine-grained soils in ISSSC system are subdivided into three categories of low, medium and high compressibility instead of two categories of low and high compressibility in USC system.
- Soils are divided into three broad divisions:
 - 1) Coarse-grained soils, when 50% or more of the total material by weight is retained on 75 micron IS sieve.
 - 2) Fine-grained soils, when more than 50% of the total material passes 75 micron IS sieve.
 - 3) If the soils are highly organic and contain a large percentage of organic matter and particles of decomposed vegetation, it is kept in a separate category marked as peat (P).

The basic soil classification is done on the basis of grain size.

Boulder	Cobble	Coarsegrainedsoil					FineGrained	
		Gravel		Sand				
		Coarse	Fine	Coarse	Medium	Fine	Silt	clay
>300	300-80	80-20	20-4.75	4.75-2.0	2.0-0.425	0.425-0.075	0.075-0.002	<0.002

Division of soil fraction on the basis of grain size (mm)

Basic soil components:-

Soil component	Symbol	Particle-size range and description
Boulders	None	Rounded to angular, bulky, hard, rock particle: average diameter more than 300 mm

Cobbles	None	Rounded to angular, bulky, hard, rock particle: average diameter smaller than 300 mm but retained on 80 mm IS sieve
Coarse-grained soils Gravel	G	Rounded to angular, bulky, hard rock particle: passing 80 mm IS sieve but retained on 4.75 mm IS sieve Coarse: 80 mm to 20 mm Fine: 20 mm to 4.75 mm
Sand	S	Rounded to angular, bulky, hard, rock particle: passing 4.75 mm IS sieve but retained on 75 micron Coarse: 4.75 mm to 2.0 mm Medium: 2.0 mm to 425 micron Fine: 425 micron to 75 micron
Fine-grained soils Silt	M	Particles smaller than 75 micron: identified by behavior, that is, slightly plastic or non-plastic regardless of moisture and exhibits little or no strength when air dried.
Clay	C	Particles smaller than 75 micron: identified by behavior, that is, it can be made to exhibit plastic properties within a certain range of moisture and exhibits considerable strength when air dried.
Organic matter	O	Organic matter in various sizes and stage of decomposition (no specific grain size)
Peat	Pt	Fibrous, spongy (no specific grain size)

CLASSIFICATION OF FINE GRAINED SOIL

CH-MH

CL-CI

OL-OH

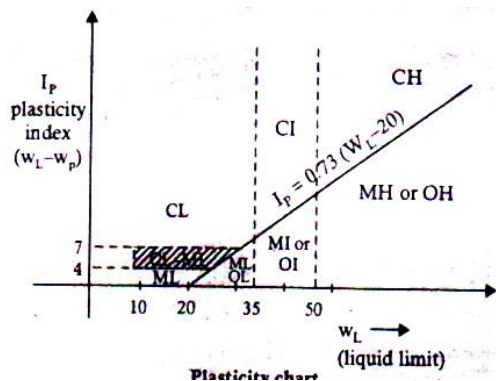
Classification of Fine Grained Soil is done on the basis of Plasticity chart.

1. First w_L & w_p are determined for sieved fraction of fine grained soil.
2. If the limit plot above A-line, classify as clay.
Further,
 - If, $35 > w_L$ classify as CL
 - If, $35 < w_L < 50$ classify as CI
 - If, $50 < w_L$ classify as CH
3. If the limit plots below A line, then we need to find out whether, soil is organic or inorganic. If its inorganic and below A line then it is silt (M).
Further,
 - If, $35 > w_L$ Classify as ML or OL
 - If, $35 < w_L < 50$ Classify as MI or OI
 - If, $50 < w_L$ Classify as MH or OH
 - GM or SM if fines are silty i.e. limit plots below A-line on plasticity chart.
 - GC or SC - limit plots above A line

Note: Dual symbols are also used when soil have equal percentage of coarse & fine grains.

- a) Determine whether the coarse-grained fraction is gravel (G) or sand (S)
- b) Determine w_L and w_p for the fine fraction.
- c) Depending on whether the limit plot above the A-line or below the A-line, classify as C or M.
- d) Based on w_L , classify as L or H
- e) Assign the dual symbols per the data obtained GM-ML, GM-MI.

GM-ML	GC-CL	SM-ML	SC-CL
GM-MI	GC-CI	SM-MI	SC-CI
GM-MH	GC-CH	SM-MH	SC-CH



- Dual symbol are used if limits plots between hatched zone i.e. IP between 4-7. Symbol = CL-ML
 - If w_L & I_p falls close to A-line, dual are used. Again if liquid limit falls close to 35% or 50% dual symbols are used. Possible dual symbols are
- | | | |
|-------|-------|-------|
| CL-ML | ML-MI | CI-CH |
| CI-MI | MI-MH | OL-OI |

PERMEABILITY

- Permeability is the ease with which water can flow through any medium.
- In soil mechanics, Permeability of soil is a soil property which describes quantitatively, the ease with which water flows through soil.
- Permeability is a very important engineering property of soils.
- Knowledge of permeability is essential in a number of soil engineering problems, such as settlement of buildings, yield of wells, seepage through and below the earth structures. It controls the hydraulic stability of soil masses.
- The permeability of soil is also required in the design of filters used to prevent piping in hydraulic structures and subgrade drainage, rate of consolidation of compressible soils and many other aspects.

DARCY'S LAW

- In one dimensional flow, discharge through fully saturated soil is given by Darcy.

$$q = k i A \text{ or } V = k i$$

Where,

q = discharge

A = Cross section area of soil corresponding to flow ' q '

k = Coefficient of permeability.

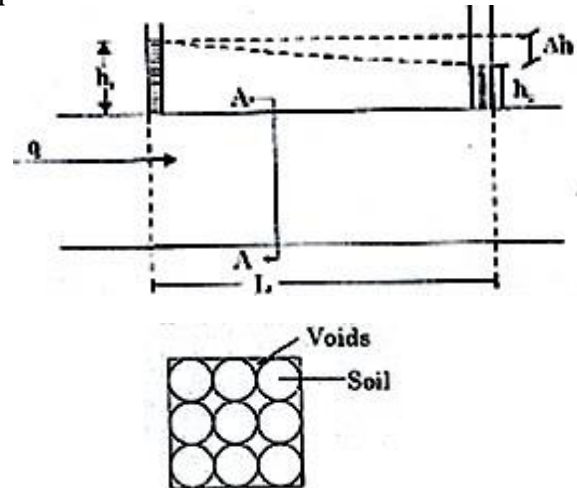
i = hydraulic gradient = $\frac{\Delta h}{L}$

$$= \frac{\text{Loss of head}}{\text{length}}$$

V = superficial velocity of flow or discharge velocity.

- The above velocity of flow is not the true velocity of flow because water flow

- only through the pore, hence area of flow should have been taken as area of pores.



Area of above cross section = A

Whereas area of pores is less than A

- The above law assumes that the flow of water takes place through the whole cross section of the soil but in reality water flows through the void present between the soil particles. Hence it is correct to call this velocity as superficial velocity.
- Superficial velocity is also called as velocity of flow or discharge velocity.
- **Actual velocity/seepage velocity (V_s)**

is given by

$$V_s = \frac{q}{A_v}, \text{ where } A_v = \text{area of voids}$$

But $q = A \cdot V$

$$A \cdot V = A_v \cdot V_s$$

$$V_s = \frac{A \cdot V}{A_v} = \frac{V}{\frac{A_v}{A}} = \frac{V}{n}$$

$$V_s = \frac{V}{n}, \text{ where } n = \text{Porosity of soil}$$

Since $n < 1$, V_s is always greater than V

- But for the sake of convenience, in engineering practice, ' V ' is used instead of ' V_s '.

COEFFICIENT OF PERMEABILITY

- Coefficient of Permeability can be defined as Superficial velocity of flow (or velocity of flow) which would occur under a unit hydraulic gradient.
- Permeability is usually expressed in the unit of velocity.
- Typical values of Permeability are as listed in the table below

Soil type	Coefficient of Permeability cm/sec	Drainage Characteristics
Gravel	>1	Previous
Sand	10^{-3}	Previous
Silt	10^{-3} – 10^{-6}	Slightly
Clay	$<10^{-6}$	Previous
		Impervious

- Coefficient of permeability divided by porosity is called coefficient of percolation (K_p)

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

Where,

K_p = Coefficient of percolation

K = Coefficient of permeability
 n = Porosity

DETERMINATION OF COEFFICIENT OF PERMEABILITY

- The coefficient of permeability of a soil can be determined using the following methods.

a) Laboratory Methods: The coefficient of permeability of a soil sample can be determined by the following methods:

- Constant-head permeability test.
- Variable-head permeability test.

- The instruments used are permeameters. The former test is suitable for relatively more pervious soils, (sand) and the latter for less pervious soils (clay).

b) Field Methods: The coefficient of permeability of a soil deposit in-situ condition can be determined by the following field methods.

- Pumping-out tests.

- Pumping-in tests.

- The pumping-out tests influence a large area around the pumping well and give an overall value of the coefficient of permeability of the soil deposit. The pumping-in test influences a small area around the hole and therefore gives a value of the coefficient of permeability of the soil surrounding the hole.

c) Indirect Methods: The coefficient of permeability of the soil can also be determined indirectly from the soil parameters by

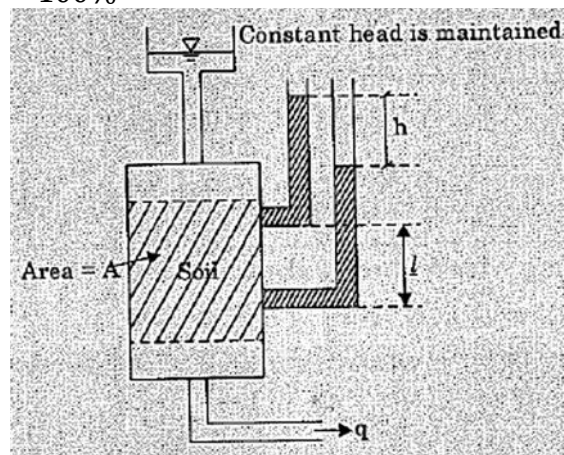
- Computation from the particle size and its specific surface.
- Computation from the consolidation test data.

- The first method is used if the particle size is known. The second method is used when the coefficient of volume change has been determined from the consolidation test of the soil.

d) Capillarity-Permeability test: The coefficient of permeability of an unsaturated soil can be determined by the capillarity-permeability test.

CONSTANT HEAD PERMEABILITY TEST

- Coefficient of permeability for **coarse soil** is determined by means of **constant-head permeability test**.
- Degree of saturation of soil should be 100%



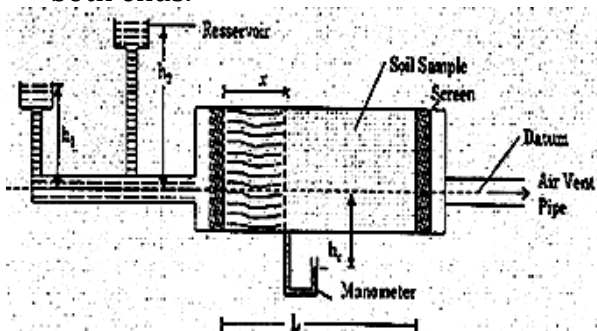
We know that, $q = kiA$

$$q = k \frac{h}{L} A$$

$$k = \frac{qL}{Ah}$$

CAPILLARITY PERMEABILITY TEST

- The coefficient of permeability for unsaturated soil can be determined by capillarity permeability test.
- A sample of partially saturated soil is placed in a 35 cm long and 4 cm diameter glass tube with screens fixed at both ends.



- Initially valve connecting to higher reservoir is closed and valve of the lower reservoir is opened.
- As the valve of lower reservoir is opened, capillary action in the soil starts drawing water into it and wetted surface advances toward the open end.
- Let the capillary head be h_c over the distance x from the left end screen. This negative capillary head causes increase in total head causing flow

$$h = h_1 + h_c$$

- Assuming uniform hydraulic gradient over the entire length x , the velocity is given by Darcy's law.

$$V = k_i = k \frac{(h_1 + h_c)}{x}$$

- The wetted surface moves further with a seepage velocity V_s . We know that

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

- But for partially saturated soil above equation is modified by taking actual

saturated porosity. As $(S \times n)$. Where S = degree of saturation.

$$V_s = \frac{k}{Sn} \left(\frac{h_1 + h_c}{x} \right)$$

$$V_s = \frac{dx}{dt} = \frac{k}{Sn} \left(\frac{h_1 + h_c}{x} \right)$$

$$x dx = \frac{k}{Sn} (h_1 + h_c) dt$$

$$\int_{x_1}^{x_2} x dx = \frac{k}{Sn} (h_1 + h_c) \int_{t_1}^{t_2} dt$$

$$\frac{x_2^2 - x_1^2}{2} = \frac{2k}{Sn} (h_1 + h_c) (t_2 - t_1)$$

- Actually capillary head is not known accurately, therefore we cannot calculate the value of k . To overcome this difficulty we close the valve connecting low reservoir and open the valve connecting high reservoir and perform the same procedure. Therefore we get a second equation also. In time t_3 to t_4 let x_3 & x_4 be the distance measured from left end.

$$\frac{x_4^2 - x_3^2}{2} = \frac{2k}{Sn} (h_1 + h_c) (t_4 - t_3)$$

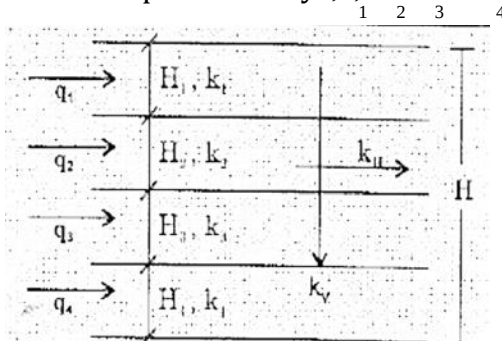
- The values of k and h_c can be obtained from above two equations.

PERMEABILITY OF STRATIFIED SOIL

- Actually in field soil is present in the form of different strata which has different permeabilities.
- For the calculation of discharge average values of permeabilities are found out for the deposit.
- There are two such average values: the average horizontal coefficient of permeability, (k_H) when the flow is parallel to the strata and the average vertical coefficient of permeability, (k_V) when the flow is normal to the strata.

- Consider a section of stratified soil as shown in figure below of varying thickness of each stratum eg:

$H_1, H_2, H_3 \& H_4$ with their respective coefficient of permeability $k_1, k_2, k_3 \& k_4$.



Note: It is assumed that each stratum, the permeability is same for both horizontal and vertical flow.

1) Horizontal flow (Parallel to bedding plane)

- Flow is taking place through all the layers at same time hence hydraulic gradient is same in each layer.

$$i_1 = i_2 = i_3 = i_4 = i \quad (\text{constant})$$

Average discharge velocity over the soil mass can be written as.

$$V = k i = \frac{1}{H} (V_1 H_1 + V_2 H_2 + V_3 H_3 + V_4 H_4) \quad 44$$

$$= \frac{1}{H} k i H + k i H + k i H + k i H \quad 111 \quad 2 \quad 22$$

$$k i_H = \frac{(k_1 H_1 + k_2 H_2 + k_3 H_3 + k_4 H_4)}{H} i$$

$$k_H = \frac{(k_1 H_1 + k_2 H_2 + k_3 H_3 + k_4 H_4)}{H_1 + H_2 + H_3 + H_4}$$

2) Vertical flow (Normal to bedding Plane)

- Let $i_1, i_2, i_3 \& i_4$ be the hydraulic gradient in different layers of thickness $H_1, H_2, H_3 \& H_4$ respectively.
- Let the total head loss be over the total thickness of soil stratum H .

- Each layer having head loss h_1, h_2, h_3, h_4 . Then constant velocity of flow is given by.

$$V = k \frac{h}{H} = k i = k i = k i = k i$$

Head loss in any layer is given by.

$$i = \frac{h_n}{H_n}$$

$$Q = k i A = k_1 i_1 A = k_2 i_2 A = k_3 i_3 A = k_4 i_4 A$$

$$k i = k_1 i_1 = k_2 i_2 = k_3 i_3 = k_4 i_4$$

$$\frac{k \times h}{H} = \frac{k_1 h_1}{H_1} = \frac{k_2 h_2}{H_2} = \frac{k_3 h_3}{H_3} = \frac{k_4 h_4}{H_4}$$

$$h_1 + h_2 + h_3 + h_4 = h$$

$$h \left(\frac{k H_1}{H k_1} + \frac{k H_2}{H k_2} + \frac{k H_3}{H k_3} + \frac{k H_4}{H k_4} \right) = h$$

$$k_v = \frac{H}{\frac{H_1}{k_1} + \frac{H_2}{k_2} + \frac{H_3}{k_3} + \frac{H_4}{k_4}}$$

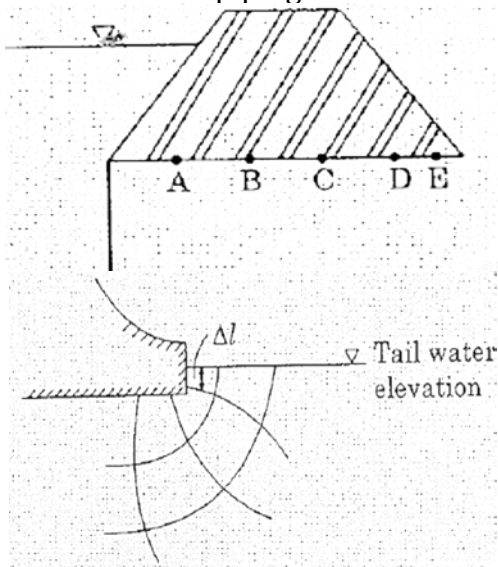
Note: k_H is always greater than k_v .

SALIENT POINTS ABOUT FLOW NET

- Shape factor is only function of boundary condition.
- Flow Net will not change if Permeability (k) of soil changes i.e. soil is changed.
- Flow net will not change if head loss during flow is changed.
- In both the above cases (i.e. (ii) & (iii)) the quantity of seepage will be different.
- Flow net is unique for a given boundary condition and if the boundary condition does not change $\frac{N_f}{N_d}$ will not change.
- Note that even if no. of flow lines or no. of equipotential lines are changed and if boundary condition does not change, $\frac{N_f}{N_d}$ will not change.

USE OF FLOW NET

1. Seepage calculation, $q = Kh \frac{N_f}{N_d}$
2. Uplift pressure calculation: Uplift pressure at any point is the pore water pressure acting vertically upward due to residual pressure head at that point.
3. Uplift pressure along the base of a masonry dam can be effectively reduced by providing vertical cut off wall at the u/s end of the base of dam. Vertical cut off wall increases the flow length and much of the head will have been lost up to A, B, C, D etc. Hence pressure at A, B, C, D reduces.
4. Calculation of exit gradient and determination of piping.



- Hydraulic gradient of flow will be max adjacent to the toe because here square is smallest in size.
- When upward hydraulic gradient approaches unity, boiling condition can occur, leading to erosion of soil.

PIPING

Piping begins near the downstream toe but may lengthen progressively towards the u/s side as seeping water gradually washes away more and more soil particles, leaving voids or pipes in the soil. Piping may work its way backwards along the base of the dam or along a bedding plane in the soil strata where the resistance is minimum.

SEEPAGE IN ANISOTROPIC SOIL

- We know that, for 2-d flow

$$K_x \frac{\partial^2 h}{\partial x^2} + K_z \frac{\partial^2 h}{\partial z^2} = 0$$

- When $K_x = K_{\text{horizontal}}$ and $K_z = K_{\text{vertical}}$, as shown in the figure

$$\frac{\partial^2 h}{K_x \partial x^2} + \frac{\partial^2 h}{\partial z^2} = 0$$

- From the above equation we can note that, when soil is anisotropic with respect to permeability i.e. $k_x \neq k_z$, the flow and equipotential lines are not necessarily to be orthogonal as because $\frac{K_z}{K_x} \neq 1$, and we cannot get the actual K_x

form of Laplace equation which is

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0$$

- But if we replace

$$x_t = x \sqrt{\frac{K_z}{K_x}}$$

$$\text{Then, } \frac{\partial^2 h}{\partial x_t^2} = \frac{\partial^2 h}{\partial x^2} \frac{K_z}{K_x}$$

$$\frac{\partial^2 h}{\partial x_t^2} + \frac{\partial^2 h}{\partial z^2} = 0$$

- Thus, new flow line and equipotential lines are drawn on transformed section with x-distance changed $x \sqrt{\frac{K_z}{K_x}}$ while keeping the vertical dimension constant.

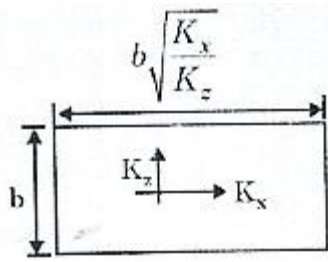
- Value of coefficient of permeability for

$$\text{transformed section } K' = \sqrt{K_x \times K_z}$$

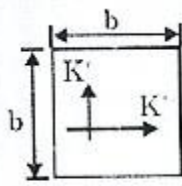
$$q = \sqrt{K_x \times K_z} \cdot H \frac{N_f}{N_d}$$

$$q = K' H \frac{N_F}{N_D}$$

Note: By taking a flow field in which flow is taking place horizontally K_x will apply



Natural section



Transformed section

$$\Delta q = K \times \frac{b}{b \sqrt{\frac{K_x}{K_z}}} \times b \times 1 = K' \times b \times 1$$

$$\boxed{\sqrt{K_x K_z} = K'}$$

ANALYTICAL METHOD

In order to find the equation of the base parabola, consider any point P on it, co-ordinates (x, y) with respect to the focus F as origin.

From the property of the parabola, we have PF = QH

$$\therefore \sqrt{x^2 + y^2} = DF + FH = x + xs \quad (\text{where } s = FH = \text{focal distance}) \quad \dots(i)$$

$$x^2 + y^2 = x^2 + s^2 + 2xs$$

$$\text{Or } x = \frac{y^2 - s^2}{2s} \quad \dots 3.1(a)$$

$$\text{And } y^2 = 2xs + s^2 \quad \dots 3.1(b)$$

This is the equation of the parabola. Taking various values of x, the values of y can be calculated from equation 3.1(b). And the parabola can be plotted. The distance can either be determined graphically or can be calculated analytically by considering the co-ordinates of point C, as follows

$$\text{From (i)} \quad s = (x^2 + \sqrt{y^2 - x^2})$$

$$ATC_x = D \text{ and } y = H$$

$$s = (\sqrt{H^2 - D}) \quad \dots 3.2$$

3.10.2 DISCHARGE THROUGH THE BODY OF THE DAM.

In order to get an expression for the discharge q through the body of the dam the present case of horizontal filter, we observe that the vertical section PQ,

$$q = KiA = k \frac{dy}{dt} y \quad (ii)$$

$$\text{But from (ii)} \quad y = (2xs + s^2)^{1/2}$$

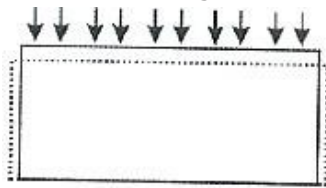
$$\therefore \frac{dy}{dt} = \frac{s}{(2xs + s^2)^{1/2}}$$

Substituting in (iii) we get

$$q = k \frac{s}{(2xs + s^2)^{1/2}} \times (2xs + s^2)^{1/2} \text{ or } q = ks$$

INTRODUCTION

- When a soil mass is subjected to a compressive force, like all other materials, its volume decreases.
- The property of the soil due to which a decrease in volume occurs under compressive forces is known as the compressibility of soil.
- The decrease in volume of soil, under stress is because of
 1. Compression and expulsion of Pore Air.
 2. Compression and Expulsion of Pore Water.
 3. Gradual readjustment of clay particles into more stable configuration.



- Reduction in volume of soil mass results in change of lateral and vertical dimension of soil mass.
- As soil being infinitely large in the lateral direction, hence the change in dimension in this direction is considered to be negligible, but there is significant change in vertical which is termed as settlement of soil.
- In other words settlement of soil is the gradual sinking of the structure due to compression of the soil below.
- Total settlement of soil is expressed as three components.

$$S_t = S_{\text{immediate}} + S_{1^{\circ}\text{-consolidation}} + S_{2^{\circ}\text{-consolidation}}$$

EFFECTIVE STRESS AND VOID RATIO RELATIONSHIP

- Equilibrium void ratio at each stress level is found out by two different methods.

1. Height of Solid Method

$$H_s = \frac{W_s}{G_s \gamma_w \cdot A}$$

$$e = \frac{H - H_s}{H_s}$$

$$e = \frac{A \times H - A \times H_s = \frac{V - V_s}{V_s}}$$

Where, W_s = dry wt of soil

G_s = Specific gravity of soil (must be known)

A = Cross section area of soil specimen

H = height of soil specimen at the end of a particular stress increment

$$H = H_1 + \Delta H$$

H_1 = Height of specimen at the beginning of stress increment

ΔH = Change in thickness under the stress increment

H_s = Height of solid

2. Change in void ratio method

Soil at the end of test is assumed to be saturated (i.e. $s = 1$)

Hence from $S \cdot e = w \cdot G_s$

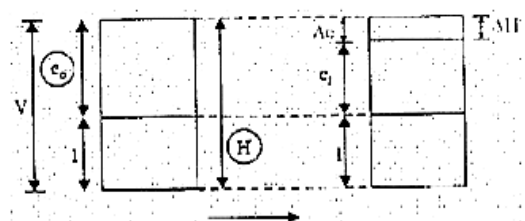
$$e_{f0} = w_{f0} \times G_s$$

Where, e_{f0} = Void ratio at the end of all stress increment i.e. at the end of the Test

w_{f0} = Water content at the end of test

(Noted after oven drying)

Due to effective stress measurement $\Delta \bar{\sigma}$



For one-dimensional consolidation

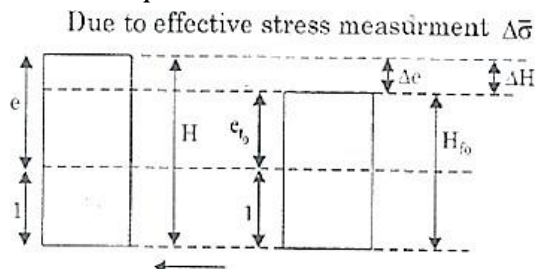
$$\frac{\Delta V}{V} = \frac{\Delta H}{H} = \frac{\Delta e}{1+e_0}$$

- Knowing H , e_0 and ΔH , can be calculated and from the above expression. Further the equilibrium void ratio at the end of every effective stress increment can be calculated as

$$e = e_0 - \Delta e$$

Another methods:

- By working backwards from the known value of e_1 & H_1 at the end of each effective stress increment we can calculate equilibrium void ratio.



$$\frac{\Delta H}{H_0} = \frac{\Delta e}{1+e_0}$$

- Knowing ΔH , H_0 & $e_0 = w_{f0} \times G_s$, Δe can be calculated. Then working backwards, $e = e_0 + \Delta e$ can be calculated.
- Thus void ratio at the end of various stress increments can be calculated.

COMPRESSIBILITY OF CLAY CAN BE REPRESENTED BY ONE OF THE FOLLOWING COEFFICIENT

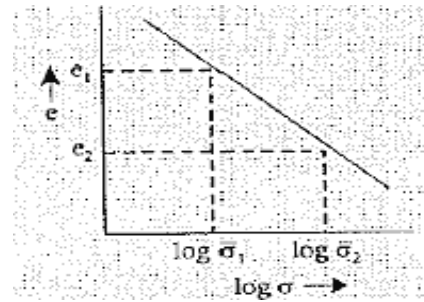
1. Compression Index (C_c):

- It is the slope of linear portion of e - $\log \sigma$

σ plot and is a dimensionless number.

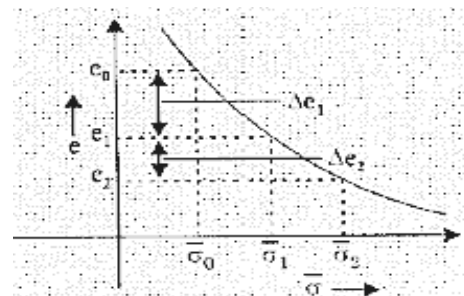
$$C_c = \frac{e_1 - e_2}{\log_{10} \sigma_2 - \log_{10} \sigma_1} = \frac{\Delta e}{\log_{10} (\sigma_2 / \sigma_1)}$$

- Not that C_c has significance only for Normally Consolidated soil.



- C_c has a constant value for a given type of soil.
- It is not a function of effective stress.
- Higher the compression index, higher is the resulting vertical deformation of clay i.e. $\Delta H \times C_c$
- The expansion part of e - $\log \sigma$ plot can be approximated to a straight line. The slope of which is called expansion index or recompression index.
- C_c is a plot between Arithmetic scale v/s log scale.

2. Coefficient of compressibility (a_v)



- Coefficient of compressibility decreases with increase in effective stress.
- With each increment of effective stress soil becomes more densified, hence resistance to further compression with same effective stress increment increases.
- a_v will be negative (-ve) as the void ratio decreases with increase in stress.
- It is a plot between arithmetic scale v/s arithmetic scale.

3. Coefficient of volume compressibility (m_v)

$$m_v = \frac{\text{Volume change per unit volume}}{\text{increase in effective stress}}$$

$$m_v = \frac{\Delta V_v / \Delta H}{\frac{\Delta e}{1+e_0} / \Delta \sigma}$$

$$m_v = \frac{\left(\frac{\Delta e}{1+e_0} \right)}{\Delta \sigma} = \frac{a_v}{1+e_0}$$

The value of m_v for a particular soil is not constant but depends on stress range over which it is calculated.

Oedometer test can also be used to calculate coefficient of consolidation (C_v), C_v indicates the rate of consolidation.

Note: Whenever possible, pre consolidation pressure for an over consolidated clay should not be exceeded construction. Note that compression will not be large if effective stress is below $\bar{\sigma}_0$ (i.e. pre consolidation stress)

COMPUTATION OF SETTLEMENT

- We will consider settlement due 1^o and 2^o consolidation in this chapter. Settlement immediate settlement will be discussed in shallow foundation.
- Below are few relationships to calculate the Primary settlement.

$$1. \frac{\Delta H}{H_0} = \frac{\Delta e}{1+e_0}$$

Find $\Delta H = S_0$ 1 consolidation

$$2. \Delta H = \frac{\Delta e}{1+e_0} \times H_0 = \frac{a_v \Delta \sigma}{1+e_0} = m_v \cdot \Delta \sigma \cdot H_0 = S_{1^{\text{st}} \text{ consolidation}}$$

- When layer thickness is more m_v & $\Delta \sigma$ changes with depth. In such a case, the layer is divided into sub layers and m_v & $\Delta \sigma$ are calculated at the centre of each sub layer.

$$S_{1^{\text{st}} \text{ consolidation}} = \sum a_{m_{vi}} \Delta \sigma_i H_i$$

- In the case of normally consolidated soil

$$\frac{\Delta H}{H_0} = \frac{\Delta e}{1+e_0}$$

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

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

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

In this case also if C_c is different in different layers of soil, then ΔH is calculated by calculating settlement in different layers and summing up the settlements to get total settlement.

- If the soil is over consolidated, then check if $\bar{\sigma}_0 + \Delta \sigma$ is greater than $\bar{\sigma}_c$ or not.

If not, use recompression index (C_r) in place of C_c .

$$\text{Hence, } \Delta H = \frac{C_r H_0}{1+e_0} \log_{10} \left(\frac{\bar{\sigma}_0 + \Delta \sigma}{\bar{\sigma}_c} \right) \text{ over}$$

consolidated

If $\bar{\sigma}_0 + \Delta \sigma > \bar{\sigma}_c$ The

n,

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

Where,

e_0 are H_0 the void ratio and height

at $\bar{\sigma}_c$.

However use of e_0 and H_0 in place of

e_0 & H_0 will not introduce much error.

Thus,

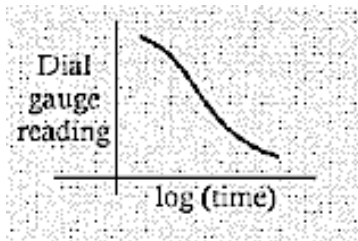
$$\Delta H = \frac{C_r H_0}{1+e_0} \log_{10} \left(\frac{\bar{\sigma}_c}{\bar{\sigma}_0} \right) + \frac{C_c H_0}{1+e_0} \log_{10} \left(\frac{\bar{\sigma}_0 + \Delta \sigma}{\bar{\sigma}_c} \right)$$

DETERMINATION OF COEFFICIENT OF CONSOLIDATION (C_v)

- The curve of deformation versus time obtained from oedometer test is similar to U versus T_v curve. This property is used to determine C_v from oedometer test by curve fitting technique.
- Two methods are used.
 - Casagrande's logarithm of time fitting method.
 - Taylor's square root of time fitting method.

(a) Casagrande's logarithm of time fitting method.

- Time for 50% condition is noted.

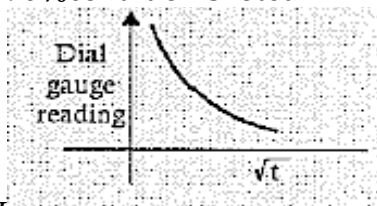


$$C_v = \frac{TH^2}{t_{50}}$$

$$T^{50} = 0.196$$

(b) In Taylor's method/Square root of time fitting method.

- Time for 90% condition is noted.

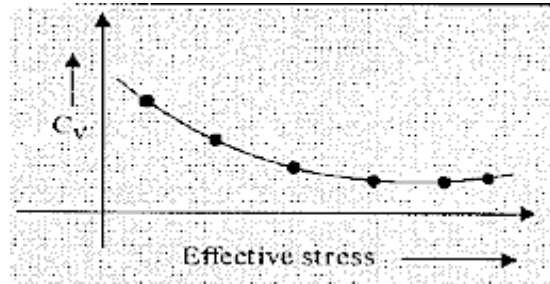


$$C_v = \frac{TH}{t_{90}}$$

$$T^{90} = 0.848$$

Note: Taylor curve is suitable because curve gets plotted as time progresses, and as soon 90% settlement point is obtained, next increment of loading is applied. Thus time to test reduces.

Note: Coefficient of consolidation can be obtained for each one of the stress increments.



C_v value obtained from any of the above curve fitting techniques is plotted against effective stress. From the above curve appropriate value of C_v for any value of effective stress can be calculated.

Note: Coefficient of consolidation can be used to calculate coefficient of permeability for fine grained soil.

- Value of C_v decreases as liquid limit of soil increases. i.e., $C_v \propto \frac{1}{w_L}$
- Value of C_v decreases as plasticity index of soil increases. i.e., $C_v \propto \frac{1}{P.I.}$

$$C_v = \frac{K(1+e_0)}{a \gamma_{vw}} = \frac{K}{m \gamma_{vw}}$$

$$K = C_v \cdot m \gamma_{vw}$$

COMPRESSION RATIO

Relative magnitude of initial compression, primary consolidation and secondary consolidation are expressed by following ratios.

(a) Initial compression ratio

$$r_0 = \frac{R_0 - R_f}{R - R_f}$$

(b) 1°-consolidation ratio (r_p)

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

(c) Secondary Compression ratio (r_s)

$$r_s = 1 - (r_0 + r_p)$$

Where,

R_i = initial dial gauge reading

R_0 = dial gauge reading corresponding

to beginning of 1° consolidation.

R_{100} = dial gauge reading corresponding

to completion of 1° consolidation. $R =$ f

final dial gauge reading $R_F - R_i$ = Can be

known from experiment.

$R_0 - R_i$ = Initial Compression

R_0, R_{90}, R_{100} are calculated from curve

fitting techniques of Taylor's method.

INITIAL COMPRESSION IS DUE TO

- (a) Compression of small quality of air in soil
- (b) Imperfect saturation
- (c) Vertical elastic compression
- (d) Lateral expansion of the specimen when imperfectly mounted.

INTRODUCTION

- Compaction of soil is the process of increasing the unit wt of soil by forcing the soil solids into a dense state and reducing the air voids.
- Compaction leads to increase in shear strength and helps improve the stability and bearing capacity of soil. It also reduces the compressibility and permeability of soil.
- This is achieved by applying static or dynamic loads to the soil.
- Compaction is measured quantitatively in terms of dry unit wt (γ_d) of the soil.
- Difference between compaction and consolidation are as tabulated below.

Compaction	Consolidation
1. Instantaneous phenomenon	1. Time dependent Phenomenon
2. Soil always partially saturated	2. Soil is completely Saturated
3. Densification due to reduction in the Volume of air voids at a given water content.	3. Volume reduction is due to expulsion of pore water from voids.
4. Specific compaction Techniques are used.	4. Consolidation occurs on account of Static load placed on the soil.

WHY DO WE NEED TO COMPACT SOIL

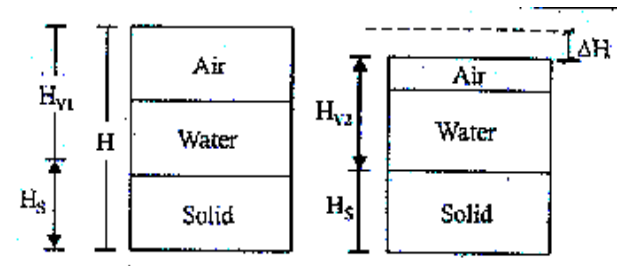
1. Max shear strength occurs at min. void ratio.
2. Large air voids if left, may to compaction under working loads causing settlement of the structure during service or may get filled with water reduces the shear strength.

3. Increase in water content is also accomplished by swelling and loss of shear strength with time.

THUS, THE VARIOUS ADVANTAGE OF COMPACTION ARE

1. Settlement can be reduced or prevented
2. Soil strength increases and stability can be improved.
3. Load carrying capacity of the pavement subgrade can be improved.
4. Undesirable volume changes (by frost action, swelling shrinkage) may be controlled.

GENERAL FORMULA FOR SETTLEMENT DUE TO COMPACTION



$$H_{v1} = e_0 H_s \quad ; e_0 = \text{initial void ratio}$$

$$H_{v2} = e_f H_s \quad ; e_f = \text{final void ratio}$$

$$\Delta H = H_{v1} - H_{v2}$$

$$\Delta H = \frac{(e_0 - e_f) H_s}{1 + e_0}$$

$$\Delta H = \frac{(e_0 - e_f) H_s}{1 + e_0}$$

COMPACTION OF COHESIONLESS SOIL

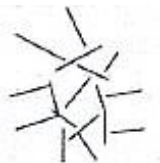
- A Cohesionless soil can be various states like
 1. Loose angular soil
 2. Dense angular soil
 3. Honey-combed state

- Soil in loose and honey-combed state must be compacted, because loose sand when subjected to vibratory loading may lead to large settlement. For example liquefaction may occur in loose-sand.
- Cohesion less soils are compacted by vibration.
- Medium and fine sands do not get compacted easily when moist because of shear strength developed by capillary forces.

COMPACTION OF COHESIVE SOIL

- Clays cannot be compacted by vibration.
- Shaking or vibration does not change the volume of clay.
- In compacting clays, position of the particles is changed by forcing the contact points along adjacent surface to a more parallel position with reduced

voids.



Loose Structure of clay before compaction (edge to face or edge to edge configuration of clay particles)



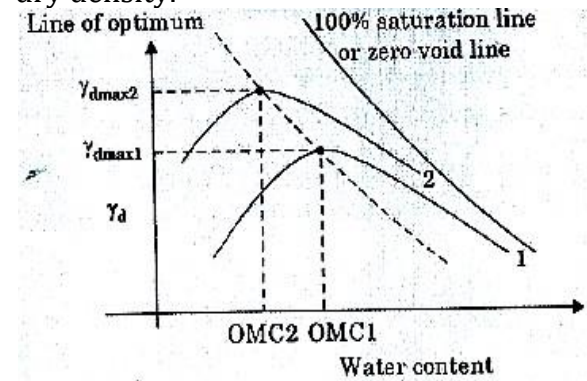
Dense Structure after compaction (Face to face configuration)

- The application of static pressure causes compaction in clayey soil.

PROCTOR TEST

- As per Proctor, a definite relationship exists between the soil moisture content and the degree of dry density to which a soil may be compacted.
- For a specific amount of compaction energy applied on the soil, there is one

moisture content termed as per Proctor optimum moisture content (OMC) at which a particular soil attains maximum dry density.



Curve 1 → Smaller compactive effort
Curve 2 → larger compactive effort

- Maximum dry unit weight obtained is a function of compactive effort and method of compaction for a particular type of soil.
- Compactive effort is a measure of mechanical energy applied to soil mass.
- Typical values of $\gamma_{dmax} = 16-20 \text{ kN/m}^3$
- Typical values of OMC = 10-20%

STANDARD PROCTOR TEST

- Weight of hammer = 2.495 kg (5.5 lb)
- Height of fall = 12" = 304.8 mm
- Volume of mould = 944 cc (1/30 cft)
- Compacted in 3 layers with 25 blows in each layer.
- Test is conducted at various moisture content and the curve as shown above is obtained.
- Compactive energy (E) applied per unit volume

$$E = \frac{NnWh}{V}$$

Where,

N = no. of blows per layer ;

n = no. of layers

w = weight of hammer; h

= height of fall

V = volume of mould.

- Standard Proctor Compactive effort per unit volume

$$= \frac{25 \times 3 \times 2.5 \times 0.305 \times 9.81 \text{ J}}{944 \times 10^{-6}} \frac{1}{\text{m}^3}$$

$$= 594.29 \text{ kJ/m}^3$$

MODIFIED PROCTOR TEST

- Weight of hammer = 4.54 kg (10 lb)
- Height of fall = 18" = 457.2 mm
- Volume of mould = 944 cc
- Compacted in 5 layers with 25 blows in each layer.
- Modified Proctor Compactive effort per unit volume

$$= \frac{25 \times 3 \times 4.54 \times 0.4572 \times 9.81 \text{ J}}{944 \times 10^{-6}} \frac{1}{\text{m}^3}$$

$$= 2696.31 \text{ kJ/m}^3$$

IMPORTANCE OF PROCTOR TEST

Before starting compaction of soil in field we must know the compaction characteristic of soil.

Proctor test gives idea about the compaction characteristics of soil.

1. It gives the density that must be achieved in the field.
2. Provides the moisture range that allows for minimum compactive effort to achieve required density.
3. Provides data on the behavior of the material in relation to various moisture content. (i.e. how permeability etc. will be affected by m/c).

LIGHT COMPACTION TEST (IS: 2720, PART VII-1974)

- Weight of hammer = 2.6 kg
- Height of fall = 310 mm
- Volume of mould = 1000 cc
- Compacted in 3 layers with 25 blows in each layer.
- Light Compaction Test Compactive effort per unit volume

$$= \frac{25 \times 3 \times 2.6 \times 0.31 \times 9.81 \text{ J}}{1000 \times 10^{-6}} \frac{1}{\text{m}^3}$$

$$= 593.014 \text{ kJ/m}^3$$

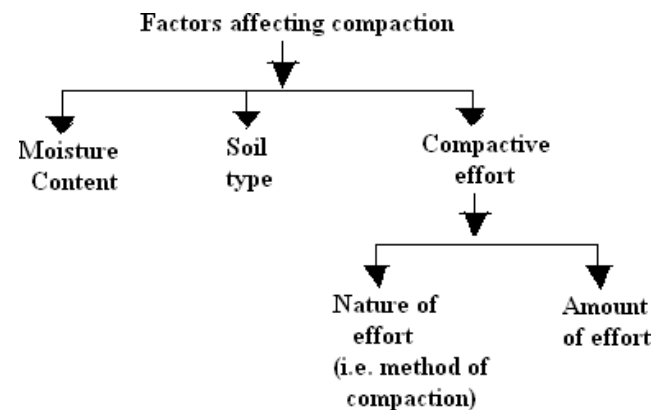
HEAVY COMPACTION TEST (IS: 2720, PART VIII- 1983)

- Weight of hammer = 4.9 kg
- Height of fall = 450 mm
- Volume of mould = 1000 cc
- Compacted in 5 layers with 25 blows in each layer.
- Heavy Compaction Test Compactive effort per unit volume

$$= \frac{25 \times 3 \times 4.9 \times 0.45 \times 9.81 \text{ J}}{1000 \times 10^{-6}} \frac{1}{\text{m}^3}$$

$$= 2703.88 \text{ kJ/m}^3$$

FACTORS AFFECTING COMPACTION



1. Moisture Content (Lubrication theory)

- At low water content soil is stiff and offers more resistance to compaction. As moisture content increases, a film of water surrounds the soil particles and which tends to lubricate the particles and make them easier to be worked around, hence they come more closer and become dense.
- This phenomenon occurs up to OMC.
- Beyond OMC water starts taking the space which would otherwise have been taken by soil,
- As γ_w is smaller than γ_s , the dry unit reduces.

LAMBE EXPLANATION

- In case of cohesive soil, there is an attractive force namely, Vanderwaal forces between two soil particles and a repulsive force due to double layer of absorbed water. While the attractive force essentially remains same, the repulsive force is more if absorbed layer thickness is more.
- If Net force is attractive flocculated structure results. However if net force is repulsive, dispersed structure results.
- At low water content, the double layer is not fully developed, i.e. repulsive force is small and net force is attractive. This makes it difficult for the particles to move closer when compactive effort is applied and a low dry unit weight is the consequence.
- As the water content increases, the double layer expands and repulsion between particles increases. The particles can easily slide over one another and get packed more easily resulting into higher dry density.
- The double layer expansion is complete at OMC, the addition of water does not cause any further expansion of double layer but water starts to occupy the space which otherwise would have been occupied by soil grains. Hence a decrease in dry unit wt. occurs beyond OMC
- This theory also explains why compaction curve is not inverted V-shape in soil which are not inverted V-shape in Cohesion less soil
- The dry unit weight can also be related to the water content and degree of Saturation.

$$\gamma_d = \frac{G\gamma_w}{1+e} = \frac{G\gamma_w}{1+\frac{wG}{S}} \quad \text{----(i)}$$
- For a given moisture content, max dry unit weight is obtained when no air voids are left. When zero air voids are achieved at a particular moisture content, degree of saturation becomes 1. Hence at various moisture content zero

air void density can be achieved by putting $S = 1$ in

$$\gamma_d = \frac{G\gamma_w}{1+\frac{wG}{S}} \cong \gamma = \frac{G\gamma_w}{1+e} \quad \text{----(ii)}$$

- When this is plotted we get zero void line.

ZERO AIR VOID DENSITY

- The maximum dry unit weight that can be achieved for a given water content by applying compaction.
- But zero air void density tough to achieve in reality because no matter how much the compaction effort is, some air voids will remain within the soil.
- This density is different from the case when water content is increased to saturate the soil.
- Zero air void line and 100% saturation line at various moisture content are same

γ_d can also be calculated as :

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

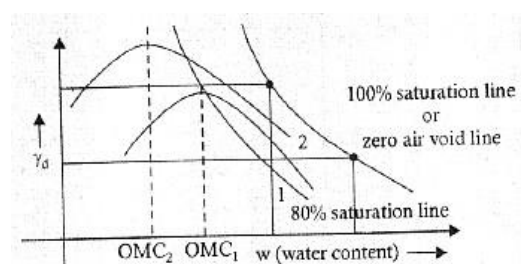
For $n_a = 0\%$ equation becomes identical to (i) equation

- It should be noted that 90% saturation line and 10% air void line are different
- 90% Saturation line

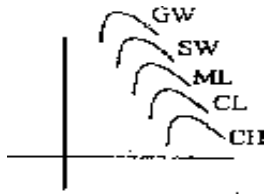
$$\gamma_d = \frac{G\gamma_w}{1+\frac{wG}{0.9}}$$

- 10% air void line

$$\gamma_d = \frac{(1-0.1)G\gamma_w}{1+wG}$$



TYPICAL COMPACTION CURVES FOR DIFFERENT SOILS



POINTS TO BE NOTED

1. Coarse grained soil is well graded and compacted to high γ_d specially if they contain some fines. However if the quantity of fines is excessive γ_d decreases.
2. Poorly graded or uniform sand lead to lowest dry unit weight values.
3. In clay soil max dry unit weight tends to decrease as plasticity increases.
4. Heavy clay with high plasticity has very high OMC.

EFFECT OF COMPACTION ON PROPERTIES OF SOILS

1. Soil structure

- The water content at which the solids is compacted plays an important role in the engineering properties of the soil. Solids compacted at a water content less than the optimum water content generally have a flocculated structure, regardless of the method of compaction.
- Solids compacted at a water content more than the optimum water content usually have a dispersed structure at point A on the dry side of the optimum, the water content is so low that the attractive forces are more predominant than the repulsive forces. This results in a flocculated structure.
- As water content is increased beyond the optimum, the repulsive forces increase and the particles get oriented into a dispersed structure.
- If the compactive effort is increased, there is a corresponding increase in the

orientation of the particles and higher dry densities are obtained, as shown by the upper curve.

2. Permeability

- The permeability of a soil depends upon the size of voids.
- Due to increase in water content for a given compactive effort there is an improved orientation of the particles and a corresponding reduction in the size of voids which cause a decrease in permeability.
- Permeability is minimum at or slightly above OMC. Beyond OMC the permeability may show a slight increase but it always remains much smaller than on dry side of optimum.
- If the compactive effort is increased, there is a corresponding increase in the orientation of the particles and higher dry densities are obtained, as shown by the upper curve.

3. Swelling

- A soil compacted dry of the optimum water content has high water deficiency and more random orientation of particles. Therefore, it imbibes more water than the sample compacted wet of the optimum, and has, therefore, more swelling.

4. Pore water pressure

- A sample compacted dry of the optimum has low water content
- The pore water pressure developed for the soil compacted dry of the optimum is therefore less than that for the same compacted wet of the optimum.

5. Shrinkage

- Soils compacted dry of the optimum shrink less on drying compared with those compacted wet of the optimum.

- The soils compacted wet of the optimum shrink more because the soil particles in the dispersed structure have nearly parallel orientation of particles and can pack more efficiently.

6. Compressibility

- The flocculated structure developed on the dry side of the optimum offers greater resistance to compression than the dispersed structure on the wet side. Therefore, the solids on the dry side are less compressible.
- It increases with an increase in the degree of saturation.
- The compressibility of a soil compacted on the wet side of the optimum is also influenced by the method of compaction. If the compaction is of kneading or impact type, it creates a more dispersed structure with a corresponding increase in the compressibility.
- If the compaction causes very large stresses, the compressibility increases due to breakdown of the structure and greater orientation of the particles.

7. Stress-Strain relationship

- The soils compacted dry of the optimum have a steeper stress-strain curve than those on the wet side.
- The modulus of elasticity for the solids compacted dry of the optimum is therefore high. Such soils have brittle failure like dense sands or oven-consolidated clays.
- The soils compacted wet of the optimum have relatively flatter stress-strain curve and a corresponding lower value of the modulus of elasticity. At higher stress level when both soils will have same strength because bonds are broken.

- At a given water content, the shear strength of the soil increases with an increase in the compactive effort till a critical of saturation is reached.
- With further increase in the compactive effort, the shear strength decreases.
- The shear strength of the compacted soils depends upon the soil type, the moulded water content, drainage conditions, the method of compaction, etc.

(d) is true for N from 4 to 10

- (a) is true for N up to 4,
 (b) is true for N from 30 to 50
 (c) is true for N from 10 to 30 and

For obtaining good quality undisturbed samples, the area ratio should be 10

percent or less.

Piston sampler causes minimum disturbance and remoulded sampler tends to disturb the soil structure completely.