BHUBANANANDAODISHASCHOOLOFENGINEERING, CUTTACK

DEPARTMENT OF CIVIL ENGINEERING

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

3RD SEMESTER

PREPARED BY : MR. KANIT PALKIA

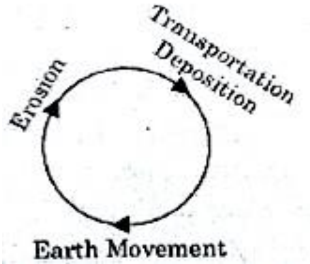
CIVIL ENGINEERING DEPT BOSE, CUTTACK

1 ORIGIN OF SOIL & SOIL WATER RELATIONSHIP

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



- Inabroadsense,soilmaybethoughtof as an incidental material in vest 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.

IMPORTAN TDEFINITIONS

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 (dryweight)ofthesoilmass.

$$w = \frac{W_w}{W_s} \times 100$$

- It is denoted by the letter symbol w and iscommonlyexpressedasapercentage. e.g.20%,40%etc.
- Theminimumvalueforwatercontentis0
- There is no upper limit for watercontent.
- 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 finegrained 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 exceed100% hence it has a upper limit of 100%.
- Both porosity & void ratio are measure of denseness (or looseness) of soil.

Note: Porosity is defined with respectto V while void ratio is defined with respect to V_s . The total volume V is a variablequantity.But,sincesolidsare incompressible, $V_s$remainsinvariantin thetotalvolumeVofthesoil.Thusin Engineeringstudies, voidratioservesas ausefulparameterforrepresenting 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.
- ItisdenotedbylettersymbolSandis commonlyexpressedasapercentage.

$V_v=V_a+V_w$

- For a fully saturated soil mass$V_v=V_w$, hence for a saturated soil mass S=100
- Forfullydrysoilmass $V_w=0$,hencefor afullydrysoilS=0%.
- For partially saturated soil mass degree of saturation varies between 0 – 100%, whichismostcommonconditionin nature.

PERCENTAGEAIRVOIDS(n_a)

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

- 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.

AIRCONTENT(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$a_c$andcommonlyexpressedasa percentage.

BULK UNIT WEIGHT(γ_t)

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

- Bulk unit weight of soil mass is defined as the weight per unit volume of soil mass.
- Itisdenotedbyletterorγor$\gamma_t$andis generallyexpressedas $\frac{kN}{m^3},\frac{N}{m^3},\frac{kgf}{cm^3}$

Note: Density(γ)ismassperunitvolume

UNIT WEIGHT OF SOLIDS(γ_s)

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

- Unit weight of solids is the weight ofsoil solids per unit volume of solids alone. It is also called as “absolute unit weight”ofa soil.
- Itisdenotedbyletterγ_s.

UNIT WEIGHTOF WATER(γ_w)

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

- Unitweightofwateristheweightper unit volume of water.
- Itisdenotedby lettersymbolγ_w.

Note: The value of $\gamma_w$changes with temperaturebutusuallywetakeγ_w=9.81 kN/m^3whichisat4°C

DRYUNITWEIGHT(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.
- Itisdenotedbysymbolγ_{sat} andhasunit kN/m^3.

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

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

- Submerged unit weight is defined as submergedunitweightofsoilsolidsper unit of total volume.
- Itisdenoted by symbol$\gamma_{sub}$and hasunit 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 BuoyantUnit Wt.

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

Note: SoilinSubmergedconditionwillbe 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 incapillary zone is in saturated condition only.

SPECIFICGRAVITYOFSOLIDS(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”.

MASSSPECIFICGRAVITYOFSOIL (G_m)

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

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

1.3.14RELATIVEDENSITY(D_r)

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

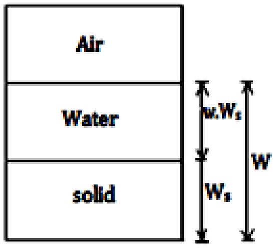
- Therelativedensityisaparameterused in **sandy and gravelly soils.**
- The value ofe_{max}&e_{min} represents the soil in very dense and loose conditions, respectively, and are determined by a standard laboratory test.
- Loose soil have values of D_r , while dense soils high values.
- Thetheoreticallylowestpossiblevalue of$D_r$is0%andhighesttheoretical possiblevalueis100%.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:

Relativedensity	Classification
0–15	Veryloose
15–35	Loose
35–65	Mediumdense
65–85	Dense
85–100	Verydense

SOMEIMPORTANTRELATIONSHIPS

Abbreviations

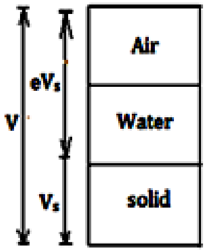
w=Water Content
W_s=Weightofsolid
W=Totalweightofsoil
V_s=Volumeofsolid
V = Total volume of soil
S=DegreeofSaturation
W_w=Weightof water
V_v=Volumeofvoids
V_w=Volumeofwater



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

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

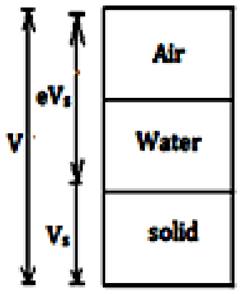
$$\Rightarrow W_s=\frac{W}{1+w}$$



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

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

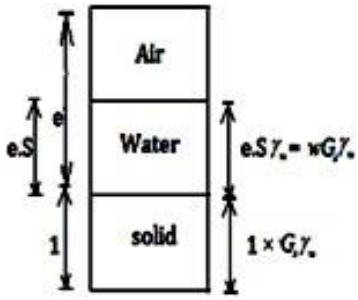
$$\Rightarrow V_s=\frac{V}{1+e}$$



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

$$\Rightarrow n=\frac{V_S}{V}=\frac{eV_S}{V_S+V_S}=\frac{eV_S}{V_S+eV_S}=\frac{e}{1+e}$$

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



$$eS=wG_S$$

$$e=\frac{V_S}{V_S}=\frac{V_S}{V_S}\times\frac{V_S}{V_S}=\frac{1}{S}\times\frac{\frac{W_w}{\gamma_w}}{\frac{W_S}{\gamma_S}}$$

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

$$\Rightarrow eS=wG_S$$

$$\Rightarrow eS=wG$$

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

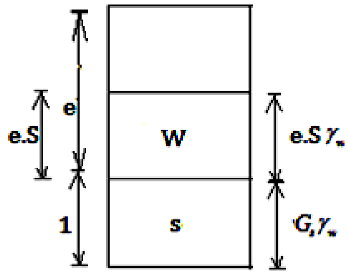
$$\gamma_t=\frac{W}{V}=\frac{W_s+W_w}{V_s+V_v}=\frac{W_s\left(1+\frac{W_w}{W_s}\right)}{V_s\left(1+\frac{V_v}{V_s}\right)}$$

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

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

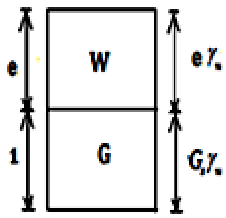
$$\Rightarrow\gamma_t=\frac{G_s+Se}{1+e}\gamma_w$$

Alternatively



$$\gamma_t=\frac{W_S+W_W}{V_S+V_V}$$

$$=\frac{G_S\gamma_w+Se\gamma_w}{1+e}\Rightarrow\gamma_t=\frac{G_2+Se}{1+e}\gamma_w$$



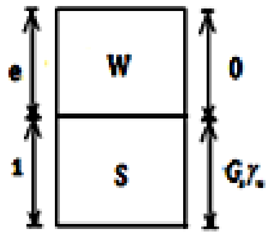
$$\gamma_{sat}=\frac{G_S+e}{1+e}\gamma_w$$

Intheexpression $\gamma_t=\frac{G_S+e}{1+e}\gamma_w$

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

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

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

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

We know that γ' = submerged unit wt = $\gamma_{sat}-\gamma_w$

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

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

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

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

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

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

We know that

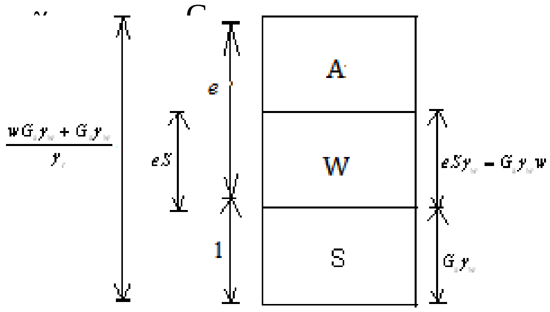
$$\gamma_d=\frac{G_s\gamma_w}{1+e}=\frac{G_s\gamma_w}{1+\frac{wG_s}{S}}$$

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

$$=\frac{\gamma_d}{G_s\gamma_w}+\frac{wW_s}{\gamma_w V}=\frac{\gamma_d}{\gamma_w}\left(w+\frac{1}{G_s}\right)$$

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

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



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

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

Summary

1. $W_s=\frac{W}{1+w}$
2. $V_s=\frac{V}{1+e}$
3. $e=\frac{n}{1-n}$
4. $n=\frac{e}{1+e}$
5. $eS=wG_s$
6. $\gamma_t=\frac{G_s+Se}{1+e}\gamma_w$
7. $\gamma_{sat}=\frac{G_s+e}{1+e}\gamma_w$

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

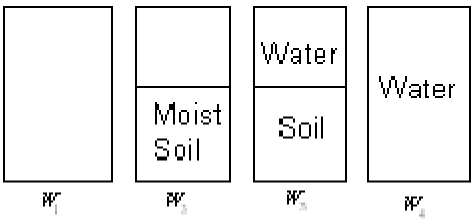
9. $\gamma' = \frac{G_s - 1}{1+e}\gamma_w$

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

11. $\gamma_d = \frac{G\gamma_w}{1+\frac{wG_s}{S}} = \frac{(1-n_a)G\gamma_w}{1+wG_s}$

PYCNOMETERMETHOD

- 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)particlesisknown.
- First, the weight of the empty pycnometer is determined (W_1) in the drycondition.Thenthesampleof**moist** soil,isplacedinthepycnometerandits weightwiththesoilisdetermined(W_2). The remaining volume of the pycnometeristhengraduallyfilledwith distilled water or kerosene. The entrappedairshouldbe removedeither by gentle heating and vigorous shaking or by applying vaccum. 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).



$W_w = W_2 - W_1 - W_s$

$$w = \frac{W_w}{W_s} = \frac{W_2 - W_1}{W_s} - 1$$

$$W_4 - W_1 = W_3 - W_1 - W_s + \frac{W_s\gamma_w}{G_s\gamma_w}$$

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

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

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

- Removal of entrapped air is difficult fromcohesivesoil.Hencethismethodis more suited for cohesionless soil.

INDEXPROPERTY OFSOIL

- 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
  a) strength
  b) load-bearingcapacity
  c) swellingandshrinkage
  d) settlementetc.
  Thesepropertiesmayberelatingto
  1. Individualsoilgrain
  2. Aggregatesoil mass

- The properties of individual particles can be determined from a remoulded, disturbed sample. These depend upon theindividualgrainstheirmineralogical composition, size and shape of grains and are independent of soil formation.

Typeofsoil	**Index property**
Coarsesoil	Particle size, Relative density, Grain shape
Finesoil	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 undisturbedsamplesorpreferablyfrom in situ tests.

PROCEDUREOFSIEVEANALYSIS

- 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 electricmotor,maybeused;about10to 15 minutes of shaking is considered adequate.
- Largerparticlesarecaughtontheupper 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 retainedon each sieveisconvertedtoapercentageofthe 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 passedthrough 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. Thisis called 'wet sieve analysis, and may be required in the case of cohesivegranular soils'.
- The resulting data are conventionally presented as a "Grain-size distribution curve"plottedonsemi-logco-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.

SEDIMENTATIONANALYSIS

- Thesoilparticleslessthan75-μsizecan be further analyzed for the distribution ofthe variousgrain-sizes ofthe orderof silt and clay by 'sedimentation analysis' or 'wet analysis'.
- Thesoilfractioniskeptinsuspensionin a liquid medium, usually water.
- Theparticlesdescendatvelocities, relatedtotheirsizes,amongother things.
- Theanalysisisbasedon'StocksLaw'.
- 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 boththese 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.Thesedimentationanalysis 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 soilper litre of water.
  iii) All the soil grains may not have the same specific gravity. However, an averagevalueisconsideredallright, since the variation may be insignificantinthecaseofparticles 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 floces are broken, the sizes calculated may be thoseoftheflocs.Flocscanbe source of erroneousresults. A deflocculatingagent,suchassodium silicate, sodium oxalate, or sodium hexametaphosphate, is used to get over this difficulty.

The general procedure for sedimentation analysis, which may beperformedeitherwiththeaid of a pipette or a hydrometer is as follows:

- An appropriate quantity of an oven-driedsoilsample,finerthan75-µsize,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 organicmatterandcalciumcompounds.
- 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 hexameta phosphate 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}, D30 and d60 are important. The d10 represent a size, in mm such that 10% of the particles are finer that this size. Similarly, the soil particles finer than d60 size are 60 per

centofthetotalmassofthesample.The size d10 is sometimes called the effective diameter.

Coefficientofuniformity

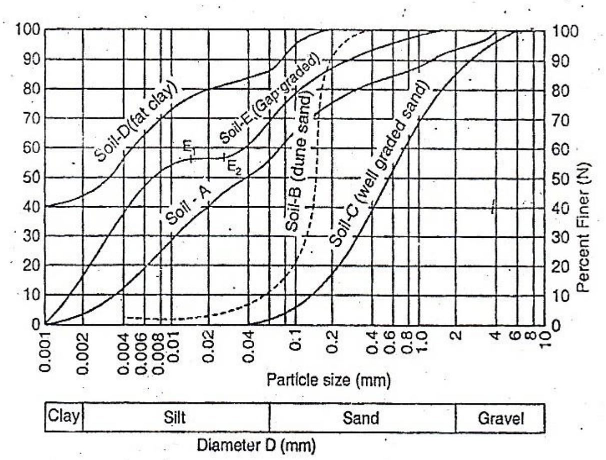
The uniformity coefficient C_u (or coefficient of uniformity) is a measure of particle size range andisgivenbytheratioofd60andd10size:

$$Cc = \frac{D60}{D10}$$

Coefficientofcurvature

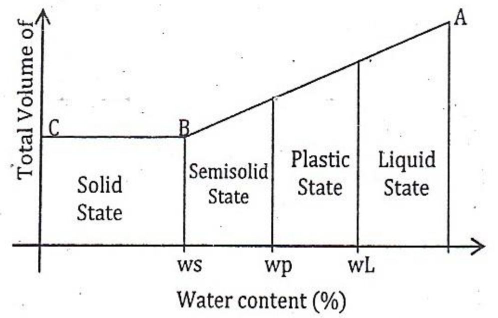
Theshapeoftheparticlesizecureis representedbytheco-efficientofthe curatureC$_c$givenby

$$C_c = \frac{(D30)}{D10XD60}$$



PARTICLE SIZE DISTRIBUTION CURVE

ConsistencyofSoil



LiquidLimit
Watercontentcorrespondingtoarbitrary limit between liquid and plastic stae of soil.

PlasticLimit
Arbitrarylimitbetweenplasticandsemi-soiledstateofconsistencyofsoil.Itis minimumwatercontentatwhichsoilstarts crumbling.

ShrinkageLimit
Maximum water content at whichreduction in water content will not cause decrease in volume of soil mass.

PlasticityIndex:$I_P = W_L - W_p$

ConsistencyIndex: $\frac{W_L - w}{Ip}$

LiquidityIndex:$I_L = \frac{w - w}{Ip}$

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

Where,
W_L: water content at liquid limit
W_P :watercontentatplasticlimit
w:natural water content
I_f :lowIndex
w_1:watercontentcorrespondingtoblowsn$_1$w$_2$:watercontentcorrespondingtoblowsn$_2$

ShrinkageLimit

$$w_s = \left[w_1 - \frac{(V_1 - V_d)\gamma w}{wd}\right] \times 100$$

Where,
w_1:watercontentoforiginalsampleof volume V_1.

Activityof Clay
Ratio of plasticity indexes to percentage by weight of ‘clay size’ the particle of size less than 2 microns.

Sensitivityof Clay

Ratio of unconfined compression strength to that of unconfined compression strength

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

2 CLASSIFICATIONOFSOIL

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% ormore of the total material by weight is retained on 75 micron IS sieve.
2) Fine-grained soils, when more than50% of the total material passes 75 micron IS sieve.
3) If the soils is highly organic and contain alargepercentageoforganicmatter andparticlesofdecomposedvegetation, it is kept in a separate category marked as peat (P).

Thebasicsoilclassificationisdoneonthe 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	0.425-075	0.075-0.002	<0.002

Divisionofsoilfractiononthebasisof grain size (mm)

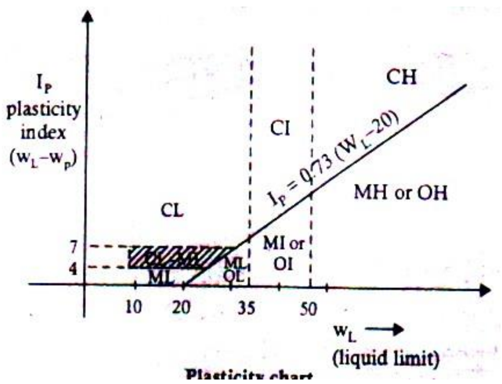
Basicsoilcomponents:-

Soil component	Symbol	Particle-sizerange and description
Boulders	None	Roundedtoangular, bulky, hard, rock particle: average diametermorethan 300 mm
Cobbles	None	Rounded to angular, bulky, hard, rock particle: average diametersmallerthan 300 mm but retained on80mmISsieve
Coarse-grainedsoils Gravel	G	Rounded to angular, bulky, hard rock particle: passing 80 mm IS sieve but retainedon4.75mmIS sieve Coarse:80mmto20 mm Fine:20mmto4.75 mm
Sand	S	Rounded to angular, bulky, hard, rock particle: passing 4.75 mm IS sieve but retainedon75micron Coarse : 4.75 mm to 2.0mm Medium:2.0mmto 425 micron Fine:425micronto75 micron
Fine-grained soils Silt	M	Particles smaller than 75 micron: identified by behavior, that is, slightlyplasticornon-plastic regardless of moisture and exhibits littleornostrength when air dried.
Clay	C	Particles smaller than 75 micron: identified by behavior, that is, it canbemadetoexhibit plastic properties withina certain range of moisture and exhibits considerable strengthwhenair dried.
Organic matter	O	Organic matter in varioussizesandstage ofdecomposition(no specific grain size)
Peat	Pt	Fibrous,spongy(no specific grain size)

CLASSIFICATIONOFFINEGRAINED SOIL

ClassificationofFineGrainedSoilisdone on the basis of Plasticity chart.

1. Firstw_L&$w_P$are determined for sieved fraction of fine grained soil.
2. IfthelimitplotaboveA-line,classifyas clay.
   Further,
   - If,35>w_L classifyas CL
   - If,35<w_L<50 classifyas Cl
   - If,50<w_L classifyas 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 belowA line then it is silt (M).
   Further,
   - If,35>w_L Classifyas ML orOL
   - If,35<w_L<50ClassifyasMIor OI
   - If,50<w_L Classifyas MH orOH
   - GM or SM if fines are silty i.e. limit plots below A-line on plasticitychart.
   - GCorSC-limitplotsaboveAline



Plasticity chart

- Dual symbol are used if limits plots between hatched zone i.e. IP between 4-7. Symbol = CL-ML
- IfW_L&I_P fallsclosertoA-line,dual are used. Again if liquid limit falls closerto35%or50%dualsymbolis used. Possible dual symbols are

CL-ML	ML-MI	CI-CH
CI-MI	MI-MH	OL-OI
CH-MH	CL-CI	OL-OH

Note: Dual symbols are also used whensoil 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) Dependingonwhetherthelimitsplot abovetheA-lineorbelowthea-line, classifyas C orM.
d) Basedonw_L,classifyasL,IorH
e) Assign the dual symbolas 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

3 EFFECTIVESTRESS,CAPILLARITY&PERMEABILITY

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 theearth 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.

$\boxed{q = kiA \text{ or } V = ki}$

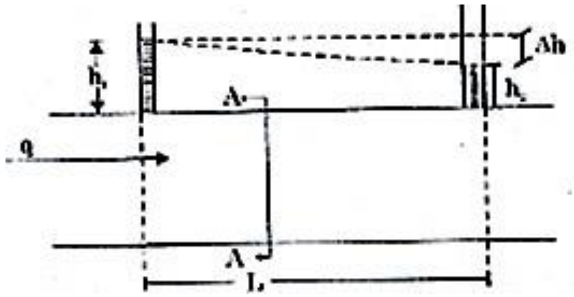
Where,
q=discharge
A = Cross section area of soil corresponding to flow 'q'
k= Coefficientof permeability.

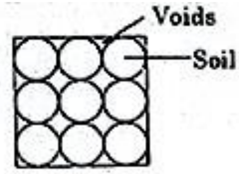
i=hydraulicgradient = $\frac{\Delta h}{L}$

$= \frac{\text{Lossofhead}}{\text{length}}$

V=superficialvelocityofflowordischarge velocity.

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





Area of above cross section =A
Whereasareaof poresislessthanA

- 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 correcttocallthisvelocityassuperficial velocity.
- Superficial velocity is also called as velocity of flow or discharge velocity.
- **Actualvelocity/seepagevelocity**(V_s) isgivenby

$V_s = \frac{q}{A_v}$,A_v=areaofvoids

Butq=A.V

$AV = A_V V_S$

$V_s = \frac{AV}{A_v} = \frac{V}{A_v/A} = \frac{V}{n}$

$\boxed{V_s = \frac{V}{n}}$,n =Porosityof soil

Sincen<1,$V_s$isalwaysgreaterthanV

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

COEFFICIENTOFPERMEABILITY

- 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

Soiltype	Coefficient of Permeability cm/sec	Drainage Characteristics
Gravel SandSi ltClay	>1 $1\text{-}10^{-3}$ 10^{-3}–10^{-6} $<10^{-6}$	Previous Previous Slightly Previous Impervious

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

$$\boxed{K_P = \frac{K}{n}}$$

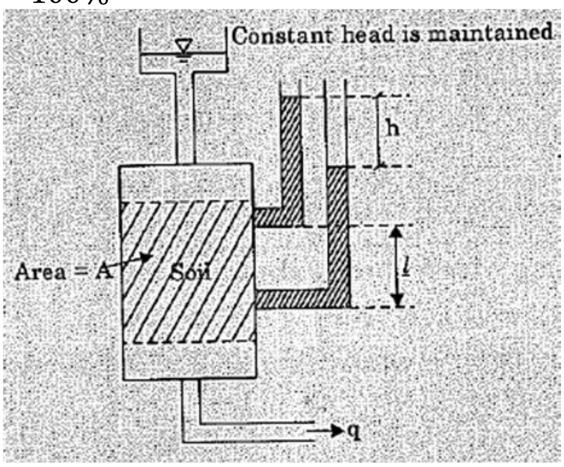
Where,
K_P=Coefficientofpercolation
K=Coefficientofpermeability n = Porosity

DETERMINATIONOFCOEFFICIENT OFPERMEABILITY

- 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:
  i) Constant–headpermeabilitytest.
  ii) Variable–headpermeabilitytest.
- 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 fields methods.
  i) Pumping-outtests.
  ii) Pumping-intests.
- 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 coefficientof permeability of the soil can alsobe determined indirectly from the soil parameters by
  i) Computation from the particle size and its specific surface.
  ii) 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 o the soil.
  d) Capillarity-Permeability test: The coefficient of permeability of an unsaturated soil can be determined by the capillarity-permeability test.

CONSTANTHEADPERMEABILITYTEST

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



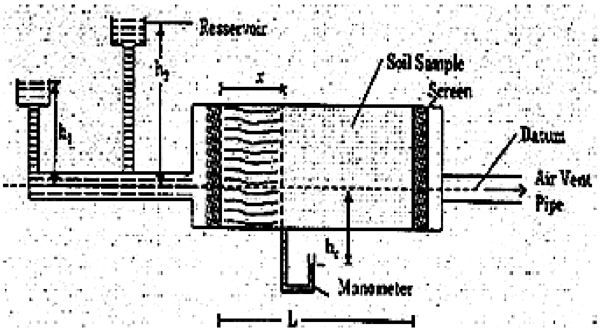
Weknowthat, q =kiA

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

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

CAPILLARITYPERMEABILITYTEST

- The coefficient of permeability for unsaturatedsoilcanbedeterminedby capillaritypermeabilitytest.
- Asampleofpartiallysaturatedsoilis placedina35cmlongand4cm diameterglasstubewithscreensfixed at both ends.



- Initiallyvalveconnectingtohigher reservoirisclosedandvalveofthe lower reservoir is opened.
- As the valve of lower reservoir is opened, capillary action in the soilstarts drawing water into it and wetted surface advancestowardsthe open end.
- Letthecapillaryheadbe$h_c$overthe 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.

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

- Thewettedsurfacemovesfurtherwith aseepagevelocityV_s weknowthat

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

- Butforpartiallysaturatedsoilabove equationismodifiedbytakingactual saturated porosity. As (S×n). Where S = degree of saturation.

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

$$V_s = \frac{dx}{dt}$$

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

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

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

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

- Actually capillary head is not known accurately, therefore we cannot calculate the valve of k. to overcomethis difficulty we close the valve connecting low reservoir and open the valve connecting high reservoir and perform the same procedure. Therefore weget asecondequationalso.Intimet_3 to t_4 let x_3&x_4 be the distance measuredfromleftend.

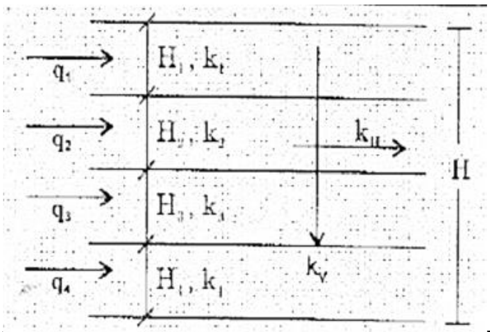
$$\boxed{\frac{x_4^2 - x_3^2}{t_4 - t_3} = \frac{2k}{Sn}(h_1 + h_c)}$$

- Thevaluesofkand$h_c$canbeobtained fromabovetwoequation.

PERMEABILITYOFSTRATIFIEDSOIL

- Actually in field soil is present in the form of different stratum 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)whentheflowis paralleltothestrataandtheaverage verticalcoefficient ofpermeability,(k_V) when the flow is normal to the strata.

- Consider a section of stratified soil as shown in figure below of varying thicknessofeachstratumeg: H_1, H_2, H_3&H_4withtheirrespective coefficientofpermeabilityk_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) Horizontalflow(Paralleltobedding plane)

- Flowistakingplacethroughallthe layersatsametimehencehydraulic gradient is same in each layer.

  $i_1 = i_2 = i_3 = i_4 = i$ (constant)

  Averagedischargevelocityoverthesoil mass can be written as.

  $$V = k_H i = \frac{1}{H}\left(V_1H_1 + V_2H_2 + V_3H_3 + V_4H_4\right)$$

  $$= \frac{1}{H}\left(k_1 i H_1 + k_2 i H_2 + k_3 i H_3 + k_4 i H_4\right)$$

  $$k_H i = \frac{(k_1H_1 + k_2H_2 + k_3H_3 + k_4H_4)}{H} i$$

  $$k_H = \frac{(k_1H_1 + k_2H_2 + k_3H_3 + k_4H_4)}{H_1 + H_2 + H_3 + H_4}$$

2) Verticalflow(Normaltobedding Plane)

- eti_1, i_2, i_3&$i_4$bethehydraulicgradient in different layers of thickness H_1, H_2, H_3&H_4respectively.
- Letthetotalheadlossbehoverthe total thickness of soil stratum H.
- Eachlayerhavingheadlossh_1, h_2, h_3, h_4. Thenconstantvelocityofflowisgiven by.

  $$V = k_V \frac{h}{H} = k_1 i_1 = k_2 i_2 = k_3 i_3 = k_4 i_4$$

  Headlossinanylayer isgivenby.

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

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

  $$ki = k_1 i_1 = k_2 i_2 = k_3 i_3 = k_4 i_4$$

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

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

  $$h\left(\frac{kH_1}{Hk_1} + \frac{kH_2}{Hk_2} + \frac{kH_3}{Hk_3} + \frac{kH_4}{Hk_4}\right) = h$$

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

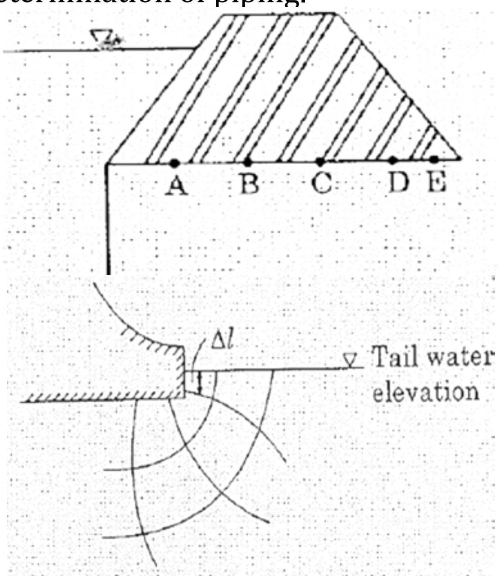
Note:k_H isalwaysgreaterthank_V.

SALIENTPOINTSABOUTFLWNET

i) Shape factor is only function of boundary condition.
ii) Flow NetwillnotchangeifPermeability (k)ofsoil changesi.e.soilischanged.
iii) Flownetwillnotchangeifheadloss during flow is changed.
iv) Inboth theabovecases(i.e.(ii)&(iii))thequantityofseepagewillbedifferent.
v) Flow net is unique for a given boundary conditionandiftheboundary,condition doesnot change $\frac{N_f}{N_d}$ willnot change.
vi) Note that even if no. of flow lines or no. ofequipotential linesarechanged and if boundary conditiondoesnotchange, $\frac{N_f}{N_d}$willnotchange.

USEOFFLOWNET

1. Seepagecalculation,$q=Kh\frac{N_f}{N_d}$
2. Uplift pressure calculation: Uplift pressure at any point is the pore water pressure acting vertically upward dueto residual pressure head at that point.
3. Upliftpressurealongthebaseofa masonary 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 lengthandmuchoftheheadwill,have beenlostuptoA,b,C,Detc.Hencepressure at A, b, C, D reduces.
4. Calculation of exit gradient and determination of piping.



- Hydraulicgradientofflowwillbemax 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 graduallywashes 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.

SEEPAGEINANISOTROPICSOIL

- Weknow that,for 2-dflow

$$K_x\frac{\partial^2 h}{dx^2}+K_z\frac{\partial^2 h}{dz^2}=0$$

- When $K_x=K_{horizontal}$ and $K_z=K_{vertical}$,as shown inthefigure

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

- 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 line are not necessarilytobeorthogonalasbecause $\frac{K_z}{K_x}\neq 1$,andwecannotgettheactual formofLaplaceequationwhichis

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

- Butif we replace

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

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

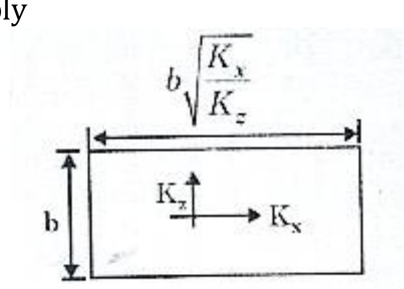
$$\boxed{\frac{\partial^2 h}{\partial x_t^2}+\frac{\partial^2 h}{\partial z^2}=0}$$

- Thus,newflowlineandequipotential lines are drawn on transformed section withx-distancechanged $x\sqrt{\frac{K_z}{K_x}}$ while keeping the vertical dimension constant.
- Valueofcoefficientofpermeabilityfor transformed section $\boxed{K'=\sqrt{K_x \times K_z}}$

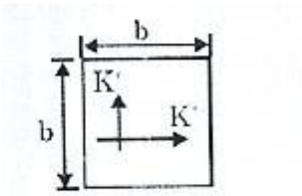
$$\boxed{q=\sqrt{K_x.K_z}H\frac{N_f}{N_d}}$$

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

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



Natural section



Transformed section

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

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

ANALYTICALMETHOD

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.

Fromthepropertyof theparabola,wehave PF = QH

$\therefore \sqrt{x^2+y^2} = DF+FH = x+xs$ (wheres= FH = focaldistance) ....(i)

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

Or $x=\dfrac{y^2-s^2}{2s}$ ....3.1(a)

And $y^2=2xs+s^2$....3.1(b)

This is the equation of the parabola. Taking variousofx,thevaluesofycanbe calculatedfromequation3.1(b).Andthe parabola can be plotted. The distance can eitherbedeterminegraphicallyorcanbe calculatedanalyticallybeconsideringthe co -of point C, as follows

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

ATC, $x=D$ and $y=H$

$s=(\sqrt{D^2+H^2}-D)$.....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,

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

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

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

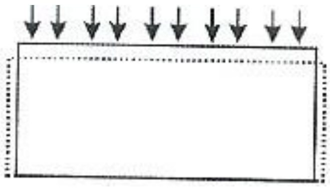
Substituting in(iii)weget

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

4 COMPRESSIBILITYANDCONSOLIDATION

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. CompressionandexpulsionofPore Air.
  2. Compression and Expulsion of Pore Water.
  3. Gradualreadjustmentofclayparticles into more stable configuration.



- Reductioninvolumeofsoilmassresults 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.
  $S_t = S_{immidiate} + S_{1^o\text{-consodilation}} + S_{2^o\text{-consodilation}}$

EFFECTIVE STRESS AND VOID RATIO RELATIONSHIP

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

1. HeightofSolidMethod

$$\boxed{H_s = \frac{W_s}{G_s \gamma_w . A}}$$

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

$$\left[e = \frac{A \times H - A \times H_s}{A \times H_s} = \frac{V - V_s}{V_s} \right]$$

Where, W_s=drywtofsoil

G_s=Specificgravityofsoil(mustbe known)

A=Crosssectionareaofsoil specimen

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

$H = H_1 + \Delta H$

H_1= Height of specimen at the beginning of stress increment

ΔH=Changeinthicknessunderthestress increment

H_s=Heightofsolid

2. Changeinvoidratio method

Soilattheendtestisassumedtbesaturated (i.e. s = 1)

Hencefrom $\boxed{S.e = w.G}$

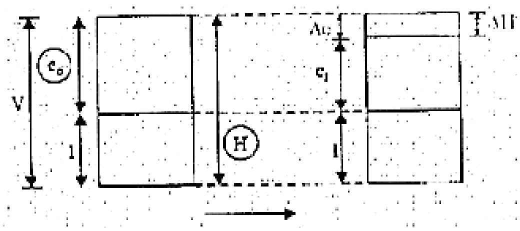
$$\boxed{e_{f0} = w_{f0} \times G_s}$$

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

w_{f0}=Watercontentattheendoftest

(Notedafterovendrying)

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



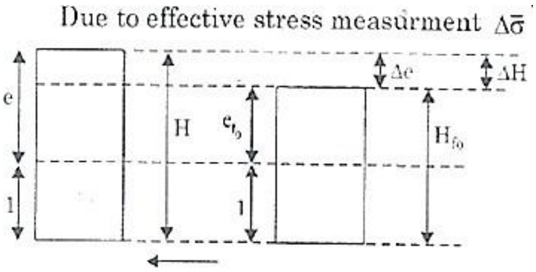
Foronedimensionalconsolidation

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

- Knowing H, e_0 and ΔH, can be calculated and from the above expression. Further the equilibriumvoid ratio at the end of every effective stressincrementcanbecalculatedas $\boxed{e=e_0-\Delta e}$

Anothermethods:

- By working backwards from the known valueofe_1&H_1attheendofeach effective stress increment we can calculate equilibrium void ratio.



$$\boxed{\frac{\Delta H}{H_{f0}}=\frac{\Delta e}{1+e_{f0}}}$$

- KnowingΔH,H_{f0}&$\boxed{e_{f0}=w_{f0}\times G_s}$, Δe canbecalculated.Thenworking backwards, $\boxed{e=e_{f0}+\Delta e}$ can be calculated.
- Thus void ratio at the end of various stress increments can be calculated.

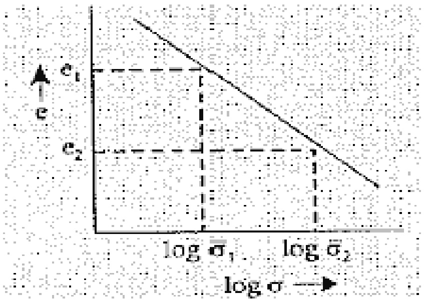
COMPRESSIBILITY OF CLAY CAN BE REPRESENTED BY ONE OF THE FOLLOWING COEFFICIENT

1. **CompressionIndex(C_c):**

- Itistheslopeoflinearportionofe-log $\bar{\sigma}$plotandisadimensionlessnumber.

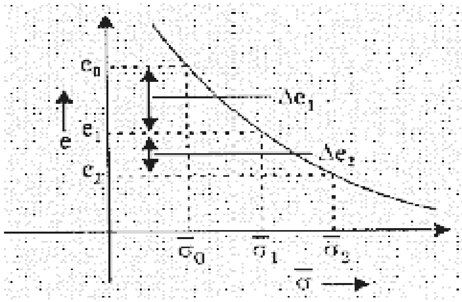
$$\boxed{C_c=\frac{e_1-e_2}{\log_{10}\bar{\sigma}_2-\log\bar{\sigma}_1}=\frac{\Delta e}{\log_{10}(\bar{\sigma}_2/\bar{\sigma}_1)}}$$

- Notethat $C_c$hassignificanceonlyfor **NormallyConsolidatedsoil.**



- C_c hasaconstantvalueforagiventype ofsoil.
- Itisnotafunctioneffectivestress.
- Higher the compression index, higher is the resulting vertical deformation of clay i.e.ΔH×C_c
- Theexpansionpartofe-log$\bar{\sigma}$plotcan be approximated to a straight line. The slope of which is called expansion index or recompression index.
- $C_c$isaplotbetweenArithmeticscale v/s log scale.

2.Coefficientofcompressibility(a_v)



- Coefficientofcompressibilitydecreases withincreaseineffective 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 voidratio decreases with increase in stress.
- Itisaplotbetweenarithmeticscalev/s arithmeticscale.

3. **Coefficientofvolumecompressibility (m_v)**

$$m_v=\frac{\text{Volumechangeperunitvolume}}{\text{increase in effective stress}}$$

$$m_v = \frac{\Delta V/V}{\Delta\bar{\sigma}} = \frac{\Delta H/H_0}{\Delta\bar{\sigma}} = \frac{\Delta e/1+e_0}{\Delta\bar{\sigma}}$$

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

The value of m_v for a particular soil is not constant but depends on stress range over which it is calculated. Odometer test can also be used to calculate coefficient of consolidation (C_v), C_v indicate the rate of consolidation.

Note: Whenever possible, pre consolidation pressure for an overconsolidated 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° and 2° consolidation in this chapter. Settlement immediate settlement will be discussed in shallow foundation.
- Below are few relationship to calculate the Primary settlement.

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

   Find $\Delta H = S_{0\,1consolidation}$

2. $\boxed{\Delta H = \frac{\Delta e}{1+e_0} \times H_0 = \frac{a_v \Delta\bar{\sigma}}{1+e_0} = m_v.\Delta\sigma.H_0 = S_{1^0 consolidation}}$

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

   $\boxed{S_{0\,1consolidation} = \sum m_{vi}\Delta\sigma_i H_i}$

4. In the case of normally consolidated soil

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

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

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

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

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

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

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

   Hence, $\Delta H = \frac{C_r H_0}{1+e} \log \frac{\bar{\sigma} + \Delta\bar{\sigma}}{\bar{\sigma}}$ over consolidated

   If $\bar{\sigma} + \Delta\bar{\sigma} > \bar{\sigma}_c$ Then,

$$\Delta H = \frac{C_r H_0}{1+e_0} \log \frac{\bar{\sigma}_c}{\bar{\sigma}_0} + \frac{C_c H'_0}{1+e_0} \log \frac{\bar{\sigma} + \Delta\bar{\sigma}}{\bar{\sigma}}$$

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,

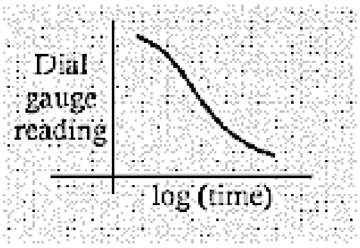
$$\Delta H = \frac{C_r H_0}{1+e_0} \log \frac{\bar{\sigma}_c}{\bar{\sigma}_0} + \frac{C_c H_0}{1+e_0} \log \frac{\bar{\sigma} + \Delta\bar{\sigma}}{\bar{\sigma}}$$

DETERMINATIONOFCOEFFICIENT OF CONSOLIDATION (C_v)

- Thecurveofdeformationversustime obtainedfromodometertestissimilar toUversusT_v curve.Thispropertyis usedtodetermine$C_v$fromoedometer test by curve fitting technique.
- Twomethodareused.
  (a) Casagrande's logarithm of time fitting method.
  (b) Taylor'ssquarerootoftimefitting method.

(a)Casagrande's logarithm of timefitting method.

- Timefor50%conditionisnoted.

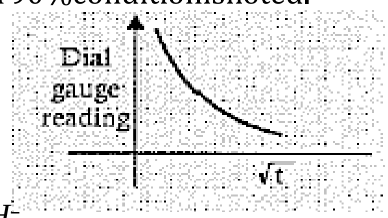


$$C_v=\frac{T_{50}H^2}{t_{50}}$$

$T_{50}=0.196$

(b) InTaylorsmethod/Squarerootof time fitting method.

- Timefor90%conditionisnoted.

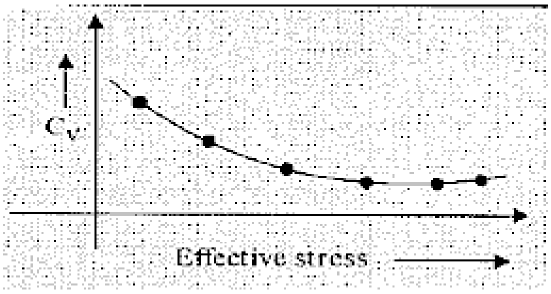


$$C_v=\frac{T_{90}H^2}{t_{90}}$$

$T_{90}=0.848$

Note: Taylorcurveissuitablebecausecurve 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 of 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 usedtocalculate coefficient ofpermeability for fine grained soil.

- ValueofC$_v$decreasesasliquidlimitof soilincreases.i.e.,$C_v \propto \frac{1}{w_L}$
- ValueofC$_v$decreasesasplasticityindex ofsoilincreases.i.e.,$C_v \propto \frac{1}{P.I}$

$$C_v=\frac{K(1+e_0)}{a_v\gamma_w}=\frac{K}{m_v\gamma_w}$$

$$K=C_v.m_v\gamma_w$$

COMPRESSIONRATIO

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

(a) Initialcompression ratio

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

(b) 1°-consolidationratio(r_p)

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

(c) SecondaryCompressionratio(r_s)

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

Where,

R_i=initialdialgaugereading
R_0=dialgaugereadingcorresponding
tobeginningof1°consolidation.
R_{100}=dialgaugereadingcorresponding
tocompletionof1°consolidation.R_f=
finaldialgaugereadingR_F–R_i=Canbe
knownfromexperiment.
R_0–R_i=InitialCompression
R_0,R_{90},R_{100} arecalculatedfromcurve
fittingtechniquesofTaylors method.

INITIALCOMPRESIONISDUETO

(a) Compressionofsmallqualityofairin soil
(b) Imperfectsaturation
(c) Verticalelasticcompression
(d) Lateralexpansionofthespecimenwhen imperfectly mounted.

5 COMPACTION

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 strengthandhelpsimprovethestability 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 dueto reduction in the Volume ofair voids at a given water content.	3. Volume reduction isdue to expulsion ofpore water from voids.
4. Specificcompaction Techniques are used.	4. Consolidationoccurson account of Static load placed on the soil.

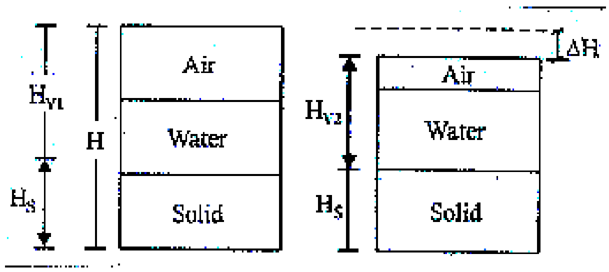
WHYDOWENEEDTOCOMPACT SOIL

1. Max shear strength occurs at min. void ratio.
2. Large air voids if left, may tocompaction 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. Settlementcanbereducedor 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



$H_{V1}=e_oH_S$; e_0=initialvoidratio

$H_{V2}=e_fH_S$; e_f=final voidratio

$$\Delta H=H_{V1}-H_{V2}$$

$$\frac{\Delta H}{H}=\frac{(e_0-e_f)H_S}{H_S+e_0H_S}=\frac{(e_0-e_f)}{1+e_0}$$

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

COMPACTION OF COHESIONLESS SOIL

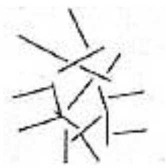
- ACohesionlesssoilcanbevarious states like
  1. Looseangularsoil
  2. Denseangularsoil
  3. Honey-combedstate

- 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 becauseofshearstrengthdevelopedbyca pillary forces.

COMPACTIONOFCOHESIVE SOIL

- Clays cannot be compacted byvibration.
- 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 amoreparallelpositionwithreduced

voids.



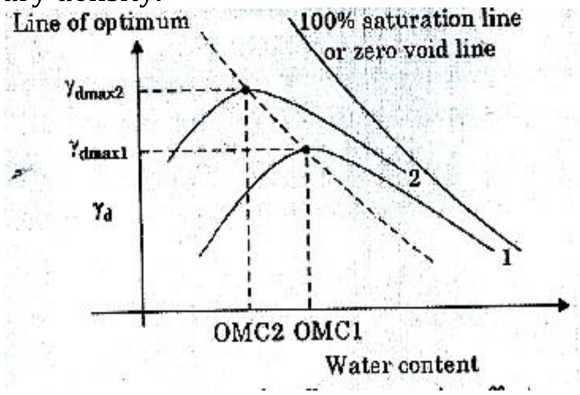
Loose Structure of clay before composition (edge to face or edge to edge configuration of clay particles)

Dense Structure after compaction (Face to face confuiguration)

- The application of static pressurecauses compaction in clayey soil.

PROCTOR TEST

- As per Proctor, a definite relationship exists between the soil moisturecontent the soil moisture content and the degree of drydensityto 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 whichaparticularsoilattainsmaximum dry density.



Curve1→Smallercompactiveeffort 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.
- Typicalvaluesof $\gamma_{d\max}$ =16–20 kN/m^3
- TypicalvaluesofOMC= 10–20%

STANDARDPROCTORTEST

- Weightofhammer =2.495 kg(5.5lb)
- Heightoffall=12″=304.8mm
- Volumeofmould=944 cc(1/30cft)
- Compacted in 3 layers with 25 blows in each layer.
- Test is conducted at various moisture content and the curve as shown aboveis obtained.
- Compactive energy (E) applied per unit volume

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

Where,
N = no. of blows par layer ;
n = no. of layers
w=weightofhammer; h
= height of fall
V=volumeof mould.

- Standard Proctor Compactive effort per unit volume

$$=\frac{25\times3\times2.5\times0.305\times9.81\,J}{944\times10^{-6}\,m^3}$$
$=594..29\ kJ/m^3$

MODIFIEDPROCTORTEST

- Weightofhammer=4.54kg(10lb)
- Heightoffall=18”=457.2mm
- Volumeofmould=944 cc
- Compacted in 5 layers with 25 blows in each layer.
- Modified Proctor Compactive effort per unit volume
$$=\frac{25\times3\times4.54\times0.4572\times9.81\,J}{944\times10^{-6}\,m^3}$$
$=2696.31\ kJ/m^3$

IMPORTANCEOFPROCTOR 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. Providesthemoisturerangethatallows 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).

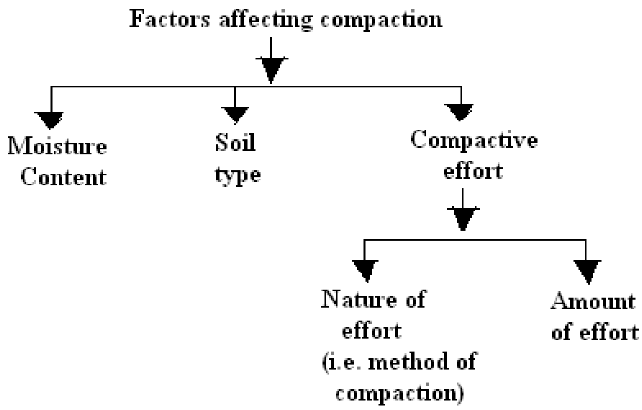
LIGHT COMPACTION TEST (IS: 2720, PART VII-1974)

- Weightofhammer =2.6kg
- Heightoffall=310mm
- Volumeofmould=1000cc
- Compacted in 3 layers with 25 blows in each layer.
- Light Compaction Test Compactive effort per unit volume
$$=\frac{25\times3\times2.6\times0.31\times9.81J}{1000\times10^{-6}\,m^3}$$
$=593.014\ kJ/m^3$

HEAVYCOMPACTIONTEST(IS:2720, PART VIII- 1983)

- Weightofhammer =4.9kg
- Heightoffall=450mm
- Volumeofmould=1000cc
- Compacted in 5 layers with25 blows in each layer.
- Heavy Compaction Test Compactive effort per unit volume
$$=\frac{25\times3\times4.9\times0.45\times9.81J}{1000\times10^{-6}\,m^3}$$
$=2703.88\ kJ/m^3$

FACTORSAFFECTING COMPACTIO



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.
- Thisphenomenonoccursupto OMC.
- Beyond OMC water starts taking the space which would otherwise havebeen taken by soil,
- As γ_w is smaller than γ_S , the dry unit reduces.
-

LAMBEEXPLANATION

- Incaseofcohesivesoil,thereisan attractiveforcenamely,Vanderwaal forces between two soil particles and a repulsive force due to double layer of absorbedwater.Whiletheattractive forceessentiallyremainssame,the repulsive force is more if absorbedlayer thickness is more.
- If Net force is attractive flocculated structureresults.Howeverifnetforceis repulsive, dispersed structure results.
- At low water content, the double layeris not fully developed, i.e. repulsiveforce is small and net force is attractive. This makes it difficult for the particlesto 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 increase. The particles can easily slide over one another and get packed more easily resulting into higher dry density.
- The double layer expansion is complete a OMC, the addition of water does not cause any further expansion of double layer but water starts to occupy the space which otherwise would havebeen occupied by soil grains. Hence a decrease in dry unit wt. occurs beyond OMC
- This theory also explains why compaction curve is not invented 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 become 1. Hence at various moisture content zero airvoiddensitycanbeachievedbyputting S = 1 **in**

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

- When this is plotted we get zero void line.

ZEROAIRVOID DENSITY

- The maximum dry unit weight that can be achieved for a given water contentby 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 lineatvariousmoisturecontentare same

γ_d can also be calculated as :

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

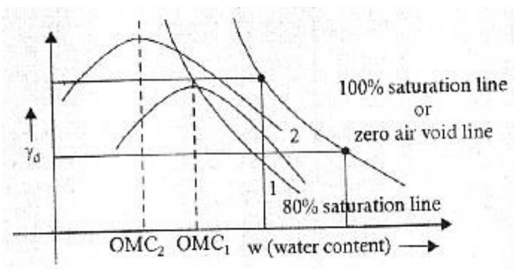
For n_a= 0% equation becomes identical to (i) equation

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

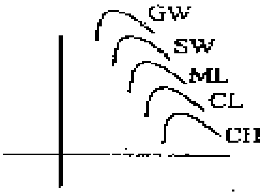
$$\gamma_d = \frac{G\gamma_W}{1+\frac{wG}{0.9}}$$

- 10%airvoid line

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



TYPICAL COMPACTION CURVES FOR DIFFERENT SOILS



POINTSTOBENOTED

1. Coarse grained soil is well graded and compactedtohigh$\gamma_d$speciallyifthey containsomefines.Howeverifthe quantity of fines is excessive $\gamma_d$decreases.
2. Poorlygradedoruniformsandleadto lowestdryunitweightvalues.
3. Inclaysoilmaxdryunitwttendtodecrease as plastically increases.
4. Heavyclaywithhighplasticallyhas very high OMC.

EFFECT OF COMPACTION ON PROPERTIES OF SOILS

1. Soilstructure

- 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 contentless than the optimum water content generally have a flocculated structure, regardlessof themethodof 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 thantherepulsiveforces.Thisresultsin 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

- Thepermeabilityofasoildependsupon 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 permeabilitymayshowaslightincrease but it always remains much smallerthan 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 watercontenthashigh waterdeficiency and more random orientation of particles. Therefore, it imbibes more water than the sample compacted wetof the optimum, and has, therefore, more swelling.

4. Porewater pressure

- A sample compacted dry of theoptimum 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 theoptimum shrink more because the soil particles in the dispersed structurehave 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 dryside are less compressible.
- It increase with an increase in the degree of saturation.
- The compressibility of a soil compacted on the west 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-Strainrelationship**

- The soils compacted dry of theoptimum have a steeper stress-strain curve than those on the west side.
- The modulus of elastically 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 theoptimum have relatively flatter stress- strain curve and a corresponding lower value of the modulus of elastically. At higher stress level when both soil wall have same strength because bonds are broken.
- At a given water content, the shear strength of the soil increases with an increase in the comp active 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.

(a)istrueforNupto 4,
(b) istrue forN from30to 50
(c) istrueforNfrom 10to30and
(d) istrueforNfrom4 to10

For obtaining good quality undisturbed samples,theareratio should be 10 percent or less.

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