LECTURE NOTES ON

GEOTECHNICALENGINEERING-I

DEPARTMENT OF CIVIL ENGG

COURSE CONTENT

1. INTRODUCTION
2. PRELIMINARY DEFINITIONS AND RELATIONSHIP
3. INDEX PROPERTIES OF SOIL
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INTRODUCTION

SOIL & SOIL ENGINEERING

Theterm"soil"canhavedifferentmeanings,dependinguponthefieldinwhichitisconsidered.

Toageologist,itisthematerialintherelativethinzoneoftheEarth'ssurfacewithinwhichrootsoccur,andwhich are formed as the products of past surface processes. The rest of the crust is grouped under the term "rock".

Toapedologist,itisthesubstanceexistingonthesurface,whichsupportsplantl

ife. To an engineer, it is a material that can be:

- builton:foundationsofbuildings,bridges
- builtin:basements,culverts,tunnels
- builtwith:embankments,roads,dams
- supported:retainingwalls

SoilMechanicsisadisciplineofCivilEngineeringinvolvingthestudyofsoil,itsbehaviorandapplicationasan engineering material.

Soil Mechanics is the application of laws of mechanics and hydraulics to engineering problems dealing with sedimentsandotherunconsolidatedaccumulationsofsolidparticles,whichareproducedbythemechanicaland chemicaldisintegrationofrocks,regardlessofwhetherornottheycontainanadmixtureoforganicconstituents.

Soilconsistsofamultiphaseaggregationofsolidparticles,water,andair.Thisfundamentalcompositiongivesrise to unique engineering properties, and the description of its mechanical behavior requires some of the most classic principles of engineering mechanics.

Engineersareconcernedwithsoil'smechanicalproperties:permeability,stiffness,andstrength.Thesedepend primarily on the nature of the soil grains, the current stress, the water content and unit weight.

FormationofSoils:

Soil is formed from rock due to erosion and weathering action. Igneous rock is the basic rock formed from the crystallization of molten magma.This rockis formed eitherinside theearthoron thesurface.These rocks undergo metamorphism under high temperature and pressure to form Metamorphic rocks. BothIgneous and metamorphicrocksareconvertedintosedimentaryrocksdueto transportationtodifferentlocations bytheagencies suchaswind,wateretc. Finally, near the surface millions ofyearsoferosion andweathering converts rocks in to soil.

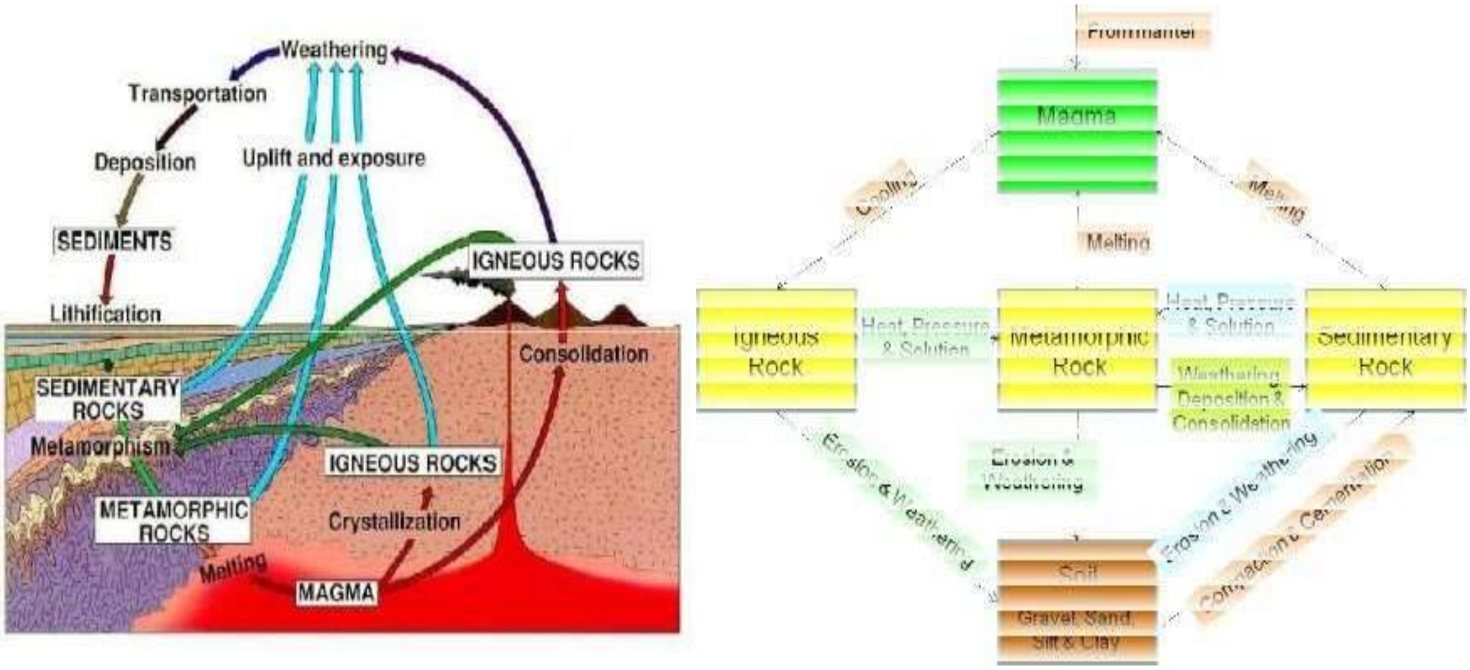


Fig.GeologicCycleofSoil

Soilsareformedfrommaterialsthathaveresultedfromthedisintegrationofrocksbyvariousprocessesofphysical and chemical weathering. The nature and structure of a given soil depends on the processes and conditions that formed it:

- Breakdownofparentrock:weathering,decomposition,erosion.
- Transportationtositeoffinaldeposition:gravity,flowingwater,ice,wind.
- Environmentoffinaldeposition:floodplain,riverterrace,glacialmoraine,lacustrineormarine.
- Subsequentconditionsofloadinganddrainage:littleornosurcharge,heavysurchargeduetoiceor overlying deposits, change from saline to freshwater, leaching, contamination.

Allsoilsoriginate,directlyorindirectly,fromdifferentrocktypes.

Weathering:

Physical weathering reduces the size of the parent rock material, without any change in the original composition of the parent rock. Physical or mechanical processes taking place on the earth's surface include the actions of water, frost, temperature changes, wind and ice. They cause disintegration and the products are mainly coarse soils.

The main processes involved are exfoliation, unloading, erosion, freezing, and thawing. The principal cause is climatic change. In exfoliation, the outer shell separates from the main rock. Heavy rain and wind cause erosion of the rock surface. Adverse temperature changes produce fragments due to different thermal coefficients of rock minerals. The effect is more for freeze-thaw cycles.

*Chemicalweathering*notonlybreaksupthematerialintosmallerparticlesbutaltersthenatureoftheoriginalparent rock itself. The main processes responsible are hydration, oxidation, and carbonation. New compounds are formed due to the chemical alterations.

Rain water that comes in contact with the rock surface reacts to form hydrated oxides, carbonates and sulphates. If there is a volume increase, the disintegration continues. Due to leaching, water-soluble materials are washed away and rocks lose their cementing properties.

Chemical weathering occurs in wet and warm conditions and consists of degradation by decomposition and/or alteration. The results of chemical weathering are generally fine soils with altered mineral grains.

Theeffectsof weatheringand transportationmainlydetermine the basic*nature*ofthe soil(size, shape,composition
anddistributionoftheparticles).

Theenvironmentintowhichdepositiontakesplace,andthesubsequentgeologicaleventsthattakeplacethere, determinethe *state* ofthesoil(density,moisturecontent)andthe *structure* orfabricofthesoil(bedding, stratification, occurrence of joints or fissures)

Transportation agencies can be combinations of gravity, flowing water or air, and moving ice. In water or air, the grains become sub-rounded or rounded, and the grain sizes get sorted so as to form poorly-graded deposits. In moving ice, grinding and crushing occur, size distribution becomes wider forming well-graded deposits.

In running water, soil can be transported in the form of suspended particles, or by rolling and sliding along the bottom. Coarser particles settle when a decrease in velocity occurs, whereas finer particles are deposited further downstream. In still water, horizontal layers of successive sediments are formed, which may change with time, even seasonally or daily.

Windcanerode,transportanddepositfine-grainedsoils.Wind-blownsoilisgenerallyuniformly-graded.

Aglaciermovesslowlybutscoursthebedrocksurfaceoverwhichitpasses.

Gravitytransportsmaterialsalongslopeswithoutcausingmuchalteration.

SoilTypes:

Soilsastheyarefoundindifferentregionscanbeclassifiedintotwobroadcategories:

(1) Residualsoils
(2) Transportedsoils

ResidualSoils:

Residualsoilsarefoundatthesamelocationwheretheyhavebeenformed.Generally,thedepthofresidualsoil s varies from 5 to 20 m.

Chemical weathering rate is greater in warm, humid regions than in cold, dry regions causing a faster breakdown of rocks. Accumulation of residual soils takes place as the rate of rock decomposition exceeds the rate of erosion or transportation oftheweathered material. In humid regions, thepresenceof surfacevegetation reduces thepossibility of soil transportation.

As leaching action due to percolating surface water decreases with depth, there is a corresponding decrease in the degree of chemical weathering from the ground surface downwards. This results in a gradual reduction of residual soil formation with depth, until unaltered rock is found.

Residualsoilscompriseofawiderangeofparticlesizes,shapesandcomposition.

TransportedSoils:

Weatheredrockmaterialscanbemovedfromtheiroriginalsitetonewlocationsbyoneormoreofthetransportation agencies to form transported soils. Transported soils are classified based on the mode of transportation and the final deposition environment.

(a) Soilsthatarecarriedanddepositedbyriversarecalled*alluvialdeposits.*

(b) Soilsthataredepositedbyflowingwaterorsurfacerunoffwhileenteringalakearecalled*lacustrinedeposits.* alternatelayersareformedindifferentseasonsdependingonflowrate.

(c) Ifthedepositsaremadebyriversinseawater,theyarecalled*marinedeposits.*Marinedepositscontainboth particulate material brought from the shore as well as organic remnants of marine life forms.

(d) Meltingofaglaciercausesthedepositionofallthematerialsscouredbyitleadingtoformationof*glacial deposits.*

(e) SoilparticlescarriedbywindandsubsequentlydepositedareknownasAeolian*deposits.*

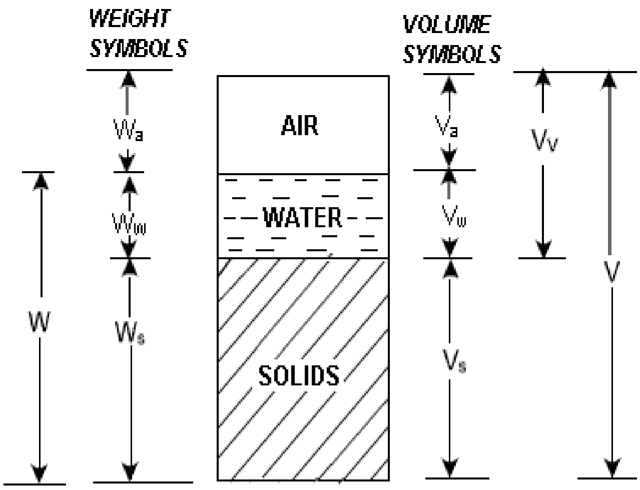
2. PRELIMINARY DEFINATION AND RELATIONSHIP

PhaseRelationsofSoils:

Soil is notacoherent solid material likesteel and concrete, but is aparticulatematerial. Soils, as theyexist in nature, consist of solid particles (mineral grains, rock fragments) with water and air in the voids between the particles.The

terandaircontentsarereadilychangedbychangesinambientconditionsandlocation.

As the relative proportions of the three phases vary in any soil deposit, it is useful to consider a soil model whichwill represent these phases distinctly and properly quantify the amount of each phase. A schematic diagram of the three-phase system is shown in terms of weight and volume symbols respectively for soil solids, water, and air. The weight of air can be neglected.



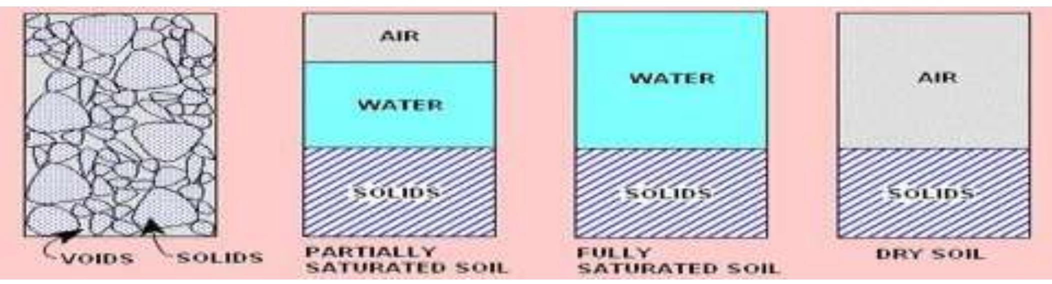
Thesoilmodelisgivendimensionalvaluesforthesolid,waterandaircompo

nents. Total volume, $V = V_s + V_w + V_v$

Three-phaseSystem:

Soilscanbepartiallysaturated(withbothairand waterpresent),orbefullysaturated(no aircontent)orbeperfectly dry (no water content).

Inasaturatedsoiloradrysoil,thethree-phasesystemthusreducestotwophasesonly,asshown.



Thevariousrelationscanbegroupedinto:

- Volumerelations
- Weightrelations
- Inter-relations

VolumeRelations:

Astheamountsofbothwaterandairarevariable,thevolumeofsolidsistakenasthereferencequantity.Thus, several relational volumetric quantities may be defined. The following are the basic volume relations:

1.Voidratio(e)istheratioofthevolumeofvoids(V_v)tothevolumeofsoilsolids(V_s),andisexpressedasa decimal.

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

2.Porosity(n)istheratioofthevolumeofvoidstothetotalvolumeofsoil(V),andisexpressedasapercentage.

$$n = \frac{V_v}{V} x\, 100$$

Voidratioandporosityareinter-

$$e = \frac{n}{1-n}$$ and $$n = \frac{e}{(1+e)}$$

3.Thevolumeofwater(V_w)inasoilcanvarybetweenzero(i.e.adrysoil) andthevolumeofvoids. Thiscanbe expressed as the degree of saturation (S) in percentage.

$$S = \frac{V_w}{V_v} x\, 100$$

Foradrysoil,S=0%,andforafullysaturatedsoil,S=100%.

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

4. Aircontent(a_c)istheratioofthevolumeofair(V_a)tothevolumeofvoids.

5. Percentageairvoids(n_a)istheratioofthevolumeofairtothetotalvolume.

$$n_a = \frac{V_a}{V} x\, 100 = n \; x \; a_c$$

WeightRelations:

Densityisameasureofthequantityofmassinaunitvolumeofmaterial.Unitweightisameasureoftheweighto fa unit volume of material. Both can be used interchangeably. The units of densityare ton/m³, kg/m³ or g/cm³. The following are the basic weight relations:

1.Theratioofthemassofwaterpresenttothemassofsolidparticlesiscalledthe watercontent(w**)**,orsometimes the moisture content**.**

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

Itsvalueis0%fordrysoilanditsmagnitudecanexceed100%.

2.Themassofsolidparticlesisusuallyexpressedintermsoftheirparticleunitweigh t orspecificgravity(G_s

$$\gamma_s = \frac{W_s}{V_s} = G_s \cdot \gamma_W$$

(γ_s)

wher γ_W =Unitweightof

Formostinorganicsoils,thevalueof$G_s$liesbetween2.60and2.80.Thepresenceoforganicmaterialreducesth e value of G_s.

3.Dryunitweight (γ_d) easureoftheamountofsolidparticlesperunitvolume

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

4.**Bulkunitweigh**n $(\gamma_t \text{ or } \gamma)$ ureoftheamountofsolidparticlespluswaterperunitvolume.

$$\gamma_t = \gamma = \frac{(W_s + W_W)}{(V_s + V_V)}$$

5. Saturatedunitweight (γ_{sat}) ualtothebulkdensitywhenthetotalvoidsisfilledupwithwater.

6. Buoyant unit weight (γ')

bmergedunitweightistheeffectivemassperunitvolumewhenthesoilis submerged below standing water or below the ground water table.

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

Inter-Relations:

It is important to quantify the state of a soil immediately after receiving in the laboratory and prior to commencing other tests. The water content and unit weight are particularly important, since they may change during transportation and storage.

Some physical state properties are calculated following the practical measurement of others. For example, dry unit weightcanbedeterminedfrombulkunitweightandwatercontent.Thefollowingaresomeinter-relations**:**

$$w = \frac{W_W}{W_s} = \frac{\gamma_W . V_W}{G_s . \gamma_W . V_s} = \frac{V_W}{G_s . V_s} = \frac{S . V_v}{G_s . V_s} = \frac{S . e}{G_s}$$

$$\gamma = \frac{(G_s + S.e).\gamma_w}{1 + e}$$

$$\gamma = \frac{(1 + w).G_s.\gamma_w}{1 + e}$$

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

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

$$\gamma' = \frac{[(G_s - 1) + (S - 1)e] \, x \, \gamma_w}{1 + e}$$

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

Example 1: Asoilhasvoidratio=0.72,moisturecontent=12%andG_s=2.72.Determine its

(a) Dryunit weight

(b) Moistunitweight,andthe

(c) Amountofwatertobeaddedperm^{3}tomakeitsatura

ted. Use $\gamma_w = 9.81 \, kN/m^3$

Solution:

(a) $\gamma_d = \frac{G_s.\gamma_w}{1 + e} = \frac{2.72 \, x \, 9.81}{1 + 0.72}$ =15.51kN/m^3

(b) $\gamma = \gamma_d(1+w)$

$$= \frac{1+0.12}{1+0.72} \, x \, 2.12 \, x \, 9.81 = 17.38 kN/m^3$$

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

$$\frac{2.72+0.72}{1+0.72} \, x \, 9.81 = 19.62 kN/m^3$$

Watertobeaddedperm^{3}tomakethesoilsaturated

$\gamma_{sat} - \gamma$ =19.62–17.38=2.24kN

Example2: Thedrydensityofasandwithporosityof0.387is1600kg/m^3.Findthevoid ratio of the soil and the specific gravity of the soil solids. [Take $\gamma_w = 1000 \, kg/m^3$

n=0.387

γ_d =1600kg/m^3

Solution:

(a)e= $\frac{n}{1-n} = \frac{0.387}{1-0.387} = 0.631$

(b) $\gamma_d = \frac{G_s . \gamma_w}{1+e}$

$\therefore$ G_s= $\frac{(1+e)}{\gamma_w} . \gamma_d = \frac{1+0.631}{1000} \, x \, 1600 = 2.61$

3. INDEX PROPERTIES OF SOIL

Soil index properties are properties which facilitate identification and classification of soils for engineering purposes. Plastic soils (clays) are normally described as cohesive as a distinction from non-plastic soils (sands and gravels) which are often called granular or non-cohesive. Plastic and cohesive are used as synonyms bearing in mind that all plastic soils are cohesive and cohesive soils are plastic. Fundamentally, electrochemical cohesion and geotechnical cohesion measured by a triaxial are very different. Cohesion in clays does not always translate to measurable cohesion that confers shear strength. The 3D network of attraction between negative particles and positive cations leads to plasticity. The nature of some properties differs for coarse- and fine-grained soils.

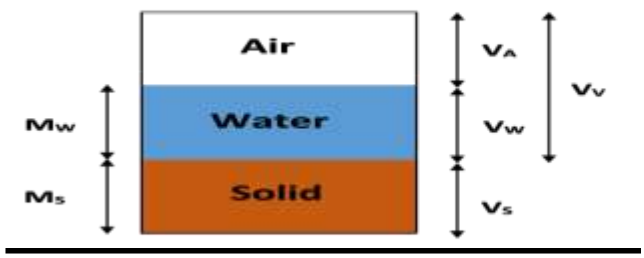
Coarse-grained (non-cohesive) soil index properties are:
- particle-size distribution
- particle shape
- relative density
- consistency
- clay and clay minerals content

Fine-grained (cohesive) soil index properties are:
- consistency
- particle shape and orientation
- clay and clay minerals content
- water content

One of the soil index properties which describe non-cohesive soils is grading otherwise known as **particle size distribution**. It gives a measure of the sizes and distribution of sizes of the particles that make up a soil and stands out to be the most fundamental of all properties especially in coarse-grained soils with little or no clay particles. Soil that contains a wide range of particle sizes is named well-graded. The opposite type of soil, which contains narrow range of particle sizes, is categorized as poorly graded. Well-graded soils can be more densely packed. **Particle shape** also influences how closely particles can be packed together. The density of soil (especially of coarse-grained) is the indication of strength and stiffness. The **relative density** is the ratio of the actual bulk density and the maximum possible density of the soil. Relative density is a good indicator of potential increases in density, and thus deformations that may occur under the different loads. The two main methods of grading soil are sieve and sedimentation analysis.

Consistency is the resistance of soils to deformation and rupture. The term consistency limits is derived from the notion that soil can exist in any four states, depending on its moisture state. Initially, the soil is in the form of a viscous liquid with no shear strength. As its moisture content is reduced, it begins to attain some shear strength but is still easily moulded, this is the plastic solid phase. Drying regimes reduces its ability to be moulded leading to cracks during moulding in the semi-solid phase. Over time the soil becomes so dry that it is a brittle solid.

Water Content (w)



The water content (w), also known as **natural water content** or **natural moisture content**, is the ratio of the weight of water to the weight of the solids in a given mass of soil. This ratio is usually expressed as percentage. When voids are completely filled with air, water content is equal to zero (dry soil).

$w(\%) = (M_w / M_s) * 100$

specific gravity

specific gravity, also called **relative density**, ratio of the density of a substance to that of a standard substance.

Particle Size Distribution

Particle Size Distribution or the percentage of grains of different sizes in a given soil is an important property of soil. Particle size analysis of coarse soils is carried out by sieve analysis or mechanical analysis whereas fine-grained soils are analysed by hydrometer analysis.

In general, a combined analysis is carried out as most soils contain both coarse and fine particles. In the combined analysis, dry soil is first analysed by sieving and then the very fine soil is analysed by hydrometer or pipette method by mixing it with water.

Importance of Particle Size Distribution.

- Particle size distribution is important for classification of soil.
- It is also used for the design of drainage filters.
- It is used for selecting filling materials for embankment, earthen dams, road sub-base etc.
- Particle size distribution is also used to estimate performance of grouting chemical injection.

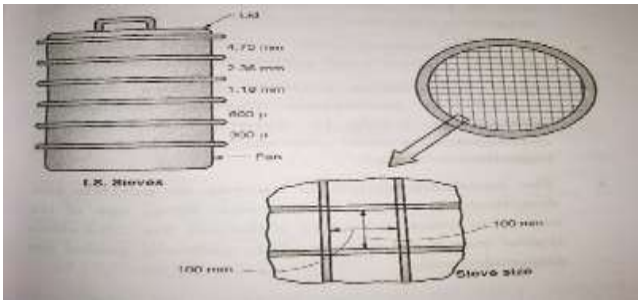
Mechanical Sieve Analysis as per IS Code : IS 2720, Part 4 – 1985 :

Sieves are wire screens having square opening. According to IS 460-1962 the sieve number is the mesh width expressed in mm for large sizes and in microns for small sizes. The 4.75 mm sieve separates the soil into 2 parts.

The fraction larger than 4.75 mm is called as coarse fraction. This fraction is analysed by the following series of sieves : 100 mm, 63 mm, 20 mm, 10 mm and 4.75 mm sieves.

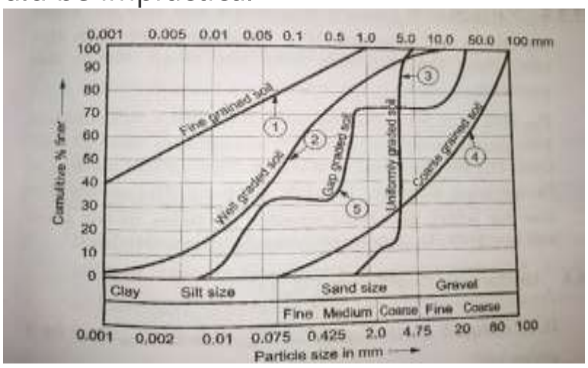
The sieves are arranged in descending sizes from top to bottom. A weighed, dry soil sample is put onto the top sieve. Generally 1000 gm sample is analysed. The top sieve is covered with lid and the bottom sieve has a pan below
it.The entire assembly is then shaken either manually with careful up-down and circular motion or in a machine known as sieve shaker. Usually 10 minutes of shaking is sufficient.The soil retained on each sieve is then weighed and the weight is recorded. The soil collected in the pan is subjected to further analysis. For this a series of 4.75 mm, 2.4 mm, 1.2 mm, 600μ (microns), 425μ, 300μ, 150μ, and 75μ sieves is used. These sieves are arranged and the sample from the pan is shaken for another 10 minutes. The weight of soil retained on each sieve is then recorded.

The soil particles passing through the 75μ sieve are collected in the pan. If the amount is significant, it is mixed with water for hydrometer or pipette analysis. A table is prepared, in which 'percentage finer' and 'cumulative percentage finer' particles corresponding to each sieve size are worked out and plotted on a semi-log graph paper.



Particle Size Distribution Curve.

As said earlier, the cumulative percentage of a particular size of particles passing through that size of sieve opening is worked out. It is plotted against the size in log scale. It is drawn semi-log graph paper, because the particle size may range from a few microns to a few hundred mm. Hence, on ordinary scale, a very long graph paper
would be required which would be impractical



Thus 'cumulative percentage finer than' or 'cumulative percentage passing through' is plotted on Y-axis in natural scale and the corresponding sieve sizes are plotted on X-axis in logarithmic scale. The resulting graph is known as **particle size distribution curve.** This curve forms one of the important index properties of the soil. Abovediagram shows typical particle size distribution curve for well graded, uniformly graded and gap graded soils

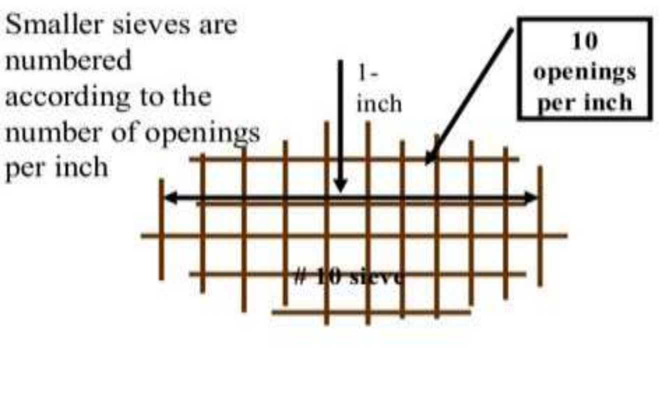
METHODS OF SIEVE SIZE ANALYSIS

Determination of article size is more important in Civil Engineering, as the particle size determines the effectiveness of final product. The characters of particle such as bulk density, physical stability, permeability and many more are decided by its size. To determine the size distribution of particles, the sieve analysis test procedure is an effective method that prevailed from the past. In sieve analysis, the particle size distribution is defined using the mass or volume. Sieve analysis is laboratory test procedure in which particles will move vertically or horizontally through sieve mesh. Depending on the needs and particle material different sieving methods are available for the application. They are manual sieving method, mechanical sieving method, dry sieving method and wet sieving method.

Manual sieving method is carried out in places where there is no electricity and mainly used in, onsite differentiation among large and small particles. Mechanical sieving method is used in laboratories to assure the quality and this is the widely using method in present days. In mechanical sieving the method can be classified into two further groups depending on their sieving movement as horizontal movement sieving method and vertical movement sieving method. The vertical movement sieving method is also known as throw-action sieving and vibratory sieving methods. Dry sieving method is considered mostly and here the testing particles (specimen) are in dry state. Wet sieving method is considered when the particle that is going to be used is already exists as wet or suspension. Here, in the sieve shaker machine a nozzle will be provided to water the upper most sample material. But need some extra about the water concentration during this wet sieving experiment.Procedure:

- Clean the sieves of sieve shaker using cleaning brush if any particles are struck in the openings.
- Record the weight of each sieve and receiving pan.
- Dry the specimen in oven for 3-4 minutes to get the dried specimen (ignore, if the specimen is already dried).
- Weigh the specimen and record its weight.
- Arrange the sieves in order as the smaller openings sieve to the last and larger openings sieve to the top. (Simply, arrange them to the ascending order of sieve numbers – No.4 sieve on top and no.200 sieve at bottom)- Sieve numbers and the particle sizes are provided below in a chart for further understanding.

Sieve Number and opening size



Sieve Number	Opening Size (mm)
4	4.750
6	3.350
8	2.360
12	1.680
16	1.180
20	0.850
30	0.600
40	0.425
50	0.300
60	0.250
80	0.180
100	0.150
140	0.106
200	0.075
270	0.053

- Keep the weight recorded specimen on the top sieve and then keep the complete sieve stack on the sieve shaker (Don’t forget to keep the lid and receiving pan).

- Allow the shaker to work 10-5 minutes – use the clock here..!Remove the sieve stack from the shaker and record the weight of each sieve and receiving pan separately
- .

Calculation:

- Mass of sieve (M1)
- Mass of sieve + retained soil (M2)
- Mass of soil (M2-M1)
- Percentage mass retained on sieve (Mass of soil / Weight of specimen)X 100
- Cumulative percent of soil retained on sieve (Zr) – It is calculate by adding the percentage retained on the sieve and all sieves above the sieve.
- Percentage finer or percent cumulative passing (100 – Zr)
- Plot the graph for log grain or sieve size VS percentage finer (Consider the log sieve or grain size on the x-axis)
- Calculate the Cc - *coefficient of curvature* and Cu - *coefficient of uniformity,* for the soil from the graph,

$$Cu = D60 \ / \ D10$$

$$Cc = (D30)^2 \ / \ (D60 \times D10)$$

(D60 = grain diameter at 60% finer, D30 = grain diameter at 30% finer, D10 = grain diameter at 10% finer).

Step 01

Step 02

Step 01 – Weigh the sieves, specimen and receiving pan separately.

Step 02 – Arrange the sieves in order and pour the specimen into the first sieve.

SIEVE ANALYSIS						
		Soil Description:				
Mass of Pan:		(g)				
Mass of Pan + Dry Soil:		(g)				
Sample Dry Mass:		(g)				
Sieve Size	Sieve Opening (mm)	Retained Soil + Pan (g)	Mass of Dry Soil (g)	% Mass Retained	% Cumulative Retained	% Finer
#4	4.750					
#10	2.000					
#20	0.850					
#40	0.425					
#60	0.250					
#100	0.150					
#200	0.075					
PAN	-					
Total Soil Mass Sieved (g)						
% Loss During Sieve Analysis						
			D_{10}:		mm	
			D_{30}:		mm	
			D_{60}:		mm	
			C_u:			
			C_c:			

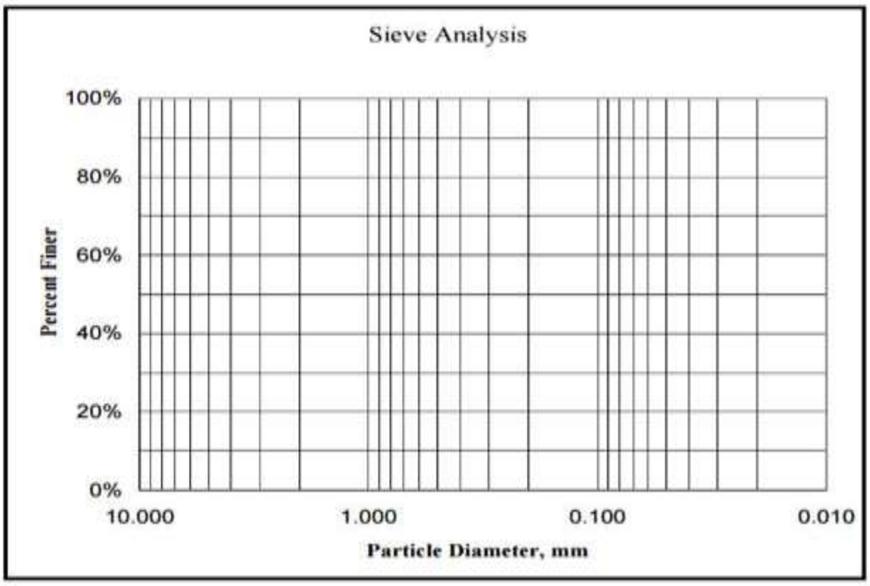
Sample Data Sheet - Sieve Analysis

Step - 04

Step - 05

Step 04 – Keep the arranged sieves on the sieve shaker and allow to shake for 0-15 mins.

Step 05 – Weigh the mass of particles on each sieve.



Graph sheet sample

SAMPLE CALCULATION

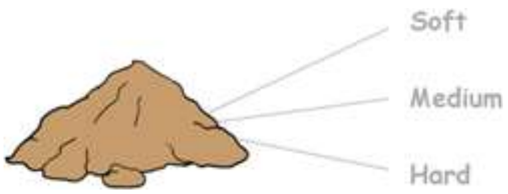
Sample calculation

Sieve Number	Sieve Size (mm)	Mass of each sieve (g) (M1)	Mass of each sieve + soil retained (M2)	Mass of soil (M2-M1)	Percentage retained on each sieve $\left(\frac{(M2-M1)}{M}\right)$ X 100	Cumulative % retained	Percent finer
4	4.750	765.5	786.5	021	4.2	4.2	95.8
10	2.360	738.25	764.25	026	5.2	9.4	90.6
16	1.180	672	716.5	044.5	8.9	18.3	81.7
30	0.600	602.5	649	046.5	9.3	27.6	72.4
40	0.425	572	684	112	22.4	50	50
60	0.300	554.5	701.5	147	29.4	79.4	20.6
100	0.150	523.5	585.5	062	12.4	91.8	8.2
200	0.075	509.5	530.5	021	4.2	96	4
Receiving pan		485	505	020	4	100	0
Total				500 (M)			

M --> Mass of specimen	M2 – M1 --> Mass of soil
M1 --> Sieve Size	Zr --> Cumulative % retained
M2 --> Mass of each sieve + soil retained	Zr - 100 --> Percent finer

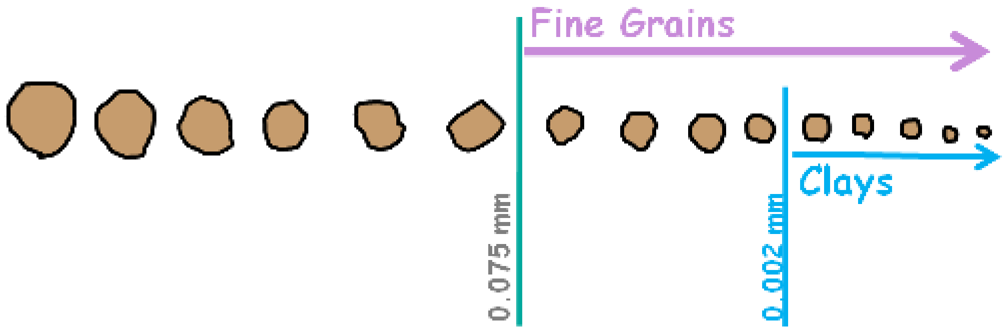
Calculate the Cc and Cu using the plotted graph for the above details.

Consistency Of Soil AndAtterberg's Limits



Consistency is a term which is used to describe the degree of firmness of soil. Consistency of a soil is indicated by terms such as soft medium and hard.

This property of consistency is defined only for fine grained soils, specially for clays and it is measured for wet, moist and dry soil samples.



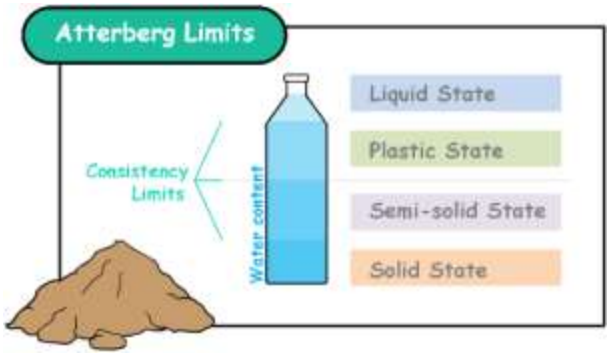
Soil consistency is the resistance of soils to deformation and rupture.

The physical properties of soil are considerably influenced by the amount of water present in them. Consistency of a soil also changes with the amount of water.

A swedish agriculture engineer Albert Atterberg mentioned that depending upon the water content the consistency of fine grained soils can be described in four states. Or in other words we may say depending upon the water content soil may appear in one of these four states

- Liquid State
- Plastic State
- Semi-solid State
- Solid State

In each state, the consistency and behavior of a soil is different and consequently so are its engineering properties. Thus the boundary between each state can be defined based on a change in the soil's behavior.

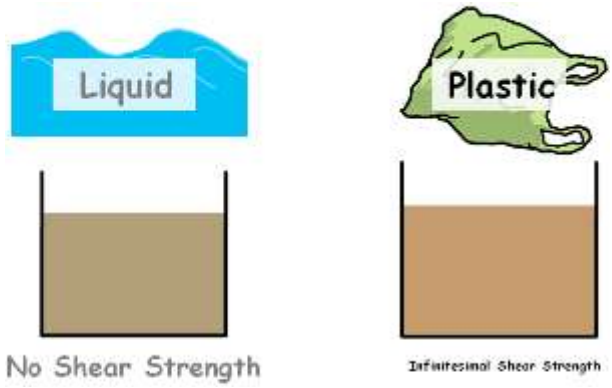


The water content at which the soil changes from one state to other is known as the Consistency limit.

Atterberg demonstrated the significance of these limits in understanding the behaviour of clays. So these limits are also called Atterberg limits.

When we mix fine grained soil in large quantities of water the resulting suspension of soil is called as liquid state of soil. In this state soil has virtually no shear strength which means soil has zero shear strength. Also it offers practically no resistance to flow and it flows like a liquid.

If the water content of the suspension is reduced, the soil becomes stiffer and starts developing resistance to shear



This is the stage when the sample changes from possessing no shear strength to possessing an infinitesimal shear strength and changes from the liquid to plastic state.

The water content at which soil changes from the liquid to plastic state is known as liquid limit.

In other words the water content at which soil stops being liquid is its liquid limit. It does not flow like a liquid anymore.

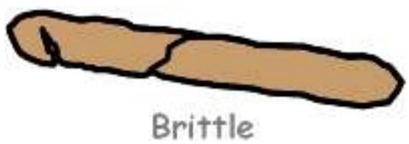
In plastic state soil can be moulded into different shapes without rupturing it due to its plasticity.

Now if we further reduce the water content of the soil, its plasticity decreases and finally soil changes its state from plastic to semi-solidIn this state if we try to mould the soil, it cracks. Soil loses its plasticity and becomes brittle.

The water content at which the soil stops being plastic and changes to semi-solid state is called plastic limit of the soil.

Upto this semi-solid state soil remains fully saturated and in with reduction of water content volume of soil mass reduces almost in equal amount.

But there comes a stage when further reduction in water content, volume of soil remains the same. At that point soil mass changes from semi solid state to solid state.



Soil converts into a soil mass which has pores partially filled with water. Now we can understand that if we reduce the water content, soil's volume remains the same but it's pore water gets reduced. Hence no volume change with water content reduction. That is soil does not shrink any more. And the water content at which soil stops shrinking, is called its shrinkage limit.

Shrinkage limit can also be defined as the lowest water content at which soil is fully saturated.

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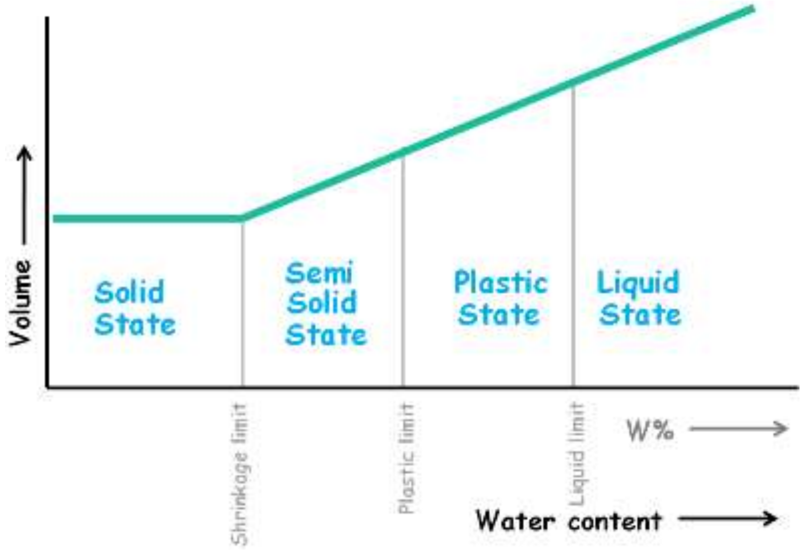
Shrinkage limit can also be defined as the lowest water content at which soil is fully saturated.

If we further reduce the water content below this limit sample begins to dry up at the surface and soil is no longer fully saturated. The colour of the sample also begins to change.



Lets represent it all in a graphical form.

Here we can see with decrease in moisture content of the soil, its volume also decreases. But below the shrinkage limit volume of soil mass remains constant. All these limits are in percentage of water content and are basic measure of the critical water contents of a fine grained soil.

4. CLASSIFICATION OF SOIL

SoilClassification:

It is necessary to adopt a formal system of soil description and classification in order to describe the various materials found in ground investigation. Such a system must be meaningful and concise in an engineering context,so that engineers will be able to understand and interpret.

Itisimportanttodistinguishbetweendescriptionandclassification:

Description of soilis a statement that describes the physical nature and state of the soil. It can be a description of a sample, or a soil*in situ*. It is arrived at by using visual examination, simple tests, observation of site conditions, geological history, etc.

Classification of soil is the separation of soil into classes or groups each having similar characteristics andpotentially similar behaviour. A classification for engineering purposes should be based mainly on mechanical properties: permeability, stiffness, strength. The class to which a soil belongs can be used in its description.

The aim of a classification system is to establish a set of conditions which will allow useful comparisons to be made betweendifferentsoils.Thesystemmustbesimple.Therelevantcriteriaforclassifyingsoilsarethe ***siz distribution*** of particles and the ***plasticity*** of the soil.

Formeasuringthedistributionofparticlesizesinasoilsample,itisnecessarytoconductdifferentparticle-sizetests.

Wet sievingis carried out for separatingfine grains from coarse grains bywashingthe soil specimen on a 75 micron sieve mesh.

Dry sieve analysis is carried out on particles coarser than 75 micron. Samples (with fines removed) are dried and shaken through a set of sieves of descending size. The weight retained in each sieve is measured. The cumulative percentage quantities finer than the sieve sizes (passing each given sieve size) are then determined.

The resultingdata is presented as a distribution curvewith grain sizealongx-axis (logscale) and percentage passing along y-axis (arithmetic scale).

Sedimentation analysisis used only for the soil fraction finer than 75 microns. Soil particles are allowed to settle from a suspension. The decreasing density of the suspension is measured at various time intervals. The procedure is based on the principle that in a suspension, the terminal velocity of a spherical particle is governed by the diameter of the particle and the properties of the suspension.

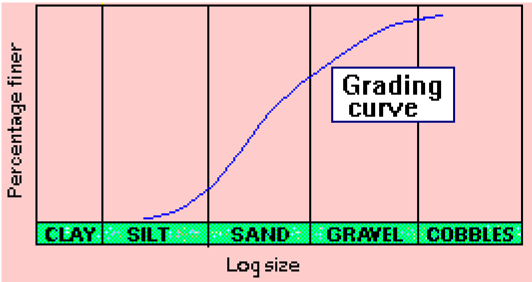
In this method, the soil is placed as a suspension in a jar filled with distilled water to which a deflocculating agent is added. The soil particles are then allowed to settle down. The concentration of particles remaining in the suspension at a particular level can be determined by using a hydrometer.

Specific gravity readings of the solution at that same levelatdifferenttimeintervalsprovideinformationaboutthe sizeofparticlesthathavesettleddown andthe massof soilremaininginsolution.

Theresultsarethenplottedbetween%finer(passing)andlogsize.

Grain-SizeDistributionCurve:

Thesizedistributioncurves,asobtainedfromcoarseandfinegrainedportions,canbecombinedtoformone complete grain-size distribution curve (also known as grading curve). A typical grading curve is shown.



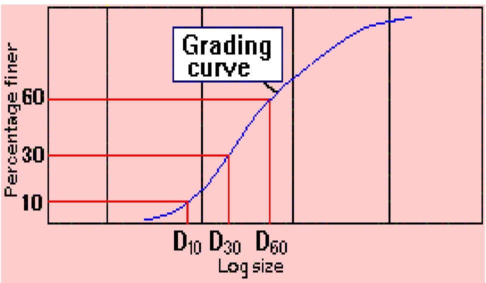
Fromthecompletegrain-sizedistributioncurve,usefulinformationcanbeobtainedsuchas:

1. **Gradingcharacteristics**,whichindicatetheuniformityandrangeingrain-sizedistribution.

2. **Percentages**(or**fractions)**ofgravel,sand,siltandclay-size.

GradingCharacteristics

Agradingcurveisausefulaidtosoildescription.Thegeometricpropertiesofagradingcurvearecalled**gradin**



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Toobtainthegradingcharacteristics,threepointsarelocatedfirstonthegradingcurv

e. D_{60} = size at 60% finer by weight
D_{30}=sizeat30%finerbywei
ght
D_{10}=sizeat10%finerbywei
ght

Thegradingcharacteristicsarethendeterminedasfollows:

1. **Effectivesize**=D_{10}

2. **Uniformitycoefficient,** $C_u = \frac{D_{60}}{D_{10}}$

3.Curvaturecoefficient, $C_c = \frac{(D_{30})^2}{D_{60} \cdot D_{10}}$

Both C_u and C_c willbe1forasingle-sizedsoil.
C_u**>5**indicatesa**well-gradedsoil**,i.e.asoilwhichhasadistributionofparticlesoverawidesizerange.

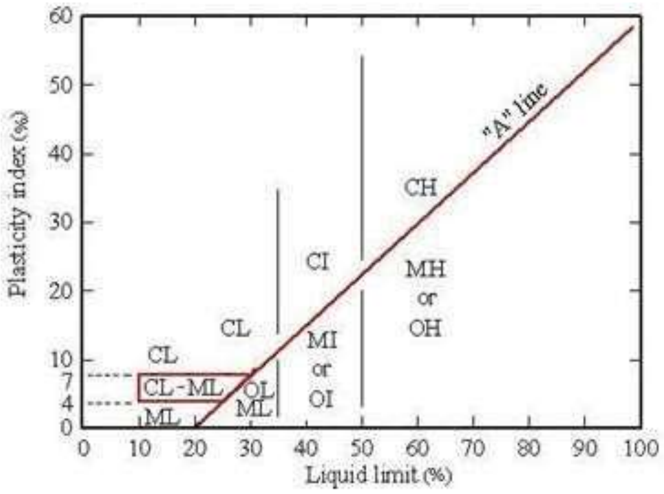
C_c**between 1 and 3** also indicates a well-graded soil.

C_u**<3**indicatesa**uniformsoil**,i.e.asoilwhichhasaverynarrowparticlesizerange.

IndianStandardSoilClassificationSystem:

Fine-grained soils are those for which more than 50% of the material has particle sizes less than 0.075 mm. Clay particles have a **flaky** shape to which water adheres, thus imparting the property of **plasticity**.

A**plasticity chart**, based on the values of liquid limit (**W_L**) and plasticity index (**I_P**), is provided in**ISSCS** to aid classification. The**'A' line** in this chart is expressed as**$I_P = 0.73 (W_L - 20)$**.



Dependingonthepointinthechart,finesoilsaredividedinto **clays(C)**, **silts(M)**,or **organic soils (O)**. The organic contentis expressed as a percentage of the mass of organicmatter in a given mass of soil to the mass of the dry soil solids.Three divisions of plasticityare also defined as follows.

Lowplasticity	**W_L< 35%**
Intermediateplasticity	**35%<W_L< 50%**
Highplasticity	**W_L> 50%**

The'**A'line**andverticallinesat**W_L**equalto**35%**and**50%**separatethesoilsintovarious classes.

Forexample,thecombinedsymbol**CH**referstoclayofhighplasticity.

Soilclassificationusinggroupsymbolsisasfollows:

GroupSymbol	**Classification**
Coarsesoils	
GW	Well-gradedGRAVEL
GP	Poorly-gradedGRAVEL
GM	SiltyGRAVEL
GC	ClayeyGRAVEL
SW	Well-gradedSAND
SP	Poorly-gradedSAND
SM	SiltySAND
SC	ClayeySAND
Finesoils	
ML	SILToflowplasticity
MI	SILTofintermediateplasticity
MH	SILTofhighplasticity
CL	CLAYoflowplasticity
CI	CLAYofintermediateplasticity
CH	CLAYofhighplasticity
OL	Organicsoiloflowplasticity
OI	Organicsoilofintermediateplasticity
OH	Organicsoilofhighplasticity
Pt	Peat

Activity:

"Clayeysoils"necessarilydonotconsistof100%claysizeparticles.Theproportionofclay mineralflakes(<0.002mmsize)inafinesoilincreasesitstendencytoswellandshrinkwith changes in water content. This is called the **activity** of the clayey soil, and it represents the degree of plasticity related to the clay content.

Activity=(PIasticityindex)/(%clayparticlesbyweight)

Classificationasperactivityis:

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

LiquidityIndex

In finesoils,especiallywithclaysizecontent,theexistingstateisdependentonthe currentwatercontent(**w**)with respecttothe consistencylimits(orAtterberglimits).The**liquidityindex(LI)**providesaquantitativemeasureof the present state.

$$LI = \frac{w - W_P}{I_P}$$

Classificationasperliquidityindexis:

Liquidityindex	Classification
>1	Liquid
0.75-1.00	Verysoft
0.50-0.75	Soft
0.25-0.50	Mediumstiff
0-0.25	Stiff
<0	Semi-solid

VisualClassification

Soilspossessanumberofphysicalcharacteristicswhichcanbeusedasaidstoidentificationinthefield.Ahandful of soil rubbed through the fingers can yield the following:

SAND(andcoarser)particlesarevisibletothenakedeye.

SILTparticlesbecomedustywhendryandareeasilybrushedoffhands.

CLAYparticlesarestickywhenwetandhardwhendry,andhavetobescrapedorwashedoffhands.

WorkedExample:

Thefollowingtestresultswereobtainedforafine-grainedsoil: W_L= 48% ; W_P = 26%
Claycontent=55% Silt content = 35%
Sandcontent=10%
*Insitu*moisturecontent=39%=w

Classifythesoil,anddetermineitsactivityandliquidityindex

Solution:

Plasticityindex,I_P=W_L–W_P=48–26=22% Liquid limit lies between 35% and 50%.

AccordingtothePlasticityChart,thesoilisclassifiedasCI,i.e.clayofintermedi

$$\Rightarrow Activity = \frac{I_P}{Clay\ content} = \frac{22}{25} = 0.88$$

Liquidityind $LI = \frac{w - W_P}{I_P} = \frac{39-26}{22} =$

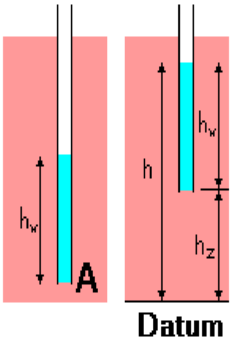
Theclayisofnormalactivityandisofsoftconsi

5. PERMEABILITY AND SEEPAGE

PermeabilityOfSoil:

Pressure,ElevationandTotalHeads

Insoils,theinterconnectedporesprovidepassage forwater.Alargenumberofsuch flowpathsacttogether,andthe averagerateofflowistermedthecoefficientofpermeability,orjustpermeability. Itisameasureoftheeasethatthe soil provides to the flow of water through its pores.



AtpointA,theporewaterpressure(**u**)canbemeasuredfromtheheightofwaterinastandpipelocated

at thatpoint. The height of the water column is the **pressure head** (**h_w**).

$h_w = u/\gamma_w$

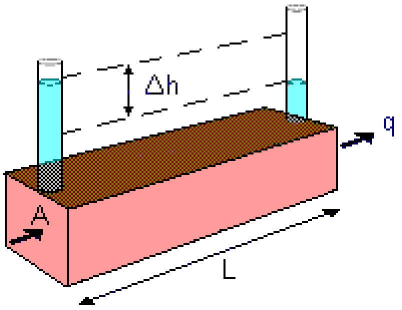
Toidentifyanydifferenceinporewaterpressureatdifferentpoints,itisnecessarytoeliminatetheeffectofthe points of measurement. With this in view, a datum is required from which locations are measured.

The**elevationhead**(**h_z**)ofanypointisitsheightabovethedatumline.Theheightofwaterlevelinthestandpipe above the datum is the **piezometric head** (**h**).

$h = h_z + h_w$

Total head consists of ***three components:*** elevation head, pressure head, and velocity head. As seepage velocity in soilsisnormallylow,velocityheadisignored,andtotalheadbecomesequaltothepiezometric head.Duetothelow seepage velocityand small size of pores, the flow of water in the pores is steadyand laminar in most cases. Water flow takes place between two points in soil due to the difference in total heads.

Darcy'slawstatesthatthereisalinearrelationshipbetweenflowvelocity(**v**)andhydraulicgradient(**i**)foranygiven saturated soil under steady laminar flow conditions.



Iftherateofflowis**q**(volume/time)throughcross-sectionalarea(**A**)ofthesoilmass,Darcy'sLawcanbeexpressed as

v=q/A=k.i

where**k**=permeabilityofthesoil
i=Δh/L
Δ**h**=differenceintotalheads
L=lengthofthesoilmass

The flow velocity (**v**) is also called the Darcian velocity or the **superficial velocity**. It is different from the actual velocityinsidethesoilpores,whichisknownasthe**seepagevelocity,v**$_S$. Attheparticulatelevel,thewaterfollowsa tortuous path through the pores. Seepage velocity is always greater than the superficial velocity, and it is expressed as:

$$v_s = \frac{q}{A_v} = \frac{q}{A_v} \cdot \frac{A}{A} \approx \frac{v}{n}$$

where**A**$_V$=Areaofvoidsonacrosssectionnormaltothedirectionofflow
n=porosityofthesoil

PermeabilityofDifferentsoils:

Permeability(**k**)isanengineeringpropertyofsoilsandisafunctionofthesoiltype. Itsvaluedependsontheaverage size of the pores and is related to the distribution of particle sizes, particle shape and soil structure. The ratio of permeabilities of typical sands/gravels to those of typical clays is of the order of **10^6**. A small proportion of fine material in a coarse-grained soil can lead to a significant reduction in permeability.

Fordifferentsoiltypesaspergrainsize,theordersofmagnitudeforpermeabilityareasfollows:

Soil	k (cm/sec)
Gravel	10^0
Coarsesand	10^0to10^{-1}

Mediumsand	10^{-1}to10^{-2}
Finesand	10^{-2}to10^{-3}
Siltysand	10^{-3}to10^{-4}
Silt	$1x10^{-5}$
Clay	10^{-7}to10^{-9}

Insoils,thepermeantorporefluidismostlywaterwhosevariationinpropertyis generallyveryless.Permeabilityof all soils is strongly influenced by the density of packing of the soil particles, which can be represented by void ratio (**e**)orporosity(**n**).

ForSands

In sands, permeability can be empirically related to the square of some representative grain size from its grain-size distribution. For filter sands, Allen Hazen in 1911 found that **k** **100(D$_{10}$)**^{2}cm/swhere**D$_{10}$**= effectivegrainsizein cm.

Differentrelationshipshavebeenattemptedrelatingvoidratioandpermeability,suchas**k** **e^2**. TheyhavebeenobtainedfromtheKozeny-Carmanequationforlaminarflowinsaturatedsoils.

$$k = \frac{1}{k_0 k_T S_S^2} \cdot \frac{e^3}{1+e} \cdot \frac{\gamma_w}{\eta}$$

where**k$_o$**and**k$_T$**arefactorsdependingontheshapeandtortuosityoftheporesrespectively,**S$_S$**isthesurface areaof the solid particles per unit volume of solid material, and areunitweightandviscosityoftheporewater.

$$k = C.\frac{e^3}{1+e} \approx C.e^2$$

ForSiltsandCla

Forsiltsandclays,theKozeny-Carmanequationdoesnotworkwell,and**logk**versus**e**plothasbeenfoundto indicate a linear relationship.

Forclays,itistypicallyfoundthat

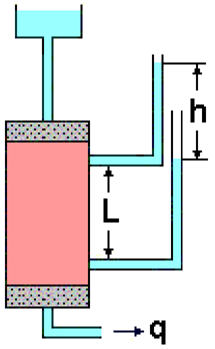
$$\log_{10} k = \frac{e - e_k}{C_k}$$

where**C$_k$**isthepermeabilitychangeindexand**e$_k$**isareferencevoidratio.

LaboratoryMeasurementofPermeability:

ConstantHeadFlow

Constant head permeameter is recommended for coarse-grained soils only since for such soils, flow rate is measurablewithadequateprecision.Aswaterflowsthroughasampleof cross-sectionareaA,steadytotalhead drop **h** is measured across length **L**.

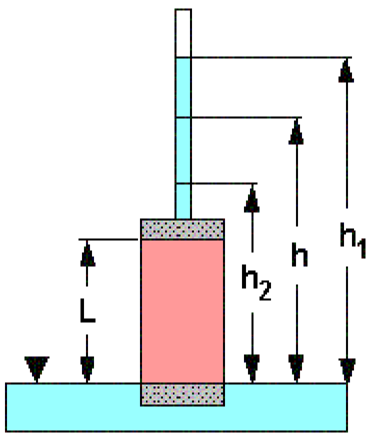


Permeability**k**isobtainedfrom:

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

FallingHeadFlow:

Fallingheadpermeameterisrecommendedforfine-grainedsoils.



Totalhead**h**instandpipeofarea**a**isallowedtofall.Hydraulicgradientvarieswithtime.Heads $\mathbf{h_1}$and$\mathbf{h_2}$are measured at times $\mathbf{t_1}$and $\mathbf{t_2}$. At any time **t**, flow through the soil sample of cross-sectional area **A** is

$$q = k.h.\frac{A}{L} \quad \text{---------------} \quad (1)$$

Flowinunittimethroughthestandpipeofcross-sectionalarea**a**is

$$a \times \left(-\frac{dh}{dt}\right) =$$ ------------------(2)

Equating **(1)** and **(2)**,

$$-a.\frac{dh}{dt} = k.h\frac{A}{L}$$

or $$-\frac{dh}{h} = \left(\frac{kA}{La}\right)dt$$

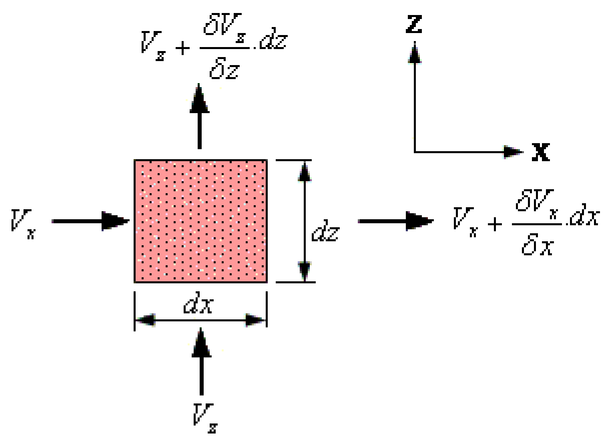
Integrating between the limits,

$$\log_e\left(\frac{h_1}{h_2}\right) = \frac{k.A}{L.a}(t_2 - t_1)$$

$$k = \frac{L.a.\log_e\left(\frac{h_1}{h_2}\right)}{A(t_2 - t_1)}$$

$$= \frac{2.3L.a\log_{10}\left(\frac{h_1}{h_2}\right)}{A(t_2 - t_1)}$$

Seepage in Soils:



A rectangular soil element is shown with dimensions ***dx*** and ***dz*** in the plane, and thickness ***dy*** perpendicuar to this plane. Consider planar flow into the rectangular soil element.

In the **x-direction,** the net amount of the water entering and leaving the element is

$$\frac{\delta V_x}{\delta x}.dx.dy.dz$$

Similarlyinthe**z-direction,**thedifferencebetweenthewaterinflowandoutflowis

$$\frac{\delta V_z}{\delta z}.dz.dx.dy$$

Foratwo-dimensionalsteadyflowofporewater, anyimbalanceinflowsintoandoutofanelement inthez-direction must be compensated by a corresponding opposite imbalance in the x-direction. Combining the above, and dividing by***dx.dy.dz*** ,the **continuity equation** is expressed as

$$\frac{\delta V_x}{\delta x}+\frac{\delta V_z}{\delta z}=0$$

FromDarcy'slaw, $V_x = \mathrm{k_x}.\frac{\delta h}{\delta x}$, $V_z = \mathrm{k_z}.\frac{\delta h}{\delta z}$,where**h**istheheadcausingflow.

WhenthecontinuityequationiscombinedwithDarcy'slaw,theequationforflowisexpressedas:

$$\mathrm{k_x}.\frac{\delta^2 h}{\delta x^2}+\mathrm{k_z}.\frac{\delta^2 h}{\delta z^2}=0$$

Foranisotropicmaterialinwhichthepermeabilityisthesameinalldirections(i.e.k_x=k_z),the**flowequation**is

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

Thisisthe**Laplaceequation**governingtwo-dimensionalsteadystateflow. Itcanbesolved***graphically, analytically, numerically, or analogically.***

Forthemoregeneralsituationinvolving***three-dimensional***steadyflow,Laplaceequationbecomes:

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

One-dimensionalFlow:

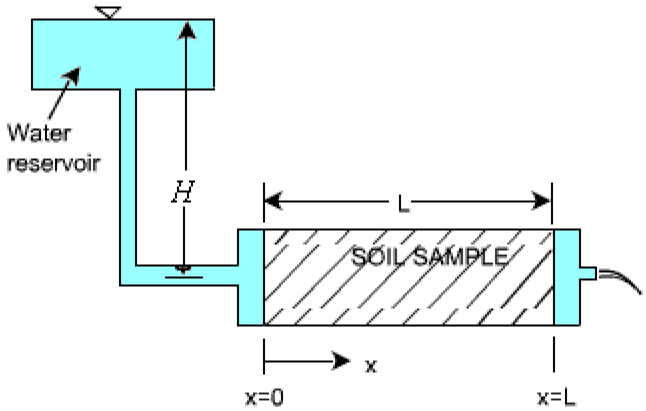
ForThis,the**LaplaceEquation**is $\frac{\delta^2 h}{\delta x^2} = 0$

Integratingtwice,ageneralsolutionisobtained.

$$\frac{\delta h}{\delta x} = c_1$$

$$h = c_2 + c_1 x$$

Thevaluesofconstantscanbedeterminedfromthespecificboundaryconditions.



Asshown,at$x$=0,h=H,andat$x$=L,h=0

Substitutingandsolving,

c_2=H, $c_1 = -\frac{H}{L}$

Thespecificsolutionforflowintheabovepermeameteris

$$h = H - \frac{H}{L}x$$

whichstatesthatheadisdissipatedinalinearlyuniformmanneroverTheentirelengthofthepermeameter.

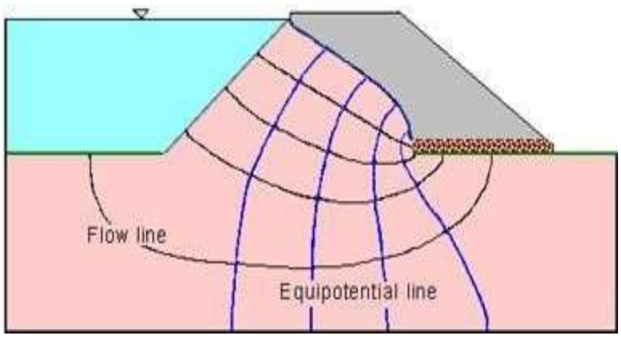
Two-dimensionalFlow:

FlowNets

Graphicalformofsolutionsto**Laplaceequation**fortwo-dimensionalseepagecanbepresentedasflownets.Two orthogonal sets of curves form a flow net:

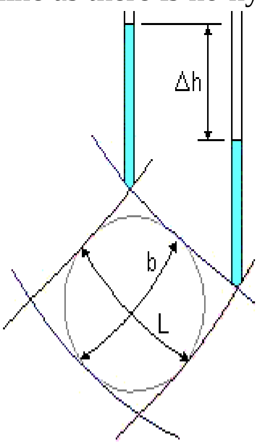
- **Equipotentiallines**connectingpointsofequaltotalhead**h**
- **Flowlines**indicatingthedirectionofseepagedownahydraulicgradient

Two flow lines can never meet and similarly, two equipotential lines can never meet. The space between two adjacentflowlinesisknownasa**flowchannel,**andthefigureformedontheflownetbetweenanytwoadjacentflow lines and two adjacent equipotential lines is referred to as a **field.** Seepage through an embankment dam is shown.



Calculationofflowinachannel

Ifstandpipepiezometerswereinsertedintothegroundwiththeirtipsonasingleequipotentialline,the nthewater would rise to the same level in each standpipe. The pore pressures would be different because of their different elevations. There can be no flow along an equipotential line as there is no hydraulic gradient.



Considerafieldoflength**L**withinaflowchannel.Thereisafalloftotalhead**Δh**.Theaveragehydraulicgradientis

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

Astheflowlinesare**b**apartandconsideringunitlengthperpendiculartofield,theflowrateis

$$\Delta q = kb\frac{\Delta h}{L}$$

Thereisanadvantageinsketchingflownetsintheformof**curvilinear'squares'**sothatacirclecanbeinscribed within each four-sided figure bounded by two equipotential lines and two flow lines.

Insuchasquare,**b=L**,andtheflowrateisobtainedas**Δq=k. Δh**

Thus the flow rate through such a flow channel is the permeability **k** multiplied by the uniform interval **h** between adjacent equipotential lines.

Calculationoftotalflow

Foracompleteproblem, theflownetcanbedrawnwiththeoverallheaddrop**h**dividedinto**N_d**sothatΔ**h=h/N_d**. If **N_f**is the no. of flow channels, then the total flow rate is

$$q = \Delta q.N_f = kh.\frac{N_f}{N_d}$$

LECTURE13

ProcedureforDrawingFlowNets:

At every**point (x,z)**where there is flow, there will be a value of head **h(x,z).** In order to represent these values, contours of equal head are drawn.

Aflownetistobedrawnbytrialanderror. Fora givensetofboundaryconditions,theflownetwillremainthesame even if the direction of flow is reversed. Flow nets are constructed such that the head lost between successive **equipotential lines**is the same, sayΔ**h**. It is useful in visualising the flow in a soil to plot the flow lines, as these are lines that are tangential to the flow at any given point. The steps of construction are:

1. Markallboundaryconditions,anddrawtheflowcrosssectiontosomeconvenientscale.

2. Draw a coarse net which is consistent with the boundary conditions and which has orthogonal equipotential andflow lines. As it is usually easier to visualise the pattern of flow, start by drawing the flow lines first.

3. Modify the mesh such that it meets the conditions outlined above and the fields between adjacent flow lines andequipotential lines are 'square'.

4. Refinetheflownetbyrepeating**step3.**

Themostcommon**boundaryconditions**are:

(a) A submerged permeable soil boundary is an equipotential line. This could have been determined by considering imaginary standpipes placed at the soil boundary, as for every point the water level in the standpipe would be the same as the water level. (Such a boundary is marked as CD and EF in the following figure.)

(b) TheboundarybetweenpermeableandimpermeablesoilmaterialsisaflowIine(ThisismarkedasABinthe same figure).

(c) Equipotentiallinesintersectingaphreaticsurfacedosoatequalverticalintervals.

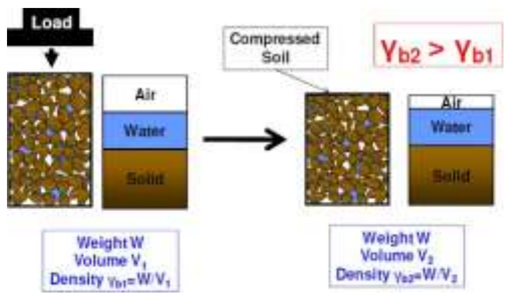
6COMPACTION

Introduction:

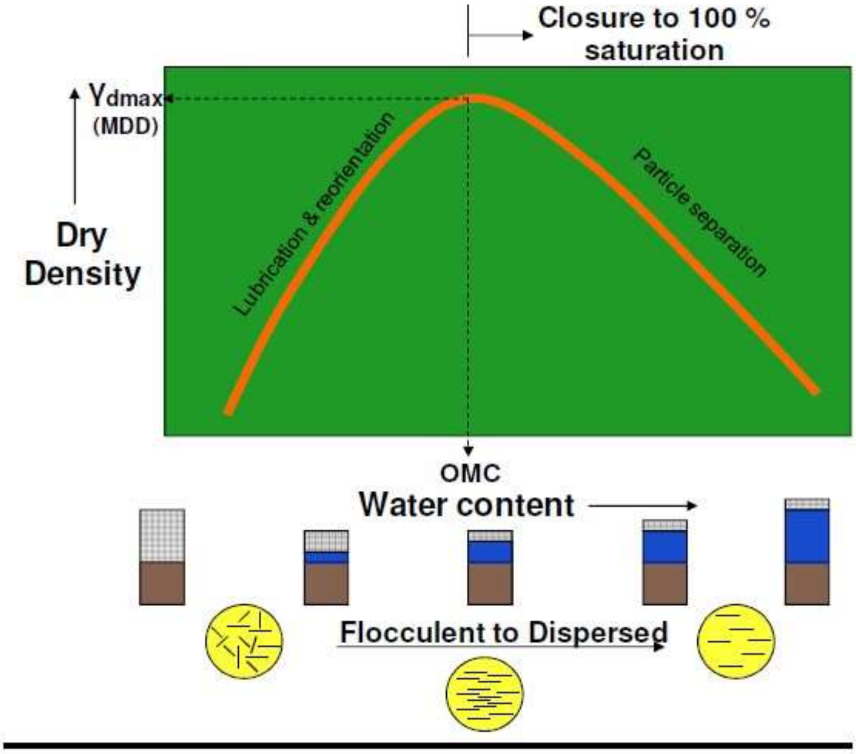
Compaction is the process of increasing the bulk density of a soil or aggregate by driving out air. For any soil, at a given compactive effort, the density obtained depends on the moisture content. An "Optimum Moisture Content" exists at which it will achievea maximum density. Compaction is themethod ofmechanicallyincreasing thedensity of soil. The densification of soil is achieved byreducing air void space. During compaction, air content reduces, but not water content It is not possible to compact saturated soil. It should be noted that higher the densityof soil mass, stronger, stiffer, more durable will be the soil mass.

Hence,Compaction

1) Increases density
2) Increasesstrengthcharacteristics
3) Increasesload-bearingcapacity
4) Decreasesundesirablesettlement
5) Increasesstabilityofslopesandembankments
6) Decreasespermeability
7) Reduceswaterseepage
8) ReducesSwelling&Shrinkage
9) Reducesfrostdamage
10) Reduceserosiondamage
11) Developshighnegativeporepressures(suctions)increasingeffectivestress



MechanismofCompaction-



Optimum Moisture Content (OMC) is the moisture content at which the maximum possible dry density is achieved for a particular compaction energy or compaction method. The corresponding dry density is called Maximum Dry Density(MDD). Water is added to lubricate the contact surfaces of soil particles and improve the compressibility of the soil matrix. It should be noted that increase in water content increases the dry density inmost soils up to onestage (Dry side). Water acts as lubrication. Beyond thislevel, any further increase in water (Wet side)will only add morevoid space,therebyreducingthe drydensity. Hence OMC indicates the boundarybetween the dryside and wet side. Hence the compaction curve as shown in figure indicates the initial upward trend up to OMC and the down ward trend.

Reasonsfortheshapeofcurve

1. On dry side of OMC,clayey soilshows highsuction,lumpsaredifficultto break orcompact.2.Increasing the water content reduces suction, softens lumps, lubricates the grains for easy compaction.

3. Aswatercontentincreases,lubricationimprovescompactionresultinginhigherdrydensity.

4. Nownearly impossibletodriveoutthelastoftheair–furtherincreaseinwatercontentresultsinreduceddry density (curve follows down parallel to the maximum possible density curve – the Zero Air Voids curve)

5. MDDandOMCdependonthecompactionenergyandarenotuniquesoilproperties.

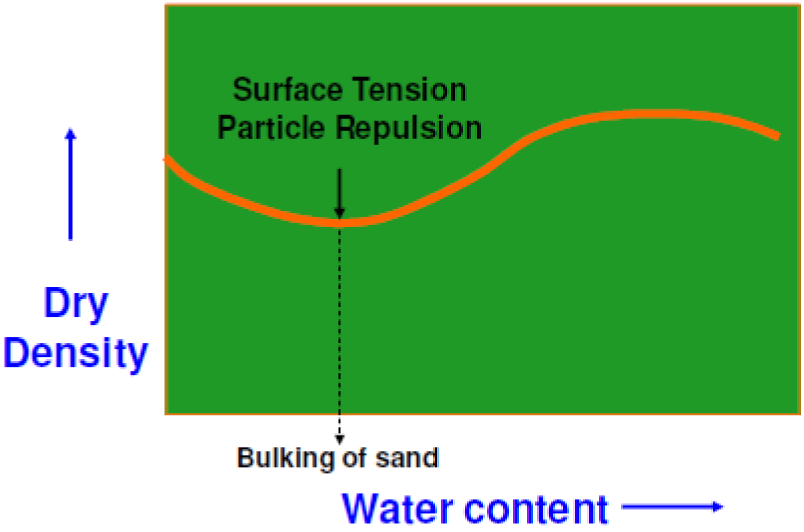
6. Forsand,suctionatlowwatercontentsalsopreventscompaction(butnotifcompletelydry)

7. Incohesionlesssoils,MDDisachievedeitherwhencompletelydry,orwhencompletelysaturated.

8. Atlowwatercontent,grainsareheldtogetherbysuction(wateratgraincontactsonly)

9. Thispreventscompaction.

10. Laboratory test for MDD on sand requires fully saturated sample, and involves vibration.



PercentAirVoids:

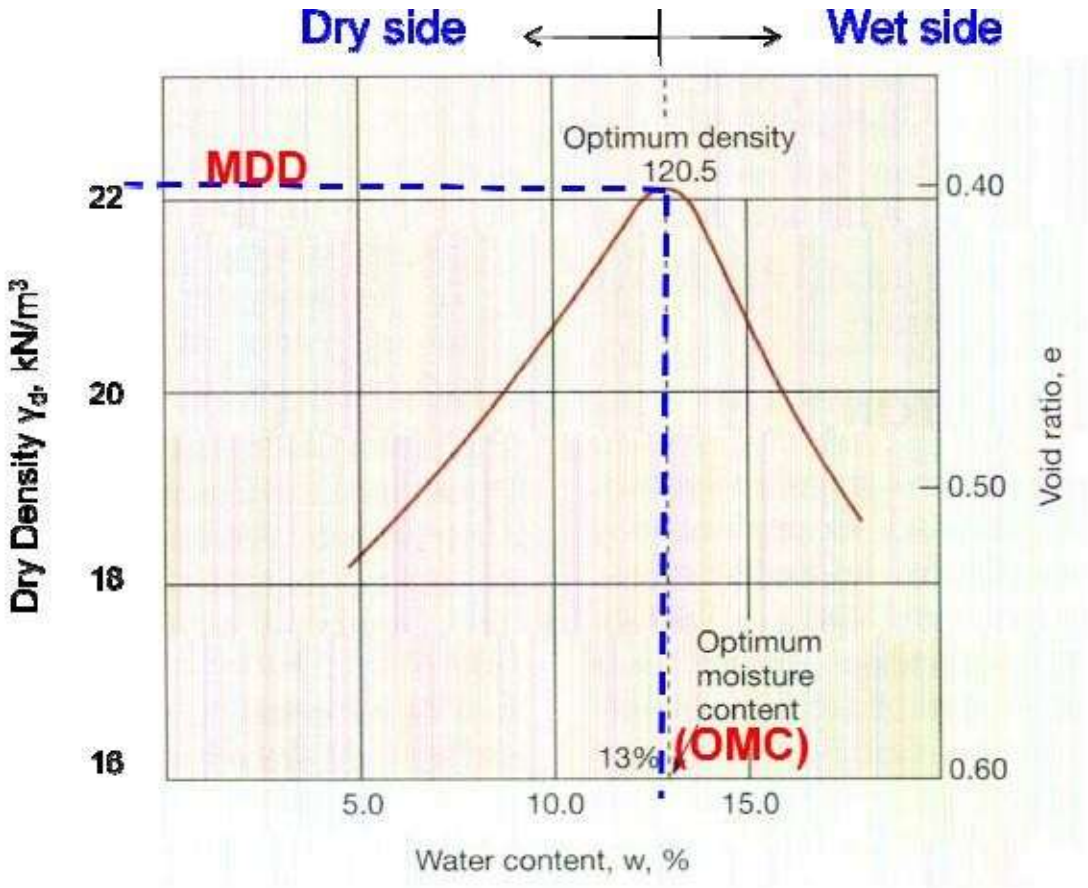
$$\gamma_d = \frac{(1-n_a)G\gamma_{\omega}}{1+\omega G}$$

FactorsaffectingCompaction-

1. WaterContent
2. AmountofCompaction
3. MethodofCompaction
4. TypeofSoil
5. AdditionofAdmixtures

EffectofWaterContent-

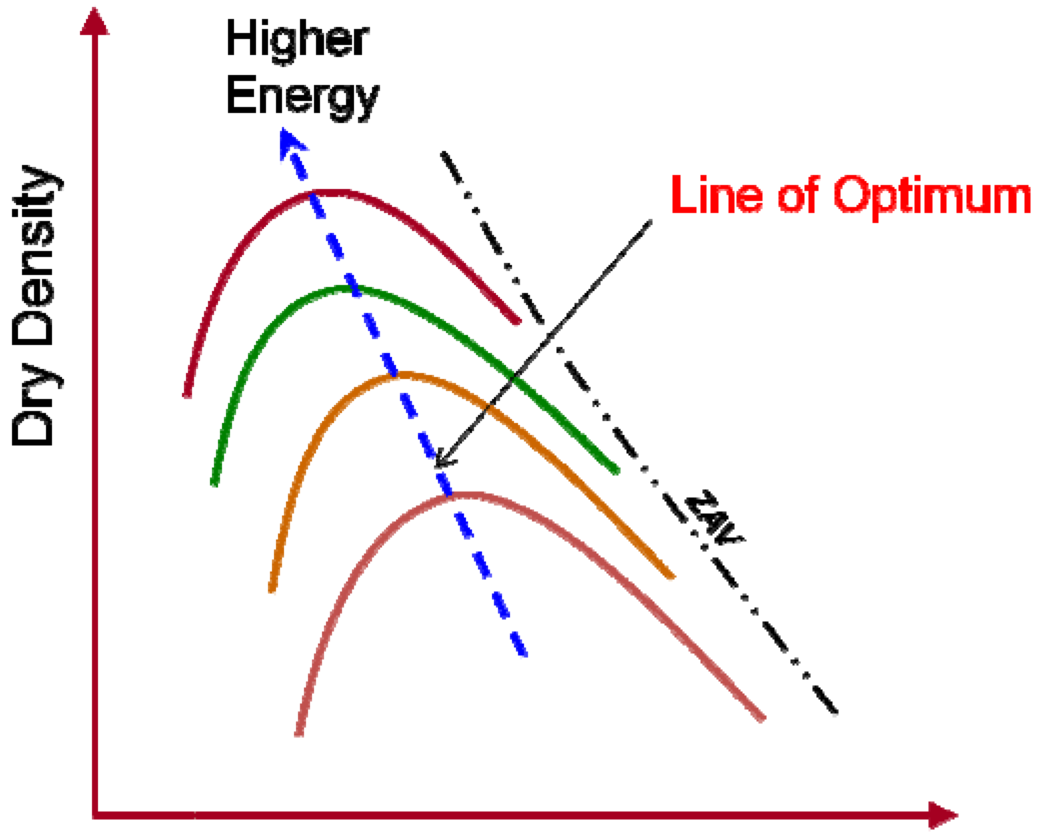
1. Withincreaseinwatercontent,compacteddensityincreasesuptoastage,beyondwhichcompacteddensity decreases.
2. ThemaximumdensityachievediscalledMDDandthecorrespondingwatercontentiscalledOMC.
3. At lower water contents than OMC, soil particles are held by the force that prevents the development of diffused double layer leading to low inter-particle repulsion.
4. Increaseinwaterresultsinexpansionofdoublelayerandreductioninnetattractiveforcebetweenparticles. Water replaces air in void space
5. Particlesslideovereachothereasilyincreasinglubrication,helpingindensepacking.
6. AfterOMCisreached,airvoidsremainconstant.Furtherincreaseinwater,increasesthevoidspace,thereby decreasing dry density.



EffectofAmountofCompaction-

1. As discussed earlier, effect of increasing compactive effort is to increase MDD And reduce OMC (Evident from Standard & Modified Proctor's Tests).

2. However,thereisnolinearrelationshipbetweencompactiveeffortandMDD.



EffectofMethodofCompaction-

Thedrydensityachievedbythesoildependsonthefollowingcharacteristicsofcompactingmethod.

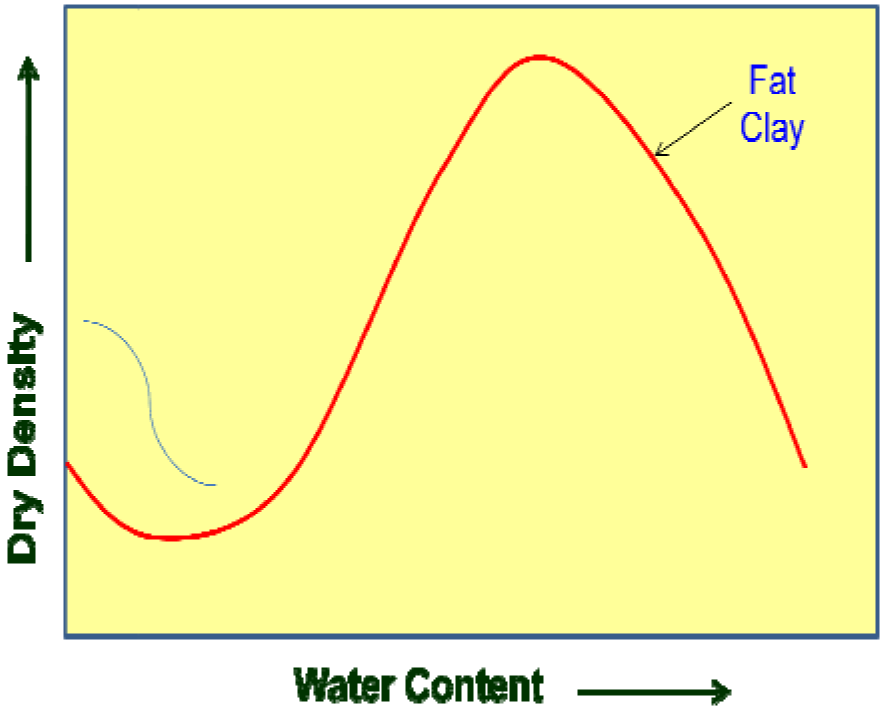
1. Weightofcompactingequipment
2. Typeofcompaction
3. Areaofcontactof
4. Timeofexposure
5. Eachoftheseapproacheswillyielddifferentcompactiveeffort.

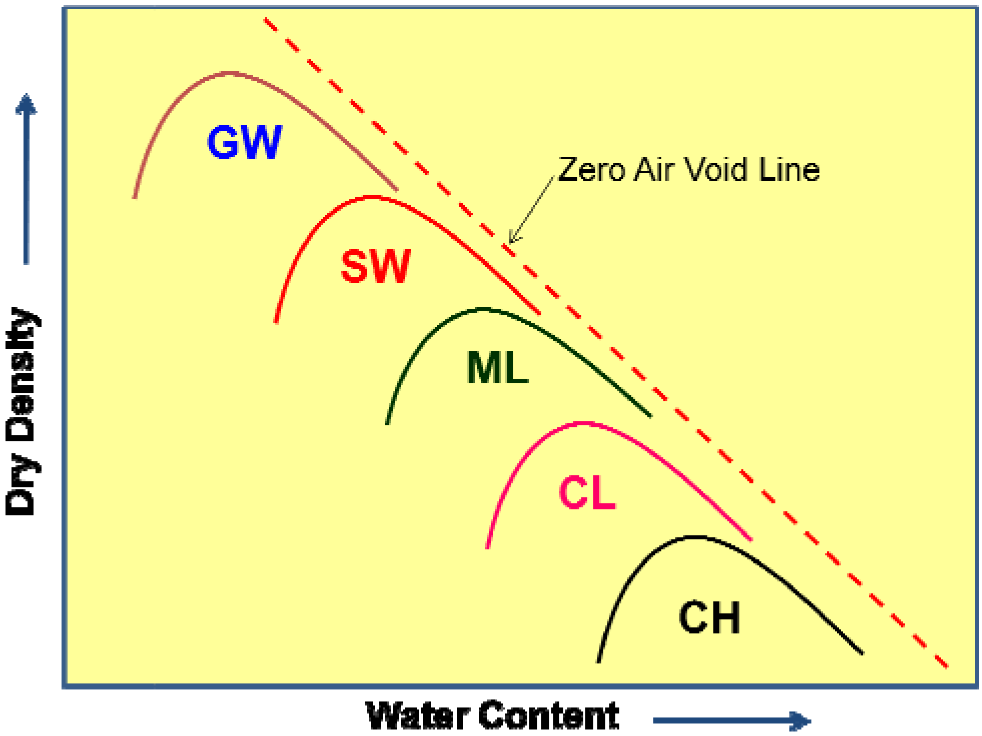
Further,suitabilityofaparticularmethoddepends ontypeofsoil.

Effect of Type of Soil

1. Maximumdensityachieveddependsontypeof
2. Coarse grained soil achieves higher density at lower water content and fine grained soil achieves lesser density, but at higher water content.

Typical Compaction Curve for Fat Clay





EffectofAdditionofAdmixtures-

1. Stabilizingagentsaretheadmixturesaddedtosoil.
2. Theeffectofaddingtheseadmixturesistostabilizethesoil.
3. Inmanycasestheyacceleratetheprocessofdensification.

Effectofcompactiononsoilproperties-

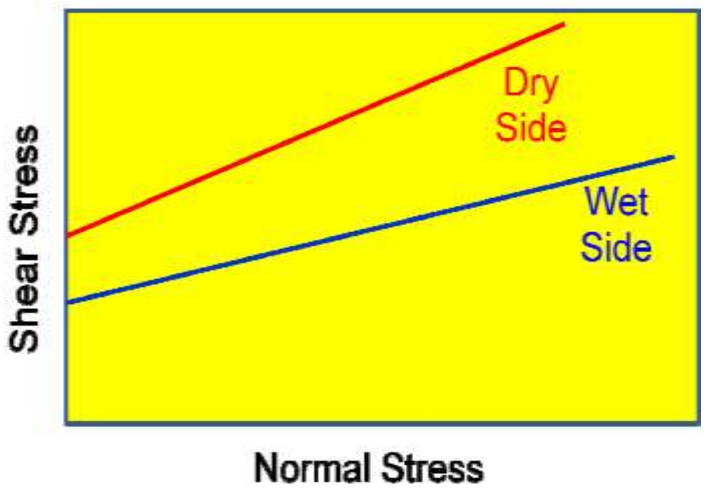
1. Density
2. Shearstrength
3. Permeability
4. BearingCapacity
5. Settlement
6. SoilStructure
7. PorePressure
8. StressStraincharacteristics
9. Swelling&Shrinkage

InfluenceonDensity:

Effect of compaction is to reduce the voids by expelling out air. This results in increasing the dry density of soil mass.

InfluenceonShearstrength:

Increase the number of contacts resulting in increased shear strength, especially in granular soils. In clays, shear strength depends on dry density, moulding water content, soil structure, method of compaction, strain drainage condition etc. Shear strength of cohesive soils compacted dry of optimum (flocculated structure) will be higher than those compacted wet of optimum (dispersed structure).



Effectofcompactiononpermeability

1. Increaseddrydensity,reducesthevoidspace,therebyreducingpermeability.
2. Atsamedensity,soilcompacteddryofoptimumismorepermeable.
3. Atsamevoidratio,soilwithbiggerparticlesizeismorepermeable.
4. Increasedcompactiveeffortreducespermeability.

EffectonBearingCapacity

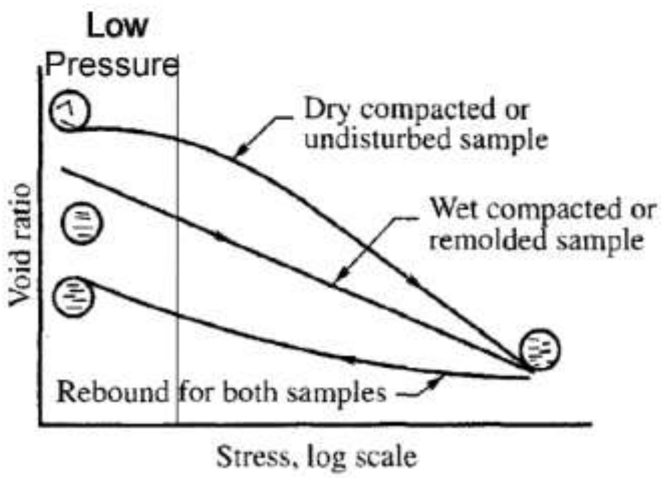
1. Increaseincompactionincreasesthedensityandnumberofcontactsbetweensoilparticles.
2. Thisresultsinincreased
3. Hencebearingcapacityincreaseswhichisafunctionofdensityand

EffectonSettlement

1. Compactionincreasesdensityanddecreasesvoidratio.
2. Thisresultsinreducedsettlement.
3. Bothelasticsettlementandconsolidationsettlementarereduced.
4. Soilcompacteddryofoptimumexperiencesgreatercompressionthanthatcompactedwetofoptimum.

EffectonCompressibility

Optimumshowsmorecompressibilitythanthatondryside.Butathigherpressure,behaviorissimilar.



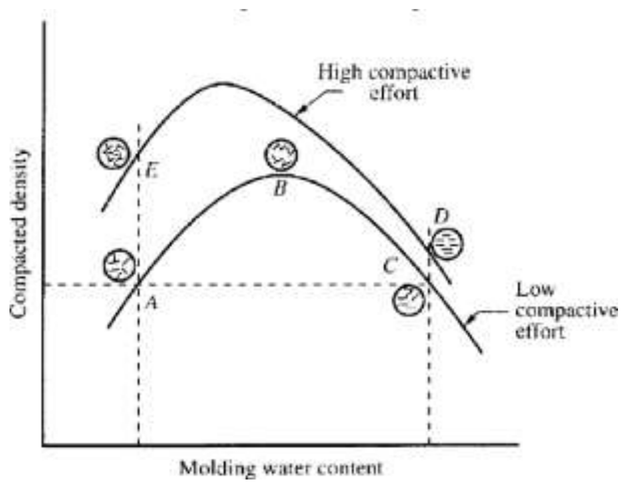
EffectonSoil Structure

Infinegrainedsoil

1. Ondrysideofoptimum,thestructureisflocculated.Theparticlesrepelanddensityisless.
2. Additionofwaterincreaseslubricationandtransformsthestructureintodispersedstructure In

coarse grained soil, single grained structure is maintained

Incompositesoil,behaviourdependsoncomposition.



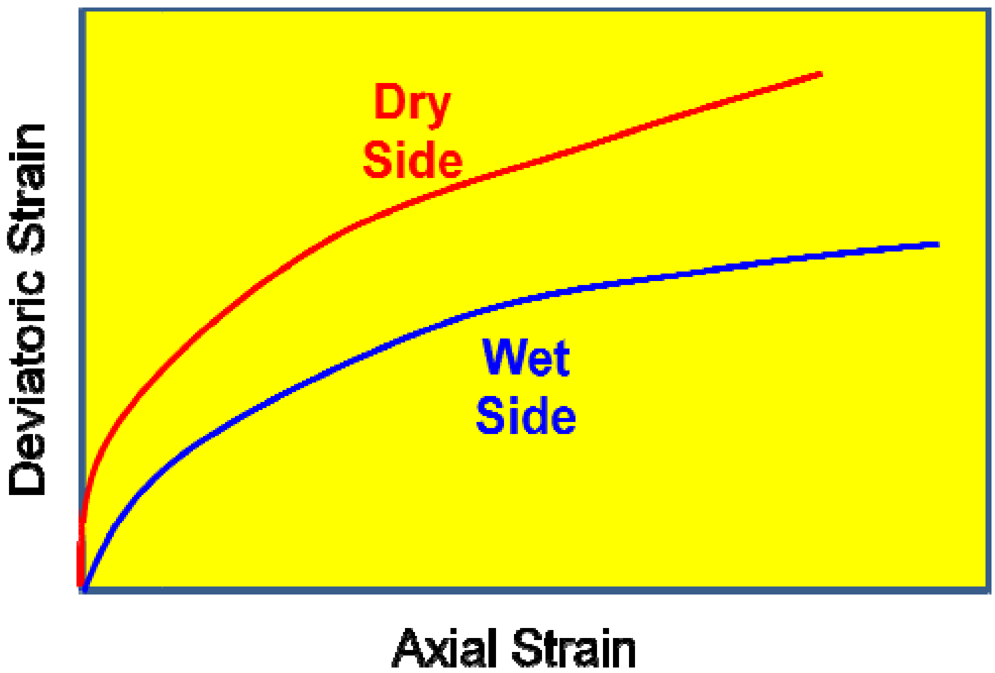
EffectonPorePressure

1. Clayeysoilcompacteddryofoptimumdevelopslessporewaterpressurethanthatcompactedwetofoptimumat the same density at low strains.

2. However,athigherstrainstheeffectisthesameinboththecases.

EffectonStressStrainCharacteristics:

The strength and modulus of elasticity of soil on the dry side of optimum will always be better than on the wet side for the same density. Soil compacted dry of optimum shows brittle failure and that compacted on wet side experiences increased strain.



EffectonSwellShrinkaspect

The effect of compaction is to reduce the void space. Hence the swelling and shrinkage are enormously reduced. Further, soil compacted dry of optimum exhibits greater swell and swell pressure than that compacted on wet side because of random orientation and deficiency in water.

StandardProctor'sCompactionTest

ReferIS2720–PartVII–1987

Apparatus

1. Cylindricalmetalmouldwithdetachablebaseplate(havinginternal diameter101.6mm,internalheight116.8mm and internal volume 945000 mm^3)
2. Collarof50mmeffectiveheight
3. Rammerofweight2.5kgf(25N)withaheightoffallof304.8mm

Procedure

1. About3kgofdrysoil,withalllumpspulverizedandpassingthrough4.75mmsieveistaken.
2. Thequantityofwatertobeaddedinthefirsttrialisdecided.(LessforCorsegrainedsoilandmoreforFine grained soil).
3. Mouldwithoutbaseplate&collarisweighed
4. Theinnersurfacesofmould,baseplateandcollararegreased.
5. Waterandsoilarethoroughlymixed.
6. Soilisplacedinmouldandcompactedinthree uniformlayers,with 25 blowsineach layer. Blowsaremaintained uniform and vertical and height of drop is controlled.

7. Aftereachlayer,topsurfaceisscratchedtomaintainintegritybetweenlayers.

8. Theheightoftoplayerissocontrolledthataftercompaction,soilslightlyprotrudesintocollar.

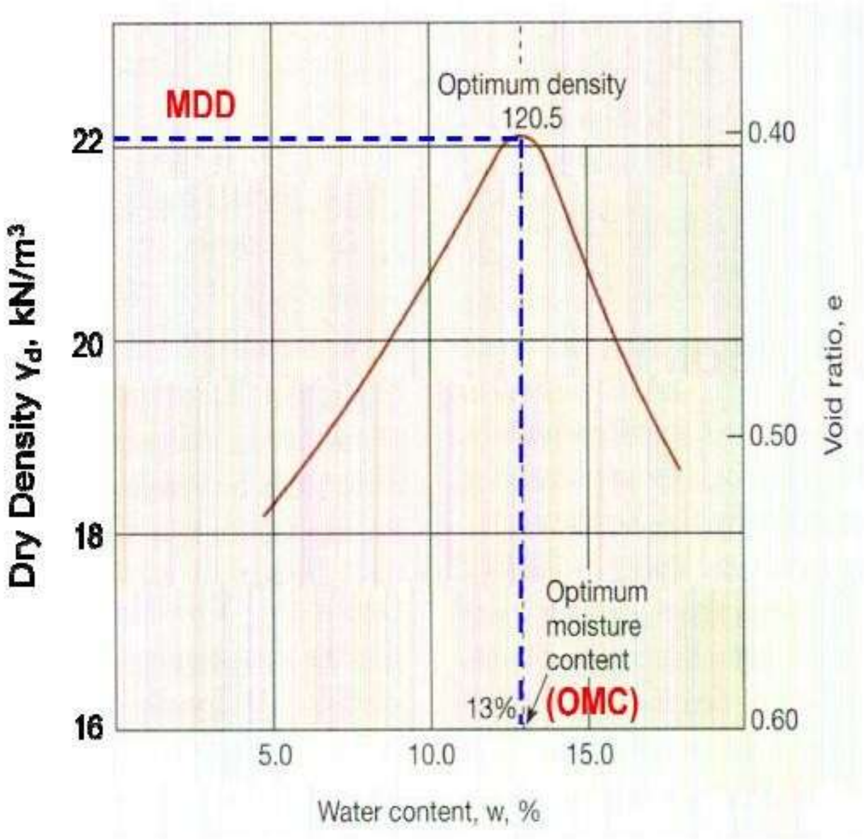
9. Excesssoilisscrapped.

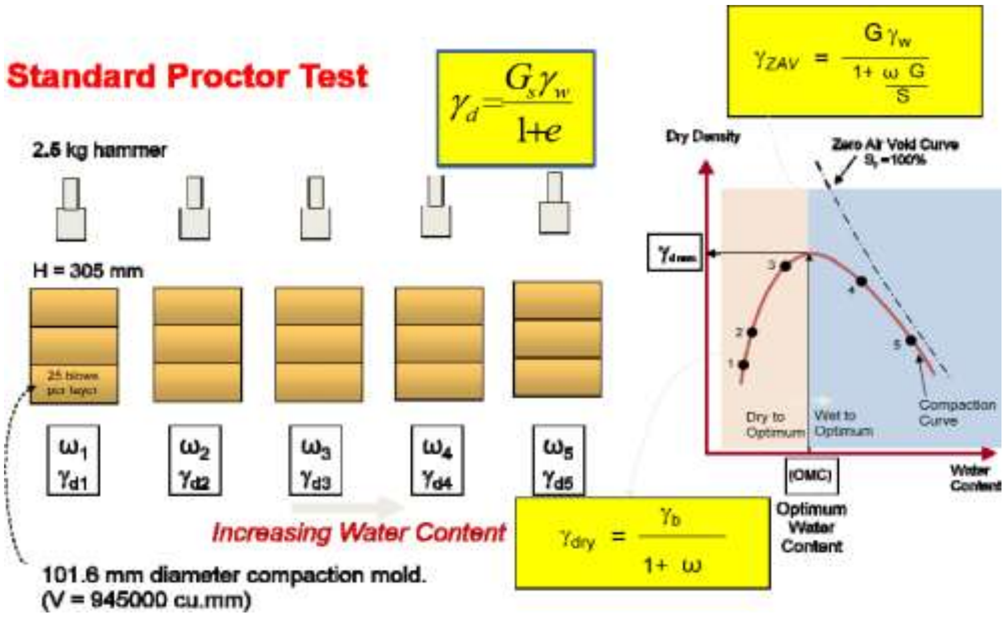
10. Mouldandsoilareweighed(W)

11. Arepresentativesamplefromthemiddleiskeptforthedeterminationofwatercontent.

12. Theprocedureisrepeatedwithincreasingwatercontent.

13. Thenumberoftrialsshallbeatleast6withafewafterthedecreasingtrendofbulkdensity.





ModifiedCompactionTest

In early days, compaction achieved in field was relatively less. With improvement in knowledge and technology, higher compactionbecamea necessityin field.HenceModifiedCompactionTestbecame relevant. It was developed during World War II by the U.S. Army Corps of Engineering tobetter represent the compaction required for airfield to support heavy aircraft.

6.4 Distinction between Standard & Modified Compaction

Standard Proctor Test	Modified Proctor Test
305 mm height of drop	450 mm height of drop
25 N hammer	45 N hammer
25 blows/layer	25 blows/layer
3 layers	5 layers
Mould size: 945 ml	Mould size: 945 ml
Energy 605160 N-mm per m^3	Energy 2726000 N-mm per m^3

Compactive energy

$$\frac{\text{No. of blows per layer} \times \text{Number of layers} \times \text{Weight of hammer} \times \text{Height of drop of hammer}}{\text{Volume of mould}}$$

TypesoffieldCompactionEquipment:

1. SmoothWheeledSteelDrumRollers
2. PneumaticTyredRollers
3. SheepsfootRollers
4. ImpactRollers
5. Vibrating Rollers
6. HandOperatedvibratingplate&rammercompactors

Smoothwheeledsteeldrumrollers

1. Capacity20kNto200kN
2. Selfpropelledortowed
3. Suitableforwellgradedsand,gravel,siltoflowplasticity
4. Unsuitableforuniformsand,siltysandandsoftclay

PneumaticTyredRollers

1. Usuallytwoaxlescarryingrubbertyredwheelsforfullwidthoftrack.
2. Deadload(water)isaddedtogiveaweightof100to400kN.
3. Suitableformostcoarse&finesoils
4. Unsuitableforverysoftclayandhighlyvariablesoil.

SheepsfootRoller

1. Selfpropelledortowed
2. Drumfittedwithprojectingclubshapedfeettoprovidekneadingaction.
3. Weightof50to80kN
4. Suitableforfinegrainedsoil,sand&gravelwithconsiderablefines.

ImpactRoller

1. Compactionbystaticpressurecombinedwithimpactofpentagonalroller.
2. Higherimpactenergybreakssoillumpandprovideskneadingaction

VibratingDrum

1. Rollerdrumfittedwithvibratorymotion.
2. Levelsandsmoothensruts

Plate&RammerCompactor:

Itisusedforbackfillingtrenches,smallerconstructionsandlessaccessiblelocations

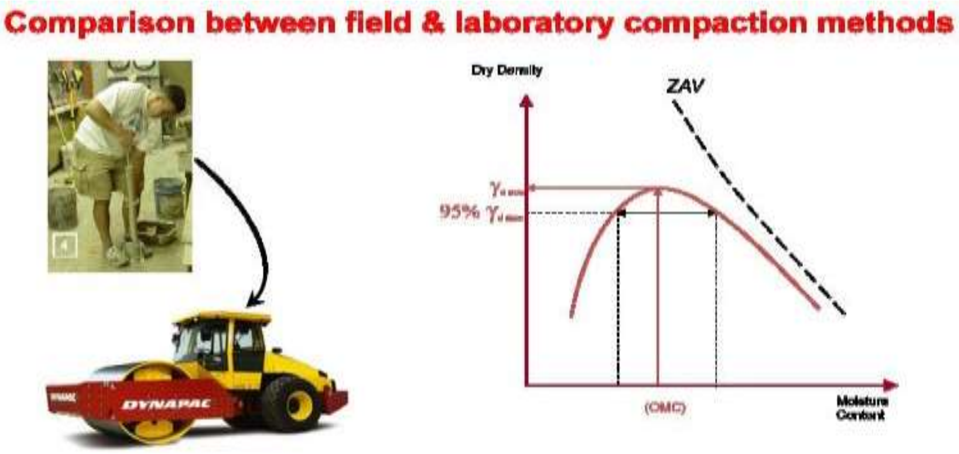
FieldCompactionControl-
Itisextremelyimportanttounderstandthefactorsaffectingcompactionin the field and to estimate the correlation between laboratory and field compaction. Field compaction control depends on

(i) Placementwatercontent,

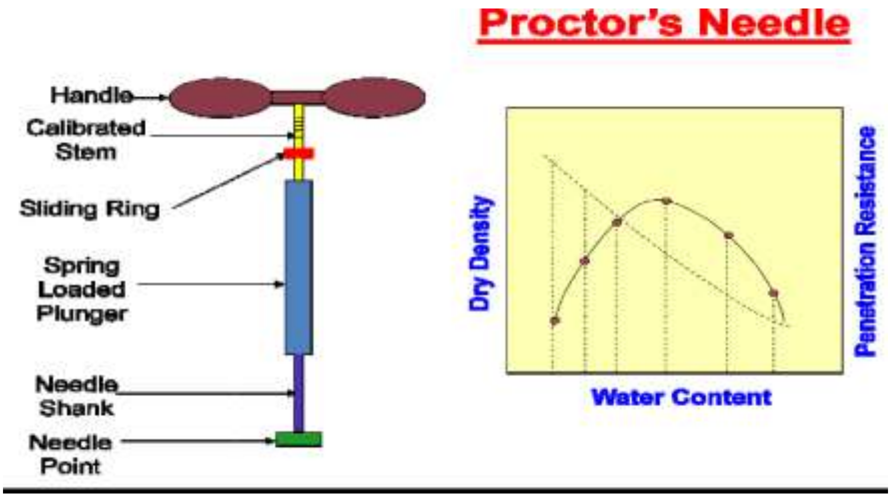
(ii) Typeofequipmentforcompaction

(iii) Liftthickness

(iv) Number of passes based on soil type & degree of c ompaction desired Placement water content isthewater content at which the ground is compacted in the field. It is desirable to compact at or close to optimum moisture content achieved in laboratory so as to increase the efficiency of compaction. However, in certain jobs the compaction is done at lower than or higher than OMC (by about 1– 2 %) depending on the desired function as detailed .



Proctor'sNeedle



1. Usedforrapiddeterminationofwatercontentofsoilinfield.
2. Rapidmoisturemeterisusedasanalternative.
3. Proctor'sneedleconsistsofapoint,attachedtograduatedneedleshankandspringloadedplunger.
4. Varyingcrosssectionsofneedlepointsareavailable.
5. Thepenetrationforceisreadonstemattop.
6. To use the needle in field Calibration in done on the specific soil in lab and calibration curve is prepared and the curve is used in the field to determine placement water content.

Compactioncontrolinfield

Therearemanyvariableswhichcontrolthevibratorycompactionordensificationofsoils.

Characteristicsofthecompactor:

(1) Mass,size

(2) Operatingfrequencyandfrequencyrange

Characteristicsofthesoil:

(1) Initialdensity

(2) Grainsizeandshape

(3) Watercontent

Constructionprocedures:

(1) Numberofpassesoftheroller

(2) Liftthickness

(3) Frequencyofoperationvibrator

(4) Towingspeed

Degreeof Compaction

Relativecompactionordegreeofcompaction

$$R.C. = \frac{\gamma_{d-field}}{\gamma_{d\max-laboratory}} \times 100\%$$

Correlation between relative compaction & relative density $R.C. = 80 + 0.2D_r$

It is a statistical result based on 47 soil samples.
Typical required R.C. >= 95%

CONSOLIDATION

Introduction

Civil Engineers build structures and the soil beneath these structures is loaded. This results in increase of stresses resulting in strain leading to settlement of stratum. The settlement is due to decrease in volume of soil mass. When water in the voids and soil particles are assumed as incompressible in a completely saturated soil system then - reduction in volume takes place due to expulsion of water from the voids. There will be rearrangement of soil particles in airvoids createdbythe outflowofwaterfromthe voids. This rearrangement reflects as avolume change leading to compression of saturated fine grained soil resulting in settlement. The rate of volume change is related to the rate at which pore water moves out which in turn depends on the permeabilityof soil. Therefore the deformation due to increase of stress depends on the "Compressibility of soils"

AsCivilEngineersweneedtoprovideanswersfor

1. Totalsettlement(volumechange)
2. Timerequiredforthesettlementofcompressiblelayer

The total settlement consists of three components

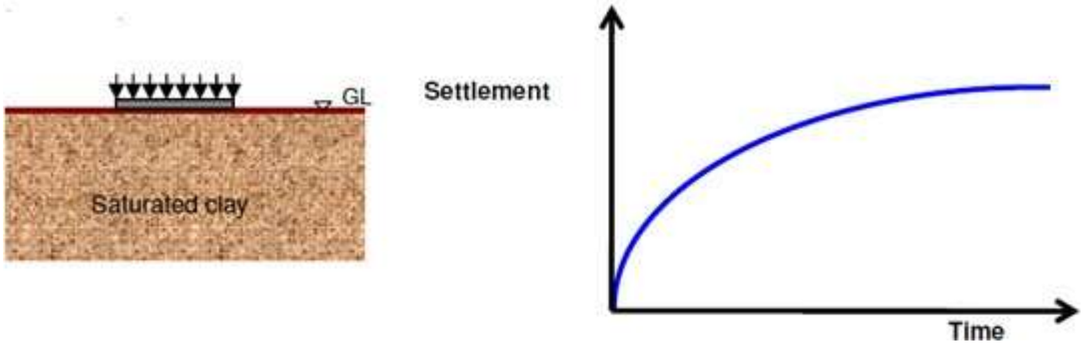
1. Immediate settlement.
2. Primaryconsolidationsettlement
3. Secondaryconsolidationsettlement(Creepsettlement

) St = Si + Sc + Ssc

ElasticSettlementorImmediateSettlement

This settlement occurs immediately after the load is applied. This is due to distortion (change in shape) at constant volume.Thereisnegligibleflowofwaterinlesspervioussoils. Incaseof pervioussoilstheflowofwaterisquickat constant volume. This is determined by elastic theory.

PrimaryConsolidationSettlement



FigureSettlementversusTime

Itoccursduetoexpulsionofporewaterfromthevoidsofasaturatedsoil.Incaseofsaturatedfinegrainedsoils, the deformation is due to squeezing of water fromthe pores leading to rearrangement of soil particles. The movementof pore waterdepends on the permeability and dissipation of pore water pressure. With thepassage of time thepore water pressure dissipates, the rate of flow decreasesand finally the flow of water ceases. During this process there is gradual dissipation of pore water pressure and a simultaneous increase of effectivestress as shown in the above Figure. The consolidation settlement occurs from the timewater begins move out from the pores to the time atwhichflowceasesfromthevoids.Thisis alsothetimefromwhichtheexcessporewaterpressurestarts reducing (effective stress increase) to the time at which complete dissipation of excess pore water pressure (total stress equal to effective stress). This timedependent compression is called “Consolidation settlement”.

Primary consolidation is a major component of settlement of fine grained saturated soils and this can be estimated from the theory of consolidation.

Incaseofsaturatedsoilmasstheappliedstressisbornebyporewateraloneintheinitialstages

$$\therefore \text{At } t = 0 \qquad \Delta\sigma = \Delta u \qquad \Delta\sigma' = 0$$

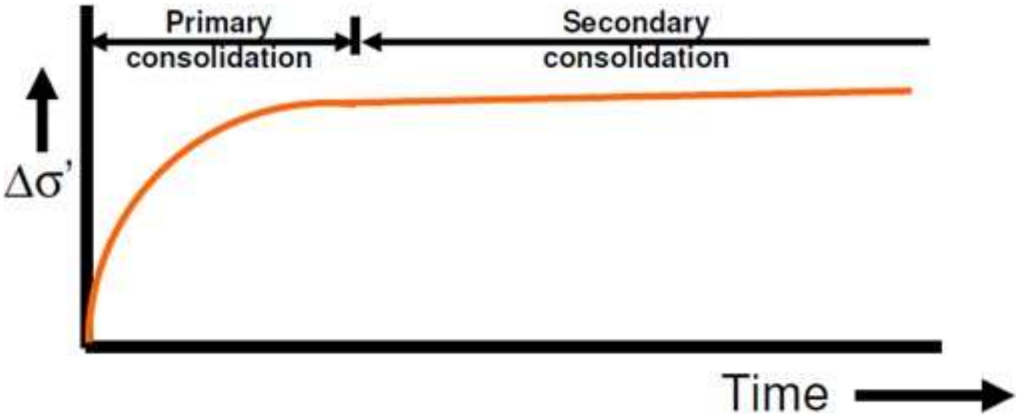
With passageoftimewaterstarts flowingout from thevoids as aresult the excess porewaterpressuredecreases and simultaneous increase in effective stress will takes place. The volume change is basically due to the change in effectivestress Afterconsiderableamountoftime(t=0)flowfromthevoidsceasestheeffectivestressstabilizesand will be is equal to external applied total stressand this stage signifies the end of primary consolidation.

$$\text{At} \quad t = t_1 \qquad \Delta\sigma = \Delta\sigma' + \Delta u$$

$$\text{At} \quad t = \infty \qquad \Delta\sigma = \Delta\sigma' \qquad \Delta u = 0 \text{ (End of primary consolidation)}$$

SecondaryConsolidationSettlement:-

ThisisalsocalledSecondarycompression(Creep). “Itisthechangeinvolumeofafinegrainedsoildueto rearrangement of soil particles (fabric) at constanteffective stress”. The rate of secondaryconsolidation is very slow when compared with primary consolidation.



FigureEffectiveStressversusTime

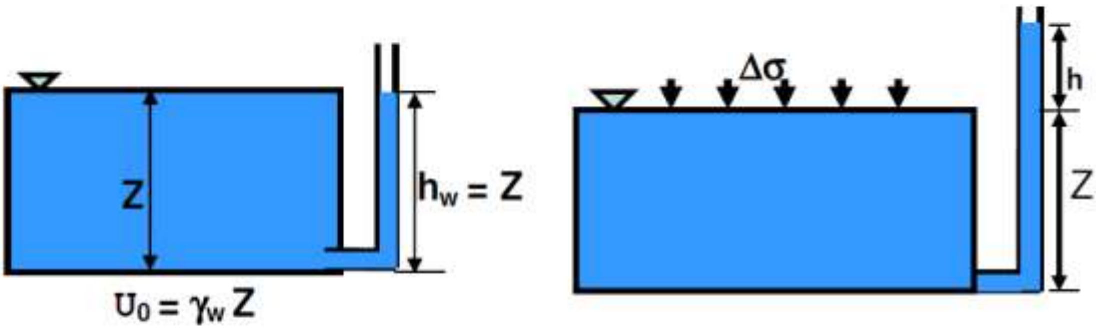
Excess Pore water Pressure (Δu)

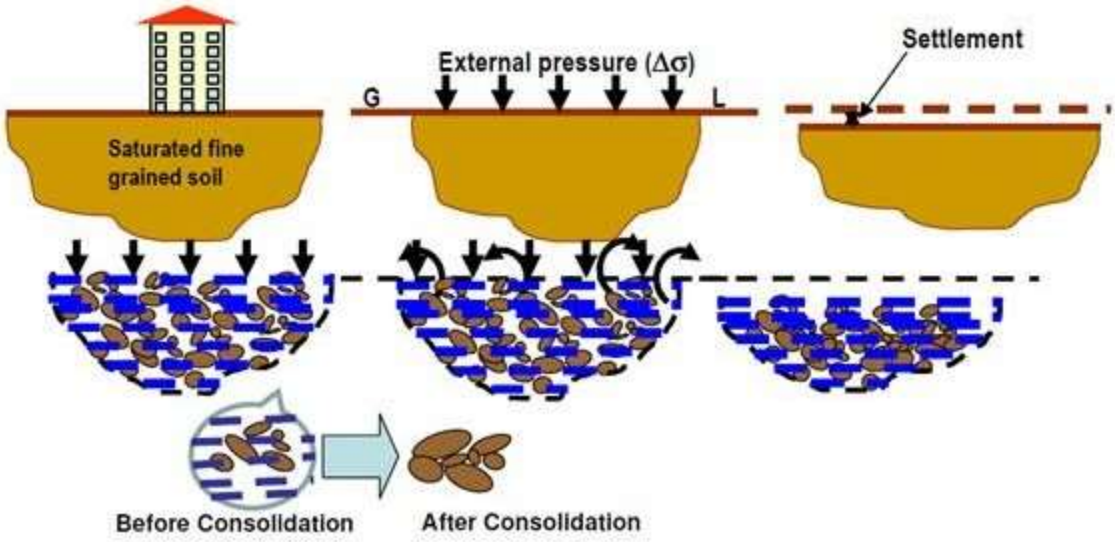
“It is the pressure in excess of the equilibrium pore water pressure”. It is represented as Δu.

$$\Delta u = h\,\gamma_w$$

Where h --- Piezometric head

γ_w --- Unit weight of water





FigureMechanismofvolumechangeinsaturatedfinegrainedsoilunderexternalloading

When saturated soil mass is subjected to external load decrease in volume takes place due to rearrangement of soil particles. Reduction in volume is due to expulsion of water from the voids. The volume change depends on the rate at which water is expelled and it is a function of permeability.

Thetotalverticaldeformation(Consolidationsettlement)dependson

1. Magnitudeofappliedpressure

2. Thicknessofthesaturateddeposi

t We are concerned with

_Measurementofvolumechange

_Thetimedurationrequiredforthevolumechange

SpringAnalogy

The consolidation process is often explained with an idealized system composed of a spring, a container with a hole in its cover, and water. In this system, the spring represents the compressibilityor the structure itself of the soil, and the water which fills the container represents the pore water in the soil.

Onfigure,thetubeontheleftofthecontainershowsthewaterpressureinthecontainer.

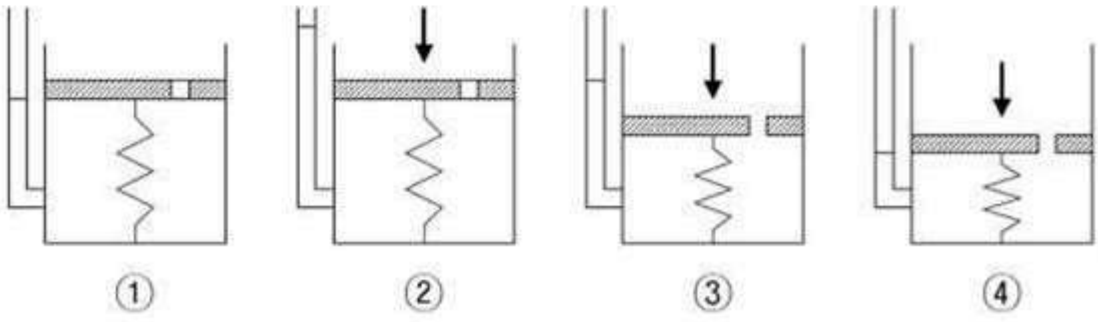


Figure:processofconsolidation

1. Thecontaineriscompletelyfilledwithwater,andtheholeisclosed.(Fullysaturatedsoil)
2. Aloadisappliedontothecover,whiletheholeisstillunopened.Atthisstage,onlythe water resists the applied load. (Development of excessive pore water pressure)
3. Assoonastheholeisopened,waterstartstodrainoutthroughthehole andthespring shortens. (Drainage of excessive pore water)
4. Aftersometime,thedrainageofwaternolongeroccurs.Now,thespringaloneresiststhe applied load. (Full dissipation of excessive pore water pressure. End of consolidation)

Terzaghi'sSpringMassAnalogy-

Terzaghi's model consists of a cylindrical vessel with a series of piston separated by springs. The space between springs is filled with water the pistons are perforated to allow for passage of water. Piezometers are inserted at the centers of different compartment to measure the pressure head due to excess pore water pressure.

Terzaghi has correlated the spring mass compression process with the consolidation of saturated clay subjected to external load. The springs and the surrounding water represent the saturated soil. The springs represent the soil skeleton networks of soil grains and water in the vessels represents the water in the voids. In this arrangement the compression is one dimensional and flow will be in the vertical direction. When pressure is applied this will beborne by water surrounding the spring

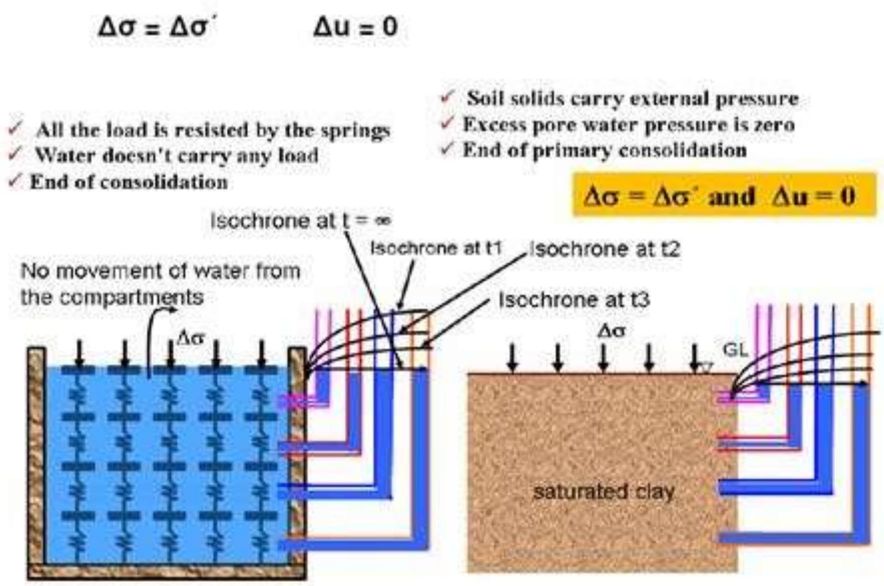
$\Delta\sigma = \Delta u$ at time t =0

Δu is called excess hydrostatic pressure due to this water level in all the Piezometer reach the same height 'h' given by $h = \dfrac{\Delta u}{\gamma_w}$

$\Delta\sigma = \Delta u$ and $\Delta\sigma' = 0$ --------- t=0

There will be no volume change. After some time't' there will be flow of water through perforation beginning from upper compartment. In the lower compartment the volume of water remains constant since the flow is in upward direction.

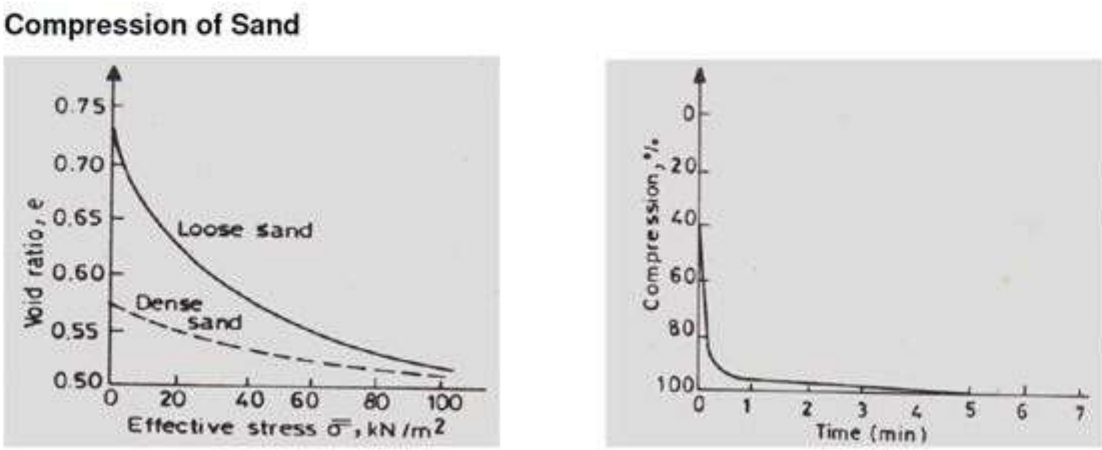
Due to flow of water in the upper segment there will be reduction in volume due to thisspring's get compressed and they being to carry a portion of the applied load. This signifies a reduction in excess hydrostatic pressure or pore water pressure and increase in effective stress in the upper segments. Whereas there will be no dissipation of excess hydrostatic pressure in lower compartments. The isochrones indicate that with passage of time there is flow of water from the lower compartments leading to gradual dissipation of excess hydrostatic pressure. At time t = 0 when no more pore water flows out the excess hydrostatic pressure will be zero in all compartments and the entire load is carried by springs.



FigureCompressionofSpringmass

The compression of a spring mass system is analogous to the consolidation of a saturated fine grained soil depositsubjected to external pressure.

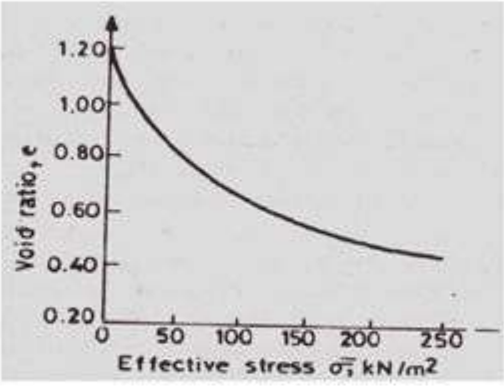
SoilCompressibility

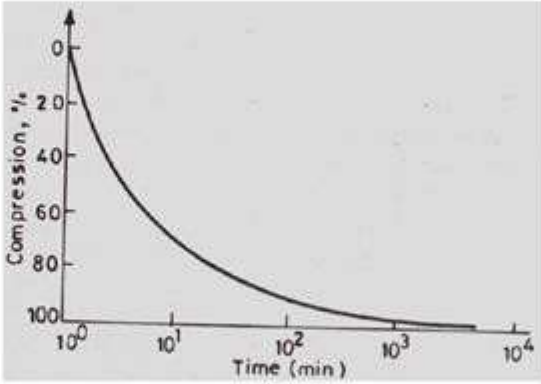


FigureVoidratio-effectivestressandcompression-timeplotsforsand

Sand deposit compresses immediatelyon load application. Loosesand compressesmorethandensesand. Loose and dense sand deposits tend towards the same void ratio.

Compressionoffinegrainedsoil(clay)



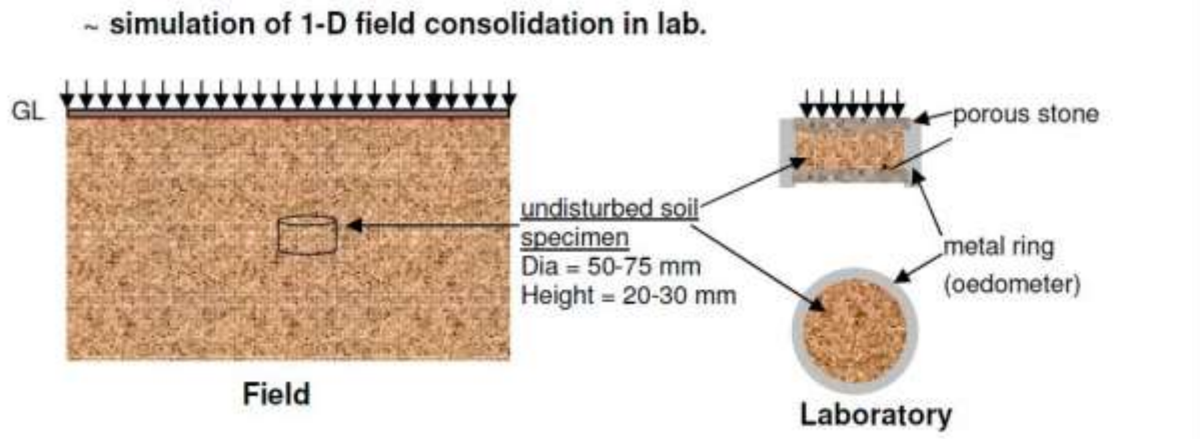


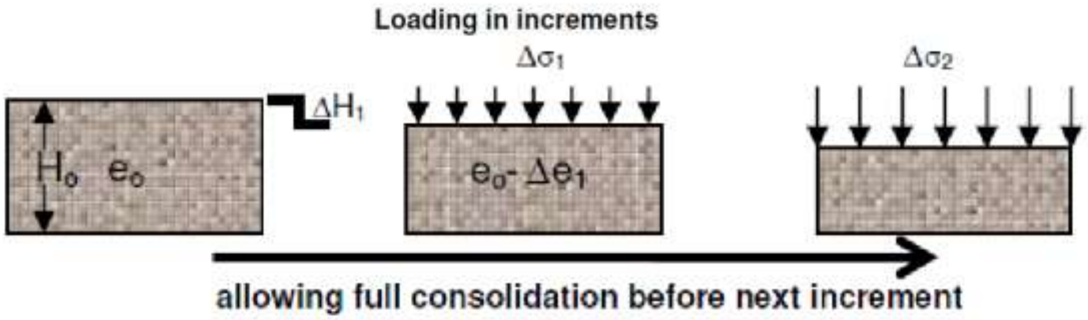
FigureVoidratio-effectivestressandcompression-timeplotsforclay

Timedependentcompressiontakeslongertimecomparedtosand.Themagnitudeofcompressionisalsolarge.

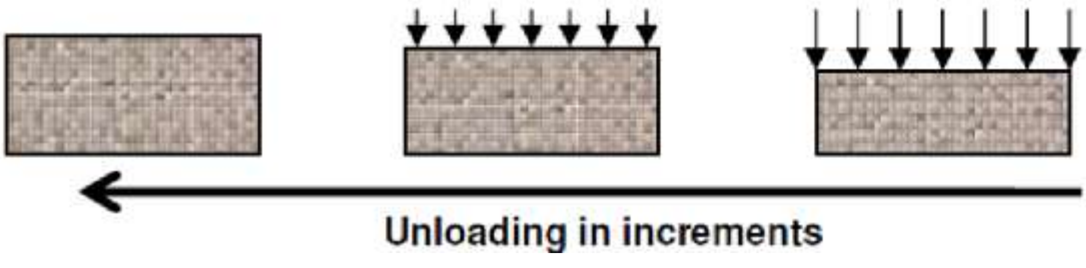
CompressionofFineGrainedSoil

Thecompressibilityoffinegrainedsoilscanbedescribedintermsofvoidsratioversuseffectivestress

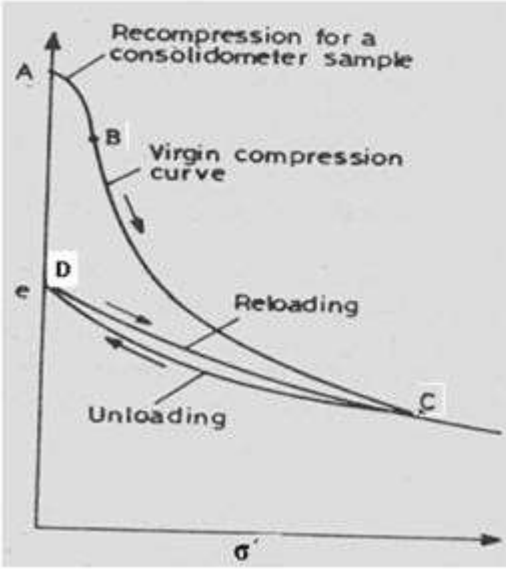




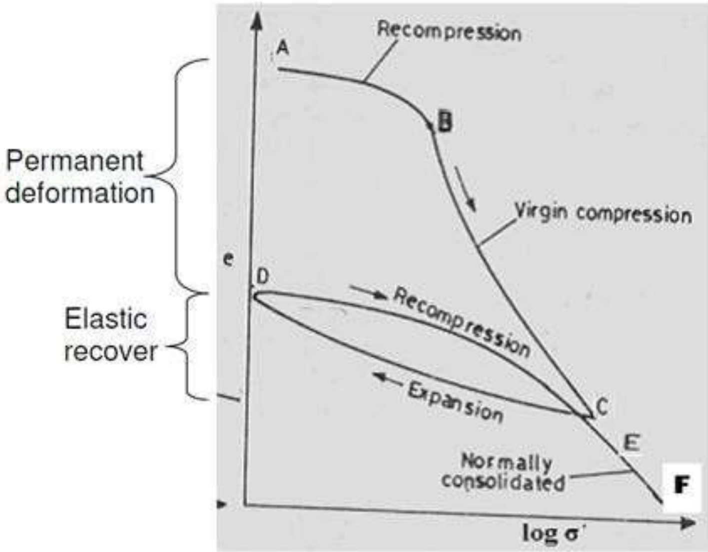
$$\Delta e_1 = \frac{\Delta H_1}{H_o}(1 + e_o) \qquad \Delta e_2 =$$



A laboratory soil specimen of dia 60mm and height 20mm is extracted from the undisturbed soil sample obtained from the field. This sample is subjected to 1D consolidation in the lad under various pressure increments. Each pressure increment is maintained for 24 hrs and equilibrium void ratio is recorded before the application of the next pressure increment. Then a plot of void ratio versus effective stress is made as shown in above figure. When the sampleisrecompressedfrompointDitfollowsDEandbeyondCitmergesalongBCFanditcompressesasit movesalongBCF



FigureVoidratioversuseffectivestress(onarithmeticplot)



FigureVoidratioversuslogarithmofeffectivestress(semi-logplot)

Duringtheinitialstages (atloweffectivestress)samplefollowsrecompression path(portionAB)andundergoes less compression. Beyond this is the virgin compression line (portion BC) also called the normal compression line and the sample undergoes large compression.

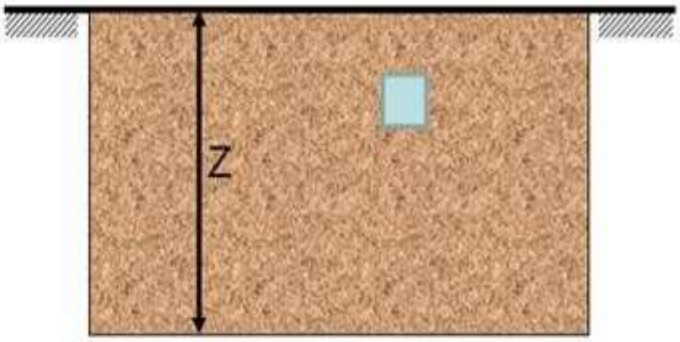
1. BC–Virgincompressioncurvealsocallednormalconsolidationline
2. From‘C’whenthesampleisunloaded,sampleexpandsandtracespathCD(expansioncurveunloading)
3. SampleundergoesPermanentstrainduetoirreversiblesoilstructureandthereisasmallelasticrecovery.
4. Thedeformationrecoveredisduetoelasticrebound
5. Whenthesampleisreloaded-reloadingcurveliesabovetherebound curveandmakes anhysteresisloopbetwee

expansionandreloadingcurves.

6. Thereloadedsoilsshowslesscompression.

7. Loadingbeyond‘C’makesthecurvetomergesmoothlyintoportionEFasifthesoilisnotunloaded.

Terzaghi's1DConsolidationEquation



FigureSaturatedsoilStrata

Assumptions:

- ✓ Thesoilmediumiscompletelysaturated
- ✓ Thesoilmediumisisotropicandhomogeneous
- ✓ Darcy'slawisvalidforflowofwater
- ✓ Flowisonedimensionalintheverticaldirection
- ✓ Thecoefficientofpermeabilityisconstant
- ✓ Thecoefficientofvolumecompressibilityisconstant
- ✓ Theincreaseinstressonthecompressiblesoildepositisconstant
- ✓ Soilparticlesandwaterareincompressible

Onedimensionaltheoryisbasedonthefollowinghypothesis

1. Thechangeinvolumeofsoilisequaltovolumeofporewaterexpelled.
2. Thevolumeofporewaterexpelledisequaltochangeinvolumeofvoids.
3. Sincecompressionisinonedirectionthechangeinvolumeisequaltochangeinheight.

Theincreaseinverticalstressatanydepthisequaltothedecreaseinexcessporewaterpressureatthedepth

$$\Delta\sigma' = \Delta u$$

This is Terzaghi's one dimensional consolidation equation

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

This equation describes the variation of excess pore water pressure with time and depth

Limitationof1Dconsolidation

1. Inthedeviationof1D equationthepermeability(Kz)andcoefficientof volumecompressibility(mv)areassumed constant,butasconsolidationprogressesvoidspacesdecreaseandthisresultsindecreaseofpermeabilityand

thereforepermeabilityisnotconstant.Thecoefficientofvolumecompressibilityalsochangeswithstresslevel. Therefore Cv is not constant

2. Theflowisassumedtobe1Dbutinrealityflowisthreedimensional

3. The application of external load is assumed to produce excess pore water pressure over the entire soil stratum but in some cases the excess pore water pressure does not develop over the entire clay stratum.

Solution of 1D consolidation

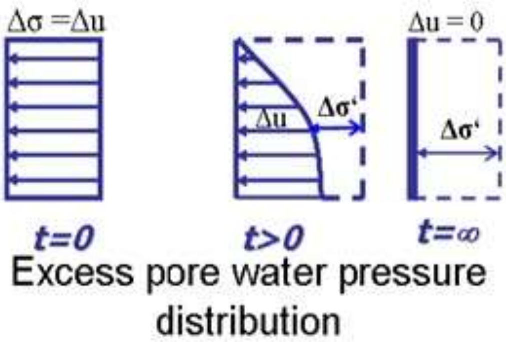
$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2} \quad \text{----------------- } 1$$

Thesolutionofvariationofexcessporewaterpressurewithdepthandtimecanbeobtainedforvariousinitial conditions.

Uniformexcessporewaterpressurewithdepth

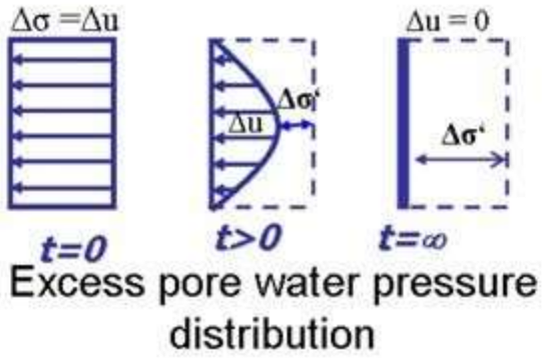
1. SingleDrainage(Drainageattopandbottomimpervious)
2. DoubleDrainage(Drainageattopandbottom)

Single Drainage (drainage at top and bottom impervious)



Excess porewater pressure distribution of single drainage

Double Drainage



Excess porewater pressure distribution of double drainage

Boundary Conditions are

i) At t = 0 $\Delta u = \Delta\sigma$ and $\Delta\sigma' = 0$

ii) At the top z = 0 $\Delta u = 0$ $\Delta\sigma = \Delta\sigma'$

iii) At the bottom z = 2Hdr $\Delta u = 0$ $\Delta\sigma = \Delta\sigma'$

A solution of equation (1) for the above boundary conditions using Fourier series is given by

$$\Delta u_{(z,t)} = \sum_{m=0}^{\infty} \frac{2\Delta u_0}{M} \sin\left(\frac{MZ}{H_{dr}}\right) e^{-M^2 T_v}$$

$M = \frac{\pi}{2}(2m+1)$ Where m = +ve integer with values from 0 to ∞

$Tv = \frac{c_v t}{H_{dr}^{\;2}}$ Where Tv =Time factor (dimensionless)

Graphical solution of 1D consolidation equation

$$\Delta u_{(z,t)} = \sum_{m=0}^{\infty} \frac{2\Delta u_0}{M} \sin\left(\frac{MZ}{H}\right) e^{-M^2 T_v}$$

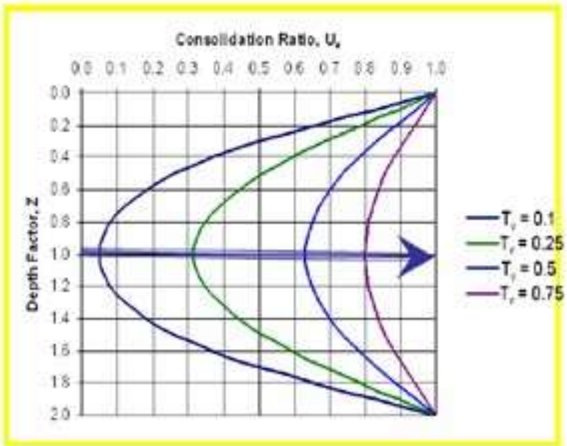
The solution of consolidation equation consists of the following three variables

1. Thedepthz
2. Theexcessporewaterpressure_u
3. Thetime(t)afterapplicationofloading

Theabovevariablesareexpressedintheformofthefollowingnon-dimensionaltermsas

Sl. No	Variables	Non-dimensional terms
1	Depth (z)	Z = z /H (Drainage path ratio)
2	Excess pore pressure (Δu)	Uz --- consolidation ratio This represents the dissipated pore water pressure to initial excess pore water pressure
3	Time (t)	T_v (Time factor)

The graphical solution of the above equation is as shown below



Terzaghi'ssolutionforone-dimentionalconsolidation

Thisindicatestheprogressofconsolidationwithtimeanddepthforagivensetofboundaryconditions.

Degree of Consolidation (U_z)

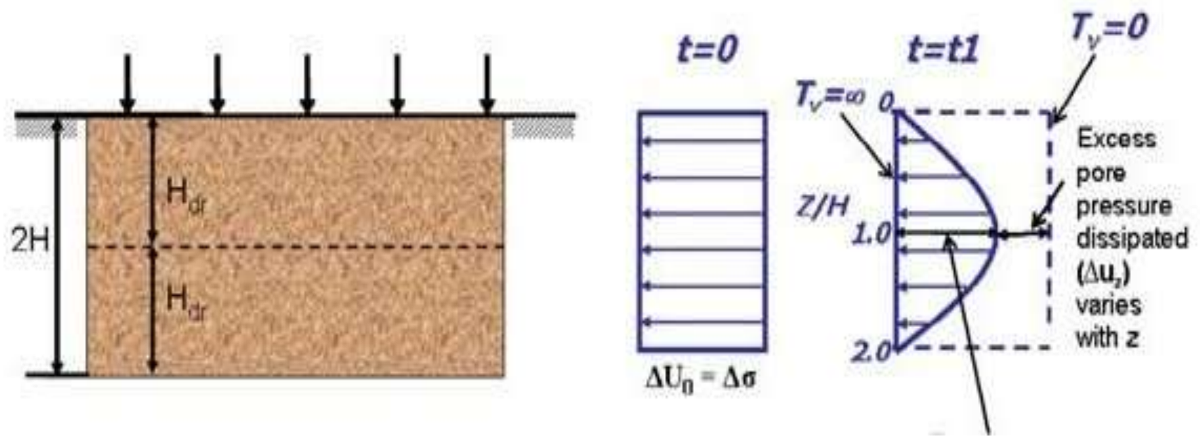


Figure:SelectionofclayayerandExcessporewaterpressuredistribution

The degree of consolidation at any depth is given by

$$U_z = \frac{\Delta u_0 - \Delta u_z}{\Delta u_0}$$

$$1 - \frac{\Delta u_z}{\Delta u_0} = \frac{\Delta \sigma'_z}{\Delta u_0}$$

Δu_o = Initial excess pore water pressure at that depth

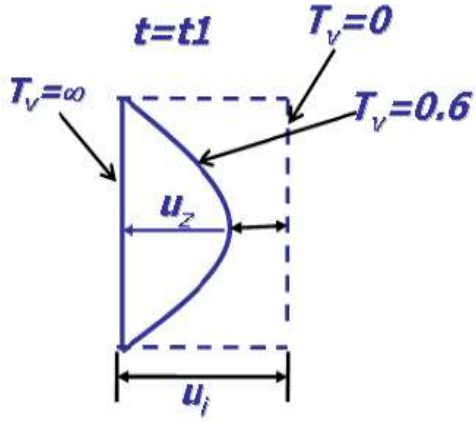
Δu_z = Excess pore water pressure at that depth

$$u_z = 1 - \frac{\Delta u_z}{\Delta u_0}$$

$$u_z = 1 - \sum_{m=0}^{\infty} \frac{2}{M} \sin\left(\frac{MZ}{H}\right) e^{-M^2 T_v}$$

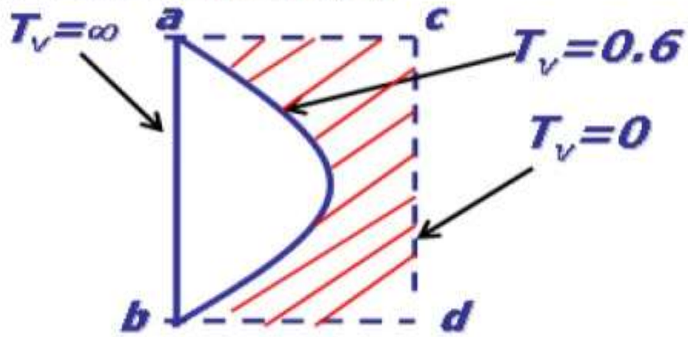
u_z=Degreeofconsolidationataparticulardepthatanygiventime

Frompracticalpointofview,theaveragedegreeofconsolidationovertheentiredepthatanygiventimeisdesirable. At any given time ***u_z****varies* with location and hence the degree of consolidation also varies.



u_z = Degree of consolidation at a certain level

Average Degree of Consolidation (U)



Theaveragedegreeofconsolidationforthewholesoildepositatanytimeisgivenby

$$U = \frac{\text{Area of the diagram of excess pore water pressure dissipated at any time}}{\text{Area of the diagram of initial excess pore water pressure}}$$

$$U = \frac{\text{Area Shaded}}{\text{Area of abcd}}$$

Mathematically $U = f(T_v)$

Consolidation of Soils

$$u = 1 - \sum_{m=o}^{\infty} \frac{2}{M} e^{-M^2 T_v}$$

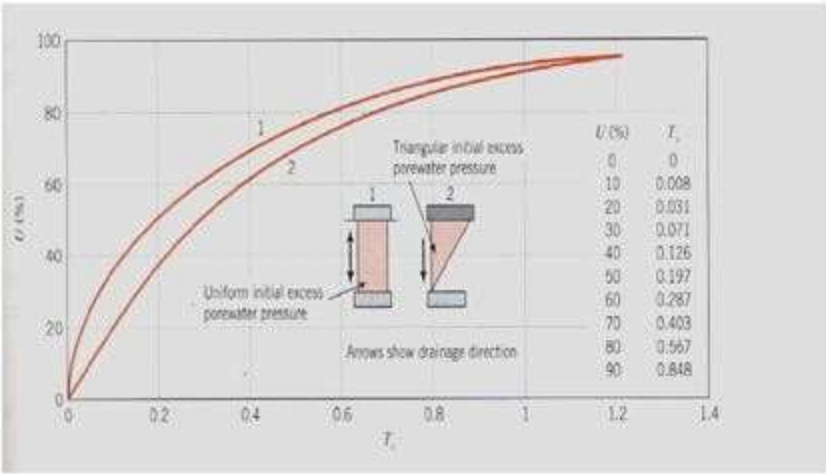


Figure:DegreeofConsolidationversustimefactor(T_v)

As per Taylor (1948) solution, the following approximation is possible

$$when \qquad U \leq 60\% \qquad Tv = \frac{\pi}{4} U^2$$

$$For \quad U > 60\% \qquad Tv = 1.781 - 0.933 \log(100 - U\%)$$

Typical values of T_v

U = 50%	Tv = 0.197
U = 60%	Tv = 0.287
U = 90%	Tv = 0.848

CompressibilityProperties-

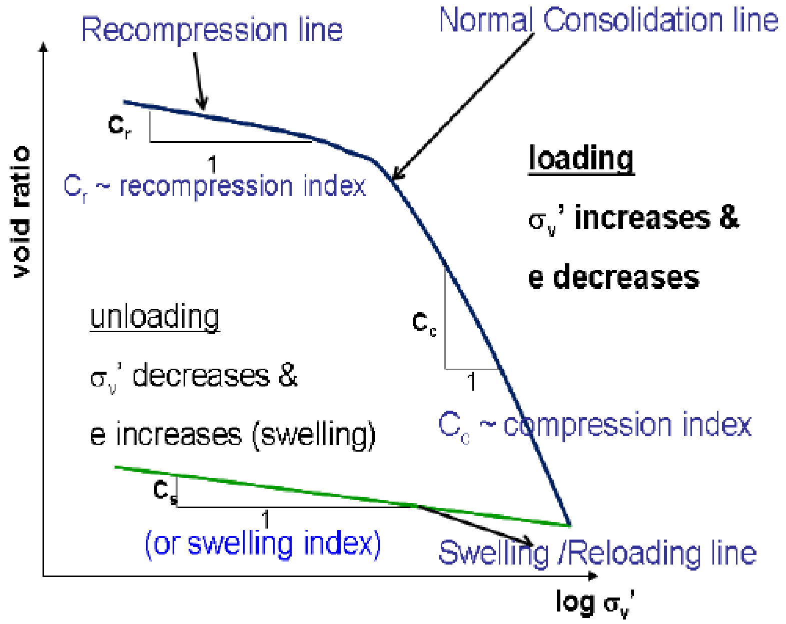


Fig. e – log σ_v' plot

Coefficient of compression/compression index (C_c)

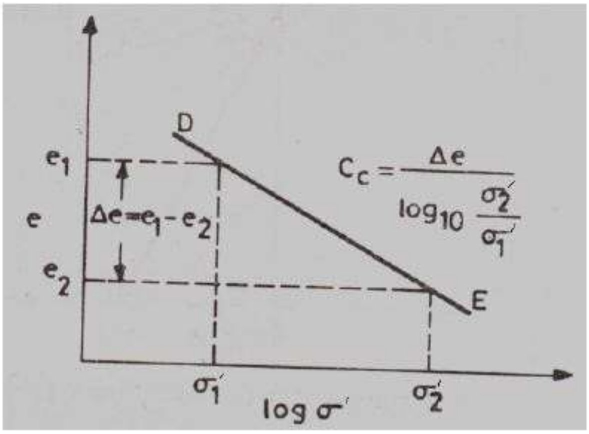


Fig. : e – log σ' plot

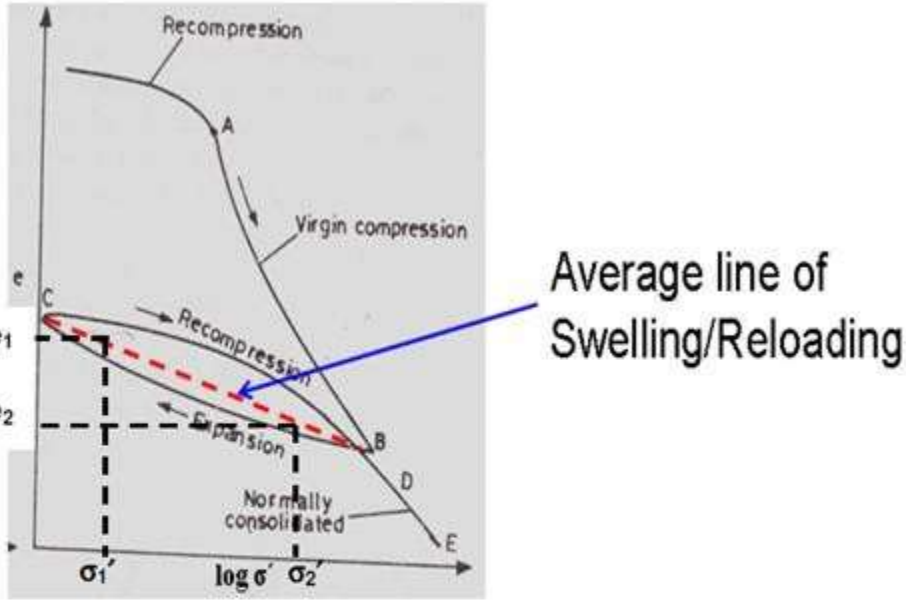


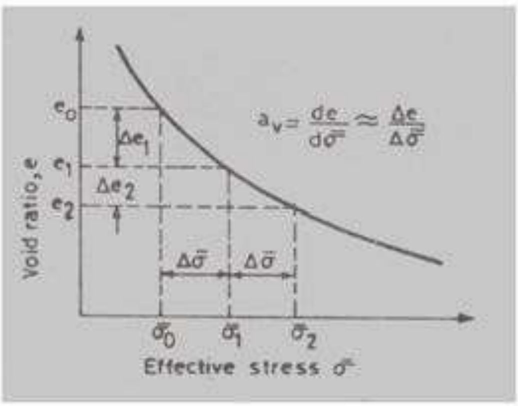
Figure:e–logσv’plot

SwellingIndex(C_s)

It is the average slope of the unloading/reloading curves in e – logσ´ plot given by

$$C_s = \frac{e_1 - e_2}{\log 10 \frac{\sigma_2'}{\sigma_1'}}$$

Co-efficient of compressibility (a_v)



Void ratio versus effective stress plot

It is the slope of the void ratio versus effective stress for a given stress increase $\Delta\sigma'$ in void ratio versus effective stress plot as shown

$$a_v = \frac{\Delta e}{\Delta \sigma'} = \frac{e_1 - e_2}{\sigma_2' - \sigma_0'}$$

a_v decrease with increase in effective stress

Co-efficient of volume compressibility (m_v)

It is the ratio of change in volume of a soil per unit initial volume due to unit increase in effective stress and is given by

$$m_v = \frac{\Delta e}{(1+e_0)} \frac{1}{\Delta \sigma'}$$

Δe = Change in void ratio
e_0 = Initial void ratio
$\Delta\sigma'$ = increase in effective stress

PreconsolidationPressure

Itisthemaximumeffectivestressexperiencedbyasoilinitsstresshistory(pastexistence)

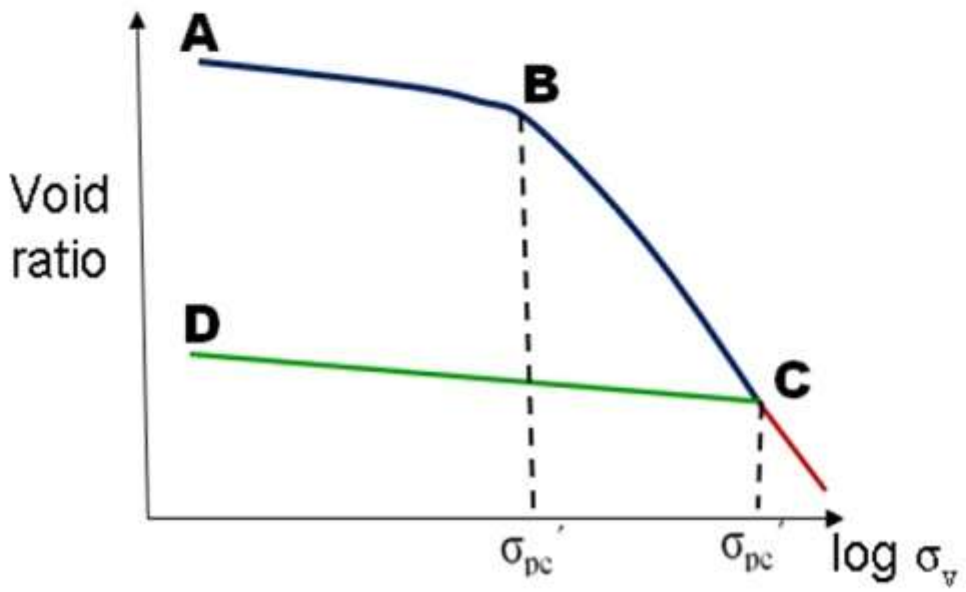


Figure:Voidratioversuseffectivestress(logscale)

ForthesoilloadedalongtherecompressioncurveABtheeffectivestressclosetopointBwillbethe preconsolidation pressure.

If the soil is compressed along BC and unloaded along CD and then reloaded along DC the effective stress close to point C will be the new preconsolidation pressure.

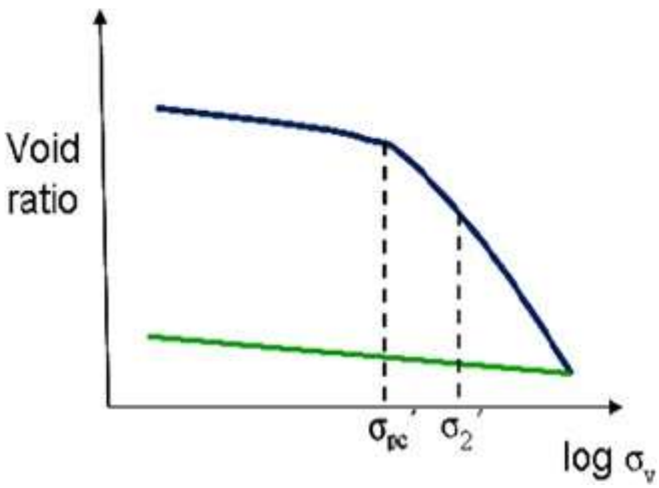
EffectofStressHistory

Basedonthestresshistory(preconsolidationpressure)soilsareclassifiedas

1. NormallyConsolidatedSoils
2. Over ConsolidatedSoils
3. Under ConsolidatedSoils

NormallyConsolidatedSoils

Itisasoildepositthathasneversubjectedtoaverticaleffectivestressgreaterthanthepresentverticalstress.



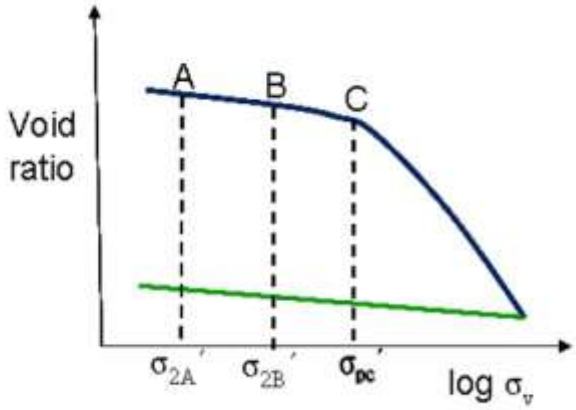
Voidratioversuseffectivestress (logscale)

UnderConsolidatedSoils

A soil deposit that has not consolidated under the present overburden pressure (effective stress) is called Under Consolidated Soil. These soils are susceptible to larger deformation and cause distress in buildings built on these deposits.

OverConsolidatedSoils

Itisasoildepositthathasbeensubjectedtoverticaleffectivestressgreaterthanthepresentverticaleffectivestress.



Void ratio versus effective stress (log scale)

The stress state σ_{2A}' and σ_{2B}' represent over consolidated soil (well with in preconsolidation pressure) Over consolidated soil deposits are less compressible and therefore structures built on these soils undergo less settlement.

7.SHEARSTRENGTHOFSOIL

NecessityofstudyingShearStrengthofsoils :

• Soilfailureusuallyoccursintheformof“shearing”alonginternalsurfacewithinthe

soil.

ShearStrength:

• Thus,structuralstrengthisprimarilyafunctionofshearstrength.

• Thestrengthofamaterialisthegreateststressitcansustain

• Thesafetyofanygeotechnicalstructureisdependentonthestrengthofthesoil

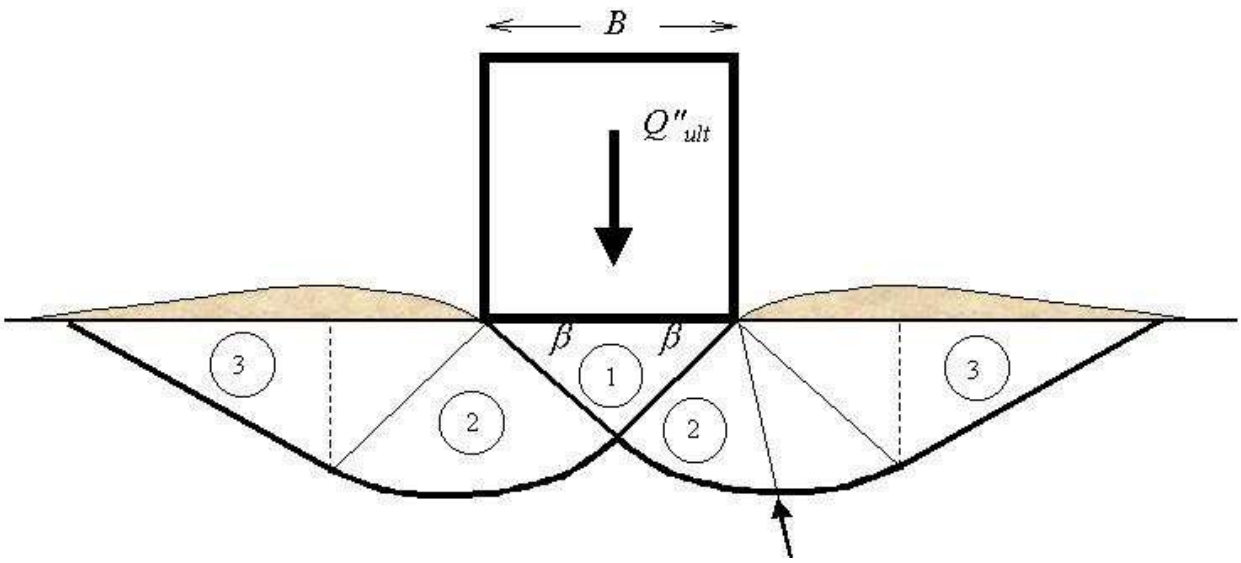
• Ifthesoilfails,thestructurefoundedonitcancollapse

Thusshearstrengthis

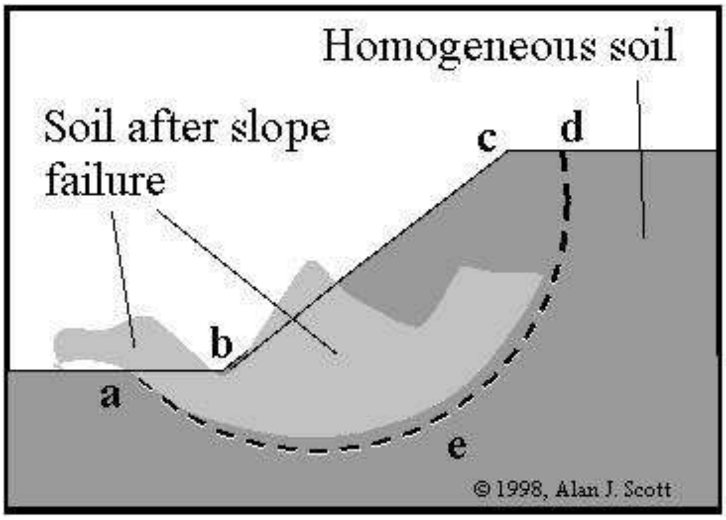
“Thecapacityofamaterialtoresisttheinternalandexternalforceswhichslide past each other”

SignificanceofShearStrength:

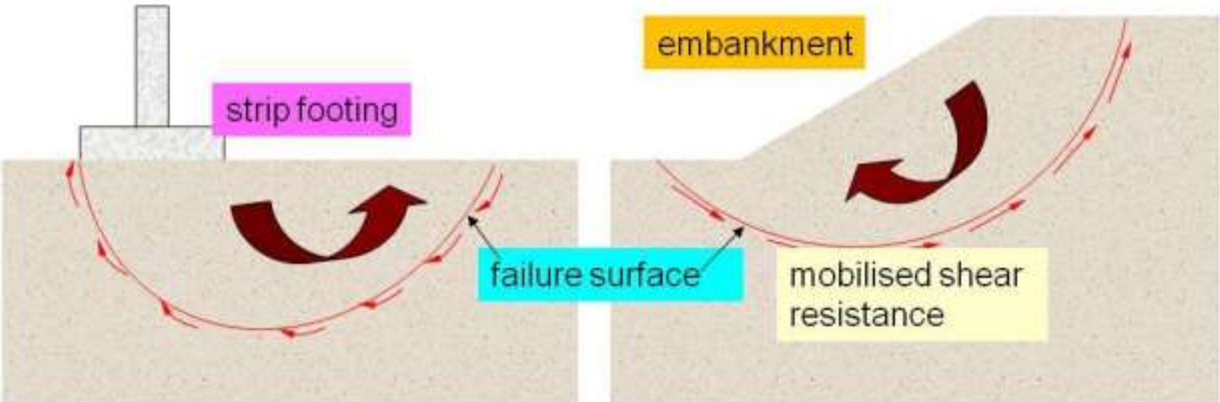
- Engineersmustunderstandthenatureofshearingresistanceinordertoanalyzesoilstabilityproblemssuchas;
- Bearingcapacity
- Slopestability
- Lateralearthpressureonearth-retainingstructure



ShearFailureunderFoundationLoad



SlopeStabilityFailureasanExampleofShearingAlongInternalSurface



Atfailure,shearstressalongthefailuresurfacereachestheshear

Thusshearstrengthofsoilis

"Thecapacityofasoiltoresisttheinternalandexternalforceswhichslidepasteach other"

ShearStrengthinSoils :

- Theshearstrengthofasoilisitsresistancetoshearingstresses.
- Itisameasureofthesoilresistancetodeformationbycontinuousdisplacement of its individual soil particles.
- Shearstrengthinsoilsdependsprimarilyoninteractionsbetweenparticles.
-Shearfailureoccurswhenthestressesbetweentheparticlesaresuchthatthey slide or roll past each other

Componentsofshearstrengthofsoils

Soilderivesitsshearstrengthfromtwosources:

Cohesionbetweenparticles(stressindependentcomponent)

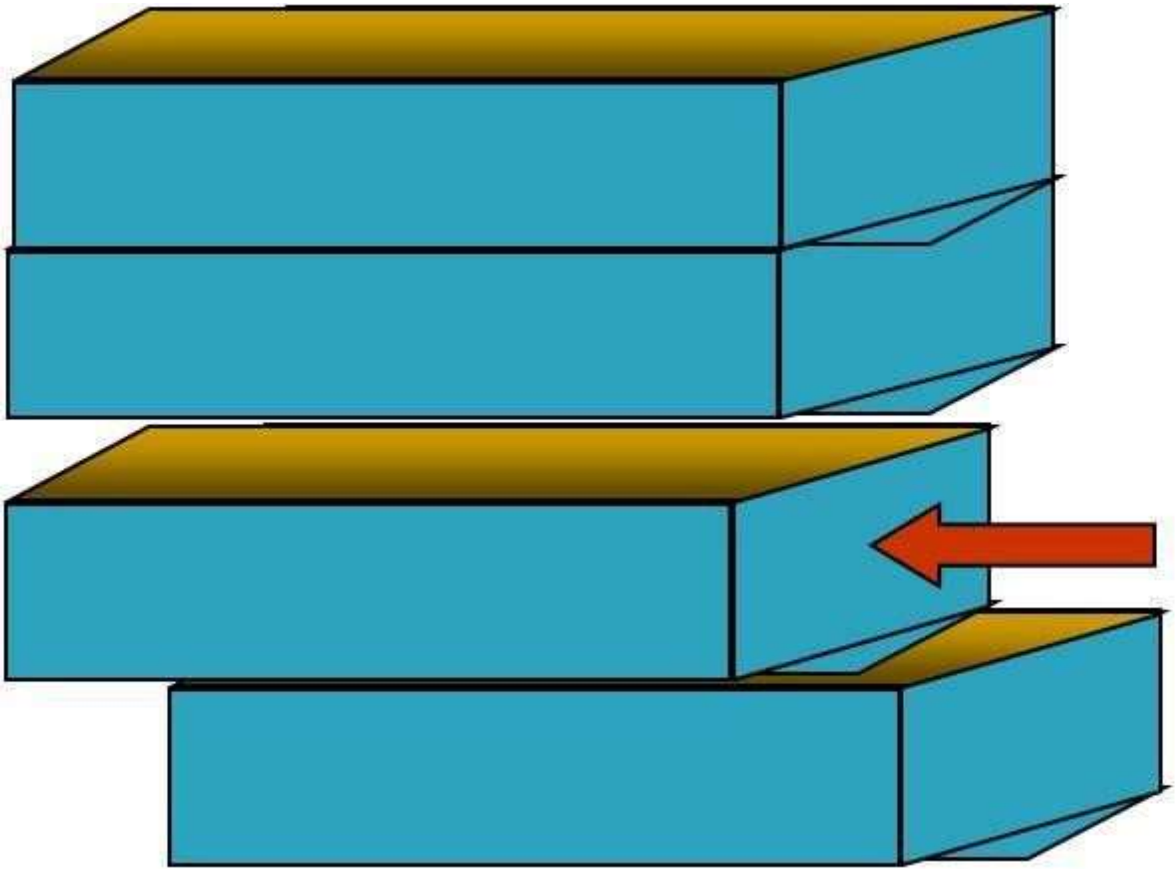
• Cementationbetweensandgrains

• Electrostaticattractionbetweenclayparticles

Frictionalresistanceandinterlockingbetweenparticles(stressdependent component)

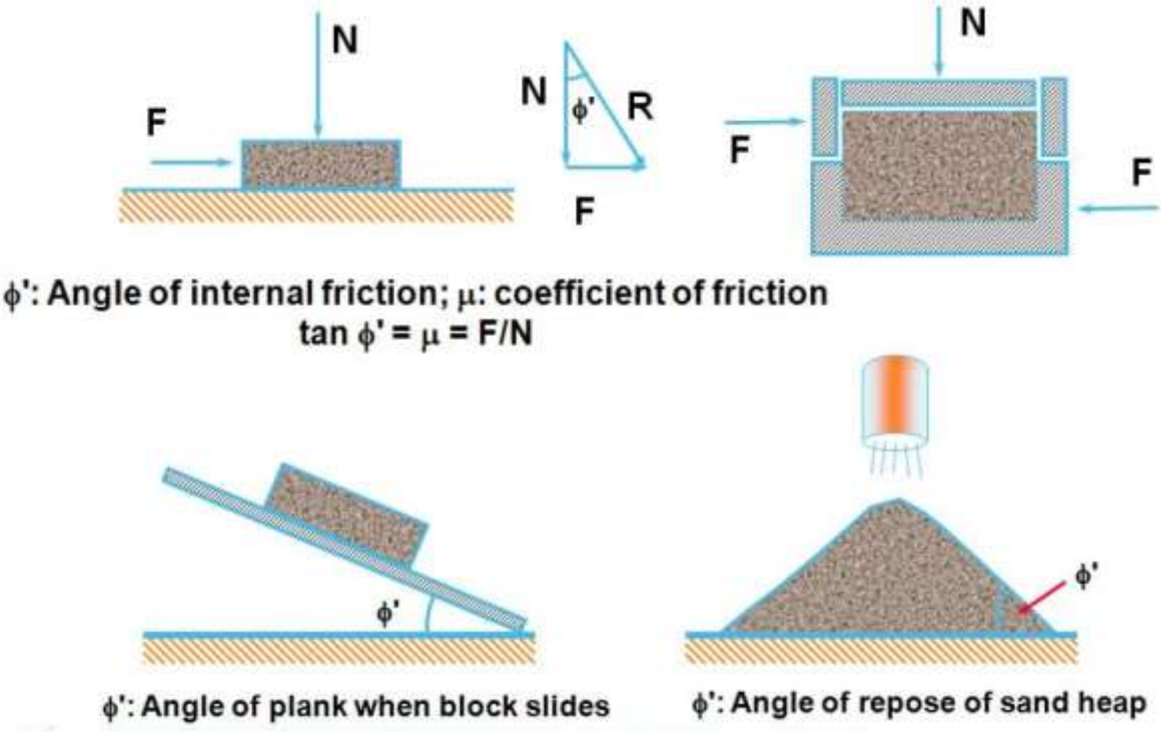
Cohesion:

Cohesion(C),isameasureoftheforcesthatcementparticlesofsoils



InternalFriction:

InternalFrictionangle(*f*),isthemeasureoftheshearstrengthofsoilsdueto



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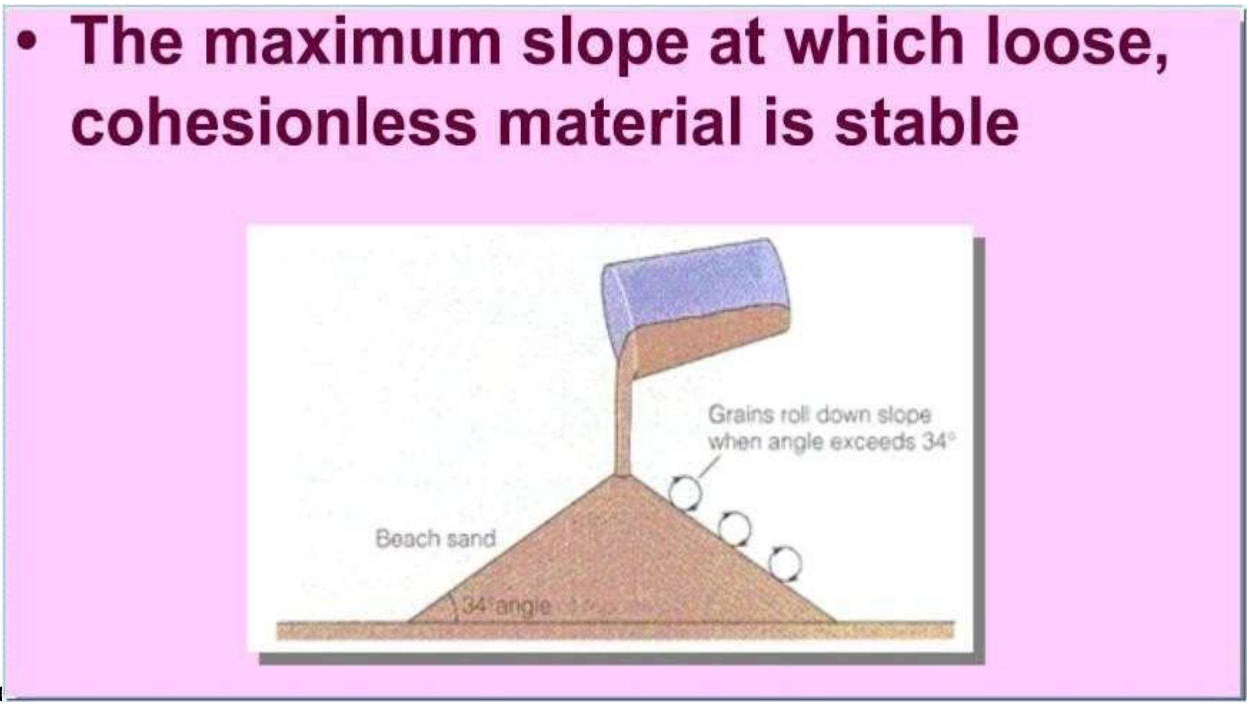


AngleofRepose

AngleofReposedeterminedby:

Particle size (higher for large particles)

Particleshape(higherforangularshapes)

Shearstrength(higherforhighershearstrength)

Stresses:

Gravitygeneratesstresses(forceperunitarea)inthegroundatdifferentpoints.Stress on a plane at a given point is viewed in terms of two components:

Normalstress(σ):actsnormaltotheplaneandtendstocompresssoilgrains towards each other (volume change)

Shearstress(τ):actstangentialtotheplaneandtendstoslidegrainsrelativetoeachother(distortionandultimately sliding failure).

FactorsInfluencingShearStrength:

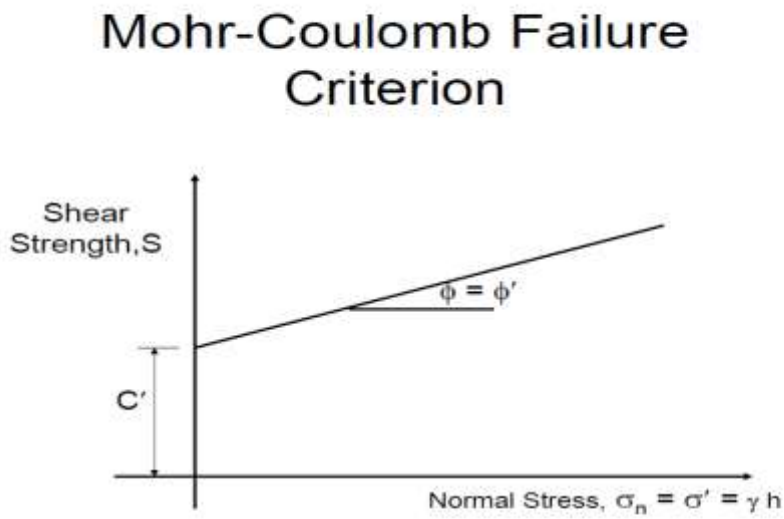
Theshearingstrength,isaffectedby:

soilcomposition:mineralogy,grainsizeandgrainsizedistribution,shapeof particles, pore fluid type and content, ions on grain and in pore fluid.

Initialstate:Statecanbedescribebytermssuchas:loose,dense,overconsolidated, normally consolidated, stiff, soft, etc.

Structure:Referstothearrangementofparticleswithinthesoilmass;the manner in which the particles are packed or distributed. Features such as layers, voids, pockets, cementation, etc, are part of the structure.

Mohr-CoulombFailureCriteria:



$$\tau_f = c + \sigma_n \tan\phi = c + \mu\sigma_n \qquad (11.2)$$

$$\tau_f = c' + \sigma_n' \tan\phi' = c' + \mu'\sigma_n' \qquad (11.3)$$

where

τ_f = shear strength

c = cohesion; c′ =effective cohesion

φ = angle of internal friction; φ′ = effective angle of internal friction

μ = coefficient of friction; μ' = effective coefficient of friction.

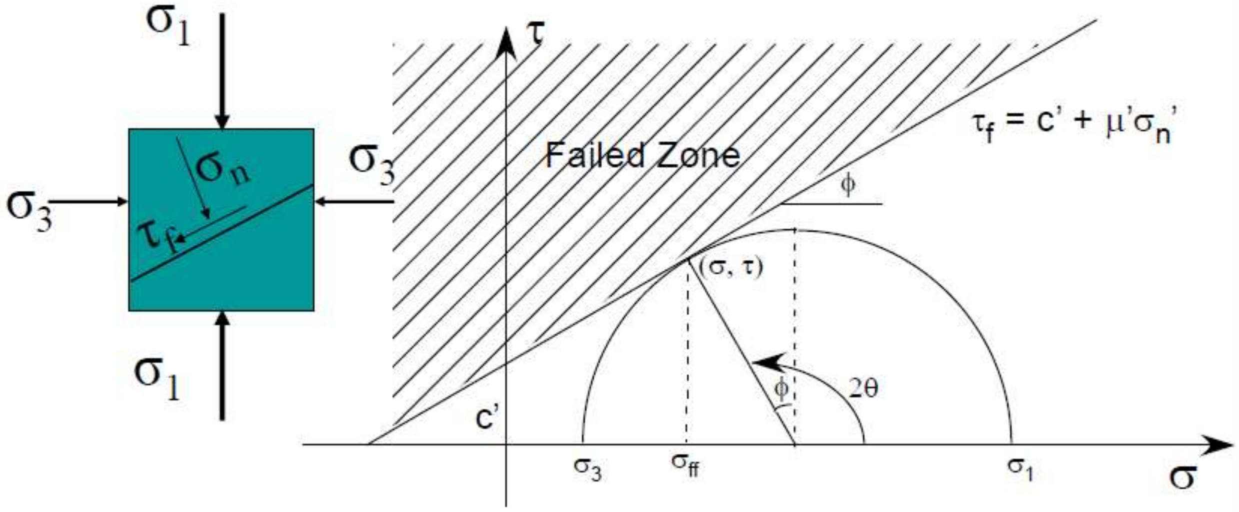
Thus, Eqs. (11.2) and (11.3) are expressions of shear strength based on tota stress and effective stress. The value of c' for sand and inorganic silt is 0. For nor mally consolidated clays, c' can be approximated at 0. Overconsolidated clays hav values of c' that are greater than 0. The angle of friction, ϕ', is sometimes referred t as the *drained angle of friction*. Typical values of ϕ' for some granular soils are give in Table 11.1

Thistheorystatesthatamaterialfailsbecauseofacriticalcombinationofnormalstressandshearstress,andnot from their either maximum normal or shear stress alone.

Table Typical Values of Drained Angle of Friction for Sands and Silts

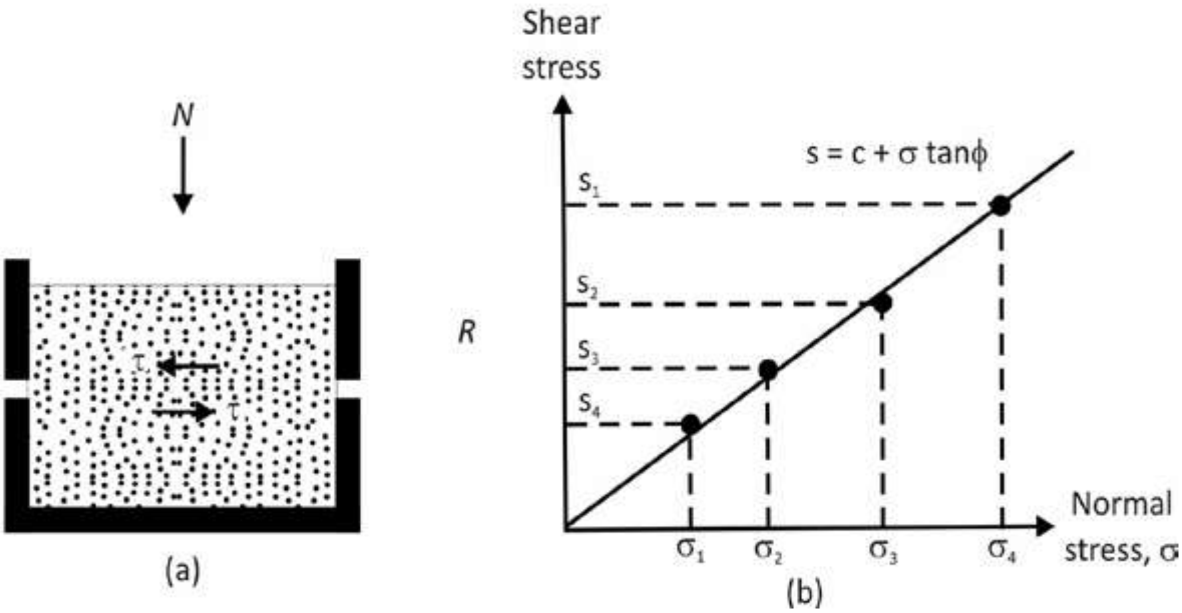
Soil type	ϕ' (deg)	$\mu=\tan\phi'$
Sand: Rounded grains		
Loose	27–30	0.51-0.58
Medium	30–35	0.58-0.70
Dense	35–38	0.70-0.78
Sand: Angular grains		
Loose	30–35	0.58-0.70
Medium	35–40	0.70-0.84
Dense	40–45	0.84-1.00
Gravel with some sand	34–48	0.67-1.11
Silts	26–35	0.49-0.70

Mohr-Coulomb shear failure criterion



DirectShearTest:

Drysandcanbeconvenientlytestedbydirectsheartests.Thesandisplacedinashearboxthatissplitintotwohalves. A normal load is first applied to the specimen. Then a shear force is applied to the top half of the shear box to cause failure in the sand. The normal and shear stresses at failure are



Direct shear test in sand: (a) schematic diagram of test equipment; (b) plot of test results to obtain thefriction angle, ϕ

$$\sigma' = \frac{N}{A}$$

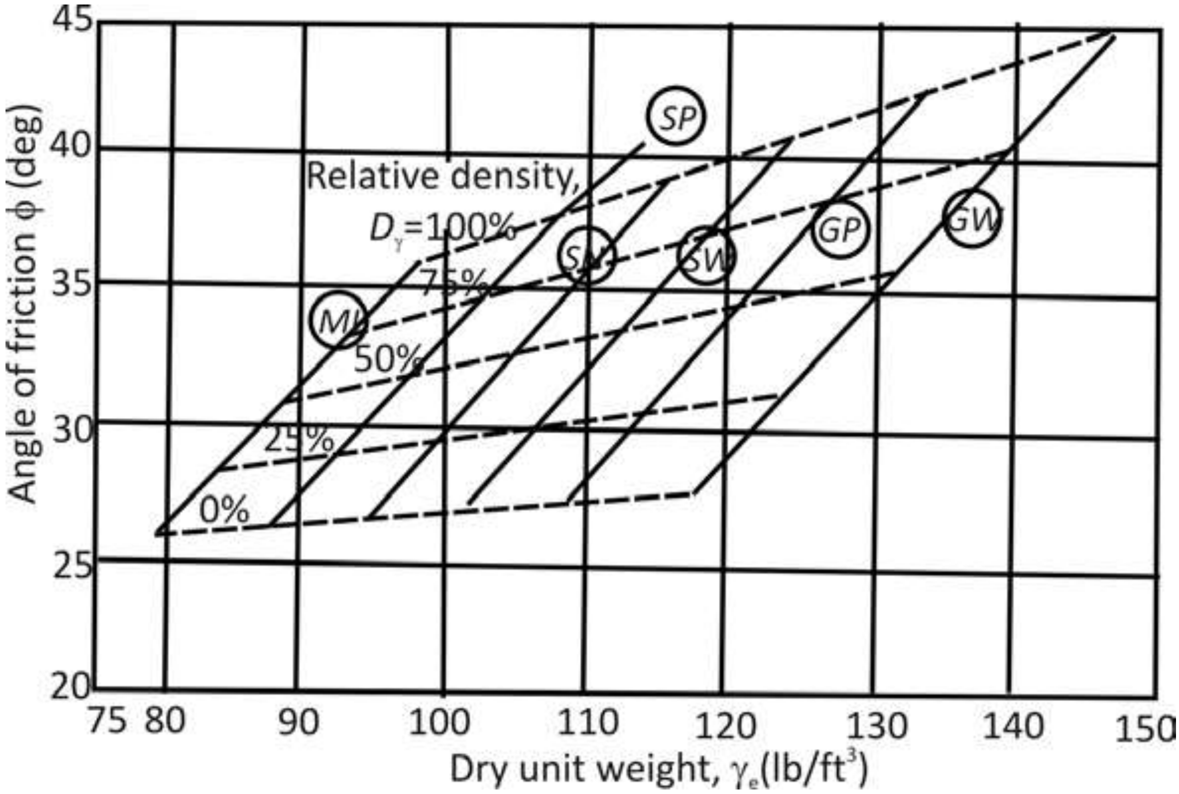
$$S = \frac{R}{A}$$

Where
A = Area of the failure plane in soil-that is, the area of cross section of the shear box

Several tests of this type can be conducted by varying the normal load. The angle of friction of the sand can be determined by plotting a graph of s against σ' ($= \sigma$)

$$\phi = \tan^{-1}\left(\frac{s}{\sigma'}\right)$$

For sands, the angle of friction usually ranges from 26° to 45°, increasing with the relative density of compaction. The approximate range of the relative density of compaction and the corresponding range of the angle of friction for various coarse-grained soils is shown in **figure** ....



Rangeofrelativedensityandcorrespondingrangeofangleoffrictionfor coarse-grainedsoil

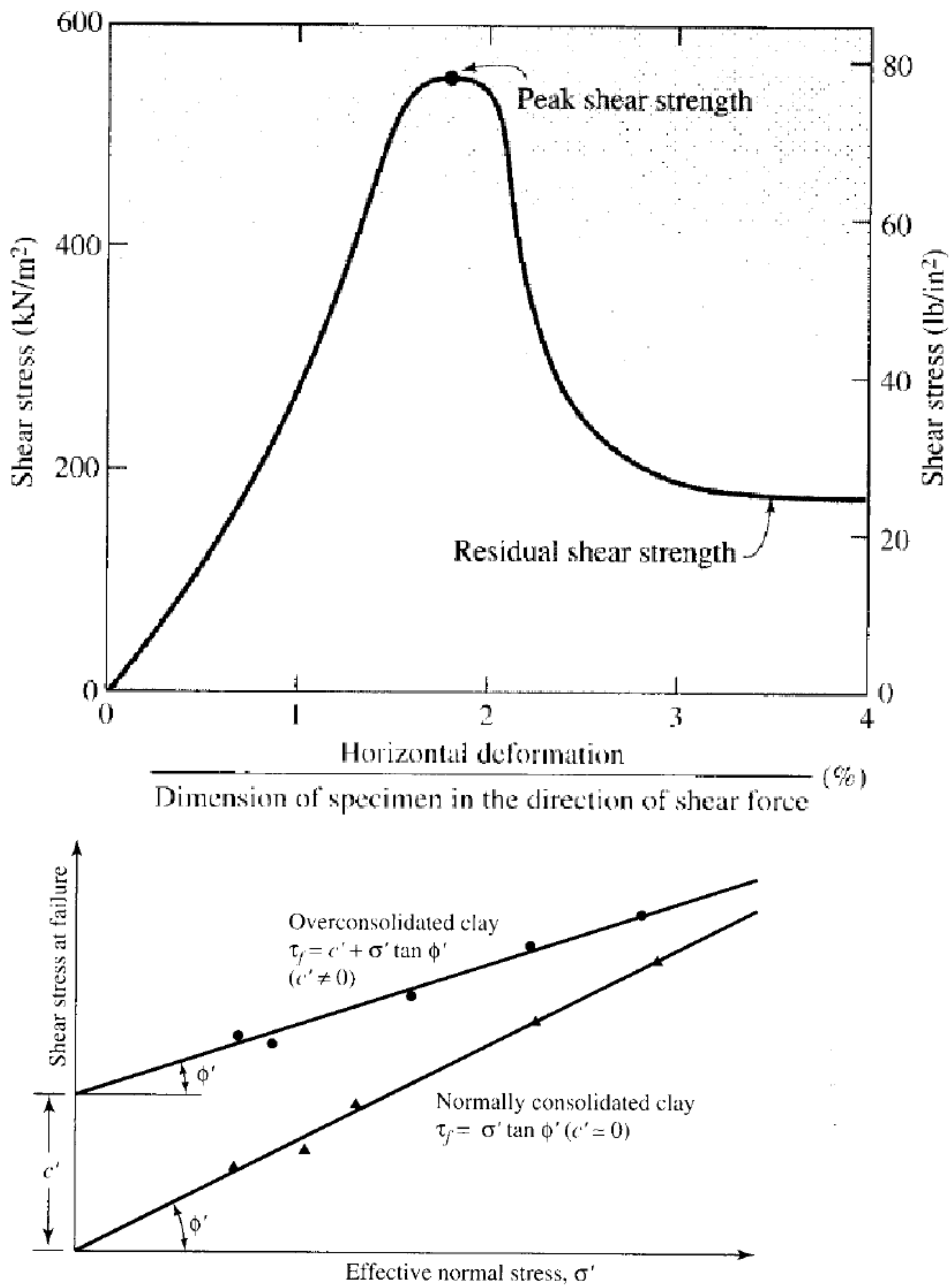


Figure 11.9 Failure envelope for clay obtained from drained direct shear tests

TriaxialTests

TriaxialcompressiontestscanbeconductedonsandsandclaysshowsaschematicdiagramoftheTriaxialtest arrangement. Essentially, it consists of placing a soil specimen confined by a rubber membrane in a Lucite chamber. An all-round confining pressure ($\sigma 3$) is applied to the specimen by means of the chamber fluid (generally water or glycerin). An added stress ($\Delta\sigma$) can also be applied to thespecimen in the axialdirection to cause failure($\Delta\sigma=\Delta\sigma f$ at failure). Drainage fromthespecimencanbe allowedor stopped,depending onthe testcondition.For clays,thre main types of tests can be conducted with Triaxial equipment:

Triaxialtest:

1. Consolidated-drainedtest(CDtest)
2. Consolidated-undrianedtest(CUtest)
3. Unconsolidated-undrained test (UU

test)MajorPrincipaleffectivestress=$\sigma 3=\Delta\sigma f=\sigma$

$1=\sigma' 1$ Minor Principal effective stress

$=\sigma 3=\Delta\sigma' 3$

Changing$\sigma 3$allowsseveraltestsofthistypetobeconductedonvariousclayspecimens.Theshearstrength parameters (c and ϕ) can now be determined by plotting Mohr's circle at failure, as shown in figure and drawing commontangenttotheMohr'scircles.ThisistheMohr-Coulombfailureenvelope.(Note:Fornormally consolidated clay, c≈0). At failure

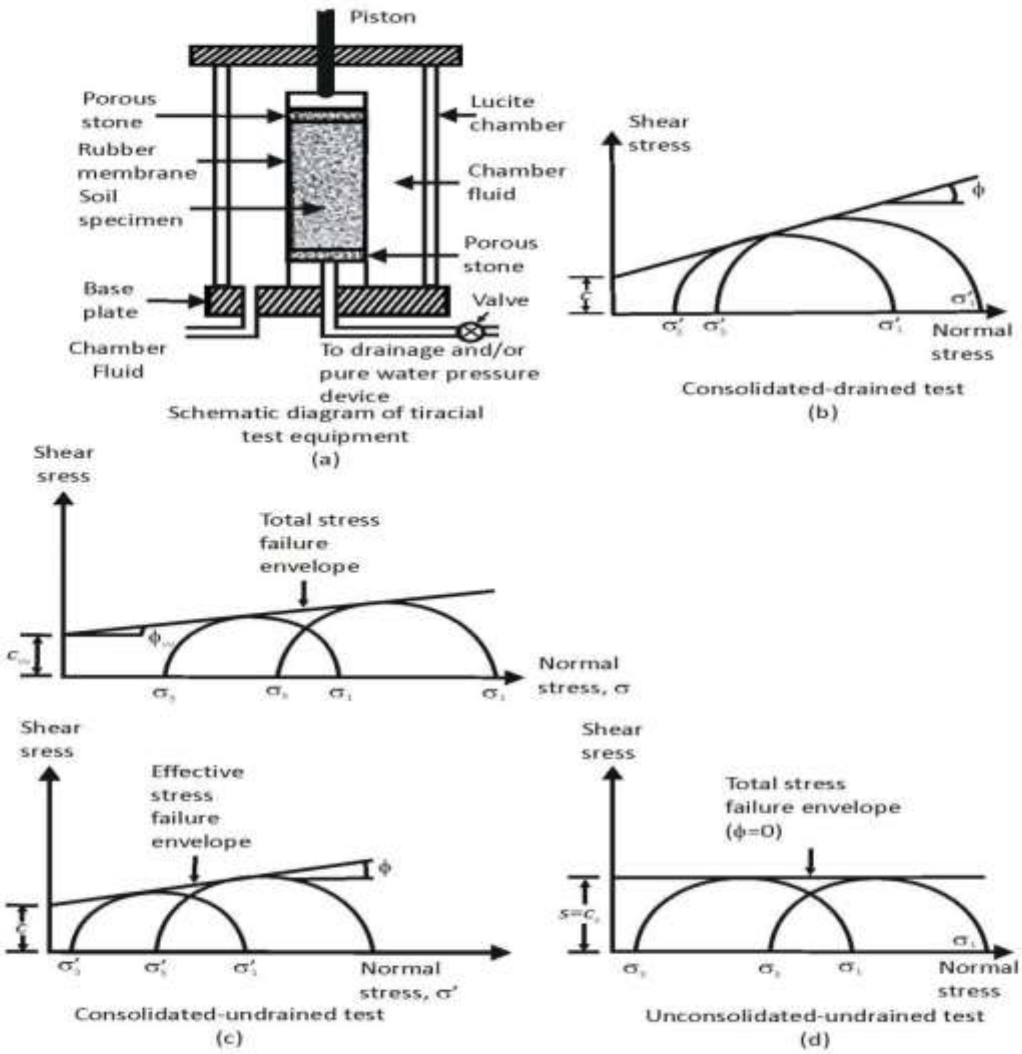
$$\sigma'_1 = \sigma'_3 \tan^2\left(45 + \frac{\phi}{2}\right) + 2c\tan\left(45 + \frac{\phi}{2}\right)$$

Forconsolidated-undrainedtests,atfailure,

Major Principal total stress $=\sigma 3=\Delta\sigma f=\sigma 1$

Minor principal total stress $=\sigma 3$

Majorprincipaleffectivestress$=(\sigma 3+\Delta\sigma f)-uf=\sigma' 1$



Minorprincipaleffectivestress$=\sigma 3-uf=\sigma' 3$

Changing$\sigma 3$ permits multiple tests of this type to be conducted on several soil specimens. The total stress Mohr's circles at failure can now be plotted, as shown infigure, and then a common tangent can be drawn to define the failure envelope. This total stress failure envelope is defined by the equation

$s=ccu+\sigma \tan\phi cu$

Whereccuandϕcuaretheconsolidated-undrainedcohesionandangleoffrictionrespectively(Note:ccu$\approx$0for normallyconsolidatedclays)

Similarly, effective stress Mohr's circles at failure can be drawn to determine the effective stress failure envelopes They follow the relation expressed in equation .

Forunconsolidated-undrainedtriaxialtests

Major principal total stress$=\sigma 3=\Delta\sigma f=\sigma 1$

Minor principal total stress $=\sigma 3$

The total stress Mohr's circle at failure can now be drawn, as shown in figure. For saturated clays, the value of $\sigma1-\sigma3=\Delta\sigma f$is a constant, irrespectiveofthechamberconfiningpressure,$\sigma3$.Thetangent to theseMohr's circles will be a horizontal line, called the $\phi=0$ condition. The shear stress for this condition is

$$s = c_u = \frac{\Delta\sigma_f}{2}$$

Where

c_u = undrained cohesion (or undrained shear strength)

The pore pressure developed in the soil specimen during the unconsolidated-undrained triaxial test is

$$u = u_a + u_d$$

The pore pressure u_a is the contribution of the hydrostatic chamber pressure, σ_3. Hence

$$u_a = B\sigma_3$$

Where

B=Skempton′sporepressureparameter

Similarly,theporepressureudistheresultofaddedaxialstress,$\Delta\sigma$,so ud=A

$\Delta\sigma$

Where

A=Skempton′sporepressureparameter

However,

$\Delta\sigma=\sigma1-\sigma3$

Combiningequationsgive

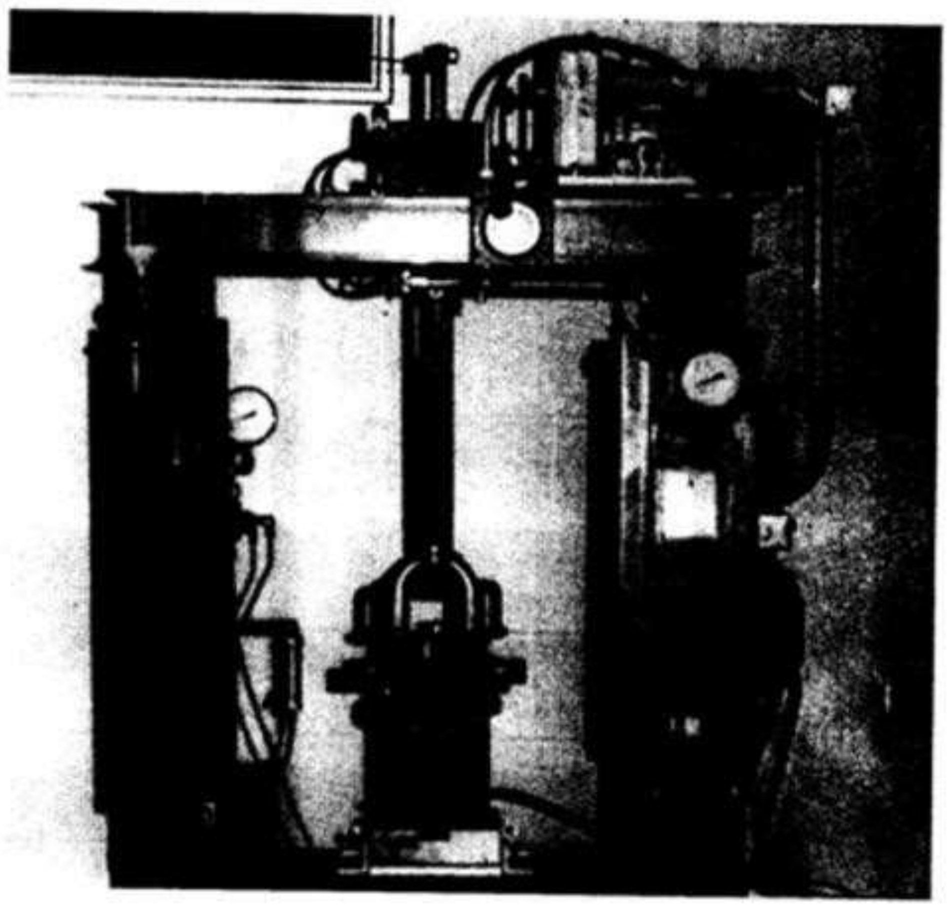
s

$u=ua+ud=B\sigma3+A\sigma1-\sigma3$

TheporewaterpressureparameterBinsoftsaturatedsoilsis1,so

$u=\sigma3+A(\sigma1-\sigma3)$

The valueoftheporewater pressureparameterAatfailure willvary withthetype of soil.Following isageneral range of the values of A at failure for various types of clayey soil encountered in nature.

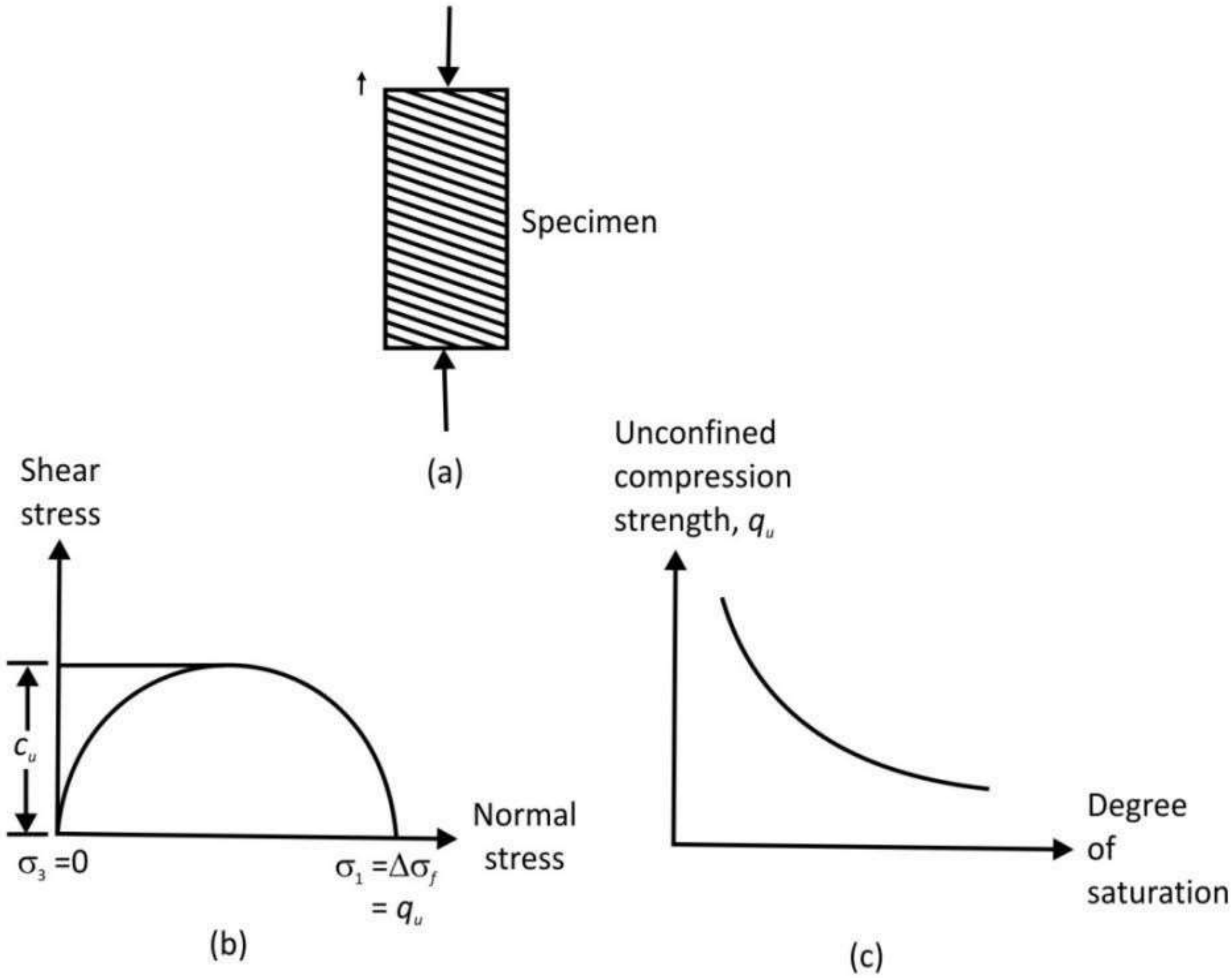


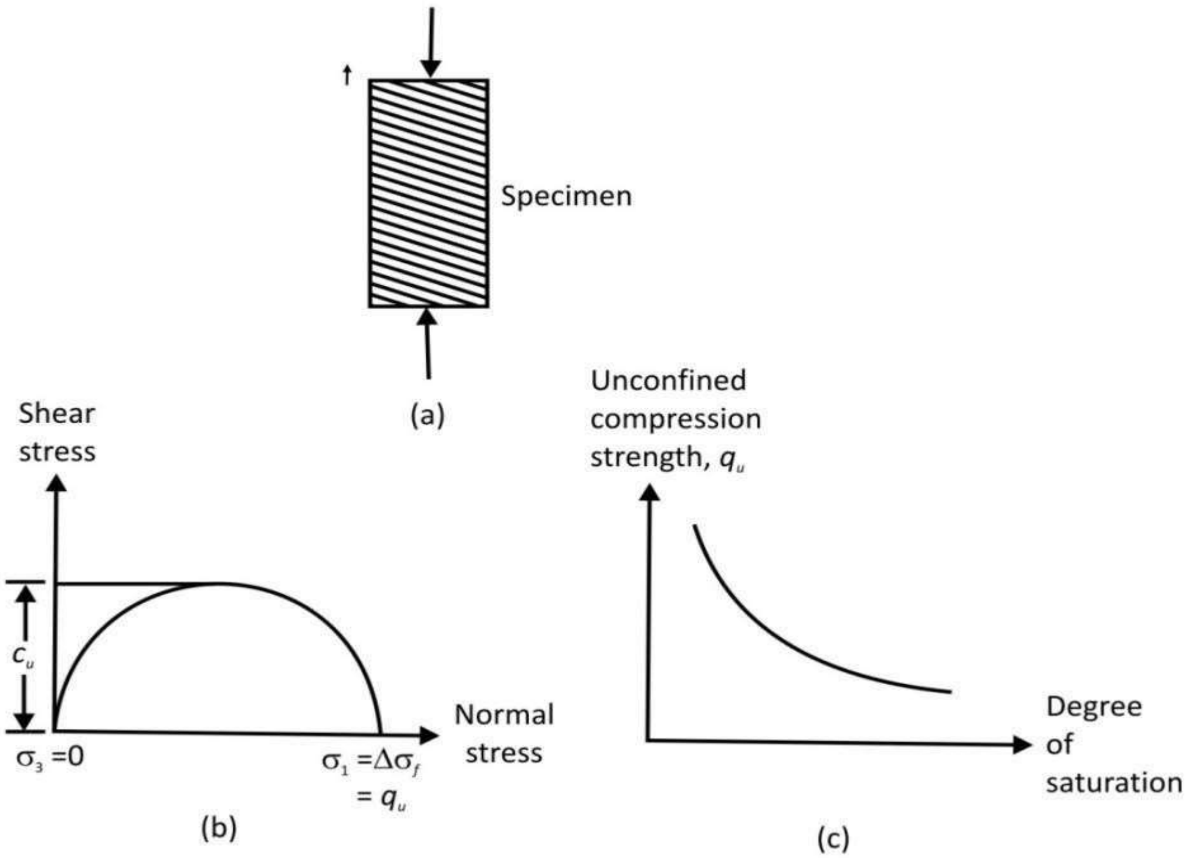
Triaxialtestequipment

UNCONFINEDCOMPRESSIONTEST

Theunconfined compression test is a special type of unconsolidated-undrainedTriaxial test in which the confining pressure$\sigma_3=0$,asshowninfigure.Inthistestanaxialstress,$\Delta\sigma$,isappliedtothespecimentocausefailure(thatis, $\Delta\sigma=\Delta\sigma_f$).

The corresponding Mohr's circle is shown in figure . Note that, for this case,u





Unconfined compressiontest: (a) soilspecimen;(b) Mohr’s circle forthetest;(c)variationof qu withthedegreeof saturation

Majorprincipaltotalstress=Δσf=qu

Minorprincipaltotalstress=0

Theaxialstressatfailure,Δσf=quisgenerallyreferredtoastheunconfinedcompressionstrength.Theshear strengthofsaturatedclaysunderthiscondition(ϕ=0,

$$s = c_u = \frac{q_u}{2}$$

The unconfined compression strength can be used as an indicator for the consistency of clays. Unconfined compression tests are sometimes conducted on unsaturated soils. With the void ratio of a soil specimen remaining constant, the unconfined compression strength rapidly decreases with the degree of saturationshows an unconfined

VaneShear Test:

Fairlyreliable results for the undrained shear strength, c,, (S : 0 concept), of verysoft to medium cohesive soils may be obtained directly from vane shear tests. The shear vane usually consists of four thin, equal-sized steel plates weldedto asteel torquerod. First, thevane is pushed into thesoil. Then torqueis applied at thetop ofthetorquerod torotatethevaneat auniformspeed. Acylinder ofsoilofheight ftand diameterr/ willresistthetorqueuntilthesoil fails. The undrained shear strength of the soil can be calculated as follows. If Iis the maximum torque applied at the head of thetorque rodto cause failure, it should beequal to thesum ofthe resistingmoment of theshear forcealong the side surface of the soil cylinder (M.) and the resisting moment of the shear force at each end (M,,)

$$T = M_s + \underbrace{M_e + M_e}_{\text{Two ends}}$$

The resisting moment can be given as

$$M_s = \underbrace{(\pi d h) c_u}_{\substack{\text{Surface}\\ \text{area}}} \quad \underbrace{(d/2)}_{\substack{\text{Moment}\\ \text{arm}}}$$

whered:diameteroftheshearvanc/z:heightofthcshearvane

For the calculation of M., investigator shavea everal t ypeso f distributicln of shear strength mobilization at thc ends of the soil cylinder:

l. Triangular. Shear strength mobilization is c,, at the periphcryof the soil cylinder and decreases lineaarlyto zero at the center.

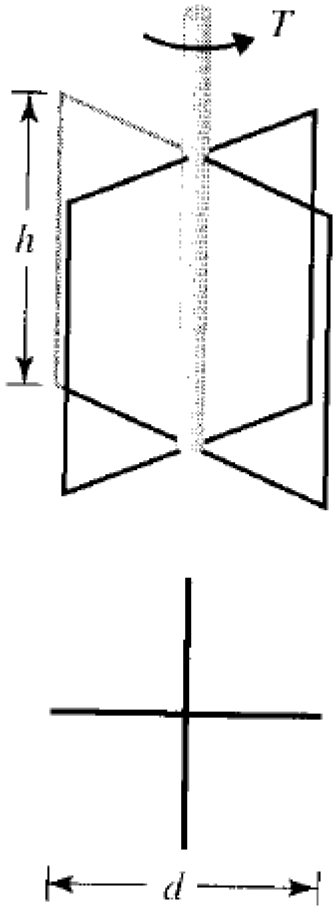
2,IJni.form.Shearstrengthmobilizationisconstant(thatis,c)fromtheperipherytothecenterofthesoilcylinder.

3. Parabolic.Shearstrengthmobilizationisc,,attheperiphery ofthesoilcylinderanddccreasesparabolically t zero at the center.

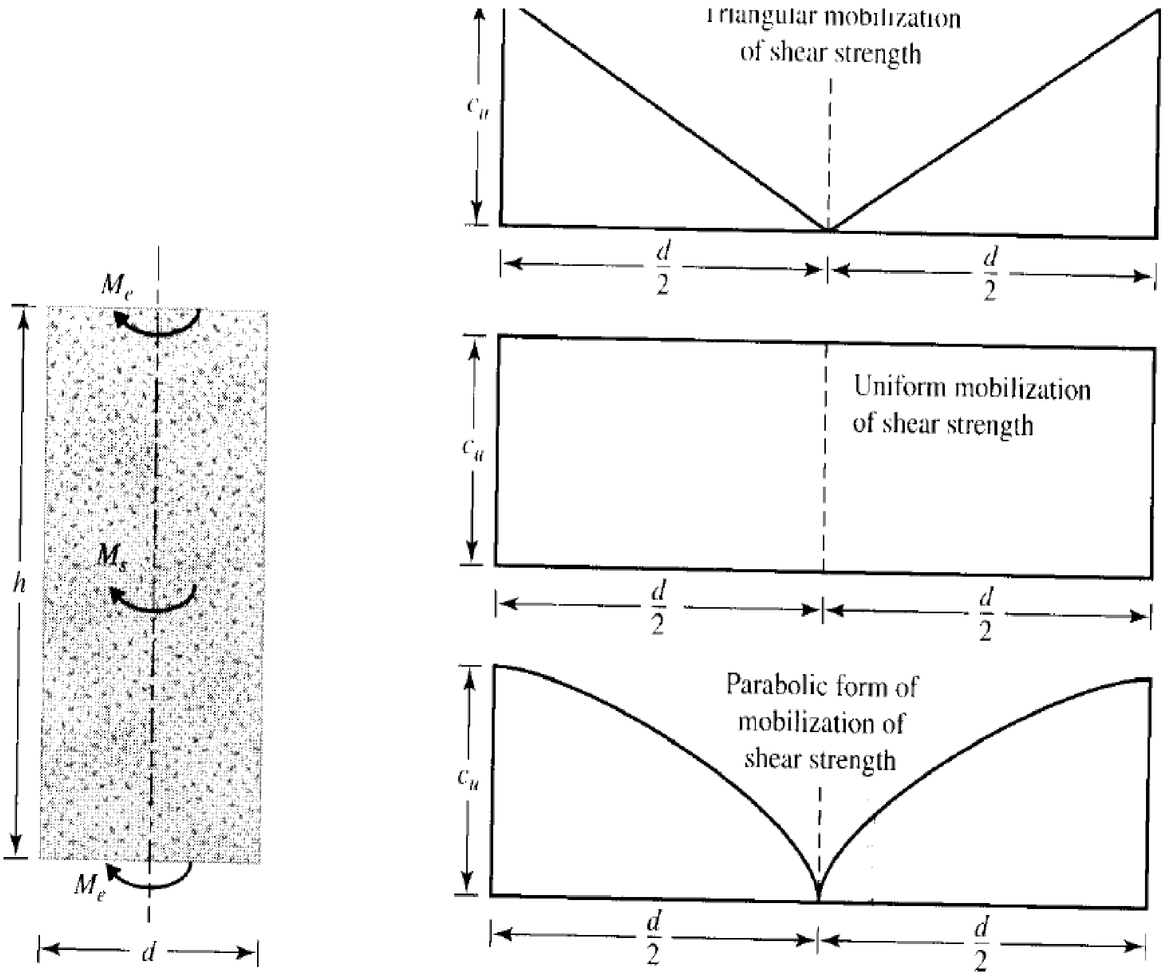
Thesevariationsinshearstrengthmobilizationareshownin Figure.Ingeneral,thetorque,Iatfailurecanbe expressed as

$$T = \pi c_u \left[\frac{d^2 h}{2} + \beta \frac{d^3}{4} \right]$$

$$c_u = \frac{T}{\pi \left[\frac{d^2 h}{2} + \beta \frac{d^3}{4} \right]}$$



I
Diagram of vane shear test equipment



(a) resistingmomentofshearforce;(b)variationsinshearstrengthmobilization

$$c_u\,(\mathrm{kN/m^2}) = \frac{T\,(\mathrm{N\cdot m})}{(366 \times 10^{-8})\underset{\substack{\uparrow\\(\mathrm{cm})}}{d^3}}$$

$$c_u\,(\mathrm{lb/ft^2}) = \frac{T\,(\mathrm{lb\cdot ft})}{0.0021\underset{\substack{\uparrow\\(\mathrm{in.})}}{d^3}}$$

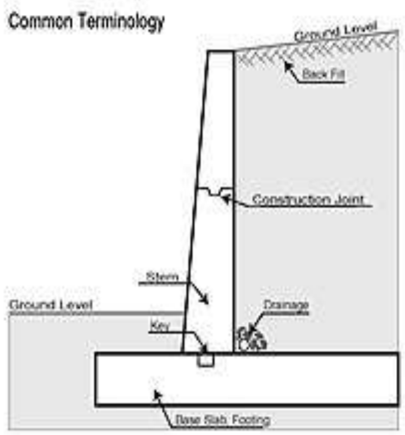
EARTH PRESSURE ON RETAINING STRUCTURES

In 1929 Terzaghi (The Father of Soil Mechanics) conducted experiments on the retaining wall and showed the relation of pressure on the wall if wall changes its position i.e to move inwards to the backfill, outwards of it or remain at its place. There are three types of earth pressures on the basis of the movement of the wall.

1. Earth Pressure at rest
2. Active Earth Pressure
3. Passive Earth Pressure

These are explained below

Pressure at rest: When the wall is at rest and the material is in its natural state then the pressure applied by material is known as Earth Pressure at Rest. It is represented by Po. Active earth pressure: When the wall moves away from the backfill, there is a decrease in the pressure on the wall and this decrease continues until a minimum value is reach after which their is no reduction in the pressure and the value will become constant. This kind of pressure is known as active earth pressure



Active earth pressure:

When the wall moves away from the backfill, there is a decrease in the pressure on the wall and this decrease continues until a minimum value is reach after which their is no reduction in the pressure and the value will become constant. This kind of pressure is known as active earth pressure.

Passive earth pressure:

When the wall moves towards the backfill, there is an increase in the pressure on the wall and this increase continues until a maximum value is reach after which their is no increase in the pressure and the value will become constant. This kind of pressure is known as passive earth pressure. This means that when the wall is about to slip due to lateral thrust from the backfill, a resistive force is applied by the soil in front of the wall.

Active Earth Pressure

For a level backfill (β =0), the following equation is used to determine the active earth pressure (pa) for all types of soils. When the retaining wall is moving away from the backfill the the ratio between lateral earth pressure and vertical earth pressure is called co-efficient of active earth pressure.

Ka = lateral pressure / vertical pressure

Passive Earth Pressure

For a level backfill (β =0), the following equation is used to determine the passive earth pressure for all types of soils. When the retaining wall is moving towards the backfill, then the ratio between the lateral earth pressure and the vertical earth pressure is called the Co-efficient of passive earth pressure.

Kp =lateral pressure / vertical pressure

Surcharge:

The material which lie above the horizontal level of the retaining structure is known as surcharge. The angle which this material makes with the retaining wall is called surcharge angle.

Assumptions made by Rankine for the derivation of earth pressure

- The soil mass is homogeneous and semi-infinite.
- The soil is dry and cohesionless.
- The ground surface is plane, which may be horizontal or inclined.
- The back of the retaining wall is smooth and vertical.
- The soil element is in a state of plastic equilibrium, i.e., at the verge of failure

ACTIVE EARTH PRESSURE:

1.Backfill with no surcharge Let us consider an element of dry soil at a depth Z below a level soil surface. Initially , the element is at rest conditions, and the horizontal pressure is given by ' h = k0 'v Where 'v is the vertical stress at C , and ' h is the horizontal stress at C. Of course, 'v = ;Z. The stresses ' h and 'v are respectively minor and major principal stresses, and are indicated by points A and B in the Mohr circle in fig 2.b . Let us now consider the case when the vertical stress remains constant while the horizontal stress is decreased. The point A shifts to position A' and the diameter of the Mohr circle increases. In the limiting condition , the point A shifts to the position A" when the Mohr circle [marked (3)] touches the failure envelope. The soil is at the verge of shear failure. It has attained the Rankine's active state of plastic equilibrium. The horizontal stress at that state is the active pressure(Pa). Fig.2.c shows the Mohr circle when active conditions are developed. Poi

CHAPTER-9
FOUNDATION ENGINEERING
9.1 FOUNDATION

It is the bottom most part of the structure that remains in direct contact with soil and transmits the load of the structure underneath. Functions of the foundation- In addition to transmitting the load of the superstructure to the soil it provides stability to the structure against overturning and erosion.

SHALLOW FOUNDATIONS

Types of Foundations may be broadly classified under two heads: shallow foundations and deep foundation is shallow if itsTerzaghi, a foundation. According to Terzaghi, a foundation is shallow if its depth is equal to or less than its width. In the case of deep foundation, the depth is equal to or greater than the width. Apart from deep strip, rectangular or square foundations, other common forms of deep foundations are: pier foundation, pier foundation are and well foundation. The shallow foundations are of the following types : spread footing (or simply, footing), strap footing, combined footing, and mat or raft footing

SPREAD FOOTING

A spread footing or simply footing is a type of shallow foundation used to transmit the load of an isolated column, or that of a wall to the subsoil. This is most common type of foundation. The base of the column or wall is enlarged or spread to provide individual support for the load.

Bearing Capacity:

Definitions:
1. Footing: A footing is a portion of the foundation of a structure that transmits loads directly to the soil.
2. Foundation: A foundation is that part of the structure which is in direct contact with and transmit loads to the ground.
3. Foundation soil : It is the upper part of the earth mass carrying the load of the structure.
4. Bearing capacity: The supporting power of a soil or rock is referred to as its bearing capacity. The term bearing capacity is defined after attaching certain qualifying prefixes.
5. Gross pressure intensity (q) : The gross pressure intensity q is the total pressure at the base of the footing due to the weight of the superstructure, self-weight of the footing and the weight of the earth fill, if any.
6. Net pressure intensity (qn). It is defined as the excess pressure, or the difference in intensities of the gross pressure after the construction of the structure and the original overburden pressure.
is the average unit weight of soil above the foundation base.
7. Ultimate bearing capacity
The ultimate bearing capacity is defined as the minimum gross pressure intensity at the base of the foundation at which the soil fails in shear. (When the term bearing capacity is used in this book without any prefix, it may be understood to be ultimate bearing capacity).
8. Net ultimate bearing capacity. It is the minimum net pressure intensity causing shear failure of soil. The ultimate bearing capacity ¥S and the net ultimate capacity are evidently connected by the following relation
9. Effective surcharge at the base level of foundation (°) . It is the intensity of vertical pressure at the vase level of foundation, computed assuming total unit weight for the portion of the soil above the water table and submerged unit weight for the portion below the water table.
10. Net safe bearing capacity (¯±). The net safe bearing capacity is the net ultimate bearing capacity divided by a factor of safety
11. Safe bearing capacity (±). The maximum pressure which the soil can carry safely without risk of shear failure is called the safe bearing capacity. It is equal to the net safe bearing capacity plus original overburden pressure