SCTE&VT | State Council for Technical Education & Vocational Training
Government of Odisha

LECTURE NOTES

ON

GEOTECNICAL ENGINEERING

TH.2

Diploma 3rd Semestar

DEPARTMENT OF CIVIL ENGINEERING

SRI Polytechnic , Komanda

CHAPTER-1

INTRODUCTION

1.0 INTRODUCTIONOFSOIL

The word “Soil” is derived from the Latin word solium which, according toWebster’s dictionary, means the upper layer of the earth that may be dug or plowedspecifically, the loose surface material of the earth in which plants grow. The abovedefinition ofsoil is used in the field of agronomywherethe main concern is in the useofsoilforraisingcrops.

In geology, earth’s crust is assumed to consist of unconsolidated sediments,called mantle or regolith, overlying rocks. The term ‘soil’ is used for the upper layerofmantlewhichcansupportplants.Thematerialwhichiscalledsoilbytheagronomist or the geologist is known as top soil in geotechnical engineering or soilengineering. The top soil contains a large quantity of organic matter and is notsuitableas a construction material or as a foundation for structures. The top soil is removedfromtheearth’ssurfacebeforetheconstructionofstructures.

Theterm‘soil’inSoilEngineeringisdefinedasanunconsolidatedmaterial,composed or solid particles, produced by the disintegration of rocks. The void spacebetween the particles may contain air, water or both the solid particles may containorganic matter. The soil particles can be separated by such mechanical means asagitationinwater.

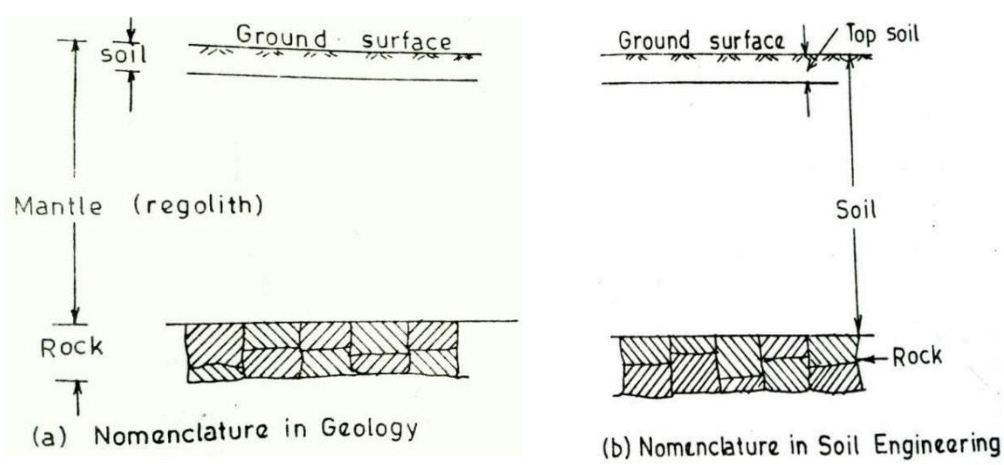


Fig.showsacross-sectionthroughtheearth’ssurface,indicatingthenomenclatureusedingeology,andinSoilEngineering.

1.1 SOILANDSOILENGINEERING:-

Theterm'soilmechanics' wascoinedbyDr. KarlTerzaghiin1925whenhisbookErdbaumechaniconthesubjectwaspublishedinGerman.

AccordingtoTerzaghi,'Soilmechanicsistheapplicationofthelawsofmechanics and hydraulics to engineering problems dealing with sediments and otherunconsolidatedaccumulationsofsolidparticlesproducedbythemechanicalandchemicaldisintegrationofrock,regardingofwhetherornottheycontainanadmixtureoforganicconstituents.

Soil mechanics is, a branch of mechanics which dealswith the action of forcesonsoilandwiththeflowof waterinsoil.

The soil consists of discrete solid particles which are neither strongly bonded as insolids nor they are as free as particles of fluids. Consequently, the behavior of soil issomewhat intermediate between that of a solid and a fluid. It is not, therefore,surprisingthatsoilmechanicsdrawsheavilyfromsolidmechanicsandfluidmechanics. As the soil is inherently a particulate system. Soil mechanics is alsocalledparticulatemechanics.

DEFINITION OF SOIL ENGINEERING AND GEOTECHNICALENGINEERING:-

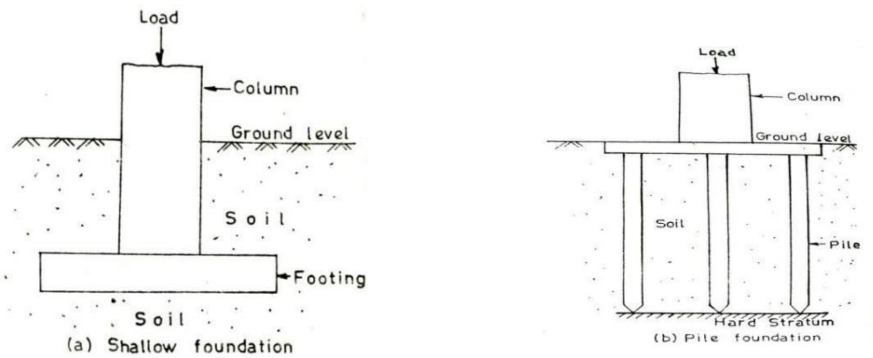
SoilEngineeringinanappliedsciencedealingwiththeapplicationsofprinciples of soil mechanics to practical problems. It has a much wider scope thansoil mechanics, as it details with all engineering problems related with soils. Itincludes site investigations, design and construction of foundations, earth-retainingstructuresand earthstructures.

1.2 SCOPEOFSOILMECHANICS:-

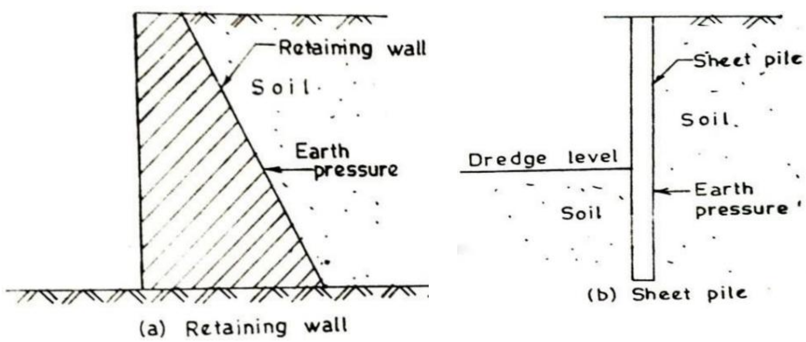
SoilEngineeringhasvastapplicationintheconstructionofvariousCivilEngineeringworks.Someoftheimportantapplicationsareasunder.

(1) **Foundations:–**Everycivilengineeringstructure,whether itisabuilding,a bridge, or a dam, is founded on or below the surface of the earth.Foundationsarerequiredtotransmittheloadofthestructuretosoilsafelyandefficiently.

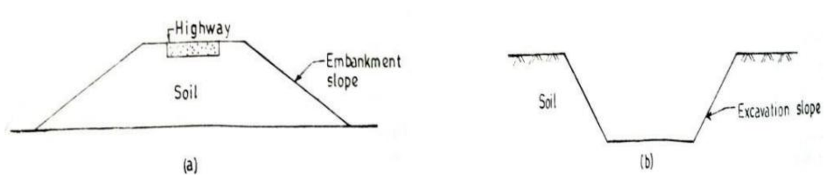
Afoundationistermedshallowfoundationwhenittransmittedtheloadtoupper strataofearth. A foundation is calleddeep foundation when the loadis transmitted to strata at considerable depth below the ground surface. Pilefoundation is a type of deep foundation. Foundation engineering is animportantbranch ofsoilengineering.



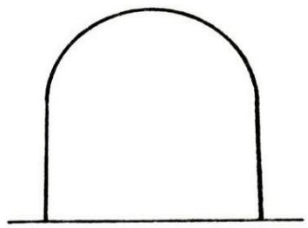
(2) **RetainingStructures:**–Whensufficientspaceisnotavailableforamassof soil to spread and form a safe slope, a structure is required to retain thesoil. An earth retaining structureis also required to keep thesoil atdifferent levels on its either side. The retaining structure may be rigidretaining wall or a sheet pile bulkhead which is relatively flexible (Fig.1.3). Soil engineering gives the theories of earth pressure on retainingstructures.



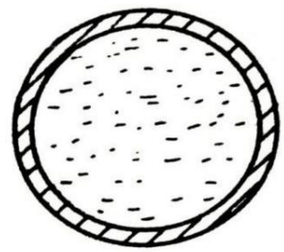
(3) **Stability of Slopes:** – If soil surface is not horizontal, thereis a componentof weight of the soil which tends to move it downward and thus causesinstability of slope. The slopes may be natural or man-made Fig. 1.4 showsslopesinfillingandcutting.Soil engineeringprovidesthemethodsforcheckingthestabilityofslopes.



(4) **Underground structures:** – The design and construction of undergroundstructures, such as tunnels, shafts, and conduits, require evaluation offorces exerted by the soil on these structures. These forces are discussed insoil engineering. Fig. 1.5 shows a tunnel constructed below the groundsurfaceandaconduitlaidbelowthegroundsurface.

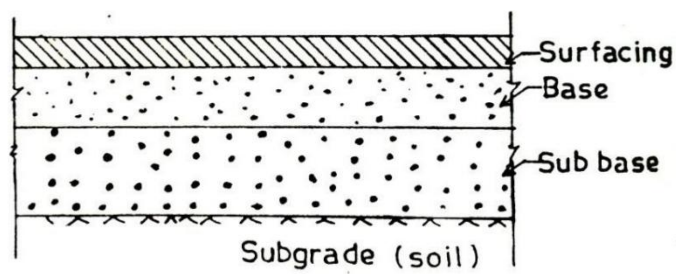


(a) Tunnel

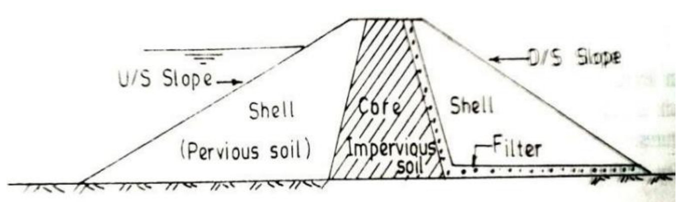


(b) Conduit

(5) **Pavement Design: –** A pavement is hard rust placed on soil (sub grade)for the purpose of proving a smooth and strong surface on which vehiclescan move. The pavement consists of surfacing, such as a bitumen layer,base and subtheme (Fig. 1.6). The behavior of sub grade under viciousconditionsofloadingandenvironmentchangeisstudiedinsoilengineering.



(6) **Earth Dom:** – Earth dams are the structures in which soil is used as aconstructionmaterial.Theearthdamsarebuiltforcreatingwaterreservoirs.Sincethefailureofanearthdammaycausewidespreadcatastrophecareistakeninitsdesignandconstruction.Itrequiresthoroughknowledgeatsoilengineering.



(7) **Miscellaneous soil: -** The geotechnical engineer has sometimes to tacklemiscellaneousproblemsrelatedwithsoil.Suchassoilheave,soilsubsidence,frostheave,shrinkageandswellingofsoils.

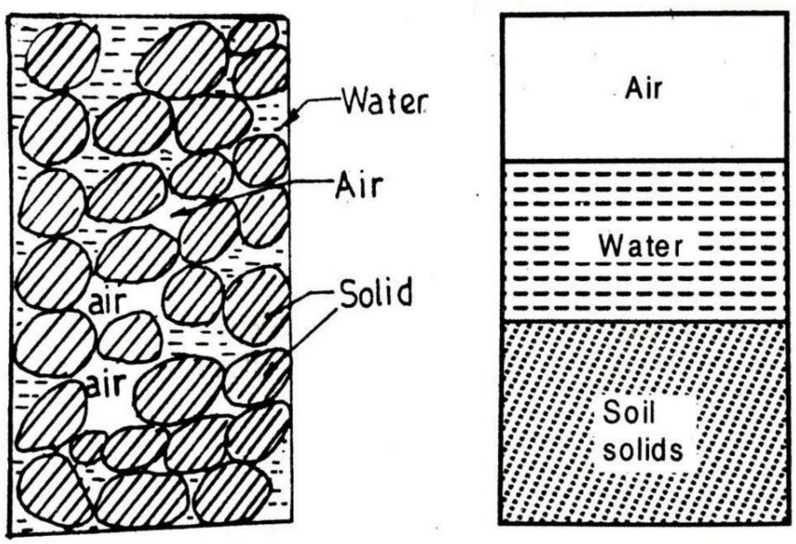
CHAPTER-2

PRELIMINARYDEFINATIONANDRELATIONSHIP

2.1 SOILASATHREEPHASE:

A soil mass consists of solid particles which from a porous structure. Thevoids in the soil mass may be filled with air, with water or partly with air and partlywithwater.In general,asoilmassconsistsofsolidparticles, waterandair.Thethreeconstituentsareblendedtogethertoformacomplexmaterial(Fig.2. 1.a).However,forconvenience,allthesolidparticlesaresegregatedandplacedinthelowerl ayerofthe three-phase diagram (Fig. 2.1b). Likewise, water and air particles are placedseparately,asshown.The3-phasediagramisalsoknownasBlockdiagram.

Itmaybenotedthattheconstituentscannotbe

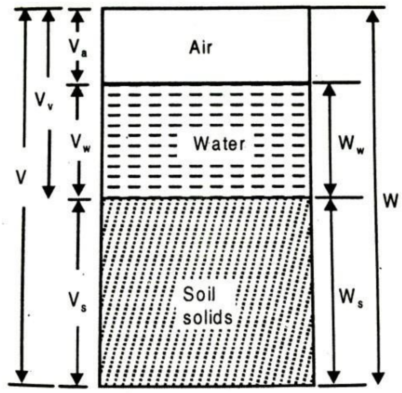


(a)

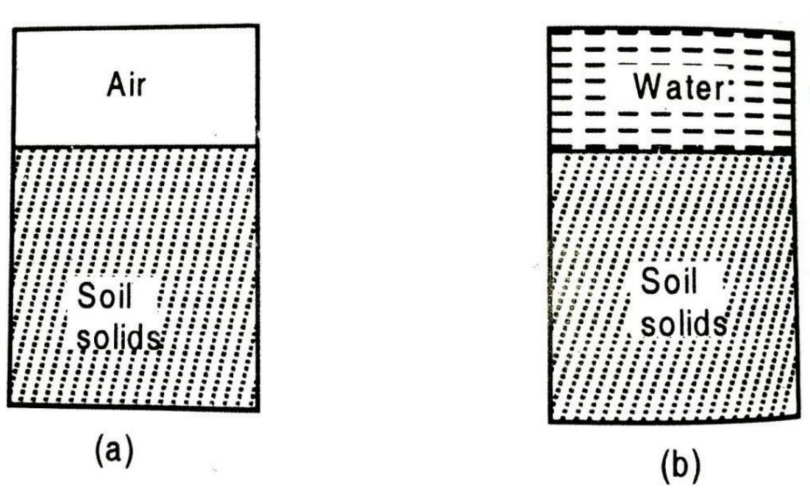
Actually segregated, as shown. A 3-phase diagram is an artifice used for easyunderstandand convenienceincalculation.

Although the soil is a three-phase system, it becomes a two-phase system in thefollowing two cases :(1) When the soil is absolutely dry, the water phase disappears(Fig. 2.2a). (2) When the soil is fully saturated three is no air phase (Fig.2.2b). It is therelative proportion of the three constituents and their interaction that governs thebehavior andproperties of soils. Thephasediagram is asimple, diagrammaticrepresentationofarealsoil,Itisextremelyusefulforstudyingthevariouster msusedinsoilengineeringandtheirinterrelationships.

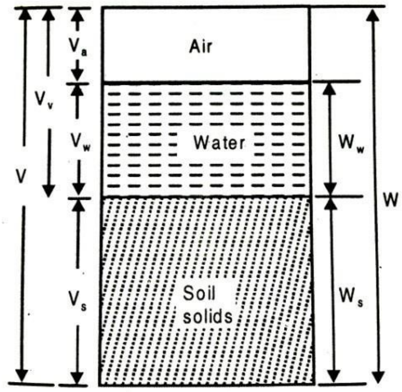
Ina3-
phasediagramitisconventionaltowritevaluesontheleftsideandthemassontherightside(Fig.2.3a).ThemailsimpleofagivessoilmassindesignatedasV.Itisequaltothesumofthevolumeofsolids(V1),thevolumeofwater(F)andthevolumeof air (Va). The volume of voids (Vv) is equal to the sum of the volumes, the waterandair



Thesoilmassofthesoil…isrepresentedasM.themassofair(ma)isverysmallandisneglected.Therefore,thesocialmassofthesoilisequaltothemassofsolids(M2)and the mass of water (Mw). Fig.2.36 shows the 3-phase diagram in which the weightsarewritten ontherightside.



2.2 WEIGHTVOLUMERELATIONSHIPS:



WATERCONTENT:-Thewatercontent(w)isdefinedastheratioofthemasswatertothe massof solids.

$$w = \frac{Ww}{Ws}$$

Thewatercontentisalsoknownasthemoisturecontent(m).Itisexpressedasapercentage,butwheneverusedinequationusedasadecimal.

The water content of the fine-grained soils, such as silts and clays, in generallymorethanthatofthecoarsegrainedsoils,suchasgravelsandsands.

The water content of some of the fine-gained soils may be even more than100%, which indicates that more than 50%of the total mass is that of water. The watercontentofasoilis animportantproperty.

SPECIFICGRAVITY(G):-

Thespecificgravityofsolidparticles(G)isdefinedastheratioofthemassofagivenvolumeofsolidstothemassofanequalvolumeofwaterat4^{0}C.Thus,the specificgravityisgivenbyG= $\frac{\rho s}{\rho w}$

*Themassdensityofwaterρwat4^{0}Cisonegm/ml,1000kg/m3or1Mg/m^3.

Thespecificgravityofsolidsformostnaturalsoilsfallsinthegeneralrangeof 2.65 to 2.80, the smaller values are for the coarse-grained soils. Table gives theaverage values of specific gravity for different soils. It may be mentioned that thespecificgravityofdifferentparticlesinasoilmassmaynotbethesame.Whenever the specific gravity of a soil mass is indicated, it is the average value of all the solidparticles presentin thesoilmass. Specificgravity ofsolids is an important parameter.Itisusedfordetermination ofvoidratioandparticlesize.

Table:TypicalValuesofG

Sl.No.	SoilType	SpecificGravity
1	Grevel	2.65-2.68
2	Sand	2.65-2.68
3	Sands	2.66-2.70
4	Slit	2.66-2.70
5	InartisticClays	2.68-2.80
6	OrganicSoils	Variable,mayfallbelow2.00

Besidesthefollowingtwotermsrelatedwiththespecificgravityarealsoused.

(1) **MassSpecificGravity(G_m):-** It isdefinedastheratioofthe massdensityofthesoiltothemassdensityofwater. Thevalueofthemassspecificgravityofasoilismuchsmallerthanthevalueofthespecific gravityofsolids. Themassspecificgravityisalsoknownastheapparentspecificgravityorthebulkspecificgravity.

(2) **AbsoluteSpecificGravity(G_a):-**Thesoilsolidsare notperfectsolidsbutcontain voids. Some of these voids are permeable through which water canenter, whereas others are impermeable. Since the permeable voids get filledwhen the soil is wet, these are in reality a part of void space in the total massand not apart of soil solids. If both the permeable and impermeable voids areexcluded from the volume of solids ,the remaining volume is the true orabsolutevolumeof thesolids.
The mass density of the absolute solids is used for the determination of theabsolute specificgravityofsolids.

VOIDS RATIO: - It is defined as the ratio of the volume of voids to the volume ofsolids.

$$e=\frac{Vv}{Vs}$$

The void ratio is expressed as a decimal, such as 0.4,0.5, etc. For coarse-grained soils, the void ratio is generally smaller than that for fine-grained soils. Forsomesoils,voidratiomayhaveavalueevengreaterthanunity.

POROSITY:-Itisdefinedastheratioofthevolumeofvoidstothetotalvolume.

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

The Porosity is generally expressed as percentage. However, in equations, itis used as a ratio. For example; a porosity of 50% will be used as 0.5 in equations.The porosity of a soil cannot exceed 100% as it would mean Vv is greater than V,whichisabsurd.Porosity isalsoknownaspercentagevoids.

Bothporosityandvoidratioaremeasuresofthedenseness(orlooseness)ofsoils.Asthe soil becomes more and more dense, their values decrease. The term porosity ismore commonly used in other disciplines such as agricultural engineering. In soilengineering, the term void ratio is more popular. It is more convenient to use voidratio than porosity. When the volume of a soil mass changes, only the numerator(i.e.Vv) in the void ratio changes and the denominator (i.e.Vs) remains constant.However, if the term porosity is used, both the numeration and the denominatorchangeanditbecome inconvenient.

Relationshipbetweenthevoidratioandtheporosityasunder.

$$\frac{1}{n} = \frac{V}{Vv} = \frac{Vv+Vs}{Vv}$$

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

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

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

$$e = \frac{n}{1-n} \text{---------(2)}$$

Theporosityshouldbeexpressedasaratio(andnotpercentage).

PERCENTAGEOFAIRVOIDS(n_a):-

Itistheratioofthevolumeofairtothetotalvolume.

$$n_a = \frac{Va}{V}$$

Itisrepresentedasapercentage.

AIRCONTENT(a_c):-

Aircontentisdefinedastheratioofthevolumeofairtot hevolumeofvoids.

$$a_c = \frac{Va}{Vv}$$

TheAircontentisusuallyexpressedasapercentage.Bothaircontentandthepercentageairvoidsarez erowhenthesoilissaturated(V_a=0).

Relationshipbetweenthepercentageairvoidsandtheaircontentcanbeobtained.

$$n_a = \frac{Va}{V} = \frac{Va}{Vv} x \frac{Vv}{V}$$

$$n_a = nxac$$

DegreeofSaturation(S)-

Thedegreeofsaturation(S)istheratioofthevolumeofwatertothevolumeofvoids.Itisalsorepre sentsasS$_r$.

$$S = \frac{Vw}{Vv}$$

Thedegreeofsaturationisgenerallyexpressedasapercentage.

It is equal to zero when the soil is absolutely dry and 100% when the soil isfullysaturated.Degreeofsaturationisusedasadecimalinexpressions.

DENSITYINDEX:-

Itthemostimportantindexpropertyofacohesionlesssoil.

It is also known as Density Index (I_D).It is also known as relative density ordegree of density. It is used to express the relative compactness of a natural soildeposit.

It is the ratio of the difference between voids ratio of the soil in its loosest stateand its natural voids ratio to the difference between voids ratio in the loosest anddensest states.

$$I_D orDr = \frac{emax-e}{emax-emin} x100$$

Wheree_{max}= Maximum void ratio of the soil in the loosest

condition.e_{min}=Minimumvoidratioofthesoilinthedensestconditio

n.

e=voidratioofthesoilinthenaturalstate.

e_{max}will found out from γmin i,e in the loosest

condition.e_{min}willfoundoutfromγmaxi,einthedensestconditi

on.ewillfoundout fromγdi,einthenaturalcondition.

$$Dr = \frac{emax - e}{emax - emin} x100$$

$$Dr = \gamma_d = \frac{\frac{G\gamma_w\gamma_m}{in} - 1}{1 + \frac{7v}{7s}} = \frac{G\gamma_w}{1+e}$$

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

e_{max} Therelativedensityofasoilgivesamoreclearideaofthedensenessthandoesthevoidratio.

BULKUNITWEIGHTY:-

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

Itisalsoknownastotalunitweight(γ_t)orwetunitweight.InSIunitsitisexpressasN/mm^{3}orKN/mm^3

DRYUNITWEIGHT:-

Thedryunitweightisdefinedastheweightofsolidspertotalvolume.

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

SATURATEDUNITWEIGHT:-

Thesaturatedunitweightisthebulkunitweightwhenthesoilisfullysaturated.

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

SUBMERGEDUNITWEIGHT:-

Whenthesoilexistsbelowwaterthanitiscalled submergedcondition.

Thesubmergedunitweight(γ')ofthesoilisdefinedasthesubmergedweightpertotal volume.

$$\gamma_{sub} = \frac{Wsub}{V}$$

RelationbetweenG,S,e,γ&γ_w:-

$$\gamma = \frac{W}{V} = \frac{Ws+Ww}{Vs+Vv} = \frac{GVs\gamma_w + Vw\gamma_w V}{Vs+Vv}$$

DividingVsbothnumerator&denominator

$$\text{Or,}\ \gamma = \frac{G\gamma_w + (7w)\gamma_w}{7s} \quad (G + \frac{7w}{7v} \times \frac{7v}{7s})\gamma_w = (G+S\,e)\gamma_w$$

$$1+\frac{7v}{7s} \quad = \quad 1+\frac{7v}{7s} \qquad 1+\frac{7v}{7s}$$

$$\text{Or}, \gamma = \frac{(G+Se)\gamma w}{1+e} \quad \text{..............................(3)}$$

Asthedegreeofsaturationis100%forasaturatedsoil,thentheS=1Or,γ_{sat}=

$$\frac{(G+e)\gamma w}{1+e} \quad \text{...(4)}$$

$$\text{Similarly,} \quad \gamma_d = \frac{Ws}{V} = \frac{Ws}{Vs+Vv} = \frac{GVs\gamma w}{Vs+Vv}$$

DividingVsbothnumerator&denominator

$$\gamma d = \frac{G\gamma w}{1+\frac{Vv}{Vs}} = \frac{G\gamma w}{1+e} \quad \text{..........(5)}$$

Weknownthat$\gamma_{sub}=\gamma_{sat}-\gamma_w$

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

$$\gamma_{sub} = \frac{(G-1)\gamma w}{1+e} \quad \text{..................................(6)}$$

<u>RelationBetweene,w,G&S:-</u>

$$\text{Weknownthat} w = \frac{Ww}{Ws} \qquad \left(\text{As} \gamma w = \frac{Ww}{Vw} \ \& \gamma s = \frac{Ws}{Vs}\right)$$

$$= \frac{Vw\gamma w}{Vs\gamma s}$$

$$= \frac{Vw}{Vv} X \frac{Vv}{Vs} X \frac{\gamma w}{\gamma s}$$

$$= S.e.\frac{1}{G} \qquad \left(\text{As } S = \frac{Vw}{Vv}, e = \frac{Vv}{Vs} \ \& G = \frac{\gamma s}{\gamma w}\right)$$

$$w.G = S.e$$

$$e = \frac{w.G}{S}$$

<u>Relationbetweene,S&n_a:-</u>

$$n_a = \frac{Va}{V} = \frac{Vv-Vw}{Vv+Vs}$$

DividingVsbothnumerator&denominator

7v _ 7w   7v7w7v – x   e−Se

=7s 7s=7s 7v7s=
7v 7s 7v 7s   e+1
—+—   —+

7s 7s   7s7s

$$n_{a=}\frac{(1-S)e}{1+e} \quad \ldots\ldots\ldots(7)$$

Relation between S & a_c:-

$$a_c = \frac{Va}{Vv} = \frac{Vv - Vw}{Vv} = 1 - \frac{Vw}{Vv} = 1 - S \text{ ....} (8)$$

Relation between Na, a_c & n:-

$$n_a = \frac{Va}{V} = \frac{Va}{Vv} \cdot \frac{Vv}{V} = a_c . n \text{ ....} (9)$$

Relation between G, n_a, e, γ_d & γ_w:-

$$V = Vs + Vv + Va$$

$$1 = \frac{Vs}{V} + \frac{Vv}{V} + \frac{Va}{V} = \frac{Vs}{V} + \frac{Vv}{V} + n_a$$

$$1 - n_a = \frac{Vs}{V} + \frac{Vv}{V} = \frac{Ws/G\gamma w}{V} + \frac{Ww/\gamma w}{V}$$

$$= \frac{\gamma d}{G\gamma w} + \frac{wWs/\gamma w}{V}$$

$$= \frac{\gamma d}{G\gamma w} + \frac{w\gamma d}{\gamma w} = \left(\frac{1}{G} + w\right) . \frac{\gamma d}{\gamma w}$$

$$\gamma d = \frac{(1-na)G\gamma w}{1+wG} \text{ ....} (10)$$

Relation between w, γd & γ:-

$$w = \frac{Ww}{Ws}$$

$$\text{Or, } 1 + w = 1 + \frac{Ww}{Ws} = \frac{Ws + Ww}{Ws}$$

$$\text{Or, } W_s = \frac{W}{1+w}$$

$$\text{Or, } \gamma_{d}.V = \frac{W}{1+w}$$

$$\text{Or, } \gamma_d = \frac{W}{(1+w)V} = \frac{y}{(1+w)}$$

$$\text{Or, } \gamma_d = \frac{y}{(1+w)} \text{ ....} (11)$$

CHAPTER-3

DETERMINATIONOFINDEXPROPERTIES

WATERCONTENTDETERMINATION:

- Thewatercontentofasoil isan importantparameterthatcontrolsitsbehaviour.
- Itisaquantitativemeasureofthewetnessofasoilmass.
- Thewatercontentofsoilmasscanbedeterminedbythefollowingmethods:

1. Ovendryingmethod
2. Pycnometermethod

(1) Ovendryingmethod:

- Theovendryingmethodisastandardlaboratorymethodandthisisaveryaccuratemethod.
- Inthismethodthesoilsampleistakeninasmall,non-corrodible,airtightcontainer.
- The mass of the sample and that of the container are obtained using anaccurateweighing balance.
- Thesoilsampleinthecontaineristhendriedinanovenatatemperatureof110°c ±5°cfor24hours.
- Thewatercontentofthesoilsampleisthencalculatedfromthefollowingequation:

$W = M_w/M_s$

$$= \frac{M_2 - M_3}{M_3 - M_1} \times 100$$

WhereM_1=massofcontainerwithlid

M_2=massofcontainer,lidandwetsoilM_3=massofcontainer,lidanddrysoil

(2) Pycnometermethod:

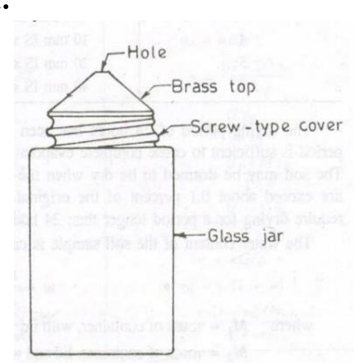


Figure3.1Pycnometer

- A pycnometer is a glass jar of about 1 litre capacity and fitted with a brassconicalcapbymeansofscrewtype cover.
- The cap has a small hole of 6 mm diameter at its apex. A rubber or fibre washerisplacedbetweenthecapandthejartopreventleakage.
- Thereisamarkonthecapandalsoonthejar.Thecapisscreweddowntothesame mark such that the volume of the pycnometer used in the calculationsremainsconstant.
- The pycnometer method for the determination of water content can be usedonlyifthespecificgravityof solidparticleisknown.
- A sample of wet soil about 200 to 400 gm is taken in the pycnometer andweighed.
- Wateristhenaddedtothesoilinthepycnometertomakeitabouthalffull.
- The contents are thoroughly mixed using a glass rod to remove the entrappedair. More and more water is added and stirring process continued till thepycnometerisfilledflushwiththeholeintheconicalcap.
- Thepycnometeriswipeddryandweighed.
- Thepycnometeristhencompletelyemptied.Itiswashedandfilledwithwater,flush withthetop hole.
- Thepycnometeriswipeddryandweighed.
- Let M_1 = mass of pycnometerM_2=massofpycnometer+ wetsoil

M_3=massofpycnometer+wetsoil+water

M_4=massofpycnometerfilledwithwateronly

ThemassM_4isequaltomassM_3minusthemassofsolids$M_s$plusthemassofanequalvolumeof water.

$$M_4=M_3-M_s+\frac{M_s}{G\rho_w}\rho_w$$

$$M_4=M_3-M_s+\frac{M_s}{G}$$

$$=M_3-M_s\left(1-\frac{1}{G}\right)$$

$$M_s=(M_3-M_4)\left(\frac{G}{G-1}\right)$$

Massofwetsoil=M_2-M_1

Thereforemassofwater$M_w=(M_2-M_1)-(M_3-M_4)\left(\frac{G}{G-1}\right)$

$$w=\frac{M_w}{M_s}\times 100$$

$$=\left[\left(\frac{M_2-M_1}{M_3-M_4}\right)\left(\frac{G-1}{G}\right)-1\right]\times 100$$

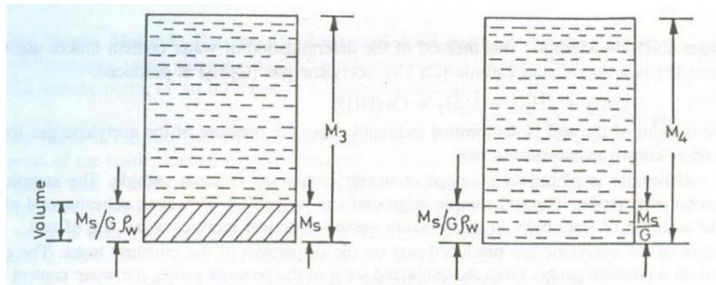


Figure3.2

PycnometermethodDerivation

- Thismethodforthedeterminationofthewatercontentissuitableforcoarsegraineds oilsfromwhichtheentrappedaircanbeeasilyremoved.

SPECIFICGRAVITY:

- Thespecificgravityofsoilsolidscanbedeterminedby:
  (i) A50mldensitybottleor
  (ii) A500mlflaskor
  (iii) Apycnometer
- The densitybottlemethod isthemostaccurateandissuitable forall typesofsoil.

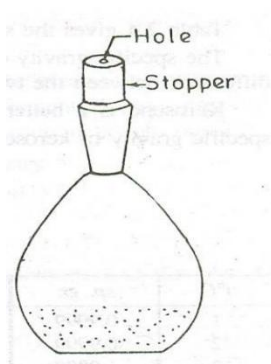


Figure3.3D

ensity bottle

- Theflaskorpycnometerisusedonlyforcoarsegrainedsoils.
- Thedensitybottlemethodisthestandardmethodusedinthelaboratory.
- Intheabovethreemethodsthesequenceofobservationissame.
- ThemassM$_1$oftheempty,dry,bottleisfirsttaken.
- Asampleofoven driedsoilcooledinadesiccatorisputinthebottleandthemass M$_2$is taken.

- Thebottleisthenfilledwithdistilledwatergraduallyremovingtheentrappedaireitherbyapplyingvacuumorbyshakingthebottle.
- ThemassM3ofthebottle,soilandwateristaken.
- Finally,thebottleisemptiedcompletelyandthoroughlywashedandcleanwaterisfilledtothetopandthemassM_4istaken.
- Ifthemassofsolid$M_s$issubtractedfromM_3andreplacedbythemassofwaterequaltothevolumeofsolidthemass M_4isobtained.

$$M_4=M_3-Ms+\frac{Ms\rho w}{G\rho w}$$

$$Or, M_s(1-\frac{1}{G})=M_3-M_4$$

But$M_s=M_2-M_1$

$$Therefore (M_2-M_1)(1-\frac{1}{G})=M_3-M_4$$

$$Or\ \frac{1}{G}(M_2-M_1)=(M_2-M_1)-(M_3-M_4)$$

$$Or G=\frac{M_2-M_1}{(M_2-M_1)-(M_3-M_4)} \text{----------------(i)}$$

$$Alternatively G=\frac{Ms}{Ms+M4-M3} \text{---------------(ii)}$$

Equation(1)givesthespecificgravityofsolidsatthetemperatureatwhichthetestwasconducted.

Specificgravityofsolidsisgenerallyreportedat27°Corat4°C.Thespecificgravity at27°Cand4°Ccanbedeterminedfromthefollowingequation

$$G_{27}=G_t\times\frac{\text{specificgravityofwateratt°C}}{\text{specificgravityofwater at27°C}}$$

And$G_4=G_t\times$specificgravityofwateratt°CWhereG_{27}= specificgravityofparticlesat27°CG_4=specificgravity ofparticlesat4°C

G_t=specificgravityofparticlesatt°C

PARTICLESIZEDISTRIBUTION:

- Thepercentageofvarioussizesofparticleinagivendrysoilsampleisfoundbya particlesizeanalysisormechanicalanalysis.
- Mechanicalanalysismeansseparationofasoilintoitsdifferentsizefractions.
- Themechanicalanalysisisperformedintwostages
  - (i) Sieveanalysis
  - (ii) Sedimentationanalysisorwetmechanicalanalysis
- The first stage is meant for coarse grained soil only while the secondstageisperformedforfinegrainedsoils.
- In general a soil sample may contain both coarse grained particles aswell as fine Particles and hence both the stages of the mechanicalanalysismaybenecessary.

SieveAnalysis:

- In the Indian standard the sieves are designated by the size of theaperatureinmm.
- Thesieveanalysiscanbedividedintotwopartsi.ethecoarseanalysisandfineanalysis.
- Anovendriedsampleofsoilisseparatedintotwofractionsbysievingitthrough a4.75mmI.Ssieve.
- The portion retained on it is termed as the gravel fraction and is keptfor the coarse analysiswhile the portion passing through it is subjectedtofinesieveanalysis.
- The following sets of sieves are used for coarse sieve analysis: IS: 100,63,20,10and4.75mm.
- The sieves used for fine sieve analysis are : IS : 2 mm, 1.0 mm, 600,425,300, 212,150and75micron.
- Sieving is performed by arranging thevarioussieves oneover the otherin the order of their mesh openings – the largest aperature sieve beingkeptatthetopandthesmallestaperaturesieveat thebottom.
- Areceiveriskeptatthebottomandacoveriskeptatthetopof wholeassembly.
- Thesoilsampleisputonthetopsieveandthewholeassemblyisfittedonasieveshakingmachine.
- The amount of shaking depends upon the shape and the number ofparticles.
- Atleast10minutesofshakingisdesirableforsoilswithsmallparticles.
- Theportionofthesoilsampleretainedoneachsieveisweighed.
- Thepercentageofsoilretainedoneachsieveiscalculatedonthebasisofthetotalmassofsoilsampletakenandfromthispercentagepassingthrougheach sieve iscalculated.

SedimentationAnalysis:

- In the wetmechanical analysisor sedimentation analysis the soilfraction finer than 75 micron size is kept in suspension in a liquid(usuallywater)medium.
- Theanalysisisbasedonstokeslawaccordingtowhichthevelocityatwhich grains settle out of suspension, all other factors being equal, isdependentupontheshape,weightandsizeofthegrain.
- However in the usual analysis it is assumed that the soil particles aresphericaland-have thesamespecificgravity.
- Withthisassumptionthecoarserparticlessettlemore quicklythanthefinerones.

- Ifvistheterminalvelocityofsinkingofasphericalparticleitisgivenby

$$v=\frac{2r^2}{9}\frac{\gamma_s-\gamma_w}{\eta}$$

$$\text{Or}\ v=\frac{1D^2}{18}\frac{\gamma_s-\gamma_w}{\eta}$$

Wherer=radiusofthesphericalparticle(m)D=di

ameterofthesphericalparticle(m)

v=terminalvelocity(m/sec)

γ_s=unitweightofparticles(KN/m^3)

γ_w=unitweightofliquidorwater(KN/m^3)

=viscosityofliquidorwater(KNs/m^2)=μ/gμ=vis

cosityinabsoluteunitsofpoise

g=accelerationduetogravity

- Ifwaterisusedasthemediumforsuspension$\gamma_w$isequalto9.81KN/m^3. Similarly

$\gamma_s=G\gamma_w$.Substitutingthisweget

$$v=\frac{1}{18}D^2\frac{(G-1)\gamma_w}{\eta}$$

Theaboveformulashouldbeexpressedintheconsistentunitsofmeters,secondsandkilonewton. Ifthediameter(D)oftheparticlesisinmmwehavev=

$$\frac{1}{18}(D/1000)^2\frac{(G-1)\gamma_w}{\eta}$$

$$=\frac{D^2\,\gamma_w\,(G-1)}{18\times10^6\,\eta}$$

Takingγ_w=9.81KN/m^3

$$\text{Weget}\ v=\frac{D^2(G-1)}{1.835\times10^6\eta}$$

$$D=\sqrt{\frac{18\times10^6\eta v}{(G-1)\gamma_w}}\,mm$$

$$D=1355\sqrt{\frac{\eta v}{G-1}}\,mm$$

Itshouldbenotedthat1poiseisequivalentto0.1Ns/m^2orto$10^{-4}KNs/m^2$IfaparticleofdiameterDmmfallsthroughaheightof$H_e$cmintminutes.v= H_e/60tcm/sec

=H_e/6000tm/sec

Substitutingintheaboveequationweget$D=\sqrt{\dfrac{18\times10^{6}\eta H_e}{(G-1)\gamma_w\times6000t}}$

$$=\sqrt{\frac{3000\eta}{(G-1)\gamma_w}}\sqrt{\frac{H_e}{t}}$$

$$D=10^{-5}F\sqrt{\frac{H_e}{t}}$$

Where$F=10^{5}\sqrt{\dfrac{3000\eta}{(G-1)\gamma_w}}$ isaconstantfactorforgivenvaluesofцandG.

At27°C,theviscosityμofthedistilledwaterisapproximately0.00855poise.Since1poiseis equivalentto10^{-4}KN-s/m^2

Wehaveц=0.00855×10^{-4}KN-s/m^2TakinganaveragevalueofG=2.68

Puttingthesevalueinv=$\dfrac{D^2(G-1)}{1.835\times10^{6}\eta}$

Wegetv=$\dfrac{D^2(9.81)(2.68-1)}{18\times10^{6}\times0.0085\times10^{-4}}$

$=1.077D^2$(m/sec)

This is an approximate version of Stoke's law and can be easily remembered for roughdetermination.

- The sedimentation analysis is done either with the help of hydrometerorapipette.
- Inboththemethods asuitableamountof ovendriedsoilsample,finerthan75micronsizeismixedwithagivenvolumeV ofdistilledwater.
- Themixtureisshakenthoroughlyandthetestisstartedbykeepingthejarcon tainingsoilwatermixture,vertical.
- Atthecommencementofsedimentationtestsoilparticlesareassumedtobe uniformlydistributedthroughoutthesuspension.
- Afteranytimeintervalt,ifasampleofsoilsuspensionistakenfromaheight H_e(measured from the top level of suspension), only thoseparticles will remain in the suspension which have not settled duringthis timeinterval.

Thediameterofthoseparticleswhicharefinerthanthosewhichhavealreadysettled canbefoundfrom$D=10^{-5}F\sqrt{\dfrac{H_e}{t}}$

- The greater the time interval t allowed for suspension to settle, the fineraretheparticlessizesretainedatthis depthH_e.
- Hencesamplingatdifferenttimeintervals,atthissamplingdepthH_e wouldgivethecontentofparticlesofdifferentsizes.
- Ifatanytimeintervalt,$M_D$isthemass,perml,ofallparticlessmallerthan the diameter D still in suspension at the depth H_e the percentagefinerthanDis givenby

$$N=\frac{M_D}{M_d/V}\times100$$

WhereN=percentagefinerthanthediameterD

M_d=totaldrymassofallparticlesputinthesuspension V=volume of suspension

Thuswiththehelpofaboveequationswecangetvariousdiameter Dandthepercentage ofparticlesfiner(N%) thanthisdiameter.

Limitationofsedimentationanalysis:

(1) The sedimentation analysis gives the particle size in terms of equivalentdiameter,whichislessthantheparticlesizegivensieveanalysis.The soilparticlesarenotspherical.Theequivalentdiameterisclosetothethickness(smallestdimension)ratherthanthelengthorwidth.
(2) As the specific gravity of solids for different particles is different, the useof an average value of G is a source of error. However as the variation ofthevalues ofGis smalltheerrorisnegligible.
(3) Stokes law is applicable only when the liquid is infinite. The presence ofwallsofthejaraffectstheresulttosomeextent.
(4) In stokes law it has been assumed that only one sphere settle and there isno interface from other spheres. In the sedimentation analysis as manyparticlessettlesimultaneouslythereissomeinterface.
(5) Thesedimentationanalysiscannotbeusedforparticleslargerthan0.2mmasturbulentconditionsdevelopandstokeslawisnotapplicable.
(6) The sedimentation method is not applicable for particles smaller than 0.2µbecauseBrownianmovementtakesplaceandtheparticlesdonotasperStokeslaw.

PIPETTEMETHOD:

- Thepipettemethodisthestandardsedimentationmethodusedinthelaboratory.
- Theequipmentconsistsofapipette,ajarandanumberofsamplingbottles.
- Generallyaboilingtubeof500mlcapacityisusedinplaceofajar.

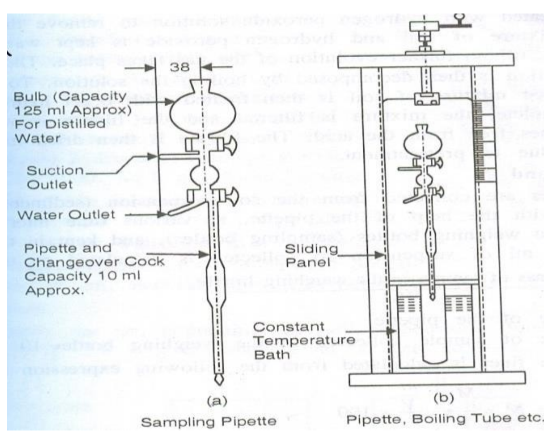


Figure3.4

- The figure shows a pipette for extracting samples from the jar from adesireddepth(H_e).
- The pipette consists of a 125 ml bulb with stop cock for keepingdistilledwater,athreewaystopcock,suctionandwastewateroutlet s,samplingpipetteof 10mlcapacity.
- The method consists in drawing off samples of soil suspension, 10 mlin volume, by means of this pipette from a depth of 10 cm (H_e) atvarioustimeintervalsafterthecommencementofthesedimentation.
- The recommended time intervals are: ½, 1, 2, 4, 8, 15 and 30 minutesand 1, 2, 4, 8, 16 and 24 hours reckoned from the commencement ofthetest.
- The pipette should be inserted in the boiling tube about 25 secondsbefore the selected time interval and the time taken for sucking thesampleshouldnotbemorethan10to20seconds.
- Each sample so taken is transferred into suitable sampling bottles anddriedinanoven.
- The mass $M_D$of solids per ml of suspension is thus found by takingthedrymass anddividingitby10.

Methodofpreparingsoilsuspension:

- Inthesedimentationanalysisonlythose particleswhicharefinerthan75micronsizeareincluded.
- About 12 to 30 gm of oven dried sample is accurately weighed andmixed with distilled water in a dish or beaker to form a smooth thinpaste. To have proper dispersion of soil a dispersing agent is added tothe soil. Some of the common dispersing agents are sodium oxalate,sodium silicate and sodium polyphosphate compounds such as tetrasodiumpyrophosphate,sodiumhexametaphosphateandsodiumtrip olyphosphate.
- IS 2720 recommends the use of dispersing solution containing 33 gmof the sodium hexametaphosphate and 7 g of sodium carbonate indistilledwatertomakeone litreof solution.
- 25 ml ofthis solution isadded to the dish (containing the soil anddistilledwater)andthemixtureiswarmedgentlyforabout10minutes.
- The contents are then transferred to the cup of a mechanical mixer,using a jet of distilled water to wash all traces of the soil out of theevaporating dish.
- Thesoilsuspensionisthenstirredwellfor15minutes.
- The suspension is then washed through 75 micron IS sieve, using jet ofdistilledwaterandthesuspension,whichhaspassedthroughthesieve,istrans ferredtothe500mlcapacityboilingtube(sedimentationtube).
- Thetubeisthenfilledtothe500mlmarkbyaddingdistilledwater.

- Thetubeisthenputinaconstanttemperaturewaterbath.
- Whenthetemperatureisthentubehasbeenstabilisedtothetemperature of the bath, the soil suspension is thoroughly shaken byinvertingthetubeseveraltimes,andthenreplacedinthebath.
- Thestopwasthenstartedandthesoilsamplesarecollectedatvarioustimeintervalswith thehelp ofpipette.

CalculationofDandN:

- 10 ml samples are collected from the soil suspension (sedimentationtube)fromadepthof10cm,withthehelpofthepipetteatvarioustimeintervals.
- Thesamplesarecollectedintotheweighingbottles(samplingbottles)andkeptintheovenfordrying.
- The mass M_D, per ml of suspension so collected is calculated as under

:M_D=drymassofsampleintheweighing bottle/V_P

WhereV_P=volumeofthepipette

= volume of sample collected in the weighing bottle = 10 mlThepercentagefineriscalculated fromthefollowingexpression

$$N' = \frac{M_D - \frac{m}{7}}{\frac{M_D}{7}} \times 100$$

Wherem=massofdispersingagentpresentintheinthetotalsuspensionofvolumeV

V=volumeofsuspension=500mlN'=p

ercentagefinerbasedonM_D

HYDROMETERMETHOD:

- The hydrometer method of sedimentation analysis differs from thepipetteanalysisinthe methodoftakingobservation.
- In the pipette analysis the mass $M_D$per ml of suspension is founddirectly by collecting a 10 ml sample of soil suspension from thesamplingdepthH_e.Howeverinthehydrometeranalysis$M_D$iscomputed indirectly by reading the density of the soil suspension at adepth$H_e$atvarioustime intervals.
- In the pipette test the sampling depth H_e is kept constant while in thehydrometer test, the sampling depth $H_e$goes on increasing as theparticles settle with the increase in the time interval. It is thereforenecessarytocalibratethehydrometer.

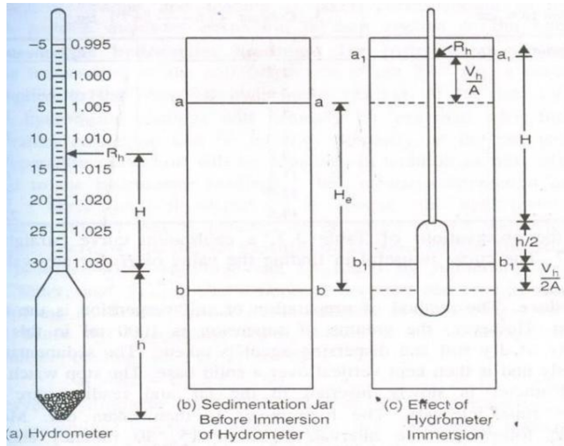


Figure3.5

Calibrationofhydrometer:

- The readings on the hydrometer stem give the density of the soilsuspensionsituatedatthecentreofthebulbatanytime.
- Forconvenience,thehydrometerreadingsarerecordedaftersubtracting 1 and multiplying the remaining digits by 1000. Such areducedreadingis designatedasR_h.
- For example, if the density reading at the intersection of horizontalsurfaceofsoilsuspensionwith thestemis1.010,itisrecordedas10 i.eR_h=10.
- As indicated in figure the hydrometer reading $R_h$is increase in thedownwarddirectiontowardsthehydrometerbulb.
- LetHbetheheightincmbetweenanyhydrometerreading$R_h$andtheneck,andhtheheightofthebulb.
- Figure(b)showsthejarcontainingthesoilsuspension.
- When the hydrometer is immersed in the jar as shown in figure (c) thewater level aa rises to a_1a_1, the rise being equal to the volume V_h ofthe hydrometer divided by the internal area of cross section A of thejar.
- Similarly the level bb rises to b_1b_1, where bb is the level situated at adepth $H_e$below the top level aa, at which the density measurements ofthesoilsuspensionarebeingtaken.
- Therisebetweenbbandb$_1$b$_1$willbeapproximatelyequaltoV$_h$/2A.
- Thelevelb1b1isnowcorrespondingtothecentreofthebulb,butthesoilparticlesatb1b1areofthesameconcentrationastheywereatbb.
- Therefore we have

$$H_e=\left(H+\frac{h}{2}+\frac{V_h}{2A}\right)-\frac{V_h}{A}$$

$$=H+\tfrac{1}{2}\left(h-\frac{V_h}{A}\right)$$

- Intheaboveexpression therearetwovariables:theeffectivedepth$H_e$andthedepthHwhichdepends uponthehydrometerreadingR_h.
- Therefore by selecting various hydrometer reading R_h, the depth H canbe measured with the help of an accurate scale and the correspondingdepth$H_e$canbefound.
- Theheighthofthebulbisconstant.Similarly$V_h$andAareconstant.
- To find the volume of the hydrometer it is weighed accurately. Themassofthehydrometeringramsgivethevolumeofthehydrometerinmi llilitres.

Testprocedure:

- The methodofpreparationofsoilsuspensionis thesameas indicatedinthepipettetest.
- Howeverthevolumeofsuspensionis1000mlinthiscaseandhencedoubles thequantityofdrysoilanddispersingagentistaken.
- Thesedimentationjarisshakenvigorouslyandisthenkeptverticaloverasol idbase.
- Thestopwatchisstartedsimultaneously.
- Thehydrometerisslowlyinsertedinthejarandreadingsaretakenat ½,1and2minutestimeinterval.Thehydrometeristhentakenout.
- More readings are then taken at the following time intervals: 4, 8, 15,30minutesand1,2,4hoursetc.
- To take the reading, the hydrometer is inserted about 30 seconds beforethegiventimeinterval,sothatitisstableatthetimewhenthereadingis tobetaken.
- Sincethesoilsuspensionisopaquethereadingistakencorrespondingtothe upperlevelofthemeniscus.

Correctiontothehydrometerreading:

Thehydrometerreadingsarecorrectedasunder:

(i) Meniscuscorrection:

- Sincethesuspensionisopaque,theobservationsaretakenatthetopofthemeniscus.
- Themeniscuscorrectionisequaltothereadingbetweenthetopofthemenisc usandthelevelofthesuspension.
- As the marking on the stem increase downward the correction ispositive.
- The meniscus correction (C_m) is determined from the readings at thetopandbottomofmeniscusinthecomparisoncylinder.Themeniscusco rrectionisconstantforahydrometer.
- If R_h’ is the hydrometer reading of the suspension at a particular time,thecorrectedhydrometer$R_h$readingisgivenby

R_h=R_h’+C_m

(ii) **Temperaturecorrection:**

- The hydrometer is generally calibrated at 27°C. If the temperature ofthe suspension is different from 27°C a temperature correction (C_t) isrequiredforthehydrometerreading.
- If the temperature is more than 27°C, the suspension is lighter and theactualreadingwillbelessthanthecorrectedreading.Thetemperatureco rrectionis positive.
- Ontheotherhand,ifthetemperatureislessthan27°Cthetemperaturecorrec tionis negative.

(iii) **Dispersionagentcorrection:**

- Addition of the dispersing agent to the soil suspension causes anincreaseinthespecificgravityofthesuspension.
- Thereforethedispersingagentcorrectionisalwaysnegative.
- The dispersing agent correction (C_d) can be determined by noting thehydrometer reading in clear water and again in the same water afteraddingthedispersingagent.
- Thus the corrected reading Rcan beobtained from theobservedreading R_h'asunder

$R=R_h'+C_m \pm C_t-C_d$

(iv) **CompositeCorrection:**

- Insteadoffindingthecorrectionindividually,itisconvenienttofindoneco mpositecorrection.
- Thecompositecorrection(C)isthealgebraicsumofallthecorrection.ThusR=

$R_h' \pm C$

- The composite correction is found directly from the readings taken in acomparisoncylinder,whichhasdistilledwaterandthedispersingagent inthesameconcentrationandhasthesametemperature.

ComputationofDandN:

- TheparticlesizeDiscalculatedfromthefollowingformulaD=10-

$^5F\sqrt{\frac{H_e}{t}}$

- To compute the percentage of the soil finer than this diameter, the massMDper ml of suspension at effective depth $H_e$is first computed asunder
- Since the hydrometer readings have been recorded by subtracting 1fromthedensity(þ)readingsandmultiplyingthemby1000,wehave

R=(þ-1)1000

Or, þ=1+R/1000 ....(i)

Where þ is the density reading actually marked on the hydrometer and R is thehydrometerreadingcorrectedforthecompositecorrection.

- Now let us consider 1 ml of soil suspension at a time interval t at theeffective depth H_e. If M_D is the mass of solids in this 1 ml suspensionthemass of waterinitwillbe

$1-M_D/G$

Totalmassof1mlsuspension$=1-\frac{M_D}{G}+M_D$

Hencedensityofthesuspension$=1-\frac{M_D}{G}+M_D$ ------------------(ii)

Equatingequation(i)and(ii)weget

$$1+R/1000 = 1-\frac{M_D}{G}+M_D$$

$$M_D=\frac{R}{1000}\left(\frac{G}{G-1}\right)$$

WhereG=specificgravityofsoilsolids

SubstitutingthesevaluesinequationN$=\frac{MD}{M_{D/7}}\times 100$

WegetN' $=\frac{\frac{R}{1000}\left(\frac{G}{G-1}\right)}{Md/V}\times 100$

TakingV=1000mlweget

$$N'=\frac{100GR}{Md(G-1)}$$

WhereN'=percentagefinerwithrespecttoM$_d$

ThusforvariousvaluesofR,N'canbecalculated

- For a combined sieve and sedimentation analysis if M is the total drymass of soil originally taken (before sieving it over 2mm sieve) theoverallpercentagefinerNis givenby

$$N=N'\times\frac{M'}{M}$$

WhereM'=cumulativemasspassing2mmsieveM=tot
aldrymassofsoilsample

Ifthesoilsampledoesnotcontainparticlescoarserthan2mmsize,NandN'willbeequal.

CONSISTENCYOFSOIL:

- Consistencymeanstherelativeeasewithwhichsoilcanbedeformed.
- Consistencydenotesdegreeoffirmnessofthesoilwhichmaybetermedass oft,firm,stifforhard.
- Finegrainedsoilmaybemixedwithwatertoformaplasticpastewhichcanb emouldedintoanyform bypressure.
- Theadditionofwaterreducesthecohesionmakingthesoilstilleasier

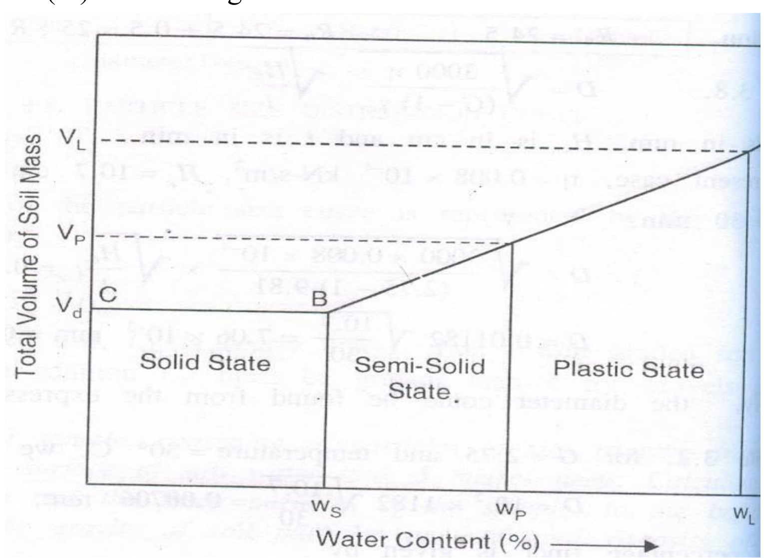
tomould.

- Furtheradditionofwaterreducestheuntilthematerialnolongerretainsitss hapeunderitsownweight,butflowsasaliquid.

- Enoughwatermaybeaddeduntilthesoilgrainsaredispersedinasuspension.
- Ifwaterisevaporatedfromsuchasoilsuspensionthesoilpassesthroughvariousstages orstatesofconsistency.
- SwedishagriculturistAtterbergdividedtheentirerangefromliquidtosolid state intofourstages:
  (i) Liquidstate
  (ii) Plasticstate
  (iii) Semi–solidstate
  (iv) Solidstate

HesetsarbitrarylimitsknownasconsistencylimitorAtterberglimitforthesedivisionsintermsofwatercontent

- Thusconsistencylimitsarethewatercontentatwhichthesoilmasspassesfromonestatetothe next.
- TheAtterberglimitswhicharemostusefulare:
  (i) Liquidlimit
  (ii) Plasticlimit
  (iii) Shrinkagelimit



Figure

3.6Differentstatesofso

LiquidLimit: **il**

- Liquid limit is the water content corresponding to the arbitrary limitbetweenliquidandplastic stateofconsistencyofasoil.
- It is defined as the minimum water content at which the soil is still intheliquidstatebuthasasmallshearingstrengthagainstflowingwhichcan bemeasuredbystandardavailablemeans.

- With reference to the standard liquid limit device, it is defined as theminimum water content at which a part of soil cut by a groove ofstandard dimension, will flow together for a distance of 12 mm underanimpactof25blowsinthedevice.

Plasticlimit:

- Plastic limit is the water content corresponding to an arbitrary limitbetweentheplasticandthesemi-solidstateofconsistencyofasoil.
- It is defined as the minimum water content at which a soil will justbegin to crumble when rolled into a thread approximately 3 mm indiameter.

Shrinkagelimit:

- Shrinkage limit is defined as the maximum water content at which areductioninwatercontentwillnotcauseadecreaseinthevolumeofasoilmass.
- It is lowest water content at which a soil can still be completelysaturated.

PlasticityIndex:

- Therangeofconsistencywithinwhichasoilexhibitplasticpropertiesiscalledplasticrangeandisindicatedbyplasticityindex.
- Plasticityindex is defined is defined asthe numerical differencebetweentheliquidlimitandtheplasticlimitofasoil.

$I_p = w_l - w_p$

- Whenplasticlimitcannotbedetermined,theplasticityindexisreportedas NP(Nonplastic).
- Whentheplasticlimitisequaltoorgreaterthantheliquidlimittheplasticityindexisreported aszero.

Plasticity:

- Plasticity is defined as that property of a soil which allows it to bedeformed rapidly, without rupture, without elastic rebound and withoutvolume change.

ConsistencyIndex:

- The consistency indexor the relative consistency is defined asthe ratioof the liquid limit minus the natural water content to the plasticityindexofsoil

$$I_c = \frac{w_l - w}{I_p}$$

Wherewisthenaturalwatercontentofthesoil

- Ifconsistencyindexofasoilisequaltounity,itisattheplasticlimit.
- SimilarlyasoilwithI$_c$equaltozeroisatitsliquidlimit.

- If I_c exceeds unity the soil is in a semi–solid state and will be stiff.

- A negative consistency index indicates that the soil has natural watercontent greater than the liquid limit and hence behaves just like aliquid.

Liquidityindex:

- Theliquidityindexorwater plasticityratioistheratio,expressedasapercentage,ofthenaturalwatercontentofasoilminusitsplasticlimittoitsplasticity index:

$$I_l = \frac{w - w_p}{I_p}$$

Wherewisthenaturalwatercontentofthesoil.

Example–1

- Duringatestforwatercontentdeterminationonasoilsamplebypycnometer,thefollowing observations weretaken
  (1) Massofwetsoilsample =1000gm
  (2) Massofpycnometerwithsoilandfilledwithwater =2000gm
  (3) Massofpycnometerfilledwithwateronly=1480gm
  (4) Specificgravityofsolids =2.67Determine

thewatercontent.

Solution:

Weknowthat$w=[(\frac{M2-M1}{M3-M4})(\frac{G-1}{G})-1]\times100$

$$=[\frac{1000}{(2000-1480)}\times(\frac{2.67-1}{2.67})-1]\times100$$

$$=20.28\%$$

Hencewatercontentofthesampleis20.28%. (Ans)

Example2:

The mass of an empty gas jar was 0.498 Kg. When completely filled with water itsmass was 1.528 Kg. An oven dried sample of soil mass 0.198 Kg was placed in the jarandwater wasaddedtofillthejaranditsmasswasfoundtobe 1.653Kg. Determinethespecificgravityofparticle.

Solution: we know

that$G=\frac{M_2-M_1}{(M_2-M_1)-(M_3-M_4)}$

0.198

$$= \frac{}{0.198-(1.653-1.528)}$$

=2.71

Hencespecificgravityofthesampleis2.71.

Example3:

A soil sample consisting of particles of size ranging from 0.5 mm to 0.01 mm, is putonthesurfaceofstillwatertank5metresdeep.Calculatethetimeofsettlementofthecoarsest and the finest particle of the sample, to the bottom of the tank. Assumeaveragespecificgravityofsoilparticlesas2.66andviscosityofwateras0.01poise.

Solution:

$$v=\frac{D^2\gamma_w(G-1)}{18\times10^6\eta}$$

$$=\frac{D^2(G-1)}{1.835\times10^6\times\eta}$$

HereG=2.66andц=$0.01\times10^{-4}=10^{-6}$KN-s/$m^2$

$$v=\frac{D^2}{1.835}\times\frac{2.66-1}{10^6(10^{-6})}$$

$=0.905D^2$

Wherevisinm/secandDisinmmFor

coarsest particle D = 0.5

mmv=$0.905(0.5)^2$=0.2263m/sec

t = h/v = 5/0.2263 = 22.1

secondsforthefinestparticle,D=0.01

mmv=$0.905(0.01)^2=9.05\times10^{-5}$m/sec

t= $\frac{5}{9.05\times10^{-5}}$=55249sec=15hours20min49seconds.

Example4:

50gramsofovendriedsoilsampleistakenforsedimentationanalysis.Thehydrometer reading in a 100 ml soil suspension 30 minutes after the commencementof sedimentation test is 24.5. The effective depth for R_h= 25, found from thecalibration curve is 10.7 cm. The meniscus correction is found to be +0.5 and thecomposite correction as – 2.50 at the test temperature of 30°C. Taking the specificgravity of particles as 2.75 and viscosity of water as 0.008 poise, calculate the smallestparticle size which would have settled during this interval of 30 minutes andthepercentage ofparticlesfinerthanthis size.

Solution:

$R_h'=24.5$ $R_h=24.5+0.5=25$ $R=24.5–2.5=22$

$$D=\sqrt{\frac{3000\eta}{(G-1)\gamma_w}}\sqrt{\frac{H_e}{t}}$$

Where D is in mm, H_e is in cm and t is in minute. Here η=

0.008×10^{-4} KN-s/m^2

$H_e=10.7$cm

G=2.75 and $\gamma_w=9.81$ KN/m^3 t=3

0min.

$$D=\sqrt{\frac{3000\times0.008\times10^{-4}}{(2.75-1)9.81}}\times\sqrt{\frac{10.7}{30}}$$

$=7.06\times10^{-3}$mm

=0.00706mm

The percentage finer is given by $N'=\frac{100G}{M_d(G-1)}R$

Where M_d=mass of dry soil=50gm

$$N'=\frac{100\times2.75}{50(2.75-1)}\times22$$

=69.1%. (Ans)

Example5:

A soil has a liquid limit of 25 % and plastic limit is 15 %. Determine the plasticity index. If the water content of the soil in its natural condition in the field is 20%, find the liquidity index and relative consistency.

Solution:

$w_l=25\%$ $w_p=15\%$ $w=20\%$

plasticity Index $I_p=w_l-w_p$

=25–15=10%

Liquidity index=$I_l=\frac{w-w_p}{I_p}\times100$

$$= \frac{0.2-0.15}{0.1} \times 100 = 50\%$$

Relativeconsistency=I_c $= \frac{wl-w}{I_p} \times 100$

$$= \frac{0.25-0.2}{0.1} \times 100 = 50\%$$ (Ans)

CHAPTER-4

CLASSIFICATIONOFSOIL

Purposeofsoilclassification:

The purpose of soil classification is to arrange various types of soil into specific groupsbased on physical properties and engineering behavior of the soils with the objective offindingthesuitabilityofsoilsfordifferentengineeringapplication,suchasintheconstructionofe arthdams,highway,andfoundationsofbuilding,etc.

For different areas of applications and withthe need for simplicity andacceptable terminology, several soil classification system s have been developed over theyears,three ofwhicharelistedbelow.

1. Highwayresearchboardclassificationsystem
2. Unifiedsoilclassificationsystem
3. Indianstandardsoilclassification.

HighwayResearchBoard (HRB)classificationSystem:

The Highway Reach Board classification system, also known as Revised Public RoadsAdministration classification system ,is used to find the suitability of a soil , as sub gradematerial in pavement construction .This classification system is based on both particle sizerangesand plasticity characteristic .soilare divided into 7 primary group designated as A -1,A-2………A-7,asshownintable 4.1.

Group A-1, is divided into two sub groups A-1 and A-1 and group A-2into four subgroups, A-2-4 , A-2-5 ,A-2-6 and A-2-7 .A characteristic group index is used to describe theperformanceofasoil as subgradematerial.

Groundindexisnotusedtoplaceasoilinaparticulargroup;itisactuallyameansof rating the value of soil as a sub grade material within its own group .The higher the valueofthegroupindex, theproperisthequalityofthematerial.

Thegroupindexofasoildependsupon.

(i) Amountofmaterialpassingthe75-micronsieve,
(ii) Liquidlimit
(iii) Plasticlimit

Groupindexisgivenbythefollowingequation:

Group index=0.2a+0.005ac+0.01bdWhere

a=that portionofpercentagepassing75micronsievegreater than35andnot exceeding 75expressedaswholenumber(0 to40)

b=that portion percentagepassing 75micron sievegreater than 15 andnotexceeding55expressedaswholenumber(0 to 40)

c=thatportionof thenumericalliquid limit greaterthan 40andnotexceeded60expressedasapositivewholenumber(0to 20) d=thatportionofthenumericalplasticityindexgreaterthan10and not exceeding30expressedasapositivewholenumber(0to20).

To classify a given soil, sieve analysis data, liquid limit and plasticity index are obtained andwe proceed from left to right in the Table 4.1 and by Process of elimination find the firstgroupfromintowhichthetestdatawillfit.ThisgivesthecorrectClassification.Theplasticityindex of A-7-5 subgroup is equal to or less than liquid limit minus 30.The plasticity index ofA-7-6subgroupisgreaterthanliquidlimit minus30.

Note: The PRA system was introduced in 1928 and revised in 1945 as HRB system. It isknown as AASHTO system since 1978 after adoption by American Association of StateHighwayandTransportation Officials.

Unifiedsoil classificationSystem

The Unified soil classification system is based on the Airfield Classification system that wasdevelopedbyACasagrande.thesystembasedonbothgrainsizeandplasticitycharacteristicof soil. The unified Soil classification (USC)system was adopted jointly by theCorpsofEngineers,U.S.armyandU.S.BureauofReclamationduring1950s.

1. Coarse-grained soils – if more than 50% by Weighty is retained on No. 200 ASTMsieve.
2. Fine–grainsoil-ifmorethan50%byweightpassesthroughNo.200ASTMsieve
3. Organicsoils.

The soil component are assigned groupsymbolsas indicated below:Coarse–grainedsoils:

Gravel:G Sand:S

Finegrainedsoils:

Silt:M Clay’s Organicsoil’s

Table4.3UnifiedSoilClassificationSystem

MajorDivision			GroupSymbols		Typical names
Coarse Grain Soils more than 50% retainedon No. 200 sieve *	Gravels 50%or moreof coarse fraction retained onNo.4 sieves	Cleargravels -200fraction <5%	GW	Wellgraded	
		Gravelwith fine-200>5% fraction	GP		
	Sand more than50% ofcross fraction passes No.4sieve	Cleansands-200<55 fraction	GM		
		Sandwith fines-200>12% fraction	GC		
Fine grained soils 50%or more passes No.200 sieve*			SW		
			SP		
			SM		
			SC		
Fine-grainedsoils 50% ormore passesNo.200 sieve.					

Table4.3givesthedetailsofUnifiedsoilClassificationsystem.Theoriginalcasagrandeplasticitychart usedforclassifingfinegraindsoilisgiveninFig 4.3
They symbolMforsilyindrivedfrom the swedishword'mo'forsilt.Example4.3.classifythesoilwithcompositionindicatedin4.2u singUSCsystem.
Solution:Sincemorethan50%ofsoilpassesthrough0.074mmsievethesoilisfinegrained.
Plasticityindex=(50-40)%=10%
Fromfig4.3
ForwL=40%ANDIp=10%thesoilcanbeclassifiedasMLOROL

INDIANSTANDARDSOILCLASSIFICATIONSYSTEM

Indian standard soil classification system (IS 1498-1970 classification andidentification of soil for general engineering purpose) is esssentially based onunified soil classification system and the salient features and given in thefollowingdiscussion.

Inthesystemsoilsarebroadly dividedinto3division

1. coarse- grained soil –if more than 50% by mass is retain on 75 micron ISsieve.
2. Finegrainedsoil–ifmorethan50%bymasspassesthrough75micronISsieve.
3. Highlyorganicsoilsandothermiscellriounssoilmaterial.Thesoilcontentlarger%o ffibrousorganicmattersuchaspeatandparticlesofdecomposevegetation. In additionshortensoilcontainingshells ,concretionscinders andothernon soilmaterialinsufficientquantities are also groupedinthisdivision.Coarsegrainedsoilsaregroupedasgravelsandsandswithgro upsymbolsGandS

Gravels(G)ifmorethan50%bymassofthecoarsegrainedfractionpassedthroug h4.75mmISsieve.

Depending on the gradation gravels(G) and sands (S) are further described usinggroupsymbolsareindicatedbelow.

GW-WellgradedgravelforwhichCu>4andCcliesbetween1&3

GP-poorlygradedgravelwhichdoesnotmeetallgraduationrequirement of GW

SW-WellgradedsandforwhichCu>6andCcliesbetween1&3GM-siltygravelif Ip> 4forfine-grainedfraction.

GC-Clayey gravel if Ip < 7 for fine-grained fraction.SM-SiltysandifIp<4forfinegrainedfraction.

SC-ClayeysandifIp>7forfine-grainedfraction.

In the case of coarse –grained soils mixed with fines if Ip lies between 4and 7 one has to use proper judgment in dealing with this border line case.Generally non-plastic classification is favored in such cases. For example asandwith10%fineswithCu>6,Ccbetween1and3andIp= 6wouldbeclassifiedasSW–SMratherthanSW-SC.

Fine-grainedsoilsaregroupedunderfollowingthreesubdivisionswithrespectivegroupsymbols:

Inorganicsiltsandveryfinesands(M)In organicclays(C)

Organicsilts,OrganicclaysandOrganicmatter(O)

Dependingonliquidlimitwhichisconsideredagoodindexofcompressibilityfinegrained soils are described as possessing (i)low compressibility (L) when liquidlimitislessthan35percent.

(ii)intermediateormediumcompressibility(I)whenliquidlimitliesbetween35percentand50percent

(iii)highcompressibility(H)whenliquidlimitisgreaterthan50percent.

The plasticity chart originally devised by A. Casagrande and slightlymodifiedbyISisusedtoclassifyfine-grainedsoilsinthelaboratory.

TheA-linehavingtheequation:

Ip=0.73(Wl-20)

Andthetwoverticallinesatwl=35andwl=50dividethechartintosixregionswithgroup symbols marked as shown in Fig.4.4 if the plotted position lies belowA-line, the soil has to be checked for organic odour by slight heating. Ifnoorganic odour is smelt than only it should be classified as inorganic silt. In case ofdoubt,thesoilshouldbeoven-driedanditsliquidlimitdetermined again.Inthecaseoforganicsoilstherewillbelargereductioninliquidlimitondrying(reductiongenerally>25%).

Soil	Soil component	Symbol	Particlesizerange anddescription
Coarse-grained Components	Boulder	none	Roundtoangular,bulkyhard,rock particle,averagediametermorethan300mm.
	Cobble	None	Roundtoangular,bulkyhard,rockparticle averagediametersmallerthan300mmbutretainedon 80mmsieve.
	Gravel	G	Roundedtoangular,bulkyhard,rock particle,passing80mmsievebutretainedon 4.75mmsieve. Coarse:80 mm to 20 mm sieveFine:20 mmto4.75mmsieve
	Sand	S	Roundedtoangularbulkyhard,rocky particle,passing4.75mmsieveretainedon75micronsieve Coarse:4.75 mmto2.0mmsieveMedium:2.0 mm to 425 micron sieveFine:425micronto75 micronsieve

	silt	M	Particlessmallerthan75-micronsieve indentifiedbybehavior;thatisslightlyplastico rnon-plasticregardlessofmoistureandexhibitslittleor nostrengthwhenair

Fine–grained Components			dried
	Clay	C	Particlessmaller than75-micron sieve identifiedbybehavior,thanis,itcanmadetoexhibitplastic propertieswithinacertainrange of moistureand exhibitsconsiderablestrengthwhen airdried

CHAPTER-5

PERMEABILITYANDSEEPAGE

5.1 ConceptofPermeability:-

- Thepropertyofsoilwhichpermitsflowofwater(orotheranyliquid)throughit is called the permeability in otherworld, thepermeabilityinthecase withwhichwatercanflowthroughit.
- Permeabilityisveryimportantengineeringpropertyofsoil.
- The knowledge of permeabilityis essentialin a numberof soil engineeringproblems such as: Settlement of Buildings, Yield of wells, Seepage through andbelowtheearthsurface.
- Permeabilitycontrolsthehydraulicstabilityofsoilmasses.
- The permeability of soils is also required in the design of filters required topreventpipinginhydraulicstructure.

Darcy'sLaw:-

Theflowoffreewaterthroughsoilisgovernedby Darcy'slaw.In1856,Darcyexperimentally that for laminar flow in a homogeneous soil, the velocity of flow (v) isgivenby

$$v=ki \longrightarrow \text{Equation no-1}$$

Where,k=coefficientofpermeability,i=hydraulicgradientandv=velocityofflowi nlaminarflowinhomogeneoussoil

TheaboveequationisknownasDarcy'slaw,whichisoneofthecornerstonesofsoilengi neering.Thedischarge'q'isobtainedbymultiplyingthevelocityofflow (v) bythetotalcross-sectionalarea(A)normaltothedirectionofflow

$$\text{Thus,}q=vA=kiA \longrightarrow \text{Equationno-2}$$

Note:-1)Thevelocityofflowisalsoknownasdischargevelocityorsuperficialvelocity.
2)TheareaAintheaboveequationincludesboththesolidsandthevoids.

Co-efficientofPermeability:-

- Thecoefficientofpermeabilitycanbedefinedusingtheequation1.Ifthehydraulicgra dientisunity,thecoefficientofpermeabilityisequaltothevelocityofflow

Or,

- The coefficient of permeability is defined as the velocity of flow which wouldoccur under unit hydraulic gradient. The co-efficient of permeability isequaltothevelocityofflow.

- Thecoefficientpermeabilityhasthedimensionsofvelocity[L/T].

- The coefficient of permeability measured in mm/sec, cm/sec, m/sec, m/dayorothervelocityunits.

- The coefficient of permeability depends upon the particle size and uponmanyfactors.

- According to USBR, the soil having co-efficient permeability greater than 10^{-3}mm/sec are classified as pervious and those with a value less than 10^{-5} to 10^{-3}mm/secaredesignatedassemi-pervious.

5.2 FactorsaffectingPermeabilityofsoils:-

Thefollowingfactorsaffectthepermeabilityofsoils.

(1) ParticleSize.
(2) Structureofsoilmass.
(3) Shapeofparticles.
(4) Voidratio.
(5) Propertiesofwater.
(6) Degreeofsaturation.
(7) Adsorbedwater.
(8) Impuritiesinwater.

(1) ParticleSize:-Co-efficient ofpermeabilityofsoilisproportionaltothesquare of particle size (D). The permeability of coarse grained soils is very large ascompared to that of fine-grained soils. The permeability of coarse sand may be more thanonemilliontimesasmuchthatofclay.

(2) Structureofsoilmass:-

ThecoefficientCtakesintoaccounttheshapeofflowpassage. The size of flow passage depends upon the structural arrangement. For samevoid ratio, the permeability is more in the case of flocculated structure as compared tothatinthedispersesstructure.

Stratifiedsoildepositshavegreaterpermeabilityparalleltotheplaneofstratification than that perpendicular to this plane. Permeability of soil deposit alsodepends upon shrinkage cracks, joints, fissures and shear zones. Loess deposits havegreaterpermeabilityintheverticaldirectionthaninthehorizontaldirection.

The permeability of natural soil deposit should be determined in undisturbedcondition. The disturbance caused during sampling may destroy the original structureandaffectthepermeability.Theeffectofdisturbanceismorepronouncesincase offine-grainedsoilsthaninthecaseofcoarse-grainedsoils.

(3) Shape of Particles: - The permeability of a soil depends upon the shape ofparticles.Angularparticleshavegreaterspecificsurfaceareascomparedwiththe

rounded particles. For the same void ratio, the soils with angular particles are lesspermeablethanthosewithroundedparticles,asthepermeabilityisinversely

proportionaltothespecificsurface.However,inanaturaldeposit,thevoidratioofora soil with angular particles may be greater than that for rounded particles, and the soilwithangularparticlesmaybeactuallymorepermeable.

(4) Void Ratio: - For a given soil, the greater the void ratio, the higher is thevalueofthecoefficientofpermeability.

Basedontheotherconcepts,ithasbeenestablishedthatthepermeabilityof soilvaries as e^2 or $e^2/(1+e)$ (figure-2). Whatever may be the exact relationship;alloilshaveeversuslogkplotasastraightline(figure-1).

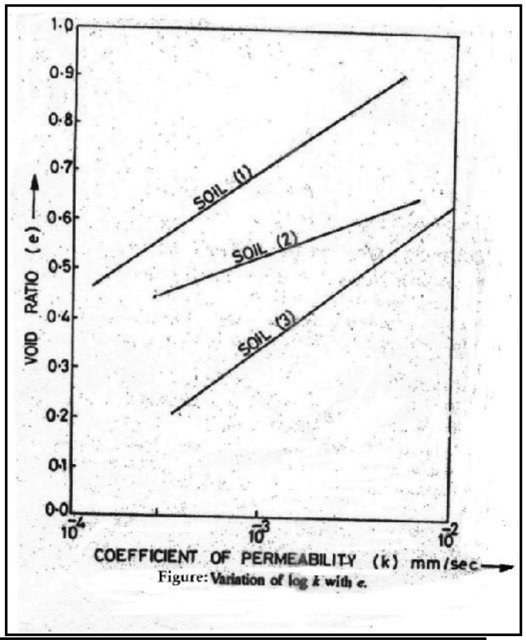
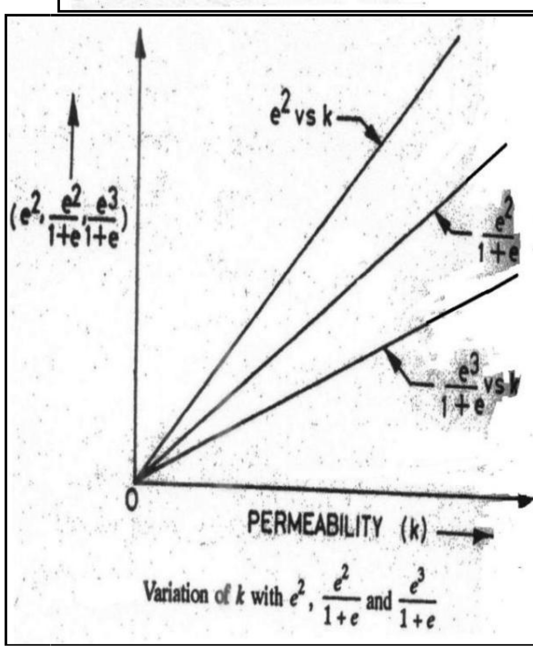


Figure: Variation of log k with e.



Variation of k with e^2, $\frac{e^2}{1+e}$ and $\frac{e^3}{1+e}$

Figure(1) Figure(2)

If the permeability of a soil at a void ratio of 0.85 isknown, itsvalueat anothervoidratioof‘e’can bedetermined usingthefollowingequation givenbyCasagrande:

$$k=1.4k_{0.85}e^2$$

Where$k_{0.85}$=permeabilityatvoidrationof0.85,k=permeabilityatavoidratio of‘e’.

(5) PropertiesofWater:-Theco-efficientofpermeabilityisdirectlyproportional to the unit weight of water (γ_w) and is inversely proportional to its viscosity(μ). The coefficient of permeability increases with an increasethe temperature due toreductionintheviscosity.

(6) Degree of Saturation:- if the soil is not fully saturated, it contains air pocketformed due to entrapped air or due to air liberated from percolating water. Whatevermay be thecauseofpresence of air in soils, thepermeability isreduced due to presenceof sir which causes blockage of passage. Consequently,the permeability of partiallysaturated soil is considerably smaller than that of fully saturated soil. In fact Darcy’s Lawnotstrictlyapplicabletosuchsoils.

(7) Adsorbed Water: - The fine grained soils have a layer of adsorbed waterstrongly attached to their surface. This adsorbed water layer is not free to move undergravity. It causes an obstruction to flow of water intheporesandhence reduces thepermeabilityofsoils.

It is difficultto estimate the void occupied by the adsorbed water. Accordingto one estimate, the void ration occupied by adsorbed water is about 0.10. The effectivevoidratioavailableforflowofwateristhus about (e - 0.1) and not ‘e’. In somecases,atverylowhydraulicgradient,thecoefficientofpermeabilityoffine-grainedsoilsbecomesnegligiblesmallduetopresenceofadsorbedwater.

(8) ImpuritiesinWater:-
Anyforeignmatterinwaterhasatendencytoplugtheflowpassageandreducetheeffectivevoidsandhencethepermeabilityofsoils.

5.3 .1- ConstantHeadPermeability Test:-

Thecoefficientofpermeabilityofarelativelymorepermeablesoilcanbedetermined inalaboratorybytheconstant–headpermeabilitytest.

1. Thetestisconductedinaninstrumentknownasconstant-headPermeameter.
2. Itconsistsofametallicmould,100mminternaldiameter,127.3mmeffectiveheightand1000mlcapcityaccordingtoIS:2720(PartXVII).
3. Themouldisprovidedwithadetachableextensioncollar,100mmdiameterand60mmhigh,requiredduringcompactionofsoil.
4. Themouldisprovidedwithdrainagebaseplatewithrecessforporousstone.
5. Themouldisfittedwithadrainagecaphavinganinletvalveandanairreleasevalve.
6. Thedrainagebaseandcaphavefittingsforclampingtothemould.

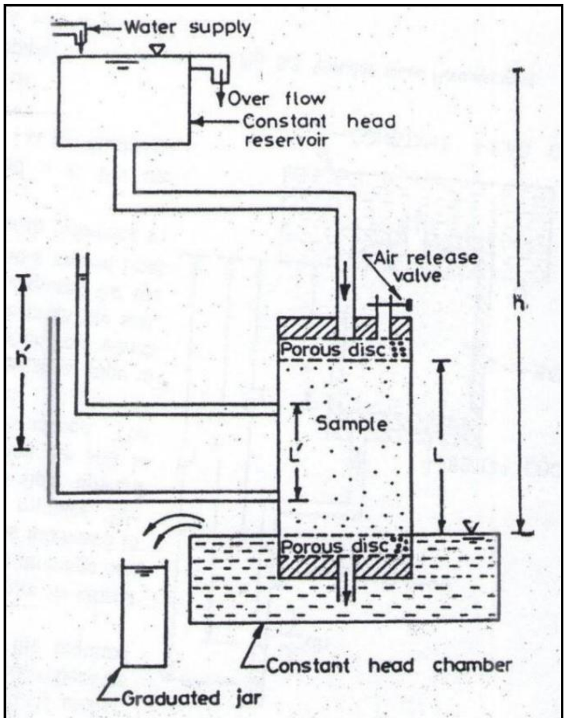


Figure:-Constant Head Permeameter

1. Theabovefigureshowsaschematicsketch.
2. Thesoilsampleisplacedinsidethemouldbetweentwoporousdiscs.
3. Theporousdiscsshouldbeatleasttentimesmorepermeablethanthesoil.
4. Theporousdiscsshouldbedeairedbeforethesearereplacedinthemould.
5. Thewatertubesshouldalsobedeaired.
6. Thesample canalso bepreparedinthepermeameterbypouringthesoilintoitandtampingittoobtainthe requireddensity.
7. Thebaseisprovidedwithadummyplate,12mmthickand108mmindiameter ,whichisusedwhenthesampleiscompactedinthemould.
8. It is essential that the sample is fully saturated. This is done by one of thefollowingthreemethods:-
    i. By pouring the soil in the permeamter filled with water and thusdepositingthesoilunderwater.
    ii. By allowing water to flow from the base to the top after the soilhasbeen placed in the mould. This is done by attaching the constant-headreservoir to the drainage base. The upward flow is maintained forsufficienttimetillalltheairhasbeenexpelledout.
    iii. By applying a vacuum pressure of about 700mm ofmercury throughthe drainage cap for about 1.5 minutes after closing the drainagevalve. Then the soil is saturated by allowingdeaired water to enterfromdrainagebase.Theair-releasevalveiskeptopenduringsaturationprocess.
9. Afterthesoilsamplehasbeensaturated,theconstant-headreservoirisconnectedtothedrainagecap.
10. Water is allowed to flow out from the drainage base for some time till a steady-stateisestablished.

11. Thewaterlevelintheconstant-headchamberinwhichthemouldisplacediskeptconstant.
12. Thechamberisfilledtothebrimatthestartoftheexperiment.
13. Thewaterwhichentersthechamberafterflowingthroughthesamplespillsoverthechamberandcollectedinagraduatedjarforconvenientperiod.
14. Theheadcausingflow(h)isequaltothedifferenceinwaterlevelsbetweentheconstant-headreservoirandtheconstant-headchamber.
15. Ifthecross-sectionalareaofthespecimenisA,thedischargeisgivenby

$$q=kiA$$
$$or, q=k(h/l)A$$
$$or, k= (ql)/(A\ h)$$

where,L=Lengthofspecimen,h=headcausingflow.

Thedischargeqisequaltothevolumeofwatercollecteddividedbytime. Thefinerparticlesofsoilspecimenhaveatendencytomigratetowardstheendfaceswhenwaterflowsthroughit.Thisresultsintheformationofafilterskinattheends.Thecoefficientofpermeabilityoftheseendportionsisquitedifferentfromthatofmiddleportion.Formoreaccurateresults,itwouldbepreferabletomeasurethelossofheadh'overalengthL'inthemiddletodeterminethehydraulicgradient(i).Thus$i=h'/L'$.

The temperature of permeating water should be preferably somewhat higherthan that of the soil sample. This will prevent release of air during the test. It also helpsin removing the entrapped air in the pores of soil. As the water cools, it has tendency toabsorbair.

Toreducethechancesofformationoflargevoidsatthepointswheretheparticles of soil touch the permeameter walls, the diameter of the permeameter is keptatleast 15to20timestheparticlessize.

Toincreasetherateofflowforthesoilsoflowpermeability,agasunderpressureis applied to the surface of water in the constant-head reservoir. The total head causingflowinthatcaseincreaseto($h+p/\gamma_w$),wherepispressureapplied.

The bulk density of the soil in the mould may be determined from the mass ofsoil in the mould and its volume. The bulk density should be equal to that in the field.Theundisturbedsamplecanalsobeusedinsteadofthecompactedsample.For accurateresults,the specimen shouldhave thesame structureasinnaturalconditions.

Theconstantheadpermeabilitytestissuitableforcleansandandgravelwith$k>10^{-2}$mm/sec.

5.3.2-FallingHead/VariableHeadPermeabilityTest:-

Forrelativelylesspermeabilitysoilsthequantityofwatercollectedinthegraduatedjaroftheconstant-headpermeabilitytestisverysmallandcannotbeaccurately. For such soils, the variable-head permeability test is used.teh permeametermouldisthesameasusedintheconstant-headpermeabilitytest.

1) Avertical,graduatedstandpipeofknowndiameterisfittedtothetopofpermeameter.

2) Thesampleisplacedbetweentwoporousdiscs.
3) The whole assembly is placed in a constant head chamber filled with water tobrimatthestartofthetest.(Seethebelowfigureshowsaschematicsketch).

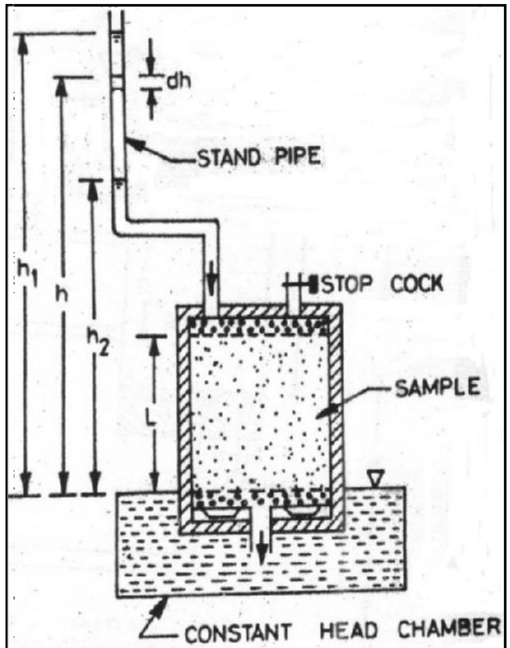


Figure:- Variable Head Permeameter.

4) Tee porous discs and water tubes be de-aired before the sample is placed. If in-situ, undisturbed sample is available, the same can be used; otherwise the soil istakeninthemouldandcompactedtorequireddensity.
5) The valve at the drainage base (not shown in figure) is closed and vacuumpressureisappliedslowlythroughthedrainagecaptoremoveairfromthesoi l.
6) The vacuum pressure is increased to 700 mm of mercury and maintained forabout15minutes.
7) The sample is saturated by allowing deaired water to flow upward from thedrainagebasewhenunder vacuum.
8) Whenthesoilissaturated,boththetopandbottomoutletsareclosed.
9) Thestandpipeisfilledwithwatertorequiredheight.
10) The test is stated by allowing the water in the stand pipe to flothrough thesampletoconstant-headchamberfromwhichitoverflowsandspillsout.
11) Asthewaterflowsthroughthesoil,thewaterlevelinthestandpipefalls.
12) The time required for the water level to fall from a known initial head (h_1) toknownasfinalhead(h_2)isdetermined.
13) Theheadismeasuredwithreferenceto thelevelof water in the constant-headchamber.

Letusconsidertheinstantwhentheheadis$h$.Fortheinfinitesimalsmalltimedt, theheadfallsbydh.
Letthedischargethroughthesamplebe$q$.

Fromcontinuityofflow, *adh=-qdt*

Where'*a*'iscross-sectionalareaofstandpipe.

Or, *a dh = - (A* X *k* X *i)* X *dt*Or, *adh=-A*X*k*X*h/L*X*dt*Or, *Akdt/aL =-dh/h*

Integrating, $\frac{AK}{aL}\int_{t1}^{t2} dt=-\int_{h1}^{h2} \frac{dh}{h}$

$\frac{Ak}{aL}(t2-t1)=\log_e(h_1/h_2)$

$k=\frac{Ak}{aL}\log_e(h_1/h_2)$

Where,*t=(t2–t1),*thetimeintervalduringwhichtheheadreducesfrom$h_1$toh_2).

Sometime$k=\frac{2.30\ a}{LAt}\log_{10}(h_1/h_2)$

Therateoffallofwaterlevelinthestandpipeandtearteofflowcanbeadjustedby changing the area of cross-section of the standpipe. The smaller diameter pipes arerequiredforlessperviоussoils.

Thecoefficientofpermeabilityisisreportedat27°CasperIS:2720(PartXVII). Thevoidratiosoil isalsogenerallydetermined.

Thevariableheadpermeameterissuitableforveryfinesandandsiltwithk=10^{-2}to10^{-5}mm/sec.

Sometime, the permeability test is conducted using the consolidometer instead of thepermeametermould.Thefixed-ringconsolidometerisusedavariable–headpermeameterbyattachingastand.

5.4 .1-SeepagePressure:-

As water flows through the soil, it extends a force on the soil. The force acts inthe direction of flows. This force is known as drag force or seepage force. The pressureinducedinthesoilisknownasseepagepressure.

Or

By virtue of the viscous friction exerted on water flowing through soil energytransfer is effected between the water and soil. The force corresponding to this energytransfer is called the seepage force or seepage pressure.Thus,seepage pressure is thepressureexertedbywateronthesoilthroughwhichpercolates.

5.4.2 -ThephenomenonofQuickSand:-

When flow takes place in an upward direction, the seepage pressure also acts inthe upward direction and the effective pressure is reduced. If the seepage pressurebecomesequaltothepressureduetosubmergedweightofthesoil,theeffectivepressure is reduced to zero, in such case, a cohesionless soil loses all its shear strength,andthesoilparticleshaveatendencytomoveupinthedirectionofflow.Thisphenomenonofliftingofsoilparticlesiscalled*quickcondition*,boilingconditionor quicksand.Thus,duringthequickcondition,

$$\sigma' = z\gamma' - p_s = 0$$

$$or, \quad p_s = z\gamma'$$

$$or \quad iz\gamma_w = z\gamma'$$

Fromwhich,

$$I = i_c = \gamma'/\gamma_w = \frac{G-1}{1+e}$$

- Thehydraulicgradientatsuchacriticalstateiscalledhydraulicgradient. For loose deposits of sand or sand or silt, if voids ratio e istaken as 0.67 and G as 2.67, the critical hydraulic gradient works outto beunity.
- Itshould benotedthat quicksand isnot a type of sandbut a flowconditionoccurringwithinacohesionlesssoilwhenitseffectivepressureis reducedtozeroduetoupwardflowofwater.

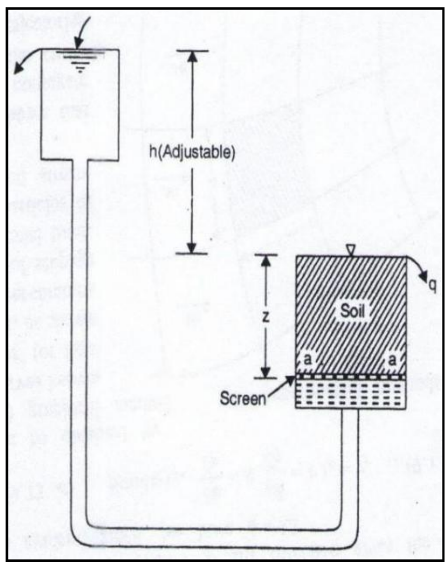


Figure:-QUICKSANDCONDITION

1. Thefigureshowsaset-uptodemonstratethephenomenonofquicksand.
2. Waterflowsinanupwarddirectionthroughasaturatedsoilsampleofthickness*'z'* underahydraulichead*'h'.*
3. This head can be increased or decreased bymoving the supply tank in theupwardordownwarddirection.
4. When the soil particles are in the state of critical equilibrium, the total upwardforce at the bottom of soil becomes equal to the total weight of all the materialsabovethesurfaceconsidered.

Equatingtheupwardanddownwardforcesatthelevel*a-a,*wehave,

$$(h+z)\gamma_w A = z\gamma_{sat} A$$

$$h\gamma_{w}=z(\gamma_{sat}-\gamma_{w})=z\gamma'$$

$$\frac{h}{z}=i_{c}=\gamma'/\gamma_{w}=\frac{G-1}{1+e}$$

5.5.1-ConceptofFlow-Net:-

1. Thegraphicalmethodofflownet construction,firstgivenbyForchheimer(1930),isbasedontrailsketching.
2. Thehydraulicboundaryconditionshaveagreateffectonthegeneralshapeoftheflow net,andhencemustbeexaminedbeforesketchingisstarted.
3. The flow net can be plotted by trial and error by observing the properties offlownetandbyfollowingpracticalsuggestiongiven byA.Casagrande.

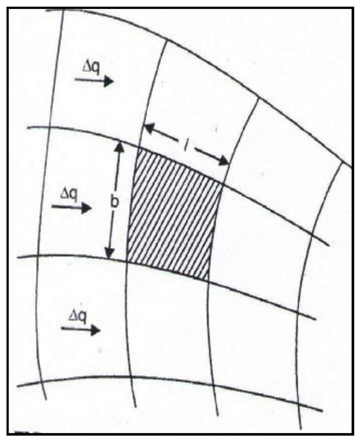


Figure:-PORTIONOFAFLOWNET

5.5.2-PropertiesofFlow-Net:-

1. Theflowlinesandequipotentiallinemeetatrightanglestooneanother.
2. Thefieldsareapproximatelysquares,sothatacirclecanbedrawntouchingallthefour sidesofthesquare.
3. Thequantityofwaterflowingthrougheachflowchannelissame,similarly,thesamep otentialdropoccursbetweentwosuccessiveequipotentiallines
4. Smallerthedimensionsofthefield,greaterwillbethehydraulicgradientandvelocity throughit.
5. In a homogeneous soil, every transition in the shape of the curves is smooth,beingeitherellipticalinshape.

5.5.3-ApplicationofFlow-Net:-

Aflownetcanbeutilizedforthefollowingpurposes:-

a) Determinationofseepage.
b) Determinationofhydrostaticpressure.
c) Determinationofseepagepressure.

d) Determinationofexistgradient.

(a) Determination of seepage:- Figure shows a portion of flow net. The portionbetween any two successive flow lines is known as flow channel. The portion enclosedbetween two successive equipotential lines and successive flow linesis known as fieldasthatshownhatchedinthefigure

Letb&lbethewidthandlengthofthefield

Δh=headdropthroughthefield

Δq=Dischargepassingthroughtheflowchannel

H=totalhydraulicheadcausingflow=differencebetweenupstreamanddown streamheads.

$$b = k H \ (N_f /$$

N_d),Where,N_f=Totalnumberofflowchannelin thenet

N_d=Totalnumberofpotentialdropsinthecompletenet

This is required expression for the discharge passing through a flow-net andvalidforisotropicsoilsinwhich$k_x=k_y=k$.

(b) Determination of hydrostatic pressure:- The hydrostatic pressure at anypointwithinthesoil mass isgivenby= $u=\gamma_w h_w$

Where$u$=hydrostaticpressure,h_w=Pizometrichead

ThehydrostaticpressureintermsofPizometrichead$h_w$iscalculatedfromtherelation –

$$h_w = h - Z$$

Whereh=Hydraulicpotentialatthepointunderconsideration.

Z= position head of the point above datum, consideredpositiveupwards.

(c)Determination of seepage pressure: - The hydraulic potential *h* at anypointlocatedafter*n*potentialdrops,eachvalue Δ his givenby

$$h = H - n\Delta h$$

the seepage pressureatany pointequals thehydraulicpotentialor balancedhydraulicheadmultipliedbyunitweightofwaterandhenceisgivenby$p_s = h\gamma_w = (H - n\Delta h)\gamma_w$

Thepressureactsinthedirectionofflow.

(d) Determination of exist gradient:- The exit gradient is hydraulic at thedownstream end of the flow line where the percolating water leaves the soilmass and emerges into the free water at the downstream. The exit gradientcan be calculated from the following expression, in which Δ h represents thepotentialdropand*l*theaveragelengthoflastfieldintheflownetatexitend:

$i_e = (\Delta h / l)$.

CHAPTER-6

COMPACTIONANDCONSOLIDATION

6.1 COMPACTION

Compactionistheapplicationofmechanicalenergytoasoilsoastorearrangeitsparticlesand reduce thevoidratio.
It is applied to improve the properties of an existing soil or in the process of placing fillsuch as in the construction of embankments, road bases, runways, earth dams, andreinforced earth walls. Compaction is also used to prepare a level surface duringconstructionofbuildings.Thereisusuallynochangeinthewatercontentandinthesizeofthein dividualsoilparticles.

Theobjectivesofcompactionare:

- Toincreasesoilshearstrengthandthereforeitsbearingcapacity.
- Toreducesubsequentsettlementunderworkingloads.
- Toreducesoilpermeabilitymakingitmoredifficultforwatertoflowthrough.

LIGHTANDHEAVYCOMPACTIONTEST

LaboratoryCompaction

Thevariationincompactionwithwatercontentandcompactiveeffortisfirstdeterminedinthela boratory.Thereareseveraltestswithstandardproceduressuchas:

- IndianStandardLightCompactionTest(similartoStandardProctorTest)
- IndianStandardHeavyCompactionTest(similartoModifiedProctorTest)

IndianStandardLightCompactionTest

Soiliscompactedintoa1000cm^{3}mouldin3equallayers,eachlayerreceiving25blowsofa2.6k grammerdroppedfromaheightof310mmabovethesoil.Thecompactionisrepeatedatvariou smoisturecontents.

IndianStandardHeavyCompactionTest

It was found that the Light Compaction Test (Standard Test) could not reproduce thedensities measured in the field under heavier loading conditions, and this led to thedevelopment of the Heavy Compaction Test (Modified Test). The equipment andprocedure are essentially the same as that used for the Standard Test except that thesoiliscompactedin5layers,eachlayeralsoreceiving25blows.Thesamemouldisalsoused. To provide the increased compactive effort, a heavier rammer of 4.9 kg and agreaterdropheightof450mmareused.

OPTIMUM MOISTURE CONTENT OF SOIL,MAXIMUM DRY DENSITY,ZEROAIRVOIDLINE

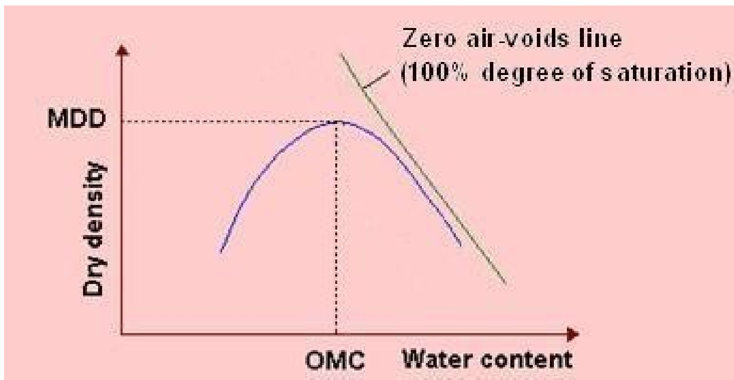
Toassessthedegreeofcompaction,itisnecessarytousethedryunitweight,whichisan indicator of compactness of solid soil particles in a given volume. The laboratorytesting ismeanttoestablishthemaximumdrydensitythatcanbeattainedforagivensoilwithastandarda mount ofcompactiveeffort.

Inthetest,thedrydensitycannotbedetermineddirectly,andassuchthebulkdensity andthemoisturecontentareobtainedfirsttocalculatethedrydensityas $\gamma_d = \frac{\gamma_t}{1+w}$

γ_t ,where

=bulkdensity,andw=watercontent.

Aseriesofsamplesofthesoilarecompactedatdifferentwatercontents,andacurveisdrawnwithaxesofdrydensityandwatercontent.Theresultingplotusuallyhasadistinct peak as shown. Such inverted “V” curves are obtained for**cohesive soils** (orsoilswithfines),andareknownascompactioncurves.



Drydensitycanberelatedtowatercontentanddegreeofsaturation(S)as

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

Thus,itcanbevisualizedthatanincreaseofdrydensitymeansadecreaseofvoidsratioandamorecompactsoil.

Similarly,drydensitycanberelatedtopercentageairvoids(n_a)as

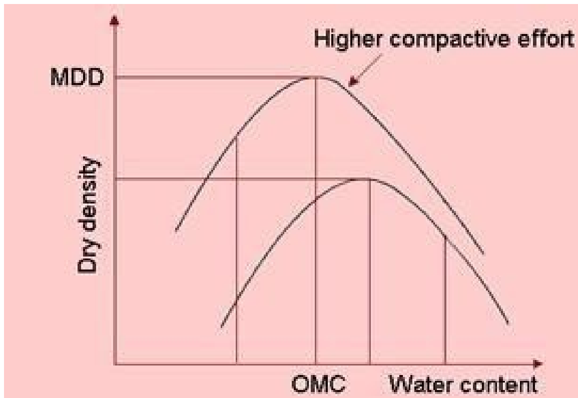
$$\gamma_d = \frac{(1-n_a)G_s.\gamma_w}{1+wG_s}$$

Therelationbetweenmoisturecontentanddryunitweightforasaturatedsoilisthe**zeroair-voids line.** It is not feasible to expel air completely by compaction, no matter howmuchcompactiveeffortisusedandinwhatevermanner.

EffectofIncreasingWaterContent
Aswaterisaddedtoasoilatlowmoisturecontents,itbecomeseasierfortheparticlestomove past one another during the application of compacting force. The particles comecloser, the voids are reduced and this causes the dry density to increase. As the watercontentincreases,thesoilparticlesdeveloplargerwaterfilms aroundthem.

This increase in dry density continues till a stage is reached where water startsoccupyingthespacethatcouldhavebeenoccupiedbythesoilgrains.Thusthewateratthisstagehindersthecloserpackingofgrainsandreducesthedryunitweight. The**maximumdrydensity(MDD)**occursatan**optimumwatercontent(OMC)**,andtheirvaluescanbeobtainedfromtheplot.

EffectofIncreasingCompactiveEffort
The effect of increasing compactive effort is shown. Different curves are obtained fordifferentcompactiveefforts.Agreatercompactiveeffortreducestheoptimummoisturecontentandincreases themaximumdrydensity.



An increase in compactive effort produces a very large increase in dry density for soilwhenitiscompactedatwatercontentsdrierthantheoptimummoisturecontent.Itshouldbe noted that for moisture contents greater than the optimum, the use of heaviercompactioneffortwillhaveonlyasmalleffectonincreasingdryunitweights.

Itcanbeseenthatthecompactioncurveisnotauniquesoilcharacteristic.Itdependsonthe compaction effort. For this reason, it is important to specify the compaction procedure(lightor heavy)whengivingvalues ofMDD andOMC.

6.2 FactorsAffectingCompaction
Thefactorsthatinfluencetheachieveddegreeofcompactioninthelaboratoryare:

- Plasticityofthesoil
- Watercontent
- Compactiveeffort

6.3 FIELDCOMPACTIONMETHODSANDTHEIRSUITABILITY

CompactionEquipment
Mostofthecompactioninthefieldis donewithrollers.Thefourmostcommontypesofrollersare
1. Smooth-wheelrollers(orsmooth-drumrollers)
2. Pneumaticrubber-tiredrollers
3. Sheepsfootrollers
4. Vibratoryrollers

Smooth-wheelrollersaresuitableforproofrollingsubgradesandfor finishingoperationoffillswithsandyandclayeysoils.Theserollersprovide100%coverageunder the wheels, with ground contact pressures as high as 310 to 380 kN/m2 (45 to 55lb/in2). They are not suitable for producing high unit weights of compaction when used onthickerlayers. Pneumaticrubber-tiredrollersarebetterinmanyrespectsthanthe smooth-wheelrollers. Theformer areheavilyloadedwithseveral rows oftires. Thesetiresare closely spaced—four to six in a row. The contact pressure under the tires can range from600 to 700 kN/m2 (85 to 100 lb/in2), and they produce about 70 to 80% coverage. Pneumaticrollerscanbeusedforsandyandclayeysoilcompaction.Compactionisachievedbyacombinationofpressureand kneadingaction.

Sheepsfootrollersaredrumswithalargenumberofprojections.Thearea ofeachprojectionmayrangefrom25to85cm^2(_4to13in^2).Theserollersaremosteffectivein compacting clayey soils. The contact pressure under the projections can range from 1400to 7000 kN/m^2 (200 to 1000 lb/in^2). During compaction in the field, the initial passes compactthelowerportionofalift.Compactionatthetopandmiddle ofaliftisdoneatalaterstage. Vibratoryrollersareextremelyefficientincompactinggranularsoils.Vibratorscan beattachedtosmooth-wheel,pneumaticrubber-tired,orsheepsfootrollerstoprovidevibratoryeffectstothesoil.

.

Handheldvibratingplatescanbeusedforeffectivecompactionofgranularsoilsover alimitedarea.Vibratingplatesarealsogang-mountedonmachines.Theseplatescanbeusedinlessrestrictedareas..

6.4 CONSOLIDATION:

According to Terzaghi (1943), “a decrease of water content of a saturated soilwithout replacement of the water by air is called a process of consolidation.” Whensaturatedclayeysoils-whichhavealowcoefficientofpermeability-aresubjectedtoacompressive stress due to a foundation loading, the ore water pressure willimmediatelyincrease;however,duetothelowpermeabilityofthesoil,therewillbeatimelagbetweentheapplicationofloadandtheextrusionoftheporewaterand,thus,thesettlement.

<u>DIFFERENCEBETWEENCOMPACTIONANDCONSOLIDATION</u>:

Consolidation and compaction are totally different process. Though both processresults a reduction in volume, it is important to know the difference between them.These are:

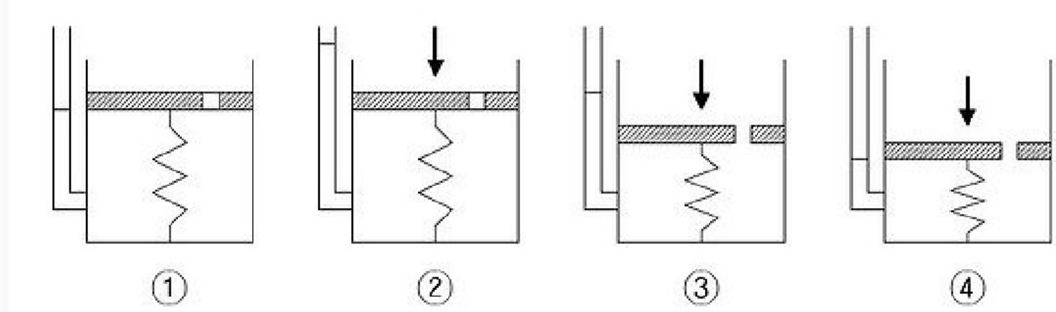
a. Compaction reduces volume of soil by rapid mechanical methods like tamping,rolling and vibration; whereas consolidation process reduces volume gradually bystatic, sustainedloading.

b. Compaction decreases volume by expelling air from partially saturated or drysoil; whereas consolidation process reduces volume by squeezing out water fromsaturated soil.Incompactionprocesswatercontentisnotaltered.

c. Compactionisahumangeneratedpressingmethodtoproducehighunitweightof soil. Thus increasing other properties to have better founding soil. In contrast,consolidation is natural process where volume of saturated soil mass reduced bystatic loads from the weight of building or other structures that is transferred tosoilthroughafoundationsystem.

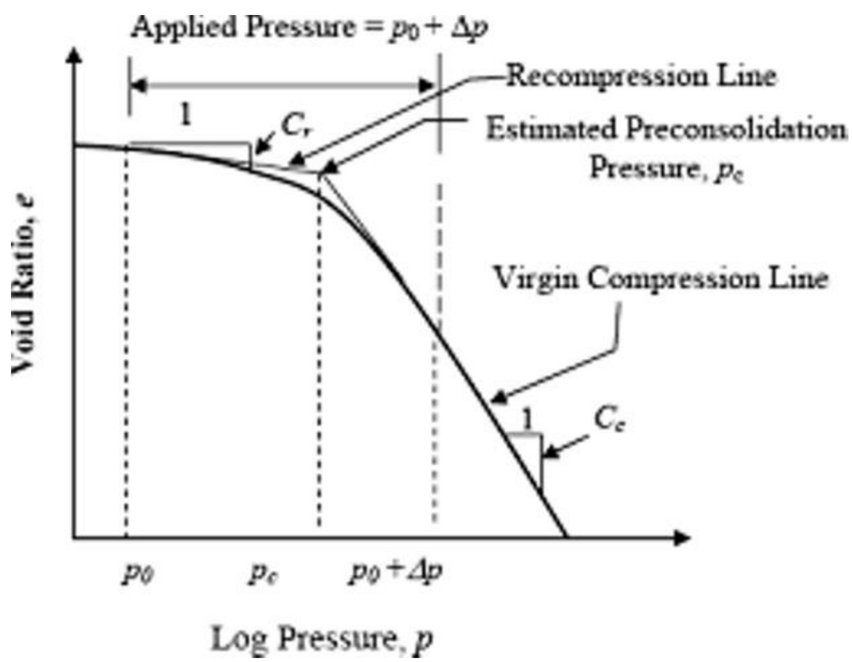
6.5 SPRINGANALOGYMETHOD

Theprocess of consolidation is often explainedwith an idealized system composedofa spring, a container with a hole in its cover, and water. In this system, the springrepresents the compressibility or the structure of the soil itself, and the water whichfillsthecontainerrepresentstheporewaterinthesoil.



1. The container is completely filled with water, and the hole is closed. (Fullysaturatedsoil)
2. A load is applied onto the cover, while the hole is still unopened. At this stage,only the water resists the applied load. (Development of excess pore waterpressure)
3. As soon as the hole is opened, water starts to drain out through the hole andthespringshortens.(Drainageofexcessporewaterpressure)
4. After some time, the drainage of water no longer occurs. Now, the spring aloneresists the applied load. (Full dissipation of excess pore water pressure.End ofconsolidation)

Pressure-VoidRatioCurve:-



Normallyconsolidated,UnderconsolidatedandOverconsolidatedsoil,

Consolidation is a process by which soils decrease in volume. According to Karl vonTerzaghi "consolidation is any process which involves a decrease in water content ofsaturated soil without replacement of water by air." In general it is the process inwhich reduction in volume takes place by expulsion of water under long term staticloads. It occurs when stress is applied to a soil that causes the soil particles to packtogethermore tightly,therefore reducingits bulkvolume.When thisoccurs inasoil that is saturated with water, water will be squeezed out of the soil. The magnitude ofconsolidation can be predicted by many different methods. In the Classical Method,developed by Terzaghi, soils are tested with an oedometer test to determine theircompressionindex.Thiscanbeusedtopredicttheamountofconsolidation.

When stress is removed from a consolidated soil, the soil will rebound, regainingsome of the volume it had lost in the consolidation process. If the stress is reapplied,thesoilwillconsolidateagainalongarecompressioncurve,definedbytherecom pression index. The soilwhichhaditsloadremovedisconsideredtobeoverconsolidated. This is the case for soils which have previously had glaciers onthem. The highest stress that it has been subjected to is termed the preconsolidationstress.TheoverconsolidationratioorOCRisdefinedasthehigheststresse xperienced divided by the current stress. A soil which is currently experiencing itshighest stress is said to be normally consolidated and to have an OCR of one. A soilcould be considered underconsolidated immediately after a new load is applied butbeforetheexcessporewaterpressure has hadtimetodissipate.

AssumptionofTerzaghi'stheoryofone-dimensionalconsolidation

1. Thesoilishomogenous(uniformincompositionthroughout).
2. Thesoilisfullysaturated(zeroairvoidsduetowatercontentbeingsohigh).
3. Thesolidparticlesandwaterareincompressible.
4. Compressionandflowareone-dimensional(verticalaxisbeingtheoneofinterest).
5. Strainsinthesoilarerelativelysmall.
6. Darcy'sLawisvalidforallhydraulicgradients.
7. The coefficient ofpermeabilityand the coefficient of volumecompressibilityremainconstantthroughout theprocess.
8. Thereisauniquerelationship,independentoftime,betweenthevoidratioandeffective stress

CoefficientofConsolidation:

TheCoefficientofconsolidationateachpressuresincrementiscalculatedbyusingthefollo

wingequations:

i. $Cv=0.197d2/t_{50}$(Logfittingmethod)

Inthelogfittingmethod,aplotismadebetweendialreadingsandlogarithmicoftime,th etimecorrespondingto50%consolidationisdetermined

ii. $C_v=0.848d2/t_{90}$(Squarefittingmethod)

Inthesquarerootfittingmethod,aplotismadebetweendialreadingsandsquare root of time and the time corresponding to 90% consolidation isdetermined.

TimeFactor:-

ThemagnitudeofconsolidationsettlementisoftencalculatedusingTerzaghi'sexpression for average degree of consolidation (U) with respect to time. Developedduringatimeoflimitedcomputingcapabilities,Terzaghi'sseriessolutiontothe one-dimensional consolidation equation was generalized using a dimensionless time factor(T), where a single U-T curve is used to describe the consolidation behavior of bothsinglyanddoublydrainedstrata.Asaresult,anycomparisonsbetweenone-andtwo-way drainage are indirect and confined to discrete values of time. By introducing amodified time factor T∗ in terms of layer thickness (D) instead of the maximumdrainage path length (Hdr), it is now possible to observe the effect of drainageconditions over a continuous range of time for a variety of asymmetric initial excesspore pressure distributions. Although two separate U-T plots are required (for singlyanddoublydrainedcases),thetimefactoratspecifictimesremainsthesameforbothcases,enablingadirectvisualcomparison.Theimportanceofarevisedtimefactorisevidentwhenobservingtheendpointofconsolidation,whichoccursasUapproaches100%. This occursat T∗≅0.5 for two-waydrainage andat T∗≅2 for one-waydrainage,anobservationnotpossibleusingthetraditionalexpressionfortimefactor.

Estimationofconsolidationsettlements

Prediction of ground settlements have always been a big challenge for the engineersthatareresponsibleforthedesignofsubwaytunnelprojects.Sincegroundsettlementis a crucial concept directly affecting the successfulness of a project, it must be takenseriouslyandshould beaccuratelyestimated.

Categories:

1. <u>Immediatesettlement</u>-elasticdeformationofdrysoilandmoistandsaturatedsoilswithoutchangetomoisture content
   a. duetohighpermeability,porepressureinclayssupporttheentireaddedloadand noimmediatesettlementoccurs
   b. generally,duetotheconstructionprocess,immediatesettlementisnotimportant
2. <u>Primaryconsolidationsettlement</u>-volumechangeinsaturatedcohesivesoilsbecauseoftheexpulsionofwaterfromvoid spaces
   a. highpermeabilityofsandy,cohesionlesssoilsresultinnearimmediatedrainageduetotheincreaseinporewaterpressureandnoprimaryconsolidationsettlementoccur

DIFFERENCEBETWEENPRIMARYANDSECONDARYCOSOLIDATION

Primaryconsolidation

Thismethodassumesconsolidationoccursinonlyone- c
dimension.Labratorydataisusedtoconstructaplotofstrainorvoidratioversuseffectivestrss wherethe
effectivestressaxisisindexorrecompression c n a logarithmic scale. The plot's slope is the compressionindex.Theequationforconsolidationsettlementofa

normallyconsolidatedsoilcanthenbedeterminedtobe:

$$\delta_c = \frac{C_c}{1+e_0} H \log\left(\frac{\sigma'_{zf}}{\sigma'_{z0}}\right)$$

where

$\delta_c$isthesettlemetduetoconsolidation.$C_c$isthecompressionindex.

e_0 is the initial void ratio.Histheheightofthesoil.

σ_{zf} is the final vertical stress.σ_{z0}istheinitialverticalstress.

C_c can be replaced by C_r (the recompression index) for use in overconsolidated soilswherethefinaleffectivestressislessthanthepreconsolidationstress.Whenthefinaleffectivestressisgreaterthanthepreconsolidationstress,thetwoequationsmustbe usedin combinationtcompressionportionoft c modelboththe recompression portion d the virgin aeconsolidationprocesses,asfollows,

$$\delta_c = \frac{C_r}{1+e_0} H \log\left(\frac{\sigma'_{zc}}{\sigma'_{z0}}\right) + \frac{C_c}{1+e_0} H \log\left(\frac{\sigma'_{zf}}{\sigma'_{zc}}\right)$$

whereσ_{zc}isthepreconsolidationstressofthesoil.

Secondaryconsolidation

Secondaryconsolidationistheconsolidationofsoilthattakesplaceafterprimary consolidation.Evenafter thereductionofhydrostaticpressuresome compression csoiltakesplaceatslowrate.Thisisknownassecondaryconsolidation.Secondary d ry

consolidation is cause bycreep, viscous behavior of the clay- ater system, consolidationoforganicmatter,andotherprocesses.In sand,settlement causedbysecondarycompression is negligible, butinpeat,itis verysignificant. Due to secondary consolidation someofthehighlyviscouswaterbetween thepointsof contactisforcedout.

Secondaryconsolidationisgivenbytheformula

$$S_s = \frac{H_0}{1+e_0} C_a \log\left(\frac{t}{t_{90}}\right)$$

Where

H_0istheheightoftheconsolidating
medium$e_0$istheinitialvoidratio
$C_a$isthesecondarycompressionindex
t is the length of time after consolidation
consideredt_{90}isthelengthoftimeforachieving90%consolida
tion

CHAPTER-7

SHEARSTRENGTH

Introduction

Soil mass when loaded may fail due to shear stress induced in it. Examples of suchfailures are sinking of soil mass under heavily loaded foundation, spalling of soilalong the edge of vertical cut, slide in an earth embankment with a steep slopemovementofbackfillbehindaweakretainingwalletc.Inalltheabovecases,thesoilfai ls essentially due to shear. When the shear stress induced in a mass of soil reacheslimiting value, shear deformation occurs, which leads to the failure of soil mass. Theresistanceofferedbythesoiltoshearisknownasshearresistance.

The maximum shearing resistance of soil against continuous shear deformation alongpotential failure plane is known as **shear strength** of soil. The plane along whichfailure of soil takes place due to sliding is known as failure plane. Failure will takeplaceontheplaneonwhichtheshearstressexceedstheshearresistance.However,ifthe soil has weak planes, the failure will be located in the weakest zone. Failure maynot take place along the plane of maximum shear stress, i.e., the plane which makes45^0withtheprincipalplanes.

The shearing resistance ofsoil is composed of two components: Normal stressdependentandnormalstressindependent.Examplesoftheabovetwocasesare:

1. Frictionalresistancebetweentheparticlesatthepointofcontact
2. Cohesion or force of attraction between soil particles. It is characteristic of soilstateandisindependentofnormalstressacrosstheplane.

The above two components can be better understood by comparing two materials,sandandclay. Considerableforceisrequiredtosheara blockofclayasshownintheFigure 1(a) even when there is no external force acting on the block. This force ishigher when the block is dry and lower with increase in water content of the soilsample.Thiscomponentiscalledcohesion.Ontheotherhand,ifwetakeasampleofsandi nasplitmouldandtrytoshearit,theforcerequiredispracticallynilwhenthereisnoexternalnorm alforce.Now,ifweapplyexternalnormalpressure,theforce required to shear the sample increases and is proportional to the normal pressureapplied. Thiscomponentiscalledfriction.

Shear strength of the cohesionless soil results from inter granular friction as above.Plastic undrained clay does not have external friction. Hence, strength of soil resultsfrom cohesion alone. In other intermediate soils, shear strength of such soil resultsfrom internalfriction aswellas cohesion.

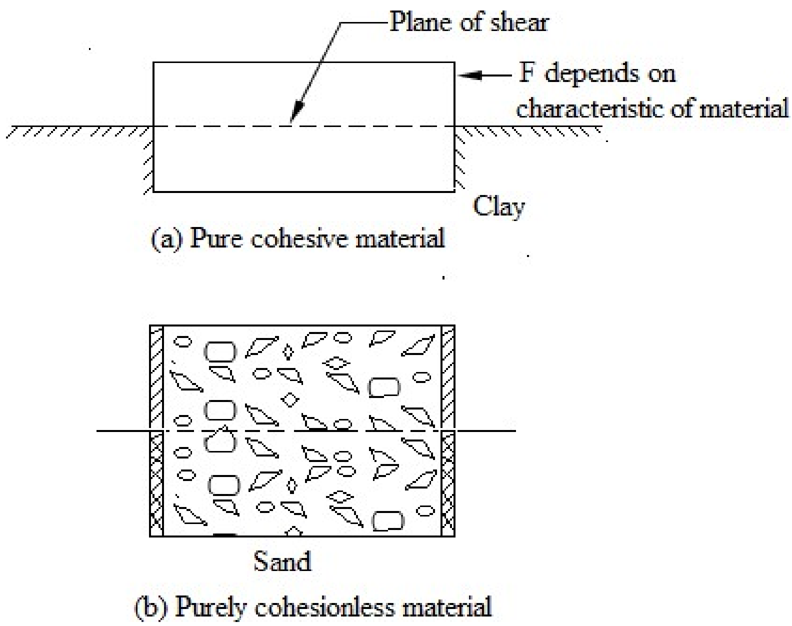


Figure1

Theoryoftwodimensionalstresssystem:Mohr'sStressCircle

Innumerableplanespassthrougheachpointinasoilmass.Thestresscomponentsoneachplanethroughthepointdependuponthedirectionoftheplane.Itisknownfromstrength of material that there exists three mutually perpendicular planes through apoint on which there is no shear stress and only normal stress acts. Such planes arecalledprincipalplanesandthenormalstresses,theprincipalstresses.Inorderoftheir

magnitude,thesestressesareknownasmajorprincipalstress(σ_1),intermediate principal stress $(\sigma_2)$and minor principal stress (σ_3). However, in most soil we dealwith, failure of soil mass is independent of intermediate stress. In such problems twodimensional stress analyses gives acceptable results for the solution of such failureproblems.

Consider the case of a soil element as in Figure 2 whose sides are principal planes i.e.,only normal stresses are acting on the faces of the element. The stress components at apointonagiveplane are givenby

$$\sigma=\frac{\sigma_1+\sigma_3}{2}+\frac{\sigma_1-\sigma_3}{2}\cos 2\alpha$$

$$\tau=\frac{\sigma_1-\sigma_3}{2}\sin 2\alpha$$

whereσandτarenormalstressandshearstresscomponentonaplaneinclinedatanangleofαwit hthemajorprincipalplane.

Theaboveresultscanberepresentedbydrawingacirclewithradius $\frac{\sigma_1-\sigma_3}{2}$ .The circle so drawn is known as Mohr's circle. Each point on the circumference of thecirclegivestwostress coordinatesatthatpointonaninclinedplane.

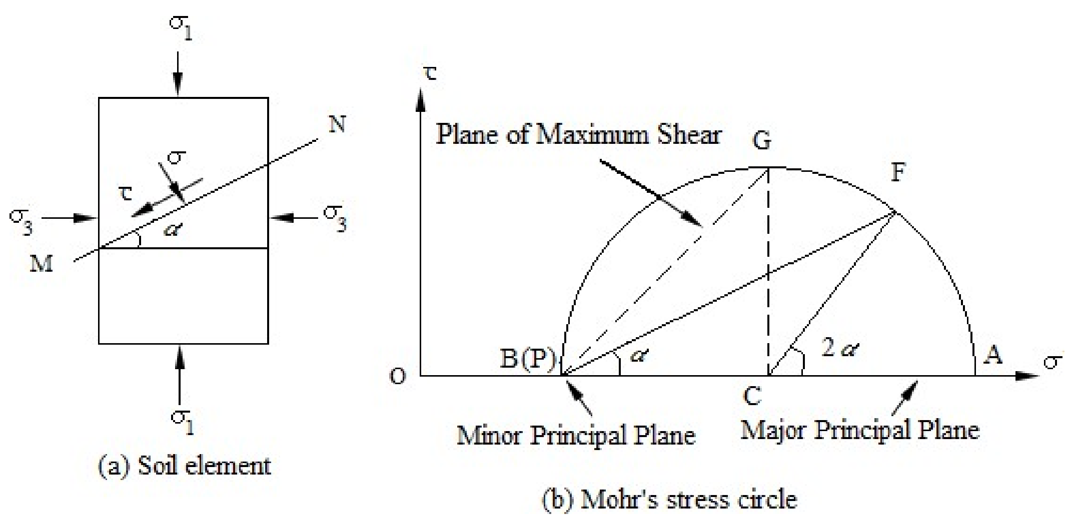


Figure2Mohr'sstresscircle

InFigure2(a),themajorprincipalplaneishorizontalandminorprincipalplaneisvertical.Point*A* intheMohr'scirclerepresentsthemajorprincipalstress$(\sigma_1,0)$and*B* representstheminorprincipalstress$(\sigma_3,0)$.Todeterminethestresscomponentsin aplanethroughthepoint,apointcalledpoleistobelocatedonthecircle.Thepoleis drawnbydrawingastraightlineparalleltotheplaneonwhich thestressconditions are known. Hence, the pole$P$ is located by drawing ahorizontal line through the pointA representing the major principal stress (σ_1).The pole can also be represented bydrawingaverticallinethrough$B$representingminorprincipalplane(σ_3).Toknow the stress on the inclined plane, a straight line PF parallel to the plane is drawnthroughthepole$P$.ThepointFonthecirclegivesthecoordinatesofthestressonthepla ne inclined at an angle $\alpha$with the direction of major principal plane. The shearstress is considered to be positive if its direction gives a clockwise moment about apointoutsidethewedgesuchaspoint$E$.

Consider another soil element as shown in Figure 3(a) in which major principal planesarenothorizontalandvertical,butareinclinedtoyandx-directions.ThecorrespondingMohr'sstresscircleisdrawnasshowninFigure3(b).PointArepresentsprincipalmajorprincipalstress$(\sigma_1,0)$andminorprincipalstress$(\sigma_3,0)$.To locatethepole,alineparalleltothemajorprincipalplaneisdrawnthrough$A$tointersect the circle at P. PB gives the direction of the minor principal plane.To determinethestresscomponentsonanyplaneMNinclinedatanangleαwiththe majorprincipalplane,alinemakingananangleofαwithPAisdrawnthrough$P$,to intersectthecircleat$F$.Thecoordinatesofpoint$F$givethestresscomponentsontheplaneMN.

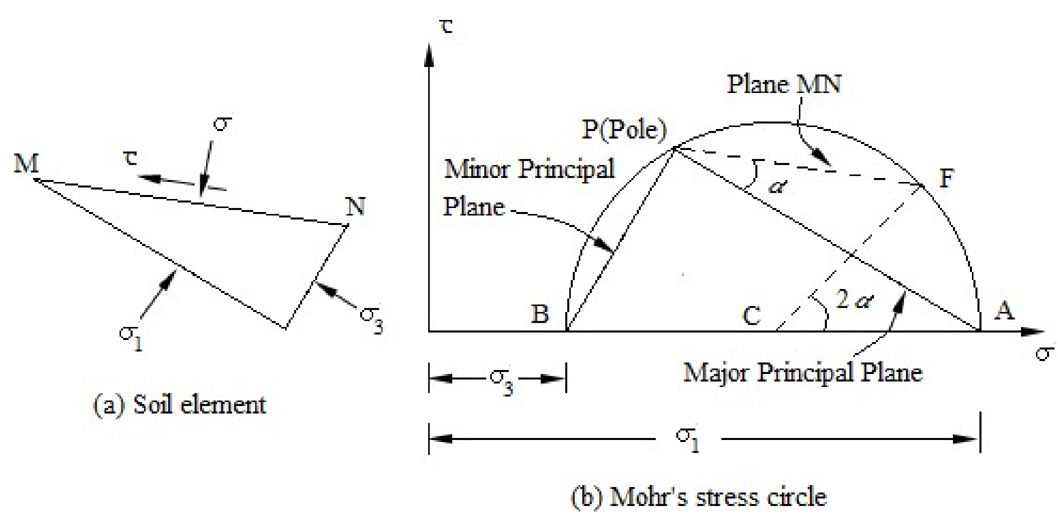


Figure3Mohr'sstresscircle

Mohr-CoulombTheoryofFailure

Various theories of failure of soil have been proposed by many soil scientists. Ofthese, the one proposed by Coulomb and generalised by Mohr has been the mostuseful for failure problems dealing with soil and hence has got wide acceptanceamongst the soil scientists. This failure theory is known as Mohr-Coulomb failuretheory.

AccordingtoMohr,thefailureofsolialongaplanetakesplacewhentheshearstresson that plane exceeds the shear resistance of the soil. The shear resistance is a functionthenormalstress onthefailureplane.Itisexpressed as

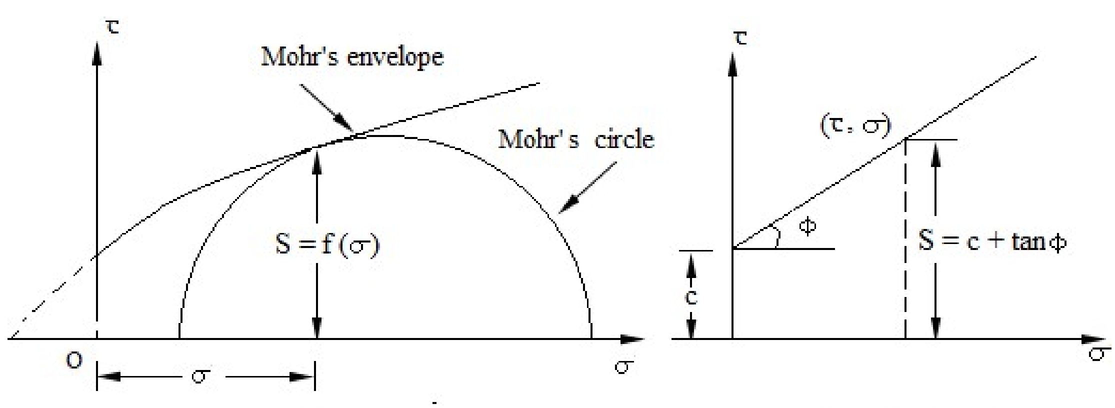
$$\tau_f = S = f(\sigma)$$

Where$\tau_f = S$=Shearstressatfailure=Shearresistance

Ifthenormalstressandshearstressareplotted,acurveisobtained.Thiscurveiscalled the shear envelope. Coulomb assumed the relationship between$\tau_f$and σaslinear andgavethe followingstrengthequation.

$$S = C + \sigma\tan\phi$$

For most of the cases of stability of soil, Mohr's failure can be approximated as astraight line for practical purposes and thus agrees with the above strength equationgivenbyCoulomb.



(a) Mohr's envelope (b) Coulomb's envelope

Figure4

C and ϕ in the expression $S = C + \sigma \tan\phi$ are empirical constants and are as cohesion and angle of friction or shearing resistance. In general the above constants are known as shear strength parameters.

Depending upon the nature of soil and the shear strength parameters, soils can be described as (i) cohesive soil, (ii) cohesion-less soil, and (iii) purely cohesive soil. The strength envelopes for the three cases are shown in the Figure 5.

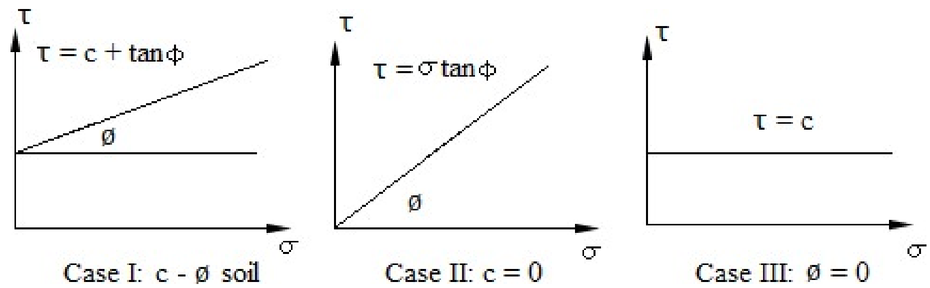


Figure 5 Strength envelopes for three types of soils

Effective stress principle

Extensive experimental studies on remoulded clays have shown the shearing strength of soil mass is controlled by the effective stress and not by the total normal stress on the plane of shear. The values of shear parameters, i.e., cohesion and angle of shearing resistance do depend upon the pore water pressure of the soil. Therefore, the Mohr-Coulomb strength equation may be expressed in terms of effective stress.

$$\tau_f = C' + \sigma' \tan\phi'$$

Where c' and ϕ' are termed as effective shear parameters. In terms of total stresses, the equation takes the form

$$\tau_f = C_u + \sigma \tan\phi_u$$

Where C_u is the apparent cohesion and ϕ_u is the apparent angle of shearing resistance

Determinationoffailureplane

Failure of soil may not take place along the plane of maximum shear stress. Thefailure will take place along the most dangerous plane called failure plane. The failureplaneistheoneonwhichthedifferencebetweenshearstrengthandshearstress,i.e., $(\tau_f-\tau)$isminimum.Todeterminetheangleoffailureplanewiththemajorprincipal plane,letusexpressthenormalstressσ'andshearstressτ'onanyplaneinclinedatanangleof$\alpha$to themajorprincipalplane.

$$\sigma'=\frac{\sigma'_1+\sigma'_3}{2}+\frac{\sigma'_1-\sigma'_3}{2}\cos 2\alpha$$

$$\tau'=\frac{\sigma'_1-\sigma'_3}{2}\sin 2\alpha$$

Theequationofshearstrengthisgivenby

$$\tau_f=C'+\sigma'\tan\phi'$$

$$=C'+\left[\frac{\sigma'_1+\sigma'_3}{2}+\frac{\sigma'_1-\sigma'_3}{2}\cos 2\alpha\right]\tan\phi'$$

So, $$(\tau_f-\tau)=C'+\frac{\sigma'_1+\sigma'_3}{2}\tan\phi'+\frac{\sigma'_1-\sigma'_3}{2}\cos 2\alpha\tan\phi'-\frac{\sigma'_1-\sigma'_3}{2}\mathrm{as}$$

Forminimumvalueof$(\tau_f-\tau)$, $\frac{d}{d\alpha}(\tau_f-\tau)=0$

Differentiating$(\tau-\tau_f)$withrespecttoα

$$\frac{d}{d\alpha}(\tau_f-\tau)=-(\sigma'_1-\sigma'_3)\sin 2\alpha\tan\phi'-(\sigma'_1-\sigma'_3)\cos 2\alpha$$

$$\cos 2\alpha=-\sin 2\alpha\tan\phi'$$

$$\cot 2\alpha=\cot(90+\phi')$$

$$\alpha=\alpha_f=45^0+\frac{\phi}{2}$$

where α_f is the angle of failure plane with respect to major principal plane.

The above expression for location of failure plane can be directly derived from Mohr'scircleshowninFigure6.*EF*representsthefailureenvelopegivenbythestraightline $\tau_f = C' + \sigma' \tan\phi'$.*P*isthepolewithstresscoordinates$(\sigma_3', 0)$.TheMohr'scircleis tangentialtotheMohrenvelopeatthepoint*F*.*PF*representsthedirectionoffailureplane,inclined at an angle $\alpha_f$with the direction of major principal plane. From the geometry ofFigure6, wegetfromtriangle*EFK*.

$$2\alpha_f = 90^0 + \phi'$$

$$\alpha_f = 45^0 + \frac{\phi'}{2}$$

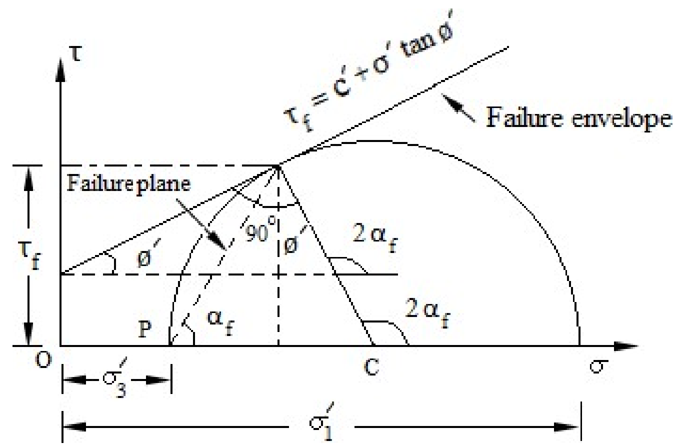


Figure6

Itmaybenotedthatanypointonthefailureenveloperepresentstwostresscomponents $\sigma'$and$\tau_f$atfailure.Andforeachσ'andτ_f,thereexiststwovaluesofprincipaleffectivestressontwoprincipalplanesforwhichfailuretakesplace.Itis evidentfromFigure5that$\tau_f$atfailureislessthanthemaximumshearstress,

correspondingtothepoint*G*,andactingontheplane*PG*.Thusthefailureplanedoesnot carry maximum shear stress, and the plane which has the maximum shear stress isnotthefailureplane.

Determinationofshearstrengthparameters

Sheartestsareconductedonundisturbedsoilsamplesobtainedfromthefield.Thetestresults areusedtoplotfailureenvelopetodeterminetheshearstrength parameters. It is to be noted that the shear strength parameters are fundamentalproperties of soil and are considered as coefficients obtained from the geometry of thestrength envelope drawn by using shear test results. So, during test on saturatedsamples,careshouldbetakentosimulatethefielddrainagecondition.

Followingfourmethodsofsheartestsarecommonlyusedinthelaboratory.

1. Directsheartest
2. Triaxialcompressiontest
3. Unconfinedcompressiontest
4. Vanesheartest

Basedonthedrainageconditions,thesheartestsareclassifiedas

1. Consolidateddrainedtest(Drainedtest/Quicktest)
2. Consolidatedundrainedtest
3. Unconsolidatedundrainedtest

1. Consolidateddrainedtest

Thisisalsoknownasdrainedtest.Inthistestdrainageofwaterisallowedduringthetest. The soil sample is first consolidated fully under the normal load (in direct sheartest) or the all round pressure (in triaxial test) and the shear stress is applied slowlyenough for the water in the sample to drain away. This simulates the long termconditions in the life of a structure, i.e., the long term stability of earth dam. Theeffectivestressparametersareused.

2. Consolidatedundrainedtest

In this test, the soil is consolidated under the normal load or the all-round pressure butshearing is done rapidly so that drainage does not take place. This simulates the suddeneffectsduringthelifeofastructure,e.g.,suddendrawdownofupstreamwaterlevelinane arth dam. The parameters used are C_u and ϕ_u. Ifporepressure measurements are madetheneffectivestressparameterscanbeused.

3. Unconsolidatedundrainedtest

In this case, the normal load or the all-round pressure as well as shear stress areapplied under conditions of no drainage. The soil is not consolidated and shearing isdonerapidly.Therefore,effectivestressesandhencetheshearstrengthofthesoildo not get mobilised. This simulates short term failure conditions in a structure, e.g.,stability of an earth dam immediately after construction. The total stress parametersareusedfor thesecases.

Directsheartest

The directshear testapparatusconsistsof(i)shearboxof squareorcircularsection, (ii)loadingyokeforapplyingnormalforce,(iii)gearedjackforapplyingshearforce, (iv) proving ring to measure shear force, and (v) strain gauges to measure horizontaldisplacementandverticaldisplacementforvolumechange.

The shear box consists of two halves which can slide relative to each other. The lowerhalf is rigidly held in position with the bottom ofthe shear box container, which slidesonrollerswhenpushedbyajackprovidedtoapplyshearforce.Thegearedjackmaybed riveneitherbyelectricmotororbyhand.Theupperhalfoftheboxisbuttagainstaprovingring .Thesoilsampleisplacedandcompactedintheshearbox.Thesampleheldinpositionbetwe enapairofmetalgridsandporousstonesorplatesasshownintheFigure7.Thegridplates,pro videdwithlinearslots,areplacedabovethetopandbelow the bottom of the specimen. To have proper grip with the soil specimen, thelinear slots in the grid plate are aligned perpendicular to the direction of the shearingforce. The soil specimen is compacted in shear box by clamping together with thehelp of two screws provided for the purpose. However, these screws are removedbefore shearing force is applied. Direct shear test may be oftwo types. Straincontrolled shear box and stress controlled shear box. The working principles of twotypesofshearboxareexplainedinthefollowing paragraphs.

In case of strain controlled shear test a normal load N is applied on the specimen bymeansofloadingyokeandiskeptconstantthroughoutthetest.Theshearingstrainisincre ased at a constant rate by pushing the lower box through the geared jack. Themovement of lower part of the box is transmitted through the specimen to the upperpart of the box. The proving ring attached to the upper part reads the shear force F atfailure. A number of tests are conducted on identical specimens with increased normalloads and the corresponding shear force recorded. A graph is plotted between theshear force F as ordinate and the normal load N as the abscissa. The plot so obtainedis knownasthefailureenvelope. Figure8(b)showsthefailureenvelopeplottedasa functionofshearstress(τ)andthenormalstress(σ).Thescaleofboth$^{\tau\text{and}\sigma\text{are}}$keptequaltomeas

uretheangleofshearingresistance(ϕ)directly fromtheplot.

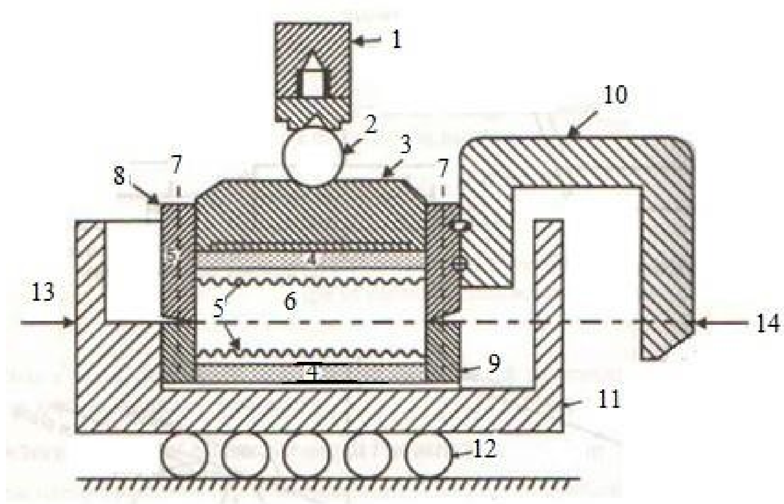


Figure7Shearboxwithaccessories

1.	Loadingyoke	8.	Upperpartofshearbox
2.	Steelball	9.	Lowerpartofshearbox
3.	Loadingpad	10.	U-arm
4.	Porousstones	11.	Containerforshearbox
5.	Metalgrids	12.	Rollers
6.	Soilspecimen	13.	Shearforceappliedbyjack
7.	Pinstofixtwohalvesofshearbox	14.	Shear resistance measured byprovingringdial gauge

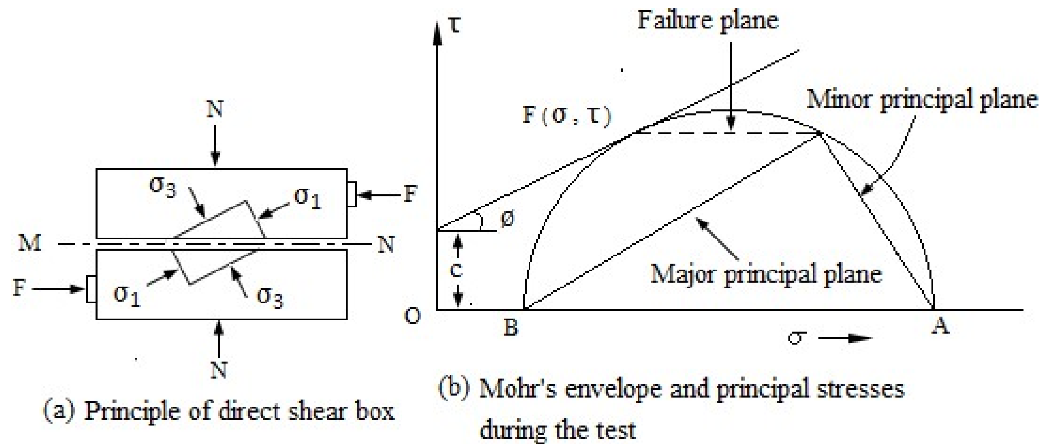


Figure8Shearboxtest

Advantagesofdirectsheartest

1. Directsheartestisasimpletestcomparedtothemorecomplextriaxialtest.
2. Asthethicknessofsampleissmall,itallowsquickdrainageand rapiddissipationofporepressureduring thetest.

Disadvantagesofdirectsheartest

1. The distribution of normal stresses and shear stresses over the potential failureplane is not uniform. The stress is more at the edges and less at the centre.Hence, progressive shear failure takes place as theshear strength is notmobilisedsimultaneouslyatallpointsonthefailureplane.
2. Thefailureplaneispredetermined,whichmaynotbetheweakestplane.
3. Theareaundersheargraduallydecreasesasthetestprogresses.Thecorrectedareaatfailure,$A_f$shouldbeusedforcomputingthevaluesofnormalstressσandshearstressτ.
4. Thereislittlecontrolonthedrainageofporewaterofsoilascomparedtothetriaxialtest.
5. Thestressonaccountoflateral restraintduetosidewallsofshearboxisnotaccountedfor inthetest.
6. Thereisnoprovisionformeasurementofporewaterpressure.

Problem1.

Fromadirect sheartesteston undisturbedsoil sample, following data havebeenobtained. Evaluate the undrained shear strength parameters. Determine shear strength,majorandminorprincipalstressesandtheirplanesinthecaseofspecimenofsample subjectedtoa normal stressof100kN/m^2.

Normalstress(kN/m^2)	70	96	114
Shear stress at failure(kN/m^2)	138	156	170

Solution.

Plot the shear stress versus normal stress to obtain the failure envelope keeping thescalesameforboththestresses.FromtheplotinFigure9,

Theangleofshearingresistance,ϕ=36^0;cohesion,c=84 kN/m^2

The shear strength corresponding to the normal stress of 100 kN/m^2 is 160 kN/m^2. Thecoordinate corresponding to (100, 160) is the failurepoint *F*. Draw theMohr's circle sothat the failure envelope is tangent to the circle at *F*. To do so, draw *FC* perpendicular tothe failure envelope. With *C* as centre and *CF* as radius, draw a circleso as to intersectthe normal load axis at *A* and *B*. Point A corresponds to the major principal stress σ_1=410kN/m^{2}andpointBcorrespondstotheminorprincipalstress σ_3=20kN/m^2.

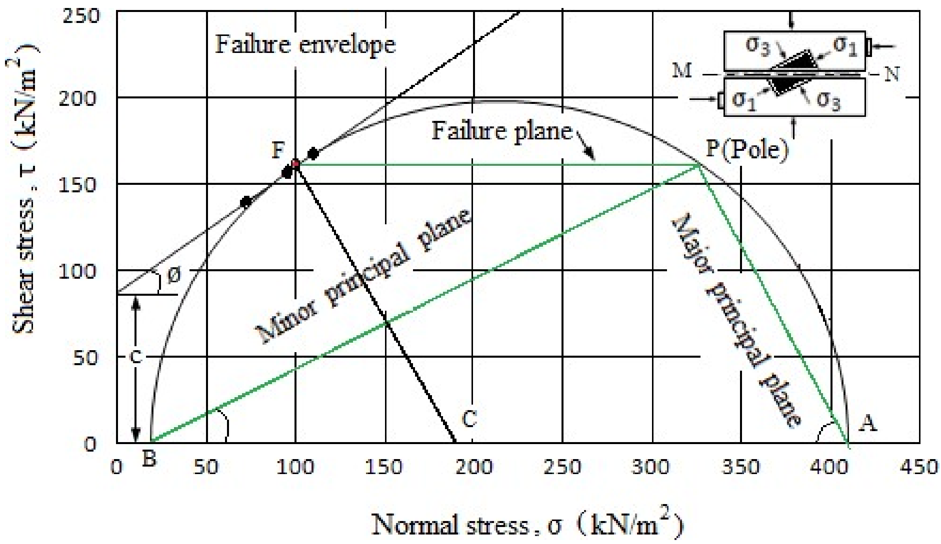


Figure9

To locate the position of the pole, draw a line *FP* parallel to the failure plane in theshear box (horizontal). *P* is the pole. *PA* is the direction of major principal plane whichmakes an angle 57 in the clockwise direction with the plane of shear. *PB* is theminorprincipal plane, making an angle of 58 in the anticlockwise direction with the planeofshearing.

Problem2.

A sample of cohesionless sand in a direct shear test fails under a shear stress of 160kN/m^2 when the normal stress is 140 kN/m^2. Find the angle of shearing resistance andtheprincipal stressatfailure.

Solution.

Plot the failure envelope passing through the origin and the point with coordinate(140,160)as normalstressandshearstresscoordinates.Thescaleforboththestressaxes arekept thesame.

FromtheplotintheFigure10,

Theangleofshearingresistance,ϕ=48.8^0;cohesion,c=0

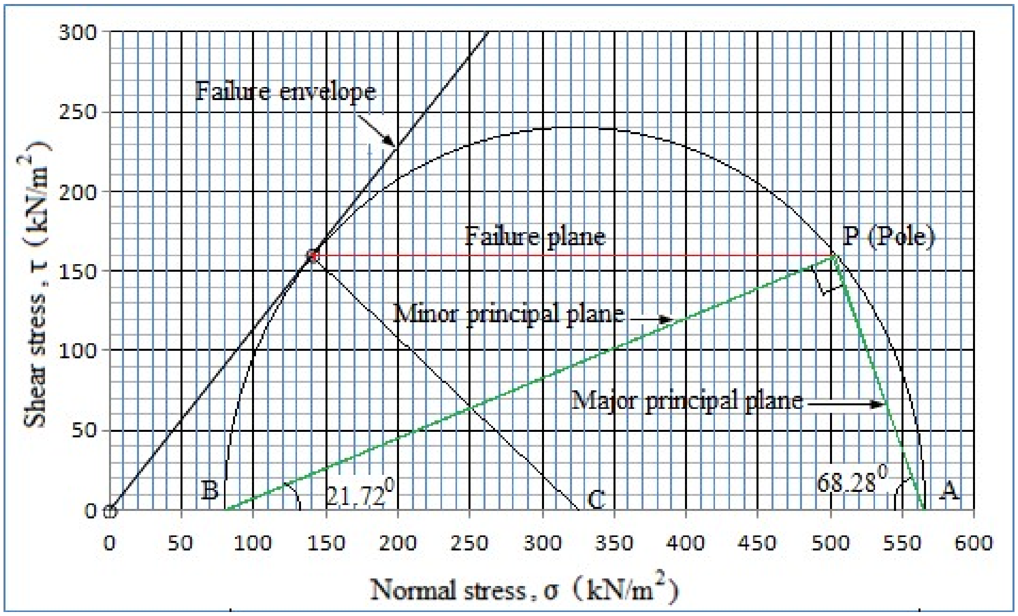


Figure10

DrawtheMohr'scirclesothatthefailureenvelopeistangenttothecircleat*F*.Todoso,draw*F* *C*perpendiculartothefailureenvelope.With*C*ascentreand*CF*asradius,drawacirclesoastointersectthenormalloadaxisat*A*and*B*.Point*A*correspondsto themajorprincipalstressσ_1 =565..81kN/m^2 andpoint*B*correspondstotheminor principalstressσ_3=80.35kN/m^2.

To locate the position of the pole, draw a line *FP* parallel to the failure plane in theshear box (horizontal). *P* is the pole. *PA* is the direction of major principal plane whichmakes an angle 68.28^0 in the clockwise direction with the plane of shear. *PB* istheminor principal plane, making an angle of 21.72^0 in the anticlockwise direction withtheplaneofshearing.

Problem3.

A cylinder of soil fails under an axial stress of 80 kN/m^2. The failure plane makes anangle of 48^0 with the horizontal. Calculate the value of cohesion and the angle ofinternalfriction ofthesoil.Verifybygraphical method.

Solution.

Asthereisonlyaxialstress,thereisnolateralstressactingonthesoil,i.e.,itis unconfinedcompressionfailure.Hence,minorprincipalstressprinci $\sigma_3=0$andmajor palstress$\sigma_1=60$.

And$\alpha=48^0$

Weknow, $\sigma_1=\sigma_3\tan^2\alpha+2c\tan\alpha$

$80=0\text{x}\tan^2\alpha+2c\tan48$

$80=2c\tan48$

$c=36.02\text{kN/m}^2$

Again, $\alpha=45^0+\frac{\phi}{2}$

$\phi=(\alpha-45)\text{x}2$

$\phi=(48-45)\text{x}2$

$\phi=6^0$

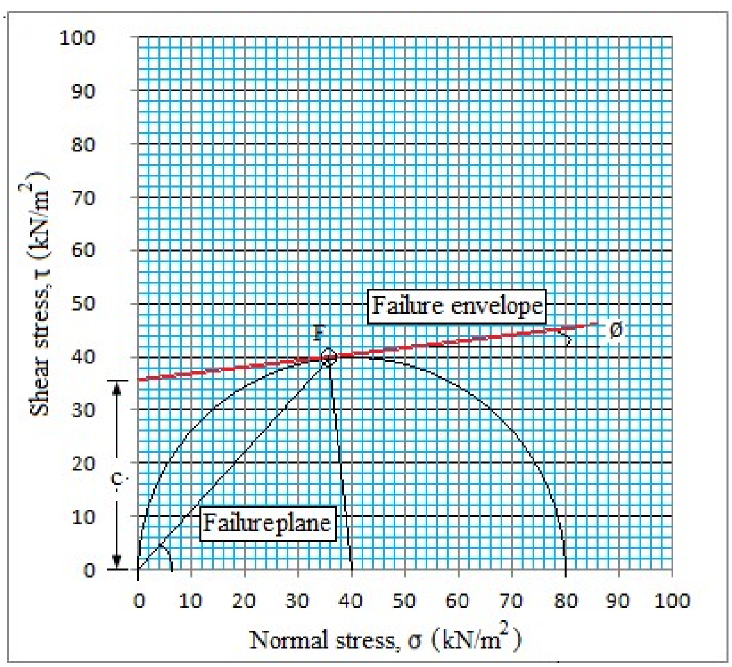


Figure11

TheMohr'sstresscircleisdrawnwithradius $\frac{\sigma_1-\sigma_3}{2}=\frac{80-0}{2}=40$.Thecirclepasses

throughtheoriginrepresentingtheminorprincipalstresswhichisalsothepole.Failure plane isdrawn through thepole*O* so asto intersect theMohr'scircle at*F*.Join *F* with the centre *C* of the Mohr's circle. Draw the failure envelope by drawing atangentat*F*onthecirclesoastointersectthey-axis.Theslope$\phi=6^0$of thefailure

plane is the angle of shearing resistance. The y-intercept c = 36.02 kN/m^2 is thecohesion.

Triaxialcompressiontest

Triaxialsheartestisthemostextensivelyusedforcomputationofshearstrengthparameters. In this test, the pecimen is compressed by applying all the three principalstresses,σ_1,$\sigma_2$andσ_3.

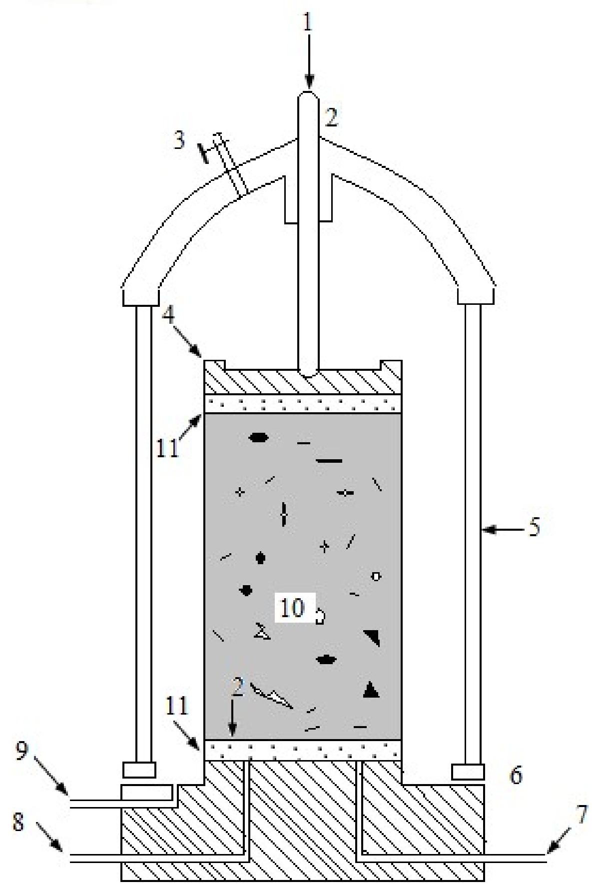


Figure12Triaxialcell

1. Axialloadmeasuredbyprovingrin gdialgauge
2. Loadingarm
3. Airreleasevalve
4. Topcap
5. Perspexcylinder
6. Sealingring
7. Porewateroutlet
8. Additionalporewateroutlet
9. Cellfluidinlet
10. Soilspecimenenclosedinrubber membrane
11. Porousdisc

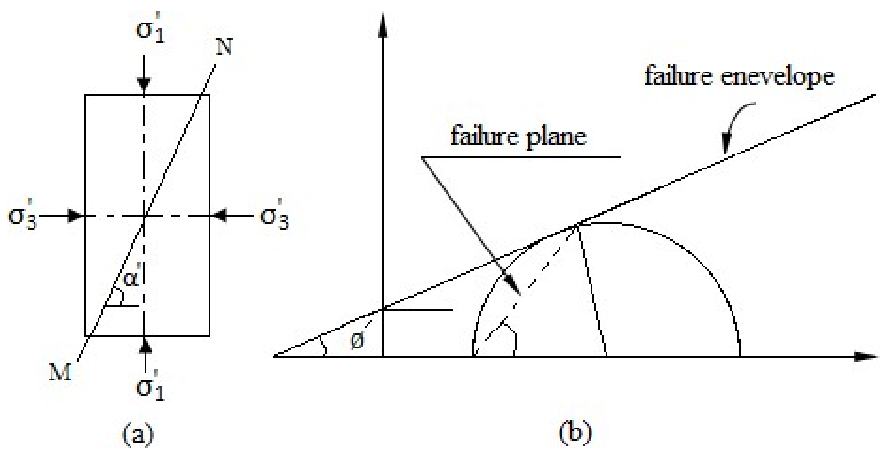
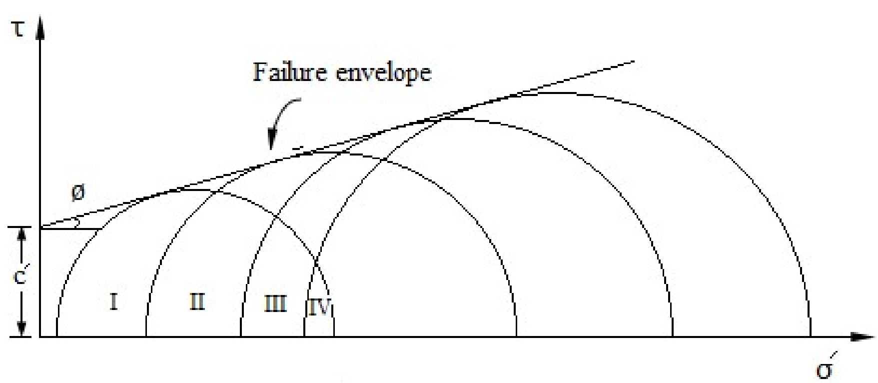


Figure13Stressconditionandfailureenvelopeintriaxialcompressiontest



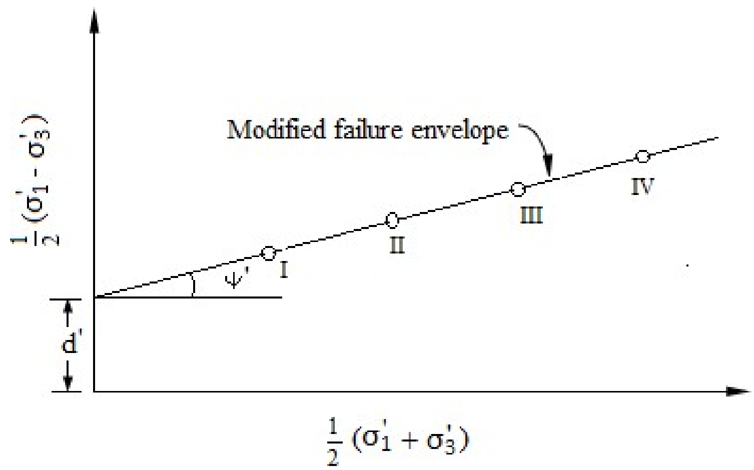


Figure14Failureenvelopes

Advantagesoftriaxialcompressiontest

Followingsaretheadvantagesoftriaxialcompressiontestoverthedirectsheartest.

1. Unlike the direct shear test in which the soil sample is forced to fail along apredeterminedplane,thespecimenintriaxialcompressionisfreetofailalongthew eakestplane.
2. Distribution of stress is uniform along the failure plane is uniform. The shearstrengthismobilizeduniformlyatallpointsonthefailureplane.
3. The test procedure has complete control of the drainage conditions. The fielddrainageconditionsarebettersimulatedintriaxialcompressiontestascompar ed todirectsheartest.
4. Precise measurements of pore pressure and volume change are possible duringthetest.
5. The effect of end restraint does not have considerable effect on the result asfailure usuallyoccurs nearthemiddleofthesample.

Unconfinedcompressiontest

Unconfinedcompressiontestisaspecialcaseoftriaxialcompressiontestinwhichnolateralorc onfiningstress $(\sigma_2=\sigma_3=0)$isapplied.Acylindricalsoilsampleof length2to2.5timesthediameterisusedastestsample.Thesoilspecimenis onlyto themajorprincipalstressσ_1 tillthespecimenfailsduetoshearingalongacritical failureplane.

Figure 15 shows the simplest form of compression testing machine. It consists of a smallload frame fitted with a proving ring to measure the vertical stress σ_1 applied to the soilspecimen.Aseparatedialgaugeis usedto measurethedeformationof thesample.

Thesampleisconicallyhollowedatitsendsandplacedbetweentwoconicalseatingsattachedtotwometalplates.Theconicalseatingsreducesendrestraintsandpreventsthe tendency of the specimen to become barrel shaped. The load is applied through acalibratedspringbymanuallyoperatedscrewjackatthetopofthemachine.Thetestsample iscompressedatuniformrateofstrainbythecompressiontestingequipment.The axial deformation and the corresponding axial compressive force are measured.the sample may undergo brittle failure or plastic failure. In case of brittle failure, adefinitemaximumloadisindicatedbytheprovingringwhichdecreasesrapidlywithfurther increase of strain. However, no definite maximum load is indicated by theprovingringdialincaseofaplasticfailure.Insuchacase, theloadcorrespondingto20% strain is arbitrarily taken as the failure load. The maximum axial compressivestress resisted by the specimen before failure is called the unconfined compressivestrength.

Theunconfinedcompressiontestisaquicktestinwhichnodrainageisallowed.Since $\sigma_3 = 0$, the Mohr's circle passes through the origin, which is the pole. Figure 16 showsthe stress conditions in a typical unconfined compression test. The equationplasticequilibriummaybeexpressedas

$$\sigma_1 = 2c_u \tan\alpha = 2c_u \tan\left(45^0 + \frac{\phi_u}{2}\right)$$

In the above equation , there are two unknownsc_u andϕ_u, and cannot be determinedby the unconfined test since a number of test on the identical specimens give the samevalueofσ_1.Hence,theunconfinedcompressiontestisgenerallyconductedon

saturatedclayforwhichtheapparentangleofshearingresistanceϕ_u=0.Hence

$$\sigma_1 = 2c_u$$

The radius of the Mohr's circle is $\frac{\sigma_1}{2} = c_u$, The failure envelope is horizontal. PF is the failure plane, and the stresses on the failure plane are

$$\sigma = \frac{\sigma_1}{2} = \frac{q_u}{2}$$

$$\tau_f = \frac{\sigma_1}{2} = \frac{q_u}{2} = c_u$$

where q_u is the unconfined compressive strength at failure. The compressive stress $q_u = \frac{F}{A_c}$ is calculated on the basis of changed cross-sectional area A_c at failure, which is given by

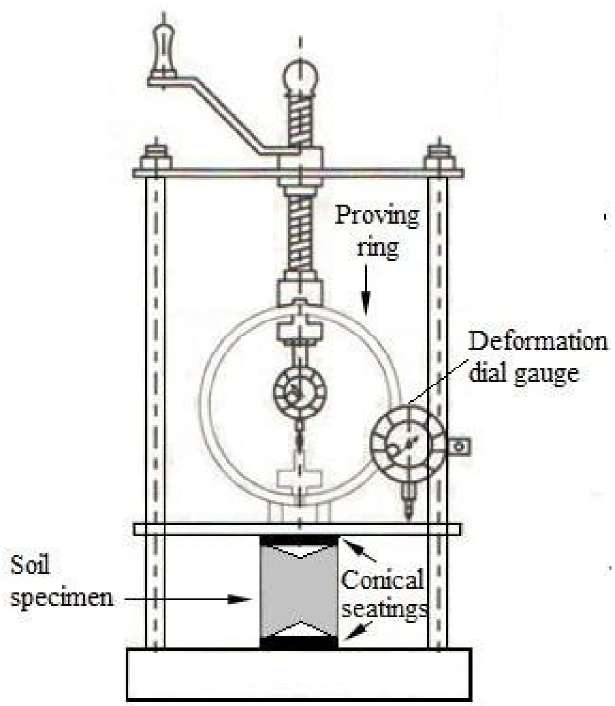


Figure 15 Unconfined Compression test setup

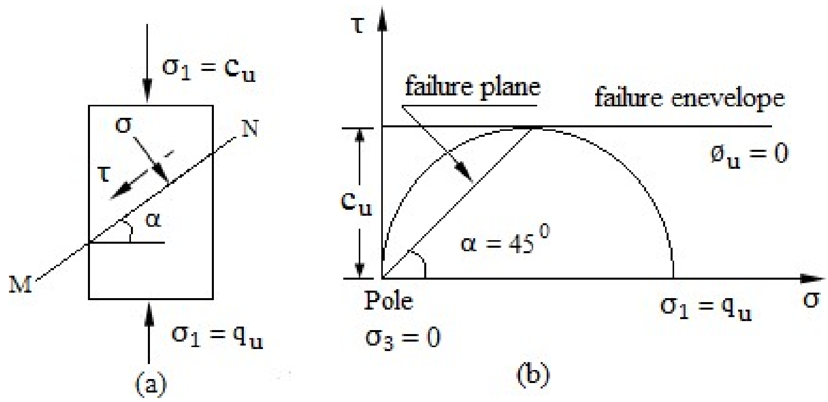


Figure16Unconfinedcompressiontest

$$A_c = \frac{V}{L_0 - \Delta L} = \frac{A_0}{1 - \frac{\Delta L}{L_0}}$$

$$A_c = \frac{A_0}{1 - \in}$$

Where A_c = corrected area of cross section specimen A_0=initialareaofcrosssectionofspecimen L_0=initiallengthofthespecimen

V=initialvolumeofthespecimen

ΔL=changeinlengthatfailure

$\in = \frac{\Delta L}{L_0}$=axialstrainatfailure