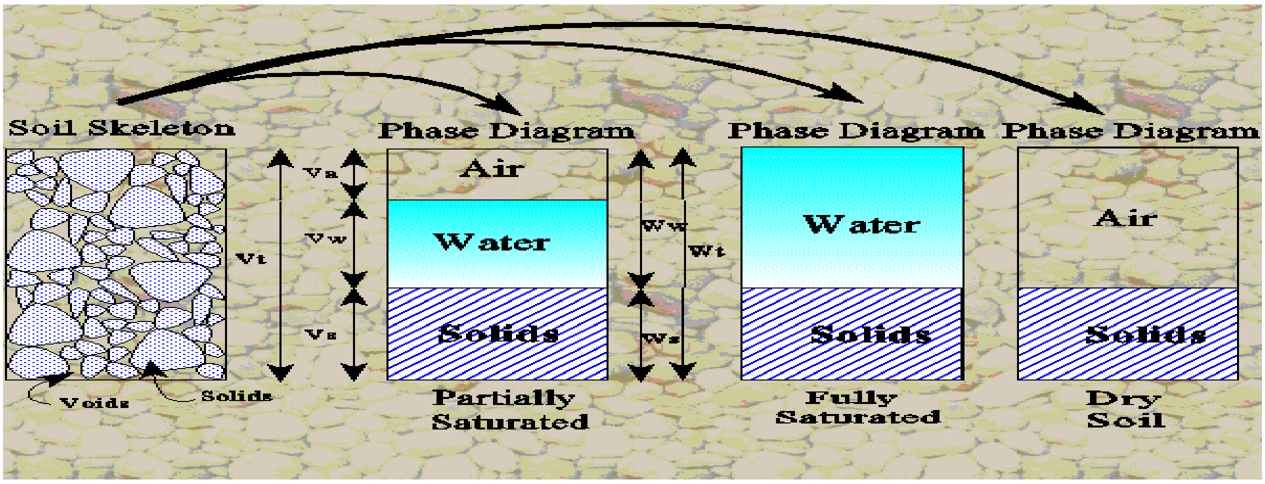
U23CET42-SOILMECHANICS

UNITI-SOILCLASSIFICATIONANDCOMPACTION

2MARKS

1	**Definetermplasticityindexandsaturatedmassdensity.(AUNOV/DEC2014)** **PLASTICITYINDEX:**itisdefined asnumericaldifferencebetween theliquid limit and plastic limit of the soil. $I_P = W_L - W_P$ **SATURATEDMASSDENSITY:** Whenthesoilmassissaturated inbulkdensity is called as saturated density.
2	**Defineshrinkageratio: (AUNOV/DEC2014)** Itisdefined astheratioofgiven volumechangeexpressed asapercentageofdry volume to the corresponding change in water content above the shrinkage limit expressed asa percentage of the weight of the over dried soil. $SR = \frac{\gamma_d}{\gamma_w}$
3	**Drawthephasediagramforcompletelydryandfullysaturatedsoilmass. (AUMAY/JUNE2014)** 
4	**List various field compaction equipments along with its suitability. (AUMAY/JUNE2014)** • Forcohesivesoil-Tamping,kneadingandimpactareeffective. • Forcohesionlesssoils-Kneading,tampingandvibrationareusefull.

5	**Differentiatebetweenvoidratioandporosity. (AUNOV/DEC2013)** **VOIDSRATIO**:Voidratioof asoilmassisratio of totalvolumeofvoidstothe volume of solids in the soil mass. It is denoted by e. $e = \frac{V_V}{V_S}$ **POROSITY:** Theporosityofa given soilsampleistheratioofthevolumeofvoids to the total volume of the given soil mass. .
6	**Themostaccuratemethodforthedeterminationofwatercontentinthe laboratory is oven drying method. (AUNOV/DEC2013)**
7	**If the liquidity index of soil is zero. Find its consistency index.(AU MAY/JUNE 2013)** $I_L=(W-W_P)/I_L$ $I_C=(W_L-$ $W)/I_P I_L=0$ $W-W_P=0$ $W=W_P$ $I_C=(W_L-W_P)/I_P$ $=I_P/I_P=1$ $I_C=1$
8	**Thedrydensityofasoilanditsspecificgravityofsolidsarerespectively18 kN/m^{3}and 2.7 find the moisture content required to have 100%saturation of the soil. (AU MAY/JUNE 2013)** $\gamma_d = 18 KN/m^3 G=2.67$ S=100%=1Find'ω' Void ratio,$e= \frac{G \cdot \gamma_\omega}{\gamma_d} - 1 = \frac{2.7 \times 9.81}{18} \quad 1$ $e=0.4715$

$e=\frac{W\cdot G}{S}$ and $\omega=\frac{e\cdot S}{G}$

$$\omega=\frac{0.4715\times1}{2.70}=0.1746=17.46\%$$

9 **Whatarealltheatterberglimitsforsoilandwhyitisnecessary? (AU NOV/DEC 2012)**

- Liquidlimit
- Plasticlimit
- Shrinkagelimit

10 **Definesieveanalysisandsedimentationanalysisandwhatisthenecessityof these two analyses? (AU NOV/DEC 2012)**

Sieve analysis is used to measure the particle size distribution of a soil bypassing the soil through the series of sieves.

Sedimentationanalysisisusedforthesoilparticlesizemeasurementandin which, the soil fraction finer than 75μ sizes is kept in suspension in a liquid medium. (Generally Water)

11 **Distinguishbetweenresidualandtransportedsoil.(AUMAY/JUNE2012)**

RESIDUALSOIL	TRANSPORTEDSOIL
• Soil remains in that place is called residual soil and usually formed by physical and chemical weathering • Characteristics of soildepend on the parent rock.	• Weather piece of rocks thathave been carried out byrolling agents like wind and water & break down into small pieces to settle down. • Characteristics of soil dependon theenvironmentalsurrounds.

12 **Givetherelationshipbetweenγ_{Sat},G ande. (AUMAY/JUNE2012)**

$\gamma=W/V=\{(\gamma_S*V_S)+(\gamma_W*V_w)\}/V$

Forpartiallysaturatesamplewehave, $V_s=1$,

$V_W=e_w$ and $V=(1+e)$

$\gamma=\{(\gamma_S*1)+(\gamma_W*e_w)\}/1+e$

but $\gamma_s=G\ \gamma_W$ and $e_{w=}e_s$

$\gamma=(G*\gamma_W+\gamma_{W.}*e_S)/1+e$

	$\gamma=\{(G+es)\gamma_{W}\}/1+e$ whenthesoilisperfectlydry,S=0 Hence γ reduced to $\gamma_{d}=G\gamma W/1+e$ whens=1,γbecomes $\gamma_{sat}=\{(G+e)$ $\gamma_{W}\}/1+e$

16MARKS

1	**A cubic meter of soil in its natural state weighs 17.75 kN; after being dried it weighs 15.08 kN. The Specific gravity of the soil is 2.70. Determine the degree of saturation, void ratio, porosity and Water content of the original soil sample.**(AU MAY/JUNE 2014) **Solution:** Volume of the soil, V=1 m^3Totalweightofthesoil,W=17.75kN Weightofsolids,W_S=15.08kN Specific gravity, G=2.70 Weightofwater, $W_W=W-W_S$ =17.75– 15.08 =2.67kN Watercontent, $\omega=\frac{W_W}{W_S}$ $=\frac{2.67}{15.08}$ =0.177=17.70 %

Unitweight, $\gamma = \frac{weight}{volume}$

$= \frac{17.75}{1}$

$= 17.75 kN/m^3$

Dryunitweight, $\gamma_d = \frac{\gamma}{1+\omega}$

$= \frac{17.75}{1+0.177}$

$= 15.081 kN/m^3$

Voidratio, $e = \frac{G \cdot \gamma_w}{\gamma_d} - 1$ $\gamma_w = 9.81$

$= \frac{2.70 \times 9.81}{15.081} - 1$

$= 0.7563$

Degreeofsaturation, $S = \frac{w \cdot G}{e}$

$= \frac{0.177 \times 2.70}{0.7563}$

$S = 0.632 (or) 63.2\%$

2	**Discuss the effects of compaction on various engineering properties of soil.(AU MAY/JUNE 2014)** When asoiliscompacted, itchangesitsengineeringpropertiesandtherebybehaves differently. Some of the engineering properties which changes on application of compactive effort is briefly described below. **1. Permeability** The effect ofcompaction is todecrease thepermeability. In the case of fine grained soilsit hasbeen found that for thesamedrydensitysoil compacted wet of optimum will be less permeable than that of compacted dry of optimum. **2. Compressibility** In case of soil samples initially saturated and having same void ratio, it has been found that in low pressure range a wet side compacted soil is more compressible than adryside compacted soil, and vice versa in high pressure range. ***3.* PorePressure** In undrained shear test conducted on saturated samples of clay it has been found that lower pore pressures develop at low strains when the sample is compacted dry of optimum, compared to thecase when the sample is compacted wet of optimum. But at high strains in both types of samplesthe development of pore pressure issame for same density and water content. **4. Stress-StrainRelation** Samples compacted dryofoptimumproduce much steeper stress-starin curves with peaks at low strains, whereas samples compacted wet ofoptimum, having the same density, produce much flatter stress-strain curves with increase in stress even at high strains. **5. ShrinkageandSwelling** At same densitya soil compacted dryofoptimumshrinks appreciablyless than that of compacted wet of optimum. Also the sol compacted dry of optimum exhibits greater swelling characteristics than samples o fthe same density compacted wet of optimum.

3.	**A sample of dry and sand having a unit weightof 16.5 kN/m^3 and a specific gravity of 2.70 is placed in the rain. During the rain the volume of the sample remains constant but the degree of saturation increases to40%. Determine the unitweightandwatercontentofthesoilafterbeingintherain. (AU MAY/JUNE 2014)**

Dryunitweight,γ_d=16.5kN/m^3

Specific gravity, G=2.70

Increase in S=40 % Initial

degree of saturation, S_1=100 S

Finaldegreeofsaturation,S_2=140 S

$$\text{Voids ratio, } e = \frac{G \cdot \gamma_w}{\gamma_d} - 1$$

$$\text{Degreeofsaturation,} S_1 = \frac{w \cdot G}{e}$$

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

$$= \left(\frac{2.70+0.6053}{1+0.6053}\times 9.81\right)$$

$$\gamma = 20.199 \text{kN/m}^3$$

$$\text{Dry unit weight, } \gamma_d = \frac{\gamma}{(1+w)}$$

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

$$= \frac{20.19}{16.5} - 1$$

$$=0.2236=22.36\ \%$$

4	**A soil sample is a mixture of cohesionless and cohesive soils.explain and discussesthemethodofdetermininggrainssizedistribution.. (AU MAY/JUNE 2014)** **DRYSIEVE ANALYSIS:** Thesoil shouldbeoven-dry, it shouldn't containanylump, if necessary, it should bepulverized.Iforganicmattersin thesoil;,ittaken air– dryinstead ofoven dry. The sample is sievedthrough a 4.75 mm IS sieve .the portionretained on the sieve is gravelfractionor plus 4.75 mm material .then gravel fraction is sieved through the set of coarse sieves manually or mechanical shaker. The minus4.75mmfraction is sieves through thesetiffinesieves .thesample is placed in the top sieves and the set of sieves is kept on a mechanical shaker. Normally, 10minutesof shacking is sufficient for most soils. The mass of soil retained on each sieve and on pan is obtained to the nearest 0.1gm **Suitability:cohesionlesssoilswithlittleornofines.** **WETSIEVEANALYSIS:** Ifthesoilcontainsasaturatedasubstantialquantityoffineparticles, A wet analysis required. A soil sample in the required quantity is taken, using a rifer an dried in an oven. The dried sample is taken in a tray and sacked with water. The samples stirred and lift soaking period of at least one hour. The slurryis taken sieved through a4.75 mm IS sieve, and washed with a jet ofwater.Thematerialretainedonthesieveisthegravelfaction.Thematerial retained on the 75 ☐ sieve is collected and dried an oven. It is then sieved throughthe set of the fine sieves ofthesize2mm,1mm,600 ☐ ,425 ☐ ,212 ☐ ,150 ☐ ,and75☐ Thematerial that would have been retained on the pan is equal to the totalmass of soil minus the sum of the masses of material retain on all sieves

5	**A sample of clay soil has liquid limit of 62% ands its plasticity index is 32%. What is the state of consistency of the soil if the soil in its natural state has a water content of 34%. Calculate the shrinkage limit & degree of saturation if thevoidratioofthesampleattheshrinkagelimitis0.70 (AU NOV/DEC 2014)** **Solution:** Liquidlimit=62%,PlasticindexI$_P$=32%,Watercontent=34%,e=0.7&G=2.7 **SHRINKAGELIMIT:** W_S=e/G W_S=0.7/2.7=0.259=25% **DEGREEOFSATURATION:** S_e=w.G S=(Wg)/e S=(0.34*2.7)/0.7 S=1.31=131%
6	**Test on a soil sample from a borrow area resulted specific gravity of 2.7, voids rati=0.65 and the water content of 15%.what is the quantity of soil required to construct an embankment volume of 8000m3,if the borrow materials compacted to achieve max dry density of 18kN/m^3 at a moisture content 18%. Calculate addition quantity of water required for every cubic meter of compacted-soil. (AUNOV/DEC2014)** **Solution**: G=2.7,e=0.65,W=15%,Volume=8000m^3,γ_d=18kN/m^3,w=18% Fortheborrowpit $\gamma(\rho)=\frac{G.\gamma(1+W)}{1+e}$ $e=\frac{G.\gamma(1+W)}{\rho}-1$

	$e=\frac{(2.7)*1(1+0.15)}{1+0.65}-1$ $e=\frac{2.7*1.15}{1.65}-1$ e=0.88 FortheFill $\rho_d(or)\gamma_d=\frac{G*\gamma w}{1+e}$ $e=\frac{G*\gamma w}{\gamma d}-1$ $e=\frac{2.7*1}{1.8}-1$ e=0.5 Volumeofsoildsremainsconstant $V_S=\frac{V1}{1+e1}=\frac{V2}{1+e2}=\frac{8000}{1+e}$ $Vs=\frac{8000}{1+0.5}$ $V1=5333.33m^3$ V1=5333.33(1+0.88) $V1=10026.6\ m^3$ Generallyw=M_W/M_d M_d=massofsolids=VsGγ_w =5333.33*2.7*1=14399.99m3 $M_{W1}=2159.99m^3$ $M_{W2}=14399.*0.18=2591.99m^3$ Watertobeadded=$M_{W2}-M_{W1}$ =2591-2159 $=72m^3$
7	S**andysoilinaborrowpithasunitweightofsolidsas25kN/m3,watercontent equalto11%andbulkunitweightequalto16kN/m3.Howmanycubic meter of**

compacted fill could be constructed of 3500 m3 of sand excavated from the borrow pit, if the required value of porosity in the compacted fill is 30%. Also compute the change in degree of saturation. (AUNOV/DEC2013)(AU NOV/DEC 2012)

Solution:

$\gamma_s = 25 kN/m^3, w = 11\% = 0.11, Porosity\ n = 30\% = 0.3$

$$\frac{V_1}{V_2} = \frac{w/\gamma_1}{w/\gamma_2}$$

$$= \frac{\gamma_2}{\gamma_1} = \frac{1+e_1}{1+e_2}$$

$$V_2 = V_1 \left(\frac{1+e_2}{1+e_1} \right)$$

$$= V_1 \left(\frac{1-n_1}{1-n_2} \right)$$

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

$$= \frac{\gamma_s}{\frac{\gamma}{(1+w)}} - 1$$

$$= \frac{\gamma_s(1+w)}{\gamma} - 1$$

$$e_1 = \frac{\gamma_s(1+w)}{\gamma_1} \quad 1$$

$$= \frac{25(1+0.11)}{16} - 1 = 0.73$$

$$e_2 = \frac{n_2}{1-n_2}$$

	$= \frac{0.3}{1-0.3} = 0.429$ $e_2 V_2 = V_1 \left(\frac{1+e_2}{1+e_1} \right)$ $= 3500 \left(\frac{1+0.429}{1+0.73} \right) = 2891.04 m^3$ $\text{Now}, S_1 = \frac{w \gamma_s}{e_1 \gamma_w}$ $= \frac{0.11 \times 25}{0.73 \times 9.81} = 0.38$ $S_2 = \frac{w\gamma_s}{e_2\gamma_w} = \frac{0.11 \times 25}{0.429 \times 9.81} = 0.65$
8	**Whatarethedifferentmethodsofcompactionadoptedinthefield? (AU NOV/DEC 2013)** **TAMPER:** Ahand operated tamper consist ofblockiron, about3to5kg ofmassattachedtowoodenrod. The tamper is lifted for about 0.30m and dropped on the soil to becompressed.MechanicalTampersoperatedbycompressedairorgasoline power **ROLLERS:** • smooth– wheelrollers • pneumatic–tiredrollers • Sheep-footrollers **SMOOTH –WHEEL ROLLERS:** Smooth–wheelrollersare useful finishing operationsaftercompactionoffillersandforcompactinggranularbasecauses of highways.

	PNEUMATIC–TIREDROLLERS • Pneumatic–tyredrollersusecompressedairtodeveloptherequired inflation pressure. • Therollercompactive thesoilprimarily by kneading action.Theserollers are effecting for compacting cohesive as well as cohesion less soils. **SHEEP–FOOTROLLERS** The sheep – foot roller consists of a hollow drum with a large number of small projections (known as feet) on its surface. The drums are mounted onasteelframe.Thedrumcanfillwithwaterorballastincreasesthemass.The contactpressureisgenerallybetween 700 to4200KN/m^2
9	**In a compaction test on a soil, the mass of wet soil when compacted in the mould was 20N. The water content of the soil was 16%. If the volume of the mould was 0.945 litres, determine the dry density, void ratio, degree of saturation and %air voids. Take G = 2.68 (AUNOV/DEC 2013)** Let us use suffix 1 for the borrow pit soil and 2 for the compacted soil. Assuming that weight and water content do not change during construction, the change in the volume can be calculated from the change in the unit weight **Solution:** Mass(or)weightofthewetsoilsample,w_1=20kN Water content,w=16% $\text{Watercontent},\omega=\frac{\text{weight ofwater}}{\text{weightofsolids}}\times 100$ $16=\frac{W_w}{W_s}\times 100$ $\text{Unitweight},\gamma=\frac{W}{V}$

$$= \frac{20}{0.945}$$

$$= 21.16 \text{kN/m}^3$$

(i) Dryunitweight, $\gamma_d = \frac{\gamma}{1+w}$

$$\gamma_d = \frac{21.016}{0.16} = 18.24 \text{ kN/m}^3 \; 1 +$$

(ii) Voidratio, $e = \frac{V_v}{V_S}$ (or)

$$e = \frac{G \cdot \gamma_w}{\gamma_d} \quad 1$$

$$e = \left(\frac{2.68 \times 9.81}{18.24} - 1\right)$$

$$e = 0.44$$

(iii)Degree of saturation, $S = \frac{V_w}{V_v}$ (or)

$$e = \frac{w \cdot G}{S} \text{ and } S = \frac{w \cdot G}{e}$$

□□□ $S = \frac{0.16 \times 2.68}{0.44} = 0.97$

(iv)Percentairvoids, $\eta_a = \frac{V_a}{V} \times 100$ and

$$\eta = \frac{e}{1 + e} = \frac{0.44}{1 + 0.44} = 0.305 \; 1$$

$$\eta = \frac{V_v}{V} \Rightarrow V_v = \eta \times V$$

$$= 0.305 \times 0.945$$

$$V_v = 0.28 \text{m}^3$$

10	**A fine grained soil has liquid limit of 60% plastic limit of 26%;classify the soil as per IS classification system. (AUNOV/DEC 2013)** Plasticityindex=liquidlimit – plasticlimit =60-26=34% PI=34%
11	**The mass of wet soil when compacted in a mould was 19.55kN. the water contentofthesoil**was**16%.ifthevolumeofthemouldwas0.95m^3.determine i) dry unit weight ii) void ratio iii) Degree of saturation& iv) percent airvoids. Take air voids G=2.68 (AU MAY/JUNE2013)(AU MAY/JUNE 2013)** **Solution:** i) Dryunitweight: $\gamma d = \frac{\gamma}{1+w}$ γ=(W/V)=(19.55/0.95) γ=20.5kN/m^3 γd=20.5/(1+0.16) γd=17.62kN/m^3 ii) Voidsratio: $e = \frac{G \cdot \gamma w}{\gamma d} \quad 1$ =[(2.68*9.81)/(17.62)]-1 =0.49 iii) Degreeofsaturation:

	$S = \frac{w \cdot G}{e}$ =(0.16*2.68)/0.49 =0.8751=87.51% iv)Percentair voids n_a =e(1-s)/(1+e) =(0.49(1-0.8751)/ (1+0.49) =0.041%
12	**Wet soil samples of mass 1.9 kg had a volume of 945 cm3after oven drying its mass as reduced to 1.7 kg. The specific gravity of solids was found to be 2.7 determine. (AU MAY/JUNE 2013)** (i) Moisturecontent (ii) Bulk density (iii) Drydensity (iv) Void ratio (v) Porosity (vi) Degreeofsaturation, (vii) Saturateddensity. Massofthewetsample,M=1.9kg=1900 g Massofthedrysample,M_d=1.7kg= 1700g Massofthewatersample,M_w=1900–1700=200g. (i) Moisturecontent, $w = \frac{M_w}{M_d}$ $= \frac{200}{1700}$ = 0.11675= 11.76% (ii) Bulkdensity,ρ = M/V=(1.9/945)=2.01*10^{-3} Kg/cm^3

(iii) Unit weight, $\gamma = \frac{mass \times g}{volume}$

$= \frac{1900 \times 9.81}{945} = 19.73 kN/m^3$

(iv) Dry density, $\gamma_d = \frac{\gamma}{1+w}$

$= \frac{19.73}{(1 + 0.11765)} = 17.65 kN/m^3$

(v) Void ratio, $e = \frac{G \cdot \gamma_w}{\gamma_d} - 1$

$= \frac{2.7 \times 9.81}{17.65} - 1 = 0.501$

(vi) Porosity: n =w.G/e

=(0.11*2.7)/0.588

=0.37=37%

(vii) Degree of saturation, $S = \frac{w \cdot G}{e}$

$= \frac{0.11765 \times 2.70}{0.501}$

=0.634(or)63.4%

(viii) Saturated Density, $\gamma_{sat} = \left(\frac{G + e}{1+e}\right)\gamma_w$

$= \left(\frac{2.7+0.11765}{1+0.11765}\right) \times 9.81$

$= 20.92 kN/m^3$

13	**ThePycnometerisusedfordeterminationofthespecificgravityof soilparticles of bothfine grained and coarse grained soils. The specific gravity of soil is determined using the relation. (AU NOV/DEC 2012)**

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

Where M_1=mass of empty Pycnometer,

M_2=massofthePycnometerwithdrysoil

M_3=mass ofthePycnometerand soiland water,

M_4=massofPycnometerfilledwithwateronly. G=

Specific gravity of soils.

EquipmentforPycnometerTest:

1. Pycnometerofabout1 litrecapacity
2. Weighingbalance,withanaccuracyof1g.

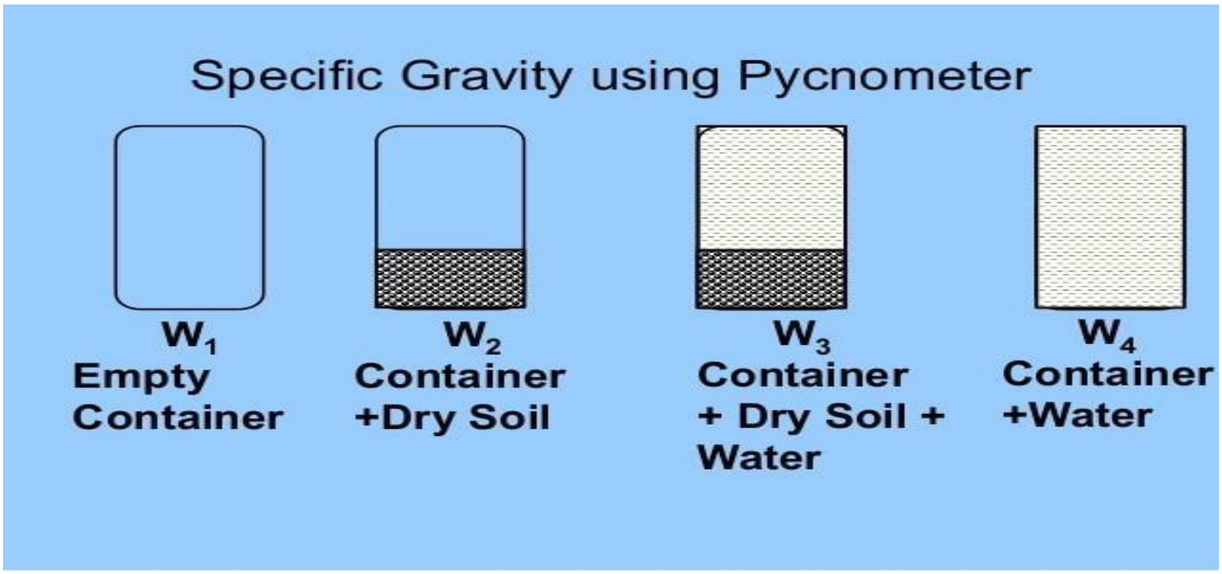
ProcedureforSpecificGravityofSoilbyPycnometer:

1. Cleanand drythe Pycnometer. Tightlyscrewitscap. Takeitsmass(M_1)tothe nearest of 0.1 g.

2. Containerwithdrysoil.Takeitsmass(M2).

3. Fillthepycnometerwithdrysoilandwater.Takeitmass(M3).

4. Fillthewaterwithwateraloneanditsmassas(M4).



CE6405-SOILMECHANICS

UNITII–SOILWATERANDWATER FLOW

2MARKS

1	**Statetheassumptionsinconstructionofflownet.(AUNOV/DEC2014)** 1. Theflowlinesandequipotentiallinesmeetatrightanglestooneanother. 2. The fieldsareapproximatelysquaressothatacirclecanbedrawntouching all the four sides of the square.
2	**StateDarcy'slawalongwithitslimitations. (AUMAY/JUNE2014)** Therateofflow'q'throughthesaturatedsoilofcross-sectionalareaA,is proportional to the hydraulic gradient 'i'. $q = kiA$ where, k-Co-efficientofpermeability(cm/sec) Darcylawisvalidonlyforlaminarflowandappliedtothesoilfractionfiner than fine gravels.
3	**Differentiateseepagevelocityfromdischargevelocity.(AUMAY/JUNE2014)** Seepagevelocityistheactualvelocity(ortruevelocity)withwhichwaterflows through the soil voids. SeepageVelocity, $v_s = \frac{v}{\eta}$ Vsisalwaysgreaterthan'v'.
4	**Whatisquicksand? (AUNOV/DEC2013)** When flow takes place inan upward direction, the seepage pressure also acts inthe upward direction and the effective pressure is reduced. If the seepage pressure becomes equal to the pressure due to submerged weight of the soil, the effective pressure is reduced to zero. In such a case, cohesion less soil loses all its shear strength, and the soil particles have a tendency to move up in the direction of flow.Thisphenomenonofliftingofsoilparticles iscalledquickcondition(or)quick sand.

5	**Whatistheimportanceofeffectivestress?(AUNOV/DEC2013)** At any point within the soil mass, the magnitude of both total stress and water pressure are dependent onthe ground water position. Witha shift in the table due to seasonal fluctuations there is a resulting change in the distribution in pore water pressure with depth. Effectivestressreducesthevoidsratioandincreasestheshearstrength.
6	**Inalaboratorypermeabilitytestonaclayeysoil,thediameter ofthestandpipe is 2cmandthe diameter of the permeameter is120cm,the heightof the mould is 130 cm. Determine the time taken for the head of water in the stand pipe to drop from 190 cm to 150 cm. (AU MAY/JUN 2013)** Solution Forclayeysoil, $K=1\times10^{-7}$ cm/sec Dia.ofstandpipe =2 cm $a=\frac{\pi}{4}\times2^2$ Dia.ofpermeameter, D=120cm $A=\frac{\pi}{4}\times120^2$ Lengthofthepermeameter, L=130cm h_1=190cm h_2=150cm $K=\frac{2.3\,aL}{At}\log_{10}\frac{h_1}{h_2}$
7	**Whatarethedifferenttypes ofsoilwater? (AUMAY/JUN2013&2012)** 1. Freewater 2. Structuralwater 3. Adsorbedwater 4. Capillarywater 5. Groundwater 6. Porewater 7. Structuralwater

8	**Whatismeantbytotalstress,neutralstressandeffectivestress?** **(AUNOV/DEC2012) (AUNOV/DEC2014)** **Effective Stress (σ′)** is defined as the stress transmitted through grain to grain at the point of contact through the soil mass. The load per unit area of soil, responsible for soil deformation is called effective stress. **NeutralStress(u)**isdefinedasthestress(or)pressuretransmittedthroughthe Pore fluid. It is also called Pore Pressure (or) Pore Water Pressure. **TotalStress(σ)** isdefinedasthestressequaltothesumofeffectivestressand neutral stress. $\sigma = \sigma' + u$
9	**Whatismeantbycapillaryriseinsoilandhowitaffectsthestresslevelin soils? (AU NOV/DEC 2012)** **Capillary rise** is defined as the rise ofwaterinthe pores ofthe soildue to existence of surface tension, which pulls the water up against the gravitational force.
10	**Listoutthevariousforms ofsoil water.** **(AUMAY/JUNE2012)** 1. Analyticalmethod 2. Grapicalmethod 3. Sandmodel 4. Electricalanalogymethod 5. Capillaryanalogymethod
11	**Differentiateseepagevelocityfromdischargevelocity.(AUMAY/JUNE2014)** Seepagevelocityistheactualvelocity(ortruevelocity)withwhichwaterflows through the soil voids. SeepageVelocity, $v_s = \frac{v}{\eta}$ where, v-flowvelocityandη-Porosity

12 **Statethenameofthemethodsoffindingfieldpermeability.**

FieldMethods

(i) Pumpingouttests

(ii) Pumpingintests

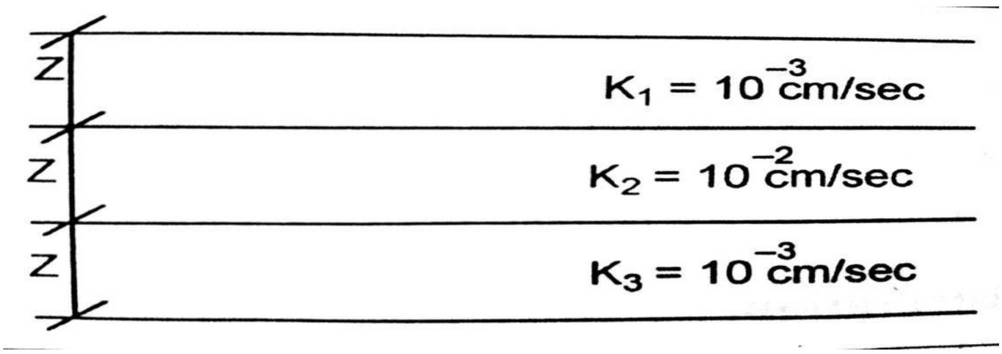
13 **Whatistheimportanceofeffectivestress?**

At any point within the soil mass, the magnitude of both total stress and water pressure are dependent onthe ground water position. Witha shift in the table due to seasonal fluctuations there is a resulting change in the distribution in pore water pressure with depth.

Effectivestressreducesthevoidsratioandincreasestheshearstrength.

16 MARKS

1 **A sand deposit contains three horizontal layers of equal thickness. The coefficient of permeability of upper and lower layer is 10^{-3} cm/sec and the intermediate layer is 10^{-2} cm/sec. what are the values of of horizontal and vertical permeability of three layers and what is the ratio. Also derive the appropriate equation require to solve above.(AU NOV/DEC2014)**



HORIZONTALFLOW:

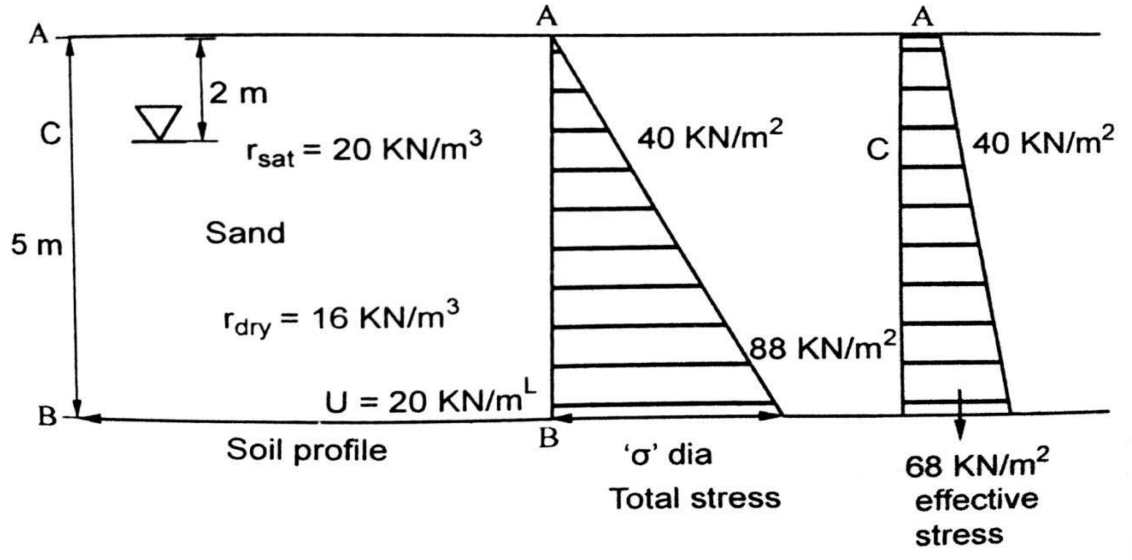
$$K_h = \frac{K1Z1 + K2Z2 + K3Z3}{Z}$$

$$K_h = \frac{10^{-3}Z1 + 10^{-2}Z2 + 10^{-3}Z3}{3Z}$$

$$K_h = \frac{Z}{3Z}.[2*10^{-3} + 10^{-2}]$$

$$K_h = \frac{1}{3}.[2*10^{-3} + 10^{-2}]$$

	$K_h = 4*10^{-3}$cm/sec **VERTICALFLOW:** $K_V = Z / \frac{Z1}{K1} + \frac{Z2}{K2} + \frac{Z3}{K3}$ $= 3Z / \frac{2Z}{10^{-3}} + \frac{Z2}{10^{-2}}$ $= 3 / \frac{2}{10^{-3}} + \frac{1}{10^{-2}}$ $= \frac{3*10^{-3}*10^{-2}}{(2*10^{-2})+(1*10^{-2})}$ $= \frac{3*10^{\wedge}-5}{0.021}$ $= 1.4*10^{-3}$ **RATIO:** $\frac{Kh}{Kv} = \frac{4*10^{-3}}{1.4*10^{-3}}$ $\frac{Kh}{Kv} = 2.86$
2	**A soil depositconsists ofsand layerof 5m thickfollowed by clay layer.He water table is at a depth of 2m from ground level and the dry and saturated unit weight of 16kN /m³respectively. Draw the variation of total, neutral and effective stress in sand layer, what will be the change do you expect in effective stressin sandy layer.(AU NOV/DEC2014)** **Solution:** **TOTALSTRESSDISTRIBUTION:** AtA,σ=0 At, Cσ=γ_{sat}* Z =20*2=40kN/m^2 At,B,σ=(γ_{sat}*Z_1)+(γ_{sat}* Z_2) =(20*2)+(16*3)

	$=88kN/m^2$ **POREPRESSUREDISTRIBUTION:** AtA,σ=0 $C, u=-H_C.\gamma_w$ $=-3*9.81=-29.43kN/m^2$ **EFFECTIVEPRESSUREDISTRIBUTION:** σ'=σ-u AtA,σ'=0 C, σ'=40kN /m² B,σ'=6.8kN/m² 
3	**Thefallingheadpermeabilitytestwasconductedonasoilsampleof 4cmdiameterand18cmlength.Theheadfellfrom1.0mto0.40min20 minutes. If the cross-sectional area of the stand pipe was 1 cm², determine the coefficient of permeability(AUMAY/JUN2014)** Lengthofthespecimen,L=18cm Diameter of the specimen,D =4 cm h_1=1.0m h_2=0.40m

	$t_1 = 0 secs$ $t_2 = 20 minutes$ $= 1200 sec$ $a_1 = 1 cm^2$ **CO-EFFICIENTOFPERMEABILITY,** $K = 2.3\frac{aL}{At}\left[\log_{10}\frac{h_1}{h_2}\right]$ $= \frac{2.3\times1\times18}{12.566\times1200}\log_{10}\left(\frac{1}{0.4}\right)$ $K = 10.925\times 10^{-4}$ cm/sec
4	**Ina falling headpermeameter testthe initial headis 40 cm.The headdrops by 5 cm in 10 minutes. Calculate the time required to run the test for the final head to be at 20 cm. If the sample is 6cm height and 50 cm2 in cross sectional area. Calculate the coefficient of permeability take area of stand pipe is 0.5 cm2** **(AUNOV/DEC2013)** In atimeinternalt=10minutes,theheaddropsfrom initialvalueofh_1=40toh_2 = 40 – 5=35cm. $K = 2.3\frac{aL}{At}\log_{10}\frac{h_1}{h_2}$ (or)$t = \frac{2.3\,aL}{AK}\log_{10}\frac{h_1}{h_2}$ $= m\log\frac{h_1}{h_2}$ $m = \frac{2.3aL}{AK}$ =constant forthesetup $10 = m\log_{10}\left(\frac{40}{35}\right)$ (or)

$$m=\frac{10}{\log_{10}\left(\frac{40}{35}\right)}$$

$$=\frac{10}{0.058}=172.5\text{units}$$

$$t=m\log_{10}\frac{h_1}{h_2}=172.5\log_{10}\frac{h_1}{h_2}$$

Now, letthetime internalrequired forthe headtodrop frominitialvalueof h_1=40 cm to a final of h_2 = 20 cm, be t minutes.

$$t=172.5\log_{10}\frac{40}{20}$$

$$=172.5\times0.301=51.9\text{minutes}$$

Again, $m = 2.3\frac{aL}{At} = 172.6$ units. $\therefore K=\frac{2.3aL}{A\times172.5}$ cm/min

(Sincetusedtocomputedmwasinminutes)

$$K=\frac{2.3\times0.5\times6}{50\times172.5\times60}\text{cm/sec}=1.335\times10^{-5}\text{cm/sec}$$

Alternatively, $K=2.3\frac{aL}{At}\log_{10}\frac{h_2}{h_1}$

$$=\frac{2.3\times0.5\times6}{50\times10\times60}\log_{10}\left(\frac{40}{35}\right)$$

$$=1.335\times10^{-5}\text{cm/sec}$$

5	**Ina site the ground water table is at existing ground level. During flooding the water level rises to2m above the ground level. Discuss the effect of raise inthe water level on effective stress. (AU NOV/DEC2013)** The ground water table is at the existing ground level. A rise in free water level above the ground surface 2m would result in an increase in the total stress at every location in that site(ie,$\sigma = 2*\gamma_w = 19.62 kN / m^2$) Similarly,theporewaterpressurewouldalsoincreaseateverylocationbythesame magnitude. (2*9.81)= 19.62kN / m^2. Thustheeffectivestressdistributionforthiscasewouldbethesame. Any fluctuation inthe leveloffree waterabove the ground surface would not result in any charge in effective stress at any depth within the soil deposit.

6

Whatisflownet?Describethemethodusedtoconstructflownet.(AU NOV/DEC2013) (AUMAY/JUN2014)

Aflownet isagraphicalrepresentationoftwodimensionalseepage&consistsof two groups of curve of flow lines and equipotential lines.

Once a flow net is constructed, its graphical properties can be used to obtain solution for many seepage problems, such as the estimation of seepage loss from reservoirs,determinationofseepagepressures,upliftpressurebelowdams,tocheck against the possibility of piping and many others.

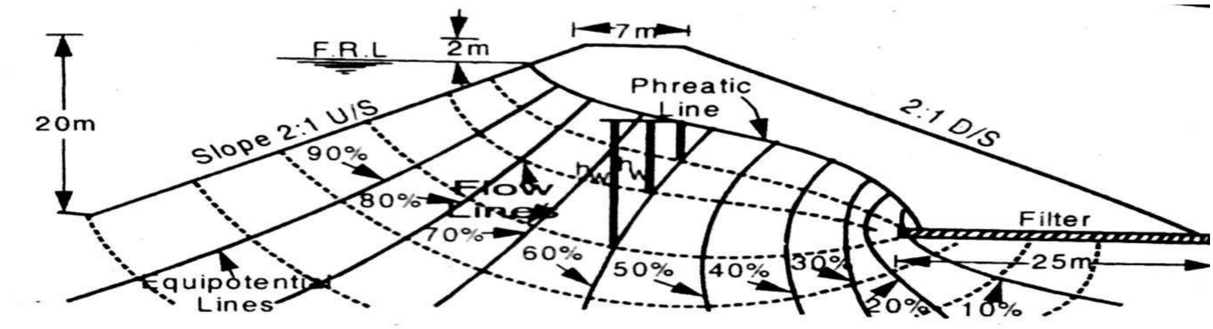


Fig. Flow Net For Steady Seepage

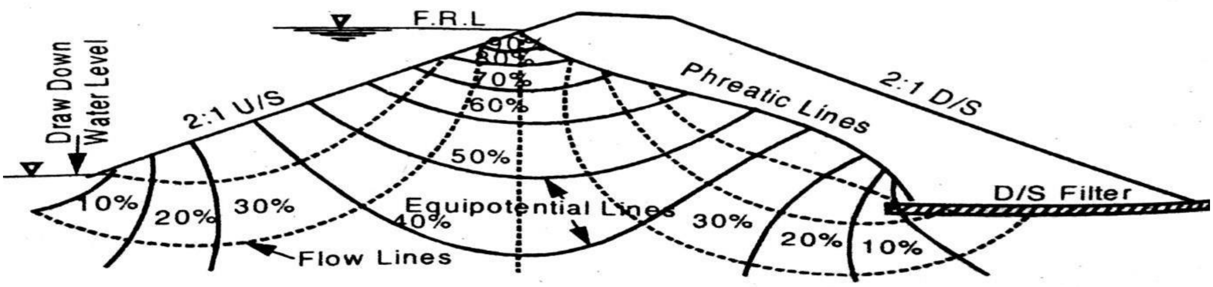


Fig. Flow Net For Sudden Draw down

METHODSFOROBTAININGFLOWNETS:

1. Analyticalmethod
2. Graphicalmethod
3. Sandmodel
4. Electricalanalogymethod
5. Capillaryanalogymethod

ANALYTICALMETHOD:

ItrequiresamathematicalsolutionoftheLaplace'sequation.

Itcanbe useinsimplecasesofflow,where theboundarycanbeexpressedin equations.

ELECTRICALANALOGYMETHOD:

Thismethodisquiteextensivelyused.

It is based onthe principle ofDarcy’s law governing flow ofwater throughsoiland is analogous to ohm’s law governing the flow of current through conductors.

The seepage being proportional to the heat dissipated is similar to that the current being proportional to the voltage drop. Both boundary conditions are similar, thus the pattern of flow of electricity obtained in the electrical model has the same geometric shape as the seepage through soils.

The seepage medium is replaced by the electric conductor consisting of water with some salt or dilute hydrochloric acid. The boundaryequipotential lines are made of the copper. The boundary flow lines are stimulated by non-conducting strips. An alternating voltage is applied across the boundary equipotential strips. A potential divided is connected in parallel with the alternating current source.

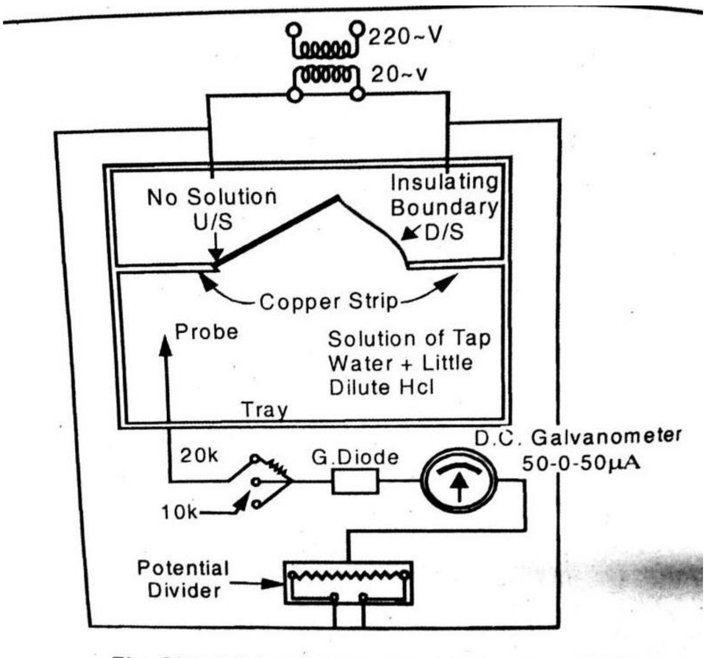


Fig. Circuit Diagram For Electric Analogy Tray.

To determine a line contour of equal potential to the potentiometer is adjusted to a percentage of the total voltage drop and the probe of galvanometer is used to find the corresponding null points. Changes in coefficient of permeability in soil zones are stimulated by changes in the coefficient of electrical conductivity.

CAPILLARYANALOGYMETHOD:

Capillary flow between two closely spaced parallel glass plates is analogous to 2D flow through soils. A model is placed between two glass plates which connect two smalltanks.Thedistanceoftheseplatesareconstantandwidthisconstant.when theboundaryconditionsofthemodelissameasseepageproblem.Theflownetcan

	becompletedbydrawingtheequipotentiallines. **SANDMODEL:** Sand model constructed in water tanks also give a visual demonstration of flow, like the capillary flow models. **GRAPICALMETHOD:** Thisisthemethodmostextensivelyused. It is based on trail sketching. The hydraulic boundary conditions must be examined before sketching. The flow net can be plotted by trail and error by observing the following properties of flow net. **PROPERTIESOFFLOWNET:** 1. The flow lines and equipotential lines are meeting at right angles to each other. 2. The field are approximately squares, so that a circle can be drawn touching the all the four sides of the square. 3. The quantity of water flowing through each flow channel is the same; likewisethesamepotentialdropoccursbetweentwosuccessiveequipotential lines. 4. Smallerthedimensions,greaterwillbehydraulicgradientandvelocity. 5. Inahomogeneoussoil,everytransitionintheshapeofthecurveissmooth.
7	**What is soil suction? How is it measured? What are the factors that affect soil suction? (AU NOV/DEC2013)** **SOILSUCTION:** Thepressuredeficiencyintheheldwateristermedassoilsuctionorsuctionessure. **MEASUREMENTOFSOILSUCTION:** Itis measuredbytheheight $h_c$towhicha watercolumncouldbedrawnbysuction in a soil mass free from external stress. pFvalue isthecommonalgorithmofheight(cm)orpressure(g/cm^2) pF=log$_{10}$(hc)

FACTORSAFFECTINGSOILSUCTION:

1. Particalsizeofsoil
2. Watercontent
3. Plasticityindexofsoil
4. Historyofdryingandwelting
5. Soilstructure
6. Temperature
7. Densenessofsoil
8. Angleofcontact
9. Dissolvedsaltsinporewater

PARTICALSIZEOFSOIL:

Smallerthesizeoftheparticles,smallerwillbetheporesizewithsmallradii, resulting in greater capillary rise and hence greater suction.

WATERCONTENT:

Smaller the water content, greater the soilsuction. Whendry, the soil suctionattain maximum.

PLASTICITYINDEXOFSOIL:

Forlesswatercontent,soilsuctionwillbegreaterinsoilwhichhasthegreater plasticity index than in the one which has low plasticity index.

HISTORYOFDRYINGANDWELTING:

Soilsuctionisgreaterduringdryingandlowersduringwetting**. SOIL**

STRUCTURE:

Thesizeofintercedesdependsonthesoilstructure.

Changeinstructureleadstochangeinsizeandthensoilsuction.

TEMPERATURE:

Riseintemperatureresultsindecreaseofsurfacetensionandtheninsoilsuction. Fall in

temperature increases the soil suction.

DENSENESSOFSOIL:

Increase in denseness of soil resultsin decrease in the size of the pores of the soiland hence increase in soil suction and vice versa.

ANGLEOFCONTACT:

Soilsuctiondecreaseswithincreasesintheangleofcontact,whichistheangle between soil and water.

Whenα=0,soilsuctionismaximum.

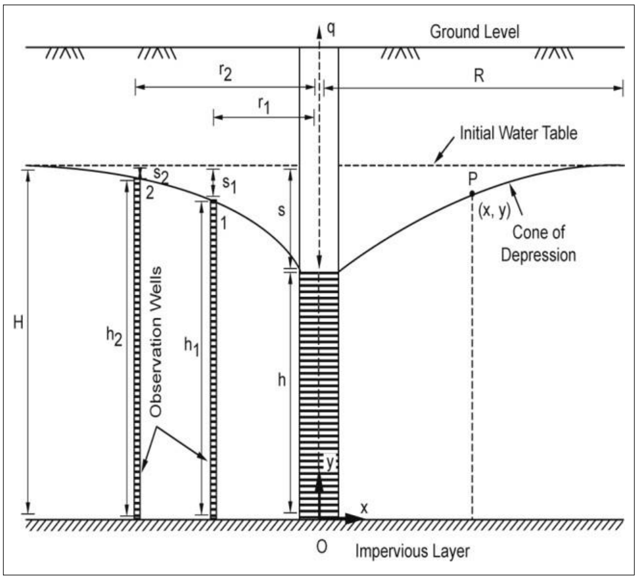
DISSOLVEDSALTSINPOREWATER:

Dissolvedsaltsincreasethesurfacetensionanditalsoincreasesthesoilsuction.

8 **Deriveanequationtodeterminethecoefficientofpermeabilityinan unconfined aquifer. (AU MAY/JUNE2013)**

Forthesakeofanalysis,weshalltaketwocases:(i)wellfullypenetratingan unconfined aquifer, (ii) well fully penetrating a confined aquifer.

Unconfined aquifer, Fig. shows a wellpenetrating an unconfined or free aquifer to its full depth



Let r= radiusofthewell

H=thickness of the aquifer,measured from the impermeable layer to the initial level of water table.

s=drawdownatthe well

h=depth of water in the well measured above the impermeable layer.

Considering the origin of co-ordinates at a point O at the centre of the well as its bottom, let the co-ordinates of any point P on the drawdown curve be (x, y).

Then from Darcy's law, Discharge $q = kA_x i_x$

Where A_x = area of cross-section of the saturated part of the aquifer at P

$= (2\pi x)y = 2\pi xy$

i_x = hydraulic gradient at P $= \frac{dy}{dx}$

Hence $q = k(2\pi xy)\frac{dy}{dx}$ or $q\frac{dx}{x} = 2\pi ky\,dy$

Integrating between the limits (R, r) for x and (H, h) for y, we get

$$q\int_r^R \frac{dx}{x} = 2\pi k\int_h^H y\,dy$$

$$\therefore\quad q(\log_e x)_r^R = 2\pi k\left(\frac{y^2}{2}\right)_h^H$$

From which $$q = \frac{\pi k(H^2 - h^2)}{\log_e \frac{R}{r}}$$

$$= \frac{1.36k(H^2 - h^2)}{\log_{10}\frac{R}{r}} \qquad \ldots(1)$$

If k is expressed in cubic meter per day per square meter (m^3/day/m^2) of the area of subsoil, the above expression for discharge will directly be in cubic meter per day units. In the above expression, R commonly known as radius of zero drawdown or maximum radius of influence, is the radius, measured from the center of the well, where the drawdown curve meets the original water table tangentially. In practice, the the selection of the radius of influence R is approximate and arbitary, but the variation in Q is small for a wide range of R. Suggested values of R fall in the range of 150 to 300 m.

Alternatively, R may be computed from the following approximate expression given by Sichardt: $R = 3000\,s\sqrt{k}$ …(2)

Where R and s are in meters, and k is in m/sec.

If there are two observation wells at radial distance r_1 and r_2 ($r_2 > r_1$) and if the depthsofwater inthemare $h_1$and $h_2$respectively. Eqn.(1)canalsobeexpressedin the following form.

$$q = \frac{\pi k(h_2^2 - h_1^2)}{\log_e \frac{r_2}{r_1}}$$

$$= \frac{1.36\, k(h_2^2 - h_1^2)}{\log_{10} \frac{r_2}{r_1}} \quad \ldots(3)$$

Ifthedrawdownsismeasuredatthewell,wehave

s=H– hand H= s+horH+h= s+2h Then, from

equation (1) , we get

$$q = \frac{\pi k(H-h)(H+h)}{\log_e \frac{R}{r}}$$

$$= \frac{\pi ks(s+2h)}{\log_e \frac{R}{r}}$$

$$= \frac{\pi ks(s+2L)}{\log_e \frac{R}{r}}$$

$$= \frac{1.36 ks(s+2L)}{\log_e \frac{R}{r}} \quad \ldots(4)$$

whereL=effectivelengthofthestrainer=h.

ASSUMPTIONSANDLIMITATIONSOFDUPUIT'STHEORY

Duputt'stheoryofflow isbasedonthe folowing assumptions:

1. The velocity of flow is proportional to the tangent of the hydraulic gradientinstead of its sine.
2. Theflowishorizontalanduniformeverywhereintheverticalsection.

3. Aquiferishomogeneous,isotropicandofinfiniteaerialextent.

4. Thewellpenetratesandreceiveswaterfromtheentirethicknessoftheaquifer.

5. Theco-efficientoftransmissibilityisconstantatallplacesandatalltimes.

6. Naturalgroundwaterregimeaffectinganaquiferremainsconstantwithtime.

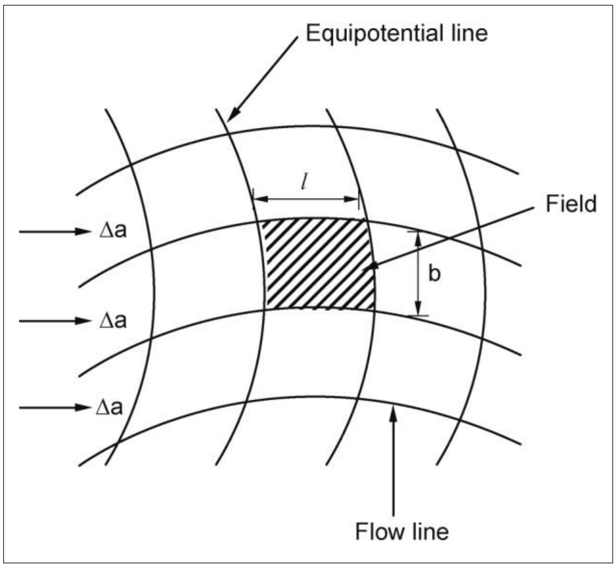
7. FlowislaminarandDarcy’slawisvalid.

Outofthese, assumptions 1,2and7are ofparticular importance. The flow is not horizontal, especially near the well. Also, the piezometric surface attains greater slope asit approaches the well boundary, with the result that assumption 1 is an approximation. Due to these reasons, the parabolic form of piezometric surface computed from the Dupuit’s theory deviates from the observed surface. This deviation is large, resulting in the formation of a seepage face. In addition to these, the velocity near the well increases, and the flow no longer remains laminar. Thus, Darcy’slawequationisnotvalidnearthewellface.

9 **Derive an equation for flow through soil and hence derive the equation to determine the seepage discharge through a flow net. Also prove the discharge through different flow channel is constant and the head loss between two different potential drop lines is constant.(AU MAY/JUNE2013)**

APPLICATIONOFFLOWNETFORDETERMINATIONOFSEEPAGE:

The figure shows a portion of flow net. The portion between any two successive flow lines is called at **flow channel**. The portion enclosed between two-sucessive equipotential lines and successive flow lines is known as field such as that shown hatched in Figure.



Letbandlbethe widthandlengthofthefield.

Δ h = head drop through the field; Δ q = discharge passing through the flow channel H = total hydraulic head causing flow = difference between upstream and downstream heads.

Then,fromDarcy'slawofflowthroughsoils:

$$\Delta q = k \cdot \frac{\Delta h}{l}(b \times 1) \text{(consideringunitthickness)}$$

If N_d=totalnumberofpotentialdropsinthecompleteflownetthen,

$$\Delta h = \frac{H}{N_d}$$

$$\text{Hence,} \Delta q = k \cdot \frac{H}{N_d}\left(\frac{b}{l}\right)$$

Thetotaldischargethroughthecompleteflownetisgivenby

$$q = \Sigma \Delta q = k \cdot \frac{H}{N_d}\left(\frac{b}{l}\right) \cdot N_f = kH \frac{N_f}{N_d}\left(\frac{b}{l}\right)$$

whereN_f=totalnumberofflowchannelsinthenet. The

field is square; hence b = l

Thus, $$b = kH \frac{N_f}{N_d}$$

This is the required expression for the discharge passing a flow net and is valid for isotropic soils in which $k_x = k_y = k$.

10 **The water table in a deposit of sand 8 m thick is at a depth of 3 m below the ground surface. Above the water table, the sand is saturated with capillary water. The bulk density of sand is 19.62 kN/m^3. Calculate the effective pressure at 1 m, 3m and 8m below the ground surface. Hence plot thevariation of total pressure, neutral pressure and effective pressure over the depth of 8 m. (AU NOV/DEC2012)**

Totalstressatthebottom,$\sigma = (3+5)\ \gamma_{sat}$

$$= 8 \times 19.62$$

$= 156.96 kN/m^2$

Neutral Pressure, $u = h_w . \gamma_w$

$= 5 \times 9.81$

$= 49.05 kN/m^2$

Effective Stress, $\sigma' = \sigma - u$

$= 15696 - 49.05$

$= 107.91 kN/m^2$

(i) STRESSATTHEGROUNDSURFACE

$\sigma = 0 \times \gamma_{sat}$

$= 0\ kN/m^2$

$u = -h_c \times \gamma_w$

$= -3 \times 9.81 = -29.43 kN/m^2$

$\sigma' = \sigma - u$

$= 0 - (-29.43) = 29.43 kN/m^2$

(ii) STRESSAT1mBELOWTHEGROUNDSURFACE

$\sigma = h.\gamma_{sat}$

$= 1 \times 19.62 = 19.62 kN/m^2$

$u = -h_c \times \gamma_w$

$= -2 \times 9.81 = -19.62 kN/m^2$

$\sigma' = \sigma - u$

$= 19.62 - (-19.62) = 39.24 kN/m^2$

	(iii)STRESSAT3mBELOWGROUNDLEVEL

$\sigma = h.\gamma_{sat}$

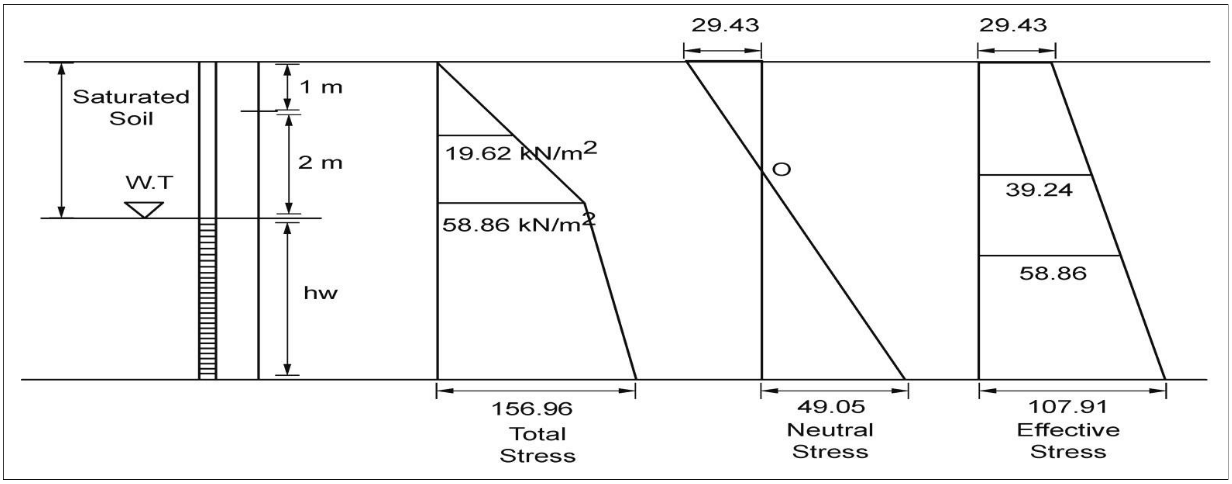
$= 3 \times 19.62 = 58.86 kN/m^2$

$u = 0 \times \gamma_w$

$= 0 kN/m^2$

$\sigma' = \sigma - u$

$= 58.86 - 0 = 58.86 kN/m^2$



11 **Write down the procedure for determinationof permeability by constant head test in the laboratory. (AU NOV/DEC2012)**

PURPOSE:

Thepurposeofthistestistodeterminethepermeability(hydraulicconductivity) ofa sandy

Soilbytheconstantheadtest method.Therearetwogeneraltypesofpermeability test methods

thatareroutinelyperformedinthelaboratory:

- Theconstantheadtestmethod,and(2)thefallingheadtestmethod.The constant head test method is used forpermeable soils ($k>10^{-4}$ cm/s)and

- Thefallingheadtestismainlyusedforlesspermeablesoils($k<10^{-4}$cm/s).

SIGNIFICANCE:

Permeability(orhydraulicconductivity)referstotheeasewithwhichwatercan flow through a soil. This property is necessary for the calculation of seepage

through earth dams or under sheet pile walls, the calculation of the seepage rate

fromwastestorage facilities(landfills,ponds,etc.),andthecalculationoftherate of

settlement of clayey soil deposits.

EQUIPMENT:

- Permeameter,
- Tamper,
- Balance,
- Scoop,
- 1000mLGraduatedcylinders,
- Watch(orStopwatch),
- Thermometer,
- Filterpaper.

TESTPROCEDURE:

(1) Measuretheinitialmassofthepanalongwiththedrysoil($\mathbf{M_1}$).

(2) Remove the cap and upper chamber of the permeameter by unscrewing the knurledcapnutsand liftingthemoffthetie rods.Measurethe insidediameterof upper and lower chambers. Calculate the average inside diameter of the permeameter (**D**).

(3) Placeoneporousstoneontheinnersupportringinthebaseofthechamberthen place a filter paper on top of the porous stone (see Photo C).

(4) Mix the soil with a sufficient quantity of distilled water to prevent the segregationofparticlesizesduringplacementintothepermeameter.Enoughwater should be added so that the mixture may flow freely (see Photo B).

(5) Usingascoop,pourthepreparedsoilintothe lowerchamber usingacircular motion to fill it to a depth of 1.5 cm. A uniform layer should be formed.

(6) Usethetampingdevicetocompactthelayerofsoil.Useapproximatelyten rams ofthe tamper per layer and provide uniformcoverage ofthe soilsurface. Repeat thecompactionprocedure untilthesoilis within2cm. ofthetopofthe lower chamber section (see Photo D).

(7) Replacetheupperchambersection,anddon’tforgettherubbergasketthatgoes between the chamber sections. Be careful not to disturb the soil that has already been compacted. Continue the placement operation until the level of the soil is about 2 cmbelow the rimof the upper chamber. Level the top surface of the soil and place a filter paper and then the upper porous stone on it (see Photo E).

(8) Placethecompressionspringontheporousstoneandreplacethechambercap and its sealing gasket. Secure the cap firmly with the cap nuts (see Photo F).

(9) Measurethesamplelengthatfourlocationsaroundthecircumferenceofthe

permeameterandcomputetheaveragelength.Recorditasthesamplelength.

(10) Keepthepanwithremainingsoilinthedryingoven.

(11) Adjustthe levelofthe funneltoallowtheconstantwater levelinittoremaina few inches above the top of the soil.

(12) Connectthe flexibletube fromthetailofthefunneltothebottomoutletofthe permeameter and keep the valves on the top of the permeameter open (see Photo G).

(13) Placetubingfromthetopoutlettothesinktocollectanywaterthat maycome out (see Photo G).

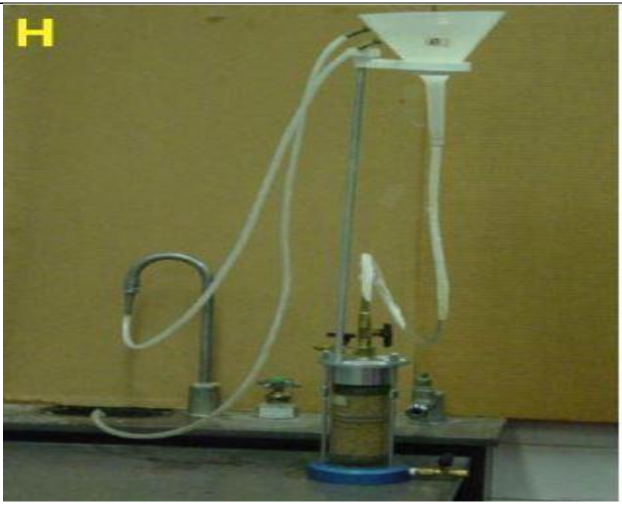
(14) Openthebottomvalveandallowthewatertoflow intothepermeameter.

(15) Assoonasthewaterbeginsto flowoutofthetopcontrol(de-airing)valve, close the control valve, letting water flow out of the outlet for some time.

(16) Closethebottomoutletvalveanddisconnectthetubingatthebottom.Connect the funnel tubing to the top side port (see Photo H).

(17) Openthebottomoutlet valveandraise the funneltoaconvenient heighttoget a reasonable steady flow of water.

(18) Allowadequatetimefortheflowpatterntostabilize(seePhotoI).



(19) Measurethetimeittakestofilla volumeof750 –1000 mlusingthe graduated cylinder, and then measure the temperature of the water. Repeat this process three times and compute the average time, average volume, and average temperature. Record the values as **t**, **Q**, and **T**, respectively (see Photo I).

(20) Measuretheverticaldistancebetweenthe funnelheadlevelandthechamber outflow level, and record the distance as **h**.

(21) Repeatstep17and18withdifferentverticaldistances.

(22) Removethepanfromthedryingovenand measurethe finalmassofthepan along with the dry soil ($\mathbf{M_2}$).

Analysis:

(1)Calculatethepermeability,usingthefollowingequation:

$\mathbf{K_T=QL/A*t*h}$

K_T=coefficientofpermeabilityattemperatureT,cm/sec.

L=lengthofspecimenincentimeters t =

time for discharge in seconds

Q=volumeofdischargeincm^3(assume1mL=1cm^3)

A=cross-sectionalareaofpermeameter(= $\mathbf{[(\pi D^2)/4]}$,D=insidediameterofthe permeameter)

h = hydraulic head difference across length L, in cm of water; or it is equal to the verticaldistancebetweentheconstantfunnelheadlevelandthechamberoverflow level.

12 **Thesubsoilstrataatasiteconsistof finesand1.8m hickoverlyingastratum of clay1.6mthick.Under theclaystratumlaysadepositofcoarsesandextending**

to a considerable depth. The water table is 1.5 m below the ground surface. Assuming the top fine sand to be saturated by capillary water, calculated the effective pressure at ground surface & at depth of 1.8m, 3.4m and 5.0m below the ground surface. Assume for fine sand G=2.65, e=0.8, and for coarse sand G=2.66, e=0.5. What will be the change in effective pressure at depth 3.4m, if no capillary water is assumed to be present in the fine sand and its bulk unit weight is assumed to be 16.68kN /m^3. The unit weight of clay may be assumed as19.32kN /m3(AU MAY/JUN2012)

Solution:

CASE1WITHOUTCAPILLARYRISE:

γ_{sat}forsand belowwT= $\frac{G+Se}{1+e}.\gamma w$

$= \frac{2.66+0.8}{1+0.8}.9.81$

$=18.8025 kN/m^3$

Totalstressat 3.4m

=(1.5*16.68)+(0.3*18.8)+(1.6*19.32)

$=61.57 kN/m^2$

Porewaterpressureat3.4m=(1.6+0.3)*9.81=18.63kN/m^2

Effective stress =61.57-18.63

$=42.93 kN/m^2$

CASE2WITHCAPILLARYRISE:

Totalstress=(1.8*18.8025)+(1.6*19.32)

$=63.35 kN/m^2$

Porepressure=(0.3*9.81)-(1.5*9.81)+(1.6*9.81)

$=3.92 kN/m^2$

Effectivestress=63.35-3.92

$=59.43 kN/m^2$

13 **In a constant head permeameter test, the following observations were taken. Distance between piezometer tappings=15cm, difference of water levels in piezometers=40cm, diameter of test sample =5cm, quantity of water collected=500ml, Duration of the test=900sec.Determine the co efficient of permeability of e soil. If the dry mass of the 15cm long sample is 486 kg and specific gravity of the solids are 2.65. Calculate seepage velocity of water during the test.(AU MAY/JUN2012)**

Solution:

Diameter d=5cm , Difference of water level in piezometer,h=40cm, Quantity Q=500ml,Duration t=900sec, Length L=15cm, Dry Mass m_d=486g, specific gravity G=2.65

COEFFICIENTOFPERMEABILITY,K:

Area,A=πr^2=19.63cm^2

$$K=\frac{Q}{L}*\frac{L}{N}*\frac{I}{A}$$

$$K=\frac{500}{900}*\frac{15}{40}*\frac{1}{19.63}$$

=10.61*10^{-3}cm/s

Seepagevelocity:

Now,

V=q/A

=500/(900*19.63)

=0.028cm/s

$P_d=M_d/V$

=486/(19.36*15)

=1.67g/cm^3

Thatis., $e=\frac{G.Pd}{Pd}-1 \quad =\frac{2.65*1}{1.67}-1$

e=0.583

	$n = \frac{e}{1+e} = \frac{0.583}{1+0.583}$ n=0.368 V_s=V/n =0.028/0.368 =$7.604*10^{-2}$cm/s

CE6405-SOILMECHANICS

UNITIIISTRESSDISTRIBUTIONANDSETTLEMENT

2MARKS:

1 **Write down the expression derived by westergaard for vertical pressure under the rectangular uniformly distributed loaded area. (AUNOV/DEC2014)**

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

2 **CompareBoussinesq’sandWestergaardanalysisforstressdistribution.**

(AUNOV/DEC2014)

Sl.No.	Boussinesq’sAnalysis	Westergaard’sAnalysis
1.	This is based on the assumptionthat,thesoil mass is homogenous.	This is based on the assumption that the soil massesaresedimentary(or) stratifieddeposits.
2.	The soil mass is isotropic.	The soil mass containing number of closely spaced horizontalsheetsofnegligible thickness.

3 **Definecoefficientofcompressibility.(AUNOV/DEC2013)**

Co-efficient of permeability (or hydraulic conductivity) is defined as the rate of flow of water under laminar flow conditions through a unit cross-sectional area of a porous medium under a unit hydraulic gradient and standard temperature conditions. The unit of ‘K’ is m/s (or) cm/s

$$q=KiA$$

4 **Whatisaninfluencediagram?Whatisitsuseinpractice?(AUNOV/DEC2013)**

A chart, consisting of number of circles and radiating lines, the influence of each area unit is the same at the centre of the circles, i.e. each area unit causes the equal verticalstressatthecentreofthediagram.Itisusedfordeterminingthevertical stressatanypointundera**uniformlyloadedarea**ofanyshape.

5 **Whatismeanbyeffectivestressofasoil?(AUMAY/JUN2013)**

Effective stress (σ′) is defined as the stress, transmitted throughgrainto grainat the pointofcontactthroughthesoilmass.Theloadperunitareaofsoilmass,

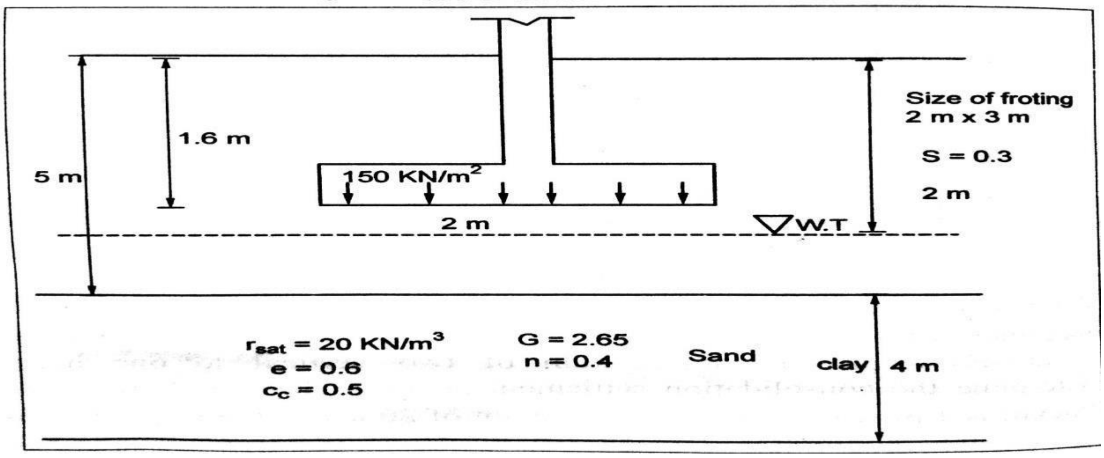
	responsibleforsoildeformationiscalledeffectivestress.
6	**Definesecondaryconsolidation.(AUMAY/JUN2013)(AUNOV/DEC2014)** When the excess neutral (water) pressure due to consolidation has been dissipated, the change in void ratio continues, and generally at a reduced rate. Thisphenomenon is called **Secondary compression** (or) **Secondary consolidation**.
7	**Write down Boussinesque equation for finding out the vertical stress under asingle concentrated load. (AUNOV/DEC2012)** Verticalstressunderapointloadσ_Z, $\sigma_Z = \frac{3Q}{2\pi Z^2}\left(\frac{1}{1+\left(\frac{r}{Z}\right)^2}\right)^{5/2}$ Where, r=Radialdistance fromaxisofloading Z=Depthatwhichthestressiscalculated
8	**Definenormallyconsolidatedclaysandoverconsolidatedclays.** **(AUNOV/DEC2012)AUNOV/DEC2014)** **Normally consolidated clay** is defined as the clayey soil, which has never been Subjected to an effective pressure greater than the present over burden pressure. **Overconsolidatedclay**istheclayeysoil,whichhasalwaysbeensubjectedtoan Effectivepressuregreaterthanthepresentoverburdenpressure.
9	**Whatisaninfluencediagram?Whatisitsuseinpractice?(AUMAY/JUN2012)** A chart, consisting of number of circles and radiating lines, the influence of each area unit is the same at the centre of the circles, i.e. each area unit causes the equal vertical stress at the centre of the diagram. It is used for determining the vertical stress at any point under a **uniformly loaded area** of any shape.
10	**Differentiatebetweencompactionandconsolidation. (AUMAY/JUN2012)**

Compaction	Consolidation
It refers to a more or less rapid reductionmainlyinair voidsundera loadingofshortduration	Consolidationisagradualprocessof volume reduction under sustained loading.

11	**Define'Isobar'.** An Isobar is a bulb shaped curve (or) contour allpaints below the ground surface of equal vertical pressure.

16 MARKS:

1	**A building column has a footing area of 2m*3m and transmits a pressure increment of 150kN/m^2 at its base embedded 1.6m below ground level. Assuming a pressure distribution of two vertical to one horizontal, determine the consolidation settlement atthe middle of the clay layer of thickness 4m with saturated unit weight of20kN/m^3 and initial void ratio of 0.6 with compression index 0.5 underlies fine sand deposit of 5m thickness. The water table is at a depth of 2m ground level. The degree of saturation of sand above water table is 30%. Take the specific gravity of fine sand as 2.65 and porosityis 40%.(AUNOV/DEC2014)** **Solution:** Forsand $\gamma_d = x = \frac{G.\gamma w}{1+e} = \frac{2.65*9.81}{1+0.6}$ $=16.25 kN/m^3$ $\gamma_{sat}=20kN/m^3$ Forclay $\gamma' = \gamma_{sat} - \gamma_w$ $= 20 - 9.81$ $=10.19kN/m^3$ (i) Initialpressureatthecentreofclaylayer: $\sigma' = (2*16.25)+3(20-9.81)+2(10.19)$ $\sigma' = 83.45kN/m^2$



(ii) Pressureincreaseatthetop middleandbottomofclay:

$(\Delta\sigma)_t = \frac{150(2*3)}{(2+3)(3+3)}$

$=30kN/m^2$

$(\Delta\sigma)_m = \frac{150(2*3)}{(2+5)(3+5)}$

$= 16.07kN/m^2$

$(\Delta\sigma)_b = \frac{150(2*3)}{(2+7)(3+7)}$

$=10kN/m^2$

Average pressure

$\Delta\sigma = 1/6[((\Delta\sigma)_t + 4(\Delta\sigma)_m + (\Delta\sigma)_b)]$

$=1/6\ [30+4(16.07)+10]$

$\Delta\sigma = \quad 17.38kN/m^2$

(iii) Finalsettlement:

$\rho_f = \frac{Cc}{1+e} H + \log_{10} \frac{\sigma+\Delta\sigma}{\sigma}$

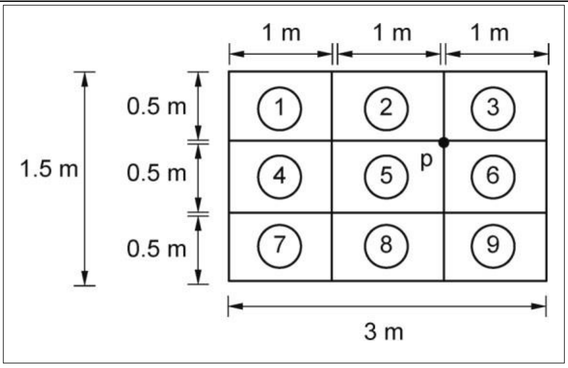
$= \frac{0.5}{1+0.6} H+4+\log_{10} \frac{83.45+17.38}{83.45}$

$=0.1m$

Settlementρ_f=100mm

2 **A concentratedload10kNactsonthesurfaceofasoilmass.UsingBoussinesq**

	analysis find the vertical stress at point (1) 3mbelowthe surface onthe axis of loading and (2) at radial distance of 2 m from axisof loading but at same depth of 3 m. (AUMAY/JUN2014) **CaseI** r= 0,z=3m,Q= 10kN $\sigma_z = \frac{3Q}{2\pi z^2}\left[\frac{1}{1+\left(\frac{r}{z}\right)^2}\right]^{5/2}$ $= \frac{3\times 10}{2\times\pi\times 3^2}\left[\frac{1}{1+\left(\frac{0}{z}\right)^2}\right]^{5/2}$ $=0.531 kN/m^2$ **CaseII** r=2mandz= 3m $\sigma_z = \frac{3Q}{2\pi z^2}\left[\frac{1}{1+\left(\frac{r}{z}\right)^2}\right]^{5/2}$ $= \frac{3\times 10}{2\times\pi\times 3^2}\left[\frac{1}{1+\left(\frac{2}{3}\right)^2}\right]^{5/2}$ $= \frac{3\times 10}{2\times\pi\times 9}[1+2.25^{5/2}]$ $=10.102 kN/m^2$
3	**A rectangular foundation 3.0 m × 1.5 m carries a uniform load of40kN/m^2.Determinetheverticalstressat'P'which is3mbelow theround surface (refer the figure). Use equivalent point load method.(AU NOV/DEC2013)**



The given foundation is divided into 9 equal and small areas of size (1.0 × 0.5) m

Loadoneacharea=40×(1.0×0.5)=20kN.

The stress at P are determined duel to 9 point loads, by using Boussinesq's Analysis.

Forloads(1)and(4), $r = \sqrt{1.5^2 + 0.25^2}$

$= 1.521$

and $\frac{r}{z} = 0.507$

Forloads(2),(3),(5)and(6),γ $\sqrt{0.5^2 + 0.25^2}$

$= 0.559$

and $\frac{r}{z} = 0.186$

Forloads(8)and(9), $V = \sqrt{0.75^2 + 0.5^2}$

$= 0.901$

and $\frac{r}{z} = 0.300$

Forload(7), $r = \sqrt{1.5^2 + 0.75^2}$

$= 1.677$

and $\frac{r}{z} = 0.559$

	andstress, $\sigma_z = \frac{\sum 3Q}{2\pi z^2} \times \frac{1}{\left[1+\left(\frac{r}{z}\right)^2\right]^{5/2}}$ Inthis problem, $\sigma_z = \frac{3 \times 20}{2\pi(3)^2}$ $=1.061(1.129+3.674+1.612+0.507)$ $=7.34 kN/m^2$
4	**Explain the Terzaghi's Theory of One Dimensional Consolidation? (AUNOV/DEC2013)(AUMAY/JUN2013)(AUNOV/DEC2012)(AUMAY/JUN 2012)** Terzaghi's theoretical concept of the consolidation process was developed by Terzaghiinthedevelopmentofthe mathematicalstatementoftheconsolidation process. The following **ASSUMPTION**: • soilhomogenousandfullysaturated • Deformationofthesoilisdueentirelytochangeinvolume • Darcy'slawforthe velocityofflowofwaterthroughsoilisperfectlyvalid. • Coefficientofpermeabilityisconstantduringconsolidation • Loadisapplieddeformationoccursonlyindirection • Thechangeinthicknessofthelayerduringconsolidationisinsignificant. $h = \frac{\bar{u}}{\gamma_w}$ .....1 Hydraulicgradient, $i = \frac{\partial h}{\partial z} = \frac{1}{\gamma_w} \cdot \frac{\partial \bar{u}}{\partial z}$ .....2 $V = Ki = K \frac{1}{\gamma_w} \cdot \frac{\partial \bar{u}}{\partial z}$ .....3 Thevelocityvwithwhichtheexcessporewaterflowatdepthz,

$V = ki$

$V = Ki = \frac{K}{\gamma w} \cdot \frac{\partial \bar{u}}{\partial z}$ 4

RateofChangeofvelocity

$\frac{\partial h}{\partial z} = \frac{K}{\gamma w} \cdot \frac{\partial \tilde{a}^2}{\partial z^2}$

Thevelocityoftheexistwillbeequalto $V + \frac{\partial v}{\partial z}.dx$

Thequantityofwater leavingsoilelements

$V + \frac{\partial v}{\partial z}.dz$

$\Delta q = \frac{\partial v}{\partial z} dxdydz$ 5

$\Delta v = -mvV_0 \Delta \sigma'$ 6

V_0=volumeofsoilelementattimet$_0$=dxdydz Change

of volume per unit time,

$\frac{\partial(\Delta v)}{\partial t} = -m_v dxdydz \frac{\partial(\Delta \sigma')}{\partial t}$ 7

Byequating(5)&(7),

$\frac{\partial v}{\partial z} = -m_v \frac{\partial(\Delta \sigma')}{\partial t}$ 8

$\Delta \sigma = \Delta \sigma' + \bar{u}$

$\frac{\partial(\Delta \sigma')}{\partial t} = \frac{-\partial u}{\partial t}$ 9

From(8)&(9)

$\frac{\partial v}{\partial z} = m_v . \frac{\partial \bar{u}}{\partial t}$ 10

Equate(6)and(10)weget

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

C_v=co-efficientofconsolidation = $\frac{k}{mv.\gamma w}$

$$C_v = \frac{k(1+e0)}{av.\gamma w}$$

Theaboveequationisthebasicdifferentialequationofconsolidation.

5.Withneatsketchesexplaintheproceduresofdeterminationofeffectivestress by newmark chart method. (AU MAY/JUN2013)(AU NOV/DEC2014)

Newmarksuggestedthata moreaccurate methodofdeterminigthe verticalstressat any point under a uniformly loaded area of any shape is with a help of influence chart or diagram

It consists of number ofcircle and radiating lines, is prepared that the influence of eachareunitwhichis formedinashapeofasectorbetweentwoconcentriccircles and two adjacent radial lines is same at the centre of circle,

Eachunitareacausesequalverticalstressatthecentreofadiagram. Let a

uniformly load circular radius r,cmbe divided in to 20sectors Vertical

pressure under a uniformly loaded area for (20 sectors)

$$\frac{\sigma_z}{20} = \frac{q}{20}\left[1 - \left\{\frac{1}{1+\left(\frac{r_1}{z}\right)^2}\right\}^{3/2}\right]$$

$$= i_f q$$

Wherei_f=influencevalue

$$1 - \left\{\frac{1}{1+\left(\frac{r_1}{z}\right)^2}\right\}^{3/2}$$

q=intensityofloading σ
= vertical pressure

z=depthzbelowthecentre

r_1=radiusoffirstconcentriccircle

eachunitsuchasOA_1B_1exertsapressureequalto$\sigma_Z/20$atcentre

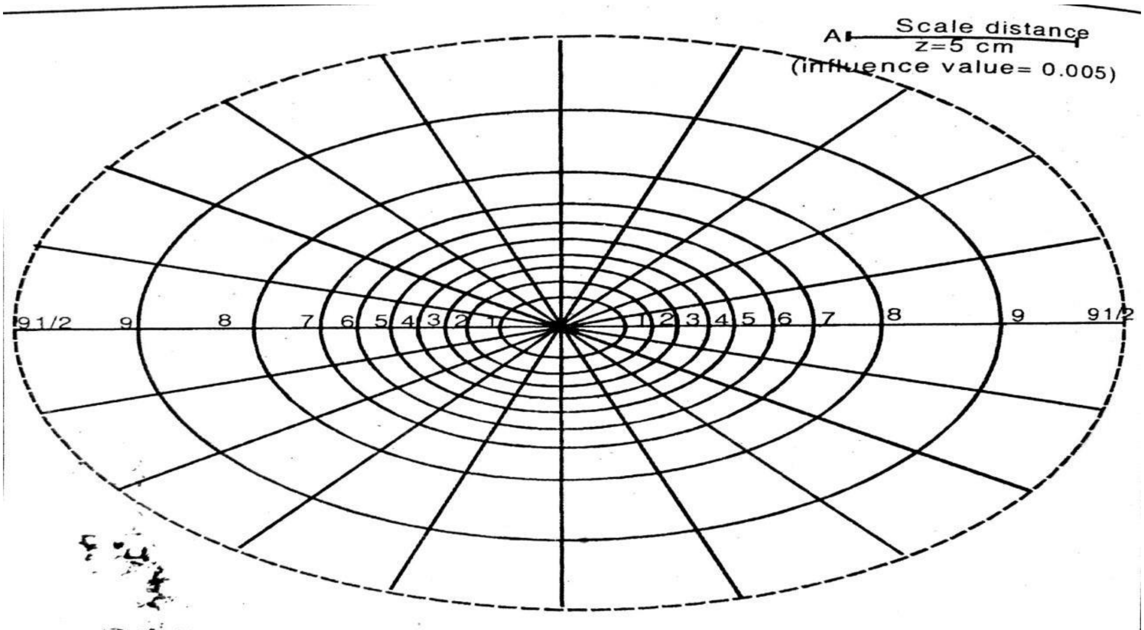


Fig. Newmark's Influence Chart

Ifi$_f$bemadeequaltoarbitrarilyfixedvalue,say0.005,wehave

$$\frac{\sigma_z}{20} = i_f q$$

$$\frac{\sigma_z}{20} = 0.005q$$

Leta$_2$betheradiusofsecondconcentriclines.Byextendingthetwentyradiallines, thespacebetweenthetwoconcentriclineisagaindivided into20sectors. $A_1A_2B_1$ B_2 is one are unit. The vertical pressure at the centre due to each ofthese area with is to be intensity 0.0059.

ThereFore.,Verticalpressuredueto

$$A_2 B_2 = \frac{q}{20}\left[1 - \left\{\frac{1}{1+\left(\frac{r_2}{2}\right)^2}\right\}^{3/2}\right] = 2 \times 0.005q$$

SubZ_1=5cm,wegetr_1=1.35,r_2=2.00cm

Similarlybyusingthe above relationwe canfindthe radiiof3rd 5th 6th 7th8th9th circles can be calculated, as tabulated in the given table.

Table:RadiiofconcentriccirclesforinfluencechartZ=5cm;i_f=0.005

No.of circle	1	2	3	4	5	6	7	8	9	10
Radius (cm)	1.35	2.00	2.59	3.18	3.83	4.59	5.54	6.94	9.54	∞

To use he chart for determining the vertical stress at any point under the loaded area,theplantofhisareais firstdrawnona tracingpapertosuchascalethatAB drawn on a chart that represents the depth to the point which the pressure is required.

The plan of the loaded area is then so placed over the chart that the point below pressure isrequiredcoincideswiththecentreofthechart.Thepointbelowwhich pressure is required may lie inside or outside the loaded area.

Thetotalnumberofarea unitscoveredbytheplanofthe loadedarea iscounted. The

vertical pressure is then calculated from the relation,

$$\sigma_A = 0.005q * N_A$$

WhereN_A=Numberofareaunitsunderthe loadedarea.

6 **A concentrated point load of 200 kN acts at the ground surface. Find the intensity of vertical pressure ata depthof 10 meters belowthe groundsurface, and situated on the axis of the loading. What will be the vertical pressure at a pointata depthof 5m andata radialdistance of 2 mfromthe axis of loading? Use Boussinesq analysis. (AU NOV/DEC2012)**

CaseI

Q=200kN

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

Where γ=0andz=10m

$$\sigma_Z = \frac{3\times 200}{2\times\pi\times 10^2}\left(\frac{1}{1+\left(\frac{0}{z}\right)^2}\right)^{5/2}$$

$$= 0.955 kN/m^2$$

	CaseII Q=200 kN z=5mandr=2m $\sigma Z=\frac{3\times Q}{2\pi z^2}\left(\frac{1}{1+\left(\frac{r}{z}\right)^2}\right)^{5/2}$ $=\frac{3\times 200}{2\times\pi\times 5^2}\left(\frac{1}{1+\left(\frac{2}{5}\right)^2}\right)^{5/2}$ $=2.636 kN/m^2$
7	**The load of continuous footing of width 2m, which may be considered to be strip load of considerable length, is 200kN /m^2.Determine the maximum principal stress at 1.5m depth below he footing, if he point lies (i) directly belowthecentreofthefooting(ii)directlybelowtheedgeofthefootingand (iii)0.8mawayfromtheedgeofthefooting.(AUMAY/JUN2012)** **(i) Directlybelowthecentreofthefooting(A_1):** $\Theta_0/2=\tan^{-1}(1/2)=0.463 rad$ $\Theta_0= 0.463*2 rad$ $\Theta_0=0.926 rad$ Principalstress, $\sigma_1=q/\pi(\Theta_0+\sin\Theta_0)$ $\sigma_1=200/\pi(0.926+0.799)$ $=109.81 kN/m^2$ Sin Θ $_0=\sin(0.926+)\ \frac{80}{\pi}$ =0.799 **(ii) Directlybelowtheedge ofthefooting(A2):** $\Theta_0=\tan^{-1}(2/2)=45^0=0.786 rad$ σ_1 = $(200/\pi).(0.785+0.706)$

	$=94.92kN/m^2$ **(iii) 0.8maway (A_3):** $\Theta_1 = \tan^{-1}(0.8/2) = 21.8 = 0.380$ rad$\Theta_2 =$ $\tan^{-1}(2.8/2) = 54.46 = 0.950$ rad$\Theta_0 = (\Theta_2 - \Theta_1) = 0.950 - 0.380 = 0.570$rad $\sigma_1 = (200/\pi).(0.570 + 0.539)$ $=70.64kN/m^2$
8	**Whatarethedifferentcomponentsofsettlement?Explainindetail.** **(AUMAY/JUN2012)** **Componentofsettlement:** A stress increase caused by the construction of foundations or other loads compresses soil layer. Thecompressioniscausedby • Deformation • Relocations • Expulsionofwaterorair Itcanbedividedintothreecategories • Immediatesettlement • Primaryconsolidationsettlement • Secondaryconsolidationsettlement **IMMEDIATESETTLEMENT:** It is also known as elastic settlement which is caused by the elastic deformation of dry soil and of moist and saturated soils without change in the moisture content. **PRIMARYCONSOLIDATIONSETTLEMENT:** Primary consolidation settlement, which is the result zero, a volume change in saturated cohesive soils because of expulsion of the water that occupies the void spaces.

	SECONDARYCONSOLIDATIONSETTLEMENT: Secondary consolidation settlementwhichisobservedin saturatedcohesivesoils andistheresultoftheplasticadjustmentofsoilfabrics.Itisan additionalform,of compressionthatoccursatconstanteffectivestress.
9	**Findintensity ofverticalpressureatapoint3mdirectlybelow25kN pointload acting on a horizontal ground surface. What will be the vertical pressure at a point 2m horizontally away from the axis of loading and at same depth of 3? Use Boussinesq's equation. (AU MAY/JUN2016)** $\sigma_z = \frac{Qr}{Z^2} * Kb$ $\sigma_z = \frac{25}{3^2} * \frac{1}{[1+(\frac{r}{z})^2]^{\frac{5}{2}}} * \frac{3}{2\pi}$ $= \frac{25}{3^2} * \frac{1}{[1+(\frac{2}{3})^2]^{\frac{5}{2}}} * \frac{3}{2\pi}$ $= 2.777 * \frac{1}{2.507} * 0.477$ $= 0.528 kN/m^2$
10	**ListtheBoussinesq'stheoryassumptionsandlimitations.(AUMAY/JUN2016)** **Assumptions of Boussineq's Theory:** The followingaretheassumptionsmadeinthesolutionbythetheoryofelasticity (i) The soil mass is an elastic medium for which the modules of elasticity Eis constant. (ii) The soil mass is homogeneous that is, allits components constituent parts (or) elements are similar and it has identical properties at every point in it in identical direction. (iii) The soil mass is isotropic that is, it has identical elastic properties in all direction through any point of it. (iv) The soil mass is semi infinite, that is it infinitely extends in all direction below a level surface **LimitationsofBoussinesq'sTheory:** By this method only vertical computations can be calculated whereas horizontal computations cannot be done since because the horizontalcomputations will not be accurate.

11	**Explainthefollowing.** **(i) Verticalstressdistributiononahorizontalplane.** **(ii) Verticalstressdistributionalongaverticalplane.** **(i) VerticalStressDistributiononaHorizontalPlane** TheverticalpressuredistributiononanyhorizontalplaneatadepthZbelow the ground surface, due to a concentrated load, is given by $\sigma_Z = K_B$ Depth zisaknowndepth.Selectingdifferent valueof horizontaldistance r, K_B can be found. Below the load, the vertical pressure will be equal to $0.4775\frac{Q}{z^2}$,anditdecreasesveryrapidlywiththeincreaseinthevalueof r, if sucha diagram is plotted for unit load (Q = 1), it is called the influence diagram for point A below the axis. **(ii) Verticalpressuredistributiononverticalline** On any vertical line distant r from the axis of the load, the variation of σ_Z can be plotted from the relation; $\sigma_Z = K_B\frac{Q}{z^2}$ and$\sigma_Z$decreasewithincreaseindepthz In the above expression, the radial distance r associated with K_B isconstant.Hencevariousvaluesofzand$\frac{r}{z}$canbeselected,and$K_B$canbe found.Then$\sigma_Z$canbecomputed,whichwillbeproportionalto K_B/z^2. Theverticalstressfirstincreases,attainsamaximumvalue,andthen decreases.

DEPARTMENT:CIVILENGINEERING
SEMESTER: IV
SUBJECTCODE/Name:C6405/SOILMECHANICS

UNIT –4

2MARKQUESTIONSANDANSWERS

1. Whatarethetestsavailablefordeterminetheshearstrength?(NOV/DEC2012)

a) Directsheartest
b) TriaxialSheartest
c) Unconfinedcompressiontest
d) Vanesheartest

2. **Whataretheadvantagesfordirectshear test? ((NOV/DEC2012)**

1. As testprogress,theareaunder sheargradually decreases.The correctedareaat failure should be used in determining the values of σ andτ.
2. Ascomparetotriaxialtest,there islittle controlon thedrainageofsoil.
3. Theplaneofshearfailureispre determined whichmaynotbe the weakestone.

3. Whatareadvantagesoftriaxialtests? (NOV/DEC2011)

1. The sheartest underallthe threedrainage conditionscanbeperformed with complete control
2. The precise measurementsoftheporepressure and volume changeduring thetest are possible.
3. Thestressdistributiononthefailureplaneisuniform
4. The stateof stress within thespecimenduring any stageof stress, aswellas at failure is completely determines.

4. Ifangleofinternalpressureofasoilis36^0.Findtheanglemade byfailureplanewith respect to minor principle plane. (NOV/DEC2011)

Theanglemadebyfailureplanewithrespecttominorprincipleplane

$$=\frac{90-36}{2}$$

$=27^0$.

5. CandΦare notfundamentalparameters.Butonlymathematicalparametersofsoil. Why? (NOV/DEC2010)

Research showed that the parameters C and Φ are not necessity fundamentalproperties of t he soil as was originally assured by Coulomb. These parameters depend upona number of factors, such as water content drainage conditions.

The current practice is to consider C and Φ as mathematical parameters whichrepresent the failure conditions for a particular soil under conditions. That is the reason why C and Φ are now called cohesion intersects and angel of shearing resistance.

6. WhatareporepressureparametersandwritedownKempton'sporepressure equation? (NOV/DEC2010)

Porepressureparametersexpressresponseof pore pressuredue change in thetotal stress under un-drained condition.

Kempton'sporepressureequation

$\Delta u = B(\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)$

WhereAandBaretheKempton's porepressure parameters

7. State coulombs equation for determination of sear strength of soil both for total and effective stress condition. (APRIL/MAY2012)

1. fortotalstress

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

Where,c-cohesion

σ-Totalstress

ϕ-Angleofinternalfriction

2. foreffectivestress

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

Where, σ'=effectivestress

8. Whatarelimitationsofdirectsheartest? (APRIL/MAY2012)

1. The stressconditionsare knownonlyatfailure .the conditionspriortofailure are undermine and there, the Mohr circle cannot be drawn

2. The stressdistributiononthefailure isnotuniform.the stressare moreat theedgesand lead to the progressive failure, like tearing of a paper.

3. The area under shear gradually decreases as the test progress. But the corrected area cannotbe determined and therefore,theoriginalarea istake isforthe computationof stress.

4. Theorientationof thefailureplaneis fixed.theplane maynotbe theweakestplane.

9. Define:normallyconsolidatedsoil (APRIL/MAY2011)

A normallyconsolidatedsoiliswhichhadnot been subjected to apressuregreaterthan the present exiting pressure.

10. Define:overconsolidatedsoil (APRIL/MAY2011)

A soilistobeoverconsolidated ithad been subjectedithad been subjected in thepast to a pressure in excess of the present pressure.

11. Whatarethebasiccomponentsareconstitutedinshearingresistanceofsoil ?

(APRIL/MAY2010)

Theshearingresistanceofsoilisconstitutedbasicallyofthefollowingcomponents:

The structuralresistance todisplacementofthe soilbecause ofthe interlocking ofthe particles,
Thefrictionalresistancetotranslocation between the individualsoilparticlesattheir contact points, and
Cohesionoradhesionbetweenthesurfacesofthesoilparticle

16MARKQUESTIONSANDANSWERS

1. **ExplaintheMohr'sstresscircle** **(APRIL/MAY2011)**

Through a point in a loaded soil mass, innumerable planes passand stress components on eachplane depends upon the directionof theplane. It canbe shown that there exist three typical planes,mutually orthogonal to each other, on which the stress is wholly normal andno shear stress acts.

These planes are called the principal planes and the normal stresses acting on these planes are called the principal stresses. In the order of decreasing magnitude of the normal stress, these planes are called major, intermediate and minor principal planes and the corresponding normal stresses on them are called major principal stress $\sigma 1$, intermediate principal stress $\sigma 2$ and minor principal stress $\sigma 3$. Many problems in soil engineering can be approximated by considering two dimensional stress conditions.

Fig. Shows a soil element subjected to two dimensional stress system. From the consideration of the equilibrium of the element, one gets the following expressions for the normal stress $\sigma$and shearing stress τon any plane MN inclined at an angle α with Thex direction:

$$\sigma=\frac{\sigma_y+\sigma_x}{2}+\left(\frac{\sigma_y-\sigma_x}{2}\right)\cos 2\alpha+\tau_{xy}\sin 2\alpha \qquad \text{...(4.1)}$$

And

$$\tau=\frac{\sigma_y-\sigma_x}{2}\sin 2\alpha+\tau_{xy}\cos 2\alpha \qquad \text{...(4.2)}$$

Whereσyandσx =normalstressesonplanesperpendiculartoyandxaxes, respectively **($\sigma y>\sigma x$),$\tau xy(=\tau yx)$=shearstressesonthesetwoplanes**

Squaring Eqs.4.1 and 4.2andadding,wegetthefollowing results:

$$\left(\sigma-\frac{\sigma_y+\sigma_x}{2}\right)^2+\tau^2=\left(\frac{\sigma_y-\sigma_x}{2}\right)^2+\tau_{xy}^2 \qquad \text{...(4.3)}$$

Eq.4.3istheequationof a circlewhose centrehasco-ordinates

$\left(\frac{\sigma_y+\sigma_x}{2},0\right)$,and whoseradiusisequalto $\sqrt{\left\{\frac{1}{2}(\sigma_y-\sigma_x)\right\}^2+\tau_{xy}^2}$

1. The co-ordinates of points on the circle represent the normal and shearing stresses on inclinedplanesatagivenpoint.ThiscircleisknownasMohr'scircleofstress(Mohr, 1870).

2. Todrawthe Mohrcircle,the normalstresses σx and σy aremarked on theabscissa, at points B and A and a circle is drawn with point C, mid-way between A and B, as the centre, with radiusequal to CB1=CA1 where BB1and AA1 arethe perpendiculars drawn at B and A of magnitude equal to τxy.

3. Shows the Mohr circle so drawn. Te co-ordinates of any point F (σ, τ) represent the stress conditions on plane which makes an angle α with the x direction.

4. If from a point B1on a circle representing the state ofstress on vertical plane, a lineis drawn parallel tothis plane (i.e vertical), it intersects the circle ata point P. Also, if from the

point A1 on the circle representing the stresses on the horizontal plane, a line is drawn parallelto thislatterplane (i.e horizontal)it willalso intersect the circle in the same pointP. In general, if through a point F representing the stresses on a given plane, a line is drawn parallel to that plane, it will also intersect the circle in the point P. The point P is therefore, a unique point called the origin of planes or the pole.

Let us now take the case of soil element whose sides are the principal planes, i.e consider the state ofstress where only normal stresses are acting on the faces ofthe element

The major principal plane is horizontal. Hence the pole P is located by drawing a horizontalline through point A this intersects the circle atB. If a linePF is drawn through Pat an angle α with the horizontal, it will intersect the circle at F which represents the stress conditions on a plane inclined at an angle α with the direction of the major principal plane.

An element in which the principal planes are not horizontal and vertical, but are inclined to y and x-directions. Point A represents the major principal stress (σ_1, 0) and B represents the minor principal stress (σ_3, 0). Hence to get the position of the pole, a line is drawnthrough a,parallelto the majorprincipalplane, to intersect the circle inP. Evidently, PB gives the direction of minor principal plane. To find the stress components on any plane MNinclined at an angle α with the major principal plane, a line is drawn through P, at an angle α with PA, to intersect the circle at F. The co-ordinates (σ, τ) of point F give the stress components on the plane MN. Analytical expression for σ, $\tau$are :

$$\sigma = \frac{\sigma_1+\sigma_3}{2} + \frac{\sigma_1-\sigma_3}{2}\cos 2\alpha \quad ...(4.4)$$

$$\tau = \frac{\sigma_1+\sigma_3}{2}\sin 2\alpha \quad ...(4.5)$$

Theresultantstress onanyplaneis$\sqrt{\sigma^2+\tau^2}$anditsangleof obliquityβis equalto $\tan^{-1}\left(\frac{\tau}{\sigma}\right)$

The maximumshear stress(pointG)τ_{max} isequalto$\frac{\sigma_1-\sigma_3}{2}$anditoccursonplanes withα=45°.InFig.4.2(b),PGshowsthedirection ofplanehavingmaximumshearstress.

Thenormalstressonthisplanewillbeequalto $\frac{\sigma_1+\sigma_3}{2}$

2. ExplaintheMohr-coulombfailuretheory (APRIL/MAY2010)

1. Materialfailsessentiallybyshear. Thecriticalshearstresscausingfailuredependsupon the properties of the material as well as on normal stress on the failure plane.
2. Theultimate strength of the materialisdeterminedbythestressesonthepotentialfailure plane (or plane of shear)
3. When the material is subjected to three dimensional principal stress (i.e.σ_1, σ_2, σ_3) the intermediate principal stress does not have any influence on the strength of material. In other words, the failure criterion is independent of the intermediate principal stress.

Note.Fordetaileddiscussionsonvarioustheoriesoffailure,seeChapter19,where the effect of the intermediate principal stress has also been discussed.

The theory was first expressed by Coulomb (1776) and later generalized by Mohr. The theory can be expressed algebraically by the equation.

$$\tau_f = s = F(\sigma)$$

Where

τf =s= shearstressonfailureplane,atfailure= shearresistanceof material F (σ) = function of normal stress

If the normal and shear stress corresponding to failure are plotted, then a curve is obtained. Theplot orthe curve iscalled thestrength envelope. Coulomb definedthefunction F (σ) as a linear function of $\sigma$and gave the following strength equation:

$$s = c + \sigma \tan\phi$$

Where, the empirical constants c and ϕrepresent respectively, the intercepts on the shear axis, and the slope of the straight line of Eq. 4.7 Fig . These parameters are usually termed as cohesion and angle of internal friction or shearing resistance respectively.
Fig. 4.3 (b) shows the Mohr's envelope, which is the graphical representation of Eq. 4.6. Coulomb considered that the relationship betweenshear strength and normal stress could be adequately represented bythe straight line. The generalized Mohrtheoryalso recognizes that the shear strength depends on the normal stress, but indicates that the relation is not linear. The strength theory upon which the Coulomb and Mohr strength lines are based indicates that definite relationship exists among the principal stresses, the angle of internal friction and the inclination of the failure plane. The curved failure envelope of Mohr is often referred to as a straight line for most of the calculations regarding the stability of soil mass. Foranidealpurefrictionmaterial, sucha straight linepasses through theorigin[Fig.4.4 (a)]. However, dense sands exhibit a slightly curved strength line, indicated by dashed line. Fig. 4.4 (b) represents purely cohesive (plastic) material, for which the straight line is parallel to the σ-axis. The strength of such a materialis independentof thenormalstress acting on the plane of failure. The way in which a straight line is fitted to a Mohr envelope will depend on the range of α which is of interest.

It can, therefore, be concluded that the Mohr envelope can be considered to be straightif theangleof internalfriction ϕisassumed to be constant. Depending upon theproperties of a material the failure envelope may be straight or curved, and it may pass through the origin of stress or it may intersect the shear stress axis.

3. Explaintheeffectivestressprinciple. (NOV/DEC2010)

In Eq. 4.7, it is assumed that the total normal stress governs the shear strength of soil. This assumption is not always correct. Extensive tests on re-mould clays have sustained beyond doubt Terzaghi's early concept that the effective normal stresses control the shearing resistance of soils. Therefore, a failure criterion of greater general applicability is obtained by expressing the shear strength as a function of the effective normal stress

σ',givenbytheequation: $\tau f = c' + \sigma' \tan\phi'$

or $\tau f = c' + (\sigma - u) \tan\phi'$

wherec'=effectivecohesionintercept;andϕ'=effectiveangleof shearing resistance In terms of total stresses, the equation takes the form:

$$\tau f = cu + \sigma \tan \phi u$$

Wherecu=apparentcohesion;ϕu =apparentangleofshearingresistance.

The normal stress σ' and shear stress τ on any plane inclined at an angle α to the major principal plane can be expressed in terms of effective major principal stress $\sigma 1'$ and effective minor principal stress $\sigma 3'$ from Eq. 4.4 And 4.5 as under:

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

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

Substituting thevaluesofσ'inEq.4.8,we get

$$\tau_f = c' + \tan\phi' \left(\frac{\sigma_1' + \sigma_3'}{2} + \frac{\sigma_1' - \sigma_3'}{2} \cos 2\alpha \right)$$

The mostdangerous plane i.e,the plane on whichfailurewilltakeplaceis theoneonwhich the difference (τf - τ), between the shear strength and shear stress is minimum.

$$(\tau_f - \tau) = c' + \frac{\sigma_1' + \sigma_3'}{2}\tan\phi' + \frac{\sigma_1' - \sigma_3'}{2}\cos 2\alpha \tan\phi' - \frac{\sigma_1' - \sigma_3'}{2}\sin 2\alpha$$

Differentiatingthiswithrespecttoα,weget

$$\frac{d}{d\alpha}(\tau_f - \tau) = -(\sigma_1' - \sigma_3')\sin 2\alpha \tan\phi' - (\sigma_1' - \sigma_3')\cos 2\alpha$$

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

Thisgives $\cos 2\alpha = -\sin 2\alpha \tan\phi'$ or $\cot 2\alpha = -\tan\phi' = \cot(90° + \phi')$

$$\alpha = \alpha_f = 45° + \frac{\phi'}{2} \quad \text{....(4.14)}$$

The above expression for the location ofthe failure plane can be directly derived from theMohrcircle (Fig.4.5). J Frepresentsthefailure envelopegivenbythe straight line$\tau f = c' + \sigma' \tan\phi'$. ThepoleP will be thepointwithstress co-ordinates as ($\sigma 3'$, 0). TheMohr circle is tangential to the Mohr envelope at the point F. PF represents the direction of the failure plane, inclined at an angle αf with the direction of the major principal plane. From the geometry of Fig. 18.5, we get from triangle JFK

$$2\alpha f = 90° + \phi' \quad \text{or} \quad \alpha f = 45° + \frac{\phi'}{2}$$

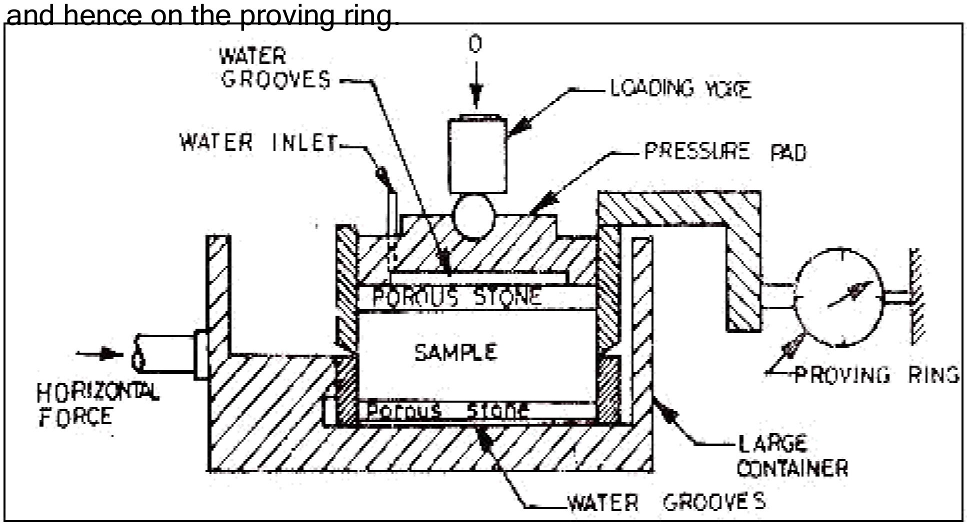
It should be noted that for any combination of the applied principle effective stress$\sigma 1'$ and $\sigma 3'$, failure will occur only of the stress circle touches the failure envelope. Also, the coordinates of thefailure point F represent the stress componentsσ' and τf at failure. As it is evident from Fig. 4.5, the τf at failure is less than the maximum shear stress, corresponding to the point G, acting on the plane PG. Thus, the failure plane does not carry maximumshear stress, and the plane which has the maximum shear stress is not the failure plane.

4. Explainthedirectshear test. (APRIL/MAY2011)

This is a simple and commonly used test and is performed in a shear-box apparatus(Fig. 4.6). The apparatus consists of a two piece shear box of square or circular cross- section. The lower half of the box is rigidly held in position in a container which rests over slides or rollers and which can be pushed forward at a constant rate by geared jack, driven either by electric motor or by hand.

The upper half of the box butts against a proving ring. The soil sample is compacted in the shear box, and is held between metal grids and porous stones (or plates). As shown in Fig. 4.6 (a), the upper half of the specimen is held in the upper box and the lower half in the lower box, and the joint between the two parts of the box is at the level of the centre of the specimen.

Normal load is applied on the specimen from a loading yoke bearing uponsteel ball of pressure pad. When a shearing force is applied to the lower box through the feared jack, the movement of lowerpart of the box is transmitted through the specimen to the upper part of the box and hence on the proving ring.



The deformation of proving ring indicates the shearforce. The volume changeduring the consolidation and during the shearing process is measured by mounting a dial gauge at the top of the box. The soil specimen can be compacted in the shear box by clamping both the parts together with the help of two screws.

These screws are, however, removed before the shearing force is applied. Metal grids, placed above the top and below the bottom of the specimen may be perforated if drained test is required, or plain if un-drained test is required. The metal grids have linear slots or

serrations to have proper grip with the soil specimen, and are so oriented that theserrations are perpendicular to the direction of the shearing force.

`Thespecimen ofthe shear boxis sheared under a normal loadN.The shearingstrain is made to increase at a constant rate, and hence the test is called the strain controlled shear box test. The other type of test is the stress controlled shear box test, in which thereis an arrangement to increase the shear stress at a desired rate and measure the shearing strain. Fig. 4.6 (a) shows the strain controlled shear box.

`The shear force, F, at failure, corresponding to the normal load N is measured with the help of the proving ring. A number of identical specimens are tested under increasing normal loads and the required maximum shear force is recorded. A graph is plottedbetween the shear force F as the ordinate and the normal load N as the abscissa. Such a plotgivesthefailureenvelope for thesoil under the giventestconditions.Fig.4.6 (c) shows such a failure envelope plotted as a function of the shear stress s and the normal stressσ. The scales of both s and $\sigma$are kept equal so that the angle of shearing resistance can be measured directly from the plot.

`Any point F (σ, τ) on the failure envelope represents the state of stress in the material during failure, under a given normal stress. In the direct shear test, the failure plane MN is

predetermined, and is horizontal. Fig. 4.6 (b) shows the stress conditions during failure. In order to find the direction of principal planes at failure, we first locate the position of thepole on the Mohr circle [Fig 4.6 (c)] on the
principle that the line joining any point on the circle to the pole P gives the direction of the plane on which the stresses are those given by the co-ordinates of that point.

` Hence, through point F a horizontal line (representing the direction of the failure plane) is drawn to intersect the circle at the point P which is the pole. Since points A and B represent respectively, the major and minor principal stresses, PA and PB give the directions of major and minor principal planes.

`Tests can be performed under all the three conditions of drainage. To conduct un-drained test, plan gridsare used. For the drained test, perforated grids are used. The same is first consolidated under the normal load, and then sheared sufficiently slowly so that complete dissipation of pore pressure takes place.

`The drained test is therefore also known as the slow test, and the shearing of cohesive soil may sometimes require 2 to 5 days. Cohesion less soils are sheared in relatively less time. For the consolidated un-drained test, perforated grids are used. The sample is permitted to consolidate under the normal load. After the completion of consolidation, the specimen is sheared quickly in about 5 to 10 minutes.

Commentsontheshearboxtest.

The direct shear test is a simple test. The relatively thin thickness of sample permits quick drainage and quick dissipation of pore pressure developed during the test. However, the test has the following disadvantages:

(1) The stress conditions across the soil sample are very complex. The distributionof normalstressesand shearingstresses overthepotentialsurfaceof slidingis not uniform. The stress is more at the edges and less in the centre. Due to this there is progressive failure of the specimen i.e., the entire strength of the soil is not mobilized simultaneously.

(2) As thetestprogresses,theareaundersheargraduallydecreases. The corrected area (Af) at failure should be used in determining the values of $\sigma$and, τ.

(3) Ascomparedtothe tri-axialtest,there islittlecontrolonthedrainageofsoil.

(4) Theplaneofshearfailureispredetermined,which maynotbetheweakest one.

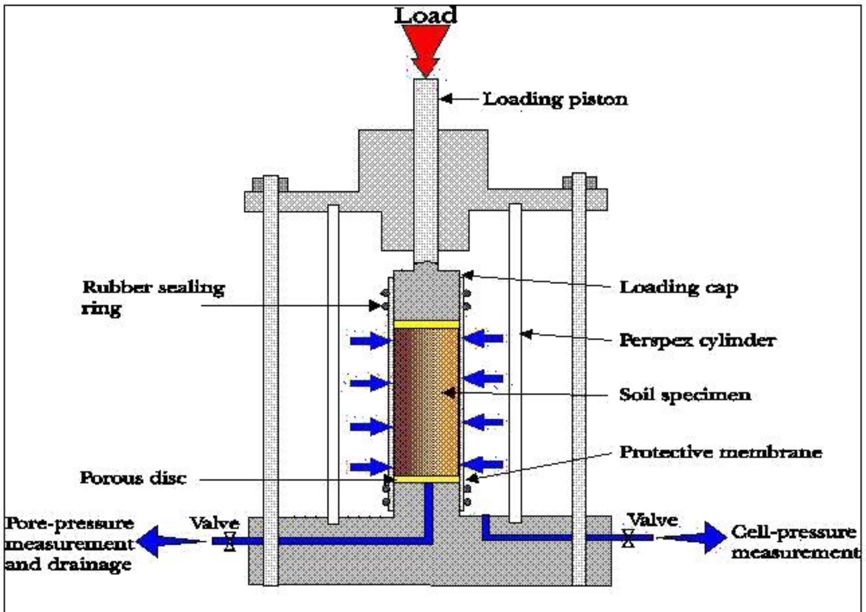
(5) Thereiseffectoflateralrestraintbytheside wallsoftheshearbox.

5. Explainthetri-axialcompressiontest (NOV/DEC2012)

The strength test more commonly used in a research laboratory today is the triaxial compression test, first introduced in the U.S.A by A. Casagrande and Karl Terzaghi. The solid specimen, cylindrical in shape, is subjected to direct stresses acting in three mutually perpendicular directions. In the common solid cylindrical specimen test, the major principal stress $\sigma 1$ is applied in the vertical direction, and the other two principal stresses $\sigma 2$ and $\sigma 3$ ($\sigma 2 = \sigma 3$) are applied in the horizontal direction by the fluid pressure round the specimen.

The test equipment specially consists of a high pressure cylindrical cell, made of Perspex or other transparent material, fitted between the base and the top cap. Three outlet connections are generally provided through the base: cell fluid inlet, pore water out let from the bottom of the specimen and the drainage outlet from the top of the specimen.

A separate compressor is used to apply fluid pressure in the cell. Pore pressure developed in the specimen during the test can be measured with the help of a separate pore pressure measuring equipment, such as Bishop's apparatus shown in Fig. 4.8. Thecylindricalspecimenisenclosedinarubbermembrane.Astainlesssteelpistonrunning through the centre of the top cap applies the vertical compressive load (called the deviator stress) on the specimen under test.



Triaxial apparatus

The load is applied through a proving ring, with the help of a mechanically operated load frame. Depending upon the drainage conditions of the test, solid nonporous discs or end caps, or porous discs are placed on the top and bottom of the specimen and the rubber membrane is sealed on to these end caps by rubber rings.

Thelengthofthespecimeniskeptabout2to2½timesitsdiameter.Thecellpressure $\sigma 3$ ($=\sigma 2$) acts all round the specimen; it acts also on the top of the specimen as well as the vertical piston meant for applying the deviator stress. The vertical stress applied by the

loading frame,throughthe provingringis equal to($\sigma 1-\sigma 3$),so that the total stress on the top of the specimen = $(\sigma 1 - \sigma 3) + \sigma 3 = \sigma 1$ = major principal stress.

This principal stress difference ($\sigma 1 - \sigma 3$) is called the deviator stress recorded on the proving ring dial. Another dial measures the vertical deformation of the sample during testing.It is desirable to maintain the cell pressure reservoir and mercury control apparatus, devisedbySkempton and Bishop (1950), as shown in Fig. 4.9 (a). Forlong duration test (lasting about a week or more), self-compensating mercury control can be used [Fig. 4.9 (c)].

A particular confining pressure $\sigma 3$ is applied during one observation, giving the value of the other stress $\sigma 1$ at failure. A Mohr circle corresponding to this set of ($\sigma 1$, $\sigma 3$) can thus be plotted. Various sets of observations are taken for different confining pressures $\sigma 3$ and the corresponding values of $\sigma 1$ are obtained. Thus, a number of Mohr circles, corresponding to failure conditions, are obtained. A curve, tangential to these stress circles, gives the failure envelope for the soil under the given drainage conditions of the test.

Sheartestscanbe performed inthe tri-axialapparatusunderallthe three drainage conditions. For un-drained test, solid (nonporous) end caps are placed on the top and bottom of the specimen. In the consolidated-un-drained test, porous discs are used. The specimen is allowed to consolidate under the desired confining pressure by keeping the pore water outlet open.

When the consolidation iscomplete, the pore water outlet is closed, and the specimen is sheared under un-drained conditions. The pore water pressure can be measured during the un-drained part of the test. In the drained test, porous discs are used, and the pore wateroutlet is kept open throughout the test. The compression test is carried out sufficiently slowlyto allow for the full drainage during the test.

Measurementofporepressureduringthetest.

It mainlyconsistsof (i)the nullindicator, (ii)the controlcylinder, (iii)pressure gauge, (iv) mercury manometer, and (v) burette.

The null indicator consisting of a single straight section of glass capillary tube dipping into an enclosed trough of mercury, is connected to the tri-axial cell through valve a by a copper tube, and to the contrl cylinder etc. through valve k. An increasein pore pressure in the sample during the test will tend to depress the mercury in the limb of the null indicator. This can be immediately balanced by adjusting the piston in the control cylinder to increase the pressure in the limb by an equal amount which is registered in the pressure gauge.Valves m, f and j are kept closed during the pore pressure measurements. In addition to the pressure gauge, a mercury manometer is also provided. This is used (i) for negative pore pressure, (ii) for accurate measurement of low positive pore pressure, and (iii) for checking the zero error of the pressure gauge.

When this manometer is connected through valves k and m, valves l and n are kept closed. The graduated tube or burette connected to the valve f is used for determining the gauge and manometer readings corresponding to zero pore pressure. In the case of fully saturated samples, this graduated tube can also be used to measure volume change during the consolidation stage of test in which drainage is permitted through the base of specimen (Bishop and Henkel 1957).

6. ExplaintheStressconditionsinsoilspecimenduringtri-axialtesting.(NOV/DEC2009)

Fig. 4.10 (a) shows the effective stresses acting on the soil specimen during tri-axial testing. The minor principal stress and the intermediate principal stress are equal. The effective minor principal stress is equal to the cell pressure minus the pore pressure. The major principal stress is equal to the deviator stress plus the cell pressure.

The effective major principal stress $\sigma 1'$ is equal to the major principal stress minus the pore pressure. The stress components on the failure plane MN are σ' and τf, and the failure plane is inclined at an angle α' to the major principal plane. Fig. 4.10 (b) shows the failure envelope JF and a Mohr circle corresponding to any failure point F. Since $\angle$JFC = 90° and the failure envelope cuts the abscissa at an angle ϕ', the angle α' of the failure plane is given by :

$$\alpha' = \frac{1}{2} \angle FCA = \frac{1}{2} (90° + \phi') = 45° + \phi'/2$$

Theprincipalstressrelationshipatfailurecanbefoundwiththehelpof Fig.4.10(b) FC = radius of Mohr circle = ½ ($\sigma 1' - \sigma 3'$) ; OC = ½ ($\sigma 1' + \sigma 3'$) ; OK = c' cot ϕ'

Hence $\sin\phi' = \frac{FC}{KC} = \frac{FC}{KO+OC} = \frac{\frac{1}{2}(\sigma_1'-\sigma_3')}{c'\cot\phi' + \frac{1}{2}(\sigma_1'+\sigma_3')} = \frac{(\sigma_1'-\sigma_3')}{2c'\cot\phi' + (\sigma_1'+\sigma_3')}$

$\therefore$ $\sigma_1'-\sigma_3' = 2c'\cot\phi' + (\sigma_1'+\sigma_3')\sin\phi'$ ...(4.15a)

or $\sigma_1'(1-\sin\phi') = \sigma_3'(1+\sin\phi') + 2c'\cot\phi'$

$\therefore$ $\sigma_1' = \sigma_3'\frac{(1+\sin\phi')}{(1-\sin\phi')} + 2c'\frac{\cos\phi'}{(1-\sin\phi')}$ ...(4.15)

or $\sigma_1' = \sigma_3'\tan^2\left(45° + \frac{\phi'}{2}\right) + 2c'\tan\left(45° + \frac{\phi'}{2}\right)$ ...(4.16)

or $\sigma_1' = \sigma_3'\tan^2\alpha' + 2c'\tan\alpha'$ ...(4.17)

or $\sigma_1' = \sigma_3' N\phi' + 2c'\sqrt{N\phi'}$ ...(4.17 a)

where $N\phi' = \tan^2\alpha' = \tan^2(45° + \phi'/2)$

Eq. 4.16 or 4.17 gives principal stress relationship. When the soil is in the state of stress defined by the Eq. 4.16 or 4.17, it is said to be in plastic equilibrium. In terms of totalstresses, 4.17 is written as

$\sigma_1' = \sigma_3\tan^2\alpha + 2c_u\tan\alpha$..(4..18)or$\sigma_1 = \sigma_3 N_\phi + 2c_u\sqrt{N_\phi}$

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

In Eq.4.16, σ_1'and σ_3'are known, andthetwounknown are ϕ'and c'. Hence two setsof observations are required to determine these two unknown parameters. In practice, a number of sets (σ_1', σ_3') at failure are observed, and Mohr circles are plotted for each set. A curve drawn tangential to these circles gives the failure envelope [Fig. 4 11 (a)].

Anothermethodof plottingthe rest results isintheformof the modifiedfailureenvelope which is a function for ½ (σ_1'+σ_3') and ½ (σ_1' - σ_3'). Rewriting Eq.4.15 (a) in the form

½ (σ_1'-σ_3') and d'+ ½ (σ_1' + σ_3') tan ψ'and

comparing itwithEq.4.5a,weobservethat

Sinϕ'=tanψ'..........(18.20a) andc'= $\frac{d'}{\cos\phi'}$

Eq.4.19representing theprincipalstressrelationship,istheequationofastraightline having itsy-coordinate represented by½(σ_1'-σ_3')andx-coordinate representedby½

(σ_1'+σ_3'). Fig. 4.11(b) shows the modified failure envelop, represented by Eq. 419, in which the slope ψ' and the intercept d' are related to ϕ' and c' through Eq. 4.20.

The line so obtained is often called the K_f line (Lambe,1969). The advantage of this method of plotting the failure envelope is that the averaging of scattered test results is facilitated to a great extent, giving the mean value of the parameters.

The calculationof thedeviatorstressmust bedone on thebasisof thechanged areaof cross-section at failure, or during any stage of the relation.

$$A2=\frac{V_1 \pm \Delta V}{L_1 - \Delta L}$$

Where V_1-initialvolume ofthespecimen; L_1=initiallength ofthespecimen
ΔV= changeinthevolumeof thespecimen
ΔL =Changeinthelengthofthespecimen

Thedeviator stress$\sigma_d$isgivenby$\sigma_d=\frac{Additionalaxiallload}{A2}$;

σ_3=fluidpressure

∴ $\sigma_1=\sigma_3+\sigma_d$

∴

Knowingσ1, E3andporepressure. σ1'andσ3'canbedetermined.

Advantagesoftri-axialtest:

(1) The sheartestunderallthethreedrainage conditionscanbeperformed with complete control.

(2) Precise measurementsofthepore pressure andvolumechangeduringthe test are possible.

(3)

(4) The stressdistributiononthefailureplaneasuniform.

(5)

(6) The stateof stress withinthe specimenduringanystageof the test,aswellas at failure is completely determinate

.

7. Explaintheun-confinedcompressiontest (NOV/DEC2013)

Theunconfinedcompressiontestisaspecialeaseoftri-axialcompressiontestinwhich σ_2 = σ_3 =0. The cell pressure in the tri-axial cell is also called the confining pressure. Due to the absence of such a confining pressure, the uni-axial test is called the unconfined compression test. The cylindrical specimen of soilis subjected to major principalstress σ1 till the specimen fails due to shearing along a critical plane of failure.

In its simplestform, the apparatus consists of a small load frame fitted with a proving ring to measure the vertical stress applied to the soil specimen. Fig4.12. (a) shows an unconfined compressiontester(GoyalandSingh,1958). Thedeformation of thesample is measured withthe helpof a separatedialgauge.Theendsof the cylindricalspecimenare hollowed in the form of cone. The cone seating reduce the tendency of the specimen to becomebarrelshapedbyreducingend-restraints.Duringthe test, load versusdeformation readings are taken and a rap is plotted.

Whenabrittlefailureoccurs,theproving ring dialindicatesadefinite maximum loadwhichdrops rapidlywiththefurther increaseof strain.In theplasticfailure,no definite maximumload isindicated. In sucha case, the loadcorrespondingto20% strain is arbitrarily taken as the failure load.

Fig. 4.12. (b), (c) shows the stress conditions, at failure, in the unconfined compression test which is essentially an un-drained test (if it is assumed that no moisture is lost from the specimen during the test). Since $\sigma_3=0$, the Mohr circle passes trough the origin which isalso the pole.

FromEq.4.18,weget$\sigma_1 2c_u \tan\sigma = 2c_u \tan \left[45^\circ \quad \frac{\phi_u}{2} \right]$

In the above equation, there are two unknowns $c_u$and ϕ_u, which cannot be determined by the unconfined test since a number of tests on the identical specimens give the same value of σ_1. Therefore, the unconfined compression test is generally applicable to saturated clays for which the apparent angle of shearing resistance ϕ_u is zero. Hence

$$\sigma_1 = 2c_u$$

Whenthe Mohrcircle isdrawn, itsradius isequalto $\sigma_1 /2 = c_u$. Thefailure envelope is horizontal. Pf is the failure plane, and the stresses on the failure plane are

$\sigma = \sigma_1 = q_u$ ....(4.24) and $t_f = \sigma_1 = q_u = c_u$

$$\frac{}{2} \quad \frac{}{2} \qquad\qquad \frac{}{2} \quad \frac{}{2}$$

Where, q_u = unconfined compressive strength at failure. The compressive stress is calculated on the basis of changed cross-sectional area A2 at failure, which is given by

$$A_2 = \frac{V}{L_1 - \Delta L} = \frac{A_1}{1 - \frac{\Delta L}{L_1}}$$

Where V - initial volume of the specimen;

L_1 = initial length of the specimen

ΔL = Change in length at failure.

8. a. Table, gives observations for normal load and maximum shear force for the specimens of sandy clay tested in the shear box, 36 cm^2 in area under un-drained conditions. Plot the failure envelope for the soil and determine the value of apparent angles of shearing resistance and the apparent cohesion. (NOV DEC/2013)

Normalload(N)	Maximumshearforce(N)
100	110
200	152
300	193
400	235

Solution

Fig. 18.13 shows the plot between the shear force F and the normal load N. From the plot, we Get ϕ_u = 22°, and total cohesive force = 70 N.

$\therefore$ **Unit apparent cohesion** $c_u = \frac{70}{36} = 1.95$ **N/cm^2**

= **19.5** kN/cm^2 = **19.5** kPa.

8.b. Samples of compacted, clean dry sand were tested in a shear box, 6 cm x 6 cm and the following results were obtained:

Normalload(N)	:	**100**	**200**	**300**	**400**
Peakshearload(N)	:	**90**	**181**	**270**	**362**
Ultimateshearload(N)	:	**55**	**152**	**277**	**300**

Determine the angle of shearing resistance of the sand in (a) the dense, and (b) the loose state.

Solution.

The value of the shearing resistance of sand, obtained from the peak stress represents the value of ϕin its initial compacted state, while that obtained from the ultimate shear corresponds to the sand when loosened by the shearing action.

Fig.4.14showsthetwoplots.Thevaluesofanglesofshearingresistancearefoundto be:

(a) dense state:ϕ=42°
(b) loosestate:ϕ=37°

9. Acylindrical specimenofsaturatedclay, 4 cmindiameterand9cminoverall length is testedinanunconfinedcompressiontester.Thespecimenhasconedendsandits lengthbetweentheapicesofconesis8cm.Findtheunconfinedcompressivestrengthof clay,ifthespecimenfailsunderanaxialloadof46.5N.Thechangeinthelengthof specimen at failure is 1 cm. (APRIL/MAY 2010)

Solution.

Originallength of specimen =9 cmoverall, and 8cmtoapicesof cones.Lengthof cylinder of the same volume and diameter (average length) L1 = 8.66 cm.

Initialcross-sectionalarea $A_1 = \dfrac{\pi \times 4^2}{4} = 12.57\,cm^2$

Changein lengthatfailure, ΔL=1cm

Areaoffailure= $A_2 = \dfrac{A_1}{1 - \dfrac{\Delta L}{L_1}} = \dfrac{12.57}{1 - \dfrac{1}{8.66}} = 14.2\,cm^2$

$\therefore$ Unconfinedcompressivestrength $q_u = \dfrac{failureload}{A_2} = \dfrac{465}{14.2} = 32.8\,kN/cm^2$

=328kN/m^2= 328kPa

$\therefore$ Shearstrength $c_u = \dfrac{q_u}{2} = \dfrac{2}{2} = 165\,kN/m^2 = 165\,kPa$

10. A cylinder of soil fails under an axial vertical stress of 160 kN/m^2, when it is laterally unconfined.Thefailureplanemakesanangleof50°withthehorizontal.Calculatethe value of cohesion and the angle of internal friction of the soil. (APRIL/MAY 2013)

Solution.

$$\alpha = 50 = 45\,; \therefore \phi u = 2(50 - 45) = 10^\circ$$

$$\tan \sigma = \tan\left[45 + \frac{\phi u}{2}\right] = \tan 50 = 1.192$$

As the sample is un-confined, σ_3 = 0

Now $\sigma_1 = \sigma_3 \tan^2\sigma + 2c\tan\sigma$

160=2ctan50°=2cx1.192

$$c = \frac{160}{2 \times 1.192} = 67.1\,kN/m^2 = 67.1\,kPa.$$

11. Two identical specimens, 4 cm in diameter and 8 cm high, of partly saturated compacted soilistested in a triaxialcellunder un-drained conditions. Thefirst specimen failed at an additional axial load (i.e. deviator load) of 720 N under a cell pressure of 100 kN/m^2. The second specimen failed at an additional axial load of 915 N under a cell pressure of 200 kN/m^2. The increase in volume of the first specimen at failure is 1.2 ml and it shortens by 0.6 cm, at failure. The increase in volume of the second specimen at failure is 1.6 ml, and it shortens by 0.8 cm at failure. Determine the value if apparent cohesion and the angle of shearing resistance (a) analytically, (b) graphically by Mohr's circle. (APRIL/MAY 2009)

Solution :

(a) Forthefirstspecimen:

Initialarea $A1 = \frac{\pi \times 42}{4} = 12.57 cm^2$

InitialVolume V1=12.57x8= 100.56cm^3

$\therefore$ Areaoffailure= $A_2 = \frac{V1 + \Delta V}{L - \Delta L_1} = \frac{100.56 + 1.2}{8 - 0.6} = 13.75 cm^2$

(b) Forthe secondSpecimen.

$\therefore$ Deviatorstressatfailure $\sigma_d = \frac{720}{123.75} = 52.4 N/cm^2 = 524 kN/m^2$

σ_3=100kN/m$_2$;σ_1=σ_3+σ_d=100 +524=624kN

/m^{2}Substitutingthe valueofσ1andσ3inEq4.18 a,we get

$$624 = 100N\phi + 2cu\sqrt{N\phi}$$

(b) Forthesecondspecimen

A_1=12.57cm^2;V_1=100.56cm^2;ΔL=0.8;ΔV=+1.6cm^3

$$A_2 = \frac{100.56+1.6}{8-0.8} = cm^2 \quad \sigma d = \frac{9151}{4.2} = 64.4 N/cm_2 \quad = 644 kN/m_2$$

σ_3=200kN/m^2;$\therefore$ σ_1=σ_3+σ_d=200+644=844kN /m^2

Substitutingthe valueof σ1 andσ3inEq.4.18a, we get

$$844 = 200n\phi + 2CU\sqrt{n\phi}$$

C_u=13 .6kN/ m^3(136kPa)

And $N_\phi = 2.2$

Now
$$N = \tan^2\left[42 + \frac{\phi_u}{2}\right] = 2.2$$

$\therefore \quad \phi_u = 22°$

12. A saturated specimen of cohesion-less sand was tested in triaxial compression and the sample failedata deviatorstressof482kN/m^2 whenthe cellpressure was100kN/m^2, under the drained conditions. Find the effective angle of shearing resistance of sand. What would be the deviator stress and the major principal stress at failure for another identical specimen of sand, if it is tested under cell pressure of 200 kN/m^2?

(NOV/DEC2010)

Solution.

In thedrained tests,theeffectivestressesareequaltothe totalstress.

$$\sigma_3' = \sigma_3 = 100 kN/m^2 \ (kPa)$$

$$\sigma_1' = \sigma_3 = \sigma_d = 100 + 482 = 582 m^2 (kPa)$$

Fig. showsthe Mohrcircle (circle1). Thefailureenvelopewillpassthroughtheorigin, since c' =0 for sand, and will be tangential to the circle. The angle of inclination of the failure envelope giveϕ' = 45°.

Alternatively,fromEq4.16

$$\sigma_1' = \sigma_3' \tan^2\left[45 + \frac{\phi'}{2}\right]$$

$$\therefore \quad 582 = 100 \times \tan^2\left[45 + \frac{\phi'}{2}\right]$$

$$\therefore \quad 45° + \frac{\phi'}{2} = \tan^{-1}(\sqrt{5.82}) = 67.5°$$

$\therefore \quad \phi' = 45°$

For the second specimen with σ_3'=200 kN/m^2, the centre of the Mohr circle passes through σ_3'=200 kN/m^2, and is tangential to the failure envelope. Circle II corresponds to this, from which σ_d'= 1160 kN/m^2

$$\sigma_2 = \sigma_1' - \sigma_3' = 960 kN/m^2 (960kPa)$$

Alternatively, σ_1' canbecalculatedfromtherelation :

$$\sigma_1' = \sigma_3' \tan^2\left[42 + \frac{\phi'}{2}\right] = 200 \tan^2\left[42 + \frac{45°}{2}\right] = 1164 kN/m^2 (1164 kPa)$$

σ_d=1164-200=964kN/m^2(964kPa).

13. Followingaretheresultsofun-drainedtri-axialcompressiontestontwo identicalsoil specimens, at failure: (APRIL/MAY 2011)

Lateralpressureσ_3(kN/m^2)	**100**	**300**
Totalverticalpressureσ_1(kN/m^2)	**440**	**760**
Porewaterpressureu(kN/m^2)	**-20**	**60**

(a) referredtototalstress,

(b) referredtoeffective stress.

Solution:

thecircles AandB with dark lines correspond to thetotalstress,andfromfailure envelope, drawn tangential to the two circles, we get

ϕ_u = 14° and c_u=110kN/m$_2$(kPa)

For the effective stress analysis, we have

σ_1' :440+20=460 and

760-60=700kN/m$_2$

σ_3' :100+20=120 and

300-60=240kN/m$_2$

The MohrcirclesA'and B'correspondingtothe effective stressesare shownbydotted lines. From the dotted failure envelope, we get

ϕ'=20°,c'=76kN/m^2(kPa)

14. Un-drainedtriaxialtestsare carriedoutonfouridenticalspecimensofsiltclay,and the following results are obtained: (APRIL/MAY 2010)

Cellpressure(kN/m^2)	**50**	**100**	**150**	**200**
Deviatorstressatfailure(kN/m^2)	**350**	**440**	**530**	**610**
Porepressure(kN/m^2)	**5**	**10**	**12**	**18**

Determine the value oftheeffective anglesopf shearingresistanceandthe cohesioninterceptbyplotting(a)conventionalfailureenvelopefromMohrcircles, (b)modifiedfailureenvelope.

Solution:Tableshowsthenecessarycalculationsofplottingthefailureenvelope

TABLE

Specimen No.	σ_3	σ_3'	σ_d	σ_1'	½($\sigma_1'+\sigma_3'$)	½($\sigma_1'-\sigma_3'$)
1	50	45	350	395	220	175
2	100	90	440	530	310	220
3	150	138	530	668	403	265
4	200	182	610	792	487	305

Fig.4.20showstheconventionalfailureenvelopefromMohr'scircles,fromwhichwegetϕ' =29.5°andc' =8kN/m^2(kPa).

Fig.4.21showsthemodifiedfailureenvelope,fromwhich we getψ'=26.5°andd'=70°

∴ $\sin\phi' = \tan\psi' = \tan 26.5°$ orϕ'=30°

$$c' = \frac{d'}{\cos\phi'} = \frac{70}{0.86} = 81 kN/m^2 \quad (kPa)$$

15. ExplainthevanesheartestAPRIL/MAY2009)

Vane shear test is a quick test, used either in the laboratory or in the field, to determine the un-drained shear strength of cohesive soil. The vane shear tester consists of four thin steel plates, called vanes, welded orthogonally to a steel rod. A torque measuring arrangement, such as a calibrated torsion spring, is attached to the rod which is rotated bya worm gear and worm wheelarrangement.Afterpushingthe vanesgently intothe soil, the torque rod is rotated at a uniform speed (usually at 1° per minute).

The rotation of the vane shears the soil along a cylindrical surface. The rotation of the spring in degrees is indicated by a pointer moving on a graduated dial attached to theworm wheel shaft. The torque T is then calculated by multiplying the dial reading with the spring constant. A typical laboratory vane is 20 mm high and 12 mm in diameter with blade thickness from 0.5 to 1 mm, the blades being made of high tensile steel. The field shear vane is from 10 to 20 cm in height and from 5 to 10 cm in diameter, with blade thickness of about 2.5 mm.

Let us assume that the top end of the vane is embedded in the soil so that both top and bottom endspartake in the shearing of the soil. Assuming that the shear resistance of the soil is developed uniformly on the cylindrical surface, the maximum total shear resistance, at failure, developed along the cylindrical surface

$$=\pi dH\tau_f$$

To find the maximum shear resistance developed at top and bottom ends, consider a radius r of the sheared surface. The shear strength of a right of thickness *dr* will be $2\pi r\, dr\, \tau_f$. Hence the total resistance of both top and bottom faces will be

$$= 2\int_0^{\pi}(2\pi r dr)\tau_f$$

To total shear strength developed will be equal to the sum of (i) and (ii). The maximum moment of the total shear resistance about the axis of torque rod equals the torque T at failure. Hence

$$T=(\tau dH\tau_f)\frac{d}{2}+2\int_0^{d/2}(2\pi r dr\tau_f)r=\pi\tau_f\left[\frac{d^2H}{2}+\frac{d^3}{6}\right]=\pi d^2\tau\left[\frac{H}{2}+\frac{d}{6}\right]$$

Ifonlythebottomendpartakesinthe shearingtheaboveequation takesthefrom:

$$T=\tau d^2\tau_f\left[\frac{H}{2}+\frac{d}{12}\right]$$

KnowingT,Handd,theshearstrength

DEPARTMENT:CIVILENGINEERING
SEMESTER: IV
SUBJECTCODE/Name:CE6405/SOILMECHANICS

UNIT -5

2MARKQUESTIONSANDANSWERS

1. DifferentiateFiniteslopeandInfiniteslope **(Nov/Dec2012),(Apr/May 2010)**

FiniteSlope

1. Limitedinextent
2. Thepropertiesofthe soilwillnotbe sameatidenticalpointsordepths

Infiniteslope

1. Largeinextent
2. Thepropertiesofthesoilwillbesameatidenticalpoint

2. SlopeProtectionMeasures **(Nov/ Dec 2012)**

Slopesthataresusceptibletoslidingshouldbeprotectedsothattheareawillbesafe. Slopeswhichhavefailed recentlyarelikelytofailunderlong-termcondition. Slopeshavebeenprotectedbyadopting some successfultechniques.In general,the corrective or protective measures involve

(i) reducingthemassorloadingwhichcontributestosliding
(ii) improvingtheshearingstrengthalongtheanticipatedzoneoffailure,and
(iii) Providingcertainmaterialswhichwillprovideresistancetomovement.

3. Whatismeantby slip? **(Apr/May2009)**

Slip orfailure zoneis athin zone of soilthat reachesthecriticalstate orresidualstate, resulting in movement of the upper

4. Definewatercontent. **(Nov/ Dec2012)**

Bydefinitionthe watercontent wisthe ratioof theweight (ormass)of the waterand the solids,

w=Ww/Wp.

5.Whatarethefactorsthataffecthydraulicconductivity? **(Apr/May2012)**

Thehydraulicconductivityisinfluencedbyanumberoffactorsincluding:

- Effectiveporosity
- Grainsizeandgrainsizedistribution
- Shapeandorientation ofparticles
- Degree ofsaturation
- Clay mineralogy

6. Whatisimmediatesettlement? (Nov/ Dec 2012)

The settlement which iscaused bytheelasticdeformationof drysoilandof moistand saturated soils without any change in moisture content.

7. Whatisprimaryconsolidationsettlement? (Nov/ Dec2013)

The settlement which results of volume change in the saturated cohesive soils because of expulsion of the water that occupies the voids space. Give the formulae to determine the verticalstress,radial stress Tangential stress,& shear stress uner uniformly distributed load load

8. Whatarethereasonsforcompressionofthesoil? (Nov/ Dec 2009)

· Compressionofsolidparticles&waterinthevoids.
· Compression&expulsionofairinthe voids.
· Expulsionofwaterinthe voids.

9. Whatarethe stagesofconsolidation? (Nov/ Dec 2011)

Thestagesofconsolidationare

· **Initialconsolidation**
· **Primaryconsolidation**
· **Secondaryconsolidation**

10. What is a principal plane?

At everypoint inastressedbody,therearethreeplanesonwhichthe shear stress are zero. These planes are known as principal planes.

11. Whatarethelimitationsofcoulomb'stheory? (MAY/JUNE2013)

Thelimitationsofcolumbtheoryare

· Itneglectstheeffectoftheintermediateprincipalstress.
· Itapproximatesthecurvedfailureenvelope byastraightline which may not give correct results.

12. GivetheCoulomb'sshearstrengthequation. (Nov/ Dec 2010)

TheCoulomb'sshearstrengthequationisgivenby,

S=c+stanj

where

C= cohesion
j= Angleofinternalfriction

13. Whatarethefactorsaffectingpermeabilitytests? (MAY/JUNE2013)

Thefollowingfivephysicalcharacteristicsinfluence theperformanceandapplicabilityof permeability tests:

(1) positionofthe waterlevel,
(2) typeofmaterial- rockor soil,
(3) depthofthetestzone,
(4) permeabilityofthetestzone, and

(5) heterogeneityandanisotropyofthe testzone.

14. Defineeffectivestress. (Nov/ Dec2010)

Effectivestressequals thetotal stress minus thepore waterpressure,or thetotalforce in the soil grains divided by the gross cross-sectional area over which the force acts.

15. DefineCriticalDepth (MAY/JUNE2014)

Ifthere is no distinct change in the character ofsubsurface strata within the critical depth, elastic solutions for layered foundations need not be considered. Critical depth is the depth belowthe foundation within which soil compression contributes significantly to surface settlements. For fine-grained compressible soils, the critical depth extends to that point where applied stress decreases to 10 percent of effective overburden pressure. In coarse-grained

material criticaldepthextends to thatpoint whereapplied stress decreasesto20percent of effective overburden pressure.

16. DefinePorosity MAY/JUNE2009)

Soils usually consist of particles, water and air. In order to describe a soil various parameters are used to describe the distribution of these three components, and theirrelative contribution to the volume of a soil. These are also useful to determine other parameters, such as the weight of the soil. They are defined in this chapter. An important basic parameter is the porosity n, defined as the ratio of the volume of the pore space andthe total volume of the soil,

16MARKQUESTIONSANDANSWERS

(AUCNov/Dec2012)

1. Explain indetailabouttheCausesofSlopefailure

Erosion: Thewindand flowingwater causeserosionoftopsurfaceofslopeand makesthe slope steep and thereby increase the tangential component of driving force.

Steady Seepage: Seepage forces in the sloping direction add to gravity forces and make theslope susceptible to instability. The pore water pressure decreases the shear strength. This condition is critical for the downstream slope.

Sudden Drawdown: in this case there is reversal in the direction flow and results in instability of side slope. Due to sudden drawdown the shear stresses are more due to saturated unit weight while the shearing resistance decreases due to pore waterpressure that does not dissipatequickly.

Rainfall: Long periods of rainfall saturate, soften, and erode soils. Water enters into existing cracks and may weaken underlying soil layers, leading to failure, for example, mud slides.

Earthquakes: They induce dynamic shear forces. In addition there is sudden buildup of pore water pressure that reduces available shear strength.

ExternalLoading: Additionalloadsplacedontopoftheslope increases thegravitationalforces that may cause the slope to fail.

Constructionactivitiesatthetoeoftheslope: Excavationatthebottomofthesloping surface will make the slopes steep and there by increase the gravitational forces which may result inslope failure.

Typesof failure

Broadlyslopefailuresareclassifiedinto3typesas.

1. Face(Slope)failure
2. Toefailure
3. Basefailure

Face(Slope)Failure: Thistypeoffailureoccurswhentheslopeangleis largeandwhenthe soil at the toe portion is strong.

ToeFailure: Inthiscasethefailure surfacepassesthroughthetoe.Thisoccurswhentheslope is steep and homogeneous

BaseFailure: Inthiscasethefailuresurfacepassesbelowthetoe

2. ExplainindetailaboutFiniteslopemethod. **(AUCNov/Dec2012)(AUCNov/Dec2011)**
Afiniteslope isonewithabaseandtopsurface,theheight being limited.The incline facesof earth dams, embankments, excavation and the like are all finite slopes.
Investigationofthestabilityoffiniteslopes involvesthefollowingsteps
1. Assumingapossibleslipsurface,

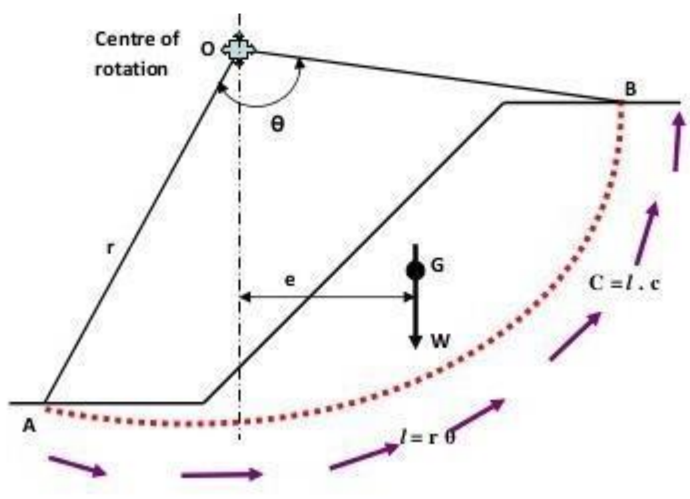
2. Studyingtheequilibriumoftheforcesacting onthissurface,and

3. Repeatingtheprocessuntiltheworstslipsurface,that is, theonewithminimum arginof safety is found.

Totalstressanalysisforpurecohesivesoil

The analysis is based on totalstresses, it is also called F= 0 analysis. It gives the abilityofan embankment immediatelyafter construction. It isassumed that the soilha time to drainand the shearstrengthparametersusedareobtained fromundrainenditionswithrespecttototalstresses. These may be obtained from either unconfined impression test or an undrained triaxial test without pore pressure measurements.
tABbeatrialslipsurface inthe formofacircular arcofradius'r'withrespect toation'O'as shown in Fig

Let`W'betheweightofthesoilwithintheslip surface



Finite slope in cohesive soils

Let`G'bethe positionofitscentre ofgravity.

TheexactpositionofGisnotrequiredanditisonlynecessarytoascertainthepositionof thelineofactionofW, this maybeobtained bydividingthefailureplane into asetofvertical slices and taking moments of area of these slices about any convenient vertical axis.

Theshearingstrengthofthesoilisc,sinceF= 0°.

Therestoringmoment(alongtheslipsurface)=clr

Incaseofcohesivesoilwhentheslope isonthevergeslippagetheredevelopsatensioncrackat the top of the slope as shown in Fig 11. the depth of tension crack is

There isno shearresistancealongthecrack.ThefailurearcreducesfromArcABtoArcAB'and the angle reduces to ´.

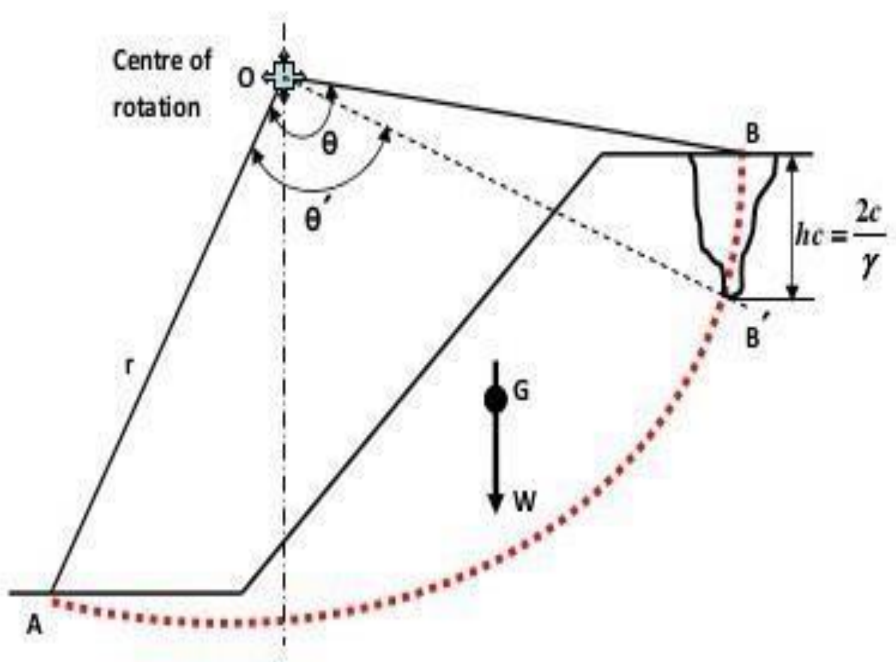
ForcomputationofFS wehave to

1. Useðinsteadofintherestoringmomentcomponent.
2. Considerthefullweight `W'ofthesoilwithintheslidingsurfaceABtocompensate for filling of water in the crack in the driving moment component

The driving moment =W.e

$$\text{Factor of safety, FS} = \frac{\text{Restoring moment}}{\text{Driving moment}} = \frac{c.r^2.\theta}{W.e}$$

Effect of Tension cracks on Stability

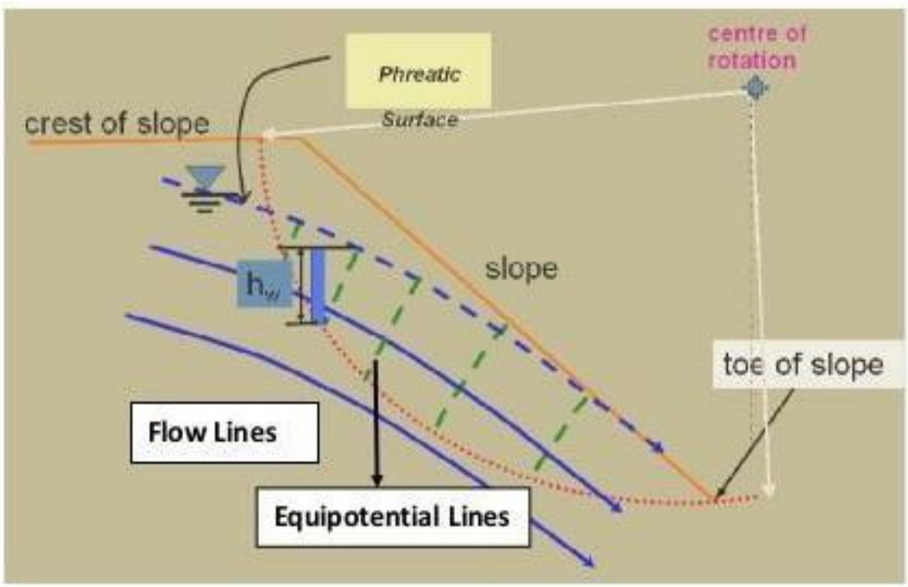


. Infinite slope with tension crack on top

3. ExplainindetailabouttheStabilityduringsteadyseepage(AUCNov/Dec2011)

When seepage occurs at a steady rate through an earth dam or embankment itrepresents critical condition for the stability of slope.
When seepage occurs pore water pressure (u) develops and this will reduce the effectivestresswhich in turn decreases the shear strength along the failure surface.
Thefollowing procedureisadoptedtoobtainstability
1. DrawtheC/Softheslope
2. Drawthepotentialfailuresurface
3. Dividethesoilmassintoslices
4. Calculate the weight W and the corresponding normal and tangential components for all theslices in the usual way
InadditionForthegivenslopeconstruct flownet (networkofequipotentialand flow lines)as shown

For the given slope construct flow net (network of equipotential and flow lines) as shown



Flow net in a finite slope under steady seepage

The average pore water pressure (u) at the bottom of the slice is given by the piezometric head (h_w) as

$$u = h_w \gamma_w$$

h_w = piezometric head above the base of the slice

The total force due to pore water pressure at the bottom of the slice

$$U = u\,\gamma_w$$

Tabulate all the values as shown below

Slice No.	Width	Area (m^2)	Weight 'W' (kN)	Normal component 'N' (kN)	Tangential componen t 'T' (kN)	Pore water pressure (u)	Total force due to pore pressure (U)
				ΣN = ___	ΣT = ___		ΣU = ___

The F.S is computed as

$$Factor\ \ of\ \ Safety, FS = \frac{(c' r\,\theta + \tan\phi' \sum(N-U)\,)}{\sum T}$$

c´ and ϕ´ - Shear parameters based on effective stress analysis obtained from drained shear tests.

If the flownet is not constructed then F.S may be computed as

$$Factor\ \ of\ \ Safety, FS = \frac{(c' r\,\theta + \tan\phi' \sum(N')\,)}{\sum T}$$

$$N' = W'\cos\delta = bZ\gamma'\cos\delta$$

N´= Weight of slice computed from effective unit weight

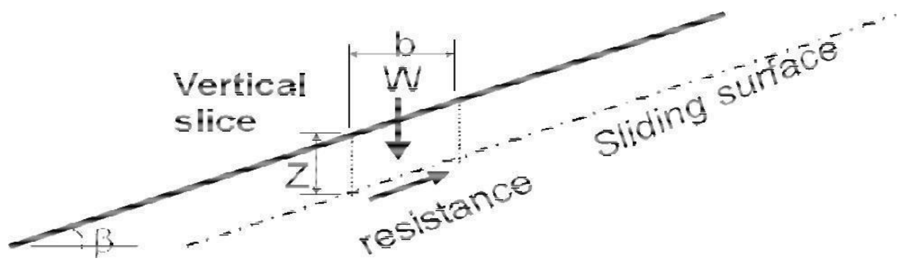
$T = W \sin\delta = b\,Z\,\gamma_{sat} \sin\delta$ - Weight of slice computed from saturated unit weight.

4. **Explainin detailaboutinfiniteslopesandfinite slopes**

(AUCNov/Dec2010)(MAY/JUN 2010)

1. InfiniteSlopes
2. FiniteSlopes

Infiniteslopes: Theyhavedimensionsthatextendovergreatdistancesandthesoilmassis inclined to the horizontal.
Finiteslopes: Afiniteslopeisonewithabaseandtopsurface,theheightbeinglimited.The inclined faces of earth dams, embankments and excavation and the like are all finite slopes.



Factorof safety
Factor of safety of aslopeisdefinedastheratioof averageshearstrength(tf)of asoiltothe average shear stress (td) developed along the potential failure surface.

ShearStrength:-

Shearstrengthofa soilisgivenby

$$t_f = c + s\tan f$$

Where,c=cohesion

=angleofinternal friction

=Normalstressonthepotentialfailuresurface Similarly,the mobilized shear strength is given by

$$t_d = c_d + s\tan f_d$$

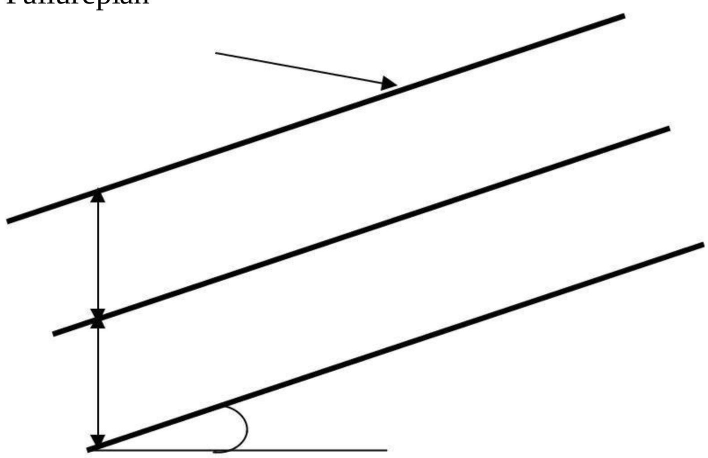
c_d and f_d

.
InfiniteSlopes:

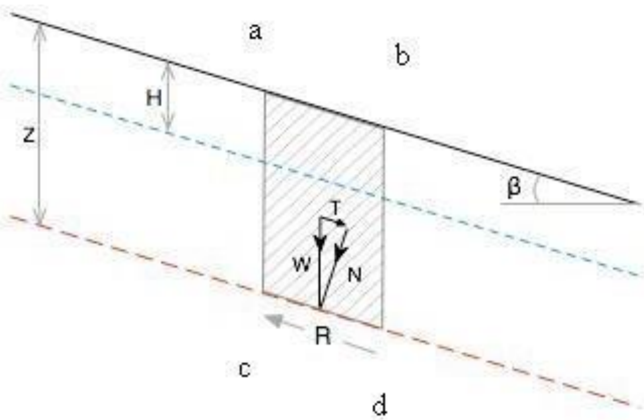
Infiniteslopeshavedimensionsthat extendedovergreat distancesandthesoilmassis inclined to the horizontal. If different strata are present strata boundaries.

areassumedto beparalleltothesurface.Failure isassumedtooccuralongaplane parallel to the surface.

Failureplan



3casesofstabilityanalysisofinfiniteslopesareconsidered Case (i) Cohesionless soil
Case(ii)Cohesive soil
Case(iii)Cohesive-frictionalsoil.



Consideraninfiniteslope inacohesionlesssoilinclinedat anangleto the horizontal as shown. Consider an element ‘abcd’ of the soil mass.

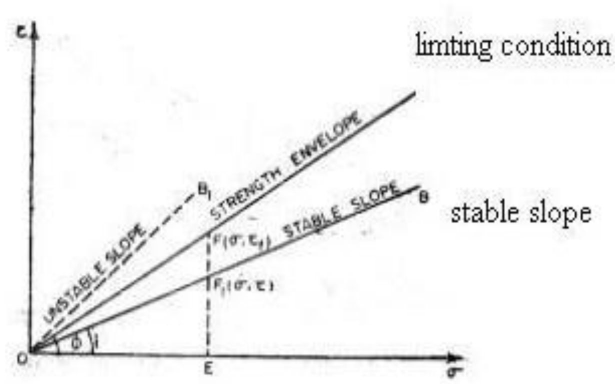
LettheweightoftheelementbeW.

The componentofWparalleltoslope =T= WSin

Thecomponent ofWperpendiculartoslope=N=W Cos
The force that causes slope to slide = T = WSin

- *OA* is Coulomb's failure envelope for cohesionless soil defined by the equation

$$\tau_f = \sigma \tan\phi$$



Failure condition for an infinite slope of cohesionless soil

5. . AlongNaturalslopecohesionless(sandysoil)isinclinedat12°tothehorizontal.Taking $\phi 30^0$ Determinethefactorof safetyoftheslope.If theslopeiscompletelysubmerged,what will be the change in the factor of safety ? Also find the seepage parelleltotheslope. $\gamma_{rat} = 19.5 KN/m^3$ **(APRIL/MAY2009)**

Solution

$$F = \frac{\tan\phi}{\tan i}$$

$$here\ \phi = 30^0, i = 12^0$$

case (i) Dry

$$F = \frac{\tan 30^0}{\tan 12^0} = 2.72$$

case(ii) Submerged

$$F = \frac{\tan 30^0}{\tan 12^0} = 2.72 = \frac{\gamma' \cos^2 \tan\phi}{\gamma' \sin i \times \cos i}$$

$$F = 2.72$$

case(iii)seepage isparellelyoslope

$$F = \frac{\gamma' \tan\phi}{\gamma_{sat} \tan i}, here\ \gamma' = \gamma_{sat} - \gamma_w$$

$$= \frac{(19.5 - 9.81)\tan 30^0}{19.5 \tan 12^0}$$

$$= 1.35$$

6. .Analyse the Slope having a slope angle 25° .It is made of cohesive (clay) having $C' = 30\ KN/m^2; \phi = 20^0$, e 0.65 ; $G = 2.7$ and underthe following condition, (Nov / Dec 2010)

ToFind

i) Whenthe soilisDry
ii) WhenwaterseepsparalleltotheSurfaceofthe Slope
iii) WhentheslopeisSubmerged

Given data

$$C' = 30 KN/m^2; \phi = 20^0; e 0.65; G = 2.7; i = 25^0$$

Solution

$$\gamma_d = \frac{G\gamma_w}{1+e} = \frac{2.7\times 9.81}{1+0.65}$$

$$\gamma_d = 16.05\ KN/m^3$$

$$\gamma_{sat} = \frac{(G+e)\gamma_w}{1+e} = \frac{(2.7+0.65)9.81}{1+0.65}$$

$$= 19.92 KN/m^3$$

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

$$= 19.92 - 9.81 10.11 KN/m^3$$

$$\gamma' = 10.11 KN/m^3$$

Case (i)ForDrySoil,StabilityNumber, S_n

$$S_n = \frac{C}{\gamma_d \times H_c} = (\tan i - \tan\phi)\cos^2 i$$

$$= (\tan 25^0 - \tan 20^0)\cos^2(25^0)$$

$$= (0.466 - 0.364)\times 0.821$$

$$= 0.084$$

$$Criticalheight, H_c = \frac{C}{\gamma_d S_n} = \frac{30}{(16.05\times 0.084)}$$

$$= 22.25 m$$

Case(ii)Forseepageparalleltosurfaceoftheslope

$$S_n = \frac{C}{\gamma_{sat} H_c} = \cos^2 i \left(\tan i - \frac{\gamma'}{\gamma_{sat}} \tan\phi \right)$$

$$= \cos^2 25^0 \left(\tan 25^0 - \frac{10.11}{19.91} \tan 20^0 \right)$$

$$= 0.821 \left(0.466 - \frac{10.11}{19.91} \times 0.364 \right)$$

$$S_n = 0.231$$

$$H_c = \frac{C}{\gamma_{sat} \times S_n} = \frac{30}{19.91 \times 0.231}$$

$$= 6.52m$$

Case(iii)ForSubmergedtype

$$S_n = \frac{C}{\gamma' \times H_c} = \cos^2 i (\tan i - \tan\phi)$$

$$= \cos^2 25^0 (\tan 25^0 - \tan 20^0)$$

$$= 0.084$$

$$\therefore \quad H_c = \frac{C}{\gamma' \times S_n} = \frac{30}{10.11 \times 0.084}$$

$$= 35.33m$$

7. Inordertofindthefactorofsafetyofd/sslopeofanearthduringsteadyseepage,the section of the dam was drawn to a scale of 1 cm=4 cm and the following result was obtained on a critical slip Circle. (Nov / Dec 2012) (AUC Nov / Dec 2014)

AreaofN-Rectangular(A_n)=14.40Sq-cm
Area of T-Rectangular (A_n) = 6.4 Sq-cm
Area of U-Rectangular (A_n)= 6.90 Sq-cm
Lengthofarc=12.6Sq-cm

Laboratorytesthavefinishedvalue of26^0 foreffective angle ofshearresistance and19.5KN/m^2forcohesion.Determinethefactorofsafetyoftheslopeunitweight of soil$\gamma=19KN/m^3$

Solution

Scale1cm=4cm

$$\therefore x=4; L=12.6\times4=50.4cm$$

$$=\frac{C \cdot L+\tan\phi\sum(N-U}{\sum T}$$

$$Consider\ 1cm\ length\ of\ dam, \phi=26^0; C'=19.5kn/m^2$$

$$\sum N=A_N\ \chi^2\ \gamma \quad =14.4\times4^2\times19=4378KN$$

$$\sum U=A_U\ \chi^2\ \gamma_w \quad =6.9\times4^2\times9.81=1083KN$$

$$\sum N=A_N\ x^2\gamma \quad =6.4\times4^2\times19=1946KN$$

$$F=\frac{(19.5\times12.6\times4)+\tan26^0\times(4378-1083)}{1946}$$

$$=1.33$$

8. . AlongNaturalslopecohesionless(sandysoil)isinclinedat12°tothehorizontal.Taking $\phi 30^0$Determinethefactorof safetyoftheslope.If theslopeiscompletelysubmerged,what will be the change in the factor of safety ? Also find the seepage (APRIL/MAY2010)

Solution

$$F=\frac{\tan\phi}{\tan i}$$

parallel to the slope. $\gamma_{rat}=19.5KN/m^3$

$$here\ \phi=30^0, i=12^0$$

case (i) Dry

$$F=\frac{\tan 30^0}{\tan 12^0}=2.72$$

case(ii) Submerged

$$F=\frac{\tan 30^0}{\tan 12^0}=2.72=\frac{\gamma' \cos^2 \tan\phi}{\gamma' \sin i\times\cos i}$$

$$G=2.72$$

case(iii) seepage is parellely o slope

$$F=\frac{\gamma' \tan\phi}{\gamma_{sat}\tan i}, here\ \gamma'=\gamma_{sat}-\gamma_w$$

$$=\frac{(19.5-9.81)\tan 30^0}{19.5\tan 12^0}$$

$$=1.35$$

9. Explainindetailaboutbishop’smethodofstabilityanalysis?

The Modified Bishop’s meth[is slightly different from the ordinary method of slices in that normal interaction forces between adjacent slices are assumed to be collinear and the resultant interslice shear force is zero. The approach was proposed byofThe constraint introduced bythe normal forces between slices makes the problem statically indeterminate. As a result, iterative methodshave to be usedto solve forthe factorofsafety. The method hasbeenshownto produce factor of safety values within a few percent of the "correct" value

Lorimer'smethod

Lorimer's Method is a technique for evaluating slope stability in cohesive soils. It differs from Bishop's Method in that it uses a slip surface in place of a circle. This mode of failure was determined experimentally to account for effects of particle cementation. The method was developed in the 1930s by Gerhardt Lorimer (Dec 20, 1894-Oct 19, 1961), a student of geotechnical pioneer

Spencer’sMethod

Spencer’s Method of analysis requires a computer program capable of cyclic algorithms, but makesslopestabilityanalysiseasier.It isnot asaccurateastheModifiedBishop’smethod,but is acceptably accurate in engineering practices.

Sarmamethod

Mainarticle:

The proposed byof is a technique used to assess the stability ofslopes under seismic conditions. It may also be used for static conditions if the value of the horizontal load is taken as zero. The method can analyse a wide range ofslope failures as it may accommodate a multi-wedge failure mechanism and therefore it is not restricted to planar or circular failure surfaces. It may provide informationaboutthe factorofsafetyor aboutthe criticalaccelerationrequired to cause collapse.

Comparisons

Theassumptions madebyanumberoflimitequilibriummethodsarelistedinthetablebelow.

Method	**Assumption**
Ordinary method of cells	Intersliceforcesareneglected
Bishop's simplified/modified	Resultantintersliceforcesarehorizontal.Therearenointersliceshear forces.
Janbu'ssimplified	Resultant interslice forces are horizontal. An empirical correction factor is used to account for interslice shear forces.
Janbu'sgeneralized	Anassumedlisusedtodefinethelocationoftheinterslicenormalforce.
Spencer	The resultant interslice forces have constant slope throughout the sliding mass.

Chugh	Same asSpencer's method but witha constant acceleration force oneach slice.
Morgenstern-Price	The direction of the resultant interslice forces is defined using an arbitrary function. The fractions of the function value needed for force and moment balance is computed.
Fredlund-Krahn(GLE)	SimilartoMorgenstern-Price.
Corpsof Engineers	The resultant interslice force is either parallel to the ground surface or equal to the average slope from the beginning to the end of the slip surface..
Loweand Karafiath	The direction of the resultantintersliceforceis equal to the average of the ground surface and the slope of the base of each slice.
Sarma	Theshearstrengthcriterionisappliedtotheshearsonthesidesand bottom of each slice.Theinclinations of the sliceinterfaces arevaried until a critical criterion is met.

10. Explainindetailaboutstabilityanalysisbymethodofslicesforsteadyseepage

The stability analysis with steady seepage involves the development of the pore pressure head diagramalongthechosentrialcircleoffailure. Thesimplest ofthemethods for knowingthepore pressure head at any point on the trial circle is by the use of flownets which is described below.

DeterminationofPorePressurewithSeepage

the section of a homogeneous dam with an arbitrarily chosen trial arc. There is steady seepage flow through the dam as represented by flow and equipotential lines. From the equipotential lines the pore pressure may be obtained at any point on the section. For example at point *a* in Fig. 10.19 the pressure head is *h*. Point *c* is determined by setting the radial distance *ac*

MethodofAnalysis(graphicalmethod)

the section of a dam with an arbitrarily chosen trial arc. The center ofrotation of the arc is 0. The pore pressure acting on the base of the arc as obtained from flow nets isWhen the soil forming the slope hasto be analyzed under a condition where full or partial drainage takes place the analysis must take into account both cohesive and frictional soil properties based on *effective stresses*. Since the effective stress acting across each elemental length of thecircular arc failure surface must be computed in this case, the method of slices is one of the convenient methods for this purpose.

Themethodofanalysis isasfollows.

Thesoil massabovetheassumedslip circleis dividedintoa number of verticalslices of equal width. The numberof slicesmay be limited to amaximum of eight to ten to facilitate computation.The forces used in the analysis acting on the slices are shown in Figs. 10.20(a) and (c). The forcesare:

1.Theweight*W*oftheslice.
2. Thenormalandtangentialcomponents oftheweight *W*actingonthebaseoftheslice. They are designated respectively as *N* and *T*.
3. Theporewaterpressure*U* acting onthebaseoftheslice.
4. Theeffectivefrictionalandcohesiveresistances actingonthebaseoftheslicewhich is Designated as *S*.

Theforcesactingonthesidesoftheslicesarestaticallyindeterminateastheydependon the Stress deformation properties of the material, and we can make only gross assumptions about theirrelative magnitudes.In the conventional slice method of analysis the lateral forces are assumed equal on both sidesof the slice. This assumption is not strictly correct. The error due to this assumption on the mass as a whole is about 15 percent (Bishop, 1955).