BAPATLA ENGINEERING COLLEGE (Autonomous):: BAPATLA

CIVIL ENGINEERING DEPARTMENT

SCHEME OF EVALUATION

II/IV B.Tech (Regular) Degree Examination (August-2022)

Subject **: SOIL MECHANICS**

Subject Code: **20CE405**

Prepared by	Verified by	Verified by
Dr.B.Satyanarayana Associate Professor	Sri P.Bapi Raju Asso. Professor	Sri M.Manoj Kumar Asst. Professor

Head of the Department

20CE405

Hall Ticket Number:

II/IV B.Tech (Regular) DEGREE EXAMINATION

August, 2022 — **Civil Engineering**

Fourth Semester — **Soil Mechanics**

Time: Three Hours — **Maximum:70 Marks**

Answer question 1 compulsory. **(14X1 = 14 Marks)**

Answer one question from each unit. **(4X14=56 Marks)**

1.	a)	What is meant by weathering?	CO1	
	b)	Define consistency index.	CO1	
	c)	How many groups of soils ate there in Indian Standard Classification System.	CO1	
	d)	State Darcy's Law.	CO2	
	e)	Define permeability of a soil mass.	CO2	
	f)	What is the role of effective stress in soil mechanics?	CO2	
	g)	Write any three clay minerals.	CO2	
	h)	What are the assumptions made in Boussinesq theory for stress distribution in soils?	CO3	
	i)	What is importance of Newmarks influence chart	CO3	
	j)	Draw Sample compaction curve.	CO3	
	k)	Differentiate between primary consolidation and secondary consolidation.	CO4	
	l)	Differentiate between consolidation and compaction.	CO4	
	m)	How soils attain their shear strength	CO4	
	n)	Name different laboratory shear tests on soils?	CO4	

Unit –I

2. a) Derive a relationship between water content, void ration, degree of saturation and specific gravity of soil solids. — CO1 — 7M

b) In a field exploration, a soil sample was collected in a sampling tube of internal diameter 5.0 cm below the ground water table. The length of the extracted sample was 10.2 cm and its mass was 387 g. If G = 2.7, and the mass of the dried sample is 313 g. Find the porosity, void ratio, degree of saturation, and the dry density of the sample. — CO1 — 7M

(OR)

3. a) A Sieve analysis test is conducted on a soil sample weighing 500 g. The results are given below. — CO1 — 7M

Sieve size (mm)	10	4.75	2.0	1.0	600μ	425μ	300μ	150μ	75μ	Pan
Soil retained (g)	29	42	40	45	89	110	68	32	35	1(

Plot the grain size analysis curve and compute Cu and Cc and classify the soil.

b) What is plasticity chart? Explain its use in soil classification. — CO1 — 7M

Unit –II

4. a) Explain Flow nets, their characteristics and uses. — CO2 — 7M

b) A vertical sheet pile penetrates 8 m into a uniform sand stratum, 15 m thick, overlying an impervious layer. It retains water for 6 m above G.L. Draw the flow net and determine the seepage under the pile. Take K = 4 x 10^{-2} m/sec. — CO2 — 7M

(OR)

5. a) What is quick sand? Derive an expression for critical hydraulic gradient. Why is quick sand condition more common in sandy soils? — CO2 — 7M

b) A falling head test was performed on a soil specimen having a diameter of 100 mm and height 120 mm. The stand pipe had a diameter of 12 mm and the water level in it dropped from 550 mm to 410 mm in 2 hours. Determine the time required for the water level in the stand pipe to come down to 200 mm. Also determine the height of water level in the — CO2 — 7M stand pipe after a period of 24 hours from the beginning of the test.

Unit –III

6.	a)	A point load of 1000kN is applied at the ground level. Calculate the vertical stress below the point of application of the load at a depth of 5meters and also calculate the stress at the same depth but at a radial distance of 6 meters. Adopt Boussinesq's approach	CO3	7M
	b)	Explain Newmark's influence chart preparation and usage.	CO3	7M
		(OR)		
7.	a)	Explain the difference between IS light and heavy compactions.	CO3	7M
	b)	Explain about various factors those affect compaction?	CO3	7M
		Unit –IV		
8.	a)	Obtain the differential equation defining the one-dimensional consolidation as given by Terzaghi.	CO4	7M
	b)	Representative sample of a layer of Silty Clay,5m thick,were tested in a consolidometer and the following results were obtained Initial Void Ratio e_0=0.90,Preconsolidation pressure σ_c =120kN/m^2,Recompression Index C_r=0.03,Compression Index C_c=0.27.Estimate the consolidation settlement if the present average over burden stress of the layer σ_0 is 70kN/m^2 and the increase in average stress in the layer is 80kN/m^2.	CO4	7M
		(OR)		
9.	a)	Explain different drainage conditions for shear testing of soils.	CO4	7M
	b)	An unconfined compression test was conducted on an undisturbed sample of clay .The sample had a diameter of 37.5mm and was 80mm long. The load at failure measured by the Proving ring was 28N and the axial deformation of the sample at failure was 13mm.Determine the Unconfined compressive strength and the undrained shear strength of the clay.	CO4	7M

SCHEME OF EVALUATION

II/IV B.Tech (Regular) Degree Examination (AUGUST- 2022)

20CE405 - SOIL MECHANICS

1. **Answer all questions** **14 x1 =14 Marks**

a) Weathering is the breaking down or dissolving of rocksand minerals on earth's surface

b) Consistency index: $\mathbf{I_c = (w_L - w_n) / I_p}$

c) **No. of soil groups : 18**

d) Darcy's Law can be expressed as **v = q/A = k.i ;** where **k =** permeability of the soil **i = Δh/L**

Δh = difference in total heads

L = length of the soil mass

e) Permeability of soil is the property of the soil which permits the passage of fluid through it and is denoted by ' k". the units are cm/sec

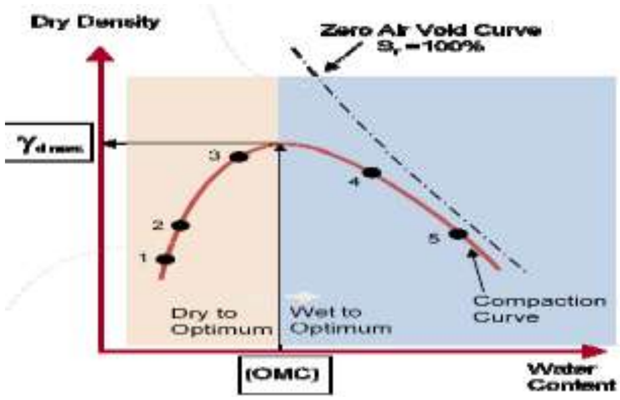
f) The effective stress controls the engineering properties of the soils. Compression and shear strength of the soil depends on effective stress.

g) Montmorillonite, chlorite, illite, kaolinite etc

h) Assumpsions in Boussinesq theory: the soil medium is elastic and continuum and has a constant value of modulus of elasticity. Soil is homogeneous, isotropic, semi-infinite.

i) New-Mark's influence chart's importance: To find vertical stress at any particular depth caused due to any shape of vertical uniformly distributed loading in the interior of an elastic, homogeneous, and isotropic medium, which is bounded by horizontal planes i.e., semi-infinite medium.

j) Sample compaction curve:



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

In case of saturated soil mass the applied stress is borne by pore water alone in the initial stages

$\therefore$ At $t = 0$ $\Delta\sigma = \Delta u$ $\Delta\sigma' = 0$

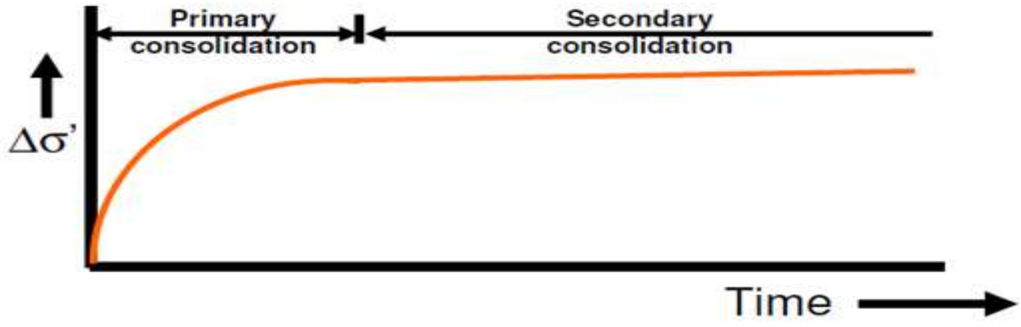
With passage of time water starts flowing out from the voids as a result the excess pore water pressure decreases and simultaneous increase in effective stress will takes place. The volume change is basically due to the change in effective stress After considerable amount of time (t =0) flow from the voids ceases the effective stress stabilizes and will be is equal to external applied total stress and this stage signifies the end of primary consolidation

At $t = t_1$ $\Delta\sigma = \Delta\sigma' + \Delta u$

At $t = \infty$ $\Delta\sigma = \Delta\sigma'$ $\Delta u = 0$ (End of primary consolidation)

Secondary Consolidation Settlement:-

This is also called Secondary compression (Creep). "It is the change in volume of a fine grained soil due to rearrangement of soil particles (fabric) at constant effective stress". The rate of secondary consolidation is very slow when compared with primary consolidation.



l) Consolidation: this is the process of expulsion of water from soil mass

Compaction: this is the process of removing air voids

m) Shear strength of soil is "The capacity of a soil to resist the internal and external forces which slide past each other" . The shear strength of a soil is its resistance to shearing stresses.

- It is a measure of the soil resistance to deformation by continuous displacement of its individual soil particles. - Shear strength in soils depends primarily on interactions between particles

n) i) Direct shear test, ii) triaxial shear test, iii) unconfined compression test and iv) vane shear test

UNIT-1

2 (a) Relationship between water content, void ration, degree of saturation and specific gravity of soil solids.

Derivation of relation between specific Gravity, water content, Degree of Saturation and void ratio (G, W, S & e)

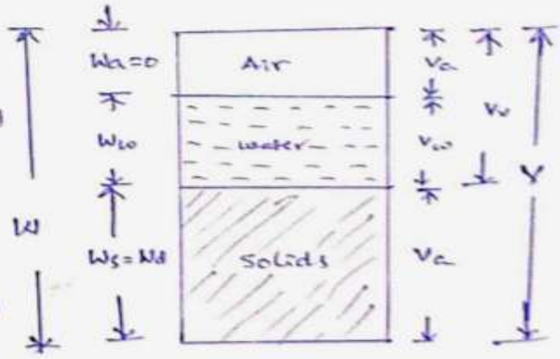
x:

We know that, water content $w = \frac{\text{wt. of water}}{\text{wt. of dry soil}}$

$\Rightarrow w = \frac{W_w}{W_d} \rightarrow (1)$

but wt of water $W_w = V_w \gamma_w \rightarrow (2)$

and wt. of dry soil W_s or $W_d = V_s \gamma_s \rightarrow (3)$



Phase diagram of partially saturated soil

~~substit~~ substitute (2) and (3) in eqn (1)

$\therefore w = \frac{V_w \gamma_w}{V_s \gamma_s} \rightarrow (4)$

degree of saturation $S = \frac{V_w}{V_v} \Rightarrow V_w = S V_v \rightarrow (5)$

sub (5) in (4)

$\therefore w = \frac{S V_v}{V_s} \cdot \frac{\gamma_w}{\gamma_s}$

$= S \cdot \frac{V_v}{V_s} \cdot \frac{\gamma_w}{\gamma_s} \rightarrow (6)$

Void ratio $e = \frac{V_v}{V_s} \rightarrow (7)$ and specific gravity $G = \frac{\gamma_s}{\gamma_w} \rightarrow (8)$

substitute (7) and (8) in (6)

$\therefore \cancel{w = S.e/G}$ $w = S.e.\frac{1}{G}$

$\Rightarrow \boxed{wG = S.e}$

if $S = 100\%$ then $e.1 = wG \Rightarrow e = wG$ //

.

2(b)

Given data: diameter of the sample d = 5.0 cm
Length of the sample = 10.2 cm
Mass of the wet sample = 387 gm
Mass of the dry sample = 313 gm
G = 2.7

To find : porosity, void ratio, degree of saturation, dry density of sample

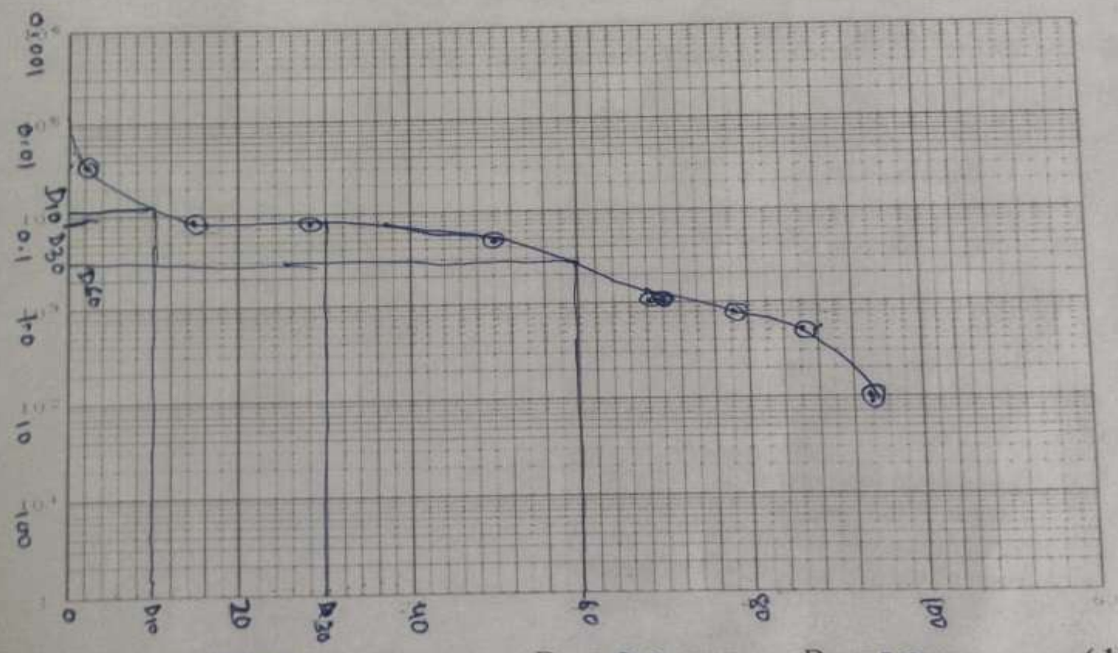
Water content w = [(387-313)/ 313]*100 = 23.64%
Void ratio e = wg = 23.64 x2.7= 0.64
Porocity n = e/(1+e) = 0.64 / (1+0.64) = 0.39
Degree of saturation: S = wG/e = 23.64 x 2.7/0.64 = 100%
Dry density γ_d = G x γ_w / (1+e) = 2.7 x 1.0 /(1+0.64) = 1.65 g/cc

3(a)

3(a) weight of sample taken = 500gm

Sieve size (mm)	Wt. retained (gm)	% wt retained	Cumulative % of wt retained	Percent finer
10	29	5.8	5.8	94.2
4.75	42	8.4	14.2	85.8
2.0	40	8.0	22.2	77.8
1.0	45	9.0	31.2	68.8
600 μ	89	17.8	49.0	51.0
425 μ	110	22.0	71.0	29.0
300 μ	68	13.6	84.6	15.4
150 μ	32	6.4	91.0	9.0
75 μ	35	7.0	98.0	2.0
Pan	10	2.0	100.0	0.0

(2M)



D_{10} = 0.09 mm , D_{30} = 0.2 mm, D_{60} = 0.8mm (1M)

Gravel G = 14.2 % Sand S = 83.8 % Fines = 2.0 % (1M)

$C_u = D_{60}/D_{10}$ = 9.0 C_c = $D_{30}^2/(D_{60} \times D_{10})$ = 0.55 (2M)

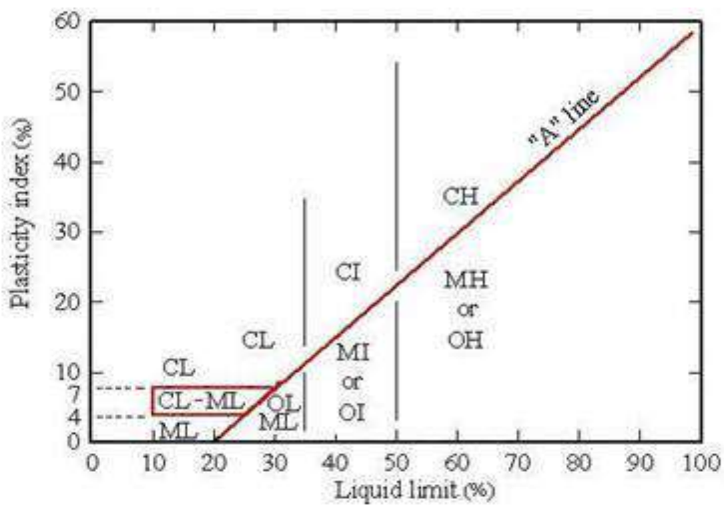
Soil Classification : SP (1M)

As Cu>6 and C_c<1, the given sand is poorly graded.

b) **Indian Standard Soil Classification System:**

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

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



Depending on the point in the chart, fine soils are divided into **clays (C)**, **silts (M)**, or **organic soils (O)**. The organic content is expressed as a percentage of the mass of organic matter in a given mass of soil to the mass of the dry soil solids.Three divisions of plasticity are also defined as follows.

Fine soils	
ML	SILT of low plasticity
MI	SILT of intermediate plasticity
MH	SILT of high plasticity
CL	CLAY of low plasticity
CI	CLAY of intermediate plasticity
CH	CLAY of high plasticity
OL	Organic soil of low plasticity
OI	Organic soil of intermediate plasticity
OH	Organic soil of high plasticity
Pt	Peat

UNIT-II

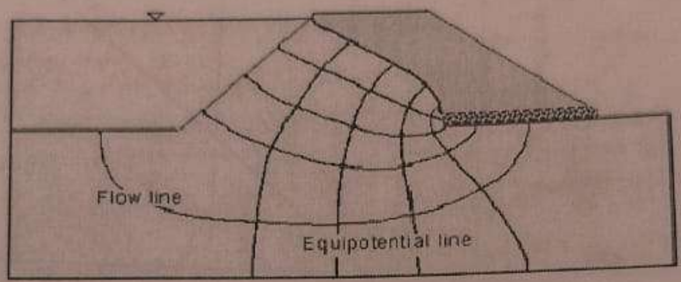
4(a)

UNIT-II

4(a) **Flow Nets** Graphical form of solutions to **Laplace equation** for two-dimensional seepage can be presented as flow nets. Two orthogonal sets of curves form a flow net:

- **Equipotential lines** connecting points of equal total head **h**
- **Flow lines** indicating the direction of seepage down a hydraulic gradient

Two flow lines can never meet and similarly, two equipotential lines can never meet. The space between two adjacent flow lines is known as a **flow channel,** and the figure formed on the flownet between any two adjacent flow lines and two adjacent equipotential lines is referred to as a **field.** Seepage through an embankment dam is shown.



Characteristics :: (1) Flow lines and equipotential lines cut each other at right angles i.e. mutually orthogonal

2) Each field is an approximate square and is a well connected flow net and it should be able to draw a circle in a field touching all the four sides

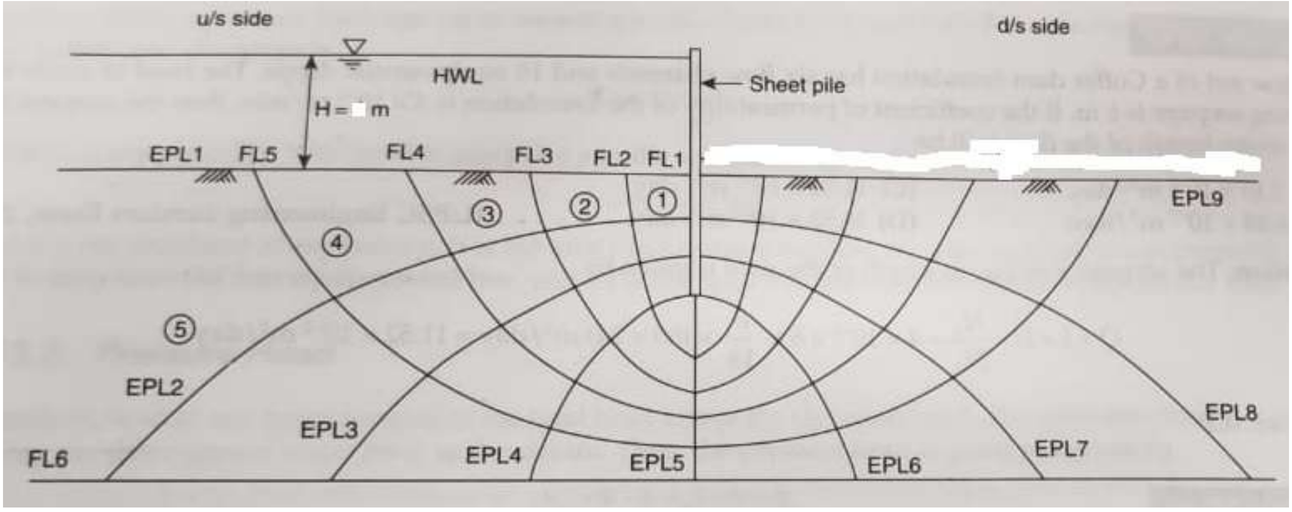
3. In a homogeneous soil, every transition in the shape of the two types of curves will be smooth, being either ellipti[c] or parabolic in shape.

4. The rate of flow through each flow channel is same

5. The same potential drop occurs between two success[ive] equipotential lines.

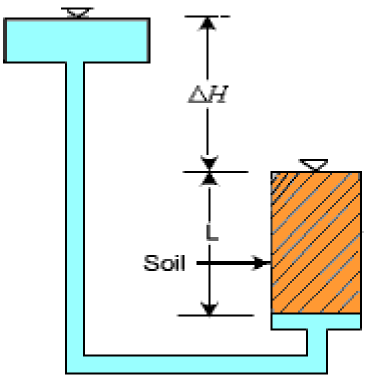
Uses: To determine (i) Quantity of seepage, (ii) seepage pressure at a point (iii) Hydrostatic pressure at a point and (iv) Exit gradient

b) A vertical sheet pile penetrates 8 m into a uniform sand stratum, 15 m thick, overlying an impervious layer. It retains water for 6 m above G.L. Draw the flow net and determine the seepage under the pile. Take $K = 4 \times 10^{-2}$ m/sec.



$K = 4 \times 10^{-2}$ m/sec.
$H = 6$m
$N_f = 5$
$N_d = 9$
Seepage $Q = KH(N_f / N_d)$
$= 4 \times 10^{-2} \times 6\ (5/9)$
$= 0.133\ m^3/sec$

5 (a) In an upward flow opposes the force of gravity and can even cause to counteract completely the contact forces. In such a situation, effective stress is reduced to zero and the soil behaves like a very viscous liquid. Such a state is known as **quick sand condition.** In nature, this condition is usually observed in coarse silt or fine sand subject to artesian conditions.



At the bottom of the soil column

$$\sigma = \gamma . L$$

$$u = \gamma_w (L + \Delta H)$$

During quick sand condition, the effective stress is reduced to zero.

$$\gamma . L = \gamma_w (L + \Delta H)$$

$$L(\gamma - \gamma_w) = \gamma_w . \Delta H$$
$$L . \gamma_s = \gamma_w . \Delta H$$
$$\frac{\Delta H}{L} = \frac{\gamma_s}{\gamma_w} = i_{cr} \approx 1$$

This shows that when water flows upward under a hydraulic gradient of about 1, it completely neutralizes the force on account of the weight of particles, and thus leaves the particles suspended in water.

b)

Solution: Permeability in the falling head test is given by

$$k = \frac{al}{At} \log_e \frac{h_1}{h_2}$$

Area of the standpipe, $a = \frac{\pi}{4} \times 12^2$

Area of the soil specimen, $A = \frac{\pi}{4} \times 100^2$

Length of the soil specimen, $l = 120$ mm.

Time for the head to fall from h_1 to h_2, $t_1 = 2$ h; $h_1 = 550$ mm; $h_2 = 410$ mm. Therefore,

$$k = \frac{(\pi/4) \times 12^2}{(\pi/4) \times 100^2} \times \frac{120}{2} \log_e \frac{550}{410} = 0.254 \text{ mm/h}$$

Time for the head to fall from h_1 to h_3:

$$k = \frac{al}{At_2} \log_e \frac{h_1}{h_3}$$

Now $h_1 = 550$ mm and $h_3 = 200$ mm. Therefore

$$0.254 = \frac{(\pi/4) \times 12^2}{(\pi/4) \times 100^2} \times \frac{120}{t_2} \log_e \frac{550}{200}$$

$$\Rightarrow t_2 = 6.882 \text{ h} = 6 \text{ h } 52.9 \text{ min}$$

Height of the water level in the standpipe after a period of $t_3 = 24$ h. We have

$$k = \frac{aL}{At_3} \log_e \frac{h_1}{h_4} \quad \Rightarrow \quad 0.254 = \frac{(\pi/4) \times 12^2}{(\pi/4) \times 100^2} \times \frac{120}{24} \log_e \frac{550}{h_4}$$

$$\Rightarrow \log_e \frac{550}{h_4} = \frac{0.254 \times 100^2 \times 24}{12^2 \times 120^2} = 3.528$$

$$\Rightarrow \frac{550}{h_4} = 34.048 \Rightarrow h_4 = 16.15 \text{ mm}$$

UNIT-III

6(a) A point load of 1000kN is applied at the ground level. Calculate the vertical stress below the point of application of the load at a depth of 5meters and also calculate the stress at the same depth but at a radial distance of 6 meters. Adopt Boussinesq's approach

Given data:

Point load Q = 1000 kN ; depth z = 5 m ; radial distance r = 6 m

Vertical stress $\sigma_z = (3Q/2\pi z^2) \{ 1/ [1 + (r/z)^2\}^{5/2}$

$= (3000/2x\pi x25) \{1/1+(6/5)^2\}^{5/2}$

$= 19.1 \{1/2.44\}^{5/2}$

$= 19.1 \{0.41\}^{5/2}$

$= 19.1 \sqrt{(0.011586)}$

$\sigma_z = 19.1 \text{ x } 0.1076 = 2.055 \text{ kN/m}^2$

b) The vertical stress at any depth in the soil due to the action of vertical load on the surface of the ground was given and explained by Boussinesq 's theory. The theory gave the formulae to calculate vertical stress at a point for different types of vertical loading , taking in to consideration only a few well defined and standard shape of loading like a point loading, line loading, strip loading etc . when some complex shape of loading , like a plan of a structure was given, itt became very cumbersome to calculate the vertical stress using these formulas. Hence a need for non simpler and faster method of stress calculation was realized. Newmark formulated a new simple graphical method to calculate the vertical stress at any particular depth caused due to any shape of vertical uniformly distributed loading in the interior of an elastic loading, homogeneous and isotropic medium which is bounded by horizontal planes.

Newmark's chart utilizes the equation given by Boussinesq for vertical stress caused due to uniformly distributed load on a circular area (vertical load) at any particular deth. $\sigma_c = I_c q$

Construction of chart:

The chart can be usd to determine the vertical stress at point P below the loaded area. A plan of the loaded area is drawn on a tracing paper to a scale such that the length AB is equal to the depth (z) of the point P below the surface. the traced plan of the loaded area is placed over the Newmark's chart such that the point P at which the pressure is required coincides with the centre of the chart. The vertical stress at point P is given by

$$\sigma_z = I \text{ x } n \text{ x } q$$

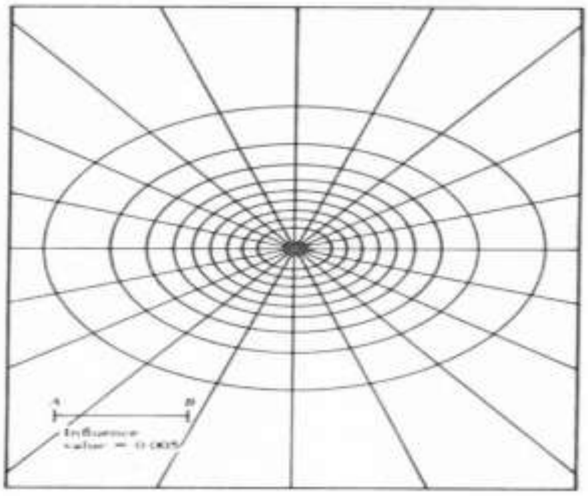
where I = influence coefficient (0.005 in this case),

n = number of small area units covered by the plaan. Each area between two successive radial lines and two successive concentric circle is taken as one unit,

q = intensity of load

Follow the 5 steps to determine the stress increase:

1. The fractions of the unit areas should also be counted and properly accounted for
2. If the plan of the loaded area extend, it may be assumed to approach the 10th circle for the purpose of counting the unit areas.
3. The point P at which the vertical stress is required may be anywhere within or outside the loaded area
4. If the depth at which the stress is required is changed, a fresh plan is required such that the new depth is equal to the distance AB on the chart.



7(a) Comparison of IS light compaction and Heavy compaction (For each 1 m)

IS Light Compaction	**Heavy Compaction**
310mm height of drop, weight – 2.6 kg	450 mm , weight – 4.9 kg
25 N hammer	45N
25 blows /layer	25 blows/layer
3 layers	5 layers
Mould capacity : 1000 cc	1000cc
Compactive energy :605160 N-mm/m^3	2726000 N-mm/m^3

b) Factors affecting compaction : (for each 1and 1/2 M)

The factors that influence the achieved degree of compaction in the laboratory are:

- Water content
- Compactive effort
- Plasticity of the soil
- Type of soil
- Method of compaction

i. Effect of Increasing Water Content

As water is added to a soil at low moisture contents, it becomes easier for the particles to move past one another during the application of compacting force. The particles come closer, the voids are reduced and this causes the dry density to increase. As the water content increases, the soil particles develop larger water films around them.

This increase in dry density continues till a stage is reached where water starts occupying the space that could have been occupied by the soil grains. Thus, the dry density increases up to a limiting moisture content (optimum moisture content), beyond which an increase in the moisture content decreases the dry density.

The **maximum dry density (MDD)** occurs at an **optimum water content (OMC),** and their values can be obtained from the plot.

The effect of the formation of a structure with increasing moisture content is another meaning given for the increase in the dry density and the subsequent decrease beyond a certain limit (Lambe, 1958).

ii. Effect of Increasing Compactive Effort

The effect of increasing compactive effort is shown. Different curves are obtained for different compactive efforts. A greater compactive effort reduces the optimum moisture content and increases the maximum dry density.

An increase in compactive effort produces a very large increase in dry density for soil when it is compacted at water contents drier than the optimum moisture content.It should be noted that for moisture contents greater than the optimum, the use of heavier compaction effort will have only a small effect on increasing dry unit weights.

It can be seen that the compaction curve is not a unique soil characteristic. It depends on the compaction effort. For this reason, it is important to specify the compaction procedure (light or heavy) when giving values of MDD and OMC.

iii) Effect of Type of soil

Well graded coarse grained soils with smooth rounded particles show a high dry density, whereas uniform sands have a low maximum dry density.

Clayey soils have lower dry densities and higher optimum moisture contents than do sands. The effect of increasing the compactive effort is also more in clayey soils.

iv) Effect of Method of Compaction

It is ideal to develop a laboratory test which could produce a reasonable moisture-density curve so as to assess the maximum dry density and optimum moisture content.

As the processes of imparting energy to the soil are different in the field and laboratory, there may be different degrees of compaction depending on the method of compaction.

Field compaction is essentially a rolling or kneading type of compaction, whereas the laboratory compaction is of the dynamic-impact type.

UNIT-IV

8(a) one-dimensional consolidation

Assumptions: **(2M)**

1. The soil medium is completely saturated
2. The soil medium is isotropic and homogeneous
3. Darcy's law is valid for flow of water
4. Flow is one dimensional in the vertical direction
5. The coefficient of permeability is constant
6. The coefficient of volume compressibility is constant
7. The increase in stress on the compressible soil deposit is constant
8. Soil particles and water are incompressible

One dimensional theory is based on the following hypothesis **(1M)**

1. The change in volume of soil is equal to volume of pore water expelled.
2. The volume of pore water expelled is equal to change in volume of voids.
3. Since compression is in one direction the change in volume is equal to change in height.

The increase in vertical stress at any depth is equal to the decrease in excess pore water pressure at the depth

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

This is Terzaghi's one dimensional consolidation equation

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

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

(FOR Solution 4M)

Solution of 1D consolidation

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

The solution of variation of excess pore water pressure with depth and time can be obtained for various initial conditions.

Uniform excess pore water pressure with depth

1. Single Drainage (Drainage at top and bottom impervious)

2. Double Drainage (Drainage at top and bottom)

Boundary Conditions are

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

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

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

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

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

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

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

Graphical solution of 1D consolidation equation

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

The solution of consolidation equation consists of the following three variables

b)

The increased stress, $\bar{\sigma}_o + \Delta\bar{\sigma}$ is equal to 70 + 80 = 150 kN/ m^2. This is greater than $\bar{\sigma}_c$ (= 120 kN/m^2).

Hence, Eq. 9.42 must be used, that is,

$$S_c = C_r \frac{H_o}{1+e_o} \log \frac{\bar{\sigma}_c}{\bar{\sigma}} + C_c \frac{H_o}{1+e_o} \log \frac{\bar{\sigma}_o + \Delta\bar{\sigma}}{\bar{\sigma}_c}$$

$$S_c = 0.03 \frac{5\times10^3}{1+0.90} \log \frac{120}{70} + 0.27 \frac{5\times10^3}{1+0.90} \log \frac{150}{120}$$

$$= 18.48 + 68.85$$

$$= 87.33 \text{ mm}$$

9(a)

The following three standard types of triaxial tests generally are conducted:

1. Consolidated-drained test or drained test (CD test)
2. Consolidated-undrained test (CU test)
3. Unconsolidated-undrained test or undrained test (UU test)

Consolidated-Drained Triaxial Test : In the CD test, the saturated specimen first is subjected to an all-around confining pressure, □3, by compression of the chamber fluid. As confining pressure is applied, the pore water pressure of the specimen increases by uc (if drainage is prevented).

Consolidated-Undrained Triaxial Test: The consolidated-undrained test is the most common type of triaxial test. In this test, the saturated soil specimen is first consolidated by an all-around chamber fluid pressure, □□3, that results in drainage. After the pore water pressure generated by the application of confining pressure is dissipated, the deviator stress, *(*Δ□d), on the specimen, is increased to cause shear failure

Unconsolidated-Undrained Triaxial Test

In unconsolidated-undrained tests, drainage from the soil specimen is not permitted during the application of chamber pressure □3. The test specimen is sheared to failure by the application of deviator stress, *(*Δ□d), and drainage is prevented. Because drainage is not allowed at any stage, the test can be performed quickly.

b)

Solution:

Initial area of the specimen $A_0 = \pi / 4 \times 37.5^2 = 1105 \text{ mm}^2$

$$\text{Axial strain } \epsilon = \frac{13}{80} = 0.162.$$

From Eq. 10.15, the corrected area of the specimen at failure is given by

$$A_f = \frac{A_0}{1-\epsilon} = \frac{1105}{1-0.162} = 1315 \text{ mm}^2$$

Unconfined compressive strength, $q_u = \dfrac{28(\text{N})}{1315(\text{mm}^2)} = 21.3 \text{ kN/m}^2.$

The undrained shear strength of the clay is one-half its unconfined compressive strength. Hence,

$$c_u = \frac{q_u}{2} = \frac{21.3}{2} = 10.7 \text{ kN/m}^2$$