CBCS SCHEME

18CV54

USN

Fifth Semester B.E. Degree Examination, July/August 2021
Basic Geotechnical Engineering

Time: 3 hrs. Max. Marks: 100

Note: ***Answer any FIVE full questions.***

1 a. With the help of 3 – phase diagram, define Void ratio , Porosity , Water content and Degree of saturation. **(08 Marks)**

b. The mass of wet soil when compacted in a mould was 19.17N. The water content of the soil mass was 16%. If the volume of mould was 0.945 litres, determine
i) Dry density ii) Void ratio iii) Degree of saturation iv) Percent air voids.
Take G = 2.68. **(12 Marks)**

2 a. Define Liquid limit, Plastic limit and Shrinkage limit ; Liquidity index and Relative consistency. **(06 Marks)**

b. Explain determination of In-situ density of soil by Sand replacement method. **(08 Marks)**

c. The liquid limit test on soil sample gives the following results. The plastic limit of the soil is 40%.

No. of Blows	12	18	22	34
Water content %	56	52	50	45

Plot a flow curve and obtain i) Liquid limit ii) Flow Index iii) Plasticity Index and iv) Toughness Index. **(06 Marks)**

3 a. List and explain various soil structures. **(06 Marks)**

b. Briefly explain the factors affecting compaction. **(06 Marks)**

c. A standard proctor test was performed on a soil sample of specific gravity 2.70, with the following results :
Maximum dry unit weight = $18 kN/m^3$; Optimum moisture content = 16%.
If the compaction effect was increased so that the maximum unit weight is $19.2\ kN/m^3$, assuming same degree of saturation, what should be the corresponding OMC. **(08 Marks)**

4 a. Explain Common clay minerals with sketches. **(06 Marks)**

b. Explain Electrical diffuse double layer and adsorbed water. **(06 Marks)**

c. A soil in the borrowpit is at a dry density of $16.67\ kN/m^3$ with water content of 12%. If the soil of $2000 m^3$ is excavated from it and compacted in an embankment with porosity of 0.32, calculate the volume of embankment which can be constructed out of this material.
Take G = 2.70. **(08 Marks)**

5 a. Explain the following : i) Effective stress analysis ii) Seepage analysis. **(06 Marks)**

b. With the help of a neat sketch, derive the equation to determine permeability by Falling Head Permeability Test. **(06 Marks)**

c. Calculate the seepage through an earth dam resting on an impervious foundation. The relevant data are given below :
Height of Dam = 60.0m ; Free Board = 2.5m ; Upstream slope = 2.75 : 1 ;
Crest width = 8.0m ; Downstream slope = 2.50 : 1 ; Length of drainage blanket = 120.0m.
Coefficient of permeability of the embankment material in x – direction = 8×10^{-7} m/s ;
y – direction = 2×10^{-7} m/s. **(08 Marks)**

6 a. What is a Flownet? What are the characteristics and uses of the Flownet? (06 Marks)
b. Describe the Casagrande's method to locate the phreatic line in a homogeneous earth dam with a horizontal filter @ its toe. (06 Marks)
c. A soil sample of height 60mm with cross sectional area 8000mm^2 was subjected to a falling head permeability test. In a time interval of 6 minutes, the head dropped from 750mm to 300mm. If the cross sectional area of stand pipe is 150mm^2, compute the coefficient of permeability. If the same sample is subjected to a constant head of 200mm, compute the total quantity of water that will get discharged through the sample in a time interval of 10 minutes. (08 Marks)

7 a. Explain Mohr – Coulomb failure theory of soil. (06 Marks)
b. List the different methods to measure the shear strength of soil. Explain any one of them. (06 Marks)
c. A shear test was carried out and the following results are recorded :

Normal stress (kN/m^2)	200	250
Shear stress (kN/m^2)	100	125

Find shear parameters, what would be the deviator stress at failure if a biaxial test is carried out from the same soil with cell pressure of 100kN/m^2. (08 Marks)

8 a. Explain the advantages of Triaxial shear test over Direct shear test. (06 Marks)
b. What are the factors affecting the shear strength of soil? (06 Marks)
c. A cylindrical specimen of saturated clay 40mm in diameter and 80mm in length is tested in an unconfined compression test. Find shear strength of clay, if the specimen fails under an axial load of 350N. The change in length of the specimen @ failure is 8mm. Also find the shear parameters if the angle made by the failure plane with horizontal is 50^0. (08 Marks)

9 a. Enumerate the assumptions and limitations of Terzaghi's Consolidation theory. (06 Marks)
b. Briefly explain normally consolidated, under consolidated and over consolidated soils. (06 Marks)
c. A soil sample 20mm thick takes 20 minutes to reach 20% consolidation. Find the time taken for a clay layer 6m thick to reach 40% consolidation. Assume double drainage in both cases. (08 Marks)

10 a. Explain Mass – Spring Analogy. (06 Marks)
b. Explain determination of coefficient of consolidation by square root of Time Fitting method. (06 Marks)
c. In a consolidation test, the void ratio of soil sample decreases from 1.20 to 1.10. When the pressure increased from 200kN/m^2 to 400kN/m^2. Calculate the coefficient of consolidation if the coefficient of permeability is 8×10^{-7} mm/s. (08 Marks)

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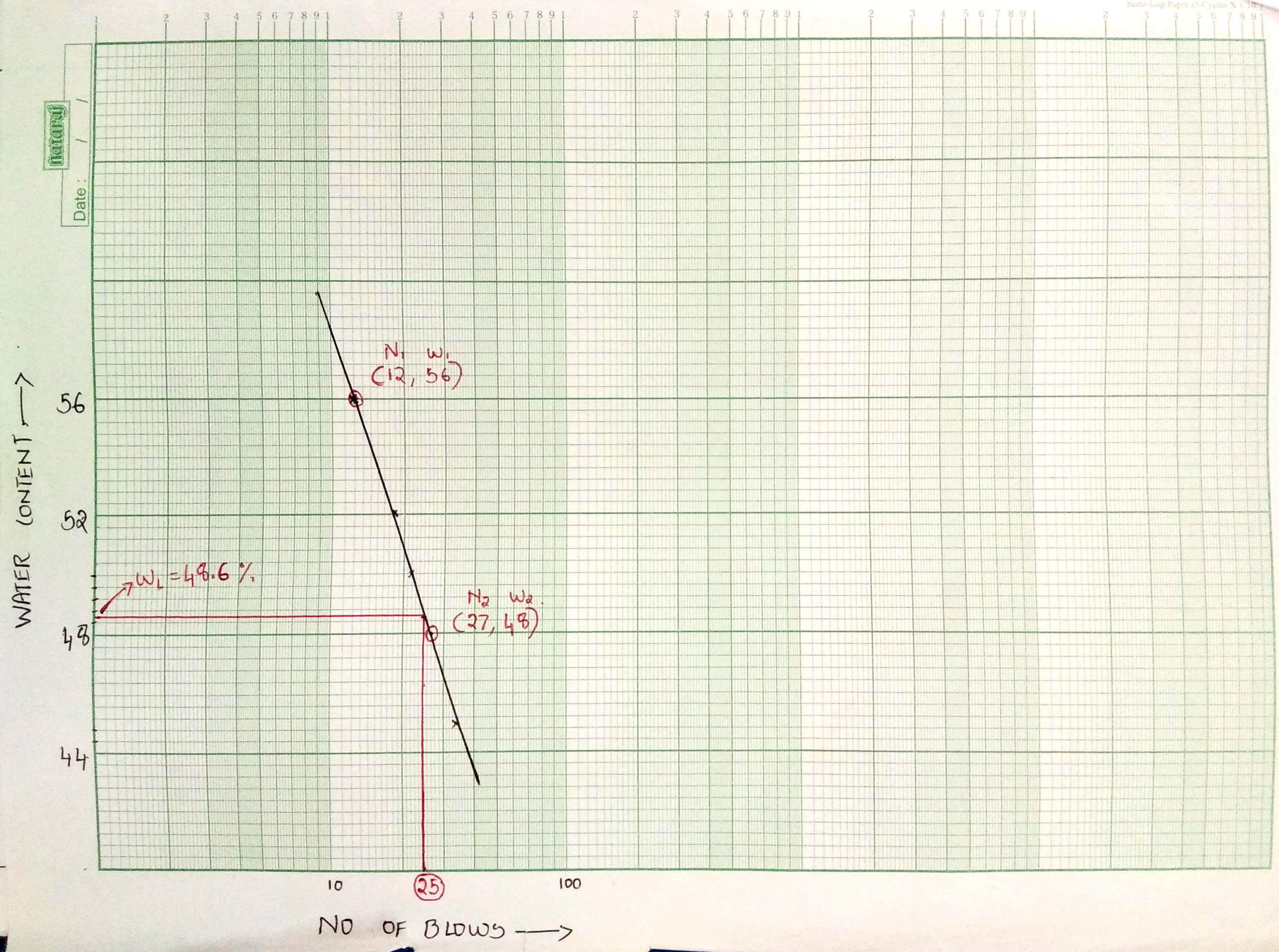
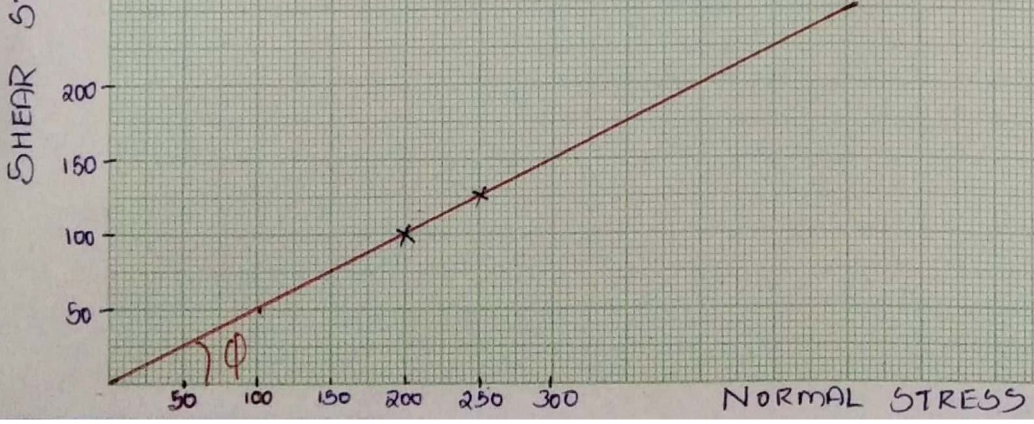
GRAPH 1



WATER CONTENT →
56
52
48
44
w_L = 48.6 %
N_1 w_1
(12, 56)
N_2 w_2
(27, 48)
10
25
100
NO OF BLOWS →

GRAPH 2

C = 0 φ = 26.56°



V SEMESTER B.E DEGREE EXAMINATION JULY-AUG 2021
BASIC GEOTECHNICAL ENGINEERING (18CV54)
QUESTION PAPER SOLUTION

Q1.a.

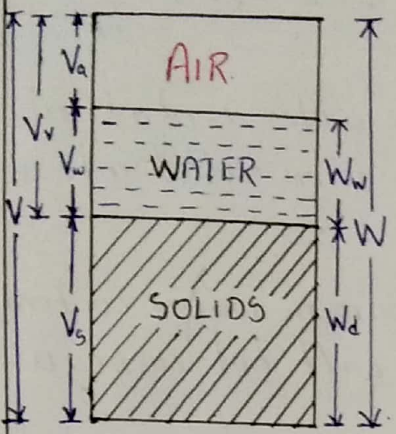


Fig 2M

① VOIDS RATIO: is defined as the ratio volume of voids to volume of solids. → 1M

$$e = \frac{V_v}{V_s}$$ → 0.5M (eqn)

② POROSITY: is defined as the ratio of volume of voids to total volume of soil. → 1M

$$n = \frac{V_v}{V} \times 100\%$$ → 0.5M

③ WATER CONTENT: is defined as the ratio of weight of water to weight of solids. → 1M

$$w = \frac{W_w}{W_d} \times 100\%$$ → 0.5M

④ DEGREE OF SATURATION: is defined as the ratio of volume of water to volume of voids. → 1M

$$S = \frac{V_w}{V_v} \times 100\%$$ → 0.5M

Q1.b.

Given, $W = 19.17N$ $w = 16\%$ $V = 0.945$ lit $G = 2.68$

$$\gamma_b = \frac{W}{V} = \frac{19.17\,N}{945 \times 10^{-6} m^3} = 20.28 \times 10^3 N/m^3 = 20.28\,kN/m^3$$

$$\gamma_d = \frac{\gamma_b}{1+w} = \frac{20.28}{1 + 16/100} = 17.48\,kN/m^3.$$

$$\gamma_d = \frac{G\gamma_w}{1+e} \Rightarrow e = \frac{G\gamma_w}{\gamma_d} - 1 = \frac{2.68 \times 9.81}{17.48} - 1 = 0.5$$

$$eS = wG \Rightarrow S = wG/e = \frac{0.16 \times 2.68}{0.5} = 0.8576 = 85.76\%$$

$$\gamma_d = (1-n_a)\frac{\gamma_w G}{1+wG} \Rightarrow n_a = 1 - \left[\frac{\gamma_d}{\gamma_w G}(1+wG)\right] = 1 - \frac{17.48}{9.81 \times 2.68}(1+0.16 \times 2.68) = 0.05$$

$$= 5\%.$$

Q2.a. LIQUID LIMIT: is the water content corresponding to the abritrary limit b/w plastic state & liquid state.
If water content is more than liquid limit then soil is in liquid state.

PLASTIC LIMIT: is defined as the minimum water content at which a soil will just begin to crumble when rolled into a thread approximately 3mm in diameter.

SHRINKAGE LIMIT: is defined as the maximum water content at which a reduction in water content will not cause a decrease in the volume of a soil mass.

LIQUIDITY INDEX: of a soil is given by $I_L = \frac{w - w_P}{I_P}$. It is used to indicate consistency of soil.

RELATIVE CONSISTENCY: of a soil is given by $I_c = \frac{w_L - w}{I_P}$.
It is useful for study of field behaviour of saturated fine grained soils.

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b. SAND REPLACEMENT METHOD.

This method consists of 2 parts

(i) Calibration of cylinder

- Measure internal dimension of calibrating ~~cylinder~~ container & compute its 'V'.
- Clean the cylinder & take its empty weight W_1
- With valve closed, fill th cylinder with sand & take weight, W_2.
- keep cylinder on glass plate. Open the valve & allow the sand to fill the cone completely. Close the valve & take weight of sand on glass plate, W_3.
- keep cylinder on top of calibrating container & open th valve & allow the sand to fill container till no movement of sand is noticed. Then cylinder is weighed (W_4).

Wt. of sand in calibrating container, $W_c = W_2 - W_3 - W_4$

Bulk Density of sand, $\rho_s = \frac{W_c}{V_c}$

ii) Determination of Soil Density.

- Level the area where density is required.
- Place the metal tray on the surface, which is having a circular hole of about 10cm in the centre.
- Dig a hole upto 15cm depth & collect all the excavated soil in the metal plate tray. Weigh the soil collected (W_s).
- Remove the plate & place the cylinder concentrically on the hole. Fill the cylinder with sand by opening the valve & close it when there is no movement of sand. Determine the weight of sand pouring cylinder (W_5).

$$\text{Volume of hole} = \frac{W_{sand}}{\rho_s} = \frac{W_2 - W_3 - W_5}{\rho_s}.$$

$$\text{Bulk density of soil} = \frac{W_{soil}}{V_{HOLE}} = \frac{W_s}{V_{HOLE}}$$

Q2.c. From graph (1) we have,

i) $w_L = 48.6\%$.

ii) $I_F = \dfrac{w_2 - w_1}{\log(N_2/N_1)} = \dfrac{48-56}{\log(27/12)} = -22.7$

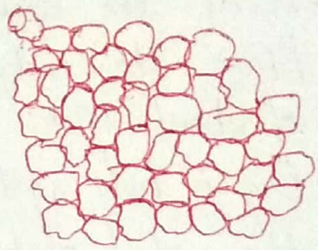
iii) Given, $w_P = 40\%$

$\therefore I_P = w_L - w_P = 48.6 - 40 = 8.6\%$.

iv) $I_T = \dfrac{I_P}{I_F} = \dfrac{8.6}{22.7} = 0.38$

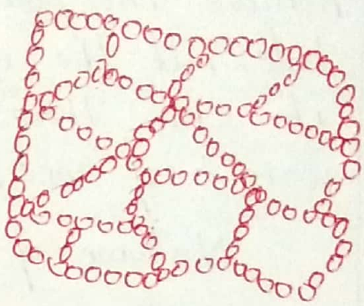
Q3.a. SINGLE GRAINED STRUCTURE.

Such structure exists in coarse grained soils which is composed of bulky grains where gravitational forces are more pre-dominant than surface force.



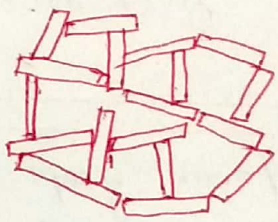
HONEYCOMB STRUCTURE.

Such a structure exists in grains of silts or rock flour smaller than 0.02mm diameter & larger than 0.0002mm.
Surface forces at contact areas are large compared to submerged wt. to prevent the grains from rolling down immediately into equilibrium positions.



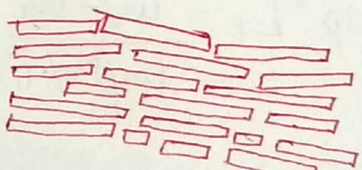
FLOCUCULENT STRUCTURE.

Such a structure is found in clay when platelets is formed when there is edge to edge contact b/w them.
A clay having flocculent structure has high voids ratio.



DISPERSED STRUCTURE.

Such a structure is found in clay when platelets have face to face contact in more or less parallel array.

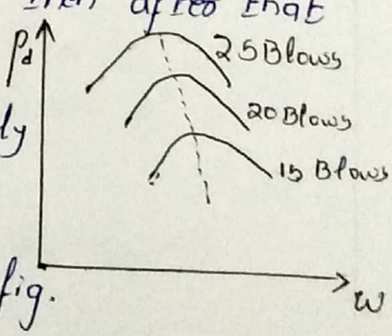


Remoulding, compacting ~~&~~ consolidation of clays tend to produce a dispersed structure.

Q3.b. The factors affecting compaction are

1> WATER CONTENT :: As water content increases the dry density increases upto a certain extent (i.e OMC) & then after that dry decreases.

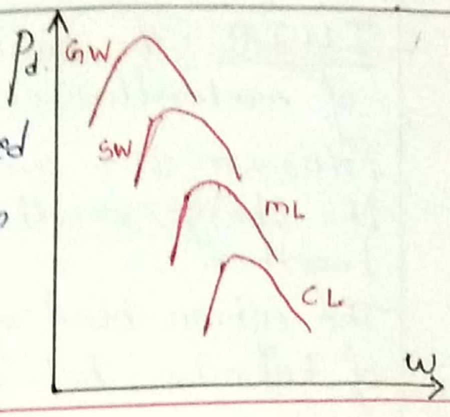
2> AMOUNT OF COMPACTION :: Amt. of compaction greatly affects the MDD and OMC of a given soil.
Increase in compactive energy results in an increase in MDD & decrease in OMC as shown in fig.



3> METHOD OF COMPACTION : The density obtained during compaction greatly depends on a> wt. of compacting equipment. b> manner of operation such as kneading, rolling, tamping, vibration etc c> time & area of contact.

4) TYPE OF SOIL

Well graded coarse soil attain a much higher density & lower OMC than fine grained soils (which require more water for lubrication because of greater specific surface).



Q3.c. Given $G = 2.7$ $\gamma_{d\,max} = 18\,kN/m^3$ $OMC_1 = w_1 = 16\%$

$\gamma_{d\,max} = 19.2\,kN/m^3$ $OMC_2 = ?$

Case 1: We have $\gamma_{d_1} = \dfrac{G\gamma_w}{1 + \frac{wG}{S}}$ $\Rightarrow \dfrac{wG}{S} = \dfrac{G\gamma_w}{\gamma_{d_1}} - 1$

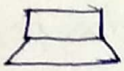
$\Rightarrow \dfrac{0.16 \times 2.7}{S} = \dfrac{2.7 \times 9.81}{18} - 1$ $\Rightarrow S = 0.916 = 91.6\%$

Case 2: $\gamma_{d_2} = \dfrac{G\gamma_w}{1 + w_2G/S}$ $\Rightarrow \dfrac{w_2G}{S} = \dfrac{G\gamma_w}{\gamma_{d_2}} - 1 \Rightarrow$

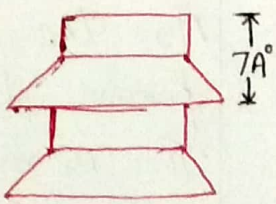
$\Rightarrow \dfrac{w_2 \times 0.916}{0.916} = \dfrac{2.7 \times 9.81}{19.2} - 1$ $\Rightarrow w_2 = 0.1287 = 12.87\%$

Q4.a. KAOLINITE: is the most common mineral of the kaolin group.

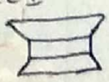
The kaolinite structural unit is made up of gibbsite sheet (with Al atoms at their centres) joined to silica sheet through the unbalanced oxygen atoms at apexes of silicas.

This structural unit is symbolised by  which is about 7A° thick.

Successive layers are held together with hydrogen bonds.

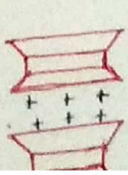


MONTMORILLONITE: is most common clay mineral in expansive clay soils.

Basic structure of each unit is made up of gibbsite sheet sandwiched b/w 2 silica sheets & is symbolised as 

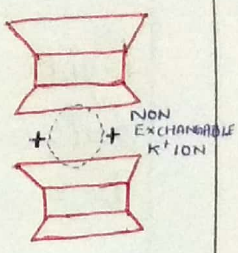
The thickness of each unit is 10 A°.

There is a weak bonding b/w successive sheets & water may enter b/w sheets causing mineral to swell.



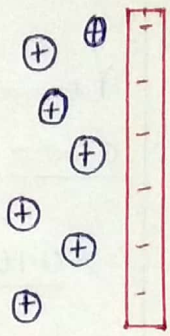
Q4.c

ILLITE: The basic structure of each unit is same as that of montmorillonite.



Potassium ions are b/w the layers serving to balance the charge resulting & to tie the sheets together.
The cation bond of illite is weaker than hydrogen bond of kalinite, but it is stronger than water bond of montmorillonite.

Q4.b. **ELECTRICAL DIFFUSE DOUBLE LAYER.**



Soil colloid suspended in water carries nearly always a net -ve charge
Since the net charge of entire soil water suspension must be zero, the charge on each colloid must be neutralized by ions from water which swarm around each colloid.
These ions are called counter ions or exchangeble ions.
The positions of counter ions are that of compromise b/w the particle charge which pulls them in & their thermal activities plus attraction by other bodies which pulls them out.
Thus counter ions constitute Diffuse Double Layer.

ADSORBED WATER.

The water held by electro-chemical forces existing on the soil surface is known as adsorbed water.
As the adsorbed water is under the influence of electrical forces, its properties are different from normal water.
It is much more viscous & its surface tension is also greater.
It is heavier than normal water.
The boiling point is higher & freezing point is lower than that of normal water.

Q4.c. Given, $\gamma_d = 16.67\,kN/m^3$ $w = 12\%$. $V = 2000\,m^3 \rightarrow$ SOURCE.

$n = 0.32$, $V = ?$ $\longrightarrow$ EMBANKMENT.

① For source.

$\gamma_d = \frac{W_d}{V} \Rightarrow W_d = \gamma_d \times V = 16.67\,kN/m^3 \times 2000\,m^3 = 33340\,kN$

$e = \frac{G\gamma_w}{\gamma_d} - 1 = \frac{2.7 \times 9.81}{16.67} - 1 = 0.59$

② For Embankment. $(n = 0.32)$

$e = \frac{n}{1-n} = \frac{0.32}{1-0.32} = 0.47$

$\gamma_d = \frac{G\gamma_w}{1+e} = \frac{2.7 \times 9.81}{1+0.47} = 18\,kN/m^3$

$\gamma_d = \frac{W_d}{V} \Rightarrow V = \frac{W_d}{\gamma_d} = \frac{33340\,kN}{18\,kN/m^3} = 1852.2\,m^3$.

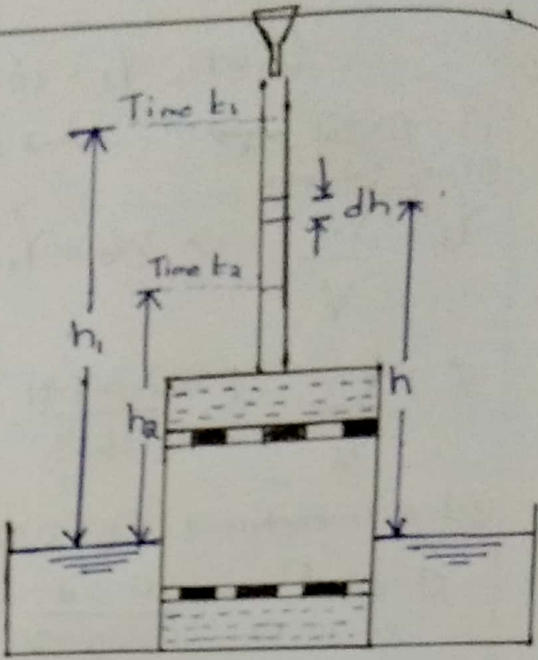
Q5.a. EFFECTIVE STRESS ANALYSIS: Effective stress is the pressure transmitted from particle through their point of contact through the soil mass above the plane.
Such a pressure, also termed as intergranular pressure is effective in decreasing the voids ratio of soil mass & in mobilising its shear strength.

SEEPAGE ANALYSIS: In seepage analysis we are determining the seepage pressure and hydrostatic pressure.
We are also determining the quantity of seepage through the soil.
Seepage analysis also involves calculation of gradient at which seepage water exits & compare it with critical gradient.

Q5.b. Let h_1 and h_2 be the heads at time intervals t_1 and t_2 ($t_2 > t_1$) respectively.

Let 'h' be the head at any intermediate time interval 't'.

Let 'dh' be the change in head in a smaller time interval 'dt'



From Darcy's Law,

$$q = va = -\frac{dh}{dt}a \quad \& \quad q = kiA$$

$$\Rightarrow -\frac{dh}{dt}a = kiA \quad \Rightarrow -dh\, a = k(h/L)A\,dt$$

$$\Rightarrow -\frac{dh}{h} = \frac{kA}{aL}dt \quad \Rightarrow \int_{t_1}^{t_2}\frac{kA}{aL}dt = \int_{h_1}^{h_2}-\frac{dh}{h}$$

$$\Rightarrow \frac{kA}{aL}[t]_{t_1}^{t_2} = -[\log h]_{h_1}^{h_2} \Rightarrow \frac{kA}{aL}(t_2-t_1) = -[\log h_2 - \log h_1]$$

$$\Rightarrow \frac{kA}{aL}(t) = \log h_1 - \log h_2 \Rightarrow k = \frac{aL}{At}\log_e(h_1/h_2)$$

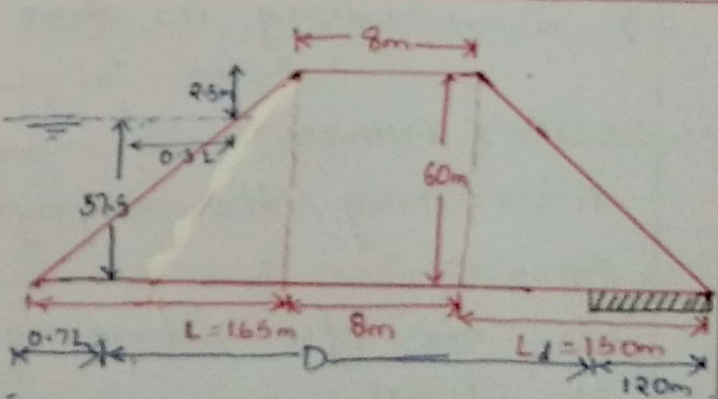
Q5.c

$1 : 2.75 :: 60 : L.$

$L = 60 \times 2.75 = 165\,m$

III'ly $L_d = 60 \times 2.5 = 150\,m$

$0.7L = 0.7 \times 165 = 115.5\,m$



D = (TOT WIDTH − 0.7L − 120
$= (165+8+150) - 115.5 - 120 = 87.5\,m.$

$S = \sqrt{D^2+H^2} - D = \sqrt{87.5^2+60^2} - 87.5 = 18.6\,m.$

Quantity of seepage per m $= kS = 4\times10^{-7}\,m/s \times 18.6\,m = 7.44\times10^{-6}\,m^3/sec.$

$k = \sqrt{k_H \times k_V} = \sqrt{8\times10^{-7}\times2\times10^{-7}} = 4\times10^{-7}\,m/s.$

Q6.a. The solution of 2 sets of curves known as equipotential lines & stream lines mutually ⊥ to each other is called as FLOWNETS.

CHARACTERISTICS.

1) Streamlines & equipotential lines meet right angles to each other.

2) The fields are approximately square, so that a circle can be drawn touching all the 4 sides of square.

3) Quantity of water flowing through each channel is the same.

4) Potential drop b/w 2 successive equipotential lines is same.

5) Smaller the dimension of field, greater the hydraulic gradient will be.

6) Every transition in the shape of the curve is smooth being either elliptical or parabolic in shape.

They are used in the determination of

i) Seepage quantity ii) Hydrostatic pressure.
iii) Seepage Pressure iv) Exit gradient.

Q6.b. 1) Let AB be the upstream face whose projection is L.

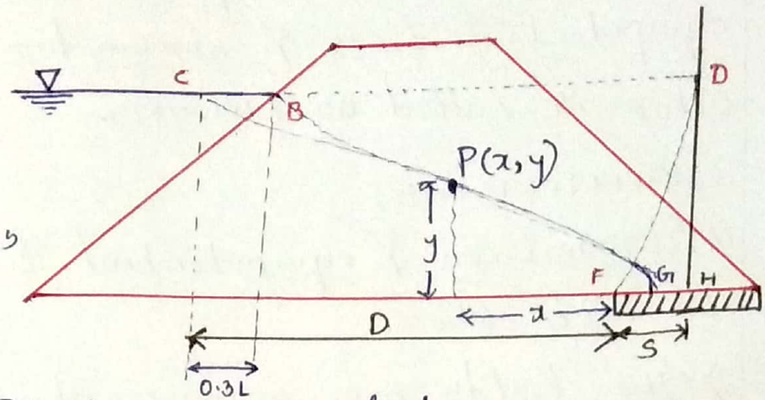
2) On water surface, measure a distance BC = 0.3L. Then 'C' is starting point of base parabola.

3) With point 'C' as centre & CF as radius, draw an arc to cut horizontal line through CB in D.

4) Draw a vertical tangent to the curve FD at 'D'. Hence DH is directrix.

5) Last point 'G' on parabola will lie mid-way b/w F and H.

6) In order to locate th intermediate points on the parabola we use the principle that its distances from focus & directrix must be equal.



To locate any point 'P' draw vertical line QP. at any distance 'x' from F. Measure QH.
With F as centre & QH as radius draw an arc to cut vertical line through Q in point 'P'.

7) Join all points to base parabola.

8) Phreatic line must start from B ∴ sketch phreatic line at B free hand s.t it starts ⊥r to AB. It should also meet d/s filter ⊥rly.

Q6.c. Given $L = 60mm$ $A = 8000mm^2$ $t = 6min$ $h_1 = 750mm$ $h_2 = 300mm$ $a = 150mm^2$ $k = ?$ → (i)
$H = 200mm$ $Q = ?$ $t = 10min$ → (ii).

Case (i)

$$k = 2.3\frac{aL}{At}\log_{10}(h_1/h_2) = 2.3\times\frac{150\times 60}{8000\times 6}\log(750/300) = 0.17\ mm/min.$$

case (ii)

$$k = \frac{QL}{hAt} \Rightarrow Q = \frac{khAt}{L} = \frac{0.17mm/min \times 200mm \times 8000mm^2 \times 6min}{60mm}$$

$$\Rightarrow Q = 27200 mm^3.$$

Q7.a. The theory was first expressed by Coulomb & later generalised by Mohr. The theory can be expressed by the eqn.

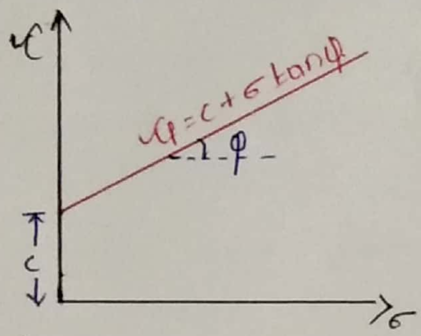
$$\tau_f = F(\sigma)$$

$$\Rightarrow \tau_f = c + \sigma \tan\phi .$$

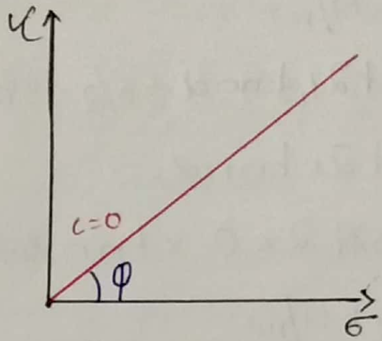
where, $\tau_f \rightarrow$ shear stress on failure plane.

$c \rightarrow$ cohesion of soil, $\phi \rightarrow$ Angle of internal friction.

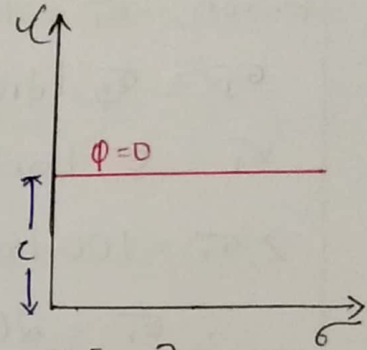
$\sigma \rightarrow$ Normal stress.



i) STRENGTH ENVELOPE



ii) FOR COHESIONLESS SOIL



iii) FOR PURELY COHESIVE SOIL.

If normal & shear stress corresponding to failure are plotted, then curve is obtained which is called as STRENGTH ENVELOPE.

Q7.b. The different methods to measure the shear strength of soil are.

i) DIRECT SHEAR TEST ii) TRIAXIAL TEST.

iii) UNCONFINED COMPRESSION TEST iv) VANE SHEAR TEST.

UNCONFINED COMPRESSION TEST.

1) Apparatus consists of small load frame fitted with proving ring.

2) Deformation of sample is measured with a dial gauge.

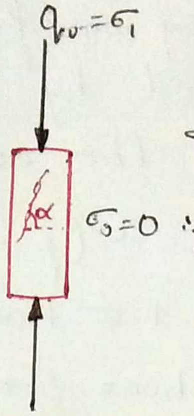
3) Cylindrical specimen of soil is subjected to vertical load gradually increasing till the specimen fails

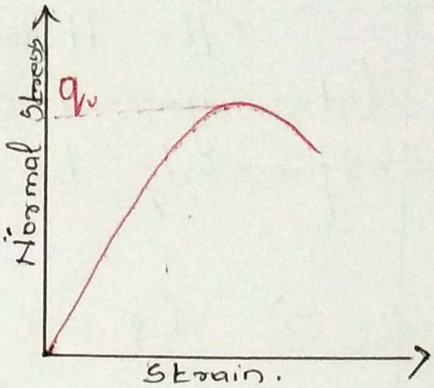
4) Load versus deformation readings are taken & graph is plotted.

In ULT, $\sigma_3 = 0$

$\sigma_1 = q_u$.

$\alpha \rightarrow$ Angle of failure plane





Q7.c. From Graph 2, $c = 0$ $\qquad \phi = 26.56°$

Given, $\sigma_c = 100\,kN/m^2$.

$$\alpha = 45° + \phi/2 = 45° + 26.56°/2 = 58.28°$$

$$\sigma_1 = \sigma_3 \tan^2\alpha + 2c\tan\alpha.$$

$$\Rightarrow \sigma_1 = 100\tan^2 58.28 + 2 \times 0 \times \tan 58.28$$

$$\therefore \sigma_1 = 261.8\,kN/m^2.$$

$$\sigma_1 = \sigma_c + \sigma_d \quad \Rightarrow \quad \sigma_d = \sigma_1 - \sigma_c = 261.8 - 100$$

$$\therefore \sigma_d = 161.8\,kN/m^2.$$

Q8.a. Advantages of Triaxial test over Direct shear test

i) There is full control of drainage.

ii) Pore pressure can be measured.

iii) Allows sample to fail along the weakest one.

iv) Distribution of stresses are uniform.

v) Effective shear strength parameters can be measured.

Q8.b. The factors affecting shear strength of soil are.

i) SHAPE OF PARTICLE: Shear strength of sand with ANGULAR particles & SHARP EDGED is greater than that of ROUNDED particles.

ii) GRADATION: Well graded sand exhibits greater shear strength than uniform sand or poorly graded sand.

iii) DENSENESS: Greater the density, greater the strength.

iv) SATURATION OF CLAY: As saturation increases, shear strength decreases.

v) CLAY CONTENT: As clay content increases in th soil mass, shear strength decreases.

vi) CONFINING PRESSURE: Shear strength increases with increase in confining pressure.

Q9.a. ASSUMPTIONS.

1) The soil is homogeneous & fully saturated.

2) Soil particles & water are incompressible.

3) The deformation of soil is entirely due to change in volume.

LIMITATIONS.

1) Flow is assumed to be 1D but in reality it is 3D.

2) As consolidation progresses voids ratio decreases hence permeability decreases. Therefore the assumption that " is constant is FALSE.

3) Application of load is assumed to produce excess porepressure, but in some case it doesn't develop over entire stratum.

Q9.b OVER CONSOLIDATED SOIL

A soil is said to be over consolidated if it has ever been subjected to a pressure in excess of its present overburden pressure.

The temporary over burden pressure to which a soil was subjected & under which it got consolidated is known as OVER CONSOLIDATED SOIL.

NORMALLY CONSOLIDATED SOIL

is one which has never been subjected to an effective pressure greater than the existing overburden pressure & which is also consolidated completely by existing overburden.

UNDER-CONSOLIDATED SOIL.

A soil which is not fully consolidated under existing overburden pressure is called an under consolidated soil.

Q9. c.

Given, $H_1 = 20mm$ $t_1 = 20min$ $U_1 = 20\%$

$t_2 = ?$ $H_2 = 6m = 6000mm$ $U_2 = 40\%$

CASE 1:

$d_1 = H_1/2 = 20/2 = 10mm$

$T_{v_1} = \pi/4 (U_1/100)^2 = \pi/4 (20/100)^2 = 0.0314$

$C_v = \dfrac{T_{v_1} d_1^2}{t_1} = \dfrac{0.0314 \times (10^2) mm^2}{20 min} = 0.157 \ mm^2/min$

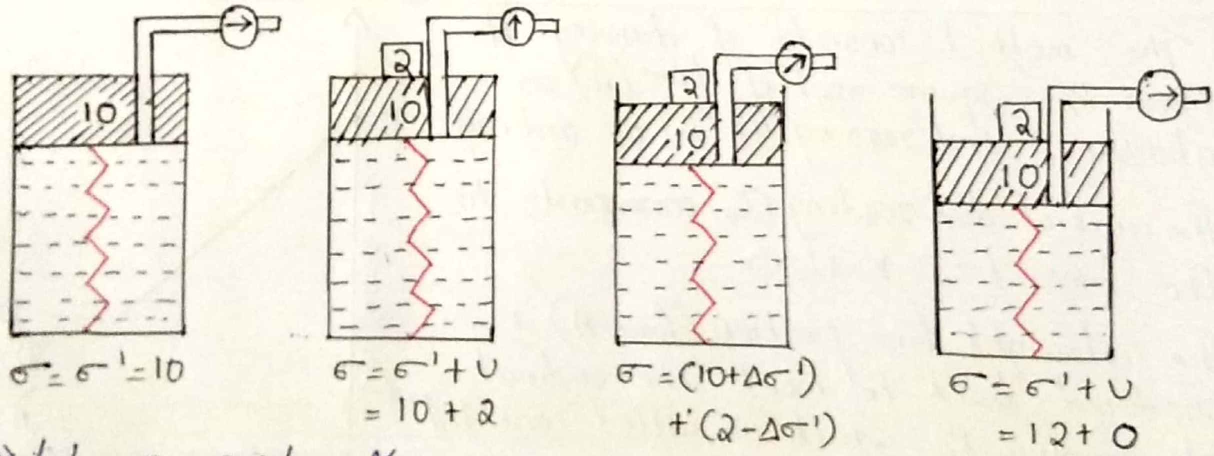
CASE 2:

$d_2 = H_2/2 = 6000mm/2 = 3000mm$

$T_{v_2} = \pi/4 (U_2/100)^2 = \pi/4 (40/100)^2 = 0.125$

$t_2 = \dfrac{T_{v_2} d_2^2}{C_v} = \dfrac{0.125 \times 3000^2 mm^2}{0.157 \ mm^2/min} = 7.2 \times 10^6 min = 5002 \ days.$

Q10. a.



1) Let a piston & spring be placed in a cylinder containing water. A valve is provided at bottom of piston.

2) If 10units of load is placed on piston, all th load is carried by spring & water is free of stress.

3) If extra 2 units of load is applied & valve is closed spring cannot deform since water is incompressible. Hence additional 2 units is entirely borne by water

$\therefore \sigma = \sigma' + u = 10 + 2 = 12.$

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4) Now, let valve be opened partially so that some water escapes & then valve is closed.
Due to this, piston moves down, spring is compressed & hence some pressure ($\Delta\sigma'$) out of 2 units is transferred to spring.

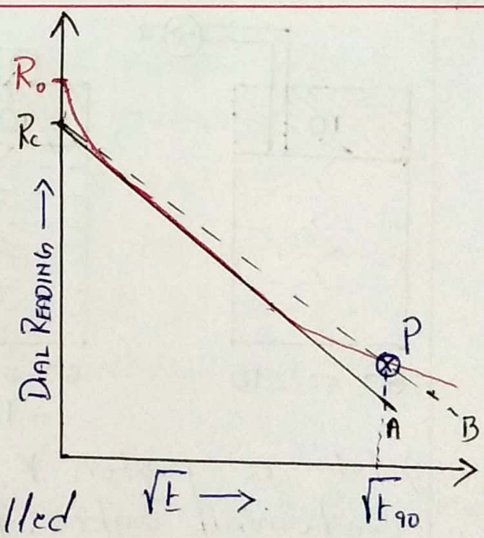
$$\therefore \sigma = (10 + \Delta\sigma') + (2 - \Delta\sigma')$$

5) If valve is opened fully, sufficient water will escape till spring length is reduced to 'z_i'. Thus whole of 2 units of pressure is transferred from water to the spring & water is free of pressure.
Entire pressure is borne by spring.

$$\sigma = \sigma' + u \Rightarrow 12 = 12 + 0$$

6) Similarly when extra load is added to soil, the pressure is first taken up by water & as water seeps out the pressure is transferred to soil grains.

Q10.b. The method consists of drawing the curve b/w square root of time ($\sqrt{t}$) as abscissa & dial reading as (R) as ordinate.



The initial dial reading (R_o) corresponds to the time $t=0$ & $U=0$.
The straight line portion (line A) is produced back to meet the ordinate at reading R_c which is called corrected zero reading.
The consolidation b/w R_o and R_c is called initial consolidation.
From R_c, another line 'B' is so drawn that its abscissa at every point is 1.15 times that of line A.
Intersection of line B with consolidation curve gives a point 'P' corresponding to 90% U whose dial reading & time may be denoted as R_{90} & t_{90} respectively.

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From the curve we get $\sqrt{t_{90}}$ & hence t_{90}.

Co-eff of consolidation is calculated from

$$C_v = \frac{(T_v)_{90} d^2}{t_{90}}$$

$T_{v_{90}}$ = time factor corr to 90% consolidation.

$$T_v = -0.9332 \log\left(1-\frac{U}{100}\right) - 0.0851 \quad \text{where } U = 90\%.$$

$T_{v_{90}} = 0.848$.

d = Drainage Path.

Q10.c. Given, $e_0 = 1.2$, $e = 1.1$, $\sigma_0' = 200 \, kN/m^2$

$\sigma' = 400 \, kN/m^2$ $k = 8 \times 10^{-7} \, mm/sec$.

$$a_v = \frac{e_0 - e}{\sigma' - \sigma_0'} = \frac{1.2 - 1.1}{400-200} = 5\times10^{-4} \, m^2/kN.$$

$$m_v = \frac{a_v}{1+e_0} = \frac{5\times10^{-4} \, m^2/kN}{1+1.2} = 2.27\times10^{-4} \, m^2/kN.$$

$$C_v = \frac{k}{m_v \gamma_w} = \frac{8\times10^{-10} \, m/sec}{2.27\times10^{-4} \, m^2/kN \times 9.81 \, kN/m^3} = 3.59\times10^{-7} \, m^2/sec.$$

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