



MUZAFFARPUR INSTITUTE OF TECHNOLOGY, MUZAFFARPUR
B.Tech 5th Semester Mid-Term Examination, 2018
Soil mechanics I (011X09)

Time: 2 hours

Full Marks: 20

Instructions: (i) Attempt any four questions. Attempt at least one question from group A and B.
(ii) Question No. 1 is compulsory.
(iii) All questions carry equal marks.

1. Chose the correct option of the following

(a) Which one is field test for the determination of the water content?

- i. Oven dry method
- ii. Torsion balance method
- iii. Pycnometer method
- iv. Radiation method

(b) Which test is not used for the determination of the field density of the soil?

- (i) Sand replacement method
- (ii) Core cutter method
- (iii) pycnometer
- (iv) Rubber balloon method

(c) Which of the following is not an engineering property of soil?

- (i) compressibility
- (ii) plasticity
- (iii) shear strength
- (iv) permeability

(d) There are some parameters used in soil mechanics like: (a). void ratio, (b). bulk density (c) relative denisty (d). mass specific gravity (e). Coefficient of permeability. Which of these parameters are dimensionless (unit less)?

- (i) a, b, c, d only
- (ii) a, c, d only
- (iii) a, c, d, e
- (iv) a and d only

(e) coefficient of permeability is inversely proportional to the which of the following parameter

- (i) void ratio
- (ii) diameter of the particle
- (iii) viscosity of the liquid
- (iv) None of these

2. What do you mean by permeability of the soil? Explain any laboratory method to find out the coefficient of the permeability.
3. Explain any method to find out the coefficient of the consolidation (C_v).
4. 1 m^3 wet soil weights 20 kN. Its dry weight is 18 kN. Specific gravity of soil solids is 2.67. Determine the water content, porosity, void ratio, and the degree of saturation. Draw a phase diagram.
5. Compare the compactive energy used in the IS heavy compaction test with that of the IS light compaction test.
6. Write the steps to draw the Newmark's influence chart for vertical stress.
7. What is quick sand condition? Explain the condition of the quick sand condition.

Solution

1.
 - a. iv
 - b. iii
 - c. ii
 - d. ii
 - e. iii
2. The permeability of a soil is a soil property which describes quantitatively, the ease with which water flows through that soil. It is a very important engineering property of a soil.
There are mainly two laboratory methods used to find out coefficient of permeability. Those are following:
 - a. Constant head permeability test
 - b. Variable head permeability test

Constant permeability test

Set up is shown in the figure below. The principle in this set-up is that the hydraulic head causing flow is maintained constant; the quantity of water flowing through a soil specimen of known cross-sectional area and length in a given time is measured. In highly impervious soils the quantity of water that can be collected will be small and, accurate measurements are difficult to make. Therefore, the constant head permeameter is mainly application cable to relatively pervious soils, although, theoretically speaking, it can be used for any type of soil. If the length of the specimen is large, the head lost over a chosen convenient length of the specimen may be obtained by inserting piezometers at the end of the specified length. If Q is the total quantity of water collected in the measuring jar after flowing through the soil in an elapsed time t , from Darcy's law,

$$q = Q/t = k.i.A$$

$$\therefore k = (Q/t).(1/iA) = (Q/t).(L/Ah) = QL/thA \dots(\text{Eq. 1})$$

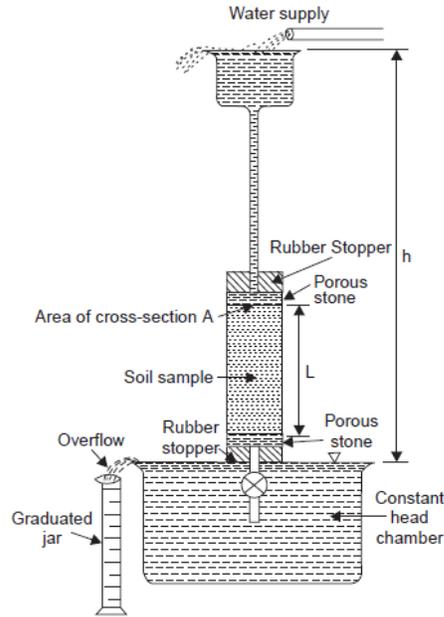
where

k = Darcy's coefficient of permeability

L and A = length and area of cross-section of soil specimen

h = hydraulic head causing flow.

The water should be collected only after a steady state of flow has been established.



Variable head permeability test

Set up is shown in the figure below. A better set-up in which the top of the standpipe is closed, with manometers and vacuum supply, may also be used to enhance the accuracy of the observations (Lambe and Whitman, 1969). The falling head permeameter is used for relatively less permeable soils where the discharge is small.

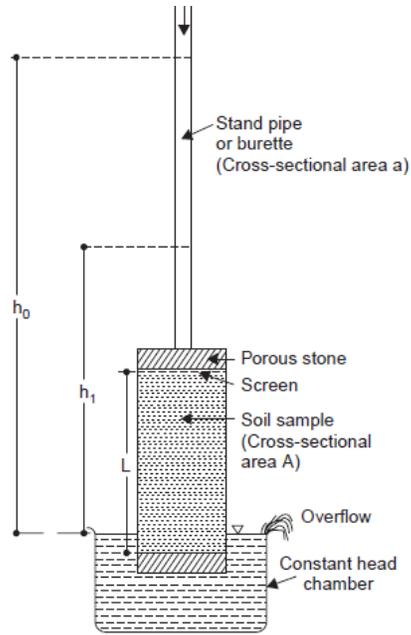
The water level in the stand-pipe falls continuously as water flows through the soil specimen. Observations should be taken after a steady state of flow has reached. If the head or height of water level in the standpipe above that in the constant head chamber falls from h_0 to h_1 , corresponding to elapsed times t_0 and t_1 , the coefficient of permeability, k , can be shown to be :

Here

$$k = \frac{2.303 aL}{A(t_1 - t_0)} \cdot \log_{10} (h_0/h_1)$$

a = area of cross-section of standpipe

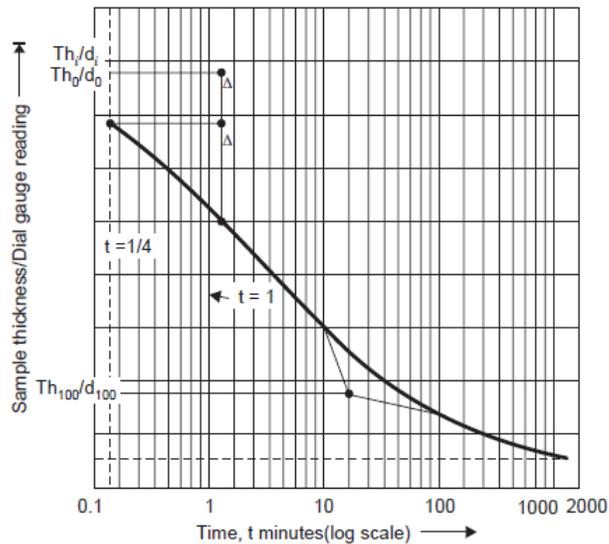
L and A = length and area of cross-section of the soil sample and the other quantities as defined.



3. Following two methods are used to find out the coefficient of consolidation
 - a. The square root time fitting methods
 - b. Logarithm of time fitting methods

Logarithm of time fitting method

This method was devised by A. Casagrande and R.E. Fadum (1939). The point corresponding to 100 per cent consolidation curve is plotted on a semi-logarithmic scale, with time factor on a logarithmic scale and degree of consolidation on arithmetic scale, the intersection of the tangent and asymptote is at the ordinate of 100% consolidation. A comparison of the theoretical and laboratory plots in this regard is shown in Figs



Since the early portion of the curve is known to approximate a parabola, the corrected zero point may be located as follows: The difference in ordinates between two points with times in the ratio of 4 to 1 is marked off; then a distance equal to this difference may be stepped off above the upper points to obtain the corrected zero point. This point may be checked by more trials, with different pairs of points on the curve. After the zero and 100% primary compression points are located, the point corresponding to 50% consolidation and its time may easily be obtained and the coefficient of consolidation computed from:

$$C_v = \frac{T_{50} H^2}{t_{50}}$$

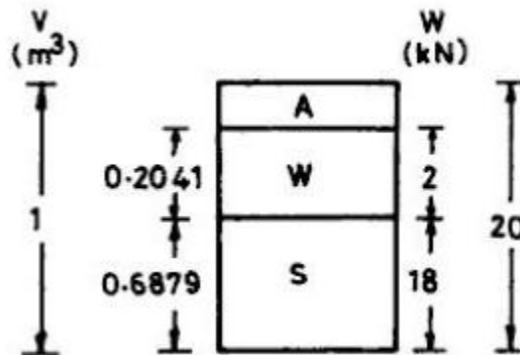
where t_{50} is read off from Fig. above

$T_{50} = 0.197$ from Terzaghi's theory, and

H is the drainage path as stated in the previous subsection.

The primary compression ratio may be obtained as given in the previous subsection.

4.



On the phase diagram, the volume of the soil (V) and its wet weight (W) and dry weight (W_s) which are given, are first shown.

The volumes and weights of the constituents are now calculated:

$$W_w = W - W_s = 20 - 18 = 2\text{kN}$$

$$V_s = \frac{W_s}{G_s \gamma_w} = \frac{18}{2.67 \times 9.8} = 0.6879 \text{ m}^3$$

$$V_w = \frac{W_w}{\gamma_w} = \frac{2}{9.8} = 0.2041 \text{ m}^3$$

$$V_v = V - V_s = 1 - 0.6879 = 0.3121 \text{ m}^3$$

Values of W_w , V_s , V_w and V_v are shown on the phase diagram. The required ratios can now be written:

$$\text{Water content, } w = \frac{W_w}{W_s} = \frac{2}{18} = 0.1111 \text{ or } 11.1 \%$$

$$\text{Porosity, } n = \frac{V_v}{V} = \frac{0.3121}{1} = 0.3121 \text{ or } 31.2 \%$$

$$\text{Void ratio, } e = \frac{V_v}{V_s} = \frac{0.3121}{0.6879} = 0.45$$

$$\text{Degree of saturation, } S = \frac{V_w}{V_v} = \frac{0.2041}{0.3121} = 0.6539 \text{ or } 65.4 \%$$

5.

Compactive energy in IS heavy compaction test

$$= \frac{4.9 \text{ (kgf)} \times 0.45 \text{ (m)} \times 5 \text{ (layers)} \times 25 \text{ (blows/layer)}}{10^3 \times 10^{-6} \text{ (m}^3\text{)}} \\ = 27562.5 \text{ kgf m/m}^3$$

Compactive energy in IS light compaction test

$$= \frac{2.6 \text{ (kgf)} \times 0.31 \text{ (m)} \times 3 \text{ (layers)} \times 25 \text{ (blows/layer)}}{10^3 \times 10^{-6} \text{ (m}^3\text{)}} \\ = 60450 \text{ kgf m/m}^3$$

IS heavy compaction test uses 4.56 times the compactive energy that is used in the IS light compaction test.

6.

It may not be possible to use Fadum's influence coefficients or chart for irregularly shaped loaded areas. Newmark (1942) devised a simple, graphical procedure for computing the vertical stress in the interior of a soil medium, loaded by uniformly distributed, vertical load at the surface. The chart devised by him for this purpose is called an 'Influence Chart'. This is applicable to a semi-infinite, homogeneous, isotropic and elastic soil mass (and not for a stratified soil).

The vertical stress underneath the centre of a uniformly loaded circular area has been shown to be:

$$\sigma_z = q \left[1 - \frac{1}{\{1 + (a/z)^2\}^{3/2}} \right] \quad \dots(\text{Eq. 10.36})$$

where a = radius of the loaded area, z = depth at which the vertical stress is required, and q = intensity of the uniform load. This equation may be rewritten in the form:

$$\frac{a}{z} = \sqrt{\left(1 - \frac{\sigma_z}{q}\right)^{-2/3} - 1} \quad \dots(\text{Eq. 10.53})$$

Here (a/z) may be interpreted as relative sizes or radii of circular-loaded areas required to cause particular values of the ratio of the vertical stress to the intensity of the uniform loading applied.

If a series of values is assigned for the ratio σ_z/q , such as 0, 0.1, 0.2, ..., 0.9, and 1.00, a corresponding set of values for the relative radii, a/z , may be obtained. If a particular depth is specified, then a series of concentric circles can be drawn. Since the first has a zero radius and the eleventh has infinite radius, in practice, only nine circles are drawn. Each ring or annular

space causes a stress of $q/10$ at a point beneath the centre at the specified depth z , since the number of annular spaces (c) is ten.

The relative radii may be tabulated as shown below:

Table 10.6 Relative radii for Newmark's influence chart

<i>S.No. of circle</i>	σ_z/q	<i>Relative radii a/z</i>	<i>Number of influence meshes per ring</i>
1	0.0	0.000	...
2	0.1	0.270	20
3	0.2	0.400	20
4	0.3	0.518	20
5	0.4	0.637	20
6	0.5	0.766	20
7	0.6	0.918	20
8	0.7	1.110	20
9	0.8	1.387	20
10	0.9	1.908	20
11	1.0	∞	...

From this table it can be seen that the widths of the annular slices or rings are greater the farther away they are from the centre. The circle for an influence of 1.0 has an infinitely large radius. Now let us assume that a set of equally spaced rays, say s in number, is drawn emanating from the centre of the circles, thus dividing each annular area into s sectors, and the total area into cs sectors. If the usual value of 20 is adopted for s , the total number of sectors in this case will be 10×20 or 200. Each sector will cause a vertical stress of 1/200th of the total value at the centre at the specified depth and is referred to as a 'mesh' or an 'influence unit'. The value 1/200 or 0.005 is said to be the 'influence value' (or 'influence factor') for the chart. Each mesh may thus be understood to represent an influence area.

The construction of Newmark's influence chart, as this is usually called, may be given somewhat as follows:

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For the specified depth z (say, 10 m), the radii of the circles, a , are calculated from the relative radii of Table 10.6 (2.70 m, 4.00 m, 5.18 m, ... and so on). The circles are then drawn to a convenient scale (say, 1 cm = 2m). A suitable number of uniformly spaced rays (say, 20) is drawn, emanating from the centre of the circles. The resulting diagram will appear as shown in Fig. 10.19; on it is drawn a vertical line ON , representing the depth z to the scale used in drawing the circles (if the scale used is 1 cm = 2 m, ON will be 5 cm). The influence value for

this chart will be $\frac{1}{10 \times 20}$ or 0.005. The diagram can be used for other values of the depth z by

simply assuming that the scale to which it is drawn alters; thus, if z is to be 5 m the line ON

now represents 5 m and the scale is therefore 1 cm = 1 m (similarly, if $z = 20$ m, the scale becomes 1 cm = 4 m).

The operation or use of the Newmark's chart is as follows:

The chart can be used for any uniformly loaded area of whatever shape that may be. First, the loaded area is drawn on a tracing paper, using the same scale to which the distance ON on the chart represents the specified depth; the point at which the vertical stress is desired is then placed over the centre of the circles on the chart. The number of influence units encompassed by or contained in the boundaries of the loaded area are counted, including fractional units, if any; let this total equivalent number be N . The stress σ_z at the specified depth at the specified point is then given by:

$$\sigma_z = I \cdot N \cdot q, \text{ where } I = \text{influence value of the chart.} \quad \dots(\text{Eq. 10.54})$$

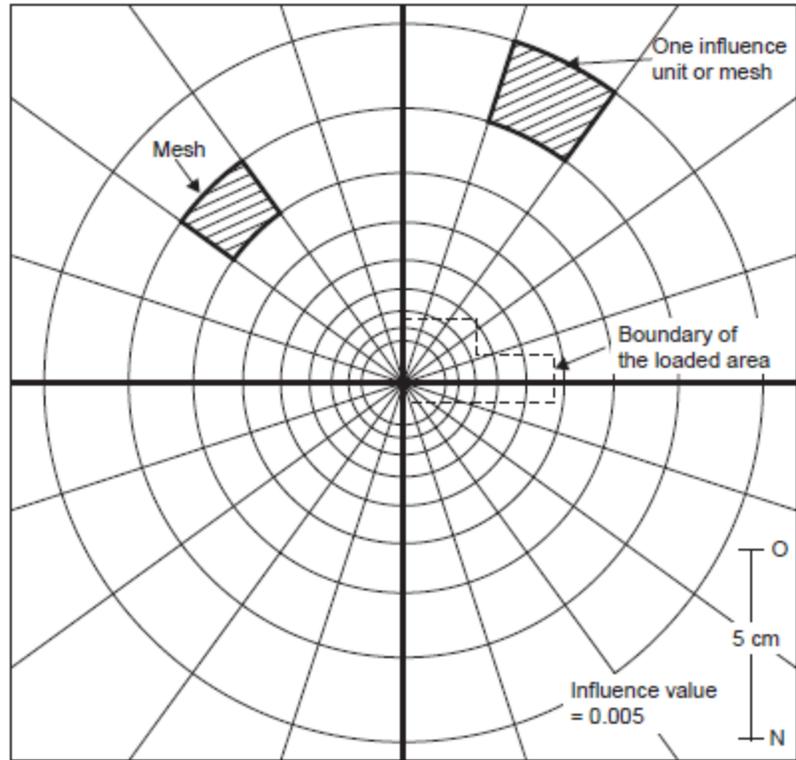
(Note: The stress may be found at any point which lies either inside or outside the loaded area with the aid of the chart).

Although it appears remarkably simple, Newmark's chart has also some inherent deficiencies:

1. Many loaded areas have to be drawn; alternatively, many influence charts have to be drawn.

2. For each different depth, counting of the influence meshes must be done. Considerable amount of guesswork may be required in estimating the influence units partially covered by the loaded area.

However, the primary advantage is that it can be used for loaded area of any shape and that it is relatively rapid. This makes it attractive.



7.

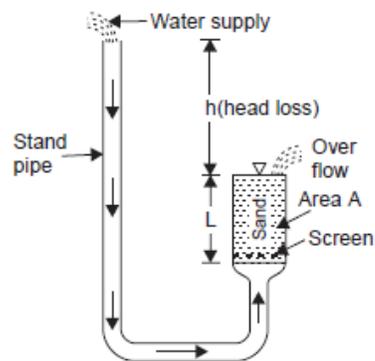


Fig. 6.26 Upward flow of water through soil

Total upward water force on the soil mass at the bottom surface

$$= (h + L)\gamma_w \cdot A$$

Total downward force at the bottom surface = Weight of the soil in the saturated condi-

tion

$$= \gamma_{\text{sat}} \cdot L \cdot A.$$

$$= \frac{(G + e)}{(1 + e)} \cdot \gamma_w \cdot L \cdot A.$$

Assuming that there is no friction at the sides, it is evident that the soil will be washed out if a sufficiently large value of h is applied. Such a boiling condition will become imminent if the upward water force just equals the weight of the material acting downward; that is,

$$(h + L) \gamma_w \cdot A = \frac{(G + e)}{(1 + e)} \cdot \gamma_w L \cdot A.$$

whence $i = h/L = (G - 1)/(1 + e)$

This means that an upward hydraulic gradient of magnitude $(G - 1)/(1 + e)$ will be just sufficient to start the phenomenon of “boiling” in sand. This gradient is commonly referred to as the “Critical hydraulic gradient”, i_c . Its value is approximately equal to unity. A saturated

sand becomes “Quick” or “Alive” at this gradient; this is only a conditions and not a type of sand.

According to Darcy’s law, the velocity at which water flows varies as the permeability, in order to maintain a specified hydraulic gradient such as unity. This explains the fact that quicksand conditions occur more commonly in fine sands with low permeability. In case of gravels with high permeability, much higher velocity of flow will be required to cause the “quicksand condition”.

Quicksand conditions are likely to occur in nature in a number of instances; however, the widespread belief that animals and man could be sucked into the quicksand is a myth, since the unit weight of the saturated sand is nearly double that of water. However, quicksand conditions present constructional difficulties. When the exit gradient for a hydraulic structure like a dam assumes the critical value, boiling occurs. This may lead to the phenomenon of progressive backward erosion in the form of a pipe or closed channel underneath the structure and ultimately failure of the structure. This is called, “piping”. The ratio of the critical gradient to the actual exit gradient is called the “factor of safety against piping”.