2.4 PERMEABILITY:

Definition:

Permeability is defined as the property of a porous material which permits the passage or seepage ofwater through its interconnecting voids.

A material having continuous voids is called permeable.

Gravel highly permeable

Stiff clay - least permeable (impermeable).

Laminar flow:

Each fluid particle travels along a definite path which never crosses the path of any other particle.

Turbulant flow:

The paths are irregular, twisting, crossing and recrossing at random.

Darcy's Law:

For laminar flow conditions in a saturated soil, the rate of flow (or) the discharge per unit time isproportional to the hydraulic gradient.

$$\boxed{q = kiA}$$

$$V = Ki = \frac{q}{A}$$

Where, q -discharge per unit time

A- Total c/s area of soil mass, perpendicular to the direction of flow.

i =Hydraulic gradient

k -Darcy's coefficient of permeability

V-Velocity of flow (or) average discharge velocity

If soil sample of length 'L' and c/s area 'A' subjected to differential head of water ($h_1 - h_2$), the hydraulic gradient 'i' will be equal to $\frac{h_1 - h_2}{L}$

$$q = \frac{K(h_1 - h_2)A}{L}$$

If 'i' is unity 'k' is equal to 'V'

Thus, coefficient of permeability is defined as the velocity of flow that will occur through the c/s area of soil under unit hydraulic gradient.

k = V = cm/sec (or) m/day (or) feet/day.

Discharge velocity and seepage velocity:

The velocity of flow 'v' is the rate of discharge of water per unit of total c/s area 'A' of soil.

$$A = A_s + A_v$$

Since, the flow takes through the voids, the actual (or) true velocity of flow will be more than the discharge velocity. This actual velocity is called the seepage velocity (V_s).

It is defined as the rate of discharge of percolating water per unit c/s area of voids perpendicular to the direction of flow.

$$q = VA = V_s A_v$$

$$V_S = \frac{VA}{A_V}$$

$$\frac{A_V}{A} = \frac{V_V}{V} = n$$

$$V_S = V.\frac{1}{n} = \frac{V}{n} = \frac{1+e}{e}.V$$

The seepage velocity V_s is also proportional to the hydraulic gradient.

$V_s = k_p\, i$ (k_p is the coefficient of percolation) From Darcy's law, $V = k\, i$

$$\frac{V_S}{V} = \frac{K_P}{K} = \frac{1}{n}$$

$$K_P = \frac{k}{n}$$

2.4.1PERMEABILITY MEASUREMENTS IN THE LABORATORY:

They are two methods

- Constant head permeability test
- Falling head permeability test

Field methods

- Pumping – out test
- Pumping - in test

Indirect methods of 'k' involving computations from

- Grain size
- Specific Surface
- Consolidation test data

1)Constant head permeability test

- Place the mould assembly in the bottom tank and fill the bottom tank with water up to its outlet.
- Connect the outlet tube of the constant head tank to the inlet nozzle of permeameter.
- Start the stopwatch, and at the same time put a beaker under the outlet of the bottom tank.
- Conduct the test for some convenient time interval. Measure the quantity of water collected inbeaker during that time.
- Repeat the test twice more, under the same head and for same time interval.

- This method is used for coarse grained soils.

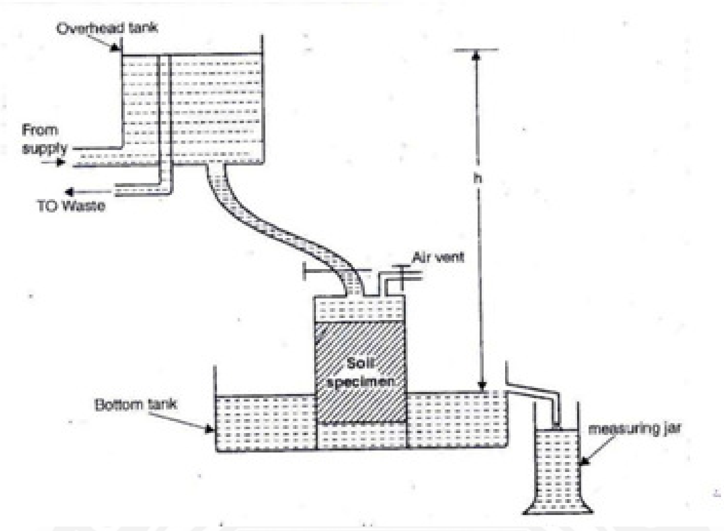


Fig 2.8 constant head diagram

Let ‘q’ be the discharge per unit time.

q=Q/t ----------------- (1)

From Darcy’s law,

q=kia---------------(2)

Equate 1 & 2,

Q / t = kiA

$$K = \frac{Q}{tia}$$

i = h / L ; k = Q / (t(h / L)A) = QL / thA

$$K = \frac{Q.L.1}{thA}$$

where,

k = coefficient of permeability (cm/s)

Q = Dischage (cm^2)

L = Length of the sample (cm)

A = Total c/s area of soil specimen (cm^2)

h = Head of water (cm)

t = Time taken (sec)

2)Falling head (or) Variable permeability test:

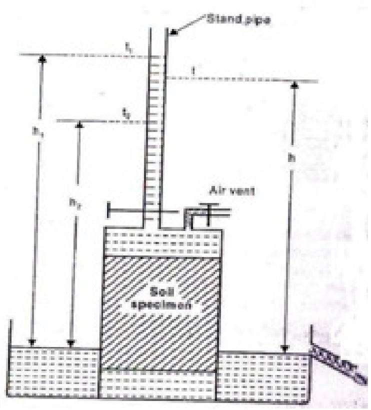


Fig 2.9 constant head diagram

- Prepare the soil specimen in the permeameter, keep the mould assembly in the bottom tank and fill the bottom tank with water.
- Connect the inlet nozzle of the mould to the stand pipe filled with water. Permit water to flow for some time till steady state of flow isreached.
- With the help of stop watch, note the time interval required for water level in the stand pipe to fall from some convenient initial value to some final value.
- Repeat the above step atleast twice and note the time interval and also note the diameter of standpipe, from which the area of stand pipe calculated.

This method is used for grained soils.From\

Darcy's law,

$$q = k\,i\,A \longrightarrow \quad 1$$

q = Q / t = Area x Rate of Velocity

$$q = -a\, dh / dt \longrightarrow \quad 2$$

' - 'sign indicates falling head height

decreases, time increases.Equate 1 & 2,

$$-a\,(dh / dt) = k\,i\,A$$

$$-a\,(dh / dt) = k\,h\,A / L$$

$$-(dh / dt) = k\,A\,h / a\,L$$

$$-\ (dh / h) = k\,A\,dt / a\,L$$

Integrating from 'h_1' to 'h_2'

and '0' to 't'.

$$-\int_{h_1}^{h_2} \frac{dh}{dt} = KA \int_0^t h$$

$$\left[\log_e h\right]_{h_1}^{h_2} = \frac{-kA}{aL}\left[t\right]_0^t$$

$\log_e h_2 - \log_e h_1 = -kAt / aL$

$\log_e h_1 - \log_e h_2 = kAt / aLk$

$= aL \log_e (h_1 / h_2)$

$$\boxed{k = \frac{2.303\, aL}{At} \log_{10} (h_1 / h_2)}$$

k = co-efficient of permeability (cm / s)

a = area of stand pipe (cm^2)

L=Length of specimen (cm)

A=Total c/s area of soil sample (cm^2)

h_1= initial head of water (cm)

h_2= final head of water (cm)

2)FIELD METHODS:

They are more reliable compared to laboratory methods for the determination of 'k'. Lab tests involves large man of soil with minimum disturbance unlike the small sample is used. The value of 'k' obtained from field tests represents an average value of 'k' for the large soil mass over a large area.

a) Pumping – out tests:

Aquifer: It is a permeable formation which allows a significant quantity of water to move through itunder field conditions.

Confined aquifers (or) Artesian aquifers: It is one in which ground water remains entrapped underpressure greater than atmospheric, by overlying relatively impermeable strata.

Unconfined Aquifers: It is one in which the ground water table is the upper surface of the zone ofsaturation and it lies within the test stratum. It is also called 'free', 'phreatic' or 'non – artesian aquifers'.

When a well is penetrated into an extensive homogeneous aquifer, the water table initially remains horizontal in the well. When the well is pumped, water is removed from the aquifer and the water table or the piezometric surface, depending upon the type of aquifer, is lowered resulting in a parabolic depression in the water table (or) piezometric surface. This depression is called the cone of depression or the drawdown curve.

In the pumping – out tests, draw downs corresponding to a steady discharge 'q', are observed at anumber of observation wells.

Pumping must continue at a uniform rate for an adequate time to establish a steady state condition,in which the draw down changes negligibly with time.

Assumptions:

1) The aquifer is homogeneous with uniform permeability and is of infinite aerial extent.
2) The flow is laminar and Darcy's law is valid.
3) The flow is horizontal and uniform at all points in the vertical section.
4) The well penetrates the entire thickness of the aquifer (and receives water).
5) Natural ground water regime affecting the aquifer remains constant with time.

6) The velocity of flow is proportional to the tangent of the hydraulic gradient (Dupuit's theory).

1)Unconfined Aquifer:

Fig., shows a well penetrating an unconfined (or) free aquifer to its full depth of an pumping – out

test.

Let r = radius of main well.

R = radius of zero drawdown known as max., radius of influence.

h = depth of water in the main well during pumping, measured above impervious layer. H = height of initial water table above impervious layer.

q = rate at which water is pumped out of well.

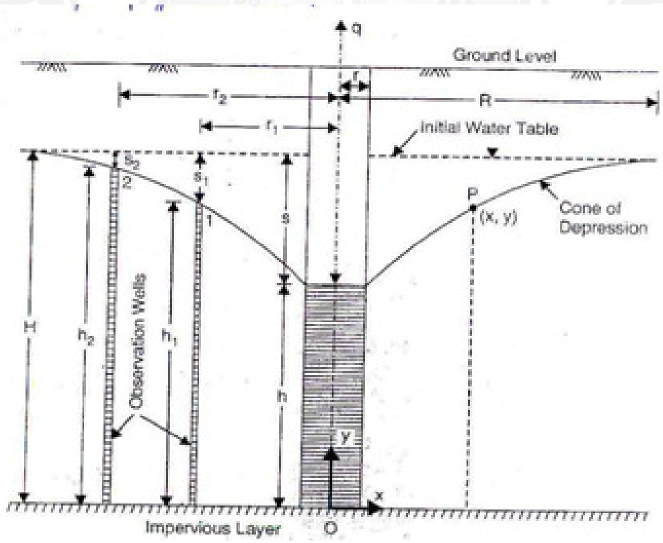


Fig 2.10 UnConfined aquifer diagram

Let 'P'(x , y) be any point on the draw down curve. The point 'o' (orgin coordinates) at the bottomof central axis of well is chosen as the orgin of reference.

Applying Darcy's law, for flow through cylindrical surface of radius 'x' and height 'y' we have,Discharge, $q = k \cdot A_x \cdot i_x$

$= k \cdot (2\pi xy)\, dy / dx$

$= q \,.\, dx / x$

$= 2\pi ky\, dy$

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

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

$$q \left[\log_e\left(\frac{1}{x}\right)\right]_r^R = 2\pi k \left[\frac{y^2}{2}\right]_h^H$$

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

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

(or)

$$\boxed{q = \frac{1.36\, k\,(H^2 - h^2)}{\log_{10} (R / r)}}$$

$$k = \frac{2.303q}{2\pi b(h_2 - h_1)} \log_{10}\left(\frac{r_2}{r_1}\right)$$

In above equation R is found to vary from 150 m to 300 m and can only be estimated crudely, as forexample, using the following equation given by Sichardt,

$$\boxed{R = 3000\, S\sqrt{k}}$$

$R = S\sqrt{k}$

K = m/sec R & S in 'm' S = H – h

To avoid the use of R, an alternative method is to measure drawdowns s_1 and

s_2 in two observations wells located at radial distances r_1 and r_2 from the axis of main well. The depths of water in the two observation wells are,

$h_1 = H - S_1$

$h_2 = H - S_2$

We now have,

$Y=h_1$,at $X=r_1$

$Y=h_2$,at $X=r_2$

$$q\int_{r_1}^{r_2}\frac{dx}{x} = 2\pi k\int_{h_1}^{h_2} y\,dy$$

$$q\left[log\ e(\frac{1}{x})\right]_{r_1}^{r_2} = 2\pi k\left[\frac{y^2}{2}\right]_{h_1}^{h_2}$$

$$K == \frac{q}{\pi(h_2^2 - h_1^2)}\log\left(\frac{r_2}{r_1}\right)$$

2)CONFINED AQUIFER:

Fig., shows a well fully penetrating a confined (or) artesian aquifer. Let (x,y) be the coordinates ofany point 'p' on the drawdown curve, measured with respect to the origin 'o'.

q = discharge or rate at which water is pumped out of main well.

b = thickness of confined aquifer.

Applying Darcy's law for flow through cylindrical surface of radius 'x' and height 'b'.

We have,$q = k\ i_x\ A_x$

where, A_x = c/s area of flow , measured at 'p' = 2 Л

x bi_x = hydraulic gradient at 'p' = dy / dx

q = k (dy / dx) (2 Л x b) (or) q (dx / x) = 2 Л x b dy

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

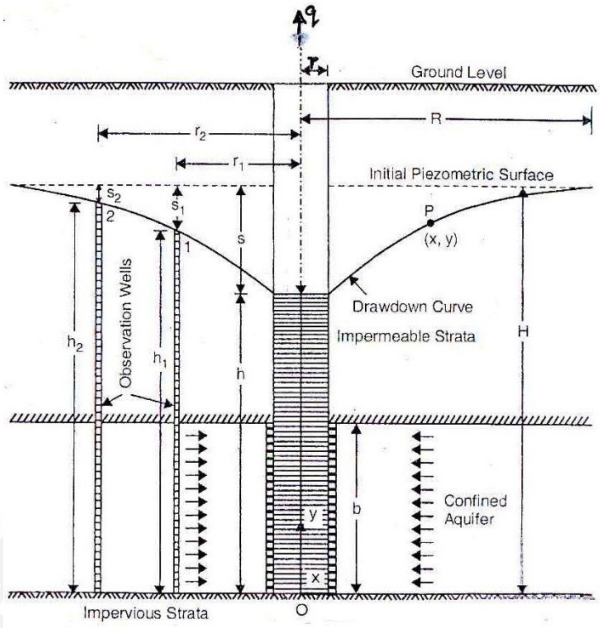


Fig 2.11 Confined aquifer diagram

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

$$q log_e \left(\frac{1}{x}\right)_r^R = 2\pi kb [y]_h^H$$

From which,

q $\log_e$ (R / r) = 2Лkb (H – h)

$$K = \frac{q}{2\pi b(H-h)} \log\left(\frac{R}{r}\right)$$

(or)

$$q = \frac{2.72T_s}{lod_{10}(\frac{R}{r})}$$

T = co-efficient of transmissibility = bk

S = drawdown at the well

Alternatively, if h_1 and h_2 are the depths of water measured above bottom impervious stratum in twoobservation wells located at radial distances r_1 and r_2 from the axis of main well, then we can write,

$$q\int_{r_1}^{r_2}\frac{dx}{x} = 2\pi Kb\int_{h_1}^{h_2} dy$$

q $\log_e$ (r_2 / r_1) = 2Лkb (h_2 – h_1)

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

(or)

$$q = \frac{2.72\,T(h_2 - h_1)}{log_{10}\left(\frac{r_2}{r_1}\right)}$$

2)PUMPING – IN TESTS:

The two methods desired by U.S. Bureau of Reclamation are,

i) Constant water level method (in open – end pipe) and

ii) Packer method (in section of borehole).

1) Constant water level method :

An open end pipe is sunk into the soil to desired depth and the soil is taken out of the pipe till its bottom end. The test is also conducted in a borehole with the pipe casing extending to the desired depth.Fig., illustrates the arrangement for the method.

In fig.,(a) and (c) the bottom end of pipe is above water table and Fig., (b) and (d) it is below water table. Water is pumped into the pipe and the rate of flow, q is adjusted to maintain water level constant in the pipe. In the case of soils of low permeability additional pressure head 'H_p' is required to be added to the gravity head 'H_g' in order to maintain constant rate of flow. The co-efficient of permeability is computed using the following equation.

$$\boxed{K = q / 5.5\, r\, H}$$

Where,

r = internal radius of pipeq = constant rate of flow

h = differential head of water (gravity plus pressure, if any)

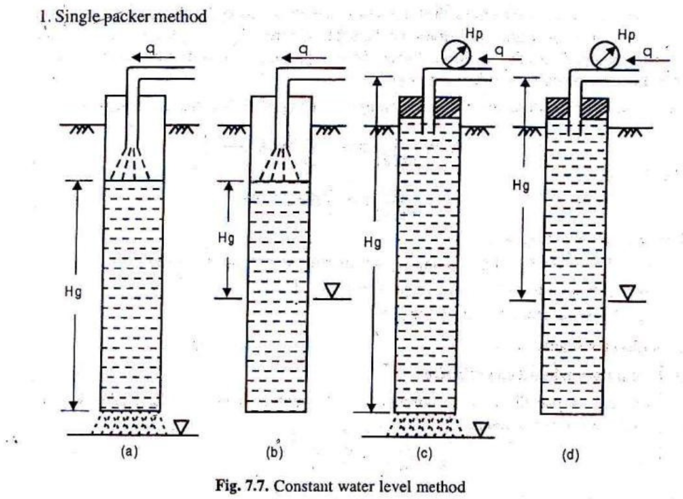


Fig 2.12 Constant water level method

2) **Packer method:**

A packer is an expandable cylindrical rubber sheeve packers are used as a means of sealing of a section of borehole. Two types of packer methods are used in practice.

i) **Single packer method:**

In single packer methods the hole is drilled to the required depth. The packer is fixed at a desired level above the bottom of the hole and the water pumped into the section below the packer. The constant rate of flow 'q' ie., attained under an applied head 'H' is found.

ii) **Double packer method:**

In double packer method, the hole is drilled to the final depth and cleaned. Two packers are fixed at a distance apart equal to 5 times the diameter of bore hole. Both packers are then expanded and water pumped into the section between the two packers. The constant rate of flow, q that is attained under an applied head, H is found

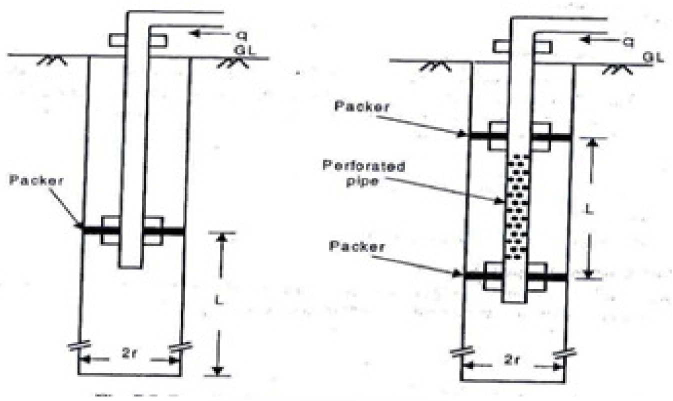


Fig 2.13 Double packer method

The co-efficient of permeability, k is computed using the following.

For, $r \leq L \leq 10\ r$

$$K = \frac{q}{2\pi LH} \sinh^{-1}\left(\frac{L}{2r}\right)$$

For, $L \geq 10\ r$

$$K = \frac{q}{2\pi LH} \log_{10}\left(\frac{L}{r}\right)$$

Where,

L = length of portion of the hole teste

dr = radius of bore hole

q = constant rate of flow into the test section

H = differential head for maintaining a constant rate of flow in test section

2.4.2PERMEABILITY OF STRATIFIED SOIL DEPOSITS:

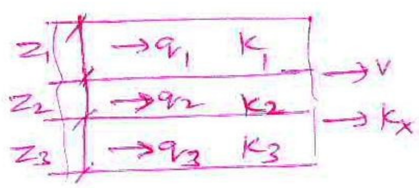
In nature soil mass may consists of so many layers deposited one above the other. Each layer is assumed to be homogeneous and isotropic. The average permeability of whole deposit will depend on the direction of flow.

Average permeability parallel to the direction of flow:

$q = q_1 + q_2 + q_3 i = i_1 = i_2 = i_3$

$kiA = k_1 i_1 A_1 + k_2 i_2 A_2 + k_3 i_3 A_3$

$A_1 = z_1$ x i, $A_2 = z_2$ x i, $A_3 = z_3$ x i, i is equal



$$K_x = \frac{K_1 Z_1 + K_2 2 + \cdots .. K_n n}{Z}$$

Average permeability perpendicular to the direction of flow:

For vertical flow, v & q is equal.

$\therefore$ i = v/k_1 , [v = ki]v = ki = kh / L

h = vL / k (or) vz / k

$$\frac{vz}{K_v} = \frac{vz_1}{K_1} + \frac{vz_2}{K_2} + \cdots . + \frac{vz_n}{K_n}$$

$$K_V = \frac{Z}{\frac{Z_1}{K_1} + \frac{Z_2}{K_2} + \cdots + \frac{Z_n}{K_n}}$$

2.4.3 Factors affecting permeability:

By comparing poiseuille's law with Darcy's law adopted for the flow through the soil pores, we get,q = k i A

$$K = D_s^2 \left(\frac{\gamma_w}{\eta}\right)\left(\frac{e^3}{1+e}\right).C \longrightarrow 1$$

Thus, the **factors affecting permeability** are,

1) Grain size
2) Properties of the pore fluid
3) Void ratio of the soil
4) Structural arrangement of the soil particles
5) Entrapped air and foreign matter
6) Adsorbed water in claying soils

1) Effect of size and shape of particles:

Permeability varies approximately as the square of the grain size. Since, soil consists of manydifferent sized grains. Some specific grain size has to be used for comparison.

a) **Allen Hazen (1892)** found in filter sands of particle size between 0.1 and 3mm.

$$\boxed{k = CD_{10}^{\ 2}}$$

K = coefficient of permeability (cm/sec)

D_{10} = Effective diameter (cm)

C = Constant = 100 (approx.,) When D_{10} in 'cm'.

b) Attempt have been made to correlate the 'k' and specific surface of the soil particles by Kozency(1907).

$$K = \frac{1}{K_K \eta S_s^2} x \frac{n^3}{1 - n^2}$$

n = porosity

s_s = Specific surface of particles (cm^2/cm^3)η = viscosity (g sec/cm^2)

k_k = constant = 5 for spherical particles

c) Loudon (1952 - 53) developed from the basis of his experiment is,

$$\boxed{\text{Log } 10(ks_s^2) = a + bn}$$

a = 1.365; b = 5.150 for '1c' at 10^0c.

2) Effect of properties of pore fluid:

From equation 1, the 'k' is directly proportional to 'γ_w' and inversely proportional to its viscosity. 'γ_w' does not change with change in temperature. Other factors remain constant.

Effect of property of water on 'k'

For a standard temperature at 27^0c

$$k_{27} = \frac{K\ \eta}{\eta_{27}}$$

k_{27} = permeability at 27^0c

η_{27} = viscosity at 27^0c

Change of 'γ_w'also taken into account.

$$\frac{K_1}{K_2} = \frac{\eta_2 \gamma_{w1}}{\eta_1 \gamma_{w2}} = \frac{\eta_2 \rho_{w1}}{\eta_1 \rho_{w2}}$$

Muscat (1937) is pointed out that in more general co-efficient of

$$k_p = \frac{K\ \eta}{\gamma_w}$$

permeability called physicalpermeability 'k_p' related to Darcy's co-efficient of permeability 'k'.

3) Effect of void ratio:

Void ratio is the opening space between the particles. If the 'e' is larger means, the water is easily flow inside their by the value of 'k' is more. k is directly proportional to the void ratio.

$$\frac{k_1}{k_2} = \left(\frac{e_1}{e_2}\right)^2 = \left(\frac{e_1^3}{}\right) \Big/ \left(\frac{e_2^3}{1+e_2}\right)$$

From the relation permeability varies as a square of void ratio.k_1 = permeability at void ratio 'e_1'

k_2 = permeability at void ratio 'e_2'

4) Structural Arrangements:

The structure of soil becomes changed depends on the method of compaction. Due to compaction particles come closer compact mass, there by pore size get reduced. So the value of 'k' get reduced. If effect of compaction is more means, the 'k' is less.

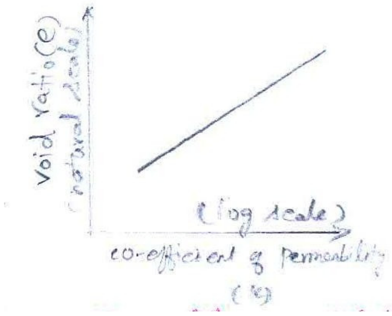


Fig2.14 PLOT OF 'e' against 'log k'

Example:

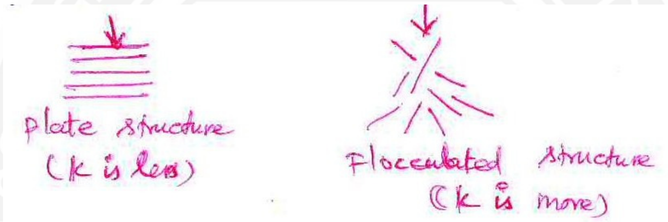


Fig 2.15 Plate structure Flocculated structure

(k is less) (k is more)

Fine grained soils ⟶ more

$\boxed{K_H > K_V}$

1) Effect of degree of saturation:

When the degree of saturation increases, the 'k' is increased. At a lower percentage of degree ofsaturation, the 'k' is increased.

Partly saturated soils ⟶ reduces 'k'

Fully saturated soils ⟶ increases 'k'.

2) Effect of presence of foreign matter:

Presence of foreign matter such as dust, reduces the value of 'k'. Because it sometimes closes the open space between the particles, there by the movement of water get reduced.

5) Effect of adsorbed water:

The adsorbed water, which is held by soil particles, is not free to move and therefore reduces theeffective pore space available for passage of water.

PROBLEMS

1) Calculate the coefficient of permeability of a soil sample, 6cm in height and $50cm^2$ in c/s area, if a quantity of water equal to 430ml passes down in 10 minutes, under an effective constant head of 40cm. On over drying, the test specimen has mass of 498g. Taking the specific gravity of soil solids as 2.65, calculate the seepage velocity of water during the test.

Given:

Q = 430ml; t = 10 x 60 = 600 seconds; A = $50cm^2$; L = 6cm; h = 40cm

Solution:

We know, co-efficient of permeability K = (Q/t) . (L/h) . (1/A)

= (430/600) . (6/40) . (1/50)

K = 2.15 x 10^{-3} cm/ sec x 864

K = 1.86 m/day [1cm/sec = 864 m/day]

Now, velocity, v = ki = 2.15 x 10^{-3} (40 / 6)

V=1.435X10^{-2} cm/sec or

v = (q/A) = (430 / (50))

= 1.435 x 10^{-2}cm/ sec

Now, p_d = (M_d / v) = (498/(50 x 6)) = 1.66 g/cm^3

$$e = \frac{G\rho_w}{\rho_d} - 1 = \frac{2.65\,x1}{1.66} - 1 = 0.595$$

$$n = \frac{e}{1+e} = \frac{0.595}{1.595} = 0.373$$

Seepage velocity, v_s = v = 1.435 x 10^{-2}

n 0.373

$v_s = 3.85 \times 10^{-2}$cm/sec

2) In a falling head permeameter test, the initial head (t=0) is 40cm. The head drops by 5cm in 10 minutes. Calculate the time required to run the test for the final head to be at 20cm. If the sample is 6cm in height and 50cm^2 in c/s area, calculate the co-efficient of permeability, taking area of stand pipe=0.5cm^2.

Given:

Falling head permeability test

h_1=40cm; t_1=0

Time, t=10min x 60 = 600 seconds. 5cm drop., so h_2= 40-5

$\boxed{h_2=35cm}$

L=6cm; a=0.5cm^2; A=50 cm^2

Solution:

Co-efficient of permeability,

$$\boxed{K=2.303\left(\frac{aL}{At}\right)\log_{10}\left(\frac{h_1}{h_2}\right)}$$

$$t = 2.3\left(\frac{aL}{AK}\right)\log_{10}\left(\frac{h_1}{h_2}\right)$$

$$t = m\log_{10}\left(\frac{h_1}{h_2}\right) \qquad m = \frac{2.3\ aL}{AK}$$

If h_1 = 40cm; h_2 = 35cm; t = 600sec600

$= m \log_{10} (40/35)$

$$\boxed{\therefore m = 10.363 \times 10^3}$$

If h1=40cm; h2=20cm

$t = 10.363 \times 10^3 \log_{10}[40/20]$

$= 3.12 \times 10^3$ sec/60

$\boxed{t = 52 \text{ minutes}}$

Now, $m = 2.3 \left(\frac{aL}{AK}\right)$

$$10.363 \times 10^3 = \frac{0.5 \times 6}{50 \times K}$$

$k = 1.33 \times 10^{-5}$cm/sec

3)A stratified soil deposit is shown in figure along in the co-efficient of permeability of the individual strata. Determine the ratio of k_H and k_v. Assume an average hydraulic gradient of 0.3 in both horizontal and vertical seepage. Find discharge and discharge velocity for each layer, for horizontal flow and hydraulic gradient and loss in head in each layer for vertical flow.

Given data:

$i = 0.3$

$k_v = ?$

$k_H = ?$

$k_H / k_v = ?$

$v_1 = v_2 = v_3 = ?$

$i_1 = i_2 = i_3 = ?$

$h_1 = h_2 = h_3 = ?$

Solution:

We know, for horizontal seepage, $k_H = \frac{k_1 z_1 + k_2 z_2 + k_3 z_3}{z_1 + z_2 + z_3}$

$$= \frac{5 \times 10^{-4} (200) + 5 \times 10^{-4} (500) + 5 \times 10^{-4} (200)}{200 + 500 + 200}$$

$$k_H = 5 \times 10^{-4} \text{ cm/sec}$$

for vertical seepage, $k_v = \dfrac{z_1 + z_2 + z_3}{\dfrac{z_1}{k_1} + \dfrac{z_2}{k_2} + \dfrac{z_3}{k_3}}$

$$= \frac{900}{\frac{200}{5X10^{-4}} + \frac{500}{5X10^{-4}} + \frac{200}{5X10^{-4}}}$$

$$K_V = 5X10^{-4} \text{ cm/Sec}$$

$$\frac{K_H}{K_V} = \frac{5X10^{-4}}{5X10^{-4}} = 1$$

i) Discharge velocity (V): (for horizontal flow)

Let q_1, q_2, q_3 be the discharge through the layer 1,2,3. V_1, V_2, V_3 be the velocity through the layer 1,2,3.

We know, $\boxed{q = kiA}$

For layer 1, $q_1 = k_1 i_1 A_1$

$= 5x10^{-4} \text{ x } 0.3 \text{ x } (200x1)$

$q_1 = 0.03 \text{ cm}^3/\text{sec}$

Also,

$q_2 = k_2 i A_2 = 5x10^{-4} \text{ x } 0.3 \text{ x } (500x1) = 0.075 \text{ cm}^3/\text{sec}$

$q_3 = k_3 i A_3 = 5x10^{-4} \text{ x } 0.3 \text{ x } (200x1) = 0.03 \text{ cm}^3/\text{sec}$

$V_1 = k_1 i = 5x10^{-4} \text{ x} 0.3 = 1.5x10^{-4} \text{ cm/sec}$

$V_2 = k_2 i = 5x10^{-4} \text{ x } 0.3 = 1.5x10^{-4} \text{ cm/sec}$

$V_3 = k_3 i = 5x10^{-4} \text{ x} 0.3 = 1.5x10^{-4} \text{ cm/sec}$

ii) Hydraulic gradient and loss in head in vertical flow:

We know, $\boxed{V = Ki}$

Hydraulic gradient ,

$$i_1 = \frac{V_1}{K_1} = \frac{1.5X10^{-4}}{5X10^{-4}} = 0.3$$
$$i_2 = \frac{V_2}{K_2} = \frac{1.5X10^{-4}}{5X10^{-4}} = 0.3$$
$$i_3 = \frac{V_3}{K_3} = \frac{1.5X10^{-4}}{5X10^{-4}} = 0.3$$

Loss of head

$\boxed{h = iz}$

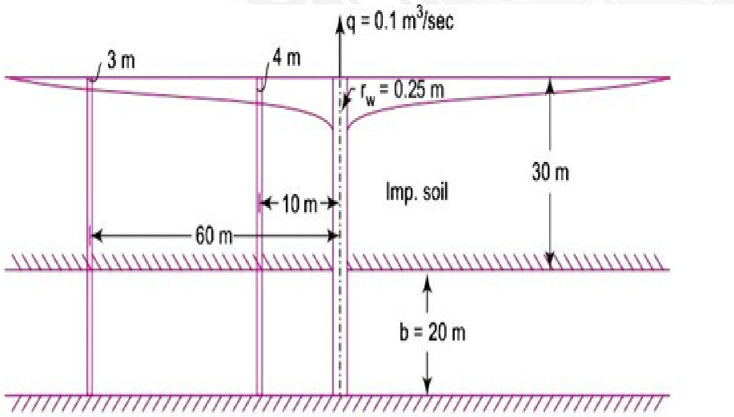
$\therefore$ i = h / (L.z) [L = 1m]

$h_1 = i_1 x z_1 = 0.3 \times 2 = 0.6m$

$h_2 = i_2 x z_2 = 0.3 \times 5 = 1.5m$

$h_3 = i_3 x z_3 = 0.3 \times 2 = 0.6m$

4).An aquifer of 20m average thickness is overlain by an impermeable layer of 30m thickness .A test well of 0.5m diameter and two observation wells at distances of 10m and 60m from the test well are drilled through the aquifer. After pumping at a rate of 0.1m3/sec for a long time, the following draw downs is stabilized in these wells: First observation well 4m; second observation well, 3m .Show the arrangement in a diagram. Determine the coefficient of permeability and draw down in the test well.



Given:

h_1=50-4=46m

h_2=50-3=47m

dia D=0.5m

q=0.1m^3/s

b=20m

$$k = \frac{2.303\, q}{2\pi b(h_2 - h_1)} \log_{10}(\frac{r_2}{r_1})$$

$$k = \frac{2.303\ x0.1}{2\pi.\ 20(47-46)}\log_{10}(\frac{60}{10})$$

$=1.43x10^{-3}$ m/sec

Draw down:

$$(h_2 - h_w) = \frac{2.303\ q}{2\pi bk}\log_{10}(\frac{r_2}{r_1})$$

$$(47 - h_w) = \frac{2.303\ x0.1}{2\pi x20x1.43\text{x}10^{-3}}\log_{10}(\frac{60}{10})$$

h_w=43.94 m

Draw down , S=50- h_w

=50-43.94=6.06m