Seepage Pressure:

By virtue of the viscous friction exerted on water flowing through soil pores, an energy transfer is effected between the water and the soil. The force corresponding to this energy transfer is called the seepage force (or) seepage pressure. Thus, seepage pressure is the pressure exerted by water on soil through which it percolates.

Seepage pressure is responsible for the phenomenon known as quick sand, and is of vital importance in the stability analysis of earth structures subjected to seepage action.

If 'h' is the hydraulic head or head lost due to frictional drag of water flowing through soil mass of thickness'z'.

∴ seepage pressure, $p_s = h\,\gamma_w$

(or) $$p_s = \frac{h}{z}\, z\,\gamma_w = i\ z\,\gamma_w$$

z = head over which the head is lost.

Seepage force, $\boxed{J = p_s \,.\, A = i\ z\,\gamma_w}$

Seepage force per unit volume $j = \dfrac{i\, z\, \gamma_w\, A}{z\, A}$

$$\boxed{j = i\,\gamma_w}$$

The vertical effective pressure may be decreased or increased due to seepage pressure depending on the direction of flow.

Effective pressure, $\sigma' = z\,\gamma' \pm p_s = \boxed{z\,\gamma' \pm iz\,\gamma_w}$

↓ Flow ⟶ σ' ⟶ increased ⟶ use "+ve".

↑ Flow ⟶ σ' ⟶ decreased ⟶ use "-ve".

UPWARD FLOW: QUICK SAND CONDITION:

When flow takes place in an upward direction, the seepage pressure also acts in the upward direction and the effective pressure is reduced.

Flow ↑ ⟶ p_s ↑ ⟶ σ′ ⟶ decreased

Flow ↓ ⟶ p_s ↓ ⟶ σ′ ⟶ increased

If $\underline{p_s = \gamma_{sat}}$ $\therefore \underline{\sigma' = 0}$

In such cases, a cohesion less soil loses all its shear strength and the soil particles have a tendency to move up in the direction of flow. This phenomenon of lifting of soil particles is called, **<u>quick condition, Boiling condition, (or) quick sand.</u>**

$\sigma' = z\,\gamma' - p_s = 0$

(or) $p_s = z\,\gamma'$ $\quad$ $i\,z\,\gamma_w = z\,\gamma'$

From which, $i = i_c = \frac{\gamma'}{\gamma'} = \frac{G-1}{1+e}\left(\frac{\gamma_{sat}-\gamma_w}{\gamma_w}\right)$

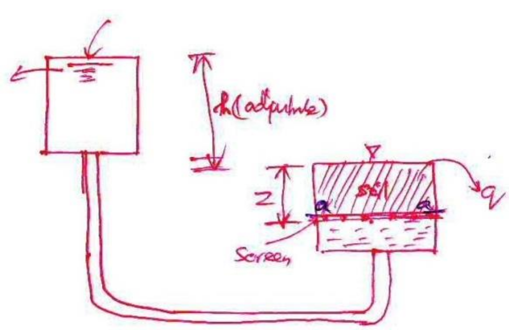
The hydraulic gradient at such a critical state is called the <u>critical hydraulic gradient</u>.

For loose sand (or) silt, e = 0.67 and G = 2.67 ie., $\boxed{i_c = 1}$

<u>Quick sand condition:</u>

It should be noted that quick sand is not a type of sand but a flow condition occurring within a cohesion less when effective pressure is reduced to zero due to upward flow of water.

Fig., shows setup of phenomenon of quick sand of thickness z, and hydraulic head, h.



Quick sand condition

When the soil particles are in the state of critical equilibrium,

The total upward force at the bottom of soil (aa) (↑) = Total weight of all the materials above the surfaceEquating them, at the level a – a,

$$(h + z)\,\gamma_w A = z\,\gamma_{sat} A$$

$$h\,\gamma_w = z\,(\gamma_{sat} - \gamma_w) = z\,\gamma'$$

$$\frac{h}{z} = i_c = \frac{\gamma'}{\gamma_w} = \boxed{\frac{G-1}{1+e}}$$

TWO DIMENSIONAL FLOW: LAPLACE EQUATION

The quantity of water flowing through a saturated soil mass, as well as the distribution of water pressure can be estimated by the theory of fluids through porous medium.

For computing these quantities with the help of theoretical analysis the following assumptions are made;

1) The saturated porous medium is incompressible. The size of the pore spaces does not change with time, regardless of water pressure.

2) The seepage water flows under 'I' (hydraulic gradient) which is due to only to gravity head loss (or) Darcy's law for flow through porous medium is valid.

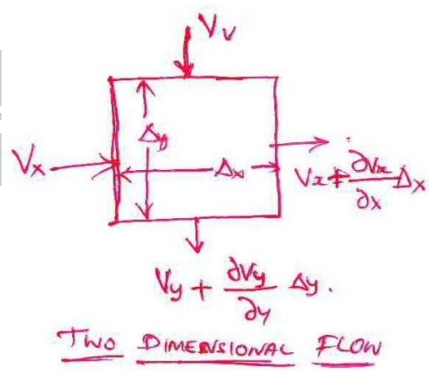
3) There is no change in the degree of saturation (s) in the zone of soil through which water seeps.

4) The quantity of water flowing into element of volume = Quantity which flows out in the same length of time.

5) The hydraulic boundary conditions at entry and exit are known.

6) Water is incompressible.

Consider an element of soil of size Δx, Δy and of unit thickness.
Let v_x, v_y are entry velocity components in 'x' and 'y' directions



$v_x + \frac{\partial v_x}{\partial x}\Delta x$

exit velocity components

$v_y + \frac{\partial v_y}{\partial y}\Delta y$

According to assumption no., 4, Entry 'v' = Exit 'v'

$$v_x(\Delta y.1) + v_y(\Delta x.1) = \left(v_x + \frac{\partial v_x}{\partial x}\Delta x\right)(\Delta y.1) + \left(v_y + \frac{\partial v_y}{\partial y}\Delta y\right)(\Delta x.1)$$

From which, $$\frac{\partial v_x}{\partial x} + \frac{\partial v_x}{\partial x} = 0 \longrightarrow 1 \text{ (continuity equation)}$$

According to assumption no., 2,

$$v_x = k_x \, . \, i_x = k_x \, . \, \frac{\partial h}{\partial x}$$

$$v_y = k_y \, . \, i_y = k_y \, . \, \frac{\partial h}{\partial y}$$

Where, h $\longrightarrow$ hydraulic head.

k_x, k_y $\longrightarrow$ co-efficient of permeability in ‘x’ and ‘y’ directions.

Substitute the value of v_x, v_y in eqn., 1

We get, $$\frac{\partial^2 (k_x \, . \, h)}{\partial x^2} + \frac{\partial^2 (k_y \, . \, h)}{\partial y^2} = 0$$

For an isotropic soil, $k_x = k_y = k$

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0$$

Substitute $\phi = kh$ = velocity potential, we get,

$$\boxed{\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} = 0}$$

This is the Laplace equation of flow in two directions.

<u>**Velocity potential (ϕ):**</u>

It is defined as a scalar function of space and time such that its derivative with respect to any direction gives the fluid velocity in that direction.

We know $\phi = kh$

$$\frac{\partial \phi}{\partial x} = k \frac{\partial h}{\partial x} = k \, i_x = v_x$$

Similarly,

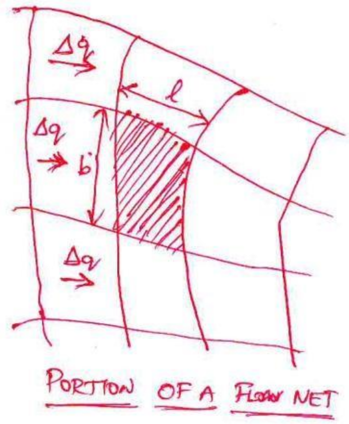
$$\frac{\partial \phi}{\partial y} = k \frac{\partial h}{\partial y} = k \, i_y = v_y$$

The solution of Laplace equation can be obtained by,

1) Analytical methods
2) Graphical methods
3) Experimental methods.

Solution gives two sets of curves known as,

a) Equipotential lines (E.L)
b) Stream lines (Flow lines) (S.L (or)F.L)



Mutually orthogonal to each other.

Equipotential line(E.L.)

It represents contours of equal head (potential).

Direction of seepage always perpendicular to E.L.

Stream lines (Flow lines)(S.L. or F.L.)

The path along which the individual particles of water seep through the soil.

Properties of flow net:

1) The flow line and equipotential line meet at right angles to one another.

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

3) The quantity of water flowing through each flow channel is the same. Similarly, the same potential drop occur between two successive equipotential lines.

4) Smaller the dimensions of the field, greater will be the hydraulic gradient and velocity of flow through it.

5) In a homogeneous soil, every transition in the shape of the curves is smooth, being either elliptical (or) parabolic in shape.

APPLICATIONS OF FLOW NET:

A flow net can be utilized for the following purposes.

1) Determination of seepage:

The portion between any two successive flow lines is known as at flow channel. The portion enclosed between two successive equipotential lines and successive flow lines is known as field (latched in figure).

Let , 'b' and 'l' be the width and length of the field.

Δh = head drop through the field

Δq = discharge passing through the flow channel

H = total hydraulic head causing flow = diff., b/w v/s and D/s heads

From Darcy's law, of flow through soils,

$\Delta q = k. \Delta h \, (b \times 1)$ (unit thickness)

$\overline{l}$

If N_d = Total no., of potential drops in the complete flow net, then

$\Delta h = \frac{H}{N_d}$; $\Delta q = k \cdot \frac{H}{N_d}\left(\frac{b}{l}\right)$

Total discharge, $q = \sum \Delta q = k \cdot \frac{H}{N_d}\left(\frac{b}{l}\right) \cdot N_f$

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

Where, N_f = Total number of flow channels in the net.

The field is square, hence b = l

Thus,

$$q = k H \frac{N_f}{N_d}$$ → Discharge expression for passing through a flow net

For isotropic soils $k_x = k_y = k$

2) <u>Determination of hydrostatic pressure:</u>

The hydrostatic pressure at any point with the soil mass is given by,

$$u = h_w \cdot \gamma_w$$

Where, u = hydrostatic pressure

h_w = piezometric head

The hydrostatic pressure interms if 'h_w' is,

$$h_w = h - z$$

Where h = hydraulic potential at that point

z = position head of the point above datum (↑ +ve)

Hence, $H = h_w + h + z$ (interms of %)

Used to plot a pressure net representing lines of equal water pressure (piezometric head).

<u>For example:</u>

Take h_w = 20%h_w = 20 = h – z

if, h = 30% H, z = 30 – 20 = 10% H

if, h = 40% H, z = 40 – 20 = 20% H

The results obtained from various points are plotted and joined by a smooth

curve to get a contour of h_w = 20% .

3) <u>Determination of seepage pressure:</u>

The hydraulic pressure 'h' at any point located after 'n' potential drops each of value Δh is given

by,

$h = H - n\Delta h$

We know, seepage pressure, $p_s = h\,\gamma_w$

$$P_s = (H - n\Delta h)\,.\,\gamma_w$$

The pressure acts in the direction of flow

4) <u>Determination of exit gradient:</u>

It is the hydraulic gradient at the d/s end of the flow line where percolating water leaves the soilmass and emerges into free water at d/s.

$$i_e = \frac{\Delta h}{l}$$

Where, Δh = potential drop

l = average length of last field in the flow net at exit

1) A homogeneous anisotropic earth dam, which is 20m high is constructed on an impermeable foundation. The coefficients of permeability of soil used for the construction of the dam. in the horizontal and vertical direction are 4.8×10^{-8} m/s and 1.6×10^{-8}m is respectively. The water level on the reservoir side is18m from the base of the dam: downstream side is dry.lt is seen that there are 4 flow channels and 18 equipotential drops is a square flow net drawn in the transformed dam section. Estimate the quantity of seepage per unit length in m^3/s through the dam.

$$q = k.H.\frac{N_f}{N_d}$$

$$k = \sqrt{k_H K_V}$$

$$= \sqrt{4.8X1.6X10^{-8}}$$

$= 2.77X10^{-8}$ m/s

$$q = 2.77\text{x}10^{-8}.18.\frac{4}{18}$$

$=11.085\text{x}10^{-8} m^3/s/m$ run