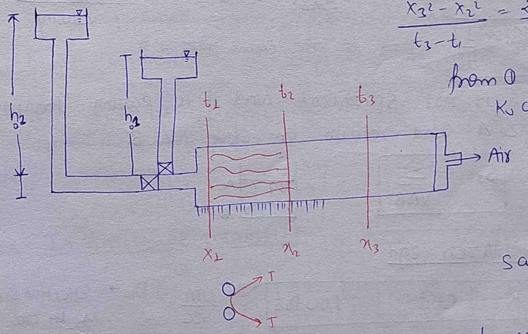
lecture
07/11/19 (Not too much imp)



$$\frac{x_2^2 - x_1^2}{t_2 - t_1} = \frac{2K_v}{S.n}(h_{o1} + h_c) \quad \text{--- (1)}$$

$$\frac{x_3^2 - x_2^2}{t_3 - t_1} = \frac{2K_v}{S.n}(h_{o2} + h_c) \quad \text{--- (2)}$$

from (1) and (2) determine K_v and h_c

K_v = permeability of saturated soil

h_c = height of capillary raise

S and n = saturation and porosity of medium

h_{o1} and h_{o2} = head under which water is flowing

(2) Field Method

(1) Well in unconfined aquifer (thiem's theory)

$$K = \frac{2.303\, q \log\left(\frac{r_2}{r_1}\right)}{\pi (h_2^2 - h_1^2)}$$

Dupit's theory

$$K = \frac{2.303\, q \log\left(\frac{R}{r_w}\right)}{\pi (H^2 - h_w^2)}$$

(1) Well in confined aquifer

Thiem's theory

$$K = \frac{2.303 \times q \log\left(\frac{r_2}{r_1}\right)}{2\pi D (h_2 - h_1)}$$

Dupit's theory

$$K = \frac{2.303\, q \log\left(\frac{R}{r_w}\right)}{2\pi D (H - h_w)}$$

Indirect Method

1) Kozney's - Carman Eqⁿ

• Permeability depend on fluid flow also

(Emphirical eqⁿ) ★★

$$\boxed{K = \frac{1}{K_K}\left(\frac{\gamma}{\mu}\right)_{fluid} \cdot \left\{\frac{e^3}{1+e} \cdot d^2\right\}_{medium} \quad \text{or} \quad K = \frac{1}{K_K'}\left(\frac{\gamma}{\mu}\right)_{fluid}\left\{\frac{e^3}{S_s^2}\right\}_{medium}}$$

K_K and K_K' → constant

γ and μ → unit weight of dynamic viscosity of fluid

e = void ratio and soil medium

d = Particle size

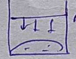
S_s = Specific surface area = $\frac{\text{surface area}}{\text{volume}}$

For spherical particle $S_s = \dfrac{4\pi r^2}{\frac{4}{3}\pi r^3} = \dfrac{3}{r} = \dfrac{6}{d}$

$\boxed{\uparrow S_s \propto \frac{1}{d\downarrow}}$ जरूरी नहीं हमेशा depend करेगा 'd' पे

If the particle is not spherical and it is passing through sieve of size A and retain on sieve size B.

$\boxed{S_s = \frac{6}{\sqrt{AB}}}$



2) Allen Hazen eqⁿ

$\boxed{K = CD_{10}^2 = 100 D_{10}^2}$ ★★ $\frac{cm}{sec}$ D_{10} → Effective size is in 'cm'

3) consolidation eqⁿ

$\boxed{K = C_v m_v \gamma_w}$ ★★

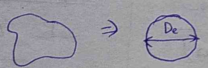
C_v = Coeff. of consolidation (m^2/sec)

m_v = Coeff. of vol^m compressibility (m^2/kN)

4) Terzaghi eqⁿ

$K = 200\, e^2 D_e^2$ cm/sec

D_e = equivalent size of sphere whose (S_s) is same as that of given particle



$(S_s)_{particle} = (S_s)_{sphere} = \frac{6}{D_e}$

Factor affecting the Permeability

(dominating factor)

1) Particle size $\boxed{K \propto d^2}$ or $\boxed{K \propto D_{10}^2}$

2) Void Ratio (Keeping all the factor constant)

Kozney's $\boxed{K \propto \frac{e^3}{1+e}}$ or Terzaghi $\boxed{K \propto e^2}$

3) Sp. surface area $\boxed{K \propto \frac{1}{S_s}}$

4) Fluid properties

$K \propto \left(\frac{\gamma}{\mu}\right)_{fluid}$

$\mu \propto \frac{1}{T^\circ}$ $\boxed{K \propto T^\circ_c}$

Note

It is difficult to compare and analyse 2 soil sample becoz permeability is dependent on both medium properties and fluid property thus coefficient of intrinsic permeability or absolute permeability is used which is dependent only upon medium properties.

$K = \frac{1}{K_K}\left(\frac{\gamma}{\mu}\right)_{fluid}\left\{\frac{e^3}{1+e} d^2\right\}_{medium}$

(Intrinsic)

Intrinsic permeability $K_o = f(\text{medium properties})$

$\boxed{K_o = \frac{K}{\left(\frac{\gamma}{\mu}\right)_{fluid}} = \frac{K\mu}{\gamma}}$ $\quad \frac{\frac{m}{sec}\cdot\frac{Ns}{m^2}}{N/m^3} = m^2$ or cm^2

5) Entrapped Gases and saturation

It is the presence of air in the voids

It is the resistance in the form of air block Hence lower is the permeability.

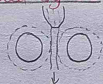
$\boxed{\downarrow K \propto \frac{1}{\text{Entrapped Gases}\uparrow}}$ $\boxed{K \propto S}$ $\because S + a_c = 1$

6) $\left\{K \propto \frac{1}{\text{foreign impurities}}\right\}$

7) Adsorbed Water

1) H↑ is the presence of adsorbed water lower it's permeability b/c lower is the area available to the flow of the fluid.

2) It is assumed that in the presence of adsorbed water void ratio of soil is approximately reduced by 0.1

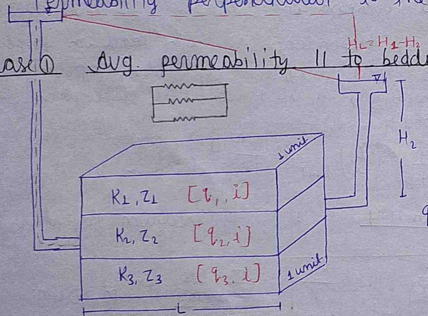


8) Stratification of soil

1) Permeability ∥ to bedding plane is more than the permeability perpendicular to the bedding plane

$H_L = H_1 - H_2$

case ① Avg. permeability ∥ to bedding plane (∥ arrangement)



total discharge

$q = q_1 + q_2 + q_3 + \dots$

$q = K_{avg}\, i\, A$

$= K_{avg}\, i\, [z_1 + z_2 + z_3 \dots] \times 1$

$= K_{avg}\, i\, \Sigma z_i$ — ①

Individual layer

$q_1 = K_1\, i\, A_1 = K_1\, i\, (z_1 \cdot 1)$

$q_2 = K_2\, i\, A_2 = K_2\, i\, (z_2 \cdot 1)$

$q_3 = K_3\, i\, A_3 = K_3\, i\, (z_3 \cdot 1)$ —— ②

from eq ① $q = q_1 + q_2 + q_3 + \dots$

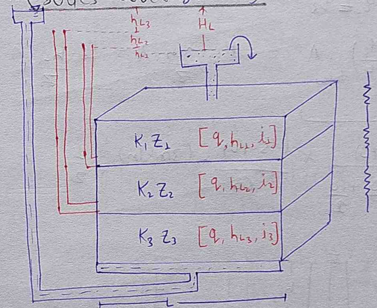
$K_{avg}\, \cancel{i}\, \Sigma z_i = K_1 \cancel{i} z_1 + K_2 \cancel{i} z_2 + K_3 \cancel{i} (z_3) + \dots$

from ① and ② $K_{avg} = \dfrac{K_1 z_1 + K_2 z_2 + K_3 z_3 + \dots}{z_1 + z_2 + z_3 + \dots}$

for Parallel $\boxed{K_{avg} = \dfrac{\Sigma K_i z_i}{\Sigma z_i}}$ ★★★

★★★ case-Ⓑ Avg. permeability perpendicular to bedding plane

(series arrangement)



Rule: series arrangment

$\boxed{\text{Discharge} \rightarrow \text{same}}$

$\boxed{i \text{ and Head loss} \rightarrow \text{different}}$

$\boxed{q = q_1 = q_2 = q_3}$

total head loss (H_L) = $h_{L1} + h_{L2} + h_{L3} + \dots$ Ⓐ

As per darcy's law

$q = K_{avg\perp}\, i\, A = K_{avg\perp} \left(\dfrac{H_L}{L}\right)_{total} A = K_{avg\perp} \dfrac{H_L}{(z_1 + z_2 + z_3 + \dots)} A$

$= K_{avg\perp} \dfrac{H_L}{\Sigma z_i} A$

$H_L = \dfrac{q}{A} \dfrac{\Sigma z_i}{K_{avg\perp}}$ --- ①

Individual

$q = q_1 = q_3 = \dots$

$q = \dfrac{K_1 h_{L1}}{z_1} A = K_2 \dfrac{h_{L2}}{z_2} A = K_3 \dfrac{h_{L3}}{z_3} A = \dots$

$h_1 = \dfrac{q}{A} \cdot \dfrac{z_1}{K_1}, \quad h_2 = \dfrac{q}{A} \cdot \dfrac{z_2}{K_2}, \quad h_3 = \dfrac{q}{A} \cdot \dfrac{z_3}{K_3}$] --- ②

from eqn ① $H_L = h_{L1} + h_{L2} + h_{L3} + \dots$

from ① and ② $\dfrac{q}{A} \cdot \dfrac{\Sigma z_i}{K_{avg\perp}} = \dfrac{q}{A} \cdot \dfrac{z_1}{K_1} + \dfrac{q}{A} \cdot \dfrac{z_2}{K_2} + \dfrac{q}{A} \cdot \dfrac{z_3}{K_3} + \dots$

$K_{avg\perp} = \dfrac{z_1 + z_2 + z_3}{\dfrac{z_1}{K_1} + \dfrac{z_2}{K_2} + \dfrac{z_3}{K_3}}$

★★ $\boxed{K_{avg\perp} = \dfrac{\Sigma z_i}{\Sigma \dfrac{z_i}{K_i}}}$

Solution (24) Page No (19)

Rule → series arrangement

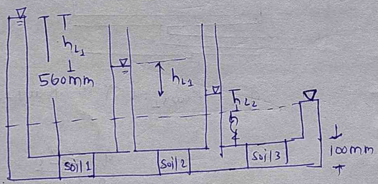
Discharge same

$q = q_1 = q_2 = q_3$

$$\underbrace{\frac{\Sigma Z_i}{\Sigma \frac{Z_i}{K_i}} \times \left(\frac{H_L}{L}\right)_{total} \cdot A}\ = K_1 \cdot \frac{h_{L_1}}{Z_1} \cdot A = K_2 \cdot \frac{h_{L_2}}{Z_2} \cdot A$$

$$\frac{(20+30+10)}{\frac{20}{2} + \frac{30}{3} + \frac{10}{1}} \times \frac{(15-10)}{60} \times (3m \times 1m) = \frac{1}{2}$$

Solution (25)



q → constant

i and Head diffinet

total head

$H_L = h_{L_1} + h_{L_2} + h_{L_3}$

$$H_L = \frac{q}{A} \cdot \frac{\Sigma Z_i}{K_{avg\,1}}$$

$$K_{avg\,1} = \frac{150+150+150}{\frac{150}{0.01} + \frac{150}{0.003} + \frac{150}{0.03}}$$

$$K_{avg\,1} = \frac{450}{70{,}000} = 6.42 \times 10^{-3}$$

$$\frac{\Sigma Z_i}{\Sigma \frac{Z_i}{K_i}} \times \left(\frac{H_L}{L}\right)_{total} \cdot A = K_1 \frac{h_{L_1}}{Z_1} \cdot A = K_2 \frac{h_{L_2}}{Z_2} \cdot A = K_3 \cdot \frac{h_{L_3}}{Z_3} \cdot A$$

$$\left[\frac{150+150+150}{\frac{150}{0.01} + \frac{150}{0.003} + \frac{150}{0.03}}\right] \times \frac{560}{(150+150+150)} \times A = 0.01 \cdot \frac{h_{L_1}}{150} \cdot A = 0.003 \cdot \frac{h_{L_2}}{150} A$$

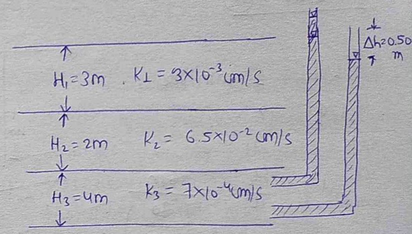
$$= 0.03 \times \frac{h_{L_3}}{150} \times A$$

$h_{L_3} = 40\,mm$

$h_{L_2} = 400\,mm$

$h_{L_1} = 120\,mm$

Solution (27)



series

q → same

i and H_L → different

$q = q_1 = q_2 = q_3$

total head loss

$H_L = h_{L_1} + h_{L_2} + h_{L_3}$

$$\left\{ H_L = \frac{q}{A} \cdot \frac{\Sigma Z_i}{K_{avg\,i}} \right\} =$$

$$\frac{K_1 \cdot h_{L_1} \cdot A}{Z_1} = \frac{K_2 h_{L_2} \cdot A}{Z_2} = K_3 \frac{h_{L_3} \cdot A}{Z_3}$$

$$K_{avg\,i} = \frac{Z_1 + Z_2 + Z_3}{\frac{Z_1}{K_1} + \frac{Z_2}{K_2} + \frac{Z_3}{K_3}}$$

$$K_{avg\,i} = \frac{3+2+4}{\frac{3}{3\times10^{-3}} + \frac{2}{6.5\times10^{-2}} + \frac{4}{7\times10^{-4}}}$$

$$K_{avg\,i} = \frac{9}{1000 + 30.77 +}$$

$$K_{avg\,i} = \frac{9}{6775.82} = 1.18 \times 10^{-3}$$

$$\frac{3+2+4}{\left(\frac{3}{3\times10^{-3}} + \frac{2}{6.5\times10^{-2}} + \frac{4}{7\times10^{-4}}\right)} \times \left(\frac{H_L}{9}\right) \cdot A$$

$$= 7\times10^{-4} \times \frac{0.5}{2} \cdot A$$

$\boxed{H_L = 1.18 m}$

Solution (26)

$$K = 2.303 \cdot \frac{a \cdot L}{A \cdot t} \cdot \log\left(\frac{h_1}{h_2}\right) = \cancel{2.303} \cdot \frac{aL}{A(5min)} \cdot \log\left(\frac{50}{48}\right)$$

$$\text{and} \quad \cancel{2.303} \cdot \frac{aL}{\cancel{A}(t)} \cdot \log\left(\frac{50}{25}\right)$$

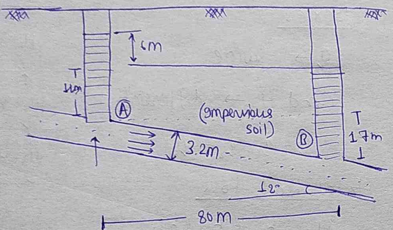
$$\frac{1}{5min} \log\left(\frac{50}{48}\right) = \frac{1}{t} \cdot \log\left(\frac{50}{25}\right)$$

$t = 84.89$ min

Solution - 34

$K = 1.2 \text{ mm/sec}$

$q = ?$

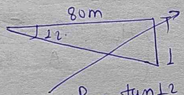


$i = \frac{H_L}{L} = \frac{6m}{\left(\frac{80}{\cos 12}\right)}$

$q = KiA$

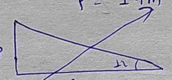
$= 1.2 \frac{mm}{sec} \times 10^{-3} m \times \frac{6m}{\left(\frac{80}{\cos 12}\right)} \times (3.2 \times 1)$

$q = 2.81 \times 10^{-4} \; m^3/s/m \text{ length}$



$\frac{P}{80} = \tan 12$

$P = 17m$

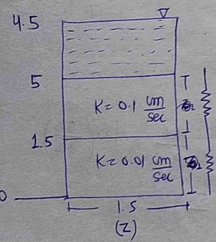


$\frac{P}{80} = \tan 12$

$P = 17$

Solution 41

Series

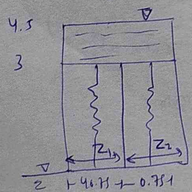


$q = K_{avg} \cdot i \cdot A$

$= \frac{\Sigma Z_i}{\Sigma \frac{Z_i}{k_i}} \left(\frac{H_L}{L}\right) \cdot A$

$= \frac{1.5 + 1.5}{\frac{1.5}{0.1} + \frac{1.5}{0.01}} \frac{cm}{sec} \times 10^{-2} m \left(\frac{4.5}{3}\right)(1.5 \times 1)$

$= 4.09 \times 10^{-4} \; m^3/s$



$q = K_{avg\parallel} \cdot \left(\frac{H_L}{L}\right) \cdot A$

$= \frac{\Sigma k_i z_i}{\Sigma z_i} \cdot \frac{H_L}{L} \cdot A$

$= \frac{0.1 \times 0.75 \times 0.01 \times 0.75}{0.75 + 0.75} \times \left(\frac{4.5}{3}\right)(1.5 \times 1)$

$= 1.23 \times 10^{-3} \; m^3/s$

CHAPTER 4 (Quick sand condition)

When water flows through saturated soil mass, total head at any point consist of pressure head, datum head and velocity head. Since velocity through the soil mass is considerary less hence velocity head is neglected in soil mass therefore, total head at any point consist of datum head and pressure head which point is also termed as hydraulic head or Piezometric head.

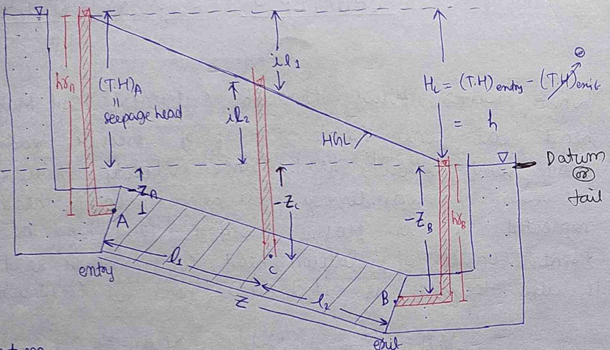
$V.H = \frac{v^2}{2g} = \frac{(Ki)^2}{2g} \rightarrow$ Neglisible

$\boxed{P.H + D.H + T.H = \text{Piezometric head} = \text{hydraulic head}}$

* Datum head at a point is a vertical distance of that point measured from assumed datum plane which is normally taken at the tail water level elevation for convenience.

* If a Piezometer or an open stand pipe is inserted at a point of flow then water would stand upto a particle height inside a piezometer termed Pressure head whereas height of Piezometric surface from datum is termed as Piezometric head

* The difference b/w total head at any two point in a soil mass through which flow is occuring represent the head loss b/w those two point, which is also termed as seepage head b/w those two point's



Datum

Rule

1. Entry side → entry ▼
2. Exit side → exit ▼
3. $i = \dfrac{\text{Head loss}}{\text{length}} = \dfrac{H_L}{Z} = \dfrac{h}{Z}$
4. Head loss = $i \times (\text{length})$
5. $(T.H)_C = (T.H)_{entry} - i l_1$ or $(T.H)_{exit} + i l_2$

Seepage Pressure when water flows through saturated soil mass it exert pressure over soil skeleton by virtue of frictional drag. this pressure exerted by water is termed as seepage pressure

→ If h is the seepage head or head diff. due to which seepage taking place then,

$$\boxed{\text{Seepage Pressure} = \text{Seepage Head} \times \gamma_w} \; *$$

$$\boxed{P_s = h \cdot \gamma_w = i\, Z \gamma_w} \; ** \qquad \because \; i = \frac{h}{Z}$$

Seepage force $(P_s) = P_s \times \text{Area} = i\, Z \gamma_w * A$

Sepecific seepage force $(P_{ss}) = \dfrac{P_s}{\text{Vol}^m} = \dfrac{i Z \gamma_w . A}{A . Z}$

$$\boxed{P_{ss} = i \gamma_w}$$

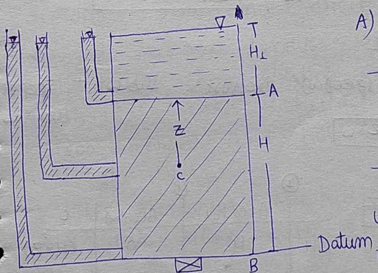
Note Seepage Pressure always acts in the dirn of flow

Hence vertical effective stress may be increased or reduced due to seepage depending upon the dirn of flow. If flow takes place in vertically downward dirn, effective stress will ↑ because seepage pressure also act in downward dirn. If flow takes place in upward dirn effective stress will ↓ because seepage pressure acts in upward dirn

Eff. Stress $(\bar{\sigma}) = Z\gamma' \pm P_s = Z\gamma' \pm i Z \gamma_w$

If flow is vertically downward $\boxed{\bar{\sigma} = Z\gamma' + i Z \gamma_w}$ **

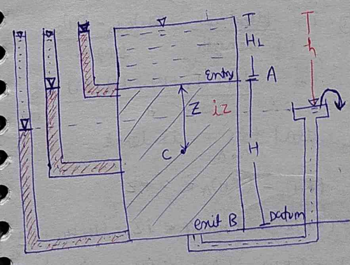
If flow is vertically upward $\boxed{\bar{\sigma} = Z\gamma' - i Z \gamma_w}$ **



A) No flow cond

	A	B	C
D.H	H [Pressure is constant in at Point]	0	H−Z
P.H	H_1	$H+H_1$	H_1+Z
T.H	$H+H_1$	$H+H_1$	$H+H_1$
σ	$H_1\gamma_w$	$H_1\gamma_w + H\gamma_{sat}$	$H_1\gamma_w + Z\gamma_{sat}$
$U = (P.H)\gamma_w$	$H_1\gamma_w$	$(H+H_1)\gamma_w$	$(H_1+Z)\gamma_w$
$\bar{\sigma} = \sigma - u$	0	$H\gamma'$	$\boxed{Z\gamma'}$

B) Downward flow cond



	① A(entry)	② B(exit)	C
D.H	H	0	(H−Z)
P.H	H_1	$(H+H_1-h)$	H_1+Z-iZ
T.H	$H+H_1$	$(H+H_1-h)$	$H+H_1-iZ$

③ $i = \dfrac{\text{Head loss}}{\text{length}} = \dfrac{(T.H)_{entry} - (T.H)_{exit}}{\text{length}}$

$= \dfrac{(H+H_1) - (H+H_1-h)}{H}$

$= \dfrac{h}{H}$

P.T.O

P.T.O

(4) Head loss = $i \times (\text{length})_{A\ to\ C}$

$$(T.H)_A - (T.H)_C = i \times Z$$

(5) $(T.H)_c = (T.H)_{entry} - iZ = (H+H_1) - iZ$

$$(PH)_c = (T.H)_c - (D.H)_c = (H+H_1-iZ) - (H-Z)$$
$$= H_1 + Z - iZ$$

Total stress approach

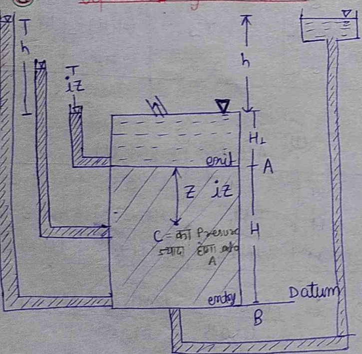
	A	B	C
σ	$H_1\gamma_w$	$H_1\gamma_w + H\gamma_{sat}$	$H_1\gamma_w + Z\gamma_{sat}$
$U=(PH)\gamma_w$	$H_1\gamma_w$	$(H+H_1-h)\gamma_w$	$(H_1+Z-iZ)\gamma_w$
$\bar\sigma=\sigma-U$	0	$H\gamma' + h\gamma_w$	$\boxed{Z\gamma' + iZ\gamma_w}$ *

Effective stress Approach

$\bar\sigma$ = Sub wt ± Seepage pressure = Sub wt ± Seepage Head × γ_w

$$\boxed{\bar\sigma = Z\gamma' + iZ\gamma_w}$$

(C) Upward flow cond



	(2) A(exit)	(1) B(entry)	C
D.H	H	0	(H−Z)
P.H	H_1	$(H+H_1+h)$	(H_1+Z+iZ)
TH	$H+H_1$	$H+H_1+h$	$H+H_1+iZ$

(3) $i = \dfrac{\text{Head loss}}{\text{length}} = \dfrac{T.H)_{entry} - T.H)_{exit}}{\text{length}}$

$$= \frac{(H+H_1+h) - (H+H_1)}{H} = \frac{h}{H}$$

(4) Head loss$_{C\to A}$ = $i \times$ length

$$(T.H)_C - (T.H)_A = i * Z$$

(5) $(T.H)_C = (T.H)_{exit} + iZ = (H+H_1) + iZ$

$$(P.H.)_c = (T.H)_c - (D.H)_c = (H+H_1+iZ) - (H-Z) = H_1 + Z + iZ$$

Total Stress Approach

	(A)	(B)	(C)
σ	$H_1\gamma_w$	$H_1\gamma_w + H\gamma_{sat}$	$H_1\gamma_w + Z\gamma_{sat}$
$U=(P.H)\gamma_w$	$H_1\gamma_w$	$(H+H_1+h)\gamma_w$	$(H_1+Z+iZ)\gamma_w$
$\bar\sigma = \sigma - U$	0	$H\gamma' - h\gamma_w$	$Z\gamma' - iZ\gamma_w$

Effective stress approach

$\bar\sigma$ = sub wt ± Seepage pressure = sub wt ± Seepage Head × γ_w

$$\bar\sigma = Z\gamma' - iZ\gamma_w$$

* Quick Sand cond

* When flow takes place in upward dirn and effective stress is reduced
* If seepage pressure equal to submerge ~~seep~~ wt of soil mass then effective stress is reduced to zero in such ~~cash~~ case cohesionless soil mass looses all it's shear strength and have the tendency to flow along with the water.
* This Phenomenon in which soil particles leave the soil mass and flow along with the water is termed as Quicksand, pipe in, sand boiling (or) Floating cond

In Quick sand cnd $\bar\sigma = \sigma - U = 0$

$$\bar\sigma = Z\gamma' - iZ\gamma_w = 0$$
$$Z\gamma' = iZ\gamma_w$$
$$i = \frac{\gamma'}{\gamma_w} = \frac{(G-1)\gamma_w}{(1+e)\gamma_w}$$

critical hydraulic gradient

$$\boxed{i_c = \frac{G-1}{(1+e)}}$$ ***

It is also termed as piping gradient (or) bursting gradient

⇒ To avoid quick sand cond^n, hydraulic gradient i should be less than i_c **

$$\boxed{(F.O.S.)_{piping} = \frac{i_c}{i} = \frac{\text{sub. wt}}{\text{Seepage pressure}}}$$

* If for fine sand $G = 2.65$ and $e \approx 0.65$

$$\boxed{i_c = \left(\frac{G+1}{1+e}\right) = \frac{2.65-1}{1+0.65} \approx 1}$$

Note Quick sand is not a type of clay bt is flow cond^n in cohesionless soil mass when effective stress is reduce to (zero) due to upward flow cond^n. Quick sand generally occurs in sand and coarse silt and is not found in gravel, clay and fine silt.

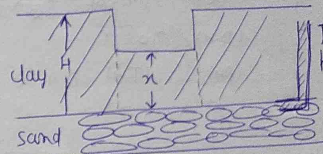
⇒ In cohesive soil like clay, shear strength is not reduce to zero even if effective stress is reduce to zero and soil particles are held together due to their inherent cohesion.

Shear strength $s = c + \bar{\sigma}_n \tan\phi$

In cohesionless soil $(c=0)$	In cohesive soil
$s = \bar{\sigma}_n \tan\phi$	$s = c + \bar{\sigma}_n \tan\phi$
In quick sand cond^n $(\bar{\sigma}_n = 0)$	In quick sand cond^n $(\bar{\sigma}_n = 0)$
$s = \cancel{\bar{\sigma}_n} \tan\phi = 0$	$s = c + \cancel{\bar{\sigma}_n} \tan\phi \neq 0$

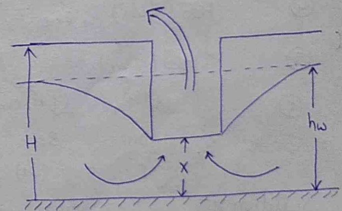
→ Generally, it does not occur in gravels and coarse sand, because high discharge is required to developed quick sand cond^n which is practically not possible in hydraulic structure.

→ Quick sand cond^n can also occurs, when sand under artesian pressure in overlained by impermeable soil like clay



Total stress Approach	Eff stress Approach	
total wt $= (P.H)\gamma_w$ $X\gamma_{sat} = h_w\gamma_w$	Sub. wt = Seepage pressure $X\gamma' = (h_w - X)\gamma_w$ Depth of excavation $(H-X)$	If F.O.S is given $F.O.S = \frac{\text{sub wt}}{\text{Seepage pressure}} = \frac{X\gamma'}{(h_w-X)\gamma_w}$

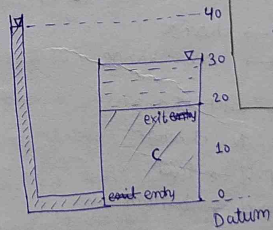
⇒ Quick sand also occurs when excavation is done below G.W.T and water is pumped out to carry out the engineering activities.



$X\gamma' = (h_w - X)\gamma_w$

$X\gamma_{sat} = h_w\gamma_w$

Ques determine datum head, pressure head, total head at entry exit and point (C)



	① entry	② exit
D.H	0	20
P.H	40	10
T.H	40	30

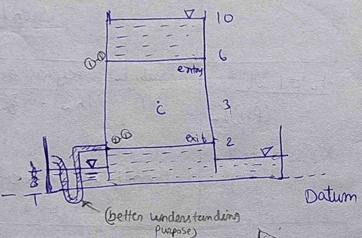
③ $i = \frac{H_L}{L} = \frac{40-30}{20} = \frac{1}{2}$

④ $(\text{Head loss})_{c \to exit} = i \times \text{length}$

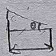
$(T.H)_c - (T.H)_{exit} = i \times (10)$

⑤ $\begin{cases} (T.H)_c = (T.H)_{exit} + \frac{1}{2} \times 10 = 30 + 5 = 35 \\ (D.H)_c = 10 \\ (PH)_c = (T.H) - D.H = 35 - 10 = 25 \end{cases}$

Solution ②



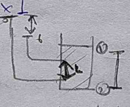
	entry ①	exit ②
D.H	6	2
P.H	4	-2
T.H	10	0



③ $i = \frac{H_L}{L} = \frac{10-0}{4} = 2.5$

④ (Head loss)$_{\hookrightarrow exit}$ = $i \times$ length = $2.5 \times (1) = 2.5$

⑤ ~~(Head loss)~~$_c$ ← $(T.H)_c - (T.H)_{exit} = i \times L$



⑤ $\begin{cases} (T.H)_c = (T.H)^0_{exit} + 2.5 \times 1 = 2.5 \\ (DH)_c = 3 \\ (P.H)_c = (T.H) - D.H = 2.5 - 3 = (-0.5) \end{cases}$

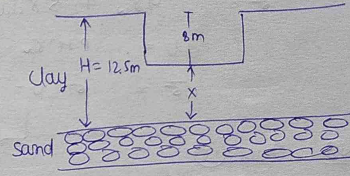
$i = \frac{H_L}{L}$

Solution ⑮ Datum → change
D.H → also change
↓
T.H → will change
PH → No change
Ist pref → tail
2nd pref → bottom

Solution ④① Also determine depth of excavation keeping FOS 1.3 having same position of W.T.

Datum head से नीचे negative Datum उसका

④① total stress Approach



I) $4.5(\gamma_{sat}) = h_w . \gamma_w$

$4.5 \times 18.5 = h_w . 9.81$

$h_w = 8.48\,m$

Effective Stress Approach

$4.5\gamma' = (h_w - 4.5)\gamma_w$

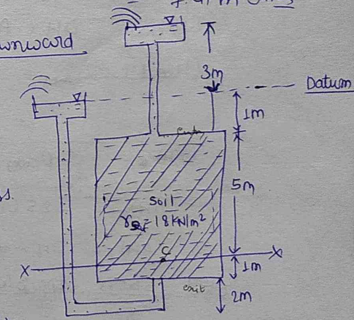
$\boxed{h_w = 8.48m}$

II) $F.OS = \frac{\text{sub wt}}{\text{seepage press}} = \frac{x\gamma'}{(h_w - x)\gamma_w}$

$1.3 = \frac{x(18.5 - 9.81)}{(8.48 - x)\,9.81}$

$\boxed{x = 5.04m}$ depth $= 12.5 - 5.04$
$= 7.41$ m Ans

Solution ③③ Downward



Effective vertical stress

or, $i = \frac{H_L}{L} = \frac{3}{6} = 0.5$

σ = sub wt + seepage press.

$= z\gamma' + iz\gamma_w$

$= 5\gamma' + \frac{1}{2} \times 5\gamma_w$

$= 5(18 - 9.81) + \frac{1}{2} \times 5 \times 9.81$

$= 65.47\ kN/m^2$

total stress approach

	entry	exit
D.H	-1	-7
P.H	4	7
T.H	3	0

③ $i = \frac{3-0}{6} = 0.5$

④ $(T.H)_x - (T.H)^0_{exit} = i \times 1 = 0.5$

$(D.H)_x = -6$

$(P.H) = T.H - D.H = 0.5 - (-6)$
$= 6.5$

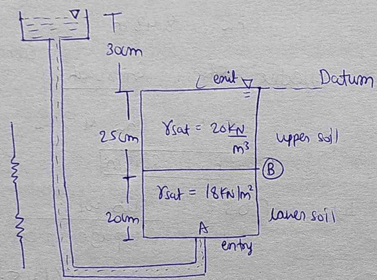
$\bar{\sigma} = \sigma - U$

$\sigma = 4\gamma_w + 5\gamma_{sat} = 4 \times 9.81 + 5 \times 18$

$U = (P.H)\gamma_w = 6.5\gamma_w$

$\bar{\sigma} = \sigma - U = 65.47\ KN/m^2$

W.B Ques-(13) PageNO (24)



30% of total head lost

total head (cm) and to Pressure head (cm) = B

ule - Series arrangement

$q \rightarrow$ same   i and H are different

$(T.H)_{entry} = DH + PH = -45 + 75 = 30cm$

$(Head\ Loss)_{lower} = \frac{h_L}{D}$ 30% of $(T.H)_A$

$= 30\% \times 30 = 9cm$

doubt

$(T.H)_B = (T.H)_{entry} - Head\ loss = 30 - 9 = 21$

$(D.H)_B = -25cm$

$(P.H) = T.H - D.H = 21 - (-25) = 46cm$

total Stress Approach

$\bar{\sigma} = \sigma - U = \sigma - (P.H)\gamma_w$

$= 0.25 \times 20 - 0.46 \times 9.81$

$= 0.487\ KN/m^2$

Effective Stress Approach

$\bar{\sigma}$ = Sub wt − Seepage Pressure

$\bar{\sigma} = 0.25\gamma' - i_2 Z \gamma_w$

$= 0.25(20 - 9.81)$

$- \left(\frac{21}{25}\right) 0.25 \cdot 9.81$

$= 0.487\ KN/m^2$

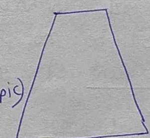
11 NOV monday : soil 6:00 PM to 9:30PM

13 NOV – 17 NOV : 6:00 to 9:30

CHAPTER-4

* Flow Net and Laplace eqn

When water flows through soil mass into dimension. It can be analyse using laplace eqn which Represent the loss of energy head in any resistive medium



$$\frac{\partial^2 H}{\partial x^2} + \frac{\partial^2 H}{\partial y^2} = 0 \quad \text{(if medium is istropic)}$$

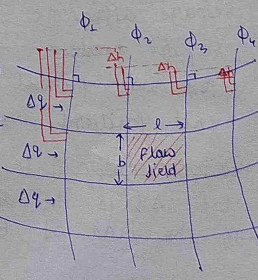
$$K_x \frac{\partial^2 H}{\partial x^2} + K_y \frac{\partial^2 H}{\partial y^2} = 0 \quad \text{(if medium is non-istropic)}$$

⇒ The graphical solution of laplace eqn is flow net which represent the description of equipotential line and stream line.

⇒ Assumption in flow net

1) Darcy's law is valid
2) Soil is Homogeneous and isotropic.
3) Pore fluid and soil solid are incompressible

Properties of flow net



1) equipotential line (φ line) and stream line (ψ lines) intersect each other ⊥ dicularly
2) Always flow takes place ∥ to flow lines and velocity of flow is always ⊥ to equipotential line
3) Loss of head b/w 2 equipotential lines is always same and is termed as equipotential drop

4) The area b/w two flow lines is known as flow channel and discharge through each flow channel is same.

5) Area bounded b/w two equipotential lines and flow line's is known as flow field, which are approximately square in isotropic medium that may be linear (or) curvilinear while in non-isotropic medium they are approximately rectangular. which may be linear (or) curvelinear

Note: If water levels are reverse on upstream and down-stream side without changing in boundary cond^n then there will be no change in flow net it means flow net is unique fun^n of given set of boundary cond.

Total discharge

$$\boxed{q = \Delta q \times N_f} \;**$$

N_f = No. of flow channels

N_f = No. of flow line - 1

Equipotential drop

$$\boxed{\Delta h = \frac{H}{N_d}} \;**$$

N_d = No. of equipotential drop

N_d = No. of equipotent line - 1

● Application of flow net

→ flow net can be used to determine

1) seepage discharge
2) seepage pressure.
3) pore water pressure (uplift pressure)
4) exit gradient

* Determination of seepage discharge

1) Let Δq is the discharge through a flow field under the head of H (&) consider unit length of dam

As per darcy's law $\Delta q = K i A$

$$i = \frac{\Delta h}{l}, \quad A = b * 1$$

$$\therefore \Delta q = K \cdot \frac{\Delta h}{l} \cdot (b \cdot 1) = K \cdot \Delta h \cdot \left(\frac{b}{l}\right)$$

Case (A) Medium is isotropic — Flow field is square $b = l$

$$\therefore \text{Discharge } \Delta q = K \cdot \Delta h \cdot \left(\frac{\cancel{b}}{\cancel{l}}\right) = K \cdot \Delta h$$

$$\therefore \text{total discharge } q = \Delta q * N_f = K \cdot \Delta h \cdot N_f \quad \left[\because \Delta h = \frac{H}{N_d}\right]$$

$$q = K \cdot \frac{H}{N_d} \cdot N_f \qquad \boxed{q = \frac{K \cdot H N_f}{N_d}} \;**$$

$m^3/s/m$ length of dam

N_f → No. of flow channels

N_d → No. of equipotential drops

$$\left[\frac{N_f}{N_d} = \text{shape factor}\right]$$

Discharge through non-isotropic medium

1) In non-isotropic medium flow field is rectangular Hence. it is transformed in isotropic medium such that flow field will be square

2) Laplace eq^n for non-isotropic medium

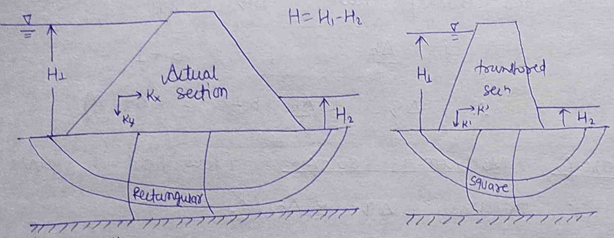
$$K_x \cdot \frac{\partial^2 H}{\partial x^2} + K_y \cdot \frac{\partial^2 H}{\partial y^2} = 0 \qquad \boxed{x = x_T \sqrt{\frac{K_x}{K_y}}} \;**$$

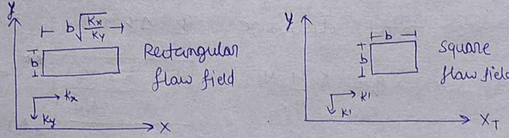
$$x^2 = x_T^2 \cdot \frac{K_x}{K_y}$$

$$\partial x^2 = \partial x_T^2 \cdot \frac{K_x}{K_y} \quad \text{then} \quad \left[K_x \cdot \frac{\partial^2 H}{\partial x_T^2 \frac{K_x}{K_y}} + K_y \cdot \frac{\partial^2 H}{\partial y^2} = 0\right]$$

$$\cancel{K_y} \cdot \frac{\partial^2 H}{\partial x_T^2} + \cancel{K_y} \frac{\partial^2 H}{\partial y^2} = 0$$

$$\left[\frac{\partial^2 H}{\partial x_T^2} + \frac{\partial^2 H}{\partial y^2}\right]$$





- Discharge through actual flow field

$$\Delta q = KiA = K_x \cdot \frac{\Delta h}{b\sqrt{\frac{K_x}{K_y}}} \cdot (b \times 1)$$

- Discharge through transformed flow field.

$$\Delta q = KiA = K' \cdot \left(\frac{\Delta h}{b}\right) \cdot (b \cdot 1)$$

- Discharge should be same

$$\Delta q = K' \cdot \frac{\Delta h}{b} (b \cdot 1) = K_x \cdot \frac{\Delta h}{b\sqrt{\frac{K_x}{K_y}}} \cdot (b \cdot 1)$$

$$K' = K_x \sqrt{\frac{K_y}{K_x}} = \sqrt{K_x K_y}$$

Equivalent Permeability

$\boxed{K' = \sqrt{K_x K_y}}$ → flow 2D flow

$K' = (K_x \cdot K_y \cdot K_z)^{\frac{1}{3}}$ → flow 3D flow

total discharge

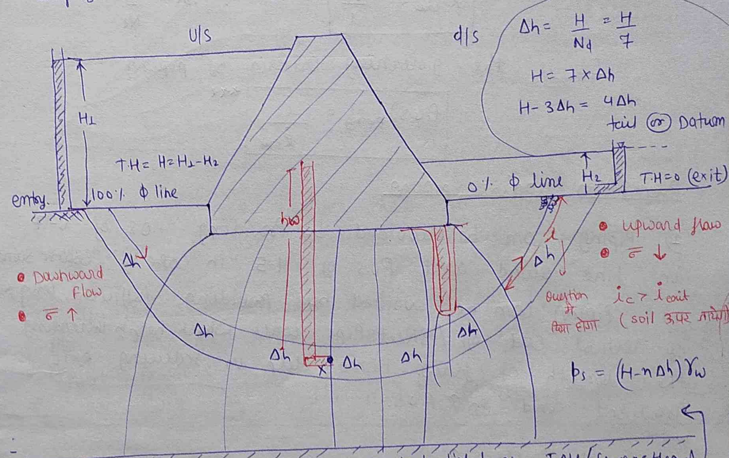
$$q = \Delta q \times N_f = K' \cdot \frac{\Delta h}{b} \cdot (b \cdot 1) N_f$$

$\boxed{q = K' H \cdot \frac{N_f}{N_d}}$ *** $\left(\Delta h = \frac{H}{N_d}\right)$

Determination of seepage pressure :

1) Let h is the seepage head after n equipotential drop then -

Seepage pressure = seepage head × γ_w



$P_s = h\gamma_w$ — After n equipotential drops — T.OH / Seepage Head $= (H - n\Delta h)$

③ Determination of hydrostatic pore water pressure

* let h_w is the pressure head after an equipotential drop then-

→ Pore water pressure = Pressure head × γ_w

$U = h_w \cdot \gamma_w$

After n equipotential drops

T.H / Seepage Head $= H - n\Delta h$

$T.H = P.H + D.H$

$P.H = T.H - D.H$

$U = \{(H - n\Delta h) - (DH)\}\gamma_w$

4) Determination of exit gradient

- It is the hydraulic gradient of last flow field from where water join the free water

$$\boxed{i_{exit} = \frac{\Delta h}{l}} \; \star \qquad \because \Delta h = \frac{H}{N_d}$$

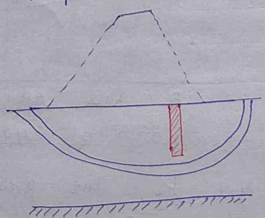
It is the governing criteria of piping

$$\boxed{F.O.S)_{piping} = \frac{i_c}{i_{exit}}} \; \star\star\star$$

Precaution against piping

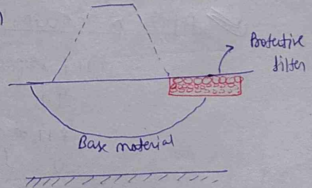
1) Piping can be avoided by providing f.o.s. of 6-7 in fine sand and F.O.S. of 4-5 in case of coarse sand

2) Piping can be avoided by providing sufficient length of vertical cut-off / sheet pipe wall which help in increasing the length of flow. Hence help in reducing exit gradient and in ↑ F.O.S.



length of flow ↑
→ i ↓
→ F.O.S ↑

3) Piping can be avoided by providing protective filter, graded filter (or) inverted filter which helps in both avoiding the erosion of soil particles and reducing the uplift pressure at the base of dam



Terzaghi's Guidelines for design of Protective Filter

1) $\dfrac{(D_{15})_{Filter}}{(D_{85})_{Base\ material}}$

2) $4 < \dfrac{(D_{15})_{Filter}}{(D_{15})_{Base\ material}}$ (Terzaghi)

(or)

$5 < \dfrac{(D_{15})_{Filter}}{(D_{15})_{Base\ material}}$ (Casagrande)

3) $\dfrac{(D_{15})_{Filter}}{(D_{15})_{Base\ material}} < 20$

$\dfrac{(D_{50})_{Filter}}{(D_{50})_{Base\ material}} < 25$

1) The first cond ensure's that no significant erosion / invasion of base material through the filter medium takes place.

2) The 2nd cond ensure's that sufficient loss of head in flow through filter takes place without build up of uplift pressure.

3) The third cond r the additional design consideration