BCE 27

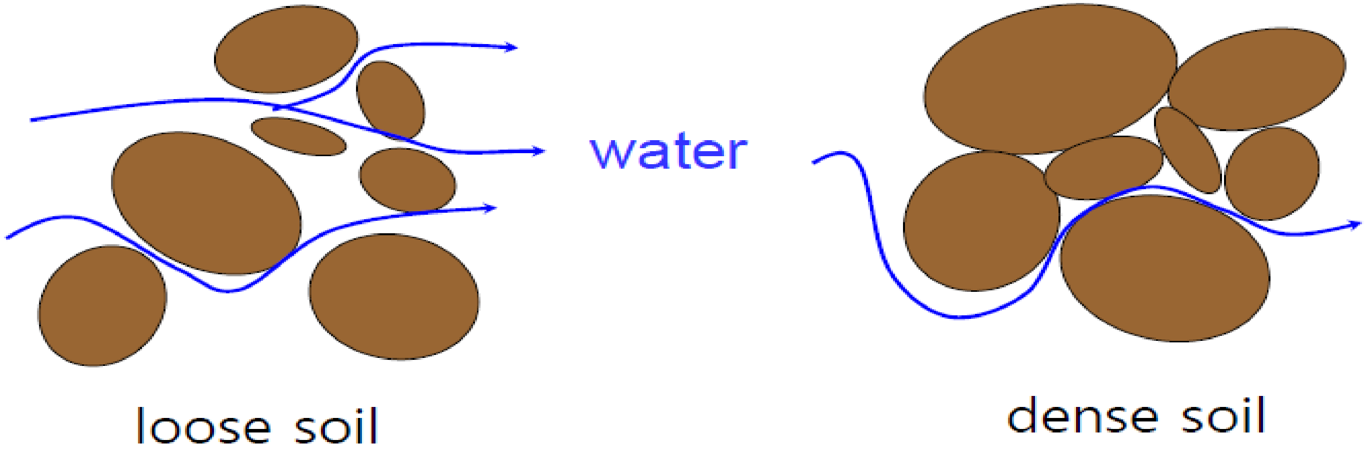
Permeability

PROF S M ALI JAWAID

Permeability

Definition

It is the property of soil which allows the flow of water through it.



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

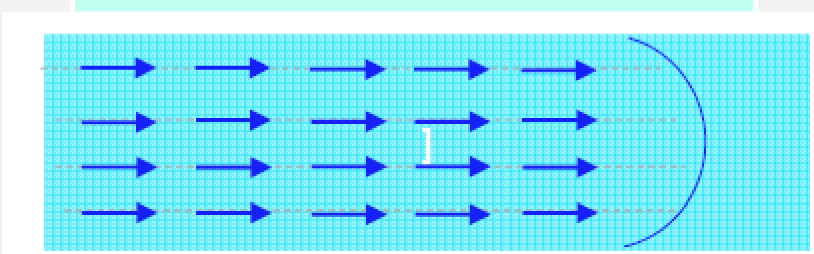
- *A soil is highly pervious when water can flow through it easily. (Gravels)*
- *In an impervious soil, the permeability is very low and water cannot easily flow through it. (Clays)*
- *Rocks are impermeable*
- *The study of the flow of water through permeable soil media is important in soil mechanics.*

Water flow through soil

Laminar flow

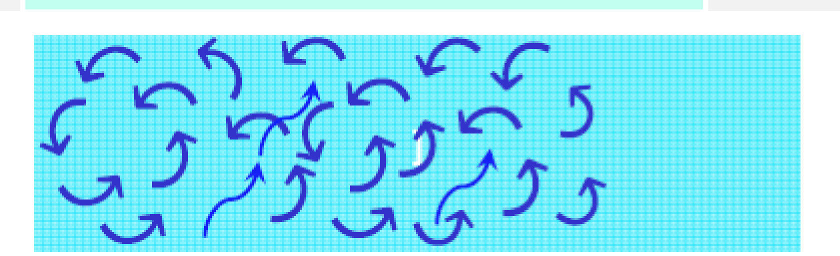
Each fluid particles travel along a definite path which never crosses the path of any other particle

(In Soils, mostly flow is laminar)



Turbulent flow

Path of travel irregular. Twisting, crossing and re-crossing at random



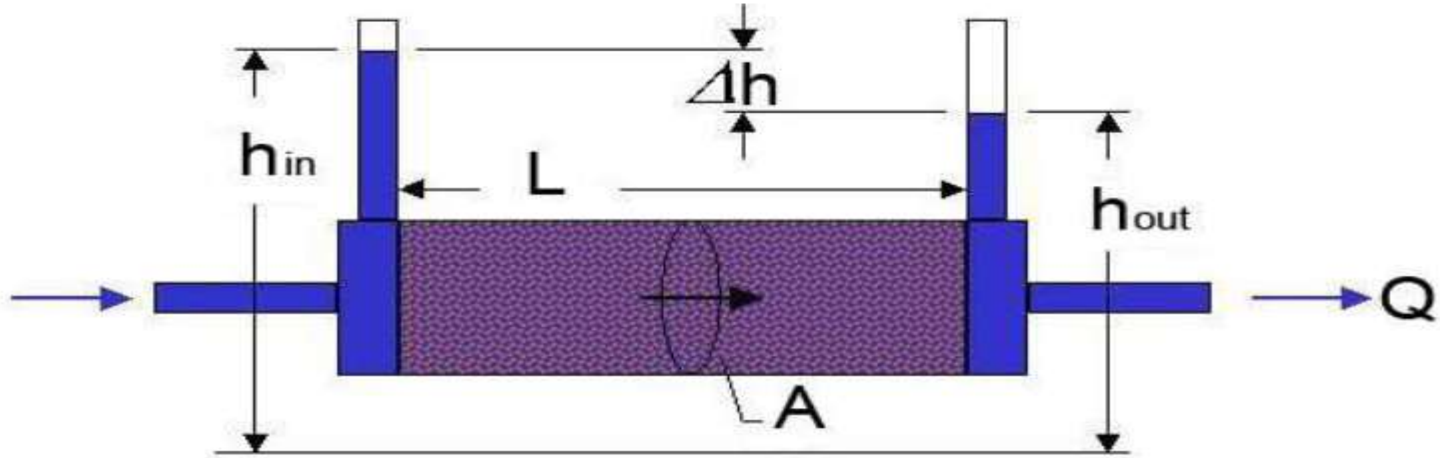
Why its study is required

- To calculate the rate of settlement of a saturated compressible soil layer.
- To calculate seepage through the body of earth dams and stability of slopes.
- To calculate the uplift pressure under the hydraulic structures and their safety against piping.
- Ground water flow towards wells and drainage of soil.

DARCY's LAW

Based on Expt study

For laminar flow condition in a saturated soil, the rate of flow/discharge per unit time across a cross-sectional area A is proportional to hydraulic gradient (under an standard temp of 27^{0}C)



$$V = q/A = ki$$

$$q = kiA$$

where $i = (\Delta h)/L$

When hydraulic gradient i=1, the V= k

So, coefficient of permeability (k) may be defined as the average velocity of flow that will occur through the total cross-sectional area of soil under unit hydraulic gradient.

k is normally expressed in cm/sec; m/day or feet/day

k (cm/sec)	Soils type	Drainage conditions
10^1 to 10^2	Clean gravels	Good
10^1	Clean sand	Good
10^{-1} to 10^{-4}	Clean sand and gravel mixtures	Good
10^{-5}	Very fine sand	Poor
10^{-6}	Silt	Poor
10^{-7} to 10^{-9}	Clay soils	Practically impervious

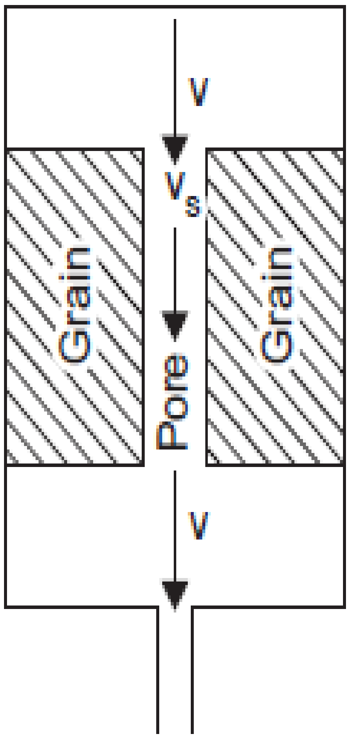
VALIDITY OF DARCY’s LAW

Darcy’s type of flow is stable in character as long as the four basic conditions are always satisfied:

- *The steady state is laminar flow*
- *Hundred percent saturation*
- *Flow fulfilling continuity conditions*
- *No volume changes occur during or as a result of flow.*

Note: For ground water flow occurring in nature and normally encountered in Soil Mech. Problems, the Darcy’s law is generally within validity limits.

Discharge/Superficial Velocity and Seepage Velocity



Since flow takes place through the voids, hence, actual velocity of flow is more than discharge velocity.

By the principle of continuity, the velocity of approach, *v*, *may* be related to the seepage velocity, v_s, *as follow :*

$$q = A \cdot v = A_v \cdot v_s$$

where A_v = area of cross-section of voids

$$\therefore \quad v_s = v \cdot \frac{A}{A_v} = v \cdot \frac{AL}{A_v L} = v \cdot \frac{V}{V_v} = \frac{v}{n}$$

$$v_s = v/n = ki/n$$

$$v_s = ki/n = (k/n)i = k_p \cdot i$$

where *kp, the constant of proportionality, is called the 'Coefficient of Percolation', and is given* by :

$$\mathbf{k_p = k/n}$$

Factors affecting permeability

An equation reflecting the influence of the characteristics of the permeant fluid and the soil on permeability was developed by Taylor (1948) based on Poiseuille's law for laminar flow through a circular capillary tube.

The flow through a porous medium is considered similar to a flow through a bundle of straight capillary tubes. The equation is :

$$k = D_s^2 \cdot \frac{\gamma}{\mu} \cdot \frac{e^3}{(1+e)} \cdot C$$

in which,

k = Darcy's coefficient of permeability

D_s = effective particle-size

γ = unit weight of permeant

μ = viscosity of permeant

e = void ratio

C = shape factor

$$k = D_s^2 \cdot \frac{\gamma}{\mu} \cdot \frac{e^3}{(1+e)} \cdot C$$

Factors affecting permeability

Grain-size

- Equ. 1 suggests that the permeability varies with the square of particle diameter.
- It is logical that the smaller the grain-size the smaller the voids and thus the lower the permeability and vice versa.
- Allen Hazen proposed;

$k = 100\ D_{10}^2$ where D_{10} *is in cm and k is in cm/s.*

Density and Viscosity

Equation 1 indicates that the permeability is influenced by both the viscosity and the unit weight of the permeant fluid.

$$k \;\alpha\; \frac{\gamma}{\mu}$$

γ & η depends on temp, hence k depends on temperature. Since effect of **temp is more on η**, neglecting variation of γ with temp Since

$$k_{27} = k_T \cdot \frac{\mu_T}{\mu_{27}}$$

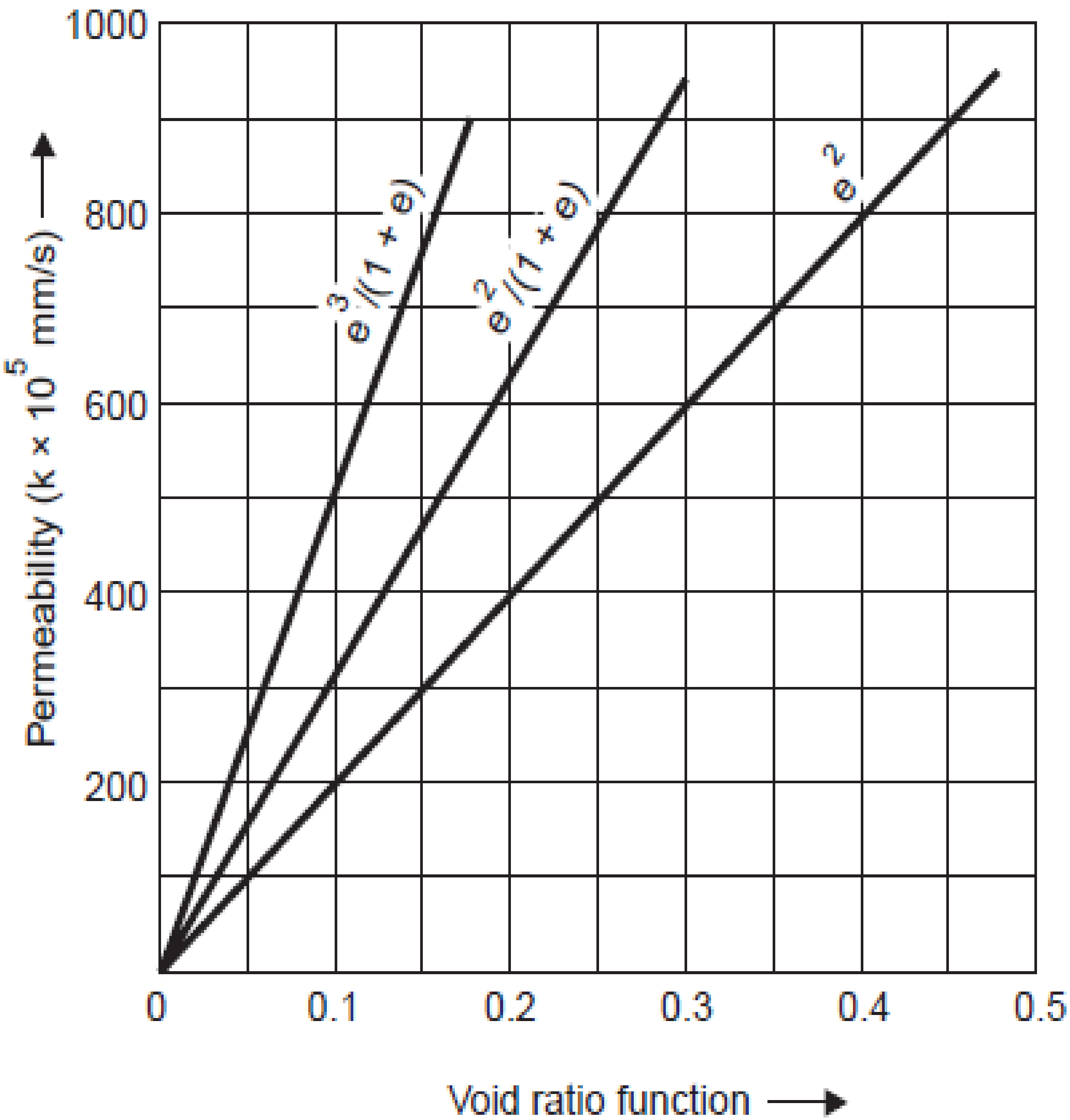
where k_T and μ_T are the permeability of soil and the viscosity of water at the test temperature of t°C and, k_{27} and μ_{27} are the permeability and viscosity at the standard temperature, i.e., 27°C.

$$k = D_s^{\,2} \cdot \frac{\gamma}{\mu} \cdot \frac{e^3}{(1+e)} \cdot C$$

Void Ratio

Equation 1 indicates that a plot of *k versus $e^3/(1 + e)$ should be a straight line.*

*This is **more** **true of coarse grained*** soils since the shape factor *C does not change appreciably with the void* ratio for these soils.



1000
800
600
400
200
0
0.1
0.2
0.3
0.4
0.5
Permeability (k × 10^5 mm/s)
Void ratio function
e^3/(1 + e)
e^2/(1 + e)
e^2

Fabric or Structural Arrangement of Particles

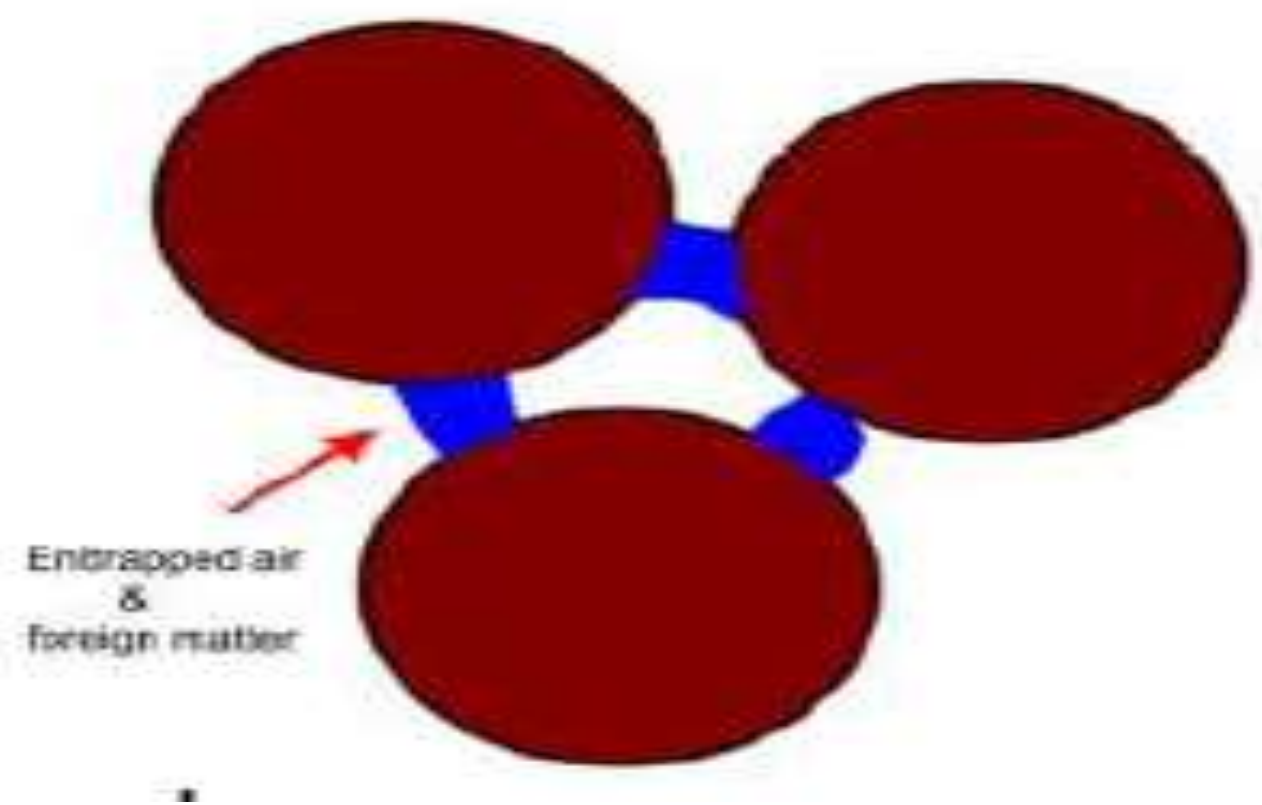
- The fabric or structural arrangement of particles is an important soil characteristic influencing permeability, especially of fine-grained soils.
- At the same void ratio, it is logical to expect a soil in the most flocculated state will have the highest permeability, and the one in the most dispersed state will have the lowest permeability.

Degree of saturation

Higher the degree of saturation, higher will be the permeability. In the case of certain sands the permeability may increase three-fold when the degree of saturation increases from 80% to 100%.

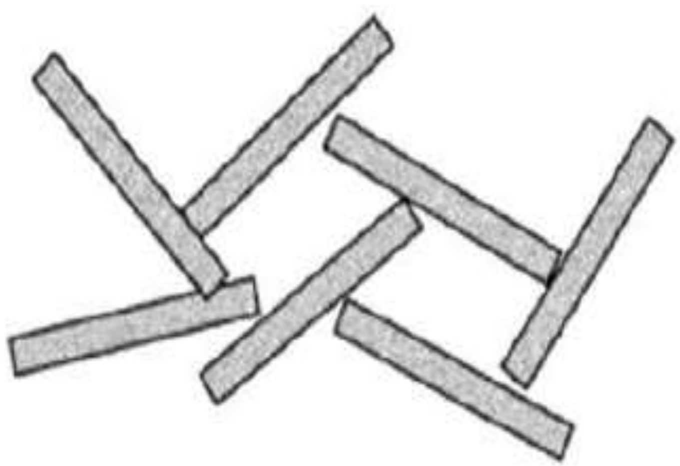
Presence of entrapped air & other foreign matter

The entrapped air and foreign matter will block the voids in soil results in decreasing in permeability

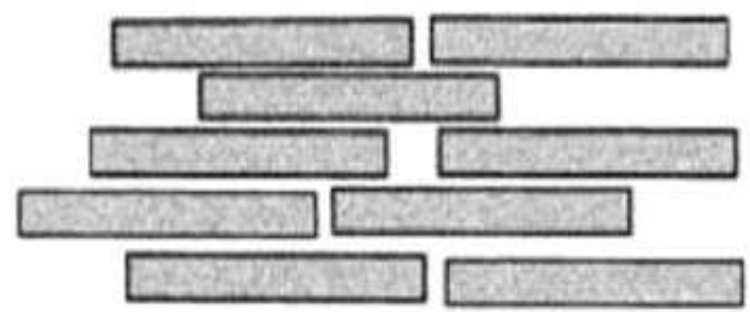


Structural arrangement

For same void ratio the permeability of the soil will be more in flocculated structure as compare to Dispersed structure.



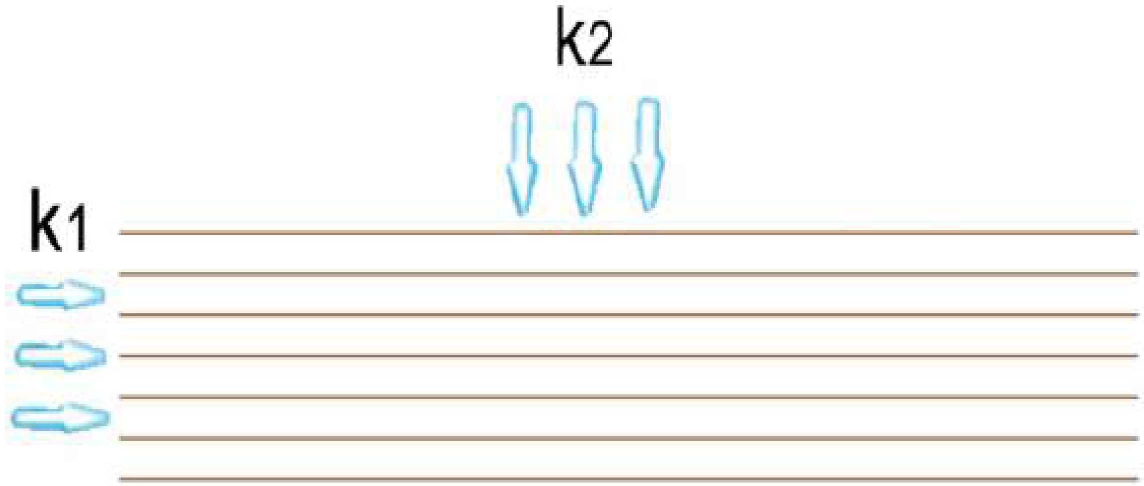
Flocculated structure



Dispersed structure

Stratification of soil

Stratified soil deposits have grater permeability parallel to the plane when compare to perpendicular to the plane.



$k1 > k2$

Laboratory Testing to find coefficient of permeability

Two standard laboratory tests are us ar to determine the coefficient of permeability of soil

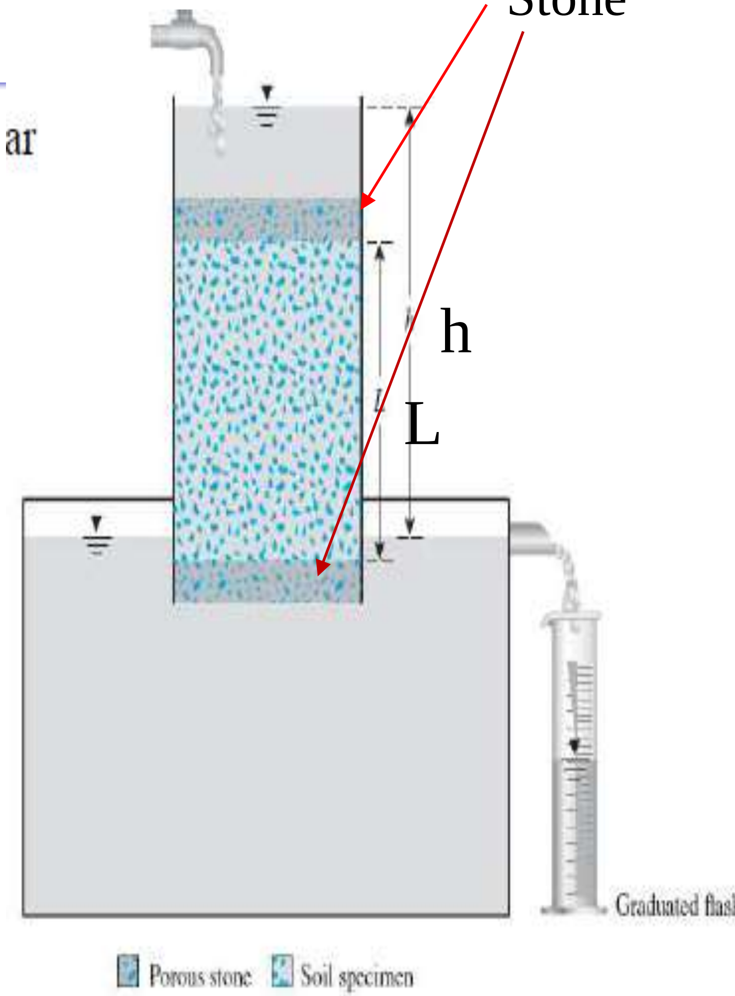
1.Constant Head Test For Sandy Soil Only

If Q is total quantity of flow in time interval t, then from Darcy's law

$$\boxed{Q = Avt = A(k \times i)t}$$

or

$$\boxed{K = \frac{QL}{Aht}}$$



When steady state of flow is reached, the total quantity of water Q in time t is collected in measuring jar.

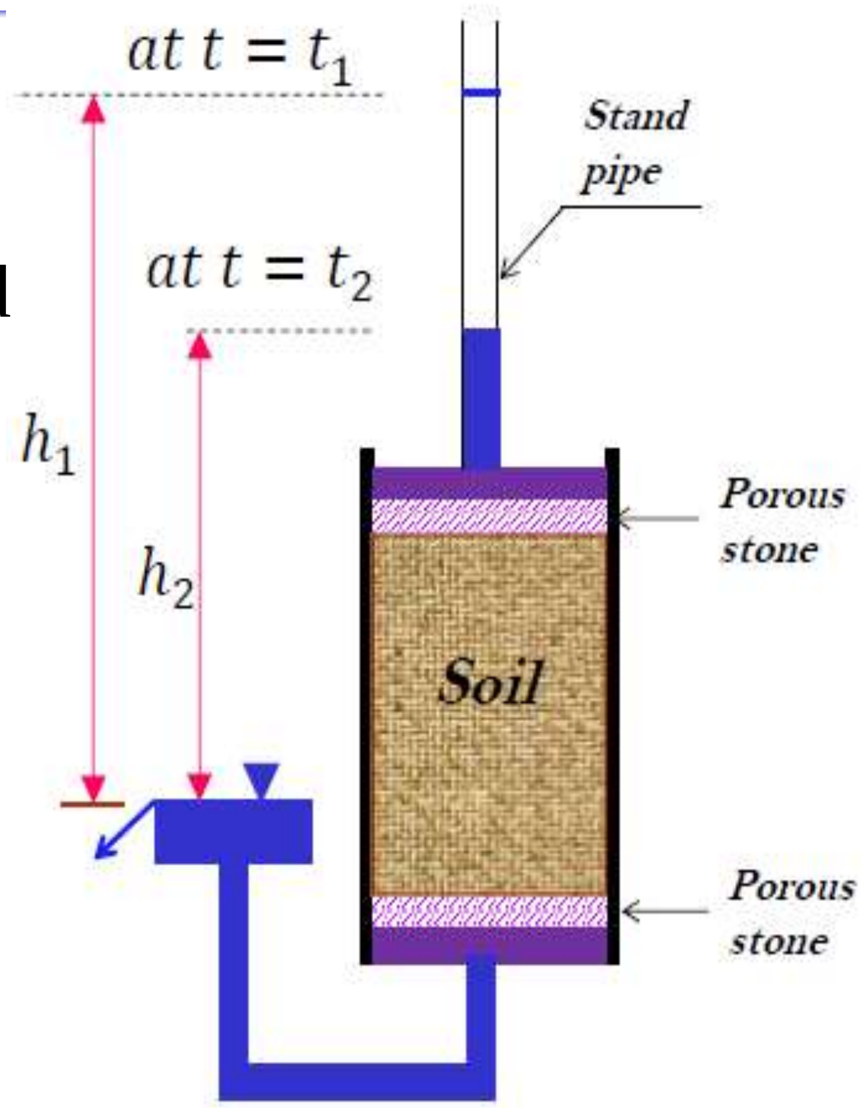
2. Falling Head Permeability test

Let h be the head at any intermediate time interval t and (-dh) be change in head in smaller time interval dt

$$v = -\frac{dh}{dt}$$

The flow into the sample is: $q_{in} = -a\dfrac{dh}{dt}$

a= area of standpipe



From Darcy's law the flow out is

$$q_{out} = k \frac{h}{L} A$$

$$\boxed{q_{out} = q_{int}} \quad \text{or} \quad k \frac{h}{L} A = -a \frac{dh}{dt}$$

Integrating between two time limits, we get

$$k\frac{A}{L}\int_{T_1}^{T_2} dt = a\int_{h_2}^{h_1}\frac{dh}{h}$$

We obtain

- Sign change by changing limit from h_2 to h_1 to h_1 to h_2

$$k = \frac{aL}{A\,\Delta t}\ln\frac{h_1}{h_2}$$

t = time

L = Length of the fine soil

A = cross section area of soil

a= cross section area of tube

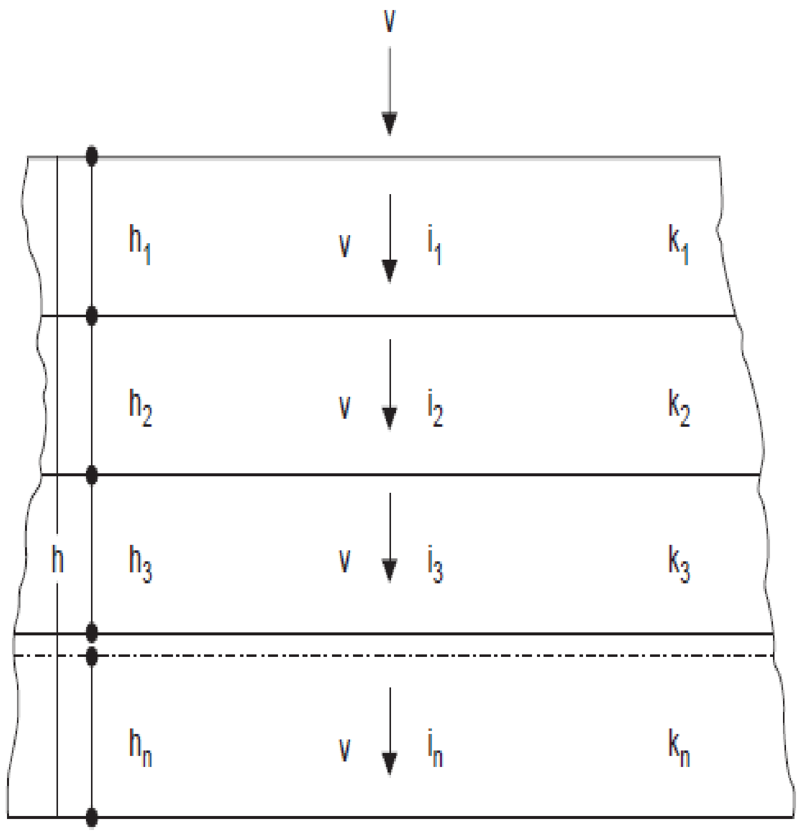
K = Coefficient of permeability

$$k = \frac{2.303\,a\,L}{A\,t}\log\frac{h_1}{h_2}$$

Permeability in Stratified Soil

Flow Perpendicular to the Bedding Planes

In this case, the velocity of flow v, and hence the discharge q, is the same through all the layers, for the continuity of flow.



Let the total head lost be Δh and the head lost in each of the layers be Δh1, Δh2, Δh3, ... Δhn.

Δh = Δh1 + Δh2 + Δh3 + ... Δhn.

The hydraulic gradients are :

$i_1 = \Delta h_1/h_1$; $i_2 = \Delta h_2/h_2$ and $i_n = \Delta h_n/h_n$

Since q is the same in all the layers, and area of cross-section of flow is the same, the velocity is the same in all layers.

Let k_z be the average permeability perpendicular to the bedding planes.

Now $$k_z \,.\, i = k_1 i_1 = k_2 i_2 = k_3 i_3 = \ldots k_n i_n = v$$

$$\therefore \quad k_z \Delta h/h = k_1 \Delta h_1/h_1 = k_2 \Delta h_2/h_2 = \ldots \frac{k_n \Delta h_n}{h_n} = v$$

Since $\Delta h = \Delta h1 + \Delta h2 + \Delta h3 + \ldots \Delta hn$.

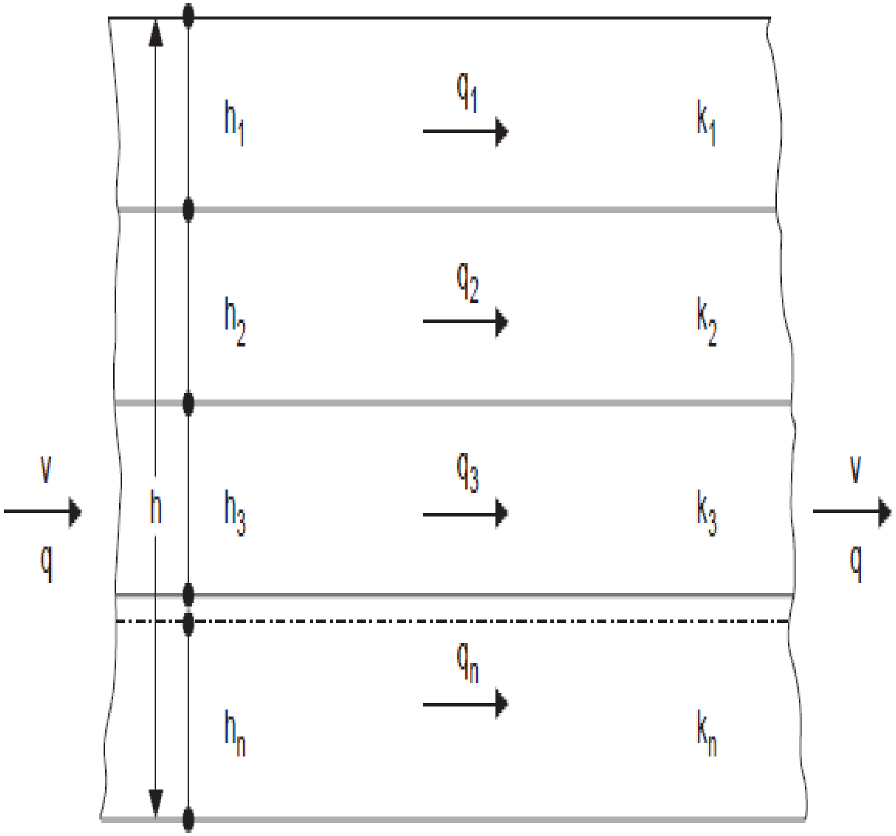
$$vh/k_z = vh_1/k_1 + vh_2/k_2 + \ldots + vh_n/k_n$$

$$k_z = \frac{h}{(h_1/k_1 + h_2/k_2 + \ldots + h_n/k_n)}$$

Flow Parallel to the Bedding Planes

The discharge through the entire deposit is equal to the sum of the discharge through the individual layers.

Assuming k_x to be the average permeability of the entire deposit parallel to the bedding planes, and applying the equation



$$q = q_1 + q_2 + \dots + q_n,$$

we have, $k_x \cdot ih = k_1 i \cdot h_1 + k_2 i \cdot h_2 + \ldots k_n i \cdot h_n.$

$$\therefore \qquad k_x = \left(\frac{k_1 h_1 + k_2 h_2 + \ldots k_n h_n}{h} \right)$$

where $h = h_1 + h_2 + \ldots + h_n.$

SEEPAGE

- Defined as the flow of water/fluid through a soil under a hydraulic gradient.

- The interaction between soils and percolating water has an important influence on:

  1 . The design of foundations and earth slopes,

  2. The quantity of water that will be lost by percolation through a water.

- The pressure that is exerted on the soil due to the seepage of water is called the *seepage force*.

- If h is the hydraulic head or head loss causes the water to flow through a soil mass of thickness, z, the seepage pressure
  - $p_s = h\,\gamma_w = (h/z)\,z\,\gamma_w = i\,z\,\gamma_w$

Hydraulic gradient

Length over which h is loss

The total seepage force transmitted to soil by seepage

$$= p_s \,.\, A = i\,z\,\gamma_w\,A$$

And seepage force per unit volume $= i\,z\,\gamma_w\,A\,/\,z\,A$

$= i\,\gamma_w$

BASIC EQUATION FOR SEEPAGE/ Laplace's Equation of continuity

The following assumptions are made:

1. Darcy's law is valid for flow through soil.

2. The hydraulic boundary conditions are known at entry and exit of the fluid (water) into the porous medium (soil).

3. Water is incompressible.

4. The porous medium is incompressible.

These assumptions have been known to be very nearly or precisely valid.

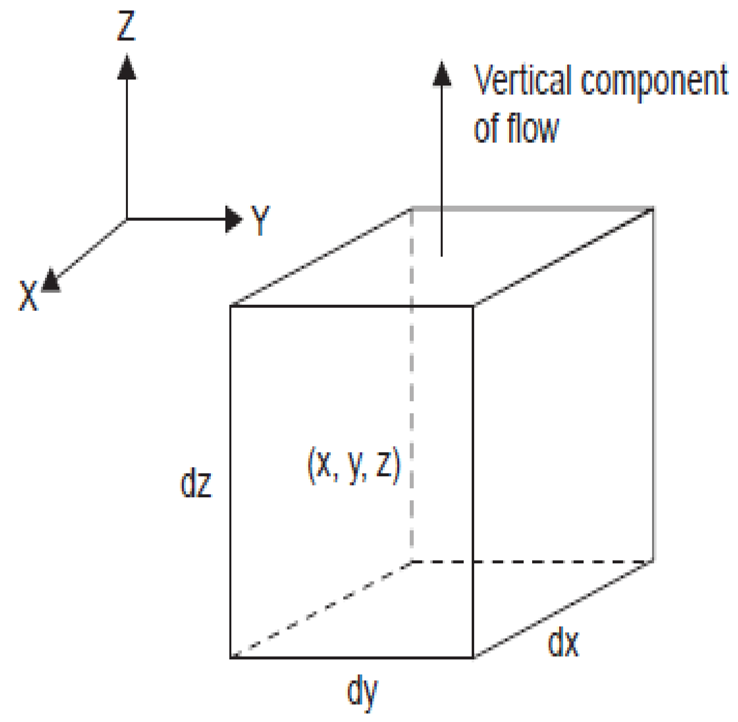
- Let us consider an element of soil through which laminar flow of water is occurring

V_x and V_z are components of discharge velocity in the horizontal and vertical direction.
The rate of flow of water into the elementary block in the horizontal direction is $V_x\, d_z\, d_y$ and in the vertical direction is $V_z\, d_x\, d_y$.
Again the rate of outflow from the block in horizontal and vertical directions are

$$V_x + (\delta V_x/\delta_x)\, d_z\, d_y$$

& $$V_z + (\delta V_z/\delta_z)\, d_x\, d_y$$

Assuming water is incompressible and no change in soil mass occurs, then total rate of inflow should be equal to total rate of outflow.



$$[V_x + (\delta V_x/\delta_x)\, d_z\, d_y] + [V_z + (\delta V_z/\delta_z)\, d_x\, d_y] - [V_x\, d_z\, d_y + V_z\, d_x\, d_y] = 0$$

Or $\delta V_x/\delta_x + \delta V_z/\delta_z = 0$

From Darcy's Law $V_x = K_x\, i_x = K_x\, (\delta h/\delta_x)$ & $V_z = K_z\, i_z = = K_z\, (\delta h/\delta_z)$

For isotropic soil $K_x = k_z$

$$\delta^2 h/\delta^2{}_x + \delta^2 h/\delta^2{}_z = 0$$

known as Continuity Equation.

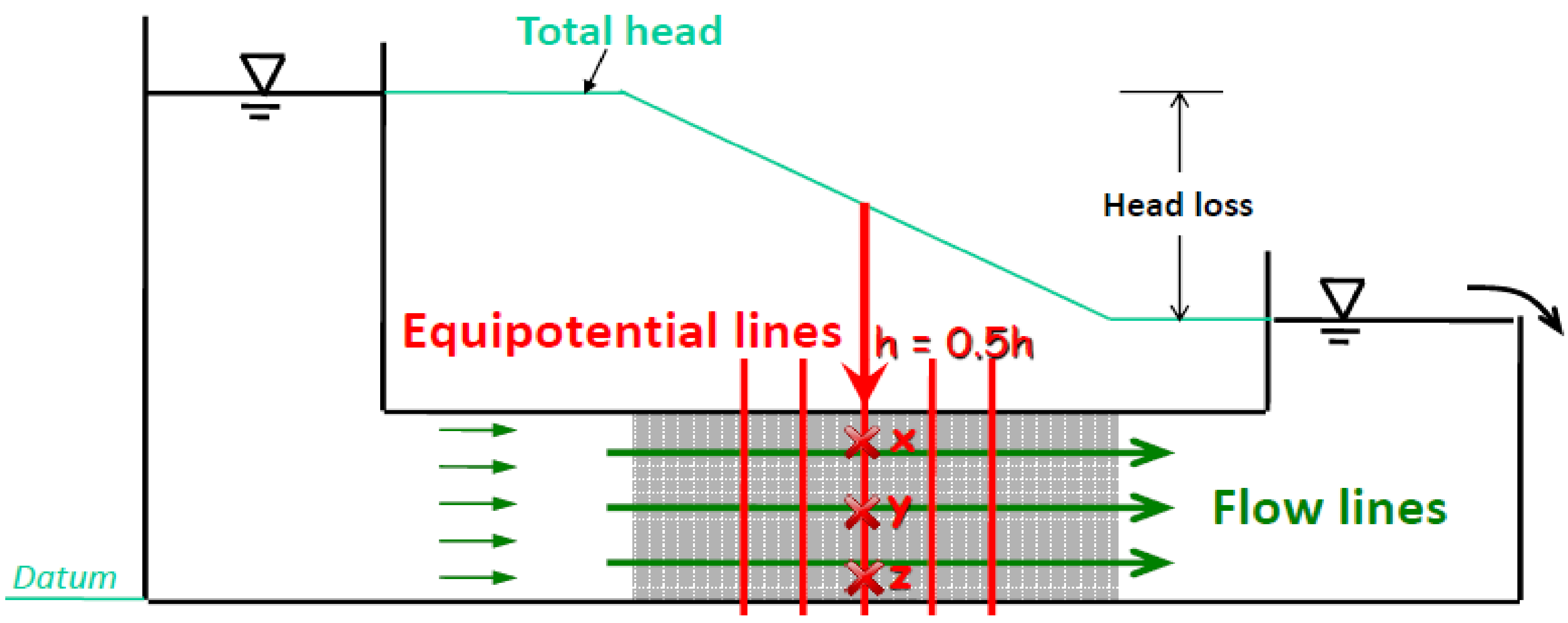
The flow net which consists of two sets of curves – a series of flow lines and of equil-potential lines–is obtained merely as a solution to the Laplace's equation.

FLOW NETS

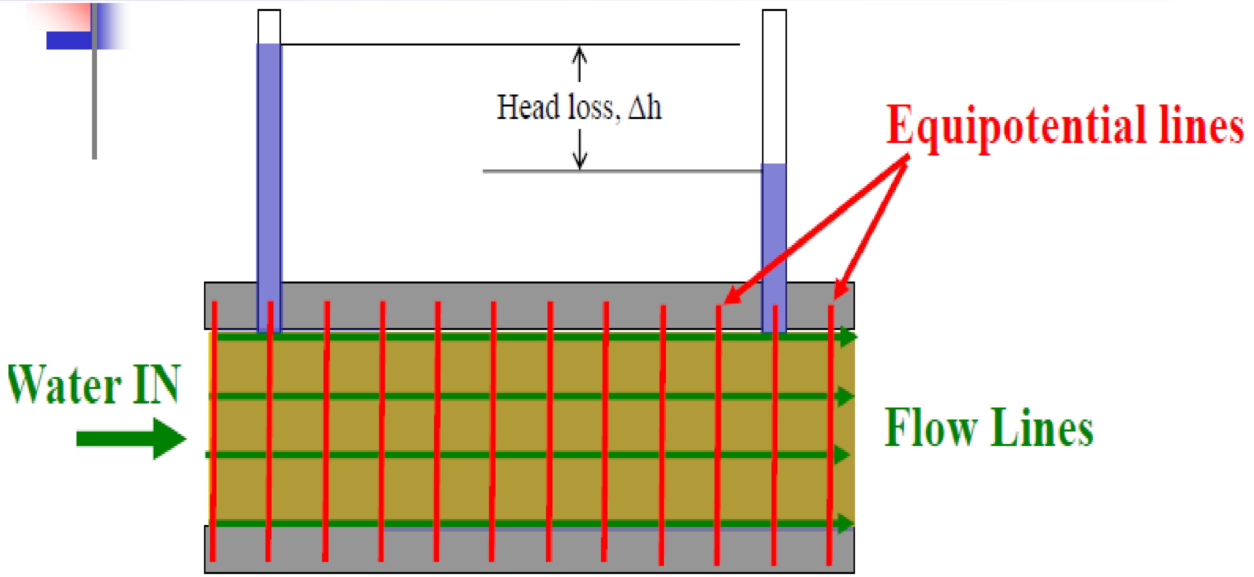
A *Flow Net consists of two groups of curves:*

Flow lines: Flow lines (aka stream lines) represent the path that a particle of water takes as it travels through the soil mass.

Equipotential lines: Equipotential lines are lines that pass through points of equal head.



DRAWING OF FLOW NETS



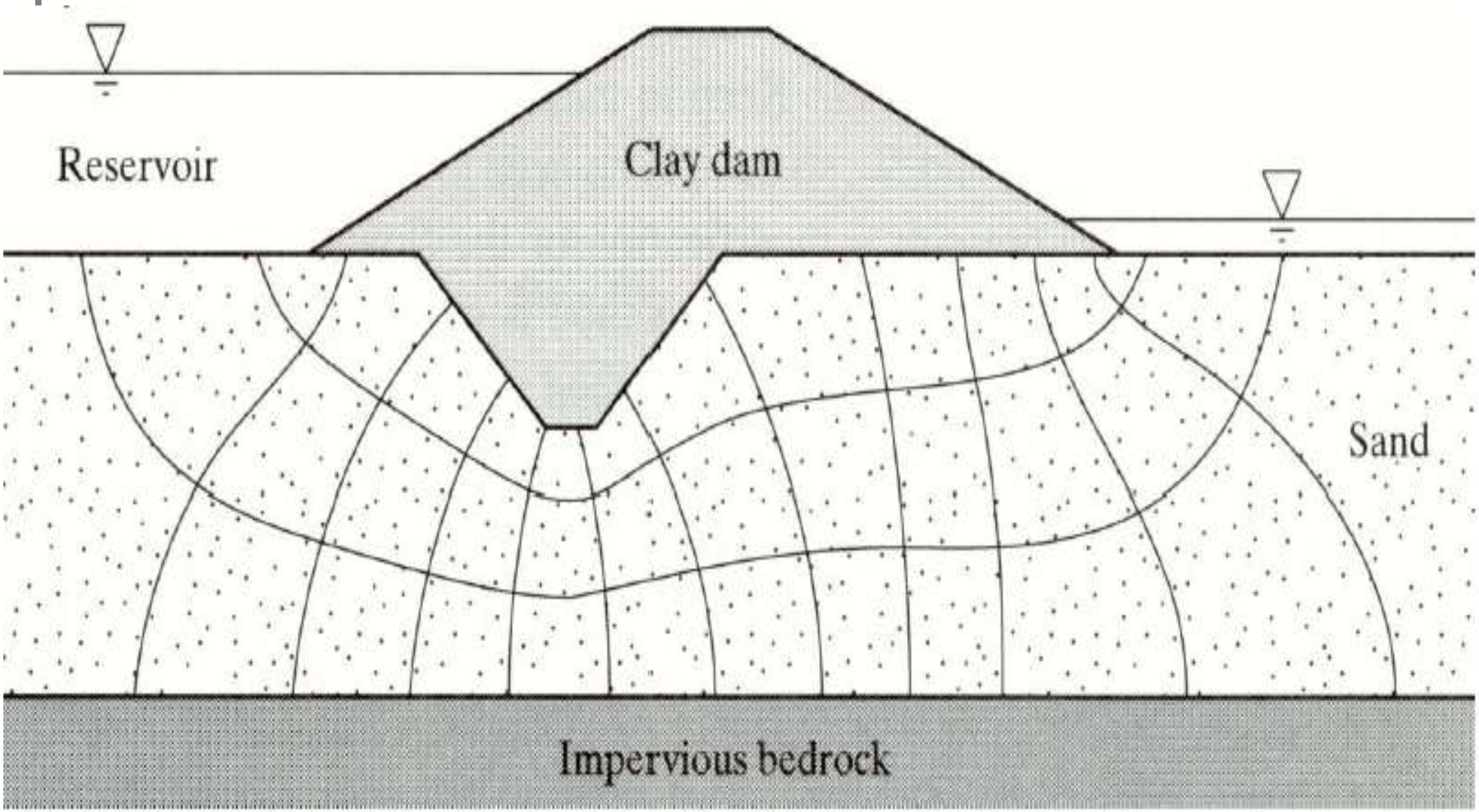
1. Flow lines and equipotential lines are at *right angles to one another.*
2. Flow lines are ‖ *to no flow boundaries*.
3. Equipotential lines are ‖ *to permeable boundaries*.
4. *Discharge through each flow path is equal.*
5. *Head loss through each equipotential space is equal.*

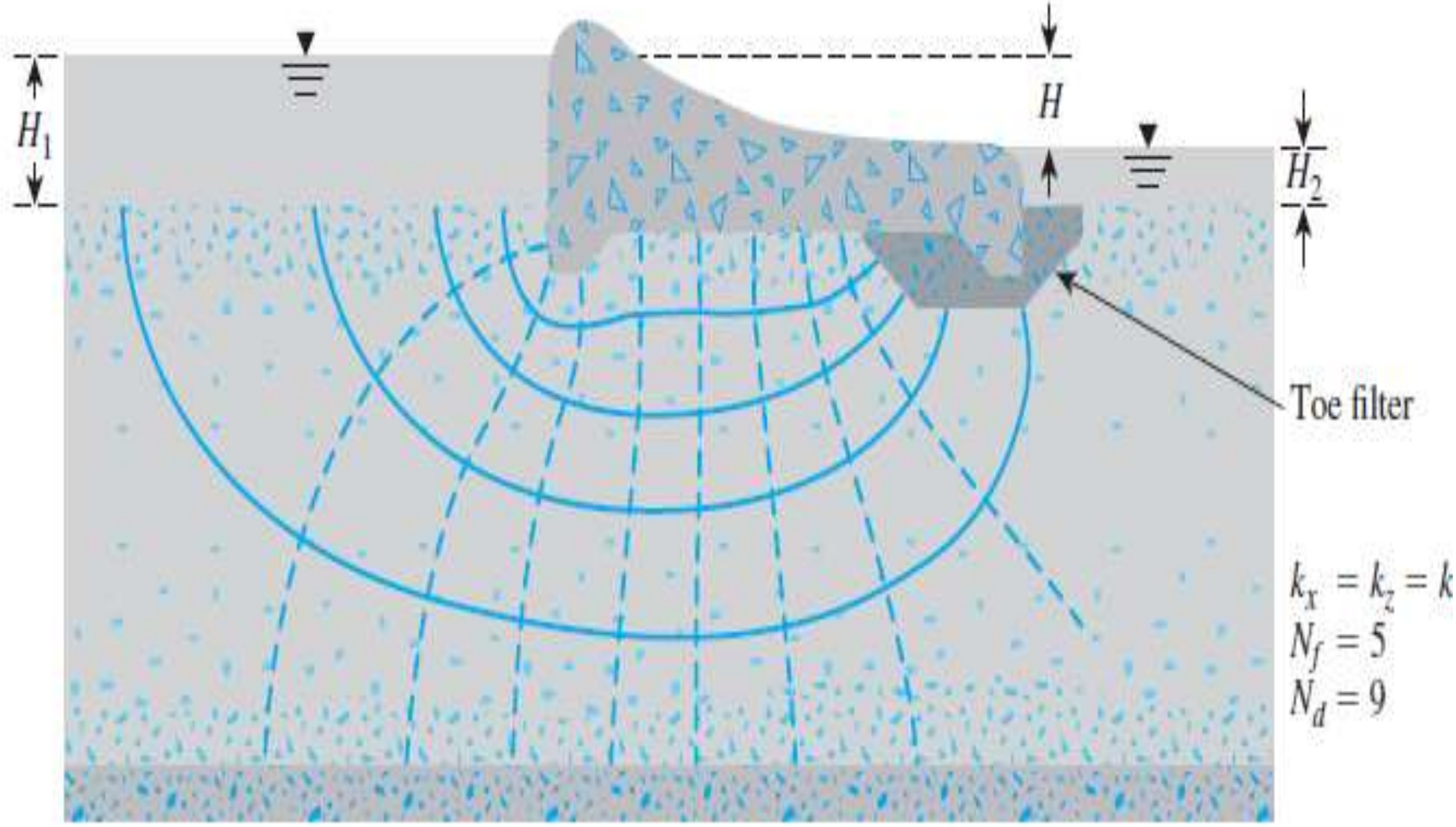
RULES FOR DRAWING FLOW NETS

1) All *impervious boundaries are flow lines*.
2) All *permeable boundaries are equipotentials*.
3) All equipotentials are at *right angles to flow lines*.
4) All parts of the flow net must have the *same geometric proportions (e.g. square or similarly shaped rectangles).*
5) Just like contour lines, flow lines cannot cross other flow lines & equipotential lines cannot cross other equipotential lines.
6) Good approximations can be obtained with 4 - 6 flow channels.

More accurate results are possible with higher numbers of flow channels, but the time taken goes up in proportion to the number of channels.

Sample flownet





Flow net under a dam with toe filter

METHODS OF OBTAINING FLOW NETS

The following methods are available for the determination of flow nets:

1. Graphical solution by sketching
2. Mathematical or analytical methods
3. Numerical analysis
4. Models
5. Analogy methods

All the methods are based on Laplace's equation.

1. Graphical Solution by Sketching

A flow net for a given cross-section is obtained by first transforming the cross-section, and then sketching by trial and error, taking note of the boundary conditions.

The properties of flow nets such as the orthogonality of the flow lines and equipotential lines, and the spaces being elementary squares, and the various rules concerning boundary conditions and smooth transitions must be observed.

Sketching by trial and error was first suggested by Forchheimer (1930) and further developed by A. Casagrande (1937).

The following suggestions are made by Casagrande for the benefit of the sketcher.

(*a*) *Every opportunity to study well-constructed flow nets should be utilised to get the* feel of the problem.
(*b*) *Four to five flow channels are usually sufficient for the first attempt.*
(*c*) *The entire flow net should be sketched roughly before details are adjusted.*
(*d*) *The fact that all transitions are smooth and are of elliptical or parabolic shape should* be borne in mind.
(*e*) *The boundary flow lines and boundary equipotentials should first be recognised and* sketched.

2. Mathematical or Analytical Methods

In a few relatively simple case the boundary conditions may be expressed by equations and solutions of Laplace’s equation may be obtained by mathematical procedure.

This approach is not popular because of the complexity of mathematics even for relatively simply problems.

Perhaps the best known theoretical solution was given by Kozeny (1933) and later extended further by A. Casagrande, for flow through an earth dam with a filter drain at the base towards the downstream side.

Another problem for which a theoretical solution is available is a sheet pile wall
(Harr, 1962).

3. Numerical Analysis

Laplace’s equation for two-dimensional flow can be solved by numerical techniques in case the mathematical solution is difficult.

Relaxation methods involving successive approximation of the total heads at various points in a mesh or net work are used.

The Laplace’s differential equation is put in its finite difference form and a digital computer is used for rapid solution.

4. Models

A flow problem may be studied by constructing a scaled model and analysing the flow in the model.

Earth dam models have been used quite frequently for the determination of flow lines.

Such models are commonly constructed between two parallel glass or lucite sheets. By the injection of spots of dye at various points, the flow lines may be traced.

This approach facilitates the direct determination of the top flow line.

Piezometer tubes may be used for the determination of the heads at various points.

5. Analogy Methods

Lapalce's equation for fluid flow also holds for electrical and heat flows.

The use of electrical models for solving complex fluid flow problems in more common.

In an electrical model, voltage corresponds to total head, current to velocity and conductivity to permeability.

Ohm's Law is analogous to Darcy's Law. Measuring voltage, one can locate the equi-potentials.

The flow pattern can be sketched later.

SEEPAGE CALCULATION

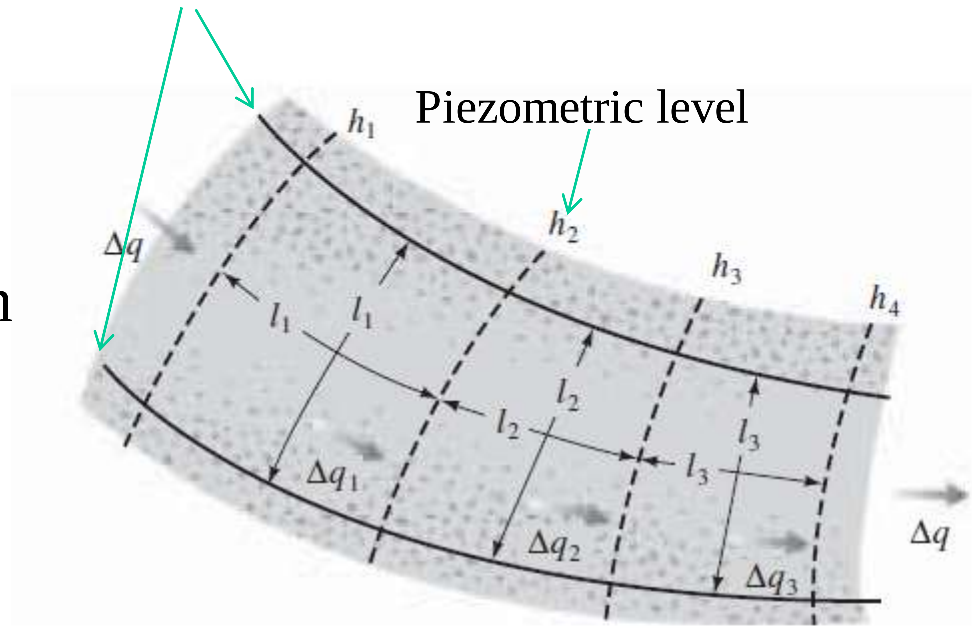
Since there is no flow across flowline

$\Delta q_1 = \Delta q_2 \ldots\ldots = \Delta q$

From Darcy's law,

The rate of flow KiA

Rate of flow per unit width



$$\Delta q = K\left(\frac{h_1 - h_2}{l_1}\right) l_1$$

$$= K\left(\frac{h_2 - h_3}{l_2}\right) l_2$$

$$= K\left(\frac{h_3 - h_4}{l_3}\right) l_3$$

Above Equation shows that if the flow elements are drawn as approximate squares, then the drop in the piezometric level between any two adjacent equipotential lines is the same. This is called the *potential drop*. *Thus*

$$h_1 - h_2 = h_2 - h_3 = h_3 - h_4 = \cdots = \frac{H}{N_d}$$

and

$$\Delta q = k\frac{H}{N_d}$$

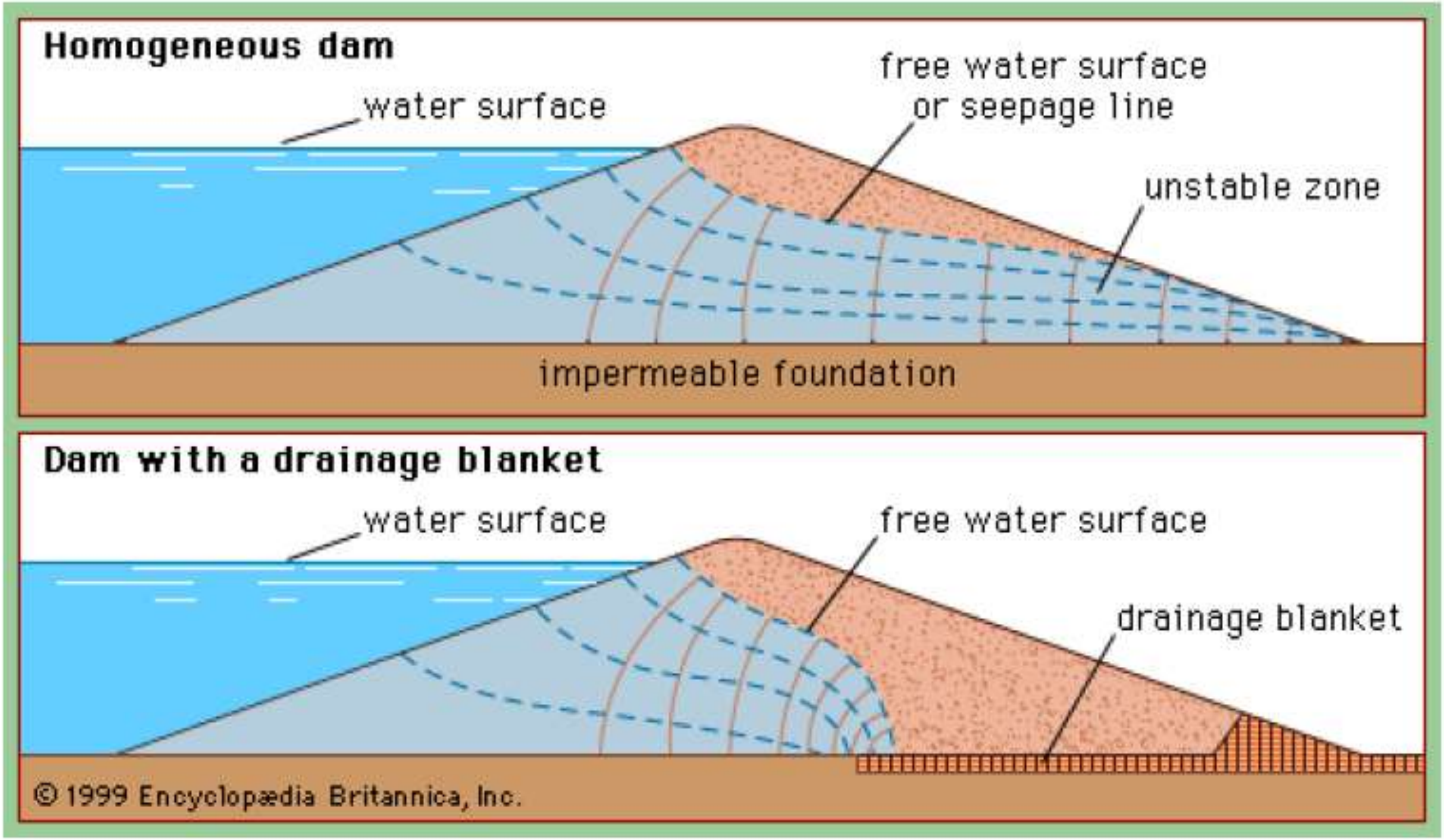
Diff of head between u/s and d/s sides

No of potential drop

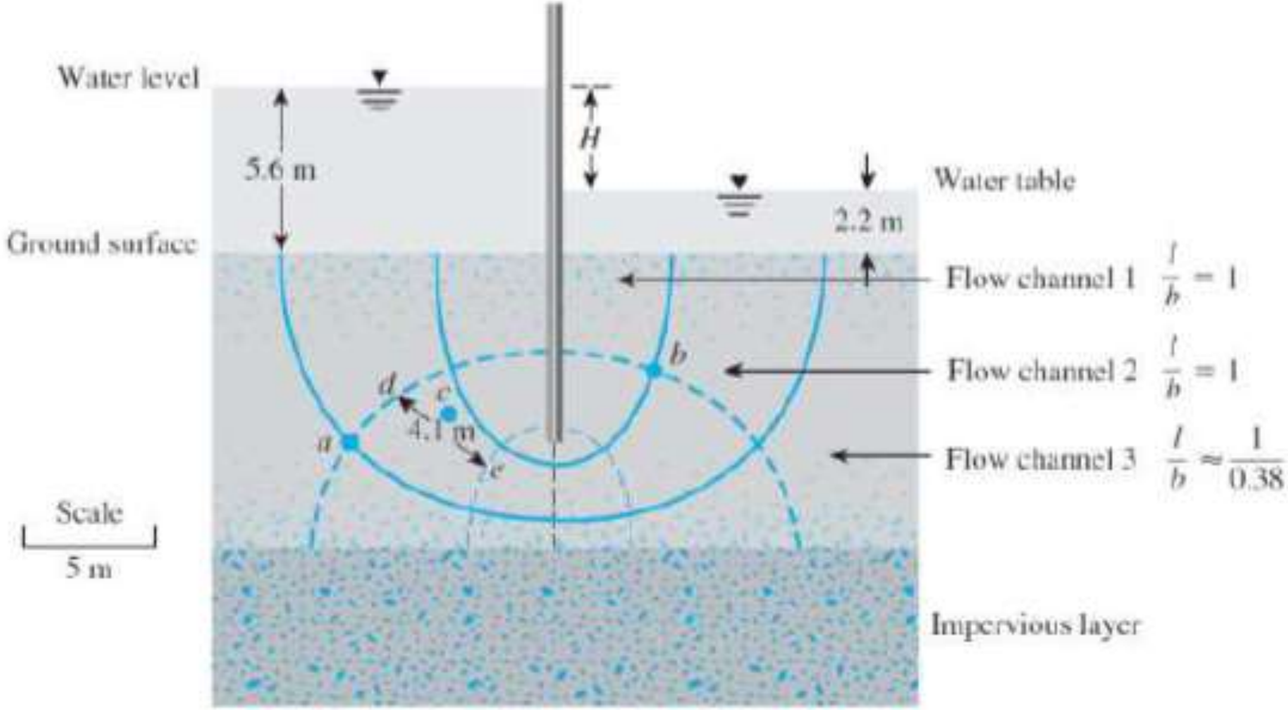
If the number of flow channels in a flow net is equal to N_f*, the total rate of flow* through all the channels per unit length can be given by

$$q = k\frac{HN_f}{N_d}$$

SEEPAGE AND FLOW NET THROUGH DAM



FLOW CALCULATIONS



$$\textit{Flow through each flow channel}, \Delta q = \frac{k \cdot H}{N_d}$$

k = permeability/hydraulic conductivity of soil

H = Head loss

N_d = Number of potential drops

$$\boxed{\textit{Total flow} = \frac{k \cdot H}{N_d} \times N_f}$$

$N_f = 1$ → for square channel

$N_f = b/l$ → for rectangular channel

Previous Figure shows a flow net for seepage around a single row of sheet piles. Note that flow channels 1 and 2 have square elements. Hence, the rate of flow through these two channels can be written as

$$\Delta q_1 + \Delta q_2 = \frac{k}{N_d}H + \frac{k}{N_d}H = \frac{2kH}{N_d}$$

However, flow channel 3 has rectangular elements. These elements have a width-to-length ratio of about 0.38; hence

$$\Delta q_3 = \frac{k}{N_d}H(0.38)$$

So, the total rate of seepage can be given as

$$q = \Delta q_1 + \Delta q_2 + \Delta q_3 = 2.38\frac{kH}{N_d}$$

Practice Problem

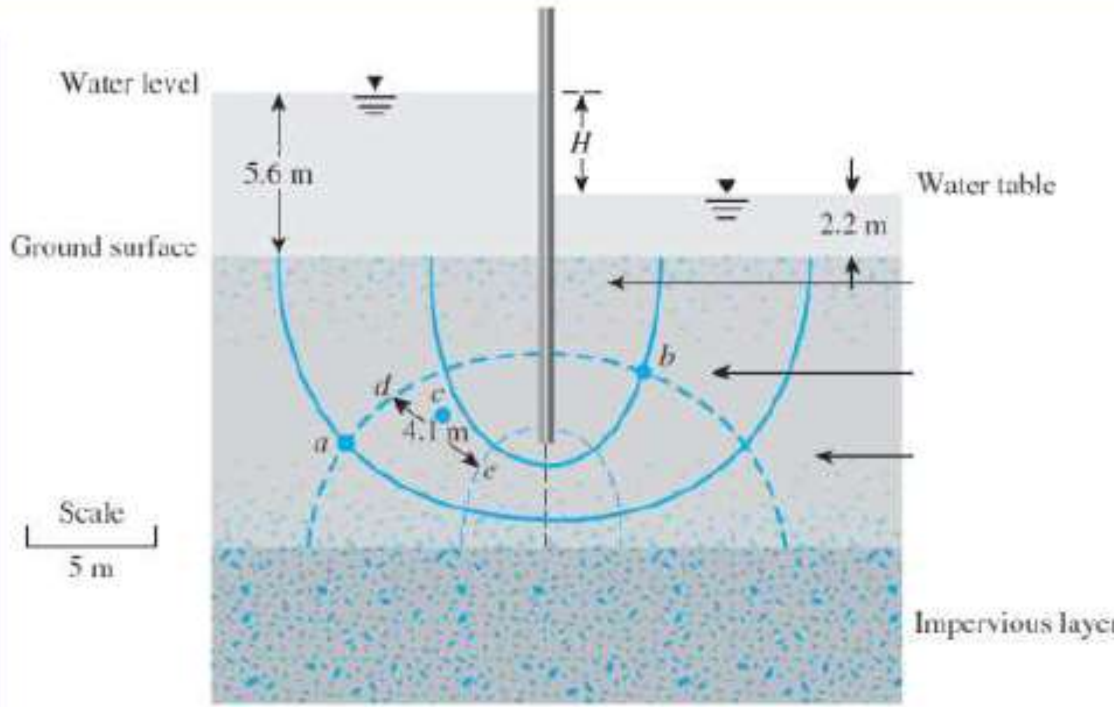
A flow net for flow around a single row of sheet piles in a permeable soil layer is shown in Figure 8.7. Given that $k_x = k_z = k = 5 \times 10^{-3}$ cm/sec, determine

a. How high (above the ground surface) the water will rise if piezometers are placed at points *a* and *b*.
b. The total rate of seepage through the permeable layer per unit length
c. The approximate average hydraulic gradient at *c*.

$$\boxed{Total\ \ flow = \frac{k \cdot H}{N_d} \times N_f}$$

$N_f = 1 \rightarrow$ for square channel
$N_f = b/1 \rightarrow$ for rectangular channel

k = permeability of soil
H = Head loss
N_d = Number of potential drops



Solution

Part a

From Figure 8.7, we have $N_d = 6$, $H_1 = 5.6$ m, and $H_2 = 2.2$ m. So the head loss c each potential drop is

$$\Delta H = \frac{H_1 - H_2}{N_d} = \frac{5.6 - 2.2}{6} = 0.567 \text{ m}$$

At point a, we have gone through one potential drop. So the water in the piezome ter will rise to an elevation of

$$(5.6 - 0.567) = \mathbf{5.033 \text{ m above the ground surface}}$$

At point b, we have five potential drops. So the water in the piezometer will ris to an elevation of

$$[5.6 - (5)(0.567)] = \mathbf{2.765 \text{ m above the ground surface}}$$

Part b

From Eq. (8.25),

$$q = 2.38 \frac{k(H_1 - H_2)}{N_d} = \frac{(2.38)(5 \times 10^{-5} \text{m/sec})(5.6 - 2.2)}{6}$$
$$= \mathbf{6.74 \times 10^{-5} m^3/sec/m}$$

Part c

The average hydraulic gradient at c can be given as

$$i = \frac{\text{head loss}}{\text{average length of flow between } d \text{ and } e} = \frac{\Delta H}{\Delta L} = \frac{0.567 \text{ m}}{4.1 \text{ m}} = \mathbf{0.138}$$

(*Note*: The average length of flow has been scaled.)

Piping

Defined as movement of soil particles by percolating water leading to development of channel

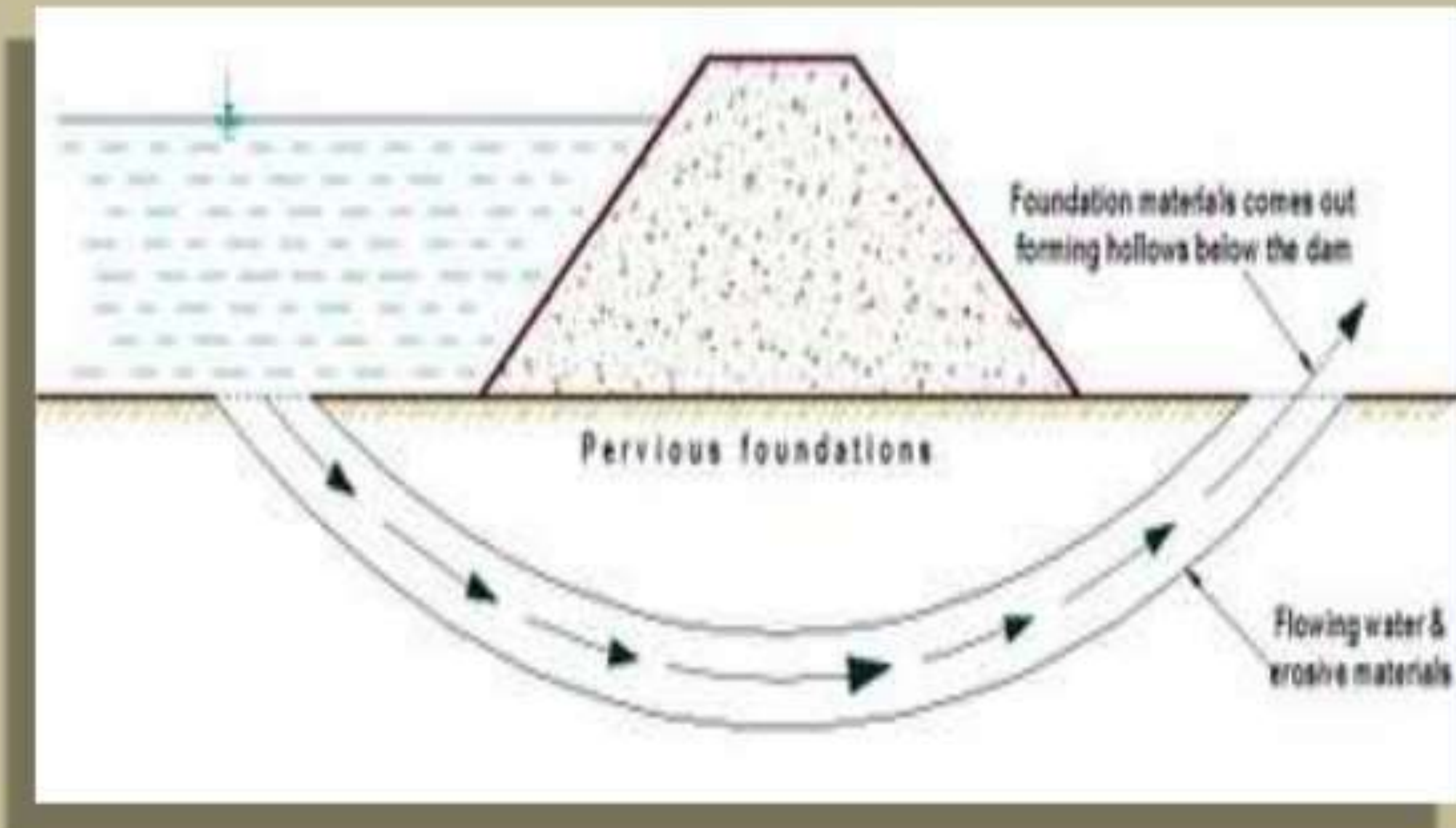
Heave Piping

When the seepage pressure of upward percolating water equal or exceeds the pressure due to submerged weight of soil at a certain level, quick condition develops, leading the blowing of soil by leaving water creating a hole/pipe in soil mass.

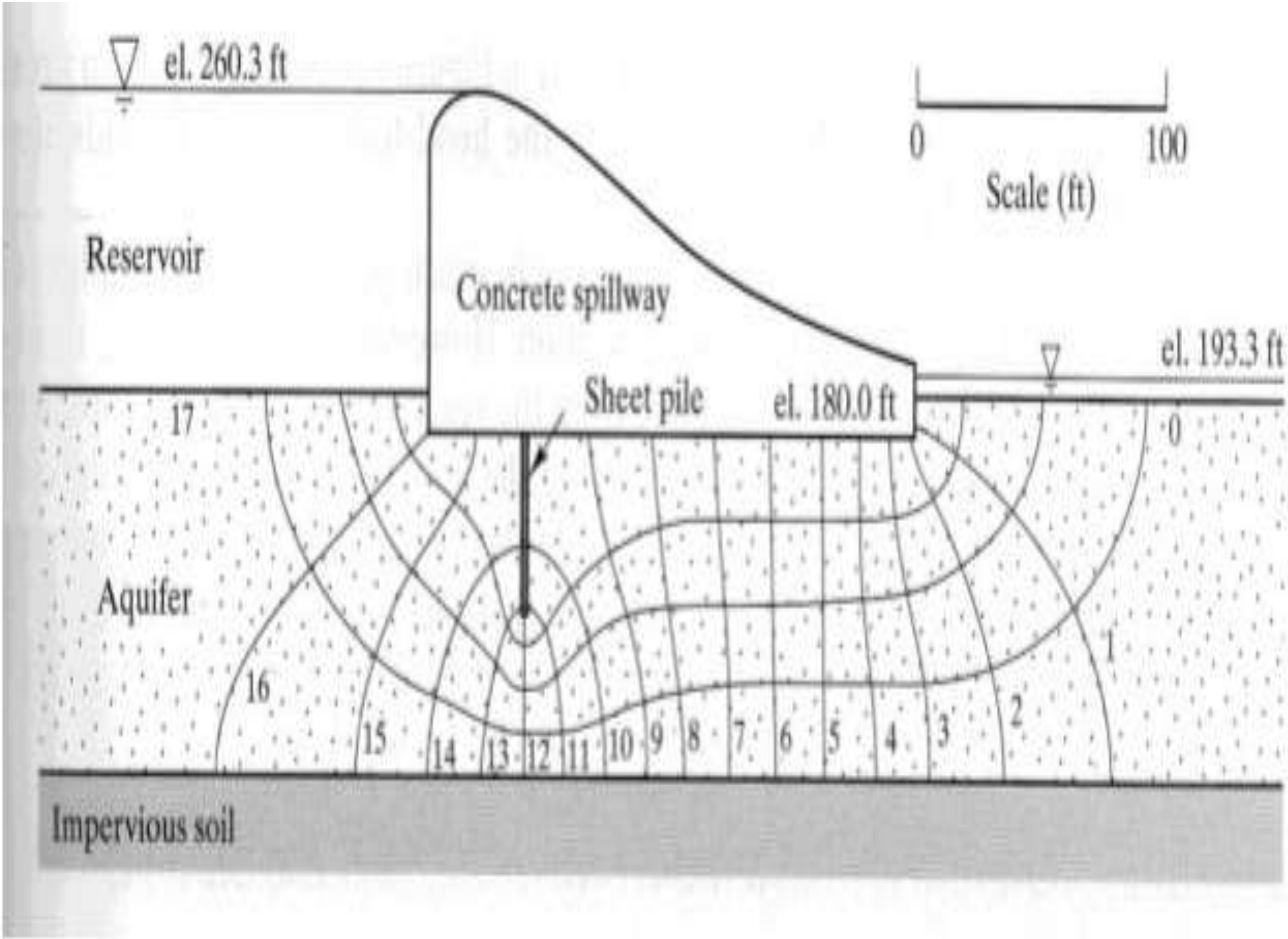
Backward erosion Piping

Starts with removal of soil at the point of exit of flowlines. Watch https://youtu.be/qEAU8abZ-uY

Piping through Foundations



Failure of Teton dam, Idaho, USA


el. 260.3 ft
0
100
Scale (ft)
Reservoir
Concrete spillway
el. 193.3 ft
Sheet pile
el. 180.0 ft
17
0
Aquifer
1
16
2
15
14
13
12
11
10
9
8
7
6
5
4
3
Impervious soil

Piping Prevention

Piping can be avoided by lengthening the flowpaths of water within the dam and its foundations.

This decreases the hydraulic gradient of the water flow and hence its velocity.

The flowpaths can be increased by: Cutoff walls.

Seepage path may be increased by providing *sheet pile, pile walls and other thin cutoffs, cutoff trenches, upstream impervious blanket .*

Piping Prevention (contd.)

By reducing Seepage

Embankment zoning (providing impervious zone or a core wall) reduces seepage within an earthdam.

By controlling the exit of seepage and reducing uplift pressure

Downstream drainage blanket, toe drain and drainage trenches are used to control the exit of seepage and allow its safe escape with out inducing piping.

Providing additional stabilizing load

Seepage may be prevented by covering the area with blanket of heavy coarse graded material called protective filters.

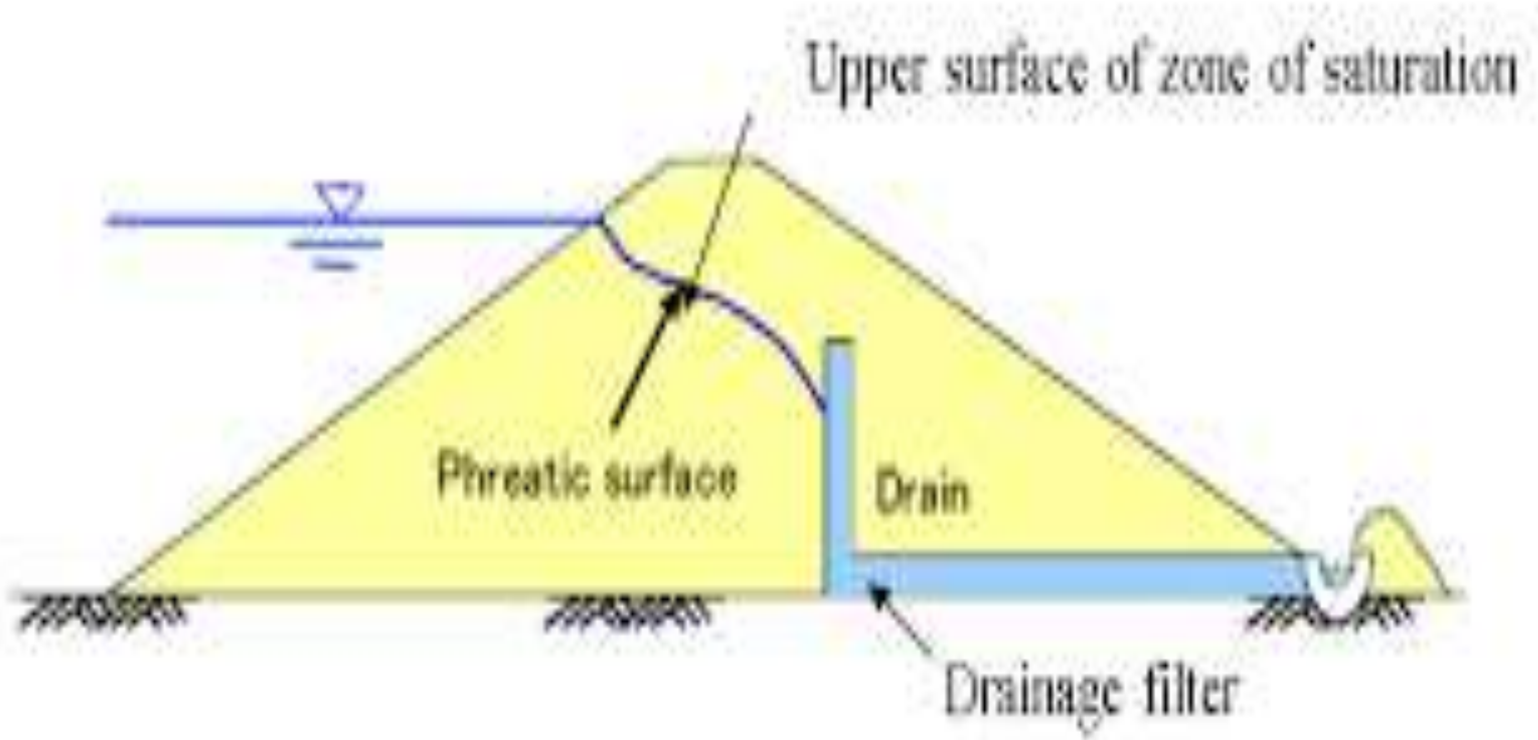


Fig. 24.1. Homogeneous dam showing filter at the downstream side.

(Source: Kunitomo, 2000)

Drainage Filter Design

- When seepage water flows from a soil with relatively fine grains into a coarser material, there is danger that the fine soil particles may wash away into the coarse material. Over a period of time, this process may clog the void spaces in the coarser material.

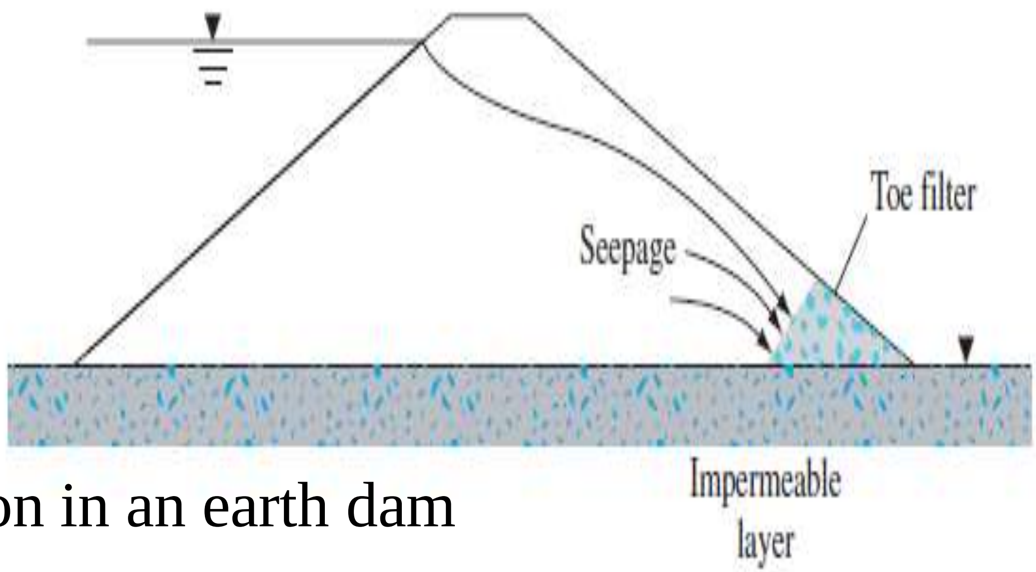
- Hence, the grain-size distribution of the coarse material should be properly manipulated to avoid this situation. A properly designed coarser material is called a *filter.*

.

For proper selection of the filter material, two conditions should be kept in mind:

Condition 1: The size of the voids in the filter material should be small enough to hold the larger particles of the protected material in place.

Condition 2: The filter material should have a high hydraulic conductivity to prevent buildup of large seepage forces and hydrostatic pressures in the filters.



Steady-state seepage condition in an earth dam which has a toe filter

FILTER DESIGN (Contd.)

Terzaghi and Peck (1948), for example, proposed the following set of criteria for an effective soil

$$\frac{D_{15(F)}}{D_{85(BS)}} < 4 \quad \text{(to prevent the filter soil from being washed out)}$$

and

$$\frac{D_{15(F)}}{D_{15(BS)}} > 4 \quad \text{(to ensure a high rate of flow of water)}$$

where F denotes filter and BS is the base soil.

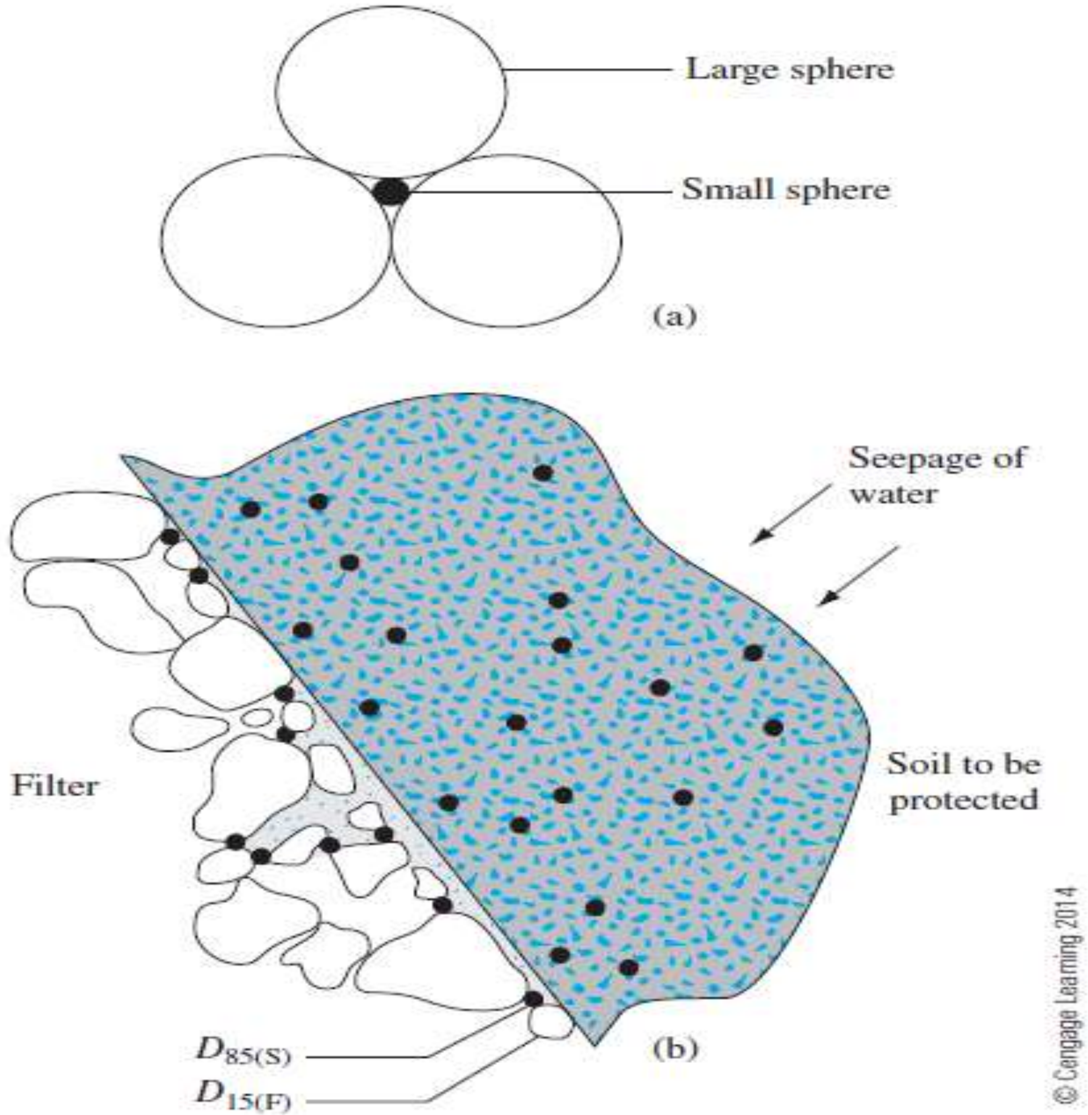
FILTER DESIGN (Contd.)

Most filters are multi layers of sand and gravel. These filters are difficult to construct, and their efficiency is easily compromised by even a small amount of fines.

Geotextiles—a permeable, polymeric material—have replaced soil filters in many construction applications.

Construction is simple and cost-effective compared with soil filters.

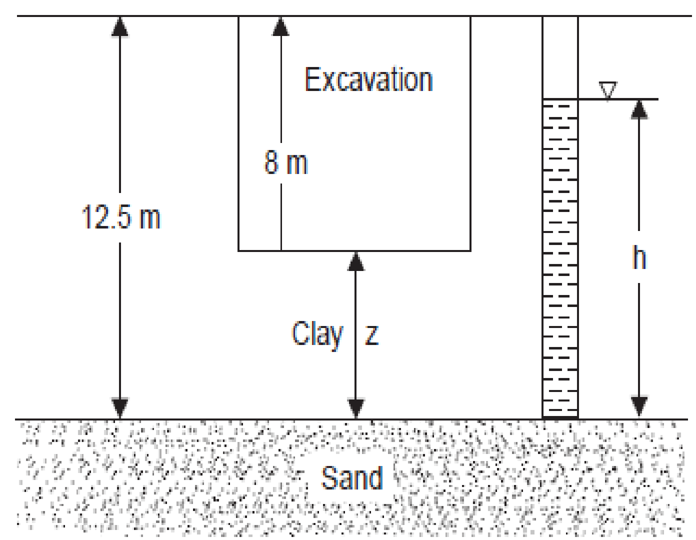
Generally, the filter is about 10 to 20 times more permeable than the soil.



(a) Large spheres with diameters of 6.5 times the diameter of the small sphere; (b)boundary between a filter and the soil to be protected

Exercise Problem

A large excavation was made in a stratum of stiff clay with a saturated unit weight of 18.64 kN/m^3. When the depth of excavation reached 8 m, the excavation failed as a mixture of sand and water rushed in. Subsequent borings indicated that the clay was underlain by a bed of sand with its top surface at a depth of 12.5 m. To what height would the water have risen above the stratum of sand into a drill hole before the excavation was started ?



the effective stress at the top of sand stratum goes on getting reduced as the excavation proceeds due to relief of stress, the neutral pressure in sand remaining constant.

The excavation would fail when the effective stress reached zero value at the top of sand.

Effective stress at the top of sand stratum,

$$\bar{\sigma} = z \cdot \gamma_{sat} - h.\gamma_w$$

If this is zero,

$$h\gamma_w = z \cdot \gamma_{sat}$$

$$h = \frac{z \cdot \gamma_{sat}}{\gamma_w} = \frac{(12.5 - 8) \times 18.64}{9.81} = 8.55 \text{ m}$$

Therefore, the water would have risen to a height of **8.55 m above the stratum of sand** into the drill hole before excavation under the influence of neutral pressure.

Compaction (Mechanical Modification)

- Soil densification by external forces.
- Densification of an unsaturated soil by reduction in volume of voids filled with air, while the volume of solids remains essentially the same.
- Major purpose of compaction of soil are:

1. To increase shear strength
2. To reduce permeability
3. To reduce compressibility
4. To reduce liquefaction potential
5. To control swelling and shrinkage

* Improvement of Engineering properties by densification is possible for natural soils as well as for soil stabilized with chemicals such as lime and cement.

- Proctors Theory of Compaction

  * Developed by R R Proctor in 1933

  * When he was supervising the construction of dam for bureau of waterworks and supply in Los Angeles, USA $\gamma_d = \frac{\gamma}{1+\omega}$

  * Based on laboratory Study using MOULD

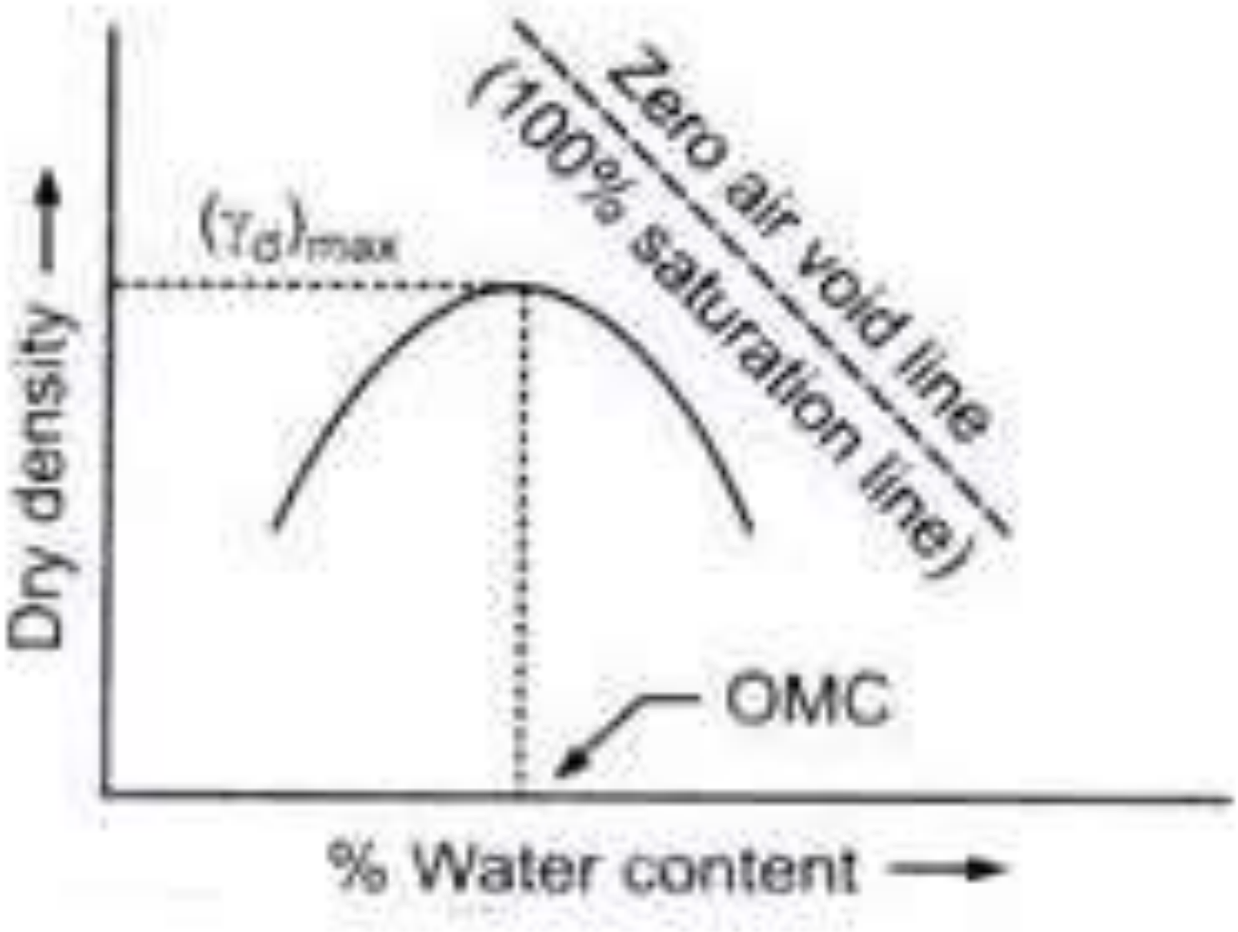
    Vol 1000 cc; I.D. = 100 mm, Ht= 127.3 mm

    Rammer 2.6 kg 310 mm drop

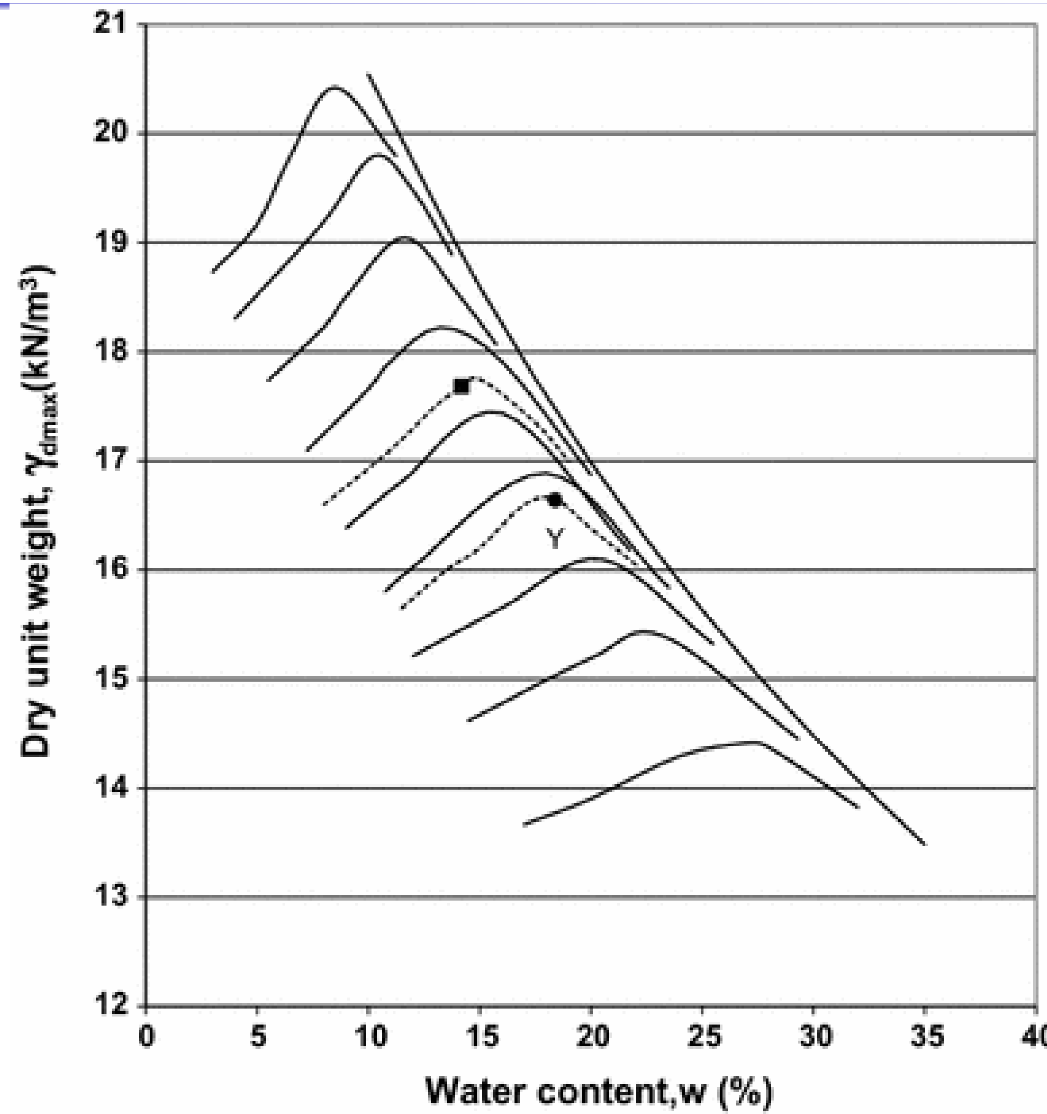
$\gamma d = (\gamma /(1+w))$

- Factors Affecting Compaction

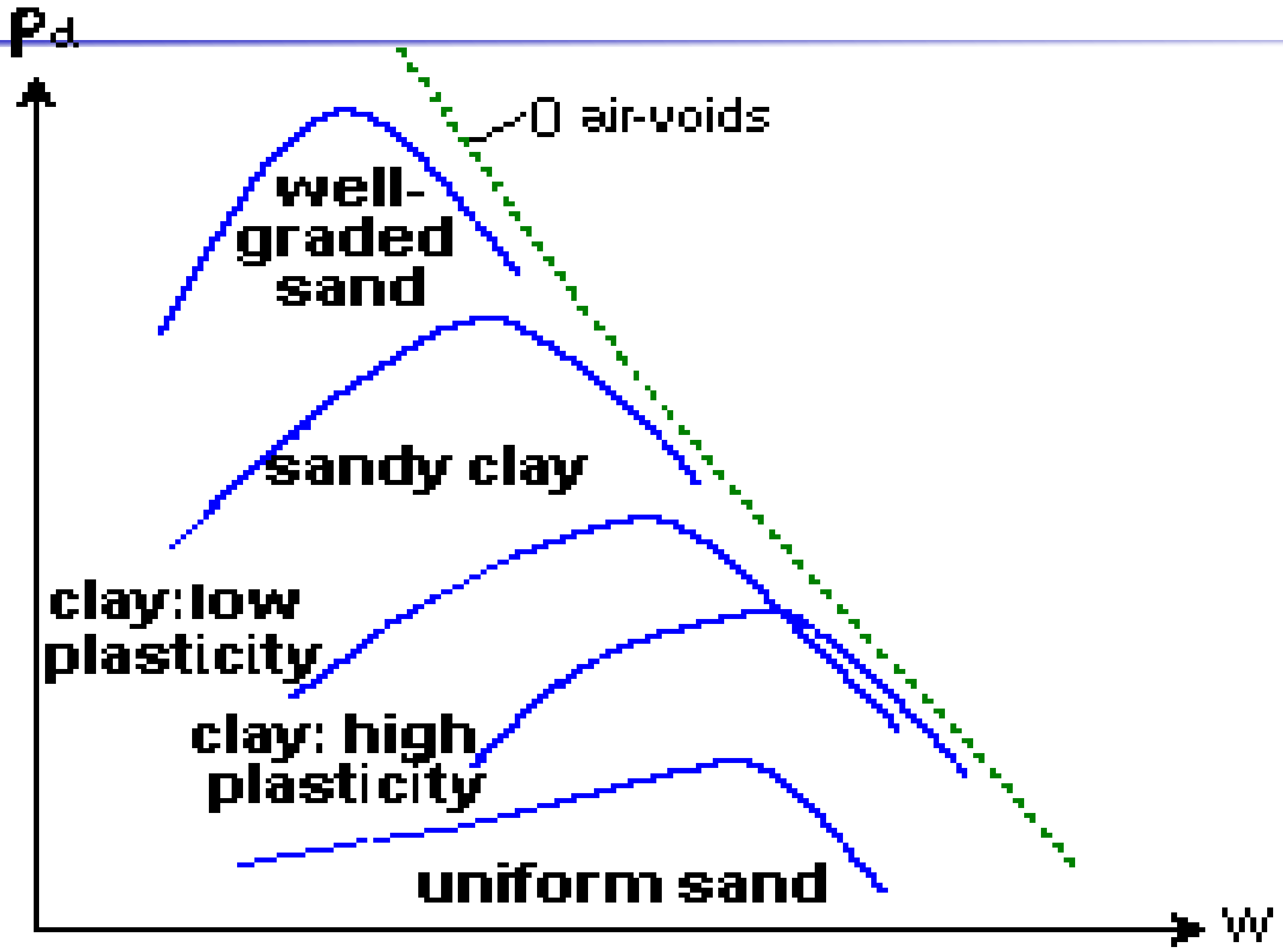
1. Moisture Content



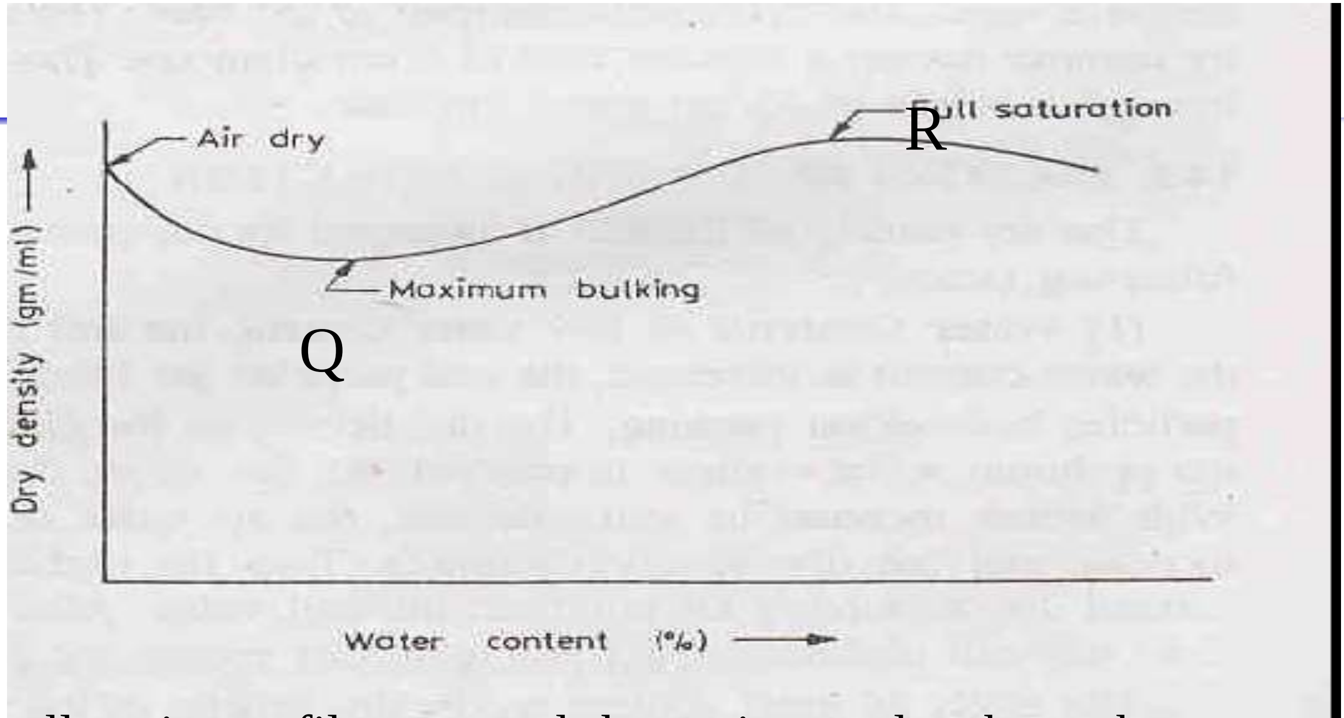
2. Effect of Compactive Effort



- Effect of Soil Type



Compaction of Sand



Small moisture films around the grains tend to keep them apart and can decrease the density up to a certain water content. The point Q on the curve indicates the minimum density.

Later on, the apparent cohesion gets reduced as the water content increases and is destroyed ultimately at 100% saturation of the sand.

Thus, the point R on the curve indicates maximum density. Thereafter, once again, the density decreases with increase in water content

- Compaction Energy

$$F = \frac{\text{No of blows/layer} * \text{No of Layers} * W * H}{\text{Vol of mould}}$$

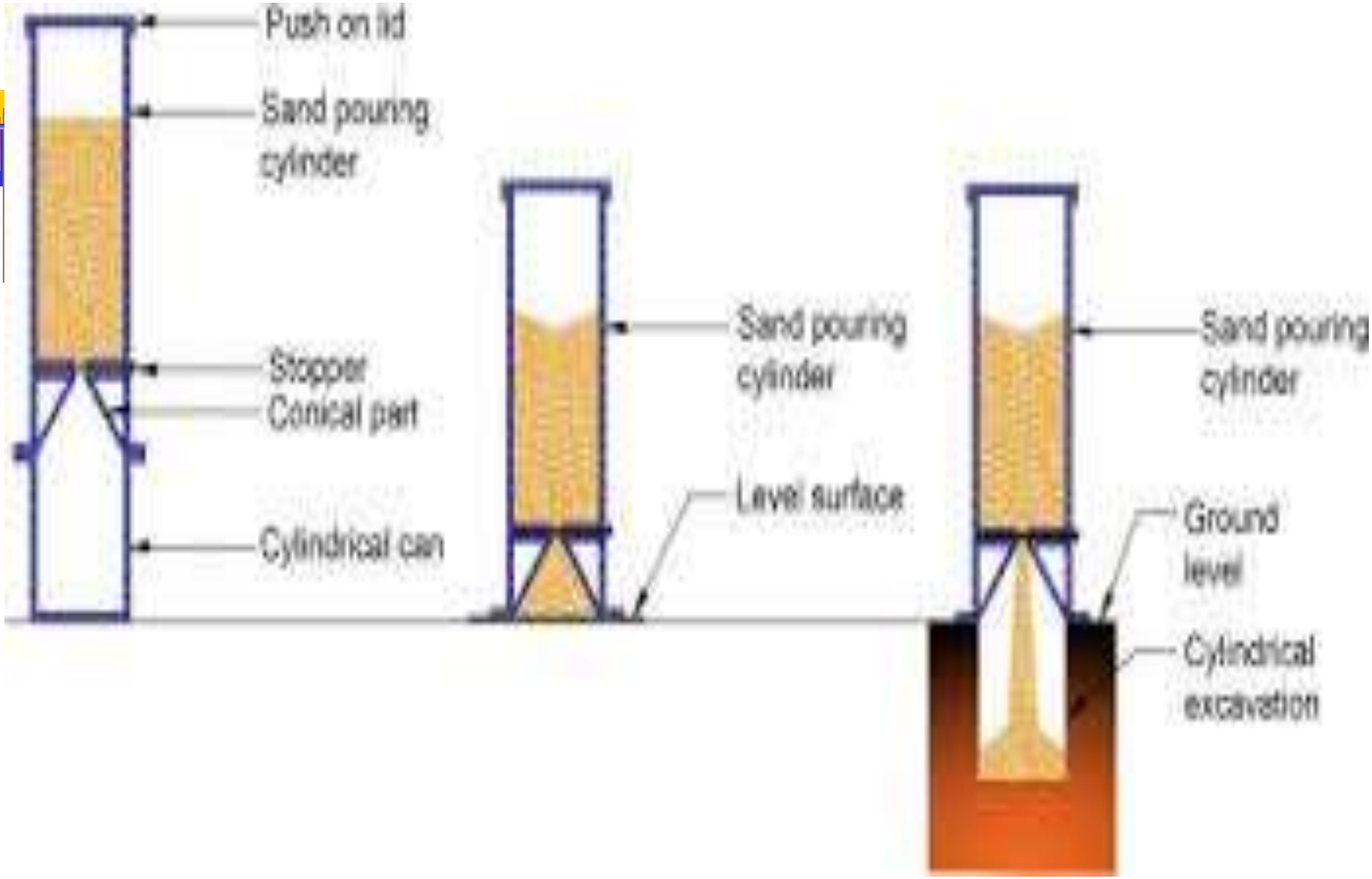
Modified Procter Test

Same as the Standard Proctor Test with the following exceptions:

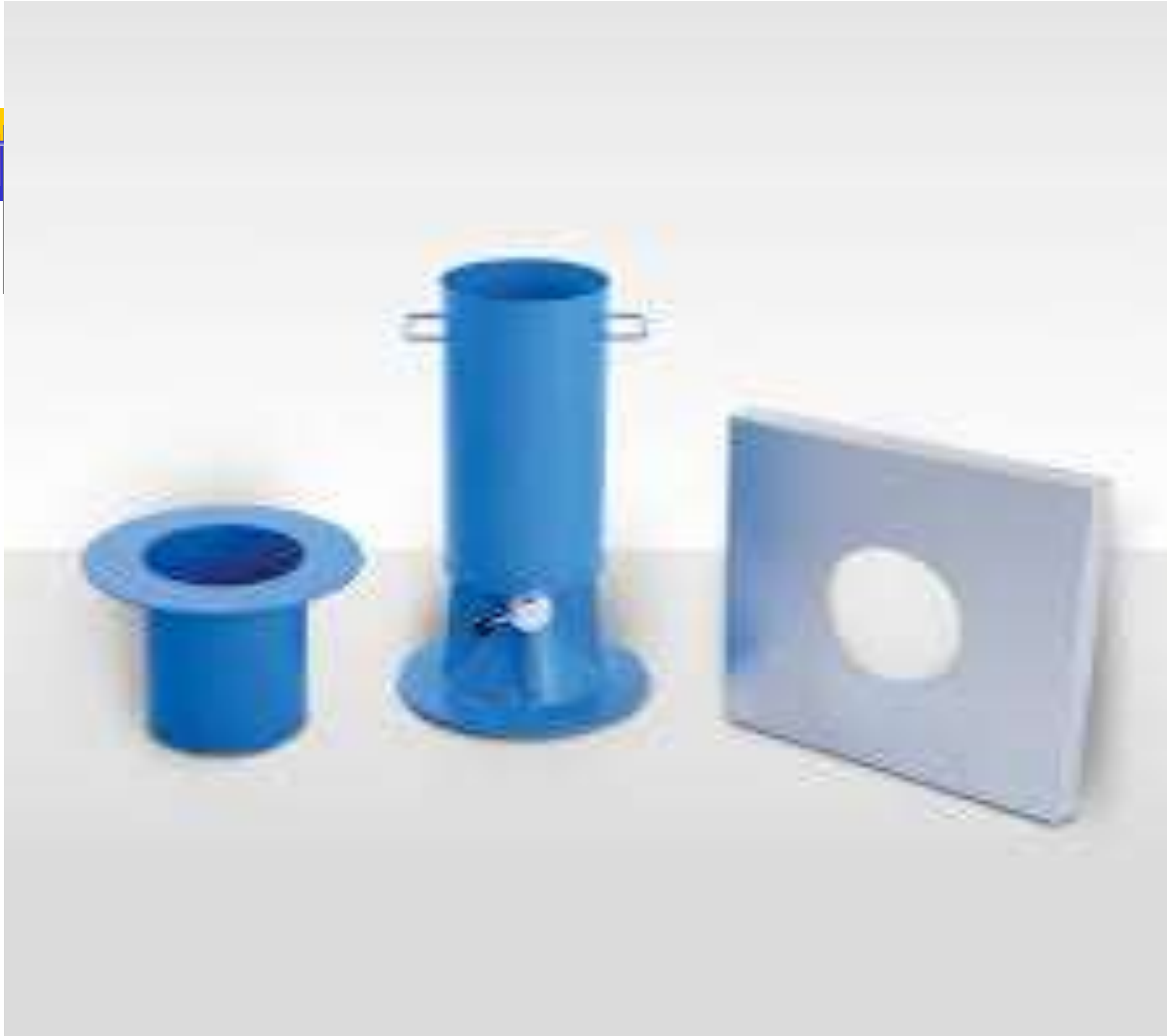
- The soil is compacted in five layers
- Hammer weight is 10 Lbs or 4.54 Kg
- Drop height h is 18 inches or 45.72cm
- Then the amount of Energy is calculated

Insitu Determination of Density

1. Core Cutter Method



2. Sand Pouring Cylinder



General Compaction Methods

	Coarse-grained soils	Fine-grained soils
Laboratory	Vibrating hammer	Falling weight and hammers Kneading compactors Static loading and press
	Vibration	Kneading
Field	❖ Hand-operated vibration plates ❖ Motorized vibratory rollers ❖ Rubber-tired equipment ❖ Free – falling weight dynamic compaction	❖ Hand-operated tampers ❖ Sheep-foot rollers ❖ Rubber-tired rollers

Field Compacton

Mostly carried out by Rollers

Content

- Introduction
- Smooth wheeled roller
- Pneumatic tyred roller
- Sheep foot roller
- Vibrating roller
- Tamping roller

Smooth wheeled roller

- Smooth wheeled rollers are of two types:

  1. Static smooth wheeled roller
  2. Vibrating smooth wheeled roller
- The compacting efficiency depends on-weight ,width and diameter of each roller.
- Suitable for- wide range of soil preferably granular soils such as sand, gravel, and crushed stones.
- When compacting cohesive soil these rollers tend to form a crust over the surface which may prevent adequate compaction in the lower portion.
- It found to be more use full where crushing action is advantageous.
- It is also effective in smoothing surface of soils that have been compacted by tamping rollers.

Smooth wheeled roller

Pneumatic tyred roller

- In this type number of pneumatic wheels are mounted on two or more axels under a loading platform ,pulled by tractor.
- This roller considered to be most suitable to compact non plastic silts and fine sands.
- In addition to the direct pressure due to rolling there is also slight kneading action.
- They may also be small or large tired units. The small tire units are usually have two tendon axle with 4 to 9 tires on each axels.
- The rear tire are spaced to travel over the surfaces between the front tire which produces a complete average of surfaces.

Cont……

- The large tired rollers are available in size varying from 15 to 200 tones gross weight because of heavy loads and high pressure . They are capable of compacting all types of soil to greater depth

Sheep foot roller

- This type of roller consists of hollow steel cylinder with projecting feet normally less than 25 cm in length.
- The weight of the roller can be increased by filling the drum with wet soil.
- The weight, width and diameter of roller may be varied and also the shape of the feet, these may be pulled by tractor.
- Varying the weight of the roller by the use of the ballast in drum will vary the foot contact pressure.
- This type of roller found most suited to clayey soil.
- During rolling operation the soil under the projecting feet get compacted and also considerable kneading action to the soil.

Cont…

- When the drum rotates, the pads out of the soil they kick to the material because of their shape.
- The thickness of compacting layers is kept about 5cm more than the length of each foot.
- About 24 or more number of passes of roller may be necessary to obtain adequate compaction, however the top layer of the sub grade may be compacted using smooth wheeled roller to get properly finished surface.
- They are suitable for compacting all fined grained material but is generally not suitable for use of cohesion less granular material.
- Therese rollers can work at speed of 6 to 10 kmph.
- Usually 6 to 10 passes will be needed to compact a 20 cm clay layer.

Sheep foot roller

Vibrating roller

- Vibratory type rollers have two smooth wheels/ drums plus the vibrators.
- One is fixed at the front and the other one is on the rear side of vibratory roller. Both drums are of the same diameter, length and also of same weight.
- Vibratory roller covers the full area under wheel. To make vibratory roller more efficient, vibrators are also fixed with smooth wheel rollers.
- Vibrations of vibrators arrange the particles by first disturbing even the arranged ones.
- Vibrators are turned off during the reversed motion of roller. In that time only static weight directly acts on the soil layer.


CASE

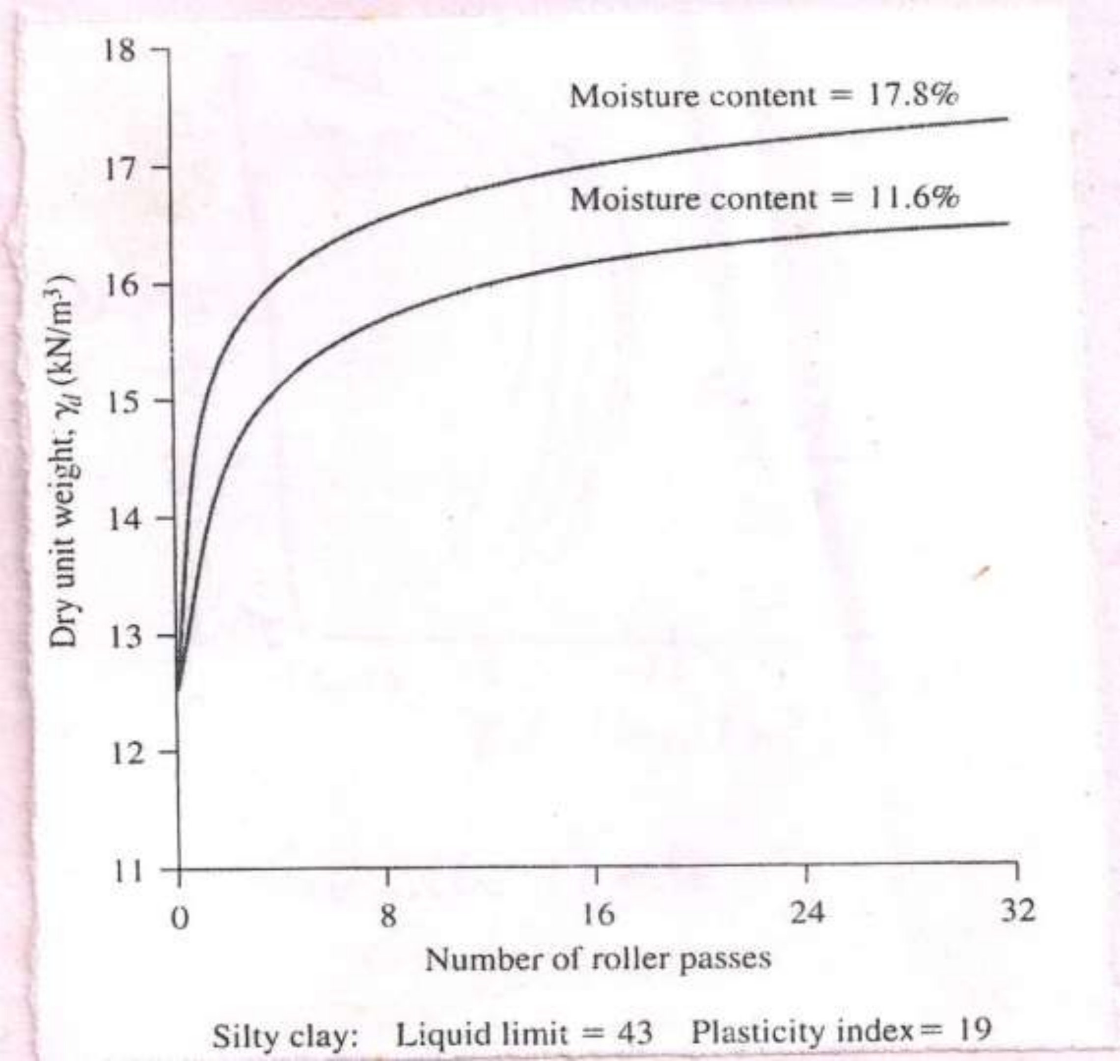
Tamping foot roller

- They are high speed , self propelled, non vibratory rollers
- These rollers usually have steel padded wheels and can be equipped with a small blade to help for leveling the layer.
- The pad are tappered with rectangular face.
- As temping rollers moves over the surface the feet penetrate into the soil to mix and compact the soil from bottom to top of the layer.
- Tamping foot roller consists of four wheels and on each wheel kneading boots/feet are fixed. Tamping roller has more coverage area i.e., about 40- 50%. Contact pressure of tamping roller varies from 1400 – 8500KPa. It is best dedicated to fine grained soils.
- A tamping foot roller is effective on all soils except clean sand.

Tamping foot roller

Factors affecting Roller Passes

1. No of roller passes



Control of Compaction in the Field

Control of compaction in the field consists of checking the water content in relation to the laboratory optimum moisture content and the dry unit weight achieved *in-situ in relation to* the laboratory maximum dry unit weight from a standard compaction test.

Typically, each layer is tested at several random locations after it has been compacted.

Relative compaction $= \gamma_{field} / \gamma_{lab}$

Relative compaction of 90 to 100 is specified and sought to be achieved.

Specifications for Field Compaction

Relative density should not be confused with relative compaction.

Correlation between relative compaction (R) and the relative density Dr

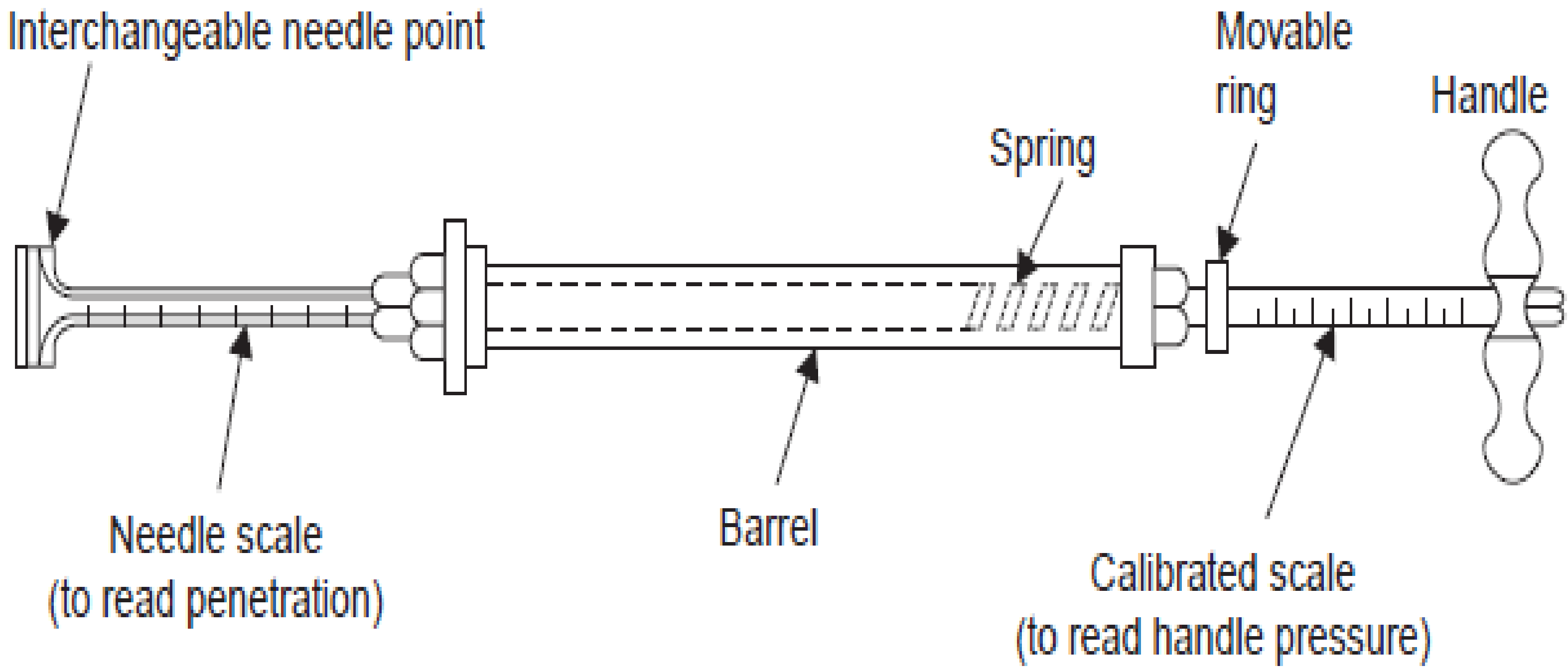
$$R = \frac{R_o}{1 - D_r(1 - R_o)}$$

where:

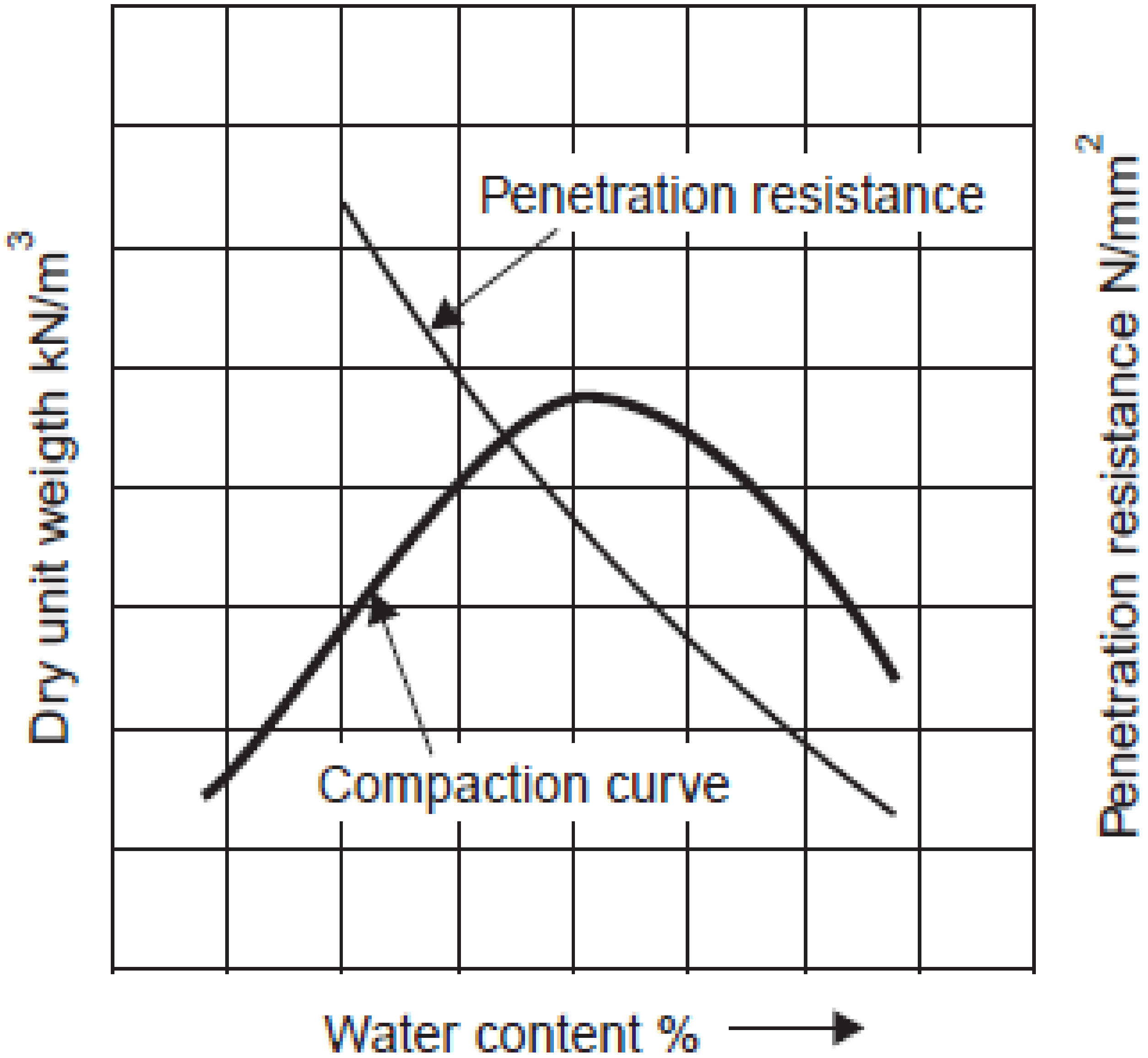
$$R_o = \frac{\gamma_{d(min)}}{\gamma_{d(\max)}}$$

Proctor Needle

The Proctor needle approach given here, is an efficient and fast one for the simultaneous determination of *in-situ unit weight and in-situ moisture content, it is also called 'penetration* needle'



Calibration curve for proctor needle



THANK YOU!

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