

Geotechnical Engineering

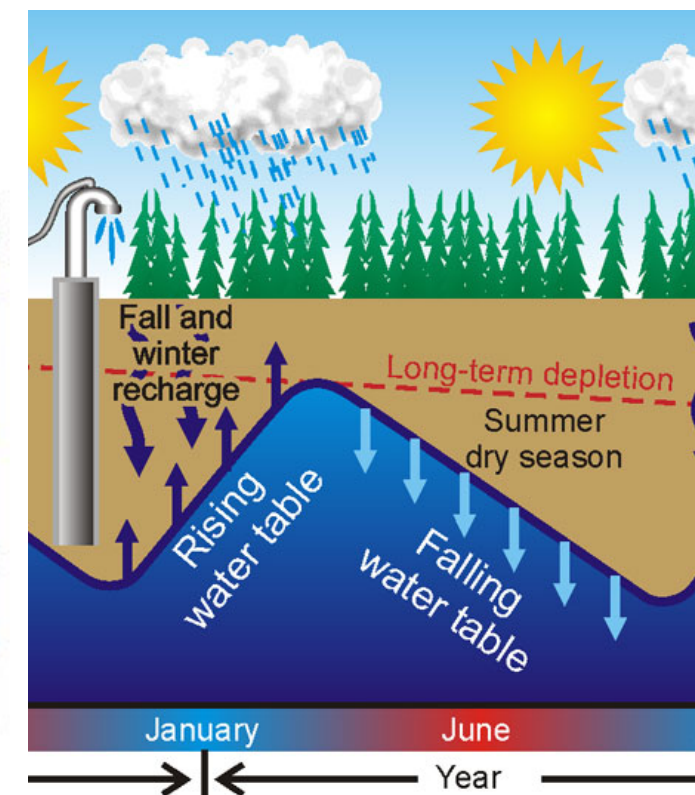
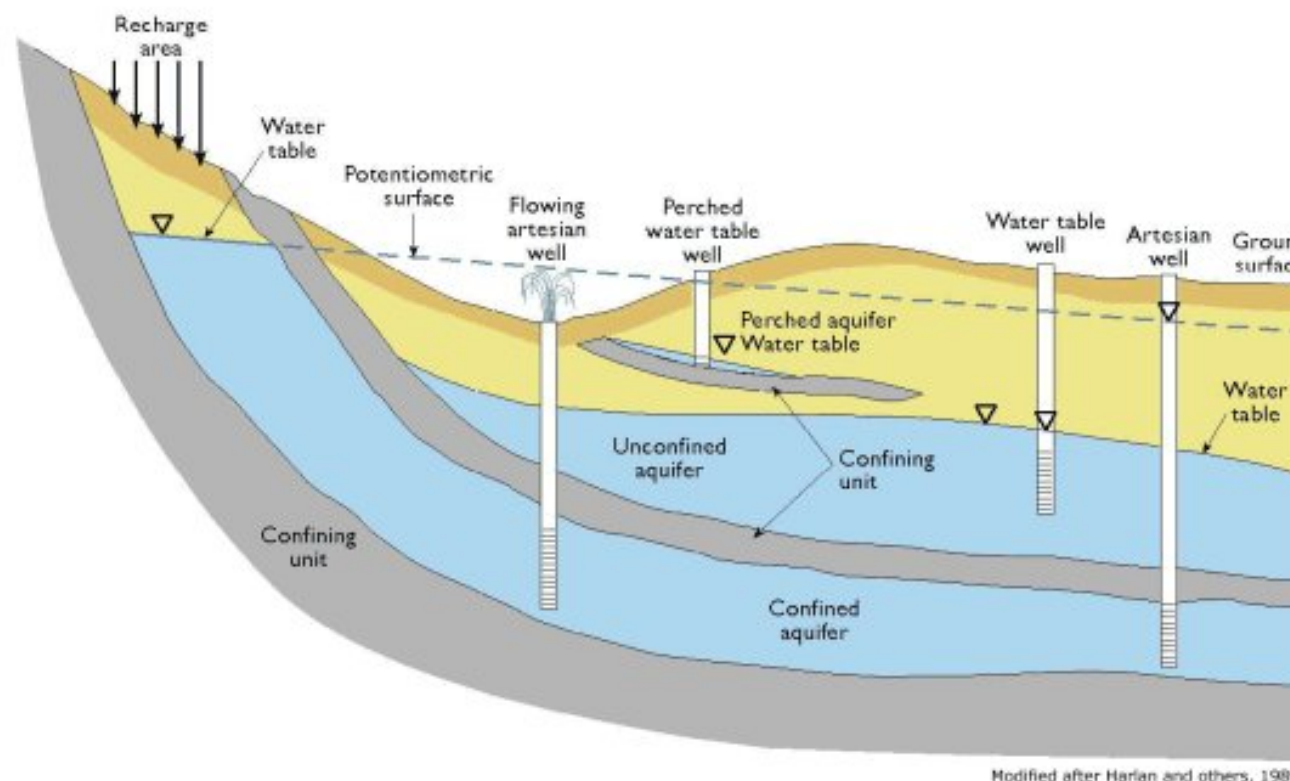
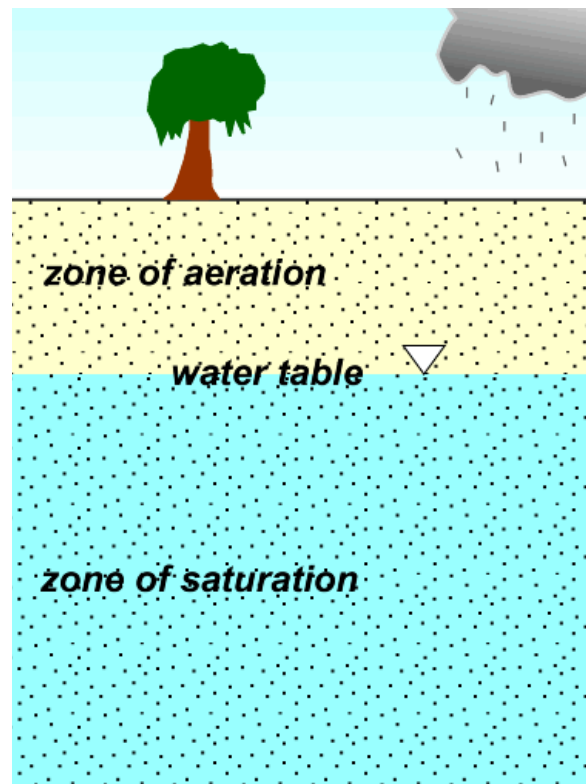


Soil Water

Text: Das Chapter 7

Subsurface water is found at all depths down to around 12,000 m below the Earth's surface. Below that depth, all voids are filled by the plastic flow of rocks under great pressure.

Soil mechanics is concerned with moisture much closer to the surface. In this region there are two zones, the *saturated zone* and the *unsaturated or aerated zone*.



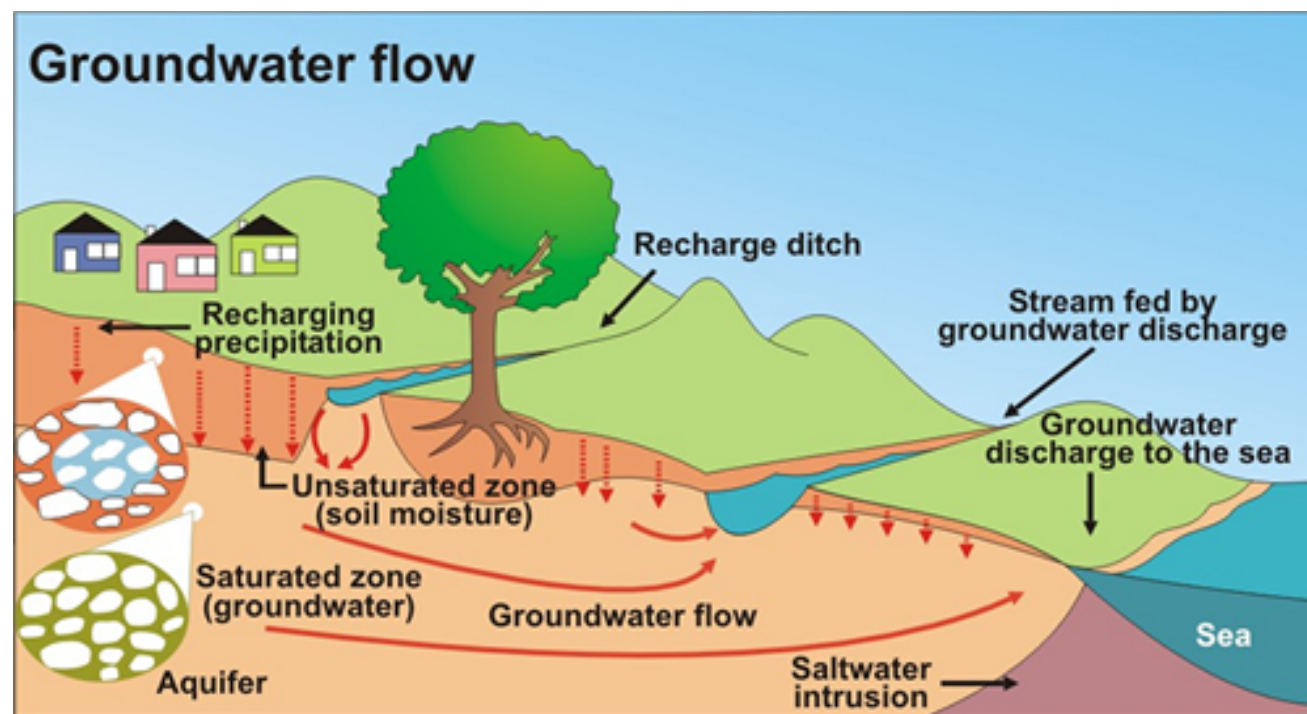
Saturated Zone: Ground Water

In the saturated zone, all the voids and fissures are filled with water under hydrostatic pressure, whereas above it the voids contain water and air.

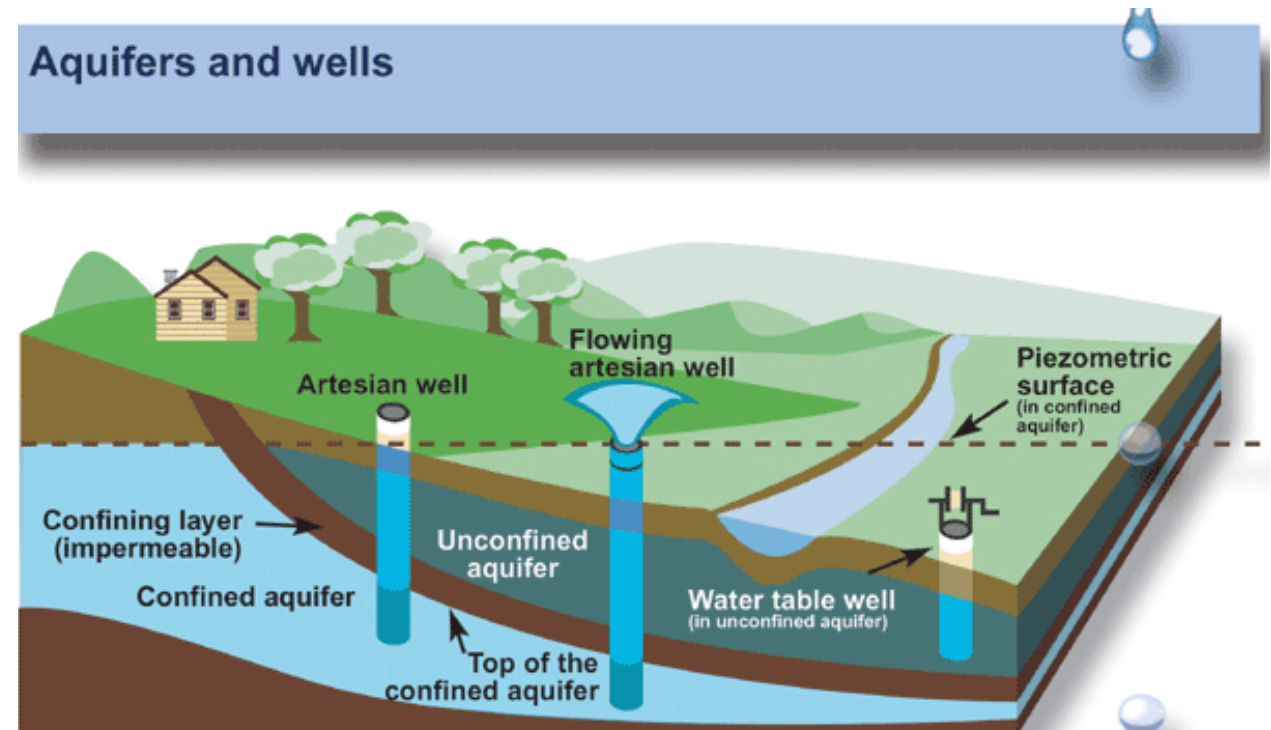
The water table or phreatic surface divides the two zones. The water along this line is at atmospheric pressure.

Generally if you dig a hole in the ground the water level in the hole defines the phreatic surface. The water table can vary seasonably with rainfall or with irrigation activity.

Where it breaks the ground surface, swamps, lakes and springs occur.



Environment Canada



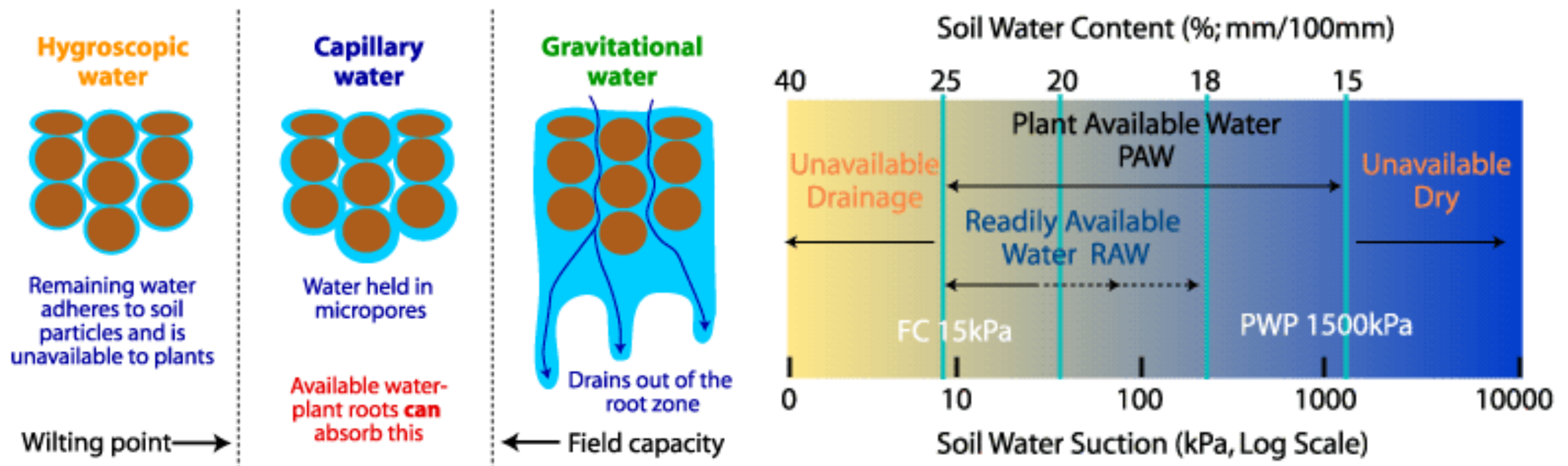
Source: Environment Canada.

Unsaturated Zone: Soil Water

In the unsaturated zone, water is bound to soil particles by a variety of physical and electrochemical forces. It takes energy to extract this water.

Conditions in the aerated zone depend on climate and biology, particularly recent rainfall, temperature and humidity and the growth activity of plants.

Depending on the degree of saturation, water may be held predominantly by hygroscopic interaction with soil minerals, or by the capillary action of free surfaces, or as both static and dynamic storage within the void spaces.

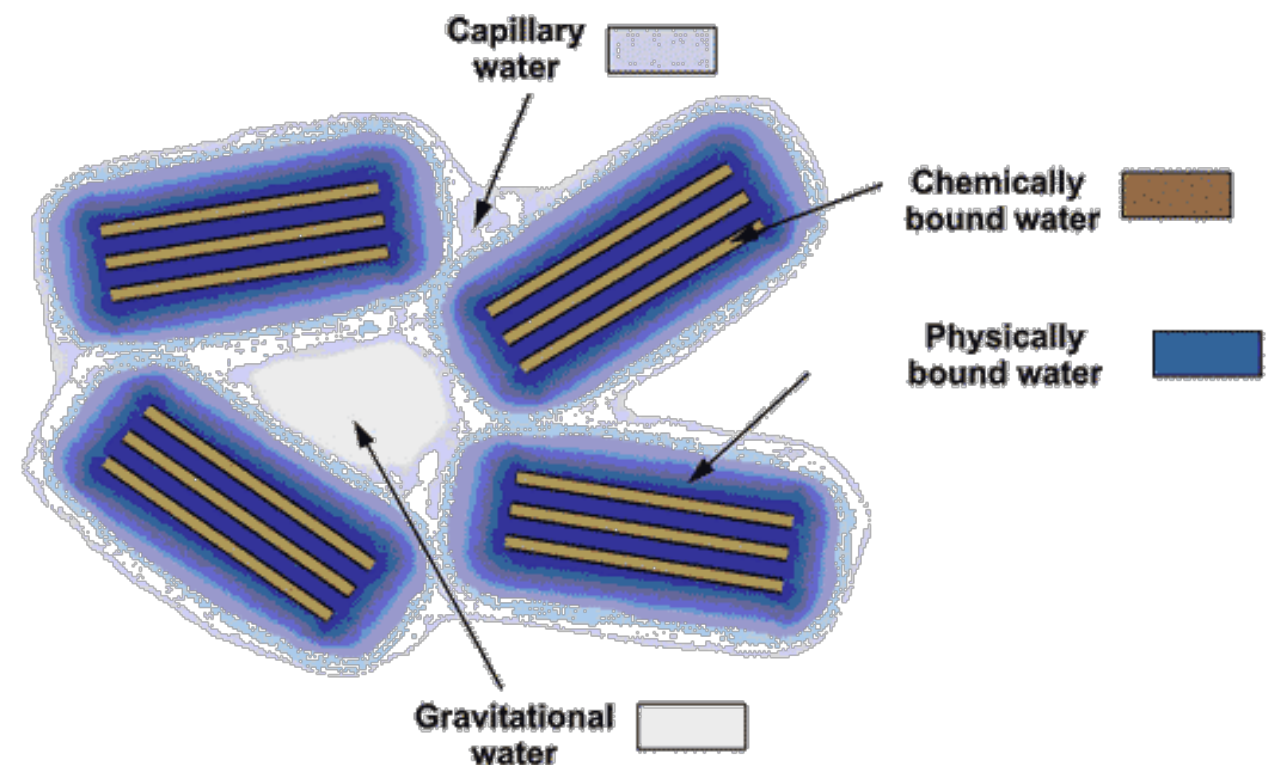
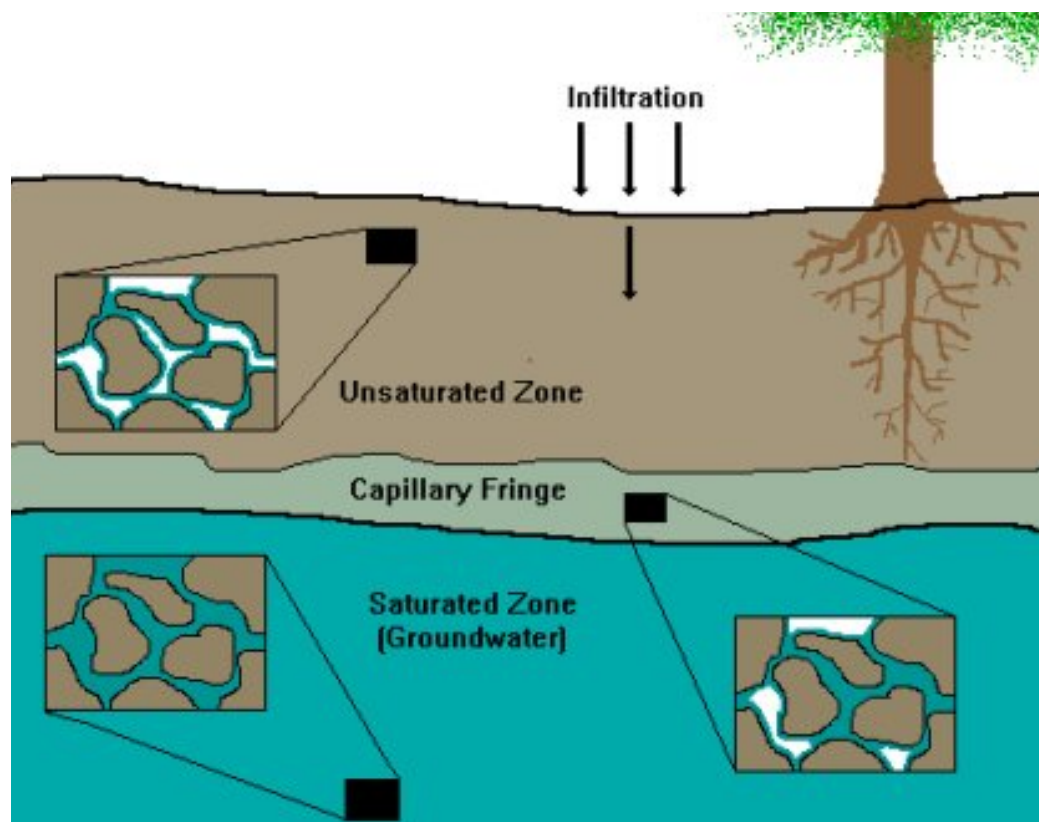


Capillary Water

Soil water at useful engineering levels of saturation is mainly bound by capillary forces. Higher levels of saturation are too wet, lower levels too dry.

The capillary fringe occurs due to surface tension in the void spaces above the water table. The smaller the particle size, and hence voids, the higher the capillary fringe will be.

Water in the capillary zone is held at a negative or suction pressure. In clays, molecular forces hold moisture even more tightly than capillary forces can exert. In silt soils the capillary fringe may exceed 2 m in depth.



Soil Water Potential

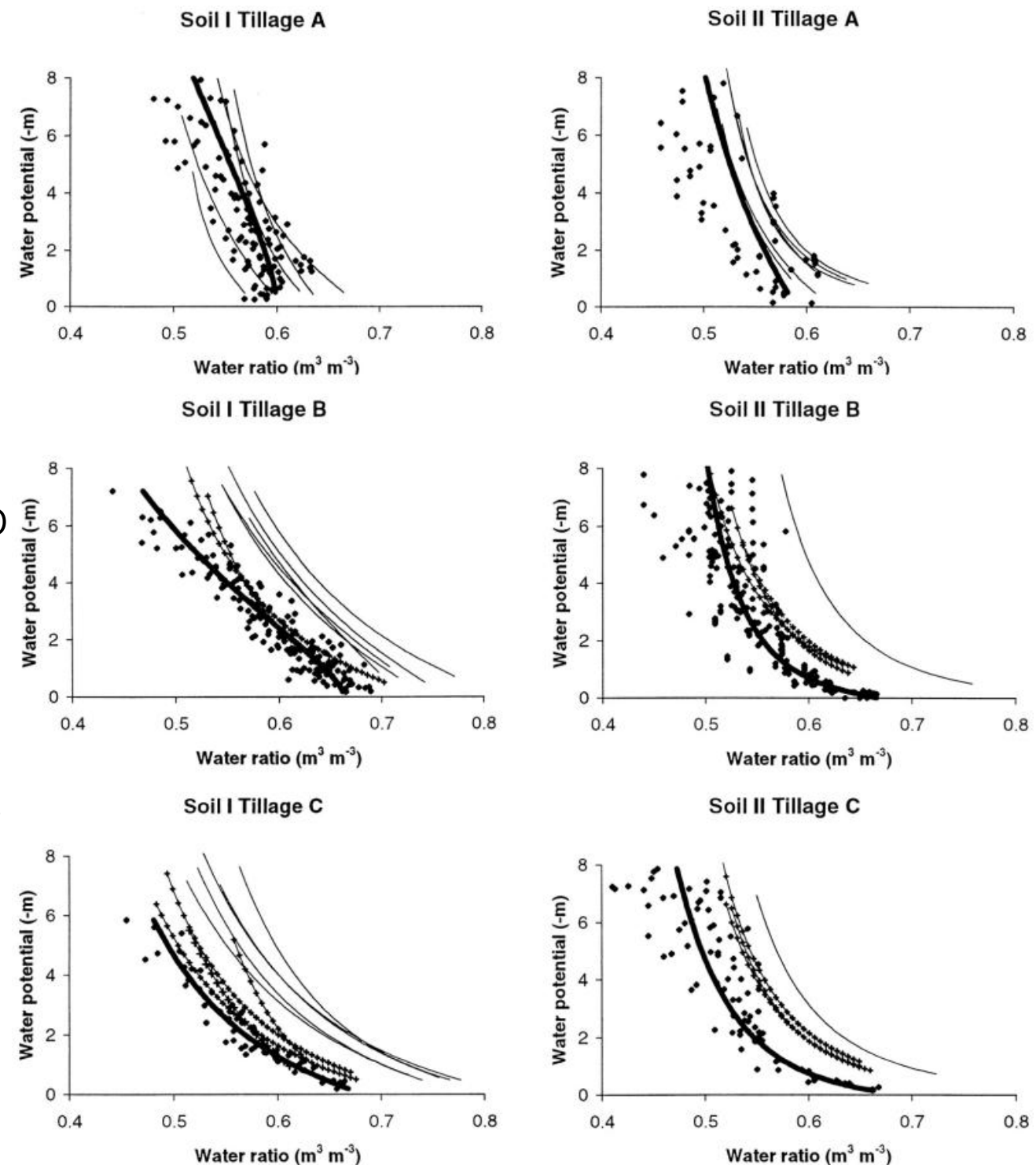
Water potential is similar to Bernoulli's principle in fluid mechanics.

Potential is measured in terms of total head (H), which is energy per weight of water. In soil there is effectively zero velocity head, but the head due to capillary and osmotic forces can be very large.

Soil water potential consists of three components: the *matric potential* h due to capillary forces, the *osmotic potential* o due to ions dissolved in the water, and the more familiar *gravitational potential* z due to elevation or depth.

In unsaturated soils, the matric potential is the largest. In saturated soils, h is close to zero while z is large.

$$H = h + o + z$$



Potential Tensionometry

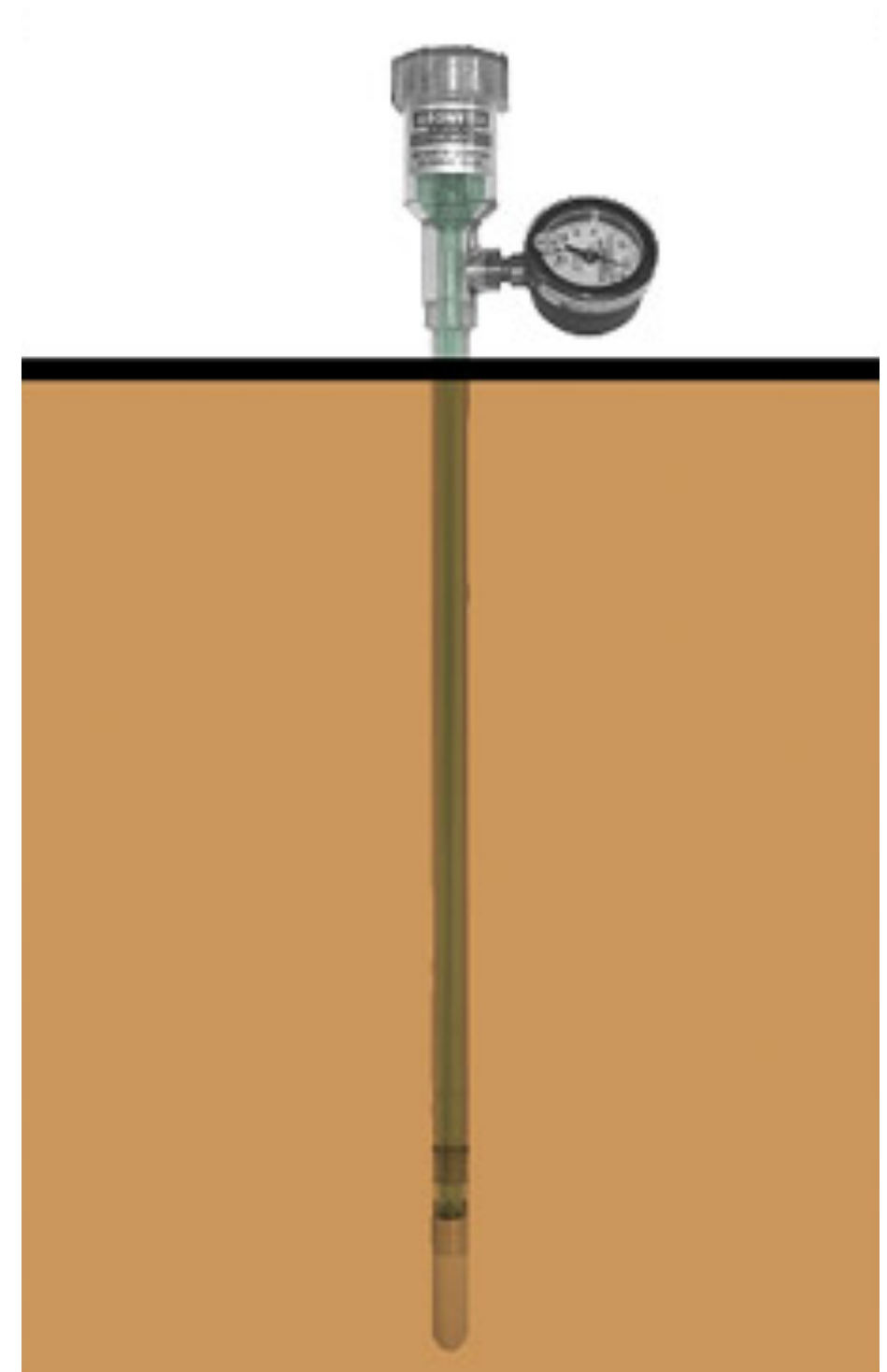
Water potential can be measured in the field by a *Tensionometer*.

A tensiometer consists of a vacuum gauge connected by a tube to a porous ceramic tip.

The tube is filled with water and is normally transparent (made of plexiglass) so the water level within it can easily be seen. The ceramic tip is permeable, and the water in the tube saturates it.

The tip is placed in contact with the soil in the root zone. Because the soil is normally not saturated, water is drawn from the tip into the soil.

As water moves from the tube into the soil, a partial vacuum is created and measured by the gauge. This measurement is not a direct measurement of soil water content. Rather, it is a measurement of soil water tension



Saturated Zone: Hydrostatic Potential

In the saturated zone the soil water potential is dominated by gravity, ie water under ordinary hydrostatic pressure.

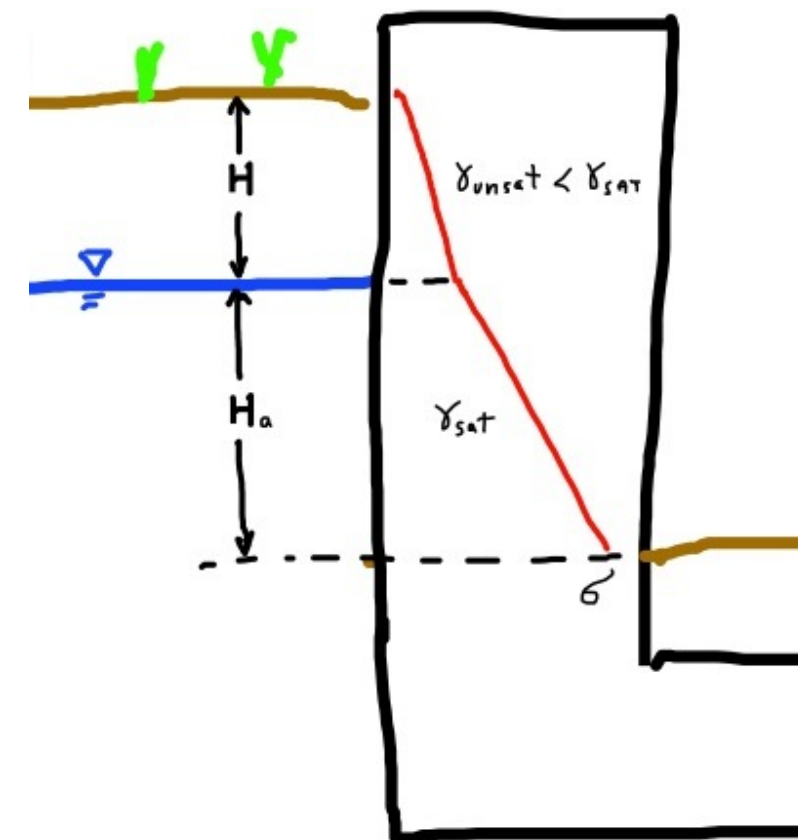
Pore water pressure in the saturated zone is the same as in a body of free water:

$$P = \rho g H_a = \gamma_w H_a$$

Where H_a is the depth below the water table. This pressure contributes to the total pressure σ exerted by the weights of saturated and unsaturated soil:

$$\sigma = \gamma_{unsat} H + \gamma_{sat} H_a$$

where H is the depth *above* the water table. γ_{sat} includes the density of soil and water. σ is thus the total pressure exerted on a supporting structure such as a retaining wall.



Saturated Zone: Effective Stress

As well as loads on supporting structures, it is often essential to know the pressure that a soil mass exerts on itself. This requires the idea of *effective stress*.

As with any submerged object, saturated soil particles are supported by buoyancy. The effective weight of each particle is reduced by its displaced volume of water:

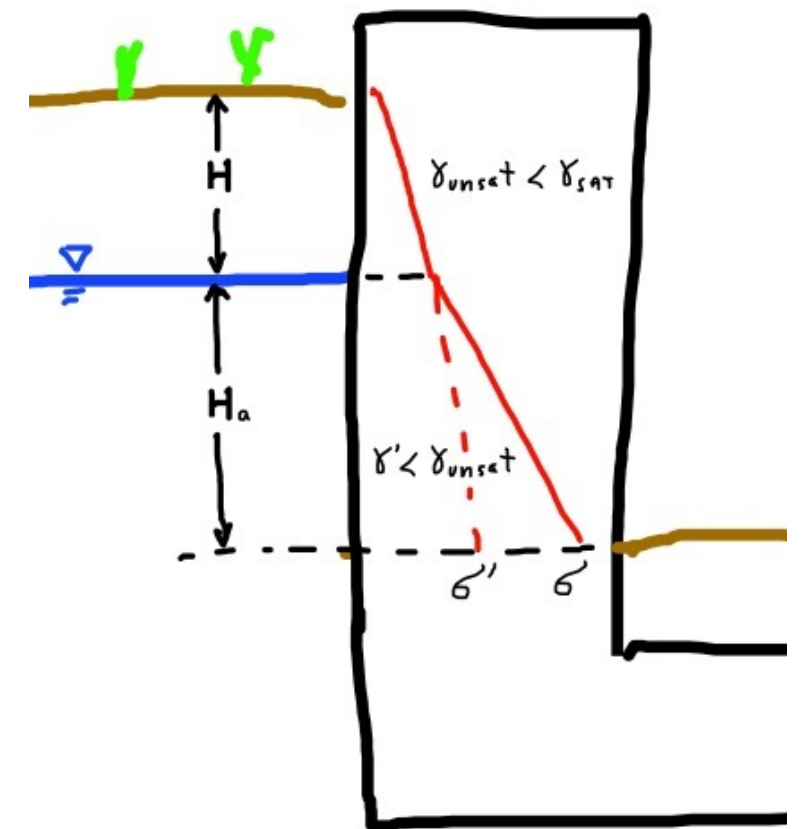
$$W = \gamma_s V - \gamma_w V = (\gamma_s - \gamma_w) V$$

The effective pressure that soil particles exert on each other is thus reduced by the same factor:

$$\begin{aligned}\sigma' &= \sigma - P = \gamma_{\text{unsat}} H + \gamma_{\text{sat}} H_a - \gamma_w H_a \\ &= \gamma_{\text{unsat}} H + (\gamma_{\text{sat}} - \gamma_w) H_a\end{aligned}$$

σ' is referred to as the *effective stress*. γ' is the *effective unit weight*.

Bearing pressures on the soil itself are determined by the effective stress, while structural pressures on any other element are given by the total stress σ .



Saturated Zone: Effective Stress

The bulk, saturated and effective unit weights of soil can be compared by the formulas developed in an earlier lecture:

Saturated soil:

$$\gamma_{sat} = \gamma_w \frac{G + e}{1 + e}$$

Unsaturated (bulk) soil:

$$\gamma_b = G \gamma_w \frac{1 + w}{1 + e} = \gamma_w \frac{G + Se}{1 + e} < \gamma_{sat} \text{ as } S < 1$$

Effective (buoyant) soil:

$$\begin{aligned} \gamma' &= \gamma_{sat} - \gamma_w = \gamma_w \left(\frac{G + e}{1 + e} - 1 \right) \\ &= \gamma_w \left(\frac{G - 1}{1 + e} \right) < \gamma_b \end{aligned}$$

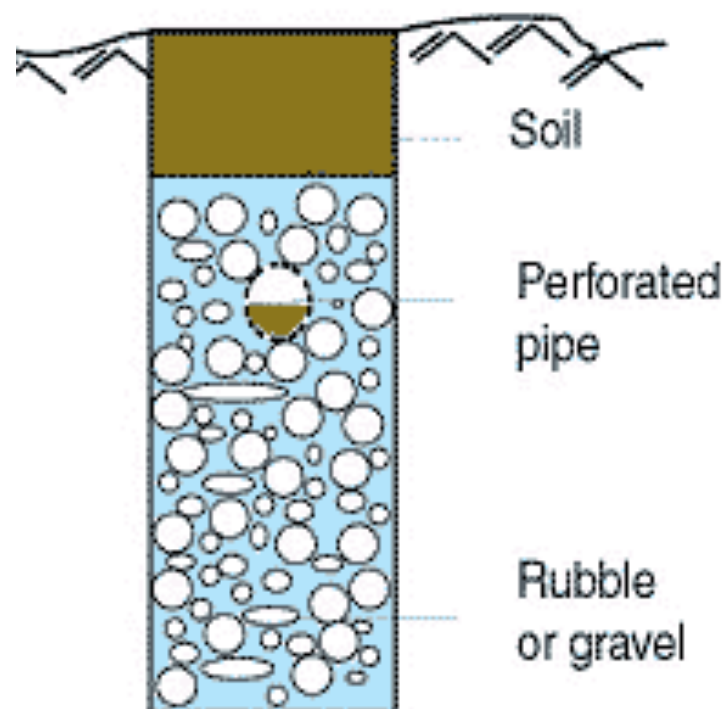
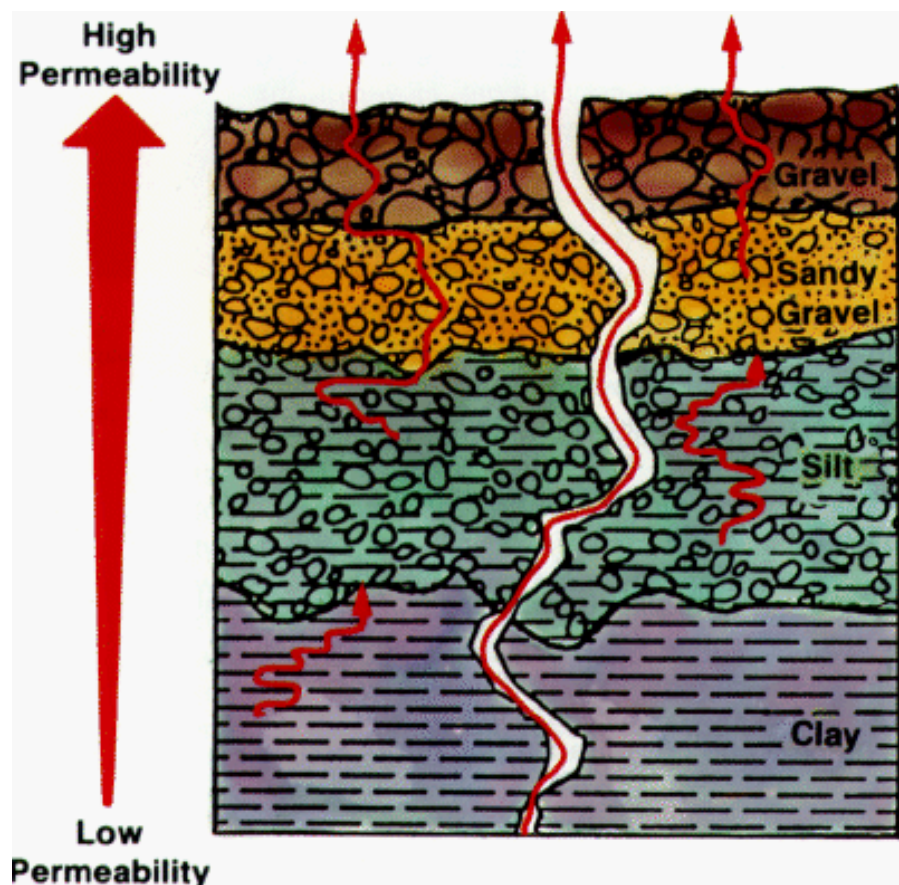
These formulas are used to calculate pressures of different kinds on structures and soil foundations. This is used not only in structural loading but in soil loading for consolidation and compaction, both of which depend on contact pressures between soil particles.

Permeability

Text: Das, 2010. *Principles of Geotechnical Engineering Ch 7*

Soils are permeable due to the existence of interconnected voids through which water can flow from points of high energy to points of low energy.

The study of the flow of water through permeable soil media is necessary for estimating the quantity of underground seepage under various hydraulic conditions, for investigating problems involving the pumping of water for underground construction, and for stability analysis of earth dams and earth retaining structures that are subject to seepage forces.



Groundwater Flow

Bernoulli equation and headloss

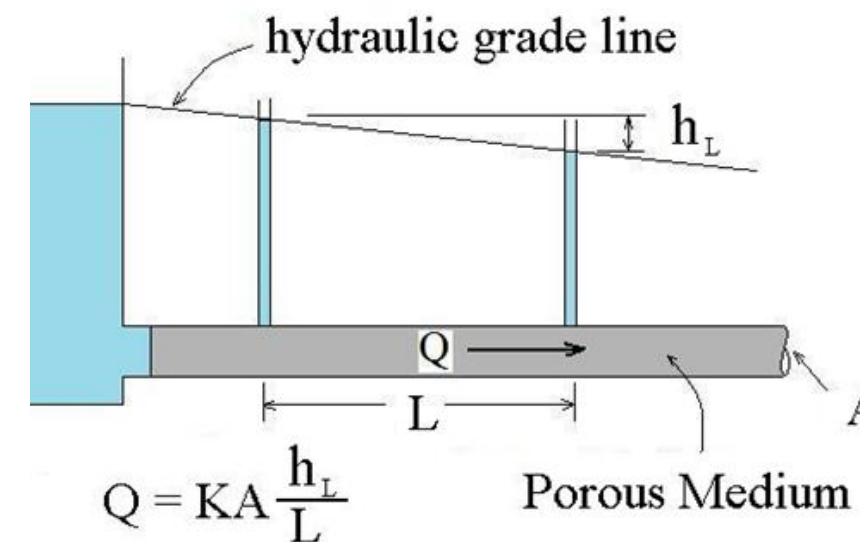
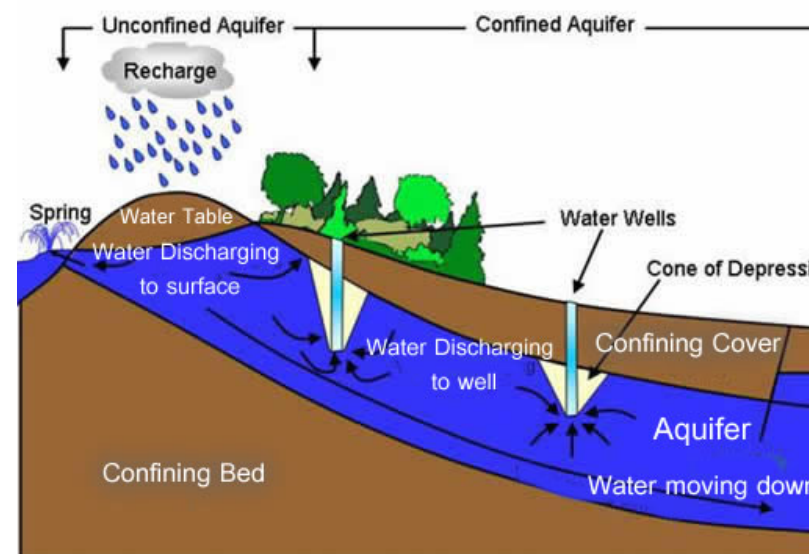
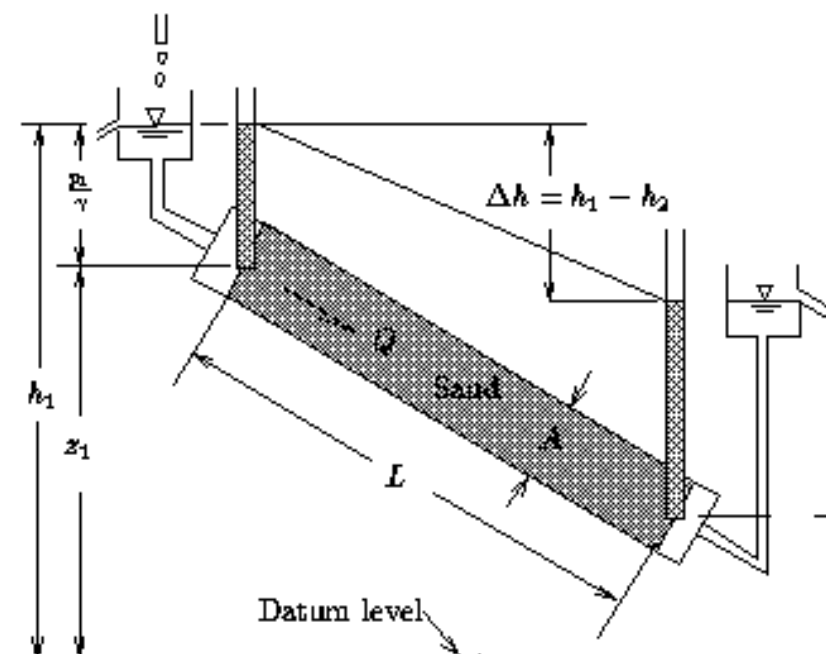
Bernoulli's equation is of limited use in groundwater flow. The total head is not constant, due to high friction losses in pores and capillary pathways.

However, a soil strata of constant porosity characteristics will show a constant rate of headloss or *hydraulic gradient*.

$$h = \frac{P}{\rho g} + Z = Z \text{ at the water table surface}$$

$$\frac{dh}{dl} = i \text{ hydraulic gradient}$$

Measurement of variations in water table (eg by observation wells) enables measurement of the hydraulic gradient.



Groundwater Flow

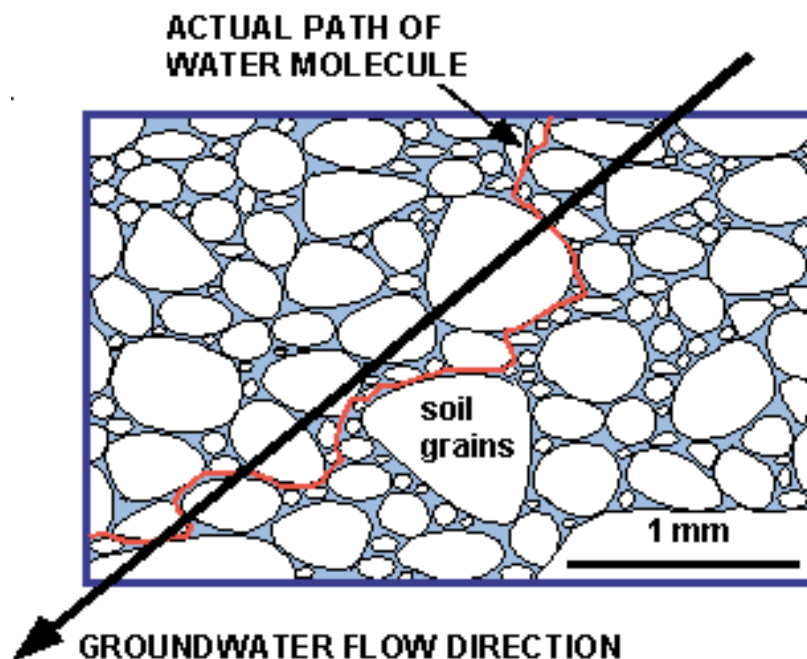
Darcy's Law

Still considering only soils with constant porosity characteristics, we can define the *hydraulic conductivity* k of a soil in terms of a constant hydraulic gradient i :

$$v = ki = -k \frac{dh}{dx}$$

The minus sign gives a +ve velocity for decreasing head, ie -ve hydraulic gradient.

v is the *discharge velocity*, defined as the average flow velocity in the direction of groundwater flow. This is less than actual flow velocities, which take long random pathways through the pore space while moving in any overall direction.



The discharge velocity is measured by the discharge and area across the flow:
 $v = Q/A$

The true or *seepage velocity* is related to the porosity and the discharge velocity:
 $v_s = v/n$

Groundwater Flow

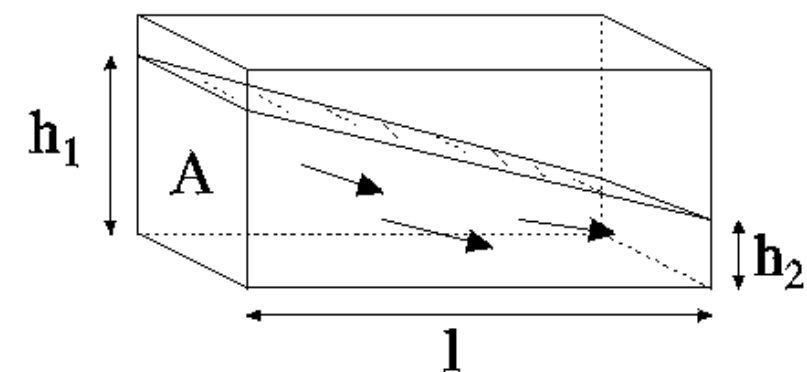
Darcy's Law

Given the hydraulic conductivity k (a property of each distinct soil stratum) and a known head difference H over a flow path length L , the flow rate through a unit area of soil can be estimated:

$$Q = kA \frac{H}{L} = k \frac{H}{L} \quad (\text{for } A = 1)$$

k has the units of velocity, since the hydraulic gradient is dimensionless. Hydraulic conductivity varies over a wide range for different soil types:

Soil Type	k cm/sec
Clean gravel	100-1.0
Coarse sand	1.0-0.01
Fine Sand	0.01-0.001
Silty clay	0.001-0.00001
Clay	<0.00001



$$\text{flow rate } Q = -K A (h_1 - h_2) / l$$

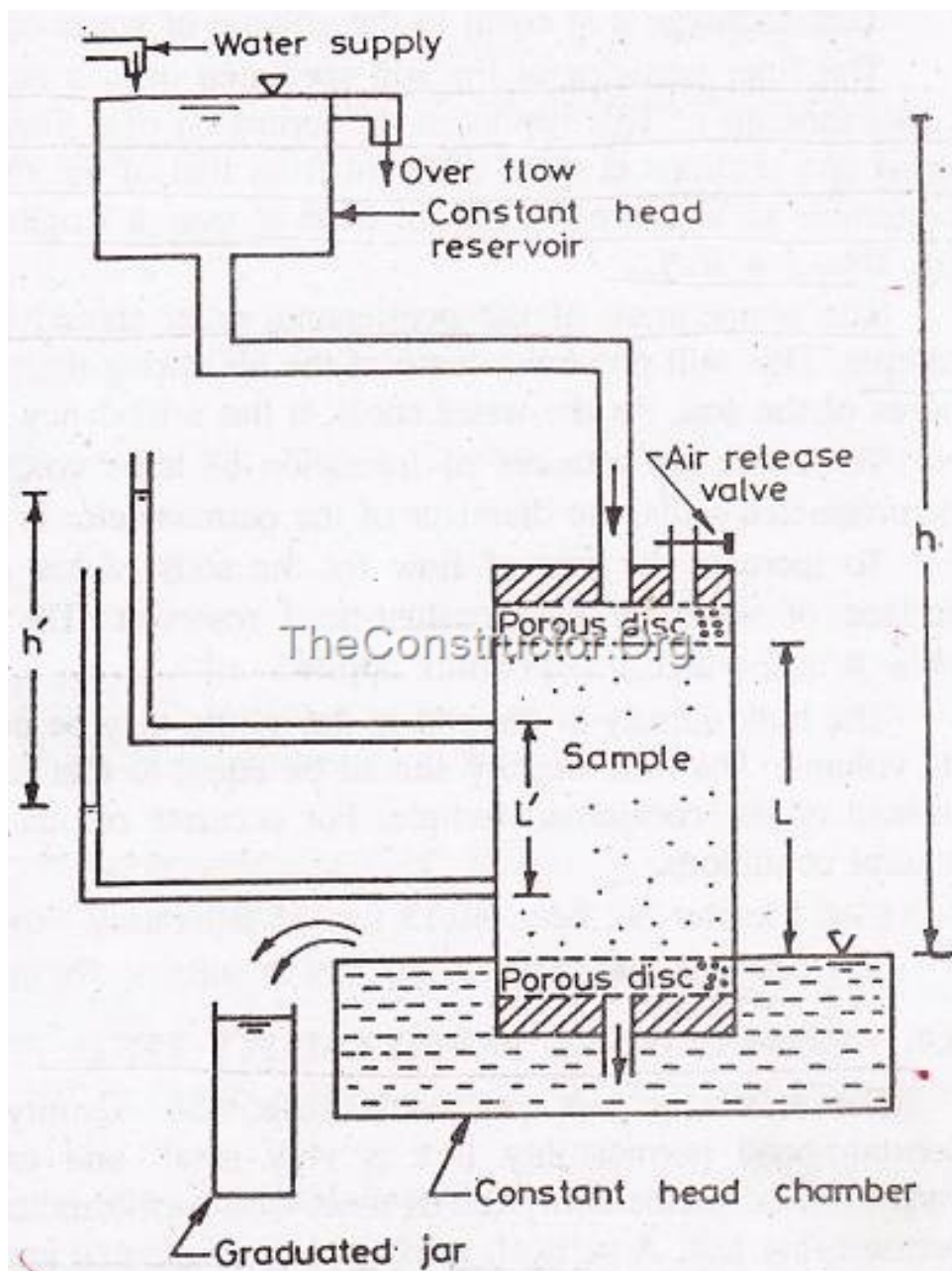
unit: L^3/T

"The velocity of flow is proportional to the hydraulic gradient"

Hydraulic Conductivity

Constant Head Permeameter

Hydraulic conductivity is measured by direct tests of flow rate for an observed head loss over a known path length. If the head difference is kept constant the measurement is known as a *constant head permeameter* test.



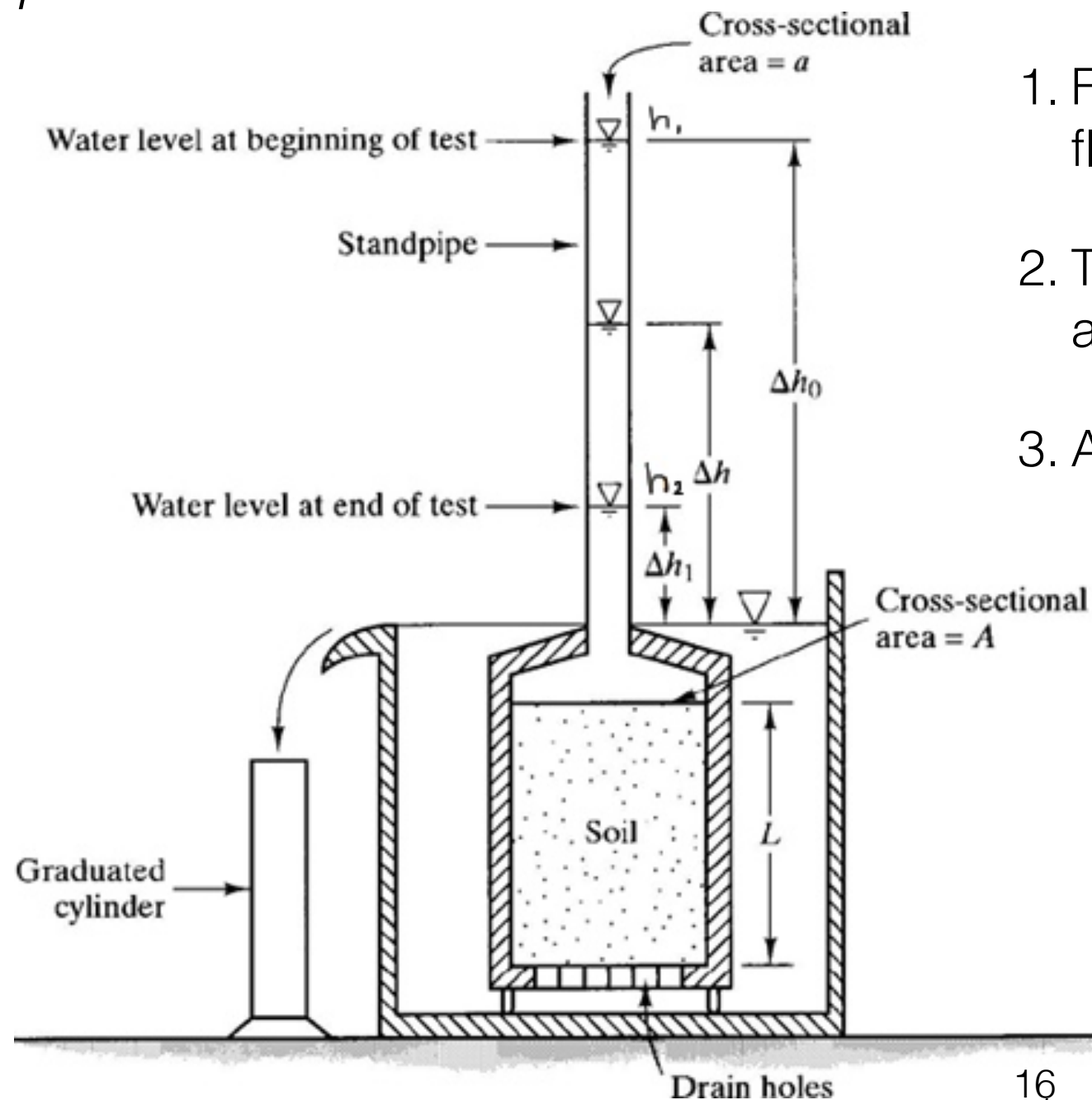
1. Flow is established with constant head and flowrate.
2. A volume V of water is collected over a measured interval t
3. Flowrate $Q = V/t = Avt = A(k_i)t = Ak(h'/L')t$
4. thus
$$k = \frac{VL'}{Ah't^2} = \frac{QL'}{Ah't}$$

A is the cross sectional area of the flow chamber

Hydraulic Conductivity

Falling Head Permeameter

An alternative arrangement tests for conductivity over a range of hydraulic gradients by allowing the head to fall as flow occurs, ie a *falling head permeameter* which drains over a finite interval:



1. Flow is established with constant head and flowrate.
2. The inflow is halted and the standpipe allowed to fall from h_1 to h_2 over time Δt .
3. At any time the flowrate Q is:

$$Q = a \frac{dh}{dt} = kA \frac{-\Delta h}{L}$$

$$\text{or } dt = \frac{aL}{Ak} \frac{-dh}{h}$$

the -ve sign accounts for the falling head

$$\text{integrating, } t = \frac{aL}{Ak} \ln \left(\frac{h_1}{h_2} \right)$$

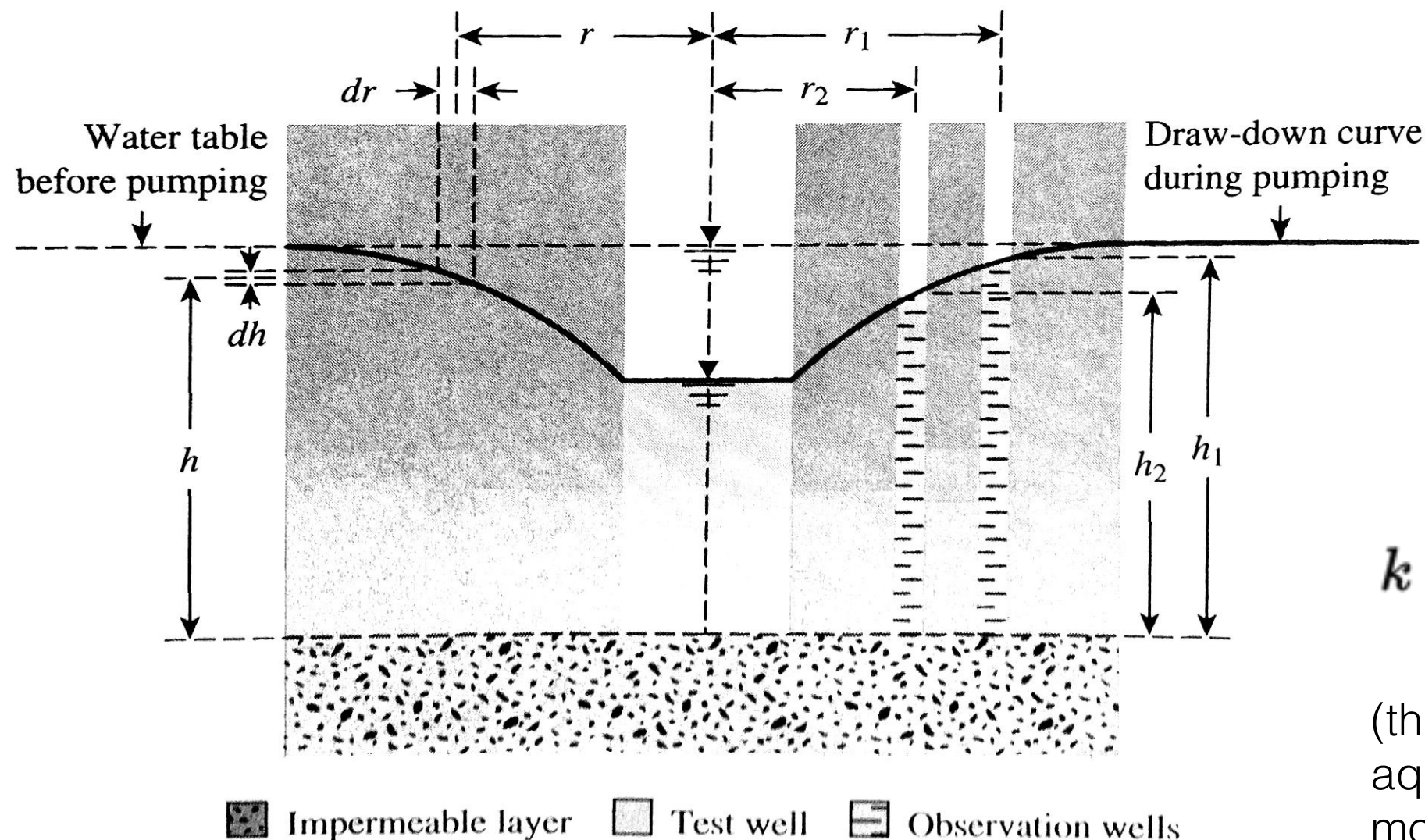
$$\text{or } k = \frac{aL}{At} \ln \left(\frac{h_1}{h_2} \right)$$

Hydraulic Conductivity

Pumping Test

It may be possible to test soil permeability directly in the field, with all local details of variability accounted for, by a pumping test.

This requires that the water table be accessible by pump wells:



1. Pump flow is established with constant head and flowrate.

2. The head at two observation wells r_1 and r_2 are recorded.

3. The conductivity k is proportional to the flowrate:

$$k = \frac{Q \ln \left(\frac{r_1}{r_2} \right)}{\pi (h_1^2 - h_2^2)}$$

(this theory depends on aquifer stratification, see more examples in the text)

Hydraulic Conductivity

Ranges and indications of k

Soils can be classified to a limited extent by hydraulic conductivity. The classification system below uses pore size as a classification correlated with drainage and soil type. The range of suitable test methods is also indicated.

K (m/s)	1	10^{-2}	10^{-4}	10^{-6}	10^{-8}	10^{-10}
Drainage	good		moderate	bad		
Classification	large	moderate	small	very small	impervious	
Soil type	gravel	clean sand	silt, clayey sand		clay	
Direct test	pumping test					
		dipool sounding				
	constant head					
		falling head				(oedometer)
Indirect test				piezocone		
				oedometer		
Correlation	grainsize distribution / density		density	index of clay		

A pumping test is conducted in the field using exploration well-drilling equipment or similar. Other indirect tests are available at the geotechnical specialist level.

Hydraulic Conductivity

Empirical and theoretical models

There are any number of models and theoretical estimates of k as a function of soil and water parameters such as porosity and viscosity.

These tend to be less than useful, as in practice individual soils do not match those parameterised in research. Conductivity is very sensitive to local details of soil formation.

This is not a problem however as a direct measurement of k is usually possible. Serious geotechnical work would not proceed without such tests.

-log K (cm/sec)	-2	-1	0	1	2	3	4	5	6	7	8	9	10	11
Permeability	Pervious				Semipervious				Impervious					
Aquifer	Good				Poor				None					
Soil	Clean gravel		Clean sand or sand & gravel			Very fine sand, silt loess, loam, solonetz								
					Peat		Stratified clay		Unweathered clay					
Rocks					Oil rocks Sandstone				Good limestone dolomite		Breccia granite			

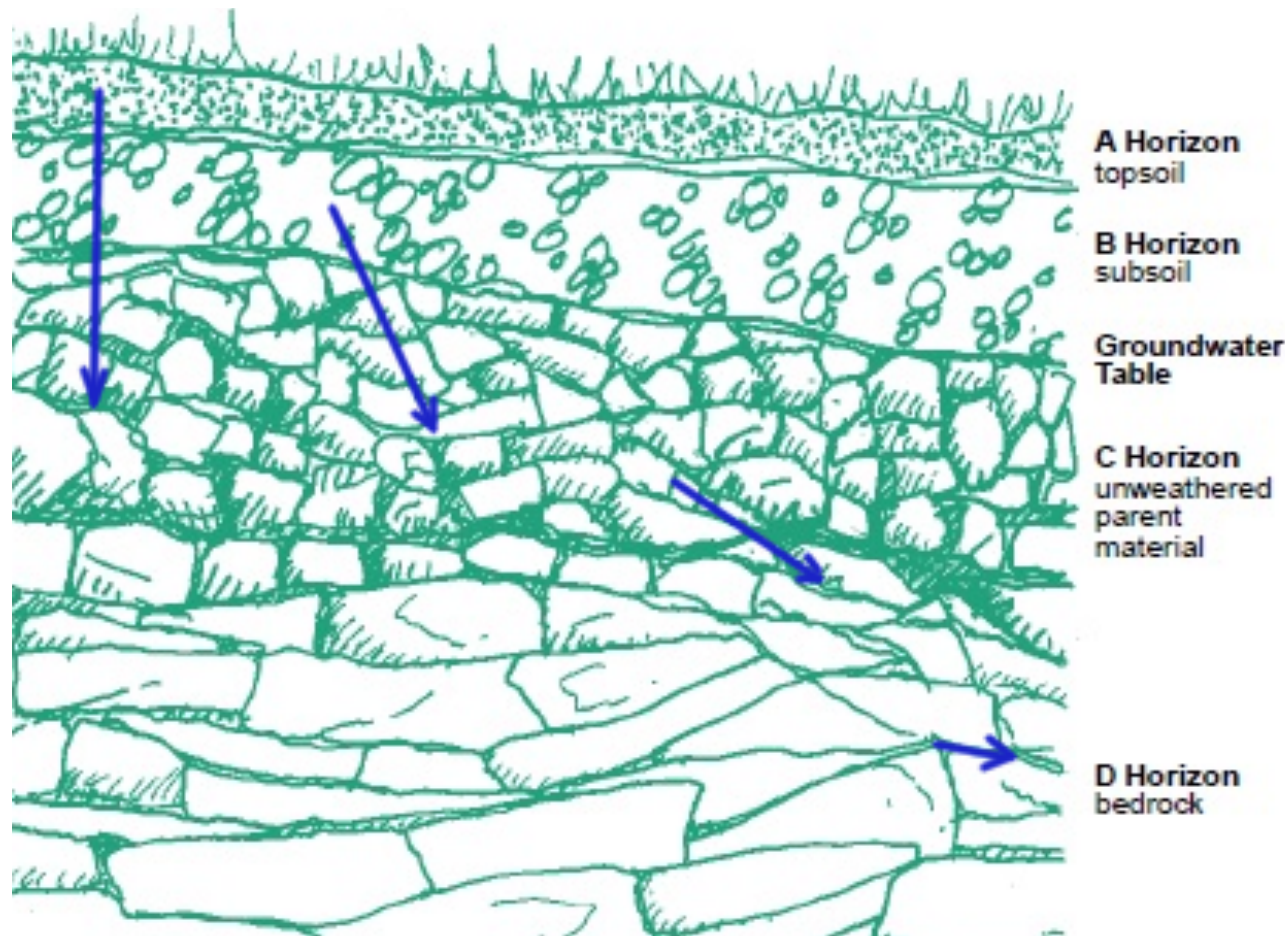
Hydraulic Conductivity

Homogeneity and Isotropy

A *homogenous* soil has a roughly constant value of hydraulic conductivity at all locations and depths in the profile and plan area of interest. This is very rare and cannot be assumed.

An *isotropic* soil has the same value of k in any direction from the point of measurement. This also is rare; most soils are anisotropic in horizontal and vertical directions due to orientations of structure.

Individual soil strata may be approximately homogenous, but drainage lines cross any number of strata at different depths. Over all strata, most soils are both *inhomogenous* and *anisotropic*.



Soil profile with typical stratified discharge velocity vectors. Conductivity decreases with depth as compaction and the proportion of native rock both increase.

Preferred flow directions vary from vertical to horizontal due to both the hydraulic gradient and the ratio of vertical to horizontal conductivity.

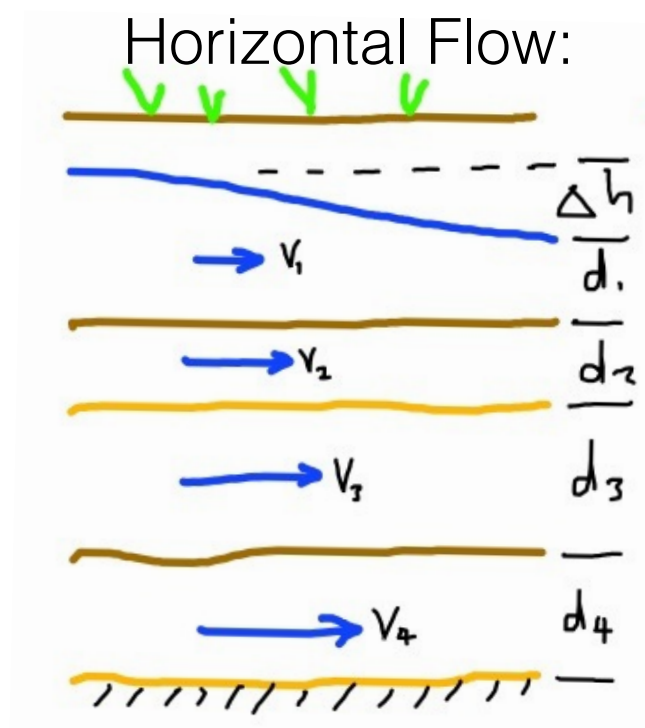
A Horizon soils have a dominant vertical flow due to vertical drying cracks and the vertical structure of macropores. C horizons tends to horizontal flow due to strata fracturing and the underlying impervious bedrock.

Hydraulic Conductivity

Anisotropic Soils

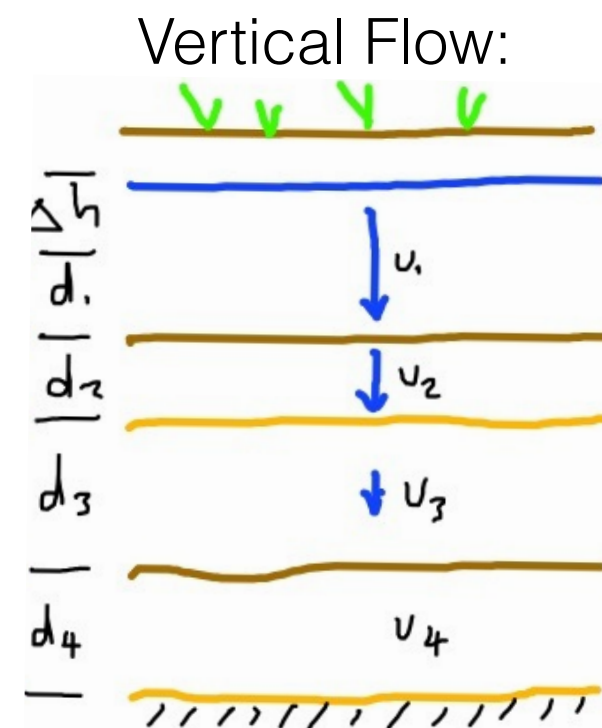
Calculations of flow across or along stratification layers can be simplified by combination formulas for *average* vertical and horizontal conductivities.

These weighted averages are analogous to electrical resistance in parallel or series. The average horizontal and vertical conductivities k_h and k_v can be estimated as:



$$Q = \sum Q_i = \sum k_i d_i \frac{dh}{dl} = k_h \frac{dh}{dl} \sum d_i$$

$$\text{so } k_h = \frac{\sum k_i d_i}{\sum d_i}$$



$$\Delta h = \sum \frac{v_i d_i}{k_i} = \frac{\bar{v}}{k_v} \sum d_i$$

$$\text{but } \bar{v} = \sum \frac{v_i d_i}{d_i} \text{ so } k_v = \frac{\sum d_i}{\sum \frac{d_i}{k_i}}$$

Hydraulic Conductivity

Example



A soil profile has the following strata:

$$d_A = 400 \text{ mm} \quad k_{hA} = 0.004 \text{ cm/s} \quad k_{vA} = 0.009 \text{ cm/s}$$

$$d_B = 700 \text{ mm} \quad k_{hB} = 0.007 \text{ cm/s} \quad k_{vB} = 0.005 \text{ cm/s}$$

$$d_C = 900 \text{ mm} \quad k_{hC} = 0.009 \text{ cm/s} \quad k_{vC} = 0.001 \text{ cm/s}$$

Calculate average horizontal and vertical hydraulic conductivities.

$$k_h = \frac{\sum k_i d_i}{\sum d_i}$$

$$k_v = \frac{\sum d_i}{\sum (d_i / k_i)}$$

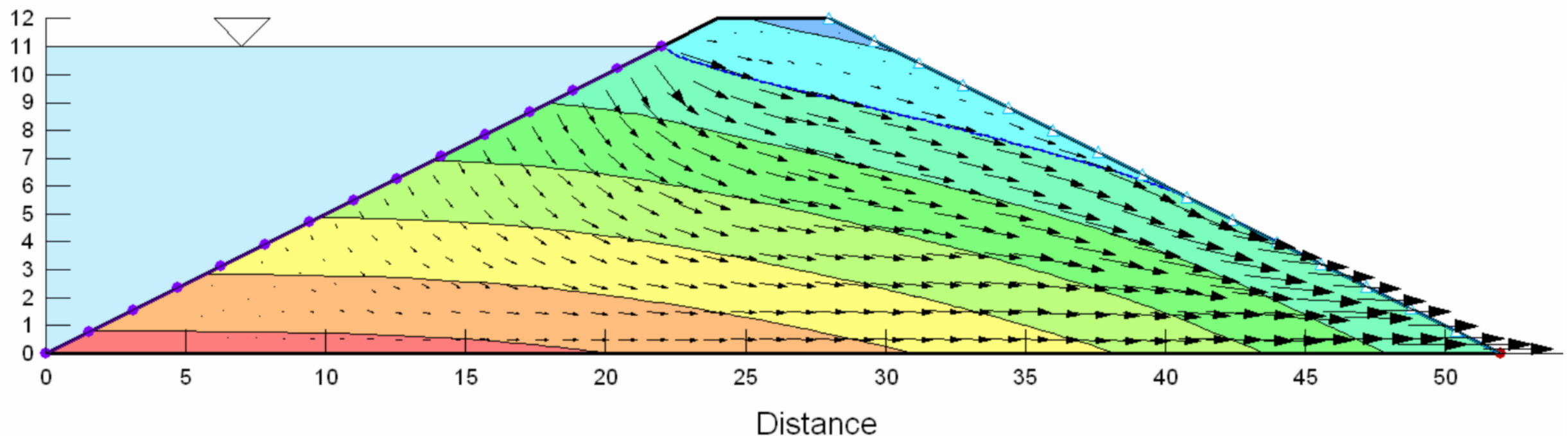
Hydraulic Conductivity

Computer Programs

Hydraulic conductivity design and analysis gets much more complex than this.

Modern geotechnical engineering makes heavy use of computer power to analyse even routine flow paths for stability.

ENG350 will make use of the student edition of Geoslope; reasonably advanced geotechnical software for seepage, slope stability and other problems.

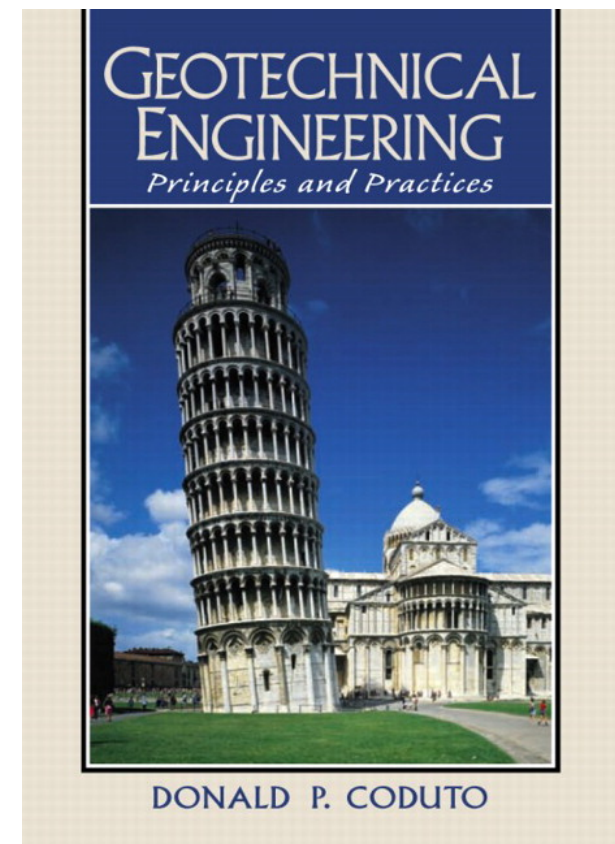
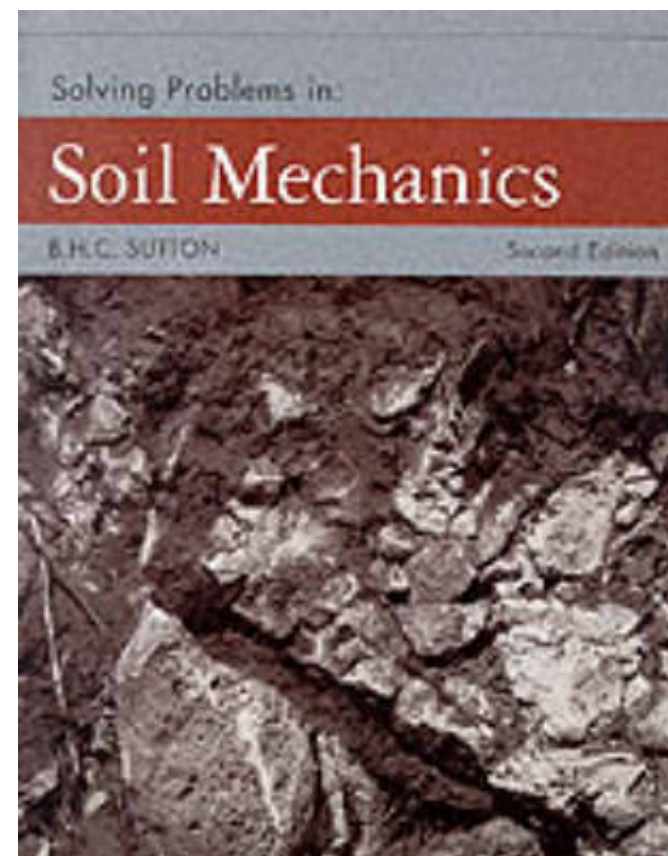
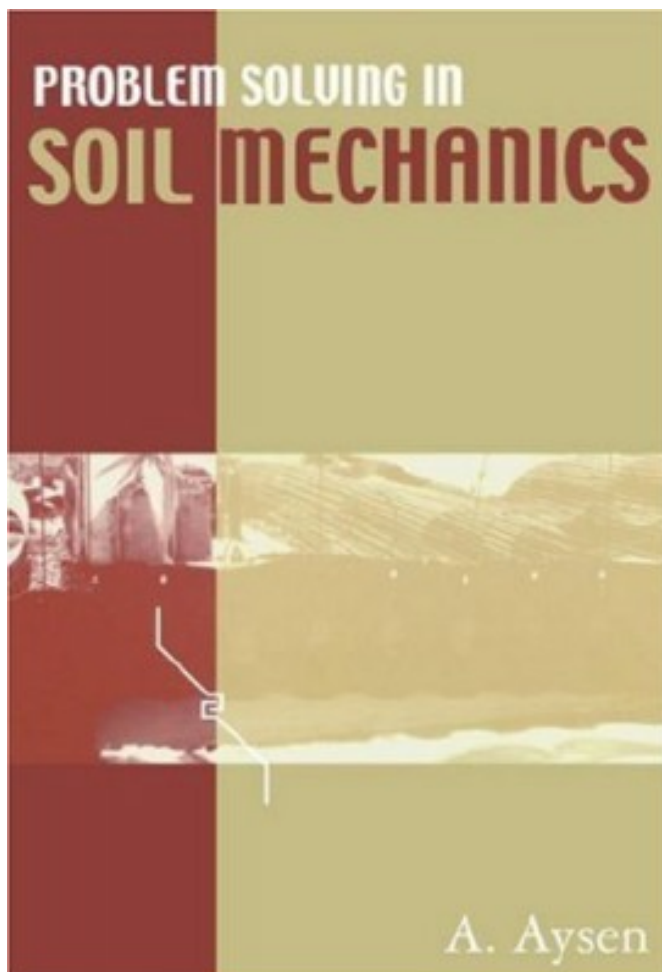
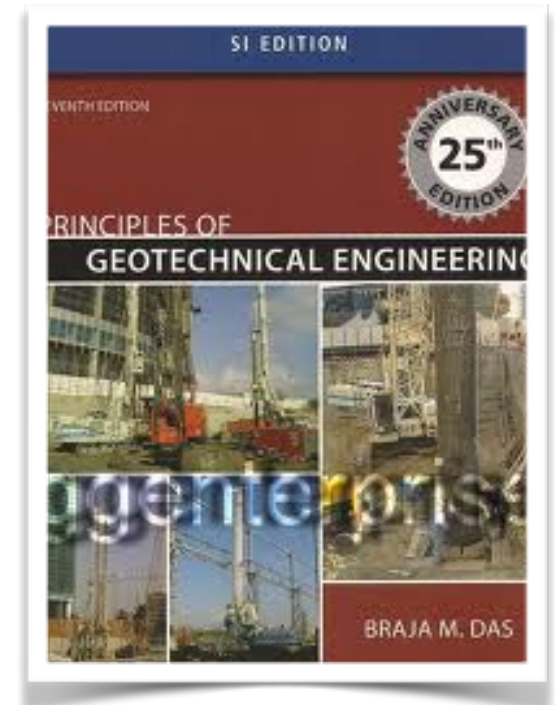


Readings

Chapter 7 Das

Glencross-Grant, 2010. *Soil Water*. Reading 6

Internet research: Geoslope student edition software.

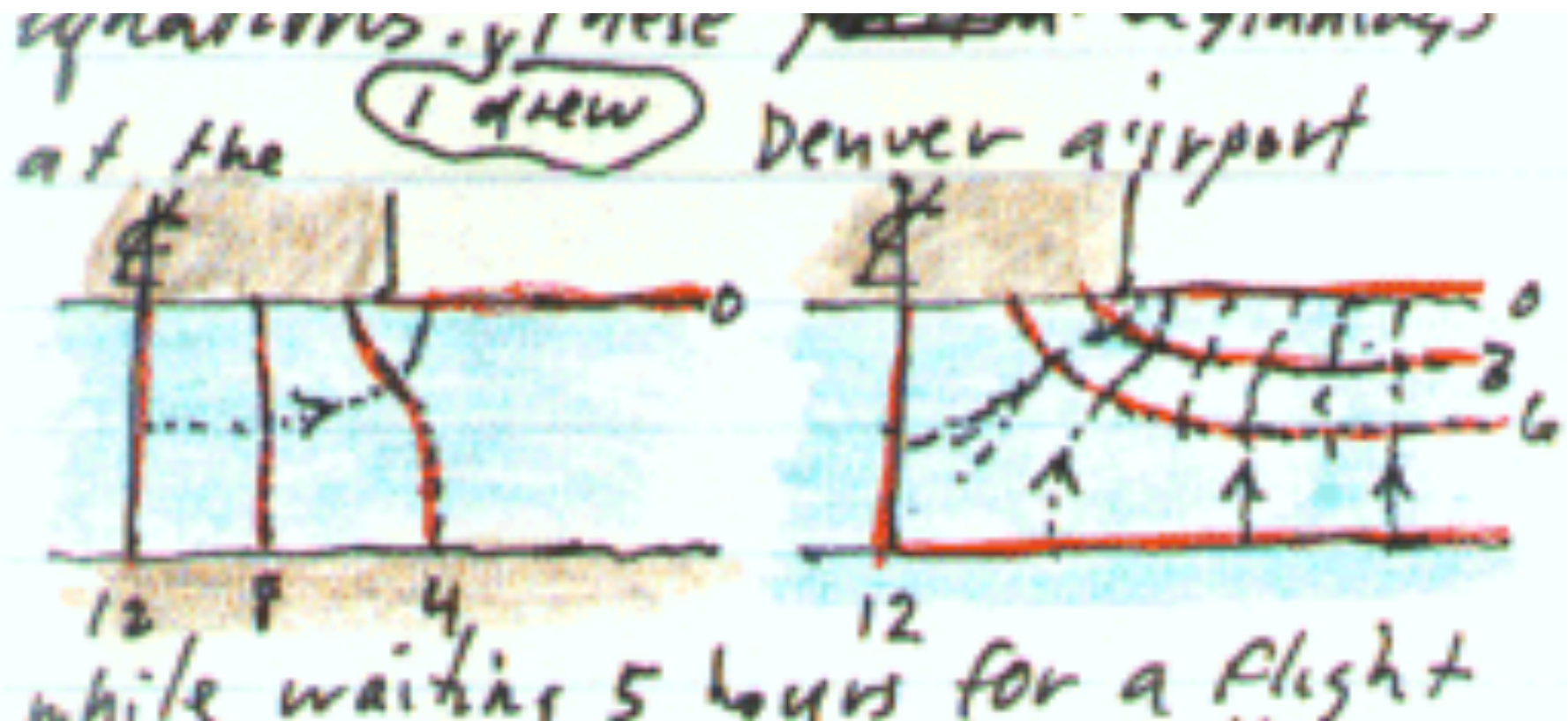
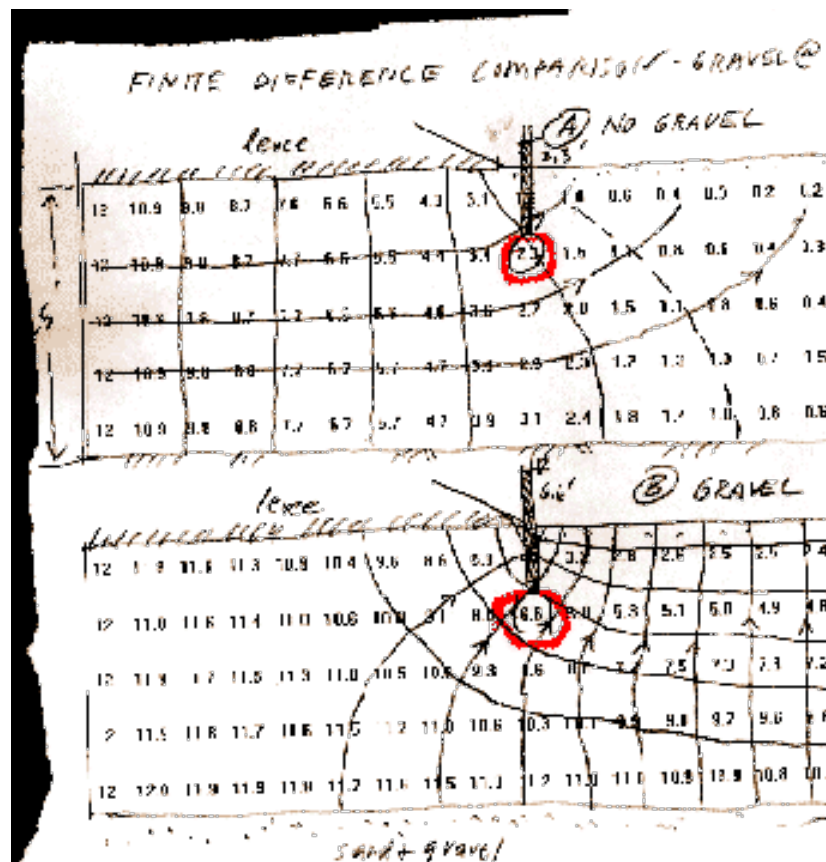


Seepage Flow Nets

Text: Das, 2010. Chapter 8

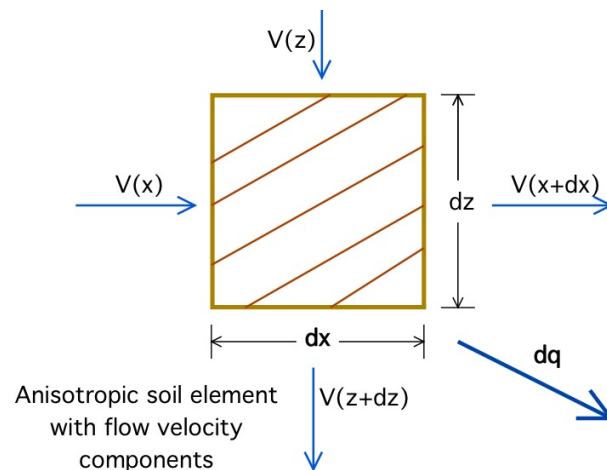
Darcy's law is generally adequate for one dimensional flow analysis, ie where the geometry of the flow is not complex and where simplifying assumptions carry no extra risk. This is rarely the case for realistic engineering design work, other than as a first estimate.

For 2nd-order analysis, a hand-drawn 2-dimensional diagram known as a *flow net*, using theory based on continuity and permeability, is a standard design detail. These are often prepared as a preliminary to a full design analysis using 3d computer models, and are essential for developing your understanding of groundwater flow.



Flow Net Potential Theory

Darcy's Law and Continuity (in the fluid mechanics sense) provide sufficient conditions for construction of a flow net diagram:



Continuity : Inflow = Outflow : $Q = VA$ or $q = V\ell$

q is the flow per unit depth into the page, ℓ is a length such as dx or dz at right angles to the flow, V is the discharge velocity as per Darcy's law.

$$dq = V_x dz + V_z dx = V_{(x+dx)} dz + V_{(z+dz)} dx$$

$$\text{so } (V_x - V_{(x+dx)}) dz = (V_{(z+dz)} - V_z) dx$$

$$\text{but } (V_{(z+dz)} - V_z) = \partial V_z$$

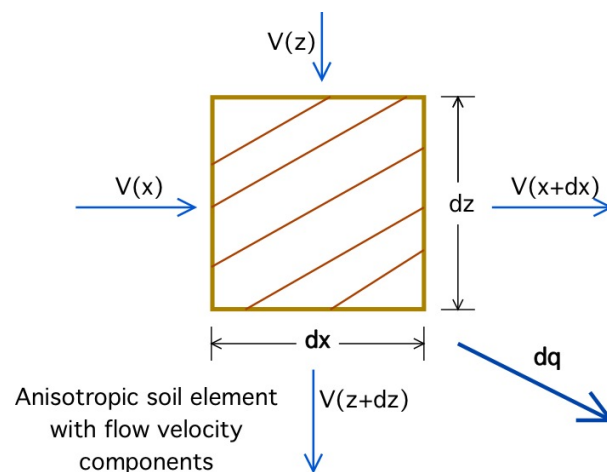
$$\text{and } (V_x - V_{(x+dx)}) = -\partial V_x$$

$$\text{so } -\frac{\partial V_x}{\partial x} = \frac{\partial V_z}{\partial z} \quad \text{or} \quad \boxed{\frac{\partial V_x}{\partial x} + \frac{\partial V_z}{\partial z} = 0}$$

The highlighted expression is a simplified form of the differential continuity equation, which is equivalent to the more familiar $Q = VA$

Flow Net Potential Theory

Darcy' law enables us to replace unknown velocities with known or measurable headloss rates and permeabilities:



$$V_x = -k_x \frac{\partial h}{\partial x} \quad \text{and} \quad V_z = -k_z \frac{\partial h}{\partial z}$$

Darcy's law: the minus signs indicate +ve velocity for decreasing head (negative hydraulic slope)

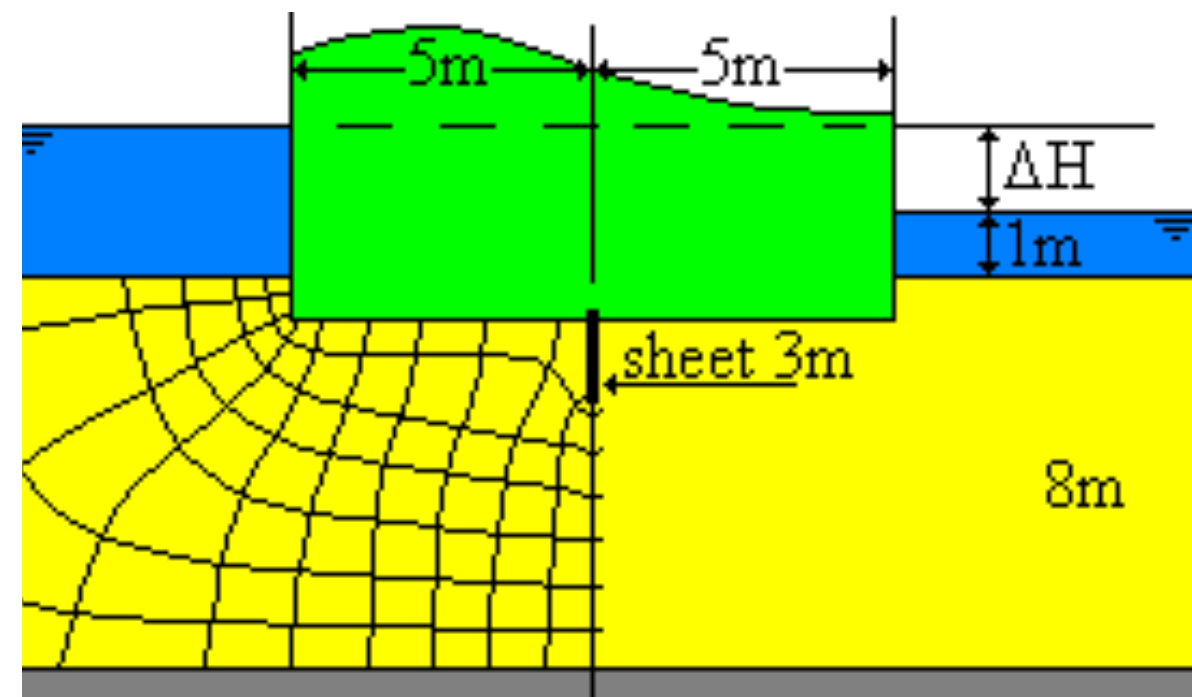
$$\text{so} \quad \frac{\partial V_x}{\partial x} + \frac{\partial V_z}{\partial z} = 0$$

$$\text{becomes} \quad k_x \frac{\partial^2 h}{\partial x^2} + k_z \frac{\partial^2 h}{\partial z^2} = 0$$

The highlighted expression is a modified form of the *Laplace equation*, a 2-dimensional differential equation which is consistent with both Darcy's law and continuity.

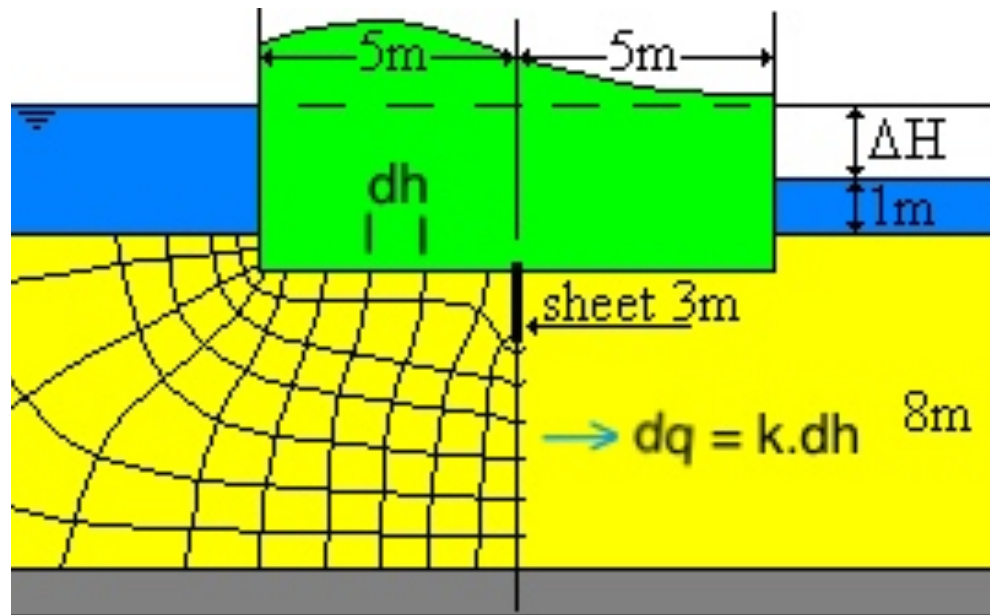
When $k_x = k_z = k$ (isotropic soil), the Laplace equation has a useful property of *orthogonality*, that is the contours of constant head are always at right angles to the streamlines.

As such the equation can be solved graphically, by sketching orthogonal flow nets with the contours always intersecting at right angles.



Flow Net Streamlines

Streamlines are never crossed by the flow. Therefore the flow within each stream tube (between pairs of streamlines) is constant.



For simplicity we draw lines of constant head at constant *increments* of head dh , and streamlines at constant increments of discharge dq .

The flow net becomes orthogonal when we also let $dq = k.dh$

Then the total headloss ΔH and total flow per unit width q are simply functions of the number of lines we include in the sketch:

$$\Delta H = N_{\phi} dh$$

$$q = N_{\psi} dq$$

where N_{ϕ} is the number of equipotential intervals (between lines of constant head) and N_{ψ} is the number of streamtubes (intervals between streamlines). (Greek letters ϕ phi and ψ psi will be explained in a moment)

but $dq = k.dh$

$$\text{so } q = N_{\psi} dh = k \frac{N_{\psi}}{N_{\phi}} \Delta H$$

A well drawn flow net with a known headloss ΔH , conductivity k and streamlines (ψ) orthogonal to potential lines (ϕ) gives a useful indication of the seepage flowrate.

Flow Net Graphical Method

A graphical flow net is sketched and re-sketched until the lines intersect at right angles.

1. Draw a long section of the surface and groundwater channels including any infrastructure such as dams or sheet piles.

2. Draw water surfaces at upstream and downstream ends of the channel

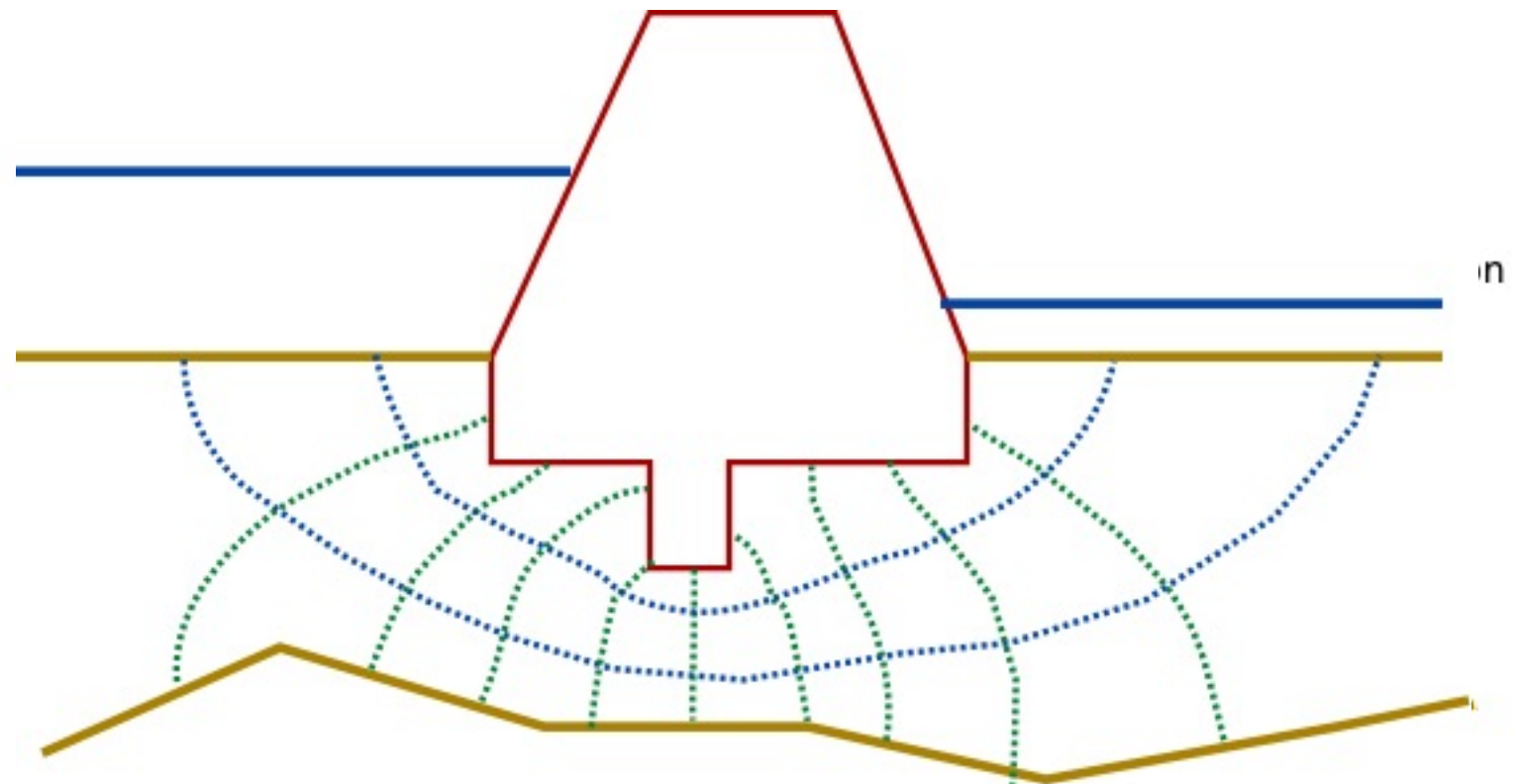
3. Identify head and headloss at start and endpoints of the flow path

4. Identify any boundary conditions such as impermeable strata (bedrock)

5. Sketch a primary streamline dividing the groundwater channel into two flow tubes. The streamline is perpendicular to any lines of constant head in the channel, eg the up- and downstream ends. THIS IS A GUESS. Use symmetry to position the line well.

6. Sketch a few lines of constant head to divide the flow path into say four distinct bands. Head lines are perpendicular to boundaries and streamlines

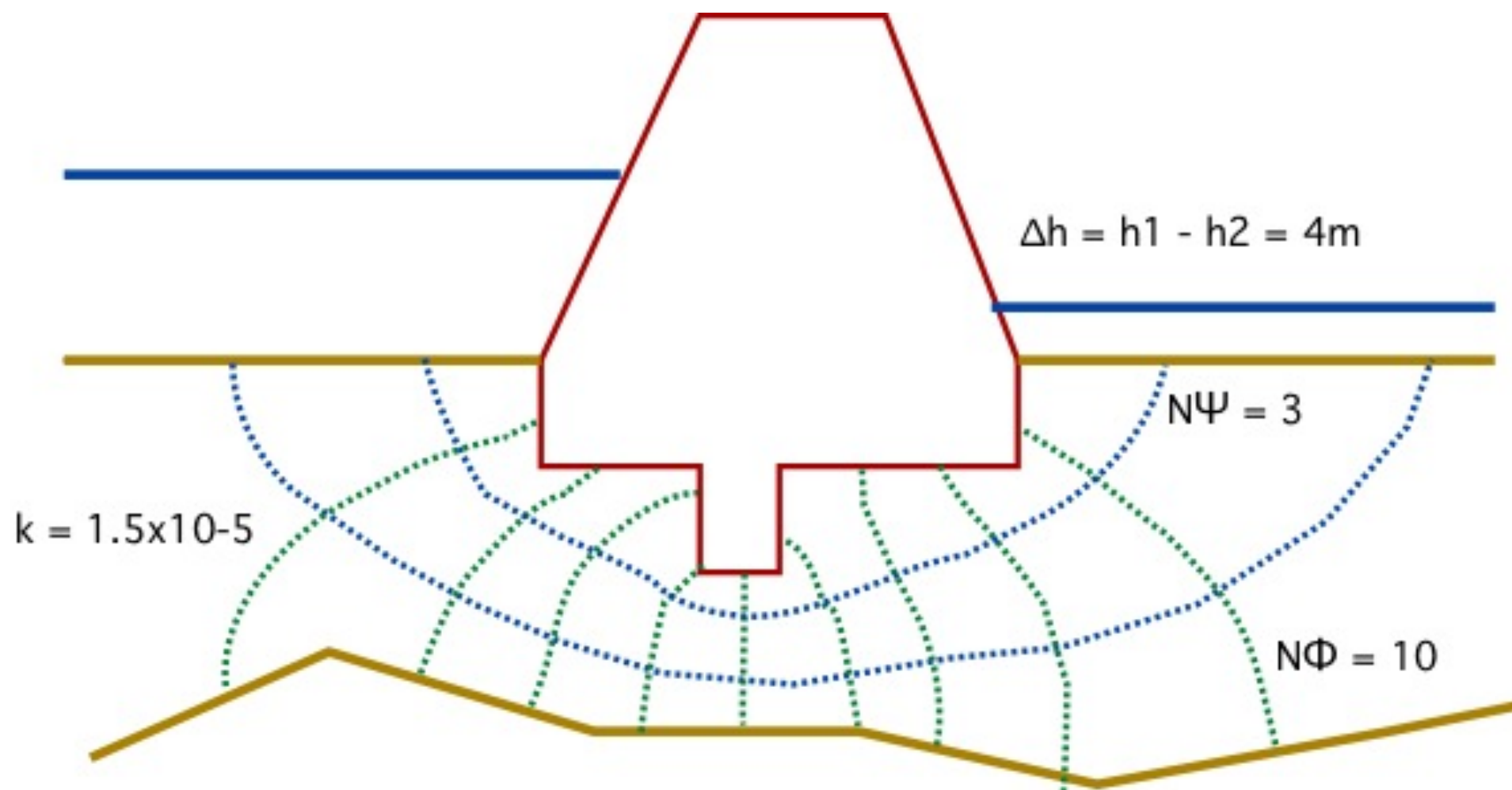
7. Continue sketching streamlines and potential lines, erasing and correcting where necessary to obtain an orthogonal-looking flow net.



A certain amount of approximation usually remains at the end. Graphical software is useful for speeding this up

Flow Net Graphical Method

To a reasonable level of accuracy, $q = k \frac{N_\psi}{N_\phi} \Delta H$



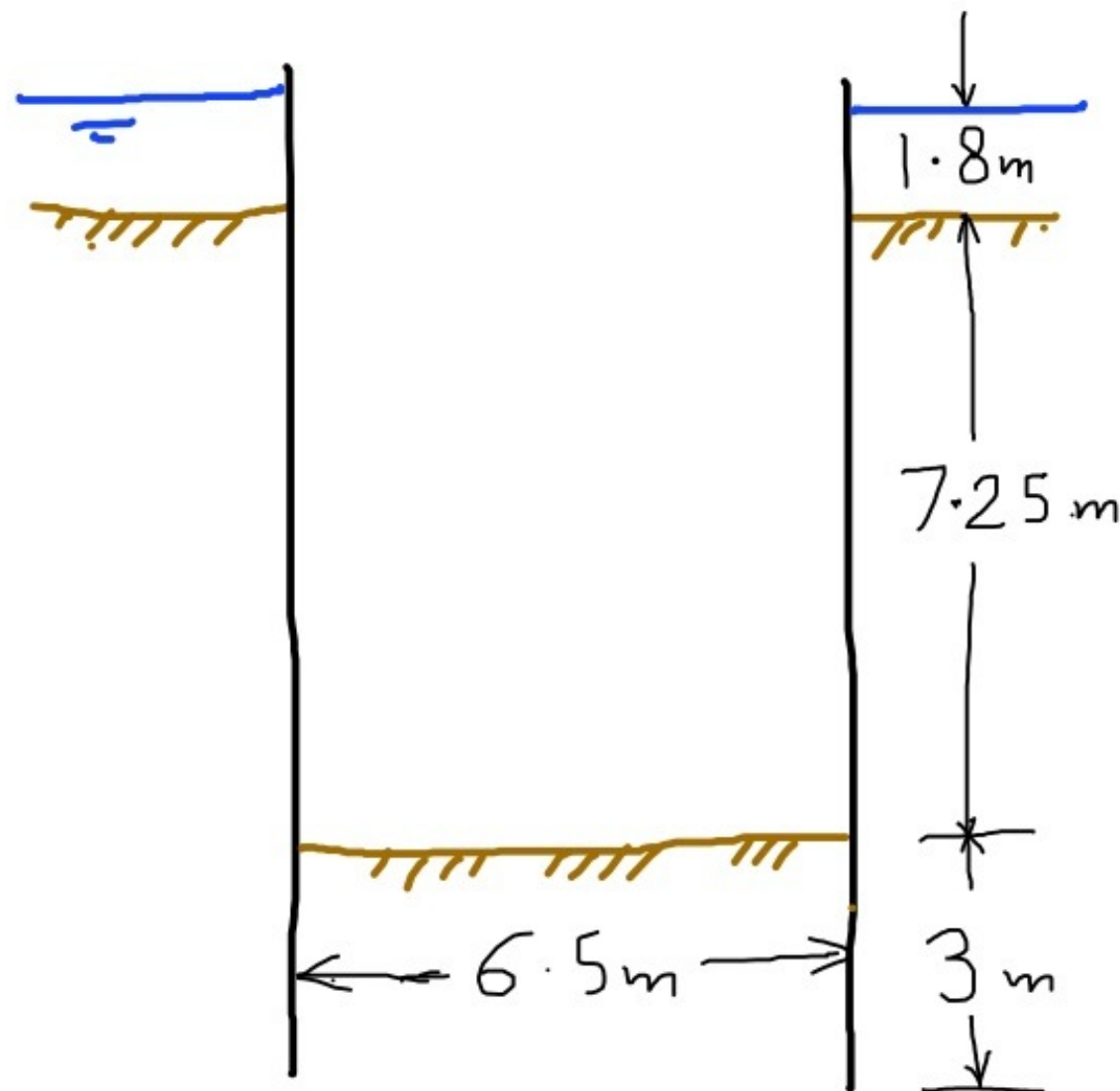
$$q = 1.5 \times 10^{-5} \times 4 \times 3 / 10$$

$$= 1.8 \times 10^{-5} \text{ m}^2/\text{s}$$

Flow Net Graphical Method

Example: The long cofferdam shown can be considered to have 2-dimensional seepage flow. Determine the seepage per metre width into the dam if the water level is maintained at the excavated lower level. Assume isotropic soil, $k = 0.015 \text{ m/s}$, $\Delta H = 9.05 \text{ m}$

$$q = k \frac{N_{\psi}}{N_{\phi}} \Delta H$$



Flow Net Graphical Method

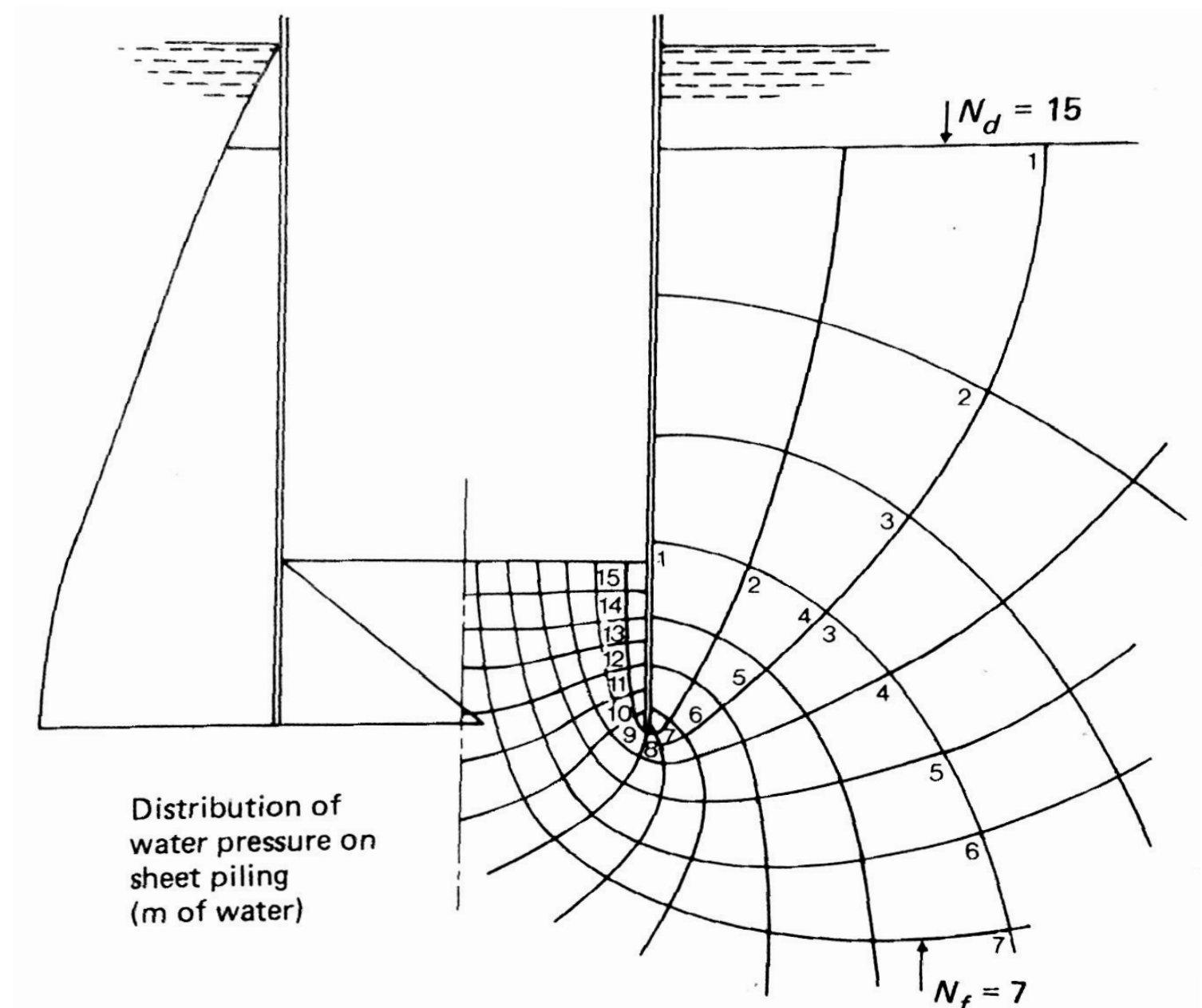
Solution: No of potential steps = 15; No of flow tubes = 7

Seepage:

$$q = 0.015 \times 9.05 \times 7/15$$
$$= 0.063 \text{ m}^3/\text{s}/\text{m}$$

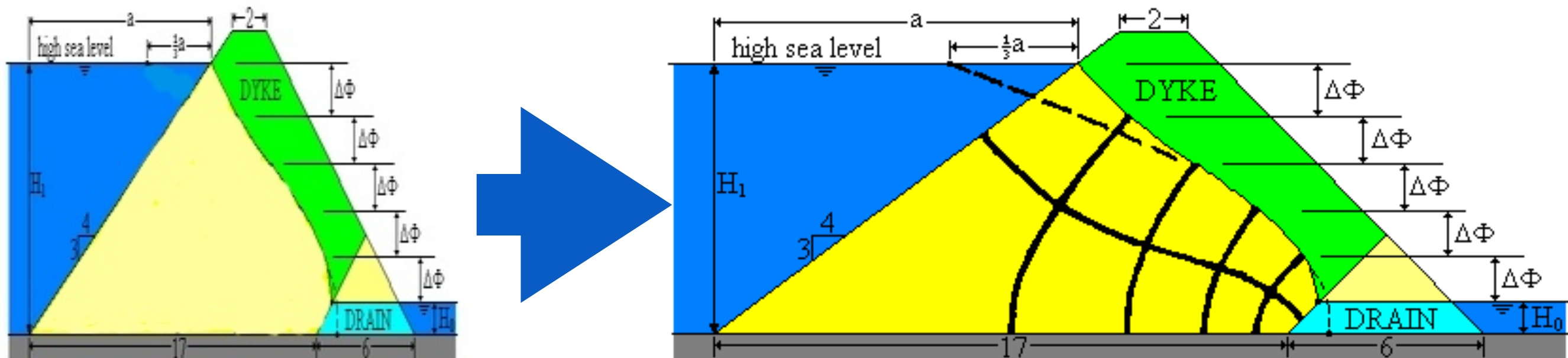
This is for half the dam ...

$$q = 0.126 \text{ m}^3/\text{s}/\text{m}$$



Flow Nets in Anisotropic Soils

When k_x and k_z are unequal, the graphical method can be applied by coordinate transformation; "stretching" of the x axis.



Channel geometry is transformed by a horizontal scale factor (about 2 in this case) then the orthogonal flow net is drawn as before.

The scale factor is:

$$x' = x \sqrt{\frac{k_z}{k_x}}$$

The seepage is then:

$$q = \sqrt{k_z k_x} \frac{N_\psi}{N_\phi} \Delta H$$

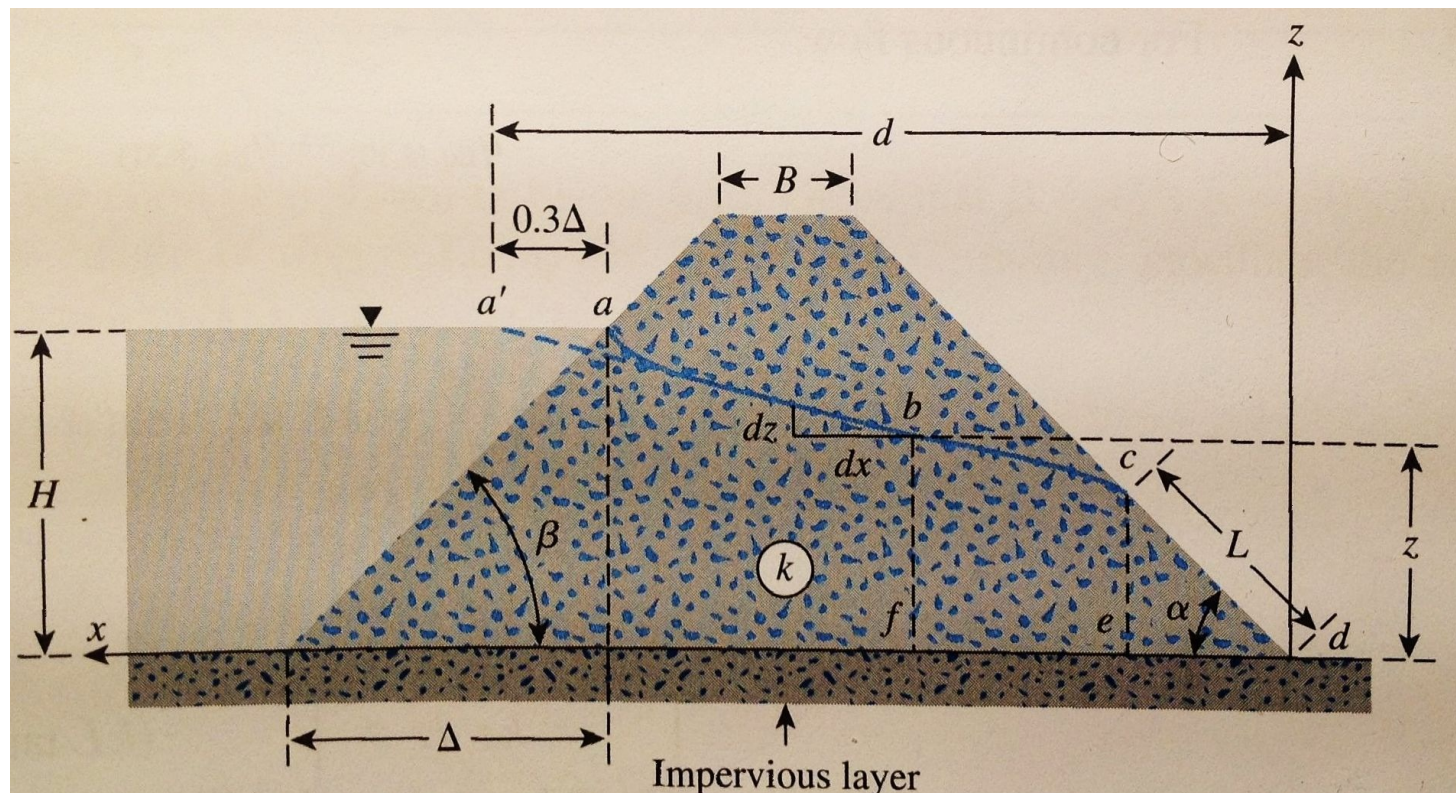
The method is modified:

1-4. Draw a longsection of the surface and groundwater channels ... *on the transformed x axis* ...

5-7. Draw an orthogonal flownet ... *on the transformed channel* ...

Earth Dam Seepage: Easy Solution

For the limited case of an earth dam constructed of isotropic soil on an impervious base (not an unrealistic assumption for disturbed or engineered soils), a simple analytic solution exists.



$$q = kiA$$

$$i = \frac{dz}{dx} = \tan(\alpha)$$

$$A = L \sin(\alpha)$$

$$q = kL \tan(\alpha) \sin(\alpha)$$

This simple method is adequate for exactly this type of earth dam, which is not uncommon in light urban or agricultural engineering. The face length L (a design unknown) can be estimated for assumed water levels (the 0.3Δ correction is due to Casagrande):

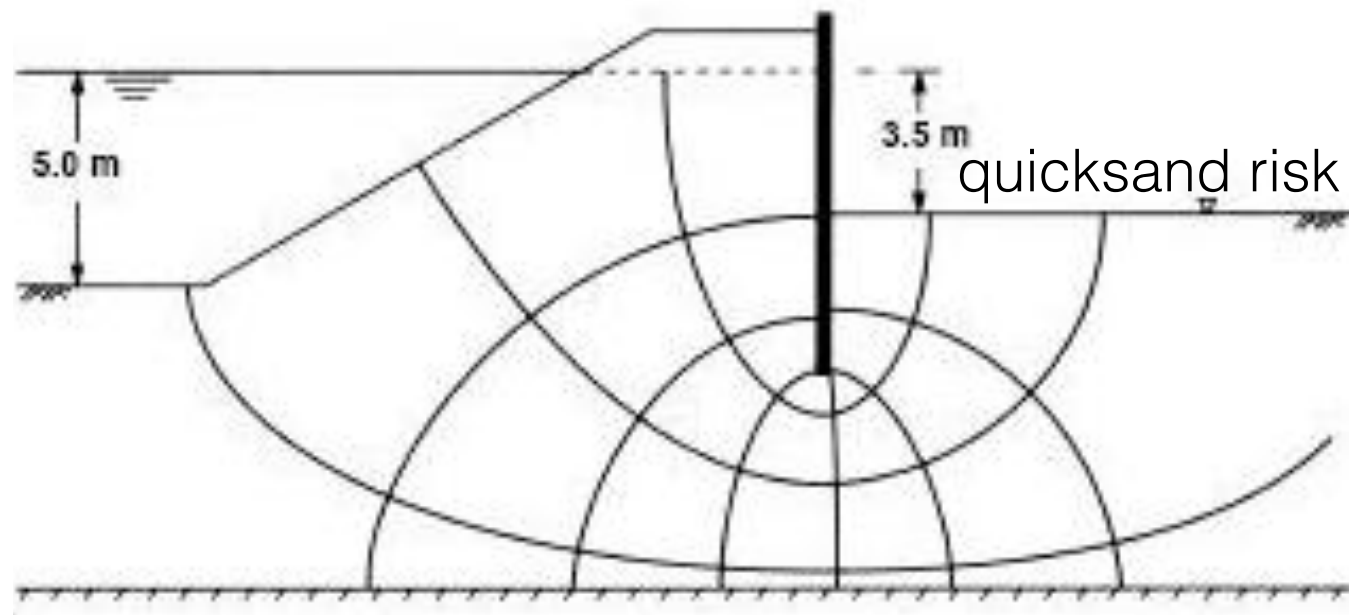
$$L = \frac{d}{\cos(\alpha)} - \sqrt{\frac{d^2}{\cos^2 \alpha} - \frac{H^2}{\sin^2 \alpha}}$$

$$d = W - \frac{0.7H}{\tan \beta}$$

W is the width from heel to toe of the base

Soil Stress due to Seepage

Groundwater flow causes frictional drag on soil particles. Rising groundwater may create sufficient lift to suspend particles in a "quick" condition.



Since $V = k \cdot dh/dl$ we find higher velocities wherever the friction slope (hydraulic gradient) is higher.

The friction slope is indicated by the spacing of the flownet grid; closer spacing indicates a higher rate of headloss, therefore higher velocity.

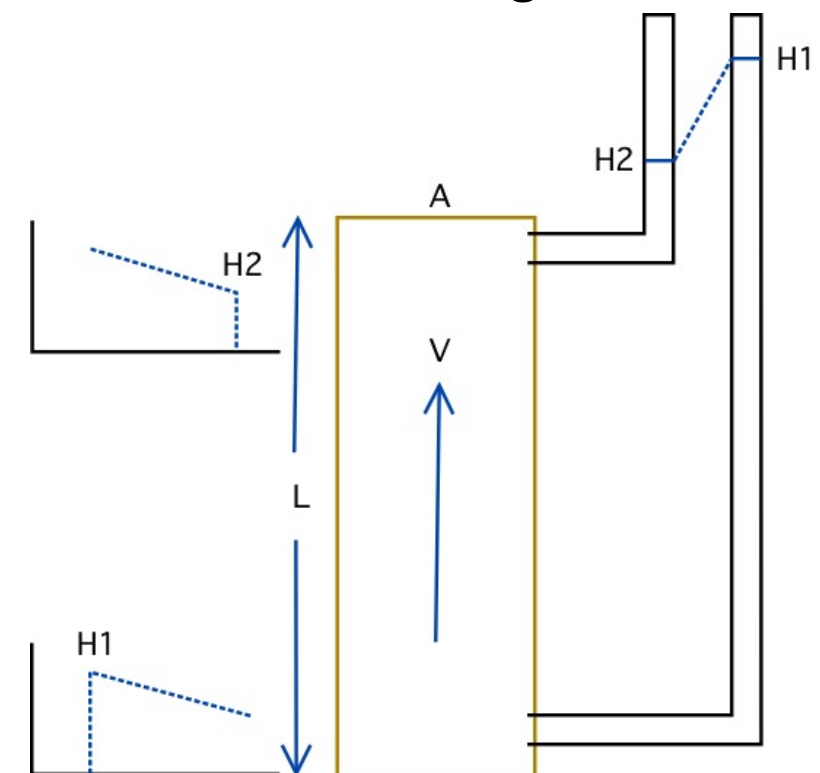
A rising flow is balanced against the submerged (buoyant or effective) weight of a soil column

Effective soil stress:
$$\sigma' = \gamma' L = \gamma_w \left(\frac{G - 1}{1 + e} \right) L$$

Pressure drop due to friction:
$$\Delta P = \gamma_w \Delta H = \gamma_w \frac{dH}{dL} L = \gamma_w i L$$

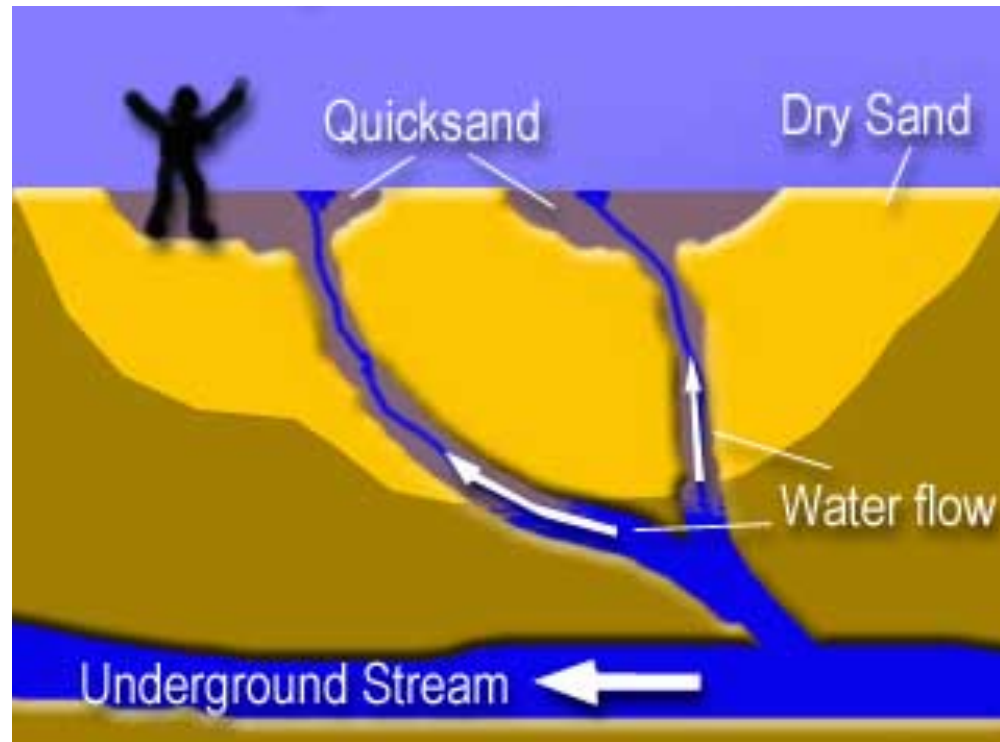
Stress = Pressure at critical hydraulic gradient:

$$\sigma' = \Delta P \quad : \quad i_c = \frac{G - 1}{1 + e}$$



Soil Stress due to Seepage

There is a critical hydraulic gradient which in rising flows will cause uplift of the soil:



$$i_c = \frac{G - 1}{1 + e}$$

This phenomenon goes by a number of names: "Quicksand"; "Piping" or "Boiling"

Soil is lifted and transported by the flow, with a loss of structural integrity of soil.

Factors of safety against piping ensure that hydraulic gradients in rising flows are well less than critical:

$$FSP = \frac{i_c}{i_{max}} \geq 1.5$$

For many soils, $G \sim 2.7$ and $e \sim 0.7$ so $i_c \sim 1$

Geotextile filters can be layered into soil wherever piping may occur. These are heavy fabric blankets that hold the soil in place while allowing flow to seep through.

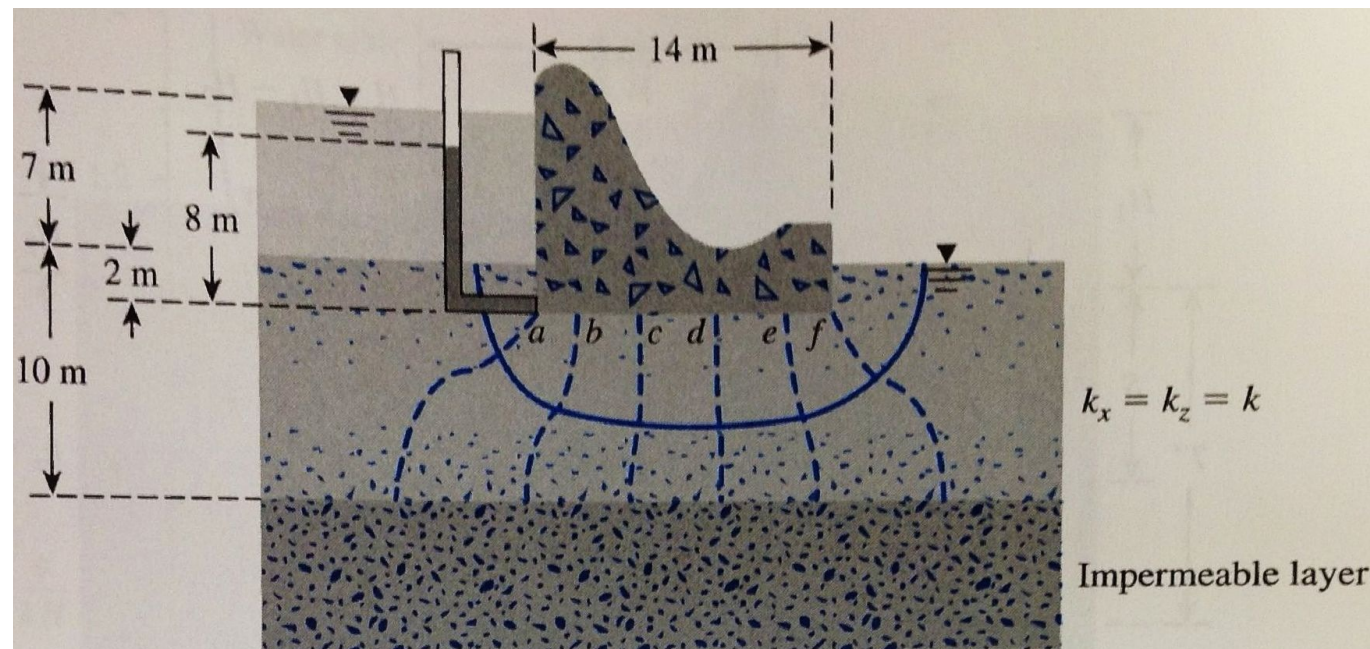
Specifications for filters were originally developed by Terzaghi. See the reading for more information.

Garah Dam Failure 1981

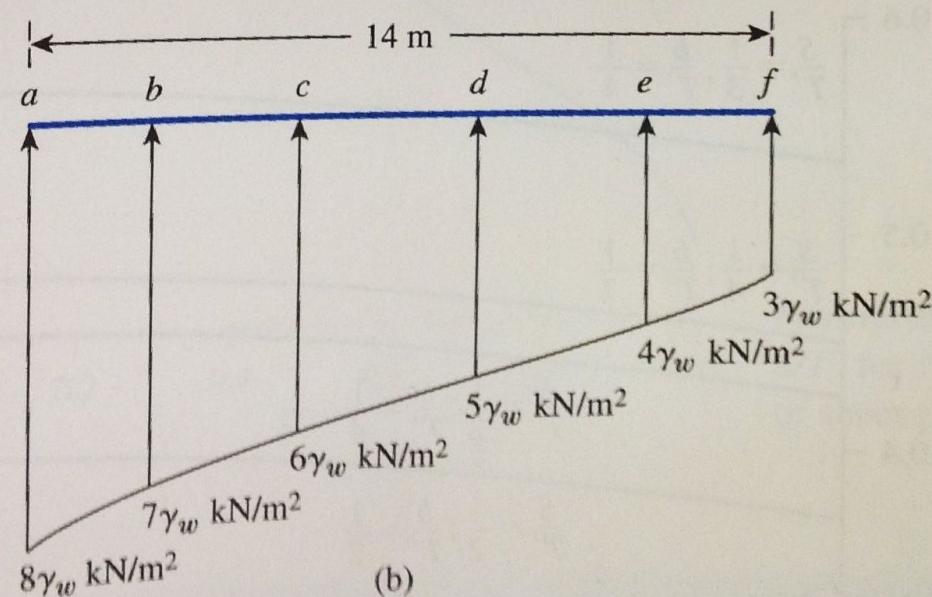


Uplift Pressure under Hydraulic Structures

Flow nets can be used to determine the uplift pressure at the base of a hydraulic structure.



(a)



(b)

There are 7 potential drops (N_ϕ) in the flownet, and the total difference in head across the net is 7 m. Each drop is 1 m.

The head at each potential line is converted directly to pressure:

$$P_a = (7+2-1)\rho g = 8\text{kPa}$$

$$P_b = (7+2-2)\rho g = 7\text{kPa} \dots \text{etc}$$

The uplift force is found by integrating pressure over area:

$$F_{ab} = \frac{P_a + P_b}{2} W L_{ab}$$

$$F = F_{ab} + F_{bc} + \dots + F_{ef}$$

Uplift failure is a significant factor in dam design.

Readings

Chapter 8 Das

Glencross-Grant, 2010: *Flow Nets* Reading 7

Internet research: Flownets, potential flow, quicksand.

