EXIT EXAM MODEL QUESTIONS

Soil Mechanics I
&
Soil Mechanics II

Prepared By Ahmed M.(M.Sc. Geotechnical Eng.)

Soil Mechanics I & Soil Mechanics II

THE *ESSENTIAL POINTS* ARE:

1. **Soils are derived from the weathering of rocks and are commonly described by textural terms such as gravels, sands, silts, and clays.**
2. **Physical weathering causes reduction in size of the parent rock without change in its composition.**
3. **Chemical weathering causes reduction in size and chemical composition that differs from the parent rock.**
4. **Clays are composed of three main types of mineral—kaolinite, illite, and montmorillonite.**
5. **The clay minerals consist of silica and alumina sheets that are combined to form layers. The bonds between layers play a very important role in the mechanical behavior of clays. The bond between the layers in montmorillonite is very weak compared with kaolinite and illite. Water can easily enter between the layers in montmorillonite, causing swelling.**
6. **A thin layer of water, called adsorbed water, is bonded to the mineral surfaces of soils. This layer significantly influences the physical and mechanical characteristics of fine-grained soils.**

Soil Mechanics I & Soil Mechanics II

THE *ESSENTIAL POINTS* ARE:

1. **Fine-grained soils have much larger surface areas than coarse-grained soils and are responsible for the major physical and mechanical differences between coarse-grained and fine-grained soils.**
2. **The engineering properties of fine-grained soils depend mainly on mineralogical factors.**
3. **Coarse-grained soils have good load-bearing capacities and good drainage qualities, and their strength and volume-change characteristics are not significantly affected by changes in moisture conditions.**
4. **Fine-grained soils have poor load-bearing capacities and poor drainage qualities, and their strength and volume-change characteristics are significantly affected by changes in moisture conditions.**

Water content **(w)** is the ratio of the weight of water to the weight of solids.

Void ratio **(e)** is the ratio of the volume of void space to the volume of solids.

Porosity **(n)** is the ratio of the volume of void to the total volume of soil.

Degree of saturation **(S)** is the ratio of the volume of water to the volume of void.

Bulk unit weight **(γ)** is the weight density, that is, the weight of a soil per unit volume.

Saturated unit weight **(γ_{sat})** is the weight of a saturated soil per unit volume.

Dry unit weight **(γ_d)** is the weight of a dry soil per unit volume.

Effective unit weight **(γ')** is the weight of soil solids in a submerged soil per unit volume.

Relative density **(D_r)** is an index that quantifies the degree of packing between the loosest and densest state of coarse-grained soils.

Swell factor **(*SF*)** is the ratio of the volume of excavated material to the volume of in situ material (sometimes called borrow pit material or bank material).

Liquid limit **(*LL*)** is the water content at which a soil changes from a plastic state to a liquid state.

Plastic limit **(*PL*)** is the water content at which a soil changes from a semisolid to a plastic state.

Shrinkage limit **(*SL*)** is the water content at which a soil changes from a solid to a semisolid state without further change in volume.

Soil Mechanics I & Soil Mechanics II

THE *ESSENTIAL POINTS* ARE:

1. **Fine-grained soils can exist in one of four states: solid, semisolid, plastic, or liquid.**
2. **Water is the agent that is responsible for changing the states of soils.**
3. **A soil gets weaker if its water content increases.**
4. **Three limits are defined based on the water content that causes a change of state. These are the liquid limit—the water content that caused the soil to change from a liquid to a plastic state; the plastic limit—the water content that caused the soil to change from a plastic to a semisolid; and the shrinkage limit—the water content that caused the soil to change from a semisolid to a solid state. All these limiting water contents are found from laboratory tests.**
5. **The plasticity index defines the range of water content for which the soil behaves like a plastic material.**
6. **The liquidity index gives a qualitative measure of strength.**
7. **The soil strength is lowest at the liquid state and highest at the solid state.**

Soil Mechanics I & Soil Mechanics II

THE *ESSENTIAL POINTS* ARE:

1. Compaction is the densification of a soil by the expulsion of air and the rearrangement of soil particles.
2. The Proctor test is used to determine the maximum dry unit weight and the optimum water content and serves as the reference for field specifications of compaction.
3. Higher compactive effort increases the maximum dry unit weight and reduces the optimum water content.
4. Compaction increases strength, lowers compressibility, and reduces the rate of flow water through soils.

Soil Mechanics I & Soil Mechanics II

1.The ratio of the volume of voids to the total volume of soil is:

(a) Voids ratio

(c) Air Content

(d) Porosity

(b) Degree Of saturation

Solution

(a) Void ratio (e) is the ratio of the volume of void space to the volume of solids. Void ratio is usually expressed as a decimal quantity.

(b) Porosity (n) is the ratio of the volume of voids to the total volume. Porosity is usually expressed as a percentage.

Clay soils can have void ratios greater than 1.

(c) Air content (a_c) Volume of air to volume of void

(d) Degree of saturation (S) is the ratio, often expressed as a percentage, of the volume of water to the volume of voids:

Soil Mechanics I & Soil Mechanics II

2.Dry density of soil is equal to the:

(a)Mass or solids to the volume of solids.

(b)Mass of solids to the total volume of soil.

(c)Density Of soil in the dried condition.

(d) None of the above.

Solution

(b)The dry mass density is defined as the mass of solids per unit total volume

As the soil may shrink during drying, the mass density may not be equal to bulk mass density of soil in dry condition.

Express the denseness of soil .high value of density indicate the soil is in compacted state.

Soil Mechanics I & Soil Mechanics II

3.The shrinkage index is equal to

(a) Liquid limit minus plastic limit.

(b) Liquid limit minus shrinkage limit.

(c) Plastic limit minus shrinkage limit.

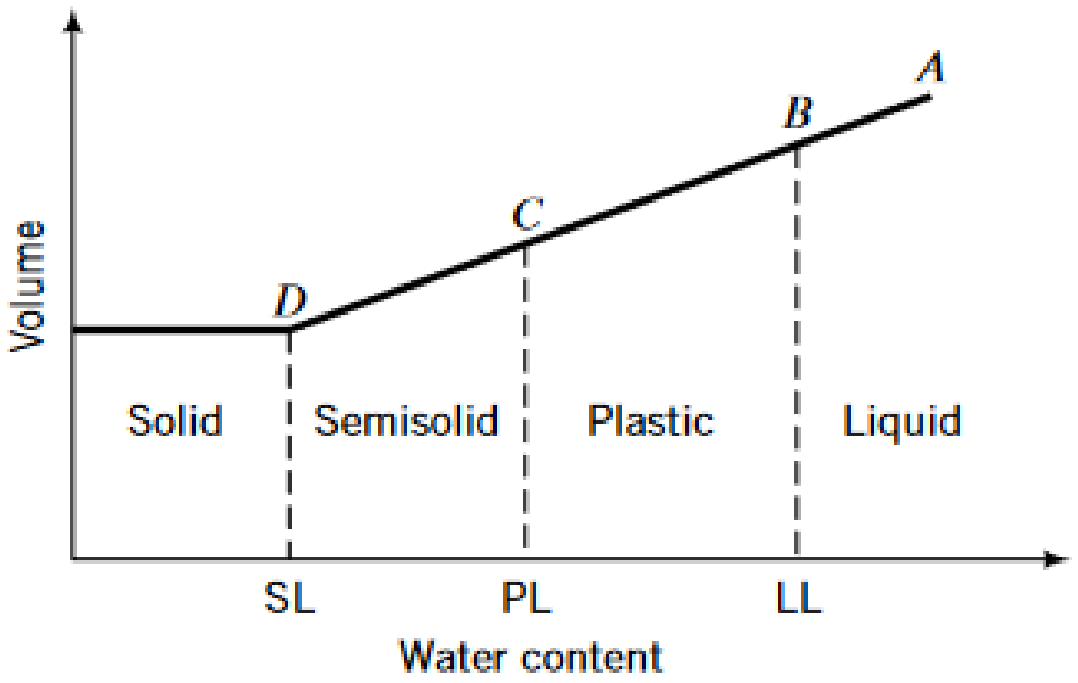
(d) None of above.

Solution

(a) The range of water contents over which the soil deforms plastically is known as the plasticity index, PI:

$$PI = LL - PL$$

$$LI = \frac{w - PL}{PI}$$



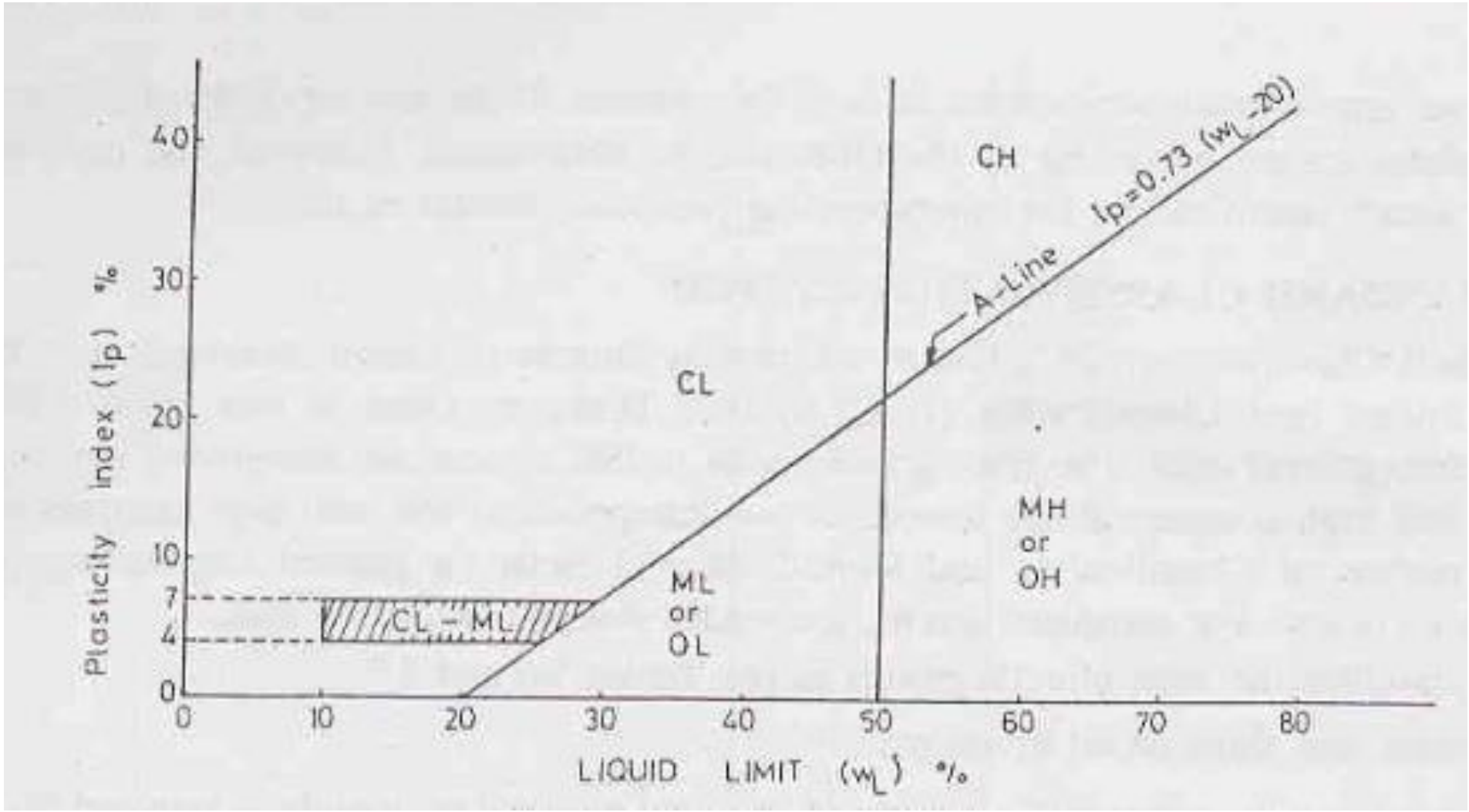
Soil Mechanics I & Soil Mechanics II

4.The soils which plot above the A line in the plasticity chart are

(a) Clays (b) silts

(c) Sands (d) organic Soils

Solution


Plasticity Index (Ip) %
40
30
20
10
7
4
0
0
10
20
30
40
50
60
70
80
LIQUID LIMIT (wL) %
CH
A-Line Ip=0.73 (wL-20)
CL
ML
or
OL
CL - ML
MH
or
OH

Soil Mechanics I & Soil Mechanics II

5. A constant-head permeameter is used for

(a) Coarse-grained soils (b) Silty soils

(c) Clayey soils (d) Organic soils

Solution

Constant-Head Test:-The constant-head test is used to determine the hydraulic conductivity of coarse-grained soils.

Falling-Head Test:-The falling-head test is used for fine-grained soils because the flow of water through these soils is too slow to get reasonable measurements from the constant-head test.

Soil Mechanics I & Soil Mechanics II

6.The shear strength of cohesion less soil is

(a) Proportional to the angle of shearing resistance.

(b) Inversely proportional to the angle of shearing resistance.

(c) Proportional to the tangent of the angle of shearing resistance

(d) None of the above

Solution

Shear strength of a soil is the maximum internal resistance to applied shearing forces.

$$\tau_f = c_{cm} + (\sigma'_n)_f \tan(\xi_o)$$

For cemented soils, Coulomb's frictional law is $\tau_f = c_{cm} + (\sigma'_n)_f \tan(\xi_o)$ where c_{cm} is called the cementation strength and ξ_o is the apparent friction angle.

Soil Mechanics I & Soil Mechanics II

7.The effective stress controls the following properties of soils;

(a) Shear strength

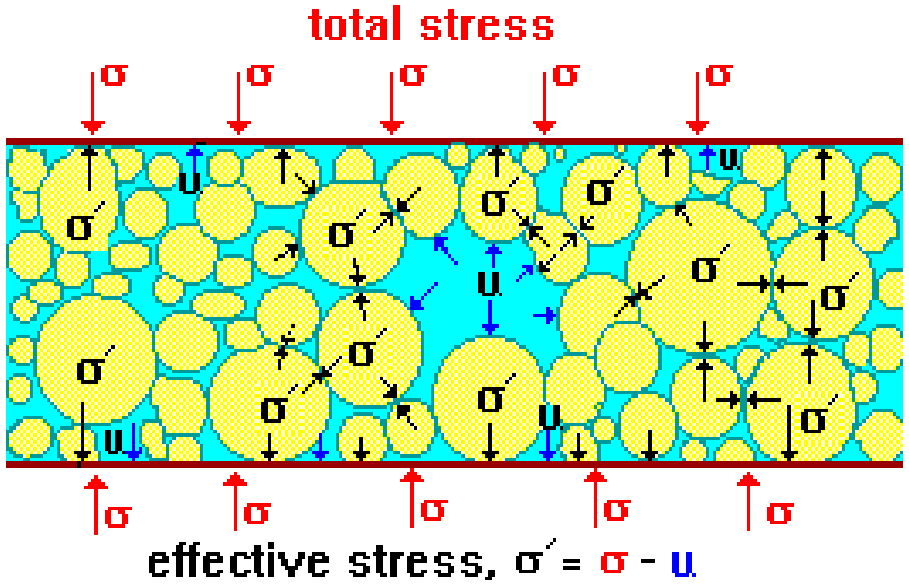
(b) Compressibility

(c) Permeability

(d) All of above

Solution

- The resistance or reaction to σ is provided by a combination of the stresses from the solids, called **effective stress (σ′)**, and from water in the pores, called **pore water pressure (u)**.



Soil Mechanics I & Soil Mechanics II

8.The range of optimum water contents for the standard proctor test for clayey soils is

(a) 6 to 10 %

(b) 8 to 12 %

(b) 12 to 15 %

(d) 14 to 20%

Soil Mechanics I & Soil Mechanics II

9. The line of optimums generally corresponds to percentage air voids of about

(a) Zero percent

(b) 5 percent

(c) 10 percent

(d) 20 percent

Soil Mechanics I & Soil Mechanics II

10.Pneumatic-tyred rollers are useful for compacting

(a) Cohesive soils

(b) Cohesion less soils

(c) Both (a) and (b)

(d) For soils in confined space

Soil Mechanics I & Soil Mechanics II

11.When drainage is permitted throughout the triaxial test ,the test is known as

(a) Quick Test

(b) Drained Test

(c) Consolidated Undrained Test

(d) None of (a),(b) and (C)

Solution

1. UU (unconsolidated undrained) test:

- In this, cell pressure is applied without allowing drainage. Then keeping cell pressure constant, deviator stress is increased to failure without drainage.

2. CU (consolidated undrained) test

In this, drainage is allowed during cell pressure application

- Then without allowing further drainage, deviator stress is increased keeping cell pressure constant.

3. CD (consolidated drained) test

This is similar to CU test except that as deviator stress is increased, drainage is permitted.

Soil Mechanics I & Soil Mechanics II

12. The shear strength of plastic undrained clay depend up on

A. Internal Friction

B. Cohesion

C. Both (a) and (b)

D. Neither (a) and (b)

Soil Mechanics I & Soil Mechanics II

13. Soil compacted dry of optimum as compared to wet of the optimum

(a) Has less permeability

(b) swells less

(c) Shrink less

(d) Has less resistance to compression

Soil Mechanics I & Soil Mechanics II

14.Standard Penetration Test is useful to measure

(a) Shear strength of soft clay

(b) shear strength of sands

(c) consistency of clay

(d) None of the above

Soil Mechanics I & Soil Mechanics II

15. In-situ vane shear Test is used to measure the shear strength of

(a) Very soft and sensitive clay

(b) stiff and fissured clay

(c) Sandy soil

(d) All of the above

Soil Mechanics I & Soil Mechanics II

16. In wet soil mass air occupies one-sixth of its volume and water occupies one-third of its volume. The void ratio of the soil is

A. 0.25

B. 0.50

C. 1.50

D. 1. 00.

Soil Mechanics I & Soil Mechanics II

17.A soil sample has a specific gravity of 2,.60 and a void ratio of 0.78. The water content required to fully saturate the soil at that void ratio will be

A. 20 %
B. 30%
C. 40%
D. 60%.

Soil Mechanics I & Soil Mechanics II

18. select the correct statement

A. Relative compaction is the same as relative density

B. Vibrofloatation is effective in the case of highly cohesive soil

C. Zero air void line and 100% saturation line are identical

Soil Mechanics I & Soil Mechanics II

16. In the stability analysis the term mobilized shear strength refers

(a) Shear strength

(b) Maximum shear stress

(c) Applied shear stress

(d) Non of the above

Soil Mechanics I & Soil Mechanics II

17.Bishop's Simplified Method of Slices Satisfies

(a) Only moment Equilibrium

(b) only vertical force Equilibrium

(c) Only horizontal Force Equilibrium

(d) all of the statics equation, except the horizontal force equilibrium

Soil Mechanics I & Soil Mechanics II

18. Identify the incorrect statement

The stability of slope is decreased by

(a) Removal of part of slope by excavation

(b) Shock caused by earthquake

(c) pore water pressure in soil

(d) Providing berm at the toe

Soil Mechanics I & Soil Mechanics II

19. The Method of slice for the stability of slop

(a) can be used for stratified soil

(b) can be used when seepage occurs and the pore pressure exists within the soil

(c) gives factor of safety based on the moments and not the force

(d) All of the above

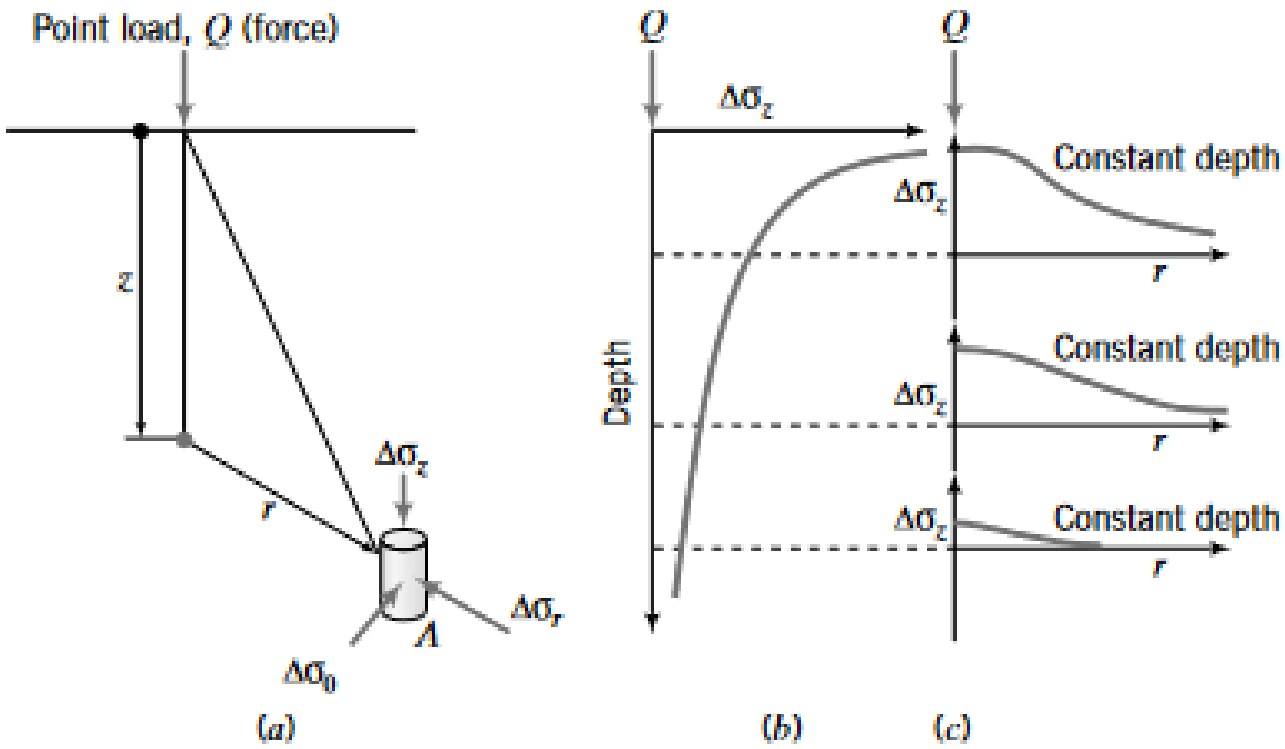
Soil Mechanics I & Soil Mechanics II

20.The stress developed at a point in the soil exactly below a point load at the surface is

A) Proportional to the depth of point
B) Proportional to the square of depth of point.
C) Inversely proportional to the depth of point.
D) Inversely proportional to the square of the depth of point.

Solution

$$\Delta\sigma_z = \frac{3Q}{2\pi z^2\left[1 + \left(\frac{r}{z}\right)^2\right]^{5/2}}$$



Solution

THE *ESSENTIAL POINTS* ARE:

1. **The increases in stresses below a surface load are found by assuming the soil is an elastic, semi-infinite mass.**
2. **Various equations are available for the increases in stresses from surface loading.**
3. **The stress increase at any depth depends on the shape and distribution of the surface load.**
4. **A stress applied at the surface of a soil mass by a loaded area decreases with depth and lateral distance away from the center of the loaded area.**
5. **The vertical stress increases are generally less than 10% of the surface stress when the depth-to-width ratio is greater than 2.**

Soil Mechanics I & Soil Mechanics II

21.The permeability of soil varies

A. Inversely as square of grain size
B. As square of grain size
C. As grain size
D. Inversely as void ratio

$$\boxed{q_j = v_j A = A k_j i} \tag{6.9}$$

The unit of measurement for q_j is m^3/s or cm^3/s. The conservation of flow (law of continuity) stipulates that the volume rate of inflow $(q_j)_{in}$ into a soil element must equal the volume rate of outflow, $(q_j)_{out}$, or, simply, inflow must equal outflow: $(q_j)_{in} = (q_j)_{out}$.

The hydraulic conductivity depends on

1. *Soil type:* Coarse-grained soils have higher hydraulic conductivities than fine-grained soils. The water in the double layer in fine-grained soils significantly reduces the seepage pore space.
2. *Particle size:* Hydraulic conductivity depends on D_{50}^2 (or D_{10}^2) for coarse-grained soils.
3. *Pore fluid properties, particularly viscosity:* $k_1 : k_2 \approx \mu_2 : \mu_1$, where μ is dynamic viscosity (dynamic viscosity of water is 1.12×10^{-3} N.s/m^2 at 15.6°C) and the subscripts 1 and 2 denote two types of pore fluids in a given soil.
4. *Void ratio:* $k_1 : k_2 \approx e_1^2 : e_2^2$, where subscripts 1 and 2 denote two types of soil fabric for coarse-grained soils. This ratio is useful in comparing the hydraulic conductivities of similar soils with different void ratios. However, two soils with the same void ratio can have different hydraulic conductivities.
5. *Pore size:* The greater the interconnected pore space, the higher the hydraulic conductivity. Large pores do not indicate high porosity. The flow of water through soils is related to the square of the pore size, and not the total pore volume.
6. *Homogeneity, layering, and fissuring:* Water tends to seep quickly through loose layers, through fissures, and along the interface of layered soils. Catastrophic failures can occur from such seepage.
7. *Entrapped gases:* Entrapped gases tend to reduce the hydraulic conductivity. It is often very difficult to get gas-free soils. Even soils that are under groundwater level and are assumed to be saturated

Soil Mechanics I & Soil Mechanics II

22.If the entire semi infinite soil mass loaded with load intensity of q at the surface ,the vertical stress at any depth is equal to

A. q
B. 0.5q
C. Zero
D. Infinity

Soil Mechanics I & Soil Mechanics II

23.The coefficient Of compressibility is

A. Change in void ratio to change in effective stress
B. Volumetric strain to change in effective stress.
C. Change in thickness to change in effective stress
D. Stress to strain.

Solution

Consolidation is the time-dependent settlement of soils resulting from the expulsion of water from the soil pores.

Primary consolidation is the change in volume of a fine-grained soil caused by the expulsion of water from the voids and the transfer of stress from the excess porewater pressure to the soil particles.

Secondary compression is the change in volume of a fine-grained soil caused by the adjustment of the soil fabric (internal structure) after primary consolidation has been completed.

Excess porewater pressure, $\Delta \mathbf{u}$, is the porewater pressure in excess of the current equilibrium porewater pressure. For example, if the porewater pressure in a soil is u_o and a load is applied to the soil so that the existing porewater pressure increases to u_1, then the excess porewater pressure is $\Delta u = u_1 - u_o$.

Drainage path, $\mathbf{H_{dr}}$, is the longest vertical path that a water particle will take to reach the drainage surface.

Past maximum vertical effective stress, $\boldsymbol{\sigma'_{zc}}$, is the maximum vertical effective stress that a soil was subjected to in the past.

Normally consolidated soil is one that has never experienced vertical effective stresses greater than its current vertical effective stress ($\sigma'_{zo} = \sigma'_{zc}$).

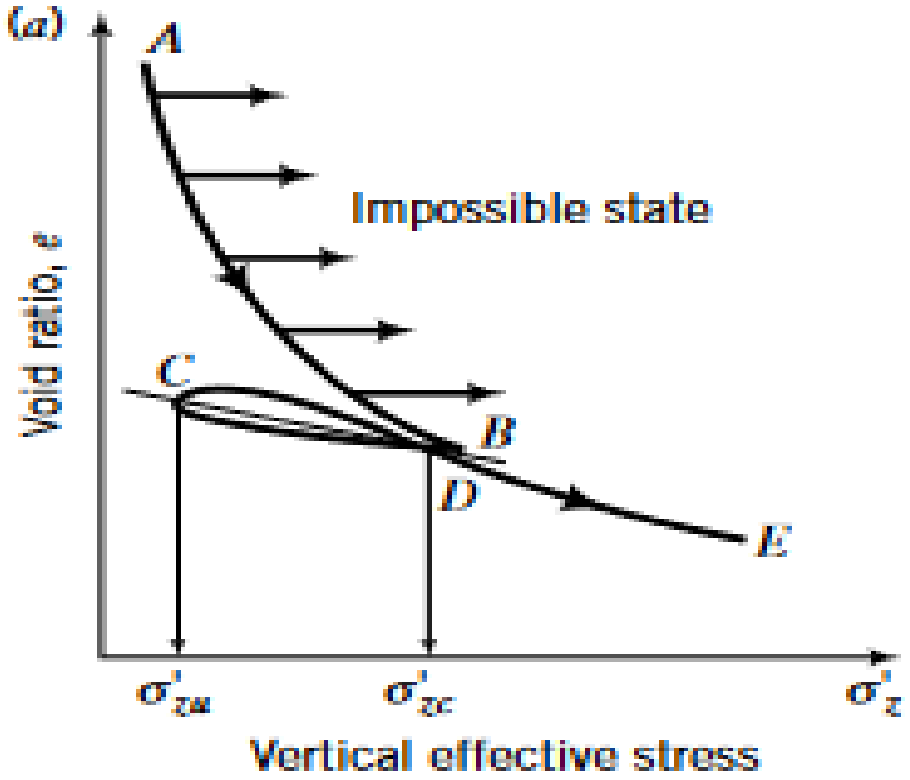
Overconsolidated soil is one that has experienced vertical effective stresses greater than its existing vertical effective stress ($\sigma'_{zo} < \sigma'_{zc}$).

Overconsolidation ratio, OCR, is the ratio by which the current vertical effective stress in the soil was exceeded in the past ($\text{OCR} = \sigma'_{zc}/\sigma'_{zo}$).

Compression index, $\mathbf{C_c}$, is the slope of the normal consolidation line in a plot of the logarithm of vertical effective stress versus void ratio.

Unloading/reloading index or ***recompression index,*** C_r, is the average slope of the unloading/reloading curves in a plot of the logarithm of vertical effective stress versus void ratio.

Modulus of volume compressibility, m_v, is the slope of the curve between two stress points in a plot of vertical effective stress versus vertical strain.



12.8. BASIC DEFINITIONS

The following basic definitions related to consolidation are of paramount importance.

(1) Coefficient of Compressibility. The coefficient of compressibility (a_v) is defined as decrease in void ratio per unit increase in effective stress. It is equal to the slope of the $e - \bar{\sigma}$ curve at the point under consideration [Fig. 12.7 (*a*)].

Thus
$$a_v = \frac{-de}{d\bar{\sigma}} = \frac{-\Delta e}{\Delta \bar{\sigma}} \qquad \ldots(12.12)$$

As the effective stress increases, the void ratio decreases, and therefore, the ratio $de/d\bar{\sigma}$ is negative. The minus sign makes a_v positive. For convenience, the coefficient of compressibility a_v is reported as positive.

As the value of a_v is different at various effective stresses, while reporting its value, the effective stress to which that value corresponds must be mentioned. The coefficient of compressibility decreases with an increase in the effective stress. In other words, the soil becomes stiffer (less compressible) as the effective stress is increased and the curve becomes flatter.

The coefficient of compressibility (a_v) has the dimensions of $[L^2/F]$. The units are m^2/kN. It may be noted that the units are inverse of that for pressure.

(2) Coefficient of Volume change. The coefficient of volume change (or volume compressibility) is defined as the volumetric strain per unit increase in effective stress. Thus

$$m_v = \frac{-\Delta V/V_0}{\Delta \bar{\sigma}} \qquad \ldots(12.13)$$

where m_v = coefficient of volume change, V_0 = initial volume,
ΔV = change in volume, and $\Delta \bar{\sigma}$ = change in effective stress.

The reader should note that the coefficient of volume change is inverse of the bulk modulus used in solid mechanics and fluid mechanics. For most clays, $m_v = 1 \times 10^{-3}$ to $1 \times 10^{-4} m^2/kN$.

The volumetric strain ($\Delta V/V_0$) can be expressed in terms of either void ratio or the thickness of the specimen as explained under:

(*a*) Let e_0 be the initial void ratio. Let the volume of solids be unity. Therefore, the initial volume V_0 is equal to $(1 + e_0)$. If Δe is the change in void ratio due to change in volume ΔV, we have $\Delta V = \Delta e$. Thus

$$\frac{\Delta V}{V_0} = \frac{\Delta e}{1 + e_0}$$

(3) Compression Index. The compression index (C_c) is equal to the slope of the linear portion of the void ratio versus log $\bar{\sigma}$ plot [Fig. 12.7 (*b*)]. Thus

$$C_c = \frac{-\Delta e}{\log_{10}(\bar{\sigma}/\bar{\sigma}_0)} \qquad \ldots(12.17)$$

where $\bar{\sigma}_0$ = initial effective stress, $\bar{\sigma}$ = final effective stress, Δe = change in void ratio

Sometimes, Eq. 12.17 is expressed as

$$C_c = \frac{-\Delta e}{\log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)} \qquad \ldots(12.18)$$

where $\Delta\bar{\sigma}$ is the change in effective stress.

The numerical value of C_c can be easily determined from the difference in void ratio corresponding to one log cycle. Thus

$$C_c = \frac{-\Delta e}{1} = -\Delta e \qquad \ldots(12.19)$$

The compression index is extremely useful for determination of the settlement in the field.

The compression index of a clay is related to its index properties, especially the liquid limit. Terzaghi and Peck gave the following empirical relationship for clays of low to medium sensitivity ($(S_t \leq 4)$.

(*a*) For undisturbed soils, $C_c = 0.009\,(w_L - 10)$

(*b*) For remoulded soils, $C_c = 0.007\,(w_L - 10)$

where w_L = liquit limit (%).

The value of C_c normally varies between 0.30 for highly plastic clays and 0.075 for low plastic clays.

The compression index is also related to the insitu void ratio e_0 or water content (w_0) as under

$$C_c = 0.54\,(e_0 - 0.35) \qquad \ldots(12.21)$$

$$C_c = 0.0054\,(2.6w_0 - 35) \qquad \ldots(12.22)$$

The coefficient of compressibility a_v may be calculated from the compression index as under :

$$a_v = 0.435\,\frac{C_c}{\bar{\sigma}_a}$$

(5) Recompression Index. Recompression is the compression of a soil which had already been loaded and unloaded. The load during recompression is less than the load to which the soil has been subjected previously. The slope of the recompression curve obtained during reloading (CFD in Fig. 12.8) when plotted as e-log $\bar{\sigma}$, is equal to the recompression index (C_r). Thus

$$C_r = \frac{-\Delta e}{\log\left(\frac{\bar{\sigma} + \Delta\bar{\sigma}}{\bar{\sigma}}\right)} \qquad \ldots(12.24)$$

The recompression index is appreciably smaller than the compression index C_c. It is usually in the range of 1/10 to 1/5 of the compression index.

(6) Normally consolidated and Over-consolidated clays. A normally consolidated soil is one which had not been subjected to a pressure greater than the present existing pressure. A soil is said to be over-consolidated if it had been subjected in the past to a pressure in excess of the present pressure.

The portion *AB* of the curve in Fig. 12.8 represents the soil in normally consolidated condition. The curve in this range is also called the *virgin compression curve*. The soil in the range *CD* when it is recompressed represents overconsolidated condition, as the soil had been previously subjected to a pressure $\bar{\sigma}_2$, which is greater than the pressure in the range *CD*.

The maximum pressure to which an over-consolidated soil had been subjected in the past divided by the present pressure is known as the overconsolidation ratio (O.C.R.). For example, the soil indicated by the condition at point *C* has an over-consolidation ratio of $\bar{\sigma}_2/\bar{\sigma}_1$.

It may be emphasized that normally consolidated soils and overconsolidated soils are not different types of soils but these are conditions in which a soil exists. The same type of soil can behave as normally consolidated in a certain pressure range and an over-consolidated in some other pressure range. For example, in Fig. 12.8, the soil which behaves as overconsolidated in the range *CD* would again behave as normally consolidated in the range *DG*.

The liquidity index of a normally consolidated clay is generally between 0.6 and 1.00, whereas that for an over-consolidated clay between 0.0 and 0.60.

As the recompression index (C_r) is very small as compared with the compression index (C_c), the soils in the overconsolidated state have smaller compressibility. The settlements of the structures built an over-consolidated clays are small.

(7) Underconsolidated clays. If the clay deposit has not reached equilibrium under the applied overburden loads, it is said to be underconsolidated. This normally occurs in areas of recent land fill.

24.With an increase in the liquid limit. compression index

A. Decreases
B. Increases
C. remain constant
D. may increase or decrease

25. The recompression index is about ------- of the compression index

A. 5 times
B. ½
C. 1/5
D. 1/20

26.When consolidation of a saturated soil sample occurs, the degree of saturation

A. Increases
B. Decreases
C. remains constant
D. may increase or decrease

27.Shallow foundation is usually defined as a foundation which has

A. Depth less than 0.6 m
B. Depth less than its width
C. Depth less than 1.0 m
D. None of the above

Foundation is a structure that transmits loads to the underlying soils.

Footing is a foundation consisting of a small slab for transmitting the structural load to the underlying soil. Footings can be individual slabs supporting single columns (Figure 12.2a) or combined to support two or more columns (Figure 12.2b), or be a long strip of concrete slab (Figure 12.2c, d; width B to length L ratio is small, i.e., it approaches zero) supporting a load-bearing wall, or a mat (Figure 12.2e).

Embedment depth ($\mathbf{D_f}$) is the depth below the ground surface where the base of the foundation rests.

Shallow foundation is one in which the ratio of the embedment depth to the minimum plan dimension, which is usually the width (B), is $D_f/B \leq 2.5$.

Ultimate bearing capacity is the maximum pressure that the soil can support.

Ultimate net bearing capacity ($\mathbf{q_u}$) is the maximum pressure that the soil can support above its current overburden pressure.

Ultimate gross bearing capacity ($\mathbf{q_{ult}}$) is the sum of the ultimate net bearing capacity and the overburden pressure above the footing base.

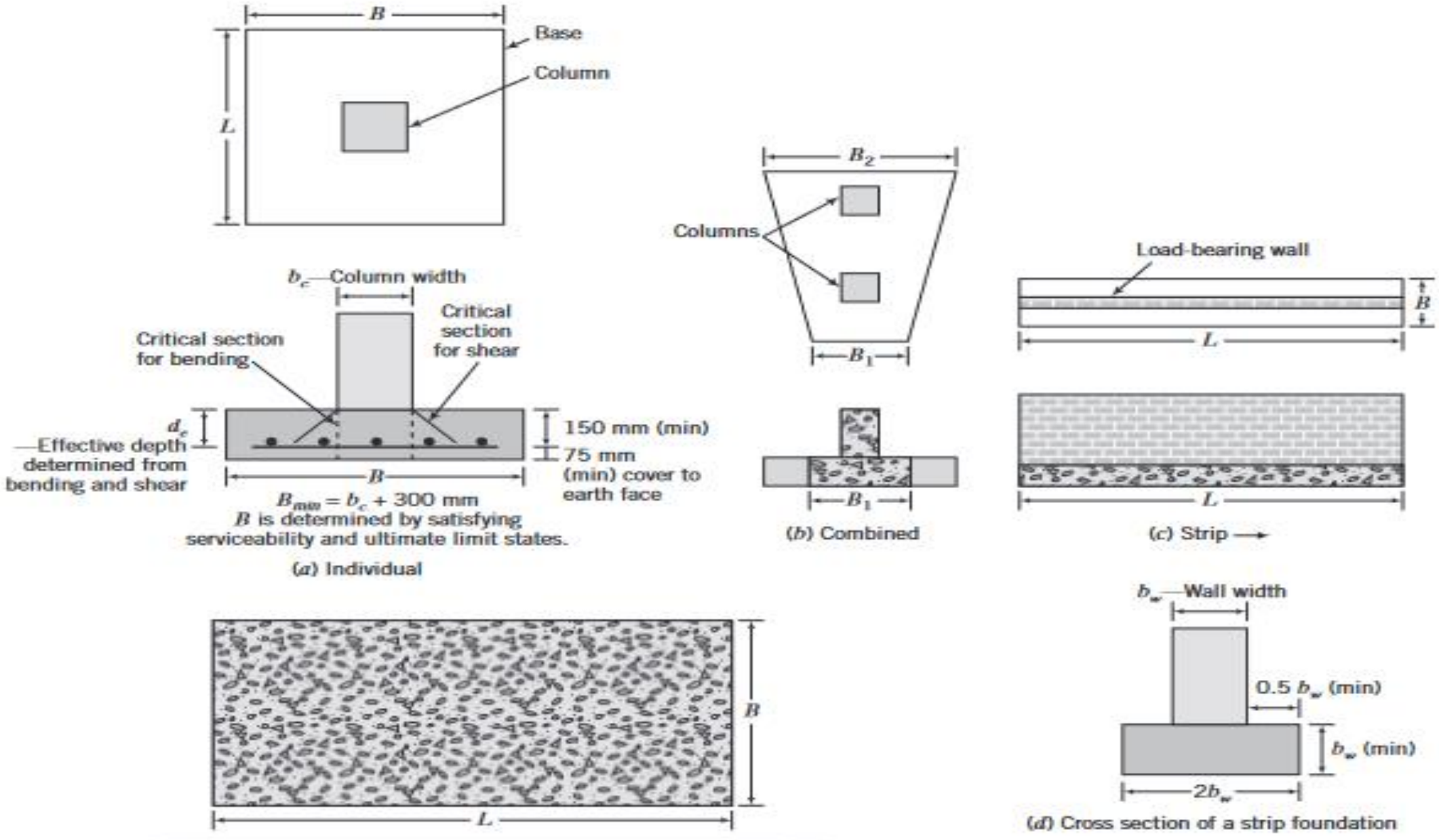


FIGURE 12.2 Type of shallow footing

Allowable bearing capacity or ***safe bearing capacity*** **(q_a)** is the working pressure that would ensure a margin of safety against collapse of the structure from shear failure. The allowable bearing capacity is usually a fraction of the ultimate net bearing capacity.

Factor of safety or ***safety factor*** **(*FS*)** is the ratio of the ultimate net bearing capacity to the allowable net bearing capacity or to the applied maximum net vertical stress. In geotechnical engineering, a factor of safety between 2 and 5 is used to calculate the allowable bearing capacity.

Ultimate limit state defines a limiting stress or force that should not be exceeded by any conceivable or anticipated loading during the design life of a foundation or any geotechnical system.

Serviceability limit state defines a limiting deformation or settlement of a foundation, which, if exceeded, will impair the function of the structure that it supports.

28.The bearing capacity of soil supporting a footing of size 3m x 3m will not be affected by the presence of water table located at a depth below the base of footing of

A. 1.0 m

B. 1.5 m

C. 3 m

D. 6 m

29.If the gross bearing capacity of a strip footing 1.5 m wide located at a depth of I m in clay is 400 KN/m^2, its net bearing capacity for 20 KN/m^2 is

A. 370 KN/m^2

B. 380 KN/m^2

C. 390 KN/m^2

D. 360 KN/m^2

30.The method of slices tor the stability of slope

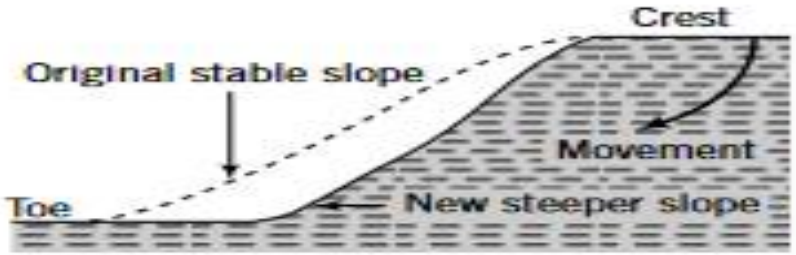
A. Can be used for stratified soils
B. Can be used when seepage occurs and the pore pressure exists within the soil.
C. Gives the factor of safety based on moments and not the forces.
D. All the above.

THE *ESSENTIAL POINTS* ARE:

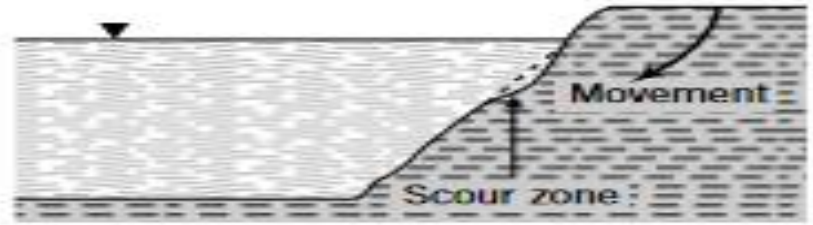
1. Geological features and environmental conditions (e.g., external loads and natural forces) are responsible for most slope failures.
2. The common modes of slope failure in soils are by translation, rotation, flow, and block movements.

THE *ESSENTIAL POINTS* ARE:

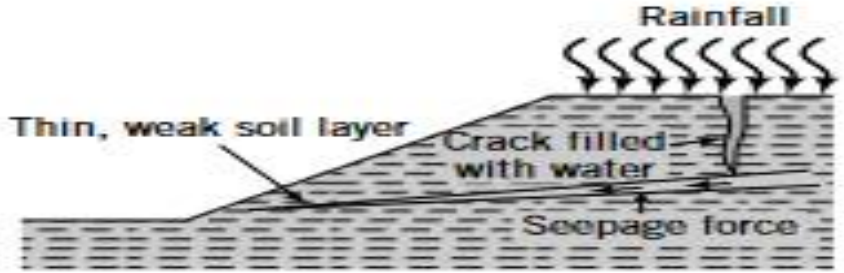
1. The maximum stable slope in a coarse-grained soil, in the absence of seepage, is equal to the friction angle.
2. The maximum stable slope in coarse-grained soils, in the presence of seepage parallel to the slope, is approximately one-half the friction angle.
3. The critical slope angle in fine-grained soils is 45° for an infinite slope failure mechanism.
4. The critical depth is the depth of tension cracks, that is, $2s_u/\gamma$.



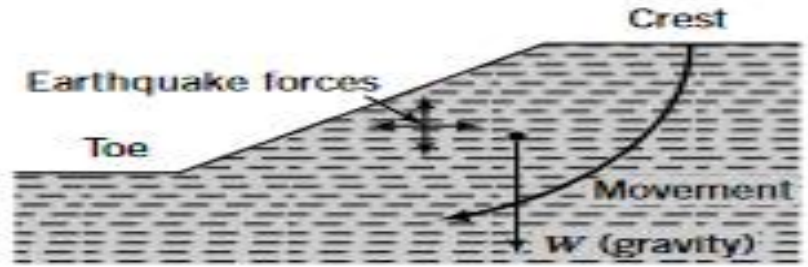
(*a*) Steepening of slope by erosion



(*b*) Scour by rivers and streams



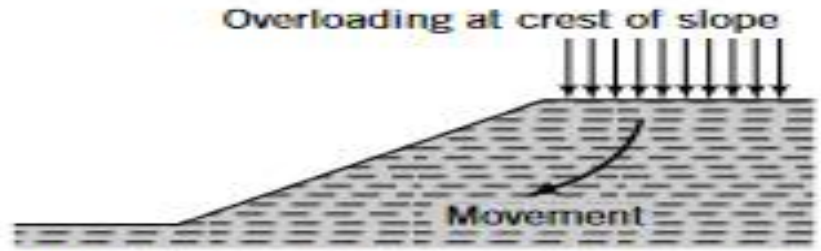
(*c*) Rainfall fills crack and introduces seepage forces in the thin, weak soil layer



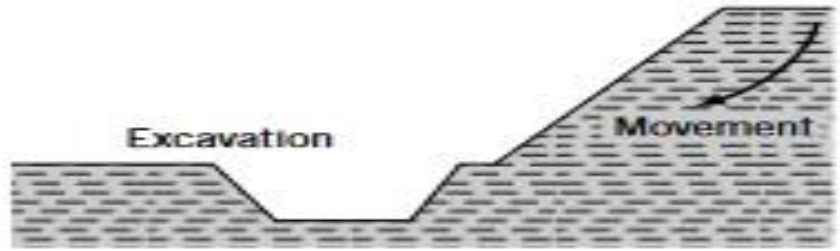
(*d*) Gravity and earthquake forces



(*e*) Geological feature—soil stratification



(*f*) Overloading at the crest of the slope



(*g*) Excavation at toe of the slope

31.In stability analysis. the term mobilized shear strength is referred to as

A. Shear strength
B. Maximum shear stress
C. Applied shear stress
D. None of the above.

32. Bishop's simplified method of slices satisfies

A. only the moments equilibrium
B. only the vertical forces equilibrium
C. only the horizontal forces equilibrium
D. All the statics equations, except the horizontal force equilibrium.

32.The factor of safety of an infinite slope in a sand deposit is 1.732 .If the angle of shearing resistance is 30. the safe slope is

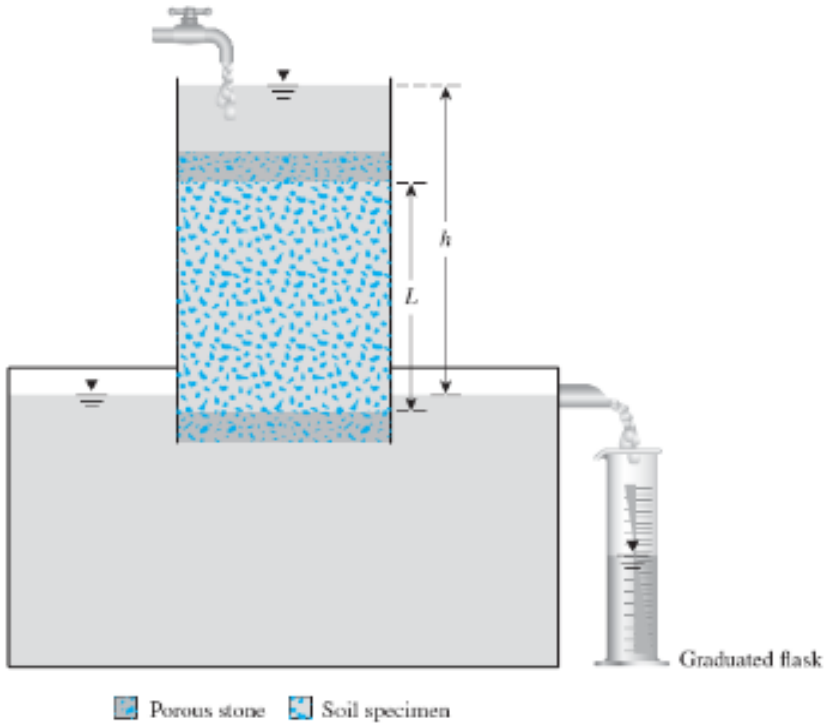
A. 19.45^0
B. 75.45^0
C. 18.4^0
D. 71.6^0

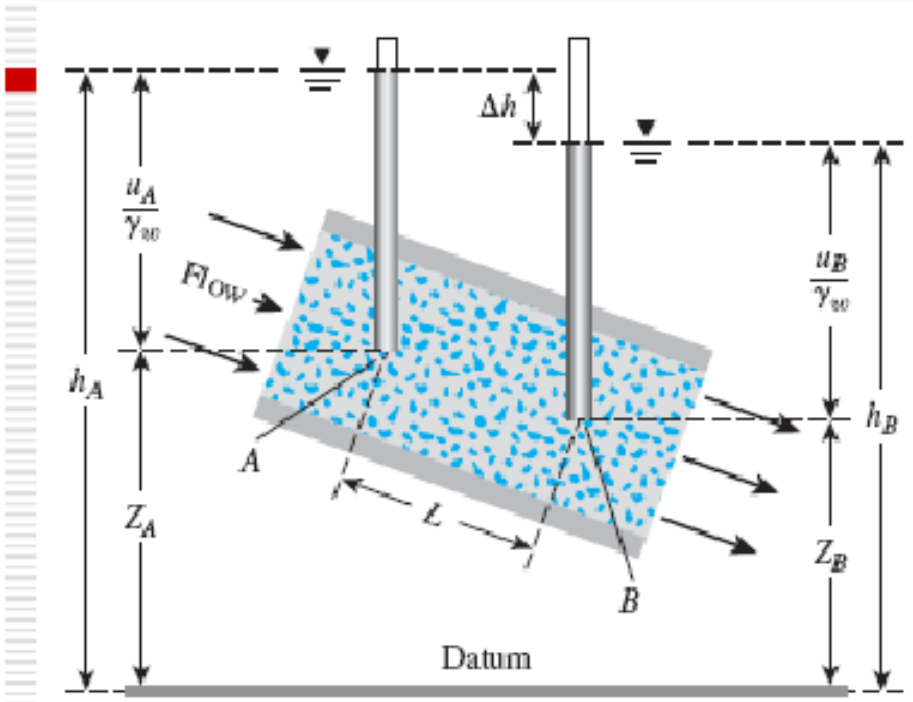
Seepage Analysis

- ✓Seepage is the flow of water under gravitational forces in a permeable medium.
- ✓Flow of water takes place from a of high head to a point of low head. The flow is genarally laminar.
- ✓The path taken by a water particle is represented by a flow line.
- ✓Although an infinite number of flow lines can be drawn, for convince, only a few are drawn.
- ✓At certain points on different flow lines, the total head will be the same.

- ✓The lines connecting to equal total head can be drawn. These lines are known as equipotential lines.
- ✓As flow always takes place along the steepest hydraulic gradient, the equipotential lines cross flow lines at right angles.
- ✓Flow lines and equipotential lines together form a flow net.
- ✓The flow lines gives a pictorial representation of the path taken by water particles and the head variation along that path.

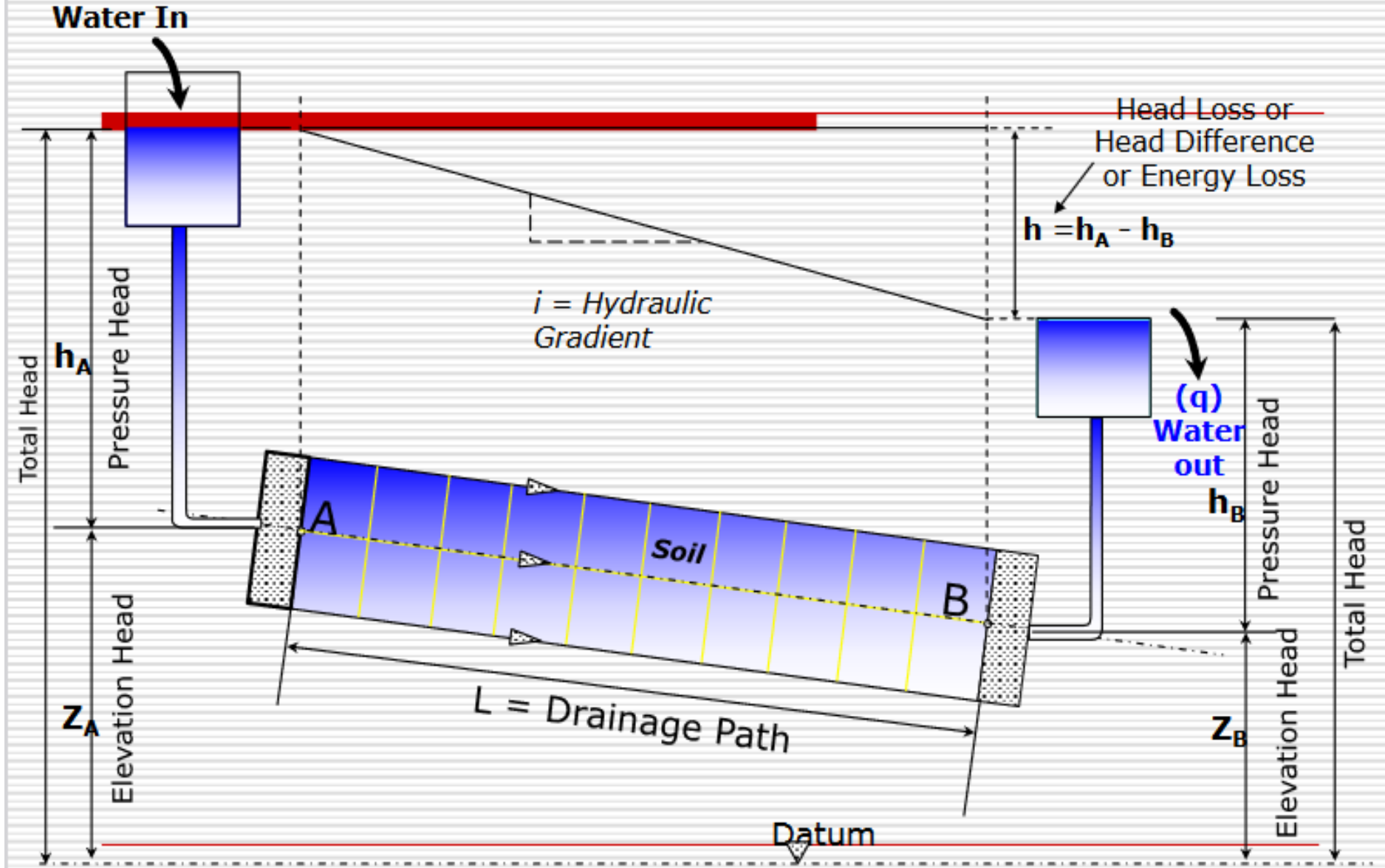
One-dimensional flow



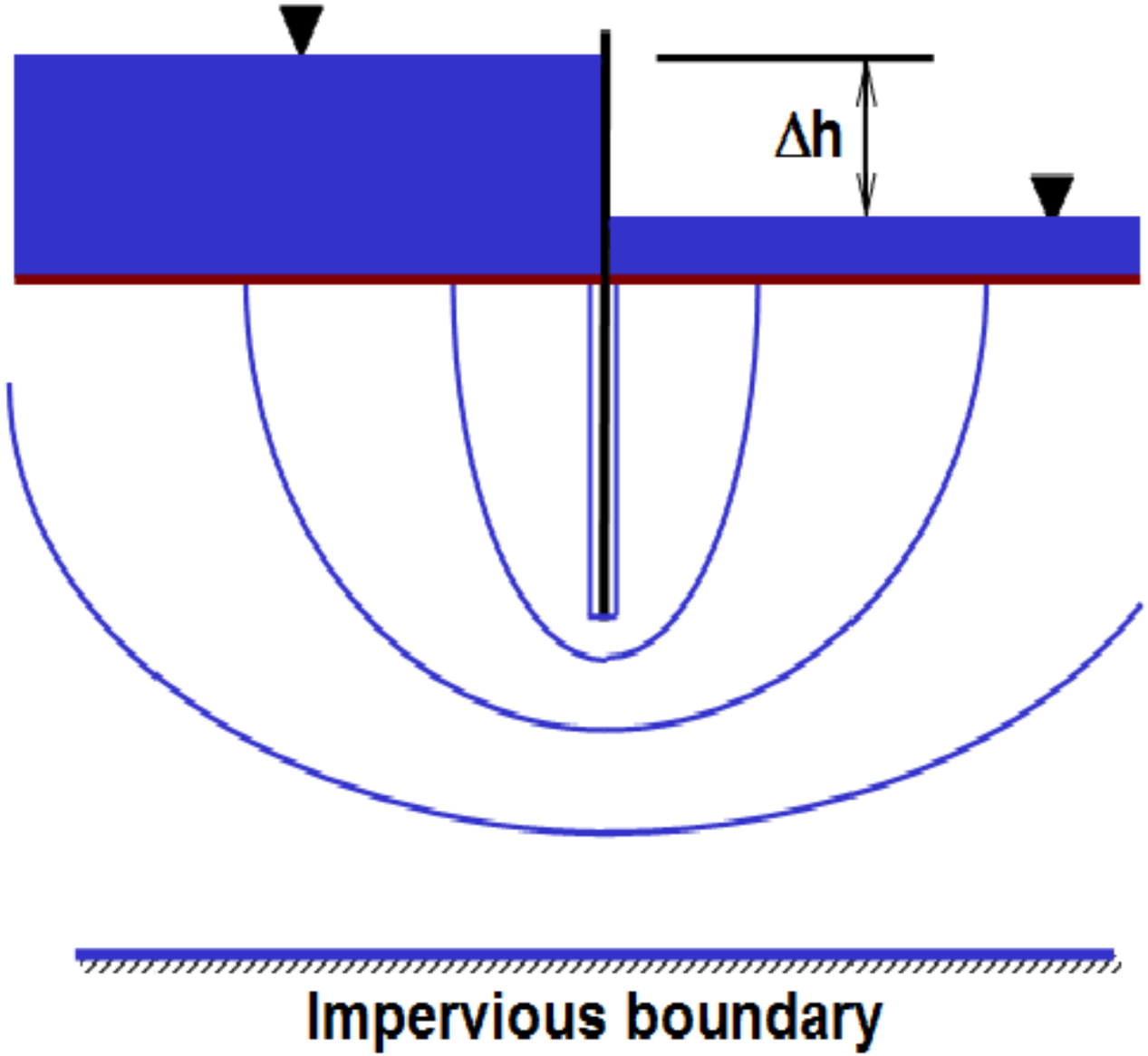


$$\text{Discharge} = Q = v.A = k.i.A = k(h/L).A$$

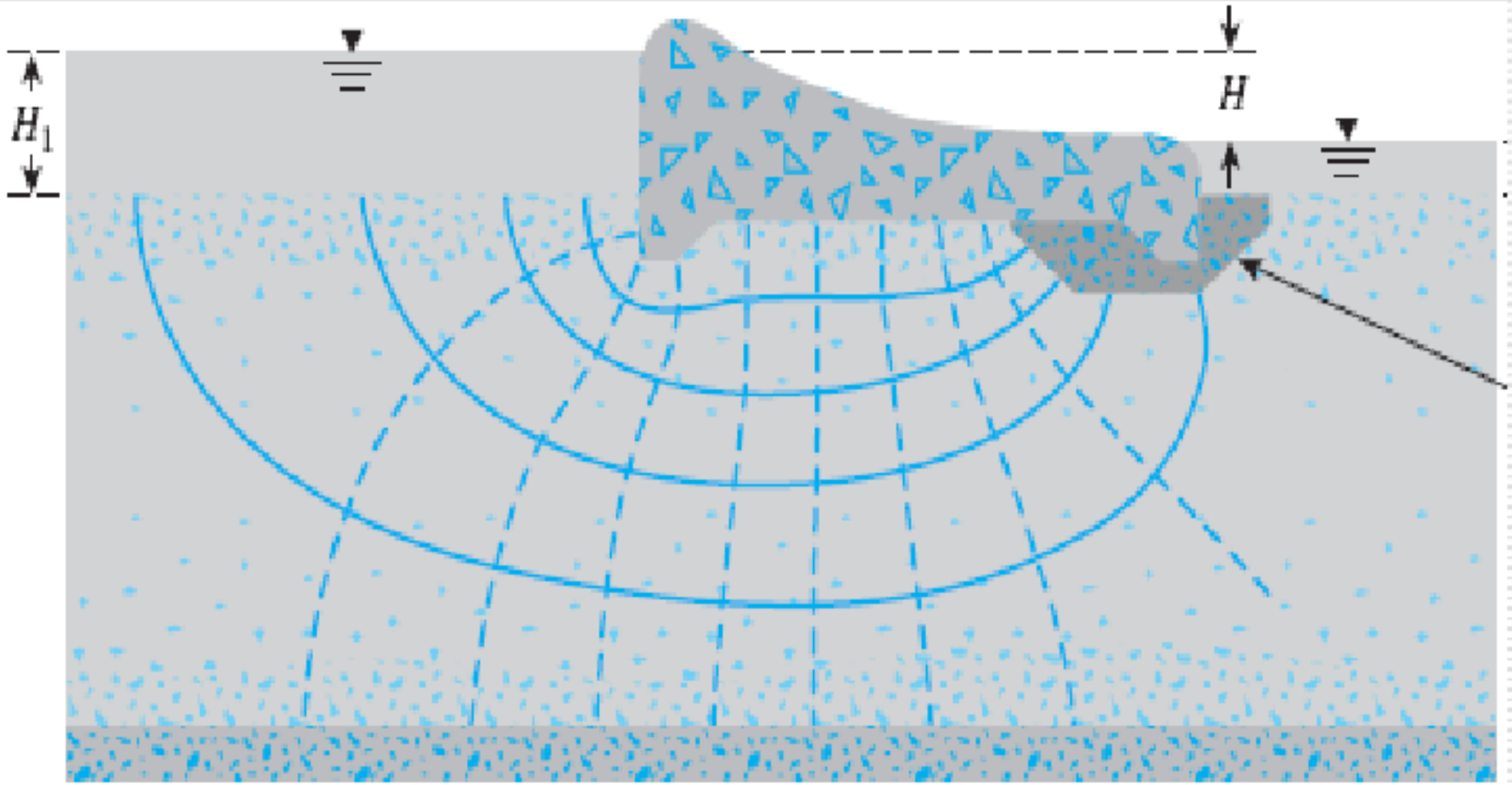
One-dimensional flow



Two-dimensional flow

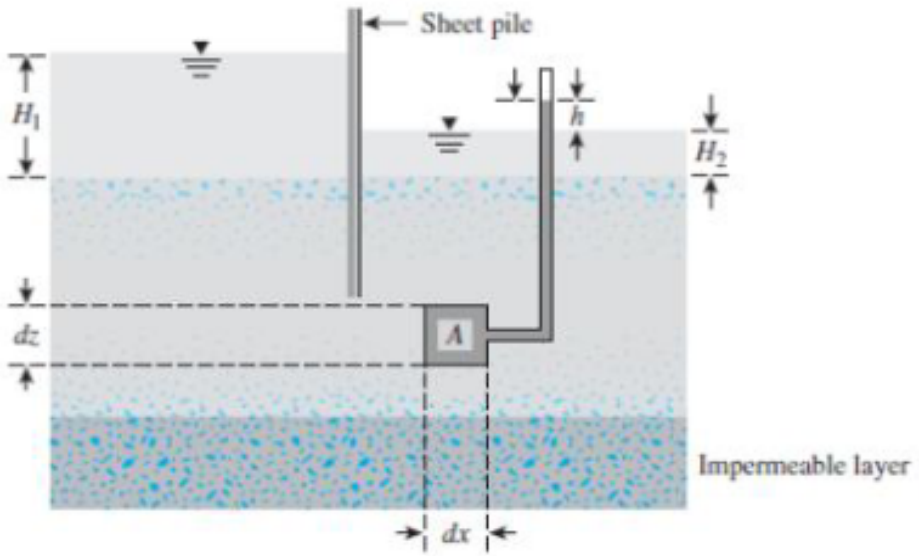


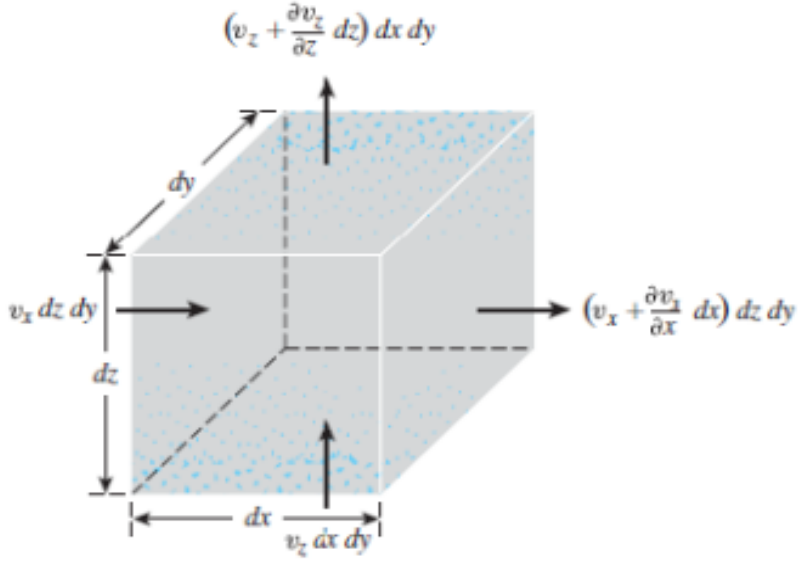
Two-dimensional flow



Laplace equation of Continuity

- In reality, the flow of water through soil is not in one direction only, nor is it uniform over the entire area perpendicular to the flow.
- The flow of water in two dimensional is described using Laplace equation.
- Laplace equation is the combination of the equation of continuity and Darcy's law.





Laplace equation of Continuity

Flow in: $v_x dydz$, $v_z dxdy$

Flow out: $\left(v_x + \frac{\partial v_x}{\partial x} dx\right) dydz$, $\left(v_z + \frac{\partial v_z}{\partial z} dz\right) dxdy$

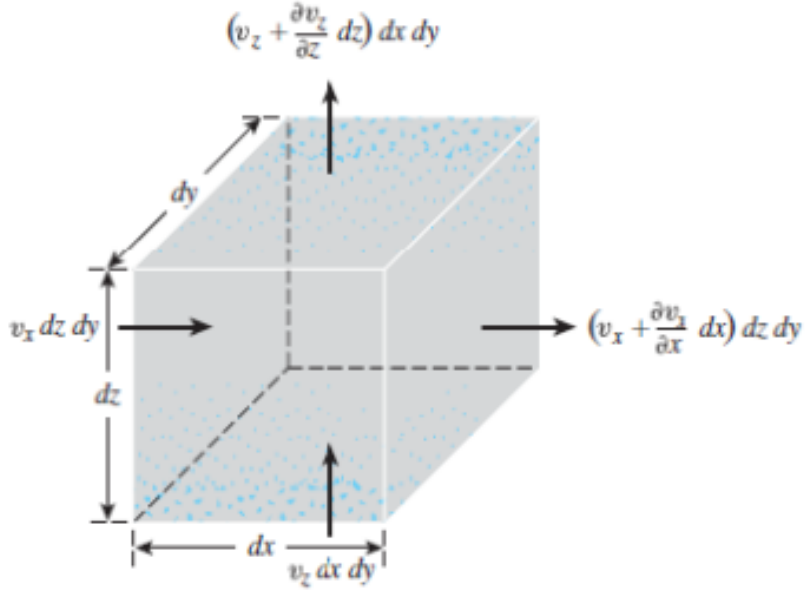
Flow out = Flow in (Continuity equation)

$$\left(v_x + \frac{\partial v_x}{\partial x} dx\right) dydz + \left(v_z + \frac{\partial v_z}{\partial z} dz\right) dxdy = v_x dydz + v_z dxdy$$

By simplification, we get

$$\frac{\partial v_x}{\partial x} dxdydz + \frac{\partial v_z}{\partial z} dxdydz = 0$$

$$\frac{\partial v_x}{\partial x} + \frac{\partial v_z}{\partial z} = 0$$



Laplace equation of Continuity

From Darcy's Law:

$$v_x = k_x \frac{dh}{dx} \quad \text{and} \quad v_z = k_z \frac{dh}{dz}$$

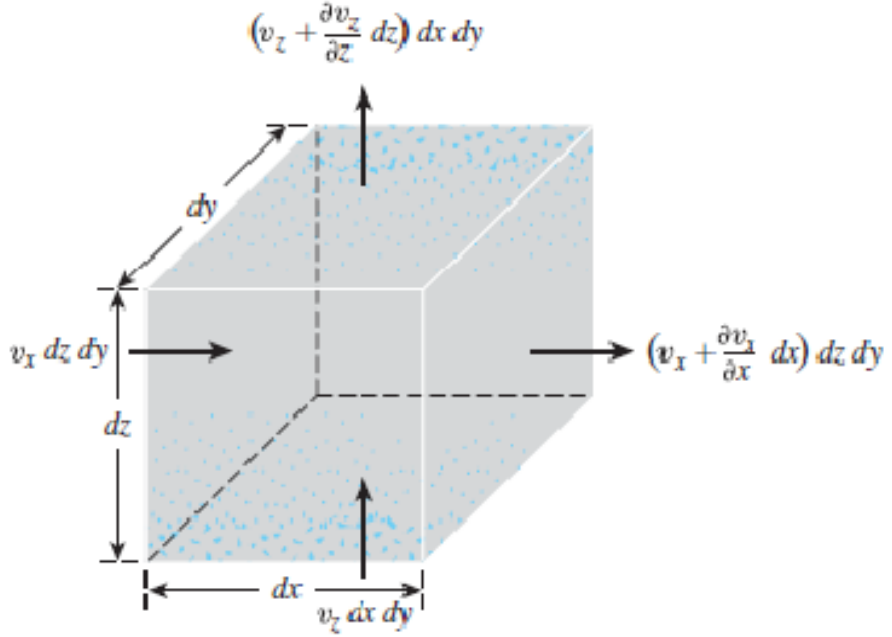
Replace in the continuity equation

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

If soil is isotropic (i.e. $k_x = k_z = k$)

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

Laplace equation

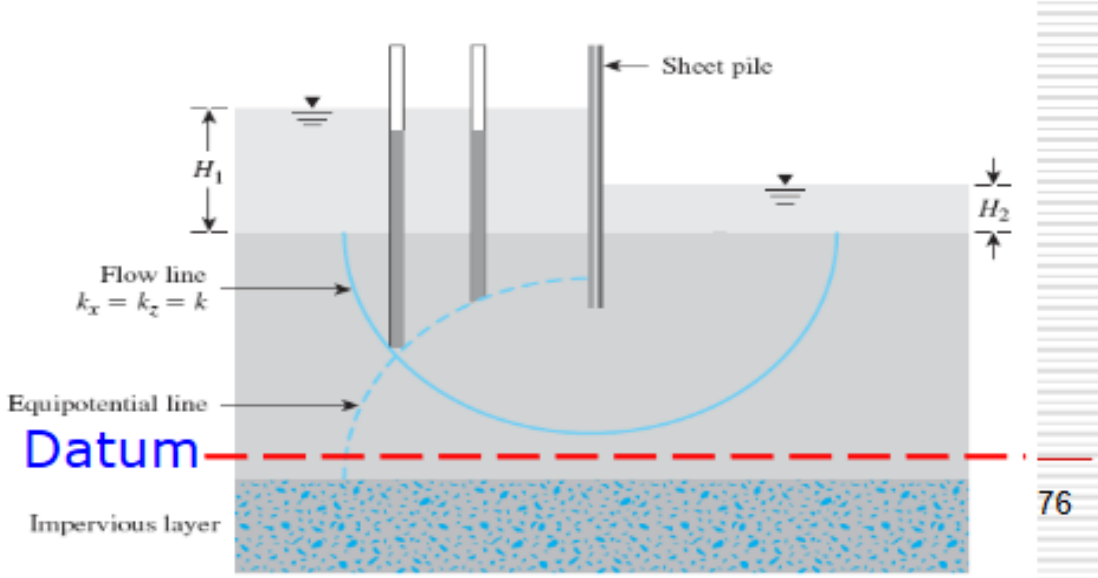


This equation governs the steady flow condition for a given point in the soil mass

Flow nets

- Flow nets are a graphical solution method of Laplace equation for 2D flow in a homogeneous, isotropic aquifer.
- In an isotropic medium, the continuity equation represents two orthogonal families of curves:

1. Flow lines: the line along which a water particle will travel from upstream to the downstream side in the permeable soil medium
2. Equipotential lines: the line along which the potential (pressure) head at all points is equal.



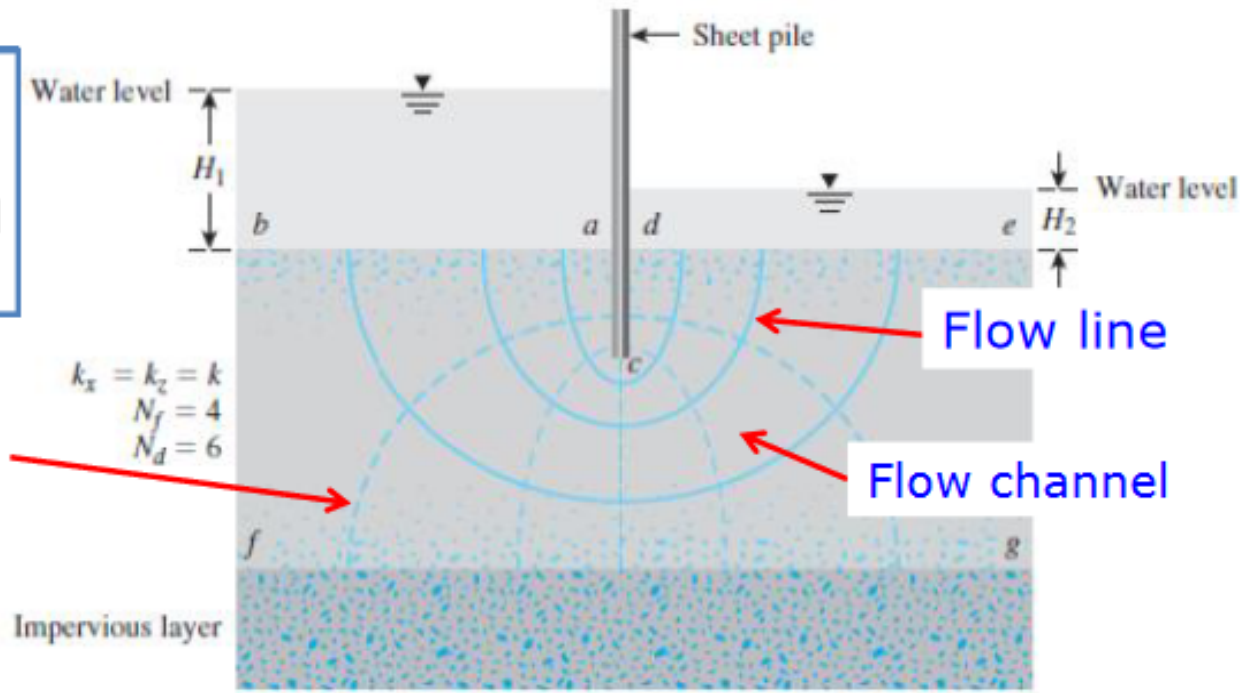
76

Flow nets

- Flow nets are the combination of flow lines and equipotential lines.
- To complete the graphic construction of a flow net, one must draw the flow and equipotential lines in such away that:
  1. The equipotential lines intersect the flow lines at right angles.
  2. The flow elements formed are approximate squares.

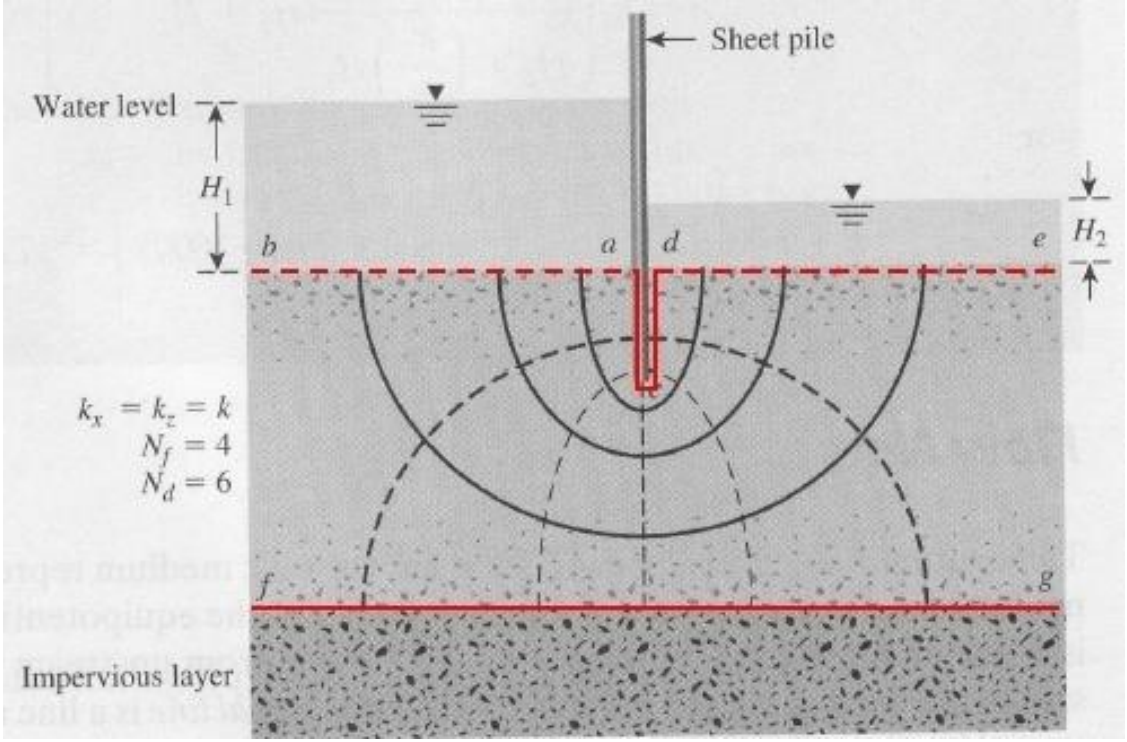
N_f: is the number of flow channels in the flow net.

N_d: is the number of potential drops.

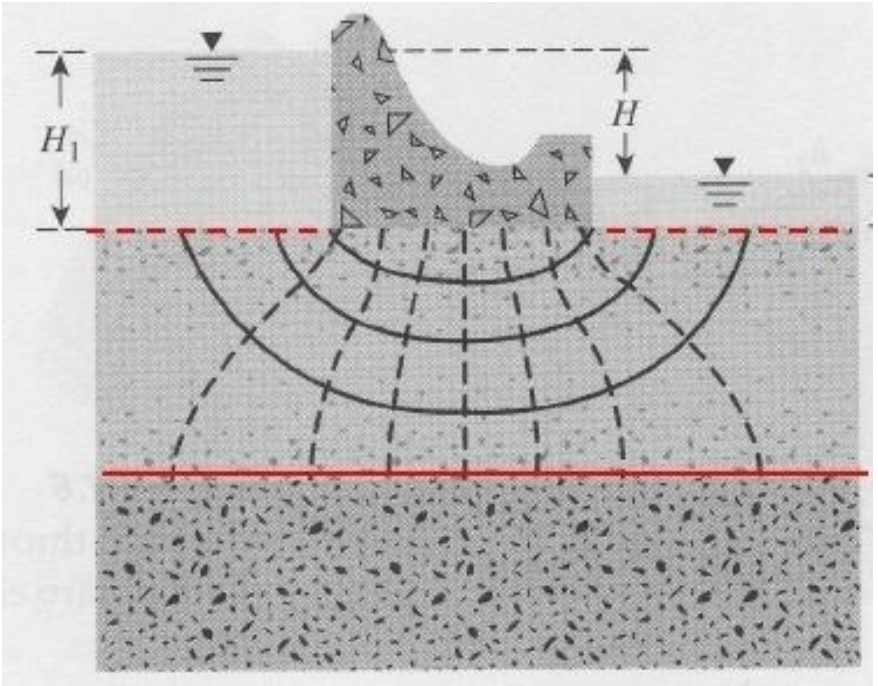


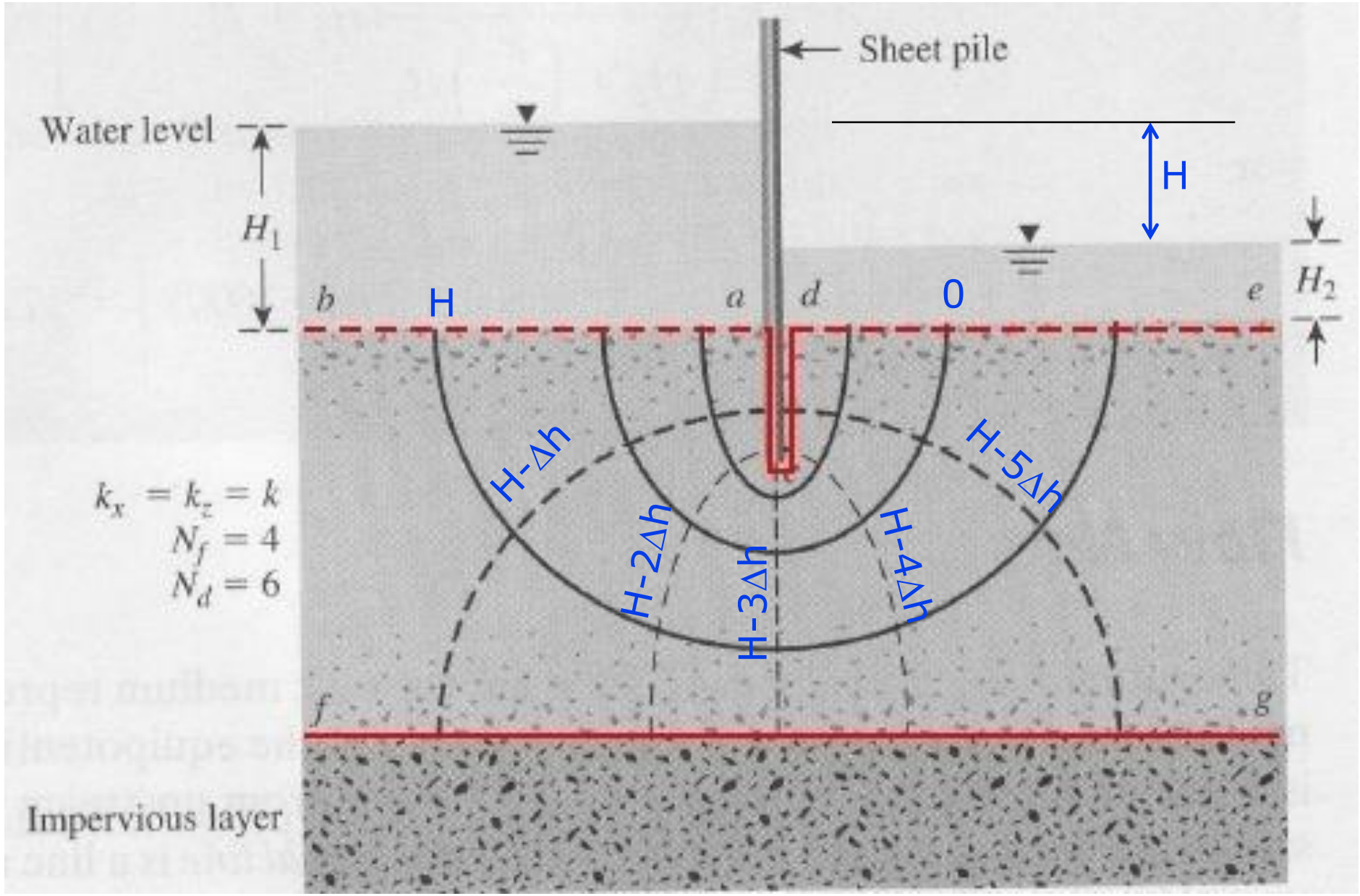
Flow Net Drawing Technique

1. Draw to a convenient scale the geometry of the problem.
2. Establish constant head and no flow boundary conditions and draw flow and equipotential lines near boundaries.
   - Constant head boundaries (water levels) represent initial or final equipotentials
   - Impermeable (no-flow) boundaries are flow lines
3. Sketch flow lines by smooth curves (3 to 5 flow lines).
   - Flow lines should not intersect each other or impervious boundary
4. Draw equipotential lines by smooth curves adhering to right angle intersections and square grids conditions (aspect ratio =1).
5. Continue sketching and re-adjusting until you get **squares almost everywhere**. Successive trials will result in a reasonably consistent flow net.

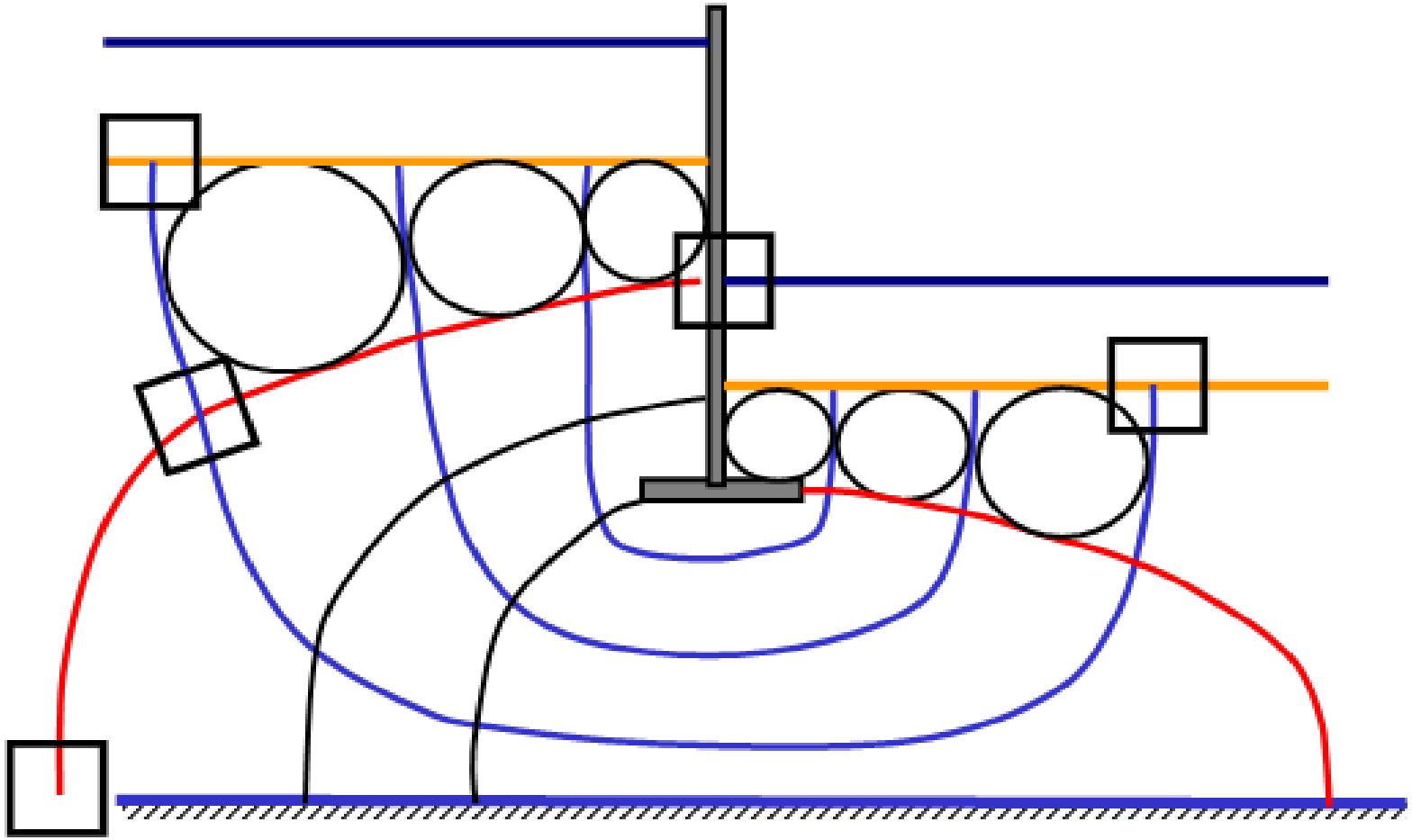


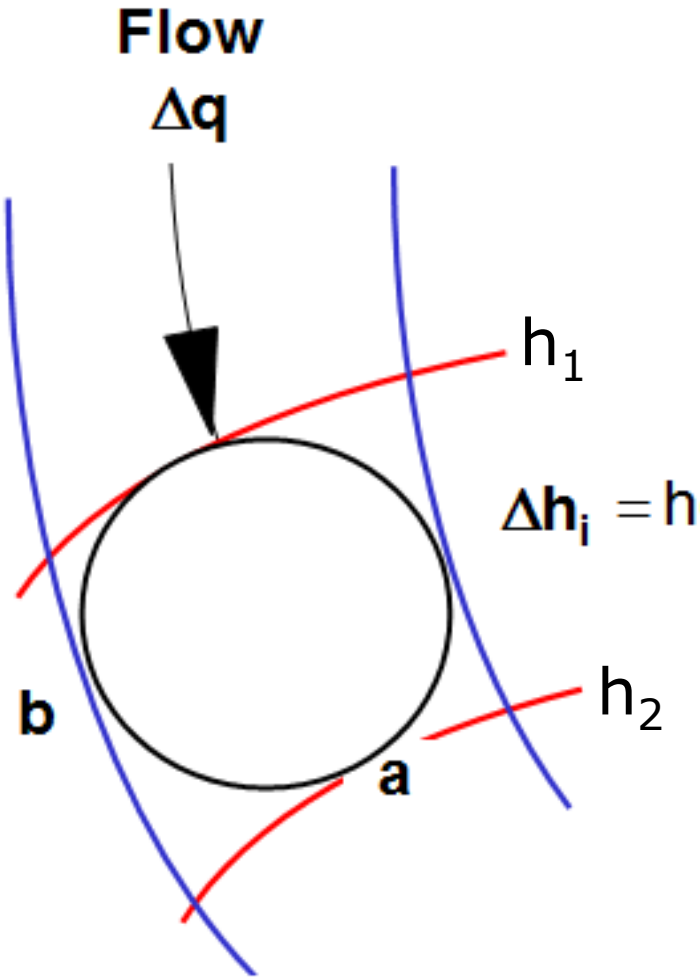
Boundary Conditions




Sheet pile
Water level
H_1
H
H_2
b
H
a
d
0
e
$k_x = k_z = k$
$N_f = 4$
$N_d = 6$
H-Δh
H-2Δh
H-3Δh
H-4Δh
H-5Δh
f
g
Impervious layer

Flow Net Construction





Δh_i = head lost between equipotentials

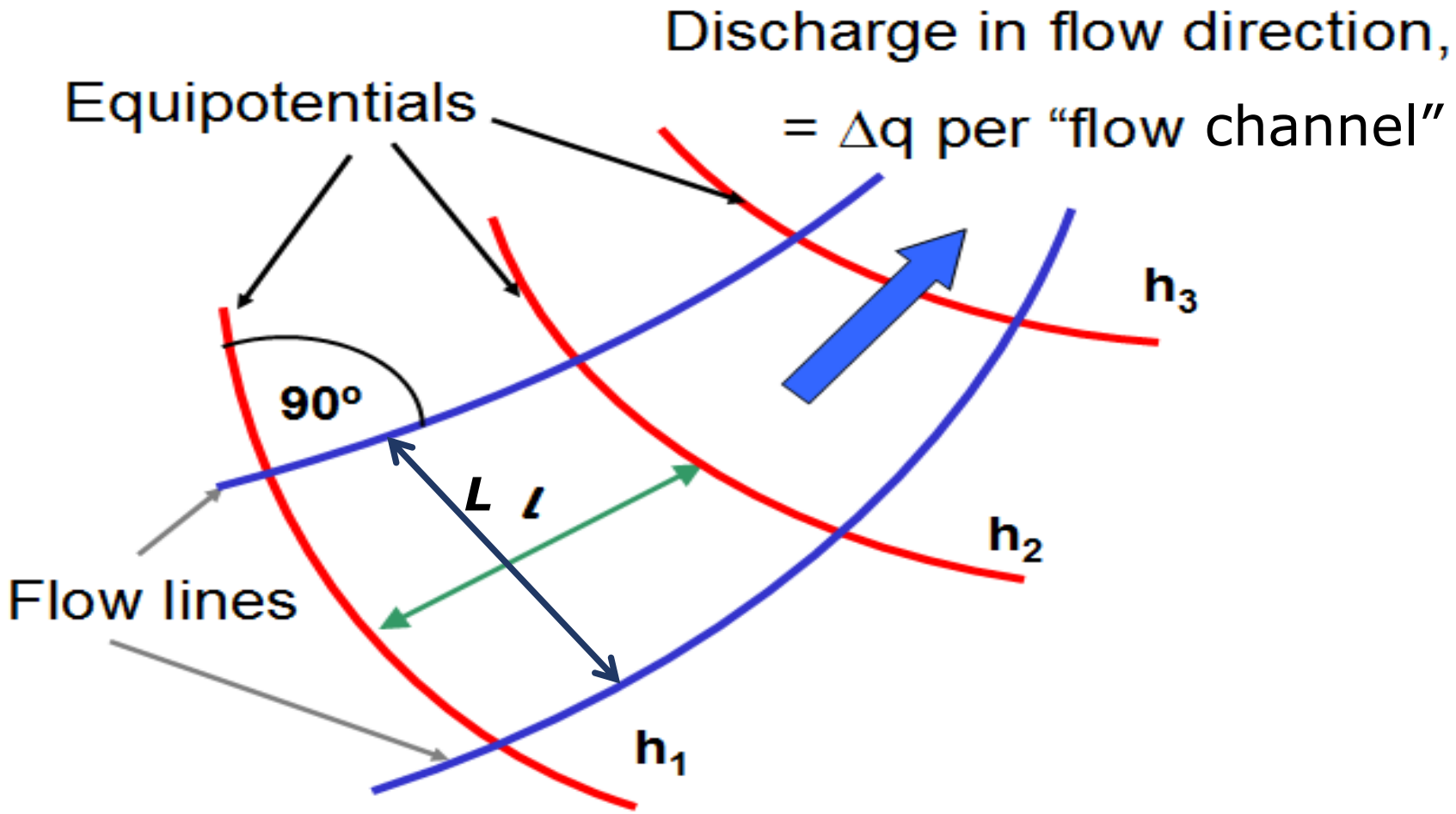
$= h_1 - h_2$

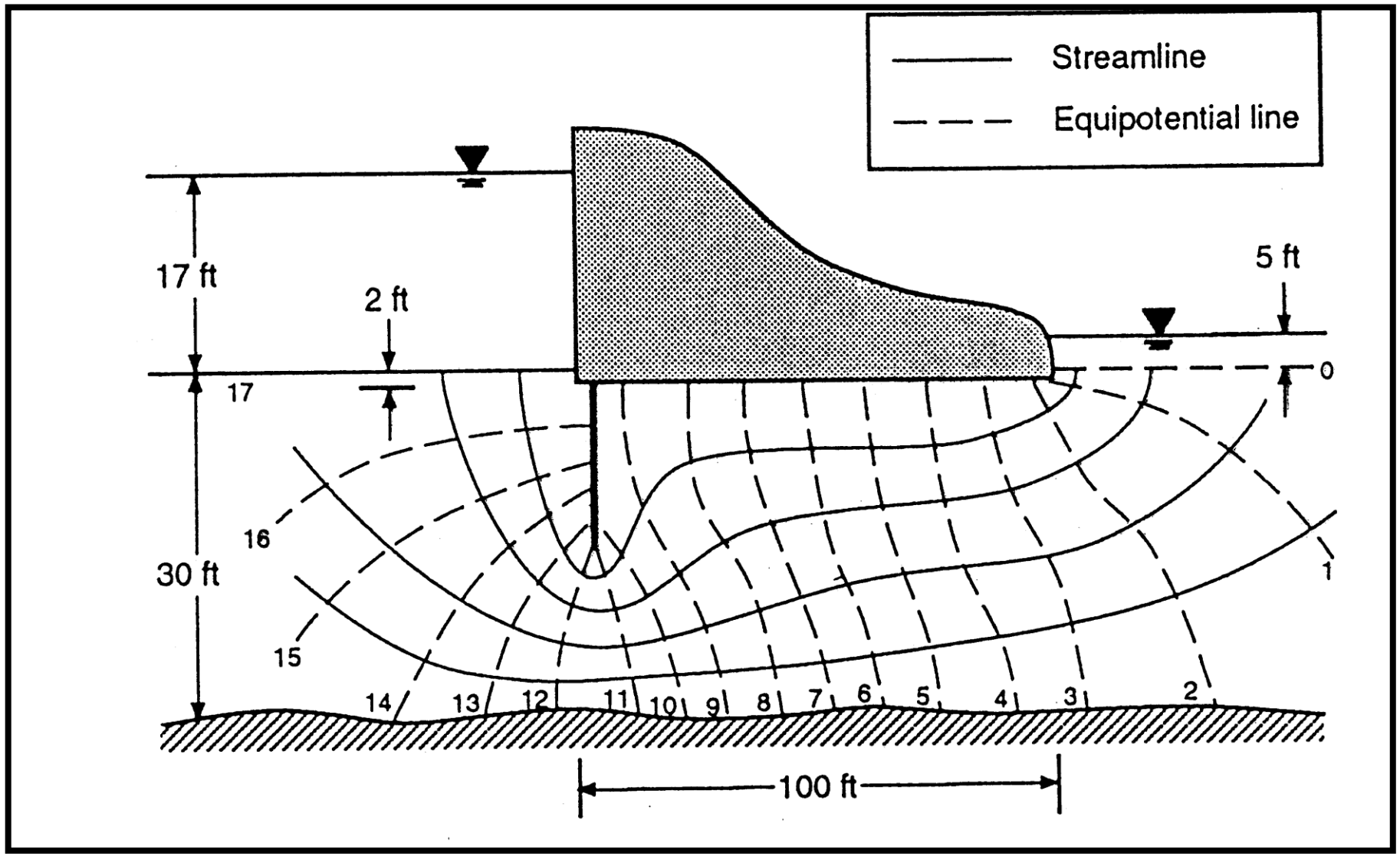
Common convention

draw "squares" with a = b

"square, M", a x b

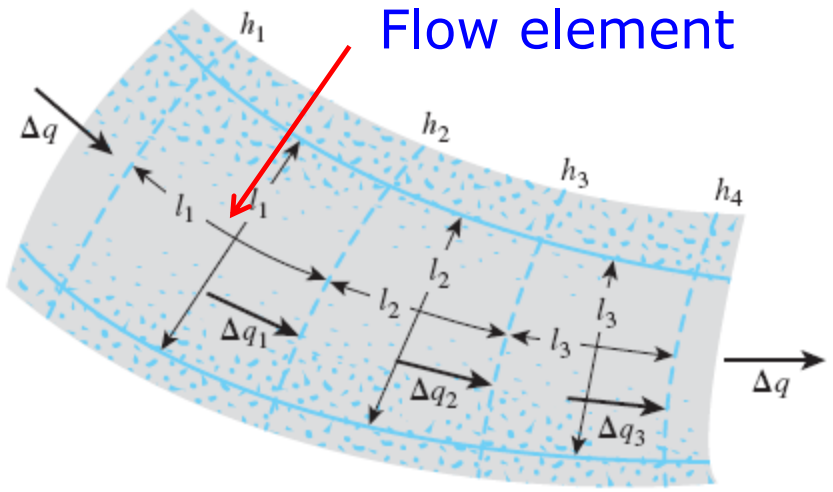
Seepage Flow




Streamline
Equipotential line
17 ft
2 ft
5 ft
30 ft
100 ft
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0

Seepage Calculation from Flow Net

- In a flow net, the strip between any two adjacent flow lines is called a flow channel.
- The drop in the total head between any two adjacent equipotential lines is called the potential drop.



- If the ratio of the sides of the flow element are the same along the flow channel, then:

1. Rate of flow through the flow channel per unit width perpendicular to the flow direction is the same.

$$\Delta q_1 = \Delta q_2 = \Delta q_3 = \Delta q$$

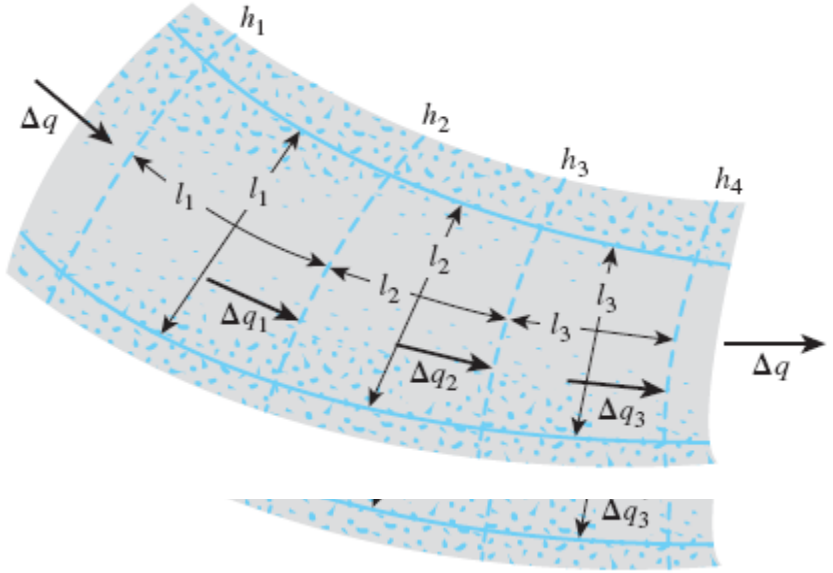
2. The potential drop is the same and equal to:

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

Where H: head difference between the upstream and downstream sides. N_d: number of potential drops.

Seepage Calculation from Flow Net

From Darcy's Equation, the rate of flow is equal to:



$$\Delta q = k\left(\frac{h_1 - h_2}{l_1}\right)l_1 = k\left(\frac{h_2 - h_3}{l_2}\right)l_2 = k\left(\frac{h_3 - h_4}{l_3}\right)l_3$$

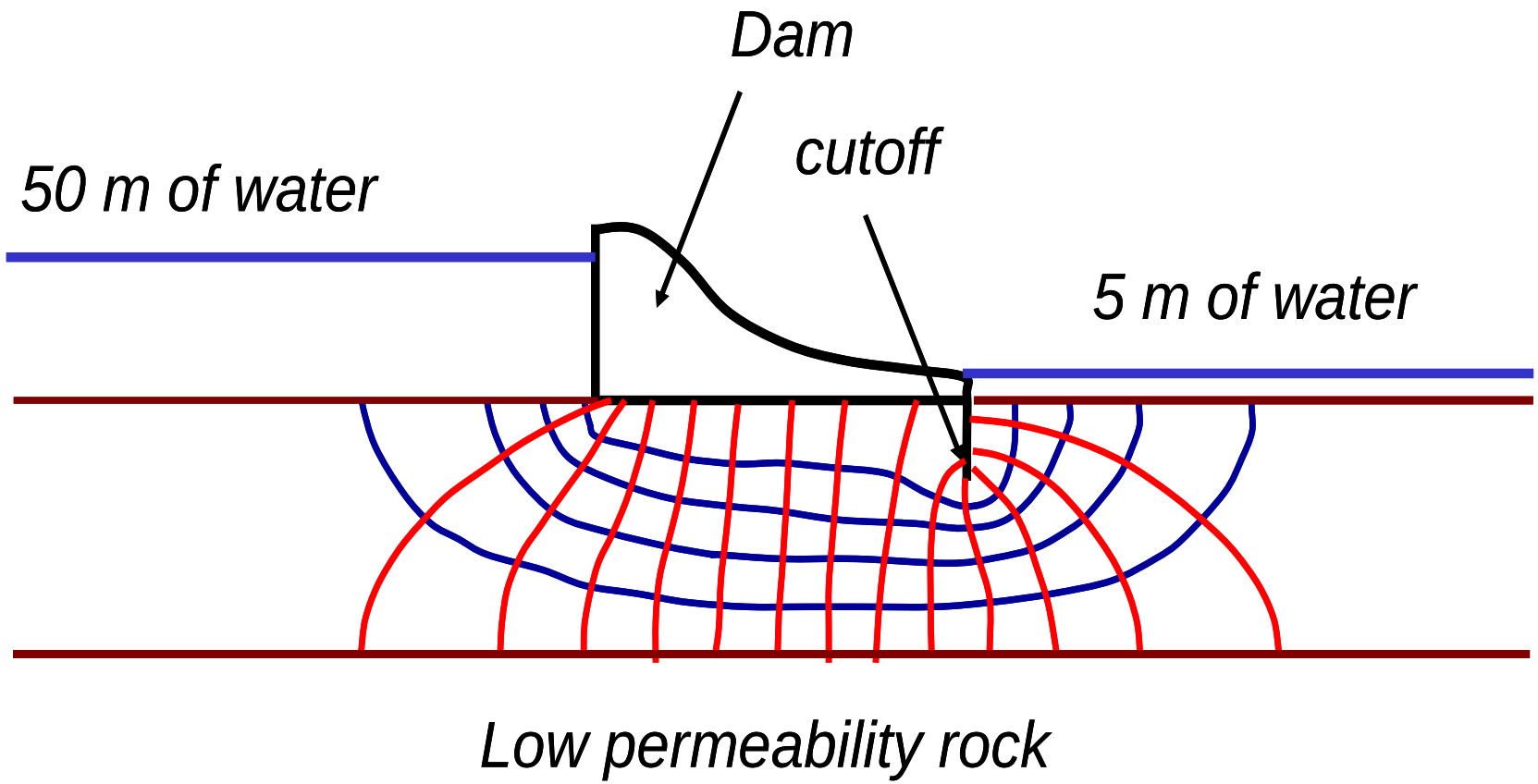
$$\Delta q = k\,H\left(\frac{1}{N_d}\right)$$

- 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:

$$\Delta q = k\,H\left(\frac{N_f}{N_d}\right)$$

Example on estimating the total flow under dam

Example: if k = 10^{-7} m/sec, what would be the flow per day over a 100 m length of wall?



Calculations

N_f = 5

N_d = 14

Δh = 45 m

k = 10^{-7} m/sec

$$q = k \frac{N_f}{N_d} \Delta h$$

Answer:

= 10^{-7}(5/14) 45 x 100 m length

= 0.000161 m^3/sec

= 13.9 m^3/day

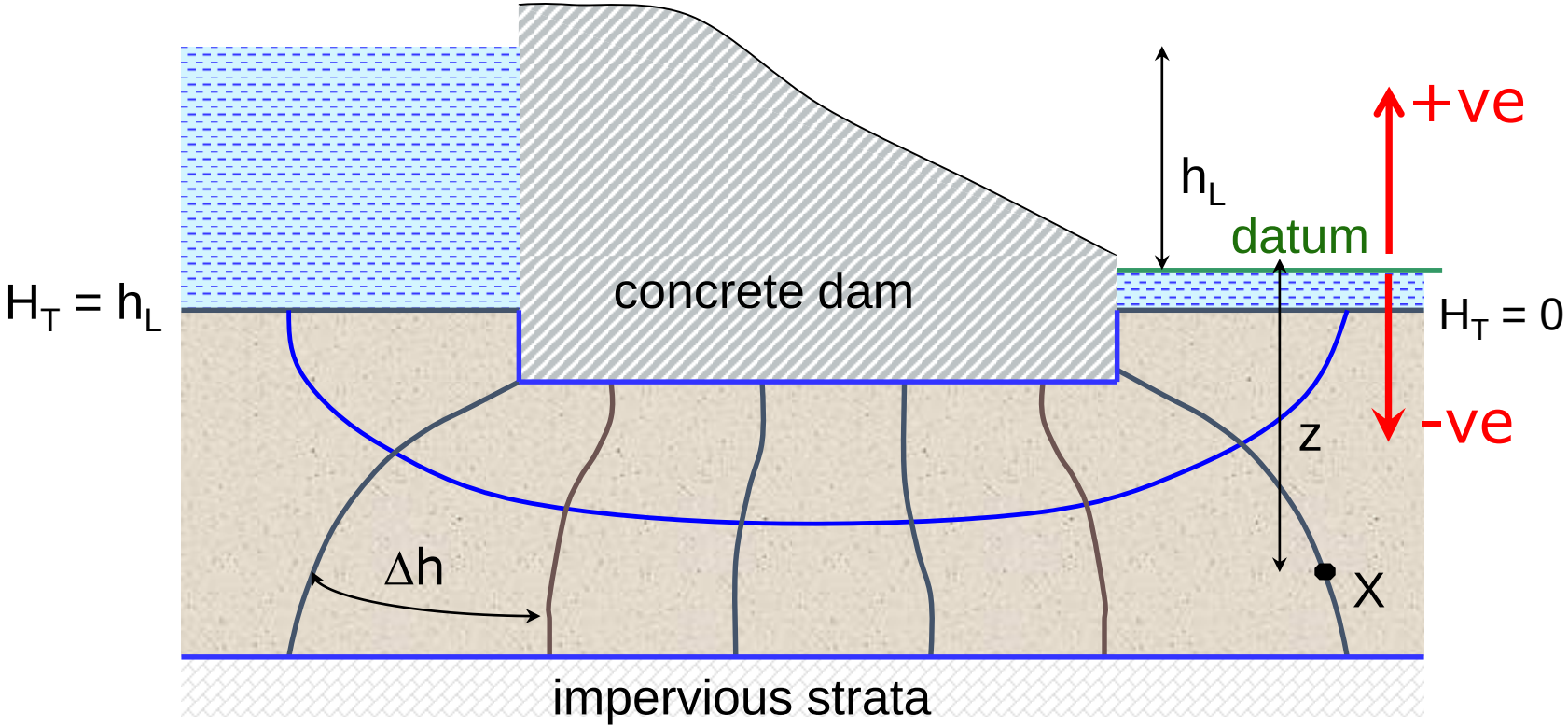
Pressure head at any point

Total head = h_L - # of drops from upstream x Δh

Elevation head = -z

Pressure head = Total head – Elevation head

$$\Delta h = \frac{h_L}{N_d}$$



Uplift Pressure under Hydraulic structures

At point a:

$u_a/\gamma_w = (7 - 1x1) - (-2)$

$= 8\ kN/m^2$

At point b:

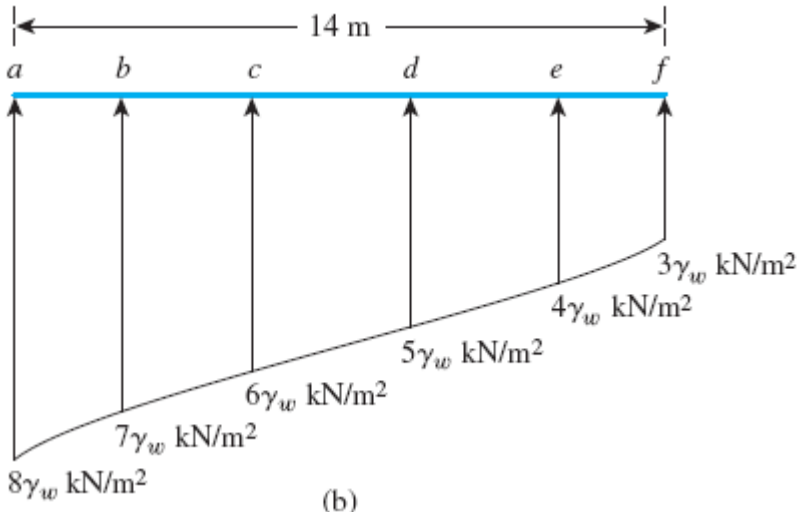
$u_a/\gamma_w = (7 - 2x1) - (-2)$

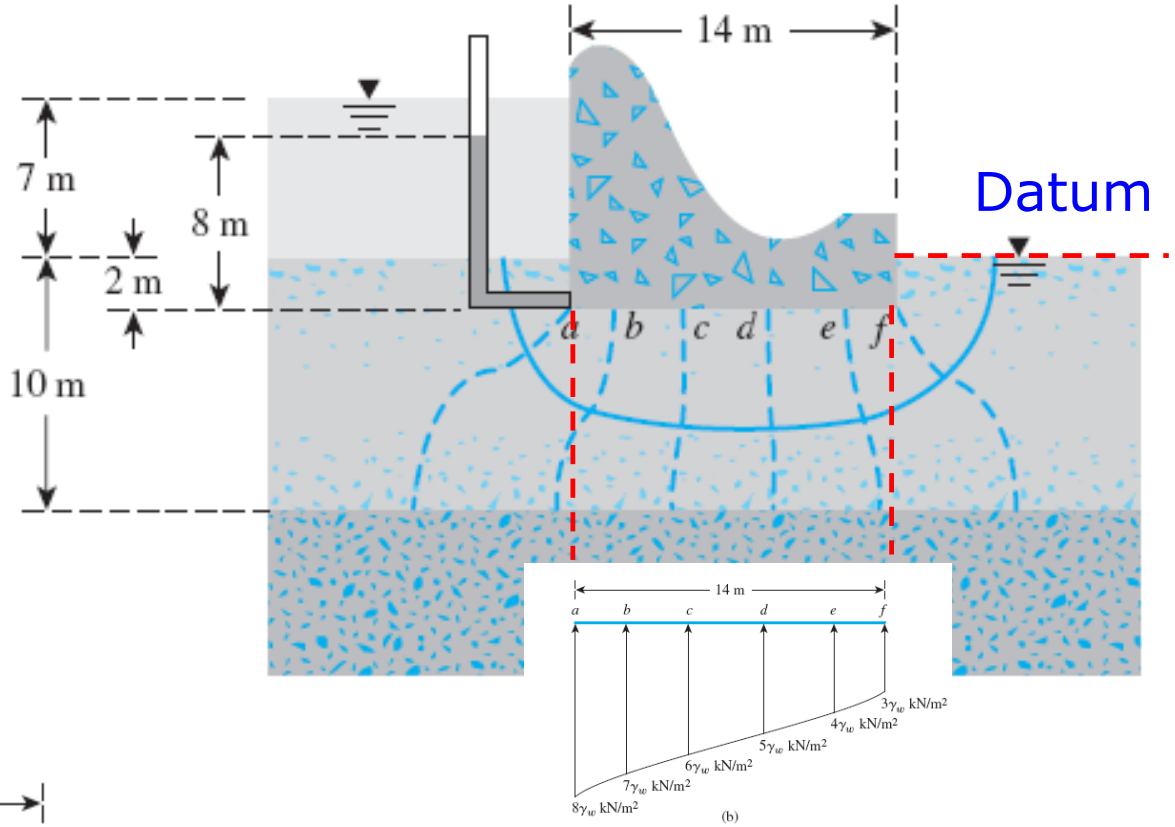
$= 7\ kN/m^2$

At point f:

$u_a/\gamma_w = (7 - 6x1) - (-2)$

$= 3\ kN/m^2$





$\Delta h = h_L/N_d = 7/7 = 1$

Example of Dam Failure

Around 7:00 am on June 5, 1976 a leak about 30 m from the top of Teton dam was observed.

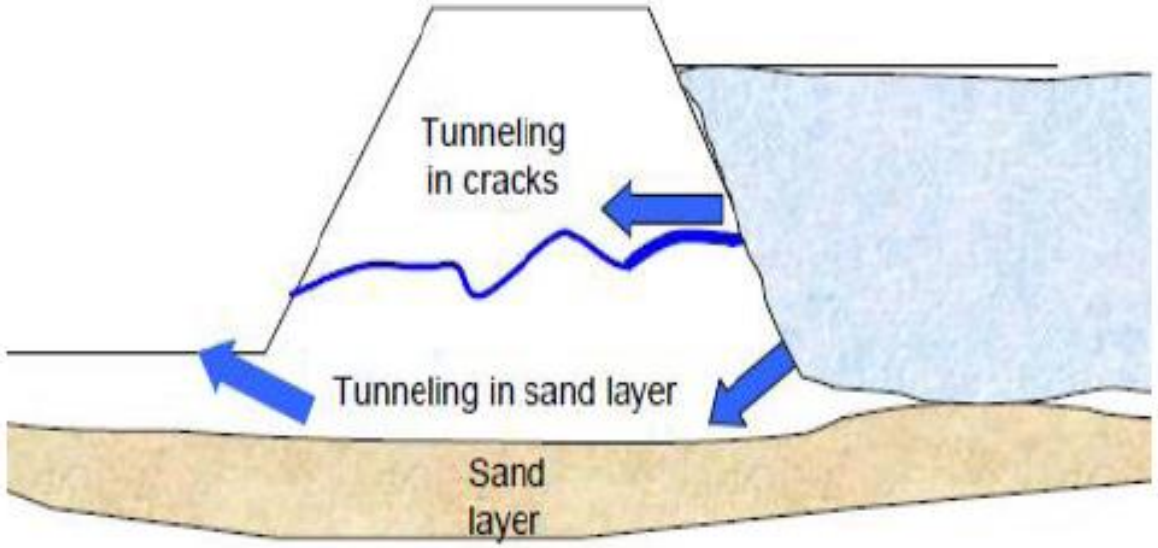


Example of Dam Failure

The Dam Broke at 11:59 AM



Example of Dam Failure



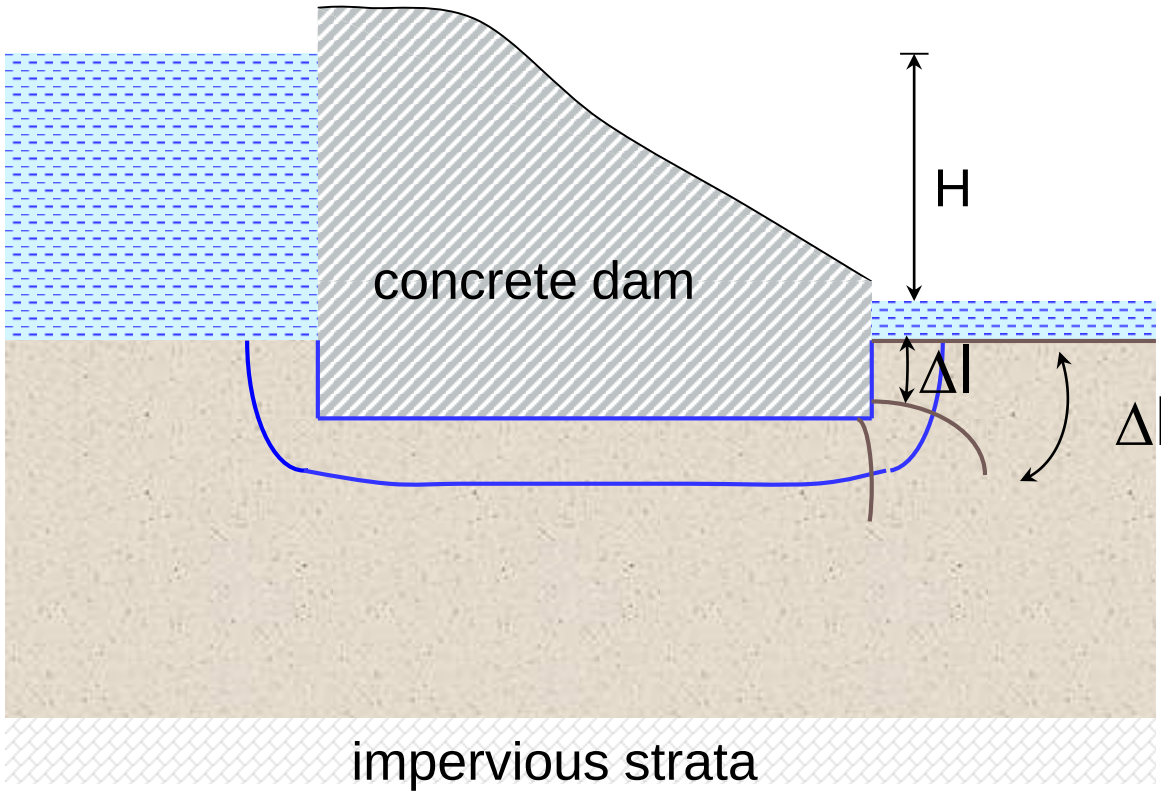
Post Failure Investigation

- Seepage piping and internal erosion
- Seepage through rock openings
- Hydraulic fracture
- Differential settlement and cracking
- Settlement in bedrock

Piping in Granular Soils

At the downstream, near the dam,

the exit hydraulic gradient $i_{exit} = \frac{\Delta h}{\Delta l}$

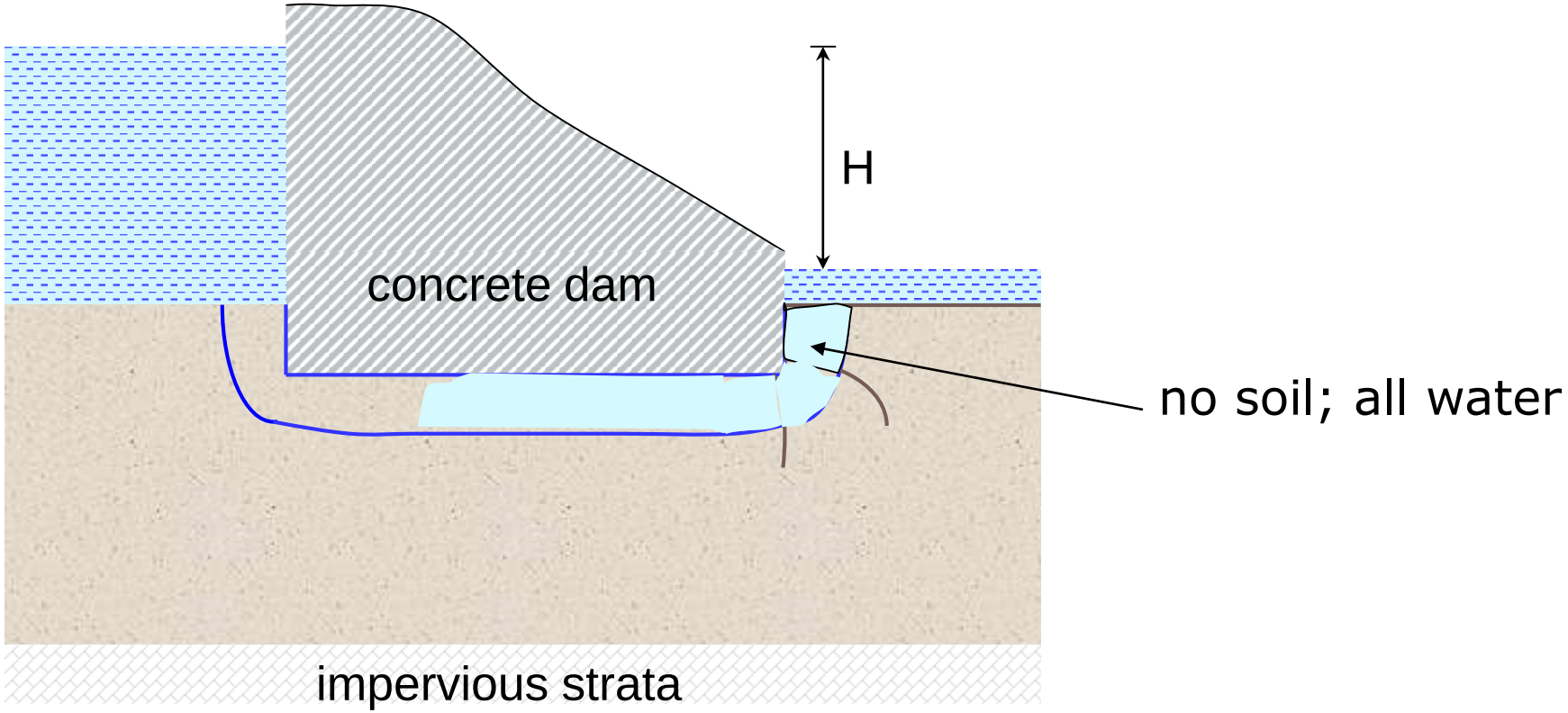


Δh = total head drop

Piping in Granular Soils

If i_{exit} exceeds the critical hydraulic gradient (i_{cr}), first the soil grains at exit get washed away.

This phenomenon progresses towards the upstream, forming a free passage of water ("pipe").



Critical hydraulic gradient, i_c

The critical hydraulic gradient (i_c),

$$i_c = \frac{\gamma'}{\gamma_w} = \frac{G_s - 1}{1 + e} \qquad \gamma' = \gamma_{sat} - \gamma_w$$

Consequences:

no stresses to hold granular soils together

$\therefore$ soil may flow $\Rightarrow$

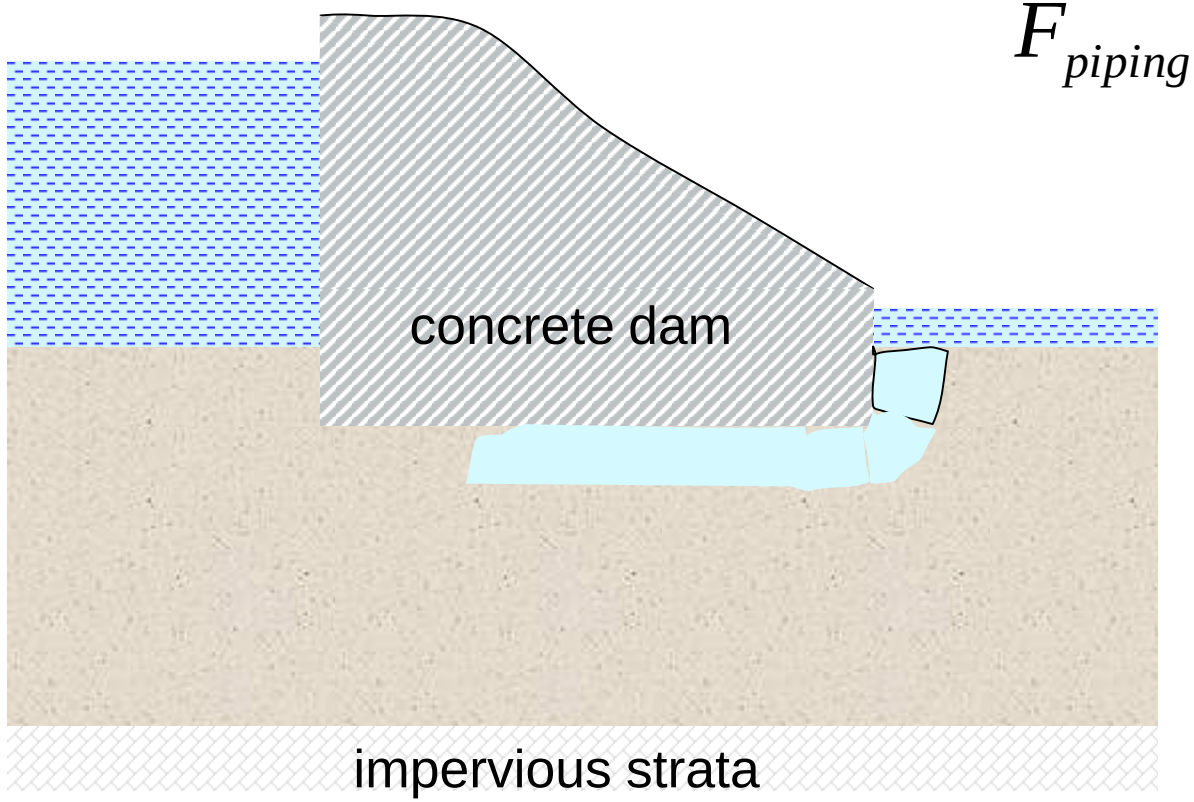
"boiling" or "piping" = EROSION

Piping in Granular Soils

Piping is a very serious problem. It leads to downstream flooding which can result in loss of lives.

Therefore, provide adequate safety factor against piping.

$$F_{piping} = \frac{i_c}{i_{exit}} > 3$$



Filters

Used for:

- facilitating drainage
- preventing fines from being washed away

Used in:

- earth dams
- retaining walls

Filter Materials:

- granular soils
- geotextiles

Granular Filter Design

Condition 1: $\frac{D_{15(filter)}}{D_{85(soil)}} \leq 4 \text{ to } 5$

Condition 2: $\frac{D_{15(filter)}}{D_{15(soil)}} \geq 4 \text{ to } 5$

after Terzaghi & Peck (1967)

$\frac{D_{15(filter)}}{D_{85(soil)}} < 5$

$\frac{D_{15(filter)}}{D_{15(soil)}} > 4$

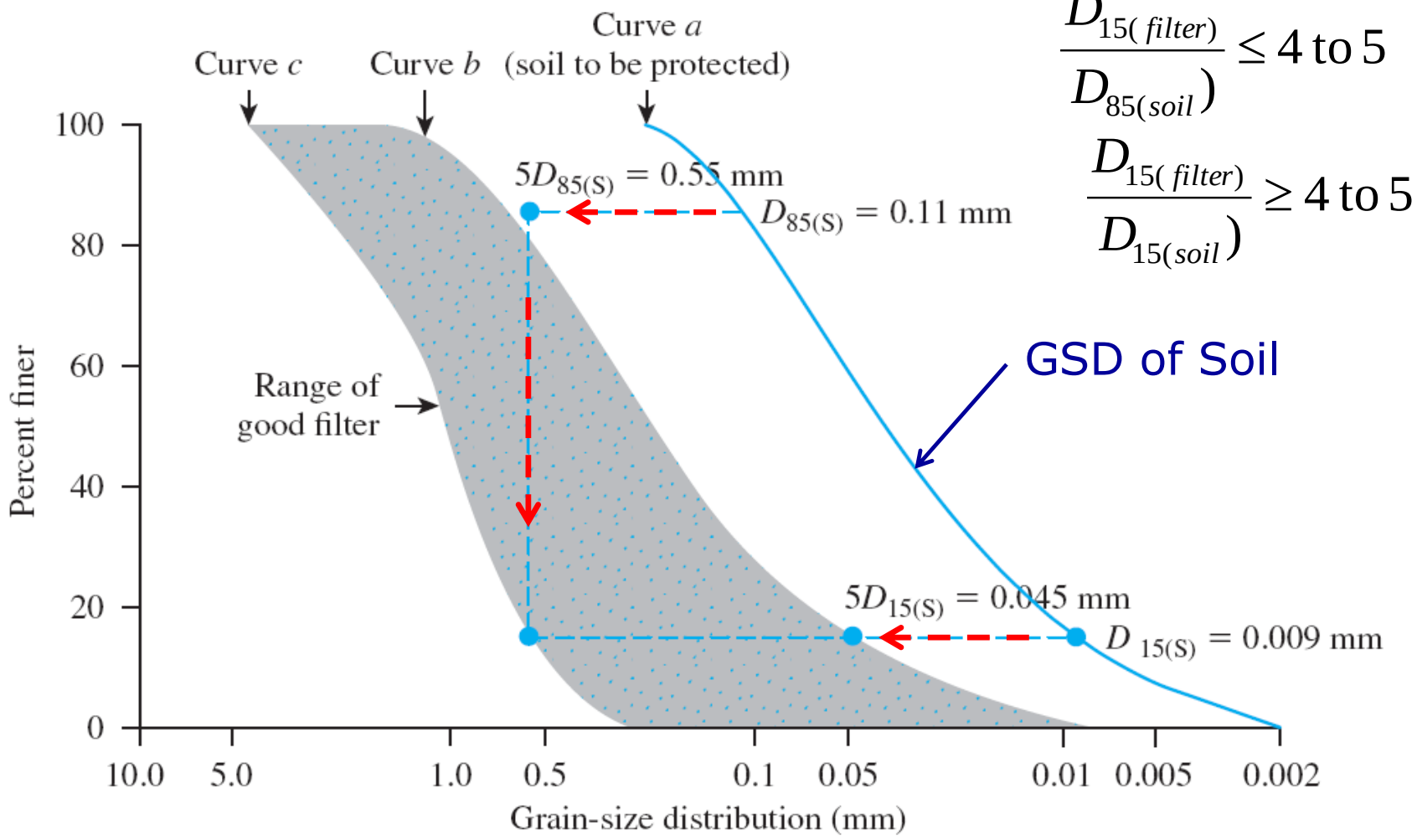
$\frac{D_{50(filter)}}{D_{50(soil)}} < 25$

$\frac{D_{15(filter)}}{D_{15(soil)}} < 20$

after US Navy (1971)

GSD Curves for the soil and filter must be parallel

Granular Filter Design



$$\frac{D_{15(\,filter)}}{D_{85(soil\,)}} \le 4 \text{ to } 5$$

$$\frac{D_{15(\,filter)}}{D_{15(soil\,)}} \ge 4 \text{ to } 5$$

Figure 8.17 Determination of grain-size distribution of filter using Eqs. (8.41) and (8.42)

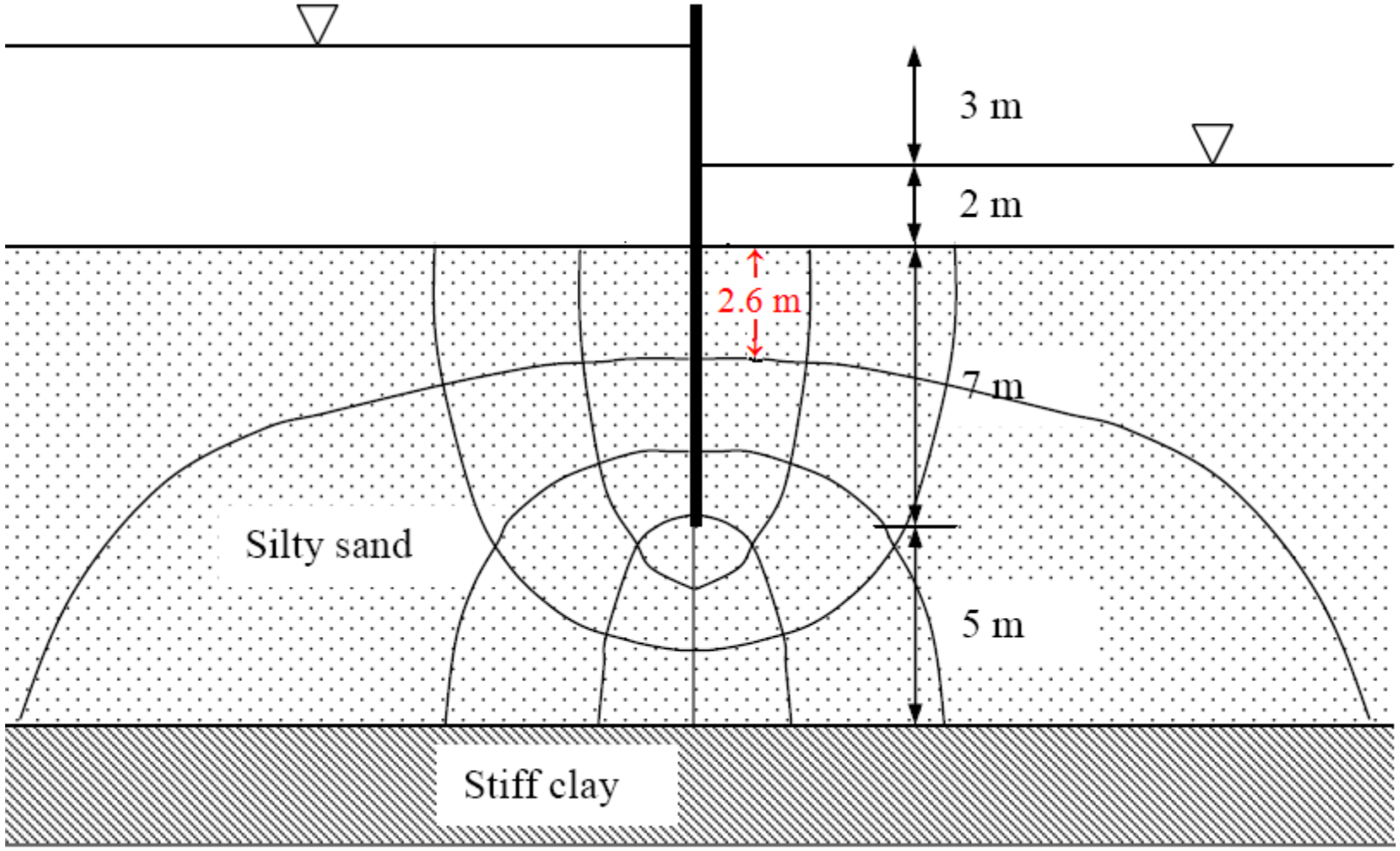
GSD Curves for the soil and filter must be parallel

EXAMPLE

- A stiff clay layer underlies a 12 m thick silty sand deposit. A sheet pile is driven into the sand to a depth of 7 m, and the upstream and downstream water levels are as shown in the figure. Permeability of the silty sand is 8.6×10^{-4}

  cm/s. The stiff clay can be assumed to be impervious. The void ratio of the silty sand is 0.72 and the specific gravity of the grains is 2.65.
- (a) Estimate the seepage beneath the sheet pile in m^3/day per meter.
- (b) What is the pore water pressure at the tip of the sheet pile?
- (c) Is the arrangement safe against piping?

EXAMPLE



Solution

(a) In the flow net, $N_f = 3$; $N_d = 8$; $\Delta h = 3$ m. The flow (Q) is given by:

$$q = kh_L \frac{N_f}{N_d} = (8.6\times10^{-6})(3)\left(\frac{3}{8}\right)(24\times3600) = 0.836\,\text{m}^3/day \text{ per metre}$$

(b) Taking downstream water level as the datum, at the tip of the sheet pile,

Total head = 1.5 m

Elevation head = -9 m

$\therefore$ Pressure head = 1.5 – (-9) = 10.5 m

Pore water pressure = (10.5)(9.81) = 103.0 kPa

(c) Head loss per equipotential drop, $\Delta h = 3/8 = 0.375$ m

The maximum exit hydraulic gradient (near the sheet pile) = 0.375/2.6 = 0.144

The critical hydraulic gradient (i_c) is given by:

$$i_c = \frac{G_s - 1}{1+e} = \frac{2.65-1}{1+0.72} = 0.96$$

$\therefore$ Safety factor with respect to piping F = 0.96/0.144 = 6.7 > 5

The arrangement is quite safe with respect to piping.

Stress in Soil Mass

General Outline



- Introduction
- Geostatic Stress
- Additional Stress
- Stress Bulbs & Contact Pressure
- Stress States & Stress Paths

1. Introduction



- Definitions
- Fundamentals of Stress-Strain
- Material Responses to Normal Loading
- Stresses in Soil Mass

Introduction

Definitions

Stress: or intensity of loading, is the load per unit area.

Strain: or intensity of deformation, is the ratio of the change in a dimension to the original dimension or the ratio of change in length to the original length.

Stiffness : resistance to deformation

Strength : ability to withstand an applied load without failure or plastic deformation

Poisson's ratio : ratio of transverse contraction strain to longitudinal extension strain in the direction of stretching force

Modulus of Elasticity: describes tendency of an object to deform along an axis when opposing forces are applied along that axis

Nominal stress = actual load/original cross-sectional area of specimen, i.e. no allowance is made for reduction in area, due to necking, as the load is increased.

Introduction

Definitions

Stress* (*strain*) *state at a point is a set of stress (strain) vectors corresponding to all planes passing through that point.

Mohr's circle is used to graphically represent stress (strain) state for two-dimensional bodies.

Isotropic means the material properties are the same in all directions, and also the loadings are the same in all directions.

Anisotropic means the material properties are different in different directions, and also the loadings are different in different directions.

Elastic materials are materials that return to their original configuration on unloading and obey Hooke's law.

Plastic materials do not return to their original configuration on unloading.

Introduction cntd

Soils are not homogeneous, elastic, rigid bodies, so the determination of stresses and strains in soils is a particularly difficult task.

One may ask: "If soils are not elastic materials, then why do I have to study elastic methods of analysis?"

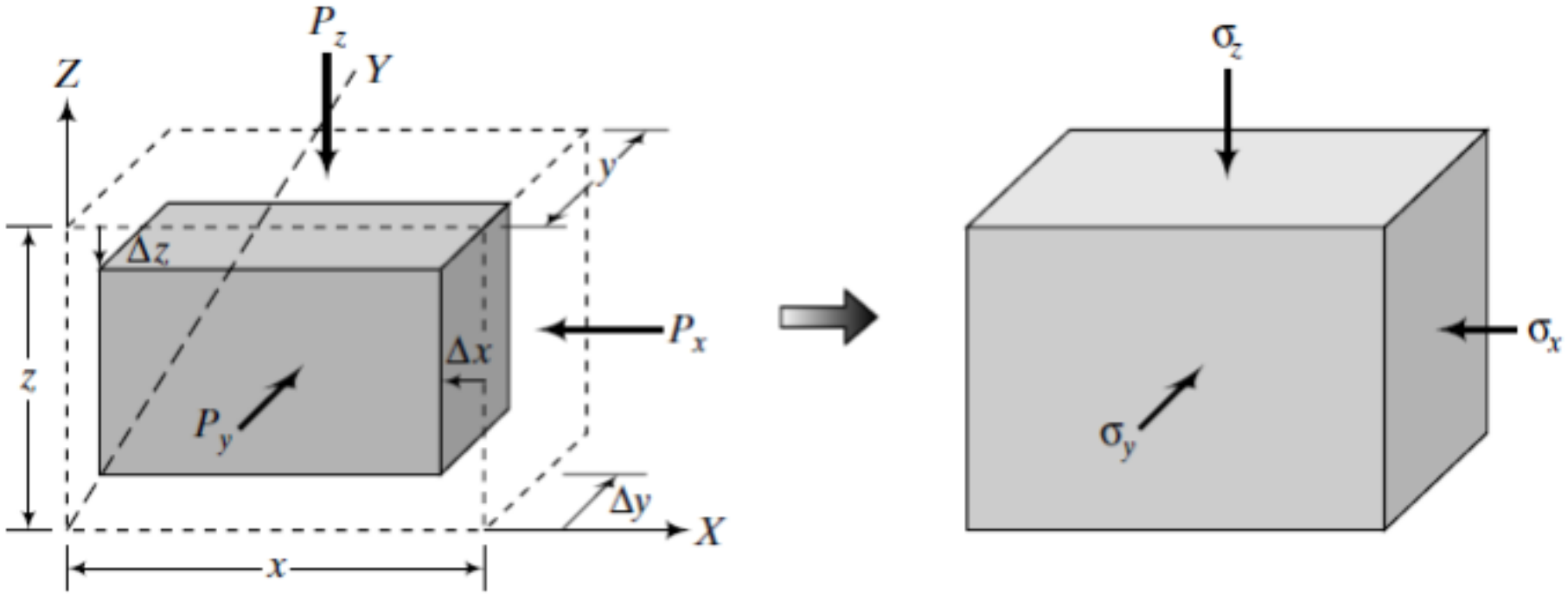
- An elastic analysis of an isotropic material involves only two constants—Young's modulus and Poisson's ratio—and thus if we assume that soils are isotropic elastic materials, then we have a powerful, but simple, analytical tool to predict a soil's response under loading. We will have to determine only the two elastic constants from our laboratory or field tests.

Introduction cntd

Stresses and Strains – A Recap of the Fundamentals

Normal Stresses and Strains

Consider a cube of dimensions $x = y = z$ that is subjected to forces $P_x = P_y = P_z$, normal to three adjacent sides.



Introduction cntd

The normal stresses:

$$\sigma_z = \frac{P_z}{xy}\,, \sigma_x = \frac{P_x}{yz}\,, \sigma_y = \frac{P_y}{xz}$$

Consider a cube of dimensions $x = y = z$ that is subjected to forces $P_x = P_y = P_z$, normal to three adjacent sides.

Let us assume that under these forces the cube compressed by $\Delta x, \Delta y, \Delta z$ in the *X*, *Y*, and *Z* directions.

The strains in these directions, assuming they are small (infinitesimal):

$$\varepsilon_z = \frac{\Delta z}{z}\,, \varepsilon_x = \frac{\Delta x}{x}\,, \varepsilon_y = \frac{\Delta y}{y}$$

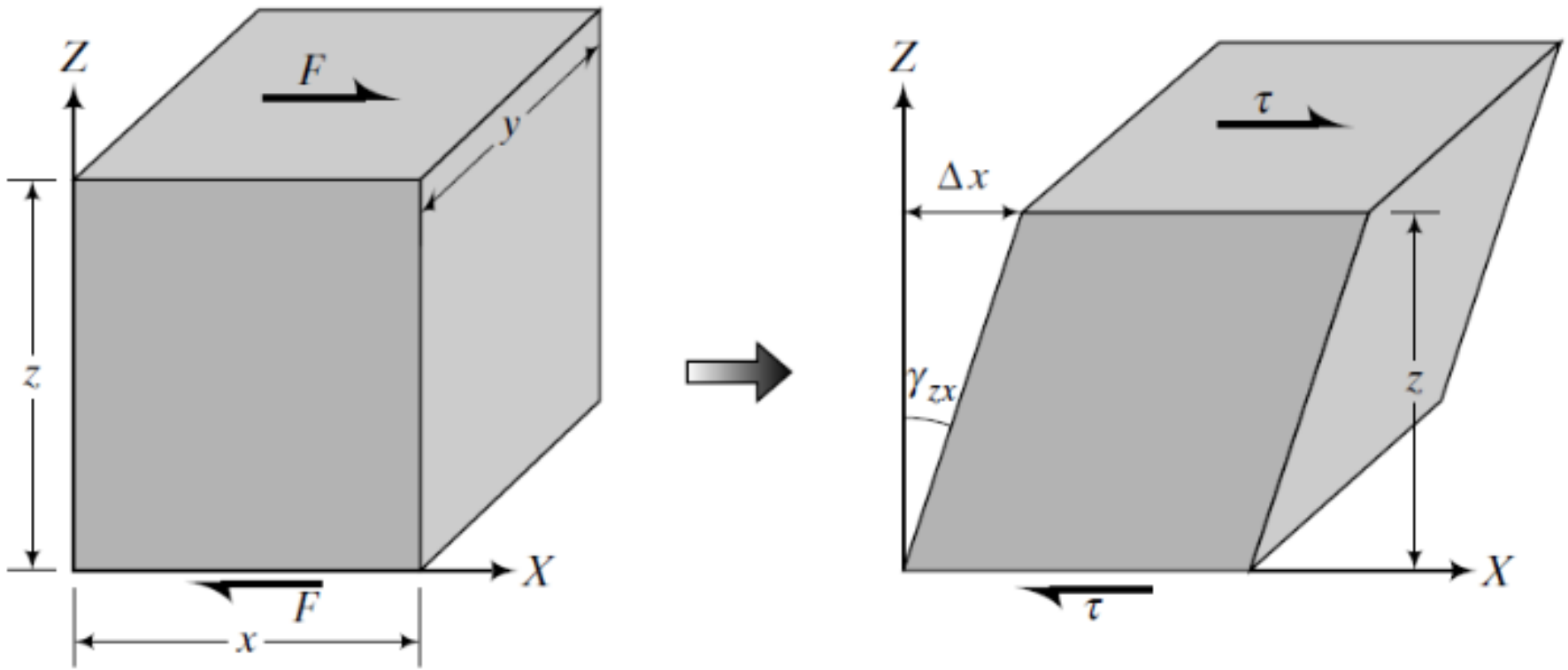
Volumeteric strain:

$$\varepsilon_p = \varepsilon_x + \varepsilon_y + \varepsilon_z$$

Introduction cntd

Shear Stresses and Shear Strains

Let us consider, for simplicity, the *XZ* plane and apply a force *F* that causes the square to distort into a parallelogram, as shown below.



Introduction cntd

The force *F* is a shearing force, and the shear stress is

$$\tau = \frac{F}{xy}$$

Simple shear strain is a measure of the angular distortion of a body by shearing forces. If the horizontal displacement is Δx, the shear strain or simple shear strain, γ_{zx}, is

$$\gamma_{zx} = \tan^{-1}\frac{\Delta x}{z}$$

For small strains, $\tan\gamma_{zx} = \gamma_{zx}$, and therefore

$$\gamma_{zx} = \frac{\Delta x}{z}$$

Introduction cntd

Material Responses to Normal Loading / Unloading

If we apply an incremental vertical load, ΔP, to a deformable cylinder of cross-sectional area *A*, the cylinder will compress by, say, Δz and the radius will increase by Δr.

The loading condition we apply here is called uniaxial loading.

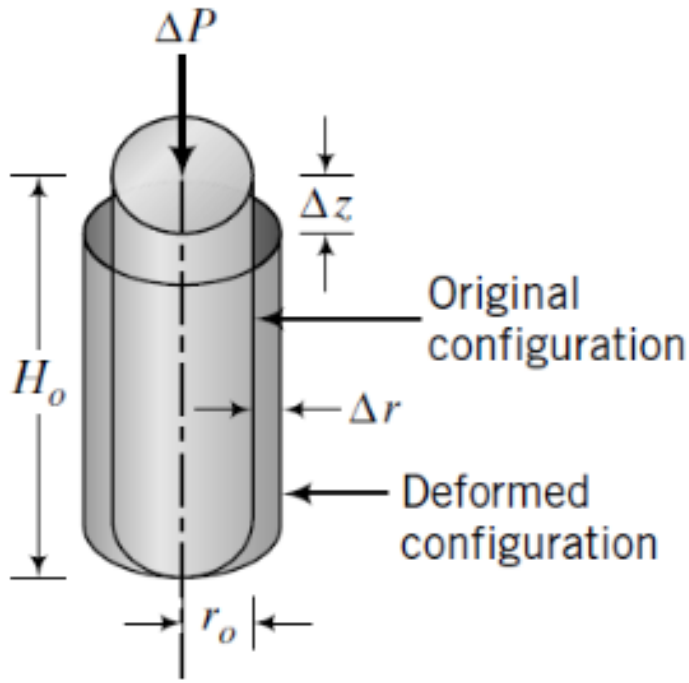
The change in vertical stress is

$$\Delta\sigma_z = \frac{\Delta P}{A}$$

The vertical and radial strains are, respectively,

$$\Delta\varepsilon_z = \frac{\Delta z}{H_o}$$

$$\Delta\varepsilon_r = \frac{\Delta r}{r_o}$$



Introduction cntd

Material Responses to Normal Loading / Unloading

The ratio of the radial (or lateral) strain to the vertical strain is called Poisson's ratio, υ, defined as

$$\upsilon = \frac{-\Delta\varepsilon_r}{\Delta\varepsilon_z}$$

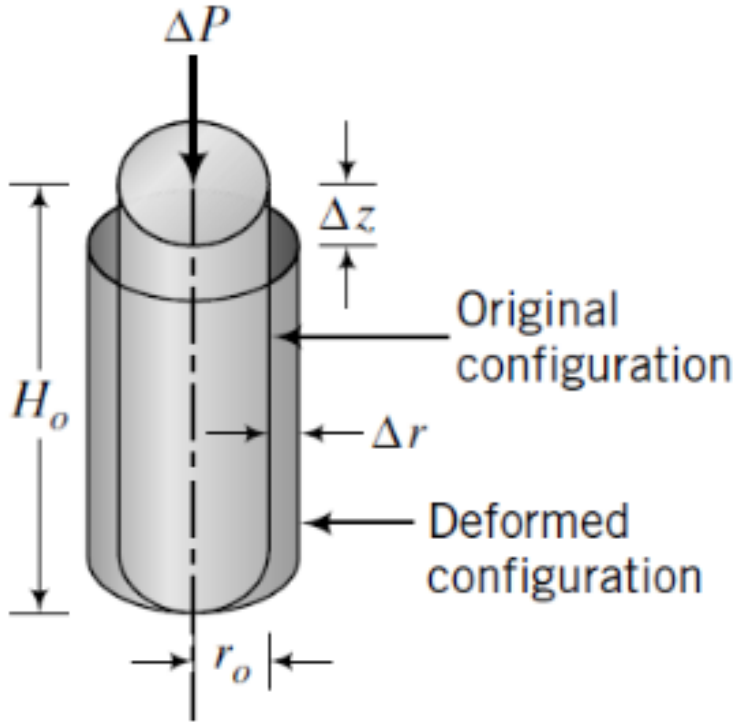


TABLE Typical Values of Poisson's Ratio

Soil type	Description	ν^a
Clay	Soft	0.35–0.40
	Medium	0.30–0.35
	Stiff	0.20–0.30
Sand	Loose	0.15–0.25
	Medium	0.25–0.30
	Dense	0.25–0.35

[a]These values are effective values, ν'.

Introduction cntd

Linear elastic materials

- For equal increments of ΔP, we get the same value of Δz.
- If at some stress point, say, at A , we unload the cylinder and it returns to its original configuration.

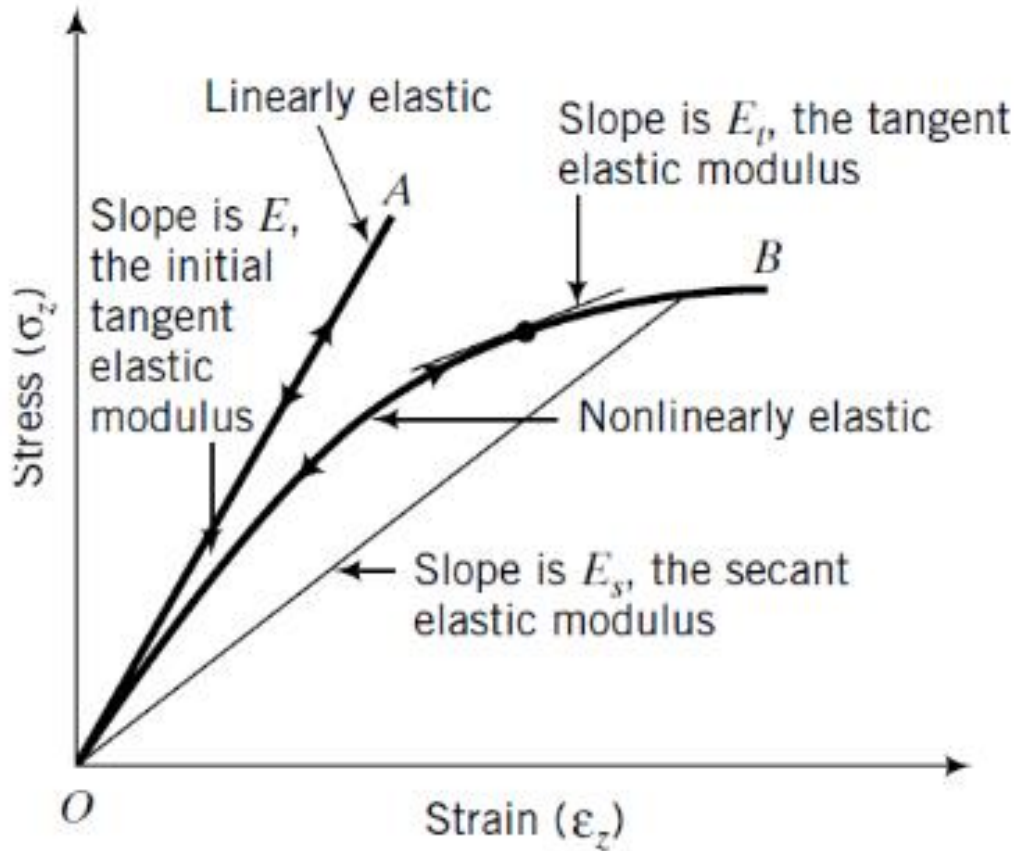
Non-linear elastic materials

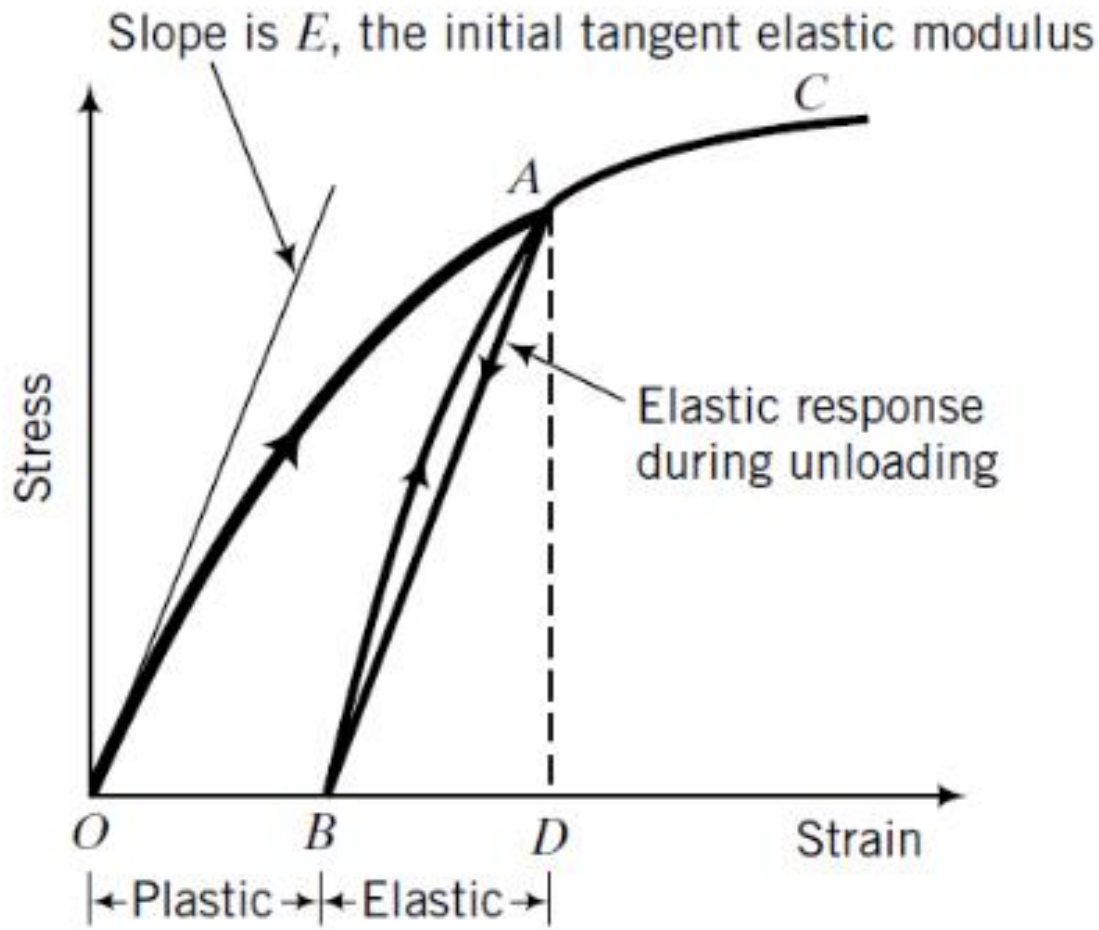
- For equal increments of ΔP, we get the different value of Δz.
- If at some stress point, say, at A , we unload the cylinder and it returns to its original configuration.

Elasto-plastic materials

- do not return to their original configurations after unloading.
- The strains that occur during loading consist an elastic or recoverable part and a plastic or unrecoverable part.

Introduction cntd





Introduction cntd

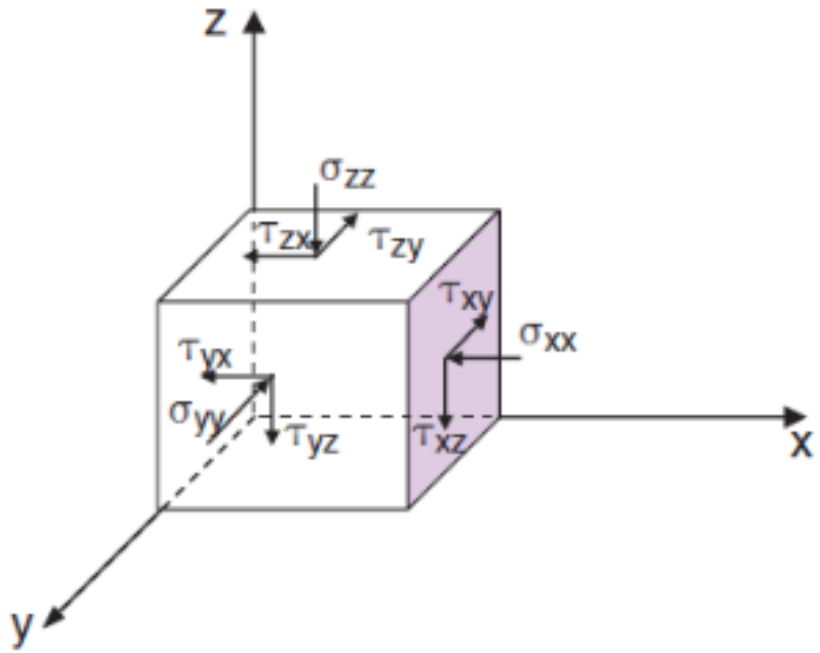
Moduli

- The elastic modulus or initial tangent elastic modulus (E) is the slope of the stress–strain line for linear isotropic material .
- The tangent elastic modulus (Et) is the slope of the tangent to the stress–strain point under consideration.
- The secant elastic modulus (Es) is the slope of the line joining the origin (0, 0) to some desired stress–strain point.

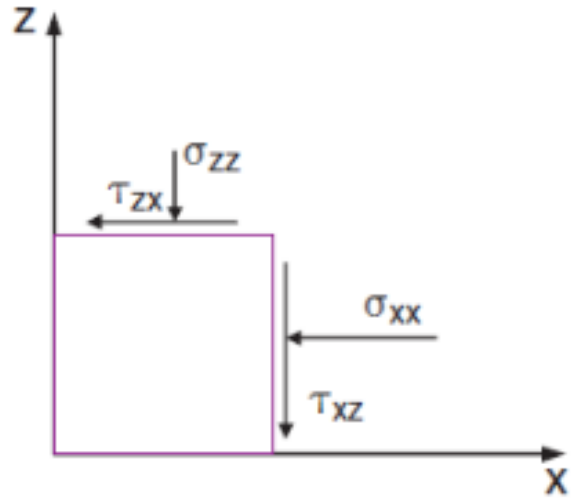
Introduction cntd

Stresses in Soil Mass

If we consider an elemental cube of soil at the point considered, then a solution by elastic theory is possible. Each plane of the cube is subjected to a stress, σ, acting normal to the plane, together with a shear stress, τ, acting parallel to the plane.



(a) 3-d stress system



(b) 2-d stress system

Introduction cntd

Stresses in Soil Mass

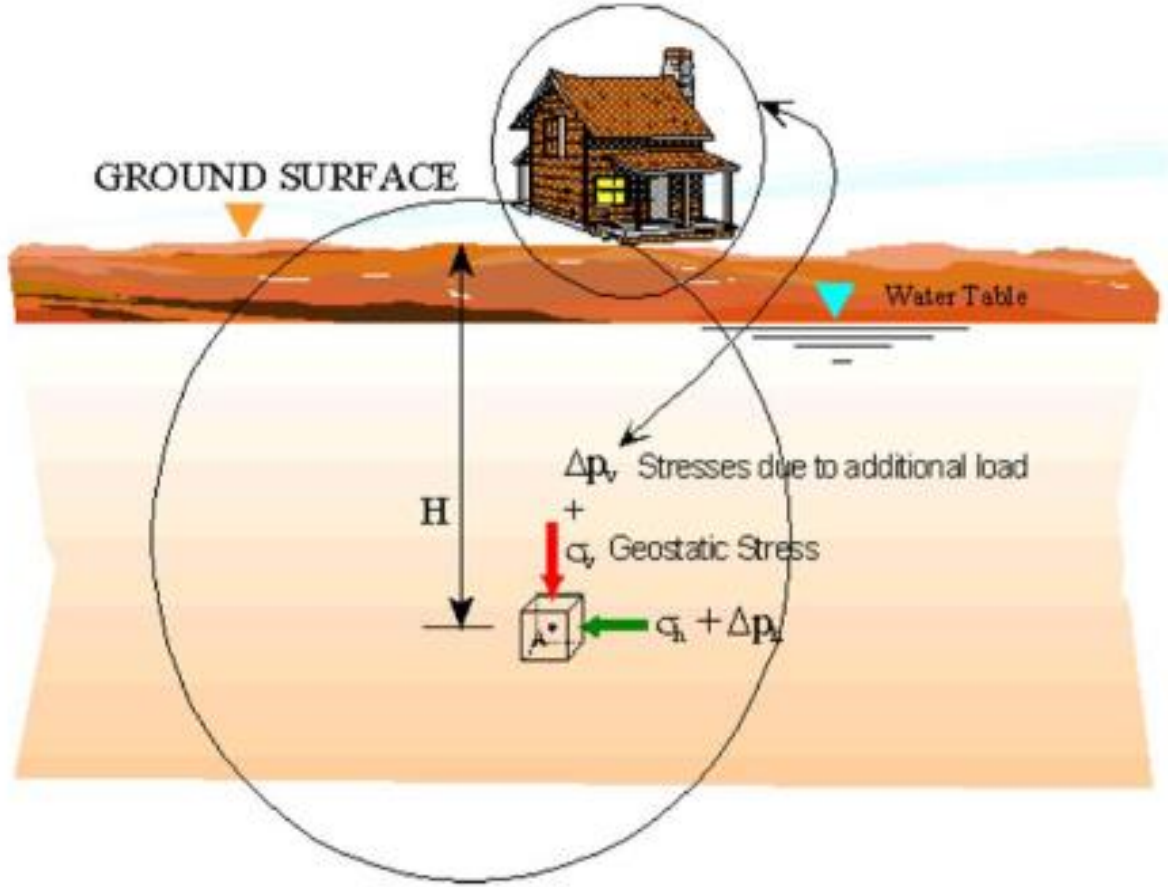
- There are therefore a total of six stress components acting on the cube.
- Once the values of these components are determined then they can be compounded to give the magnitudes and directions of the principal stresses acting at the point considered.
- Many geotechnical structures operate in a state of plane strain, i.e. one dimension of the structure is large enough for end effects to be ignored and the problem can be regarded as one of two dimensions.

Introduction cntd

Stresses in Soil Masses

- Geostatic stresses: from self weight of soil
- Additional stress: from imposed load

- Vertical stress
- Horizontal stress
- Shear stress



Introduction cntd

Vertical pressure at a point below ground surface is normally induced in four ways:

- **Geostatic Pressure/Overburden:** from self weight of soil; increases linearly with depth
  - **Hydrostatic Pressure:** from ground water; increases linearly with depth
- **Additional stress**: from an imposed structural load; decreases with depth in a non-linear manner
  - **Surcharge load**: a load of infinite extent on the surface; does not vary with depth.

2. Geostatic Stress



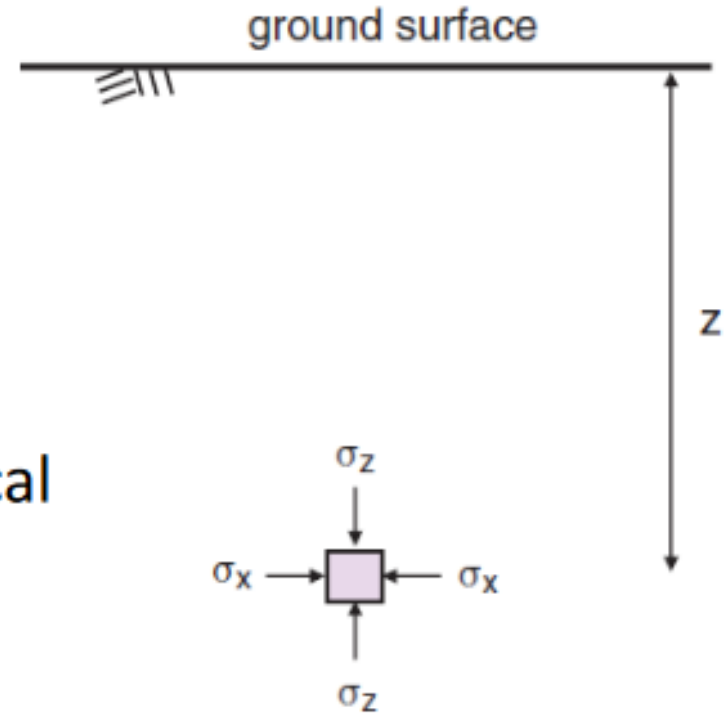
- Total Stress
- Pore Pressure
- Effective Stress
  - Effects of Capillarity
  - Effects of Seepage

Geostatic Stress

Total Stress

- is due to the weight of everything that lies above that point including soil, water and any load applied to the soil surface.

- increases with depth and with unit weight and the total vertical stress at depth z in the soil due to the weight of the soil acting above.



$$\sigma_z = \gamma z$$

where γ = unit weight of the soil.

If the soil is multi-layered, the total vertical stress is determined by summing the stresses induced by each layer of soil.

Geostatic Stress cntd

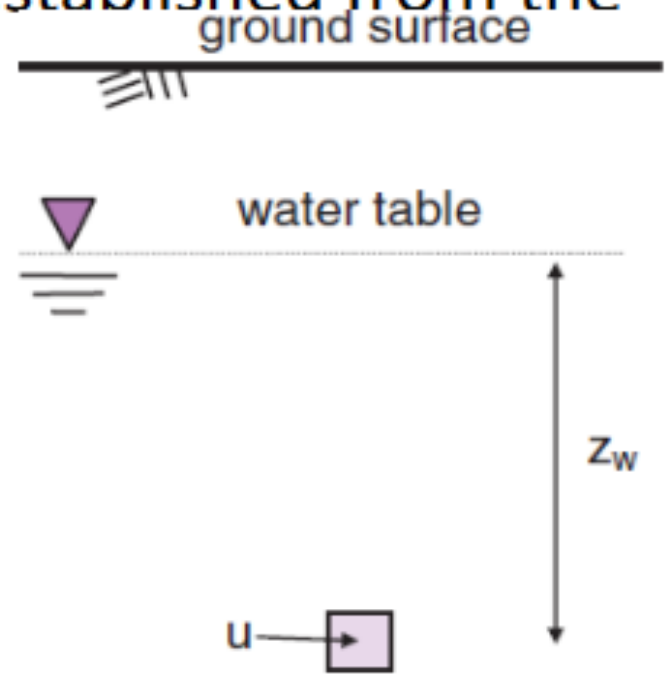
Pore Pressure

The water within the voids of a soil is known as pore water.

Pore water experiences pressure known as the pore pressure or pore water pressure, u.

In the case of a horizontal ground water table, we may be able to assume that no flow is taking place and the pore pressure at a point beneath the ground water table can be established from the hydrostatic pressure acting.

The magnitude of the pore pressure at the water table is zero.



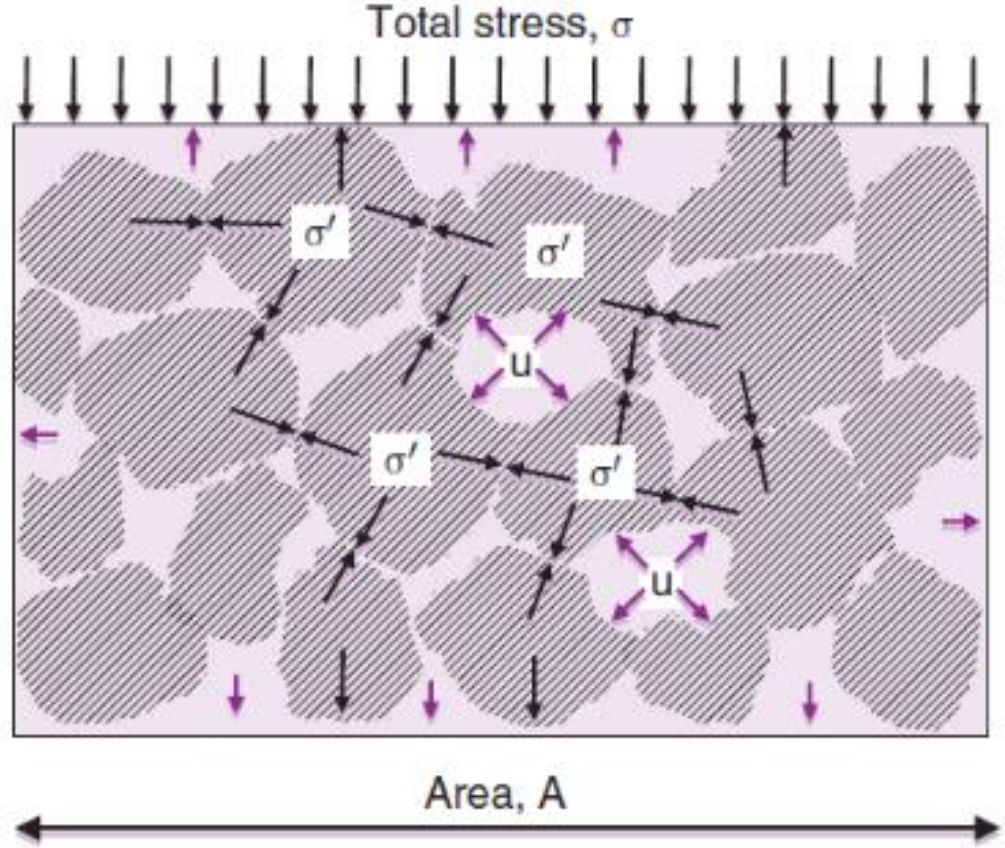
$u = \gamma_w z_w$

where z_w = the depth below the water table.

Geostatic Stress cntd

Effective Stress

The stress that controls changes in the volume and strength of a soil is known as the *effective stress.*



Consider saturated soils only.

When a load is applied to such a soil, it will be carried by the water in the soil voids (causing an increase in the pore water pressure) or by the soil skeleton (in the form of grain to grain contact stresses), or else it will be shared between the water and the soil skeleton.

The portion of the total stress carried by the soil particles is known as the effective stress, σ'.

Geostatic Stress cntd

Effective Stress

Terzaghi first presented the concept of effective stress in 1925.

He showed, from the results of many soil tests, that when an undrained saturated soil is subjected to an increase in applied normal stress, $\Delta\sigma$, the pore water pressure within the soil increases by Δu, and the value of Δu is equal to the value of $\Delta\sigma$. This increase in u caused no measurable changes in either the volumes or the strengths of the soils tested, and Terzaghi therefore used the term neutral stress to describe u, instead of the now more popular term pore water pressure.

$$\sigma' = \sigma - u$$

where the prime represents 'effective stress'.

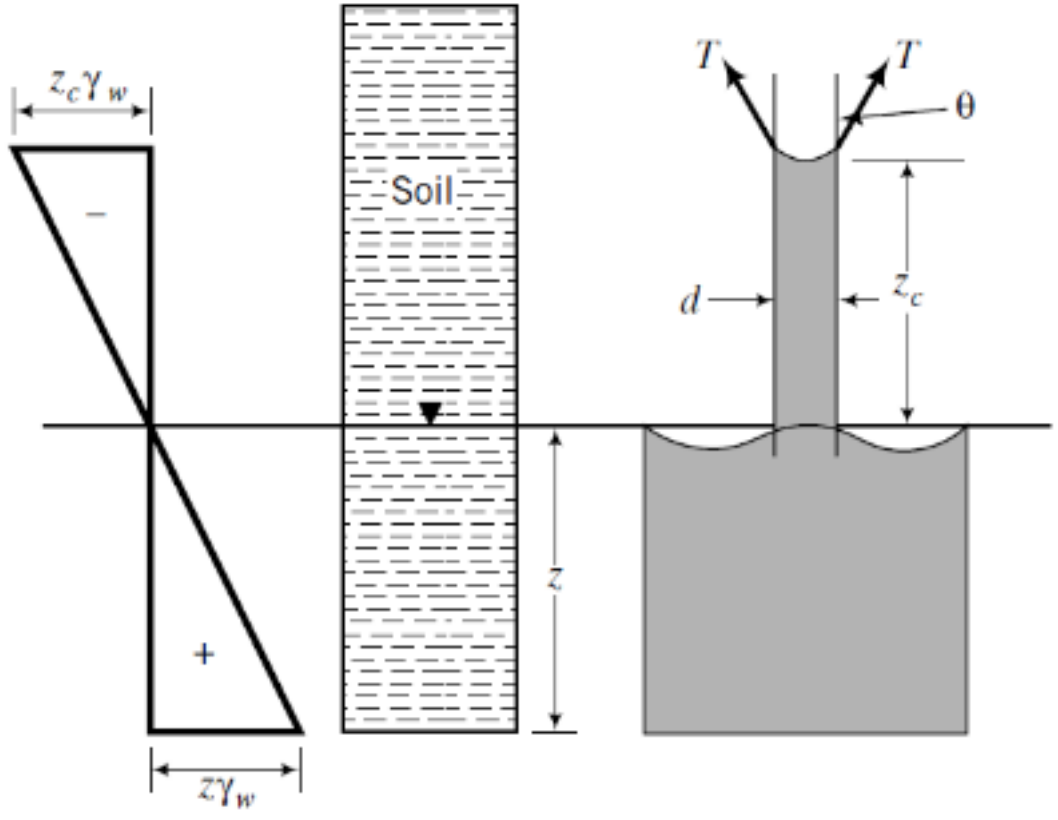
NB. This equation is applicable to all saturated soils.

Geostatic Stress cntd

Effect of Capillary

The porewater pressure due to capillarity is negative (suction), and is a function of the size of the soil pores and the water content.

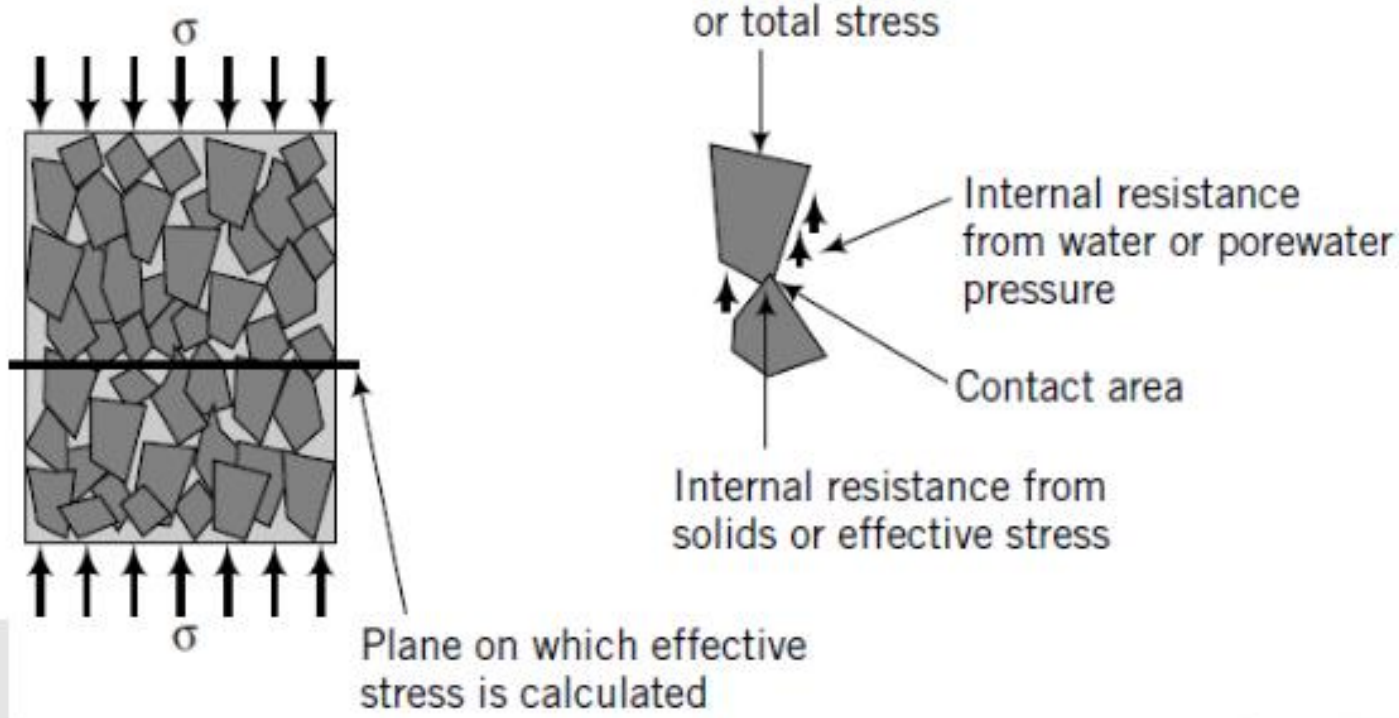
At the groundwater level, the porewater pressure is zero and decreases (becomes negative) as you move up the capillary zone.



Geostatic Stress cntd

The effective stress increases because the porewater pressure is negative.

For example, for the capillary zone, zc, the porewater pressure at the top is $-z_c\gamma_w$ and the effective stress is $\sigma' = \sigma - (-z_c\gamma_w) = \sigma + z_c\gamma_w$



Geostatic Stress cntd

Effect of Seepage

As water flows through soil it exerts a frictional drag on the soil particles, resulting in head losses.

The frictional drag is called seepage force in soil mechanics.

It is often convenient to define seepage as the seepage force per unit volume (it has units similar to unit weight), which we will denote by j_s.

If the head loss over a flow distance, L, is Dh, the seepage force is

$$j_s = \frac{\Delta h \gamma_w}{L} = i\gamma_w$$

Geostatic Stress cntd

If seepage occurs downward, then the seepage stresses are in the same direction as the gravitational effective stresses.

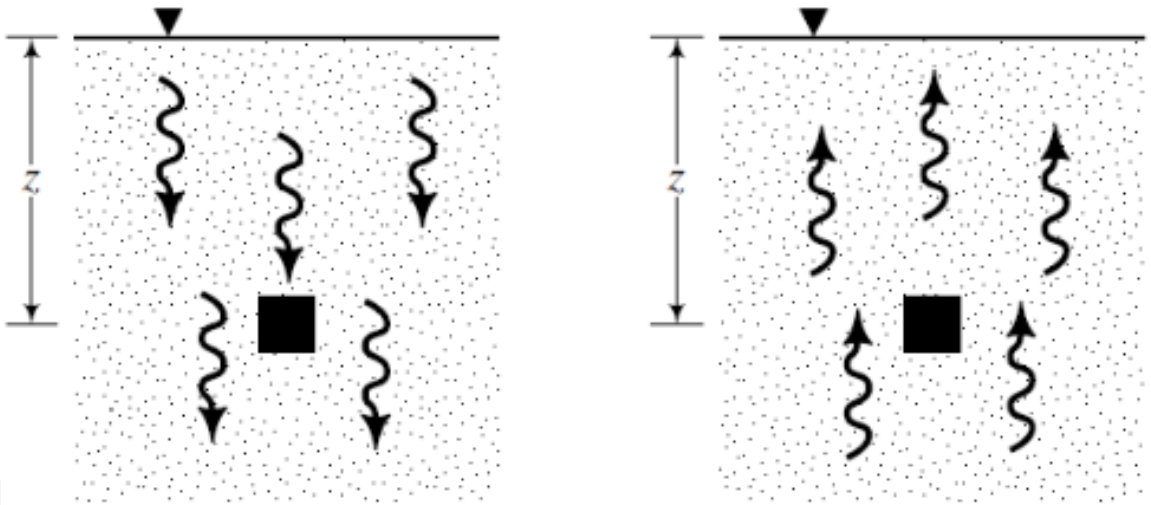
From static equilibrium, the resultant vertical effective stress is

$$\sigma' = \gamma' z + iz\gamma_w = \gamma' z + j_s z$$

If seepage occurs upward, then the seepage stresses are in the opposite direction to the gravitational effective stresses.

From static equilibrium, the resultant vertical effective stress is

$$\sigma' = \gamma' z - iz\gamma_w = \gamma' z - j_s z$$

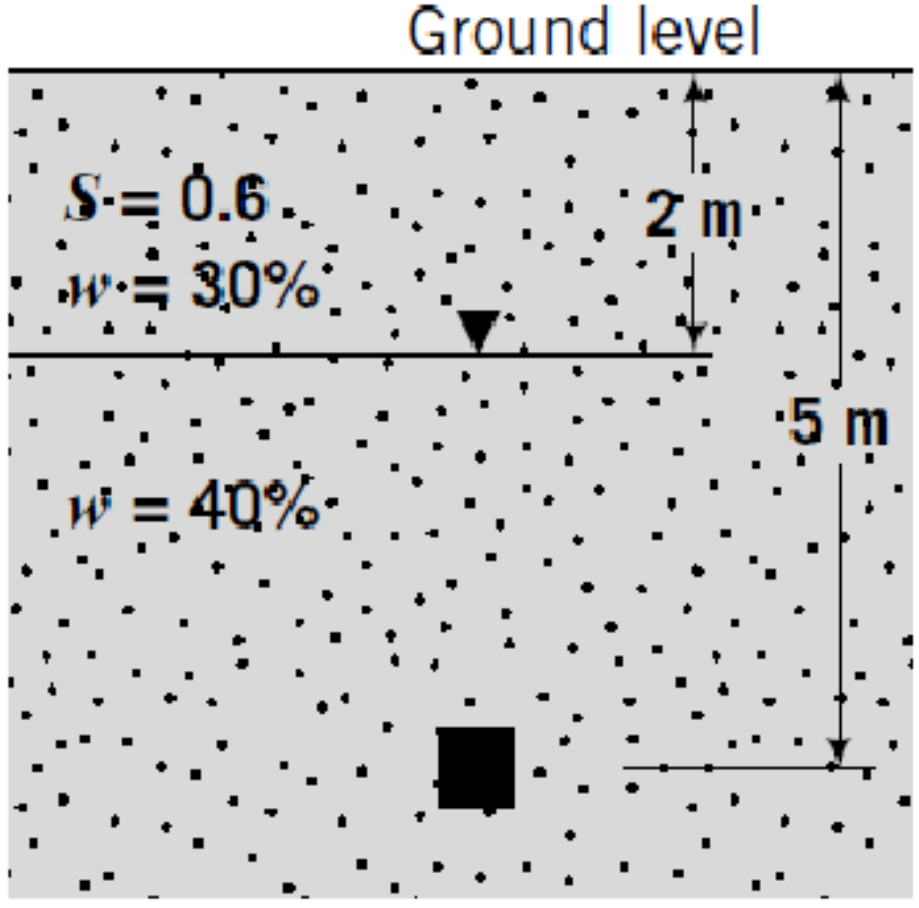


(*a*) Downward seepage (*b*) Upward seepage

Geostatic Stress cntd

EXERCISE 6.2.1 – EFFECTIVE STRESS

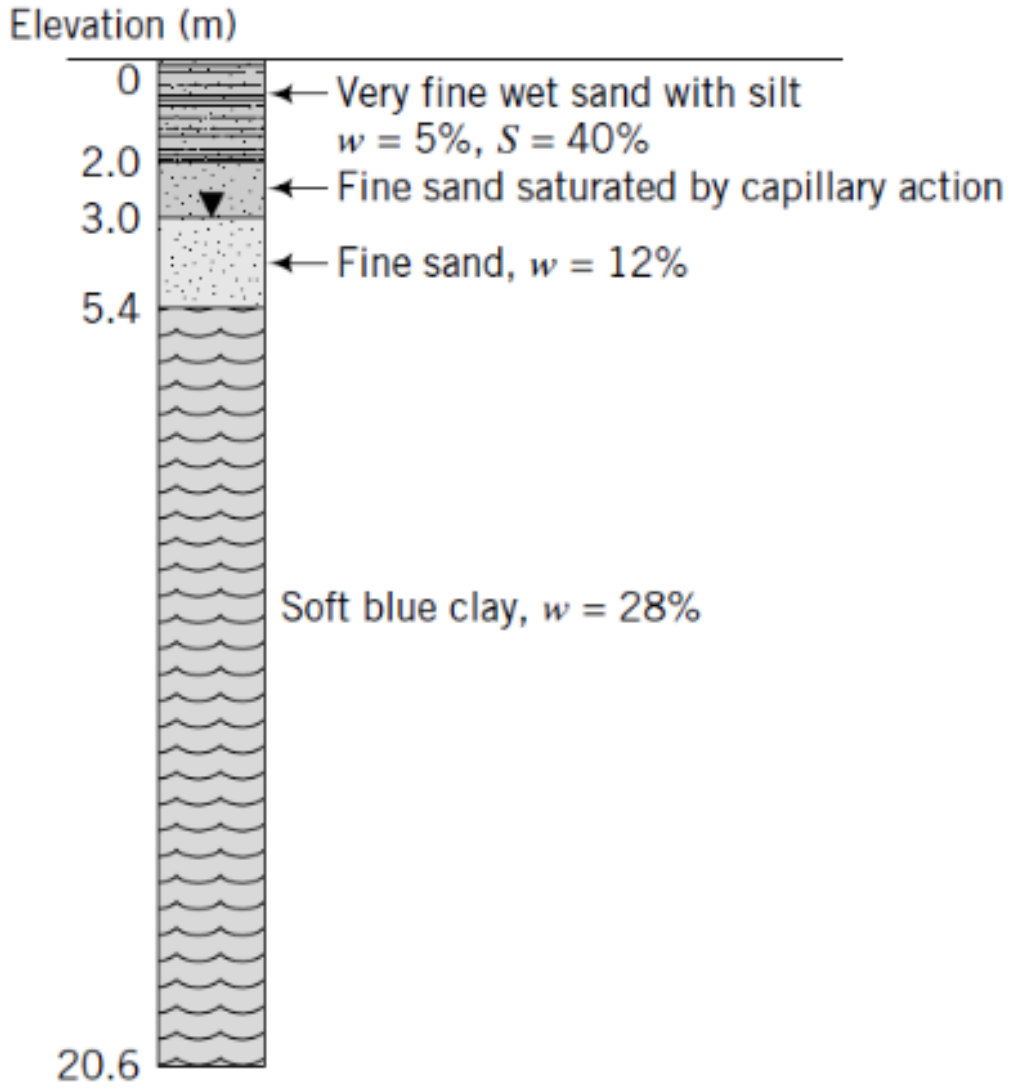
Calculate the effective stress for a soil element at depth 5 m in a uniform deposit of soil, as shown in figure below.



Geostatic Stress cntd

EXERCISE 6.2.2 – EFFECTIVE STRESS DISTRIBUTION

A borehole at a site reveals the soil profile shown in figure.
Plot the distribution of vertical total and effective stresses with depth.

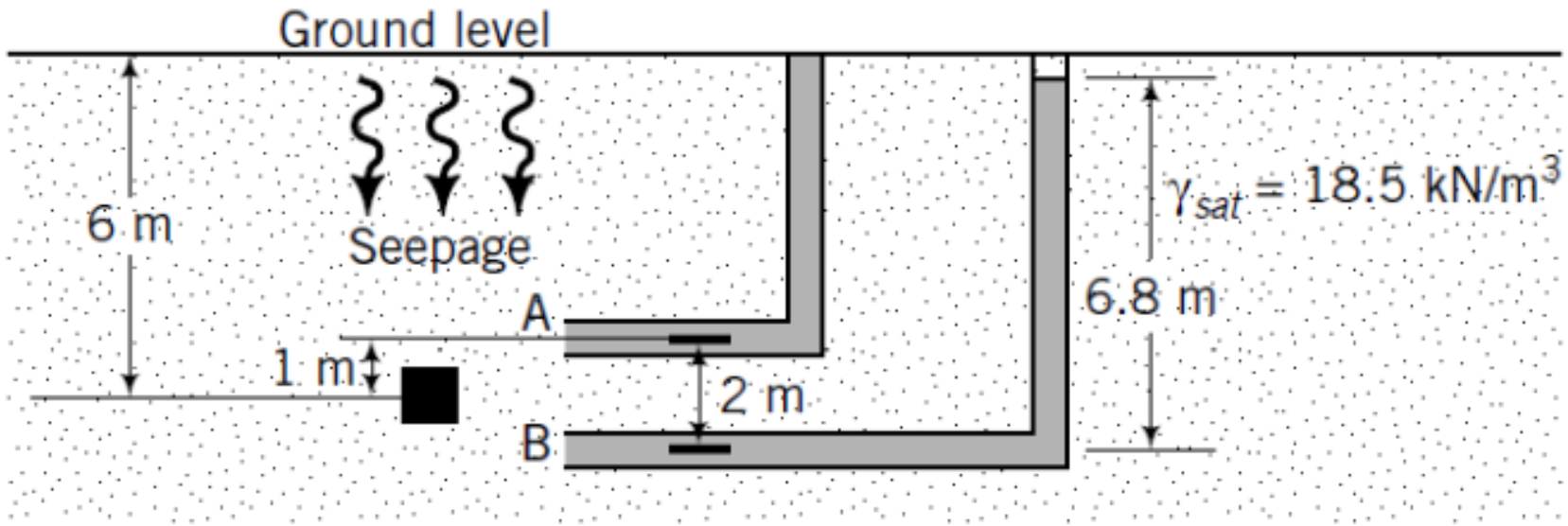


Geostatic Stress cntd

EXERCISE 6.2.3 – EFFECTS OF SEEPAGE

Water is seeping downward through a soil layer, as shown in figure. Two piezometers (A and B) located 2 m apart (vertically) showed a head loss of 0.2 m.

Calculate the resultant vertical effective stress for a soil element at a depth of 6 m.



Geostatic Stress cntd

Horizontal Components of Geostatic Stresses

The ratio of the horizontal principal effective stress to the vertical principal effective stress is called the lateral earth pressure coefficient at rest (Ko), that is,

$$K_o = \frac{\sigma_3'}{\sigma_1'}$$

K_o applies only to effective principal, not total principal, stresses.

To find the lateral total stress, one must add the porewater pressure.

NB. The porewater pressure is hydrostatic and, at any given depth, the porewater pressures in all directions are equal.

Geostatic Stress cntd

EXERCISE 6.2.4 – Horizontal Geostatic Stresses

Calculate the horizontal effective stress and the horizontal total stress for the soil element at 5 m in Exercise 6.2.1 if Ko = 0.5.

Geostatic Stress cntd

Porewater Pressure Distribution in Flow Regime

The porewater pressure at any point *j* within the flow domain (flownet) is calculated as follows:

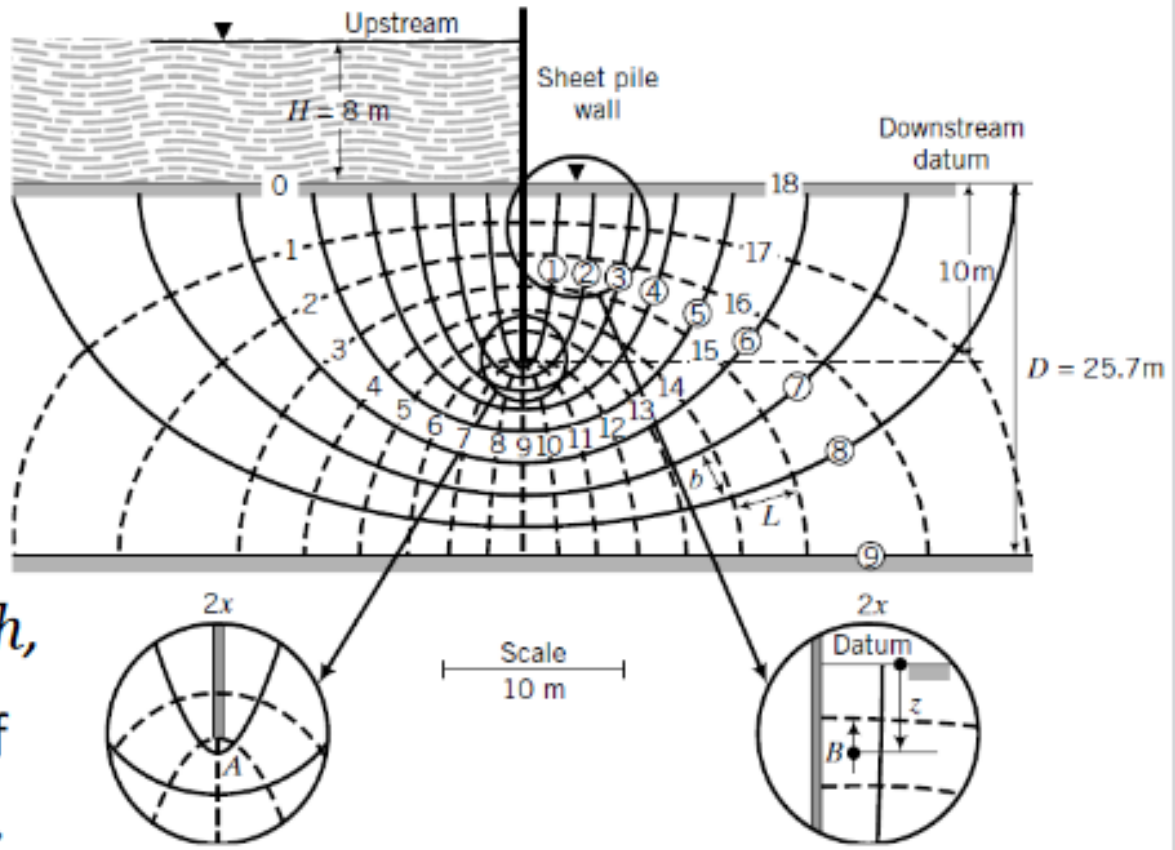
1. Select a datum.

[Downstream water level is chosen as the datum in the figure here]

2. Determine the total head at *j*: $H_j = \Delta H - (N_d)_j \Delta h$,

where $(N_d)_j$ is the number of equipotential drops at point *j*;

$(N_d)_j$ can be fractional. For example, at *B*, $H_B = \Delta H - 16.5\Delta h$



Geostatic Stress cntd

3. Subtract the elevation head at point *j* from the total head H_j to get the pressure head.

For point *B* , the elevation head h_z is -*z* (point *B* is below the datum). The pressure head is then $(h_p)_j = \Delta H - (N_d)_j \Delta h - h_z$

4. The porewater pressure is $u_j = (h_p)_j \gamma_w$

The uplift force per unit length (length is normal to the *XZ* plane) is found by calculating the porewater pressure at discrete points along the base and then finding the area under the porewater pressure distribution diagram, that is,

$$P_w = \sum_{j=1}^{n} u_j \, \Delta x_j$$

Uplift Forces under Dams

The uplift force per unit length (length is normal to the *XZ* plane) is found by calculating the porewater pressure at discrete points along the base and then finding the area under the porewater pressure distribution diagram, that is,

$$P_w = \sum_{j=1}^{n} u_j \, \Delta x_j$$

where P_w is the uplift force per unit length, u_j is the average porewater pressure over an interval Δx_j, and *n* is the number of intervals.

It is convenient to use Simpson's rule to calculate P_w :

$$P_w = \frac{\Delta x}{3}\left(u_1 + u_n + 2\sum_{i=3}^{n} u_i + 4\sum_{i=2}^{n} u_i\right)$$

3. Additional Stress



- Infinite Loads
  - Surcharge Load
- Finite Loads
  - Point Load
  - Strip Load
    - Embankment Loads
  - Circular Load
  - Rectangular Load
    - Approximate Methods
  - Irregularly Shaped Areas

Additional Stress

Introduction

- The determination of the stress distributions created by various applied loads has occupied researchers for many years.
- The basic assumption used in all their analyses is that the soil mass acts as a continuous, homogeneous and elastic medium.
- The assumption of elasticity obviously introduces errors but it leads to stress values that are of the right order and are suitable for most routine design work.

Additional Stress

- In most foundation problems, however, it is only necessary to be acquainted with the increase in vertical stresses (for settlement analysis) and the increase in shear stresses (for shear strength analysis)
- Surface loads are divided into two general classes, finite and infinite. However, these are qualitative classes and are subject to interpretation.
- Examples of finite loads are point loads, circular loads, and rectangular loads. Examples of infinite loads are fills and surcharges.

Additional Stress

- The increases in soil stresses from surface loads are total stresses. These increases in stresses are resisted initially by both the porewater and the soil particles.
- Most soils exist in layers with finite thicknesses. The solution based on a semi-infinite soil mass will not be accurate for these layered soils.

Additional Stress

- ***Elastic Half Space***: the space is bounded by a horizontal plane and consists of a homogeneous, isotropic, and elastic material extending endlessly downwards and horizontally below the horizontal plane.
- ***Elastic Full Space***: consists of a homogeneous, isotropic, and elastic material but the application of the load is isnside the space; typically used for structures buried in the ground (eg. Tunnels)
- ***Elastic Quarter Space***: has an important application in determining stresses at the back of retaining wall due to surface loads.

Additional Stress

- The distribution of stresses within a soil from applied surface loads or stresses is determined by assuming that the soil is a semi-infinite, homogeneous, linear, isotropic, elastic material.
- A semi-infinite mass is bounded on one side and extends infinitely in all other directions; this is also called an “elastic half-space.”
- Because of the assumption of a linear elastic soil mass, we can use the principle of superposition. That is, the stress increase at a given point in a soil mass in a certain direction from different loads can be added together.

Additional Stress cntd

Stresses induced by uniform surface surcharge

- Road, railway, fill, ice, etc
- In the case of a uniform surcharge spread over a large area it can be assumed that the increase in vertical stress resulting from the surcharge is constant throughout the soil.
- Here, the vertical total stress at depth z, is given by

$$\sigma_z = \gamma z + q$$

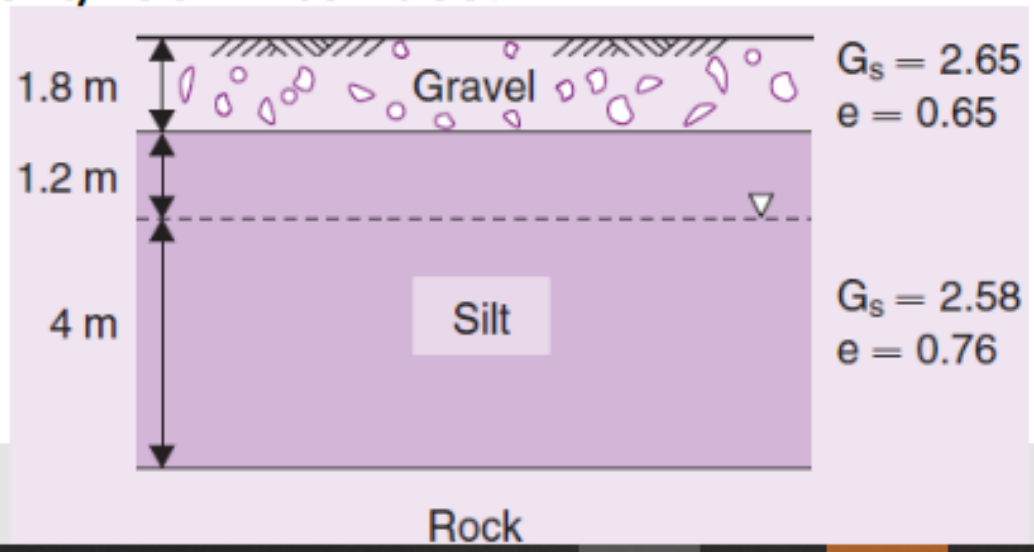
where q is the magnitude of the surcharge (kPa).

Additional Stress cntd

EXERCISE 6.3.1 – SURCHARGE LOAD ON EFFECTIVE STRESS

Details of the subsoil conditions at a site are shown in figure below together with details of the soil properties. The ground surface is subjected to a uniform loading of 60 kPa and the groundwater level is 1.2 m below the upper surface of the silt. It can be assumed that the gravel has a degree of saturation of 50% and that the silt layer is fully saturated.

Determine the vertical effective stress acting at a point 1 m above the silt/rock interface.



Additional Stress cntd

Stresses induced by point load

- Electric line pole, light stand, column, etc
- Boussinesq (1885) evolved equations that can be used to determine the six stress components that act at a point in a semi-infinite elastic medium due to the action of a vertical point load applied on the horizontal surface of the medium.
- His expression for the increase in vertical stress is:

$$\Delta\sigma_z = \frac{3Q}{2\pi z^2 \left[1 + \left(\frac{r}{z}\right)^2\right]^{5/2}}$$

Additional Stress cntd

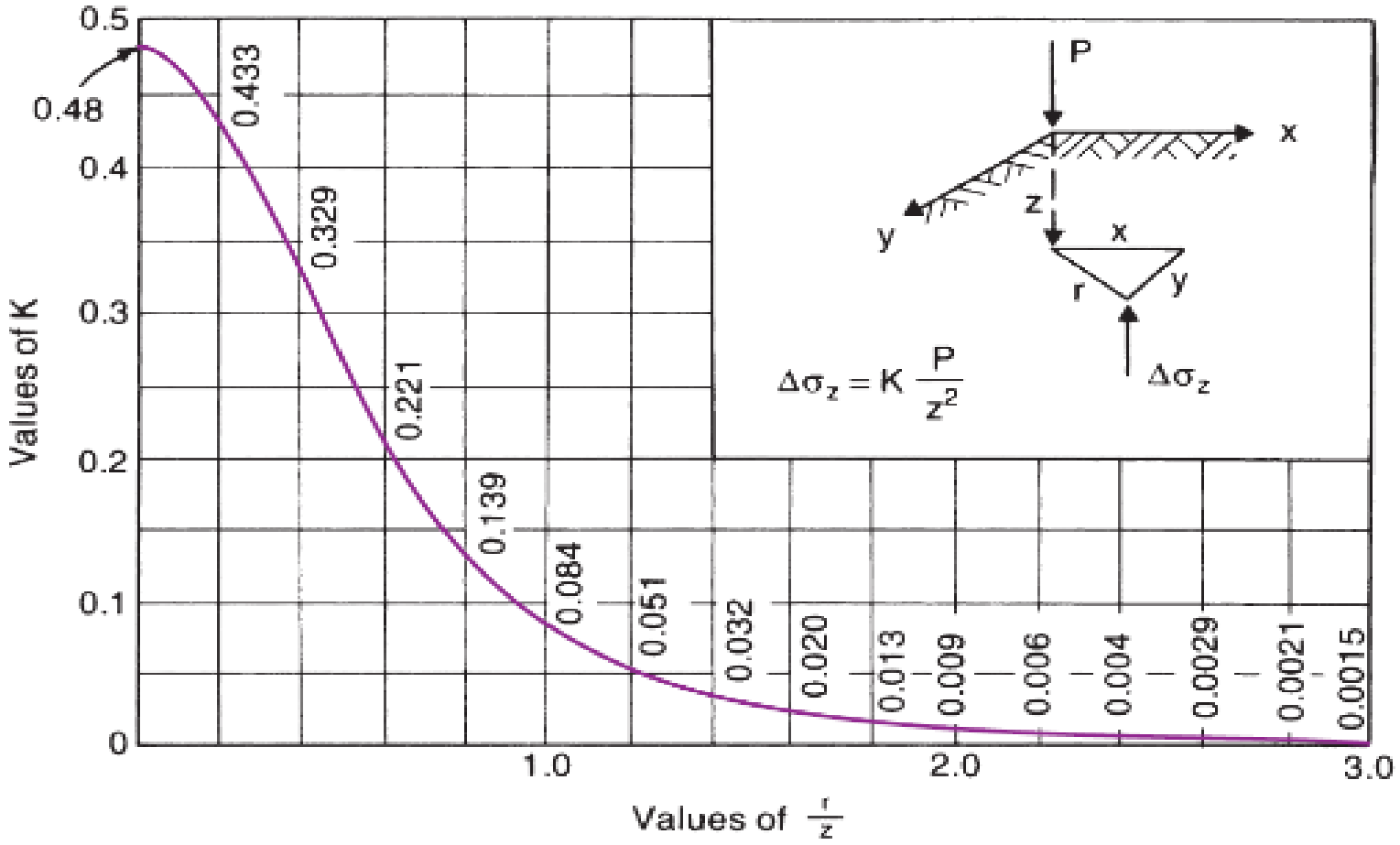
Stresses induced by point load

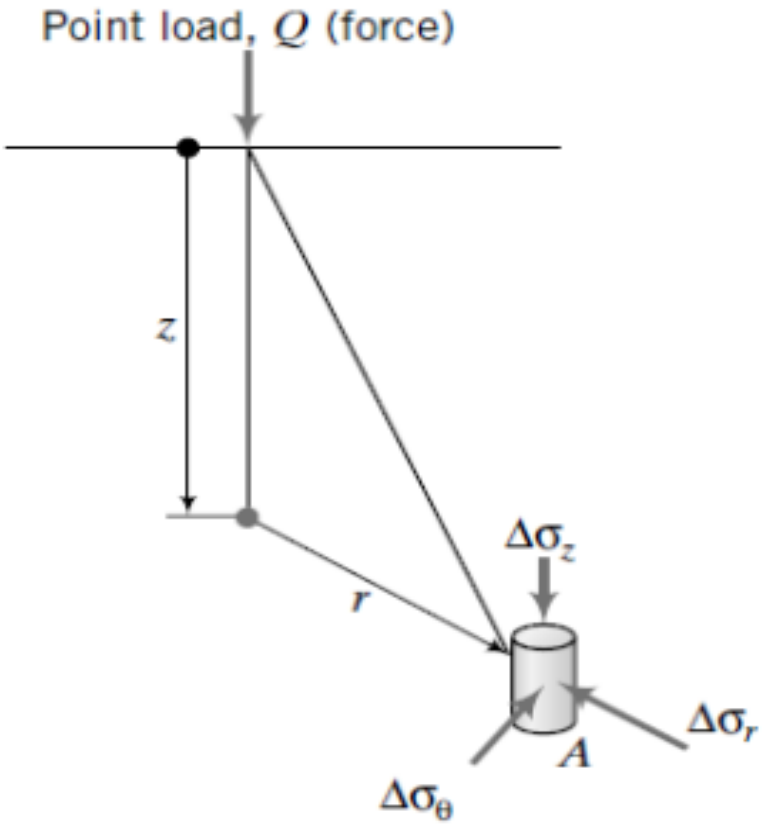
The expression has been simplified to:

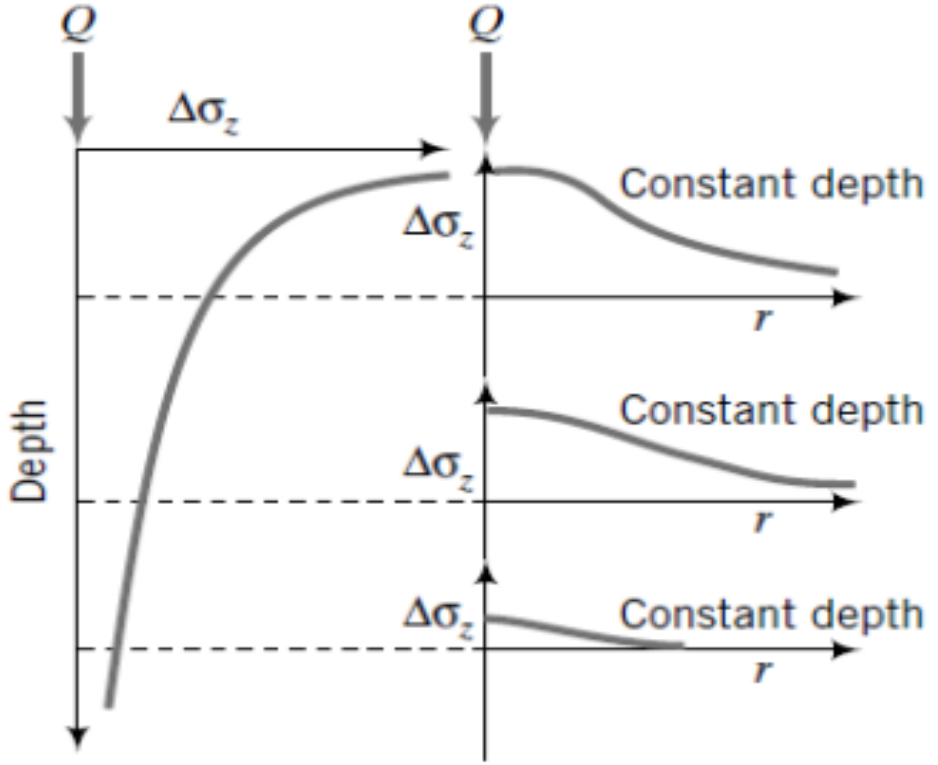
$$\Delta\sigma_z = \frac{Q}{z^2} K$$

where K is an influence factor.

$$K = \frac{3}{2\pi} \frac{1}{\left[1 + \left(\frac{r}{z}\right)^2\right]^{5/2}}$$







$$\Delta\sigma_r = \frac{Q}{2\pi}\left(\frac{3r^2z}{(r^2+z^2)^{5/2}} - \frac{1-2\nu}{r^2+z^2+z(r^2+z^2)^{1/2}}\right)$$

$$\Delta\sigma_\theta = \frac{Q}{2\pi}(1-2\nu)\left(\frac{z}{(r^2+z^2)^{3/2}} - \frac{1}{r^2+z^2+z(r^2+z^2)^{1/2}}\right)$$

$$\Delta\tau_{rz} = \frac{3Q}{2\pi}\left[\frac{rz^2}{(r^2+z^2)^{5/2}}\right]$$

Additional Stress cntd

EXERCISE 6.3.2 - Point load

a) A concentrated load of 400 kN acts on the surface of a soil.

Determine the vertical stress increments at points directly beneath the load to a depth of 10 m.

b) A pole carries a vertical load of 200 kN. Determine the vertical total stress increase at a depth 5 m (a) directly below the pole and (b) at a radial distance of 2 m.

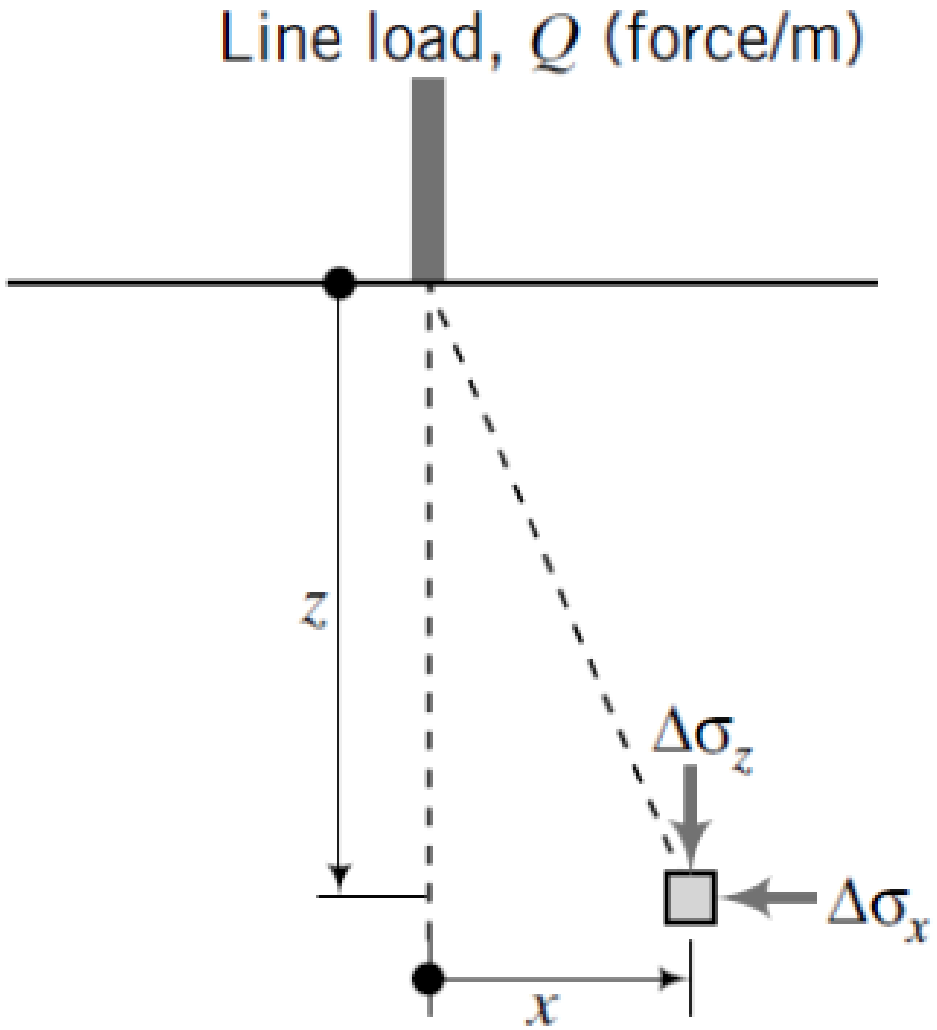
Stresses induced by uniform line load

- Rack or rail loading, load from a long brick wall.

$$\Delta\sigma_z = \frac{2Qz^3}{\pi(x^2+z^2)^2}$$

$$\Delta\sigma_x = \frac{2Qx^2z}{\pi(x^2+z^2)^2}$$

$$\Delta\tau_{zx} = \frac{2Qxz^2}{\pi(x^2+z^2)^2}$$



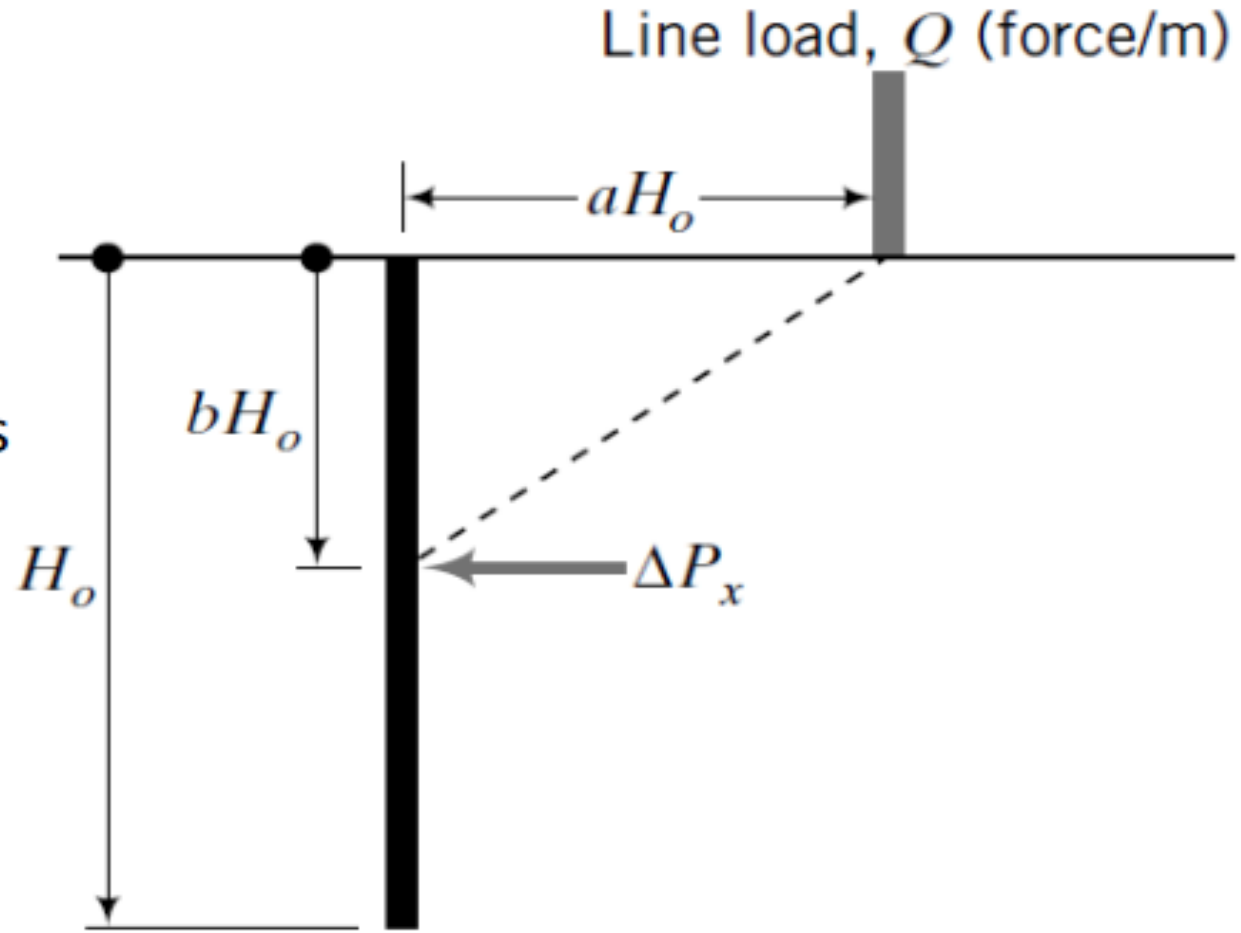
Additional Stress cntd

Line Load Near a Buried Earth-Retaining Structure

$$\Delta\sigma_x = \frac{2Qa^2b}{\pi H_o(a^2+b^2)^2}$$

The increase in lateral force is

$$\Delta P_x = \frac{2Q}{\pi\,(a^2+1)^2}$$



Stresses induced by strip load

- Strip foundation - a structure of finite width and infinite length on a soil surface.
- Two types of strip loads are common in geotechnical engineering.
- One is a load that imposes a uniform stress on the soil, for example, the middle section of a long embankment.
- The other is a load that induces a triangular stress distribution over an area of width B . For example the stress under the side of an embankment.

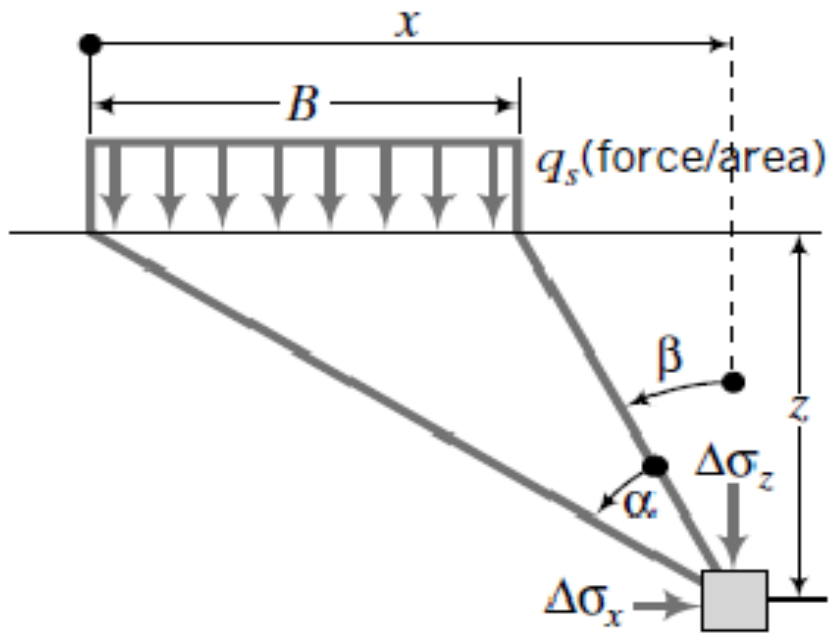
Additional Stress

Uniform surface stress

$$\Delta\sigma_z = \frac{q_s}{\pi}[\alpha + \sin\alpha\cos(\alpha + 2\beta)]$$

$$\Delta\sigma_x = \frac{q_s}{\pi}[\alpha - \sin\alpha\cos(\alpha + 2\beta)]$$

$$\Delta\tau_{zx} = \frac{q_s}{\pi}[\sin\alpha\sin(\alpha + 2\beta)]$$

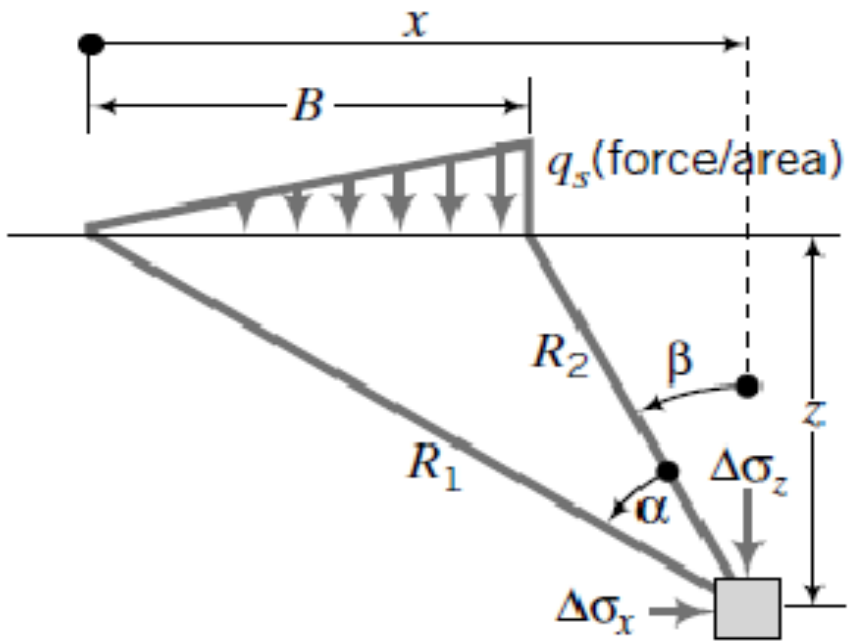


Linear varying surface stress

$$\Delta\sigma_z = \frac{q_s}{\pi}\left[\frac{x}{B}\alpha - \frac{1}{2}\sin 2\beta\right]$$

$$\Delta\sigma_x = \frac{q_s}{\pi}\left[\frac{x}{B}\alpha - \frac{z}{B}\ln\frac{R_1^2}{R_2^2} + \frac{1}{2}\sin 2\beta\right]$$

$$\Delta\tau_{zx} = \frac{q_s}{2\pi}\left[1 + \cos 2\beta - 2\frac{z}{B}\alpha\right]$$



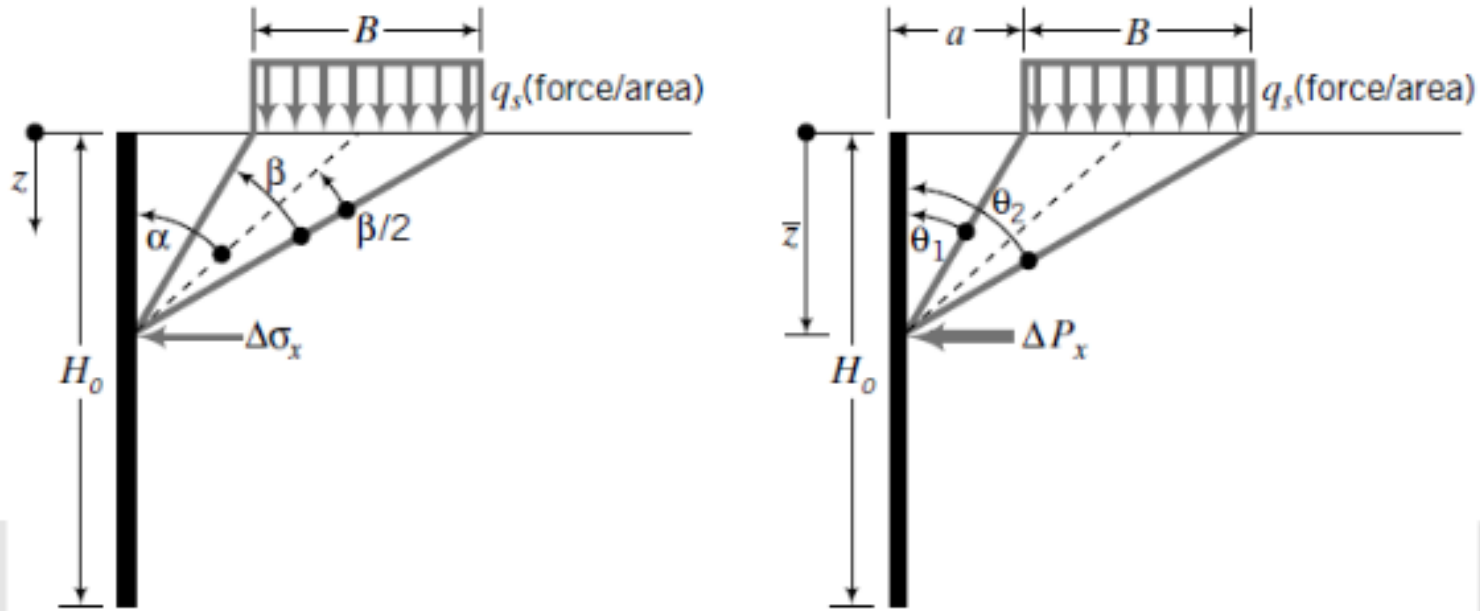
Uniform surface stress near retaining wall

$$\Delta\sigma_x = \frac{2q_s}{\pi}[\beta - \sin\beta\cos 2\alpha] \;\&\; \Delta P_x = \frac{q_s}{90}[H_o(\theta_2 - \theta_1)]$$

$$\bar{z} = \frac{H_o^2(\theta_2 - \theta_1) - (R_2 - R_1) + 57.3BH_o}{2H_o(\theta_2 - \theta_1)}$$

where $\theta_1 = \tan^{-1}\left(\frac{a}{H_o}\right), \theta_2 = \tan^{-1}\left(\frac{a+B}{H_o}\right)$

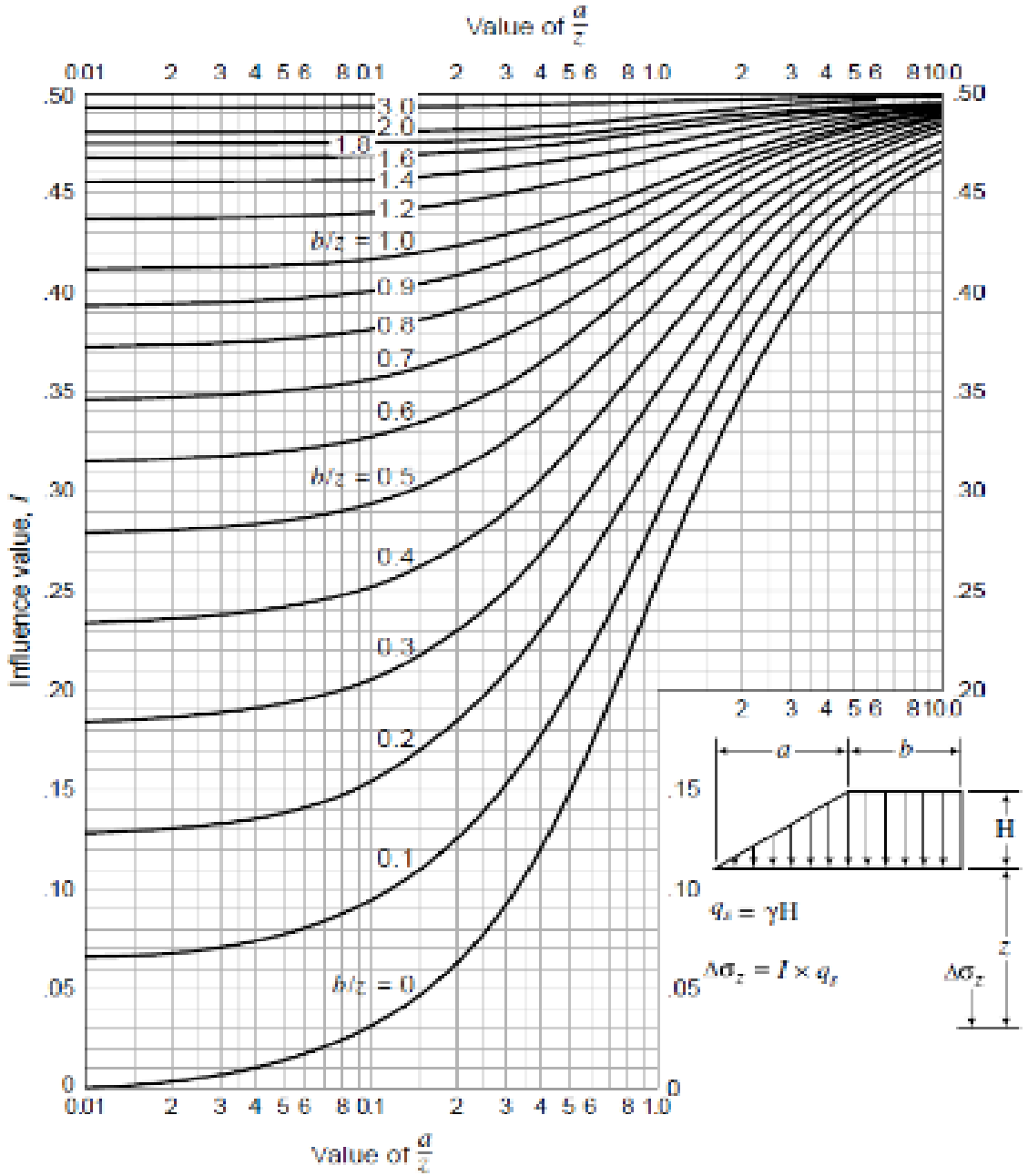
$R_1 = (a+B)^2(90-\theta_2), and\ R_2 = a^2(90-\theta_1)$



Stresses induced by embankment load

Loads from an embankment can be considered as a combination of a rectangle and two triangular strip loads.

- $\Delta\sigma_z = 2I'q_o$



Additional Stress cntd

Stress induced due to a uniformly loaded circular area

- circular foundation of an oil or water tank.

$$\Delta\sigma_z = q_s\left[1-\left(\frac{1}{1+(r_o/z)^2}\right)^{3/2}\right] = q_s I_c$$

$$\Delta\sigma_r = \Delta\sigma_\theta = \frac{q_s}{2}\left[(1+2v)-\frac{4(1+v)}{[1+(r_o/z)^2]^{1/2}}+\frac{1}{[1+(r_o/z)^2]^{3/2}}\right]$$

