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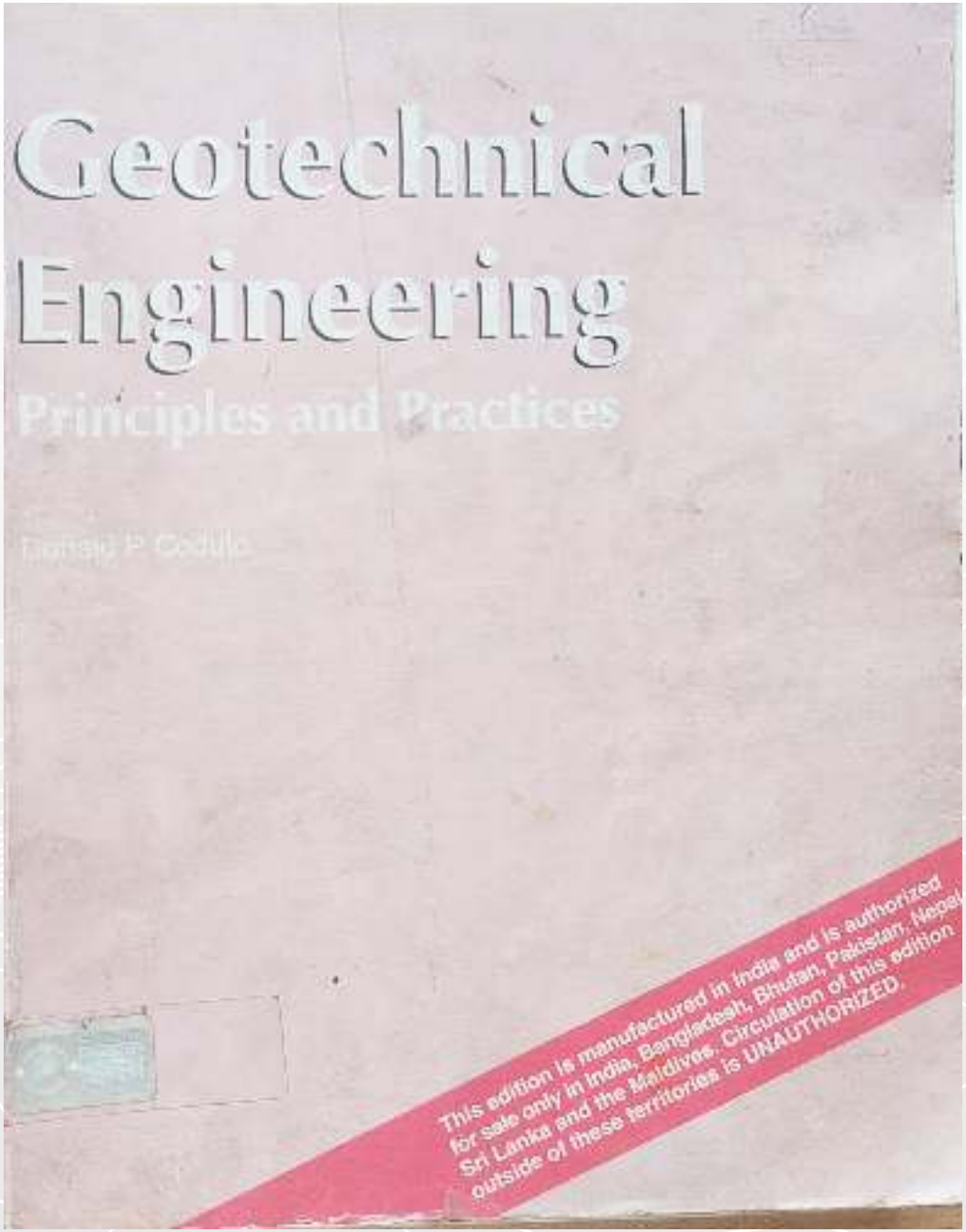
Principles of Soil Mechanics

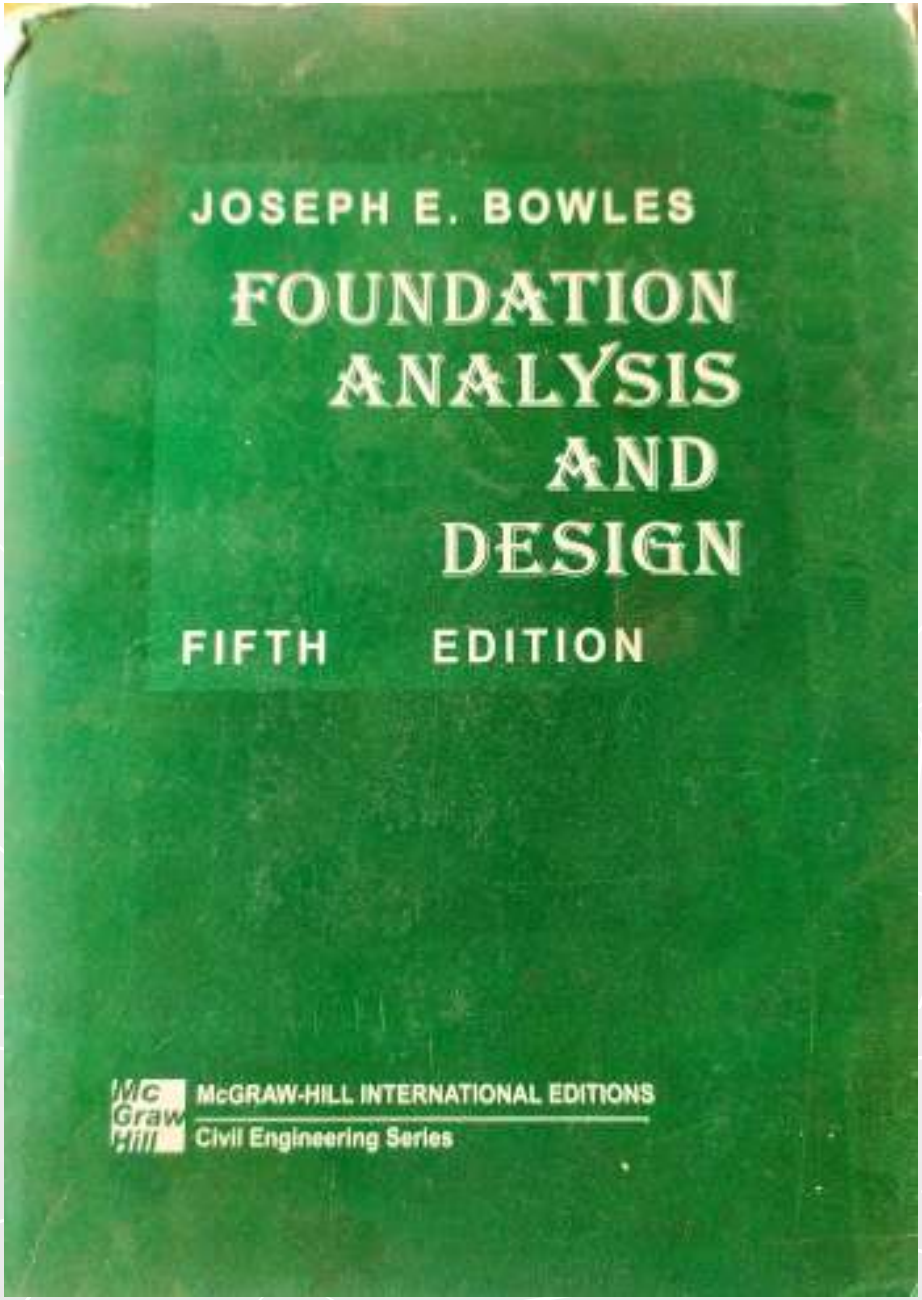
Syllabus

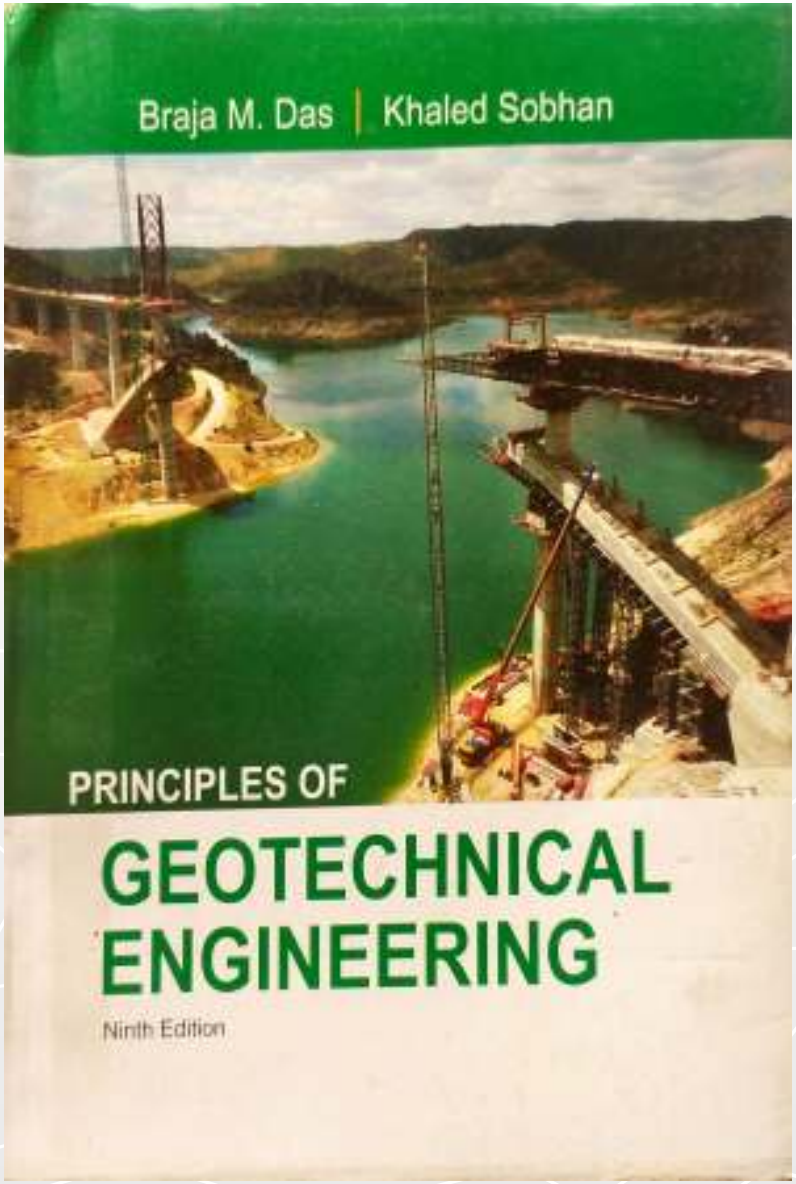
- **Scope** of Geotechnical Engineering
- Soil Mechanics and Foundation Engineering
- Soil Composition
- Soil **Compaction**
- Principles of total and effective **stresses**
- **Permeability** and **Seepage**
- **Compressibility** and **settlement** behavior of soil.

Reference Books

- Geotechnical Engineering, Principles and Practices, by **Donald P. Coduto**.
- Principles of Geotechnical Engineering, by **Braja M. Das, Khaled Sobhan.**
- Introduction to Soil Mechanics, by Brajs M Das.
- Advanced Soil Mechanics, by B M Das
- Physical and Geotechnical properties of Soils, 2nd Edition by **Joseph E. Vowels**
- A Text Book of Soil Mechanics & Foundation Engineering, 4th Edition, by VNS Murthy.
- Soil Mechanics and Foundation Engineering, by KR Arora.


Geotechnical
Engineering
Principles and Practices
Donald P Coduto
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JOSEPH E. BOWLES
FOUNDATION ANALYSIS AND DESIGN
FIFTH EDITION
McGRAW-HILL INTERNATIONAL EDITIONS
Civil Engineering Series


Braja M. Das | Khaled Sobhan
PRINCIPLES OF
GEOTECHNICAL
ENGINEERING
Ninth Edition

What we are going to discuss?

- **Soil & Rock and Soil as materials.**
- Soil Mechanics and its **need to study**.
- **Foundation Engineering and its scope**.
- Geotechnical Engineering, its **importance** and **concerned issues**.
- **Differences** between Soils & Rocks and other Civil Engineering Materials.
- Soil and Rock sizes.

What is soil?

What is Soil?

Soils are **aggregates of minerals/solid particles** and together with **air** and /or **water** in the void spaces, they form of **Three-Phase** system.

Aggregates- granular particles

Minerals- a solid element or compound that occurs naturally.



Soil as Materials and Rocks

Soil as materials-

Geologist:

- **Loose surface of the earth** produced by the gradual weathering of rock to produce sediments as distinguished from solid bedrock.

Agriculturist:

- **A** dynamic **natural body** on the surface of the earth in which **plants grow**, composed of **mineral** and **organic materials** and living forms.

Engineers:

- **Un-cemented aggregate** of mineral grains (**solid particles**) and **decayed organic matters** with **liquid and gas** in the empty spaces between the solid particles.
- **Support the foundation**.

Rocks: A rock is any **naturally occurring solid mass** or **consolidated aggregates of minerals or mineraloid.**

Mineraloids - A mineraloid is a naturally occurring mineral -like substance that **does not demonstrate crystallinity**.

Crystallinity - refers to the **degree of structural order** in a solid.



Soil Mechanics

Soil Mechanics

Definition:

Branch of Geotechnical Engineering that deals with the **engineering properties** of soils and its **behavior** under stress and strains.

Engineering Properties of soils comprises of:

- **Physical properties** (color, texture, structure etc.)
- **Index properties** (Sp. Gr, water content, particle size distribution etc.)
- **Shear strength parameters** (cohesion, angle of internal friction, vertical normal stress)
- **Permeability characteristics** (ability of soil to allow water to pass through it)
- **Consolidation** parameters.

Why do you need to Study Soil Mechanics?



Why do you need to study Soil Mechanics?

- Easily available **construction materials**.

  -Embankments, dams, road way and other similar structures.

- Safely **carry the loads** of any civil engineering structure.

Foundation Engineering

Foundation Engineering

Definition:

Branch of geotechnical engineering which **applies soil mechanics, structural engineering,** and **project serviceability requirements for design, construction, and maintenance** of foundations.





Scope of Foundation Engineering

Scope of Foundation Engineering:

- **Obtaining information** about the **superstructure** involved and the **loads** acting there on.
- **Collecting subsurface soil**.
- **Investigating** the **engineering properties** of the soil.
- Determine the **geotechnical input** for foundation design.
- **Studying the types of foundations required** in relation to the **load or structure**, **subsurface soil condition**.
- **Design** the foundation

Geotechnical Engineering

Geotechnical Engineering

Definition:

- Branch of **civil engineering** that deals with **soils**, **rocks** and **underground water**; and **their relation** to design, construction and operations of engineering projects.

Importance of studying of Geotechnical Engineering

- Almost all of the **civil engineering project** must be **supported by the ground**, hence all of us require at **least some knowledge** on geotechnical engineering.

Issues to be Addressed by Geotechnical Engineers

Issues to be addressed by the Geotechnical Engineers: (**SSG –IEFR-TEC**)

- What are the **soils and rocks** in the **subsurface** at the construction site?
- Can the soils and rocks beneath the construction site **can safely support** the projects?
- What **groundwater conditions** currently exist, how might they **change** in the future, and what **impact** do they have on the projects?
- What will be the **impact of any planned excavation**, **grading** and **filling**.
- Are the **natural or proposed earth slopes** are stable? If not, what must to be done to stabilize.

Issues to be addressed (cont..)

- What **kinds of foundations are necessary** to support the planned structures, and how **should to design them**?
- If the **project requires retaining walls**, what **kind** would be best and how to **design** them?
- If the project **requires a tunnel** or **underground opening**, how should it be excavated?
- How will the **site respond to potential earthquake**?
- Has the **ground become contaminated** with chemical or biological materials? Do they these materials represent **health or safety hazard**? If so, what must be done to rectify them?

Issues to be addressed by Geotechnical Engineers (summary): (SSG –IEFR-TEC)

- **Subsurface** conditions
- **Safely support** the project
- **Groundwater** conditions
- **Impact** planned soil disturbance
- **Earth slopes** stable
- **Foundation** Type
- **Retaining wall**
- **Tunnel** or opening in the ground
- **Earthquake**
- **Contaminated** ground

How Soil Mechanics help to Design Foundations

How does soil mechanics help to design the Foundation of Structures/Buildings?

All structures/buildings have to **be built on soils**.

- **Soil mechanics lays down certain principles, theories and procedures** for the design of a safe and sound substructure.

- **Foundation engineering deals with the design of various types of substructures/foundations** under different soil and environmental condition.

- **During design**, designer has to make use of the **properties of soils**, **theories pertaining to the design** and **his own practical experience** to **adjust** the design to suit the field condition.

- **Designer** has to **deal with the natural soil deposits**, which perform the engineering function of supporting the foundation and the superstructure above it.

- Soil deposits in nature are **available in an extremely erratic manner**, **producing** thereby, an **infinite variety** of possible combination which would **affect the choice and design of foundation.**

- Foundation engineer must have the ability to **interpret the principles of soil mechanics** to suit the field conditions.

Difference between Soil & Rocks and Other Engineering Materials

Differences in composition between Soils & rocks and other civil engineering materials:

- Soils and rocks are **natural materials**, **not manufactured** ones; most of the others are **manufactured** materials.

- Soils & rocks specially soils are **particulate materials** that consists of individual particles; most of the others are **continuous mass**.

- Soils & rocks can contain all the **three phases of matter (ie solid, liquid and gas) simultaneously** and can be present in varying proportion, where other materials are **solid mass**.

- Soil and rocks are **natural materials**, **not manufactured** ones;

most of the other civil engineering materials are manufactured materials.

-**Engineering properties** **of soil vary from place to place** even across a single building site.

-we cannot **sort out soils and rocks** but **must accommodate** what is present on site.

- Soil is a **particulate materials** that consists of individual particles.

-It is **not a continuous mass**.

- Some **rock strata** specially sedimentary rocks can also be treated as particulates.

-Even rock that appear to be continuous mass virtually **always contain cracks and fissures** that divide it into blocks.

- Soil can contain all the **three phases of matter** **(ie Solid, Liquid and Gas) simultaneously** and can be present in varying proportion.

-Rocks can also contain all three phases, although the liquid and gas phases **may be confined to the fissures**.

Soils as Particulate materials - Explanation

- **Most of the engineering materials** consists of:

  -**Continuous mass** held together with molecular bond, and

  -**Mechanical properties** of such materials depend on their **chemical make up** and the **nature of these bond**.

Example: Reinforcement bar

-**Shear strength of steel** depend on the strength of their molecular bond,

-their **shear failure** requires breaking them off.



Soil is a particulate material that consists of individual particles assembled together.

Its **engineering properties** depend on:

-**largely on interaction** between these particles.

-only **secondarily** on their **internal properties**.

Example: Gravel, sand, silts

- When **soils fail in shear**, they do:

-because of the **particles begin to roll and slide past each others**.

-**not because of the particles beak**, **breakage of individual particles** is typically minimal.

- **Shear strength of soil** depend on the following factors:

  -**coefficient of friction** between particles and

  -**tightness** of the of packing ie **compaction**.

  -Cohesion

- **Clays also have particulate structure** but the **nature of the particles is quite different.**

  -**much more interaction** between the particles and the pore water.

  -**behavior is more complex** than that of other.

Particle Sizes

Soil Particle Size

Name of the Organization	Grain Size (mm)			
	Gravel	Sand	Silt	Clay
Massachusetts Institute of Technology (MIT)	> 2	2 to 0.06	0.06 to 0.002	<0.002
US Department of Agriculture (USDA)	> 2	2 to 0.05	0.05 to 0.002	<0.002
American Association of State Highway and Transportation Officials (AASHTO)	76.2 to 2	2 to 0.075	0.075 to 0.002	<0.002
Unified Soil Classification System (USCS) (US Army Corps of Engineers, US Bureau of Reclamation, and American Society for Testing and Materials (ASTM))	**76.2 to 4.75**	**4.75 to 0.075**	**Fines (i.e. silts and clay)** **<0.075**	

Sieve opening of US **Sieve No. 4 is 4.75mm**. Sieve No. 10, 2mm and **Sieve No. 200**, is 0.075mm







Boulder > 256mm

Cobbles **76.2-256mm**

Gravel 76.2 to 4.75mm

Rock Fragments



Sand 4.75 to 0.075mm



Fines <0.075mm

(0.075m -0.002mm & < 0.002mm)

Thanks

Soils Composition

Soils: Formed by **weathering** of rocks.

Rock: Defined as naturally occurring consolidated aggregates of geological material, called minerals.

Physical Properties of Soils: Primarily dictated by the minerals that constitute the soil particles.

Rock Cycle & Origin of Soils

Soils:

- Mineral grains that form from the solid phase of a soil aggregate are the **product** of the rock weathering.
- **Size** of the individual mineral grains varies over a wide range.
- Many of the physical properties are determined by the **size, shape and chemical composition** of the mineral grains.

Types of Rocks

On the basis of the **mode of origin**:

- **Igneous Rocks-** by cooling magma/lava.
- **Sedimentary Rock-** by lithification of sediments
- **Metamorphic Rock- by metamorphism**

Weathering

Process of breaking down of the rocks by mechanical and chemical process into smaller pieces.

- **Mechanical Weathering (disintegration)**: Caused by the expansion and contraction of rocks from the continuous gain and loss of heat, which result in ultimate disintegration. Mechanical erosion due to ocean wave and wind at Yehliu, Taiwan.

- **Chemical weathering (decomposition)**: Original rock minerals are transformed into new minerals by chemical reaction. Honey comb structure



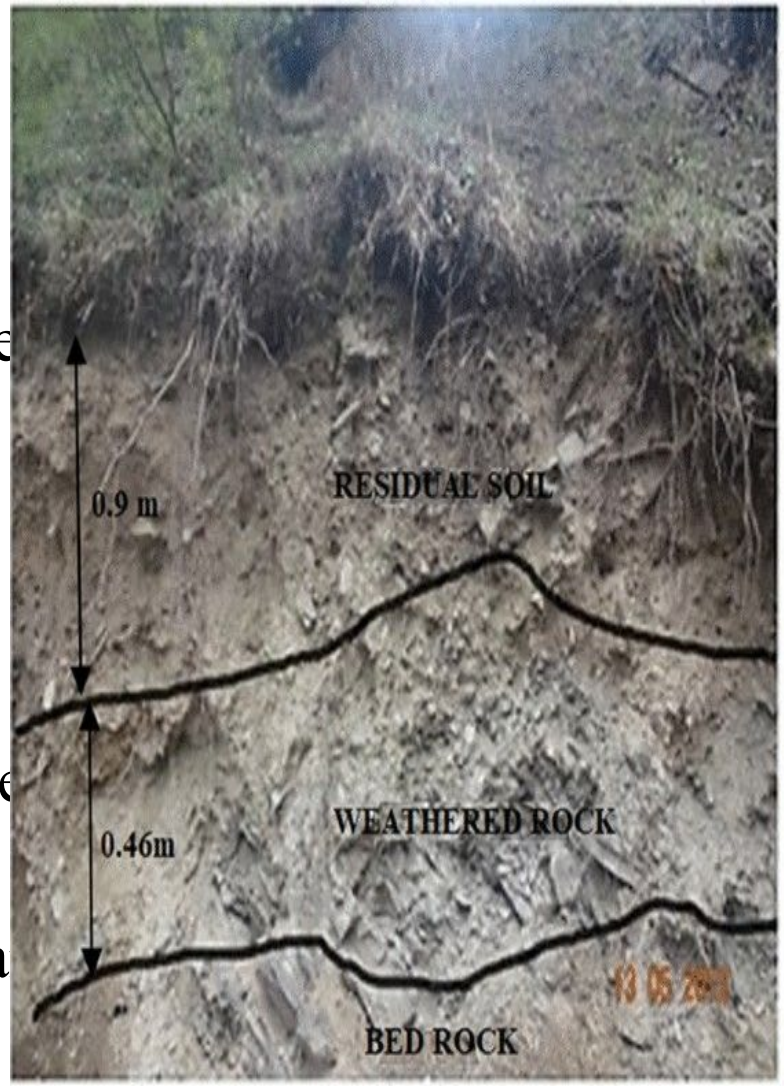


Transportation of Weathering Product

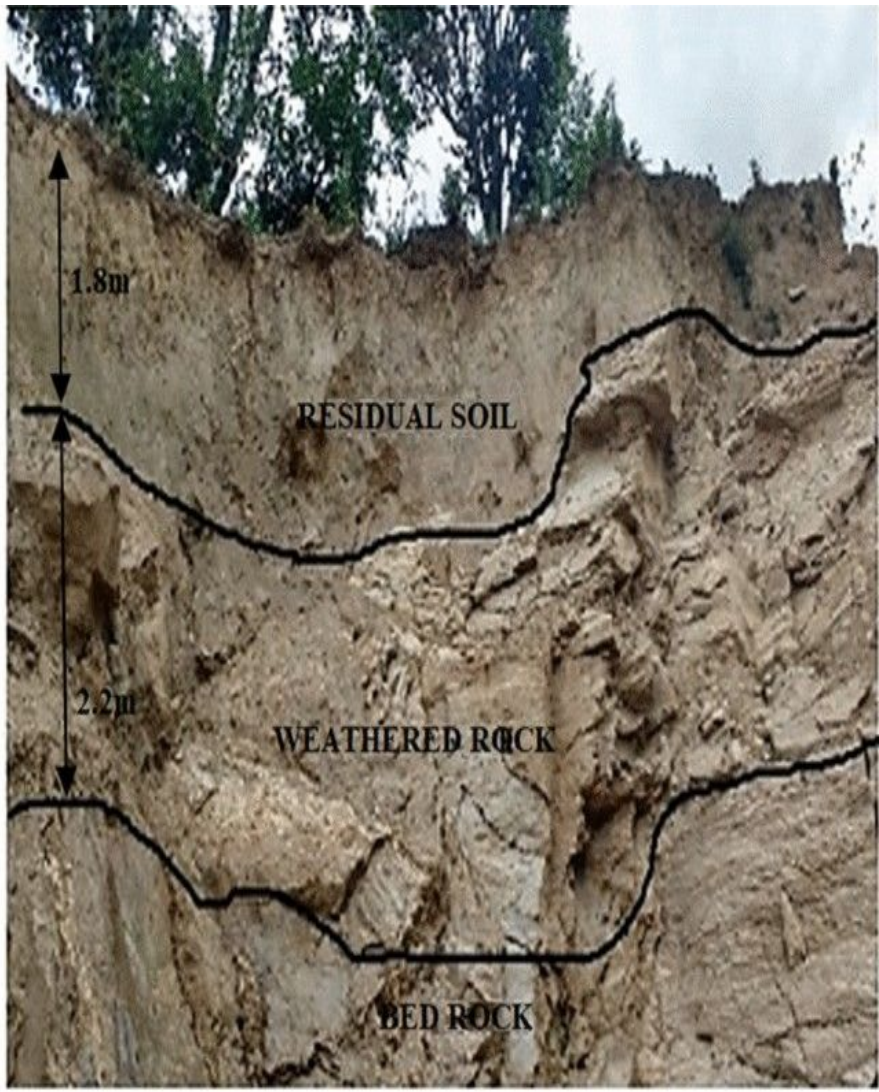
Products of weathering **may stay** in the **same place or may be moved** to other place by ice, wind and gravity.

At Place of Origin:

- Soils formed by the weathered product at place of origin are calle **residual soils**.
- An important characteristics of of residual soils is the **gradation** of particle size.
- Fine grained soils is found at the surface, grain size increases with depth and at greater depth, angula fragments may also be found.



(a)



(b)

Classification of Transported Soils

Depending on mode of transportation and deposition, transported soils may be classified into several groups:

- **Glacier soils** – by glaciers
- **Alluvial soils** – by streams
- **Lacustrine soils** – by lakes
- **Marine soils** – by seas
- **Aeolian soils** – by wind
- **Colluvial soils** – by gravity



Sedimentary Rock

Mechanical Sedimentary Rocks:

- Deposits of gravel, sand, silts and clay formed by weathering may become **compacted** by overburden pressure and **cemented** by agents like iron oxide, calcite, dolomite and quartz.
- Cementing agents are generally **carried** in solution by ground water.
- Such solutions **fill spaces** between particles and form sedimentary rocks known as detrial rocks.
- **Example**: Conglomerate, Sandstone, mudstone, shale.



Chemical Sedimentary Rocks

- Formed by the chemical process.
- **Examples**: Limestone, halite, dolomite and gypsum







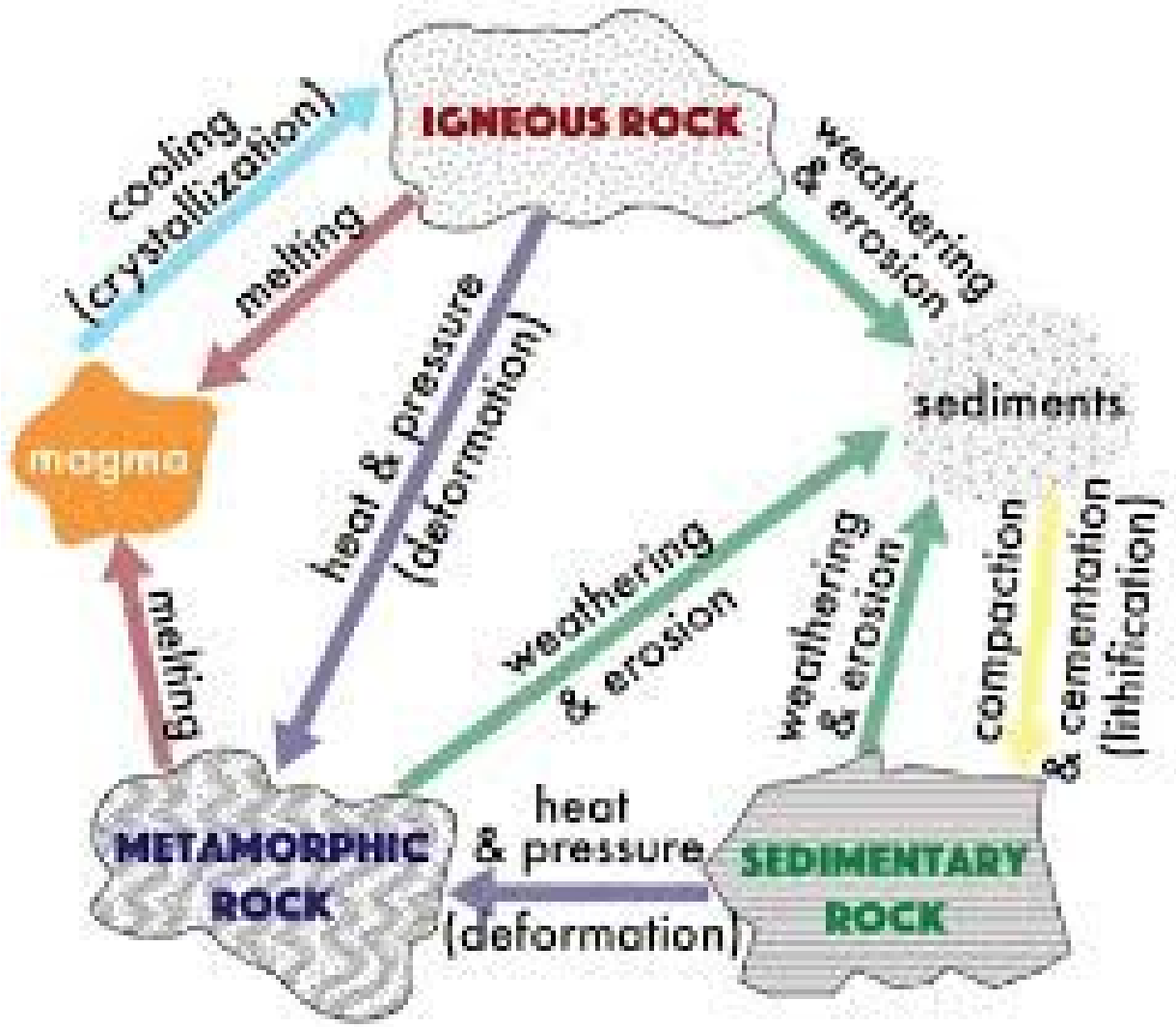


Metamorphic Rocks

- Process of changing the composition and the texture of rock by heat and pressure (without melting).
- Process known as metamorphism.
- Examples: Gneiss, Phylite, Schist.



Rock Cycle


IGNEOUS ROCK
cooling (crystallization)
melting
weathering & erosion
magma
heat & pressure (deformation)
sediments
melting
weathering & erosion
weathering & erosion
compaction & cementation (lithification)
METAMORPHIC ROCK
heat & pressure (deformation)
SEDIMENTARY ROCK

Soil Particle Shape

Shape of particles is equally important as particle size distribution, because it has significant influence on the physical properties of a given soil.

Divided into 3 major categories:

- Bulky
- Flaky
- Needle shaped.



Bulky

- Formed mostly by mechanical weathering of rock and minerals.
- Shapes of the bulky particles are termed as:

  Angular

  Sub-angular

  Sub-rounded

  Rounded

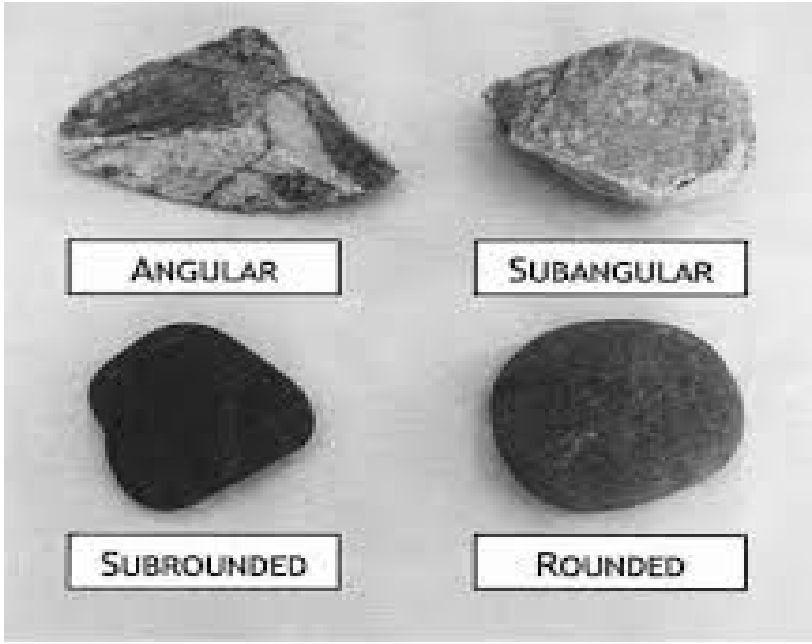


Figure 2.22 Shape of bulky particles (Courtesy of Braja M

- Small sand particles located close to their origin are generally **very angular**.
- Sand particles carried by wind and water for a long distance can be **sub-angular to rounded shape**.
- Shape of angular particles in a soil mass has a **great influence on the physical properties** of the soil, such as

  maximum and minimum void ratio

  shear strength parameters,

  compressibility etc.

Flaky Particles

- When the particle thickness is small when compared with the width and length of that particles.
- Particles are predominantly clay minerals.

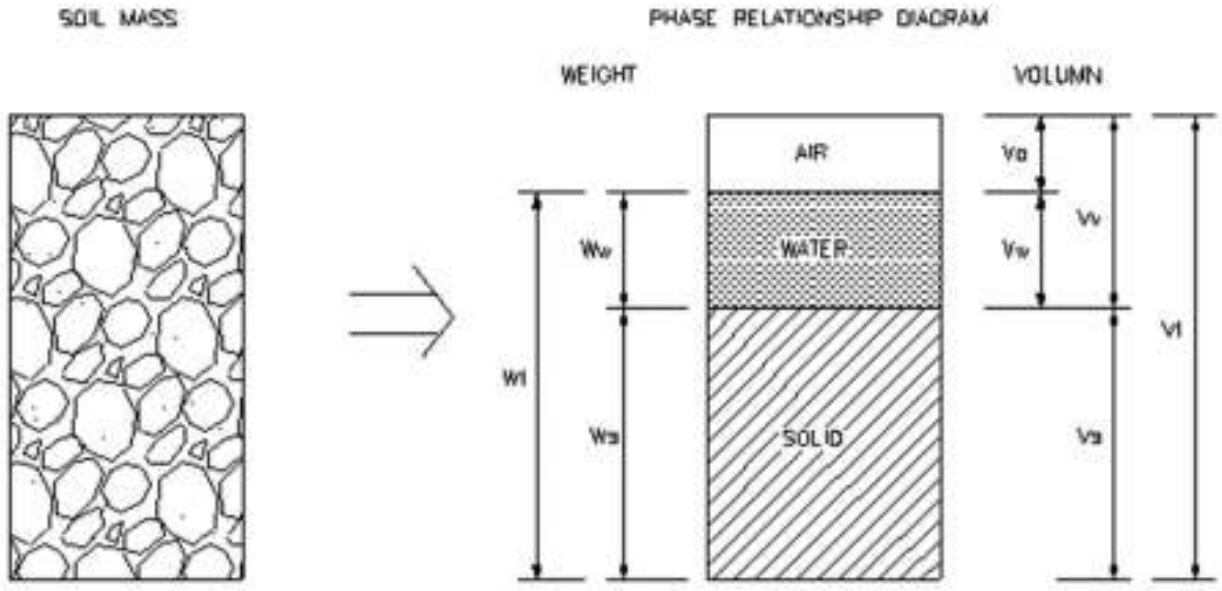


Elongated Particles:

A particle having a length to width ratio greater than 3.

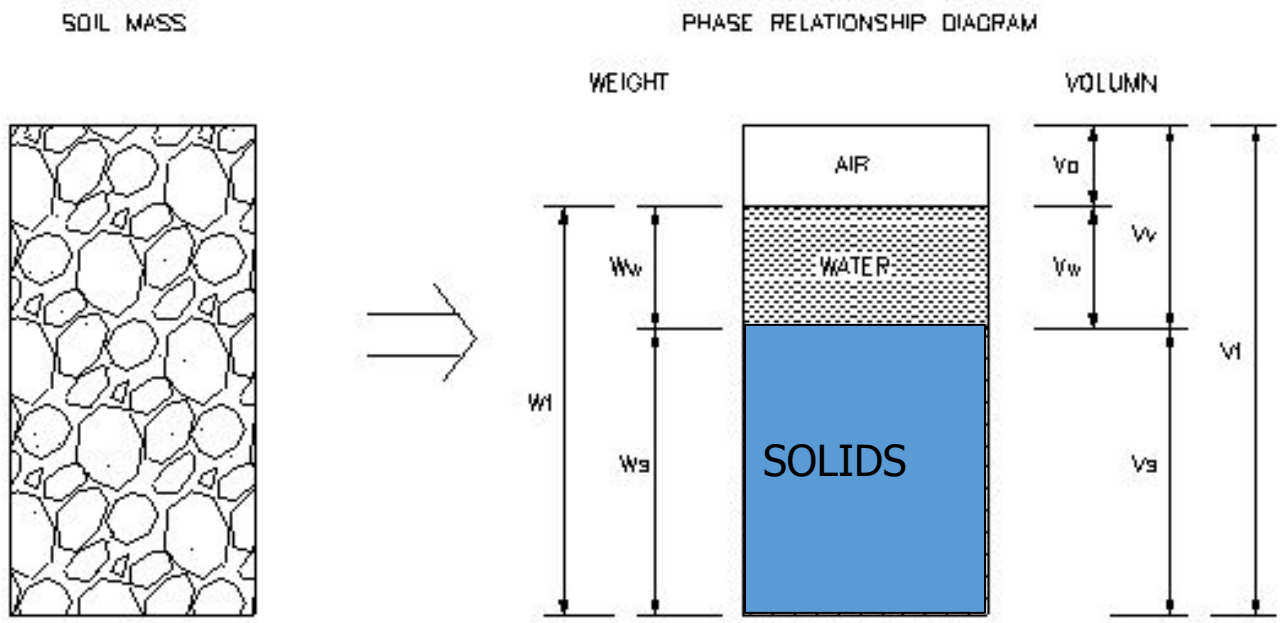
Three Phase System

- **Soils are different** from most of the civil engineering materials that it can contain **solid, liquid and gas phases.**
- **Liquid and gas phases** are contained in **voids or pores** between solid particles.
- **Three phases are often interact** and **these interactions have significant effects on soils behavior**.

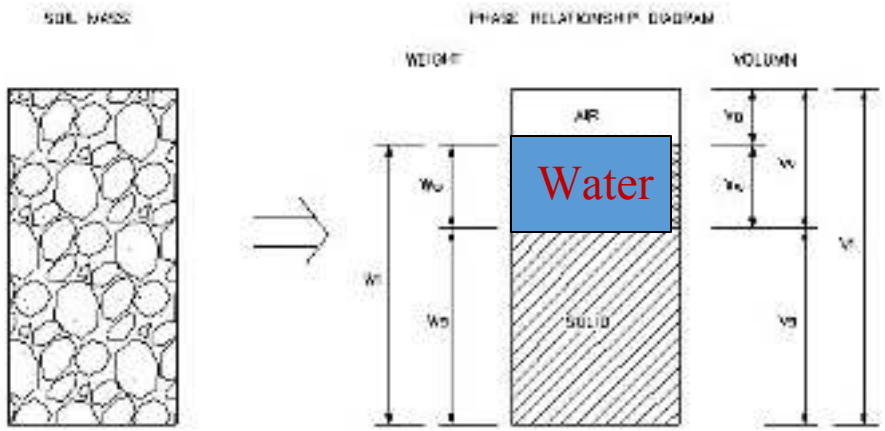


Solid Phase

- **Always present** in soils.
- **Consists of particles** derived from the weathering of rocks.
- Can **include organic materials**.



Liquid Phase



- **Usually present**, most often consists **of water**.

Can include other materials also:

- **Gasoline or other chemicals**

  -have **leaked out** of underground tanks or pipelines

  -or **infiltrated** from the ground surface.

clean up of such contaminants can be a significant problem.

- **Leachate**

  -Liquid that has escaped from the **sanitary landfill**.

Can **contaminate GW** and make it unfit for drinking.

- **Sea Water**

  -sea water moving inland through soil.

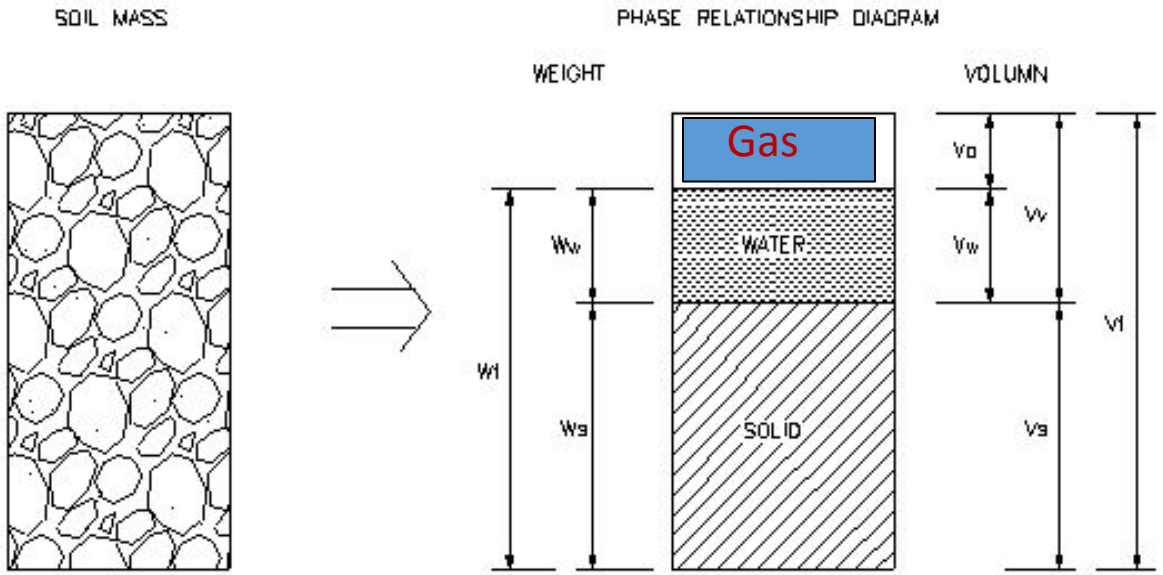
  -when wells are installed near ocean to pump out fresh water, get salt water.

- **Natural Petroleum Seeps**

Importance: Although these constituents usually present a **small fraction** of the liquid in a soil, they can be **very important,** specially if **the GW is to be pumped out and used** for domestic purposes, but becomes contaminated.

Gas Phase

Gas Phase



If the **liquid phase does not completely fill** the voids, the remaining, the remaining phase is occupied by the gas.

- **Usually air**, can include **other gases** also.

- **Methane and Carbon dioxide:**

  -Form from the **decomposition of organic materials**

  -often found from **the sanitary land fills**, and **near highly organic natural soils.**

- **Petroleum Vapor**: from the **contaminated soils**.

- **Hydrogen Sulfide**:

  -from the **decomposition of sulfide bearing materials.**

  -sometimes found **near sanitary landfills**, **sewerage facilities** and **coal deposits**.

Hazardous Effects of Gas in soils.

Gases other than air can be important because they sometimes **pose safety hazard to the workers** in **utility vaults**, **small excavation**, **mines** and **underground areas**.

Toxic and Inflammable:

- **Carbon dioxide**: form from the **decomposition of organic materials** and **toxic.**
- **Hydrogen Sulphide: poisonous**
- **Methane:** Also **form from decomposition of organic materials** and **inflammable**. If trapped, and accidentally ignited, create explosion.

- **Gases neither toxic nor inflammable**, can be dangerous because they displaced Oxygen, thus **suffocating workers**.

- This problems has **caused injury and death**, but can be avoided by using special detection equipment and adequate ventilation.

- Components other than air are very important, but air generally represent **a small portion of the soil weight and volume.**

- Simply refer to the liquid and gas phases as **'Pore water'** and **'Pore air'**.

Soils as Particulate materials

Most of the engineering materials consists of:

- **Continuous mass** held together with **molecular bond**, and

  -**Mechanical properties** of such materials depend on their **chemical make up** and the **nature of these bond.**

Example: Steel

-Shear strength of steel depend on the strength of their molecular bond,

-their shear failure requires breaking them off.

- Soil is a **particulate material that consists of individual particles assembled together**. Its engineering properties depend on:

  -**primarily** on **interaction** between these particles.

  -**only secondarily** on their **internal properties**.

Example: Gravel, sand, silts

When soil **fails in shear**, they do:

-because of the particles began to **roll and slide past** each others.

-**not because of the particles beak**, breakage of individual particles is typically minimal.

- **Shear strength of soil** depend on the following factors:
  - **coefficient of friction** between particles and
  - **tightness** of the of packing ie compaction.

- **Clays also have particulate structure** but the nature of the particles is quite different.
  - **much more interaction between the particles** and the pore water.
  - **behavior is more complex** than that of other.

Thanks

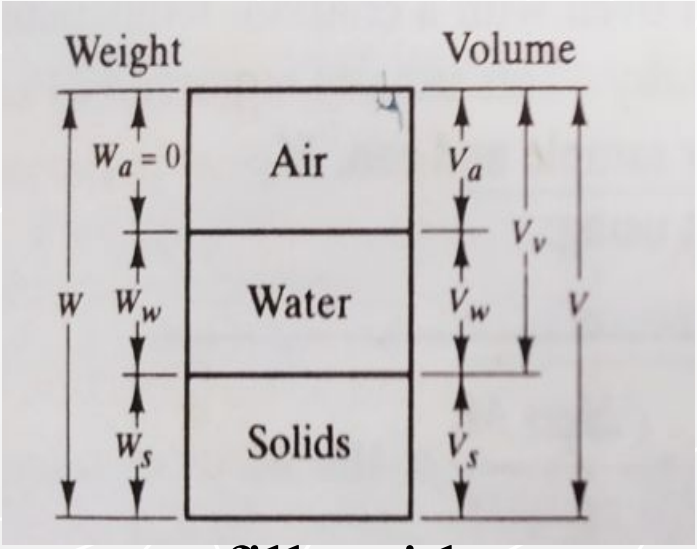
EWCE 341

Principles of Soil Mechanics

Weight–Volume Relationship

What we discussed last class?

- **Differences between soils & rocks and Other engineering Materials**.
- Soils as particulate materials.
- **Three Phase system of soils**



  * Solid phase – always present
  * Liquid phase – usually present
  * Gas phase – present when liquid does not fill voids completely

- **Hazardous effects of gas in soils other than air**

What we will discuss today?

- Phase diagram or weight-volume relationship.
- Different soil parameters

Weight–Volume Relationship

- Weight-Volume relationship is to **identify the relative proportions of solids, water and air** in a soil, because these proportions have a **significant effect on its behavior**.

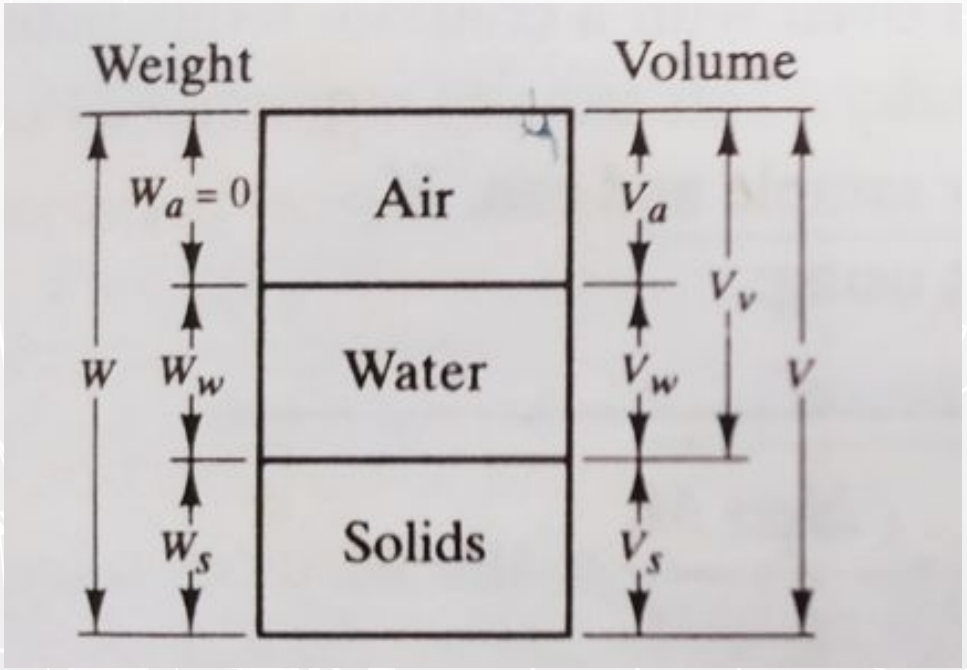
- Geotechnical engineers have developed **quantitative methods** of assessing these components.

Phase Diagrams

- Phase diagram **indicate the relative proportions of solids, water and air in a soil.**

- Left side of the diagram indicate the **weight or mass of each component,** while those on the **right side indicate their volumes.**

Three-Phase of Soil



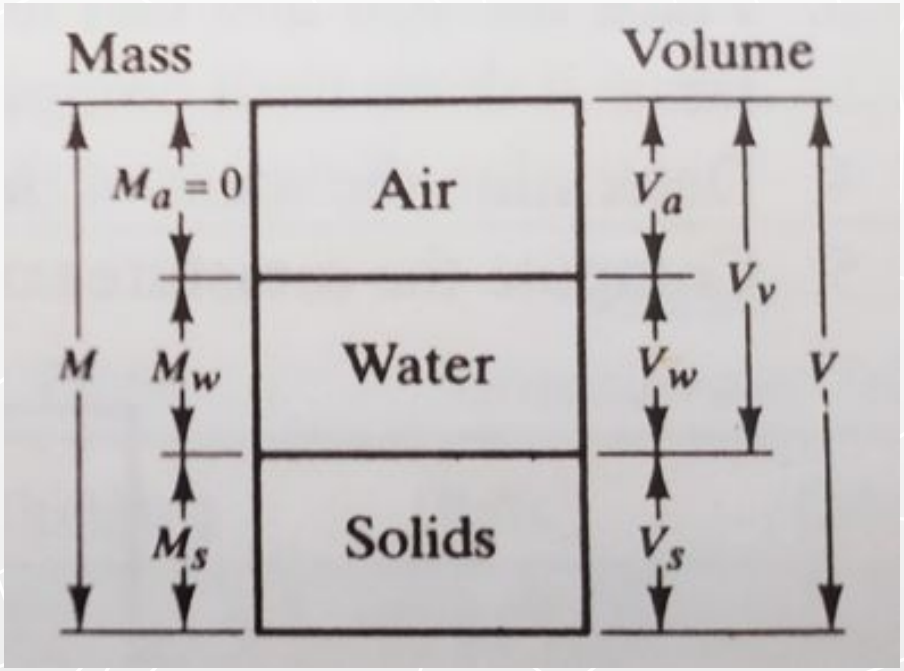
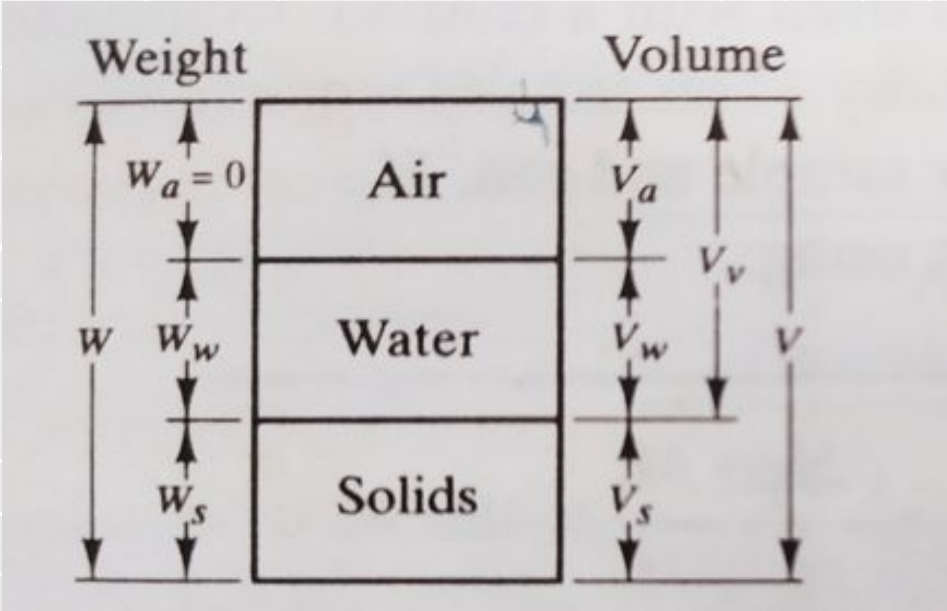


Figure: Phase diagram describe the relative proportions of solids, water and air in a soil. One side presents **weights or masses**, while other side presents **volumes**.

Definition of Weight-Volume Parameter

- Geotechnical engineers have defined **several weight-volume parameters** based on the dimensions shown in the phase diagram.
- Parameters give **important information** on the **composition of a particular soil**, and form part of the basic language of soil mechanics.



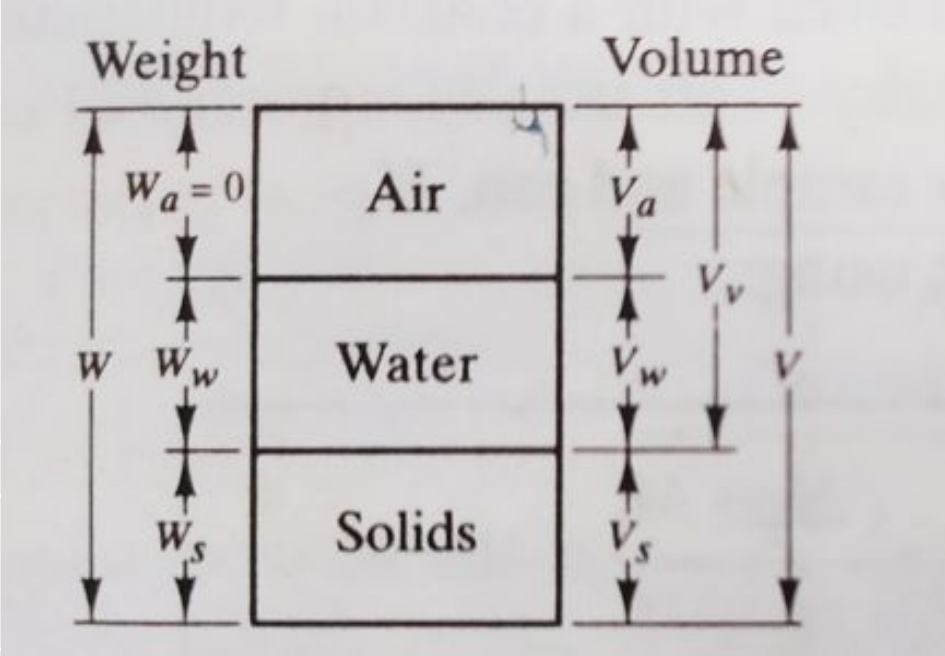
Moisture Content and Degree of Saturation

Moisture Content or Water Content (w):

- One of the most common soil parameters, which can be expressed either in terms of weights or mass.

$$w = \frac{W_w}{W_s} \times 100\%$$

$$w = \frac{M_w}{M_s} \times 100\%$$



- Values in the field between 3 and 70%.
- Values greater than 100%, are sometimes found in soft soils below the GWT, which simply means such soils have more water than solids.
- Moisture content can be easily **measured in the lab** by conducting moisture content test (ASTM D2216).

$$w = \frac{M_1 - M_2}{M_2 - M_c} \times 100\%$$

Where, M_1 = Mass of Soil+ Can, Mass of M_2= Dry soil+ Can

M_c= Mass of Can

- If there is no water w=0

Degree of Saturation (S)

- Percentage of void filled with water.

$$S = \frac{V_w}{V_v} \times 100\%$$

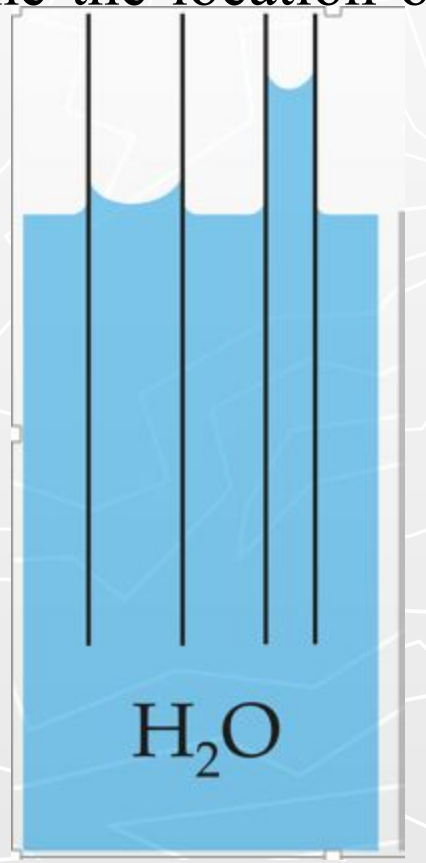
- Holding as much moisture as possible
- S=0 when there is no water..

- S has the maximum value 100%, when all of the voids are filled with water called ***saturated* condition**. **Below the GWT**, soils are generally saturated. (*When soil is being called saturated condition*?)
- **Values above GWT** are usually between 5 to 100%, although value approaching zero can be found in very arid region.

Why saturation conditions are not used to determine the GWT?

- In saturated condition, **capillary effects** can draw water upward from the GWT, often producing soils with S=100% well above the GWT.
- Saturation computations are not used to determine the location of GWT.

Capillary action/effect:
Capillary action is the ability of a liquid to flow in narrow spaces without the assistance of, or even in opposition to, external forces like gravity.



Unit weight and Density

- Geotechnical engineers often need to know the unit weight, γ (F-21)

$$\gamma = \frac{W}{V}$$

Why unit weight measurements should not even attempt on poor quality soil sample?

- Unit weight of **undisturbed soil samples** can easily be determined in the lab by measuring their physical dimensions and weighing them.
- Methods produce reliable measurements of γ for many soils.
- **Value of γ is affected by sample disturbance**, specially in sandy and gravely soils.
- Sometimes *unit* weight measurements are made supposedly on undisturbed soil samples that in reality have significant soil disturbance.

Unit weight and Density

- Such measurements are very misleading, so it is best to not even attempt unit weight measurements on poor quality samples.
- Following table presents typical ranges of γ for various soils.

Soil Type and Unified Soil Classification (See Section 5.3)	Typical Unit Weight, γ			
	Above Groundwater Table		Below Groundwater Table	
	(lb/ft^3)	(kN/m^3)	(lb/ft^3)	(kN/m^3)
GP — Poorly graded gravel	110 - 130	17.5 - 20.5	125 - 140	19.5 - 22.0
GW — Well graded gravel	110 - 140	17.5 - 22.0	125 - 150	19.5 - 23.5
GM — Silty gravel	100 - 130	16.0 - 20.5	125 - 140	19.5 - 22.0
GC — Clayey gravel	100 - 130	16.0 - 20.5	125 - 140	19.5 - 22.0
SP — Poorly graded sand	95 - 125	15.0 - 19.5	120 - 135	19.0 - 21.0
SW — Well graded sand	95 - 135	15.0 - 21.0	120 - 145	19.0 - 23.0
SM — Silty sand	80 - 135	12.5 - 21.0	110 - 140	17.5 - 22.0
SC — Clayey sand	85 - 130	13.5 - 20.5	110 - 135	17.5 - 21.0
ML — Low plasticity silt	75 - 110	11.5 - 17.5	80 - 130	12.5 - 20.5
MH — High plasticity silt	75 - 110	11.5 - 17.5	75 - 130	11.5 - 20.5
CL — Low plasticity clay	80 - 110	12.5 - 17.5	75 - 130	11.5 - 20.5
CH — High plasticity clay	80 - 110	12.5 - 17.5	70 - 125	11.0 - 19.5

Variations of Unit Weight

Two variations of unit weight are commonly used:

- Dry unit weight of soil: $\gamma_d = \frac{W_s}{V}$ soils in dry condition
- $\gamma_d = \frac{\gamma}{1+w}$
- Unit weight of water: $\gamma_w = \frac{W_w}{V_w}$

Normally, values of γ_w= **9.81 kN/m^3 = 62.4 lb/ft^3** for fresh water and γ_w= **10.1 kN/m^3 =64.0 lb/ft^3** are used.

- $\gamma_s = \frac{W_s}{V_s}$ unit weight of soil solids
- $\gamma_{sat} = \frac{W}{V}$ soils in saturation

$\gamma_{sub} = \frac{W_{submerged}}{V}$ soils in submerged condition = γ_{sat} - γ_w

- Buoyant Unit Weight, $\gamma_b = \gamma - \gamma_w$ For soils below GWT, γ_b is being used

$$\gamma = \frac{W}{V} = \frac{W_s + W_w}{V} = \frac{W_s + wW_s}{V}$$

as we know, $w = \frac{W_w}{W_s}$

$$= \frac{W_s(1++w)}{V} = \gamma_d(1+w)$$

now, $\gamma_d = \frac{\gamma}{1+w}$ (proved)

Density

- Similar parameters can be defined based on mass instead of weight. These becomes density ρ, dry density ρ_d, and density of water ρ_w:

$$\rho = \frac{M}{V}$$

$$\rho_d = \frac{M_s}{V}$$

$$\rho_w = \frac{M_w}{V_w}$$

- Design value of $\boldsymbol{\rho_w = 1000\ kg/m^3}$ **and** $\boldsymbol{62.4\ lb/ft^3}$ for fresh water.

- **Relation between unit weight and density:** $\boldsymbol{\gamma = \rho\ g}$, where g= acceleration due to gravity= 9.81 m/s^2 = 32.2 ft/s^2

Why unit weight is more useful than density in geotechnical computations?

- Unit weight is **used to compute stresses** due to weight of the soil.
- In **dynamic analysis**, it is required to consider inertial (***Inertia*** *is the resistance of any physical object to any change in its velocity*) effects that are best presented in terms of masses.
- **For example**, test programs are performed in order t**o study general statistical trends in the response of soils under simulated earthquake conditions.**

Specific Gravity of Solids

- Specific Gravity of any material is the ratio of its density to that density of water.
- In case of soils, it is required to **compute Sp Gr for the solid phase only**, and express as the specific of solids, G_s:

$$G_s = \frac{M_s}{V_s \rho_w}$$

$$G_s = \frac{W_s}{V_s \gamma_w}$$

- $\gamma_s = \frac{W_s}{V_s}$
- $G_s = \frac{\gamma_s}{\gamma_w}$

- Specific Gravity of soil solids is quite different from the specific

Typical values of Specific Gravity Values of Soil Solids

- Nearly all real soils have $2.60 < G_s < 2.80$, which is generally a very narrow range.

- Typical Values:

Soil Type	Specific Gravity
Sand	2.65 – 2.67
Silty Sand	2.67 – 2.70
Inorganic clay	2.70 – 2.80
Soils with mica or iron	2.75 – 3.00
Organic soils	Quite variable, as low as 2.0

- Some unusual soils have G_s values well outside these limits. For example **Olivine sands in Hawaii have $G_s$values as high as 4.50,** while organic soils have G_s values, some times less than 2.00.as high as







Olivine Sand (green) Beach in Hawaii

Void Ratio and Porosity

Void ratio:

- Relative volumes of voids and solids may be expressed using the void ratio, e:

$$e = \frac{V_v}{V_s}$$

- **Densely packed soils** have a low void ratio.
- Typical values in the **field range from 0.1 to 2.5**

Porosity:

- Relative volume of voids to the volume of soil mass.

$$n = \frac{V_v}{V} \text{ x } 100\%$$

- Values ranges 9 to 70%.

Relation between porosity (n) and void ratio (e)

- We know, $\mathbf{n = \frac{V_v}{V} \times 100\%}$

Again $\mathbf{e = \frac{V_v}{V_s}}$, $V_v = e\, V_s$, and $V = V_v + V_s$

$$n = \frac{eV_s}{V_v + V_s} \times 100\%$$

$$= \frac{eV_s}{e\,V_s + V_s} \times 100\%$$

$$= \frac{V_s\,(e)}{V_s(e+1)} \times 100\%$$

$$= \mathbf{\frac{e}{e+1} \times 100\%}$$

Or, $ne + n = e$ or $e\,(1-n) = n$ or $\mathbf{e = \frac{n}{1-n}}$

Relative Density

- Relative density is a **special weight-volume** parameter used in sandy and gravely soils.
- Defined as: $$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \text{ x } 100\%$$

Where, D_r = Relative density

e = void ratio

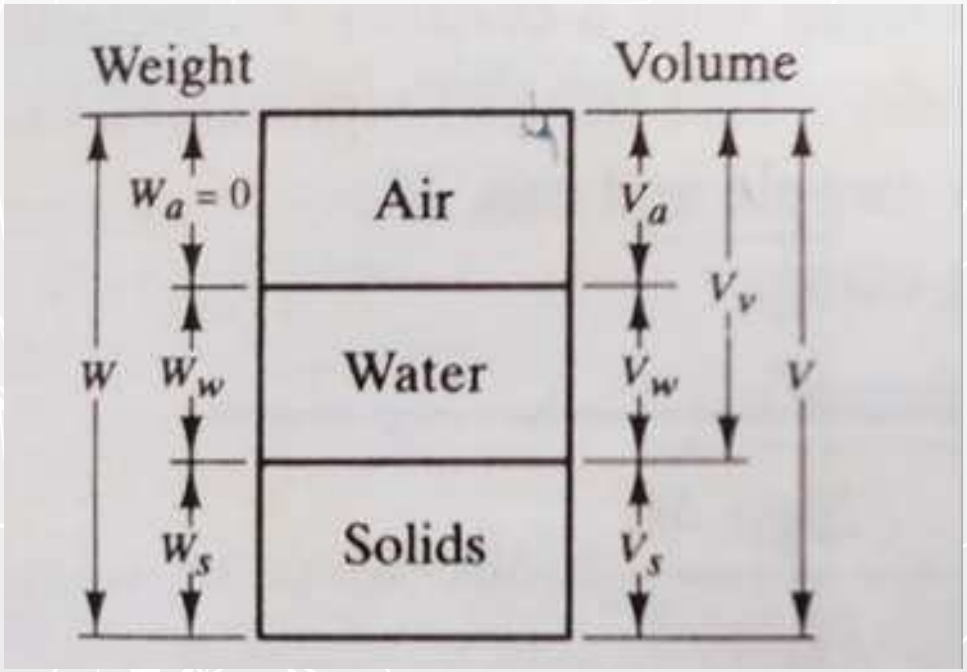
e_{min} = Minimum void ratio

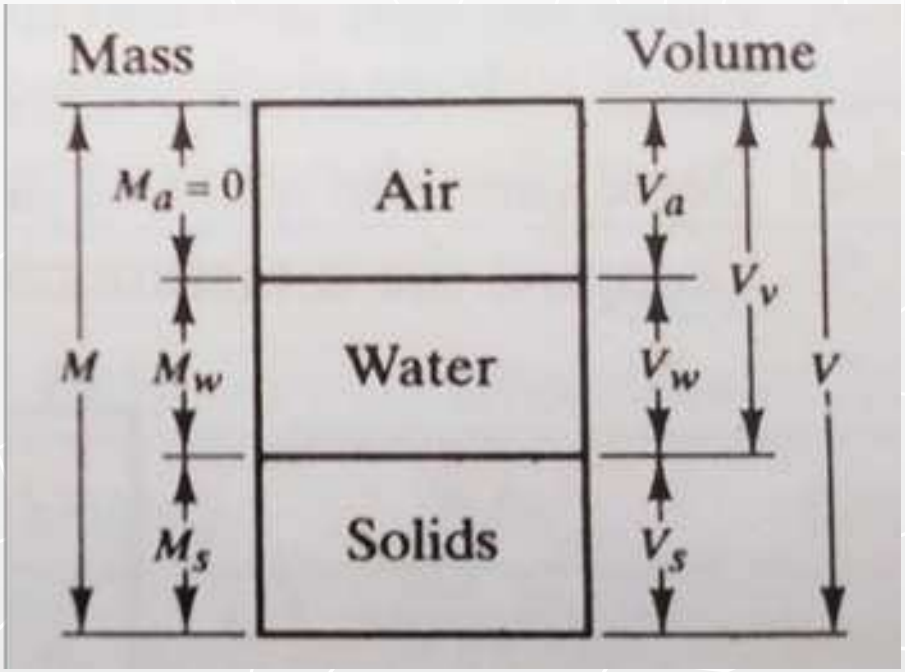
e_{max} = Maximum void ratio

- Values of e_{max} **and** e_{min} **represent the soil in very dense and very loose conditions** respectively and determined by standard laboratory test (ASTM D4253 and D4254).
- Loose soils have low values of D_r and while dense soils have high values, in theory **it varies from 0% to 100%.**

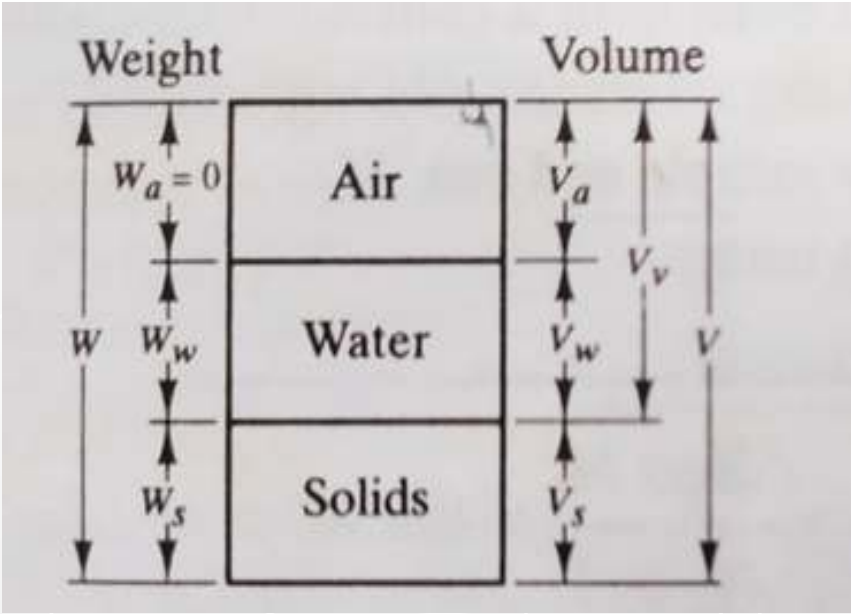
Derived Equation:

By combining definitions and the phase diagrams, we can derive a series of new weight-volume equations:





Prove that $S = \frac{w G_s}{e} \times 100\%$. (F-21)



Solution: Let $V_s = 1$

$e = \frac{V_v}{V_s}$ gives $V_v = e$

$S = \frac{V_w}{V_v} \times 100\%$

$= \frac{V_w}{e} \times 100\%$ gives $V_w = Se$

$\gamma_w = \frac{W_w}{V_w} = \frac{W_w}{Se}$ gives $W_w = \gamma_w Se$

$G_s = \frac{W_s}{V_s \gamma_w} = \frac{W_s}{(V_s = 1)\gamma_w}$ gives $W_s = G_s \gamma_w$

Now, $w = \frac{\gamma_w Se}{G_s \gamma_w}$ or $w = \frac{Se}{G_s} \times 100\%$

$G_s = \frac{Se}{w}$ or $S = \frac{w G_s}{e} \times 100\%$ proved.

Prove that $\gamma_d = \frac{G_s \gamma_w}{1+e}$ when water content is 100%

Solution: Let $V_s = 1$

$$\gamma_d = \frac{W_s}{V}$$

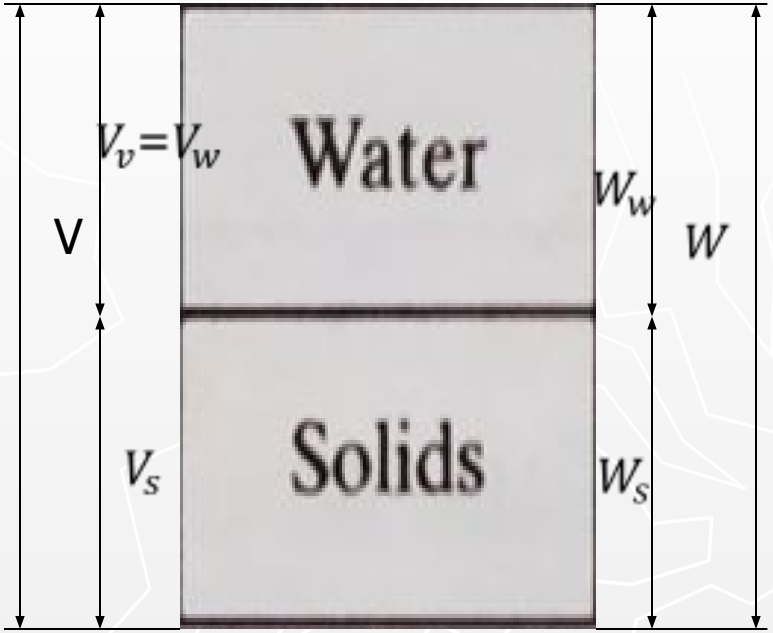
$$G_s = \frac{W_s}{V_s \gamma_w} = \frac{W_s}{(V_s = 1)\gamma_w} = \frac{W_s}{\gamma_w}$$

gives $W_s = G_s \gamma_w$

Again, $e = \frac{V_v}{V_s} = \frac{V_v}{(V_s = 1)} = V_v$ or $e = V_w$

Water content, $w = \frac{W_w}{W_s}$ gives $W_w = wW_s$

$$V = V_s + V_w = 1 + V_w \text{ or } V = 1 + V_v$$



- $\gamma_d = \frac{W_s}{V} = \frac{G_s \gamma_w}{1+e}$ **Proved**

Again, $G_s = \frac{W_s}{V_s \gamma_w}$ **, Similarly** $G_w = \frac{W_w}{V_w \gamma_w}$

$$V_w = \frac{W_w}{G_w \gamma_w} = \frac{(W_w =) w W_s}{G_w \gamma_w} = \frac{w G_s \gamma_w}{G_w \gamma_w} = \mathbf{w} G_s \qquad (G_w = 1) \text{------- (1)}$$

- $\gamma_d = \frac{W_s}{V} = \frac{G_s \gamma_w}{1+V_w} = \frac{G_s \gamma_w}{1+\mathbf{w} G_s} = \frac{G_s \gamma_w}{1+\mathbf{w} G_s}$ **proved**

Again, $e = \frac{V_v}{V_s} = \frac{V_v}{(V_s=1)} = V_v = V_w$ **(from Figure)**

$e = V_w = \mathbf{w} G_s$ **---- (from 1)**

$$\gamma_d = \frac{G_s \gamma_w}{1+e} = \frac{G_s \gamma_w}{1+\mathbf{w} G_s}$$ **Proved.**

Thanks

EWCE 341

Principles of Soil Mechanics

Weight–Volume Relationship

Void Ratio and Porosity

Void ratio:

- Ratio of volumes of voids and solids may be expressed using the void ratio, e:

$$e = \frac{V_v}{V_s}$$

- **Densely packed soils** have a low void ratio.
- Typical values in the **field range from 0.1 to 2.5**

Porosity:

- Relative volume of voids to the volume of soil mass.

$$n = \frac{V_v}{V} \times 100\%$$

- Values ranges 9 to 70%.

Relation between porosity (n) and void ratio (e)

We know, $\mathbf{n = \frac{V_v}{V} \times 100\%}$

Again $\mathbf{e = \frac{V_v}{V_s}}$, $V_v = e\ V_s$, and $V = V_v + V_s$

$$n = \frac{eV_s}{V_v + V_s} \times 100\%$$

$$= \frac{eV_s}{e\ V_s + V_s} \times 100\%$$

$$= \frac{V_s\,(e)}{V_s(e + 1)} \times 100\%$$

$$= \mathbf{\frac{e}{e + 1} \times 100\%}$$

Or, ne + n = e or e (1-n) = n or $\mathbf{e = \frac{n}{1 - n}}$

Relative Density

- Relative density is a **special weight-volume** parameter used in sandy and gravely soils.
- Defined as: $$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \text{ x } 100\%$$

Where, D_r = Relative density

e = void ratio

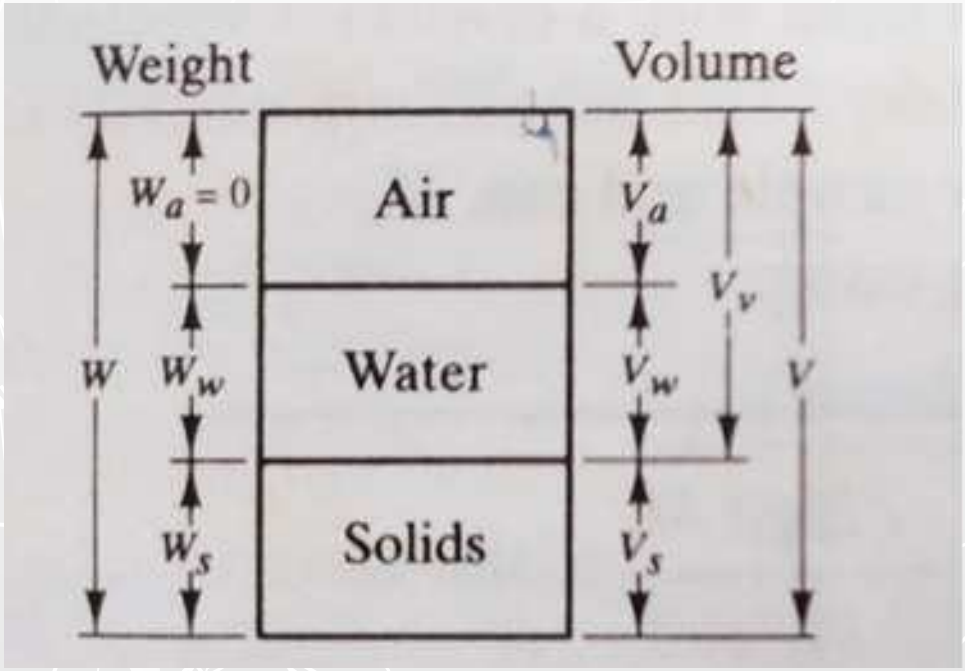
e_{min} = Minimum void ratio

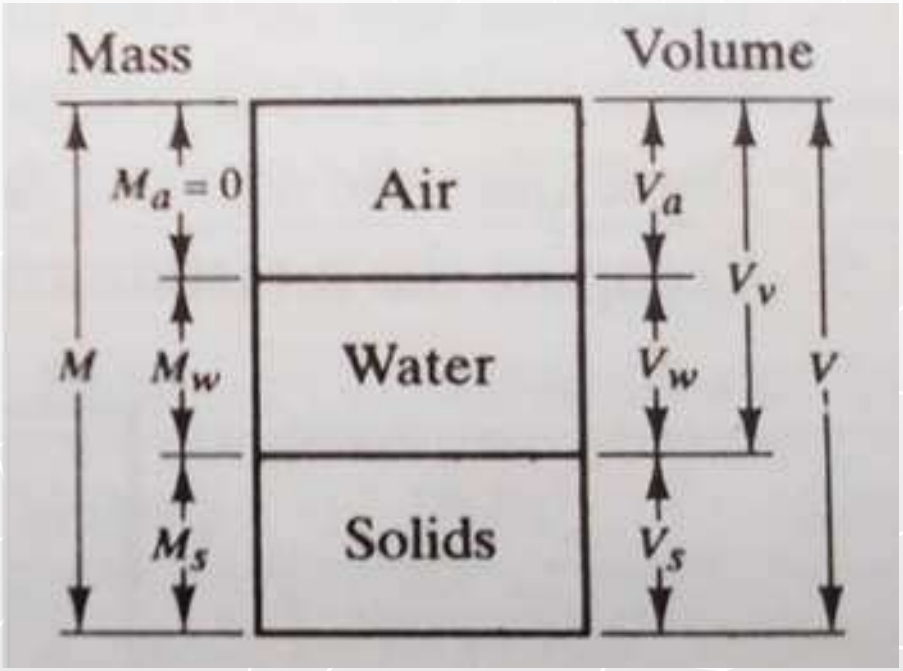
e_{max} = Maximum void ratio

- Values of $\boldsymbol{e_{max}}$ **and** $\boldsymbol{e_{min}}$ **represent the soil in very loose and very dense conditions** respectively and determined by standard laboratory test (ASTM D4253 and D4254).
- Loose soils have low values of D_r and while dense soils have high values, in theory **it varies from 0% to 100%.**

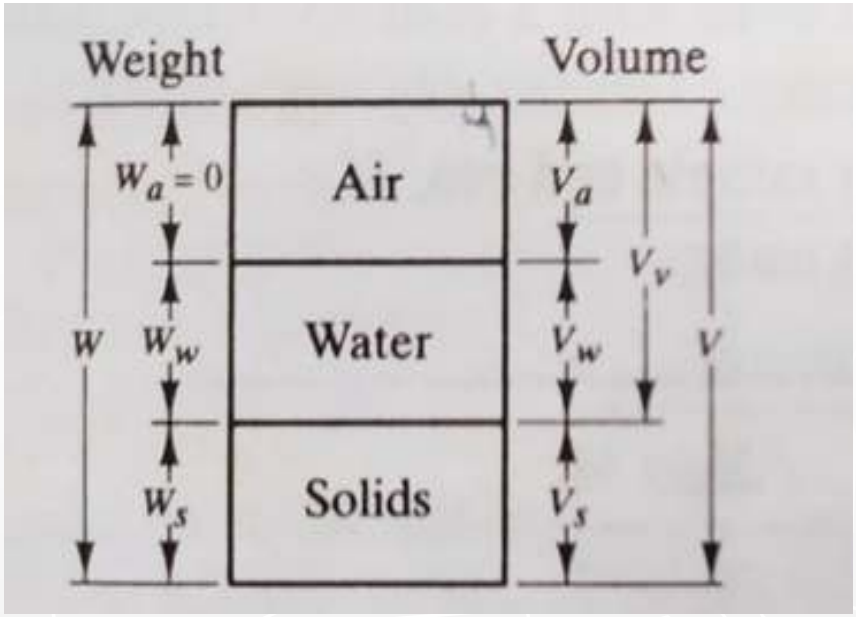
Derived Equation:

By combining definitions and the phase diagrams, we can derive a series of new weight-volume equations:





Prove that $S=\frac{w G_s}{e}$ **x 100%. (F-21)**



Solution: Let V_s=1

$e = \frac{V_v}{V_s}$ gives V_v=e

$S= \frac{V_w}{V_v}$ x 100%

$=\frac{V_w}{e}$ x 100% gives V_w=Se

$\gamma_w = \frac{W_w}{V_w} = \frac{W_w}{Se}$ gives $W_w = \gamma_w Se$

$G_s = \frac{W_s}{V_s \gamma_w} = \frac{W_s}{(V_s=1)\gamma_w}$ gives $W_s = G_s \gamma_w$

Now, $w= \frac{W_w}{W_s}$ x 100% $= \frac{\gamma_w Se}{G_s \gamma_w}$ x 100% or $w= \frac{Se}{G_s}$ x 100%

$G_s= \frac{Se}{w}$ x 100% or $S=\frac{wG_s}{e}$ x 100% proved.

Prove that $\gamma_d = \frac{G_s \gamma_w}{1+e}$ when water content is 100%

Solution: Let $V_s = 1$

$$\gamma_d = \frac{W_s}{V}$$

$$G_s = \frac{W_s}{V_s \gamma_w} = \frac{W_s}{(V_s = 1)\gamma_w} = \frac{W_s}{\gamma_w}$$

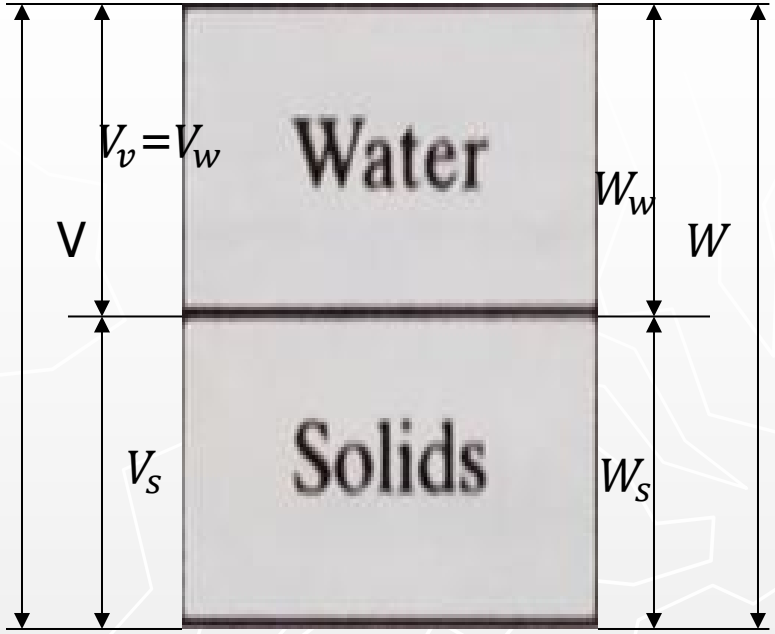
gives $W_s = G_s \gamma_w$

Again, $e = \frac{V_v}{V_s} = \frac{V_v}{(V_s = 1)} = V_v$ or $e = V_w$

Water content, $w = \frac{W_w}{W_s}$ gives $W_w = wW_s$

$V = V_s + V_w = 1 + V_w$ or $V = 1 + V_v$

$= 1 + e$



- $\gamma_d = \frac{W_s}{V} = \frac{G_s \gamma_w}{1+e}$ **Proved**

Again, $G_s = \frac{W_s}{V_s \gamma_w}$ **, Similarly** $G_w = \frac{W_w}{V_w \gamma_w}$

$$V_w = \frac{W_w}{G_w \gamma_w} = \frac{(W_w =) w W_s}{G_w \gamma_w} = \frac{w G_s \gamma_w}{G_w \gamma_w} = \mathbf{w} G_s \qquad (G_w = 1) \text{------- (1)}$$

- $\gamma_d = \frac{W_s}{V} = \frac{G_s \gamma_w}{1+V_w} = \frac{G_s \gamma_w}{1+\mathbf{w} G_s} = \frac{G_s \gamma_w}{1+\mathbf{w} G_s}$ **proved**

Again, $e = \frac{V_v}{V_s} = \frac{V_v}{(V_s = 1)} = V_v = V_w$ **(from Figure)**

$e = V_w = \mathbf{w} G_s$ **---- (from 1)**

$\gamma_d = \frac{G_s \gamma_w}{1+e} = \frac{G_s \gamma_w}{1+\mathbf{w} G_s}$ **Proved.**

Following equations may be used:

$$\gamma_d = \frac{\gamma}{1 + w}$$

$$\gamma_d = \frac{G_s \gamma_w}{1 + w G_s / S}$$

$$W_s = \frac{W}{1 + w}$$

$$M_s = \frac{M}{1 + w}$$

$$e = \frac{G_s \gamma_w}{\gamma_d} - 1$$

$$w = S\left[\frac{\gamma_w}{\gamma_d} - \frac{1}{G_s}\right] \times 100\%$$

$$S = \frac{w}{\frac{\gamma_w}{\gamma_d} - \frac{1}{G_s}} \times 100\%$$

Solving Weight – Volume problems

Two methods:

- Using derived equations
- Using phase diagrams

Using derived equations

A 27.50 lb soil sample has a volume of **0.220 ft^3**, a moisture content of **10.2%**, and specific gravity of solids of **2.65.** Compute the unit weight, dry unit weight, degree of saturation, void ratio, and porosity.

Solutions:

Using derived equations:

$$\gamma = \frac{W}{V} = \frac{27.50\ lb}{0.220\ ft^3} = 125.0\ \text{lb}/ft^3$$

$$\gamma_d = \frac{\gamma}{1+w} = \frac{125.0\ lb/ft^3}{1+0.102} = 113.4\ \text{lb}/ft^3$$

$$S = \frac{w}{\frac{\gamma_w}{\gamma_d} - \frac{1}{G_s}} \text{ x } 100\% = \frac{0.102}{\frac{62.4\ lb/ft^3}{113.0\ lb/ft^3} - \frac{1}{2.65}} \text{ x } 100 = 59\%$$

$$e = \frac{G_s \gamma_w}{\gamma_d} - 1 = \frac{2.65\ x\ 62.4\ lb/ft^3}{113.0\ lb/ft^3} - 1 = 0.4582$$

$$n = \frac{e}{1+e} \text{ x } 100\% = \frac{0.458}{1+0.458} \text{ x } 100\% = 31.4\%$$

Using Phase Diagram – Procedure:

- Draw a phase diagram and annotate all of the dimensions presented in the problem statement. Remember, set W_a=0. If the soil is saturated, also set V_a=0.
- If **no dimensions** are given, a quantity may be assumed. For example V= 1 m^3 or 1 ft^3
- **Addition and subtraction** may be done to determine other dimensions.
- **Compute the required parameters** from these dimensions.

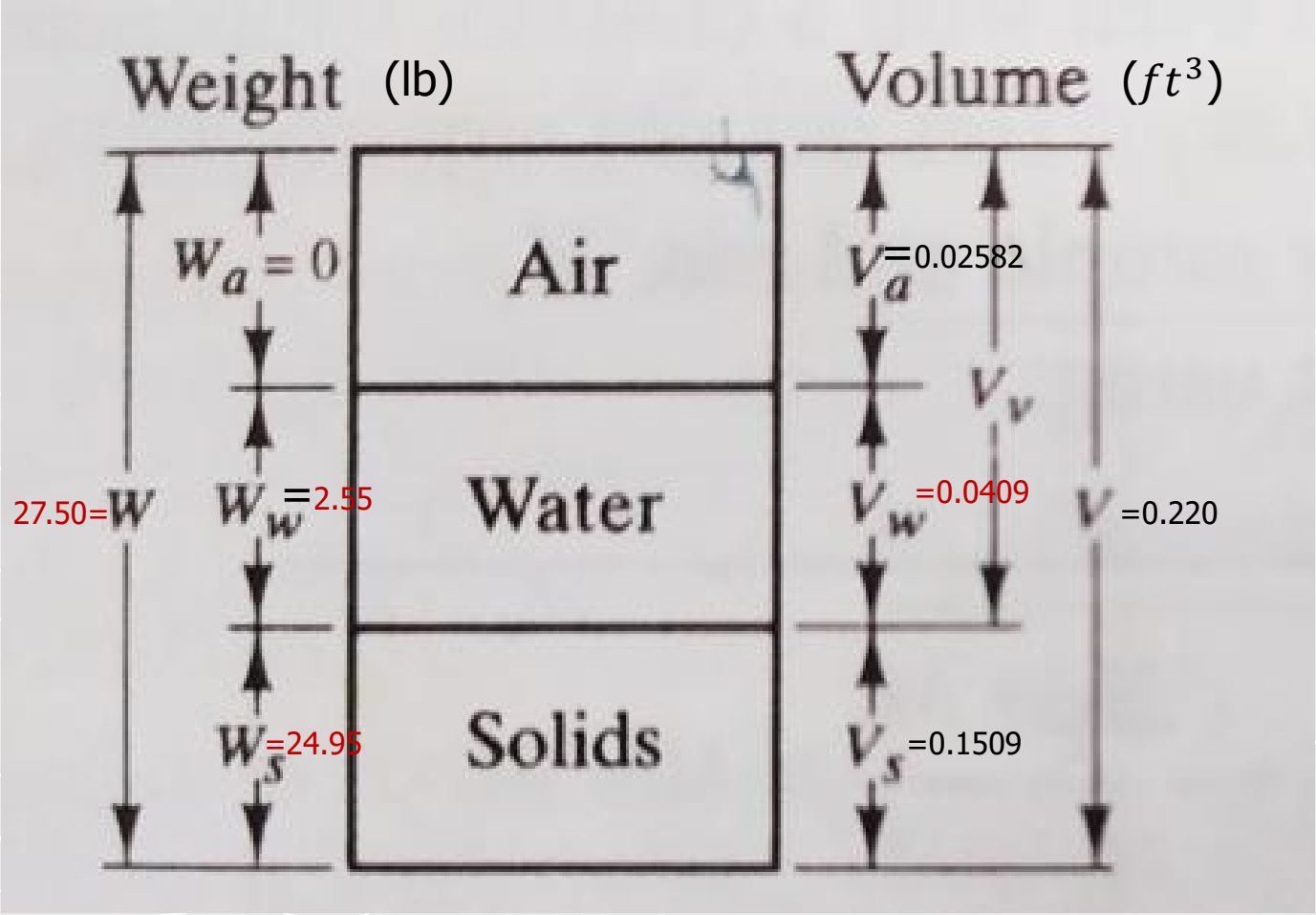
- $\gamma = \frac{W}{V}$ $\gamma_d = \frac{W_s}{V}$ $S=\frac{V_w}{V_v}x100\%$ $e=\frac{V_v}{V_s}$ $n=\frac{V_v}{V}x100\%$

Using Phase Diagram:

A 27.50 lb soil sample has a volume of 0.220 ft^3, a moisture content of 10.2%, and specific gravity of solids of 2.65. Compute the unit weight, dry unit weight, degree of saturation, void ratio, and porosity.

Solutions:

Step-1:Draw and annotate a phase diagram.



- Step-2: Assume dimension - na
- Step-3: Determine dimensions in phase diagram:

$$W_s = \frac{W}{1+w} = \frac{27.50}{1+0.102} = 24.95 \text{ lb}$$

$$W_w = W - W_s = 27.50 - 24.95 = 2.55 \text{ lb}$$

$$G_s = \frac{W_s}{V_s \gamma_w} \text{ or } \quad 2.65 = \frac{24.95}{V_s\,(6.24\ lb/ft^3)} \text{ or } V_s = 0.1509 ft^3$$

$$\gamma_w = \frac{W_w}{V_w} = \frac{2.55\ lb}{V_w} \qquad \text{or} \qquad V_w = 0.0409\ ft^3$$

$$V_a = V - V_w - V_s = 0.220\ ft^3 - 0.0409\ ft^3 - 0.1509 ft^3 = 0.0282 ft^3$$

Step-4: Compute parameters

$$\gamma = \frac{W}{V} = \frac{27.50\ lb}{0.220\ ft^3} = 125.0\ \text{lb}/ft^3$$

$$n = \frac{V_v}{V} \text{ X } 100\% = \frac{0.0282ft^3 + 0.0409ft^3}{0.220ft^3} \text{ X } 100\% = 31.4\%$$

$$e = \frac{V_v}{V_s} = \frac{0.0409ft^3 + 0.0282ft^3}{0.1509ft^3} = 0.458$$

$$\gamma_d = \frac{W_s}{V} = \frac{24.95\ lb}{0.220ft^3} = 113.0\ \text{lb}/ft^3$$

$$S = \frac{V_w}{V_v} \text{ x } 100\% = \frac{0.0409ft^3}{0.0409ft^3 + 0.0282ft^3} \text{ x } 100\% = 59\%$$

A certain soil has the following properties:

G_s= 2.71

n=41.9%

w=21.3%

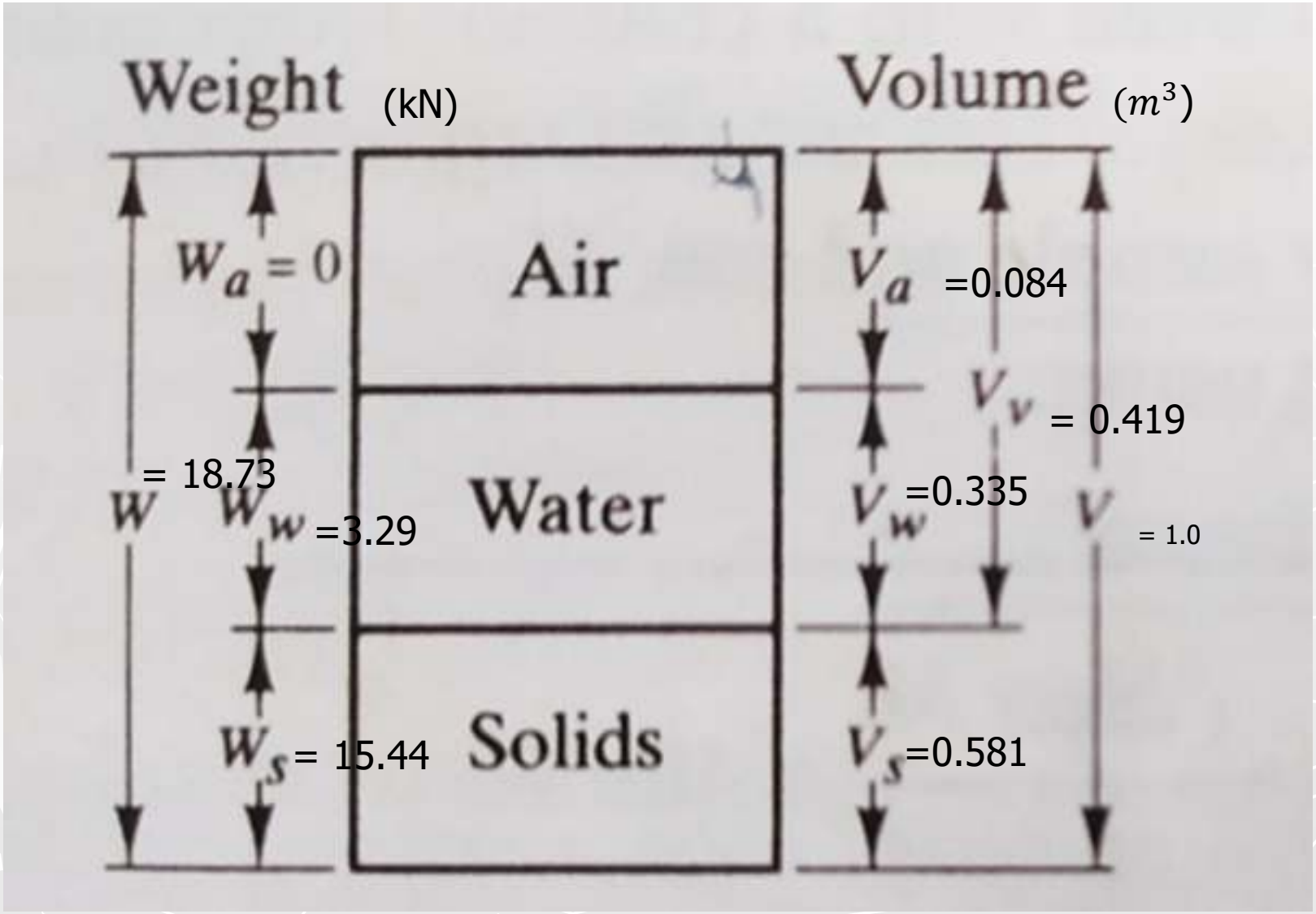
Find the degree of saturation S, and unit weight, γ, using phase diagrams.

Solution:

Step-1: Draw and annotate phase diagram.

Step-2: Assume dimensions.

- No volumes or weighs are stated.
- All data applies to entire soil strata.
- Assume V= 1 m^3


Weight (kN)
Volume (m^3)
$W_a = 0$
Air
V_a =0.084
V_v = 0.419
W = 18.73
W_w =3.29
Water
V_w =0.335
V = 1.0
W_s = 15.44
Solids
V_s =0.581

Step-3: Determine dimensions in phase diagram.

$$\mathbf{n}=\frac{V_v}{V} \quad \text{or } 0.419=\frac{V_v}{1} \qquad \text{or } V_v=0.419\ m^3$$

$$V_s=\mathrm{V}-V_v \quad \text{or } V_s=1-0.419=0.581\ m^3$$

$$G_s=\frac{W_s}{V_s\gamma_w} \quad \text{or } 2.71=\frac{W_s}{(0.581m^3)(9.81\frac{kN}{m^3})} \qquad \text{or } W_s=15.44\ \mathrm{kN}$$

$$w=\frac{W_w}{W_s}\mathbf{x100\%} \quad \text{or } 0.213=\frac{W_w}{15.44\ kN} \qquad \text{or } W_w=3.29\ \mathrm{kN}$$

$$\mathrm{W}=W_s+W_w=15.44\ \mathrm{kN}+3.29\mathrm{kN} \quad \text{or } \mathrm{W}=18.73\ \mathrm{kN}$$

$$\gamma_w=\frac{W_w}{V_w} \quad \text{or } 9.81\mathrm{kN}/m^3=\frac{3.29kN}{V_w} \qquad \text{or } V_w=0.335\ m^3$$

$$V_a=V_v-V_w=0.419m^3-0.335m^3=0.084m^3$$

Step-4: Compute parameters.

$$S=\frac{V_w}{V_v}\ x100\% = \frac{0.335m^3}{0.335m^3+0.084m^3}\ x100\% = 80.0\%$$

$$\gamma=\frac{W}{V} = \frac{18.73kN}{1m^3} = 18.73\ kN/m^3$$

Thanks

EWCE 341

Principles of Soil Mechanics

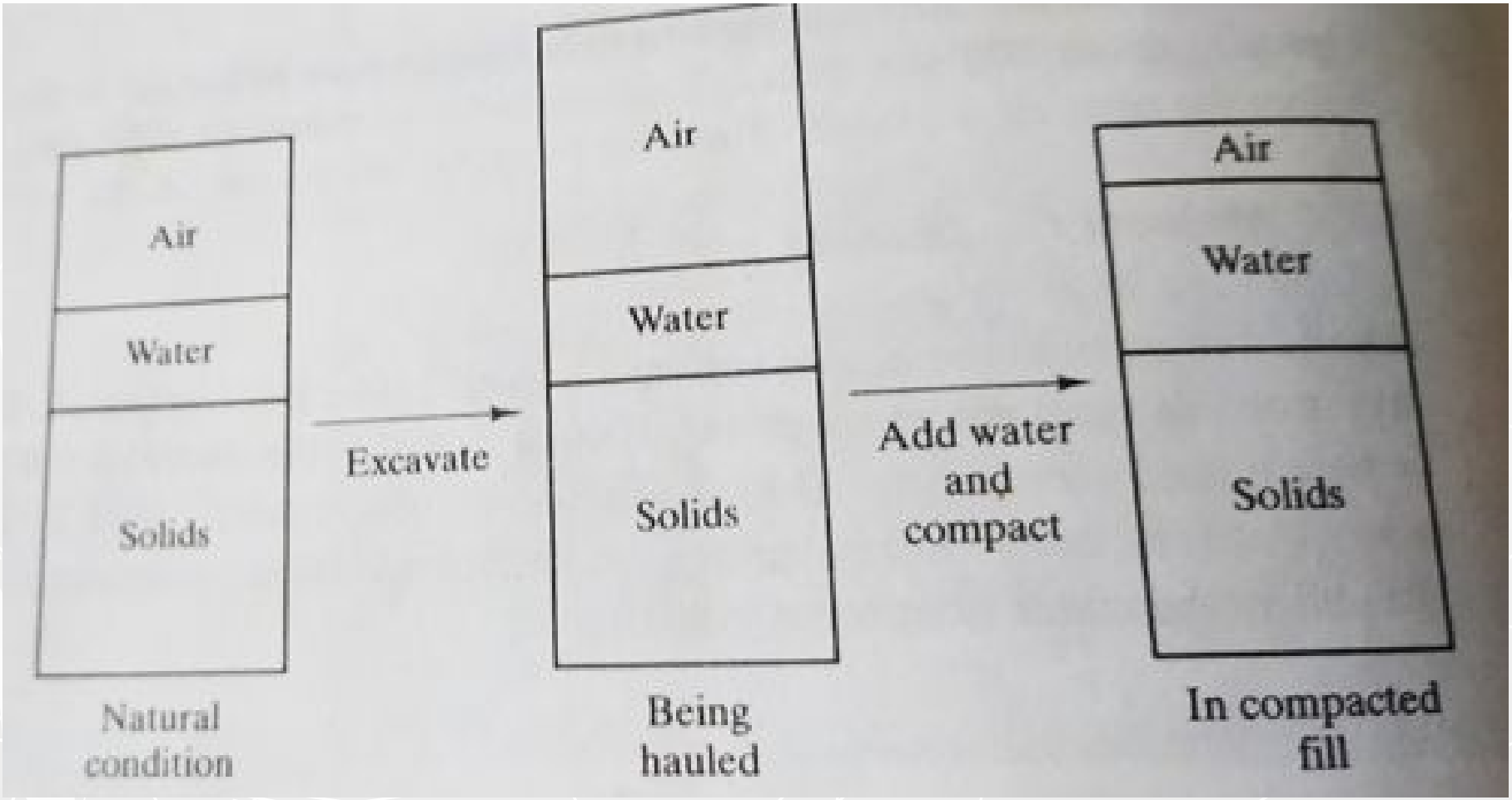
What is soil Compaction?

- **Use of equipment** to **compress soil** into **smaller volume**, thus increasing its dry unit weight and improving its engineering properties.

- Solids and water are virtually **incompressible**, so compaction produces a **reduction in volume of air**.

Definition of soil Compaction

- Pressing the soil particles close each other **by mechanical methods.**
- During compaction, **air is expelled from the voids** in the soil mass and **moisture content is fixed**.



Phase Diagram showing the changes in soil as it moves from its natural location to a compaction fill

Purpose of soil Compaction:

- Improve **engineering properties** of soils.
- Increase **shear strength** of soil.
- Increase **stability** and **bearing capacity**.
- Reduce **compressibility**.
- Reduce the **permeability**.

Requirements of soil compaction in Engineering works:

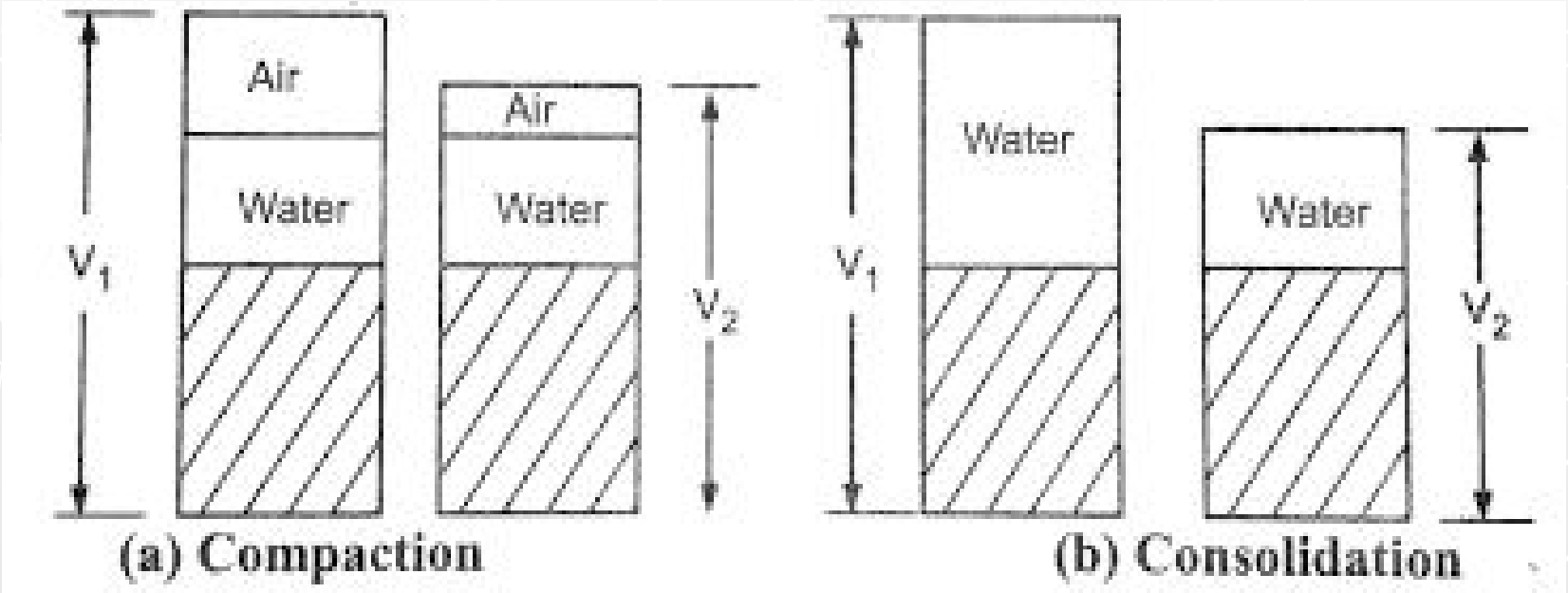


- Earth dam
- Canal embankment
- Highways, railways and runways
- Many other engineering applications.

Differences between consolidation and compaction of soils:

- Both the processes caused the **reduction of volume** in the soil mass.
- **Consolidation**: **Gradual process** of reduction of volume under **sustained static loading**.

Compaction: **Rapid process** of reduction of volume by **mechanical methods** like rolling, tamping, vibration etc.



- **Consolidation**: Causes a reduction of volume of **saturated soils/partially saturated soils** due to **squeezing out of water** from the soils.

Compaction: Volume of **limited saturated soils** decreases due to **expulsion of air** from the voids at **unaltered water content**.

- **Consolidation**: Process **occurs in nature** when saturated soil deposits are subjected to **static loads**, caused by the weight of buildings and other structures.

Compaction: **Artificial process** which is done to increase the density (unit weight) of the soils **to improve its properties** before it put into any use.

Ensuring Proper Compaction of Fill:

- Civil Engineers have to **include criteria for placement and compaction of fill** in the plans and specifications.

- Contractors are then **obliged to satisfy the criteria** while implementing projects.

History of development of compaction of fill:

- Early fill were used **without any special effort to compact** the fill.
- Some engineers **recognized the importance** of compaction in 19th century.
- **Animals were used as compaction equipment** on some project, a team of 115 goats used to compact an earth dam.

- Then **heavy rollers began to be used**; initially these rollers were pulled by a teams of **horses,** but by 1920 the horses being replaced with the **tractors**.

- During 20th century, development of enhanced the **capabilities of compaction equipment**.

- Now, a **wide varieties of effective equipment** is available.

Incidental Compaction:

- **All of equipment that drives over the fill**, from pickup trucks to loaded scrapers, **contribute to the compaction**.

- As an civil engineer, generally you **can not rely on** this incidental compaction because:

-Most construction equipment is **intentionally designed to have low contact pressure** between the tires or tracts and soils. Such pressures are too low to produce the required compaction in normal-thickness lifts.

-**Incidental traffics usually follows common routes**, so their compactive effort is not **uniformly distributed across the fill**. Thus some areas may receive sufficient compaction, while other receive virtually none.

Necessity of Specialized Compaction Equipment:

- Necessary to use specialized compaction equipment specifically designed for this task.

- Such equipment is **much more efficient and effective**.

Methods used by the Compaction Equipment for compaction:

Pressure:

- **Contact pressure** between the equipment and the ground is probably the **most important factor** in resulting compaction of under lying soils.
- A typical **smooth drum roller** has contact pressure of about 3500 kPa (500lb/sq inch), which is far greater than the track mounted equipment (83 kPa or 12 lb/sq inch)



Impact:

- Some equipment **imparts a series of blows** to the soils, such as by dropping a weight.
- This **adds a dynamic component** to the Compactive efforts.



Vibration:

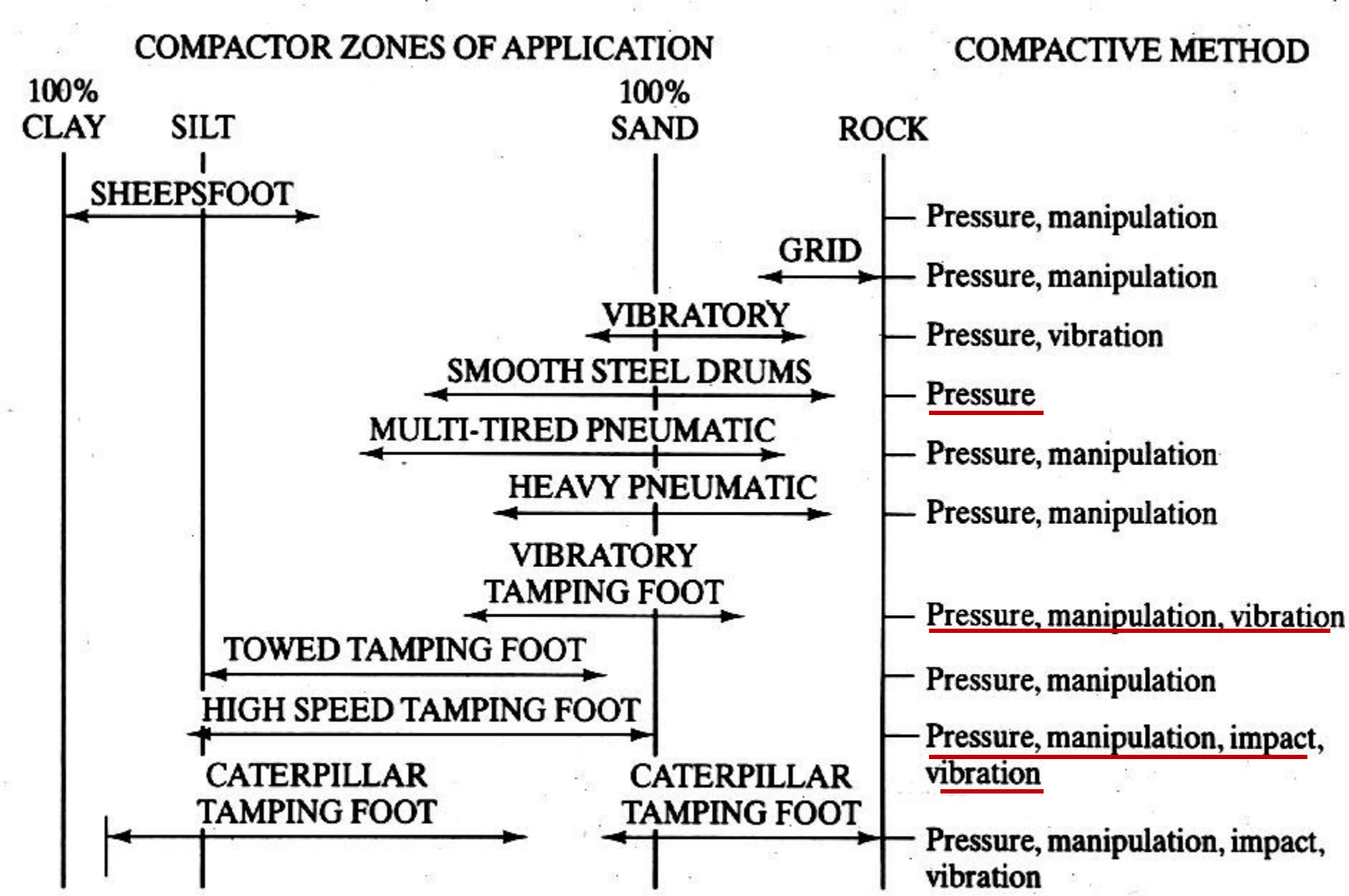
- Vibratory compaction equipment utilizes eccentric weighs or some other device to **induce strong vibrations** in to the soils, which can **enhance the compaction**.
- Vibrations typically have a **frequency of 1000-3500 cycles per minutes.**



Manipulation/kneading:

- Compaction equipment that **imparts some shearing forces to the soils** can also contribute to better compaction.
- This **action is called kneading** or manipulation.
- Excessive manipulation, such as in **an overlay wet fill**, can be detrimental.
- When such fills are **simply being moved around with no compaction occurring**, condition that called **pumping**.





Soil types best suited for various kinds of compaction equipment (Adapted from Caterpillar,1993).

- Also called **tamping rollers**, have rectangular-shaped **lugs, or "feet**,"(150mm – 250mm).

- Type of Soils: **Heavy clay and silty clay**.

- Compactive Method: **Pressure and manipulation**

Sheep foot Rollers



Grid Rollers





Type of soils: **Gravels and Gravely soils**.

Compactive method: **Pressure and Manipulation**

Vibratory Rollers



- Surface compaction of granular soils is mainly conducted using vibratory rollers.
- Type of Soils: **Sandy soils**.
- Compactive Method: **Pressure and vibration**

Smooth Steel Drum Roller





- Surface compaction of granular soils.
- Type of Soils: **Silt, Sands and Gravel**
- Compactive Method: **Pressure**

Multi-Tired Pneumatic Rollers



- Use air to create the required inflation pressure which goes from 200 to 1000 kN/sq.m.
- Type of Soils: **Silt, Sands and Gravel**
- Compactive Method: **Pressure and Manipulation**

Heavy Pneumatic Roller



- Type of Soils: **Sands and Sandy Gravel**
- Compactive Method: **Pressure and Manipulation**

Vibratory Tamping Foot Roller



- Type of Soils: **Silty Sands and Sand**
- Compactive Method: **Pressure, Manipulation and Vibration**

Towed Tamping Foot Roller



- Type of Soils: **Silts to sands**
- Compactive Method: **Pressure, Manipulation**

High Speed Tamping Foot Roller



- Type of Soils: **Silts to sands**
- Compactive Method: **Pressure, Manipulation, impact, Vibration**

Caterpillar Tamping Foot Roller



- Type of Soils: **All Types of Soils**
- Compactive Method: **Pressure, Manipulation, impact, Vibration**

Compaction Improves engineering properties of fill:

Compaction is one of the **most important parts** of earthwork construction.

Compaction **improves the engineering properties** in many ways:

Increase Shear strength,

- **Reduces the potential for slope stability problems**, such as landslides,
- **Enhances the fill capacity for supporting loads**, such as foundations.

Decrease the compressibility

- Reduces the potential for excessive settlement.

Decrease hydraulic conductivity

- **Inhibits the flow of water through the soils**; may be desirable, depending on situation.

Decrease void ratio

- **Reduces the amount of water can be held in soils**;
- Helps in maintaining the **desirable strength parameters**.

Increase erosion resistance

- Helps to maintain the **ground surface in a serviceable condition**.

Soil Compaction

Why dry unit weight ($\gamma_d = \frac{W_s}{V}$) of soils is an important parameter for soil compaction?

- Nearly all **compaction specifications are based on** achieving a certain value of dry unit weight.
- Geotechnical Engineers **measure the unit weight of soils in the field to verify the contractors compliance** with the specification.
- **Dry unit weight** is an **indicators of soils quality, easy to measure** and **correlate** with desirable engineering properties.
- **Engineering properties** ie. Favorable strength, compressibility, erosion control etc **to be obtained when dry unit weight criteria has been achieved.**

Cylindrical Rollers:



Rollers function admirably while utilizing surface dressings to create an even smoother and flatter surface.

Thanks

EWCE 341

Principles of Soil Mechanics

Proctor Compaction Test

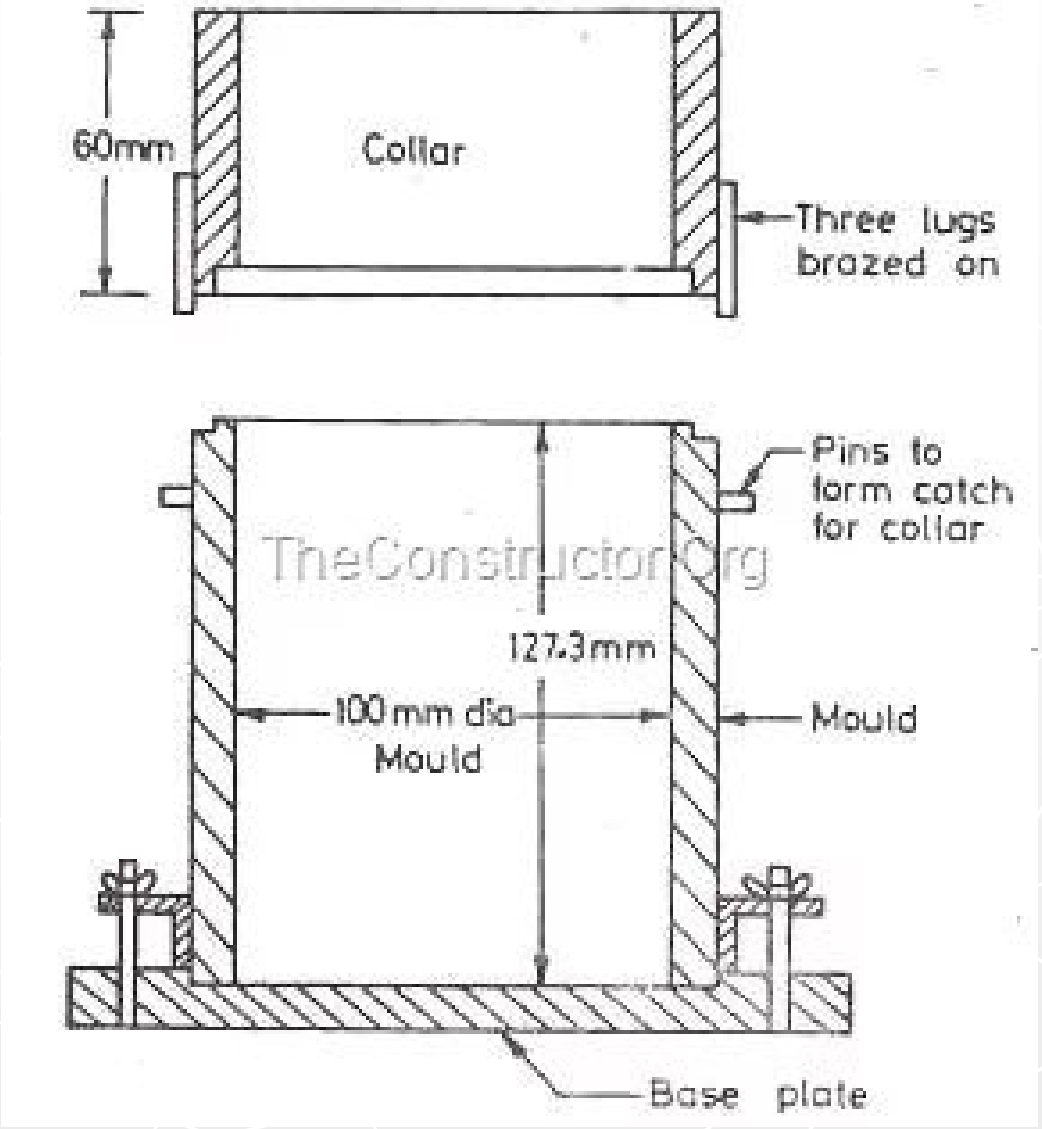
What we learned in the last class?

- What is soil Compaction?
- Purpose of soil Compaction.
- Requirements of soil compaction in Engineering works.
- Differences between consolidation and compaction of soils.
- Ensuring Proper Compaction of Fill
- History of development of compaction of fill
- Necessity of Specialize Compaction Equipment
- Methods used by the Compaction Equipment for compaction.
- Compaction improves engineering properties of fill.
- Why dry unit weight ($\gamma_d = \frac{W_s}{V}$) of soils is an important parameter for soil compaction?

Laboratory Compaction Test of Soils



Standard Proctor Test Apparatus (Mold and Rammer)


60mm
Collar
Three lugs brazed on
Pins to form catch for collar
TheConstructor.org
127.3mm
100 mm dia Mould
Mould
Base plate

Proctor Compaction Test

What is Proctor Compaction Test?

- During 1930s, **R R Proctor, an engineer** of LA City, USA developed a **method for assessing the compaction of fills**, becomes a nearly universal.

- **Method consists of :**

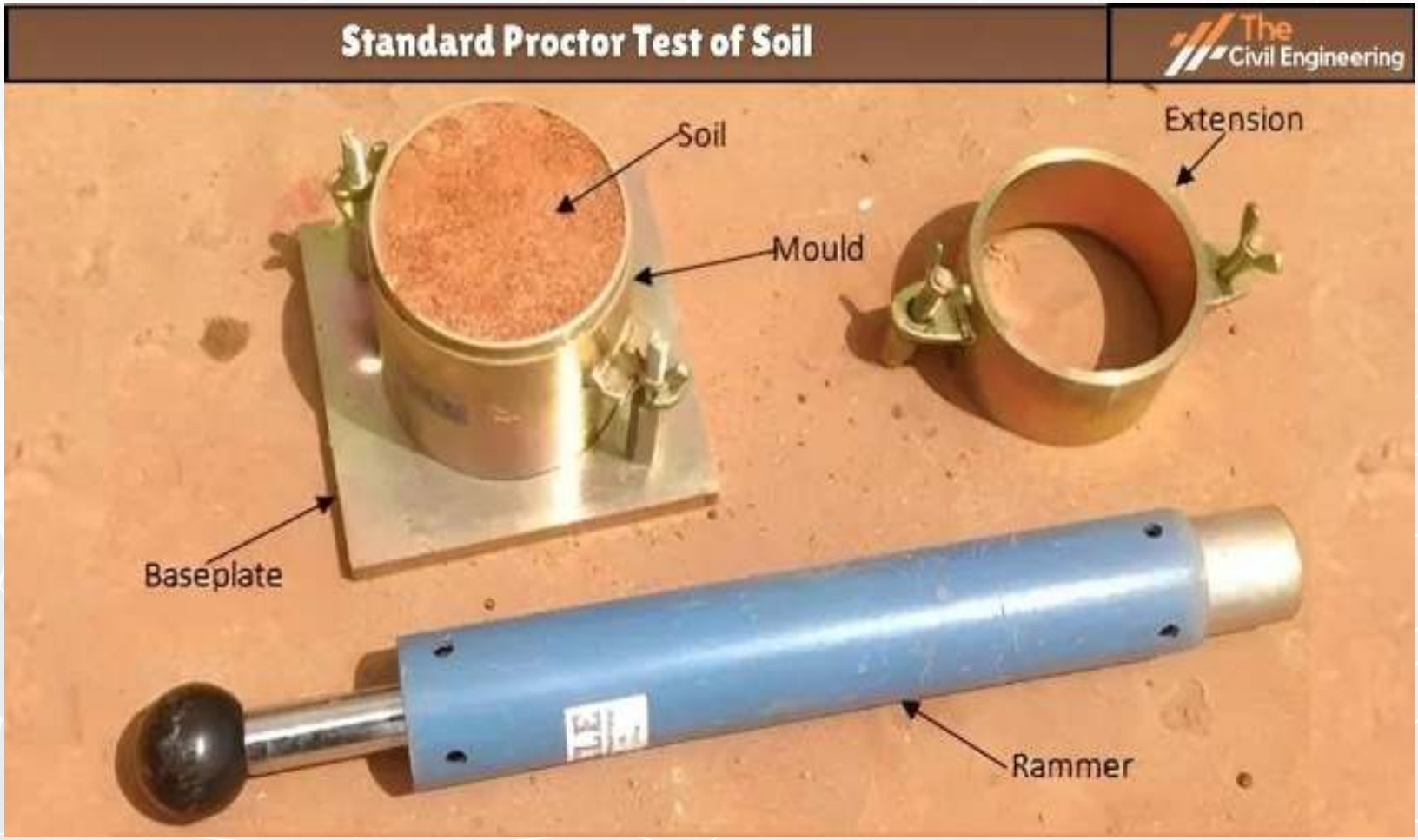
  -Compacting soils in the lab to **attain** $(\gamma_d)_{max}$

  -Requiring the contractor to **achieve some specified % of this value in the field**.

Types of Proctor Compaction Test:

- **Standard Proctor Test**: Original version of laboratory test.

- **Modified Proctor Test**: Revised version of Standard Proctor Test.



Modified Proctor Test:

- During 1940s and 1950s, geotechnical engineers found that 'Standard Proctor Test' in **no way sufficient** for airport and highways because fills were not providing adequate support for heavy trucks and aircraft.

- **US Army Corps of Engineers** addressed the problem by developing 'Modified Proctor Test' which **used greater level of compaction** and thus producing higher values of $(\gamma_d)_{max}$.



Differences between Standard Proctor Compaction Test and Modified Proctor compaction test

Events	Standard P C test	Modified P C Test
Standardised	ASTM D698, AASHTO T-99	ASTM D1557 and AASHTO T-180
Hammer Weight	5.5 lb (2.49kg)	10 lb
Hammer drop height	12” (305mm)	18” (457mm)
Number of Soil Layers	3	5
Number of hammer blows/ Layers	25	25
Energy Imparted from hammer for each sample		

Test Procedure:

- About **3.0kg of air-dried** soil sample is taken.
- Water is added to bring its **water contain 5%** if soil is coarse grained, **8% if its fine grained soils.**
- **Water content should be much less** than the expected moisture content:

  Sand - 6-10%

  Sandy silt or silty sand- 8 –12%

  Silt- 12 -16%

  Clay - 14-20%
- Soil is mixed thoroughly and covered with a **wet cloth** and left for **about 15-30mins.**
- Mold is **cleaned, dried** and **greased** lightly.

- Place some of the prepared soil into the **standard cylindrical steel mold** until it is **filled about 40% full**.

- Compact the soil by **applying 25 blows** from a special **5.5 lb (2.49 kg) hammer** that **drops from a height of 12"** (305mm).

- Place a **second layer** of prepared soil into the mold until it is **75% full.** Compact the soil by **applying 25 blows**.

- Place the **3rd layer** of the prepared soil into the mold and compact it by **applying 25 blows** in the same fashion. Thus applied a total of **75 blows of hammer**.

- **Trim the sample** so that the **volume is exactly 1/30** ft^3 **(9.44x10^{-4}**m^3 and then **weight it**.

Soil Compaction



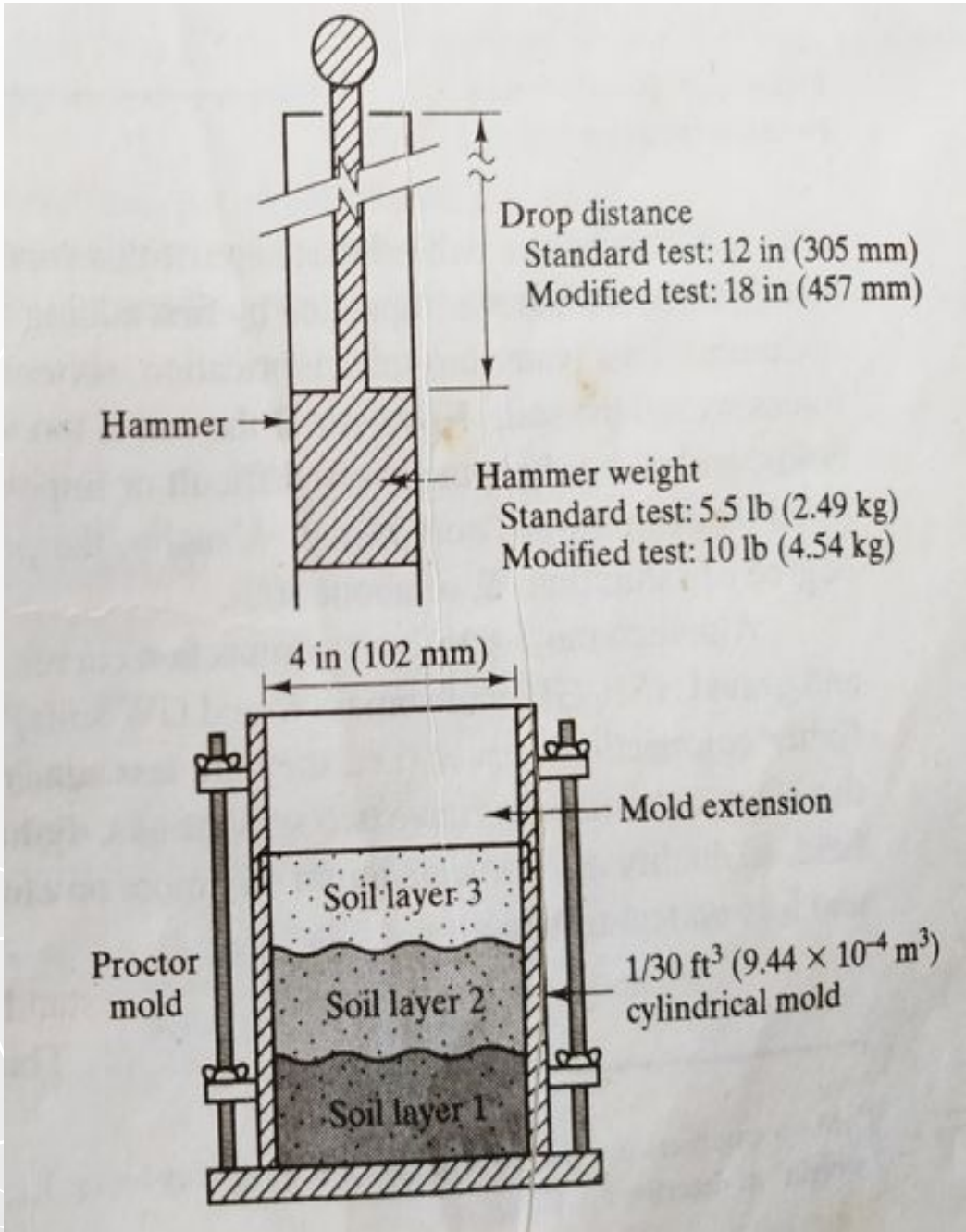


Figure: Mold and Hammer for Proctor Compaction test. Standard Test uses three layers.

- **Unit weight** $\gamma = \frac{W_{ms} - W_m}{V_m}$

where, W_{ms} = weight of mold +soil

W_m = weight of mold

V_m = volume of mold ie. $\frac{1}{30} ft^3$ or $9.49 \times 10^{-4} m^3$

- Perform the **moisture content test** on the **representative portions of the compacted sample,**

$$w = \frac{W_w}{W_s} = \frac{W_2 - W_3}{W_3 - W_1} \times 100\%$$

where, W_1 = weight of can

W_2 = weight of can + wet soil

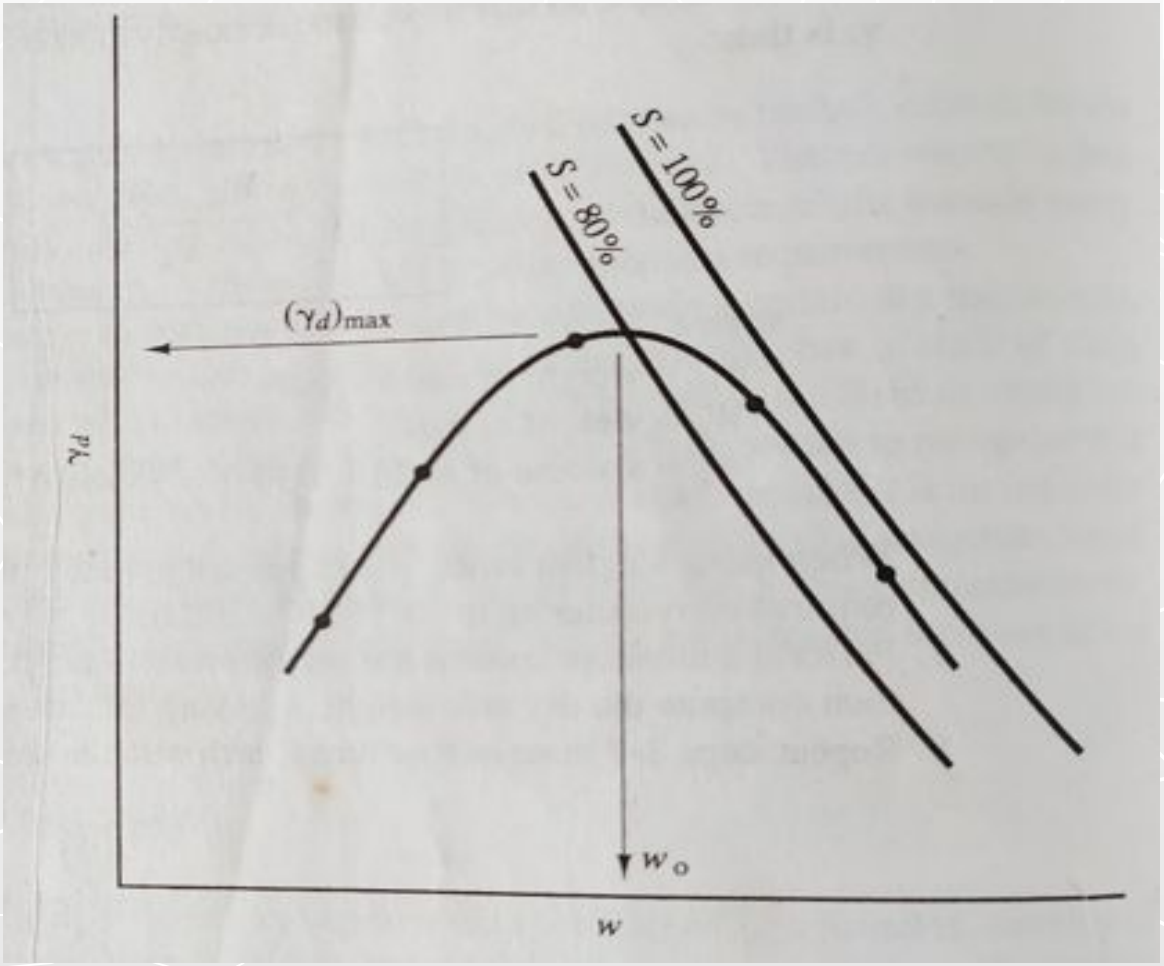
W_3 = weight of can + dry soil ($110 \pm 5°C$ for 24 hours).

- Compute the dry unit weight of the soil sample:

$$\gamma_d = \frac{\gamma}{1+w}$$

- **Repeat the test 3-4 times** with soils at a different moisture content.
- Then plot the graph in **log paper γ_d vs. w%.**

Results from Proctor Compaction Test



What is Proctor Compaction Curve?

- Plot γ_d, achieved by compacting soils at various **moisture content w.**
- Curve connecting data points is called compaction curve.
- In **dry soils, better compaction** is achieved **by adding water** to raise its moisture content to near maximum.
- Water provides **lubrication, softens clay bonds**, and **reduces surface tension forces** within the soils.

Compaction Curve

- Thereby**, expelled air to reduce volume**.
- If soils become **too wet**, no voids is left, very **difficult/impossible to compact.**
- Peak of compaction curve usually at degree of saturation **S=80%.**
- **S=100%** curve known as **zero void curve is out side** the compaction curve.
- **Most of the soils have the similar curves**, but clean sands and gravels do not show such curves.

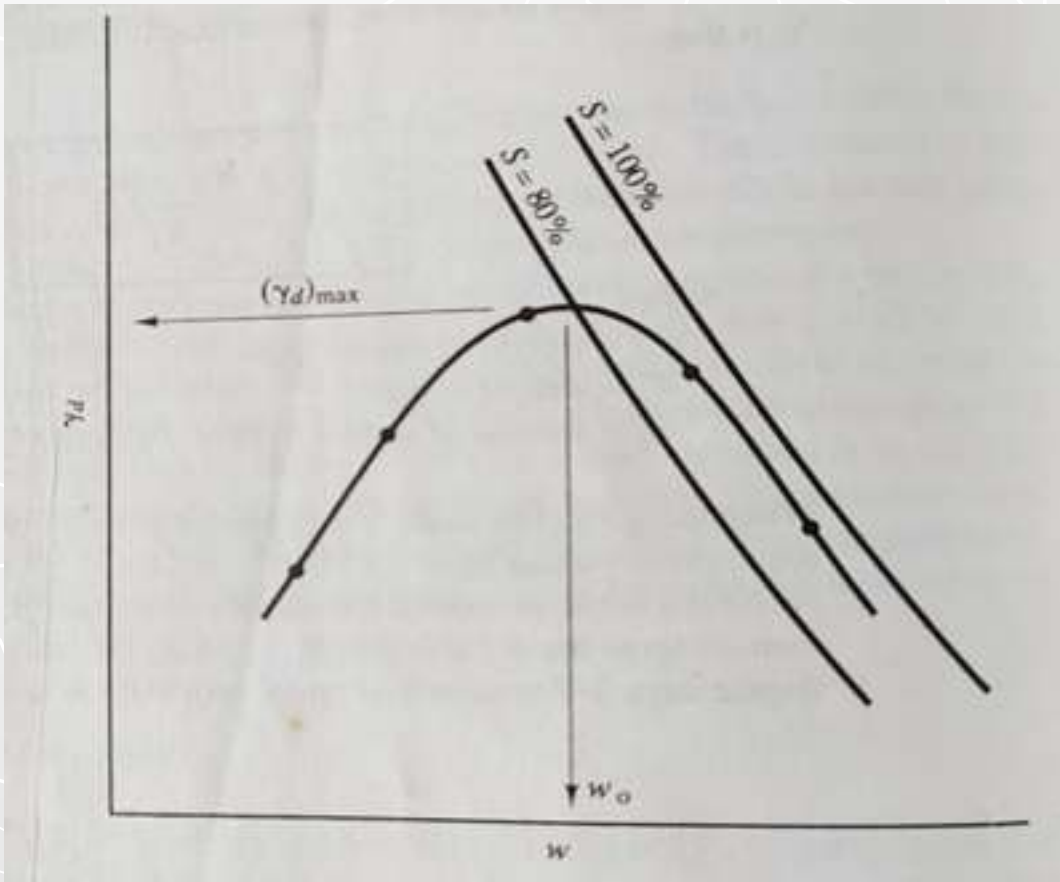
Compaction Curve

- Clean sand and gravel **have better compaction curves because they are less sensitive to moisture contents.**
- Curves of such soils have **some times two peaks** and **compacted easily in the field**.
- So quality assessment of compaction of clean sands and gravels **might rely on observation of contractor procedures** and less on test results.

How to draw saturation Curve?

From the equation, $w = S\left[\frac{\gamma_w}{\gamma_d} - \frac{1}{G_s}\right] \times 100\%$

- **Select γ_d in the range of test data.**
- **Compute the corresponding values of w** for S=80% and S=100%.
- Plot data point.



How the Earthwork Specification are given?

- **From the graph**, get the $\gamma_{d_{max}}$ and moisture content.
- Determine the value of γ_d **from the field after compaction**.
- Then express the degree of compaction ie **Relative compaction**

$$C_R = \frac{\gamma_d}{\gamma_{d_{max}}} \text{x } 100\%$$

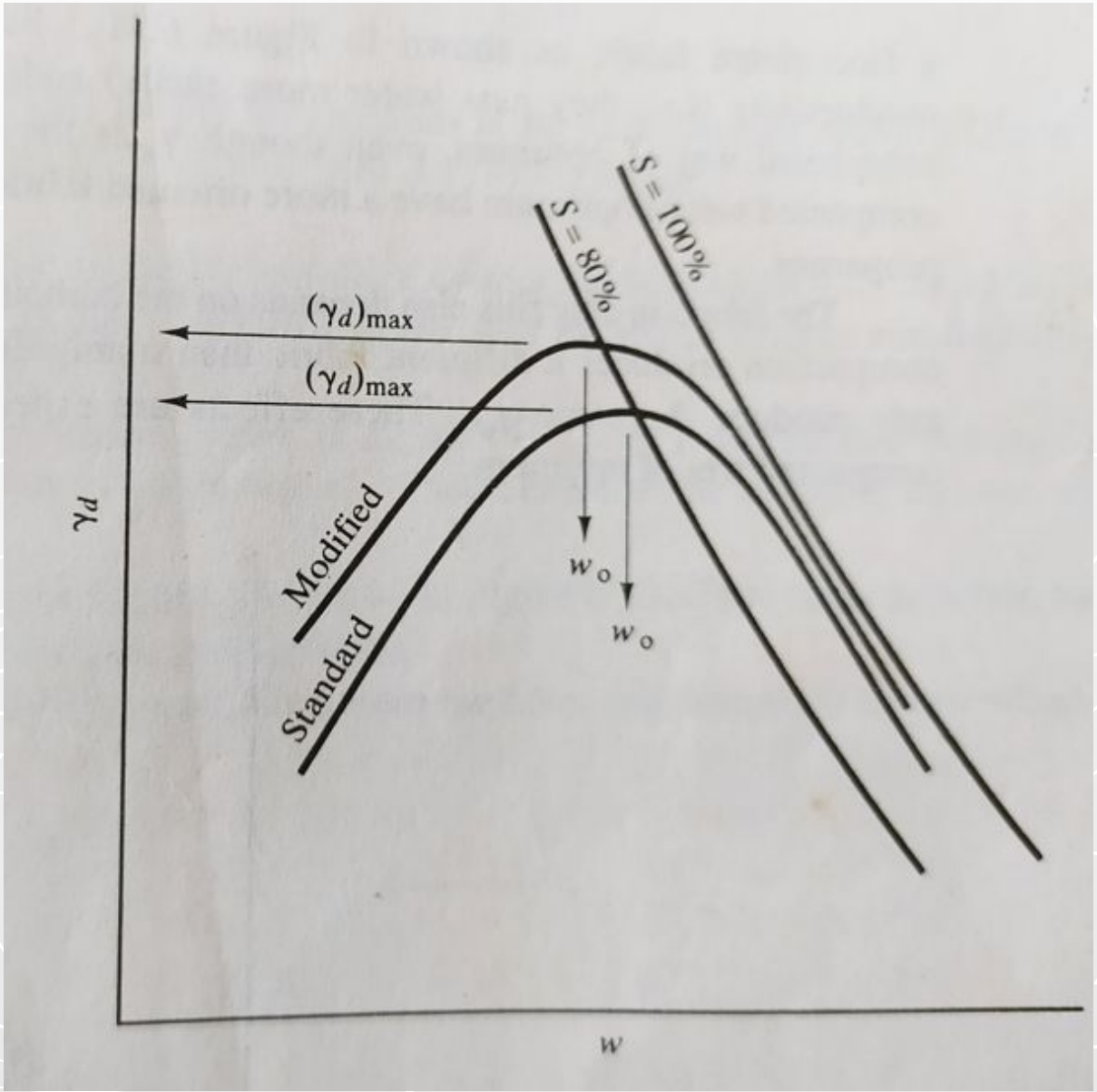
- **Most Earthwork specifications are written in terms of C_R** and require the contractor **to achieve at least certain value of C_R**.

- For example, if a certain soil has $\gamma_{d_{max}} = 120$ **lb/ft^3** and project specification is **required** $C_R \geq 90\%$, then contractor must compact the soils in the field **until** $\gamma_d \geq 108$ **lb/ft^3**.

Typical Compaction Specification

Type of project	Minimum Required Relative Compaction
Fills to support buildings or roadways	90%
Upper 150mm of subgrade below roadways	95%
Aggregate base material below road ways	95%
Earth dams	100%

Comparison Between standard Proctor test and modified Proctor test results of same soil:



- From the figure the term **maximum dry unit weight is misleading** because the standard and the modified tests have two different "maximums".
- For each test, $\gamma_{d_{max}}$ is the greatest dry unit weight achieved for the level of compaction effort (ie. For so many blows of a certain hammer).
- **Neither these values represents the largest possible dry density**.
- Figure also shows, how the optimum moisture content for the modified test is **slightly less** than that for the standard test.

Ensuring Proper Compaction in the Field:

- γ_d achieved in the field will ***generally less*** than $\gamma_{d_{max}}$, even the moisture content is exactly equal to w_0, because the **contractor usually *applies less compactive*** efforts (ie. so many passes of a certain kind of Roller).

- **This is acceptable**, so long as the project specifications permit relative compaction is less than 100%, which they usually do.

- Sometimes, contractors **applies large effort that exceeds** $\gamma_{d_{max}}$, thus producing a relative compaction of more than 100%; specially on earth dam where high level of compaction are required.

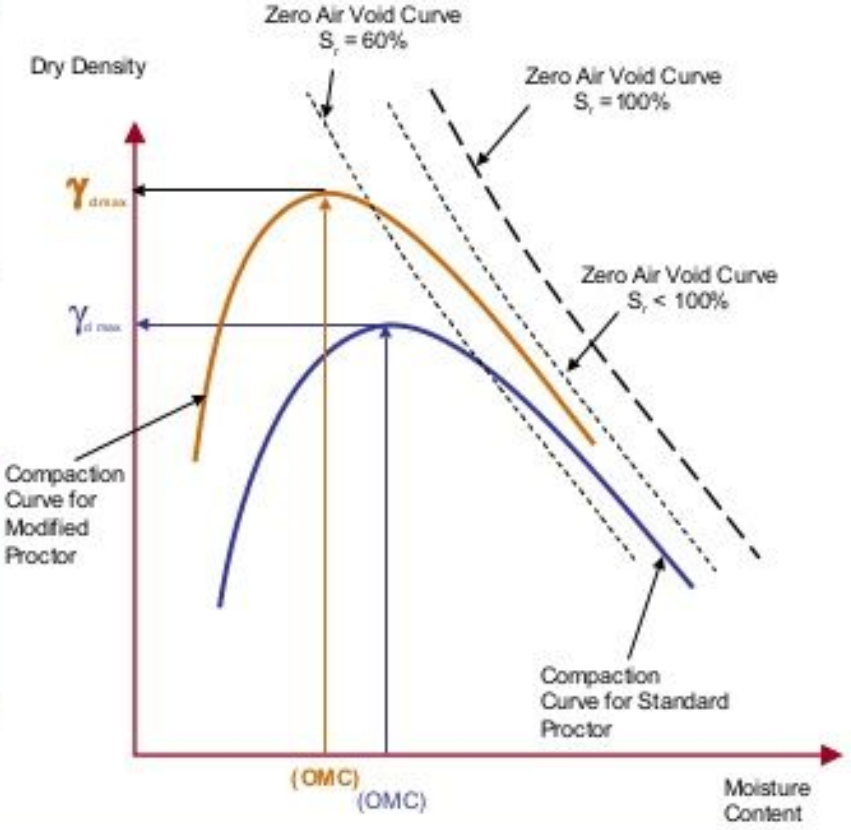
Soil Compaction in the Lab:

1- Standard Proctor Test
ASTM D-698 or AASHTO T-99

Energy = 12,375 foot-pounds per cubic foot

2- Modified Proctor Test
ASTM D-1557 or AASHTO T-180

Energy = 56,520 foot-pounds per cubic foot



$$\text{Energy} = \frac{\text{Number of blows per layer} \times \text{Number of layers} \times \text{Weight of hammer} \times \text{Height of drop hammer}}{\text{Volume of mold}}$$

S=100% and S=80% Curve

- **S=100% curve represent an upper limit** for the proctor data, because it is impossible to compact a soil having S>100%.

- At S=100% **no voids remains for air**, so it is **called zero voids curve.**

- **S=80% curve should go near through the peak** in proctor compaction curve.

- Right limb of proctor curve should **be slightly to the left of S=100% curve,** because compaction does not remove all the air.

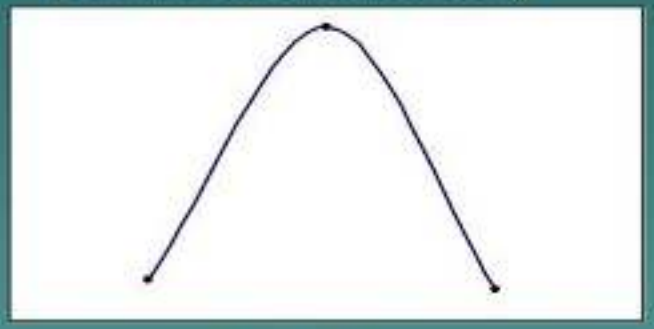
Different Shapes of Compaction Curve

- **Well graded sands** have **better compaction curves** **because they are less sensitive to moisture contents.**
- **Poorly graded sands** have **some times two peaks** and **compacted easily in the field**.
- So quality assessment of **compaction of clean sands and gravels** **might rely on observation of contractor procedures** and less on test results.
- **Silt or poorly graded sand** with silts have **odd shape curve**.
- **Clay** has one and a **half peak curve**.

Compaction Curves for Different Types of Soil

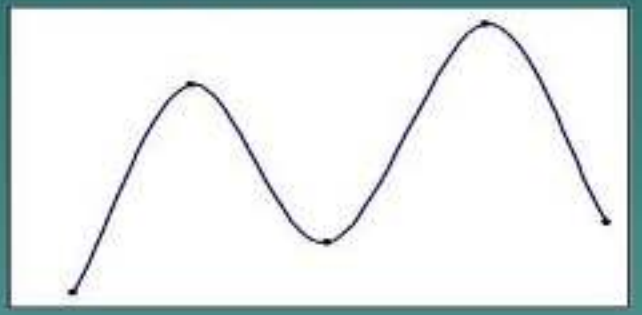
One peak

(Well Graded Sand)



Double peak

(Poorly Graded Sand)



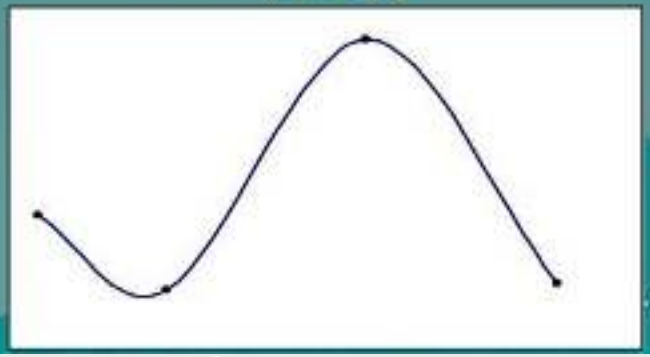
Odd Shape

(Silt OR Poorly Graded Sand with Silt)



One & half peak

(Clay)



Compaction

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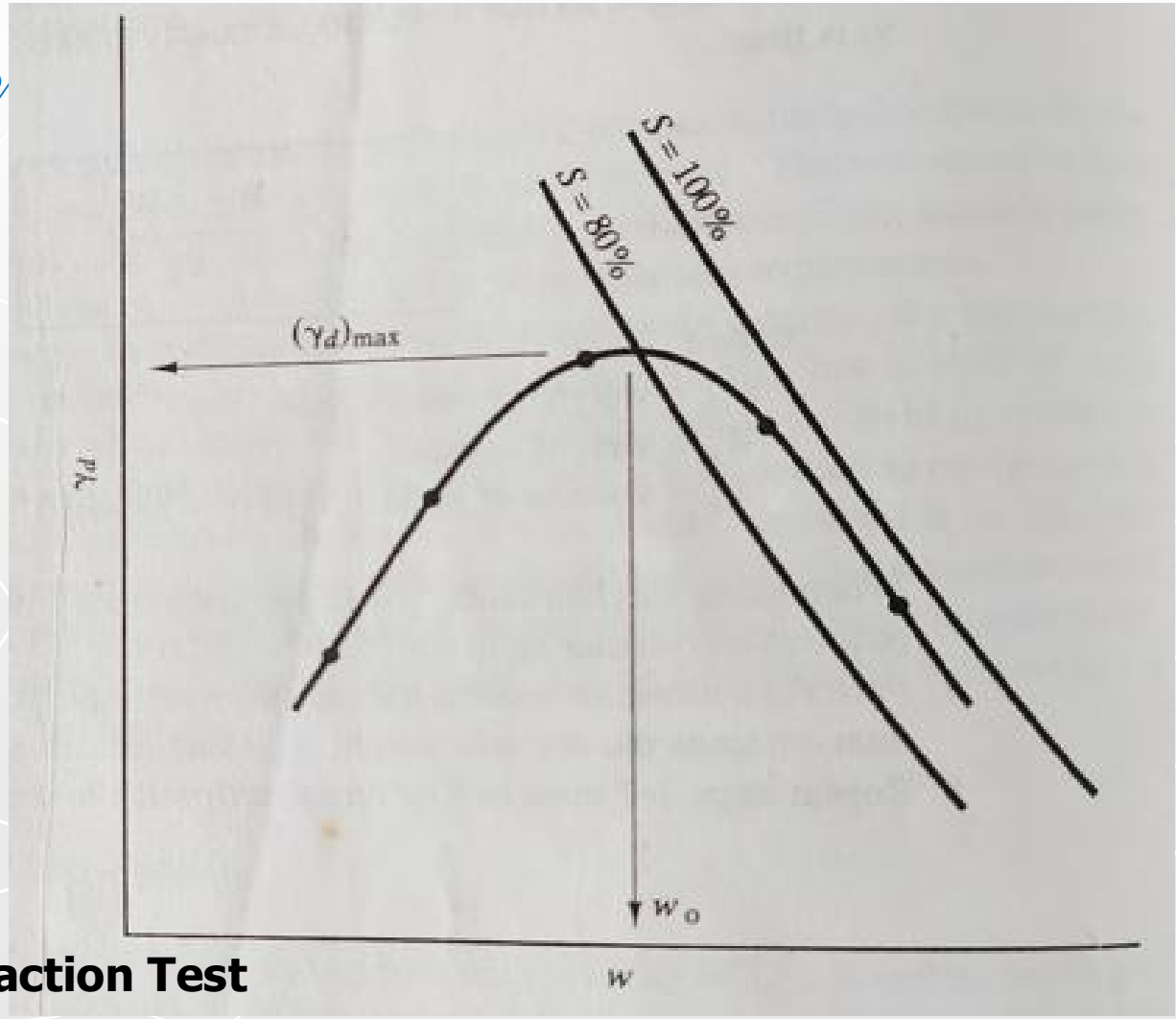
Principles of Soil Mechanics

Soil Compaction

- $\gamma = \frac{W_{ms} - W_m}{V_m}$

$$w = S\left[\frac{\gamma_w}{\gamma_d} - \frac{1}{G_s}\right] x 100\%$$

- $w = \frac{W_w}{W_s} = \frac{W_2 - W_3}{W_3 - W_1}$ x 100%
-
- $\gamma_d = \frac{\gamma}{1+w}$
- Then plot the graph in **log paper γ_d vs w.**
- $C_R = \frac{\gamma_d}{\gamma_{d_{max}}}$ x 100%



Results from Proctor Compaction Test

A proctor compaction test has been performed on a soil sample. The **test result** were as follows:

Data Point nos.	1	2	3	4	5
• **Mass of compacted Soils and mold(kg)**	**3.762**	**3.921**	**4.034**	**4.091**	**4.040**
• Mass of can (g)	20.11	21.24	19.81	20.30	20.99
• Mass of can + wet soil(g)	240.85	227.03	263.45	267.01	240.29
• Mass of can + dry soil(g)	231.32	212.65	241.14	238.81	209.33

Mass of the compaction mold was 2.031kg. Moisture tests were performed on small portion of compacted soils sample.

- Compute γ_d **and w** for each data point and plot these result.
- **Develop and plot curves** for **S =80%** and **S= 100%** using a specific gravity of 2.69.
- Using data from a and b, **draw the proctor compaction curve** and **determine** $\gamma_{d_{max}}$ **and** w_0.

Solution:

Computation for data point no. 1:

For unit weight, $\gamma = \frac{W_{ms} - W_m}{V_m} = \frac{(M_{ms} - M_m)g}{V_m}$

$$= \frac{(3.762kg - 2.031kg)9.81m/s^2}{9.44x\ 10^{-4}\ m^3} \text{x (1N/1000kN)}$$

$$= 17.99\ \text{kN}/m^3$$

For water content, $w = \frac{M_1 - M_2}{M_2 - M_c}$ x100%

$$= \frac{240.85 - 231.32}{231.32 - 20.11} \text{x100\%}$$

$$= 4.5\%$$

$$\gamma_d = \frac{\gamma}{1+w} = \frac{17.99}{1+0.045} = 17.22\ \text{kN}/m^3$$

Thus, **this data point plots** at w =4.5% and $\gamma_d = 17.22\ \text{kN}/m^3$

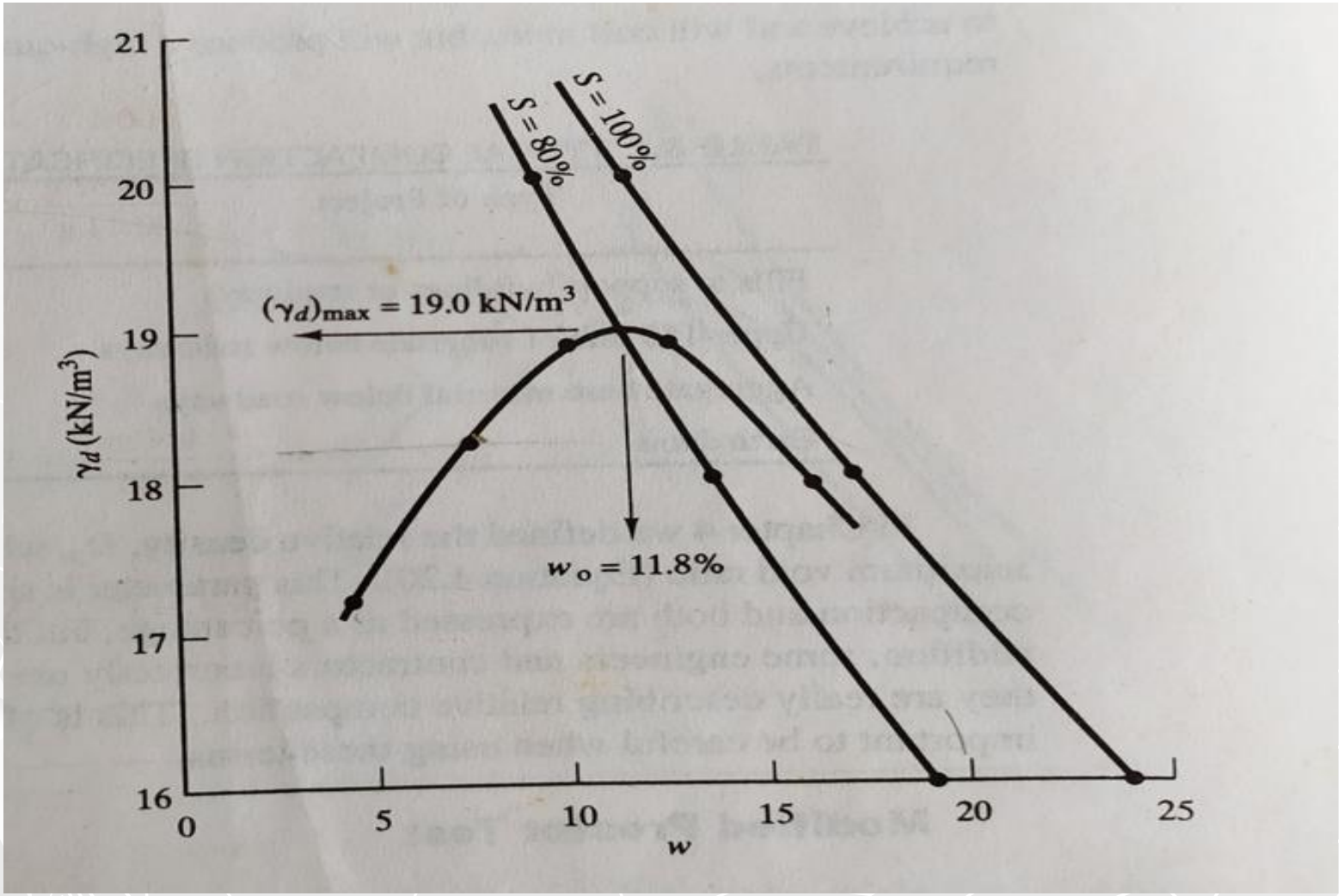
Following the same procedures for other data points gives:

Data point No.	1	2	3	4	5
γ kN/m^3	17.99	19.64	20.81	21.41	20.88
w	4.5%	7.5%	10.1%	12.9%	16.4%
γ_d kN/m^3	17.22	18.27	18.90	18.96	17.94

- From equation, $w = S\left[\frac{\gamma_w}{\gamma_d} - \frac{1}{G_s}\right]$ $\gamma_w = 9.807\ \mathrm{kN/m^3}$
- Select value of γ_d in the range of test data and compute the corresponding values of w foe S=80% and S=100%:

γ_d **kN/m^3**	**w**	
	@ S=80%	@ S=100%
16.00	19.3%	24.1%
18.00	13.9%	17.3%
20.00	9.5%	11.9%

Plots data points and draw the compaction curves.



Final Results are: $\gamma_{d_{max}}$ = 19.0 kN/m^3 and w_0=11.8%

Field Density/Unit Weight Tests:

- Final link in assessing fill compaction by Proctor Compaction Test method is to measure γ_d in the fill after compaction and thus enabling to use equation $C_R = \frac{\gamma_d}{\gamma_{d_{max}}} \text{x} 100\%$

- Several methods have been developed to do so and known as ***Field Density Tests/Field Unit Weight Test***.

- All of these methods are **performed in the field**, thus making it possible to immediately present the test results to the contractor.

- Such **rapid feed back is important**, because contractor must rectify an inadequately compacted zones before they are buried by additional fill.

Sand Cone Test

- One of the most common field density test method.

Procedures:

- Prepare a level surface in the fill and dig a cylindrical hole about 125mm (5 inch) in dia and 125mm (5 inch) deep.
- **Save all the soi**ls that comes out of the hole and determine its weight W.
- **Fill the sand cone apparatus** with special free flowing SP (Southern Specific) sand (Density Test Sand). Sand Cone Test Sand, Model: HM-106)
- Determine **weight of cone apparatus and sand= W_1**

- **Place sand cone over the hole**, open the valve and allow the sand to fill the hole and the cone.
- **Close the valve**, remove the sand cone from the hole and **determine the new weight =W_2** .

Volm of hole, $V=\frac{W_1 - W_2}{\gamma_{sand}}$ - Vcone

$$\gamma = \frac{W}{V}$$

- **Need to measure w**
- $\gamma_d = \frac{\gamma}{1+w}$

$$CR = \frac{\gamma_d}{\gamma_{d_{max}}} \times 100\%$$

Where,

V= volume of test hole

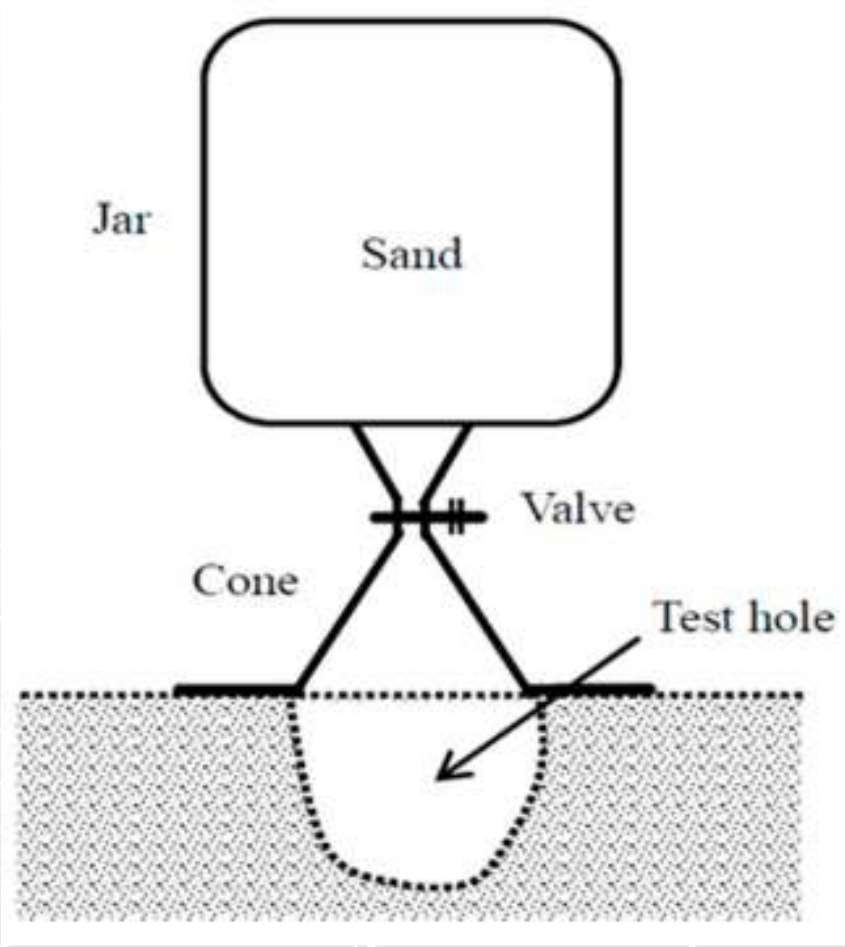
W_1= Initial weight of sand cone apparatus

W_2= Final weight of sand cone apparatus

γ_{sand}= unit weight of sand used in sand cone

V_{cone}= volume of sand cone below the valve

W= weight of soil removed from the test hole.


Jar
Sand
Valve
Cone
Test hole


gettyimages
JEFFREY_JK



- Naturally **rounded silica sands** of nearly **pure quartz** are mined from the Ottawa, Illinois area. Three different grading's are available, each designed for a specific type of test.

- **HM-106 Density Test Sand** (Sand Cone Test Sand) for sand cone density testing of soils is clean, uniform, un-cemented and free-flowing, and meets **ASTM D1556/1556M** and **AASHTO T 191**.



Determination of $w\%$ in the field:

- **Stir-frying the soil sample** in a **frying pan** over **a portable propane stove**
- **Speedy Moisture Tester**
- **Drive Cylinder Test**
- **Water Ring Test**

Measurement of Moisture Content:

- To measure moisture content, normally it requires placing the soils sample in the oven overnight, which is unacceptably slow.

Special rapid methods to measure the moisture content in the field:

- **Stir-frying the soil sample** in a **frying pan** over **a portable propane stove.**

  -Method is fast and gives **reasonably accurate results** in clean sand and gravel.

  -Much hotter than laboratory oven, thus producing **erroneous results in clay and organic soils**; w will be too high.

Speedy Moisture Tester:

- Mixing the soil with **calcium carbide** in a **special sealed container** known as a ***speedy moisture tester***.

  -Water in the soil reacts with CaC_2 to produce **acetylene gas (C_2H_2)** and reaction continues until water is depleted.

  -A **pressure gauge measures** the resulting pressures generated by the production of gas and calibrated to **compute w.**

  -Results are **not as accurate** as a standard laboratory moisture test, but are usually sufficient for field density tests.

 # A sand cone test has been performed in the compacted fill of which $\gamma_d$max= 19.0 kN/m³ . The test results are as follows:

Initial mass of sand cone apparatus = **5.912** kg

Final mass of sand cone apparatus = **2.378** kg

Mass of soil recovered from the hole = **2.883** kg

Moisture contents of soil from hole **= 7.0%**

Density of sand = 1300 kg/m³.

Volume of cone below the valve = **1.114x 10^{-3} m³**

The project specification requires to a relative compaction of at least **90%.**

Compute γ_d and C_R, and determine whether project specification has been meet.

Solutions:

$M_{sand\ (cone+hole)} = 5.912\ \text{kg} - 2.378\text{kg} = 3.534\ \text{kg}$

$$V_{cone+hole} = \frac{(M_{sand)\ cone+hole})}{\rho_s} = \frac{3.534\ kg}{1300 kg/m^3} = 2.718\text{x}\ 10^{-3} m^3$$

$$V_{hole} = V_{cone+hole} - V_{cone} = 2.718\text{x}\ 10^{-3} m^3 - 1.114\text{x}\ 10^{-3} m^3$$

$$= 1.604\ \text{x}\ 10^{-3} m^3$$

$W_{soil} = M_{soil}\ g = 2.883\text{kgx}\ 9.81\text{m/s}^2\ \text{x}\ 0.001 = 2.828\text{x}10^{-2}\text{kN}$

$$\gamma = \frac{W_{soil}}{V_{hole}} = \frac{2.828\text{x}10^{-2}\text{kN}}{1.604\ \text{x}\ 10^{-3} m^3} = 17.63\ \text{kN}/m^3$$

$$\gamma_d = \frac{\gamma}{1+w} = \frac{17.63\ \text{kN}/m^3}{1+0.07} = 16.5\ \text{kN}/m^3$$

$$C_R = \frac{\gamma_d}{\gamma_{(d)\,max}} = \frac{16.5\ \text{kN}/m^3}{19.0\ \text{kN}/m^3} = 86.8\%$$

Nuclear Density Test (ASTM D2922)

- Consists of **Special Device** which **emits gamma rays** and **detects how they travel through soils**
- **Amount of gamma rays received back** into the device **correlates** with unit weight of soils.
- Measurements of w and γ **depends on empirical correlation** and which must be ultimately be programmed into the device.
- Results **display on digital electronic** **re**adouts.
- In spite of its use of hi-tech equipment, this method is **slightly less accurate** than sand cone, because it is based on empirical correlations with transmission of radiation.



Performing nuclear density test



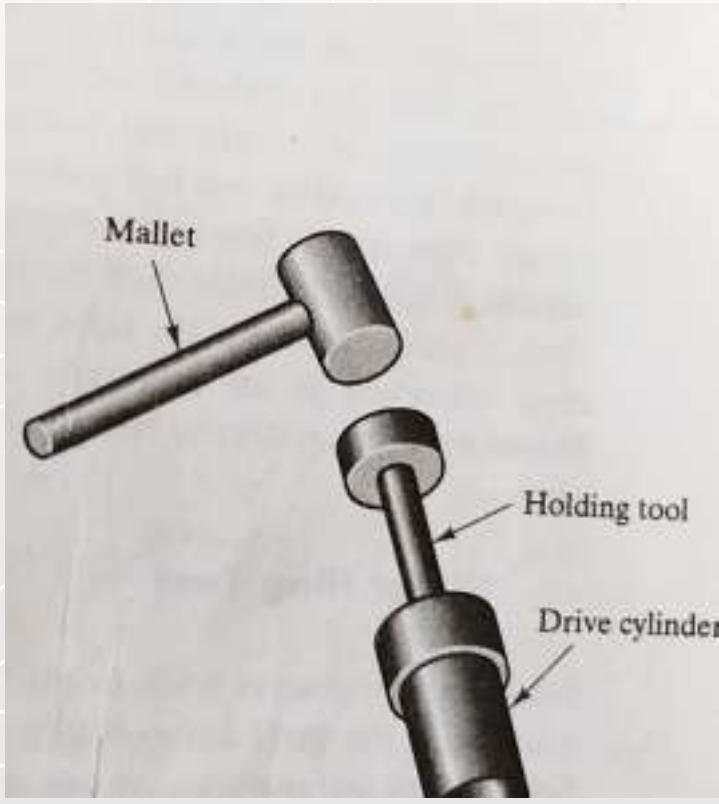




Performing nuclear density test

Drive Cylinder Test (ASTM D2937):

- Consists of **driving a thin-wall steel tube** in to the soil using **special drive head** and **a mallet**.
- **Cylinder is then dugout** of the fill using a shovel, soil is **trimmed smooth** and it is **weighed**.
- γ of the fill is the computed based on this **weight** and the **volume** of the cylinder.
- Then **w is determined**.
- **Much faster** than sand cone but slightly less precise.
- **Only suitable** for **fills with sufficient silt and clay** to provide enough dry strength to keep the sample inside the cylinder.

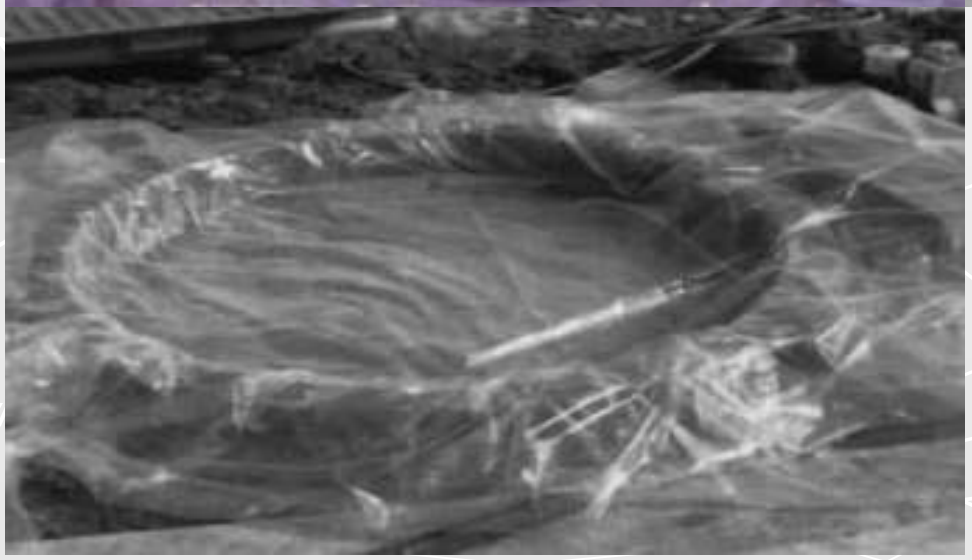


Water Ring Tests (ASTM D5030):

- Sometimes **engineers wish to place fills** that **contains cobbles** (2.5–10.1 inch) and **small boulders**.
- Some **earth dams contains** zones made of such soils.
- **Conventional field density test methods are not applicable** because of the large particle sizes.
- One way to evaluate such fills is the **water ring test**.
- Test uses a **much larger sample** and **uses water** instead of sand to measure the volume.

- Holes has been dug (a ring is **typically 1800mm in diameter)** and excavated soils has been **weighed**.
- Hole has been lined with plastic and **filled with water to measure its volume.**








Thank you

EWCE 341

Principles of Soil Mechanics

EWCE 341

Stresses in Soils

EWCE 341

Stresses in soils

What we are going to discuss today?

- What is stress in soils?
- Stresses on a plane
- Types of stresses
- What is shear failure in soils?
- Types of shear failures
- Computation of vertical stress
- Problem

What is stress in soils ?

Stress:

- All civil Engineering projects **impart** loads on the ground that supports them.
- Loads produces **stresses in soils** ie **compressive stress**, **shear stress** and **even tensile stress**.
- A **stress** on the **soil** depends on the **load per unit area**.
- Construction of a foundation mainly increases the **stresses** on the **soil**.
- As the **stress** increases in the **soil**, the **soil** can be deformed by the **stress**. There are two **stresses** can acts on a plane: normal **stress** (σ) and shear **stress**(τ) .

Why stresses in soils are important to the Geotechnical Engineers?

Stresses in Soils

Induce Stresses:

- When a building is constructed, its weight is transmitted to the ground through the foundation and thus **induces stresses** in the under lying strata.

Additional Stresses:

- **Additional stresses** exist in the ground due to the weight of the overlying soils.

Importance of stresses to Geotechnical Engineers:

- **Induce stresses** might cause problems such as **shear failures** or **excessive settlement**.
- **Additional stresses** might cause problems such as **land slide**, **stability of slopes**.
- Both **depends on the differences** between these **stresses and the strength of soils.**

Shear Failure in Soils

- A shear failure is **defined as a foundation failure** that occurs when the **shear stresses in the soil exceed the shear strength of the soil.**

- When the **ultimate bearing capacity** of the soil is reached, it may fail in one of the following **three failure mode** depending upon the **type of soil** and **depth to width ratio** of the footing (i.e. D/B):

  -General Shear Failure
  -Local Shear Failure
  -Punching Shear Failure

General Shear Failure:

A slight downward movement of the footing **develops fully plastic zones** and a **sudden failure takes place** with a **considerable bulging** of the ground surface adjacent to the footing.

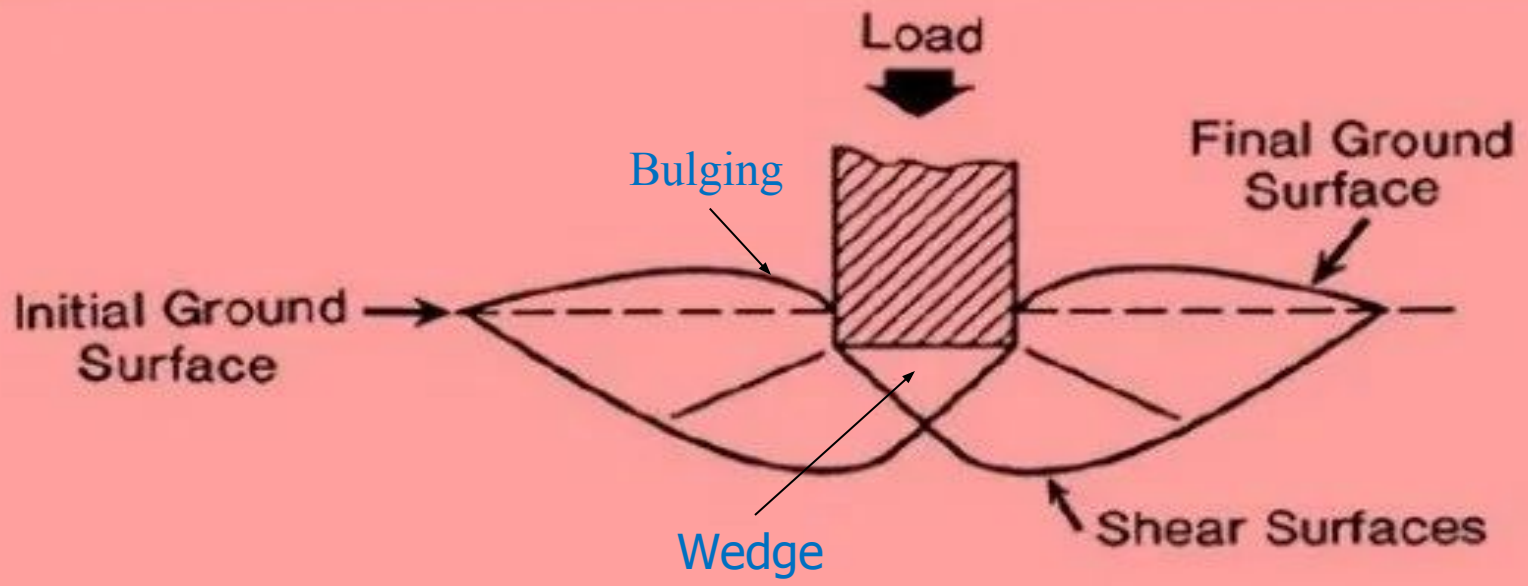
Characteristics of General Shear failure:

- **Well defined failure pattern**, consisting of a **wedge** and **slip surface** and **bulging (heaving) of soil** surface adjacent to the footing.

- **Sudden collapse occurs**, accompanied by tilting of the footing.

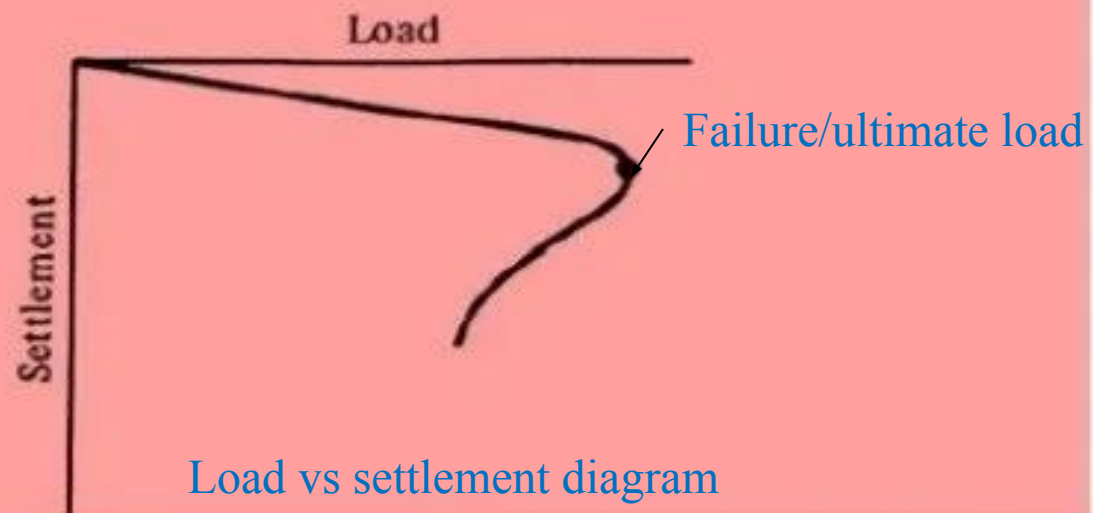
- Occurs in case of **dense sand or stiff cohesive soil** supporting the footing.
- **Failure load is well defined** in this case.
- **Load-settlement diagram** is similar to stress-strain for dense sand or over-consolidated clay as shown.
- **Ultimate load is well defined**.

General Shear Failure



Dense sand or stiff cohesive soils

/Slip surface



Load vs settlement diagram



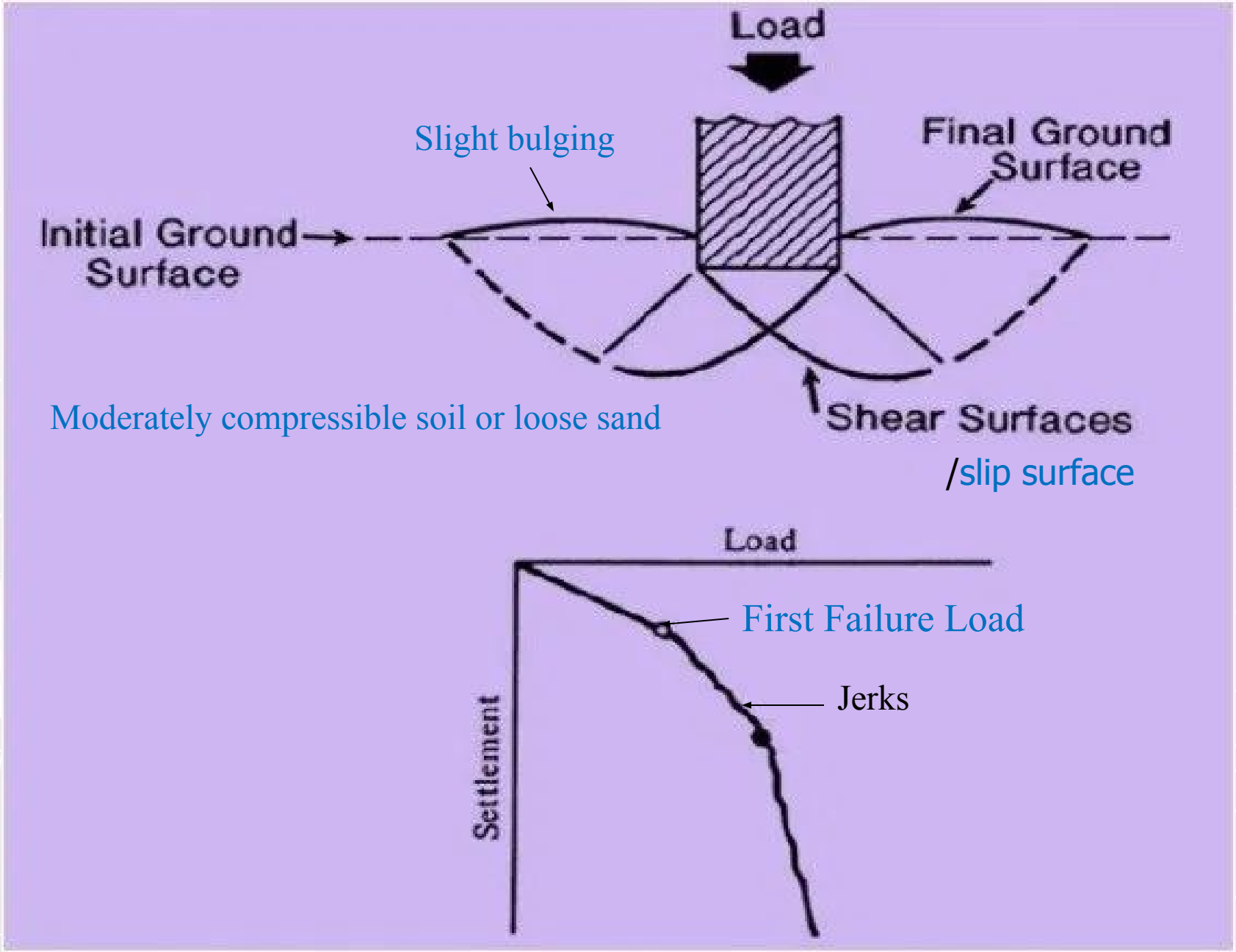
Local Shear Failure:

- Failure pattern consists of **wedge** and **slip surface** but is **well defined only under the footing**.
- **Slight bulging** of soil surface occurs:
- A **large deformation takes place under the footing** before the development of failure zones, i.e. large vertical settlement takes place before slight bulging of the ground surface.

Characteristics of Local Shear Failure:

- **Ultimate load is not well defined.**
- Occurs in **moderately compressible soils or loose sand**.
- **Yielding takes place** close to the lower edges of the footing.
- **Several yield developments may occur** accompanied by settlement in a series of **jerks**.
- Bearing pressure at which the first yield takes place is referred to as the **first-failure pressure or first failure load**.

Local Shear Failure







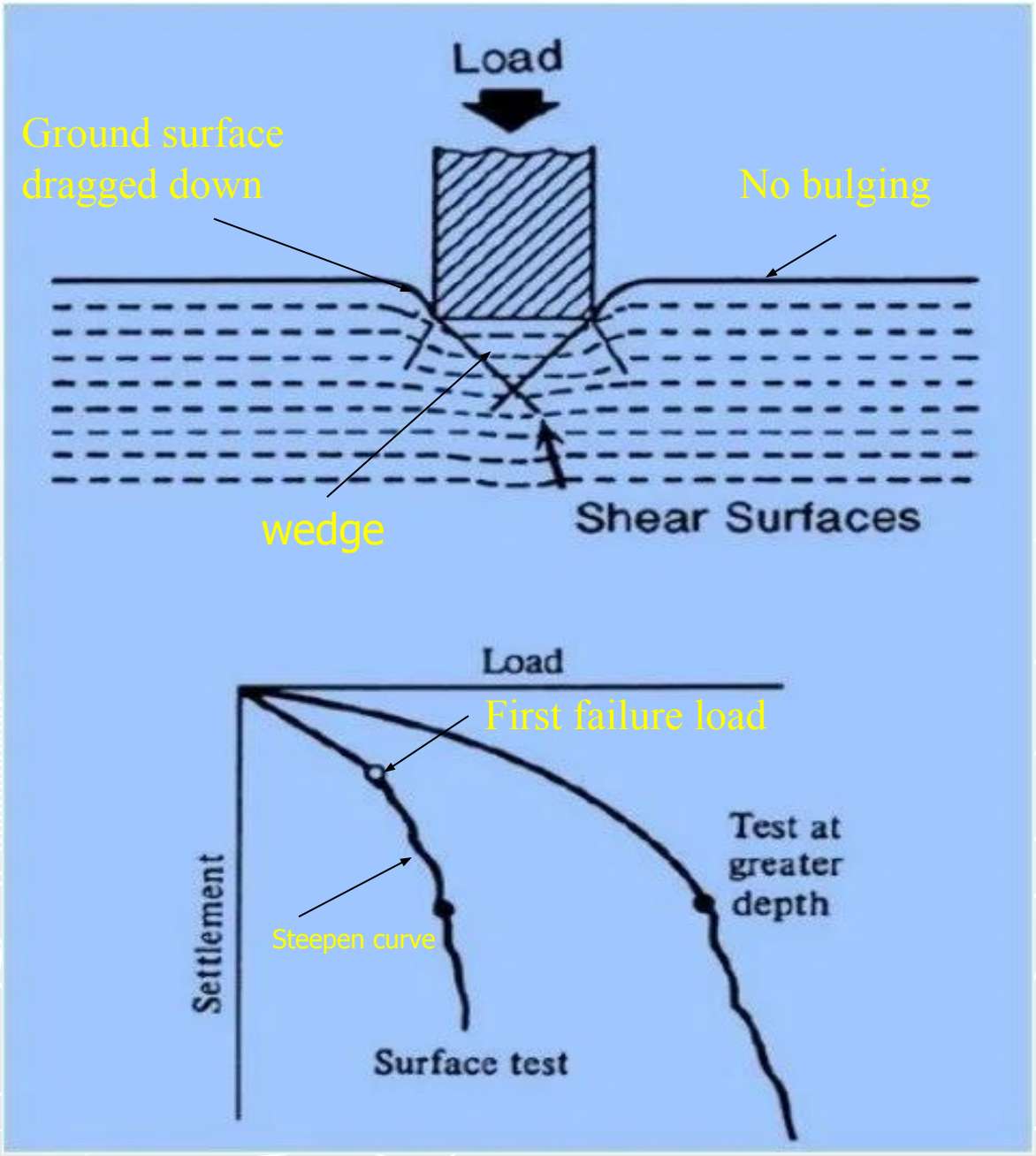
Punching Shear Failure:

- **Failure pattern is not well defined** in punching shear failure of soil.
- Failure takes place in weak compressible soils with considerable vertical settlement.

Characteristics of Punching Shear Failure:

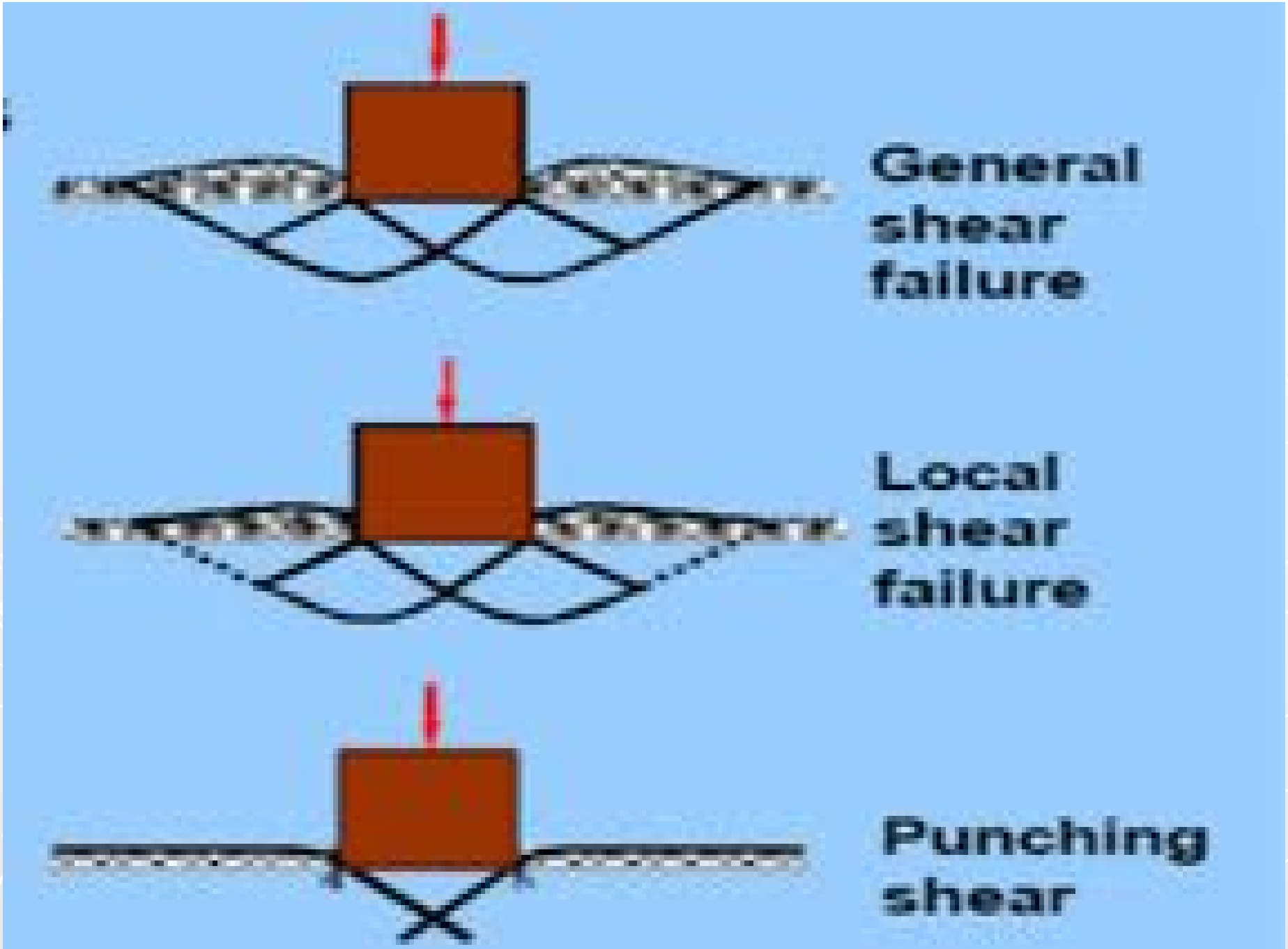
- **Yield surfaces are** adjacent to the sides of the foundation.
- **Failure take place immediately below footing** and surrounding soil remains relatively unaffected
- **Ground surface may be dragged down** thus, **no bulging** of the surface takes place.
- **Large settlements-ultimate load is not well defined**.
- After the first yield the **load-settlement curve** will steepen slightly.

Punching Shear Failure



Punching Shear Failure




General shear failure
Local shear failure
Punching shear

Land slide



Slope Stability



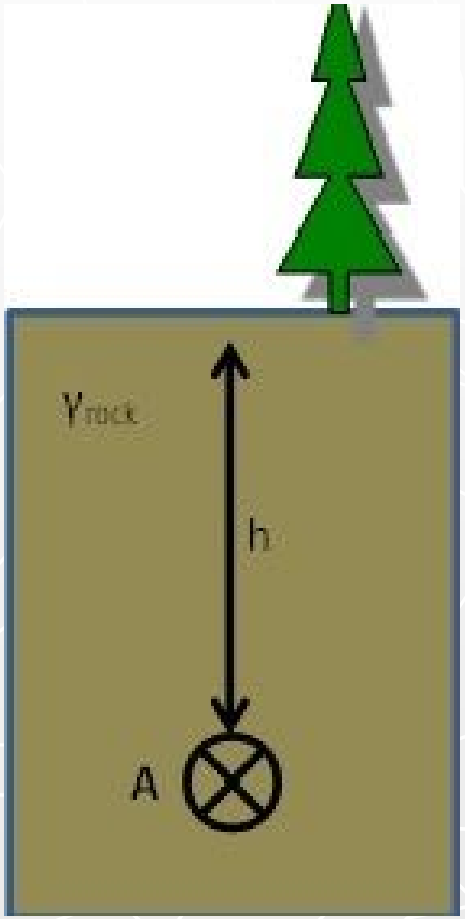
Describe the different sources of stresses in the ground.

Sources of Stresses in the Ground: Two broad categories:

Geostatic Stresses:

- Sometimes called **body stresses.**

- Occur due to the **weight of soil** above the point of being evaluated.



- **Naturally present** in the ground.

- Ac**tivity**, like placing of fills or making an excavation can cause them change.

- **Induced Stresses:**

  -Caused by the **external loads**.

  -Example: Structural foundation, vehicles etc.

  -**Usually caused by human activity**.

Only consider static stresses.

What are the geostatic stresses? Why these are important to geotechnical engineers?

Geostatic Stresses:

- **Caused by the gravity**, acting on the soils or rocks, so the **direct result** in a vertical stresses.
- Has **significant impact** on the engineering behavior of soils and frequently need to be computed.
- **Vertical normal stress** indirectly produces horizontal normal stress and shear stress.
- **Normal stress and shearing stress** are important to geotechnical engineers.

Geostatic stresses may be:

- Vertical stresses
- Horizontal Stresses
- Shear stresses.

How do you compute the geostatic vertical stress at a particular point in the ground?

Computation of Vertical Geostatic Stresses:

- Compute geostatic vertical stress at point P.
- Consider a column of soil that extends from the ground surface down to the point P.
- Column intercepts soil strata with unit weight $\gamma_1, \gamma_2, \gamma_3$

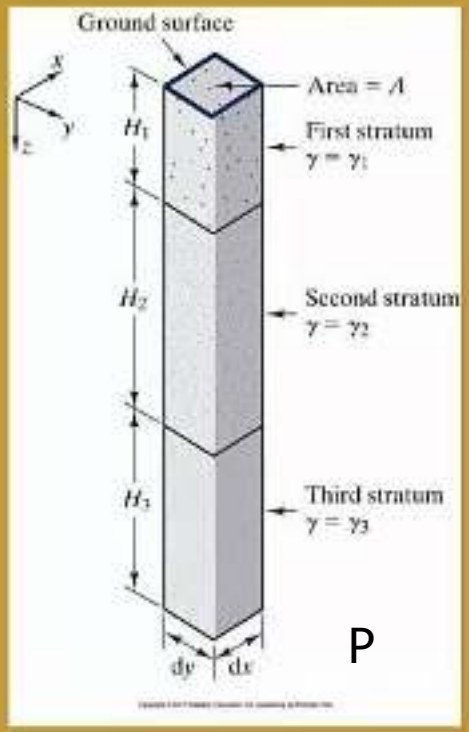
so weight of the soil column:

$$W = dxdy \sum \gamma H = A \sum \gamma H$$

- Geostatic vertical stress at bottom of the soil column,

$$\sigma_z = \frac{W}{A} = \frac{A \cdot \sum \gamma \cdot H}{A} = \sum \gamma H$$

Computing vertical geostatic stress



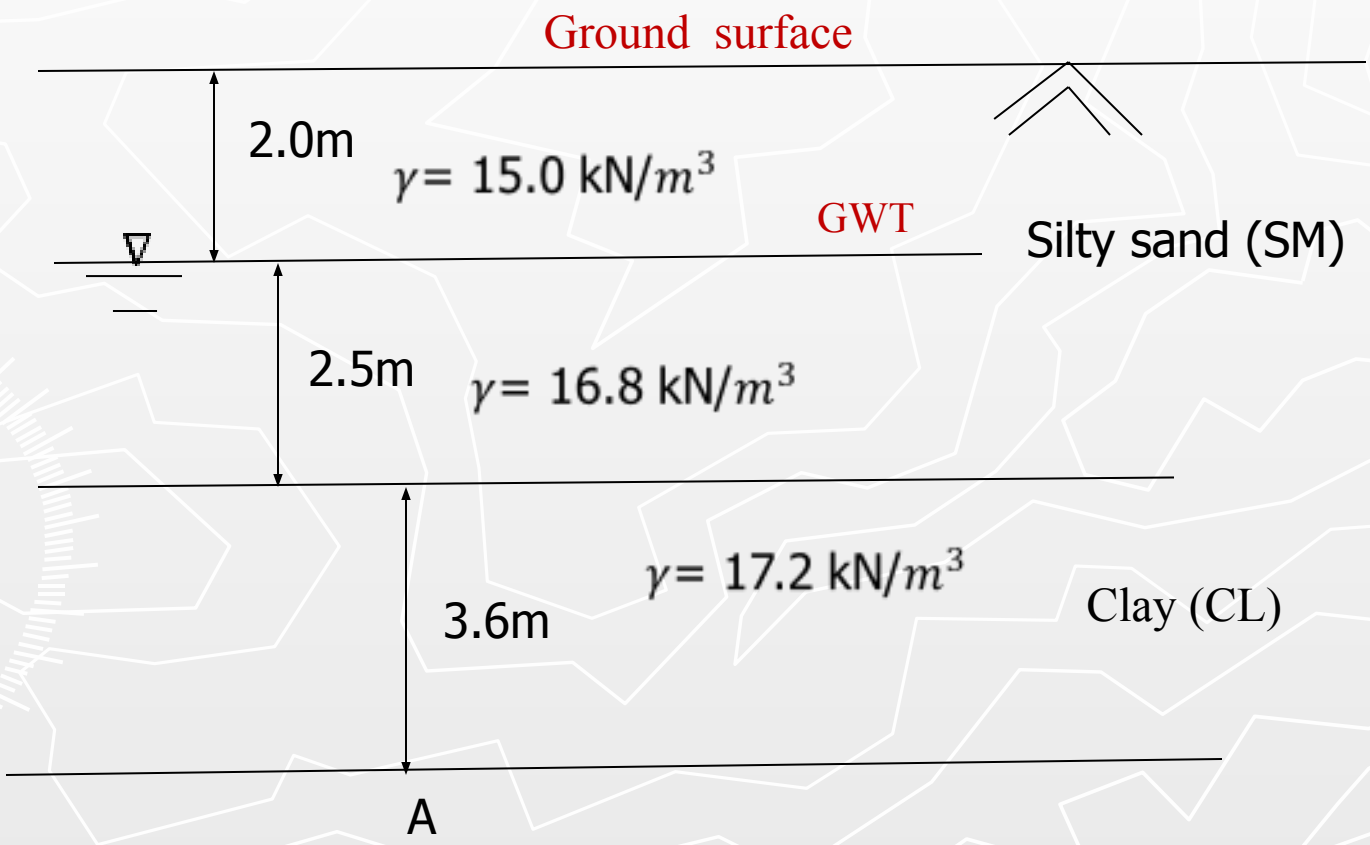
- Total weight of soil column, W

$$W = \sum V_i \gamma_i$$

$$V_i = H_i dx dy$$

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Compute vertical normal stress at point A, shown in the following figure (soil profile).



Solution:

$$\sigma_z = \sum \gamma H = 15.0 \times 2.0 + 16.8 \times 2.5 + 17.2 \times 3.6$$
$$= 133.92 \approx 134 \text{ kN/m}^2 \text{(kPa)}$$

EWC 341

Stresses in soils

What we have discussed today?

- What is stress in soils?
- Stresses on a plane
- Types of stresses
- What is shear failure in soils?
- Types of shear failures
- Computation of vertical stress
- Problem

Thank you

EWCE 341

Principles of Soil Mechanics

Stresses in Soils

What we learned in the last class?

- What is stress in soils ?
- Shear Failures in Soils.
- Sources of stresses in soils
- Computation of Vertical Geostatic Stress.

What we are going to learn today?

- What is stress in soils ?
- Simplifying assumptions for computation of stresses.
- Designation of stress and strain
- Elastic parameters of soils ie. E, G and ν
- Mohr Circle

What is stress in soils ?

Stress:

- All civil Engineering projects impart loads on the ground that supports them.
- **Loads produces stresses** in soils ie compressive stress, shear stress and even tensile stress.

Simplifying Assumptions

For **computing stresses, soils or rocks will have to have following assumptions:**

- It is **continuous material** or not a particulate materials.
- It is **homogenous** ie the relevant engineering properties are **same at all locations**. In stress analysis E, G and ν to be constant ie. there are no '**hard spots**' or '**soft spots**' which can significantly alter the stress distribution.
- It is **isotropic** ie the engineering properties are same in all directions with exceptional is the bedded sedimentary rocks.
- It has **linear elastic stress-strain properties** ie stress is proportional to strain.

Homogeneous vs Isotropic

Homogeneous refers to the uniformity of the structure of the matter.	Isotropic refers to the property of having uniform in every direction.
Structure of homogeneous material is uniform.	Structure of isotropic material can be either homogeneous or non-homogeneous.
Properties of homogeneous matter does not depend on the direction.	Properties of isotropic matter depends on the direction.
Air, clear solutions are example.	Water, glass are examples.

Designation of Stress & Strain

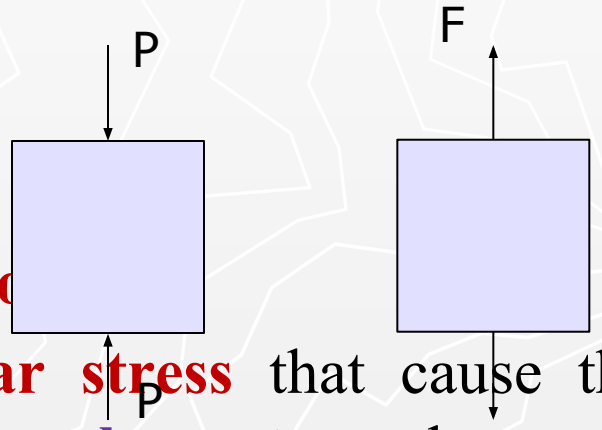
Stress:

- **Each face of the element of a material** is **subjected to a normal stress**, which could be either tension or compression.
- Geotechnical engineers use:

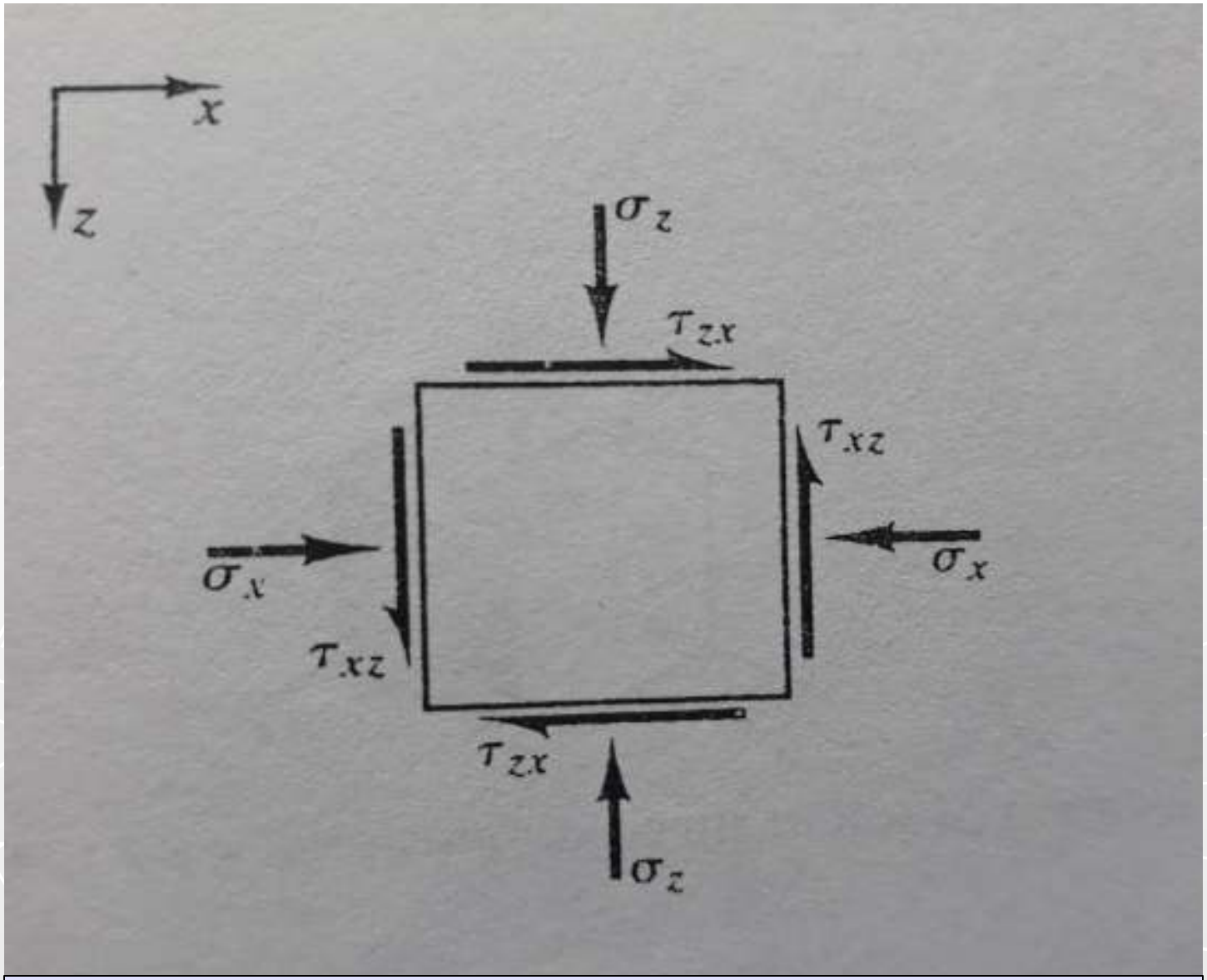
  + = compression

  - = tension

Because they **always deal with the compressio**



- For two dimensional problems, **+ve shear stress** that cause the element to **rotate counter clockwise and –ve shear stress** that cause it to rotate clock wise.



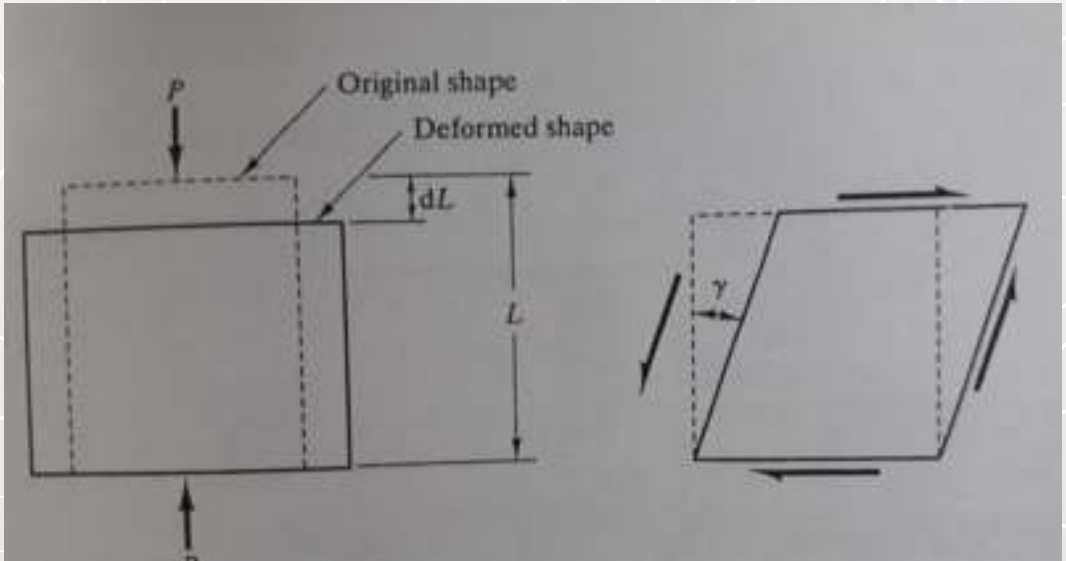
A two dimensional soil element with x-axis and z-axis

Strain:

- When material are **subjected to a stress**, **they respond to deformation** called strain.
- Normal stresses produce normal strain and shear stresses produce shear strain.
- **Normal Strain**, ϵ, is the change in length divided by the initial length.

$$\epsilon = \frac{dL}{L}$$

- **Shear Strain**, γ, is the angle of deformation, expressed in radians.



Modulus of Elasticity, Shear Modulus and Poisson's Ratio: In stress analysis E, G and ν to be constant

Modulus of Elasticity (E):

- Also known as Young's Modulus.

(the modulus takes its name from British scientist Thomas Young, who described its calculation in his *Course of Lectures on Natural Philosophy and the Mechanical Arts* in 1807).

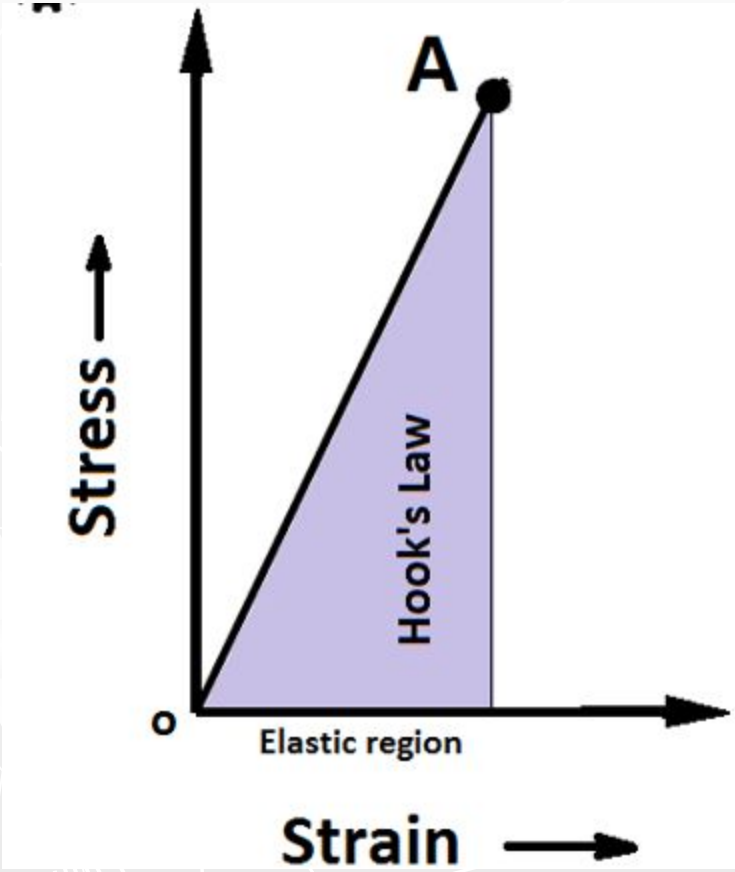
- $E = \frac{\sigma}{\epsilon} = \frac{Normal\ Stress}{Normal\ Strain}$

$\sigma = \frac{P}{A}$ $\epsilon = \frac{dL}{L}$

Large value of E indicates a material that is **very stiff** and does not experience much deformation under applied load.

Type of Soil	Strength of soil	Elastic modulus, E (MPa)
Fine sand	Loose	5–10
	Medium	10–25
	Dense	25–50
Silt	Soft	<10
	Stiff	10–20
	Hard	>20
Clay	Very soft	<3
	Soft	2–7
	Firm	5–12
	Stiff	10–25
	Very stiff	20–50
	Hard	40–80

Hook's Law: Law stating that the strain in a solid is proportional to the applied stress within the elastic limit of that solid.



$$\text{Stress} \propto \text{Strain}$$

$$\sigma \propto \varepsilon$$

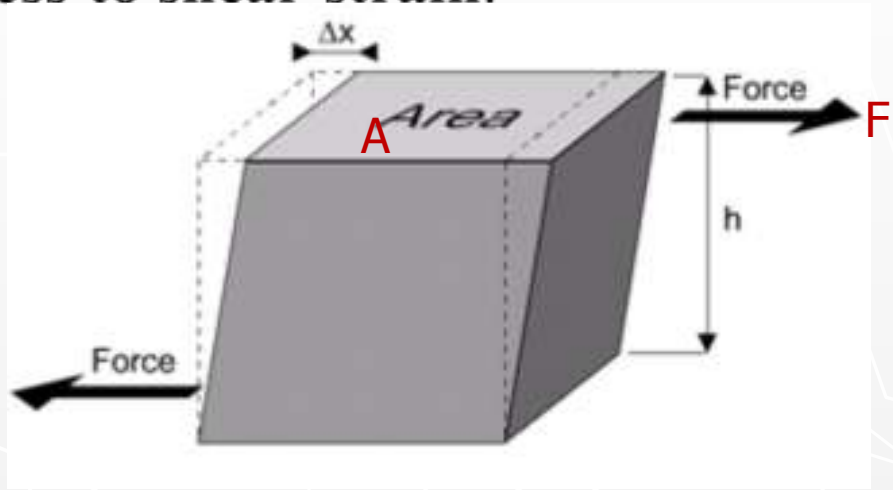
$$\sigma = E\,\varepsilon$$

$$E = \frac{\sigma}{\varepsilon}$$

* E = Young's modulus of elasticity

Shear Modulus (G or η):

- Also known as **modulus of rigidity**.
- Defined as **the ratio of shear stress to shear strain**.
- May be denoted by G .



$$G = \frac{\tau}{\gamma} = \frac{Shear\ Stress}{Shear\ Strain} = \frac{\frac{F}{A}}{\frac{\Delta x}{h}}$$

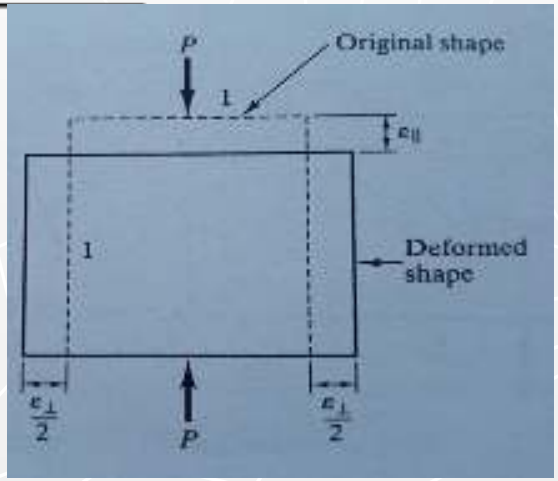
$$= \frac{\tau}{\gamma} = \frac{E}{2(1+\nu)}$$

- **A large** value indicates **a soil is highly rigid**.
- A **small G value** indicates a **soil is soft or flexible**.
- One **definition of a fluid** is **a substance with a shear modulus of zero.**

Soil Type	Shear Modulus, (kPa)
Dense Sands & Gravels	69,000 - 345,000
Silty Sand	27,600 - 138,000
Medium Stiff Clay	6900 - 34,500
Soft Clays	2750 - 13,750

Poisson's Ratio :

- **Figure shows** the deformation of an unconfined element.
- Compressive strain parallel to stress is $\epsilon_{||}$
- Tensile strain perpendicular to stress $\epsilon_{\perp}$
- $$\nu = \frac{\epsilon_{\perp}}{\epsilon_{||}} = \frac{Strain\ in\ the\ direction\ perpendicular\ to\ load}{Strain\ in\ the\ direction\ of\ load} = \frac{lateral\ strain}{Longitudinal\ strain}$$



$\nu = 0.0 - 0.5$

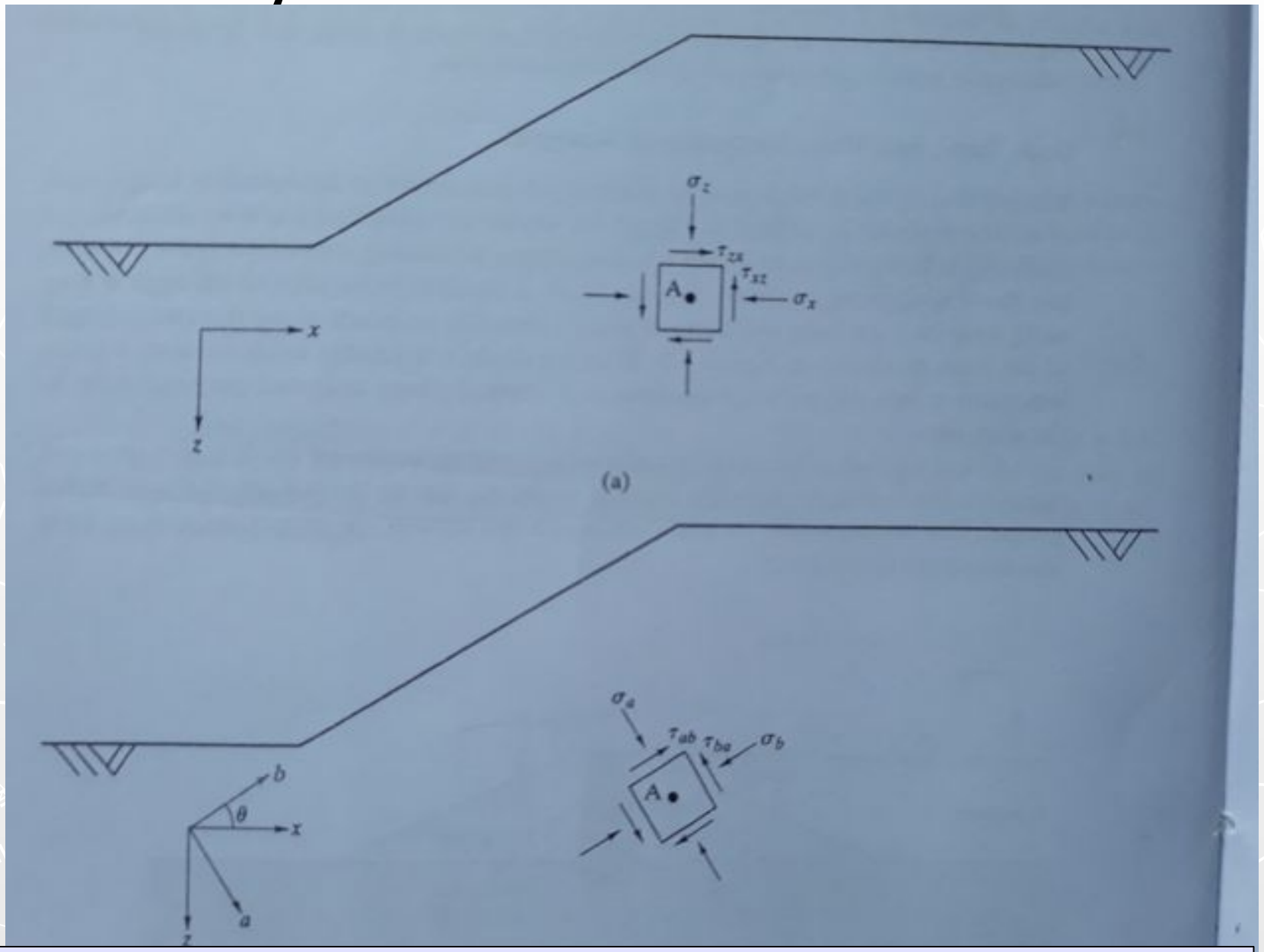
Concrete $\nu = 0.1 - 0.2$; Cork $\nu = 0.0$,

Rubber, $\nu = 0.49999$

For soils: 0.15 to 0.45 (soft clay to dense sand)

Material	*Poisson's ratio*
rubber	≈0.5
gold	0.42
saturated clay	0.40–0.50
magnesium	0.35
titanium	0.34
copper	0.33
aluminium-alloy	0.33
clay	0.30–0.45

Mohr's Circle Analysis



State of stress at a point under the ground represented in two different coordinate systems.

Mohr Circle

- At the point A, **under slope**, the **normal and shear stresses are acting on any two orthogonal planes**, define the stresses at this point.

- However, there are an **infinite number of pairs of orthogonal planes that can pass through point A**, each with their own set of normal and shear stresses.

- One of the **way to use graphical representation** **called Mohr Circle**, **to transform stresses from one pair of orthogonal planes** to any other pair of orthogonal planes.

- Mohr Circle was developed by **German Engineer Otto Mohr**.

- Mohr Circle **provides**, **how stresses are changed from one coordinate system to another.**

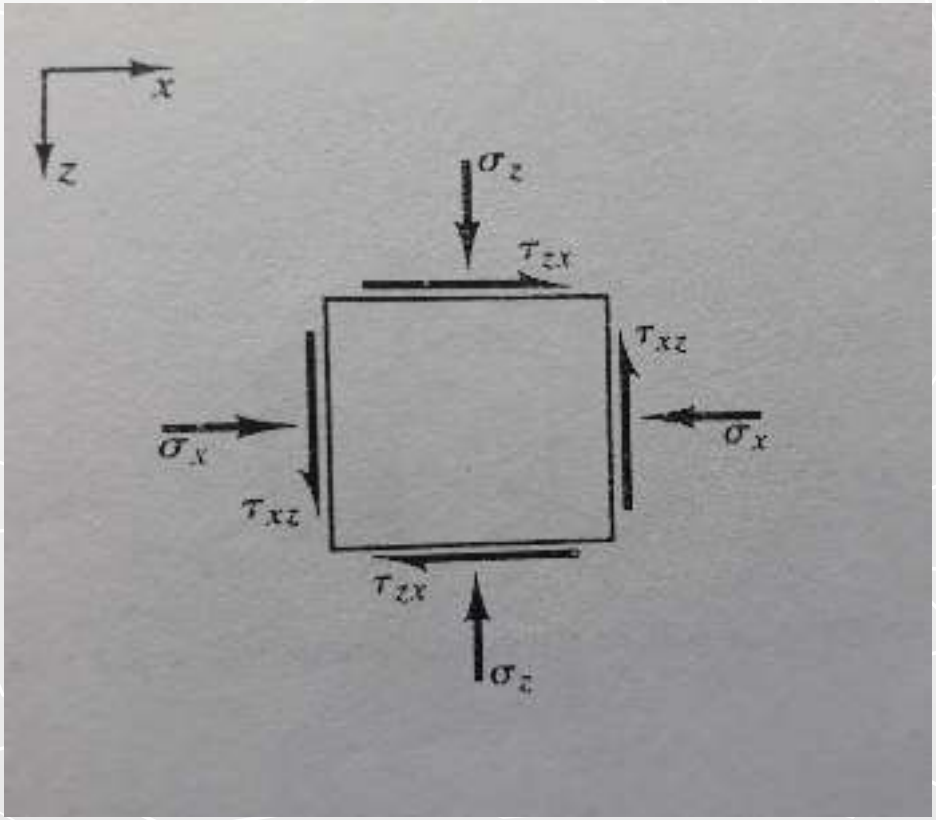
What is Mohr Circle? Write down the importance of Mohr Circle in the analysis of stresses in soils.

Mohr Circle?

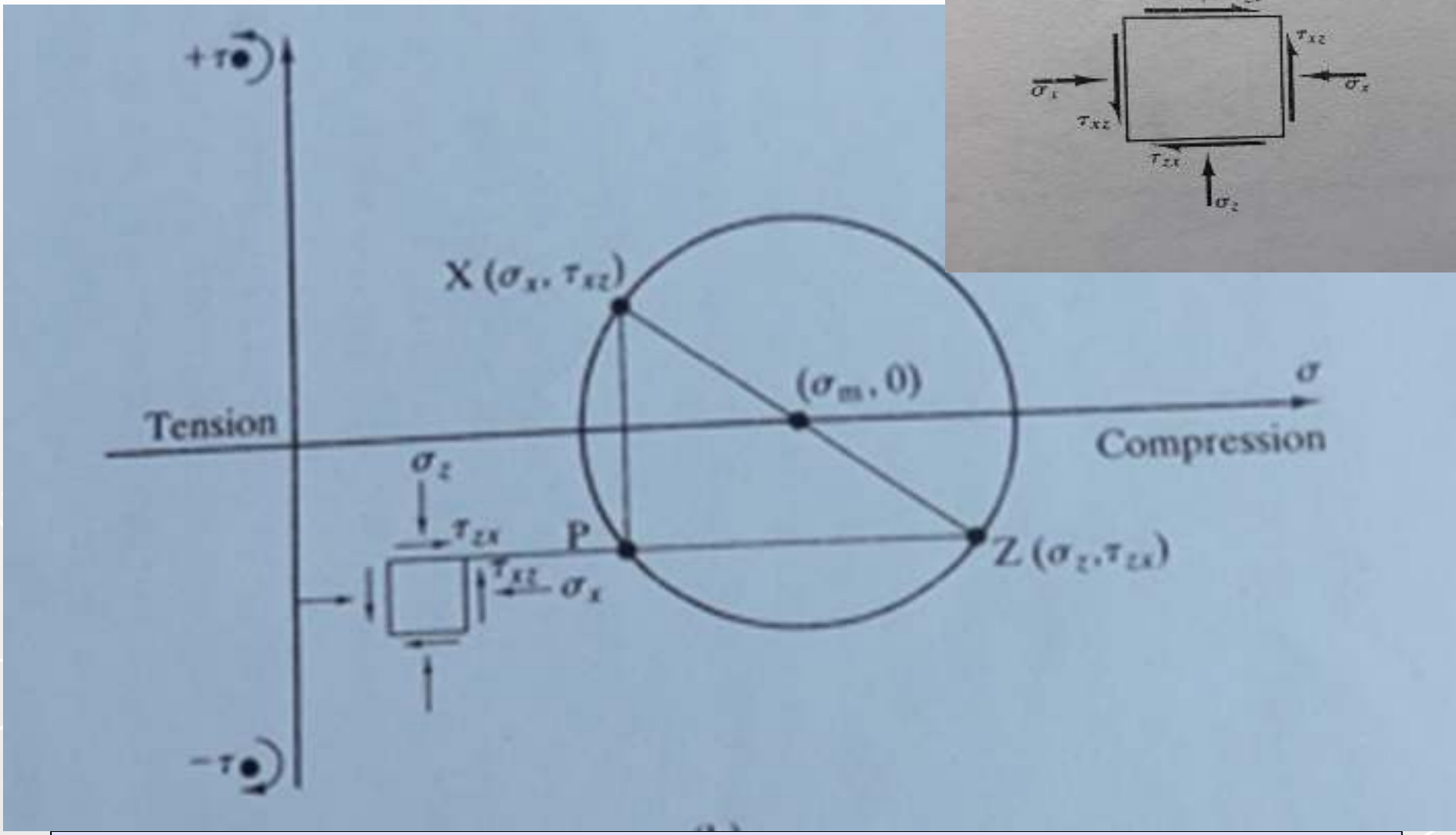
- A **graphical representation** of stresses developed by the German Engineer Otto Mohr.
- Mohr Circle **describes the two-dimensional state of stresses at a point on a circle** plotted on a normal vs shear stress diagram.
- Location of the **center and radius of the circle represent the state of stresses**, and
- **Points on the circle represent the shear and the normal stresses** for axes system rotated at different angle.

How to draw Mohr Circle-Pole Method?

- **Applied stresses in one coordinate system** ie the normal and shear stresses on two orthogonal planes in soil element shown in the figure.
- **Assume** the **value of stresses are known** and normal vertical stress is greater than normal horizontal stress ie $\sigma_z > \sigma_x$.



Mohr Circle of stress and the Pole Method



Points Z and X on the Circle represent the stresses for soil element shown above.

Creation of Mohr Circle

Following steps are followed to create Mohr Circle:

Step-1: Locate the center of circle on the $\sigma - \tau$ **plot**.

- Circle is **always centered on** σ **–axis** at the point $(\sigma_m, 0)$,

Step -2: Locate the points representing the known stresses and compete the circle:

- Stresses acting on the Z and X planes will be represented by two **points lying on the circle**.
- Here τ_{zx} **is** $-$ **ve** because of the element rotates clockwise and τ_{xz} **is +ve** for counterclockwise rotation.

- Assuming $\sigma_z > \sigma_x$, plot point Z, representing the stresses on the Z-plane (plane perpendicular to Z-axis) and point X representing stresses on the X-plane (plane parallel to X-axis).

- Knowing the **coordinates of points Z and X**, we can compute the radius of the circle.

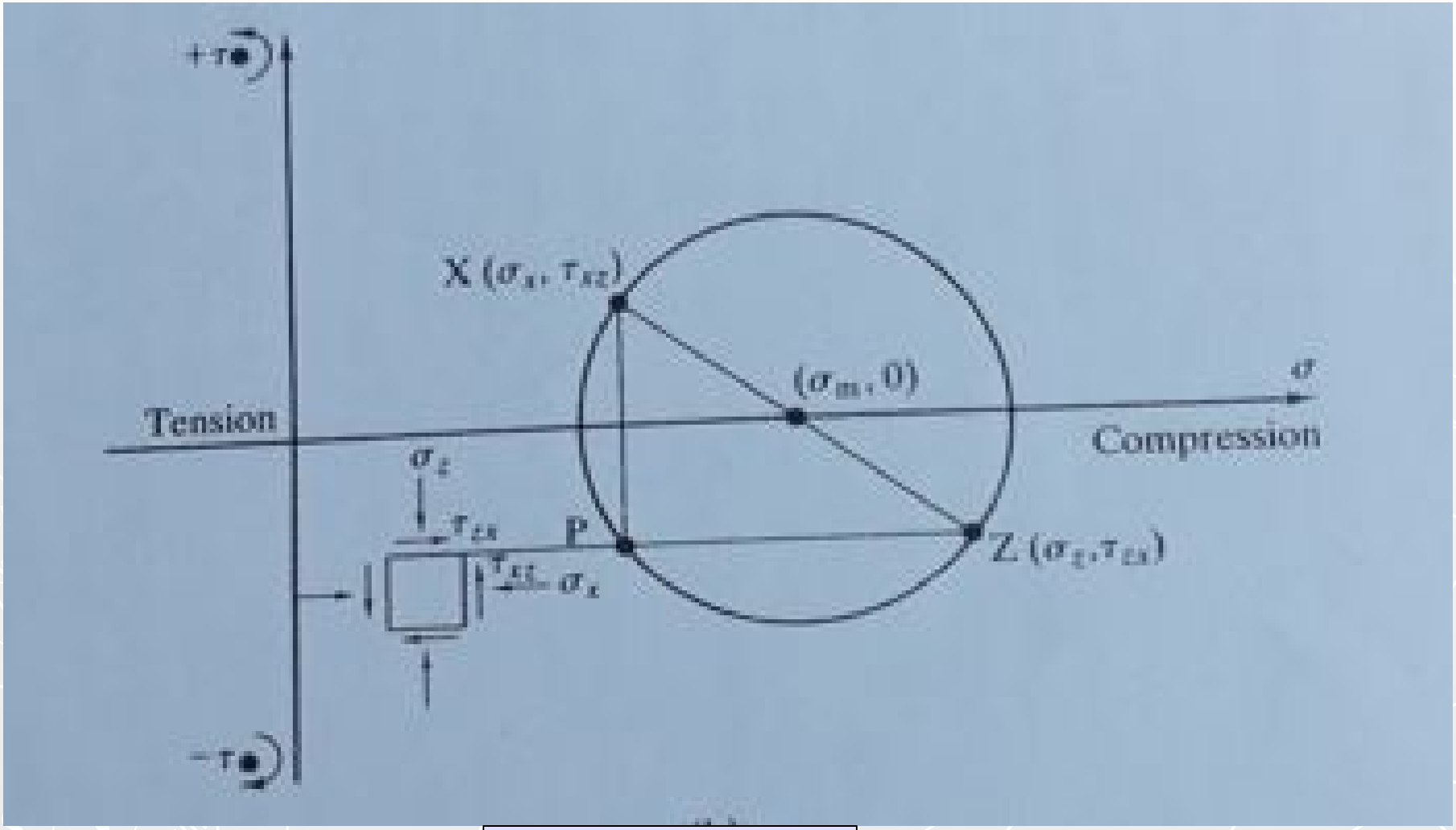
$$R = \frac{\sqrt{(\sigma_z - \sigma_x)2 + (2\tau_{zx})2}}{2} = \sqrt{(\sigma_z - \sigma_x)^2/2 + (\tau_{zx})^2}$$

- Now, **Mohr Circle is created**.

Step - 3: Locate the pole or **origin of plane** on the Mohr Circle.

- Pole is a **special point** on Mohr Circle.
- "A straight line drawn **through** the pole, parallel to a given plane will **intersect the Mohr Circle at the point** which represents the stresses acting on the plane".
- **We know point Z** (plane parallel to Z-axis and plane on which σ_z and τ_{zx} act), represents the stresses on the Z-plane, **locate the pole by drawing a horizontal line from point Z ,** to where it intersect the circle at point P**.**

Mohr Circle of stress and the Pole Method



Location of Pole 'P'

Location of Pole 'P'

- Similarly, if we **draw a vertical line from point X**, (plane perpendicular to X-axis and plane on which σ_x and τ_{xz} act), it will also **intersect the circle at point P**.

- **Point P is the pole.**

- **Location of Pole on given Mohr Circle is unique.**

- Pole method **only works** if the sign conventions are compression is +ve for normal stress and clockwise is –ve for shear stresses.

Advantages of Pole Method

- **Soil element can be drawn** on the figure with its proper orientation.

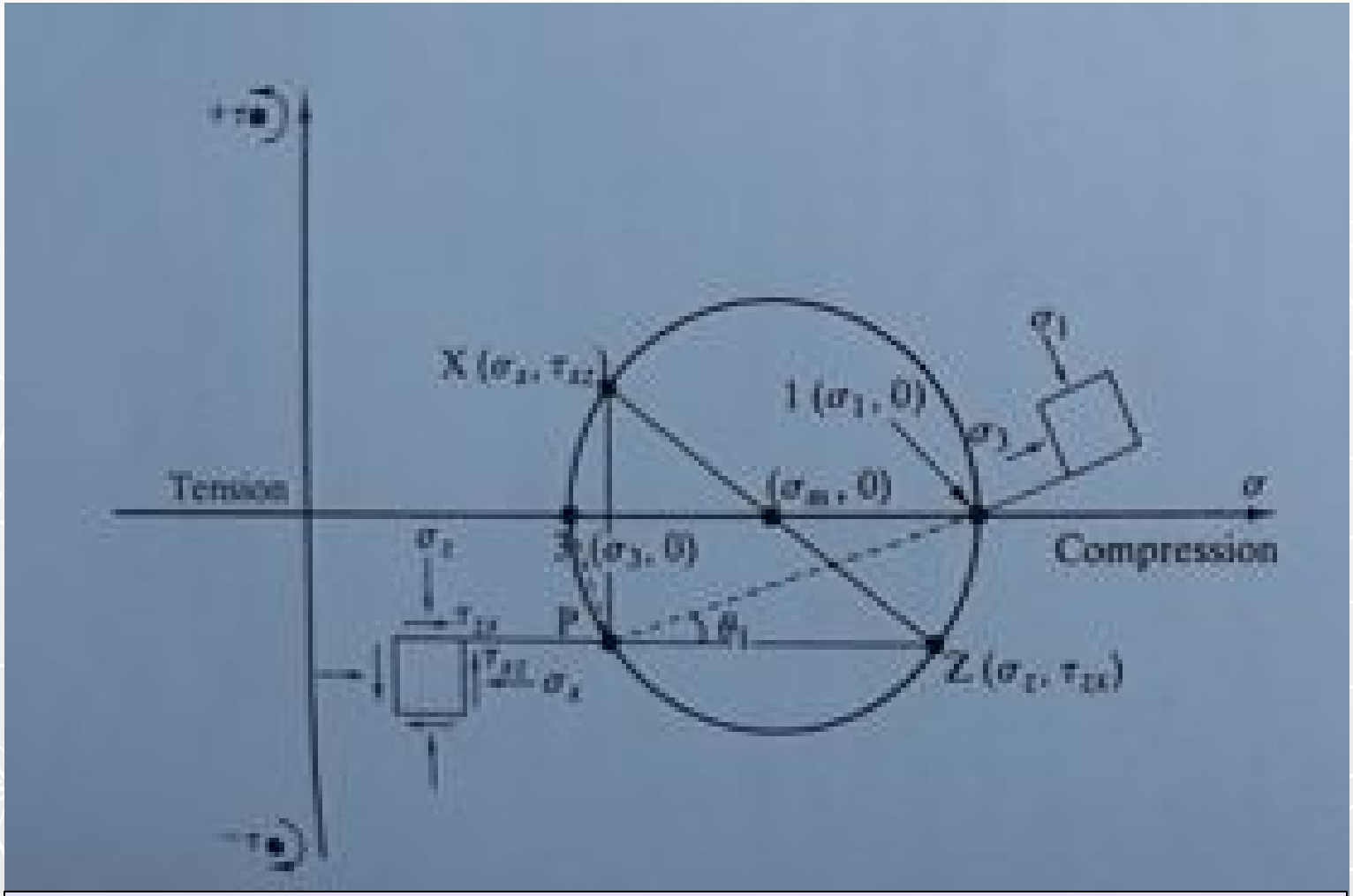
- Easy to ensure that **stresses are shown in proper direction**, using correct sign convention.

What is Pole Method of creating a Mohr Circle?

Describe the Pole method to draw a Mohr Circle with net diagram.

What do you mean by the Principal Stresses described in Mohr Circle? How you can determine the magnitude of principal stresses?

Principal Stresses



Determining the principal stresses by drawing lines from the pole to points 1 and 3.

Principal Planes and Principal Stresses:

If the **soil element is rotated to certain angle**, the **shear stresses will be zero on all four sides.**

- Plane of this element are represented by **point 1** and **3**, known as **Principle Planes**.

Stresses acting on Principles Planes are known as Principle Stresses:

- **Major principal Stress, σ_1**: The greatest normal stress that acts on any plane.
- **Minor Principal Stress, σ_3**: The smallest normal stress that acts on any plane.
- These two stresses **σ_1 and σ_3 act at right angle** to each other.

- Principal Stresses can be determined by the following formula:

$$\sigma_1 = \frac{\sigma_x + \sigma_z}{2} + \sqrt{\left(\frac{\sigma_x - \sigma_z}{2}\right)^2 + \tau_{zx}^2}$$

$$\sigma_3 = \frac{\sigma_x + \sigma_z}{2} - \sqrt{\left(\frac{\sigma_x - \sigma_z}{2}\right)^2 + \tau_{zx}^2}$$

Where, $\boldsymbol{\sigma_1}$= major principal stress

$\boldsymbol{\sigma_3}$= minor principal stress

$\boldsymbol{\sigma_x}$=horizontal normal stress

$\boldsymbol{\sigma_z}$ = vertical normal stress

$\boldsymbol{\tau_{zx}}$ =shear stress acting on horizontal plane.

The vertical and the horizontal stresses at a certain point in a soil are as follows:

σ_x=2100 lb/ft2

σ_z=3000 lb/ft2

τ_{zx} = 300 lb/ft2

Determine the magnitudes and the direction of the major and minor principal stresses.

Solution:

- Construction of the Mohr Circle

Center is located at

$$\sigma_m = \frac{\sigma_1+\sigma_3}{2} = \frac{2100+3000}{2} = 2550 \text{ lb/ft2}$$

Radius of the circle is given by

$$r = \sqrt{\left(\frac{\sigma_z-\sigma_x}{2}\right)^2+(\tau_{zx})^2}$$

$$= \sqrt{\left(\frac{3000-2100}{2}\right)^2+(300)^2}$$

$$= 300 \text{ lb/ft2}$$

- Principal Stresses can be determined by the following formula:

$$\sigma_1 = \frac{\sigma_x + \sigma_z}{2} + \sqrt{\left(\frac{\sigma_x - \sigma_z}{2}\right)^2 + \tau_{zx}^2}$$

$$\sigma_3 = \frac{\sigma_x + \sigma_z}{2} - \sqrt{\left(\frac{\sigma_x - \sigma_z}{2}\right)^2 + \tau_{zx}^2}$$

Where, $\boldsymbol{\sigma_1}$= major principal stress

$\boldsymbol{\sigma_3}$= minor principal stress

$\boldsymbol{\sigma_x}$=horizontal normal stress

$\boldsymbol{\sigma_z}$ = vertical normal stress

$\boldsymbol{\tau_{zx}}$ =shear stress acting on horizontal plane.

- Compute the principal stresses:

  $\boldsymbol{\sigma_1}$= 2550+541= 3091lb/ft2

  $\boldsymbol{\sigma_3}$= 2550-541 = 2009 lb/ft2

- Direction of principal stresses are:

$$\theta_1 = \frac{1}{2}\cos^{-1}\sqrt{\frac{1}{1 + [2\tau_{zx}/(\sigma_z - \sigma_x)]^2}}$$

θ_1=16.8º

Thank you

EWCE 341

Principles of Soil Mechanics

Stresses in Soils

What we are going to discuss today?

-Stresses in soils

-Horizontal stress

-Measurement of horizontal stress

(in-situ & lateral earth pressure)

-Measurement of Geostatic stress and Induced Stress)

-Boussinesq's Equation (Point and Line Load)

-Newmark's Solution (Area load)

-Chart solution

What is stress in soils ?

Stress:

- All civil Engineering projects impart loads on the ground that supports them.
- **Loads produces stresses in soils** ie **compressive stress**, **shear stress** and **even tensile stress**.

Horizontal Stress:

- How horizontal stresses are:

-**Direct result** of **applied external loads** such as the braking forces from the wheels of a large track.

-**Indirect result** of **applied vertical geostatic stresses and induced stresses.**

Importance:

- **Important** for many engineering analysis.
- **Design of Retaining Walls (RW)** depends on the horizontal stresses in the soils being retained.









How can you measure the horizontal stresses in the ground?

Measure of horizontal stress:

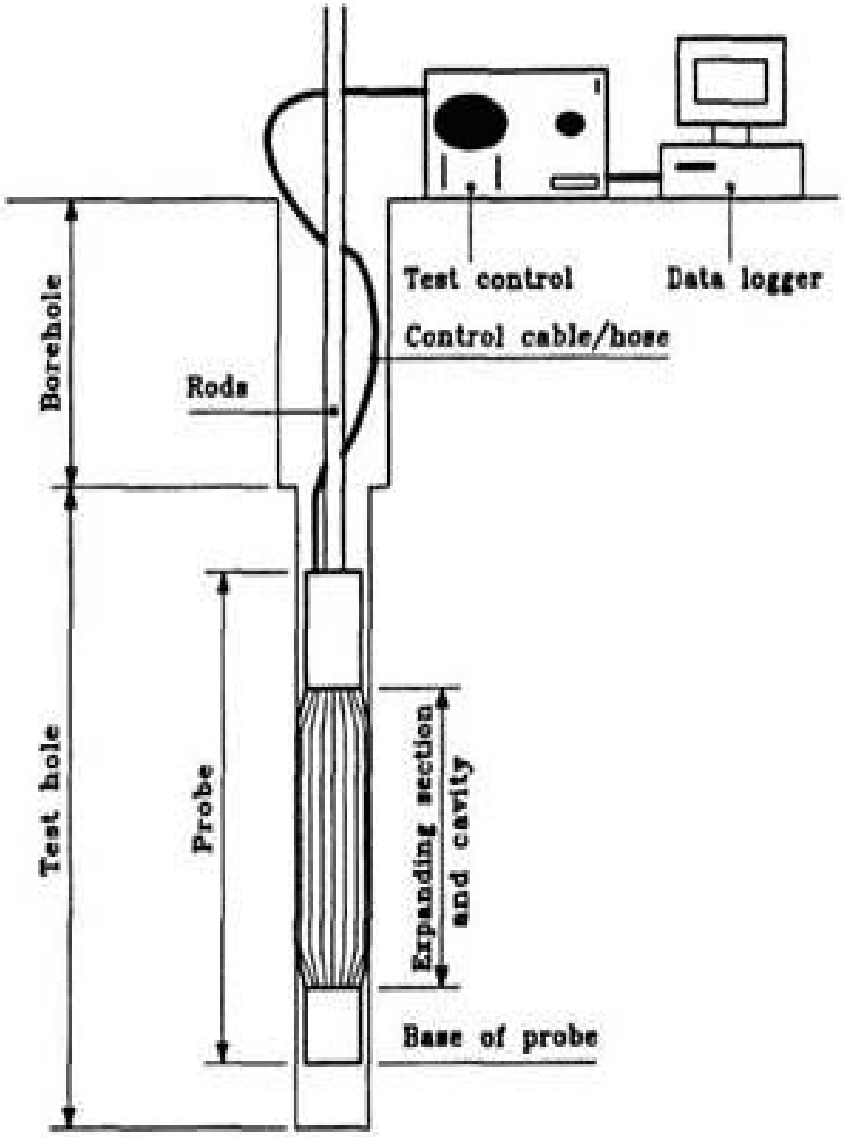
- **In-situ measurement** by:
  - -**PMT**: Pressure Meter Test
  - -**DMT**: Dilatometer Test
- **Estimated from the vertical stress**

using of lateral earth pressure.

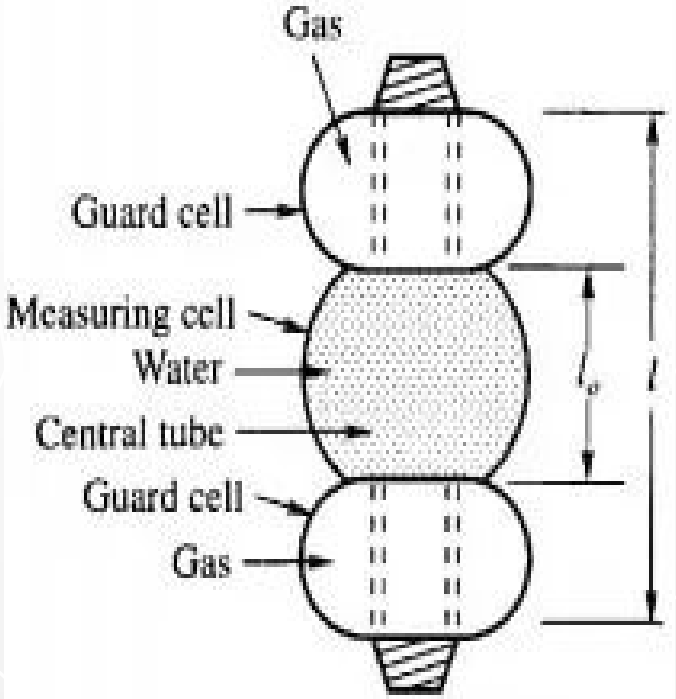
If **ground is horizontal** then $\sigma_x = \sigma_y$

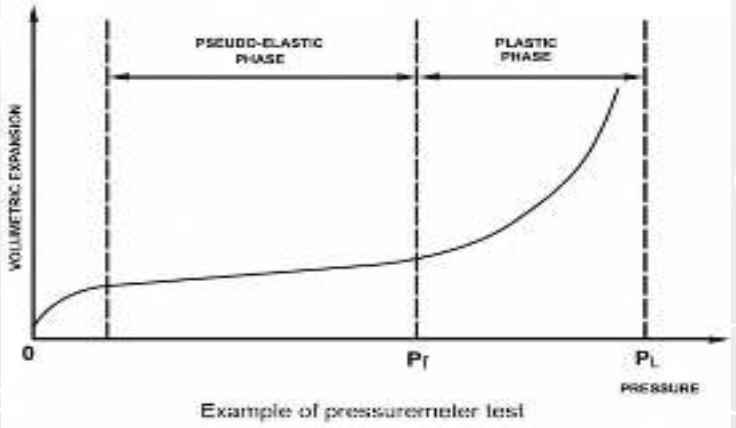
for three dimensional analyses.





Pressure meter

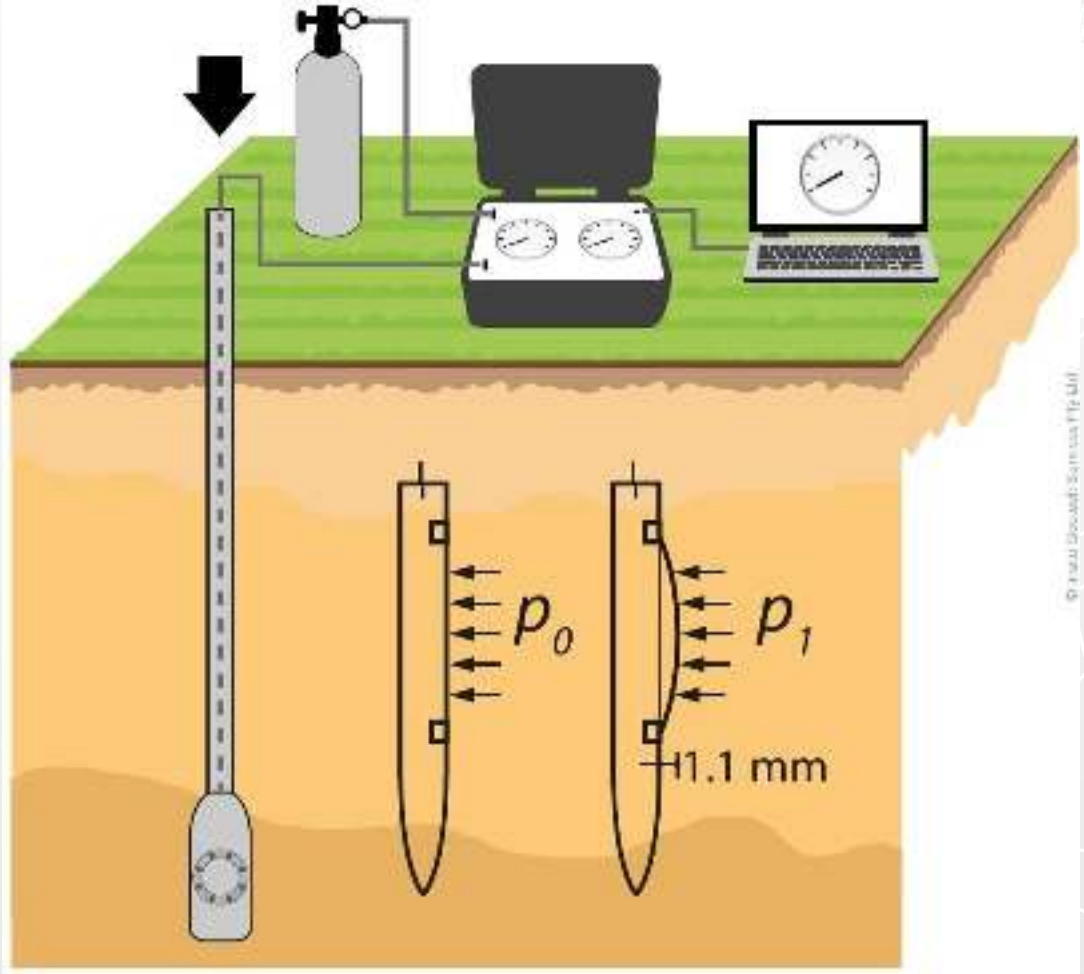




Example of pressuremeter test



Dilatometer Test



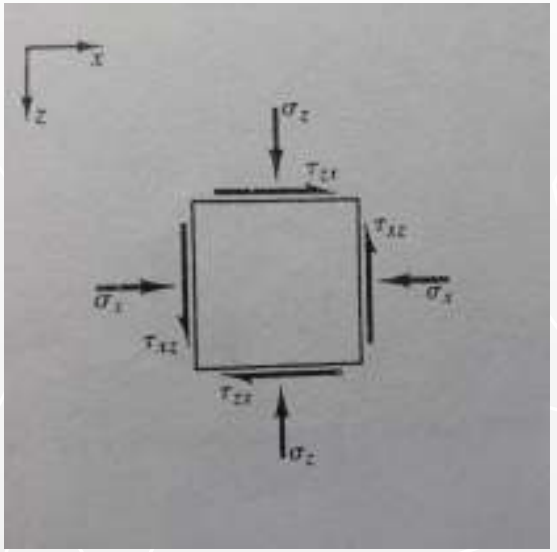




Shear stress:

- If the **ground surface is horizontal**, the **geostatic shear stresses** on **horizontal and vertical planes** are all equal to zero.

ie $\tau_{xz} = \tau_{zx} = 0$



- Shear stresses may be present on **other planes**.
- Shear stresses conditions are more complex if ground surface is inclined.

Induced Stresses:

- Civil Engineering projects often **induce external loads on to the ground**, thus producing induced stresses.
- **External loads includes**, structural foundations, vehicles, stock piles and many others.
- Resulting induced stresses are significant because these **can be the sources of excessive settlement, shear failure** and **other problems**.

Measurement of Induced Stresses

Load System:

- **Application of Point Loads**
- Application of Line Loads.
- Application of Area Loads.

Solution System:

- **Analytical solutions**
- Numerical solutions
- **Charts Solution**

Measurement of induce stresses:

Boussinesq's Method:

- French Mathematician Joseph **Boussinesq developed** method of computing induced stresses in an ***infinite elastic half-space*** due to an applied external load.

Elastic Half-space

- Each **discrete particle of soil**, in contact with another, **will deform when loaded**, depending on the force that is "driving" contact between the two particles.
- **Elastic half-space** is the volume in which this deformation takes place.

Soil is a little more complicated because as **each particle is loaded**, it has two choices

- **to move into an unoccupied space, or**
- **deform in contact with another particle**.

Deform in contact with another particle:

- **If soil deforms**, it does so in the **"half space"** defined by the **contact area that increases** with deformation until the stress stabilizes.
- This **part of soil movement is recoverable** and is known as **"elastic deformation"** or "**elastic settlement".**

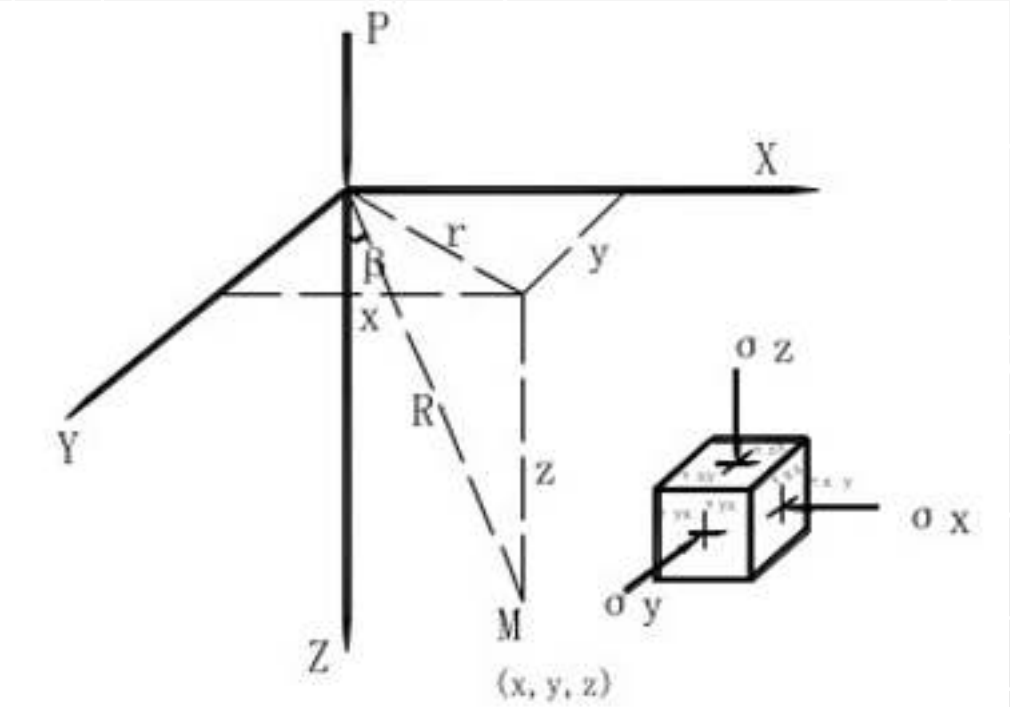
Move into an unoccupied space:

- It is **re-orientation of the soil grains** which is **generally not recoverable** and results in **permanent or plastic deformation** of the soil mass.

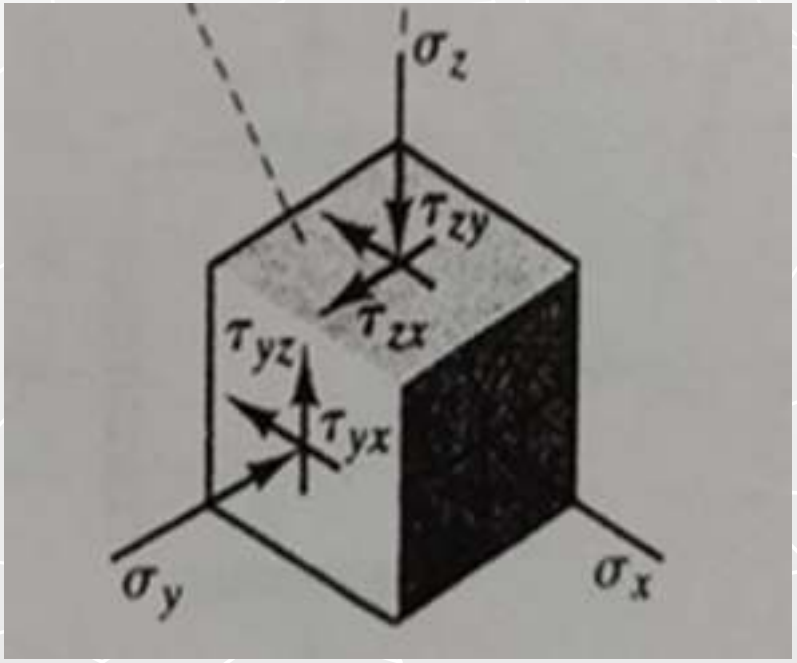
- Bousinesq solve the problem **where a point load (vertical load) P, is perpendicular to this plane** (ground surface). Figure below.

- According to his solution, such a load will induce the **following stresses at a point in the ground** shown below.

- For three dimensional analyses in the x-y-z plane, we only require $\triangle \sigma_x$, $\triangle \sigma_y$, $\triangle \sigma_z$ and τ_{zx}, τ_{zy}; τ_{yz}, τ_{yx}; τ_{xz}, τ_{xy}.



Stresses due to concentrated load



$$\Delta\sigma_x = \frac{P}{2\pi}\left[\frac{3x_f^2 z_f}{R^5} - (1-2\nu)\left(\frac{x_f^2 - y_f^2}{Rr^2(R+z_f)} + \frac{y_f^2 z_f}{R^3 r^2}\right)\right]$$

$$\Delta\sigma_y = \frac{P}{2\pi}\left[\frac{3y_f^2 z_f}{R^5} - (1-2\nu)\left(\frac{y_f^2 - x_f^2}{Rr^2(R+z_f)} + \frac{x_f^2 z_f}{R^3 r^2}\right)\right]$$

$$\Delta\sigma_z = \frac{3Pz_f^3}{2\pi R^5}$$

$$\Delta\tau_{zx} = -\Delta\tau_{xz} = \frac{3Pz_f^2 x_f}{2\pi R^5}$$

$$\Delta\tau_{yx} = -\Delta\tau_{xy} = \frac{P}{2\pi}\left[\frac{3x_f y_f z_f}{R^5} - (1-2\nu)\left(\frac{(2R+2)x_f y_f)}{(R+z_f^2)R^3}\right)\right]$$

$$\Delta\tau_{yz} = -\Delta\tau_{zy} = \frac{3Pz_f^2 y_f}{2\pi R^5}$$

$$R = \sqrt{x_f^2 + y_f^2 + z_f^2}$$

$$r = \sqrt{x_f^2 + y_f^2}$$

Application of Line Load:

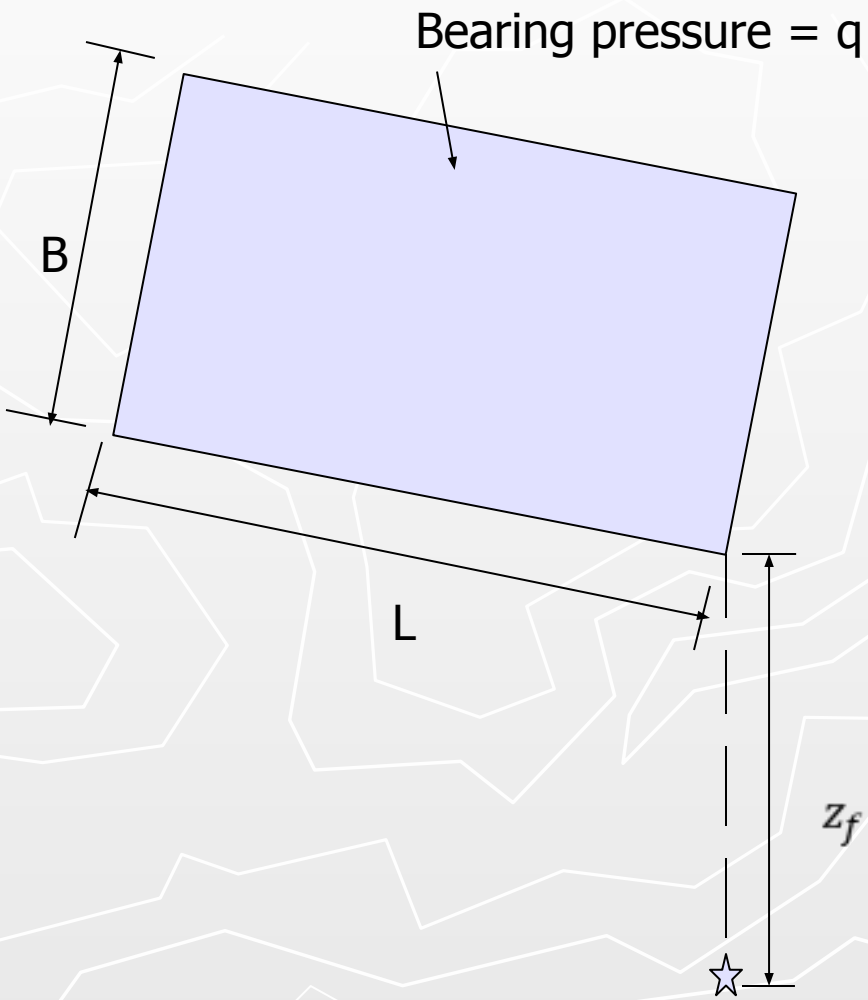
A line load of infinite length parallel to y axis,

$$\sigma_z = \frac{2\, z_f^3 \frac{P}{b}}{\pi\left(x_f^2 + z_f^2\right)^2}$$

P=vertical load

b= unit length along the line

Newmark's Solution for Induced Vertical Stresses beneath the corner of a rectangular Loaded area.



Equation for analytical solution for **vertical induced stress** at depth z beneath the corner of a loaded Rectangle.

If $B^2 + L^2 + z_i^2 < B^2L^2/z_i^2$,

$$I_\sigma = \frac{1}{4\pi}\left[\left(\frac{2BLz_i\sqrt{B^2+L^2+z_i^2}}{z_i^2(B^2+L^2+z_i^2)+B^2L^2}\right)\left(\frac{B^2+L^2+2z_i^2}{B^2+L^2+z_i^2}\right)\right.$$

$$\left. + \pi - \sin^{-1}\frac{2BLz_i\sqrt{B^2+L^2+z_i^2}}{z_i^2(B^2+L^2+z_i^2)+B^2L^2}\right]$$

Otherwise,

$$I_\sigma = \frac{1}{4\pi}\left[\left(\frac{2BLz_i\sqrt{B^2+L^2+z_i^2}}{z_i^2(B^2+L^2+z_i^2)+B^2L^2}\right)\left(\frac{B^2+L^2+2z_i^2}{B^2+L^2+z_i^2}\right)\right.$$

$$\left. + \sin^{-1}\frac{2BLz_i\sqrt{B^2+L^2+z_i^2}}{z_i^2(B^2+L^2+z_i^2)+B^2L^2}\right]$$

Equation for analytical solution for **vertical induced stress** at depth z beneath the corner of a loaded Rectangle.

where:

I_σ = influence factor for vertical stress at a point beneath the corner of the loaded rectangle

B = width of loaded rectangle

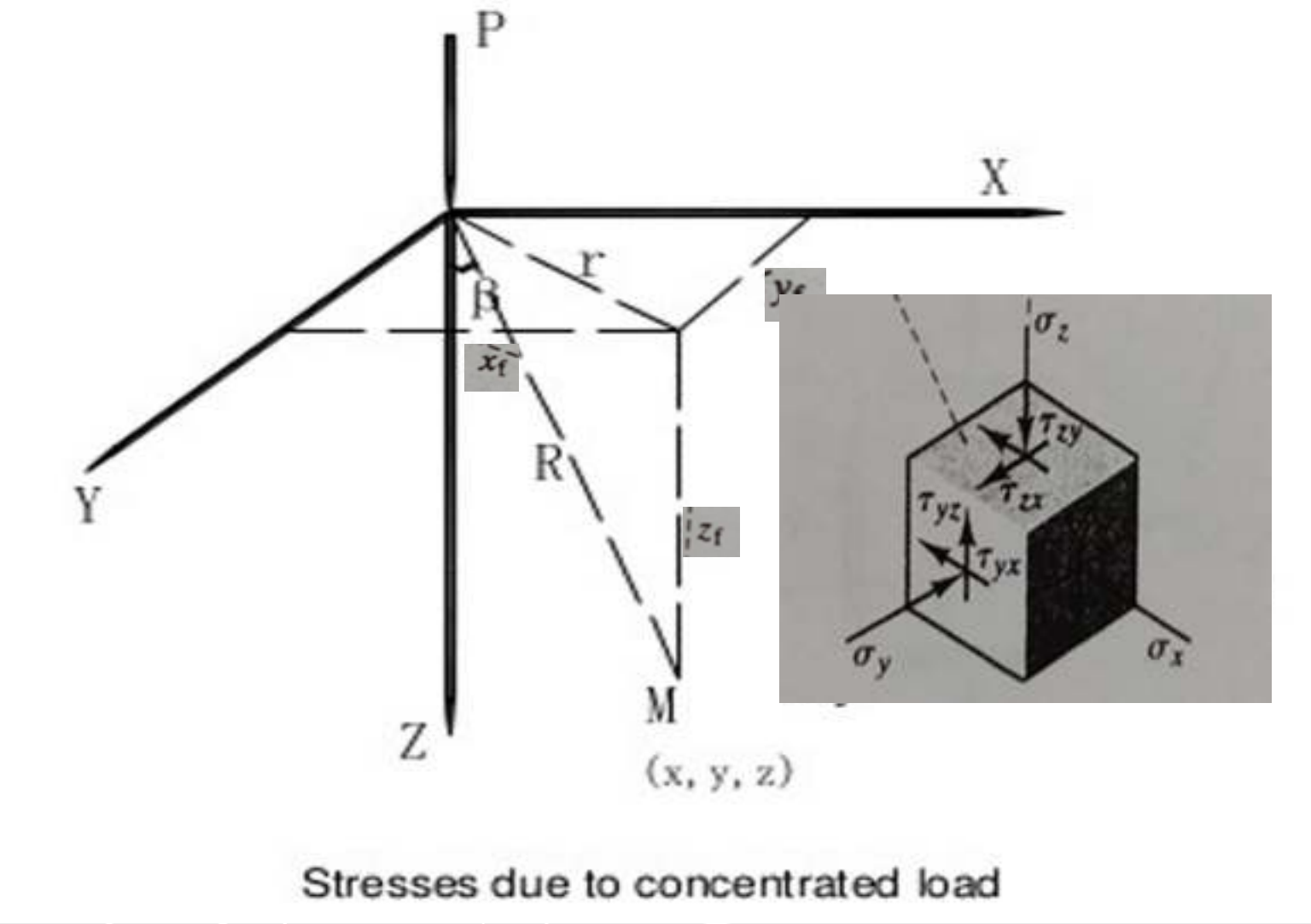
L = length of loaded rectangle

z_f = vertical distance from loaded rectangle to the point (always > 0)

Notes:

1. The $\sin^{-1}$ term must be expressed in radians.
2. Newmark's solution is often presented as a single equation with a $\tan^{-1}$ term, but that equation is incorrect when $B^2 + L^2 + z_f^2 < B^2L^2/z_f^2$.
3. It is customary to use B as the shorter dimension and L as the longer dimension.

Ex. 1: The dimension in Figure below are x_f=10.0 ft, y_f=0.0 and z_f=15.0 ft. The load P is **132 K,** and the soil is a partially saturated clay. Compute the induced stresses $\triangle \sigma_x$, $\triangle \sigma_z$ and τ_{zx} in the soil element.



Stresses due to concentrated load

Fig : Element of soil in three-dimensional x-y-z space/plane.

Solution (Ex. 1):

- For partially saturated clay, assume poison's ratio ν=0.35

$$R = \sqrt{(x_f^2 + y_f^2 + z_f^2)}$$

$$= \sqrt{(10.0^2 + 0.0^2 + 15.0^2)}$$

$$= 18.0 \text{ ft}$$

$$r = \sqrt{(x_f^2 + y_f^2)}$$

$$= \sqrt{(10.0^2 + 0.0^2)}$$

$$= 10.0 \text{ ft}$$

Solution – Ex.1 (cont..):

$$\Delta\sigma_x = \frac{P}{2\pi}\left[\frac{3x_f^2 z_f}{R^5} - (1-2\nu)\left(\frac{x_f^2 - y_f^2}{Rr^2(R+z_f)} + \frac{y_f^2 z_f}{R^3 r^2}\right)\right]$$

Putting the values, P=132000 lb, other value, we get,

$\Delta\sigma_x = 183.75\ \text{lb}/ft^2$ Ans. (ν=0.30)

$$\Delta\sigma_z = \frac{3Pz_f^3}{2\pi R^5} = \frac{3(132{,}000)(15.0)^3}{2\pi(18.0)^5} = 115.05\ \text{lb}/ft^2 \text{ Ans.}$$

$$\Delta\tau_{zx} = \frac{3Pz_f^2 x_f}{2\pi R^5} = \frac{3(132{,}000)(15.0)^2(10.0)}{2\pi(18.0)^5} = 75.0\ \text{lb}/ft^2 \text{ Ans.}$$

Chart Solutions

Pressure/Stress bulbs/Curves:

- Curves which **connect the points of equal induced stresses** are sometimes called **pressure bulbs or stress bulbs.**

- **Charts provide a visual sense** of how the stresses are distributed.

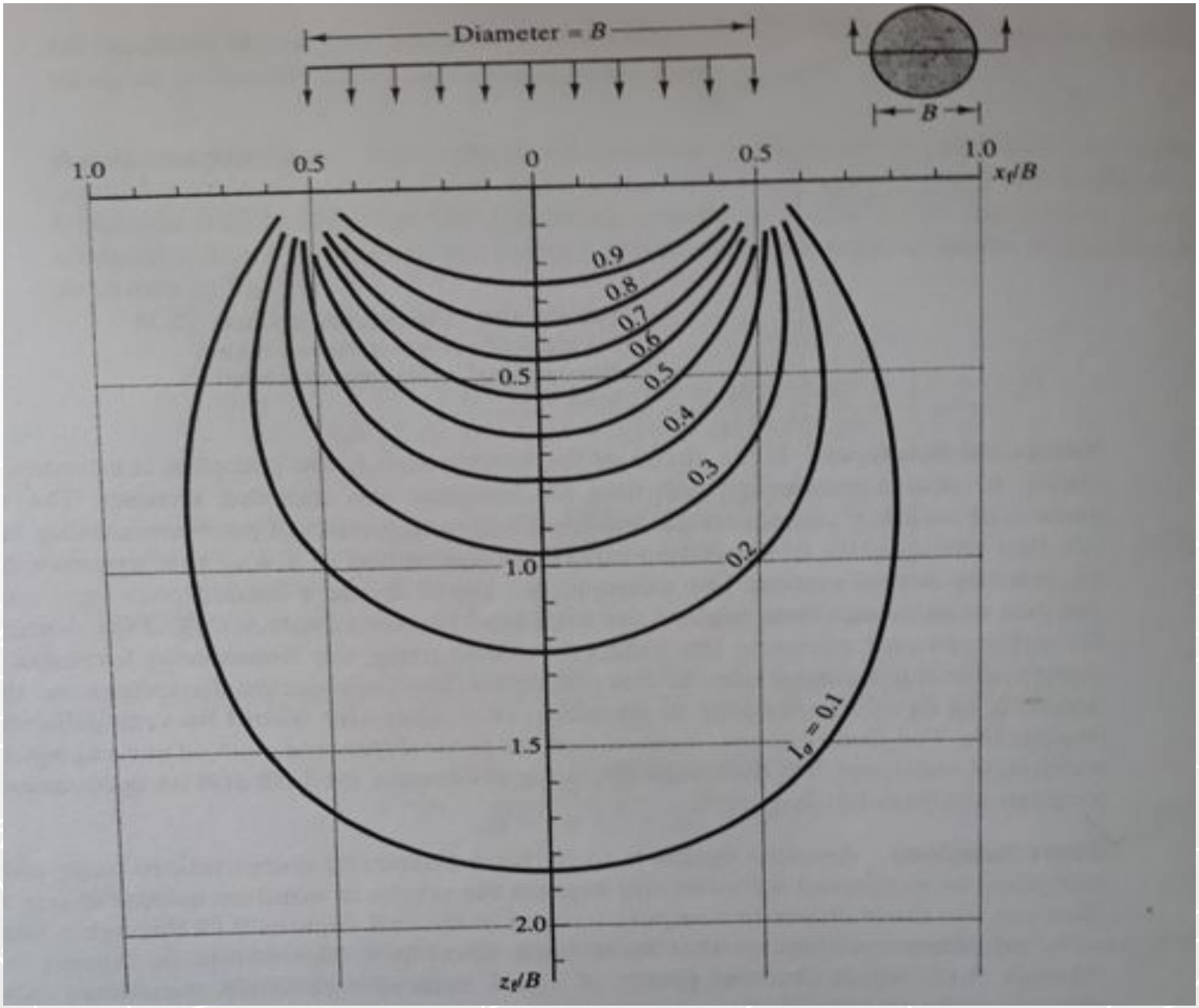


Fig 1: Influence factor for induced vertical stress under **circular loaded area**, as per Boussinesq.

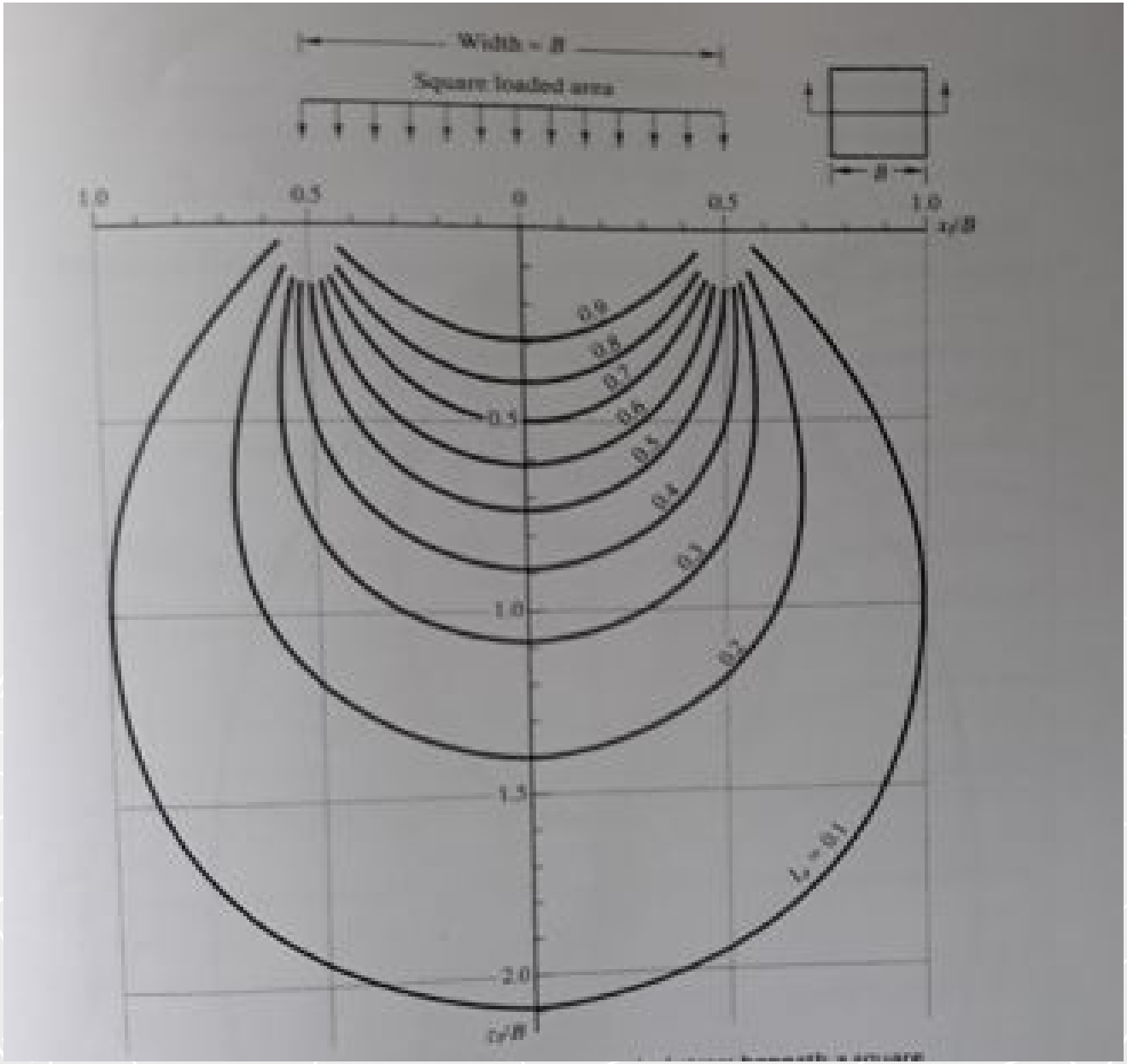


Fig 2: Influence factor for induced vertical stress under square loaded area, as per Boussinesq.

(of a line or solid) limited in one direction and stretching to infinity in the other.

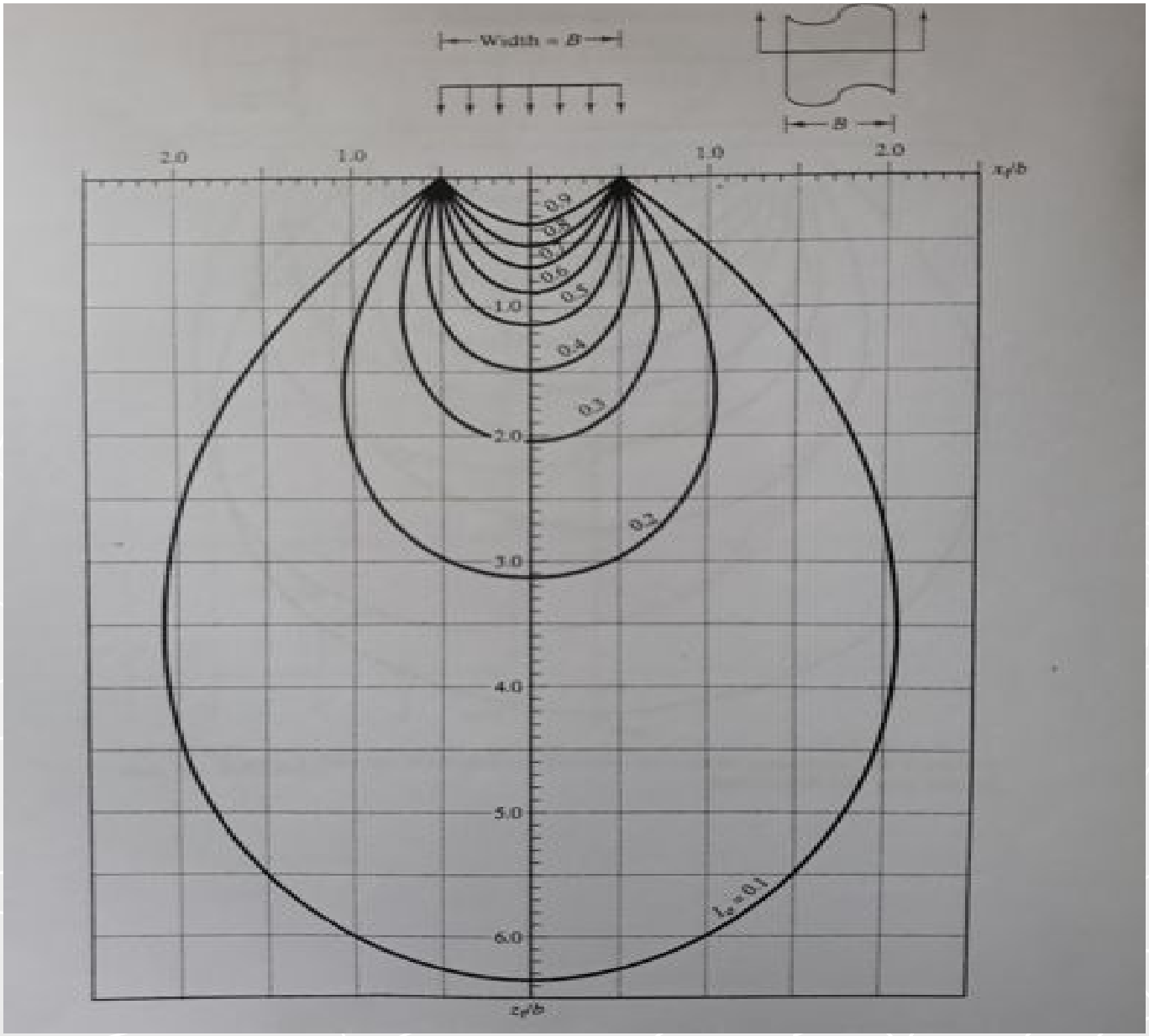
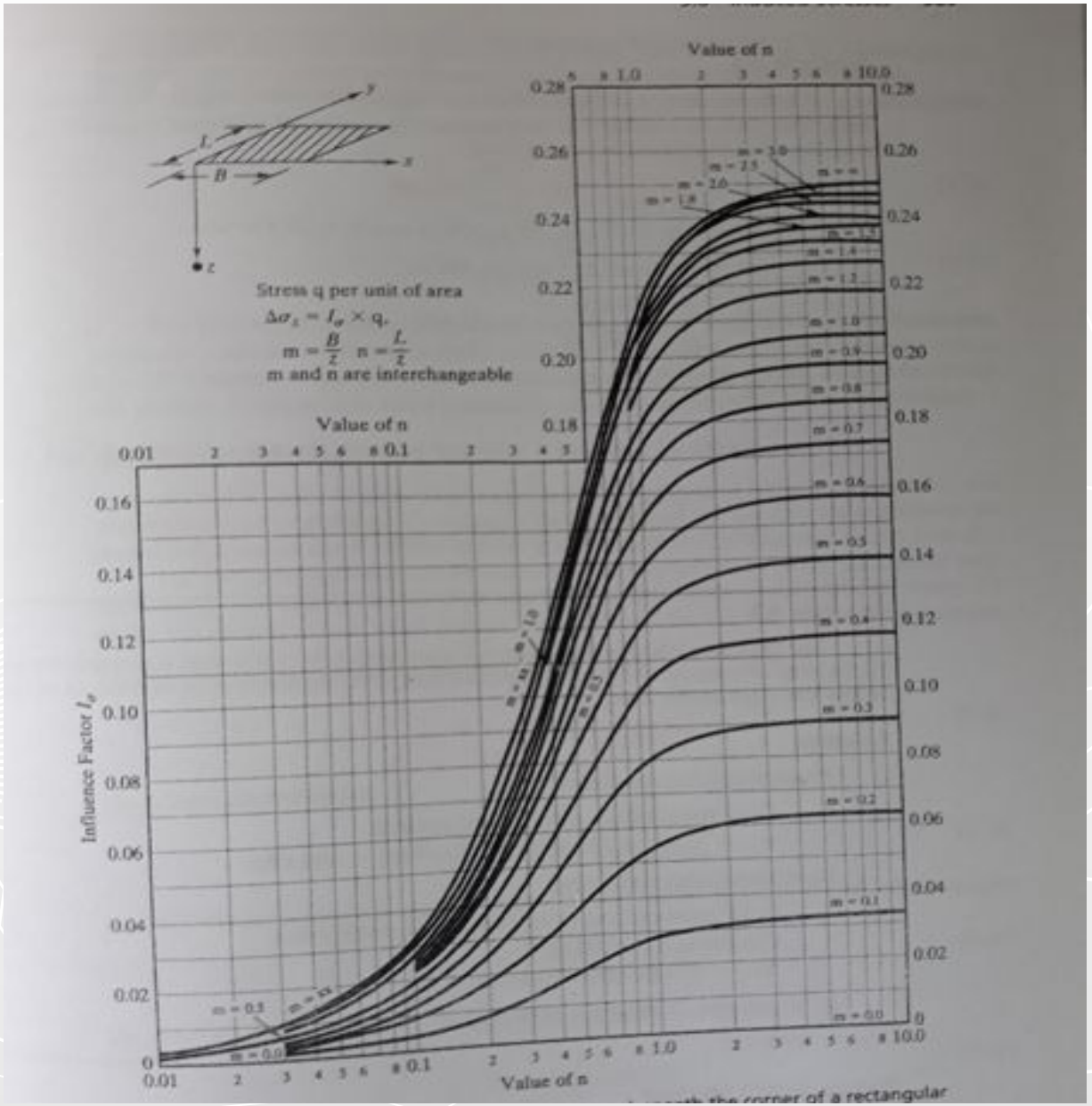


Fig 3: Influence factor for induced vertical stress under **semi infinite strip load**, as per Boussinesq.

Fig 3: Influence factor for induced vertical stress **beneath the corner of a rectangular loaded area**.



Ex. 2: Compute $\triangle \sigma_z$ at a depth of **10.0m** below the edge of a **25.0m** diameter of footing of **water tank**. The tank, its foundation and it contents have a **total mass of $6.1X10^6$ kg,** which is uniformly distributed across its base.

Solution (Ex.2):

$W = mg = (6.1 \times 10^6 \text{ kg})(0.00981) = 59{,}841 \text{ kN}$

$A = \pi r^2 = \pi (12.5)^2 = 491\ m^2$

$q = \frac{P}{A} = \frac{59{,}841 kN}{491 m^2} = 122 \text{ kPa}$

$\frac{z_f}{B} = \frac{10m}{25m} = 0.40$

From Fig. of Ex. 2: at $\frac{z_f}{B} = \frac{10}{25} = 0.40$ and beneath the edge of the tank, $\frac{x_f}{B} = \frac{12.5}{25} = 0.50$;

$I_\sigma = 0.40$

$\Delta \sigma_z = I_\sigma \times q = 0.40 \times (122 \text{ kPa}) = 49 \text{ kPa}$ Ans.

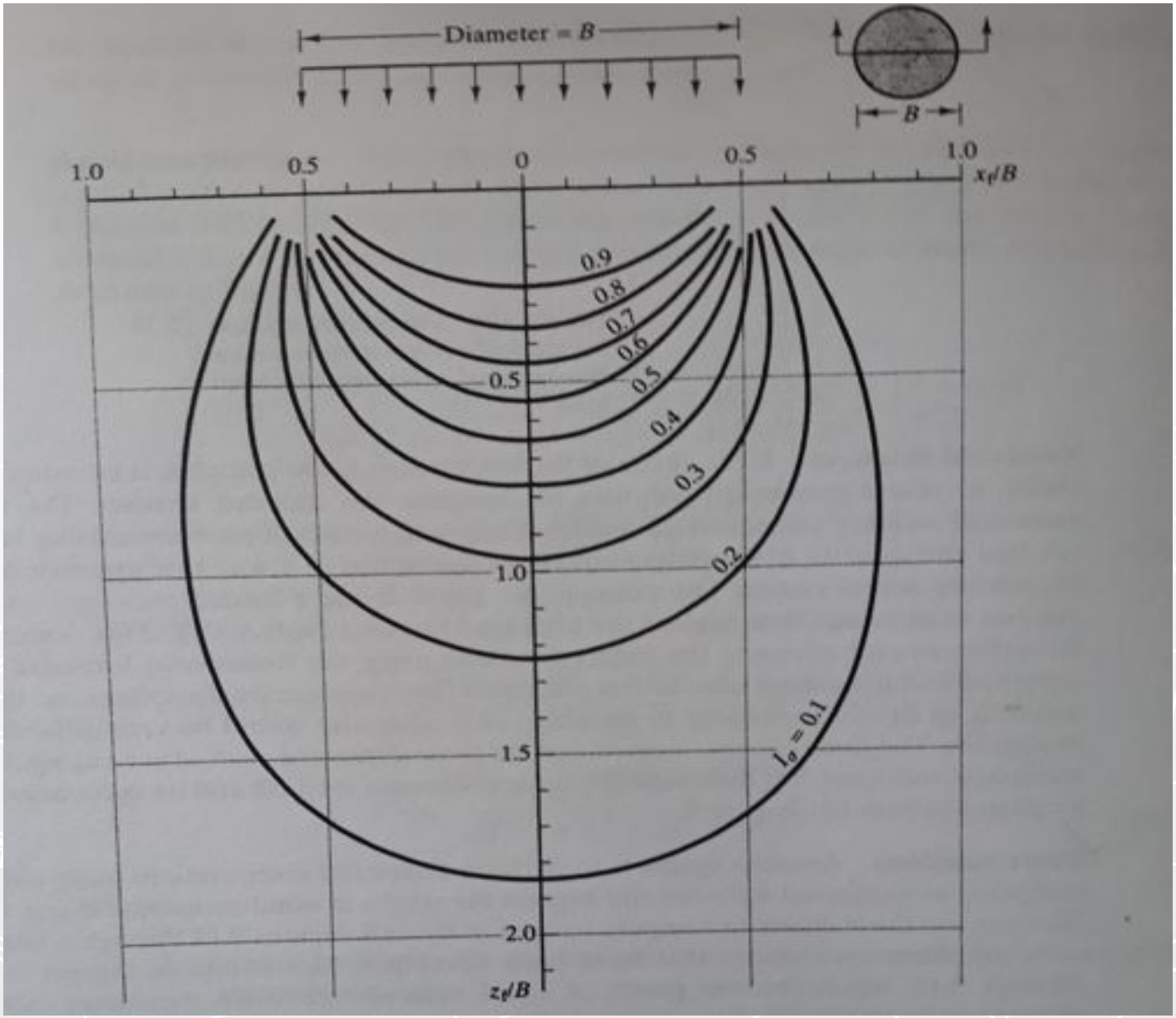


Fig of Ex. 2: Influence factor for induced vertical stress under circular loaded area, per Boussinesq.

Ex. 3: Compute $\triangle \sigma_z$ at a depth **of 10.0m** below the **edge of a 25.0m** diameter water tank. The tank, its foundation and it contents have a total mass of $6.1X10^6$ kg, which is uniformly distributed across its base. The tank is underlain by a soil that has a unit weight of **18.0 kN/m^3**. Compute the **preconstruction** and **post construction** σ_z at a depth of 10.0m below the edge of the tank.

Solution (Ex. 3):

Preconstruction Condition:

Only the geostatic stresses are present,

$\sigma_z = \sum \boldsymbol{\gamma H} = (18.0\text{kN}/m^3)\text{x}\ (10.0\text{m}) = 180\ \text{kPa}$

Post-construction Condition:

Both geostatic and induced stresses are present,

- **Geostatic stress σ_z**= 180 kPa

Solution Ex.3: (cont..)

- **Induced stress:**

$$W = mg = (6.1 \times 10^6 \text{ kg})(0.00981) = 59{,}841 \text{ kN}$$

$$A = \pi r^2 = \pi (12.5)^2 = 491\ m^2$$

$$q = \frac{P}{A} = \frac{59{,}841 kN}{491 m^2} = 122 \text{ kPa}$$

$$\frac{z_f}{B} = \frac{10m}{25m} = 0.40$$

From Fig. of Ex. 2: at $\frac{z_f}{B} = 0.40$ and beneath the edge of the tank, $\frac{x_f}{B} = \frac{12.5}{25} = 0.50$;

$$I_\sigma = 0.40$$

$$\Delta \sigma_z = I_\sigma \times q = 0.40 \times (122 \text{ kPa}) = 49 \text{ kPa Ans.}$$

Thereby, Post-construction stress $\sigma_z = 180 \text{ kPa} + 49 \text{ kPa} = 229 \text{ kPa}$ Ans.

Westergaard's Method

- Westergaard, Harold M (1938), a **Danish structural Engineer**, solved the same problem, Boussinesq addressed but with slightly different assumptions.

- **Instead of using a perfectly elastic material**, he assumed one that **contained closely spaced horizontal reinforcement members** of **infinitesimal thickness**.

- In such cases, **the horizontal strain zero at all points**.

- It is a **precise representation of layered soils** where some layers are much stiffer than others.

Based on Westergaard's method, Terzaghi presented the following formula for σ_z, due to vertical point load P.

$$\Delta\sigma_z = \frac{PC}{2\pi z_f^2}\left[\frac{1}{C^2 + (r/z_f)^2}\right]^{1.5}$$

$$C = \sqrt{\frac{1-2\nu}{2(1-\nu)}}$$

- Westergaard's solution produces σ_z values **equal to or less** than the Boussinesq values.
- **As ν increases, the computed stress becomes smaller, eventually reaching zero at ν=0.5.**
- Some Geo Tech Engineers prefer Westergaard, at least for certain soil.
- Boussinesq is more commonly used and appropriate for most problems.

Approximate method:

- Sometimes it is useful to use such methods of computing stresses in soils.

- **Useful when an quick answer is needed** or **when a computer is not available.**

- Following formulas compute the **influence factor** I_σ for induced vertical stress beneath the **center of an area load.**
- They **produce values that are within 5% of Boussinesq values,** which is more than sufficient for virtually all practical problems.

For circular loaded areas (Poulos and Davis, 1974),

$$I_\sigma = 1 - \left(\frac{1}{1 + (B/2z_f)^2}\right)^{1.5}$$

For square loaded areas,

$$I_\sigma = 1 - \left(\frac{1}{1 + (B/2z_f)^2}\right)^{1.76}$$

For continuous loaded areas (also known as *strip loads*) of width B and infinite length,

$$I_\sigma = 1 - \left(\frac{1}{1 + (B/2z_f)^{1.38}}\right)^{2.60}$$

For rectangular loaded areas of width B and length L,

$$I_\sigma = 1 - \left(\frac{1}{1 + (B/2z_f)^{1.38+0.62B/L}}\right)^{2.60-0.84B/L}$$

where:

I_σ = influence factor for vertical stress beneath the center of a loaded area

z_f = depth from bottom of loaded area to point

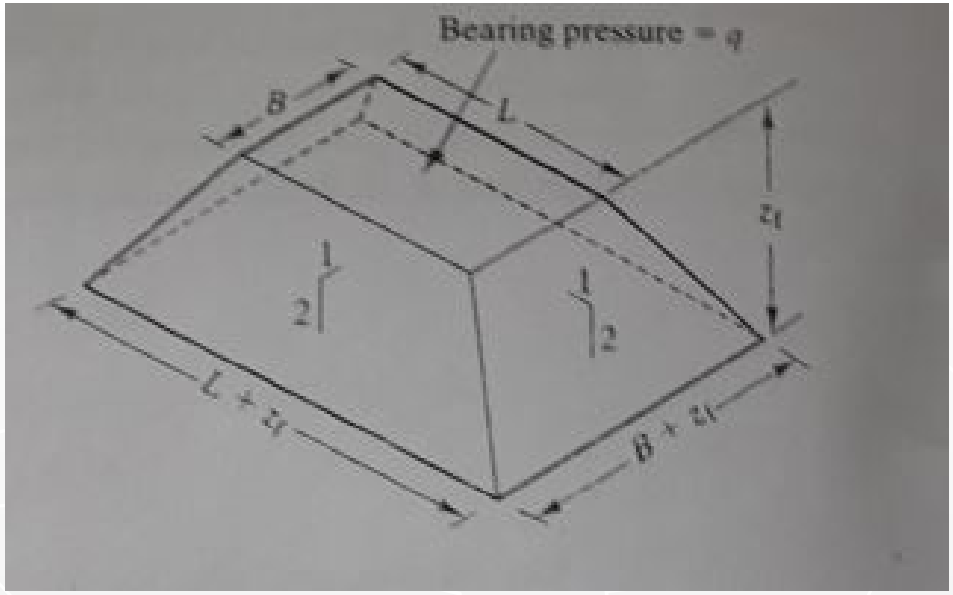


Fig.: Use of 1:2 method to compute the average $\Delta \sigma_z$ at a specified depth below the loaded area.

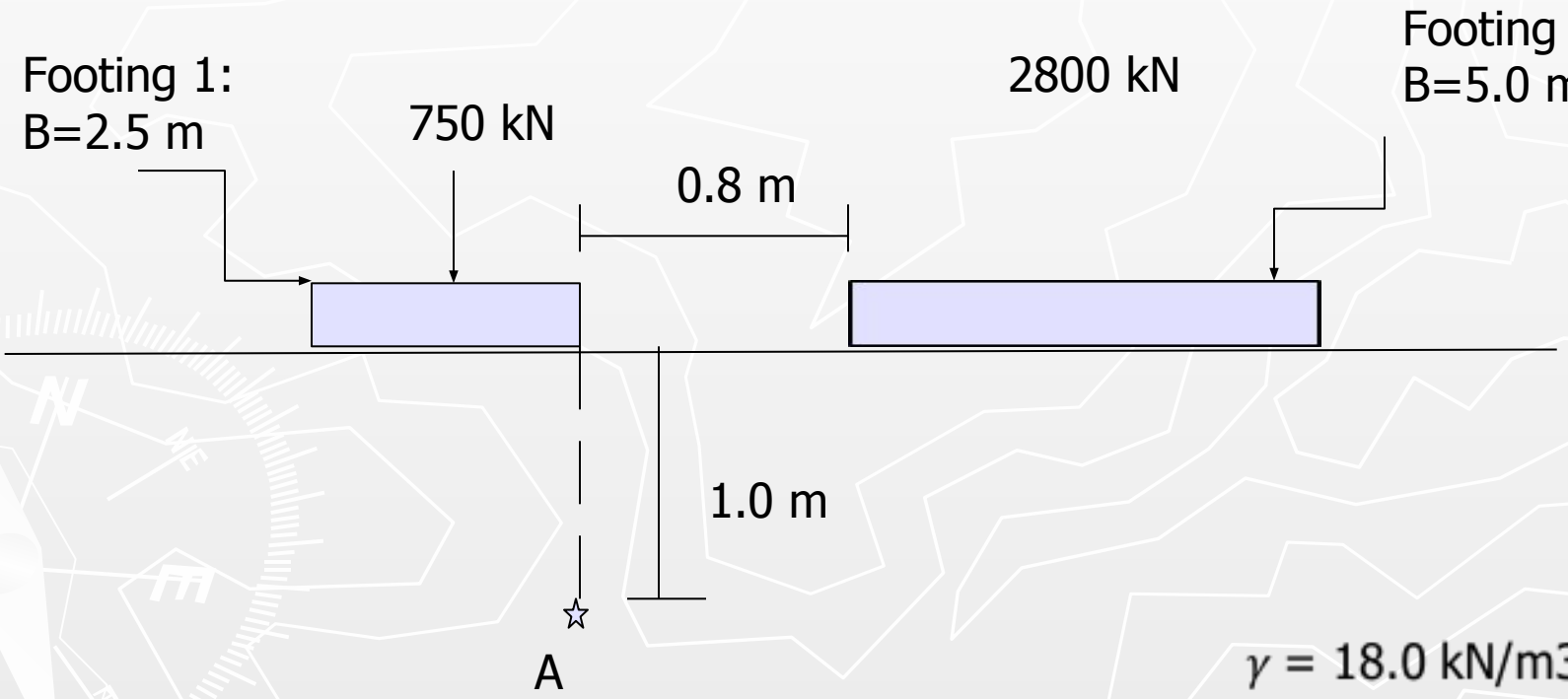
When applied to a rectangular loaded area of $B \times L$, the 1:2 method produces the following formula for the influence factor for vertical stress, I_σ, at a depth z_t:

$$I_\sigma = \frac{BL}{(B + z_t)(L + z_t)}$$

How to compute Stresses in soils

- Boussinesq's Method
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Ex. 4: Two square footings **carry vertical loads** as shown in the following figure. Compute the **magnitude of σ_z** at Point A under the edge of the smaller footing, considering both geostatic and induced stress.



Footing: 1

Footing: 2

Solution (Ex.4):

- Geostatic stress:

$$\sigma_z = \sum \gamma H = (18.0\ \mathrm{kN}/m^2)(1.0\mathrm{m}) = 18\mathrm{kPa}$$

- Bearing Pressures:

Footing 1: $q_1 = \frac{P}{A} = \frac{750kN}{(2.5\ m)^2} = 120\mathrm{kPa}$

Footing 2: $q_2 = \frac{P}{A} = \frac{2800kN}{(5.0\ m)^2} = 112\mathrm{kPa}$

Solution (Ex.4) (cont..):

- Relative z and x coordinates for point of interest:

Footing 1:

$$\frac{z_f}{B} = \frac{1.0m}{2.5m} = 0.40$$

From the figure, beneath the edge of footing $\frac{x_f}{B} = \frac{1.25}{2.5} = 0.50$

Therefore, $I_\sigma = 0.45$

$$\Delta \sigma_{z_1} = I_\sigma \, q_1 = 0.45 \text{ x } (120.0 \text{ kPa}) = 54 \text{ kPa}$$

Footing 2:

$$\frac{z_f}{B} = \frac{1.0m}{5.0m} = 0.20$$

From the figure, beneath the edge of footing $\frac{x_f}{B} = \frac{2.5+0.8}{5} = 0.66$

Solution (Ex.4):

Therefore, $I_\sigma = 0.15$

$$\Delta \sigma_{z_2} = I_\sigma \, q_1 = 0.15 \text{ x } (112.0 \text{ kPa}) = 16.8 \text{ kPa}$$

$$\Delta \sigma_{z_{final}} = \sigma_z + \Delta \sigma_{z1} + \Delta \sigma_{z2}$$

$$= 18 \text{ kPa} + 54 \text{ kPa} + 16.8 \text{ kPa}$$

$$= 88.8 \text{ kPa} \quad \text{Ans.}$$

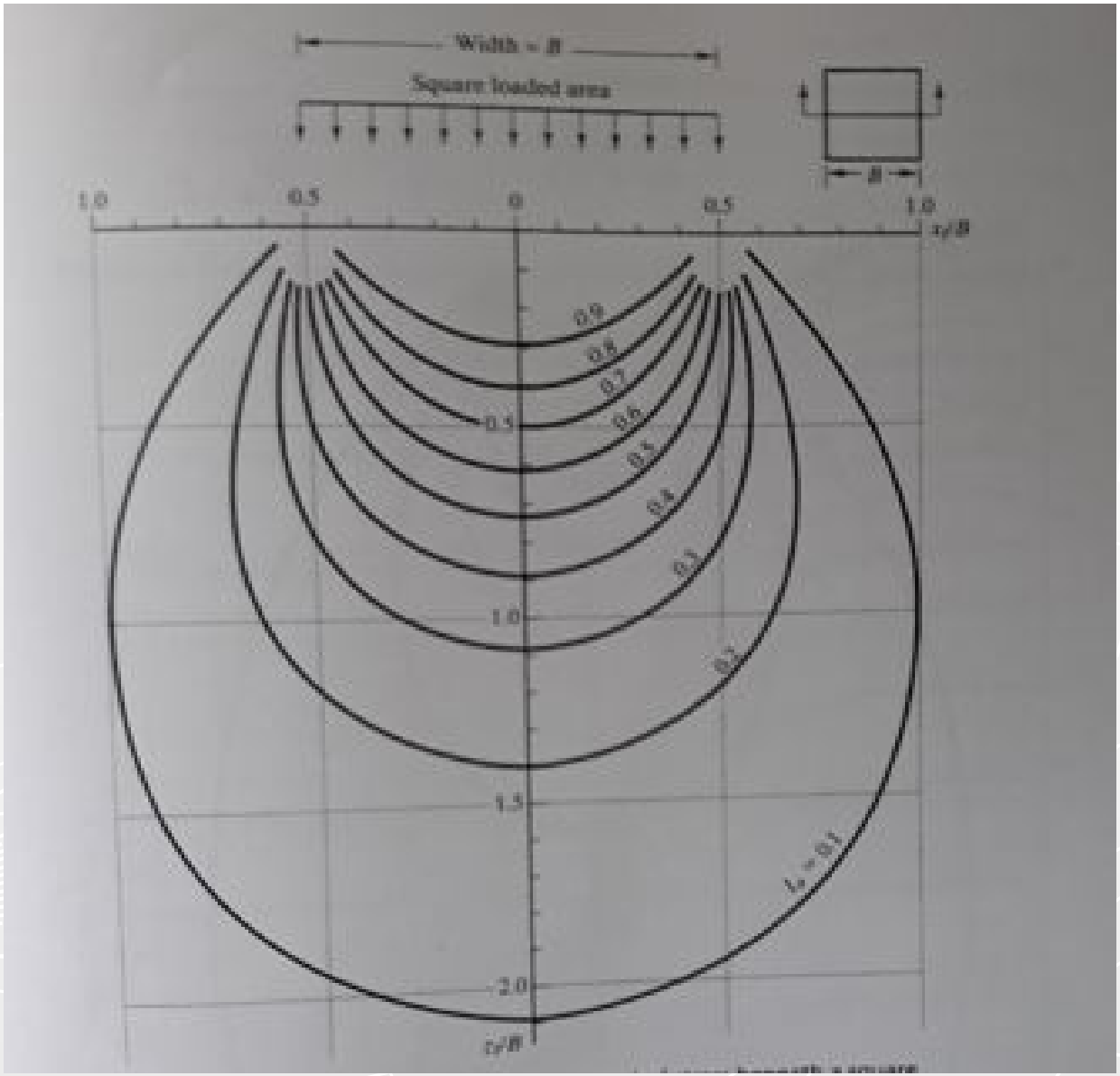
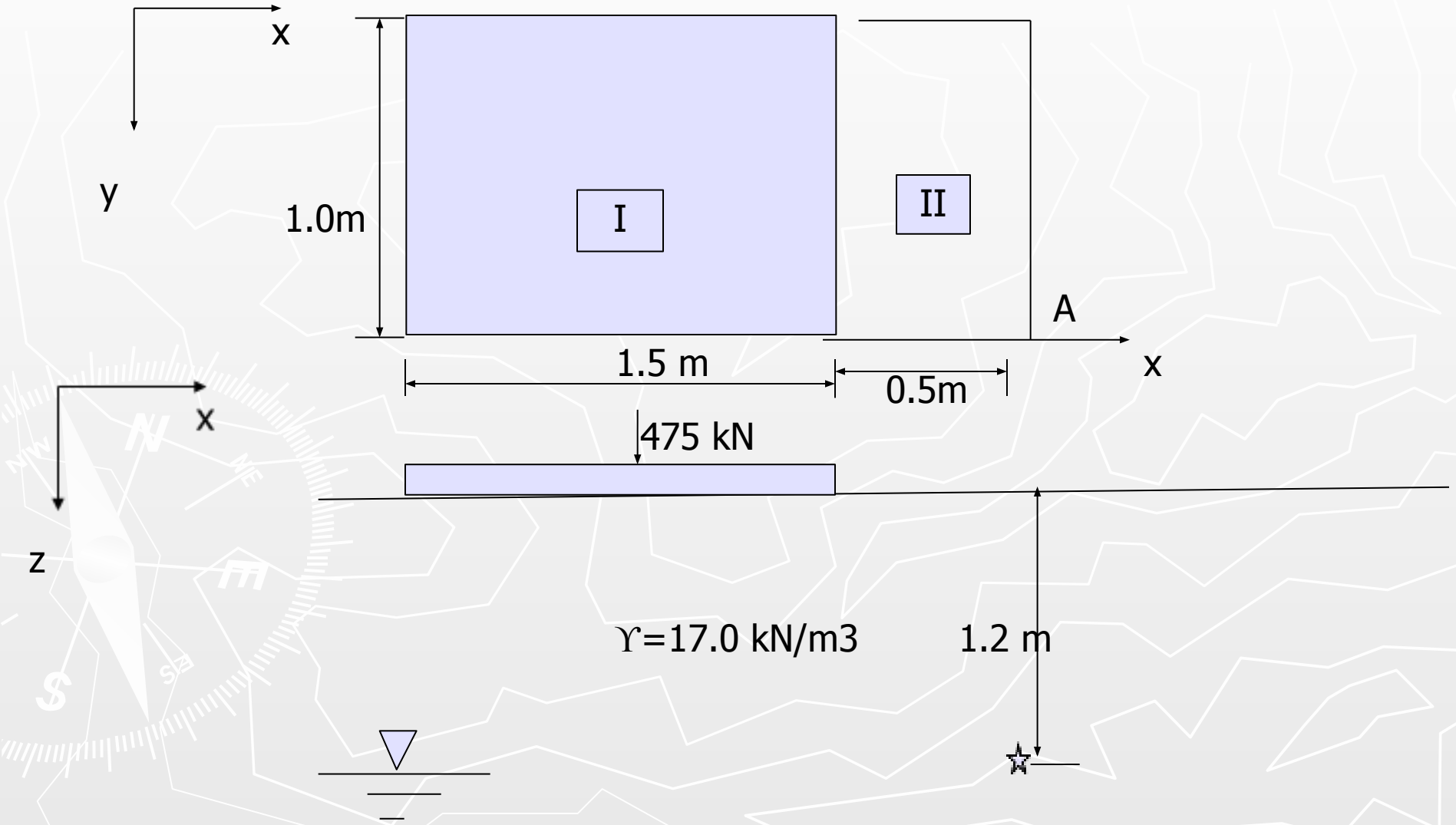


Figure of Ex. 4: Influence factor for induced vertical stress under square loaded area, per Boussinesq.

Ex. 5: The **1.0m X 1.5m** footing shown in the following figure, supports a **vertical load of 475 kN**. Compute the magnitude of σ_z at point A, considering both **geostatic and induced stresses**.



Solutions of Ex. 5:

- **Geostatic stress**:

$$\sigma_z = \sum \gamma \mathbf{H} = (17.0\ \text{kN}/m^2)(1.2\text{m}) = 20.4\ \text{kPa}$$

Induced stress:

- Equations below used to compute σ_z, below the corner of a loaded rectangle, but point A is not beneath the corner.
- Stress can be computed beneath a **1st fictitious footing** (I + II), which is 1.0m wide and (1.5m+0.5m=) 2.0m long.
- **2nd fictitious footing II**, which is 0.5m wide and 1.0m long.
- **Both have a corner over point A.**
- Have the **same bearing pressure as the real footing**.

Solutions of Ex. 5 (cont..)

- **Bearing pressures,**

$$q = \frac{P}{A} = \frac{475\ kN}{(1.0m)(1.5m)} = 317\ \text{kPa}$$

- **Footing (I+II):**

B=1.0. L=2.0, z_f= 1.2, $B^2 + L^2 + z_f^2$=6.44 > B^2L^2/z_f^2=3.34

Hence, equation 2 to be used.

Therefore, I_σ=**0.182**

σ_z =(0.182)(317)= **57.7kPa**

- **Footing (II):**

B=0.5. L=1.0, z_f= 1.2, $B^2 + L^2 + z_f^2$ =2.69> B^2L^2/z_f^2=0.02

Hence, equation 2 to be used.

Therefore, I_σ=**0.098**

σ_z =(0.098)(317)= **31.1 kPa**

Hence, $\Delta\sigma_z$ = **57.7 – 31.1 = 26.6 kPa**

Combined results σ_z= **20.4+ 26.6** = 47.0 kPa Ans.

Thank you

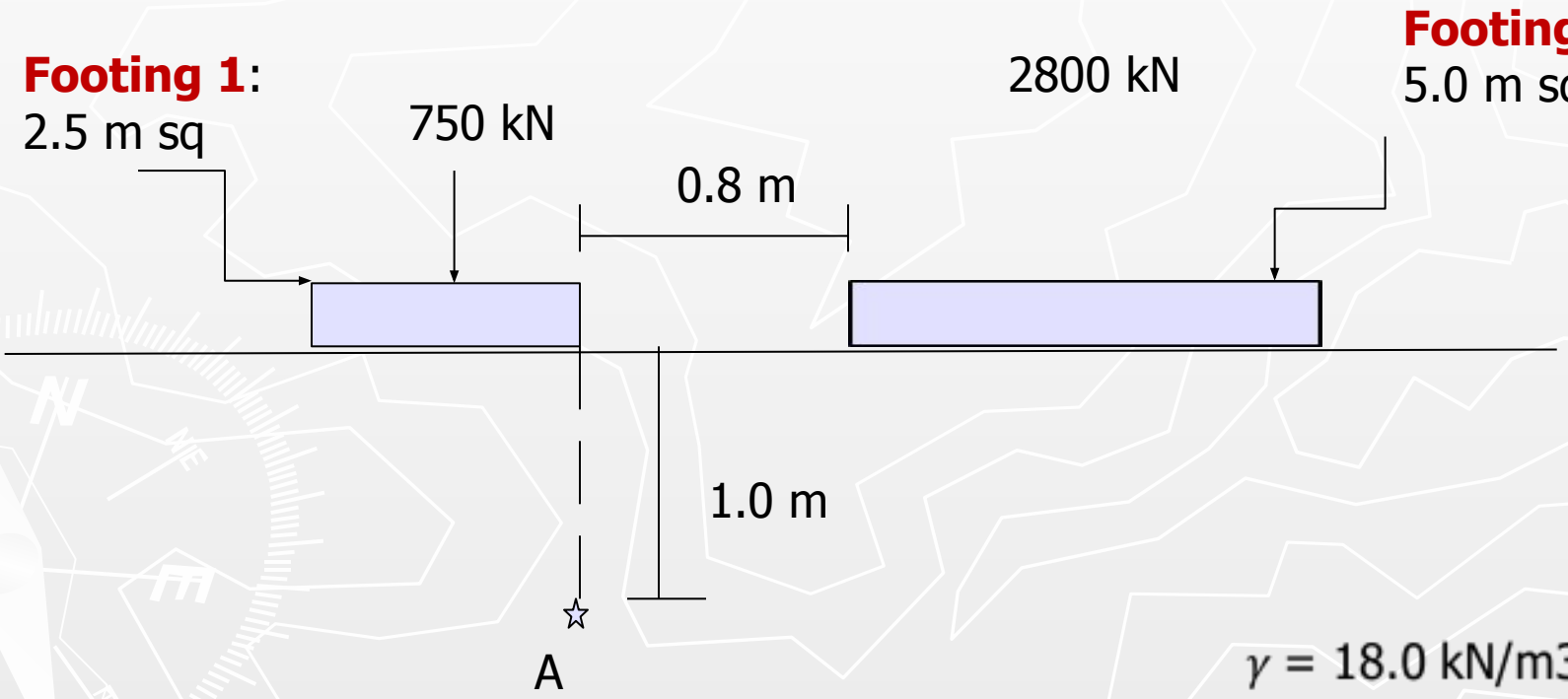
EWCE 341

Principles of Soil Mechanics

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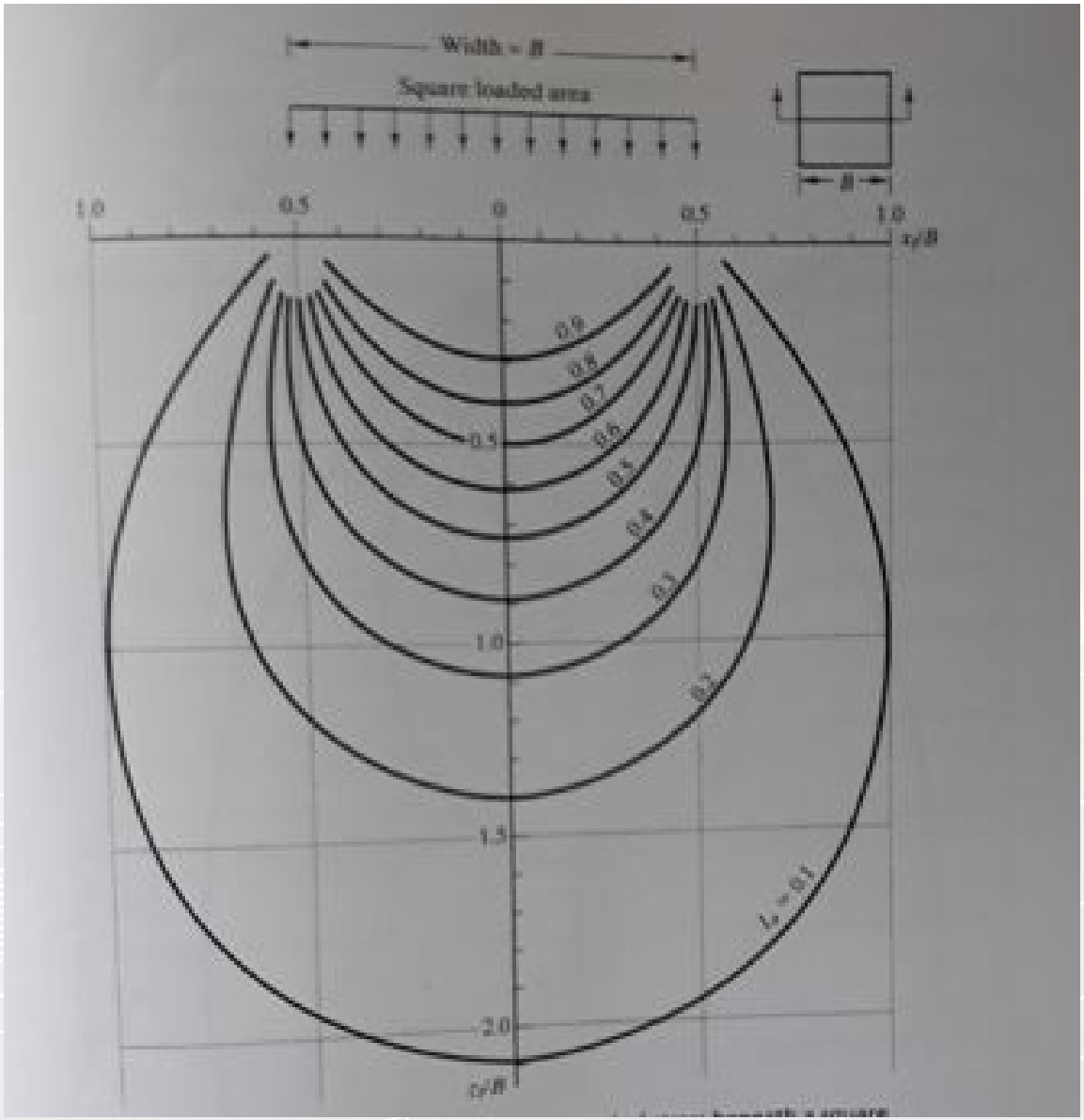
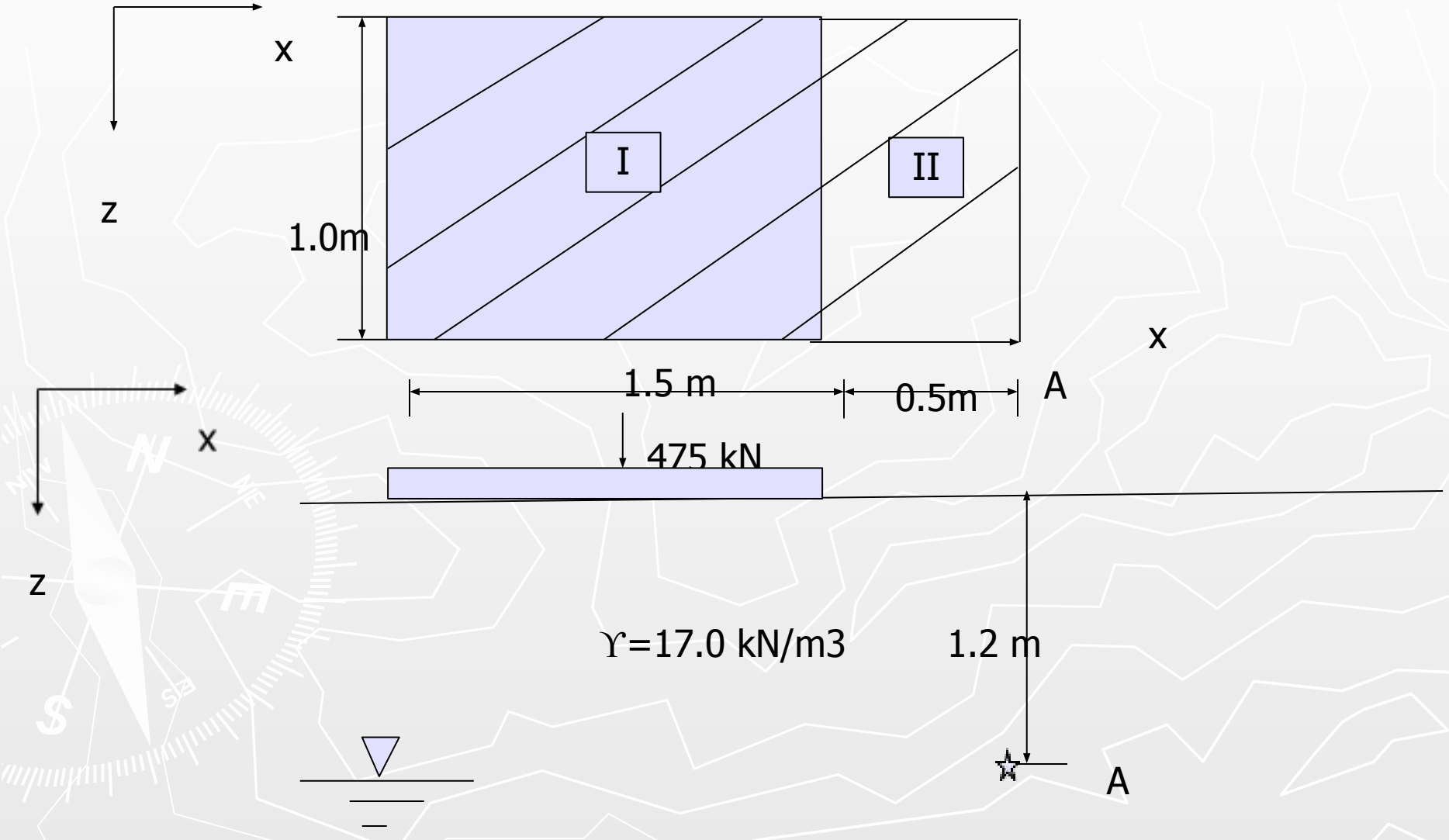


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Combined results σ_z= **20.4+ 26.6** = 47.0 kPa Ans.

If $B^2 + L^2 + z_t^2 < B^2L^2/z_t^2$,

$$I_\sigma = \frac{1}{4\pi}\left[\left(\frac{2BLz_t\sqrt{B^2+L^2+z_t^2}}{z_t^2(B^2+L^2+z_t^2)+B^2L^2}\right)\left(\frac{B^2+L^2+2z_t^2}{B^2+L^2+z_t^2}\right)\right.$$

$$\left. + \pi - \sin^{-1}\frac{2BLz_t\sqrt{B^2+L^2+z_t^2}}{z_t^2(B^2+L^2+z_t^2)+B^2L^2}\right]$$

Equation-1

Otherwise,

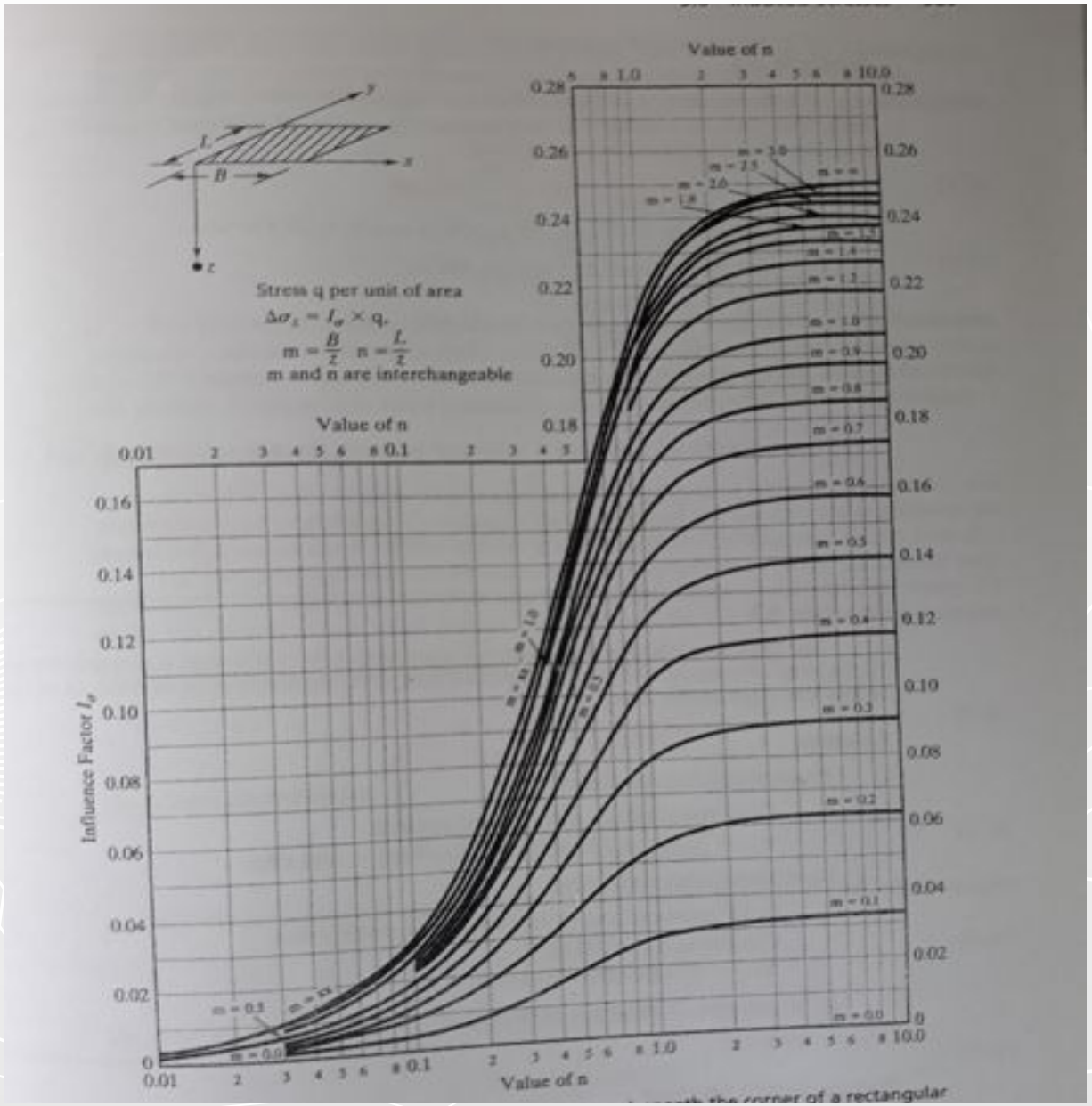
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$$\left. + \sin^{-1}\frac{2BLz_t\sqrt{B^2+L^2+z_t^2}}{z_t^2(B^2+L^2+z_t^2)+B^2L^2}\right]$$

Equation-2

Figure of Ex.5

Figure 3: **Influence factor for induced vertical stress beneath the corner of a rectangular loaded area**.



Stresses in soils:

- All civil Engineering projects impart loads on the ground that supports them.
- **Loads produces stresses in soils** ie **compressive stress**, **shear stress** and **even tensile stress**.

- **Compressive stresses** ie geostatic stresses and induced stresses developed in soils , **are carried partially by the solid particles and partially by the pore water**.

Total stress:

- **Sum of the stresses** which are carried by the two phases of soils ie solid and water.
- Total stress can be very useful if we **divide into two parts**.

What is effective stress in soils?

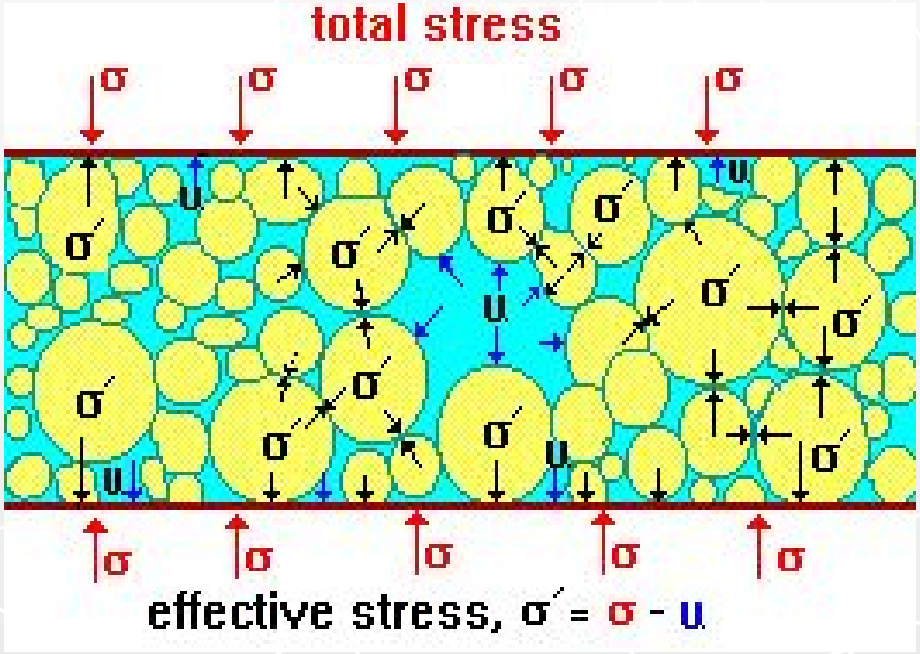
Total stress can be divided into two parts:

Effective stress (σ')

- Portion of stresses that are carried by the solid particles of soils.

Pore water pressure (u):

- Portion of stresses carried by the pore water.



Pore Water Pressures (cont..)

- Denoted by the symbol **u**.

$$\mathbf{u} = \gamma_w z_w$$

where, γ_w = unit weight of water,

z_w = depth of water in soils.

- Used for **finding the effective stress** beneath the soil.

- Difference between pore water stress and total stress is said to be effective stress.

- **Pore pressure in soil is divided into two categories:**
  - -Negative pore pressure and
  - -Positive pore pressure.

Positive pore pressure:

- **Z**one of soil that is below water table will be fully saturated.
- **Pressure in the pore is greater than atmospheric pressure.**
- Hence, it is said to be positive pore pressure.

Negative pore pressure:

•Above the water table, **soil may remain saturated or unsaturated and pressure in pore is lower than the atmospheric pressure**.

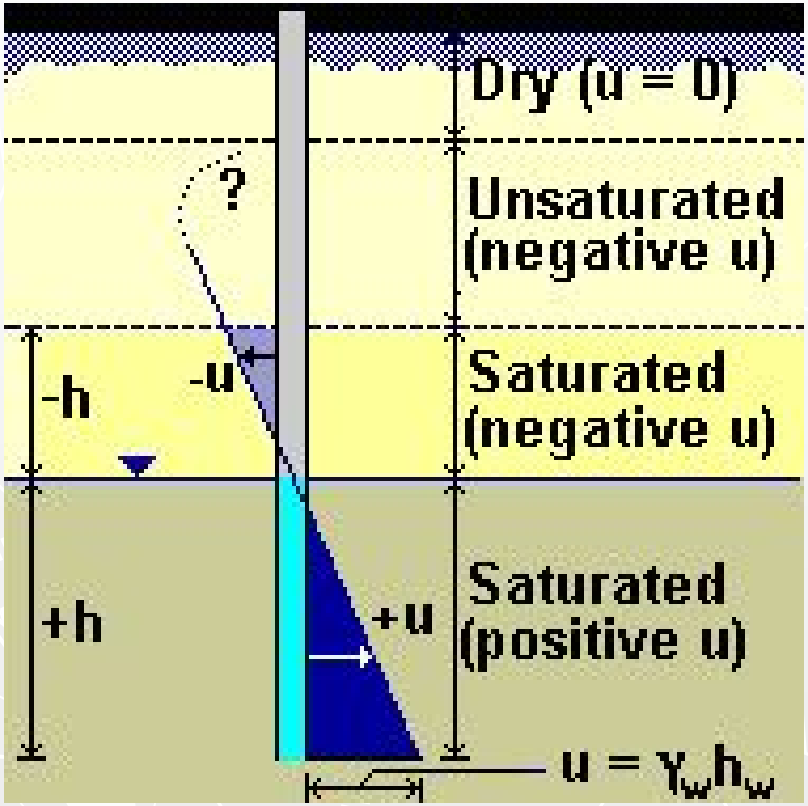
•Hence, it is said to be negative pore pressure.

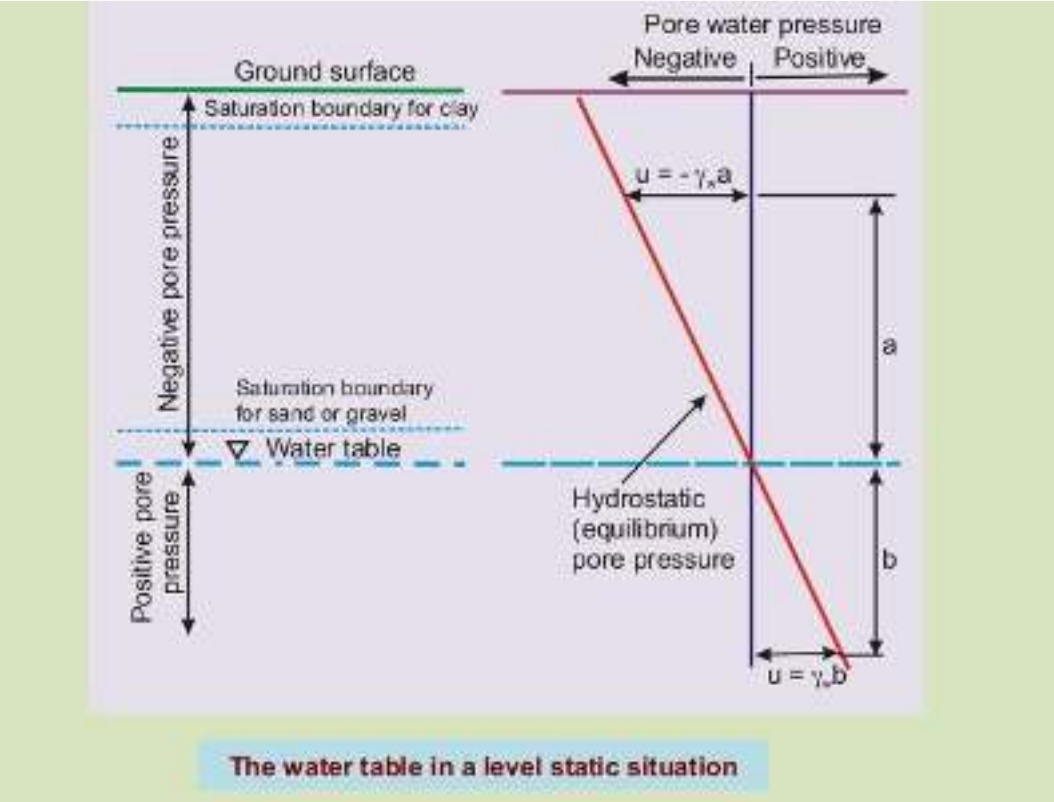
Below the water table, pore pressure is **positive;**

In dry soils and at the level of the water table, pore pressure is **zero;**

Above the water table, soil can remain saturated due to capillary rise but with **negative** pore pressure (suction).

Ideal Scenario for +ve and –ve Pore Pressure (u)





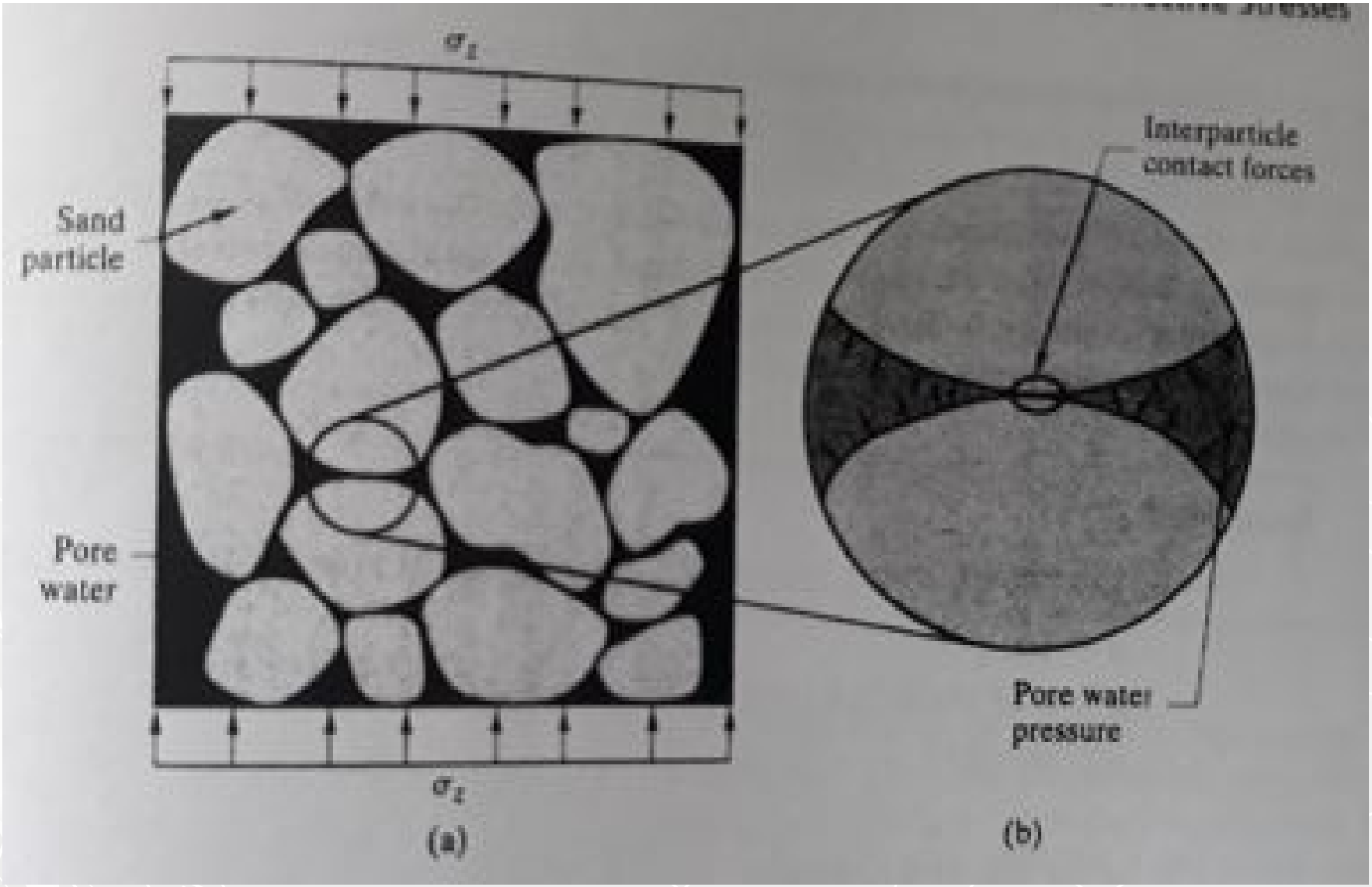


Figure: Division of total stress into effective stress and pore water pressure.

Determination of vertical Effective Stress

where,

σ'_z = vertical effective stress

σ_z = vertical total stress

u = pore water pressure

- If only **geostatic stresses** are present,

$$\sigma'_z = \sum \gamma H - u$$

If only **Induced stresses** are present,

$$\sigma'_z = \sum \Delta\sigma_z - u$$

- If both **geostatic and induced stress** are present,

$$\sigma'_z = \sum \gamma H + \sum \Delta\sigma_z - u$$

=Total stresses – Pore water pressure

Horizontal Effective stresses:

- Ratio of the horizontal to vertical effective stresses is defined as the ***coefficient of lateral earth pressure, K.***

- For **geostatic stresses beneath a level ground surface**, is normally assumed that k in the x direction is equal to that in the y direction.

$$\mathbf{K} = \frac{\sigma'_x}{\sigma'_z}$$

- If ground surface is inclined, or if induced stresses are present, k may be different in x and y direction.

- Values of k varies about 0.3 to 3.0.

Stresses beneath the bodies of water:

Importance:

- Geotechnical engineers need to **compute stresses beneath the water** such as lakes, rivers or oceans.
- Might be necessary while **designing foundation for a bridge.**
- Might be necessary when evaluating the **stability of slopes that extends under water.**

Computation of Effective Stresses beneath the bodies of water:

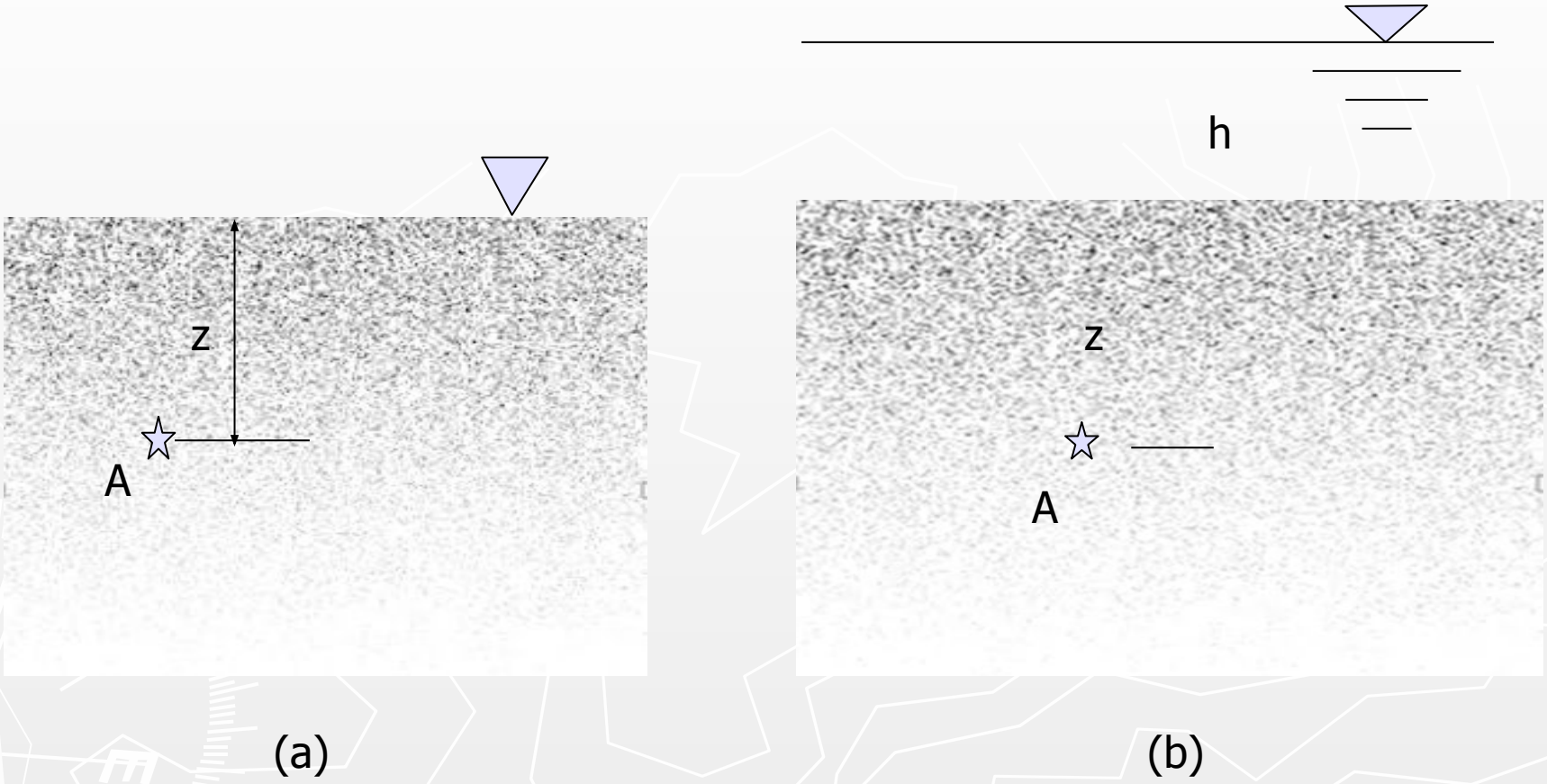


Figure: Computation of effective stress beneath bodies of water

Stresses beneath the bodies of water (cont..):

When GWT at ground surface, stress at Point A :

- **Fig. (a)**

$$\sigma_z = \gamma z$$

$$u = \gamma_w z$$

$$\sigma'_z = \sigma_z - u = \gamma z - \gamma_w z = z(\gamma - \gamma_w) = z\gamma_b$$

- **Fig. (b)**

$$\sigma_z = \gamma z + \gamma_w h$$

$$u = \gamma_w (z + h)$$

$$\sigma'_z = \sigma_z - u = (\gamma z + \gamma_w h) - \gamma_w (z + h) = z(\gamma - \gamma_w) = z\gamma_b$$

- **Water above the ground surface does not change the effective stress,** because it **contributes equally** to both the total stress and pore water pressure.

- We know, the buoyant unit, γ_b of a soil is equal to the total unit weight of soil less the unit weight of water.

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

$$z\,\gamma_b = z\,\gamma_{sub}$$

Q: Explain with example.

Explain the stress conditions with negative pore water pressure in soils.

Stress conditions with negative pore water pressure or soil suction:

- Most geotechnical analyses **assume** that the pore water pressure above the GWT is zero. Therefore, effective stress is equal to the total stress.
- In reality, **soils above GWT generally have –ve pore water** pressures also known as soil suction, which means the effective stress is greater than the total stress.

Vadose Zone

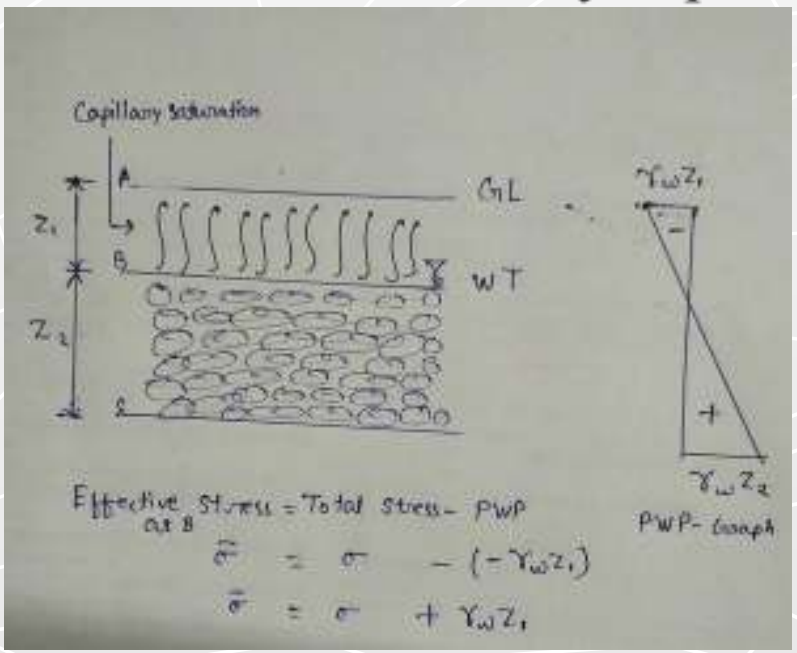
- In the figure, within the **vadose zone**, the soil remains saturated in the zone of capillary rise.

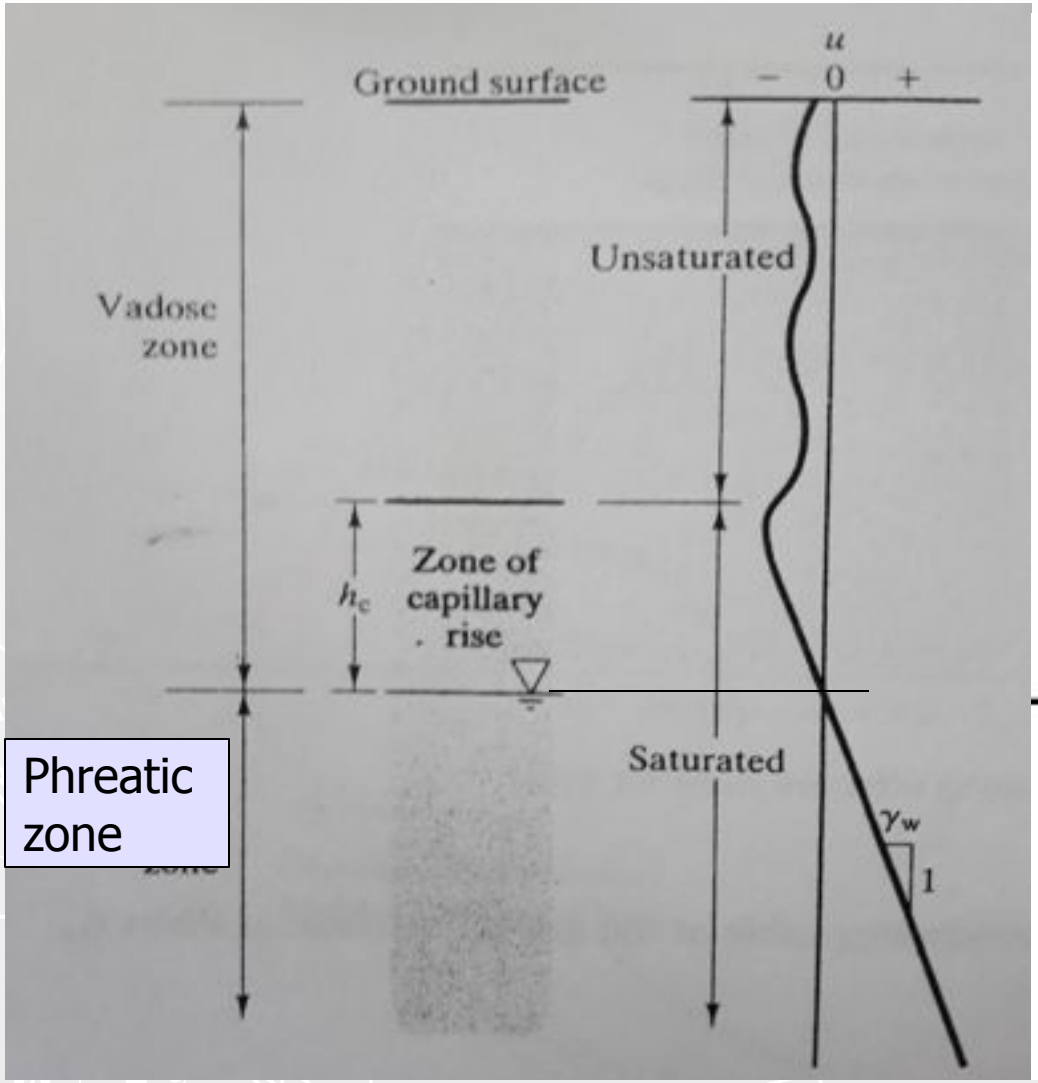
Zone of Capillary Rise:

- In the zone of capillary rise, the u may be computed by using the equation $\mathbf{u=\gamma_w z_w}$, except $\mathbf{z_w}$ **is now negative**.

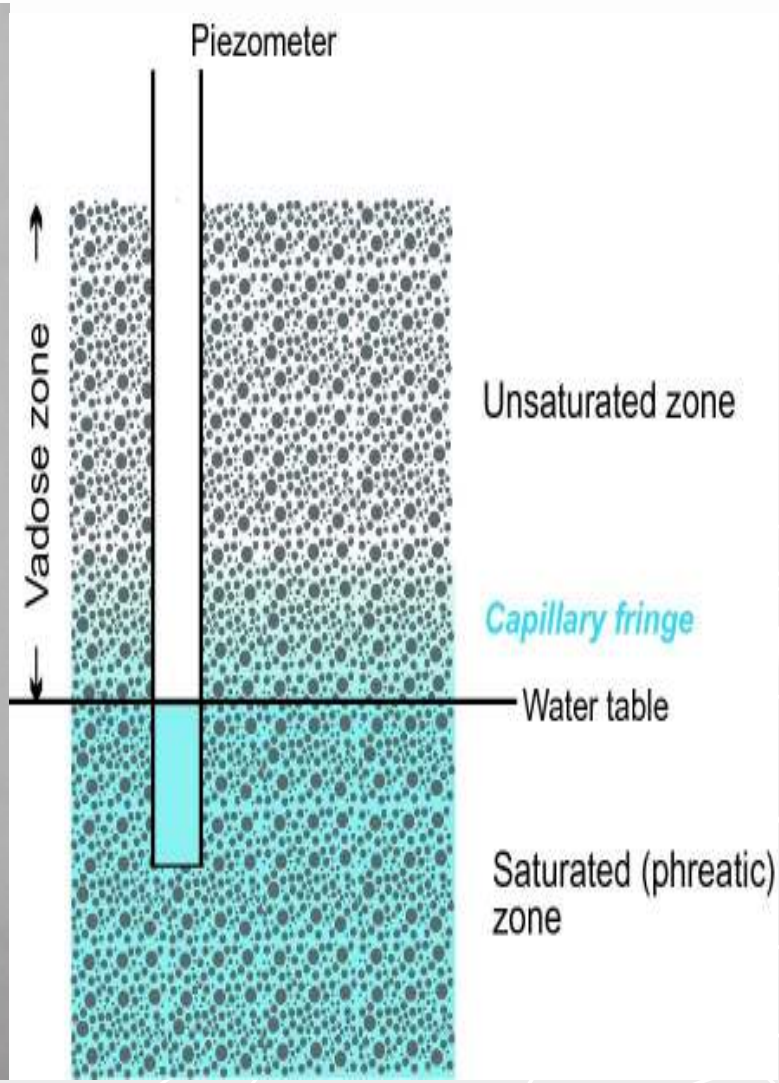
Capillary zone: Immediately above the water-table, into which **water may be drawn upwards as a consequence of capillary action** (intermolecular forces between the liquid and surrounding solid surfaces).

- This **produces a negative u** ie the pore water in the capillary zone may be visualized as a column of water held in tension by capillary forces.





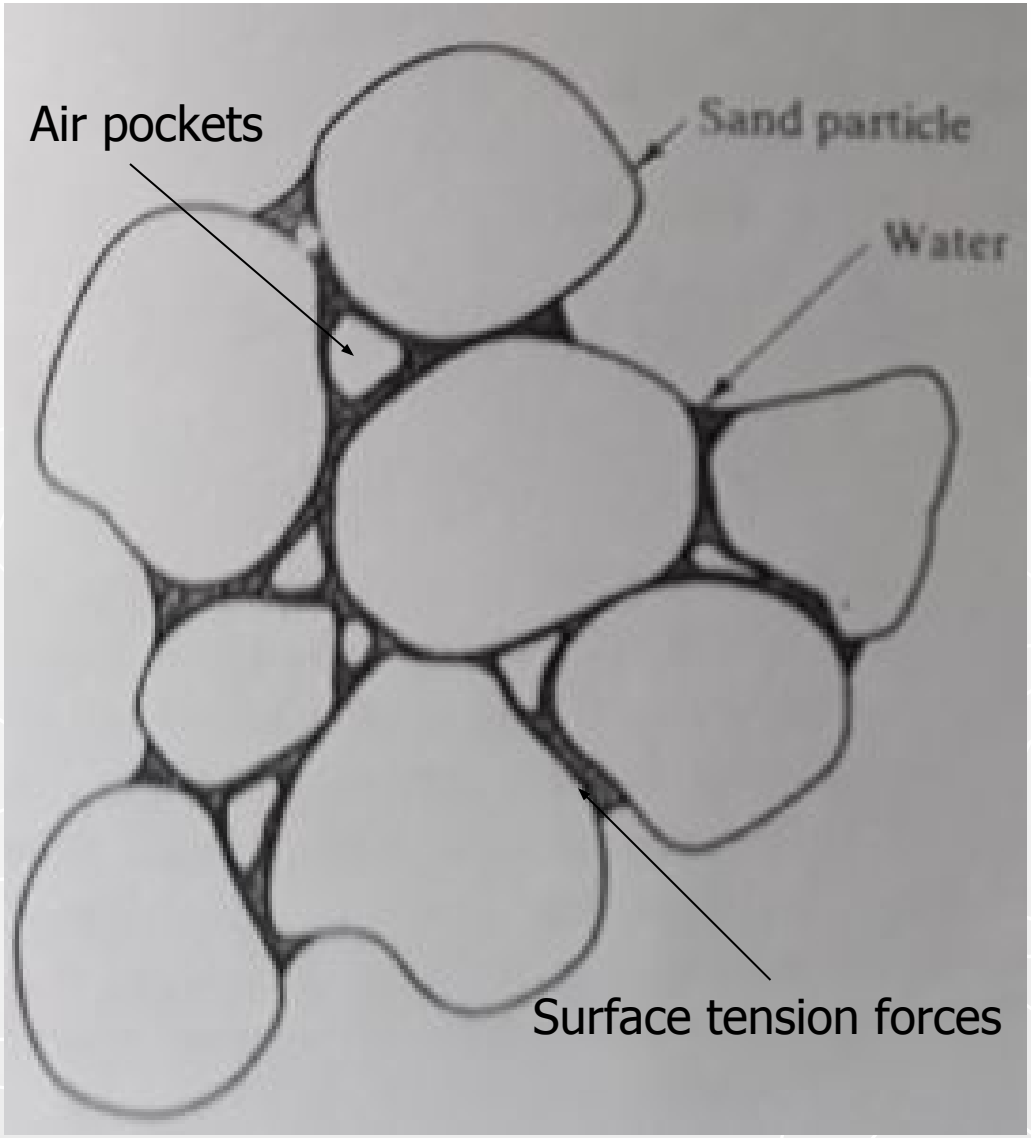
Phreatic zone



A plot of pore water pressure vs elevation.

Unsaturated zone:

- Above the Zone of Capillary Rise, the **degree of saturation falls well below 100% .**
- Pore water **collects into small capillary zones adjacent to particles contact points.**
- **Surface tension forces develop** between **this water and the particles,** creating tensile forces (-ve u) in the water.
- In the **air pockets, the pressure is atmospheric**.
- Negative pore water pressures can be measured in situ or in the laboratories.



Above the capillary zone, the pore water retreats to particle contact points.

Thank you

EWCE 341

Principles of Soil Mechanics

Compressibility and Settlement

Compressibility and Settlement

- **Settlement and Subsidence**
- **Issues of Geotechnical Engineers**
- **Criteria to compute the Magnitude of Settlement**
- **Different Types of Settlement:**
- **Changes in Vertical Effective Stress**

Compressibility and Settlement

Settlement and Subsidence:

Settlement:

- Many civil engineering projects **impart loads** onto the ground, which produce corresponding **increase in the vertical effective stress, σ'_z.**
- Stress increases **induce vertical strains, ε_z** in the soils, which cause the **ground surface to move downward**.
- Downward movement is called **Settlement**.

Subsidence

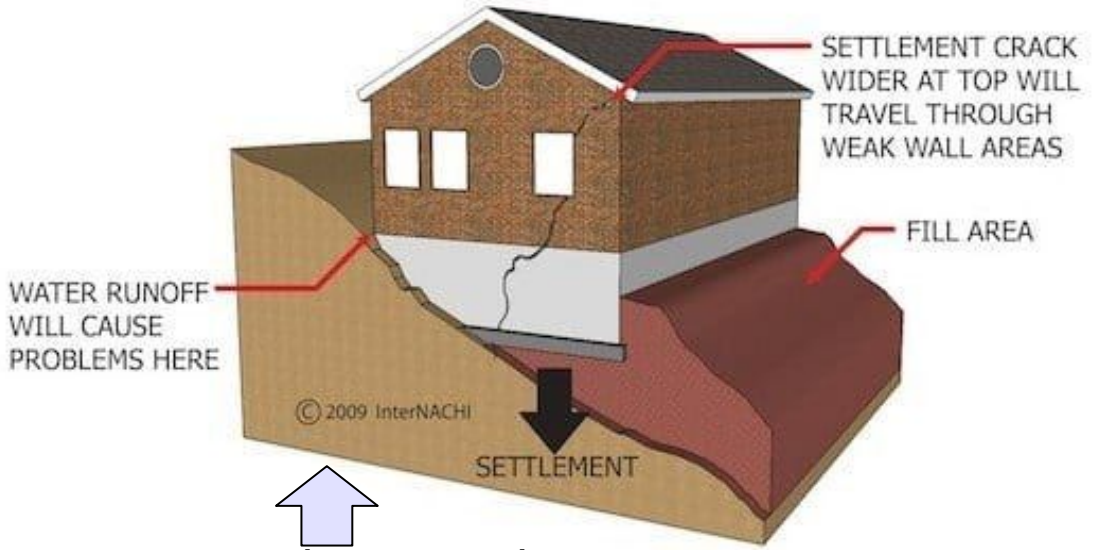
- Whenever **settlement occurs over a large area**, it is sometimes called **subsidence**.
- **Land subsidence** is a **gradual settling** or sudden sinking of the Earth's surface
- Often caused by **human activities** like excessive groundwater pumping or **natural events** such as earthquakes or soil compaction.

Indonesia - July 21 2019 : land subsidence



Building settlement





Foundation Settlement



Subsidence opening up the earth, threatening local infrastructure and people's homes.

Issues of Geotechnical engineers related to soil settlement:

- Whenever σ'_z **increases**, there is always will be a **corresponding settlement** δ.
- **Issue facing a geotechnical engineer is not if settlement will occur,** but rather:

  -magnitudes of these settlements and

  -how they compare with the tolerable limits.

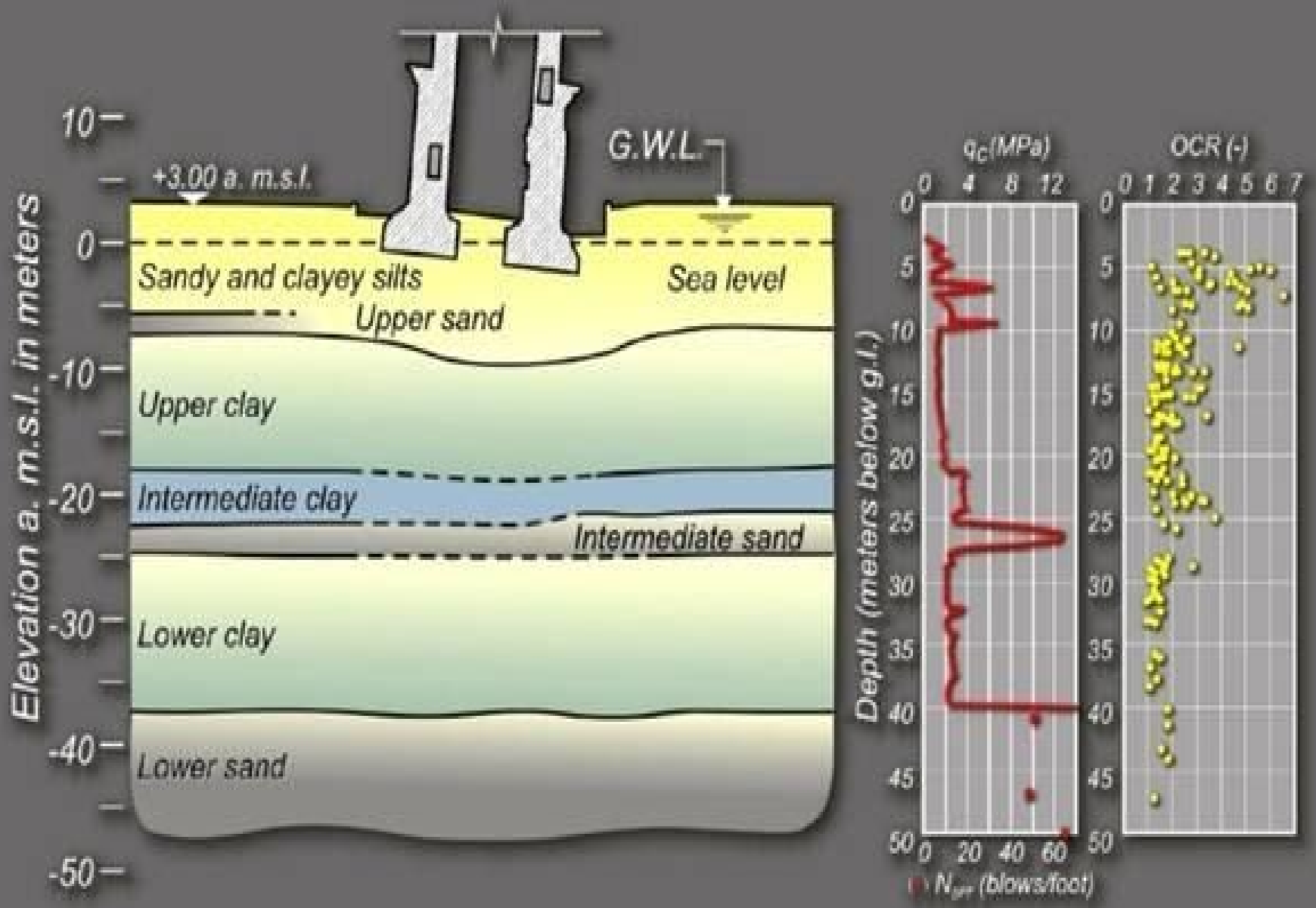
A classic example of **total settlement: Palace of Fine Arts,** cultural center in Mexico City, which has sunk **more than 15 feet (4.6m)** since its construction in the early 20th century.





Example of differential settlement: **Leaning Tower of Pisa**, which leans due to **insufficient bearing capacity** of the soil beneath one side of the **tower**.

SUBSOIL PROFILE AND SOIL PROPERTIES



Criteria to compute the Magnitude of Settlement.

- **Settlement is a displacement caused by a change in stress.**

- **Settlement can be computed if we know:**
  - -the **magnitude of the change in stress** and
  - -the **stress-strain properties of soils**.

Different Types of Settlement:

- **Distortion Settlement (δ_d)**
- **Consolidation settlement (δ_c)**)/Primary Consolidation settlement.
- **Secondary Compression Settlement (δ_s).**

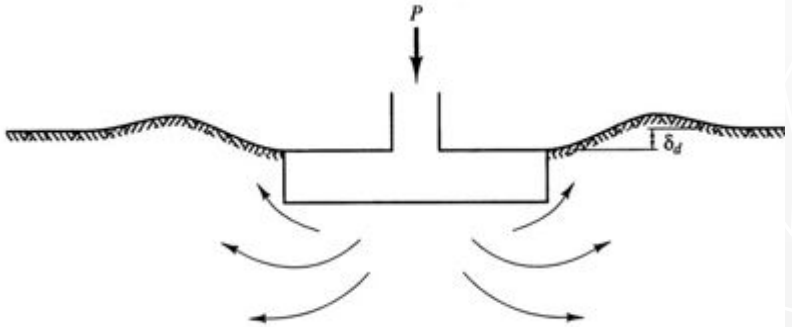
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Different Types of Settlement:

Distortion Settlement (δ_d):

Distortion Settlement



- Results **from lateral movements of the soils** in response to **changes in** σ'_z.
- Occurs **without volume change**.
- **Similar to Poisson's effect** where an object loaded with in the vertical direction **expands laterally**.
- Primarily occurs when **the load is confined to a small area**, such as **structural foundation** or **near the edges of large loaded areas**, such as **embankment**.

Settlement of Soil



Consolidation settlement (δ_c):

- Also known as **primary consolidation settlement**.
- Occurs when a soli is subjected to an **increase in σ'_z and individual particles respond by arranging into tighter packing.**
- **Volume of the solid particles** themselves remains **virtually unchanged**.
- **Change in total volume** (and the resulting change) is due to a **decrease in the volume of voids, V_v**.
- **If soil is saturated**, the **reduction V_v can occur if some of the pore water is squeezed out of the soils**.

Consolidation Settlement (cont…)

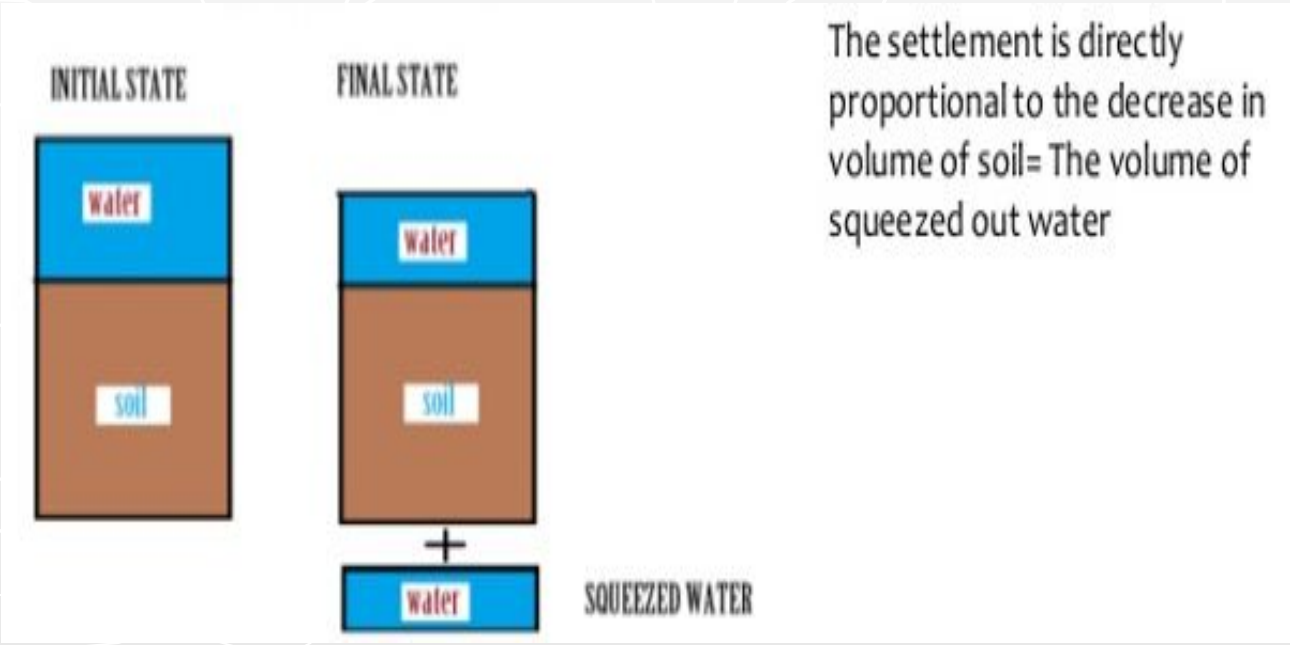
- In some cases, the process occurs as **quickly** as load is applied, but in other situation it occurs **very slowly**.

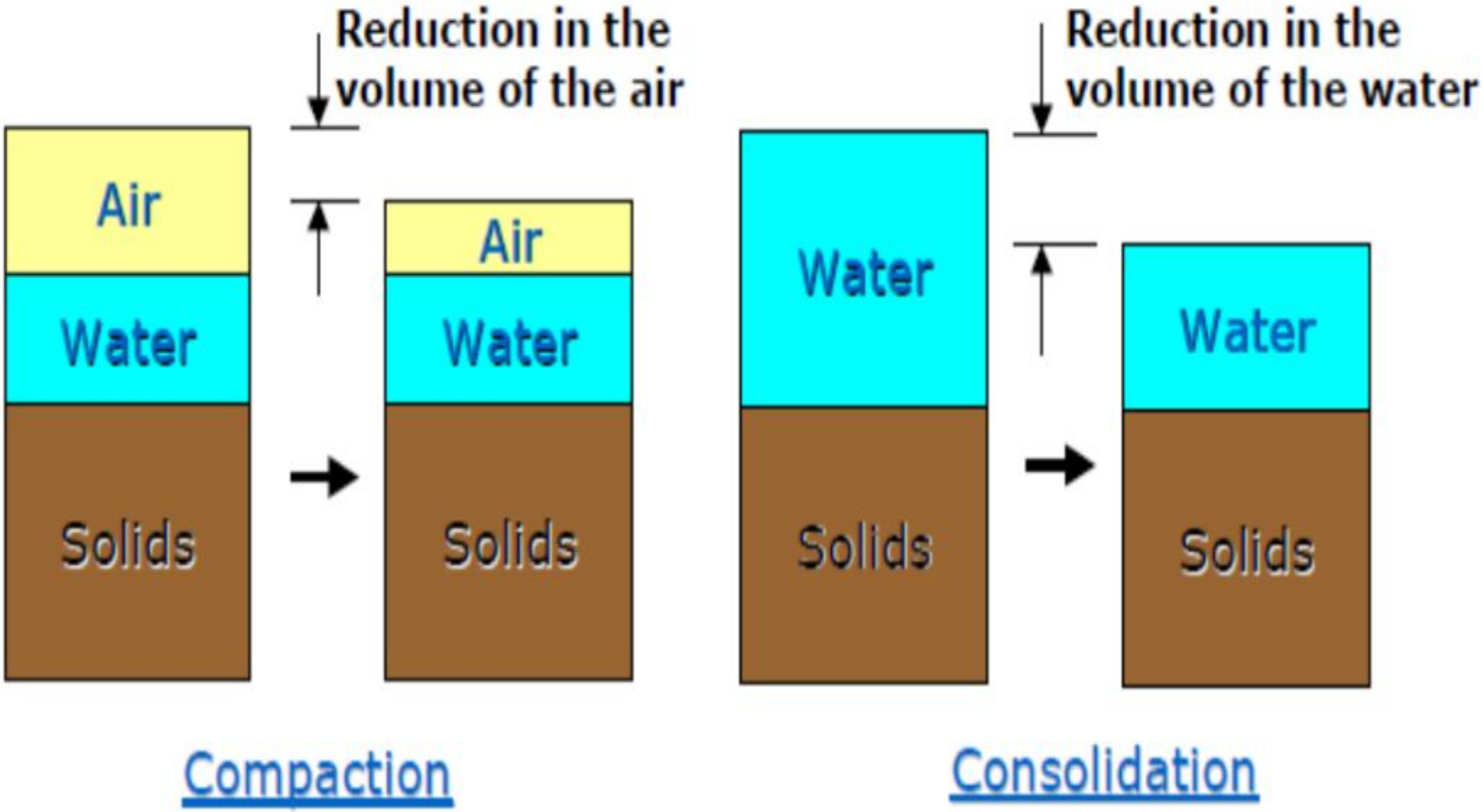
- **Rate of consolidation depends on**:

  -the degree of saturation of soils,

  -its hydraulic properties, and

  -other factors




Reduction in the volume of the air
Air
Water
Solids
Air
Water
Solids
Compaction
Reduction in the volume of the water
Water
Solids
Water
Solids
Consolidation

Secondary Compression Settlement (δ_s):

- Primarily due to **particle reorientation, creep, and decomposition of organic materials.**
- Particle **reorientation cause a reduction in the volume of voids**, like consolidation settlement.
- δ_s is not due change in σ'_z; it **occurs at a constant** σ'_z.
- δ_s is **always time dependent** and can be significant in highly plastic clays, organic soils and sanitary landfills.

Types of Settlement basing on the Duration:

Short-term settlement:

- Also known as *Immediate settlement.*
- Occurs as quickly as the load is applied.

Long-term settlement:

- Also known as delayed settlement.
- Occurs over some longer period.
- Many engineers associate consolidation settlement solely with **the long-term settlement of clay soils**.

Changes in Vertical Effective Stress:

Most Settlement is due to change in the vertical stress.

Initial condition:

- **Initial vertical effective stress, σ'_{z0}**, at a point in the soils **before the event** that causes settlement occurs.
- Initial condition consist of **geostatic stresses only** and estimated using the equation:

$$\sigma'_z = \sum \gamma H - u$$

Final condition:

- **Final vertical effective stress, σ'_{zf}**, is the value after the settlement process is complete.
- Method of computing σ'_{zf} depends on the **kind of event** that is causing the stresses to increase.

Changes in Vertical Effective Stress: (cont…)

- Most common events are:

  -**Placement of fill,**

  -Placement of external load, and

  -Changes in ground water elevation.

Stress Changes Due to Placement of Fill:

- When a **fill is placed** on the ground, σ_z' in the underlying soil increases due to the weight of the fill.

- If the **length and the width of the fill are large compared to the depth of the point** at which stresses to be computed, the fill can be called an **aerial fill and evaluation of the fill to be done as an Point load.**

- When the **point beneath the central area of the fill**, then σ'_{zf} can be computed by simply adding another layer to the $\sum \gamma H$.

$$\sigma'_{zf} = \sigma'_{z0} + \gamma_{fill} H_{fill}$$

$$\sigma'_{z0} = \sum \gamma H - u$$

Stress Changes Due to Placement of Fill (cont…)

Where,

σ'_{z0}= initial vertical effective stress

σ'_{zf}= final vertical effective stress

γ_{fill}= unit weight of the fill

H_{fill}= thickness of the fill

- If the **length or width of the fill are less than the twice the depth to the point** at which the stresses are to be computed as a point load, or

- If this point is **near the edge of the fill**, then **evaluation of the fill to be done as an area load.**

Ex. 1: A **5.0 ft** thick fill to be placed on a site underlain by medium clays as shown in the following figure 1, compute initial vertical effective stress, σ'_{z0} and final vertical effective stress, σ'_{zf} at point A.

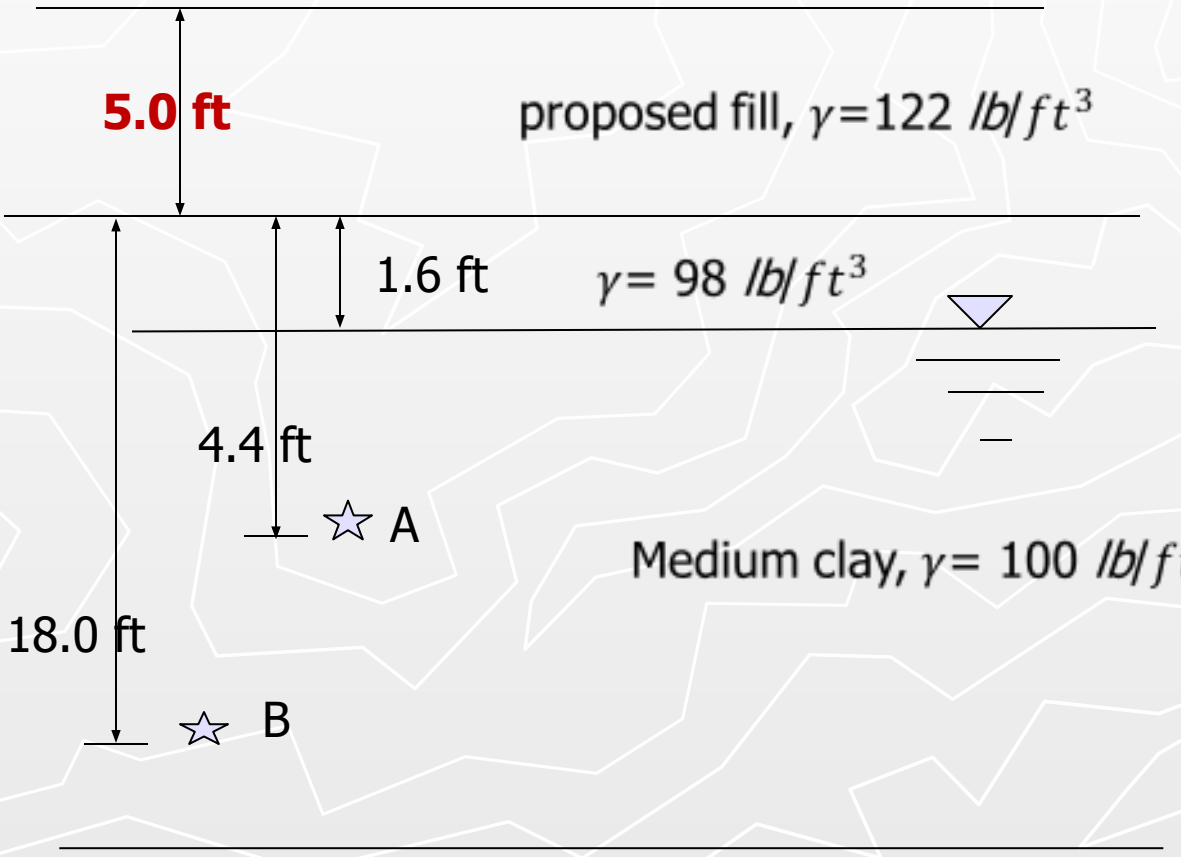


Fig. 1: Soil profile

Solution:

$\sigma'_{z0} = \sum \gamma H - u$

$= (98.0 \text{ lb}/ft^3)x(1.6ft) + (100.0 \text{ lb}/ft^3)x(4.4ft) - (62.4 \text{lb}/ft^3)x(4.4ft)$

$= 322 \text{ lb/ft}^2$ Ans.

$\sigma'_{zf} = \sigma'_{z0} + \gamma_{fill} H_{fill}$

$= 322 \text{ lb/ft}^2 + (122 \text{ lb}/ft^3)x(5.0ft)$

$= 932 \text{ lb/ft}^2$ Ans.

Stress Changes Due to Placement of External Loads:

- External loads such as structural foundations, produce increases in σ'_z.
- In such case: $\sigma'_{zf} = \sigma'_{z0} + \Delta\sigma_z$

where, $\Delta\sigma_z = I_\sigma q$

$\Delta\sigma_z$= induced vertical stress

I_σ= influence factor for vertical stress

q=bearing pressure for vertical stress

$q = \frac{P}{A}$ where, P=applied vertical load

A= area upon which the load acts.

Stress Changes Due to Changes in GWT Elevation:

Change of value of u:

- When the **GW table changes** from one elevation to some **lower elevation**, **u in the underlying soils decreases** and σ_z' increases.

- **More subtle process**, because there is no visible source of loading at the ground surface, and yet it can be and has been the **cause of significant settlements**.

- **Example**: Some of the settlement problems in some City, have been due to excessive pumping from the water supply wells and corresponding drops in GWT.
- Here, $\sigma_z' = \sum \gamma H - \mathbf{u}$, with the final GW position.

Stress Changes Due to Changes in GWT Elevation (cont…):

Change of value of γ:

- **Changes in GWT elevation may change the γ of soils.**
- Soils that is now above the GWT will probably have **lower moisture content, have a lower γ than before.**
- Zone of soils between **the initial** and **final** GWTs may have **one γ for the σ'_{z0} computation and another γ for σ'_{zf} computation.**
- Some civil engineering projects include **multiple causes of settlement** acting simultaneously.

Example: A project might include both placement of fill and construction of multiple structural foundation.

Here, σ'_{zf} can be computed by $\sigma'_{zf}=\sum\gamma H + \Delta\sigma_z - u$, with post construction conditions.

Thank you

EWCE 341

Principles of Soil Mechanics

Compressibility and Settlement

Distortion Settlement

- When **heavy loads are applied** over a small area, **soil can deform laterally.**

- **Lateral deformations produce additional settlement at ground surface**, which is called **distortion settlement,** δ_d.

- Distortion settlement is generally **much smaller than consolidation settlement, and can be ignored**.

- It is sometimes **considered in the design of spread footing foundations.**

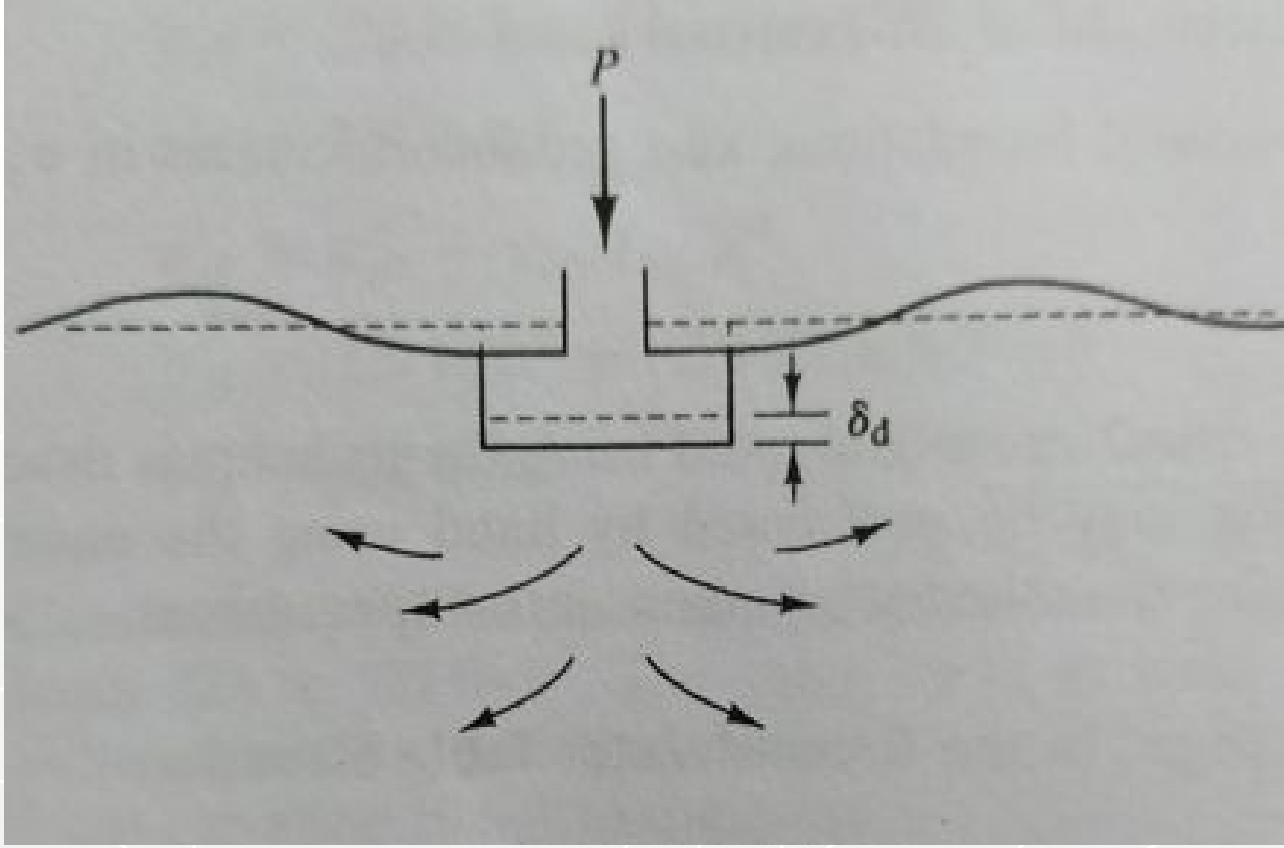




Figure: Distortion settlement beneath a small loaded area.

Consolidation Settlement:

- Term ***consolidation*** is used to described the **pressing of soil particles into a tighter packing** in response to in increase in effective stress.
- It is assumed that the **volume of solids remain constant** ie **the compression of individual particles is negligible**; only the **volume of voids changes.**

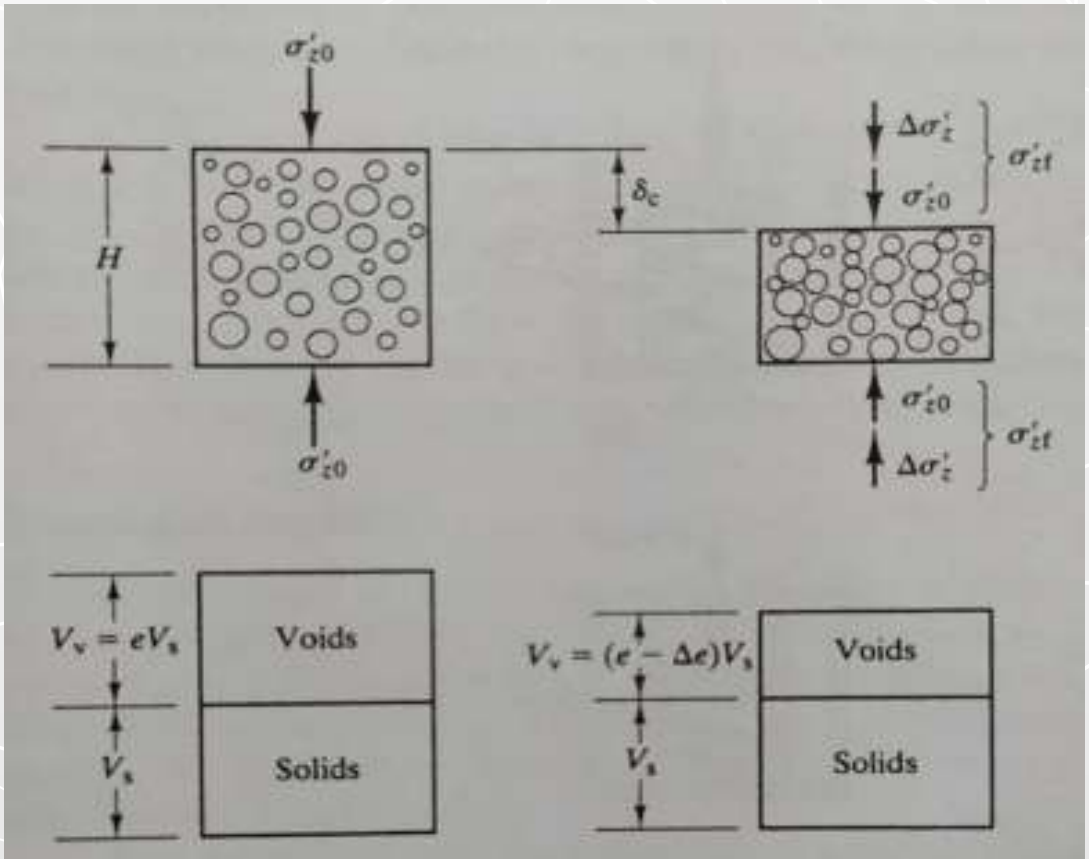


Figure: Consolidation of soil particles under the influence of an increasing vertical effective stress.

$$e = \frac{V_v}{V_s}$$

Consolidation Settlement (cont..)

- Resulting settlement is known as ***consolidation settlement*** δ_c.
- **Most important source of settlement in soils** and its analysis is an important cornerstone of geotechnical engineering.

Question: What is consolidation settlement in soils? Why it is important to geotechnical engineering?

Understanding the Consolidation analyses.

- Consolidation analyses usually **focus on saturated soils (S=100%)**, which means the **voids are completely filled** with water.
- Both the **water and solids are virtually incompressible**.
- So consolidation **can occur only as some of the water is squeezed out of the voids**.
- Process can be **demonstrated by taking a saturated kitchen sponge and squeezing it**; the sponge compresses, the water is pushed out.

Question: Explain the physical process of consolidation and its relationship with the flow of pore water.

Physical process of consolidation and its relationship with the flow of pore water:

Piston and Spring Analogy:

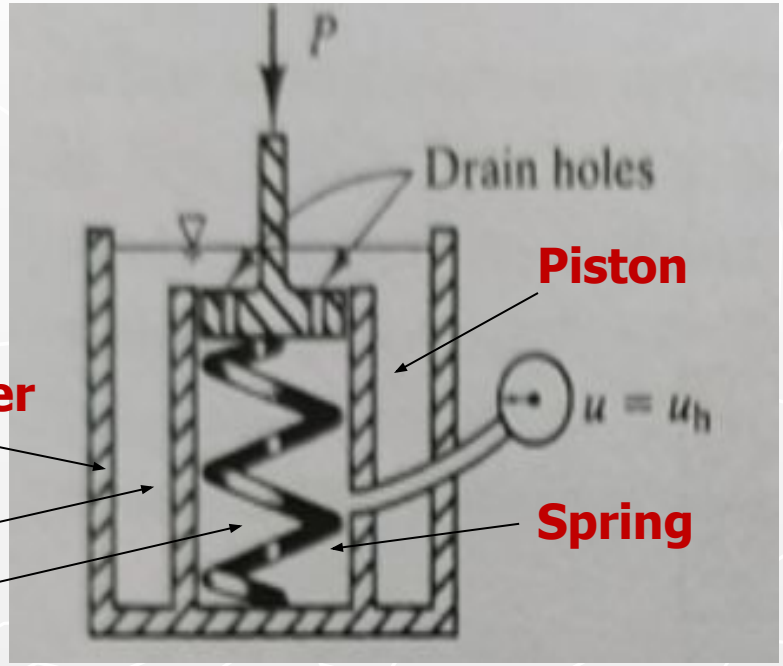
- Consider the mechanical piston and spring analogy.

- **Device consists of a piston and spring located inside a cylinder.**

- **Cylinder** is filled with **water** and a **small drain hole** present in the **piston.**

Piston and Spring Analogy (cont…):

- **Components represent an element of soil** at some depth in the ground.
- **Spring** representing the soil solids.



- **Water** representing the pore water, and
- **Drain holes** representing the soil voids through which the pore water must flow.

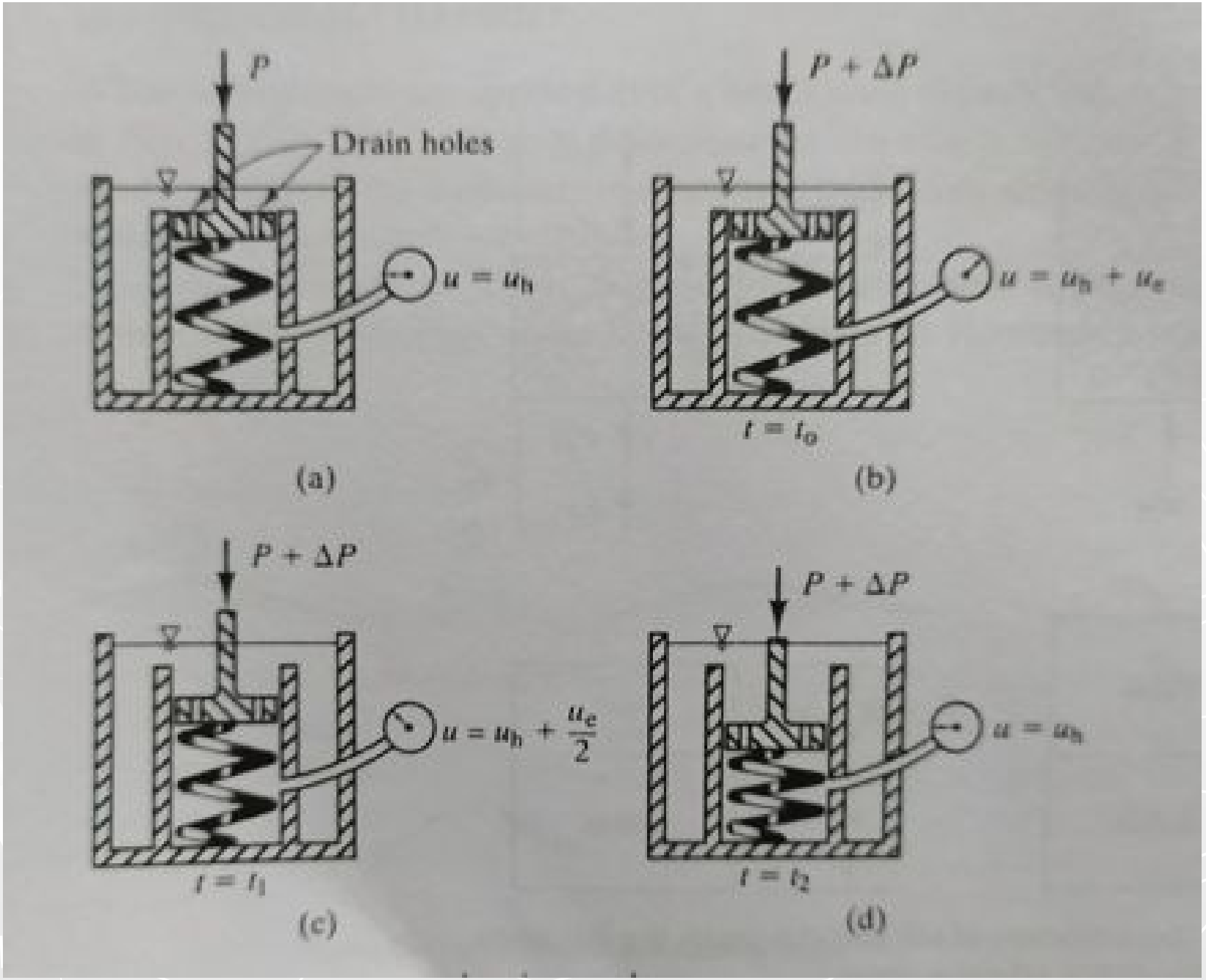


Figure: Piston and Spring Analogy.

Figure (a).

- Piston is in **static equilibrium at top** under a certain vertical load P.

- Assembly is **submerged in a cylinder**, so the water is subjected to a hydrostatic pressure that represents the **hydrostatic pore water pressure** u_h, in the soils.

- **Water pressure** on top and bottom are equal, so that applied **load on piston is carried entirely by the spring**.

- Load divided by the cross-sectional area of the cylinder represents the **initialvertical effective stress**, σ'_{z0}.

$$\sigma'_{z0} = \frac{Load}{}$$

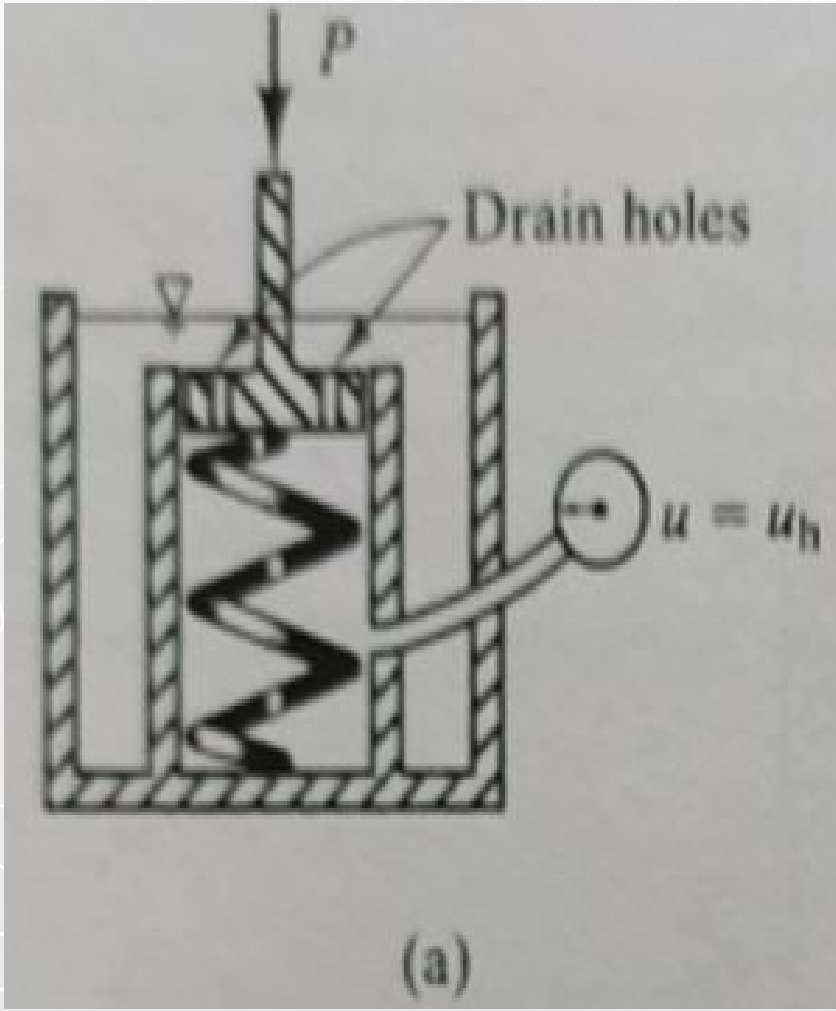
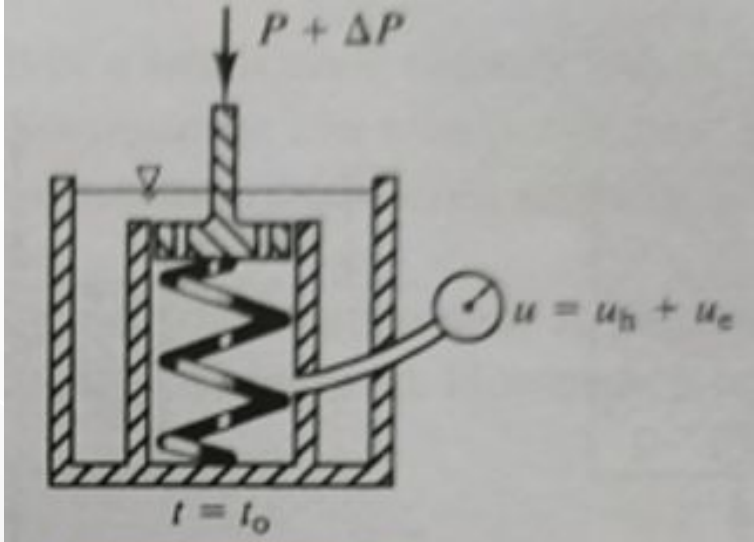


Figure (b):



Figure; (b)

- At time **$t=t_0$** an additional load **ΔP** is applied to the piston, which represent additional total vertical stress $\Delta\sigma_z$ in a soil, such as induced **by a new fill**.

- Causes a very small **downward movement** of the piston, but this movement is **resisted** by **both the spring (soil)** and **water (pore water)**.

- **Water** is much stiffer than spring, it **carries virtually all the additional load** and the **water pressure increases**.

- Additional pressure is known as *excess pore water pressure*, u_e.

- *Water pressure* ***u*** **in side the cylinder** now equals to $u = u_h + u_e$.

- **Water pressure inside the cylinder is now greater than outside**, so some of the **water began to flow** through the holes.

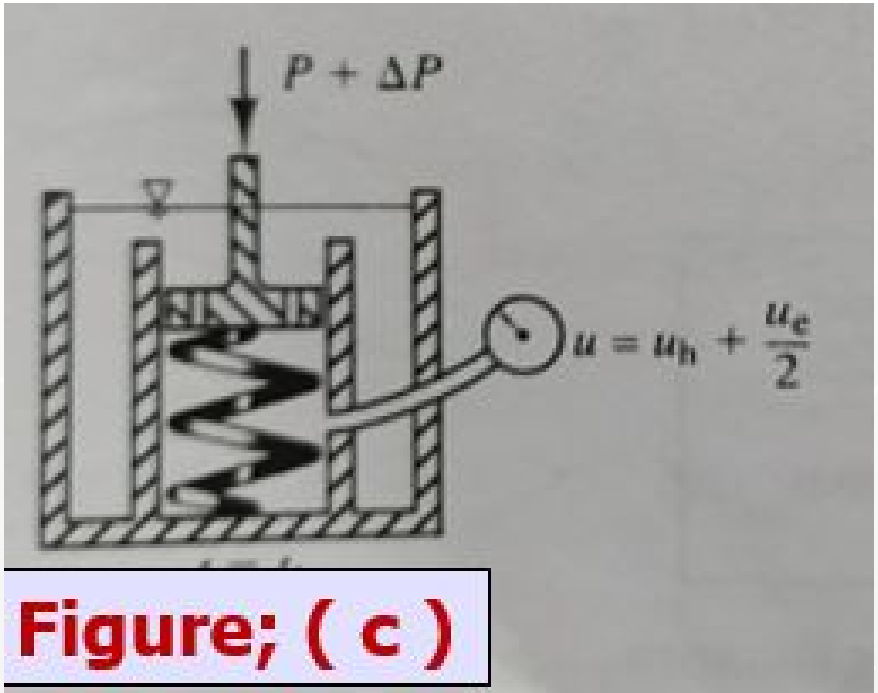
Piston and Spring Analogy: (cont..)

- Holes are very small, so the **flow rate through them is also small**, but eventually **a certain quantity of water passes through.**
- **Outward flow of water allows the piston to move further downward,** thus **compressing the spring** and **relieving some of the load from water**.
- **Process represents** the **gradual transfer of stress from** the pore water to the soil solids**.**

Question: Understanding the relationship between compression of the spring and dissipation of water is key to solving the consolidation problems. explain

Figure (c):

- At time t=t_1, **half of** $\Delta\sigma_z$ has been transferred to the soil solids and **half of them** is still being carried by pore water.



Figure; (c)

Figure (d):

- Process continues until the **spring has compressed sufficiently** to **accommodate the original effective stress** plus **the additional effective stress** at **t=t_2**.

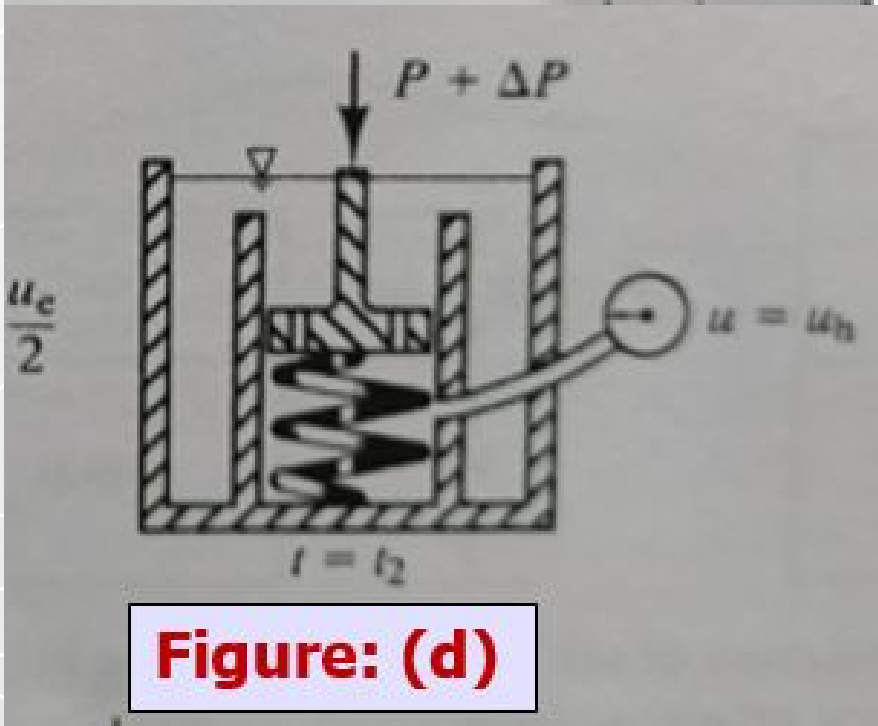


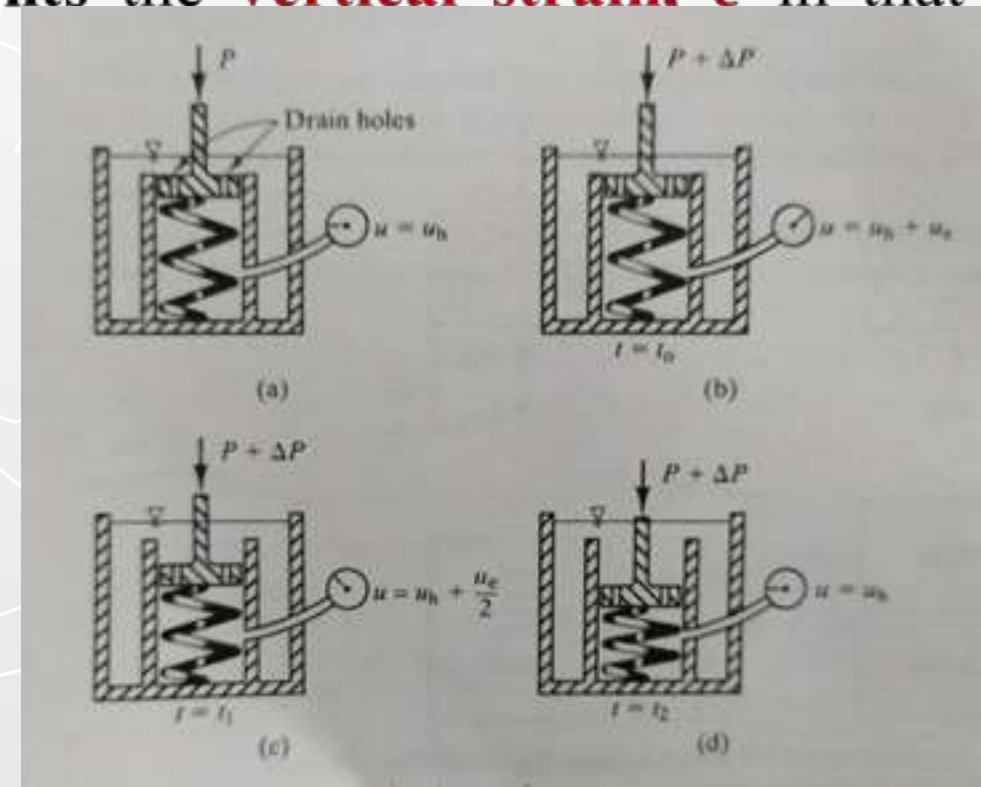
Figure: (d)

Figure (d):

- Excess pore water pressure is now zero (u_e=0), so flow through the holes ceases.

- Components returned to **static equilibrium**, but **the piston is in lower position and** $u=u_h$.

- **Change in position represents** the **vertical strain,** ϵ in that element of soil in the field.



Question: Describe the consolidation processes in the field.

Processes in the field:

- **Initial build up of excess pore water pressures** in soils is **more complex** because it depends on the changes of **normal stress** and **deviator stress**.

- Problems can be **simplified** by **assuming the excess pore water pressure** u_e, immediately after loading is equal to $\Delta\sigma_z$. $(u_e=\Delta\sigma_z)$

- **Increase in pore water pressure produces** a **hydraulic gradient in the soil**, causing some of the **pore water to flow away.**

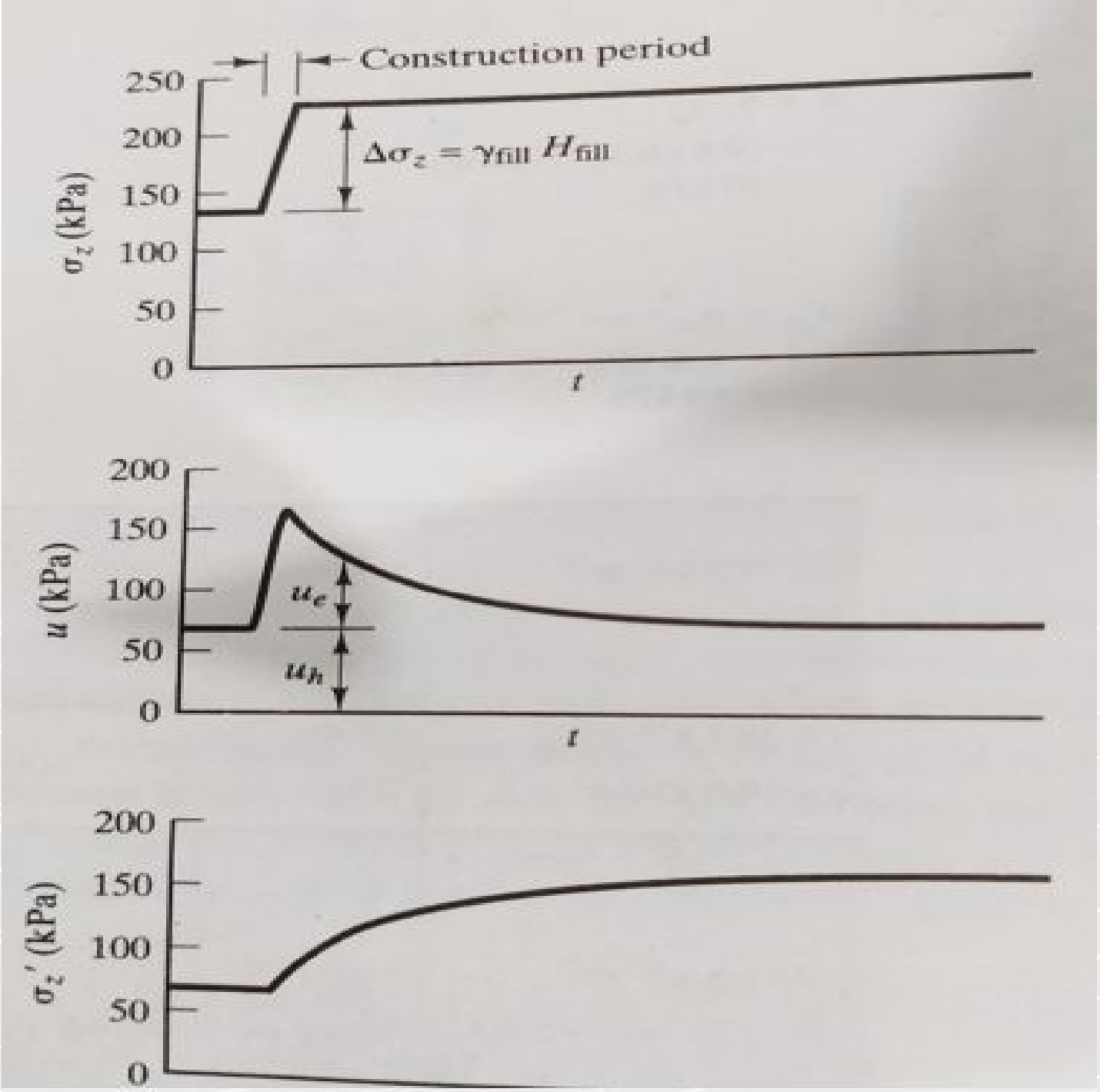
Describe the consolidation processes in the field (cont..).

- As each **increment of stress, water is discharged**, **the solid particles consolidate** and **begin to carry part of the new load.**

- Thus, $\Delta\sigma_z$ **is gradually transferred** from the **pore water to the soil solids**, and the vertical effective stress $\Delta\sigma'_z$ **rises**.

- **Eventually, all of the new load is carried by the solids**, the **pore water pressure returned to its hydrostatic value** u_h, and the **flow of pore water ceases**.

Question: The transfer of load from water to solids is one of the most important processes in geotechnical engineering. Explain.


Construction period
250
200
150
100
50
0
σ_z (kPa)
$\Delta\sigma_z = \gamma_{fill} H_{fill}$
t
200
150
100
50
0
u (kPa)
u_e
u_h
t
200
150
100
50
0
σ_z' (kPa)

Ex. 2: Find out the **vertical effective stress** in soil element at point A, when a 5-m deep fill is placed shown in the following figure; after the consolidation is complete. Explain the consolidation process in the field.

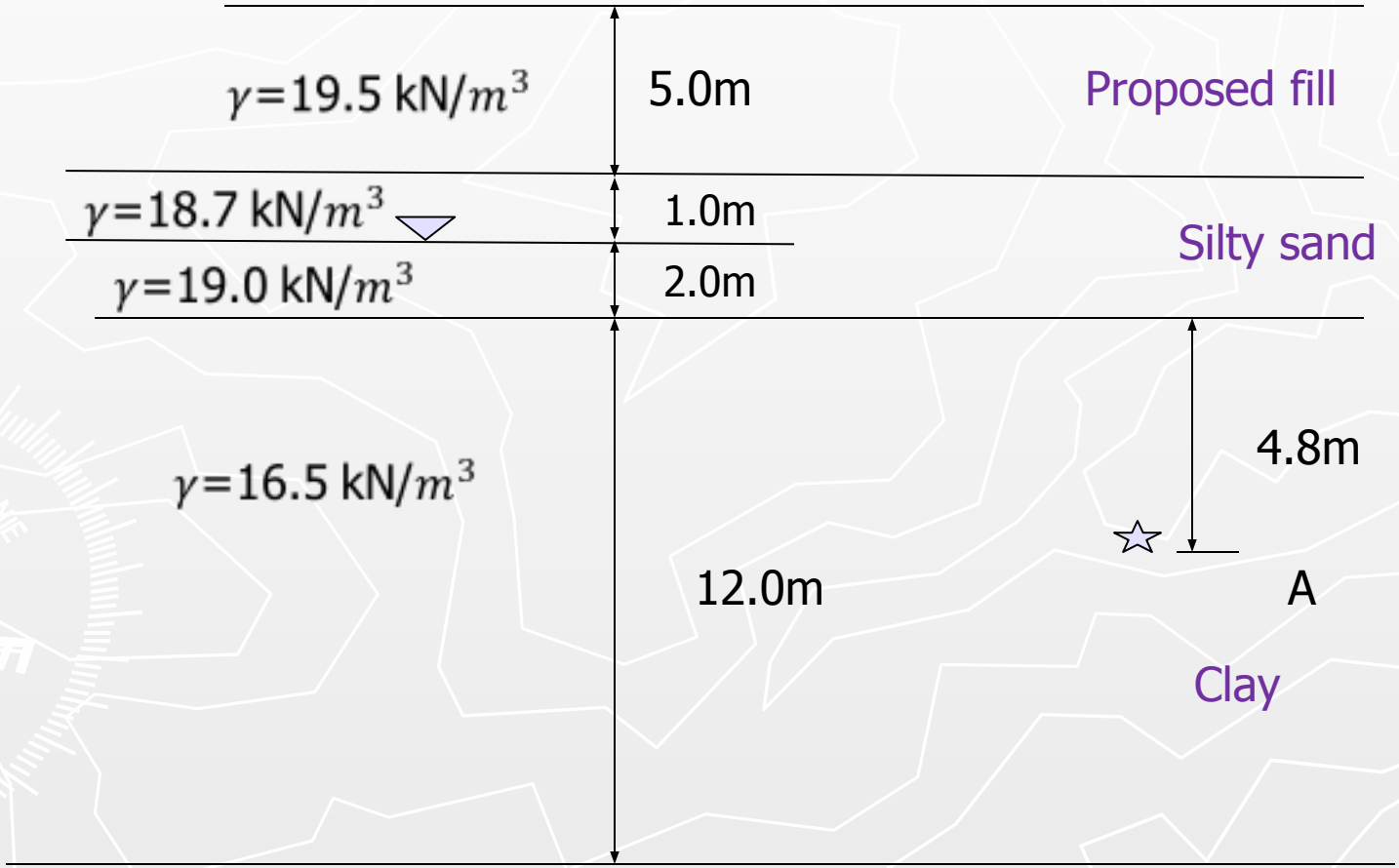


Figure: Soil Profile

Solution:

- Soil element is initially subjected to the following stresses:

$$\sigma'_{z0} = \sigma_{z0} - u$$

$$\sigma_{z0} = \sum \gamma H$$

$$= 18.7 \times 1.0 + 19.0 \times 2.0 + 16.5 \times 4.8$$

$$= 136 \text{ kPa}$$

$$u = \gamma_w z_w$$

$$= 9.8 \times (2.0 + 4.8)$$

$$= \mathbf{67.0 \text{ kPa}}$$

$$\sigma'_{z0} = \sigma_{z0} - u$$

$$= 136 - 67 = \mathbf{69 \text{ kPa}}.$$

Solution (cont..):

- **5-m deep fill has been placed**. These increases the vertical total stress:

$$\sigma_{zf} = \sigma_{z0} + \gamma_{fill} H_{fill}$$

$$= 136\text{kPa} + 19.5\text{x}5.0 = 234\ \text{kPa}$$

- **Initially the applied load** (fill) is carried entirely by the **pore water**. So the pore water pressure becomes:

$$\mathbf{u} = u_h + u_e$$

$$= 67\text{kPa} + 19.5\text{x}5$$

$(\sigma_{zf} = u_e, \textit{excess load carried by pore water})$

$$67 + 47.5 = 165\text{kPa}$$

Solution (cont…)

- But the **vertical effective stress remain unchanged**,

σ'_z $= \sigma_z$- u (ie σ_{zf} -u)

$=234 - 165$

$=$**69 kPa**

- As some of the pore water drains away, soil element consolidates and $\Delta\sigma_z$ **is gradually transferred from the pore water to the solids.**

- After sufficient long time, u_e=0 and consolidation complete.

u $= u_h + u_e = 67\text{kPa} + 0 =$**67kPa**

σ'_z $= \sigma_z$- u

$=234 - 67 =$ **167 kPa (Answer).**

Thank you

EWCE 341

Principles of Soil Mechanics

Consolidation

Ex. 2: Find out the **vertical effective stress** in soil element at point A, when a 5-m deep fill is placed shown in the following figure; after the consolidation is completed in the field.

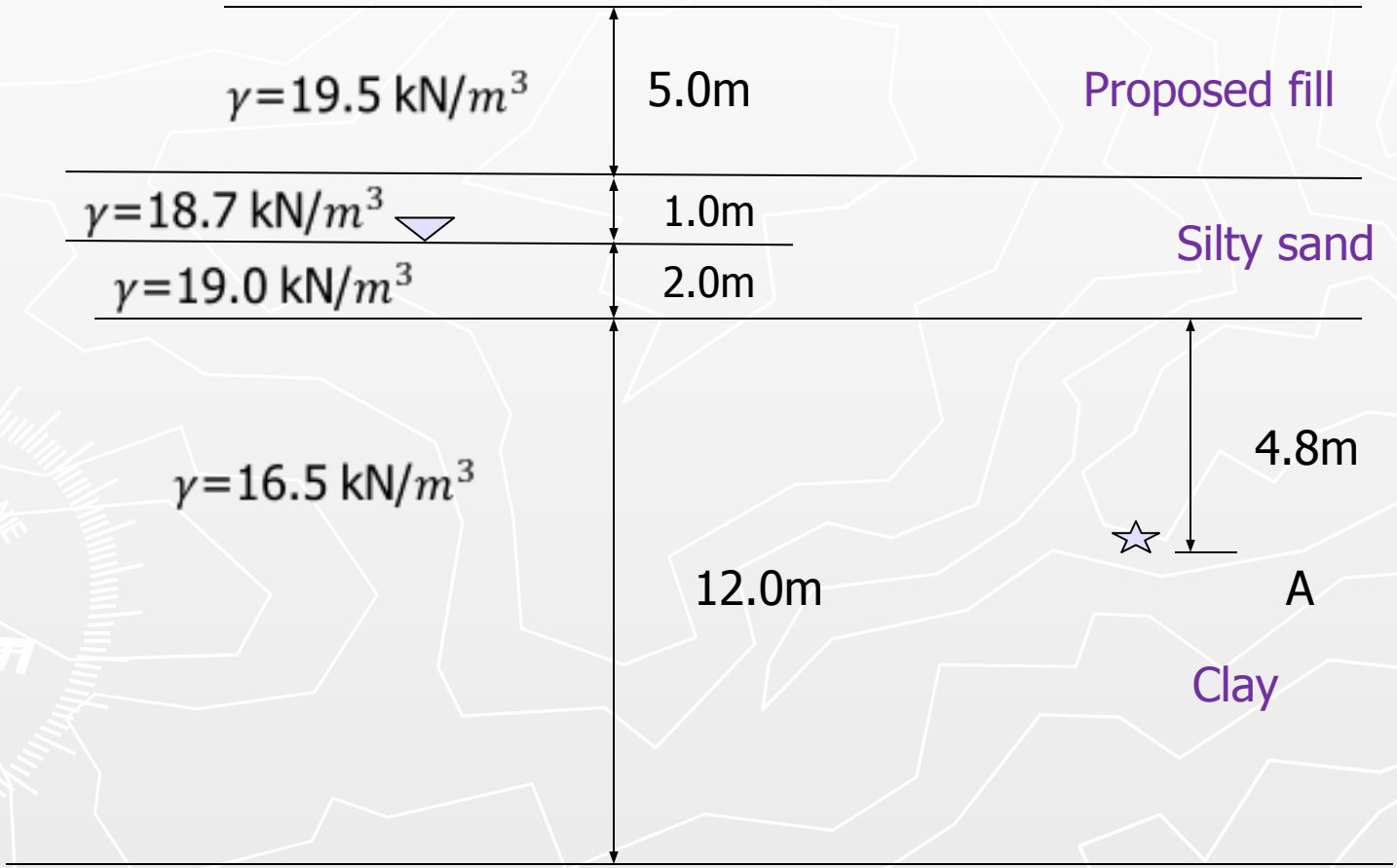


Figure: Soil Profile

Solution:

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$\sigma_{z0} = \sum \gamma H$

$= 18.7 \times 1.0 + 19.0 \times 2.0 + 16.5 \times 4.8$

$= 136$ kPa

$u = \gamma_w z_w$

$= 9.8 \times (2.0 + 4.8)$

$= \mathbf{67.0\ kPa}$

$\boldsymbol{\sigma'_{z0} = \sigma_{z0} - u}$

$= 136 - 67 = \mathbf{69\ kPa}.$

Solution (cont..):

- 5-m deep fill has been placed. These increases the vertical total stress:

$$\sigma_{zf} = \sigma_{z0} + \gamma_{fill} H_{fill}$$

$$= 136\text{kPa} + 19.5 \times 5.0 = 234\ \text{kPa}$$

- **Initially the applied load** (fill) is carried entirely by the pore water. So the pore water pressure becomes:

$$u = u_h + u_e$$

$$= 67\text{kPa} + 19.5 \times 5$$

$$= 165\text{kPa}$$

Solution (cont…)

- But the **vertical effective stress remain unchanged**,

σ'_z = σ_z- u (ie σ_{zf} -u)

=234 – 165

=**69 kPa**

- As some of the pore water drains away, soil element consolidates and $\Delta\sigma_z$ **is gradually transferred from the pore water to the solids.**
- After sufficient long time, u_e =0 and consolidation complete.

u = u_h + u_e = 67kPa + 0 =**67kPa**

σ'_z = σ_z- u

=234 – 67 = **167 kPa (Answer).**

What we are going to learn today?

- **What is Consolidation Test Sample?**
- **Test Procedures**
- **Loading Sequence**
- **Methods of presenting test result**
- **Plastic and elastic deformation**
- **Pre-consolidation Stress**

Consolidation Test Sample

- Tests are usually performed on **a high quality *undisturbed* samples (natural)**, of soft to medium clays and tests results are reliable.

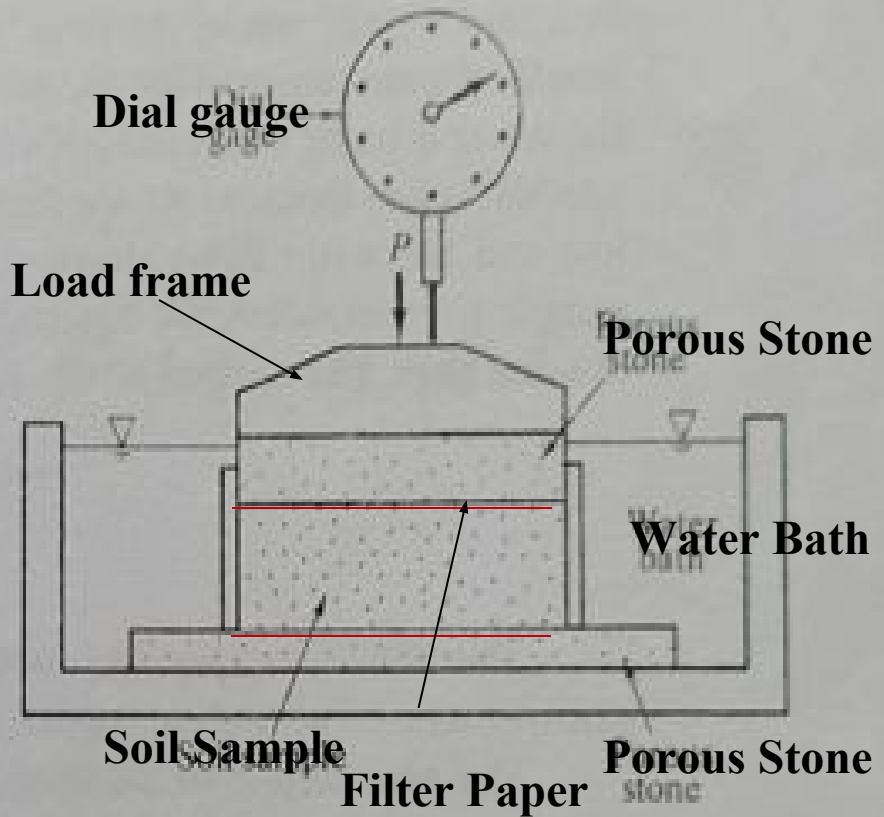
- It is virtually **impossible to obtain** high quality undisturbed samples and thereby **uses empirical correlations /in-situ tests** to asses the stress-strain properties.

- It is important for samples that were **saturated in the field** to remain so during storage and testing.

- Sometimes engineers need to evaluate the consolidation characteristics of **proposed compacted fills**, and do so by **performing consolidation tests** on **samples that has been remolded and compacted in the laboratory.**

Consolidation Test in the laboratory





Components

Specimen Ring:

- **Soil specimen**, which has the shape of an upright cylinder, is **placed in the consolidometer, and surrounded** by a brass or stainless **ring**.

Purpose of Rings:

-To maintain zero horizontal strain, producing one-dimensional consolidation.





Test Procedure

Porous Stones:

- **Porous stones** are placed above and below the specimen, which is **strong enough** to carry the applied loads and **porous enough** to allow water to pass through freely.

Filter Papers:

- Placed on top and bottom of soil specimen of specimen ring.

Water Bath:

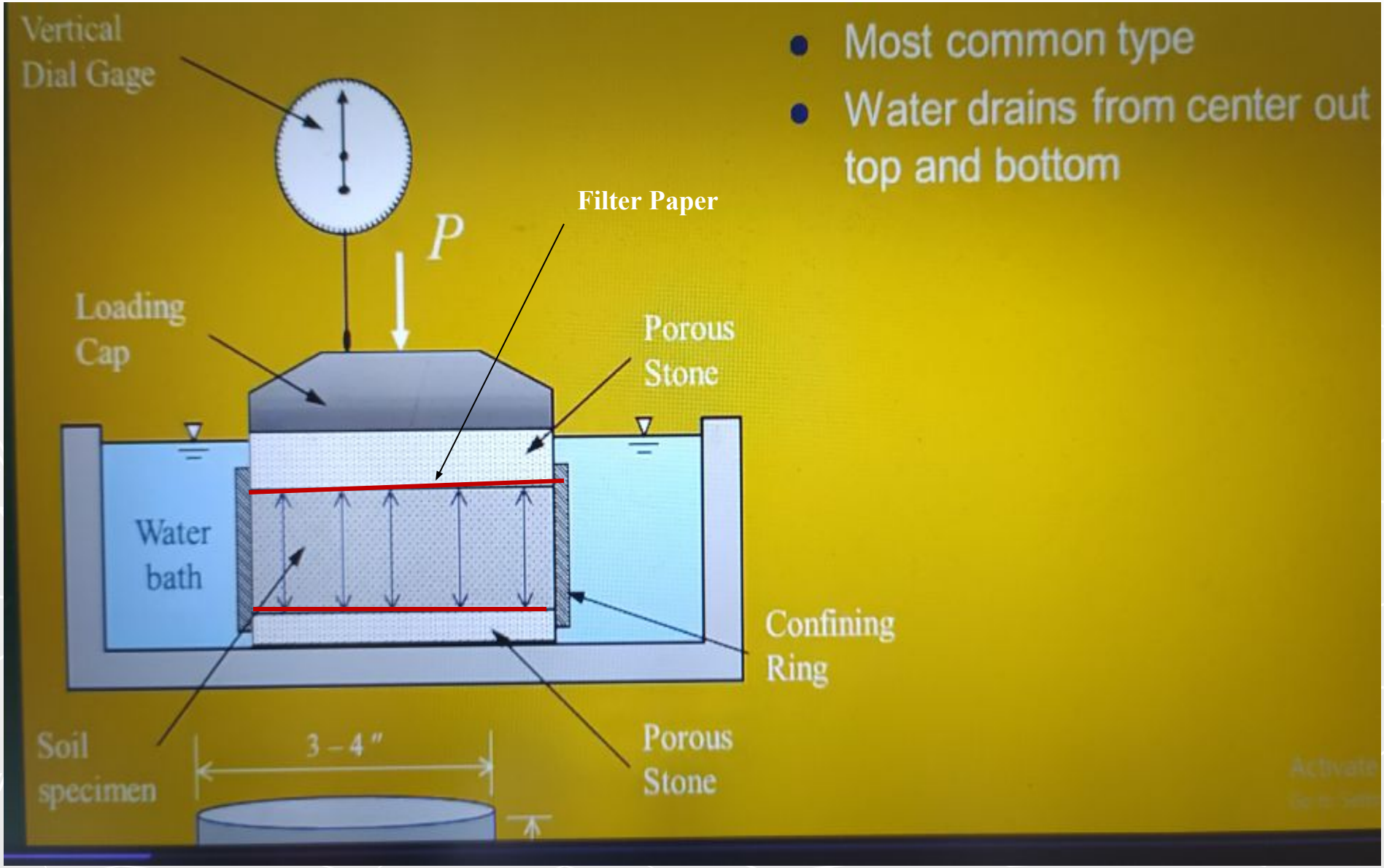
- Specimen, rings and porous stones are **submerged in a water bath**, to keep the soil saturated.

Device for Measuring Deformation:

- **A Dial gauge or device** is placed above the specimen to measure its compression as the test progresses.
- **Loading Cap:**
- **Loading Device:**



1-D Consolidation





Test procedure (cont…)

Loading Sequence

Step 1.

- **Test begins** by applying a vertical load, P, which produces a vertical effective stress, σ'_z.

$$\sigma'_z = \frac{P}{A} - u$$

Where, σ'_z = vertical effective stress

P= applied load

A= x-sectional area of whole specimen

u = pore water pressure inside the specimen.

- Water bath barely cover the specimen, **so u is very small** compares to σ'_z and thus may be ignored.

Test procedure (cont…)

- **Specimen is allowed to consolidate,** may require several hours or more.
- Completion of consolidation is determined by **monitoring the dial gauge. Vertical strain** $\in_z$, upon completion of consolidation is:

$$\in_z = \frac{change\ in\ dial\ gauge\ reading}{initial\ height\ of\ sample}$$

$$\varepsilon = \frac{\Delta L}{L_0}$$

where

- ε strain
- ΔL total elongation [m]
- L_0 original length [m]

Now, we have **1st data point** $(\sigma'_z, \in_z)$.

Step 2:

- **Increase the normal load** to some higher value and allow the soil to consolidate again, thus obtaining **2nd data point** (σ'_z, $\in_z$).
- **Process continues** until desired peak vertical stress is reached.
- From this loading sequence, **curve ABC** is obtained.

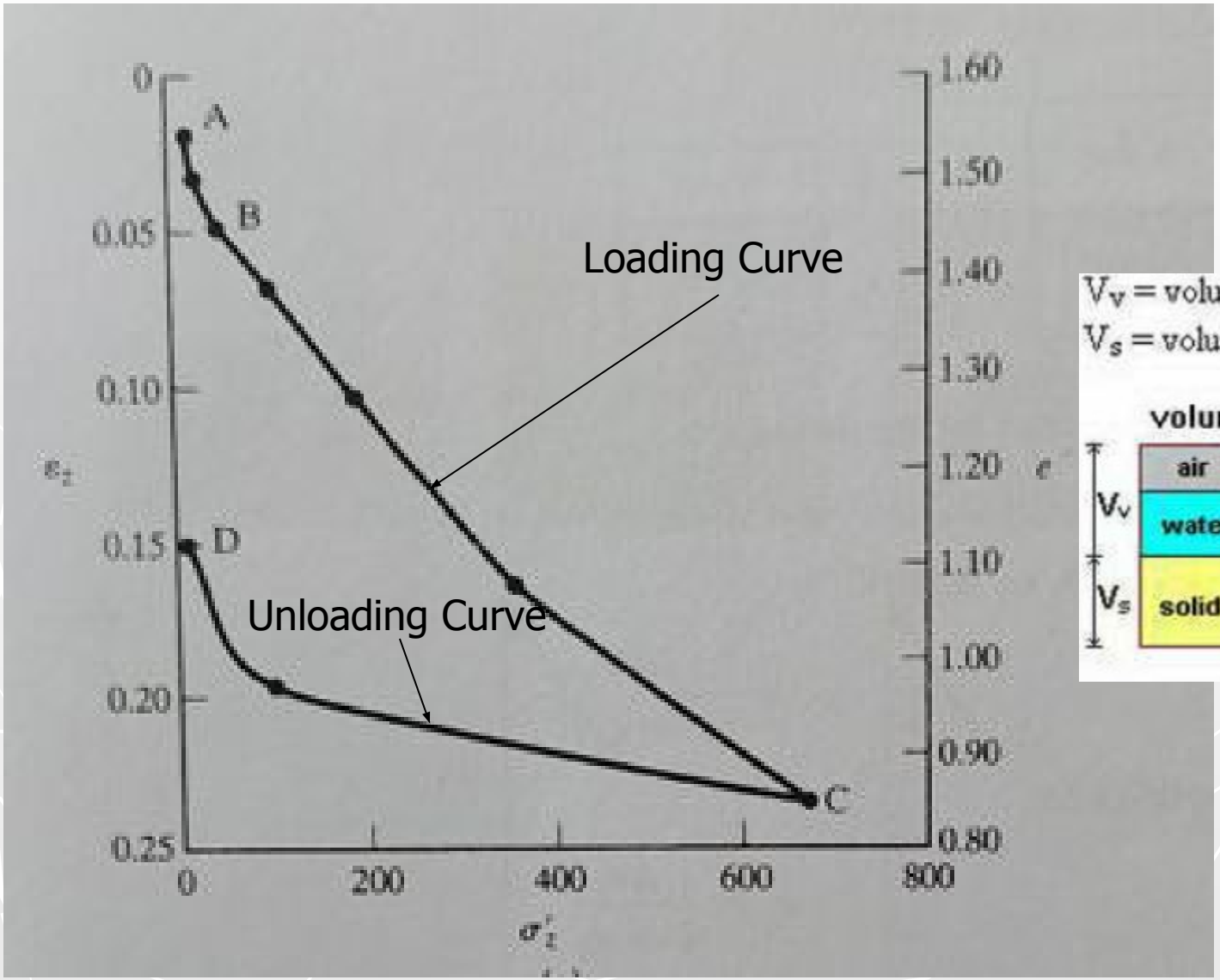
Unloading sequence:

- Now, **incrementally unload** the sample and allow it to rebound, thus producing the **unloading curve CD** (Figure).

- **Stress-strain curve for soil is decidedly non-linear** when plotted in arithmetic space.

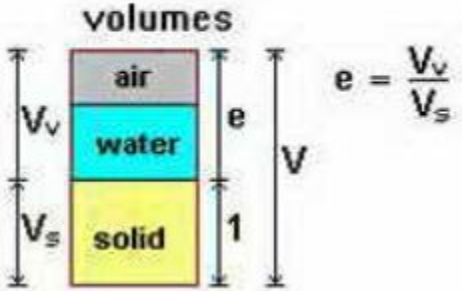
- But, when **presented on a semi-logarithmic plot** (Figure), the same data are much easier to interpret.

Figure: Result of laboratory consolidation test- data plotted on arithmetic scale. The initial void ratio, e_0 is 1.60.



V_v = volume of voids

V_s = volume of solid grains.



- In **semi logarithmic plot space**, the **stress-strain curve has three distinct segments** and each is nearly linear.

- Segment **AB** is known as ***reloading curve*** or ***recompression curve,*** ***BC*** is the ***virgin curve,*** and **CD** is the ***unloading curve*** or ***rebound curve***.

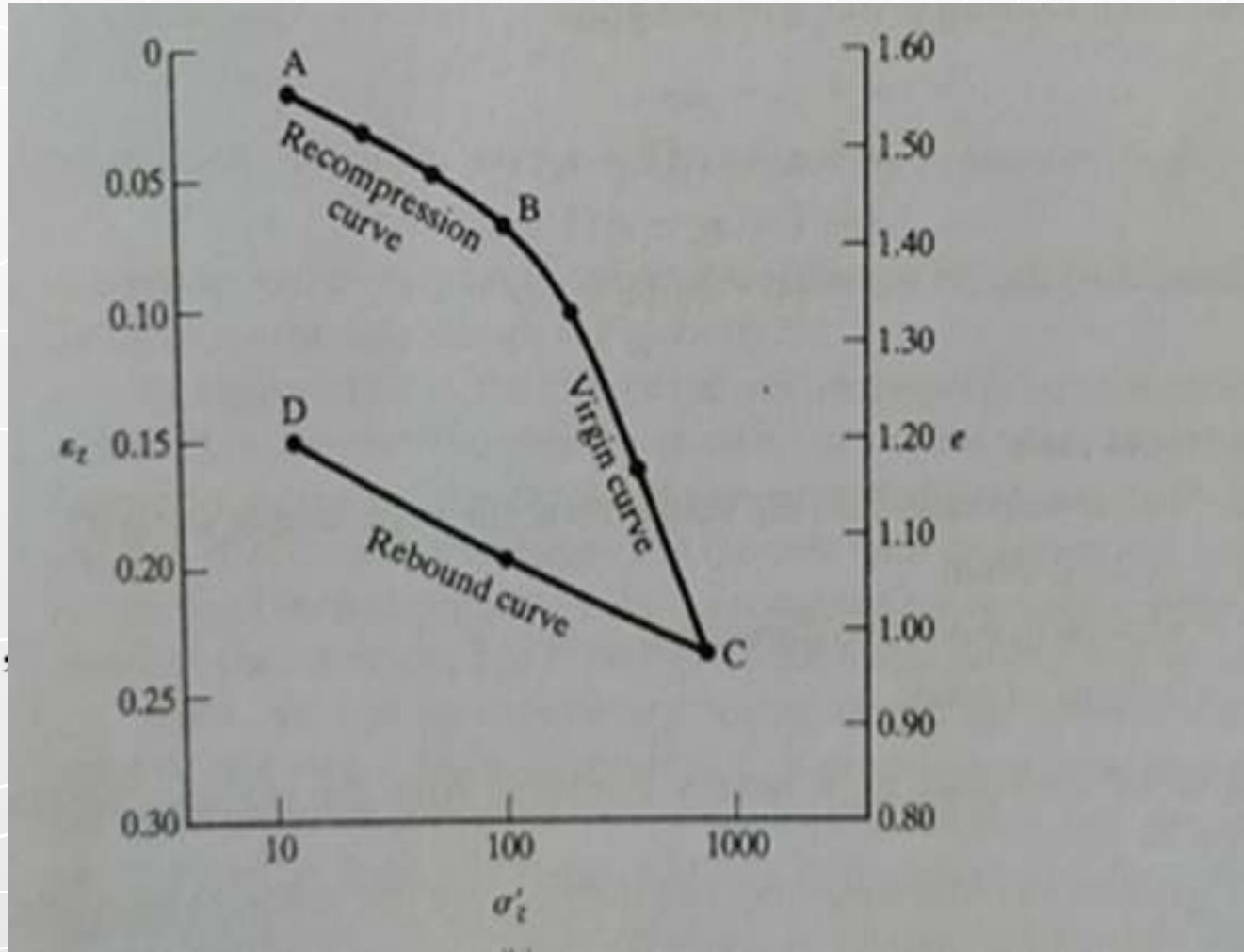


Figure: Result of laboratory consolidation test-data plotted on semi-logarithmic scale. Initial void ratio, ***e_0*** **is 1.60**.

Methods of presenting consolidation test results:

Two different methods of presenting consolidation test results, could be presented in either form or both..

A strain plot, 1st method:

- **Plot of $\in_z$ versus log σ'_z**, and thus a **direct representation of the data** obtained in the lab.
- **Most straight forward approach**, because the purpose of a consolidation test is to measure the stress-strain properties of soils.

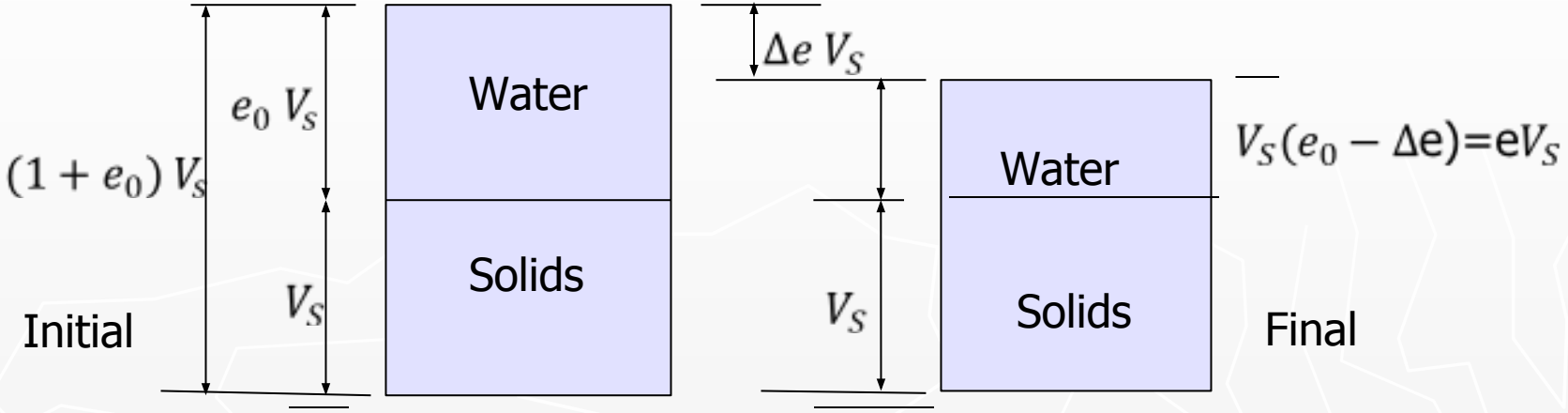
A void ratio plot, 2nd method:

- **Plot of void ratio *e* versus log σ'_z**, and **not a direct representation of the data obtained in the lab.**
- Method is **used by Terzaghi**, presumably **because**, it emphasizes the **reduction in void size** that occurs during consolidation.

To compute the void ratio at various stages of the test, **develop an**

Derivation of an equation that relates void ratio with strain

$$e = \frac{V_v}{V_s}$$



$$eV_S = e_0\, V_s - \Delta eV_S \quad \text{---------- (1)}$$

$$\epsilon_z = \frac{\Delta eV_S}{(1+e_0)\, V_s}$$

$$= \frac{\Delta e}{(1+e_0)} \quad \text{---------- (2)}$$

$$\varepsilon = \frac{\Delta L}{L_0}$$

where

- ε strain
- ΔL total elongation [m]
- L_0 original length [m]

Combining the equation (1) and (2),

$$e = e_0 - \Delta e$$

$$= e_0 - \epsilon_z\,(1+ e_0)$$

$$= 1+ e_0 - \epsilon_z\,(1+ e_0) - 1$$

$$= (1 - \epsilon_z)\,(1 + e_0) - 1 \quad \text{---------- (3)}$$

Where,

e = void ratio

e_0= initial void ratio (ie. void ratio at beginning of the test).

ϵ_z = vertical strain

Δe = change in void ratio during test= e_0 -e

V_S = volume of solids

- Initial void ratio, e_0, is usually computed from the moisture content using,

$$S = \frac{wG_S}{e} \times 100\%$$

Plastic and Elastic Deformation:



Question: Define plastic and elastic deformation.

Plastic deformation:

- All materials deform when subjected to an applied load.
- If all this **deformation** **is retained when the load is released**, it is said to have experienced *plastic deformation.*
- Example: **bending of an copper wire**.

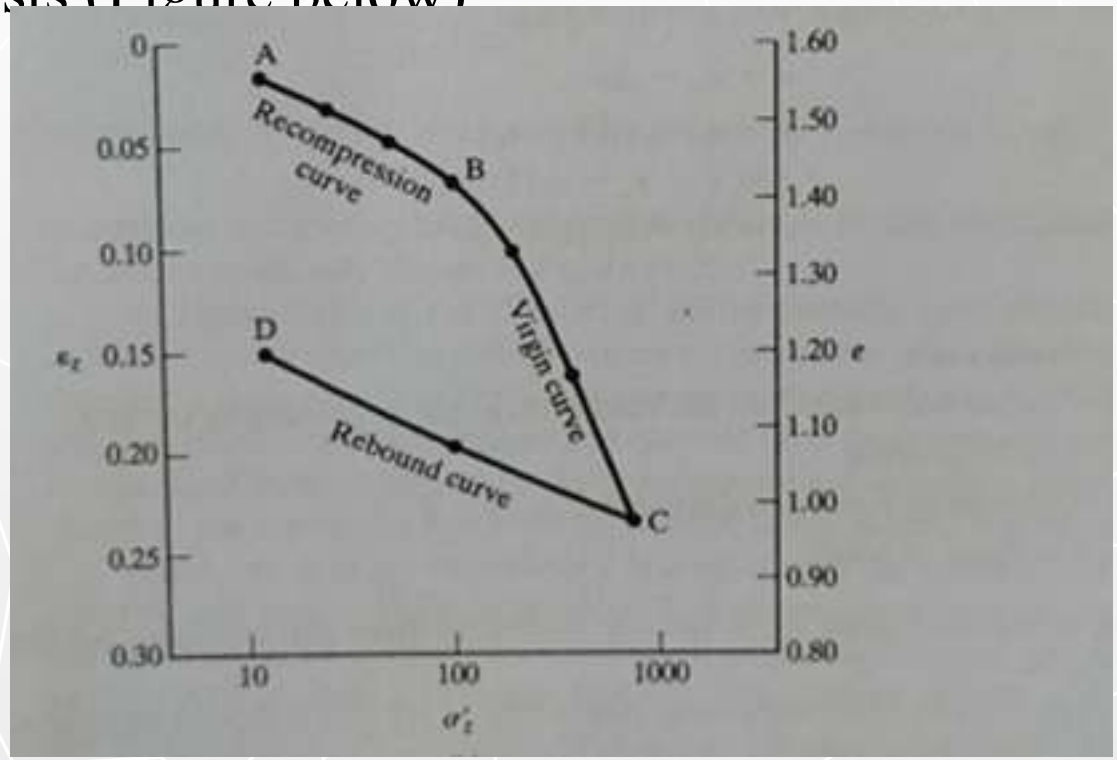


Elastic deformation:

- All materials deform when subjected to an applied load.
- If the **material** **return to its original size and shape when the load is released,** it is said to have experienced *Elastic deformation.*
- Example: bending of a rubber hose.

Question: Describe the plastic and elastic deformation of soils in the results of laboratory consolidation test.

- **Soil exhibits both plastic and elastic deformations**, this is why we see two slopes in the loading and unloading curves in the results of laboratory consolidation tests (Figure below)



Plastic and Elastic Deformation of soils under Applied Load:

Question: Explain the plastic and elastic deformations of soil under applied load.

To understand the plastic and elastic deformations of soil under applied load: Let us consider **soil element A** shown in the figure and **consolidation data** for this element.

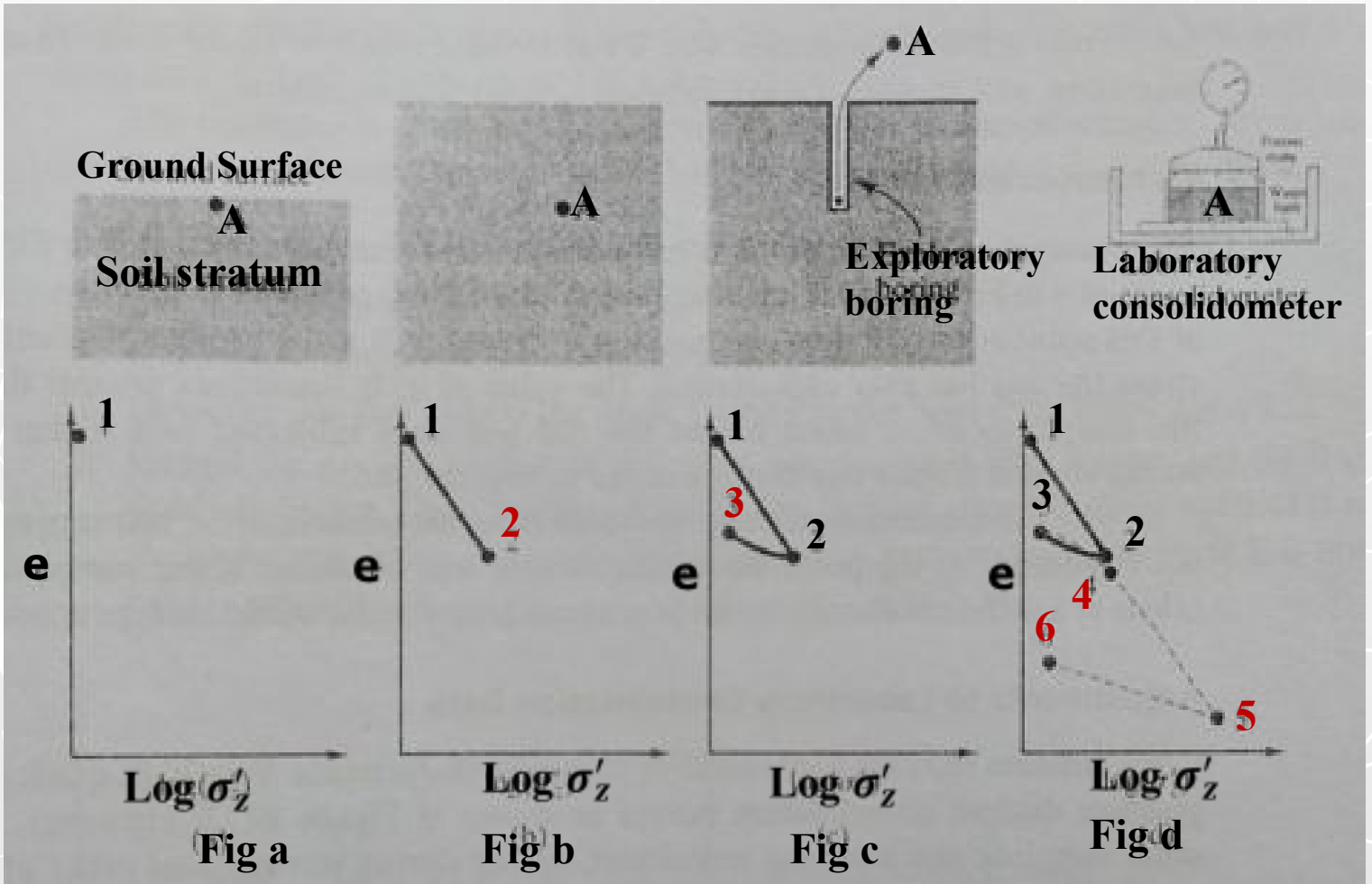


Figure (a):

- **Element of soil A**, has **recently been deposited** and is described by point 1 on the plot.
- Effective stress σ'_z, **is low** and the void ratio **e, is high**.

Figure (b):

- **Additional deposition occurs** and the element A, becomes progressively buried by the newly deposited soils.
- Effective stress σ'_z, **increases** and the void ratio **e, decreases (ie consolidation occurs)**.
- At this stage, **both plastic and elastic deformations are occurring.**
- **Point 2, describes the conditions** that **existed just prior to our drilling and sampling effort.**

Figure (c):

- We **drill an exploratory boring**, obtain an undisturbed sample from element A, and take into the laboratory.

- This **process removes the overburden stress**, so he **effective stress drops** and the **samples expands** slightly (ie. the **void ratio e increases**) as described by **Point 3**.

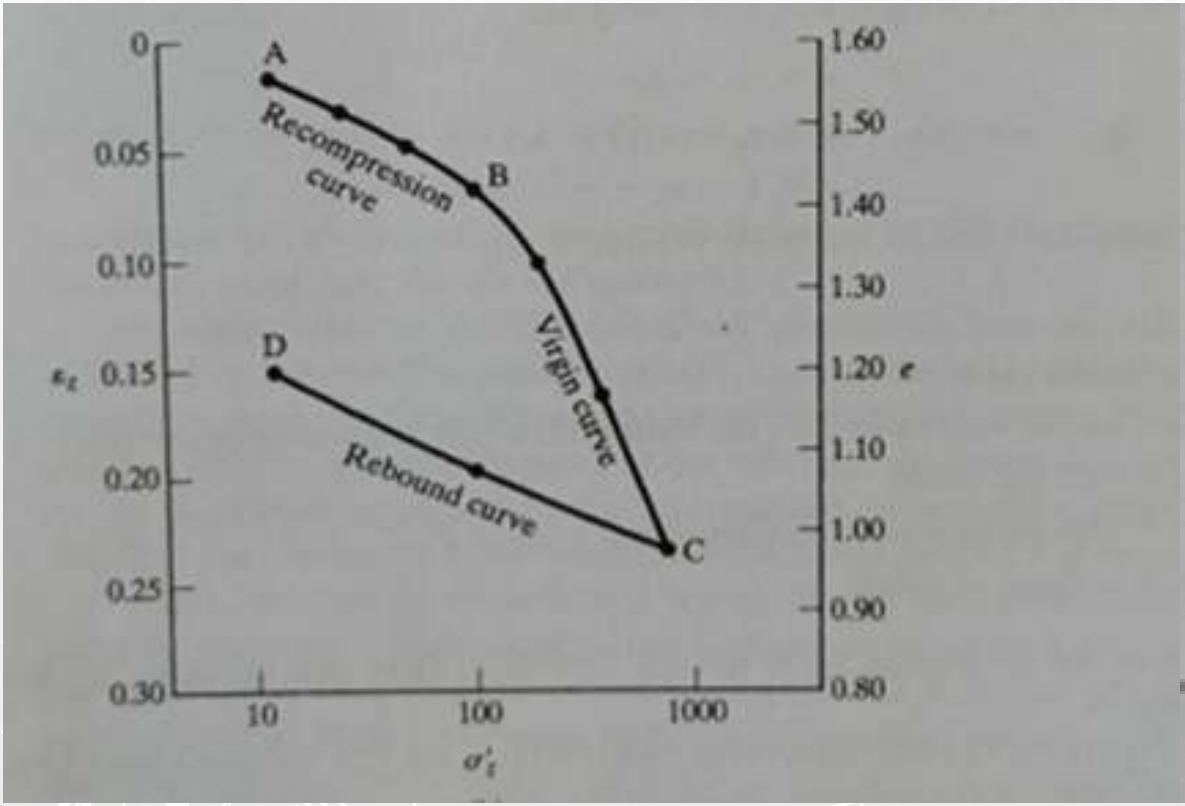
- **This expansion** reflects the **elastic portion** of the compression that occurred naturally in the field.

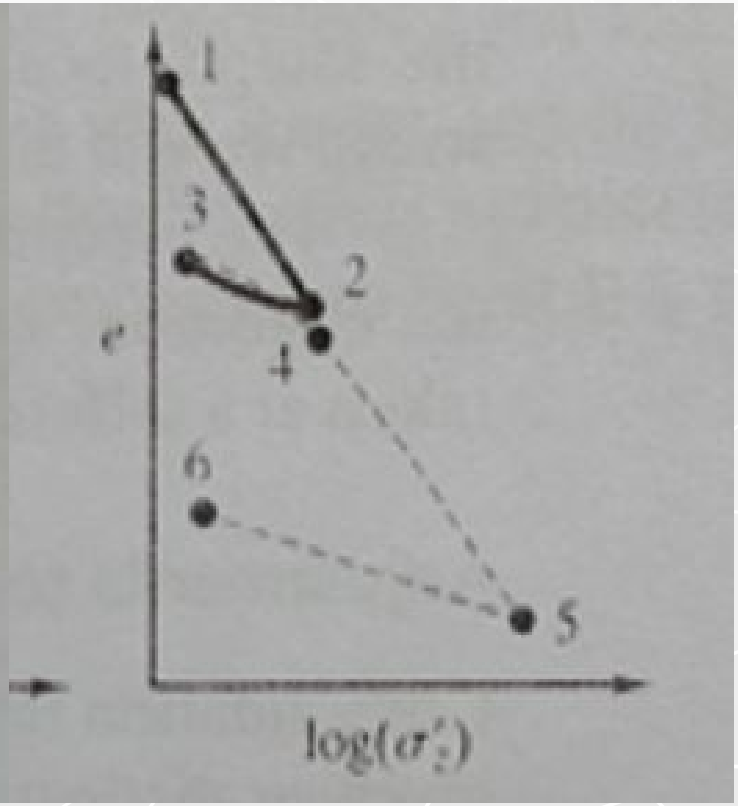
- Although, there has been **some elastic rebound**, **most of the compression was plastic**, so the unloading **curve 2-3 is much flatter** than the loading **curve 1-2.**

- Sample is reloaded in the lab, at point 3.
- **Curve 3-4** is nearly parallel to **curve 1-2** and **reflects elastic compression only**.
- Effective stress σ'_z, **is less than the maximum past effective stress,** so no new plastic deformation occurs.
- **When the curve reaches Point 4**, its slope suddenly changes, and curve **4-5 reflects new** plastic deformations, which occur only, when the σ'_z, **is higher than ever before**. **Curve 4-5** is the virgin curve.
- Finally, when unload the specimen in the lab and form **Curve 5-6,** is the unloading or rebound curve.
- **Curve 5-6** is nearly parallel to **curves 2-3 and 2-3** and reflects the elastic component only.

Pre-consolidation Stress:

- Point where the **slope of the consolidation curve changes** (i.e. point B or Point 4) is an important event in the consolidation process.





Pre-consolidation Stress (cont..)

- Stress at this point is called the ***pre-consolidation stress*** σ'_c.
- It is the **greatest vertical effective stress** the soil has ever experienced.
- Value of $\sigma'_c > \sigma'_{z0}$ at the sample location, which mean the soil was once subjected to a higher effective stress.
- **Pre-consolidation stress** obtained from the consolidation test **represents only the conditions of soil** at that point.
- If the sample has been taken **at different elevation**, the pre-consolidation stress would change accordingly.

What we have learned today?

- **What is Consolidation Test?**
- **Test Procedures**
- **Loading Sequence**
- **Methods of presenting test result**
- **Plastic and elastic deformation**
- **Pre-consolidation Stress**

Thank you

EWCE 341

Principles of Soil Mechanics

Consolidation

EWCE 341

What we are going to learn today?

- Sample disturbance in consolidation test.
- Adjustments of laboratory consolidation data.
- Compression/recompression index and ratio.
- Consolidation status in the field.
- Problems

Sample Disturbance in Consolidation Test:

Question: Consolidation tests are very sensitive to sample disturbance. Explain

Question: Describe the effects of soil sampling in consolidation test results.

- Consolidation tests are **very sensitive** to sample disturbance.
- **Very high quality samples produce distinct consolidation curves,** shown in the figure:

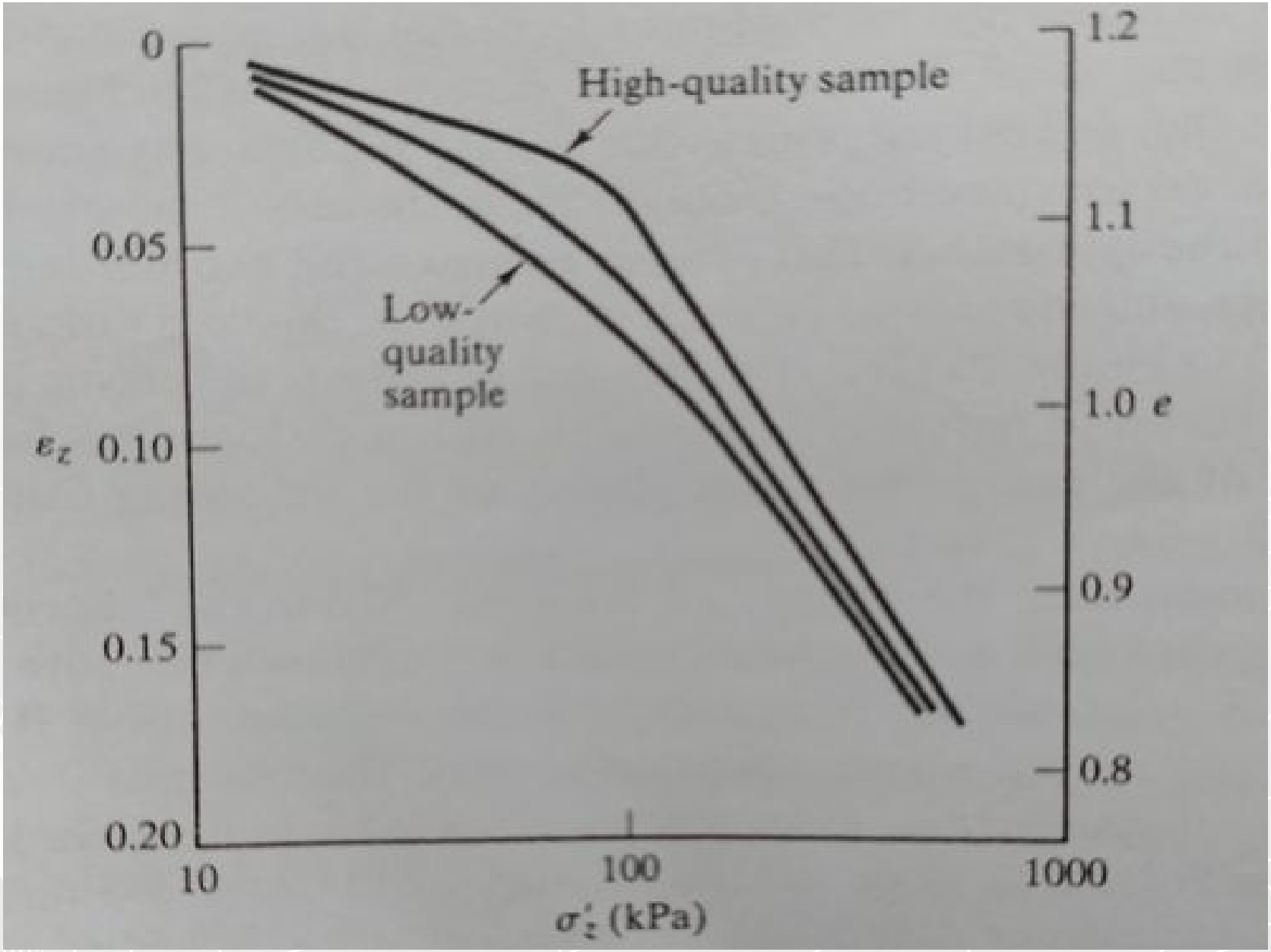


Figure: Effect of sample disturbance on consolidation test results.

Effects of Soil Sampling in Consolidation Test.

- **(a). Less than the ideal sampling**, (b). **handling techniques**, (c). **drying during storage** and (e). **other effects can alter the sample** and **make the test results** more obscure and difficult to interpret.

- (f). Makes **difficult to explain obtained** σ_c' from the poor quality samples because the **curve near the transition** between the **recompression** and **virgin curves becomes much more rounded.**

- Should be **very careful** with samples intended for consolidation tests.

Adjustment of Laboratory Consolidation Data:

Casagrande Procedure (to determine the σ'_c):

Locate the **point of minimum radius** on the consolidation curve (Point A).

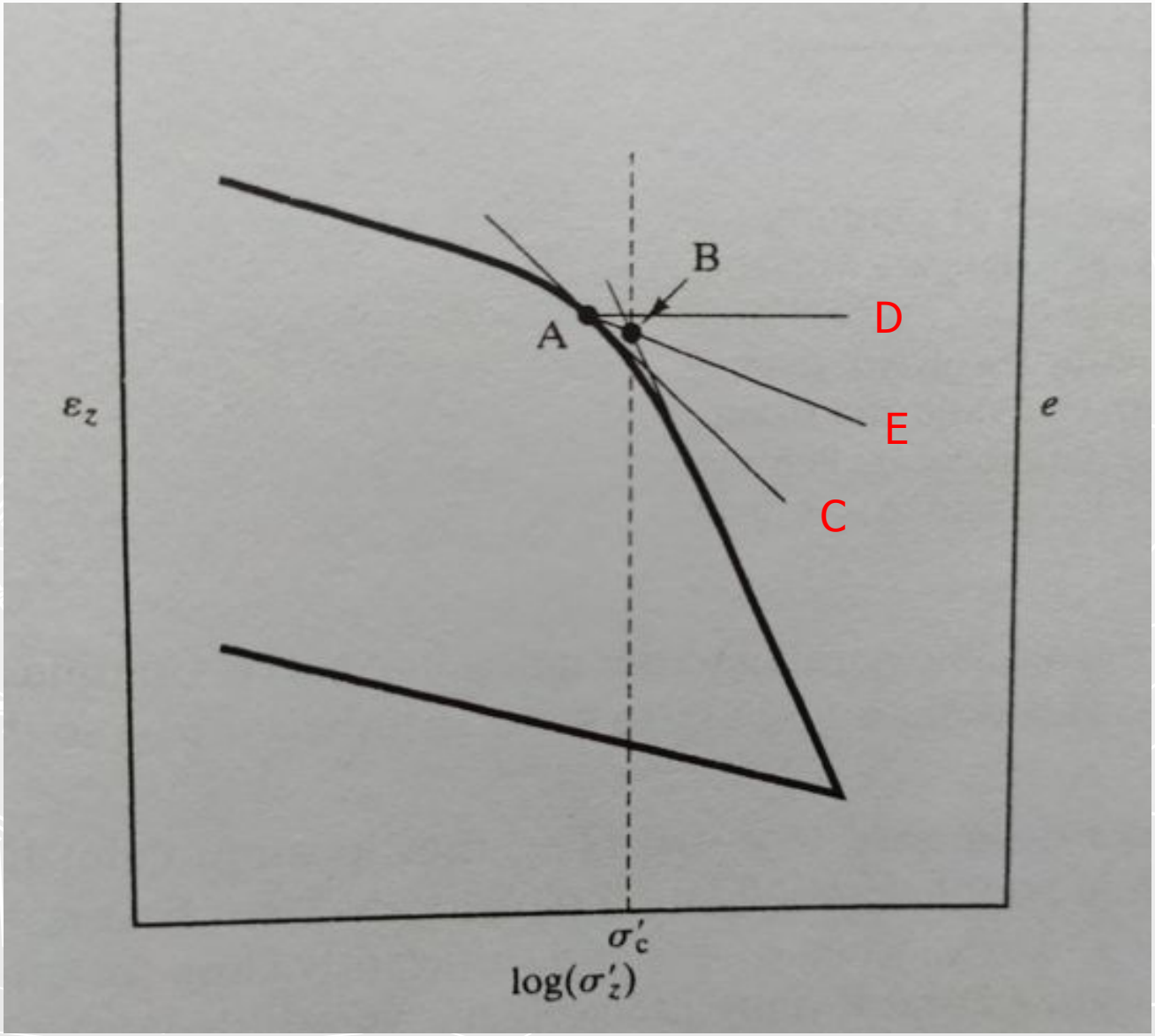
- Draw a **horizontal line** through Point A.
- Draw a **line tangent** to the laboratory curve at Point A.
- Draw the **line that bisects the angle** formed by the lines from upper two steps.

Adjustment of Laboratory Consolidation Data (cont…):

- **Extends the straight portion of the virgin curve** upward until it intersects the line drawn in last steps.
- This **identifies Point B.**
- **Vertical effective stress at Point B** is the **pre-consolidation stress σ_c'.**
- e or ε_z corresponding to point B is not used in any analysis.

Question: Describe the Casagrande's procedure to determine the pre-consolidation stress from laboratory data.

Figure: Casagrande's method of finding the pre-consolidation stress.



Adjustment of Laboratory Consolidation Data:

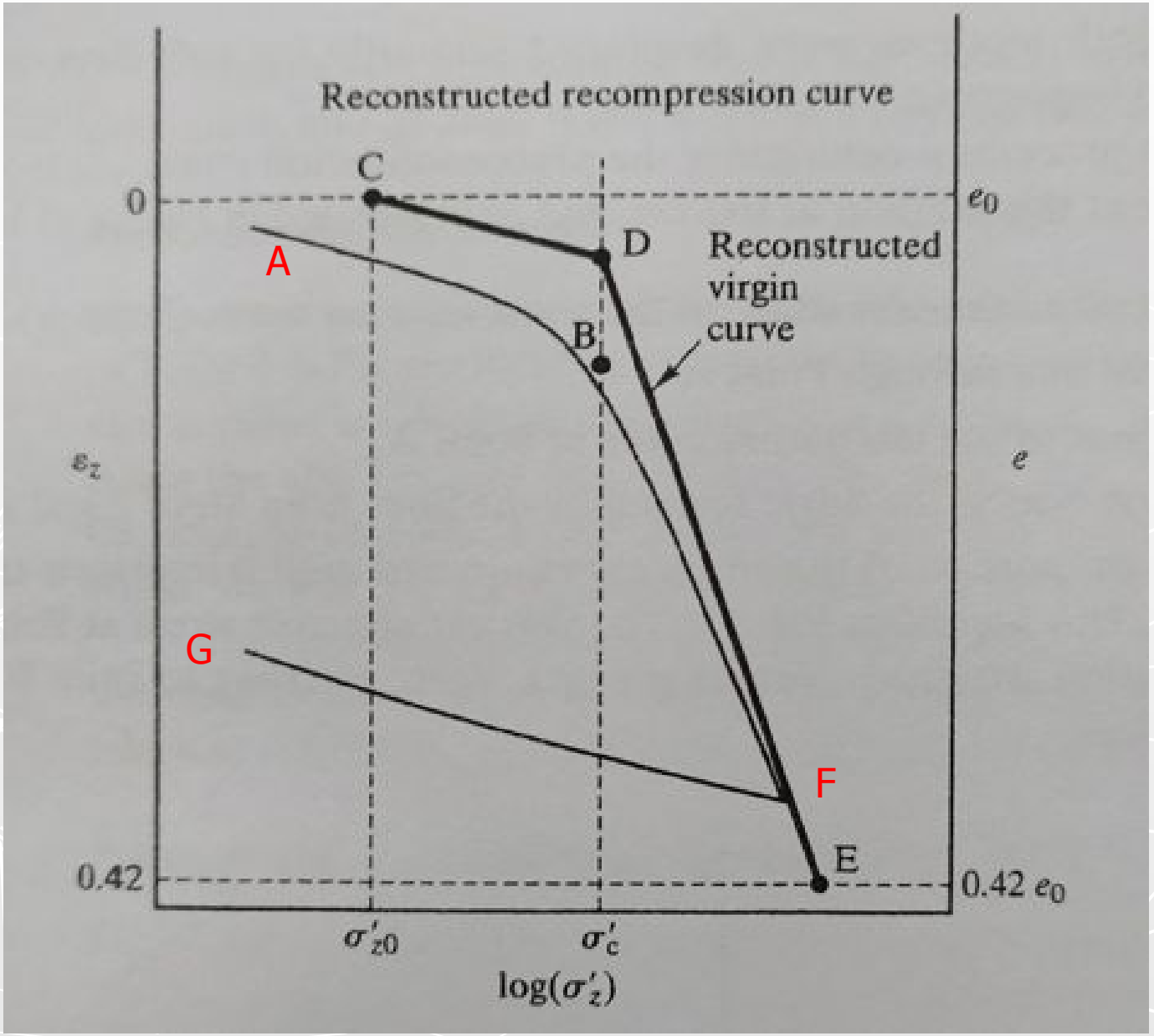
Schmertmann Procedure: (Reconstruction of Field Consolidation Curve)

Sample disturbance affects the slope of the curves, so this procedure is an attempt to **reconstruct the field consolidation curve.**

- **Determine σ'_c**, using Casagrande procedure.
- **Compute the initial vertical effective σ'_{zo}**, at the sample depth. This is the vertical effective stress prior to placement of the proposed load.
- Draw a horizontal line at **e = e_0** or **ε_z=0**, from the vertical axis to **σ'_{zo}**. This locate point C.

Question: Describe the Scmertmann's procedure to reconstruct the field consolidation curve.

Figure: Schmertmann's method of adjusting consolidation test results.



Adjustment of Laboratory Consolidation Data (cont…):

- Beginning at Point C, draw line **parallel to un-loading curve.** Continue to the right until reaching σ'_c line. This defines **Point D**. In some cases, $\sigma'_c \approx \sigma'_{zo}$; this steps is unnecessary.

- Extend the virgin curve downward to **e = 0.42e_0,** when e_0=2, thus locating the Point E or ε_z=0.42.

- **Draw a line connecting points D and E.** This is the **reconstruction of virgin curve.**

Soil Compressibility:

Question: Describe the compressibility of soil in terms of consolidation data plot.

- **Slopes on the consolidation plot reflect the compressibility of the soils.**

- **Highly Compressible**: **Steep slopes mean**, a given increase σ'_z will cause a large strain ε_z (for a large change in **e),** so such soils are said to be *highly compressible*.

- **Slightly Compressible**: **Shallow slopes** indicate that the **same increase** in σ'_z will produce less strain, so the soil is *slightly compressible.*

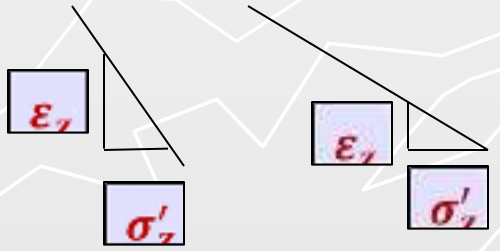
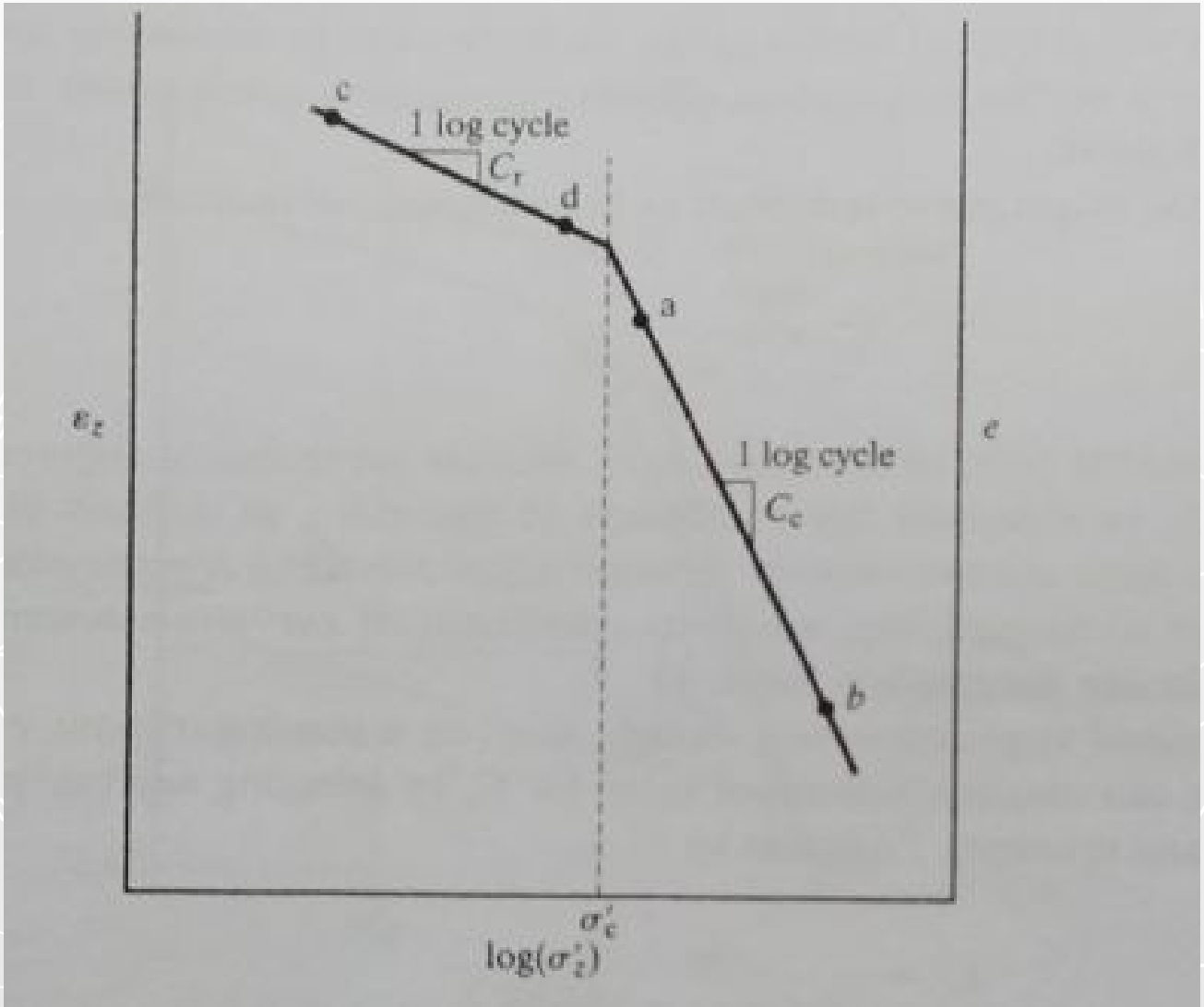


Figure: Slope of consolidation curves.



EWCE 341

What learned in the last class?

- Sample disturbance in consolidation test.
- Adjustments of laboratory consolidation data.

EWC 341

What we are going to learn today?

- Compression/recompression index and ratio.
- Consolidation status in the field.
- Problems

Question: Define compression index and compression ratio.

Compression Index and compression ratio:

Compression Index (C_c):

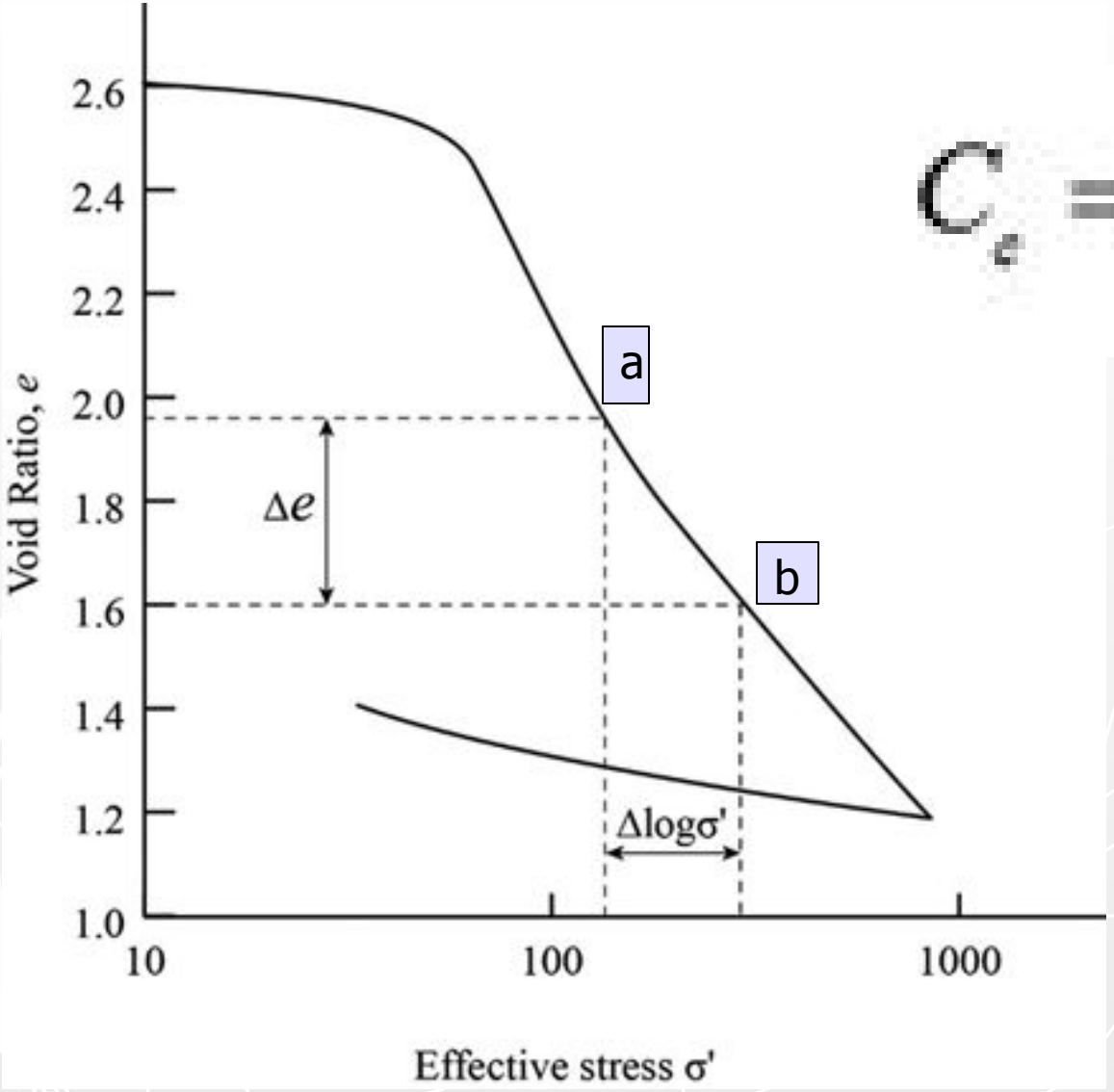
- Slope of the void ratio and log of effective stress in **virgin curve** is defined as the compression index, C_c.

$$C_c = \frac{de}{d \log \sigma'_z} = 0.05 \text{ to } 0.6$$

- For NC clay=0.2 to 0.6 and silts=0.2 to 0.4
- Higher value indicates greater compressibility and settlement potential.

Compression index is a measure of **compressibility of soil** but, **coefficient of curvature** describes the shape of the particle size distribution curve. Both has the same symbol C_c.

- $\mathbf{C_c = (D_{30})^2/D_{60}.D_{10}}$



$$C_e = \frac{\Delta e}{\Delta \log \sigma'}$$

- Reconstruction curve is straight line on a semi logarithmic e vs log σ'_z plot, so we can obtain a numerical value for C_c by selecting any two points, a & b, on the virgin curve.
- Now,

$$C_c = \frac{e_a - e_b}{(\log \sigma'_z)_b - (\log \sigma'_z)_a}$$

Compression Ratio ($\frac{C_c}{1+e_0}$):

- If the consolidation data are plotted only in ε_z - log σ'_z form ie **no void ratio data given**, then the slope of the virgin curve is:

$$\frac{C_c}{1+e_0} = \frac{(\varepsilon_z)_b - (\varepsilon_z)_a}{(\log \sigma'_z)_b - (\log \sigma'_z)_a}$$

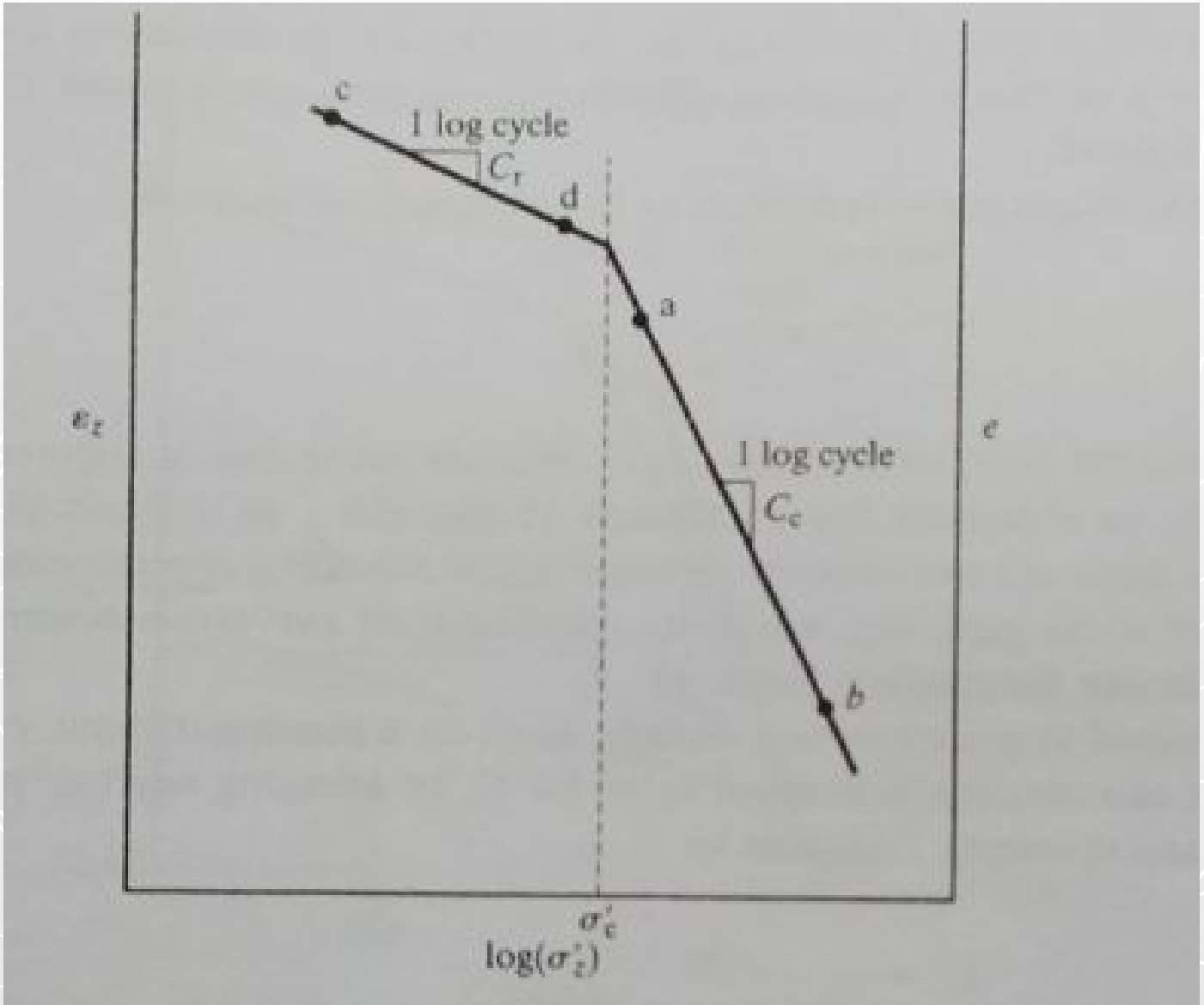
Here the parameter $\frac{C_c}{1+e_0}$ is **called compression ratio**.

If the reconstructed virgin curve is sufficiently long, it is convenient to **select points a and b** such that

$$(log\, \sigma'_z)_b = 10\, (log\, \sigma'_z)_a.$$

- It demonstrates that C_c **could be defined as the reduction in void ratio per 10-fold increase** (one log cycle) in σ'_z.
- Similarly, $\frac{C_c}{1+e_0}$ ie compression ratio is the strain ε_z per 10-fold increase in σ'_z.

Figure: Slope of consolidation curves.



Question: Define recompression index and recompression ratio.

Recompression Index and recompression ratio:

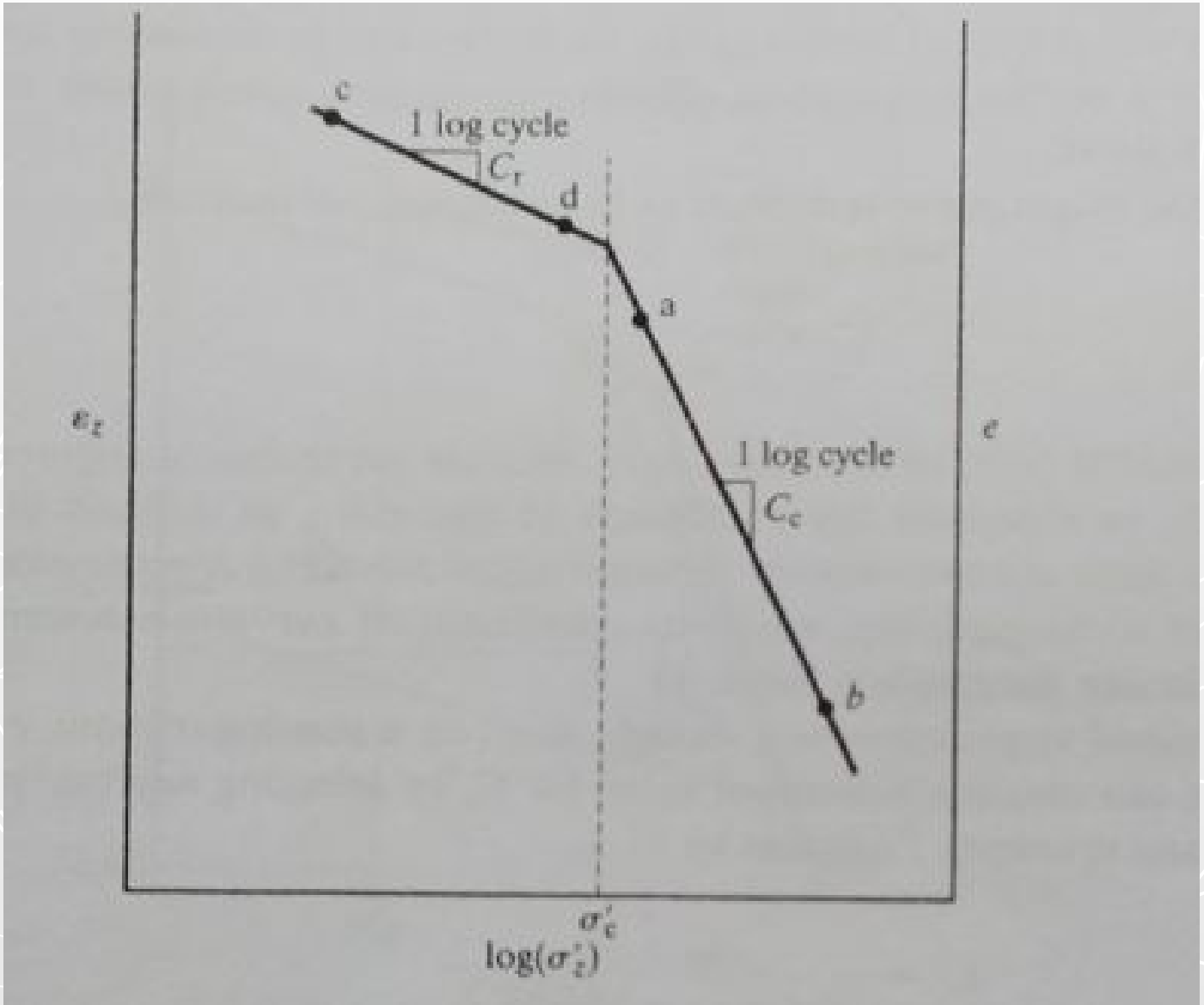
Recompression Index (C_r):

- **Slope of the recompression curve** is defined as the recompression index , C_r.
- We can obtain a numerical value for C_r by selecting any two points, c & d, on the **Re-loading curve**.

$$C_r = \frac{e_c - e_d}{(\log \sigma'_z)_d - (\log \sigma'_z)_c} = 0.04 \text{ to } 0.2$$

Represent stiffness of OC soils, which is less compressible because it has already undergone

Figure: Slope of consolidation curves.



Recompression Ratio ($\frac{C_r}{1+e_0}$):

- If the consolidation data are plotted only in ε_z - σ'_z form ie **no void ratio data given**, then the slope of the recompression curve is:
- $$\frac{C_r}{1+e_0} = \frac{(\varepsilon_z)_d - (\varepsilon_z)_c}{(\log \sigma'_z)_d - (\log \sigma'_z)_c}$$
- Here, the slope is $\frac{C_r}{1+e_0}$, which is called recompression ratio.

Consolidation Status in the Field

- When performing consolidation analyses, need to compare:

  -the pre-consolidation stress σ'_c,

  - with the initial vertical stress σ'_{zo}.

- **Pre-consolidation stress σ'_c,** is determined from the laboratory test data and **initial vertical stress σ'_{zo},** is determined using equation $\sigma_z = \sum \gamma H - u$ with the original field condition (ie without the new load) and original hydrostatic pressure.

- **Both values must be determined at the same depth**, it is the depth of sample on which consolidation test was performed.

Conditions of Soils Exists in the Field:

On the values of σ'_c and σ'_{zo} determined, we need to assess the **three conditions of soils exist in the field**:

Normally Consolidated Soils:

- If $\sigma'_{zo} \approx \sigma'_c$, then the vertical effective stress in the field has **never been higher** than its current magnitude.
- Condition is known as being *normally consolidated* ***(NC)*.**

Example- soils at the **bottom of lake,** where sediments brought in by a river have slowly accumulated.

- Soils will be normally consolidated if σ'_{zo} and σ'_c are **equal within about $\pm$20%.**

Over-consolidated Soils:

- If $\sigma'_{zo} < \sigma'_c$, then the vertical effective stress in the field was **once higher** than its current magnitude.

- Condition is known as being over-consolidated **(OC).**

Processes that can cause a soil to become over-consolidated:

- **Extensive erosion or excavation** such that ground surface elevation is now much lower than it once was.

- **Surcharge loading from a glacier**, which has since melted.

- **Surcharge loading from a structure**, such as storage tank, **which has since been removed.**

Under-consolidated Soils:

- If $\sigma'_{zo} > \sigma'_c$, the soil is said to be **under-consolidated,** which means the soil is still in the process of consolidating under the previously applied load.

- Soil is still in the **process of losing water** from its pores and its volume is decreasing, which is why it is known as an "under-consolidated" soil.

- Soils include **soft clays, organic-rich soils, and peat** that have not fully settled under their own weight or recent loads.

- Example: **Rapidly deposited sediments in riverbeds** or ocean floors or **soils from recently placed embankments** or landfill fills.

Ex. 2: The element of soil **at point A**, shown in the following Figure 1. A consolidation test has been performed on a sample of soil obtained from Point A. The test results are plotted and adjusted by Casagrande procedure shown in Figure 2. Determine **whether the soil at Point A is normally consolidated or over-consolidated.** The proposed fill in the Figure 1 has not yet been placed. Finally compute C_c and C_r.

Solution:

- Element of soil at point A is initially subjected to following stress:

$$\sigma'_{z0} = \sum \gamma H - u$$

$$\sigma_{z0} = \sum \gamma H$$

$$= (18.7\ \text{kN/m}^3)(1.0\text{m}) + (19.0\ \text{kN/m}^3)(2.0\text{m}) + (16.5\ \text{kN/m}^3)(4.8\text{m}) = \mathbf{136\ kPa}$$

Solution:

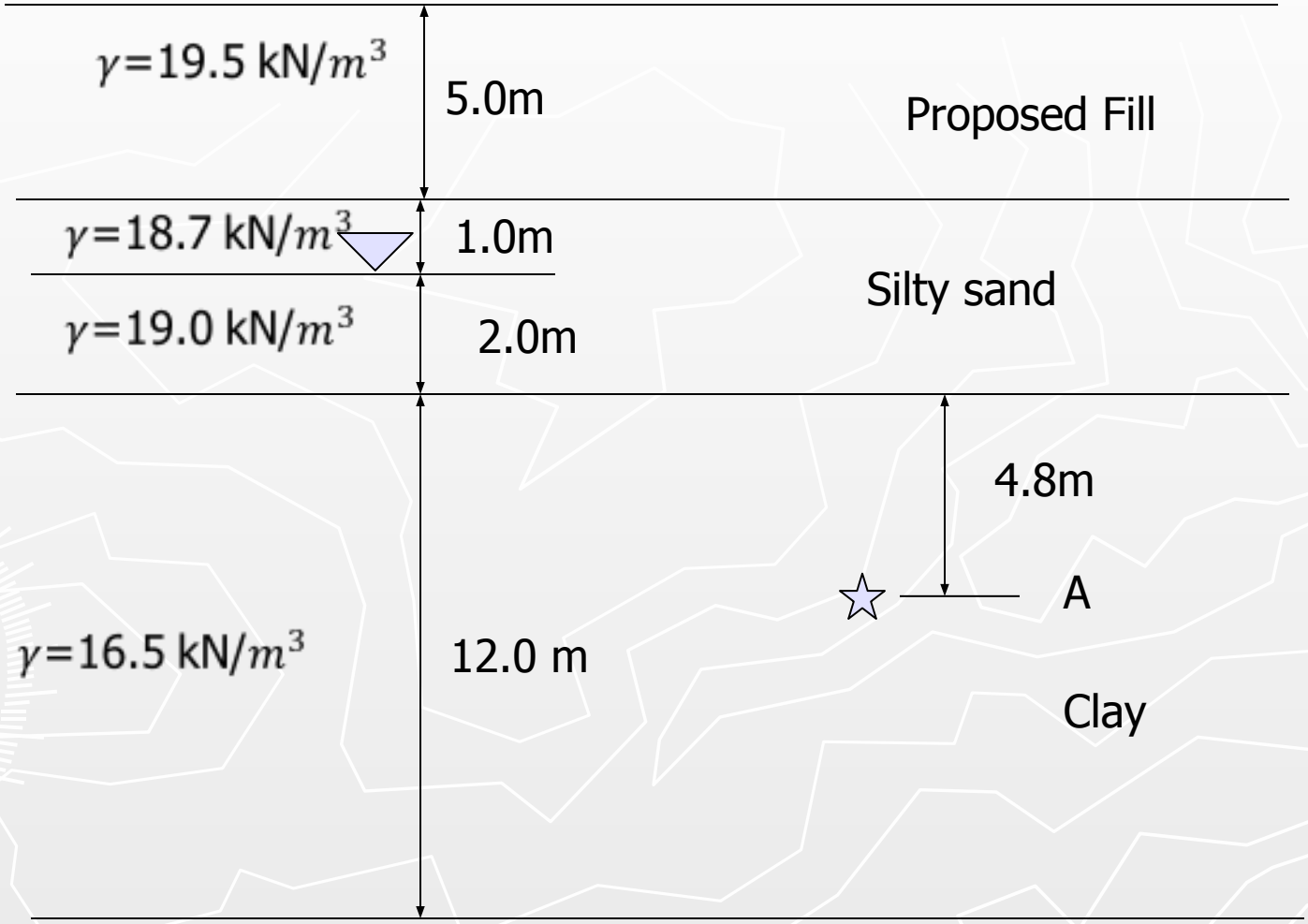


Figure 1: Soil profile

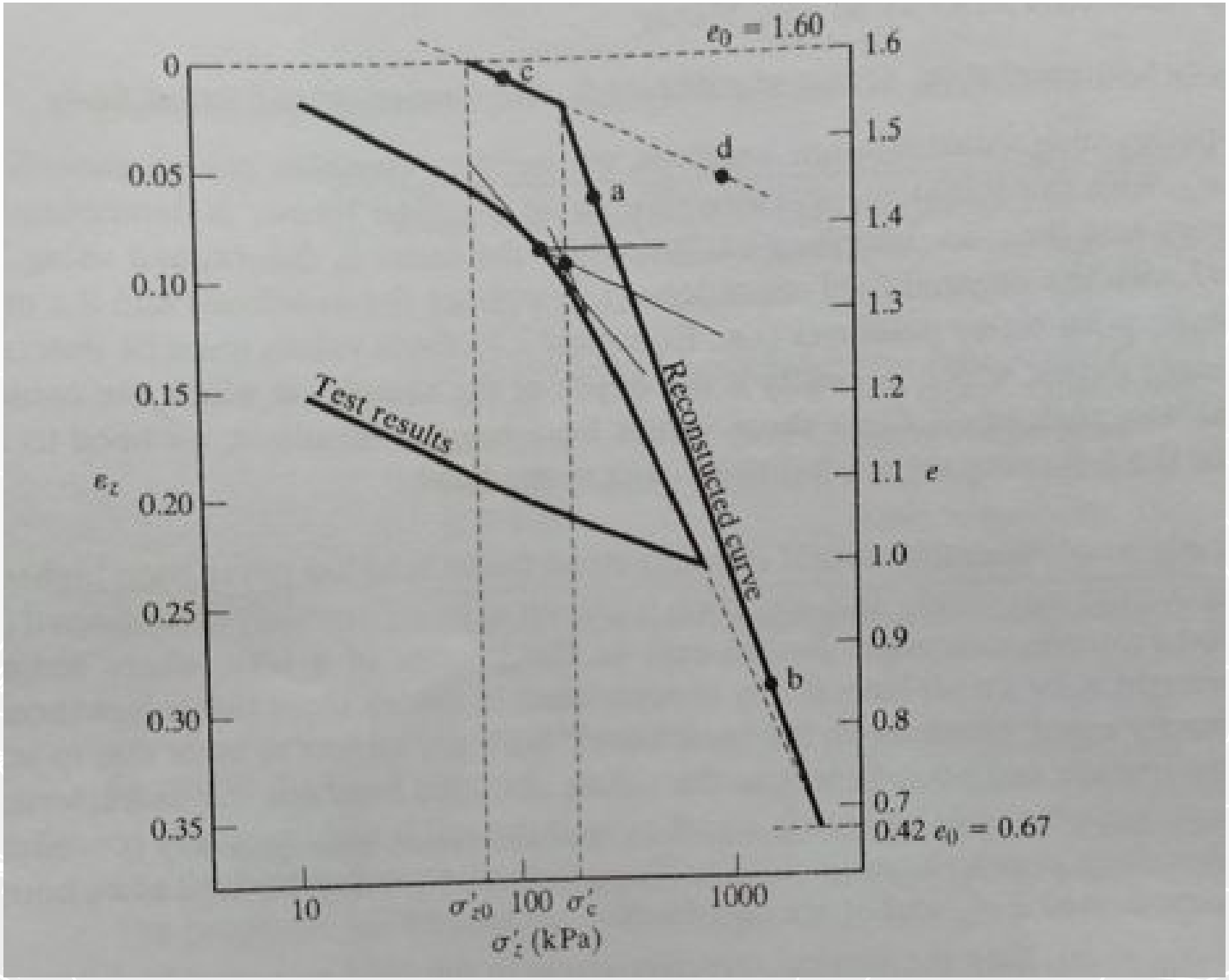


Figure 2: Adjusted consolidation data from laboratory data

$\mathbf{u = \gamma_w h_w}$

$= (9.8\ \text{kN}/m^3)(6.8.0\text{m}) = \mathbf{67\ kPa}$

$\mathbf{\sigma'_{z0} = \sigma_{z0} - u}$

$= (136 - 67)\ \text{kPa} = \mathbf{69\ kPa}$

From Figure 2. Computed $\mathbf{\sigma'_c = 140\ kPa}$

Here, $\mathbf{\sigma'_{z0} < \sigma'_c}$ **by more than 20% (51%),** so the soil is over – consolidated **(OC).**

Solution:

Slope of the reconstructed lines:

$$C_c = \frac{e_a - e_b}{(\log \sigma'_z)_b - (\log \sigma'_z)_a}$$

$$C_c = \frac{1.43 - 0.84}{(\log \sigma'_z)_b - (\log \sigma'_z)_a}$$

$$= \frac{1.43 - 0.84}{\log 248\ kN/m^3 - \log 40\ kN/m^3} = \mathbf{0.74}\ (0.05 \text{ to } 0.6)$$

Highly compressible clay

$$C_r = \frac{e_c - e_d}{(\log \sigma'_z)_d - (\log \sigma'_z)_c}$$

$$= \frac{1.58 - 1.46}{\log 157 \frac{kN}{m^3} - \log 16\ kN/m^3} = \frac{0.12}{1} = \mathbf{0.12}\ (0.02 \text{ to } 0.2)$$

Stiffer soil

Thank you

EWCE 341: Principles of Soil Mechanics

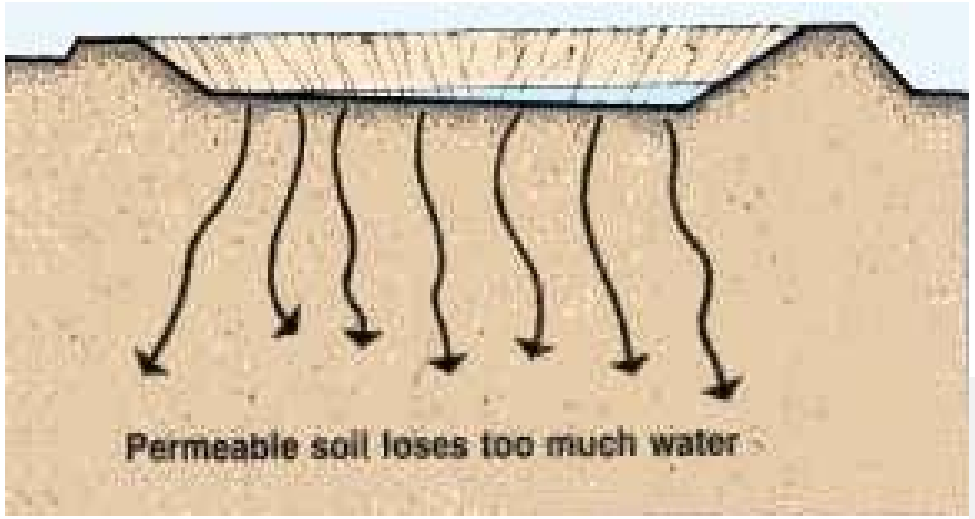
Permeability of Soil

What we are going to learn today?

- What is permeability of soil?
- Permeability is an important engineering property of soils.
- Hydraulic heads.
- Factors Affecting Permeability of soils
- Uses of Permeability of Soils

What is permeability of soil?

- **Property of soil** which ***permits flow of water*** through the interconnected voids/pores of it**.**
- Permeability is the **ease with which water can flow through voids/pores of soils**.



Soil type	Permeability k (m/sec)	Reducing permeability
Gravel	>0.1	↓
Sand	$0.1 - 10^{-4}$	
Silt	$10^{-4} - 10^{-8}$	
Clay	$<10^{-8}$	



Permeability is an important engineering property of soils?

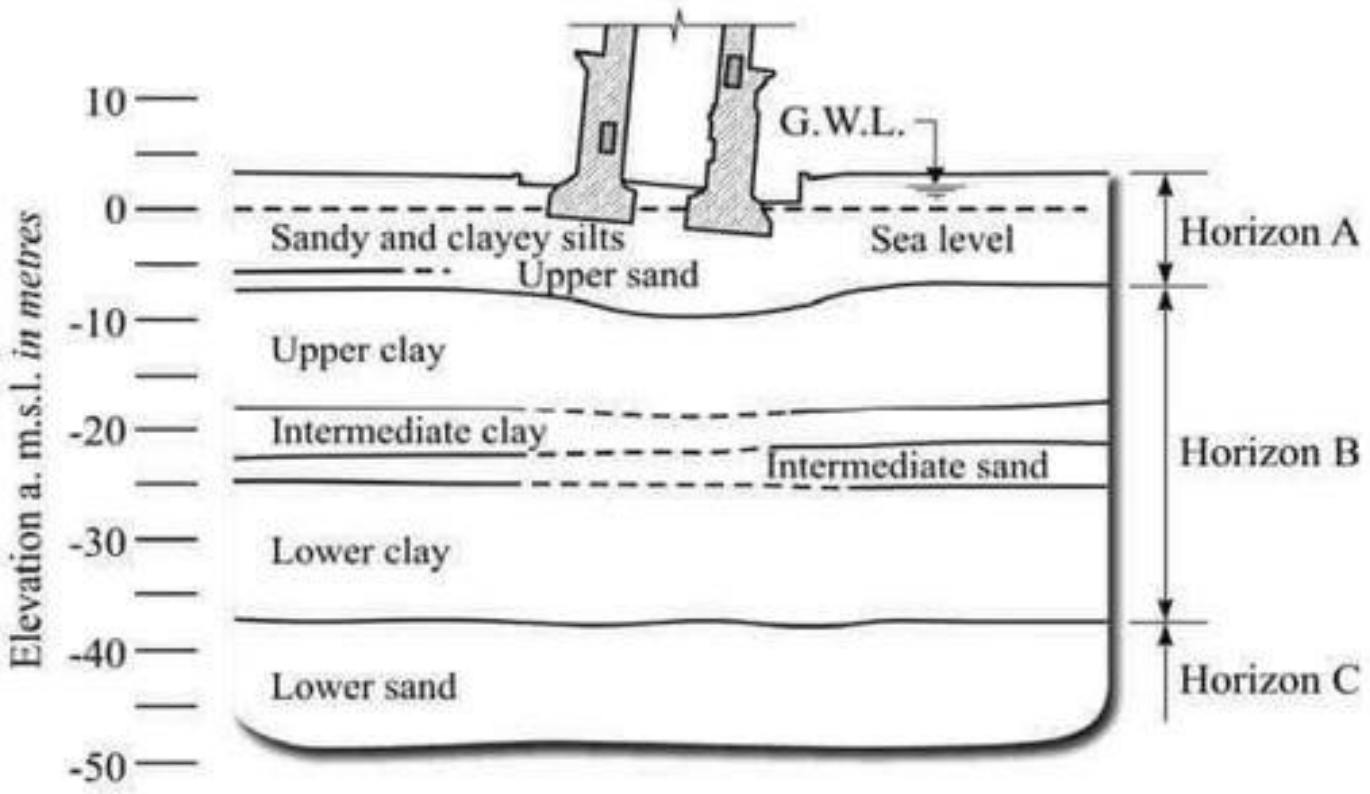
A **knowledge of permeability is essential** in a **number of soil engineering problems:**

- **Influences** the **rate of settlement** of a saturated soils under load.
- **Stability of slopes and retaining structures** can be greatly affected by the permeability of soils.
- **Design of earth dams** is very much based upon the permeability of soils used.
- **Design of Filters (to prevent piping and erosion)** are based upon their permeability.
- **Rate of flow of wells** from an aquifer is dependent on permeability.
- **Performance of land fill liners** is based upon their permeability.

Question: Why permeability is an important engineering property of soil?

- **Influences** the **rate of settlement** of a saturated soils under load.





Settlement of a structure

- **Stability of slopes and retaining structures** can be greatly affected by the permeability of soils.

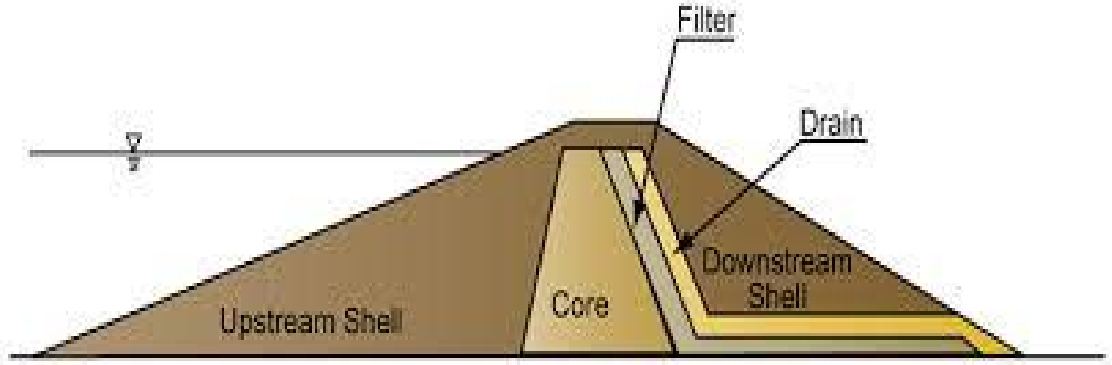


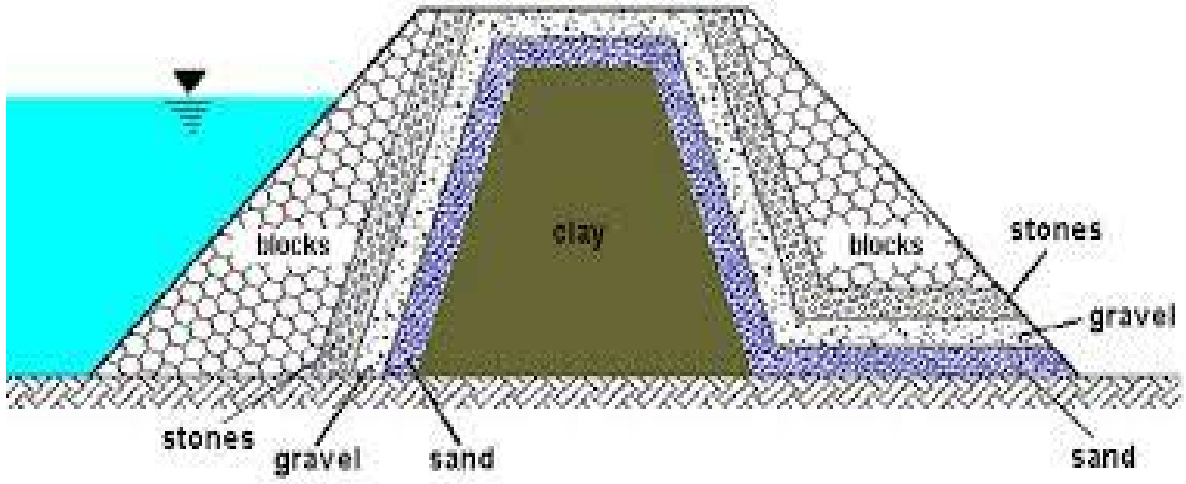




Stability of Slope and Retaining Structure

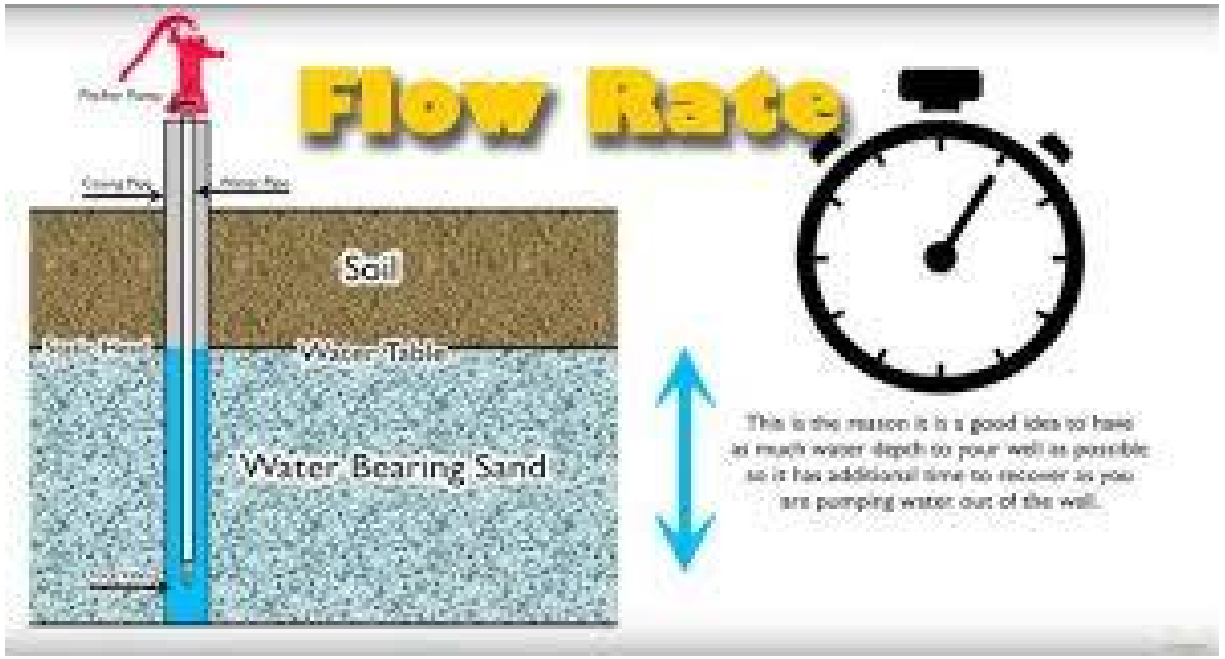
- **Design of earth dams** is very much based upon the permeability of soils used.
- **Design of Filters (to prevent piping and erosion)** are based upon their permeability.







Earth Dam and its Typical X-Section



- **Rate of flow of wells** from an aquifer is dependent on permeability.
- **Performance of land fill liners** is based upon their permeability.

Landfill Liners: A **landfill liner**, is intended to **be a low permeable barrier**, which is laid down under **engineered landfill** sites.

A landfill bottom liner system can consist of:

- **A base layer of clay** that has low permeability.
- **A synthetic membrane bottom liner.**



Hydrology

Definition: Study of water movement across the earth.

Surface Water Hydrology: Assessment of rainfall intensities, streams flow, lake water level

Ground Water Hydrology: Study of Ground Water





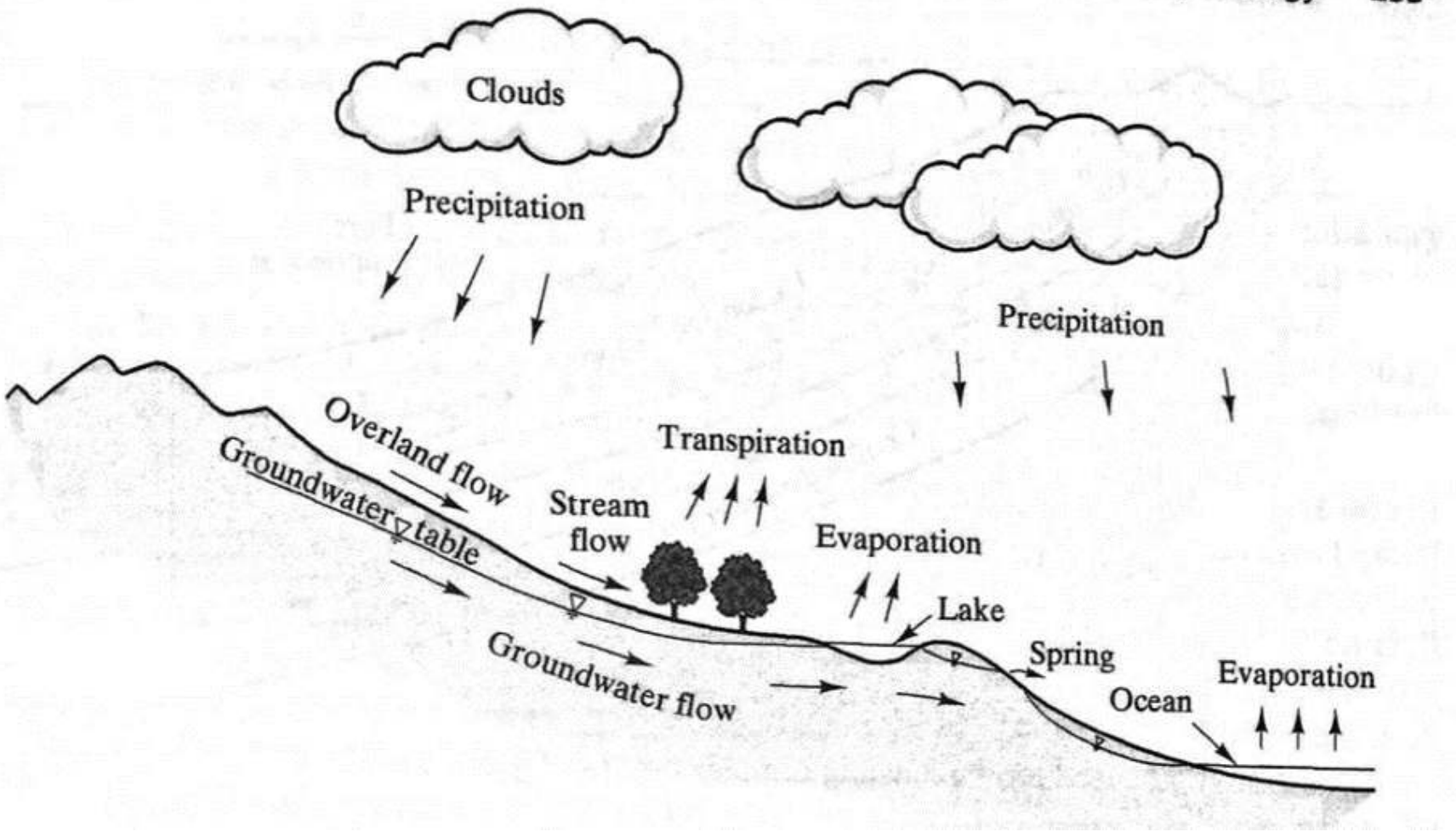
Hydrologic Cycle

Movement of water across the earth is ultimately driven by energy received from the sun.

- **Evaporation and Transpiration:** – water rises in to the sky and form clouds
- **Precipitation**:- (rain, sleet, hail, snow)
- **Infiltration**:- recharge GW. Some remains near the GS but much of it reaches GW level, fully saturated zone

Process occurs both in high land and low lands. **GW eventually gains enough potential energy and begins to flow, reappears** on ground surface as spring or seeps to low spots then to ocean.

Hydrologic Cycle



Groundwater Hydrology

Geotechnical Engineers are **mostly concerned with that portion of hydrologic cycle that occur underground**.

- **Subsurface water**: all under ground water, specially all water located within **soil voids** and **rock fissures**.

GWT:-Most important of **subsurface water**.

- GWT location is important and finding it is one of the primary objectives of **site characterization program.**
- Located by **installing observation wells** and allowing GW to seep into them until it reaches equilibrium.
- **Water elevation** inside wells is GWT.
- Soil **profiles represent GWT** as a line marked with a triangle.
- GWT often **changes with time**, depending on the season of the year, recent patterns of rainfall, irrigation practices, pumping activities and other factors. Fluctuation may very from 1m to 10m or even more in only a year or two.

Groundwater Hydrology (cont--)

- GWT conditions **encountered in an exploratory boring** are not necessarily those we use for design.
- Often need to use observed conditions as a basis for **estimating the worst condition** that are likely to occur during the project life.

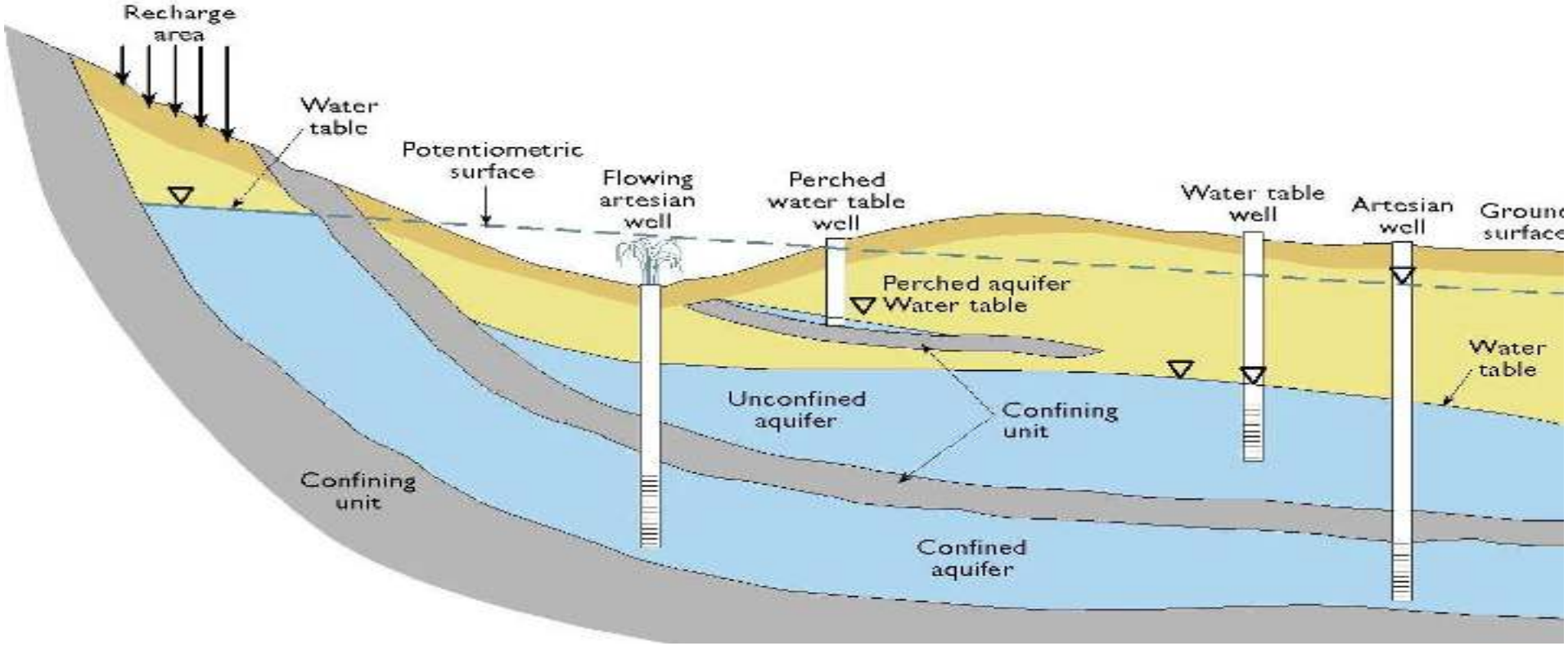


Figure: Soil Profile showing complex nature of groundwater

Groundwater Hydrology (cont…)

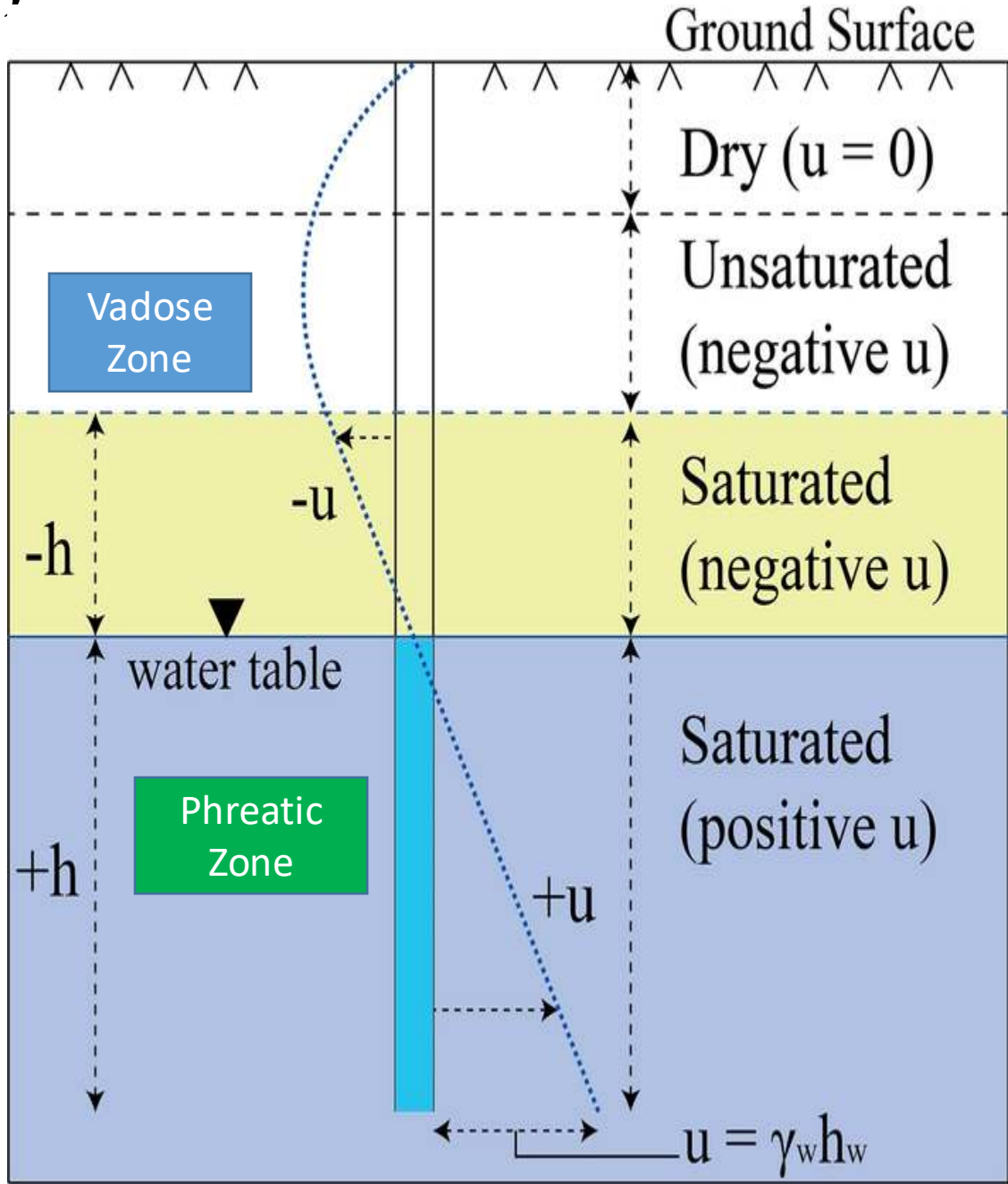
Division of subsurface water:

Phreatic Zone:

- Portion below the GWT
- Water is subjected to +ve pressure as a result of weight of overlying water and other causes.
- Most subsurface water is in phreatic zone.

Vadose Zone:

- Portion **above** GWT
- Water has **-ve pressure**.
- Held in place by capillary action and other forces present in soils.



Groundwater Hydrology (cont..)

Technically **Only water in phreatic zone is true GW**. But often the term GW to describe all subsurface water

- **Aquifers**: Soils that can **transmit large quantities** of water. (**sands and gravels**).
- **Aquicludes**: Transmit water slowly, **clays**
- **Aquitards**: Flow of water at **low to moderate rate**, silty sands.

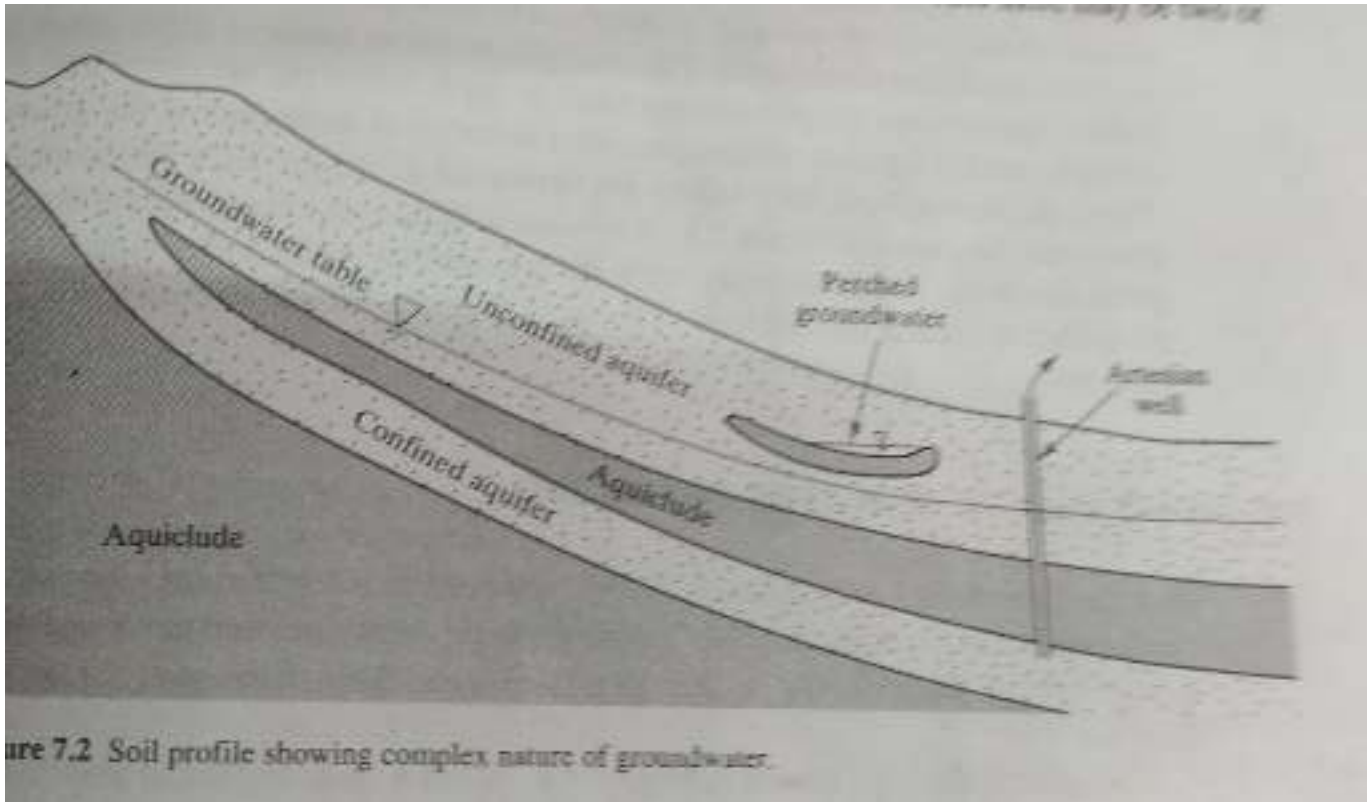
All three may be in a single soil profile, flow can be more complex.

Unconfined Aquifers:

- Upper aquifer in which **bottom flow** boundary is defined by an **aquiclude** but **upper boundary** is free to reach it own natural level.
- **GW occupies lower portion of aquifer**.
- **Flow regime-** zone of soils through the water flows.
- If GW arrives at site, GWT would rise accordingly.

Confined aquifers:

- **Lower aquifer** in which **both lower and upper flow boundaries** are defined by aquiclude and **similar to a pipe** that is flowing full.
- **Most are artesian**, **water at top of aquifers i**s under pressure.
- People often drill wells into because **water will rise up** through the aquiclude without pumping.
- If water pressure is high enough, artesian wells **deliver water all the way to GS without pumping.**



ure 7.2 Soil profile showing complex nature of groundwater.

GW Exploration and Monitoring

- Presence of water in soil pores and rock fissures has a very significant impact on engineering behaviors of soils.
- **Site characterization program needs to assess GW conditions.**

How to assess:

- **Drilling a Boring or Excavation**

i. May **observe small seeps**, with moisture tickling in to hole.
ii. May be due to small **non-uniformities in soil conditions** that have **trapped water** at certain level.

Perched GW –Large zone of trapped water.

- **Continuous Drilling**

i. To a great enough depth, **GW eventually encountered**.
ii. It is the level to which **water fills open boring**.

Determination of GWT



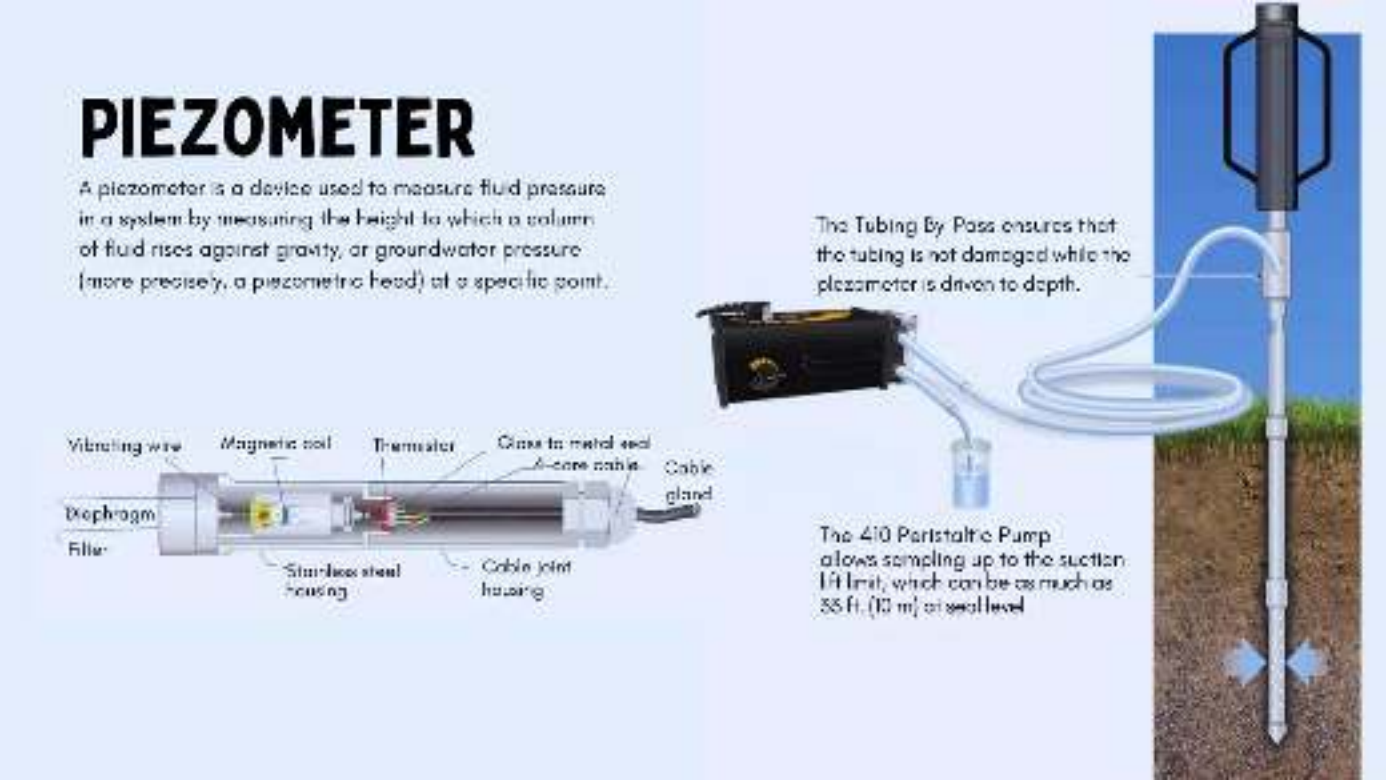
Open Boring

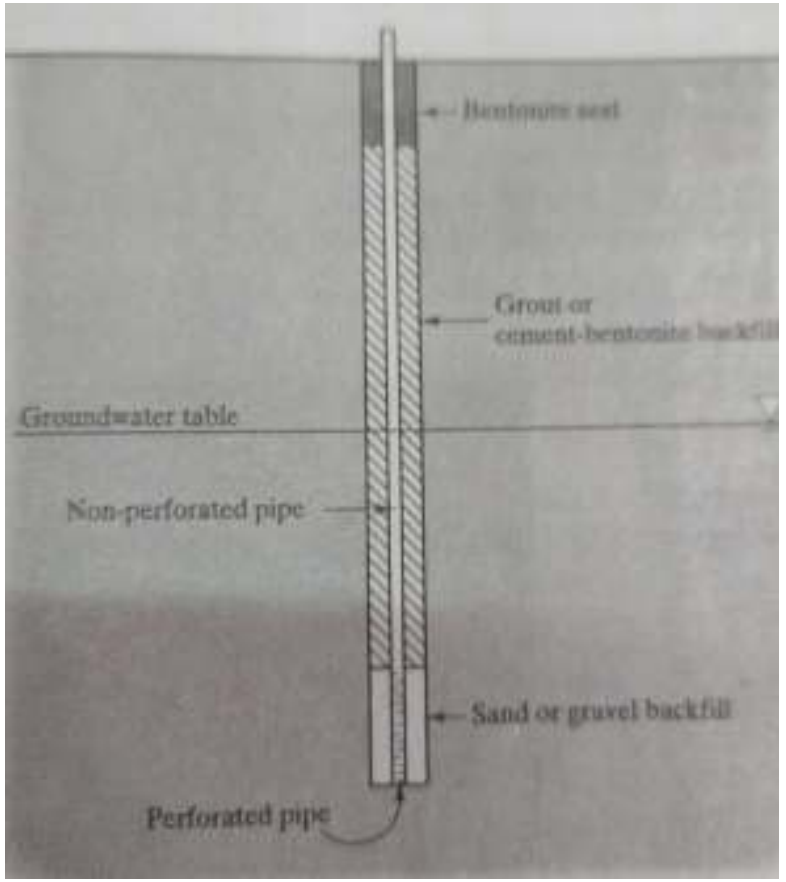
- Sometimes, water **quickly flows** (**sands, gravels**) into hole, **reaching equilibrium** in an hour or less.
- In these case, **GWT can be located** in the open hole before it is backfilled.
- In silty or clayey soil, **may require hours** or even days to reach equilibrium.
- Leaving hole open long time, may **pose safety problems**.
- GWT table often **changes with time** and may require to monitor these changes.
- Both problems can be **solved by installing instrument** in the observation well in the field.

Field Instrumentation – to monitor GWT

- **Install piezometers** in the observation well of ground.

- **Open stand pipe piezometer**, similar to observation well except the inside of pipe is hydraulically connected on one point in the soils.

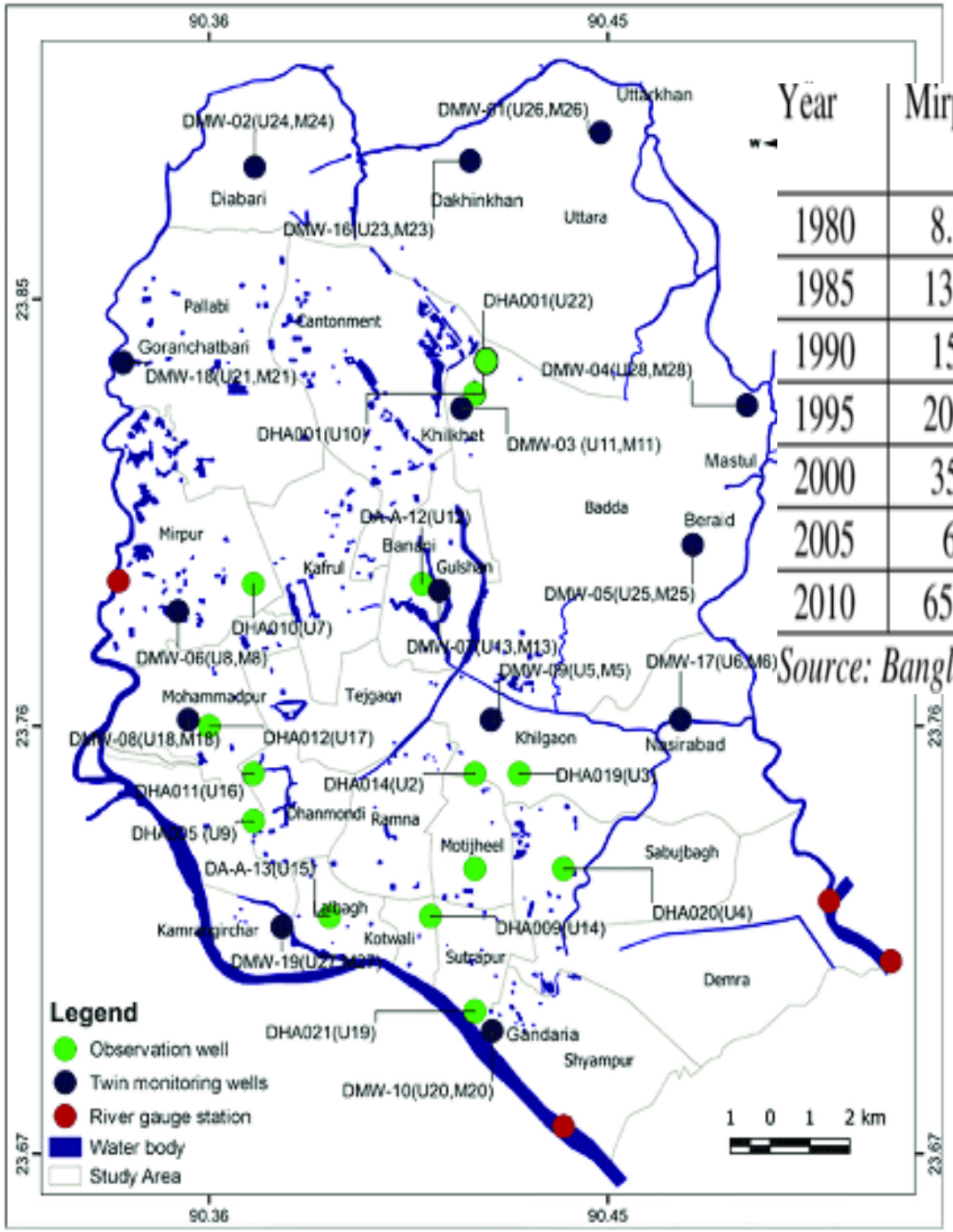




- **Observation well**, which is hydraulically open along the entire length.

GW is able to **flow freely** in and out of the pipe, so the water level inside is GWT

Depth can be **measured by electronic probe / electronic** WL indicator or simply **lowering a cloth tape**.



Year	Mirpur	Mohammadpur	Dhanmondi	Gulshan	Cantonment	Lalbag	Sabujbag	Sutrapur
1980	8.36	9.3	10.12	9.91	-	1.67	3.13	5.88
1985	13.57	13.5	12.76	9.95	-	16.06	13.63	8.93
1990	15.3	14.52	15.4	10	-	17.71	21.49	11.08
1995	20.76	20.48	24.27	24.25	18.46	19.78	26.43	15.67
2000	35.2	20.87	40.12	27.59	27.88	36.57	46.45	18.79
2005	63	32.21	60.85	37.3	27.8	44.77	54.4	20.12
2010	65.97	31.24	66.32	42.66	27.81	44.77	54.4	21.1

Source: Bangladesh Water Development Board, 2014

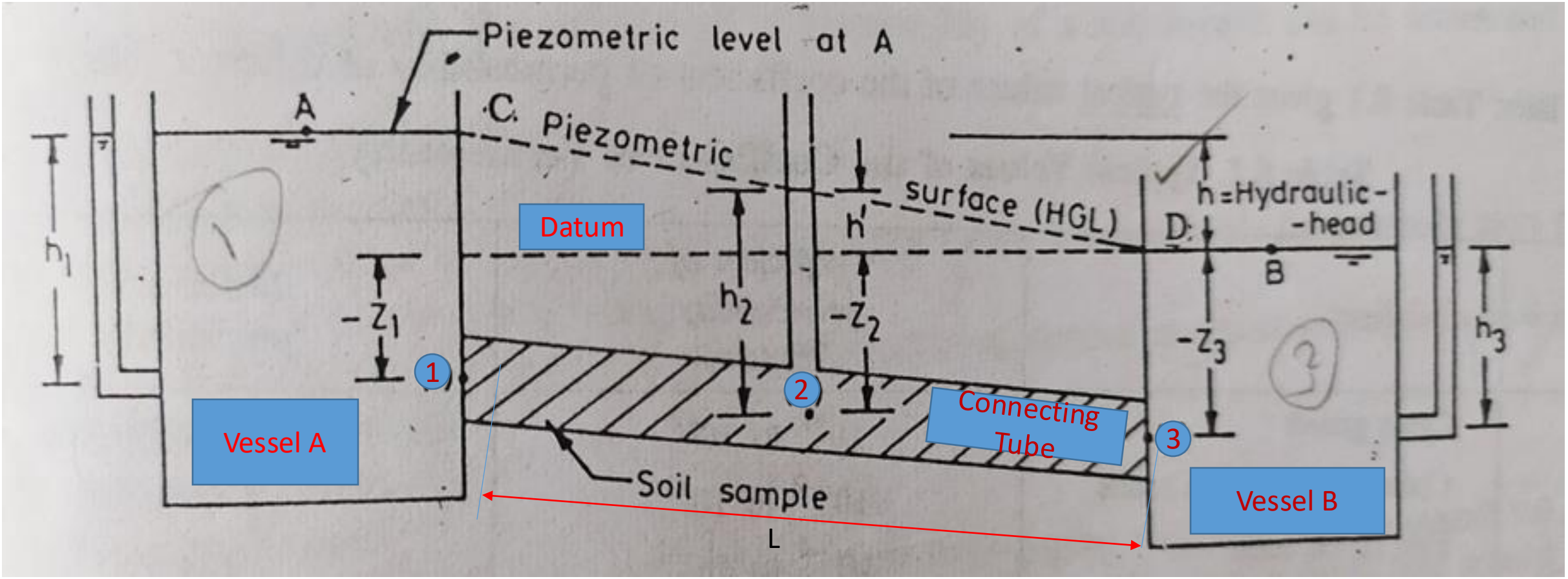
Hydraulic head or **piezometric head:** It is a specific measurement of liquid pressure above a vertical of datum.

Different Types Hydraulic Head:

- **Total Head** at any point in a flowing fluid
  = **elevation (datum) head**+ **pressure head** + **velocity head**.
- **Elevation Head** = vertical distance of the point from the **datum**.
- **Pressure Head**= head indicated by a **piezometer.**
- **Velocity Head**= $\frac{v^2}{2g}$, for flow of water through soils, as velocity is extremely small, the velocity head is **neglected**.

Total Head at any point in a flowing water= **elevation (datum) head**+ **pressure head**

Various Heads



Point	Elevation Head	Pressure Head	Total Head
1.	$-z_1$	h_1	$h_1 + (-z_1)$
2.	$-z_2$	h_2	$h_2 - z_2$
3.	$-z_3$	h_3	0

- Two **vessels** A & B containing water at different levels and connected by a **small tube** (of length L) containing soil sample.
- **Flow takes** place from vessel A (with high head) to vessel B (with low head) through the tube.
- **Datum** at the water level in vessel B.

- Elevation head, pressure head and total head **at points 1, 2 and 3** are shown in the figure.

Hydraulic Head:

- **Total head** at point at 1, $\boldsymbol{h_1 - z_1 is\ h}$ and at point 3, is zero $\boldsymbol{ie. h_3 - z_3}$.
- Head **h** is known as the **hydraulic head** for which water flows from **vessel A to vessel B.**
- **h** = differences in elevations of water levels at the entry and exit points of soil mass (h_1 - h_3) at point 3.
- h'= **Loss of head** through soil at point 2, also known as **effective head**.

Hydraulic Gradient:

- **Loss of head per unit length of flow** through the soils.
- $i = \frac{h}{L}$, h= hydraulic head, L= length of the soil specimen.
- A large hydraulic gradient reflects extensive friction.

Hydraulic Gradient Line (HGL):

- Line CD drawn by joining the points of head variations.
- Also known as **Pressure Gradient Line**.
- If piezometer is inserted at he intermediate point 2, water will rise up to the level of HGL, CD, which is also known as **Piezometric Surface**.
- Assumed that **the loss of head over the length** over the length of soil sample is **uniform** and **variation is linear**.

Factors Affecting Permeability of soils

- **Grain or particle size**

  $k=CD_{10}^{2}$

Alan Hazen equation

- **Void Ratio (e)**

  More void ratio more permeability, $k \propto e^{2}$

- **Soil Composition**

  Gravel, sands and Silts – **Presence of mica** can decrease the permeability

  Clay – water attracted between clay particles reduce permeability

- **Structural Arrangement**

  Remolding of natural soils reduces permeability

- **Stratification**

  Water flowing parallel to strata, permeability will be more.

- **Presence of Foreign Particles and Entrapped Air**

  Reduces void spaces and block interconnected pores, affects permeability.

- **Degree of Saturation**

  If soil is **saturated or dry**, permeability will be more or less.

What we have discussed today?

- What is permeability of soil?
- Permeability is an important engineering property of soils.
- Hydraulic heads.
- Factors Affecting Permeability of soils

Thank You

EWCE 341: Principles of Soil Mechanics

Permeability

What we discussed in previous class?

- What is Permeability?
- Important in a no. of soil engineering problems.
- GW Hydrology?
- Assessment of GWT
- Factors affecting permeability of soils

What we are going to learn today?

- Darcy's Law
- Determination of K?
- Problems

Darcy's Law:

- **Flow of free water through soil** is governed by the Darcy's Law.

- In 1856, Darcy demonstrated experimentally that for **laminar flow** in **homogenous soil,** the velocity of flow (v) is given by:

  $v = ki$ *where, k= coefficient permeability, i = hydraulic gradient*

  $i = \frac{h}{L}$, h= hydraulic head, L= length of the soil specimen

Velocity of flow v=**discharge velocity**.

Discharge is obtained by multiplying the velocity of flow by the total x-sectional area of soil (A) **normal to the direction flow**.

Q= $vA = kiA$ **Area A includes both solids and voids**.

Darcy's Law found valid for a **wide range of soil types from clay to coarse sands**.

Limitations:
For **clean gravels**, **accuracy of flow diminishes** as a **turbulent flow**.
For **clay**, with low hydraulic gradient, **accuracy diminishes,** as **flow rate is so small**.

Coefficient of Permeability:

- Defined by equation, $v=ki$

If $i = \frac{h}{L} = 1.0$ **then v = k**

Coefficient permeability **(k) is defined as the velocity of flow which would occur under unit hydraulic gradient,** expressed as velocity unit (cm/sec).

k of a homogeneous and isotropic soils depends on primarily:

- Viscosity of pore water (η)
- Void ratio (e) of the soil
- Size and shape of soil particles
- Degree saturation (S)

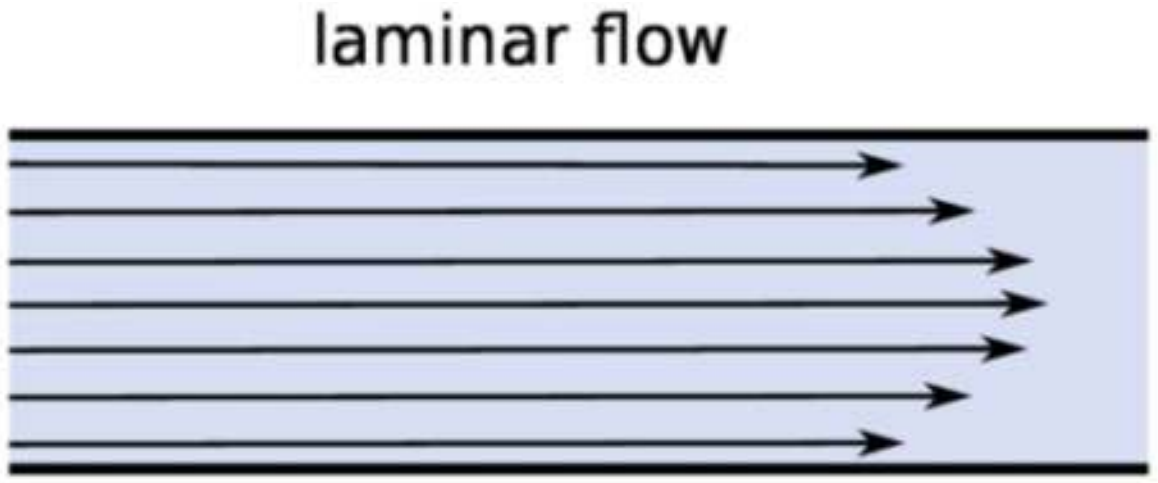
Some Typical Values of *k in Soils:*

Soil	*K* cm/sec	Degree of Permeability
Gravel	$> 10^{-1}$	Very High
Sandy gravel, clean sand, fine sand	$10^{-1} > k > 10^{-3}$	High to Medium
Sand, dirty sand, silty sand	$10^{-3} > k > 10^{-5}$	Low
Silt, silty clay	$10^{-5} > k > 10^{-7}$	Very Low
Clay	$< 10^{-7}$	Virtually Impermeable

GW Flow:

Laminar Flow

- **Laminar flow** occurs when a **fluid flows in parallel layers**, with no disruption between the layers.
- At **low velocities**, the fluid **tends to flow without lateral mixing**, and **adjacent layers slide past one another** like playing cards.
- In laminar flow, the **motion of the particles of the fluid is very orderly** with all particles moving in straight lines parallel.

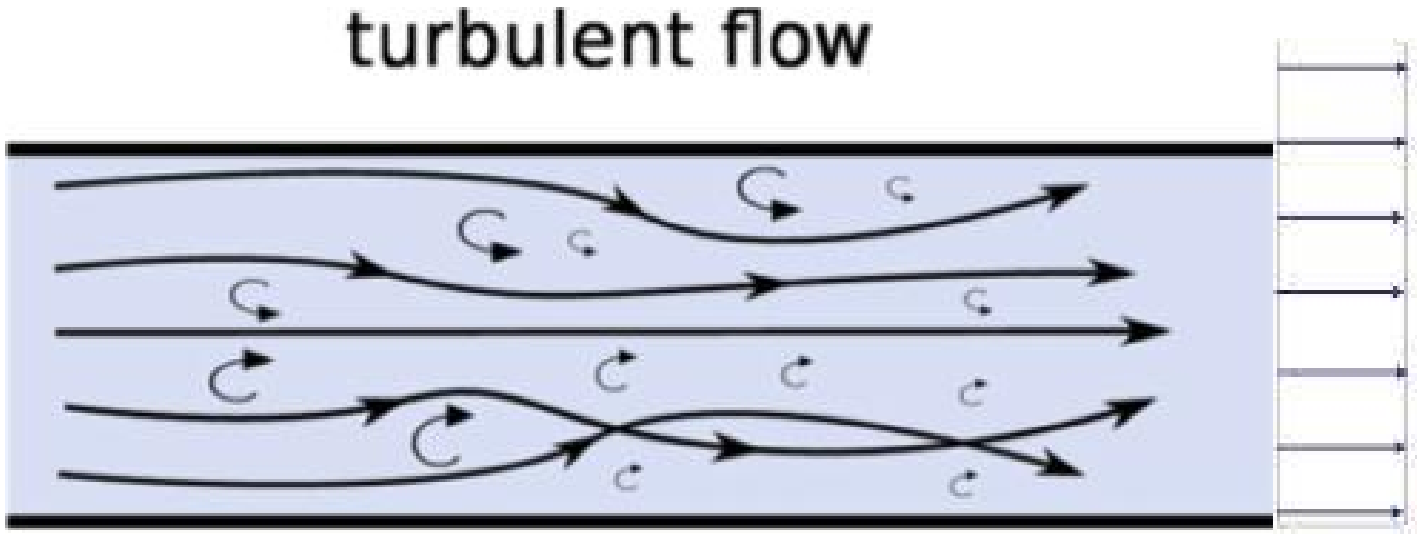


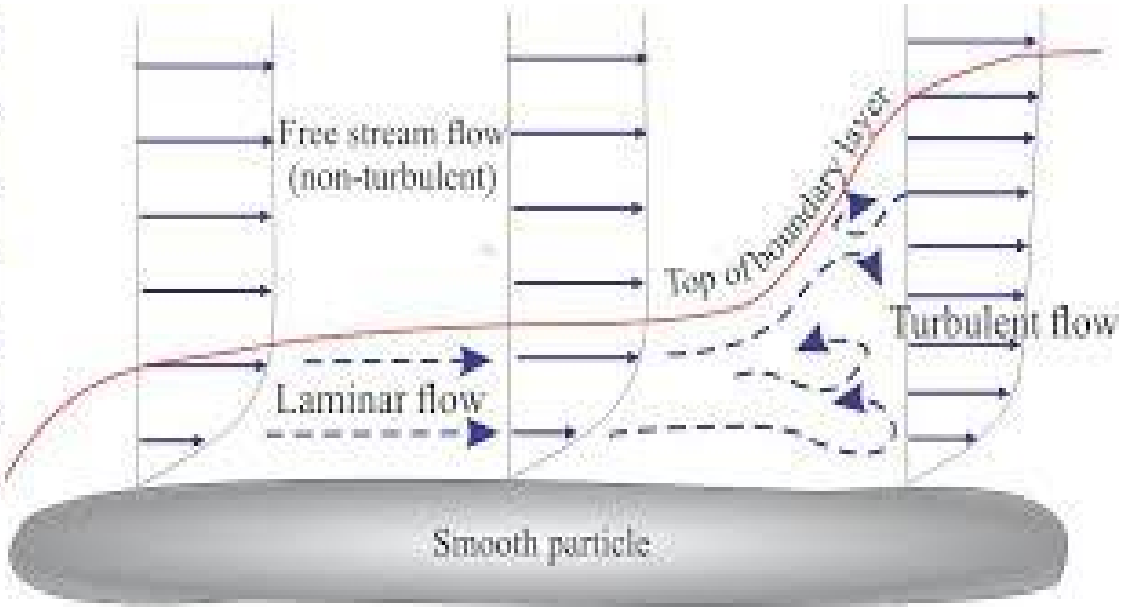
Turbulent Flow

- **Turbulent flow** is characterized by **chaotic property changes**.

includes **rapid variation** of **pressure** and flow **velocity** in space and time.

- Fluid **no longer travels in layers** and **mixing across**.
- Flows at **Reynolds numbers** **larger than 1** are typically (but not necessarily) **turbulent**, while those at low Reynolds numbers below 1 usually remain **laminar**.
- Flow in the range of Reynolds numbers in between and known as **transition**.





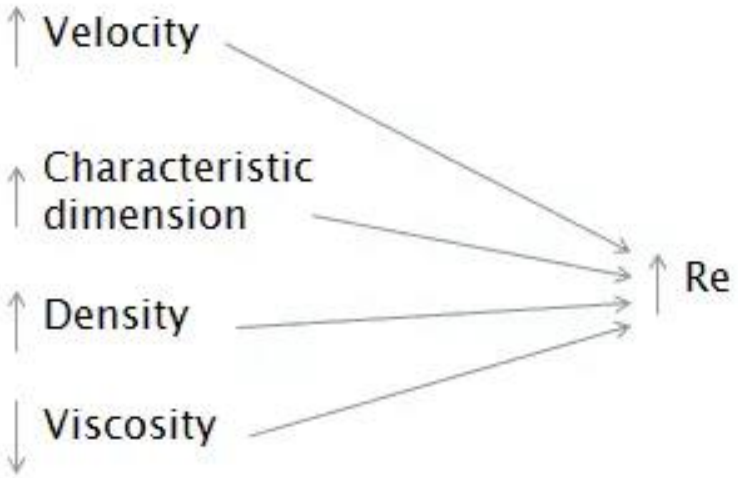
Reynolds Number

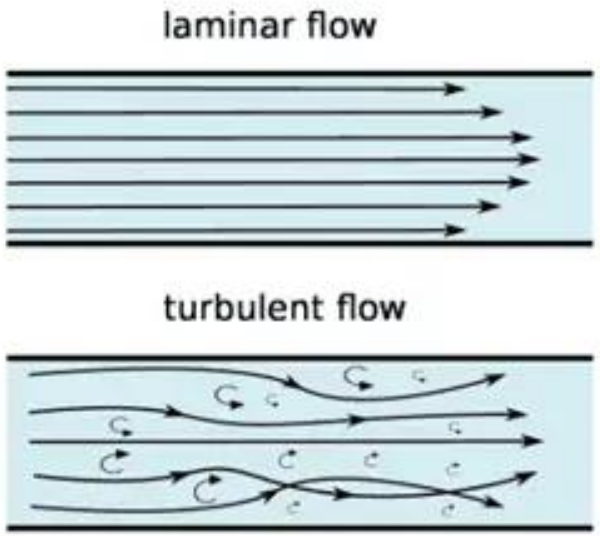
- Reynolds number, **referred to as Re**, is used to determine whether the fluid flow is laminar or turbulent.
- Although the Reynolds number **comprises** both static and kinetic properties of fluids, it is **specified as a flow property** since dynamic conditions are investigated.
- Reynolds number is **the ratio** of the inertial forces to the viscous forces.

$$\mathbf{Re} = \frac{inertial\ force}{viscous\ force}$$

- Ratio helps to **categorize laminar flows from the turbulent ones.**
- $\mathbf{Re} = \frac{\rho V L}{\eta}$ or $\frac{\rho V D}{\eta}$

$$Re = \frac{\text{inertia forces}}{\text{viscous forces}} = \frac{\rho \cdot u \cdot D}{\mu}$$





Reynolds Number through **pipe flow**

Flow Type	Reynolds Number range
Laminar	Upto Re-2300
Transition	2300 <Re<4000
Turbulent	Re> 4000

Reynolds Number – flow of water through soils

- For the flow of water through soil, the flow is laminar if Reynold's number **is less than 1**. In **fine-grained soil**, flow is laminar.

- In **coarse-grained soil**, the flow becomes turbulent when the size of particles is very large i.e. in gravel range

Validity of Darcy's Law

Darcy's Law is valid if the flow through soils is laminar flow:

- Flow of water through **soils** depends upon the **dimensions of interstices**, which in turn **depends on particles size**.
- **In fine-grained soils**, the **dimensions of interstices are very small** and flow is also generally laminar.
- In **very coarse-grained soils**, such as coarse gravel, flow may be turbulent.

For Flow of water through pipes, the flow is laminar when the Reynolds Number is less than 2300.

For flow of water through soils:

- Flow is **laminar** if the Reynolds Number is **less than unity**.
- **Characteristics length** of in the Reynolds number is taken as the average particle diameter (D).
- $\frac{\rho V D}{\eta} \leq 1$, maximum diameter of the particles for the flow to be laminar is about 0.50mm.
- Value of **Re $\leq$ 1** is however conservative.
- Flow **remains laminar** even up to **Re is 75**.

Validity of Darcy's law

- Darcy's law is **valid** for flow in **clays, silts and fine sand**, flow is laminar
- Darcy's Law **may not be applicable**, in **coarse sands, gravels and boulders**, flow is turbulent.
- In **extremely fine grained soils**, such as colloidal clay, **the interstices are very small,** Darcy's law is **not valid.**

Range of the validity of Darcy's law:

- **Difficult to predict** exact range of validity of Darcy's law.
- Best method is to **conduct experiments** and **determine the actual relationship** between velocity v and hydraulic gradient i.
- For Darcy's law to be valid, this **relationship ie. v and i to be approximately linear**.
- For coarse grained sands, gravels and boulders, v and i relationship is **non-linear.**

Determination of k

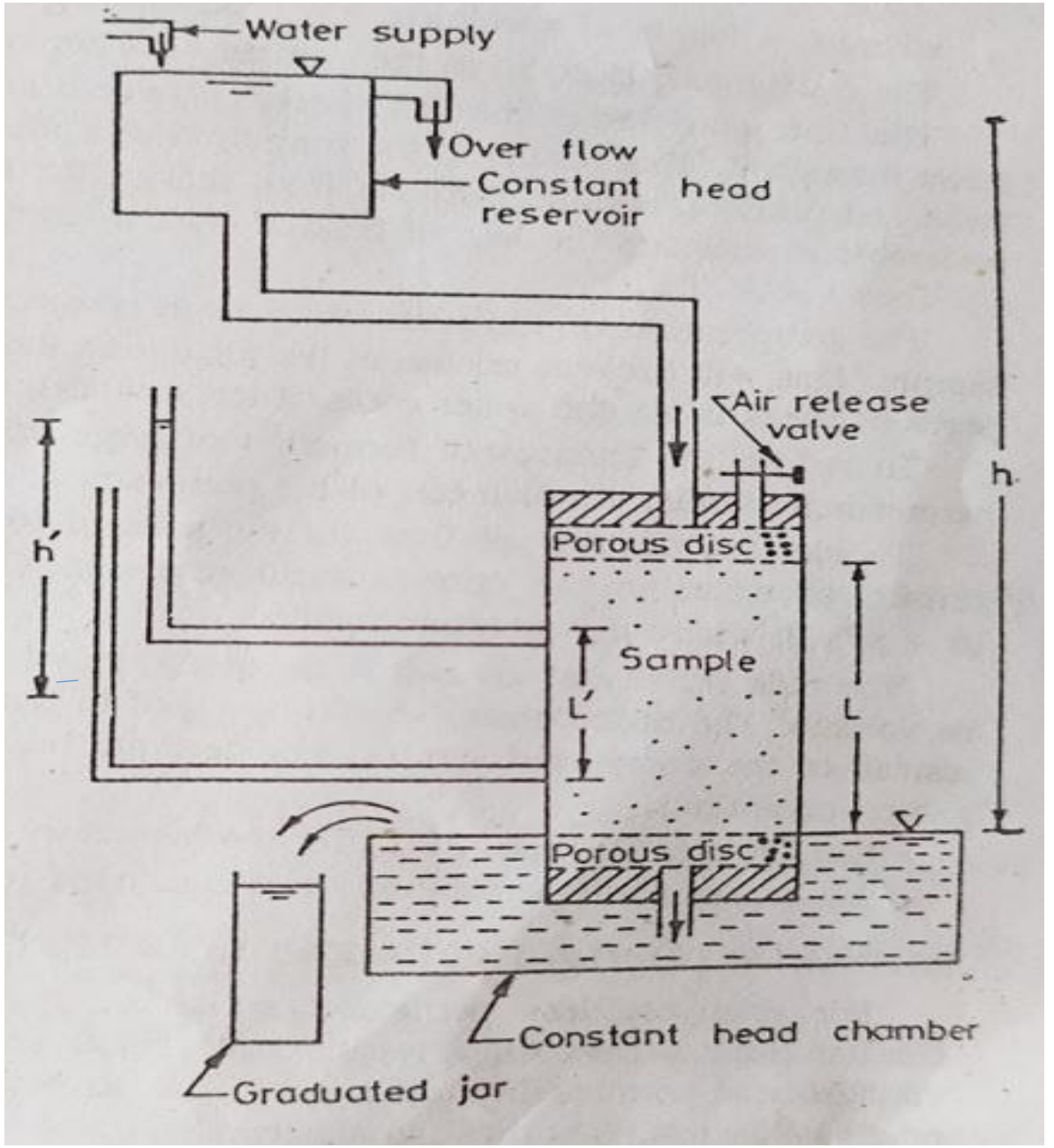
K of a soil sample can be determined by the following **Laboratory** methods:

- Constant Head Permeability Test
- Variable Permeability Test

Constant Head Permeability Test:

- Figure shows a schematic sketch.
- **Soil sample** is placed inside the mold between two porous disc.
- Soil sample is **saturated** and water is **allowed to flow out until a steady state is established.**
- Water level in the **constant-head reservoir** in which mold is placed is **kept constant**.

Constant Head Permeability Test (cont…):



- **Constant head chamber** is filled to the brim at the start of experiment.
- Water which **enters the chamber after flowing through the soil sample**, **spills over** the chamber and collected in a **graduated jar** over a period (time).
- **Head causing flow (h)** is equal to the difference in water levels between constant head reservoir and constant head chamber.
- A= x-sectional area of specimen A.
- Discharge, **q= kiA**

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

$$\mathbf{k}=\frac{\boldsymbol{qL}}{\boldsymbol{Ah}}$$

- **Finer particles of the soil specimen have a tendency to migrate** towards the end faces when water flows through it.
- For more accurate results, **i=** $\frac{h'}{L'}$
- **Suitable for clean sand and clean gravel**.

Clean sand (USCS):

- **Less than 5%** of fines
- **SW, SP**.

Clean Gravel

- Less than 5% Fines
- GW, GP

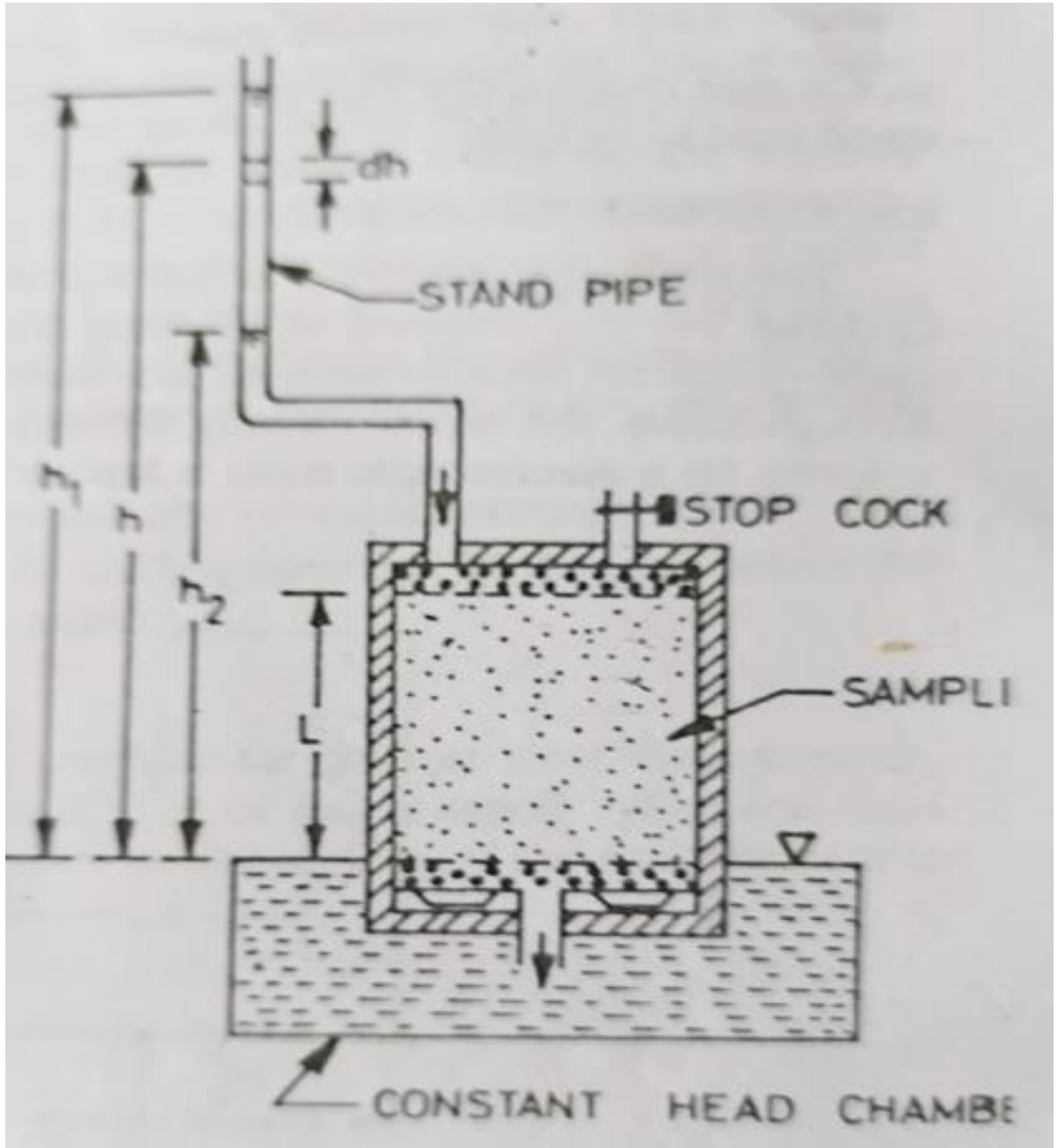
Variable Head Permeability Test:

- For **relatively less permeable soils**, **quantity water** collected in the graduated jar of the constant head permeability test is **very small** and **can not be measured** accurately.
- For such soils variable-head permeability test is used.

Test is started by **allowing water in the stand-pipe to flow through the sample** to the constant-head chamber from **which it over flows**.

- As water flows through the soil, the **water level in the stand-pipe falls**.
- Time required for the water level to fall from the known **initial head (h_1)** to a known **final head (h_2)** is determined.
- **Head is measured** with reference to the level of water in the constant head chamber.

Variable Head Permeability Test (cont…):



- Let us consider the **instant** (time), when **head is h**.
- For the infinitesimal **small time dt**, the **head falls dh**.
- Let the **discharge** through the sample be q.
- From the continuity of flow,

$$\mathbf{a.dh = - q.dt}$$

$$= -(\text{Ax k x i}) \text{ x dt}$$

$$= - \text{Ak} \frac{h}{L} \text{ x dt}$$

$$\frac{\boldsymbol{Ak\,dt}}{\boldsymbol{aL}} = -\frac{\boldsymbol{dh}}{\boldsymbol{h}}$$

Or, $\frac{Ak}{aL} \int_{t_1}^{t_2} dt = - \int_{h_1}^{h_2} \frac{dh}{h}$ or $\frac{Ak}{aL} (t_2 - t_1) = log_e \left(\frac{h_1}{h_2} \right)$

Or, $\mathbf{k = \frac{aL}{At} log_e \left(\frac{h_1}{h_2} \right) = \frac{2.30aL}{At} log_{10} \left(\frac{h_1}{h_2} \right)}$ where, $t_2 - t_1 = t$

- Rate of fall of water level in the stand pipe and rate of flow can be **adjusted** by **changing the area of the x-section of the stand pipe**.

- **Smaller diameter pipes are required for less pervious soils.**

- Variable permeameter is suitable for **fine sand and silt** with k = 10^{-2} to 10^{-5}mm/sec.

Ex. 1: A **constant head** permeability test has been performed with **3 standpipe tubes** (piezometers). The graduated cylinder collects **892ml** of water in **1120 seconds**. The dimensions are:

Soil sample **dia=1.8cm**

Elevation of water in **upper piezometer=181.0cm**

Elevation of water **in lower piezometer=161.0cm**

Piezometers **tips are spaced=16.7cm on center**

Compute the hydraulic gradient and coefficient of permeability of the soil sample.

Solution:

$$\mathbf{q} = \frac{\boldsymbol{Vol}}{\boldsymbol{t}} = \frac{892}{1120} = 0.796\ cm^3/\text{sec}$$

$$\mathbf{A} = \frac{\boldsymbol{\pi}}{\boldsymbol{4}}\boldsymbol{D^2} = \frac{\pi}{4}1.8^2 = 2.54\ cm^2$$

$$\mathbf{i} = \frac{\boldsymbol{\Delta l}}{\boldsymbol{\Delta h}} = \frac{161.6-181.0}{3x16.7} = 0.39$$

q=kiA

$$k = \frac{q}{iA} = \frac{0.796}{0.39x2.54} = 0.8\ \text{cm/sec}$$

Ex. 2: A constant head permeability test has been conducted on a **100mm diameter** 270mm tall **soil sample** in a permeameter. The upper and the lower reservoir elevation were 2010mm and 1671mm above the laboratory floor. The piezometers, whose **tips are spaced 200mm** apart, head readings of **1809mm** and **1578mm** and the graduated collected **910 ml of water** in 25 min 25 sec. Compute the permeability of the soil sample. Give your comments on the result.

Solution:

$$\mathbf{q} = \frac{\boldsymbol{Vol}}{\boldsymbol{t}} = \frac{910}{1525} = 0.597\ cm^3/\text{sec}$$

$$\mathbf{A} = \frac{\boldsymbol{\pi}}{\mathbf{4}}\boldsymbol{D^2} = \frac{\pi}{4}1^2 = 0.785\ cm^2$$

$$\mathbf{i} = \frac{\Delta \boldsymbol{l}}{\Delta \boldsymbol{h}} = \frac{1809-1578}{3x200} = 0.385$$

$$\mathbf{q = kiA}$$

$$k = \frac{q}{iA} = \frac{0.597}{0.385x0.785} = 1.98\ \text{cm/sec}$$

Gravel 1.0 -100 cm/sec

Fine sand: 0.01 – 0.001

Soil	*K* cm/sec	Degree of Permeability
Gravel	$> 10^{-1}$	Very High
Sandy gravel, clean sand, fine sand	$10^{-1} > k > 10^{-3}$	High to Medium
Sand, dirty sand, silty sand	$10^{-3} > k > 10^{-5}$	Low
Silt, silty clay	$10^{-5} > k > 10^{-7}$	Very Low
Clay	$< 10^{-7}$	Virtually Impermeable

Ex. 3: A **3.2m** thick silty sand stratum **intersect of a reservoir** shown in the figure. This stratum has a hydraulic conductivity of **$4x10^{-2}$cm/sec** and extends along the entire **1000m** length of the reservoir. An observation well has been installed in this strata as shown. Compute the **seepage loss** **per month** from the reservoir through the stratum

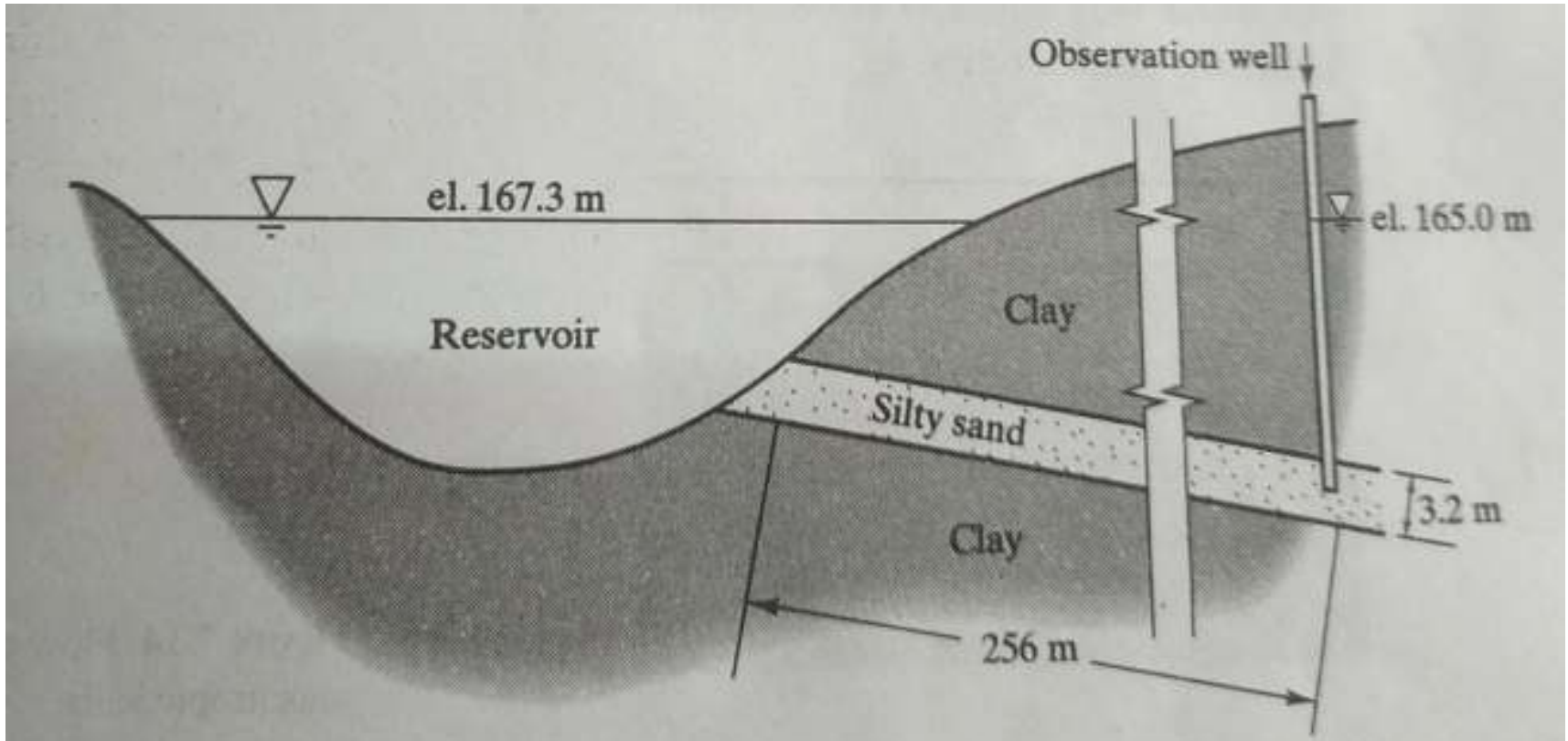


Figure 7.13 Cross-section through reservoir showing silty sand stratum. This cross-section is oriented parallel to the direction of flow.

Solution:

Observation well indicates a WL above the top of silty stratum, so it is an **confined aquifer**.

$$\mathbf{i}=\frac{\Delta h}{L}=\frac{167.3-165.0}{256}=0.0090$$

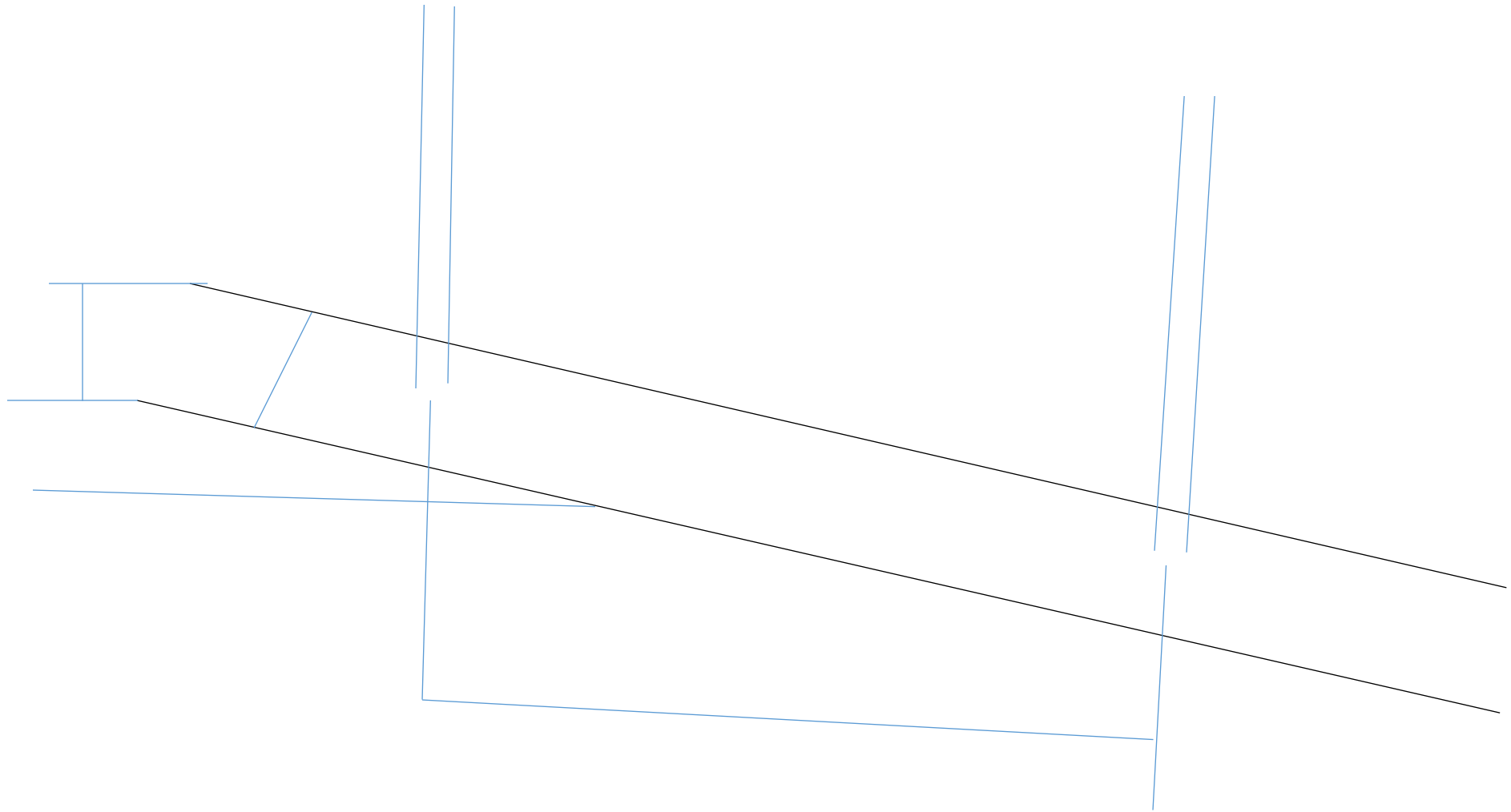
$$\mathbf{A=3.2x1000}=3200\ m^2$$

$$\mathbf{K=4x10^{-2}cm/sec}=4\text{x}\ \frac{3600x24x30}{100x100}=1036.8\ \text{m/month}$$

Q=kiA

$= 1036.8\text{x}0.0090\text{x}3200$

$=30{,}000\ m^3/\text{month}$



Thank You

EWCE 341: Principles of Soil Mechanics

Seepage Analysis

What we are going to discuss today:

- What is seepage?
- Importance of Seepage Flow
- Flow line, Equipotential line, Flow net, Flow channel
- Study from the flow net
- Basic Flow net construction –Vertical Flow
- Properties of Flow net
- Example of flow net below sheet pile

What is Seepage?

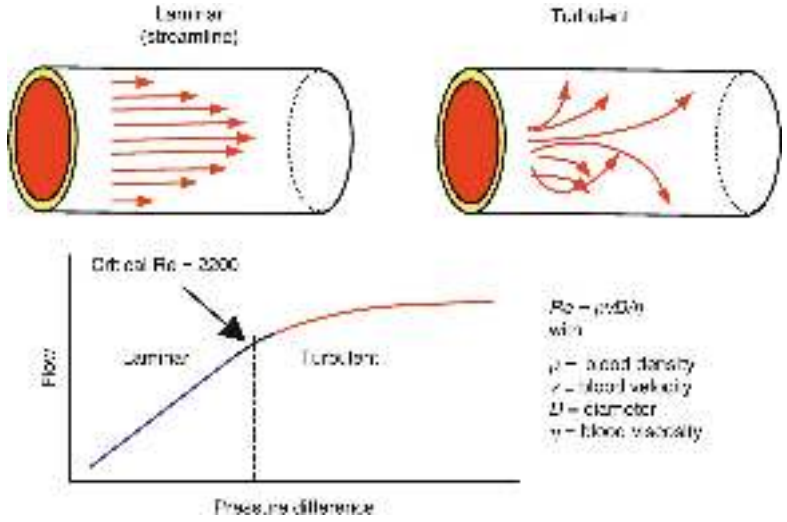
Seepage Analysis

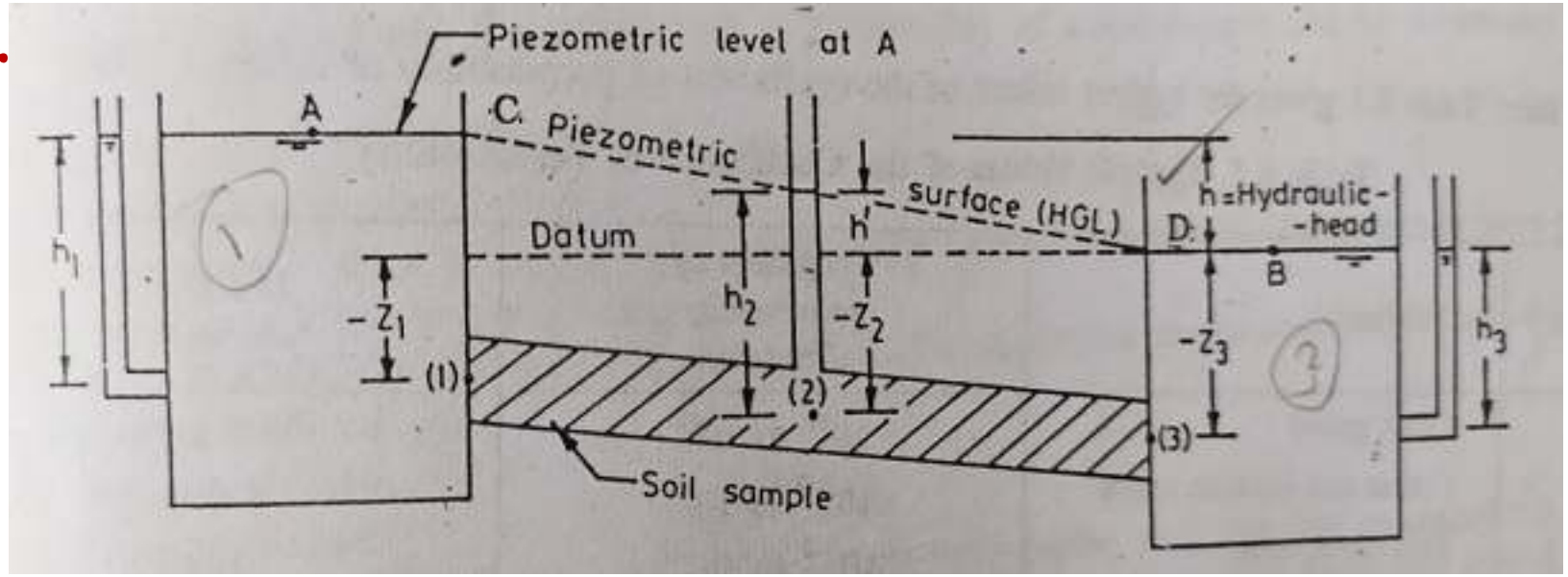
What is seepage?

- Seepage is the **flow (slow escape) of water under gravitational forces** in **permeable medium (soil).**

- **Flow of water takes place** from a point of **high head** to a point of **low head**.

- Flow is generally **laminar**



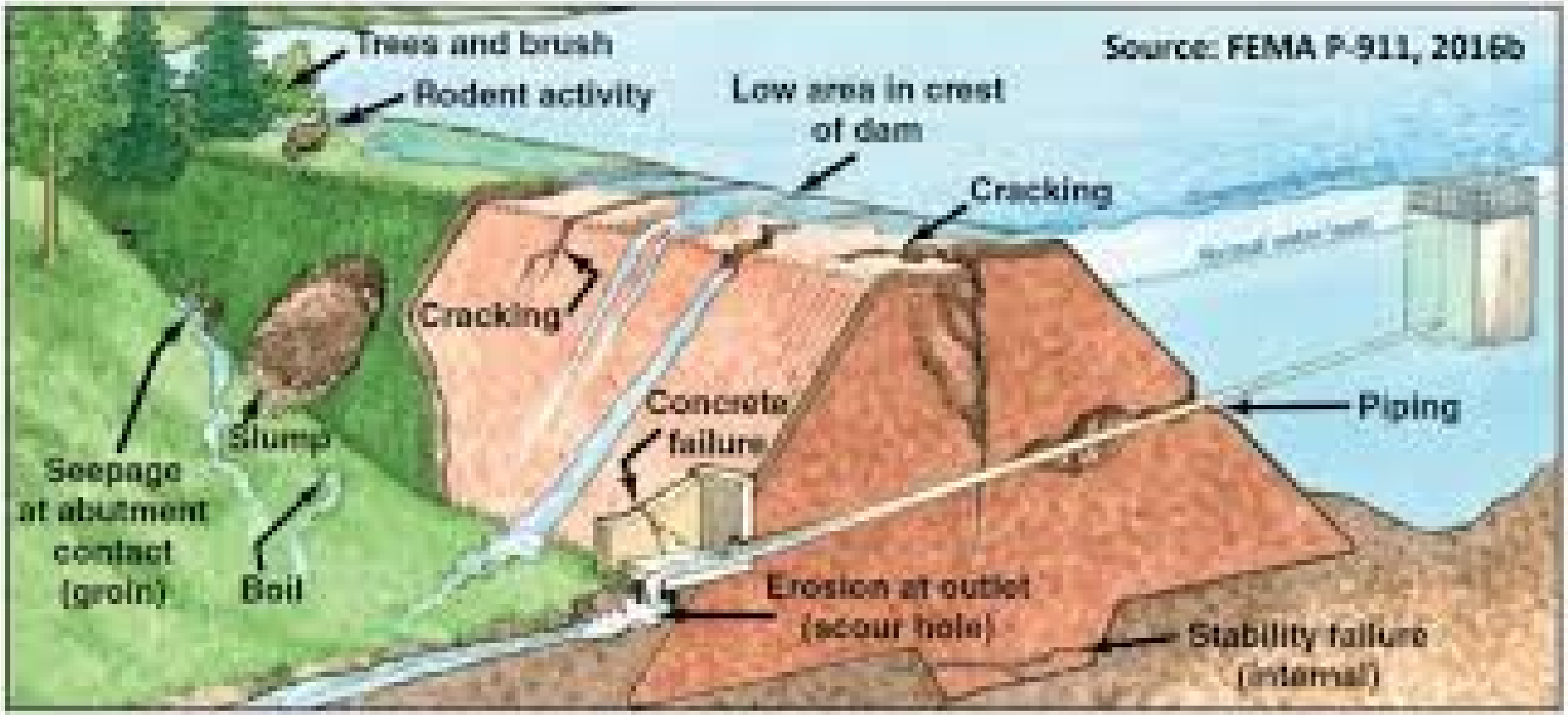


Importance of Seepage Flow:

Importance of Seepage Flow:

- **Interaction between soils and percolating water** has an important influence on:
  -**Design** of foundations and earth slopes.
  -**Quantity of water** that will be lost by percolation through a dam or subsoil.

- **Foundation failures** due to ***piping*** is quit common.



- **In stability of slopes**, the **seepage force** is an important factor.
- **Shear strength of soils get reduced** due to the development of neutral stress or pressure.

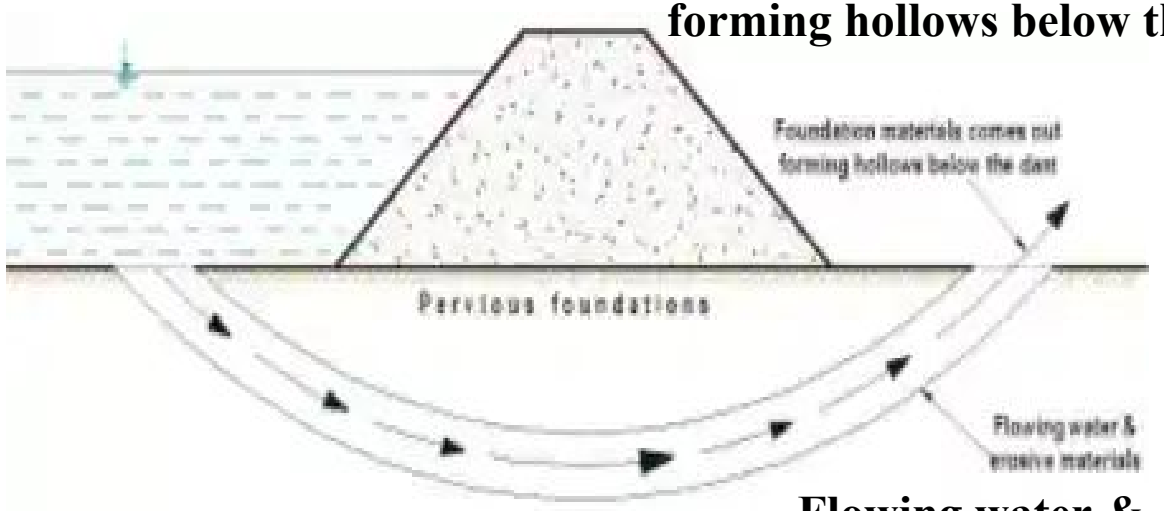
A detailed understanding of **hydraulic conditions** is therefore essential for the satisfactory design of structures.

Piping

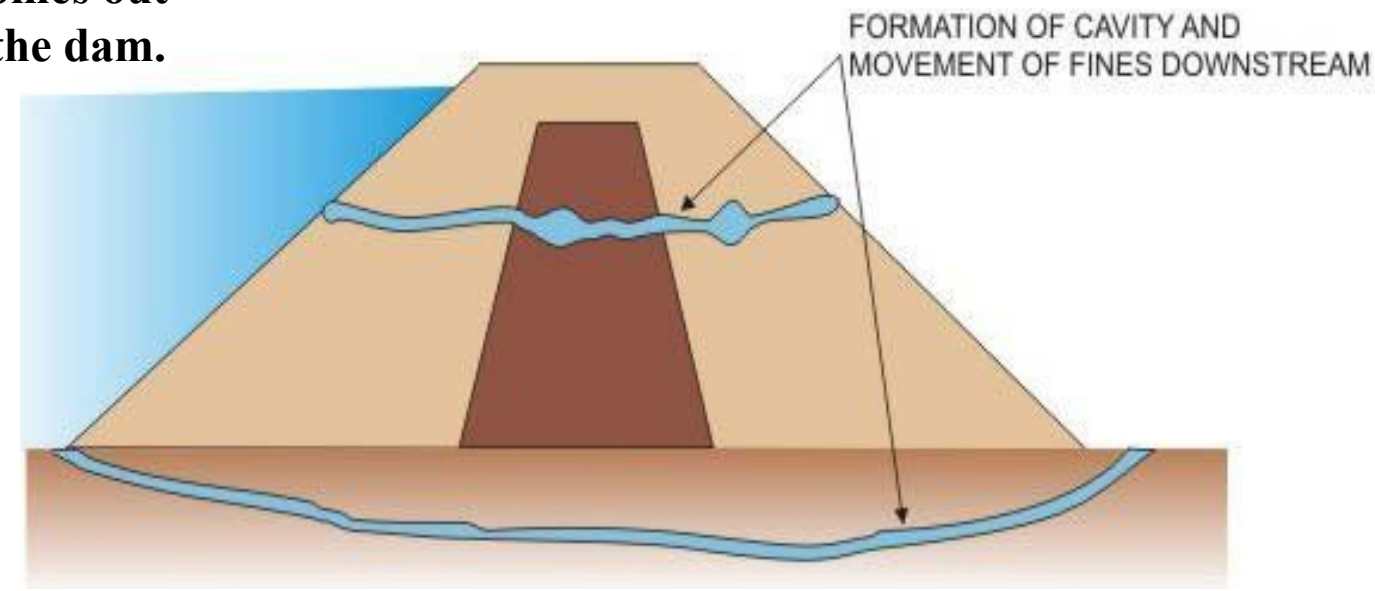
What is piping?

- Progressive development of **internal erosion** by **seepage**, **appearing downstream as a hole discharging water**.
- **Induced by regressive erosion of particles** from downstream and along the upstream line **towards an outside environment** until a continuous pipe is formed.
- **Internal erosion** is the **formation of voids within a soil** caused by the removal of material by seepage.

Foundation materials comes out forming hollows below the dam.



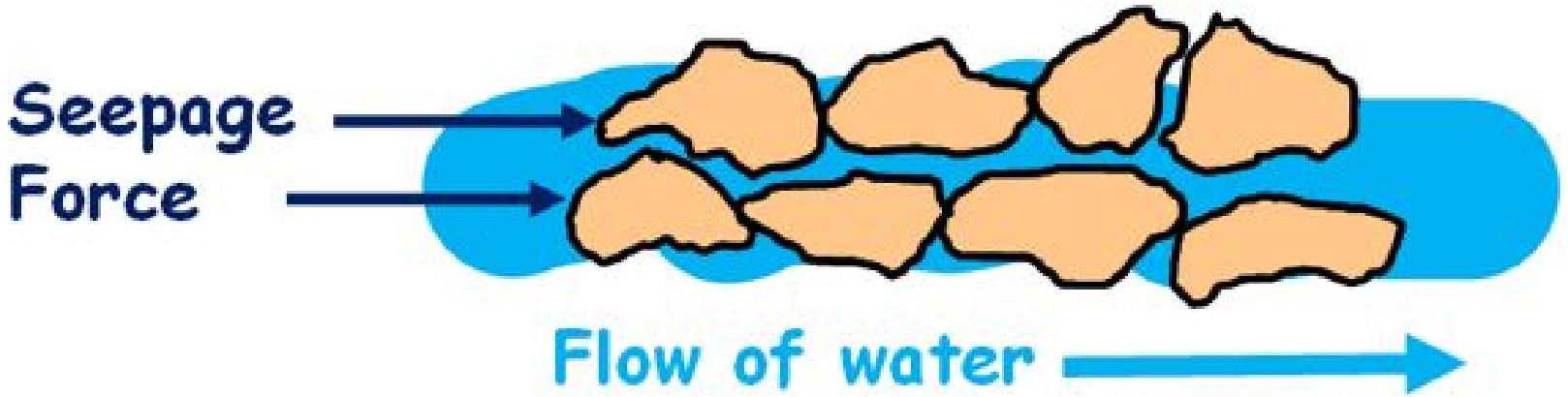
Flowing water & Carries materials



Seepage: Phenomenon of flow of water or movement of water through the **soil** is called **seepage.**

Seepage Force: **Force** applied by the water on the **soil** particles is called the **seepage force.**

Seepage pressure: Because of Force, pressure developed in the **soil** is termed as **seepage** pressure

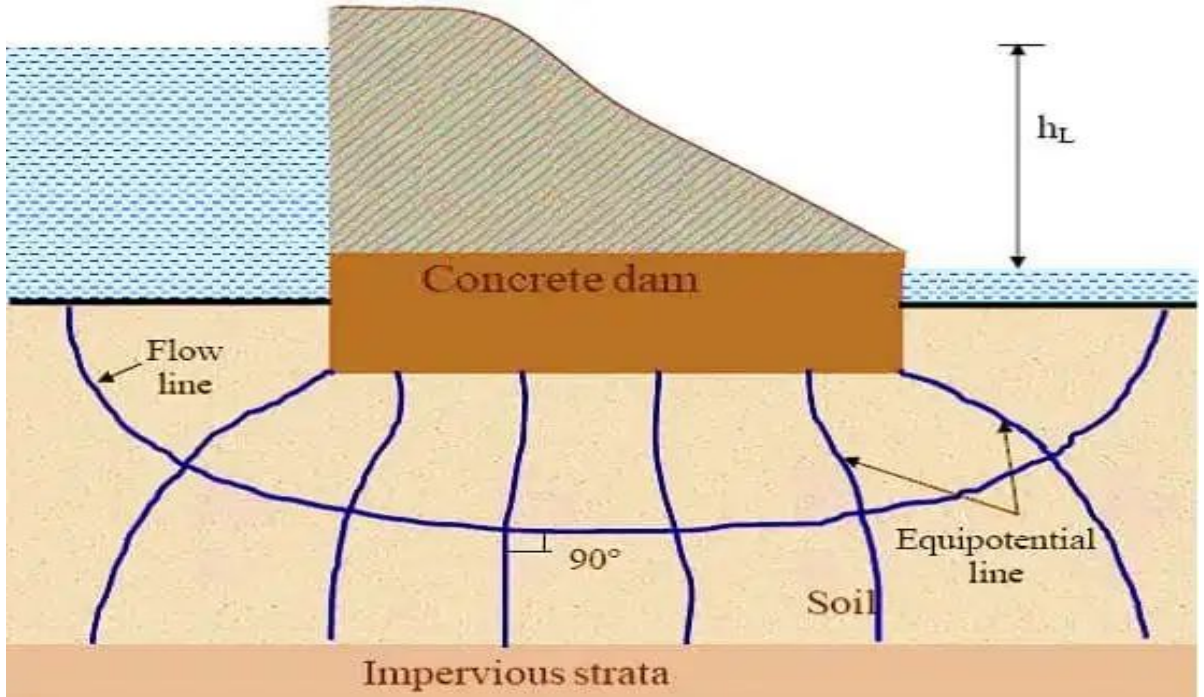


Flow line, Equipotential line, **Flow net**, Flow channel/tube, **Flow element/field**

Flow Line:

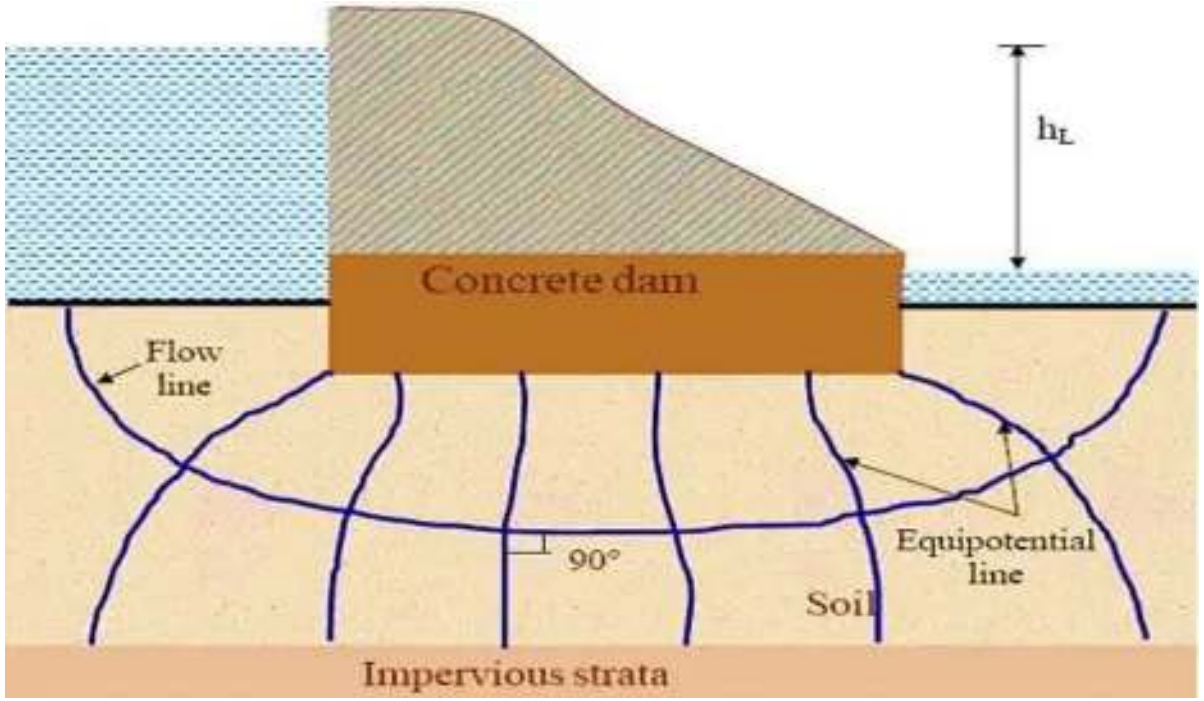
- **Path** which **a particle of water follows** in its course of seepage **through a saturated soil mass** is called a **flow line**.

- Although **an infinite number of flow lines** can be drawn, but for convenience, only a few are drawn.
- **At certain points on different flow lines,** the **total head** will be the same.



Equipotential Line:

- Lines connecting **points of equal head** can be drawn.
- At all points along an equipotential line, water would rise in piezometric tube to a certain elevation known as piezometric level.
- As flow always take place along the steepest hydraulic gradient, **equipotential lines cross flow lines at right angle**.



Flow net:

- **Flow lines** and **equipotential lines** **form a flow net.**
- Gives **pictorial representation** of the **path taken** by water particles and **head variation** along that path.
- Network of flow lines and equipotential lines **intersecting at right angles** to each other.

Path which a particle of water follows in its course of seepage through a saturated soil mass is called a **flow line**.

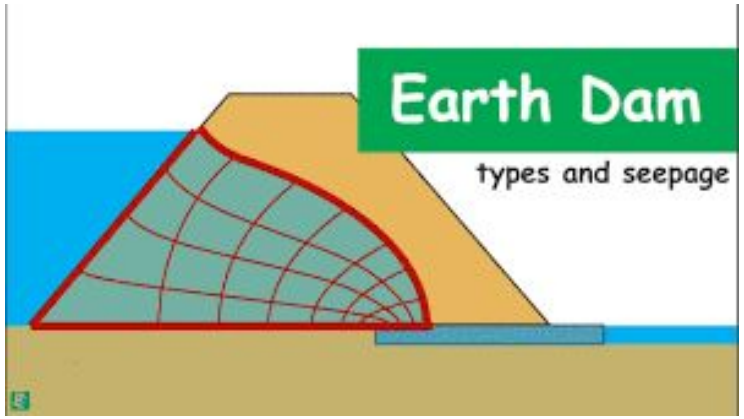
Equipotential lines are lines that intersect the flow lines at **right angles**.

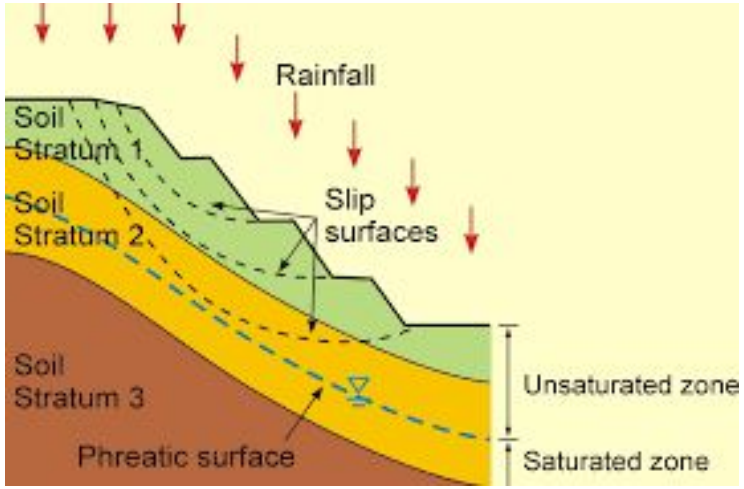
At all points along an equipotential line, the water would rise in piezometric tubes to the same elevation known as the piezometric head .

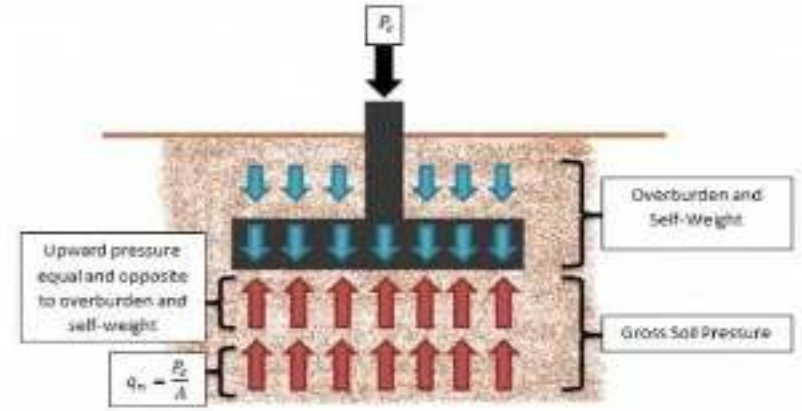
Requirements of Studying Flow net

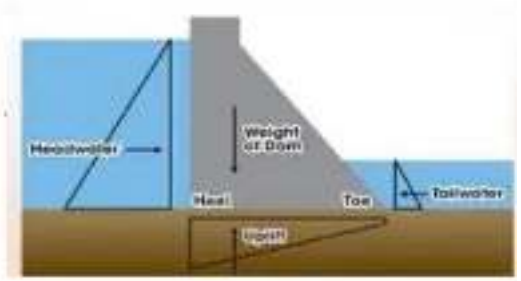
Following can be studied by the construction of Flow Net:

- Computation of **seepage lost** under or through a dam,
- **Estimate of Uplift pressures** caused by the water on the base of a concrete dam and
- **Study of the Effect of seepage** on the stability of earth slopes





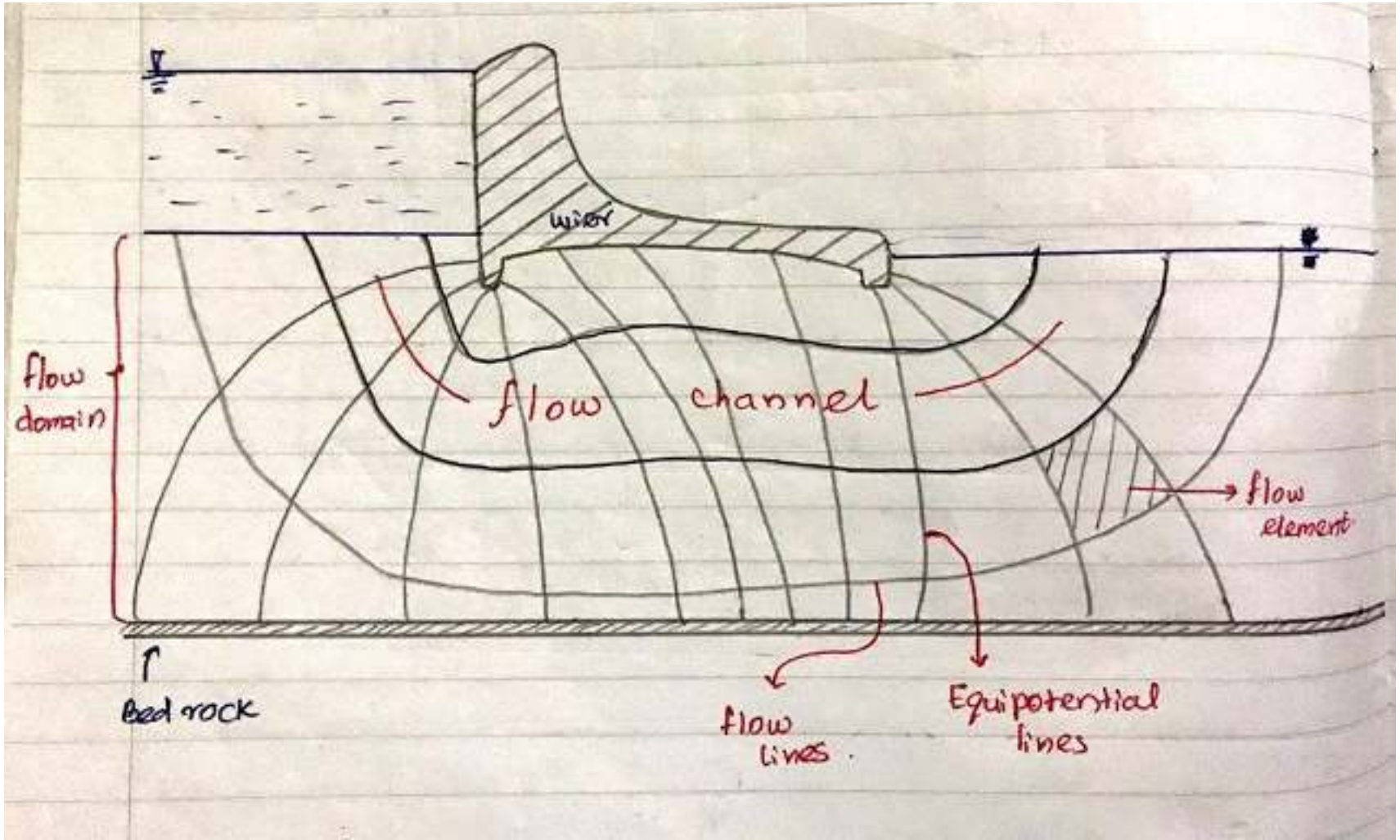




Uplift Pressure- Effects, and Prevention Strategies

Flow Channel:

- A **flow channel** is a **space between two flow lines**.
- **Head lost** is the same between two adjacent potential lines and independent of the permeability and the head causing **flow.**



Weir:

- Basically **an obstruction in the flow path** in open channel.
- Will cause an **increase in water depth** as water flow over the weir.
- The greater the flow rate, the greater will be the increase the depth of flow.

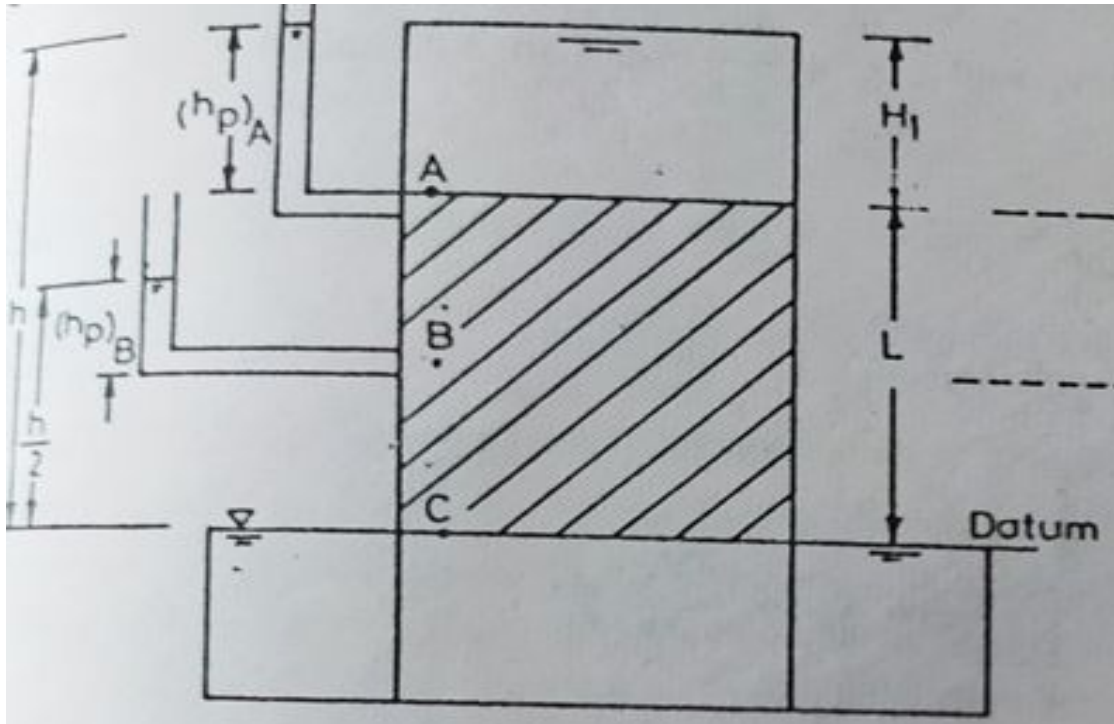


Basic Flow Net Construction -Vertical Flow through soil:

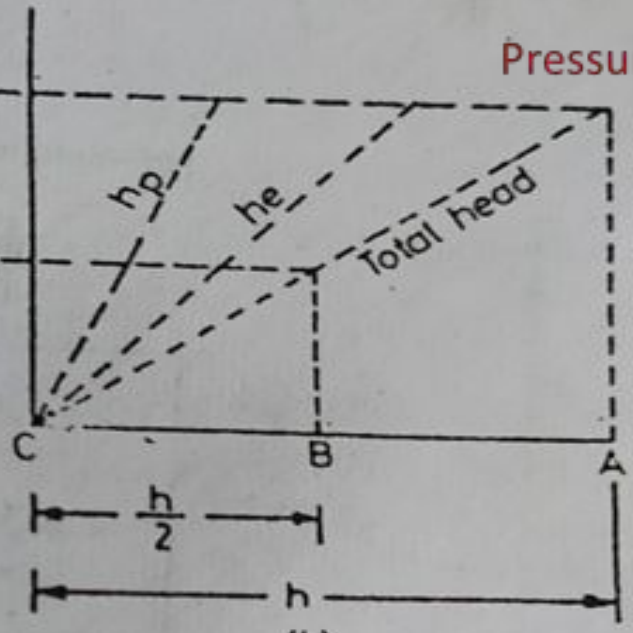
- Figure (a) shows that a **glass cylinder** containing soil sample of length L.
- A **steady flow occurs** vertically downward through the soil sample under a **head h.**
- **Elevation head, pressure head** and **total head** at points at A, B and C can be worked out as shown in Figure (b) and Figure (c).

Elevation Head h_e, *from datum*

Pressure Head h_p, *from piezometer*



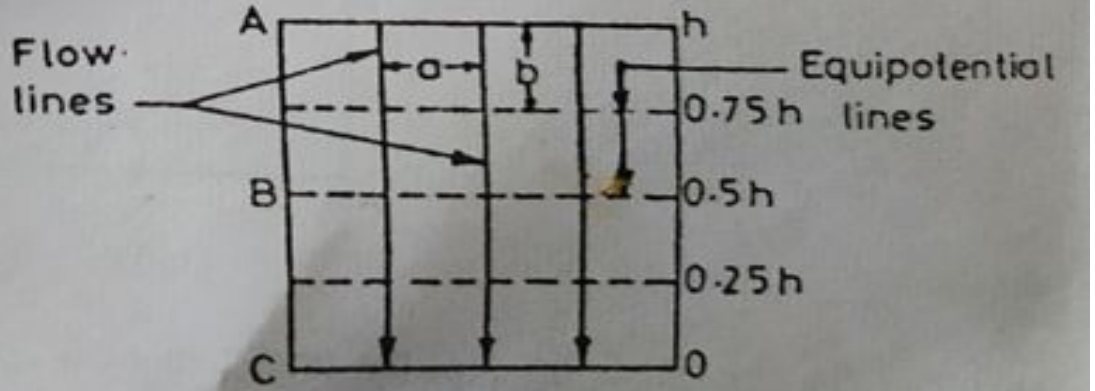
(a)



(b)

Point	Elevation head (he)	Pressure head (hp)	Total head (H)
A	L	$h_{pA}=H_1$	$L+H_1=h$
B	0.5 L	$0.5L+H_1-0.5h$ $h_{pB}=0.5h-0.5L$	$L+H_1-0.5h$ $=0.5h$
C	0	$h_{pC}=0$	0

(c)



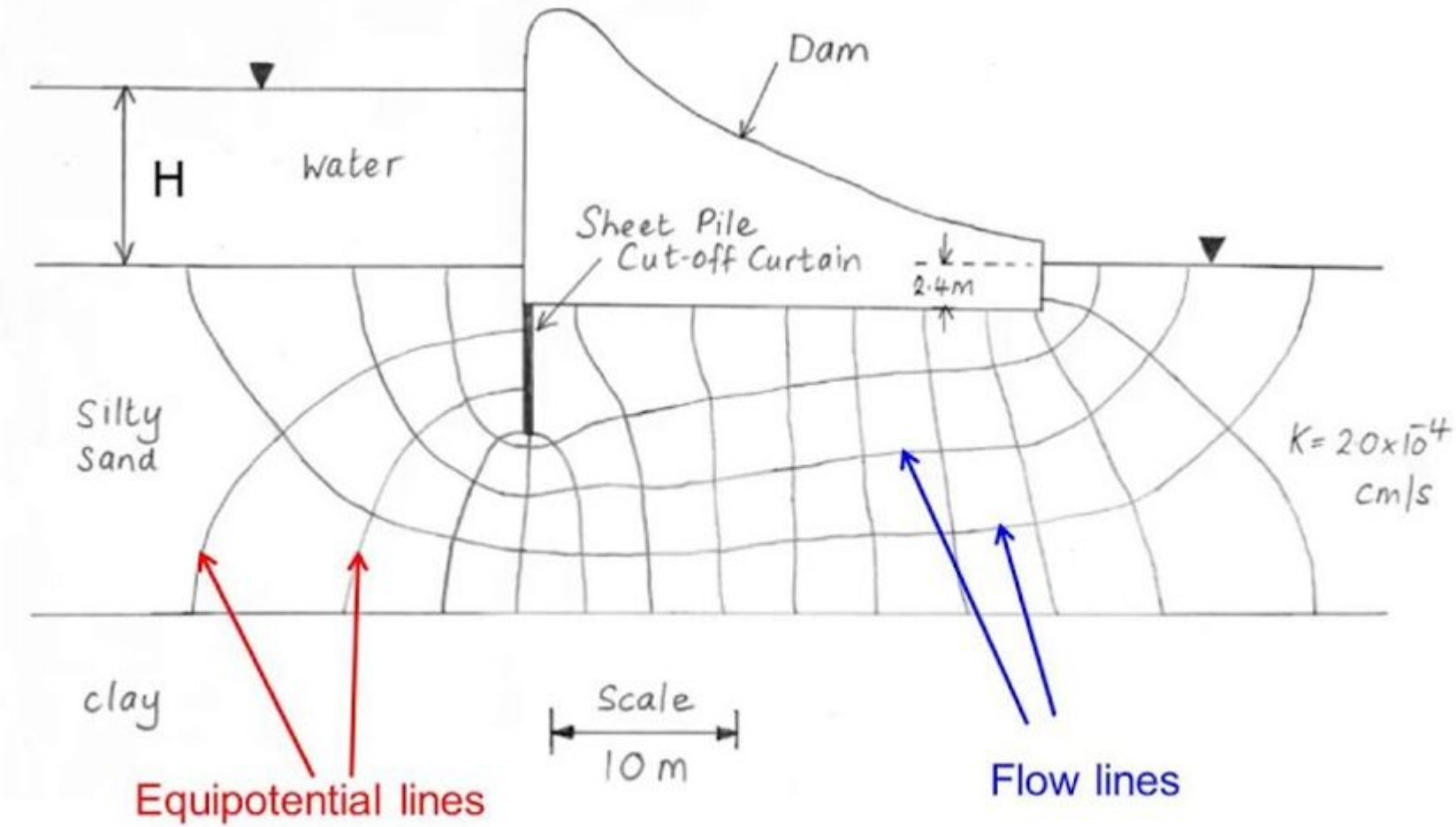
Flow net

(d)

0.5L + (h+L)-0.5h

- **Point B** is at a height of **0.5L above the datum**.
- As the rate of loss of head is linear, the loss of head up to point B is $\frac{h}{2}$ and **total head at point B is also $\frac{h}{2}$.**
- **Figure (d)** shows a simple flow net, in which **5 flow lines** and an **equal number of equipotential lines** are drawn.
- **Equipotential lines are horizontal** and **flow lines are vertical** in this case.
- If a **dye is inserted** at a few points on the top of the soil sample, **paths taken by the dye represent the flow lines.**
- But flow nets in actual soil engineering problems are not as simple as shown.

Flow Net Construction

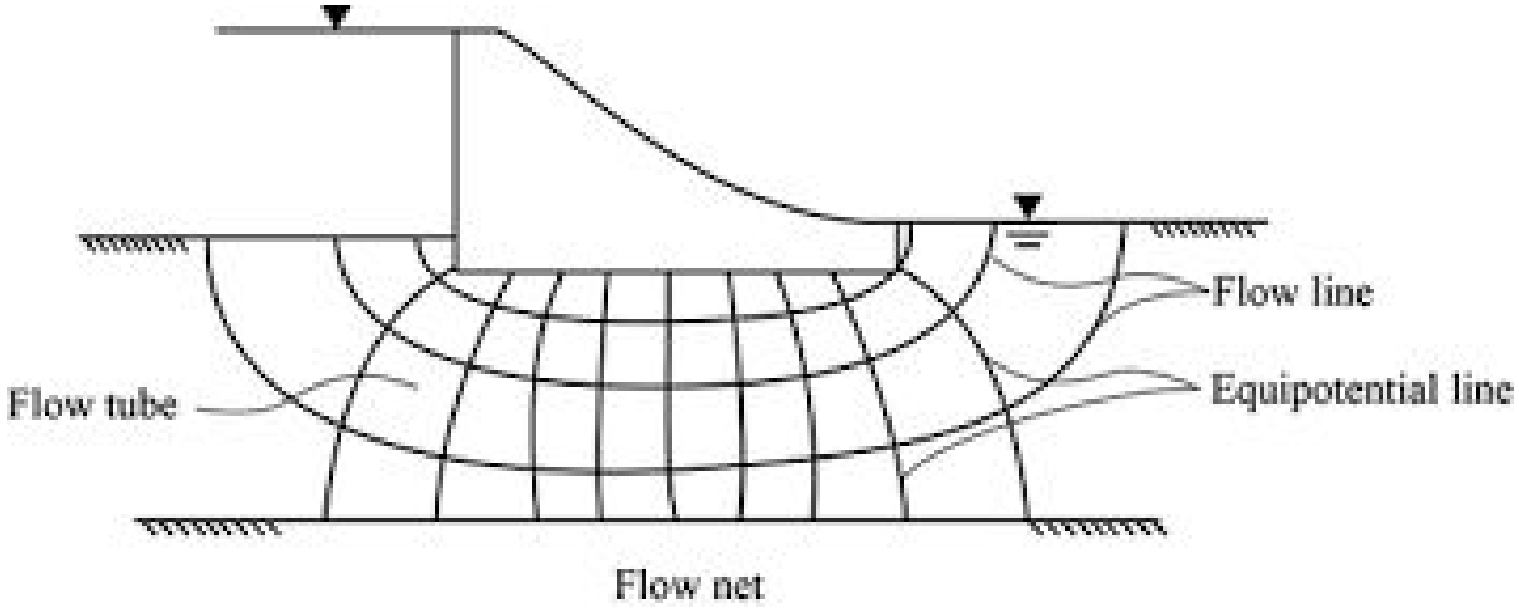


Properties of Flow Net

Flow Net Construction:

Properties of a flow net:

- Flow and equipotential lines are **smooth curves.**
- Flow lines and equipotential lines meet **at right angles** to each other.
- No two flow lines **cross** each other.
- No two flow or equipotential lines **start** from the same point.

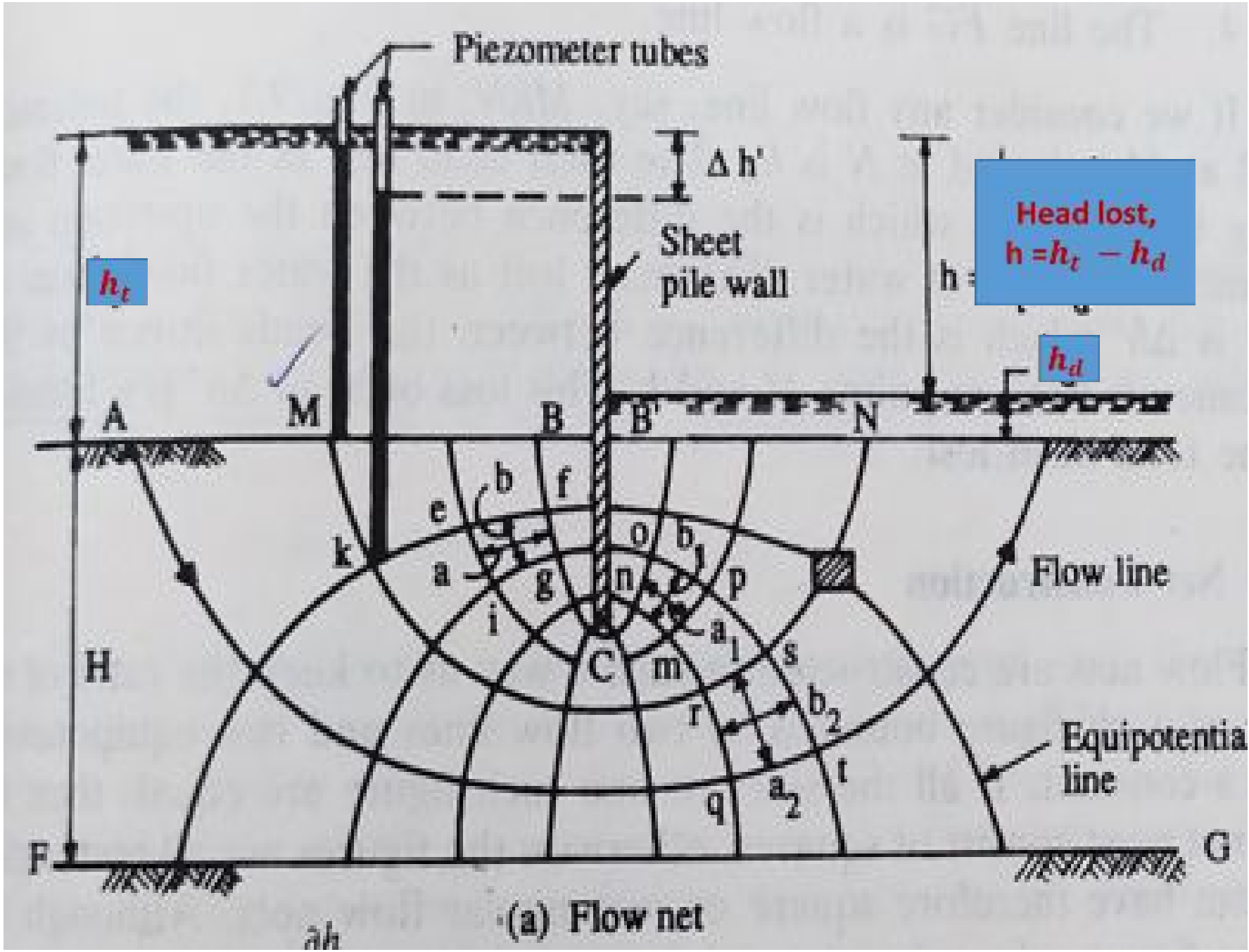


Flow net

Example of a Flow net for the flow below a sheet pile.

Figure: A typical example of a Flow net for the flow below a sheet pile.

- Head of water on the up-stream side of the sheet pile is h_t.
- Head on the downstream side is h_d.
- **Head lost** as water flows from up steam to down stream Side is h.



What have discussed today?

- What is seepage?
- Importance of Seepage Flow
- Flow line, Equipotential line, Flow net, Flow channel
- Study from the flow net
- Basic Flow net construction –Vertical Flow
-Properties of flow net
-Example of flow net below sheet pile.

Thank You

EWCE 341: Seepage Analysis

What learned in the last class?

- What is seepage?
- Importance of Seepage Flow
- Flow line, Equipotential line, Flow net, Flow channel
- Study from the flow net
- Basic Flow net construction –Vertical Flow

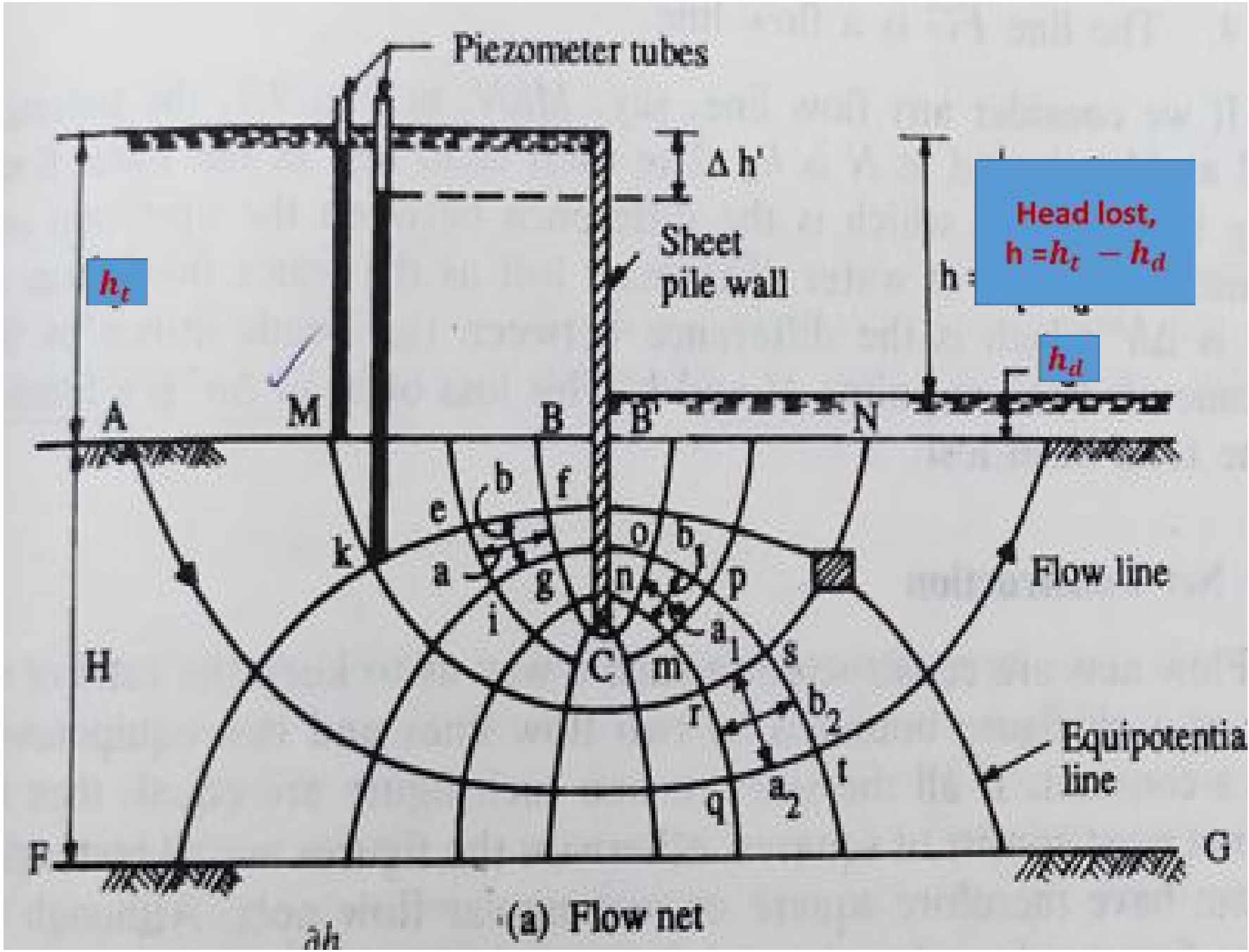
What we are going to learn today:

- Boundary conditions.
- Elements to Construct Flow Net.
- Methods for the construction of flow nets

Example of a Flow net for the flow below a sheet pile.

Figure: A typical example of a Flow net for the flow below a sheet pile.

- Head of water on the up-stream side of the sheet pile is h_t.
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Boundary conditions: Consists of statements of head or flow conditions at all boundary points.

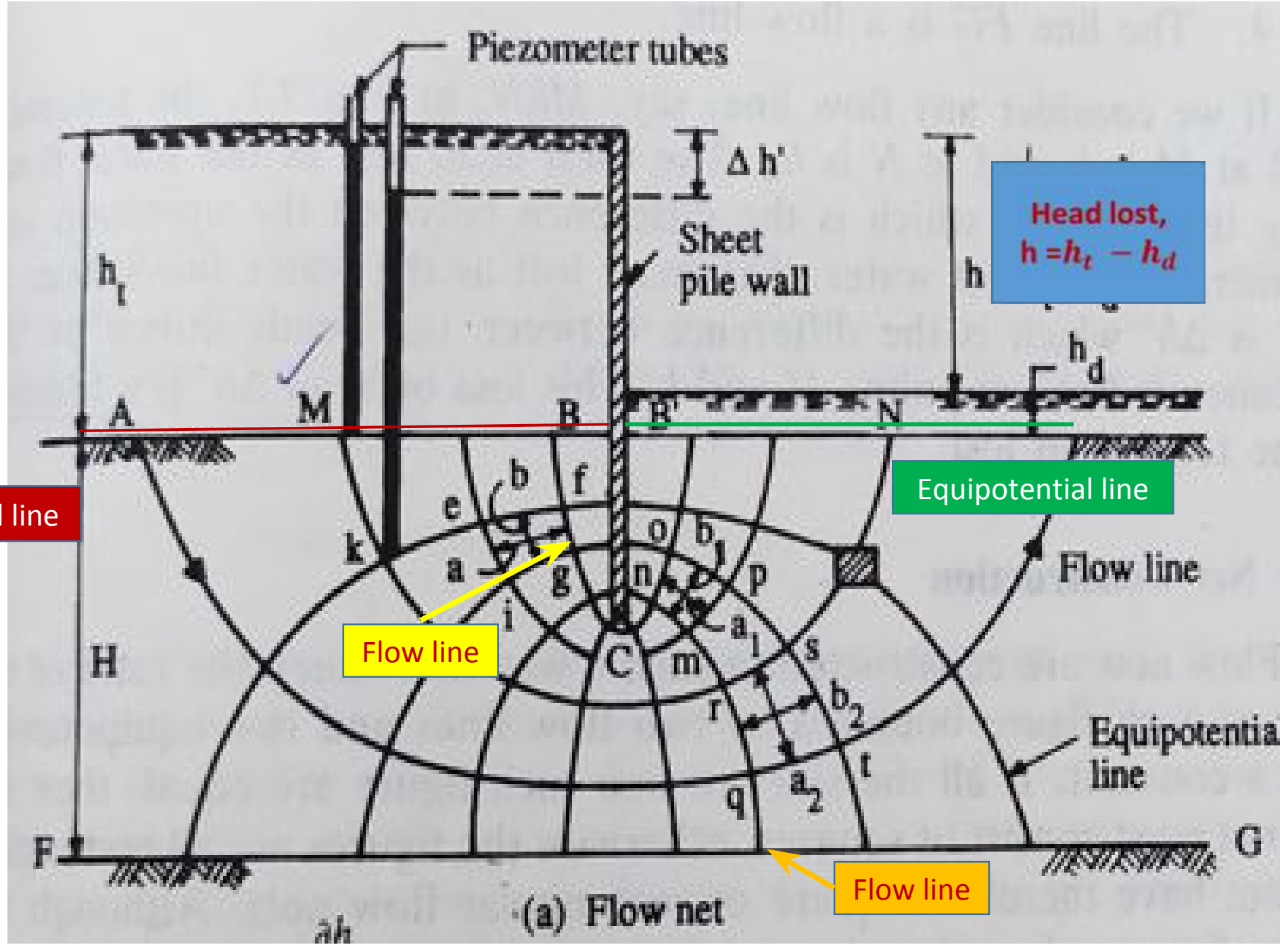
- Assume that the **flow is two dimensional**, ie. all flow lines are parallel to the plane of the figure.
- All flows are in the **steady state case**.

Steady-state flow:

- Refers to the condition where the **fluid properties** at any single point in the system **do not change over time**.
- Fluid properties include **temperature, pressure, and velocity**.
- One of the most significant properties that **system mass flow rate is constant.**

Figure: A typical example of a Flow net for the flow below a sheet pile.

- Head of water on the up-stream side of the sheet pile is h_t.
- Head on the downstream side is h_d.
- Head lost as water flows from up steam to down stream side is h.



Boundary Conditions- Generally in four in number:

- Line **AB is an equipotential line** along which the **head is h_t**.
- **Line BCB'** along the sheet pile wall is a **flow line**.
- Line **B'N is an equipotential** line along which the head is equal to h_d.
- Line **FG is a flow line**.

Elements to Construct Flow Net:

Head Lost:

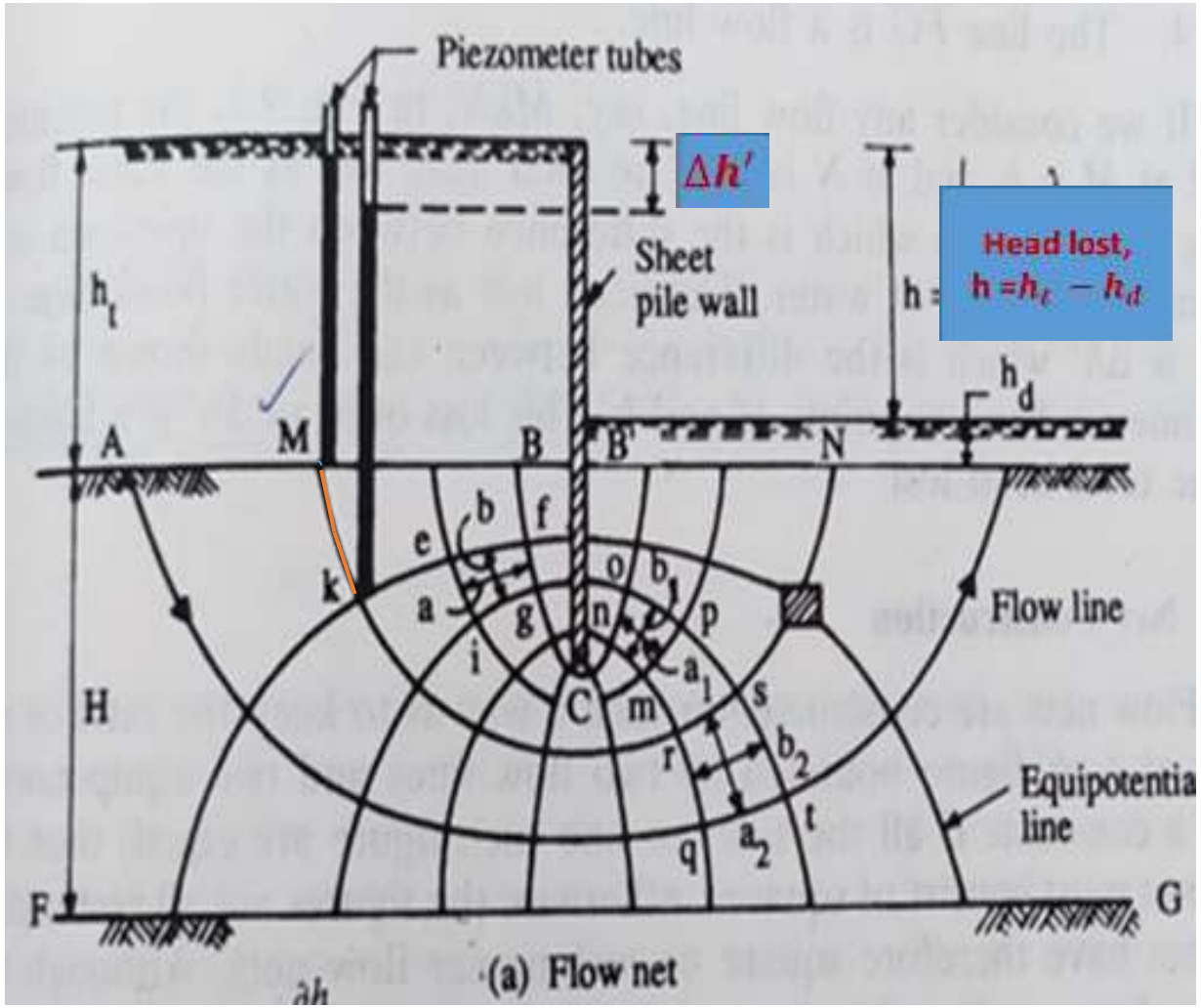
- If we consider any **flow line** **say**, MkN, **potential head** at M is h_t and at N is h_d.
- **Total head lost** as water flows from

M to N is h= h_t - h_d.

- **Head lost** as water flows from

M to k is $\Delta h'$, difference between head shown by piezometers fixed at points at **M and k.**

- **Loss of head $\Delta h'$** is a **fraction of the total head lost.**



Ratio of Sides of Flow Elements/Fields:

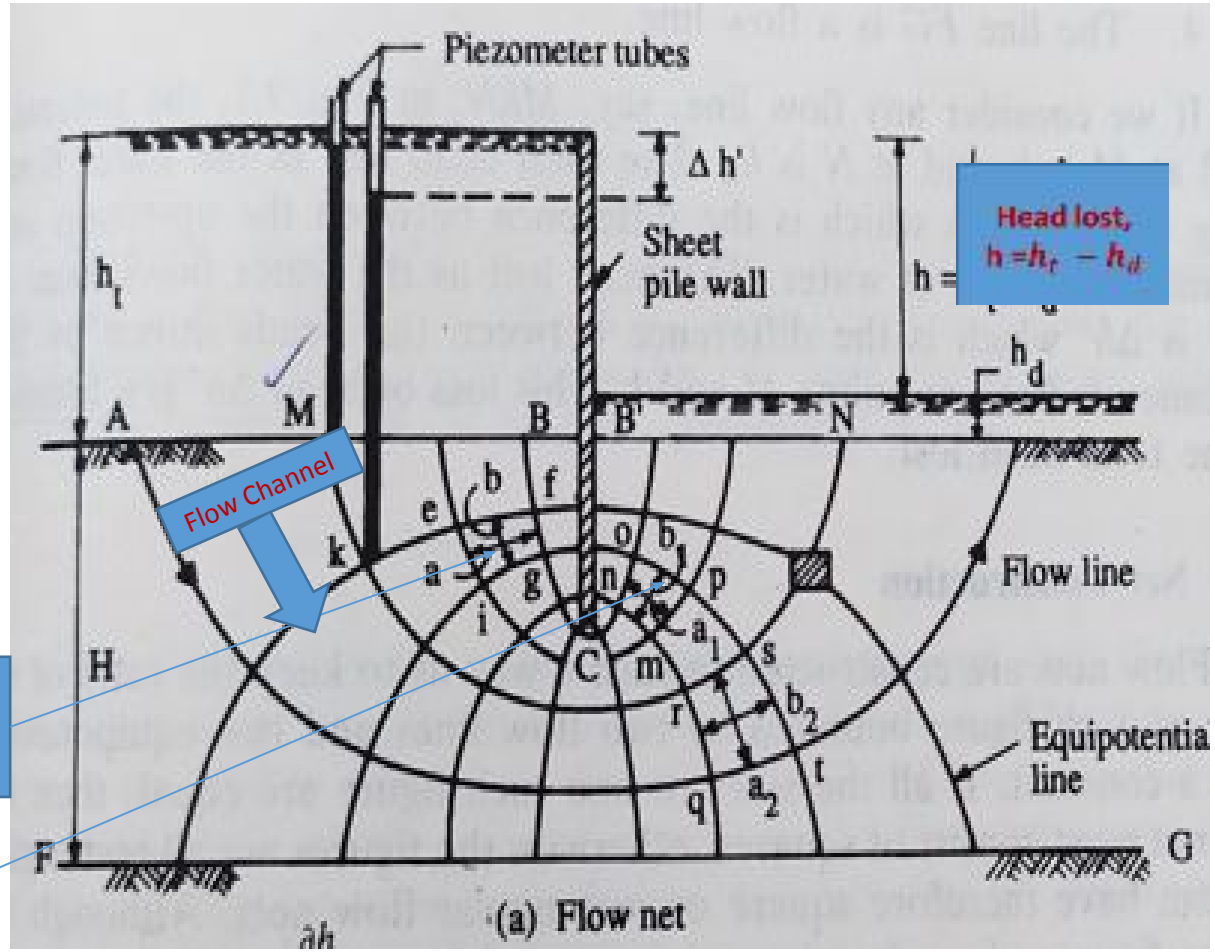
- Flow nets are constructed in such a way as to keep the **ratio of sides of each figure** bounded by two flow lines and two equipotential lines, **a constant ie.** ($\frac{a}{b}$=constant).

- If all the **sides of one such figure/field are equal**, then the flow net must consist of **square**, otherwise **rectangles**.

- **Average width of the square are equal**

.

- In the figure, width 'a' of **efgi** is equal to its length 'b'.



Flow element: efgi

Flow Element: nopm

Flow channel:

- **Area bounded by any two neighboring flow lines** is called as a flow channel.

If the flow net is constructed, in such a way that **ration a/b remains the same** for all figures/flow elements/fields, then it can be shown that there is the **same quantity of seepage** in each flow channel.

- Consider two figures/flow element/field, ***efgi* and *nopm* in one flow channel**.
- Figure ***rstq* is chosen** in such a way that it lines within the **same equipotential line** that bound the figure ***nopm***.

- Darcy's law for the **discharge through** any Flow element/fields/figure such as **efgi per unit length of the section** may be written as,

  $\Delta q = kiA = k\frac{\Delta h}{b}a = k\Delta h\ \frac{a}{b}$, where, Δh=head loss in crossing the Flow Element/Field, **efgi** .

- For three Flow Element/Fields, expression in this form are:

$$\Delta q = k \Delta h \frac{a}{b}, \qquad \Delta q_1 = k \Delta h_1 \frac{a_1}{b_1}, \Delta q_2 = k \Delta h_2 \frac{a_2}{b_2}$$

- Value of **k remains same** for all three figures. If the Flow Elements/Fields are **all squares** then,

$$\frac{a}{b} = \frac{a_1}{b_1} = \frac{a_2}{b_2} = 1$$

- Since the Flow Element/Fields **efgi and nopm are in the same flow channel**, then $\Delta q = \Delta q_1$
- Since **nopm and rstq** are **within the same equipotential lines**, then, $\Delta h_1 = \Delta h_2$
- We get the following relationship, $\Delta q = \Delta q_2$ and $\Delta h = \Delta h_2$.

Here **proves** that the **same quantity flows** through each Flow Element/Field and there is **same head drop/lost** in crossing each Flow Element/Fields, **if all the Flow Elements/Fields are squares or posses the same ratio a/b.** (Question)

- Flow nets can be rectangular or square. But square flow nets are generally used in practice since this is easier to construct.

Methods for the construction of flow nets: Important methods,

- Analytical method
- Electrical analog method
- Scaled modeled method
- **Graphical method**.

Graphical method:

Most commonly used method.

- **Boundary conditions** to be identified and established.
- **Reasonably good flow net** can be drawn even **by a novice but require practice**.
- Accuracy of **hydraulic quantities** such as discharge, pore water pressure does **not depend on the exactness of flow net.**

Following Points should be kept in mind while sketching the flow net:

- **Many flow channels** distract the attention from the essential features. **Normally three to four channels are sufficient.**
- Appearance **of entire flow net to be watched** and not that part of it. Small details can be adjusted after the entire flow net has been roughly drawn.
- **Curve** should be roughly **elliptical or parabolic** in shape.
- **All transitions** should be **smooth**.
- Flow lines and equipotential lines **should be orthogonal** and **form approximate squares.**
- **Size of the squares** in a **flow channel should be changed** gradually from upstream to the down stream.

Procedures for drawing the flow net:

Step-1:

- Identify the hydraulic **boundary condition**:

-Upstream bed level GDAK represent 100% **potential line** and down stream bed level CFJ, 0% **potential line**.

-First **flow line KLM**, hugs the hydraulic structures and KLM is formed by the flow of water on the upstream of the sheet pile, the downstream sheet pile and at the interface of the base of the dam and soil surface.

-Last **flow line** is indicated by the impervious stratum NP.

Step-2:

- Draw a **trial flow line ABC**, adjacent the boundary line. Line must be at right angles to the upstream and down stream beds.

Location of first trial line is **determined from experience**

Step-3:

- **Starting from the up stream end**, **divide the first flow channel** into approximate squares **by the equipotential lines**. **Size of the square should change gradually**.
- Some of the squares may be quite irregular. Such squares are called singular squares.

Step-4:

- **Extend downward the equipotential lines** forming the sides of the squares. These extensions, point out approximates **width of the squares**, such as squares marked (1) and (2).
- **Other sides of the square are set equal to the widths** as determined above.
- Irregularities are smoothened out and **next flow line DF** is drawn joining these bases. While sketching the flow line, care should be taken to make flow fields as approximate squares through out.

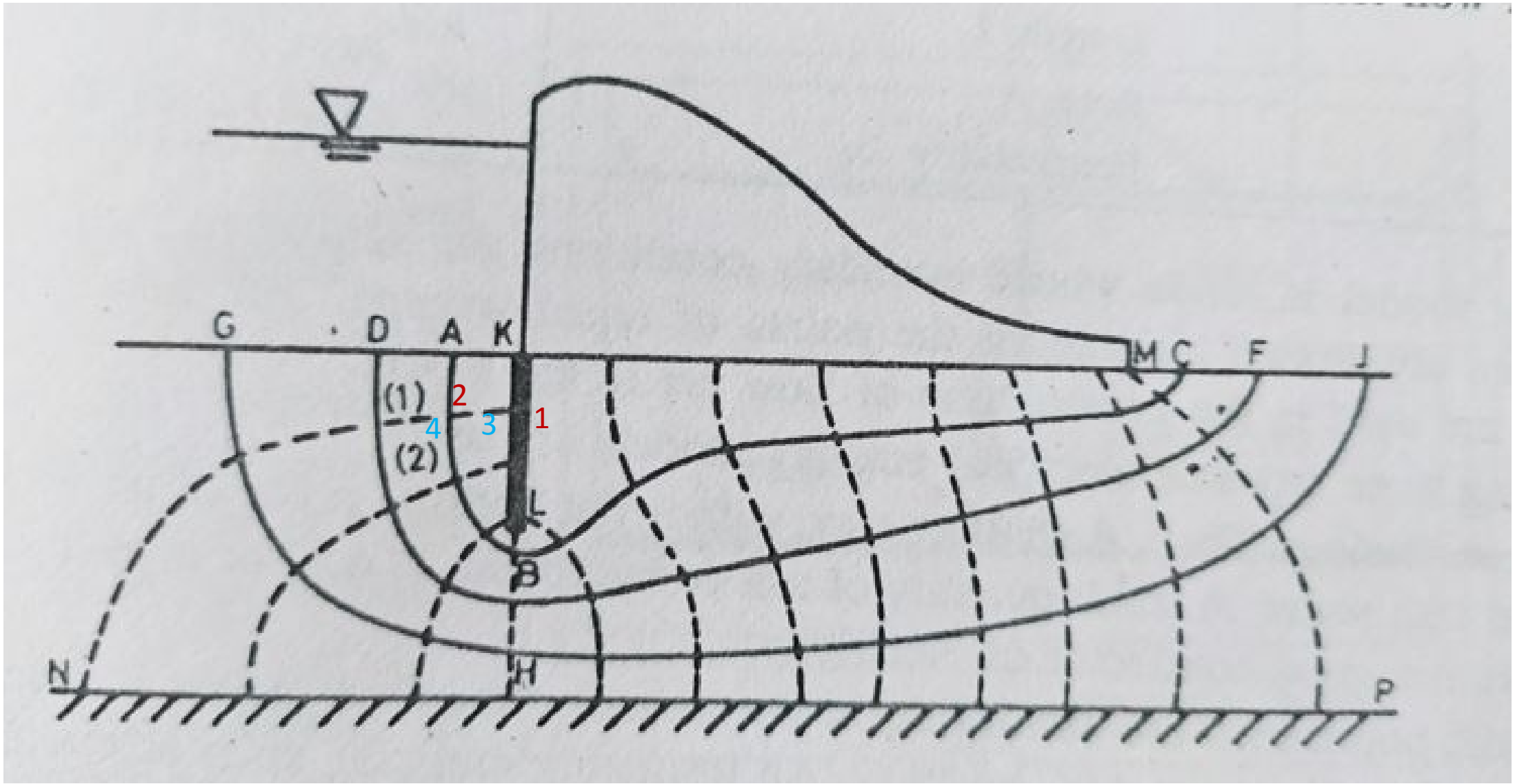
Step-5:

- **Equipotential lines are further extended downward**, and one more **flow line GHJ is drawn**, repeating steps 4.

Step-6:

- If the **flow fields in the last flow ch**annel are **inconsistent** with **actual boundary** conditions, the whole procedure is repeated after taking a new trial flow line.

It is **not necessary that the flow elements/fields** of the last flow channel should make complete squares. Flow elements/fields in the last flow channel may be approximately rectangles with the **same length to width ratio**. In this case number of flow channels would not be full integer. Flow channels will be an integer only by chance.



Flow net construction by graphical method

Draw a flow net for a single row sheet pile driven in to a permeable soil as shown in the figure, given following data:

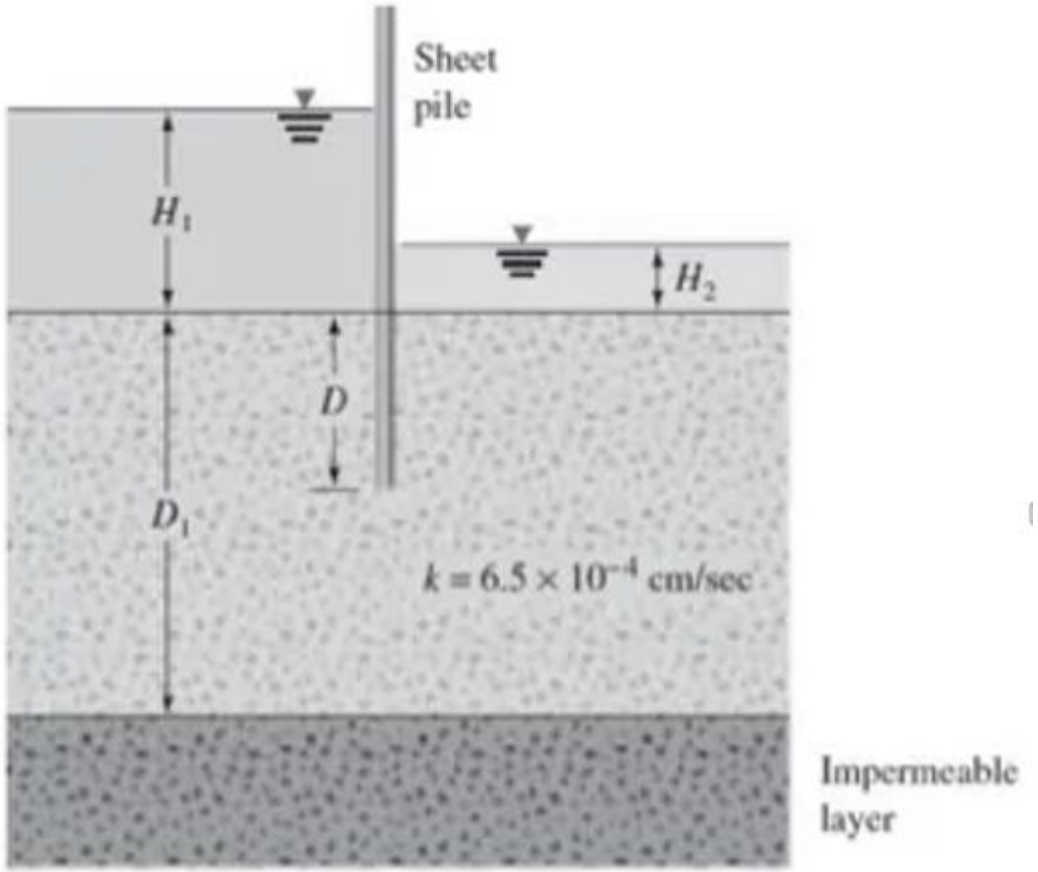
H_1=5.o m

H_2=0.4 m

D_1=10.0 m

D=4.0 m

Also calculate the seepage loss per meter length of sheet pile (at right angle to the cross section).



Total Head loss

H= $H_1 - H_2$=5.o m–0.4 m= 4.6 m

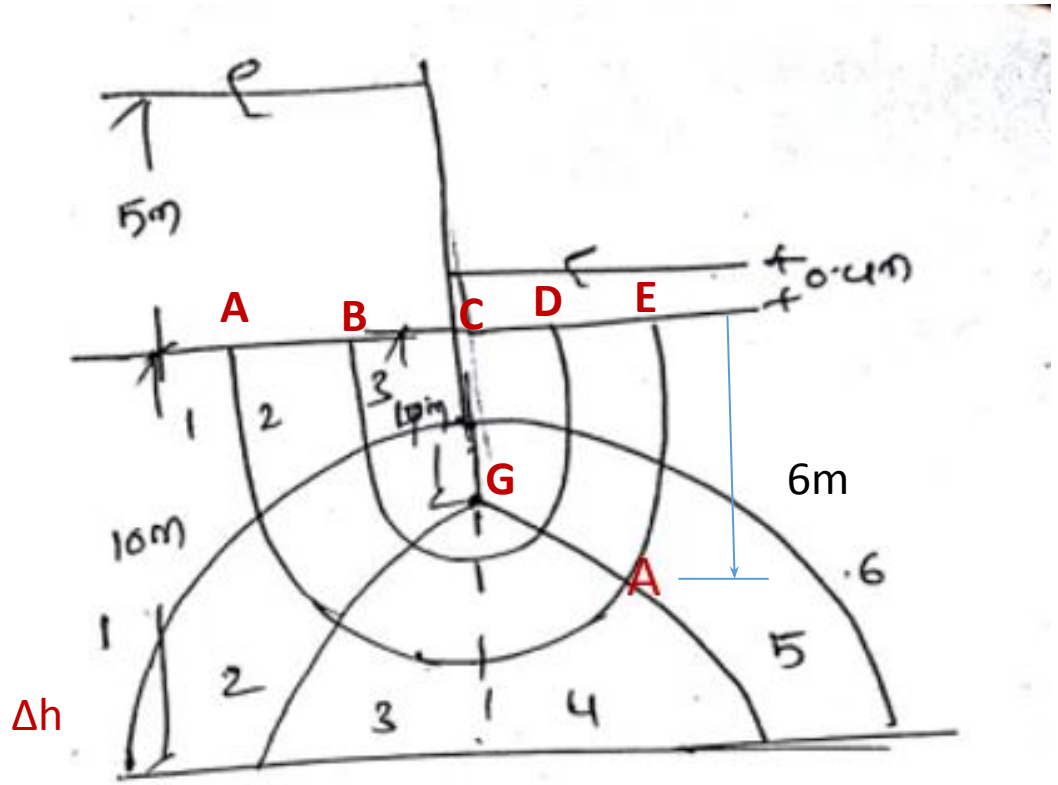
Number of flow Channel, N_f=3

Number of Equipotential drop, N_d=6

Potential drop between two adjacent potential line, $\Delta h = \frac{H_1}{N_d} = \frac{5}{6} =$ **0.833m**

Square flow element, $\frac{b}{a}$=1

K= 6.5x 10^{-4}cm/sec = 6.5x 10^{-6} m/sec



Seepage loss, $q = k \frac{H}{N_d} N_f \frac{b}{a} = k \Delta h N_f$

$q = 6.5x\ 10^{-6} x\ 0.833\ x3\ x1 = 16.24 \times 10^{-6}$ m^3/sec/m =14.03 m^3/day/m

Pore Water Pressure at Pont A

u_A = ?

Need to know the water head at point A

Water coming from upstream, how many potential drop to reach at A

Experiencing number of potential drop $N_d = 4$

Each time you cross a equipotential line total head loss Is equal to Δh

At point A water experiencing $4\Delta h$ **(from u/s)**

Total Head at A, h_A= Initial Head at u/s –Head Loss = H_1 - 4Δh =5.0 -4x0.767= 1.93m
(w.r.t above GS, GS as our datum)

Now the elevation head, z_A= directly from the graph = - 6.0 m (negative)

Pressure at A, $u_A = \gamma_w$ x (h_a - z_a) = 9.81x(1.93 – (-6))= 9.81x7.93=77.79 kN/m^2

Thank You

