University of Global Village (UGV), Barishal

Geotechnical Engineering-I

Content of Theory Course

Prepared By
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Program: B.Sc. in CE

BASIC COURSE INFORMATION

Course Title	Geotechnical Engineering-I
Course Code	CE 0732-3203
Credits	04
CIE Marks	120
SEE Marks	80
Exam Hours	2 hours (Mid Exam) 3 hours (Semester Final Exam)
Level	6th Semester

Geotechnical Engineering-I

Course Code: CE 0732-3203 **CREDIT:04**

Course Teacher: Rubieyat Bin Ali **TOTAL MARKS:200**

Mid Exam Hours 2 **CIE MARKS: 120**

Semester End Exam Hours 3 **SEE MARKS: 80**

Course Learning Outcomes (CLOs): After completing this course successfully, the students will be able to-

CLO 1	**Understand** the soil formation (i.e. Science) and determination of its physical and engineering properties using mathematical and engineering knowledge.
CLO 2	**Figure** out solution of soil related problems faced by the engineering society using both theoretical knowledge and soil testing.
CLO 3	**Use** existing knowledge/experimental setup for understanding the complex mechanisms within the soil.
CLO 4	**Gain** the ability to solve a range of soil related problems, especially those involving water flow and soil settlement.
CLO 5	**Develop** an understanding regarding responsibilities of professional engineers at the site and dealing with expected legal and cultural issues at the site**.**

SL	Content of Course	Hrs	CLOs
1	Details Definition of soil, Soil Mechanics and Geotechnical engineering, Compressibility of soil, Biography of the Father of Soil Mechanics, Mechanical and Chemical weathering process, Details classification of soil, MIT, USDA and AASHTO classification, Textural Classification Chart, Details about sandy soil, Details about clayey soil, Details about residual soil, Transported and organic soil, Single grain and honeycomb structure, Flocculent and dispersed structure, Montmorillonite, Illite and kaolinite.	8	CLO1
2	3 phase system, Void-ratio, porosity, water content and degree of saturation, Different relationships, Mathematical problems, Liquid Limit, Plastic Limit, Shrinkage Limit, Disturbed soil and undisturbed soil.	8	CLO2, CLO3
3	Compaction, General Principle, Optimum moisture content and zero air void line, Standard and modified proctor test, Factors and Field Compaction, Total stress applied to soil, Stress taken by soil, Pressure taken by pore of water, Mathematical problems.	9	CLO3, CLO4
4	Soil permeability: Factors affecting, Constant head and falling head method, Flow property of fluid, Seepage pressure through earthen embankment, Active and passive earth pressure, Determining active earth pressure for different cases.	9	CLO4, CLO5

Text Book:

1. “Principles of Geotechnical Engineering” 5th Edition By Braja M. Das.
2. “Soil Mechanics and Foundations” by Punmia, Dr. B. C.; Jain, Ashok Kumar and Jain, A. K.
3. “Soil Mechanics and Foundation Engineering” by Dr. K.R. Arora
4. “Principles of Foundation Engineering” by Braja M. Das.

ASSESSMENT PATTERN
CIE- Continuous Internal Evaluation (120 Marks)
SEE- Semester End Examination (80 Marks)

Bloom's Category	Tests
Remember	**10**
Understand	**15**
Apply	**15**
Analyze	**15**
Evaluate	**15**
Create	**10**

Bloom's Category Marks (out of 120)	Tests (min 4) (60)	Assignments (min 2) (20)	Quizzes (20)	External Participation in Curricular/Co-Curricular Activities (20)
Remember	10		10	Attendance/Field Visit/Presentation /Project Exhibition (20)
Understand	10		10	
Apply	10			
Analyze	10			
Evaluate	10			
Create	10	20		

Course plan specifying content, CLOs, teaching learning and assessment strategy mapped with CLOs

Week	Topic	Teaching-Learning Strategy	Assessment Strategy	Corresponding CLOs
1	Soil Mechanics Details, Karl Von Terzaghi, Weathering Process	Lecture, discussion, group work	Quiz, Written Exam	CLO1
2-3	Origin and classification of Soil	Oral Presentation, debate	Assignment, Written, Quiz	CLO1
4	Details types of soil (Sandy and clayey soil)	Video lecture, Field visit	Report writing, Demonstration	CLO1
5	Soil structure and Clay Mineralogy	Lecture	Viva, Quiz	CLO1, CLO3
6	Index Property of Soil	Project exhibition	Project, Field visit	CLO1
7	Mathematical Problems Related to soil three phase system	Discussion, Video Presentation	Quiz, Written Exam	CLO1
8	Atterberg limit (LL, PL and SL)	Case-based Learning, Demonstration	Assignment, Written, Quiz	CLO1, CLO4
9	Soil sample collection techniques	Lecture, discussion, group work	Report writing, Demonstration	CLO3, CLO4
10	Soil Compaction I	Oral Presentation, debate	Viva, Quiz	CLO2
11	Soil Compaction II	Video lecture	Project, Field visit	CLO2

Course plan specifying content, CLOs, teaching learning and assessment strategy mapped with CLOs

Week	Topic	Teaching-Learning Strategy	Assessment Strategy	Corresponding CLOs
12	Total stress (TS), Effective stress (ES) and pore water pressure (PWP)	Lecture	Quiz, Written Exam	CLO3, CLO5
13	Mathematics (TS, ES and PWP)	Project exhibition	Assignment, Written, Quiz	CLO3, CLO3
14	Soil Permeability	Discussion, Video Presentation	Report writing, Demonstration	CLO1
15	Seepage of Soil	Case-based Learning, Demonstration	Viva, Quiz	CLO1
16	Active and passive earth pressure of soil I	Lecture, discussion, group work	Project, Field visit	CLO3, CLO5
17	Practice and Exercise, Practice and Exercise, Practice and Exercise	Video lecture	Assignment, Written, Quiz	CLO3, CLO5

Introduction to Soil Mechanics

Week 1

Pages 9-42

Soil:

For engineering purposes, soil is defined as the uncemented aggregate of mineral grains and decayed organic matter (solid particles) with liquid and gas in the empty spaces between the solid particles. Soil is used as a construction material in various civil engineering projects, and it supports structural foundations. Thus, civil engineers must study the properties of soil, such as its origin, grain-size distribution, ability to drain water, compressibility, shear strength, and load-bearing capacity.

Soil

Soil Mechanics:

Soil mechanics is the branch of science that deals with the study of the physical properties of soil and the behavior of soil masses subjected to various types of forces.

Soil Engineering:

Soils engineering is the application of the principles of soil mechanics to practical problems.

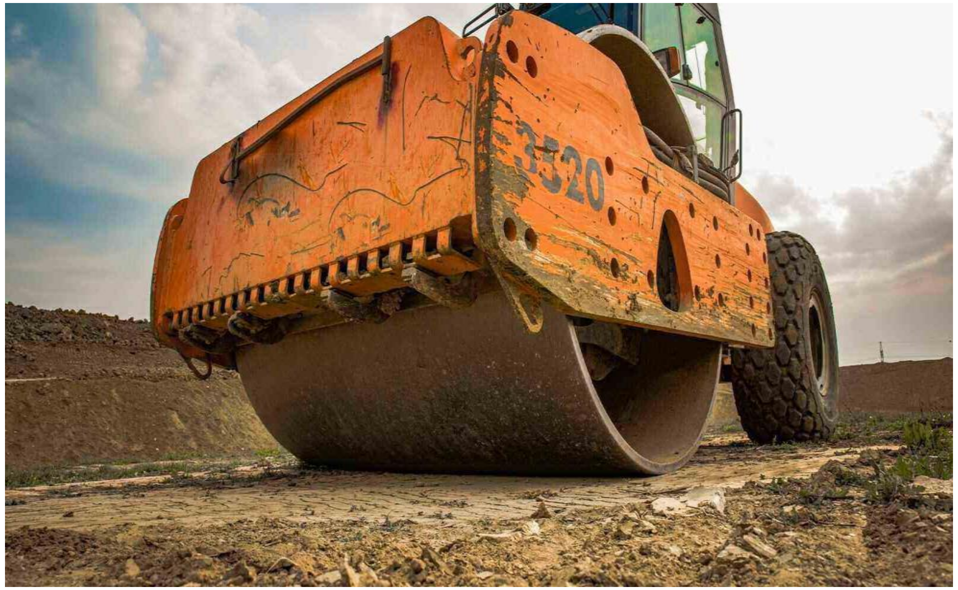
Geotechnical Engineering:

Geotechnical engineering is the sub discipline of civil engineering that involves natural materials found close to the surface of the earth. It includes the application of the principles of soil mechanics and rock mechanics to the design of foundations, retaining structures, and earth structures.

Geotechnical Engineering

Soil Compressibility:

Soil compressibility is the degree to which a soil's volume decreases when a mechanical load is applied. It's an important parameter that describes the relationship between the effective stress and the void ratio of a soil.



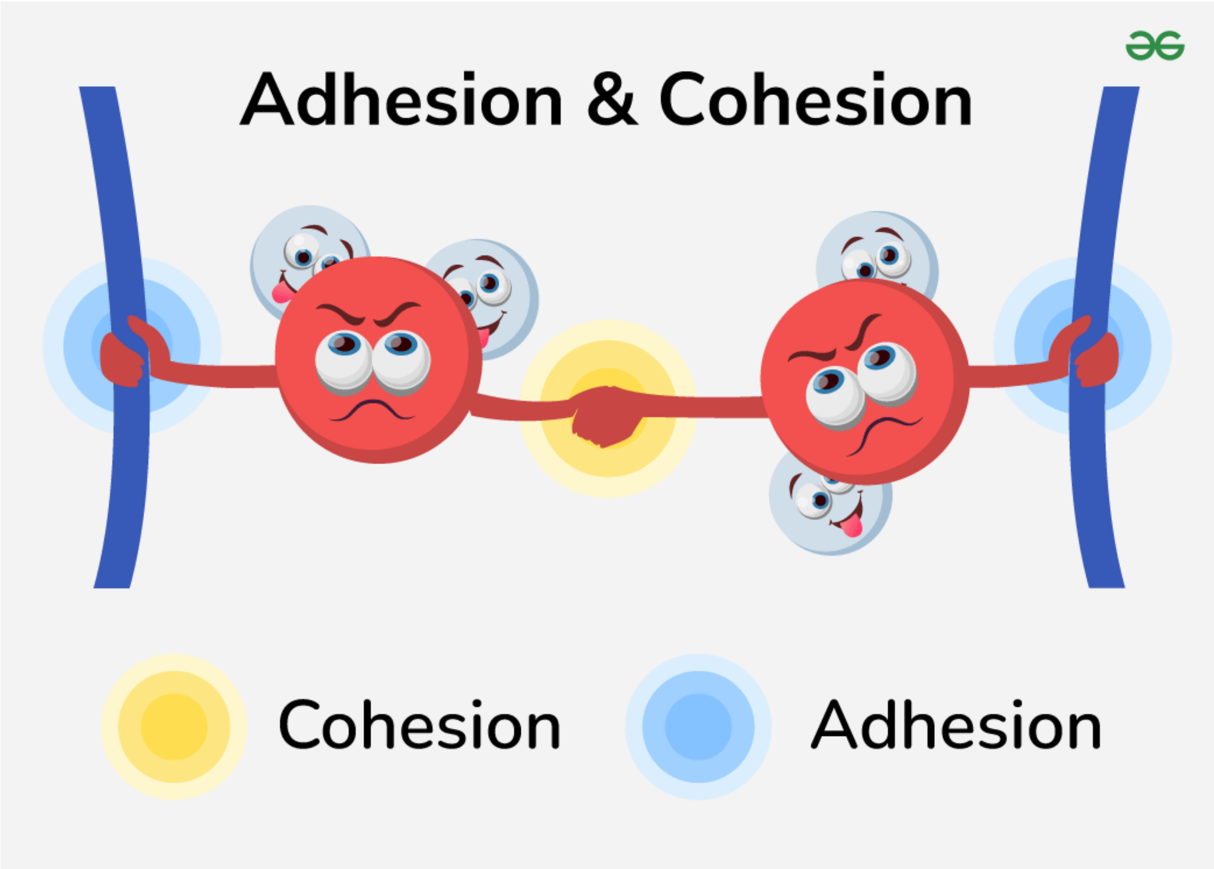
Soil Compressibility

Causes of compression:

Soil compresses when air or water is excluded from void spaces, soil particles rearrange, solid particles compress and deform, and liquids and gases within the voids compress. The compressibility of soil varies depending on its type and origin. For example, soft clay has a very high degree of compressibility, while silt has a low degree of compressibility. Example: Clay.

Cohesion Vs. Adhesion

Cohesion is the property of molecules (of the same substance) to stick to each other due to mutual attraction. Adhesion is the property of different molecules or surfaces to cling to each other. For example, solids have high cohesive properties so they do not stick to the surfaces they come in contact.



Cohesion Vs. Adhesion

Karl Terzaghi: The Father of Soil Mechanics

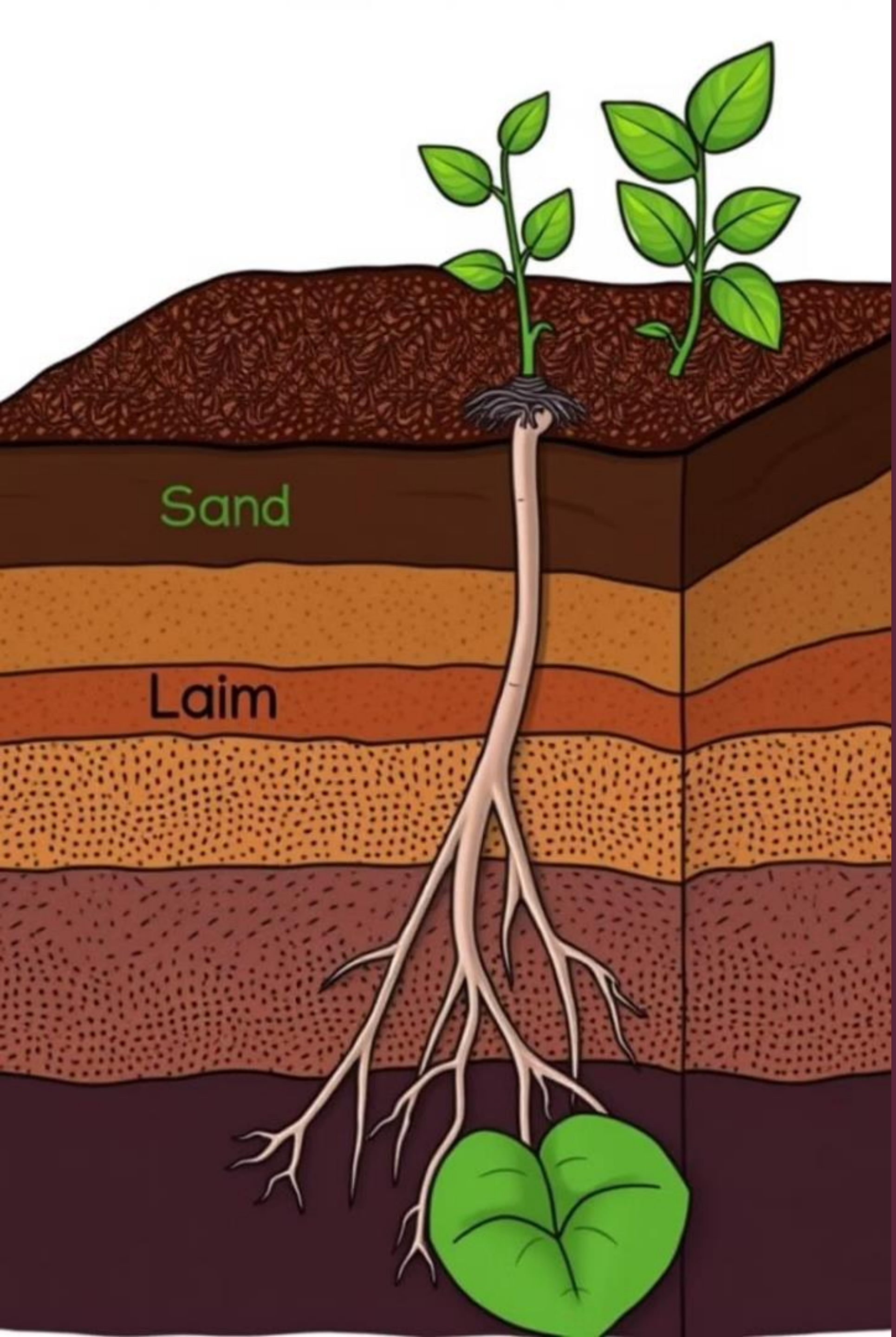
Karl Terzaghi revolutionized the understanding of soil behavior and laid the foundation for modern geotechnical engineering.

R **by Rubieyat Bin Ali**

Early Life and Education

Born in Prague, Austria-Hungary, in 1883, Terzaghi exhibited a passion for science from a young age.

He earned a doctorate in engineering from the Technical University of Vienna, specializing in hydraulics and soil mechanics.



Pioneering Work in Soil Mechanics

1 **Early Research**

Terzaghi's early research focused on the behavior of soil under load and its implications for foundations, dams, and other structures.

2 **Field Experiments**

He conducted extensive field experiments to validate his theories and develop practical solutions for real-world problems.

3 **Groundbreaking Publications**

Terzaghi published seminal works, including "Erdbaumechanik" in 1925, which established soil mechanics as a distinct field of study.

Key Contributions and Innovations

Consolidation Theory

He developed a theory to predict the settlement of soils over time under applied loads.

Effective Stress Principle

Terzaghi's groundbreaking principle explained how soil strength is influenced by pore water pressure.

Shear Strength Parameters

Terzaghi introduced the concept of cohesion and angle of internal friction to characterize soil strength.

Effective Stress Principle and its Impact

Improved Designs

The effective stress principle enabled engineers to design foundations and structures with greater accuracy and safety.

Optimized Construction

It provided a framework for predicting and controlling soil behavior during construction, leading to more efficient and cost-effective projects.

Enhanced Safety

The principle contributed significantly to the safety of structures and infrastructure, protecting lives and property.

Observational Method and Construction Monitoring

1 Terzaghi emphasized the importance of monitoring soil behavior during construction to validate design assumptions and identify potential problems.

2 He advocated for the use of instruments like piezometers and inclinometers to measure ground movement and pore water pressure.

3 The observational method allowed for timely adjustments to construction plans based on real-time data, minimizing risks and ensuring project success.

Global Impact and Consulting Work

1

Terzaghi's expertise was sought after globally, and he consulted on numerous major projects worldwide.

2

He played a key role in designing and constructing dams, tunnels, and other infrastructure projects in diverse geological settings.

3

His influence extended to developing countries, contributing to the advancement of geotechnical engineering practices in new regions.

Personality and Interests

1

Passionate

Terzaghi was deeply passionate about soil mechanics and geotechnical engineering, dedicated to advancing the field.

2

Intellectual

He was known for his sharp intellect and ability to solve complex problems with innovative solutions.

3

Pragmatic

Terzaghi's approach was practical and grounded in real-world observations and experiments.

4

Teacher

He was a gifted teacher and mentor, inspiring generations of geotechnical engineers.

Pragmatic Philosophy and Methods

1

Observation

Terzaghi emphasized the importance of careful observation and data collection to understand soil behavior.

2

Experimentation

He believed in conducting rigorous experiments to validate theories and develop practical solutions.

3

Simplicity

Terzaghi preferred simple, practical approaches over overly complex theoretical models.

Terzaghi's Legacy and Influence

100+

Years of Impact

Terzaghi's contributions have shaped the field of geotechnical engineering for over a century.

1000+

Projects

His work has been applied to countless infrastructure projects worldwide, ensuring safety and efficiency.

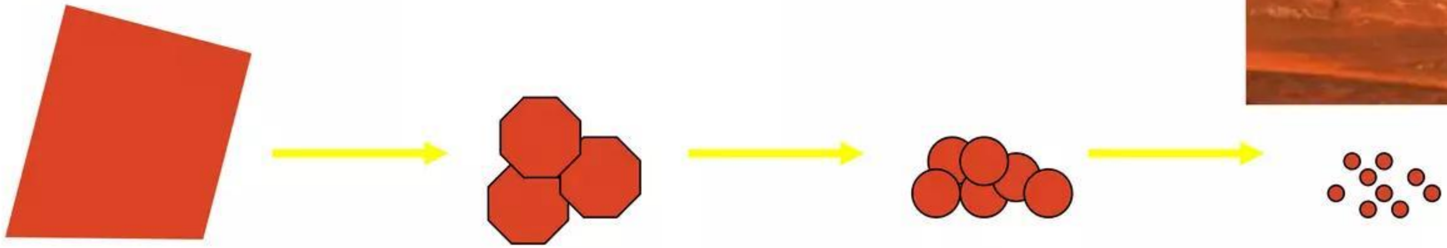
100000+

Engineers

His legacy continues to inspire generations of geotechnical engineers to push the boundaries of the field.

Weathering

- Process of Decay & Disintegration of rocks under the influence of certain physical & chemical agencies of atmosphere like wind, water, ice, sun etc.

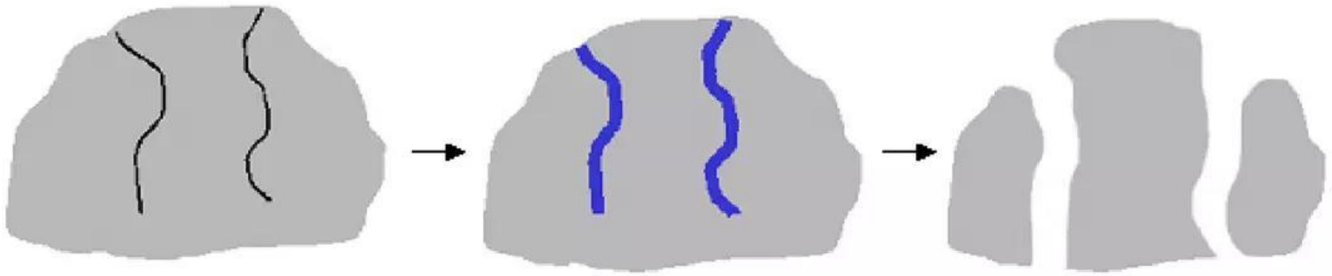


Weathering

➢ Terminology

1) Disintegration

➢ The Process by which the rock is broken into small piece by mechanical action of the physical agents without chemical change.



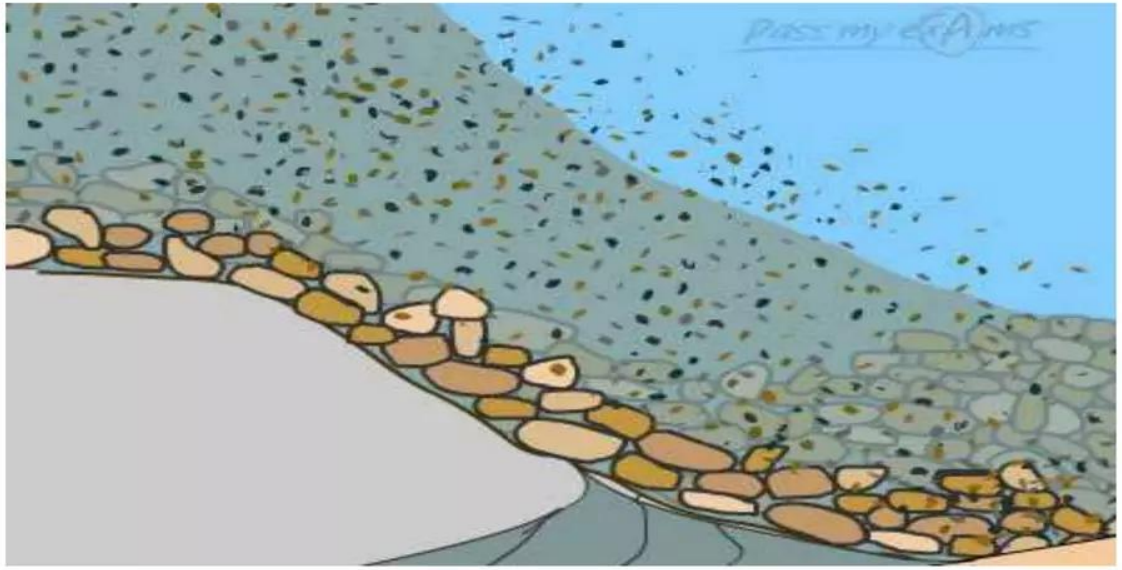
The black lines in the rock represent fractures that are occurring in the rock.

The blue lines in the rock represent water soaking into the fractures.

The water freezes and expands. If this cycle of freezing, expansion, and thawing continues, the rock will gradually disintegrate.

Weathering

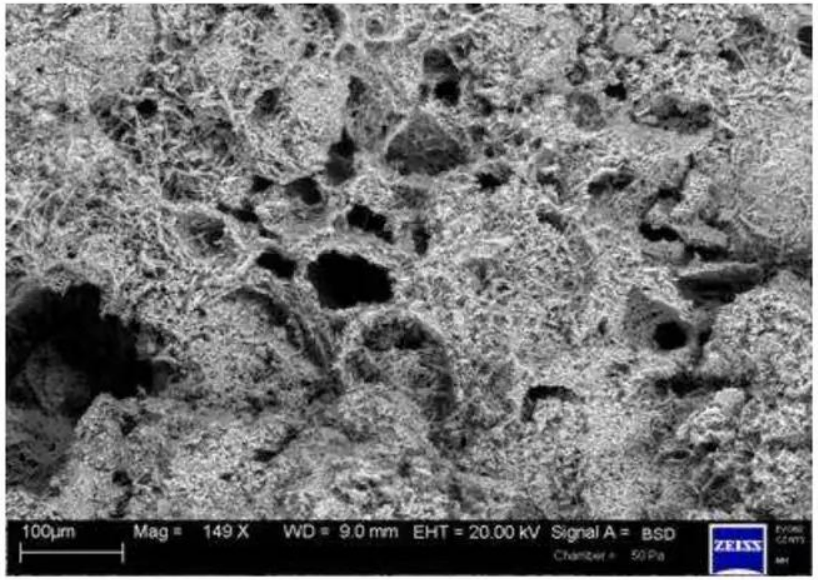
➢ Terminology



2) Deposition

- ➢ Process by which the transported broken pieces of rocks are deposited at other place with or without chemical change.

Weathering



➢ **Terminology**

3) Decomposition

➢ Process by which rocks are broken into small pieces by destructive chemical action of the physical agents is called decomposition.

Weathering

➢ Terminology

4) Erosion

➢ Process which includes destruction of existing rocks and removal of product from site of destruction.

➢ Terminology

5) Deflation

- Process by which rock particles are removed by impact of strong wind.

Weathering

➢ **Types of Weathering**

1) Mechanical Weathering
2) Chemical Weathering
3) Biological Weathering

Weathering

➢**Mechanical Weathering**

- Wind

Weathering

➢**Mechanical Weathering**

- River

Weathering

➢Mechanical Weathering

- Glaciers

Weathering

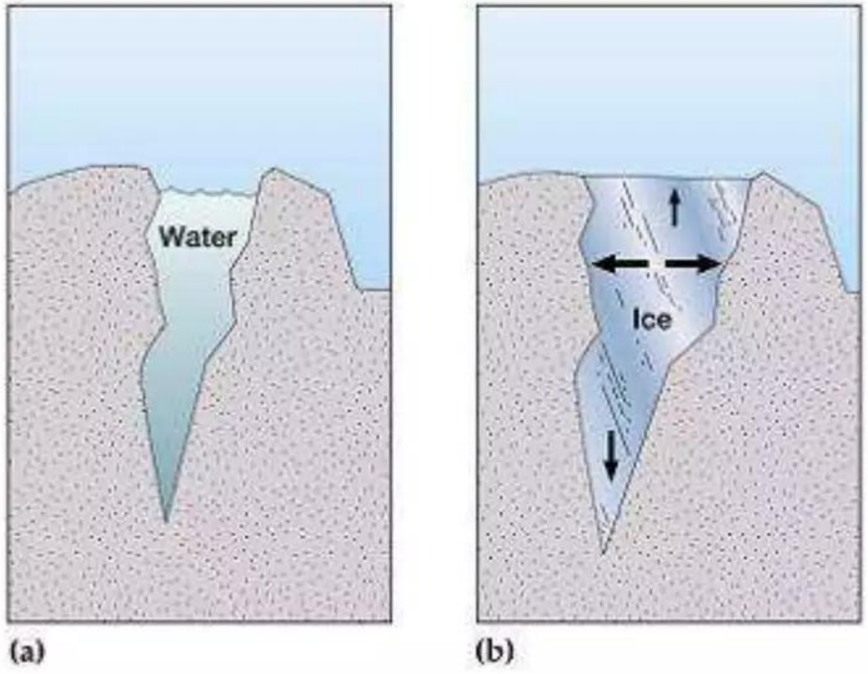
➢ Mechanical Weathering

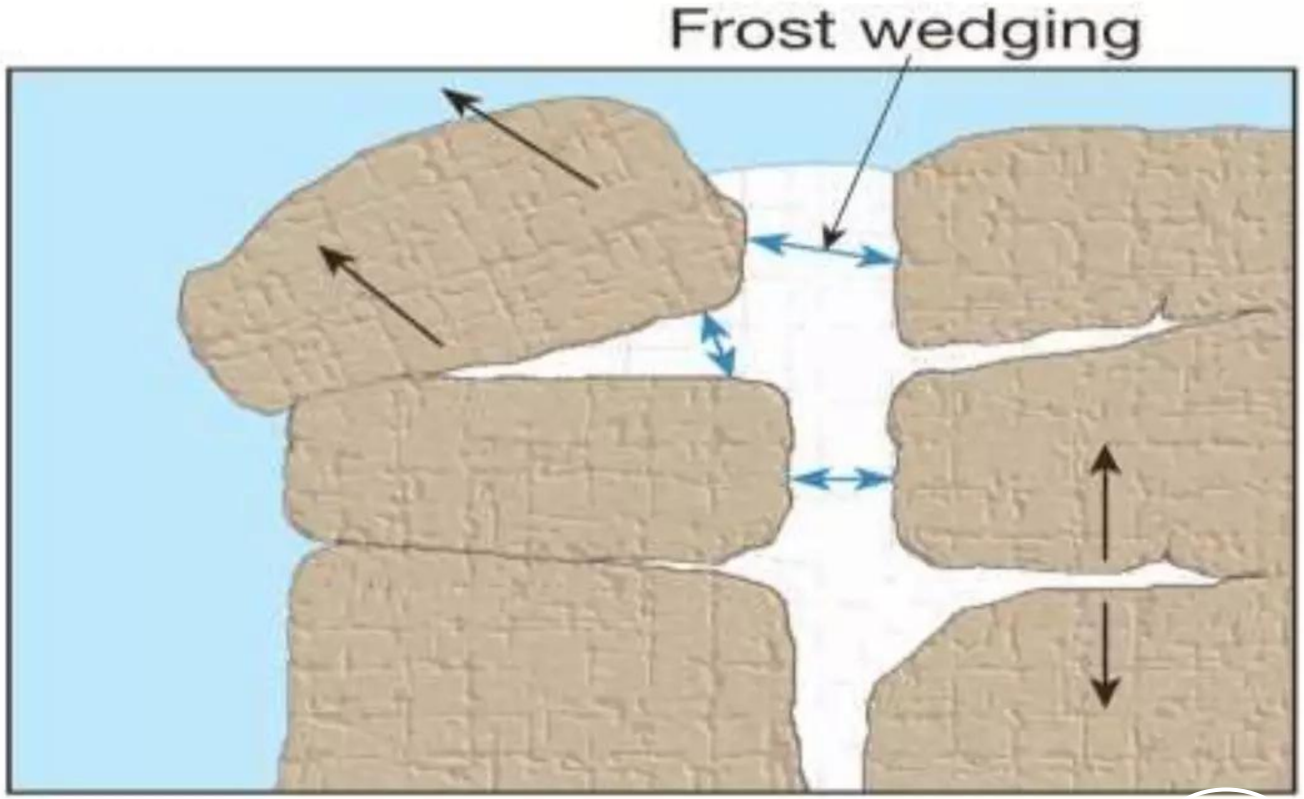
- Gravity

Weathering

➢**Mechanical Weathering**

- Wedging Action of Ice





Weathering

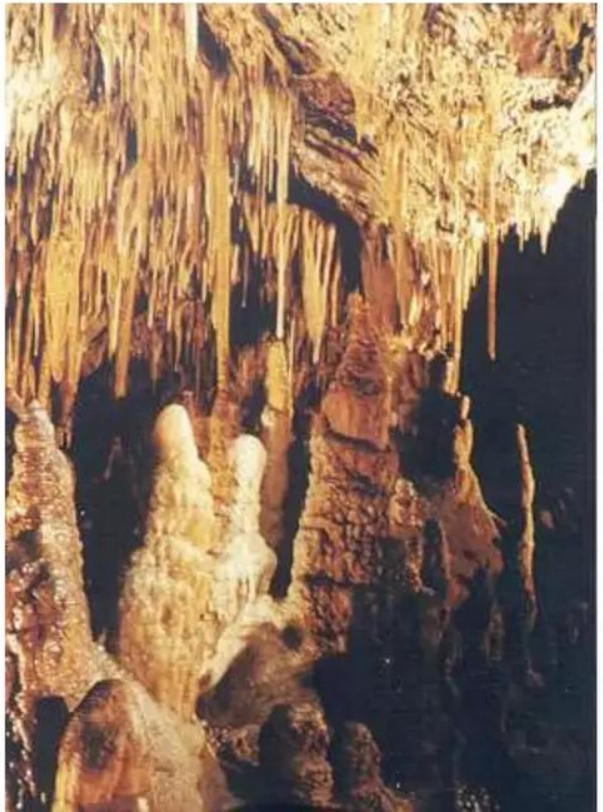
➢ Chemical Weathering

• Oxidation

Weathering

➢ **Chemical Weathering**

- Carbonation



Weathering

➢Chemical Weathering

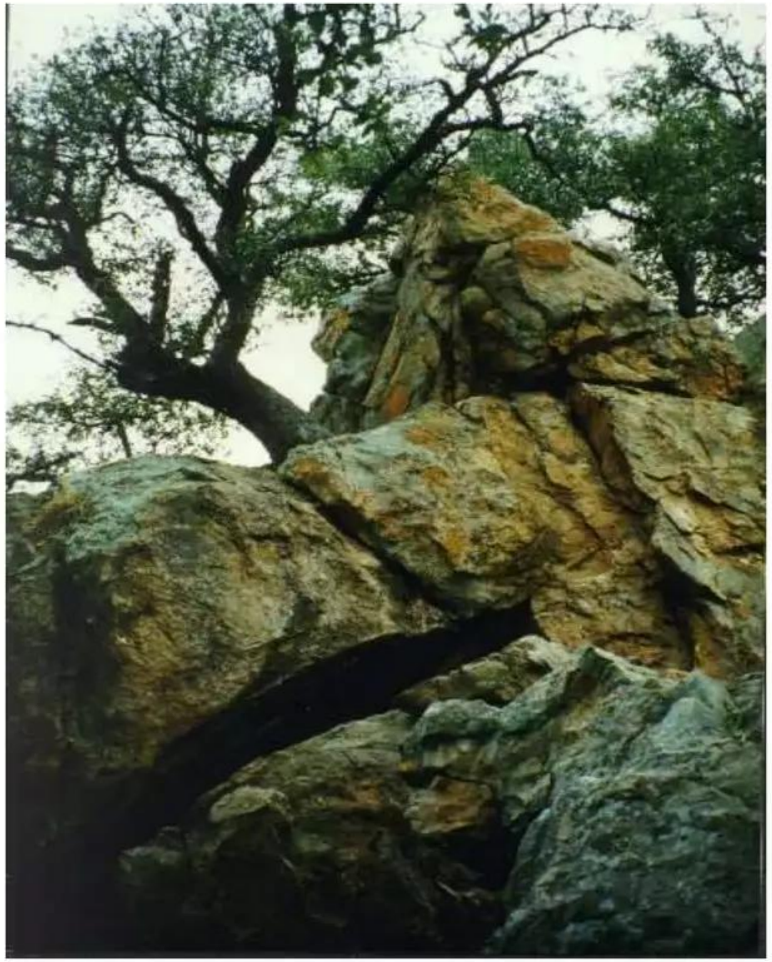
- Hydration

Stalactite formation - evaporation of water leaves small amounts of calcite behind.

Weathering

➢ **Biological Weathering**

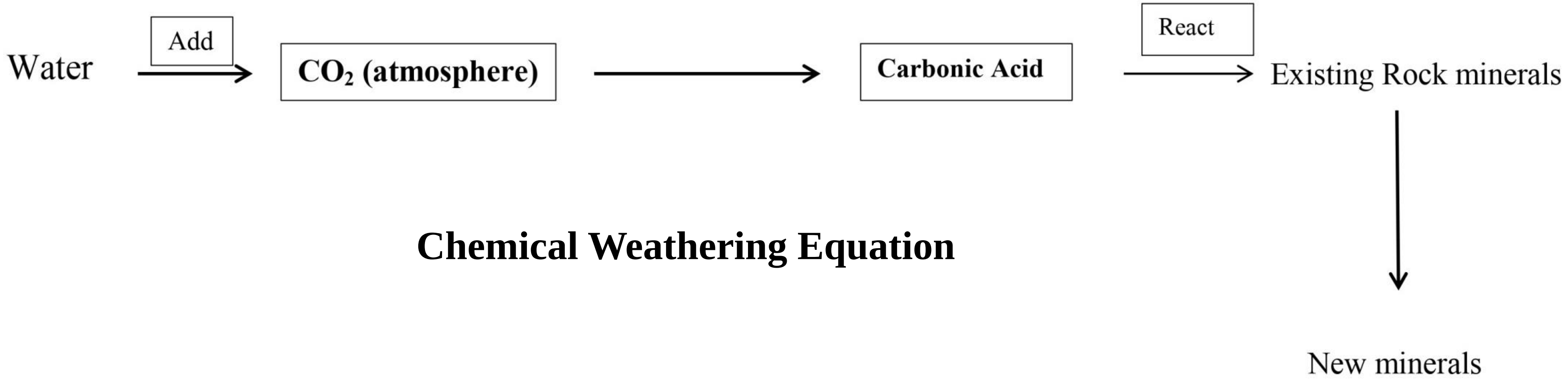
- Trees & Plants



Weathering

➢ Biological Weathering

- **Manmade**
- During Construction of Tunnels & Mining



Chemical Weathering Equation

Weathering

➢ Biological Weathering

- **Animals**
- Some Animals make burrows underground & help in weathering of rocks.

Weathering

➢ Factors Affecting Weathering

1) Nature of Rock (e.g. Granite & Sandstone)
2) Climate (Climatic Conditions)
3) Physical Environment (Topography)
4) Animals (Burrowing Organisms)

Weathering

➢ Importance of Weathering

1) Weathered Soil is fertile for agriculture
2) Weathering makes rocks porous and permeable, which is important for ground water occurrence
3) Develops cheap building Stones
4) Economic minerals deposits like bauxites formed due to weathering

Classification of Soil

Week 2-3

Pages 43-84

CONTENTS

Purpose of Soil Classification

1. Soils in nature rarely exist separately as gravel, sand, silt, clay or organic matter, but are usually found as mixtures with varying proportions of these components.
2. Classifying soils into groups with similar behavior, in terms of simple indices, can provide geotechnical engineers a general guidance about engineering properties of the soils through the accumulated experience.
3. Two commonly used approach for Classifying soils is based on particle distribution and Atterberg limits

gravels soil

organic soil

Soil Classification Systems

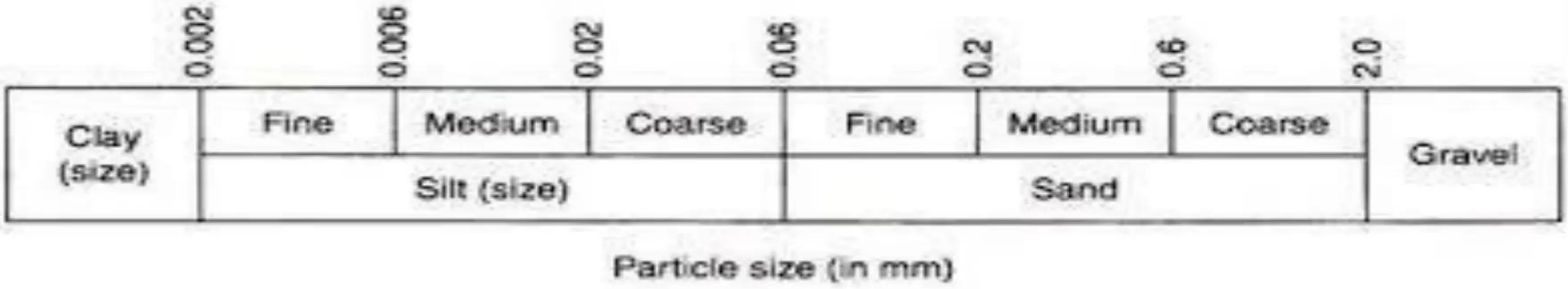
1. Particle Size Classification
2. Textural Classification
3. Highway Research Board (HRB) Classification
4. Unified Soil Classification System (USCS)
5. Indian Standard Classification System (ISCS)

Particle Size Classification

Introduction

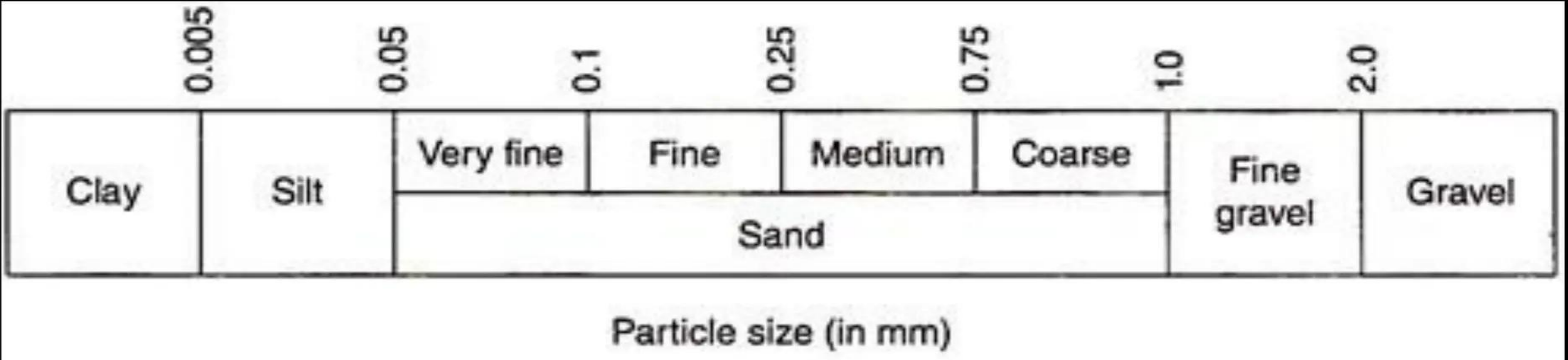
- In this system soils arranged according to grain sizes.
- Terms such as **gravel, sand, silt and clay** are used to indicate grain sizes.
- These terms are used only as designation of particle sizes.
- They do not signify naturally occurring soil types, as naturally occurring soil are mixture of particles of different sizes.
- Plasticity characteristics of soil is not accounted for soil classification in this system.
- In India Particle size classification is done on the basis of IS 1498:1970

MIT Soil Classification



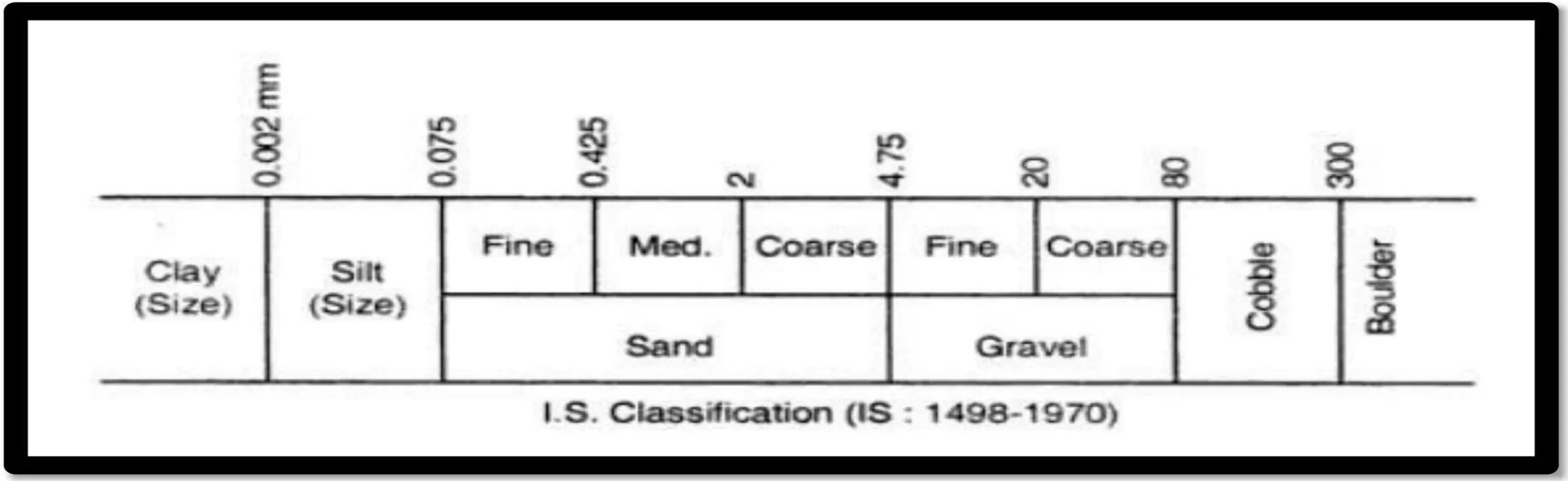
MIT soil classification.

US Bureau Soil Classification



US Bureau of soil classification.

IS Classification of Grain Size



I.S. Classification (IS : 1498-1970)

Particle Size Distribution

- The percentage of various sizes of soil particles in a given dry soil sample is found by mechanical analysis or particle size analysis.
- By mechanical sieve analysis is meant for separation of a soil into its different size fractions.
- The mechanical sieve analysis is performed in two stages as shown below:

Mechanical Analysis

- **Sieve Analysis**
- **Sedimentation or wet mechanical analysis**

Sieve Analysis:

- Sieve analysis consists of shaking the soil sample through a set of sieves that have progressively smaller openings.
- In the Indian Standard (IS: 460-1962), the sieves are designated by size of opening in (mm) as given.
- The complete sieve analysis can be divided into two parts- the coarse analysis and fine analysis.

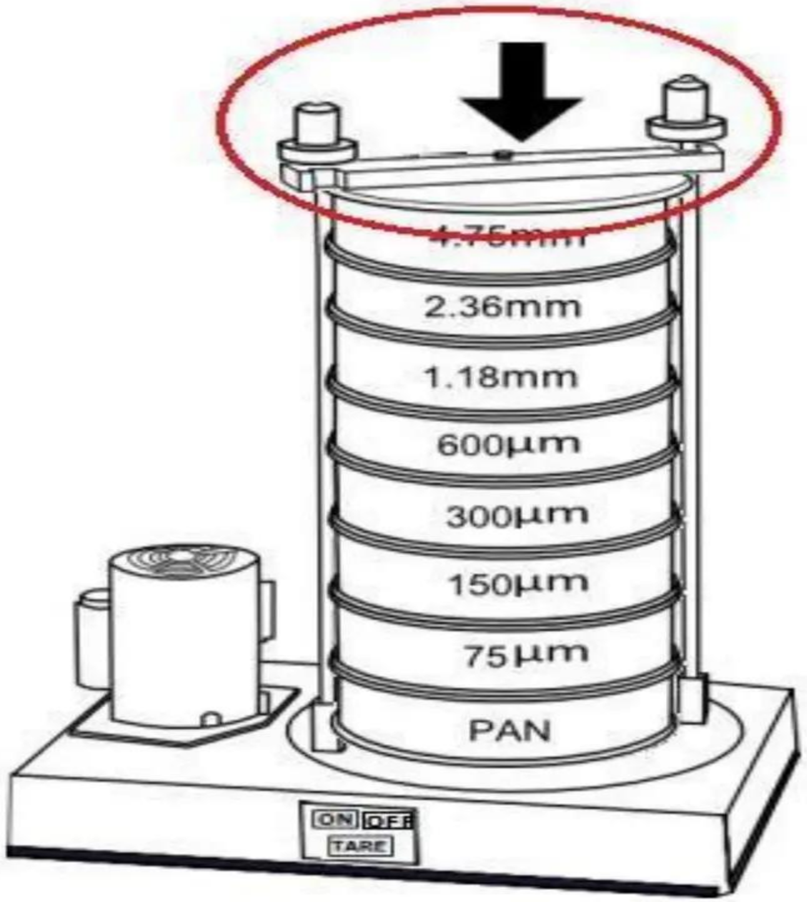
Sieve Number	Size of Opening (mm)
4	4.750
6	3.350
8	2.360
12	1.680
16	1.180
20	0.850
30	0.600
40	0.425
50	0.300
60	0.250
80	0.180
100	0.150
140	0.106
200	0.075

Sieve Analysis

- An oven dried sample of soil is separated into two fractions by sieving it through a 4.75mm IS sieve.
- The portion retained on it is termed as *gravel fraction* and is kept for coarse analysis, while portion passing through it is subjected to fine sieve analysis.
- Soil passing 4.75mm I.S. Sieve and retained on 75micron I.S. Sieve contains no fines. Those soils can be directly dry sieved rather than wet sieving.

Dry Sieving:

- Take 500gm of the soil sample after taking representative sample by quartering.
- Conduct sieve analysis using a set of standard sieves as given in the data sheet.
- The sieving may be done either by hand or by mechanical sieve shaker for 10minutes.Weigh the material retained on each sieve.
- The percentage retained on each sieve is calculated on the basis of the total weight of the soil sample taken.
- From these results the percentage passing through each of the sieves is calculated.
- Draw the grain size curve for the soil in the semi- logarithmic graph provided.



Mechanical Sieve Shaker

Wet Sieving:

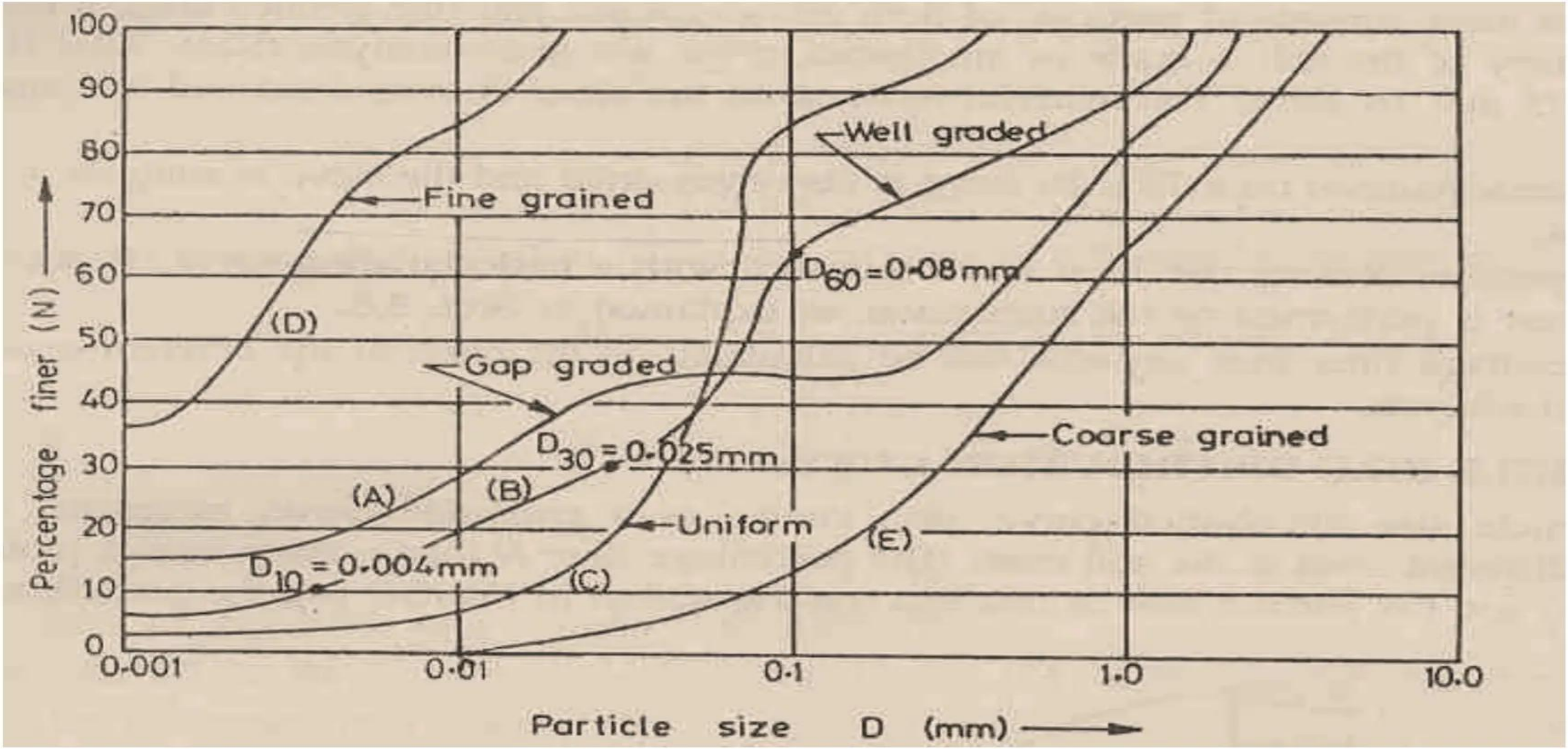
If the soil contains substantial quantity (say more than 5%) of fine particles, a wet a sieve analysis is required. All lumps are broken into individual particles.

- Take 200gm of oven dried soil sample and soaked with water.
- If deflocculation is required, 2% sodium hexametaphosphate solution is used instead of water.
- The sample is stirred and left for soaking period of at least 1 hour.
- The slurry is then sieved through 4.75 mm sieve and washed with a jet of water.
- The material retained on the sieve is the gravel fraction, which should be dried in oven and weighed.

Wet Sieving:

- The material passing through 4.75mm sieve is sieved through 75μ sieve.
- The material is washed until the water filtered becomes clear.
- The soil retained on 75-micron sieve is collected and dried in oven.
- It is then sieved through the sieve shaker for ten minutes and retained material on each sieve is collected and weighed.
- The material that would have been retained on pan is equal to the total mass of soil minus the sum of the masses of material retained on all sieves.
- Draw the curve for the soil in the semi-logarithmic graph to obtain grain size distribution curve.

Grain-Size Distribution Curves



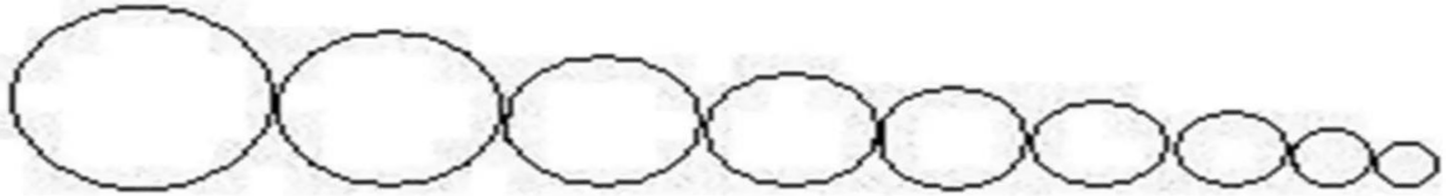


Figure 1. Well Graded

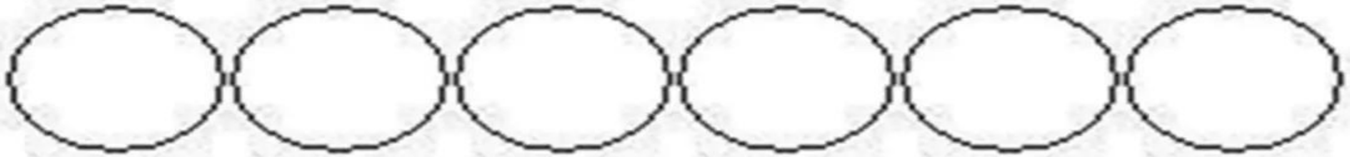


Figure 2. Uniformly-Graded

Figure 3. Gap-Graded

- **Well graded Soil** will have particles from almost all the size ranges and will give a stable mix.
- **Gap graded Soil** will have deficiency of certain particles of a particular size range.
- **Uniformly or poorly graded Soil** will have almost same size or from narrow range of particle sizes of soil in the sample. This sample will contain single size soil.

Coefficients of Uniformity and Curvature

Coefficient of uniformity

$$C_u = \frac{D_{60}}{D_{10}}$$

Coefficient of curvature

$$C_c = \frac{(D_{30})^2}{D_{60} D_{10}}$$

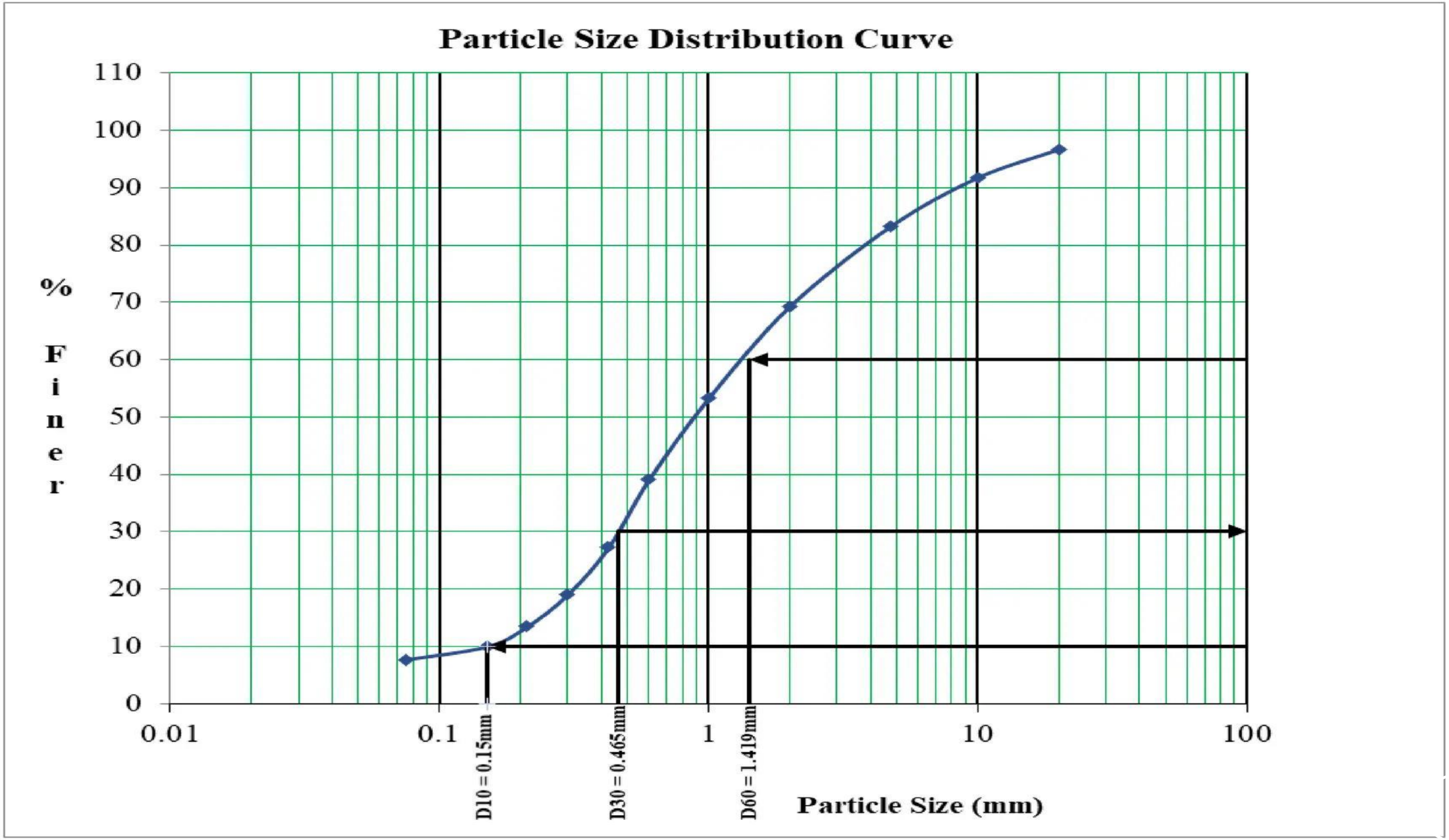
Cu should be more than 4 for gravel and more than 6 for sand.

Coefficient of curvature (Cc) should lie between 1 and 3 for well grade gravel and sand.

- Sieve analysis was performed on 1000 gm of dry soil sample and the following observations were made as given: Use particle size classification system and classify the soil, also find Cu and Cc

Sieve Size (mm)	Mass Retained (gm)
20	33
10	49
4.75	85
2	140
1	160
0.6	142
0.425	118
0.3	82
0.212	56
0.15	35
0.075	23

Sieve Size (mm)	Mass Retained (gm)	% Mass Retained	% Cumulative Retained	% Finer
20	33	3.3	3.3	96.7
10	49	4.9	8.2	91.8
4.75	85	8.5	16.7	83.3
2	140	14	30.7	69.3
1	160	16	46.7	53.3
0.6	142	14.2	60.9	39.1
0.425	118	11.8	72.7	27.3
0.3	82	8.2	80.9	19.1
0.212	56	5.6	86.5	13.5
0.15	35	3.5	90	10
0.075	23	2.3	92.3	7.7


Particle Size Distribution Curve
110
100
90
80
70
60
50
40
30
20
10
0
% Finer
0.01
0.1
1
10
100
D10 = 0.15mm
D30 = 0.465mm
D60 = 1.419mm
Particle Size (mm)

D60	1.419
D30	0.465
D10	0.150

$Cu = D60/D10 = 1.419/0.150 = 9.46 > 6$, Hence it is Sand

$Cc = D30^2/(D60*D10) = 0.465^2/(1.419*0.150) = 1.01 > 1$, Hence it is well graded

The soil may be classified as **Well Graded Sand (SW)**

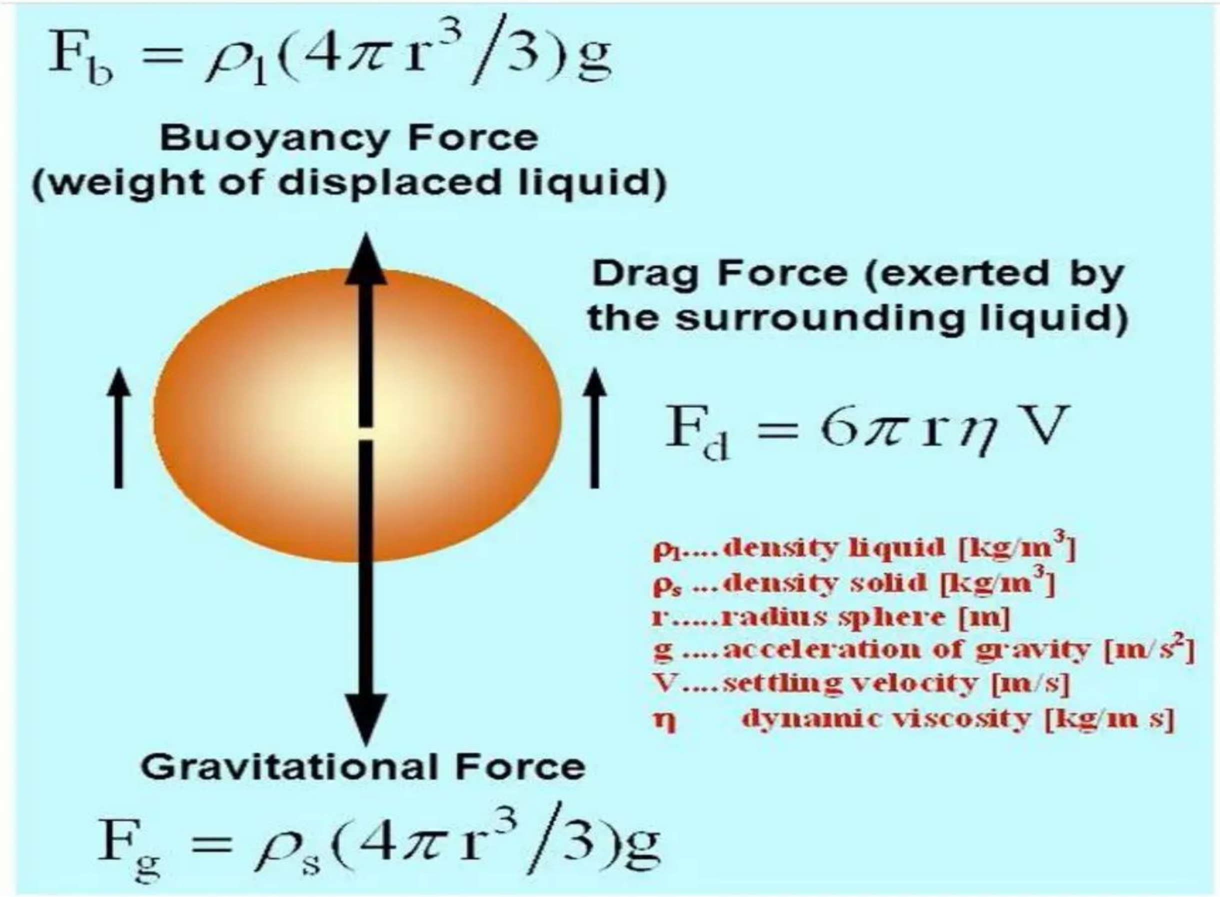
Importance of Particle Size Distribution

1. Used for the soil classification.
2. Used to design drainage filter.
3. Used to select fill materials of embankments, earth dams, road sub-base materials.
4. Used to estimate performance of grouting, chemical injection and dynamic compaction.
5. Effective Size, D_{10}, can be correlated with the hydraulic conductivity.
6. Estimate approximate value of co-efficient of permeability.
7. Used to determine susceptibility of soil to frost action.
8. Used of Soil stabilization and pavement design.

Particle Size Distribution - Sedimentation

- For particles $\leq$ 0.075 mm (silt and clay fractions) sedimentation methods based on Stokes law are used to deduce particle size distribution.
- Soil particles settle in aqueous solution attaining terminal velocities proportional to their mass and size.
- The amount of suspended soil after a given settling time is used to determine particle size fractions.
- The amount of soil in suspension is determined by either extracting a sample by the pipette method or from a direct hydrometer reading.

Stokes Law Three forces acting on a spherical particle.



- Buoyancy and drag forces act against the gravitational force.
- A spherical soil particle D=5 um reaches 99% of its terminal velocity in aqueous solution within 0.017ms, and for D=1 mm the time is 0.68s.
- Buoyancy Force (weight of displaced liquid)Drag Force (exerted by the surrounding liquid) Gravitational Force

Stokes Law

The three forces acting upon the settling particle quickly equilibrate and the particle reaches a constant settling velocity.
We can solve the force balance equation to obtain the settling velocity

$$\sum F_i = 0 = F_g - F_b - F_d$$

$$0 = \rho_s \left(\frac{4\pi r^3}{3} \right) g - \rho_l \left(\frac{4\pi r^3}{3} \right) g - 6\pi r \eta V \quad \Rightarrow \quad V = \frac{(\rho_s - \rho_l) d^2 g}{18\eta}$$

Since we know that velocity equals length per time we can calculate the time particles of a certain size need to settle through a distance h:

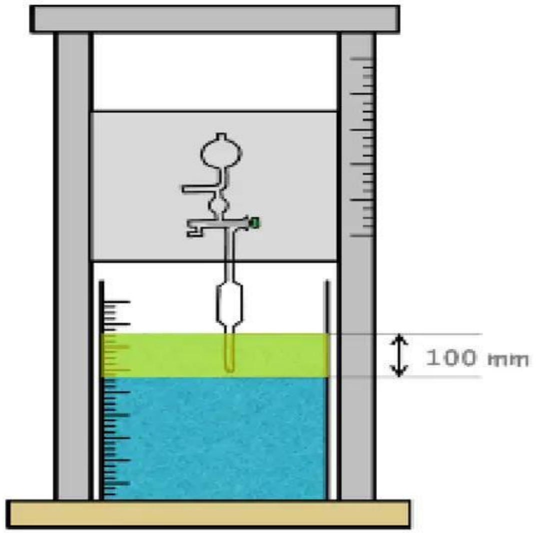
$$\boxed{V = \frac{h}{t} = \frac{(\rho_s - \rho_l) d^2 g}{18\eta} \quad \Rightarrow \quad t = \frac{18\eta h}{(\rho_s - \rho_l) d^2 g}}$$

t is the time required for particles of a certain size to settle below a certain depth h.

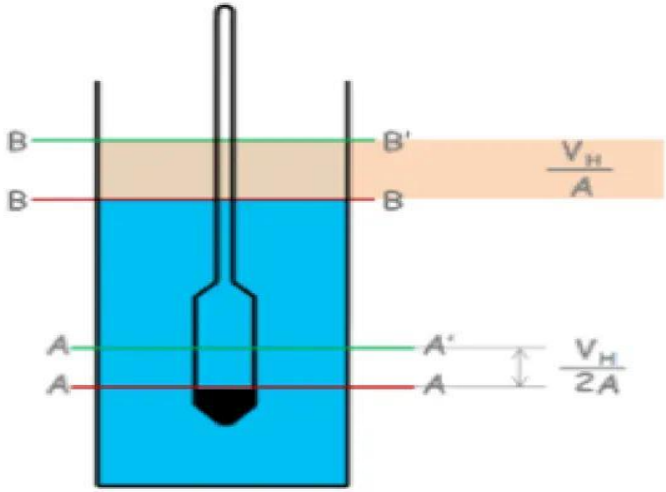
Stokes Law - Limitations

- Particles are large enough to be unaffected by the thermal (Brownian) motion of the fluid molecules.
- All particles are rigid, spherical, and smooth.
- All particles have the same density.
- The suspension is dilute enough that particles do not interfere with each other
- Fluid flow around the particles is laminar. That means no particle exceeds the critical velocity for the onset of turbulence.
- In practice we know that soil particles are neither spherical nor smooth. Hence the diameter calculated from Stokes law does not necessarily correspond to the actual dimensions of the particles. We rather receive an effective or equivalent settling diameter.

Methods of Sedimentation Analysis



Pipette Method



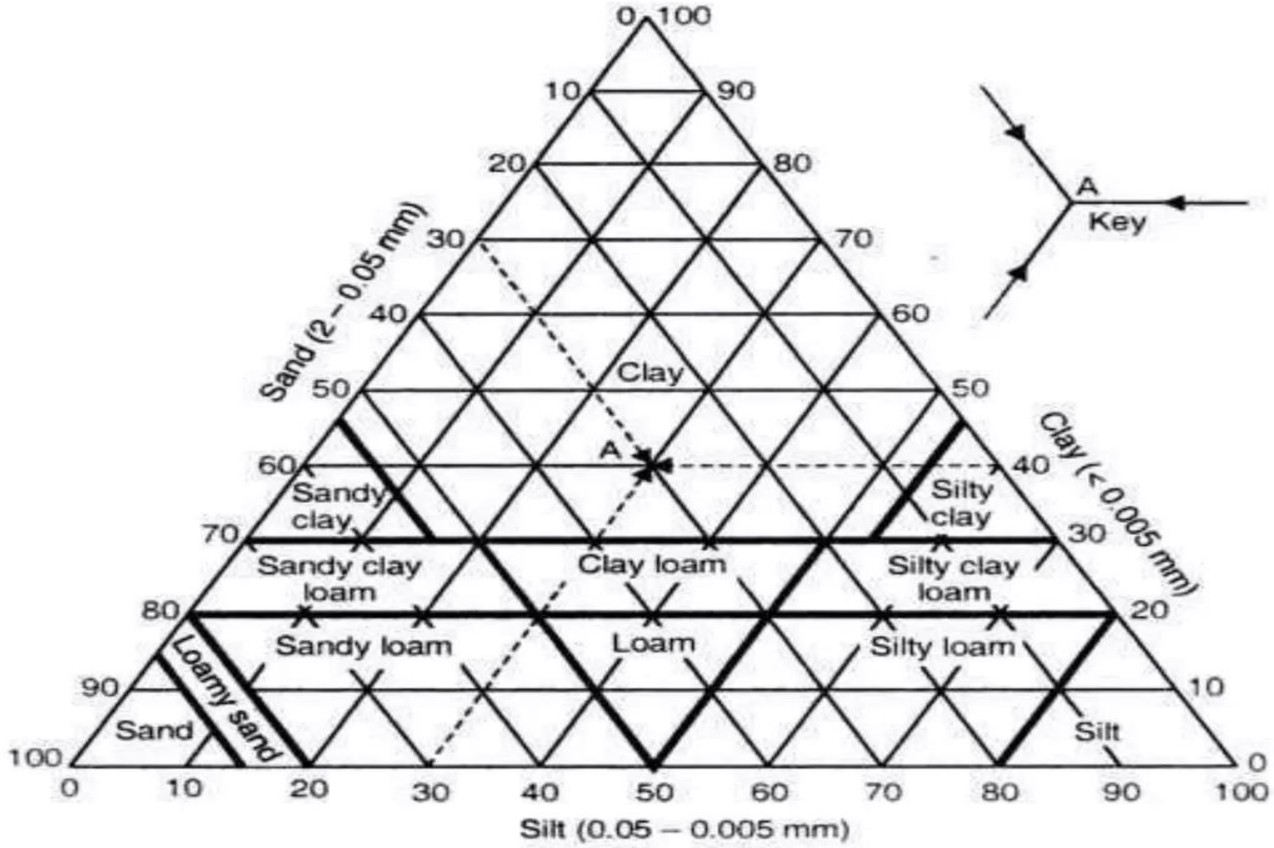
Hydrometer Method

(ii) Textural Classification of Soil

The classification of soil exclusively based on particle size and their percentage distribution is known as textural classification system. This system specifically names the soil depending on the percentage of sand, silt and clay. The triangular charts are used to classify soil by this system.

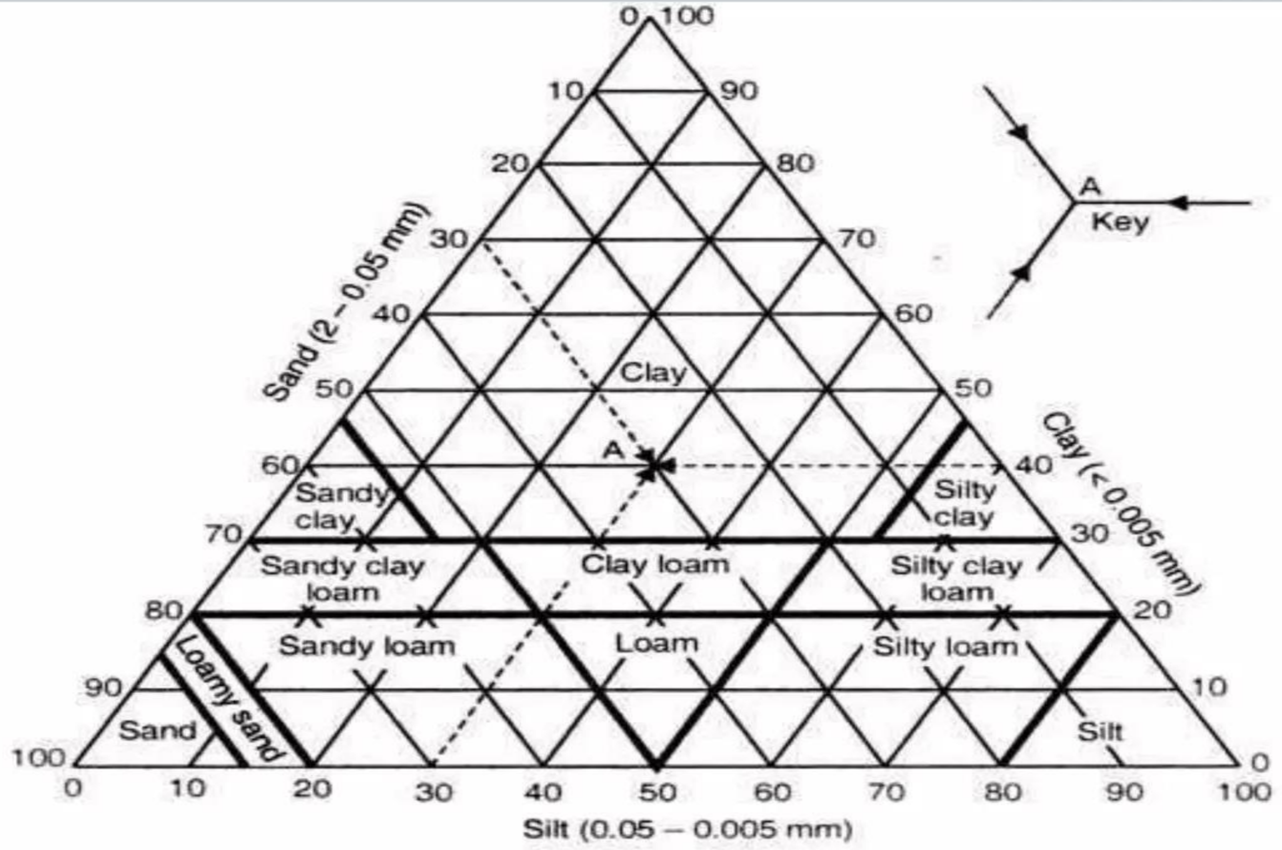
Figure – 1 shows the typical textural classification system.

Textural Classification Chart



TEXTURAL CLASSIFICATION CHART
(ADAPTED FROM U.S. PUBLIC ROADS ADMINISTRATION)

Example

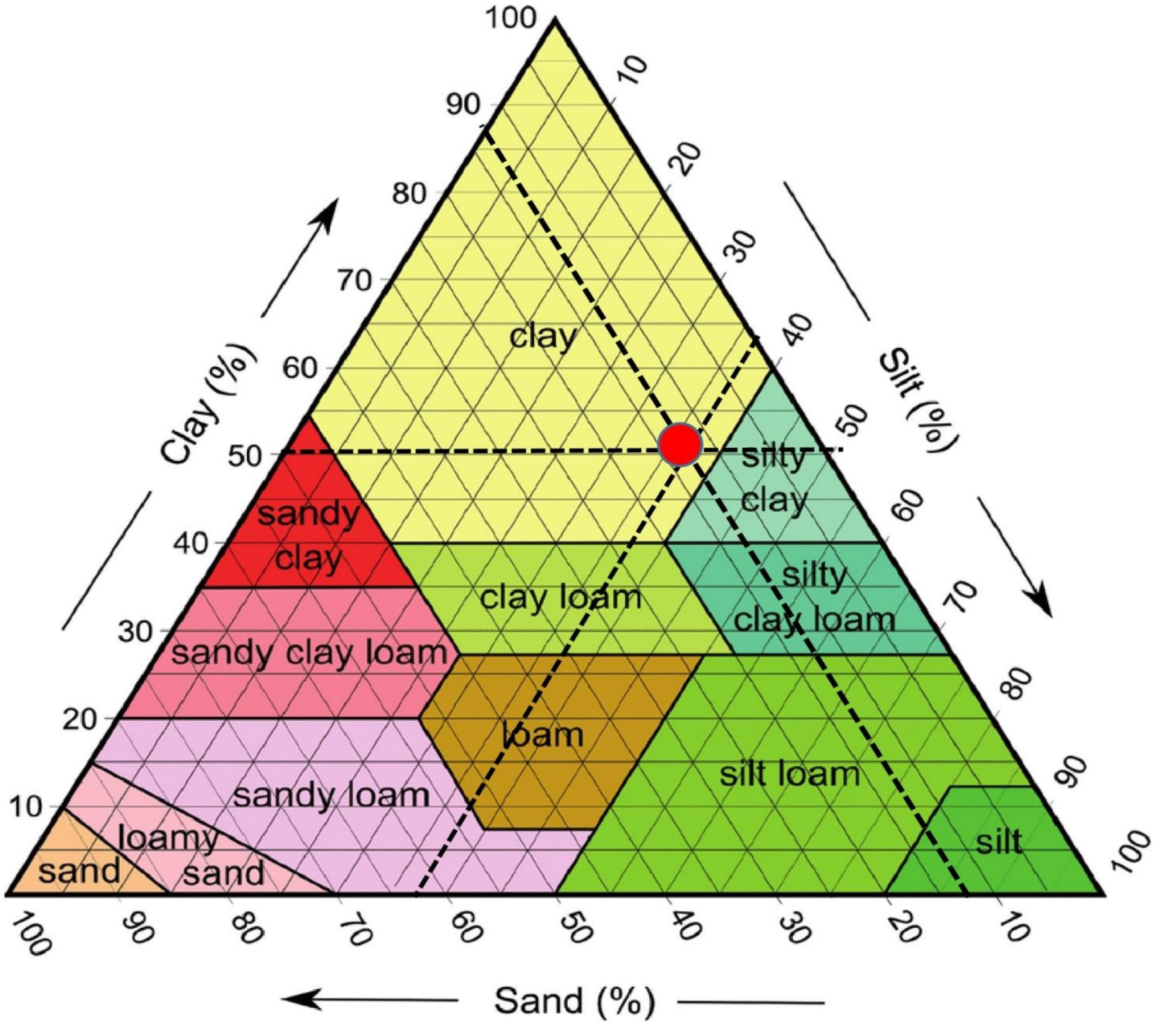


TEXTURAL CLASSIFICATION CHART
(ADAPTED FROM U.S. PUBLIC ROADS ADMINISTRATION)

30% sand
30% silt
40% clay

Soil Type

- Clay


100
90
80
70
60
50
40
30
20
10
Clay (%)
Silt (%)
Sand (%)
clay
sandy clay
silty clay
clay loam
silty clay loam
sandy clay loam
loam
silt loam
sandy loam
loamy sand
sand
silt

(iii) AASHTO classification system of Soil

80 20 4.75 0.075 0.002

ISI	C	F	C	M	F	Silt	Clay
	Gravel		Sand				

AASHTO classification, (table-2) is otherwise known as PRA classification system. It was originally developed in 1920 by the U.S. Bureau of Public Roads for the classification of soil for highway subgrade use.

This system is developed based on particle size and plasticity characteristics of soil mass. After some revision, this system was adopted by the AASHTO in 1945.

(iv) Unified Soil Classification System

Unified soil classification system was originally developed by Casagrande (1948) and was known as airfield classification system. It was adopted with some modification by the U.S. Bureau of Reclamation and the U.S. Corps of Engineers.

This system is based on both grain size and plasticity characteristics of soil. The same system with minor modification was adopted by ISI for general engineering purpose (IS 1498 – 1970).

FOUR MAJOR DIVISIONS

Course Grained

Organic Soil

Fine Grained

Peat

IS SOIL CLASSIFICATION SYSTEM

IS: 1498-1970 describes the Indian Standard on Classification and Identification of Soils for general engineering purposes. It is similar to U.S. Soil classification system except that the U.S. Soil classification system has 15 groups while this system has 18 groups Significant provisions of this system are given below:

Soils shall be broadly divided into three divisions:

1. Coarse-grained Soils: More than 50% of the total material by weight is larger than 75μ IS Sieve size.
2. Fine-grained Soils: More than 50% of the total material by weight is smaller than 75μ IS Sieve size.
3. Highly Organic Soils and Other Miscellaneous Soil Materials: These soils contain large percentages of fibrous organic matter, such as peat, and particles of decomposed vegetation. In addition, certain soils containing shells, concretions, cinders and other non-soil materials in sufficient quantities are also grouped in this division.

Coarse-grained soils shall be divided into two sub-divisions:

- Gravels: More than 50% of coarse fraction (+ 75 μ) is larger than 4.75 mm IS Sieve size.
- Sands: More than 50% of Coarse fraction (+ 75 μ) is smaller than 4.75 mm IS Sieve size.

Fine-grained soils shall be divided into three sub-divisions:

- ❖ Silts and clays of low compressibility: Liquid limit less than 35% (L).
- ❖ Silts and clays of medium compressibility: Liquid limit greater than 35% and less than 50% (I).
- ❖ Silts and clays of high compressibility: Liquid limit greater than 50 (H).

The coarse-grained soils shall be further sub-divided into eight basic soil groups, and the fine- grained soils into nine basic soil groups; highly organic soils and other miscellaneous soil materials shall be placed in one group

The symbols used to designate soil in this system are

GW – well graded gravel

GP – poorly graded gravel

GM – silty gravel

GC – clayey gravel

SW – well graded sand

SP – poorly graded sand

SM – silty sand

SC – clayey sand

CL – clay of low plasticity

CI – clay of medium plasticity

CH – clay of high plasticity

ML – silt of medium plasticity

MI – silt of medium plasticity

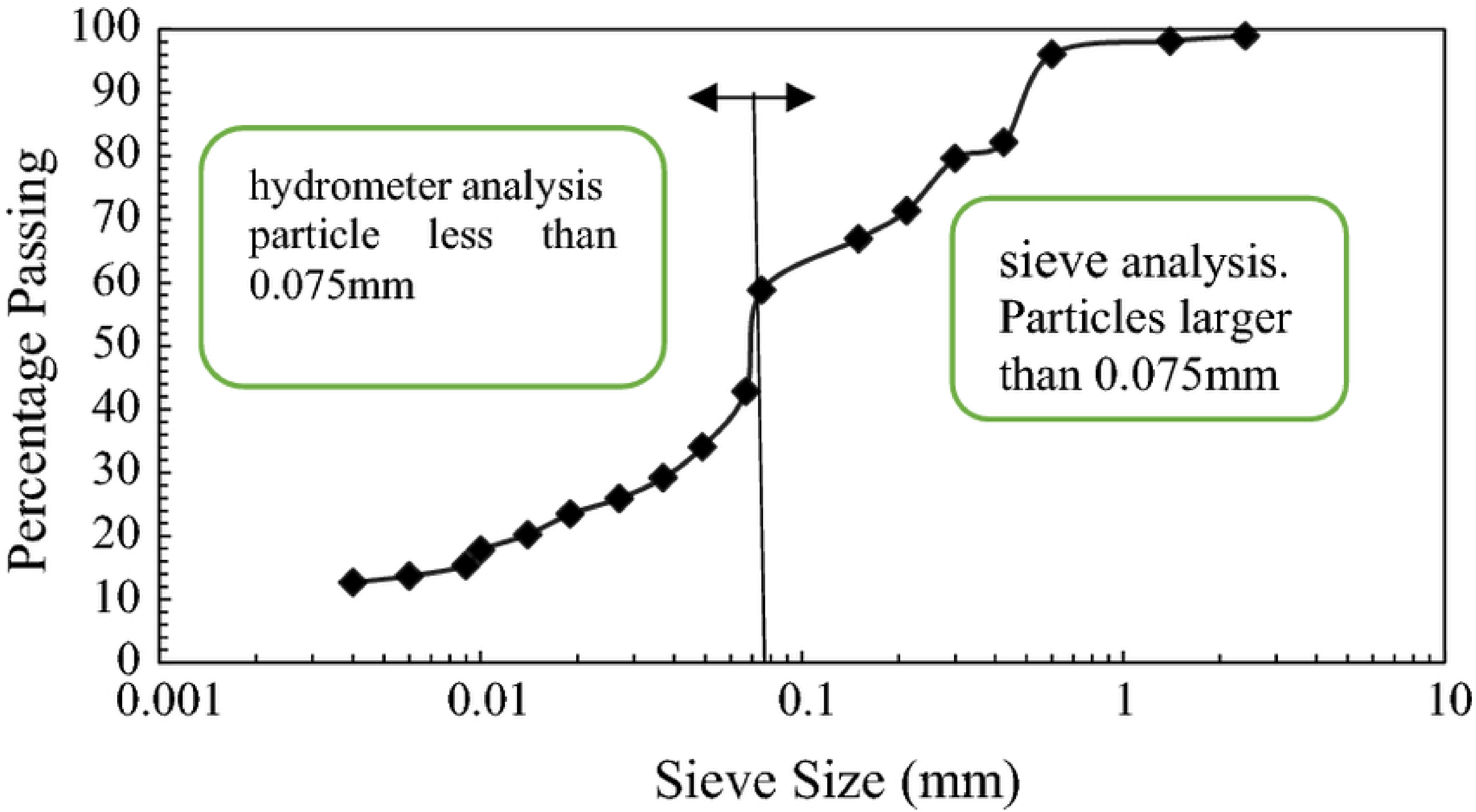
MH – silt of high plasticity

OL – organic silt and clays of low plasticity

OI – organic silt and clays of medium plasticity

OH – organic silt and clays of high plasticity

Pt – peat


Percentage Passing
100
90
80
70
60
50
40
30
20
10
0
0.001
0.01
0.1
1
10
Sieve Size (mm)
hydrometer analysis particle less than 0.075mm
sieve analysis. Particles larger than 0.075mm

Click the following video link

Hydrometer Analysis

% of Gravel, Sand, Silt and Clay from grain size analysis Curve

Soil-Foundation Interaction

Week 4

Pages 85-104

Cohesionless soil is a type of soil that doesn't stick together and is made up of particles that depend on friction to hold them together. Cohesionless soils are also known as non cohesive or granular soils.

Cohesionless soil/Sandy Soil

Characteristics

Friction

Cohesionless soils are free-running and their strength depends on the friction between their particles.

Water transmission

Cohesionless soils transmit water easily and have a relatively high hydraulic conductivity.

Shear strength

Cohesionless soils have shear strength that only has a friction component.

Grain size

Cohesionless soils are coarse-grained, meaning they are made up of larger grains.

Examples sand and gravel.

Cohesive soil is a type of soil that sticks together because its particles are attracted to each other. Cohesive soils are fine-grained, low-strength, and easily deformed.

Importance

Cohesive soils are important to consider when studying erosion and stormwater runoff because they are less likely to erode than other types of soil.

Cohesive soil/clayey Soil

Characteristics

Plasticity: Cohesive soils can be molded and retain their shape when moist

Shrinkage and expansion: Cohesive soils are prone to shrinkage and expansion because they hold water

Hardness: Cohesive soils can be hard when dry, similar to cement

Cohesion: Cohesive soils exhibit significant cohesion when submerged

Examples

clay, silty clay, sandy clay, clayey silt, organic clay, peat, and most loam.

TABLE 2-DEWATERING ATTRIBUTES OF COHESIVE AND NON COHESIVE SOILS	
COHESIVE -loam, clay, silt	NON COHESIVE-sands and gravels
water has an electronic bond to clay particles saturated clays have some free water also	water is not part of the soil particle
does not give up or drain water easily	gives up water easily
dewater from inside the excavation using drainage path and sumps	dewater from outside the excavation using dewater wells
stronger	dewatering changes the soil strength to stronger
dewatering is usually much less expensive	depending on permeability can be very expensive or impossible to dewater by pumping
soft clays are susceptable to bottom heave	wet sands are susceptable to piping and boiling
dewatering causes settlement	dewatering causes settlement

Differences between cohesive and cohesionless soil

(Click this link)

DIFFERENCE BETWEEN CLAY SOIL AND SANDY SOIL

CLAY SOIL

- Composed of fine mineral particles that are less than 0.002 mm in diameter.
- Holds onto water and nutrients well due to its dense structure.
- Drainage is often poor and the soil can become waterlogged.
- Heavier and more compact than sandy soil, making it harder to work with.
- Can become hard and compacted when dry, making it difficult for roots to penetrate.

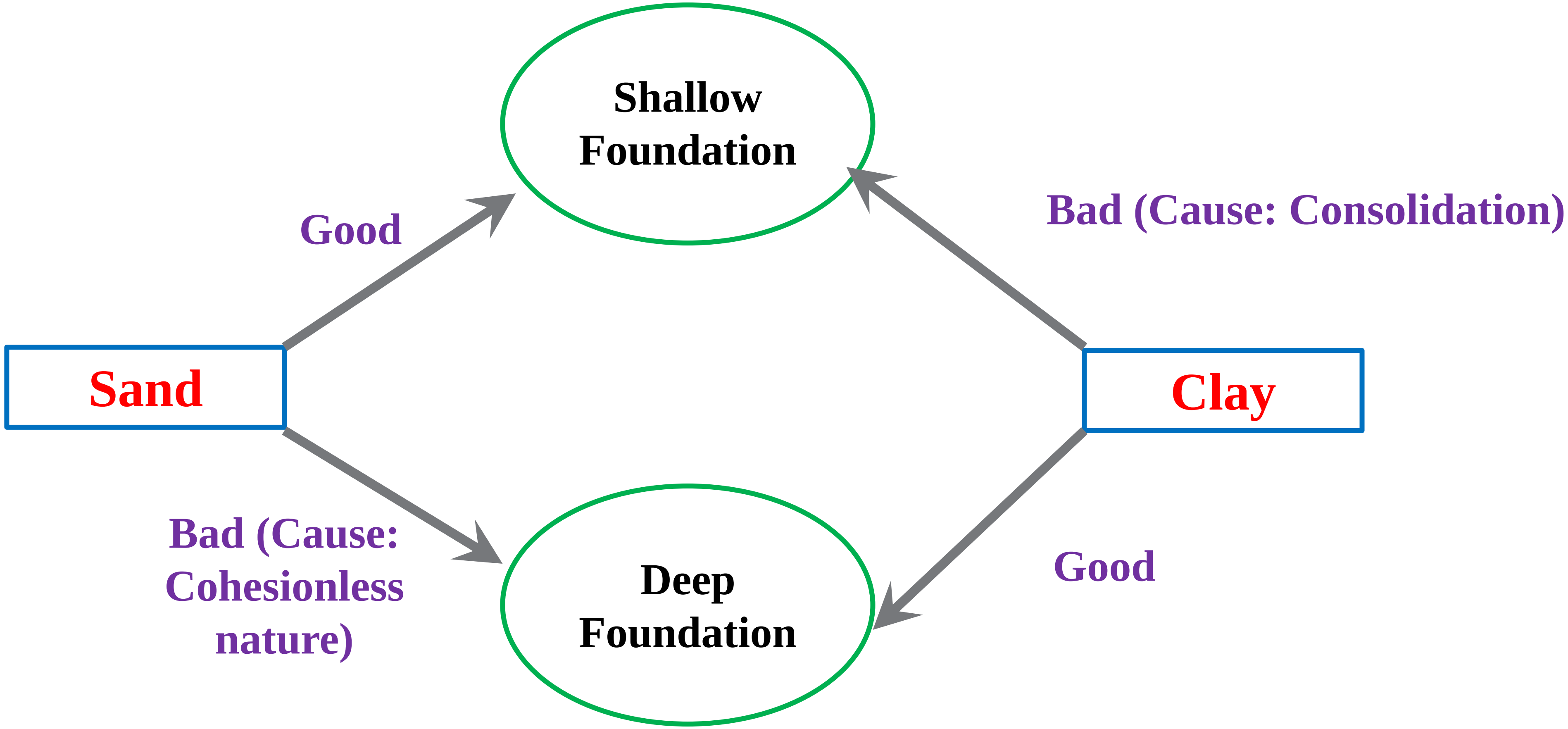
SANDY SOIL

- Composed of larger mineral particles that range in size from 0.05 mm to 2 mm.
- Has a low water-holding capacity and nutrients can easily leach away.
- Has good drainage and is less likely to become waterlogged.
- Lighter and easier to work with due to its loose structure.
- Doesn't compact easily and allows roots to penetrate easily.
- Warms up quickly in the spring and cools down quickly in the fall, making it warmer overall.

Clay Soil means Soft soil

Clay Soil means Soft soil

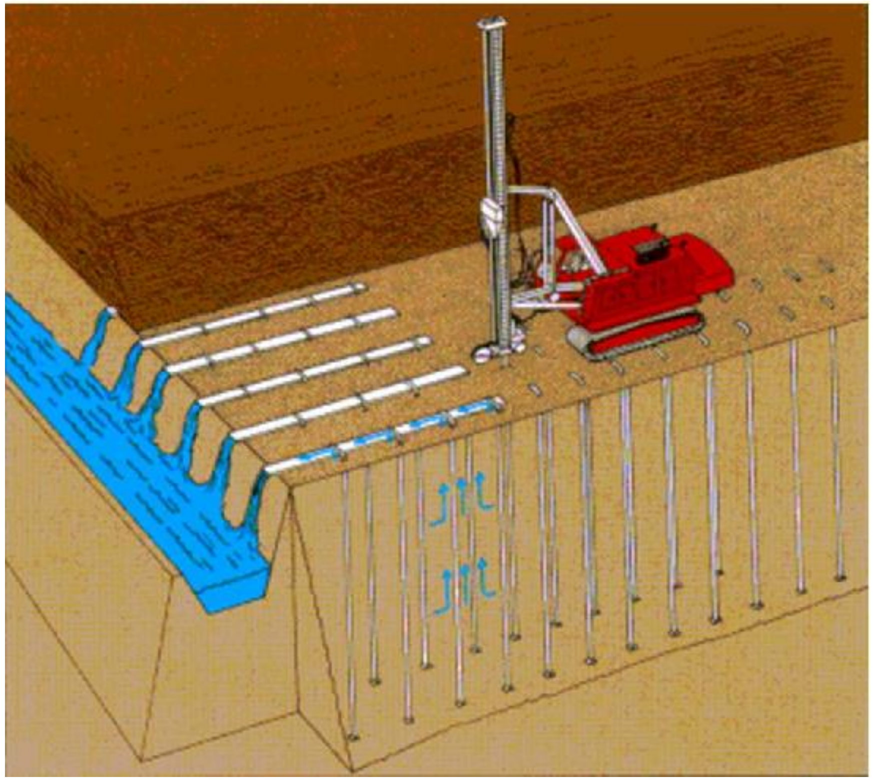
Which soil is suitable for which type of foundation…



Explanation….. Given in face to face class…… so must present this class

Soil Improvement techniques

A prefabricated vertical drain (PVD) is a pipe made of plastic and geotextile that helps improve soft soils. PVDs are also called wick drains or band drains.



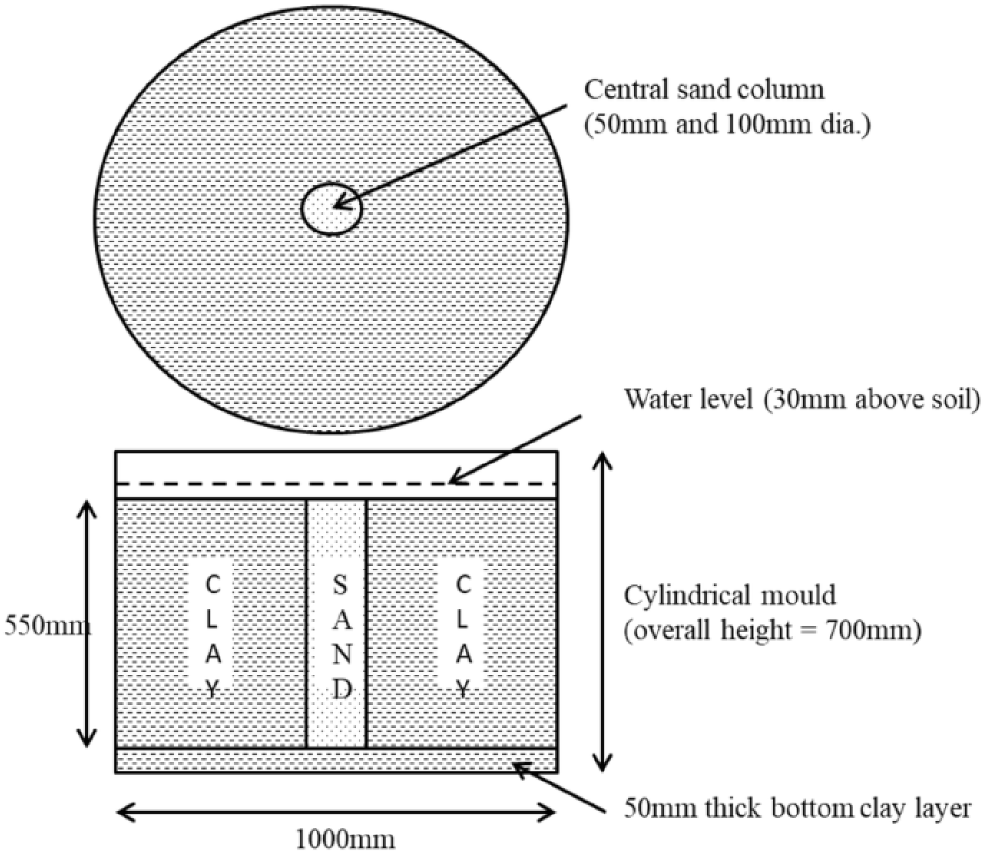
Prefabricated vertical drain

How they work

- PVDs are installed in soft soils, such as clay, to help them consolidate faster
- The PVDs create a path for water to flow out of the soil
- This above process is called consolidation
- PVDs are often used with surcharging, which is when a temporary load is applied to the ground
- The surcharge increases pore water pressure, which forces water out of the soil through the PVDs

Soil Improvement techniques

A **sand column** is a column of sand that is installed in soft soil to improve its bearing capacity. Sand columns can also help prevent liquefaction and seepage.



Sand column

Functions of sand columns

Prevent seepage: Sand columns prevent fine soils from escaping through seepage water.

Reduce liquefaction: Sand columns reduce damage from liquefaction of saturated granular soils during earthquakes.

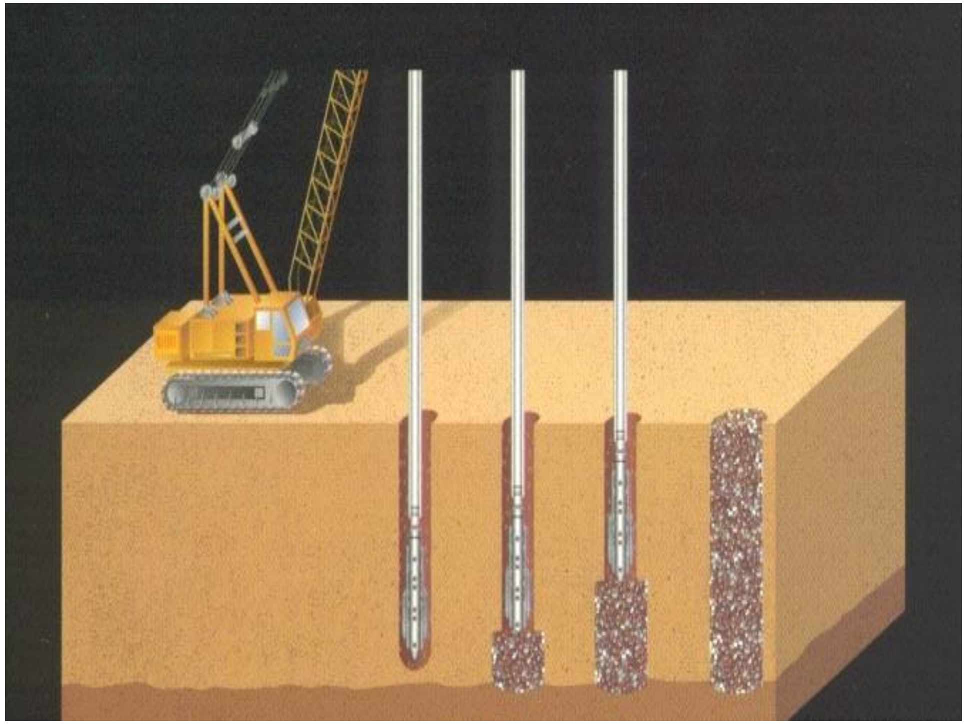
Improve bearing capacity: Sand columns improve the ultimate bearing capacity of soft soil.

Accelerate stabilization: Sand columns accelerate the stabilization of soft soil.

This process increases the rate of consolidation and improves the bearing capacity of the soil.

Soil Improvement techniques

Stone columns are a ground improvement technique that uses compacted stone to strengthen soil. They are used to reduce settlement, increase bearing capacity, and improve slope stability. stone columns can be used in loose sandy soil. Stone columns are a cost-effective way to improve the stability of loose or soft soils, including sandy soils.



Stone column

Benefits of stone columns

•Stone columns can increase the bearing capacity of the soil.

•They can reduce the total and differential settlements of the soil.

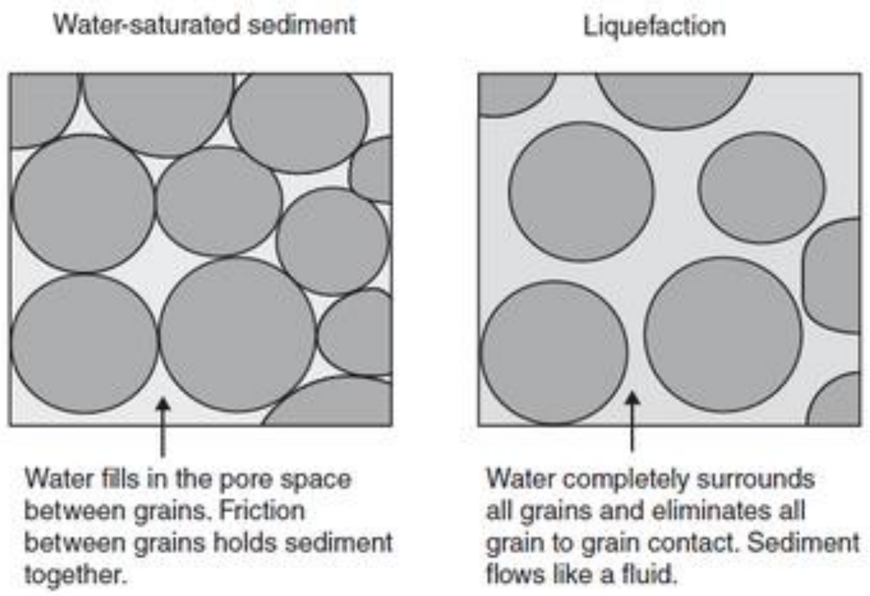
•They can reduce the liquefaction potential of sands.

•They can prevent ground movement.

•This columns act as vertical drains, allowing excess pore water pressure to dissipate quickly.

•Stone columns can resist lateral loads to some degree, but they may not be able to withstand high lateral loads or seismic forces.

Soil liquefaction is a natural hazard that occurs when soil loses its strength and stiffness and behaves like a liquid. It happens when saturated or partially saturated soil is subjected to a sudden change in stress, such as shaking from an earthquake.



Causes

Soil liquefaction is caused by increased pore pressure and reduced effective stress. It's most likely to happen in loosely packed, water-logged soils, like sandy or silty soils.

Effects

When soil liquefies, it can cause buildings and other structures to sink, tilt, float, or slide. It can also lead to ground settlement and land sliding.

Examples

Some examples of soil liquefaction include:

Niigata, Japan: The 1964 Niigata earthquake caused widespread liquefaction that destroyed many buildings.

San Francisco: The 1989 Loma Prieta earthquake caused liquefaction in the Marina District, damaging or destroying more than 30 buildings.

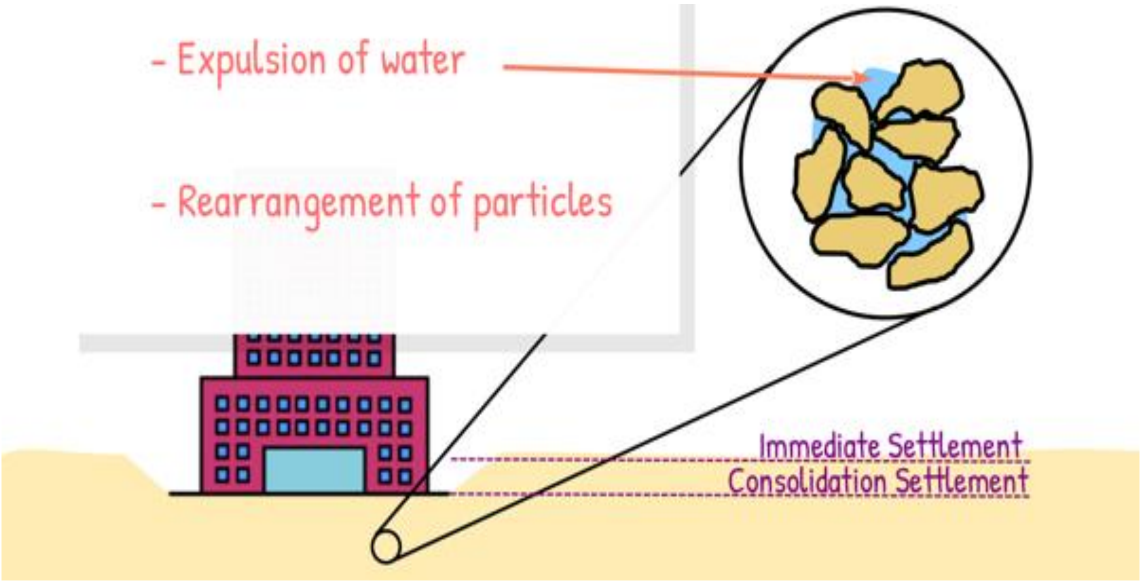
Deadly Nature of Soil Liquefaction

Consolidation

In geotechnical engineering, **consolidation** is the process of soil volume reduction due to applied stress. This can be caused by natural processes or human activity. It's a mechanical process that occurs when water is expelled from soil voids.

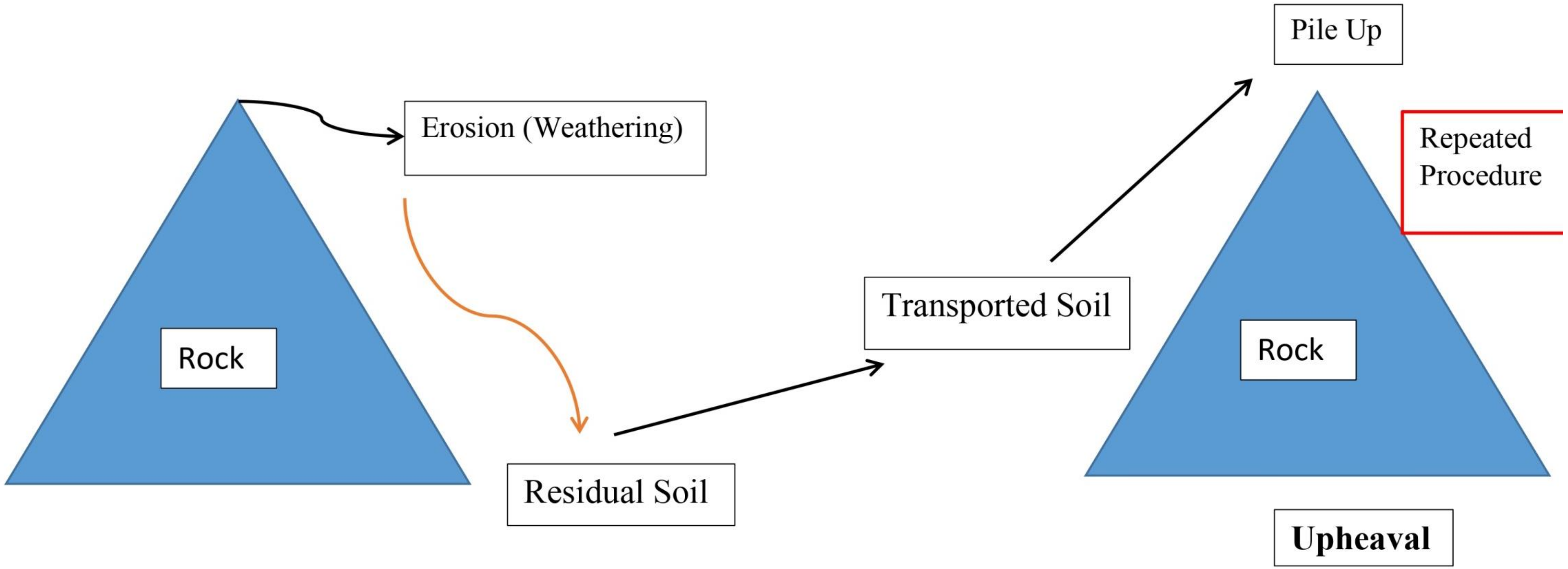
Importance

Consolidation is important in geotechnical engineering because it can affect the stability and durability of structures built on top of soil.



Consolidation

Geological Soil Formation



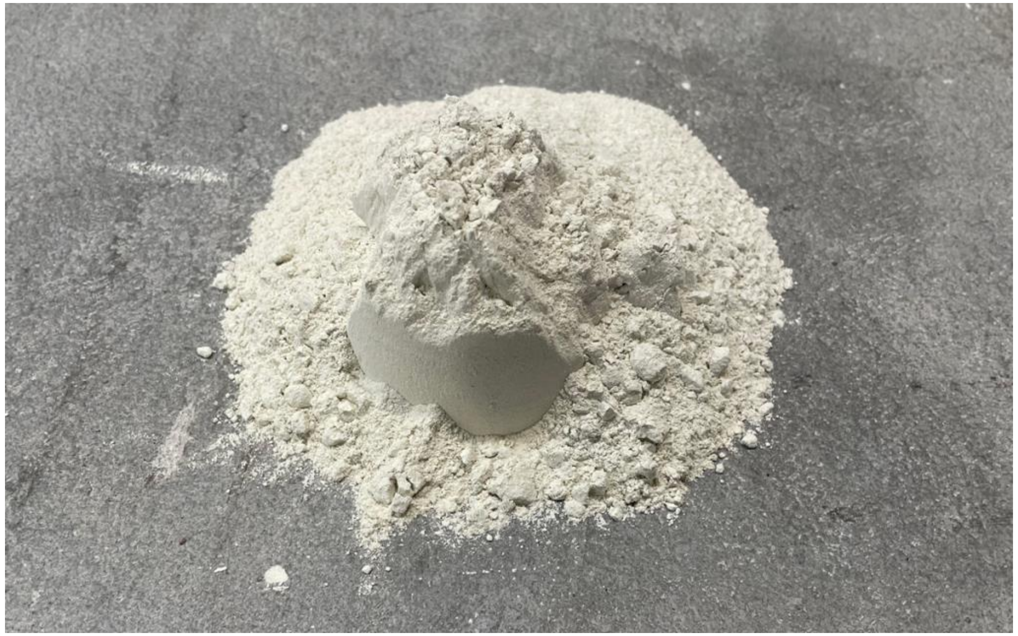
Weathering Effect

Residual Soil

Residual soil can be defined as a soil material which is the result of weathering and decomposition of rocks that has not been transported from its original place. There are many types of residual soil which are discussed below.

a) Bentonite soil:

Bentonite is an absorbent swelling clay consisting mostly of montmorillonite. It usually forms from weathering of volcanic ash in seawater, which converts the volcanic glass present in the ash to clay minerals. Bentonite beds are white or pale blue or green in fresh exposures, turning to a cream color and then yellow, red, or brown as the exposure is weathered further.



Bentonite soil

b) Black cotton soil:

Black cotton soils are medium to high compressibility inorganic clays and form a significant soil group in India. They're distinguished by high properties of shrinkage and swelling. This Black Cotton soil mostly occurs in the central and western parts and covers about 20 percent of India's total territory.

c) Laterite soil:

Laterite is both a soil and a rock type rich in iron and aluminium and is commonly considered to have formed in hot and wet tropical areas. Nearly all laterites are of rusty-red coloration, because of high iron oxide content. They develop by intensive and prolonged weathering of the underlying parent rock, usually when there are conditions of high temperatures and heavy rainfall with alternate wet and dry periods.

Black cotton soil

Laterite soil

Transported Soil:

Transported soils are those materials that have been moved from their place of origin, by gravity, wind, water, glaciers, or human activity – either singularly or in combination. There are many types of transported soil which are discussed below.

a) Alluvial Soil:

Alluvial soils are soils deposited by surface water (Especially River). This can be found along rivers, in floodplains and deltas, stream terraces, and areas called alluvial fans. Approximately 28% of Indian soils are alluvial soils.

b) Aeolian Soil:

A type of soil that is transported from one place to another by the wind is called "Aeolian soil." Generally, eolian (or aeolian) sediments are wind-deposited materials that consist primarily of sand or silt-sized particles. These materials tend to be extremely well sorted and free of coarse fragments. Some rounding and frosting of mineral grains is detectable. Example: sand dune.

Aeolian Soil

c) Glacial Soil:

Soil composed of boulder clays, sands, etc., which were formed by the action of ice during the Pleistocene age. Glacial soil is found in high Himalayan regions having rocky terrain with ice blocks. They are covered with snow for most of the year. The soil is much less exposed to the air due to snow cover.

Glacial Soil

d) Lacustrine Soil:

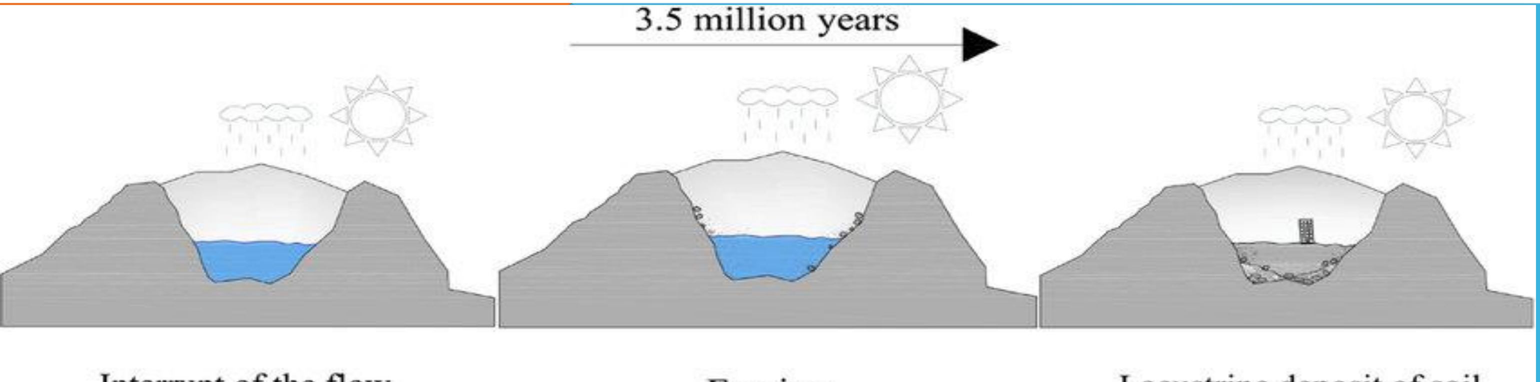
Lacustrine soils are soils that formed on a lake bottom. After the lake waters have receded, the remaining soils are called lacustrine soil. This is uniform in texture but variable in chemical composition and that has been formed by deposits in lakes which have become extinct. Lakes deposits are mostly clays and silts.

Lacustrine Soil

e) Marine soil:

Soil deposits on ocean beds are known as marine deposits or marine soils. Though oceans can be very violent, the seabeds are very calm for the most part. Hence, very small particles would deposit on seabeds. The texture and composition depends on the proximity to land and biological matter.

Marine Soil



f) Colluvial Soil/Talus Soil:

Colluvium is defined as "a superficial deposit" which is transported predominantly by gravity and contains 50% of material > 60 mm in size (i.e., cobbles) from a hill. In general, colluvial soils such as talus have been deposited by the gravity. Talus consists of irregular, coarse particles. It is a good source of broken rock pieces and coarse-grained soils for many engineering works.

Colluvial Soil

g) Tough soil/Volcanic ash soil:

A type of soil that is transported from the area of volcanic explosion by the wind or water is called " Tough soil/Volcanic ash soil".

Volcanic ash Soil

Organic Soil:

Organic soil is a soil that is created by the decomposition of plant and animal materials to create a nutrient and mineral rich mini-ecosystem with microorganisms that feed and breathe life back into the soil.

a) Muck Soil: (OC = 45-60%)

b) Peat Soil: (OC>70%)

Organic Soil

When Organic content (OC) > 20%, this soil is not recommended for civil engineering constructional purposes

Soil Structure and Clay Mineralogy

Week 5

Pages 106-108

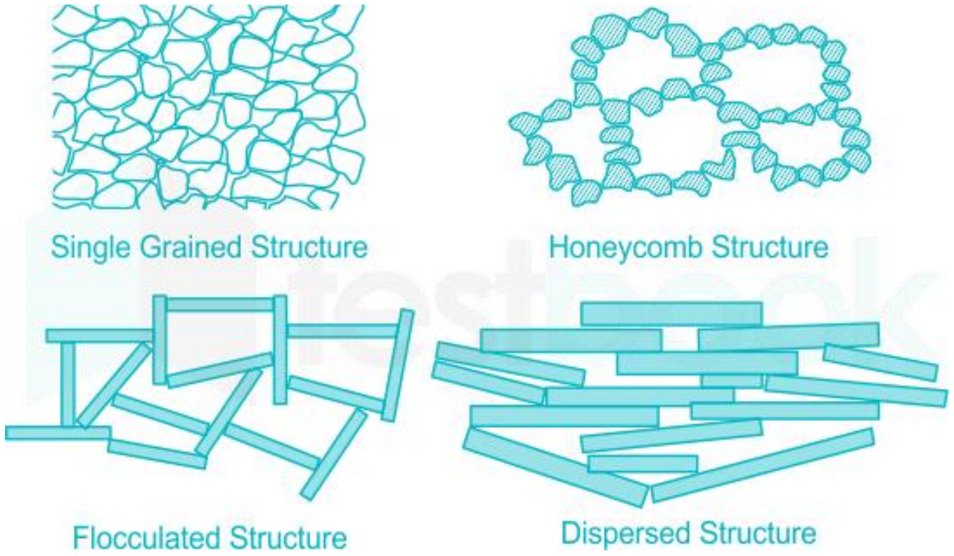
Single grain structure is a type of soil structure that occurs in coarse-grained soils, while honeycomb structure occurs in silt deposits.

Single grain structure

- Occurs in coarse-grained soils like sand
- Particles settle independently without contact
- Gravitational force is predominant over surface electric forces
- Usually associated with Textural Group I soils

Honeycomb structure

- Occurs in silt deposits
- Particles attract to each other and form arches that combine into a honeycomb pattern
- Both gravitational and surface electric forces are important
- Can be found in fine sand and silt deposits
- Has a high void ratio and can carry a relatively heavy load



Types of Soil

Flocculent and dispersed structures are types of soil structure that are formed by the arrangement of fine particles in soil. Flocculent structures are formed when particles aggregate together, while dispersed structures are formed when particles settle individually.

Flocculent structure

Formation: Clay particles in water come together due to electrostatic attraction, forming visible flocs

Characteristics: Particles are held together edge-to-edge or edges-to-face

Properties: Lightweight, high void ratio, and formed due to attractive forces

Importance: Water moves through large pores, and plant roots grow in pore space

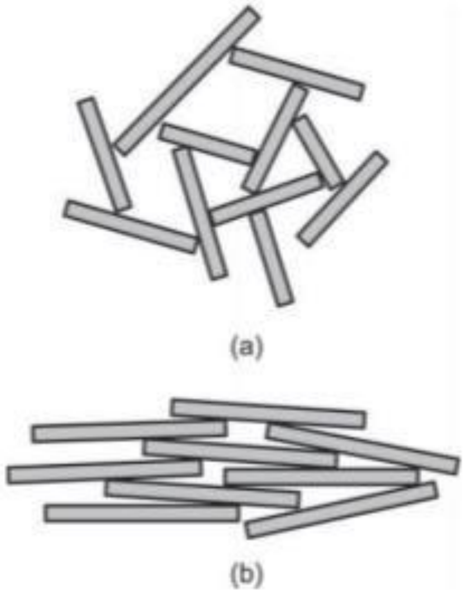
Dispersed structure

Formation: Particles settle individually, with particles pointing face-to-face

Characteristics: Particles are more or less parallel to each other, and formed due to repulsive forces

Properties: Relatively fewer voids than flocculent structures

Importance: Can plug soil pores and impede water movement and soil drainage



Types of Soil

Montmorillonite, illite, and kaolinite are all clay minerals with different properties and uses.

Montmorillonite

- **Characteristics**: A swelling clay that absorbs water and other fluids between layers of atoms
- **Uses**: Used in masks, poultices, kitty litter, and to clean up spilled oil
- **Composition**: A member of the smectite group of clays

Illite

- **Characteristics**: A common mica and clay mineral that is grayish-white to silvery-gray in color
- **Composition**: Similar to muscovite, but with more silicon and less potassium
- **Uses**: A major component of sedimentary rocks older than 100 million years

Kaolinite

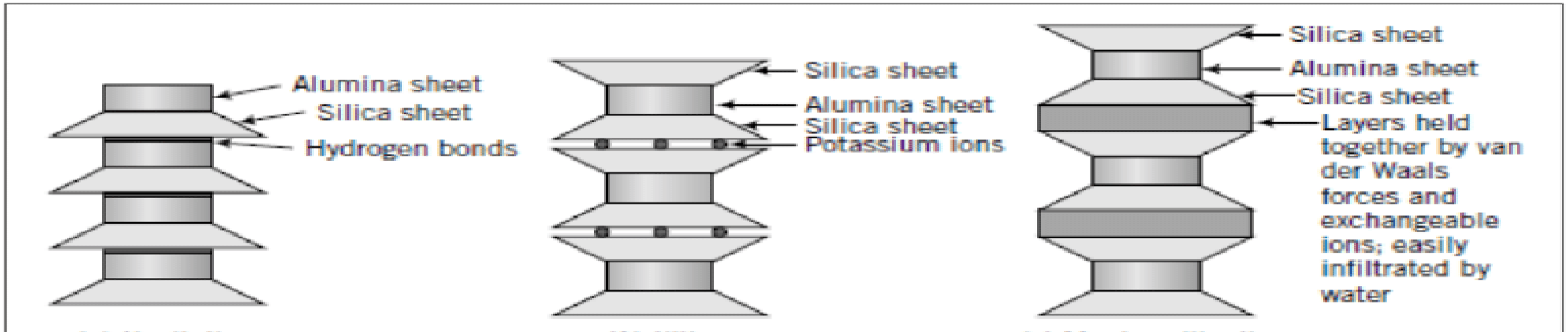
- **Characteristics**: The main clay used to make ceramic ware because it remains white when fired in a kiln
- **Uses**: Used to make ceramic ware

Uses in animal feed

- A mixture of illite, montmorillonite, and kaolinite can be used as a binder and anticaking agent in animal feed

Uses in the environment

- Clays like montmorillonite and vermiculite can be used to clean up spilled oil and lighten potting soil



Phase Relationship of Soil

Week 6-7

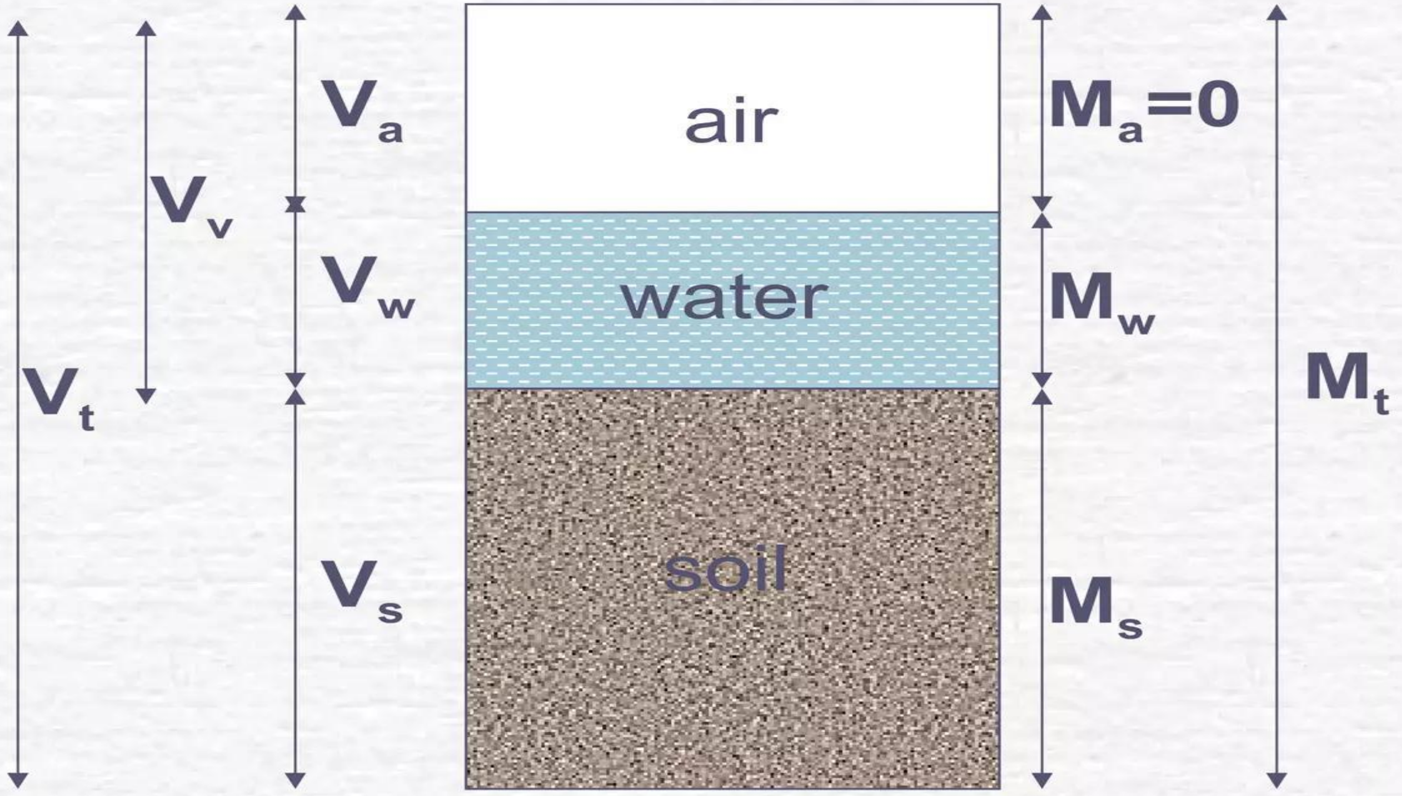
Pages 106-143

Objectives

To compute the masses (or weights) and volumes of the three different phases.

Notation

M = mass or weight
V = volume
s = soil grains
w = water
a = air
v = voids
t = total



Phase Diagram

2

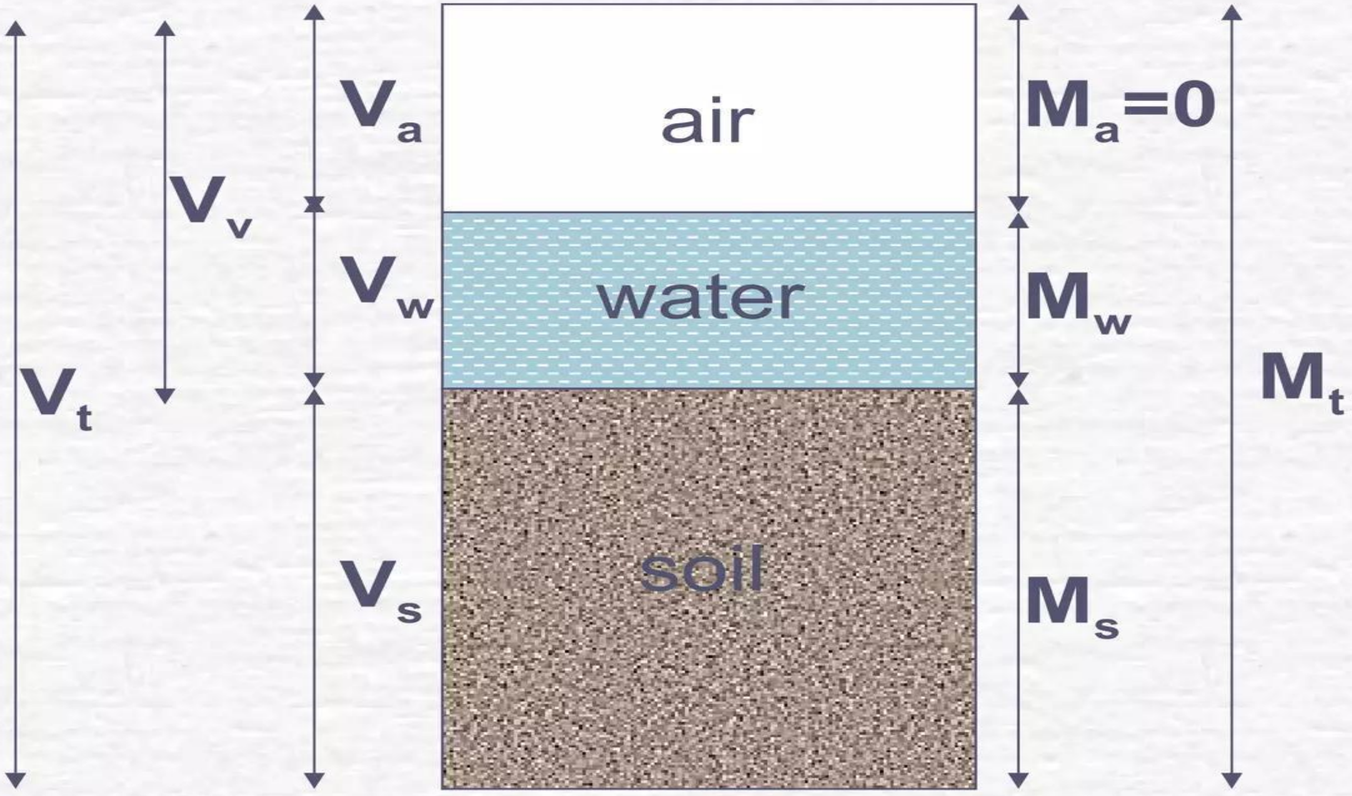
Definitions

Water content (w) is a measure of the water present in the soil.

$$w = \frac{M_W}{M_S} \times 100\%$$

Expressed as percentage.

Range = 0 – 100+%.

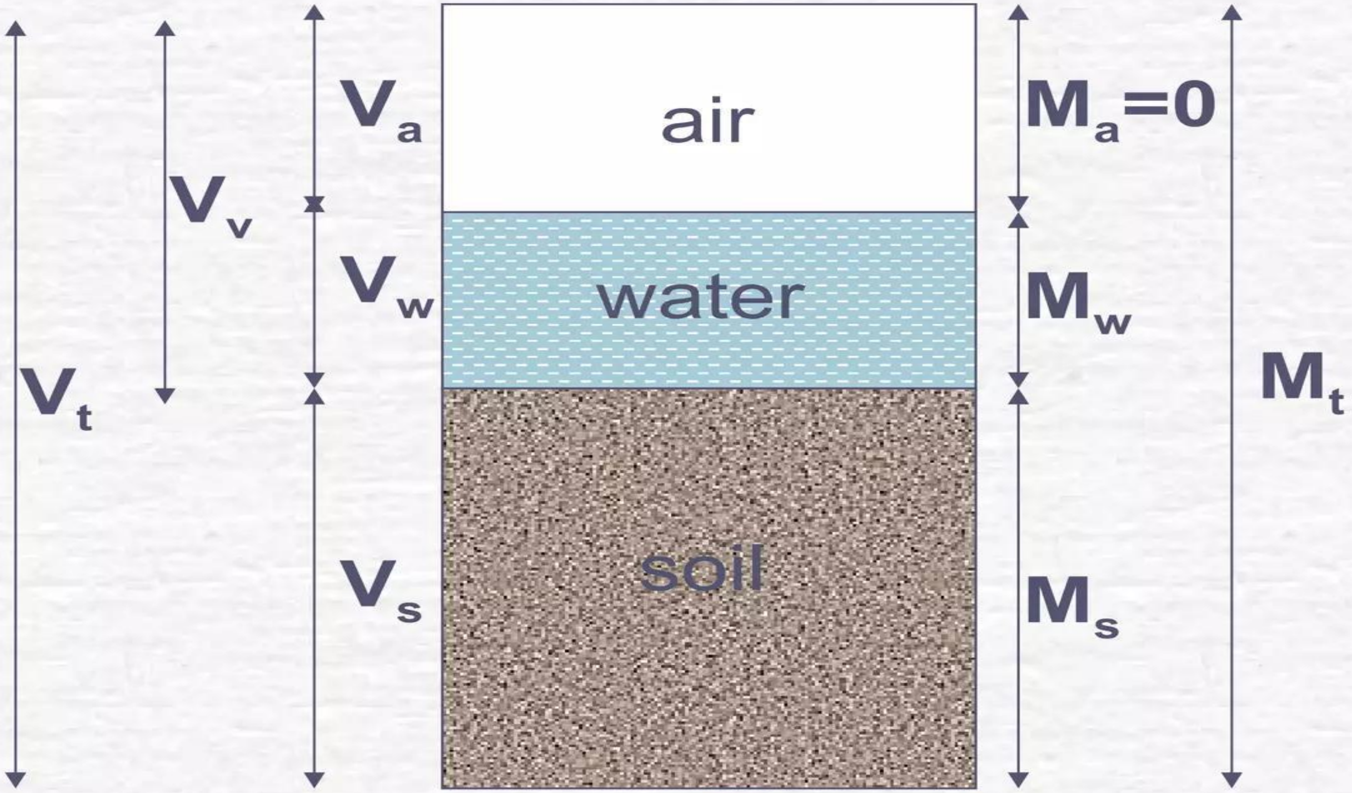


Phase Diagram

Definitions

Void ratio (e) is a measure of the void volume.

$$e = \frac{V_V}{V_S}$$



Phase Diagram

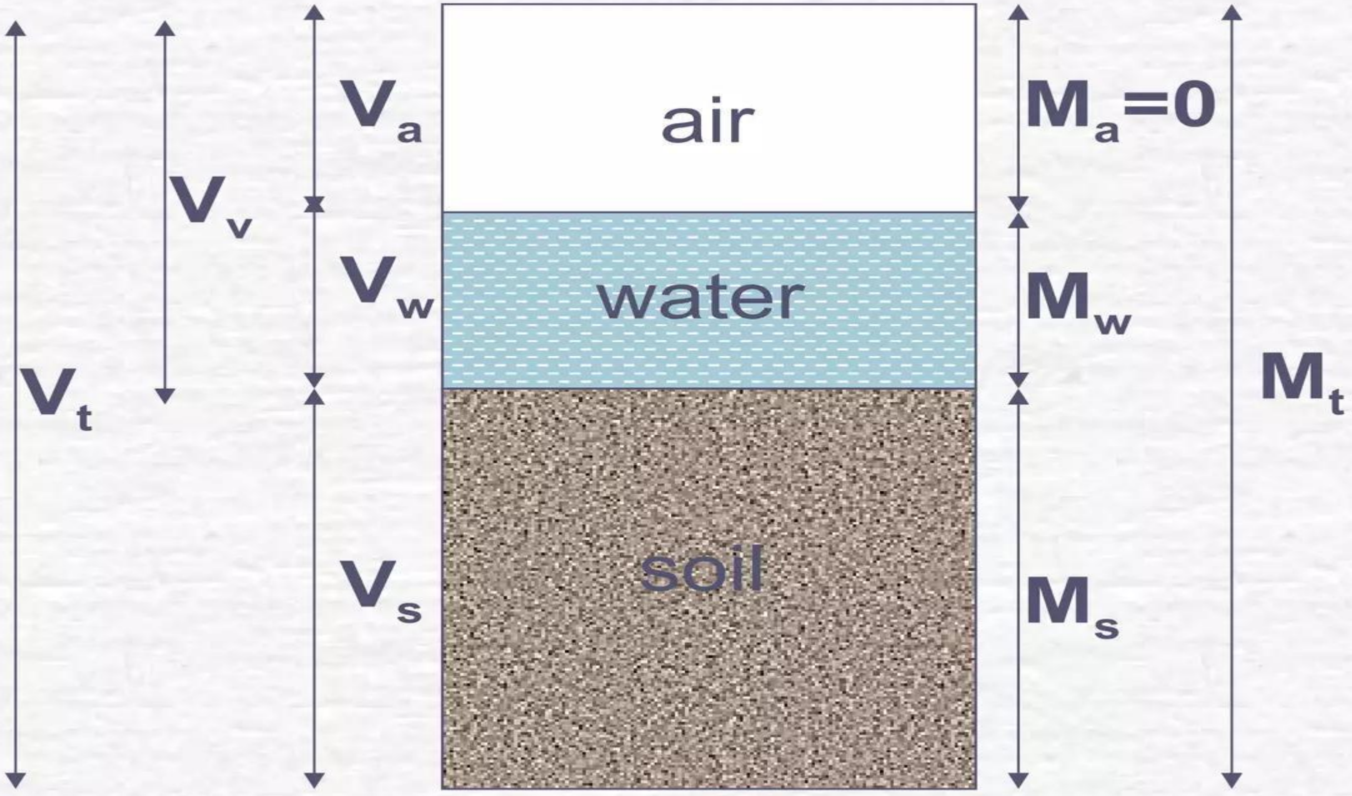
4

Definitions

Porosity (n) is also a measure of the void volume, expressed as a percentage.

$$n = \frac{V_V}{V_T} \times 100\%$$

Theoretical range: 0 – 100%



Phase Diagram

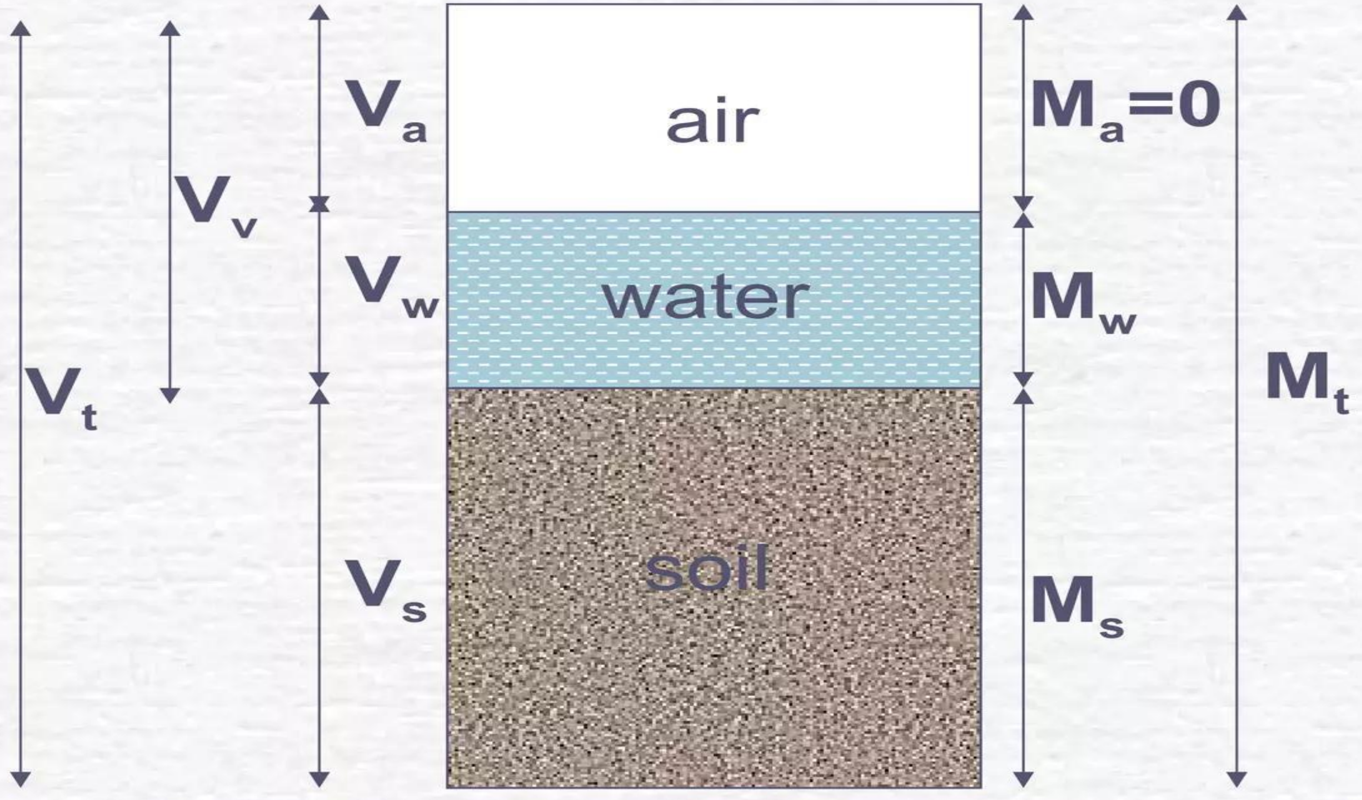
5

Definitions

Degree of saturation (S) is the percentage of the void volume filled by water.

$$S = \frac{V_W}{V_V} \times 100\%$$

Range: 0 – 100%



Phase Diagram

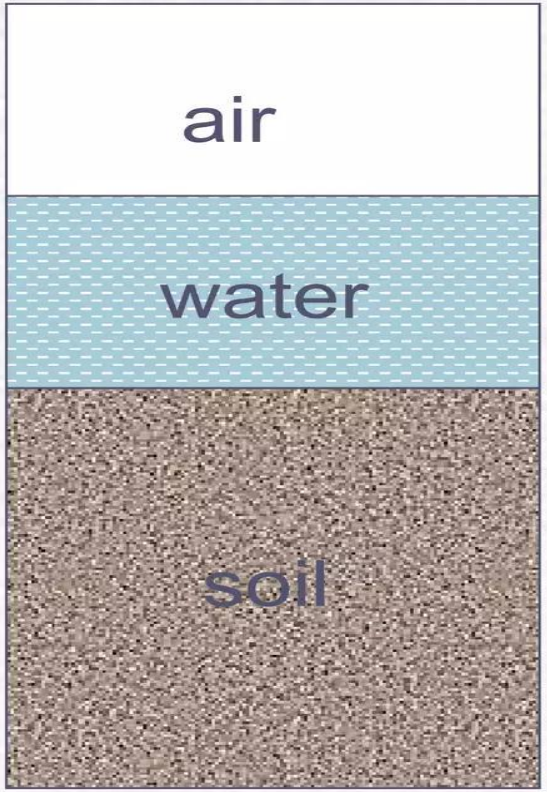
A Simple Example

In this illustration,

$e = 1$

$n = 50\%$

$S = 50\%$

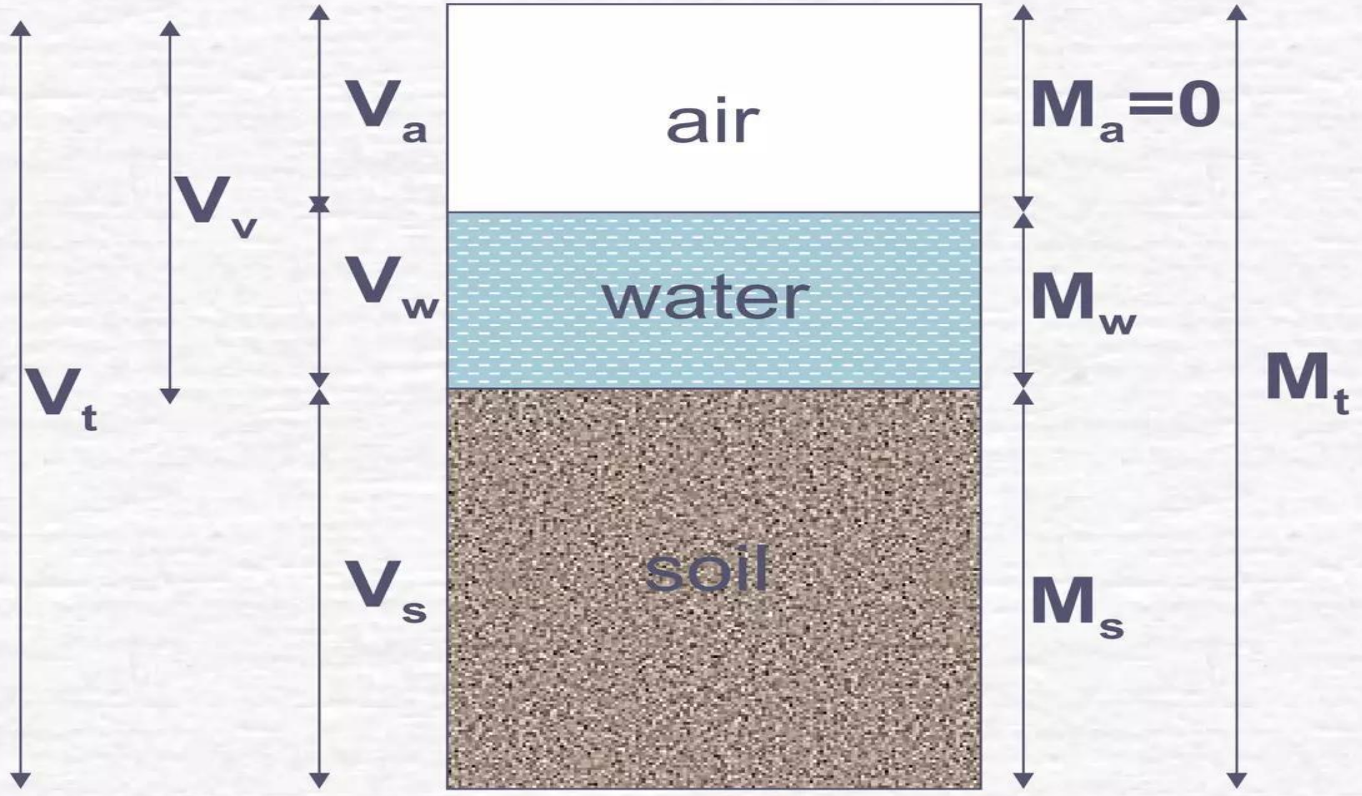


Definitions

Bulk density (ρ_m) is the density of the soil in the current state.

$$\rho_m = \frac{M_T}{V_T}$$

Units: t/m³, g/ml, kg/m³



Phase Diagram

8

Definitions

Saturated density (ρ_{sat}) is the density of the soil when the voids are filled with water.

Submerged density (ρ') is the effective density of the soil when it is submerged.

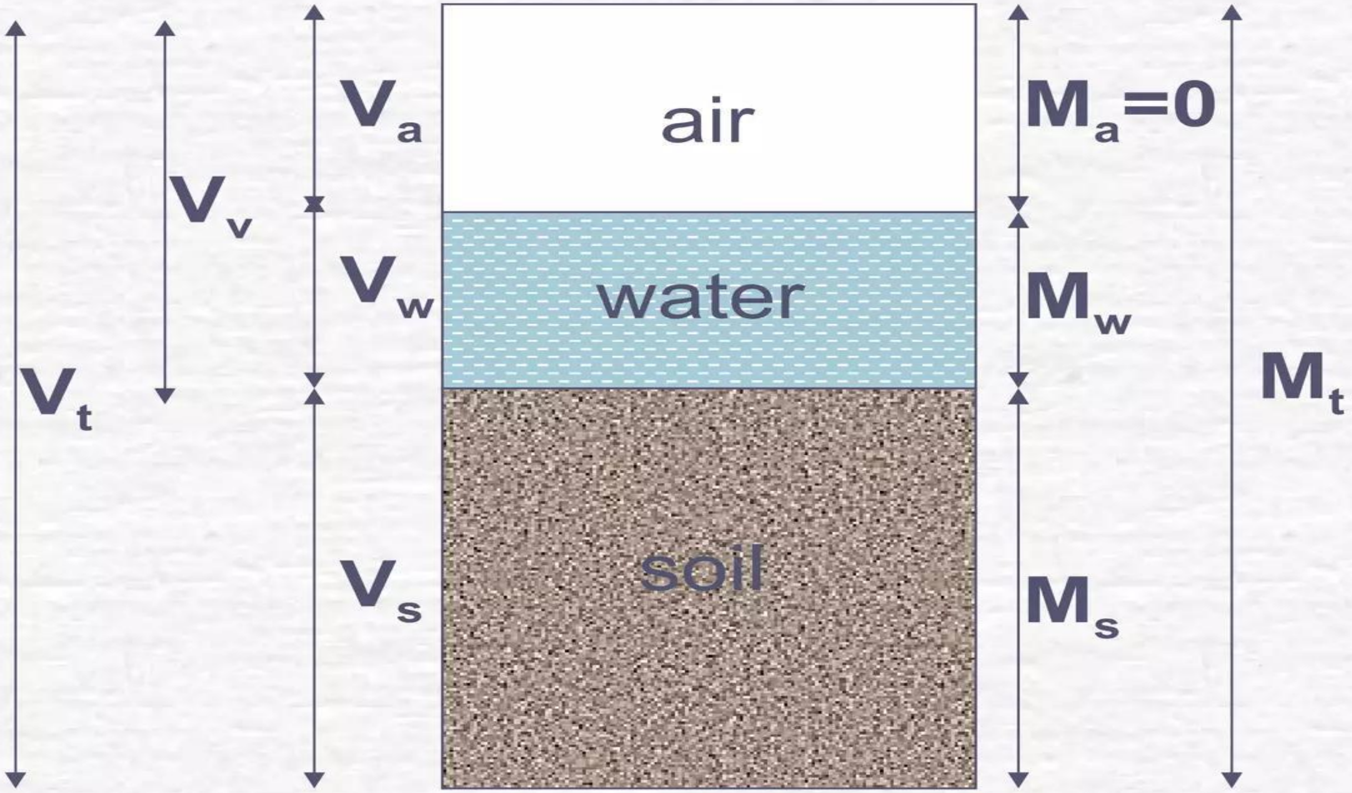
$$\rho' = \rho_{sat} - \rho_w$$

Definitions

Dry density (ρ_d) is the density of the soil in dry state.

$$\rho_d = \frac{M_S}{V_T}$$

Units: t/m^3, g/ml, kg/m^3



Phase Diagram

Definitions

Bulk, saturated, dry and submerged unit weights (γ) are defined in a similar manner.

Here, use weight (kN) instead of mass (kg).

$$\underbrace{\gamma}_{N/m^3} = \underbrace{\rho}_{kg/m^3} \underbrace{g}_{m/s^2}$$

Specific gravity of the soil grains (G_s) typically varies between 2.6 and 2.8.

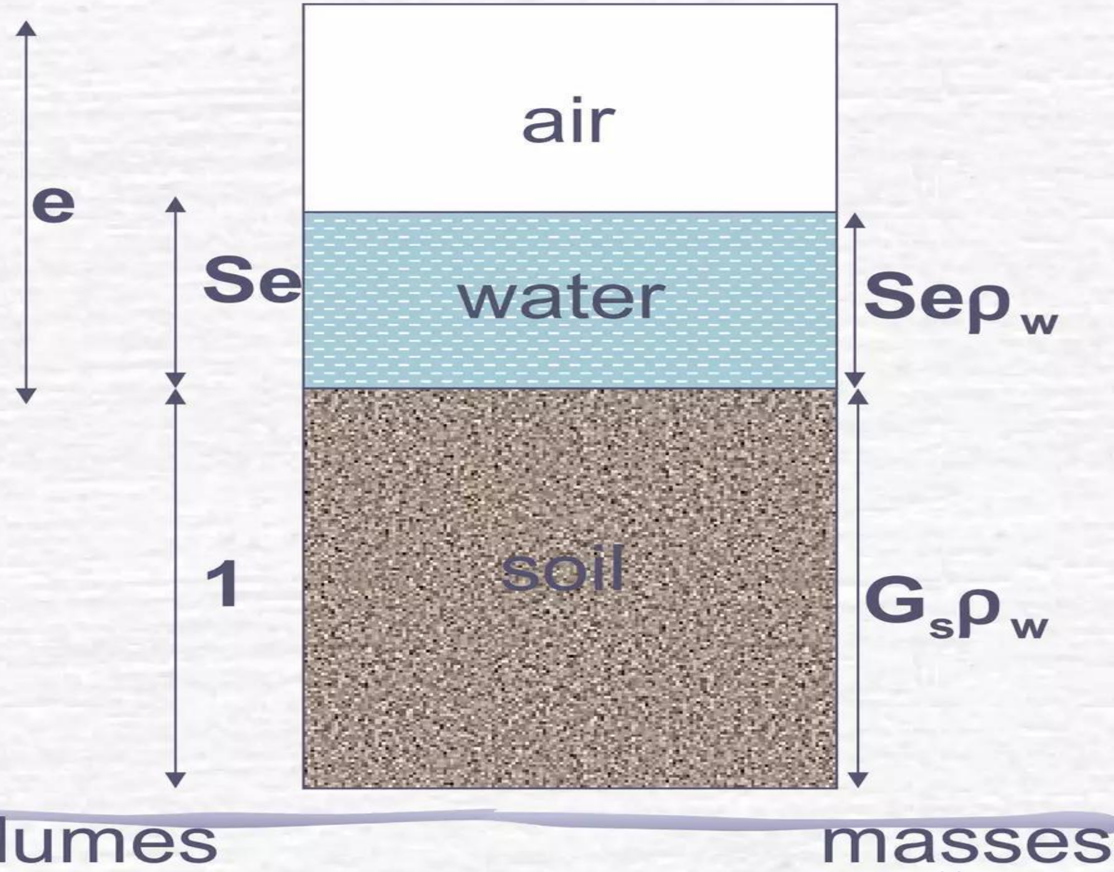
Phase Relations

Consider a fraction of the soil where $V_s = 1$.

The other volumes can be obtained from the previous definitions.

The masses can be obtained from:

Mass = Density x Volume



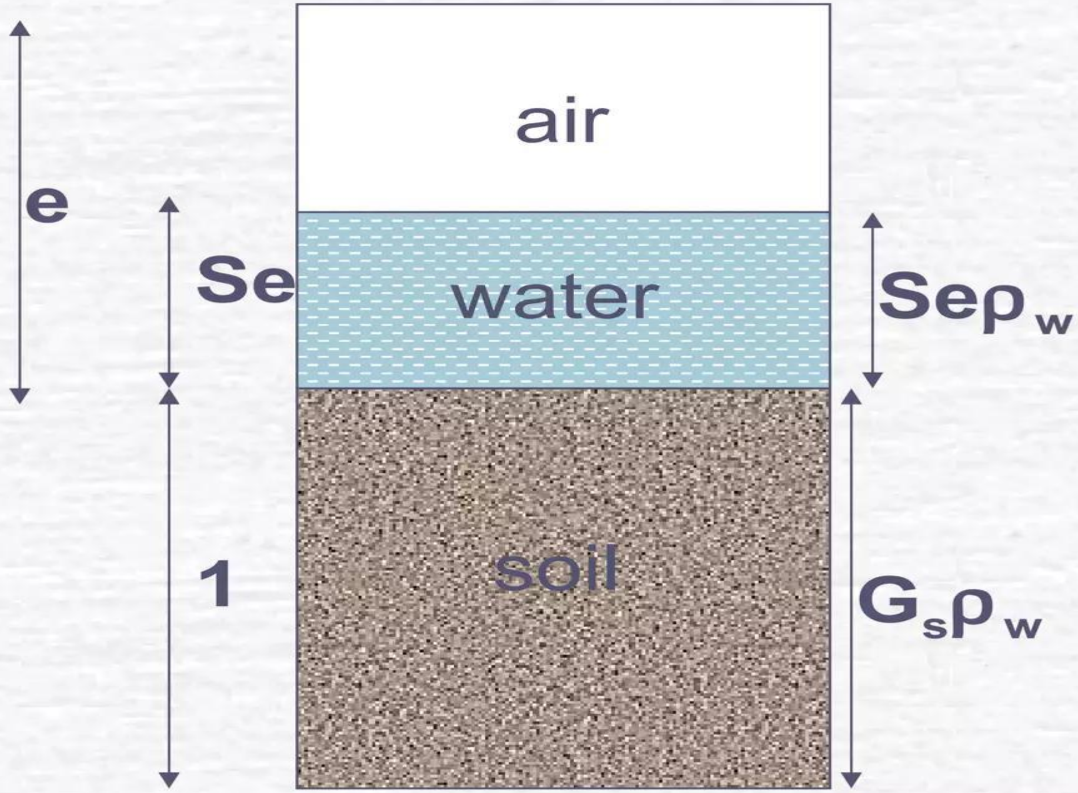
Phase Diagram

Phase Relations

From the previous definitions,

$$w = \frac{M_W}{M_S} = \frac{Se}{G_S}$$

$$n = \frac{V_V}{V_T} = \frac{e}{1+e}$$



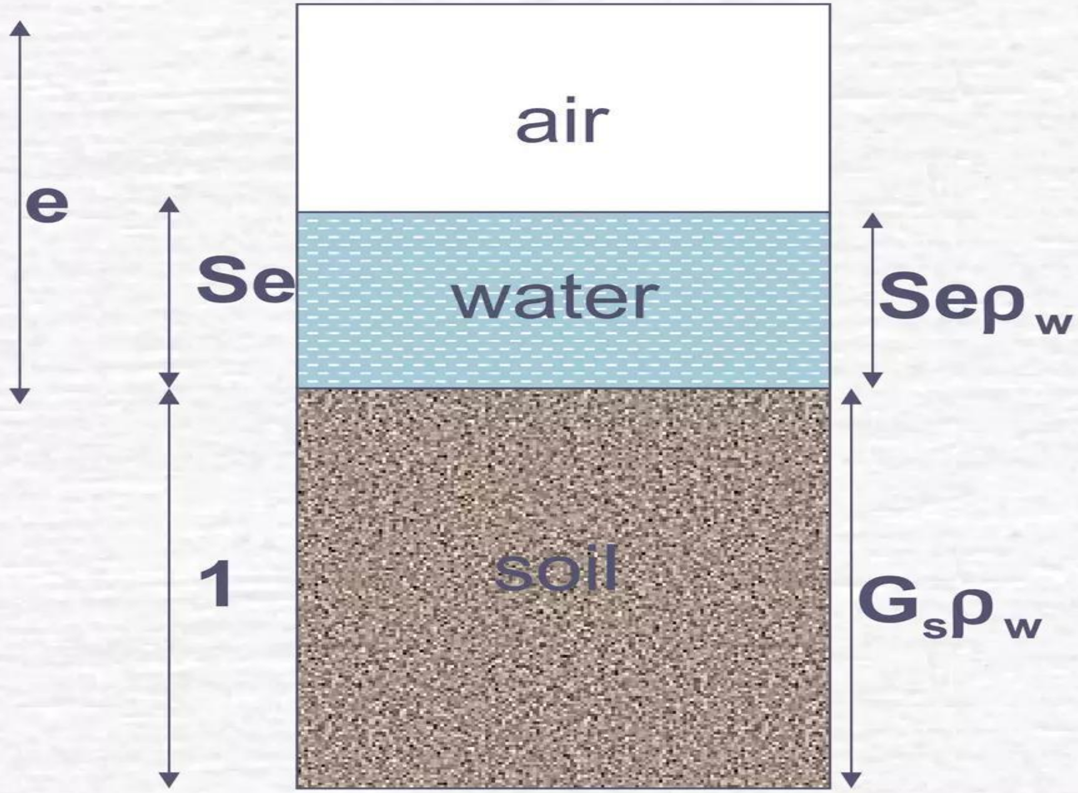
Phase Diagram

Phase Relations

$$\rho_m = \frac{M_T}{V_T} = \frac{G_S + Se}{1+e}\rho_W$$

$$\rho_{sat} = \frac{M_T}{V_T} = \frac{G_S + e}{1+e}\rho_W$$

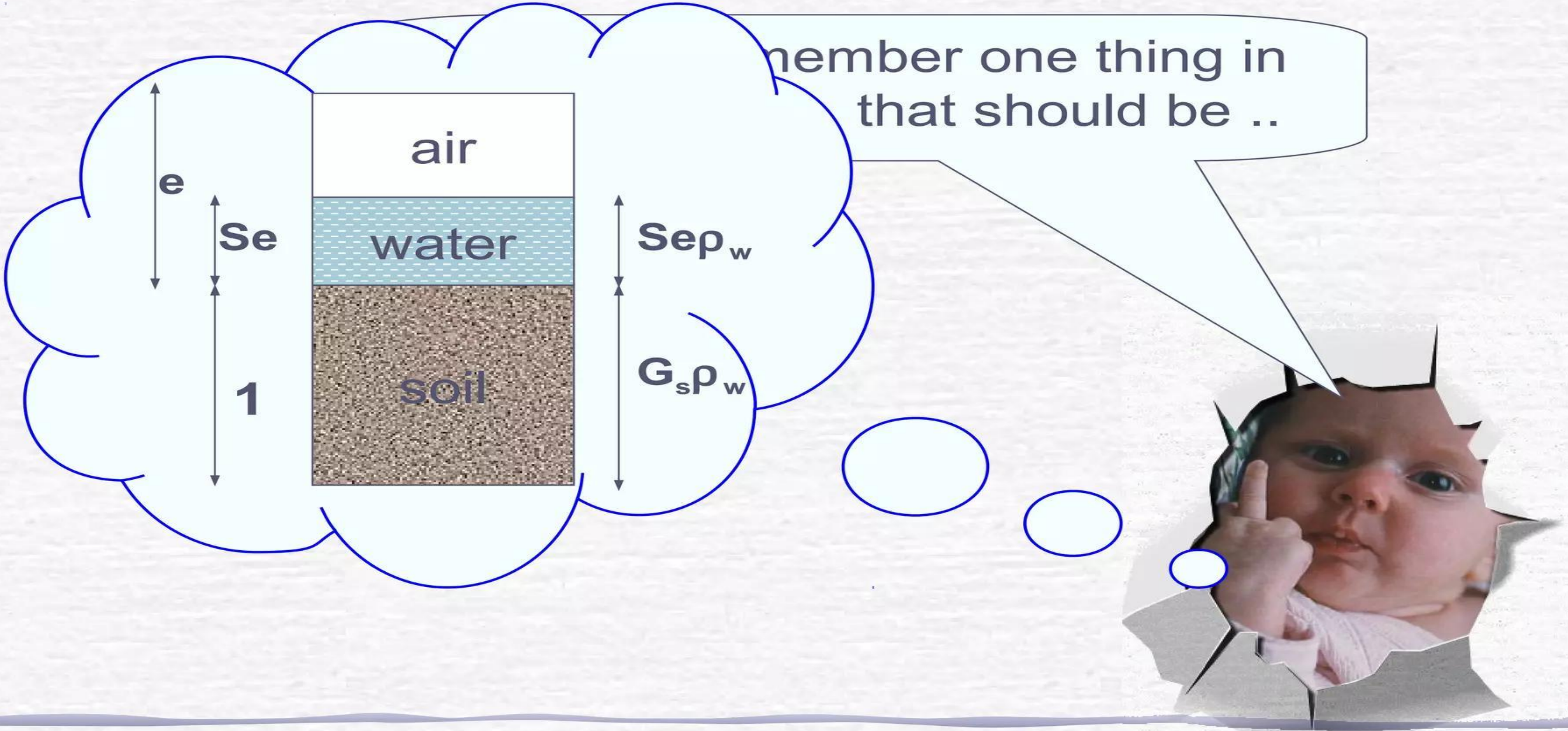
$$\rho_d = \frac{M_S}{V_T} = \frac{G_S}{1+e}\rho_W$$



Phase Diagram

- Try not to *memorise* the equations. ***Understand*** the definitions, and develop the relations from the phase diagram with $V_S = 1$;
- Assume G_S (2.6-2.8) when not given;
- Do not mix densities and unit weights;
- Soil grains are incompressible. Their mass and volume remain the same at any void ratio.

A Suggestion..



Testing Times...

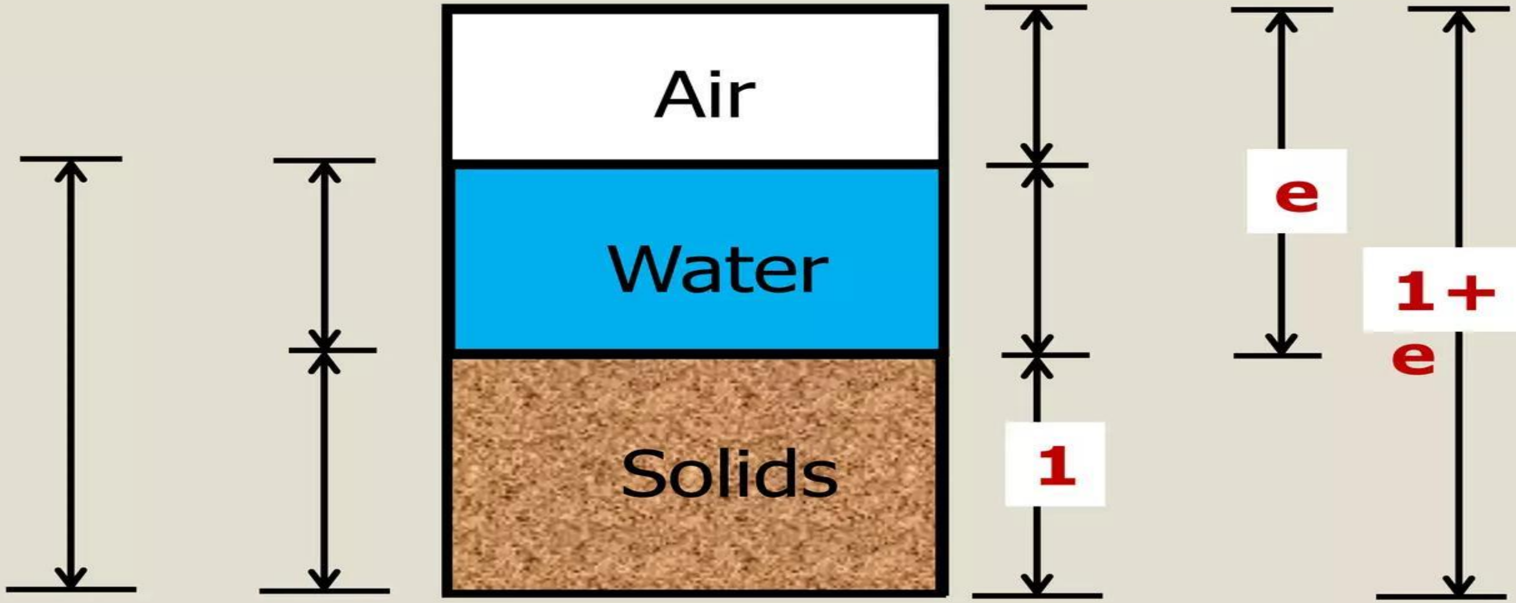
State whether the following are true or false?

(a) Dry density is greater than submerged density True

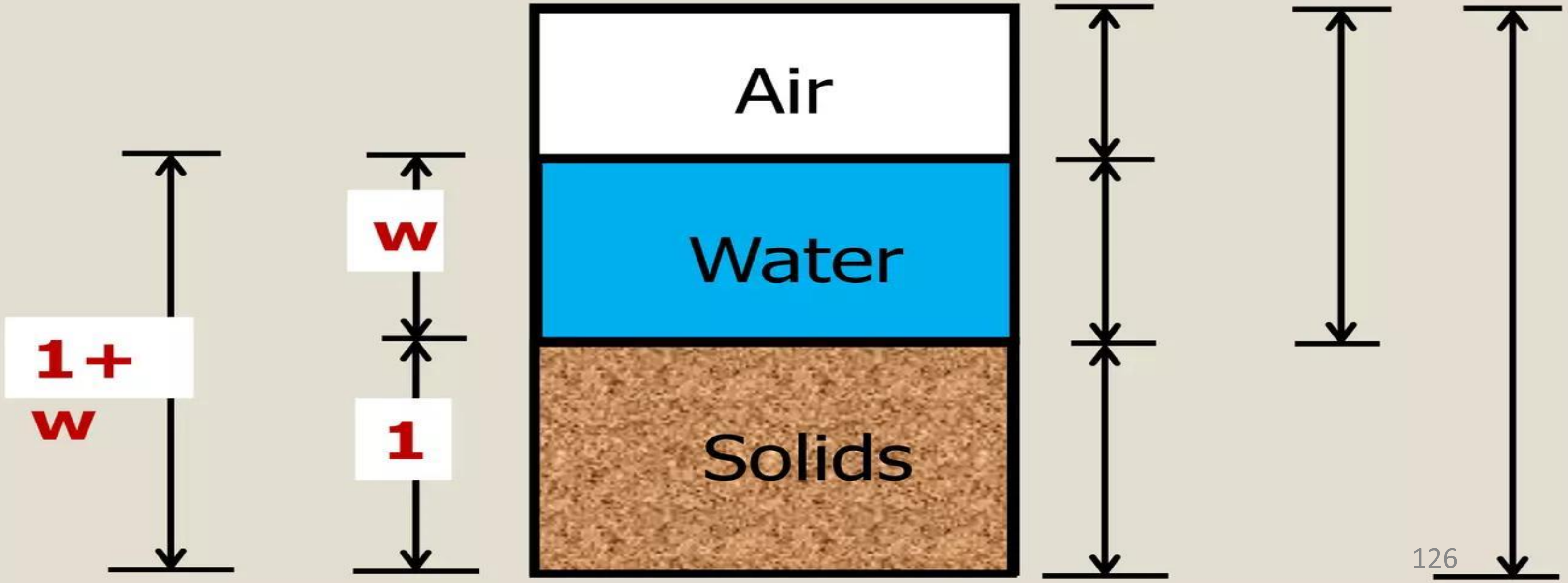
(b) In unsaturated soils, water content is always less than 100%. False

(c) Larger the void ratio, larger the porosity. True

If given e, assume $V_s = 1$



If given w, assume $W_s = 1$



1. Relationship between e and n

$$e = \frac{V_v}{V_s} = \frac{V_v}{V - V_v} = \frac{\left(\frac{V_v}{V}\right)}{1 - \left(\frac{V_v}{V}\right)} = \frac{n}{1 - n} \tag{3.6}$$

Also, from Eq. (3.6),

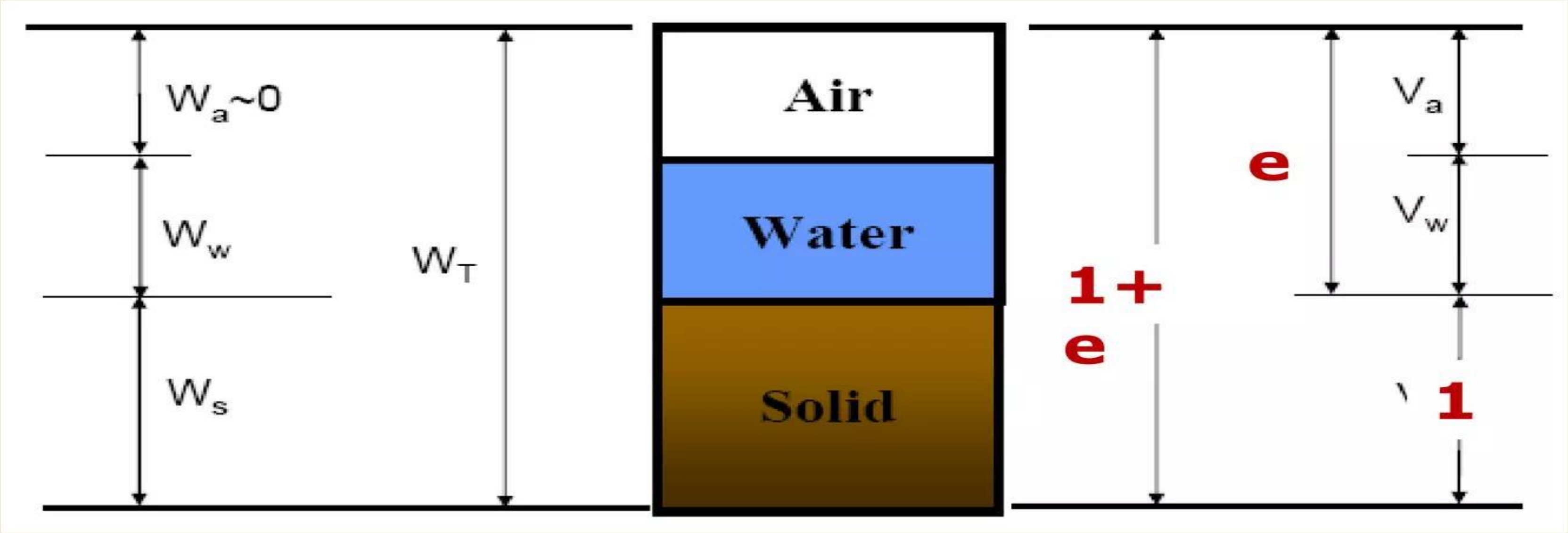
$$n = \frac{e}{1 + e} \tag{3.7}$$

<u>Using phase diagram</u>

Given : e

required: n

$$n = \frac{V_v}{V_t} = \frac{e}{1 + e}$$



2. Relationship among e, S, w, and Gs

$$w = \frac{w_w}{w_s} = \frac{\gamma_w V_w}{\gamma_s V_s} = \frac{\gamma_w V_w}{\gamma_w G_s V_s} = \frac{V_w}{G_s V_s}$$

- **Dividing the denominator and numerator of the R.H.S. by V_v *yields:***

$$\boxed{Se = wG_s}$$

- **This is a very useful relation for solving THREE-PHASE RELATIONSHIPS.**

2. Relationship among e, S, w, and Gs

- **Textbook derivation**

$$\gamma = \frac{W}{V} = \frac{W_s + W_w}{V} = \frac{G_s\gamma_w + wG_s\gamma_w}{1+e} = \frac{(1+w)G_s\gamma_w}{1+e} \tag{3.15}$$

and

$$\gamma_d = \frac{W_s}{V} = \frac{G_s\gamma_w}{1+e} \tag{3.16}$$

or

$$e = \frac{G_s\gamma_w}{\gamma_d} - 1 \tag{3.17}$$

Because the weight of water for the soil element under consideration is $wG_s\gamma_w$, the volume occupied by water is

$$V_w = \frac{W_w}{\gamma_w} = \frac{wG_s\gamma_w}{\gamma_w} = wG_s$$

Hence, from the definition of degree of saturation [Eq. (3.5)],

$$S = \frac{V_w}{V_v} = \frac{wG_s}{e}$$

or

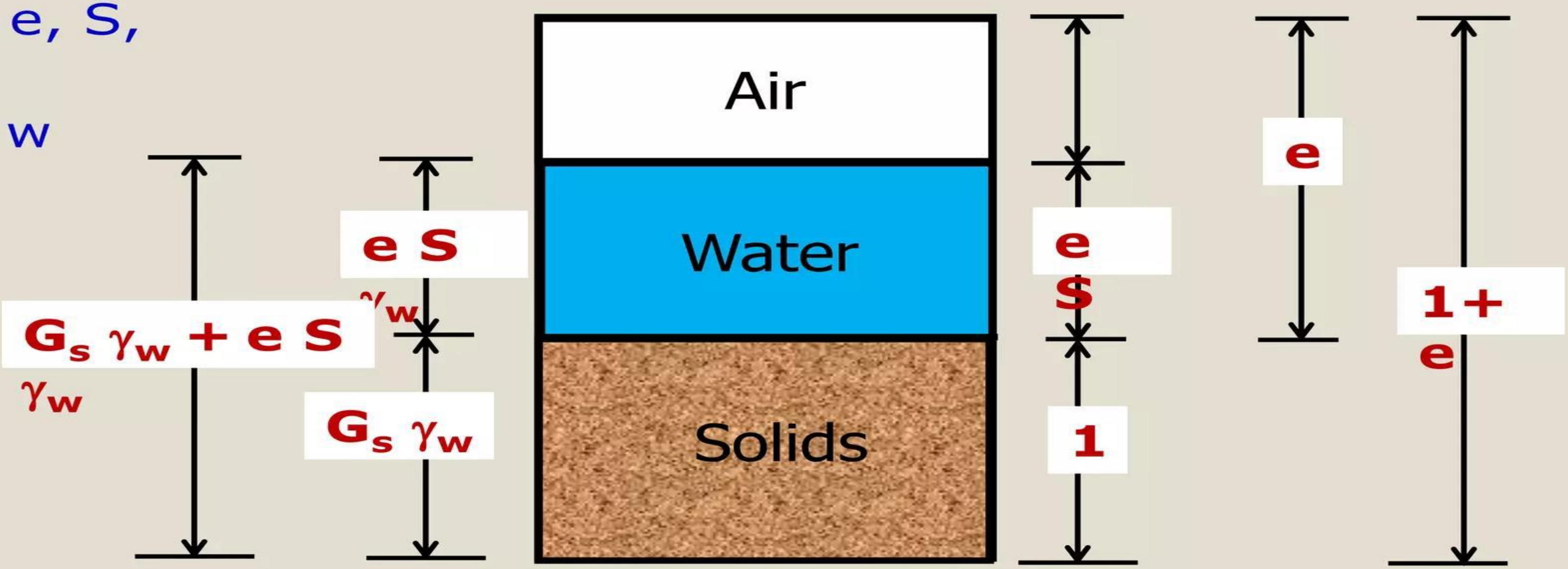
$$\boxed{Se = wG_s} \tag{3.18}$$

2. Relationship among e, S, w, and Gs

Using phase diagram

Given : e, S, G_s

required: w



$$w = \frac{W_w}{W_s} = \frac{e\,S\,\gamma_w}{G_s\,\gamma_w} = \frac{e\,S}{G_s} \qquad \therefore\ w\,G_s = e\,S$$

3. Relationship among γ, e, S and G_s

$$\gamma = \frac{W}{V} = \frac{W_w + W_s}{V_s + V_v} = \frac{\gamma_w V_w + \gamma_s V_s}{V_s + V_v} = \frac{\gamma_w V_w + \gamma_w G_s V_s}{V_s + V_v}$$

$$\boxed{\gamma = \frac{(Se + G_s)}{1+e}\gamma_w}$$

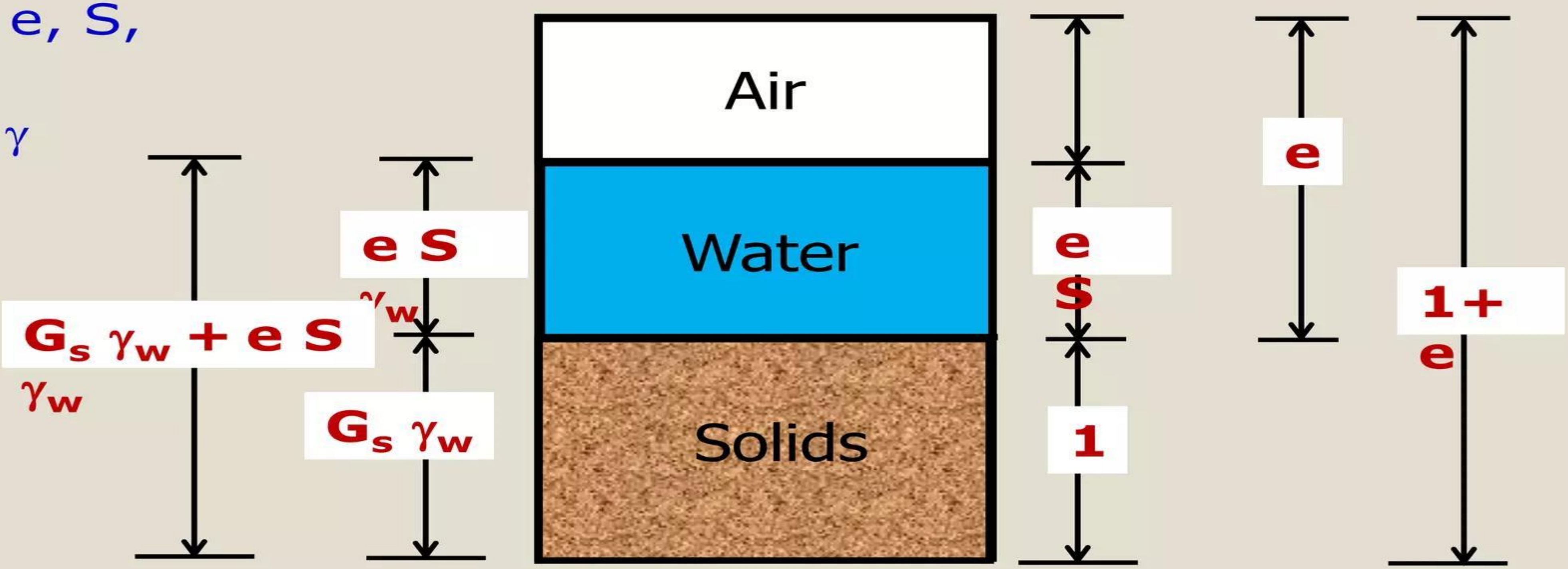
- **Notes:**
  - **Unit weights for dry, fully saturated and submerged cases can be derived from the upper equation**
  - **Water content can be used instead of degree of saturation.**

3. Relationship among γ, e, S and G_s

Using phase diagram

Given : e, S, G_s

required: γ



$$\gamma = \frac{W_t}{V_t} = \frac{(G_s + eS)\gamma_w}{1+e}$$

$$\gamma_d = \frac{W_s}{V_t} = \frac{G_s \gamma_w}{1+e}$$

$$\gamma_d = \frac{W_t(\text{water filled with wate r})}{V_t} = \frac{(G_s + e)\gamma_w}{1+e}$$

•Various Unit Weight Relationships

Table 3.1 Various Forms of Relationships for γ, γ_d, and γ_{sat}

Moist unit weight (γ)

Given	Relationship
w, G_s, e	$\dfrac{(1+w)G_s\gamma_w}{1+e}$
S, G_s, e	$\dfrac{(G_s+Se)\gamma_w}{1+e}$
w, G_s, S	$\dfrac{(1+w)G_s\gamma_w}{1+\dfrac{wG_s}{S}}$
w, G_s, n	$G_s\gamma_w(1-n)(1+w)$
S, G_s, n	$G_s\gamma_w(1-n)+nS\gamma_w$

Dry unit weight (γ_d)

Given	Relationship
γ, w	$\dfrac{\gamma}{1+w}$
G_s, e	$\dfrac{G_s\gamma_w}{1+e}$
G_s, n	$G_s\gamma_w(1-n)$
G_s, w, S	$\dfrac{G_s\gamma_w}{1+\left(\dfrac{wG_s}{S}\right)}$
e, w, S	$\dfrac{eS\gamma_w}{(1+e)w}$
γ_{sat}, e	$\gamma_{sat}-\dfrac{e\gamma_w}{1+e}$
γ_{sat}, n	$\gamma_{sat}-n\gamma_w$
γ_{sat}, G_s	$\dfrac{(\gamma_{sat}-\gamma_w)G_s}{(G_s-1)}$

Saturated unit weight (γ_{sat})

Given	Relationship
G_s, e	$\dfrac{(G_s+e)\gamma_w}{1+e}$
G_s, n	$[(1-n)G_s+n]\gamma_w$
G_s, w_{sat}	$\left(\dfrac{1+w_{sat}}{1+w_{sat}G_s}\right)G_s\gamma_w$
e, w_{sat}	$\left(\dfrac{e}{w_{sat}}\right)\left(\dfrac{1+w_{sat}}{1+e}\right)\gamma_w$
n, w_{sat}	$n\left(\dfrac{1+w_{sat}}{w_{sat}}\right)\gamma_w$
γ_d, e	$\gamma_d+\left(\dfrac{e}{1+e}\right)\gamma_w$
γ_d, n	$\gamma_d+n\gamma_w$
γ_d, S	$\left(1-\dfrac{1}{G_s}\right)\gamma_d+\gamma_w$
γ_d, w_{sat}	$\gamma_d(1+w_{sat})$

Solution of Phase Problems

Method 1: Memorize relationships

$$Se = wG_s \qquad \gamma = \frac{(Se + G_s)}{1+e}\gamma_w$$

$$n = \frac{e}{1+e} \qquad \gamma_d = \frac{\gamma}{1+w}$$

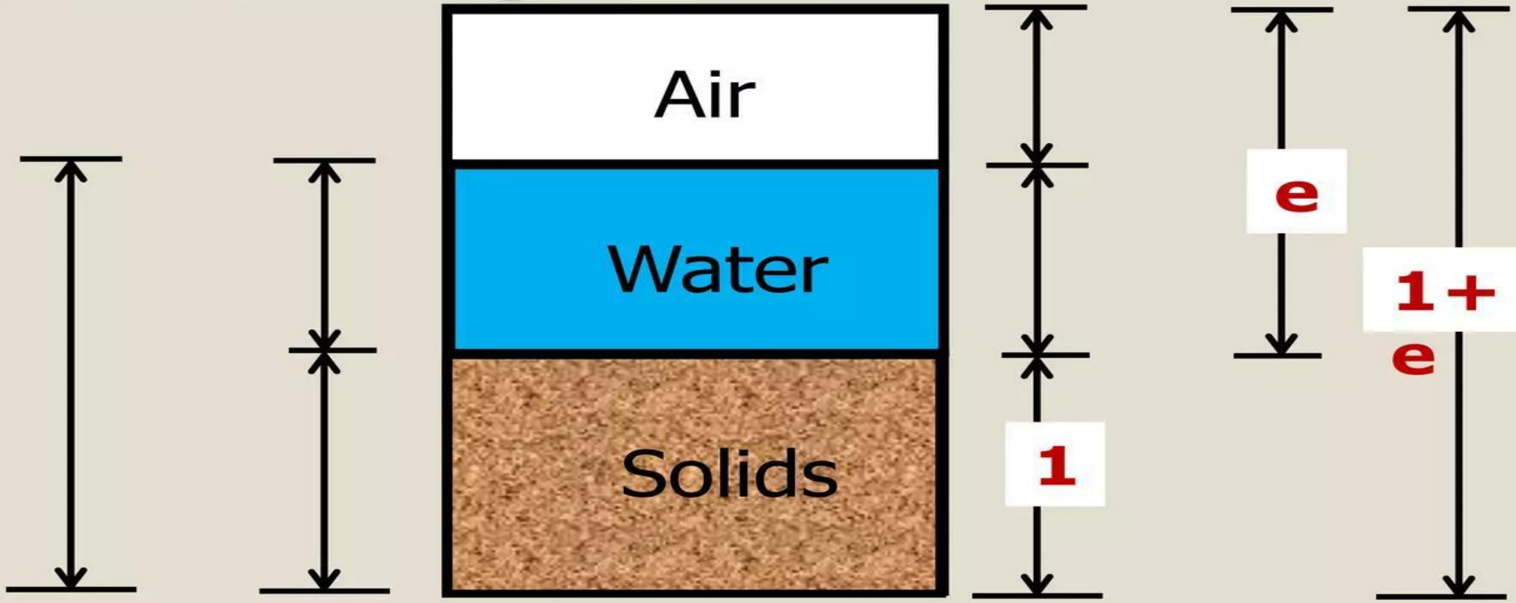
Method 2: Return to Basics

Remember the following simple rules:

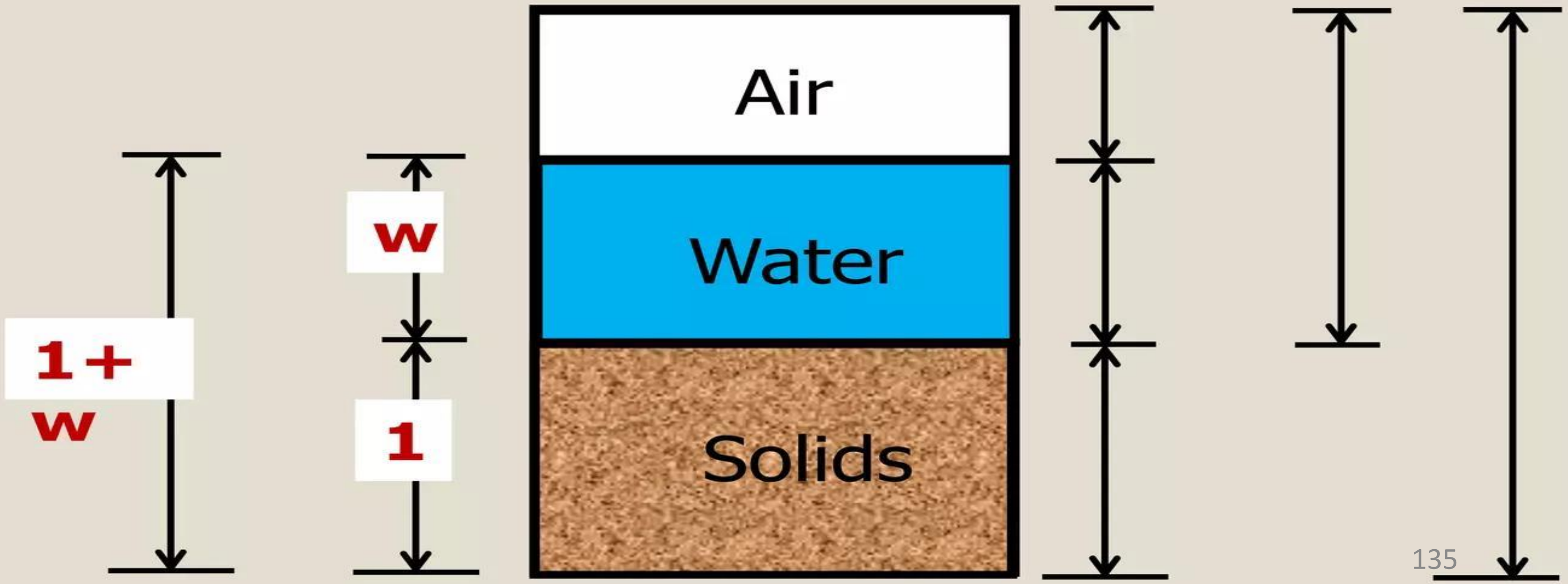
1. Remember the basic definitions of *w*, e, G_s, S, etc.
2. Draw a phase diagram.
3. Assume either V_s=1 or V_t=1 or w_s = 1 depending on given values.
4. Solve the phase diagram.

Method 2: Problem assumptions

If given e, assume $V_s = 1$



If given w, assume $W_s = 1$



Example 2

The moist unit weight of a soil is 19.2 kN/m^3. Given that G_s 2.69 and $w = 9.8\%$, determine

a. Dry unit weight
b. Void ratio
c. Porosity
d. Degree of saturation

Method 1a

a. $$\gamma_d = \frac{\gamma}{1+w} = \frac{19.2}{1+\frac{9.8}{100}} = \mathbf{17.5\ kN/m^3}$$

b. $$\gamma_d = 17.5 = \frac{G_s\gamma_w}{1+e} = \frac{(2.69)(9.81)}{1+e}; \quad e = \mathbf{0.51}$$

c. $$n = \frac{e}{1+e} = \frac{0.51}{1+0.51} = \mathbf{0.338}$$

d. $$S = \frac{wG_s}{e} = \frac{(0.098)(2.69)}{0.51} \times 100 = \mathbf{51.7\%}$$

Method 1b

$w = 9.8/100 = w_w/w_s = Se\,\gamma_w\,/\,G_s\,\gamma_w = Se/2.69 \rightarrow Se = 0.26362$

$\gamma = [w_w + w_s]/V = [Gs\,\gamma_w + Se\,\gamma_w]/\,(1+e)$

$19.2 = [(2.69 \times 9.807) + (0.26362 \times 9.807)]/1+e \rightarrow e = 0.50852$

$\gamma_d = w_s/v = G_s\,\gamma_w\,/1+e = (2.69 \times 9.807)/1+0.50852 = 17.4878\ kN/m^3$

$n = V_v/V = e/1+e = 0.50852/(1+0.50852) = 0.338$

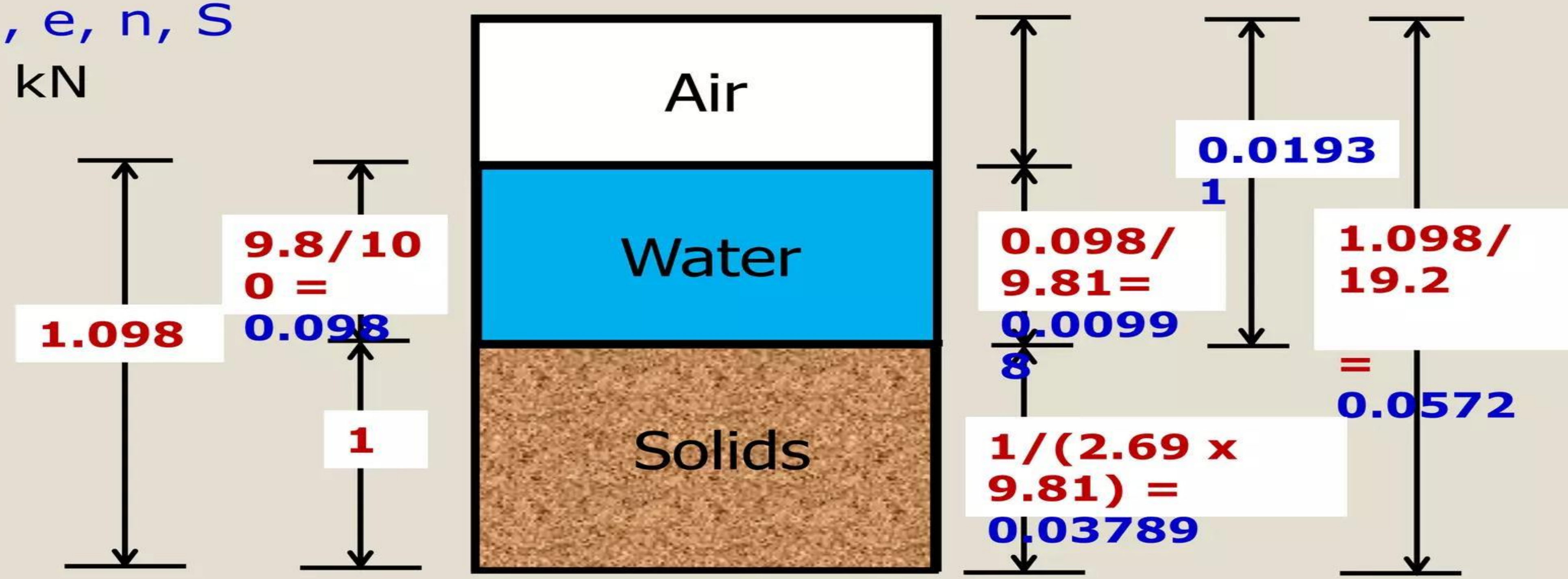
$Se = 0.26362 \rightarrow S = 0.523479 \times 100 = 52.3479\%$

Method 2:

Given : w = 9.8% , $\gamma = 19.2$ kN/m^3, G_s = 2.69

required: γ_d , e, n, S

assume W_s =1 kN



$$\gamma_d = \frac{W_s}{V_t} = \frac{1}{0.0572} = 17.48 \text{ kN/m}^3$$

$$n = \frac{V_v}{V_t} = \frac{0.01931}{0.0572} \times 100 = 33.76\%$$

$$e = \frac{V_V}{V_S} = \frac{0.01931}{0.03789} = 0.51 \text{ kN/m}^3$$

$$S = \frac{V_w}{V_v} = \frac{0.00998}{0.01931} \times 100 = 51.68\%$$

Field density testing (e.g., sand replacement method) has shown bulk density of a compacted road base to be 2.06 t/m^3 with a water content of 11.6%. Specific gravity of the soil grains is 2.69. Calculate the dry density, porosity, void ratio and degree of saturation.

E

Solution:

$$w = \frac{Se}{G_s}$$

$$\therefore \mathrm{Se} = (0.116)(2.69) = 0.312$$

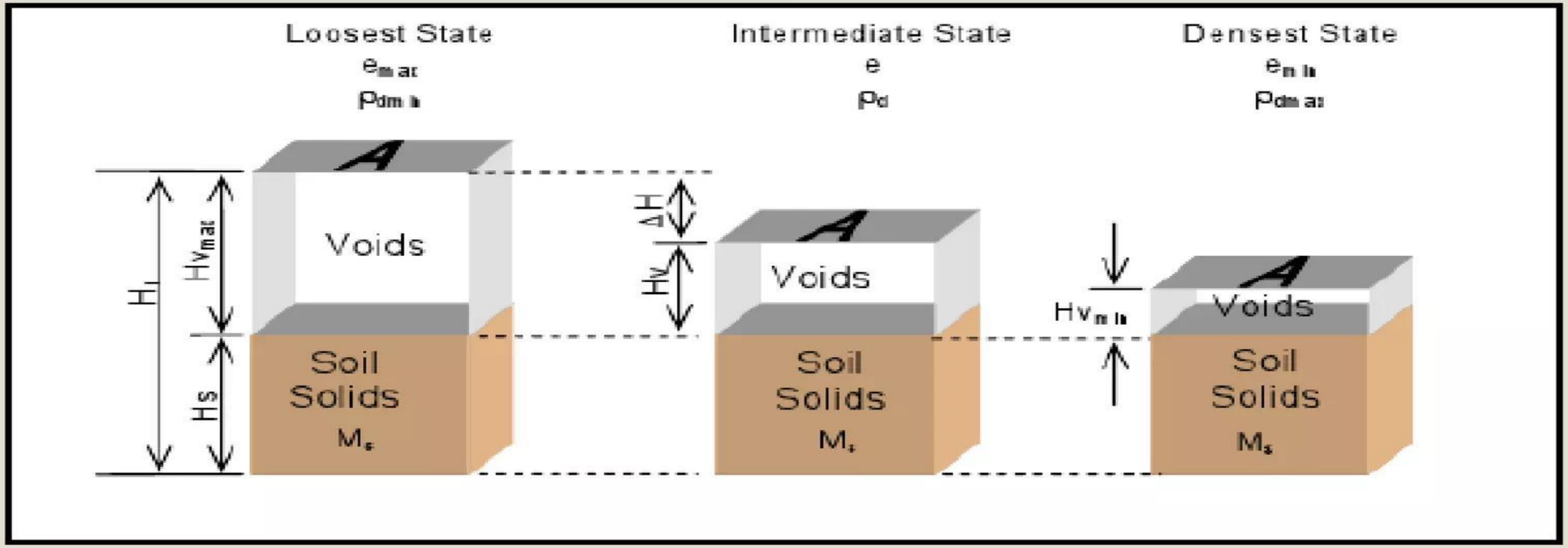
$$\rho_m = \frac{G_s + Se}{1+e}\rho_w$$

$$\therefore 2.06 = \frac{2.69 + 0.312}{1+e} \times 1.0$$

$$\therefore \mathrm{e} = 0.457$$

•Relative Density

- The relative density is the parameter that compare the volume reduction achieved from compaction to the maximum possible volume reduction
- The relative density Dr, also called density index is commonly used to indicate the IN SITU denseness or looseness of granular soil.



Volume reduction from compaction of granular soil

- D_r can be expressed either in terms of **void ratios** or **dry densities**.

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}}$$

where D_r = relative density, usually given as a percentage
e = *in situ* void ratio of the soil
e_{max} = void ratio of the soil in the loosest state
e_{min} = void ratio of the soil in the densest state

$$D_r = \frac{\left[\frac{1}{\gamma_{d(min)}}\right] - \left[\frac{1}{\gamma_d}\right]}{\left[\frac{1}{\gamma_{d(min)}}\right] - \left[\frac{1}{\gamma_{d(max)}}\right]} = \left[\frac{\gamma_d - \gamma_{d(min)}}{\gamma_{d(max)} - \gamma_{d(min)}}\right]\left[\frac{\gamma_{d(max)}}{\gamma_d}\right]$$

where $\gamma_{d(min)}$ = dry unit weight in the loosest condition (at a void ratio of e_{max})
γ_d = *in situ* dry unit weight (at a void ratio of e)
$\gamma_{d(max)}$ = dry unit weight in the densest condition (at a void ratio of e_{min})

•Remarks

- The relative density of a natural soil very strongly affect its engineering behavior.
- The range of values of D_r may vary from a minimum of zero for very LOOSE soil to a maximum of 100% for a very DENSE soil.
- Because of the irregular size and shape of granular particles, it is not possible to obtain a ZERO volume of voids. (Do you remember well-graded vs. poorly-graded!!)
- ASTM test designations D-4253 and D-4254 (2007) provide procedure for determining maximum and minimum dry unit weights of granular soils.

- Granular soils are qualitatively described according to their relative densities as shown below

Relative Density (%)	Description of soil deposit
0-15	Very loose
15-50	Loose
50-70	Medium
70-85	Dense
85-100	Very dense

- The use of relative density has been restricted to granular soils because of the difficulty of determining e_{max} in clayey soils. Liquidity Index in fine-grained soils is of similar use as D_r in granular soils.

Atterberg Limit

Week 8

Pages 145-154

Introduction:

The Atterberg limits are a basic measure of the critical water contents of a fine-grained soil: its shrinkage limit, plastic limit, and liquid limit. The Atterberg limits can be used to distinguish between silt and clay, and to distinguish between different types of silts and clays. The water content at which the soils changes from one state to the other are known as consistency limits or Atterberg's limit.

Liquid Limit:

Liquid Limit is defined as the minimum water content at percent on dry bases at which the soil changes its state from Liquid to Plastic State.

Plastic Limit:

Plastic Limit is defined as the minimum water content at percent on dry bases at which a soil enters from Semi-Solid State to Plastic State.

Shrinkage Limit:

Shrinkage limit is defined as, the maximum water content at which a reduction in water content will not cause a decrease in the volume of a soil mass.

It is lowest water content at which soil can still be completely saturated.

Determination of Liquid Limit of Soil:

The liquid limit is determined in the laboratory with the help of the standard liquid limit apparatus designed by Casagrande.

Apparatus:

1. Casagrande appratus.
2. Sieve No. 40 (ASTM).
3. Spatula.
4. Containers.
5. Tray.
6. Electric Oven.
7. Balance.

Procedure:

1. About 120 g of the specimen passing through 425 micron sieve is mixed thoroughly with distilled water in the evaporation dish or on a marble plate to form a uniform paste.
2. A portion of the paste is placed in the cup over the spot where the cup rests on the base, squeezed down and spread into position and the groove is cut in the soil pat.
3. The handle is rotated at a rate about 2 revolutions per second, and the number of blows is counted until the two parts of the soil sample come into contact at the bottom of the groove along a distance of 10 mm.

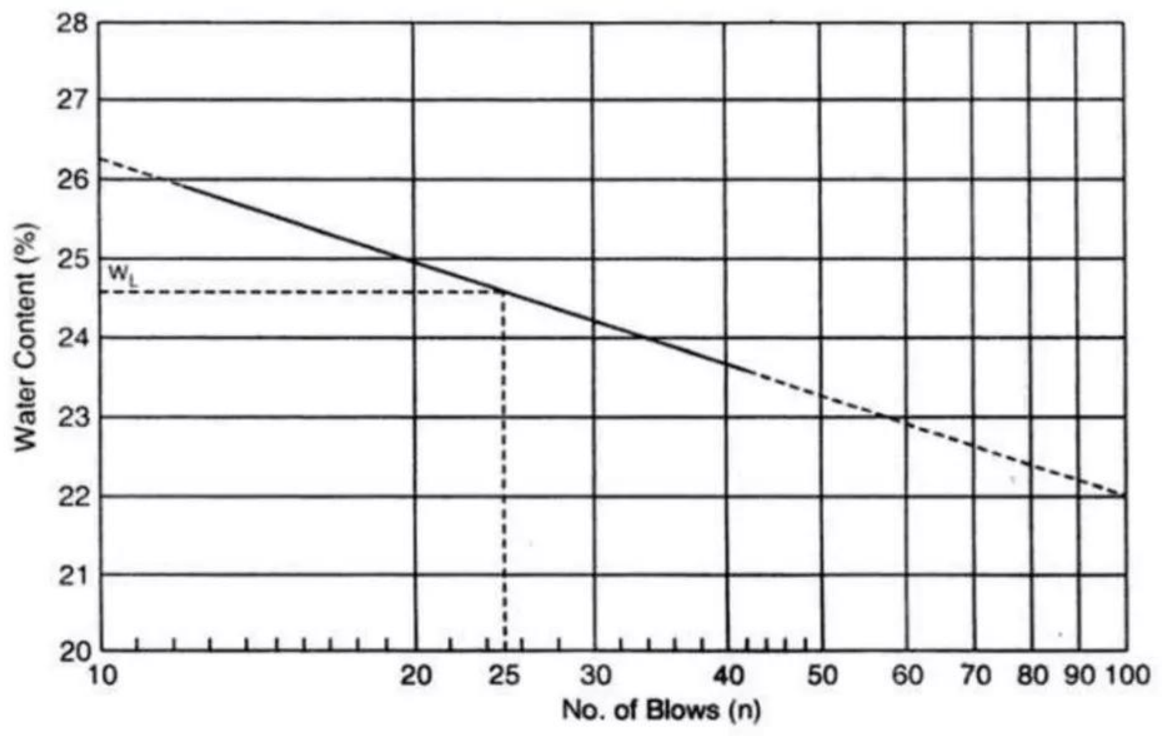
Some soils tend to slide on the surface of the cup instead of the flowing. If this occurs, the result should be discarded, and the test repeated until flowing does not occur.

Procedure:

4. After recording the number of blows, approximately 10 gram of soil from near the closed groove is taken for water content determination.

   Since it is difficult to adjust the water content precisely equal to the liquid limit when the groove should close in 25 blows, the liquid limit is determined by plotting a graph between the number of blows as abscissa on a logarithmic scale and the corresponding water content as ordinate.

Procedure:



FLOW CURVE.

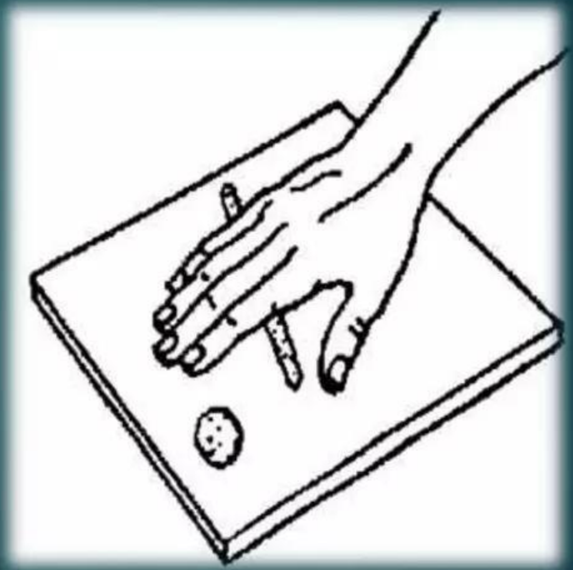
For plotting the flow curve, at least four to five sets of reading in the range of 10 to 50 blows should be taken.

The water content corresponding to 25 blows is taken as the liquid limit.

Determination of Plastic Limit of Soil.

Apparatus:

1. Glass plat (2 x 2 feet).
2. Spatula.
3. Sieve No. 40 (ASTM).
4. Electric oven.
5. Balance.



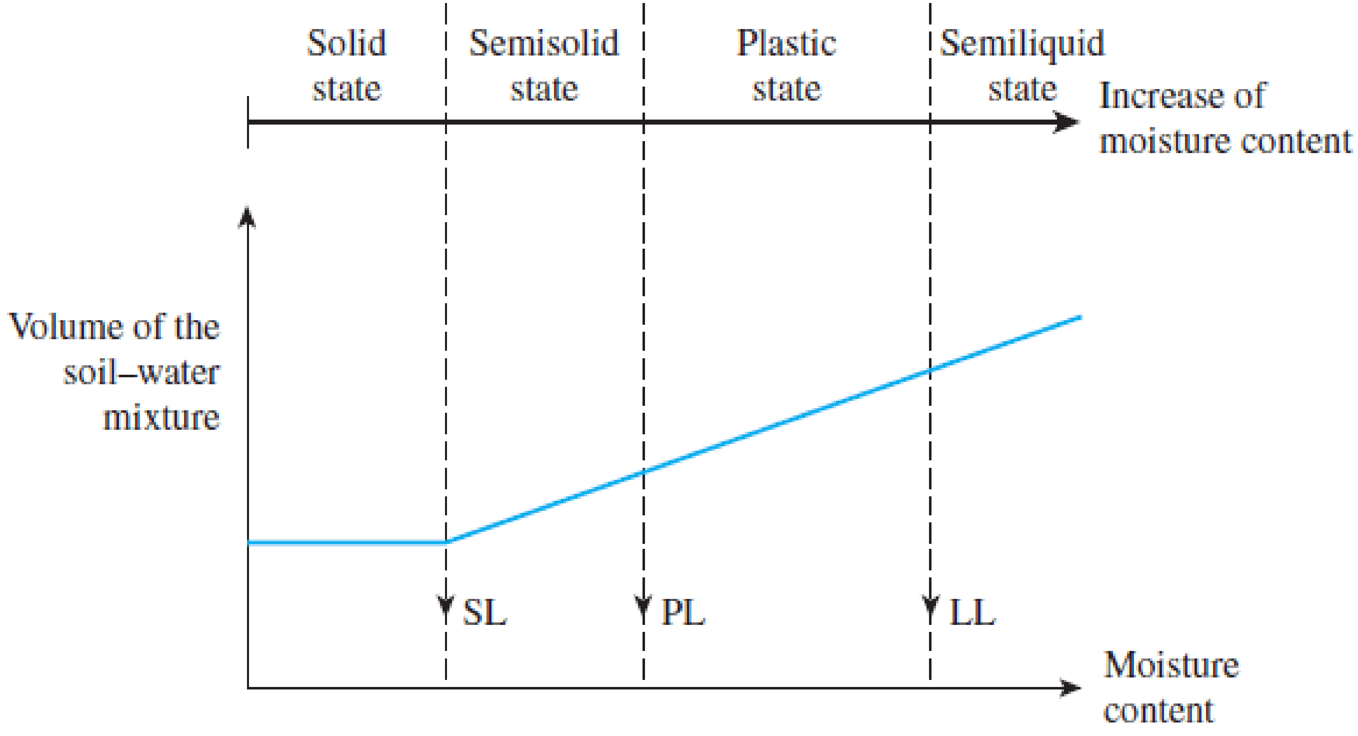
Procedure:

1. To determine the plastic limit, the soil specimen, passing 425 micron sieve, is mixed thoroughly with distilled water until the soil mass becomes plastic enough to be easily molded with fingers.
2. The plastic soil mass should be left for enough time to allow water to permeate through the soil mass.
3. A ball is formed with about 8 g of this plastic soil mass and rolled between the fingers and a glass plate (or marble plate) with just sufficient pressure to roll the mass into a thread of uniform diameter throughout its length.
4. When a diameter of 3 mm is reached, the soil is re-molded again into a ball.

Procedure:

5. This process of rolling and re-molding is repeated until the thread starts just crumbling at a diameter of 3 mm.
6. The crumbled threads are kept for water content determination.
7. The test is repeated twice more with fresh samples.
8. The plastic limit (PL) is then taken as the average of three water contents.

The plasticity index is calculated from the relation: PI = LL – PL.



Atterberg Limit

Soil Sample Collection

Week 9

Pages 156-157

Disturbed soil samples are soil samples that have been collected in a way that alters their natural structure and properties.

How are disturbed soil samples collected?

- **Auger boring**: An auger is hammered into the soil, and the soil is scooped out
- **Wash boring**: A method of collecting disturbed soil samples
- **Rotary drilling**: A method of collecting disturbed soil samples
- **Percussion drilling**: A method of collecting disturbed soil samples
- **Direct excavation**: A method of soil exploration that produces disturbed samples

What are disturbed soil samples used for?

Determining soil grain size, Determining soil plasticity characteristics, Determining soil specific gravity, Studying soil grain properties, and CBR test for highway construction.

Spilt spoon sampler

Spilt spoon sampler is used for collecting disturbed soil samples

An **undisturbed soil sample** is a soil sample that is taken from below the ground surface without disturbing its structure, water content, density, or stress state. These samples are used to determine the soil's physical properties, such as its density, permeability, shear strength, and compressive strength.

Shelby tube is used for collecting undisturbed soil samples.

Shelby Tube

Soil Compaction

Week 10-11

Pages 159-195

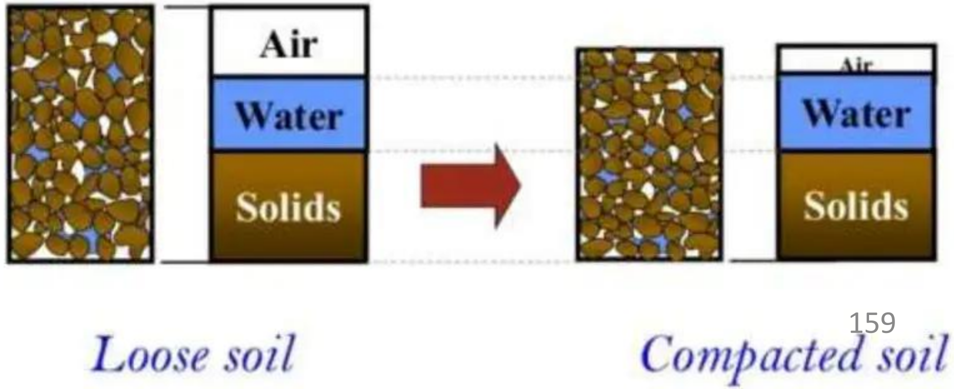
4. Soil Compaction

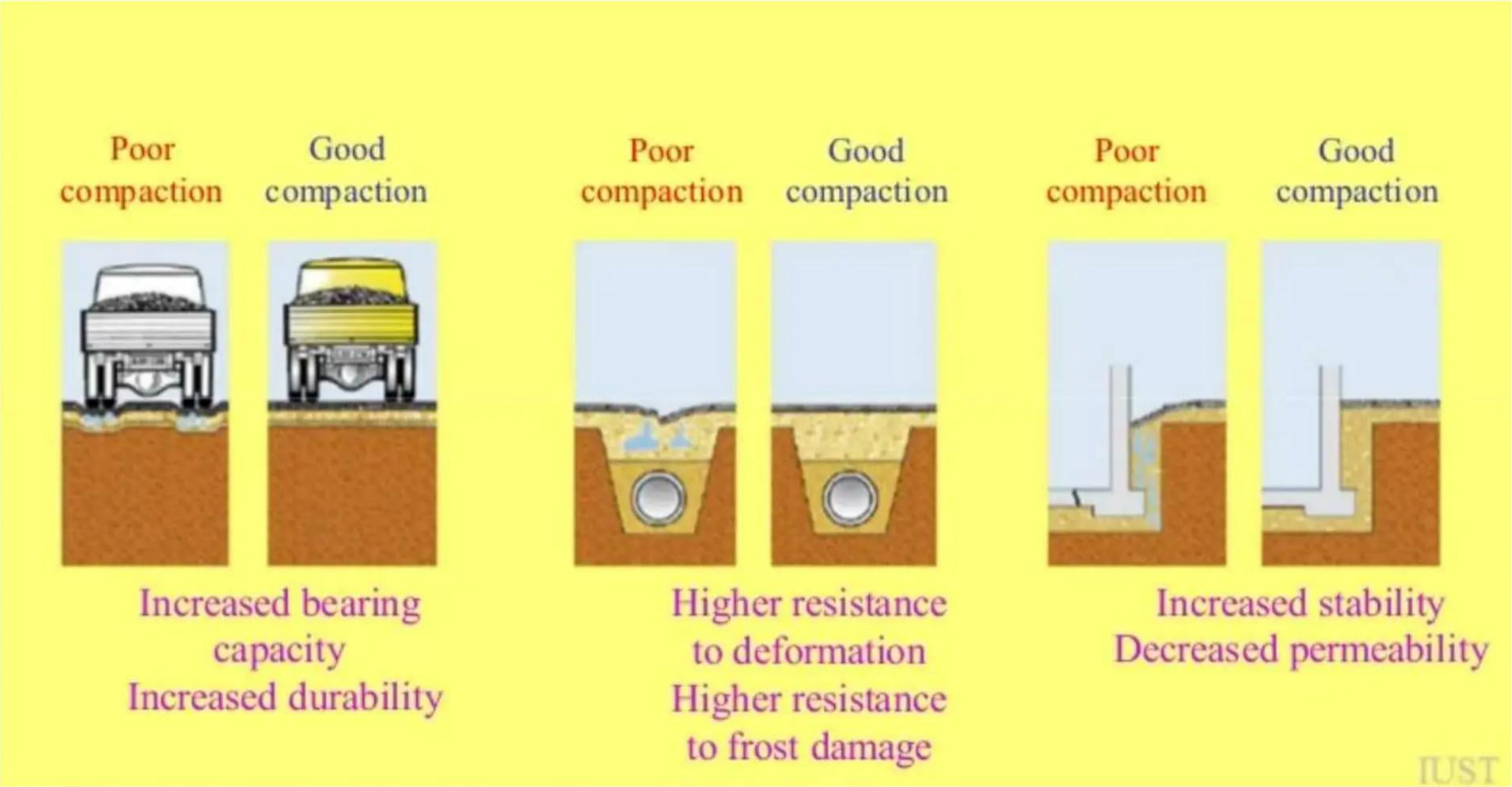
4.1 Definition and purpose of compaction

- The process of **reduction of the air void space** from the soil by densification of soil through the application of mechanical energy is known as Soil Compaction.

Purpose of Soil Compaction:

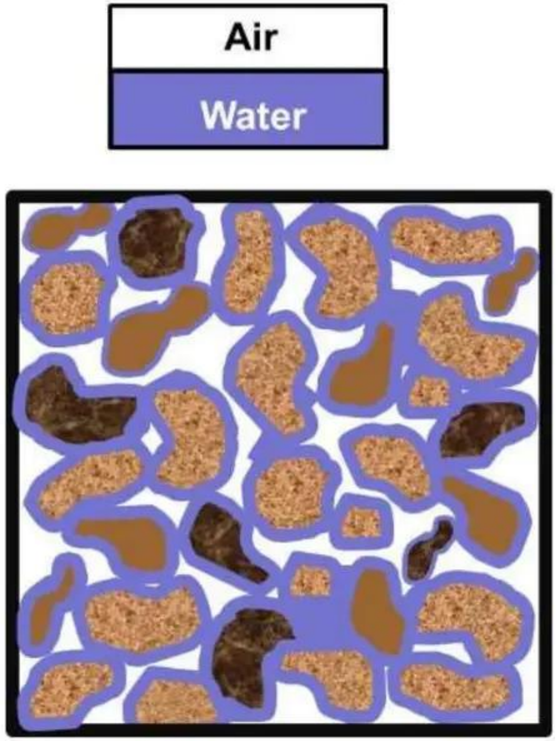
- To reduce the void space or void ratio in soil mass.
- To increase soil strength and load bearing capacity.
- Prevent unwanted soil settlement.
- Provide stability.
- Reduce permeability and water seepage.

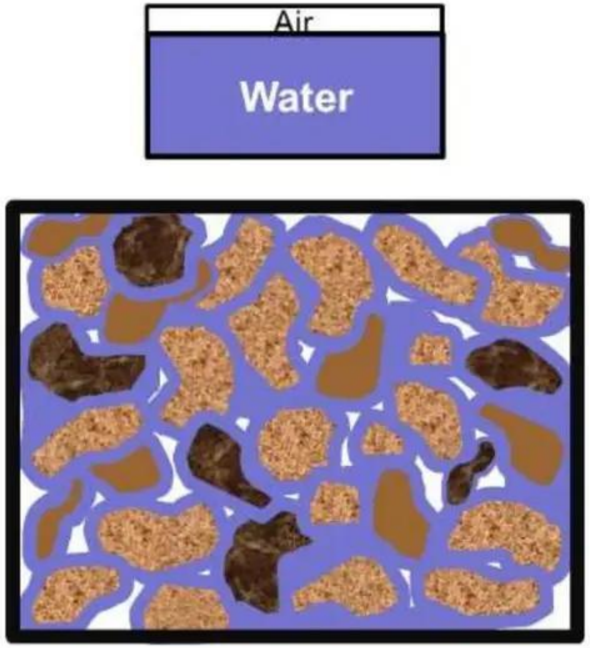



Poor compaction
Good compaction
Poor compaction
Good compaction
Poor compaction
Good compaction
Increased bearing capacity
Increased durability
Higher resistance to deformation
Higher resistance to frost damage
Increased stability
Decreased permeability
IUST

4.2 Dry density and water content relationship

Role of water in compaction





Water acts as a **lubricant** for soil particles to slide, i.e. decreases friction, and soil particles easily move closer into a denser configuration.

Adding water to the soil beyond a point, the water starts to replace the voids, and soil particles slide apart from each other. The density thus decreases (**gw** **<<gs**)

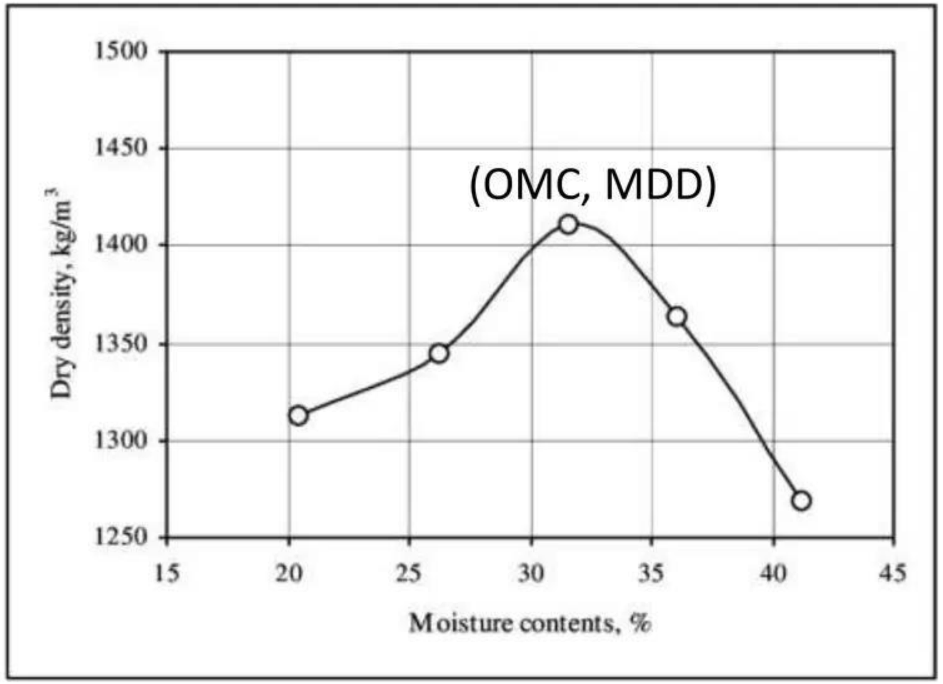
4.2 Dry density and water content relationship

$$\gamma = \frac{G + Se}{1 + e}\gamma_w$$

For compaction study, we use only the dry density, as we only need the closer arrangement of soil solids. So for dry density, S = 0.

$$\gamma_d = \frac{G.\gamma_w}{1 + e}$$

Also we know, $\gamma_d = \frac{\gamma}{1+w}$



MDD=Maximum Dry Density, OMC=Optimum Moisture Content

5.3 Laboratory test to obtain compaction characteristics

- Standard Proctor Test (ASTM)
- Modified Proctor Test (US Army Corps of Engineers)
- Indian Standard Light Compaction
- Indian Standard Heavy Compaction

5.3 Laboratory test to obtain compaction characteristics

Standard Proctor Test	Modified Proctor Test	IS Light Compaction Test	IS Heavy Compaction Test	Description
1/30 Ft^3	1/30 Ft^3	1000 cm^3	1000 cm^3	Volume of mould
5.5 pounds	10 pounds	2.6 kg	4.9 kg	Weight of hammer
12 inch	18 inch	310 mm	450 mm	Height of fall
3	5	3	5	No. of layers
25	25	25	25	No. of blows per layer

5.3 Laboratory test to obtain compaction characteristics

Procedure for any compaction test (Standard Proctor Test):

1. Take about 16-18 kg of dried soil sample in a large container.
2. Add water to about 4% of the soil mass if the soil is sandy and to about 8% if the soil is clayey. Thoroughly mix the soil.
3. Keep the soil in an air-tight container for about 18 to 20 hours for maturing. Mix the soil thoroughly. Divide the processed soil into 6 to 8 parts.
4. Take a clean and well greased standard Proctor mould with base plate and collar. Note the empty weight of the standard proctor mould.
5. Take about 2.5 kg of the processed soil, and hence place it in the mould in 3 equal layers. Take about one-third the quantity first, and compact it by giving 25 blows of the rammer. The blows should be uniformly distributed over the surface of each layer.

5.3 Laboratory test to obtain compaction characteristics

Procedure for any compaction test (Standard Proctor Test):

6. The top surface of the first layer be scratched with spatula before placing the second layer. The second layer should also be compacted by 25 blows of rammer. Likewise, place the third layer and compact it.
7. The amount of the soil used should be just sufficient to fill the mould and leaving about 5 mm above the top of the mould.
8. Remove the collar and base plate and trim off the excess soil projecting above the mould using a straight edge.
9. Clean the mould from outside. Weigh it to the nearest gram.
10. Remove the soil from the mould.

5.3 Laboratory test to obtain compaction characteristics

Procedure for any compaction test (Standard Proctor Test):

11. Take the soil samples for the water content determination from the top, middle and bottom portions. Determine the water content.
12. Add about 2-3% of the water to a fresh portion of the processed soil, and repeat the steps 5 to 9.

5.3 Laboratory test to obtain compaction characteristics

S.No.	Mass of empty mould (1)	Mass of mould + wet soil (2)	Mass of soil mass M = (2) – (1)	Bulk density ρ = M/V	Water Content w	Dry density ρ_d = $\rho/(1+w)$	Void ratio e	Degree of saturation S
1								
2								
3								
4								
5								

Density = M/V

Unit weight = W/V = M.g/V

5.3 Laboratory test to obtain compaction characteristics

- **Zero Air Void Line:**
- $\gamma_d = \frac{(1-n_a)G.\gamma_w}{1+w.G}$
- **For zero air voids, i.e. n_a= 0%**

$$\gamma_d = \frac{G.\gamma_w}{1+w.G}$$

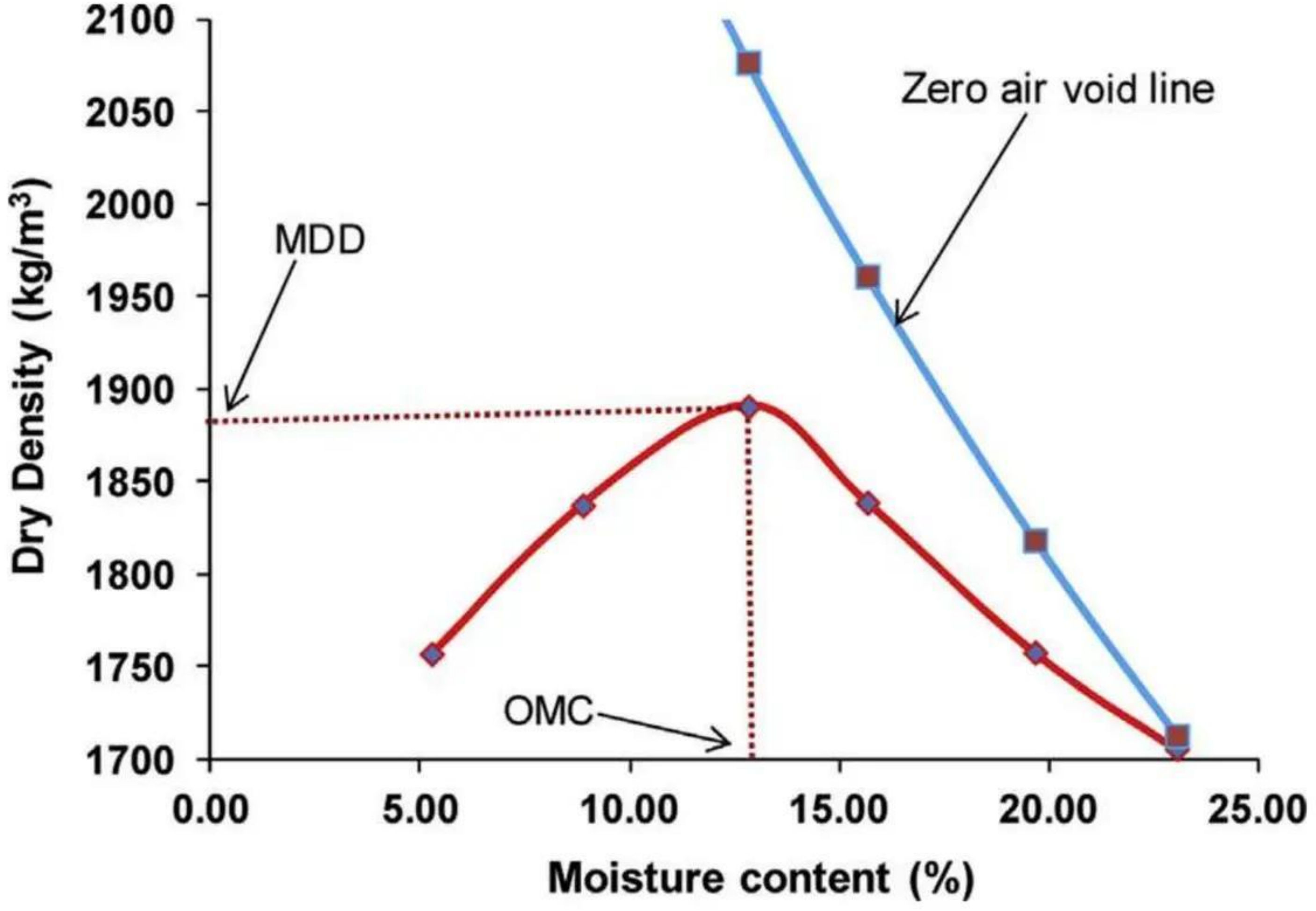
Also for 100% saturated line,

$$\gamma_d = \frac{G.\gamma_w}{1+e}$$

Since, e= w.G/S, and for S=1,

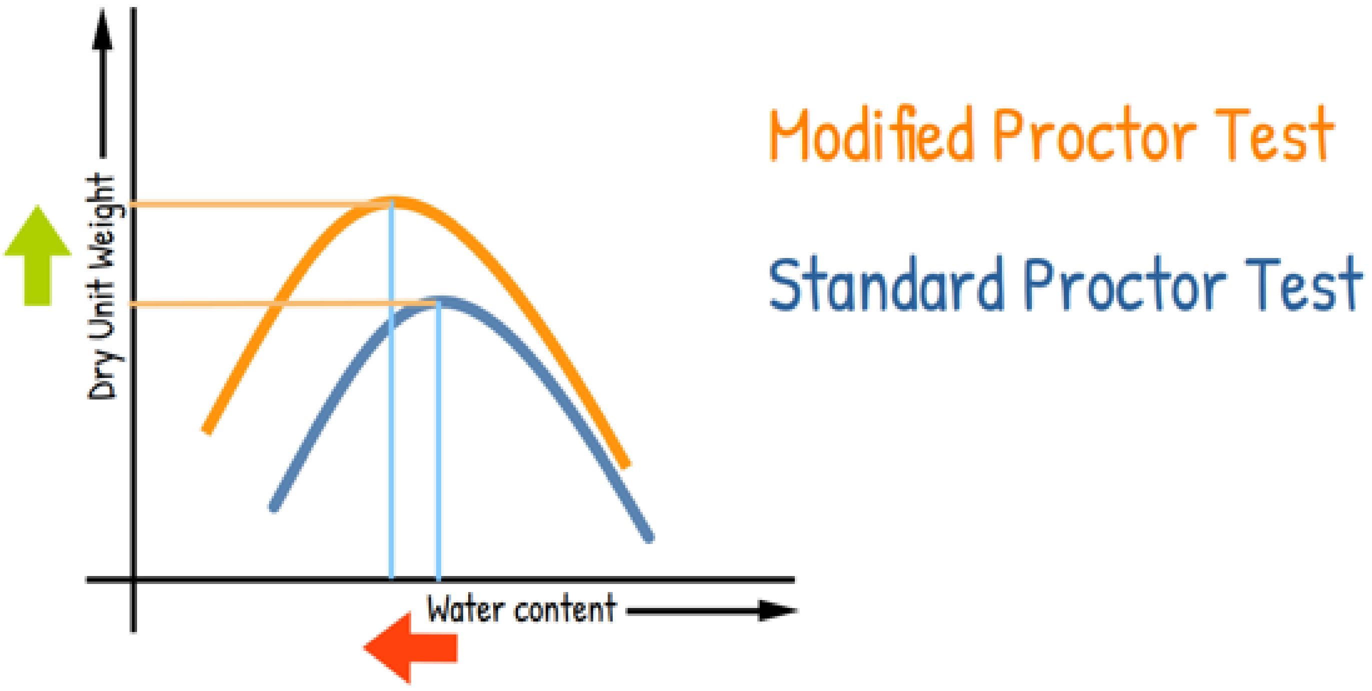
$$\gamma_d = \frac{G.\gamma_w}{1+w.G}$$

This is the equation of zero air void line or 100% saturation line.



5.4 Factors affecting Compaction

1. Water Content
2. Compactive effort
3. Type of Soil
4. Method of Compaction



5.4 Factors affecting Compaction

1. Water Content:

- At lower water content, the soil is stiff and the soil grains offer more resistance to compaction.
- As the water content increases the particles develop larger and larger water films around them, which tend to lubricate the particles and make them easier to be worked around, to move closer into a denser configuration, resulting in a higher dry unit weight and lower air voids.
- The dry unit weight continues to increase till the optimum moisture content is reached, where the lubrication effect is the maximum.
- With further increase in the water content, the water starts to fill the space of soil particles and increases volume. Since **gw<<gs**, the unit weight of soil mass decreases.

5.4 Factors affecting Compaction

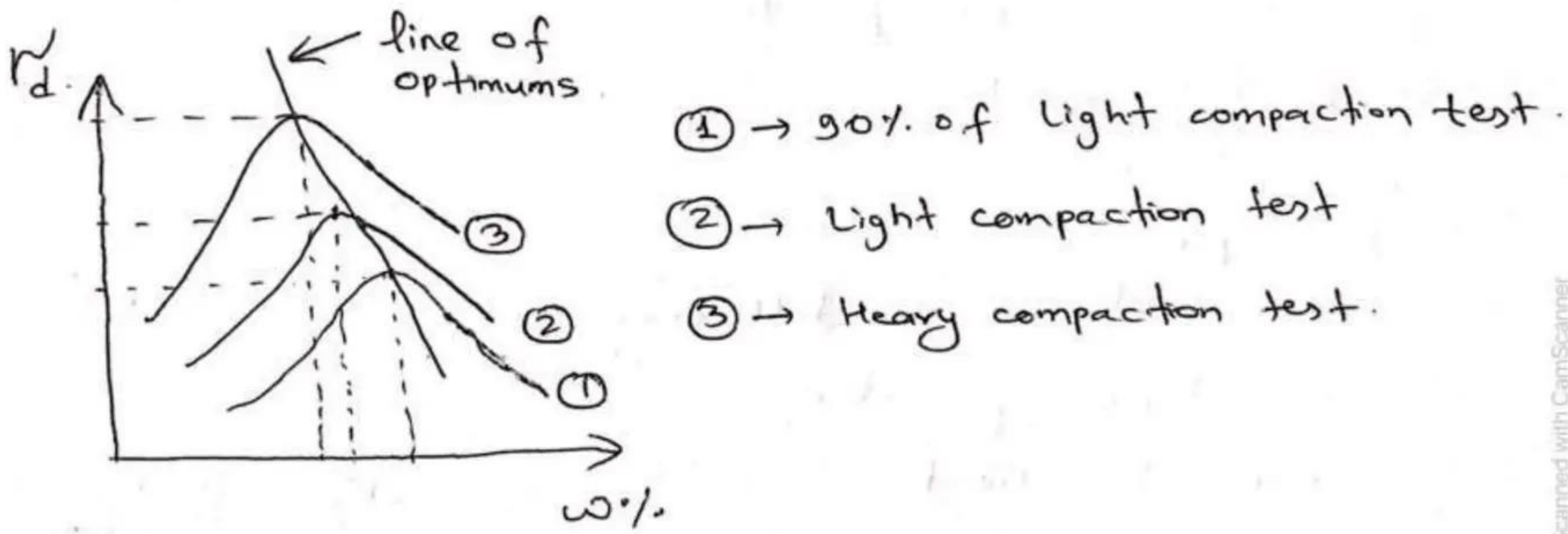
2. Compactive Effort:

Compactive energy = (Mass of hammer x g x height of fall) x number of layers x number of blows per layer

- For IS light compaction test:
  - Compactive energy = 2.6 kg x 9.81 m/s^2 x 0.310 m x (25 x 3) = **593.0145 Joule**
- For IS heavy compaction test:
  - Compactive energy = 4.9 kg x 9.81 m/s^2 x 0.450 m x (25 x 5) = **2703.88125 Joule**
- Thus, Heavy compaction applies roughly **4.5** times the energy as the light compaction.

5.4 Factors affecting Compaction

2. Compactive Effort:



- With the increase in compactive effort, the maximum dry density increases while the optimum moisture content decreases.
- If the peaks of compaction curves for different compactive efforts are joined together, a “line of optimum” is obtained

- The line of optimum is nearly parallel to the zero air void curve.
- Thus, higher compactive effort also does not result in higher efficiency of compaction.

5.4 Factors affecting Compaction

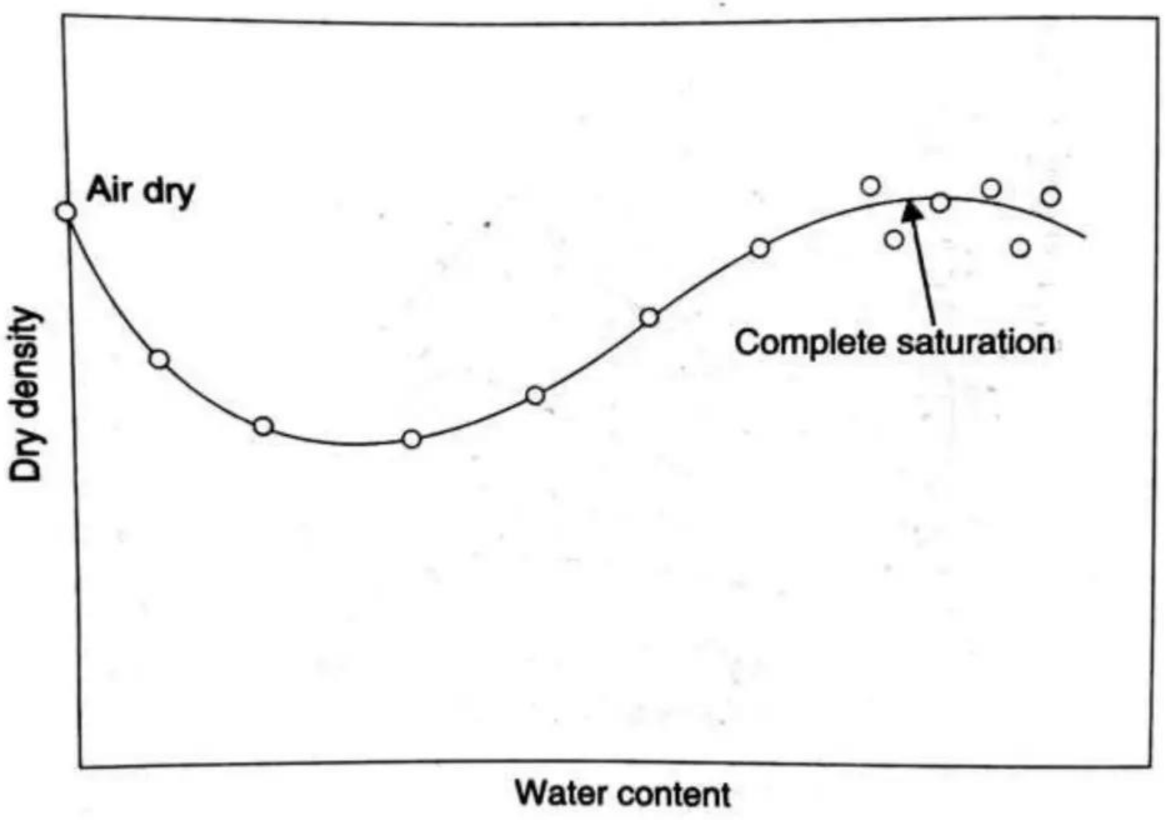
3. Soil types:

- Well graded coarse-grained soils, compact to high dry unit weight, specially if they also contain some fines. However, if the quantity of fines is excessive, the MDD decreases.
- Poorly graded or uniform sized soils/sands have the lowest dry unit weight.
- In clays, the MDD tends to decrease as plasticity index increases.
- Clays with high plasticity have very low MDD and very high OMC. (Eg. Montmorillonite)

5.4 Factors affecting Compaction

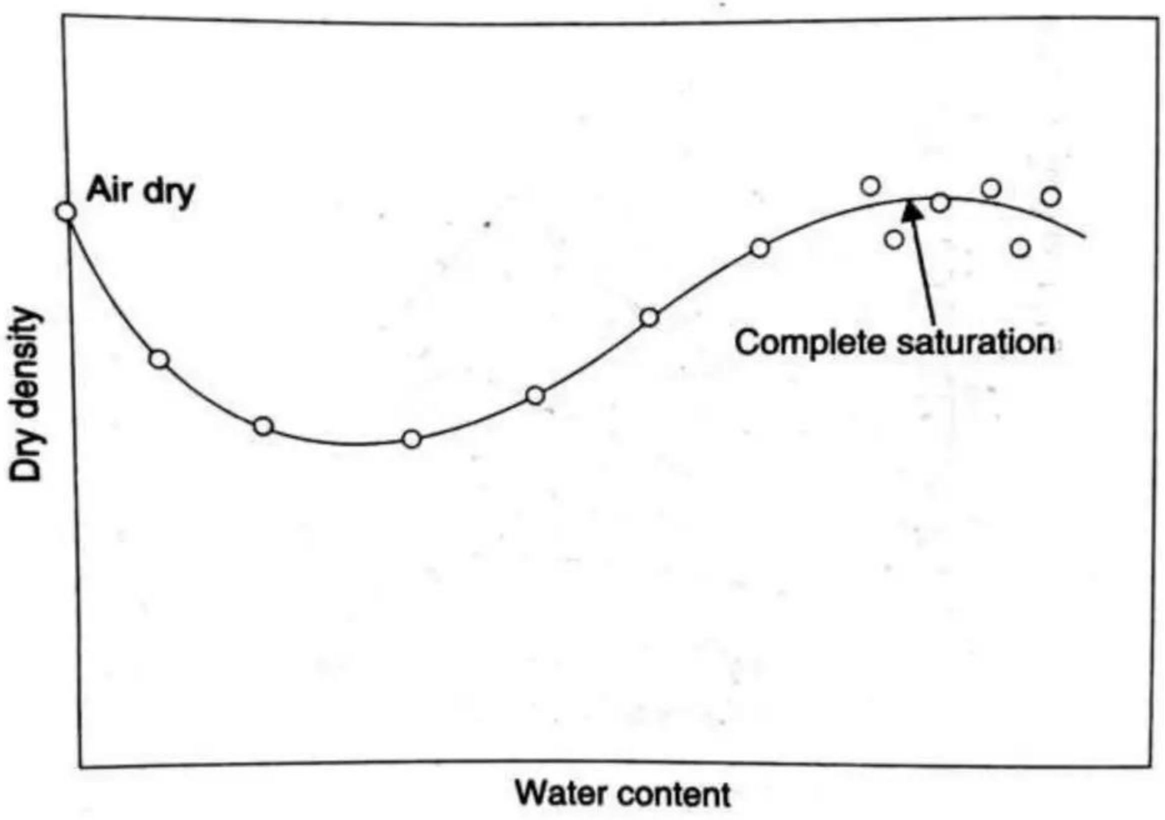
3. Soil types:

- The typical inverted U shape of the compaction curve applies only to soils with some amount of plasticity.
- In the case of **pure sandy soils** (i.e. no plasticity), the curve is obtained as shown:



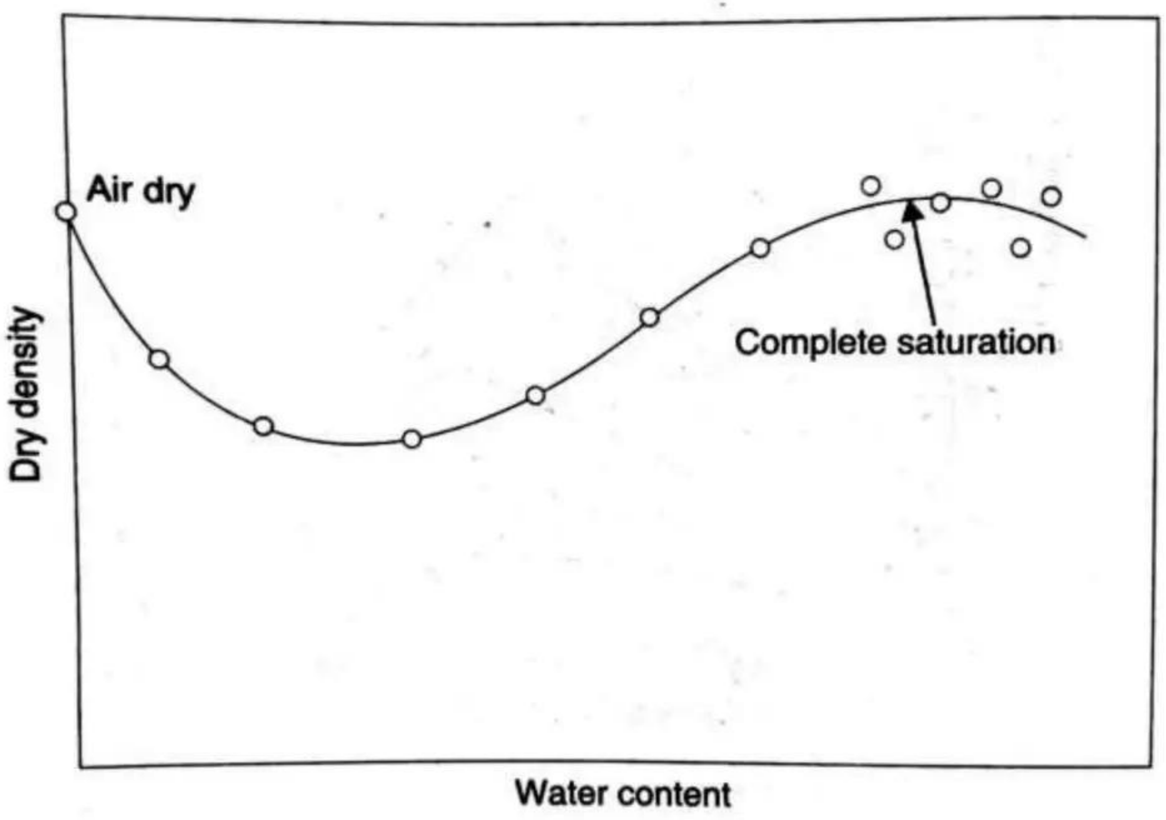
5.4 Factors affecting Compaction

- First, the dry unit weight decreases, i.e. the volume increases, with increase in water content.
- This is due to the capillary tension in pore water which prevents the soil particles from coming closer. This phenomenon is known as "bulking of sands".
- The maximum bulking occurs at a water content of 4-5%.



5.4 Factors affecting Compaction

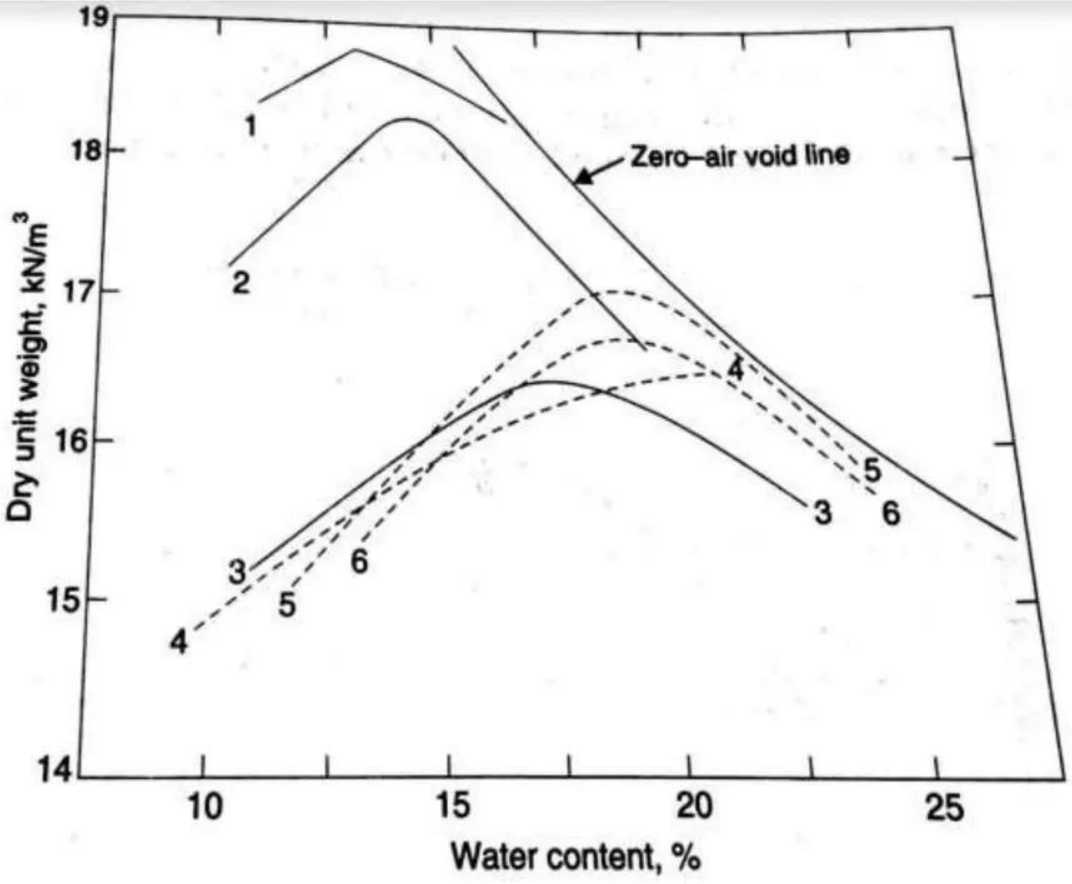
- With increase in water content, the water menisci are broken and the particles are able to move and adapt a closer packing.
- The dry unit weight becomes maximum when the soil is fully saturated.
- Hence, for pure sands, maximum density can be achieved when compacted in either dry state or saturated state.



5.4 Factors affecting Compaction

4. Method of soil compaction:

- The dry density not only depends upon the amount of compactive effort but also on the method of compaction.
- For the same amount of compactive effort/energy, the dry density will depend whether the method of compaction utilizes kneading action, dynamic action or static action.



1. Laboratory static compaction at 13790 kPa
2. Modified AASHTO compaction
3. Standard proctor compaction
4. Laboratory static compaction at 1379 kPa
5. Field compaction rubber tyred loaded, 6 passes
6. Field compaction sheep's foot rollers, 6 passes

5.5 Effect of compaction on engineering behavior of fine grained soil

At the field exact OMC is difficult to achieve, and the soil are compacted at dry of optimum or at wet of optimum.

- **Soil Structure:**

✓When compacted at wet of optimum, impact creates greater alignment of particles, “Dispersive“ soil.

✓However, if the soil becomes too aligned, then the soil can behave similar to a viscous fluid. This is called Pumping. Pumping can occur due to:

- ❖Too much water and/or
- ❖Overcompaction

✓When compacted at dry of optimum, due to lower water content, the diffuse double layer is thinner, so attraction between clay particles prevails. This causes more “Flocculated” soil structure

5.5 Effect of compaction on engineering behavior of fine grained soil

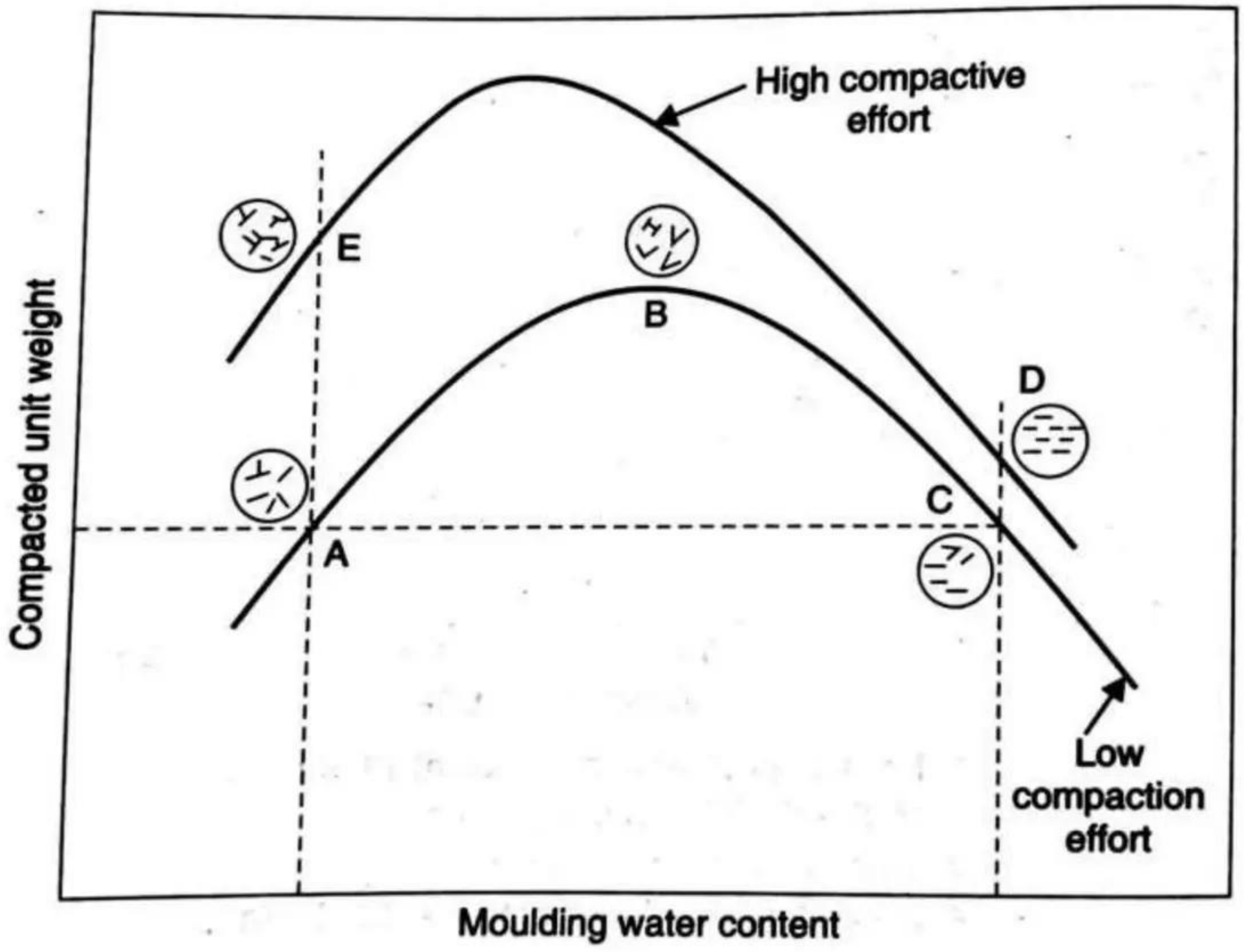


Fig. Role of over compaction

5.5 Effect of compaction on engineering behavior of fine grained soil

Property	Dry of optimum	Wet of optimum
Soil structure	Flocculated	Dispersed
Permeability	More	Less
Swelling	High	Low
Shrinkage	Low	High
Stress-strain behavior	Brittle	Ductile
Compressive Strength	Higher	Lower

5.5 Effect of compaction on engineering behavior of fine grained soil

Project	Compaction water content	Reason
Core of an earth dam	Wet of optimum	To reduce permeability and prevent cracking in core
Homogeneous earth dam	Dry of optimum	To have a stronger soil and prevent buildup of high pore water pressures
Subgrade of pavement	Wet of optimum	To limit volume changes, and lower permeability

5.6 Method of compaction and compaction control at field

Types of compaction equipment

❑ <u>Handheld compactors:</u>
- ✓ Compaction in tight spaces
- ✓ Usually requires smaller lifts

❑ <u>Sheepsfoot Roller:</u>
- ✓ Best for cohesive soil, poor for cohesionless soil
- ✓ Compaction using "kneading" action

❑ <u>Smooth Drum Roller:</u>
- ✓ Best for sandy clays
- ✓ Typically used for finishing operations

5.6 Method of compaction and compaction control at field

Types of compaction equipment

- **Pneumatic Roller:**
  - ✓ Four or five closely spaced rubber tires
  - ✓ Best for clayey sands, sandy clays and silt
  - ✓ Applies great pressure to a small area

- **Vibratory Drum Roller:**
  - ✓ Best for granular or cohesionless soil
  - ✓ Compaction using pressure + vibration
  - ✓ Can overcompact fine grained soils

5.6 Method of compaction and compaction control at field

- The selection of equipment and the procedure of compaction depends on the characteristics of the soil to be compacted.
- The compaction achieved in the field would depend on:

a. Thickness of layer
b. Type of roller
c. Number of passes of the roller
d. Intensity of pressure on the soil

5.6 Compaction control at field

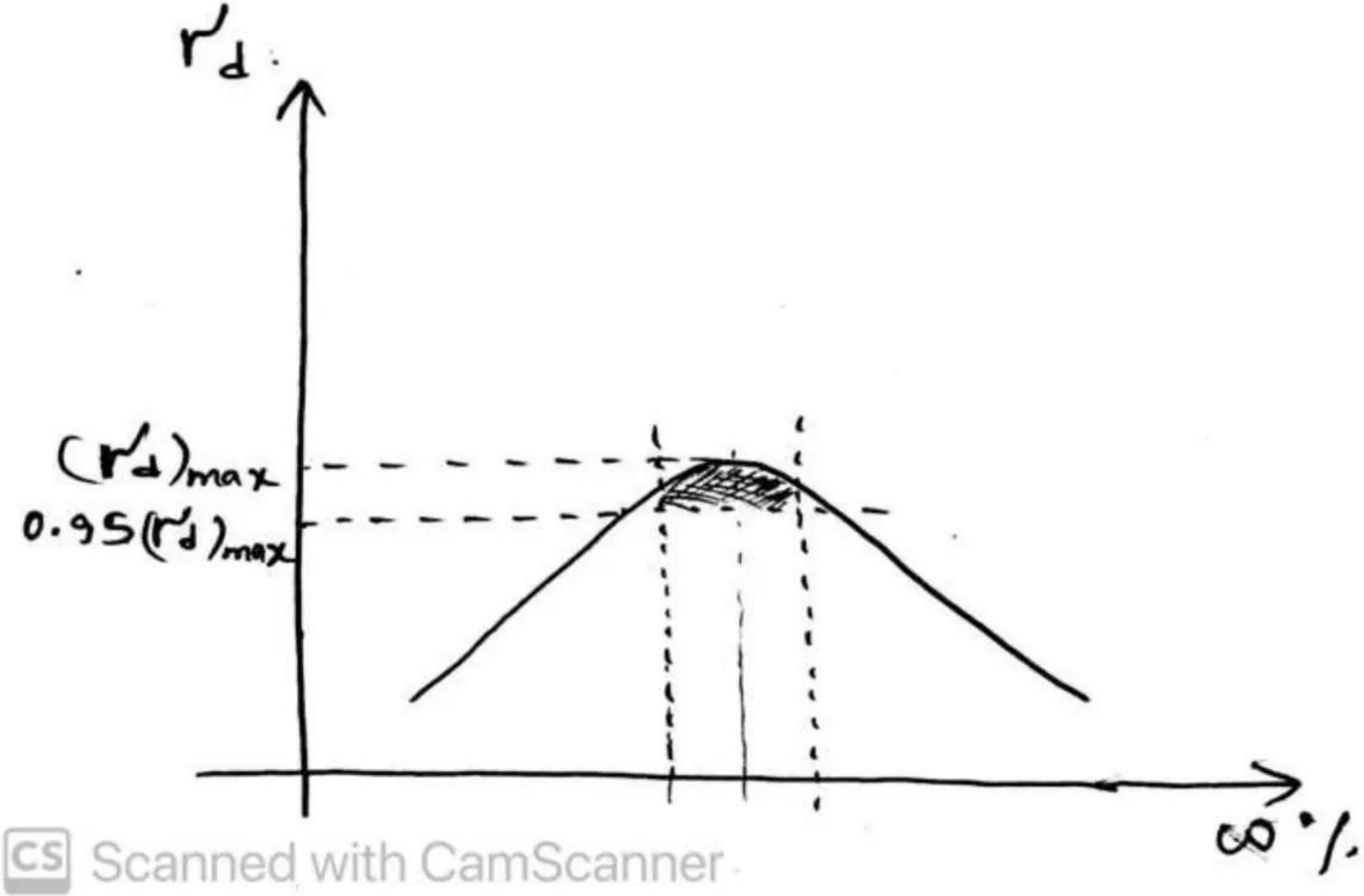
- In the field, engineers will measure dry density and water content of the compacted soil and compare them with the specifications.
- Generally, specification should mention the compaction requirement in terms of **Relative Compaction** and a **Water Content Tolerance**
- Example: " The backfill material shall be compacted to achieve a dry density of at least 95% of maximum dry density and within ±3% of optimum moisture content"

$$\text{Relative Compaction} = R = \frac{(\gamma_d)_{field}}{(\gamma_d)_{\max_{proctor}}} \times 100\%$$

The fill material shall be placed in layers (not exceeding 150 mm thickness unless otherwise Directed by the Engineer) parallel to the formation level and shall be built up evenly over the whole area of the work after proper compaction of each layer. Sufficient slope shall be maintained at all times to ensure adequate drainage. Construction traffic shall be routed over the whole area generally and shall not be confined to narrow widths. Damage caused by construction traffic shall be made good at once.

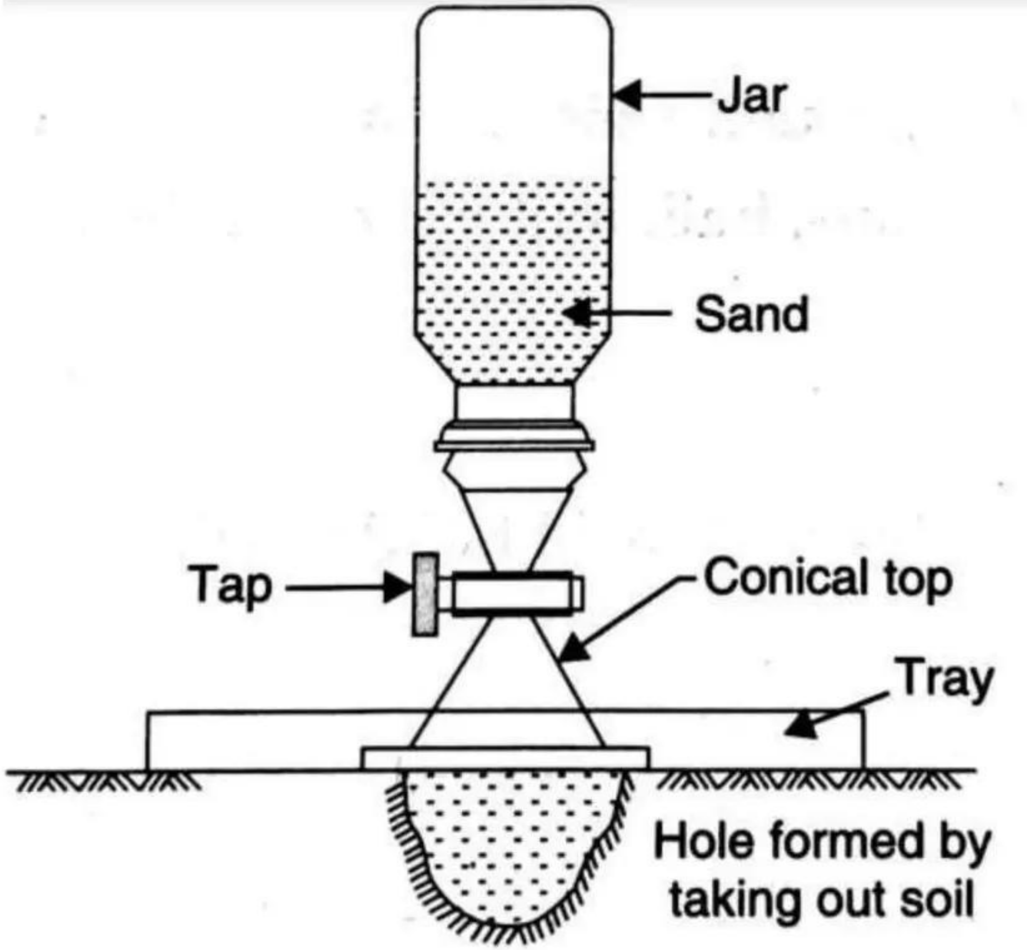
Where the Permanent Structures shall be erected on fill surface, the fill material shall be compacted in such a manner as to ensure maximum density of the material. The material shall be placed in layers of compacted thickness not exceeding 150 mm unless otherwise directed by the Engineer. Unless otherwise specified, the layers of fill material shall be compacted throughout to achieve a dry density of at least 95% of the maximum dry density.

5.6 Compaction control at field



Measurement of field density
Sand Cone Method:

- Clear the site and place a square tray with central hole on the cleaned surface.
- An excavation of diameter equal to the diameter of hole in the tray and depth of about 10-15 cm is made on ground.
- The excavated soil is collected in tray and weighed (W_1).
- The sand cone apparatus with two thirds full of clean, dry sand is weighed (W_2) and placed upside down centrally over the hole.
- The tap/valve is opened and sand is allowed to run to fill the excavated hole and conical end.



Measurement of field density

Sand Cone Method:

- The tap is closed after no further flow of sand, and the sand cone is weighed again. (W_3)
- The sand cone is placed over a level surface and weight of sand filling the cone is measured (W_4)
- Also the unit weight of sand is measured with the help of a measuring cylinder of known volume. (γ_{sand})
- Weight of sand filling the hole (W_{sand})= W_2-W_3-W_4
- Volume of sand filling the hole = volume of hole (V) = W_{sand} / γ_{sand}
- Thus, Insitu density = W_1 / V

Measurement of field density
Sand Cone Method:

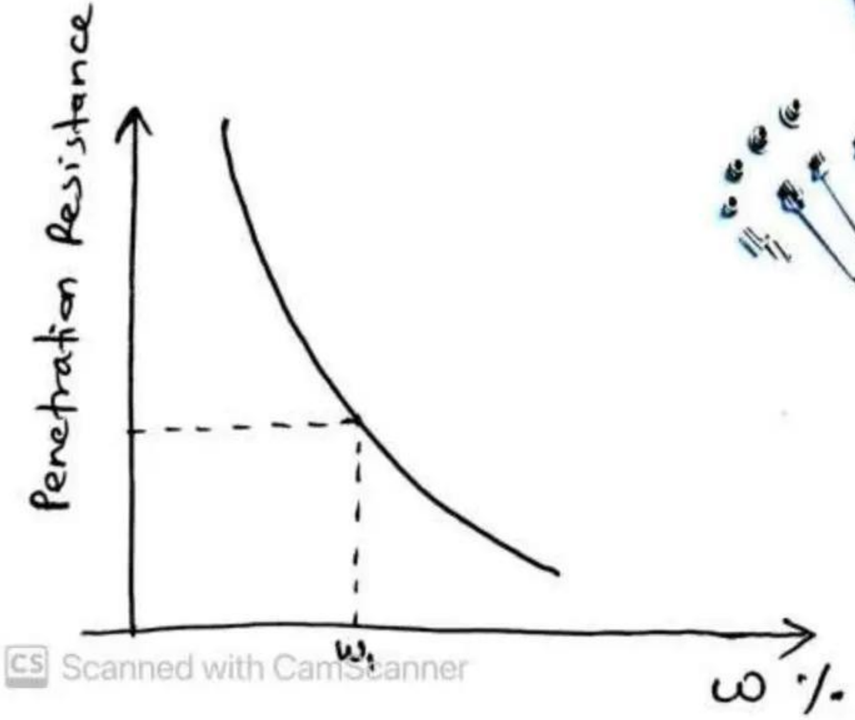
- Time consuming test method
- Accurate measurement of field dry density
- Requires laboratory equipment
- Use of moisture meter can save time

Measurement of field density
Proctor's Compaction Needle Method:

- Used for rapid determination of moisture content in-situ

- Equipment consists of a needle attached to a spring loaded plunger, calibrated to read penetration resistance in kg/cm^2

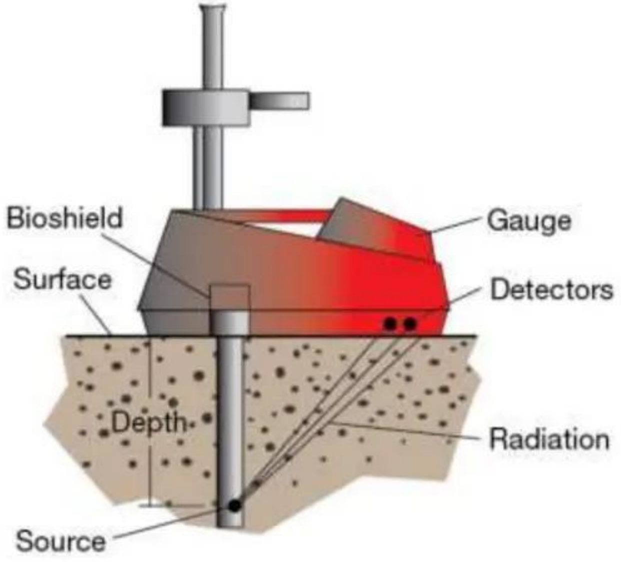
- Requires lab based calibration at the beginning



Measurement of field density
Nuclear Density Gauge Method:

- Probe has a radioactive source that emits gamma rays and fast neutrons
- The denser the soil, the fewer gamma rays are detected
- The wetter the soil, the fewer fast neutrons are detected.

- Requires timely calibration
- High initial cost
- Potential danger to the field personnel from radiation



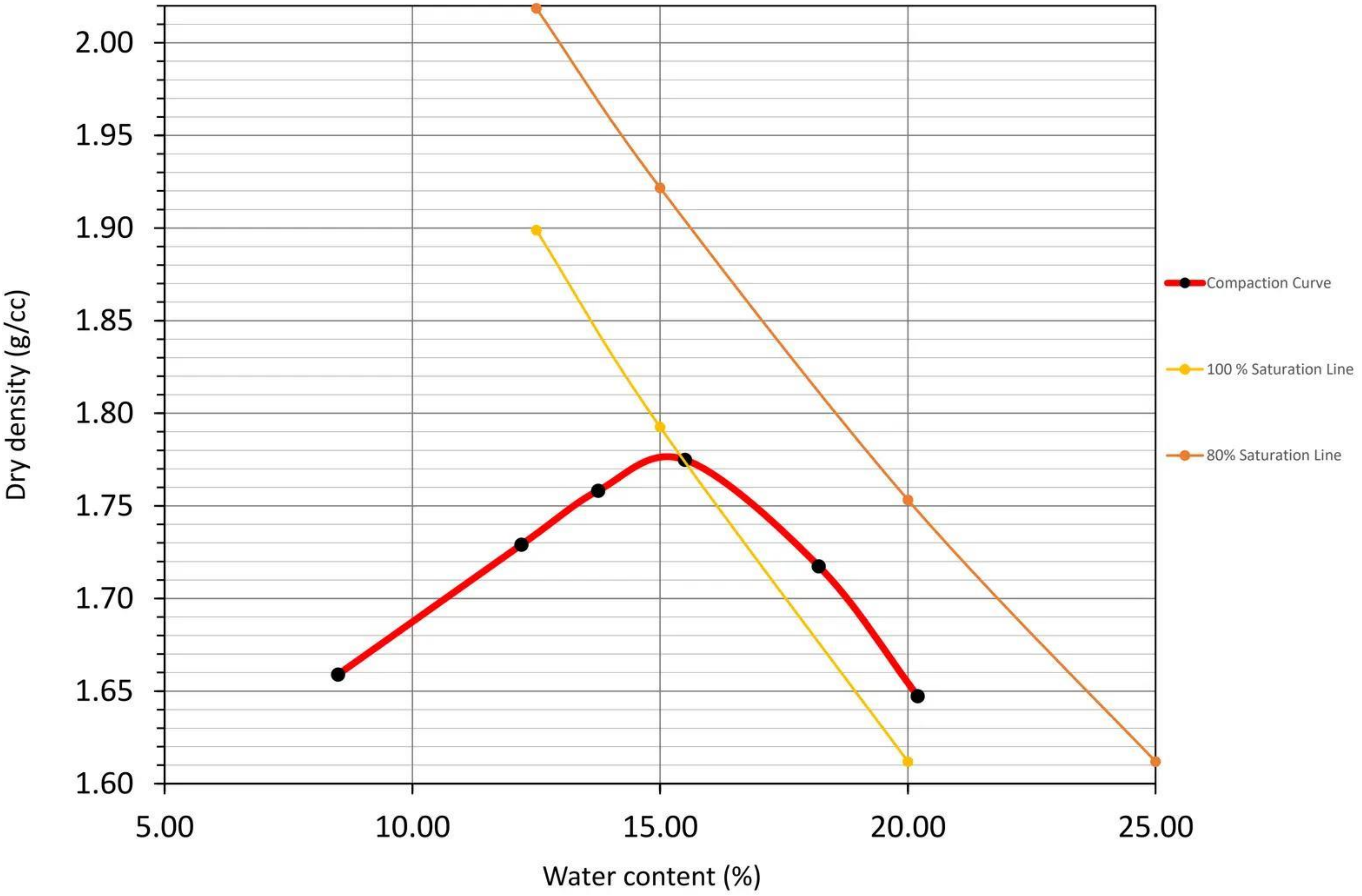
Numerical

1. The in-situ void ratio of a granular soil deposit is 0.50. The maximum and minimum void ratios of the soil were determined to be 0.75 and 0.35. Gs =2.67. Determine the relative density and relative compaction of the deposit.
2. The following data refers to a compaction test:

Water content %	8.5	12.2	13.75	15.5	18.2	20.2
Weight of wet sample per (kg)	1.80	1.94	2.00	2.05	2.03	1.98

If the specific gravity of soil grains was 2.7, volume of mould = 1000 cc,

a. Plot the compaction curve and obtain the maximum dry unit weight and the optimum moisture content.
b. Plot the 80% and 100% saturation lines.
c. If it is proposed to secure a relative compaction of 95% in the field, what is the range of water content that can be allowed.
d. Would the 20% air voids curve be the same as the 80% saturation curve?



2.00
1.95
1.90
1.85
1.80
1.75
1.70
1.65
1.60
5.00
10.00
15.00
20.00
25.00
Dry density (g/cc)
Water content (%)
Compaction Curve
100 % Saturation Line
80% Saturation Line

Stresses in Soil

Week 12-13

Pages 196-197

Total stress

•The total weight of the soil and water above a given point, per unit area of soil. It also includes any externally applied loads.

•**Pore water pressure**

•The pressure of water in the soil pores. This pressure depends on the depth and the flow of water.

•**Effective stress**

•The difference between total stress and pore water pressure. It's the amount of stress that the soil particles resist.

Explanation

•The total stress on soil is carried by both the soil grains and the water.

•The effective stress is the stress carried by the soil skeleton.

•The combined effect of total stress and pore water pressure controls how soil behaves, such as its strength, compression, and distortion.

•Changes in total stress or pore water pressure can cause ground movements and instabilities. For example, rainfall can increase pore water pressure, which can cause slopes to fail.

Click here

Total Stress, Effective and Pore water pressure related math

Permeability of Soil

Week 14

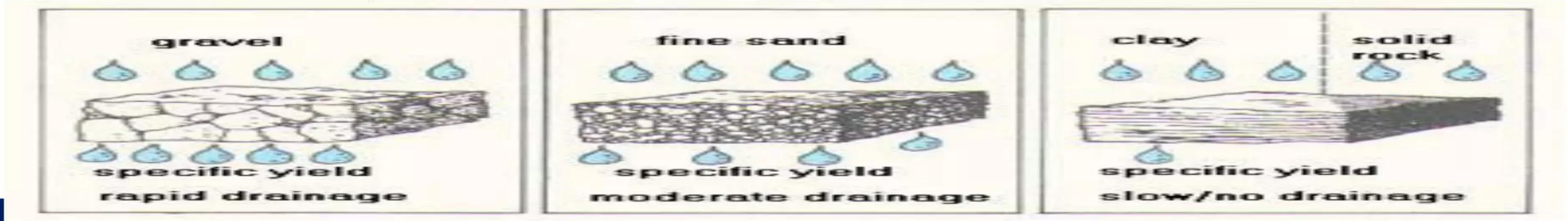
Pages 199-238

WHAT IS PERMEABILITY?

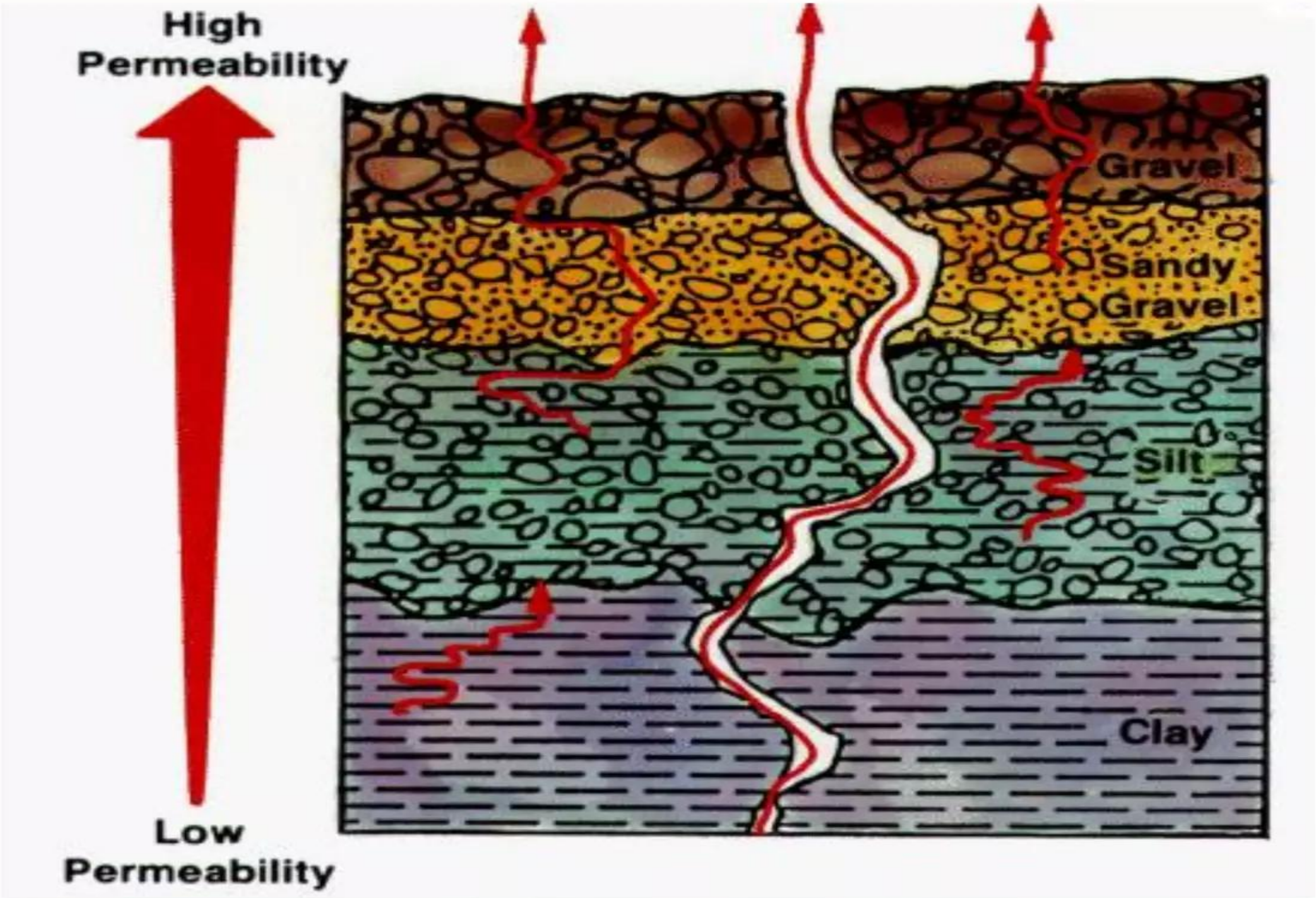
- Permeability is defined as the property of a porous material which permits the passage or seepage of water through its interconnecting voids.
- A measure of how easily a fluid (e.g., water) can pass through a porous medium (e.g., soils)
- It is a very important Engineering property

gravels $\longrightarrow$ highly permeable

stiff clay $\longrightarrow$ least permeable

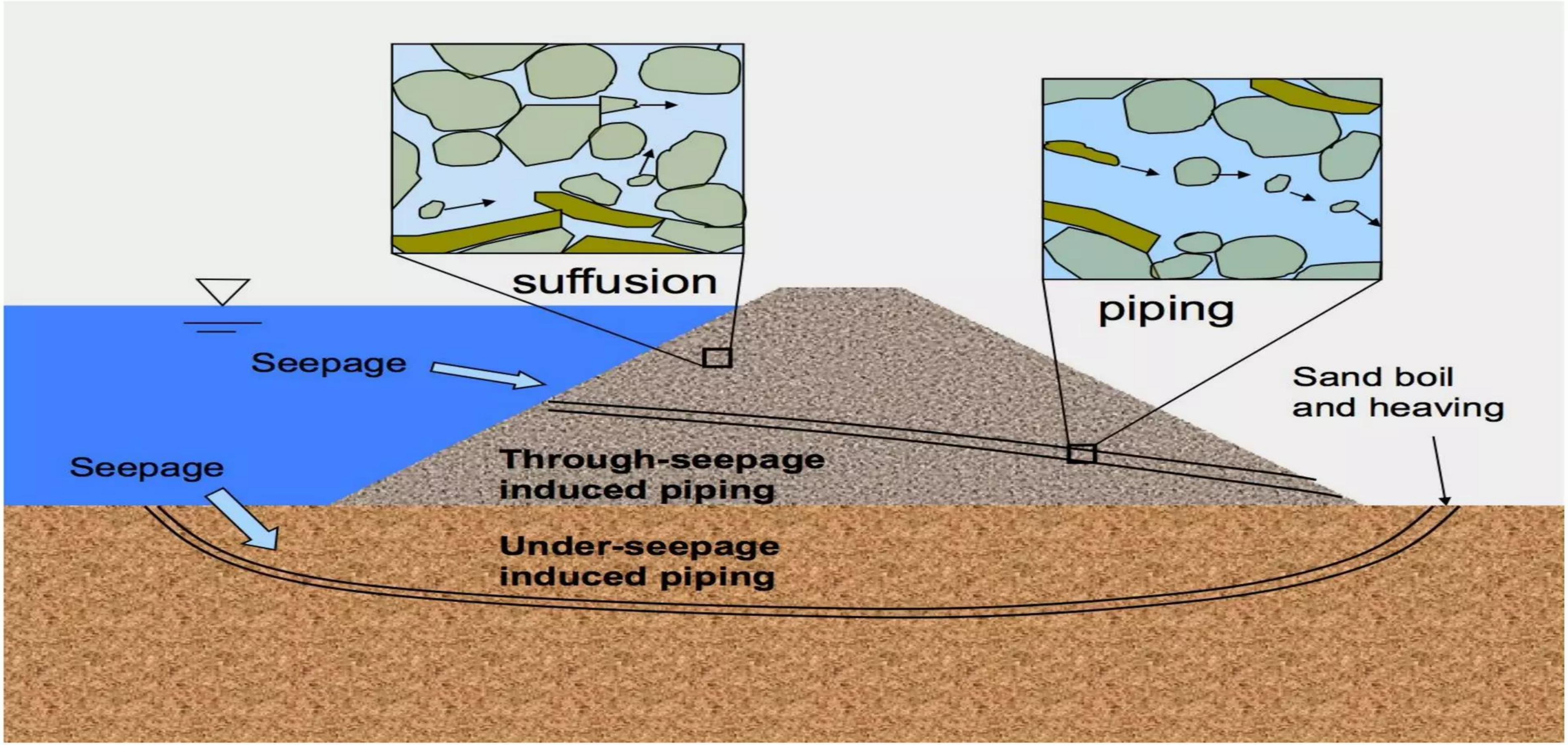



Compare
2 minutes
2 hours
200 days
200 years
1 meter
Gravel
Sand
Silt
Clay


High
Permeability
Gravel
Sandy
Gravel
Silt
Clay
Low
Permeability

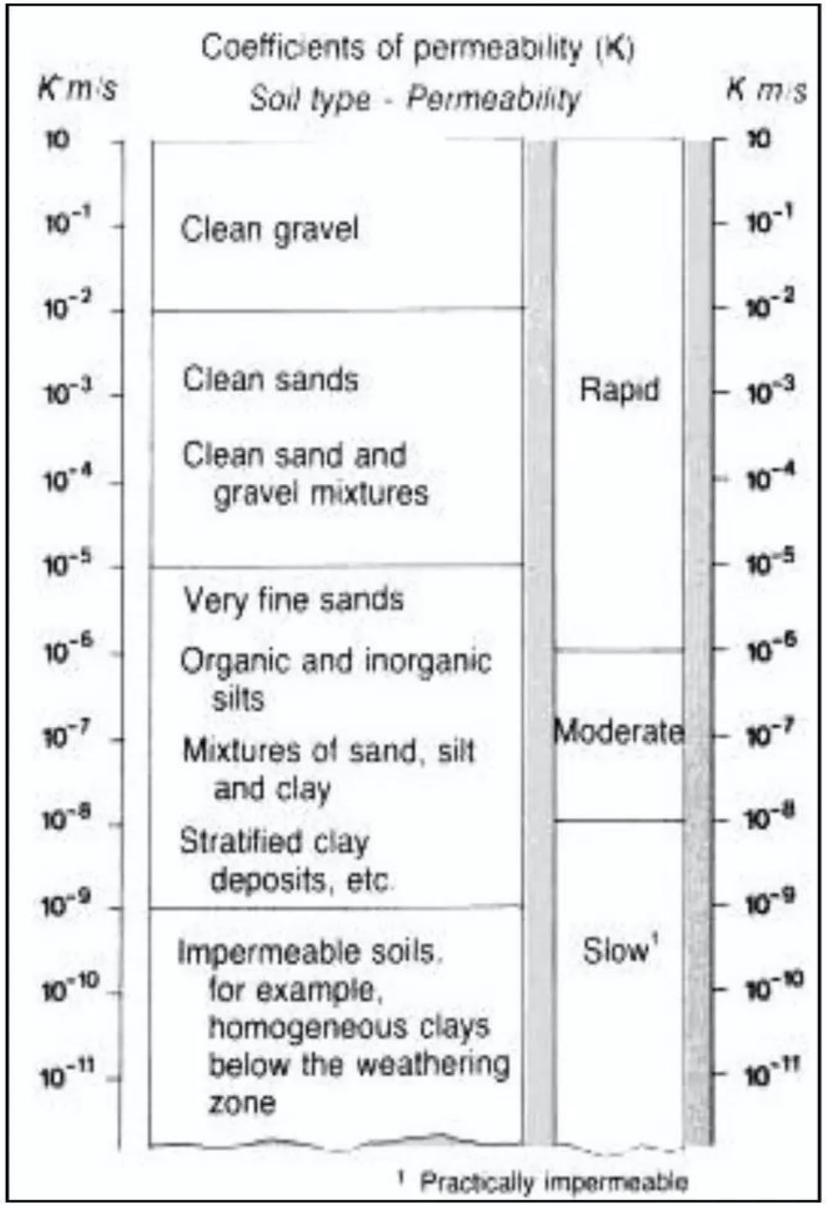
Permeability through soil is important for the following engineering problems:

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



Phenomenon of piping action

Failed slope surface



How permeability is measured?

- Permeability is measured in terms of rate of flow of water through the soil in a given period of time.
- It is expressed in cm/day or cm/hour or m/sec.

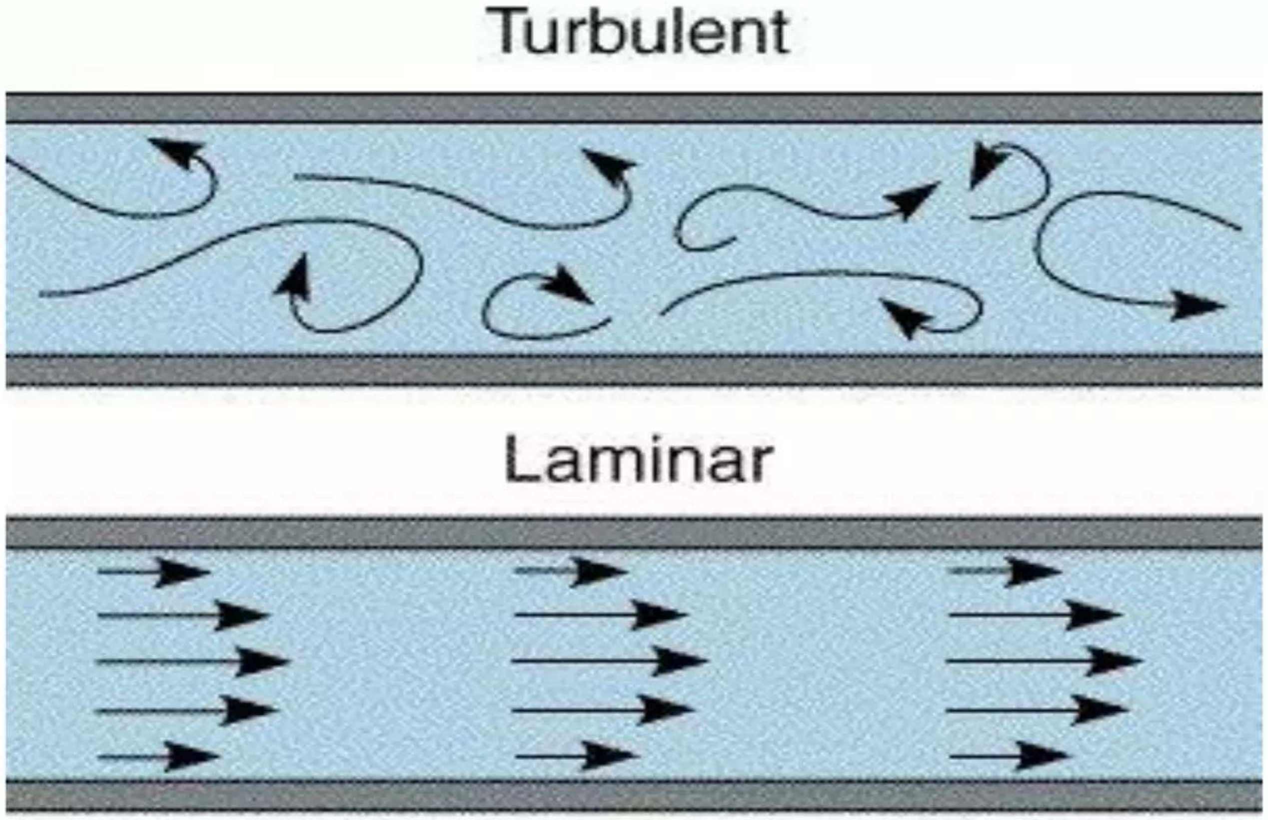
Soil type	k (cm/sec)	k (ft/min)
Clean gravel	100–1.0	200–2.0
Coarse sand	1.0–0.01	2.0–0.02
Fine sand	0.01–0.001	0.02–0.002
Silty clay	0.001–0.00001	0.002–0.00002
Clay	<0.000001	<0.000002

Flow of water through Soils

Flow of water through soils may either be a **laminar flow** or a **turbulent flow**

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

Paths are irregular and twisting, crossing at random



***Henry Darcy* (1803-1858), Hydraulic Engineer.**

His law is a foundation stone for several fields of study

Assumptions of Darcy's law :-

- The following assumptions are made in Darcy's law.

➢ The soil is saturated.

➢ The flow through soil is laminar.

➢ The flow is continuous and steady.

➢ The total cross sectional area of soil mass is considered.

➢ The temperature at the time of testing is 27^0C.

Henry Darcy **(1803-1858), Hydraulic Engineer.**

His law is a foundation stone for several fields of study

Darcy's Law: Darcy demonstrated experimentally that for laminar flow conditions in a saturated soil, the rate of flow or the discharge per unit time is proportional to the hydraulic gradient

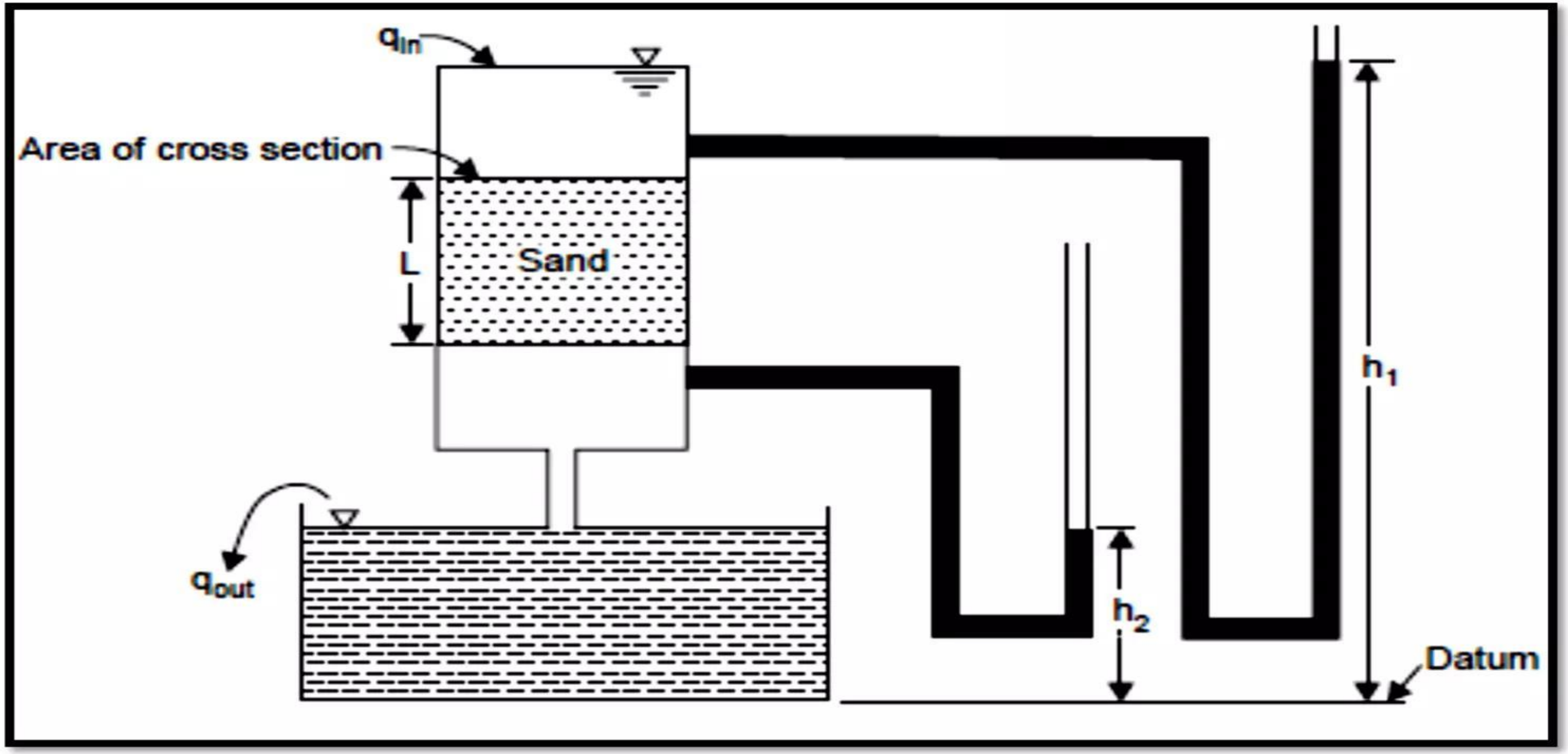
$$q = vA$$

$$v = ki$$

$$q = kiA$$

Validity of Darcy's law - When flow is laminar

Darcy's Law



Darcy's Experiment.

VALIDITY OF DARCY'S LAW

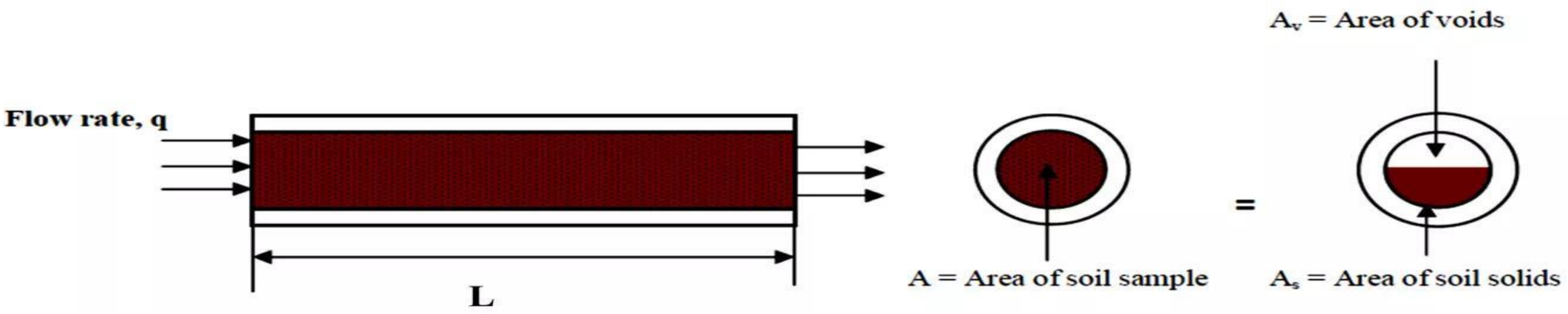
1) Darcy's law is valid if the flow through soils is laminar:

- The flow of water through soils depends upon the dimension of particles. In fine grained soils the dimensions of the interstices (voids) are very small and flow is necessarily laminar.
- In course- grained soil, the flow is also laminar. However, in very coarse-grained soils, such as gravels, the flow may be turbulent.
- For flow through soils, the flow is laminar if the Reynolds number is less than unity.

VALIDITY OF DARCY'S LAW

2) As per Allen Hazen, the maximum diameter of the particle for the flow to be laminar is about 0.50 mm.
3) It is valid for flow in clays, slits and fine sands. In coarse sands, gravels and boulders, the flow may be turbulent, and Darcy's law may not be applicable.
4) For Darcy's law to be valid, the relationship between velocity (v) and hydraulic gradient (i) should be linear.
5) In extremely fine-grained soils, such as colloidal clay, the interstices are very small. The velocity is therefore very small. In such soils, the Darcy's law is not valid.

Relation between Discharge Velocity and Seepage Velocity:



From Continuity equation we have, q = v.A = k.i.A

But, A = total area including voids and solids

But in actual practice flow of water through soil takes place through voids only, the actual velocity will be more than discharge velocity. This actual velocity is called as *seepage velocity* (vs), and is defined as rate of discharge of percolating water per unit cross-sectional area of voids perpendicular to the direction of flow.

Relation between Discharge Velocity and Seepage Velocity:

Therefore,

$$q = v.A = vs.Av$$

$$vs = v \times A/Av$$

But, $A/Av = V/Vv = 1/n$

$$vs = v \times 1/n$$

from definition we have, $n = e/1+e$

$$vs = v \times (1 + e/e)$$

Hence from above equation it is clear that seepage velocity > discharge velocity.

Factors affecting Permeability of soil

There is an equation known as **Poiseuille's equation**, which when compared with the Darcy's law gives the value of coefficient of permeability,

$$\mathbf{k} = \frac{\gamma_w}{n} * D_s^2 * \frac{e^3}{(1+e)} \times \mathbf{C}$$

In above equation, the factor k depends on:

1. **Grain Size:** Allen Hazen found that the co-efficient of permeability can be expressed as

$$\text{k (in cm)} = C \times D_{10}^2$$

where, D_{10}= effective diameter(cm), C = constant, approximately equal to 100.

From the above relation is clear that greater the grain size of soil greater will be the permeability and vice-versa

Factors affecting Permeability of soil

2. **Properties of pore fluid:** Viscosity of the fluid affects the flow, more the viscosity less is the fluidity, so it is inversely proportional to the viscosity.

$$k_1/k_2 = n_2/n_1$$

3. **Void ratio:** Based on the mean hydraulic radius it can be written that,

$$k_1/k_2 = (e_1/e_2)^2$$

From the above relation it is clear that co-efficient of permeability is directly proportional to void ratio. So, greater the void ratio greater will be the permeability of soil and vice-versa.

Factors affecting Permeability of soil

4. **Structural arrangements:** Depending upon the method of compaction and condition existing during deposition, the structural arrangement of soil mass may vary at the same void ratio and consequently permeability also varies. it is defined by the factor C is the above equation. For stratified deposits the flow is different for flow parallel and perpendicular or stratification.
5. **Degree of saturation and other foreign matter**:

The permeability is affected by the partial saturation of the soil, when it has entrapped the air inside. Other foreign matter has the general tendency to flow towards the critical flow channel and thus choking it.

Factors affecting Permeability of soil

6. **Adsorbed water:** The adsorbed water held on the surface of colloidal particles are highly cohesive and is immobile to normal hydrodynamic forces. Because of this the area of effective void space is reduced and thus permeability is reduced.

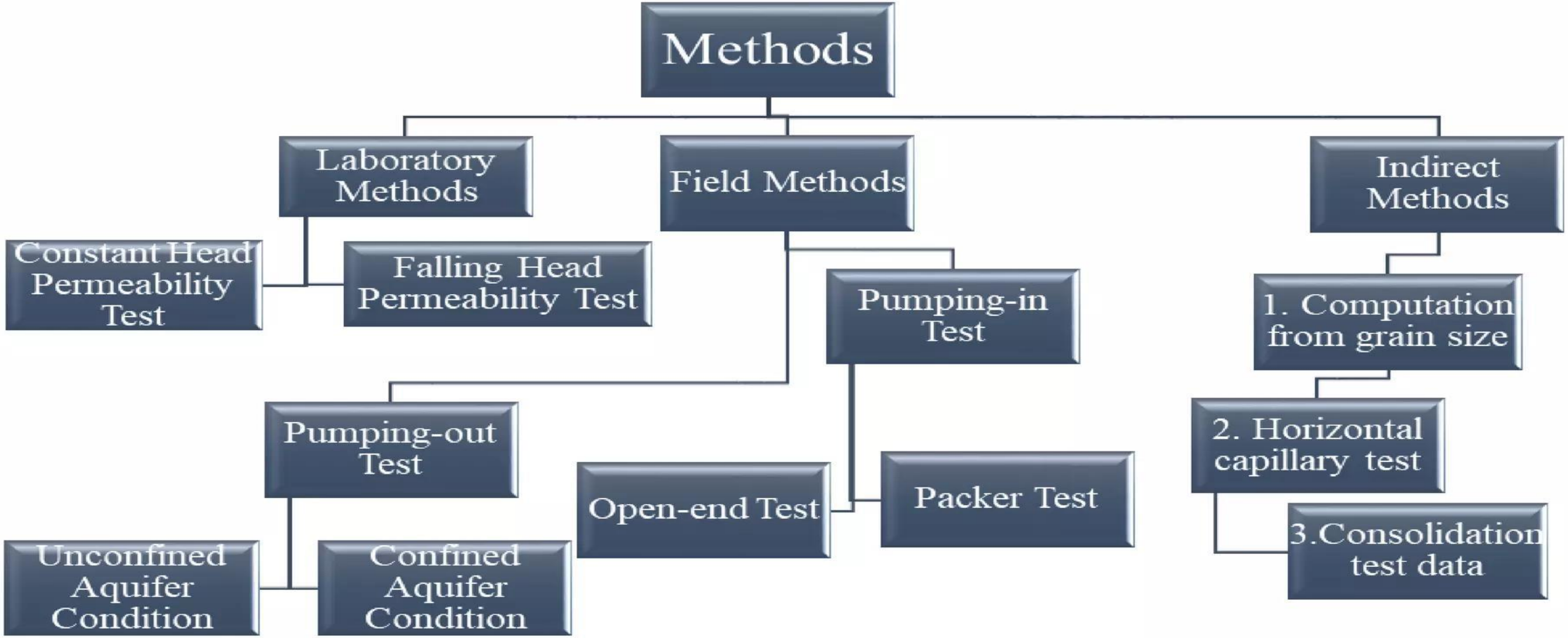
Absolute permeability

- Independent of the properties of water
- It depends only on the characteristics of soil
- The absolute permeability only depends on the geometry of the pore-channel system.

Relative permeability

It is the ratio of effective permeability of a particular fluid to its absolute permeability.

Methods of Determination of co-efficient of Permeability:



Constant Head Permeability Test

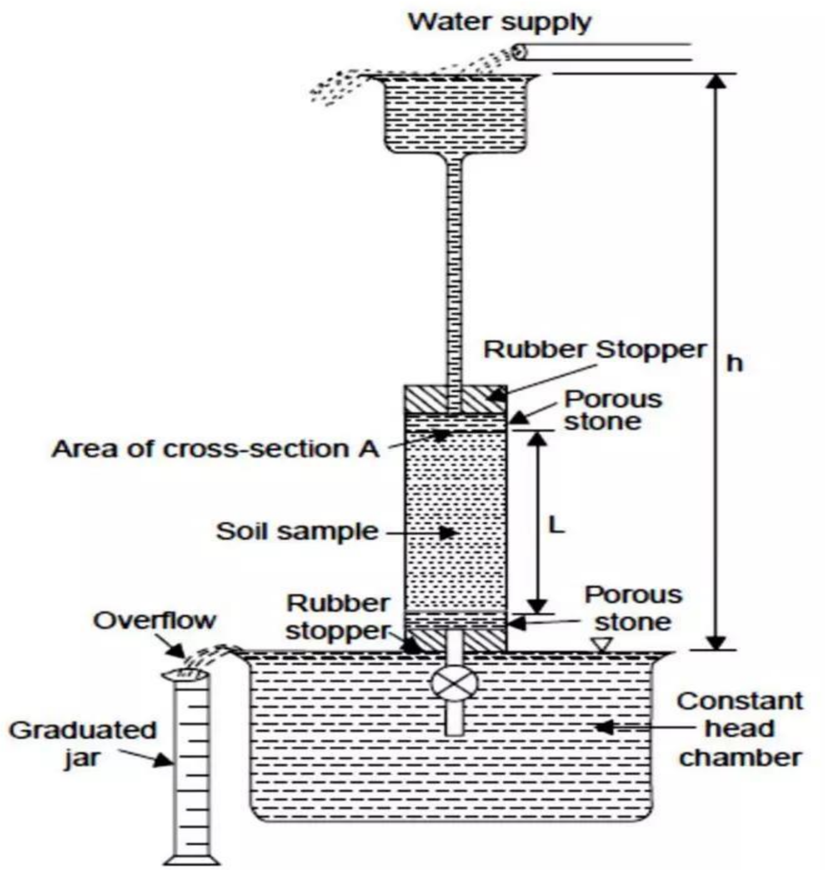
- Quantity of water that flows under a given hydraulic gradient through a soil sample of known length & cross sectional area in a given time
- Water is allowed to flow through the cylindrical sample of soil under a constant head
- For testing of pervious, coarse grained soils

$$k = \frac{QL}{Aht}$$

K = Coefficient of permeability

Q = total quantity of water

t = time, L = Length of the coarse soil



Setup for Constant Head Test

Constant Head Permeability Test

Test Procedure:-

1. Measure internal dimensions of the mould. Apply a little grease on the inside to the mould.
2. Take about 2.5kg of the soil, from a thoroughly mixed wet soil, in the mould. Compact the soil at the required dry density using a suitable compacting device.
3. Remove the collar and base plate. Trim the excess soil level with the top of the mould.
4. Clean the outside of the mould. Find the mass of the soil in the mould. Take a small specimen of the soil in container for the water.

Constant Head Permeability Test

6. Place the porous stone (disc) on the drainage base and keep a filter paper on the porous stone. Place a filter paper and a porous stone on the top of specimen.
7. Connect the constant head tank to the drainage cap inlet.
8. Open the stop cock, and allow the water downward so that all the air is removed, then close the stop cock.
9. Now, again open the stop cock and at the same time start the stopwatch. Collect the water flowing out of the base in a measuring flask for some convenient time interval.
10. Measure the difference of head (h) in levels between the constant head tank and the outlet in the base.

Falling Head Permeability Test

- Relatively for less permeable soils
- Water flows through the sample from a standpipe attached to the top of the cylinder.
- The head of water (h) changes with time as flow occurs through the soil. At different times the head of water is recorded.

$$k = \frac{2.30aL}{At}\log_{10}\frac{h_1}{h_2}$$

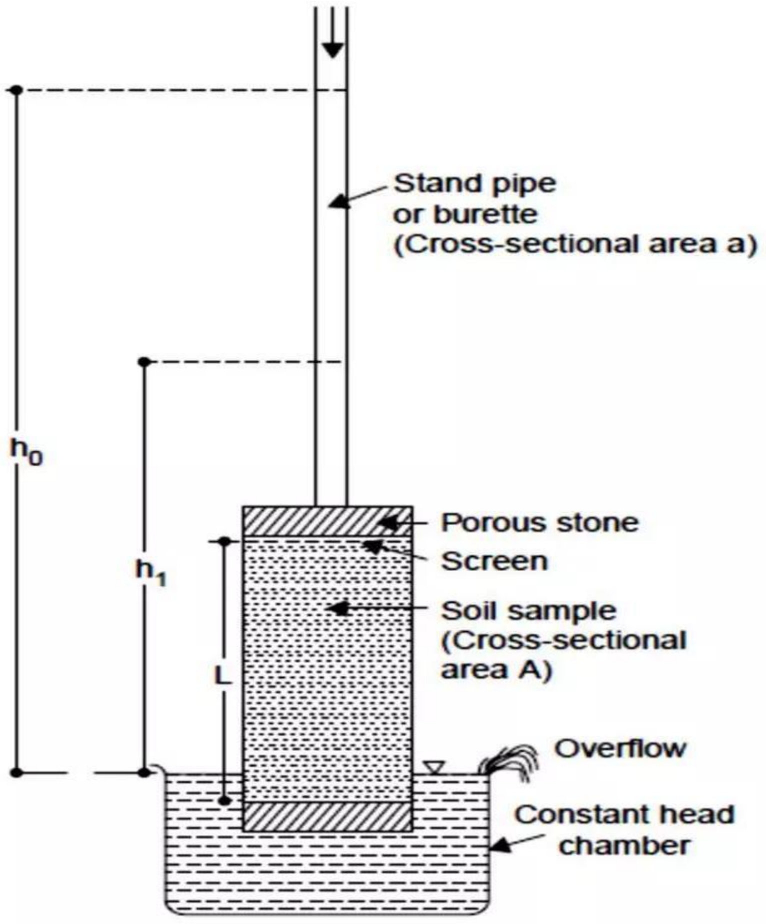
t = time

L = Length of the fine soil

A = cross section area of soil

a = cross section area of tube

K = Coefficient of permeability



Setup for Falling Head Test

Falling Head Permeability Test

Test Procedure:-

1. Prepare the remoulded soil specimen in the permeameter and saturate it.
2. Keep the permeameter mould in the bottom tank and fill the bottom tank with water up to its outlet.
3. Connect the water inlet nozzle of the mould to the stand pipe filled with water.
4. Permit water to flow for some time till steady state of flow is reached.
5. Now open the valve of stand pipe and record the time (t) to fall the head from h_1 to h_2. Repeat this step at least twice.

By Indirect Method

Allen hazen's formula

$$k = cD_{10}{}^2$$

Loudon's formula

$$\log_{10}(kS^2) = a + bn$$

Consolidation test data

$$k = C_v \gamma_w m_v$$

Kozeny carman equation

$$k = \frac{g\rho_w}{(C_s \mu S^2)T^2} \frac{e^3}{1+e}$$

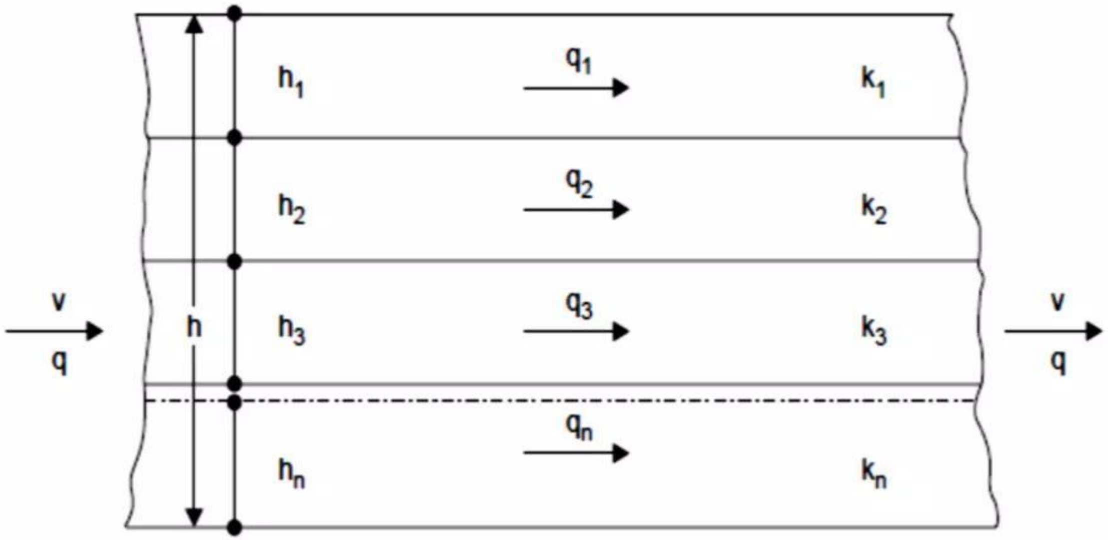
Constants, a = 1.365
b = 5.15
c = value b/w 100 & 150
T= Tortuosity
S = surface area

Permeability in Stratified Soils

Flow parallel to the plane of stratification

Let h_1, h_2, h_3 ...hn be the thicknesses of each of the n layers which constitute the deposit, of total thickness h. Letk_1, k_2, k_3 ... k_n be the Darcy coefficients of permeability of these layers respectively.

Let kx = average permeability of soil deposit parallel to bedding plane.



Total discharge through the soil deposit = sum of discharge through individual layers

$$q = q_1 + q_2 + \cdots q_n$$

$$k_x . i . h = k_1 . i . h_1 + k_2 . i . h_2 + \cdots k_n . i . h_n$$

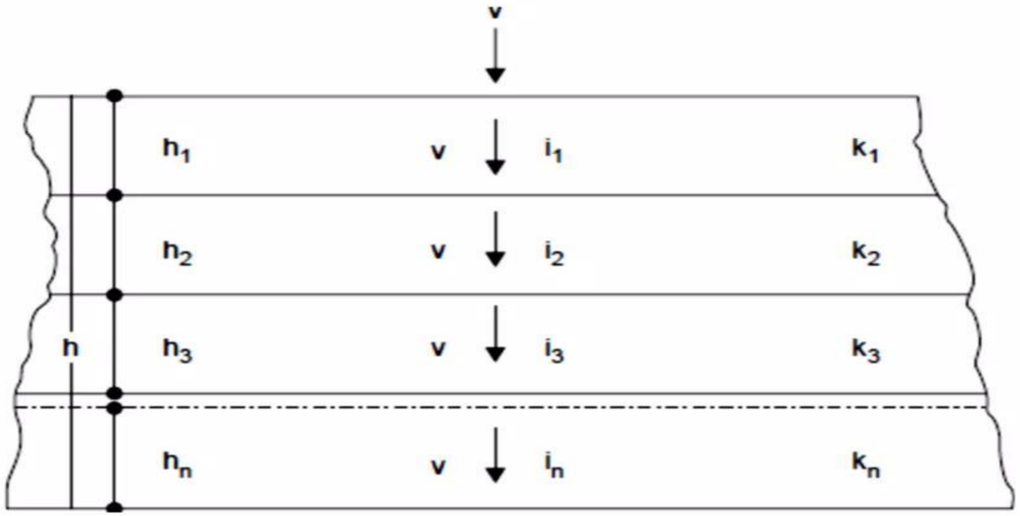
$$k_x = \frac{k_1 . h_1 + k_2 . h_2 + \cdots k_n . h_n}{h}$$

Permeability in Stratified Soils

Flow normal to the plane of stratification

Let h_1, h_2, h_3 ...h_n be the thicknesses of each of the n layers which constitute the deposit, of total thickness h. Let k_1, k_2, k_3 ... k_n be the Darcy coefficients of permeability of these layers respectively.

Let k_z = average permeability of soil deposit perpendicular to bedding plane.



In this case, $h = h1 + h2 + \ldots h_n$

But $h_1 = i_1 * h_1$, $h_2 = i_2 * h_2$, Similarly $h_n = i_n * h_n$

We know, v= $i*kz$, Hence h= v$*hkz$

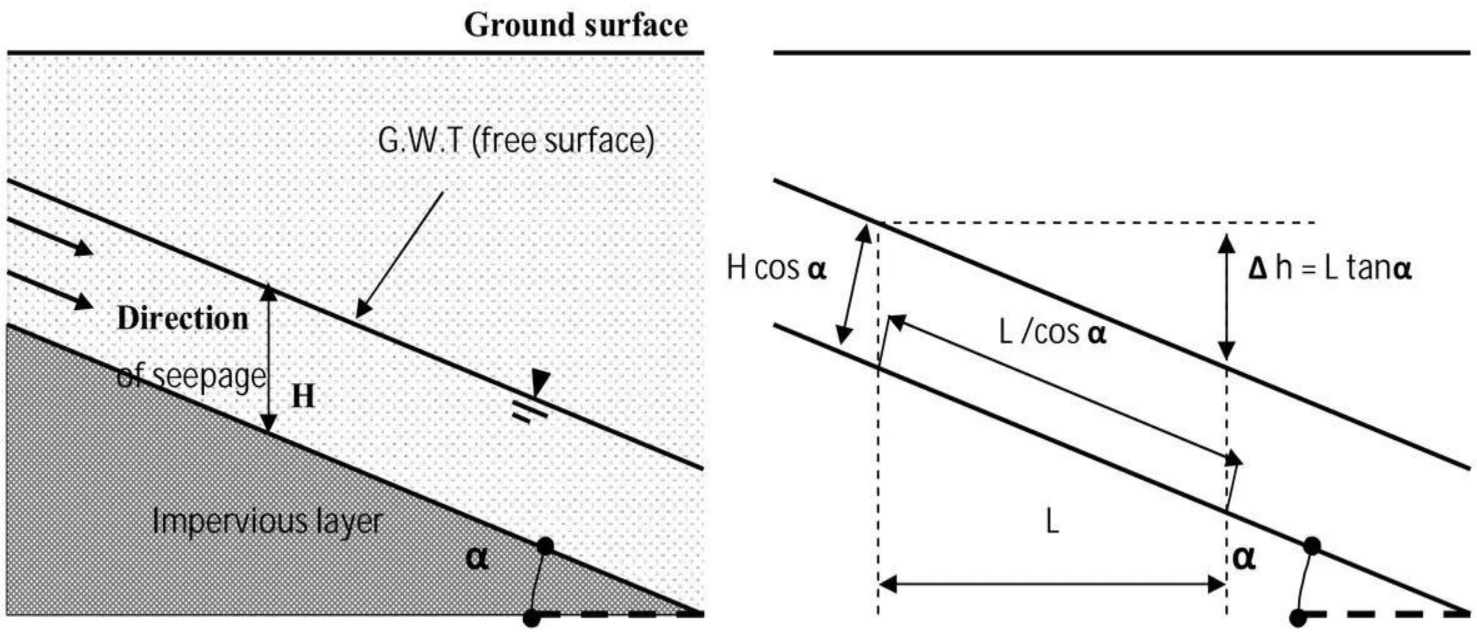
Therefore, v$*hkz$= v$*h_1k_1$ + v$*h_2k_2$ +...... v$*h_nk_n$

$kz = hh_1k_1 + h_2k_2 + \ldots h_nk_n$

$$k_z = \frac{h}{\frac{h_1}{k_1} + \frac{h_2}{k_2} + \cdots \frac{h_n}{k_n}}$$

Examples

1. An impervious layer as shown in the figure underlies a permeable soil layer. With k = 4.8x10^{-3} cm/sec for the permeable layer, calculate the rate of seepage through it in cm^3/sec/cm length width. Given H = 3 m and α = 5^o .



Solution

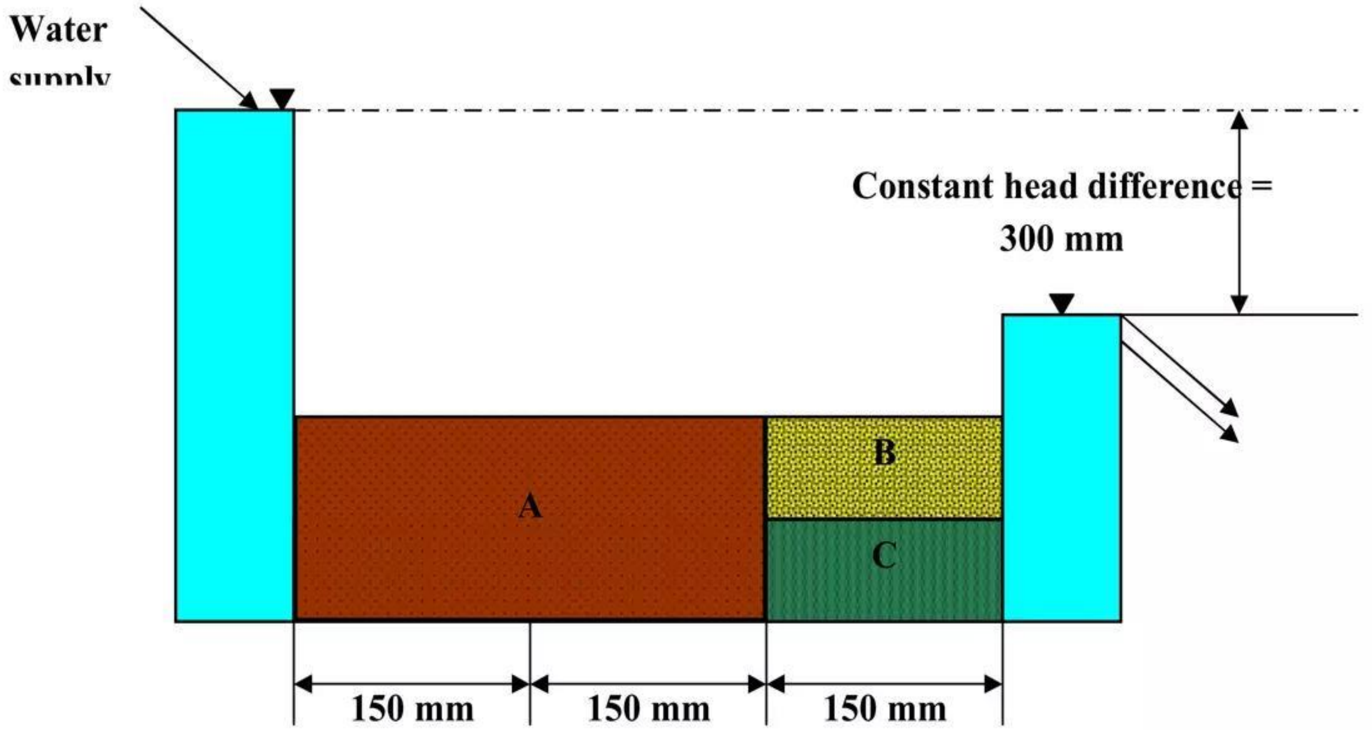
From the above figure

$$\boldsymbol{i}=\frac{\boldsymbol{headloss}}{\boldsymbol{length}}=\frac{\boldsymbol{L}\tan\alpha}{\left(\dfrac{\boldsymbol{L}}{\cos\alpha}\right)}=\sin\alpha$$

$$\boldsymbol{q}=\boldsymbol{kiA}=(\boldsymbol{k})(\sin\alpha)(\boldsymbol{H}\cos\alpha.1)=\left(4.8\boldsymbol{x}10^{-4}\right)(\sin 5).(3\cos 5.)=12.5\boldsymbol{x}10^{-4}$$

$\boldsymbol{q}=12.5$ cm^3/sec/cm length

2. *The following figure shows the layers of soil in a tube 100mmx100mm in cross – section. Water is supplied to maintain a constant head difference of 300 mm across the sample. The permeability coefficient of the soils in the direction of flow through them are as follows: Find the rate of supply.*



Soil	k (cm/sec)
A	$1x10^{-2}$
B	$3x10^{-3}$
C	$5x10^{-4}$

Solution

For the soil layers B & C (the flow is parallel to the stratification)

$$k_{H(eq)} = \frac{1}{H}\left(k_{h1}H_1 + k_{h2}H_2\right) = \frac{1}{10}(3x10^{-3}(5) + 5x10^{-4}(5)) = 1.75x10^{-3} \quad \text{cm/sec}$$

For the layer A with equivalent layer of B&C

$$\therefore k_{eq} = \frac{H}{\frac{H_1}{k_1} + \frac{H_2}{k_2}} = \frac{45}{\frac{30}{1x10^{-2}} + \frac{15}{1.75x10^{-3}}} = 3.8x10^{-3}$$

$$k_{eq} = 0.003888 cm/\sec$$

$$q = k_{eq} iA = 0.003888 \frac{300}{450} (10)^2 = 0.259\ cm^3/\sec$$

3. The permeability coefficient of a sand at a void ratio of 0.55 is 0.1 ft/min. estimate its permeability coefficient at avoid ratio of 0.7. Use Casagrande empirical relationship

Solution

From Casagrande relation $k = 1.4e^2 k_{0.85} \Rightarrow k \propto e^2$ ***.So***

$$\frac{k_1}{k_2} = \frac{e_1^2}{e_2^2} \Rightarrow \frac{0.1}{k_2} = \frac{(0.55)^2}{(0.7)^2} \Rightarrow k_2 = \frac{(0.1)(0.7)^2}{(0.55)^2} = 0.16 \text{ ft/min at } e = 0.7$$

4.for normally consolidated clay soil, the following are given:

Void ratio	*k (cm/sec)*
1.1	0.302×10^{-7}
0.9	0.12×10^{-7}

Estimate the permeability coefficient of clay at void ratio of 1.2 .

Use Samarasingh et. al. relation.

Solution

Samarasingh et.al. eq. $k = C_3 \dfrac{e^n}{1+e}$

$$\therefore \frac{k_1}{k_2} = \frac{\left(\dfrac{e_1^n}{1+e_1}\right)}{\left(\dfrac{e_2^n}{1+e}\right)}$$

$$\frac{03.02x10^{-7}}{0.12x10^{-7}} = \frac{\dfrac{(1.1)^n}{1+1.1}}{\dfrac{(0.9)^n}{1+0.9}} \Rightarrow 2.517 = \left(\frac{1.9}{2.1}\right)\left(\frac{1.1}{0.9}\right)^n$$

$$\therefore 2.782 = (1.222)^{n}$$

$$n = \frac{\log(2.782)}{\log(1.222)} = \frac{0.444}{0.087} = 5.1$$

So

$$k = C_3\left(\frac{e^{5.1}}{1+e}\right)$$

To find C_3

$$0.302x10^{-7} = C_3\left[\frac{(1.1)^{5.1}}{1=1.1}\right] = \left(\frac{1.626}{2.1}\right)C_3$$

$$C_3 = \frac{\left(0.302x10^{-7}\right)(2.1)}{1.626} = 0.39x10^{-7}\,cm/\sec$$

Hence

$$k = \left(0.39x10^{-7}\right)\left(\frac{e^{5.1}}{1+e}\right)$$

At a void ratio of 1.2

$$k = \left(0.39x10^{-7}\right)\left(\frac{1.2^{5.1}}{1+1.2}\right) = 0.449x10^{-7}\ \text{cm/sec.}$$

5. pumping test from Gravity well in a permeable layer underlain by an impervious stratum was made. When steady state was reached, the following observations were made q = 100 gpm; h_1 = 20 ft; h_2 = 15 ft; r_1 = 150 ft; and r_2 = 50 ft. Determine the permeability coefficient of the permeable layer.

<u>Solution</u>

Since $k = \dfrac{2.303q\log_{10}\left(\dfrac{r_1}{r_2}\right)}{\pi\left(h_1^2 - h_2^2\right)}$

Given: q = 100gpm = 13.37 ft^3 / min, so

$$k = \frac{2.303x13.37\log_{10}\left(\frac{150}{50}\right)}{\pi\left(20^2 - 15^2\right)} = 0.0267\, ft/\min \approx 0.027\, ft/\min$$

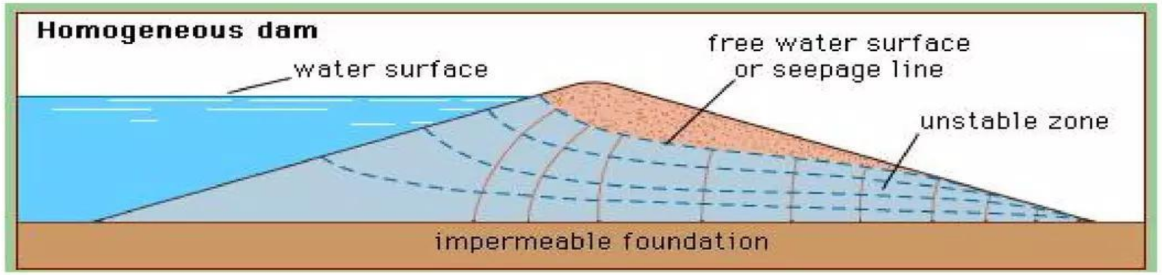
Seepage

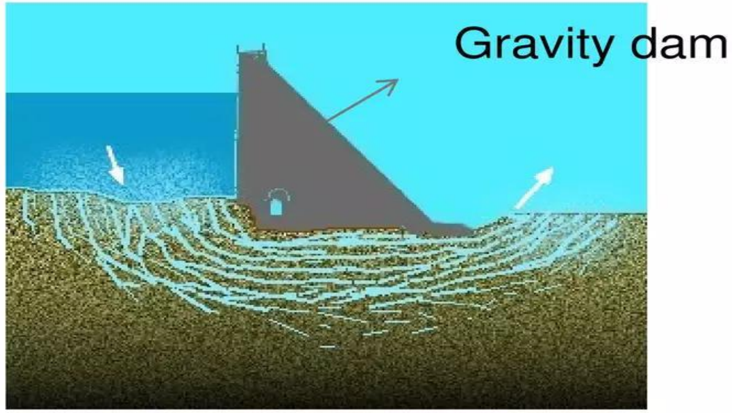
Week 15

Pages 240-261

SEEPAGE

***Seepage* is the slow escape of a liquid or gas through porous material or small holes**





Seepage is one of the applications of ground water hydraulic. The aims of studying seepage in civil engineering are:

1- To find the discharge of seepage through and beneath the structure

2- To find up lift pressure under the structure

3- To find solution for foundation and piping failures

PHERATIC LINES

- Phreatic line also known as the seepage line or saturation line is an imaginary line which separates saturated zone from unsaturated zone in an earthen dam. It is also known as Hydraulic gradient line.
- Along the phreatic line, the atmospheric pressure exists (which is equal to zero) at the face of phreatic line.
- Above the phreatic line there is a negative hydrostatic pressure in the dam section and below which there is positive hydrostatic pressure.

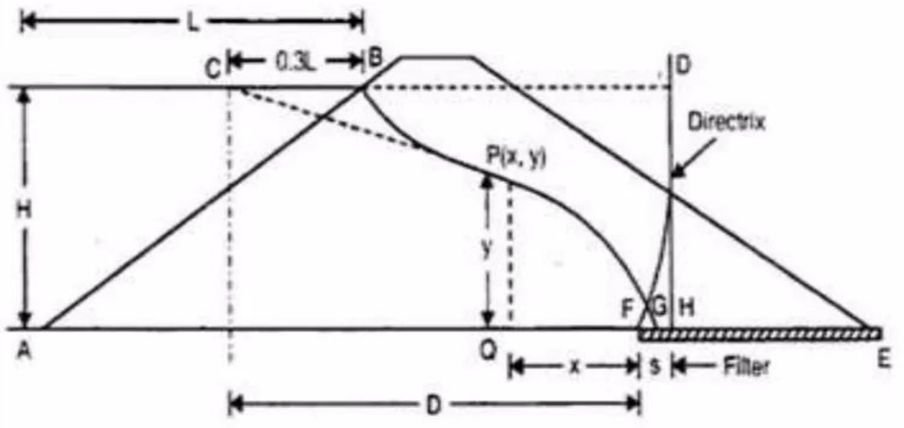
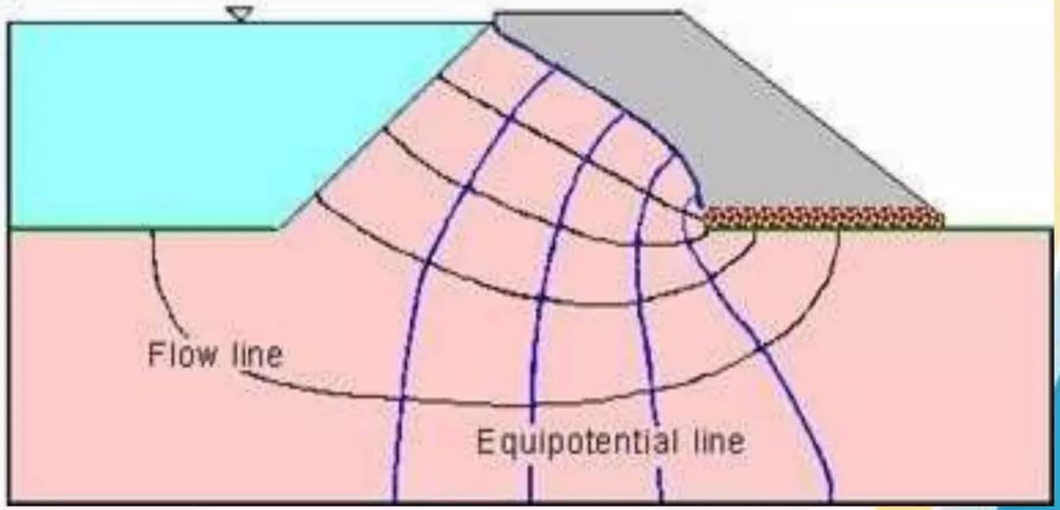


Fig. 26.2. Phreatic line with horizontal drainage filter.

(Source: Punmia, 1994)

FLOWNETS

- When water flows through the interconnecting voids of a soil mass it follows a certain path.
- Flownet is the graphical representation of flow of water thgrough the interconnecting voids of the soil mass.
- A flownets consists of two groups of curves. These are flow lines and equipotential lines. These curves bear a fixed relationship to each other.



Some Definitions

- FLOW LINES--are the paths which water particle follow in the course of seepage through a soil mass under laminer flow comdition. Water flows from the point of higher head to low head. Flow lines represent the pattern of flow.
- EQUIPOTENTIAL LINES--are the lines formed by joining the points of same head or potential on the flow lines.
- FLOW NETS -If we draw the flow lines and the equipotential lines for a given flow of water through a given soil we get a net like sketch which is known as flow net.
- Flow Channels- The space between two adjacent flow lines is called flow path or channel.
- Field- The figure formed on the flow net between any two adjacent flow lines and adjacent equipotential lines is refferd to as field.

Properties of Flow net:

1. Flow lines and equal potential lines intersect each other at 90 degrees.
2. The areas bounded by the flow lines and equal potential lines form approximate squares.
3. Flow nets must satisfy the boundary conditions of flow field.
4. Quantity of water flowing through each flow channel is the same.
5. The potential drop in any two consecutive equal potential lines is same/constant.
6. Flow lines and equal potential lines are smooth curves.
7. Flow lines do show refraction at the interface between two soils having different coefficient of permeability.
8. Darcy's law is valid.

APPLICATION OF FLOWNETS

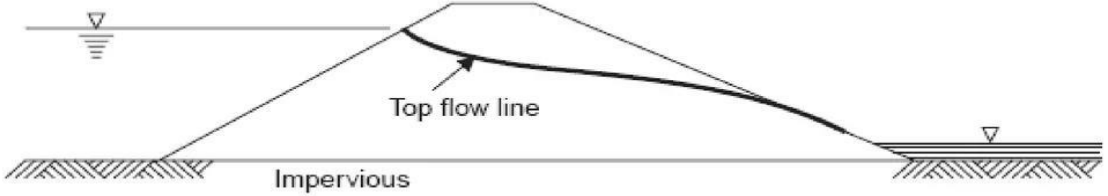
1. **Estimation of seepage losses from reservoirs:** It is possible to use the flow net in the transformed space to calculate the flow underneath the dam.

2. **Determination of uplift pressures below dams:** From the flow net, the pressure head at any point at the base of the dam can be determined. The uplift pressure distribution along the base can be drawn and then summed up.

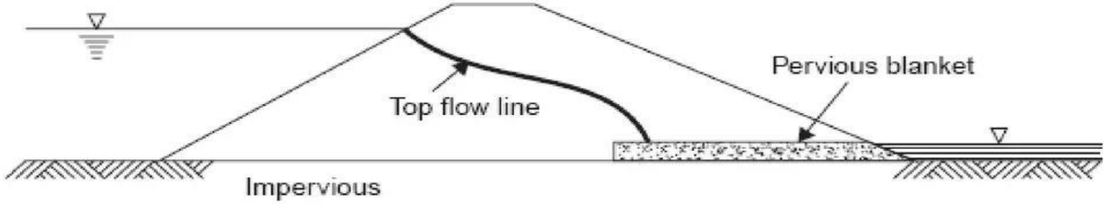
3. **Checking the possibility of piping beneath dams:** At the toe of a dam when the upward exit hydraulic gradient approaches unity, boiling condition can occur leading to erosion in soil and consequent piping. Many dams on soil foundations have failed because of a sudden formation of a piped shaped discharge channel.

- Seepage through an Earth Dam

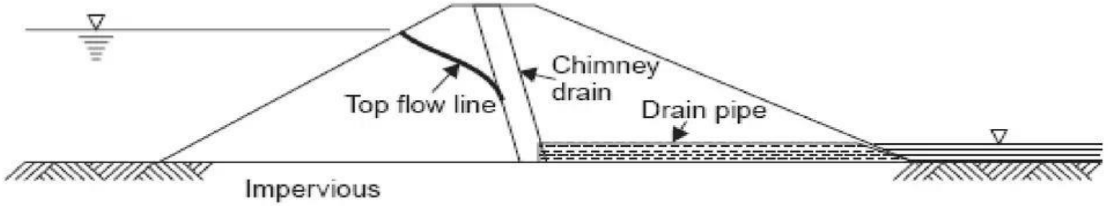
The flow through an earth dam differs from the other cases in that the top flow line is not know in advance of sketching the flow net. Thus, it is a case of ***unconfined flow***. The top flow line as well as the flow net will be dependent upon the nature of internal drainage for the earth dam. Typical cases are shown in Fig. 6.8; the top flow line only is shown.



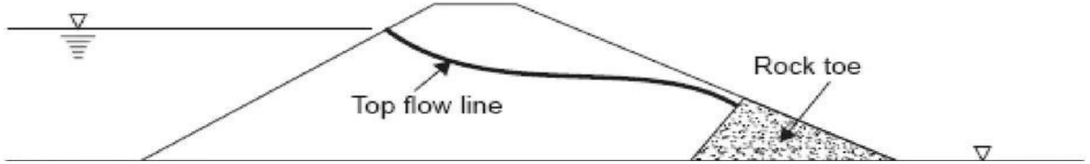
(a) Homogeneous dam without internal drain



(b) Homogeneous dam with underdrain on pervious blanket



(c) Homogeneous dam with chimney drain



Assuming that the top flow line is determined, a typical flow net for an earth dam with a rock toe, resting on an impervious foundation is shown in Fig. 6.9:

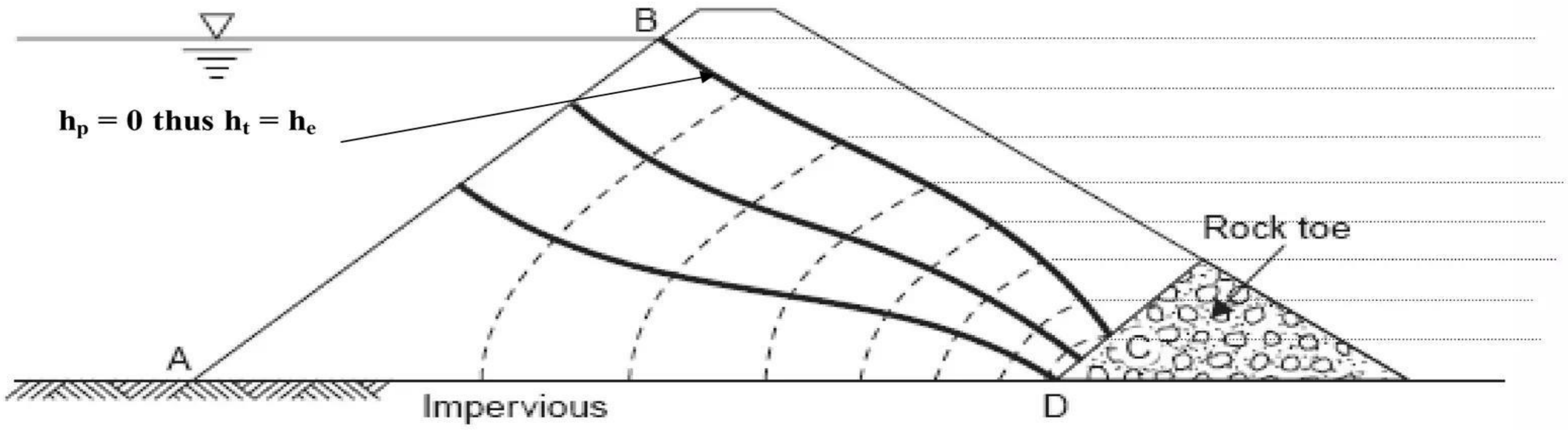


Fig. 6.9 Flow net for an earth dam with rock toe (for steady state seepage)

AB is known to be an equipotential and **AD** a flow line. **BC** is the top flow line; at all points of this line the pressure head is zero. Thus **BC** is also the 'phreatic line'; or, on this line, the total head is equal to the elevation head. Line **CD** is neither an equipotential nor a flow line, but the total head equals the elevation head at all points of **CD**.

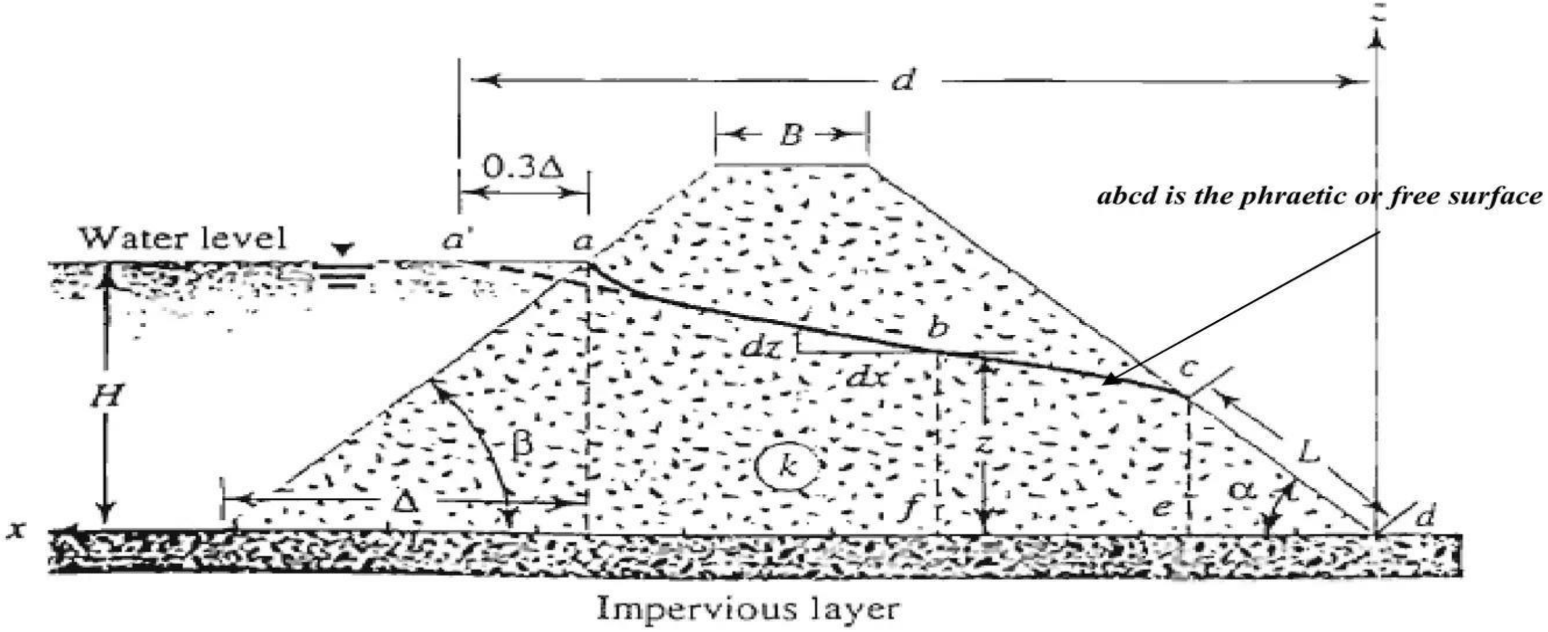


Figure 7.14 Flow through an earth dam constructed over an impervious base

Schaffernak's solution

using Dupuit's assumption $\quad i \cong \dfrac{dz}{dx} = \sin\alpha$

Considering $\triangle$cde

$$q = kiA$$

$$i = \frac{dz}{dx}$$

$$A = (\overline{ce})(1) = L\sin\alpha$$

so

$$q = k(\tan\alpha)(L\sin\alpha) = kL\tan\alpha\sin\alpha \ldots\ldots\ldots\ldots(1)$$

Again,

$$q = kiA = k\left(\frac{dz}{dx}\right)(zx1) = kz\frac{dz}{dx} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

For continuous flow

$$q_1 = q_2$$

$$kz\frac{dz}{dx} = kL\tan\alpha\sin\alpha$$

$$\int_{z=L\sin\alpha}^{z=H} kz\,dz = \int_{z=L\cos\alpha}^{x=d} (kL\tan\alpha\sin\alpha)dx$$

....

....

$$L = \frac{d}{\cos\alpha} - \sqrt{\frac{d^2}{\cos^2\alpha} - \frac{H^2}{\sin^2\alpha}} \text{.....................}(3)$$

Steps to find rate of seepage q (per m length of the dam)

1. obtain α
2. calculate Δ(see the Fig.) and then 0.3Δ
3. calculate d
4. with known values of α and d , calculate L from Eq. 3
5. with known values of L, calculate q from Eq.1

L. Casagrande's Solution

Casagrande show that when α is more than 30^o the deviation from Dupuit's

Assumption is more noticeable, he suggested that

$$i = \frac{dz}{ds} = \sin\alpha \qquad \text{where } ds = \sqrt{dx^2 + dz^2}$$

$$q = kiA = k\sin\alpha)L\sin\alpha) = kL\sin^2\alpha$$

$$again$$

$$q = kiA = k\left(\frac{dz}{ds}\right)(1xz)$$

Combining these questions e get,

$$\int_{z=L\sin\alpha}^{z=H} kz\,dz = \int_{L}^{s} L\sin^2\alpha$$ where s = length of the curve a$^{/}$bc

$$\boldsymbol{L = s - \sqrt{s^2 - \frac{H^2}{\sin^2\alpha}}}$$

With an error about 4-5%, e can write

$$s = \sqrt{d^2 + H^2}$$

Then

$$L = \sqrt{d^2 + H^2} - \sqrt{d^2 - H^2 \cot^2 \alpha}$$

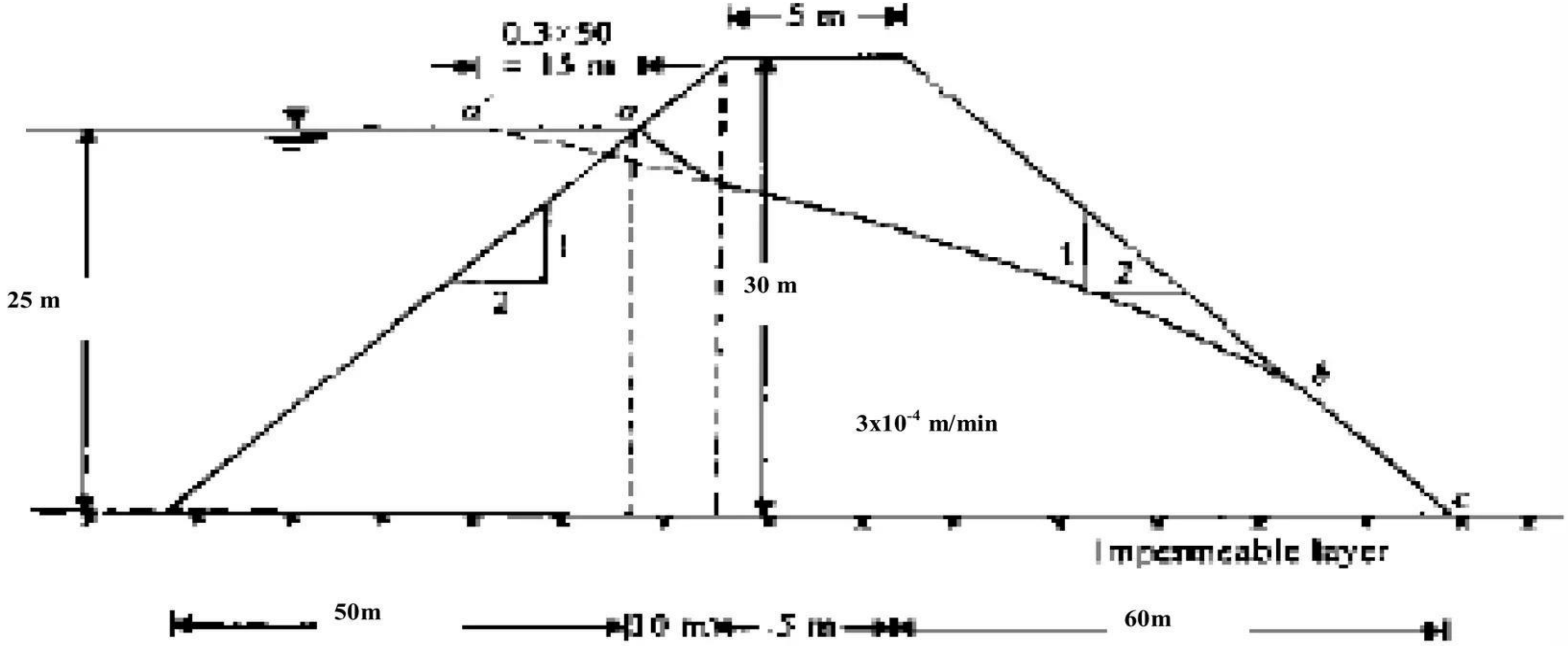
Once L is known, the rate of seepage can be calculated

$$q = kL \sin^2 \alpha$$

Example

The cross-section of an earth dam is shown in Figure. Calculate the rate of seepage through the dam [q in m^3/min ·m] by

1. Schaffernak's solution
2. L. Casagrande's method;


5 m
0.3 x 50
= 15 m
30 m
25 m
1
2
3x10-4 m/min
Impermeable layer
50m
10 m
5 m
60m

Schaffernak's solution

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

$$L = \frac{90}{\cos\alpha} - \sqrt{\frac{90^2}{\cos^2 26.57} - \frac{25^2}{\sin^2 26.57}} = 16.95$$

d = 125 – 0.7x50=

90 m; α = 26.57°

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

$$q = 3x10^{-4}(16.95)(\tan 26.57)(sn 26.57) = 11.37x10^{-4}$$

L. Casagrande's method;

$$L = \sqrt{d^2 + H^2} - \sqrt{d^2 - H^2 \cot^2 \alpha}$$

$$L = \sqrt{90^2 + 25^2} - \sqrt{90^2 - 25^2 \cot^2 26.57} = 19m$$

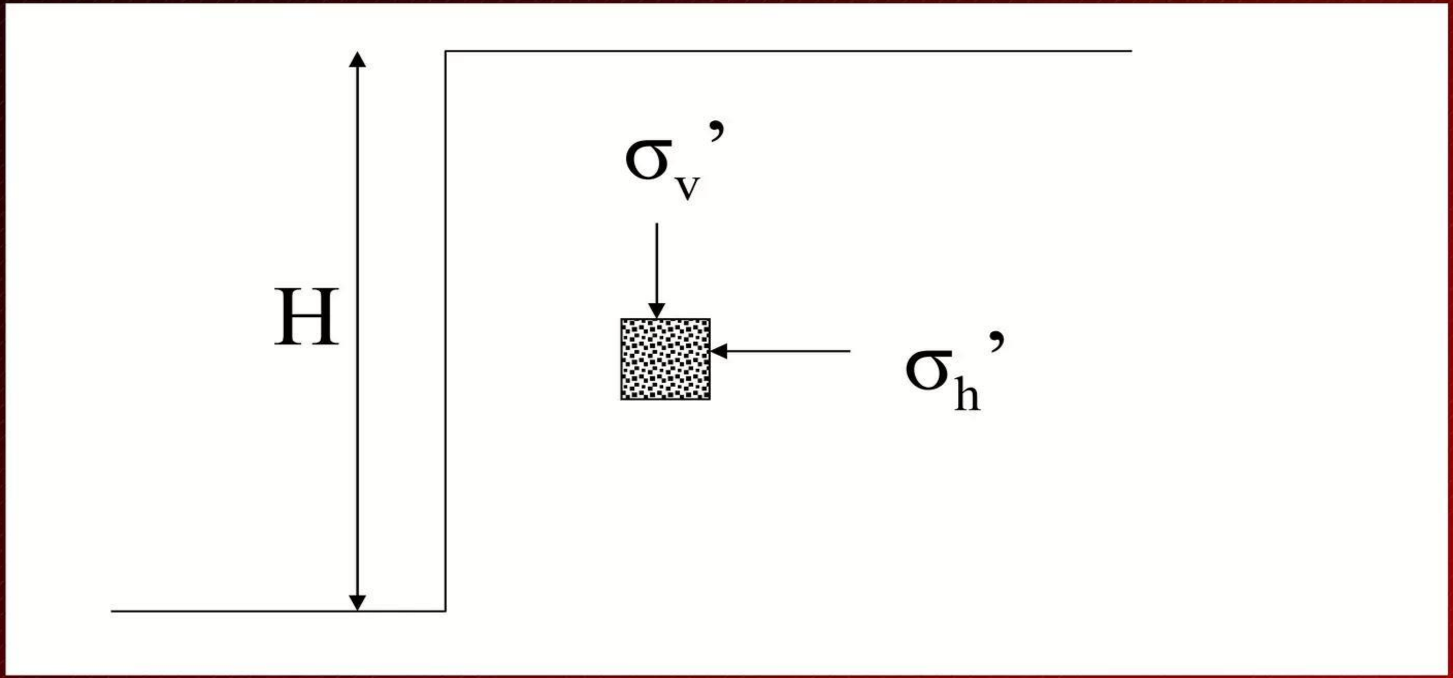
$$q = kL \sin^2 \alpha = 3x10^{-4}(19)(\sin 26.57) = 11.4x10^{-4} \ldots m^3/(m.\min)$$

Earth Pressure

Week 16

Pages 263-276

Lateral Earth Pressure



- We can calculate σ_v'
- Now, calculate σ_h' which is the horizontal stress
- σ_h'/ σ_v' = K
- Therefore, σ_h' = Kσ_v' (σ_v' is what?)

Lateral Earth Pressure

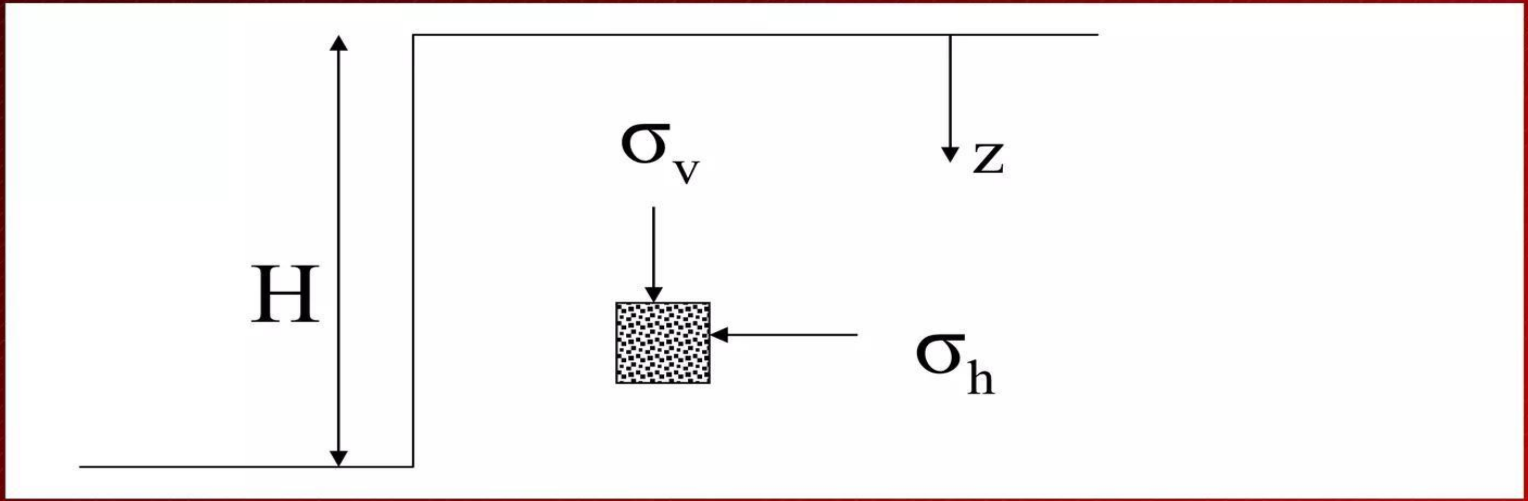
- There are 3 states of lateral earth pressure

  K_o = At Rest

  K_a = Active Earth Pressure (wall moves away from soil)

  K_p = Passive Earth Pressure (wall moves into soil)

  Passive is more like a resistance



At Rest Earth Pressure

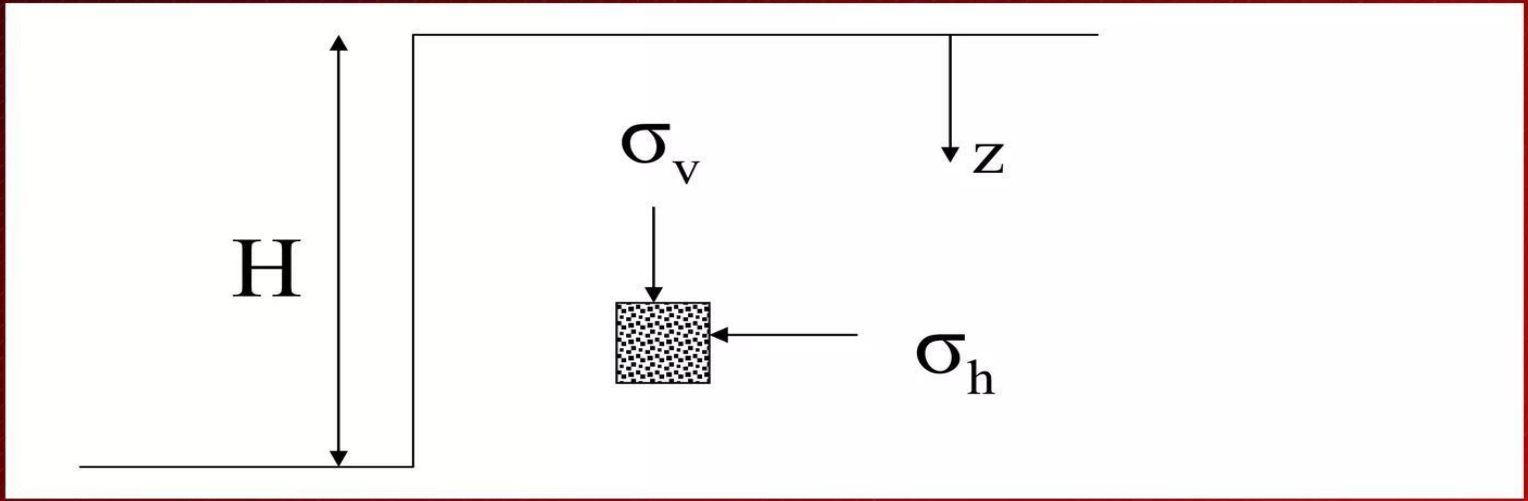
At rest earth pressure occur when there is no wall rotation such as in a braced wall (basement wall for example)

K_o can be calculated as follows:

$K_o = 1 - \sin \varphi$ for coarse grained soils

$K_o = .44 + .42 \, [PI / 100]$ for NC soils

$K_{o\,(oc)} = K_{o\,(NC)} \, (OCR)^{1/2}$ for OC soils



Active Earth Pressure

Active earth pressure occurs when the wall tilts away from the soil (a typical free standing retaining wall)

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Active Earth Pressure

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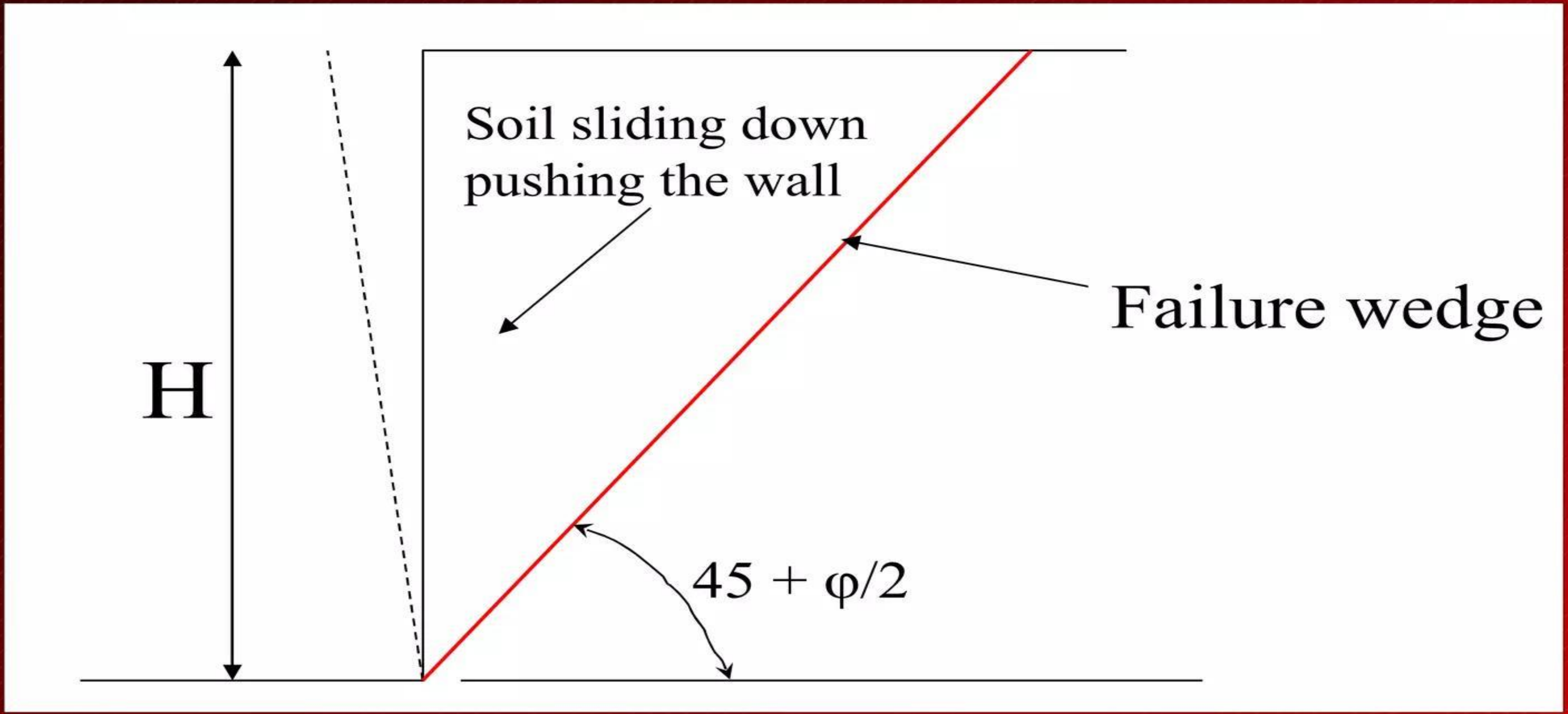
Active Earth Pressure

Active earth pressure occurs when the wall tilts away from the soil (a typical free standing retaining wall)

K_a can be calculated as follows:

$$K_a = \tan^2 (45 - \varphi/2)$$

thus: $\sigma_a{}' = K_a \sigma_v{}' - 2c(K_a)^{1/2}$



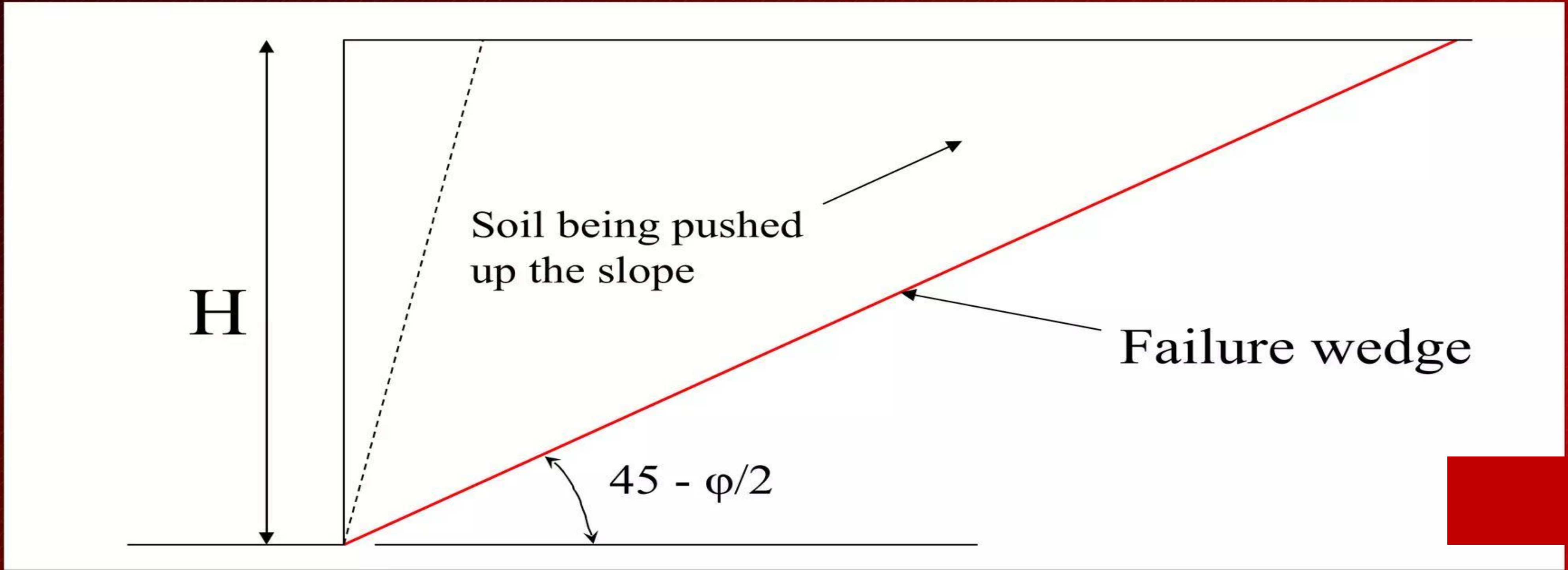
Passive Earth Pressure

Passive earth pressure occurs when the wall is pushed into the soil (typically a seismic load pushing the wall into the soil or a foundation pushing into the soil)

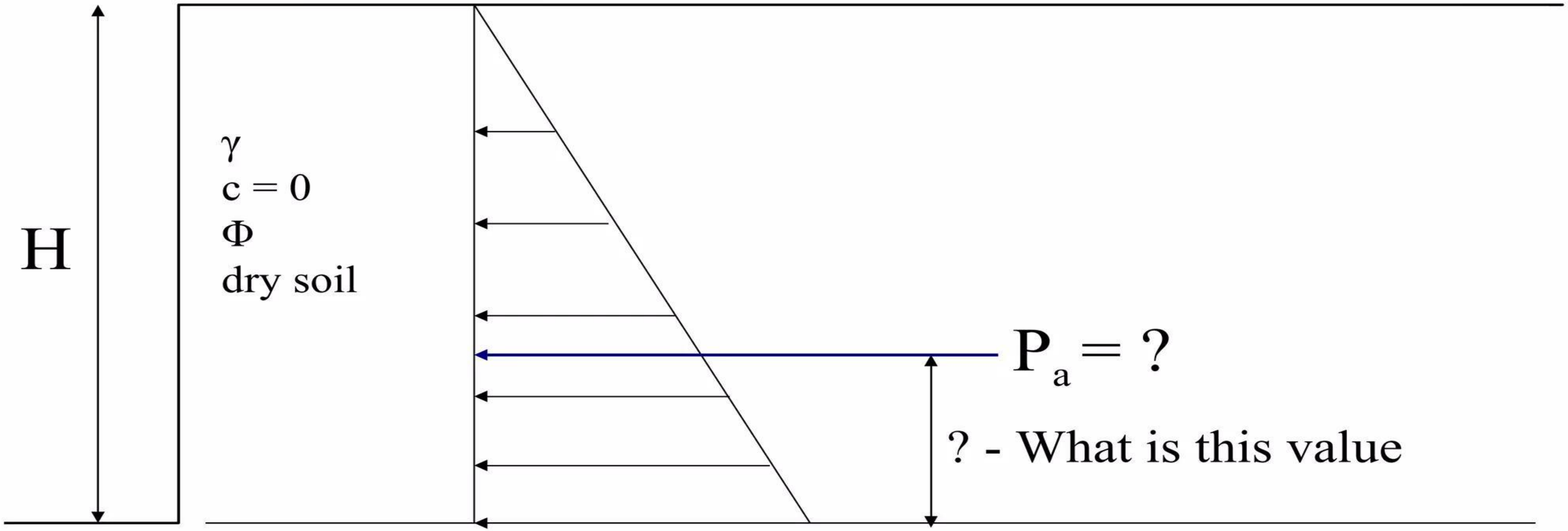
K_p can be calculated as follows:

$$K_p = \tan^2 (45 + \varphi/2)$$

thus: $\sigma_p' = K_p \sigma_v' + 2c (K_p)^{1/2}$



Active Stress Distribution (c = 0)



$\sigma_a' = K_a \sigma_v' - \cancel{2c(K_a)^{1/2}} \to 0$

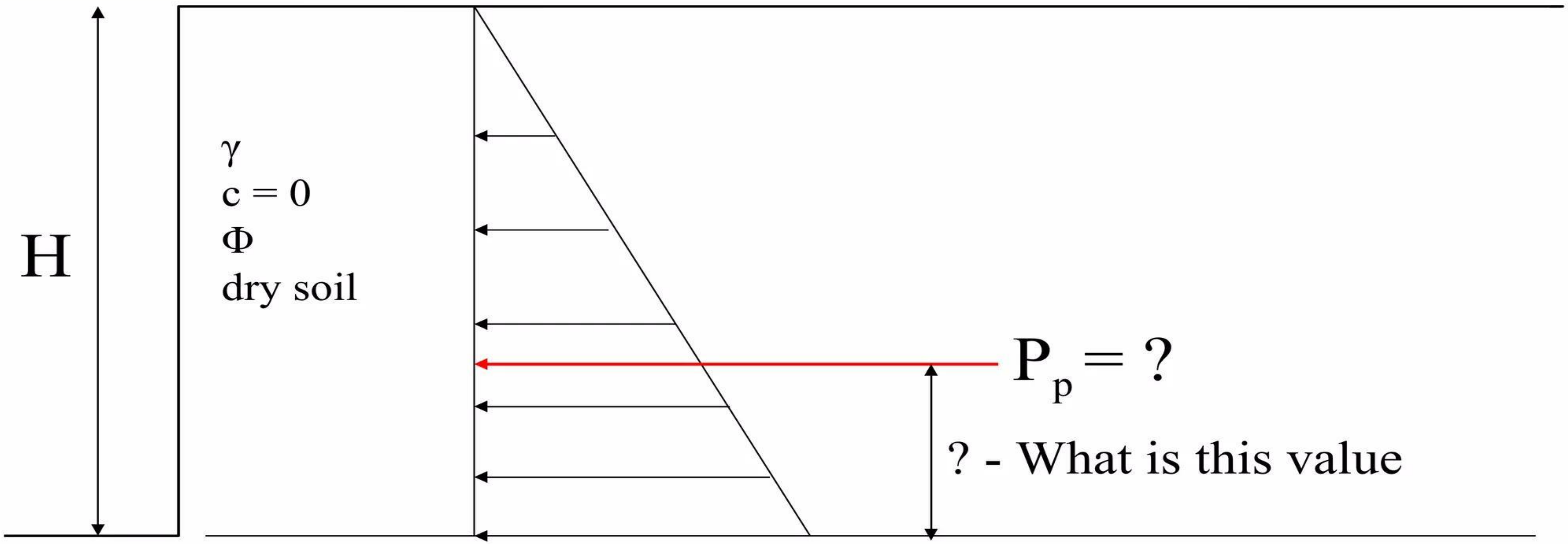
$\sigma_a' = K_a \sigma_v'$

σ_a' is the stress distribution

P_a is the force on the wall (per foot of wall)

How is P_a found?

Passive Stress Distribution (c = 0)



$$\sigma_p' = K_p\, \sigma_v' - \cancel{2\, c\, (K_p)^{1/2}} \rightarrow 0$$

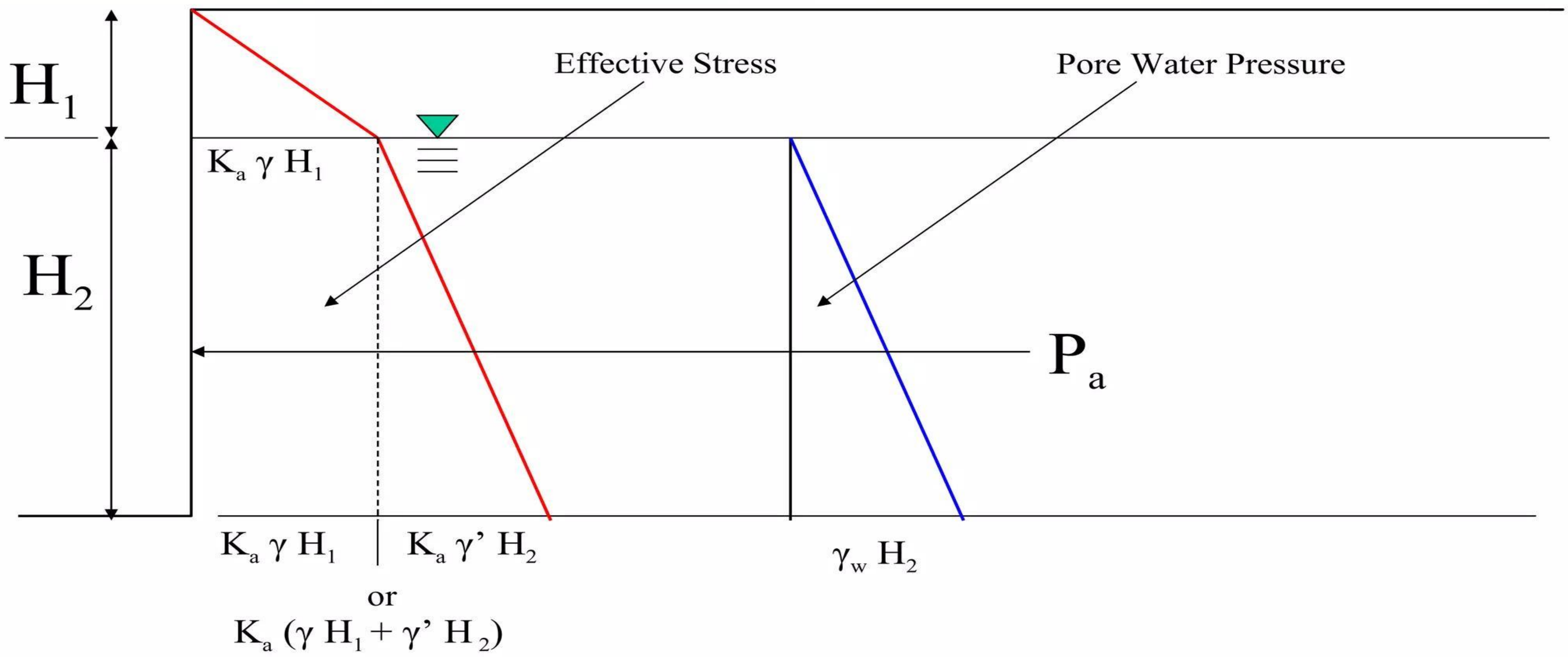
$$\sigma_p' = K_p\, \sigma_v'$$

σ_p' is the stress distribution

P_p is the force on the wall (per foot of wall)

How is P_p found?

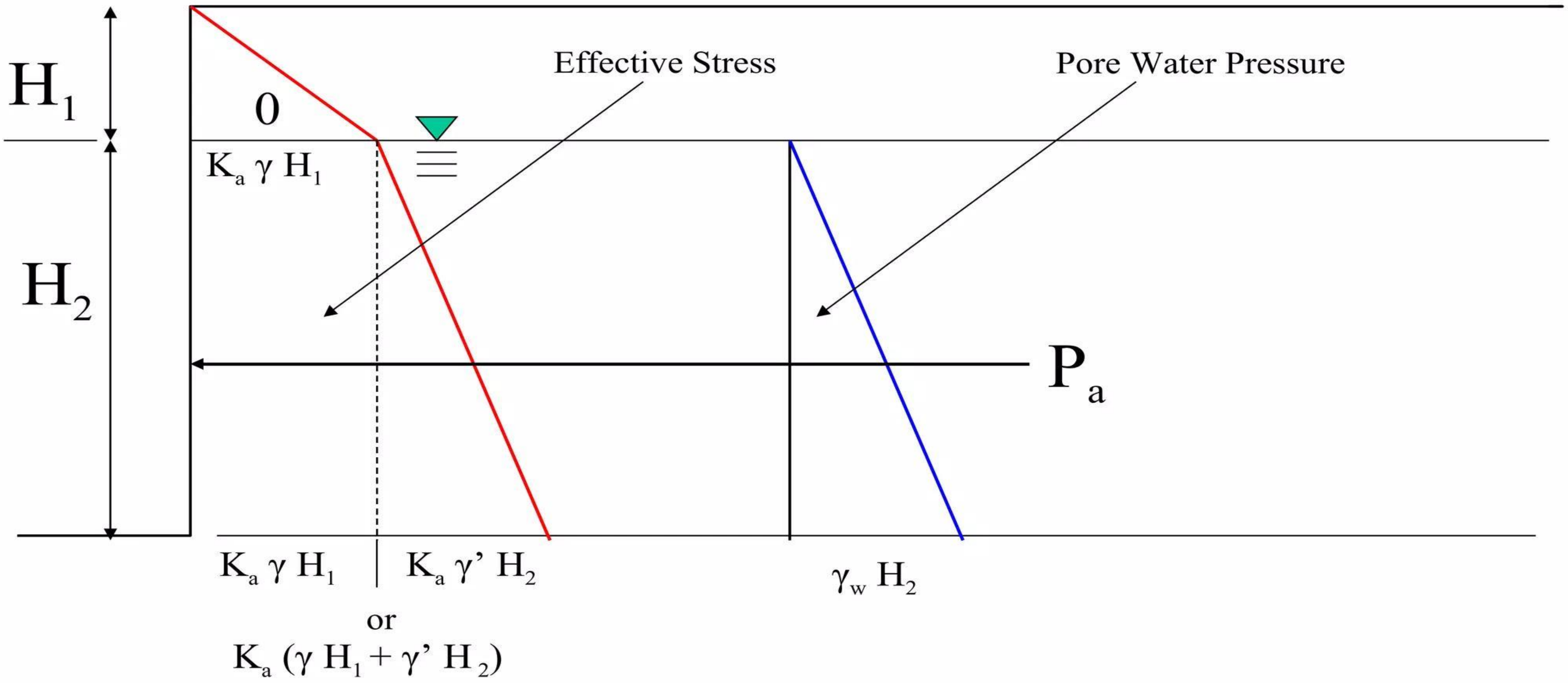
Stress Distribution - Water Table (c = 0)



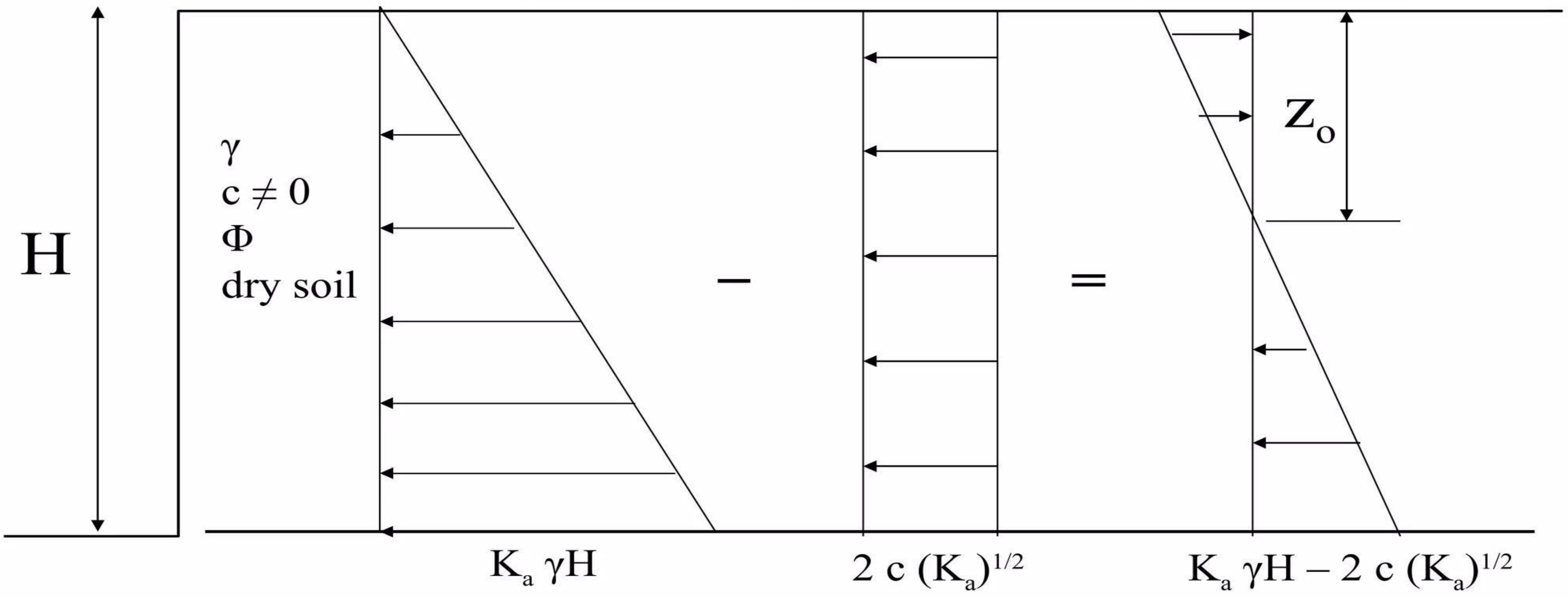
$P_a = \Sigma$ areas $= ½\, K_a\, \gamma H_1^2 + Ka\, \gamma H_1 H_2 + ½\, K_a\, \gamma' H_2^2 + 1/2 \gamma_w H_2^2$

Stress Distribution With Water Table

Why is the water pressure considered separately? (K)



Active Stress Distribution ($c \neq 0$)



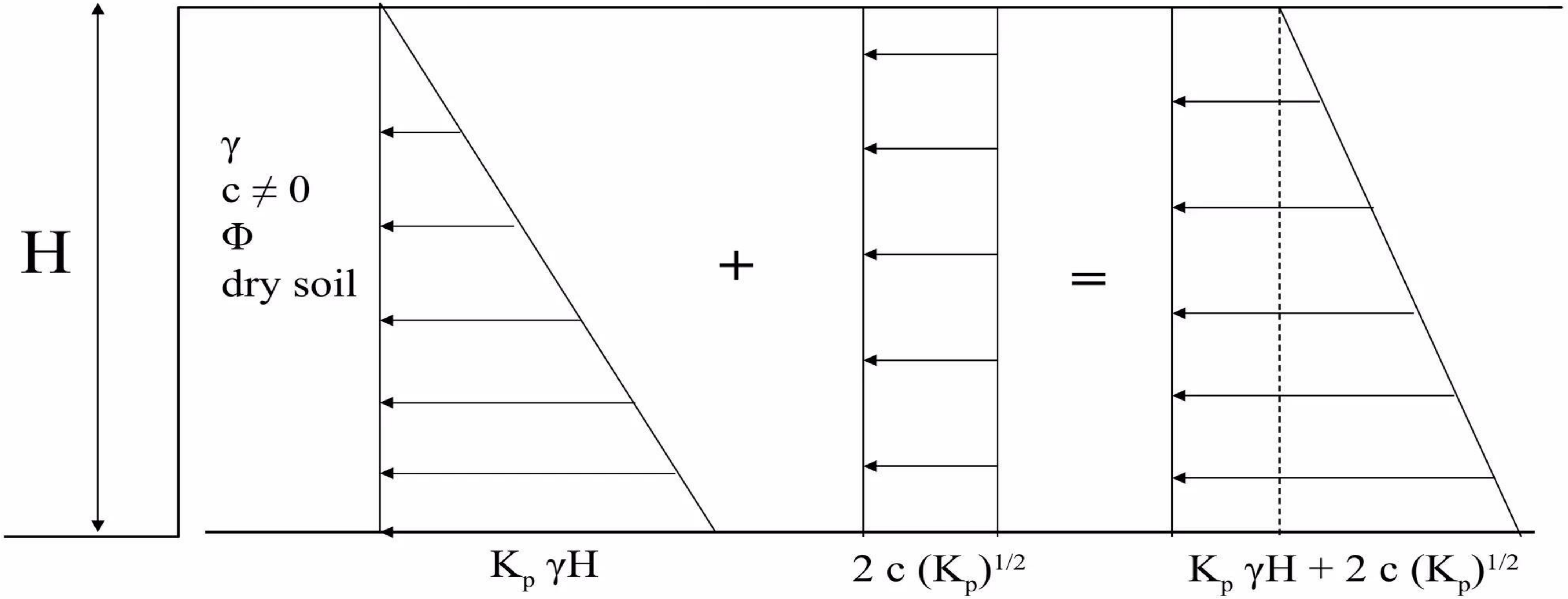
Find z_o:

$K_a \gamma z_o - 2 c (K_a)^{1/2} = 0$

$Zo = 2c / \gamma (K_a)^{1/2}$

Pa = ?

Passive Stress Distribution $(c \neq 0)$



$Pp = ?$

Week 17

Any Questions?

Thank you for your kind attention