Soil Mechanics

Course Code: CLE 205,

Class No :1565, C-Slot, GDN-128

Instructor: Dr. Mahendra Gattu

Lecture Hours: Mon/ Wed 2pm – 340pm

Tue/Thu 4pm – 540 pm

Fri 2pm – 250pm

Office: GDN 102

Soil Mechanics

<u>Textbooks</u>:

1. Basic and Applied Soil Mechanics : Gopal Ranjan and A. S. R. Rao,
2. Geotechnical Engineering: C. Venkatramiah.
3. Dr.K.R.Arora (2001), Soil Mechanics and Foundation Engineering, Standard Publishers, Delhi – 110006

<u>Mode of Evaluation:</u>

Quiz, CAT, Term End.

About Instructor

Education

- PhD Northwestern University, Evanston(Chicago), USA
- MS University of Virginia, Charlottesville, USA
- B.Tech Civil Engineering, IIT-Madras

Industry

- L&T Construction, Metallurgical and Material Handling IC,

  Chennai, Kolkata.
- GMS LLP, Building Envelope and Structural Consulting Firm, New York City, USA.

Course Objectives

- **To facilitate learning about soil and its importance in civil engineering:**
  - What it is soil?
  - How does it behave mechanically?
  - Why does it behave that way?
  - How to measure its properties?
  - How to apply this understanding to solve some basic engineering problems.

Course Objectives

- **Methods for achieving learning objectives:**
  - Reading/studying the materials
  - Class periods and notes (questions during class are encouraged)
  - Solving homework problems
  - Laboratories, both performing the experiments and processing data for the write-ups
  - Studying for exams

Course Syllabus

- **UNIT I : Weight volume relations and Index properties (9 hours)**

  Importance of geotechnical engineering – 3-phase diagram – Weight-volume relations – Index properties of soils – Simple soil engineering tests - Atterberg's limits – Classification of soils – Theory of compaction.

- **UNIT II: Soil water and Permeability (9 hours)**

  Soil water - Effective and neutral stresses – Flow of water through soils – Permeability – Darcy's law –Seepage and flow-nets - Quick sand.

Course Syllabus

- **UNIT III: Stress distribution in soils (9 hours)**

  Vertical pressure distribution: Boussinesq's equation for point load and uniformly distributed loads of different shapes– Newmark's influence chart – Westergaard's equation – Isobar diagram – Pressure bulb - Contact pressure

- **UNIT IV: Compressibility and Consolidation (9 hours)**

  Compressibility – e-log p curve – Preconsolidation pressure - Primary consolidation – Terzaghi's consolidation theory - Laboratory consolidation test – Determination of C_v by Taylor's and Casagrande's methods

Course Syllabus

- **UNIT V: Shear strength of soils (9 hours)**

  Stress analysis by Mohr’s circle - Mohr’s strength theory – Shear strength of soils – Mohr-Coloumb strength envelope – Laboratory shear tests – Direct shear test – Triaxial compression – Unconfined compression test – Vane shear test – Shear strength of saturated cohesive soils – Shear strength of cohesionless soils - conditions for liquefaction

Exam Schedule

CAT-I : 8-9 June

CAT- II : 20 – 21 June

Term End: 3 – 8 July

Lecture 1

- Importance of Geotechnical Engineering
- 3-Phase Diagram

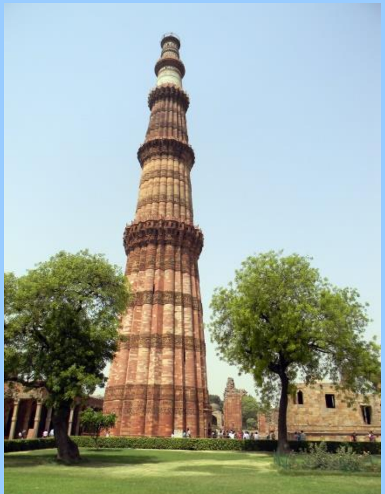
Geotechnical Engineering

- Branch of civil engineering dealing with behavior of earth materials.
- Principles of Soil mechanics + Rock Mechanics
- Wide range of applications in structures used in
  - Military: Underground bunkers
  - Mining: Excavation of coal mines
  - Oil and natural gas exploration: Underground drilling
  - Transportation: Roads, Railway lines including tunnels esp on hilly terrains.

Importance of Geotechnical Engineering

- Soil is the ultimate foundation material which supports the structure.

Leaning Tower of Pisa



Importance of Geotechnical Engineering

- Understanding stability of natural and manmade slopes

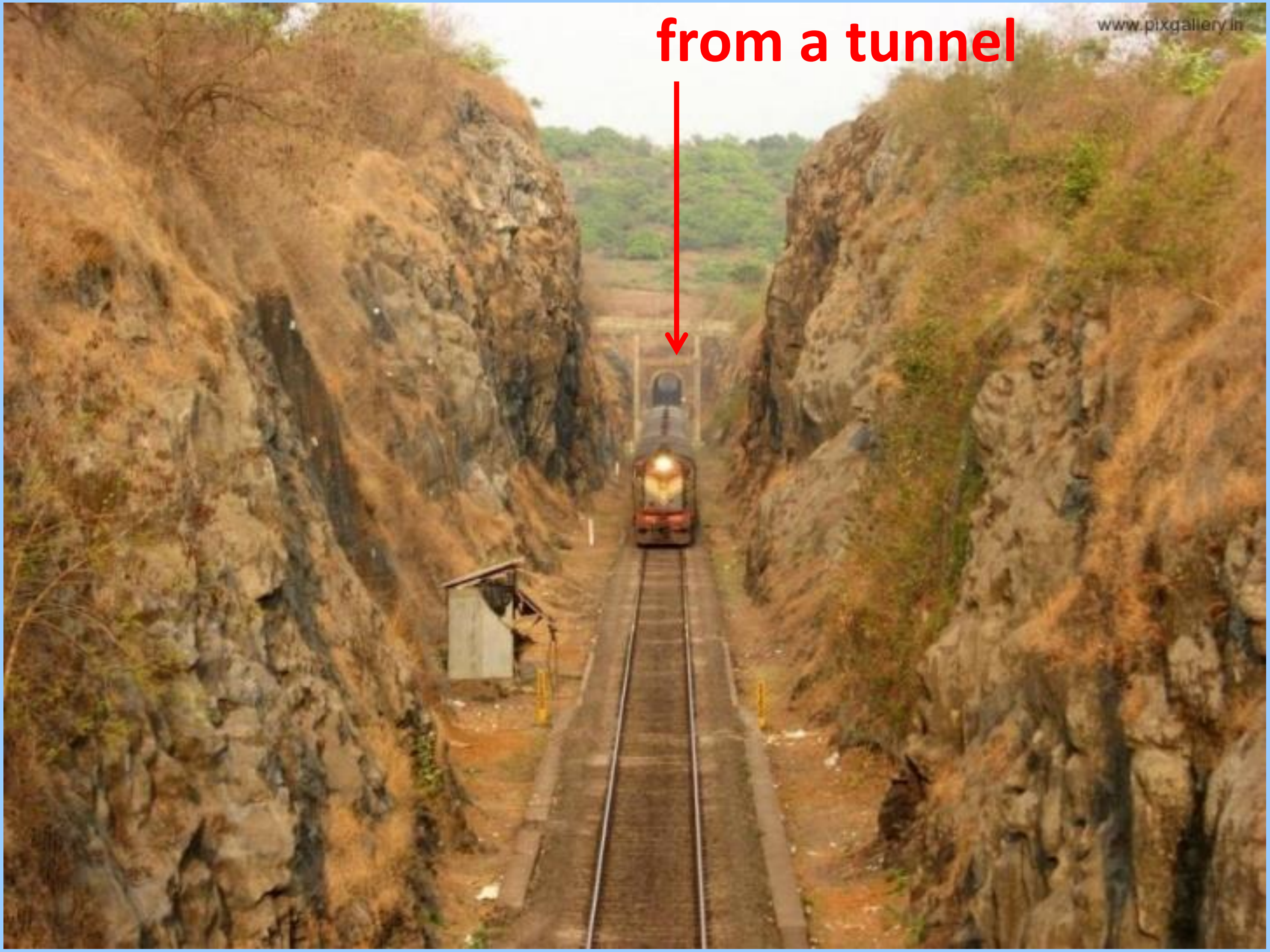
Rock Sliding

Mud Sliding in Sri Lanka

Importance of Geotechnical Engineering

- Tunnels, Conduits

Konkan Railway Jan Shatabdhi Express emerging from a tunnel



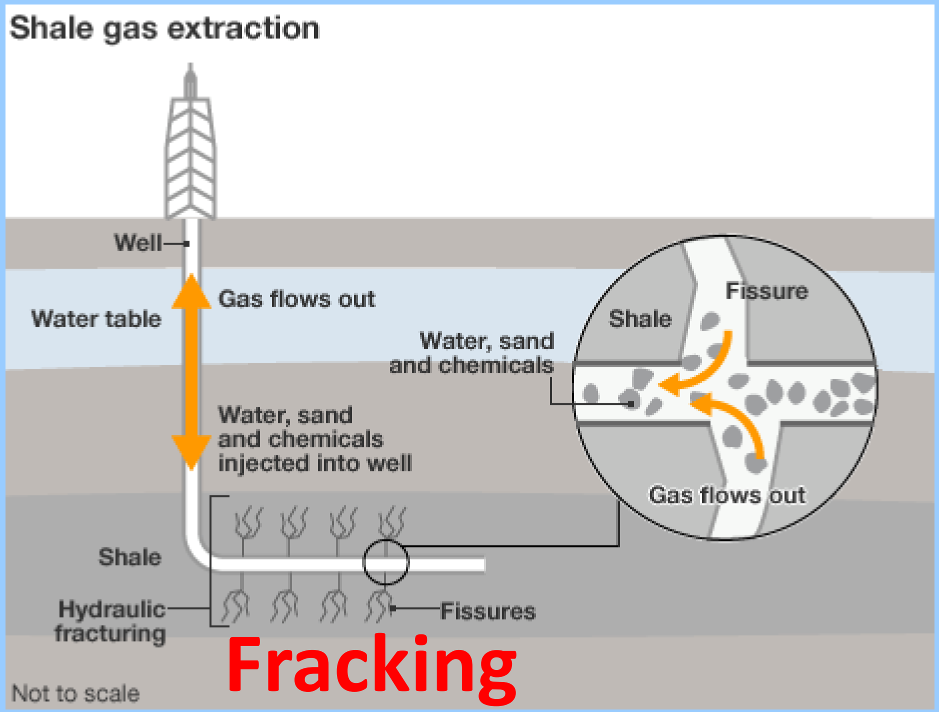
Importance of Geotechnical Engineering

- Machine Foundations
- Dynamic Loading like earthquake, blast loading influence soil behavior

Soil Liquefaction

Importance of Geotechnical Engineering

- Oil and Natural Gas industry
  - Shale gas industry using hydraulic fracturing and principle of rock mechanics.



Crude oil price

Per barrel

June 2014: $107

6 months later

Jan 5 2015: $**50**

- Offshore Platforms →

Importance of Geotechnical Engineering

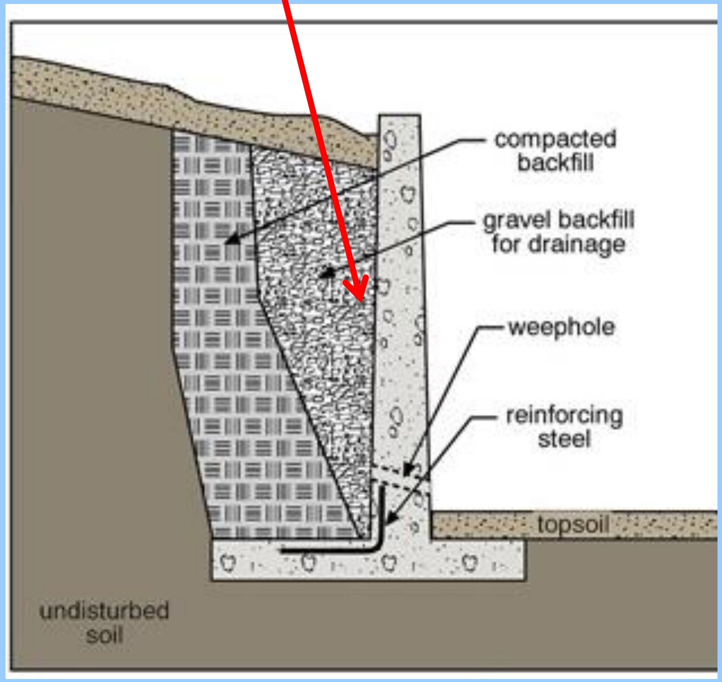
- Soil-Structure Interaction

Process in which response of soil influences the structure behavior and response of structure influence soil behavior.

Hanshin Expressway, Kobe Earthquake, Japan 1995

Importance of Geotechnical Engineering

- Earth retaining structures:, Wall with counterforts. Cantilever wall, Gravity Walls – Determining Lateral loads.



- Construction material

Soil as 3 Phase Material

- Use of Classroom Board

Lecture 3: Index Properties and Classification Test

Dr. Mahendra Gattu

CLE 205 Soil Mechanics

Today's Agenda

- Relationship $Se = WG$
- Relationship between $\gamma_{bulk}, \gamma_{dry}, e, G, w, S$
- Unit Phase Diagram
- Problems

Relationship

- S : Degree of Saturation = V_w/V_v
- e : Void Ratio = V_v/ V_s
- w : Water Content = w_w/w_s
- G : Specific Gravity

$$Se = wG$$

$$Se = \frac{V_w}{V_V}\frac{V_v}{V_s}$$

$$= \frac{V_w}{V_s}$$

$$= \left(\frac{\rho_w G}{\rho_s}\right)\frac{V_w}{V_s}$$

$$= \left(\frac{\rho_w V_w}{\rho_s V_s}\right)G$$

$$= \frac{w_w}{w_s}G$$

$$= wG$$

Relationship

- Bulk Unit Weight, Dry Unit Weight

$$\gamma_{bulk} = \frac{\gamma_w (G + se)}{(1+e)} = \frac{\gamma_w G(1+w)}{(1+e)}$$

$$\gamma_{dry} = \frac{\gamma_w G}{(1+e)}$$

$$\gamma_{bulk} = \frac{W}{V}$$

$$= \frac{W_w + W_s}{V_s + V_V}$$

$$= \frac{W_s(1+\frac{W_w}{W_s})}{V_s(1+\frac{V_s}{V_V})}$$

$$= \frac{G\gamma_w(1+w)}{(1+e)}$$

$$= \frac{\gamma_w(G+se)}{(1+e)}$$

Problem

- $1m^3$ of soil weighs 19.80 kN. The specific gravity of soil particle is 2.70 and water content is 11%. Find void ratio, dry density, degree of saturation

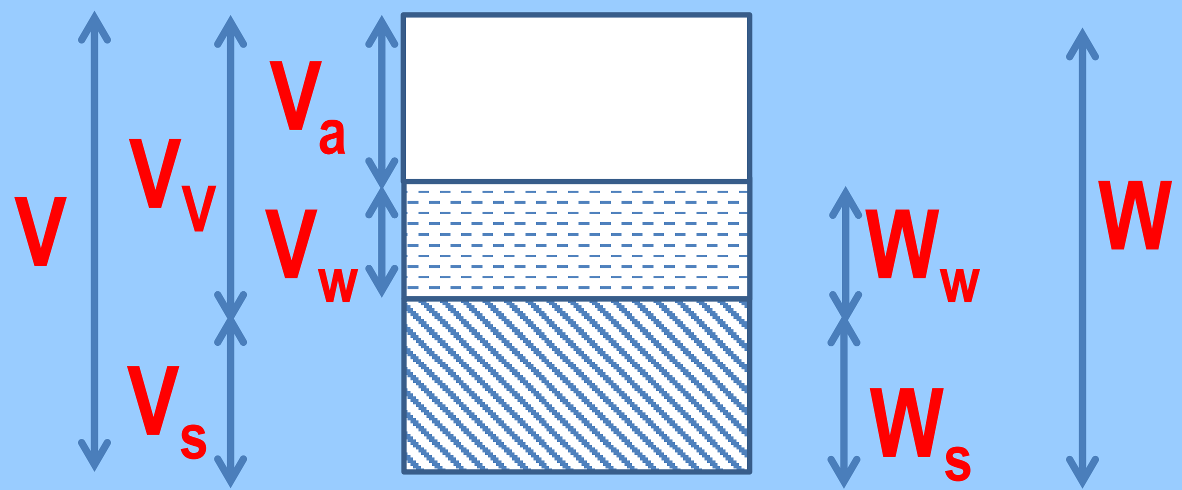
i) Using first principles.

ii) Using formulae

Problem

- The porosity of a soil sample is 35%.The specific gravity of soil particles is 2.7. Calculate its void ratio, dry density, saturated density, submerged density.

Phase Diagram for Soil



$$V_s = 1$$

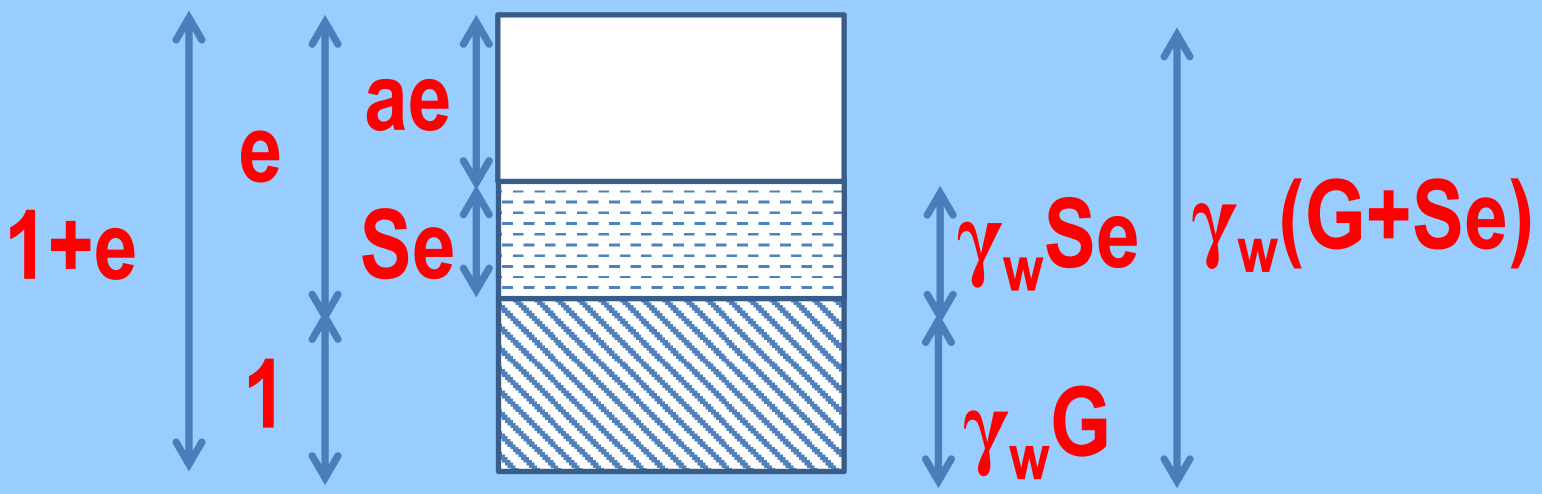
$$V_v = e$$

$$V_w = Se$$

$$W_w = \gamma_w V_w = \gamma_w Se$$

$$W_s = \gamma_s V_s = G\gamma_w$$

Unit Phase Diagram for Soil



Lecture 4: Water Content, Specific Gravity Test, Classification Tests: Index Properties

Dr. Mahendra Gattu

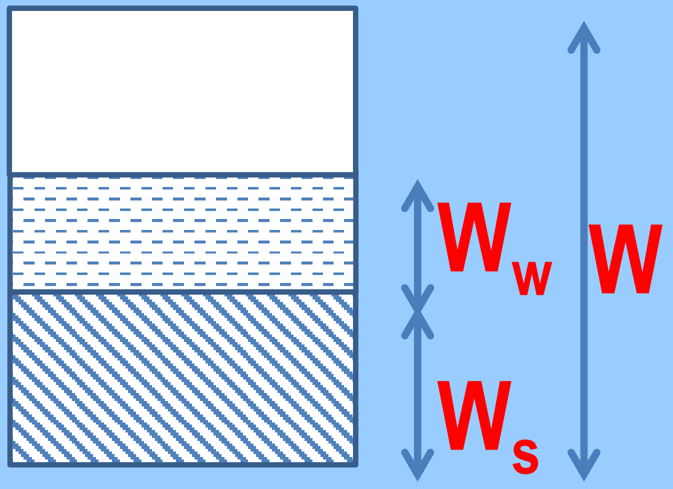
CLE 205 Soil Mechanics

Today’s Agenda

- Determination of Water content
  - Oven Drying Method
  - Pycnometer Method
  - Rapid Moisture Meter Method , Sand Bath Method
- Determination of Specific gravity of Solids
- In-Situ Unit Weight determination
  - Core Cutter Method
  - Sand Replacement Method
  - Water Displacement Method

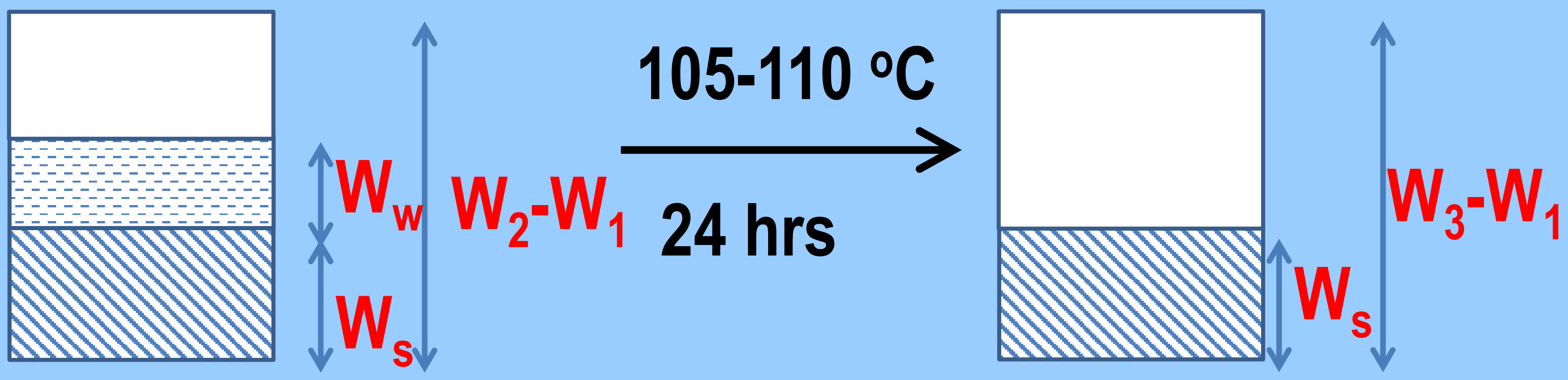
Determination of water content

- Oven Drying Method
- Pycnometer Method
- Rapid Moisture Test Method
- Sand Bath Method



$$w = \frac{W_w}{W_s}$$

Oven Drying Method



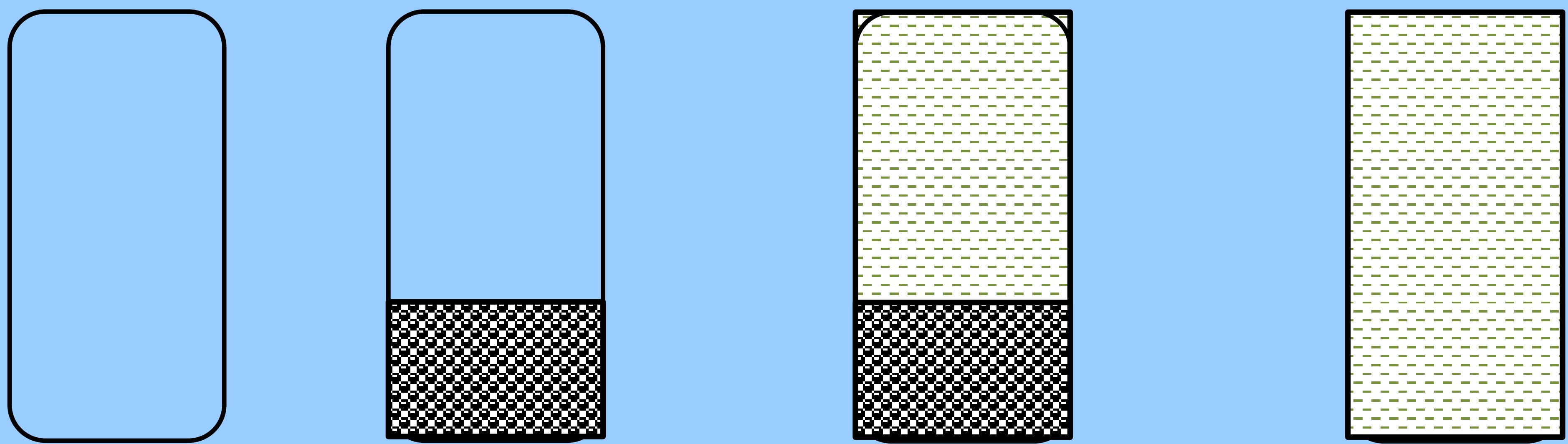
- At temp > 110 deg.C, crystal structure of clay breaks down.
- For organic soils, temp is maintained at 60 deg.C
- W_1: Wt. of Container
- W_2: Container + Moist Soil
- W_3: Container+ Oven Dried Soil

$$W_w = W_2 - W_3$$

$$W_s = W_3 - W_1$$

Water Content: Pycnometer Method

G of soil is known



W_1
Empty Container

W_2
Container + Moist Soil

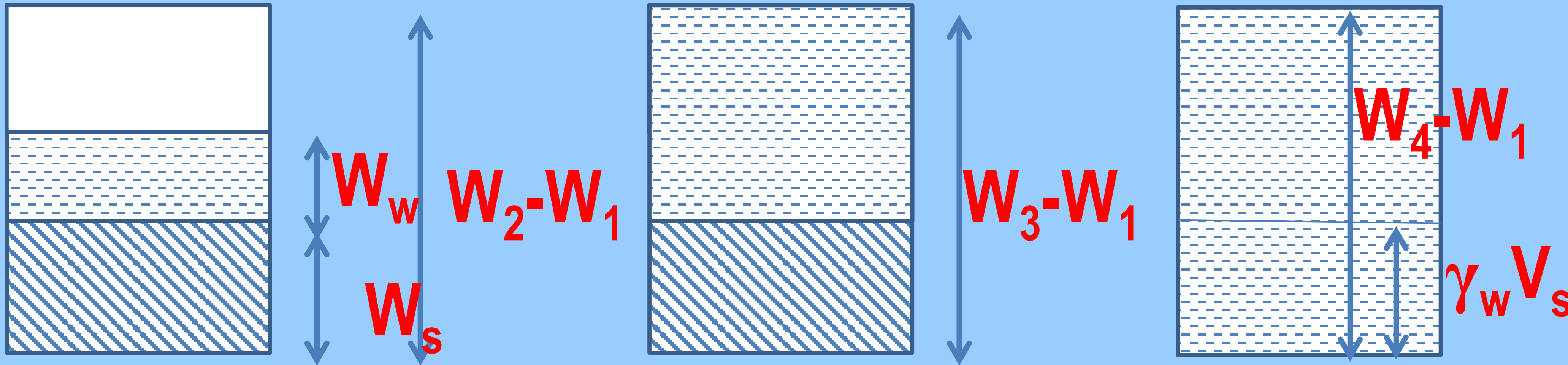
W_3
Container + Moist Soil + Water

W_4
Container +Water

Pycnometer Method: Phase Diagram

Moist Soil **Moist Soil + Water** **Water**

$$W_w = (W_2 - W_1) - W_s$$



$$V_s = \frac{W_s}{\gamma_s} = \frac{W_s}{G\gamma_w}$$

$$(W_3 - W_1) - W_s + \gamma_w V_s = (W_4 - W_1)$$

$$W_s = (W_3 - W_4)\frac{G}{G-1}$$

Pycnometer Method

$$W_w = W_2 - W_1 - W_s$$

$$W_s = (W_3 - W_4)\frac{G}{G-1}$$

$$w = \frac{W_w}{W_s} = \frac{(W_2 - W_1)}{W_s} - 1$$

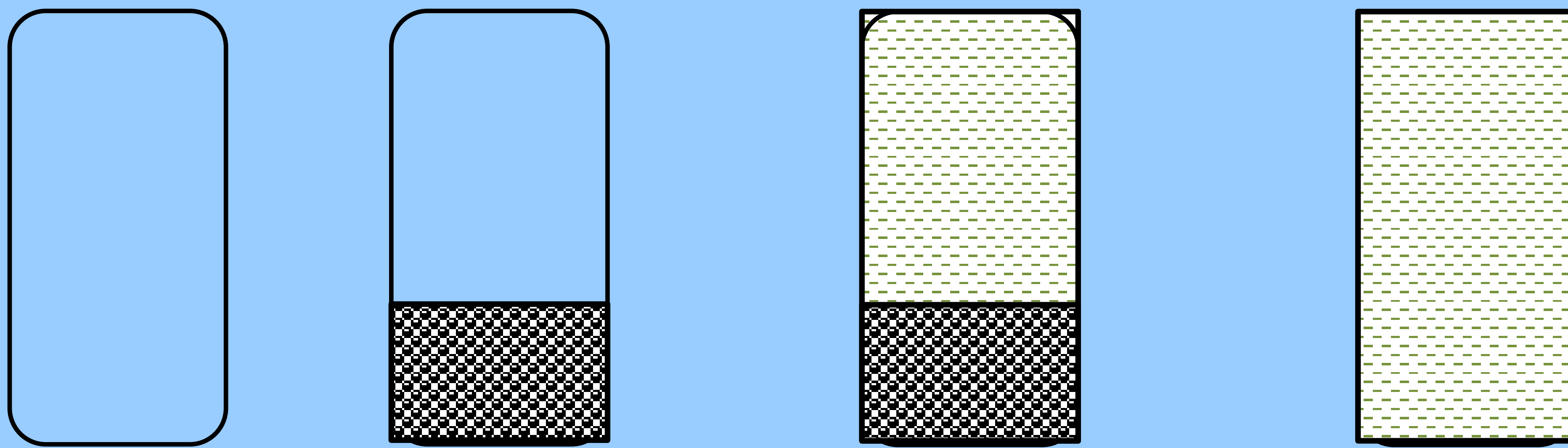
$$w = \frac{(W_2 - W_1)}{(W_3 - W_4)}\frac{G-1}{G} - 1$$

Specific Gravity

- G is the ratio of specific gravity of the solid to the specific gravity of water. It can obtained by measuring the weight of solid to the weight of water occupying equivalent volume of water.

$$G = \frac{\rho_s}{\rho_w} = \frac{\gamma_s}{\gamma_w}$$

Specific Gravity using Pycnometer



W_1
Empty Container

W_2
Container +Dry Soil

W_3
Container + Dry Soil + Water

W_4
Container +Water

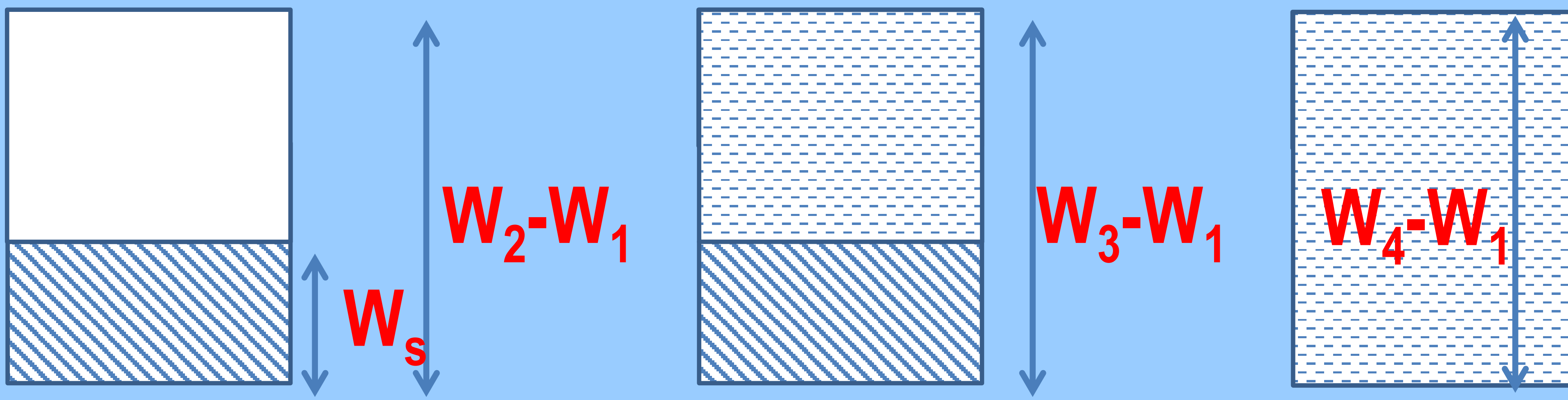
Specific Gravity: Phase Diagram

Dry Soil

$$W_s = W_2 - W_1$$

Dry Soil + Water

Water



$$V_s = \frac{W_s}{\gamma_s} = \frac{W_s}{G\gamma_w}$$

$$(W_3 - W_1) - W_s + \gamma_w V_s = (W_4 - W_1)$$

Specific Gravity Calculations

$$W_s = (W_3 - W_4)\frac{G}{G-1}$$

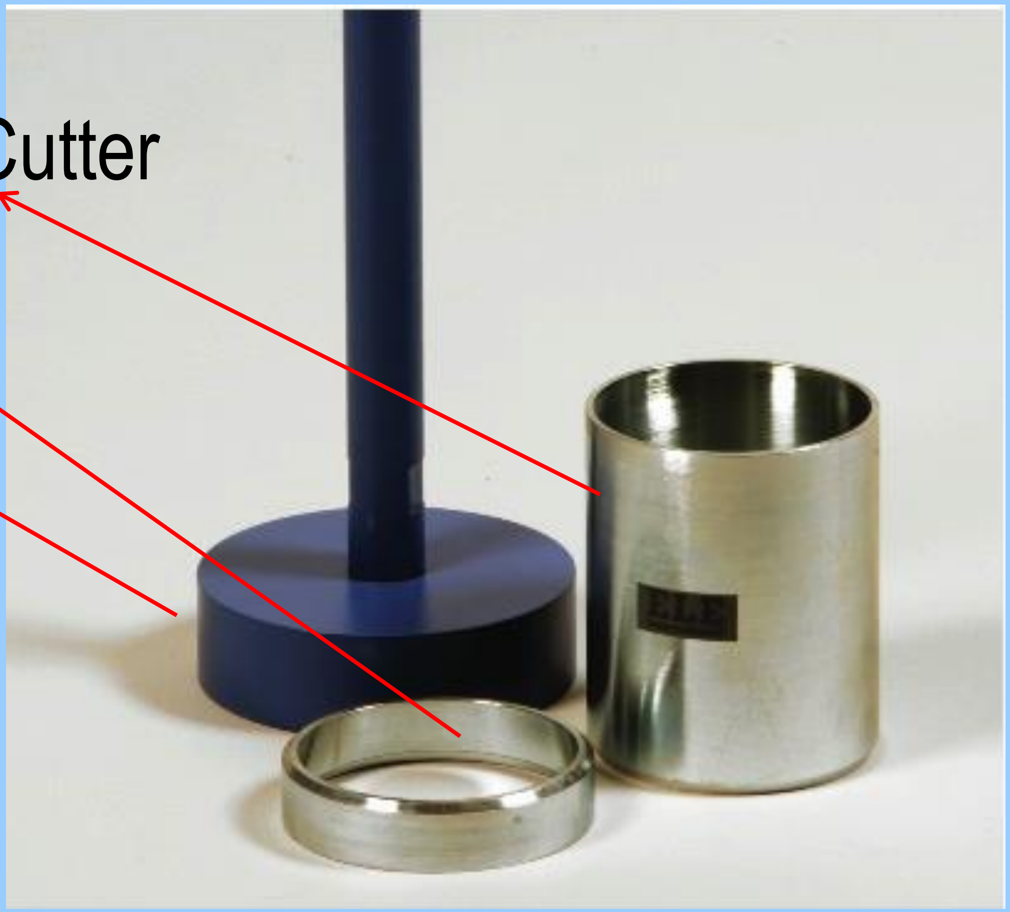
$$W_2 - W_1 = (W_3 - W_4)\frac{G}{G-1}$$

$$\frac{G}{W_2 - W_1} = \frac{G-1}{W_3 - W_4} = \frac{G-(G-1)}{(W_2 - W_1)-(W_3 - W_4)}$$

$$G = \frac{W_2 - W_1}{(W_2 - W_3)-(W_1 - W_4)}$$

In-Situ Unit Weight: Core Cutter Method

- Core Cutter Method
- Soft Soils, Non-Cohesive Soils
- Video
- Rammer, Dolly, Core Cutter



In-Situ Unit Weight: Sand Replacement Method

- Sand Replacement Method
- Hard, Gravely Soils
- Video

In-Situ Unit Weight Determination

- Water Displacement Method
  - Cohesive Soil is weighed : W_1
  - It is coated with paraffin and weighed again: W_2
  - The system of soil coated with paraffin is now immersed in water. The volume displaced is measured, V_{displ}

$$V_{Paraffin} = \frac{W_2 - W_1}{\gamma_{paraffin}}$$

$$\gamma = \frac{W_1}{V}$$

$$V = V_{displ} - V_{Paraffin}$$

Index Properties of Soil

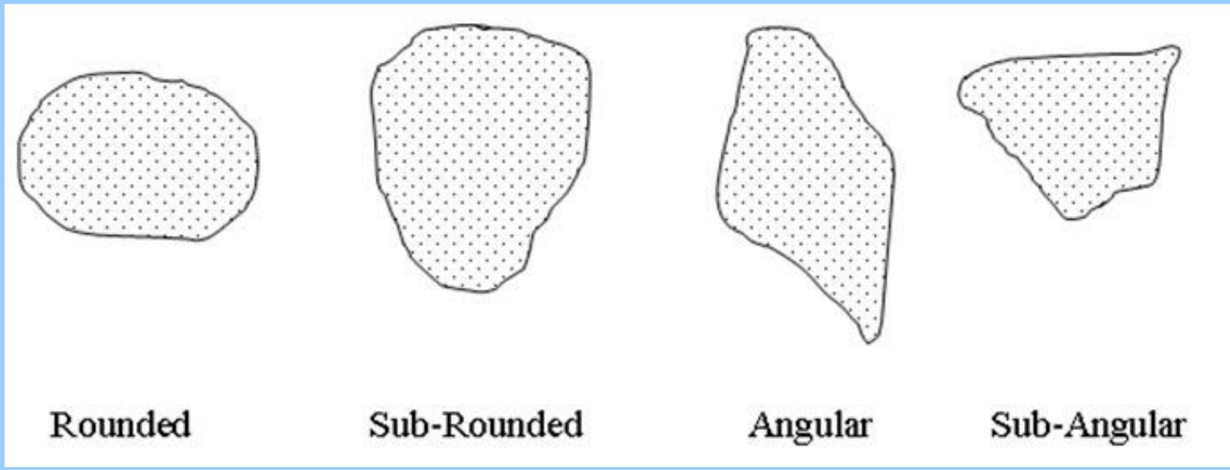
- Tests carried out in order to classify a soil are classification tests.
- The results obtained on the basis of these tests are alled index properties of soil.
- Index properties of soil are divided into
  - Soil Grain Properties
  - Soil Aggregate Properties

Grain Shape

- Bulky Grains:
  - All dimensions are more or less the same.
  - Sandy and gravel soils.
  - Formed by mechanical break-down of parent rocks.
  - Angular → Sub-Angular → Sub-Rounded →Rounded transformation occurs during transportation
  - Sphericity, De: Equivalent diameter of the particle, L is length of the particle,

$$S = \frac{D_e}{L}$$

$$D_e = \left(\frac{6V}{\pi}\right)^{1/3}$$



Grain Shape

- Flaky Grains:
  - One Dimension is very small compared to the remaining two dimensions. Ex: A large sheet of paper.
  - During sedimentation, these grains fall like leaves floating in air.
- Needle-Shape Grains
  - One dimension of grain is fully developed and larger than other two.

Engineering Characterization of Soils

Particle Size

coarse-grained

- Particle/Grain Size Distribution
- Particle Shape

fine-grained

- Soil Plasticity

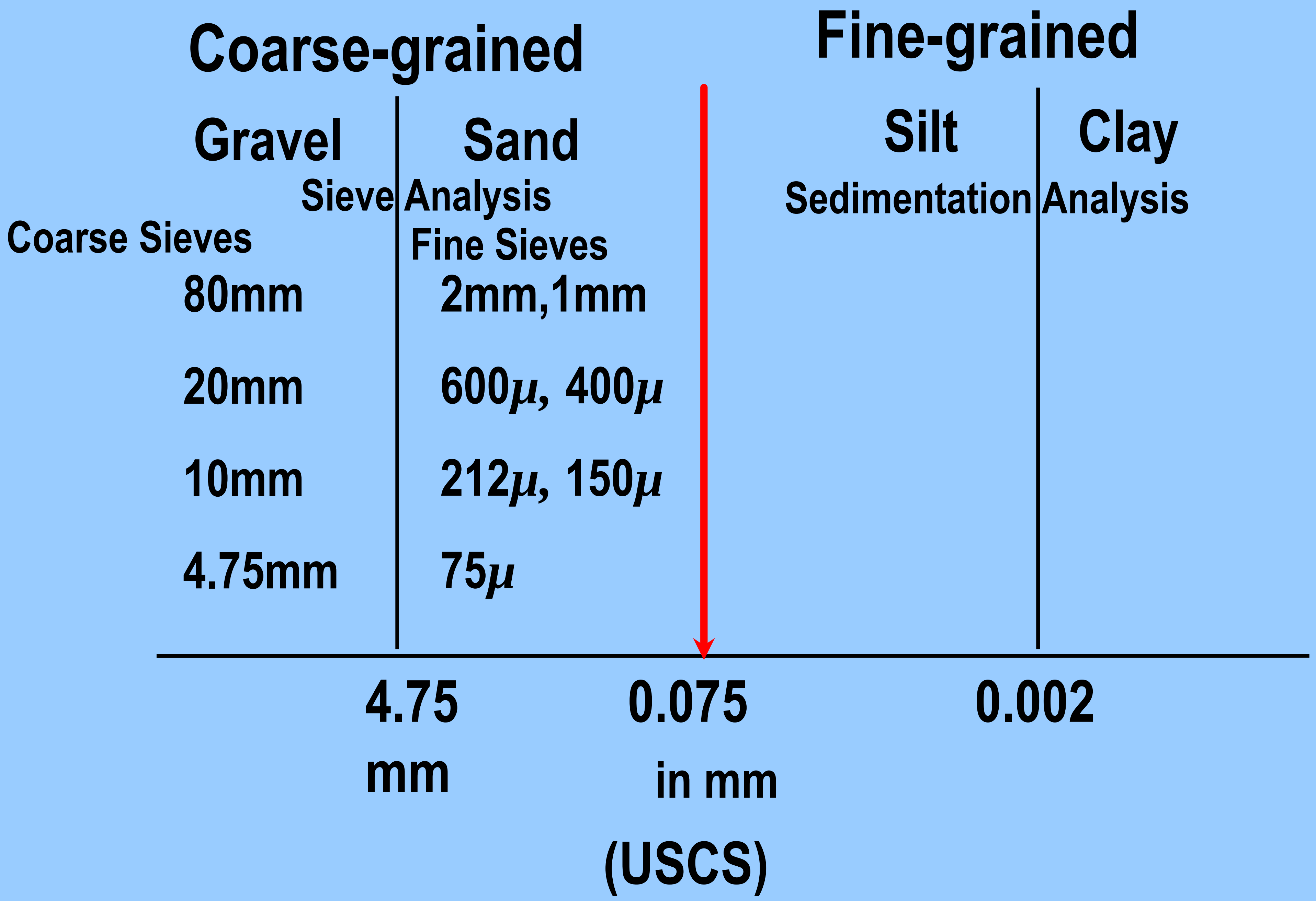
Grain Size Distribution

Why?

- Range of sizes of particles
- % of particles of each of the size ranges.

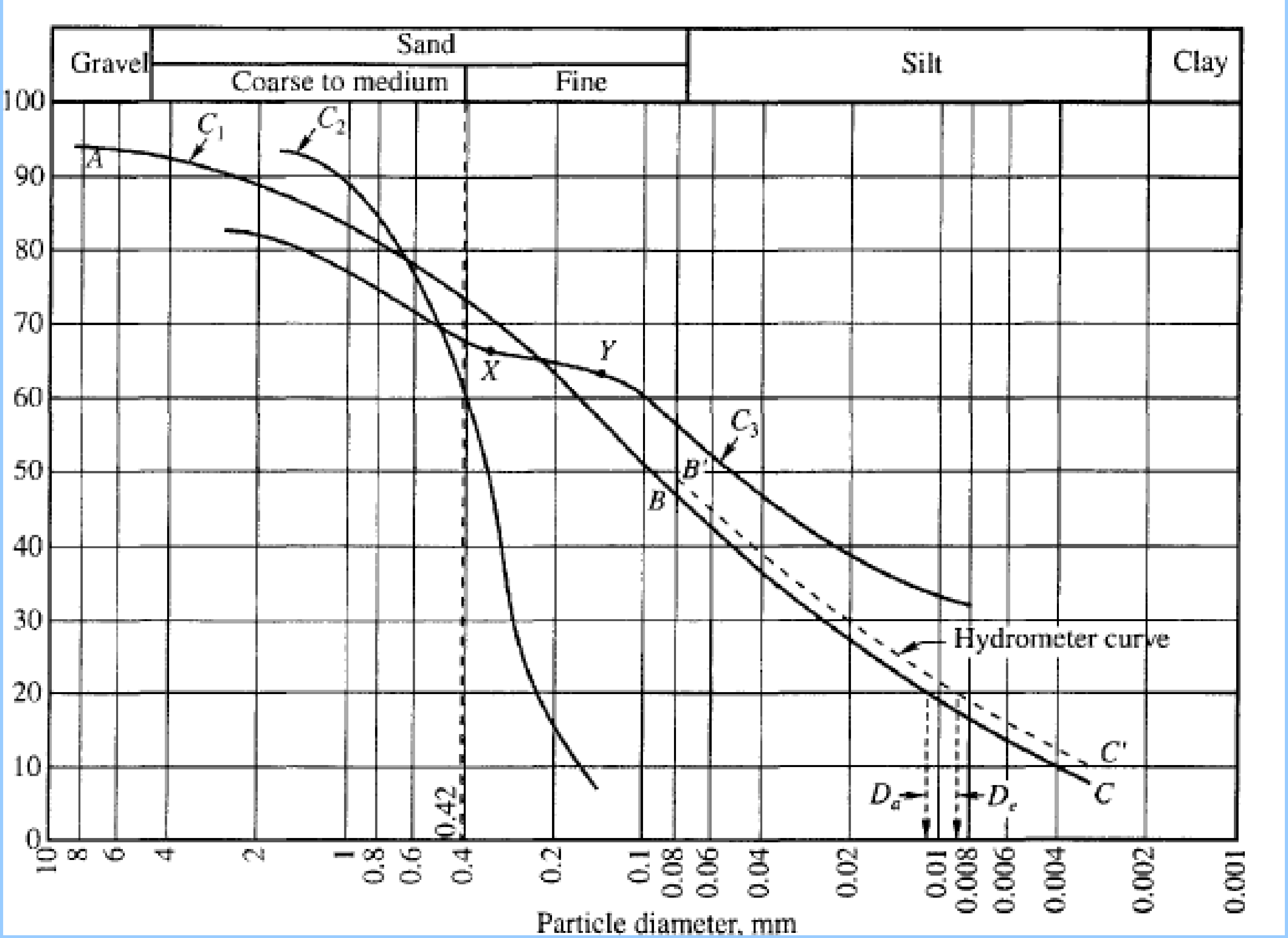
How?

- Sieve analysis for coarse fraction
- Sedimentation analysis for fine fraction.


Coarse-grained
Fine-grained
Gravel
Sand
Silt
Clay
Sieve Analysis
Sedimentation Analysis
Coarse Sieves
Fine Sieves
80mm
2mm,1mm
20mm
600μ, 400μ
10mm
212μ, 150μ
4.75mm
75μ
4.75
mm
0.075
in mm
0.002
(USCS)

Grain Size Distribution Curve

% of particles (by wt) finer than Diameter D



Particle Diameter D

Grain Size Distribution Curve

- Well Graded
- Poorly Graded,
  - Gap Graded,
  - Uniformly Graded
- Effective Size D_{10}
- Coefficient of Uniformity,

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

- Coefficient of Curvature

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

Sedimentation Analysis: Stoke's Law

$$v = \frac{\gamma_s - \gamma_w}{18\mu} D^2$$

- *v = Terminal Velocity*
- *D= Diameter of Sphere*
- *μ= Viscosity of Water*
- *Brownian Motion <0.0002 mm < D < 0.2 mm <Turbulence*
- *Assumptions: Spherical particle, Rate of fall not influenced by adjacent particles, Average value of sp. Gravity G of grains is used to calculate γ_s, Floc formation may occur resulting in larger diameter of the particles.*

Sedimentation Analysis Hydrometer Method

- What is Hydrometer?
- Archimedes Principle: Hydrometer floats in the suspension. The level of immersion depends on the density of fluid.
- $G_s = 1 + R_h/1000$
- R_h= Reading of the Hydrometer
- G_s= Corresponding Specific Gravity

Calibration of Hydrometer: Effective depth L and Hydrometer Reading R or L_1,L_2

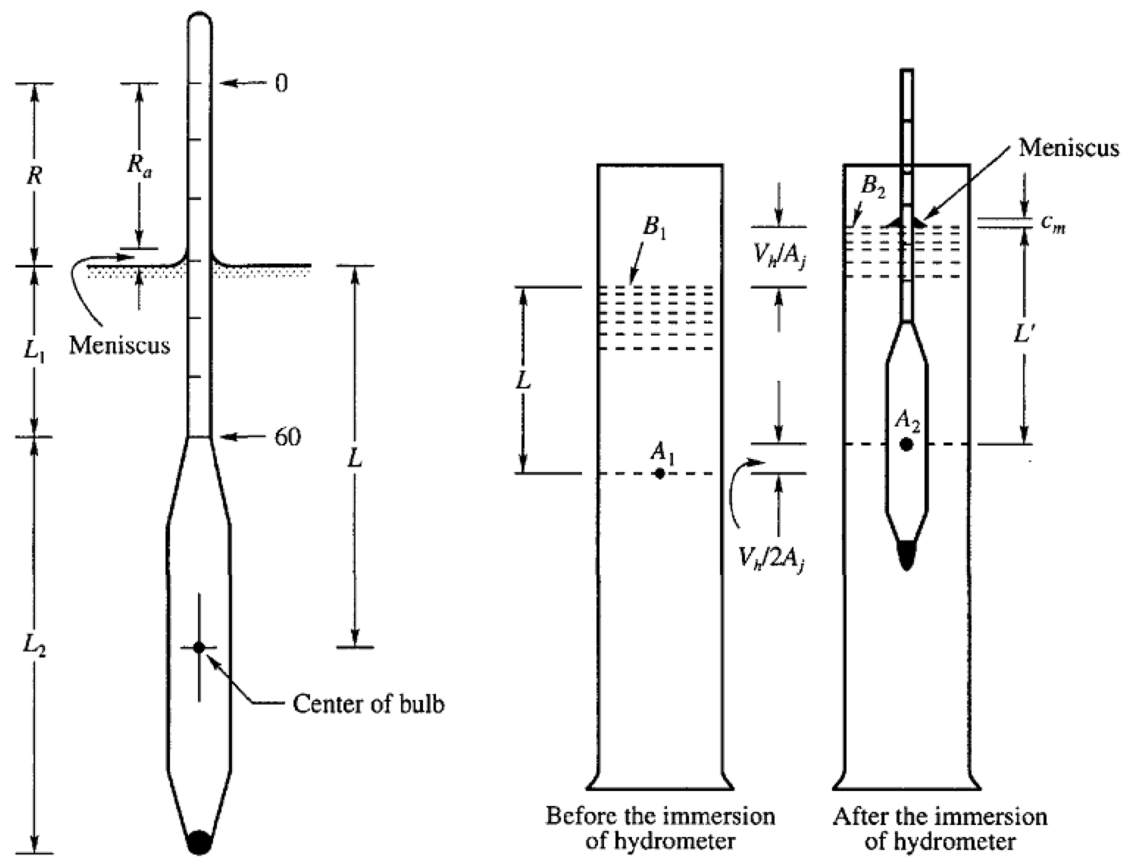


Figure 3.5 ASTM 152 H type hydrometer

Figure 3.6 Immersion correction

$$L = L_1 + \frac{L_2}{2} + \frac{V_h}{2A_j} - \frac{V_h}{A_j}$$

Corrections to Hydrometer Reading

- Meniscus Correction, Cm >0
- Temperature Correction
  - High Temp, Ct>0
  - Low Temp, Ct<0
- Deflocculating agent correction, Cd>0

Procedure of Hydrometer Analysis

t	Rh	Rc	L	D	N
15 sec	-	*	*		
30 sec	-	*	*		
1min	-	*	*		
2min	-	*	*		
5min	-	*	*		
10 min	-	*	*		

w_s is weight of soil grains in a soil of suspension of 1000cc.

$$D = \sqrt{\frac{18\mu(\frac{L}{t})}{\gamma_s - \gamma_w}}$$

$$N = \frac{G_s}{G_s - 1} \frac{R_c}{w_s} \times 100\%$$

Stages of Consistency

- Consistency is a term which is used to describe the degree of firmness of a soil using description such as soft , firm , stiff or hard. It indicates the *relative ease with which soil can be deformed*.

  The property of consistency is associated only with fine grained soils, since fine grained soils are considerably influenced by the amount of water content present in them. Depending on water content, the following four stages are used to describe the consistency of clay soil :

  - Liquid State
  - Plastic State
  - Semi-Solid State
  - Solid State

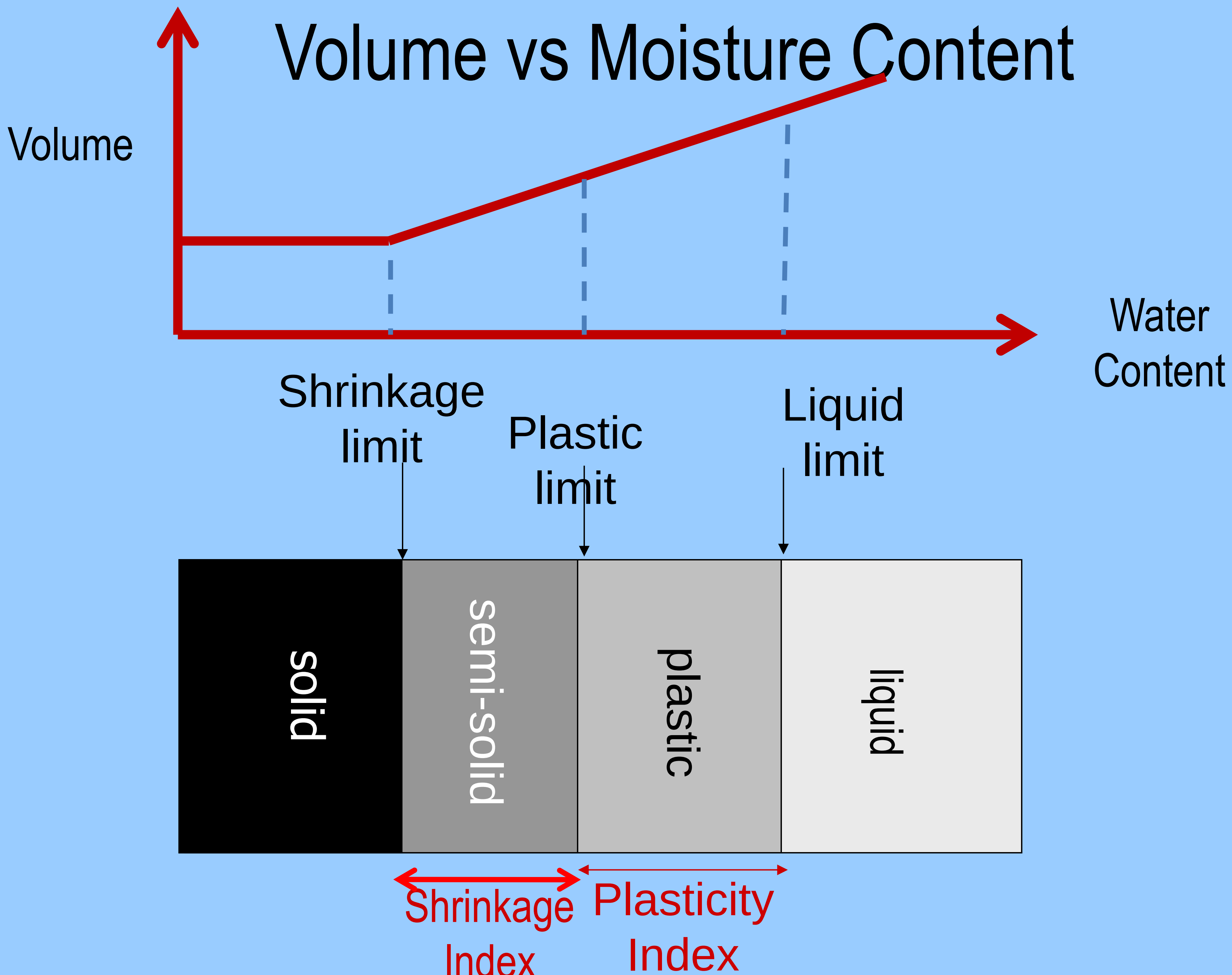
- The boundary water contents at which soil undergoes changes from one state to another are called " **consistency limits**."

Consistency Limits

Atterberg, a Swiss soil scientist first demonstrated the significance of the consistency limits in 1911.

- Liquid Limit
- Plastic Limit
- Shrinkage Limit

Volume vs Moisture Content



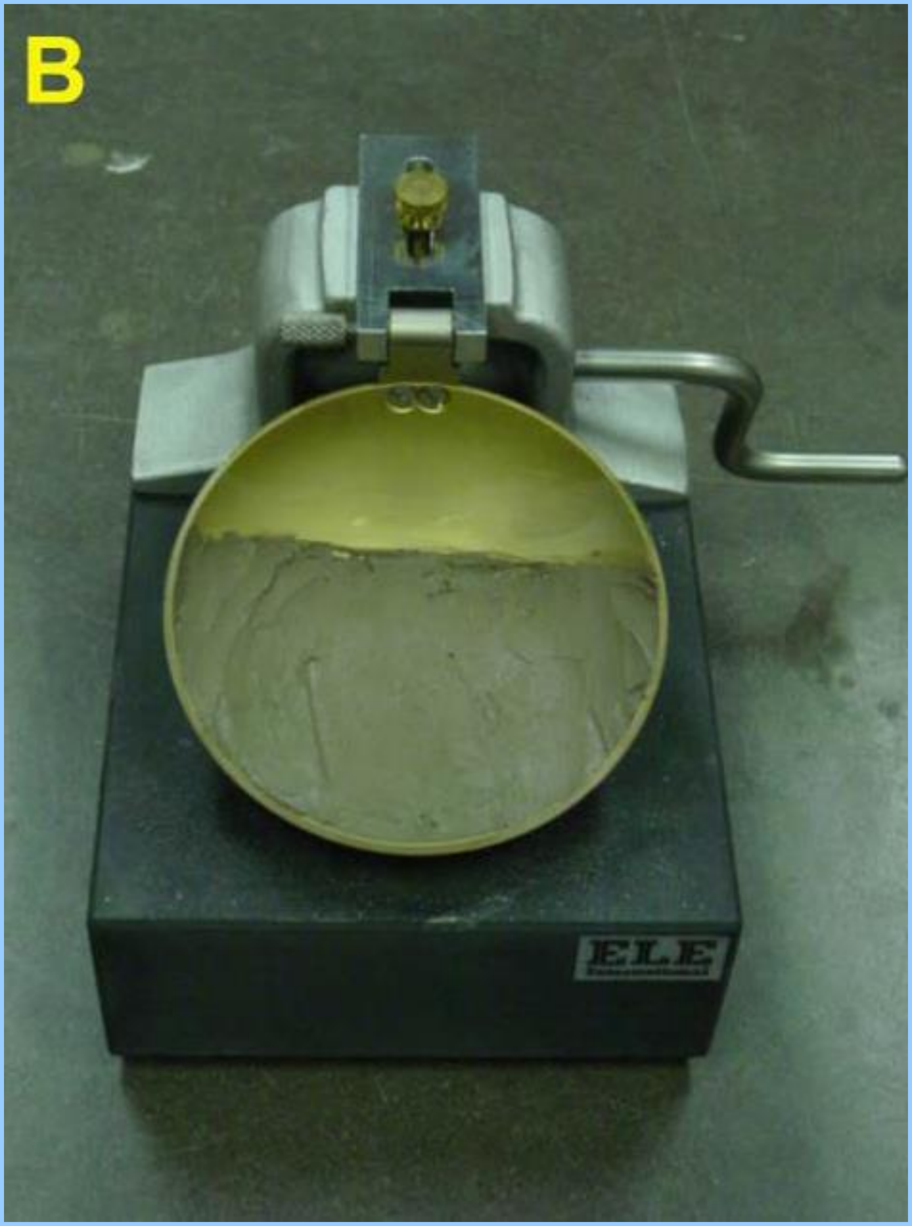
Liquid Limit Test

- Cone Penetration Method
  - Water Content at w_α for which cone penetration α between 20mm and 30 mm is obtained

$$w_L = w_\alpha + 0.01(25 - \alpha)(w_\alpha + 15)$$

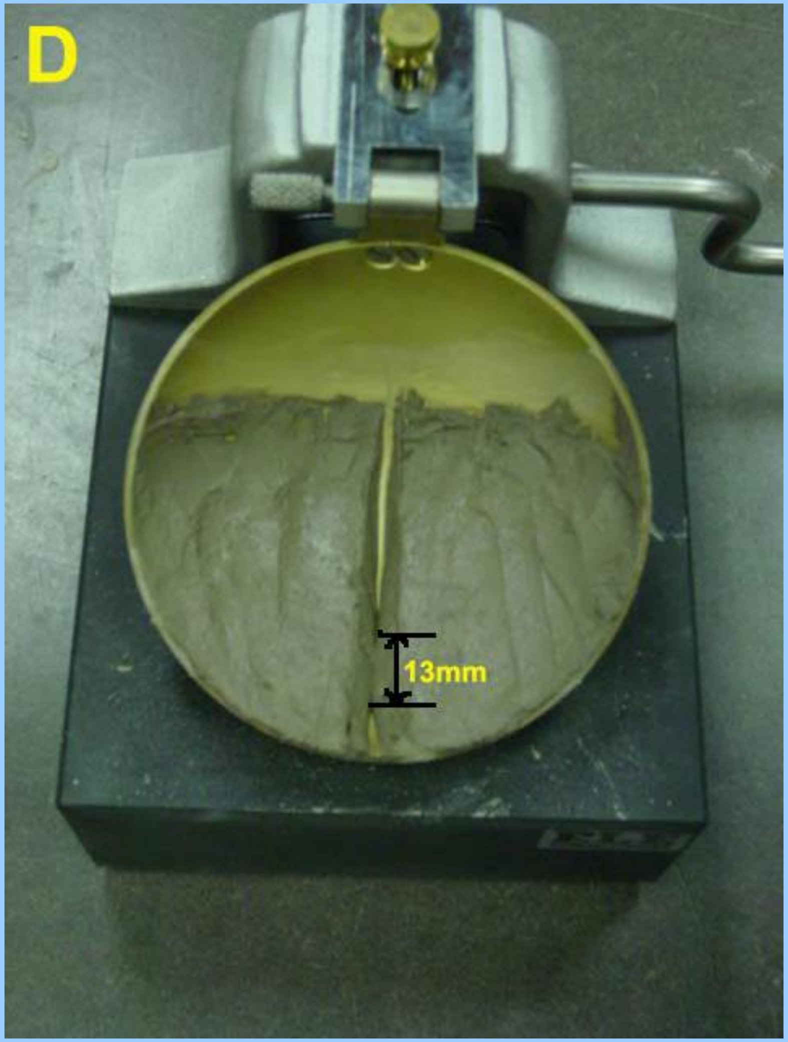
- Casagrande Liquid Limit Device
  - Flow Index, I_f
  - One Point Method
    - Toughness Index.

Liquid limit device



Liquid Limit Device

- Video



Liquid Limit Device

- Video

$$I_f = \frac{w_2 - w_1}{\log(\frac{N_2}{N_1})}$$

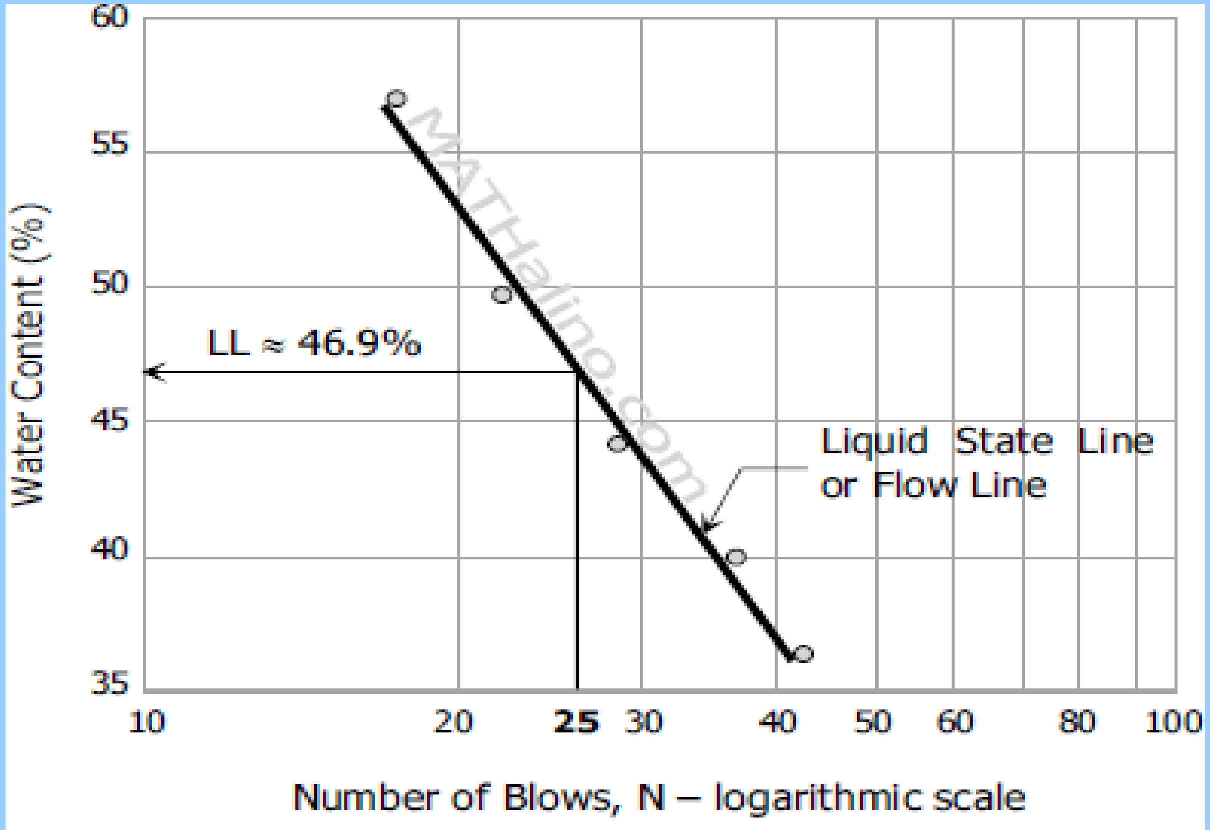


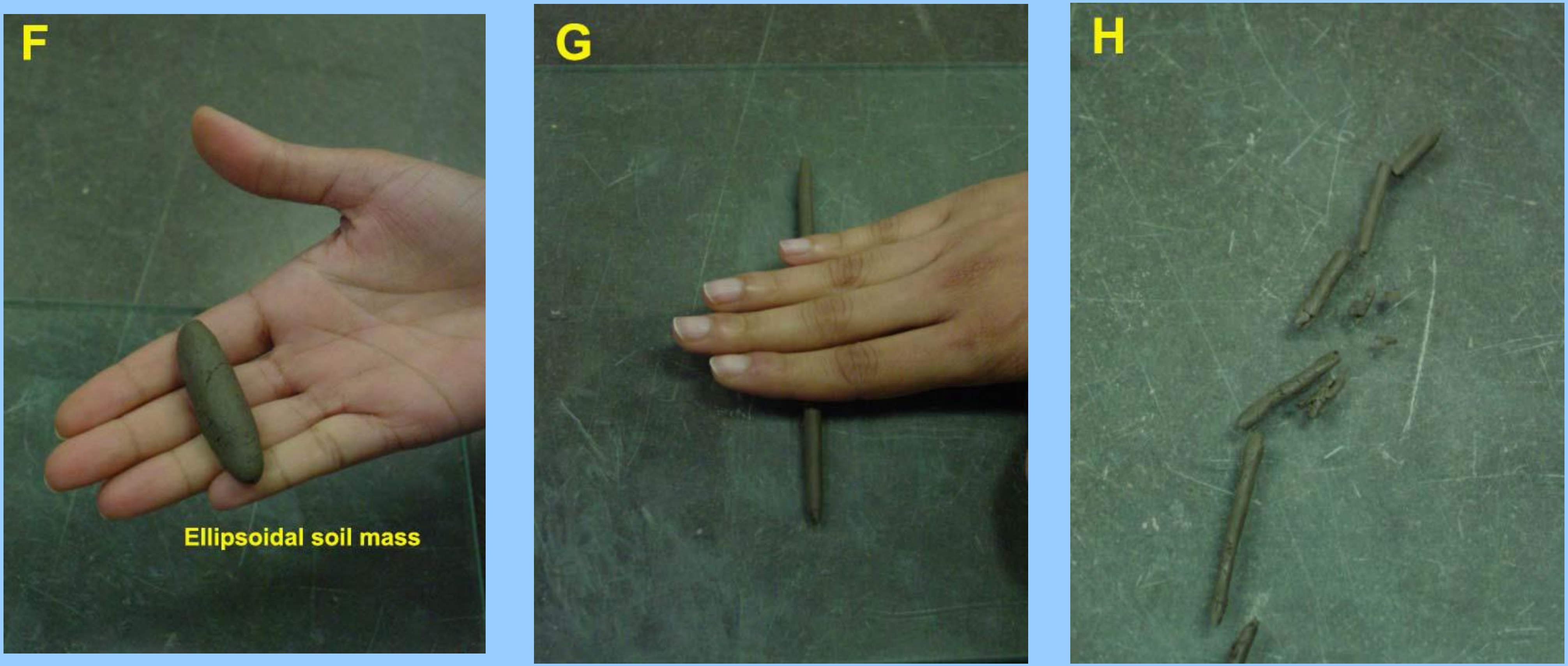
Figure 3 Typical liquid limit results from the Casagrande cup method.

Plastic Limit

- The moisture content at which a thread of soil just begins to crack and crumble when rolled to a diameter of 3 mm
- Minimum water content at which change in shape of soil is accompanied by visible cracks.

Plastic Limit

- Video



Indices

- Plasticity Index

$$PI = LL - PL$$

- Consistency Index

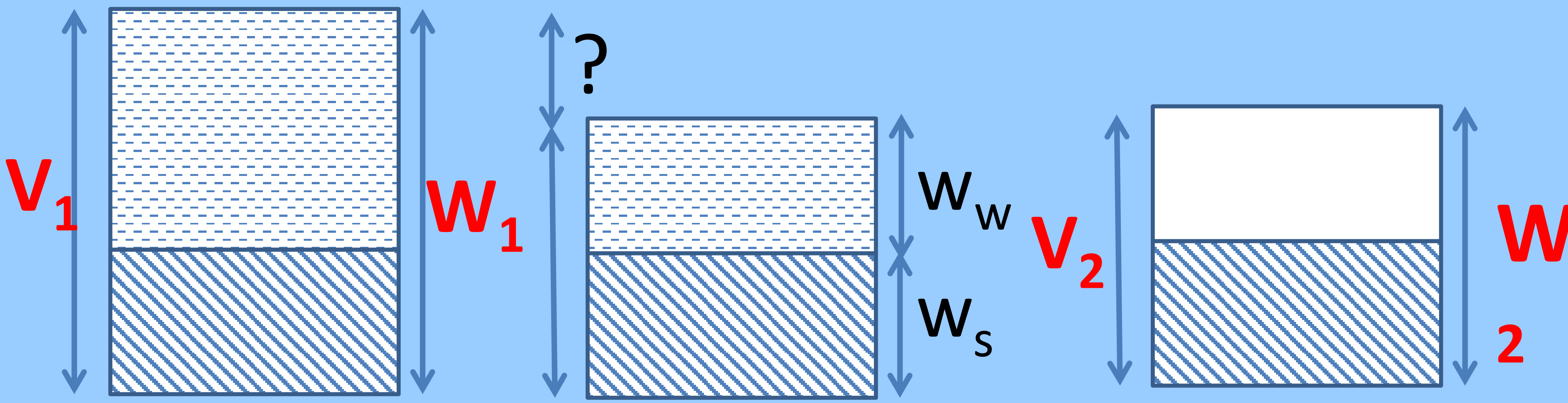
$$CI = \frac{LL - w}{LL - PL}$$

- Liquidity Index

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

- Problem:
- A sample of soil with LL of 72.8 was found to have a liquidity index of 1.21 and water content of 81.3. What are its plastic limit and plasticity index? Comment on the consistency of the soil.

Shrinkage Limit



$$SL = \frac{w_w}{w_s} = \frac{(W_1 - W_2) - (V_1 - V_2)\gamma_w}{W_2}$$

Shrinkage Limit

- Determination of G

$$G = \frac{W_2 / V_s}{\gamma_w} \quad , V_s = V_1 - \left[\frac{W_1 - W_2}{\gamma_w} \right]$$

$$G = \frac{W_2}{V_1 \gamma_w - (W_1 - W_2)}$$

Numerical on Shrinkage Limit

- In a shrinkage limit test, a container of volume 9.6cc was filled with soil slurry. The weight of saturated soil was 17.46 g. The slurry was then dried, first in atmosphere and then in oven at a constant temperature of 110 degC. The wt and volume of the dried soil was 11.58g and 5.22 cc respectively. Determine the shrinkage limit of the soil and its shrinkage ratio.

Shrinkage Ratio, R

- R, Ratio of given change in volume of soil expressed as % of dry volume to the corresponding volume change in water content above the shrinkage limit.

$$R = \frac{\dfrac{V_1 - V_2}{V_d} \times 100}{W_1 - W_2}$$

- Volumetric Shrinkage, Vs
- Problem

$$V_s = \frac{V_i - V_d}{V_d} \times 100$$

Activity of Clays

$$Activity = \frac{PI}{\% - Wt - \ finer - than - 2\mu}$$

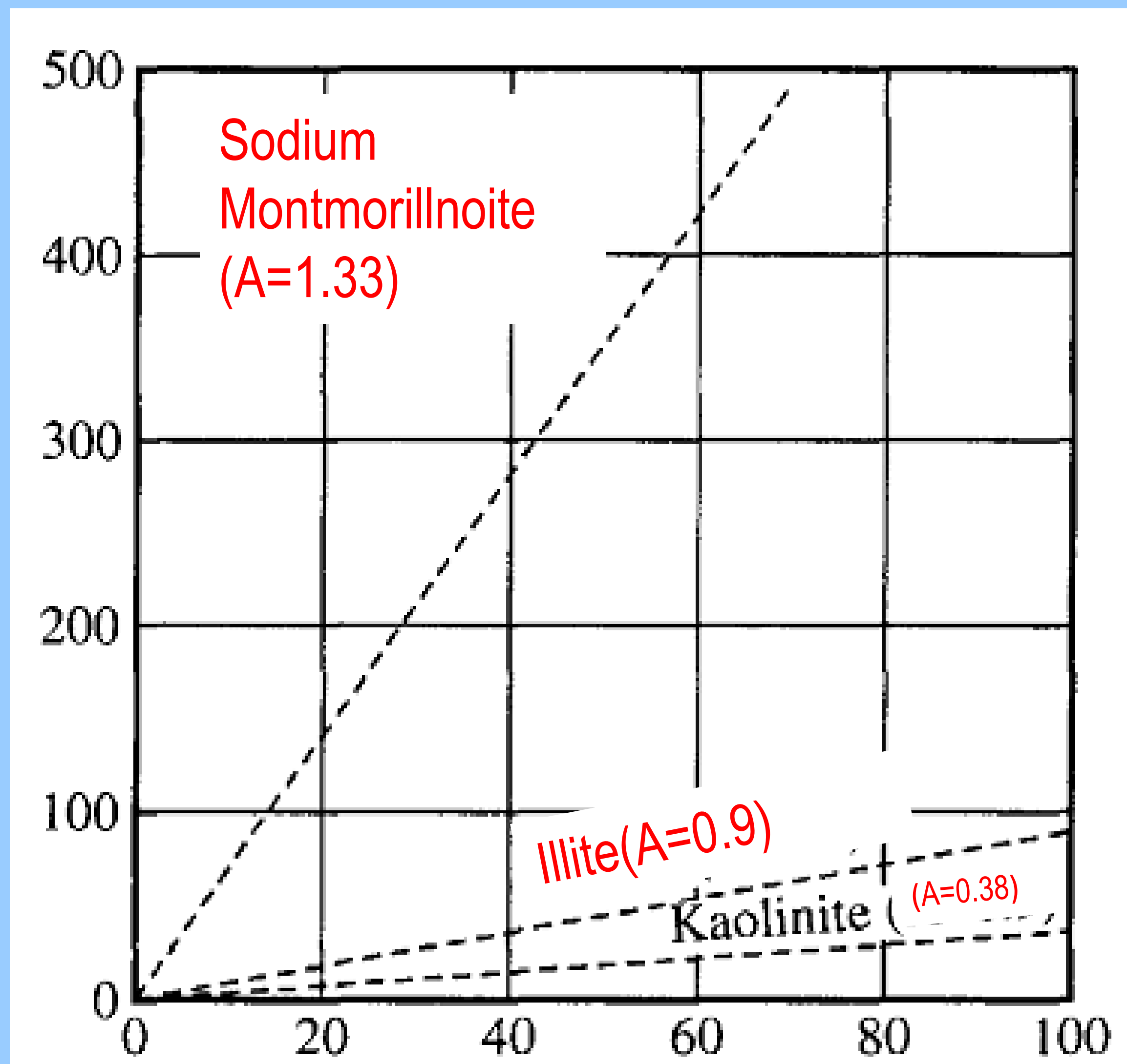
- Higher activity, more clay like behavior, higher swelling, higher shrinkage, higher compressibility
- Graph

Activity	Classification
< 0.75	Inactive
0.75-1.25	Normal
>1.25	Active

Activity of Clays

- Graph

Plasticity Index



% finer than 2μ

Numerical on Activity of Clays

- Two Clays have the following characteristics. Calculate their Activity Values. Compare their engineering behavior.

	Clay A	Clay B
LL	60	50
PL	25	30
% finer then 2μ	25	40

Numerical on clay behavior

- Two Clays have the following characteristics. Which of the soil is more plastic? Which of them is softer in consistency?

	Clay A	Clay B
LL	44	55
PL	20	35
Water content, w	25	50

Unconfined Compressive Strength

- Explanation: Video
- Mohr's Circle

Consistency	Strength kN/m^2
Very Soft	25
Soft	25-50
Medium	50-100
Stiff	100-200
Very Stiff	200-400
Hard	>400

Sensitivity

- Ratio of Unconfined compressive strength of an undisturbed soil specimen to its unconfined strength after remoulding.

$$s_t = \frac{(q_u)_{undisturbed}}{(q_u)_{remoulded}}$$

St	Classification
1-4	Normal
4-8	Sensitive
8-15	Extra-Sensitive
> 15	Quick

Thixotropy

- Property of certain clays by virtue of which they regain part of strength lost due to remoulding.
- Application: Driving pile foundations causes the soil to get disturbed, the soil structure loses strength. After a month, the soil regains part of its strength.

Lecture 7: Classification of Soils, Compaction, Consolidation

Dr. Mahendra Gattu

CLE 205 Soil Mechanics

Classification of Soils

- Unified Soil Classification System
  - Casagrande (1948) intended for use in airfield construction during World-War II
  - Coarse grain soils classified on the basis of GSD and fine grained soils on the basis of plasticity characteristics
    - Coarse-grained soils: Less than 50% passing through 75μ sieve
      - Gravel: If more than 50% retained on 4.75 mm sieve
      - Sand: More than 50% pass through 4.75mm sieve
    - Fine Grained soils: Greater then 50% passing through 75μ sieve
      - A-line of Plasticity Chart separates Clay and Silt.
      - LL> 50 High Compressibility
      - LL<50 Low Compressibility

Classification of Soils

- Unified Soil Classification System

Soil Type	Prefix	Sub Group	Suffix
Gravel	G	Well graded	W
Sand	S	Poorly graded	P
Silt	M	Silty	M
Clay	C	Clayey	C
Organic	O	LL<50	L
Peat	Pt	LL>50	H

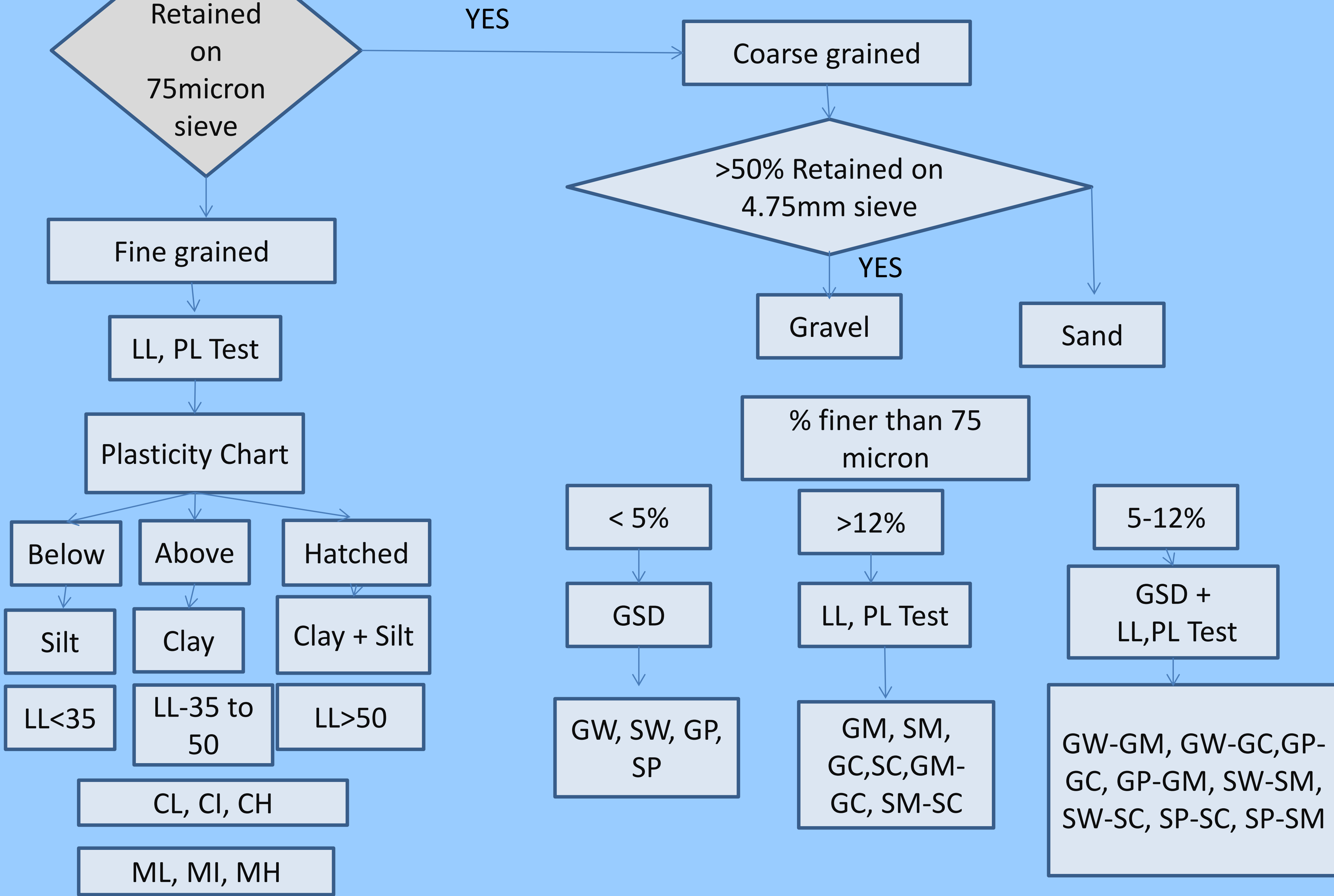
Classification of Soils

- AASHTO (American Association of State Highway Transport Officials)
- Developed by US Bureau of Public Roads(now Federal Highway Administration).
- Group Index, GI = 0.2 a + 0.0005 ac + 0.01 bd
  - a: min(% passing 75 μ -35, 75)
  - b: min(% passing 75 μ -15, 55)
  - c: min(LL-40, 60)
  - d: min(PI-10, 30)
    - GI =0 Good subgrade material,
    - GI> 20 Poor Subgrade material.

Classification of Soils

- INDIAN Soil Classification System
- 1959, Revised 1970.
- Based on USCS: Main Difference Fine Grained Soils classified as Low, Medium and High Compressibility.
- Step by Step Procedure:
  - Flow Chart
  - A-line

Flow chart



>50% Retained on 75micron sieve
YES
Coarse grained
>50% Retained on 4.75mm sieve
YES
Gravel
Sand
Fine grained
LL, PL Test
Plasticity Chart
Below
Above
Hatched
Silt
Clay
Clay + Silt
LL<35
LL-35 to 50
LL>50
CL, CI, CH
ML, MI, MH
% finer than 75 micron
< 5%
>12%
5-12%
GSD
LL, PL Test
GSD + LL,PL Test
GW, SW, GP, SP
GM, SM, GC,SC,GM-GC, SM-SC
GW-GM, GW-GC,GP-GC, GP-GM, SW-SM, SW-SC, SP-SC, SP-SM

Soil Compaction

- Compaction vs Consolidation
- Laboratory Tests:
- Proctor Test, Modified Proctor Test
- Dry Density vs Moisture Content
- Zero Air Void Line
- Compactive Effort
- Types of Soil
- Compaction curve for sand
- Problem

Compaction

Smooth wheeled rollers

Pneumatic Tyre rollers

Compaction

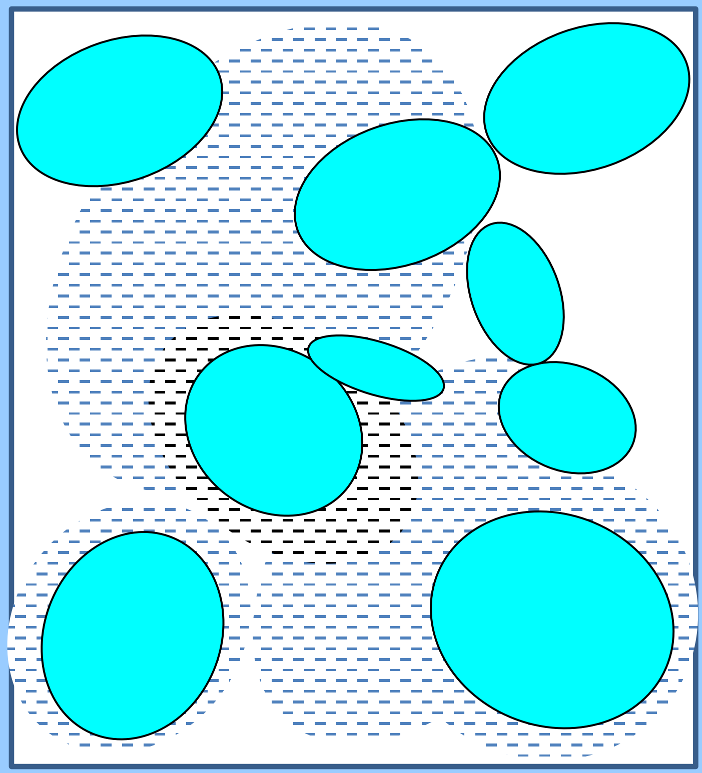
Dynamic/
Impact
compaction

Thought Experiment- 1

- Imagine you have a round plastic bottle in your hand.
- You pour round shaped pepper mint into it, many in number.
- Then you shake the bottle vigorously
- What happens to the pepper mints and the void space?

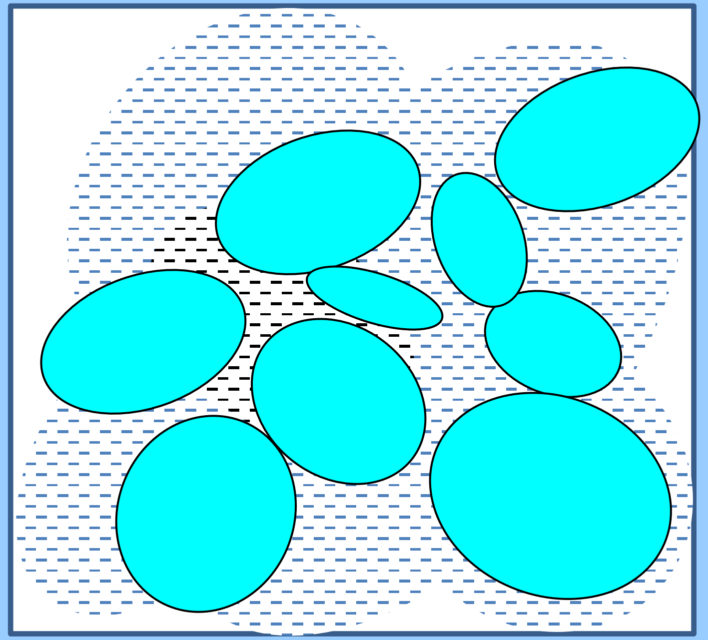
- Fill a small quantity of water in the plastic bottle.
- Again Add peppermints and shake the bottle?
- Do you require the same amount of effort now to reduce the void space?

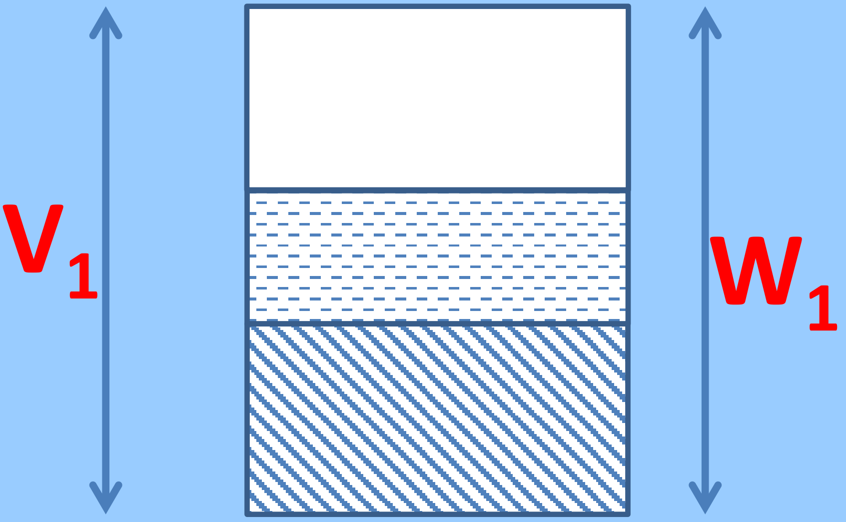
Compaction

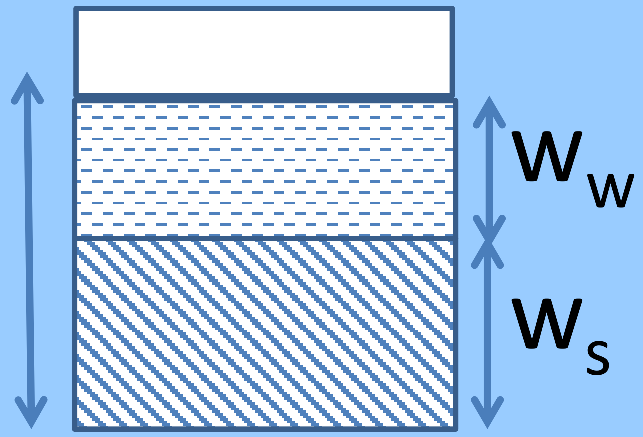


Compactive effort

Reduction in Void space





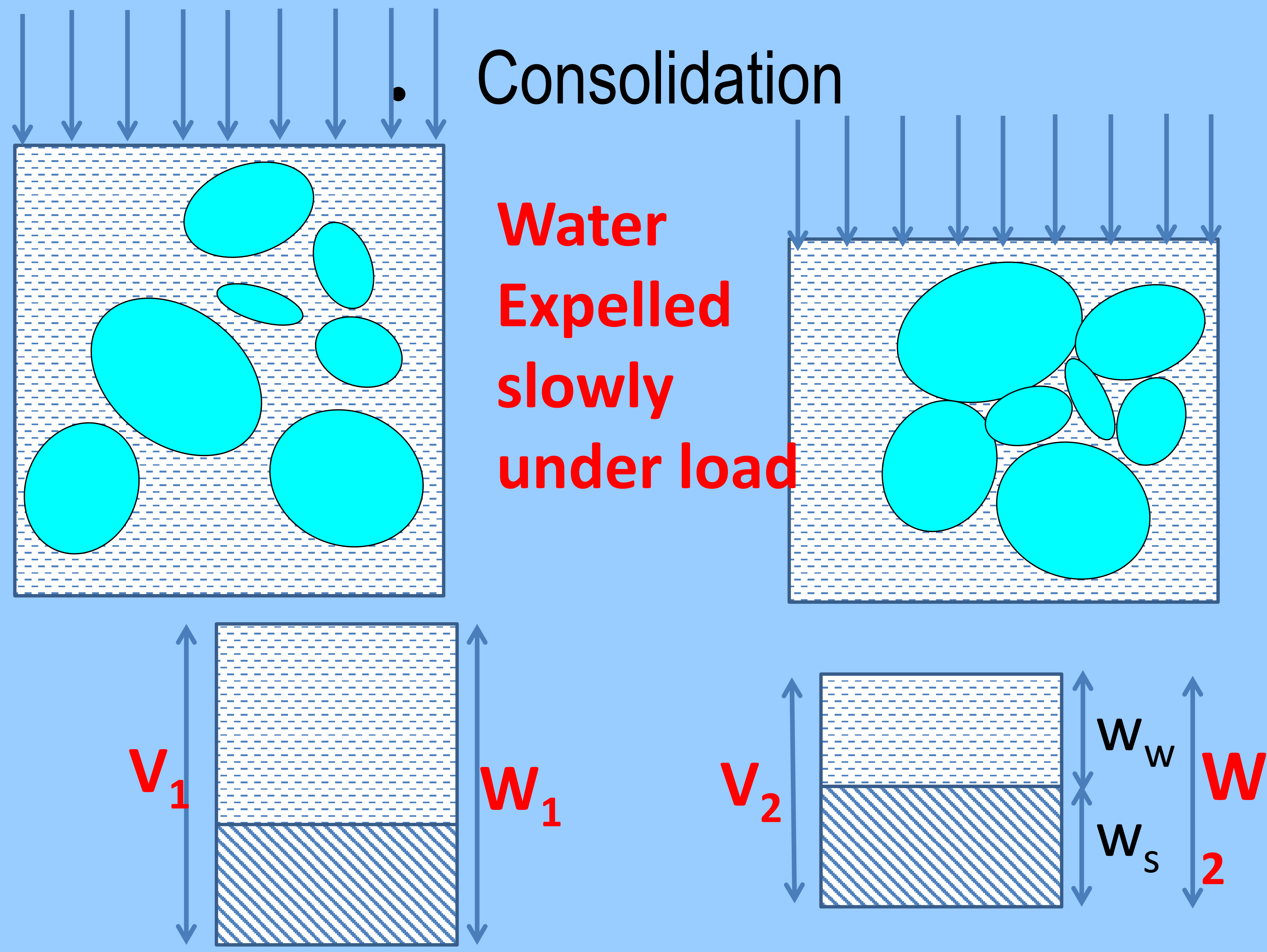


Compaction Advantages

- Grain to Grain Contact Increases—Increased shear strength
- Reduces Permeability of Soil by decreasing the void space.
- Increases bearing capacity of soil
- Prevents settlement of soil and undesirable volume changes.
- Prevents build up of large pore pressures that cause soil to liquefy under earthquakes.

Thought Experiment-2

- Take a rectangular sponge block.
- Immerse it in water for some time.
- Remove the sponge block and place it beside a measure scale.
- Very Slowly start applying load on the sponge.
- What happens?


Consolidation
Water Expelled slowly under load
V1
W1
V2
Ww
Ws
W2

Compaction vs Consolidation

- Soil is always unsaturated
- Densification due to reduction of air voids at a given water content
- Instantaneous phenomenon.
- Occurs due to compaction technique.

- Soil: Completely Saturated
- Volume reduction is due to expulsion of water from voids
- Time-dependent phenomenon.
- Occurs due to load placed on soil

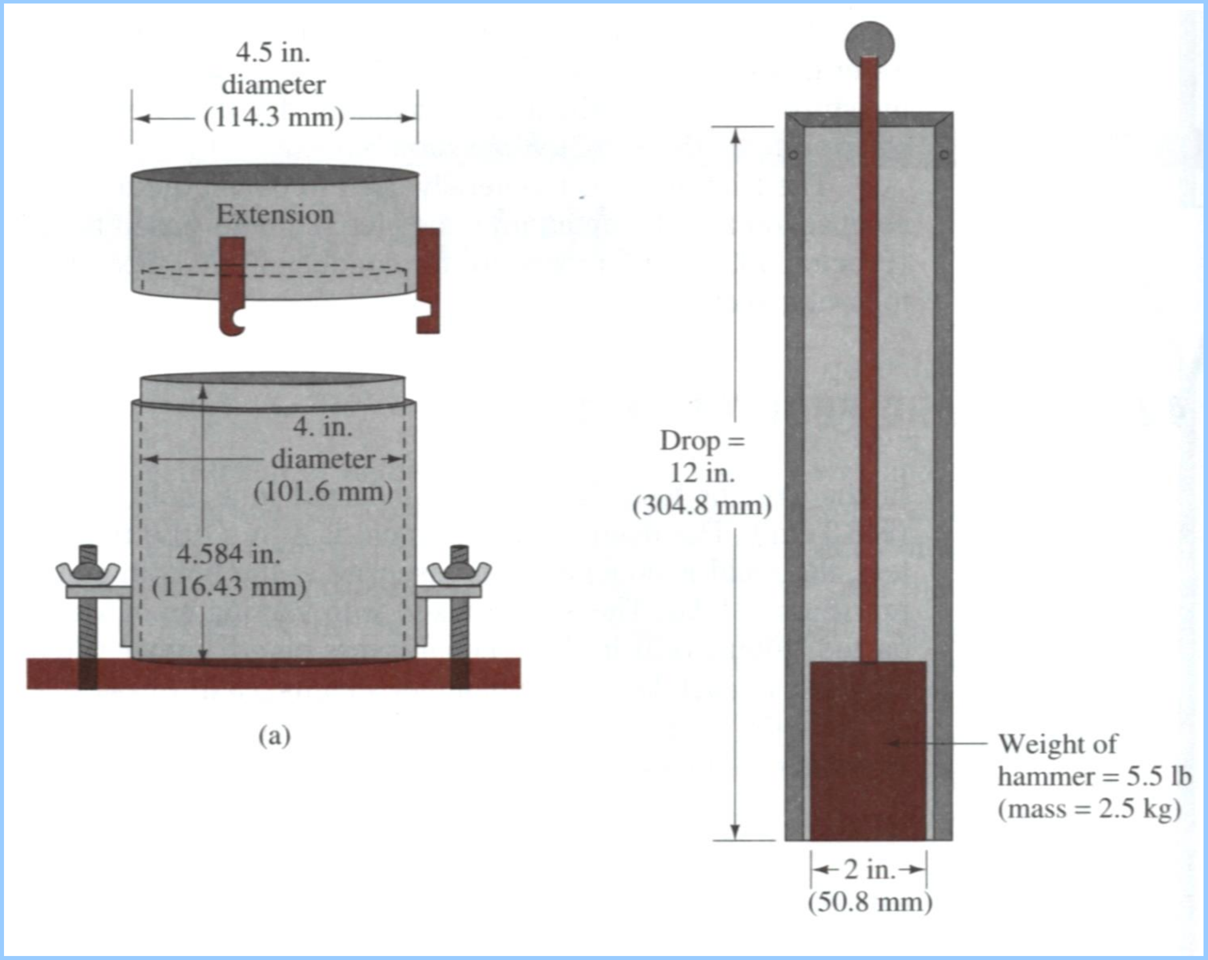
Compaction Laboratory tests

<u>Purpose</u>

To determine the *<u>proper amount of mixing water</u>* to use when compacting the soil in the field to achieve maximum density.

- RR Proctor :
  - Standard Proctor Test,
  - Modified Proctor Test
- IS equivalent of Standard Proctor Test is Light Compaction Test
- IS equivalent of Modified Proctor Test is Heavy Compaction Test

Test Equipment



Test Procedure

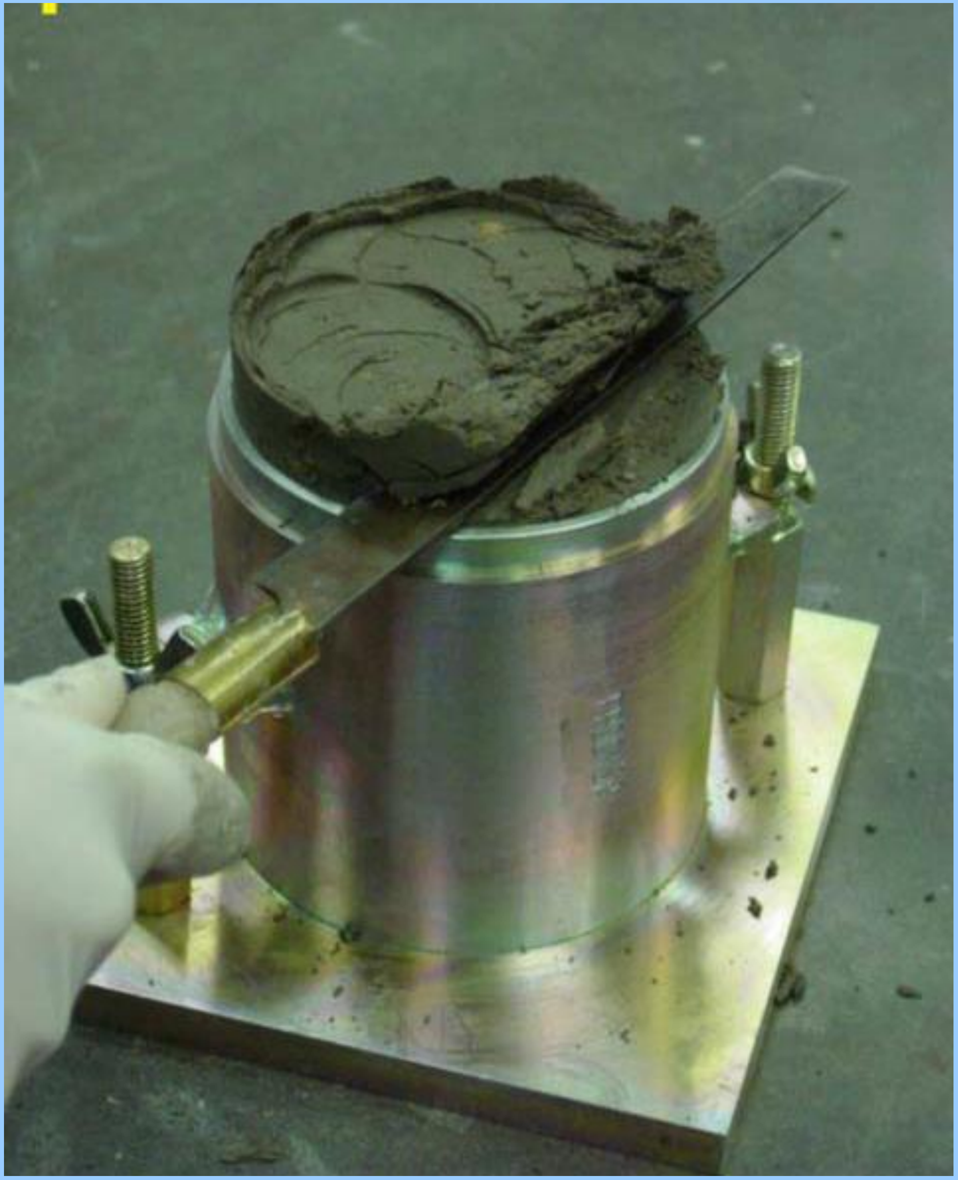
Test Procedure

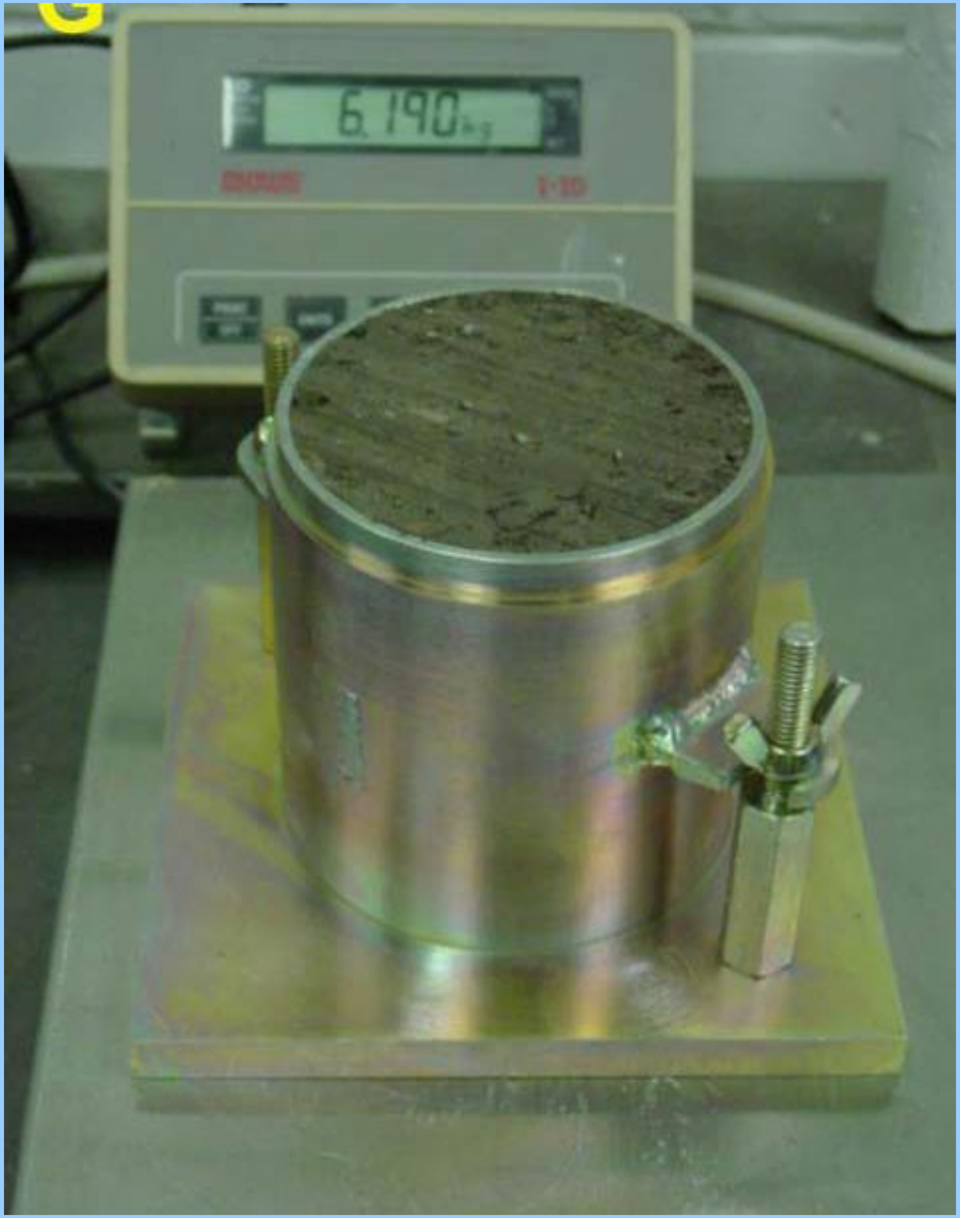
Standard Proctor Test

3 layers

- 25 blows per layer
- 2.7Kg hammer
- 300mm drop

Test Procedure





Test Procedure

Oven Dried to determine

Water Content.

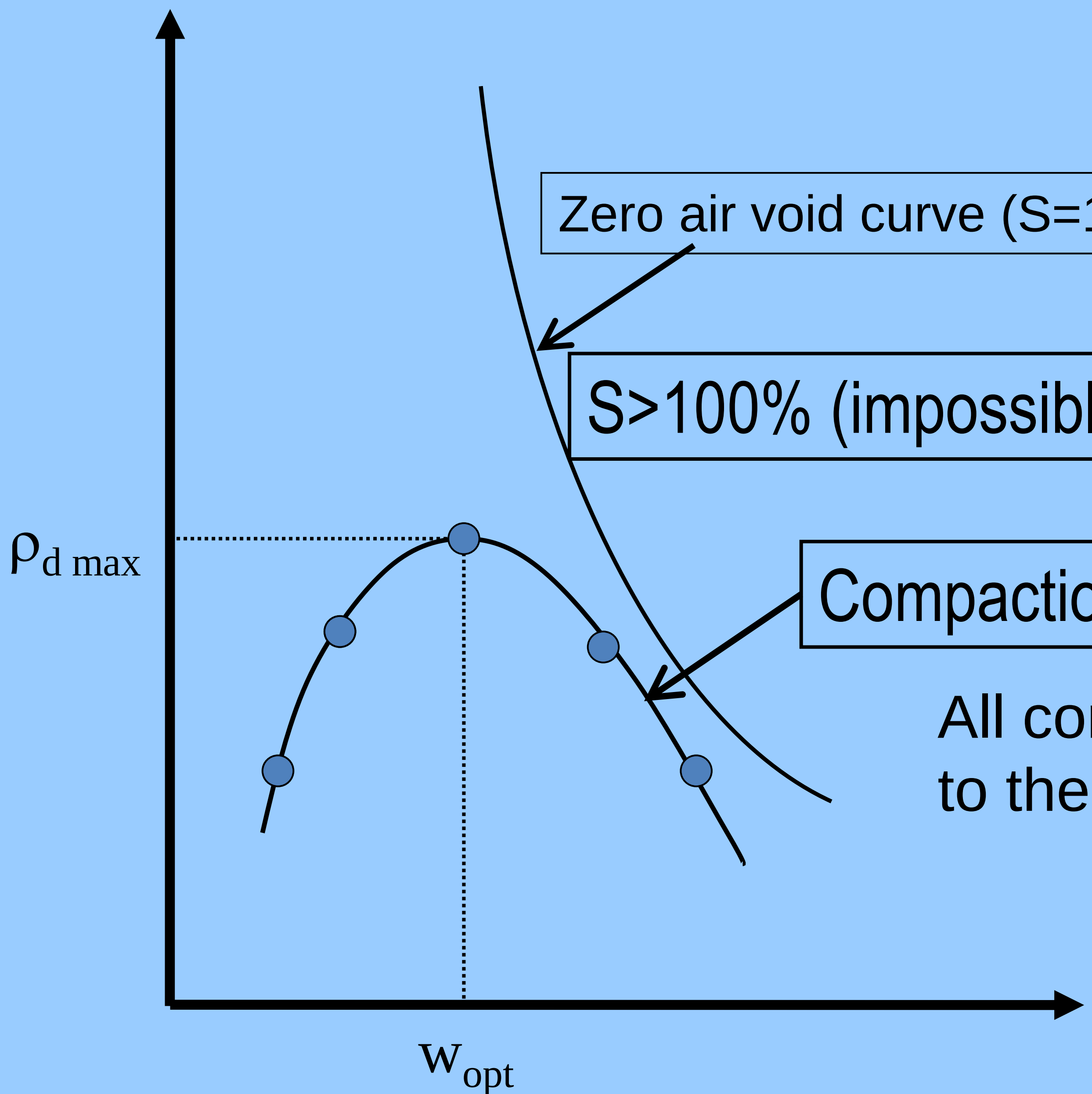
Test Procedure

- For different water content of soil, weight of the compacted soil in the mould is measured.
- The dry weight is then determined as

$$\gamma_d = \frac{\gamma_{bulk}}{1 + w}$$

- Plot: Max dry density-OMC

Test Procedure: Results



All compaction points should lie to the left of ZAV curve

Laboratory tests

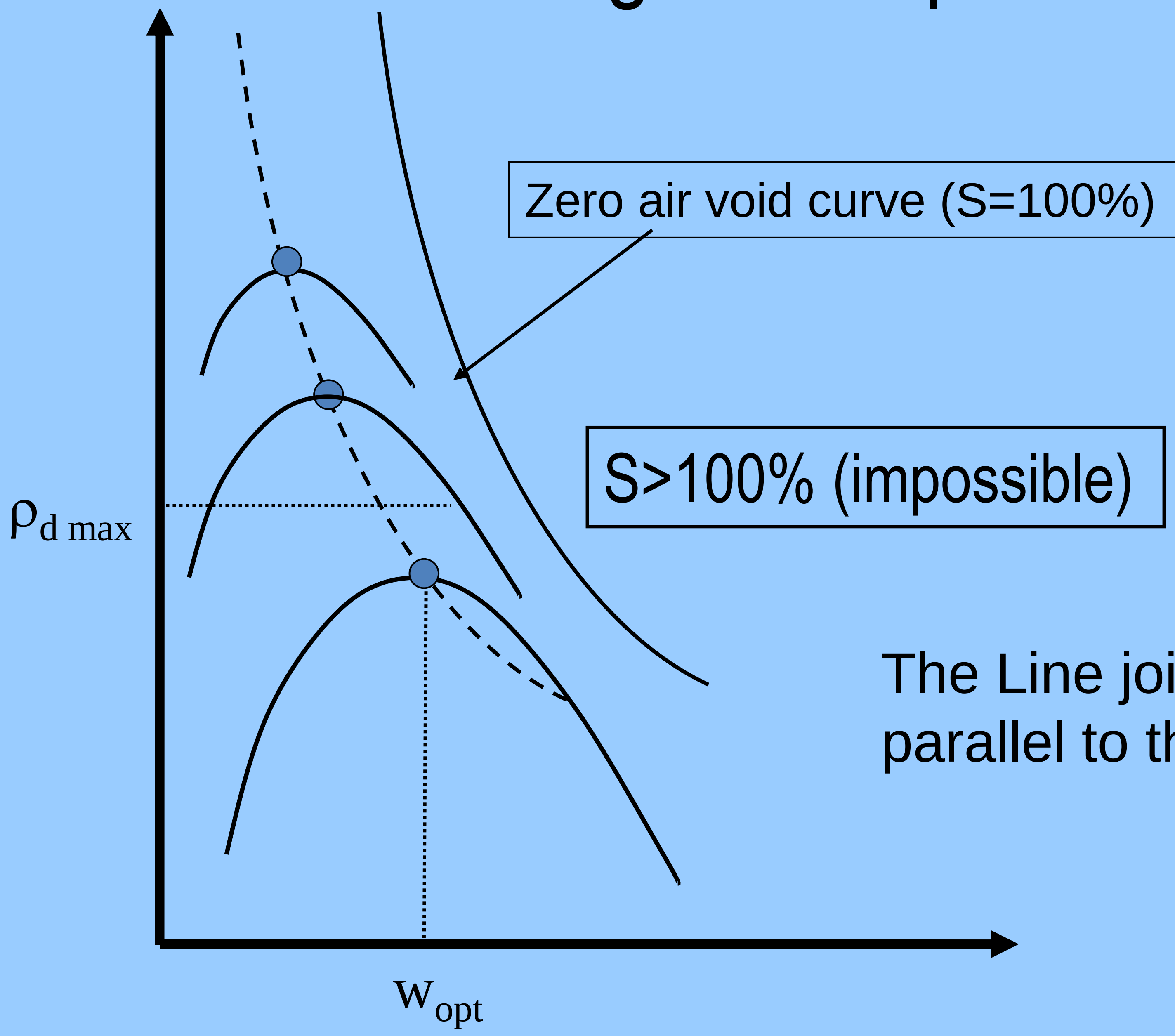
IS Code	Light Compaction Test	Heavy Compaction Test
Volume of Mould	1000 cc	1000 cc
Weight of Hammer	2.6 kg	4.9 kg
Drop of Hammer	310 mm	450 mm
Layers x Blows	3 layers x 25 blows	5 layers x 25 blows
Compaction Energy per unit volume	?	?

Compactive Effort

With increasing compactive effort,

- What happens to γ_{dmax}?
- What happens to OMC?
- Given $\gamma^3_{dmax} > \gamma^2_{dmax} > \gamma^1_{dmax}$ and $OMC_3 < OMC_2 < OMC_1$
  - Which is greater $\gamma^2_{dmax} - \gamma^1_{dmax}$ or $\gamma^3_{dmax} - \gamma^2_{dmax}$?
  - What is greater $OMC_1 – OMC_2$ or $OMC_2 – OMC_3$?
- To which line is line joining the peak parallel to? What does it mean?

Increasing in Compaction Effort

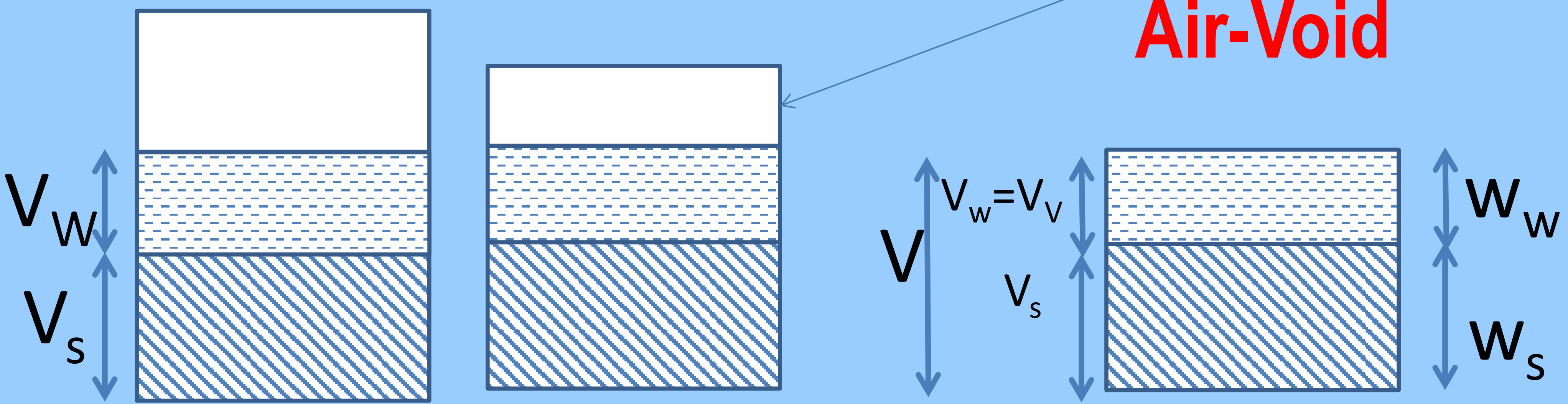


The Line joining the maximum is parallel to the ZAV

Zero Air-Void Line

- For a given soil, theoretically obtained maximum dry unit weight for a soil at a given water content.
- Explanation using phase diagram

Zero Air-Void



Air Voids reduce on application of compaction energy and theoretically can reach zero

Zero Air-Void Line

$$\gamma_d = \frac{W_s}{V} = \frac{G\gamma_w V_s}{V_v + V_s}$$

$$= \frac{G\gamma_w}{\frac{V_v}{V_s} + 1}$$

$$= \frac{G\gamma_w}{\frac{G\gamma_w V_v}{G\gamma_w V_s} + 1}$$

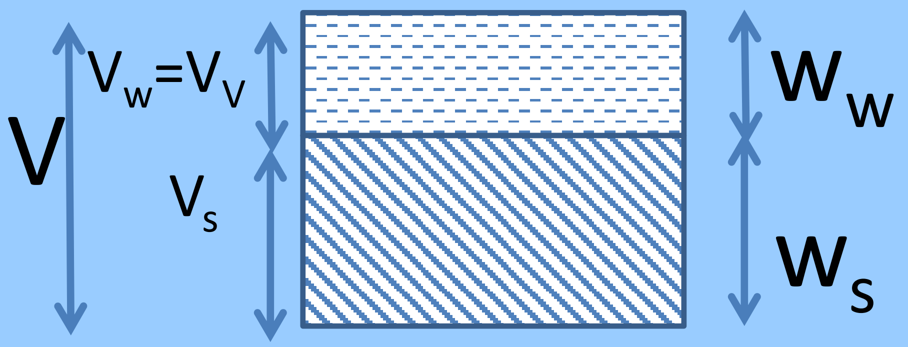
$$= \frac{G\gamma_w}{\frac{G\gamma_w V_w}{W_s} + 1}$$

$$= \frac{G\gamma_w}{\frac{GW_w}{W_s} + 1}$$

$$= \frac{G\gamma_w}{Gw + 1}$$

- Derivation of Zero Air Void Line from first principles for different water content w

Zero Air-Void



Lecture 8: Total Stress, Effective Stress, Neutral Stress

Dr. Mahendra Gattu

CLE 205 Soil Mechanics

Today's Agenda

- Total Stress, Pore Pressure, Effective Stress
- Problem
- Capillary Action in soils
- Problem

Total Stress

- Stress = Force/Area
- Vertical Stress = Mass x g / Area
- Vertical Stress at depth h for soil of unit weight γ.

$$\sigma = h\gamma$$

- Can be measured physically.

Pore Pressure

- Denoted by u
- Acts equally on all sides: Hydrostatic pressure
- Can be measured physically.

$$u = h_w \gamma_w$$

Effective Stress

- Denoted by $\overline{\sigma}$
- Measure of intergranular stresses
- Mechanical behavior is linked to effective stress
  - Shear Strength, Void Ratio, Compressibility

$$\overline{\sigma} = \sigma - u$$

- Cannot be measured physically

Physical Meaning of Effective Stress

- Diagram
- Plane passing through points of inter-particle contact.
- Resolving the forces into tangential T and normal N’ directions and equillibrium in Normal Direction

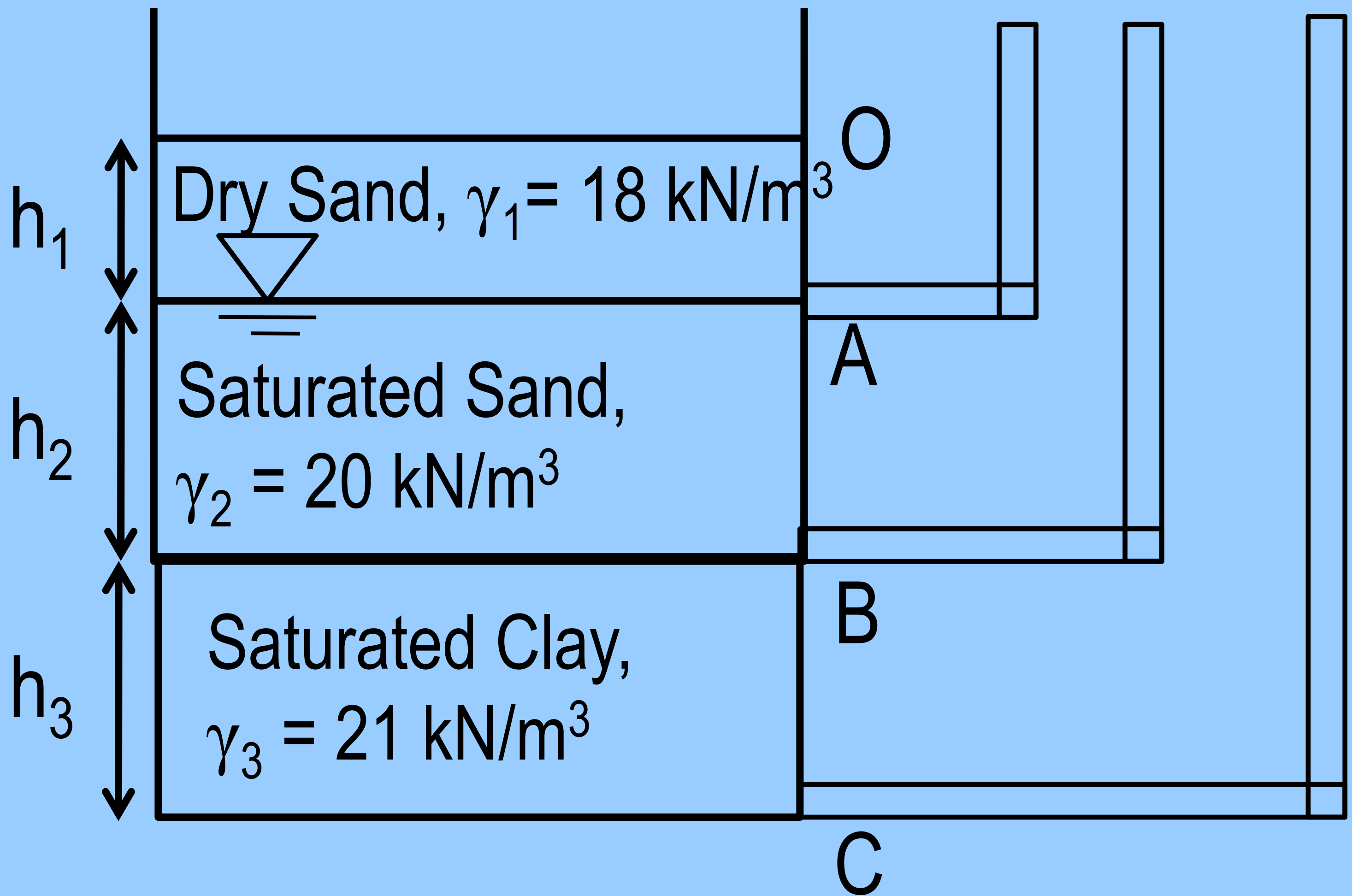
$$P = \sum N' + uA_w$$

$$\sigma = \frac{P}{A} \quad \bar{\sigma} = \frac{\Sigma N'}{A} \quad \frac{A_w}{A} \approx 1$$

$$\sigma = \bar{\sigma} + u$$

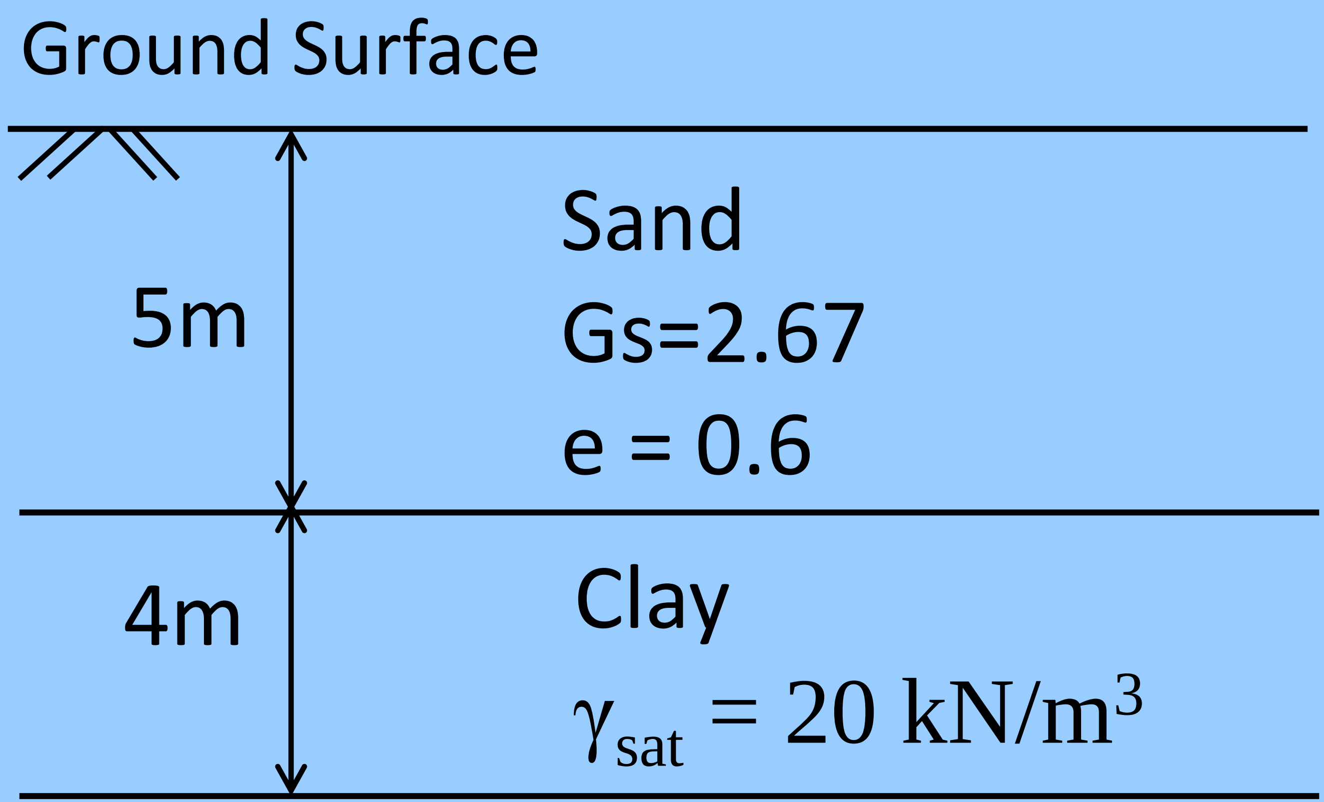
Problem 1

- Determine Neutral Stress and Total pressure at A, B,C

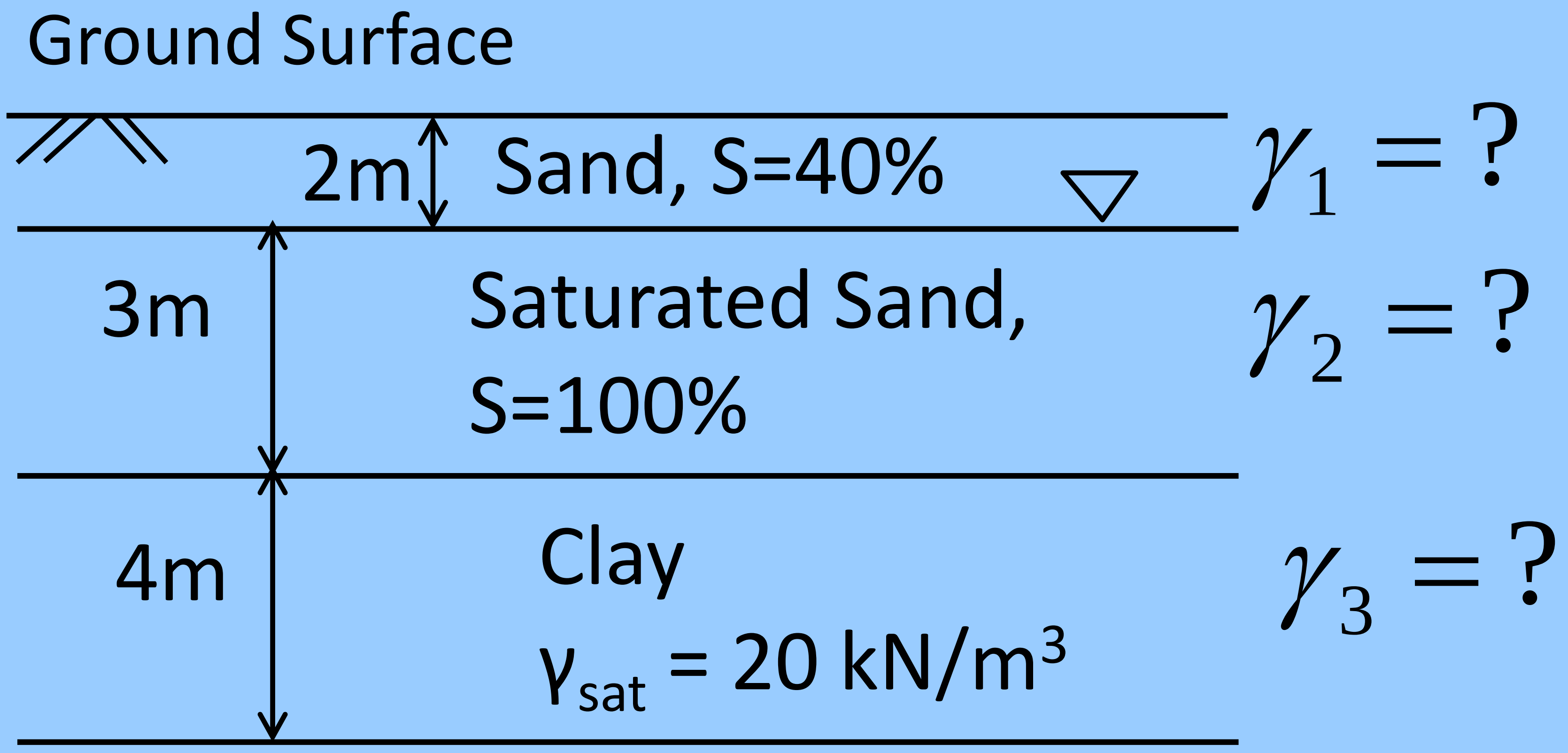


Problem 2

For the subsoil conditions shown in figure, plot the total, neutral and effective stress distribution upto the bottom of the clay layer, when the water table is at 2m below the ground surface (take S = 50% above the water table).



Solution



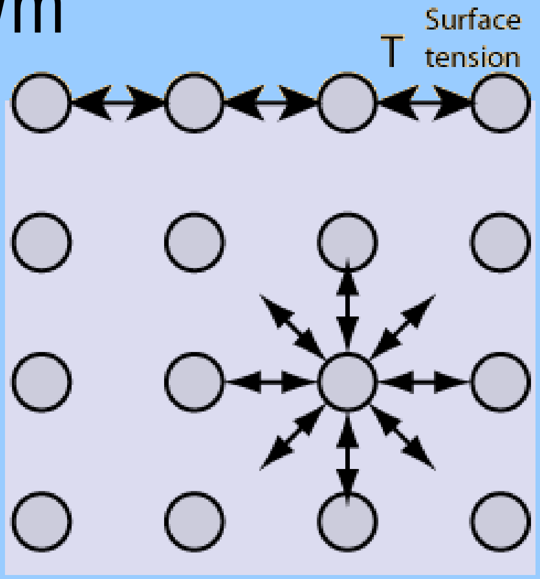
Ground Surface
2m
Sand, S=40%
3m
Saturated Sand,
S=100%
4m
Clay
γ_{sat} = 20 kN/m^3
$\gamma_1 = ?$
$\gamma_2 = ?$
$\gamma_3 = ?$

Capillary water in Soils

- What is surface tension?
- How does capillary rise of water takes place in soils?
- What is the effect of capillary rise in soils?
- Problem

Surface Tension

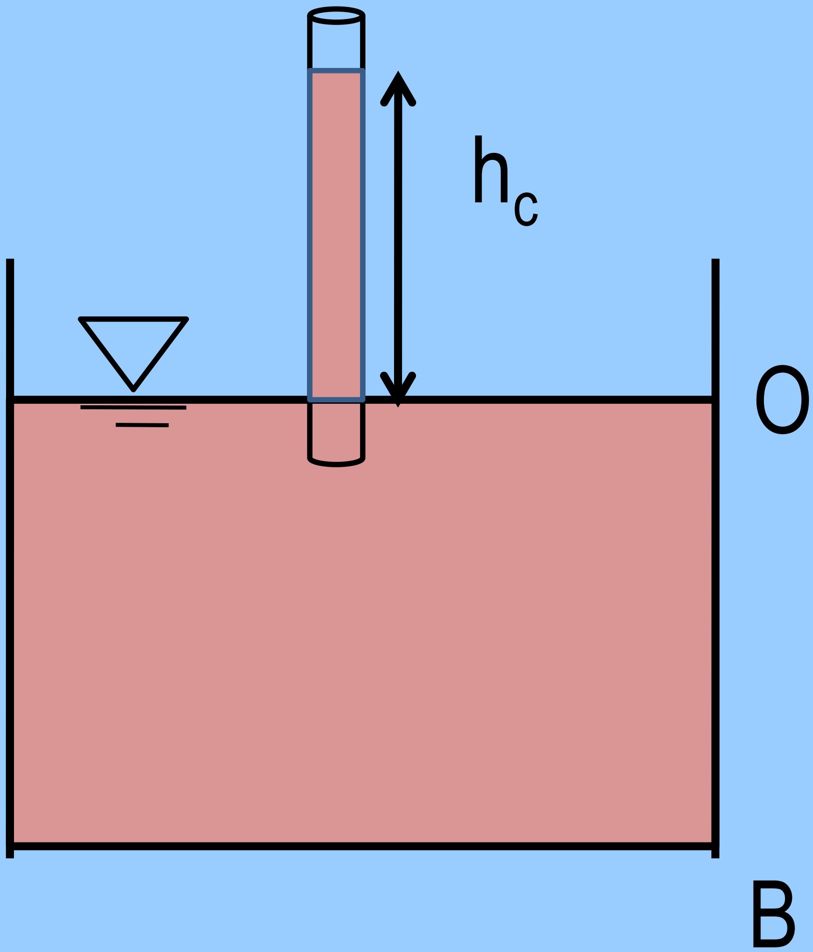
- Attractive forces/ Cohesive forces are present between molecules. Those on the surface have no neighboring atoms above, and exhibit stronger attractive forces upon their nearest neighbors on the surface. This enhancement of the intermolecular attractive forces at the surface is called surface tension.
- Unit Force/Unit Length i.e N/m



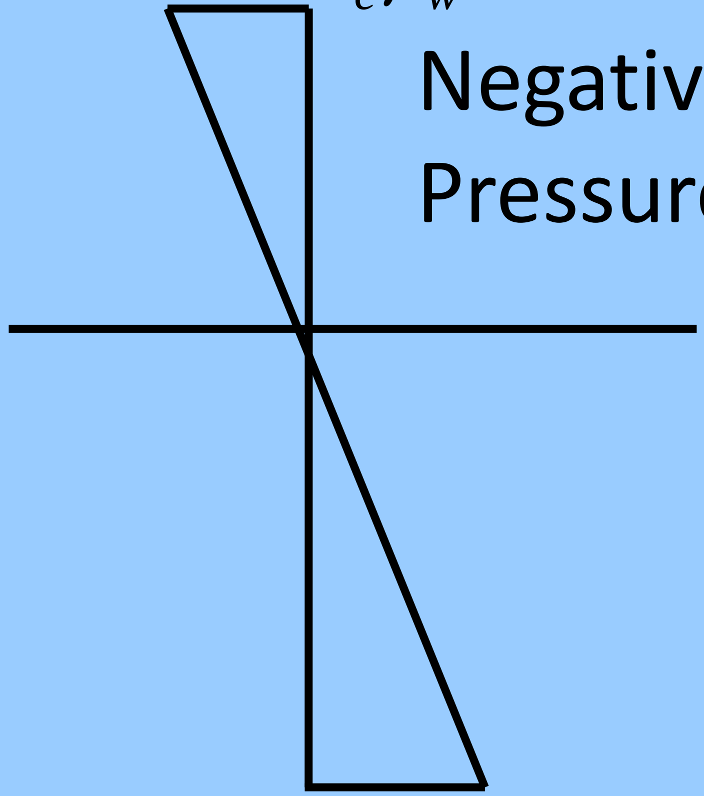
Capillary Rise in pipette

- Derivation

$$h_c = \frac{4T}{\gamma_w d}$$



$$u = -h_c \gamma_w$$

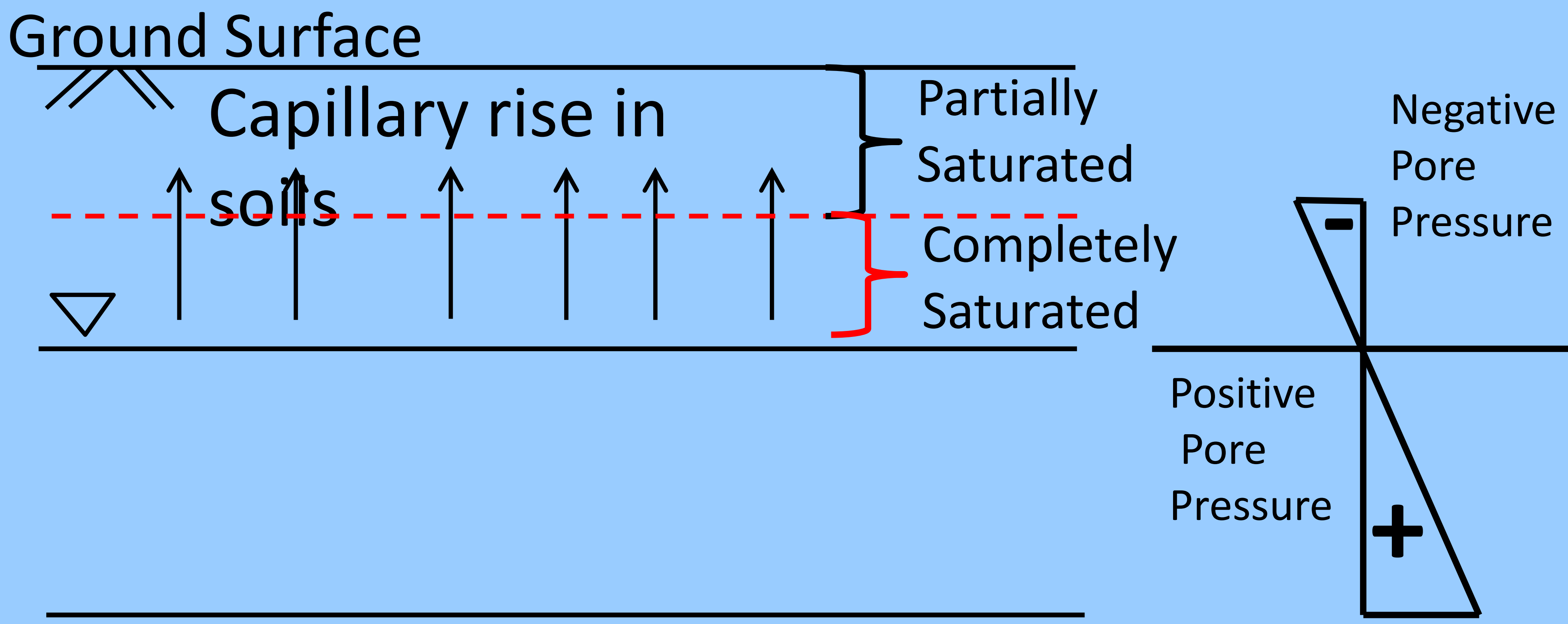


Negative Pore Pressure

Capillary Rise in Soils

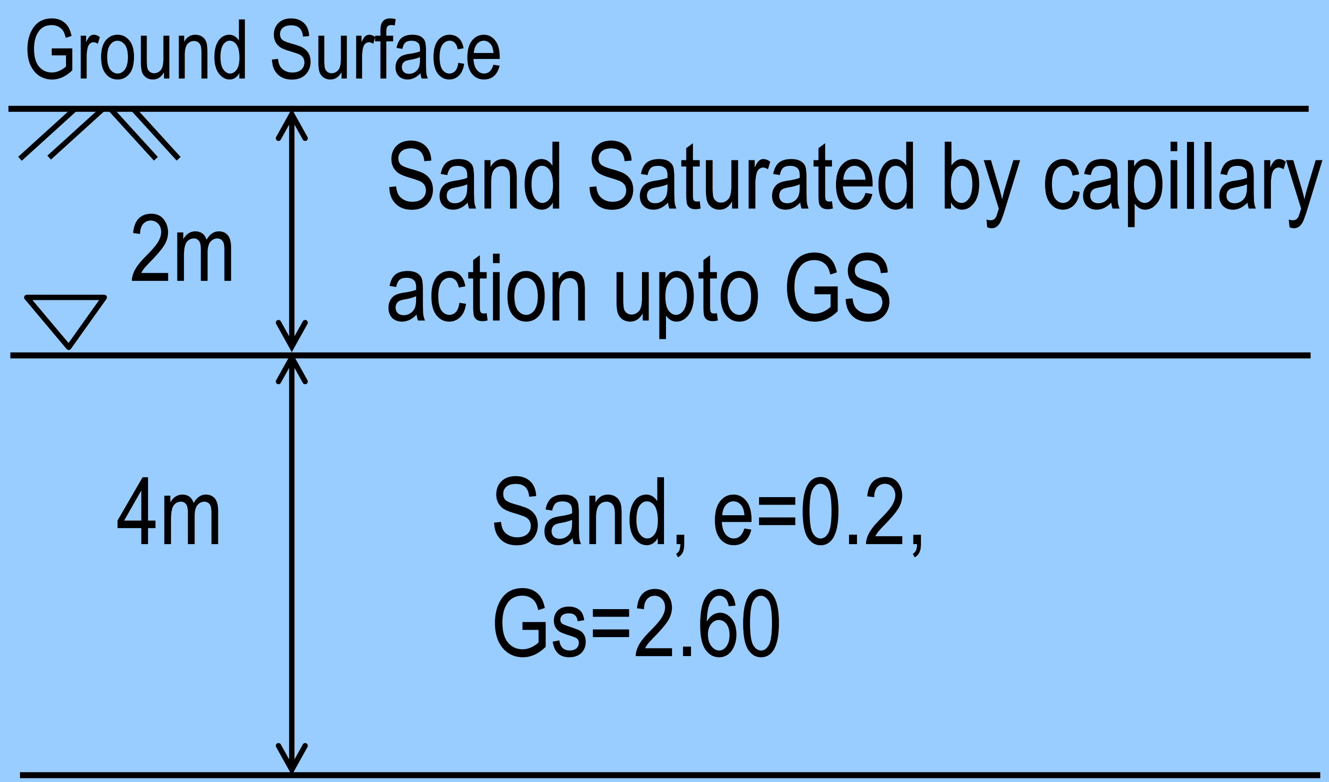
- Negative pore water pressure
- Increase in effective stress

$$\bar{\sigma} = \sigma - (-u)$$



Problem

- For the subsoil conditions shown in the figure what are the effective stress values at 1m, 2m and 4m depths.



$$\gamma_{sat} = ?$$

Lecture 9: Permeability

Dr. Mahendra Gattu

CLE 205 Soil Mechanics

Today's Agenda

- What is permeability?
- Why is it important in soil mechanics?
- What is Darcy's law?
- How is permeability measured in laboratory?
- How is permeability measured in field?

Permeability

- Permeability is a soil property which describes quantitatively the ease with which soil allows water to flow through it.
- More is the permeability, more is the ease with water can flow through that soil.
- Permeability of Gravel > Sand > Silt > Clay. Permeability decreasing with decreasing grain size.

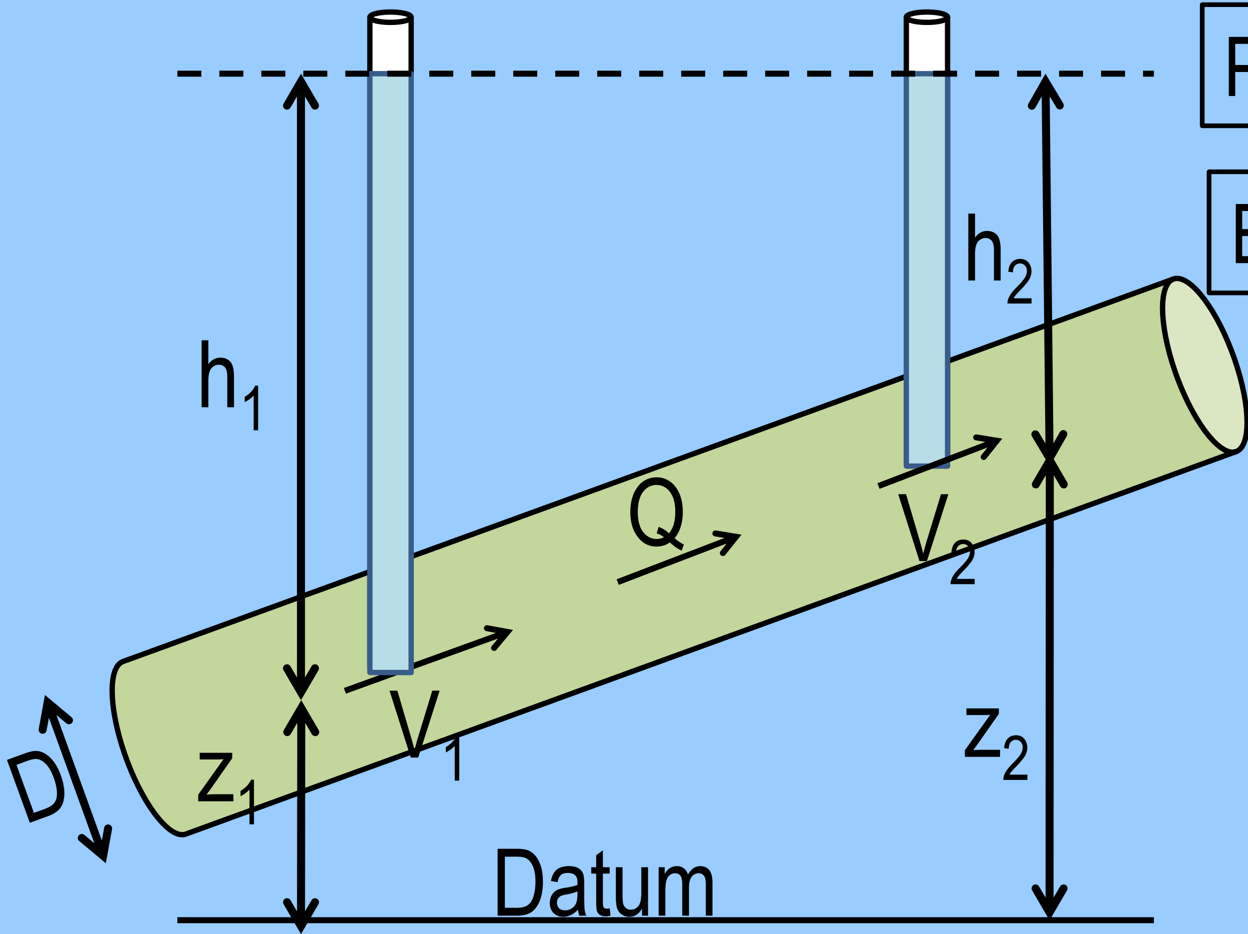
Importance of permeability in soil mechanics

- Excavations of open cuts in sand below water table
- Seepage through earth embankment dams
- Subgrade drainage
- Rate of consolidation of compressible soils.

Fluid Mechanics Basics

- Bernoulli's equation

$$\frac{p_1}{\gamma} + \frac{V_1^{\,2}}{2g} + z_1 = \frac{p_2}{\gamma} + \frac{V_2^{\,2}}{2g} + z_2$$



Pressure Head $\frac{p}{\gamma_w} = h$

Elevation Head z

Velocity Head $\frac{V^2}{2g}$

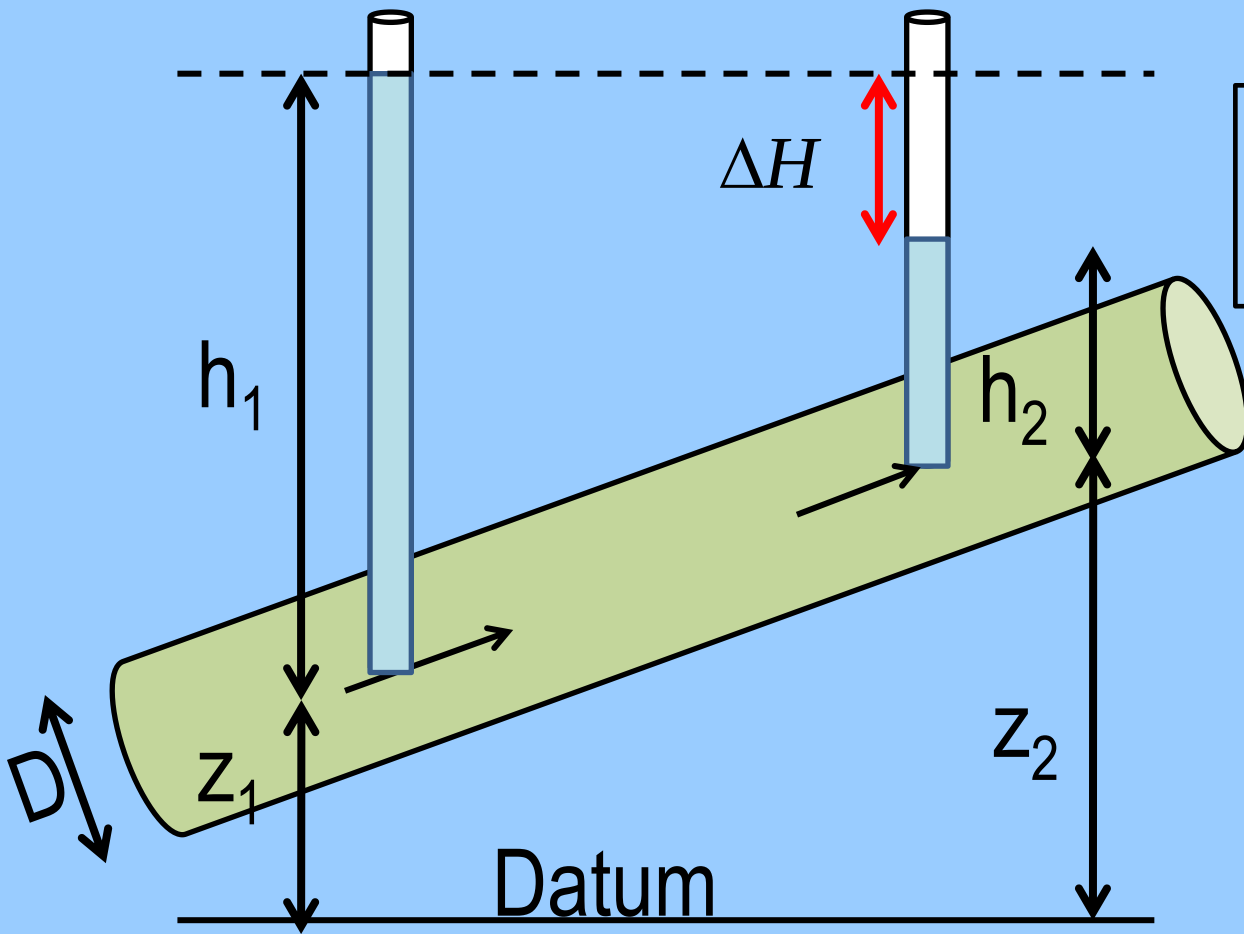
$$\frac{p_1}{\gamma} + z_1 = \frac{p_2}{\gamma} + z_2$$

$$h_1 + z_1 = h_2 + z_2$$

Fluid Mechanics Basics

- Head Loss

$$\Delta H_L = \left(\frac{p_1}{\gamma} + \frac{V_1^{\,2}}{2g} + z_1 \right) - \left(\frac{p_2}{\gamma} + \frac{V_2^{\,2}}{2g} + z_2 \right)$$



For Pipes of uniform cross-section

$$\Delta H = \left(\frac{p_1}{\gamma} + z_1 \right) - \left(\frac{p_2}{\gamma} + z_2 \right)$$

$$\Delta H = \left(h_1 + z_1 \right) - \left(h_2 + z_2 \right)$$

Fluid Flow in Soils

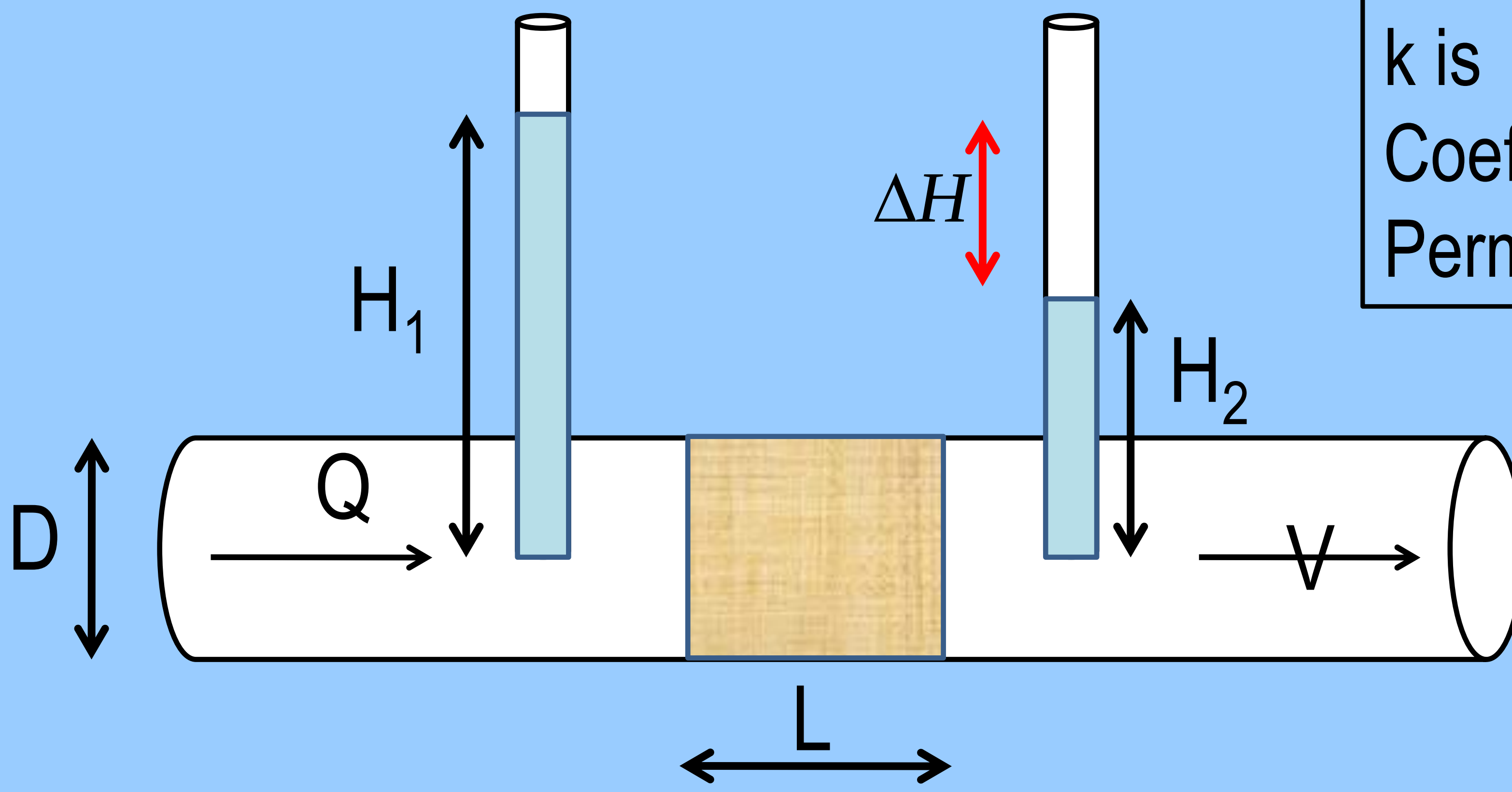
- Darcy's Law $Q \propto \dfrac{\Delta H}{L} A$

$$Q = k\left[\frac{\Delta H}{L}\right]A$$

Hydraulic Gradient $i = \dfrac{\Delta H}{L}$

$$V = ki$$

k is Coefficient of Permeability



Problem

- Determine k

Given:

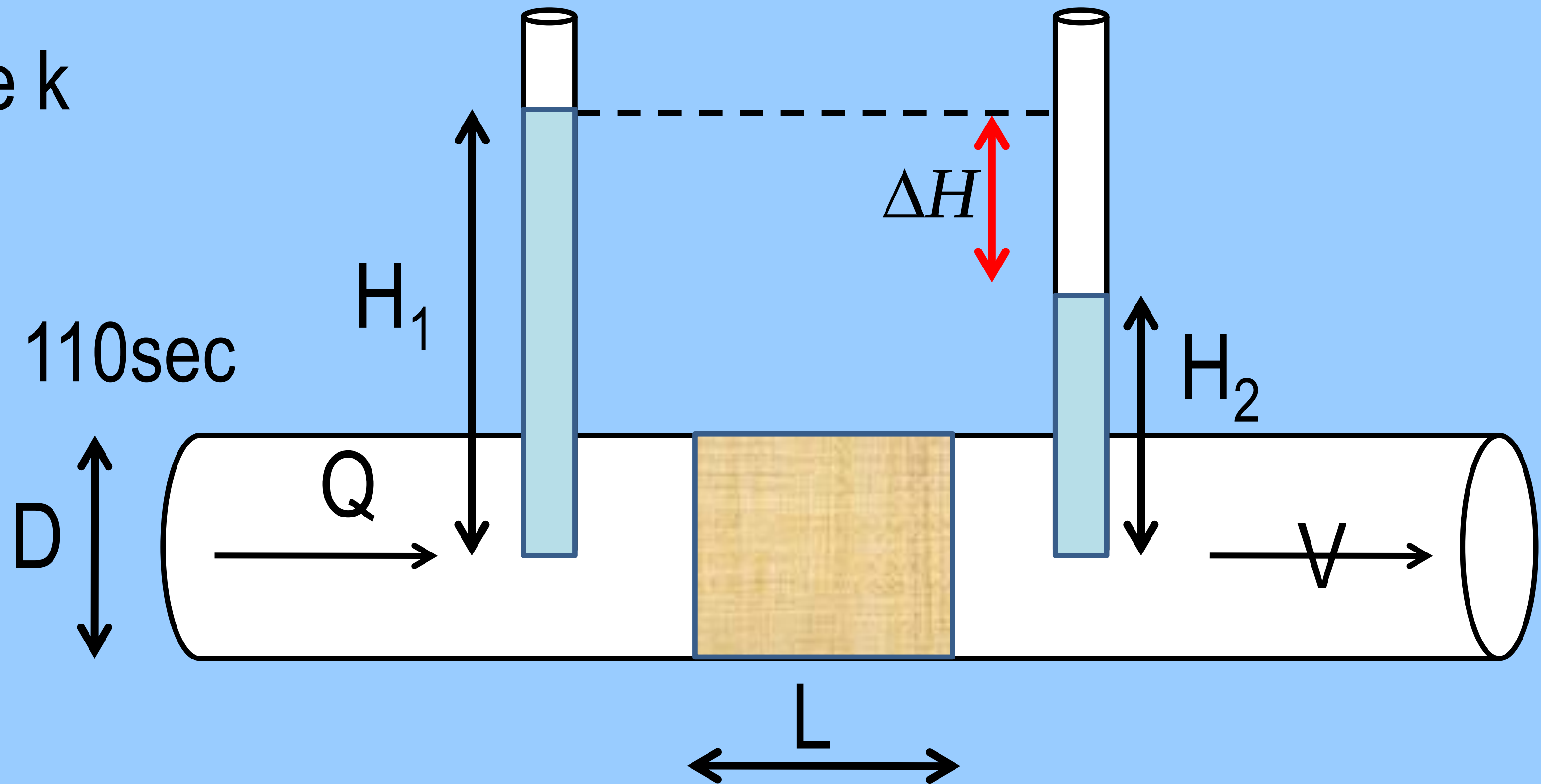
Q = 200ml in 110sec

$A = 30\ cm^2$

L = 25 cm

$H_1 = 70$ cm

$H_2 = 30$ cm



Homework Problem

- Determine h_2

Given:

$Q = 1.82\ cm^3/sec$

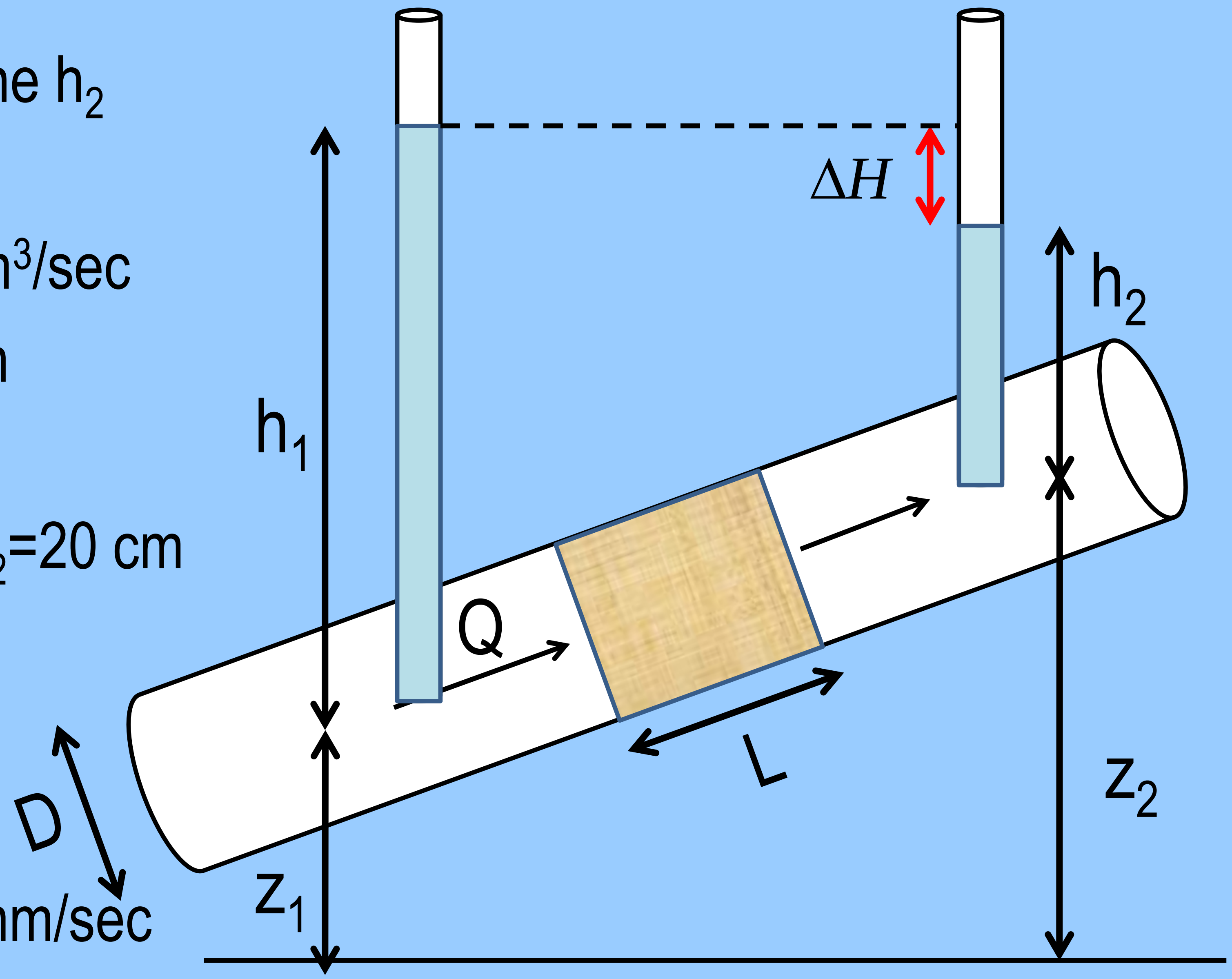
$D = 6.18\ cm$

$L = 25\ cm$

$h_2=30cm,\ z_2=20\ cm$

$z_1=10\ cm$

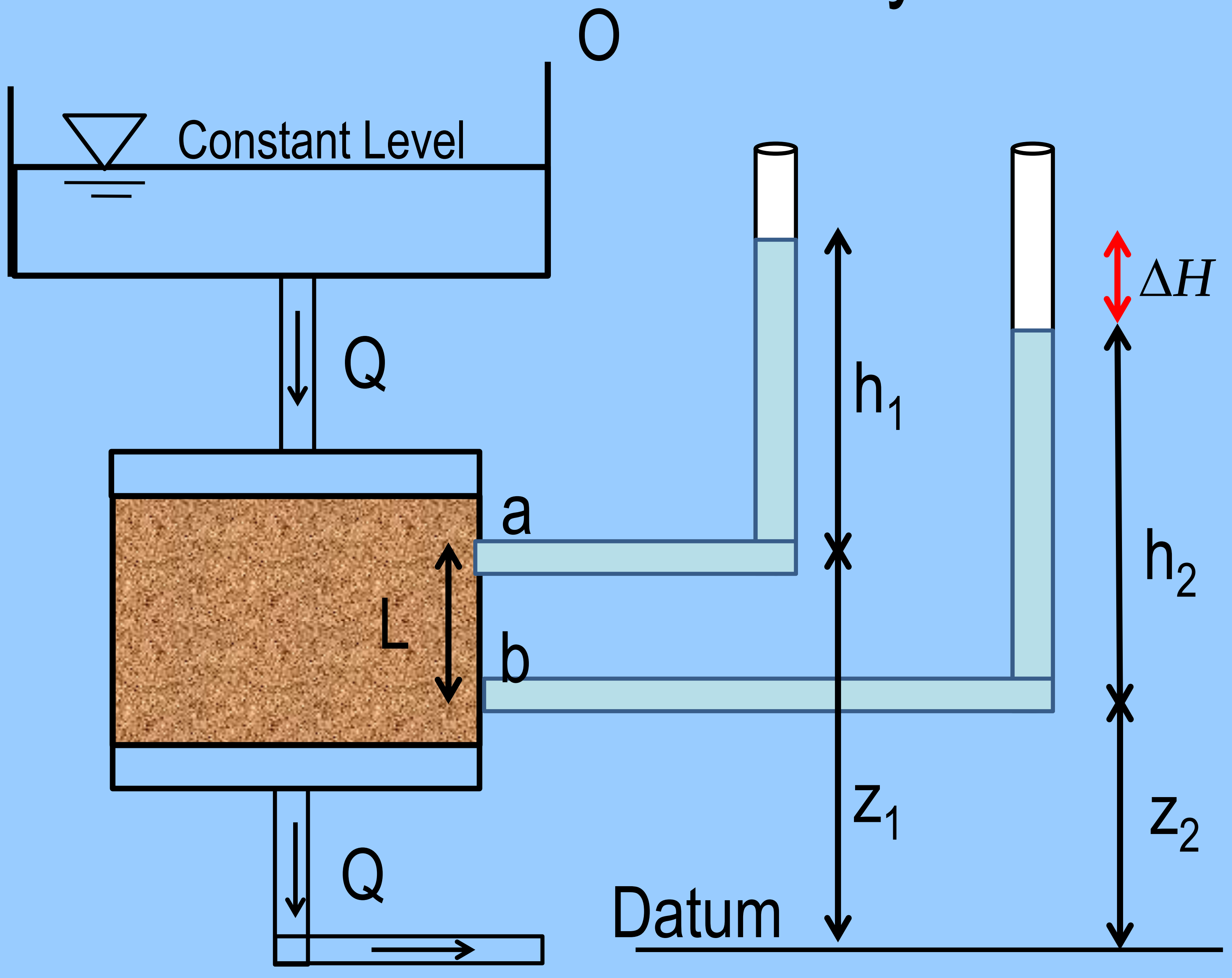
$K=0.1894\ mm/sec$



Methods to measure permeability

- Laboratory:
  - Constant Head Permeability Test
  - Variable Head Permeability Test
- Field
  - Unconfined Aquifer Test
  - Confined Aquifer Test

Constant Head Permeability Test



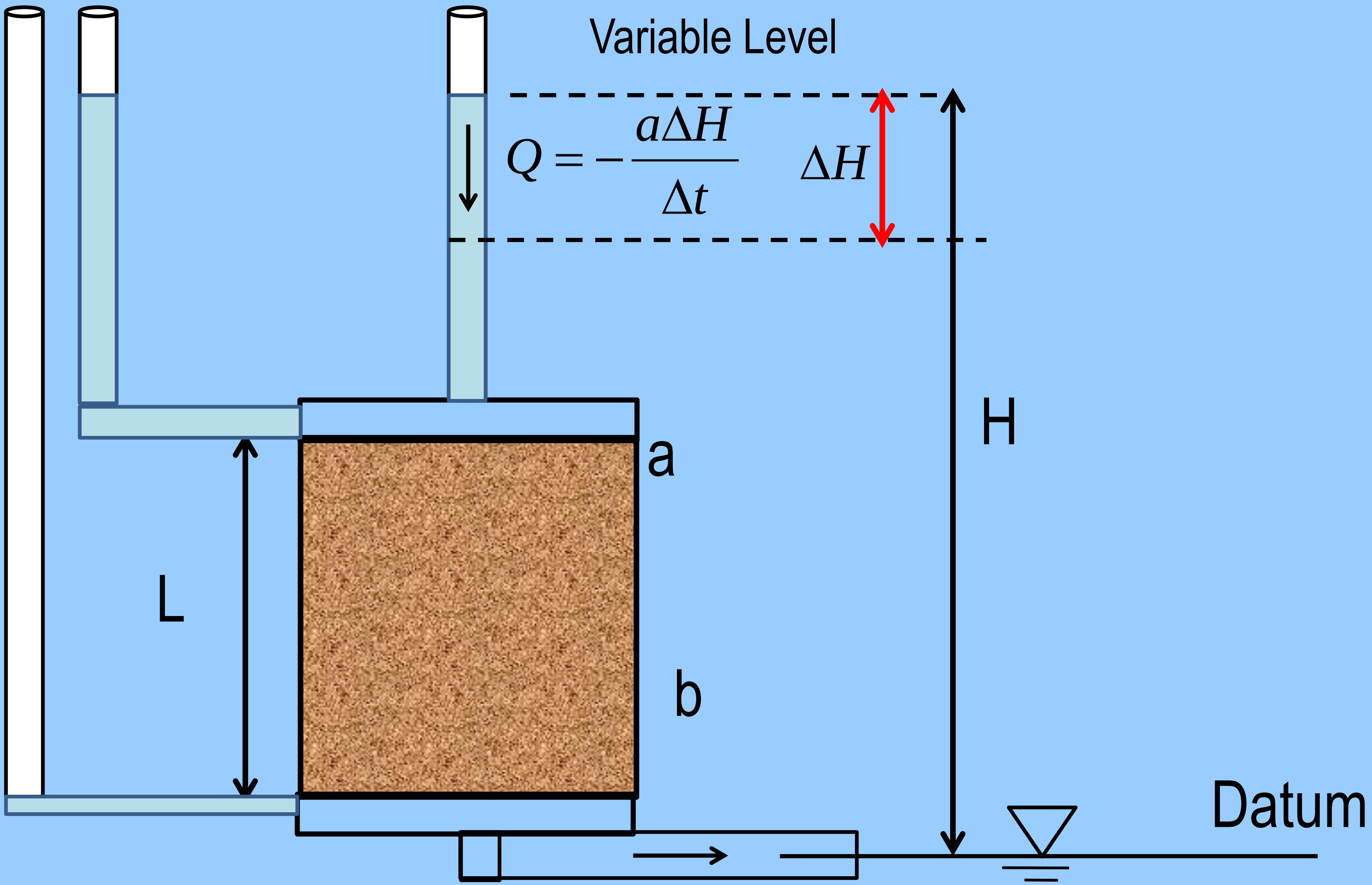
Constant Head Permeability Test

- Pervious, Coarse Grained Soils
- K- Darcy's Coefficient of Permeability, m/sec
- Q: Rate of Flow or Discharge, m^3/sec
- L : Length, m
- A : Area of Cross –Section, m^2

$$Q = k\left[\frac{\Delta H}{L}\right]A$$

$$k = \frac{QL}{\Delta H \times A}$$

Variable Head Permeability Test



Variable Head Permeability Test

- Fine Grained Soils
- K- Darcy's Coefficient of Permeability, m/sec
- h_1: Height in standpipe at time t_1
- h_2: Height in standpipe at time t_2
- L : Length, m
- a : Area of Cross –Section of Stand pipe, m^2

Variable Head Permeability Test

$$Q = KiA$$

$$-a\frac{dh}{dt} = k\frac{h}{L}A$$

$$-a\int_{h_1}^{h_2}\frac{dh}{h} = \frac{kA}{L}\int_{t_1}^{t_2}dt$$

$$a\ln\left(\frac{h_1}{h_2}\right) = \frac{kA}{L}(t_2 - t_1)$$

$$k = \frac{aL}{A(t_2 - t_1)}\ln\left(\frac{h_1}{h_2}\right)$$

$$k = \frac{2.303aL}{A(t_2 - t_1)}\log\left(\frac{h_1}{h_2}\right)$$

Problem

- In a falling head permeability test, head causing flow was initially 50 cm and it drops 2cm in 5 minutes. How much time is required for the head to fall to 25 cm.

Homework Problem

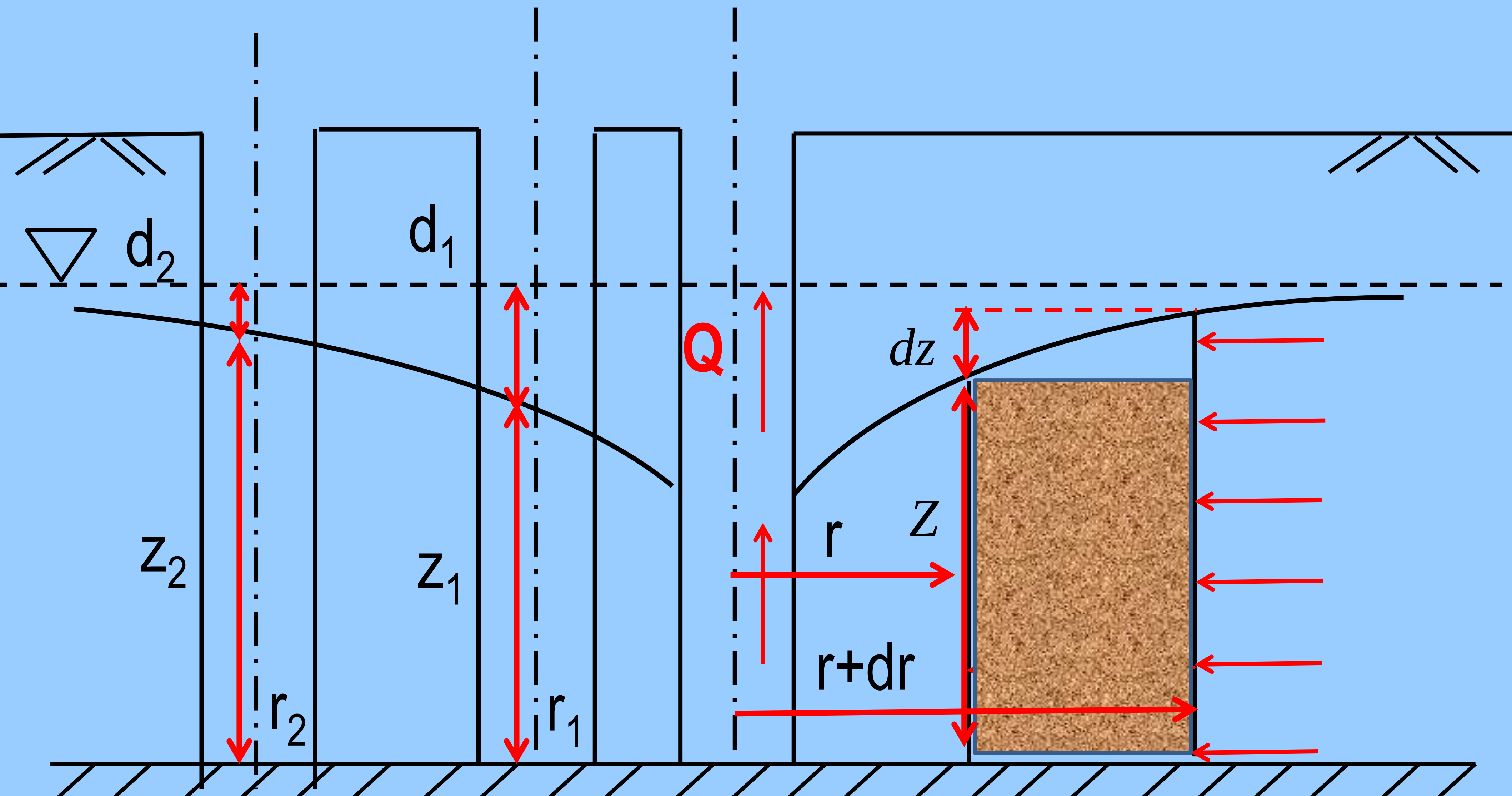
- A falling head permeability test was performed on a sand sample and the following data are recorded:
- Cross-sectional area of permeameter = 100 cm^2; length of the soil sample =15cm, area of stand pipe = 1cm^2, time taken for the head to fall from 150 cm to 50 cm = 8 minutes, temperature of water = 25^oC; dry mass of the soil specimen = 2.2 kg and its Gs=2.68.
- Calculate its void ratio e_{25}.
- Compute the coefficient of permeability of soil.
- What is the coefficient of permeability of the soil at void ratio of e_{20} =0.70 and standard temperature of 20^oC?

Note: γ_w @25 ^oC = γ_w @20 ^oC,

μ_w @25^oC =890/10^6 N sec/m^2, μ_w @20^oC =1002/10^6 N sec/m^2.

$$k \propto \frac{e^3}{1+e}\frac{\gamma_w}{\mu_w}$$

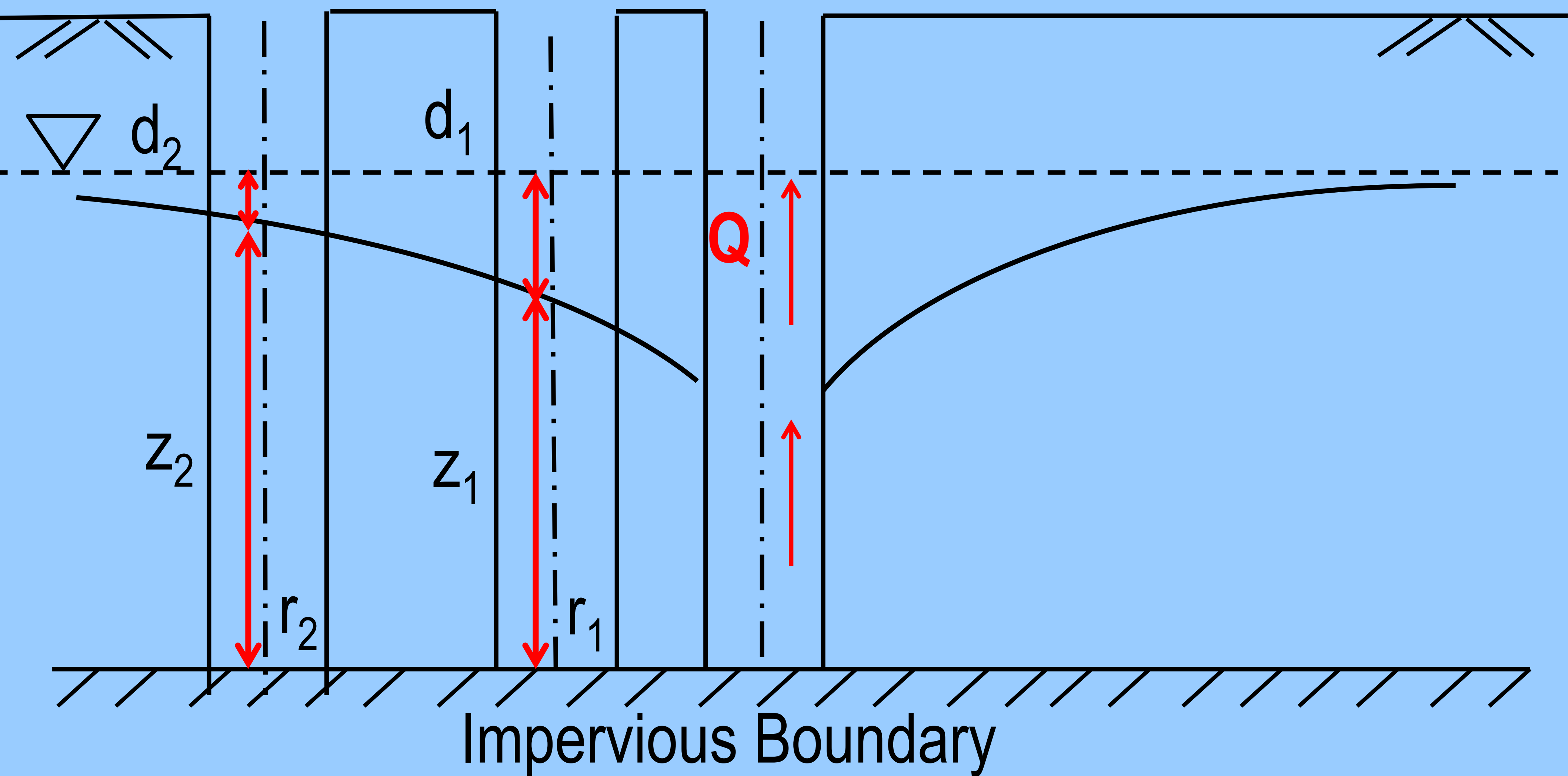
Field Permeability Tests: Unconfined Aquifer



Impervious Boundary

Unconfined Aquifer

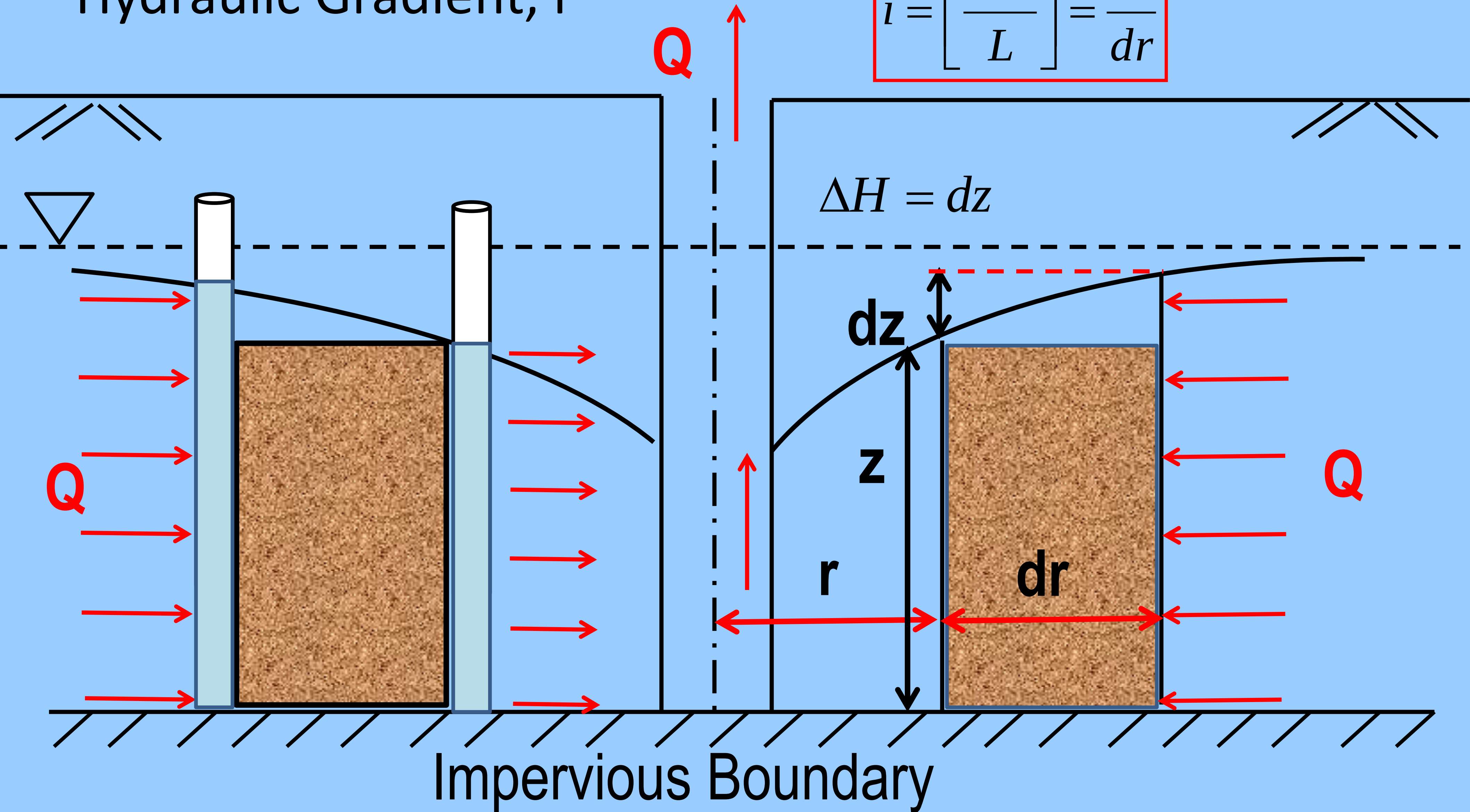
Observation Wells dug at r_1 and r_2.Water pumped through main well at Q m^3/sec



Field Permeability Tests: Unconfined Aquifer

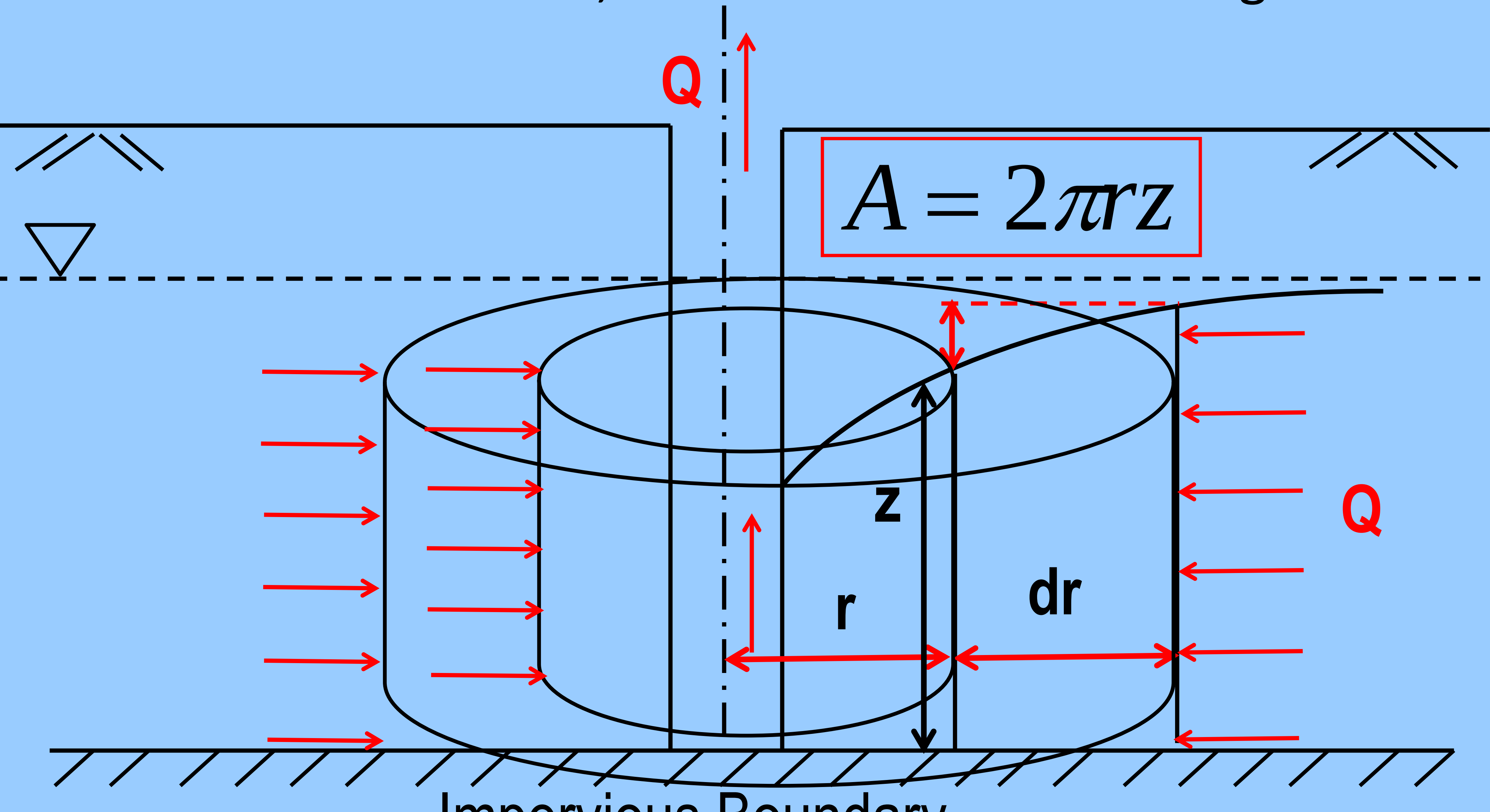
Hydraulic Gradient, i

$$i = \left[\frac{\Delta H}{L}\right] = \frac{dz}{dr}$$



Unconfined Aquifer

Cross-sectional Area, A = Cirucumference x Height



Unconfined Aquifer Test

$$Q = KiA$$

$$= K\frac{dz}{dr}2\pi rz$$

$$\frac{Q}{2\pi}\frac{dr}{r} = K \cdot zdz$$

$$\frac{Q}{2\pi}\int_{r_1}^{r_2}\frac{dr}{r} = K\int_{z_1}^{z_2} zdz$$

$$\frac{Q}{2\pi}\ln\left(\frac{r_2}{r_1}\right) = K\frac{(z_2^2 - z_1^2)}{2}$$

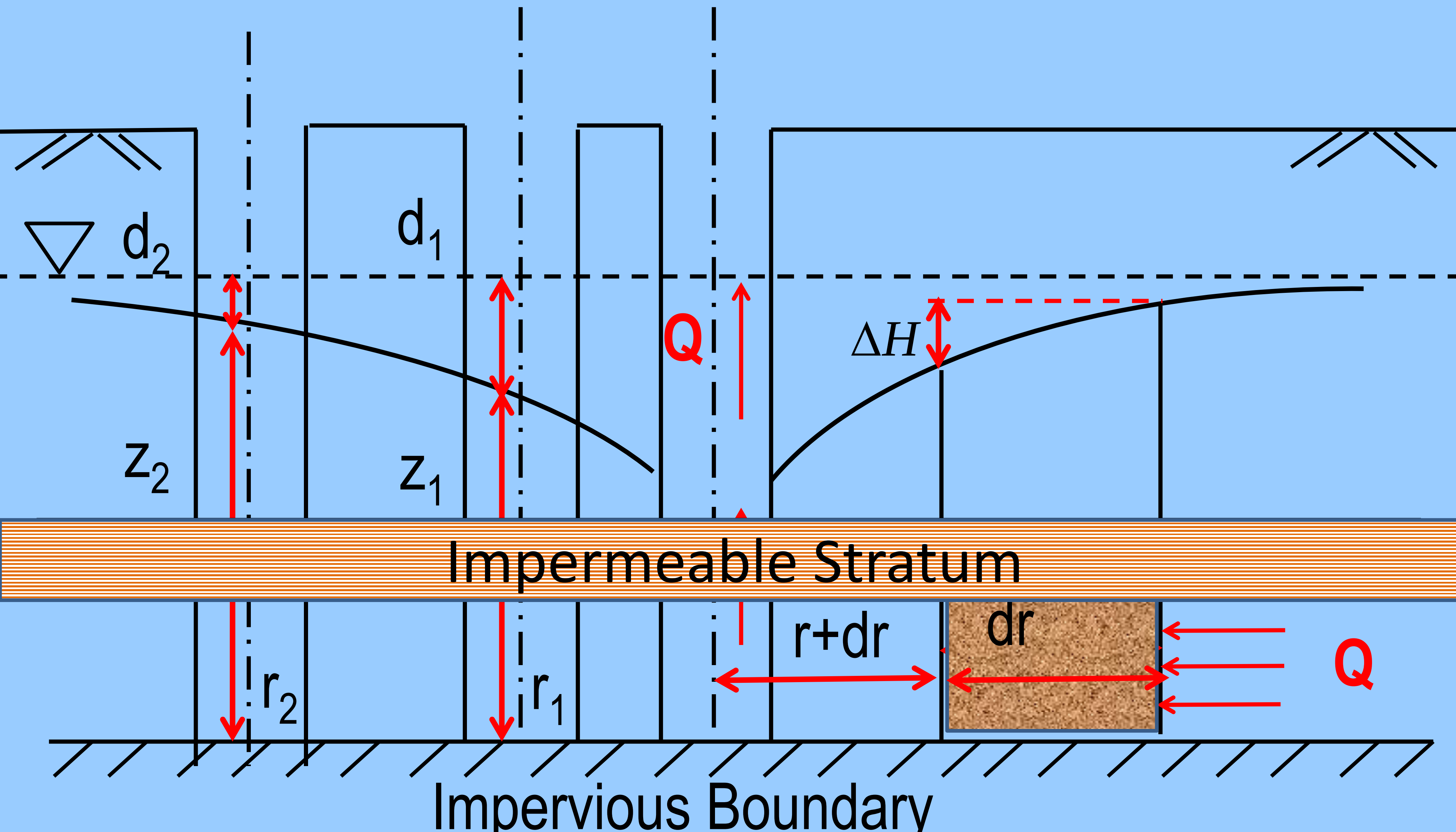
$$k = \frac{Q}{\pi}\frac{\ln\left(\frac{r_2}{r_1}\right)}{(Z_2^2 - Z_1^2)}$$

$$k = \frac{Q}{\pi}\frac{2.303\log\left(\frac{r_2}{r_1}\right)}{(Z_2^2 - Z_1^2)}$$

Homework Problem

- For a field pumping test, a well was sunk through a horizontal stratum of sand 14.5 m thick and underlain by a impermeable clay stratum. Two observation wells were sunk at horizontal distance of 16m and 34 m respectively from the pumping well. The initial position of the water table was 2.2 m below ground level. At steady state pumping of 925 litres/minute, the drawdowns (depths measured from water table) in the observation wells were found to be 2.45 m and 1.20 m respectively. Calculate the coefficient of horizontal permeability of the sand. Note: 1000 litre = $1 m^3$.

Field Permeability Tests: Confined Aquifer



Confined Aquifer Test

$$Q = KiA$$

$$= K\frac{dz}{dr}2\pi rH$$

$$\frac{Q}{2\pi}\frac{dr}{r} = K \cdot Hdz$$

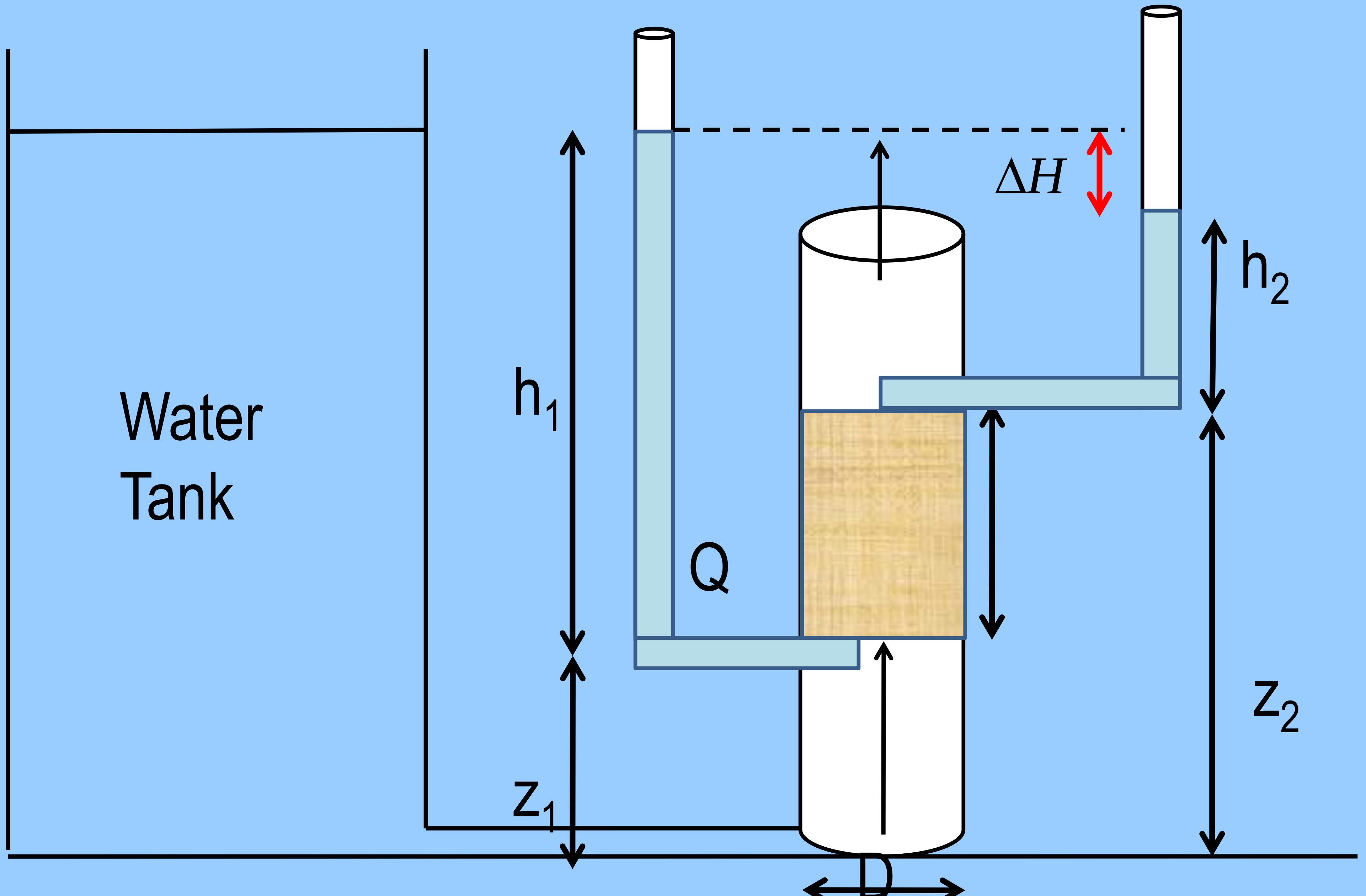
$$\frac{Q}{2\pi}\int_{r_1}^{r_2}\frac{dr}{r} = K \cdot H\int_{z_1}^{z_2} dz$$

$$\frac{Q}{2\pi}\ln\left(\frac{r_2}{r_1}\right) = KH(z_2 - z_1)$$

$$k = \frac{Q}{\pi}\frac{\ln\left(\frac{r_2}{r_1}\right)}{2H(Z_2 - Z_1)}$$

$$k = \frac{Q}{\pi}\frac{2.303\log\left(\frac{r_2}{r_1}\right)}{2H(Z_2 - Z_1)}$$

Flow Upward



Lecture 10: Seepage, Flow Nets

Dr. Mahendra Gattu

CLE 205 Soil Mechanics

Today's Agenda

- Problem 1 and Problem 2
- Quick Sand Condition
- Laplace equation for two-dimensional flow
- Flow Net
  - Properties
  - Use
- Problem

Problem

- Determine the pressure head, elevation head, total head and head loss at the entering end, exit end and point Xin the sample.

Lecture 11: Consolidation

Dr. Mahendra Gattu

CLE 205 Soil Mechanics

Today’s Agenda

- Consolidation review
- Total Settlement, Immediate, Primary, Secondary
- Relationship between void ratio and settlement
- Problem

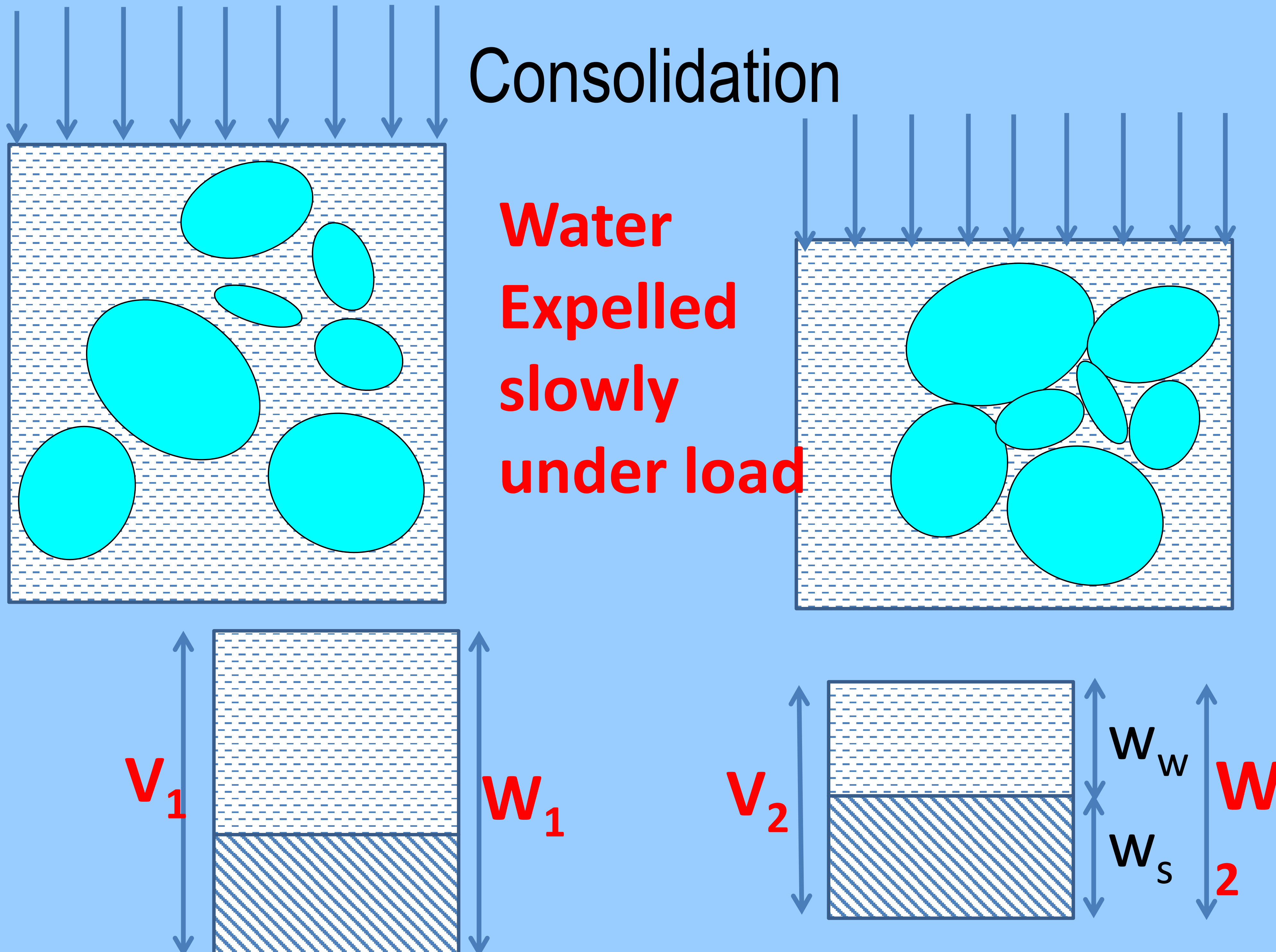
Today's Agenda

- Definitions
  - Compression Index
  - Coefficient of Compressibility
  - Coefficient of Volume Compressibility
- Role of Stress History
  - Normal Consolidation
  - Over consolidation Stress
  - Over-Consolidation Ratio
- Problem

Consolidation Review

- Time Dependent Phenomenon
- Amount of Settlement = function (Time of Settlement)
- Our approach
  - Total/ Final Settlement (Independent of Time)
  - Intermediate Settlement (Function of Time): Pore pressure dissipation, Mechanics of Consolidation, Terzaghi's 1-D consolidation Theory

Consolidation



Amount of Settlement

- Total Settlement = Immediate Settlement+ Primary Consolidation Settlement + Secondary Settlment

$$S_t = S_i + S_p + S_s$$

- Immediate Settlment: Negligible flow of water out of soil mass and volume remains same, Occurs almost immediately after the load is imposed

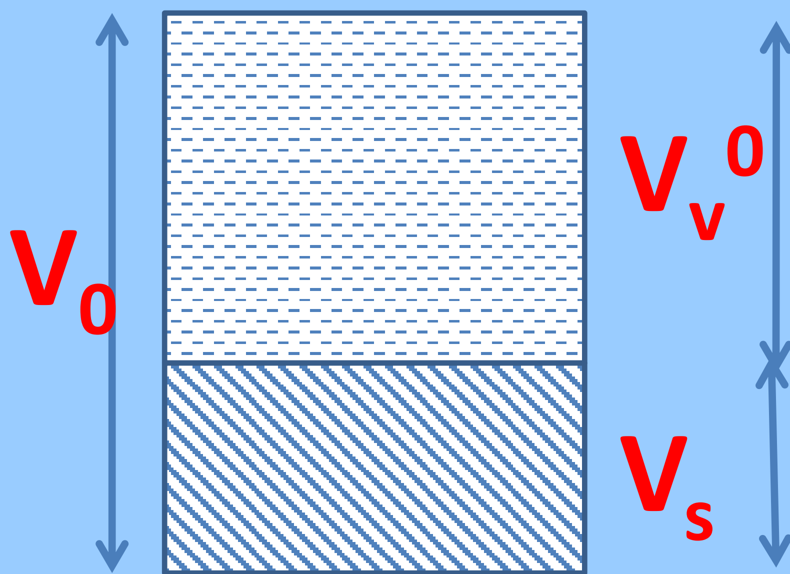
•Primary Consolidation

- Primary Consolidation: Squeezing out of pore water from a loaded saturated soil causing a time dependent decrease in volume
- Rate of Flow is controlled by permeability, compressibility, pore pressure
- With passage of time, as pore pressures u dissipate, the rate of flow will decrease and will eventually stop leading to constant effective stress, signifying the end of primary consolidation.

Secondary Consolidation

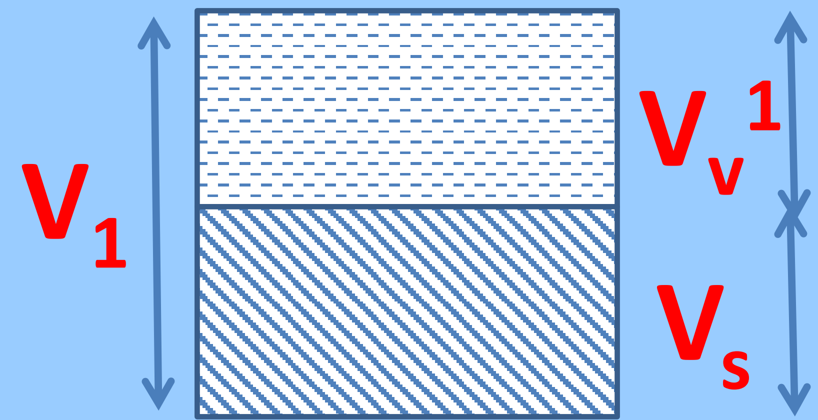
- The time dependent settlement exhibited by soils post primary consolidation at constant effective stress.

Relationship between void ratio and primary consolidation settlement



$$e_0 = \frac{V_V}{V_s}$$

$$e_1 = \frac{(V_V)^1}{V_s}$$



$$V_S^o = \frac{1}{1+e_o} V$$

$$V_W^0 = eV_S = \frac{e_0}{1+e_0} V$$

$$V_S^1 = V_S^0$$

$$V_W^1 = e_1 V_S^0 = \frac{e_1}{1+e_0} V_0$$

Relationship between void ratio and primary consolidation settlement

$$\Delta V = V_1 - V_0$$

$$= V_V^1 - V_V^0$$

$$= V_W^1 - V_W^0$$

$$= \frac{e_1 - e_0}{1 + e_0} V$$

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

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

$$\Delta H = \frac{\Delta e}{1 + e_0} H$$

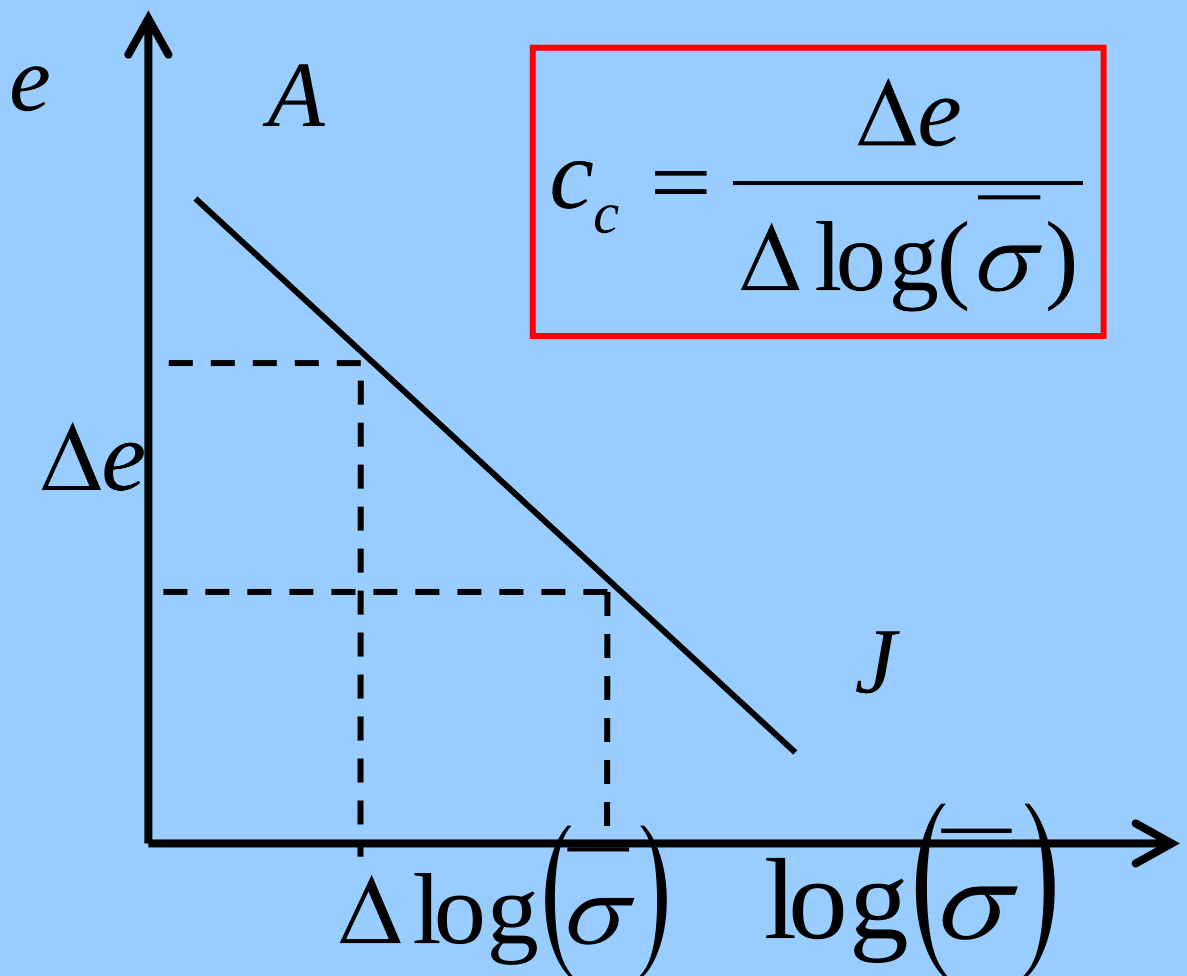
Problem

A compressible stratum is 6m thick and its void ratio is 1.70. If the final void ratio after the construction of the building is expected to be 1.61, what will be the probable ultimate settlement of the building?

$\bar{\sigma}_2$

Definitions

Compression Index

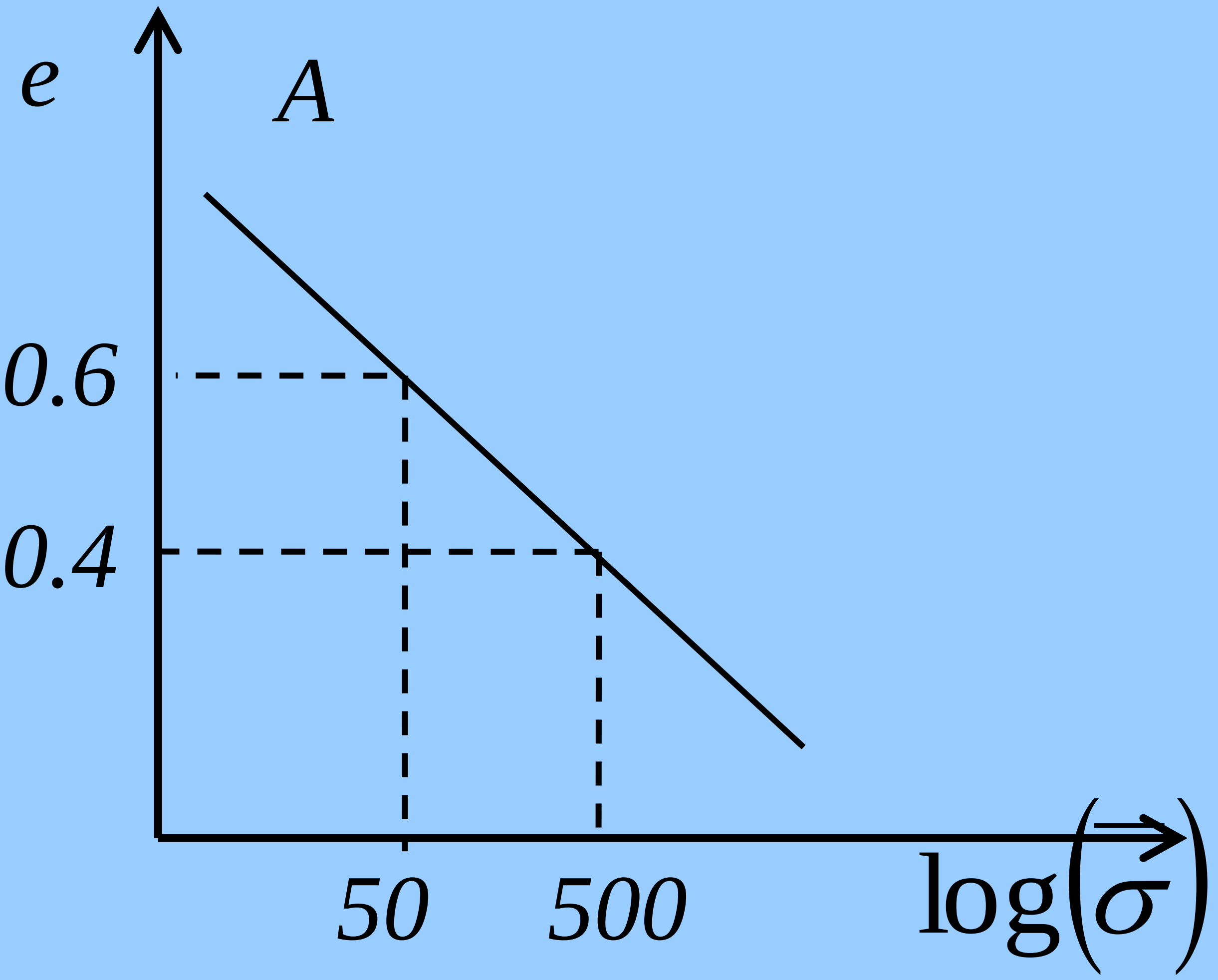


$$c_c = \frac{\Delta e}{\Delta \log(\bar{\sigma})}$$

$$c_c = \frac{e_2 - e_1}{\log\left(\dfrac{\bar{\sigma}_2}{\bar{\sigma}_1}\right)}$$

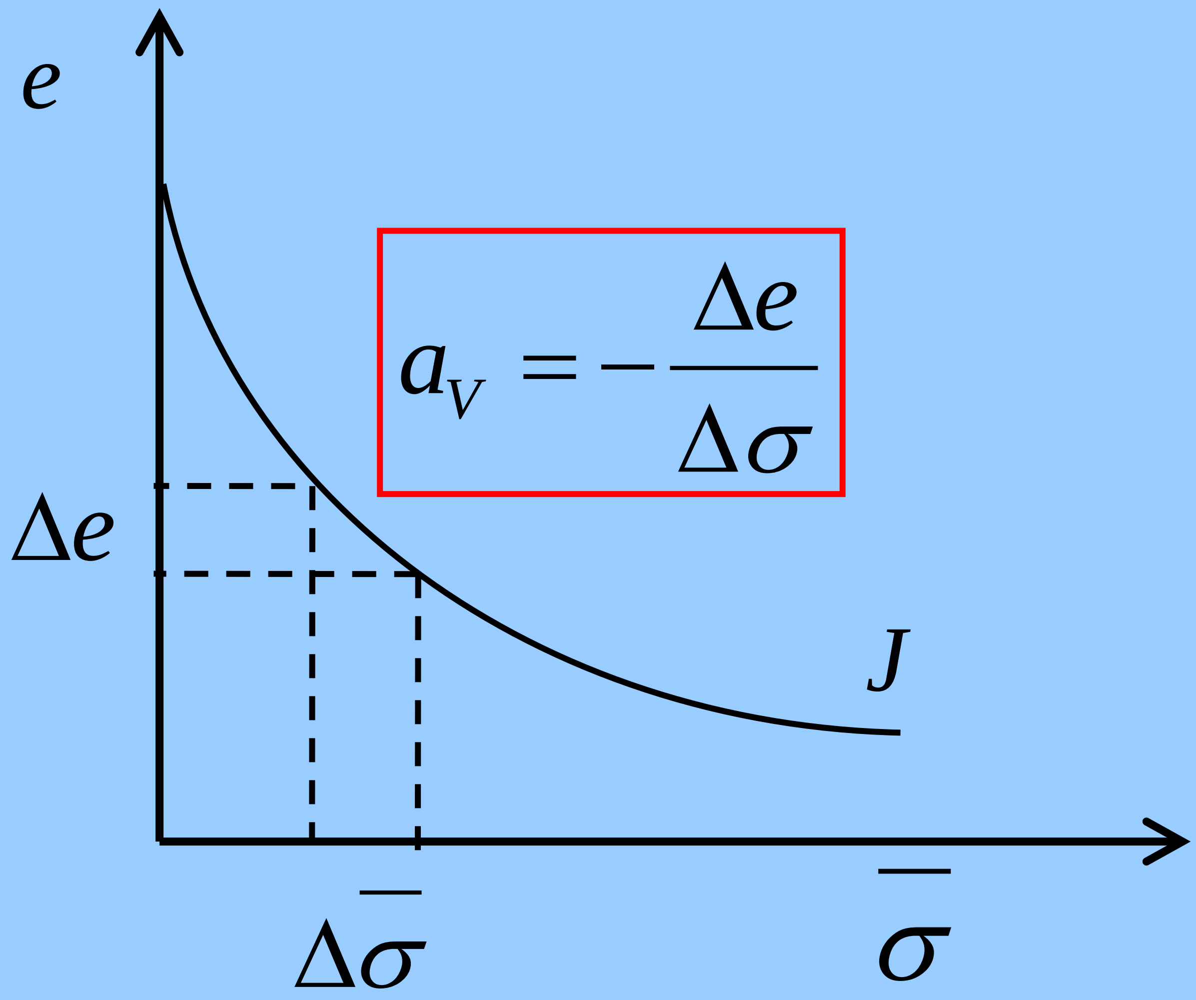
Problem

Calculate compression index.



Definitions

Coefficient of Compressibility



$$a_V = -\frac{\Delta e}{\Delta \sigma}$$

Definitions

Coefficient of Volume Compressibility

$$m_V = -\frac{\Delta V}{V} \Big/ \Delta\overline{\sigma}$$

$$m_V = -\frac{\Delta e}{1+e_0} \Big/ \Delta\overline{\sigma} = \frac{a_V}{1+e_0}$$

Problem

A clay soils tested in a consolidometer showed a decrease in void ratio from 1.20 to 1.10 when pressure was increased from 0.25 to 0.50 kgf/cm^2.

a) Calculate the coefficient of compressibility and coefficient of volume compressibility.

b) If the sample tested at a site was taken from a clay layer 3m in thickness, determine the consolidation settlement resulting from the stress increment.

Definitions

Preconsolidation stress is the maximum stress the soil has ever experienced.

$$\bar{\sigma}_{pre} = \bar{\sigma}_{\max}$$

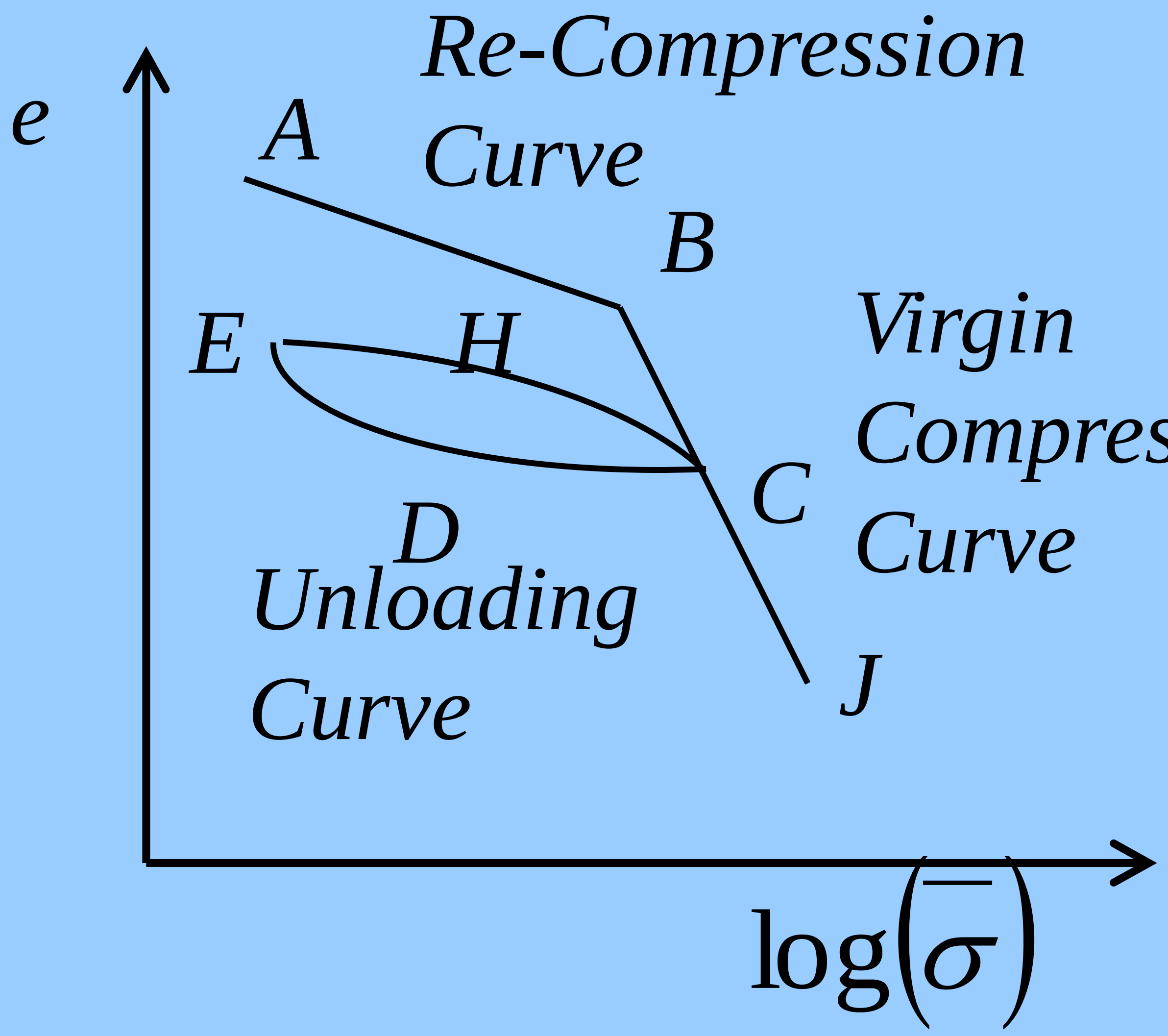
A soil is said to be <u>normally consolidated</u> when the effective stress $\bar{\sigma}$ is the maximum it has ever experienced in its stress history i.e.

$$\bar{\sigma}_{current} = \bar{\sigma}_{pre}$$

A soil is said to be <u>over-consolidated</u> if the current effective stress is less than the preconsolidation stress i.e

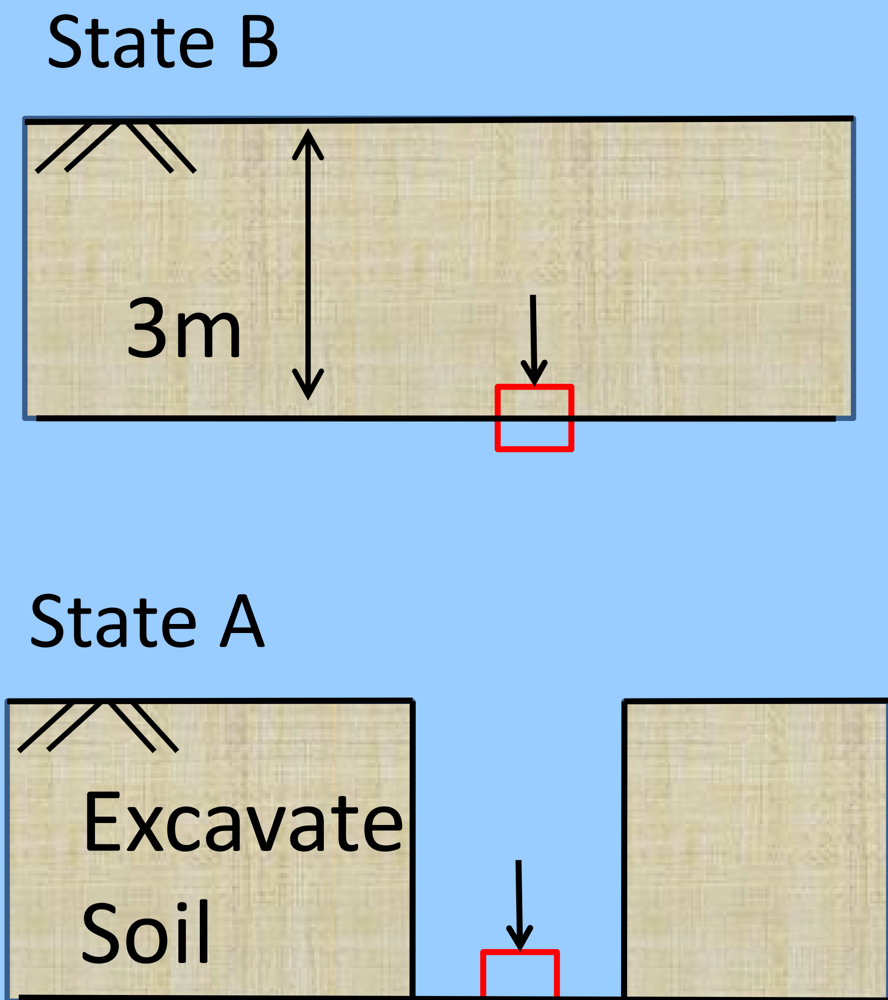
$$\bar{\sigma}_{current} < \bar{\sigma}_{pre}$$

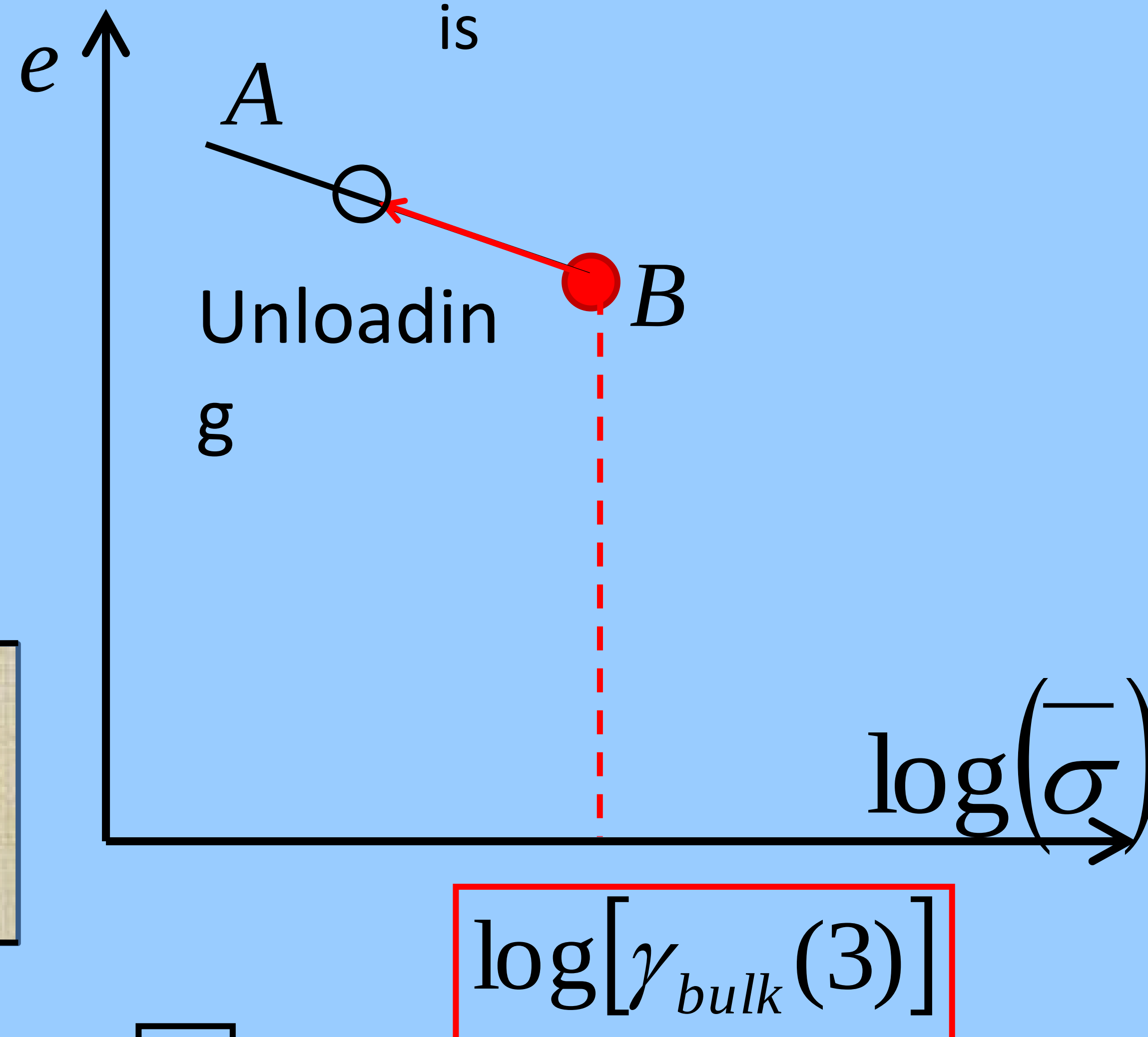
Role of Stress History



Role of Stress History

$\overline{\sigma}_{pre}$ ● Maximum Stress $[\gamma_{bulk}(3)]$ experienced by Soil is



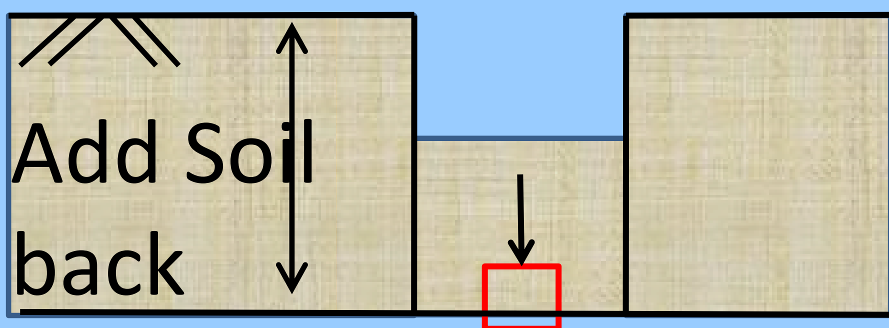


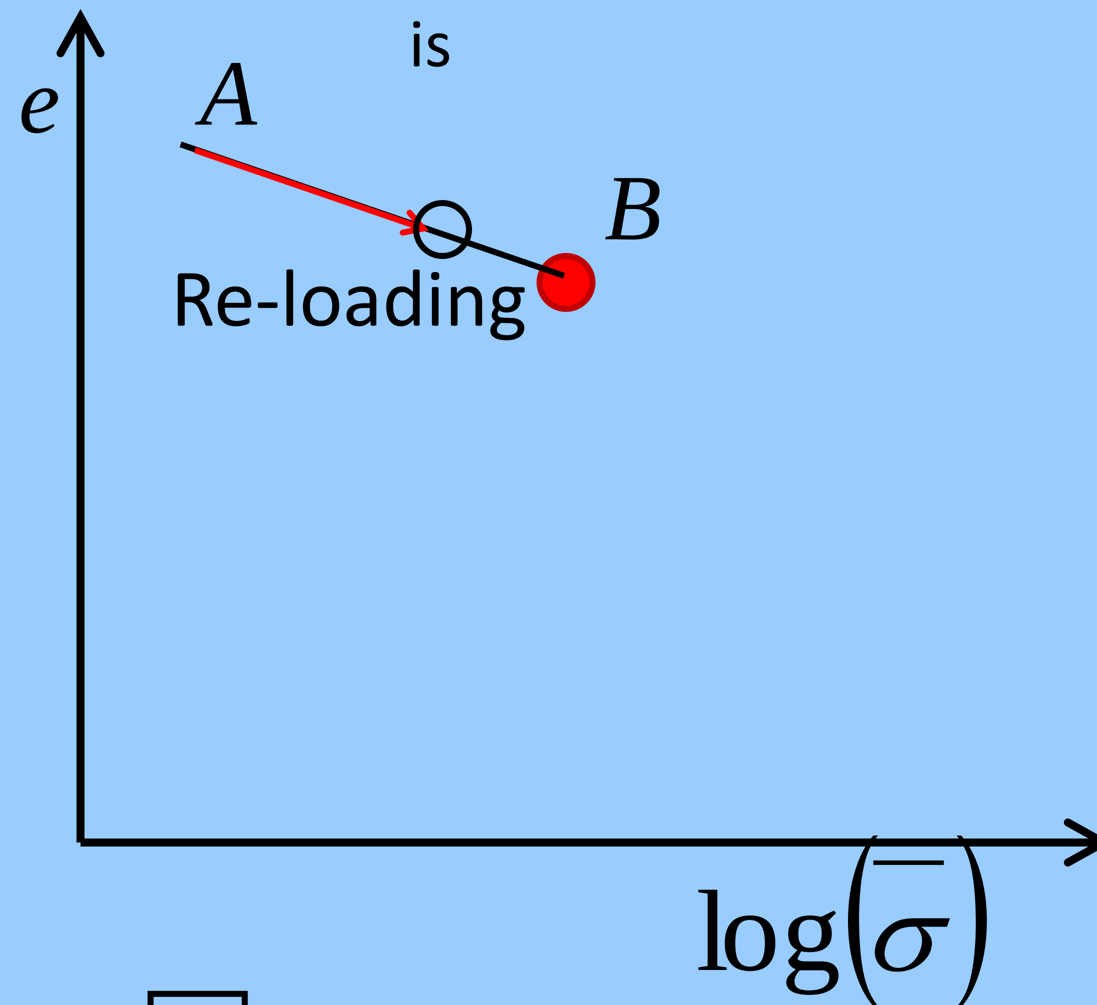
$\overline{\sigma}$ ○ Current Stress

Role of Stress History

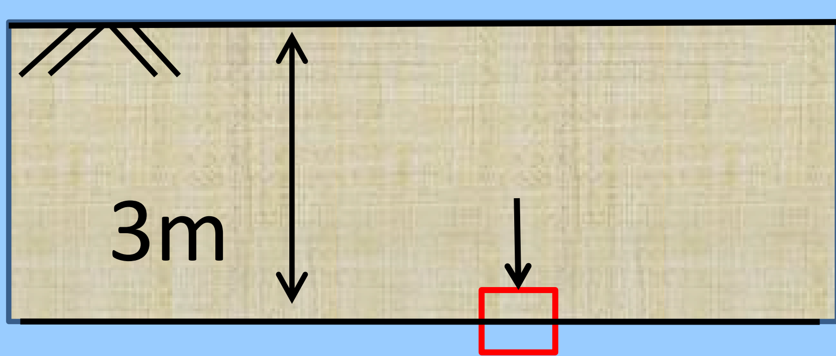
$\overline{\sigma}_{pre}$ ● Maximum Stress $[\gamma_{bulk}(3)]$ experienced by Soil is

State A





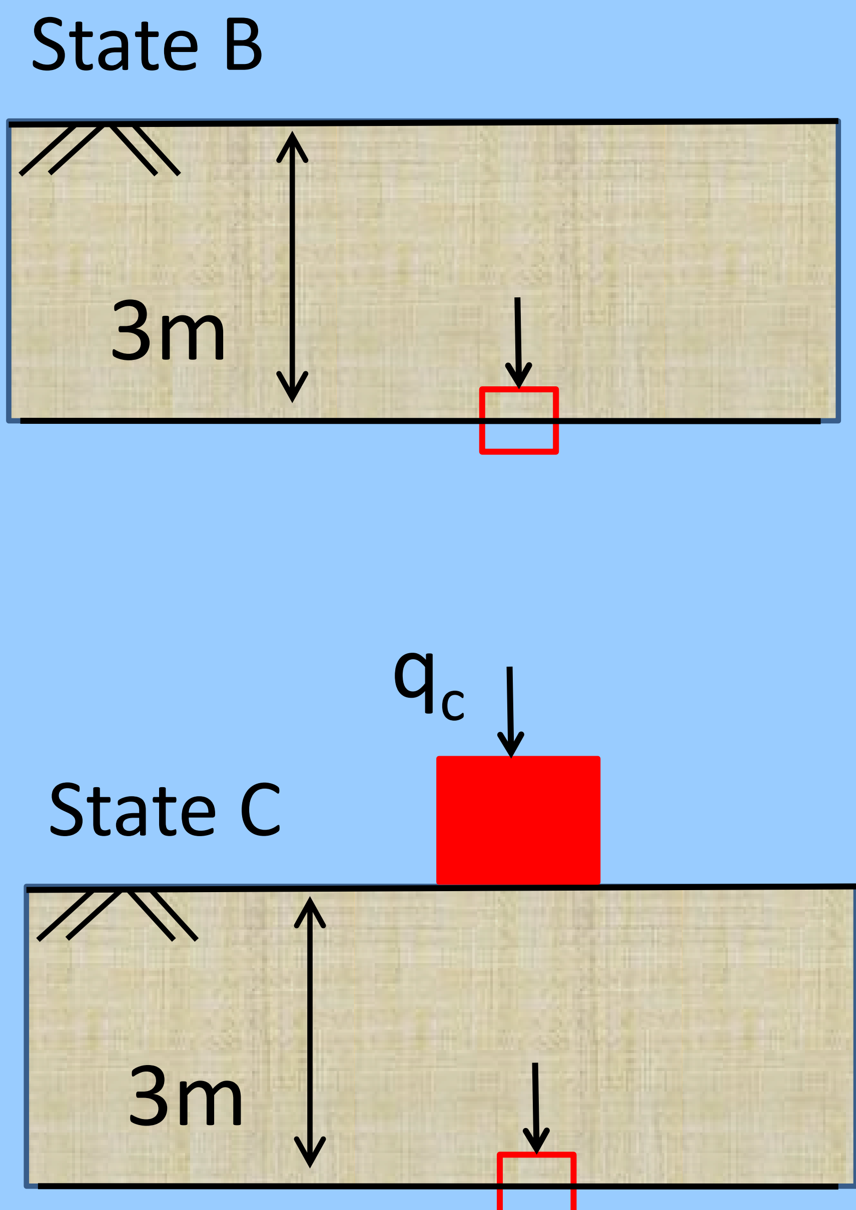
State B

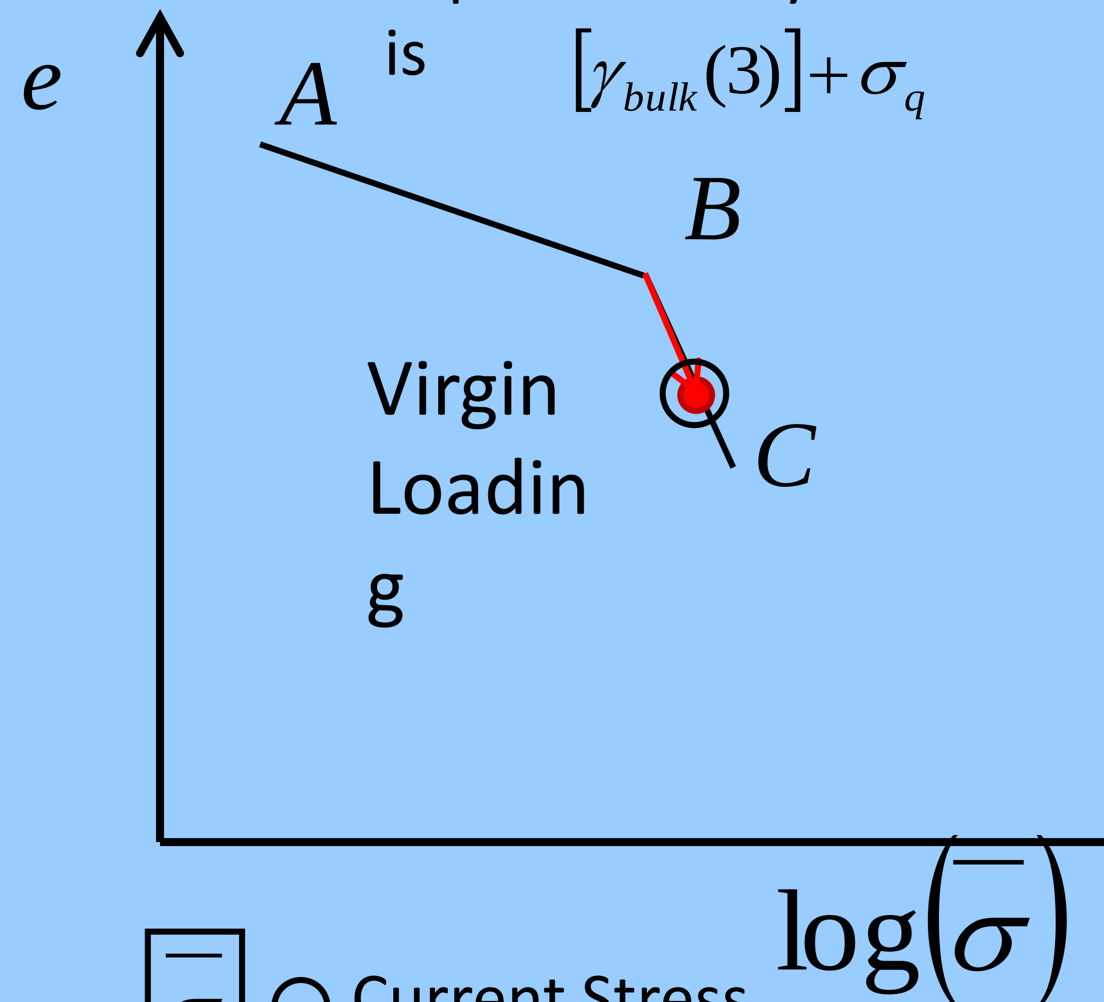


$\overline{\sigma}$ ○ Current Stress

Role of Stress History

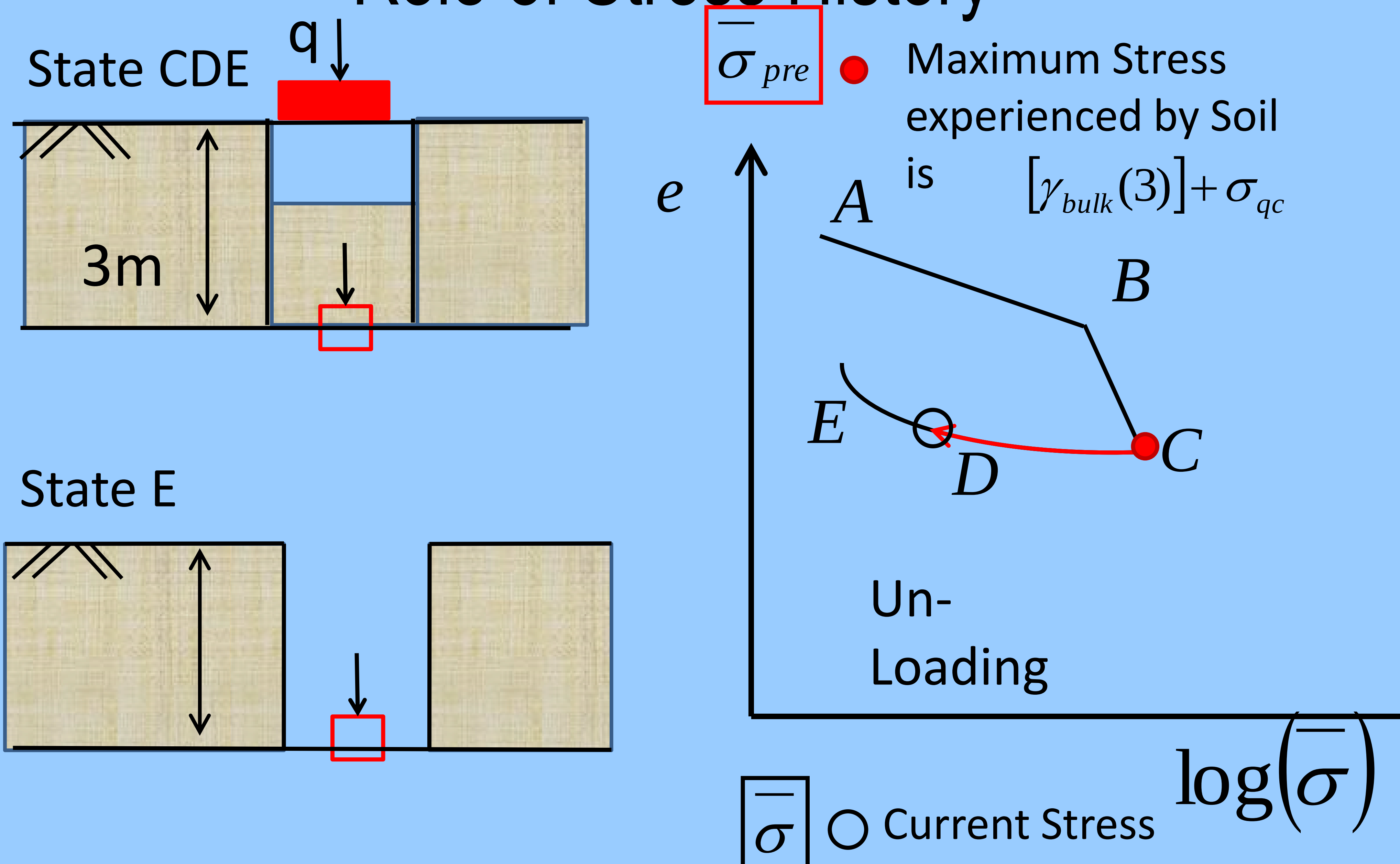
$\overline{\sigma}_{pre}$ ● Maximum Stress experienced by Soil is $[\gamma_{bulk}(3)]+\sigma_q$



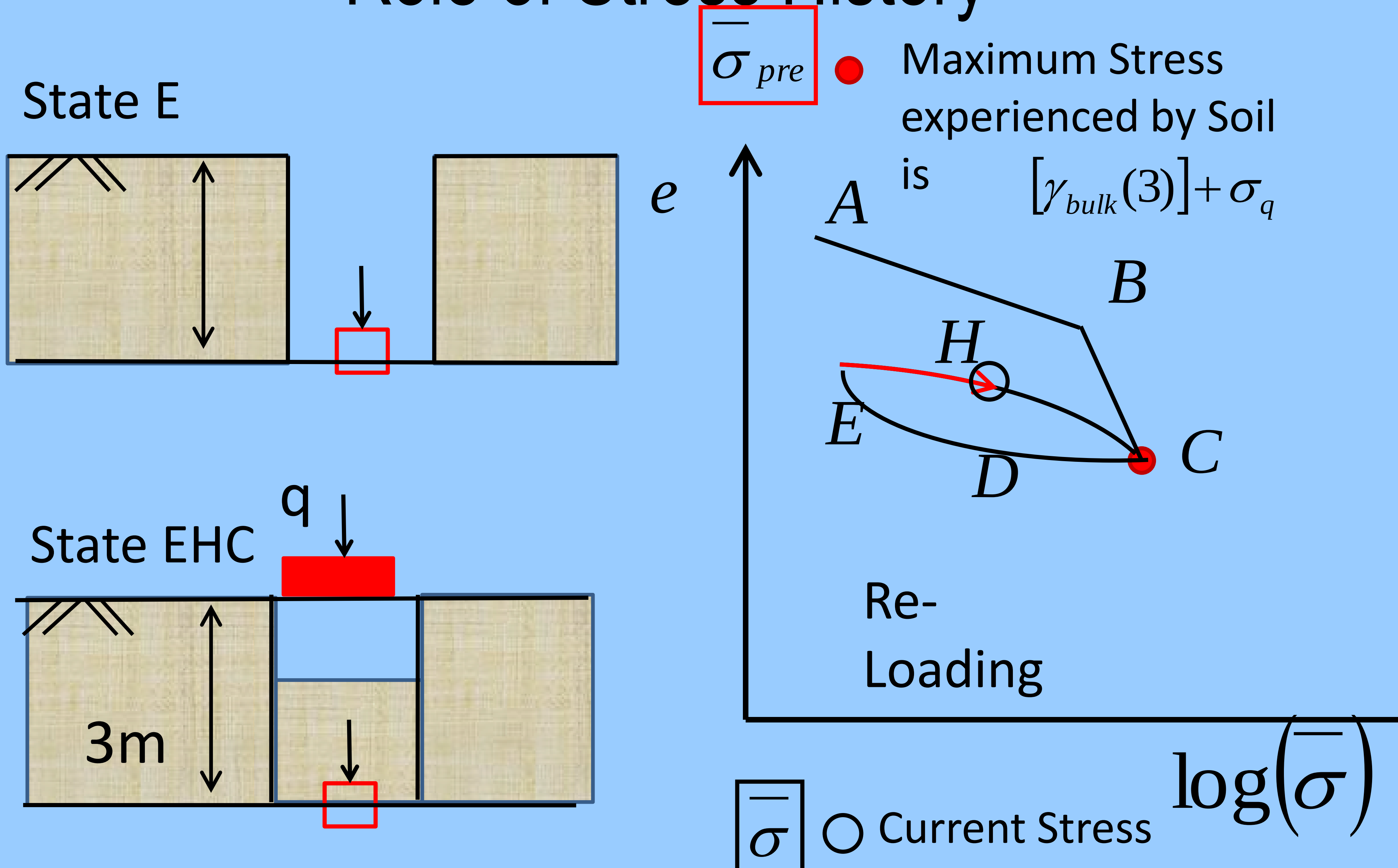


$\overline{\sigma}$ ○ Current Stress

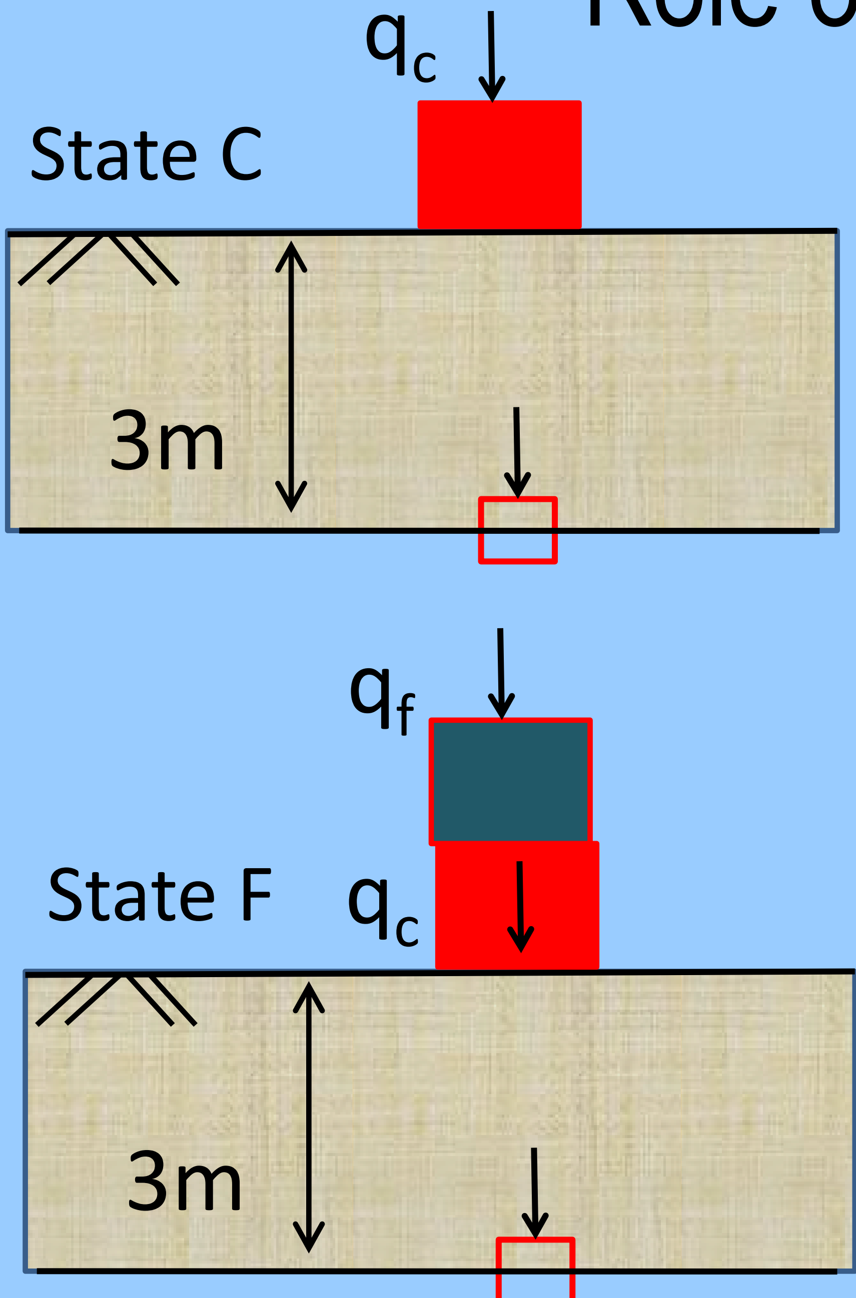
Role of Stress History



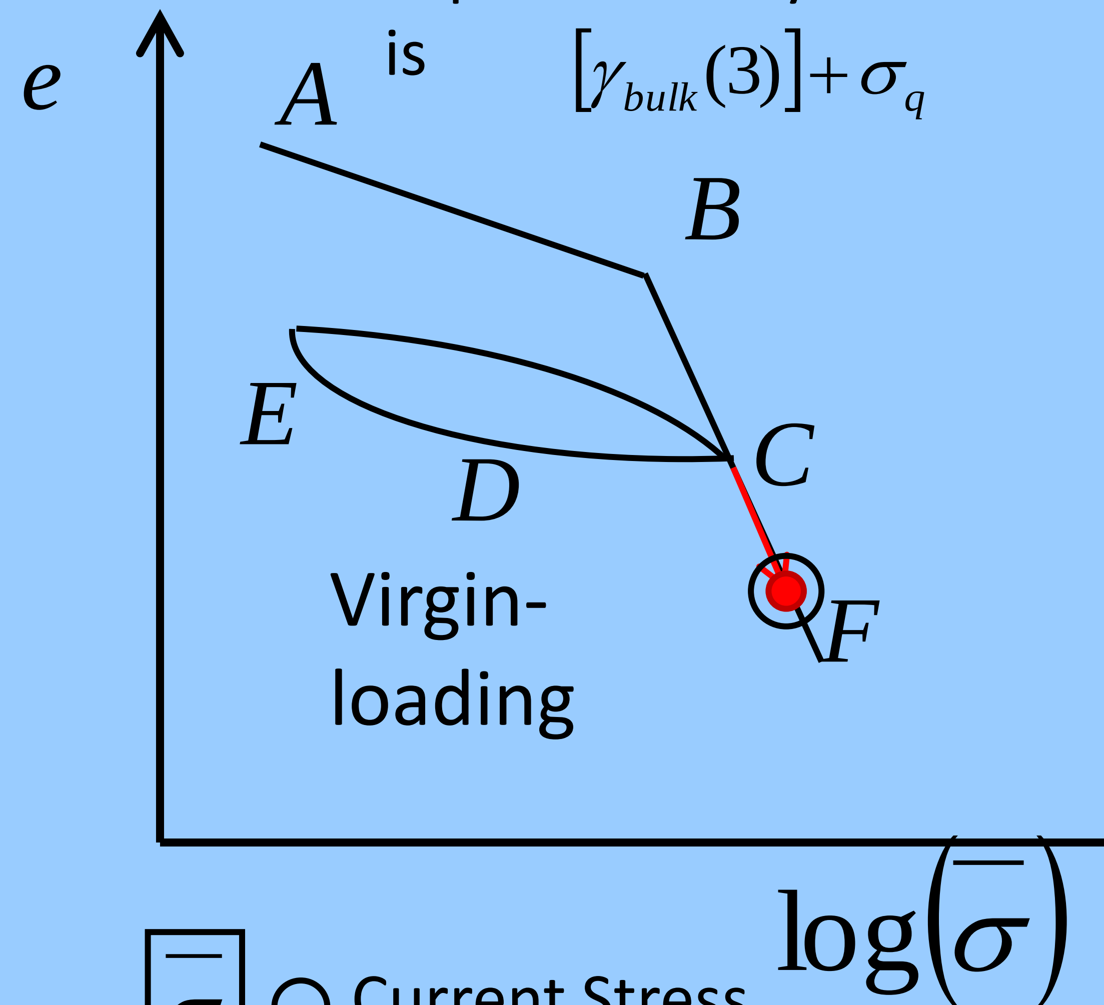
Role of Stress History



Role of Stress History



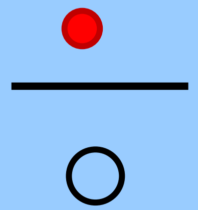
$\overline{\sigma}_{pre}$ ● Maximum Stress experienced by Soil is $\left[\gamma_{bulk}(3)\right]+\sigma_q$

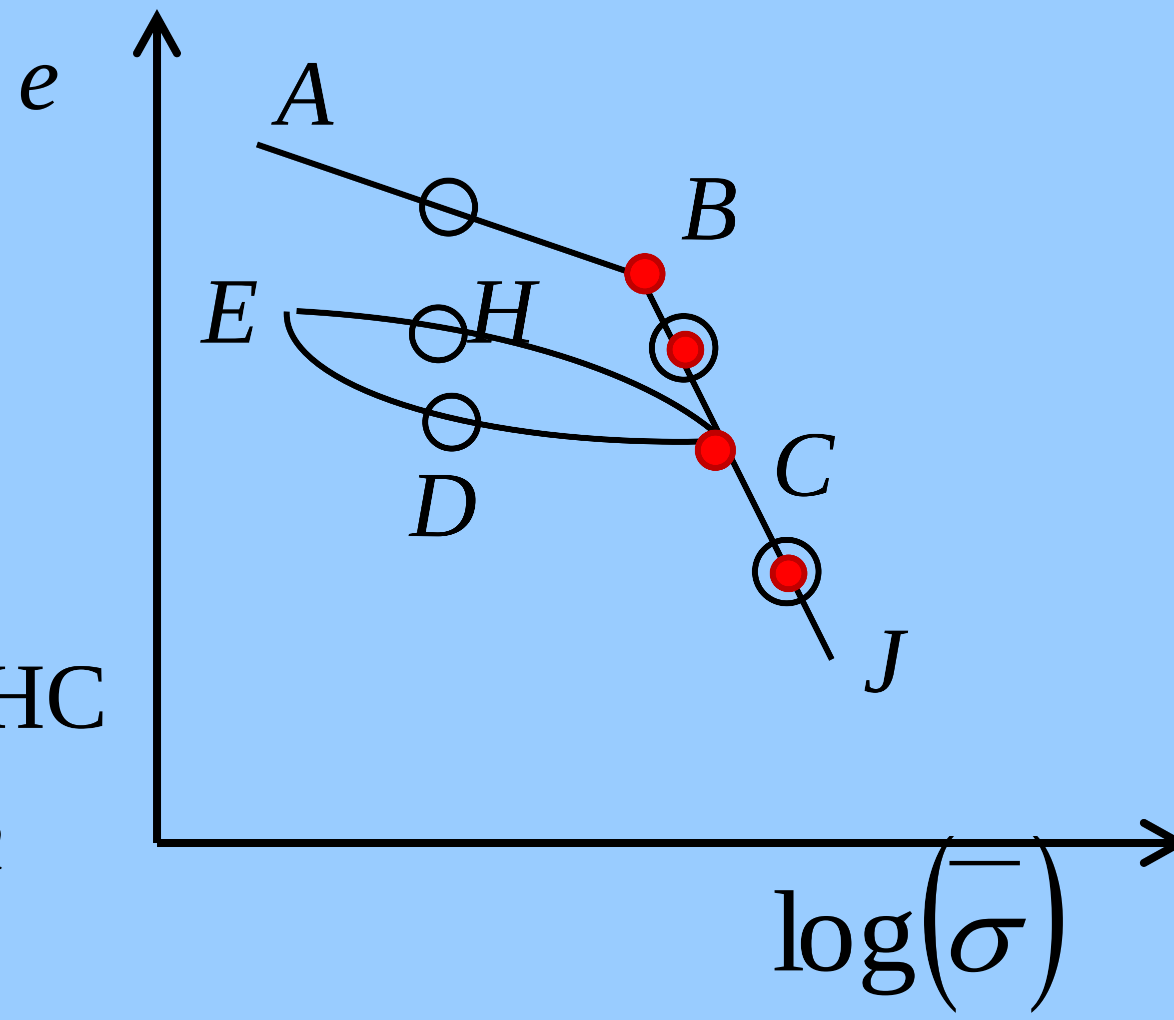


$\overline{\sigma}$ ○ Current Stress

Over-Consolidation Ratio, OCR

$$OCR = \frac{\bar{\sigma}_{pre}}{\bar{\sigma}_{current}}$$





Over Consolidation

OCR > 1, AB, CDE, EHC

Normal Consolidation

OCR = 1, BC and CJ

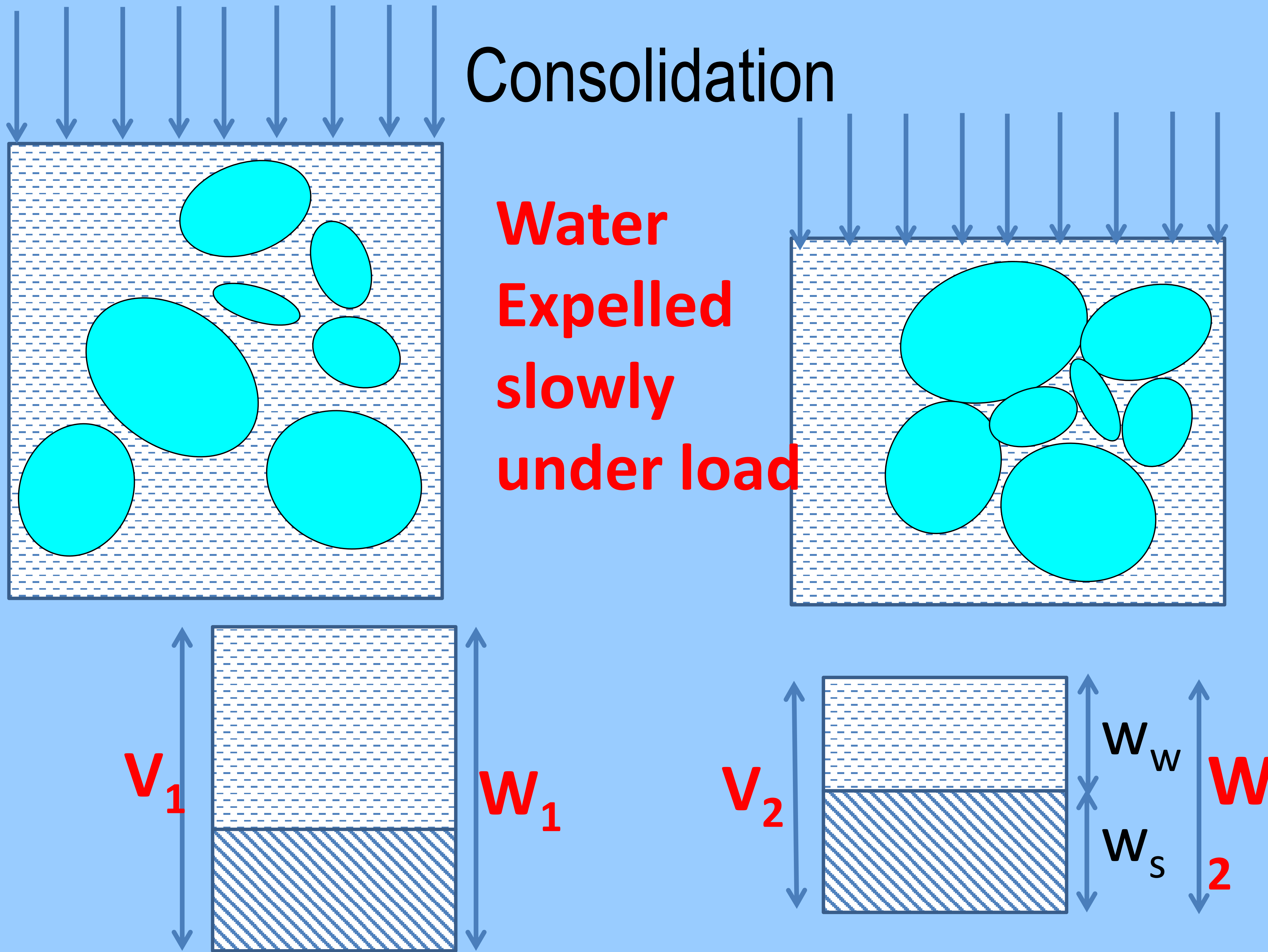
Lecture 12: Consolidation

Dr. Mahendra Gattu

CLE 205 Soil Mechanics

Consolidation Review

- Time Dependent Phenomenon
- Amount of Settlement = function (Time of Settlement)
- Our approach
  - Total/ Final Settlement (Independent of Time)
  - **Intermediate Settlement (Function of Time): Pore pressure dissipation, Mechanics of Consolidation, Terzaghi's 1-D consolidation Theory**


Consolidation
Water
Expelled
slowly
under load
V1
W1
V2
Ww
Ws
W
2

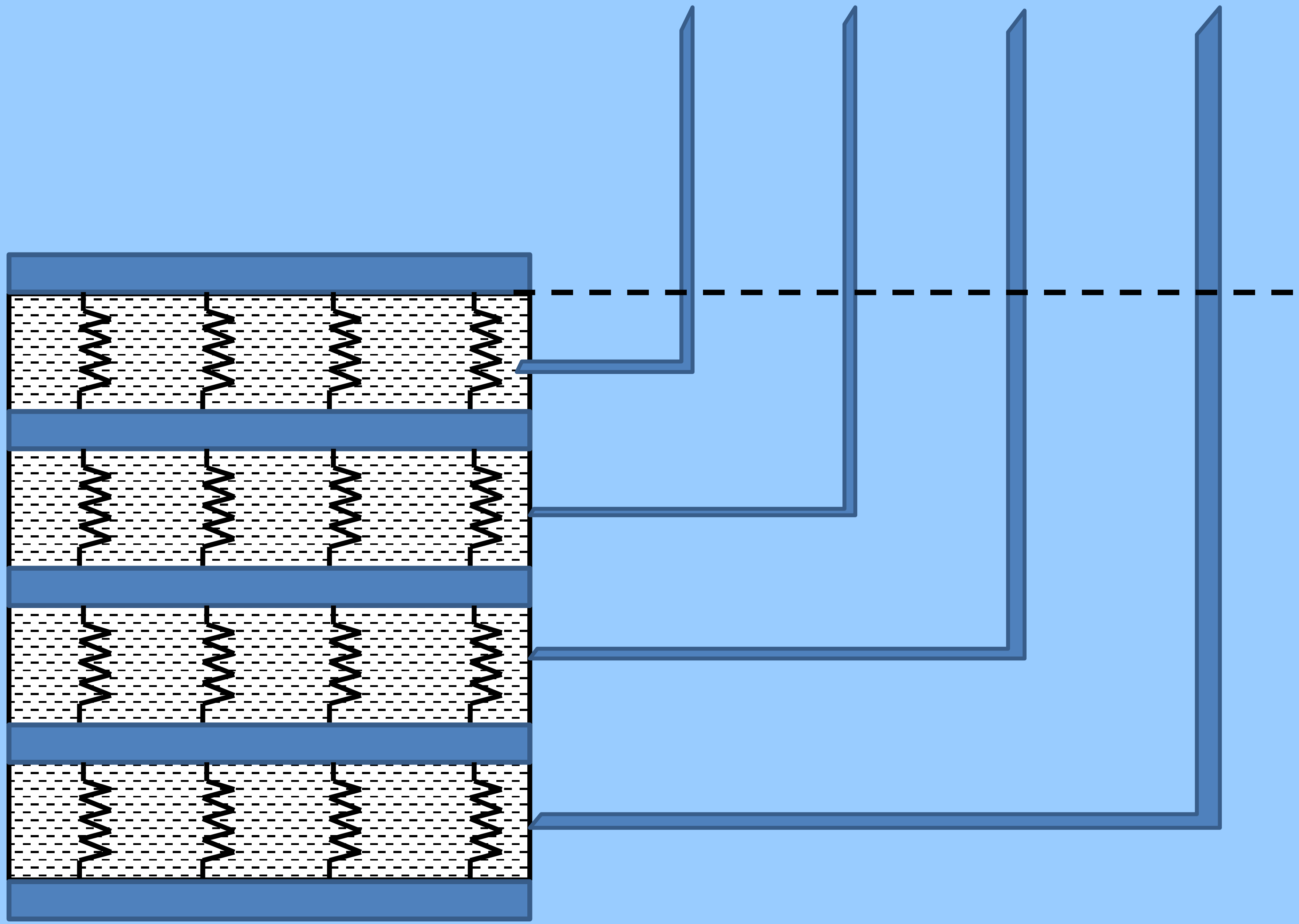
Today's Agenda

- Mechanics of Consolidation: Spring Dashpot-Model
- Terzaghi's 1-D Consolidation Theory
- Problems

Mechanics of Consolidation

- Spring Piston Model
- A cylindrical vessel with compartments marked by pistons separated by springs
- Pistons has perforations to allow water to flow through
- Piezometers inserted at middle of each compartment
- Space between springs filled with water.

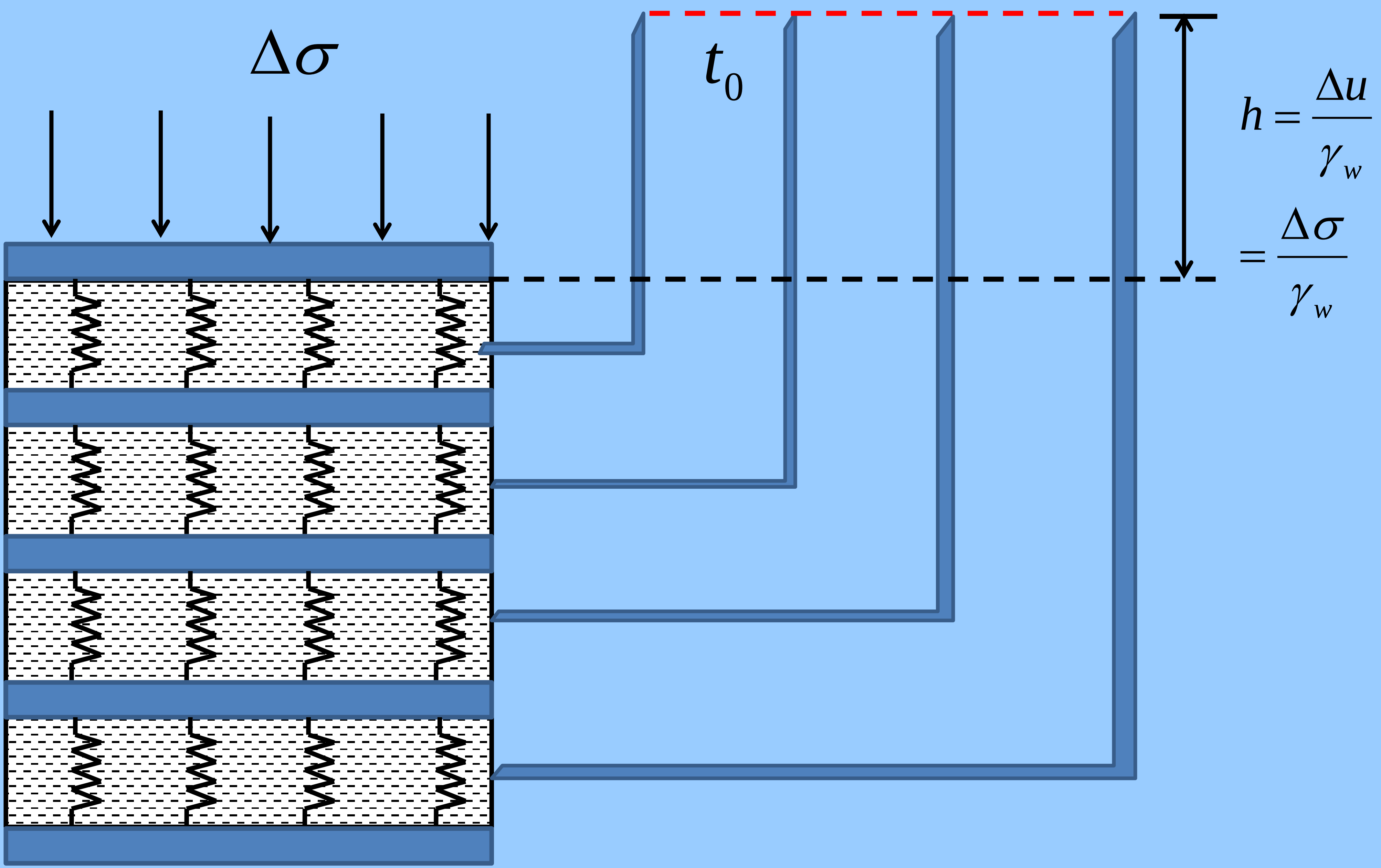
Spring-Piston model



Mechanics of Consolidation

- Apply pressure $\Delta\sigma$ on the top most piston.
- Immediately on application of load,
  - Length of springs remain unchanged
  - As a result, the entire $\Delta\sigma$ is borne by water in the vessel. Δu is the excess hydrostatic pressure
  - Initial rise in water level $h = \dfrac{\Delta\sigma}{\gamma_w}$

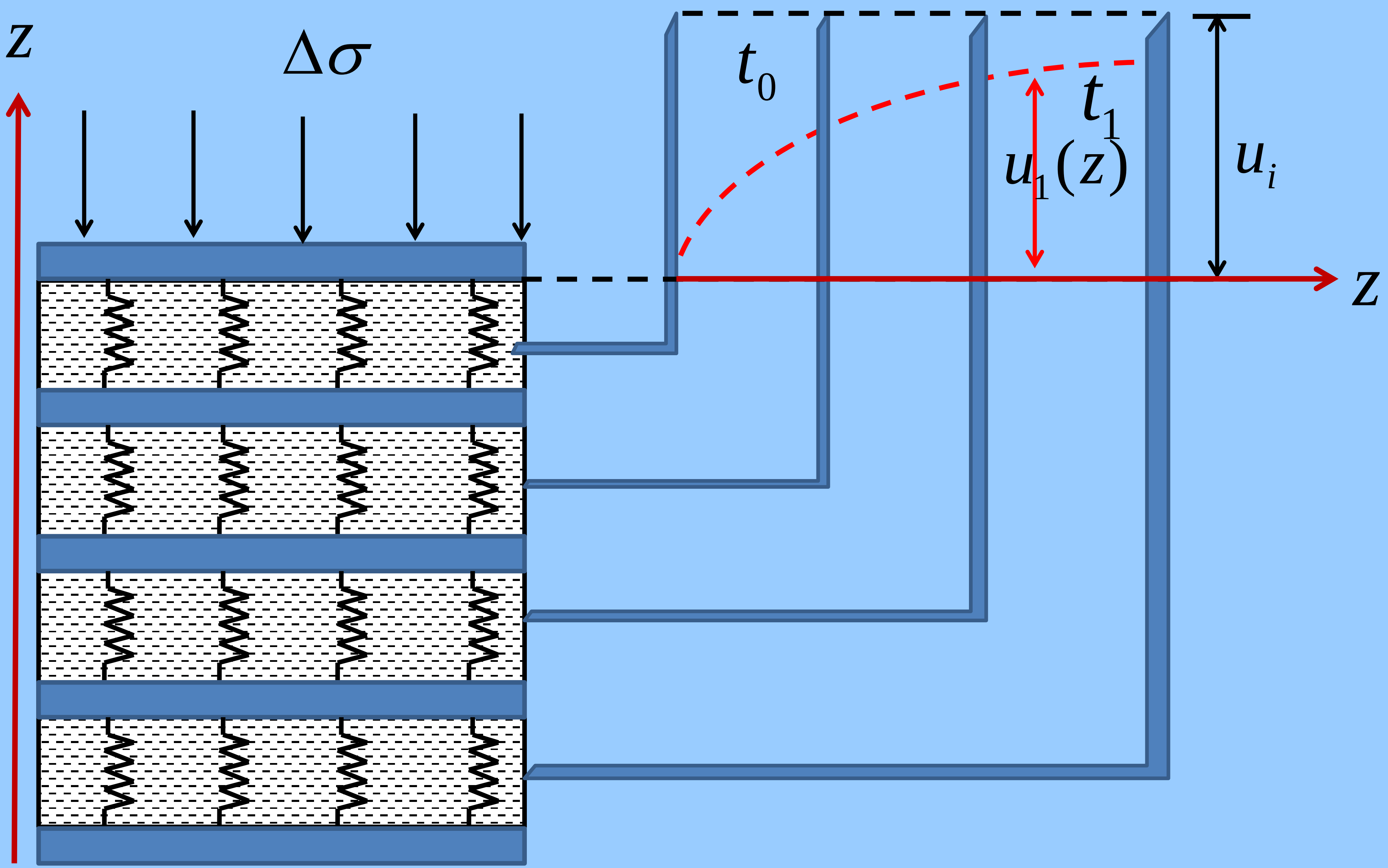
Pore pressure at time t=0



Mechanics of Consolidation

- After time t, flow of water through the perforations has begun at upper compartments. There is a corresponding decrease in volume, the upper springs have compressed a little.
- They carry portion of applied loads and a drop in pressure occurs. The isochrone for $t=t_1$ represents the pore distribution at upper and lower compartments.

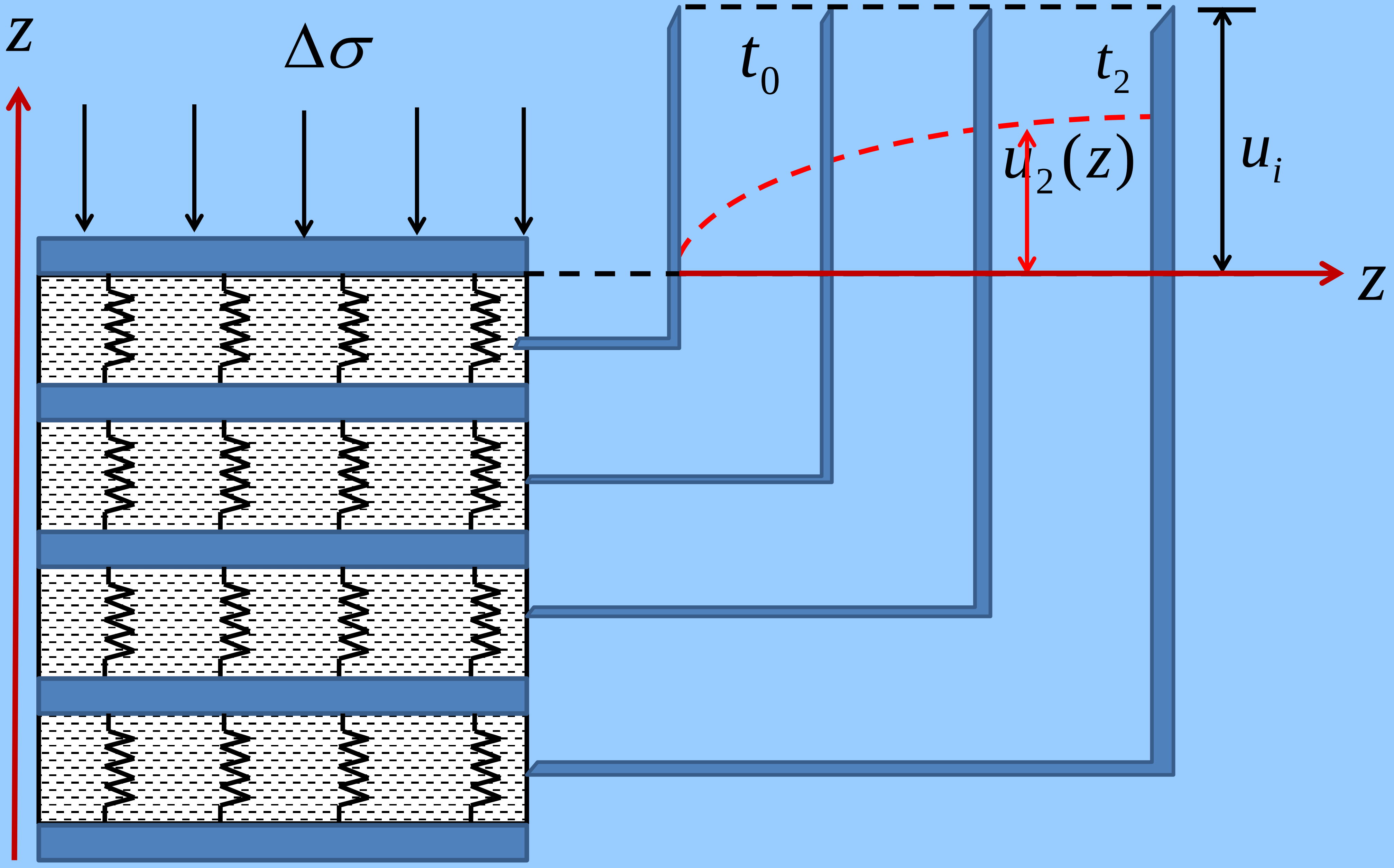
Pore pressure at time t>0



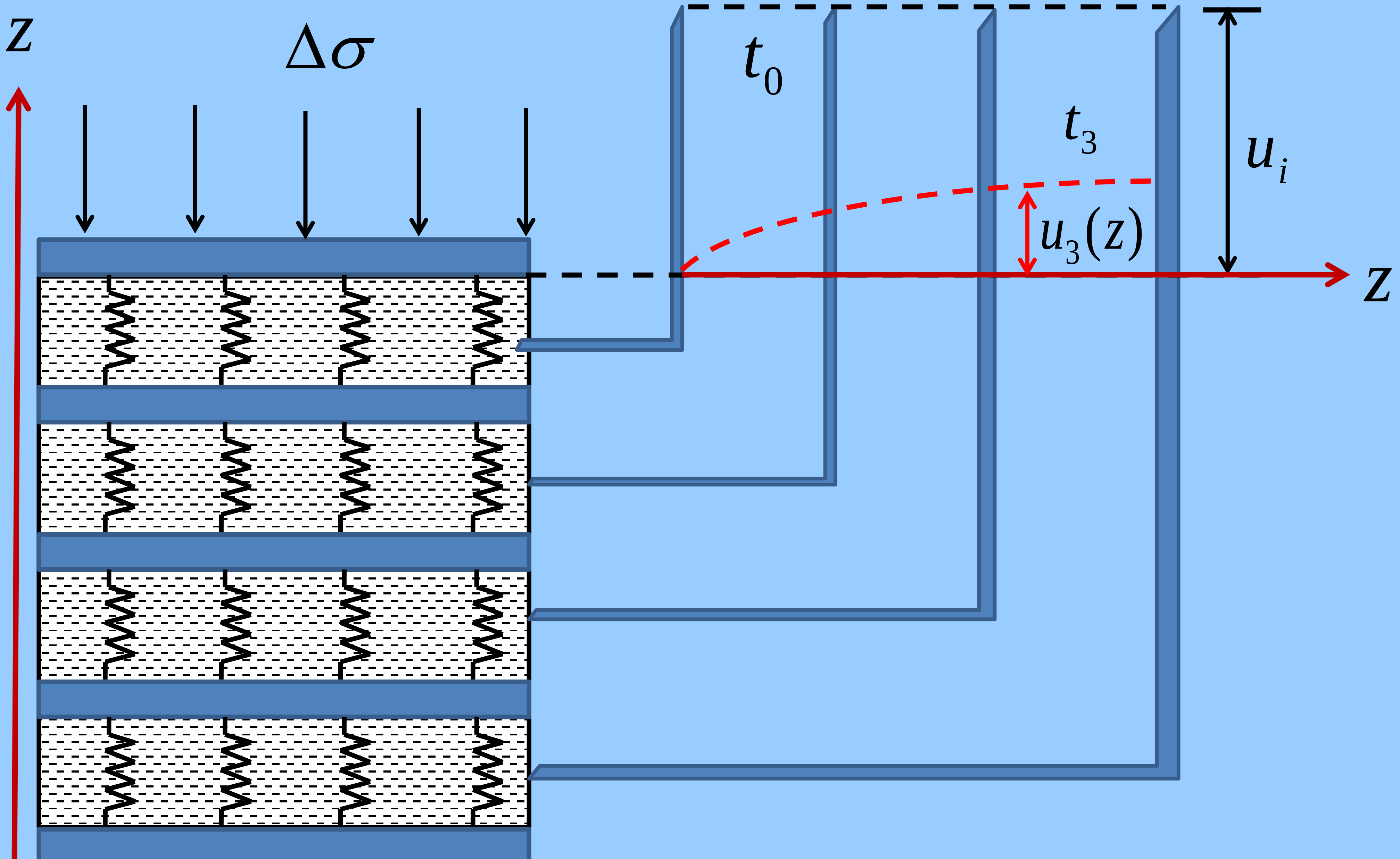
Mechanics of Consolidation

- As time progresses, the springs in the lower compartments are also compressed leading to increase in $\Delta\sigma$ and drop in Δu
- Finally after long time, excess pore pressure Δu drops to zero and the entire load is carried by springs.

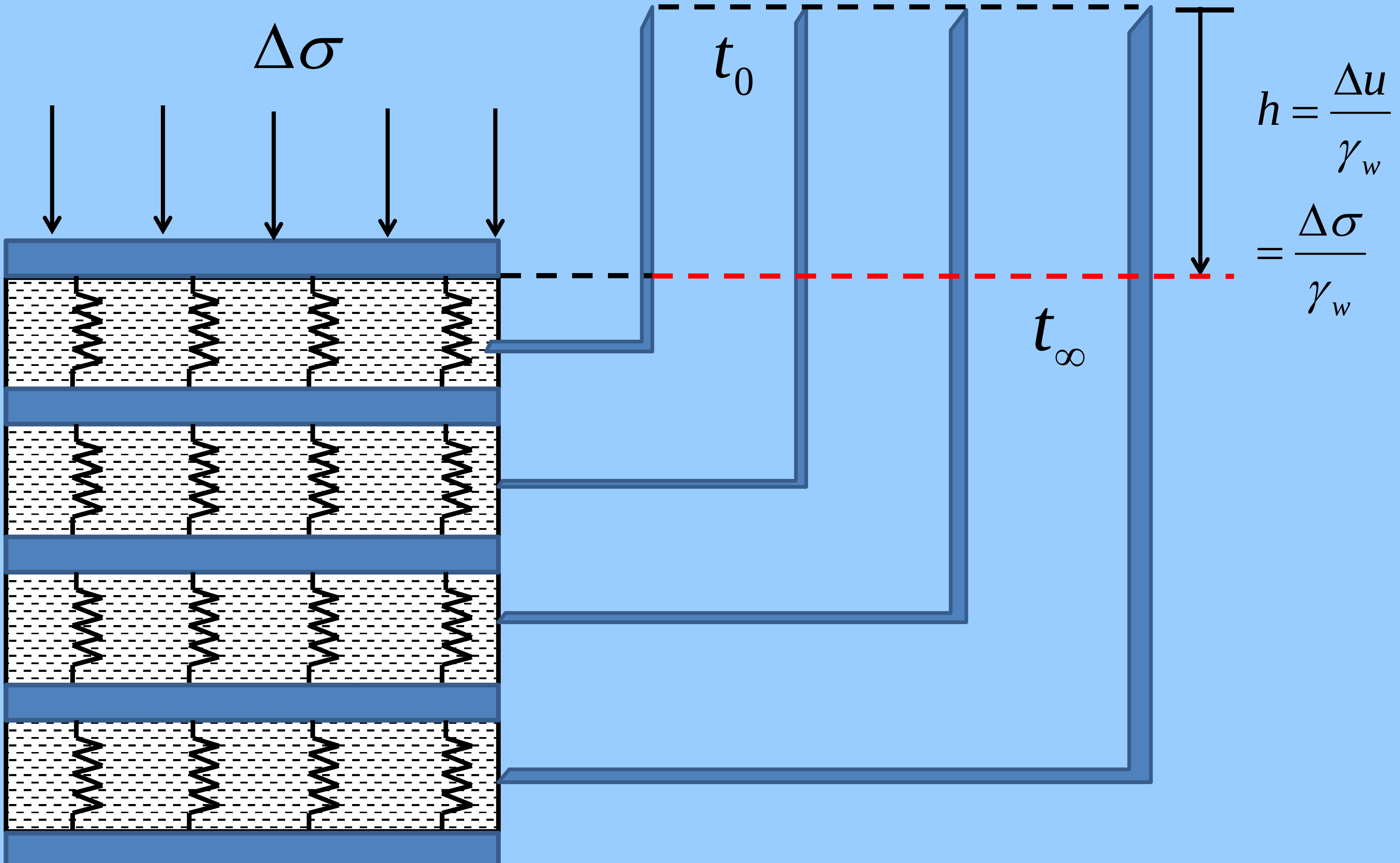
•Pore pressure at time t>0



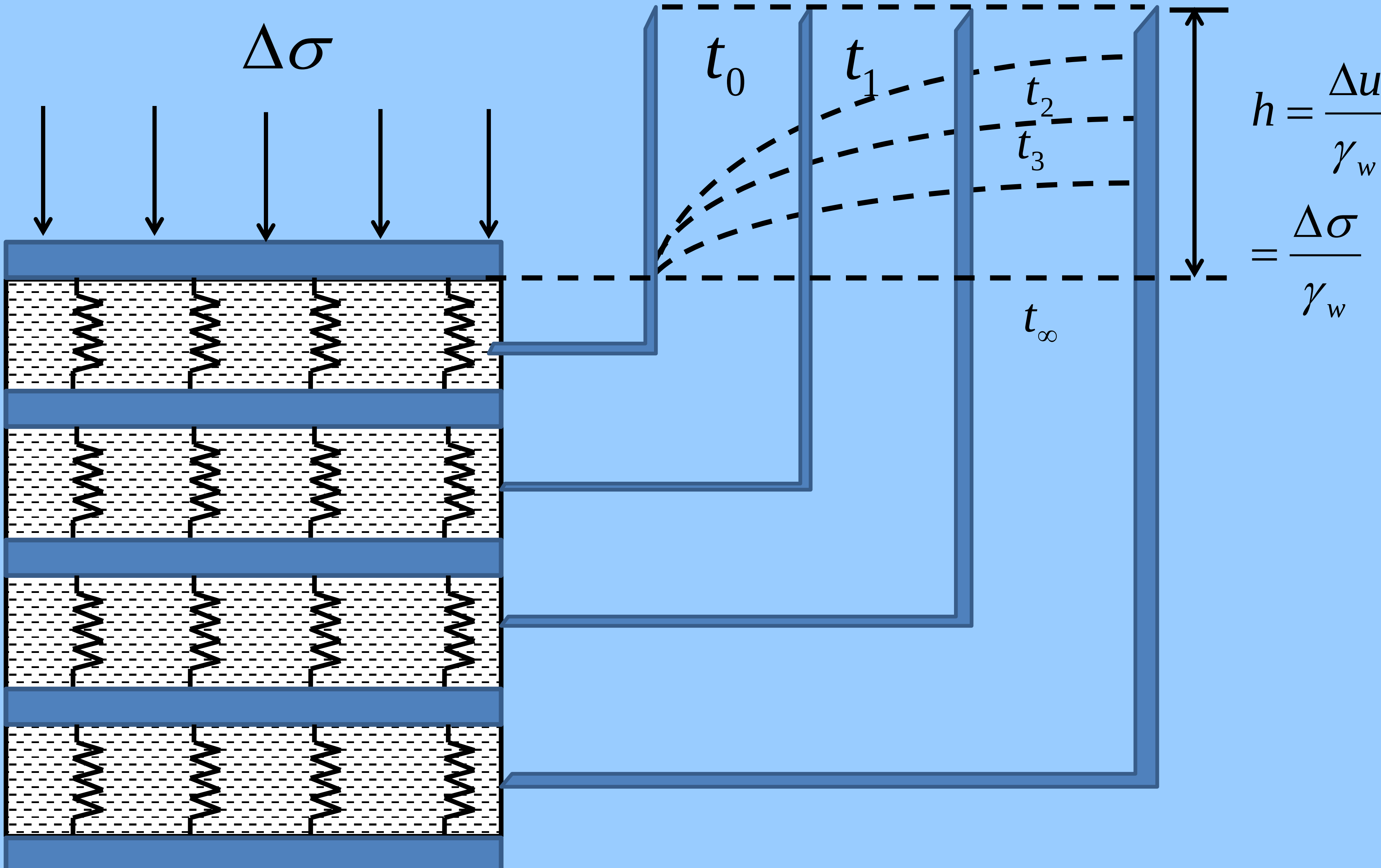
Pore pressure at time t>0



Pore pressure at time t→Infinity



Spring-Piston model



Mechanics of Consolidation

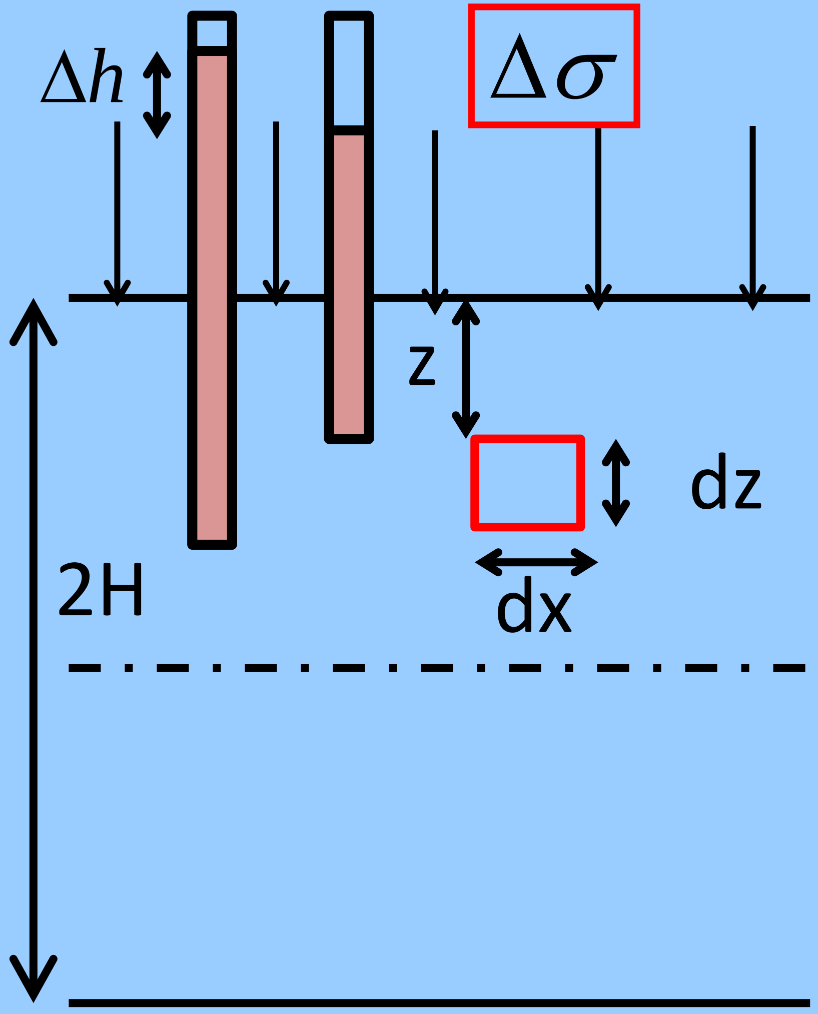
- The decrease in soil volume by the squeezing out of pore water on account of gradual dissipation of excess hydrostatic pressure induced by an imposed total stress is called consolidation.

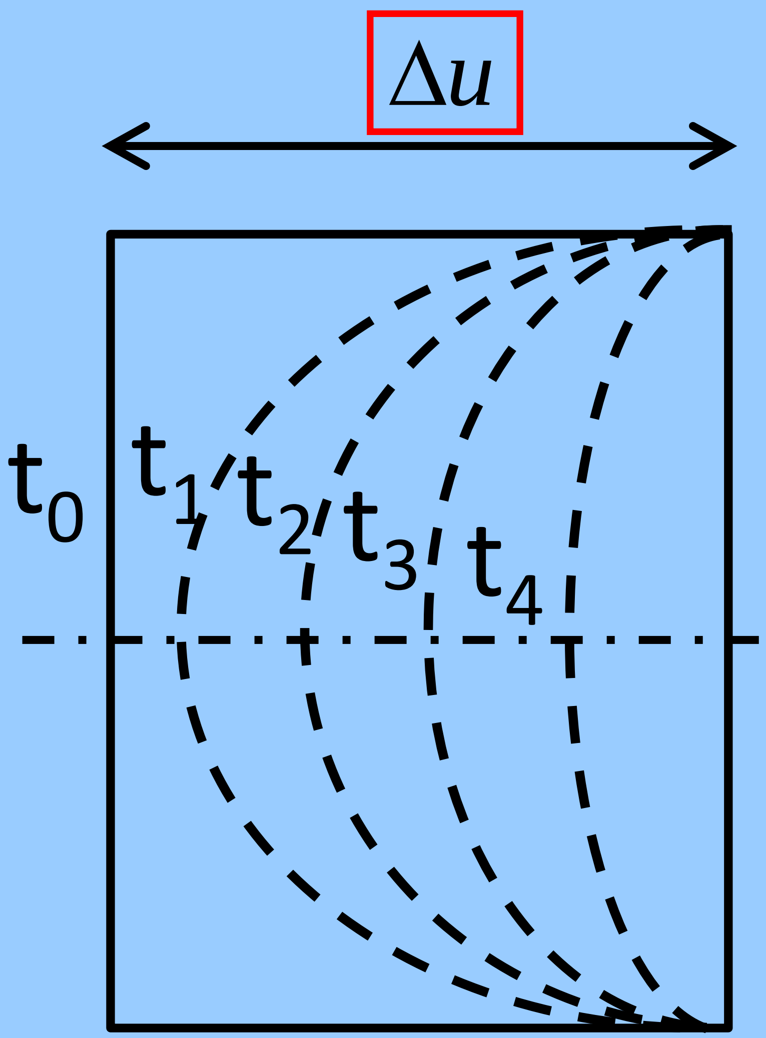
Terzaghi's 1-D Theory of Consolidation

- A Theory to predict the pore pressures at any elapsed time and at any location is required to predict the time rate of consolidation of a consolidating layer.
- Assumptions:

1. Compression and flow is 1-D
2. Darcy's Law is valid
3. Soil is homogeneous and completely saturated
4. Soil grains and water are both incompressible
5. Strains are small. Applied del sigma produce virtually no changes in thickness,
6. k, av are constant.
7. No secondary compression.

Terzaghi's 1-D Theory of Consolidation





Terzaghi's 1-D Theory of Consolidation

- Continuity Equation $$\rho \nabla \bullet v = \rho \frac{\partial w}{\partial t}$$

$$\left(\frac{\partial v_x}{\partial x} + \frac{\partial v_y}{\partial y} + \frac{\partial v_z}{\partial z} \right) dV = \frac{\partial w}{\partial t}$$

$$\left(\frac{\partial v_z}{\partial z} \right) dV = \frac{\partial w}{\partial t}$$

- Darcy's Law $$v_z = k_z \frac{\partial h}{\partial z} = \frac{k_z}{\gamma_w} \frac{\partial u}{\partial z}$$

- Continuity Equation

becomes

$$\frac{k_z}{\gamma_w} \left(\frac{\partial^2 u}{\partial z^2} \right) dV = \frac{\partial w}{\partial t}$$

Terzaghi's 1-D Theory of Consolidation

- Change in Volume and void ratio

$$\frac{\partial w}{\partial t} = \frac{dV}{1+e_0}\left(\frac{\partial e}{\partial t}\right) \qquad de = -a_V d\overline{\sigma}$$

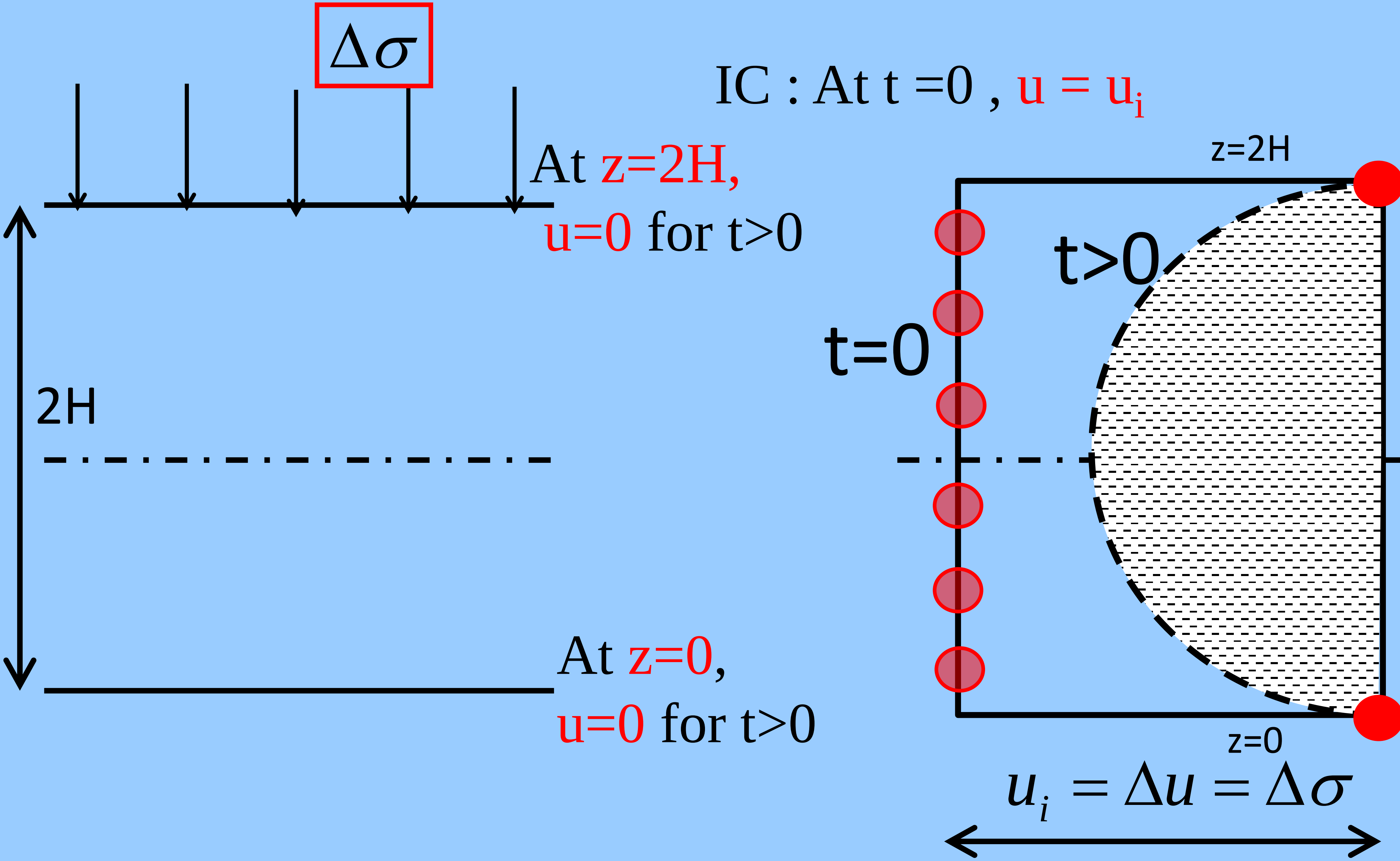
$$= a_V du$$

$$\frac{k_z}{\gamma_w}\left(\frac{\partial^2 u}{\partial z^2}\right) dV = \frac{a_V}{1+e_0}\left(\frac{\partial u}{\partial t}\right) dV$$

$$\frac{k_z(1+e_0)}{\gamma_w a_V}\frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t}$$

$$\boxed{C_V \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t}}$$

Boundary and Initial Conditions



Dimensionless Parameters

- Drainage Path Ratio $$Z = \frac{z}{H}$$
- Time factor $$T_V = \frac{C_V t}{H^2}$$
- Degree of Consolidation $$U_z$$

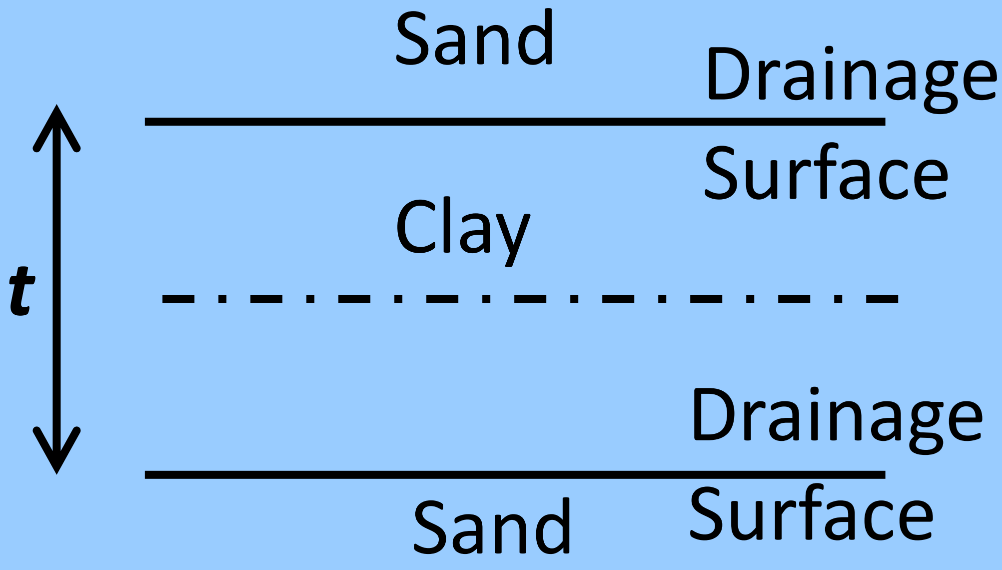
Drainage path ratio Z=z/H

What is value of the length of drainage path H?

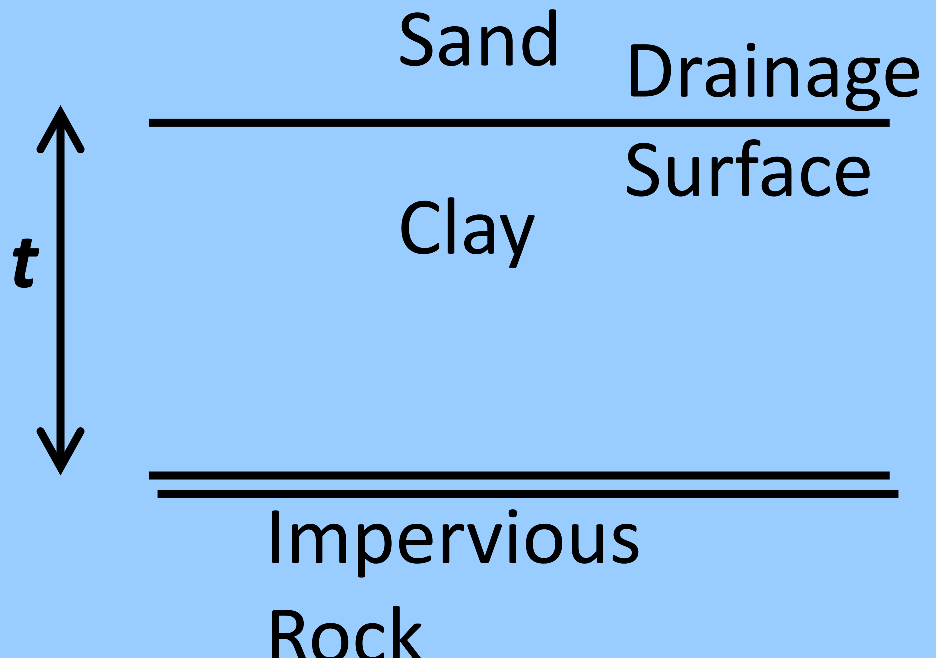
- The maximum distance the water has to travel to reach the drainage surface is used as H in Terzaghi’s 1-D solution.
- t is the thickness of the consolidating layer.
- Depending on drainage conditions, H is taken as t (Single drainage) and t/2(double drainage)

Types of Drainage

- Double Drainage
  - H= t/2



- Single Drainage
  - H=t



Time factor Tv

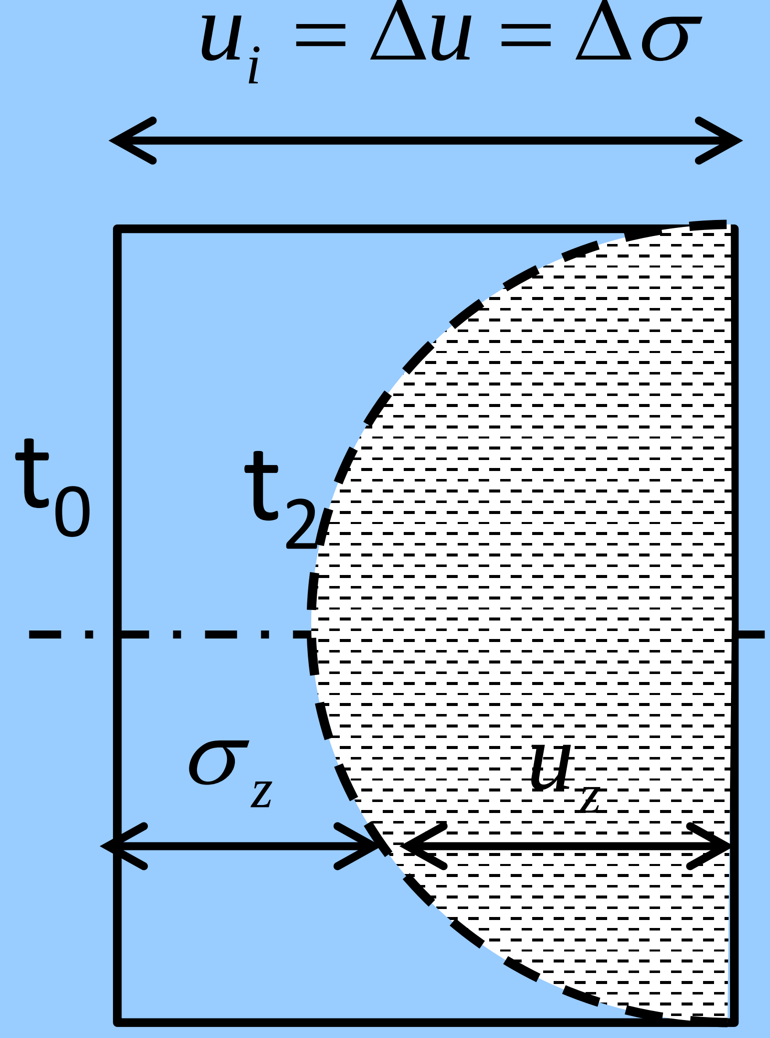
- $T_v = C_v \, t / H^2$, Time Factor
- The time required for a soil to reach a given degree of consolidation is directly proportional to the square of the length of the drainage path and inversely proportional to the coefficient of consolidation.

Degree of Consolidation

Degree of Consolidation

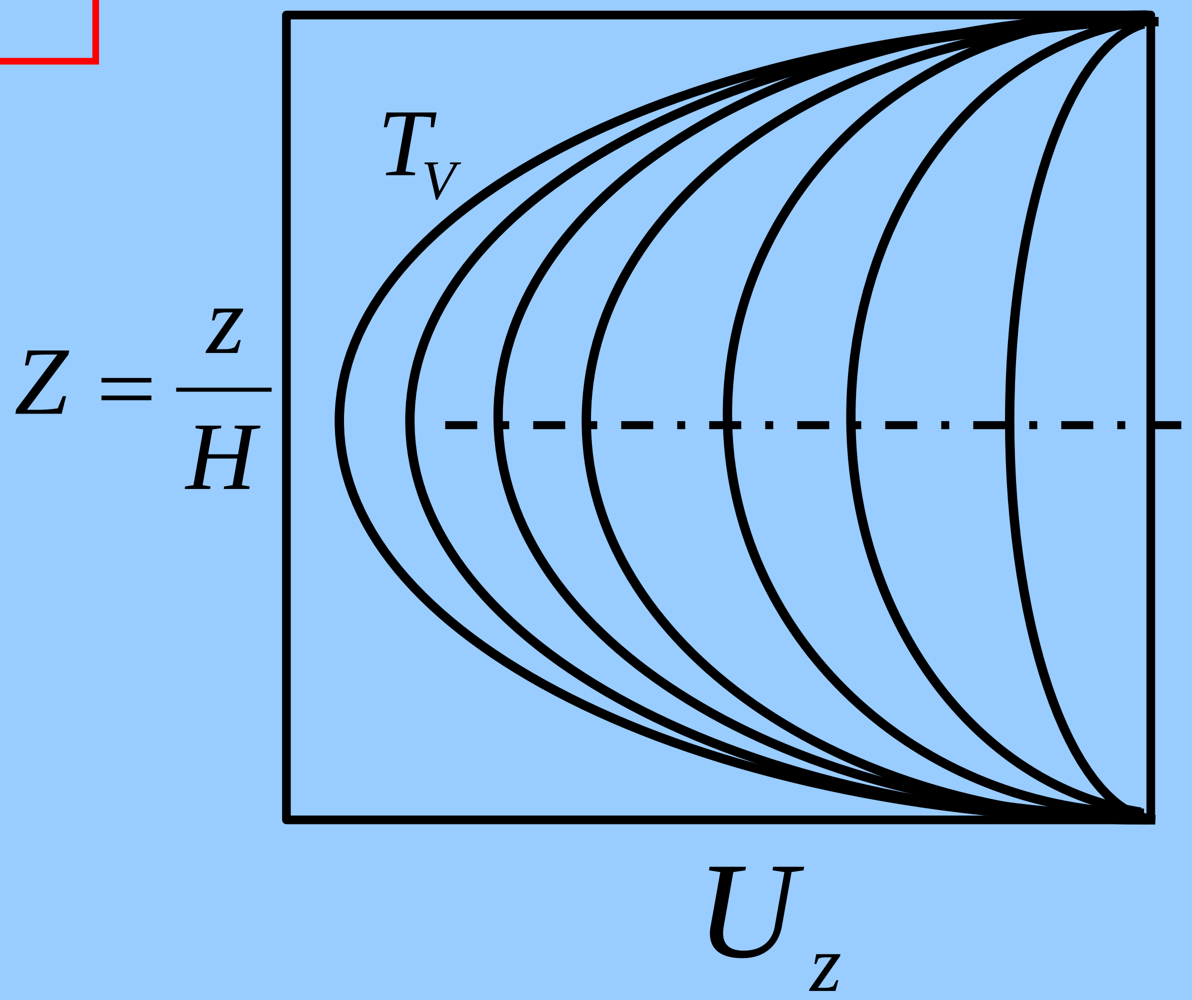
$$U_Z = \frac{u_i - u_z}{u_i}$$

- Represents the stage of consolidation at a certain location in the consolidating layer.
- Ratio of dissipate pore pressure by initial pore pressure at a given location.



Solution of the pde

$$U_z = 1 - \sum_{n=0}^{\infty} f_1(Z)\, f_2(T_V)$$



Average Degree of Consolidation

$$U = \frac{\text{Pore Pressure Dissipated}}{\text{Initial Pore Pressure}}$$

$$U = \frac{Hatched}{Total}$$

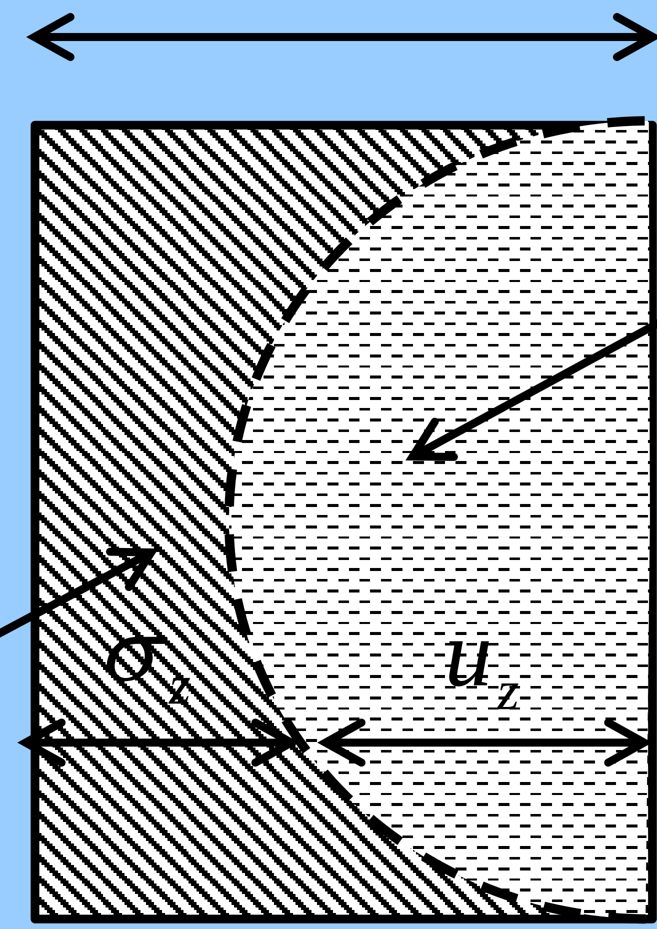
At t =0,
Hatched = 0
U=0

At t →infinity,
Hatched = Total
U=1

$$u_i = \Delta u = \Delta \sigma$$

$$Z = \frac{z}{H}$$

Pore
Pressure
Dissipated



Excess pore
pressure
remaining

$$U_z$$

Taylor's simplified Solution

- U is average degree of consolidation.
- Tv is the time factor

$$T_V = \begin{cases} \frac{\pi}{4} U^2, U \leq 60\% \\ 1.781 - 0.933 \log(100 - U\%), U > 60\% \end{cases}$$

Terzaghi's 1-D Theory of Consolidation Summary

- C_v is coefficient of consolidation, Units m^2/sec or m^2/day
- Boundary Conditions and Initial Conditions
- Non-dimensional Factors
- Z=z/H, Drainage path Ratio
- $T_v = C_v\, t / H^2$, Time Factor
- Degree of Consolidation or Consolidation ratio at given z
- U: Average degree of Consolidation at a given time t

$$U_Z = \frac{u_i - u_z}{u_i}$$

$$U = \frac{\text{Pore Pressure Dissipated}}{\text{Initial Pore Pressure}}$$

Problem

- A 8m thick clay layer with single drainage settles by 120mm in 2 years. The coefficient of consolidation for this clay was found to be 0.006 cm2/sec. Calculate the likely ultimate consolidation settlement and find out how long it will take to undergo 90% of this settlement.

Problem

- A certain clay layer has thickness of 5m. After 1 year, when the clay was 50% consolidated, 8cm of settlement has occurred. For a similar clay and loading conditions, how much settlement would occur at the end of 1 year and 4 years respectively, if thickness of new layer were 25m?

Problem

- In a laboratory consolidation test on a 20mm thick sample of saturated clay taken from a site, 50% consolidation was reached in 10 minutes. Estimate the time required for clay layer 5m thickness at site for 50% consolidation if drainage is only towards the top. Assume the laboratory sample and clay layer are subjected to same increase in stress. How much time is required for clay layer for 50% consolidation. How much time is required to reach 50% consolidation for double drainage of clay layer?

Homework Problem

1. A normally consolidated clay settled by 20 mm when the effective stress was increased from 25 to 50 kN/m2. What will be its settlement when the effective stress is increased from 50 to 100 kN/m^2?

Lecture 14: Shear Strength

Dr. Mahendra Gattu

CLE 205 Soil Mechanics

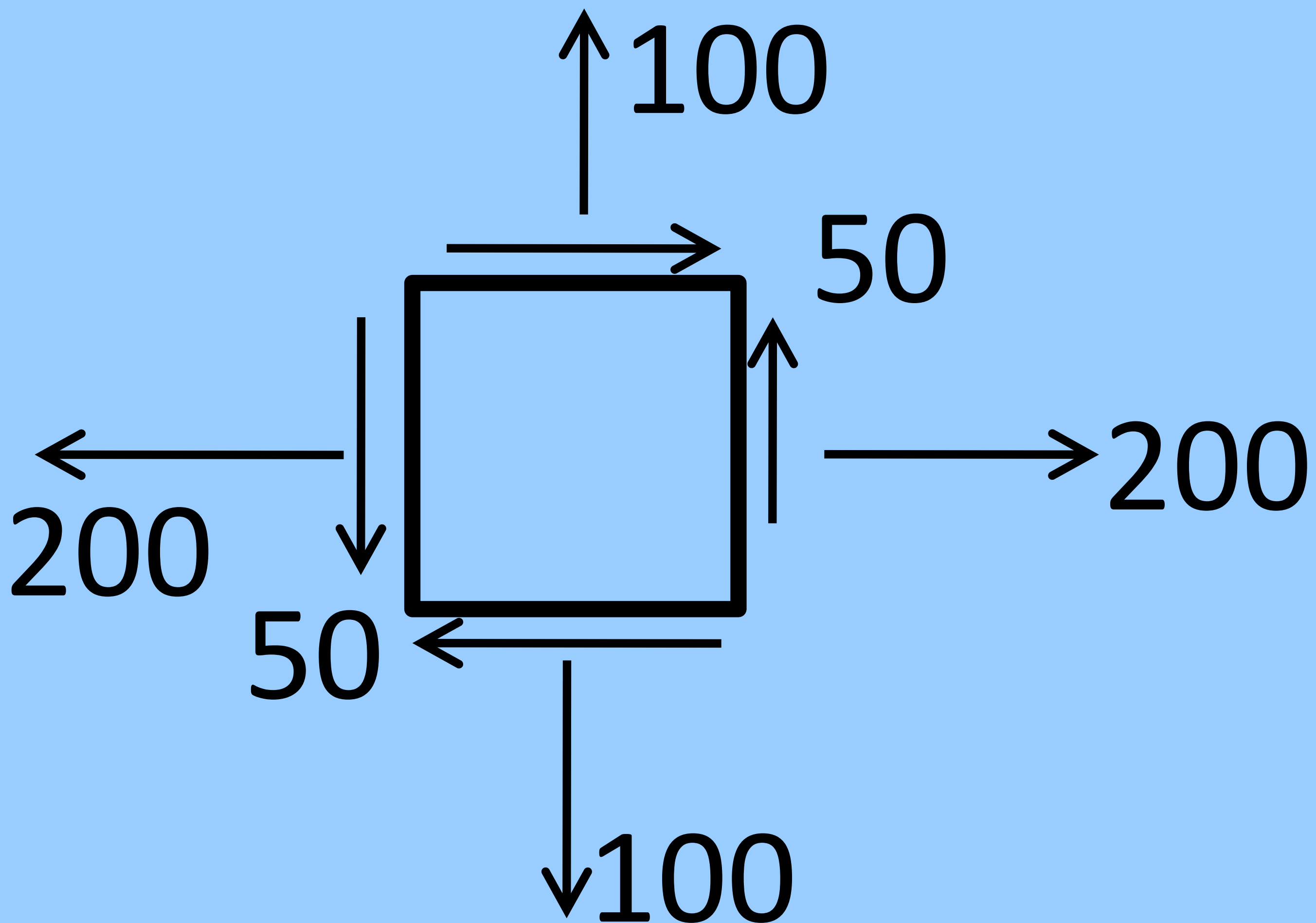
Today's Agenda

- Mohr's Circle
- Shear Strength of Soils
- Mechanism of shear resistance
- Mohr-Coulomb Failure Criterion
  - Modification for soils
- Direct Shear Test
- Triaxial Shear Test

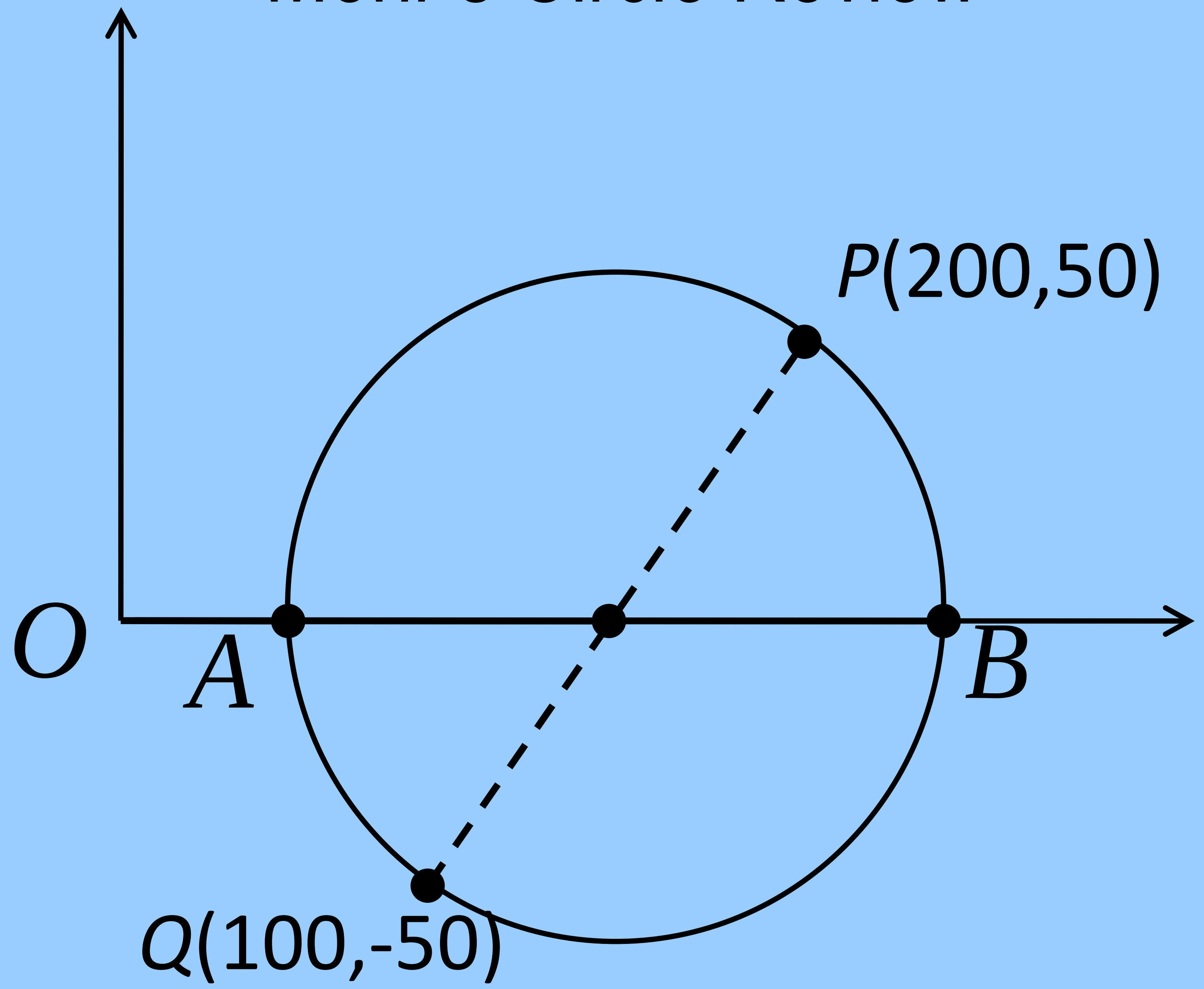
Today's Agenda

- Types of Triaxial Shear Test
  - UU , CU, CD
- Vane Shear Test
- Shear Strength of Clay Soils
  - UU Test
  - CU Test
  - CD Test

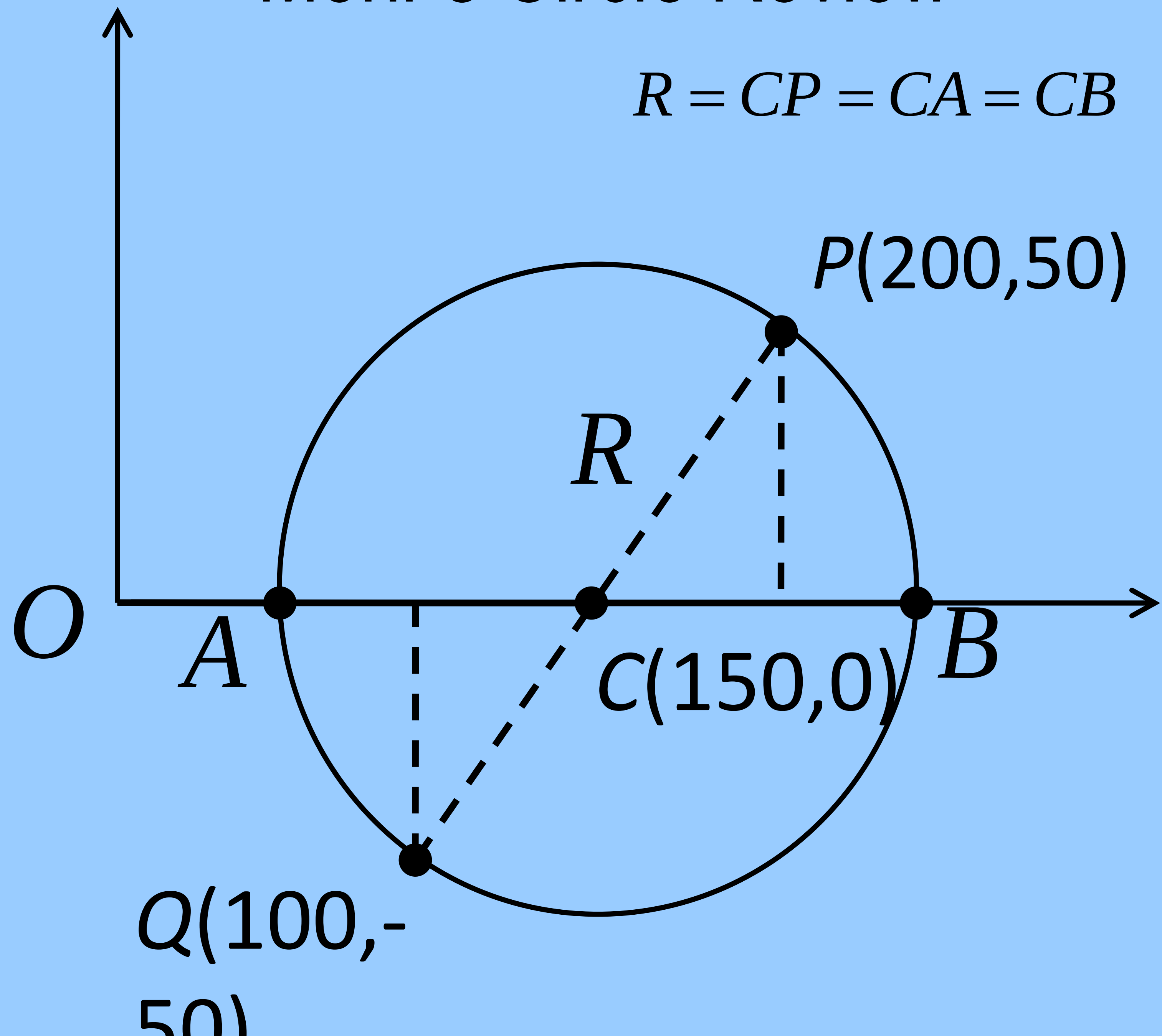
Mohr's Circle Review



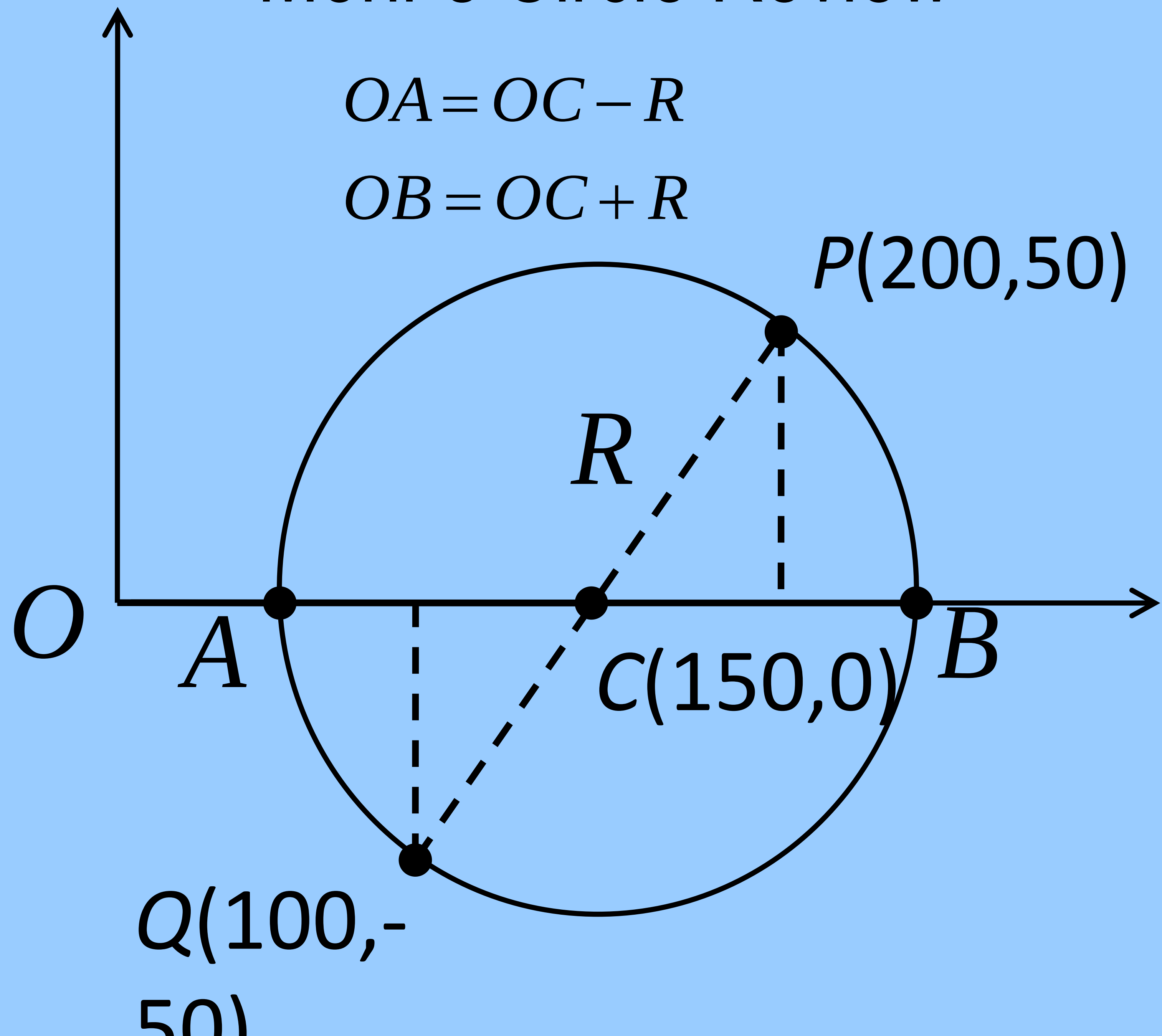
Mohr's Circle Review



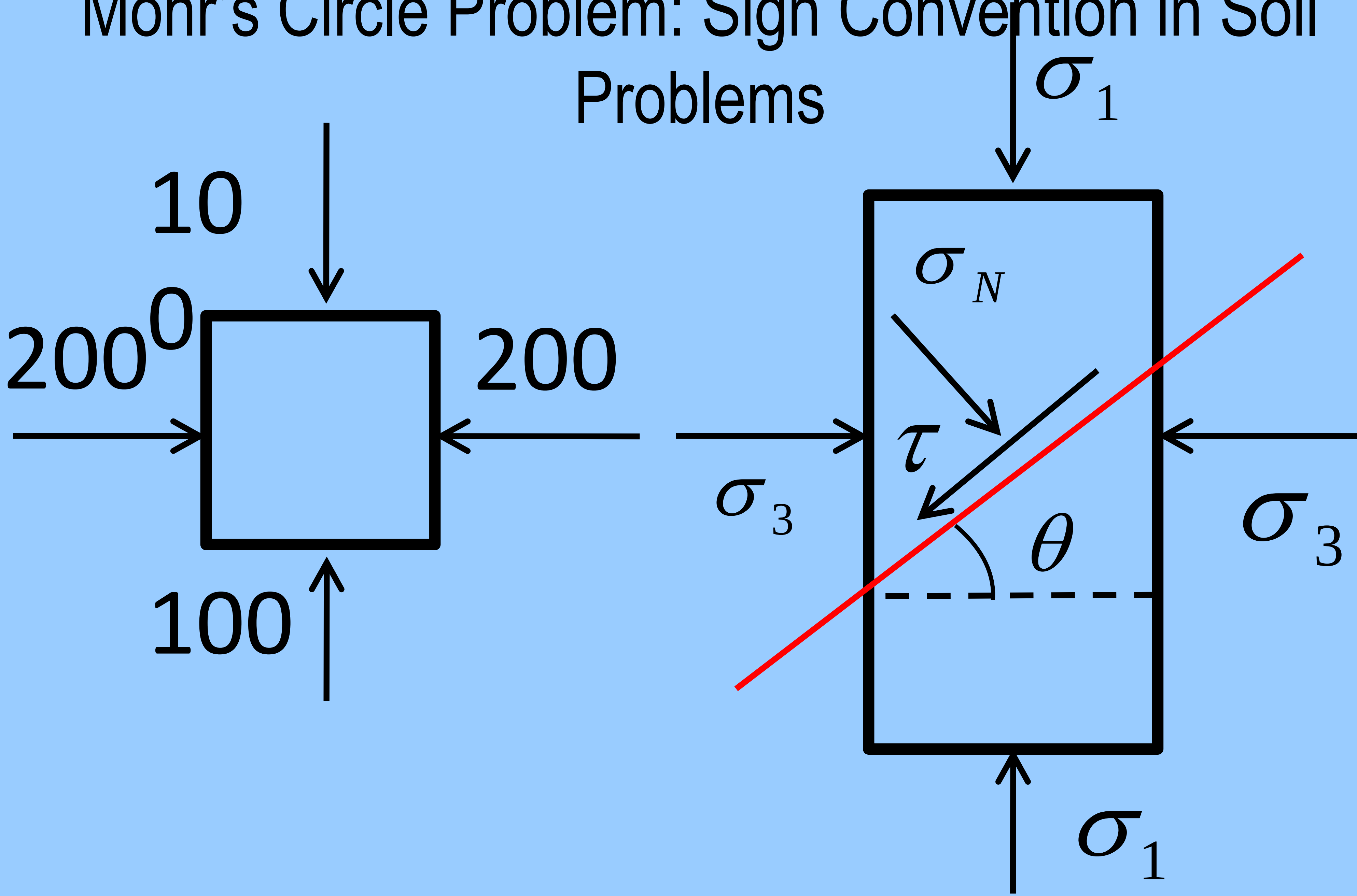
Mohr's Circle Review



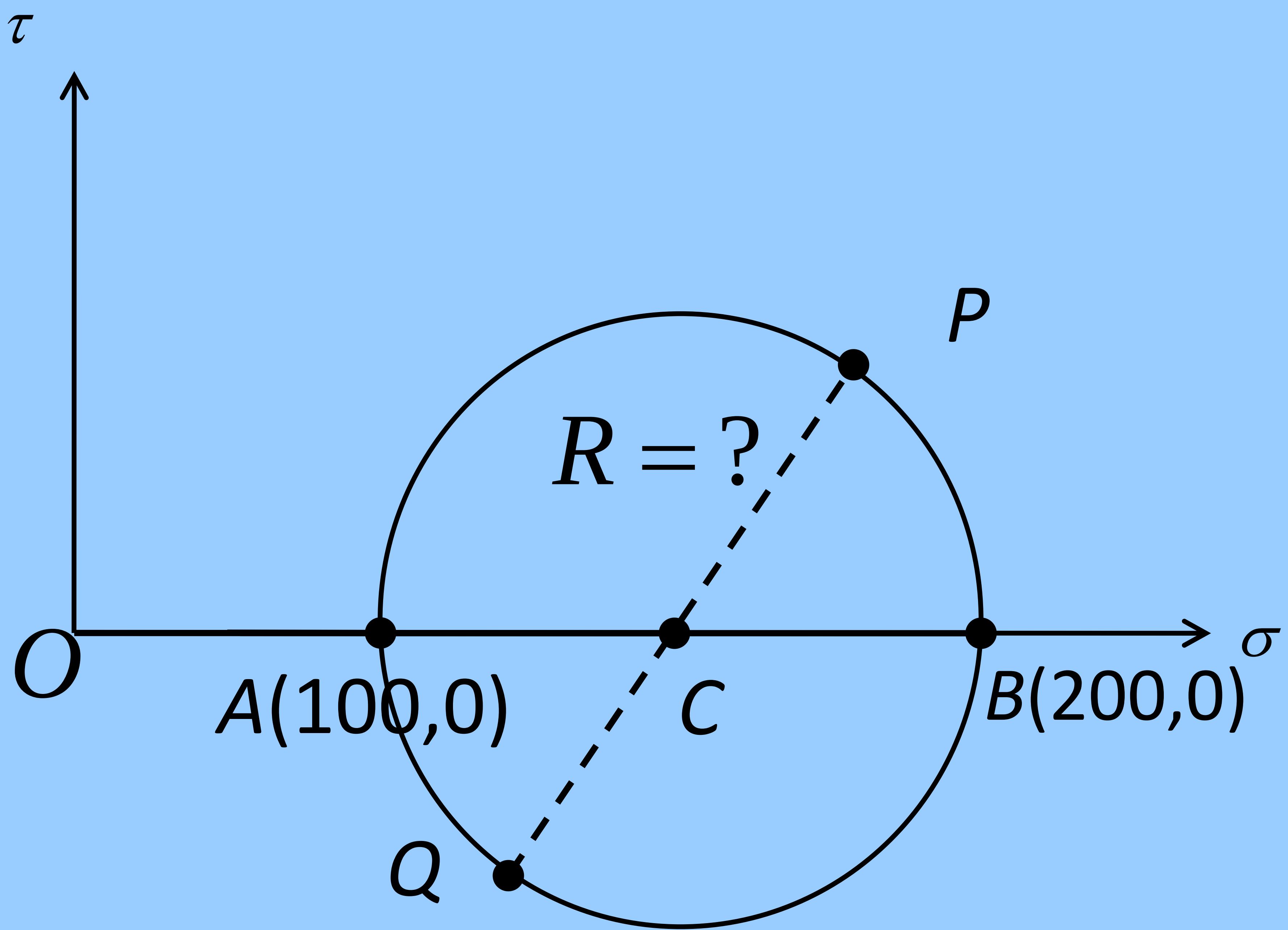
Mohr's Circle Review



Mohr's Circle Problem: Sign Convention in Soil Problems



Mohr's Circle Review



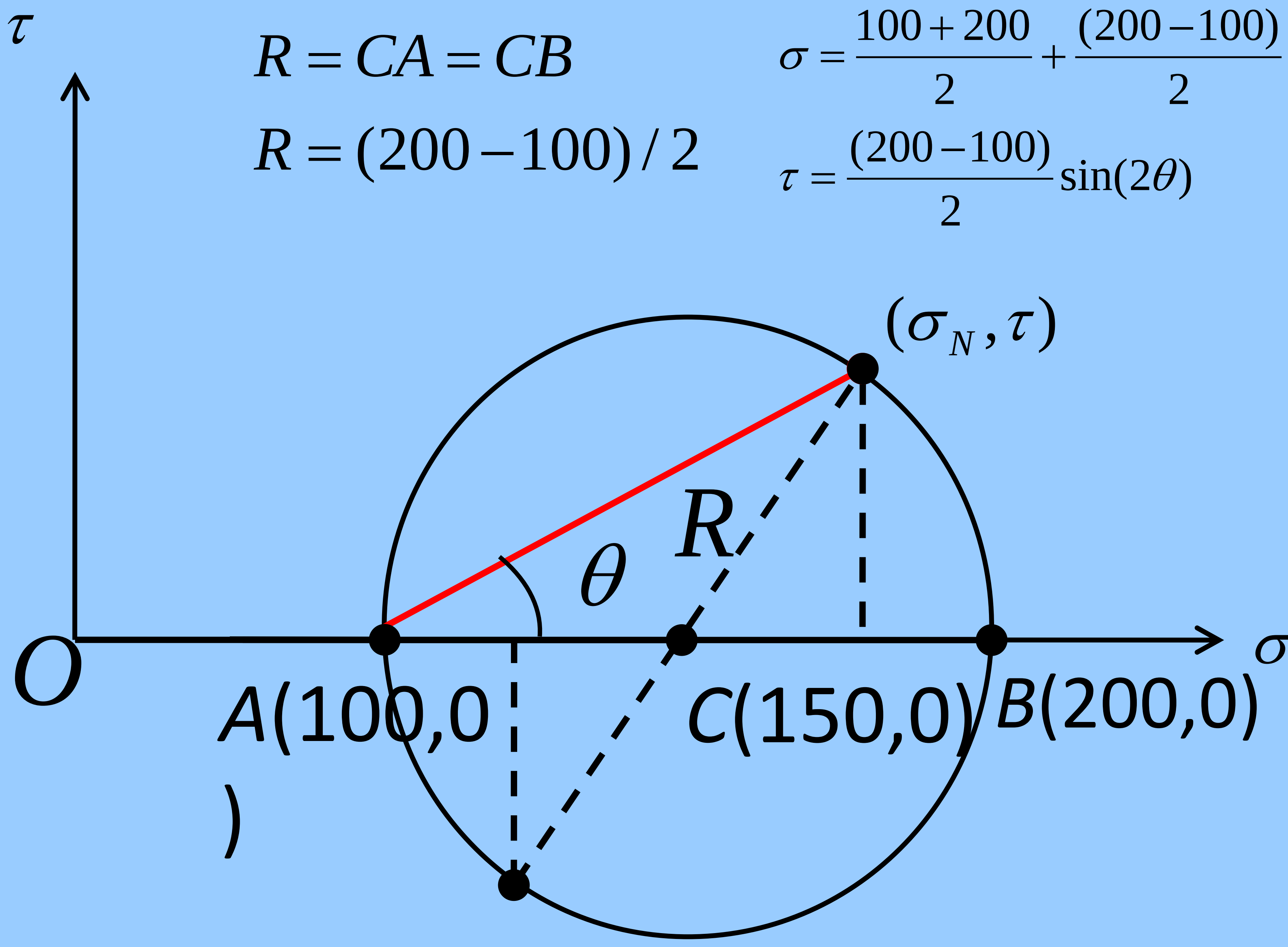
•Mohr's Circle Review

$$R = CA = CB$$

$$R = (200 - 100)/2$$

$$\sigma = \frac{100+200}{2} + \frac{(200-100)}{2}\cos(2\theta)$$

$$\tau = \frac{(200-100)}{2}\sin(2\theta)$$



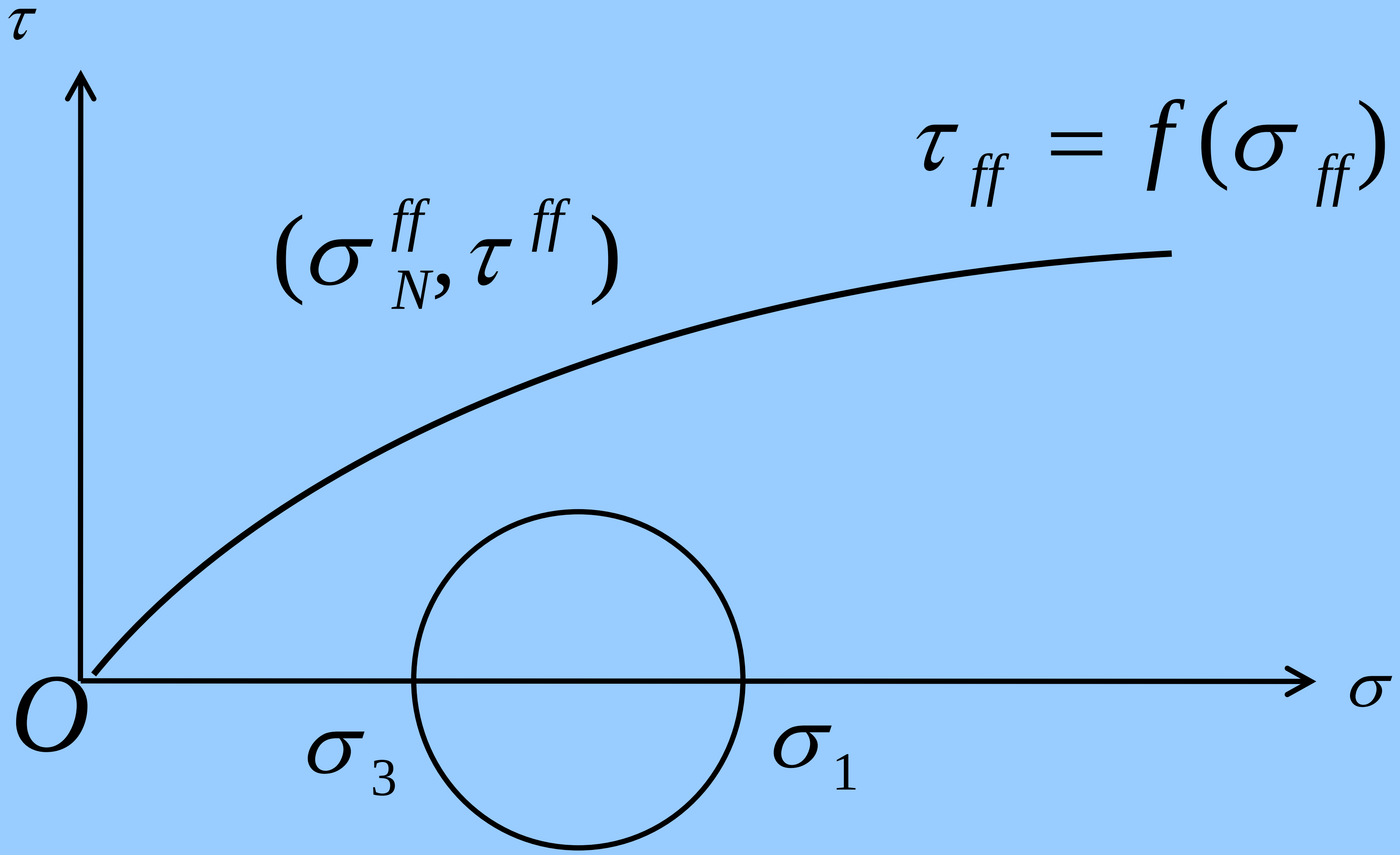
Mechanism of Shear Resistance

- Friction
- Interlocking of particles contributes appreciably to frictional resistance

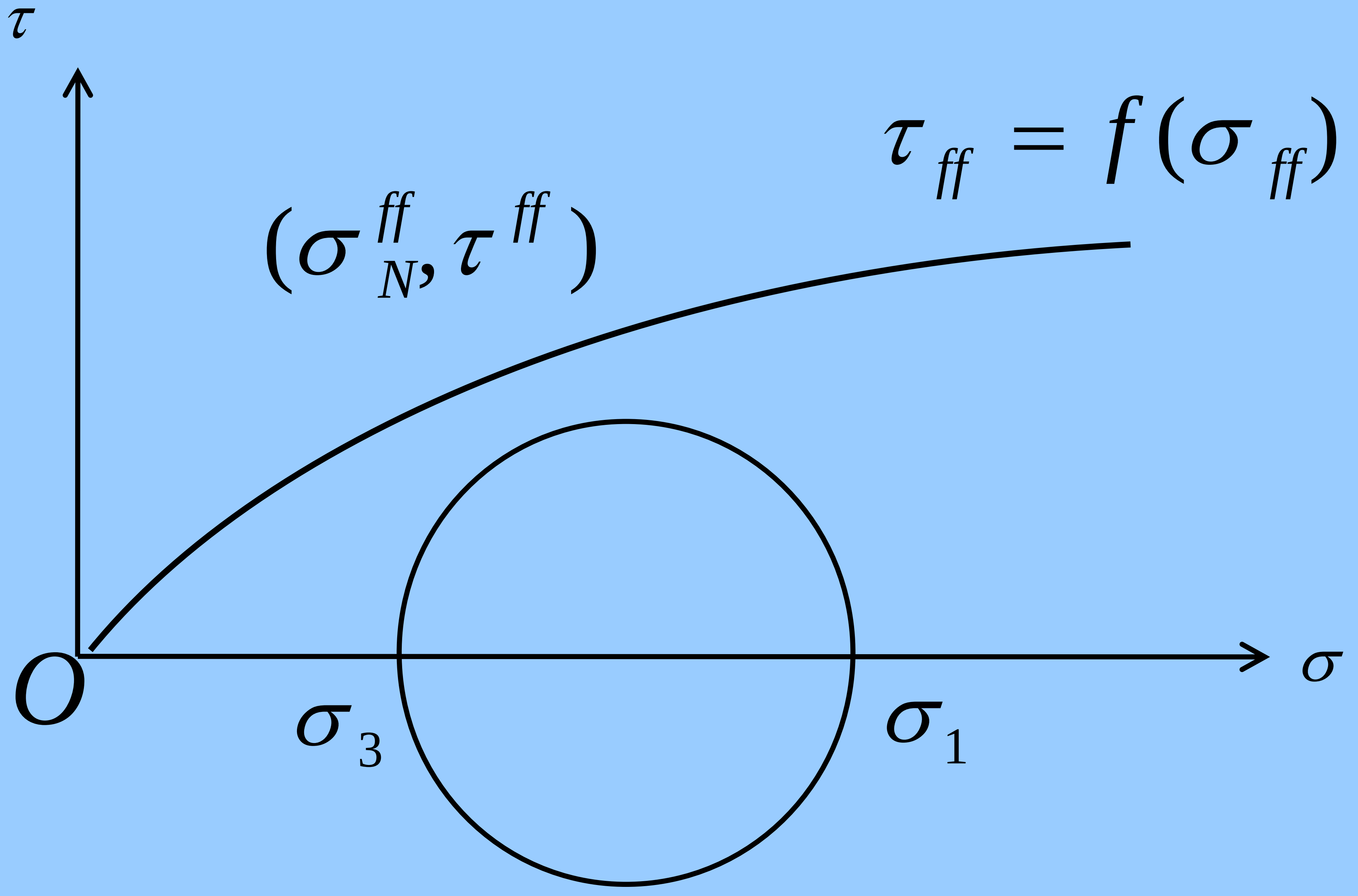
Mohr-Coulomb Failure Criterion

- According to Mohr's failure theory, the materials fail when the shear stress on the failure plane at failure reaches a value which is unique function of the normal stress on that plane.

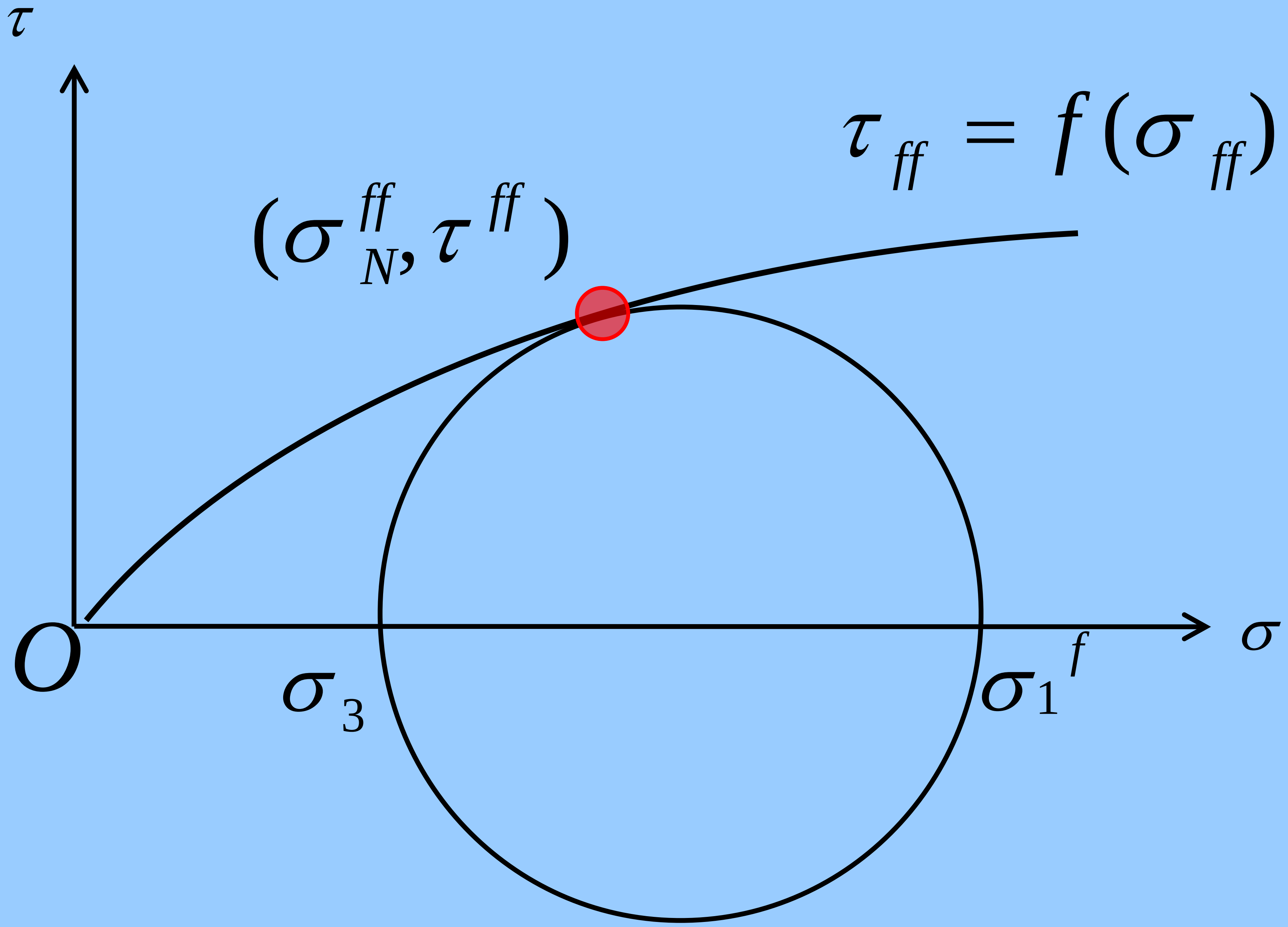
Mohr Coulomb Failure Envelope



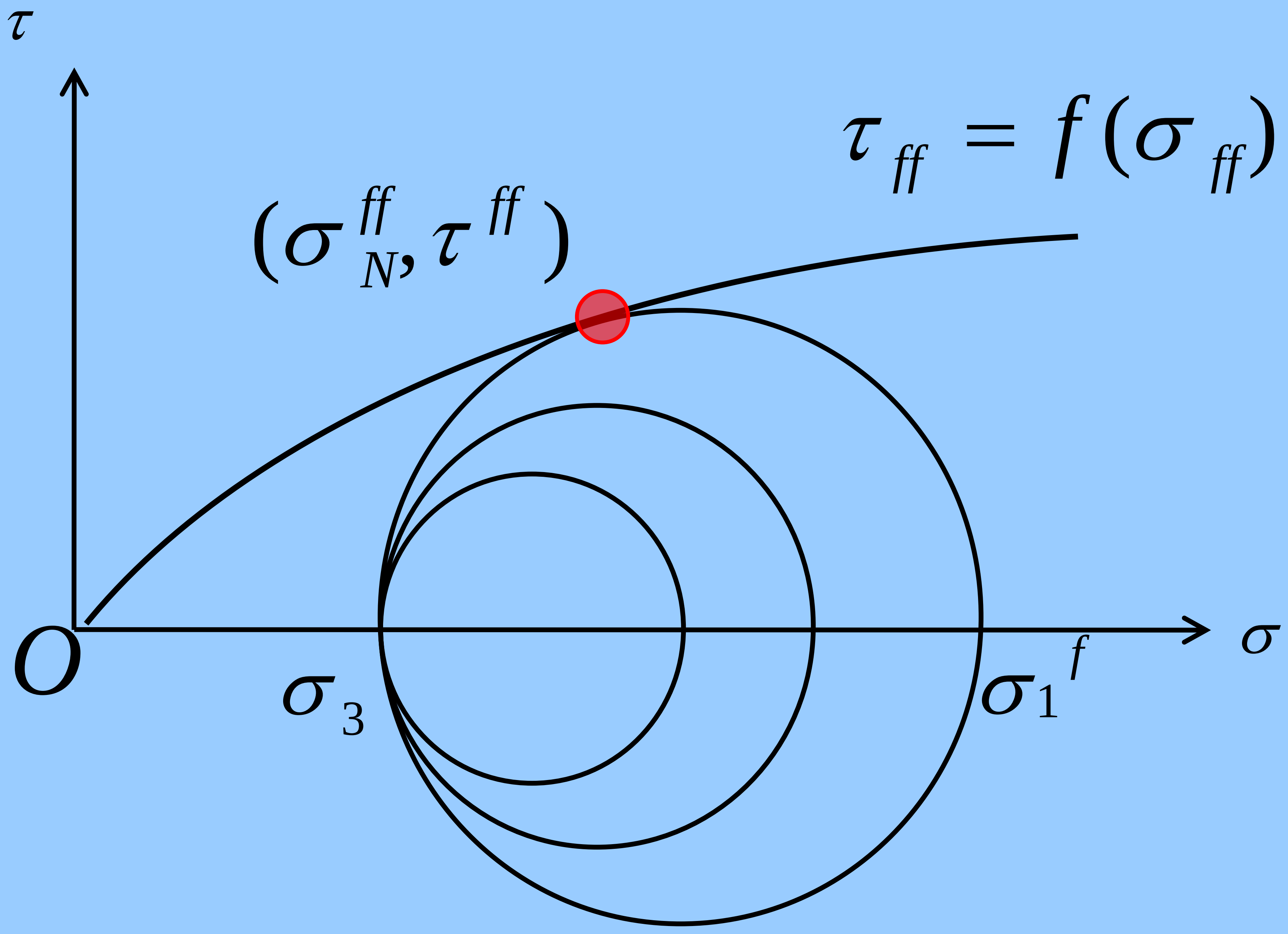
Mohr Coulomb Failure Envelope



Mohr Coulomb Failure Envelope



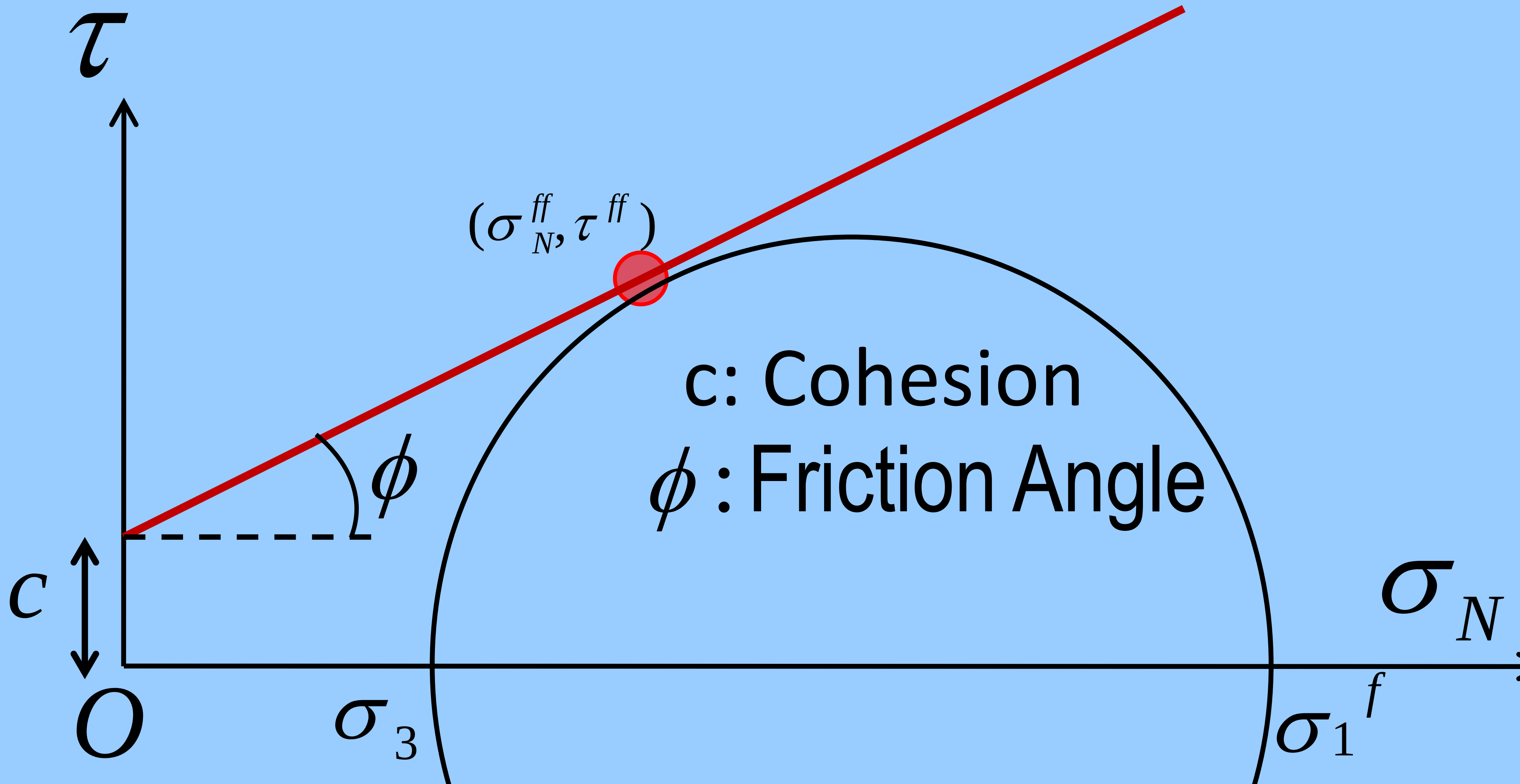
Mohr Coulomb Failure Envelope



Coulomb Equation

$$\tau = c + \sigma_N \tan(\phi)$$

Total Stress



Problem

- Sketch the Coulomb Equation for the following two cases on the $\sigma - \tau$ plot.

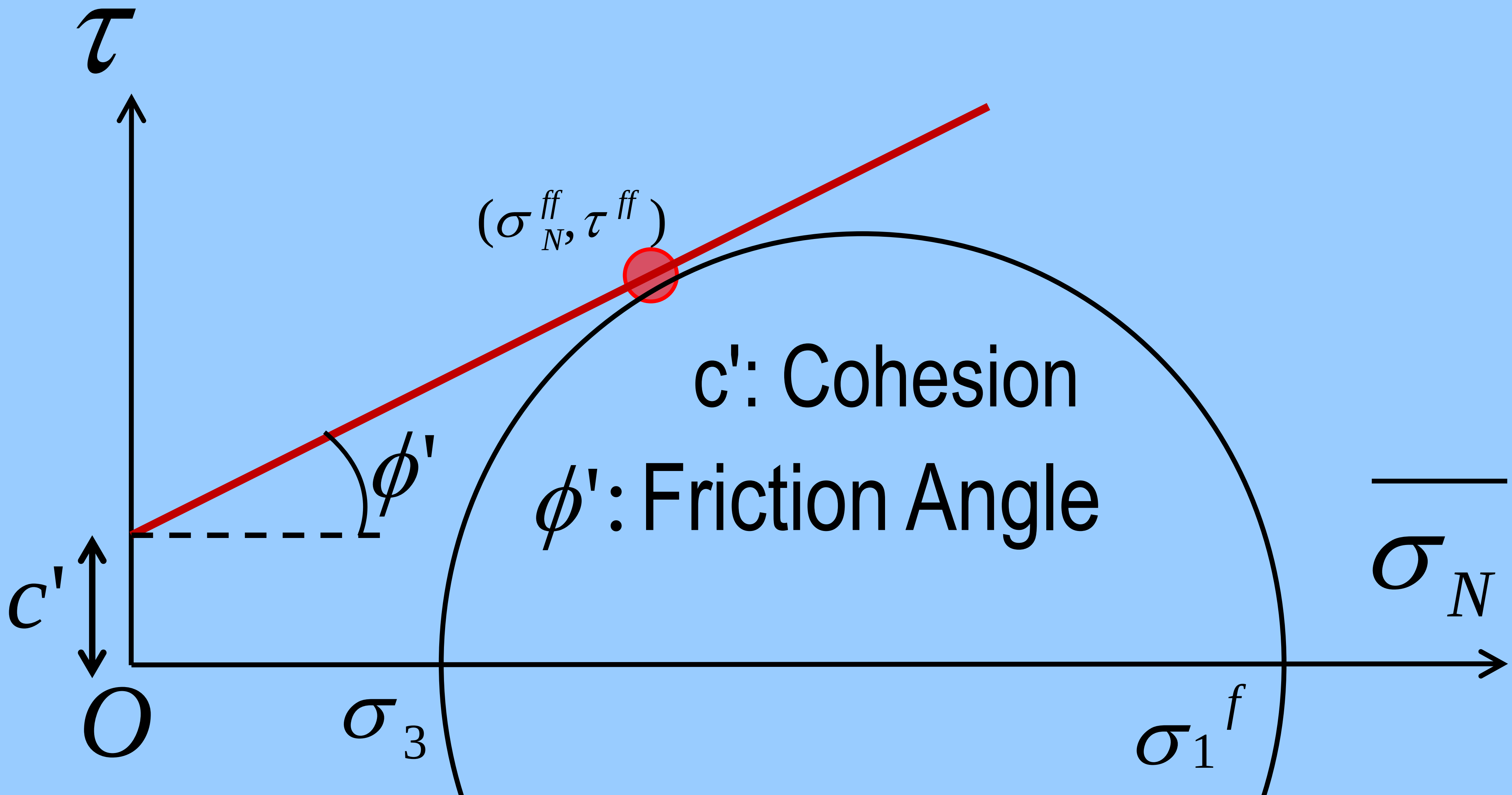
i) c=0, $\Phi = 30deg$

ii) $c=2kN/m^2$, $\Phi = 45deg$

Modified Coulomb Equation

$$\tau = c' + \overline{\sigma}_N \tan(\phi')$$

Effective Stress



Problem

- Determine the shear strength in terms of effective stress on plane within a saturated soil mass at a point where the total normal stress is 200kN/m^2 and the pore water pressure is 80 kN/m^2. The effective shear strength parameters for the soil are

$c' = 2 kN/m^2, \Phi = 30 deg$

Problem

- The stress at failure on a failure plane in a cohesionless soil were Normal Stress=15 kN/m^2, Shear Stress = 6 kN/m^2. Determine the angle of internal friction of soil, major and minor principal stresses.

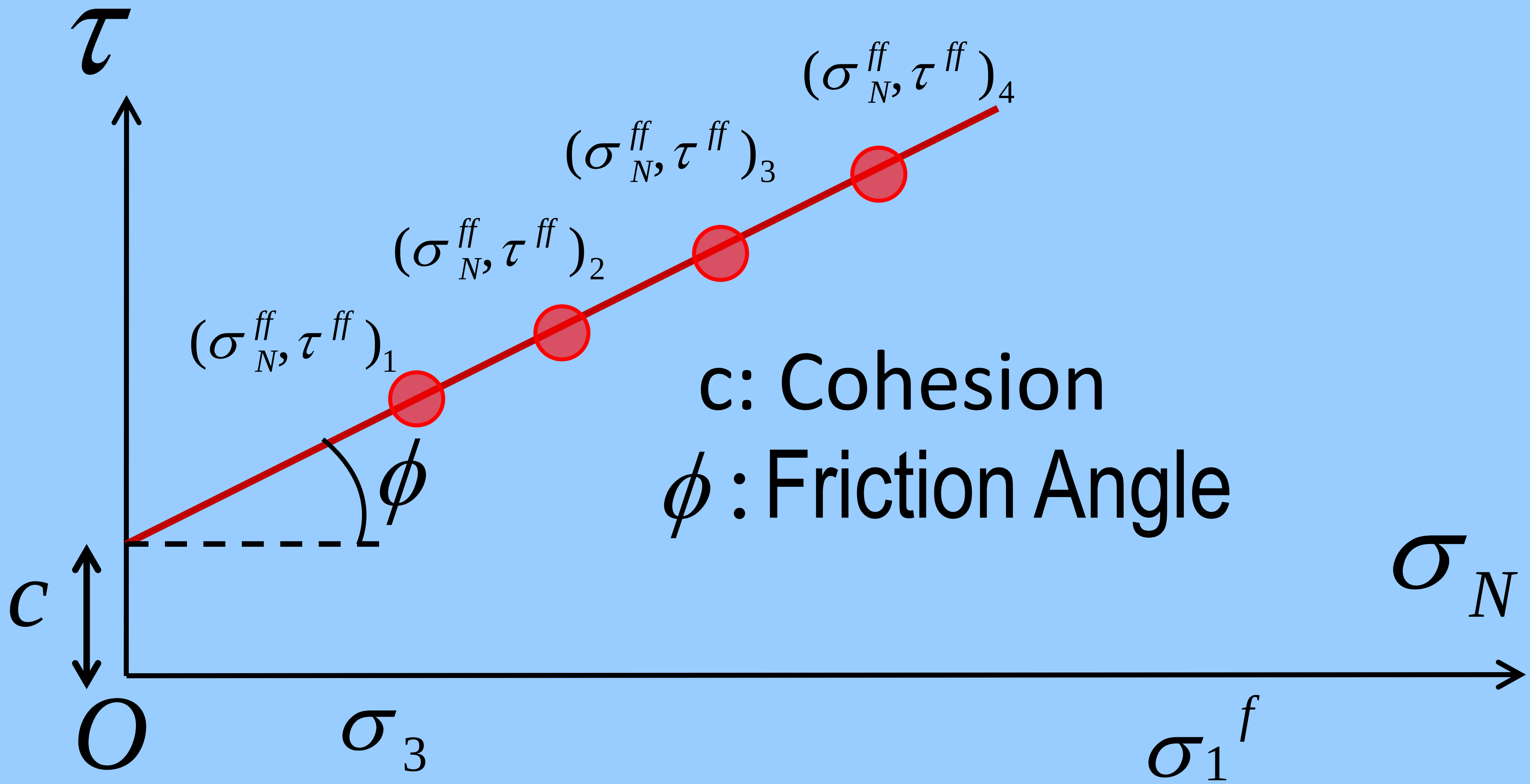
Direct Shear Test

- Metal box of square cross-section split into two halves approximately.
- Shear is applied at constant rate.
- Shear load vs shear deformation for different vertical load. (σ,τ) at failure determined.
- Failure Shear stress for given normal stress determined.
- The test is repeated for different normal stresses.

Direct Shear Test

$$\tau = c + \sigma_N \tan(\phi)$$

Total Stress



Problem

- The following data was obtained from direct shear test. Normal Pressure= 20kN/m^2, Tangential Pressure = 16 kN/m^2. Angle of internal friction = 30^o, cohesion= 4 kN/m^2. Represent the data on the Mohr's circle and compute the principal stresses.

Direct Shear Test

- Advantages
  - Quick, Inexpensive Test.
  - Ease of preparation of sample
- Disadvantages
  - Drainage conditions cannot be controlled
  - Pore water pressure cannot be measured
  - Failure plane is always horizontal
  - Not suitable for fine-grained soils for which drainage conditions play an important role.

Advantages of Triaxial Test

- Most versatile of all shear testing methods.
- Drainage conditions can be controlled..
- Pore water measurements can be made accurately.
- Failure plane is not forced.

Triaxial Test

- Test Procedure
- Types of Triaxial Test
- Unconfined Compression Test

Test Procedure

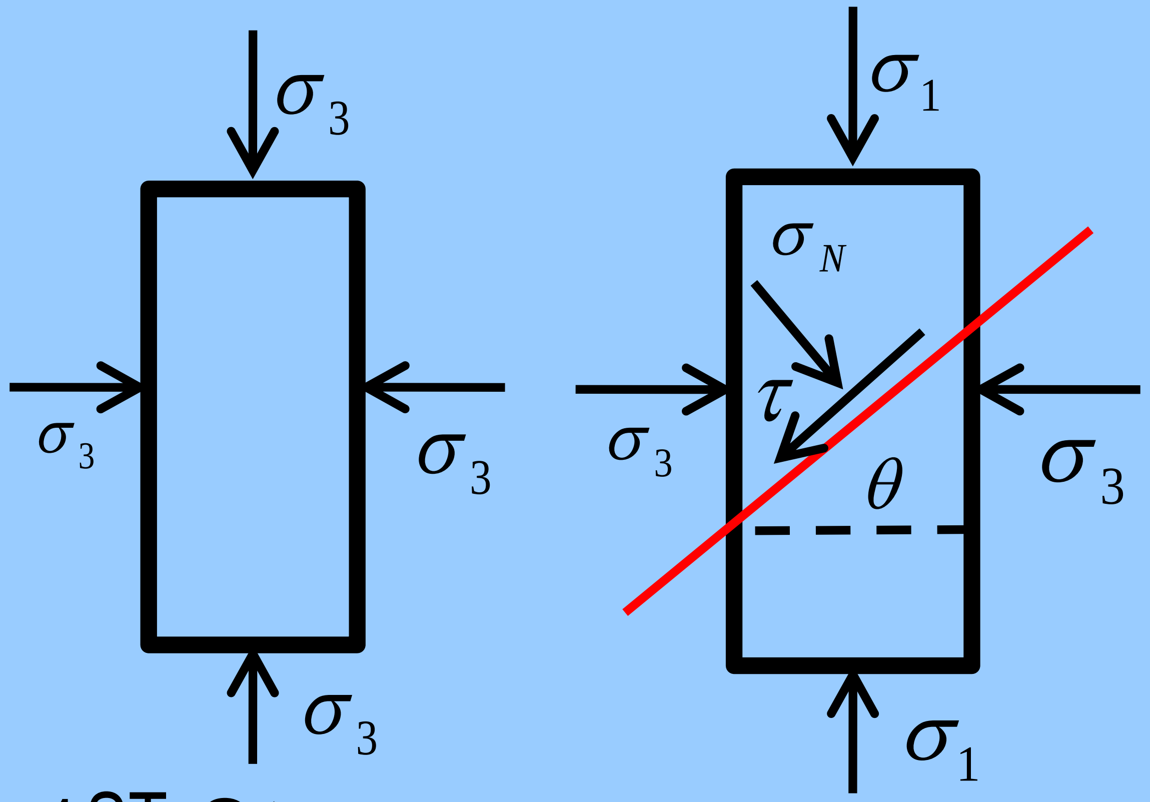
- Cylindrical specimen placed on porous disc resting on pedestal of triaxial cell
- Specimen enclosed in rubber membrane and sealed to the pedestal at the bottom as well as to the loading cap at the top by rubber O-rings.
- Triaxial cell is filled with water at required pressure. This all round pressure is called cell pressure or confining pressure. It acts radially on the vertical surface and on the top and bottom surfaces.
- With Cell pressure held constant, additional axial stress is applied through the ram gradually until the sample fails at an additional axial stress σ_a.

Measurements made in Triaxial Test

- Axial strain is determined by measuring the change in length using either a dial gauge or a displacement transducer.
- Pore water measuring apparatus used to measure pore pressure operated only under conditions of no flow by closing the valve in the drainage line.
- If development of pore water pressure is not allowed, the valve in the pore water line is closed and the valve in the drainage line is opened. Thus allowing flow of water from the soil through the drainage line into a burrette attached to this line. The volume of water expelled from the specimen is measured in burrette.

Stages of Loading

- 1st Stage is Application of Cell pressure
- 2nd Stage is Application of Axial Load



1ST Stage 2nd Stage

Types of Triaxial Test

	1 st Stage (Application of Cell Pressure)	2nd Stage (Application of Axial Load to Failure)
UU Test	No Drainage, No Consolidation UNCONSOLIDATED	No Drainage UNDRAINED
CU Test	Drainage Allowed Consolidation takes place CONSOLIDATED	No Drainage UNDRAINED
CD Test	Drainage Allowed Consolidation takes place CONSOLIDATED	Drainage Allowed DRAINED

Types of Triaxial Test

- UU: Unconsolidated Undrained Test
  - Takes 15 minutes to complete
  - Enables determination of Cu, *Φu*.
- CU: Consolidated Undrained Test
  - Measures total stress parameters C_{cu}, Φ_{cu} and effective stress parameters C'_{cu}, Φ'_{cu} .
  - Takes 24 hours for consolidation 1st stage and 2hour for axial loading to failure 2nd stage.

Types of Triaxial Test

- CD: Consolidated Drained Test
  - Measures effective stress parameters C'_{cu}, Φ'_{cu} .
  - Very Slow and takes upto 2 weeks
- Unconfined Compression Test
  - Cell pressure applied is zero. First stage loading is zero.
  - $C_u = q_u/2$
  - $\Phi_u = 0$.

Vane Shear Test

- Why is it used?
  - For measuring shear resistance of soft saturated clay deposits in the field directly by pushing the vane directly into the soil upto required depth and applying torque.
  - This overcomes the difficulty of obtaining undisturbed soil sample whose shear strength may alter significantly during the process of sampling and handing.

Problem

- In an in situ vane shear test on a saturated clay, a torque of 35 N.m was required to shear the soil. The diameter of the vane was 50mm and length 100mm. Calculate the undrained shear strength of clay.

- The vane was rotated rapidly to cause remoulding of soil. The torque required to shear the soil in remoulded state was 5kNm. Determine the sensitivity of clay.