

Geotechnical Engineering-I

Lecture 1 & 2

**Presented by
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Civil engineering

- Civil engineering is the oldest engineering discipline after military engineering, and it was defined to distinguish non-military engineering from military engineering.
- Civil engineering is a professional engineering discipline that deals with the design, construction, and maintenance of the physical and naturally built environment, including works like roads, bridges, canals, dams, and buildings



Specialization of Civil engineering

Civil Engineering

Structure Engineering

Geotechnical Engineering

Transportation Engineering

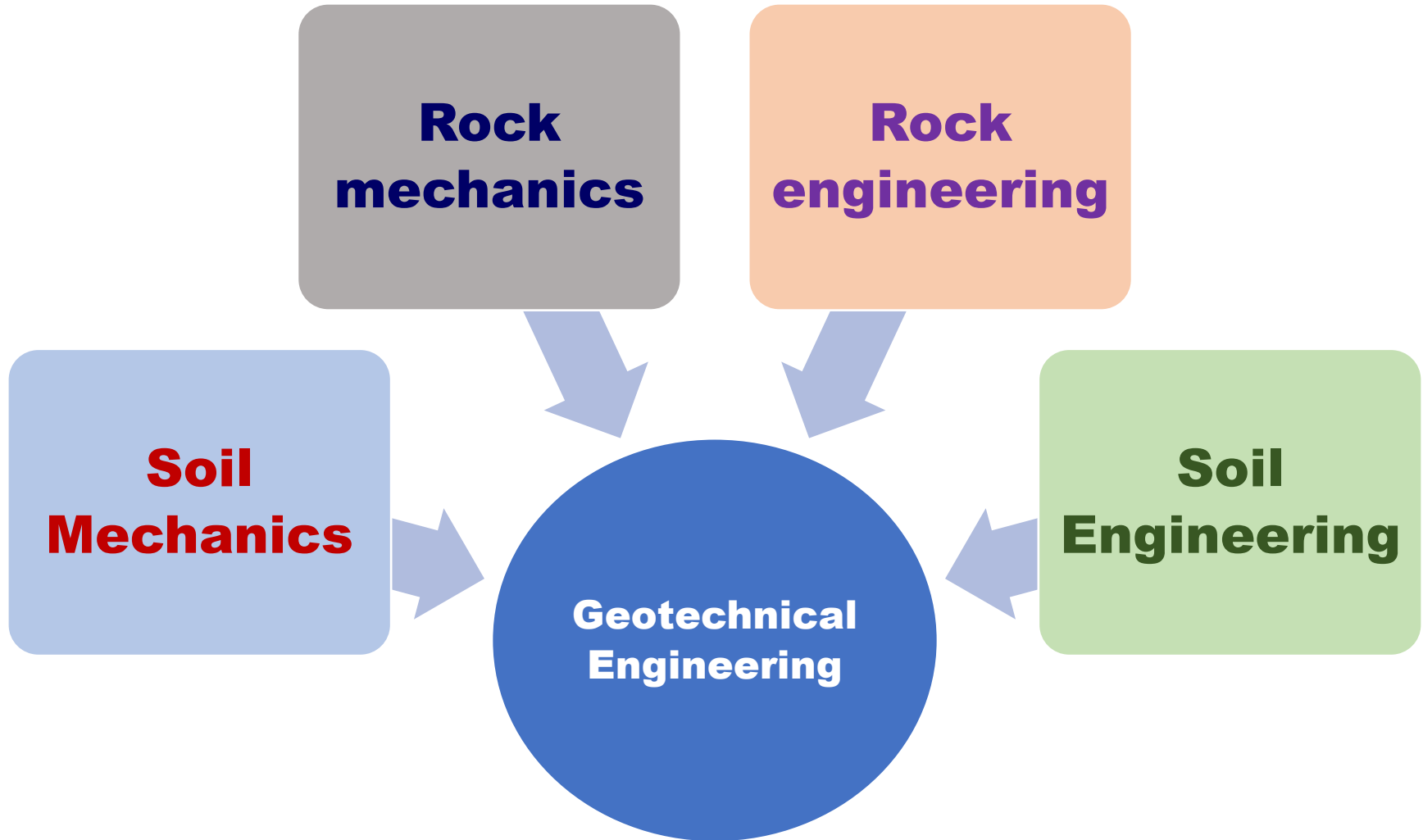
Environmental Engineering

Water resource engineering

Construction Planning and Management



Geotechnical Engineering



GEOTECHNICAL ENGINEERS AT WORK

BUILDINGS

- High-rises
- Hospitals
- Schools
- Stadiums



TRANSPORT

- Roads
- Railways
- Airports
- Tunnels



WATER RESOURCES

- Dams
- Canals
- Levees
- Reservoirs



ENERGY

- Wind farms
- Solar farms
- Nuclear plants
- Pipelines



ENVIRONMENT

- Landfills
- Contaminated site cleanup
- Tailings dams
- Mining



COASTAL

- Sea walls
- Ports
- Jetties
- Reclamation
- Offshore wind



INDIAN CONTEXT:



- **Metro rails** (Delhi, Mumbai, Bengaluru, Kolkata, Chennai)



- **Bullet train** (Ahmedabad-Mumbai corridor)



- **Char Dham Highway** (Uttarakhand – challenging Himalayan soil)



- **Bharatmala Pariyojana** (nationwide highway development)



- **Sagarmala** (port-led development along coastline)



Application
of Geotechnical engineering

Buildings and Foundation

Application Area	Major Geotechnical Challenges	Soil Mechanics Concepts Involved
High-rise buildings	Weak foundation soil	Bearing capacity
Hospitals and schools	Uneven settlement and cracking	Consolidation and settlement
Basements and underground parking	Groundwater seepage	Permeability and seepage
Urban construction sites	Excavation collapse	Shear strength
Earthquake-prone areas	Liquefaction risk	Soil dynamics
Large commercial complexes	Load transfer to soil	Stress distribution

Transportation infrastructure

Application Area	Major Geotechnical Challenges	Soil Mechanics Concepts Involved
Roads and highways	Weak subgrade and rutting	Compaction and CBR
Railways and metro tracks	Track settlement	Consolidation
Hill roads and embankments	Landslides and slope failure	Shear strength
Airports	Heavy repeated loading	Cyclic loading behavior
Tunnels	Ground collapse during excavation	Earth pressure theory
Drainage systems	Waterlogging and reduced strength	Permeability

Water Resource Engineering

Application Area	Major Geotechnical Challenges	Soil Mechanics Concepts Involved
Dams	Seepage through embankment	Permeability
Canals	Piping and erosion	Seepage pressure
Reservoirs	Foundation instability	Bearing capacity
Flood embankments	Slope failure	Shear strength
Hydraulic structures	Excess pore water pressure	Effective stress
Earthen dams	Internal erosion	Soil-water interaction

Energy Infrastructure

Application Area	Major Geotechnical Challenges	Soil Mechanics Concepts Involved
Wind farms	Heavy tower loading	Bearing capacity
Solar farms	Differential settlement	Compressibility
Nuclear plants	Dynamic machine loading	Soil dynamics
Pipelines	Soil movement and instability	Lateral earth pressure
Transmission towers	Foundation uplift	Shear strength
Remote energy projects	Difficult ground conditions	Soil investigation

Environmental geotechnics

Application Area	Major Geotechnical Challenges	Soil Mechanics Concepts Involved
Landfills	Long-term waste settlement	Consolidation
Contaminated sites	Pollutant migration	Permeability
Tailings dams	Embankment failure	Shear strength
Mining areas	Slope instability	Slope stability
Waste containment systems	Leakage prevention	Seepage analysis
Remediation projects	Soil protection	Compacted soil behavior

Coastal and Offshore Engineering

Application Area	Major Geotechnical Challenges	Soil Mechanics Concepts Involved
Sea walls	Wave-induced erosion	Soil erosion
Ports and harbors	Soft marine clay	Compressibility
Jetties	Scour around foundations	Seepage and erosion
Land reclamation	Long-term settlement	Consolidation
Offshore wind farms	Cyclic wave loading	Soil dynamics
Coastal foundations	Weak saturated soil	Bearing capacity

Case Study NH-66 Collapse, Kollam, Kerala (December 2025)

The incident:

A 4-5 meter high embankment on National Highway 66 collapsed at Mylakkadu near Kottiyam. The entire fill section failed catastrophically.

Geotechnical cause:

NHAI assessments confirmed a "**deep-seated shear/bearing capacity failure**" – the soft, compressible marine clay foundation was too weak to support the embankment load. The fill added 10-11 tonnes per square meter, far exceeding the soil's shear strength.

The critical error:

The contractor proceeded without adequate geotechnical investigation. Residents had repeatedly warned authorities that earth-fill embankments were unsuitable for Kerala's monsoon-prone wetlands with soft alluvial deposits



Case Study Land Subsidence, Baddi-Ramshehar Belt, Himachal Pradesh (2023-2025)

The incident

Since 2023, recurring land subsidence has damaged 70+ houses in Sheel Sunani village, followed by 8 more houses in Jamun Da Dohra village. In Ramshehar, 60 villagers were rendered homeless when debris flowed down the hill.

Geotechnical cause

CSIR geotechnical investigations confirmed that **friable rocks and weak soil formations** in the area could not withstand the record rainfall (325 mm in July, 308 mm in August). The region's net safe bearing capacity was grossly overestimated.

The consequence

Villages have been declared "sinking zones" and brought under the Adarsh Resilient Village Programme for rehabilitation





Flood Damage & Embankment
Breaches, North Bengal
(October 2025)



Flash Flood & Debris Flow, Dharali,
Uttarakhand (August 2025)

Some other worldwide cases

Failure

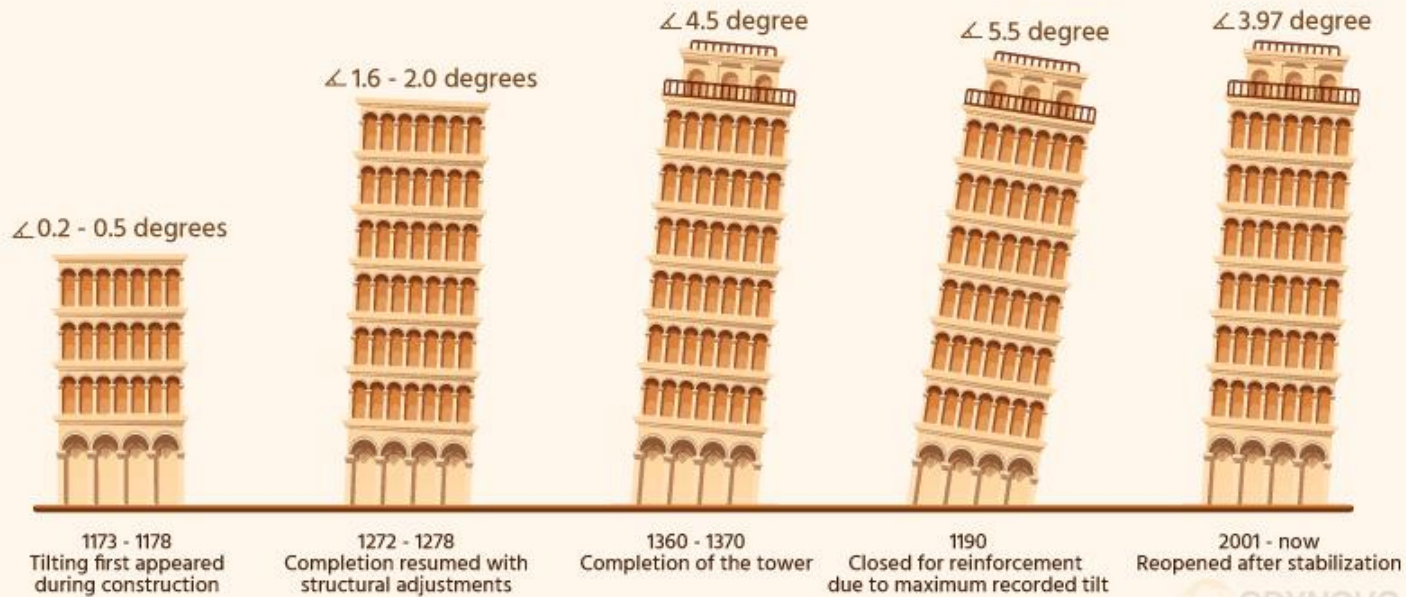
St. Francis
Dam

Teton Dam

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Leaning T

From 0.5° to 5.5° – The Leaning Tower of Pisa's Timeline



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Syllabus of Geotechnical Engineering I

Course code- 101403 (3 0 0 3)

Unit- 1.0:

7hrs

Introduction–Types of soils, their formation and deposition, Definitions: soil mechanics, soil engineering, rock mechanics, geotechnical engineering. Scope of soil engineering. Comparison and difference between soil and rock. Basic Definitions and Relationships-Soil as three-phase system in terms of weight, volume, voids ratio, and porosity. Definitions: moisture content, unit weights, degree of saturation, voids ratio, porosity, specific gravity, mass specific gravity, etc. Relationship between volume weight, voids ratio- moisture content, unit weight- percent air voids, saturation- moisture content, moisture content specific gravity etc. Determination of various parameters such as: Moisture content by oven dry method, pycnometer, sand bath method, torsional balance method, nuclear method, alcohol method and sensors. Specific gravity by density bottle method, pycnometer method, measuring flask method. Unit weight by water displacement method, submerged weight method, core-cutter method, sand-replacement method.

On completion of this Unit-, the students must be able to:

- Understand the different types of soil based on their formation mechanism;
- Understand the various phase diagrams and derive various phase relationships of the soil; Perform various laboratory experiments to determine moisture content, specific gravity; Perform field experiments to estimate the field density of the soil mass

Unit- 2.0

7hrs

Plasticity Characteristics of Soil - Introduction to definitions of: plasticity of soil, consistency limits-liquid limit, plastic limit, shrinkage limit, plasticity, liquidity and consistency indices, flow & toughness indices, definitions of activity and sensitivity. Determination of: liquid limit, plastic limit and shrinkage limit. Use of consistency limits. Classification of Soils-Introduction of soil classification: particle size classification, textural classification, unified soil classification system, Indian standard soil classification system. Identification: field identification of soils, general characteristics of soil in different groups.

On completion of this Unit-, the students must be able to:

- Understand the behaviour of soils based on their moisture contents;
- Perform laboratory experiments to estimate various Atterberg limits and evaluate index properties of soils;
- Classify any soils based on their particle size distribution and index properties

Unit- 3.0

7hrs

Permeability of Soil - Darcy's law, validity of Darcy's law. Determination of coefficient of permeability: Laboratory method: constant-head method, falling-head method. Field method: pumping- in test, pumping- out test. Permeability aspects: permeability of stratified soils, factors affecting permeability of soil. Seepage Analysis- Introduction, stream and potential functions, characteristics of flow nets, graphical method to plot flow nets.

On completion of this Unit-, the student must be able to:

- Determine the permeability of soils through various laboratory and field tests;
- Analytically calculate the effective permeability of anisotropic soil mass;
- Determine the seepage quantities and pore water pressures below the ground;
- Graphically plot the equipotential lines and flow lines in a seepage flow

Unit- 4.0

7hrs

Effective Stress Principle - Introduction, effective stress principle, nature of effective stress, effect of water table. Fluctuations of effective stress, effective stress in soils saturated by capillary action, seepage pressure, quick sand condition.

Unit- 5.0

7hrs

Compaction of Soil-Introduction, theory of compaction, laboratory determination of optimum moisture content and maximum dry density. Compaction in field, compaction specifications and field control.

Unit- 6.0

7hrs

Stresses in soils – Introduction, stresses due to point load, line load, strip load, uniformly loaded circular area, rectangular loaded area. Influence factors, Isobars, Boussinesq's equation, Newmark's Influence Chart. Contact pressure under rigid and flexible area, computation of displacements from elastic theory. On completion of this Unit-, the student must be able to:

- Analytically compute the vertical stress in a semi-infinite soil mass due to various loading conditions; Plot isobars due various loading conditions.

Course Outcomes

1. Evaluate physical and basic properties of the soil.
2. Evaluate the plasticity behaviour of the soil
3. Apply concepts of the permeability behaviour of soil
4. Apply concepts of compaction in different civil engineering applications
5. Apply the concepts of effective stress in field conditions.
6. Calculate the stresses due to the external load on the soil

Text books/References

1. Soil Mechanics by Craig R.F., Chapman & Hall
2. Fundamentals of Soil Engineering by Taylor, John Wiley & Sons
3. An Introduction to Geotechnical Engineering, by Holtz R.D. and Kovacs, W.D., Prentice Hall, NJ
4. Principles of Geotechnical Engineering, by Braja M. Das, Cengage Learning
5. Principles of Foundation Engineering, by Braja M. Das, Cengage Learning
6. Essentials of Soil Mechanics and Foundations: Basic Geotechnics by David F. McCarthy
7. Soil Mechanics in Engineering Practice by Karl Terzaghi, Ralph B. Peck, and Gholamreza Mesri.
8. Geotechnical Engineering: Principles and Practices of Soil Mechanics and Foundation Engineering (Civil and Environmental Engineering) by V.N.S. Murthy.

Soil

- Soil is derived from latin word solium.
- The upper layer of the earth that may be dug or plowed, specially the loose surface material of earth in which plants grow. (Agronomy)
- The upper layer of earth crust is assumed to be consist of consolidated sediments, which support plants. (Geology)
- All natural occuring, relative unconsolidated earth material-organic or inorganic character that lies above bed rock. (civil engineering)

FORMATION OF SOIL

Soil is formed by the breakdown of rocks (parent material) through physical and chemical processes, along with the action of living organisms, water, air and time.

SOIL FORMATION PROCESS



FACTORS

- Climate
- Organisms
- Relief (Topography)
- Parent Material
- Time

1. MECHANICAL DISINTEGRATION (PHYSICAL WEATHERING)

Breakdown of rocks into smaller pieces without changing their chemical composition.

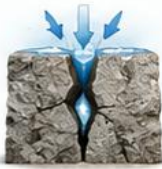
TEMPERATURE VARIATION

Heating and cooling causes expansion and contraction of rocks.



FREEZE-THAW ACTION

Water enters cracks, freezes, expands and widens the cracks.



SALT CRYSTALLIZATION

Salt crystals grow in cracks and break the rock.



ABRASION

Wind, water, ice or waves wear away rock surfaces.



PRESSURE RELIEF

Removal of overlying pressure causes rocks to expand and crack.



Parent rock



Large fragments



Small fragments



Fine particles

RESULT

Rocks are broken into smaller pieces (mechanically), increasing the surface area for chemical action.

2. CHEMICAL DECOMPOSITION (CHEMICAL WEATHERING)

Minerals in rocks are changed into new substances by chemical reactions.

HYDRATION

Minerals absorb water and swell, breaking the rock.

Example:
Anhydrite → Gypsum



Slightly weathered rock

HYDROLYSIS

Reaction with water changes minerals into clay minerals and releases other ions.

Example:
Feldspar → Clay



Highly weathered rock

OXIDATION

Minerals react with oxygen, forming oxides (rust).

Example:
Iron → Iron oxide



Mineral alteration

CARBONATION

Carbonic acid in water reacts with minerals, dissolving them.

Example:
Calcite → Calcium bicarbonate



DISSOLUTION

Soluble minerals dissolve in water and are removed.

Example:
Halite dissolves in water



Soil



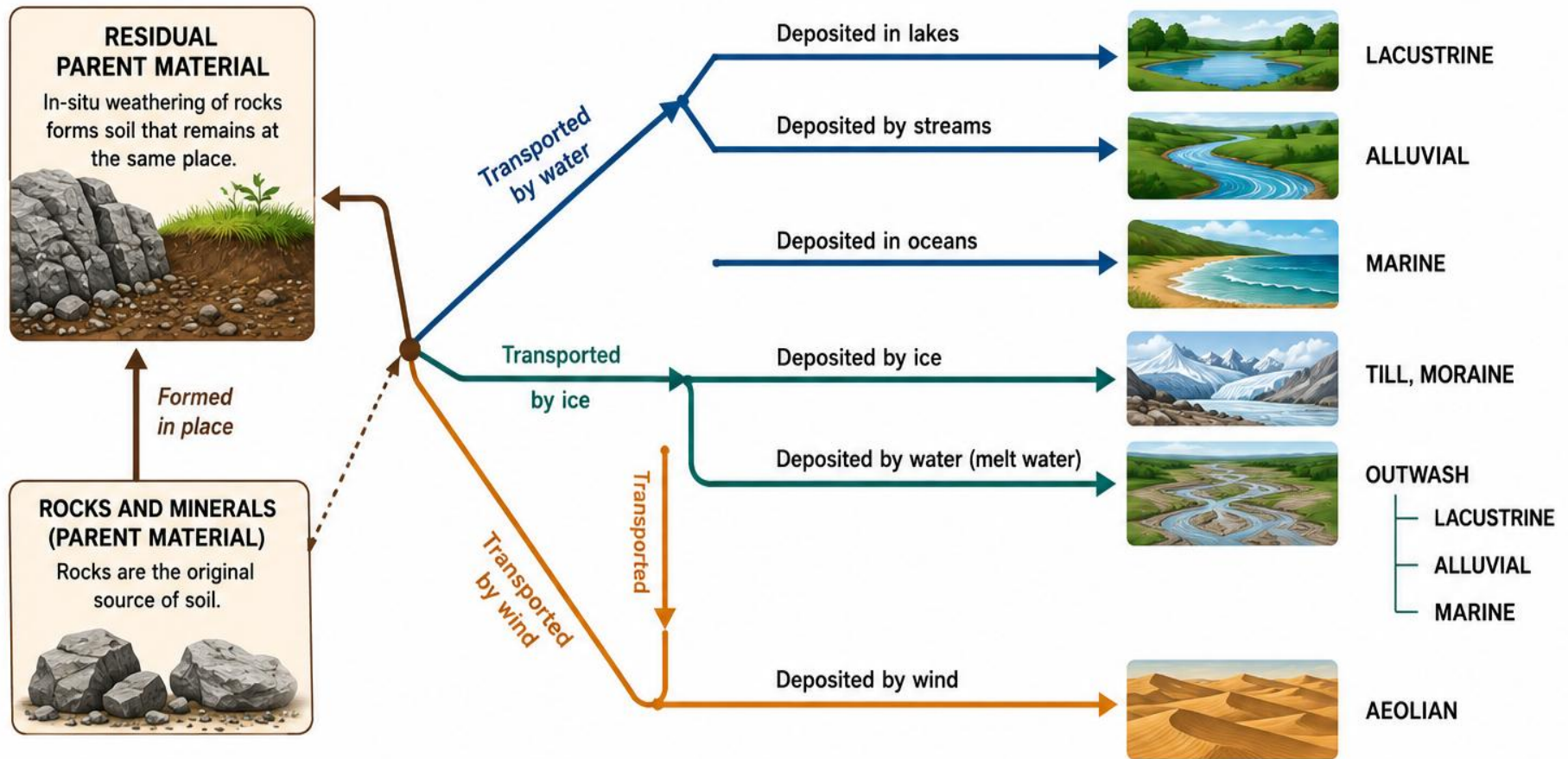
RESULT

Minerals are decomposed and transformed into new materials; soil becomes finer and more stable.

The combined action of MECHANICAL DISINTEGRATION and CHEMICAL DECOMPOSITION, along with living organisms, produces SOIL over a long period of time.

ORIGIN OF SOIL

Soil originates from the breakdown of rocks (parent material) and is transported and deposited by natural agents to form different types of soil.



KEY POINTS



Soil is formed by the weathering of rocks (residual) or by deposition of transported materials.



Water, ice and wind are the major natural agents that transport and deposit soil particles.



Different modes of transport and deposition result in different soil types.



These soils further evolve over time with the help of climate, organisms, topography and time.

TYPES OF SOIL IN INDIA

Different regions of India have different types of soil based on their origin, climate, topography and drainage.

ALLUVIAL SOIL

Found in the Indo-Gangetic plains, Brahmaputra valley, coastal plains and river deltas.

Regions: Punjab, Haryana, Uttar Pradesh, Bihar, West Bengal, Assam, Odisha, Coastal Andhra Pradesh, Tamil Nadu deltas

BLACK SOIL (REGUR SOIL)

Formed from weathering of basaltic rocks, rich in clay.

Regions: Maharashtra, Madhya Pradesh, Gujarat, Telangana, Karnataka (parts), Tamil Nadu (parts)

RED & YELLOW SOIL

Developed from igneous and metamorphic rocks, red due to iron oxide.

Regions: Tamil Nadu, Karnataka, Andhra Pradesh, Telangana, Odisha, Chhattisgarh, Jharkhand, parts of Maharashtra and MP

LATERITE SOIL

Formed under high rainfall and high temperature, leached and acidic.

Regions: Western Ghats, Kerala, Karnataka, Goa, Odisha, Assam, West Bengal, North-Eastern States

OTHER IMPORTANT SOILS

LITHOSOLS

Very shallow soils over hard rocks.

Regions: Hilly and mountainous areas

GREY SOIL

Formed in areas of low rainfall and poor drainage.

Regions: Parts of Punjab, Haryana, Uttar Pradesh, Rajasthan

MAJOR TYPES OF SOIL

- Alluvial Soil
- Black Soil (Regur)
- Red & Yellow Soil
- Laterite Soil
- Arid Soil
- Forest & Mountain Soil
- Peaty & Marshy Soil
- Saline & Alkaline Soil
- Others / Mixed

ARID SOIL (DESERT SOIL)

Sandy, low in organic matter and humus, formed in arid climate.

Regions: Rajasthan, parts of Gujarat, Haryana, Punjab

FOREST & MOUNTAIN SOIL

Shallow, loamy, rich in organic matter in forested and hilly areas.

Regions: Himalayan region, North-East India, Western Ghats, hilly areas of Uttarakhand, Himachal Pradesh, Jammu & Kashmir

PEATY & MARSHY SOIL

Formed in waterlogged and marshy areas, high in organic matter.

Regions: Sundarbans (West Bengal), Kuttanad (Kerala), Terai regions, some parts of Assam

SALINE & ALKALINE SOIL

Contains excess salts and alkalis, often in arid and semi-arid regions.

Regions: Gujarat (Rann of Kutch), Rajasthan, Haryana, Uttar Pradesh (some parts), coastal areas

KEY TAKEAWAYS



Soil types vary widely across India due to climate, relief, parent material and drainage.



Alluvial soils are fertile and support major agriculture in the northern plains.



High rainfall areas have laterite and forest soils.



Arid and coastal regions have arid, saline and alkaline soils.

Soil Type	Geotechnical Characteristics	Engineering Problems	Suitable Engineering Applications
Alluvial Soil	Mostly sandy to silty; moderately compressible; variable density	Settlement under heavy loads; liquefaction in loose deposits	Foundations, embankments, highways after compaction
Black Cotton Soil (Regur Soil)	Highly expansive clay; high shrink-swell behavior; low permeability	Severe swelling and shrinkage causing foundation cracks	Requires soil stabilization and special foundations
Red & Yellow Soil	Sandy to clayey; moderate bearing capacity; porous	Erosion and low cohesion in dry state	Roads, light structures, embankments
Laterite Soil	Porous and heavily weathered; low fertility; hardens on exposure	High compressibility when wet; erosion problems	Road subgrade, building blocks after treatment
Arid/Desert Soil	Loose sandy soil; low cohesion; high permeability	Wind erosion, dune movement, low bearing capacity	Pavement works with stabilization
Forest & Mountain Soil	Shallow soil; high organic content; unstable on slopes	Landslides and slope instability	Hill roads with retaining structures
Peaty & Marshy Soil	Very high compressibility; high water content; low strength	Excessive settlement and low bearing capacity	Usually avoided or improved before construction
Saline & Alkaline Soil	Presence of salts affects soil behavior and durability	Corrosion of foundations; reduced strength	Requires chemical treatment and drainage control
Marine Soil	Soft clayey deposits; highly compressible; low shear strength	Large settlement and stability issues	Port and coastal structures with deep foundations
Glacial Soil	Mixed particle sizes; irregular deposits	Unpredictable engineering behavior	Limited construction after investigation
Aeolian Soil	Fine sand deposits transported by wind; loose structure	Wind erosion and instability	Stabilized foundations and pavements
Colluvial Soil	Heterogeneous and loosely deposited	Slope failure and landslides	Requires detailed slope stability analysis

Thank You

Geotechnical Engineering-I

Lecture 3 & 4

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DETERMINATION OF SPECIFIC GRAVITY OF SOIL

PYCNOMETER METHOD & DENSITY BOTTLE METHOD

1. PYCNOMETER METHOD

APPARATUS

- Pycnometer (with capillary hole in stopper)
- Balance (accuracy 0.01 g)
- Oven
- Thermometer
- Distilled water
- Funnel
- Tissue / Filter paper



Density bottle



Oven dried soil

PROCEDURE

- 1 Clean and dry the pycnometer.
- 2 Weigh the empty pycnometer with stopper (W_1).
- 3 Fill the pycnometer with distilled water up to the capillary hole in the stopper. Wipe the outside and weigh (W_2).
- 4 Take about 50 g of oven dried soil and weigh (W_3).
- 5 Put the soil in the pycnometer and add distilled water. Remove entrapped air by gentle shaking and tapping.
- 6 Fill with water up to the capillary hole. Wipe and weigh (W_4).
- 7 Determine temperature and note.

PRECAUTIONS

- Fill water up to the capillary hole accurately.
- Remove all air bubbles from soil.
- Maintain constant temperature ($\pm 1^\circ\text{C}$).

CALCULATION

Specific gravity of soil solids, $G_s =$

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

where,

- W_1 = mass of empty pycnometer (g)
 W_2 = mass of pycnometer + water (g)
 W_3 = mass of pycnometer + dry soil (g)
 W_4 = mass of pycnometer + dry soil + water (g)

TYPICAL VALUES

Specific gravity of most mineral soils ranges from 2.60 to 2.75.

2. DENSITY BOTTLE METHOD

APPARATUS

- Density bottle (50 or 100 ml)
- Balance (accuracy 0.01 g)
- Oven
- Distilled water
- Funnel
- Thermometer
- Tissue / Filter paper



Pycnometer



Oven dried soil

PROCEDURE

- 1 Clean and dry the density bottle.
- 2 Weigh the empty bottle with stopper (W_1).
- 3 Fill the bottle with distilled water, remove air bubbles, insert stopper, wipe outside and weigh (W_2).
- 4 Take about 25–30 g of oven dried soil and weigh (W_3).
- 5 Put the soil in the bottle and add water. Remove entrapped air by shaking.
- 6 Fill the bottle completely with water, insert stopper, wipe outside and weigh (W_4).
- 7 Note the temperature.

PRECAUTIONS

- Fill completely with water; no air bubbles.
- Wipe outside surface dry before weighing.
- Maintain constant temperature ($\pm 1^\circ\text{C}$).

CALCULATION

Specific gravity of soil solids, $G_s =$

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

where,

- W_1 = mass of empty density bottle (g)
 W_2 = mass of bottle + water (g)
 W_3 = mass of bottle + dry soil (g)
 W_4 = mass of bottle + dry soil + water (g)

TYPICAL VALUES

Specific gravity of most mineral soils ranges from 2.60 to 2.75.

NOTE: Use distilled water and maintain the same temperature for all weighings.

G_s = Specific gravity of soil solids (dimensionless)

Temperature correction for water density may be applied if temperature differs from 27°C significantly.

DIFFERENT METHODS OF DETERMINATION OF WATER CONTENT

1. OVEN DRYING METHOD



1. Weigh wet soil (W_1)
2. Dry in oven at 105–110°C
3. Cool in desiccator and weigh (W_2)

Water content, $w = \frac{W_1 - W_2}{W_2} \times 100\%$
Most common and standard method.

2. TORSION BALANCE METHOD



The torque required to twist a wire is proportional to the weight loss due to drying of soil.

3. PYCNOMETER METHOD



1. Weigh empty pycnometer (W_1)
2. Add wet soil and water, weigh (W_2)
3. Fill with water, weigh (W_3)

Based on the specific gravity of soil particles to determine water content.

4. SAND BATH METHOD



The soil sample is heated in a sand bath and loss in weight gives the water content.

5. ALCOHOL METHOD



1. Mix soil with methylated spirit
2. Shake thoroughly
3. Allow to stand and decant
4. Determine water content from volume/weight relationship

Alcohol displaces water; the amount of water is determined by difference.

6. CALCIUM CARBIDE METHOD



1. Add CaC_2 to the sample
2. Water reacts to form acetylene gas
3. Pressure developed is proportional to water content

Quick method based on the reaction of water with calcium carbide.

7. RADIATION METHOD



Utilizes gamma rays to determine water content based on the scattering/absorption by hydrogen atoms in water.



Note: Selection of method depends on accuracy required, type of soil, available equipment, time and purpose of test.

OVEN DRYING METHOD

Determination of Water Content in Soil

PRINCIPLE

A known mass of moist soil is dried in an oven at 105–110°C until a constant mass is obtained. The loss in mass is considered as the water content.



APPARATUS

- Oven (105–110°C)
- Weighing balance (accuracy 0.01 g)
- Moisture cans with lids
- Desiccator
- Tongs / forceps
- Spatula

PROCEDURE

- 1 Clean and dry the moisture can. Weigh it with lid and record the mass (W_1).
- 2 Place a representative soil sample in the can and weigh with lid. Record the mass (W_2).
- 3 Remove the lid and place the can in the oven maintained at 105–110°C.
- 4 Dry for about 24 hours.
- 5 Remove the can, cover with lid and place in desiccator to cool to room temperature.
- 6 Weigh the can with dry soil and lid. Record the mass (W_3).
- 7 Repeat drying for 1–2 hours, cool and weigh again until the difference between two successive masses is less than 0.1 g (constant mass).

DEFINITION

Water content (w) is the ratio of mass of water in the soil to mass of dry soil, expressed as a percentage.

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

PRECAUTIONS

- Use a representative soil sample.
- Maintain oven temperature at 105–110°C.
- Cool the sample in desiccator to avoid absorption of moisture.
- Dry to constant mass for accurate results.

CALCULATION STEPS

- 1 Calculate mass of water (loss in mass):

$$M_w = W_2 - W_3$$

- 2 Calculate mass of dry soil:

$$M_d = W_3 - W_1$$

- 3 Calculate water content (w):

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

or

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

W_1 = mass of empty can with lid (g)

W_2 = mass of can with wet soil and lid (g)

W_3 = mass of can with dry soil and lid (g)

APPLICATION

Most common and standard method for laboratory determination of water content.

DETERMINATION OF WATER CONTENT BY TORSION BALANCE TEST

As per IS 2720 (Part II) – 1973, Section 4

1. TITLE / INTRODUCTION

This method provides rapid determination of water content of soil using an infra-red lamp torsion balance moisture meter. The meter gives percentage water on wet basis directly from the scale, which is then converted to water content on dry basis.

2. APPARATUS



- Infra-red lamp torsion balance moisture meter
- Palette knife / steel spatula
- Thermometer (40°C to 150°C)
- Suitable container for keeping sample

Instrument includes infra-red lamp, torsion balance, calibrated drum, pointer, variac heat control and magnetic damper.

3. SOIL SPECIMEN



- Representative soil sample
- Sample mass: about 25 g
- Maximum particle size: less than 2 mm

4. PRINCIPLE

A moist soil specimen is dried by infra-red heating in the torsion balance moisture meter. Loss of water causes a change in balance, and the calibrated drum gives percentage water on wet basis (m). This is converted to water content on dry basis (w).

5. PROCEDURE

1



Keep the test sample in a suitable container so that moisture is not lost.

2



Set the 100% scale division of the calibrated drum in line with the index mark.

3



With the empty pan on the pivot, adjust the pointer to the index line using the initial adjustment knob.

4



Rotate the drum drive knob anticlockwise to bring the 0% scale division in line with the index mark.

5



Raise the lamp housing and place the soil evenly on the sample pan until the pointer returns to the index mark (about 25 g).

6



Lower the lamp housing, switch on the infra-red lamp, insert the thermometer, and adjust the variac so that drying temperature is about $110 \pm 5^\circ\text{C}$.

7



As drying proceeds, rotate the drum drive knob to bring the pointer back to the index. When the pointer remains steady, note the final reading m on wet basis.

6. FORMULA FOR CALCULATION

Meter reading is on wet basis.

$$\text{Water content on dry basis: } w = \frac{m}{100 - m} \times 100$$

m = water content read directly from moisture balance scale on wet basis (%)

w = water content on dry basis (%)

7. EXAMPLE CALCULATION

Given meter reading, $m = 18\%$

$$w = \frac{18}{100 - 18} \times 100$$

$$w = \frac{18}{82} \times 100 = 21.95\%$$

Result: Water content, $w = 21.95\%$ ($\approx 22\%$)

8. NOTES / PRECAUTIONS

- ✓ Maintain drying temperature around $110 \pm 5^\circ\text{C}$.
- ✓ Spread soil uniformly in the pan.
- ✓ Use clean and cool pan for each test.
- ✓ Final reading is taken only when the pointer remains steady.
- ✓ This is a rapid method; oven drying is the standard method.



Rapid, convenient and suitable for field and laboratory use.

DETERMINATION OF WATER CONTENT BY PYCNOMETER METHOD

(Using Known Specific Gravity, G)



For a moist soil sample of known specific gravity G , water content is determined from pycnometer weighings.

APPARATUS

- pycnometer with stopper
- balance
- distilled water
- drying oven
- tray/spatula

PROCEDURE

- 1 Weigh empty pycnometer = W_1
- 2 Place moist soil in pycnometer and weigh = W_2
- 3 Add water to the moist soil in the pycnometer, remove air bubbles, fill to the mark, and weigh = W_3
- 4 Fill the clean pycnometer completely with water and weigh = W_4
- 5 Take specific gravity of soil solids as known, G

FORMULA FOR CALCULATION

$$w = \left[\left(\frac{W_2 - W_1}{W_3 - W_4} \right) \times \left(\frac{G - 1}{G} \right) - 1 \right] \times 100$$

WHERE / SYMBOLS

- W_1 = Mass of the empty pycnometer
 W_2 = Mass of the pycnometer + moist soil
 W_3 = Mass of the pycnometer + moist soil + water
 W_4 = Mass of the pycnometer completely filled with water
 G = Specific gravity of soil solids
 w = Water content (%)

(a) Empty pycnometer



Mass = W_1

(b) Pycnometer + moist soil



Mass = W_2

(c) Pycnometer + moist soil + water



Mass = W_3

(d) Pycnometer completely filled with water



Mass = W_4

EXAMPLE CALCULATION

Given:

$G = 2.65$
 $W_1 = 32.40 \text{ g}$
 $W_2 = 62.40 \text{ g}$
 $W_3 = 148.00 \text{ g}$
 $W_4 = 132.40 \text{ g}$

$$W_2 - W_1 = 30.00 \text{ g}$$

$$W_3 - W_4 = 15.60 \text{ g}$$

$$\begin{aligned}
 w &= \left[\left(\frac{30.00}{15.60} \right) \times \left(\frac{1.65}{2.65} \right) - 1 \right] \times 100 \\
 &= [1.923 \times 0.623 - 1] \times 100 \\
 &= \mathbf{19.7\%}
 \end{aligned}$$



RESULT: Water content, $w = 19.7\%$

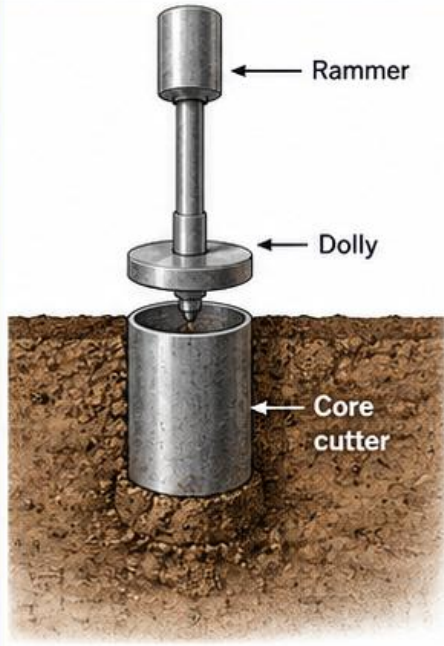


Use the same temperature water, remove entrapped air, and take all weights accurately.

DETERMINATION OF FIELD DENSITY OF SOIL

Core Cutter Method and Sand Replacement Method

1. CORE CUTTER METHOD



Suitable for:

Fine-grained cohesive soils; not suitable for gravelly, very hard, or very soft soils.



Apparatus:

Core cutter, dolly, rammer, steel rule, trimming knife, balance.



Procedure

1. Level the ground surface.
2. Drive the core cutter vertically into the soil using dolly and rammer.
3. Excavate the cutter carefully and trim both ends flush.
4. Weigh empty cutter = W_1 and cutter + soil = W_2 .
5. Calculate cutter volume $V = \pi D^2 H / 4$.

Formula



Bulk density,

$$\rho = \frac{W_2 - W_1}{V}$$

Dry density,

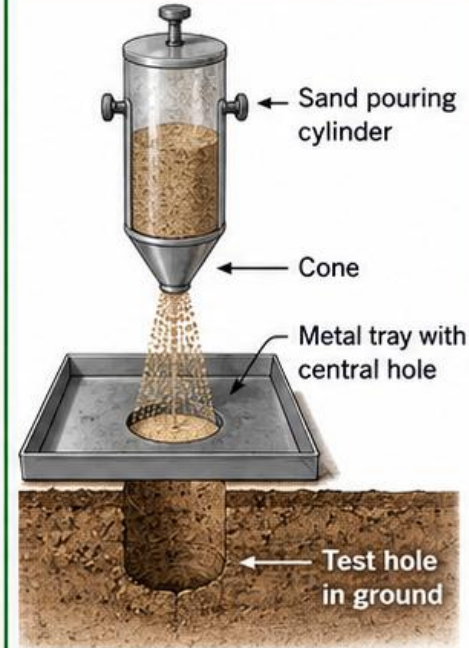
$$\rho_d = \frac{\rho}{1 + w}$$



Main point:

Volume of soil is obtained directly from the known volume of the cutter.

2. SAND REPLACEMENT METHOD



Suitable for:

Sandy and gravelly soils; where core cutter cannot be used.



Apparatus:

Sand pouring cylinder, calibration sand, metal tray with central hole, balance, excavating tools, container.



Procedure

1. Level the ground and place the tray.
2. Dig a small hole and collect the excavated moist soil; mass of soil = W .
3. Fill the sand pouring cylinder with dry calibrated sand.
4. Allow sand to fill the cone and the hole.
5. Find mass of sand filling the hole, M_h .
6. Compute hole volume $V = M_h / \rho_s$, where ρ_s is the bulk density of calibration sand.

Formula



Bulk density,

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

Dry density,

$$\rho_d = \frac{\rho}{1 + w}$$



Main point:

Volume of soil is obtained indirectly from the volume of sand filling the hole.

QUICK COMPARISON



Core cutter:

Faster, simpler, direct volume.



Sand replacement:

More versatile, indirect volume.



Both methods determine field bulk density.



Dry density is obtained after moisture content correction.

Thank You

Geotechnical Engineering-I

Lecture 5

**Presented by
Dr. Akash Priyadarshee
Department of Civil Engineering
Muzaffarpur Institute of Technology, Muzaffarpur**

CONCEPT OF SEDIMENTATION ANALYSIS

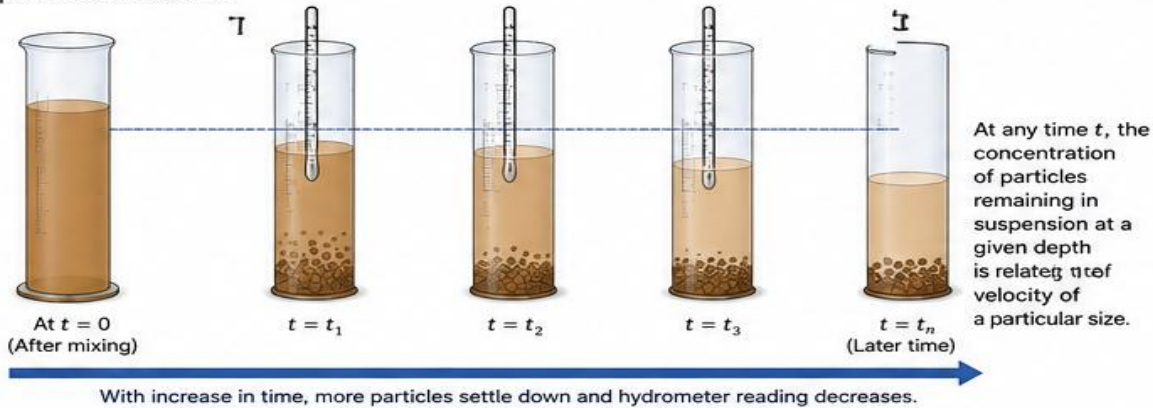
Sedimentation analysis is based on the principle that soil particles dispersed in a fluid settle under gravity.

The rate at which particles settle depends on their size, shape, density difference and fluid properties.

By measuring the concentration of particles remaining in suspension with time, the particle size distribution of fine soil is obtained.

PRINCIPLE

When a soil–water suspension is allowed to stand, particles settle at different velocities. Larger particles settle faster and smaller particles settle slower.



SEDIMENTATION ANALYSIS – CONCEPTUAL WORKFLOW

1. Dispersion



Soil is mixed with water and dispersing agent and stirred thoroughly.

2. Transfer



Suspension is transferred to a sedimentation cylinder and volume made up to the mark.

3. Sedimentation

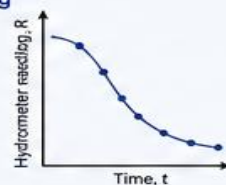


Cylinder is kept undisturbed for sedimentation.

4. Sampling / Hydrometer Reading



Hydrometer reading is taken at selected time intervals at a standard depth.



From readings, particle diameter corresponding to time t is calculated using Stokes' law.

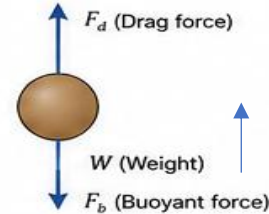
Hydrometer reading at time t gives concentration of particles finer than the equivalent particle diameter that has settling velocity equal to the depth of hydrometer divided by time t .

ASSUMPTIONS IN STOKES' LAW

- Particles are spherical and of uniform size.
- Flow around particle is laminar ($Re < 1$).
- Particle settles in an infinite medium (no wall effect).
- No flocculation of particles.
- Fluid is Newtonian and at constant temperature.

STOKES' LAW AND TERMINAL VELOCITY

Consider a spherical particle of diameter d settling in a fluid of viscosity μ .



Where,

W = weight of particle

F_b = buoyant force

F_d = drag force

DERIVATION (Stokes' Law)

For laminar flow around a sphere ($Re < 1$), Stokes showed that the drag force,

$$F_d = 3 \pi \mu d v \quad \dots(1)$$

At terminal velocity (v_t), the particle falls with constant velocity.

Hence, by Newton's second law,

$$W - F_b - F_d = 0$$

or,

$$(\rho_s - \rho_f) g \left(\frac{\pi d^3}{6} \right) - (\rho_f g \left(\frac{\pi d^3}{6} \right) - 3 \pi \mu d v_t = 0$$

Simplifying,

$$v_t = \frac{(\rho_s - \rho_f) g d^2}{18 \mu} \quad \dots \text{Stokes' Law}$$

where,

v_t = terminal settling velocity (m/s)

ρ_s = density of soil particles (kg/m^3)

ρ_f = density of fluid (kg/m^3)

g = acceleration due to gravity (9.81 m/s^2)

μ = dynamic viscosity of fluid (Pa-s)

d = particle diameter (m)

TERMINAL VELOCITY

The terminal velocity is the constant velocity attained by a particle when the net force on it becomes zero.

$$v_t = \frac{(\rho_s - \rho_f) g d^2}{18 \mu}$$

Hence,

$$v_t \propto d^2$$

Larger particles (large d) have higher terminal velocity and settle faster.

REYNOLDS NUMBER FOR SETTLING PARTICLE

Reynolds number,

$$Re = \frac{\rho_f v_t d}{\mu}$$

For Stokes' law to be valid,

$$Re < 1 \text{ (laminar flow)}$$

If $Re > 1$, correction factors must be applied and Stokes' law is not applicable.

APPLICATION IN SEDIMENTATION ANALYSIS

From Stokes' law,

$$d = \sqrt{\frac{18 \mu L}{(\rho_s - \rho_f) g t}}$$

where,

L = effective depth of hydrometer below meniscus (m)

t = time (s)

Thus, for a given time t , a particle of diameter d has just reached depth L . Particles smaller than d remain in suspension; larger particles have settled.

SAMPLE PREPARATION OF SOIL SAMPLE FOR HYDROMETER ANALYSIS

1. Collection of Soil Sample



Disturbed soil sample is collected and air dried.

2. Oven Drying



Soil is oven dried at 105–110 °C to remove moisture.

3. Pulverizing



Dried soil is gently crushed using a mortar and pestle.

4. Sieving



Soil is sieved through 75 μm (No. 200) IS sieve to remove coarse particles.

5. Weighing



A known weight of soil (usually 50 g) is taken for the test.

6. Mixing with Dispersion Agent



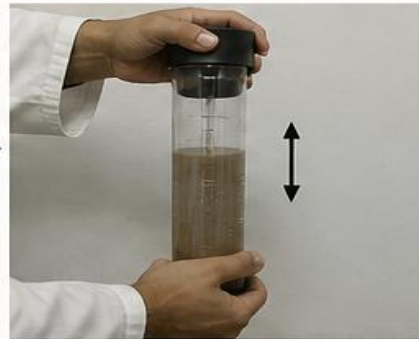
Soil is mixed with distilled water and a few drops of dispersing agent (e.g., Calgon) and stirred thoroughly.

7. Transfer to Sedimentation Cylinder



The mixture is transferred to a 1 litre sedimentation cylinder and the volume is made up to the mark with distilled water.

8. Shaking



Cylinder is tightly stoppered and shaken vigorously for 1–2 minutes.

9. Sedimentation



Cylinder is placed undisturbed and hydrometer readings are taken at specified time intervals.

10. Hydrometer Analysis



Hydrometer is inserted gently and readings are recorded to determine particle size distribution.

PIPETTE METHOD (STANDARD METHOD) FOR GRAIN SIZE ANALYSIS OF SOILS

AS PER IS 2720 (PART 4) – 1985

1. SAMPLE PREPARATION

Sample soil passing 4.75 mm IS sieve. Oven dry and take 25–50 g (as per soil type) for analysis.



Soil passing 4.75 mm IS sieve

2. PRE-TREATMENT (REMOVE ORGANIC MATTER & SODIUM SALTS)

- Add 75 ml H_2O_2 (20 volume)
- Boil gently till reaction ceases (reduce to ~30 ml)
- If calcium present, add ~10 ml N HCl, stir and allow to stand till acid reaction to litmus.



3. WASHING

Filter and wash with warm distilled water till filtrate gives no acid reaction to litmus.



4. DISPERSION

Transfer soil to evaporating dish. Add 25 ml sodium hexametaphosphate solution + ~25 ml distilled water. Warm gently for 10 minutes.



5. MECHANICAL MIXING

Transfer to dispersion cup and mix with mechanical stirrer for 15 minutes (8 000 – 10 000 rpm).



6. WASH THROUGH 75-micron IS SIEVE

Wash the suspension through 75-micron IS sieve using distilled water.



7. MAKE UP TO VOLUME

Transfer suspension to sedimentation tube and make up to 500 ml with distilled water. Mix thoroughly.



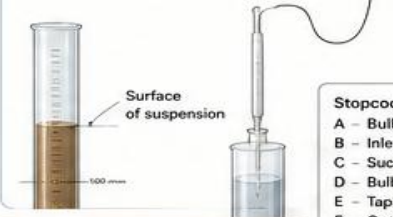
8. SEDIMENTATION

- Place sedimentation tube in constant temperature bath at $27 \pm 0.1^\circ\text{C}$ (preferable).
- Shake thoroughly and start stopwatch simultaneously.

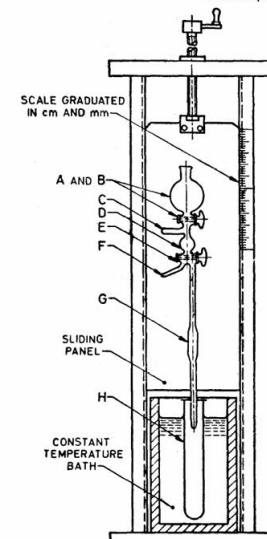


9. SAMPLING WITH PIPETTE (at selected time intervals)

Insert pipette to 100 mm below surface at the required time. Draw sample, close tap, remove, wash outside, and discharge into weighed bottle.



Stopcock
A – Bulb
B – Inlet
C – Suction
D – Bulb
E – Tap
F – Outlet



10. OVEN DRY & WEIGH

Evaporate the contents in weighed bottle at $105 - 110^\circ\text{C}$. Cool in desiccator and weigh to nearest 0.001 g.



TIME INTERVALS FOR SAMPLING (Commonly used)

Time after shaking	Particle size, D (mm) (for $G_s = 2.68$, $H = 100$ mm, $T = 27^\circ\text{C}$)
30 seconds	0.020
1 minute	0.006
2 minutes	0.002
4 minutes	0.001
8 minutes	0.0005
15 minutes	0.0003
30 minutes	0.0002
1 hour	0.0001
2 hours	0.00007
4 hours	0.00005
24 hours	0.00002

Note: Particle size varies with temperature and specific gravity. Use actual T and G_s in equation.

CALCULATIONS

1. Diameter of particle, D (mm) (Stokes' Law)

$$D = \sqrt{\frac{18\eta H}{(\rho_s - \rho_w) g t}}$$

Where,

- η = coefficient of viscosity of water (poise)
- H = height of fall (100 mm = 10 cm)
- ρ_s = density of soil particles = $G_s \times \rho_w$
- g = acceleration due to gravity (981 cm/s^2)
- t = time of sedimentation (min)
- ρ_w = density of water at temperature T

At 27°C , $G_s = 2.68$, values of D for various times are given in Table 1 of IS 2720 (Part 4).

2. Percentage finer than D

$$W = \frac{(M_i - M_s)}{W_b} \times 100$$

Where,

- M_i = mass of soil in 500 ml taken at time t (g)
- M_s = mass of dispersing agent in 500 ml (g)
- W_b = mass of soil after pretreatment (g)

3. Combined gradation

Combine results of:

- Sieve analysis ($> 75 \mu\text{m}$)
 - Pipette analysis ($< 75 \mu\text{m}$)
- on the basis of total mass of soil sample taken.

IMPORTANT NOTES

- Use sodium hexametaphosphate solution as dispersant (33 g + 7 g Na_2CO_3 per litre) or other suitable dispersant.
- Maintain temperature of suspension at $27 \pm 0.1^\circ\text{C}$ for maximum accuracy.
- Take readings at specified time intervals carefully without disturbing the suspension.
- Hydrometer method is an alternative (subsidiary method).
- Method not applicable if $< 10\%$ passes 75-micron IS sieve.

APPARATUS ESSENTIALS



This method determines the particle size distribution of the fines ($< 75 \mu\text{m}$) based on the principle of sedimentation and Stokes' Law.

HYDROMETER METHOD (SUBSIDIARY METHOD) FOR GRAIN SIZE ANALYSIS OF SOILS

AS PER IS 2720 (PART 4) – 1985

1. PRE-TREATMENT (See Cl. 5.2.4.1)

- Remove organic matter with H_2O_2 (20 volume).
- For soils containing calcium, add N HCl, allow to stand and wash till no acid reaction to litmus.
- Dry at $105 - 110^\circ C$, cool in desiccator and weigh (W_b).

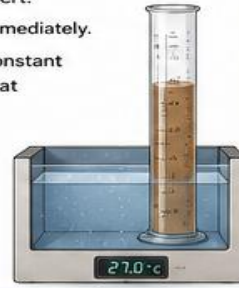
2. DISPERSION (See Cl. 5.2.4.2)

- Add 100 ml sodium hexametaphosphate solution to the pre-treated soil.
- Mix mechanically for 15 minutes at 8 000 – 10 000 rpm.
- Wash through 75-micron IS sieve.
- Transfer suspension to 1000 ml measuring cylinder and make up to 1000 ml with distilled water.



3. SEDIMENTATION (See Cl. 5.2.4.3)

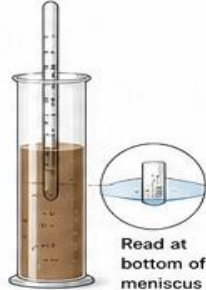
- Stopper the cylinder, shake thoroughly and invert.
- Start stopwatch immediately.
- Keep cylinder in constant temperature bath at $27 \pm 0.1^\circ C$ (recommended).



4. HYDROMETER READINGS (See Cl. 5.2.4.3)

Time of Readings

- 0.5, 1, 2, 4 minutes
- 8, 15, 30 minutes
- 1, 2, 4 hours
- 24 hours (once or twice within 24 h)
- Also take reading in pure distilled water at the same temperature for temperature correction (Δt).



Precautions

- Insert and withdraw hydrometer slowly (10 s each).
- Avoid vibration and disturbance of suspension.
- Maintain temperature; note at each reading ($\pm 0.5^\circ C$).
- Remove, rinse and keep hydrometer in distilled water after each reading.

Corrections to Apply

- Meniscus correction (C_m)
- Temperature correction (Δt)
- Dispersing agent correction (x)
- These are applied to observed hydrometer reading (R) to obtain corrected reading (R_c).

5.1 Loss in Mass in Pre-treatment (See Cl. 5.1.4.1)

$$P = 100 - \frac{W_b(100 + w)}{W_a}$$

Where,

- P = loss in mass in percentage
- W_b = mass of soil after pre-treatment (g)
- w = moisture content of original soil (%)
- W_a = mass of air dry soil used (g)

5.2 Concentration of Soil in Suspension (See Cl. 5.2.5.2)

$$C = \frac{R_c}{H_R \times K}$$

Where,

- C = concentration of soil in suspension (g/ml)
- R_c = corrected hydrometer reading
- H_R = effective depth corresponding to reading R_c (cm)
- K = constant = 1 (since suspension volume = 1000 ml)

5.2.1 Corrected Hydrometer Reading (R_c) (See Cl. 5.2.5.1)

$$R_c = R + C_m + \Delta t - x$$

Where,

- R = observed hydrometer reading
- C_m = meniscus correction
- Δt = temperature correction
- x = dispersing agent correction

5.2.2 Effective Depth, H_R (See Cl. 5.2.3.2)

$$H_R = H_h - \frac{V_h}{A} - \frac{h}{2}$$

Where,

- H_h = length from neck to reading R_h (cm)
- V_h = volume of hydrometer (ml)
- A = area of cylinder (cm^2)
- h = height of bulb (cm)

Plot H_h vs R_h for all major readings to obtain smooth calibration curve.
Use curve to get H_R for each corrected reading.

5. CALCULATIONS

5.3 Equivalent Diameter of Particle, D (See Cl. 5.2.5.3)

$$D = \sqrt{\frac{18\eta H}{(G_s - 1) g t}}$$

Where,

- D = particle diameter (mm)
- η = coefficient of viscosity of water (poise)
- H = effective depth (cm)
- G_s = specific gravity of soil fraction (passing 75-micron IS sieve)
- g = acceleration due to gravity = 981 cm/s^2
- t = time of sedimentation (min)

Values for Viscosity of Water (η) (Approximate values)

Temperature ($^\circ C$)	10	15	20	25	27	30	35
η (poise)	0.0131	0.0114	0.0101	0.0089	0.0084	0.0079	0.0071

(Use η corresponding to temperature of suspension)

5.4 Percentage Finer than D (See Cl. 5.2.5.4)

$$W = \frac{M_d \times 100}{W_b}$$

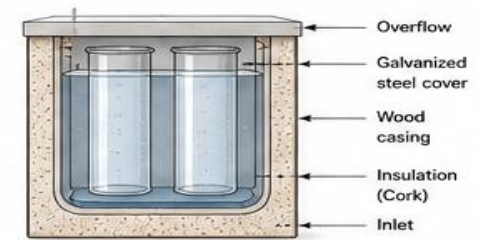
Where,

- W = percentage finer than D
- M_d = mass of soil in suspension (g) ($= C \times 1000 \text{ ml} = C \times 1000$)
- W_b = mass of soil after pre-treatment (g)

APPARATUS ESSENTIALS

- Hydrometer
- 1000 ml measuring cylinder
- Constant temperature bath ($27 \pm 0.1^\circ C$)
- Thermometer ($0 - 50^\circ C$, least count $0.5^\circ C$)
- Mechanical stirrer
- 75-micron IS sieve
- Balance (accuracy 0.01 g)
- Oven ($105 - 110^\circ C$)
- Stop watch
- Sodium hexametaphosphate solution
- Distilled water, Wash bottle, etc.

CONSTANT TEMPERATURE BATH (Typical)



Maintain temperature at $27 \pm 0.1^\circ C$

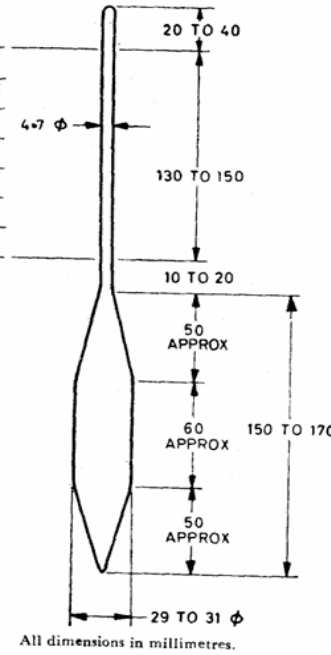


FIG. 5 HYDROMETER

5.5 Mass of Soil in Suspension

$$M_d = C \times 1000$$

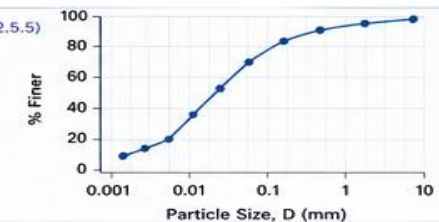
Where,

- M_d = mass of soil in suspension (g)
- C = concentration of soil in suspension (g/ml) (for 1000 ml suspension)

6. COMBINED GRADATION (See Cl. 5.1.4.4 / 5.2.5.5)

Combine results of:

- Sieve analysis ($> 75 \mu m$)
 - Hydrometer analysis ($< 75 \mu m$)
- on the basis of total mass of soil sample taken.
Plot % finer (W) versus particle size (D , mm) on semi-log graph to obtain grain size distribution curve.



NOTE : Method applies only if $\geq 10\%$ of material passes 75-micron IS sieve. Maintain temperature at $27 \pm 0.1^\circ C$ for maximum accuracy.

Thank You

Geotechnical Engineering-I

Lecture 6 & 7

**Presented by
Dr. Akash Priyadarshee
Department of Civil Engineering
Muzaffarpur Institute of Technology, Muzaffarpur**

CONSISTENCY LIMITS & ITS IMPORTANCE

Consistency limits, also called **Atterberg limits**, define the water content boundaries at which a fine-grained soil changes its consistency or state. These limits are crucial for identification, classification and engineering performance.

CONSISTENCY LIMITS

LIQUID LIMIT (LL)

Water content at which soil changes from liquid to plastic state.



LIQUID STATE

PLASTIC LIMIT (PL)

Water content at which soil changes from plastic to semi-solid state.



PLASTIC STATE

SHRINKAGE LIMIT (SL)

Water content at which soil changes from semi-solid to solid state.



SEMI-SOLID STATE

SOLID STATE

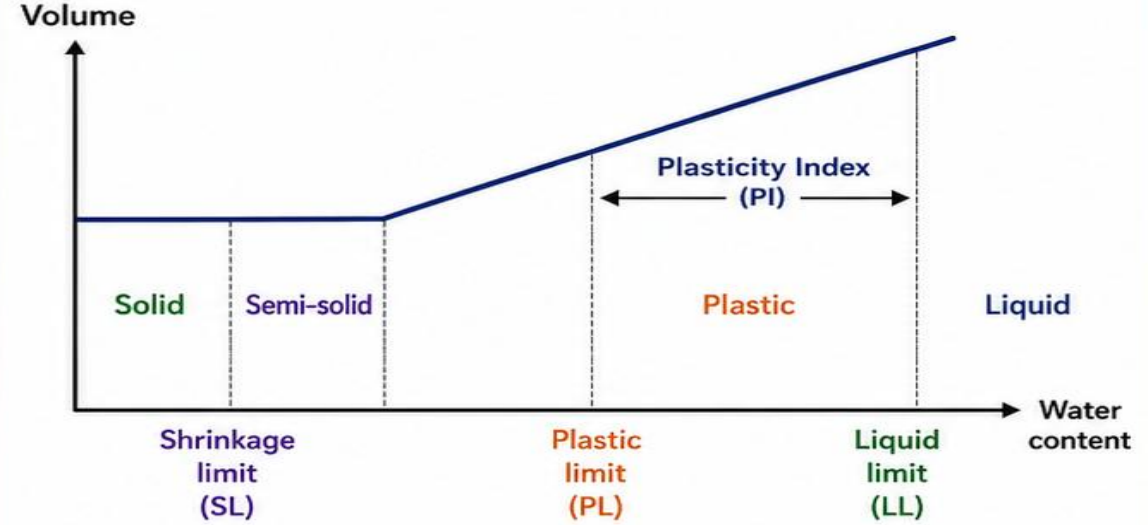
Below shrinkage limit, soil behaves as a solid.



SOLID STATE

DECREASING WATER CONTENT (w)

VOLUME – WATER CONTENT RELATIONSHIP & ATTERBERG LIMITS



IMPORTANCE OF ATTERBERG LIMITS

1. SOIL IDENTIFICATION & CLASSIFICATION



Help in identifying fine-grained soils and classifying them using systems like USCS and IS Classification.

2. PREDICTS ENGINEERING BEHAVIOR



Indicates soil plasticity, compressibility, shrink-swell potential and strength characteristics.

3. FOUNDATION & PAVEMENT DESIGN



Helps in assessing settlement potential, bearing capacity and suitability of soils for foundations and pavements.

4. EARTHWORKS & CONSTRUCTION



Useful in deciding excavation feasibility, compaction requirements and optimum moisture content.

5. VOLUME CHANGE PREDICTION



Helps predict swelling and shrinkage behavior of soils, important for structures on expansive soils.

6. QUALITY CONTROL & MATERIAL SELECTION



Ensures consistency in soil properties for borrow material, embankments and other geotechnical applications.

Atterberg limits are simple laboratory tests that provide vital information about fine-grained soils and are essential for safe, economical and reliable geotechnical design and construction.

DETERMINATION OF LIQUID LIMIT BY CASAGRANDE'S APPARATUS

As per IS 2720 (Part 5): 1985

1. INTRODUCTION / DEFINITION



Liquid limit (LL) is the water content at which a standard groove in a soil pat closes for about 12 mm under 25 blows in the Casagrande device.



Used for classification and evaluation of fine-grained soils.

2. APPARATUS REQUIRED



425-micron
IS sieve



Mixing dish /
Evaporating dish



Spatula



Moisture cans



Balance



Oven



Wash bottle /
Water dropper

GROOVING TOOLS & GROOVE SHAPE (Top View)

Casagrande / IS
Grooving Tool
(for clayey soils)



Narrow groove
per IS)

ASTM Grooving Tool
(for sandy soils)

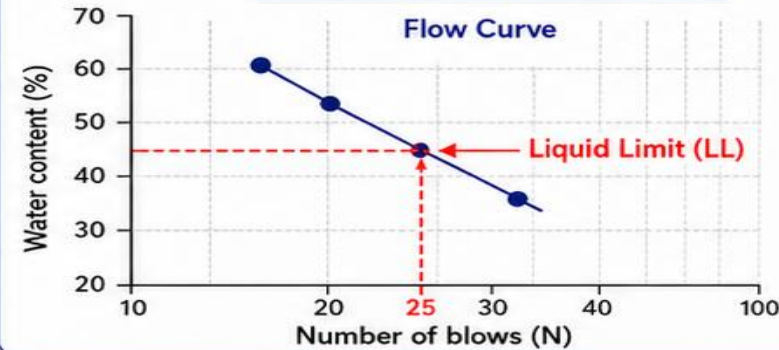


Wider U-shaped
groove
(as per ASTM)

3. PROCEDURE

- 1 Take about 120 g air-dried soil passing 425-micron sieve.
- 2 Mix with distilled water to form a uniform paste.
- 3 For clayey soils, use Casagrande / IS grooving tool. For sandy soils, use ASTM tool.
- 4 Place paste in Casagrande cup and level it to about 1 cm thickness at the deepest point.
- 5 Cut standard groove with the appropriate grooving tool.
- 6 Rotate the crank at about 2 revolutions per second.
- 7 Record the blows when the groove closes for about 12 mm.
- 8 Take a sample for water content determination and repeat for at least 4 trials between 15 and 35 blows.

4. OBSERVATION & GRAPH



Flow Index, I_f

$$I_f = \frac{w_1 - w_2}{\log_{10} \left(\frac{N_2}{N_1} \right)}$$

Liquid limit = water content corresponding to 25 blows from the flow curve.

5. KEY POINTS / IMPORTANCE



Helps classify fine-grained soil.



Indicates consistency and plasticity.



Useful in foundation and pavement design.



Helps assess settlement and compressibility.



Assists in identifying expansive soil behavior.



Important for laboratory quality control.



Important test conditions: soil passing 425-micron sieve, groove closure 12 mm, crank speed about 2 rev/s, and LL obtained from the flow curve at 25 blows.

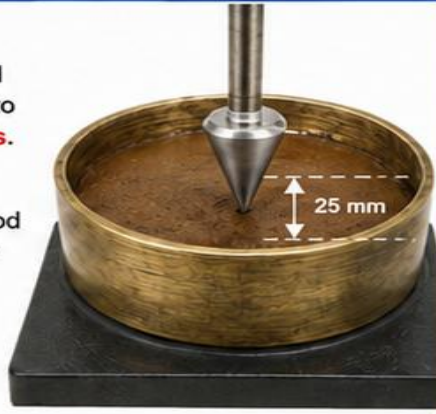
DETERMINATION OF LIQUID LIMIT BY CONE PENETROMETER METHOD

As per IS 2720 (Part 5): 1985

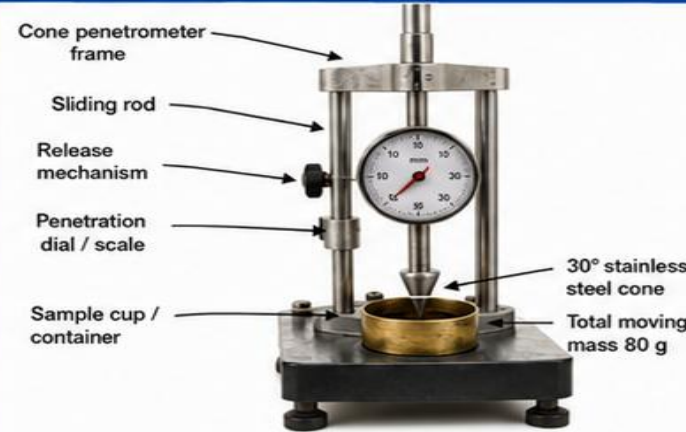
1. INTRODUCTION / PRINCIPLE

Liquid limit is the water content at which a standard cone penetrates **25 mm** into the soil paste in **5 seconds**.

The cone penetration method is an alternative method for determining liquid limit of fine-grained soils.



2. APPARATUS REQUIRED



Cone just touching the soil surface before release

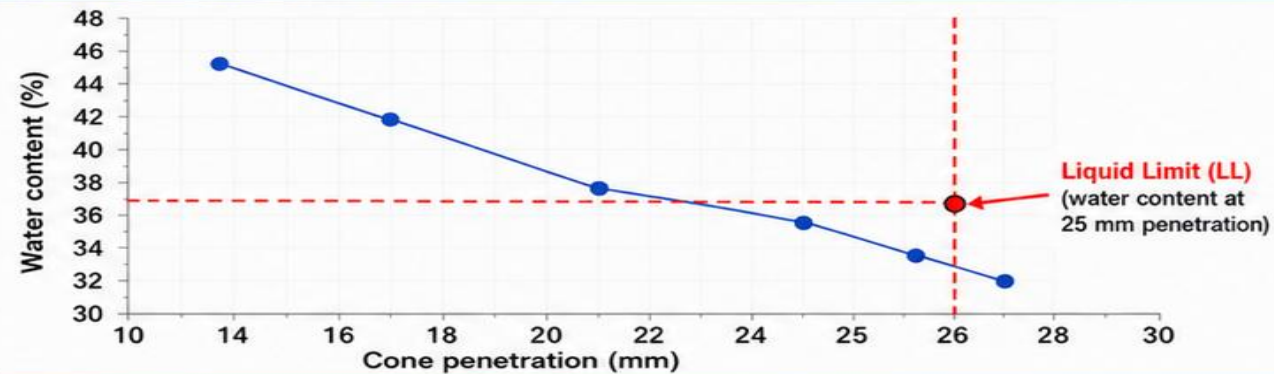


Tip of the 30° cone just touches the smooth soil surface.

3. PROCEDURE

1. Take about 150 g of air-dried soil passing 425-micron sieve.
2. Mix with distilled water to form a uniform paste.
3. For highly clayey soil, allow the paste to stand for about 24 hours.
4. Remix thoroughly and place the paste in the cup, level the surface smoothly.
5. Lower the 30° cone so that its tip just touches the soil surface.
6. 5 s Release the cone and allow penetration for 5 seconds.
7. Record cone penetration in mm and take a sample for water content determination.
8. Repeat for at least 4 trials with penetrations in the range of about 14–28 mm.

4. OBSERVATION & GRAPH



Report LL to the nearest first decimal place.

One-Point Method (Note)

If an accepted trial gives penetration between 16 mm and 26 mm, the liquid limit may be estimated from that accepted trial.

5. KEY POINTS / IMPORTANCE

Soil Classification
Helps in classifying fine-grained soils (ML, CL, CH, etc.).

Consistency / Plasticity Assessment
Indicates the liquid state boundary and plastic behaviour of the soil.

Compressibility Indication
Higher LL generally indicates higher compressibility.

Foundation & Pavement Design
Essential for design of foundations, earthworks and pavement systems.

Swelling / Shrinkage Assessment
Helps in predicting volume change potential of clayey soils.

Laboratory Quality Control
Ensures uniformity and reliability in geotechnical testing.



Important test conditions:

- soil passing 425-micron sieve
- 30° cone, 80 g total mass
- penetration observed after 5 seconds
- LL = water content at 25 mm penetration

SHRINKAGE LIMIT TEST (IS 2720 – PART 6)

Shrinkage limit is the water content at which a further loss of water does not cause any further decrease in volume of a soil mass.

PRINCIPLE

A soil pat is allowed to dry in a shrinkage dish until it just begins to crumble. The water content at this stage is taken as the shrinkage limit.

APPARATUS

- Shrinkage dish (brass dish) – ϕ 44 mm, height 12.7 mm
- Glass plate
- Spatula
- Oven
- Balance (accuracy 0.01 g)
- Desiccator



PROCEDURE

- Take a representative soil sample passing 425 μ m IS sieve and mix with water to form a uniform paste.

- Place the soil paste in the shrinkage dish on a glass plate and level it to a uniform thickness.

- Weigh the dish + wet soil (W_1).

- Allow the dish to dry in room condition.

- When the soil just begins to crumble at the edges or along the surface, weigh the dish + soil (W_2).

- Remove the remaining soil from the dish and oven dry the soil. Weigh the oven dry soil (W_3).

- Calculate the water content. Repeat for 2–3 trials and take the average.

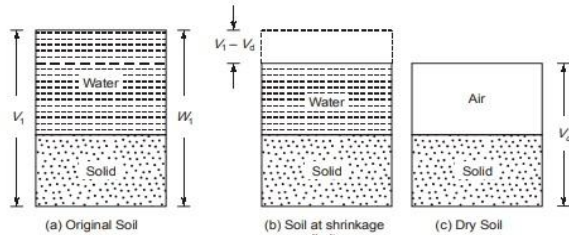
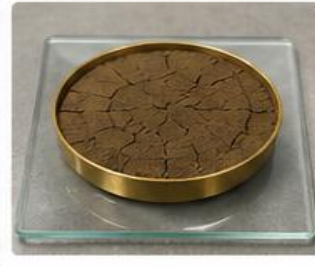


Fig. Determination of Shrinkage limit

where, V_1 = Original volume of soil, V_d = Dry volume of soil
 W_1 = Original weight of soil, $W_d = W_s$ = Dry weight of soil (solids)
 w_1 = Natural water content

Shrinkage limit,

$$w_s = \frac{W_w}{W_s} = \frac{(W_1 - W_s) - (V_1 - V_d)\gamma_w}{W_d} = \frac{(W_1 - W_s)}{W_d} - \frac{(V_1 - V_d)\gamma_w}{W_d}$$

$$w_s = w_1 - \left(\frac{V_1 - V_d}{W_d} \right) \times \gamma_w$$

- Shrinkage limit test can be used to determine sp. gravity of soil solids

$$G = \frac{1}{\left(\frac{\gamma_w}{\gamma_d} - \frac{w_s}{100} \right)}$$

$$w_s = \frac{1}{R} - \frac{1}{G}$$

where, γ_w = Unit weight of water, γ_d = Dry unit weight of soil, w_s = % shrinkage limit

- If specific gravity G and void ratio e are known, then

$$w_s = \frac{Se}{G} = \frac{e}{G} \quad (\because S = 1)$$

IDENTIFICATION OF SHRINKAGE LIMIT

The shrinkage limit is reached when the soil mass just begins to crumble, i.e., a further loss of water does not cause any decrease in volume.

BEFORE – volume decreases with loss of water



AT SHRINKAGE LIMIT – no further decrease in volume



SPECIFICATION (IS 2720 – PART 6)

The test shall be conducted in accordance with IS 2720 (Part 6): 1972 – Determination of Shrinkage Limit.

REPORT

- Shrinkage limit, w_s (%)
- Number of trials
- Any deviations from the method

Note: The shrinkage limit is always less than the plastic limit and indicates the boundary between semi-solid and solid states.

CONSISTENCY LIMITS – IMPORTANT INDICES AND THEIR EQUATIONS

1. VOLUMETRIC SHRINKAGE (S_v)

It is the decrease in volume of soil mass as it dries from the liquid limit to the shrinkage limit.



At Liquid Limit
(Volume = V_L)

At Shrinkage Limit
(Volume = V_s)

$$\text{Volumetric shrinkage} = \frac{V_1 - V_d}{V_d} \times 100$$

Where, V_1 = Original soil volume

V_d = dry volume of soil

2. SHRINKAGE RATIO (S_r)

It is the ratio of volume of soil at shrinkage limit to the volume at liquid limit.

$$\text{Shrinkage Ratio} = \frac{V_1 - V_2}{V_d - w_2}$$

Where, V_1 = Original soil volume

V_2 = volume after drying

V_d = dry volume of soil

w_1 = Original weight of soil

w_2 = Water content at volume V_2

3. PLASTICITY INDEX (I_p)

It is the range of water content over which the soil remains in plastic state.

$$I_p = w_L - w_P$$

Where,

w_L = Liquid limit (%)

w_P = Plastic limit (%)

Higher I_p indicates higher plasticity of the soil.

4. SHRINKAGE INDEX (I_s)

It is the range of water content over which the soil remains in semi-solid state.

$$I_s = w_P - w_S$$

Where,

w_P = Plastic limit (%)

w_S = Shrinkage limit (%)

Higher I_s indicates greater tendency of the soil to shrink in semi-solid state.

5. LIQUIDITY INDEX (L_I)

It indicates the relative position of natural water content with respect to the consistency limits.

$$L_I = \frac{w - w_P}{I_p}$$

Where,

w = Natural water content (%)

w_P = Plastic limit (%)

I_p = Plasticity index (%)

Interpretation:

- $L_I < 0$ → Semi-solid state
- $L_I = 0$ → At plastic limit
- $0 < L_I < 1$ → Plastic state
- $L_I = 1$ → At liquid limit
- $L_I > 1$ → Liquid state

6. TOUGHNESS INDEX (I_T)

It is the ratio of plasticity index to liquidity index.

$$\text{Toughness index, } I_T = \frac{I_p}{L_I}$$

Where,

I_p = Plasticity index (%)

L_I = Liquidity index

Interpretation:

- Higher I_T → Soil is tougher (requires more energy to remould)
- Lower I_T → Soil is less tough

7. SENSITIVITY (S_t)

It is the ratio of the undisturbed shear strength to the remoulded shear strength of a soil.

$$S_t = \frac{S_u}{S_r}$$

Where,

S_u = Undisturbed shear strength

S_r = Remoulded shear strength

Interpretation:

- $S_t < 2$ → Low sensitivity
- $2 \leq S_t \leq 4$ → Medium sensitivity
- $S_t > 4$ → High sensitivity

8. ACTIVITY NUMBER (A)

It indicates the clay activity by relating plasticity index to the percentage of clay-size particles.

$$A = \frac{I_p}{\% \text{ clay fraction}}$$

Where,

I_p = Plasticity index (%)

% clay fraction = Percentage of particles smaller than 2 μm

Interpretation:

- $A < 0.75$ → Inactive clay
- $0.75 \leq A \leq 1.25$ → Normal clay
- $A > 1.25$ → Active clay

SYMBOLS SUMMARY

w_L = Liquid limit (%)

w_P = Plastic limit (%)

w_S = Shrinkage limit (%)

w = Natural water content (%)

I_p = Plasticity index (%)

I_s = Shrinkage index (%)

V_L = Volume at liquid limit

V_S = Volume at shrinkage limit

S_v = Volumetric shrinkage (%)

These indices help in assessing the consistency, engineering behavior and classification of fine-grained soils.



ENGINEERING INTERPRETATION OF ATTERBERG LIMIT INDICES

1. PLASTICITY INDEX (PI) AND SOIL PLASTICITY

I_p	Consistency
0	Non plastic
0-7	Low plastic
7-17	Medium plastic
>17	Highly plastic

Plasticity

2. LIQUIDITY INDEX (LI) AND CONSISTENCY OF SOIL

$$LI = \frac{w - w_p}{w_L - w_p}$$

Liquidity Index (LI)	Consistency
$LI < 0$	Hard
$0 - 0.25$	Stiff
$0.25 - 0.50$	Firm
$0.50 - 0.75$	Soft
$0.75 - 1.00$	Very Soft
> 1.00	Liquid State

where, w = Natural water content
 w_p = Plastic limit, w_L = Liquid limit

3. CONSISTENCY INDEX (I_c) AND CONSISTENCY OF SOIL

$$I_c = \frac{w_L - w}{w_L - w_p} = 1 - LI$$

Consistency Index (I_c)	Consistency
> 1.0	Hard
$0.75 - 1.0$	Stiff
$0.50 - 0.75$	Firm
$0.25 - 0.50$	Soft
$0 - 0.25$	Very Soft
< 0	Liquid State

where, w = Natural water content
 w_p = Plastic limit, w_L = Liquid limit

4. SENSITIVITY CLASSIFICATION

$$S_t = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$

Sensitivity (S_t)	Classification
< 2	Low Sensitivity
$2 - 4$	Medium Sensitivity
$4 - 8$	Sensitive
$8 - 16$	Very Sensitive
$16 - 32$	Extra Sensitive
> 32	Quick Clay / Extremely Sensitive

Higher sensitivity indicates greater loss of strength on remoulding and poorer stability.

5. ACTIVITY NUMBER CLASSIFICATION (Skempton, 1953)

$$A = \frac{PI}{\% \text{ clay fraction } (< 2 \mu\text{m})}$$

Activity (A)	Clay Activity
< 0.75	Inactive Clay
$0.75 - 1.25$	Normal Clay
> 1.25	Active Clay

Typical Activity of Common Clay Minerals

Clay Mineral	Activity
Kaolinite	$0.3 - 0.5$
Illite	$0.5 - 1.0$
Montmorillonite	$1.5 - 7.0$

Higher activity indicates greater influence of clay minerals on plasticity and volume change.

6. TOUGHNESS INDEX CLASSIFICATION

$$I_T = \frac{PI}{I_F}$$

where I_F = Flow Index

Toughness Index (I_T)	Soil Behavior
< 0.5	Very Brittle
$0.5 - 1.0$	Low Toughness
$1.0 - 2.0$	Moderate Toughness
> 2.0	High Toughness

Higher toughness means soil requires more energy to remould (more cohesive / plastic soil).

Symbols: w = Natural water content (%) w_L = Liquid limit (%) w_p = Plastic limit (%) PI = Plasticity index (%) I_F = Flow index
 I_c = Consistency index LI = Liquidity index S_t = Sensitivity A = Activity number

RELATIVE DENSITY OF SOIL

DEFINITION

Relative density (D_r) is a measure of the compactness of cohesionless soils (sands and gravels). It is the ratio of the density index (or void ratio index) of a soil to that for the loosest and densest states.

FORMULA

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100\% \quad \text{or} \quad D_r = \frac{\gamma_d - \gamma_{d(min)}}{\gamma_{d(max)} - \gamma_{d(min)}} \times 100\%$$

Where,

D_r = Relative density (%)

e = Natural void ratio of the soil

e_{max} = Maximum (loosest) void ratio

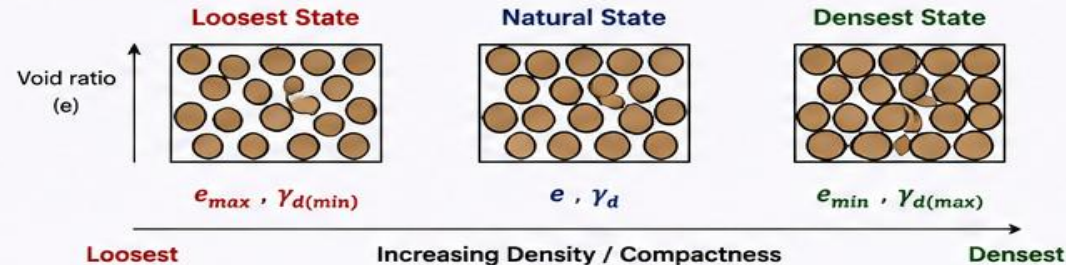
e_{min} = Minimum (densest) void ratio

γ_d = Natural dry unit weight

$\gamma_{d(max)}$ = Maximum (densest) dry unit weight

$\gamma_{d(min)}$ = Minimum (loosest) dry unit weight

CONCEPTUAL DIAGRAM



DETERMINATION

Relative density can be determined by:

- **Laboratory method:** Using maximum and minimum void ratio (from Standard Proctor / relative density test as per IS 2720 Part 28)
- **Field method:** Using in-situ density tests and correlations with N-value (SPT), CPT, or other empirical methods.

IMPORTANT NOTES

- Relative density is applicable only to cohesionless soils.
- It is independent of the type of sand but dependent on gradation, particle shape, and test method.

CLASSIFICATION OF SOIL BASED ON RELATIVE DENSITY (For Sands)

Relative Density (D_r) (%)	Description	Field Condition	N-value (Approx.)
0 – 15	Very Loose	Flows like a liquid when shaken, very little resistance to penetration.	0 – 4
15 – 35	Loose	Easily penetrated by standard sampler, low resistance.	4 – 10
35 – 65	Medium Dense	Moderate resistance to penetration.	10 – 30
65 – 85	Dense	High resistance to penetration.	30 – 50
85 – 100	Very Dense	Very high resistance to penetration, difficult to drive sampler.	> 50

Note: The above N-values are approximate and may vary with equipment, overburden pressure, and soil type.

TYPICAL RANGE OF VOID RATIO AND DRY UNIT WEIGHT FOR SANDS

Description	Void Ratio (e)	Dry Unit Weight, γ_d (kN/m ³)
	Typical Range	Typical Range
Very Loose	0.80 – 1.10	12 – 14
Loose	0.60 – 0.80	14 – 16.5
Medium Dense	0.40 – 0.60	16.5 – 19
Dense	0.25 – 0.40	19 – 20.5
Very Dense	0.15 – 0.25	> 20.5

HIGHER THE RELATIVE DENSITY → GREATER THE STRENGTH, STIFFNESS AND DILATANCY, LOWER COMPRESSIBILITY AND SETTLEMENT

Thank You

Geotechnical Engineering-I

Lecture 8

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CLASSIFICATION OF SOIL BASED ON PARTICLE SIZE

General geotechnical particle-size groups commonly used in engineering

Larger particles ←

→ Smaller particles

BOULDER

> 300 mm



COBBLE

80 mm to 300 mm



GRAVEL

4.75 mm to 80 mm



Coarse gravel

20 mm to 80 mm

Fine gravel

4.75 mm to 20 mm

SAND

0.075 mm to 4.75 mm



Coarse sand

2.0 mm to 4.75 mm

Medium sand

0.425 mm to 2.0 mm

Fine sand

0.075 mm to 0.425 mm

FINES

< 0.075 mm (75 microns)



TEST METHODS



Sieve analysis:
used for coarse fractions (gravel and sand)



Hydrometer / pipette analysis:
used for fines

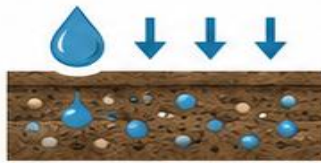


Important note: In the IS soil classification system, particles finer than 75 microns are called *fines*. Silt and clay are further distinguished mainly by plasticity characteristics, not only by particle size.

IMPORTANCE OF PARTICLE-SIZE CLASSIFICATION



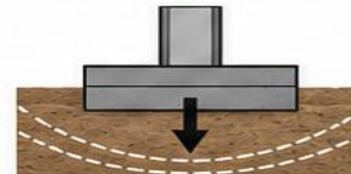
Soil identification and description



Permeability and drainage behavior



Compaction and workability



Bearing capacity and settlement behavior



Selection of construction and pavement materials

Particle-size boundaries may vary slightly between different standards.

SOIL CLASSIFICATION BASED ON GROUP INDEX

AASHTO system for evaluating subgrade quality

1. INTRODUCTION / PRINCIPLE

Group Index (GI) is an empirical number used in the AASHTO soil classification system to indicate the quality of a soil as a highway subgrade material.

Lower GI = better subgrade quality;
higher GI = poorer performance.



2. PARAMETERS USED



F = percentage passing
0.075 mm sieve
(No. 200 sieve)



LL = liquid limit



PI = plasticity index

3. GROUP INDEX FORMULA

$$GI = (F - 35)[0.2 + 0.005(LL - 40)] + 0.01(F - 15)(PI - 10)$$

- If any bracketed quantity is negative, take it as zero.
- GI is reported as a whole number.
- Typical GI range: 0 to 20 or more.

4. SOIL CLASSIFICATION / INTERPRETATION

GI = 0	GI = 1-4	GI = 5-9	GI = 10-19	GI = 20 or more
Excellent to good	Good to fair	Fair	Poor	Very poor

Increasing GI → decreasing subgrade quality

5. LINK WITH AASHTO SOIL GROUPS

A-1, A-2, A-3

Granular soils
(generally better subgrade)

A-4, A-5

Silty soils

A-6, A-7

Clayey soils
(generally poorer subgrade)

The Group Index is mainly used to compare soils within these AASHTO groups.

6. IMPORTANCE / APPLICATIONS



Helps evaluate
subgrade suitability
for highways



Useful in
pavement design
and material
selection



Indicates relative
load-supporting
capacity



Helps compare
fine-grained
soils



Aids in identifying
poor subgrade
requiring
improvement



Useful for borrow
material selection
and quality
control

7. QUICK EXAMPLE



Given: $F = 60$, $LL = 45$, $PI = 15$

$$GI = (60 - 35)[0.2 + 0.005(45 - 40)] + 0.01(60 - 15)(15 - 10)$$

$$GI = 25(0.225) + 0.01(45)(5)$$

$$GI = 5.625 + 2.25 = 7.875 \approx 8$$



GI ≈ 8



Fair subgrade



Key idea: As the amount of fines and plasticity increase, the Group Index usually increases, indicating weaker subgrade performance.



CLASSIFICATION OF SOIL BASED ON TEXTURE

Based on the relative proportion of sand, silt and clay in soil

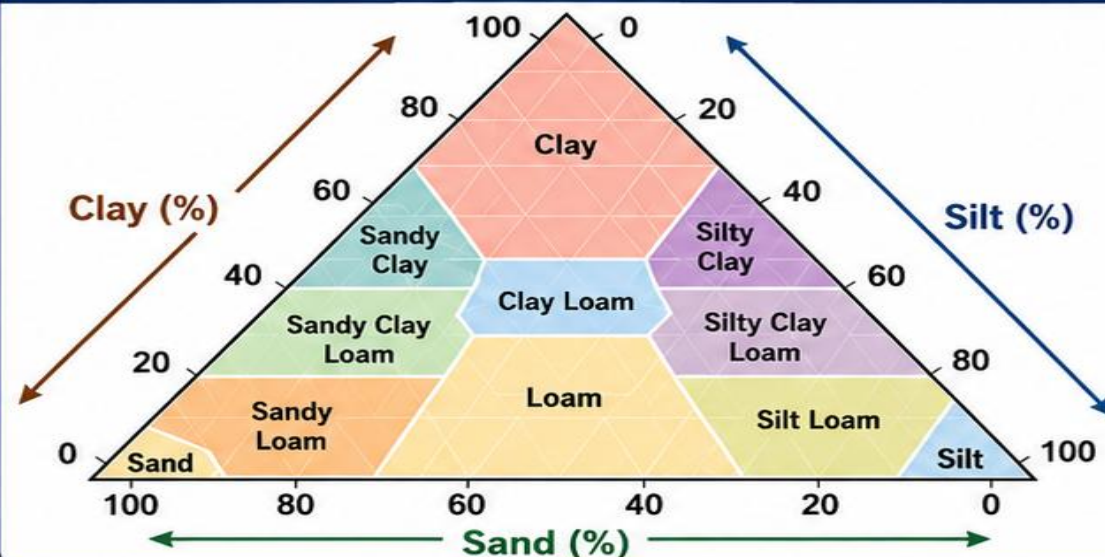
1. INTRODUCTION / PRINCIPLE



Soil texture is the relative proportion of sand, silt and clay-sized particles in a soil mass.

Texture influences water retention, drainage, aeration, workability and fertility.

2. SOIL TEXTURE TRIANGLE (USDA)



3. MAJOR TEXTURAL GROUPS

Coarse-textured soils

- sand
- loamy sand
- sandy loam

Medium-textured soils

- loam
- silt loam
- silt
- clay loam
- silty clay loam
- sandy clay loam

Fine-textured soils

- sandy clay
- silty clay
- clay

4. FIELD FEEL / GENERAL BEHAVIOR

Sandy soil



Gritty feel



High drainage



Low water holding

Silty soil



Smooth / floury feel



Moderate water holding

Clayey soil



Sticky and plastic when wet



High water holding



Low drainage

Loam



Balanced mixture



Good for cultivation

5. IMPORTANCE OF TEXTURAL CLASSIFICATION



Soil identification and description



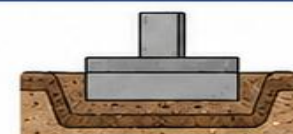
Irrigation and drainage planning



Crop suitability and fertility management



Compaction and workability



Foundation and earthwork behavior



Permeability and water retention assessment



Texture is determined from the percentages of sand, silt and clay; textural class is commonly obtained using the soil texture triangle.

Thank You

Geotechnical Engineering-I

Lecture 9

**Presented by
Dr. Akash Priyadarshee
Department of Civil Engineering
Muzaffarpur Institute of Technology, Muzaffarpur**

CLAY MINERALS OF SOIL & THEIR ENGINEERING IMPORTANCE

1. INTRODUCTION / PRINCIPLE



Clay minerals are very fine hydrous aluminosilicate minerals, usually smaller than 2 microns, formed by weathering.

Their plate-like structure and surface activity strongly influence plasticity, swelling, compressibility, permeability, and strength of soils.

2. MAJOR CLAY MINERALS

KAOLINITE



- 1:1 layer mineral
- Low plasticity
- Low swelling
- Low cation exchange capacity
- Comparatively stable

Engineering behavior:
More stable, lower volume change

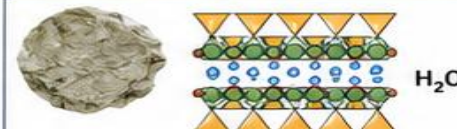
ILLITE



- 2:1 non-expanding mineral
- Medium plasticity
- Low to moderate swelling
- Potassium-bonded layers
- Low to moderate CEC

Engineering behavior:
Moderate plasticity and compressibility

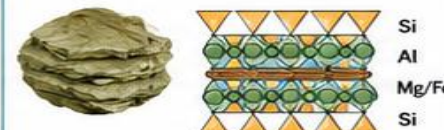
MONTMORILLONITE / SMECTITE



- 2:1 expanding mineral
- Very high swelling
- High to very high plasticity
- High water adsorption
- Very high cation exchange capacity

Engineering behavior:
Expansive soil problems, major shrink-swell potential

CHLORITE



- 2:1:1 layer mineral
- Low to moderate swelling
- Low to medium plasticity
- Low to moderate CEC
- Comparatively stable

Engineering behavior:
Relatively stable behavior

3. BASIC LAYER STRUCTURE

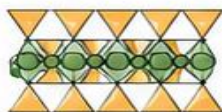
1:1 STRUCTURE



One silica tetrahedral sheet + one alumina octahedral sheet.

- ▲ Silica tetrahedral sheet (SiO_2)
- Alumina octahedral sheet ($\text{Al}(\text{OH})_6$)
- Interlayer cation (e.g., K^+)
- Water molecule (H_2O)

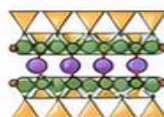
2:1 STRUCTURE



Two silica tetrahedral sheets + one alumina octahedral sheet.

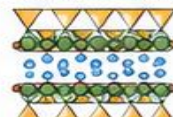
EXPANDING vs NON-EXPANDING 2:1 LAYERS

NON-EXPANDING (e.g., Illite)



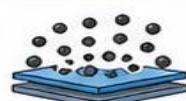
Tight interlayer held by K^+ ions. No water entry, no significant swelling.

EXPANDING (e.g., Smectite)



Water enters interlayer space easily. Layers separate, causing swelling.

4. KEY PROPERTIES CONTROLLED BY CLAY MINERALS



SPECIFIC SURFACE AREA
Higher surface area increases interaction with water and ions.

CATION EXCHANGE CAPACITY (CEC)
Ability to adsorb and exchange cations; controls plasticity, swelling and strength.



WATER ADSORPTION
Controls moisture holding capacity and affects consistency.



PLASTICITY
Determines ease of molding and deformability when wet.



SHRINK-SWELL BEHAVIOR
Volume change with moisture variation; critical in expansive soils.



COMPRESSIBILITY
Influences settlement magnitude and time rate.



PERMEABILITY
Controls seepage and drainage; affects pore water pressures.



SHEAR STRENGTH / COHESION
Cohesive strength largely controlled by clay type and mineral activity.

5. COMPARATIVE ENGINEERING BEHAVIOR

Mineral	Layer type	Swelling	Plasticity	Engineering significance
Kaolinite	1:1	Low	Low to medium	More stable, lower volume change
Illite	2:1	Low to moderate	Medium	Moderate plasticity and compressibility
Montmorillonite	2:1 expanding	Very high	High to very high	Expansive soil problems, major shrink-swell
Chlorite	2:1:1	Low	Low to medium	Relatively stable behavior

6. ENGINEERING IMPORTANCE / APPLICATIONS



Soil classification and identification
Identify clay type for proper engineering use.



Foundation design and settlement assessment
Predict compressibility and long-term settlements.



Expansive soil / shrink-swell prediction
Assess volume change potential and mitigation needs.



Pavement and subgrade performance
Control moisture sensitivity and strength.



Slope stability and shear strength evaluation
Evaluate stability and factor of safety.



Soil stabilization using lime/cement & material selection
Improve strength, reduce plasticity and swelling.



Landfill liners / impermeable barriers and embankments
Ensure low permeability and better performance.



7. KEY IDEA / CONCLUSION

Clay mineralogy governs the engineering behavior of fine-grained soils.

Soils rich in montmorillonite generally show the highest swelling and plasticity, while kaolinitic soils are comparatively more stable.

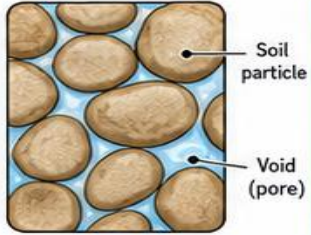


SOIL STRUCTURE & ITS ENGINEERING IMPORTANCE

1. INTRODUCTION / PRINCIPLE

Soil structure is the arrangement and bonding of soil particles and the voids between them.

It strongly affects strength, compressibility, permeability, and settlement behavior.



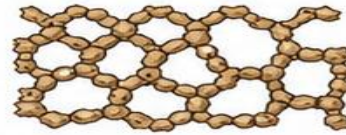
2. MAJOR TYPES OF SOIL STRUCTURE

SINGLE-GRAINED STRUCTURE



- Coarse-grained soils such as sand and gravel
- Particles act independently
- High permeability
- Low compressibility

HONEYCOMB STRUCTURE



- Mainly silt
- Open loose framework
- Higher compressibility
- Moderate instability

FLOCCULENT STRUCTURE



- Clay particles arranged edge-to-face in a random open structure
- Relatively higher void ratio
- More compressible

DISPERSED STRUCTURE



- Clay particles arranged face-to-face in a more parallel orientation
- Denser arrangement
- Lower permeability
- Different consolidation behavior

3. STRUCTURE FORMATION / COMPARISON

GRAIN SIZE



Larger grains form open frameworks; finer grains fill voids and create different structures.

PARTICLE SHAPE



Shape influences contact pattern, orientation and void geometry.

ELECTROCHEMICAL EFFECTS



Surface charges, cations and double-layer thickness control attraction, flocculation or dispersion.

DEPOSITION ENVIRONMENT



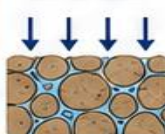
Energy of water, rate of deposition, chemistry and history affect particle arrangement and bonding.



Structure controls void ratio, contact pattern, drainage path, and resistance to deformation.

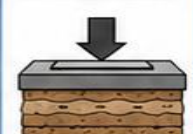
4. ENGINEERING PROPERTIES AFFECTED BY SOIL STRUCTURE

PERMEABILITY



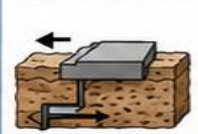
Void size and connectivity control ease of water flow.

COMPRESSIBILITY



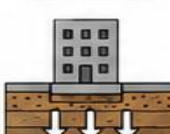
Structure governs void ratio change under load.

SHEAR STRENGTH



Particle contacts and bonding influence shear resistance.

SETTLEMENT



Affects magnitude and rate of settlement.

SHRINK-SWELL BEHAVIOR



Structure and mineralogy govern volume change with moisture variation.

COMPACTION RESPONSE



Determines achievable dry density and optimum moisture content.

5. ENGINEERING IMPORTANCE / APPLICATIONS

FOUNDATION DESIGN & SETTLEMENT ASSESSMENT



Predicts settlement magnitude and differential behavior.

EMBANKMENTS & EARTH DAMS



Controls stability, compressibility and seepage paths.

PAVEMENT SUBGRADE BEHAVIOR



Affects stiffness, resilient response and rutting.

SLOPE STABILITY & EROSION BEHAVIOR



Influences shear strength and susceptibility to erosion.

SEEPAGE & DRAINAGE PERFORMANCE



Controls flow networks and excess pore pressure.

GROUND IMPROVEMENT / STABILIZATION PLANNING



Helps select suitable techniques and materials.

SOIL CLASSIFICATION & SITE INVESTIGATION INTERPRETATION



Improves interpretation of field and laboratory data.

6. COMPARATIVE TABLE

TYPE	TYPICAL SOIL	PERMEABILITY	COMPRESSIBILITY	STABILITY
SINGLE-GRAINED STRUCTURE	Sand, Gravel	High	Low	High
HONEYCOMB STRUCTURE	Silt	Moderate	Moderate to High	Moderate
FLOCCULENT STRUCTURE	Clay (flocculated)	Moderate	High	Low to Moderate
DISPERSED STRUCTURE	Clay (dispersed)	Low	Moderate to High (slower consolidation)	Low



7. KEY IDEA / CONCLUSION

Soil structure is a key factor controlling the engineering behavior of soils.

Even soils with similar particle size may behave very differently because of differences in particle arrangement and bonding.

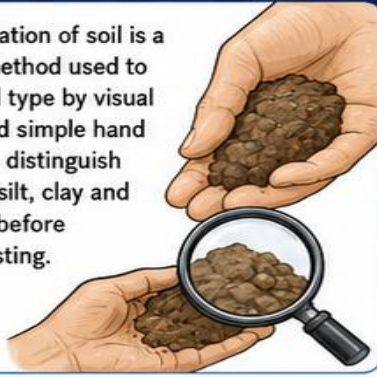


FIELD IDENTIFICATION METHODS FOR SOIL

Rapid visual and manual tests used for preliminary soil classification in the field

1. INTRODUCTION / PRINCIPLE

Field identification of soil is a preliminary method used to recognize soil type by visual inspection and simple hand tests. It helps distinguish gravel, sand, silt, clay and organic soils before laboratory testing.



2. COMMON FIELD IDENTIFICATION METHODS

1 VISUAL INSPECTION



Observe color, grain size, angularity, stratification, roots and organic matter.

2 FEEL TEST



Gritty feel indicates sand; smooth or floury feel indicates silt; sticky feel indicates clay.

3 PARTICLE-SIZE OBSERVATION



Identify coarse-grained (gravel, sand) vs fine-grained (silt, clay) soil.

4 DRY STRENGTH TEST



Dried pat crushed by fingers; high dry strength suggests clay, low suggests silt.

5 DILATANCY / SHAKING TEST



Moist soil in palm, shaken; quick water appearance and disappearance suggests silt, slow or none suggests clay.

6 PLASTICITY / THREAD TEST



Roll moist soil into a 3 mm thread; ability to form thread indicates plastic clayey soil.

TOUGHNESS TEST



Resistance of the thread near plastic limit; higher toughness indicates clay.

ORGANIC CONTENT / ODOR TEST



Dark color, fibrous matter, and decayed odor suggest organic soil.

3. TYPICAL FIELD CLUES FOR DIFFERENT SOILS

GRAVEL



- Visible large particles
- Very gritty
- Non-plastic
- Excellent drainage

SAND



- Visible grains
- Gritty
- Non-plastic to slightly plastic
- Rapid drainage

SILT



- Smooth / floury feel
- Low dry strength
- Quick dilatancy
- Slight plasticity

CLAY



- Sticky, plastic
- High dry strength
- No or slow dilatancy
- Shrink-swell tendency

ORGANIC SOIL



- Dark color
- Fibrous texture
- Odor (decayed)
- Compressible

4. SIMPLE FIELD TESTS – WHAT THEY INDICATE

GRITTY FEEL



Sand content high

SMOOTH FEEL



Silt content likely

STICKY & MOLDABLE



Clay present

STRONG DRY PAT



Higher clay fraction

QUICK WATER SHEEN ON SHAKING



Silty soil

THREAD CAN BE ROLLED



Plastic soil

5. ENGINEERING IMPORTANCE / APPLICATIONS

PRELIMINARY SOIL CLASSIFICATION



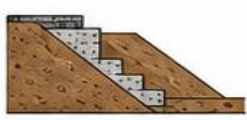
Quickly classify soil in the field for decision making.

SELECTION OF BORROW MATERIAL



Helps in selecting suitable borrow materials on site.

FOUNDATION AND EMBANKMENT ASSESSMENT



Assists in evaluating soil suitability and stability.

PAVEMENT SUBGRADE EVALUATION



Indicates subgrade quality for pavement design.

SITE INVESTIGATION PLANNING



Guides planning of sampling and testing locations.

DECIDING NEED FOR LABORATORY TESTING



Helps decide extent of laboratory tests required.

6. LIMITATIONS



Field identification is approximate and should be confirmed by laboratory tests such as sieve analysis, Atterberg limits, and moisture content determination.



Field identification methods provide quick, low-cost and useful preliminary information about soil behavior, but final design should rely on laboratory and geotechnical investigation results.



Thank You

Permeability of soil

Types of water in soil

- Two type:
 - Free water (moves under influence of gravity)
 - Held Water (don't moves under influence of gravity)
 - Structural water (water present in mineral structure)
 - Adsorbed water (hygroscopic water, air-dried)
 - Capillary water (due to surface tension)

Capillary rise in tube

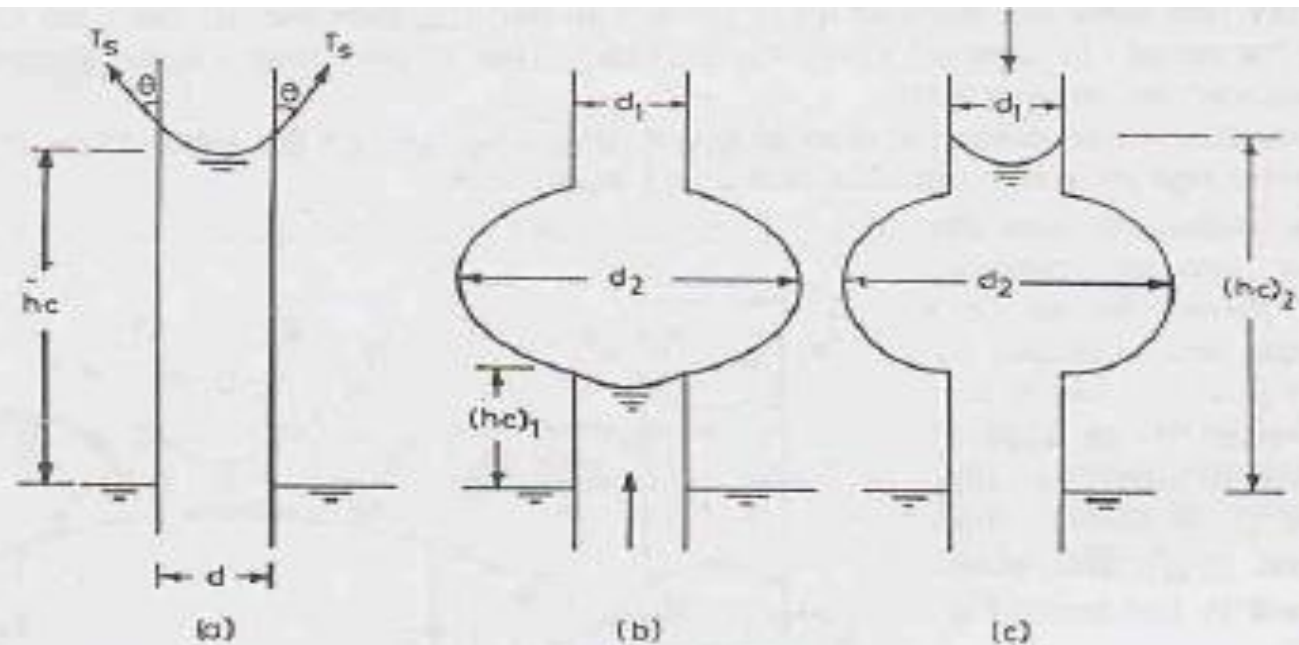


Fig. 7.2. Capillary Rise

F_u = Upward pull due to surface tension = $(T_s \cos \theta) \pi d$

where T_s = surface tension and d diameter of the tube.

F_d = Downward force due to mass of water in the tube

$$= \gamma_w (\pi/4 d^2) \times h_c$$

where h_c = height of capillary rise.

For equilibrium,

$$F_u = F_d$$

or

$$(T_s \cos \theta) \pi d = \gamma_w (\pi/4 d^2) h_c$$

or

$$h_c = \frac{4 T_s \cos \theta}{\gamma_w d} = \frac{4 T_s \cos \theta}{g \rho_w d}$$

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

Taking $T_s = 0.073 \text{ N/m}$, $\gamma_w = 9810 \text{ N/m}^3$,

$$h_c = \frac{4 \times 0.073}{9810 d} = \frac{3 \times 10^{-5}}{d} \text{ metres}$$

where d is in metres.

If d is in centimeters,

$$h_c = \frac{3 \times 10^{-3}}{d} \text{ metres}$$

If h_c and d both are in cm,

$$h_c = \frac{0.3}{d} \text{ cm}$$

- *pressure due to capillary rise* $= -\gamma_w h_c$

Capillary rise in soil

- $d = 0.2D_{10}$
- Terzaghi and Peck (1948)
- $$h_{cmax} = \frac{C}{eD_{10}}$$
- C= constant
depending upon the
shape of grain and
impurities

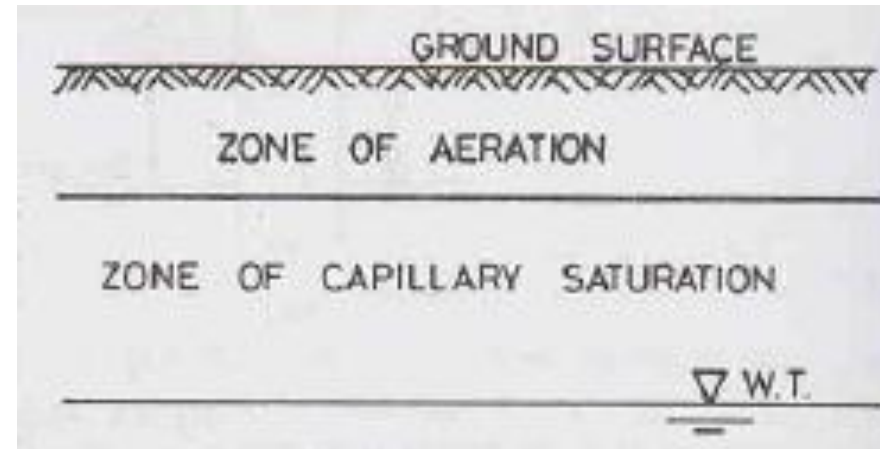


TABLE 7.11 Representative Heights of Capillary Rise

<i>S.No.</i>	<i>Soil Type</i>	<i>Capillary rise (m)</i>
1.	Fine gravel	0.02 to 0.10
2.	Coarse sand	0.10 to 0.15
3.	Fine sand	0.30 to 1.00
4.	Silt	1.0 to 10.0
5.	Clay	10.0 to 30.0
6.	Colloid	more than 30.0

Permeability of soils

- Property of soil which allow water to flow through it.
- Flow of water takes due to head difference.
- Total head (h) at a point is sum of elevation (datum) head, the pressure head and velocity head.
- Velocity in soil is very small so, velocity head is neglected.
- Total head loss per unit length is known as hydraulic gradient (i).
- $i = h/L$

Darcy's Law

- $V = ki$
- Where k = coefficient of permeability
 - i = hydraulic gradient
 - V = discharge velocity or superficial velocity
 - *Discharge* (q) = kiA

<i>S. No.</i>	<i>Soil Type</i>	<i>Coefficient of permeability (mm/sec)</i>	<i>Drainage properties</i>
1.	Clean gravel	10^{+1} to 10^{+2}	Very good
2.	Coarse and medium sands	10^{-2} to 10^{+1}	Good
3.	Fine sands, loose silt	10^{-4} to 10^{-2}	Fair
4.	Dense silt, clayey silts	10^{-5} to 10^{-4}	Poor
5.	Silty clay, clay	10^{-8} to 10^{-5}	Very poor

Determination of coefficient of permeability

- Laboratory Methods
 - Constant-head permeability test
 - Variable-head permeability test
- Field Methods
 - Pumping out tests
 - Pumping in tests

Constant Head Permeability Test

If the cross-sectional area of the specimen is A , the discharge is given by (Eq. 8.3)

$$q = k i A$$

or

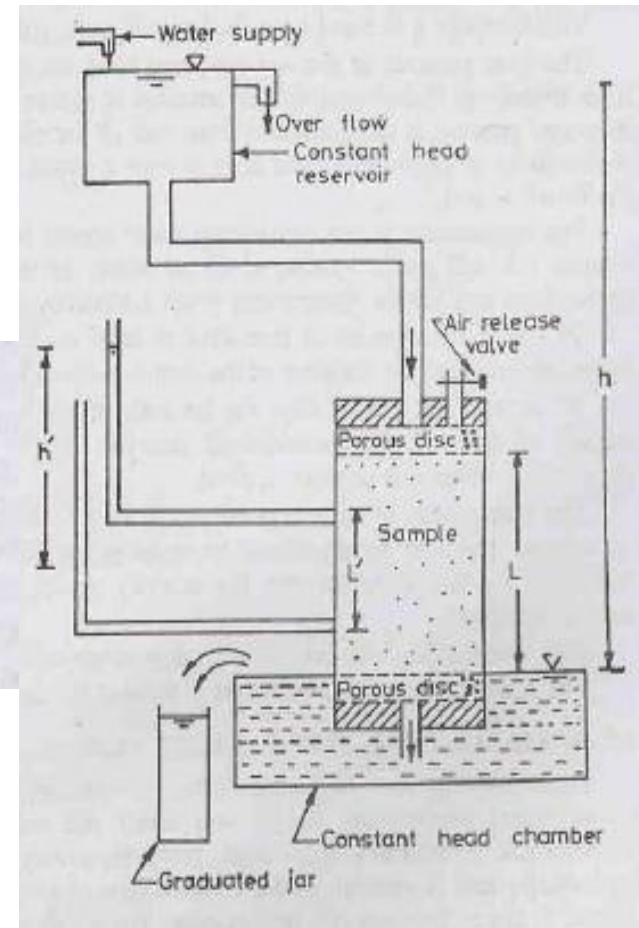
$$q = k \frac{h}{L} A$$

or

$$k = \frac{qL}{Ah}$$

where L = length of specimen, h = head causing flow.

The discharge q is equal to the volume of water collected divided by time.



Variable Head Permeability Test

$$a \, dh = -q \, dt$$

where a is cross-sectional area of the standpipe.

$$\text{or} \quad a \, dh = -(A \times k \times i) \times dt$$

$$\text{or} \quad a \, dh = -A k \times \frac{h}{L} \times dt$$

$$\text{or} \quad \frac{A k \, dt}{a L} = \frac{-dh}{h}$$

$$\text{Integrating,} \quad \frac{A k}{a L} \int_{t_1}^{t_2} dt = - \int_{h_1}^{h_2} \frac{dh}{h}$$

$$\text{or} \quad \frac{A k}{a L} (t_2 - t_1) = \log_e (h_1/h_2)$$

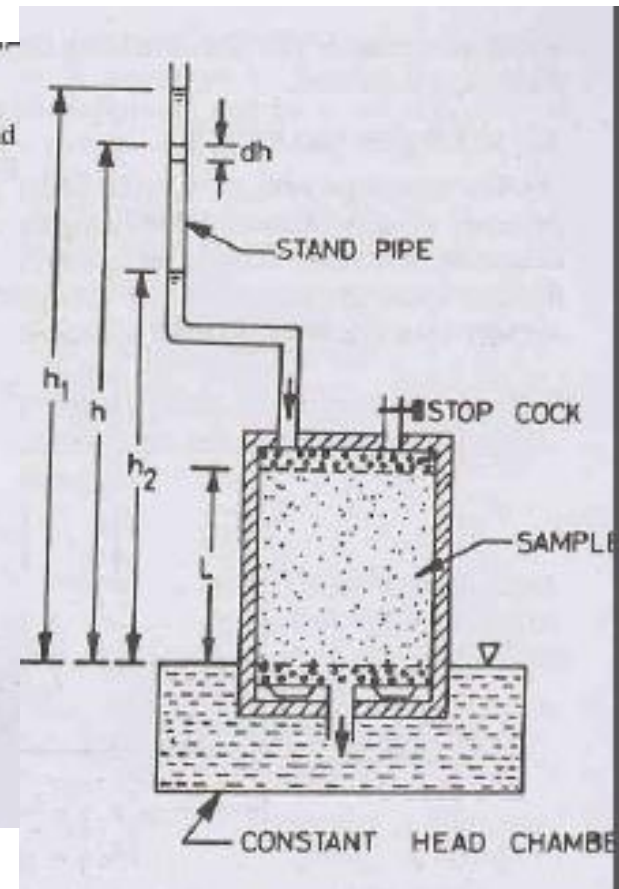
$$\text{or} \quad k = \frac{a L}{A t} \log_e (h_1/h_2)$$

where $t = (t_2 - t_1)$, the time interval during which the head reduces from h_1 to h_2 .

Eq. 8.6 is sometimes written as

$$k = \frac{2.30 a L}{A t} \log_{10} (h_1/h_2)$$

Fig. 8.3. Variable Head



Geotechnical Engineering-I

Lecture 12

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PUMP OUT TEST FOR UNCONFINED AQUIFER

In an unconfined aquifer, water is pumped out from a well and the drawdown in water level with radial distance from the well is measured at steady state. The coefficient of permeability is evaluated using Dupuit's assumption.

TEST PROCEDURE

1. A tube well of radius r_w is drilled in an unconfined aquifer.
2. The well penetrates the entire thickness of the aquifer.
3. Water is pumped out at a constant rate Q until steady state is reached.
4. The drawdown (s) at various radial distances ($r_1, r_2, \dots$) from the well is measured using piezometers.
5. The coefficient of permeability (k) is calculated using Dupuit's assumption.

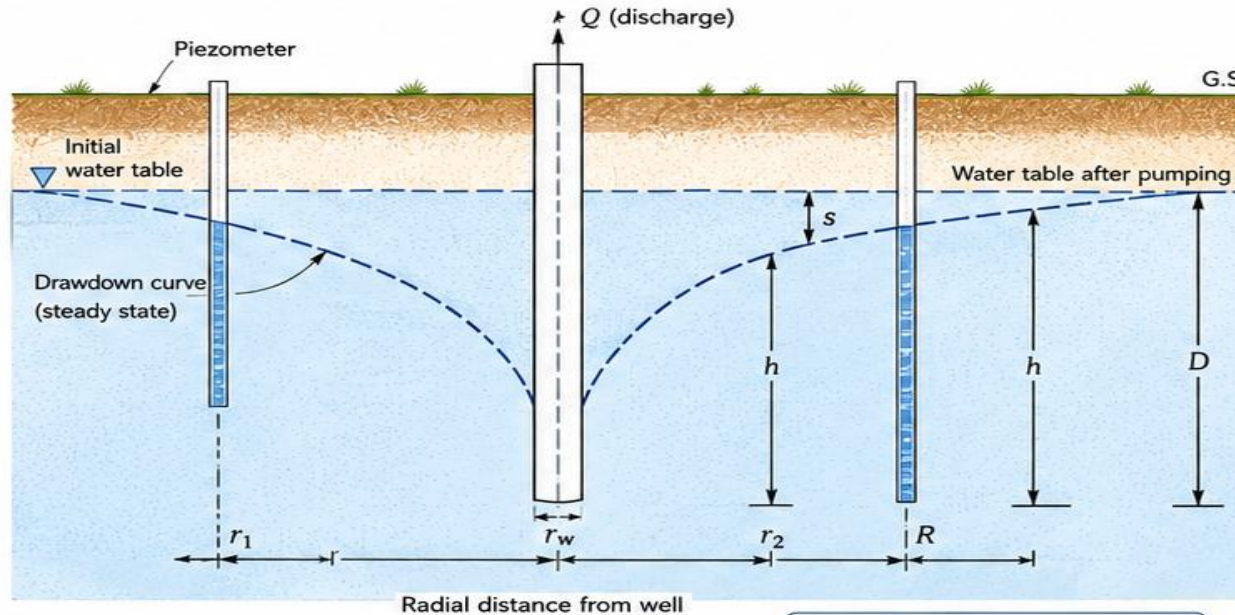
ASSUMPTIONS

1. Flow is steady.
2. Flow is laminar and Darcy's law is valid.
3. Soil mass is homogeneous, isotropic and fully saturated.
4. The coefficient of permeability is constant.
5. Flow is radial and horizontal.
6. Natural ground water table is horizontal.
7. Hydraulic gradient i is small.
8. The slope of the hydraulic gradient line is small and can be taken as the tangent at the point of measurement.

DUPUIT'S ASSUMPTION

The vertical component of flow is negligible and all the flow lines are horizontal. The vertical hydraulic gradient is assumed zero and the flow occupies the entire thickness h of the aquifer.

SCHEMATIC DIAGRAM



CONE OF DEPRESSION

The lowering of the water table around the well due to pumping forms a cone of depression. The maximum depression occurs at the well.

PLAN VIEW (FLOW NET)



EXPRESSION FOR COEFFICIENT OF PERMEABILITY (k)

Using Dupuit's assumption,

$$k = \frac{2.30 Q}{\pi (h_2^2 - h_1^2)} \log_{10} \left(\frac{r_2}{r_1} \right) \quad (\text{cm/s})$$

where,

Q = Discharge at steady state (cm^3/s)

r_1, r_2 = Radial distances from well (cm)
($r_2 > r_1$)

h_1, h_2 = Saturated thickness of aquifer at r_1 and r_2 respectively (cm)

$h = D - s$ = Saturated thickness of aquifer at a distance r (cm)

D = Original saturated thickness of aquifer (cm)

s = Drawdown at a distance r (cm)

k = Coefficient of permeability (cm/s)

NOTES

- Pumping is continued until steady state conditions are reached.
- Piezometers should be thin and properly screened.
- Radius of influence (R) should be large enough so that drawdown at R is small.
- All units must be consistent.

UNITS

k in cm/s (or m/s)
 $1 \text{ cm/s} = 10^{-2} \text{ m/s}$

DERIVATION SUMMARY

Discharge through a cylindrical surface of radius r and thickness h

$$q = k i A$$

$$= k \left(\frac{dh}{dr} \right) (2\pi r h)$$

At steady state, q is constant

$$q = 2\pi k h \frac{dh}{dr}$$

Integrating between r_1 and r_2 and h_1 and h_2

$$\int_{h_1}^{h_2} h dh = \frac{q}{2\pi k} \int_{r_1}^{r_2} \frac{dr}{r}$$

Solving for k

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

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

CONFINED AQUIFER AND PIEZOMETRIC SURFACE

CONFINED AQUIFER

A confined aquifer is a water-bearing layer confined between two impervious aquitards. The piezometric surface (P.S.) is the water table equivalent for a confined aquifer.

The water pressure at any point in the aquifer is indicated by the piezometric surface.

PIEZOMETRIC SURFACE (P.S.)

The piezometric surface is the level to which water will rise in a well tapping the confined aquifer. Initially, it is horizontal. When pumping starts, it is depressed and a cone of depression is formed around the well.

DERIVATION FROM DARCY'S LAW

Consider discharge through a cylindrical surface at radial distance r from the centre of the well and of height z . From Darcy's law,

$$q = kiA \quad \text{or} \quad q = k \left(\frac{dz}{dr} \right) (2\pi r b) \quad \dots(a)$$

Integrating,

$$[\log_e(r)]_{r_1}^{r_2} = \frac{2\pi k b}{q} [z]_{z_1}^{z_2}$$

or

$$k = \frac{q \log_e(r_2/r_1)}{2\pi b(z_2 - z_1)} \quad \dots(8.24)$$

or

$$k = \frac{2.30 q \log_{10}(r_2/r_1)}{2\pi b(z_2 - z_1)} \quad \dots(8.25)$$

where z_1 = height of water level in observation well (1) at radial distance of r_1
 z_2 = height of water level in observation well (2) at radial distance of r_2

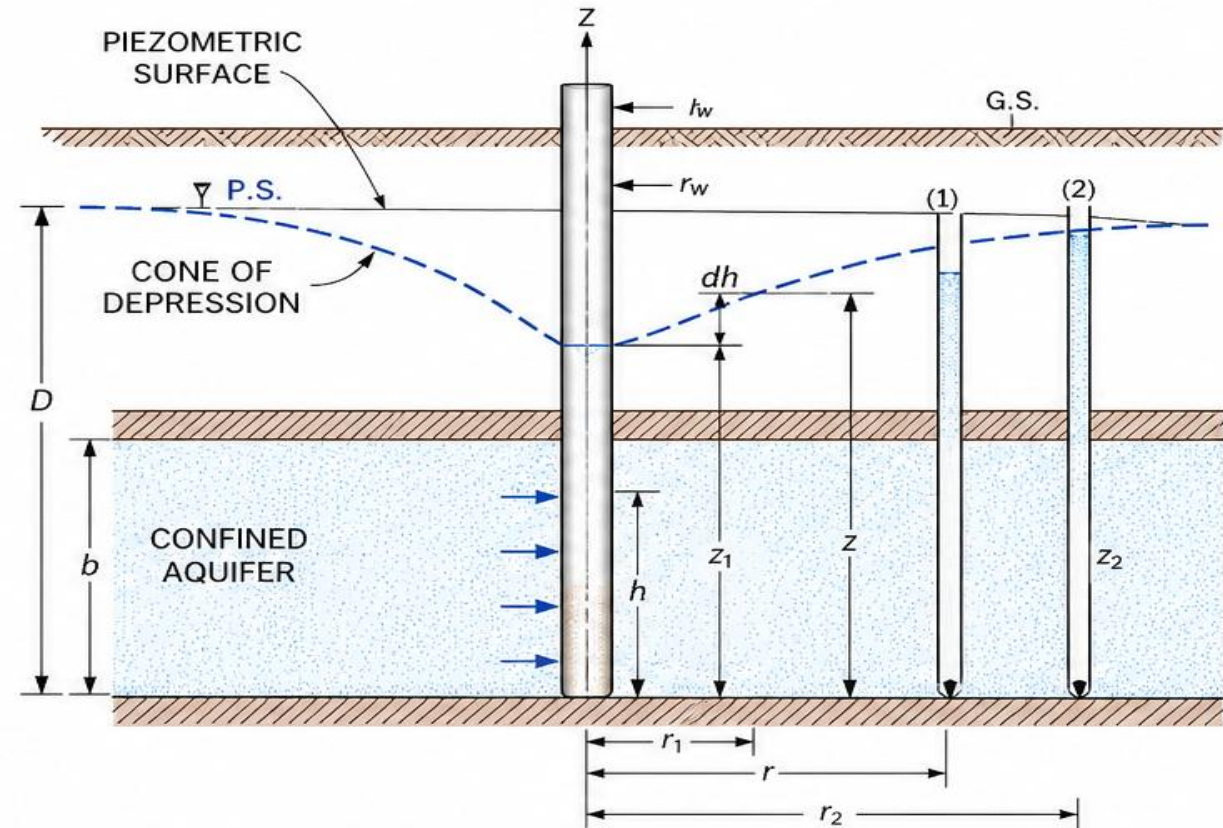
As in the case of an unconfined aquifer, an approximate value of k can be determined if the radius of influence R is known or estimated. In this case,

$$k = \frac{q \log_e(R/r_w)}{2\pi b(D - h)} \quad \dots(8.26)$$

Key Symbols

D = Depth of aquifer
 b = Thickness of aquifer
 z = Height of point above base of aquifer
 h = Depth of water in well
 R = Radius of influence
 r = Radial distance

CROSS-SECTION OF CONFINED AQUIFER



SUMMARY

- The piezometric surface is the level to which water rises in a tightly cased well tapping a confined aquifer.
- Initial piezometric surface is horizontal.
- Pumping lowers the piezometric surface forming a cone of depression.
- Coefficient of permeability (k) can be determined using observation wells or radius of influence.

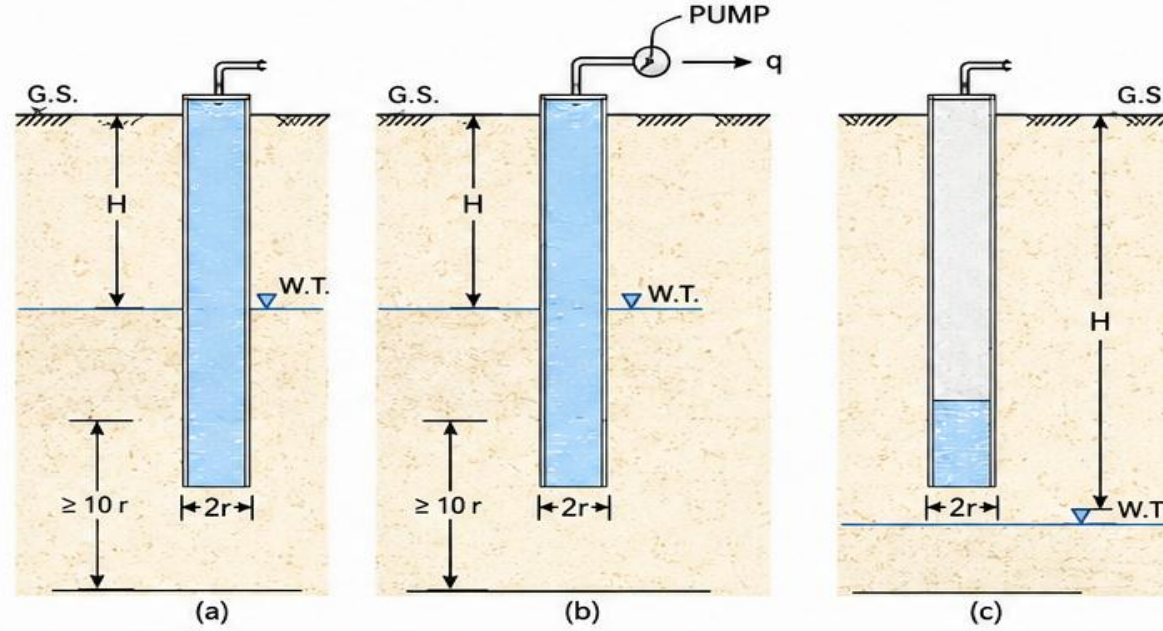
PUMPING OUT TEST (OPEN-END TEST) FOR PERMEABILITY OF SOIL

Pumping-out tests give more reliable values of the coefficient of permeability of stratum just close to the hole. In open-end test, water is pumped out from the bore hole and the rate of fall of water level is measured.

TEST PROCEDURE

1. A bore hole of diameter $2r$ is drilled to the desired depth and cleaned.
2. A pipe casing with water inlet is inserted into the hole and water is added to keep the water level at the top of the hole.
3. The water is pumped out and the rate of fall of water level is measured by a metering system.
4. The discharge (q) at steady state is noted corresponding to the difference in levels (H) between the inlet to the casing and the water table.
5. The coefficient of permeability (k) is calculated using the following expression.

SCHEMATIC DIAGRAM



$2r$ = Diameter of bore hole (cm)
 H = Difference of levels between inlet to the casing and water table (cm)
W.T. = Water table
G.S. = Ground surface
 q = Discharge (cm^3/s)

EXPRESSION

The coefficient of permeability is given by

$$k = \frac{q}{5.5 r H} \quad (\text{cm/s})$$

where,

k = coefficient of permeability (cm/s)

q = discharge at steady state (cm^3/s)

r = inside radius of the casing (cm)

H = difference of levels between inlet to the casing and water table (cm)

NOTES

- The test gives the permeability of the soil surrounding the bore hole.
- The lower end of the pipe should be at a distance of not less than $10r$ from the top as well as from the bottom of the stratum.

TYPICAL OBSERVATIONS

Time (t) (s)	Water level in casing (cm) (h)	Discharge collected (cm^3) (V)	Discharge (q) (cm^3/s)
t_1	h_1	—	—
t_2	h_2	V	$q = V / (t_2 - t_1)$
$\vdots$	$\vdots$	$\vdots$	$\vdots$
t_n	h_n	V	$q = V / (t_n - t_{n-1})$

DEFINITION OF TERMS

Steady condition : A condition in which the rate of fall of water level in the casing becomes uniform with time.

Discharge (q) : Quantity of water pumped out per unit time at steady condition.

Permeability (k) : A measure of the ease with which water can flow through soil.

UNITS

k in cm/s (or m/s)

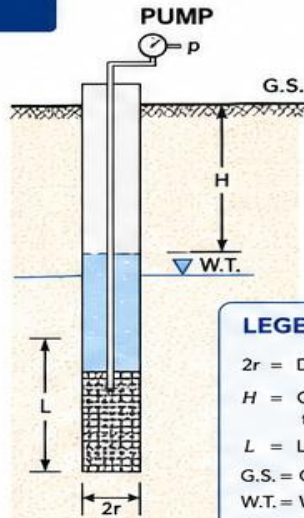
1 cm/s = 10^{-2} m/s

PUMPING IN TESTS FOR PERMEABILITY OF SOIL

In pumping-in tests, water is pumped into the bore hole under pressure and the head required to maintain a constant rate of flow is measured.

1. OPEN-END TESTS

1. A pipe casing is inserted into the bore hole to the desired depth and the hole is cleaned.
2. Water is pumped into the hole and the rate of rise of water level is measured.
3. The water level is maintained constant at a height H above the ground water table.
4. The coefficient of permeability (k) is calculated using the following equations.



(a) Open-end test

LEGEND

$2r$ = Diameter of bore hole (cm)
 H = Constant head above water table (cm)
 L = Length of hole tested (cm)
 G.S. = Ground surface
 W.T. = Water table
 p = Total head ($H + p/\gamma_w$)
 q = Discharge at steady state (cm^3/s)

EQUATION (OPEN-END TEST)

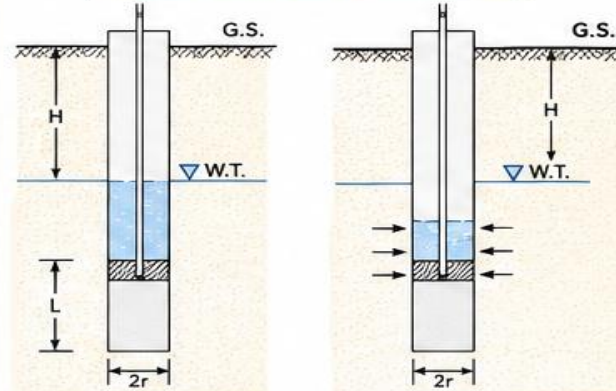
For $L \geq 10r$ For $10r > L \geq r$

$$k = \frac{q}{2\pi LH} \log_e \left(\frac{L}{r} \right) \quad k = \frac{q}{2\pi LH} \sinh^{-1} \left(\frac{L}{2r} \right) \quad (\text{cm/s})$$

where q = discharge at steady state (cm^3/s)

2. PACKER TESTS

(a) SINGLE-PACKER TEST



(a) Before pumping

(b) During pumping

EQUATION (SINGLE-PACKER TEST)

For $L \geq 10r$

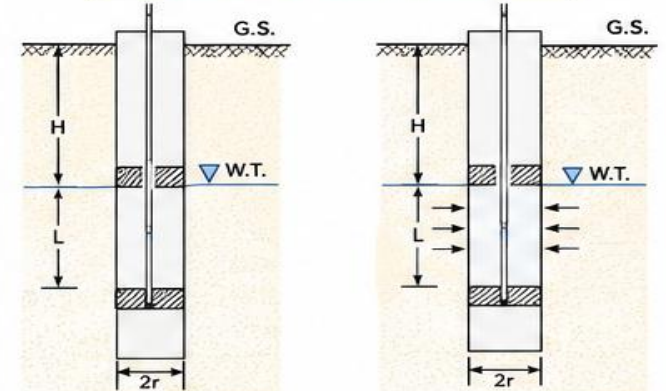
$$k = \frac{q}{2\pi LH} \log_e \left(\frac{L}{r} \right) \quad (\text{cm/s})$$

For $10r > L \geq r$

$$k = \frac{q}{2\pi LH} \sinh^{-1} \left(\frac{L}{2r} \right) \quad (\text{cm/s})$$

where,
 q = discharge at steady state (cm^3/s)

(b) DOUBLE-PACKER TEST



(a) Before pumping

(b) During pumping

EQUATION (DOUBLE-PACKER TEST)

For $L \geq 10r$

$$k = \frac{q}{2\pi LH} \log_e \left(\frac{L}{r} \right) \quad (\text{cm/s})$$

For $10r > L \geq r$

$$k = \frac{q}{2\pi LH} \sinh^{-1} \left(\frac{L}{2r} \right) \quad (\text{cm/s})$$

where,
 q = discharge at steady state (cm^3/s)

GENERAL NOTES

- The lower end of the pipe should be at a distance of not less than $10r$ from the top as well as from the bottom of the stratum.
 - In packer tests, the casing should be properly sealed with packers to prevent leakage.
 - When pumping is under pressure (p), the measured head becomes ($H + p/\gamma_w$).
- γ_w = Unit weight of water

COMPARISON OF TESTS

Test Type	Open-end test	Single-packer test	Double-packer test
Casing	No packer	One packer	Two packers
Flow area	Whole open length	Lower end (tested interval)	Between two packers
Use	Permeability of unconfined stratum	Permeability of bottom portion	Permeability of any selected layer
Accuracy	Less accurate	More accurate	Most accurate

DEFINITION OF TERMS

$2r$: Diameter of bore hole (cm)
 H : Difference of levels between entry (or top packer) and water table (cm)
 L : Length of hole or test section (cm)
 p : Pressure of water applied (kg/cm^2)
 p/γ_w : Equivalent head due to applied pressure (cm)
 $H + p/\gamma_w$: Total head during pumping (cm)
 q : Discharge at steady state (cm^3/s)
 k : Coefficient of permeability (cm/s)
 r : Inside radius of bore hole (cm)

UNITS

k in cm/s (or m/s)
 $1 \text{ cm/s} = 10^{-2} \text{ m/s}$

COEFFICIENT OF PERMEABILITY BY INDIRECT METHODS

The value of the coefficient of permeability of a soil can be estimated by using indirect methods, without conducting laboratory or field tests. The following methods are commonly used.

1. Allen Hazen's Formula

Allen Hazen conducted a large number of tests on filter sands of particle size between 0.1 mm and 3.0 mm, having a coefficient of uniformity of less than 5, and gave the following relation :

$$k = C D_{10}^2 \quad (\text{cm/sec}) \quad \dots (8.30)$$

where k = coefficient of permeability (cm/sec)
 D_{10} = effective size (cm)
 C = constant, with a value between 100 and 150

- If k and D_{10} are taken in mm/sec and mm, respectively, the value of the constant C lies between 10 and 15.
- Although Allen Hazen formula was derived for uniform sands in a loose state of compaction, it is frequently used for other soils. However, the computed values may be in error by $\pm 100\%$.

2. Kozeny-Carman Equation

The coefficient of the permeability of a soil can be estimated using the Kozeny-Carman equation:

$$k = \frac{g \rho_w}{(C_1 \mu S)^2} \cdot \frac{e^3}{1 + e} \quad (\text{cm/sec}) \quad \dots (8.31)$$

where k = coefficient of permeability (cm/sec) $g = 981 \text{ cm/sec}^2$,
 ρ_w = mass density of water (gm/ml) S = specific surface (cm^2/cm^3)
 C_1 = shape factor, which can be taken as 2.5 for granular soils
 μ = coefficient of viscosity (poise)
 e = void ratio

3. Loudon's Formula

Loudon gave the following empirical formula:

$$\log_{10}(kS^n) = a + bn \quad (\text{cm/sec}) \quad \dots (8.35)$$

where k = coefficient of permeability (cm/sec) a = constant, with an average value of 1.365 at 10°C
 S = specific surface (cm^2/cm^3) b = constant, with an average value of 0.7
 n = porosity, expressed as a ratio

- The value of f depends upon the angularity of the particles. This value is usually taken as 1.1 for rounded sands, 1.25 for sands of medium angularity and 1.4 for angular sands.

- If $M_1, M_2, \dots, M_n$ are the percentage of the total soil sample retained on different sieves, the overall specific surface of the total sample is given by

$$S = f(M_1 S_1 + M_2 S_2 + \dots + M_n S_n) \quad \dots (8.34)$$

where $S_1, S_2, \dots, S_n$ are the specific surface of spheres uniformly distributed within the corresponding sieves.

SPECIFIC SURFACE (S)

For computation of k from Eq. (8.31), the value of specific surface S is required.

- For a particle of any shape, if the specific surface (S) of the particle is equal to the surface area of the particle per unit volume.
- For a spherical particle of diameter D , specific surface (S) is given by

$$S = \frac{6}{D} \quad (\text{cm}^2/\text{cm}^3) \quad \dots (8.32)$$

The specific surface of spheres uniformly distributed in size between the mesh size a and b is given by

$$S = \frac{6\sqrt{ab}}{ab} \quad (\text{cm}^2/\text{cm}^3) \quad \dots (8.33)$$

For accurate results, the ratio a/b should not be greater than 2.

FACTOR (f) FOR PARTICLE SHAPE (ANGULARITY)

Particle shape (angularity)	Factor, f
Rounded sands	1.1
Medium angular sands	1.25
Angular sands	1.40

UNITS

k in cm/s (or m/s)
 $1 \text{ cm/s} = 10^{-2} \text{ m/s}$

CAPILLARITY-PERMEABILITY TEST



1. OBJECTIVE

Determine the coefficient of permeability (k) of soil in unsaturated condition using capillary flow.



2. APPARATUS

- Transparent tube to hold dry soil sample
- Lower water reservoir and upper water reservoir
- Screen at the outlet end
- Air vent pipe at the outlet end
- Tube/valve arrangement to connect reservoirs and maintain water level
- Imaginary manometer to read capillary head



3. WORKING PRINCIPLE

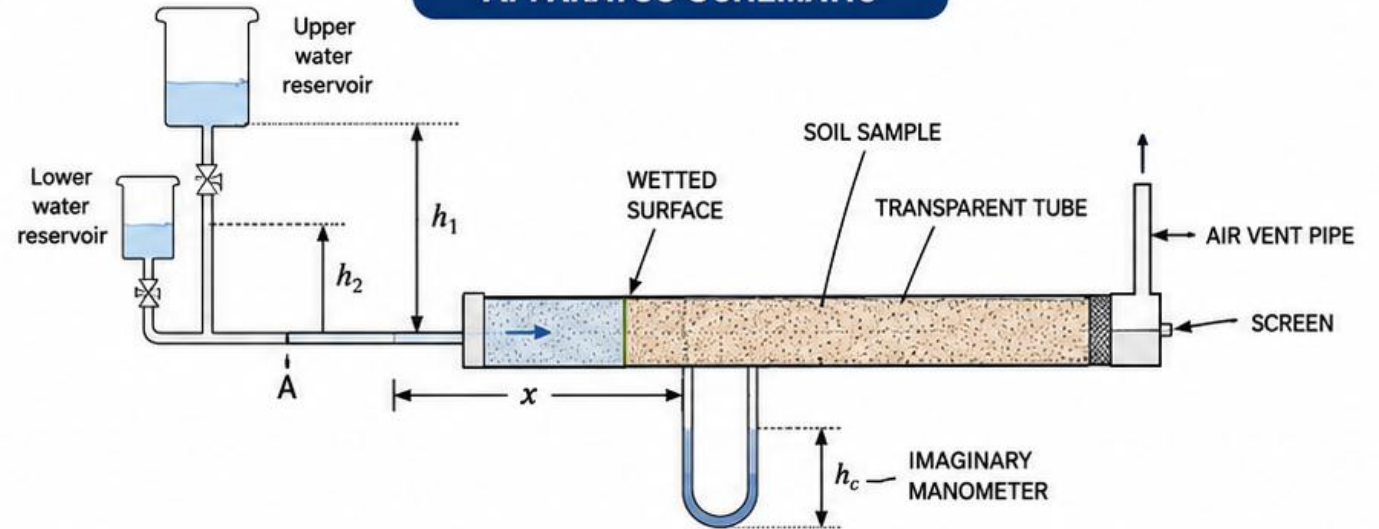
- Water from the lower reservoir enters the transparent tube and wets the soil at the inlet.
- The wetted surface (wetting front) advances through the soil sample for a distance x with time.
- The soil is in unsaturated condition; hence the head in the tube is negative and is considered accordingly.
- Coefficient of permeability is obtained by observing the movement of the wetting front with time.



4. KEY ASSUMPTIONS

- Uniform hydraulic gradient across the length x .
- Darcy's law is applicable to the unsaturated flow.

APPARATUS SCHEMATIC



MAIN EQUATIONS

Total head causing flow (negative):

$$h = h_1 + h_c$$

Darcy's law:

$$v = ki = k \frac{(h_1 + h_c)}{x}$$

Seepage velocity:

$$v_s = \frac{v}{n}$$

For partially saturated soil:

$$v_s = \frac{k_u}{S_n} \frac{(h_1 + h_c)}{x}$$

Rate of advance of wetting front:

$$\frac{dx}{dt} = \frac{k_u}{S_n} \frac{(h_1 + h_c)}{x}$$

Integrated form (between t_1, x_1 and t_2, x_2):

$$\frac{x_2^2 - x_1^2}{t_2 - t_1} = \frac{2k_u (h_1 + h_c)}{S_n}$$

A second equation is obtained by changing the head from h_1 to h_2 and repeating the test. Solving the two equations gives k_u and h_c .

SYMBOLS

k_u	Unsaturated coefficient of permeability
S	Degree of saturation
n	Porosity
x	Distance traveled by wetting front from inlet
h_1	Head in the tube for the first test (negative)
h_2	Head in the tube for the second test (negative)
h_c	Capillary head (negative)



TAKEAWAY

By measuring the advance of the wetting front (x) with time (t) under known heads (h_1, h_2) and capillary head (h_c), the unsaturated coefficient of permeability (k_u) of soil can be estimated.

Thank You

Geotechnical Engineering-I

Lecture 13

**Presented by
Dr. Akash Priyadarshee
Department of Civil Engineering
Muzaffarpur Institute of Technology, Muzaffarpur**

PERMEABILITY OF STRATIFIED SOIL DEPOSITS



CONCEPT



A stratified soil deposit consists of several soil layers having different permeabilities.

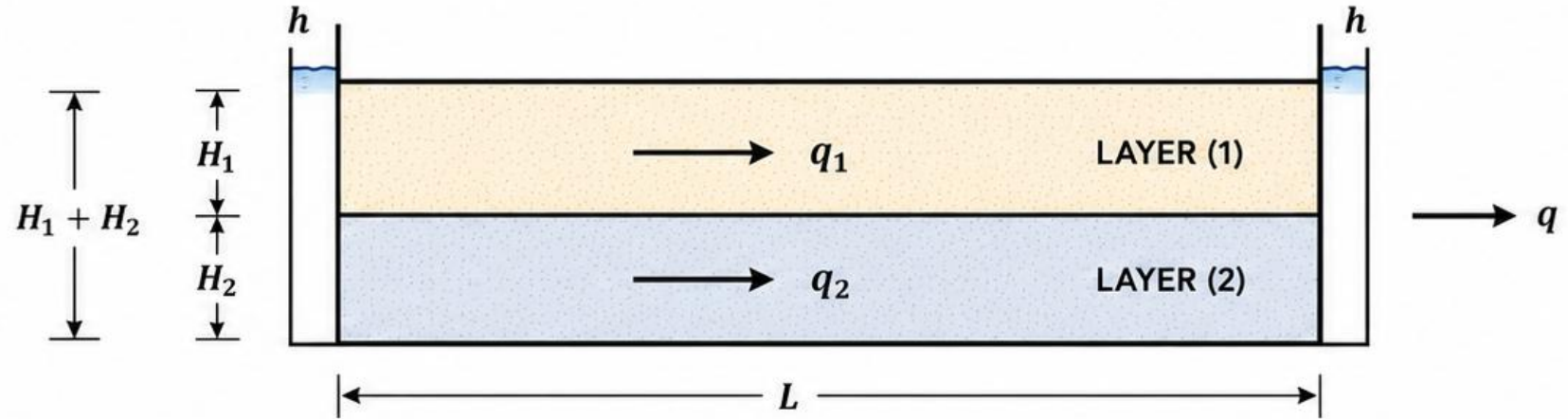


The average permeability depends on the direction of flow relative to the planes of stratification.



In this slide, we focus on **Flow Parallel to Planes of Stratification**.

FLOW PARALLEL TO PLANES OF STRATIFICATION



H_1, H_2 = Thickness of layers (1) and (2)

h = Head at inlet and outlet (same head loss in each layer)

L = Length of deposit

q_1, q_2 = Discharge per unit width in each layer

q = Total discharge per unit width



KEY RELATIONS

- 1 Total discharge per unit width:
- 2 Using Darcy's law for parallel flow:
- 3 Equivalent permeability for two layers:
- 4 For n layers:

$$q = q_1 + q_2$$

$$k_h \cdot i \cdot (H_1 + H_2) = k_1 \cdot i \cdot H_1 + k_2 \cdot i \cdot H_2$$

$$k_h = \frac{k_1 H_1 + k_2 H_2}{H_1 + H_2}$$

$$k_h = \frac{k_1 H_1 + k_2 H_2 + \dots + k_n H_n}{H_1 + H_2 + \dots + H_n}$$



IMPORTANT IDEA

- ✓ For flow parallel to stratification, the hydraulic gradient is the **same in each layer**.
- ✓ The total discharge equals the **sum of the discharges** through individual layers.
- ✓ Therefore, the equivalent permeability is the **thickness-weighted arithmetic mean**.



TAKEAWAY

For flow parallel to the planes of stratification, equivalent permeability is obtained by weighting each layer permeability with its thickness.

PERMEABILITY OF STRATIFIED SOIL DEPOSITS

Flow Normal to the Plane of Stratification

CONCEPT



A stratified soil deposit contains multiple soil layers with different permeabilities.

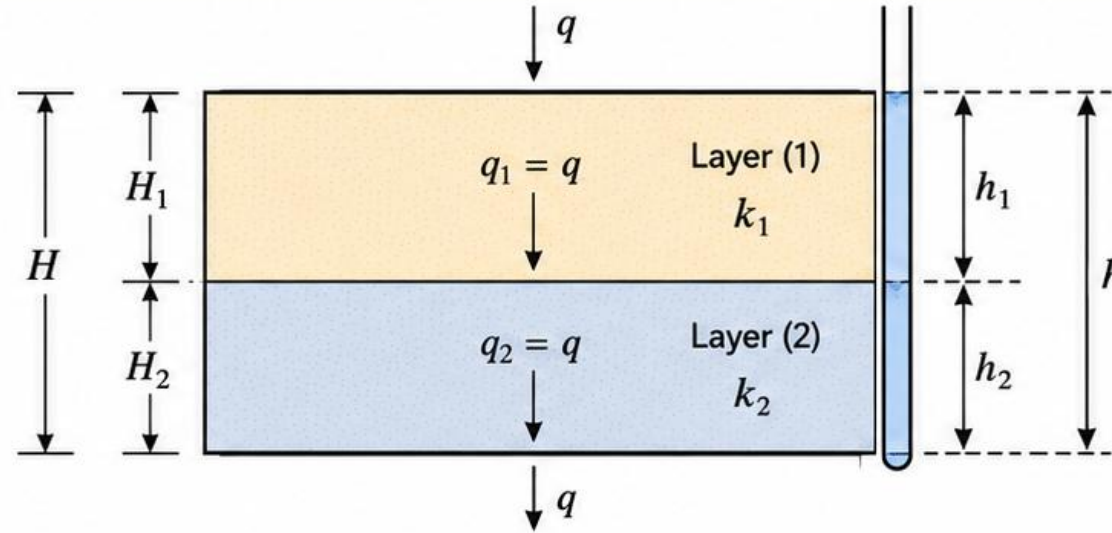


In this case, flow occurs normal (perpendicular) to the planes of stratification.



The discharge through each layer is the same, similar to electrical resistances in series.

FLOW NORMAL TO PLANES OF STRATIFICATION



IMPORTANT IDEA

- ✓ For flow normal to stratification, discharge is the same through every layer.
- ✓ Head loss is shared among the layers.
- ✓ Therefore, the equivalent permeability is the **thickness-weighted harmonic mean** of the layer permeabilities.
- ✓ This condition is analogous to resistances in series.

KEY RELATIONS



$$q = q_1 = q_2$$

$$k_v i_v \times 1 = k_1 i_1 \times 1 = k_2 i_2 \times 1$$

$$i_1 = \frac{k_v}{k_1} i_v$$

$$i_2 = \frac{k_v}{k_2} i_v$$

$$h = h_1 + h_2$$

$$i_v H = i_1 H_1 + i_2 H_2$$

$$k_v = \frac{H_1 + H_2}{\frac{H_1}{k_1} + \frac{H_2}{k_2}}$$

For n layers:

$$k_v = \frac{H_1 + H_2 + \dots + H_n}{\frac{H_1}{k_1} + \frac{H_2}{k_2} + \dots + \frac{H_n}{k_n}}$$



SYMBOLS

k_1, k_2	permeability of layers 1 and 2
k_v	equivalent permeability normal to stratification
i_1, i_2	hydraulic gradients in layers 1 and 2
i_v	overall hydraulic gradient
H_1, H_2	thicknesses of layers 1 and 2
H	total thickness of deposit
q	discharge per unit width
h, h_1, h_2	total and individual head losses



TAKEAWAY

For flow perpendicular to soil layering, the equivalent permeability is obtained by dividing total thickness by the sum of thickness-to-permeability ratios.

LAMINAR FLOW THROUGH POROUS MEDIA

Generalized Hagen–Poiseuille approach for soil permeability

1. CONCEPT

- For laminar flow through porous media, the generalized Hagen–Poiseuille equation can be applied.
- In porous media, the area of flow passage equals the porosity times the total cross-sectional area.
- The hydraulic radius relates the void geometry to the seepage flow.
- This leads to a general expression for the coefficient of permeability of soil.

2. GENERALIZED FLOW EQUATION

$$q = C_1 \left(\frac{\gamma_w i}{\mu} \right) R_H^2 A \quad \dots(8.14)$$

C_1 depends on the shape of the conduit

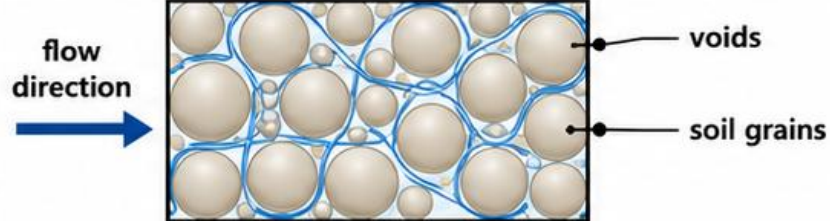
$$q = C_1 \left(\frac{\gamma_w i}{\mu} \right) R_H^2 (nA)$$

n = porosity

7. SYMBOLS

- q = discharge
 v = Darcy velocity
 k = coefficient of permeability
 i = hydraulic gradient
 μ = dynamic viscosity of water
 γ_w = unit weight of water
 R_H = hydraulic radius
 A = total cross-sectional area
 A_v = area of void flow
 P_v = wetted perimeter
 a_s or A_s = surface area of flow channel
 n = porosity
 e = void ratio
 D = equivalent grain diameter
 C, C_1 = shape coefficients

3. HYDRAULIC RADIUS

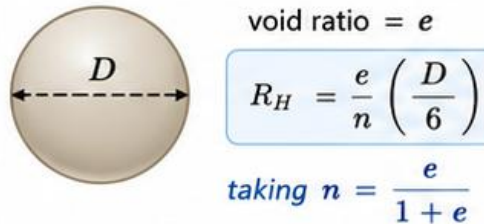


$$R_H = \frac{\text{area of flow}}{\text{wetted perimeter}} = \frac{A_v}{P_v}$$

$$R_H = \frac{A_v \times L}{P_v \times L}$$

$$R_H = \frac{\text{volume of flow channel}}{\text{surface area of flow channel}} = \frac{V_v}{A_s}$$

4. FOR SPHERICAL GRAINS



5. DERIVED EXPRESSIONS

$$q = C_1 \left(\frac{\gamma_w i}{\mu} \right) \left(\frac{eD}{6} \right)^2 \left(\frac{e}{1+e} \right) iA$$

Replacing $\frac{C_1}{36}$ by C

$$q = \frac{C_1}{36} \left(\frac{\gamma_w i}{\mu} \right) \left(\frac{e^3}{1+e} \right) D^2 iA$$

$$q = C \left(\frac{\gamma_w i}{\mu} \right) \left(\frac{e^3}{1+e} \right) D^2 iA$$

$$v = \frac{q}{A} = C \left(\frac{\gamma_w i}{\mu} \right) \left(\frac{e^3}{1+e} \right) D^2 i = ki$$

$$k = C \left(\frac{\gamma_w i}{\mu} \right) \left(\frac{e^3}{1+e} \right) D^2 \quad \dots(8.15)$$



For laminar flow through porous soil, permeability is proportional to grain size squared and strongly influenced by void ratio and fluid properties.

Thank You

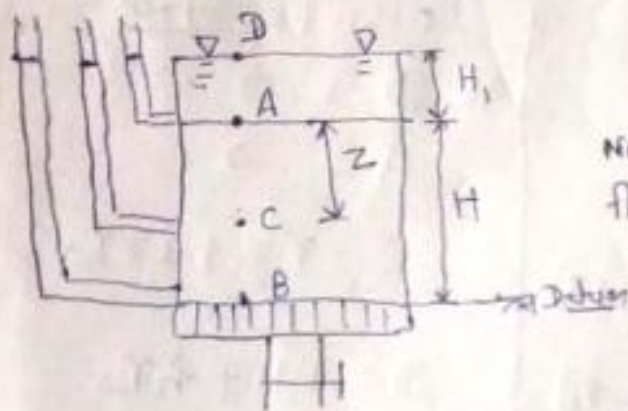
Unit 4 Effective stress

Total Stress(σ) = Effective stress(σ') + Pore pressure(u)

$$\sigma = \sigma' + u$$

Effective stress for different flow condition:-

No-flow condition:-



	$\frac{T}{H}$	$\frac{P}{H}$	$\frac{D}{H}$	
	H_1+H	0	(H_1+H)	D
		H_1	H	A
No flow	$H+H_1$	H_1+Z	$(H-Z)$	C
	$H+H_1$	H_1+H	0	B

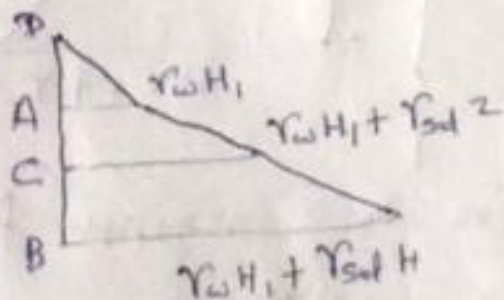
Total Stress

$$\text{At } D, = 0$$

$$A = \gamma_w H_1$$

$$C = \gamma_w H_1 + \gamma_{sat} Z$$

$$B = \gamma_w H_1 + \gamma_{sat} H$$



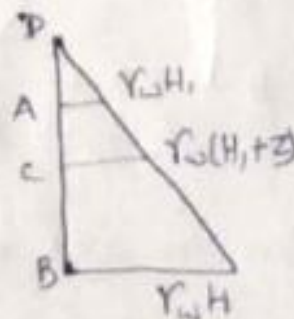
Pore pressure

$$\text{At } D = 0$$

$$A = \gamma_w H_1$$

$$C = \gamma_w (H_1 + Z)$$

$$B = \gamma_w H$$



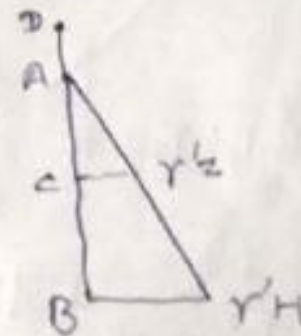
Effective Stress

$$\text{At } D = 0$$

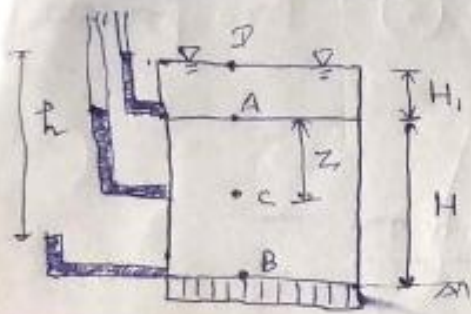
$$A = 0$$

$$C = \gamma' Z$$

$$B = \gamma' H$$



Downward flow: →



	Datum Head	PH	γH
D	$H_1 + H$	0	$H_1 + H$
A	H	H_1	$H_1 + H$
C	$H - z$	$H_1 + z - iz$	$H_1 + H - iz$
B	0	$H_1 + H - h$	$H_1 + H - h$

$$i = \frac{h}{H}$$

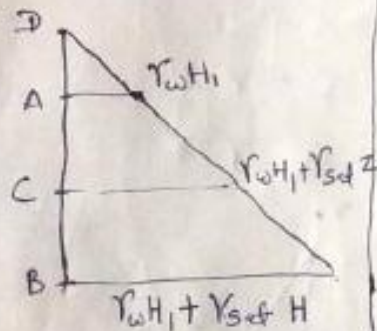
Total Stress

$$At, D = 0$$

$$A = \gamma_w H_1$$

$$C = \gamma_w H_1 + \gamma_{sat} z$$

$$B = \gamma_w H_1 + \gamma_{sat} H$$



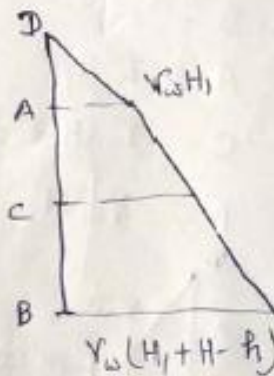
Pore Pressure

$$At, D = 0$$

$$A = \gamma_w H_1$$

$$C = \gamma_w (H_1 + z - iz)$$

$$D = \gamma_w (H_1 + H - h)$$



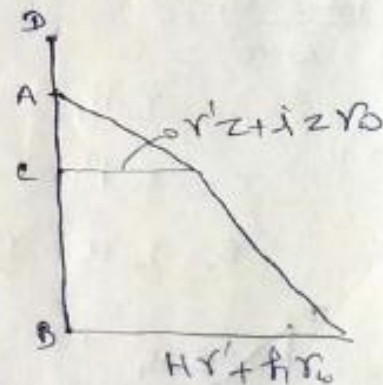
Effective Stress

$$At, D = 0$$

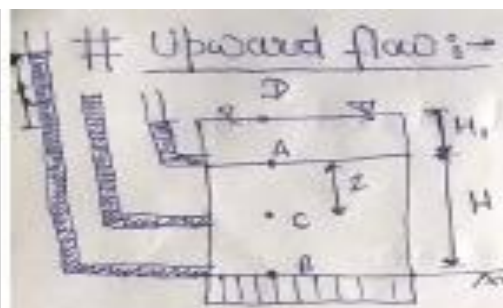
$$A = 0$$

$$C = \gamma' z + iz \gamma_w$$

$$D = \gamma' H + h \gamma_w$$



So, Due to Seepage Downwards effective stress increases for z depth by $\boxed{iz \gamma_w}$ → for H depth → $iH \gamma_w$



	DH	PH	TH	$i = \frac{h}{z}$
D	$H_1 + H$	0	$H_1 + H$	
A	H	H_1	$H_1 + H$	
C	$H - z$	$H_1 + i z$	$H_1 + H + i z$	
B	0	$H_1 + H + h$	$H_1 + H + h$	

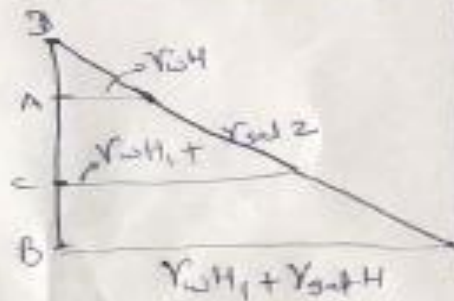
Total Stress

At, $D = 0$

$A = \gamma_w H_1$

$C = \gamma_w H_1 + \gamma_w z$

$B = \gamma_w H_1 + \gamma_w H$



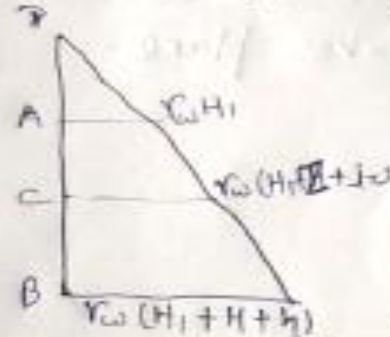
Pore Pressure

At, $D = 0$

$A = \gamma_w H_1$

$C = \gamma_w (H_1 + i z)$

$B = \gamma_w (H_1 + H + h)$



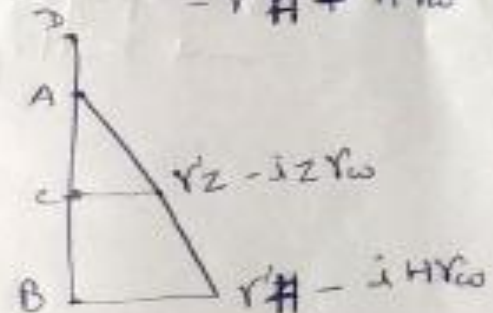
Effective Stress

At, $D = 0$

$A = 0$

$C = \gamma' z - i z \gamma_w$

$B = \gamma' H - i H \gamma_w$
 $= \gamma' H - h \gamma_w$



* So, Due to seepage upward effective stress decreases for z depth by $i z \gamma_w \rightarrow$ for H depth $\rightarrow i H \gamma_w$

$$\text{Seepage force per unit volume} = \frac{i H \gamma_w \times A}{A \times H} = i \gamma_w$$

Quick Sand Conditions →

* When seepage take place in upward direction this type of phenomenon takes place.

$$\gamma' H - i H \gamma_w = 0 = \text{effective stress zero}$$

$$\Rightarrow \gamma' - i \gamma_w = 0$$

$$\Rightarrow i = \frac{\gamma'}{\gamma_w} = \frac{(G-1) \gamma_w}{(1+e) \gamma_w} = \frac{G-1}{1+e}$$

$$\therefore \boxed{i_c = \frac{G-1}{1+e}} = \text{critical hydraulic gradient}$$

* If seepage pressure exceed weight of soil Sand boiling takes place.

Unit-5

Compaction Test

Outline of presentation

- What is Compaction?
- Laboratory test of compaction
- Factors affecting compaction
- How to do compaction in Field?
- Impact of compaction on the properties of soil

Compaction

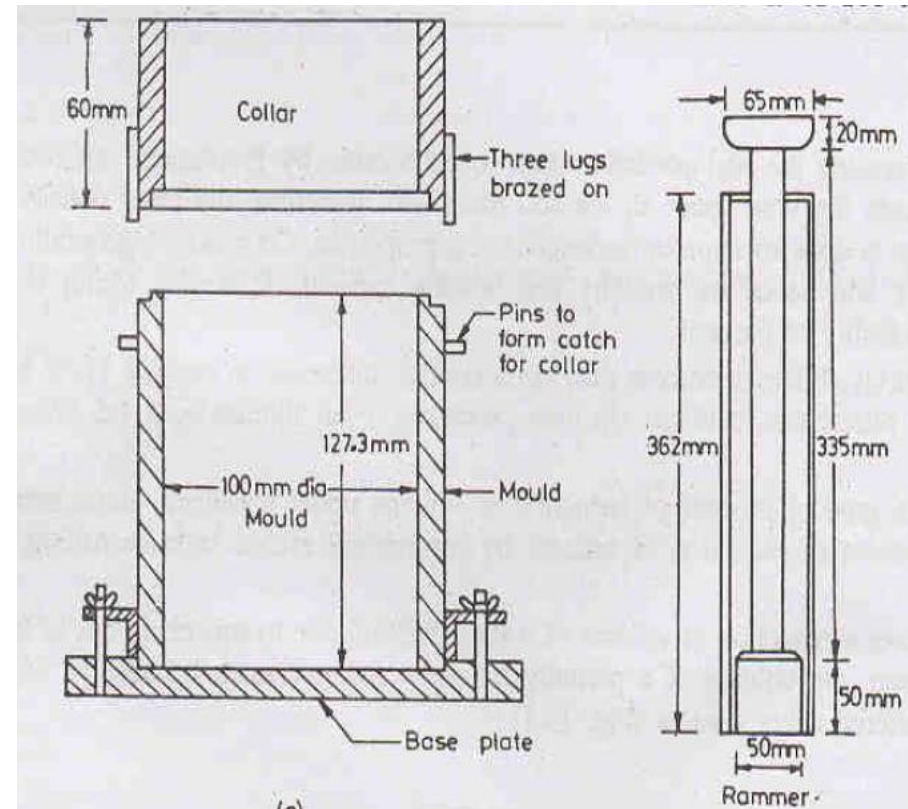
- Compaction is pressing of soil during which expulsion of air takes places.
- The difference between the phenomenon of compaction and consolidation is that in consolidation expulsion of water takes place while in compaction expulsion of air takes place.

Laboratory test for compaction

- Standard Proctor test
- Modified Proctor test

Standard Proctor test

- In this test soil is compacted in 3 layers, each layer is given 25 blows with 2.6 kg rammer with free fall of 310 mm.
- In this test water content is changed and corresponding density is measured.
- Compaction effort is kept fixed.



$$\rho_d = \frac{G \rho_w}{1 + e}$$

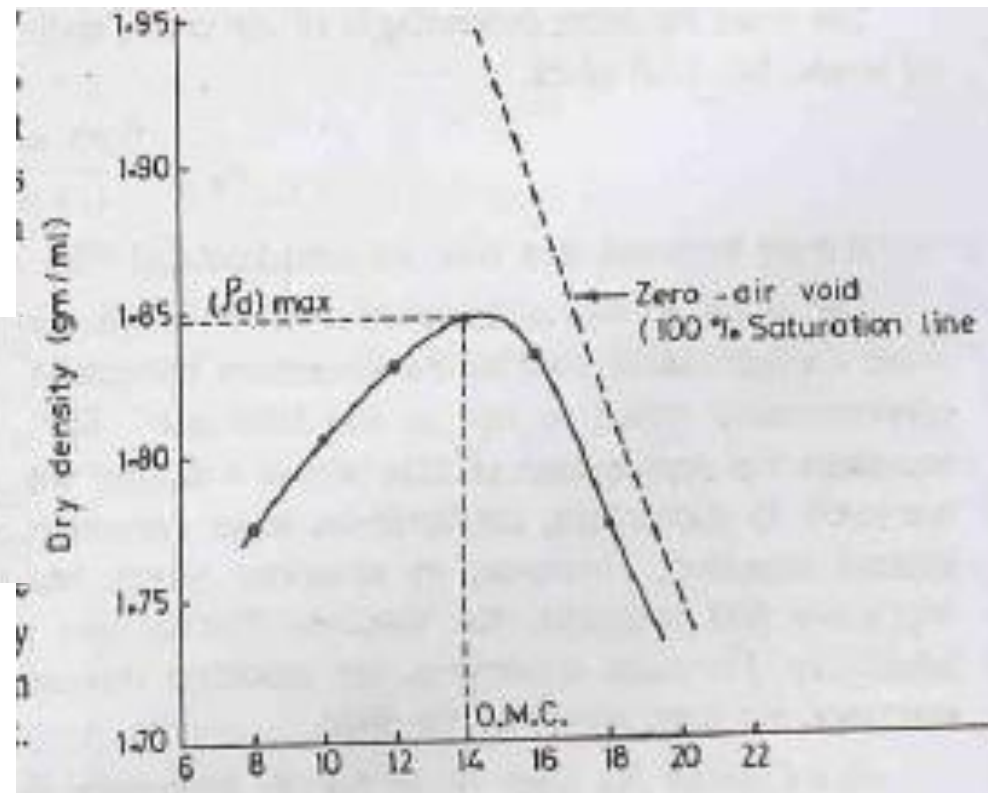
As $e = wG/S$,

$$\rho_d = \frac{G \rho_w}{1 + (wG/S)}$$

For theoretical maximum density, $n_a = 0$. Therefore,

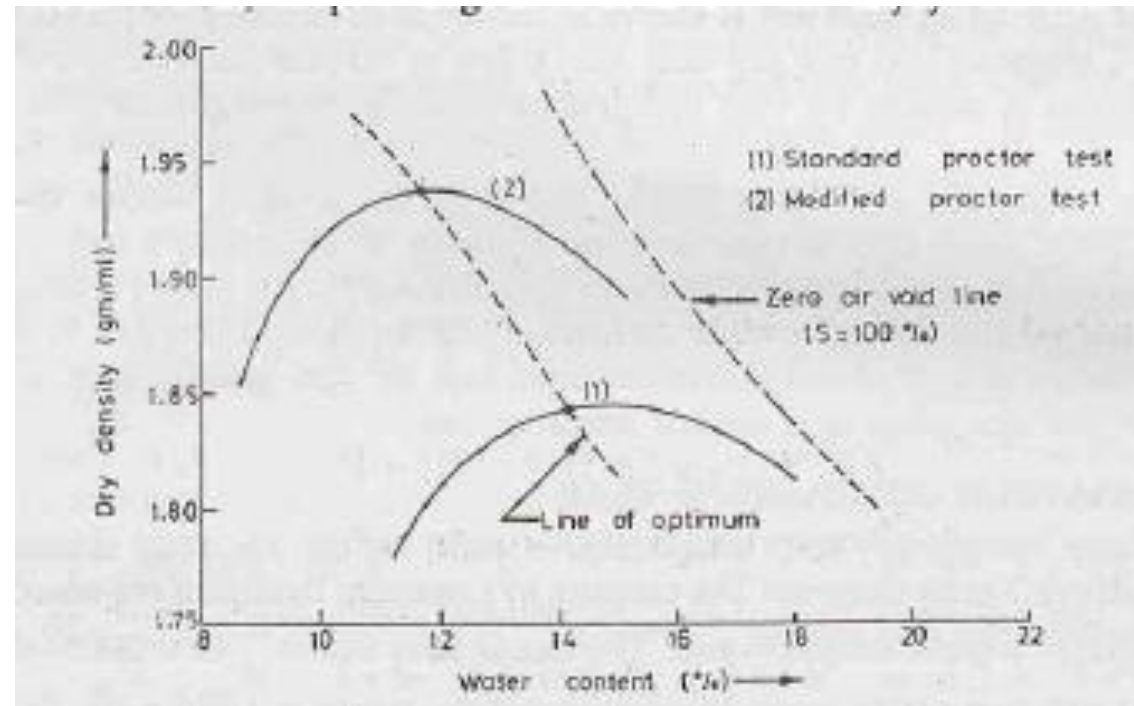
$$(\rho_d)_{theor max} = \frac{G \rho_w}{1 + wG}$$

Thus, the zero-air void line and 100% saturation line are identical.



Modified Proctor test

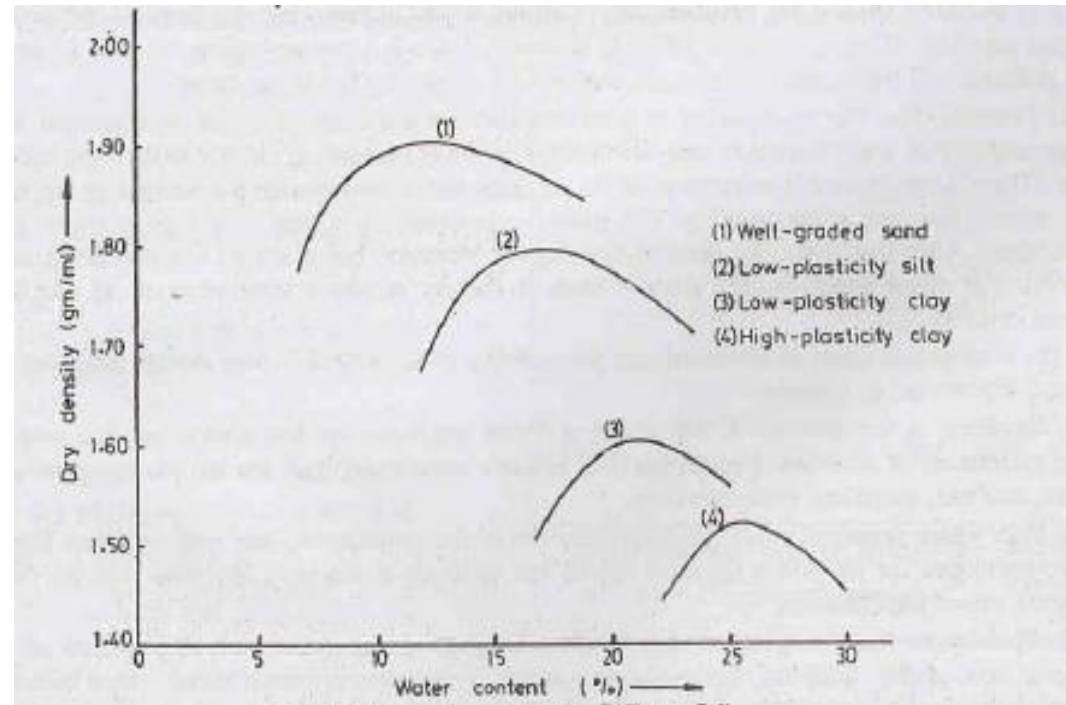
- In modified proctor test compaction effort get increased about 4.56 times
- Weight of rammer 4.98 kg, free drop 450 mm, 5 no. of layers, 25 blows



Factor affecting compaction

- Water content
- Amount of compaction
- Type of soil
- Method of compaction
- Admixture

- For same effort of compactive method, the compaction changes the compacting action.
- Kneading action, dynamic action and static action are different form of the compaction



Field compaction apparatus

- Rammers or Tampers: These are used for the structure having confined area or for the soil adjacent to the existing structures.



Rollers



Other method

- Vibroflotation
- Compaction by ponding
- Compaction by explosive
- Compaction piles etc.

Suitability of various methods

- Cohesionless soils only : smooth wheel rollers, vibratory rollers, vibroflotation, blasting compaction piles can be used
- Cohesive soil only: Sheep-foot rollers are suitable, precompression can also be used.
- Both cohesive and cohesionless soil: Tampers, Pneumatic tyred rollers, Ponding method.

Relative comaction

$$\text{Relative compaction} = \frac{\rho_d \text{ in the field}}{(\rho_d)_{\max} \text{ in the laboratory}} \times 100$$

Impact of compaction

Table 6.12 Comparison of Dry of Optimum and Wet of Optimum Compaction

<i>Property</i>	<i>Dry of Optimum</i>	<i>Wet of optimum</i>
Structure after compaction	Flocculated (random)	Dispersed (oriented)
Water deficiency	More	Less
Permeability	More, isotropic	Less, anisotropic ($k_H > k_v$)
Compressibility at low stress at high stress	Low High	Higher Lower
Swellability	High	Low
Shrinkage	Low	High
Stress-strain behaviour	Brittle; high peak higher elastic modulus	Ductile; no peak, lower elastic modulus
Construction pore water pressures	Low	High
Strength (undrained) as moulded, after saturation	High somewhat higher if swelling prevented	Much lower Low

References

- Basic and applied soil mechanics by Gopal Ranjan
- Soil Mechanics and Foundation Engineering by K R Arora

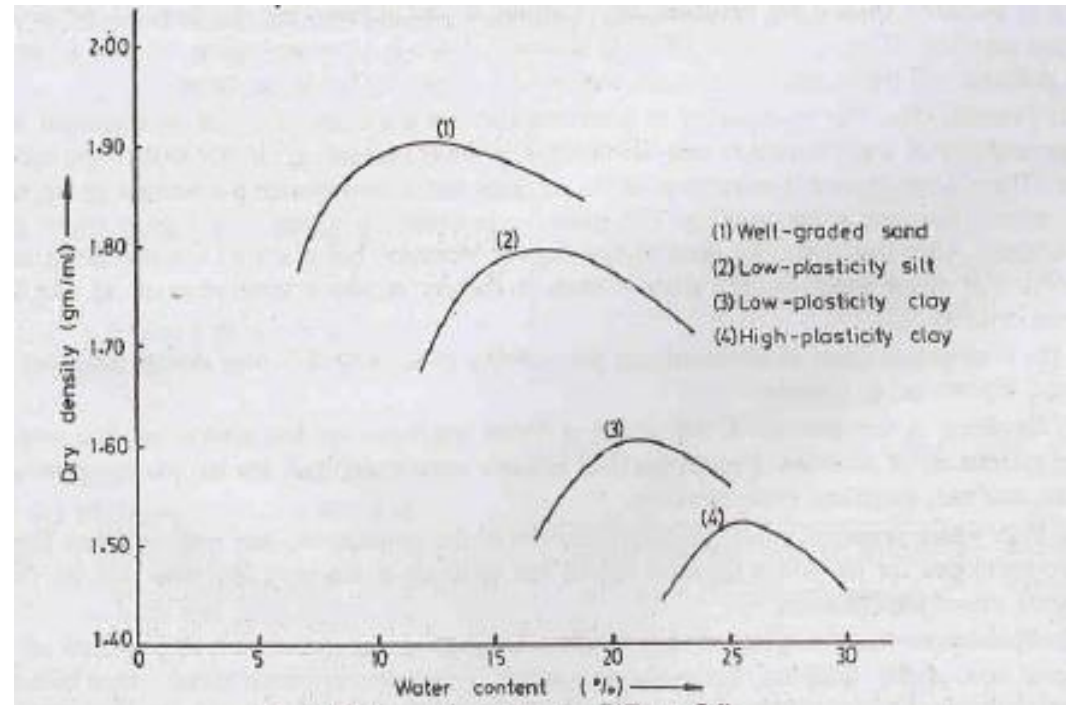
Unit-5

Compaction test

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- Type of soil
- Method of compaction
- Admixture

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