Introduction to Geotechnical Engineering

Geotechnical Engineering

- Geotechnical Engineering deals with the application of Civil Engineering Technology to some aspects of earth.

- Geotechnical Commission of Swedish State Railways (1914-1922) was the first to use the word Geotechnical in the sense that we know it today: the combination of Civil Engineering technology and Geology.

- Geotechnical Engineering deals with;
  - Design of Foundation
  - Stability of Slopes and Cuts
  - Design of Earth Structures
  - Design of Roads and Airfield etc

Soil Mechanics

- Soil Mechanics is defined as the branch of engineering science which enables an engineer to know theoretically or experimentally the behavior of soil under the action of
  1. Loads (static or dynamic),
  2. Gravitational forces,
  3. Water and
  4. Temperature.

- According to Karl Terzaghi, Soil Mechanics is the applications of **Laws of Hydraulics** and **Mechanics** to engineering problem dealing with sediments and other unconsolidated accumulations of solid particles produced by Mechanical and Chemical Disintegration of rocks.

Historical Perspective of Soil Mechanics and Geotechnical Engineering

The record of the first use of soil as a construction material by man kind is lost in antiquity.

In true engineering sense, there is no 'Geotechnical Engineering' prior to the 18th Century.

One of the most famous example of problems related to soil bearing capacity and foundations in the construction of structures prior to 18^{th} century is the Leaning Tower of Pisa in Italy. The construction of the Tower began in 1173 A.D. and last over 200 years.

Soil Mechanics (Explanation)

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

- Civil Engineer must study the properties of Soil, such as its origin, grain size distribution, ability to drain water, compressibility, shear strength, and load bearing capacity.

- Geotechnical Engineering is the sub discipline of Civil Engineering that involves applications of the principles of Soil Mechanics and Rock Mechanics to design of foundations, retaining structures and earth structures.

The Leaning Tower of Pisa, Italy.
Morning, 1 March 2004. SW view

Height: 54 m;
Max tilt: 5 m out of plumb
Tilt direction: E, N, W, and S.
Weight: 15,700 tons;
Base: ϕ = 20 m;
Reason: a weak clay layer at 11 m depth
Solution: excavation of soil from north side for about 70 tons.

Born: October 2, 1883 in Prague

Died: October 25, 1963 in Winchester, Massachusetts

He was married to Ruth D. Terzaghi, a geologist.

He won the Norman Medal of ASCE four times (1930, 1943, 1946, and 1955).

He was given nine honorary doctorate degrees from universities in eight different countries.

He started modern soil mechanics with his theories of consolidation, lateral earth pressures, bearing capacity, and stability.

Karl Terzzaghi (1883 - 1963)

- Karl Terzaghi has often been called *the father of Soil Mechanics.*
- Academically, he earned an under graduate degree in Mechanical Engineering.
- In 1925, he accepted lectureship at MIT.
- In 1939, he accepted professorship at Harvard University till his death.
- His recognition and formulation of the effective stress principle and its influence on settlement analysis, strength, permeability and erosion of soils was his most prominent contribution. But Terzaghi also pioneered a great range of methods and procedures for investigation, analysis, testing, instrumentation, and practice that defined much of the field we currently know as geotechnical engineering.

Karl Terzaghi (1883 - 1963)

Unfortunately, soils are made by nature and not by man, and the product of nature are always complex... As soon as we pass from steel and concrete to earth, the omnipotence of theory ceases to exist. Natural soil is never uniform. Its properties changes from point to point while our knowledge of its properties are limited to those few spots of which the samples have been collected. In soil mechanics, the accuracy of computed results never exceeds that of crude estimate, and the principal function of theory consists in teaching us in what and how to observe in the field. (Karl Terzaghi)

Why we study Soil Mechanics?

- Virtually every structure is supported by soil or rock. Those that aren't either fly, float or fall over.[1]

- Various reasons to study the properties of Soil:[2]
  1. Foundation to support Structures and Embankments
  2. Construction Material
  3. Slopes and Landslides
  4. Earth Retaining Structures
  5. Special Problems

Ref. 1. Pg # 1. Geotechnical Engg. (Principles and Practices) by Donald P. Coduto. 2nd Ed.
2. Pg # 3 to 16 , Chap # 1. Soil Mechanics by T. William Lambe.

Why we study Soil Mechanics?

- Various reasons to study the properties of Soil:

1. Foundation to support Structures and Embankments
   - Effects of static loading on soil mass
     - Shear failure of the foundation soil
     - Settlement of structures
   - Stability criteria (Solution)
     - There should be no shear failure of the foundation soil.
     - The settlement should remain within permissible limits.
   - Firm Soil -> Spread Footing (Spread Foundation)
   - Soft Soil -> Pile Foundation (Vertical members transferring load of structure to ground i.e. rock)

Why we study Soil Mechanics?

- Various reasons to study the properties of Soil:
  1. Foundation to support Structures and Embankments
     - Effects of dynamic loading on soil mass
       - For Design and construction of roads following must be considered:
         - Compaction Characteristics
         - Moisture Variation

Soil subjected to dynamic load.

Why we study Soil Mechanics?

- Various reasons to study the properties of Soil:
  2. Construction Material
     - Subgrade of highway pavement
     - Land reclamation (Dubai Palm City)
     - Earthen dam

Why we study Soil Mechanics?

- Various reasons to study the properties of Soil:
  3. Slopes and Landslides
     - Major cause is the moisture variation resulting in;
       - Reduction of shear strength
       - Increase of moisture
       - Increase in unit weight
     - Excavation of trenches for buildings require braced excavation.

Why we study Soil Mechanics?

Landslide of a parking area at the edge of a steep slope, mainly due to increase in moisture content.

Why we study Soil Mechanics?

- Various reasons to study the properties of Soil:
  4. Earth Retaining Structures
     - Earth retaining structure (e.g., Retaining walls)are constructed to retains (holds back) any material (usually earth) and prevents it from sliding or eroding away.

Why we study Soil Mechanics?

- Various reasons to study the properties of Soil:

5. Special Problems

   i. Effects of river water on soil mass

      a) Scouring

      Causes:

      - Increased flow velocity due to obstruction
      - Fineness of riverbed material

      Stability criteria:

      - The foundation of pier must be below the scour depth

   ii. Land Erosion

Why we study Soil Mechanics?

- Various reasons to study the properties of Soil:

5. Special Problems

    iii. Effects of frost action on soil mass

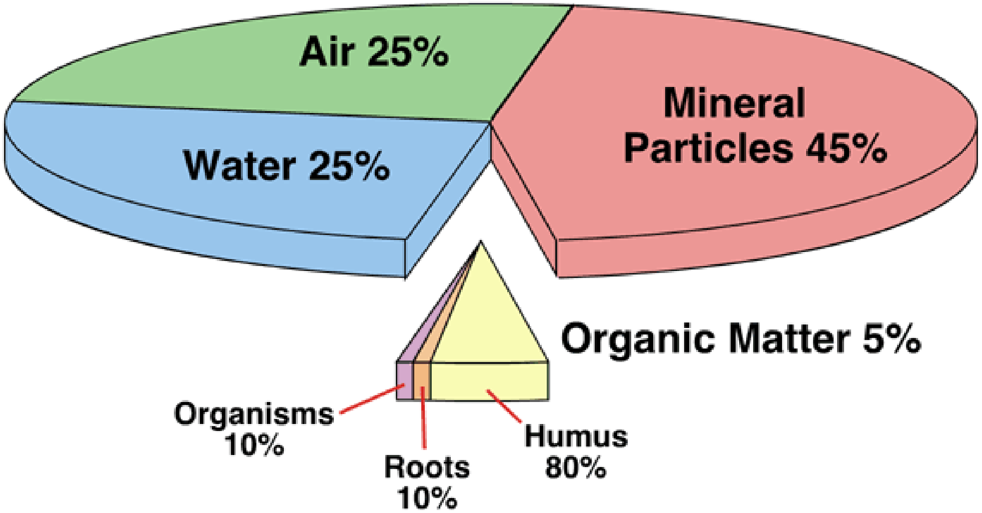
    - Reduction Of Shear Strength
    - Settlement Of Structure In Summer
    - Lifting Up Of Structure In Winter

    Causes:

    - Heaving (due to formation of ice lenses)
    - Increase of moisture due to thawing (MELTING)

Soil Descriptions

- Soil is an unconsolidated agglomerate of minerals with or without organic matter found at or near the surface of the earth crust, with which or upon which civil engineers build their structures



WEATHERING OF ROCKS

- Weathering of Rocks
- Physical Weathering
- Chemical Weathering
- Rock Cycle

Weathering of Rocks

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Weathering of Rocks

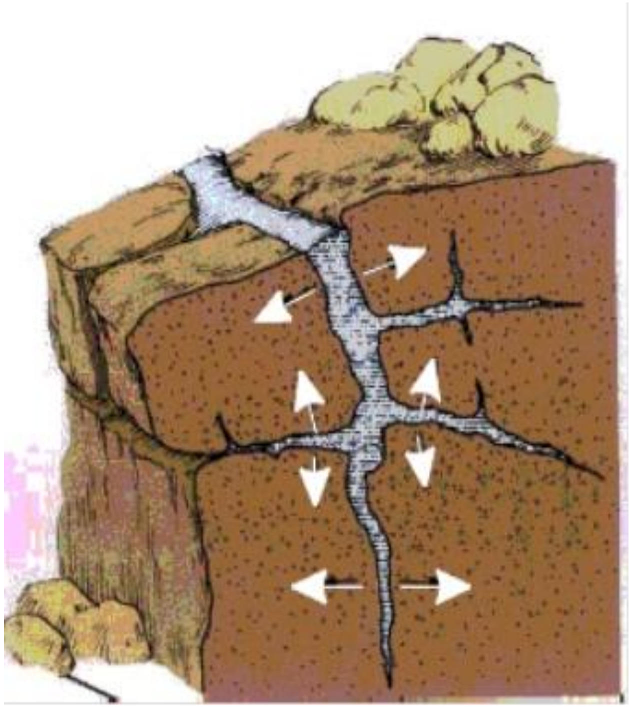
- Weathering is the process of breaking down rocks by physical and chemical process into smaller particles.[1]

- There are two main types of weathering processes:
  - Physical (or mechanical) Weathering
  - Chemical Weathering

- Biological weathering is caused by activities of living organisms - for example, the growth of roots or the burrowing of animals. Tree roots are probably the most occurring, but can often be by animals!

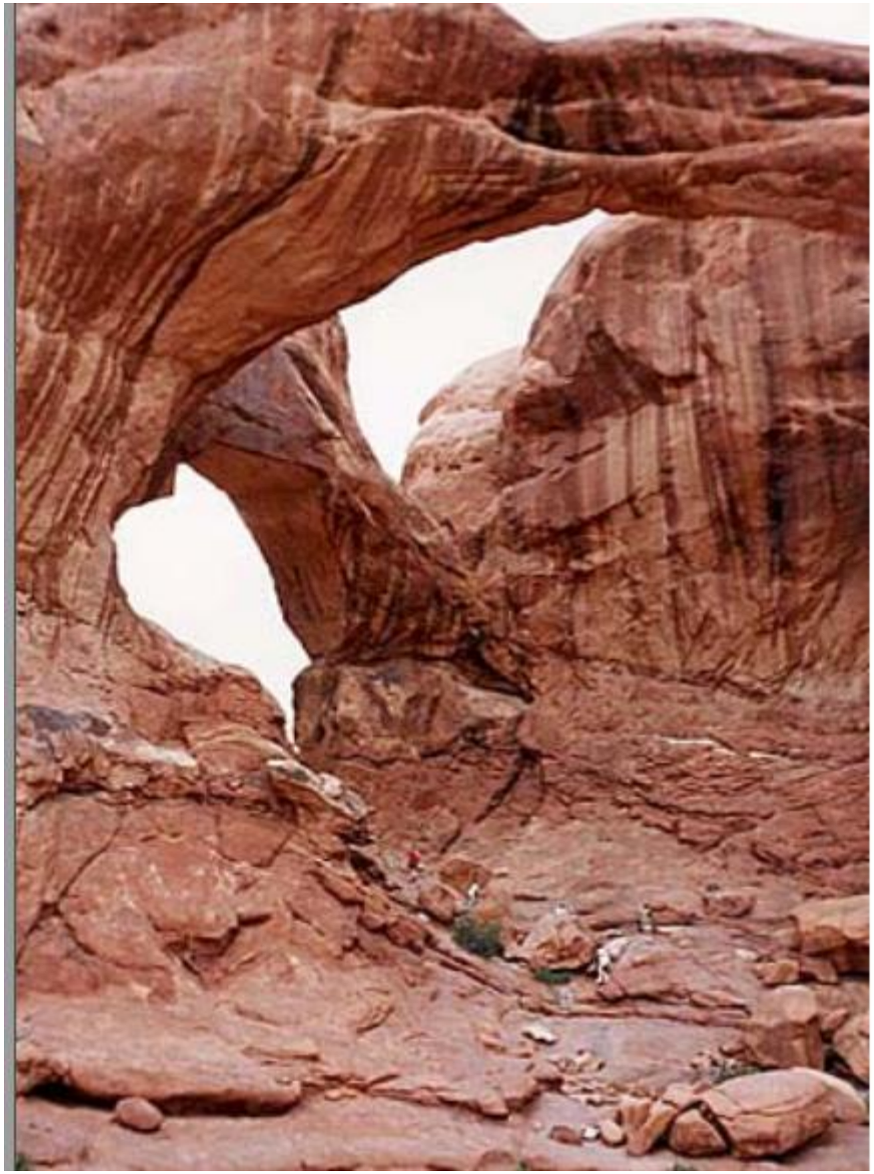
Physical Weathering

- Physical (or mechanical) Weathering is the disintegration of rocks into smaller particles through physical processes, including:
  - The erosive action of water, ice and wind.
  - Opening of cracks as a result of unloading due to erosion of overlying soil and rock.
  - Loosening through the percolation and subsequent freezing (and expansion) of water.
  - Thermal Expansion and contraction from day to day and season to season.
  - Landslides and rockfalls.
  - Abrasion from the downhill movement of nearby rock and soil.



Mechanical/Physical weathering

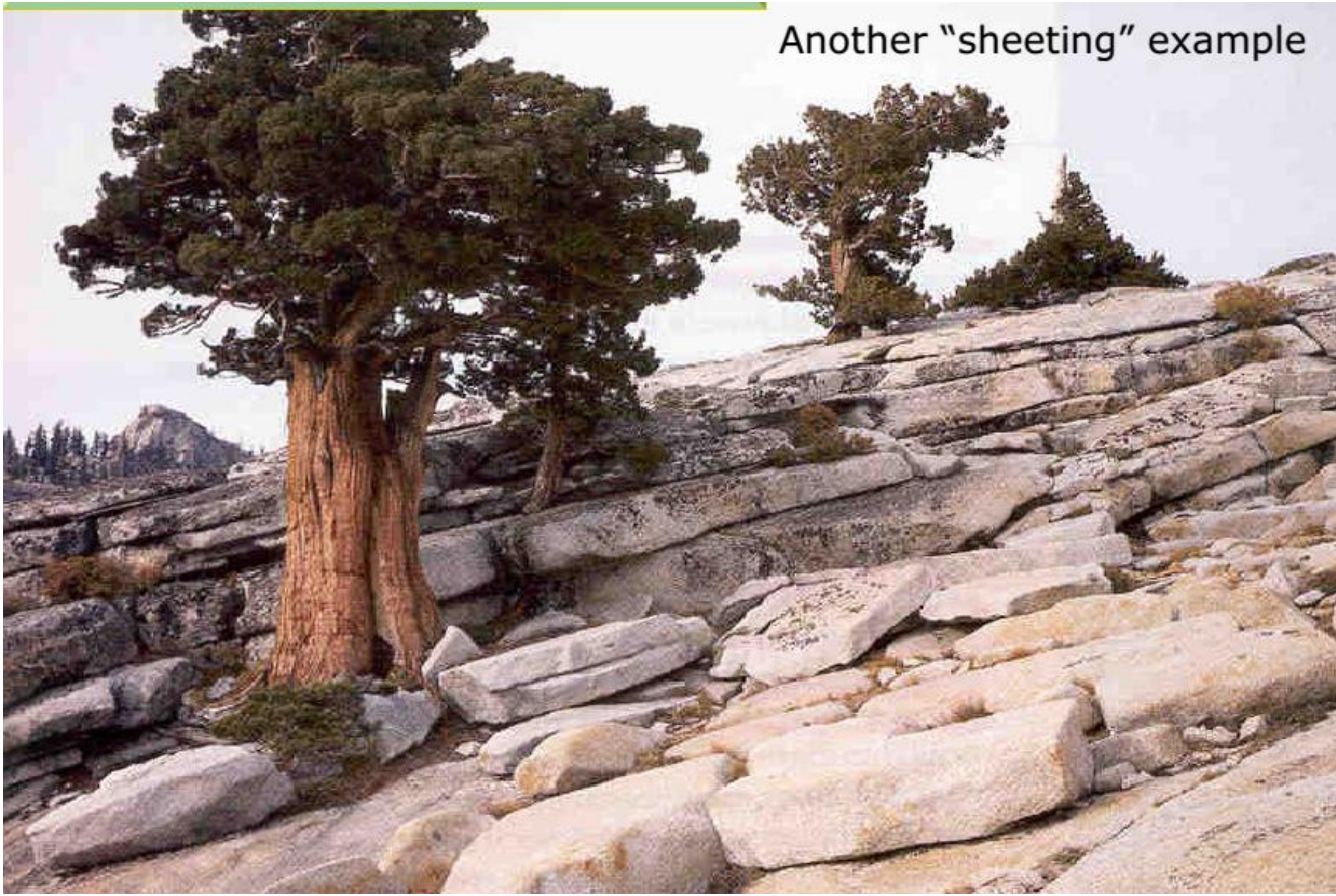
Differential weathering



Twin Tower, God's Garden, Colorado

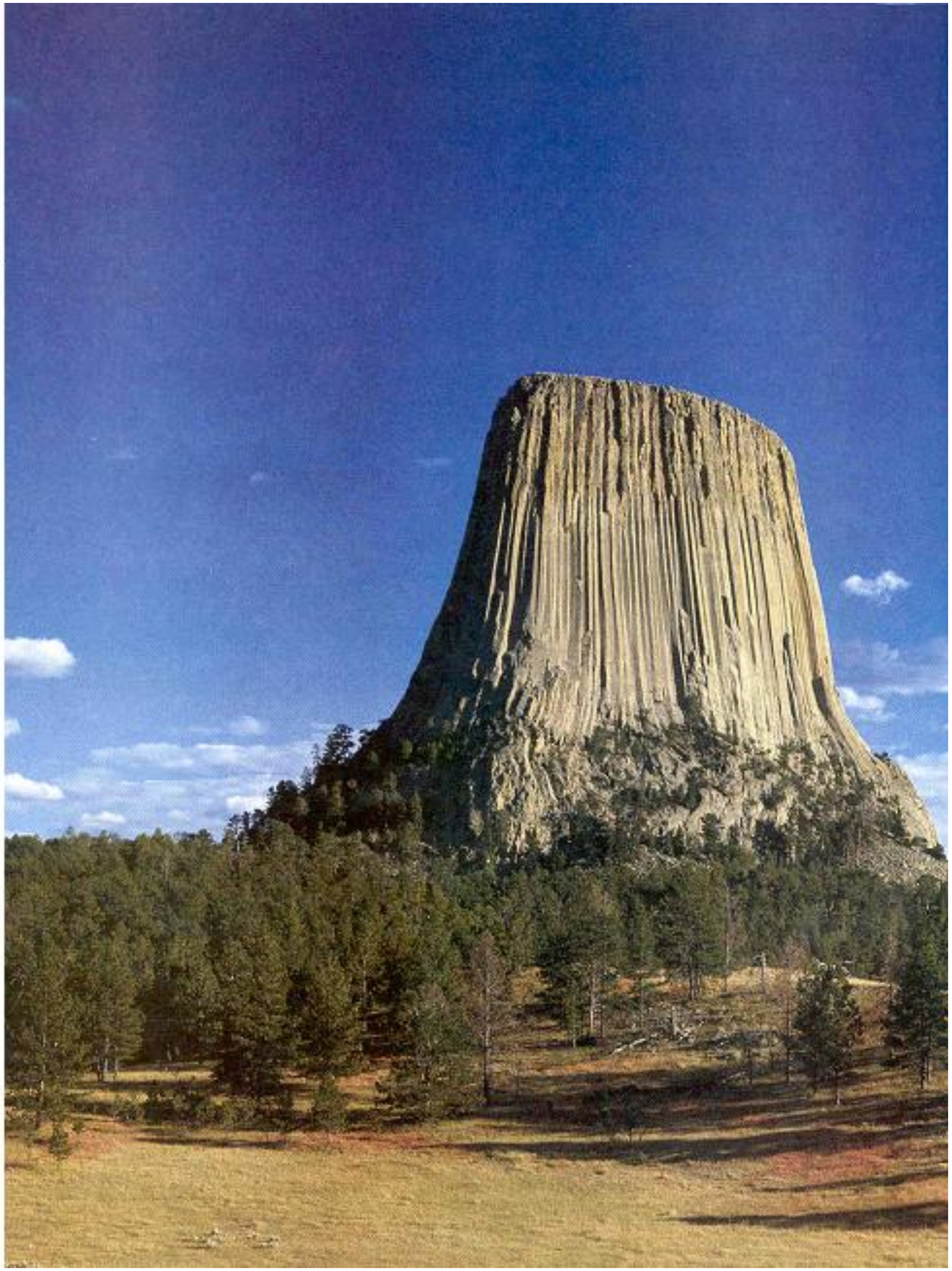
joints are parallel cracks in which rocks on either side are not offset; **Sheeting** rock layers peel like layers of an onion

Another “sheeting” example



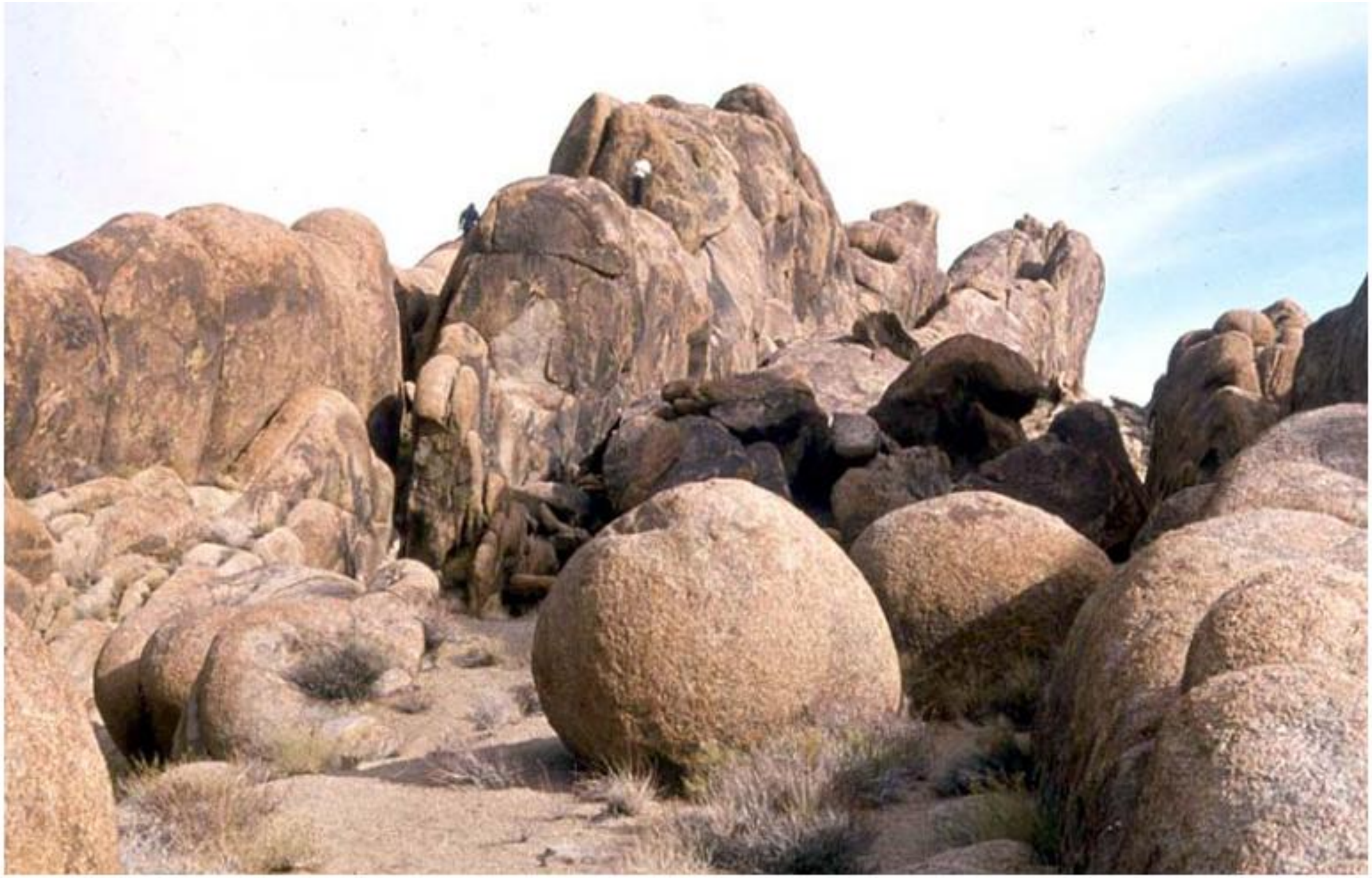
Vertical columns from magma cooling & shrinking

"columnar jointing"



Columnar jointing in basalt

Glacially polished basalt columns
(end view)



Spheroidal Weathering. Granite illustrates weathering forms quite well. Chemical weathering attacks to granite along joints and makes rounded boulders (Alabama Hills near Lone Pine) .

Chemical Weathering

- Chemical Weathering is the disintegration of rock through chemical reactions between the minerals in the rocks, water, and oxygen in the atmosphere.[1]

- An example of the chemical weathering orthoclase to form clay minerals, silica and soluble potassium carbonate follows:[1]

$$H_2O + CO_2 \rightarrow H_2CO_3 \rightarrow H^+ + (HCO_3)^-$$

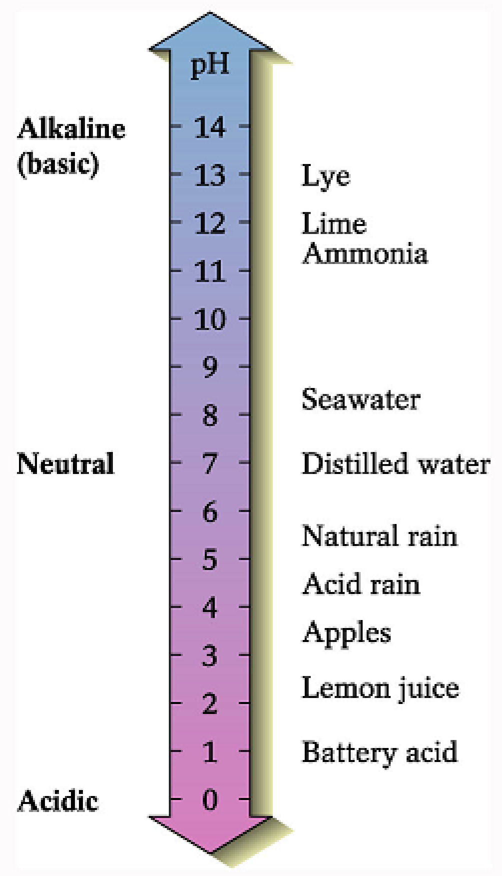
$$\underset{\text{Orthoclase}}{2K(AlSi_3O_8)} + 2H^+ + H_2O \rightarrow 2K^+ + \underset{\text{Silica}}{4SiO_2} + \underset{\substack{\text{Kaolinite} \\ \text{(Clay mineral)}}}{Al_2Si_5(OH)_4}$$

Chemical Weathering

- Chemical weathering rate depends on
  1. Temperature
  2. Amount of surface area
  3. Availability of water or natural acid

- **Thus, rocks in tropical environment experience most severe chemical weathering.**

Acidity of Natural Waters



Water is a good solvent.

Acidic water is better!

pH of most natural waters ranges from 4 to 9

pH > 9 or < 4 occurs in extreme environments

Why is rainwater naturally acidic?

Rainwater contains dissolved CO_2 from atmosphere.

Dissolved CO_2 reacts with water to form carbonic acid (H_2CO_3)

$$CO_2 + H_2O \Leftrightarrow H_2CO_3$$

Carbonic acid dissociates to produce hydrogen ion (H^+) and bicarbonate

$$H_2CO_3 \Leftrightarrow H^+ + HCO_3^-$$

Marble tombstones and carvings are particularly susceptible to chemical weathering by dissolution. Note that the urn and tops of ledges are heavily weathered, but the inscriptions are somewhat sheltered and remain legible. Photo taken in one New Orleans graveyard.

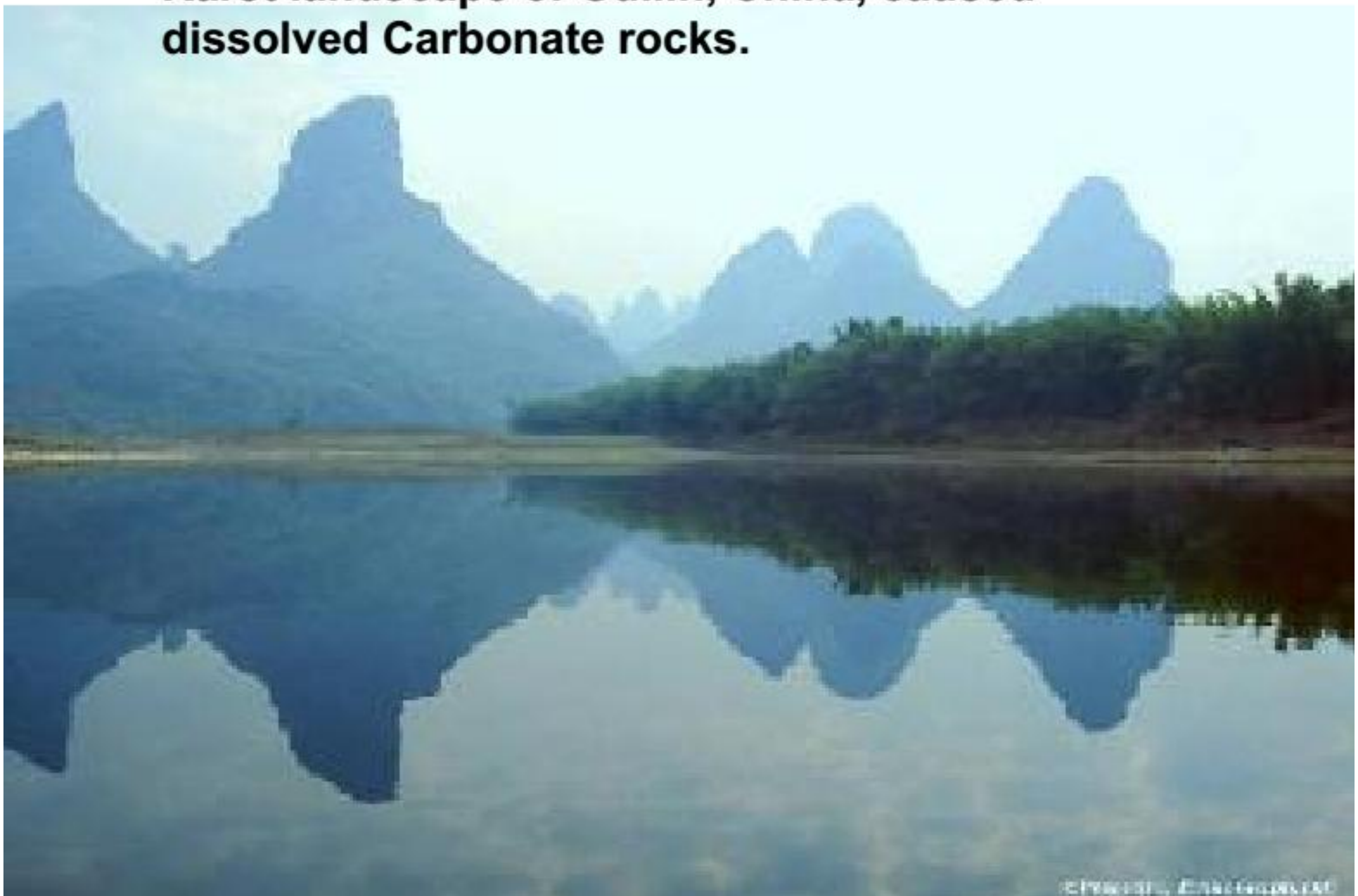
A 16th-century monastery in Mexico shows the ravages of weathering mostly from wind and wind-driven rain. The rock is volcanic tuff.

Rock Cycle

All rock at or near Earth's surface is bei
ng modified by the processes of meta
morphism, melting,
crystallization, lithification and
weathering. These processes move
rock material through the states of met
amorphic rock, igneous
rock, sedimentary rock, melts and
sediment. The natural and
continuous cycling of rock
materials through these states is
known as the Rock Cycle.[1]

Ref. 1. General Dictionary of Geology (Environmental Geographic Student Association (EGSA))

Karst landscape of Guilin, China, caused dissolved Carbonate rocks.



SOIL AND ITS TYPES

- What is Soil?
- Formation of Soil
- Types of Soil
  - Geological Consideration
  - Engineering Consideration

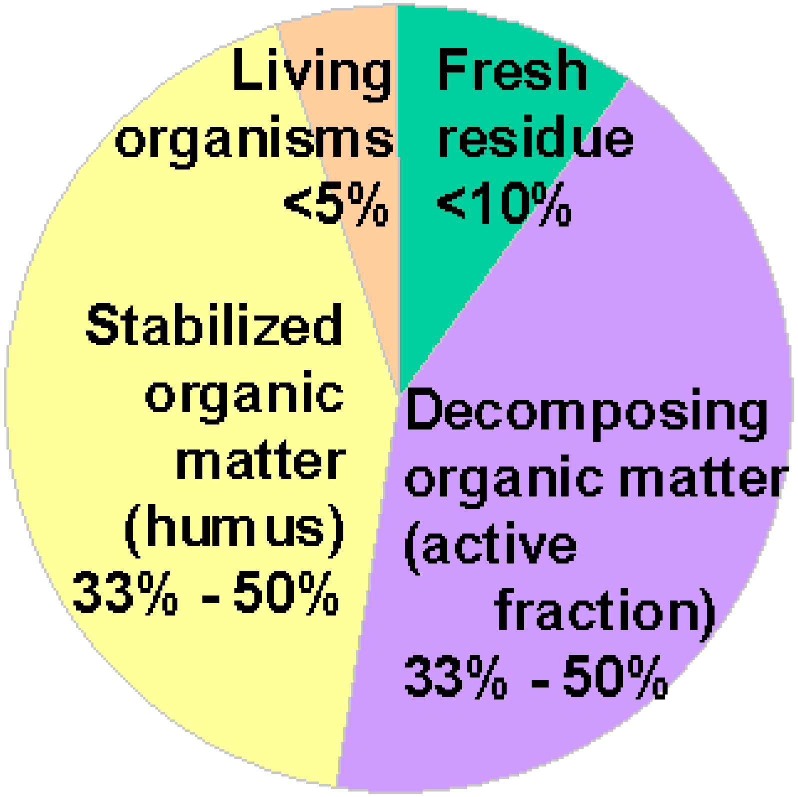
What is Soil? (Definitions)

- Soil is defined as the weathered and fragmented outer layer (crust) of the earth's terrestrial surface.[1]

- The term Soil, according to engineering point of view, is defined as the material by means of which and upon which engineers builds their structures.[2]

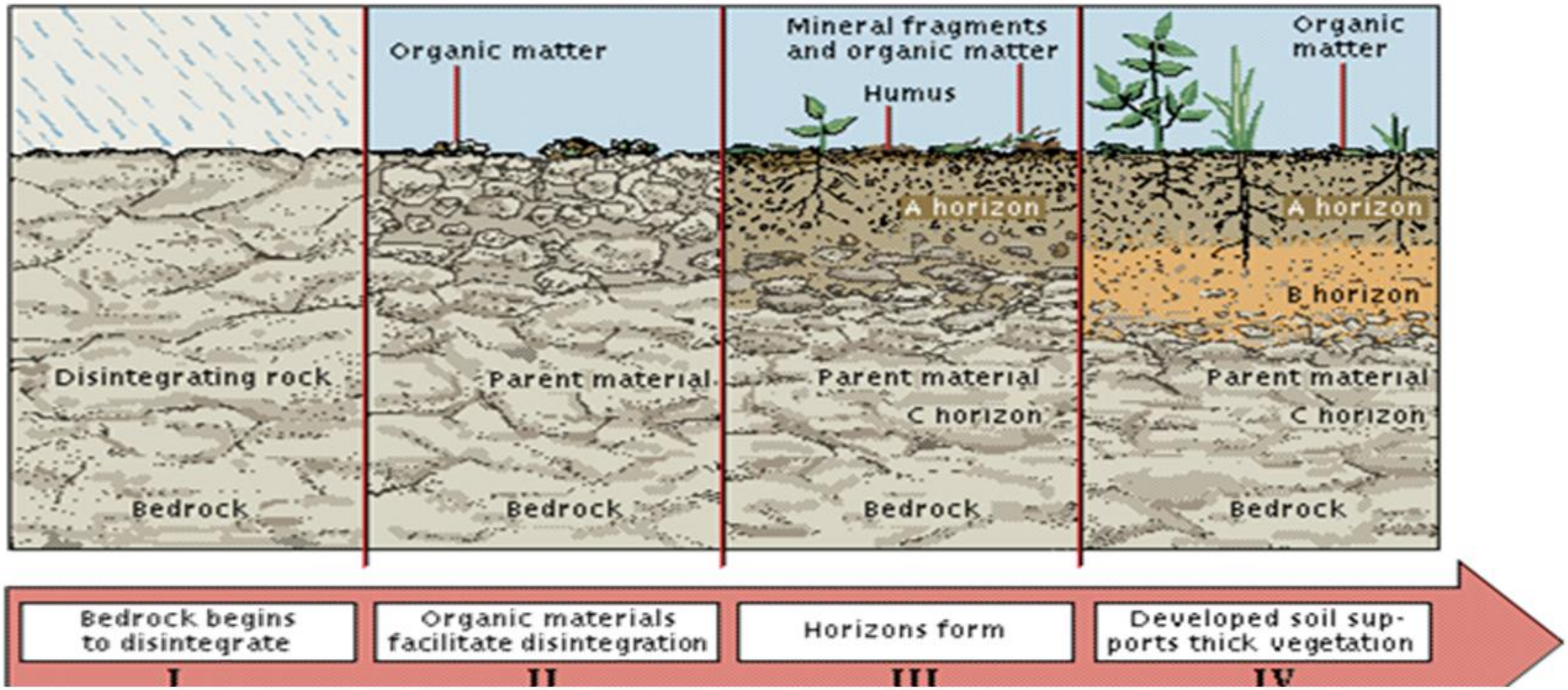
- For engineering purpose Soil is defined as the uncemented aggregate of mineral grains and decayed organic material (solid particle) with liquid and gas in the empty spaces between the solid particles.[3]

Ref. 1. Pg # 3. Introduction to Env. Soil Physics by Daniel Hillel (Elsevier Academic Press)
2. Pg # 2. Soil Mechanics for Transprtation Engg. by Prof. Shaukat Ali Khan
3. Pg # 1. Principles of Geotechnical Engg. By Braja M. Das. 5th Ed. (CENGAGE Learning)

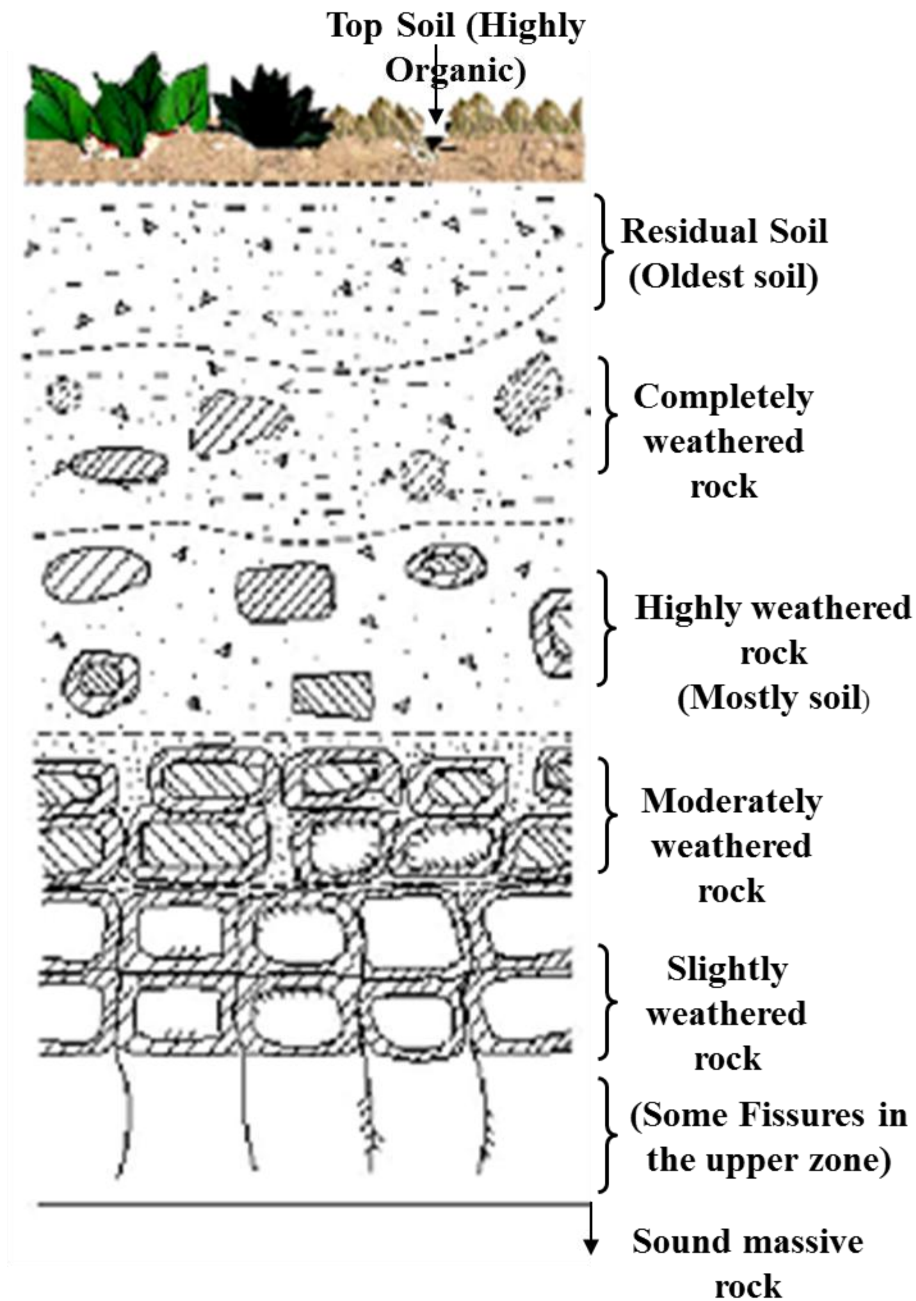

Living organisms <5%
Fresh residue <10%
Stabilized organic matter (humus) 33% - 50%
Decomposing organic matter (active fraction) 33% - 50%

Formation of Soil

- Soil is generally formed by disintegration and decomposition (weathering) of rocks through the action of physical (or mechanical) and chemical agents which break them into smaller and smaller particles.



Different stages of weathering of rocks and formation of soil.



Types of Soil

- Soil types, based on geological and engineering view points, are separately discussed below:
  1. Geological consideration:

     Geologist classify soil into two major categories: residual soils and transported soil

     i. **Residual Soils:**

        When the rock weathering is faster than the transport process induced by water, wind and gravity, much of the soil remains in place. It is known as residual soil.

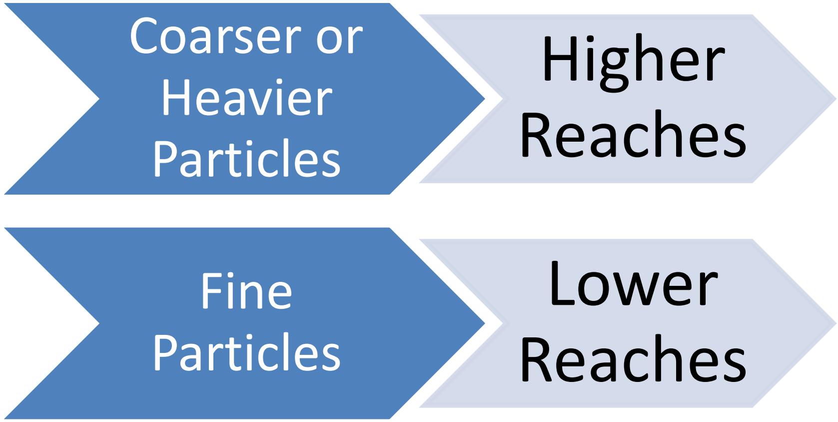
Types of Soil

1. Geological consideration:

   ii. **Transported Soil:**

      i. **Glacial Soil:** This type of soil is developed, transported and deposited by the actions of glaciers. These deposits consists of rocks fragments, boulders, gravels, sand, silt and clay in various proportions (i.e., a heterogeneous mixture of all sizes of particles).

      ii. **Alluvial Soil:** This type of soil (also known as fluvial soil or alluvium) is transported and deposited to their present position by streams and rivers.



Types of Soil

1. Geological consideration:

   ii. **Transported Soil:**

      iii. **Aeolian Soil:** The soil transported by geological agent ‘wind’ and subsequently deposited is known as wind blown soil or Aeolian Soil.

      iv. **Colluvial Soil**: A colluvial soil is one transported downslope by gravity. There are two types of downslope movement – slow (creep – mm/yr) and rapid (e.g., landslide)

      v. **Lacustrine and Marine Soil:**

         i. Lacustrine Soil is deposited beneath the lakes.

         ii. Marine Soil is also deposited underwater i.e., in the Ocean.

Types of Soil

2. Engineering consideration:

Nomenclature (Soil Type)	Range of Sizes	
	ASTM	AASHTO
Gravel	75 mm to 4.75 mm (3in Sieve to No. 4 sieve)	Larger than 2 mm
Coarse Sand	4.75 mm to 2 mm (No. 4 to No. 10 sieve)	2mm to 0.425 mm
Medium Sand	2 mm to 0.425 mm (No. 10 to No. 40 sieve)	------------------------
Find Sand	0.425 mm to 0.075 mm (No. 40 to No. 200 sieve)	0.425 mm to 0.075 mm
Silt	0.075 mm to 0.005 mm (No. 200 to .005 mm)	0.075 mm to .002 mm
Clay	Smaller than 0.005 mm	Smaller than 0.002 mm
Colloids	Smaller than 0.001 mm	Smaller than 0.001 mm

Types of Soil

2. Engineering consideration (MIT):

i. Clay: (< .002mm)

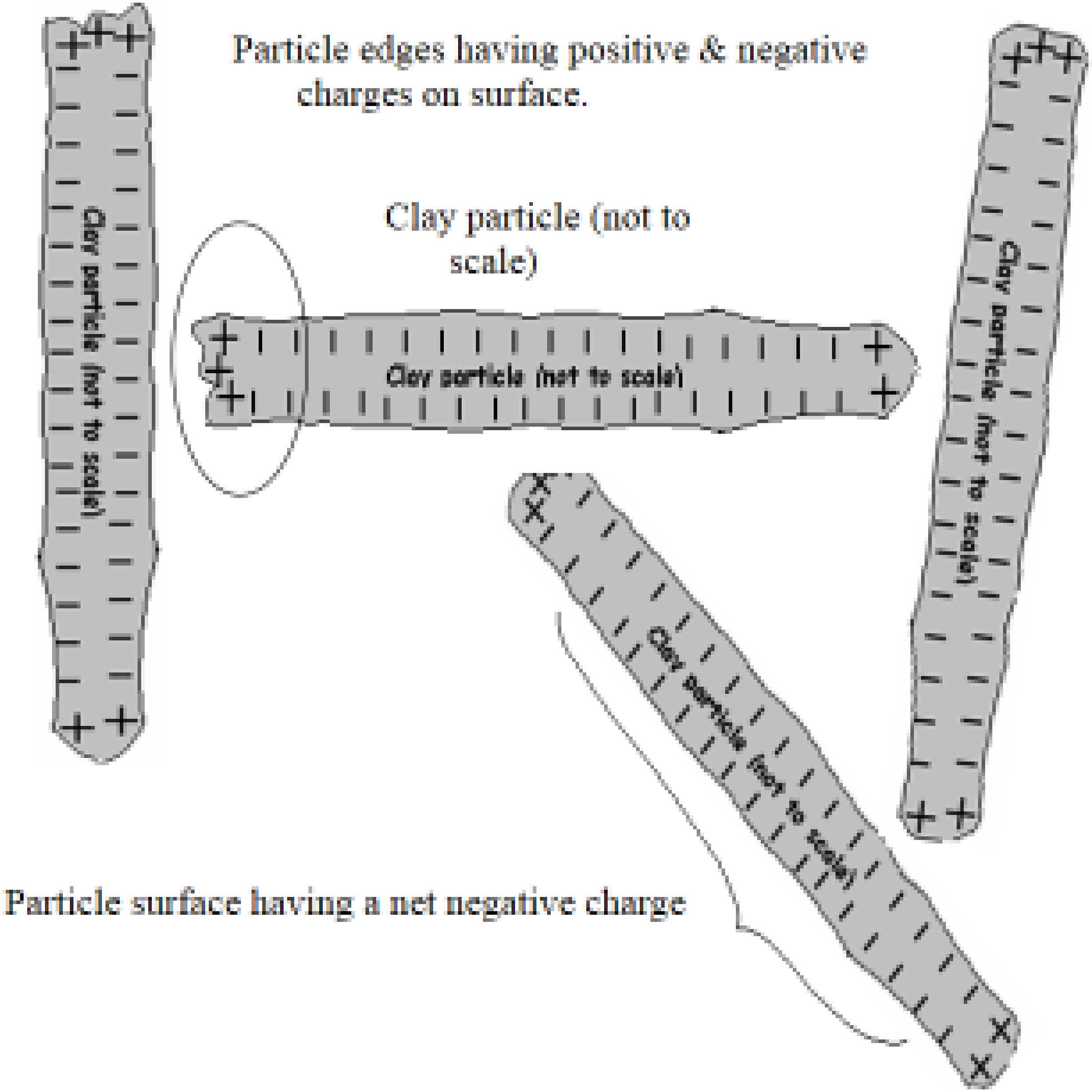
- In moist condition, clay becomes sticky and can be rolled into threads.
- High dry strength, low erosion, low permeability, good workability and compaction under moist condition. Also susceptible to shrinkage and swelling.

ii. Silt: (.002mm < Size < .06mm)

- High capillarity, no plasticity and very low dry strength
- It possesses properties of both clay and sand.

iii. Sand: (.06mm < Size < 2mm)

- Particle shape varies from rounded to angular
- No plasticity, considerable frictional resistance, high permeability and low capillarity
- Abundant quantities of sand are available in deserts and riverbeds



Electrical charge on clay particles and inter-particle bonding

Types of Soil

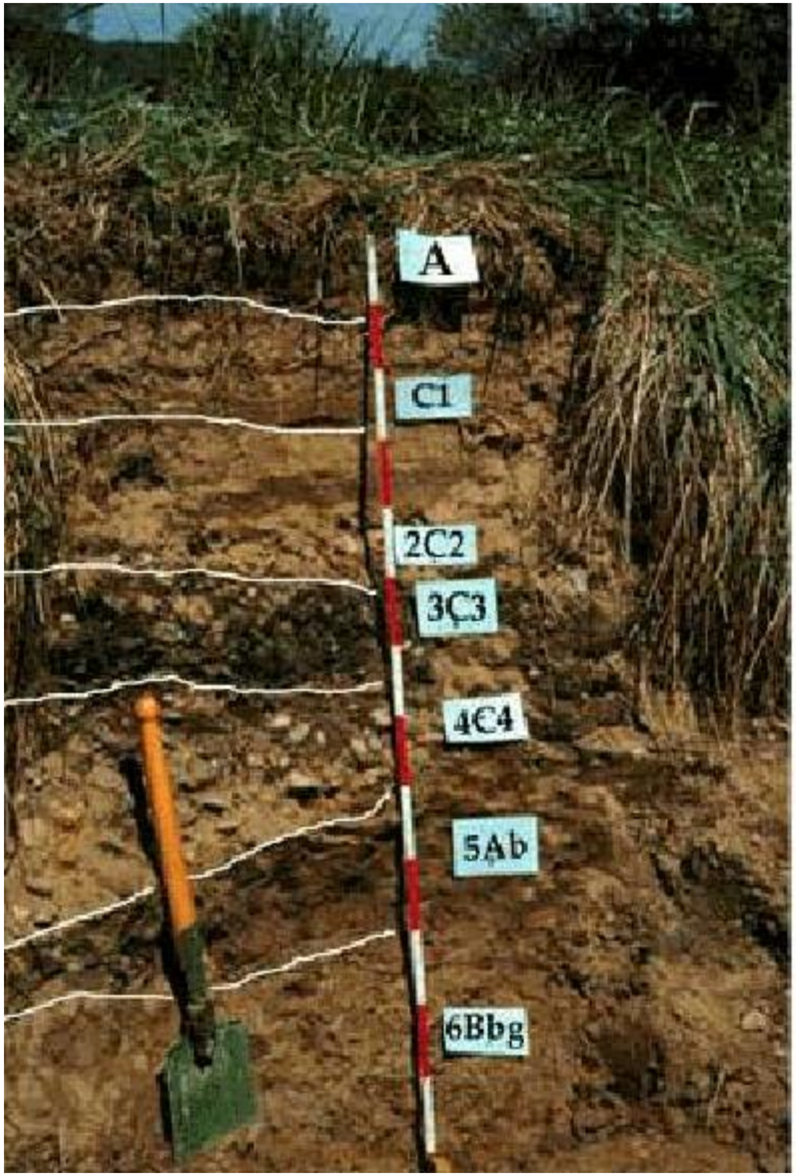
2. Engineering consideration (MIT):

   iv. **Gravels: (2mm < Size < 60mm)**

   - **They form a good foundation material.**
   - **The gravels produced by crushing of rocks are angular in shape while those taken from riverbeds are sub-rounded to rounded.**

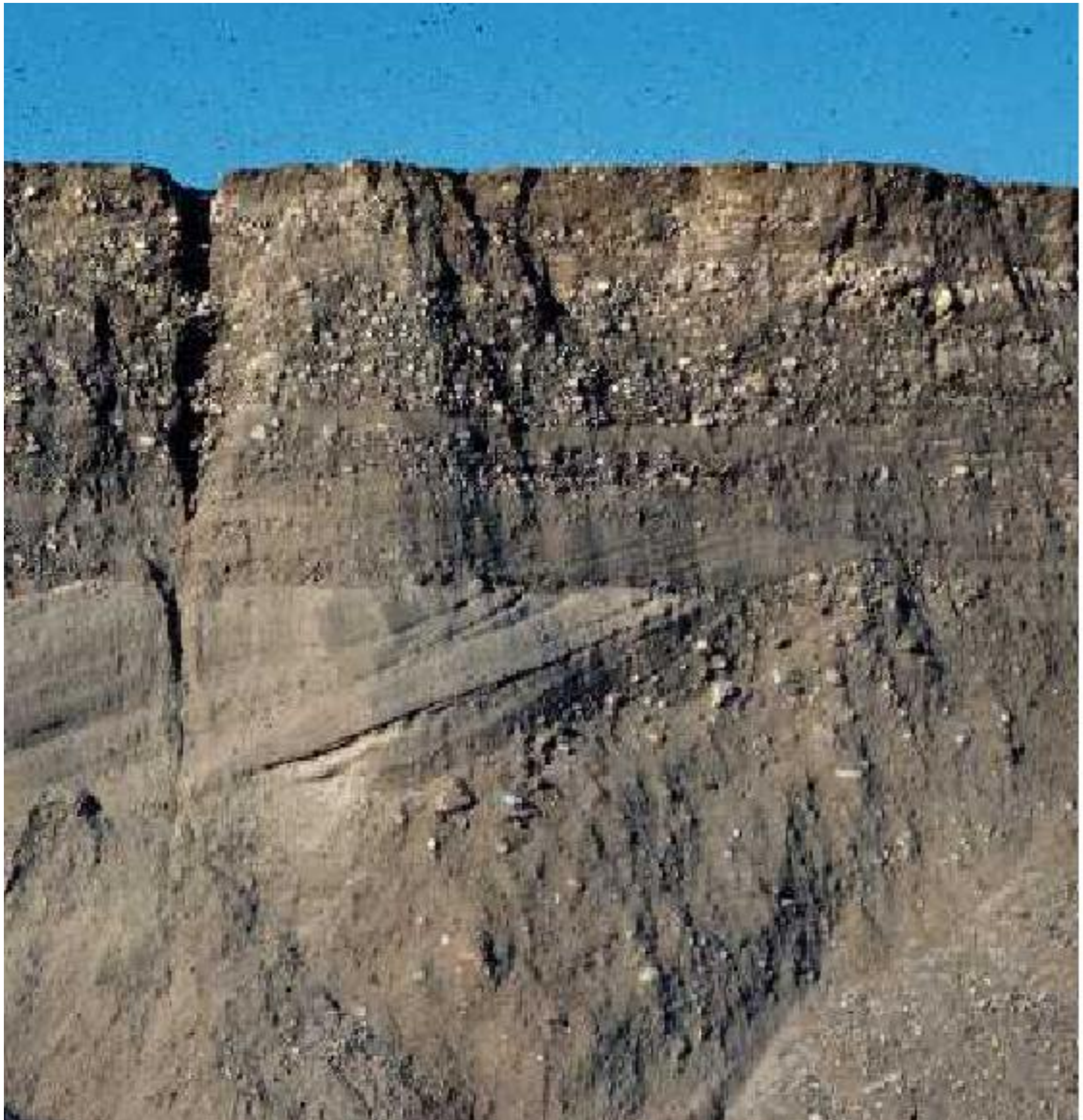
   v. **Cobbles and Boulder:**

   - Particles larger than gravels are commonly known as cobbles and boulders.
   - Cobbles generally range in size 60mm t0 200mm.
   - The materials larger than 200mm is designated as boulders.



Typical Soil Profile

This photo is an outcrop of a glacial till deposit. Glacial till is a heterogeneous mixture of clay to boulder size particles deposited within or beneath glacial ice. The till type on this photo is a dense or basal till with lenses of looser, sandy material (sandy till), the soil type mapped in this area is the Montauk series [the solum (the A and B horizons) has been removed on this photo. photo location: Fearing Hill, Wareham, MA].



A photo of a glacial fluvial deposit (the topsoil and subsoil of a Hinckley soil has been removed) from a gravel pit. This photo shows the horizontal stratified layers of sand and gravel on the top of the photo called the topset beds or delta plain. The inclined or dipping layers of fine and coarse sand (visible on left part of photo) are called the foreset beds or delta slope. The foreset beds were deposited into a glacial lake, the contact of the topset and foreset beds (delta plain/delta slope) marks the former water level of the lake. town of Raynham, MA.

Summary

Soil come from weathering of rocks.

Mechanical weathering is accomplished by physical forces that break rock into smaller and smaller pieces without changing the rock's mineral composition.

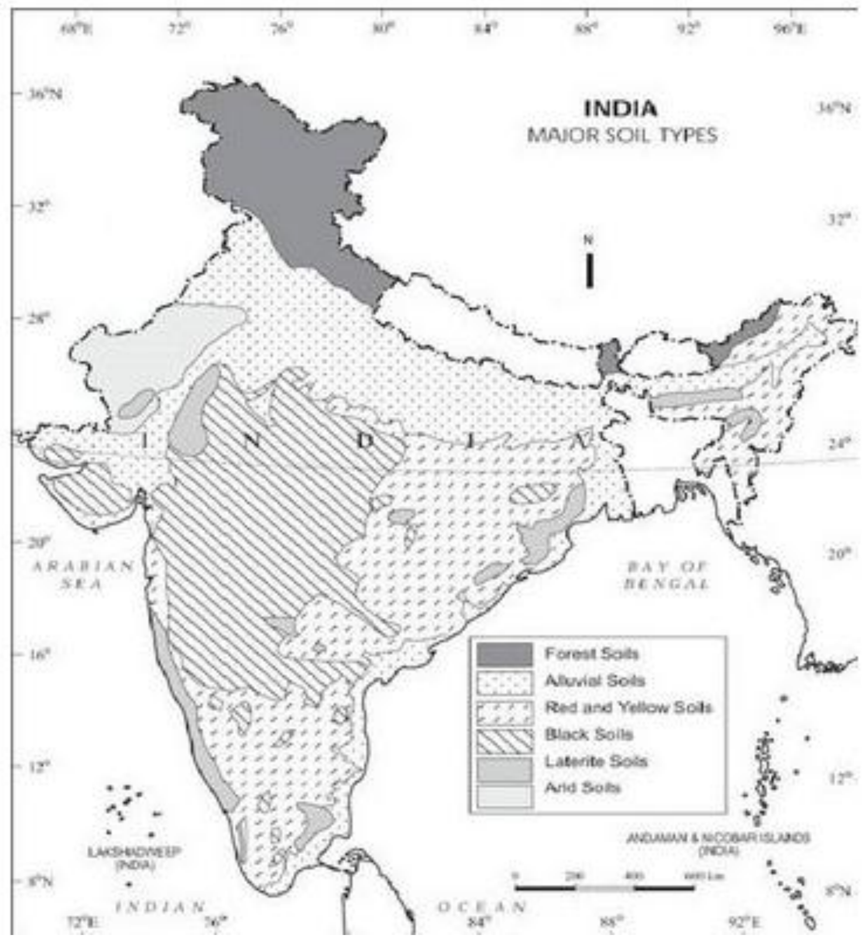
Chemical weathering involves breaking down rock components and internal structure and forming new compounds.

Whereas weathering breaks rocks apart, erosion removes rock debris by mobile agents such as water, wind, or ice.

Principle Soil Types

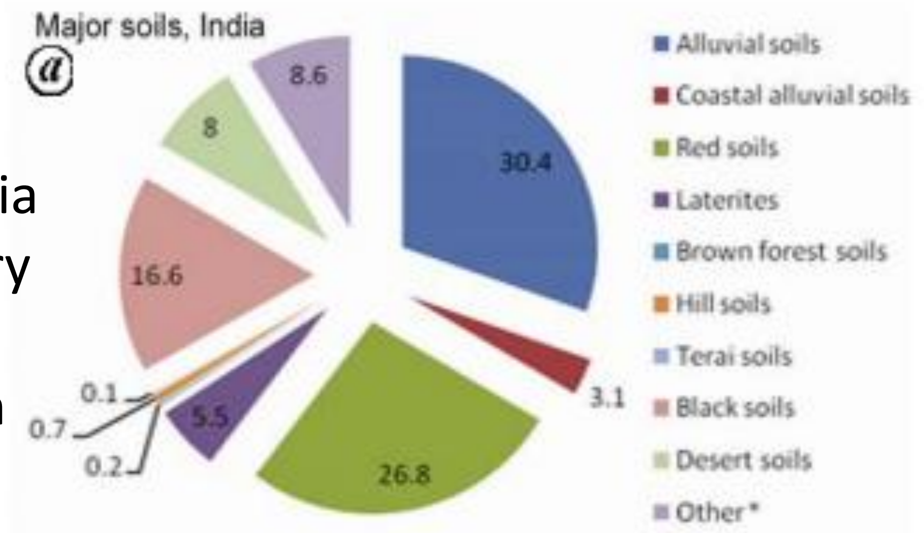
On basis of genesis, colour, composition and location, soil types in India are:

- Alluvial Soil
- Red and yellow Soil
- Laterite Soil
- Saline Soil
- Forest Soil
- Black Soil
- Arid Soil
- Peaty Soil



Classification of types of Soil in India:

- On the basis of USDA soil taxonomy, the types of Soil in India are:
- Inceptisols – covers 39% area in India
- Entisols – covers 28% area in India
- Alfisols – 13 % area in the country
- Vertisols – 8% area in India
- Aridisols 0.4% area of the soils in India
- Ultisols – 2% area in India
- Mollisols – 0.4% area in India



➢ **ALLUVIAL SOILS**:

ALLUVIAL soil are formed when a river gradually loses its carrying capacity with decreasing velocity by slowing down and does not have sufficient power to keep the large particles of soil then these particles settle to the riverbed. Further decrease in velocity causes smaller particles to settle. As the river becomes slow and sluggish it holds only the extremely fine particles in suspension. These particles are deposited finally at the mouth of the river, where they form DELTAS of fine-grained soil. This is the most important and widespread category. It covers 40% of the land area. In fact the entire Northern Plains are made up of these soils. They have been brought down and deposited by three great Himalayan rivers Sutlej, Ganga and Brahmaputra and their tributaries. Through a narrow corridor in Rajasthan they extend into the plains of Gujarat. They are common in eastern coastal plains and in the deltas of Mahanadi, Godavari, Krishna and cauveri.

- **REGUR SOILS:**

  A rich soil consisting of a mixture of sand and clay and decaying organic materials**.** These soils are black in colour and are also known as black soils. Since, they are ideal for growing cotton, they are also called cotton soils, in addition to their normal nomenclature of regur soils. These soils are most typical of the Deccan trap (Basalt) region spread over north-west Deccan plateau and are made up of lava flows. They cover the plateaus of Mahrashtra, Saurashtra, Malwa and southern Madhya Pradesh and extends eastwards in the south along the Godavari and Krishna Valleys.

- **RED SOILS:**

  The ancient crystalline and metamorphic rocks on weathering have given rise to the red soils. The red color is due to the wide diffusion of iron. They are generally poor in nitrogen, phosphorus and humus. These soils are poorer in lime, potash, iron oxide and phosphorus than the regur soils. The clay fraction of red soils is rich in Kaolinite. Red soils are also found under forest vegetation. Red and yellow soils are also seen side by side. The yellow color is due to the high degree of hydration of the ferric oxide in them than that in the red soils. The soils comprise vast areas of Tamil nadu, Karnataka, south eastern Maharashtra, Madhya Pradesh, orissa and Chhotanagpur. In the north it includes the Santhal Paraganas in Bihar, the Birbhum district of West Bengal, the Mirzapur, Jhansi and hamirpur district of uttar Pradesh

- **LATERITE SOILS:**

  The laterite soils is the result of intense leaching owing to heavy tropical rains. They are found along the edge of plateau in the east covering small parts of Tamil Nadu, and Orissa and a small part of Chhotanagpur in the north and Meghalaya in the north-east

- **Desert soils:**

  Desert soils tend to be low in organic matter, and high in alkaline and salts**.** These soils are mostly sandy to loamy fine sand with brown to yellow brown colour, contains large amounts of soluable salt and lime with ph ranging 8.0 to 8.5 nitrogen content is low. The presence of phosphates and nitrates make the desert soil fertail. They are distributed in Haryana, Punjab, rajasthan.

State name	Types of soils present	% of area	Floura and fauna
1.Andhra pradesh	1. Red sandy soils 2. Mixed red and black soil 3. Alluvial soils 4. Desert soils 5. Black soils	70% 10% 8% 10% 2%	dry deciduous mixed type forests trees like sandlewood redsander etc there are 1500 species belonging to 176 families tiger panther spotted deer monkey mangoose wild dogs deers etc
2.Arunachal pradesh	1. Glacier soils 2. Red loamy soils 3. Alluvial soils	20% 75% 5%	Mainly consists of tropical and sub tropical dry and moist broad leaf forests, himalayan subtropical broad leaf forests and species of oak, deodar, pine, fir etc.. It is a well known habitat to a variety of animals. There are around 1200 bird and 359 animal species in the state such as western tragopan, snow leapord etc..

Uses of Soil

- As a supporting material to bear the loads of structures resting on earth
- As a raw constructional material for construction of earth structures (Dams, levees, roads)
- As a processed material (Burnt bricks, concrete mix etc.)
- In Pottery (Kaolinite)
- Kaolinite is also used in paper paint and pharmaceuticals
- Bentonite is used in drilling

Clay Minerology and Soil Structure

Outline

1. Clay Minerals
2. Identification of Clay Minerals
3. Specific Surface (S_s)
4. Interaction of Water and Clay Minerals
5. Interaction of Clay Particles
6. Soil Structure and Fabric
7. Soil Fabric-Natural Soil
8. Soil Fabric-Clay Soils
9. Soil Fabrics-Granular Soils
10. Loess
11. Suggested Homework
12. References

Clay Minerals

Origin of Clay Minerals

•"The contact of rocks and water produces clays, either at or near the surface of the earth" (from Velde, 1995)

Rock +Water $\rightarrow$ Clay

For example,

•The CO_2 gas can dissolve in water and form carbonic acid, which will become hydrogen ions H^+ and bicarbonate ions, and make water slightly acidic.

$$CO_2 + H_2O \rightarrow H_2CO_3 \rightarrow H^+ + HCO_3^-$$

•The acidic water will react with the rock surfaces and tend to dissolve the K ion and silica from the feldspar. Finally, the feldspar is transformed into kaolinite.

Feldspar + hydrogen ions+ water $\rightarrow$ clay (kaolinite) + cations, dissolved silica

$$2KAlSi_3O_8 + 2H^+ + H_2O \rightarrow Al_2Si_2O_5(OH)_4 + 2K^+ + 4SiO_2$$

Note that the hydrogen ion displaces the cations.

Origin of Clay Minerals (Cont.)

- The alternation of feldspar into kaolinite is very common in the decomposed granite.

- The clay minerals are common in the filling materials of joints and faults (fault gouge, seam) in the rock mass. *Weak plane!*

1.8 Mixed Layer Clays

- Different types of clay minerals have similar structures (tetrahedral and octahedral sheets) so that interstratification of layers of different clay minerals can be observed.

- In general, the mixed layer clays are composed of interstratification of expanded water-bearing layers and non-water-bearing layers. Montmorillonite-illite is most common, and chlorite-vermiculite and chlorite-montmorillonite are often found.

(Mitchell, 1993)

Weight-Volume Relationships

For this chapter, we need to know the following

- Mass (M) is a measure of a body's inertia, or its "quantity of matter". Mass does not change at different places.

- Weight (W) is the force of gravity acting on a body.

$$W = M \cdot g \qquad \text{where } g : acceleration\, due\, to\, gravity = 9.81\, {}^{m}\!/\!{}_{\sec^2}$$

$$Density,\ \rho = \frac{Mass}{Volume}$$

$$Unit\, weight,\ \gamma = \frac{Weight}{Volume} = \frac{Mass \cdot g}{Volume}$$

$$So \quad \gamma = \rho \cdot g$$

- The unit weight is frequently used in geotechnical engineering than the density (e.g. in calculating the overburden pressure).

Units of unit weight and density

- ✓ The SI unit of mass density (ρ) is kilograms per cubic meter (kg/m^3).
- ✓ The SI unit of force is Newton, therefore, the unit weights of soils are typically expressed in kN/m^3

Relationship between unit weight and density

The unit weights of soil in kN/m^3 can be obtained from densities in kg/m^3 as

$$\boxed{\gamma\,(\mathrm{kN/m^3}) = \frac{g\rho(\mathrm{kg/m^3})}{1000}}$$

- ✓ The density of water ρ_w varies slightly, depending on the temperature. At 4C^o , water's density is equal to 1000 kg/m^3 or 1 g/cm^3

$$\text{unit weight of water}\,, \gamma_w = 9.81\, {kN}/{m^3}$$

Soil Phases

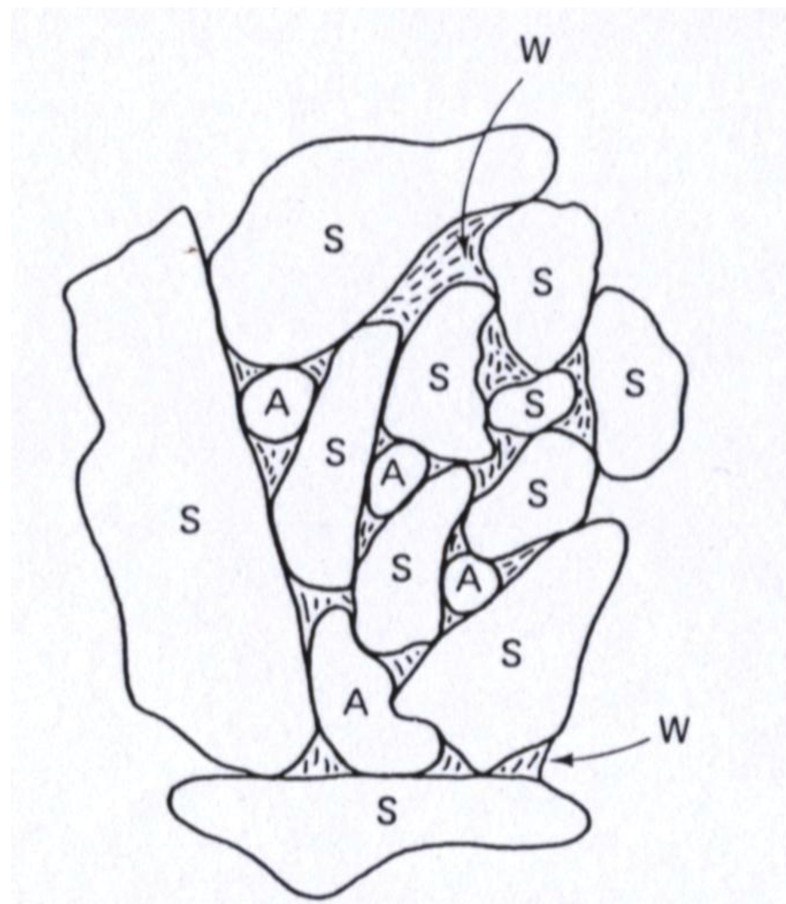
- Soil deposits comprise the accumulated solid particles plus the void space between the particles

- The void spaces are partially or completely filled with water or other liquid.

- Voids space not occupied by fluid are filled with air or other gas.

- Hence soil deposits are referred to as three-phase system, i.e. Solid + Liquid (water) + Gas (air)


mm


W
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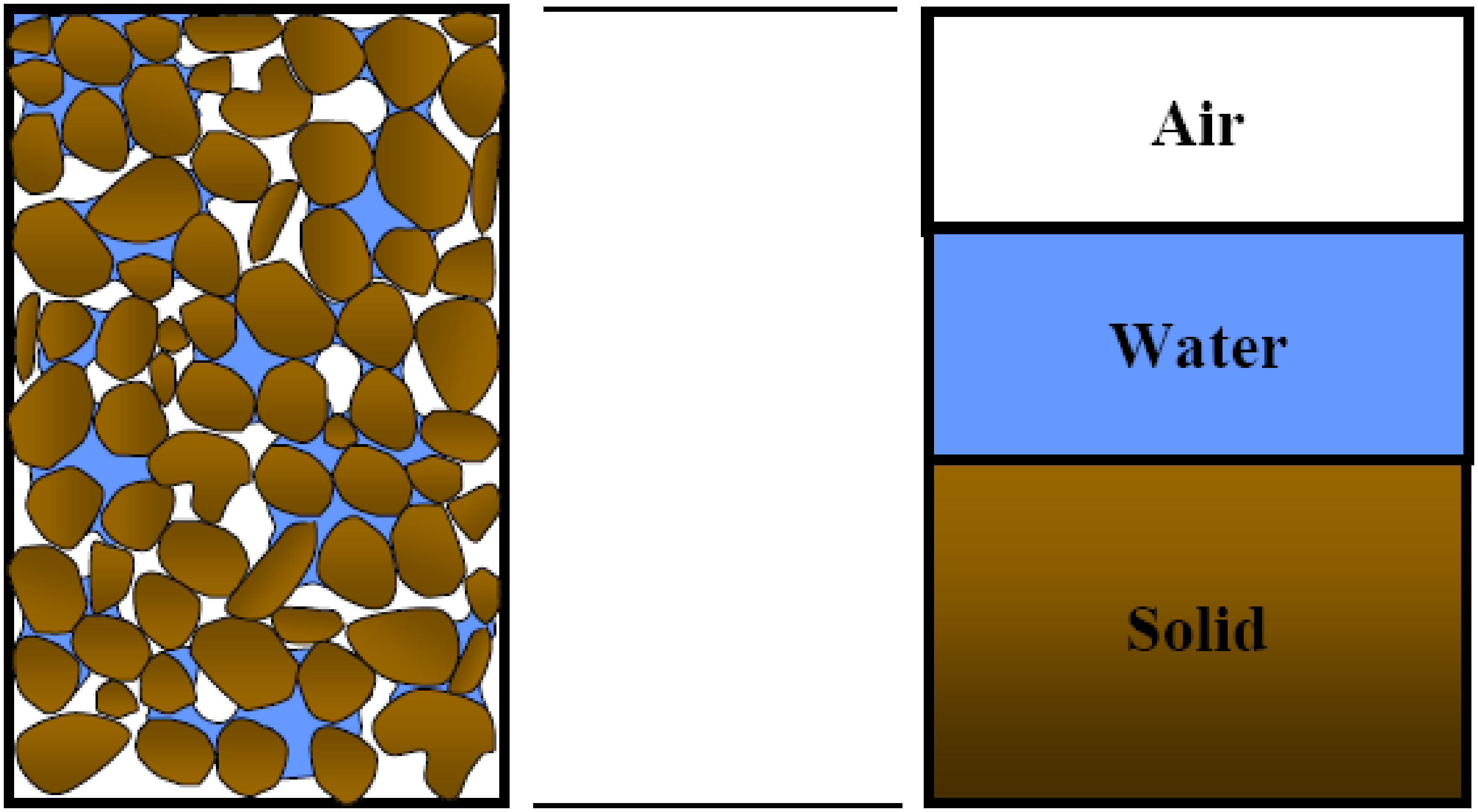
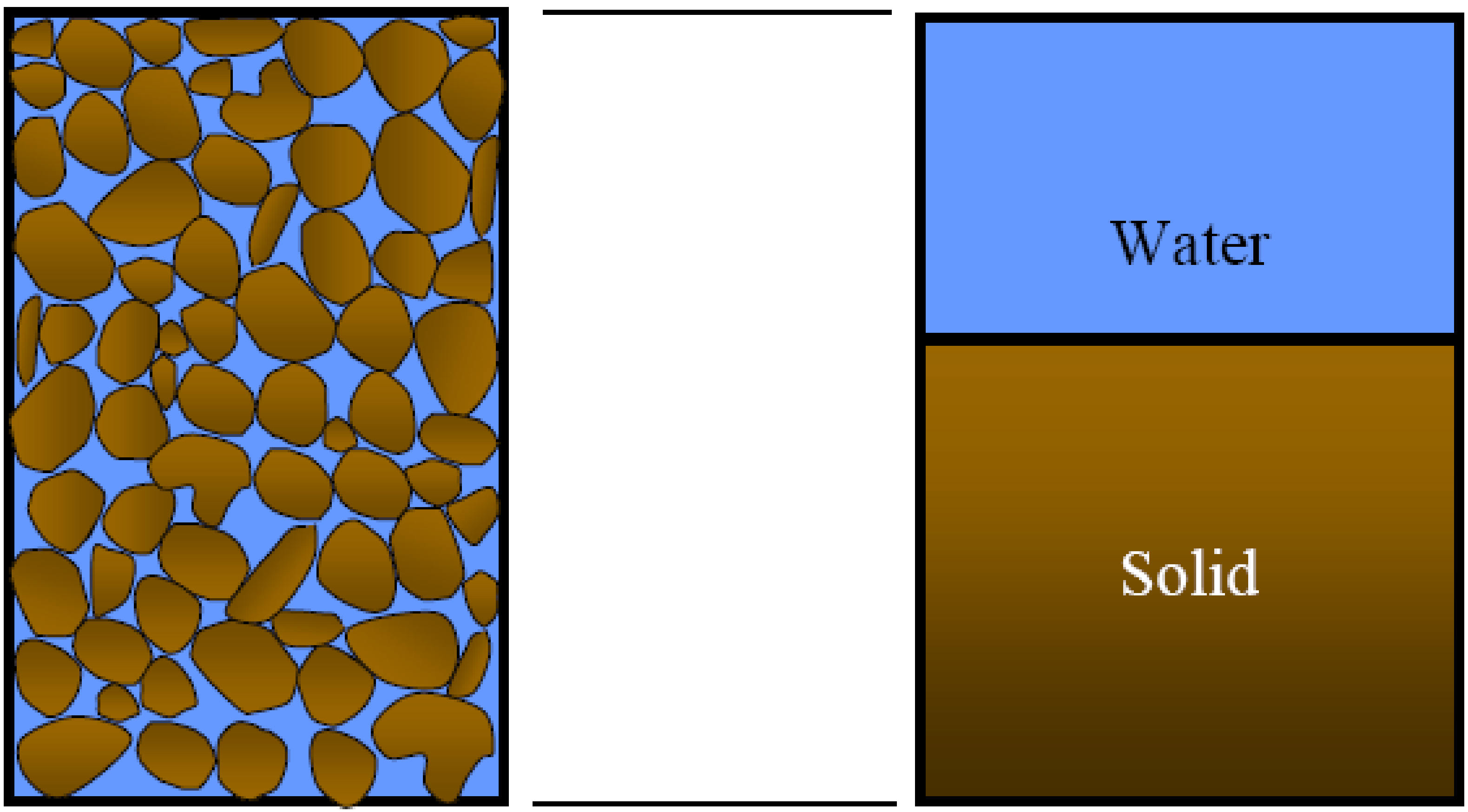
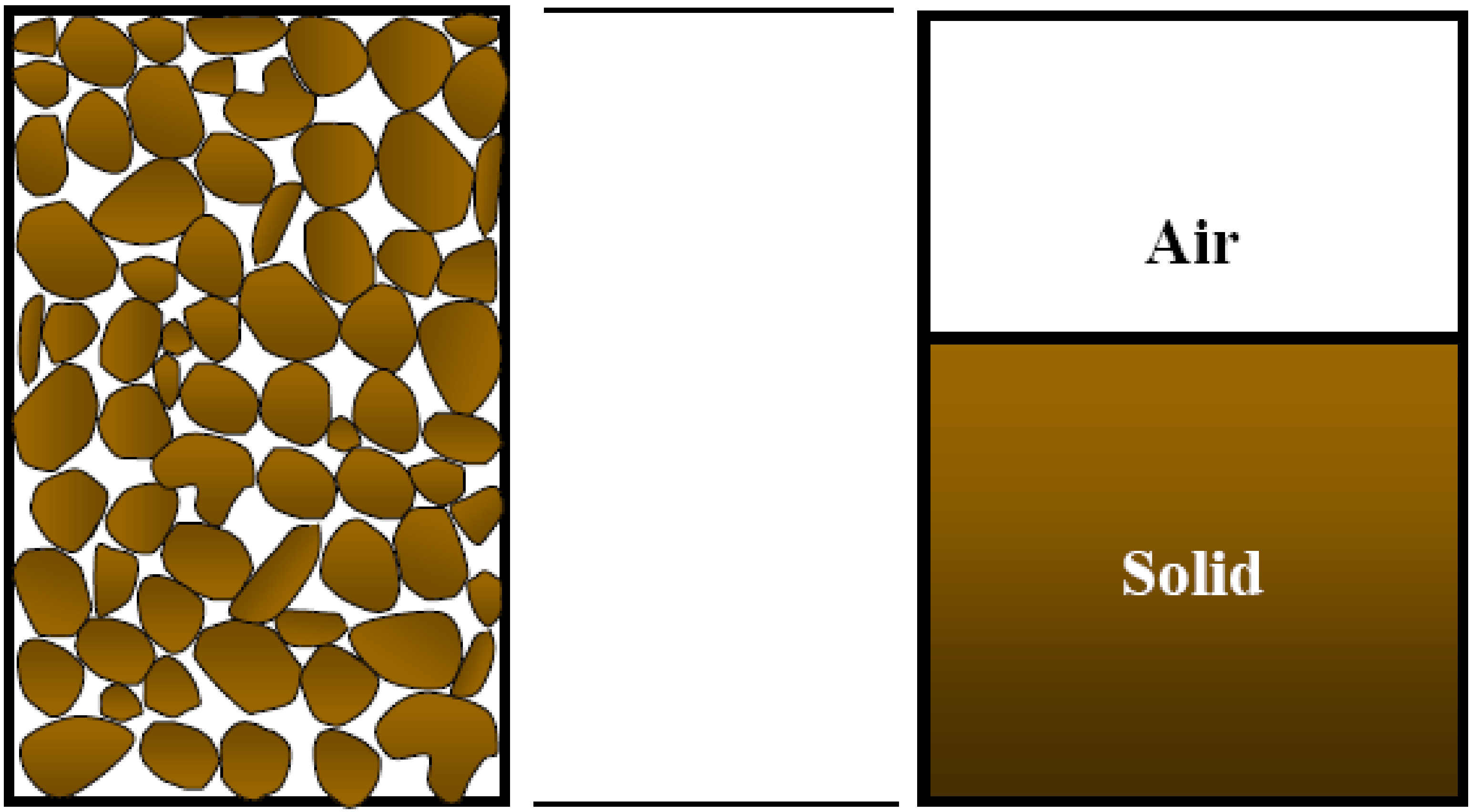
Three Phase Diagram



Fully Saturated Soils (**Two** phase)

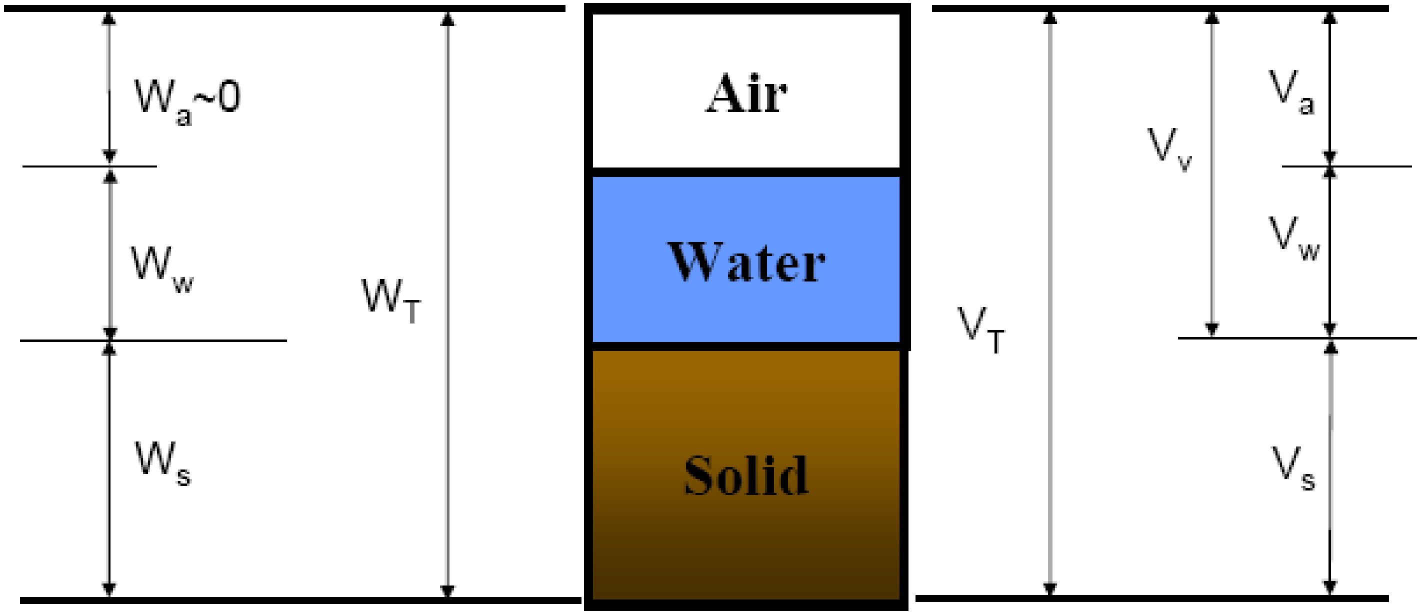


Dry Soils (Two phase) [Oven Dried]



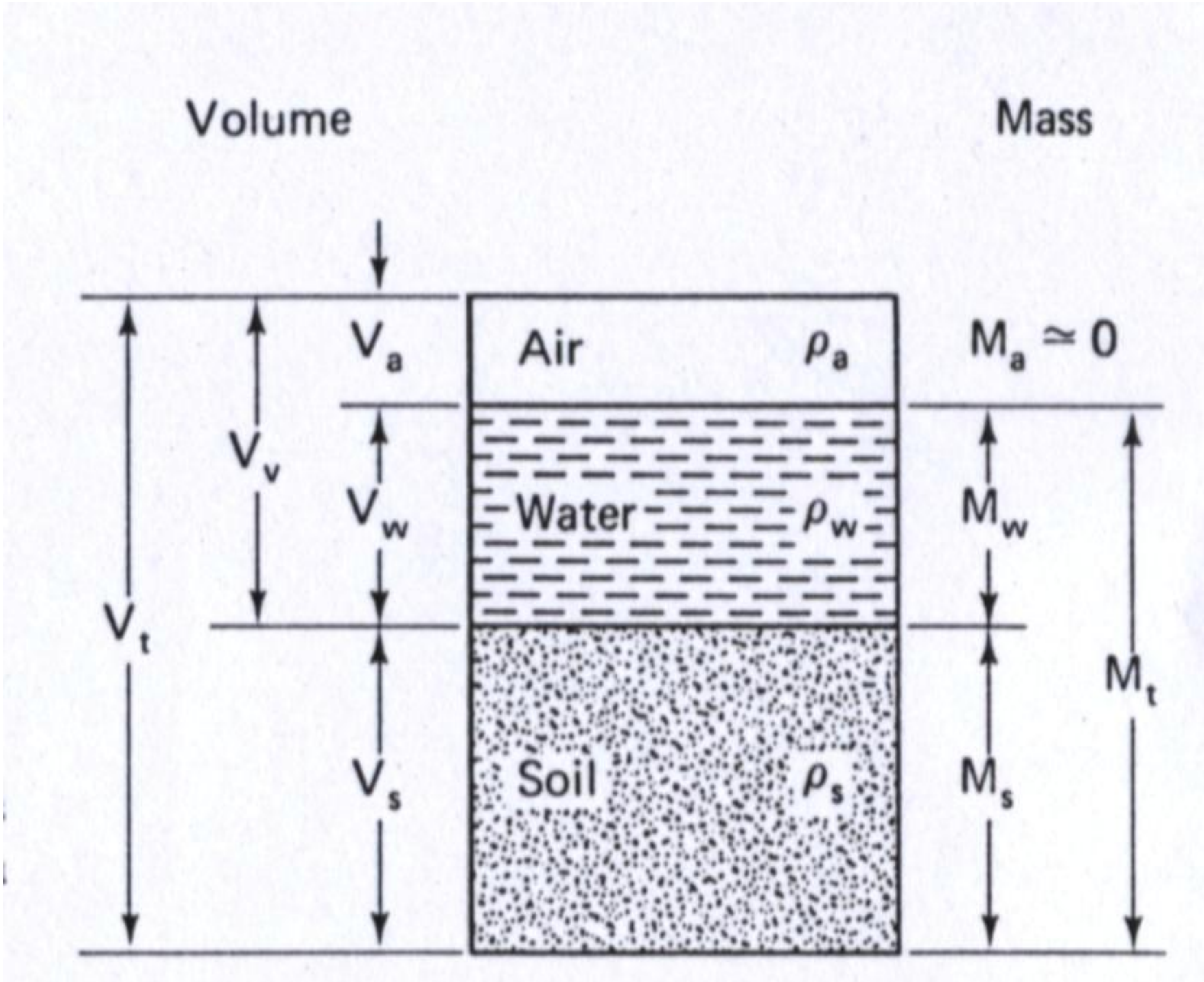
PHASE DIAGRAM

For purpose of study and analysis, it is convenient to represent the soil by a PHASE DIAGRAM, with part of the diagram representing the solid particles, part representing water or liquid, and another part air or other gas.



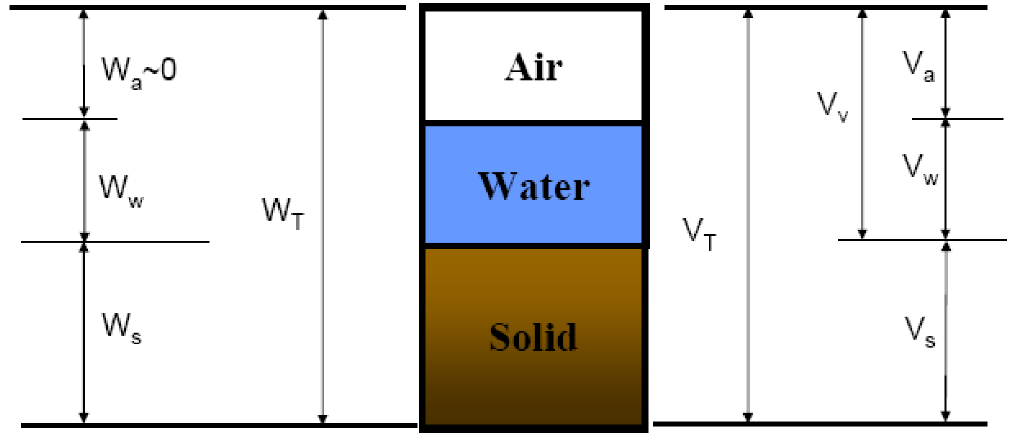
Wt: total weight
Ws: weight of solid
Ww: weight of water
Wa: weight of air = 0

Vt: total volume
Vs: volume of solid
Vw: volume of water
Vv: volume of the void



Volume
Mass
V_a
Air
ρ_a
$M_a \simeq 0$
V_v
V_w
Water
ρ_w
M_w
V_t
M_t
V_s
Soil
ρ_s
M_s

Volumetri



- (1) Void ratio **e**

$$e = \frac{Volume\ of\ \ voids}{Volume\ of\ \ solids} = \frac{V_v}{V_s}$$

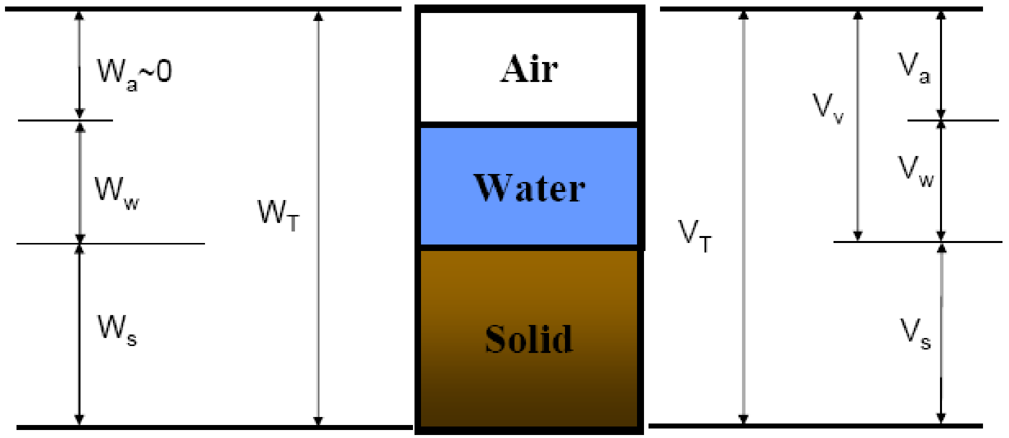
- (2) Porosity **n%**

$$n = \frac{Volume\ of\ \ voids}{Total\ volume\ of\ \ soil\ \ sample} = \frac{V_v}{V_t} \times 100$$

- (3) Degree of Saturation **S% (0 – 100%)**

$$S = \frac{Total\ volume\ of\ \ voids\ contains\ water}{Total\ volume\ of\ \ voids} = \frac{V_w}{V_v} \times 100\%$$

Weight Ratios

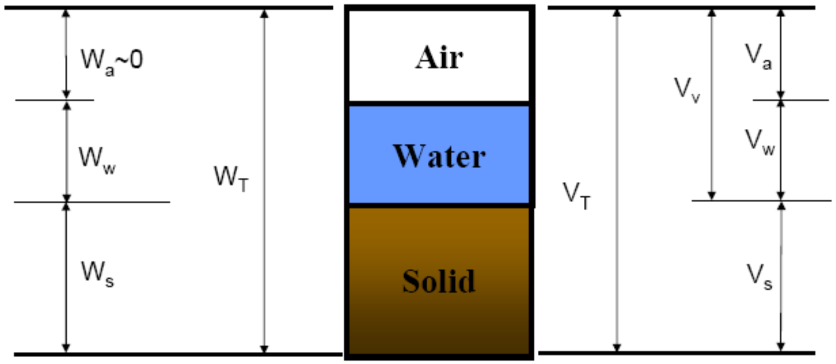


- (1) Water Content ***w%***

$$w = \frac{Weight\ of\ water}{Weight\ of\ \ soil\ solids} = \frac{W_w}{W_s} \cdot 100\%$$

-
-

Soil unit weights



- (1) Dry unit weight

$$\gamma_d = \frac{Weight\ of\ soil\ solids}{Total\ volume\ of\ soil} = \frac{W_s}{V_t}$$

- (2) Total, Wet, Bulk, or Moist unit weight

$$\gamma = \frac{Total\ \mathrm{weight}\ of\ soil}{Total\ volume\ of\ soil} = \frac{W_s + W_w}{V_t}$$

- Saturated unit weight (considering S=100%, V_a =0)

$$\gamma_{sat} = \frac{Weight\ of\ soil\ solids + water}{Total\ volume\ of\ soil} = \frac{W_s + W_w}{V_t}$$

<u>Note:</u> The density/or unit weight are ratios which connects the volumetric side of the PHASE DIAGRAM with the mass/or weight side.

Specific gravity, G_s

The ratio of the mass of a solid particles to the mass of an equal volume of distilled water at 4°C

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

i.e., the specific gravity of a certain material is ratio of the unit weight of that material to the unit weight of water at 4° C.

The specific gravity of soil solids is often needed for various calculations in soil mechanics.

$$G_s = \frac{\gamma_s}{\gamma_w}$$

$G_{w = 1}$

$G_{mercury} = 13.6$

Typical Values of Specific Gravity

Table 3.1 Specific Gravities of Minerals

Quartz	2.65
K-Feldspars	2.54–2.57
Na–Ca-Feldspars	2.62–2.76
Calcite	2.72
Dolomite	2.85
Muscovite	2.7–3.1
Biotite	2.8–3.2
Chlorite	2.6–2.9
Pyrophyllite	2.84
Serpentine	2.2–2.7
Kaolinite	2.61[a]
	2.64 ± 0.02
Halloysite (2 H_2O)	2.55
Illite	2.84[a]
	2.60–2.86
Montmorillonite	2.74[a]
	2.75–2.78
Attapulgite	2.30

[a] Calculated from crystal structure.

(Lambe and Whitman, 1979)

Table 2.2 *Specific Gravities of Common Minerals*[a]

Mineral	*G*
Halite	2.1–2.6
Gypsum	2.3–2.4
Serpentine	2.3–2.6
Orthoclase	2.5–2.6
Chalcedony	2.6–2.64
Quartz	2.65
Plagioclase	2.6–2.8
Chlorite and illite	2.6–3.0
Calcite	2.7
Muscovite	2.7–3.0
Biotite	2.8–3.1
Dolomite	2.8–3.1
Anhydrite	2.9–3.0
Pyroxene	3.2–3.6
Olivine	3.2–3.6
Barite	4.3–4.6
Magnetite	4.4–5.2
Pyrite	4.9–5.2
Galena	7.4–7.6

[a] A. N. Winchell (1942).

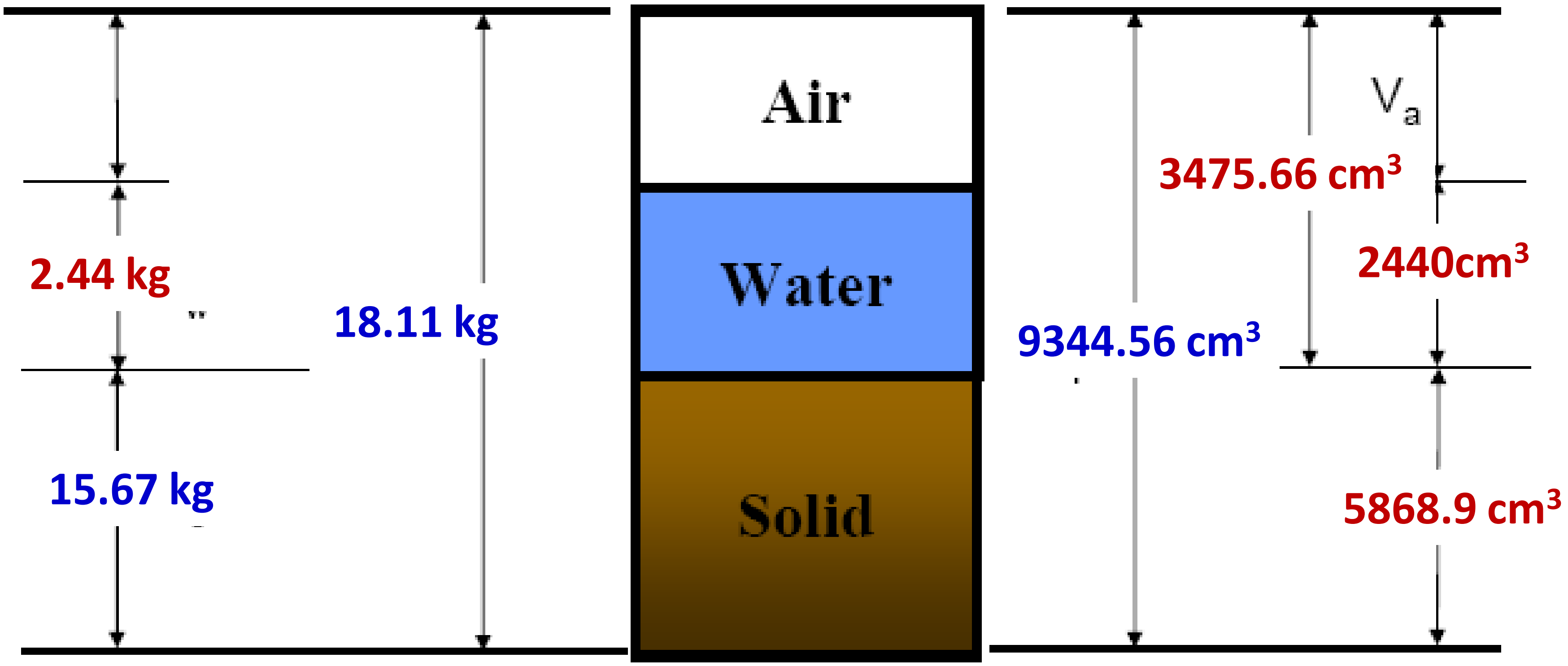
(Goodman, 1989)

- Expected Value for Gs

Type of Soil	Gs
Sand	2.65 - 2.67
Silty sand	2.67 – 2.70
Inorganic clay	2.70 – 2.80
Soils with mica or iron	2.75 – 3.00
Organic soils	< 2.00

Example 1:

- In its natural state, a moist soil has a total volume of 9344.56 cm^3 and a mass 18.11 kg. The oven-dry mass of soil is 15.67 kg. If G_s = 2.67, calculate the moisture content, moist unit weight, void ratio and degree of saturation.



Relationships Between Various Physical Properties

All the weight- volume relationships needed in soil mechanics can be derived from appropriate combinations of six fundamental definitions. They are:

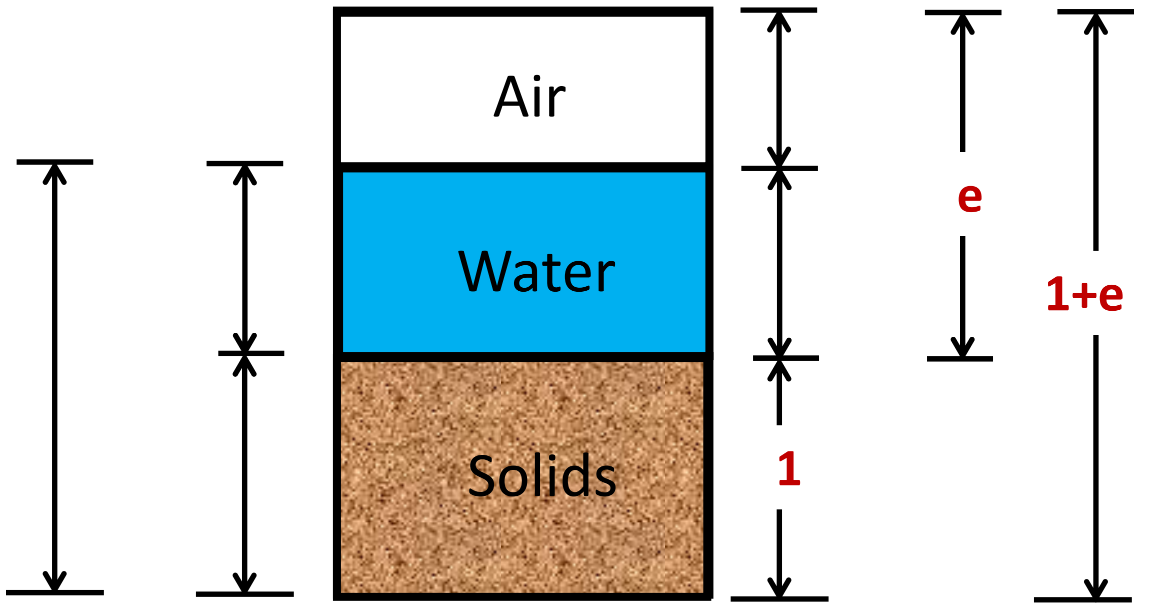
1. **Void ratio**
2. **Porosity**
3. **Degree of saturation**
4. **Water content**
5. **Unit weight**
6. **Specific gravity**

Use of Phase diagrams in finding relationships physical properties

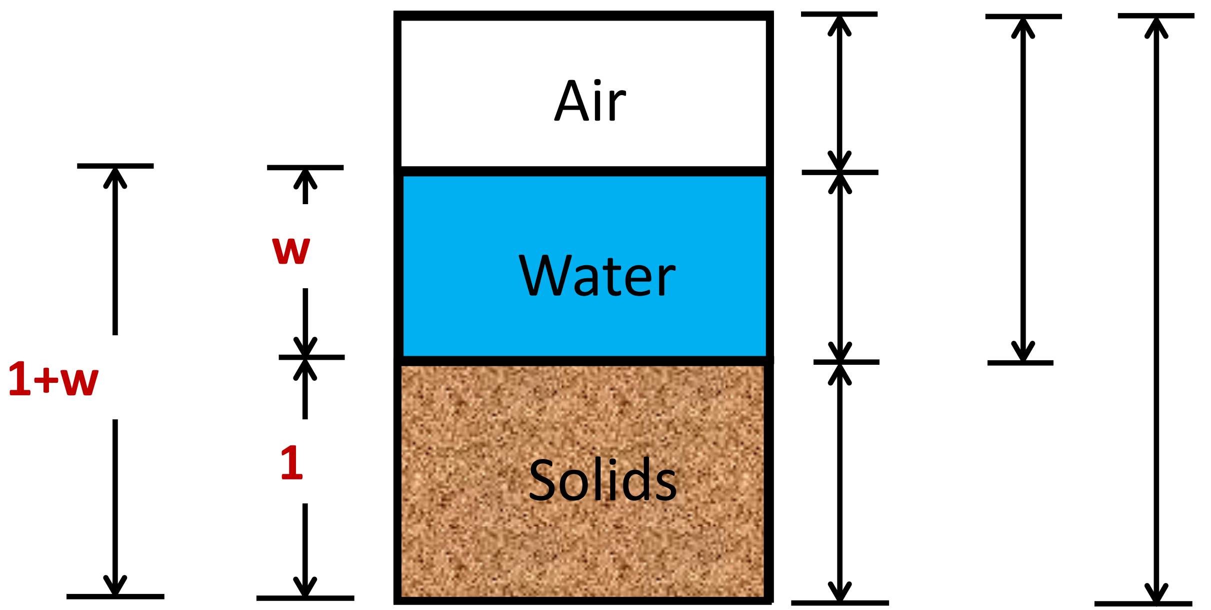
- *Remember the following simple rules:*

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

If given e, assume V_s =1



If given w, assume W_s =1



1. Relationship between e and n

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

Also, from Eq. (3.6),

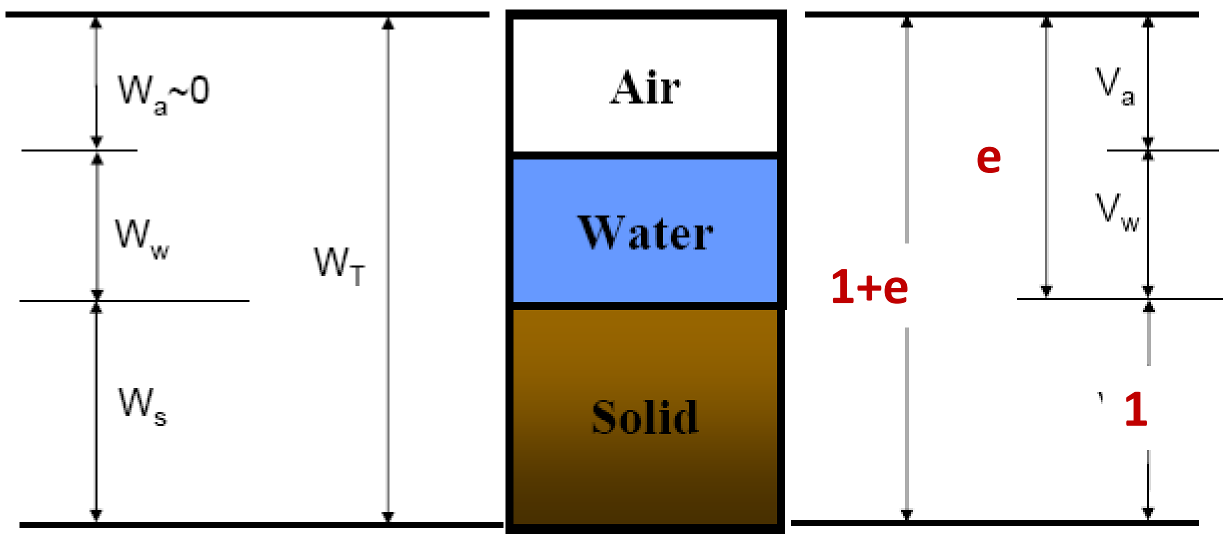
$$n = \frac{e}{1 + e} \tag{3.7}$$

Using phase diagram

Given : e

required: n

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



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

$$w = \frac{W_w}{W_s} = \frac{\gamma_w V_w}{\gamma_s V_s} = \frac{\gamma_w V_w}{\gamma_w G_s V_s} = \frac{V_w}{G_s V_s}$$

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

$$\boxed{Se = wG_s}$$

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

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

Textbook derivation

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

and

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

or

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

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

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

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

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

or

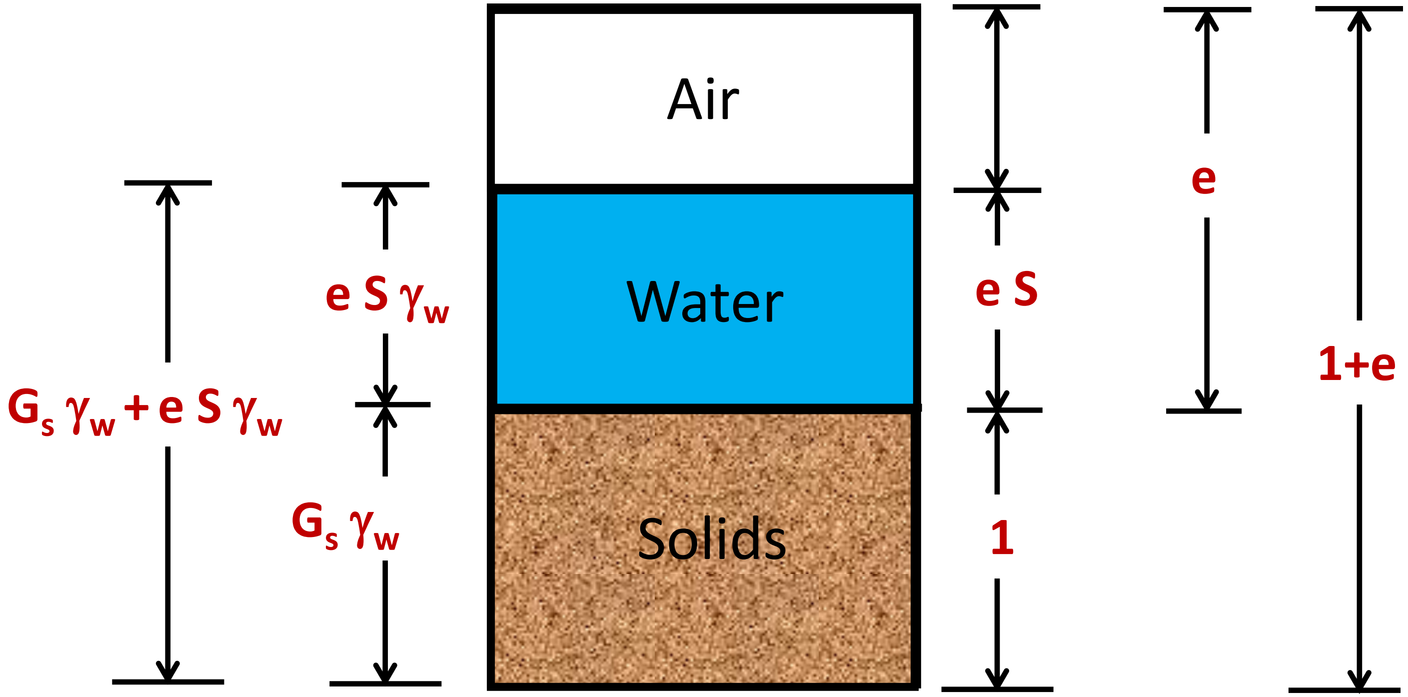
$$\boxed{Se = wG_s} \tag{3.18}$$

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

Using phase diagram

Given : e, S, G_s

required: w



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

3. Relationship among γ, e, S and G_s

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

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

Notes:

Unit weights for dry, fully saturated and submerged cases can be derived from the upper equation

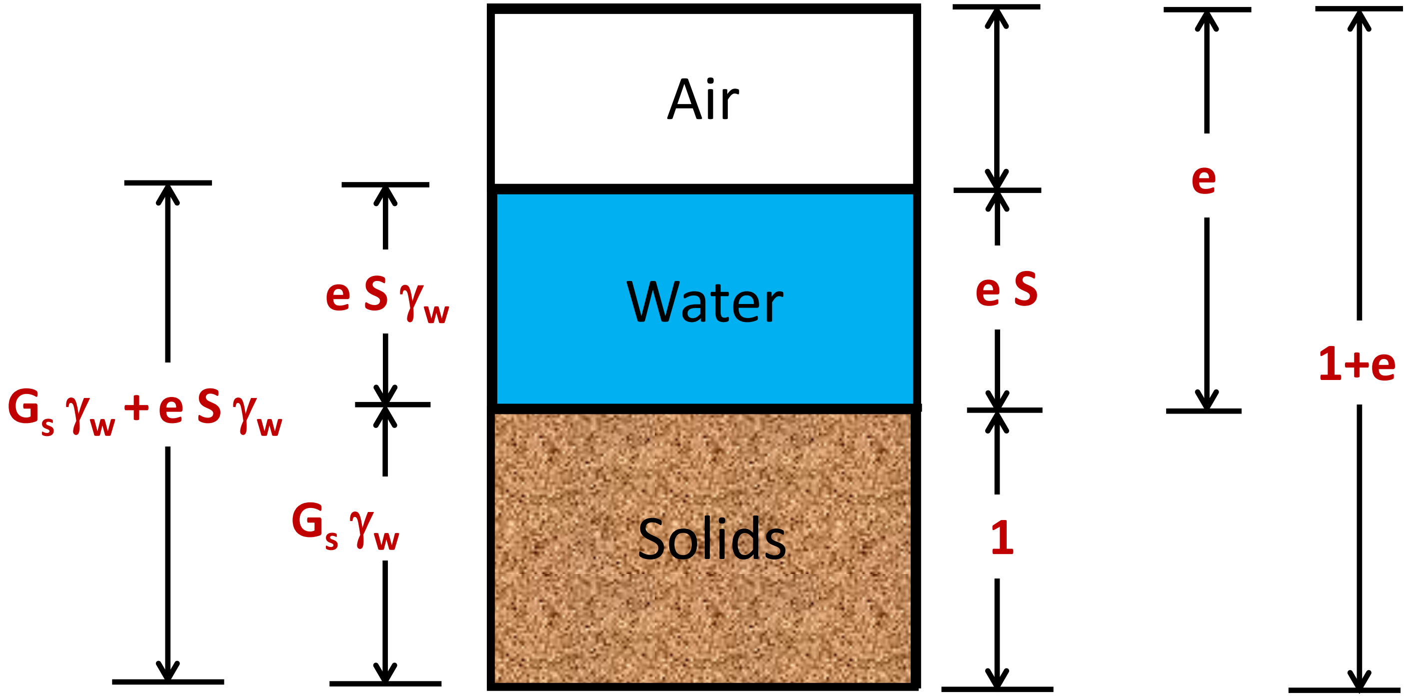
Water content can be used instead of degree of saturation.

3. Relationship among γ, e, S and G_s

Using phase diagram

Given : e, S, G_s

required: γ



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

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

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

Various Unit Weight Relationships

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

Moist unit weight (γ)		Dry unit weight (γ_d)		Saturated unit weight (γ_{sat})	
Given	**Relationship**	**Given**	**Relationship**	**Given**	**Relationship**
w, G_s, e	$\dfrac{(1+w)G_s\gamma_w}{1+e}$	γ, w	$\dfrac{\gamma}{1+w}$	G_s, e	$\dfrac{(G_s+e)\gamma_w}{1+e}$
S, G_s, e	$\dfrac{(G_s+Se)\gamma_w}{1+e}$	G_s, e	$\dfrac{G_s\gamma_w}{1+e}$	G_s, n	$[(1-n)G_s+n]\gamma_w$
w, G_s, S	$\dfrac{(1+w)G_s\gamma_w}{1+\dfrac{wG_s}{S}}$	G_s, n	$G_s\gamma_w(1-n)$	G_s, w_{sat}	$\left(\dfrac{1+w_{sat}}{1+w_{sat}G_s}\right)G_s\gamma_w$
w, G_s, n	$G_s\gamma_w(1-n)(1+w)$	G_s, w, S	$\dfrac{G_s\gamma_w}{1+\left(\dfrac{wG_s}{S}\right)}$	e, w_{sat}	$\left(\dfrac{e}{w_{sat}}\right)\left(\dfrac{1+w_{sat}}{1+e}\right)\gamma_w$
S, G_s, n	$G_s\gamma_w(1-n)+nS\gamma_w$	e, w, S	$\dfrac{eS\gamma_w}{(1+e)w}$	n, w_{sat}	$n\left(\dfrac{1+w_{sat}}{w_{sat}}\right)\gamma_w$
		γ_{sat}, e	$\gamma_{sat}-\dfrac{e\gamma_w}{1+e}$	γ_d, e	$\gamma_d+\left(\dfrac{e}{1+e}\right)\gamma_w$
		γ_{sat}, n	$\gamma_{sat}-n\gamma_w$	γ_d, n	$\gamma_d+n\gamma_w$
		γ_{sat}, G_s	$\dfrac{(\gamma_{sat}-\gamma_w)G_s}{(G_s-1)}$	γ_d, S	$\left(1-\dfrac{1}{G_s}\right)\gamma_d+\gamma_w$
				γ_d, w_{sat}	$\gamma_d(1+w_{sat})$

Solution of Phase Problems

- ***Method 1: Memorize relationships***

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

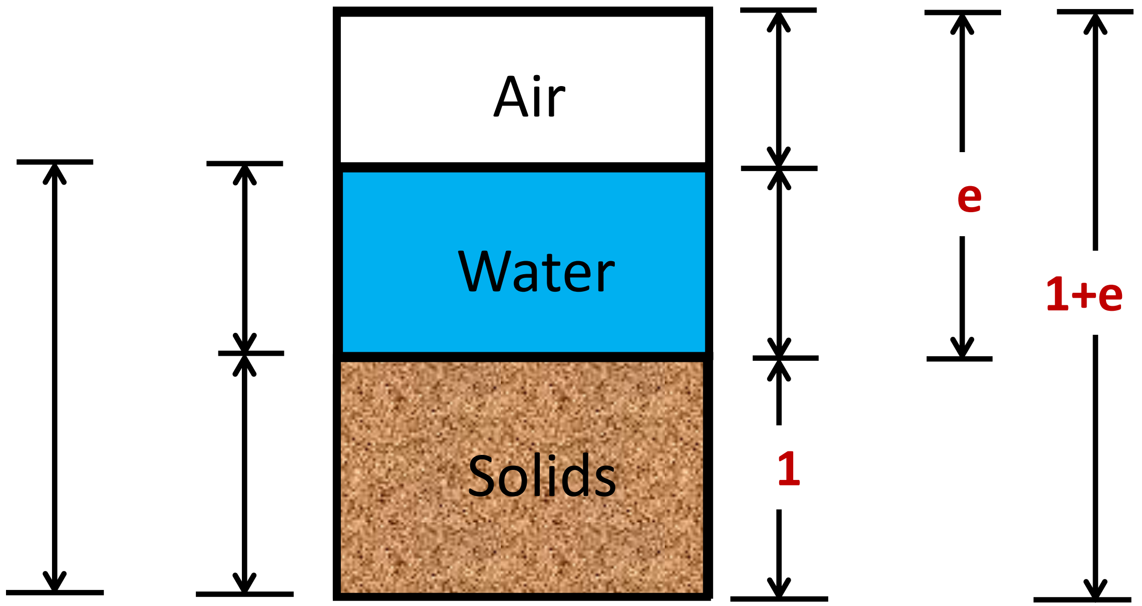
$$n = \frac{e}{1+e} \qquad \gamma_d = \frac{\gamma}{1+w}$$

- ***Method 2: Return to Basics***
- *Remember the following simple rules:*

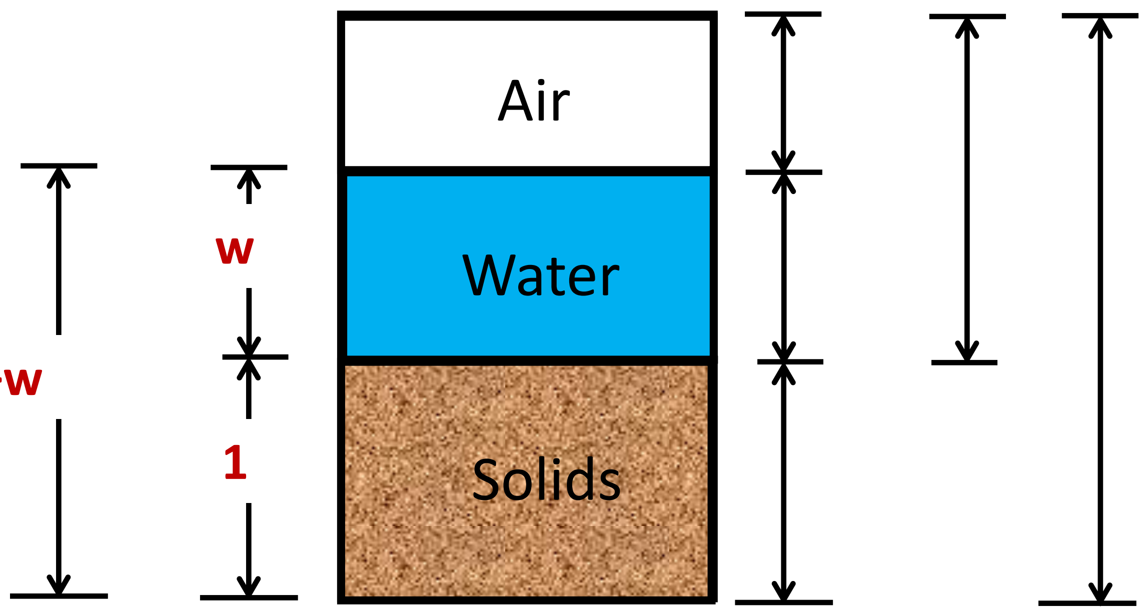
1. Remember the basic definitions of *w*, e, G_s, S, etc.
2. Draw a phase diagram.
3. Assume either V_s=1 or V_t=1 or w_s = 1 depending on given values.
4. Solve the phase diagram.

Method 2: Problem assumptions

If given e, assume $V_s = 1$



If given w, assume $W_s = 1$



Example 2

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

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

Method 1a

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

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

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

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

Method 1b

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

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

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

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

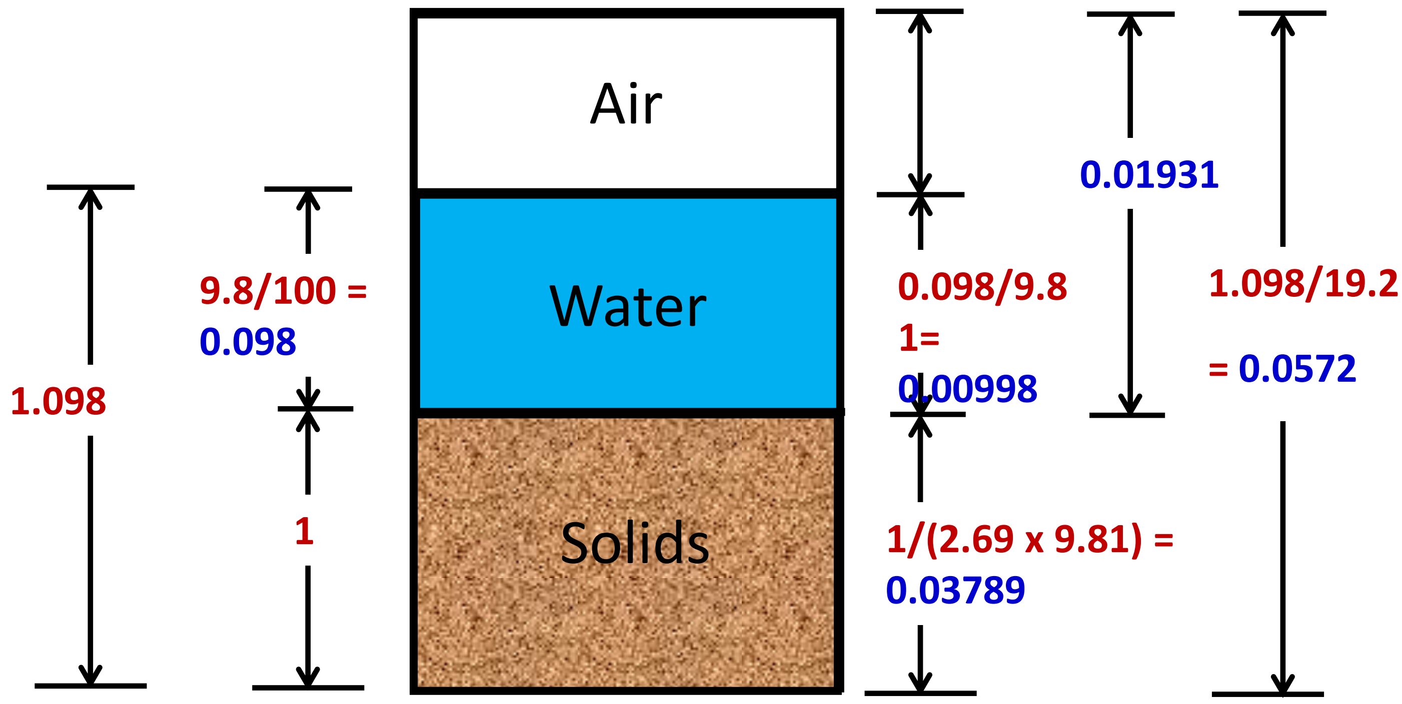
$n = V_v/V = e/1+e = 0.50852/(1+0.50852) = 0.338$

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

Method 2:

Given : w = 9.8% , $\gamma = 19.2$ kN/m^3, G_s = 2.69
required: γ_d , e, n, S

assume W_s =1 kN



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

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

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

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

Example 3

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

Solution:

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

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

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

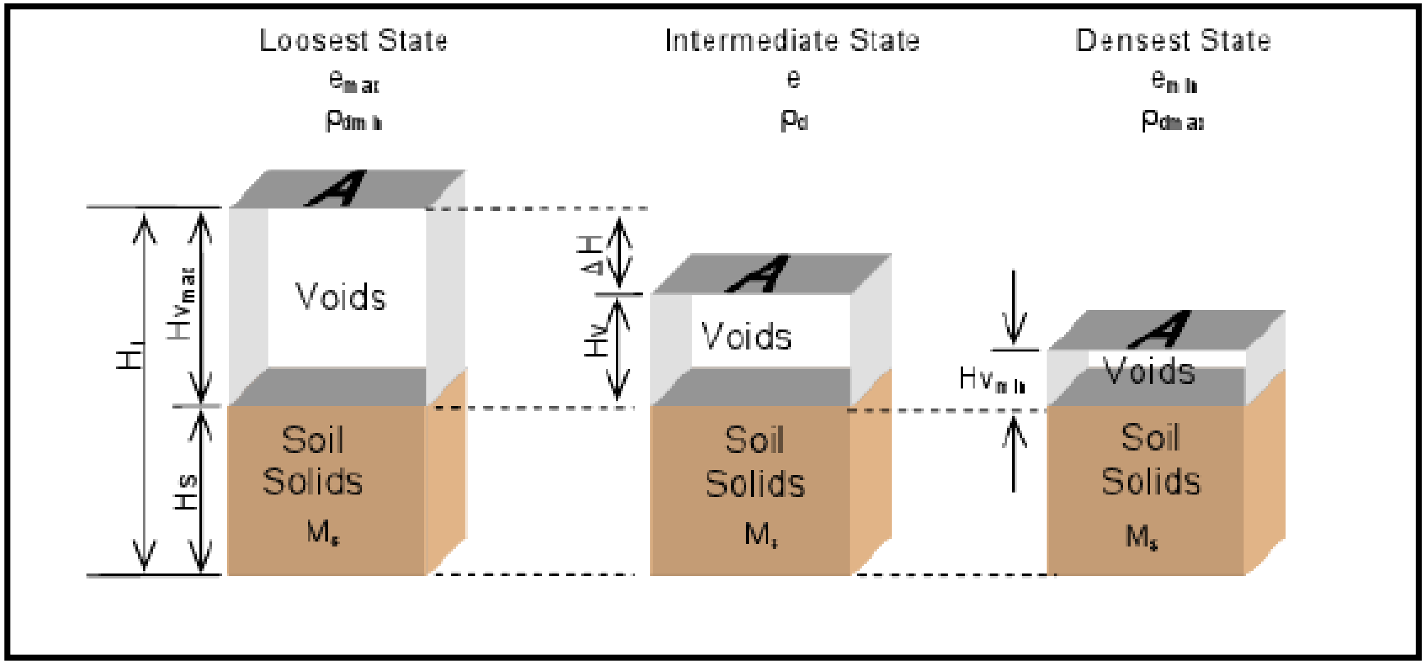
$$\therefore 2.06 = \frac{2.69 + 0.312}{1+e} \times 1.0$$

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

Relative Density

The relative density is the parameter that compare the volume reduction achieved from compaction to the maximum possible volume reduction

The relative density Dr, also called density index is commonly used to indicate the IN SITU denseness or looseness of granular soil.



Volume reduction from compaction of granular soil

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

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

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

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

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

Derivation

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100\%$$

$$= \frac{\gamma_{d\,max}}{\gamma_d} \times \frac{\gamma_d - \gamma_{d\,min}}{\gamma_{d\,max} - \gamma_{d\,min}} \times 100\%$$

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

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

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

Remarks

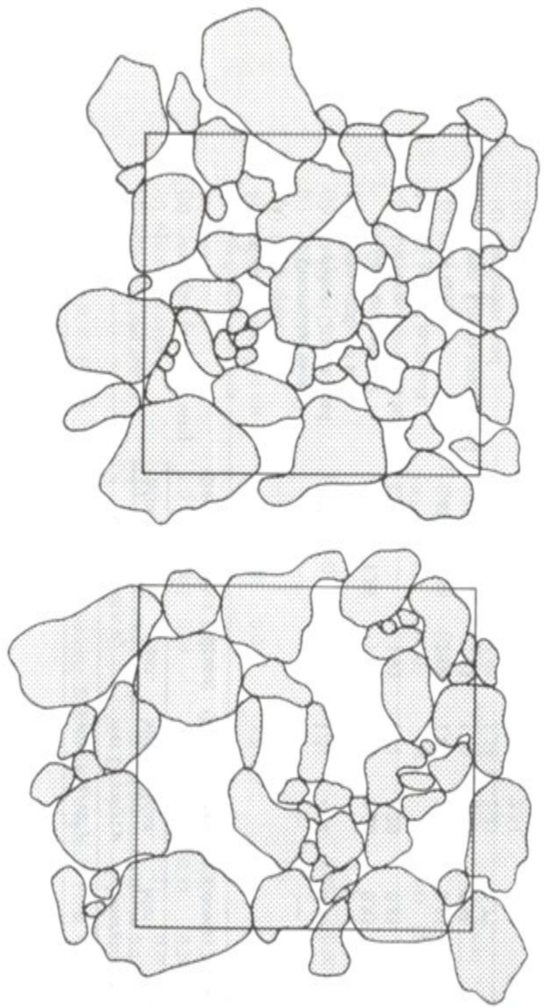
The relative density of a natural soil very strongly affect its engineering behavior.

The range of values of D_r may vary from a minimum of zero for very LOOSE soil to a maximum of 100% for a very DENSE soil.

Because of the irregular size and shape of granular particles, it is not possible to obtain a ZERO volume of voids. (Do you remember well-graded vs. poorly-graded!!)

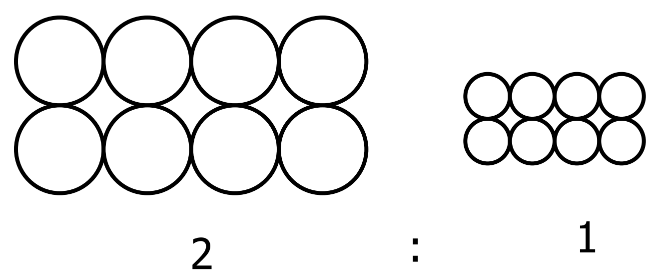
ASTM test designations D-4253 and D-4254 (2007) provide procedure for determining maximum and minimum dry unit weights of granular soils.

The Relative Density (Dr) (Cont.)



Holtz and Kovacs, 1981

"The relative density (or void ratio) alone is not sufficient to characterize the engineering properties of granular soils" (Holtz and Kovacs, 1981). Two soils with the same relative density (or void ratio) may contain very different pore sizes. That is, the pore size distribution probably is a better parameter to correlate with the engineering properties (Santamarina et al., 2001).

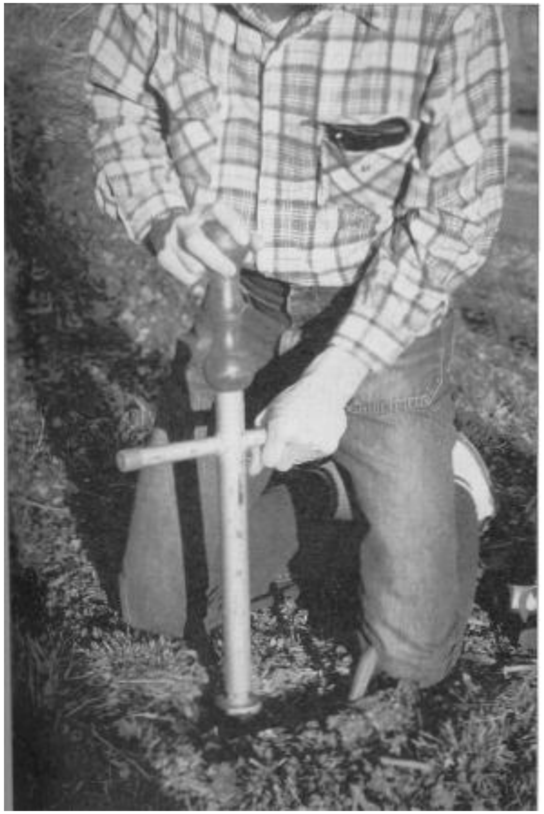


- The relative density of a natural soil deposit very strongly affects its engineering behavior. Consequently, it is important to conduct laboratory tests on samples of the sand at the same relative density as in the field (from Holtz and Kovacs, 1981). (compaction)

Determination of Bulk Density

To determine bulk density we need to measure the dry mass and the total volume occupied by the soil sample.

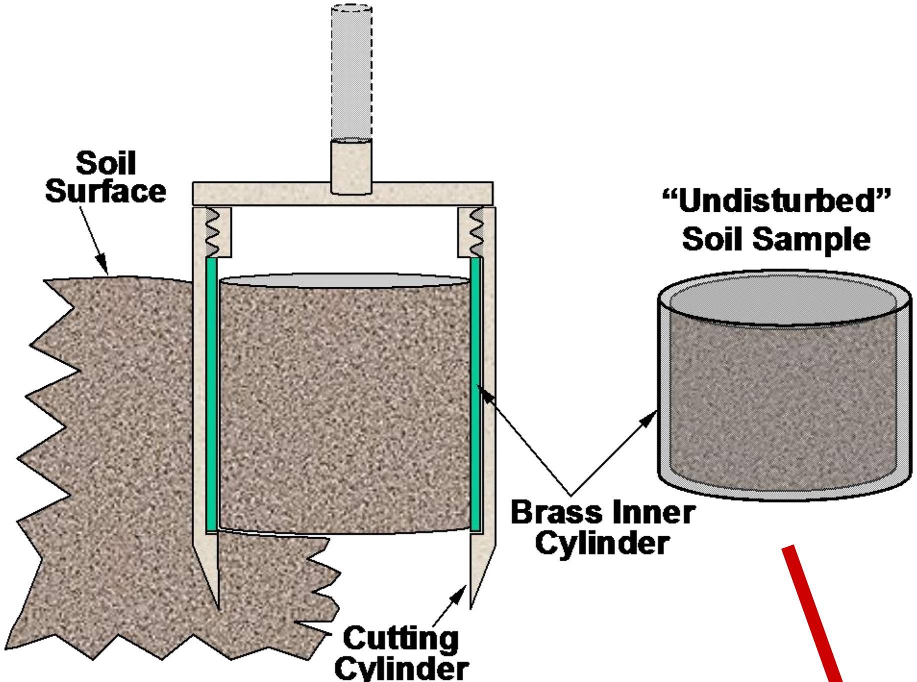
CORE METHOD:



A cylindrical metal sampler is driven into the soil to remove a known volume (core).

The core (soil + brass cylinder) is oven-dried at 105 ^{o}C to remove non-structural soil water till the mass remains constant (usually after 24–48 hrs).

Core Method



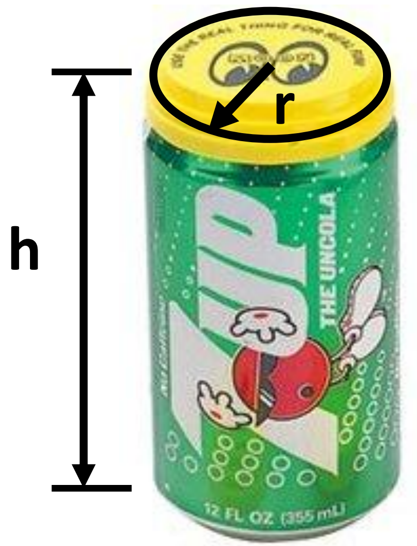
Dry Mass Sample

Oven
105°C

Volume Cylinder

$$V_c = V_t = r^2 \cdot \pi \cdot h$$

Core Method - Example



r = 3 cm

h = 12 cm

M_s = 480 g (oven-dry mass)

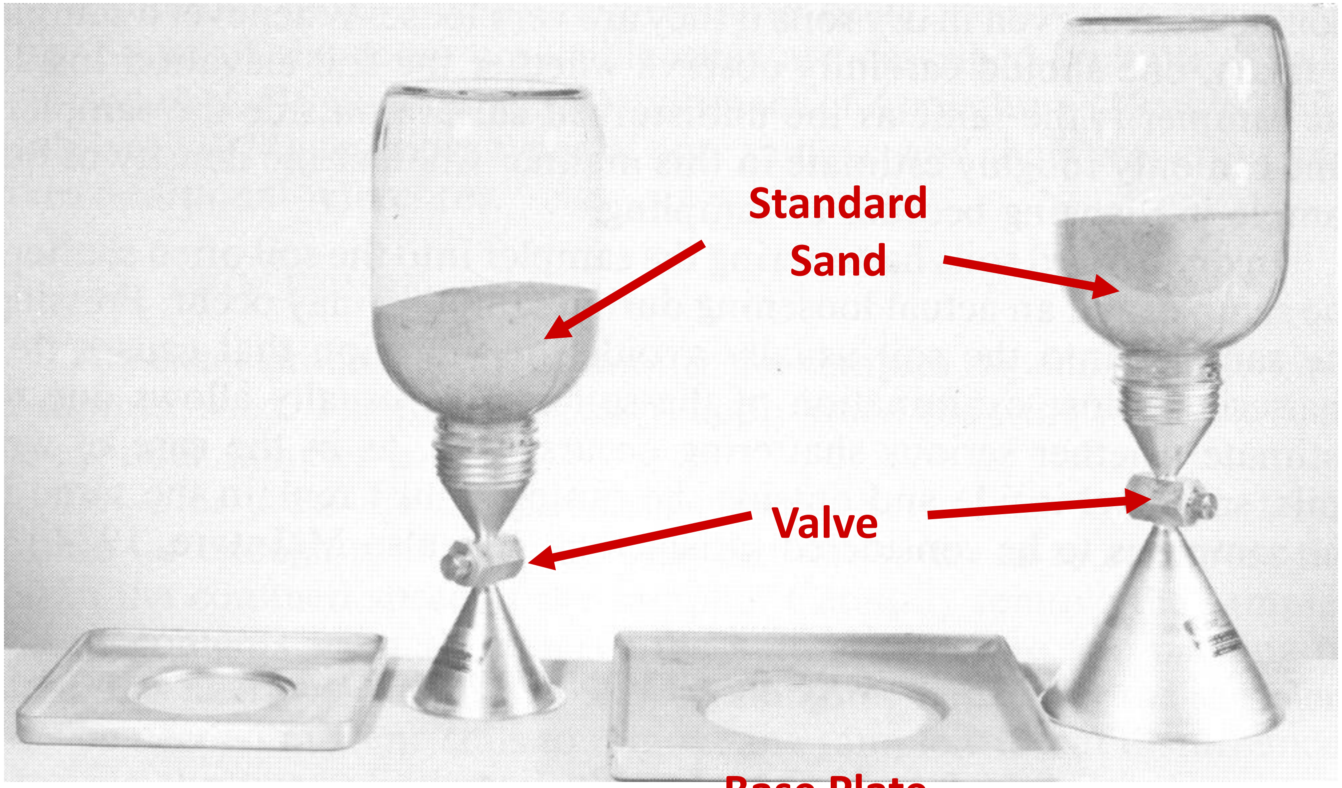
$$V_t = r^2 \cdot \pi \cdot h = 3^2 \cdot 3.14 \cdot 12 = 339\,cm^3$$

$$\rho_b = {M_s}/{V_t} = {480g}/{339cm^3} = 1.42\,g/cm^3$$

Excavation Methods – Sand Funnel

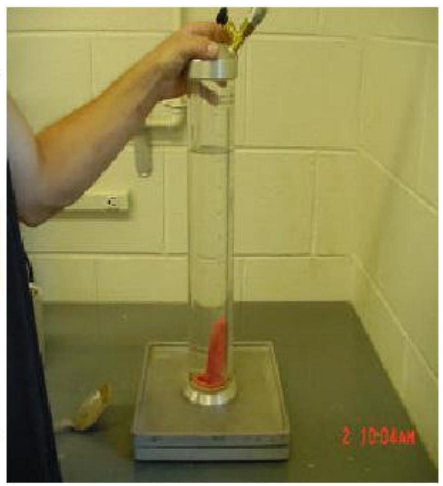
A quantity of soil is excavated in the field, dried at 105 ºC and weight.

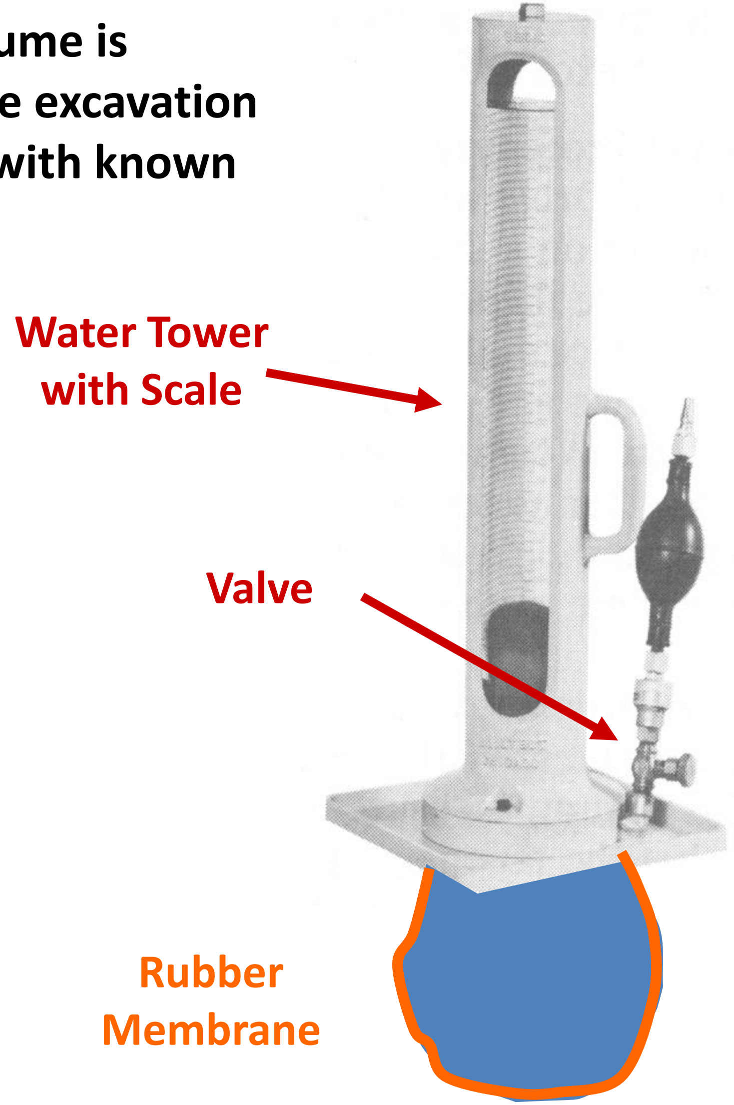
The volume is determined by filling the excavated hole with a well defined standard sand of which the volume per unit mass is known.
(SAND-FUNNEL Method)



Excavation Methods – Rubber Balloon

In the RUBBER BALLOON Method the volume is determined by inserting a balloon into the excavation and filling it with water or an other fluid with known density.





Engineering Classification of Soils

Purpose

- Classifying soils into groups or sub-groups with similar engineering behavior.
- Classification systems were developed in terms of *simple* indices (GSD and plasticity).
- These classifications can provide geotechnical engineers with general guidance about engineering properties of the soils through the *accumulated experience*.

Communicate between engineers

Simple indices

GSD, LL, PI → Classification system (Language) → Estimate engineering properties → Achieve engineering purposes

Use the accumulated experience

I. Overview

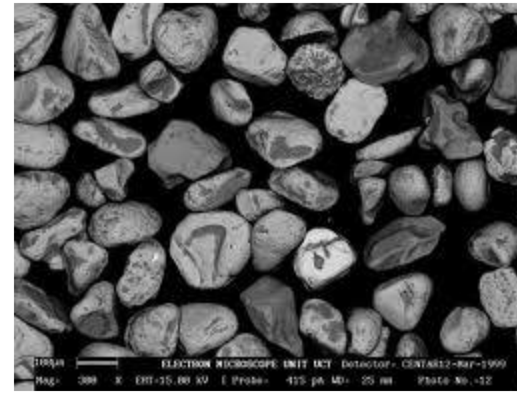
A. Two Systems of Classification

1. Pedagogical Classifications

(soil weathering, texture, chemistry, profile thickness, etc.)

2. Engineering Classifications

– soil texture
– degree of plasticity (Atterberg's Limits)


100μm
ELECTRON MICROSCOPE UNIT UCT
Mag= 300 X EHT=15.00 kV
Photo No.=12

If I give you a bag of 1-Kg soil taken from an under construction site and ask you the following questions.

1. What is the most basic classification of soil?
2. What are the methods of soil gradation or grain size distribution?
3. How do you define the soil types? Clay, Silt, Sand, Gravel or cobble and boulder
4. Calculate $\mathbf{D_{10}}$, $\mathbf{D_{30}}$ **and** $\mathbf{D_{60}}$ of this soil using the sieve analysis?
5. Calculate both the $\mathbf{C_u}$ **and** $\mathbf{C_C}$ of this soil?
6. Is this soil poorly, gap or well graded, Liquid limit and Plastic limit? How do you define theses terms?

You will learn in today's class

Sieve Analysis

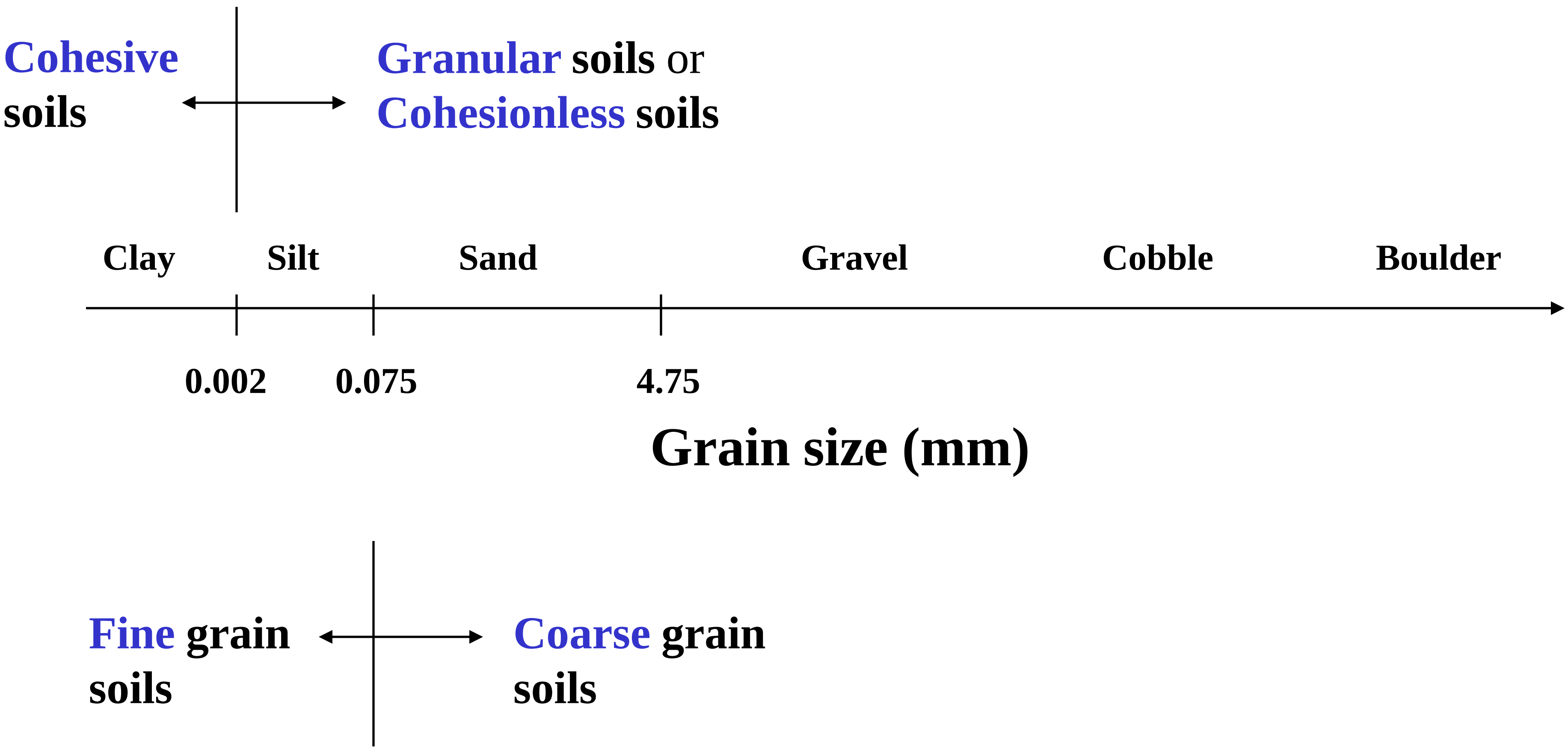
Purpose:

- This test is performed to determine the percentage of different grain sizes contained within a soil.
- The mechanical or sieve analysis is performed to determine the distribution of the coarser, larger-sized particles, and the hydrometer method is used to determine the distribution of the finer particles.

Significance:

- The distribution of different grain sizes affects the engineering properties of soil.
- Grain size analysis provides the grain size distribution, and it is required in classifying the soil.

Major Soil Groups



Soil-Particle Size Classification

Table 2.3 Particle-Size Classifications

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

Grain Size Distribution

<u>Significance of GSD:</u>

- **To know the relative proportions of different grain sizes.**

  - An important factor influencing the geotechnical characteristics of a **coarse** grain soil.

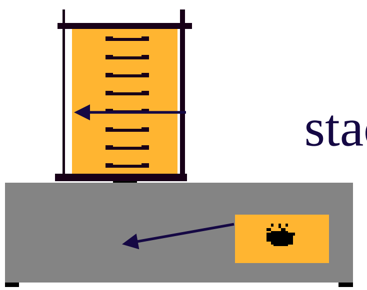
  - Not important in fine grain soils.

Grain Size Distribution

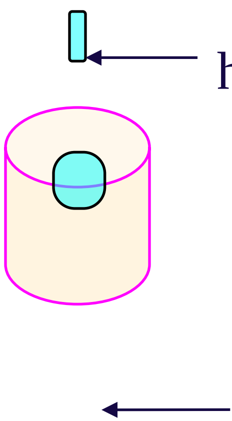
Determination of GSD:

- In coarse grain soils …... By sieve analysis

- fine grain soils …... By hydrometer analysisIn

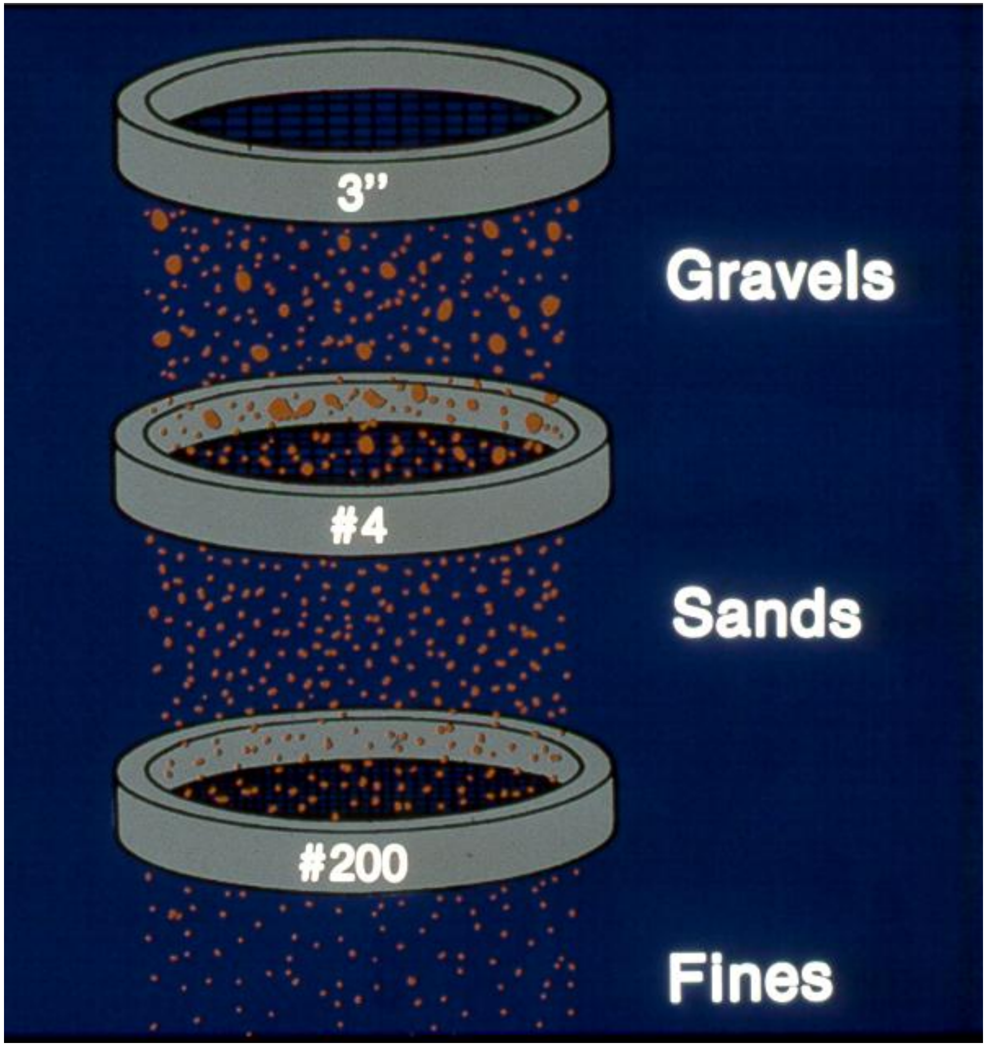


Sieve Analysis



Hydrometer Analysis

Sieve Analyses



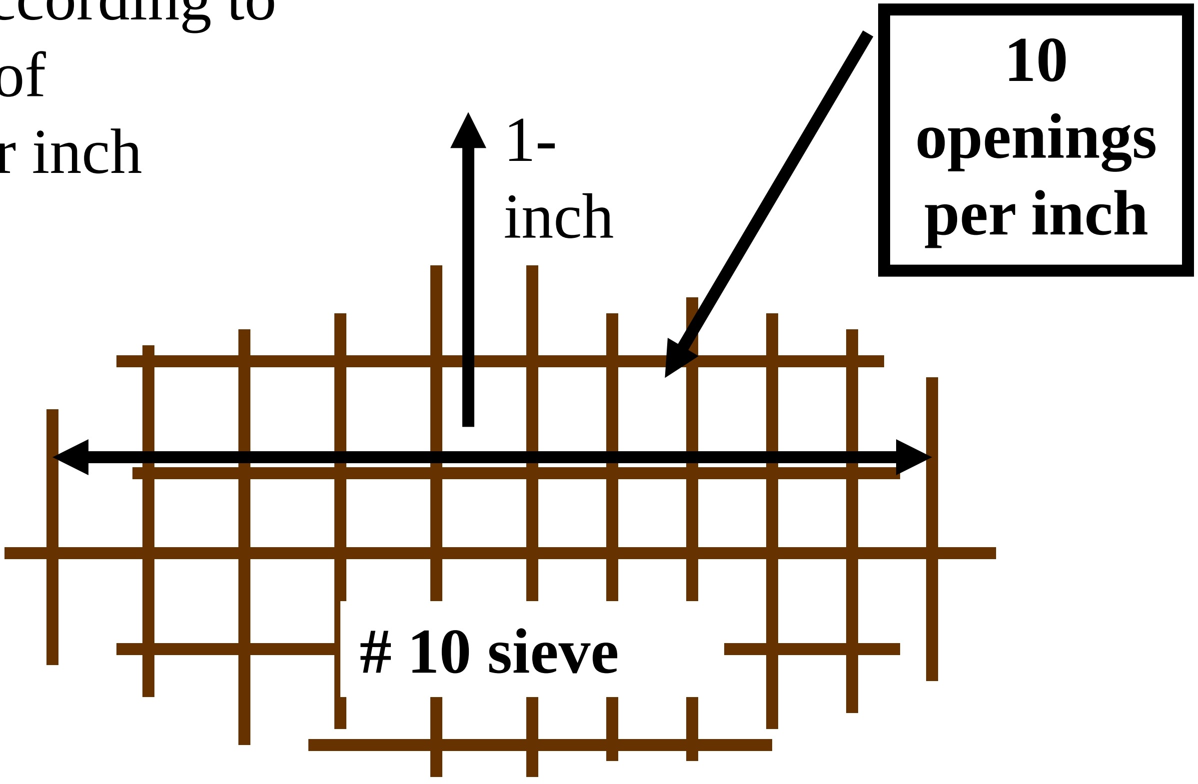
Sieve Analysis

Sieve Designation - Large

Sieves larger than the #4 sieve are designated by the size of the openings in the sieve

Sieve Designation - Smaller

Smaller sieves are numbered according to the number of openings per inch



Sieving procedure

(1) Write down the weight of each sieve as well as the bottom pan to be used in the analysis.

(2) Record the weight of the given dry soil sample.

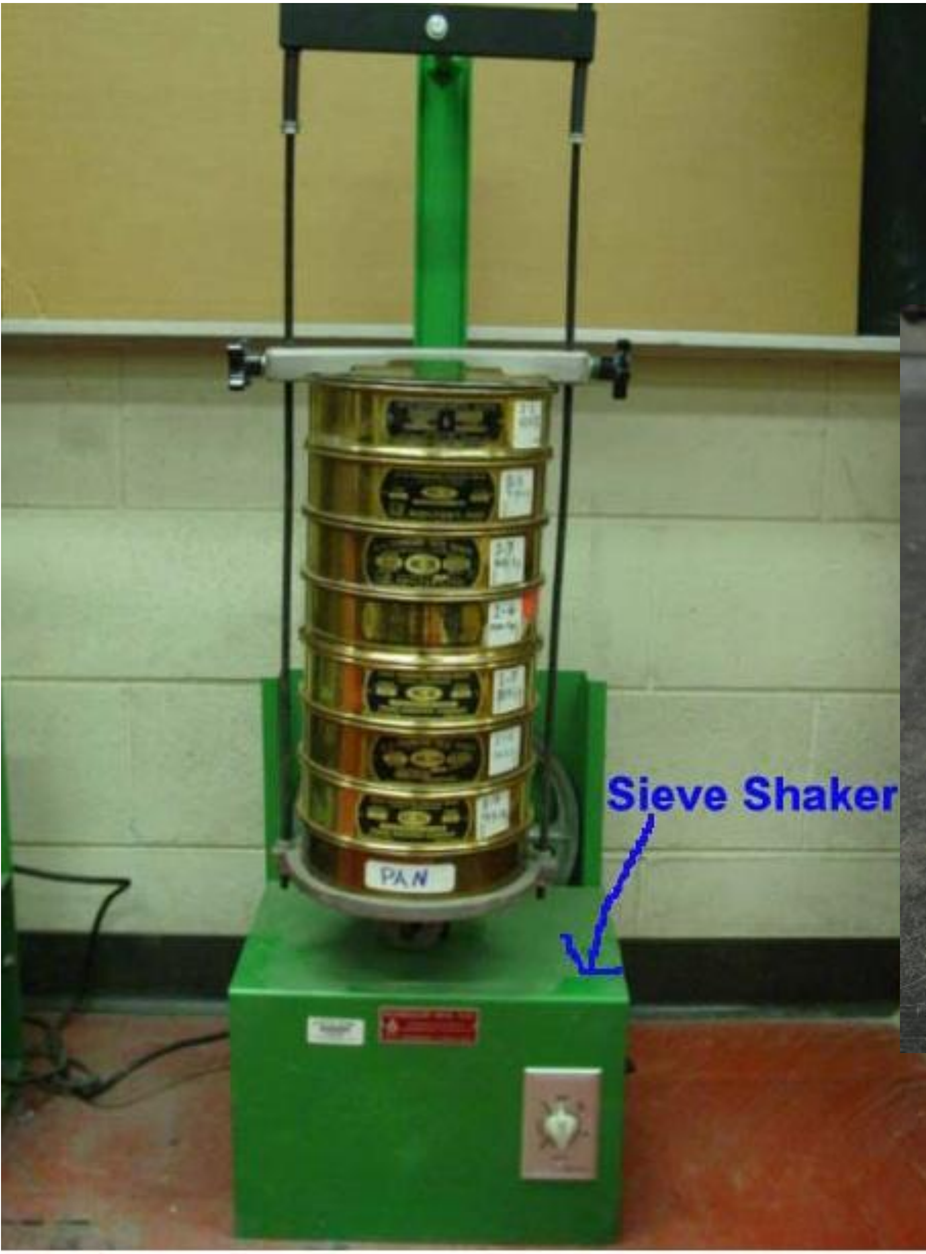
(3) Make sure that all the sieves are clean, and assemble them in the ascending order of sieve numbers (#4 sieve at top and #200 sieve at bottom). Place the pan below #200 sieve. Carefully pour the soil sample into the top sieve and place the cap over it.

(4) Place the sieve stack in the mechanical shaker and shake for 10 minutes.

(5) Remove the stack from the shaker and carefully weigh and record the weight of each sieve with its retained soil. In addition, remember to weigh and record the weight of the bottom pan with its retained fine soil.


Set of Sieves
Soil Specimen
Balance
PAN


PAN


PAN
Sieve Shaker


ELECTRONIC SOLUTION BALANCE U-1027
0.824
lb
lb/oz
kg
MODE
OFF
SAMPLE
SET
ON
PCS

Data Analysis:

(1) Obtain the mass of soil retained on each sieve by subtracting the weight of the empty sieve from the mass of the sieve + retained soil, and record this mass as the weight retained on the data sheet. The sum of these retained masses should be approximately equals the initial mass of the soil sample. A loss of more than two percent is unsatisfactory.

(2) Calculate the percent retained on each sieve by dividing the weight retained on each sieve by the original sample mass.

(3) Calculate the percent passing (or percent finer) by starting with 100 percent and subtracting the percent retained on each sieve as a cumulative procedure.

Sieve Sizes

▼ **TABLE 1.5** U.S. Standard Sieve Sizes

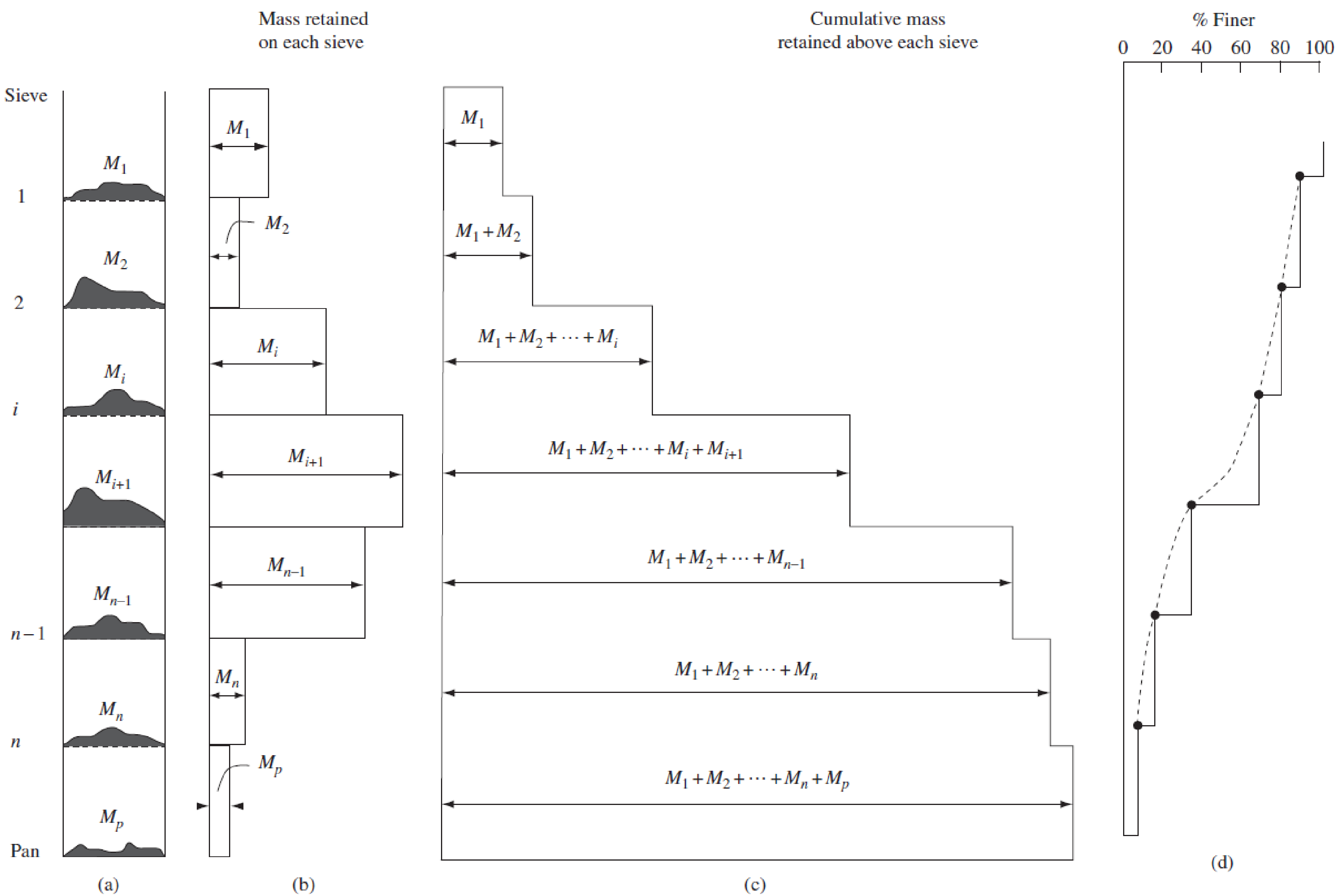
Sieve no.	Opening (mm)
4	4.75
5	4.00
6	3.35
7	2.80
8	2.36
10	2.00
12	1.70
14	1.40
16	1.18
18	1.00
20	0.850
25	0.710
30	0.600
35	0.500
40	0.425
50	0.355
60	0.250
70	0.212
80	0.180
100	0.150
120	0.125
140	0.106
170	0.090
200	0.075
270	0.053

(Das, 1998)

Table 4.5(a). METRIC SIEVES (BS)

Construction	*Aperture size: Full Set (A)*	*'Standard' set (B)*	*'Short' set C)*
Perforated steel place (square hole)	75 mm	+	
	63	+	+
	50		
	37.5	+	
	28		
	20	+	+
	14		
	10	+	
	6.3	+	+
	5		
	3.35	+	
	2	+	+
	1.18	+	
	600 μm	+	+
	425		
	300	+	
	212		+
	150	+	
	63	+	+
Lid and receiver	+	+	+
	19 sieves	13 sieves	7 sieves

(Head, 1992)


Mass retained on each sieve
Cumulative mass retained above each sieve
% Finer
0 20 40 60 80 100
Sieve
1
2
i
$n-1$
n
Pan
M_1
M_2
M_i
M_{i+1}
M_{n-1}
M_n
M_p
$M_1 + M_2$
$M_1 + M_2 + \cdots + M_i$
$M_1 + M_2 + \cdots + M_i + M_{i+1}$
$M_1 + M_2 + \cdots + M_{n-1}$
$M_1 + M_2 + \cdots + M_n$
$M_1 + M_2 + \cdots + M_n + M_p$
(a)
(b)
(c)
(d)

Sieve Number	Diameter (mm)	Mass of Empty Sieve (g)	Mass of Sieve+Soil Retained (g)	Soil Retained (g)	Percent Retained	Percent Passing
4	4.75	116.23	166.13	49.9	9.5	90.5
10	2.0	99.27	135.77	36.5	7.0	83.5
20	0.84	97.58	139.68	42.1	8.0	75.5
40	0.425	98.96	138.96	40.0	7.6	67.8
60	0.25	91.46	114.46	23.0	4.4	63.4
140	0.106	93.15	184.15	91.0	17.4	46.1
200	0.075	90.92	101.12	10.2	1.9	44.1
Pan	---	70.19	301.19	231.0	44.1	0.0
			Total Weight=	523.7		

For example: Total mass = 500 g,

Mass retained on No. 4 sieve = 9.7 g

For the No.4 sieve:

Quantity passing = Total mass - Mass retained

= 500 - 9.7 = 490.3 g

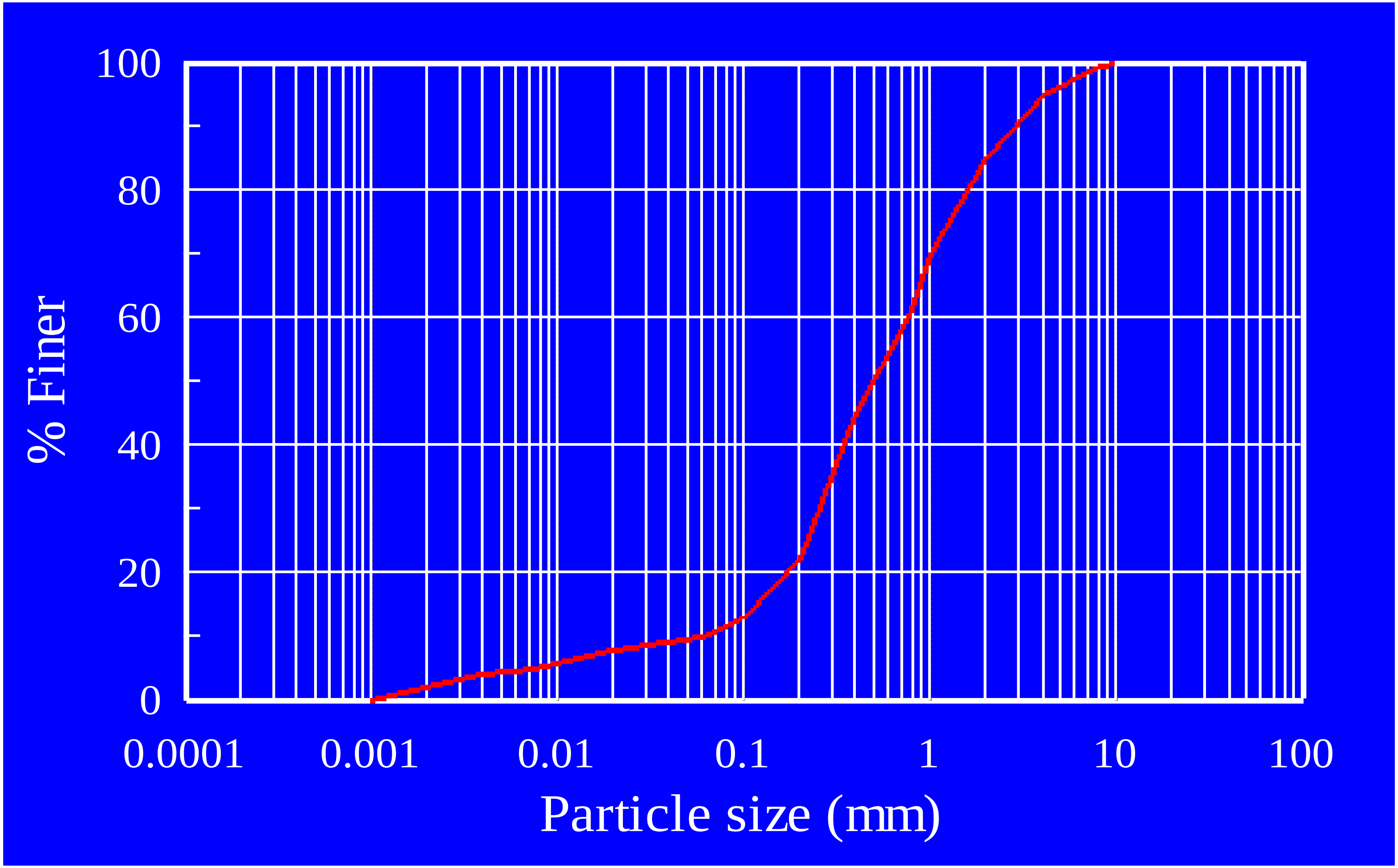
The percent retained is calculated as;

% retained = Mass retained/Total mass

= (9.7/500) X 100 = 1.9 %

From this, the % passing = 100 - 1.9 = 98.1 %

Grain size distribution



Unified Soil Classification

- Each soil is given a 2 letter classification (e.g. SW). The following procedure is used.
  - Coarse grained (>50% larger than 75 mm)
    - Prefix S if > 50% of coarse is Sand
    - Prefix G if > 50% of coarse is Gravel
    - Suffix depends on %fines
    - if %fines < 5% suffix is either W or P
    - if %fines > 12% suffix is either M or C
    - if 5% < %fines < 12% Dual symbols are used

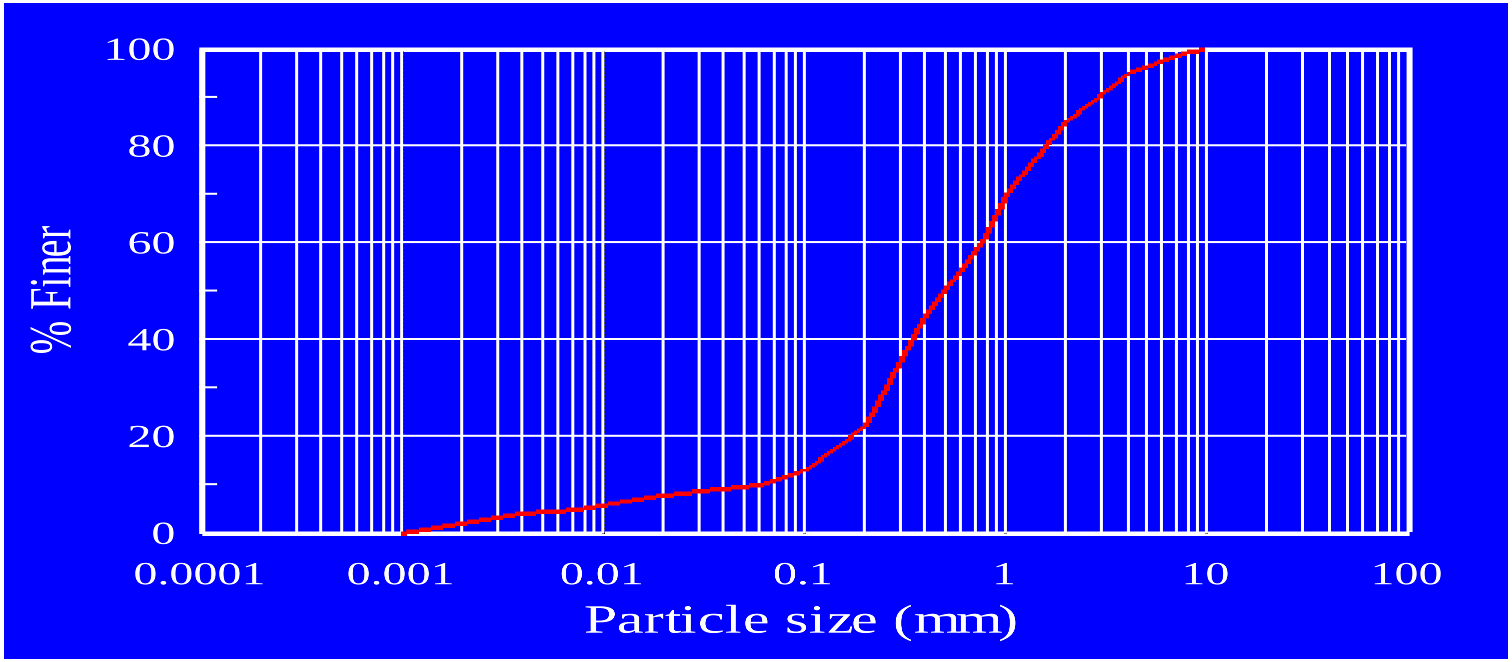
Unified Soil Classification

To determine W or P, calculate C_u and C_c

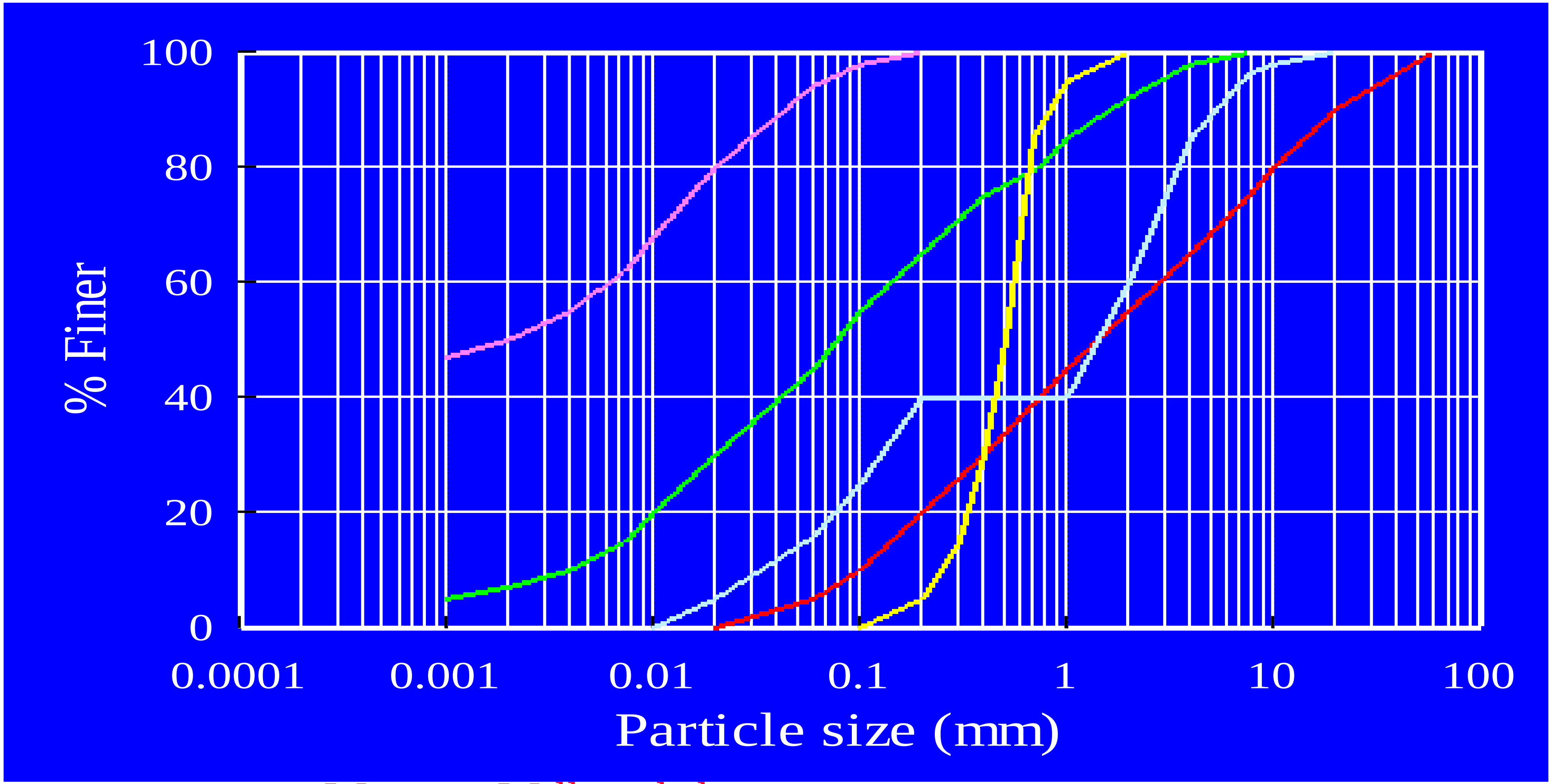
$$C_u = \frac{D_{60}}{D_{10}}$$

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

x% of the soil has particles smaller than D_x



Grading curves



W	Well graded
U	Uniform
P	Poorly graded
C	Well graded with some clay
F	Well graded with an excess of fines

Procedure for grain size determination

- Sieving - used for particles > 75 μm
- Hydrometer test - used for smaller particles
  - Analysis based on Stoke's Law, velocity proportional to diameter

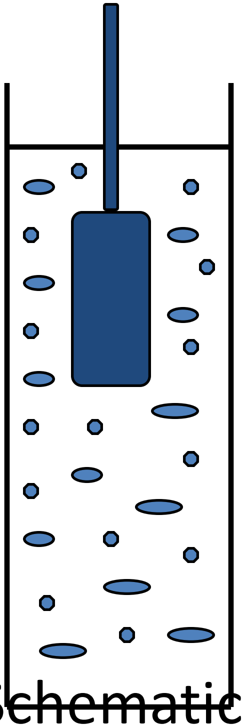


Figure 1 Schematic diagram of hydrometer test

Procedure for grain size determination

- Sieving - used for particles > 75 μm

- Hydrometer test - used for smaller particles
  - Analysis based on Stoke's Law, velocity proportional to diameter

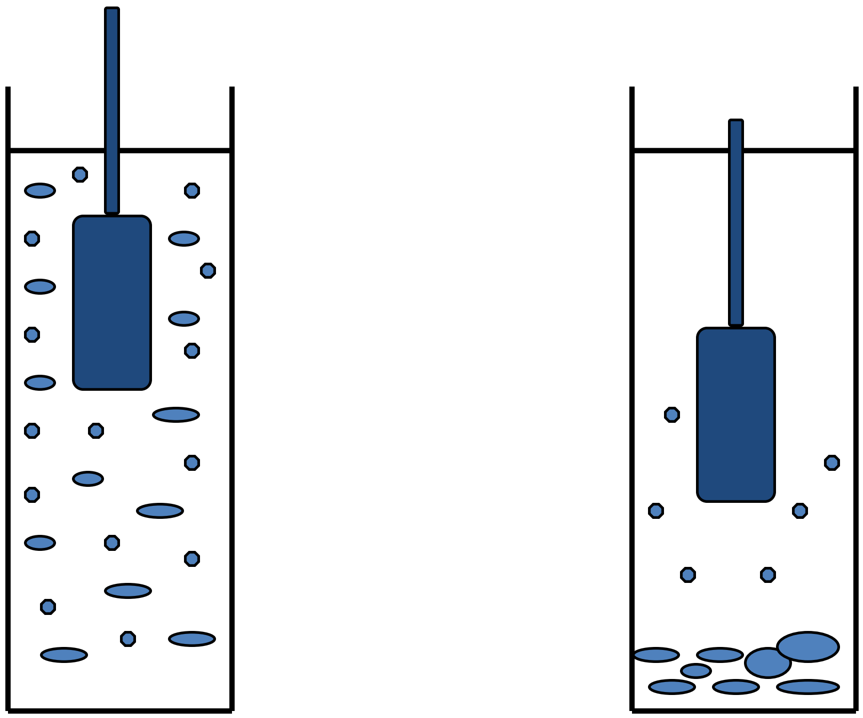
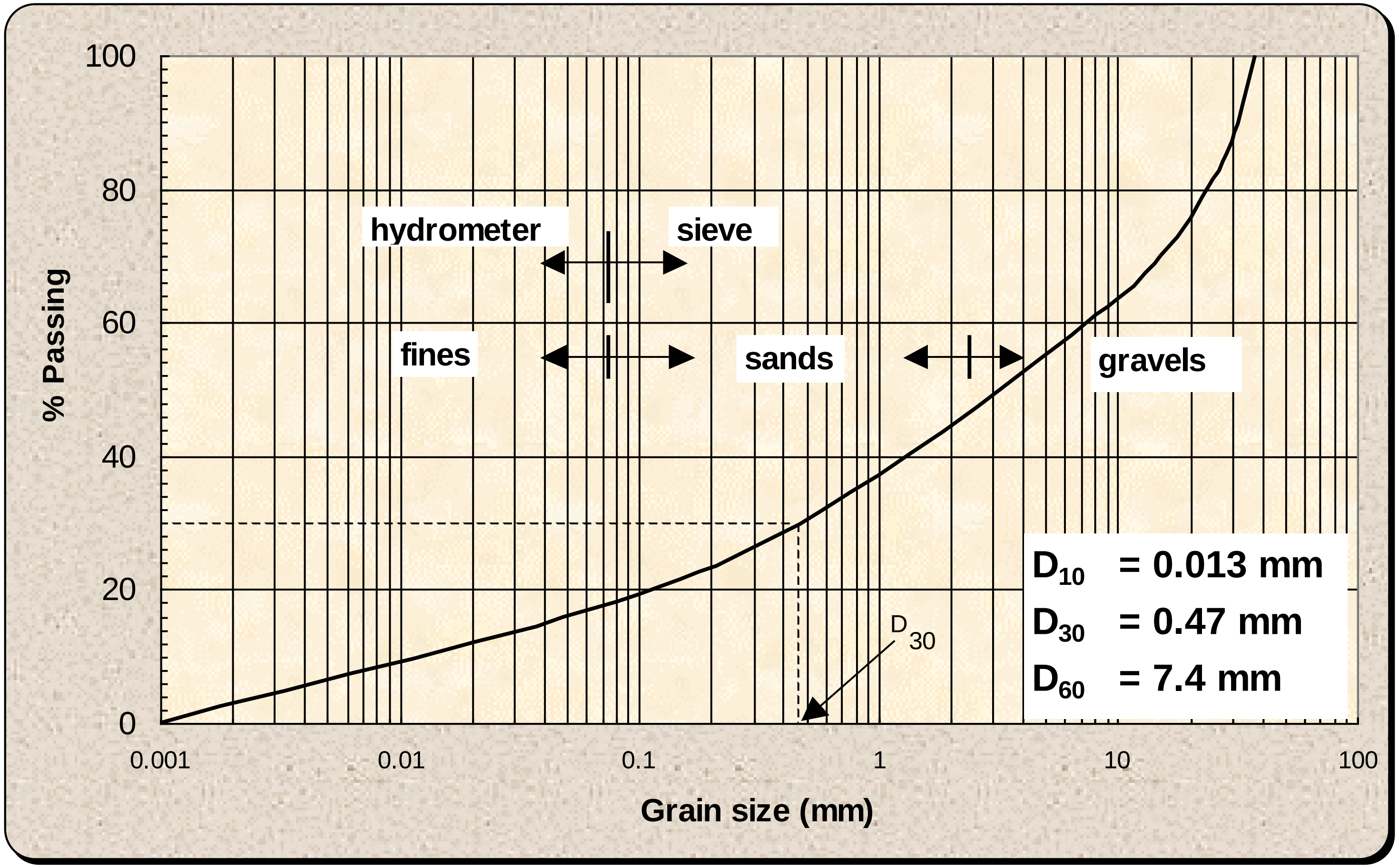


Figure 1 Schematic diagram of hydrometer test



Grain Size Distribution Curve

- can find % of gravels, sands, fines

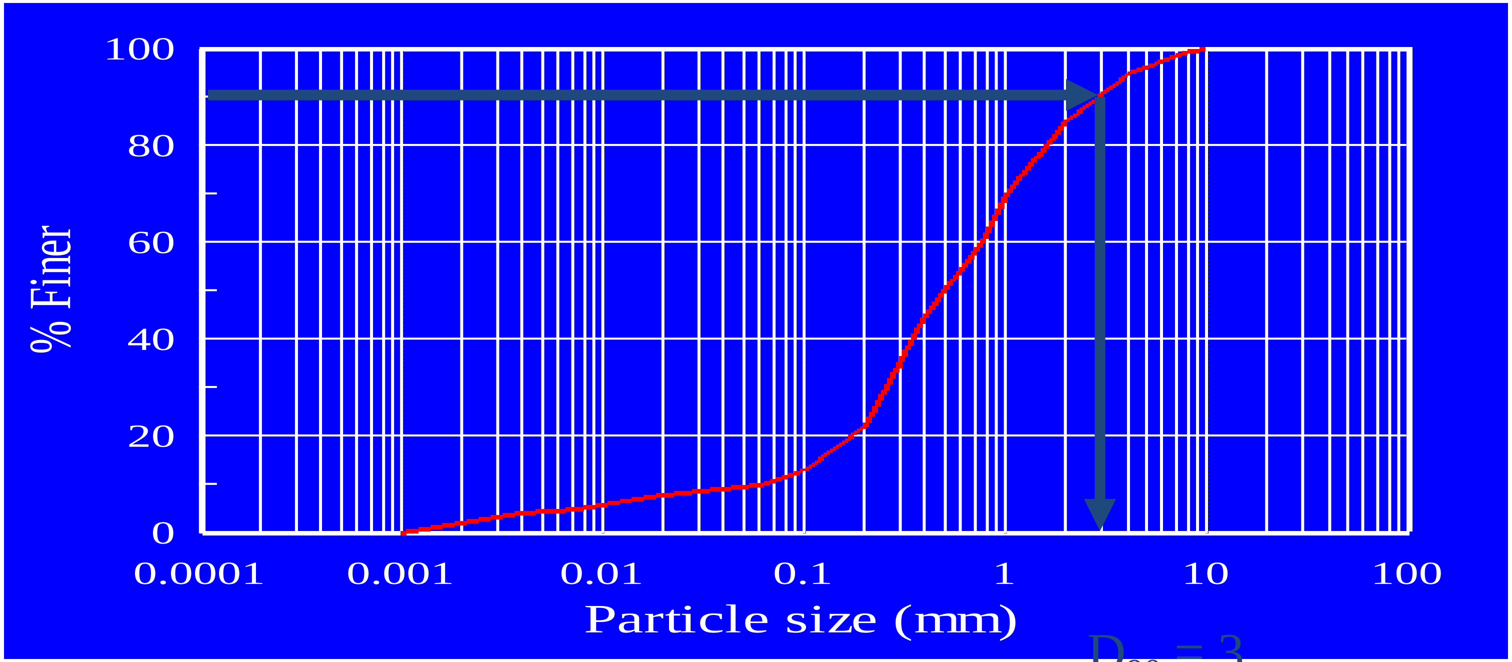
- define D_{10}, D_{30}, D_{60}.. as above.

To determine W or P, calculate C_u and C_c

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

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

x% of the soil has particles smaller than D_x



D_{90} = 3 mm

Well or Poorly Graded Soils

Well Graded Soils

Wide range of grain sizes present

Gravels: C_c = 1-3 & C_u >4

Sands: C_c = 1-3 & C_u >6

Poorly Graded Soils

Others, including two special cases:

(a) Uniform soils – grains of same size

(b) Gap graded soils – no grains in a specific size range

Grain Size Distribution (Cont.)

- **Describe the shape**

Example: well graded

$$D_{10} = 0.02\,\text{mm}\,(\text{effective size})$$
$$D_{30} = 0.6\,\text{mm}$$
$$D_{60} = 9\,\text{mm}$$

Coefficient of uniformity

$$C_u = \frac{D_{60}}{D_{10}} = \frac{9}{0.02} = 450$$

Coefficient of curvature

$$C_c = \frac{(D_{30})^2}{(D_{10})(D_{60})} = \frac{(0.6)^2}{(0.02)(9)} = 2$$

- **Criteria**

Well – graded soil

$$1 < C_c < 3 \text{ and } C_u \geq 4$$
$$(\text{for gravels})$$
$$1 < C_c < 3 \text{ and } C_u \geq 6$$
$$(\text{for sands})$$

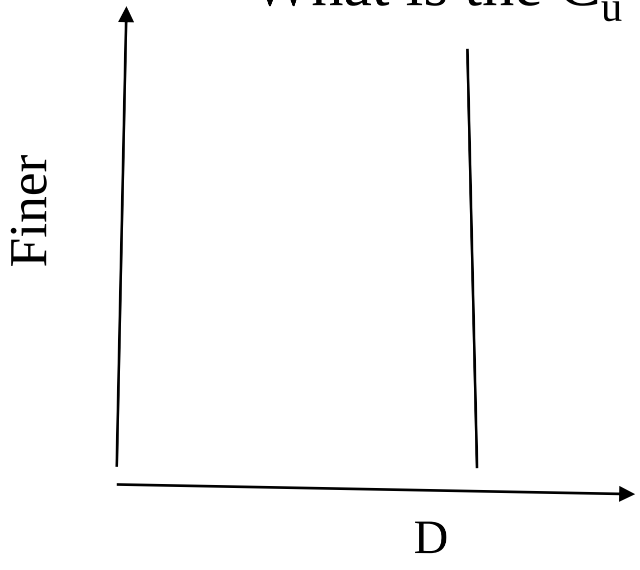
- **Question**

What is the C_u for a soil with only one grain size?

Answer

- Question

What is the C_u for a soil with only one grain size?



Grain size distribution

Coefficient of uniformity

$$C_u = \frac{D_{60}}{D_{10}} = 1$$

SAMPLE PROBLEM

1. For a soil with D_{60} =0.42mm, and D_{30} =0.21mm, and D_{10} =0.16,   calculate and the coefficient of gradation.

Sol'n;

$C_U = D_{60} / D_{10} = 0.42mm/0.16mm = 2.625$

$C_C = (D_{30})^2 / (D_{10})(D_{60}) = (0.21)^2 / (0.16)(0.42) = 0.66$

2. The following are the results of a sieve analysis:

IS Sieve (mm) (1)	Mass of Soil Retained on Each Sieve (g) (2)	% Retained (3)	Cumulative % retained (4)	% Finer = 100- (4)
20	35	3.89	3.89	100
10	40	4.44	8.33	
4.75	80	8.89		
2.0	150			
1.0	150			
0.6	140			
425 m	115			
212 m	55			
150 m	35			
75 m	25			
PAN	75			

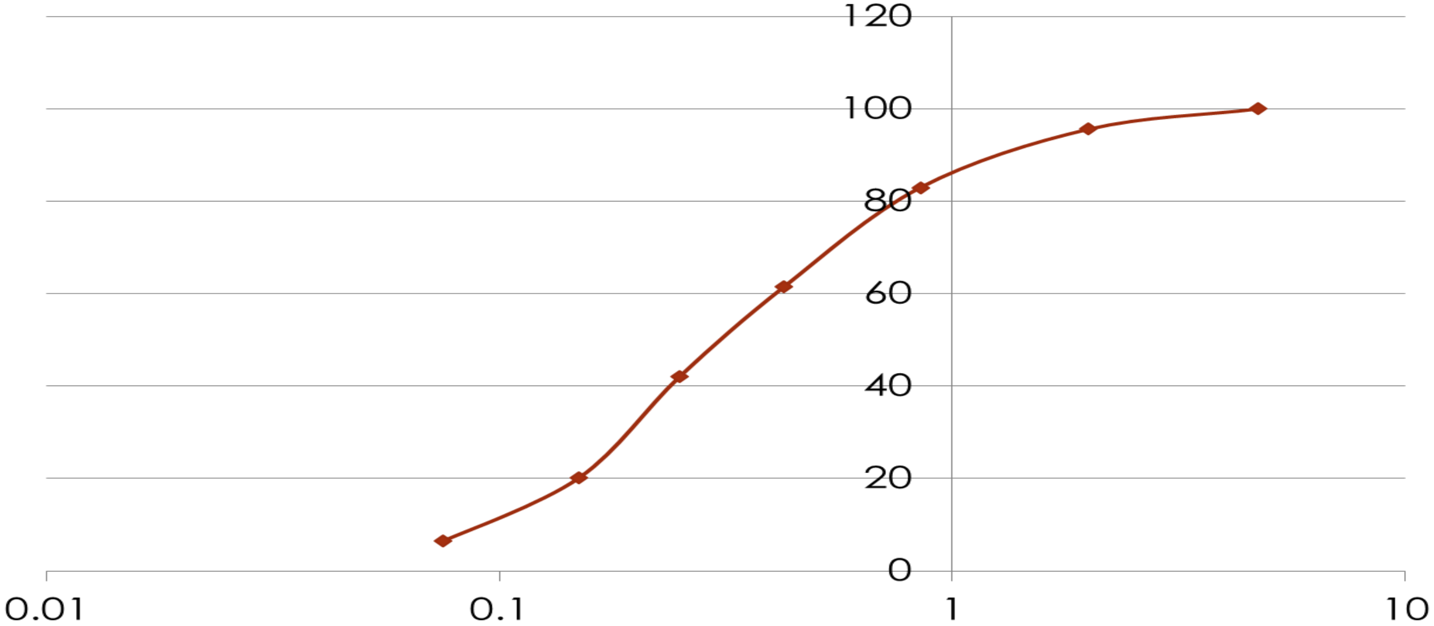
a.) Determine the percent finer than each sieve size and plot a grain-size distribution curve.

b.) Determine D_{10} , D_{30} and D_{60} from the grain-size distribution curve.

c.) Calculate the uniformity coefficient, C_u.

d.) Calculate the coefficient of graduation, C_c.

Grain-size distribution curve



Solution

c.) $C_u = D_{60}/D_{10} = 0.4/0.12 = 3.33$

d.) $C_C = (D)^2/(D_{60})(D_{10}) = 1.01$

3.The particle-size characteristics of a soil are given in the table.

Sieve no.	Opening	% Passing
4	4.75	100
10	2.00	90
20	0.850	64
40	0.425	38
80	0.180	18
200	0.075	13

Calculate the uniformity coefficient (C_u) and coefficient of gradation (C_c).

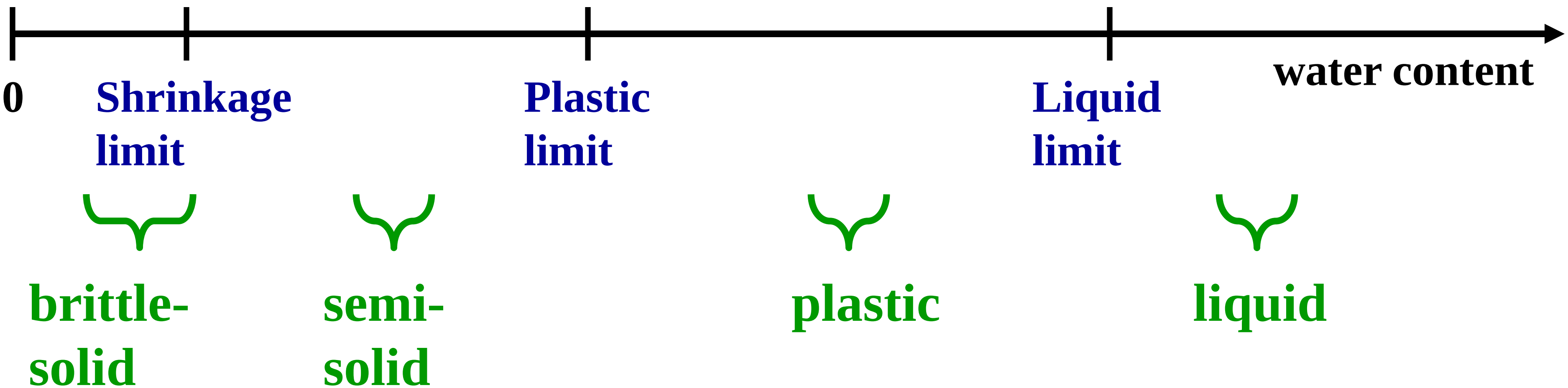
Sol'n:

$$C_u = D_{60} / D_{10} = 0.73/0.019 = 38.421$$

$$C_c = (0.29)^2 / (0.73)(.019) = 0.063$$

Atterberg Limits

- Border line **water contents**, separating the different **states** of a fine grained soil



Purpose:

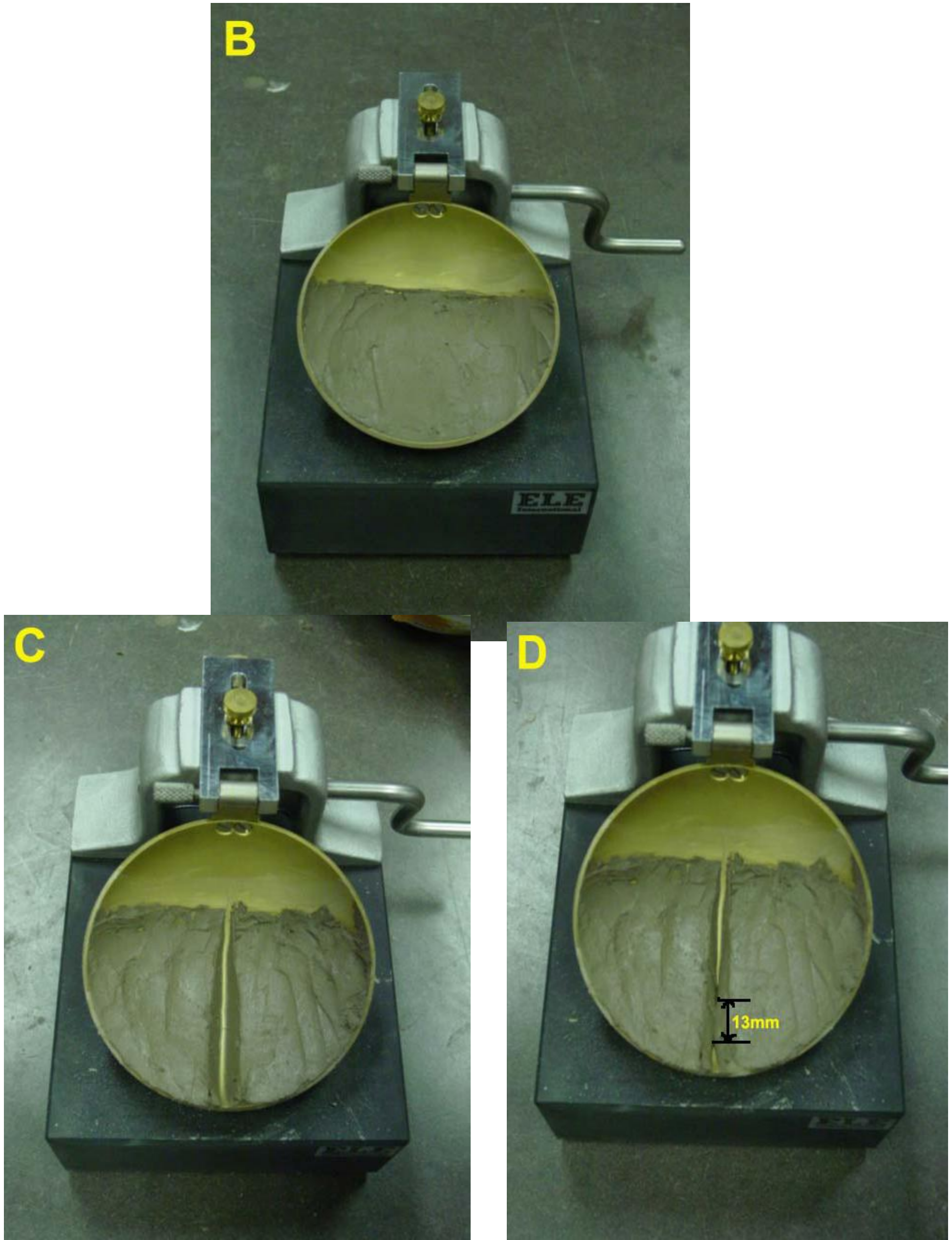
This lab is performed to determine the plastic and liquid limits of a fine grained soil. The Atterberg's limits are based on the moisture content of the soil. Defined by Laboratory Test concept developed by Atterberg in 1911.

The plastic limit: is the moisture content that defines where the soil changes from a semi-solid to a plastic (flexible) state.

The liquid limit: is the moisture content that defines where the soil changes from a plastic to a viscous fluid state.


A
Spatula
Liquid limit device
Grooving
tool
Soil specimen
Moisture cans

- **Defined by Laboratory Test concept developed by Atterberg in 1911.**



The liquid limit (LL) is arbitrarily defined as the water content, in percent, at which a pat of soil in a standard cup and cut by a groove of standard dimensions will flow together at the base of the groove for a distance of 12 mm under the impact of 25 blows in the devise.

The cup being dropped 10 mm in a standard liquid limit apparatus operated at a rate of two shocks per second.

Atterberg Limits

Liquid Limit (w_L or LL):

Clay flows like liquid when $w > LL$

Plastic Limit (w_P or PL):

Lowest water content where the clay is still plastic

Shrinkage Limit (w_S or SL):

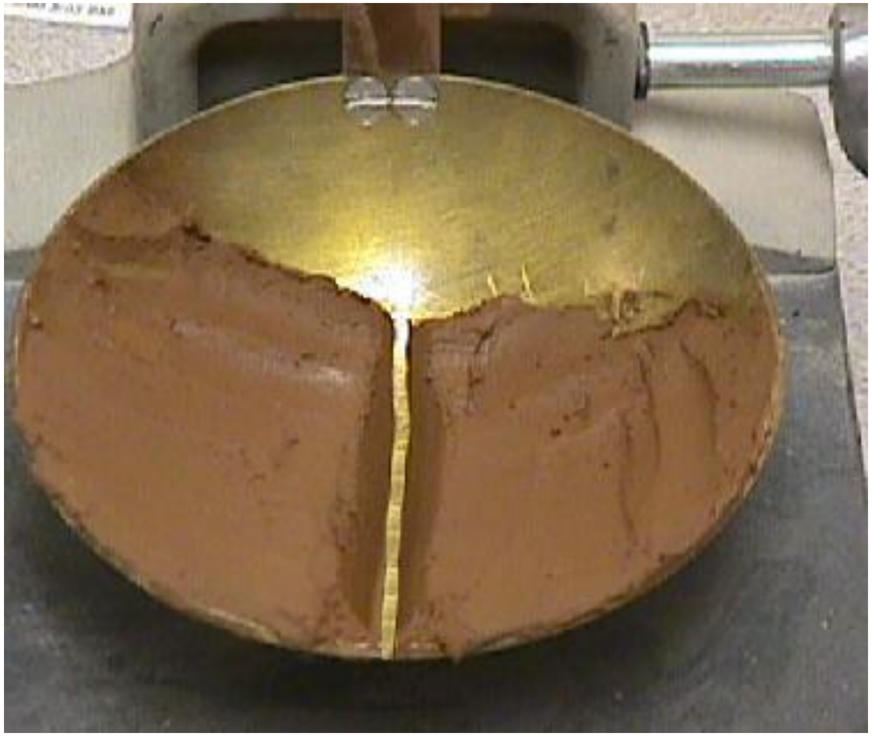
At $w<SL$, no volume reduction on drying

LL Test Procedure

- Prepare paste of soil finer than 425 micron sieve
- Place Soil in Cup

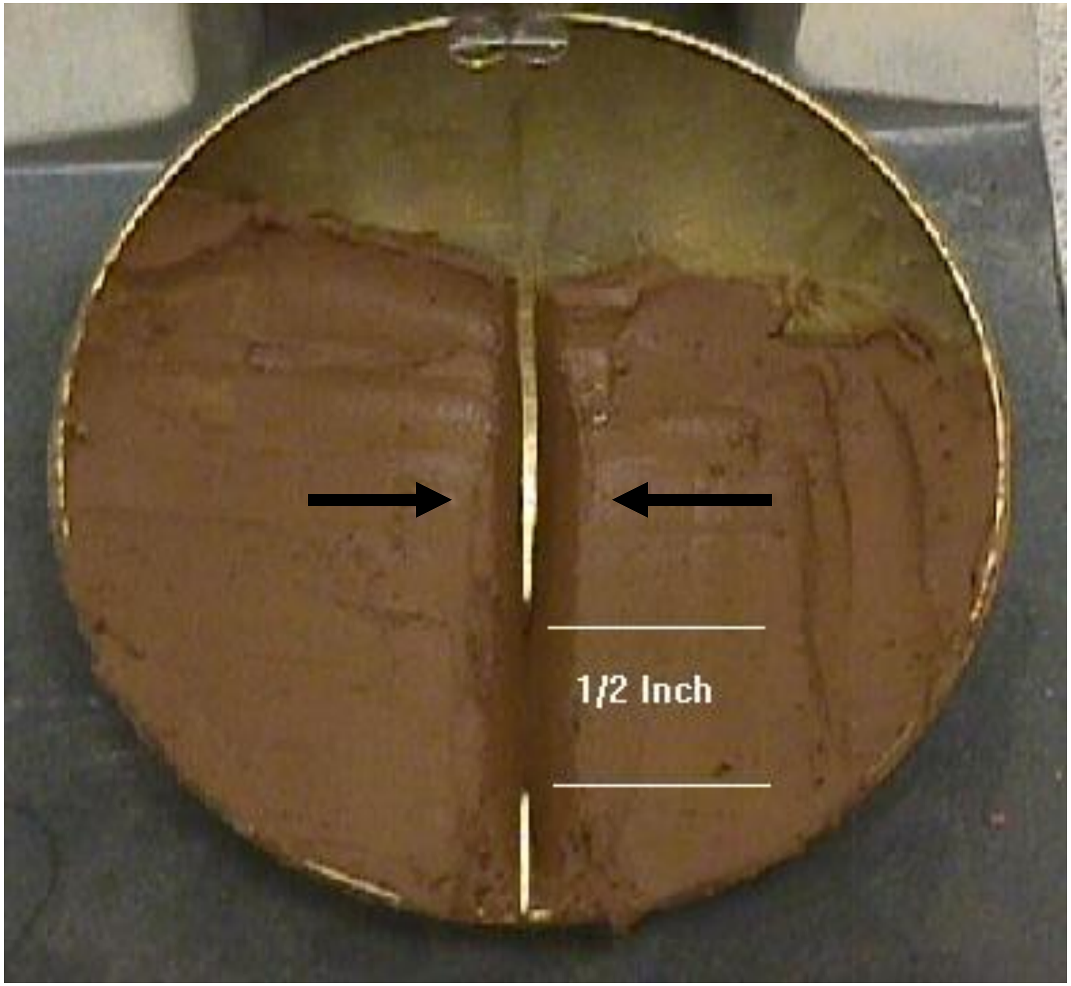
LL Test Procedure

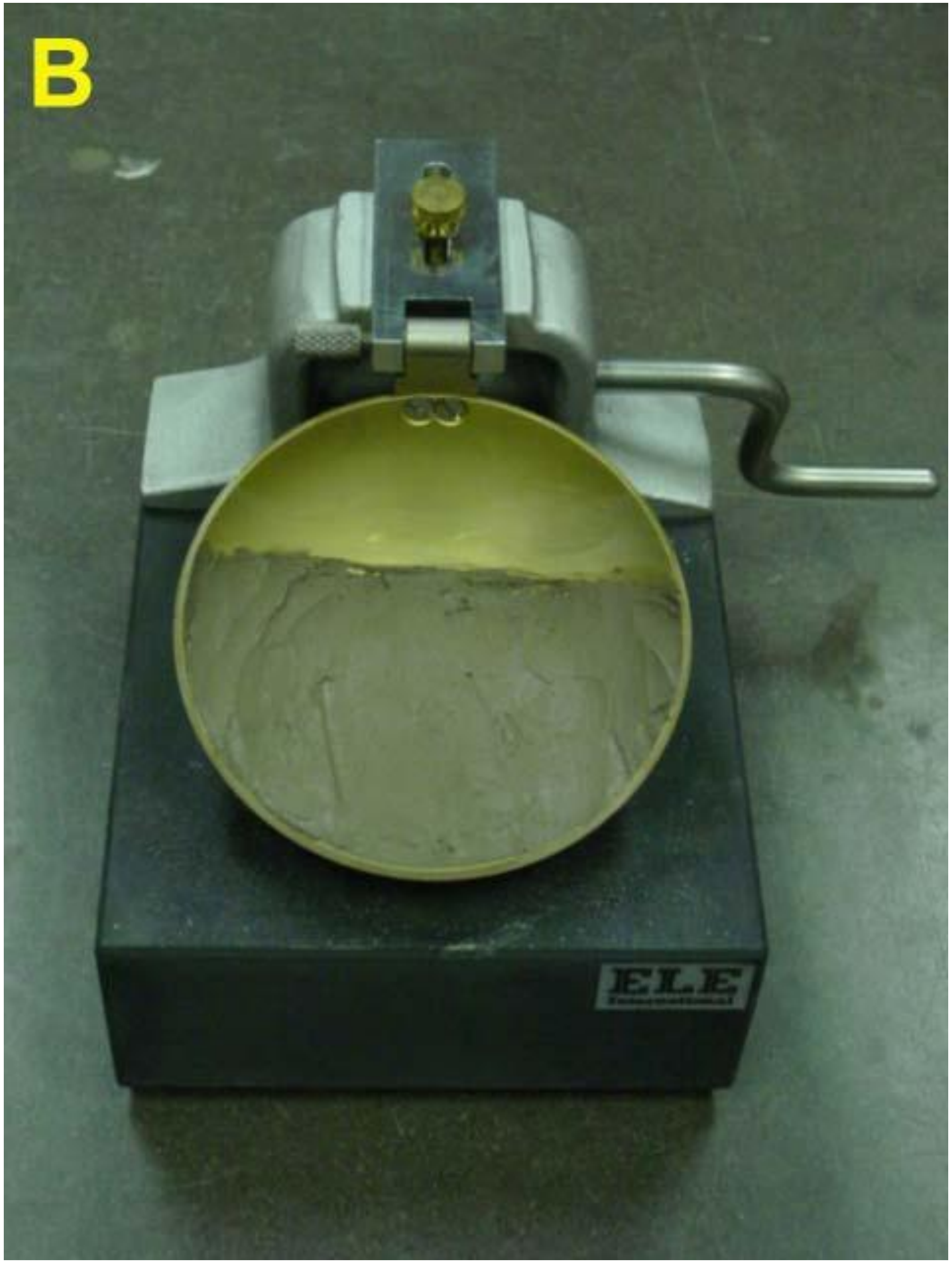
- Cut groove in soil paste with standard grooving tool



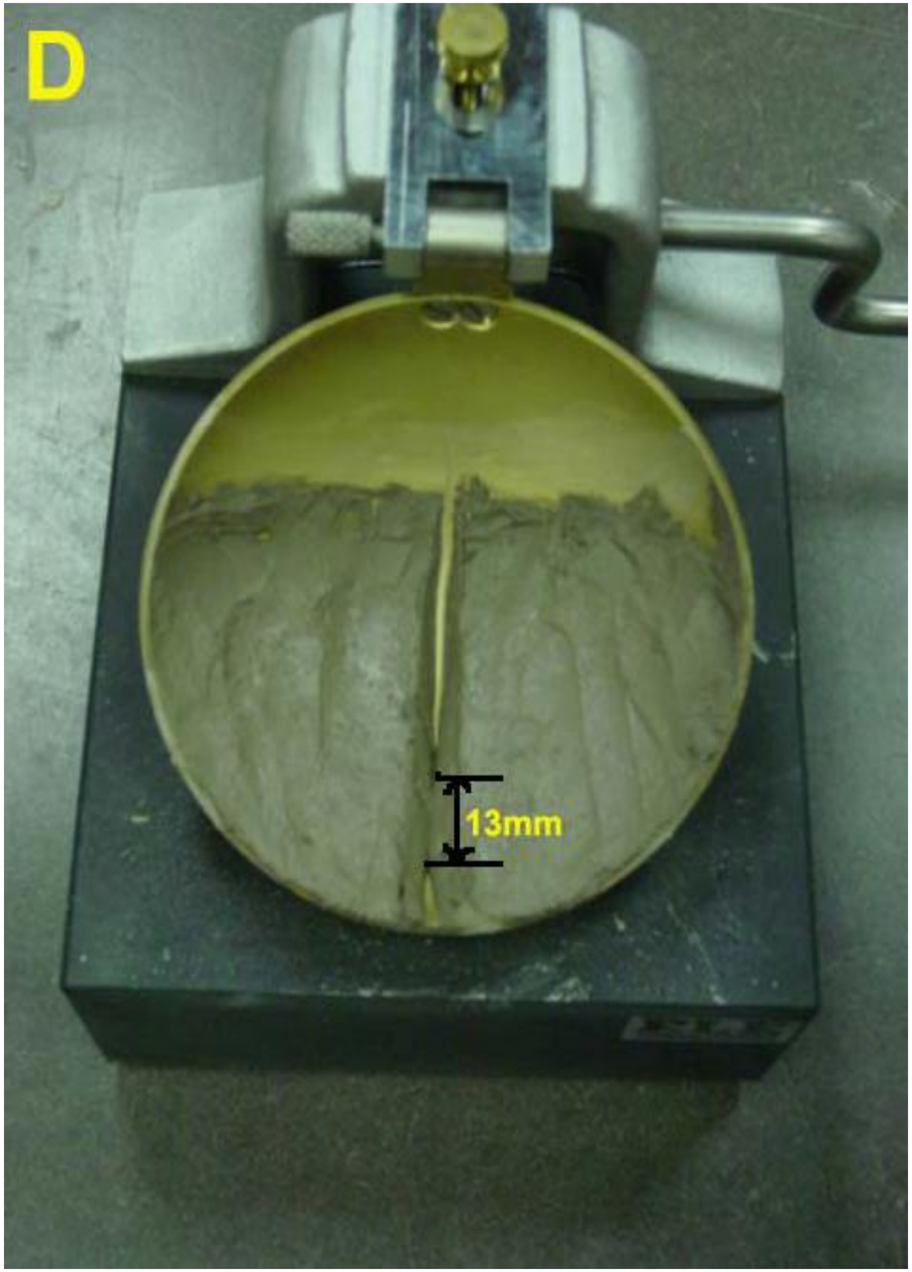
LL Test Procedure

- Rotate cam and count number of blows of cup required to close groove by 1/2"




B
ELE
International

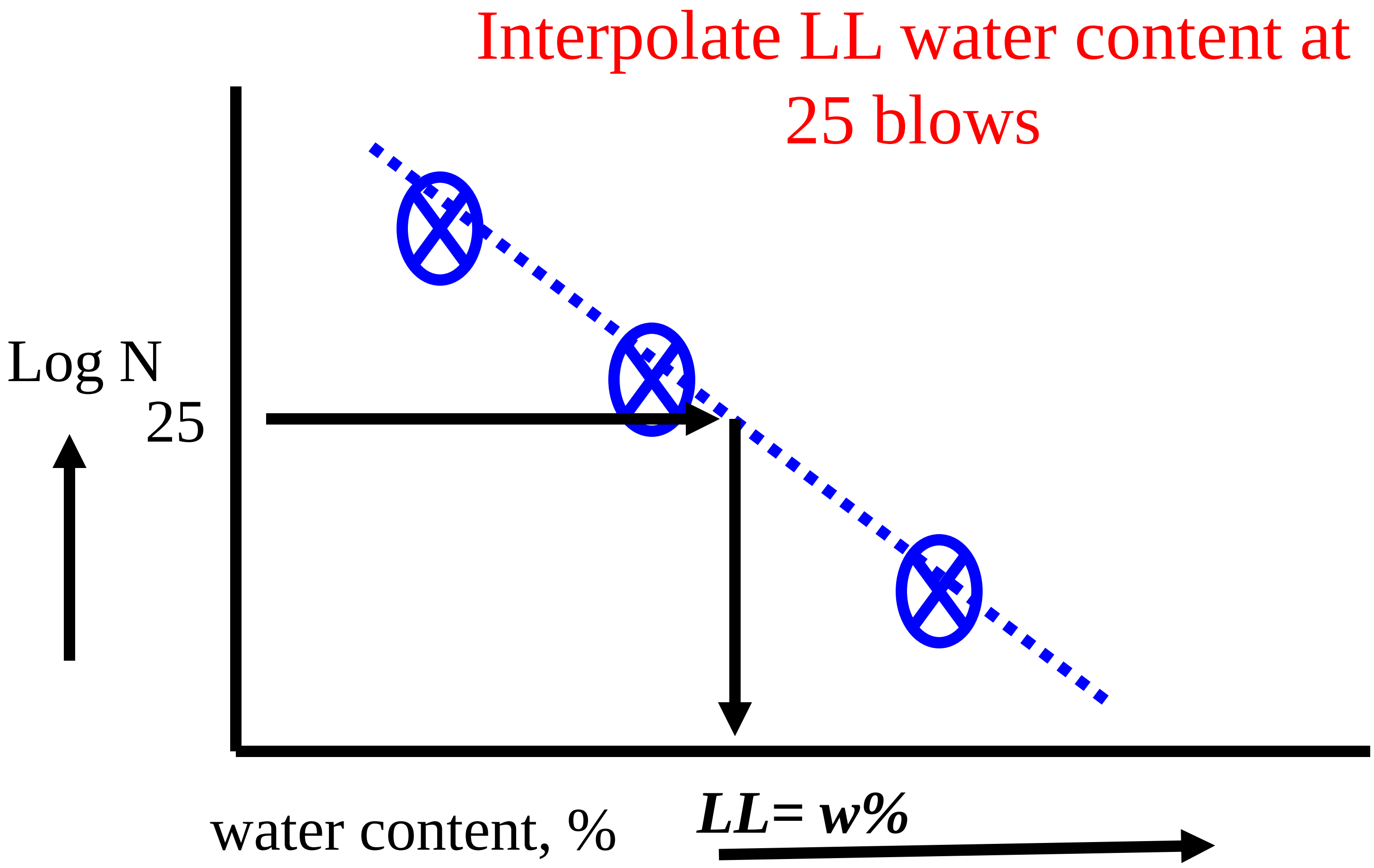

C


D
13mm

LL Test Procedure

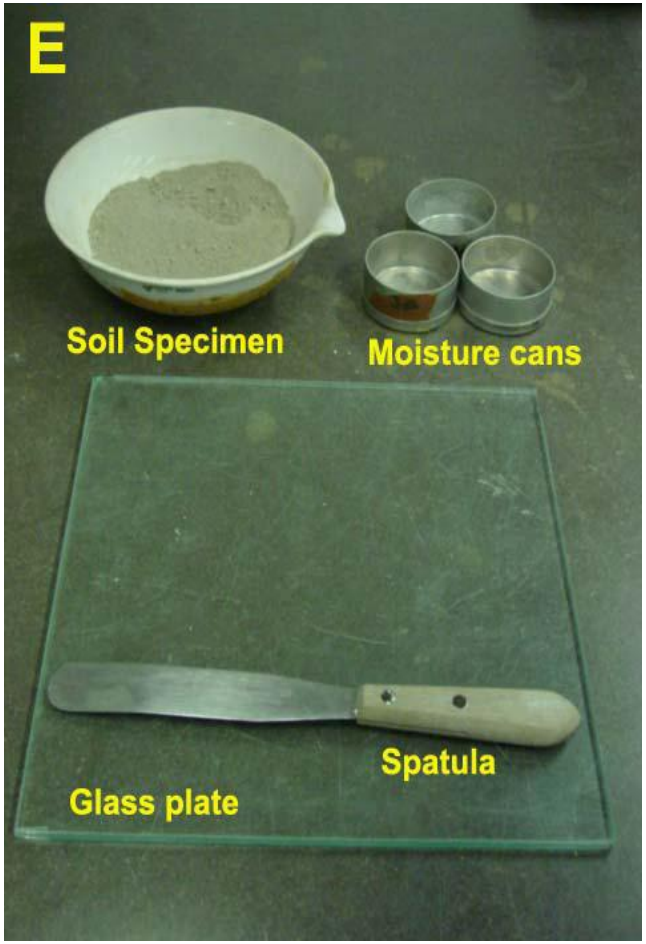
- Perform on 3 to 4 specimens that bracket 25 blows to close groove
- Obtain water content for each test
- Plot water content versus number of blows on semi-log paper

LL Test Results



Plastic Limit

The minimum water content at which a soil will just begin to crumble when it is rolled into a thread of approximately 3 mm in diameter.



Plastic Limit w% procedure

- Using paste from LL test, begin drying
- May add dry soil or spread on plate and air-dry

Plastic Limit w% procedure

When point is reached where thread is cracking and cannot be re-rolled to 3 mm diameter, collect at least 6 grams and measure water content. Defined plastic limit


F
Ellipsoidal soil mass

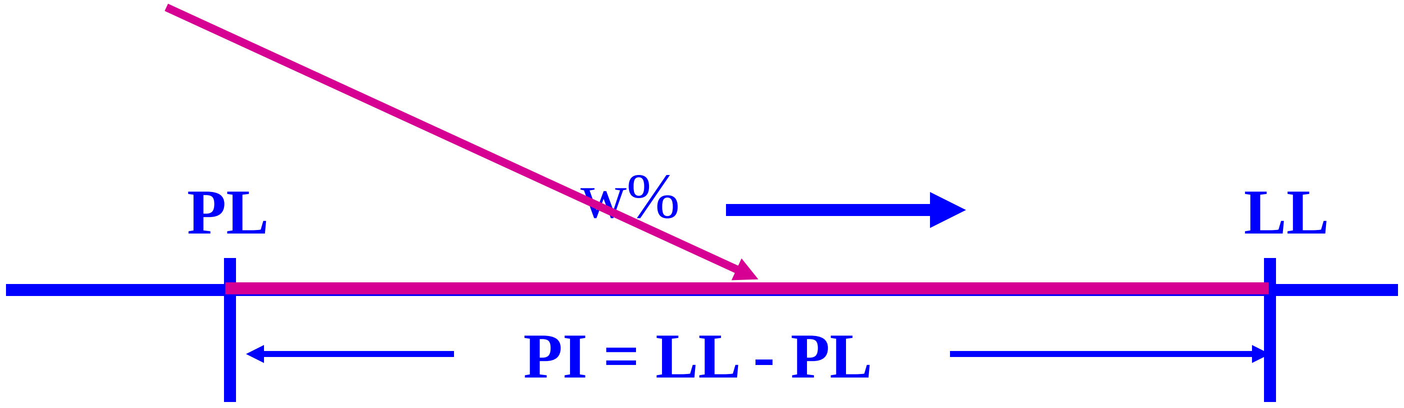

G

1. Calculate the water content of each of the plastic limit moisture cans after they have been in the oven for at least 16 hours.
2. Compute the average of the water contents to determine the plastic limit, PL.

Definition of Plasticity Index

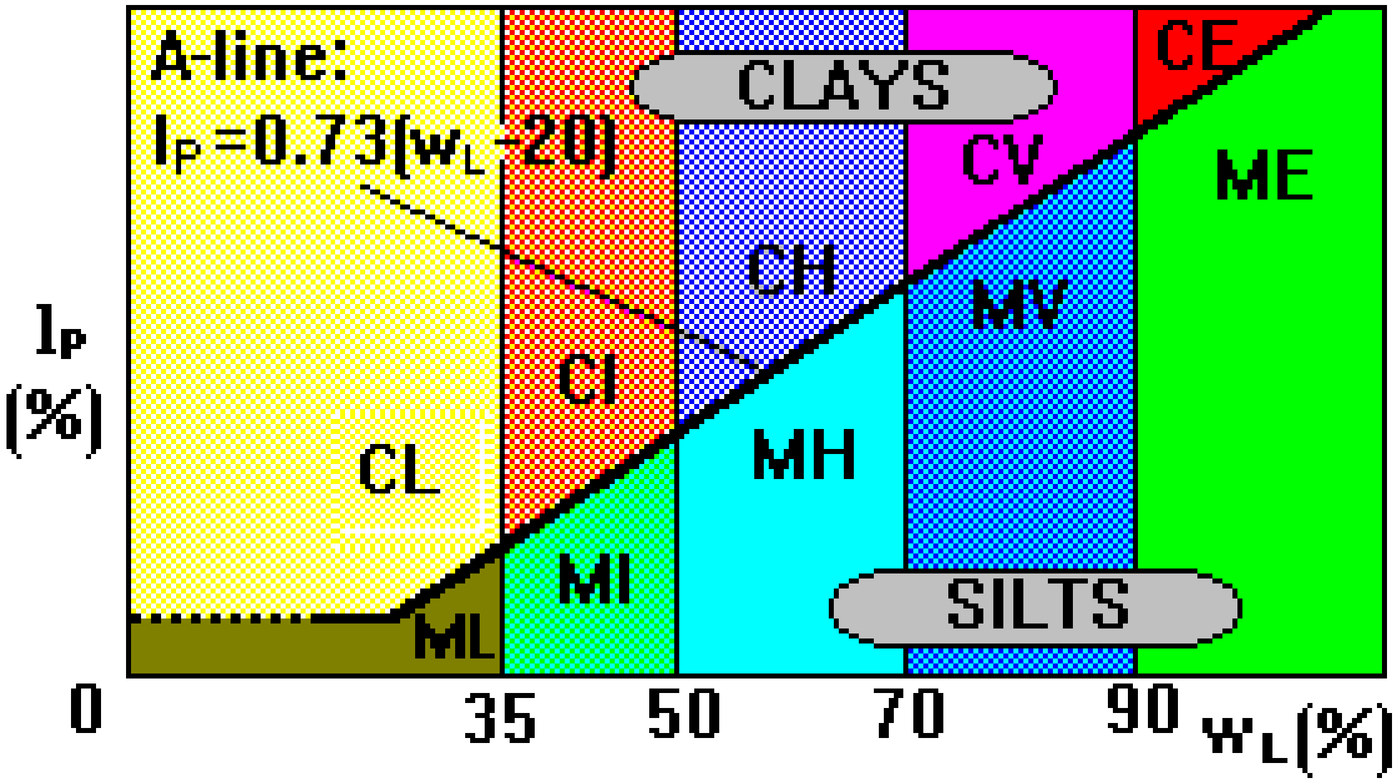
- Plasticity Index is the numerical difference between the Liquid Limit w% and the Plastic Limit w%

Plasticity Index = Liquid Limit – Plastic Limit



plastic (remoldable)

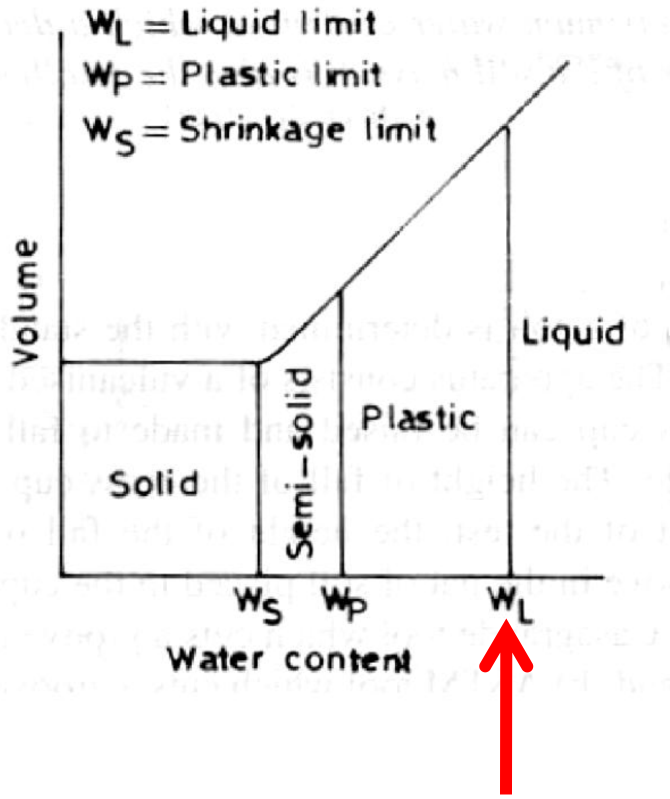
Plasticity Chart



Low plasticity	w_L = < 35%
Intermediate plasticity	w_L = 35 - 50%
High plasticity	w_L = 50 - 70%
Very high plasticity	w_L = 70 - 90%
Extremely high plasticity	w_L = > 90%

C. Liquid Limit

- *Soil is practically a liquid*
- *Shows minimal shear strength*
- *Defined as the moisture content required to close a distance of 0.5 inch along the bottom of a groove after 25 blows of the liquid limit device.*

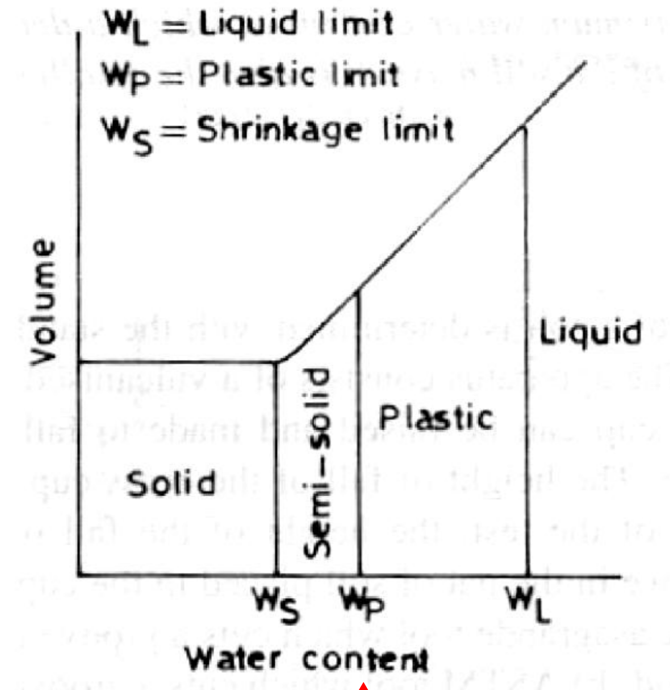


animation

C. Liquid Limit

D. Plastic Limit

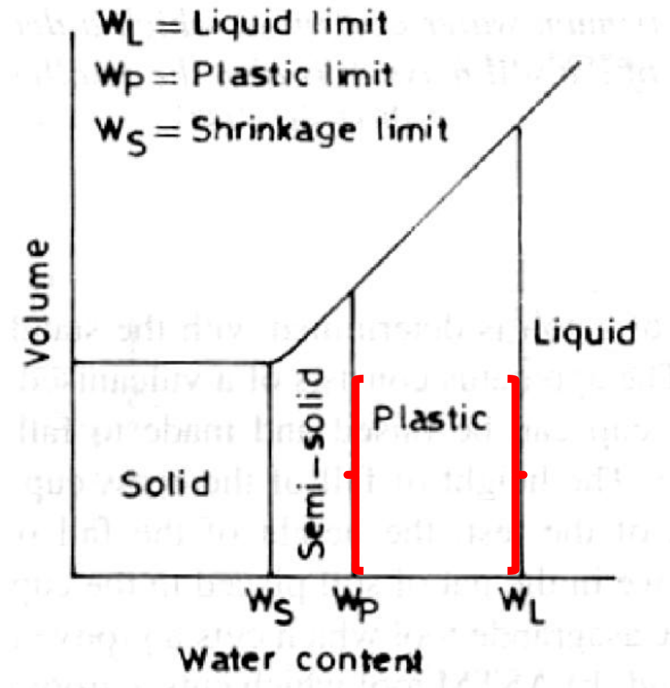
- *Water content at which the soil is a plastic*
- *Less water content than liquid limit*
- *Wide range of shear strengths at plastic limit*
- *Defined as the moisture content % at which the soil begins to crumble when rolled into 1/8” diameter threads*



animation

C. Liquid Limit

D. Plastic Limit



E. Plasticity Index (PI)

- *Difference between Liquid Limit and Plastic Limit*
- *Important measure of plastic behavior*

C. Liquid Limit

In general….

PI	Degree of Plasticity
0	Nonplastic
1-5	Slightly plastic
5-10	Low plasticity
10-20	Medium plasticity
20-40	High plasticity
40+	Very high plasticity

(from Burmister, 1949)

D. Plastic Limit

E. Plasticity Index (PI)

- *Difference between Liquid Limit and Plastic Limit*
- *Important measure of plastic behavior*

Classification Systems

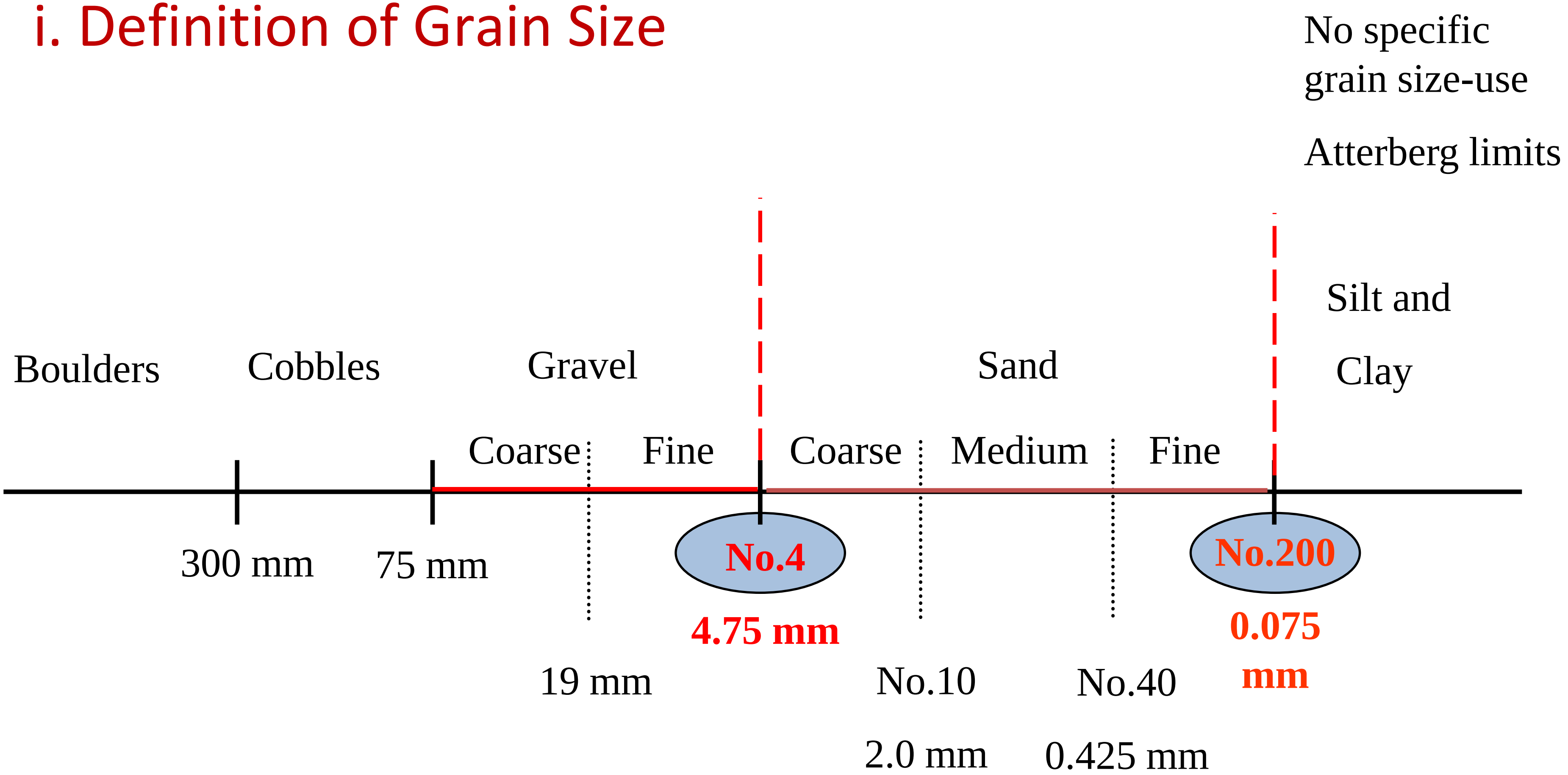
Two commonly classification system used are:

1. Unified Soil Classification System (USCS) (preferred by geotechnical engineers).

2. American Association of State Highway and Transportation Officials (AASHTO) System (preferred by Transportation engineers).

1. Unified Soil Classification System (USCS)

i. Definition of Grain Size



1. Unified Soil Classification System (USCS)

% Passing sieve No. 200 (0.075 mm)

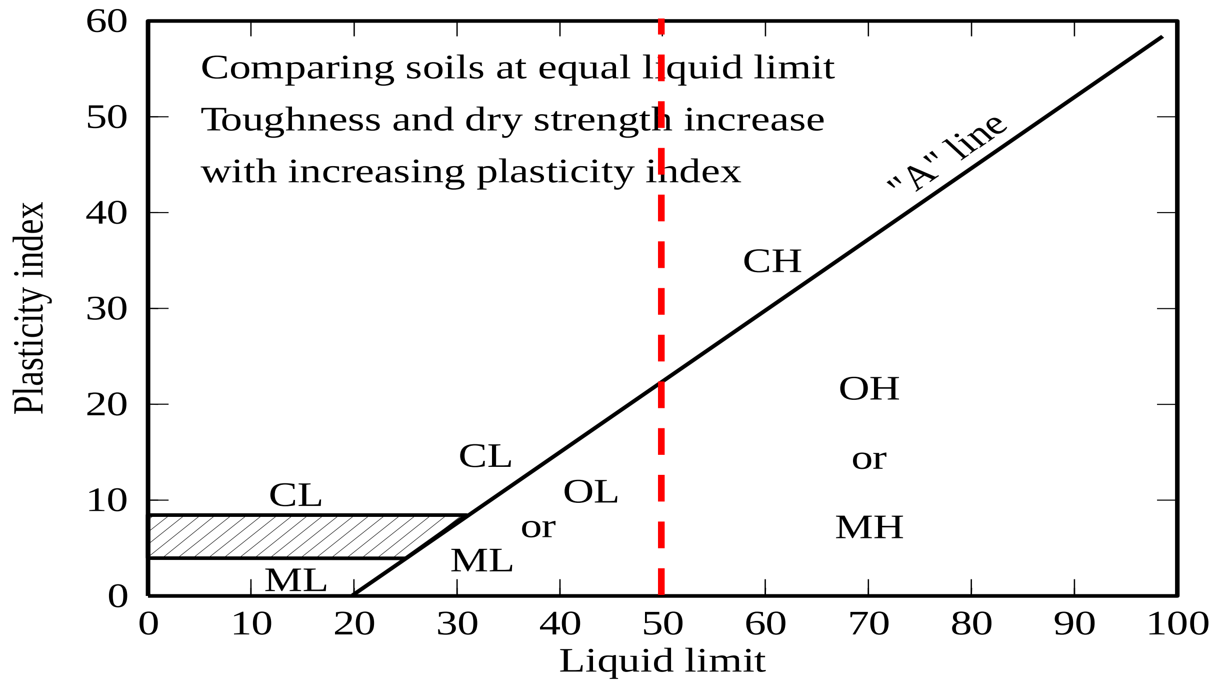
< 50%	> 50%
Coarse-grained soils	Fine-grained soils Silt (M) Clay (C)
•Grain size distribution	•Use Plasticity chart •LL, PL

Required tests: Sieve analysis

Atterberg limit

1. Unified Soil Classification System (USCS)

Used for Fine grained soils to determine whether silt (M) or clay (C)



Plasticity chart

Below A-line is silt – use symbol M

Above A-line is clay – use symbol C

LL > 50 → High plasticity

LL< 50 → low plasticity

1. Unified Soil Classification System (USCS)

% Passing sieve No. 200 (0.075 mm)

< 50%

Coarse-grained soils

•Grain size distribution

> 50%

Fine-grained soils

Silt (M)

Clay (C)

•Use Plasticity chart

ML, MH, CL, CH

Required tests: Sieve analysis

Atterberg limit

1. Unified Soil Classification System (USCS)

% Passing sieve No. 200 (0.075 mm)

< 50%

Coarse-grained soils

% Coarse soil (Co) = 100 - % Passing # 200

% Gravel (G) = 100 - % Passing # 4

G > 1/2 Co
Gravel (G)

G < 1/2 Co
Sand (S)

% Passing sieve No. 200

- < 5%: GW, GP, SW or SP — Use → C_u, C_c W : well graded P: poorly graded
- 5% -12 %: GW-GM, GW-GC, GP-GM, GP-GC, SW-SM, SW-SC, SP-SM, SP-SC
- > 12%: GM, GC, SM, SC — Use → plasticity charts

> 50%

Fine-grained soils
Silt (M)
Clay (C)

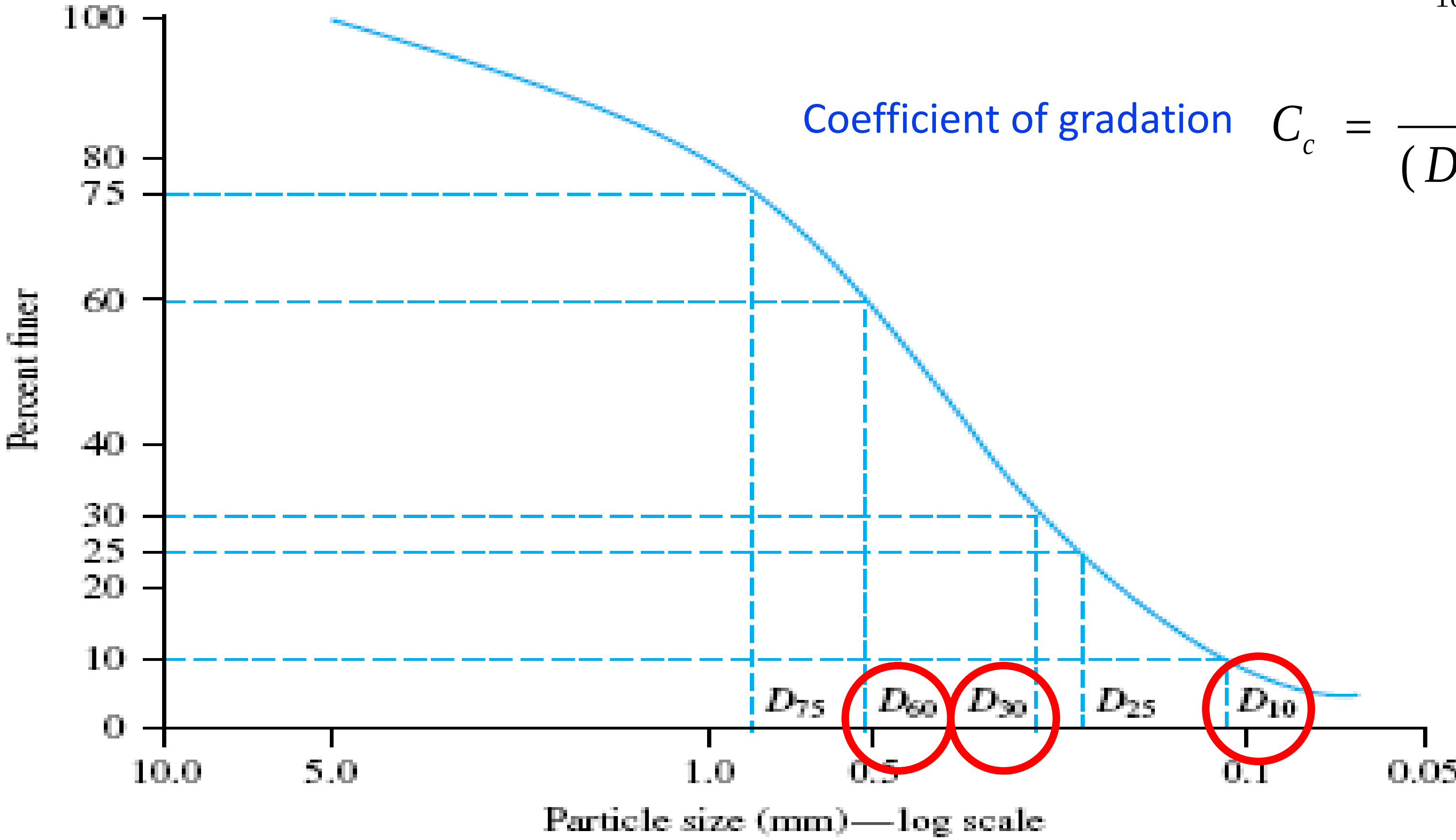
- Use Plasticity chart
- LL, PL

1. Unified Soil Classification System (USCS)

To determine if well graded (W) or poorly graded (P), calculate C_u and C_c

Coefficient of uniformity $C_u = \frac{D_{60}}{D_{10}}$

Coefficient of gradation $C_c = \frac{D_{30}^2}{(D_{60} \times D_{10})}$



1. Unified Soil Classification System (USCS)

Coefficient of uniformity $$C_u = \frac{D_{60}}{D_{10}}$$

Coefficient of gradation $$C_c = \frac{D_{30}^2}{(D_{60} \times D_{10})}$$

Conditions for Well-graded soils

For gravels → $C_u > 4$ **and** C_c is between 1 and 3

For Sand → W if $C_u > 6$ **and** C_c is between 1 and 3

1. Unified Soil Classification System (USCS)

Table 5.2 Unified Soil Classification System (Based on Material Passing 76.2-mm Sieve)

Criteria for assigning group symbols				Group symbol
Coarse-grained soils More than 50% of retained on No. 200 sieve	**Gravels** More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[a]	$C_u \geq 4$ and $1 \leq C_c \leq 3$[c]	GW
			$C_u < 4$ and/or $1 > C_c > 3$[c]	GP
		Gravels with Fines More than 12% fines[a,d]	$PI < 4$ or plots below "A" line (Figure 5.3)	GM
			$PI > 7$ and plots on or above "A" line (Figure 5.3)	GC
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines[b]	$C_u \geq 6$ and $1 \leq C_c \leq 3$[c]	SW
			$C_u < 6$ and/or $1 > C_c > 3$[c]	SP
		Sands with Fines More than 12% fines[b,d]	$PI < 4$ or plots below "A" line (Figure 5.3)	SM
			$PI > 7$ and plots on or above "A" line (Figure 5.3)	SC
Fine-grained soils 50% or more passes No. 200 sieve	**Silts and clays** Liquid limit less than 50	Inorganic	$PI > 7$ and plots on or above "A" line (Figure 5.3)[e]	CL
			$PI < 4$ or plots below "A" line (Figure 5.3)[e]	ML
		Organic	$\dfrac{\text{Liquid limit — oven dried}}{\text{Liquid limit — not dried}} < 0.75$; see Figure 5.3; OL zone	OL
	Silts and clays Liquid limit 50 or more	Inorganic	PI plots on or above "A" line (Figure 5.3)	CH
			PI plots below "A" line (Figure 5.3)	MH
		Organic	$\dfrac{\text{Liquid limit — oven dried}}{\text{Liquid limit — not dried}} < 0.75$; see Figure 5.3; OH zone	OH
Highly Organic Soils	Primarily organic matter, dark in color, and organic odor			Pt

[a] Gravels with 5 to 12% fine require dual symbols: GW-GM, GW-GC, GP-GM, GP-GC.

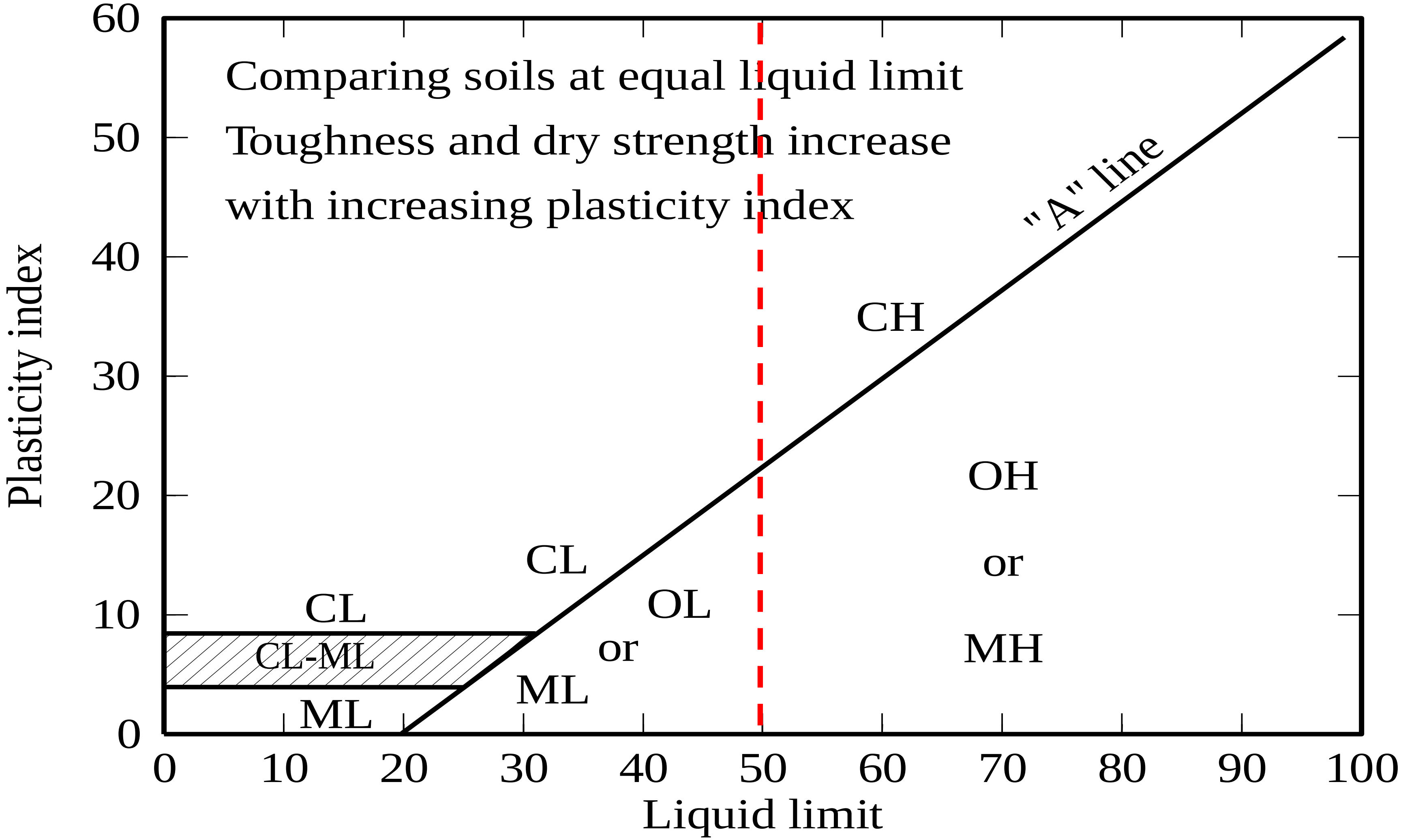
[b] Sands with 5 to 12% fines require dual symbols: SW-SM, SW-SC, SP-SM, SP-SC.

[c] $C_u = \dfrac{D_{60}}{D_{10}}; \quad C_c = \dfrac{(D_{30})^2}{D_{60} \times D_{10}}$

[d] If $4 \leq PI \leq 7$ and plots in the hatched area in Figure 5.3, use dual symbol GC-GM or SC-SM.

[e] If $4 \leq PI \leq 7$ and plots in the hatched area in Figure 5.3, use dual symbol CL-ML.

1. Unified Soil Classification System (USCS)



Plasticity chart

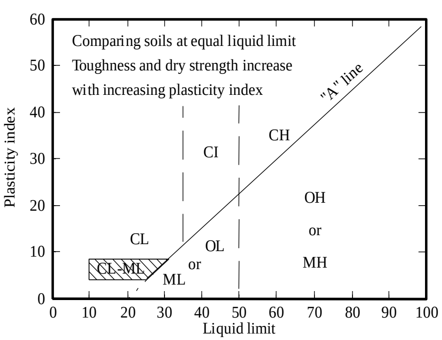
Unified soil classification (including identification and description)

Note (for all rows): The .075mm sieve size is about the smallest particle visible to the naked eye. Use grain size curve in identifying the fractions as given under field identification.

Major divisions			*Field identification procedures* (Excluding particles larger than 75mm and basing fractions on estimated weights)	*Group symbols* 1	*Typical names*	*Information required for describing soils*	*Laboratory classification criteria*	
Coarse grained soils *More than half of material is larger than .075mm sieve size*	*Gravels* *More than half of coarse fraction is larger than 2.36mm*	*Clean gravels (little or no fines)*	Wide range of grain size and substantial amounts of all intermediate particle sizes	GW	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical names: indicate approximate percentages of sand and gravel: maximum size: angularity, surface condition, and hardness of the coarse grains: local or geological name and other pertinent descriptive information and symbol in parentheses. For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics. *Example:* Silty sand, gravelly; about 20% hard angular gravel particles 12.5mm maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic lines with low dry strength; well compacted and moist in places; alluvial sand; (SM)	Determine percentages of gravel and sand from grain size curve Depending on percentages of fines (fraction smaller than .075mm sieve size) coarse grained soils are classified as follows Less than 5% GW, GP, SW, SP More than 12% GM, GC, SM, SC 5% to 12% Borderline case requiring use of dual symbols	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 4 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing	GP	Poorly graded gravels, gravel-sand mixtures, little or no fines			Not meeting all gradation requirements for GW
		Gravels with fines (appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below)	GM	Silty gravels, poorly graded gravel-sand-silt mixtures			Atterberg limits below "A" line or PI less than 4 — Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols
			Plastic fines (for identification procedures see CL below)	GC	Clayey gravels, poorly graded gravel-sand-clay mixtures			Atterberg limits above "A" line with PI greater than 7
	Sands *More than half of coarse fraction is smaller than 2.36mm*	*Clean sands (little or no fines)*	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	SW	Well graded sands, gravelly sands, little or no fines			$C_U = \frac{D_{60}}{D_{10}}$ Greater than 6 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantely one size or a range of sizes with some intermediate sizes missing	SP	Poorly graded sands, gravelly sands, little or no fines			Not meeting all gradation requirements for SW
		Sands with fines (appreciable amount of fines)	Non-plastic fines (for identification procedures, see ML below)	SM	Silty sands, poorly graded sand-silt mixtures			Atterberg limits below "A" line or PI less than 4 — Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols
			Plastic fines (for identification procedures, see CL below)	SC	Clayey sands, poorly graded sand-clay mixtures			Atterberg limits above "A" line with PI greater than 7

Identification procedure on fraction smaller than .425mm sieve size

Major divisions		*Dry strength crushing characteristics*	*Dilatency (reaction to shaking)*	*Toughness (consistency near plastic limit)*	Group symbols	Typical names	Information required for describing soils
Fine grained soils *More than half of material is smaller than .075mm sieve size*	*Silts and clays liquid limit less than 50*	None to slight	Quick to slow	None	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains: colour in wet condition, odour if any, local or geological name, and other pertinent descriptive information, and symbol in parentheses For undisturbed soils add information on structure, stratification, consistency and undisturbed and remoulded states, moisture and drainage conditions *Example* Clayey silt, brown: slightly plastic: small percentage of fine sand: numerous vertical root holes: firm and dry in places; loess; (ML)
		Medium to high	None to very slow	Medium	CL,CI	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
		Slight to medium	Slow	Slight	OL	Organic silts and organic silt-clays of low plasticity	
	Silts and clays liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	MH	inorganic silts, micaceous or dictomaceous fine sandy or silty soils, elastic silts	
		High to very high	None	High	CH	Inorganic clays of high plasticity, fat clays	
		Medium to high	None to very high	Slight to medium	OH	Organic clays of medium to high plasticity	
Highly organic soils		Readily identified by colour, odour spongy feel and frequently by fibrous texture			Pt	Peat and other highly organic soils	



Plasticity chart
for laboratory classification of fine grained soils

Organic Soils

- Highly organic soils- Peat (Group symbol PT)
  - A sample composed primarily of vegetable tissue in various stages of decomposition and has a fibrous to amorphous texture, a dark-brown to black color, and an organic odor should be designated as a highly organic soil and shall be classified as peat, PT.

- Organic clay or silt (group symbol OL or OH):
  - "The soil's liquid limit (LL) after oven drying is less than 75 % of its liquid limit before oven drying." If the above statement is true, then the first symbol is O.
  - The second symbol is obtained by locating the values of PI and LL (not oven dried) in the plasticity chart.

Borderline Cases (Dual Symbols)

Coarse-grained soils with 5% - 12% fines.

- About 7 % fines can change the hydraulic conductivity of the coarse-grained media by orders of magnitude.
- The first symbol indicates whether the coarse fraction is well or poorly graded. The second symbol describe the contained fines. For example: SP-SM, poorly graded sand with silt.

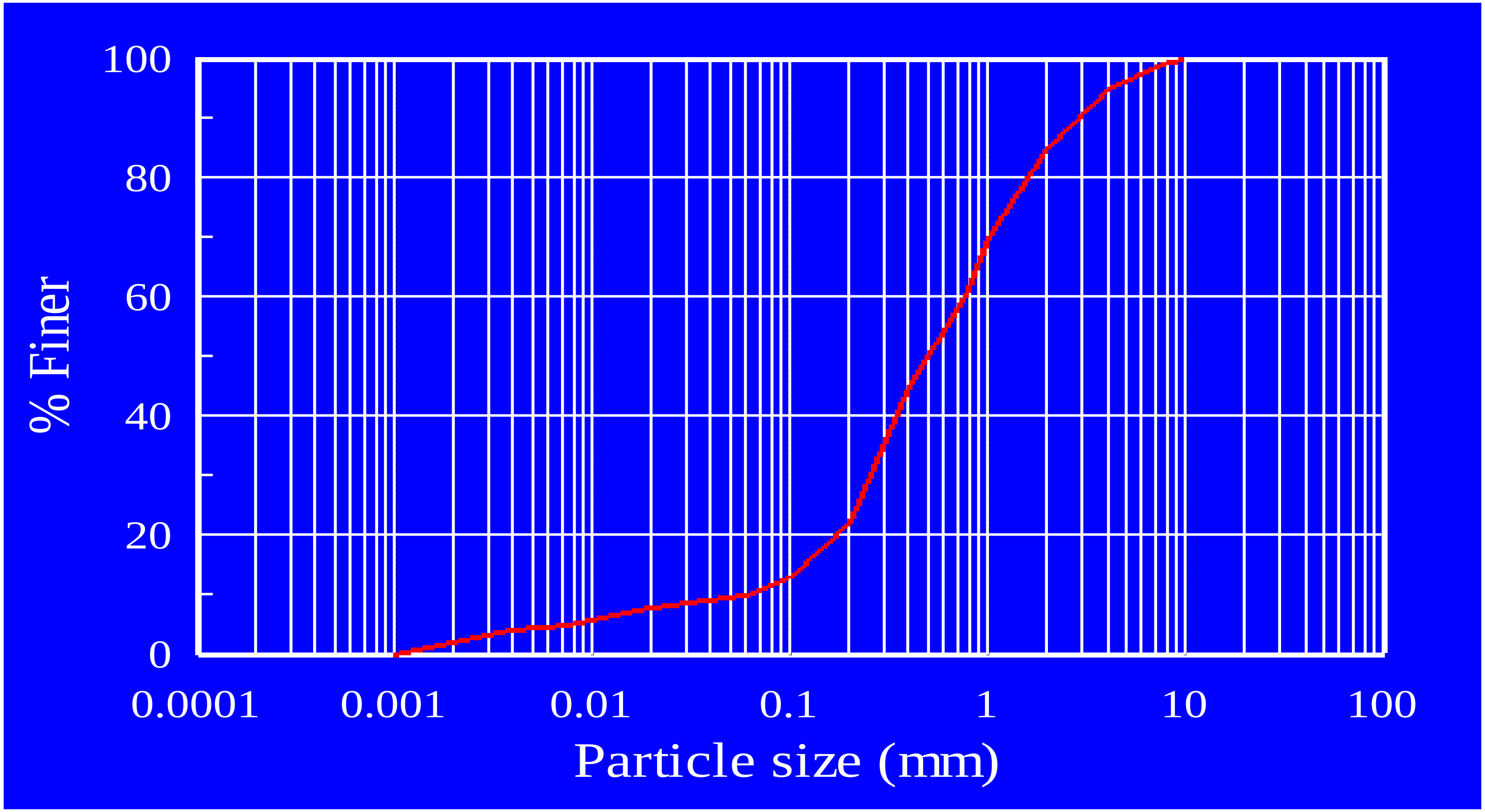
Fine-grained soils with limits within the shaded zone. (PI between 4 and 7 and LL between about 12 and 25).

- It is hard to distinguish between the silty and more claylike materials.
- CL-ML: Silty clay,   SC-SM: Silty, clayed sand.

Soil contain similar fines and coarse-grained fractions.

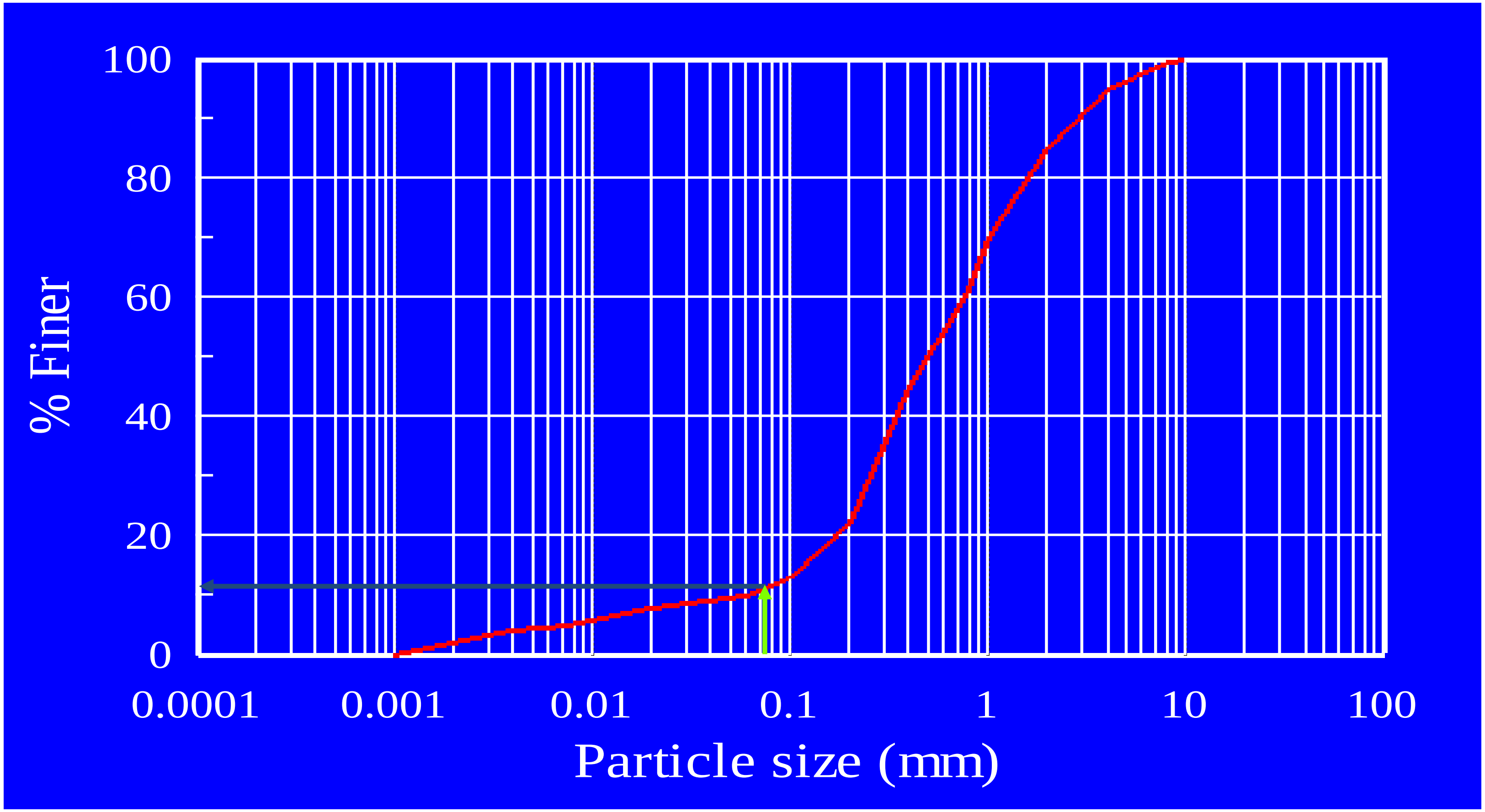
- possible dual symbols GM-ML

Example 1



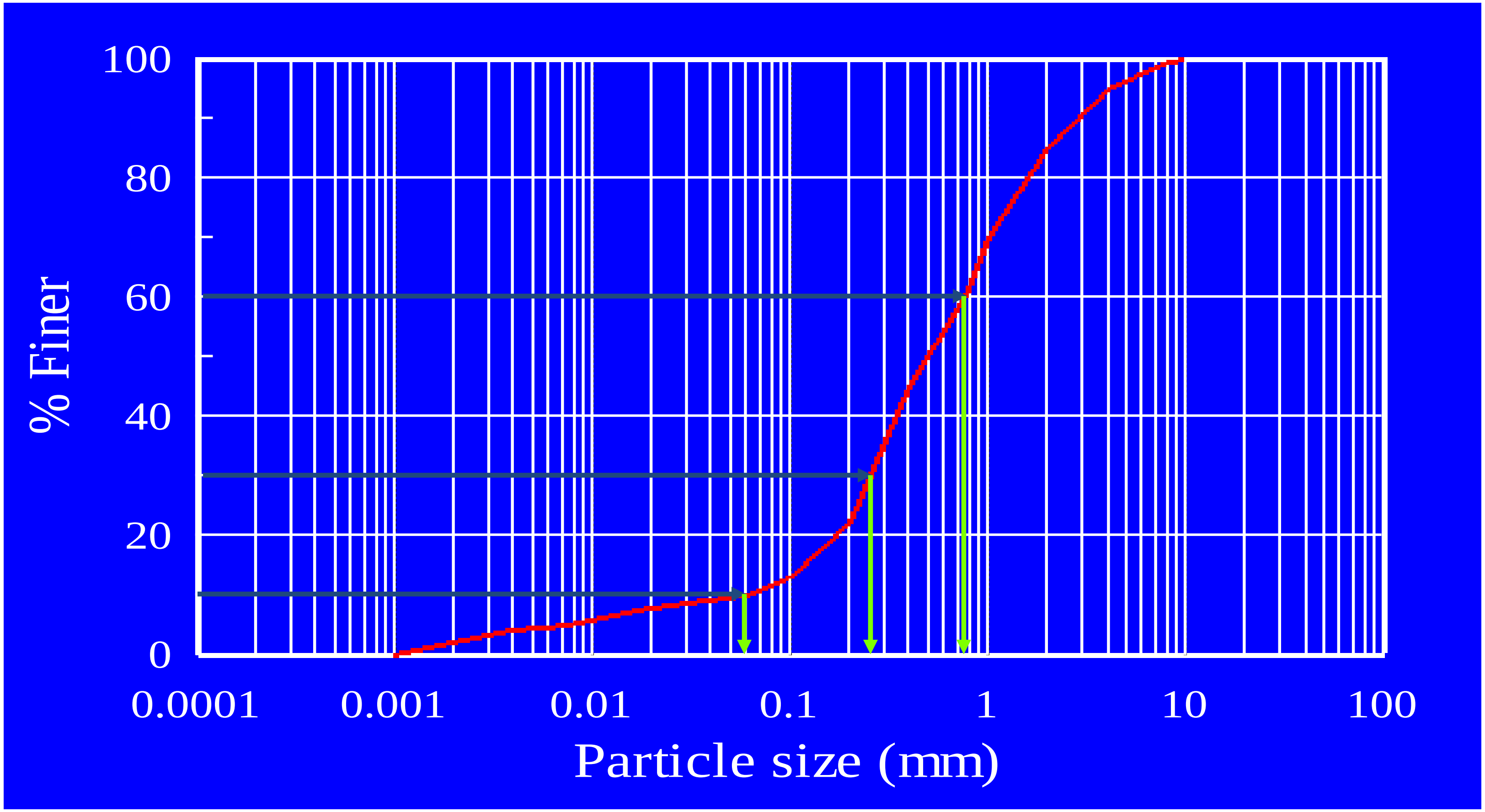
100
80
60
40
20
0
% Finer
0.0001
0.001
0.01
0.1
1
10
100
Particle size (mm)

Example 1



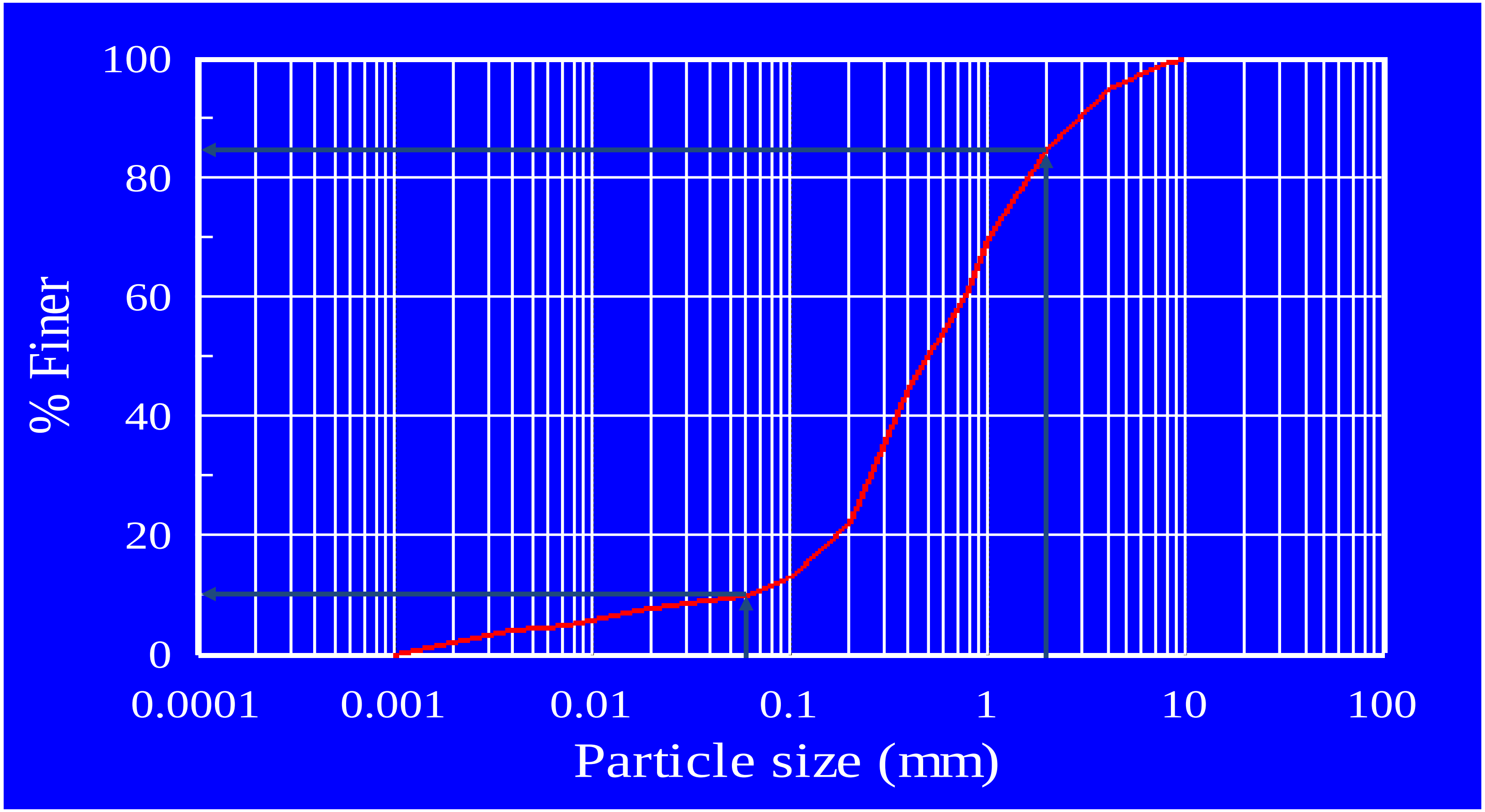
- %fines (% finer than 75 μm) = 11% - Dual symbols required

Example 1



- %fines (% finer than 75 μm) = 11% - Dual symbols required
- D_{10} = 0.06 mm, D_{30} = 0.25 mm, D_{60} = 0.75 mm

Example 1



Particle size fractions:	Gravel	17%
	Sand	73%
	Silt and Clay	10%

Example 1

Of the coarse fraction about 80% is sand, hence Prefix is S

$C_u = 12.5$, $C_c = 1.38$

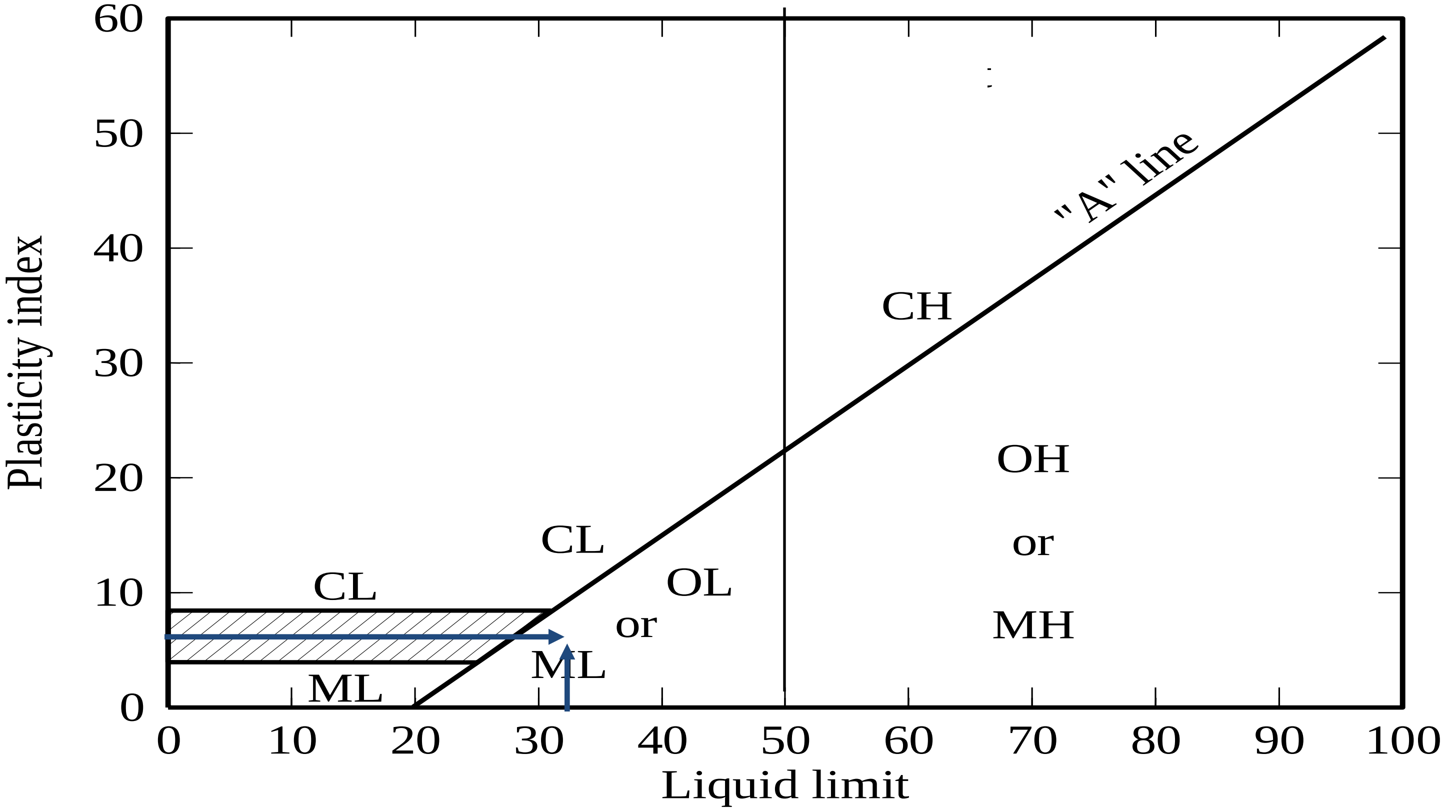
$\text{Suffix}_1 = W$

From Atterberg Tests

LL = 32, PL = 26

$I_p = 32 - 26 = 6$

Example 1



Plasticity chart
for laboratory classification of fine grained soils

Example 1

Of the coarse fraction about 80% is sand, hence Prefix is S

$C_u = 12.5$, $C_c = 1.38$

Suffix$_1$ = W

From Atterberg Tests

LL = 32, PL = 26 & $I_p = 32 - 26 = 6$

From Plasticity Chart point lies below A-line

Suffix$_2$ = M

Example 1

Of the coarse fraction about 80% is sand, hence Prefix is S

$C_u = 12.5$, $C_c = 1.38$

$Suffix_1 = W$

From Atterberg Tests

LL = 32, PL = 26

$I_p = 32 - 26 = 6$

From Plasticity Chart point lies below A-line

$Suffix_2 = M$

Dual Symbols are SW-SM

Example 1

Of the coarse fraction about 80% is sand, hence Prefix is S

$C_u = 12.5$, $C_c = 1.38$

$Suffix_1 = W$

From Atterberg Tests

LL = 32, PL = 26 & $I_p = 32 - 26 = 6$

From Plasticity Chart point lies below A-line

$Suffix_2 = M$

Dual Symbols are SW-SM

To complete the classification the Symbols should be accompanied by a description

Example 2

Classify the following soils Using Unified Classification System.

Soil	No. 4 Sieve	No. 200 Sieve	LL	PI
(cumulative % passing)				
A	92	48	30	10
B	99	76	60	32
C	80	35	24	2

1. Unified Soil Classification System (USCS)

% Passing sieve No. 200 (0.075 mm)

< 50%

Coarse-grained soils

% Coarse soil (Co) = 100 - % Passing # 200

% Gravel (G) = 100 - % Passing # 4

- G > 1/2 Co — Gravel (G)
- G < 1/2 Co — Sand (S)

% Passing sieve No. 200

- < 5% — GW, GP, SW or SP — Use → C_u, C_c — W : well graded P: poorly graded
- 5% -12 % — GW-GM, GW-GC, GP-GM, GP-GC, SW-SM, SW-SC, SP-SM, SP-SC
- > 12% — GM, GC, SM, SC — Use → plasticity charts

> 50%

Fine-grained soils
Silt (M)
Clay (C)

- Use Plasticity chart
- LL, PL

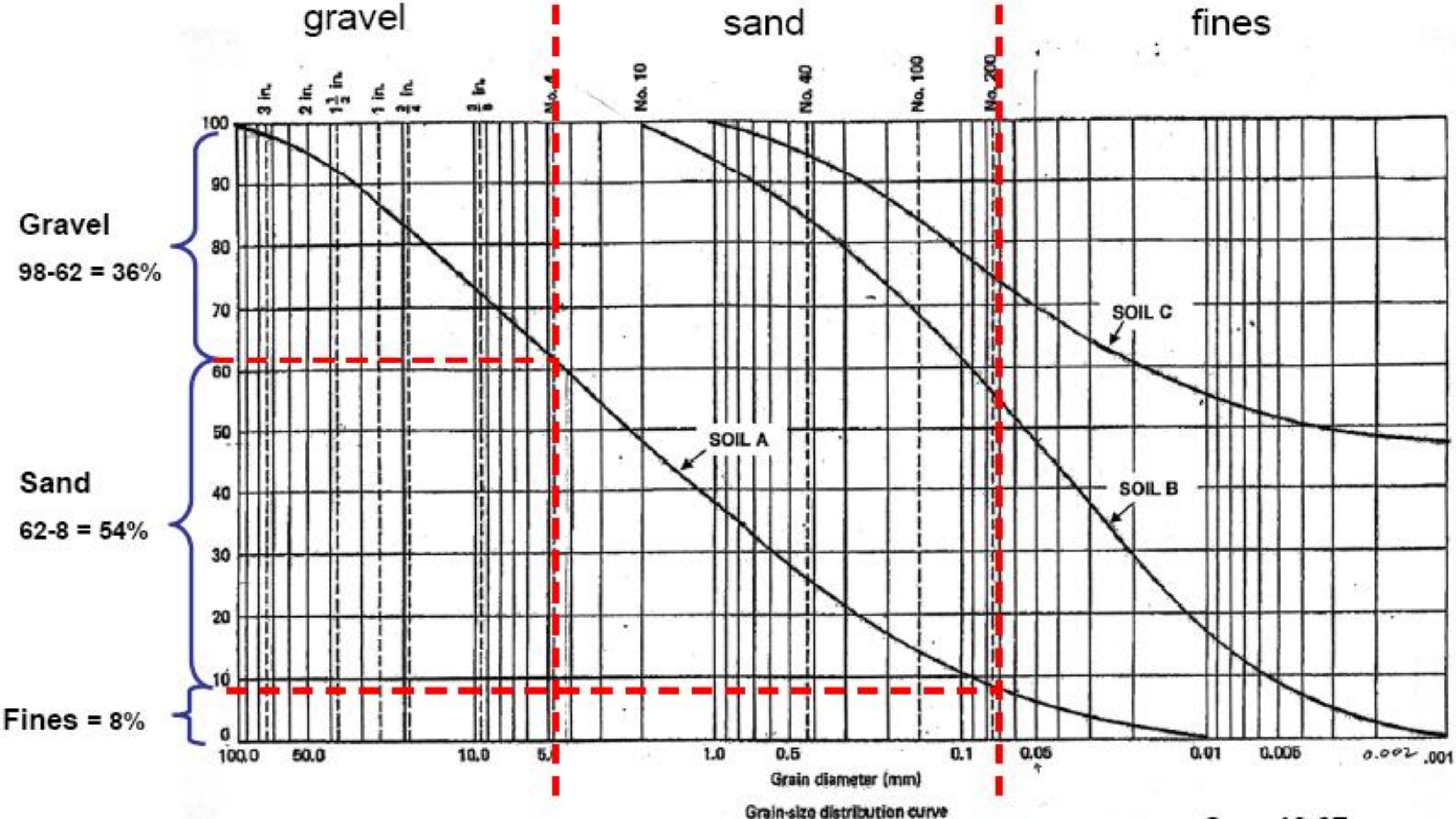
Example 2

Soil A
Coarse = 100-48 = 52% (retained on N o. 2 0 0), so COARSE-GRAINED SOIL
8% retained on No. 4, vs. 52% coarse,
8/52 = 15% (<50%), so SAND
Using the LL and PL values in the USAC
Atterberg limits above line A, so Clay
Classification SC, clayey sand

Soil B
Coarse = 100 - 76 = 24%, so FINE-GRAINED SOIL
LL = 60, and P I = 32 Using Casagrandi Chart
Classification CH, inorganic clay with high plasticity

Soil C
Coarse = 100 - 35 = 65%, so COARSE-GRAINED SOIL
20% retained on No. 4, vs. 65%
coarse, 20/65 = 31% (<50%), so SAND
Using Casagrandi Chart
Classification SM, Silty sand

Example 3 – Soil A

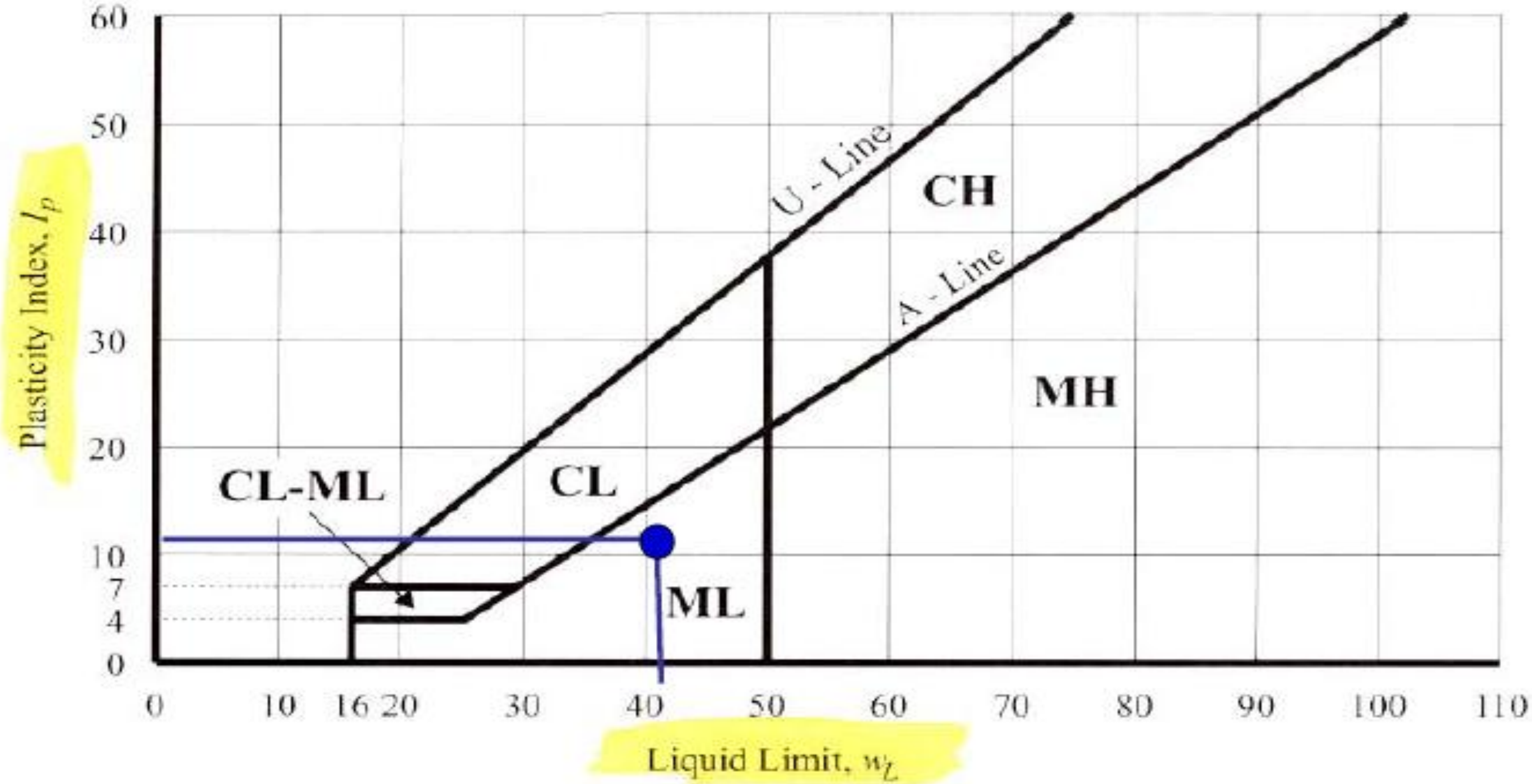


Soil A: D_{60} = 4.2 mm , D_{30} = 0.6 mm, D_{10} = 0.09 mm

Cu = 46.67
Cc = 0.95

LL = 42
PL = 31
PI = 42-31 = 11

Example 3 – Soil A (Cont.)



LL = 42
PL = 31
PI = 42-31 = 11

$\rightarrow$ $\boxed{\text{ML}}$

Example 3

Soil A is then classified as

SP-SM – Poorly-grades sand with silt and gravel

Example 4

Passing No.200 sieve 30 %

Passing No.4 sieve 70 %

LL= 33

PI= 12

PI= 0.73(LL-20), A-line

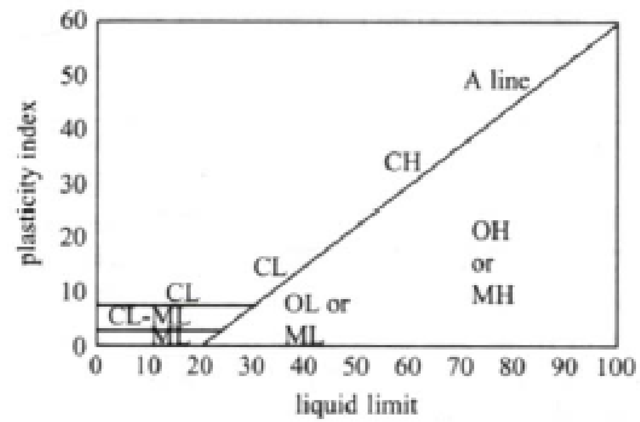
PI=0.73(33-20)=9.49

SC

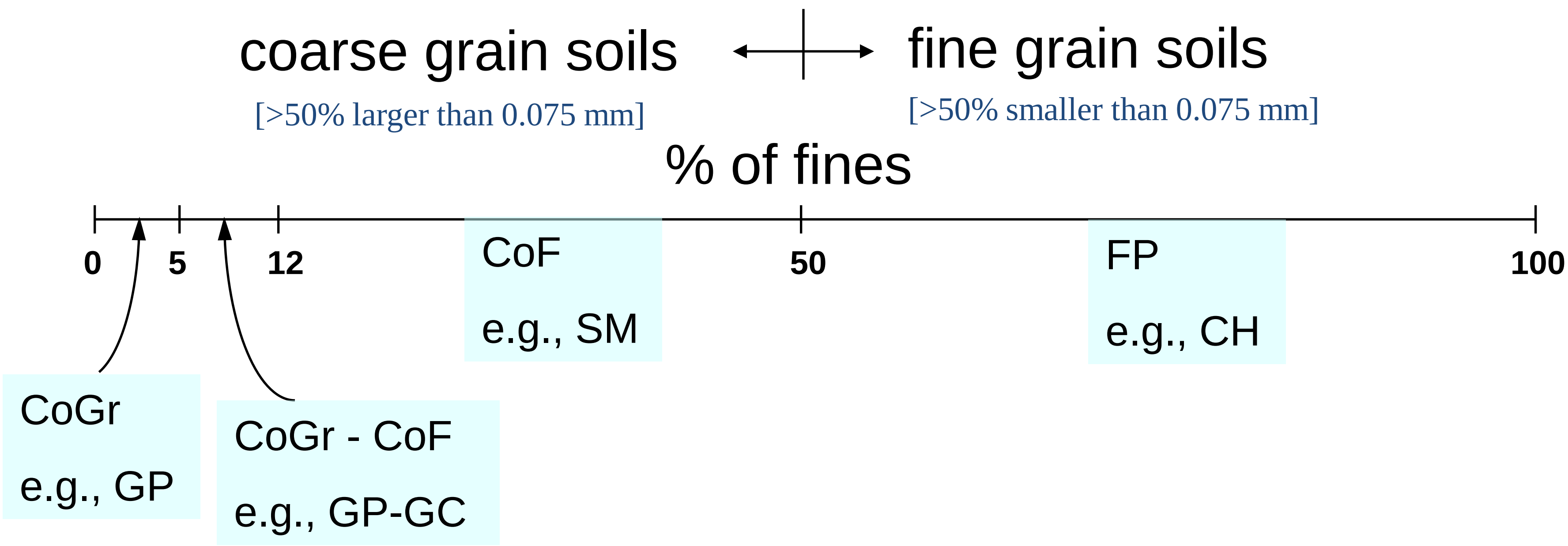
(≥15% gravel)

Clayey sand with gravel

COARSE More than 50% retained sieve #200	Gravel: more than 50% coarse fraction retained on sieve #4	Less than 5% fines	$C_u > 4,\ 1 \le C_c \le 3$	→ GW
			Not satisfying GW	→ GP
		More than 12% fines	Below 'A' line	→ GM
			Above 'A' line	→ GC
	Sand: less than 50% coarse fraction retained on sieve #4	Less than 5% fines	$C_u > 6,\ 1 \le C_c \le 3$	→ SW
			Not satisfying SW	→ SP
		More than 12% fines	Below 'A' line	→ SM
			Above 'A' line	→ SC
FINE Less than 50% retained sieve #200	$LL < 50$			ML
				CL
				OL
	$LL > 50$			MH
				CH
				OH



Summary of the USCS



<u>**Co**: Coarse</u>

G = Gravel

S = Sands

<u>**F**: Fines</u>

M = Silts

C = Clays

<u>**Gr**: Gradation</u>

W = well graded

P = poorly graded

<u>**P**: Plasticity</u>

H = LL > 50

L = LL < 50

Symbols

Soil symbols:

G: Gravel

S: Sand

M: Silt

C: Clay

O: Organic

Pt: Peat

Liquid limit symbols:

H: High LL (LL>50)

L: Low LL (LL<50)

Gradation symbols:

W: Well-graded

P : Poorly-graded

Example: SW, Well-graded sand

SC, Clayey sand

SM, Silty sand,

MH, Elastic silt

$$\text{Well} - \text{graded soil}$$

$$1 < C_c < 3 \text{ and } C_u \geq 4$$

$$\text{(for gravels)}$$

$$1 < C_c < 3 \text{ and } C_u \geq 6$$

$$\text{(for sands)}$$

2. American Association of State Highway and Transportation Officials system (AASHTO)

Origin of AASHTO: (For road construction)

- This system was originally developed by Hogentogler and Terzaghi in 1929 as the Public Roads Classification System.

- Afterwards, there are several revisions. The present AASHTO (1978) system is primarily based on the version in 1945. (Holtz and Kovacs, 1981)

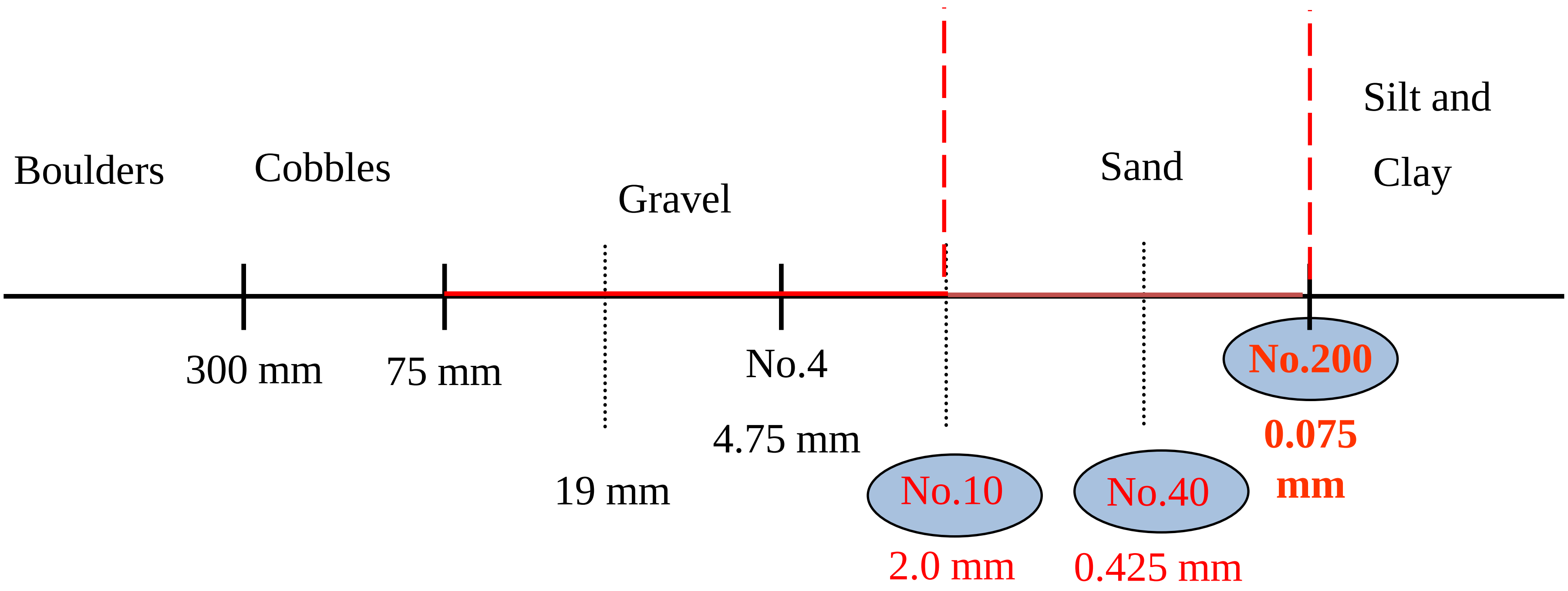
edure for AASHTO Classification (American Association of State Highway and Transportation Officials)

Developed in 1929 as the Public Road Administration Classification System
Modified by the Highway Research Board (1945)

2. AASHTO

i. Definition of Grain Size

No specific grain size-use Atterberg limits



Procedure for AASHTO Classification

- Determine the percentage of soil passing the #200 sieve
- Determine the subgroups
  - For coarse-grained soils (gravel and sand), determine the percent passing the #10, 40, and 200 sieves, **AND**
  - Determine the liquid limit and plasticity index
  - **THEN**, determine soil group or subgroup from Table 9.1

– For coarse-grained soils (gravel and sand), determine the percent passing the #10, 40, and 200 sieves.

	Cobble
3 "	Gravel
#10	Very Coarse to Med Sand
#40	Fine/Very Fine Sand
#200	Silt/Clay

#10

#40

#200

2. AASHTO

ii. General guidance

- 8 major groups: A1~ A7 (with several subgroups) and organic soils A8
- The required tests are sieve analysis and Atterberg limits.
- The group index, an empirical formula, is used to further evaluate soils within a group (subgroups).

A1 ~ A3	A4 ~ A7
Granular Materials	Silt-clay Materials
$\leq$ 35% pass No. 200 sieve	$\geq$ 36% pass No. 200 sieve
Using LL and PI separates silty materials from clayey materials (only for A2 group)	Using LL and PI separates silty materials from clayey materials

- The original purpose of this classification system is used for road construction (subgrade rating).

2. AASHTO

iii. Classification

General classification	Granular materials (35% or less of total sample passing No. 200)						
	A-1			A-2			
Group classification	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7
Sieve analysis (percentage passing)							
No. 10	50 max.						
No. 40	30 max.	50 max.	51 min.				
No. 200	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.
Characteristics of fraction passing No. 40							
Liquid limit				40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		NP	10 max.	10 max.	11 min.	11 min.
Usual types of significant constituent materials	Stone fragments, gravel, and sand		Fine sand	Silty or clayey gravel and sand			
General subgrade rating	Excellent to good						

Classification starts from left to right

2. AASHTO

iii. Classification

General classification	Silt-clay materials (more than 35% of total sample passing No. 200)			
Group classification	A-4	A-5	A-6	A-7 A-7-5[a] A-7-6[b]
Sieve analysis (percentage passing)				
No. 10				
No. 40				
No. 200	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No. 40				
Liquid limit	40 max.	41 min.	40 max.	41 min.
Plasticity index	10 max.	10 max.	11 min.	11 min.
Usual types of significant constituent materials	Silty soils		Clayey soils	
General subgrade rating	Fair to poor			

[a]For A-7-5, $PI \leq LL - 30$
[b]For A-7-6, $PI > LL - 30$

Note:

The first group from the left to fit the test data is the correct AASHTO classification.

Group Index

- Used to evaluate the quality of a soil as a highway subgrade material.
- This index is written in parentheses after the group or subgroup designation [e.g. A-4(3)].

The first term is determined by the LL

$$GI = (F_{200} - 35)[0.2 + 0.005(LL - 40)] + 0.01(F_{200} - 15)(PI - 10) \qquad (1)$$

The second term is determined by the PI

<u>For Groups A-2-6 and A-2-7</u>

$$GI = 0.01(F_{200} - 15)(PI - 10)$$ use the second term only

F200: percentage passing through the No.200 sieve

In general, the rating for a pavement subgrade is inversely proportional to the group index, GI.

Some rules of Group Index GI

1. If Eq. (1) yields a negative value for GI, it is taken as 0.

2. The group index is rounded off to the nearest whole number (for example, GI 3.4 is rounded off to 3; GI 3.5 is rounded off to 4).

3. There is no upper limit for the group index.

4. The group index of soils belonging to groups A-1-a, A-1-b, A-2-4, A-2-5, and A-3 is always 0.

Example 1

Classify the following soils by the AASHTO classification system.

	Soil				
Description	A	B	C	D	E
Percent finer than No. 10 sieve	83	100	48	90	100
Percent finer than No. 40 sieve	48	92	28	76	82
Percent finer than No. 200 sieve	20	86	6	34	38
Liquid limit[a]	20	70	—	37	42
Plasticity index[a]	5	32	Nonplastic	12	23

Example 1 [Soil B]

Passing No.200 86%

LL=70, PI=32

LL-30=40 > PI=32

$$GI = (F_{200} - 35)[0.2 + 0.005(LL - 40)] + 0.01(F_{200} - 15)(PI - 10) = 33.47 \cong 33 \quad \textit{Round off}$$

$\boxed{A\text{-}7\text{-}5(33)}$

General classification	Silt-clay materials (more than 35% of total sample passing No. 200)			
Group classification	*A-4*	*A-5*	*A-6*	*A-7* *A-7-5*[a] *A-7-6*[b]
Sieve analysis (percentage passing)				
No. 10				
No. 40				
No. 200	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No. 40				
Liquid limit	40 max.	41 min.	40 max.	41 min.
Plasticity index	10 max.	10 max.	11 min.	11 min.
Usual types of significant constituent materials	Silty soils		Clayey soils	
General subgrade rating	Fair to poor			

[a] For A-7-5, $PI \le LL - 30$

[b] For A-7-6, $PI > LL - 30$

Example 2

- Classify the following soil Using AASHTO System.
- Given:
- % passing No. 10 = 100;
- % passing No. 40 = 80;
- '% passing No.200 = 58
- LL = 30; PI = 10.

Example 2

From Table, the group classification is A-4.

Now we calculate the Group Index

GI= (F-3S) [0.2 + 0.005(LL-40)] + 0.01(F-1)(PI-10)

Using the given data, F= 58, LL=30 & PI=10

GI= (58-23) [0.2 + 0.005(30-40)] + 0.01(58-15) (10-10)

GI= (35) [0.2 + 0.005(-10)] + 0.01(58-15) (0)

GI= 5.25

Then GI= 5.25 - - > 5

Thus, the AASHTO Classification is A- 4 (5); Silty Soil; Fair to poor as subgrade material

- Soil property changes from place to place. Even in the same place it may not be uniform at various depths. The soil property may vary from season to season due to variation in moisture content. The load from the structure is to be safely transferred to soil. For this, safe bearing capacity of the soil is to be properly assessed.

At one time, Terzaghi stated:

"There will be no soil mechanics without water."

PERMEABILITY AND SEEPAGE

- Soil water,
- permeability definition and necessity of its study,
- Darcy's law,
- factors affecting permeability.
- Laboratory measurement of permeability – Constant head method and Falling head method as per IS 2720.
- Field test for determination of permeability- Pumping in test and Pumping out test as per IS 5529part-I.
- Permeability of stratified soil deposits.

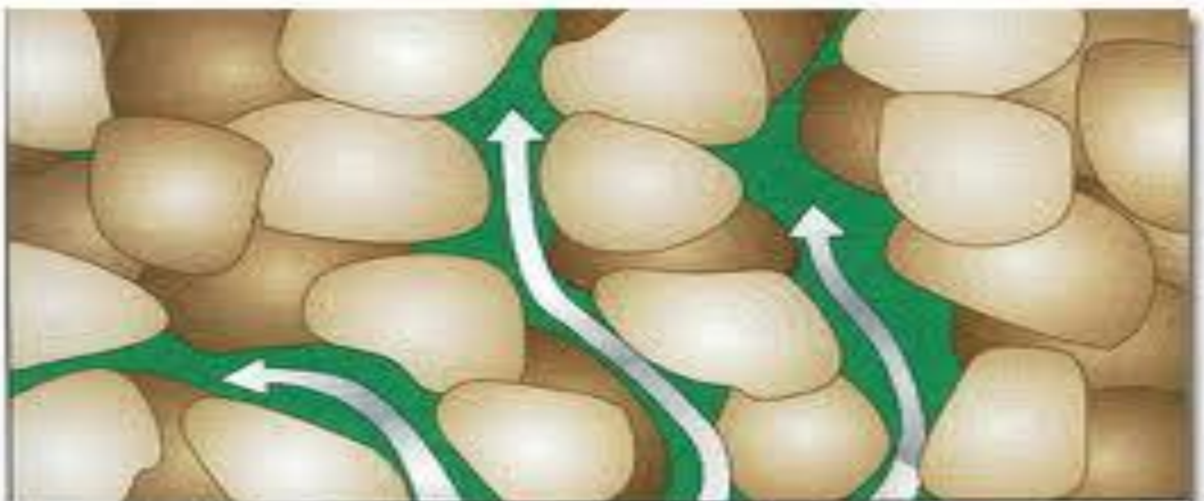
- Seepage and Seepage Pressure,
- quick sand phenomenon,
- critical hydraulic gradient,
- General flow equation for 2-D flow (Laplace equation), Flow Net, properties and application, Flow Net construction for flow under sheet pile and earthen dam.

SOIL-WATER SYSTEM

Soil properties :

Soil is a complex mass of mineral and organic particles. The important properties that classify soil according to its relevance to making crop production (which in turn affects the decision making process of irrigation engineering) are:

- Soil texture
- Soil structure



Connected pores give a rock permeability.

SOIL-WATER SYSTEM

Soil texture:

- This refers to the relative sizes of soil particles in a given soil. According to their sizes, soil particles are grouped into gravel, sand, silt and day.

- The relative proportions of sand, silt and clay is a soil mass determines the soil texture.

SOIL-WATER SYSTEM

- Engineering soil properties and parameters describe the behavior of soil under induced stress and environmental changes.

- Of interest to most geotechnical applications are the **strength, deformability, and permeability** of in situ and compacted soils.

PERMEABILITY AND SEEPAGE

- **The coefficient of permeability (or permeability) in soil mechanics is a measure of how easily a fluid (water) can flow through a porous medium (soil).**

- **The flow of water through soils, called seepage**, occurs when there is a difference in the water level (energy) on the two sides of a structure such as a dam or a sheet pile.

SOIL PERMEABILITY

- The coefficient of permeability k is a measure of the rate of flow of water through saturated soil under a given hydraulic gradient i, cm/cm, and is defined in accordance with Darcy's law as:

- Assumption : Flow is laminar

DARCY'S LAW:

When the flow through soil is laminar, Darcy's law (Darcy, 1856) applies:

The darcy's law states that for laminar flow the velocity of flow, v is proportional to the hydraulic gradient i.

V=Ki **(k is constant of proportionality)**

$$q = v \, . \, A = k \, i \, A$$

Where

q = rate of flow, cm^3/s.

A = cross-sectional area of soil conveying flow, cm^2

k is dependent on the grain-size distribution, void ratio, and soil fabric and typically may vary from as much as 10 cm/s for gravel to less than 10^{-7} cm/s for clays.

For typical soil deposits, k for horizontal flow is greater than k for vertical flow, often by an order of magnitude.

DARCY'S LAW:

V=Ki **(k is constant of proportionality)**

$$q = v \cdot A = k\,i\,A$$

The rate of flow under laminar flow conditions through a unit cross sectional are of porous medium under unit hydraulic gradient is defined as coefficient of permeability.

Hydraulic gradient is the total head loss per unit length.

SOIL PERMEABILITY

- Permittivity: Like permeability, a measure of the capacity of a geosynthetic to allow a fluid to move through its voids or interstices, as represented by the amount of fluid that passes through a unit surface area of the material in a unit time per unit thickness under a unit pressure gradient, with laminar flow in the direction of the thickness of the material.

- For evaluation of geotextiles, use of permittivity, being independent of thickness, is preferred to permeability.

SOIL PERMEABILITY

FACTORS AFFECTING PERMEABILITY:

- GRAIN SIZE
- PROPERTIES OF PORE FLUID
- TEMPERATURE
- VOID RATIO
- DEGREE OF SATURATION
- ABSORBED WATER
- SHAPE OF PARTICLES

GRAIN SIZE

Grain size of the soil , or the effective size D10 is one of the factors which affect permeability . Allen Hazen gave the relation

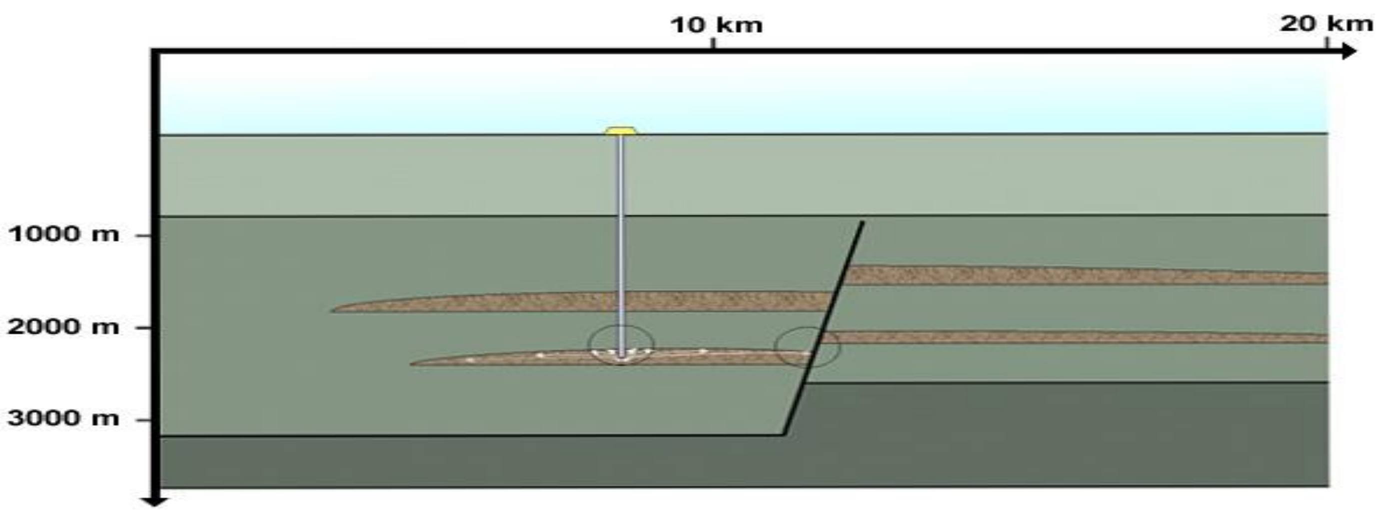
K=100(D10)2

Where, k= coeff of permeability in cm/s and D10 is the effective grain size of the soil.

The permeability of coarse grained soil is more than that of fine grained soil.

➢Properties of pore fluid

The permeability is directly proportional to the unit weight of water and inversely proportional to its viscosity. Though the unit weight of water does not change in temp , there is grate variation in viscosity with temp.

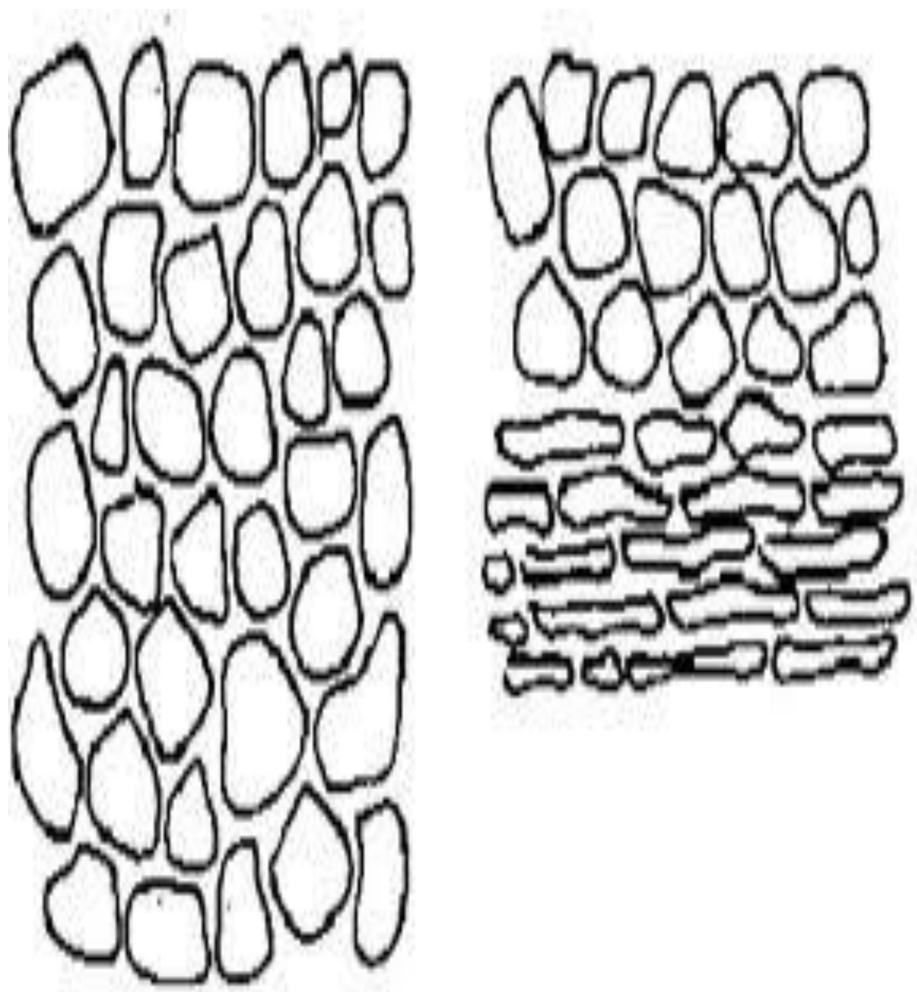


➢ temperature

Since viscosity of pore fluid decreases with the temp , permeability increases with temp , as unit weight of the pore fluid does not change much with change in temp .

➢ Void ratio

Increases in void ratio increase the area available for flow , hence the permeability increases with critical condition



VOID RATIO

➢Degree of saturation

The permeability of partially saturated soil is less than that of fully saturated soil

➢Absorbed water

The absorbed water surrounding the fine- soil particles is not free to move , and reduces the effective pore space available for the passage of water .

Shape of particles

Permeability is inversely proportional to the specific surface e.g. the angular particles have more specific surface area as compare to rounded particles.

Example 9.1 A 0.5-m-long soil specimen is subjected to a steady-state flow with a constant head loss $h_L = 1.5$ m as shown in Figure 9.4. Calculate the total head at point C.

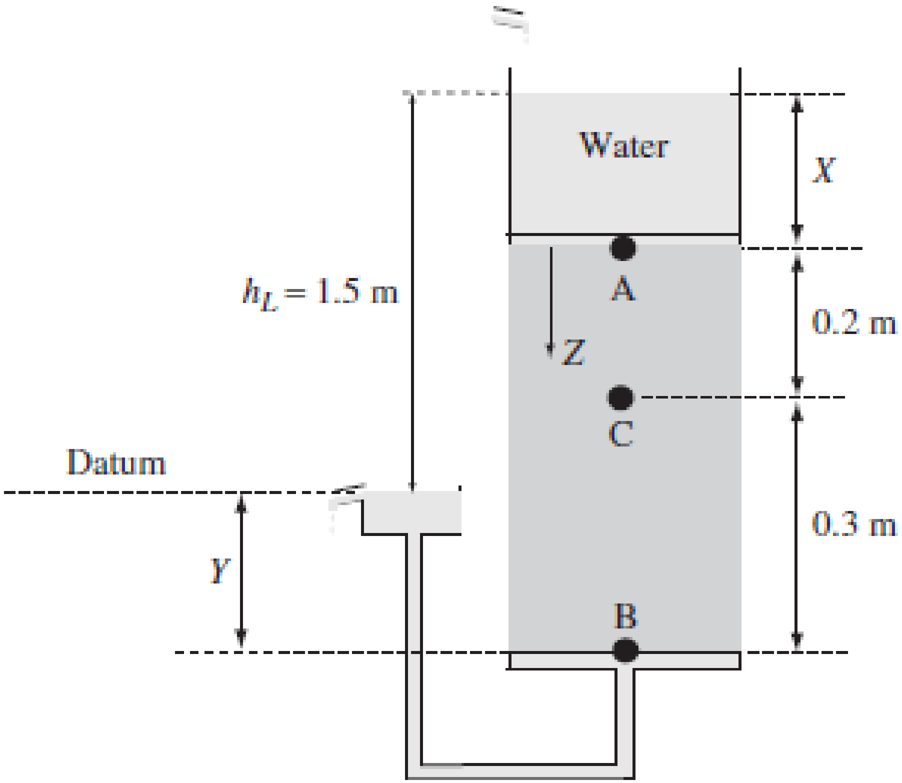


FIGURE 9.4 Steady-state flow with a constant head loss.

Example 9.2 Consider the one-dimensional flow condition shown in Figure 9.5. Determine the hydraulic gradient between points A and B and the flow direction knowing that the pore pressures at points A and B are 2.943 and 11.772 kPa, respectively. With respect to the datum shown in the figure, the elevation heads of points A and B are 0.5 and 0.2 m, respectively.

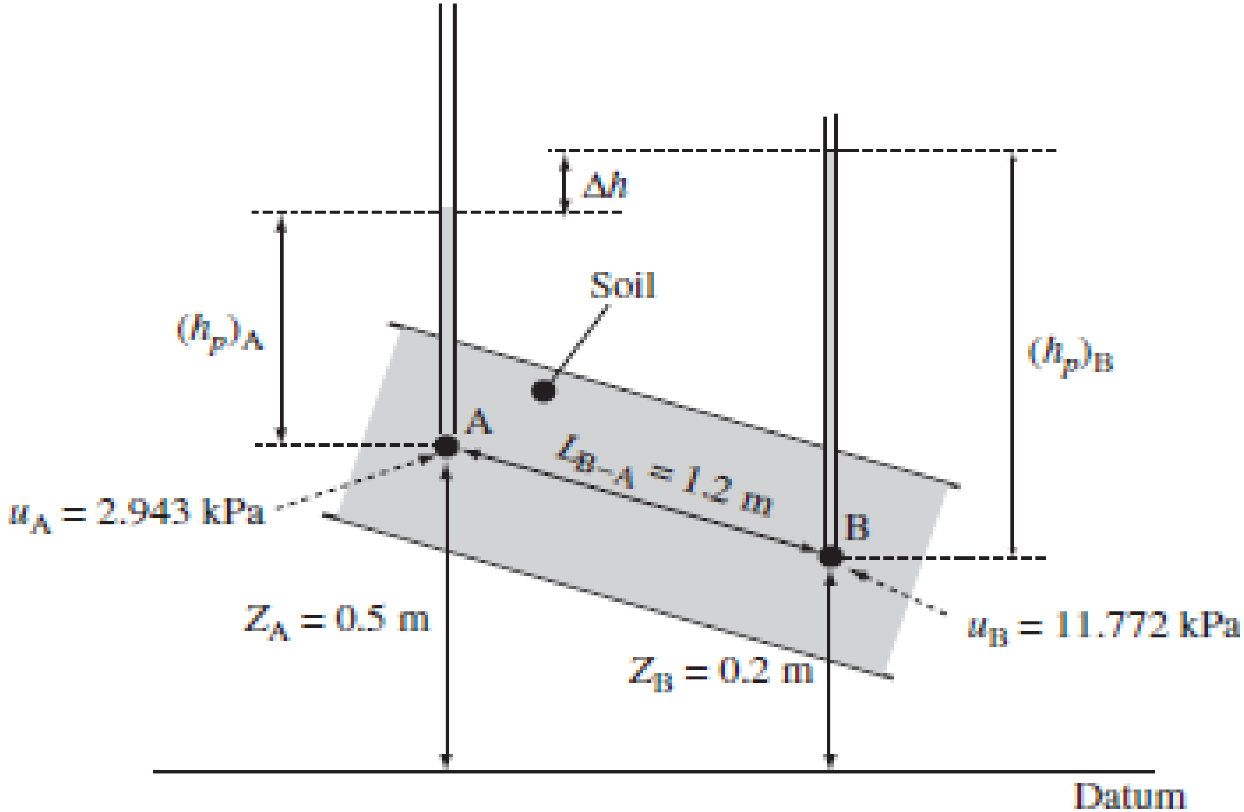


FIGURE 9.5 One-dimensional seepage.

LABORATORY DETERMINATION OF PERMEABILITY

- Constant head permeability test
- Variable head permeability test

- Permeability range for this test – 10^{-3} to 10^{-7} cm/sec

CONSTANT HEAD PERMEABILITY TEST :

- The coefficient of permeability of a coarse-grained soil can be determined in the laboratory using a constant-head permeability test.

- The test includes a cylindrical soil specimen that is subjected to a constant head as shown in Figure 9.6.

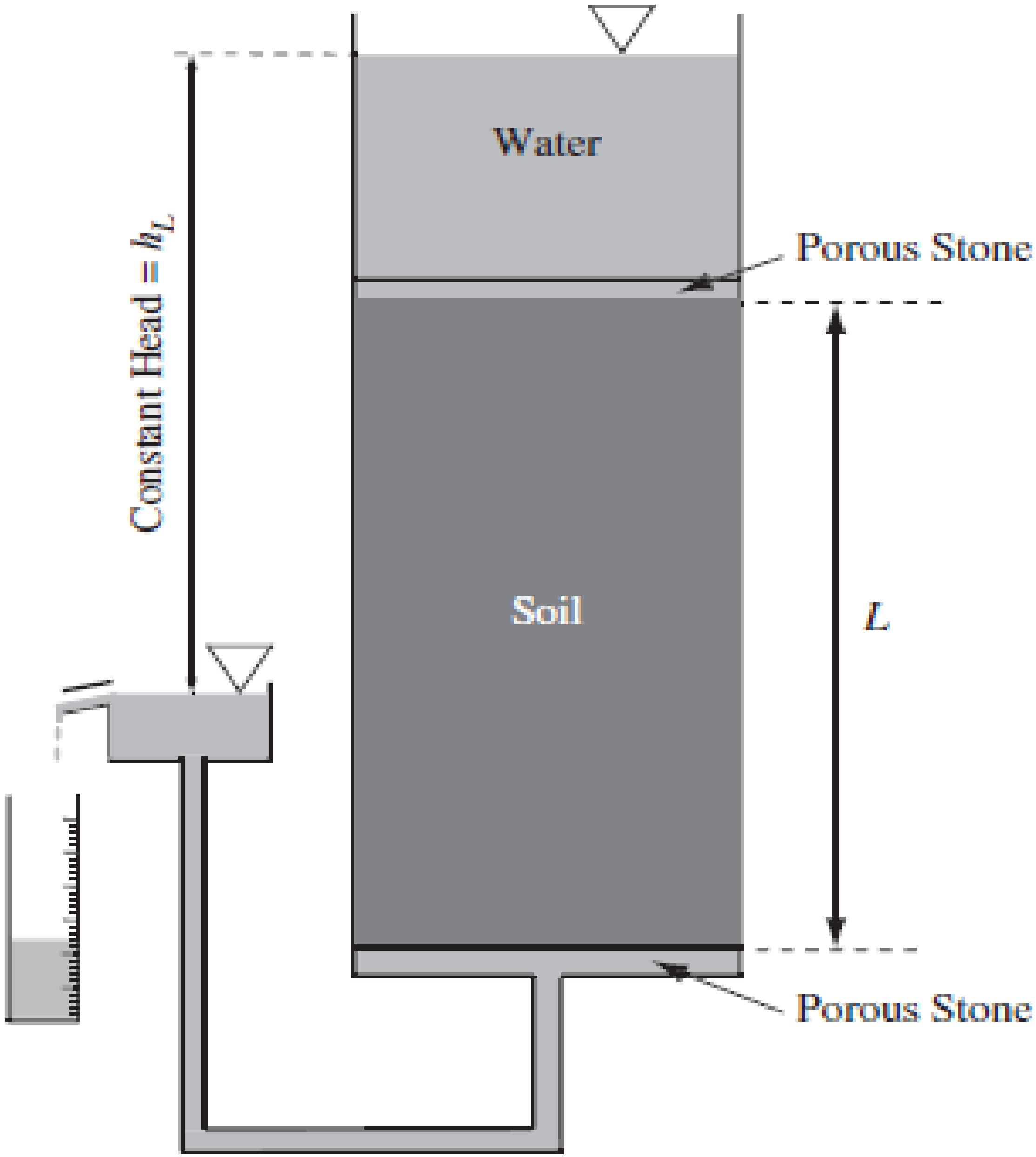


FIGURE 9.6 Constant-head permeability test.

CONSTANT HEAD PERMEABILITY TEST

- The length of the soil specimen is L and its cross-sectional area is A.

- The total head loss (h_L) along the soil specimen is equal to the constant head, which is the difference in elevation between the water levels in the upper and lower reservoirs as shown in the figure.

- A constant head implies that we have reached a steady-state condition in which the flow rate is constant (i.e., does not vary with time). Using a graduated flask, we can collect a volume of water (Q) in a period of time (t). From this we can calculate the flow rate q(= Q/t).

LABORATORY DETERMINATION OF PERMEABILITY BY

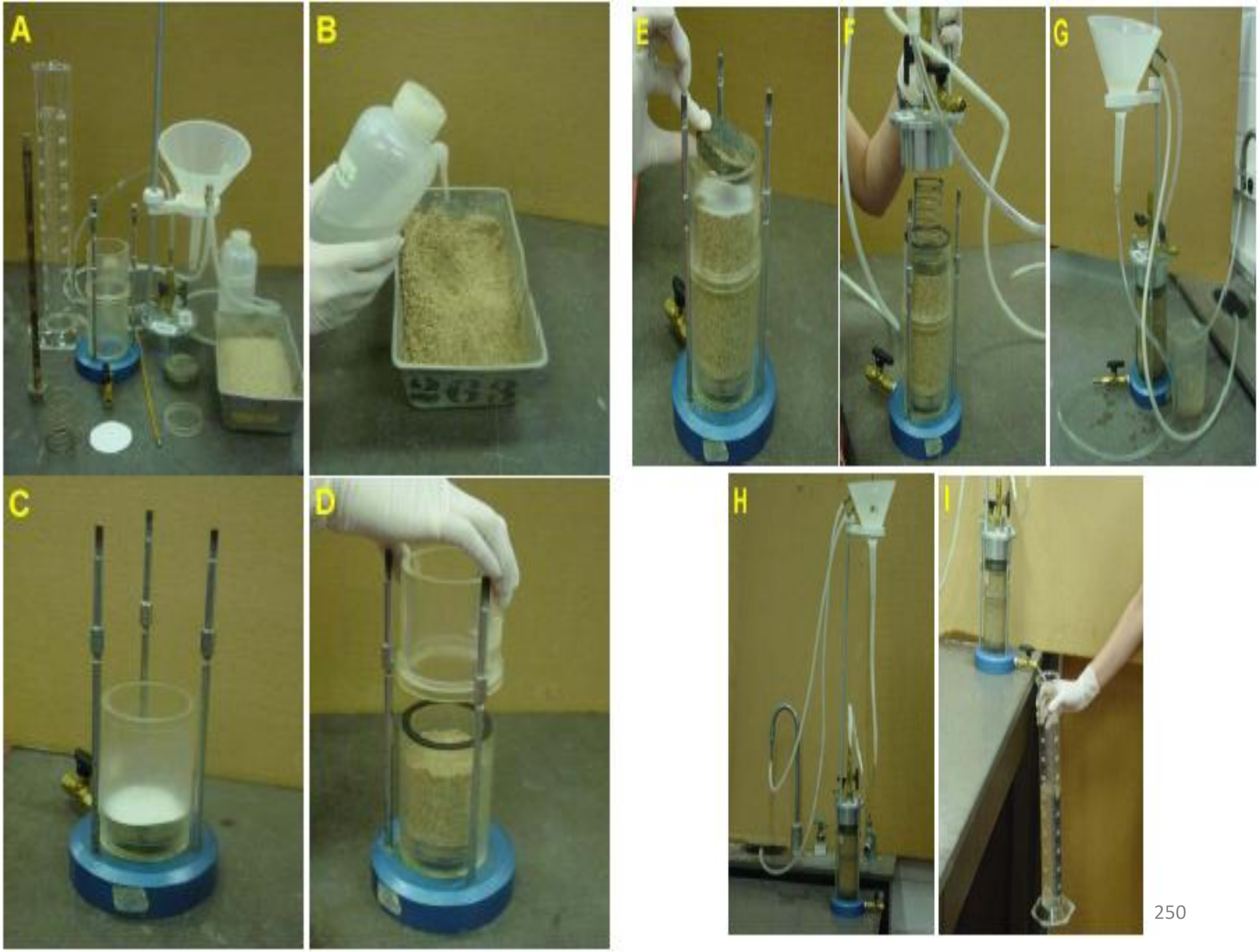
CONSTANT HEAD PERMEABILITY TEST (IS 2720 PART 17)

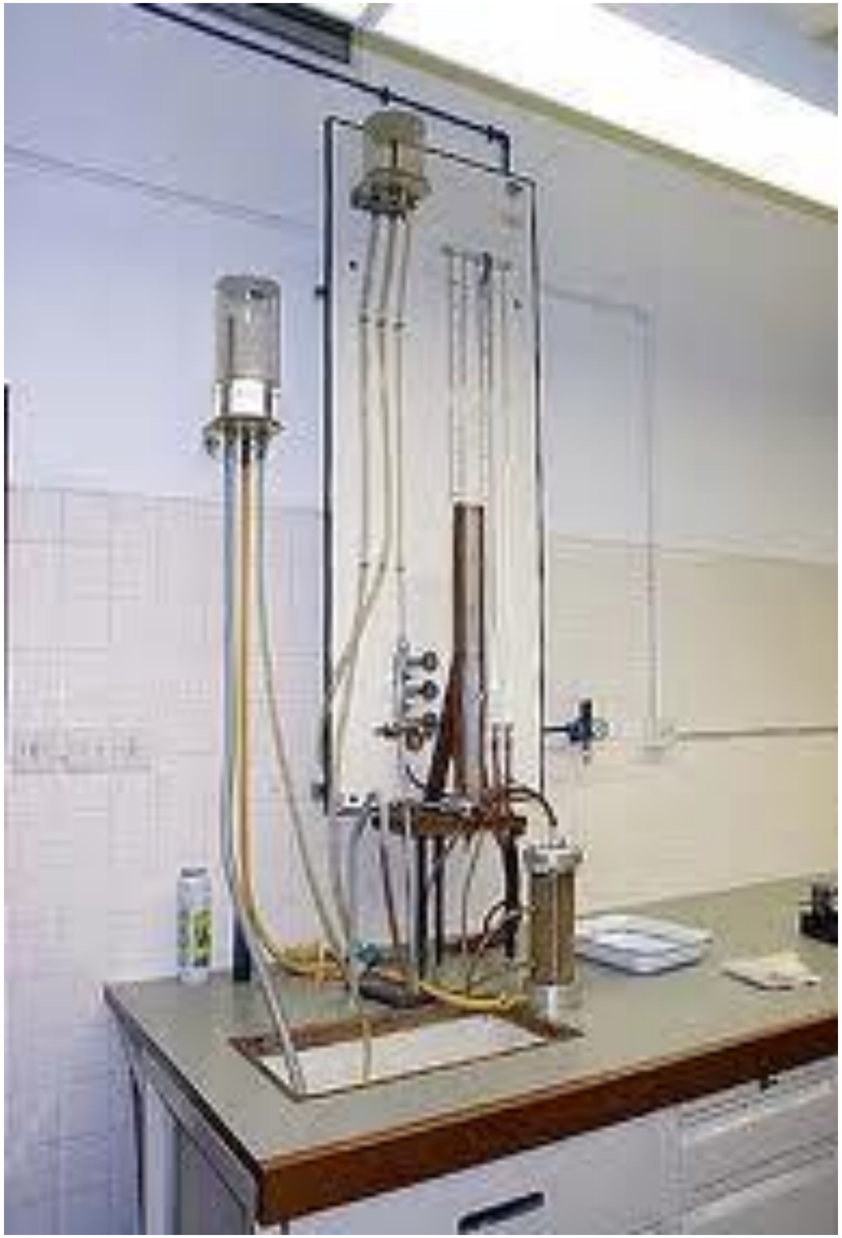
PREPARATION OF TEST **SPECIMEN:**

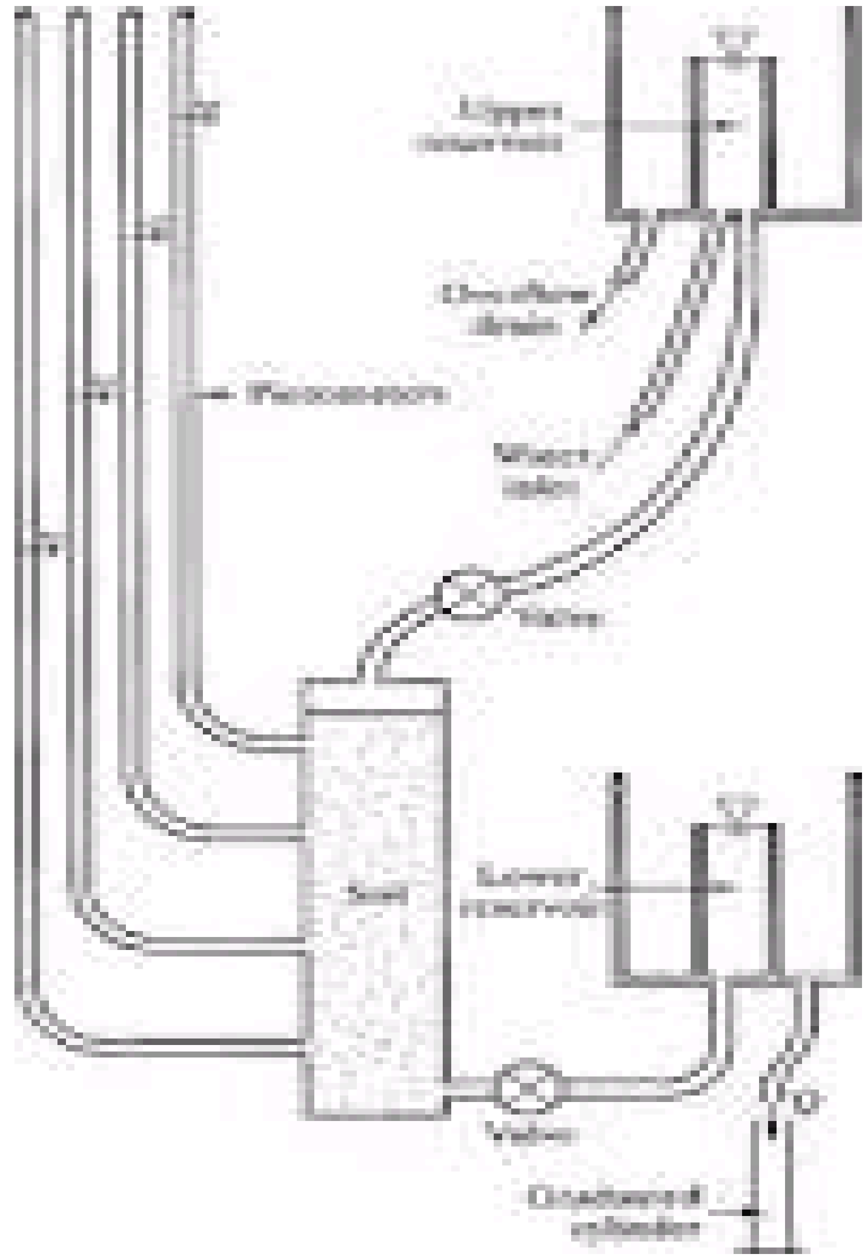
- A 2.5-kg sample shall be taken from a thoroughly mixed air-dried or oven-dried material.

- The moisture content of the 2.5-kg sample shall be determined.

- The sample shall be placed in an airtight container. The quantity of water to be added to the stored sample to give the desired moisture content shall be computed and spread evenly over the sample, and after thoroughly mixing, the material shall again be placed in the storage container. The moisture content of the sample shall again be determined and the entire process repeated until the actual moisture content is within 0.5 percent of that desired.


A
B
C
D
E
F
G
H
I





LABORATORY DETERMINATION OF PERMEABILITY BY

CONSTANT HEAD PERMEABILITY TEST (IS 2720 PART 17)

PREPARATION OF TEST **SPECIMEN:**

- The permeameter mould shall be weighed empty to the nearest gram. After greasing lightly the inside of the mould it shall be clamped between the compaction base plate and the extension collar. The assembly shall be kept on a solid base.

- The mould with the specimen inside shall be assembled to the drainage base and cap having porous discs. The porous discs shall be saturated before assembling the mould.

LABORATORY DETERMINATION OF PERMEABILITY BY

CONSTANT HEAD PERMEABILITY TEST (IS 2720 PART 17)

PROCEDURE :

- For a constant head test arrangement, the specimen shall be connected through the top inlet to the constant head water reservoir. The bottom outlet shall be opened and when the steady state of flow has been established, the quantity of flow for a convenient time interval shall be collected and weighed or measured.

- Alternatively, the inlet may be at the bottom and water may be collected from the outlet at the top. The collection of the quantity of flow for the same time interval shall be repeated thrice.

PROCEDURE :

- The linearity (of Darcy's law) between the hydraulic gradient and the average velocity of flow for the soil under test should be established by performing the test over a range of hydraulic gradients.

- The hydraulic gradients in the permeability test should preferably include the hydraulic gradient likely to occur in the field and any deviation

RECORD OF OBSERVATION:

- The inside diameter and the height of the permeameter are measured and recorded as diameter D and length ***L of the specimen*** loss h. The heights ***H_1 and H_2 are measured to determine the head*** The temperature of water ***T is also measured and recorded.***

- During the test, observations are made of

RECORD OF OBSERVATION:

- For the purpose of getting a quantitative description of the state of the sample, after the test, the weight of wet soil specimen W, is measured and recorded. Its dry weight W, is measured after drying for 24 hours.

- The water content, w is computed and noted. From the knowledge of the specific gravity G, of

CALCULATIONS:

The permeability ***K_T at temperature T is calculated as:***

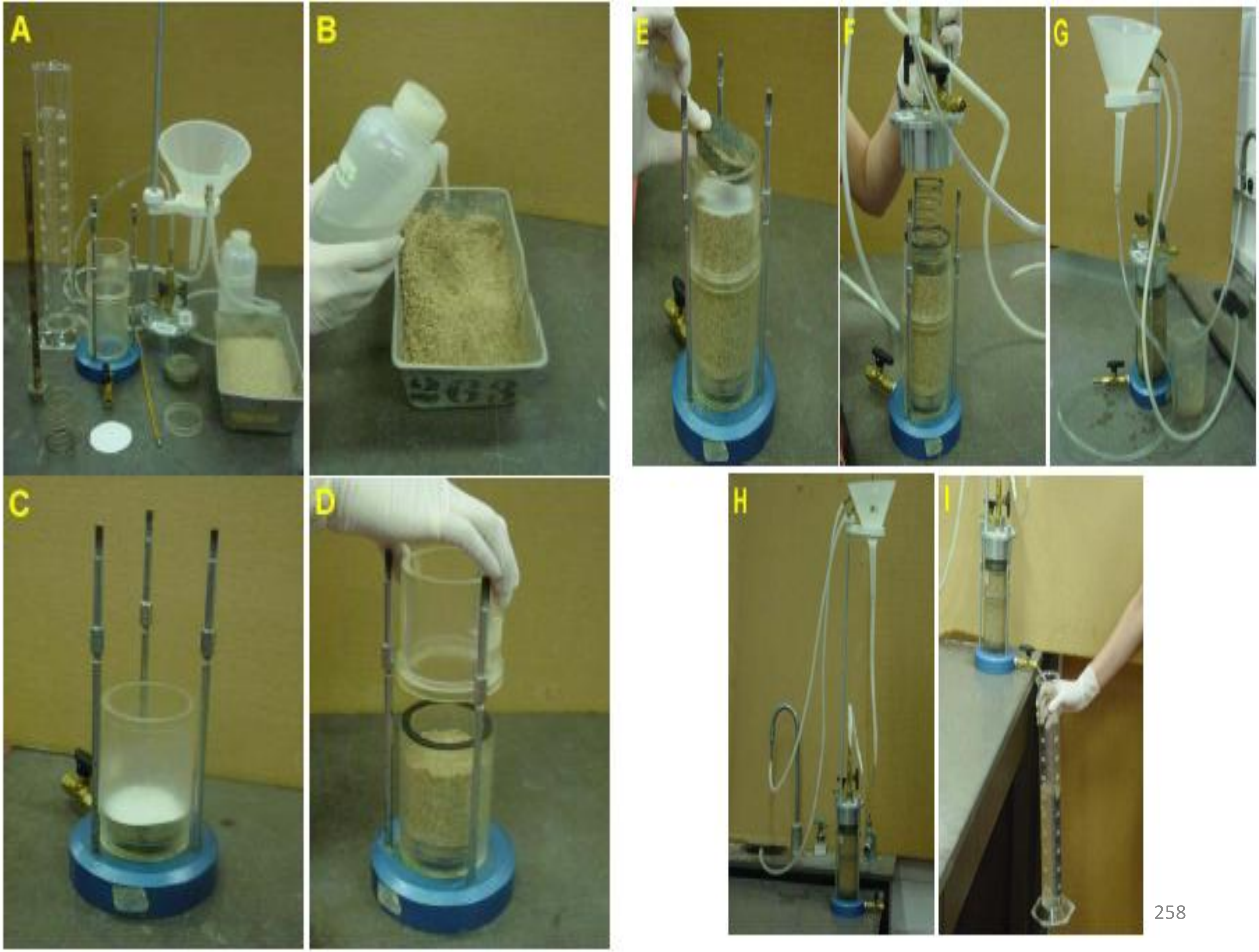
$$K_T = Q / Ait$$

Q -quantity in cm ^{3},

A- area of specimen in cm^2

i -hydraulic gradient, and

t- time in seconds.


A
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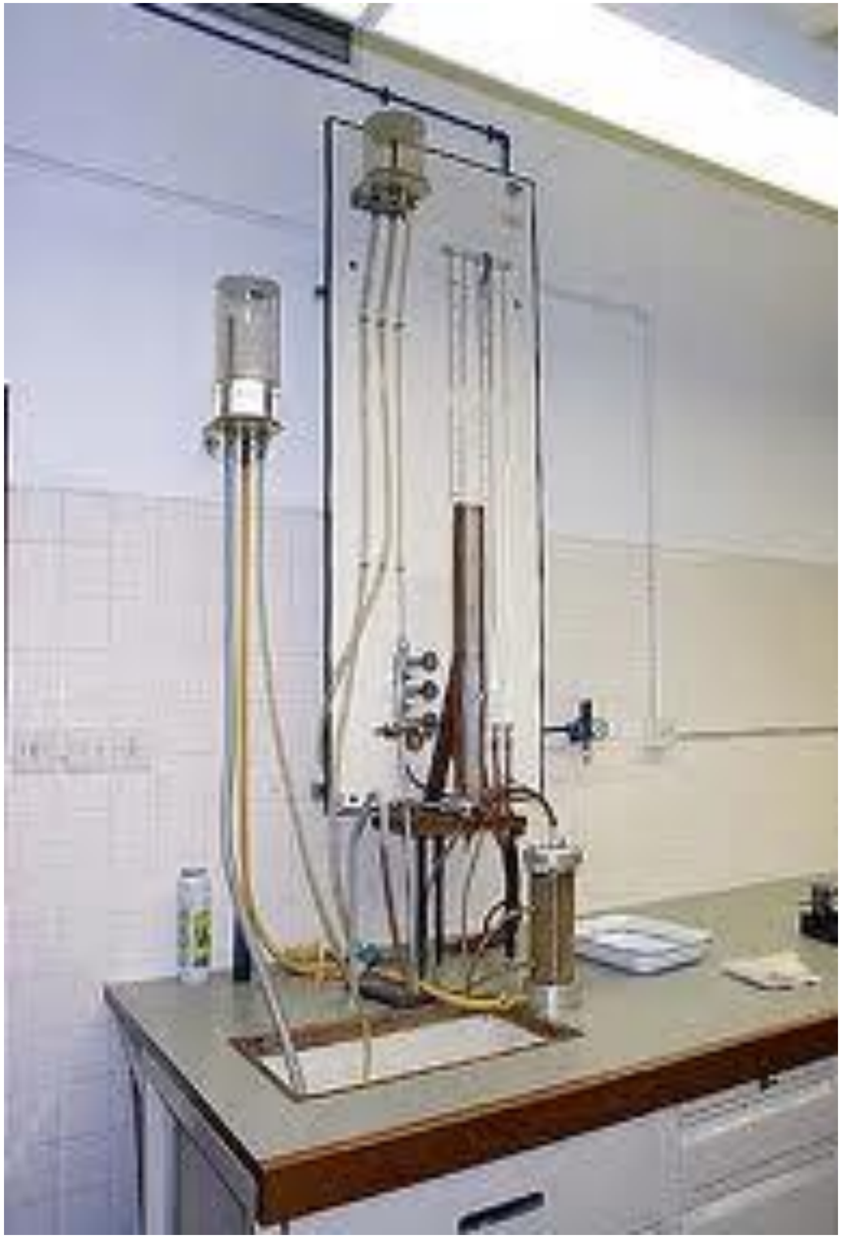
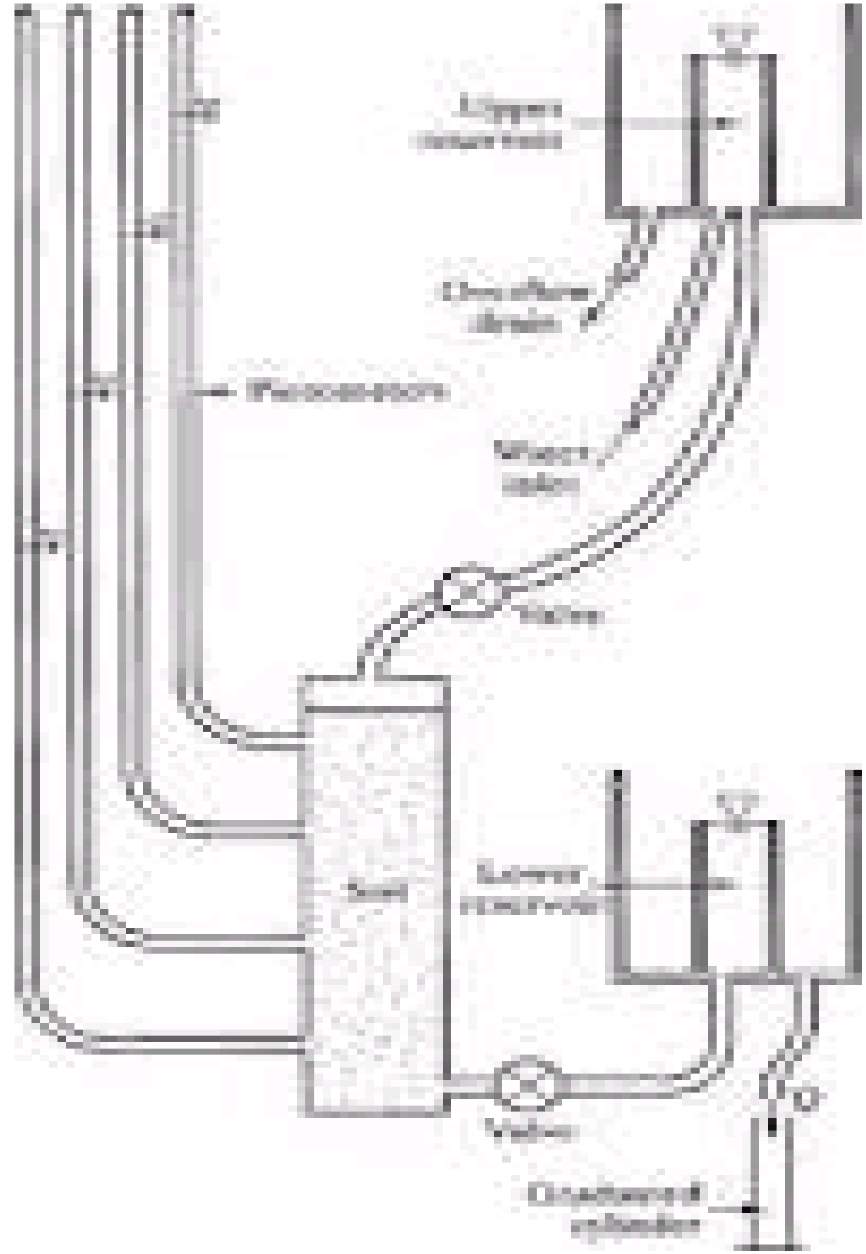
➢DETERMINATION OF COEFF PERMEABILITY

BY CONSTANT HEAD TEST

❖*APPARATUS*

1. **Permeater mould with 100mm dia. 127.3 mm high.**
2. **accessories of the permeater like cover, base, detachable collar , porous stones, dummy plate.**
3. **compaction hammer 2.6kg**
4. **constant head water tank .**
5. **I.S. sieve 4.75mm**





➢procedure

1) Take 2.5 kg of oven dried soil passing 10mm i.s. sieve and add water to bring the moisture content to the desired level.

2) Apply grease inside the mould , base plate and collar.

3) clamp the mould between compaction base plate extension collar.

4) Prepare soil specimen by filling it in 3 layers , each layer rammed 25 times with a 2.6 kg rammer filling through 310 mm.

5) Remove the collar and excess soil.

6) Determine the dry density of the soil specimen those prepared .

7) **Saturate the porous discs in boiling in water.**

8) **Covers the specimen by filling at both ends with filter paper and assemble the mould with soil specimen to the drainage base and cup having porous discs.**

9) **Immerse the mould with the specimen in a water tank for saturation 12-24 hours.**

10) **Connect the specimen through the top inlet to the constant head water tank.**
11) **Open the outlet and allow de-aired water to flow till a steady flow is established.**
12) **Measure the head causing flow 'h'**
13) **Collect the quantity of water (Q) in a measuring cylinder for convenient time interval (t) for this head 'h'.**
14) **Calculate the coeff of permeability (k).**
15) **Record the test temp.**

Soil type	Limits of grain size, mm	Size at which permeability is measured, mm	Coefficient of permeability	
			cm/sec	ft/yr
Gravel		4	1	10^6
—	2.0			
Sand		0.6	10^{-2}	10^4
—	0.06	0.06	10^{-4}	10^2
Silt		0.008	10^{-6}	1
—	0.002			
Clay		0.001	10^{-8}	10^{-2}

Coefficients of Permeability of Clay Minerals

Clay mineral	Exchangeable cation	Void ratio	Coefficient of permeability, cm/sec
Montmorillonite	Na	15	8×10^{-8}
		3	1.5×10^{-8}
	K	11	5×10^{-8}
		7	8×10^{-9}
	Ca	8	1×10^{-5}
		4	1×10^{-7}
	H	9	2×10^{-6}
		3	1×10^{-7}
Kaolinite	Na	1.6–0.5	1.5×10^{-6} to 8×10^{-7}
	K	1.6–1.1	3×10^{-6} to 9×10^{-7}
	Ca	1.6–1.3	1×10^{-5} to 1.5×10^{-6}
	H	1.4–1.0	1×10^{-5} to 1.5×10^{-6}

Permeabilities obtained by falling-head test on samples in consolidation apparatus. Results indicate the following:

For montmorillonite at void ratio 8 the order of permeability in terms of the exchangeable ion present is

$$K < Na < H < Ca;$$

for kaolinite at void ratio 1.5 the order is

$$Na < K < Ca < H.$$

At a given value of consolidation pressure (effective stress), the order of permeability is

$$Na < K < Mg < Ca.$$

Dispersion has the effect of considerably decreasing the permeability of a soil.

Quiz

1. What is permeability?
2. What is the unit of permeability ?
3. Use of permeability in soil mechanics?
4. What is Darcy's law ?
5. Darcy's law is applicable for which kind of flow?
6. Factor affecting on permeability?
7. What is the different method to determine the permeability in laboratory?

QUIZ

8. What is constant head method to find the permeability ?

9. What is the variable head method to find the permeability?

10. With the help Constant head method we determine the permeability of …………soil.

11. With the help of variable head method we determine the permeability of …..soil.

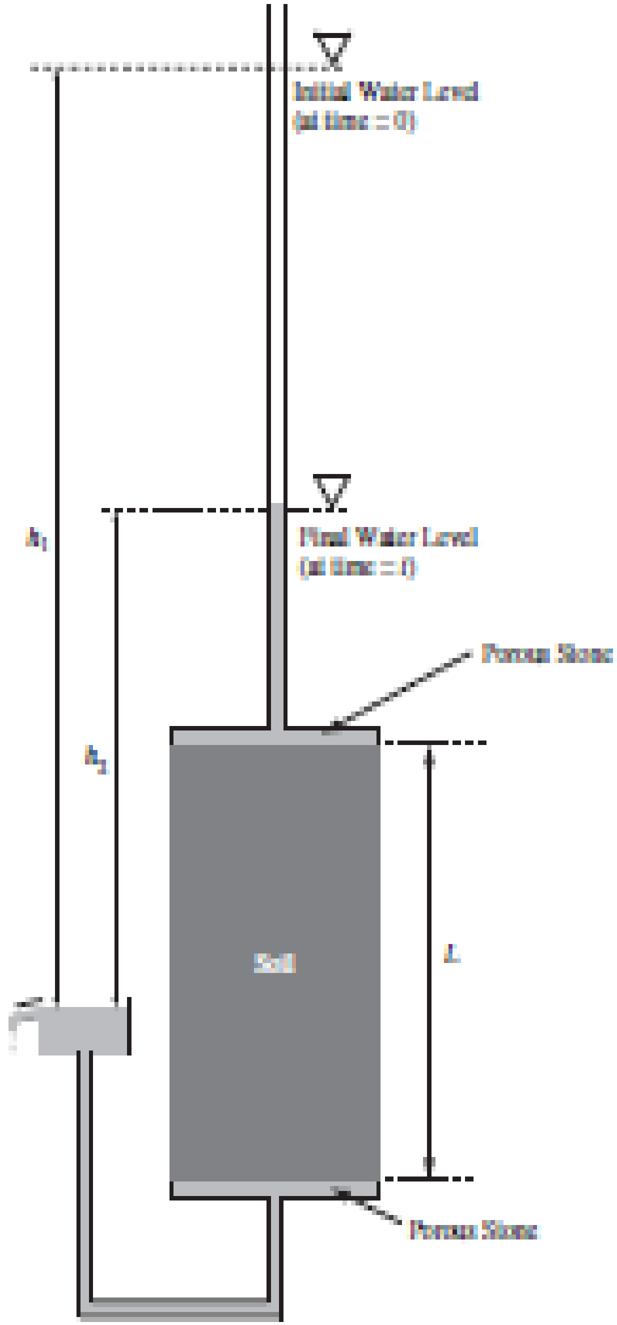
Temperature **- *t* -** **(°C)**	**Density** **- *ρ* -** **(kg/m³)**	**Specific Weight** **- *γ* -** **(kN/m³)**
0	**999.8**	**9.806**
4	**1000**	**9.807**
10	**999.7**	**9.804**
20	**998.2**	**9.789**
30	**995.7**	**9.765**
40	**992.2**	**9.731**
50	**988.1**	**9.690**
60	**983.2**	**9.642**
70	**977.8**	**9.589**
80	**971.8**	**9.530**
90	**965.3**	**9.467**
100	**958.4**	**9.399**

LABORATORY DETERMINATION OF PERMEABILITY BY FALLING HEAD PERMEABILITY TEST (IS 2720 PART 17)

PROCEDURE:

- For a falling head test arrangement the specimen shall be connected through the top inlet to selected stand-pipe. The bottom outlet shall be opened and the time interval required for the water level to fall from a known initial head to a known final head as measured above the centre of the outlet shall be recorded.

- The stand-pipe shall be refilled with water and the



Initial Water Level
(at time = 0)
Final Water Level
(at time = t)
Porous Stone
h_1
h_2
Soil
L
Porous Stone

LABORATORY DETERMINATION OF PERMEABILITY BY

FALLING HEAD PERMEABILITY TEST (IS 2720 PART 17)

PROCEDURE:

- Alternatively, after selecting the suitable initial and final heads, h_1 and h_2 respectively, time intervals shall be noted for the head to fall from h_1, to $\sqrt{h_1 h_2}$, and similarly from $\sqrt{h_1 h_2}$ ***to*** h_2. ***The time intervals should be the same;*** otherwise the observation shall be repeated after refilling the stand-pipe.

LABORATORY DETERMINATION OF PERMEABILITY BY

FALLING HEAD PERMEABILITY TEST (IS 2720 PART 17)

RECORD OF OBSERVATION:

The dimensions of specimen, length L and diameter D, are measured.

Area a of stand-pipe is recorded.

The temperature T, of water is also measured and recorded.

During the test, observations are made of initial time t_i, final time t_f initial head h_1 final head h_2

LABORATORY DETERMINATION OF PERMEABILITY BY FALLING HEAD PERMEABILITY TEST (IS 2720 PART 17)

RECORD OF OBSERVATION:

The permeability K_T is calculated and recorded.

At the end of the test, the weight of wet soil specimen Wt is measured and recorded. Then the sample is dried in the oven for 24 hours and the dry weight W_s is measured and recorded. The water content, W is computed and noted. Void ratio, e, and degree of saturation S are

CALCULATIONS :

At temperature **T of water, the permeability K_T is calculated as:**

$$kT = 2.303\ aL / A\ (\mathbf{t_f - t_i}) \quad * \quad \mathbf{log_{10}}\ (h_1/h_2)$$

- **In the FLOW of WATER in SOILS**

$$Q = \frac{kA}{\ell}(h_1 - h_2)$$

Where Q is the water flow rate
h_1 is the inlet head
h_2 is the outlet head
A is the cross-section area
k is the permeability
ℓ is the path length

1) **Mathematical solutions**
   a) **exact solutions for certain simple situations**
   b) **solutions by successive approximate - e.g. relaxation methods**
2) **Graphical solutions**
3) **Solutions using the electrical analogue**
4) **Solutions using models**

Only graphical methods will be used in this course

FIELD TEST FOR DETERMINATION OF PERMEABILITY(IS 5529 PART 1)

The field permeability test carried out to determine the permeability of each surface strata encounter up to bed rock as well as to ascertain overall permeability of strata. This test are carried out in standard drill holes where subsurface exploration for foundation are carried out by drilling.

The test also carried out in auger holes or bore holes of larger size for depth up to 30m.

The test carried out are either pumping in or the pumping out type.

FIELD TEST FOR DETERMINATION OF PERMEABILITY(IS 5529 PART 1)

PUMPING IN TEST (Gravity feed in drill holes or bore holes)

- Constant head method (cased well , open end test)
- Falling head method (uncased well)
- Slug method

PUMPING OUT TEST :

- Unsteady state
- Steady state
- Bailor method

FIELD TEST FOR DETERMINATION OF PERMEABILITY(IS 5529 PART 1)

PUMPING OUT TEST :

The pumping out test is an accurate method for finding out in-situ permeability of the strata below water table or below river sand.

This method is best suited for all ground water problems where accurate values of permeability representative of the entire aquifer are required for designing cut off or planning excavation.

FIELD TEST FOR DETERMINATION OF PERMEABILITY(IS 5529 PART 1)

PUMPING IN TEST:

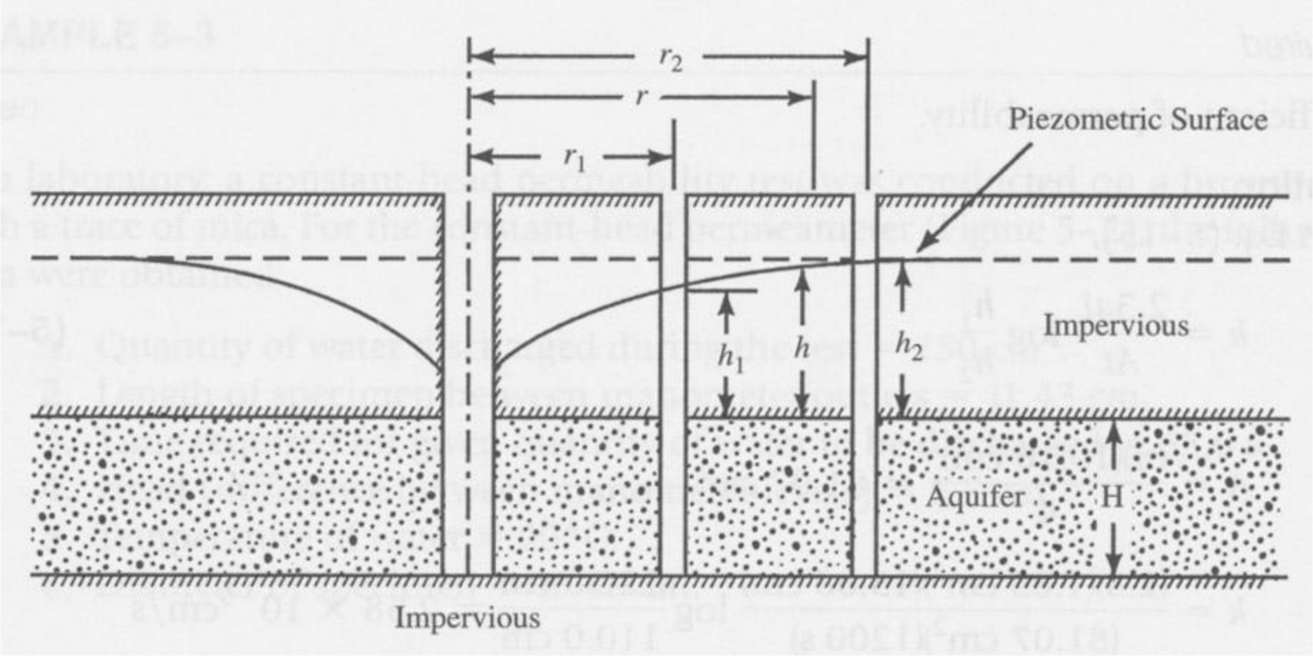
The pumping in test method is applicable for strata above water table.

The test is especially performed in formation of low permeability and limited thickness where adequate in formation of low permeability and limited thickness where adequate yield is not available for pump out test.

Coefficient of Permeability

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

: ______________________________ aquifer

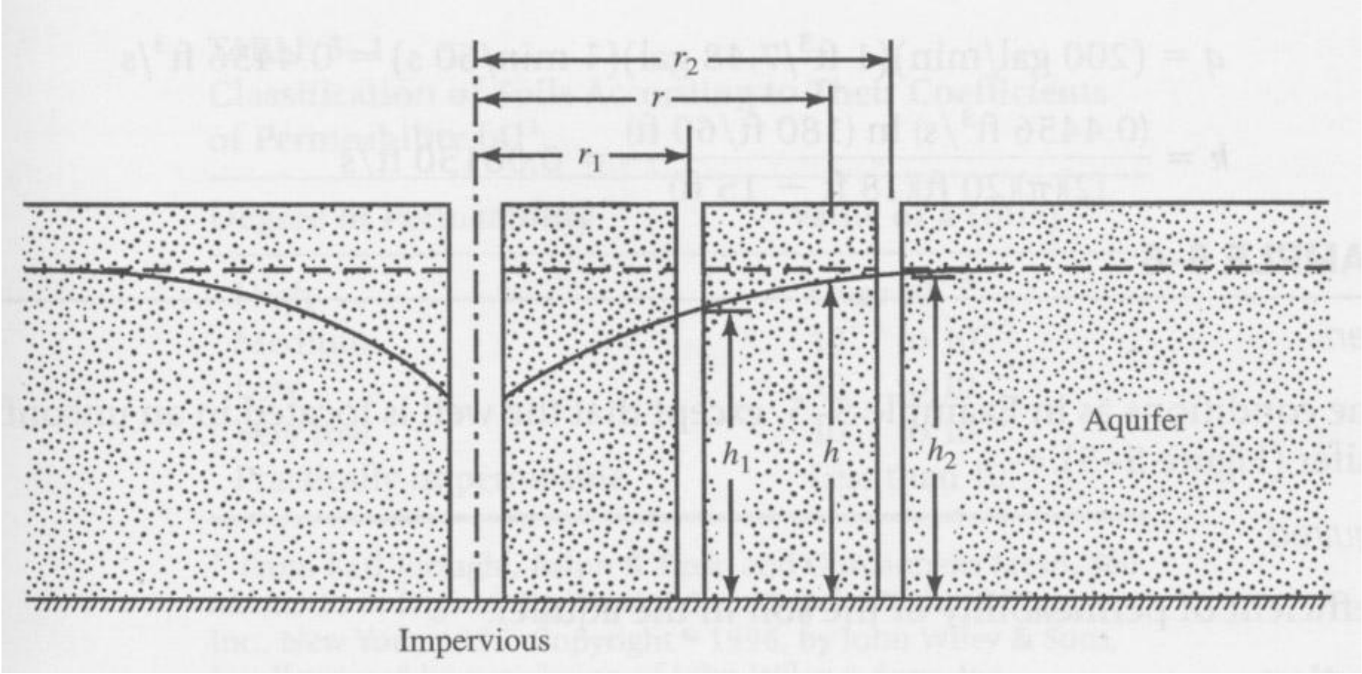


CONFINED FLOW PUMPING TEST

Coefficient of Permeability

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

: ______________________________ aquifer

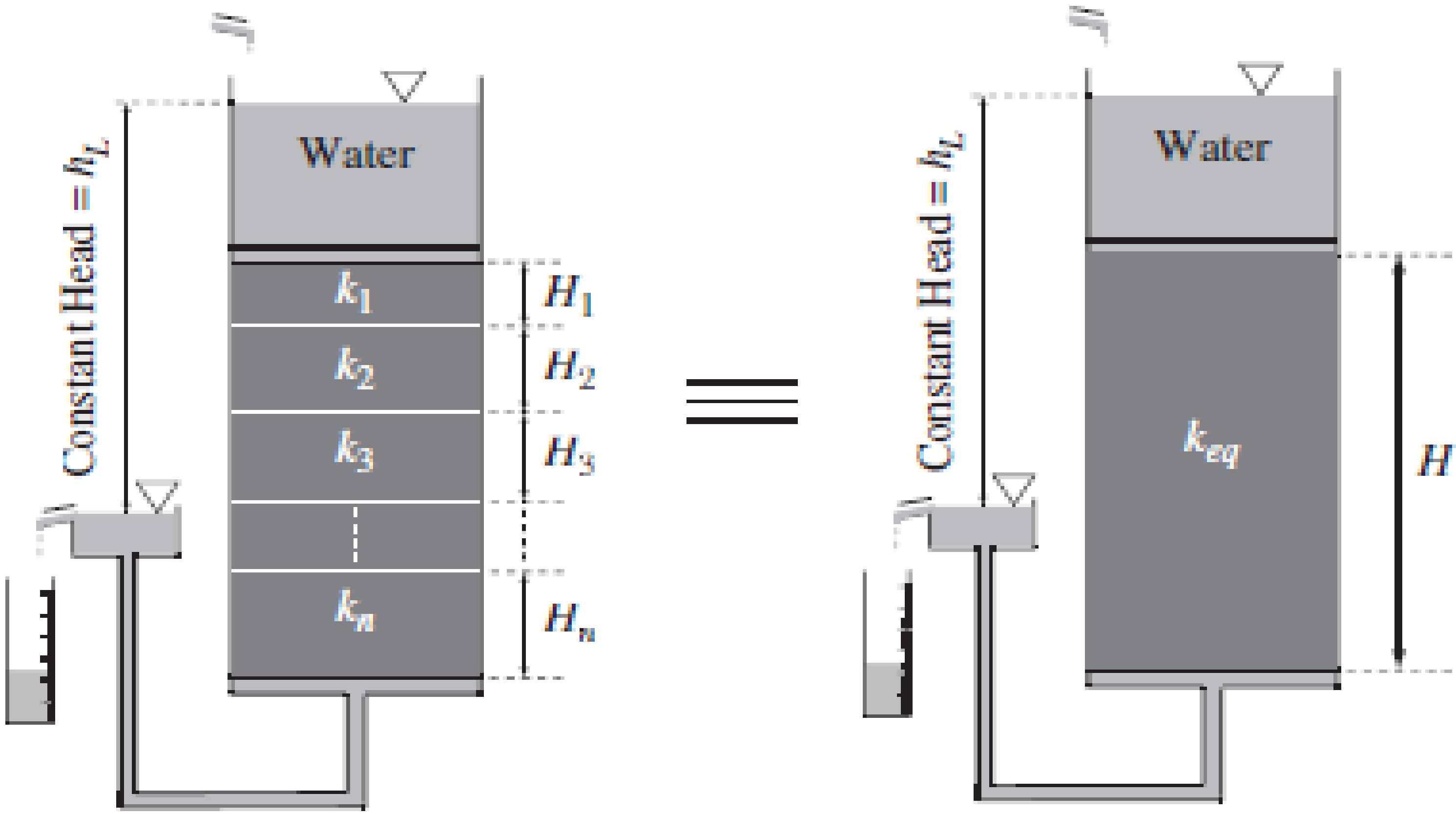


UNCONFINED FLOW PUMPING TEST

PERMEABILITY OF STRATIFIED SOILS

- Consider a stratified soil having horizontal layers of thickness H_1,H_2,H_3, . . .,H_n with coefficients of permeability k_1, k_2, k_3, . . , k_n, as shown in Figure .

- For flow perpendicular to soil stratification, as shown in the figure, the flow rate q through area A of each layer is the same. Therefore, the head loss across the n layers is given as



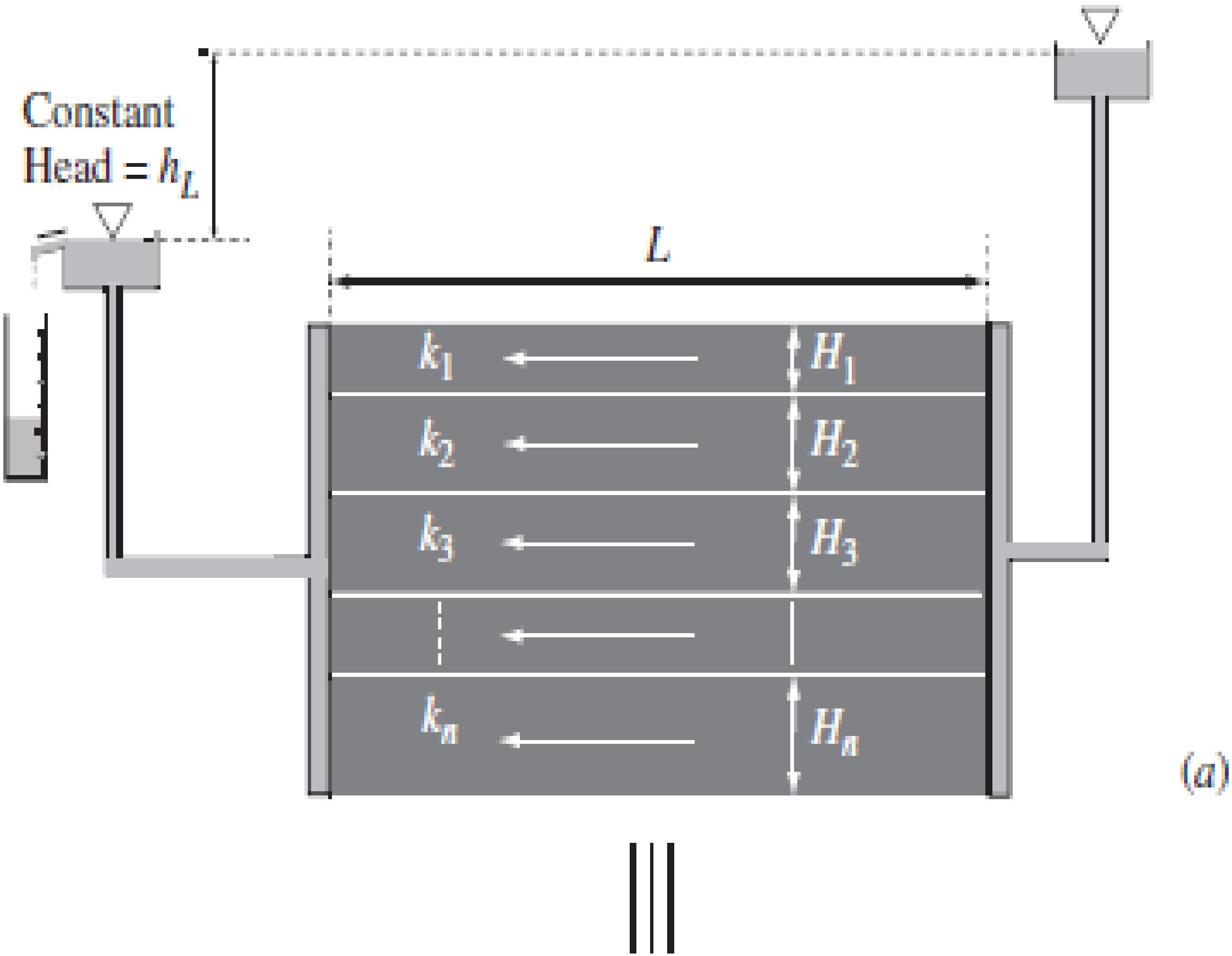
(a) (b)

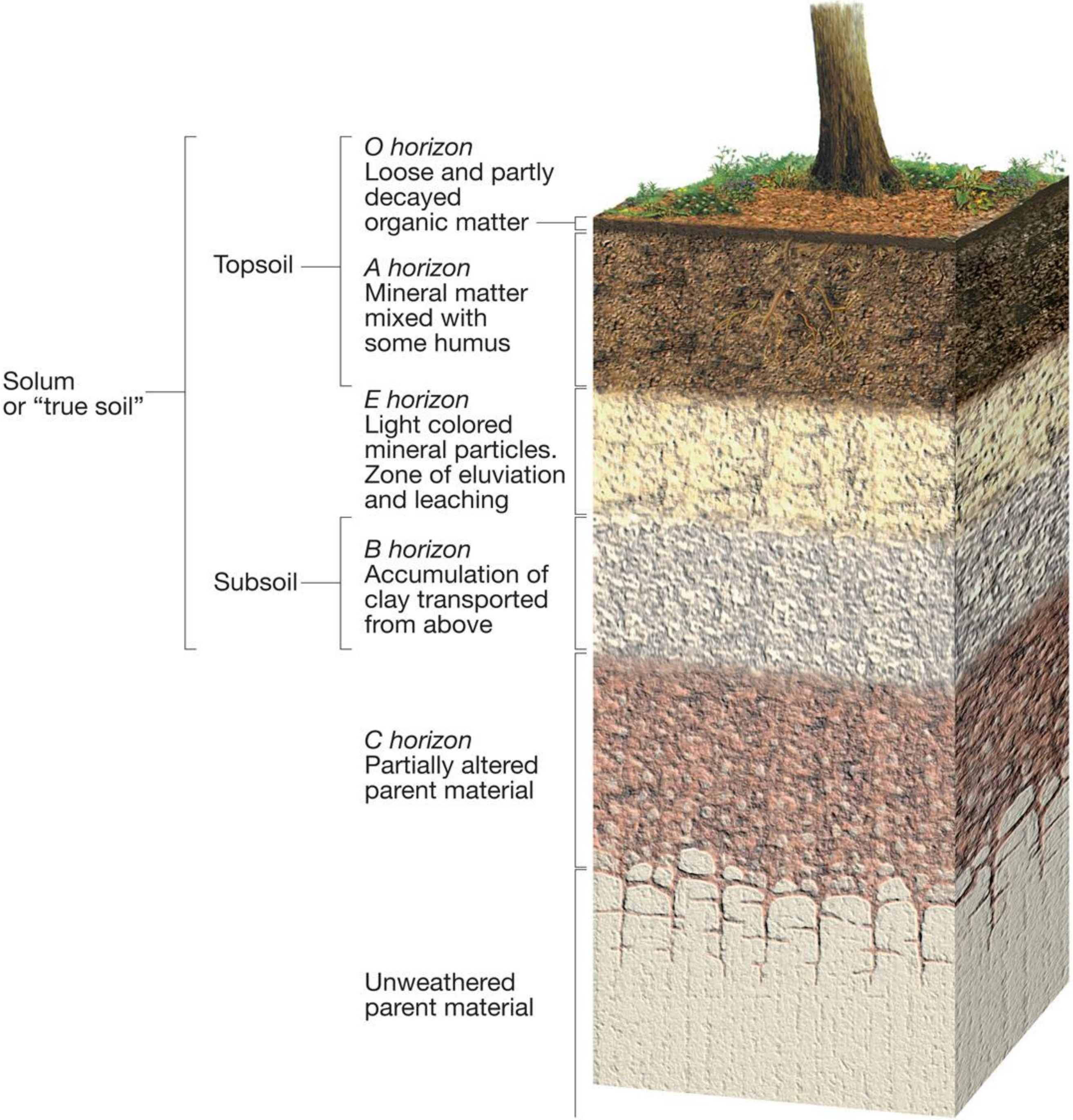
$$k_{eq} = \frac{H}{H_1/k_1 + H_2/k_2 + H_3/k_3 + \cdots + H_n/k_n}$$

PERMEABILITY OF STRATIFIED SOILS

- For a flow that is parallel to soil stratification, such as the one shown in Figure , the head loss h_L over the same flow path length L will be the same for each layer. Thus, $i_1 = i_2 = i_3 = \cdot \cdot \cdot = i_n$. The flow rate through a layered system (with width = 1 unit) is

$$k_{eq} = \frac{k_1 H_1 + k_2 H_2 + k_3 H_3 + \cdots + k_n H_n}{H}$$

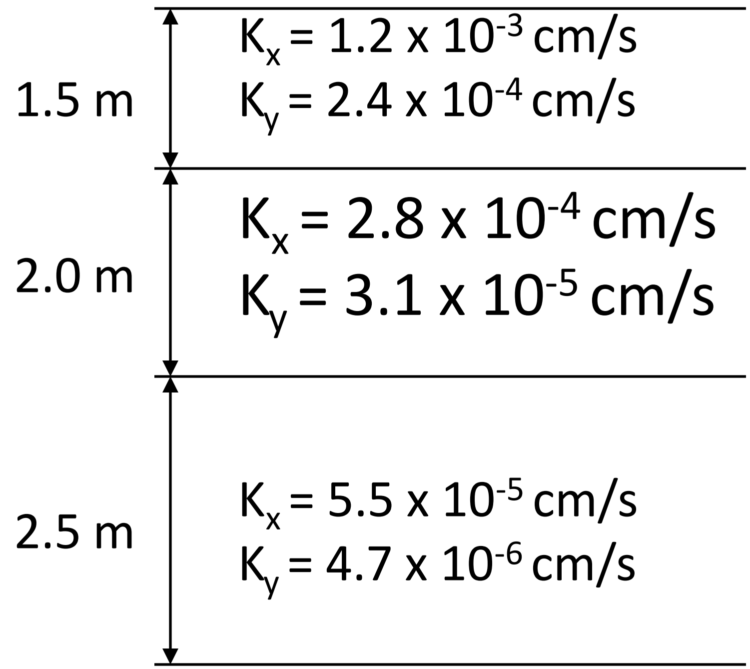

Constant
Head = h_L
L
k_1
H_1
k_2
H_2
k_3
H_3
k_n
H_n
(a)



An idealized soil profile

Example

- A nonhomogeneous soil consisting of layers of soil with different permeabilities.
- Average coefficient of permeability in the horizontal direction and



Quiz

- The permeability of soil varies …………..

- The maximum particle size for which Darcy's law in applicable is ………..

- The coefficient of permeability of clay is generally ……….

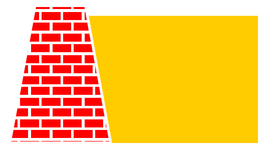
- Constant head method is used for ………..

- A soil has discharge velocity of 6 x 10^{-7} m/sec and

Soil Compaction Theory

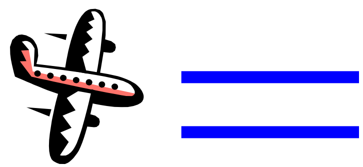
Compaction

- **Soil is used as a basic material for construction**
  - *Retaining walls,*

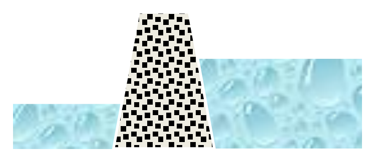


  - *Highways, Embankments, Ramps*

  - *Airports,*



  - *Dams, Dikes, etc.*



- **The advantages of using soil are:**

1. *Is generally available everywhere*
2. *Is durable - it will last for a long time*
3. *Has a comparatively low cost*

What is Compaction?

- In most instances in civil engineering and/or construction practice, whenever soils are imported or excavated and re-applied, they are **compacted**.
- The terms **compaction** and **consolidation** may sound as though they describe the same thing, but in reality they do not.

What is Compaction

Heavy Weight

What do you think about this live compaction machine

- When loose soils are applied to a construction site, compressive mechanical energy is applied to the soil using special equipment to densify the soil (or reduce the void ratio).
- Typically applies to soils that are being applied or re-applied to a site.

What is Consolidation

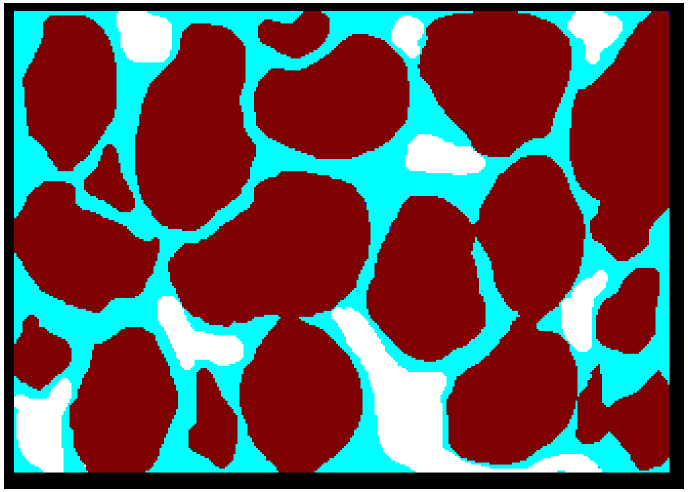
- When a Static loads are applied to saturated soils, and over a period of time the increased stresses are transferred to the soil skeleton, leading to a reduction in void ratio.

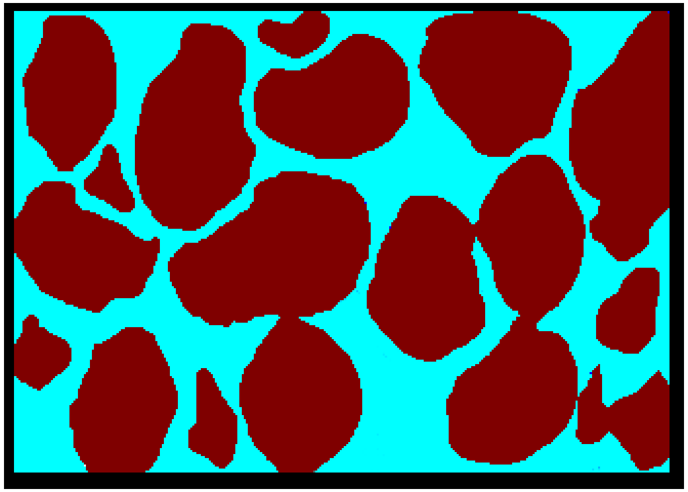
- Depending on the permeability of the soil and the magnitude of the drainage distance, this can be a very time-consuming process.

- Typically applies to existing, undisturbed soil deposits that has appreciable amount of clay.

Compaction - Consolidation

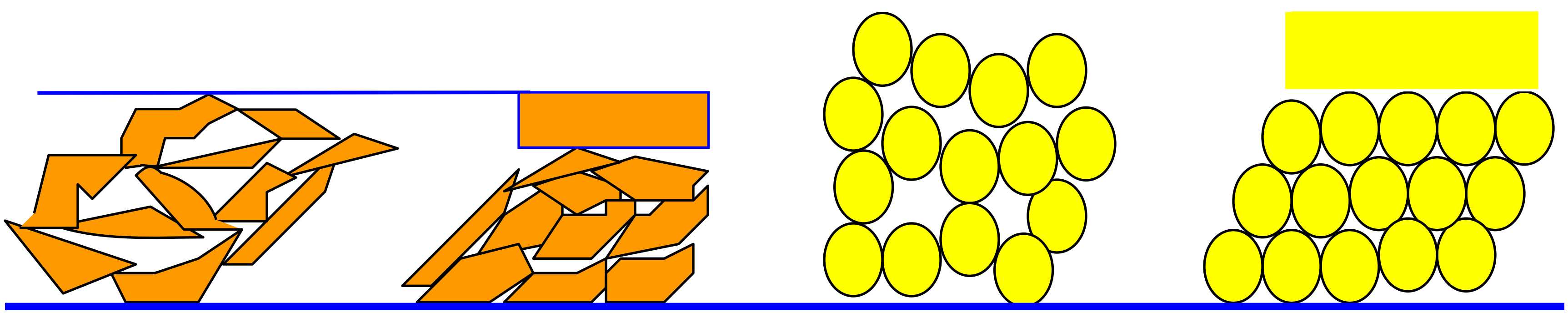
- Compaction means the removal of air-filled porosity.
- Consolidation means the removal of water-filled porosity.





Principles of Compaction

Compaction of soils is achieved by . It is assumed that the compaction process does not decrease the volume of the solids or soil grains.



The Goal of Compaction

Phase Diagram

- Reduce air-void volume V_a in soils as much as is possible.
- For a given water content w, the max. degree of compaction that can be achieved is when all of the air voids have been removed, that is (S=1).
  - Since $S = wG_s/e$, the corresponding void ratio
  - (for S=1) will be: $e = wG_s$

Principles of Compaction

- **The degree of compaction of a soil is measured by the dry unit weight of the skeleton.**

- The dry unit weight correlates with the degree of packing of the soil grains.

Recall that $g_d = G_s g_w/(1+e)$ ·

- The more compacted a soil is:
  - ✓ the smaller its void ratio (e) will be.
  - ✓ the higher its dry unit weight (g_d) will be

Typical Calculation (γ_d)

Phase Diagram

BACK

- **block diagram shown**
- Total Mass $M = M_w + M_s$
- Total Volume $V = V_w + V_s$
- Void ratio $e = V_v / V_s$
- Water content $w = M_w / M_s$
- Saturation $S = V_w / V_v$
- Moist unit weight
  - $\gamma = (M w + M_s) / V$
  - $= (w + 1) M_s / V = (1+w) \gamma_d$
  - $\gamma_d = \gamma / (1+w) =$
  - $\gamma_d = G_s g_w/(1+e)$

- 1) Increased Shear Strength
  - This means that larger loads can be applied to compacted soils since they are typically stronger.
- 2) Reduced Permeability
  - This inhibits soils' ability to absorb water, and therefore reduces the tendency to expand/shrink and potentially liquefy
- 3) Reduced Compressibility
  - This also means that larger loads can be applied to compacted soils since they will produce smaller settlements.
- 4) Control Swelling & Shrinking
- 5) Reduce Liquefaction Potential

Various Types of compaction test

Type of Test	Mould	Hammer mass (kg)	Drop (mm)	No of layers	Blows per layer
BS "Light"	One Liter	2.5	300	3	27
	CBR	2.5	300	3	62
ASTM (5.5lb)	4 in	2.49	305	3	25
	6 in	2.49	305	3	56
BS "Heavy"	One Liter	4.5	450	5	27
	CBR	4.5	450	5	62
ASTM (10lb)	4 in	4.54	457	5	25
	6 in	4.54	457	5	56
BS Vibration hammer	CBR	32 to 41	Vibration	3	1 minute

General Compaction Methods

	Coarse-grained soils	Fine-grained soils
Laboratory	•Vibrating hammer (BS)	•Falling weight and hammers •Kneading compactors •Static loading and press
Field	•Hand-operated vibration plates •Motorized vibratory rollers •Rubber-tired equipment •Free-falling weight; dynamic compaction (low frequency vibration, 4~10 Hz)	•Hand-operated tampers •Sheepsfoot rollers •Rubber-tired rollers
	Vibration	Kneading

The Standard Proctor Test

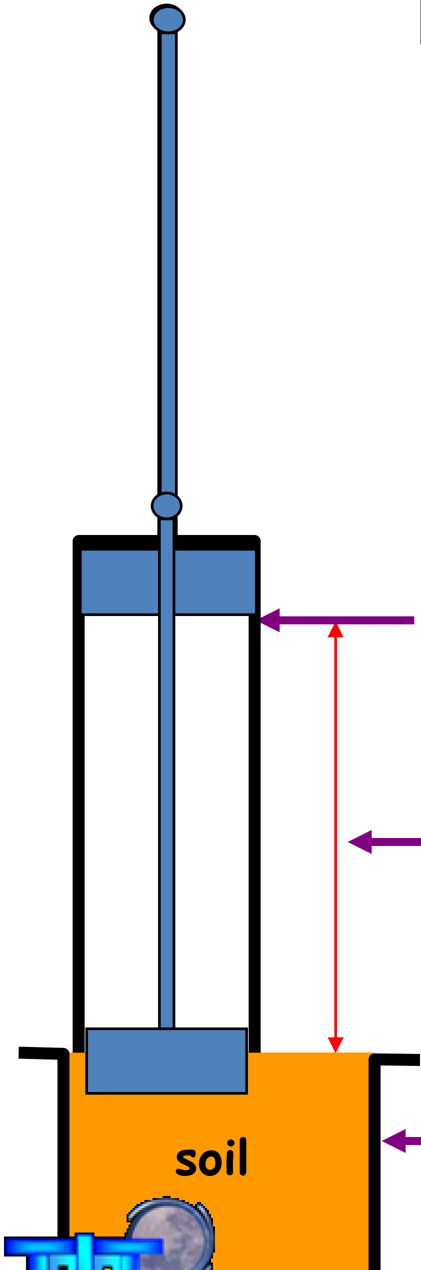
- **R.R. Proctor in the early 1930's was building dams for the old Bureau of Waterworks and Supply in Los Angeles, and he developed the principles of compaction in a series of articles in Engineering News-Record.**

Variables of Compaction

Proctor established that compaction is a function of four variables:

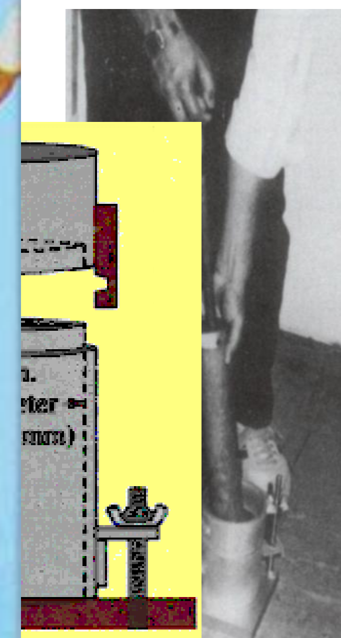
- Dry density (ρ_d) or dry unit weight γ_d.
- Water content w
- Compactive effort (energy E)
- Soil type (gradation, presence of clay minerals, etc.)

The Standard Proctor Test Equipments



Hammer Weight 5.5 lb

Drop Height h=12"



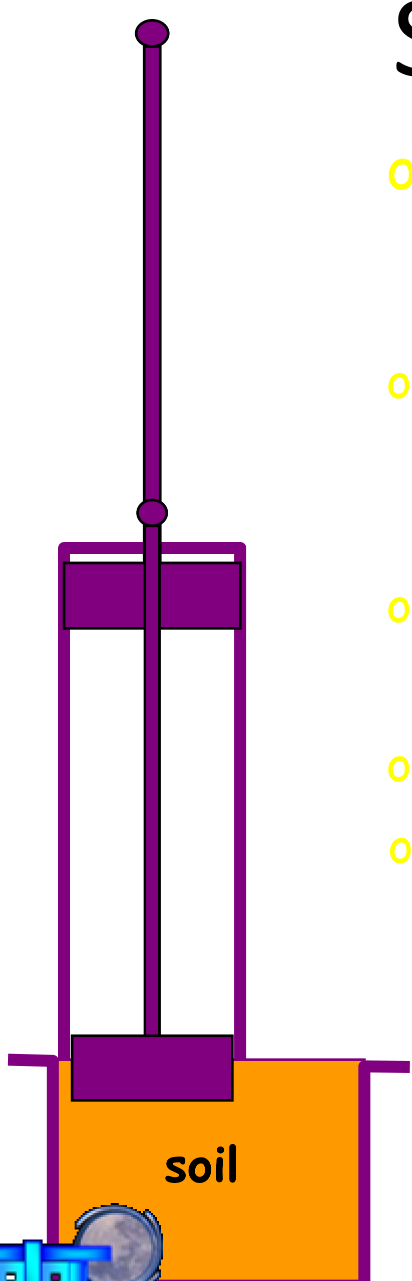
Volume 1/30 ft^3 or 944 cm^3
Diameter 4 in or 10.16 cm
Height 4.584 in or11.643cm

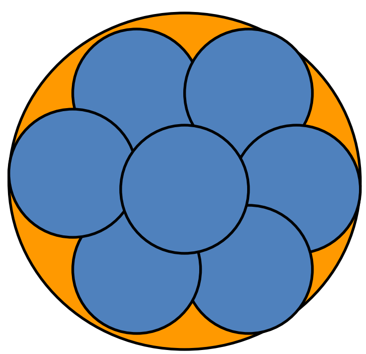
Equipments Needed For Compaction

ASTM D-698 / D-1557		AASHTO T-99 / T-180	
For determining moisture - dnesity relationship.			
SO-351	Standard Proctor Mold	*Machined steel, galvanized, 4" i.d., 4.584" height, 2" height of collar*	1 Pc
SO-352	Standard Proctor Mold	*Machined steel, galvanized, 6" i.d., 4.584" height, 2" height of collar*	1 Pc
SO-353	Standard Proctor Hammer	*Machined steel, galvanized, 2" i.d., 12" drop height, 5,5 lbs weight*	1 Pc
SO-354	Standard Proctor Hammer	*Machined steel, galvanized, 2" i.d., 18" drop height, 10 lbs weight*	1 Pc
SO-355	Extruder	*Steel frame, hydraulic jack*	1 Set
GE-303	Square Pan	*Galvanized steel, l 65 x 65 x 7.5 cm*	1 Pc
GE-390	Thin Box	*Alumunium, 60 gr capacity*	12 Pcs
GE-405A	Graduated Cylinder	*Plastic, 1.000 ml capacity*	1 Pc
GE-801	Scoop	*Cast Alumunium*	1 Pc
GE-871	Trowel	*Pointed type*	1 Pc
GE-890	Straight Edge	*30 cm length*	1 Pc
GE-900	Rubber Mallet	*Wooden handle*	1 Pc
GE-920	Steel Wiire Brush	*Wooden handle*	1 Pc

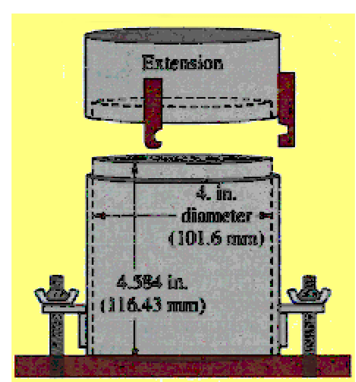
Standard Proctor Test

- The soil is mixed with varying amounts of water to achieve different water contents.
- For each water content, the soil is compacted by dropping a hammer 25 times onto the confined soil
- The soil is in mold will be divided into three **lifts**
- Each **Lift** is compacted 25 times
- This is don 4-6 times from dry-wet





25 Blows/Layer



Layer or lift # 3
Layer or lift # 2
Layer or lift # 1

Standard Energy

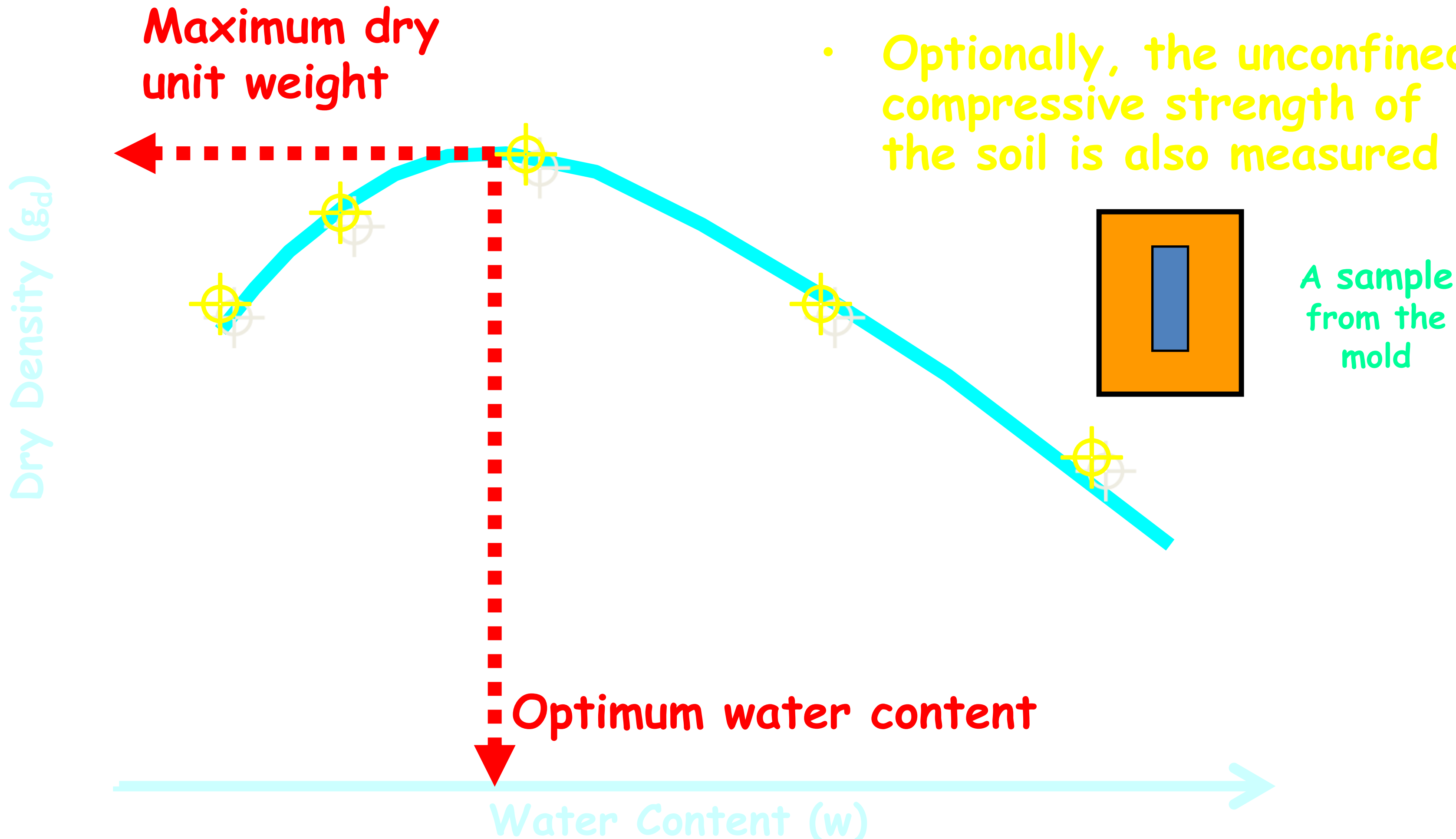
- Compactive (E) applied to soil per unit volume:

$$E = \frac{(\#\ \text{blows/layer}) * (\#\ \text{of layers}) * (\text{hammer weight}) * (\text{height of drop})}{\text{Volume of mold}}$$

$$E_{SP} = \frac{(25\ \text{blows/layer}) * (3\ \text{of layers}) * (5.5\ \text{lbs}) * (1.0\ \text{ft})}{(1/30)\ \text{ft}^3} = 12{,}375\ ft - lb/\ ft^3$$

Results from Standard Proctor Test

- Optionally, the unconfined compressive strength of the soil is also measured



Proctor's principle of compaction

- Using a standard energy, if a series of specimens of a soil are compacted at increasing water contents, the resultant dry density of the specimens will vary. The density will increase to a peak value, then decrease.

Principle of Compaction

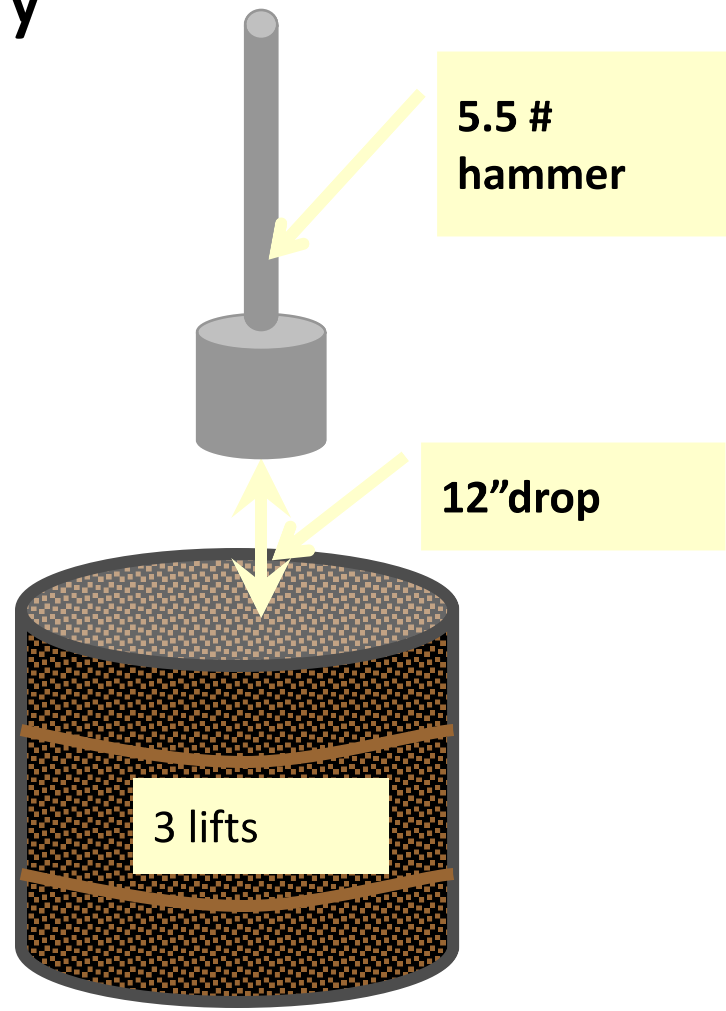
- A plot of the dry density versus the water content from a compaction test will be parabolic in shape.
- The peak of the curve is termed the maximum dry density, and the water content at which the peak occurs is the optimum water content.

Standard Proctor Energies

- Several standard energies are used for laboratory compaction tests
  - Standard – 12,400 ft-lbs/ft^3
  - Modified – 56,000 ft-lbs/ft^3
  - California – 20,300 ft-lbs/ft^3

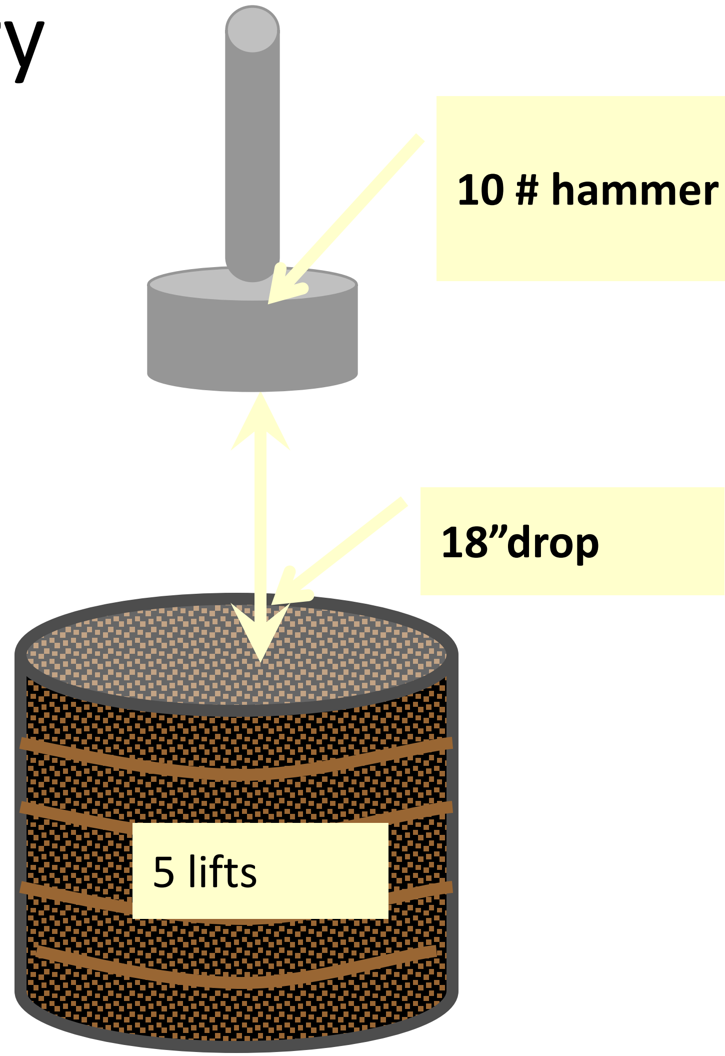
Standard Proctor Compaction Test Summary

- Uses 5.5 pound hammer
- dropped 12 inches
- mold filled in 3 lifts
- 25 blows of hammer per lift
- Total energy is $\approx$12,400 ft-lbs/ft^3



Modified Proctor Compaction Test Summary

- Uses 10 pound hammer
- dropped 12 inches
- mold filled in 5 lifts
- 25 blows of hammer per lift
- Total energy is $\approx$12,400 ft-lbs/ft^3



Proctor Compaction Test Summary

- Several Standard molds are used depending on maximum particle size in sample
  - 4”diameter mold (1/30 ft^3) used for soils with low gravel contents
  - Method A for soils with < 20 % gravel
  - Method B for soils with > 20 % gravel and < 20 % larger than 3/8”

Proctor Compaction Test Summary

- Several Standard molds are used depending on maximum particle size in sample
  - 6”diameter mold (1/13.33 ft^3) used for soils with significant gravel contents
  - More than 20 % gravel larger than 3/8”
  - Must have less than 30 % larger than 3/4”

Proctor Compaction Test Summary

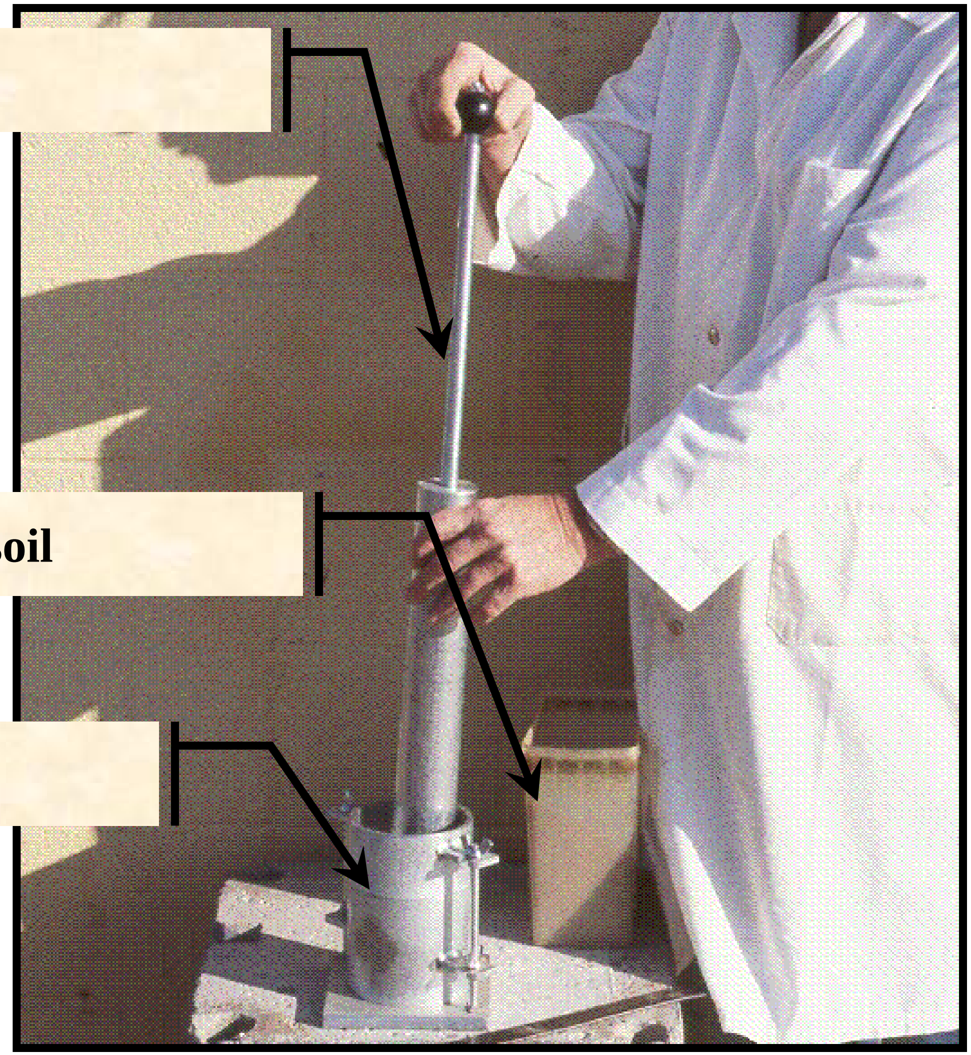
- Standardized tests are not available for soils with more than 30 percent by weight of the total sample being larger than 3/4”in diameter gravels
- ASTM Compaction Test Methods are
  - D698A D1557A
  - D698B D1557**B**
  - D698C D1557C

Proctor Compaction Test Summary

- Prepare 4 to 5 specimens at increasing water contents about 2 % apart.   Example - prepared samples at 14, 16, 18, and 20 percent. Use range of moistures based on feel and experience.

Proctor Compaction Test Summary

- Then, compact each sample into a steel mold with standard procedures



Proctor Compaction Test Summary

- Then, strike off excess soil so the mold has a known volume of soil.

II. Laboratory Methods for Determining OM and MD

The Proctor Test (after Ralph R. Proctor, 1933)

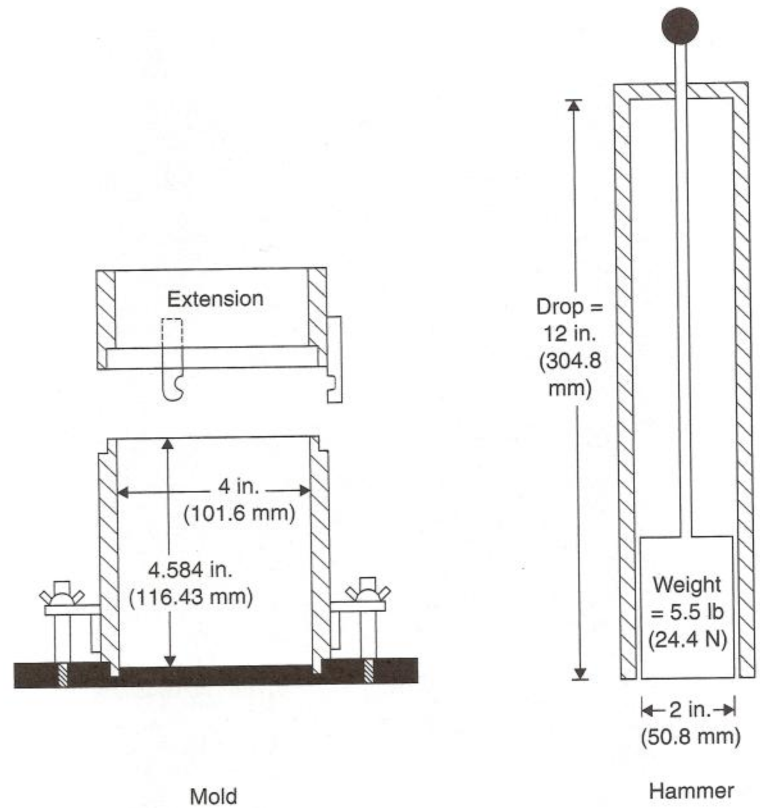


Figure 12.2. Standard Proctor mold and hammer.

II. Laboratory Methods for Determining OM and MD

The Proctor Test (after Ralph R. Proctor, 1933)

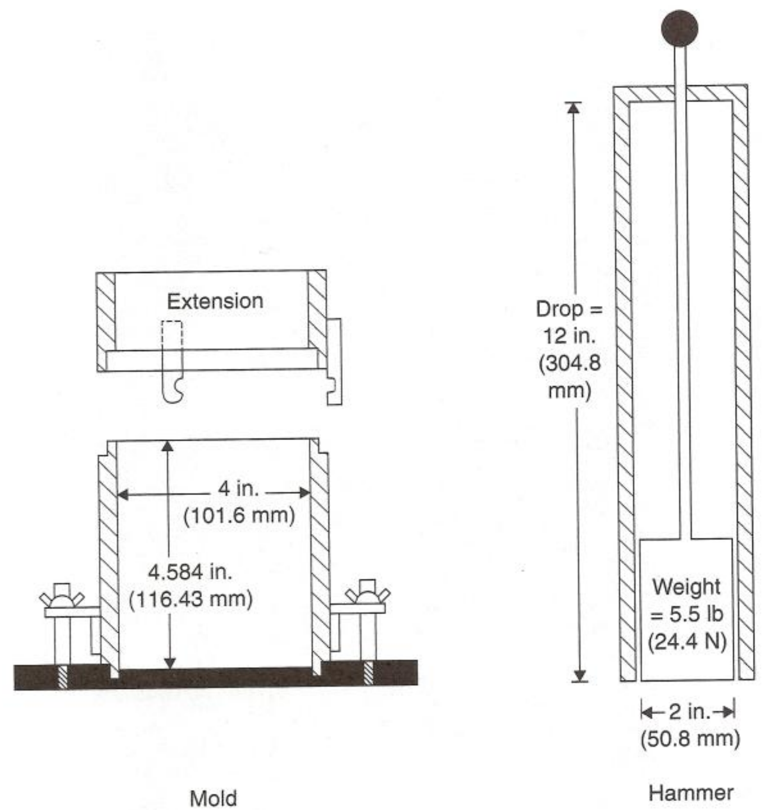


Figure 12.2. Standard Proctor mold and hammer.

II. The Method

The Proctor Test (after Ralph R. Proctor, 1933)

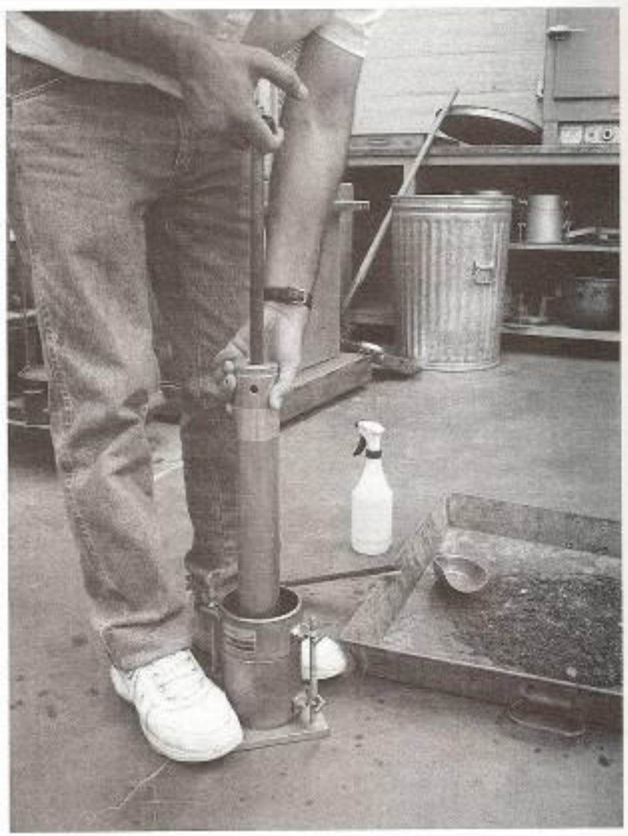
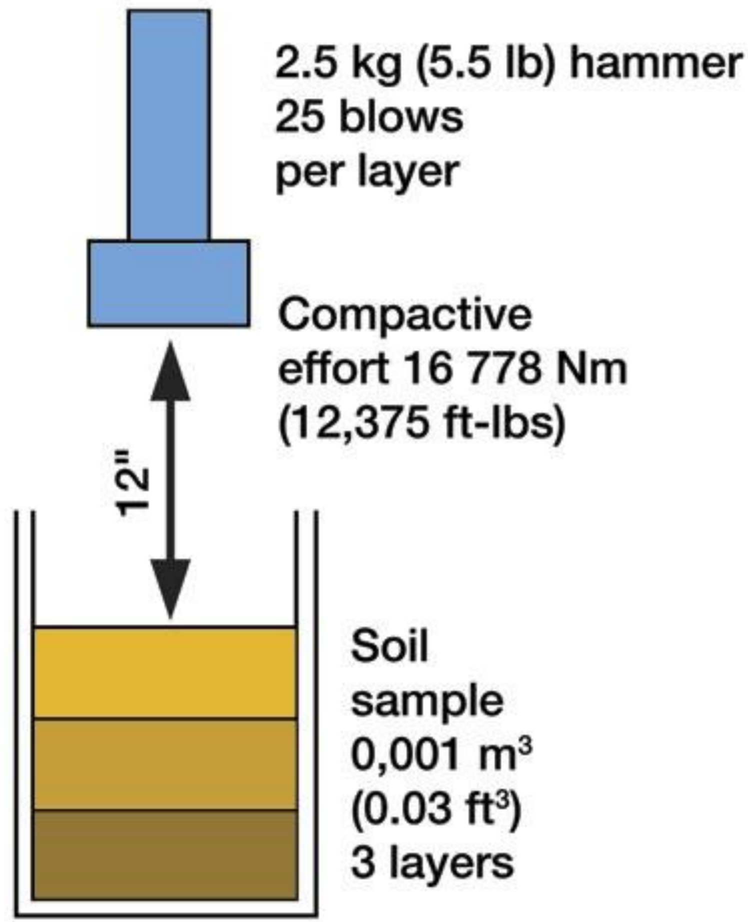


Figure 12.3. Compaction of soil in Proctor mold.



II. The Method

The Proctor Test (after Ralph R. Proctor, 1933)

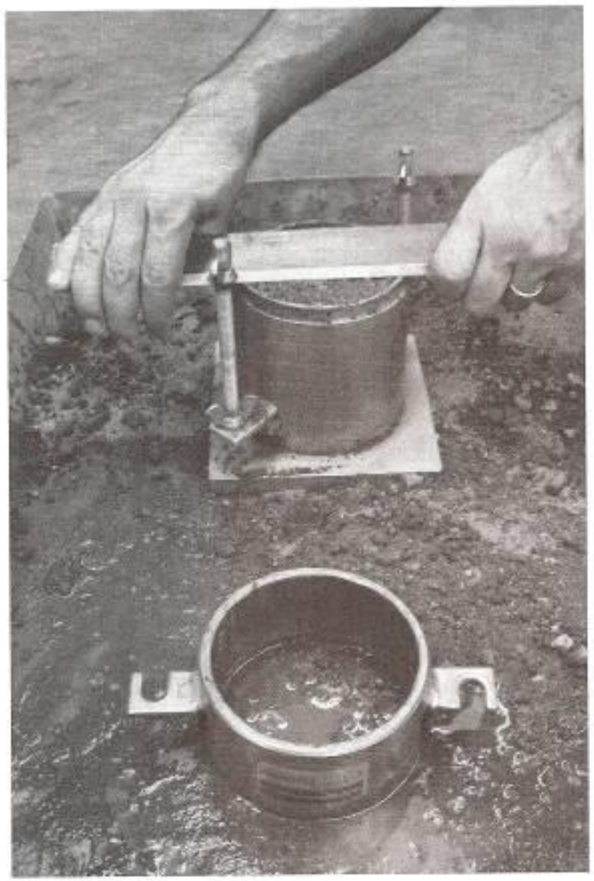
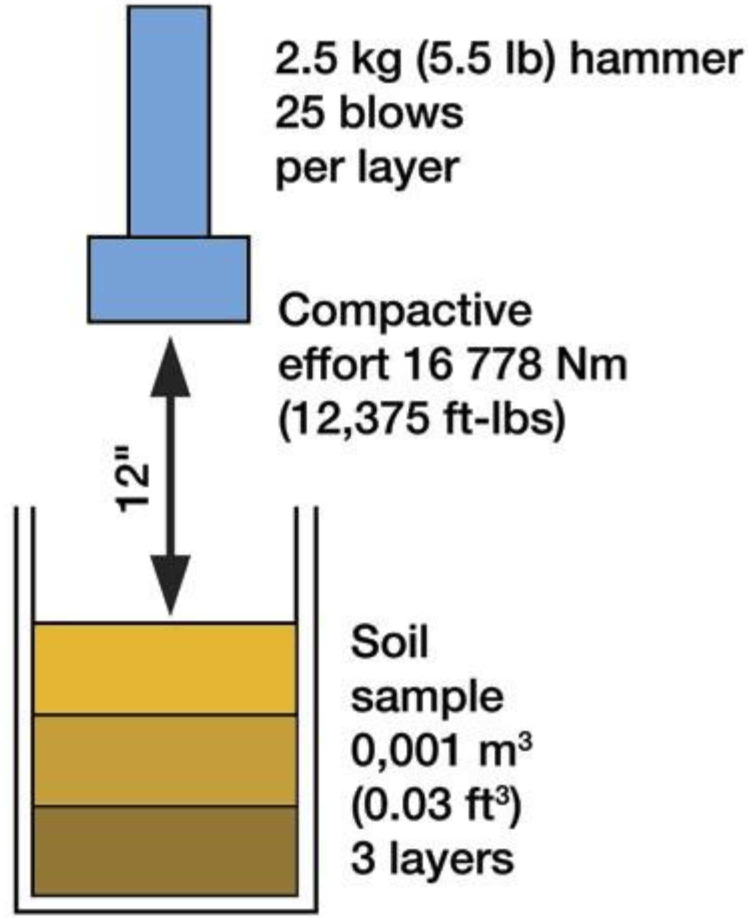


Figure 12.4. Excess soil being trimmed (Step 8).



Proctor Compaction Test Summary

- For each sample, measure the weight and the water content of the soil in the mold
- The mold volume and weight are pre-measured. <u>Don't assume nominal volume</u> of 1/30 ft^3 or 1/13.33 ft^3
- Calculate moist density
- Calculate dry density
- Plot dry density and water content for each point

Class Problem

- Calculate Moist density, dry density

$$\gamma_{\mathbf{moist}} = \frac{\mathbf{Weight}_{\mathbf{Moist}}}{\mathbf{Volume\_Mold}}$$

$$\gamma_{dry} = \frac{\gamma_{moist}}{1 + \frac{w\%}{100}}$$

Class Problem

Mold wt = 4.26 #, Mold Vol. = 0.03314 ft^3

Dry Unit Weight

- **The compacted soil is removed from the mold and its dry density (or dry unit weight) is measured.**

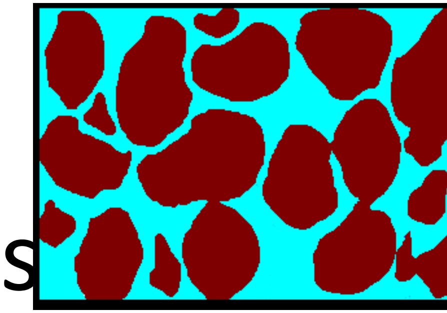
$$\gamma_d = \frac{\gamma_m}{1+\omega}$$

Where

$$\gamma_m = \frac{Mg}{V}$$

- γ_d =Dry Unit weight
- γ_m =Bulk Density
- ω =Water Content
- V =Total Soil Volume
- M =Total Wet Soil Mass
- g =Gravitational Acceleration

Water Role in Compaction Process

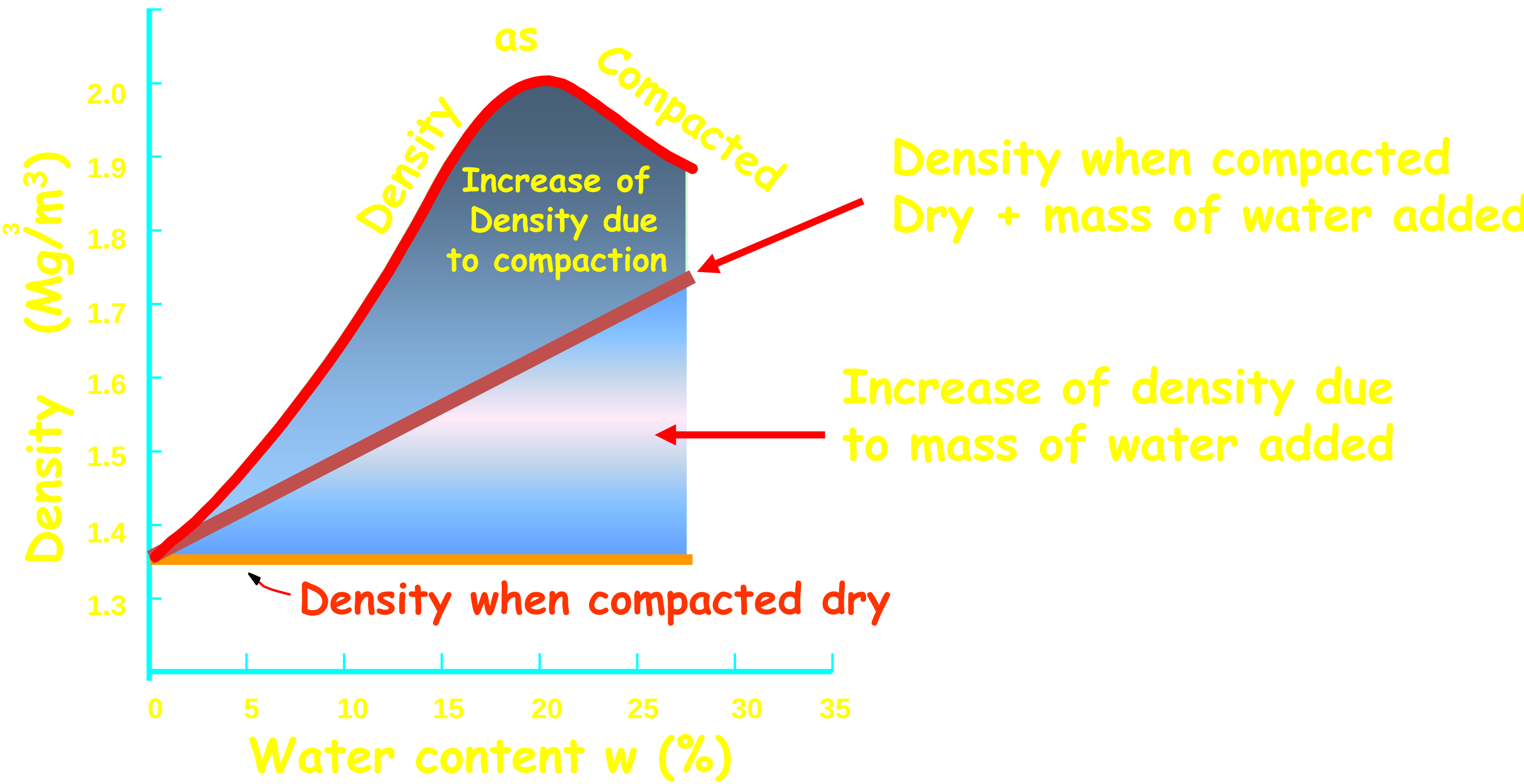


- **Water lubricates the soil grains so that they slide more easily over each other and can thus achieve a more densely packed arrangement.**

  - **A little bit of water facilitates compaction**

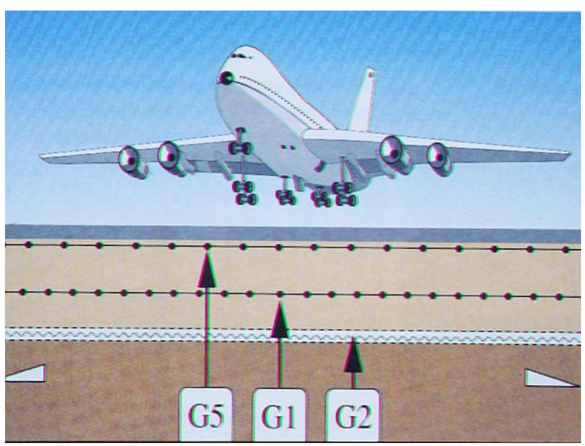
  - **too much water inhibits compaction.**

Dry Unit Weight

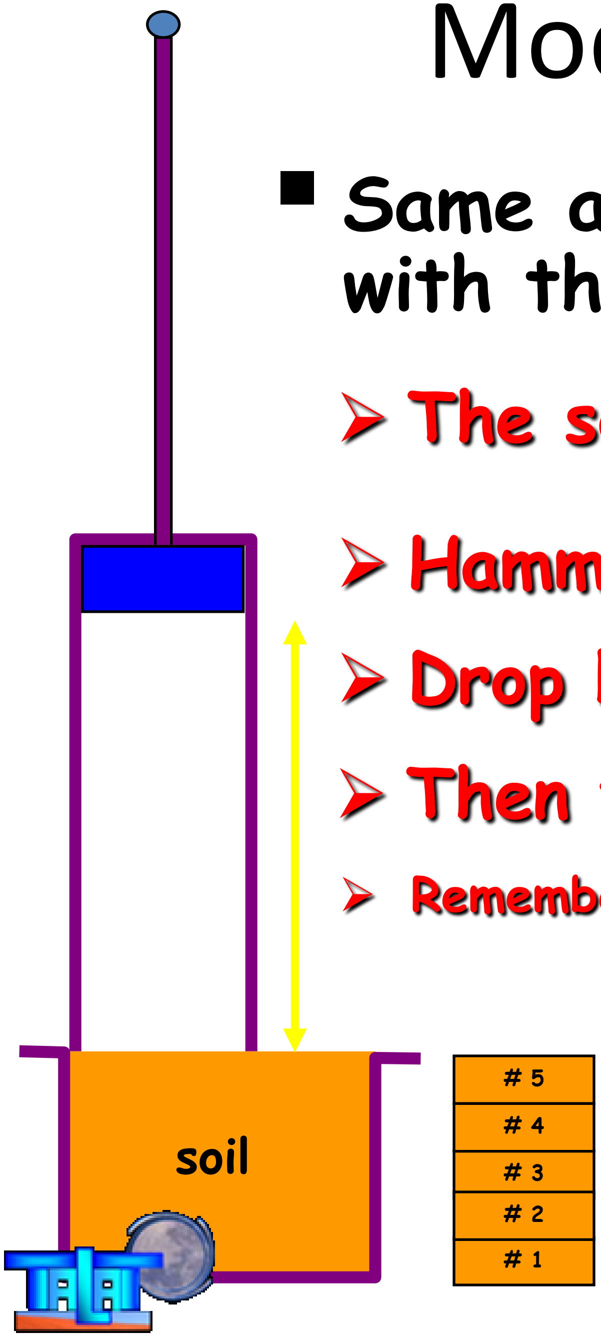


Modified Proctor Test

- **Was developed during World War II**
- **By the U.S. Army Corps of Engineering**
  - **For a better representation of the compaction required for airfield to support heavy aircraft.**



Modified Proctor Test



- **Same as the Standard Proctor Test with the following exceptions:**
  - **The soil is compacted in five layers**
  - **Hammer weight is 10 Lbs or 4.54 Kg**
  - **Drop height h is 18 inches or 45.72cm**
  - **Then the amount of Energy is calculated**
  - **Remember Standard Proctor Energy**

$$E_{SP} = 12{,}375\, ft-lb/\ ft^3$$

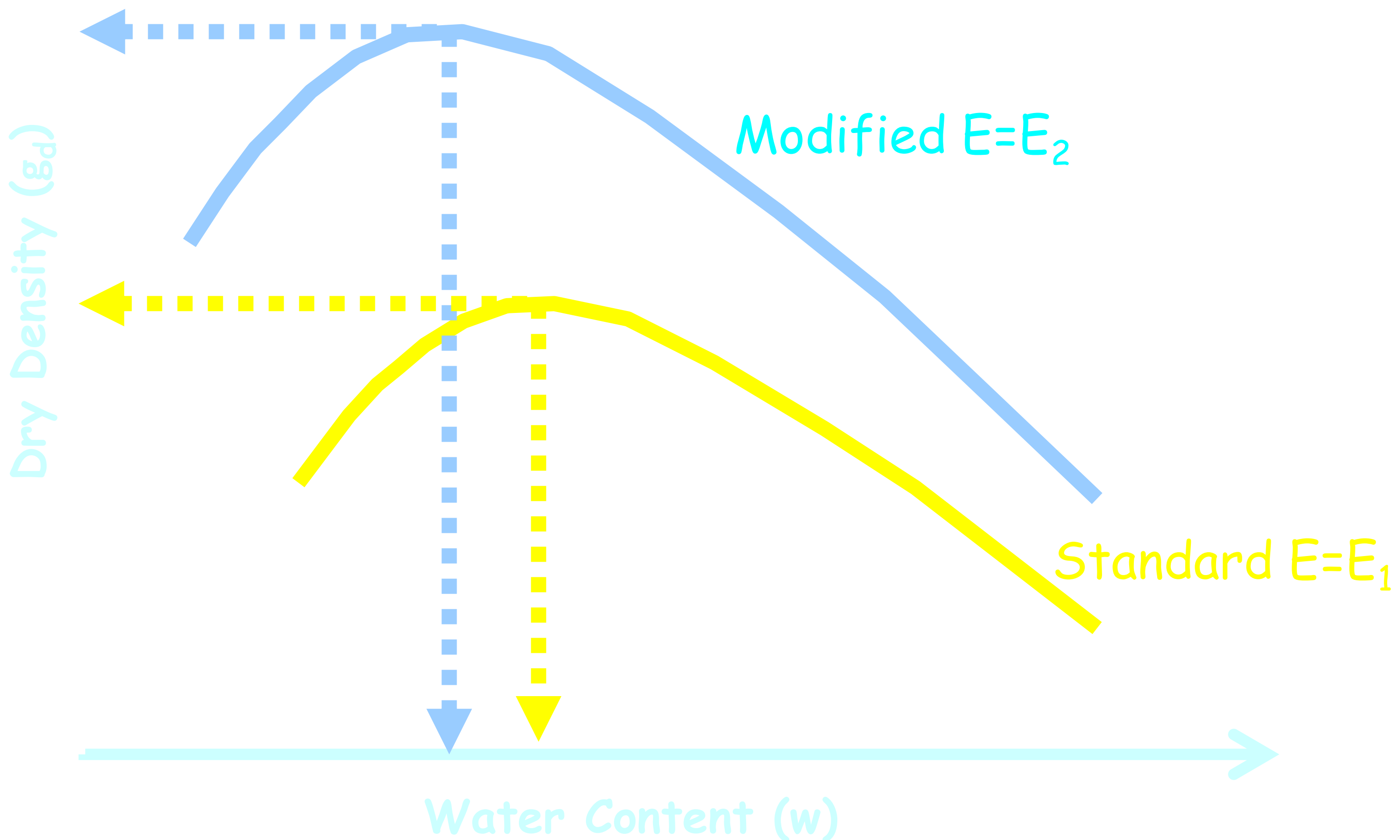
$$E_{MP} = \frac{(25\text{blows/layer}) * (5 \text{ of layers}) * (10\,\text{lbs}) * (1.5\,\text{ft})}{(1/30)\text{ft}^3}$$

$$E_{MP} = 56{,}250\, ft-lb/\ ft^3$$

$$\frac{E_{MP}}{E_{SP}} = \frac{56{,}250\, ft-lb/\ ft^3}{12{,}375\, ft-lb/\ ft^3} = 4.55$$

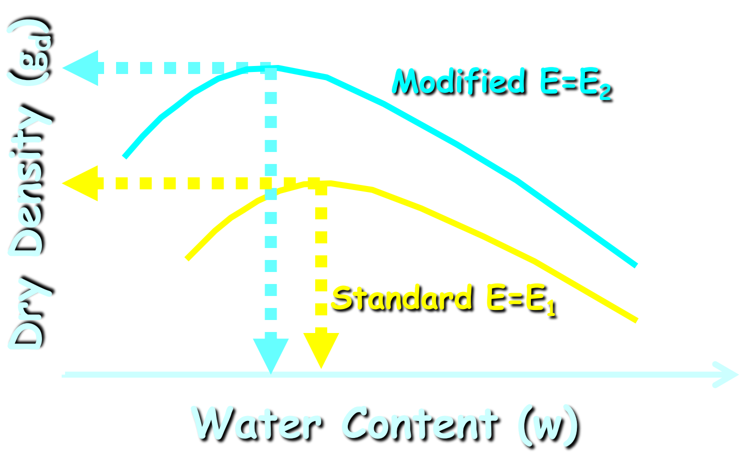
Effect of Energy on Compaction

$E_2 > E_1$

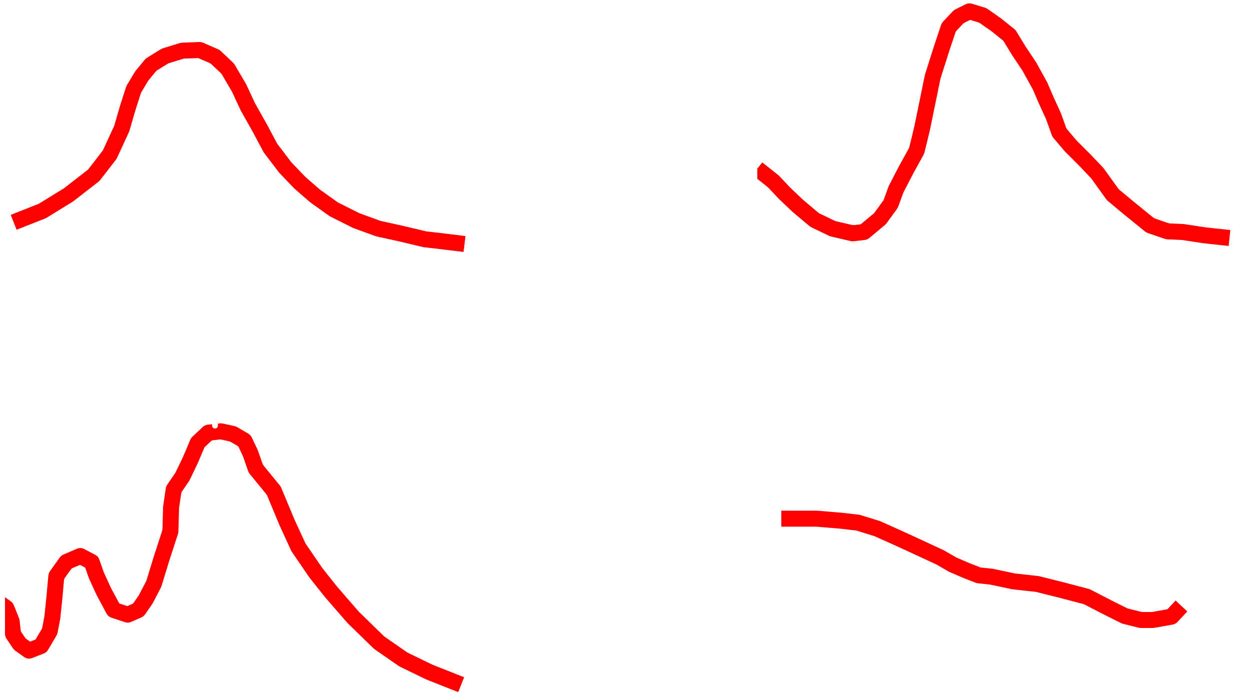


Comparison-Summary

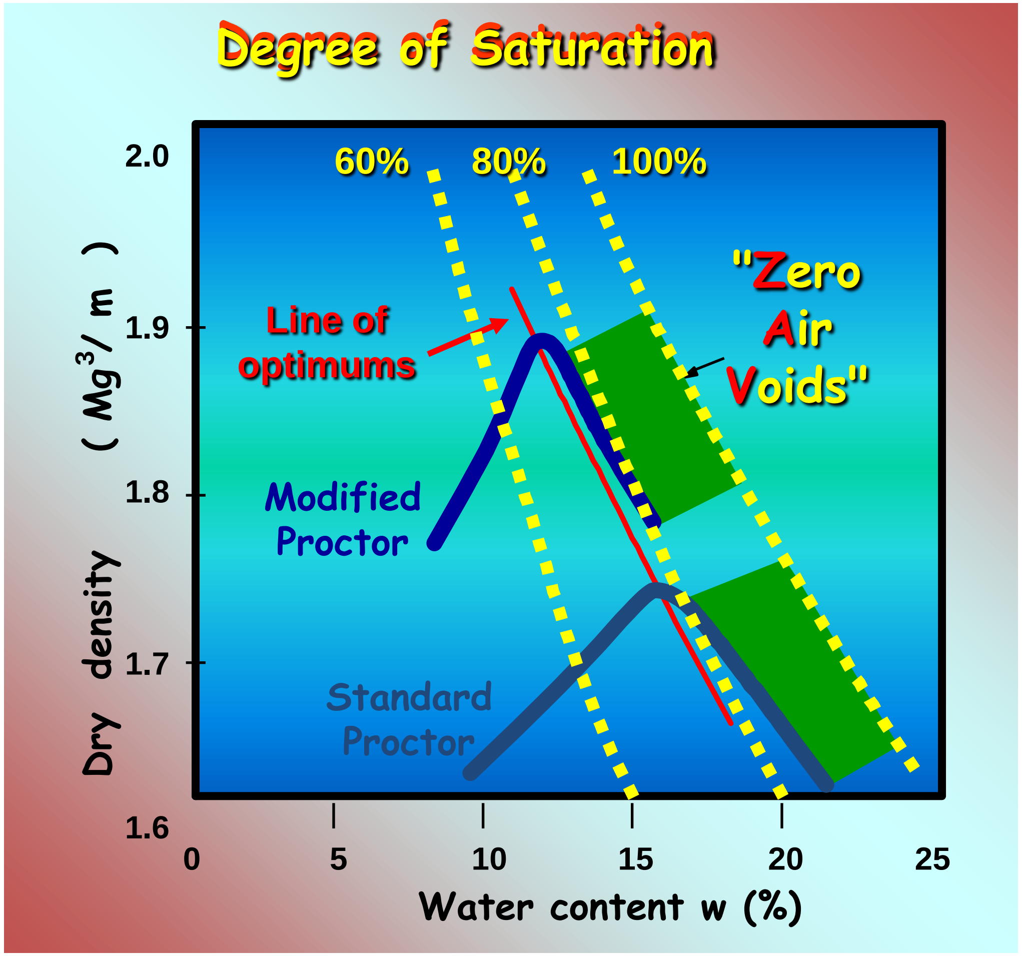
Standard Proctor Test	Modified Proctor Test
Mold size: 1/30 ft^3	Mold size: 1/30 ft^3
12 in height of drop	18 in height of drop
5.5 lb hammer	10 lb hammer
3 layers	5 layers
25 blows/layer	25 blows/layer
Energy 12,375 $ft \cdot lb/ft^3$	Energy 56,250 $ft \cdot lb/ft^3$



Common Compaction Curves Encountered in Practice



Zero-Air-Void



Points from the ZAV curve can be calculated from:

$$\rho_{dry} = G_s \rho_w / 1 + e$$

ZAV: The curve represents the fully saturated condition (S=100%).

ZAV cannot be reached by compaction.

Line of Optimum: A line drawn through the peak points of several compaction curves at different compactive efforts for the same soil will be almost parallel to a 100 % S curve

Entrapped Air: is the distance between the wet side of the compaction curve and the line of 100% saturation.

Results-Explanation

Below w_{omc}

Dry of Optimum

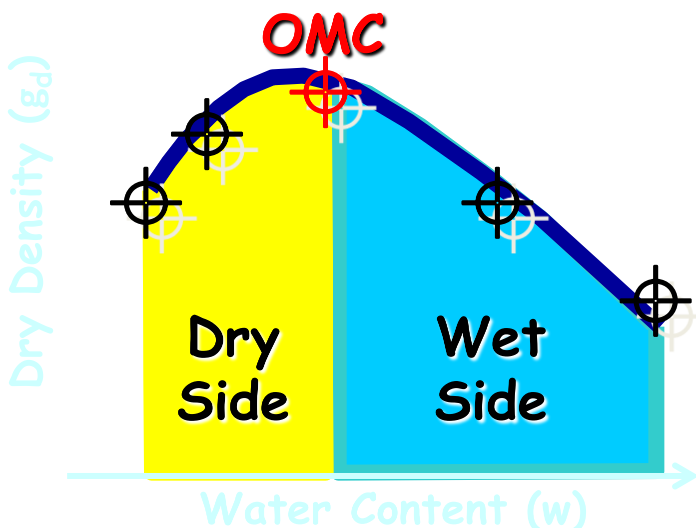
• As the water content increases, the particles develop larger and larger water films around them, which tend to "lubricate" the particles and make them easier to be moved about and reoriented into a denser configuration.

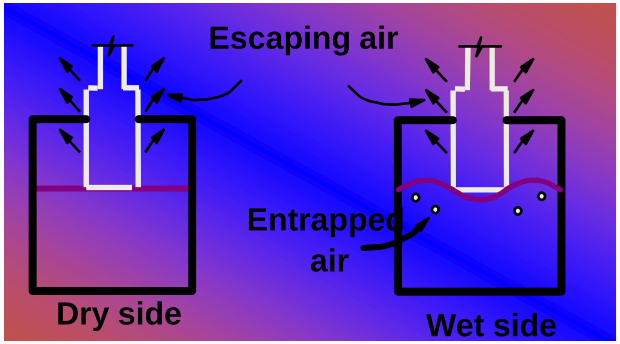
Hammer Impact

• Air expelled from the soil upon impact in quantities larger than the volume of water added.

At w_{omc}

The density is at the maximum, and it does not increase any further.





Above w_{omc}

Wet of Optimum

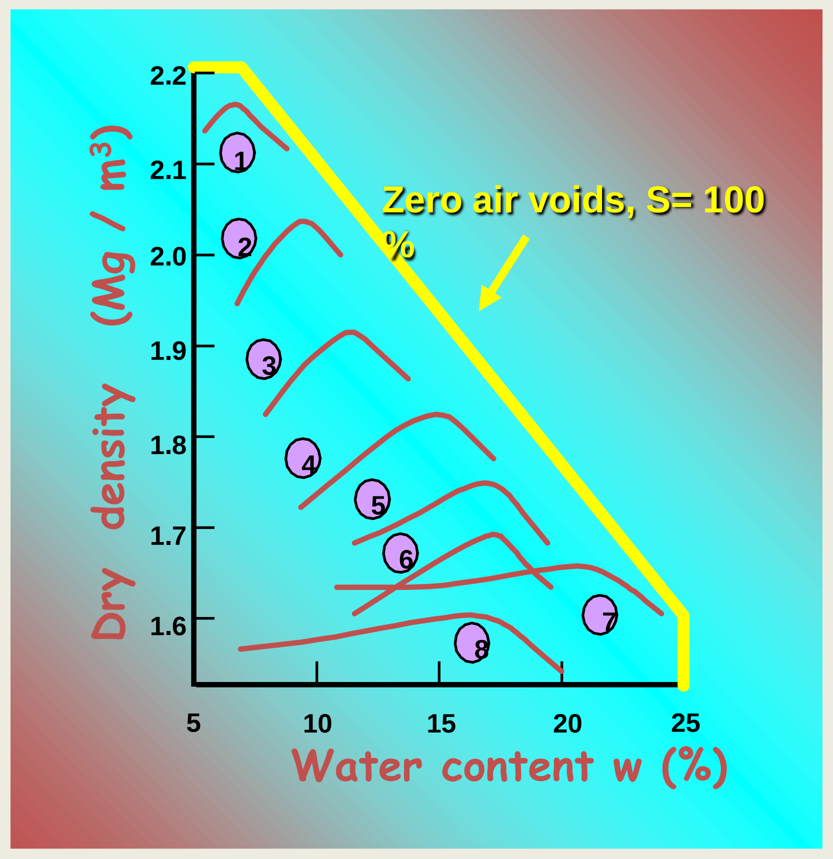
Water starts to replace soil particles in the mold, and since $\rho_w << \rho_s$ the dry density starts to decrease.

Hammer Impact

Moisture cannot escape under impact of the hammer. Instead, the entrapped air is energized and lifts the soil in the region around the hammer.

Effects of Soil Types on Compaction

The soil type-that is, grain-size distribution, shape of the soil grains, specific gravity of soil solids, and amount and type of clay minerals present



Soil texture and Plasticity data						
NO	Description	Sand	Silt	Clay	LL	PI
1	Well graded loamy sand	88	10	2	16	NP
2	Well graded sandy loam	72	15	13	16	NP
3	Med graded sandy loam	73	9	18	22	4
4	Lean sandy silty clay	32	33	35	28	9
5	Lean silty clay	5	64	31	36	15
6	Loessial silt	5	85	10	26	2
7	Heavy clay	6	22	72	67	40
8	Poorly graded sand	94	6	6	NP	NP

Compaction Characteristics

Unified Soil Classification

Group Symbol	Compaction Characteristics
GW	Good
GP	
GM	
GC	
SW	
SP	
SM	
SC	Good to Fair
CL	
ML	Good to Poor
OL, MH, CH, OH, PT	Fair to Poor

Embankment Materials

Unified Soil Classification

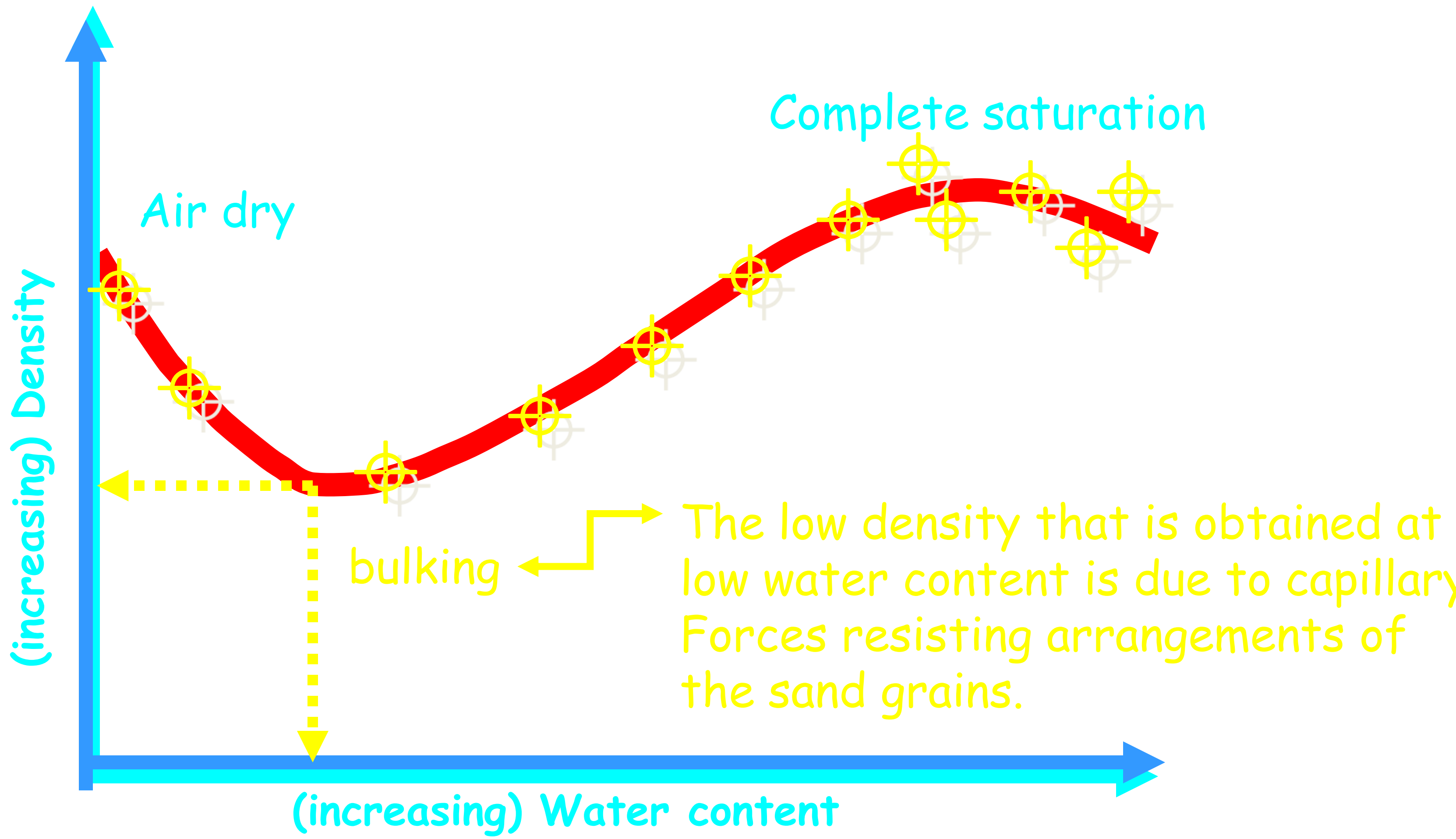
Group Symbol	Value as Embankment Material
GW	Very Stable
SW	
CL	Stable
GP	Reasonably Stable
GM	
GC	
SC	
SP	Reasonably Stable when Dense
SM	
ML	Poor, gets better with high density
OL, MH, CH, OH, PT	Poor, Unstable

Subgrade Materials

Unified Soil Classification

Group Symbol	Value as Subgrade Material
GW	Excellent
GP	Excellent to Good
GM	
GC	Good
SW	
SP	Good to Fair
SM	
SC	
ML	Fair to Poor
CL	
OL, MH, CH, OH, PT	Poor to Not Suitable

Typical Compaction Curve for Cohesionless Sands & Sandy Gravel

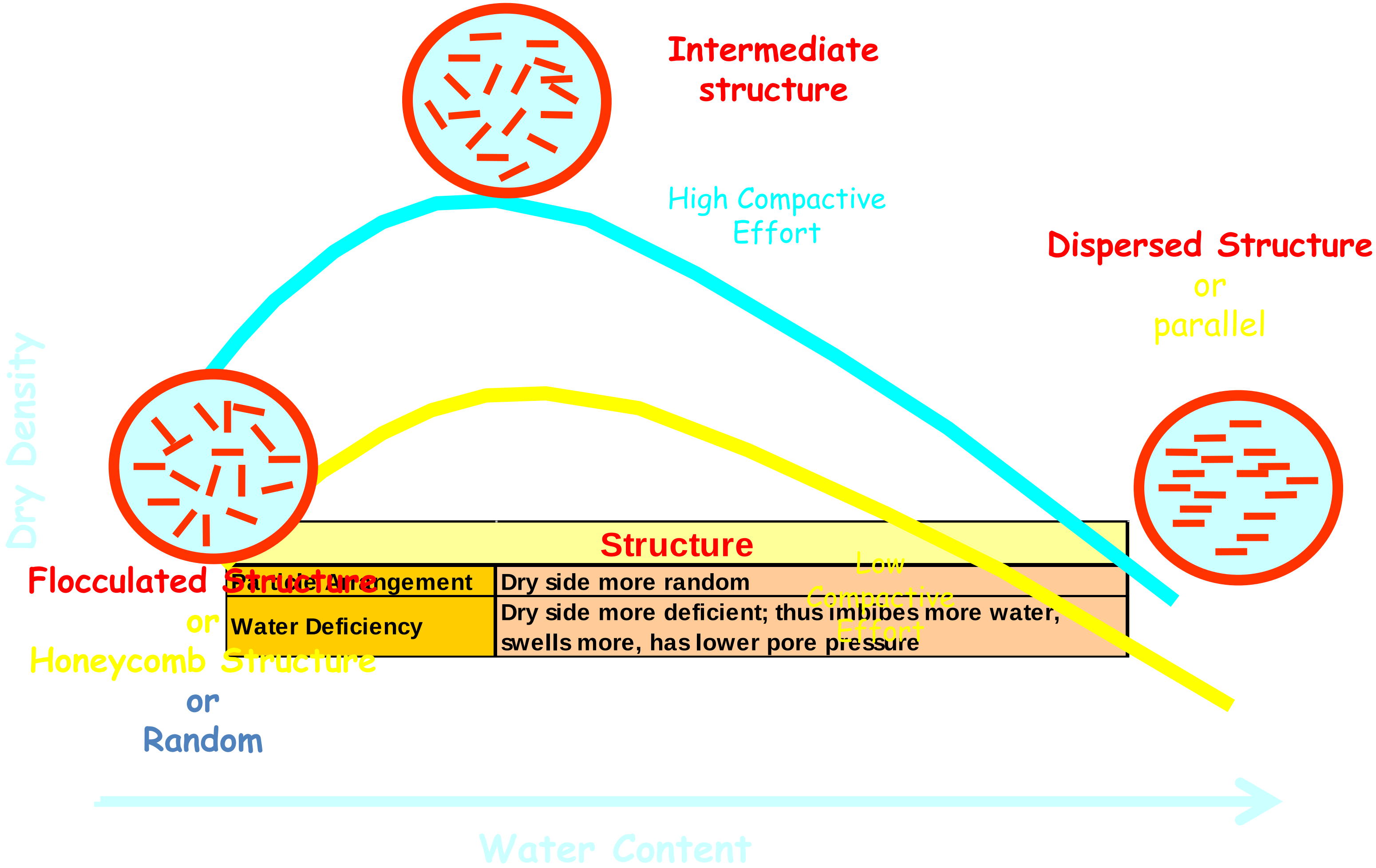


Water & Compaction

Remember what is the Affect

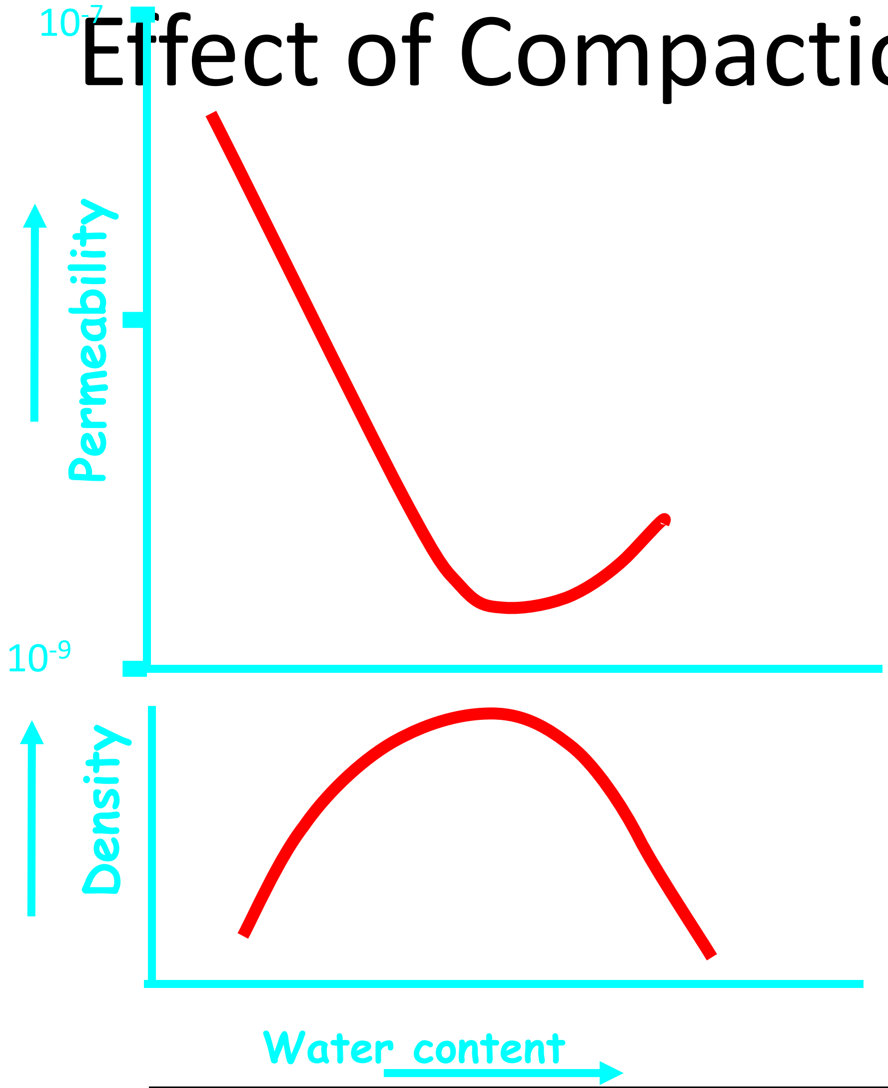
- **Increasing the water content at which soil is compacted:**
  - **Increases the likelihood of obtaining dispersed soil structure with reduced shear strengths.**
  - **Increases the pore pressure in the soil, decreasing the short term shear strength.**

Structure of Compacted Clay



Structure	
Particle Arrangement	Dry side more random
Water Deficiency	Dry side more deficient; thus imbibes more water, swells more, has lower pore pressure

Effect of Compaction on permeability

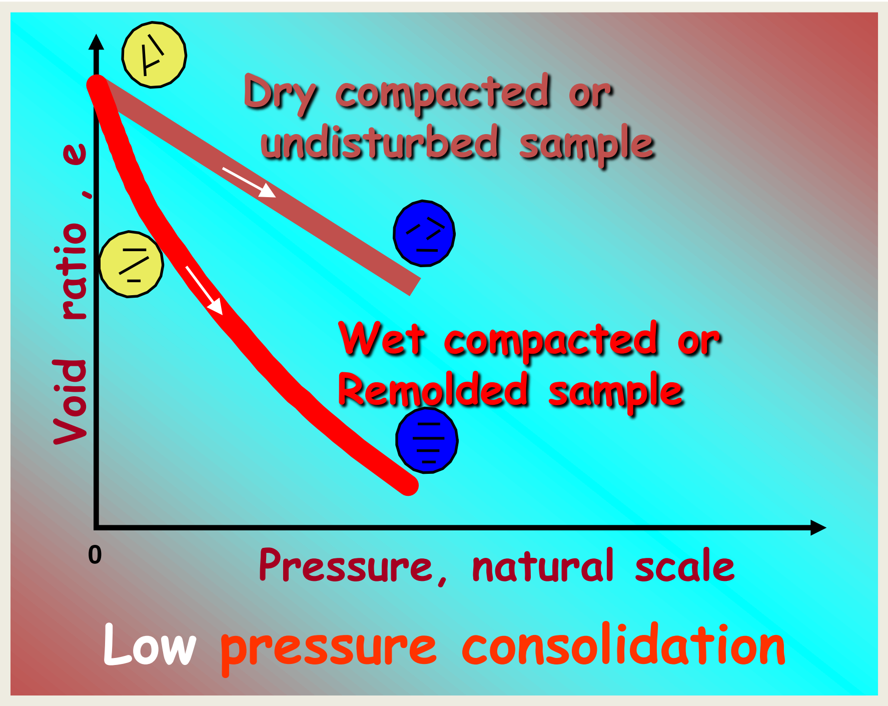


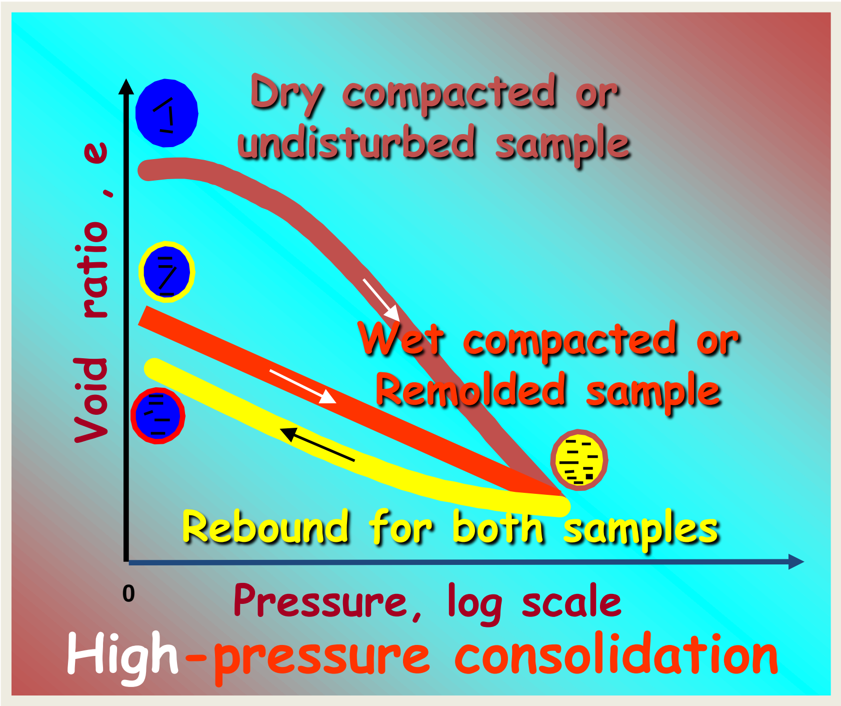
Permeability at constant compactive effort decreases with increasing water content and reaches a minimum at about the optimum.

If compactive effort is increased, the permeability decreases because the void ratio decreases.

Permeability	
Magnitude	**Dry side more permeable**
Permanence	**Dry side permeability reduced much more by permeation**

Effect of Compressibility





Compressibility of compacted clays is function of stress level.

Low stress level: Clay compacted wet of optimum are more compressible.

High stress level: The opposite is true

Compressibility	
Magnitude	Wet side more compressible in low pressure range, dry side in high pressure range
Rate	Dry side consolidates more rapidly

Compressibility & Expansion

Unified Soil Classification

Group Symbol	Compressibility and Expansion
GW	Very Little
GP	
SW	
SP	
GM	Slight
GC	
SM	
SC	Slight to Medium
ML	
CL	Medium
OL, MH, CH, OH, PT	High

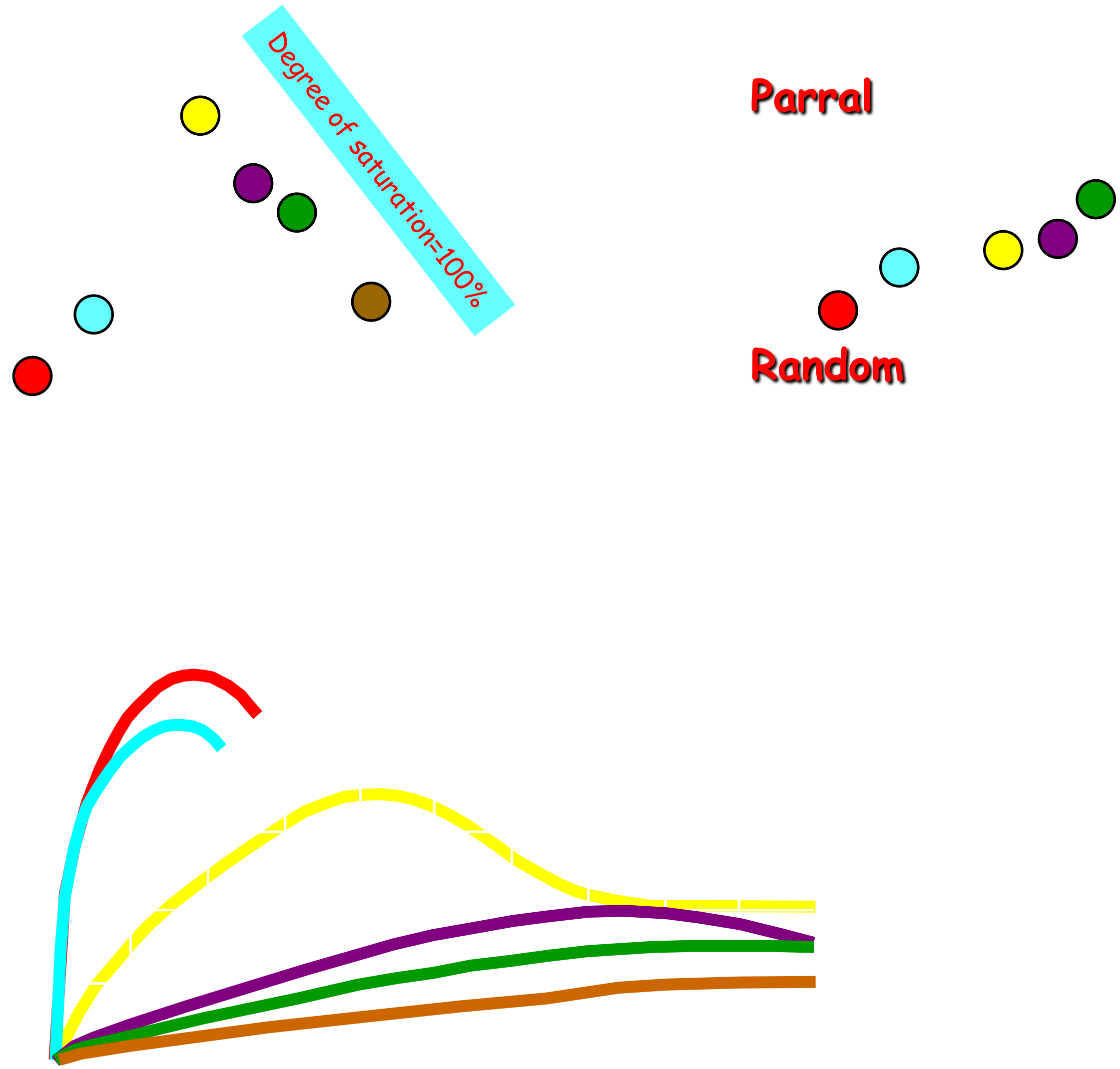
Compressibility & Expansion

Unified Soil Classification

Group Symbol	Compressibility and Expansion
GW	Very Little
GP	
GM	Slight
GC	
SW	Very Little
SP	
SM	Slight
SC	Slight to Medium
ML	
CL	Medium
OL, MH, CH, OH, PT	High

Effect of Strength

Samples (Kaolinite) compacted dry of optimum tend to be more rigid and stronger than samples compacted wet of optimum



Effect of Strength (con)

The CBR (California bearing ratio)

CBR= resistance required to penetrate a 3-in^2 piston into the compacted specimen/ resistance required to penetrate the same depth into a standard sample of crushed stone.

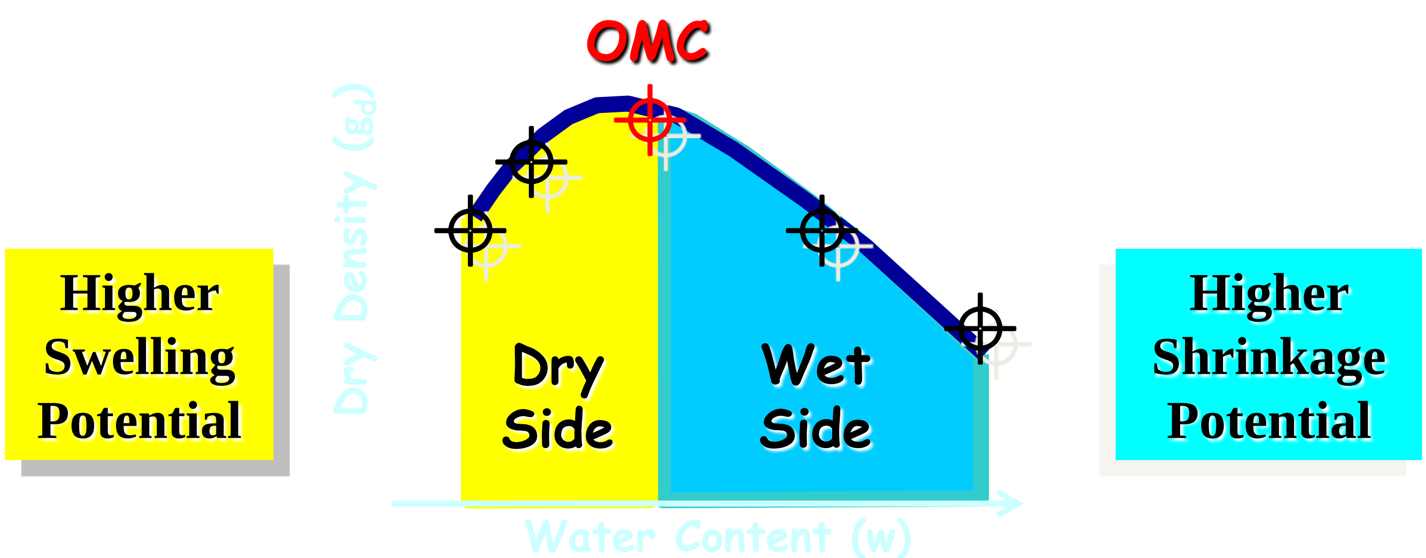
A greater compactive effort produces a greater CBR for the dry of optimum. However, the CBR is actually less for the wet of optimum for the higher compaction energies (overcompaction).

Comparison of Soil Properties
Dry of Optimum & Wet of Optimum Compaction

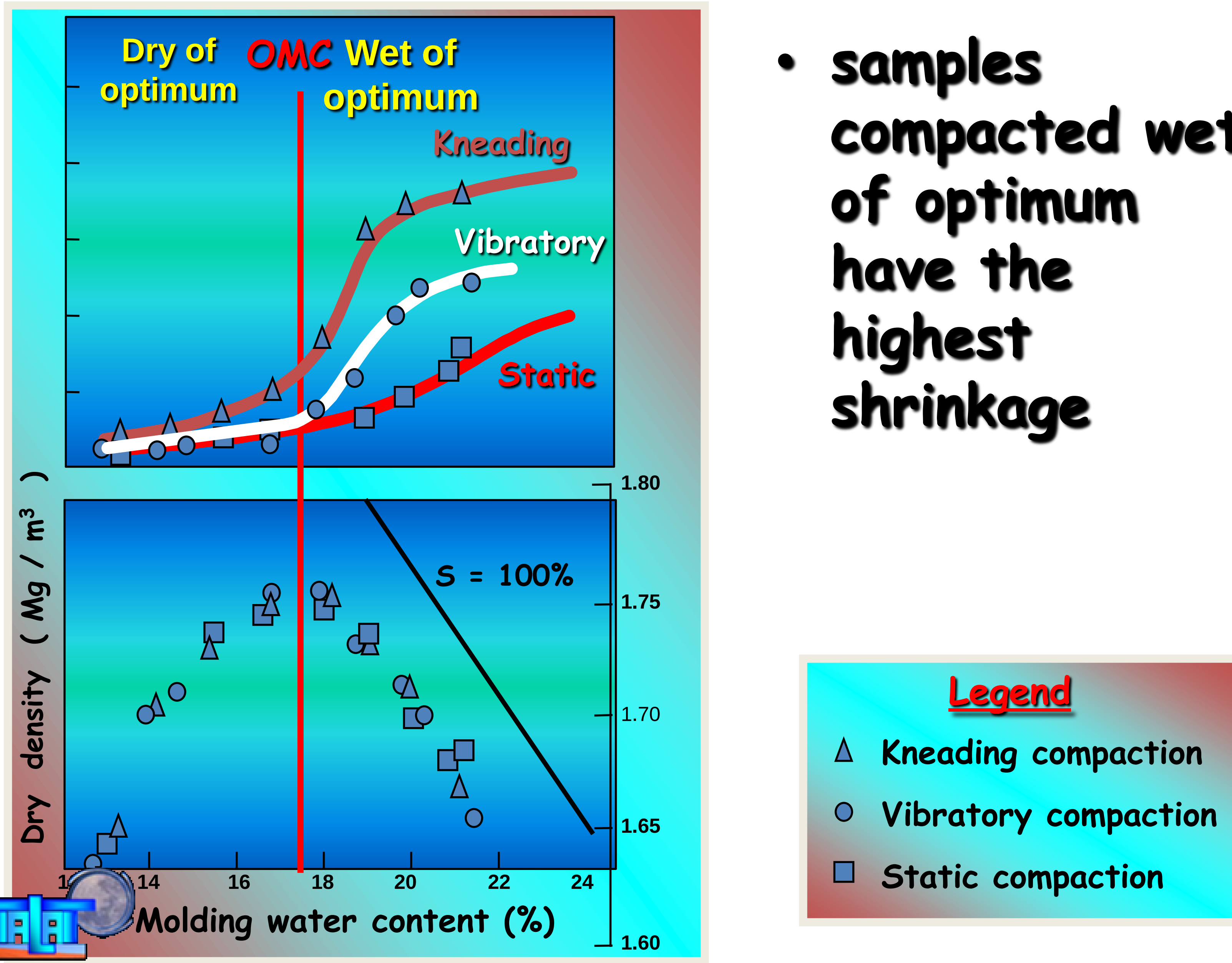
	Strength
As molded	
a :Undrained	Dry side is much higher
b :Drained	Dry side is some how higher
After saturation	
a :Undrained	Dry side higher if swelling prevented,wet sidecan be hiher if swelling is permitted
b :Drained	dry side the same or slpghtly hiher
Stress-strain modulus	Dry side much greater
Sensitivity	Dry side more apt to be sensitive

Effect of Swelling

- Swelling of compacted clays is greater for those compacted dry of optimum. They have a relatively greater deficiency of water and therefore have a greater tendency to adsorb water and thus swell more.



Compaction and Shrinkage



- samples compacted wet of optimum have the highest shrinkage

Engineering Properties Summary

Properties	Dry side	Wet side
Structure	More random	More oriented (parallel)
Permeability	More permeable	
Compressibility	More compressible in *high* pressure range	More compressible in *low* pressure range
Swelling	Swell more, higher water deficiency	*Shrinkage more
Strength	Higher	

Summary

<table>
<tr><th>UCS Group Symbol</th><th>Compaction Characteristics</th><th>Compressibility and Expansion</th><th>Value as Embankment Material</th><th>Value as Subgrade Material</th></tr>
<tr><td>GW</td><td rowspan="7">Good</td><td rowspan="2">Very Little</td><td>Very Stable</td><td>Excellent</td></tr>
<tr><td>GP</td><td rowspan="3">Reasonably Stable</td><td rowspan="2">Excellent to Good</td></tr>
<tr><td>GM</td><td rowspan="2">Slight</td></tr>
<tr><td>GC</td><td rowspan="2">Good</td></tr>
<tr><td>SW</td><td rowspan="2">Very Little</td><td>Very Stable</td></tr>
<tr><td>SP</td><td rowspan="2">Reasonably Stable when Dense</td><td rowspan="3">Good to Fair</td></tr>
<tr><td>SM</td><td>Slight</td></tr>
<tr><td>SC</td><td>Good to Fair</td><td rowspan="3">Slight to Medium</td><td>Reasonably Stable</td></tr>
<tr><td>ML</td><td>Good to Poor</td><td>Poor, gets better with high density</td><td rowspan="2">Fair to Poor</td></tr>
<tr><td>CL</td><td>Good to Fair</td><td>Stable</td></tr>
<tr><td>OL, MH, CH, OH, PT</td><td>Fair to Poor</td><td>High</td><td>Poor, Unstable</td><td>Poor to Not Suitable</td></tr>
</table>

Appendix

Hand Out 03_2

ABLE 5-1 Comparison of Soil Properties between Dry of Optimum and Wet of Optimum Compaction*

Property	Comparison
1. Structure:	
A. Particle arrangement	Dry side more random
B. Water deficiency	Dry side more deficient; thus imbibes more water, swells more, has lower pore pressure
C. Permanence	Dry side structure sensitive to change
2. Permeability:	
A. Magnitude	Dry side more permeable
B. Permanence	Dry side permeability reduced much more by permeation
3. Compressibility:	
A. Magnitude	Wet side more compressible in low pressure range, dry side in high pressure range
B. Rate	Dry side consolidates more rapidly
4. Strength:	
A. As molded:	
(a) Undrained	Dry side much higher
(b) Drained	Dry side somewhat higher
B. After saturation:	
(a) Undrained	Dry side somewhat higher if swelling prevented; wet side can be higher if swelling permitted
(b) Drained	Dry side about the same or slightly greater
C. Pore water pressure at failure	Wet side higher
D. Stress-strain modulus	Dry side much greater
E. Sensitivity	Dry side more apt to be sensitive

Hand Out 03_3

Compaction and Earth Dam

- The engineer must consider not only the behavior of the soil as compacted but the behavior of the soil in the completed structure, especially at the time when the stability or deformation of the structure is most critical.

- For example, consider an element of compacted soil in a dam core. As the height of the dam increases, the total stresses on the soil element increase. When the dam is performing its intended function of retaining water, the percent saturation of the compacted soil element is increased by the permeating water. Thus the engineer designing the earth dam must consider not only the strength and compressibility of the soil element as compacted, but also its properties after is has been subjected to increased total stresses and saturated by permeating water.

Field Compaction Equipment and Procedures

Objective of field Compaction & Control Parameters

- The objective of compaction is to stabilize soils and improve their engineering behavior
- It is important to keep in mind the desired engineering properties of the fill, dam, road, etc…
- The density of the fill in addition to the water content should be observed.

From Holtz and Kovacs, 1981

Design Procedure for field Compaction

Design and Selection of Fill Materials

1) Survey the local soil sources that could possibly be used.

2) Obtain soil samples from each source (borrow), and perform the necessary laboratory tests to classify the soil via either <u>**AASHTO or the UCS systems to define the properties required for design.**</u>

   - The classification itself will often tell whether or not a given soil is suitable for an intended application.
   - Soils with large shrinkage ratios (SR) should be avoided.
   - Soils with high plasticity indices (PI) should be avoided, since they indicate a tendency to shrink/expand.
   - Organic matter which can decay should generally be avoided as fill material unless special precautions are taken.

Design and Selection of Fill Materials

3) Once a soil is found to be suitable, for an intended application, perform the necessary moisture-density study.

4) If local codes/guidelines are not provided, a study would be needed to determine the minimum relative field compaction of the soil. Factors would be:

   - ✓ required shear strength of the soil
   - ✓ maximum allowable settlements under design loads.

5) After the earth structure is designed,
   - The compaction specifications are written.
   - Field compaction *control tests* are specified,
   - And the results of these become the standard for controlling the project.

How to set your Specification

From Holtz and Kovacs, 1981

Specifications

1) End-product specifications

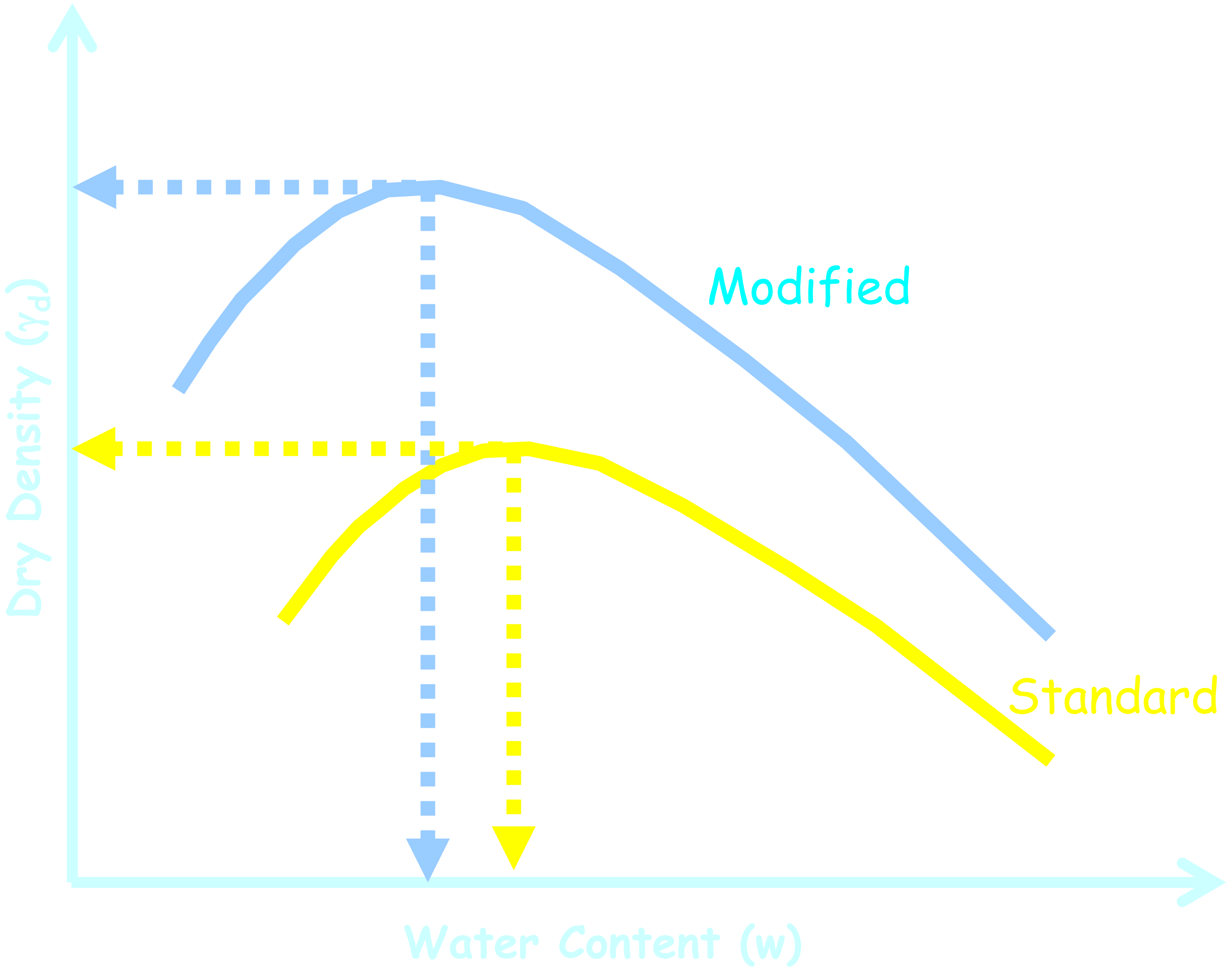
- This specification is used for most highways and building foundation, as long as the contractor is able to obtain the specified ***relative compaction*** , how he obtains it doesn't matter, nor does the equipment he uses.
- *Care the results only !*

2) Method specifications

- The type and weight of roller, the number of passes of that roller, as well as the lift thickness are specified. A maximum allowable size of material may also be specified.
- ***It is typically used for large compaction project.***

Results From Laboratory Test

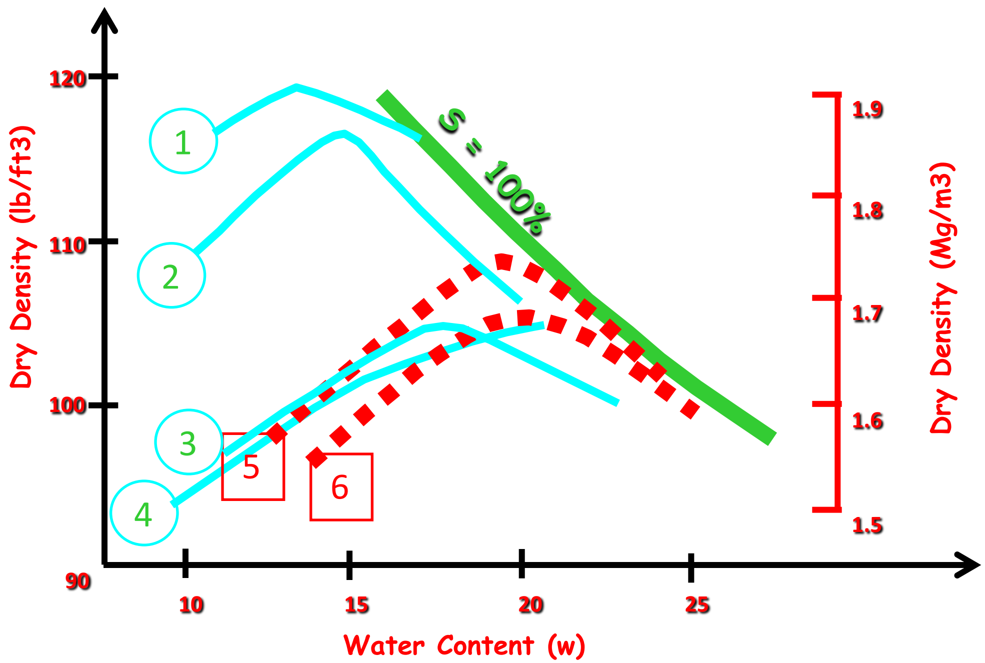
Standard & Modified Proctor Test



(From Lambe and Whitman, 1979)

Field & Laboratory Compaction

- It is difficult to choose a laboratory test that reproduces a given field compaction procedure.
- The laboratory curves generally yield a somewhat lower optimum water content than the actual field optimum.
- The majority of field compaction is controlled by the *dynamic laboratory tests*.



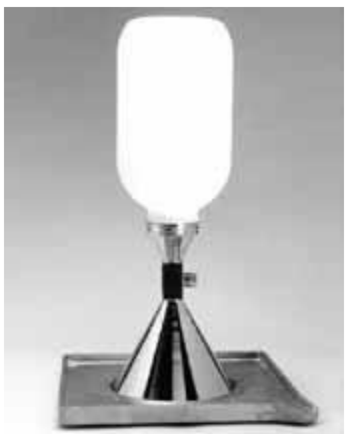
Curve 1, 2,3,4: laboratory compaction

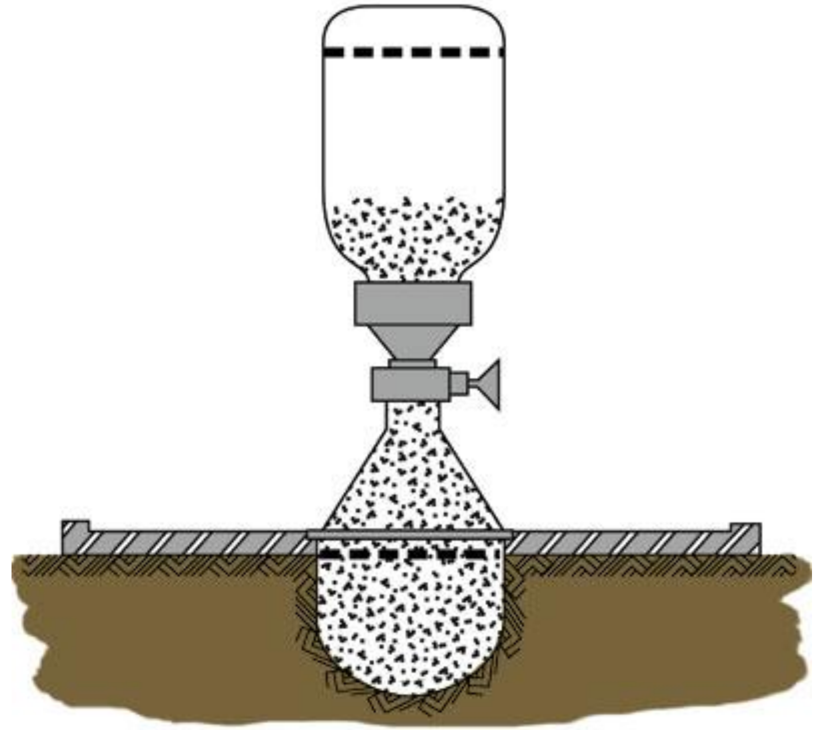
Curve 5, 6: Field compaction

Field Compaction Equipments

IV. Field Methods of Determining if OM & MD are achieved

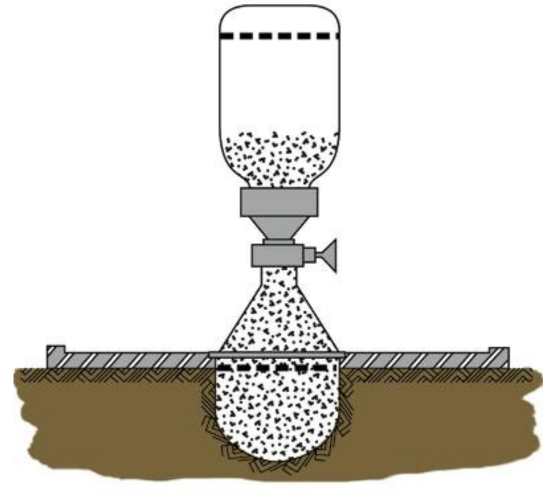
A. Sand Cone Method





IV. Field Methods of Determining if OM & MD have been achieved

A. Sand Cone Method



Unit Weight: (moist) = $\frac{\text{weight of soil and water}}{\text{volume total of soil}}$ $\quad \boldsymbol{\gamma_w} = \frac{Ws+Ww}{Vt} = \frac{Ww}{Vt}$

Moisture Content: = $\frac{\text{weight of water}}{\text{weight of soil}}$ $\quad \mathbf{w} = \frac{\mathbf{Ww}}{\mathbf{Ws}}$

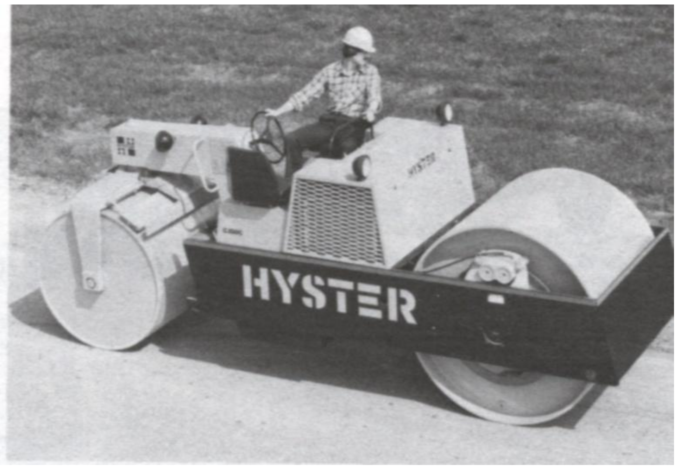
Bulldozers

Watering

Compaction
Field Equipments

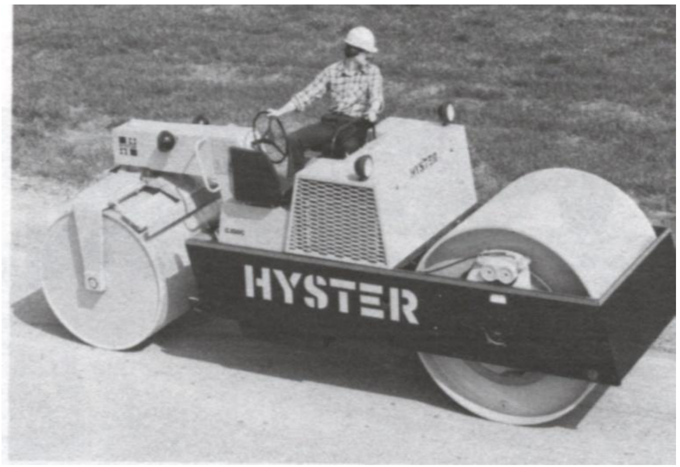
Smooth-wheel roller (drum)

- 100% coverage under the wheel
- Contact pressure up to 380 kPa
- Can be used on all soil types except for rocky soils.
- Compactive effort: static weight
- The most common use of large smooth wheel rollers is for proof-rolling subgrades and compacting asphalt pavement.



Smooth-wheel roller (drum)

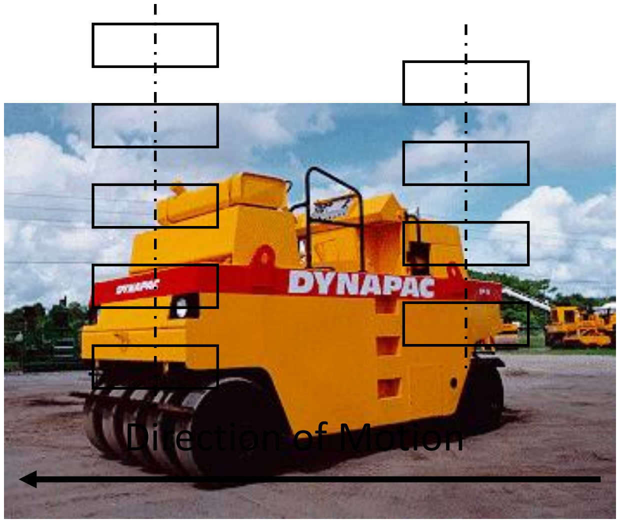
- **Suitable for:**
  - well-graded sands and gravels
  - silts and clays of low plasticity



- **Unsuitable for:**
  - uniform sands;
  - silty sands;
  - soft clays

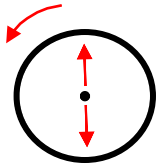
Pneumatic (or rubber-tired) roller

- 80% coverage under the wheel
- Contact pressure up to 700 kPa
- 7 to 13 wheels are arranged in two rows.
- Compactive effort: static weight and kneading.



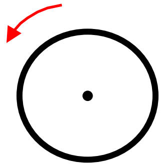
Pneumatic (or rubber-tired) roller

- Small Tired Roller
  - Straight rolling
  - Wobble Action



- Heavy Tire Roller
  - 50 to 100 tons
  - Tire pressure 90-150 psi
  - Additional weight
    - Water, Sand or Steel

- Can be used for both granular and fine-grained soils.

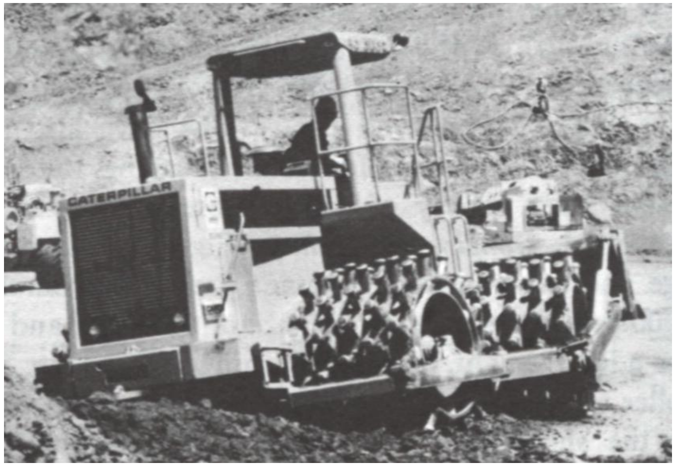


- Can be used for highway fills or earth dam construction.

Pneumatic (or rubber-tired) roller

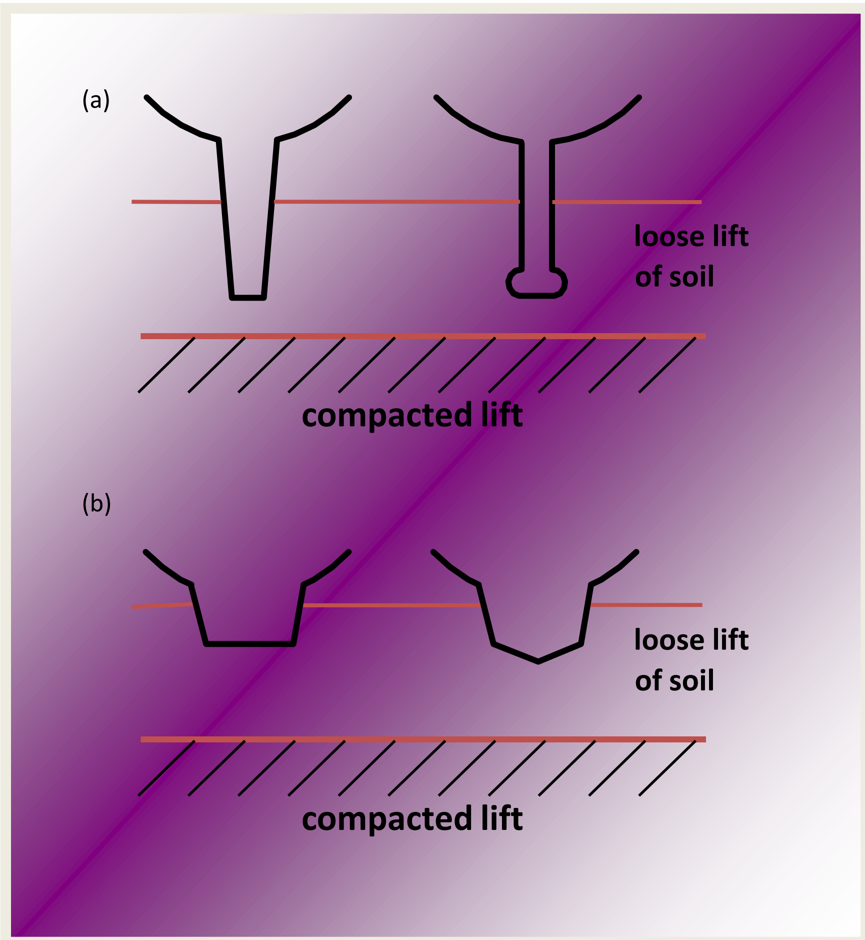
- Suitable for: most
  - Coarse
  - fine soils.
- Unsuitable for:
  - very soft clay
  - highly variable soils

Sheepsfoot rollers



- Has many round or rectangular shaped protrusions or "feet" attached to a steel drum
- 8% ~ 12 % coverage
- Contact pressure is from 1400 to 7000 kPa
- It is best suited for clayed soils.
- Compactive effort: static weight and kneading.

foot

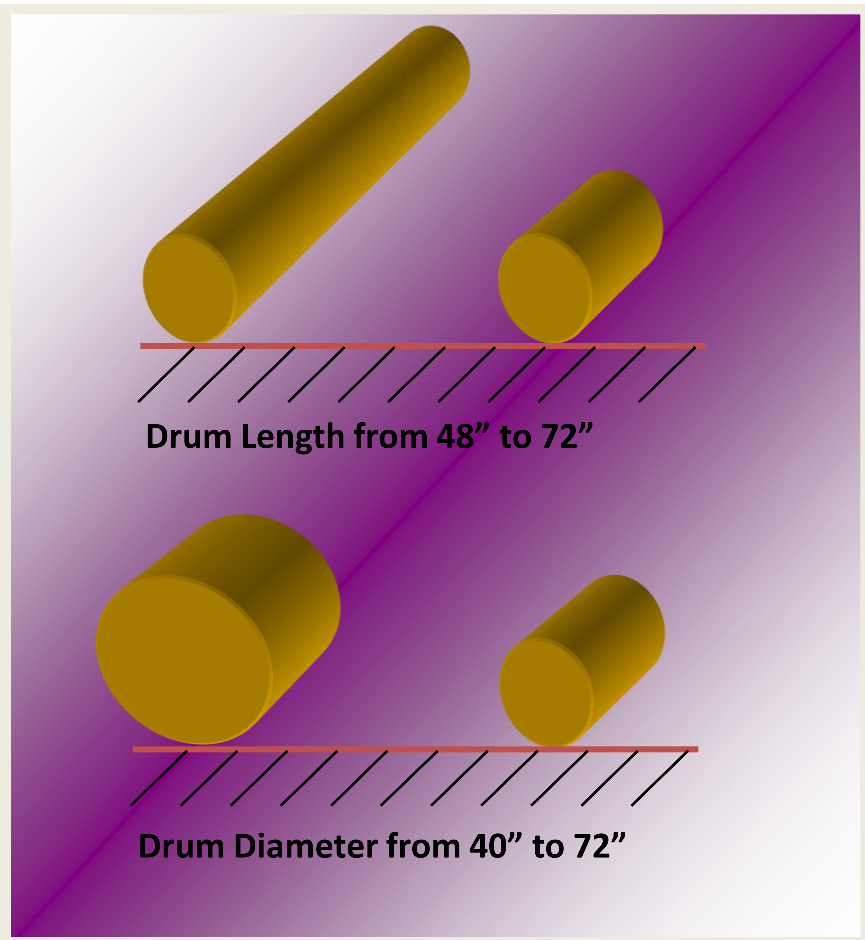


Heavy footed compactors with large feet that fully penetrate a loose lift of soil are ideal.

- Minimum specifications:

- weight : 18000 kg
- foot length: 18 cm to 20 cm
- number of passes: 5

Drum



Drum may be loaded with

Water

Soil

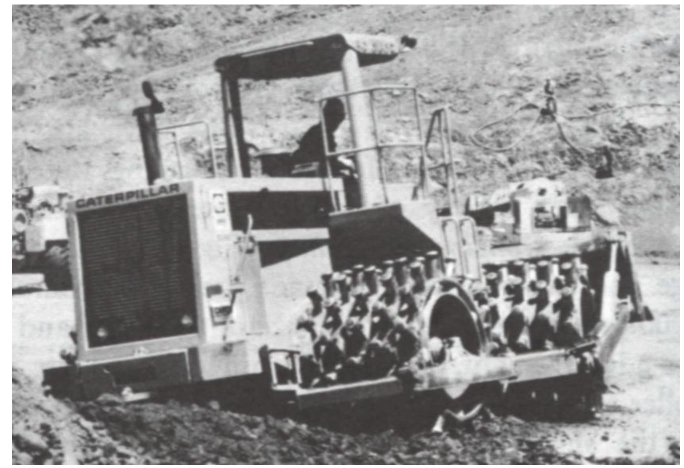
Loaded weight Vary from

6,000 #

80,000 #

Sheepsfoot rollers

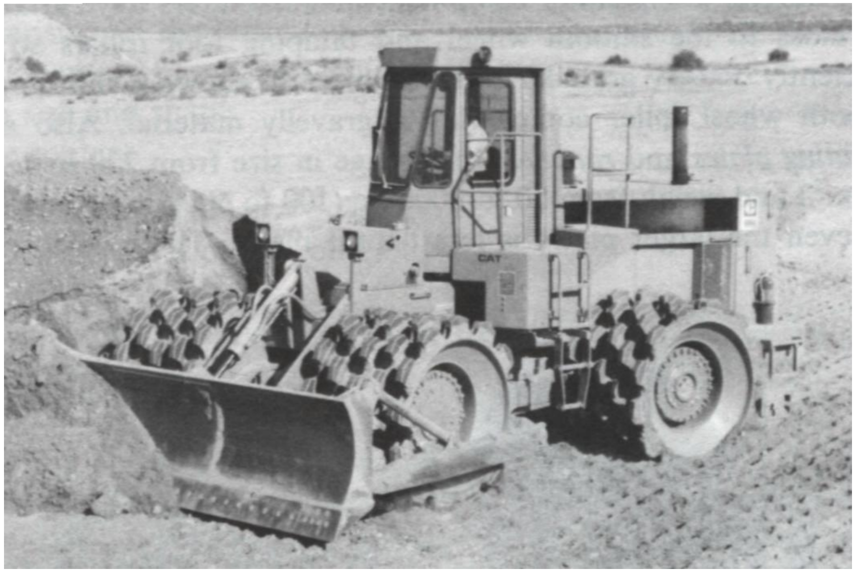
- Suitable for:
  - fine grained soils
  - sands and gravels, with >20% fines



- Unsuitable for:
  - very coarse soils
  - uniform gravels

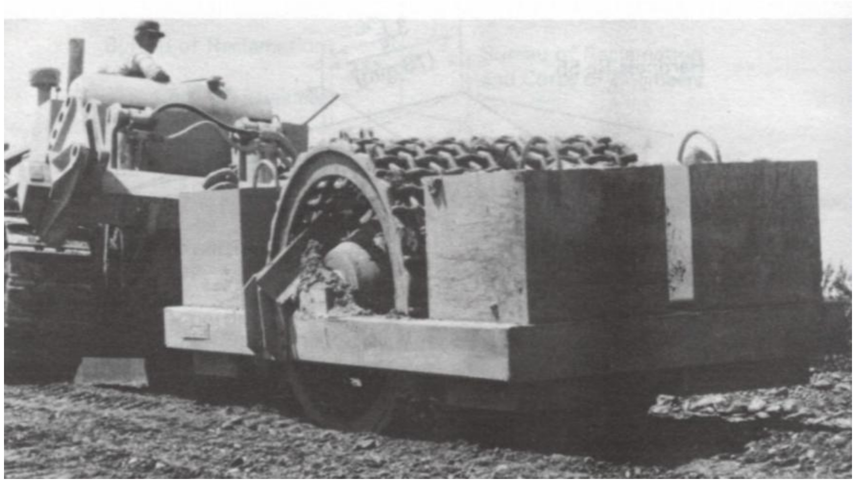
Tamping foot roller

- About 40% coverage
- Contact pressure is from 1400 to 8400 kPa
- It is best for compacting fine-grained soils (silt and clay).
- Compactive effort: static weight and kneading.



Mesh (or grid pattern) roller

- 50% coverage
- Masses range from 5-12 Tones
- Contact pressure is from 1400 to 6200 kPa
- Compactive effort: static weight and vibration.
- High towing speed, the material is vibrated, crushed, and impacted.

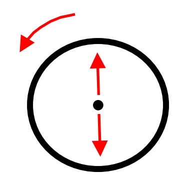


- Suitable for:
  - well-graded sands
  - soft rocks
  - stony soils with fine fractions

- **Unsuitable for:**
  - **uniform sands**
  - **Silty sands**
  - **Very soft clays**

Vibrating drum on smooth-wheel roller

- Vertical vibrator attached to smooth wheel rollers.
- The best explanation of why roller vibration causes densification of granular soils is that particle rearrangement occurs due to cyclic deformation of the soil produced by the oscillations of the roller.
- Compactive effort: static weight and vibration.
- Suitable for granular soils



Vibrating Plate & Power Rammer

- Range from hand-guided machines to larger roller combinations
- Suitable for:
  - most soils with low to moderate fines content
- Unsuitable for:
  - large volume work
  - wet clayey soils

- Also called a 'trench tamper'
- Hand-guided pneumatic tamper
- Suitable for:
  - trench back-fill
  - work in confined areas
- Unsuitable for:
  - large volume work

Compactor Zones of Application

100 % Clay — 100 % Sand — Rock	Compactive Effort
Sheepsfoot	Static wt, Kneading
Grid	Static wt, Kneading
Vibratory	Static wt, Vibration
Smooth steel drums	Static wt
Multitired pneumatic	Static wt, Kneading
Heavy pneumatic	Static wt, Kneading
Towed tamping foot	Static wt, Kneading
High speed tamping foot	Static wt, Kneading, impact, Vibratio

Equipments & Soil Type

- Special compaction equipment is then used to compact this lift of soil:

Equipment Type	Soil Type
Smooth-Wheeled Rollers	sands & gravels
Pneumatic Rubber-Tired Rollers	silts & clays
Sheepsfoot Rollers	silts & clays
Vibratory Rollers	sands & gravels
Vibratory Tampers	sands & gravels

Variables-Vibratory Compaction

- There are many variables which control the vibratory compaction or densification of soils.

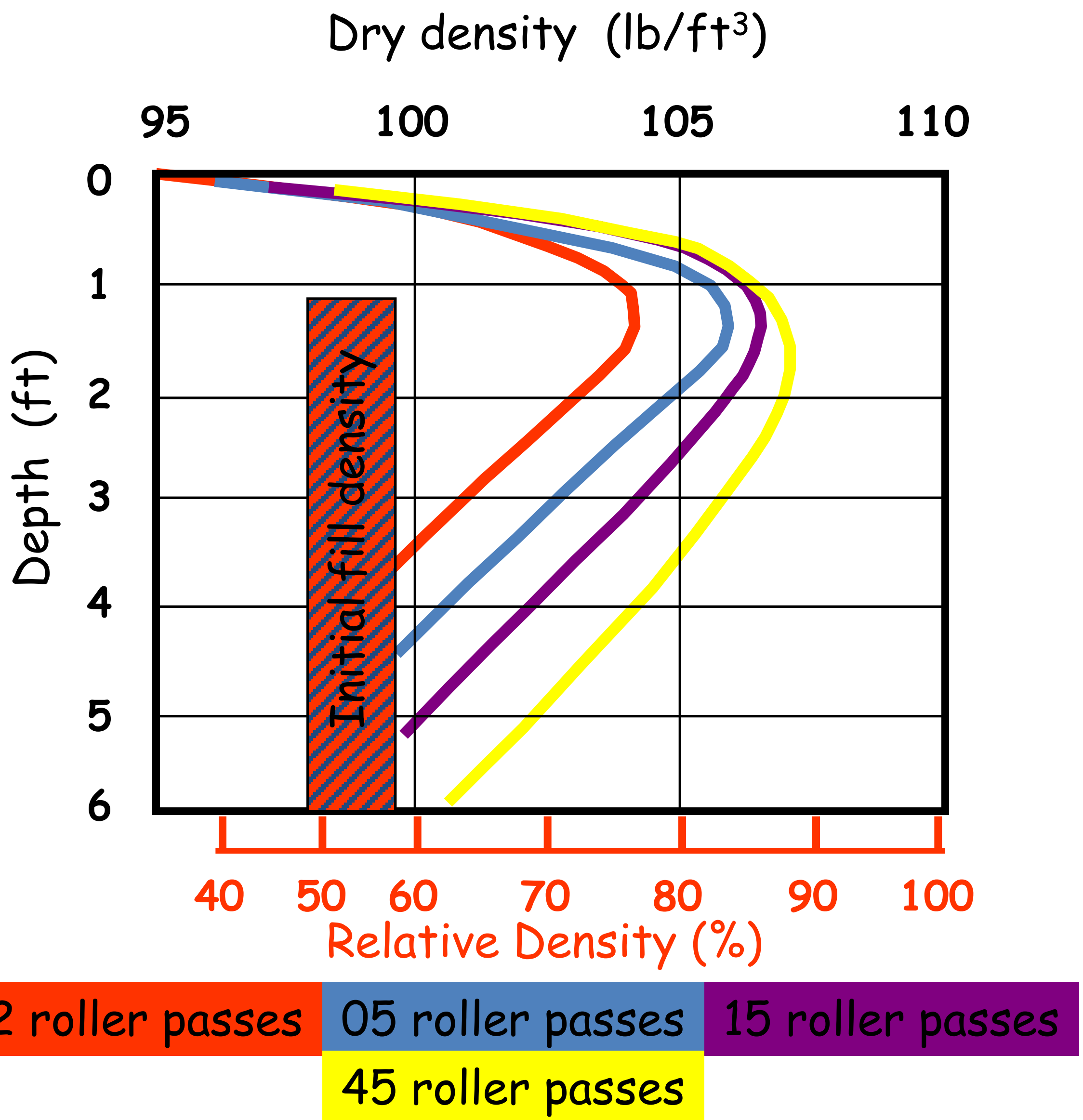
  Characteristics of the compactor:
  - (1) Mass, size
  - (2) Operating frequency and frequency range

  Characteristics of the soil:
  - (1) Initial density
  - (2) Grain size and shape
  - (3) Water content

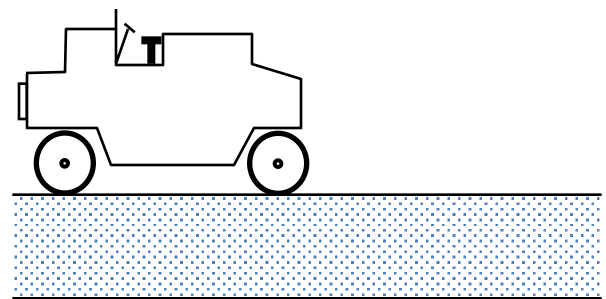
  Construction procedures:
  - (1) Number of passes of the roller
  - (2) Lift thickness
  - (3) Frequency of operation vibrator
  - (4) Towing speed

Holtz and Kovacs, 1981

Roller Passes



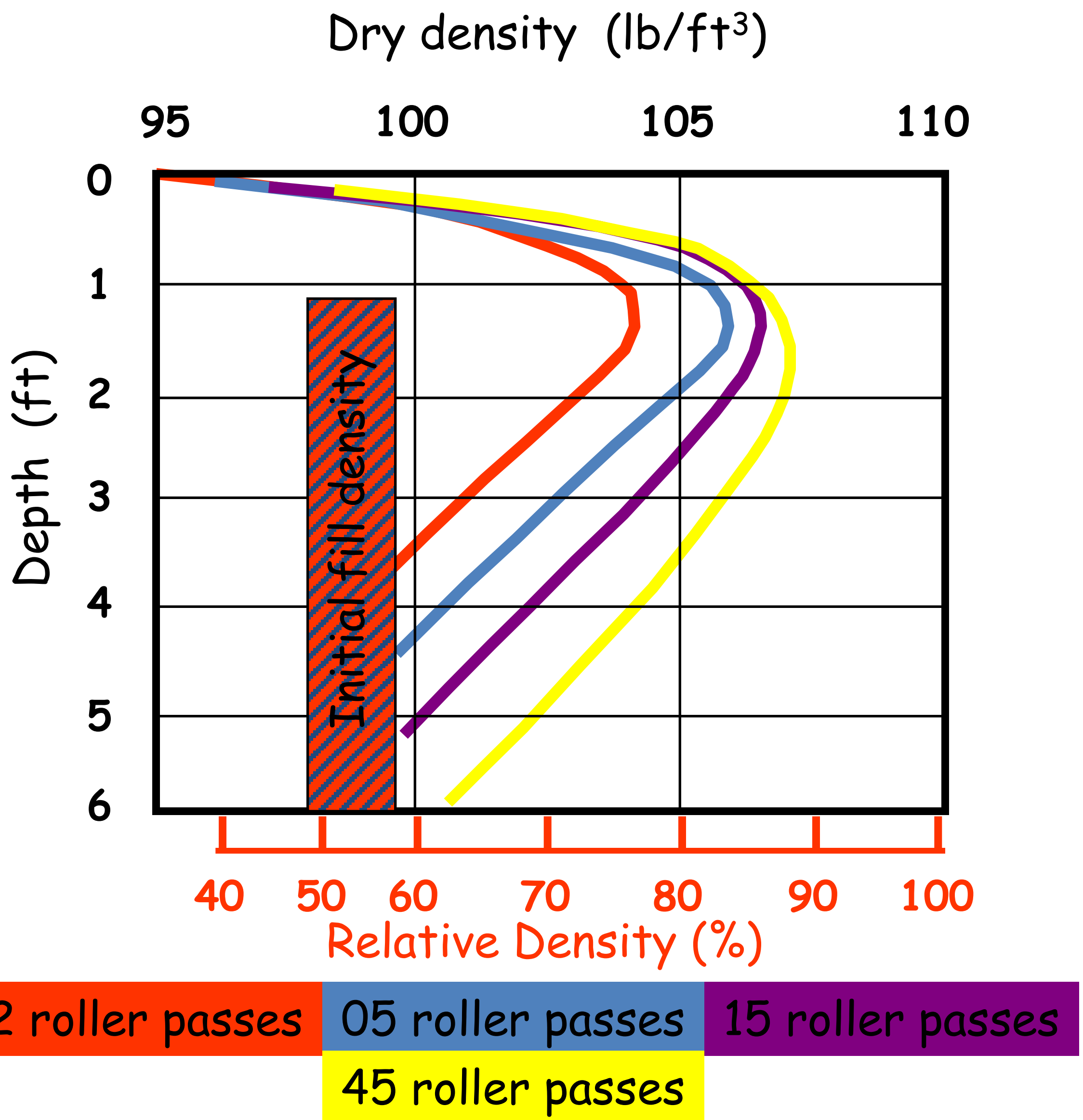
When compacting past five or so coverage's, there is not a great increase in density



- 240 cm think layer of northern Indiana dune sand
- 5670 kg roller operating at a frequency of 27.5 Hz.

Holtz and Kovacs, 1981

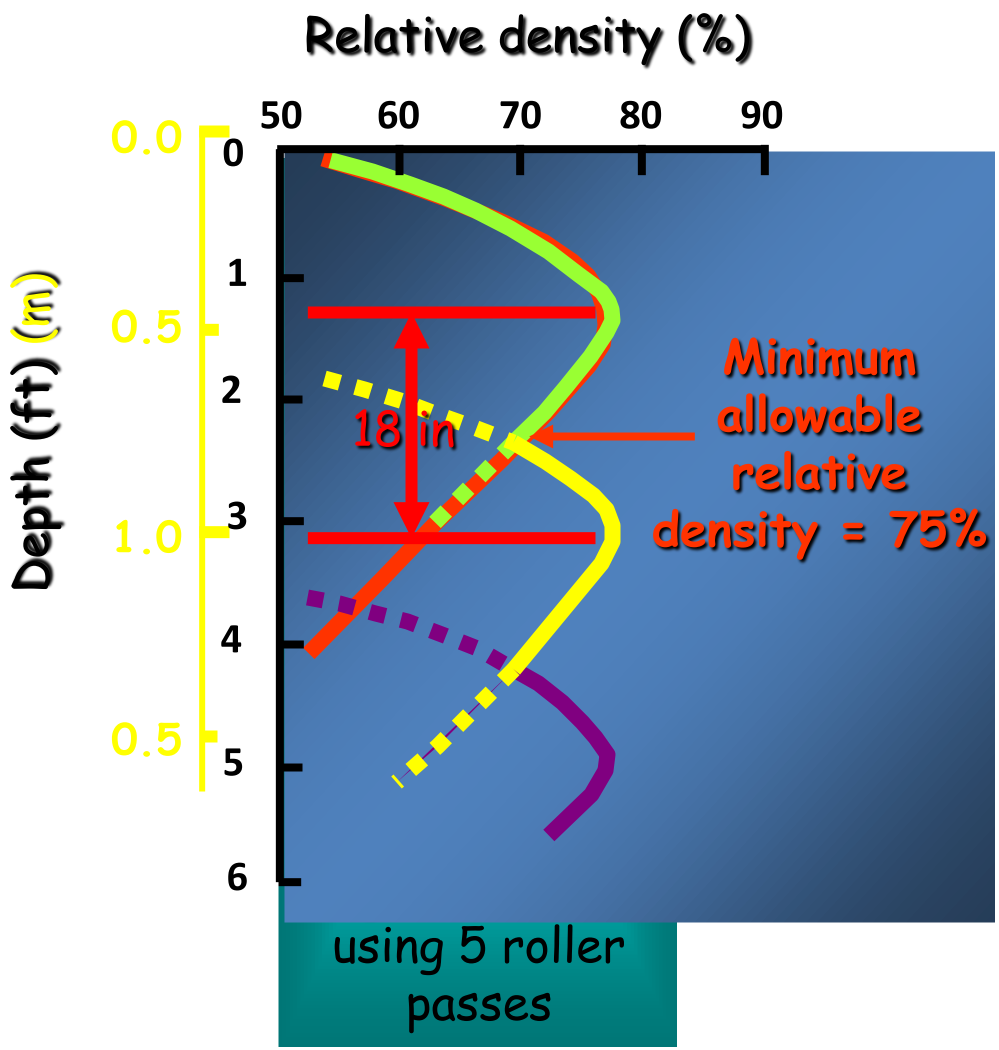
Roller Passes



- Low Compaction at the surface
- Max. Dr. is approximately ½ meter bellow the surface.
- Most effective compaction is done during the first 5-7 passes.

Determine the Lift Height

- For most compaction equipment, lift thicknesses should typically be on the order of six inches (6") or 15cm if no experience or testing



Determine the Lift Height

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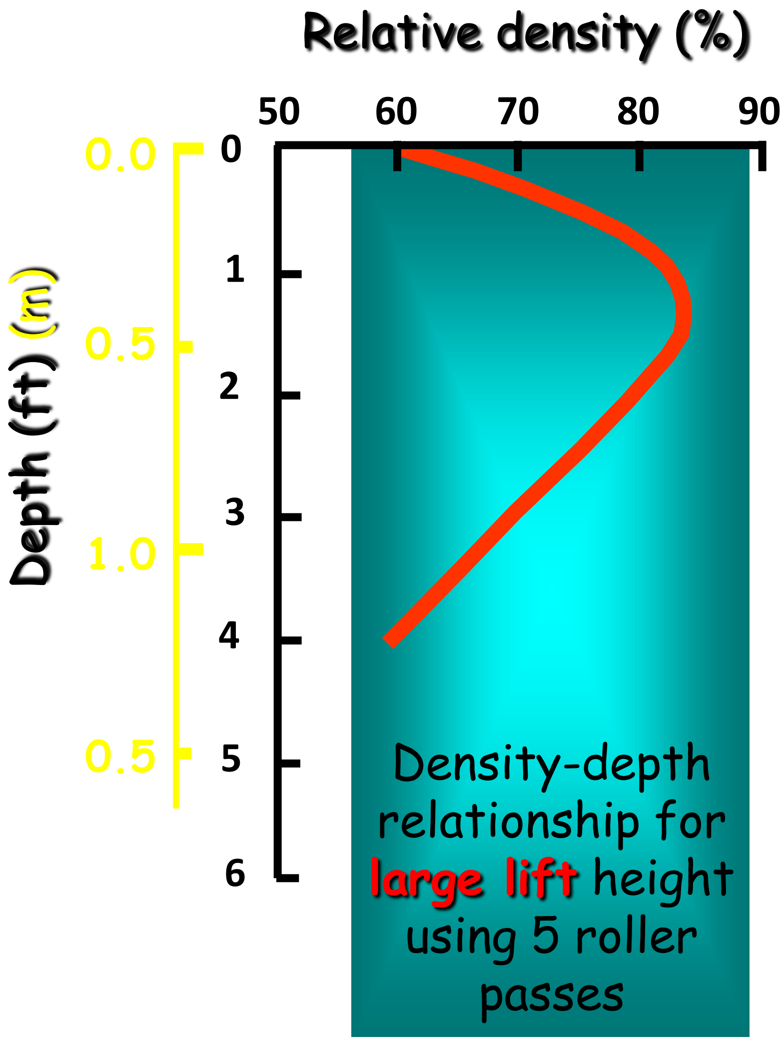
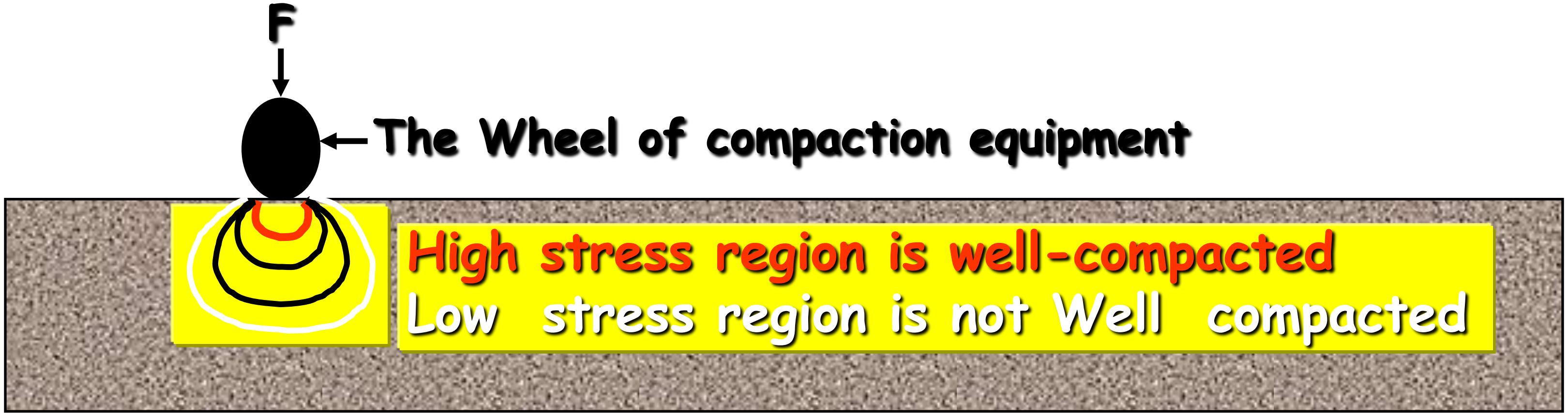
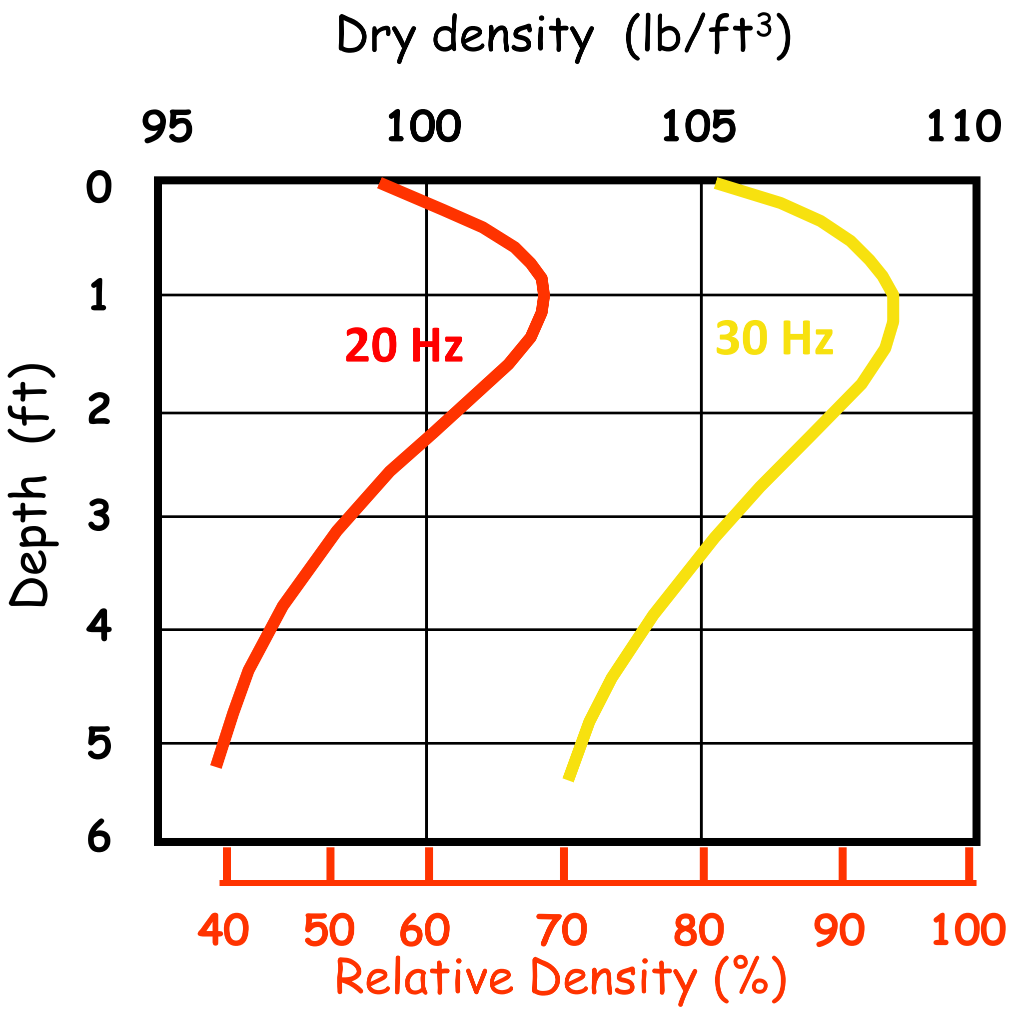


Lift Thickness

- If lift thicknesses are too large:
  - Soil at the top of the lift will be well-compacted.
  - Soil at the bottom of the lift will not be compacted. Why?
  - This is sometimes called the Oreo-Cookie effect.

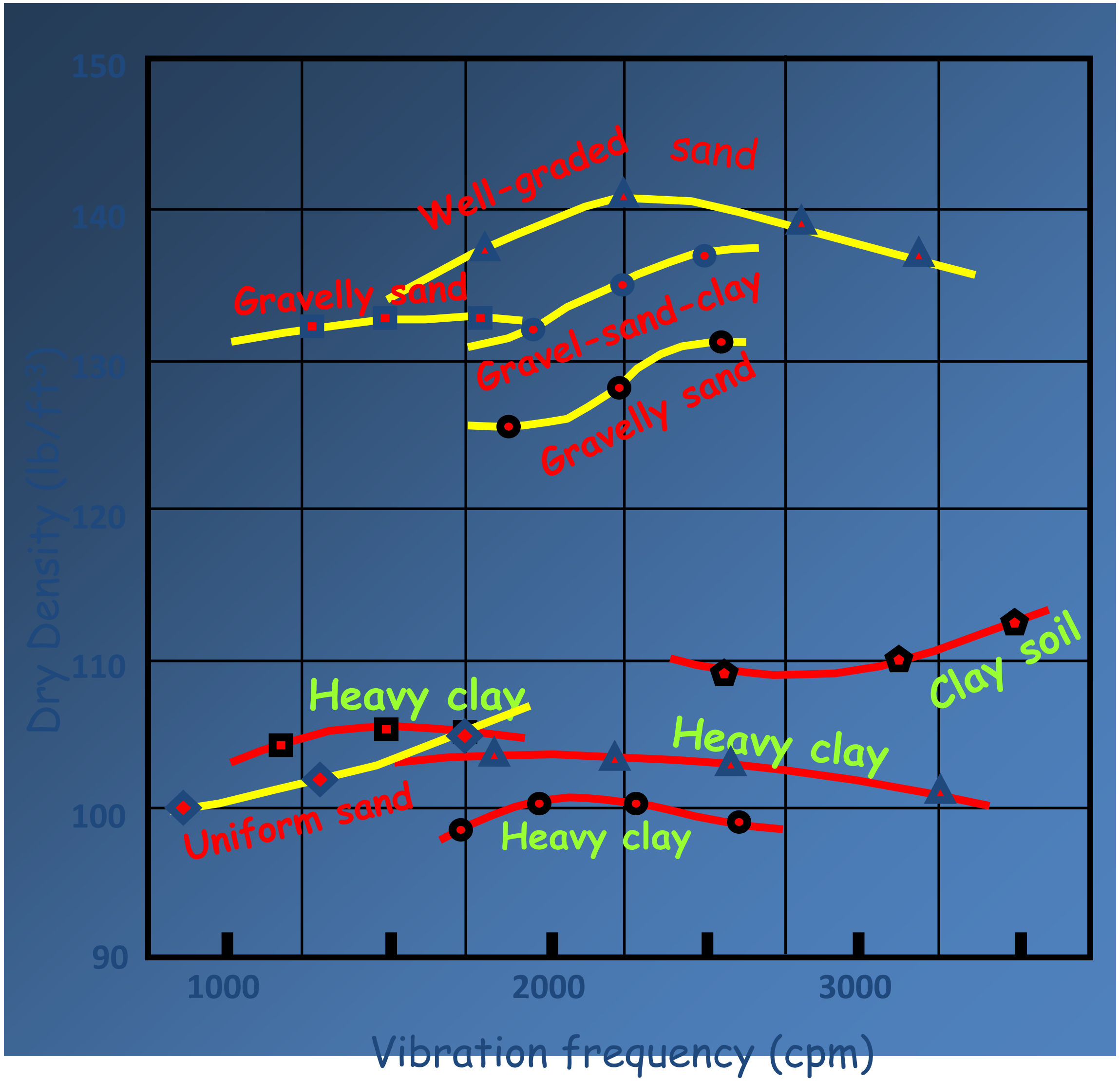


Frequency

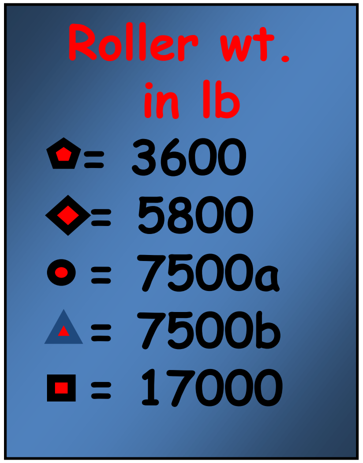


- Compacted Density increases with increasing operation frequency.
- The operating frequency should be at least as large as the resonant frequency to obtain the most efficient use of the

Frequency



The frequency at which a maximum density is achieved is called the optimum frequency.

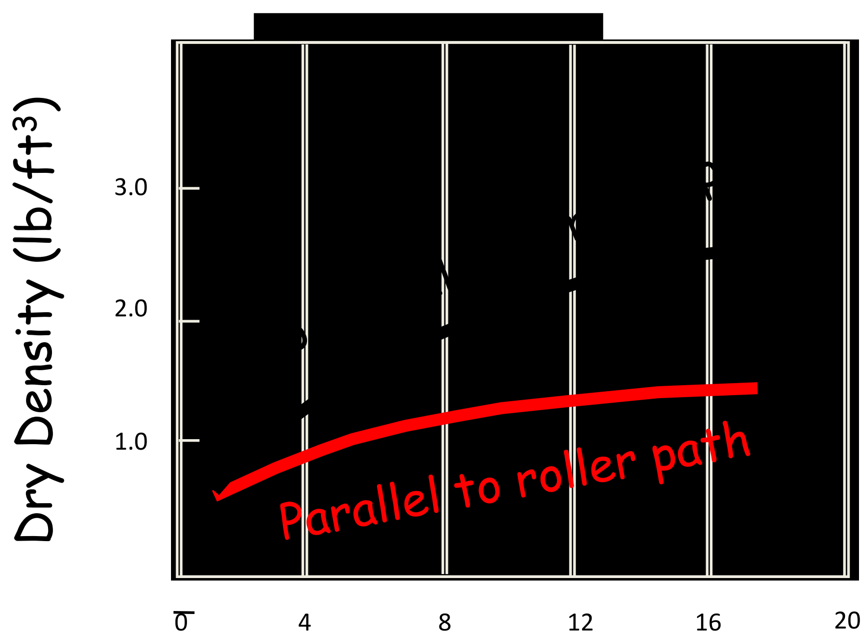


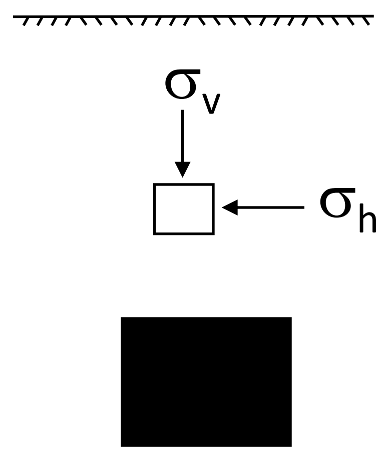
Stresses

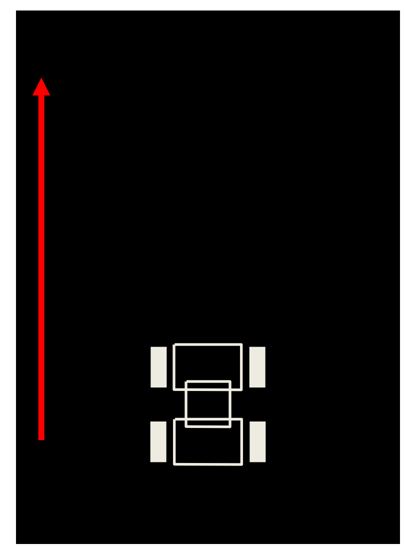
Horizontal stress measurements indicated that lateral stresses are much grater than at rest condition.

Lateral stresses were found to increase with

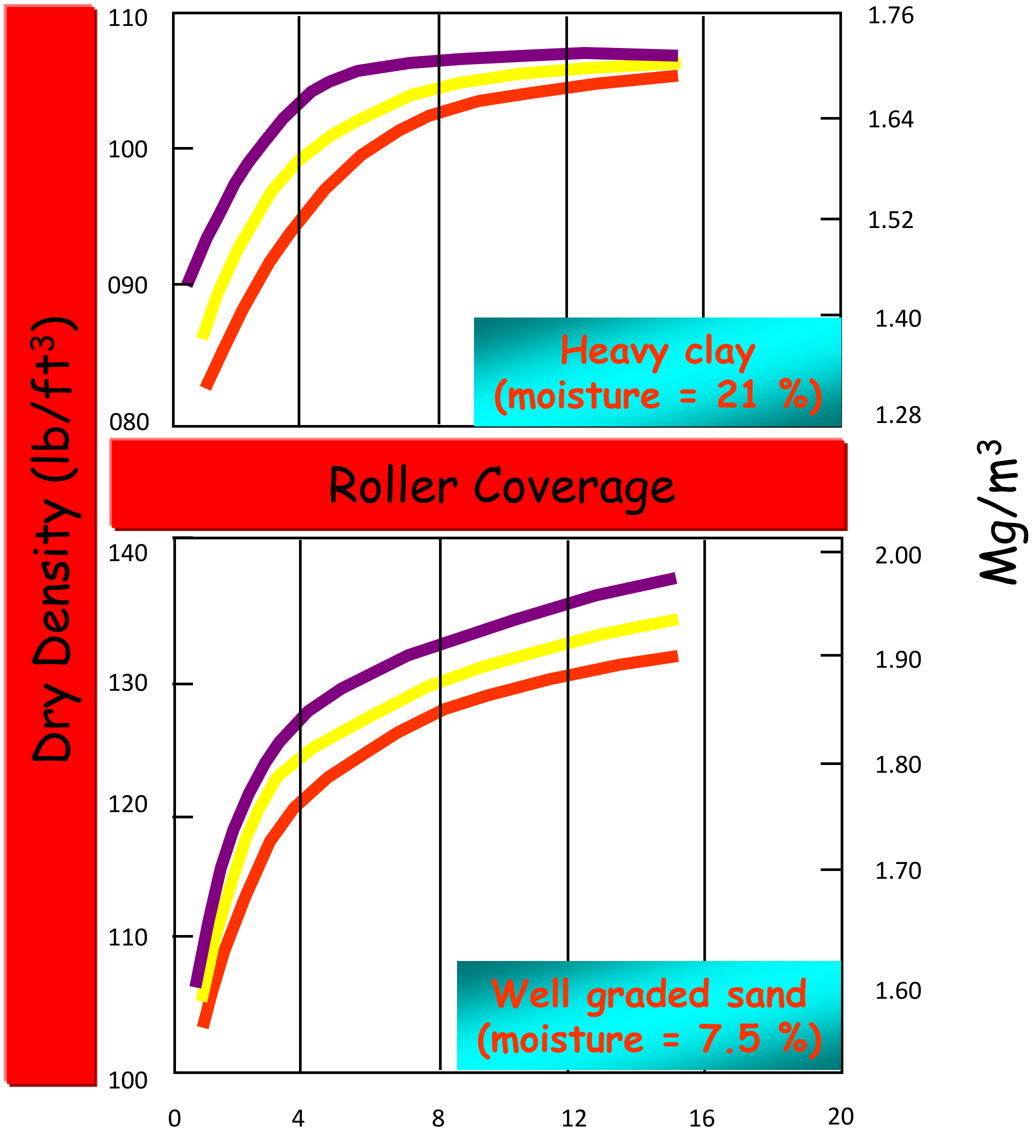
- Number of passes
- Operating frequencies







Roller Travel Speed



For a given number of passes, a higher density is obtained if the vibrator is towed more slowly.

For the same speed, the higher the number of passes the higher the density

Settlement and Consolidation

§ 4 *Settlement and Consolidation*

General
Oedometer test
Preconsolidation pressure
Consolidation settlement
Terzaghi's theory
Degree of consolidation

§ 4 *Settlement and Consolidation*

§ 4.1 General

- *compressibility*

–volume changes in a soil when subjected to pressure –giving *AMOUNTS* of settlement

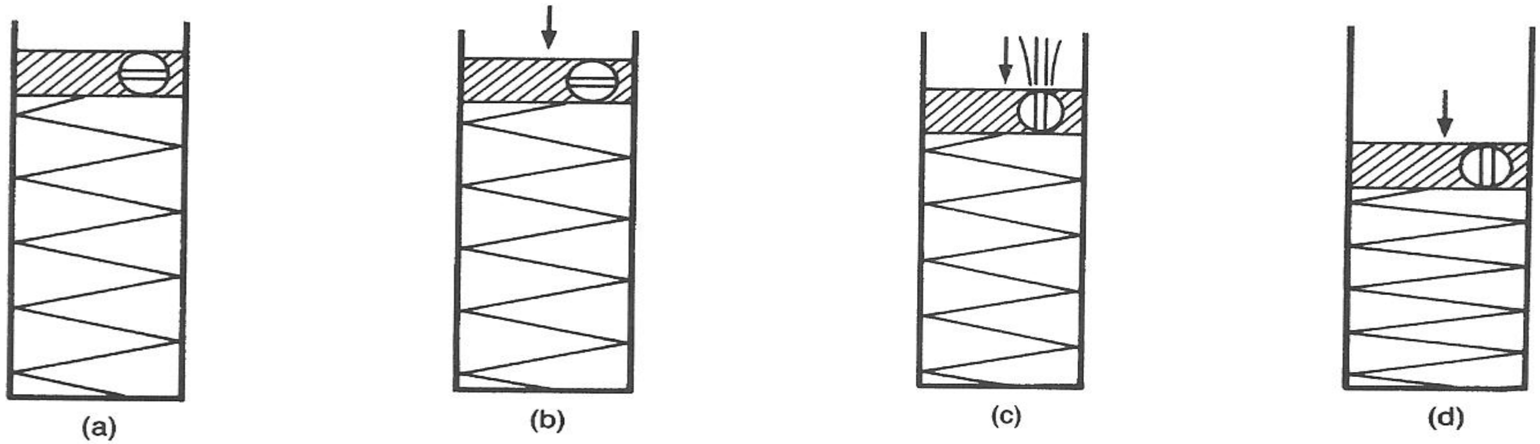
- *consolidation*

-rate of volume change with time –giving TIME to produce an amount of settlement required

One-dimensional consolidation

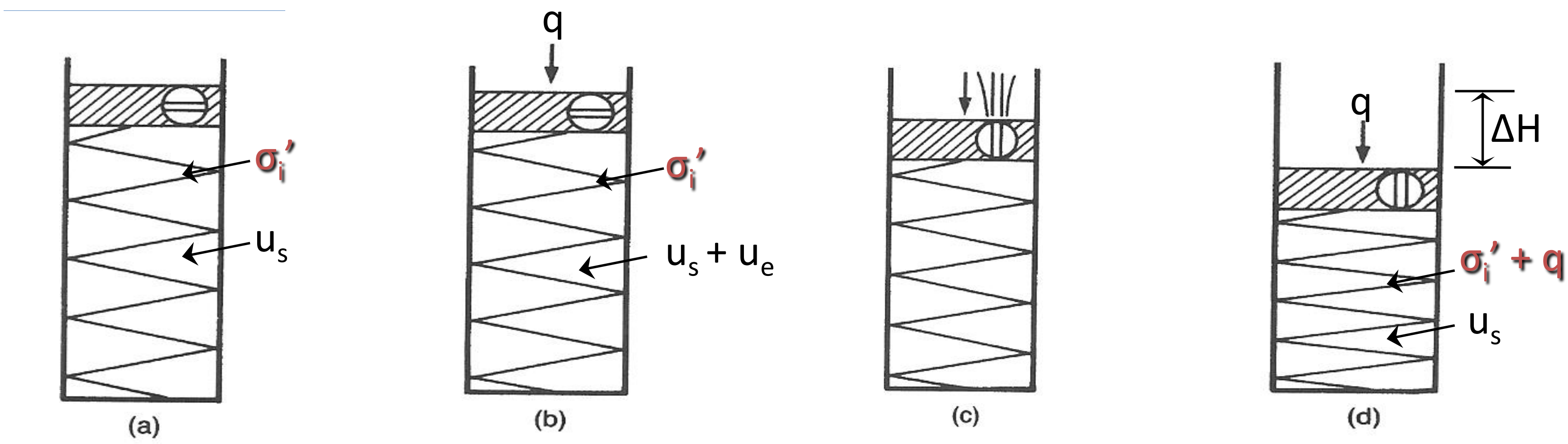
- simplest case
- load applied over a small portion of total clay layer area
- zero lateral strain
- swelling?
- opposite of consolidation
- Gradual volume increase
- Negative excess pore water pressure

Analogy (Craig, Figure 3.2)



- housed in a sealed chamber filled with water
- spring (soil skeleton) starts to compress
- the valve in the seal is closed
- the spring (soil skeleton) feels nothing

in terms of pore water pressure:



- as water drains, the spring (skeleton) gradually carries more of the load and the effective stress on soil $\sigma_i' + \ldots$, excess pore water pressure dissipates
- The only thing we really notice is that, at the end of this (months, years), the soil has settled by ΔH
- finally soil skeleton carries all the load and the effective stress on the soil is $\sigma_f' = \sigma_i' + q$ and $u = u_s$ again

Stress History

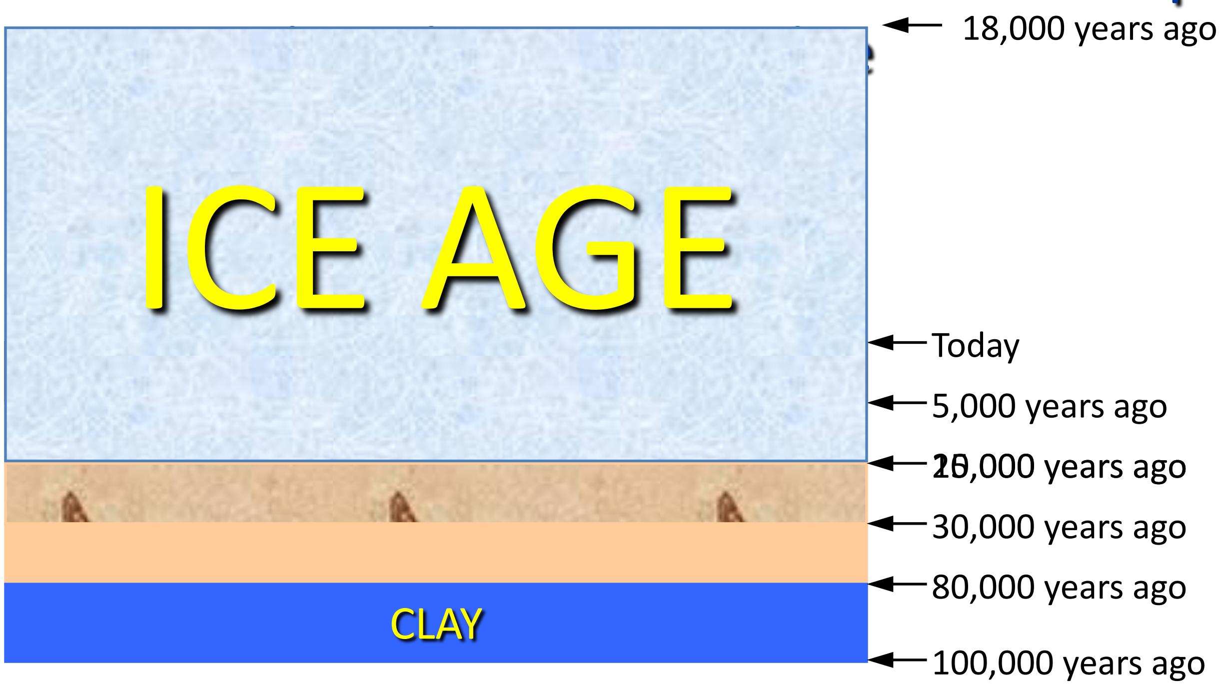
Normally Consolidated Clay

- If the present effective stress is the maximum to which the clay has ever been subjected

Stress History

Overconsolidated Clay

- If the effective stress at some time in the past has been



Preconsolidation Pressure, σ'_c

- The maximum effective stress that has acted on the clay in the past
- Obtained through analysis of laboratory test data (Casagrande's graphical method)
- σ'_c should not be exceeded during construction to ensure minimum compression

Overconsolidation Ratio, OCR

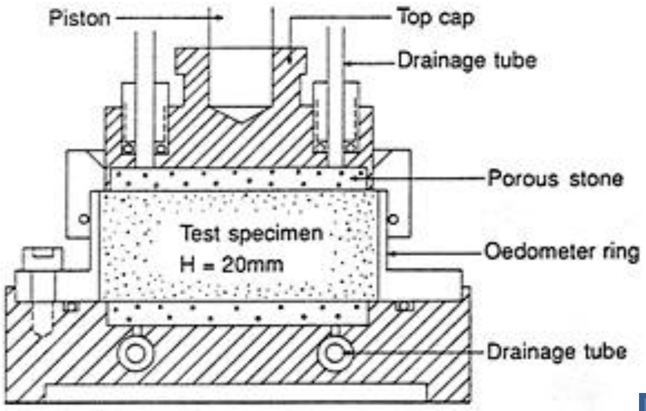
$$OCR = \frac{\sigma'_c}{\sigma'_{present}}$$

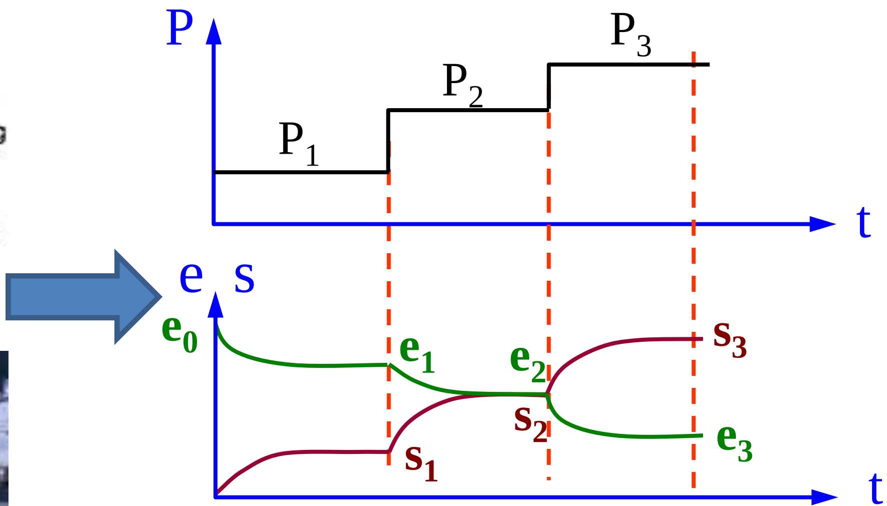
- Normally Consolidated: OCR =1
- Overconsolidated: OCR > 1

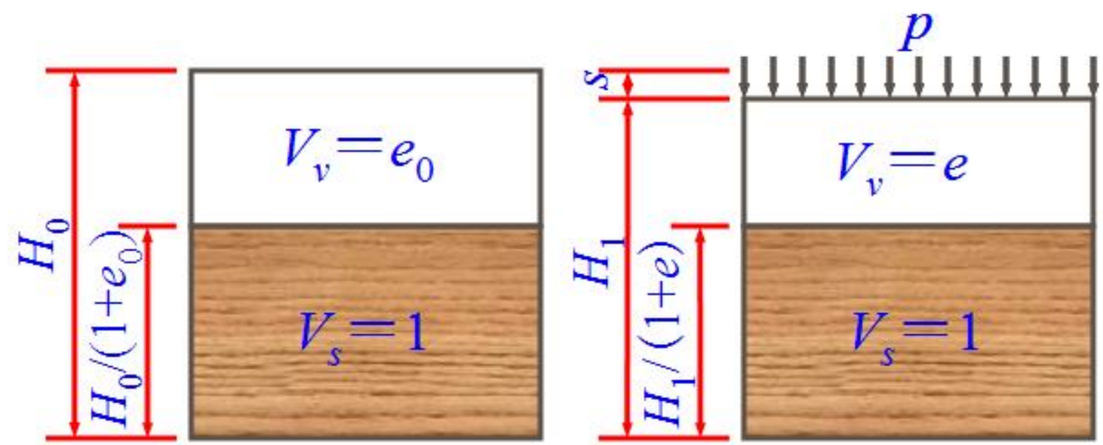
How can OCR > 1?

- Erosion of overburden
- Recession of glacial ice sheets
- Permanent rise of water table

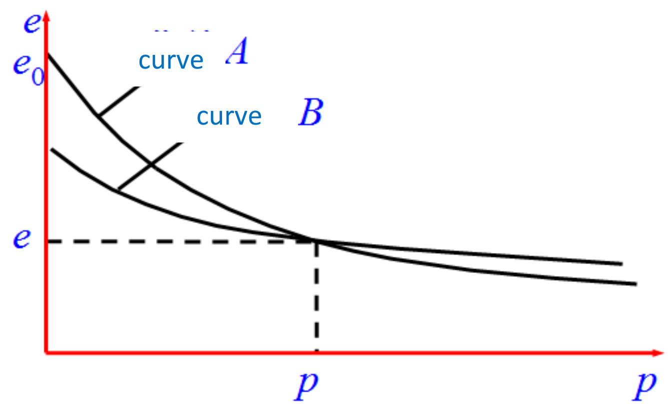
§ 4.2 Oedometer test





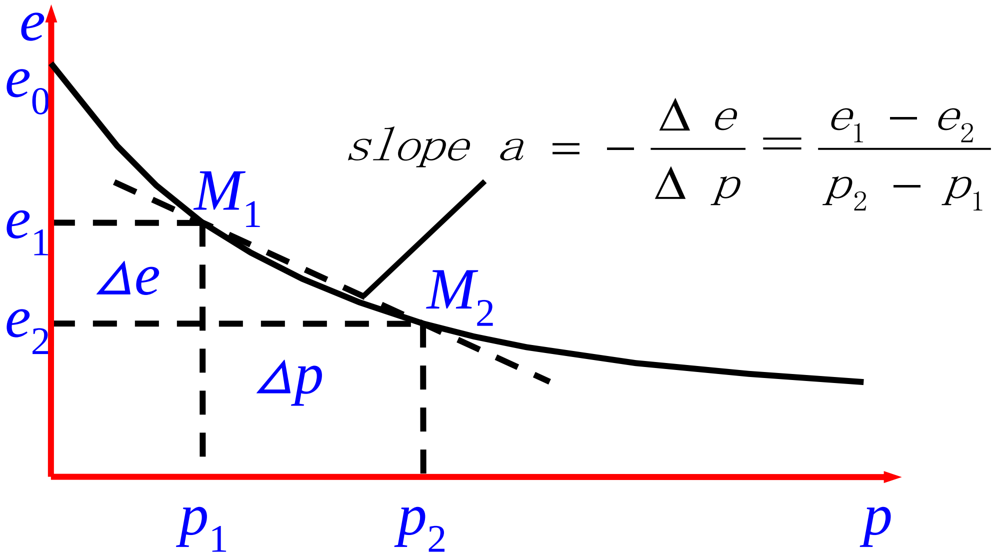


$$\frac{H_0}{1+e_0} = \frac{H_1}{1+e} \quad \Longrightarrow \quad e = e_0 - \frac{s}{H_0}(1+e_0)$$



e–p curve

Compression coefficient a



curve $e-p$

$$a = -\frac{de}{dp}$$

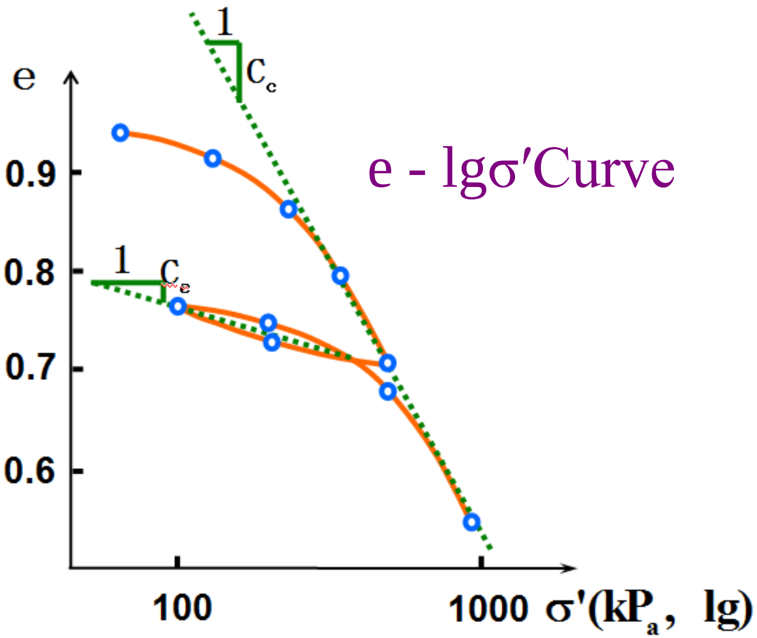
$$a = -\frac{\Delta e}{\Delta p} = \frac{e_1 - e_2}{p_2 - p_1}$$

Evaluation of compression with $a_{1\text{-}2}$

- $a_{1\text{-}2} < 0.1\text{MPa}^{-1}$, **Low compressibility**
- $0.1\text{MPa}^{-1} \leq a_{1\text{-}2} < 0.5\text{MPa}^{-1}$, **Middle compressibility**
- $a_{1\text{-}2} \geq 0.5\text{MPa}^{-1}$, **High compressibility**

compression index Cc

$$C_c = \frac{e_1 - e_2}{\lg p_2 - \lg p_1} = -\frac{(e_1 - e_2)}{\lg \dfrac{p_2}{p_1}}$$



$$C_c = \frac{e_1 - e_2}{\lg \dfrac{p}{p_c}} = -\frac{de}{\lg \dfrac{dp}{p}}$$

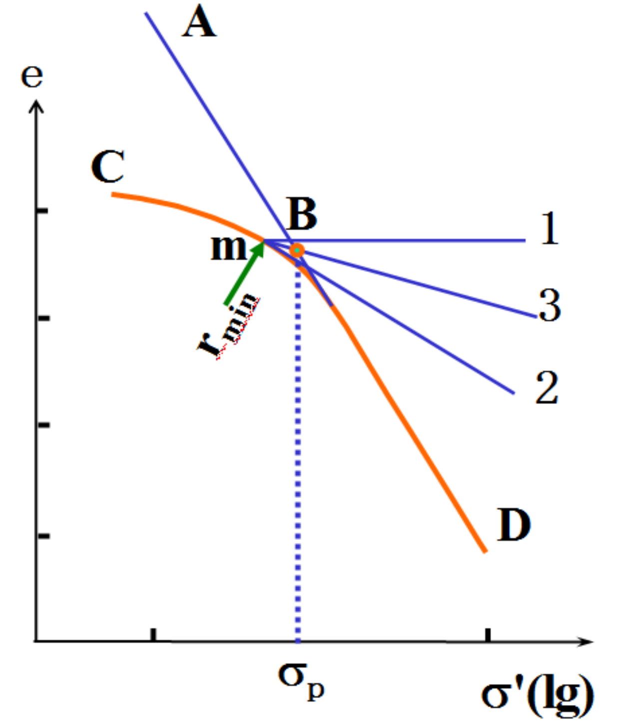
$$a_v = -\frac{de}{dp} = \frac{C_c}{p}$$

§ 4.3 Preconsolidation pressure

Preconsolidation pressure-the maximum effective vertical stress that has acted on the clay in the past

$$OCR = \frac{\sigma_p}{\sigma_s}$$

OCR=1 : lack consolidation
OCR>1 : normal consolidation
OCR<1 : over consolidation



How to obtain the preconsolidation pressure:

1 Produce back the straight-line part (BC) .

2 Determine the point (D) of maximum curvature on the recompression part (AB) of the curve.

3 Draw the tangent to the curve at D and bisect the angle between the tangent and the horizontal through D.

4 The vertical through intersection point of the bisector and CB produced gives the approximate

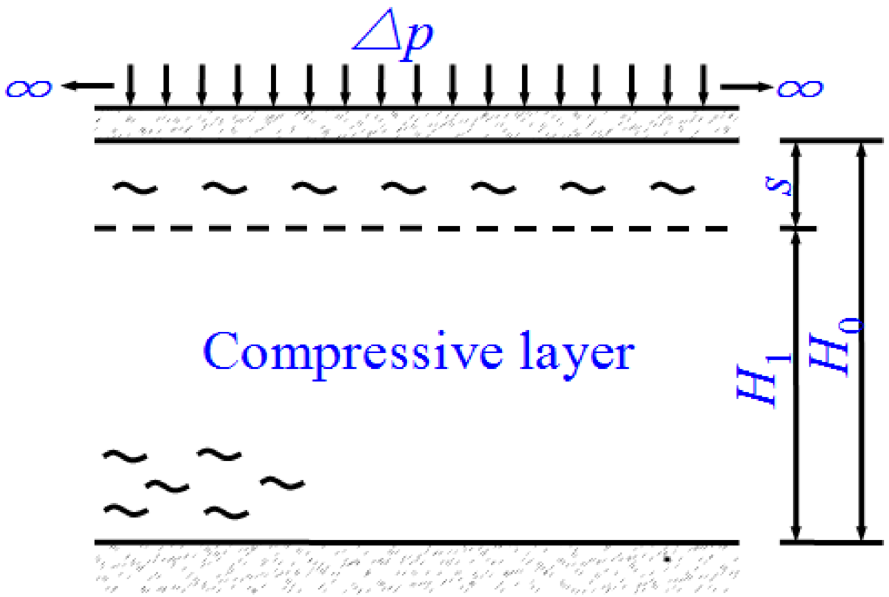
§ 4.4 Consolidation settlement

$$\mathrm{S} = \mathrm{S_d} + \mathrm{S_c} + \mathrm{S_s}$$

$$s = H_1 - H_2 = \frac{e_1 - e_2}{1 + e_1} H_1$$

$$a = -\frac{\Delta e}{\Delta p} = \frac{e_1 - e_2}{p_2 - p_1}$$

$$s = \frac{a}{1 + e_1}(p_2 - p_1)H_1 = \frac{\Delta p}{E_s} H_1$$



- ✓ coefficient of volume compressibility or the compression index
- ✓ Consider a layer of saturated clay of thickness H.
- ✓ an elemental layer of thickness dz at depth z.

◆Curve of gravity stress and additional stress at the central of base

◆Determine the calculation depth

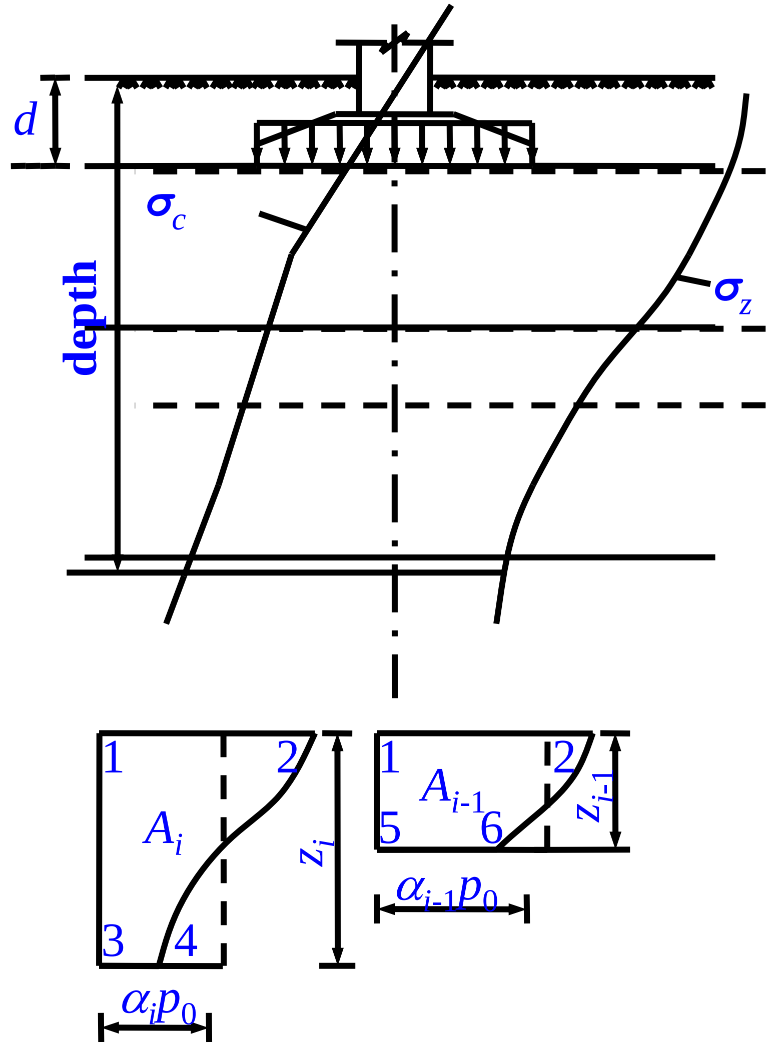
◆Determine the layer

◆The settlement of each layer

$$\Delta s_i = \frac{e_{1i} - e_{2i}}{1 + e_{1i}} h_i$$

◆The whole settlement

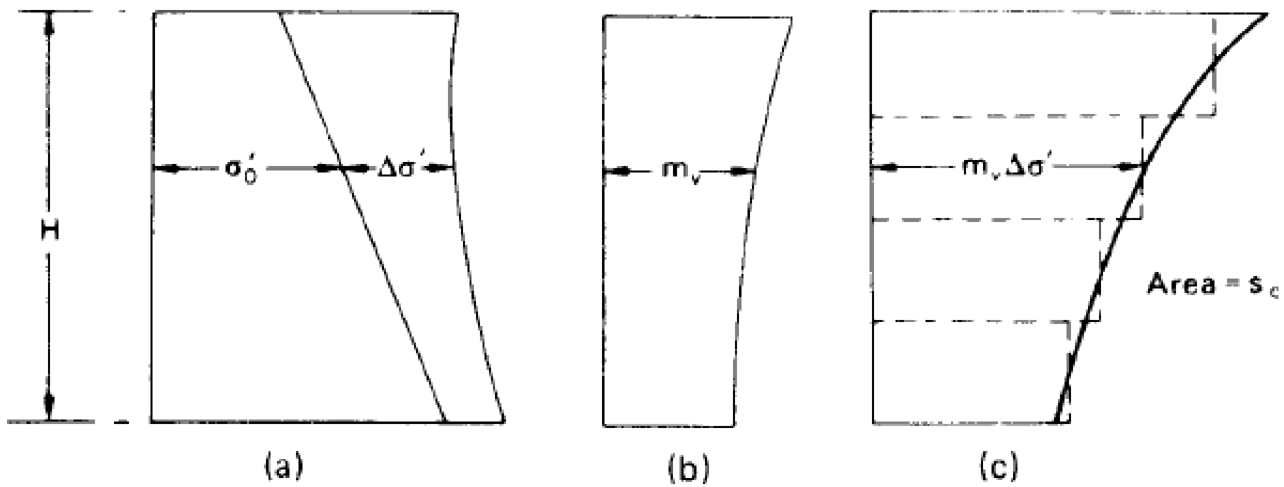
$$s = \sum_{i=1}^{n} \Delta s_i$$

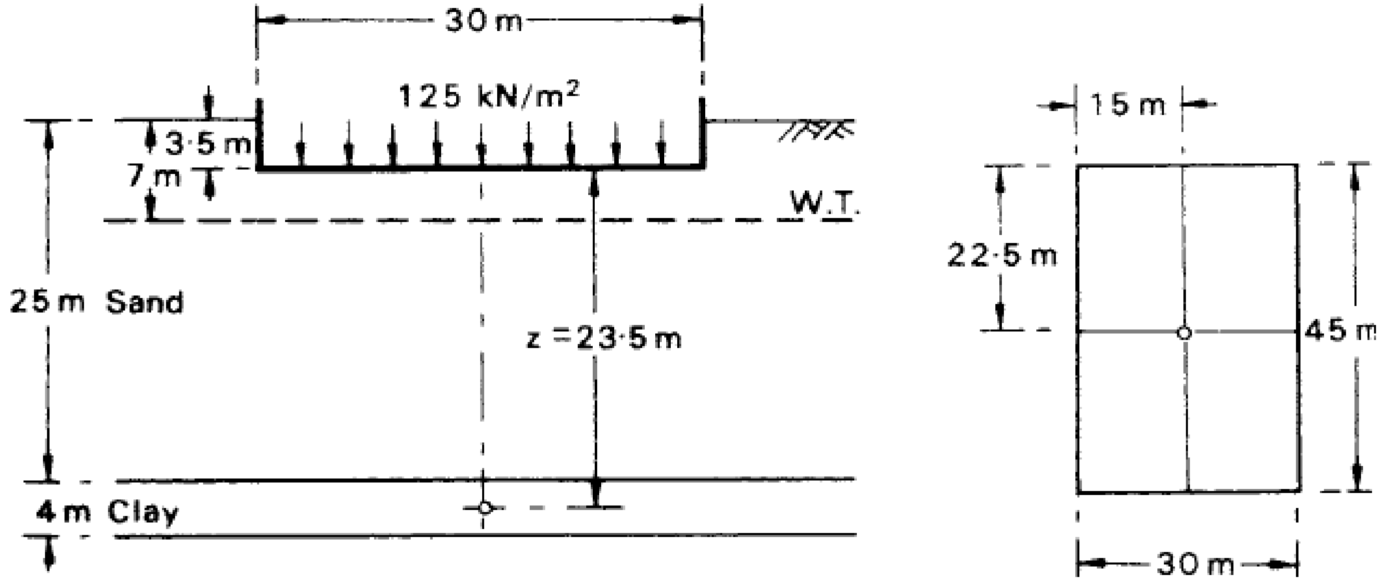


Example 1

A building is supported on a raft 45×30 m, the net foundation pressure (assumed to be uniformly distributed) being $125\ \text{kN/m}^2$. The soil profile is as shown in Figure 7.9. The value of m_v for the clay is $0.35\ \text{m}^2/\text{MN}$. Determine the final settlement under the centre of the raft due to consolidation of the clay.

The clay layer is thin relative to the dimensions of the raft, therefore it can be assumed that consolidation is approximately one-dimensional. In this case it will be





sufficiently accurate to consider the clay layer as a whole. Because the consolidation settlement is to be calculated in terms of m_v, only the effective stress *increment* at mid-depth of the layer is required (the increment being assumed constant over the depth of the layer). Also, $\Delta\sigma' = \Delta\sigma$ for one-dimensional consolidation and can be evaluated from Figure 5.10.

At mid-depth of the layer, $z = 23.5\,\text{m}$. Below the centre of the raft:

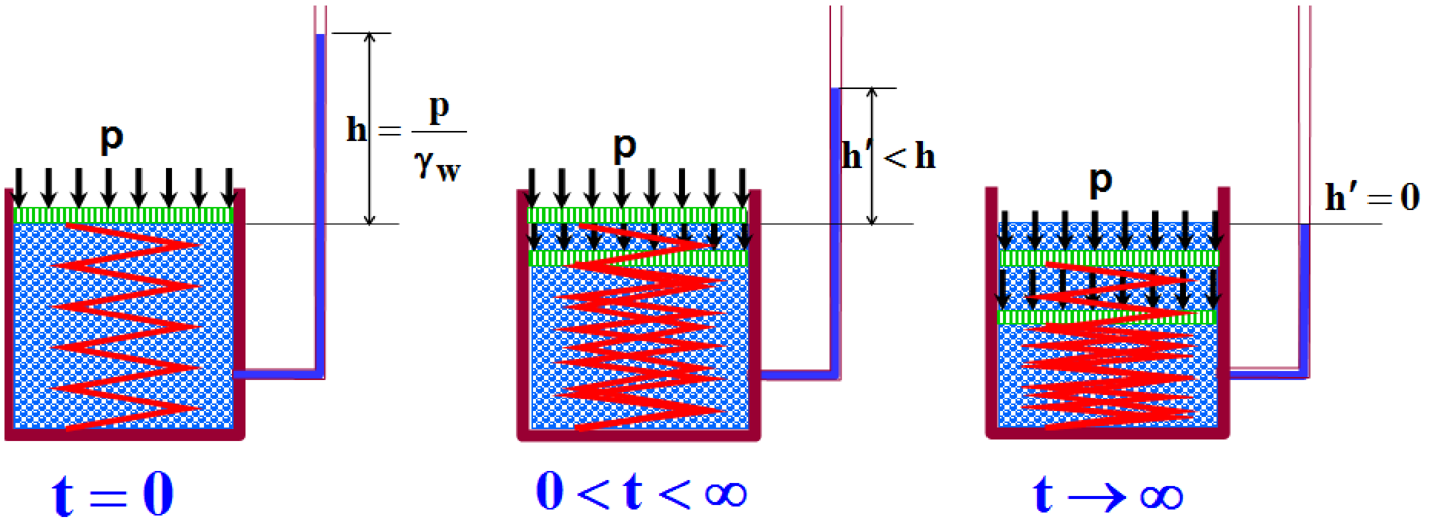
$$m = \frac{22.5}{23.5} = 0.96 \qquad n = \frac{15}{23.5} = 0.64 \qquad I_r = 0.140$$

$$\Delta\sigma' = 4 \times 0.140 \times 125 = 70\,\text{kN/m}^2$$

$$s_c = m_v \Delta\sigma' H = 0.35 \times 70 \times 4 = 98\,\text{mm}$$

§ 4 *Settlement and Consolidation*

§ 4.5 Terzaghis theory of one-dimensional consolidation



The assumptions made in the theory are:

◆1 The soil is homogeneous and fully saturated.

◆2 There is a unique relationship, independent of time, between void ratio and effective stress.

◆3 The solid particles and water are incompressible.

◆4 Compression and flow are one-dimensional (vertical).

◆5 Strains are small.

◆6 Darcy’s law is valid at all hydraulic gradients.

◆7 The coefficient of permeability and volume compressibility remain constant.

The total stress increment

⇩

soil skeleton

⇩

increasing effective stress

⇩

the excess pore water pressure decreases

$$\frac{\partial u}{\partial t} = \frac{k(1+e_1)}{\gamma_w a} \frac{\partial^2 u}{\partial z^2} \qquad c_v = \frac{k(1+e_1)}{a\gamma_\omega}$$

$$u_e = u_i \quad \text{for} \quad 0 \le z \le 2d \quad \text{when} \quad t = 0$$

$$u_e = 0 \quad \text{for} \quad z = 0 \quad \text{and} \quad z = 2d \quad \text{when} \quad t > 0$$

$$u_{z,t} = \frac{4}{\pi}\sigma_z \sum_{m=1}^{\infty} \frac{1}{m} \sin\frac{m\pi^2}{2H} \exp(-\pi^2 m^2 T_v / 4)$$

$$T_v = \frac{c_v}{H^2} t$$

§ 4.6 Degree of consolidation

the consolidation settlement at time t being given

✓by the product of U and

✓the final settlement.

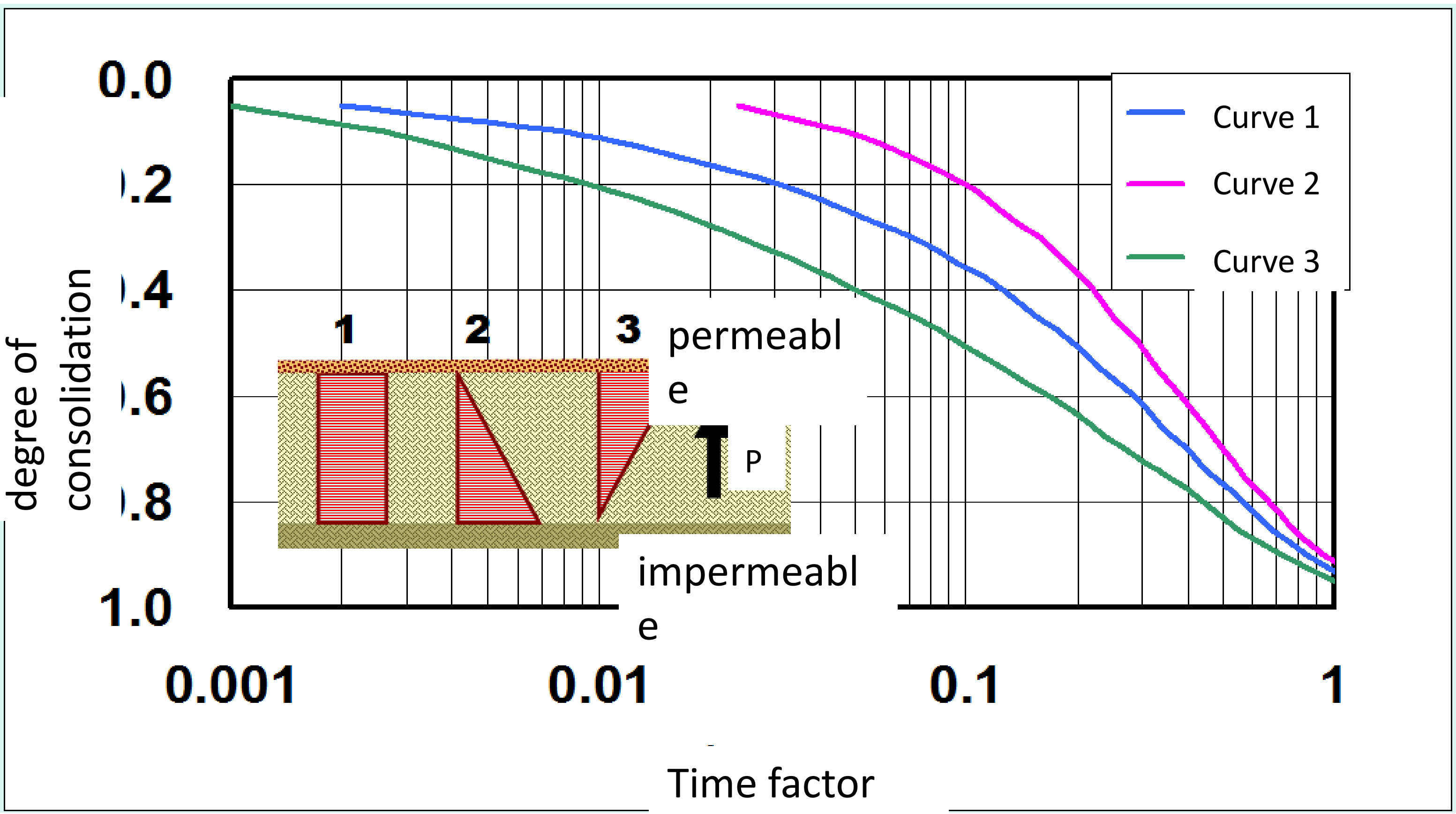
$$U = \frac{s_{ct}}{s_c}$$

$$U_t = \frac{\int \sigma'_{z,t} dz}{\int \sigma_z dz} = \frac{\int \frac{a\sigma'_{z,t}}{1+e_1} dz}{\frac{a\sigma_z}{1+e_1} H} = \frac{S_t}{S_\infty}$$

$$\text{for} \quad U < 0.60, \quad T_v = \frac{\pi}{4} U^2$$

$$\text{for} \quad U > 0.60, \quad T_v = -0.933 \log(1 - U) - 0.085$$

Determine the degree of consolidation



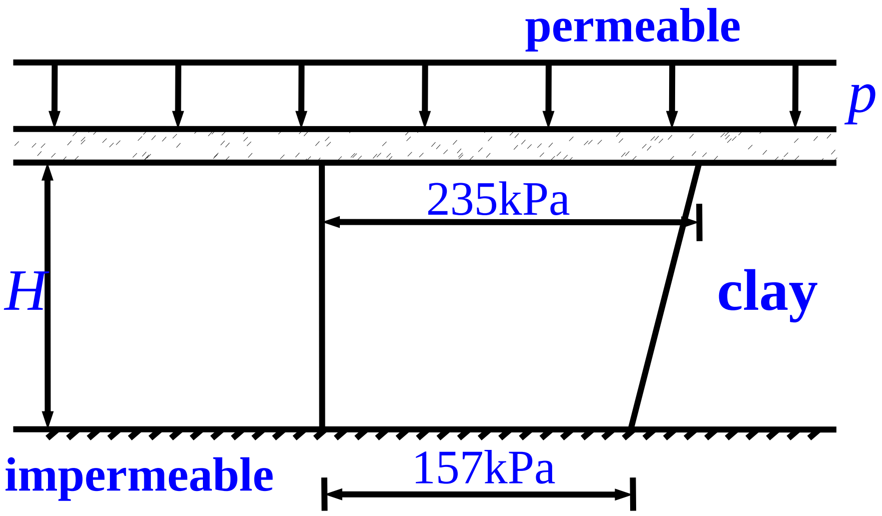
Example 2

✓ H=10m ; e_1=0.8; a=0.00025kPa^{-1}; k=0.02m/**year**

? ① **settlement after one year** S_t

② **time(**t) *when* U_z=0.75

③ **if the bottom layer is permeable, time(**t) *when* U_z=0.75



Solution

- 1. When t=1 year

$$S = \frac{a}{1+e_1}\sigma_z H = 273mm$$

$$c_v = \frac{k(1+e_1)}{a\gamma_w} = 14.4m^2 / year \quad \alpha = \frac{235}{157} = 1.5$$

$$T_v = \frac{c_v}{H^2}t = 0.144$$ From figure U_t=0.45

$$S_t = U_z S = 123mm$$

- 2. If Uz=0.75 From U_z=0.75, α=1.5 then Tv=0.47

$$t = \frac{T_v H^2}{c_v} = 3.26\text{year}$$

- 3. If open layer, Uz=0.75

From U_z=0.75, α=1, H=5m then T_v=0.49

$$t = \frac{T_v H^2}{c_v} = 0.85\text{year}$$

Soil Settlement

Soil Settlement:

Total Soil Settlement = Elastic Settlement + Consolidation Settlement

$S_{total} = S_e + S_c$

Elastic Settlement or Immediate Settlement depends on $\begin{cases} \text{Load Type (Rigid; Flexible)} \\ \text{Settlement Location (Center or Corner)} \end{cases}$

Elastic Settlement $\begin{cases} \text{Theory of Elasticity} \\ \text{Time Depended Elastic Settlement (Schmertman \& Hartman Method (1978)} \end{cases}$

Elastic settlement occurs in sandy, silty, and clayey soils.

Consolidation Settlement (Time Dependent Settlement)

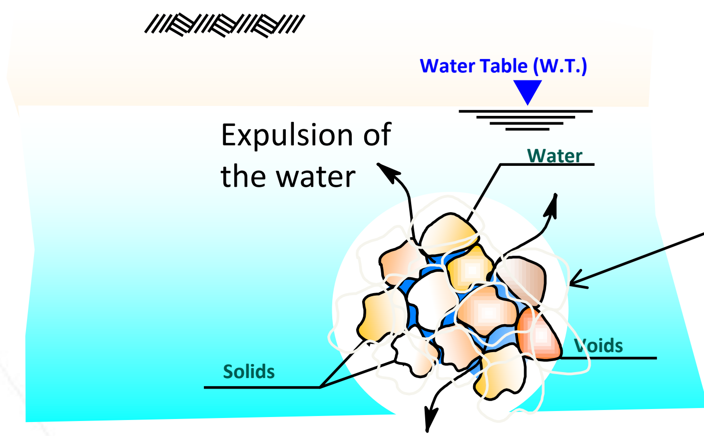
* Consolidation settlement occurs in cohesive soils due to the expulsion of the water from the voids.
* Because of the soil permeability the rate of settlement may varied from soil to another.
* Also the variation in the rate of consolidation settlement depends on the boundary conditions.

$$S_{Consolidation} = S_{primary} + S_{secondary}$$

Primary Consolidation — Volume change is due to reduction in pore water pressure

Secondary Consolidation — Volume change is due to the rearrangement of the soil particles

(No pore water pressure change, $\Delta u = 0$, occurs after the primary consolidation)



When the water in the voids starts to flow out of the soil matrix due to consolidation of the clay layer. Consequently, the excess pore water pressure (Δu) will reduce, and the void ratio (e) of the soil matrix will reduce too.

Elastic Settlement

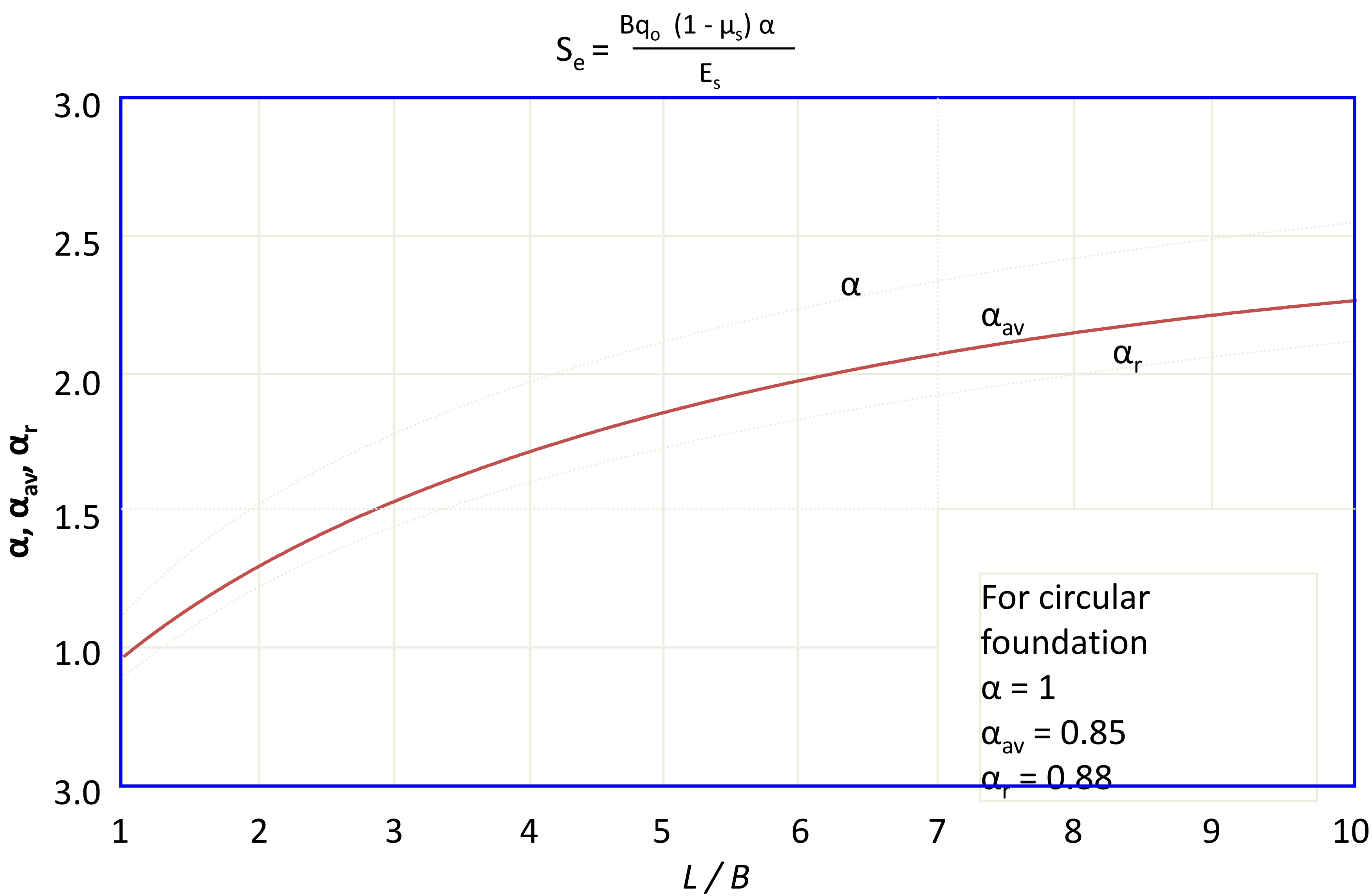
$$S_e = \frac{Bq_o}{E_s}(1 - \mu_s^2)\frac{\alpha}{2}$$ (corner of the flexible foundation)

$$S_e = \frac{Bq_o}{E_s}(1 - \mu_s^2)\alpha$$ (center of the flexible foundation)

Where $\alpha = \frac{1}{\pi}\left[\ln\left(\sqrt{1+m^2} + m \,/\, \sqrt{1+m^2} - m \right) + m.\ln\left(\sqrt{1+m^2} + 1 \,/\, \sqrt{1+m^2} - 1 \right) \right.$

$m = B/L$

B = width of foundation
L = length of foundation



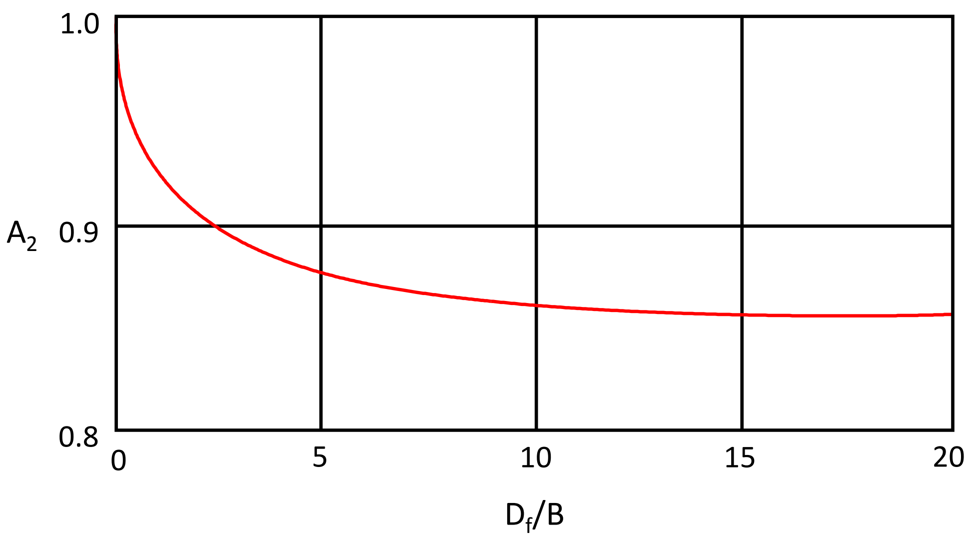
Values of α, α_{av}, and α_r

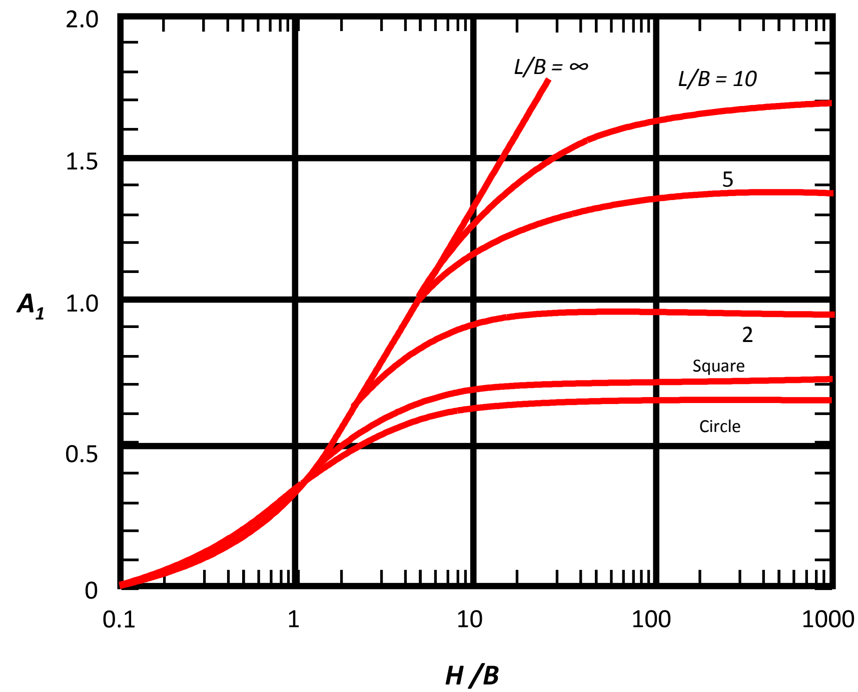
Elastic Settlement of Foundation on Saturated Clay

Janbu, Bjerrum, and Kjaernsli (1956) proposed an equation for evaluation of the average elastic settlement of flexible foundations on saturated clay soils (Poisson's ratio, $\mu_s = 0.5$). Referring to Figure 1 for notations, this equation can be written as

$$S_e = A_1 A_2 q_o B/E_s$$

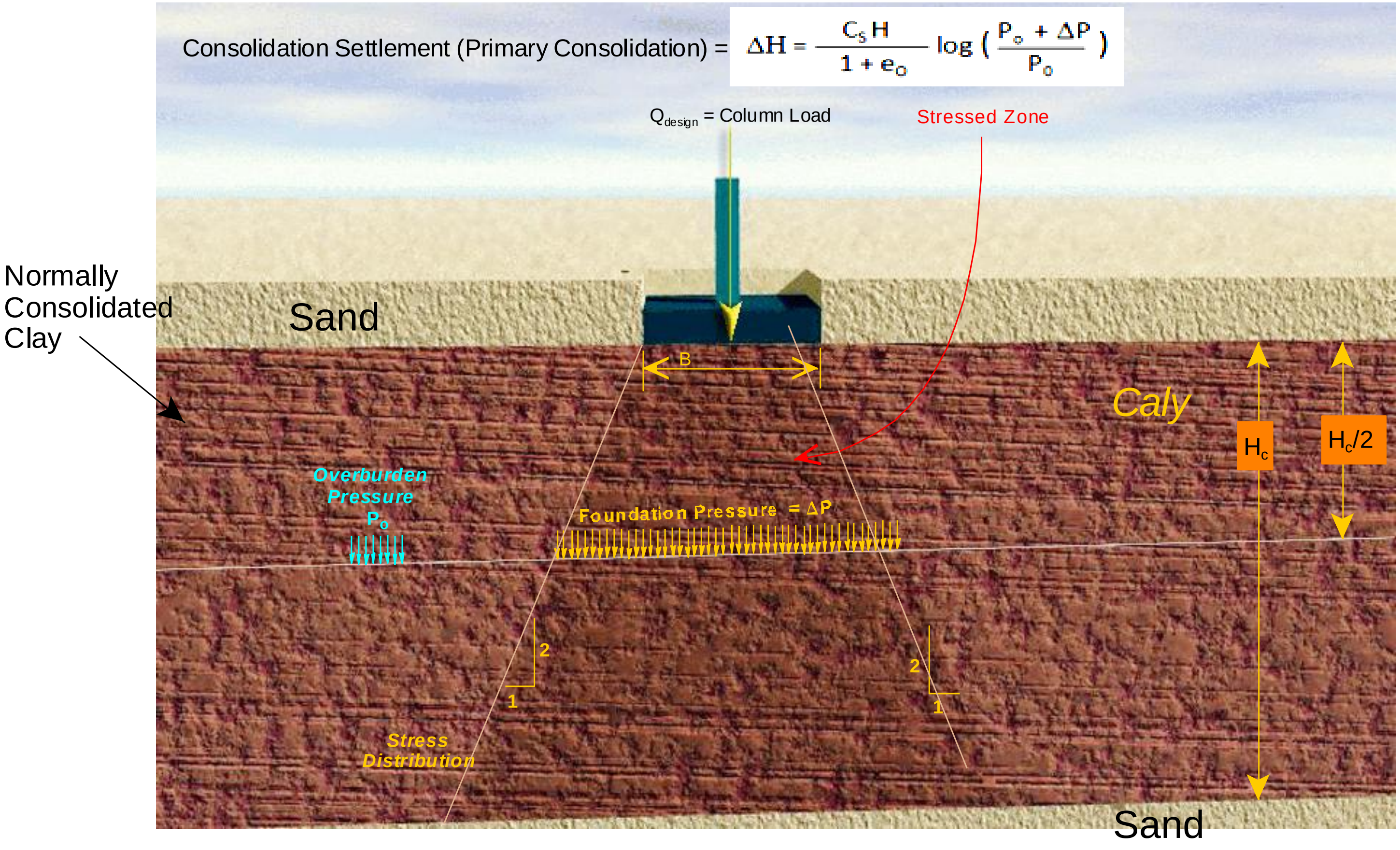
where A_1 is a function H/B and L/B, and is a function of D_f/B.
Christian and Carrier (1978) have modified the values of A_1 and A_2 to some extent, and these are presented in Figure 2.





Values of A1 and A2 for elastic settlement calculation (after Christian and Carrier, 1978)

Consolidation Settlement



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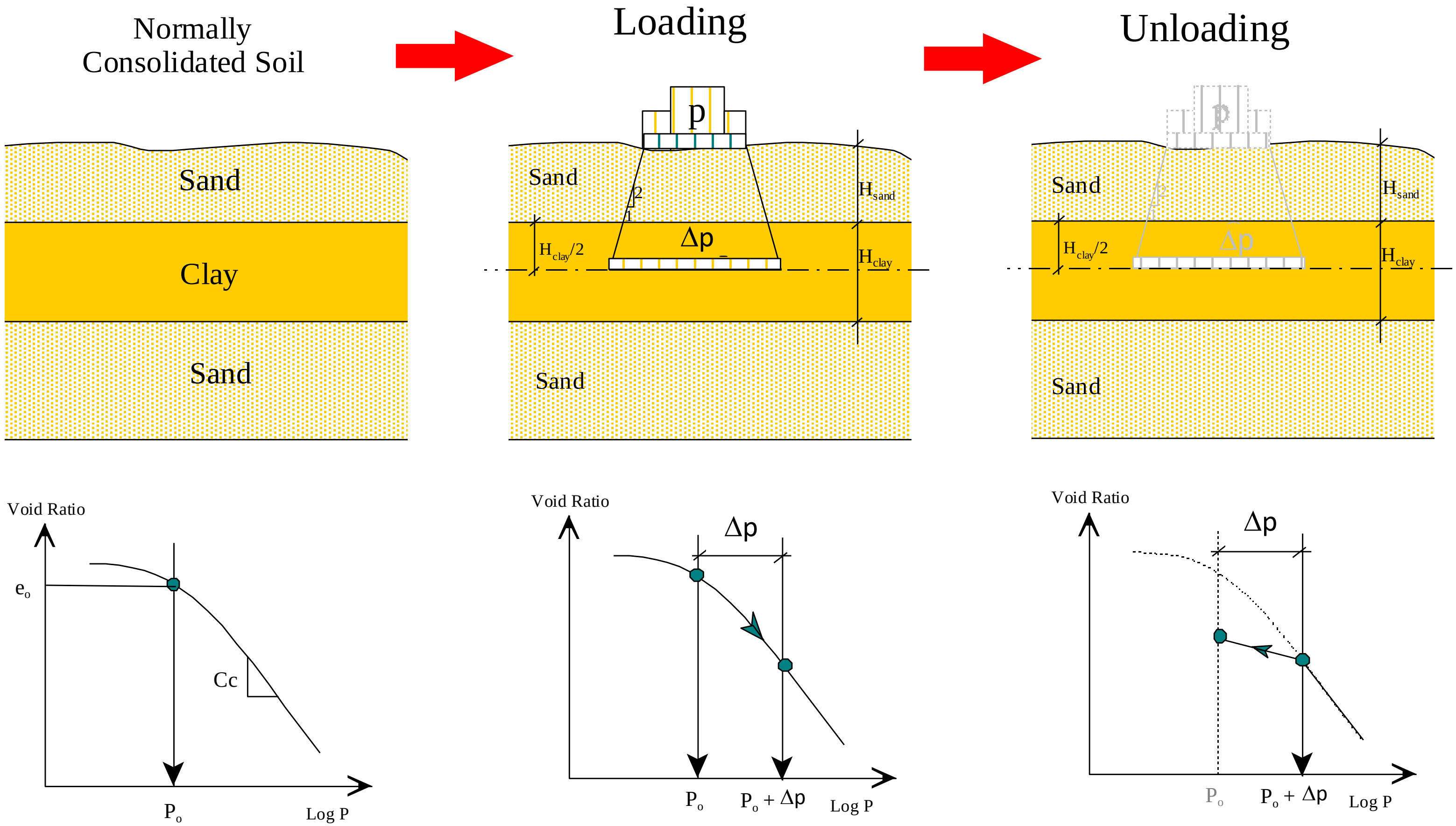
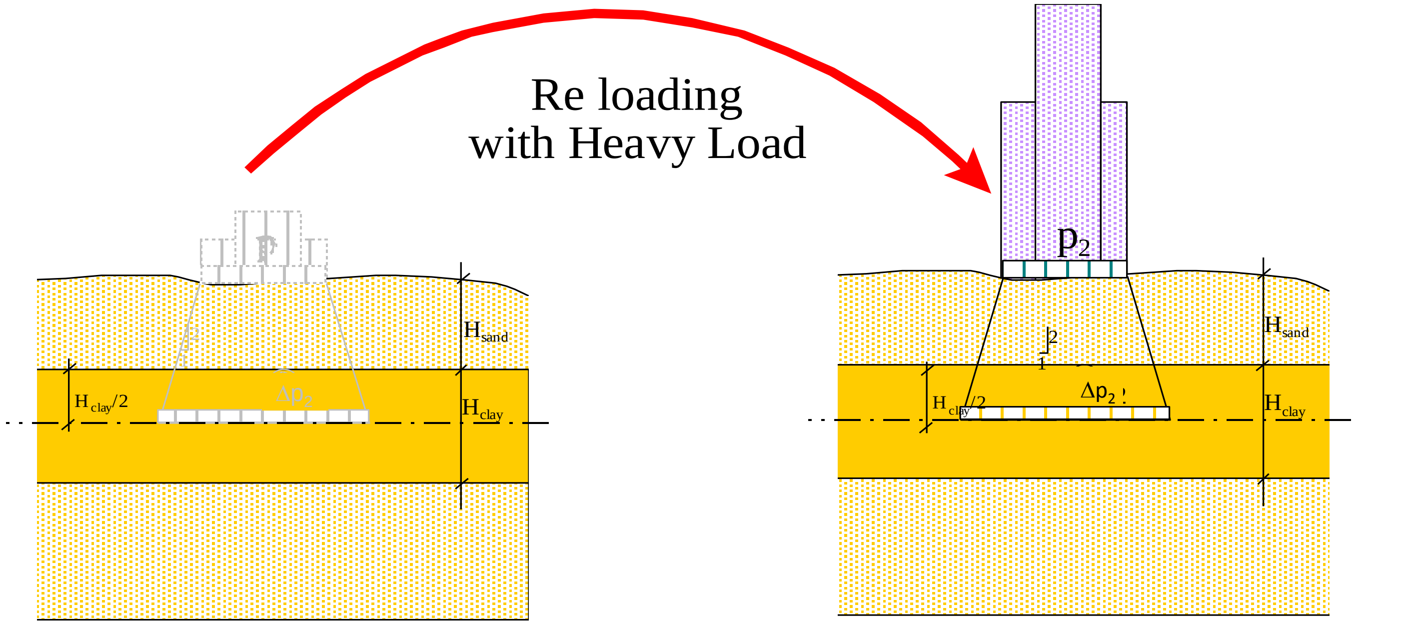
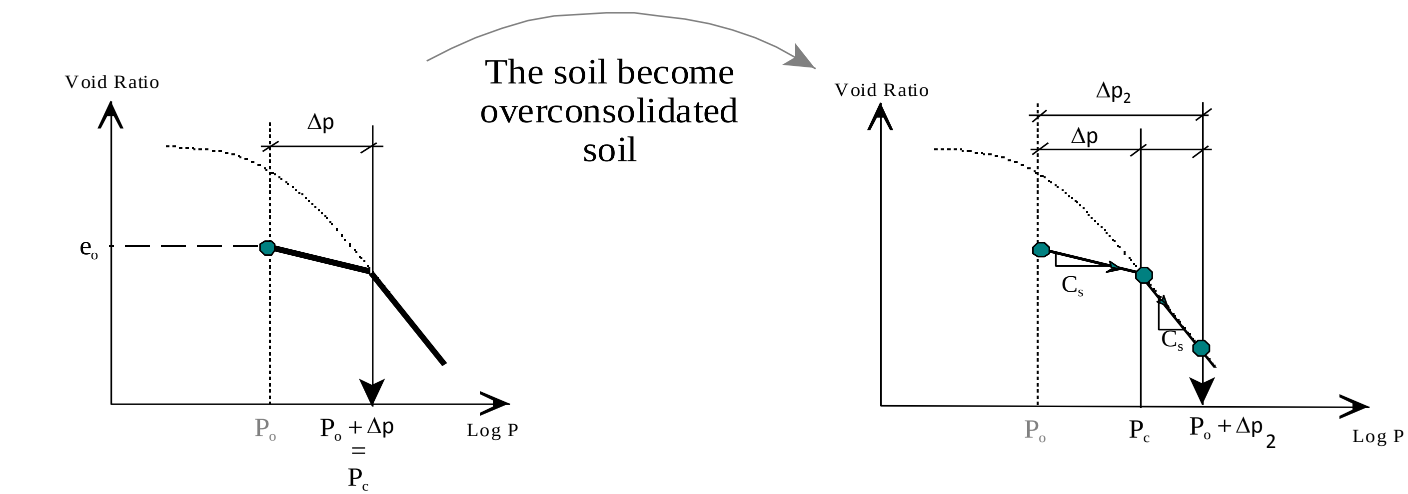
Consolidation Settlement



$$\Delta H = \frac{Cc\,H}{1 + e_o} \log \left(\frac{P_o + \Delta P}{P_0} \right)$$

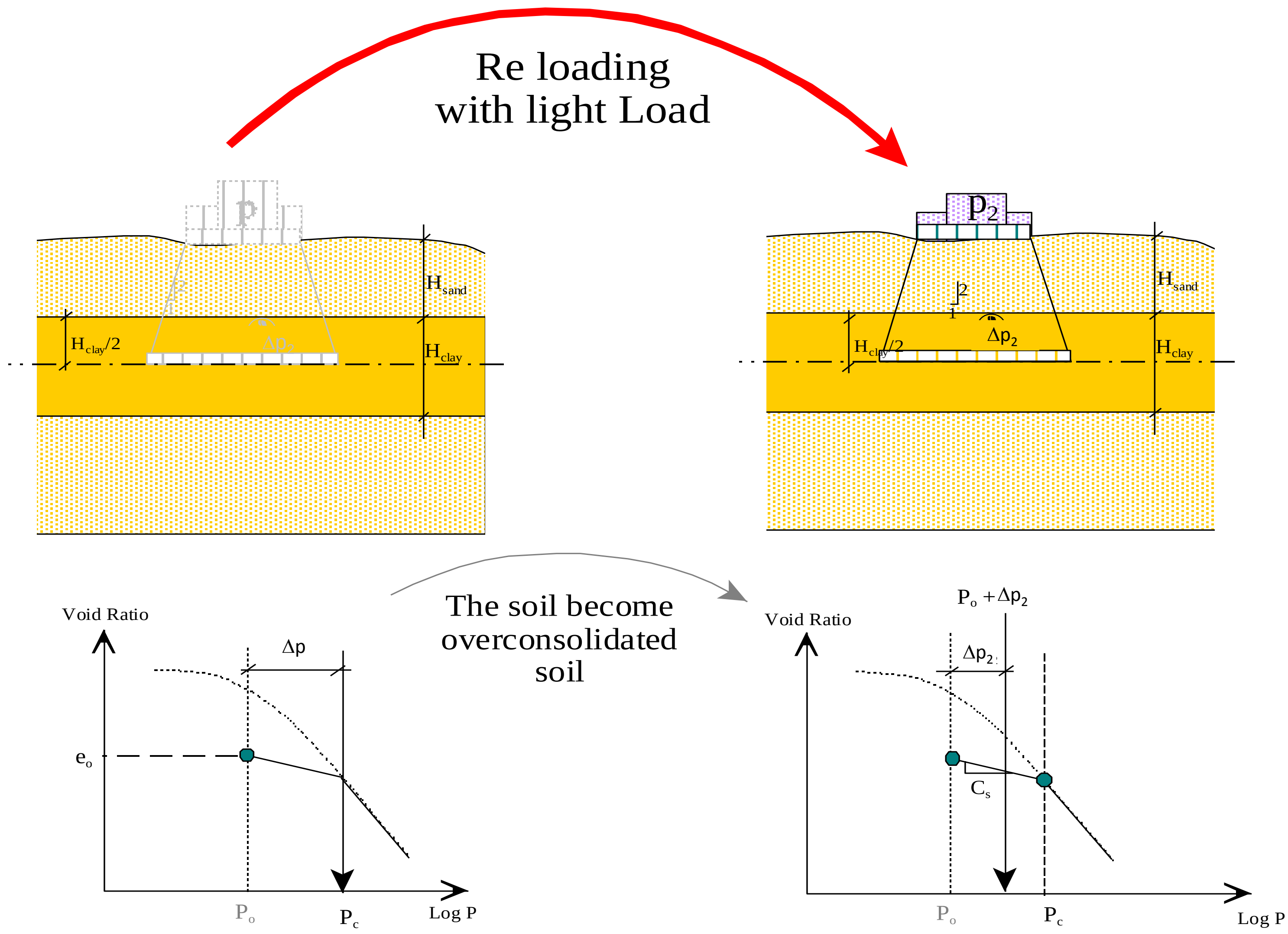
By: Kamal Tawfiq, Ph.D., P.E.





$$\Delta H = \frac{C_S H}{1 + e_o} \log\left(\frac{P_C}{P_o}\right) + \frac{C_C H}{1 + e_o} \log\left(\frac{P_o + \Delta P_2}{P_C}\right)$$

By: Kamal Tawfiq, Ph.D., P.E.



$$\Delta H = \frac{C_S H}{1 + e_O} \log \left(\frac{P_o + \Delta P_2}{P_o}\right)$$

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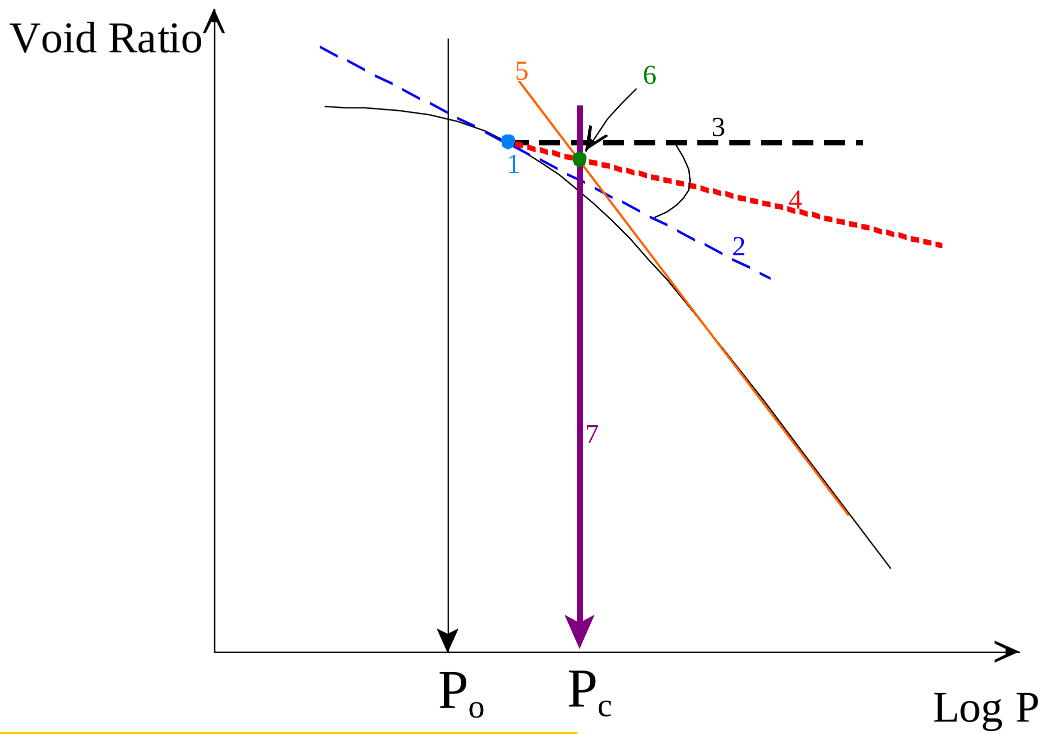
Determining The Preconsolidation Pressure (Pc)

Cassagrande Graphical Method



$OCR = P_c/P_o$

OCR = 1 → Normally Consolidated

OCR > 1 → Over Consolidated

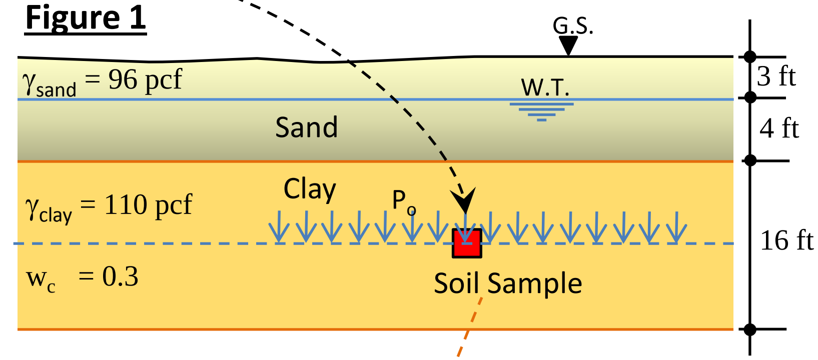
OCR > 4 → Heavily Over Consolidated

By: Kamal Tawfiq, Ph.D., P.E.

Example:

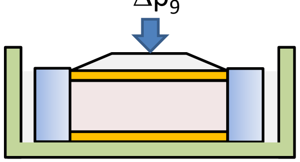
1. Soil sample was obtained from the clay layer
2. Conduct consolidation test [9 load increments]
3. Plot e vs. log (p) (Figure 2)
4. Determine Compression Index (C_c) & Swelling Index (C_s)

Figure 1



In the lab and after removing the soil sample from the ground, the stresses on the soil sample = 0

Stress Increments Δp_1 Δp Δp_7 Δp Δp_9



In the lab the stresses are added to the soil sample

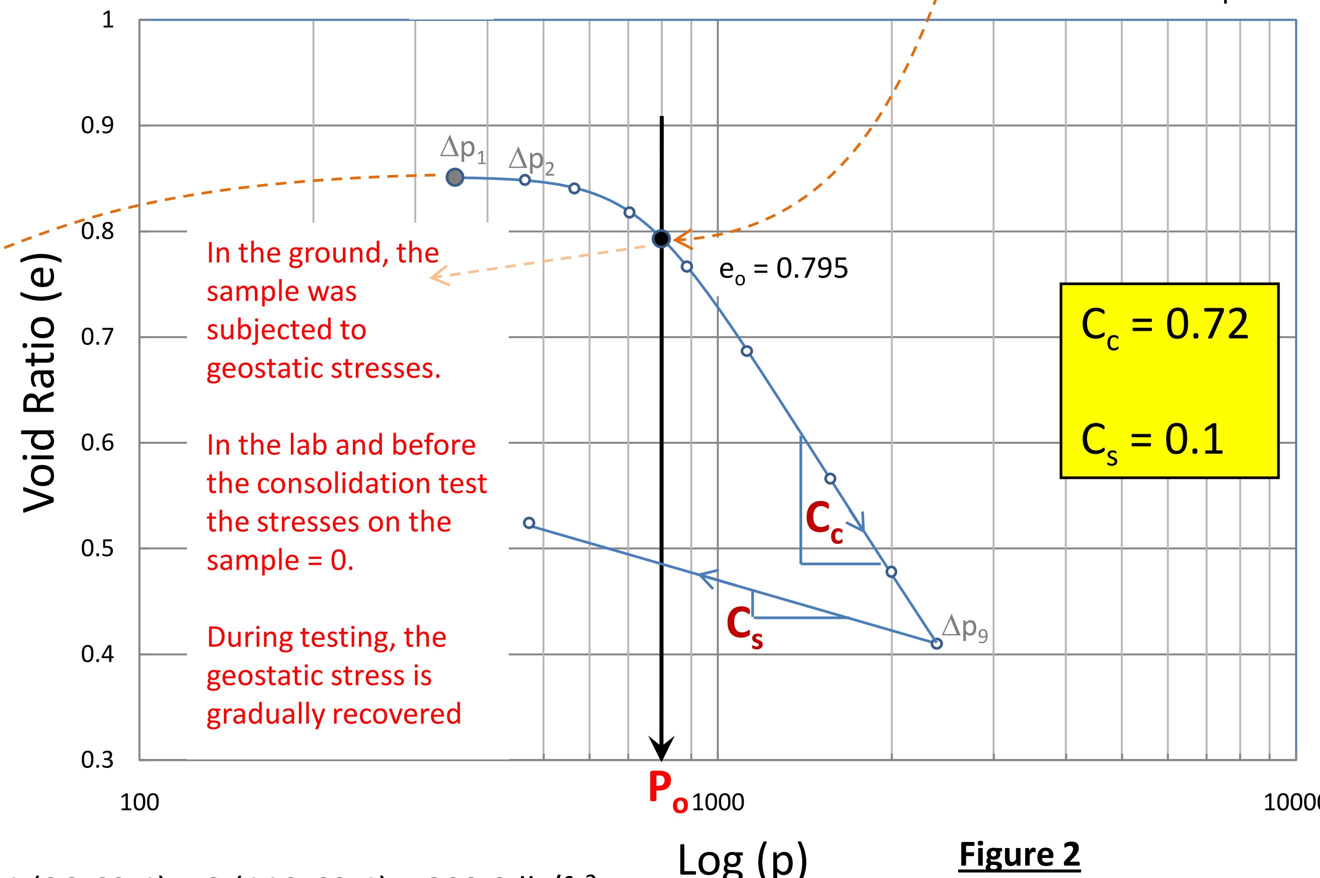


Figure 2

5. Determine $P_o = 3.(96) + 4.(96-62.4) + 8.(110-62.4) = 803.2 \text{ lb/ft}^2$

Tangent to point 1

Example:

6. Using Casagrande's Method to determine Pc

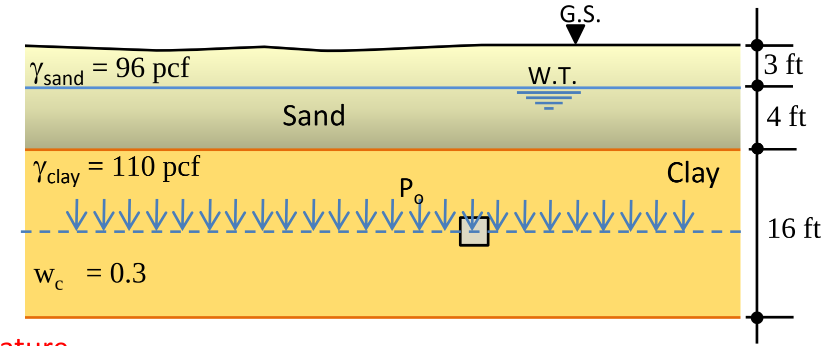
$P_c = 800 \text{ lb/ft}^2$

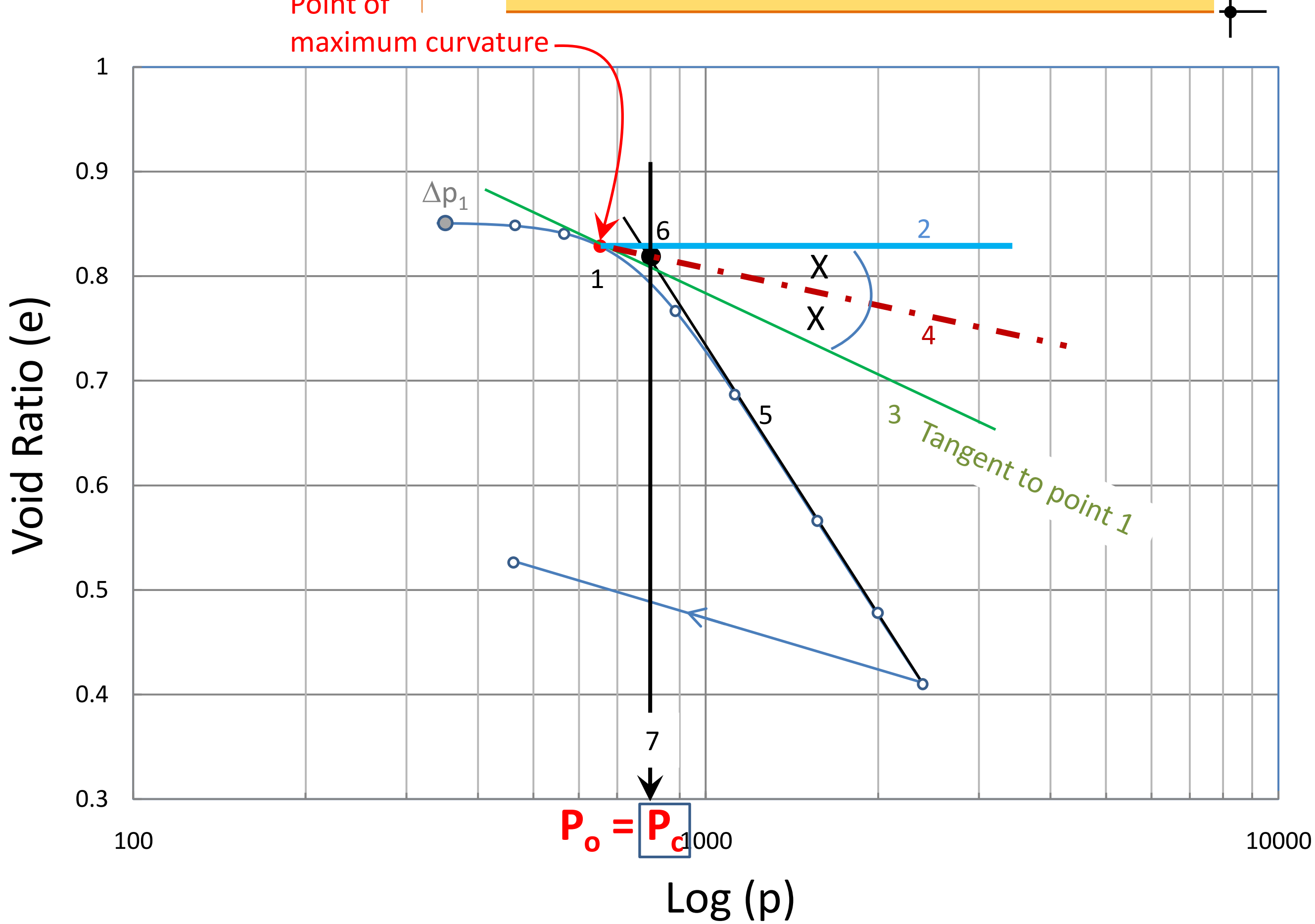
Overconsolidation Ratio

$$OCR = \frac{P_c}{P_o} = 1$$

The soil is

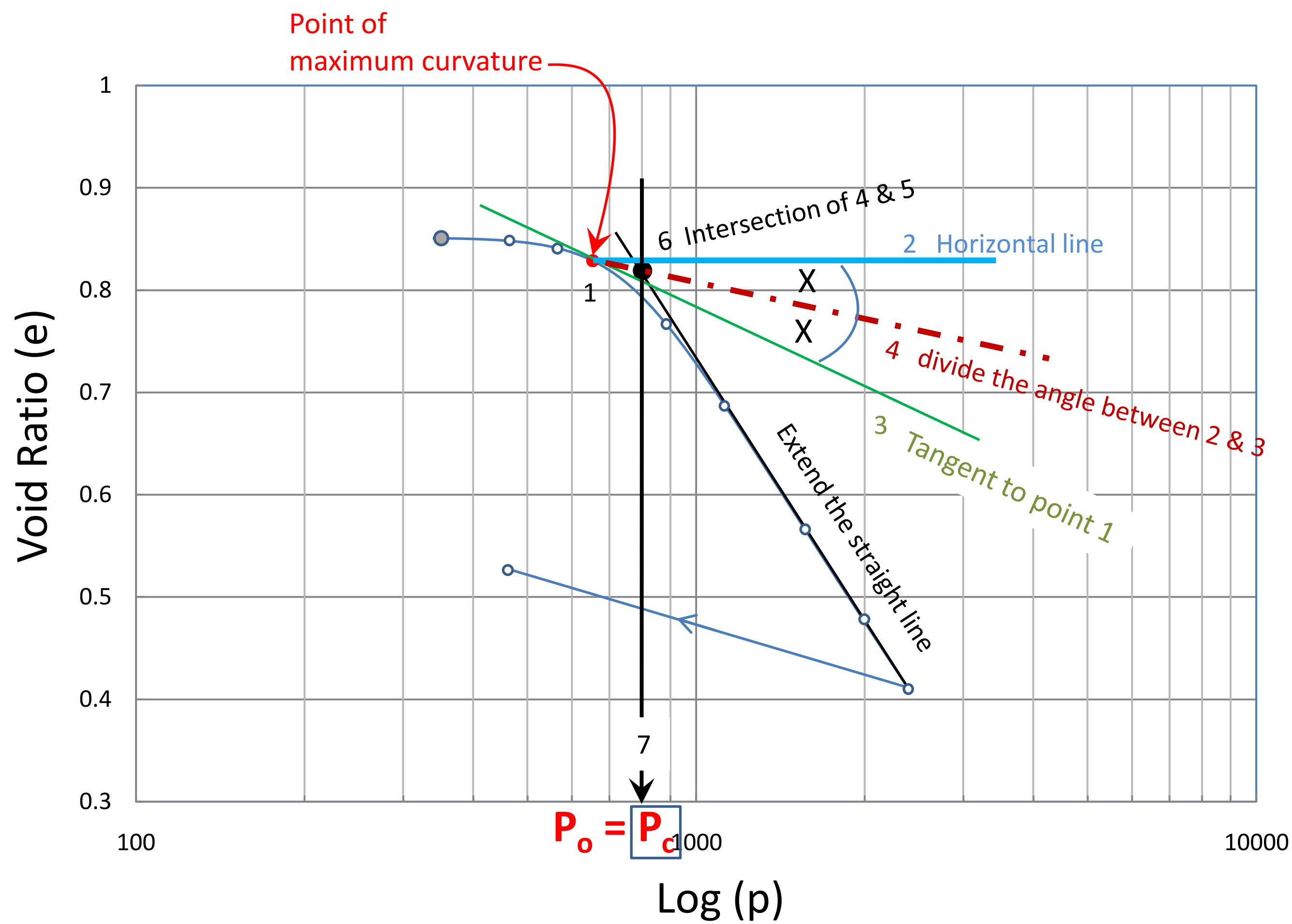
Normally Consolidated
N.C. soil





Casagrande's Method to Determine Preconsolidation Pressure (Pc)

1 Normally Consolidated Soil

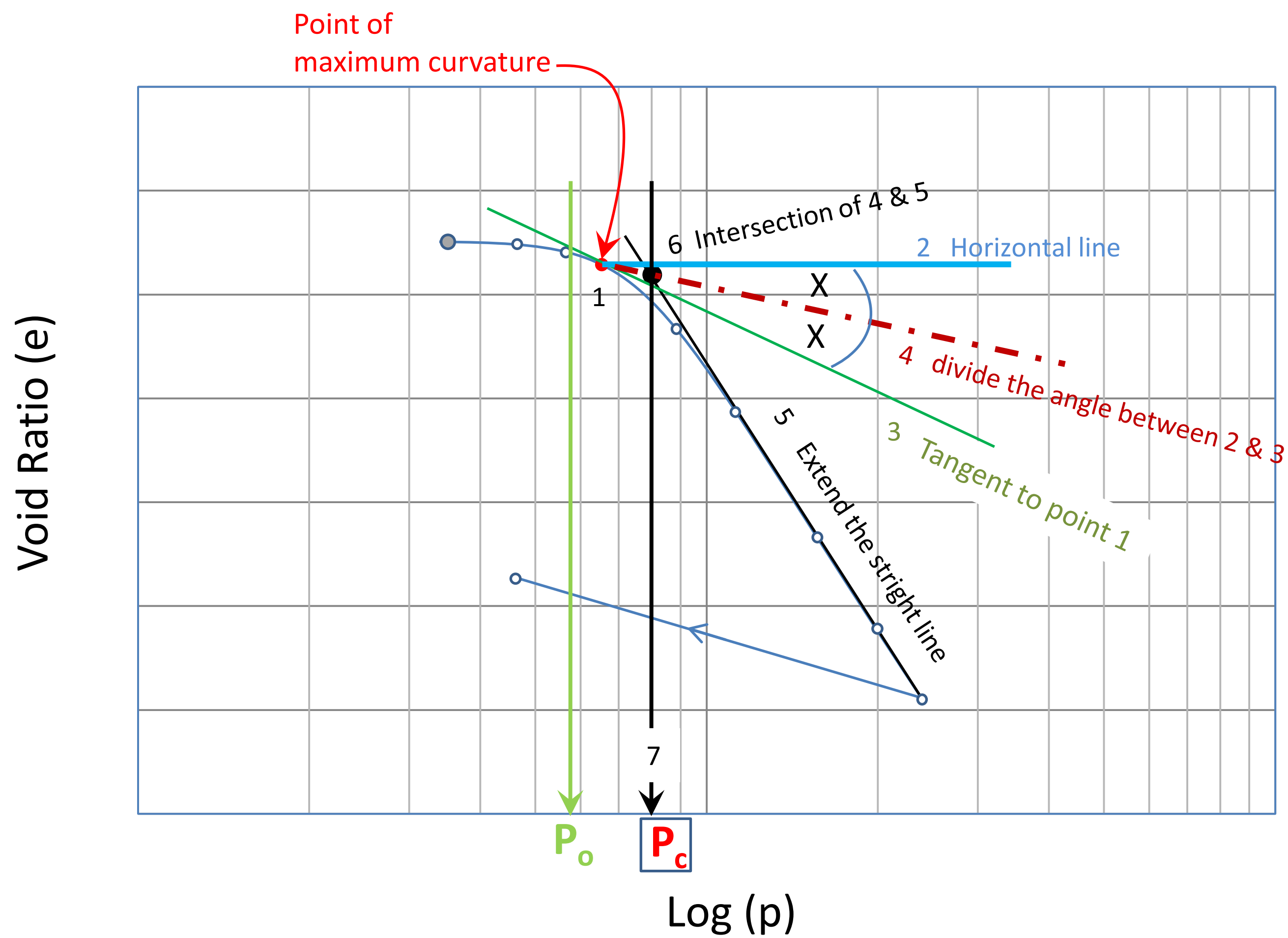


Overconsolidation Ratio OCR = $\frac{P_c}{P_o}$ = 1

The soil is **Normally Consolidated** (N.C.) soil

Casagrande's Method to Determine Pc

2 Overconsolidated Soil



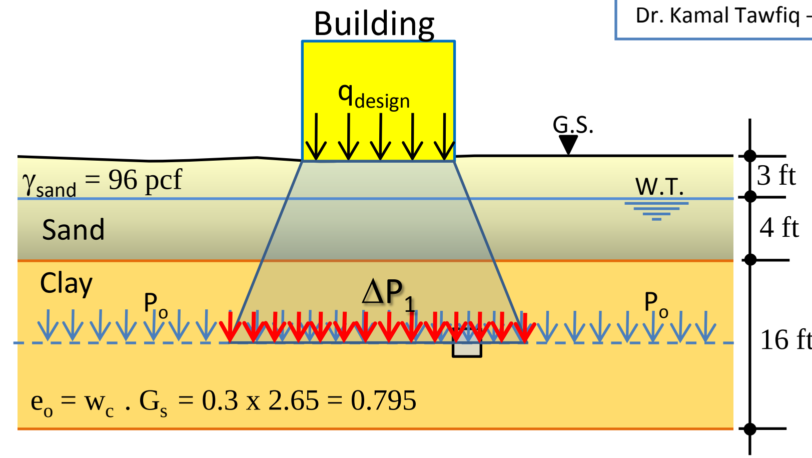
Overconsolidation Ratio OCR $= \dfrac{P_c}{P_o} > 1$

The soil is **<u>oversonsolidated</u>** (O.C.) soil

Example:

A 150’ x 100’ building will be constructed at the site. The vertical stress due to the addition of the building q_{design} =1000 lb/ft^2

The weight of the building Q_{design} will be transferred to the mid height of the clay layer



Q_{design} = 15,000,000 lb

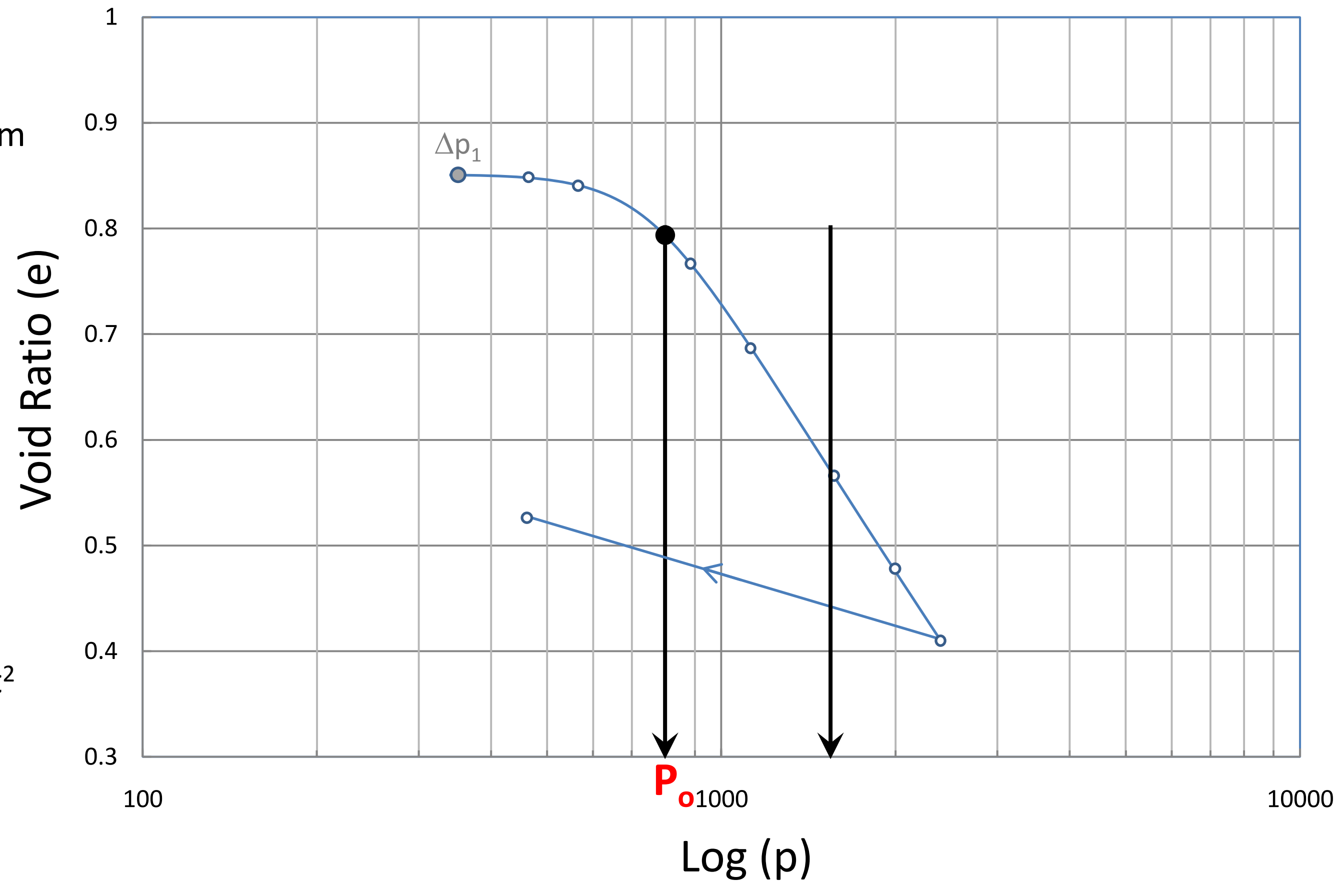
The added stress at 15’ from the ground surface is

$$\Delta p = \frac{15{,}000{,}000 \text{ lb}}{(150+15) \times (100+15)}$$

ΔP = 790.51 lb/ft$_2$

$\Delta P + P_o$ =

790.51 + 803 = 1593.51 lb/ft^2



Example:

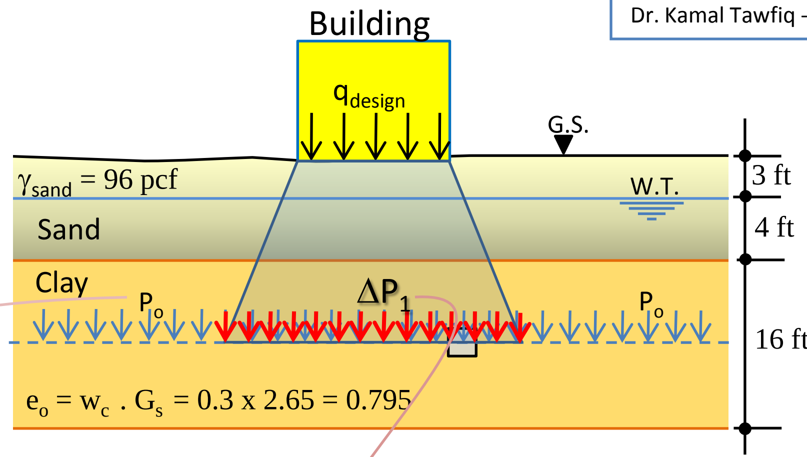
$\Delta P + P_o = 790.51 + 803 = 1593.51 \text{ lb/ft}^2$

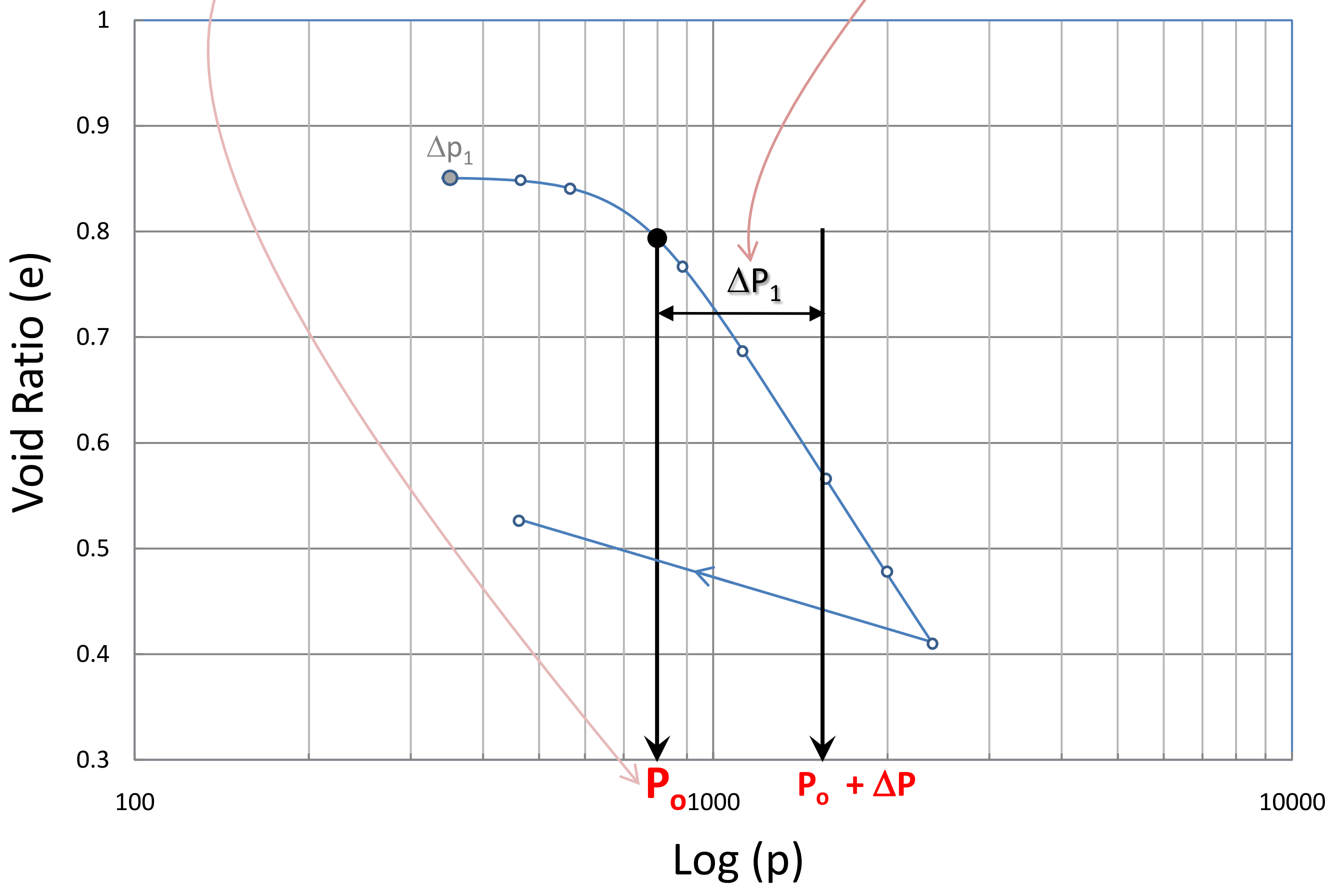
Consolidation Settlement

$$\Delta H = \frac{C_c H}{1 + e_O} \log \left(\frac{P_o + \Delta P}{P_o} \right)$$

$$\Delta H = \frac{0.72 \times 16}{1 + 0.795} \log \left(\frac{1593.51}{803} \right)$$

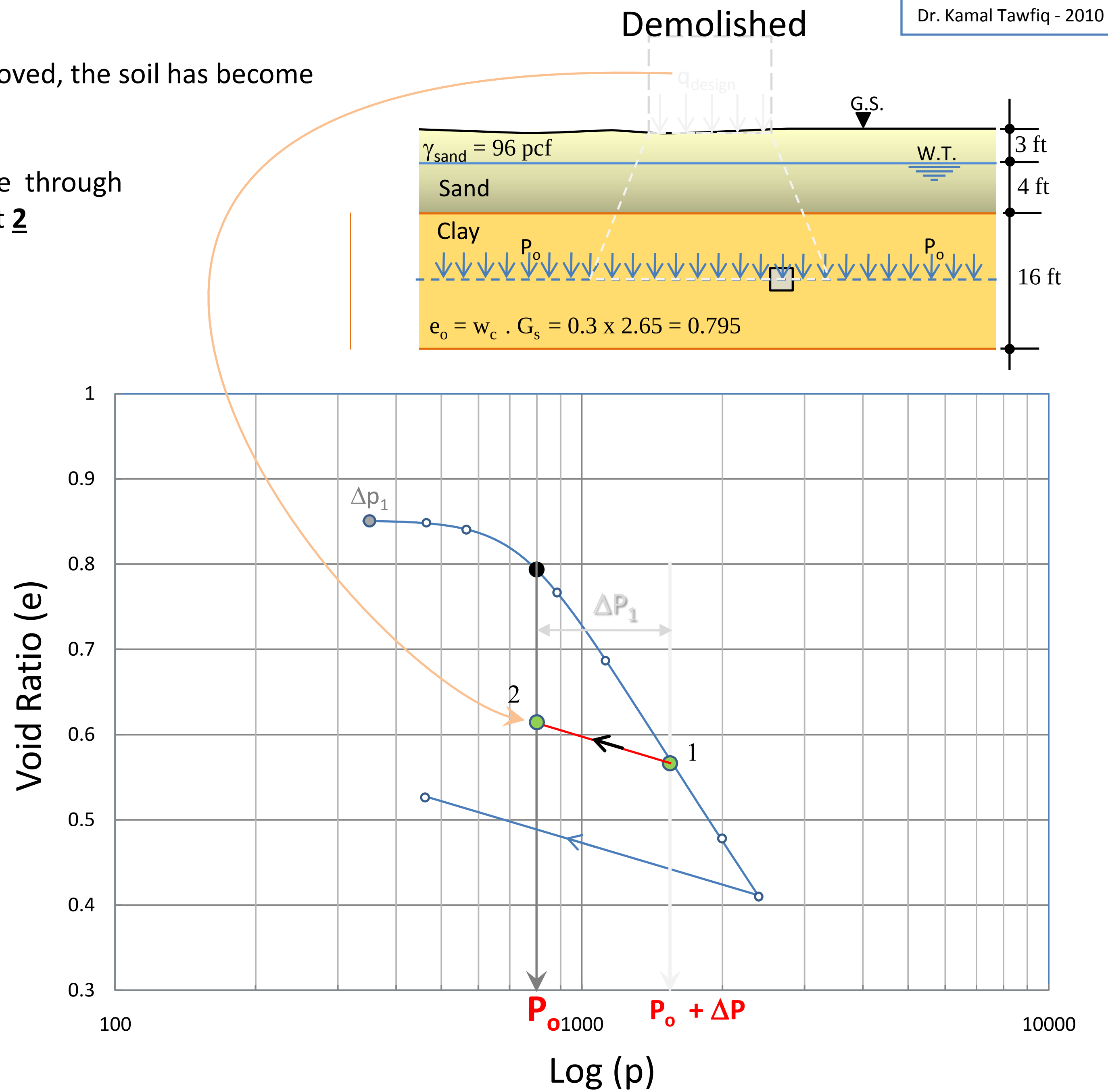
$$\Delta H = 1.9 \text{ ft}$$





When the building was removed, the soil has become an overconsolidated clay.

The rebound has taken place through swelling from pint **1** to point **2**



Dr. Kamal Tawfiq - 2010

Scenario #1

The soil now is overconsolidated Soil:

The new building is heavier in weight

$$\Delta H = \frac{C_s H}{1+e_o} \log\left(\frac{P_c}{P_o}\right) + \frac{C_c H}{1+e_o} \log\left(\frac{P_o + \Delta P}{P_c}\right)$$

$e_o = 0.61$

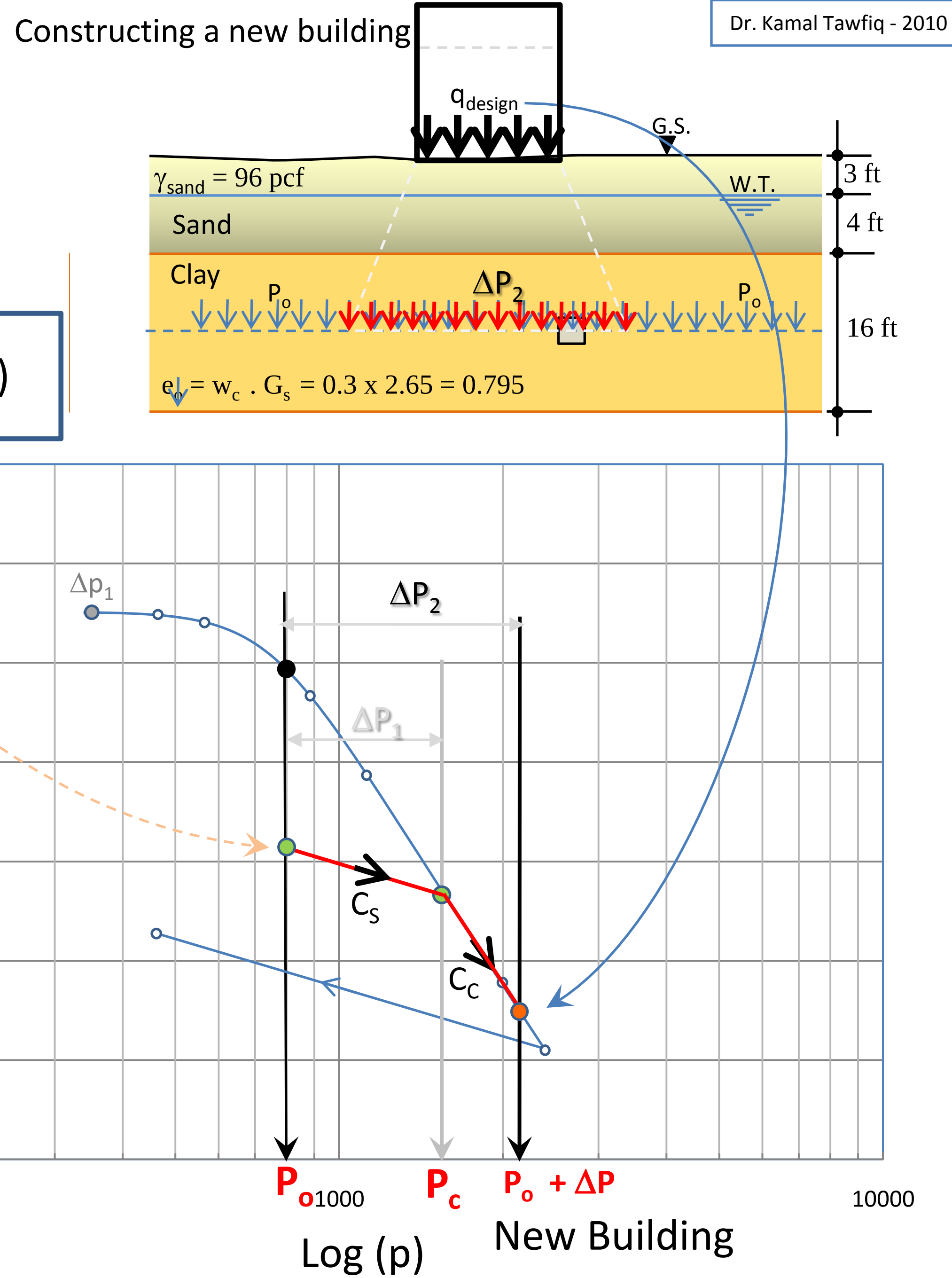
Assume $P_o + \Delta p_2 = 2100$ psf

$$\Delta H = \frac{0.1 \times 16}{1+0.61} \log\left(\frac{1593.51}{803}\right)$$

$$+$$

$$\frac{0.72 \times 16}{1+0.61} \log\left(\frac{2100}{1593.51}\right)$$

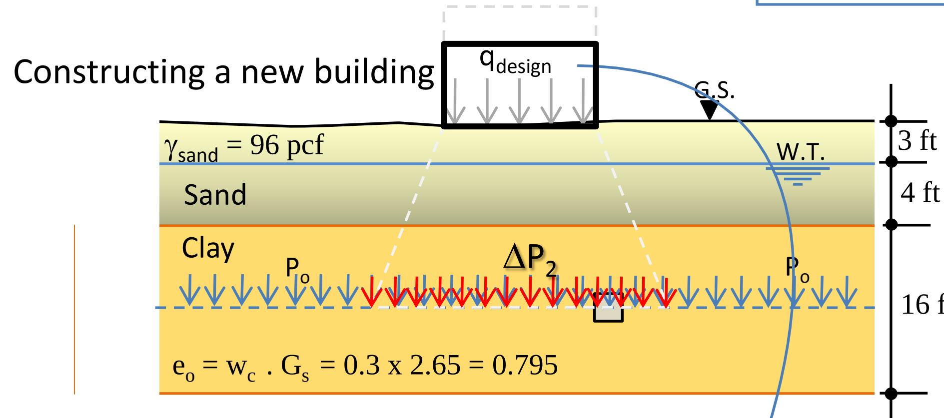
$$=$$



Scenario # 2

The soil now is overconsolidated Soil:

The new building is lighter in weight



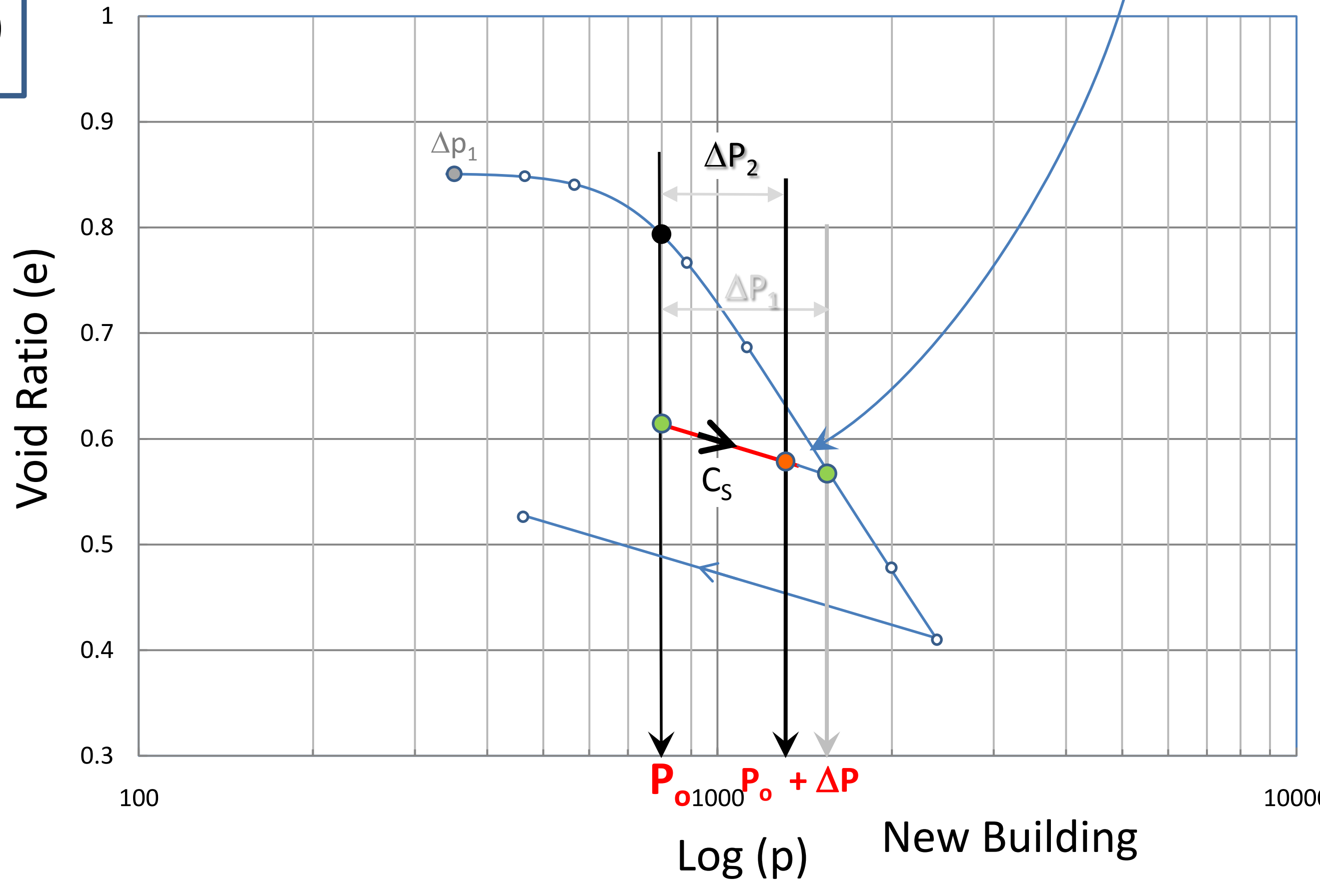
$$\Delta H = \frac{C_S H}{1 + e_O} \log\left(\frac{P_o + \Delta P}{P_0}\right)$$

$e_o = 0.61$

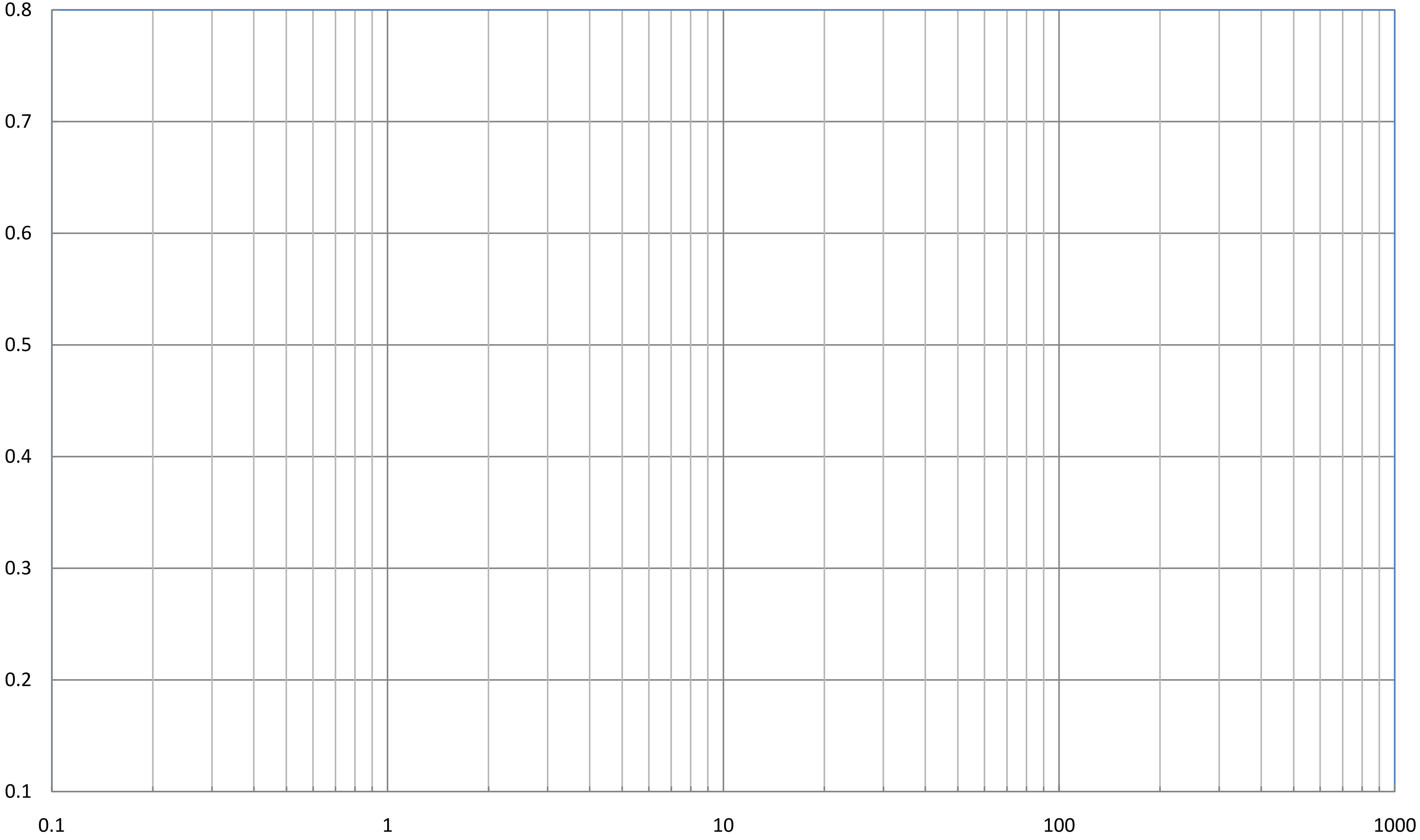
Assume $P_o + \Delta p_2 = 1600$ psf

$$\Delta H = \frac{0.1 \times 16}{1 + 0.61} \log\left(\frac{1600}{1593.51}\right)$$

=



Example of Semi-log Scale



Rate of Consolidation

Settlement at any time = S_{time}

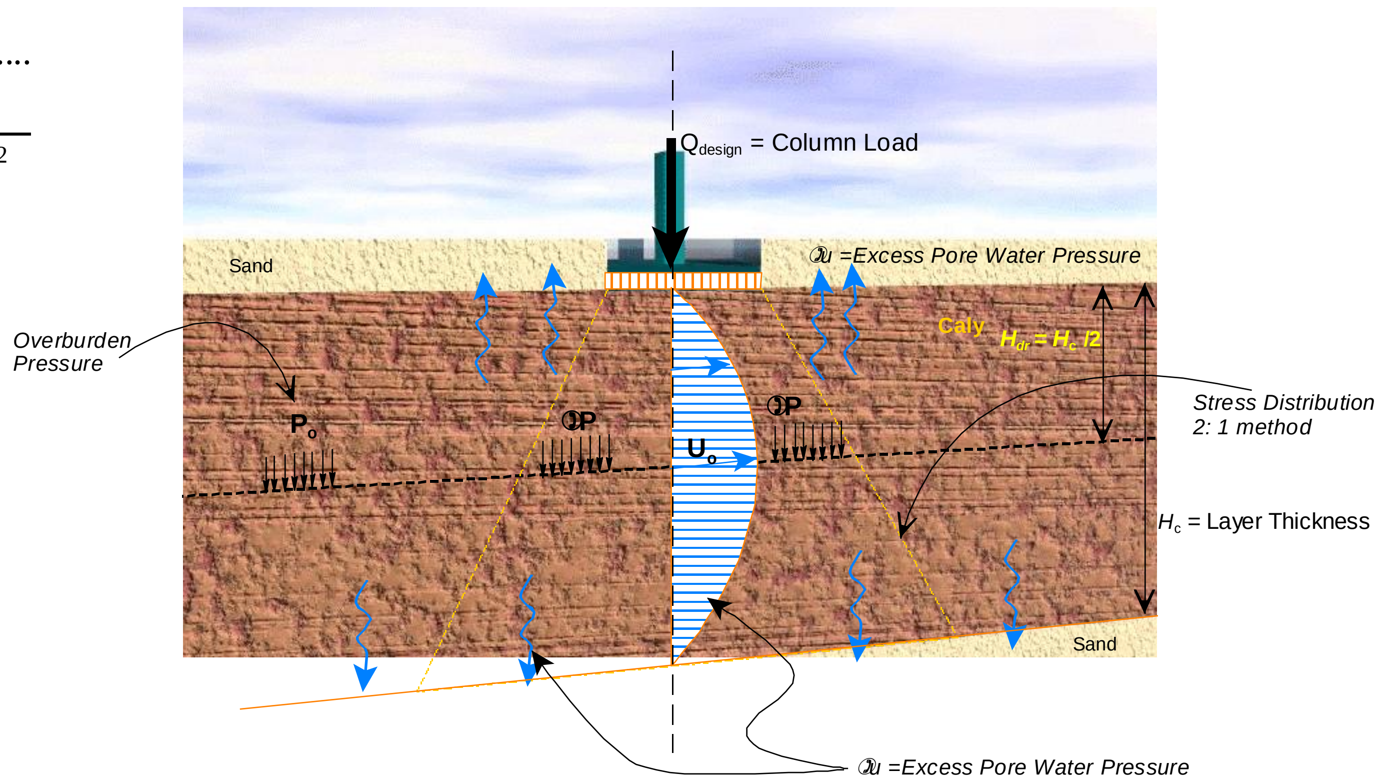
$$S_{time} = S_{ultimate} * U\%$$

$$S_{ultimate} = (C_c/1+e_o)\ H_c\ .\ \log [(P_o + \Delta P)/P_o]$$

$U\% = f\ (T_v)$ ....

$T_v = f\ (c_v)$ ......

$$T_v = \frac{c_v\ .\ t}{(H_{dr})^2}$$



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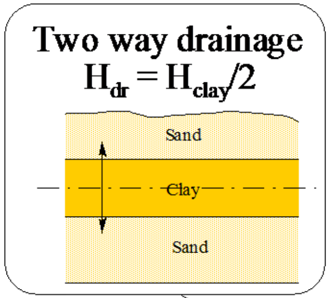
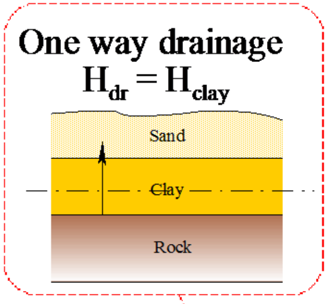
Rate of Consolidation

Settlement at any time = S_{time}

$$S_{time} = S_{ultimate} * U\%$$





$$\underbrace{U\%}_{\text{From Tables}} = f(T_v) \ldots\ldots \qquad T_v = \text{Time Factor}$$

$$T_v = f(c_v) \ldots\ldots \qquad C_v = \text{Coefficient of Consolidation}$$

$$\boxed{T_v = \frac{c_v \, . \, t}{(H_{dr})^2}}$$

t = time (month, day, or year)

$(H_{dr})^2$= Drainage Path

H_{dr} = H or H/2

C_v is obtained from laboratory testing

Once you determine T_v use the table shown on the page to determine U%

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Degree of Consolidation (U%) vs. Time Factor (T_v)

U (%)	T_v	U (%)	T_v	U (%)	T_v	U (%)	T_v
0	0	26	0.0531	52	0.212	78	0.529
1	0.00008	27	0.0572	53	0.221	79	0.547
2	0.0003	28	0.0615	54	0.230	80	0.567
3	0.00071	29	0.0660	55	0.239	81	0.588
4	0.00126	30	0.0707	56	0.248	82	0.610
5	0.00196	31	0.0754	57	0.257	83	0.633
6	0.00283	32	0.0803	58	0.267	84	0.658
7	0.00385	33	0.0855	59	0.276	85	0.684
8	0.00502	34	0.0907	60	0.286	86	0.712
9	0.00636	35	0.0962	61	0.297	87	0.742
10	0.00785	36	0.102	62	0.307	88	0.774
11	0.0095	37	0.107	63	0.318	89	0.809
12	0.0113	38	0.113	64	0.329	90	0.848
13	0.0133	39	0.119	65	0.304	91	0.891
14	0.0154	40	0.126	66	0.352	92	0.938
15	0.0177	41	0.132	67	0.364	93	0.993
16	0.0201	42	0.138	68	0.377	94	1.055
17	0.0227	43	0.145	69	0.390	95	1.129
18	0.0254	44	0.152	70	0.403	96	1.219
19	0.0283	45	0.159	71	0.417	97	1.336
20	0.0314	46	0.166	72	0.431	98	1.500
21	0.0346	47	0.173	73	0.446	99	1.781
22	0.0380	48	0.181	74	0.461	100	∞
23	0.0415	49	0.188	75	0.477		
24	0.0452	50	0.197	76	0.493		
25	0.0491	51	0.204	77	0.511		

Shear Strength of Soil

Strength of different materials

Steel

Tensile strength

Concrete

Compressive strength

Soil

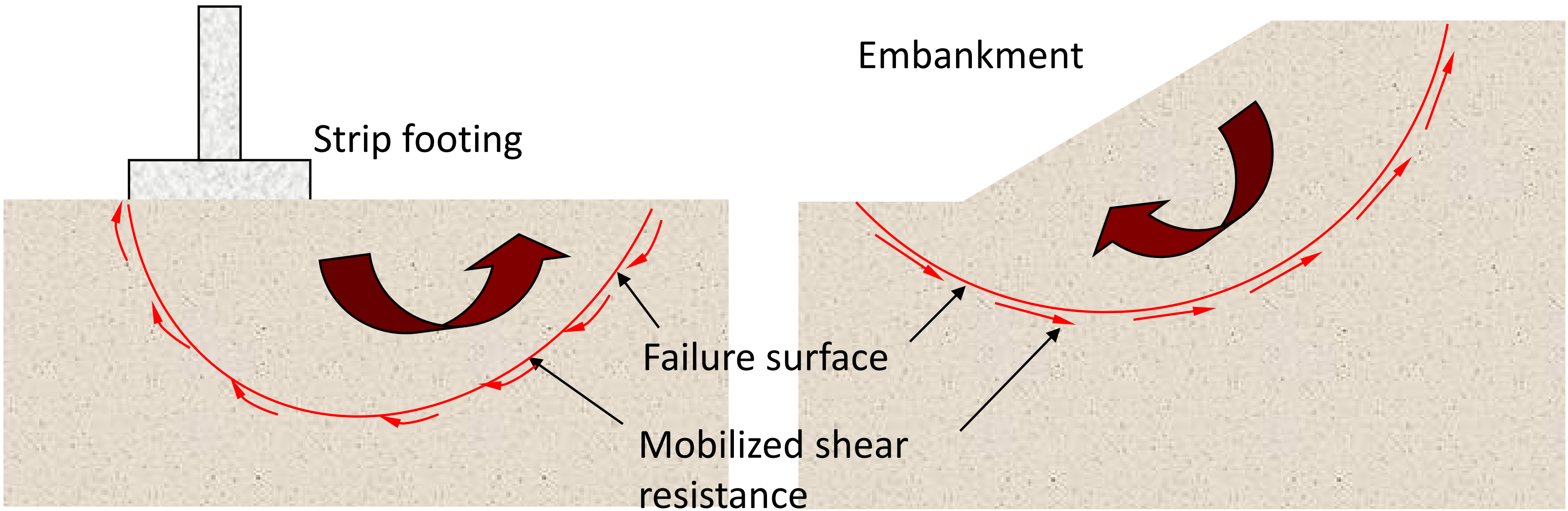
Shear strength

Presence of pore water

Complex behavior

Shear failure of soils

Soils generally fail in <u>shear</u>



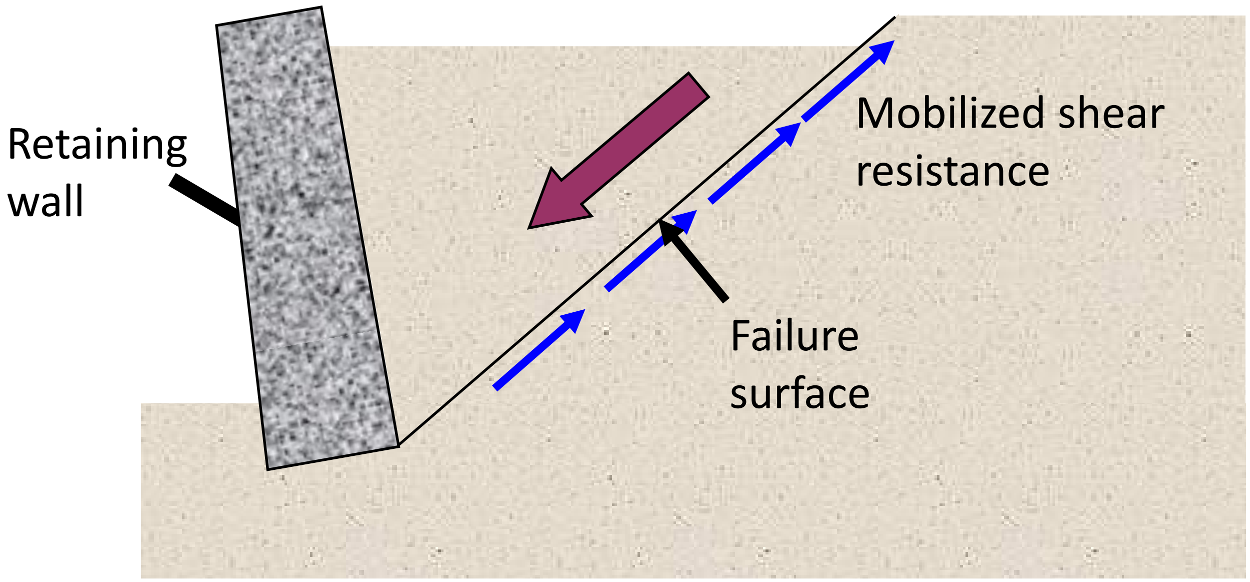
At failure, shear stress along the failure surface (mobilized shear resistance) reaches the shear strength.

Shear failure of soils

Soils generally fail in shear

Shear failure of soils

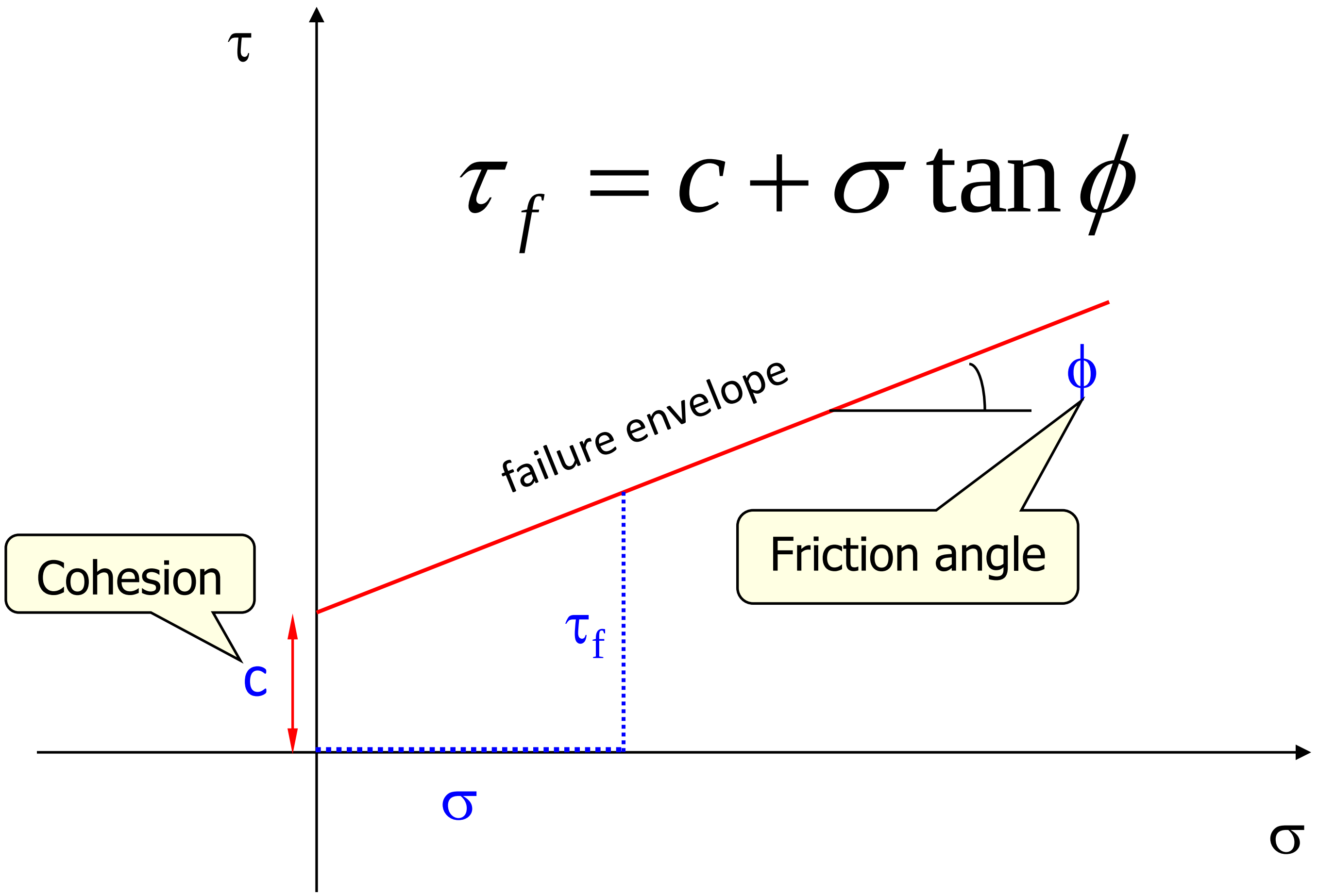
Soils generally fail in <u>shear</u>



At failure, shear stress along the failure surface (mobilized shear resistance) reaches the shear strength.

Mohr-Coulomb Failure Criterion

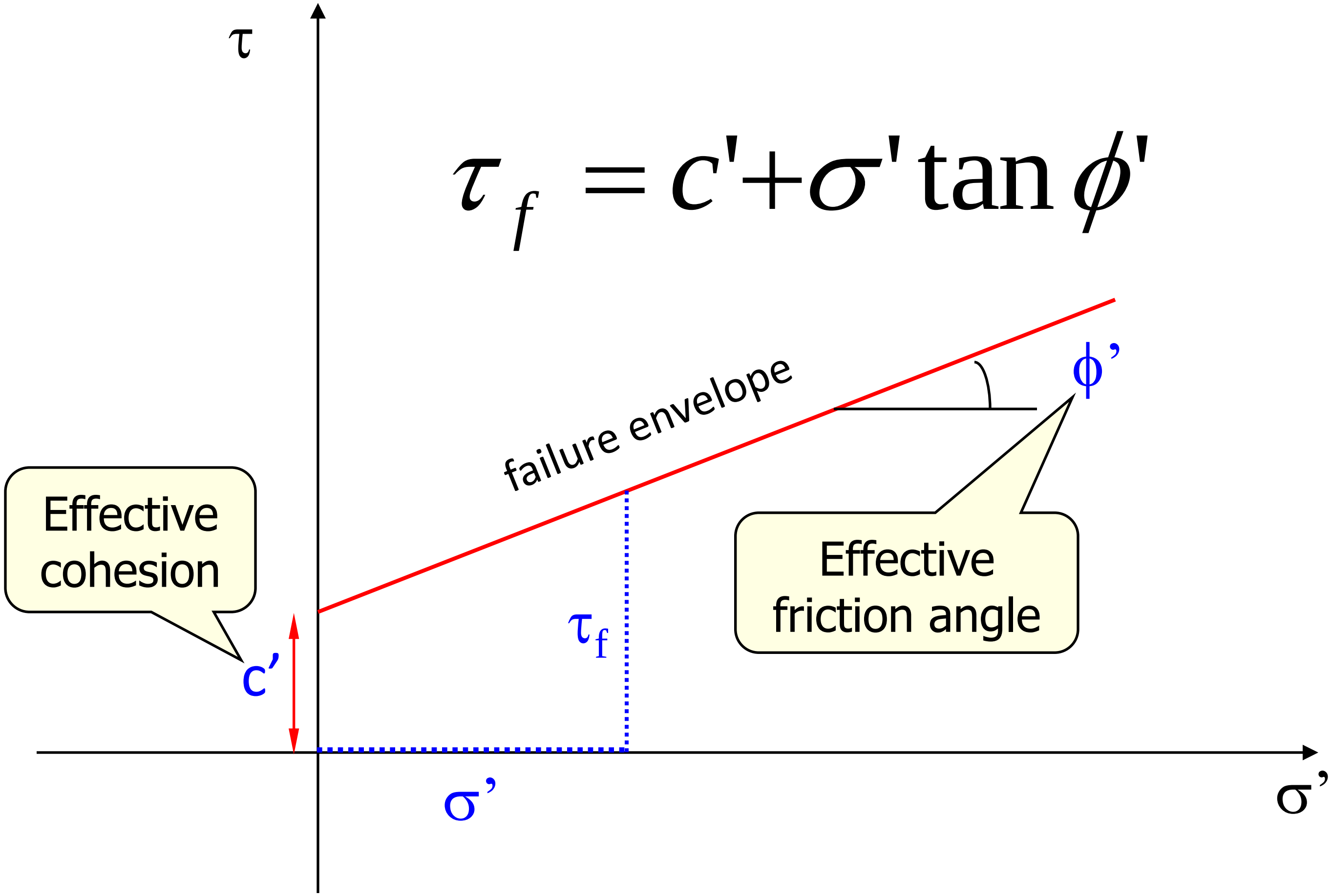
(in terms of total stresses)



τ_f is the maximum shear stress the soil can take without failure, under normal stress of σ.

Mohr-Coulomb Failure Criterion

(in terms of effective stresses)



$$\tau_f = c' + \sigma' \tan \phi'$$

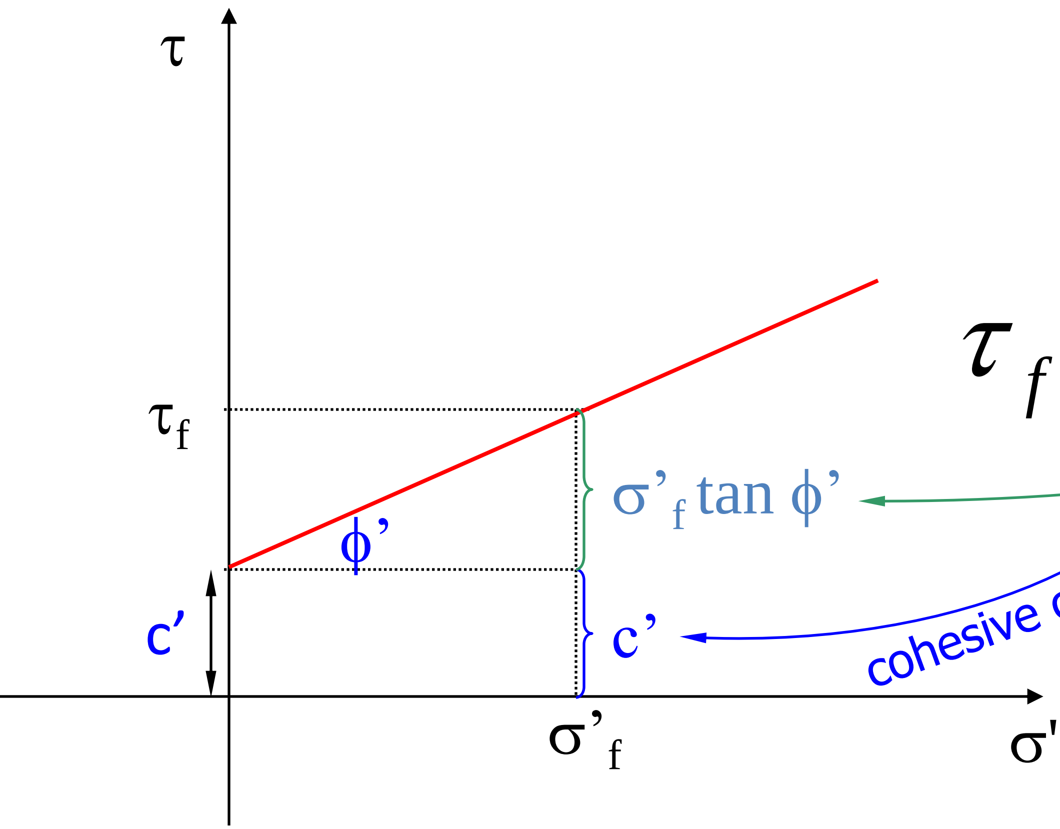
$$\sigma' = \sigma - u$$

u = pore water pressure

τ_f is the maximum shear stress the soil can take without failure, under normal effective stress of σ'.

Mohr-Coulomb Failure Criterion

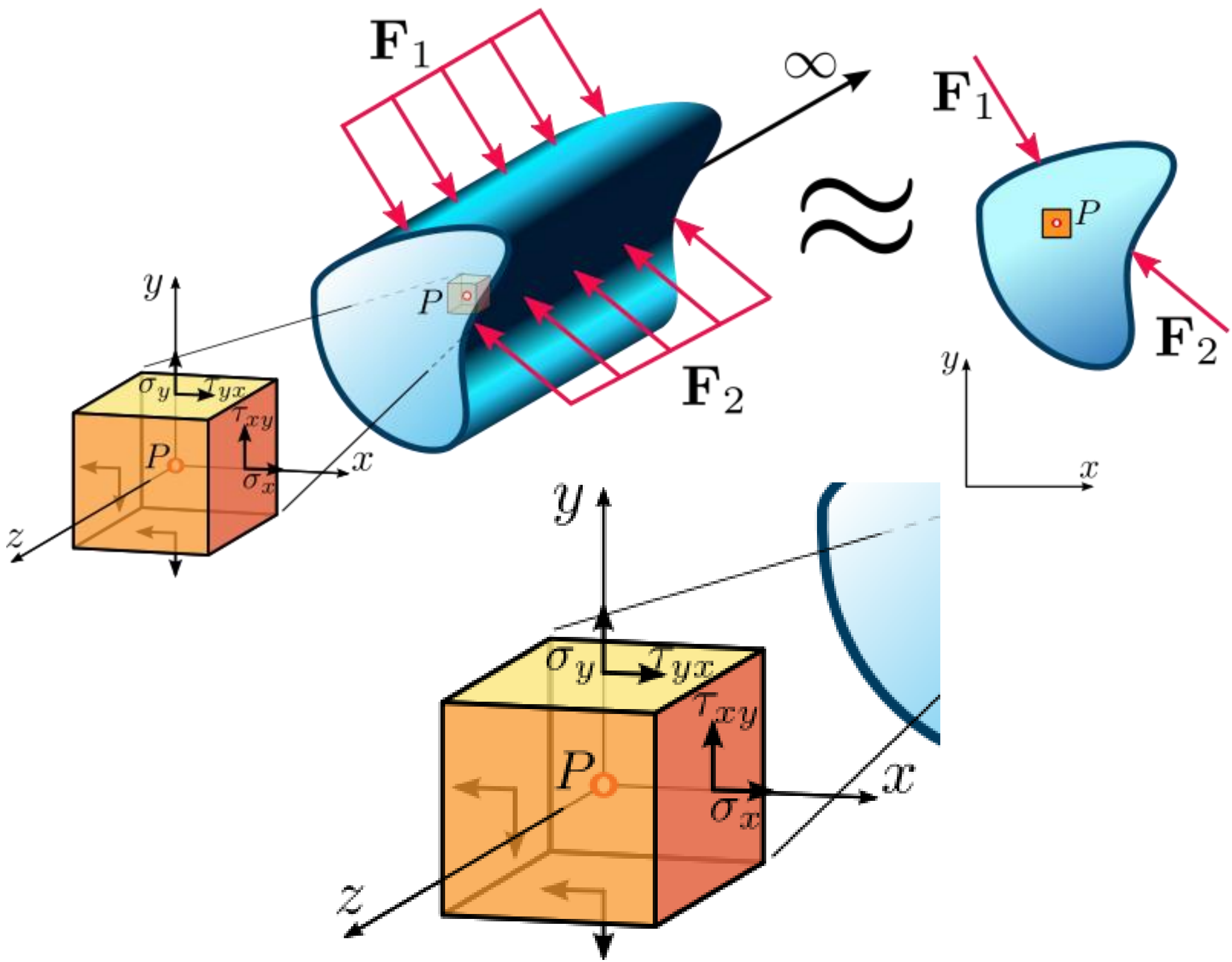
Shear strength consists of two components: cohesive and frictional.



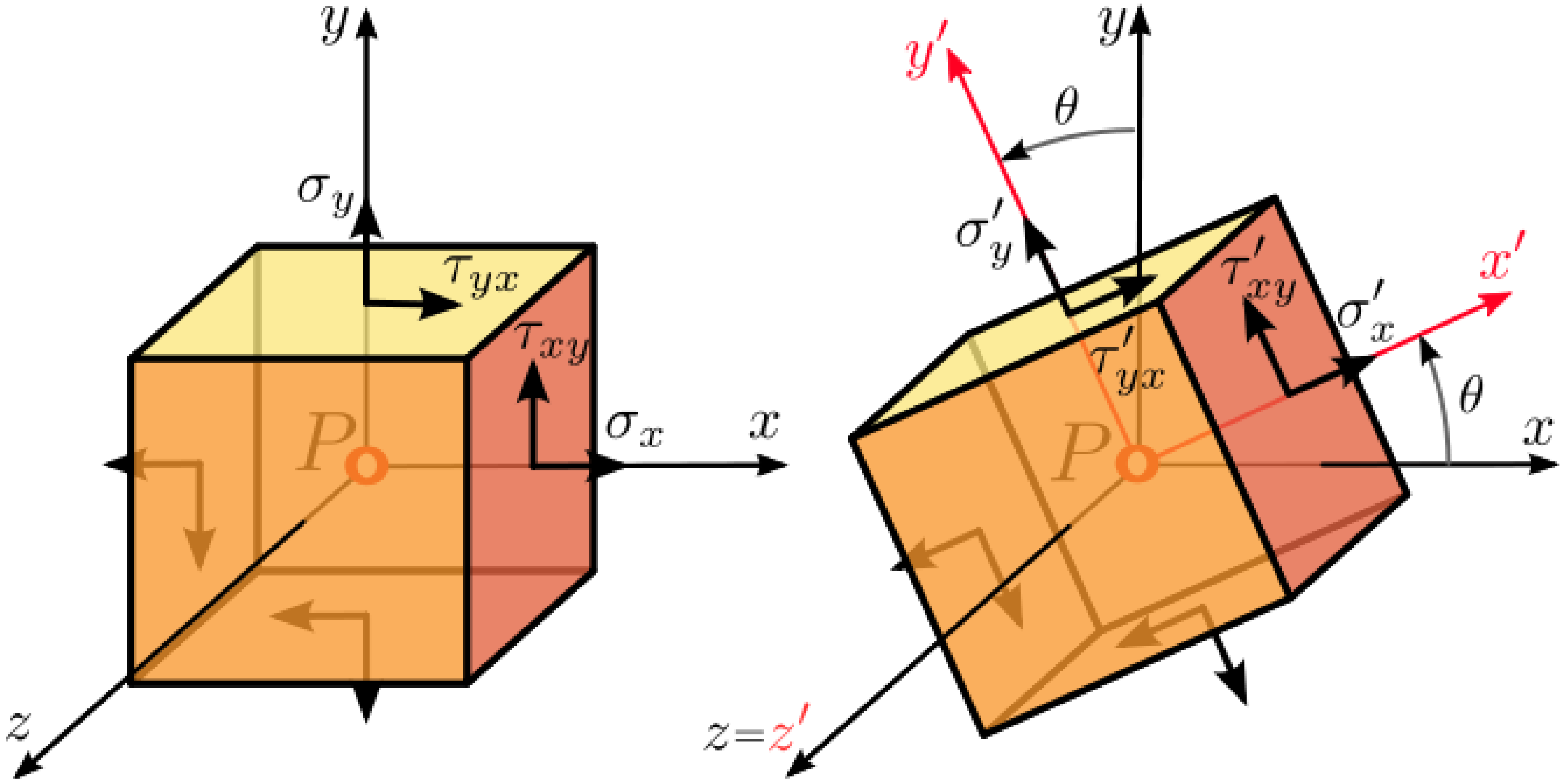
$$\tau_f = c' + \sigma'_f \tan\phi'$$

frictional component

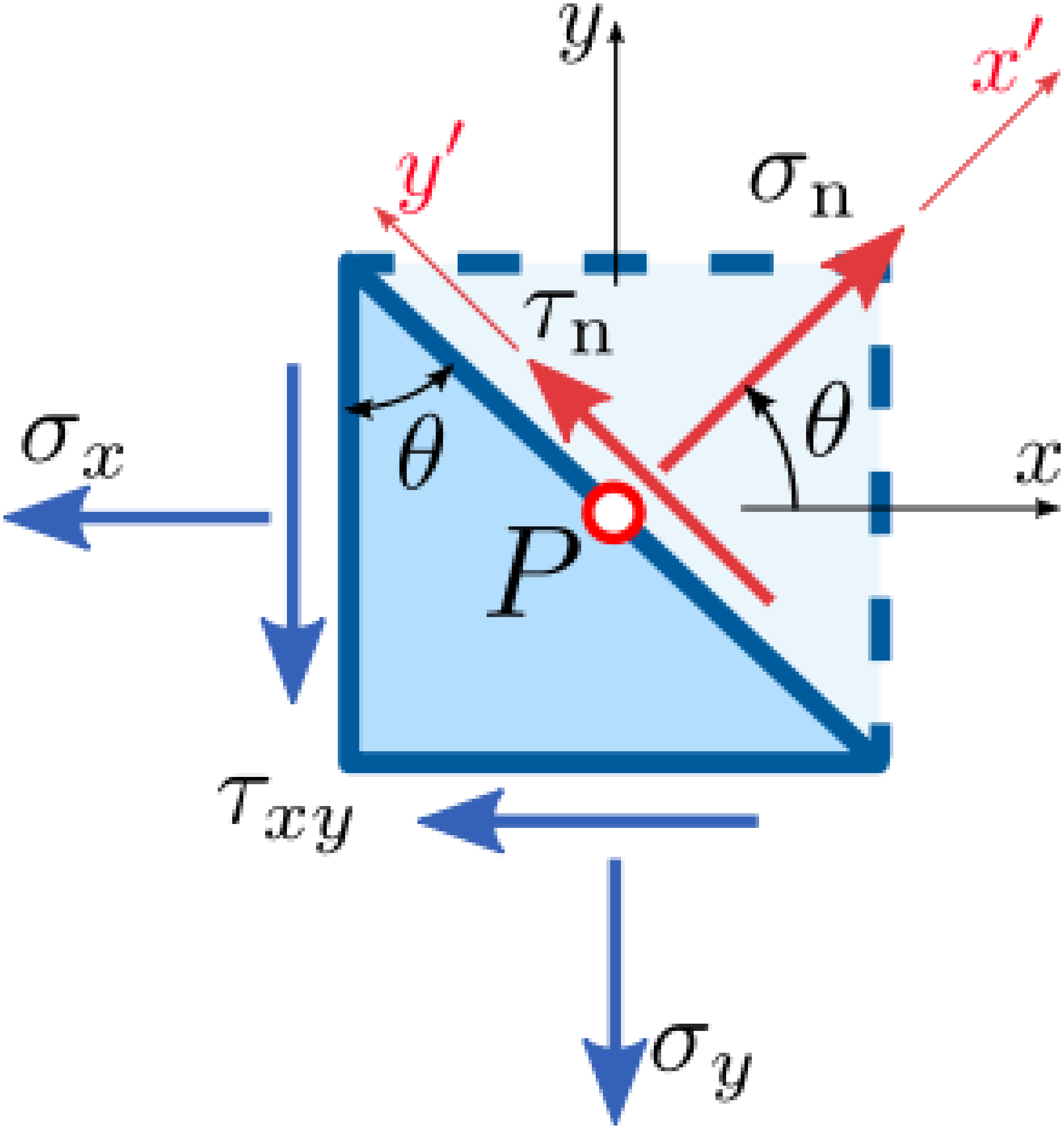
cohesive component



$\mathbf{F}_1$
∞
$\mathbf{F}_1$
$\approx$
P
P
$\mathbf{F}_2$
$\mathbf{F}_2$
y
x
y
σ_y
τ_{yx}
τ_{xy}
P
σ_x
x
z
y
σ_y
τ_{yx}
τ_{xy}
P
σ_x
x
z



y
σ_y
τ_{yx}
τ_{xy}
σ_x
x
P
z
y'
y
θ
σ'_y
τ'_{xy}
x'
σ'_x
τ'_{yx}
θ
x
P
$z = z'$



y
x'
y'
$\sigma_{\rm n}$
$\tau_{\rm n}$
θ
σ_x
θ
x
P
τ_{xy}
σ_y

Normal stresses and shear stresses on any plane can be obtained with the following equations

$$\sigma_{\mathrm{n}} = \frac{1}{2}(\sigma_x + \sigma_y) + \frac{1}{2}(\sigma_x - \sigma_y)\cos 2\theta + \tau_{xy}\sin 2\theta$$

$$\tau_{\mathrm{n}} = -\frac{1}{2}(\sigma_x - \sigma_y)\sin 2\theta + \tau_{xy}\cos 2\theta$$

Principal stresses

$$\tau_{\mathrm{n}} = -\frac{1}{2}(\sigma_x - \sigma_y)\sin 2\theta + \tau_{xy}\cos 2\theta = 0$$

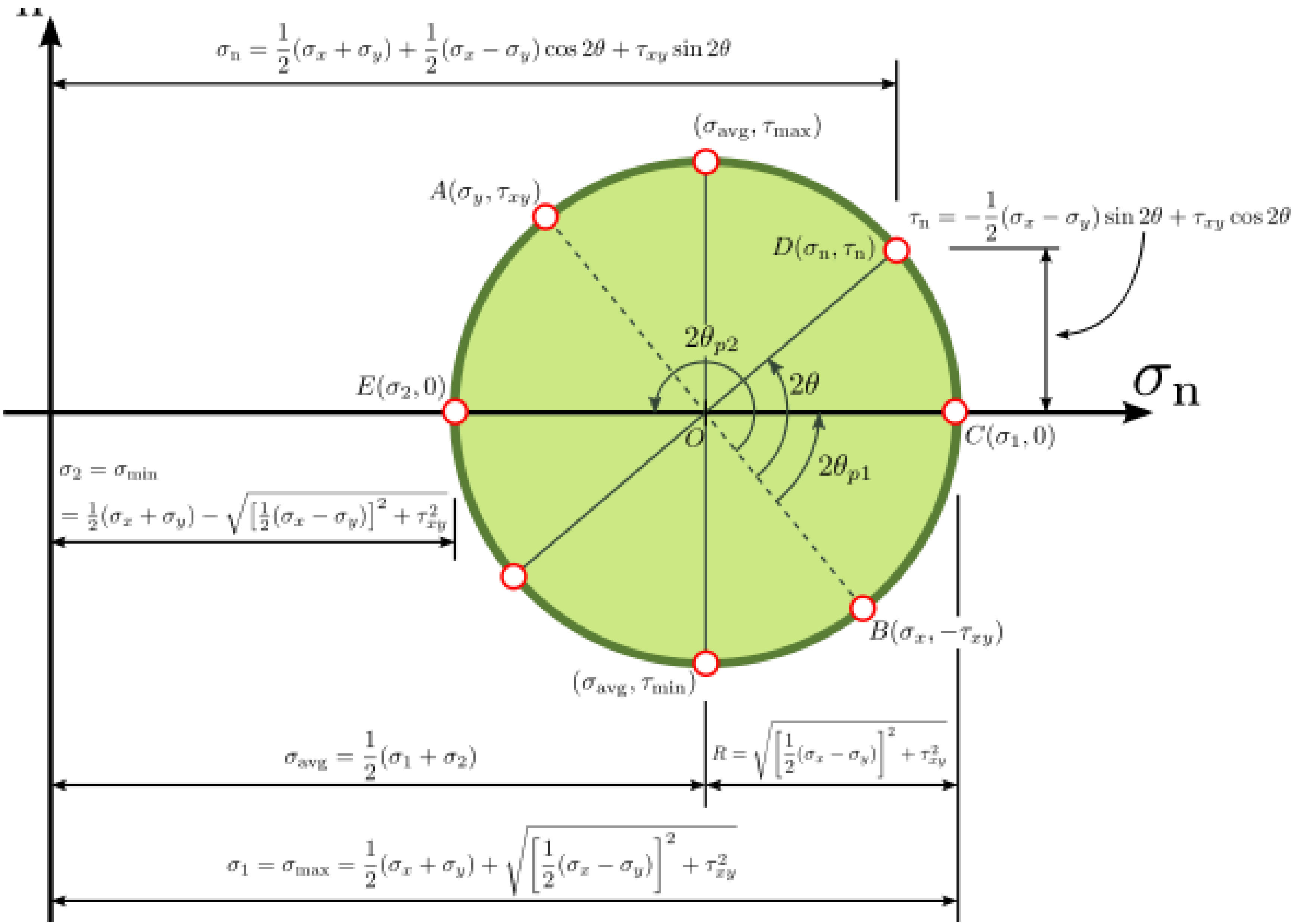
or

$$\frac{d\sigma_{\mathrm{n}}}{d\theta} = 0$$

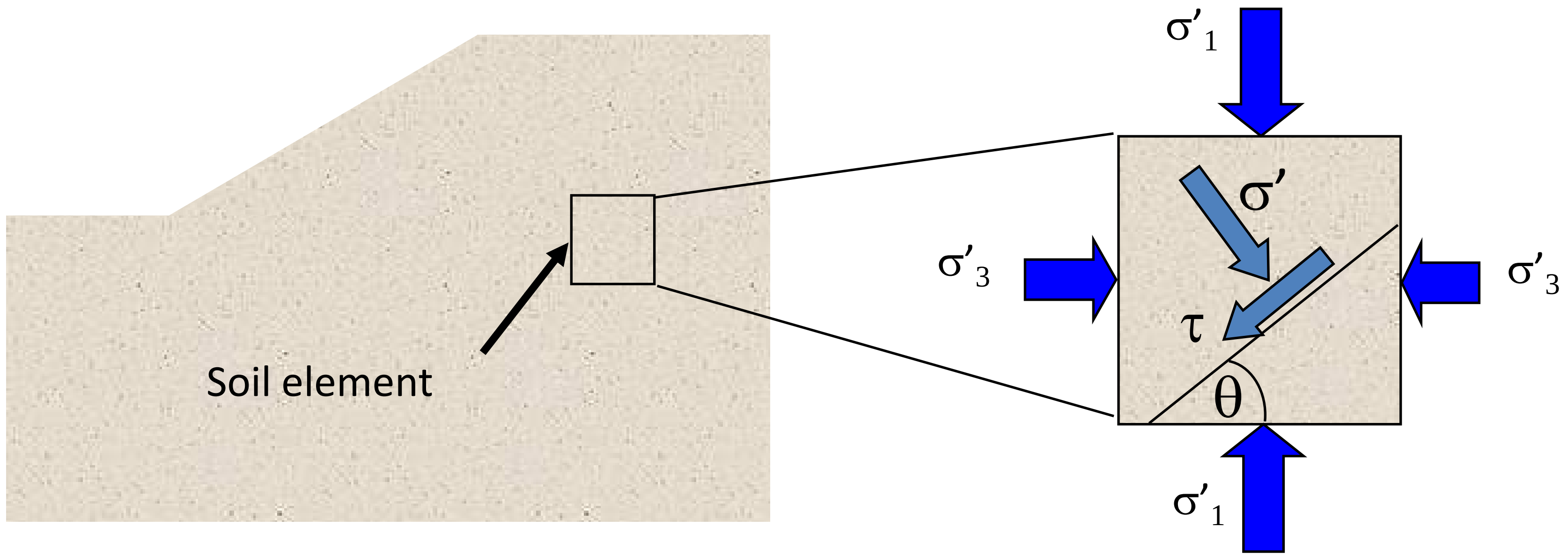
$$\tan 2\theta_{\mathrm{p}} = \frac{2\tau_{xy}}{\sigma_x - \sigma_y}$$

$$\boxed{\sigma_1 = \sigma_{\max} = \tfrac{1}{2}(\sigma_x + \sigma_y) + \sqrt{\left[\tfrac{1}{2}(\sigma_x - \sigma_y)\right]^2 + \tau_{xy}^2}}$$

$$\boxed{\sigma_2 = \sigma_{\min} = \tfrac{1}{2}(\sigma_x + \sigma_y) - \sqrt{\left[\tfrac{1}{2}(\sigma_x - \sigma_y)\right]^2 + \tau_{xy}^2}}$$


$\sigma_n = \frac{1}{2}(\sigma_x + \sigma_y) + \frac{1}{2}(\sigma_x - \sigma_y)\cos 2\theta + \tau_{xy}\sin 2\theta$
$(\sigma_{avg}, \tau_{max})$
$A(\sigma_y, \tau_{xy})$
$\tau_n = -\frac{1}{2}(\sigma_x - \sigma_y)\sin 2\theta + \tau_{xy}\cos 2\theta$
$D(\sigma_n, \tau_n)$
$2\theta_{p2}$
2θ
$E(\sigma_2, 0)$
σ_n
O
$C(\sigma_1, 0)$
$2\theta_{p1}$
$\sigma_2 = \sigma_{min}$
$= \frac{1}{2}(\sigma_x + \sigma_y) - \sqrt{\left[\frac{1}{2}(\sigma_x - \sigma_y)\right]^2 + \tau_{xy}^2}$
$B(\sigma_x, -\tau_{xy})$
$(\sigma_{avg}, \tau_{min})$
$\sigma_{avg} = \frac{1}{2}(\sigma_1 + \sigma_2)$
$R = \sqrt{\left[\frac{1}{2}(\sigma_x - \sigma_y)\right]^2 + \tau_{xy}^2}$
$\sigma_1 = \sigma_{max} = \frac{1}{2}(\sigma_x + \sigma_y) + \sqrt{\left[\frac{1}{2}(\sigma_x - \sigma_y)\right]^2 + \tau_{xy}^2}$

Mohr Circle of stress



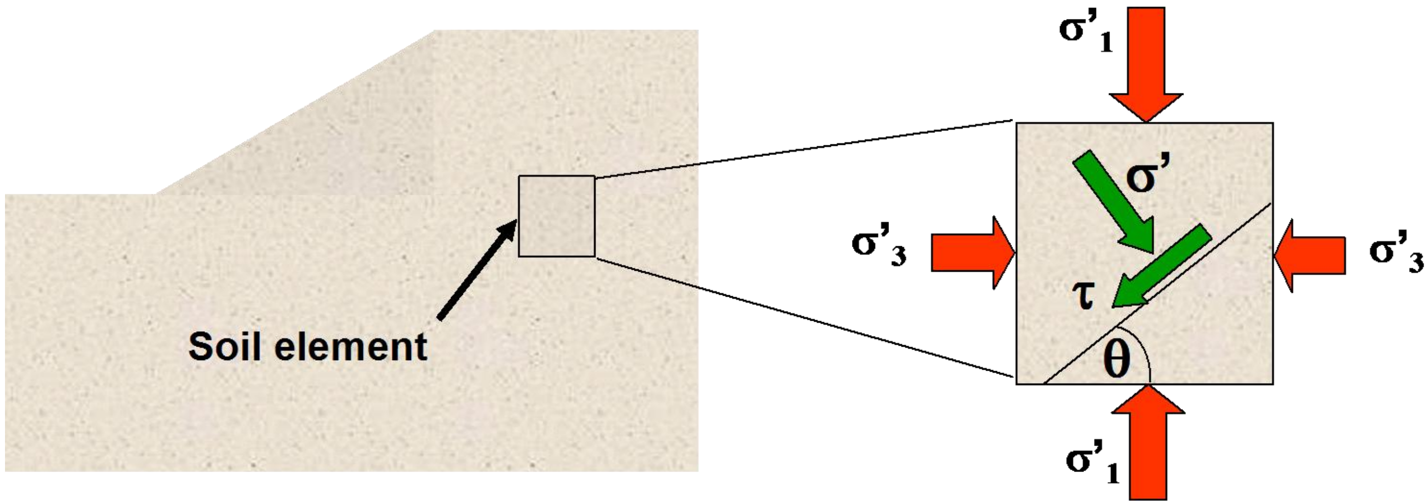
Resolving forces in σ and τ directions,

$$\tau = \frac{\sigma_1^{'} - \sigma_3^{'}}{2} Sin2\theta$$

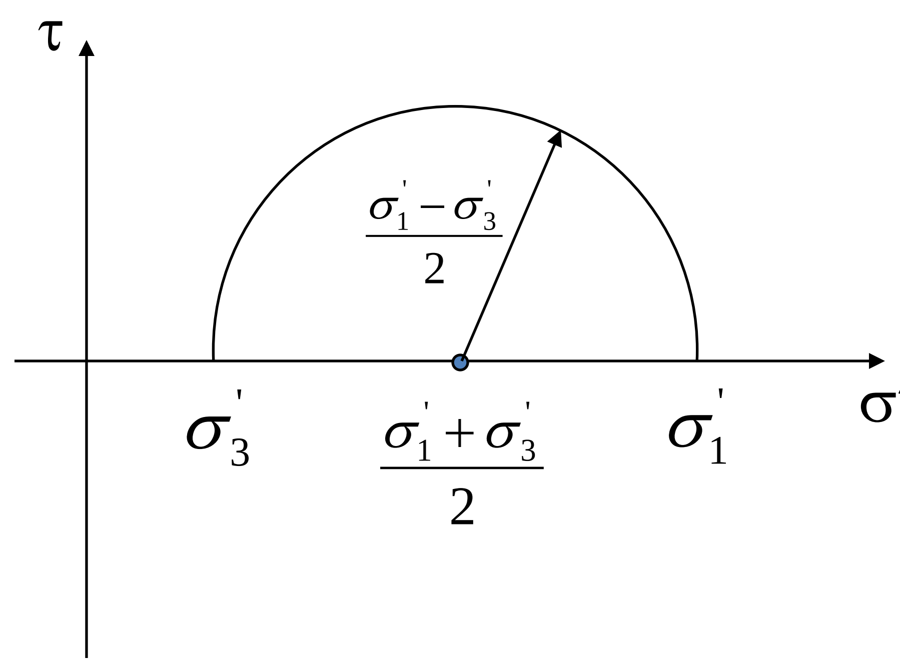
$$\sigma^{'} = \frac{\sigma_1^{'} + \sigma_3^{'}}{2} + \frac{\sigma_1^{'} - \sigma_3^{'}}{2} Cos2\theta$$

$$\tau^2 + \left(\sigma^{'} - \frac{\sigma_1^{'} + \sigma_3^{'}}{2} \right)^2 = \left(\frac{\sigma_1^{'} - \sigma_3^{'}}{2} \right)^2$$

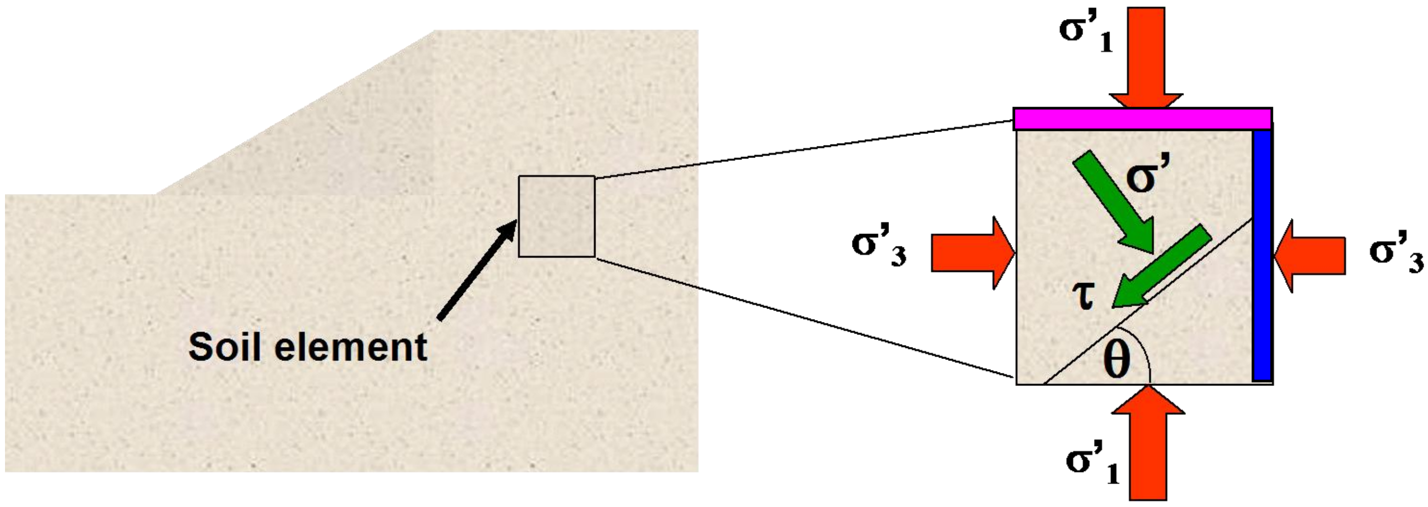
Mohr Circle of stress



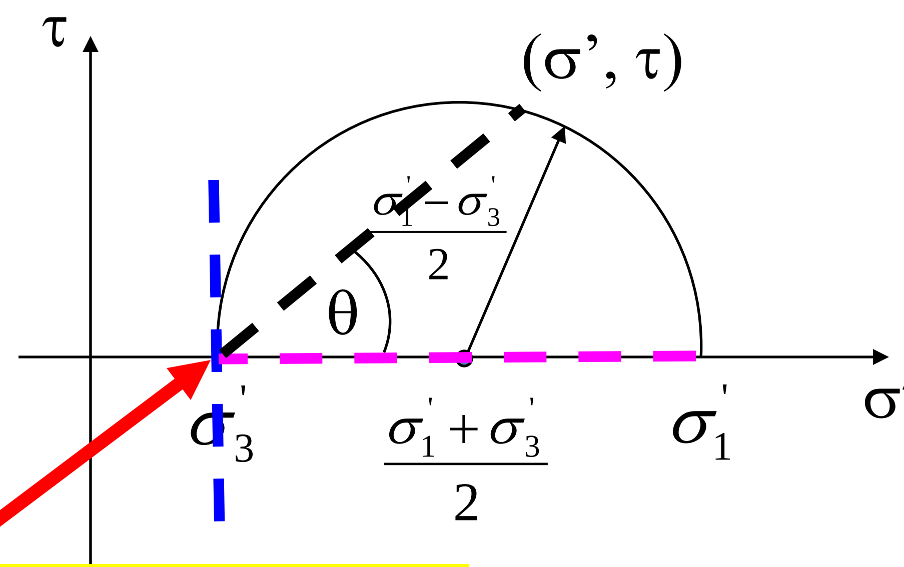
$$\tau^2 + \left(\sigma' - \frac{\sigma_1' + \sigma_3'}{2}\right)^2 = \left(\frac{\sigma_1' - \sigma_3'}{2}\right)^2$$



Mohr Circle of stress

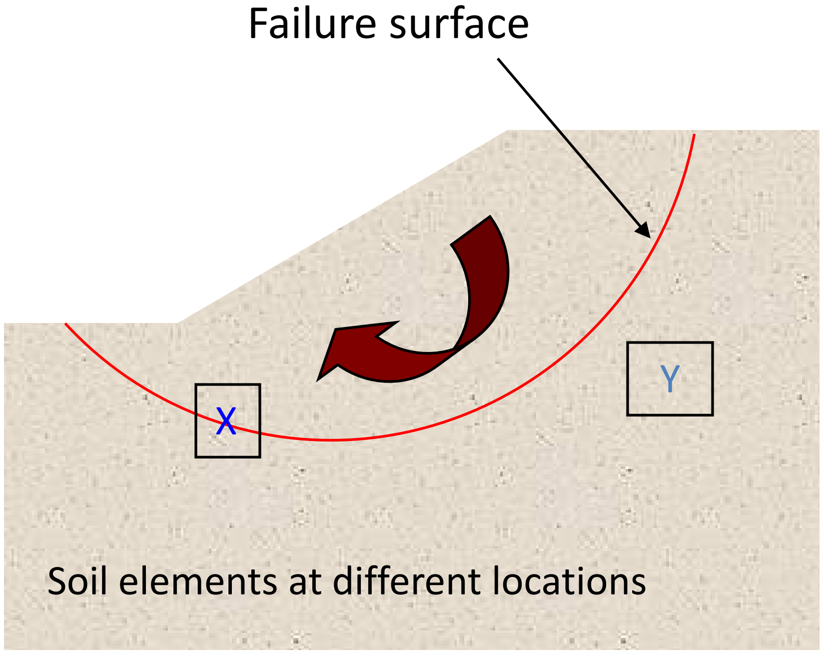


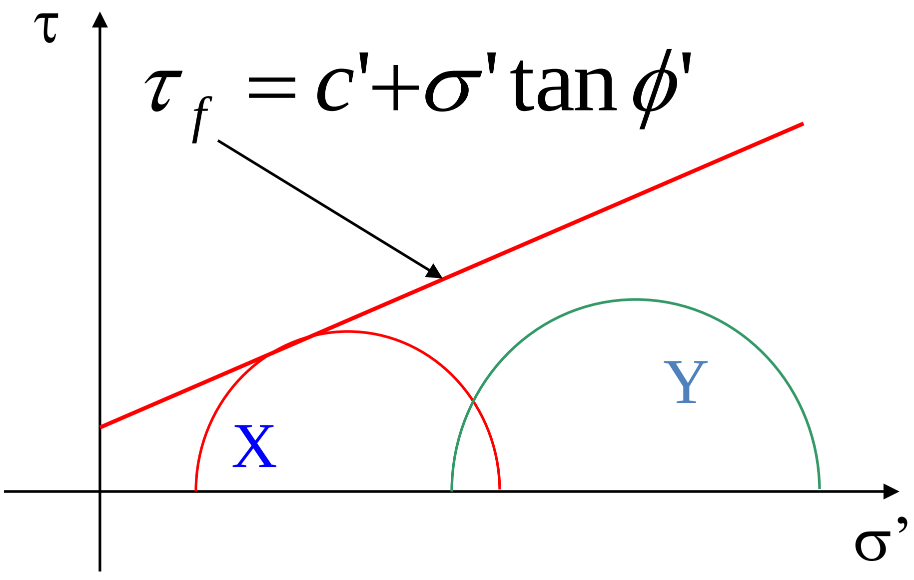
$$\tau^2 + \left(\sigma' - \frac{\sigma_1' + \sigma_3'}{2}\right)^2 = \left(\frac{\sigma_1' - \sigma_3'}{2}\right)^2$$



P_D = Pole w.r.t. plane

Mohr Circles & Failure Envelope





Y ~ stable

X ~ failure

Determination of shear strength parameters of soils (c, ϕ or c', ϕ')

Laboratory tests on specimens taken from representative undisturbed samples

Most common laboratory tests to determine the shear strength parameters are,

1. Direct shear test
2. Triaxial shear test
3. Vane shear test

Other laboratory tests include,
torsional ring shear test, plane strain triaxial test, laboratory vane shear test, laboratory fall cone test

Field tests

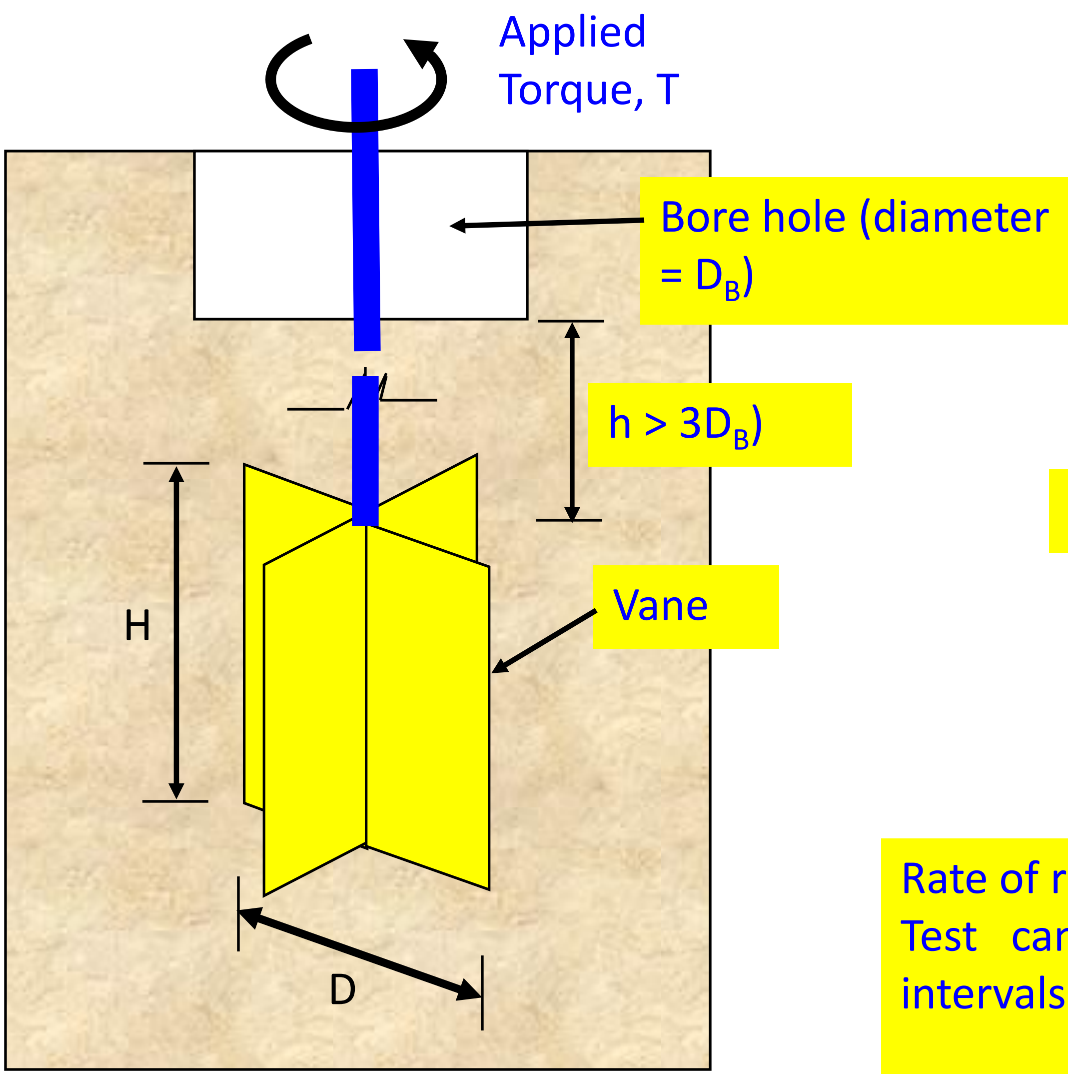
1. Vane shear test
2. Torvane
3. Pocket penetrometer
4. Fall cone
5. Pressuremeter
6. Static cone penetrometer
7. Standard penetration test

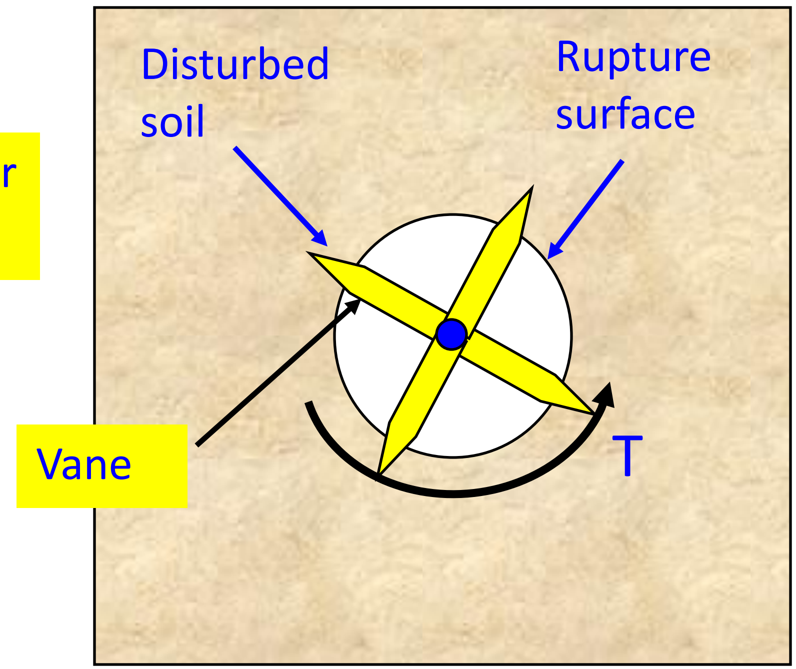
Vane shear test

- What does Vane Shear Test measure?
  - Shear strength
    - a term used to describe the maximum strength of soil at which point significant plastic deformation or yielding occurs due to an applied shear stress.
  - Undrained shear strength
    - refers to a shear condition where water does not enter or leave the cohesive soil during the shearing process
  - Remolded undrained shear strength
    - is the peak undrained shearing resistance measured during the initial rotation of the vane.
  - Peak undrained shear strength
    - is the shear strength after significant failure and remolding of the initial soil structure.
  - Sensitivity
    - is the effect of remolding on the consistency of cohesive soil.

Vane shear test

This is one of the most versatile and widely used devices used for investigating <u>undrained shear strength</u> (C_u) and sensitivity of soft clays

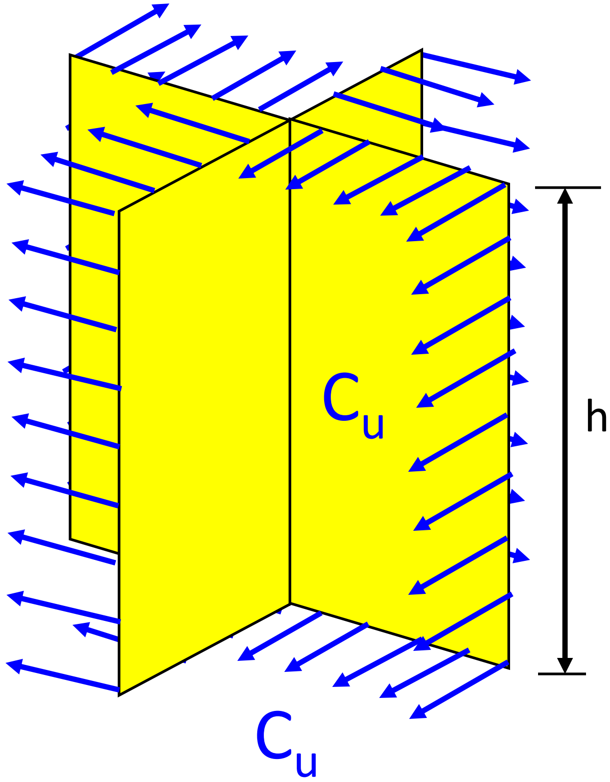




PLAN VIEW

Rate of rotation : $6^0 – 12^0$ per minute
Test can be conducted at 0.5 m vertical intervals

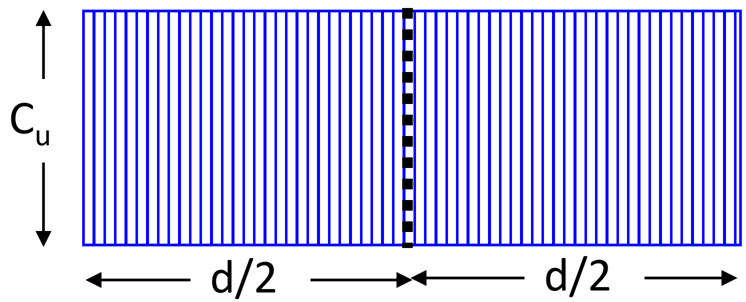
Vane shear test



Since the test is very fast, Unconsolidated Undrained (UU) can be expected

$$T = M_s + M_e + M_e = M_s + 2M_e$$

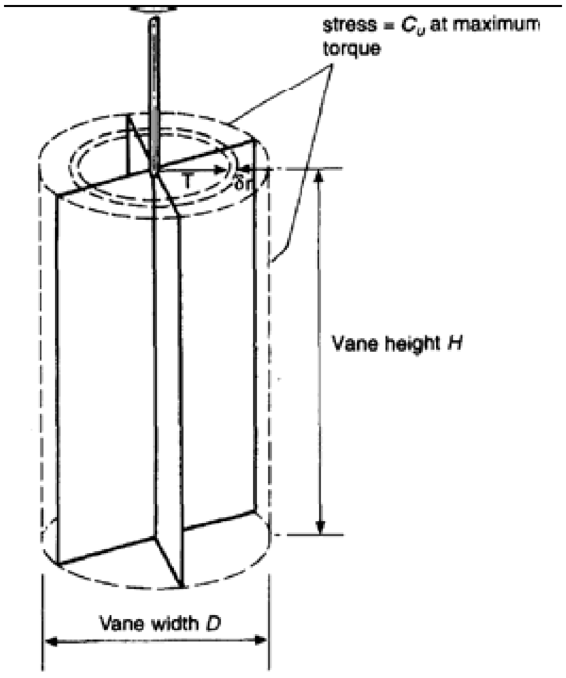
M_e – *Assuming a uniform distribution of shear strength*



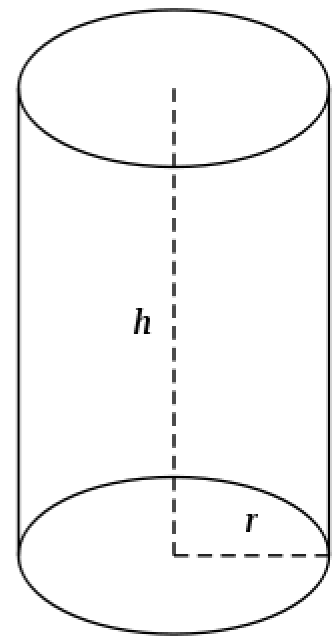
$$M_e = \int_0^{\frac{d}{2}} (2\pi r dr).C_u r$$

$$M_e = 2\pi C_u \int_0^{\frac{d}{2}} r^2 dr = 2\pi C_u \left[\frac{r^3}{3}\right]_0^{\frac{d}{2}}$$

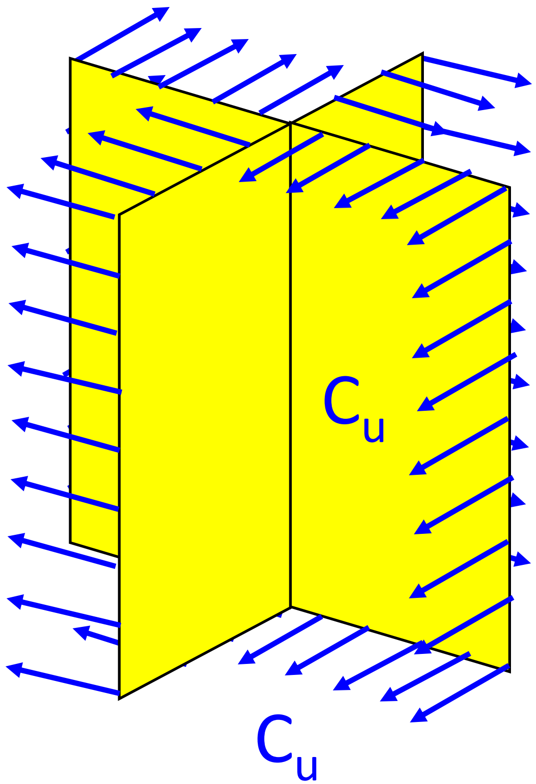
$$M_e = \frac{2\pi C_u}{3}\left[\frac{d^3}{8}\right] = \frac{\pi C_u d^3}{12}$$



Surface area of the cylinder = 2πrh= πdh



Vane shear test



$T = M_s + M_e + M_e = M_s + 2M_e$

M_s – Shaft shear resistance along the circumference

$$M_s = \pi dh C_u \frac{d}{2} = \pi C_u \frac{d^2 h}{2}$$

$$T = \pi C_u \frac{d^2 h}{2} + \frac{\pi C_u d^3}{12} \times 2$$

$$T = \pi C_u \left(\frac{d^2 h}{2} + \frac{d^3}{6} \right)$$

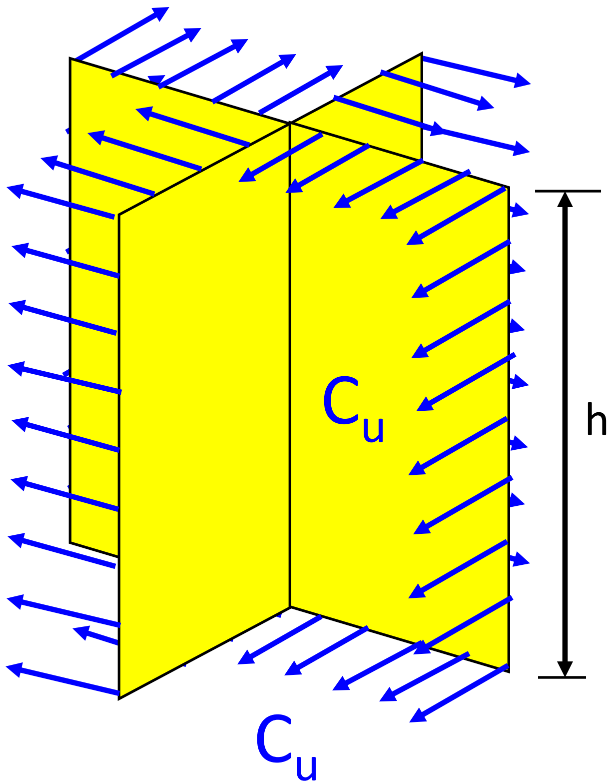
$$C_u = \frac{T}{\pi \left(\frac{d^2 h}{2} + \frac{d^3}{6} \right)}$$

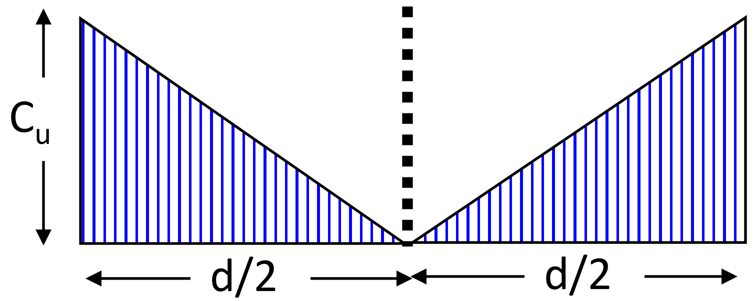
Since the test is very fast, Unconsolidated Undrained (UU) can be expected

Vane shear test

$T = M_s + M_e + M_e = M_s + 2M_e$

M_e – *Assuming a triangular distribution of shear strength*





$$C_u = \frac{T}{\pi\left(\frac{d^2h}{2} + \frac{d^3}{8}\right)}$$

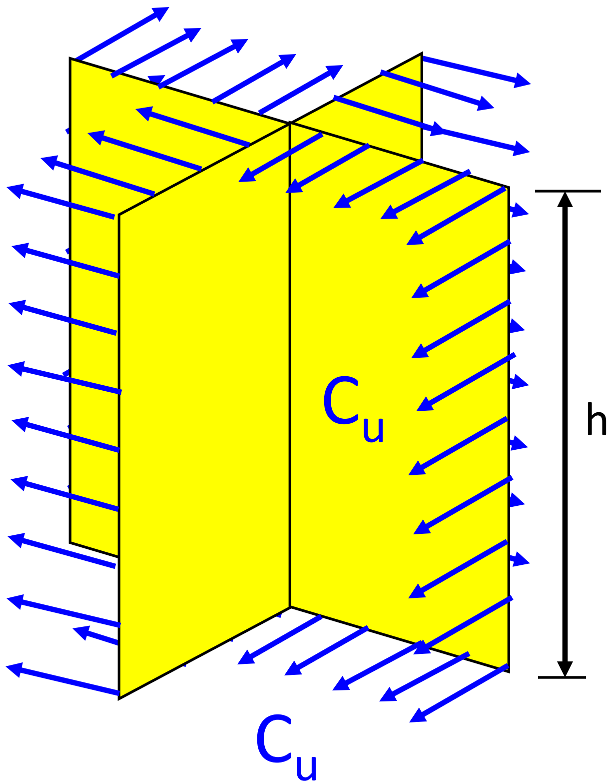
Since the test is very fast, Unconsolidated Undrained (UU) can be expected

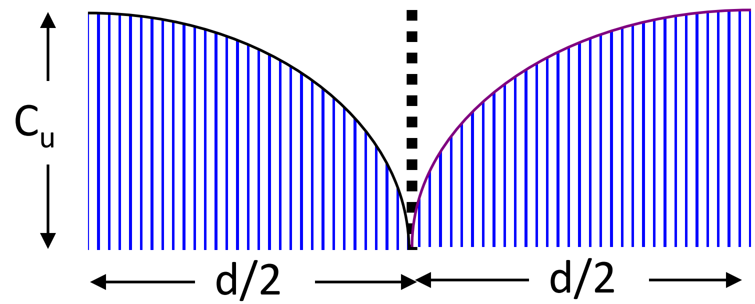
Can you derive this ???

Vane shear test

$T = M_s + M_e + M_e = M_s + 2M_e$

M_e – Assuming a parabolic distribution of shear strength



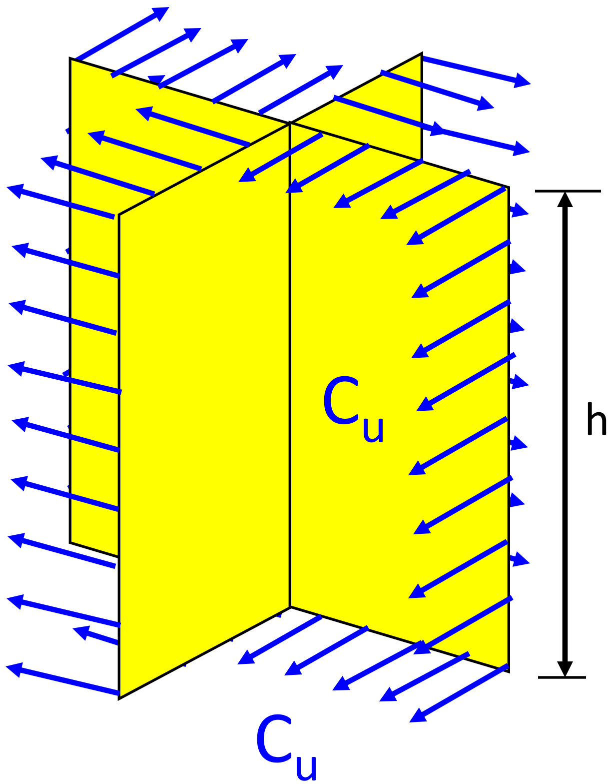


$$C_u = \frac{T}{\pi\left(\frac{d^2h}{2} + \frac{3d^3}{20}\right)}$$

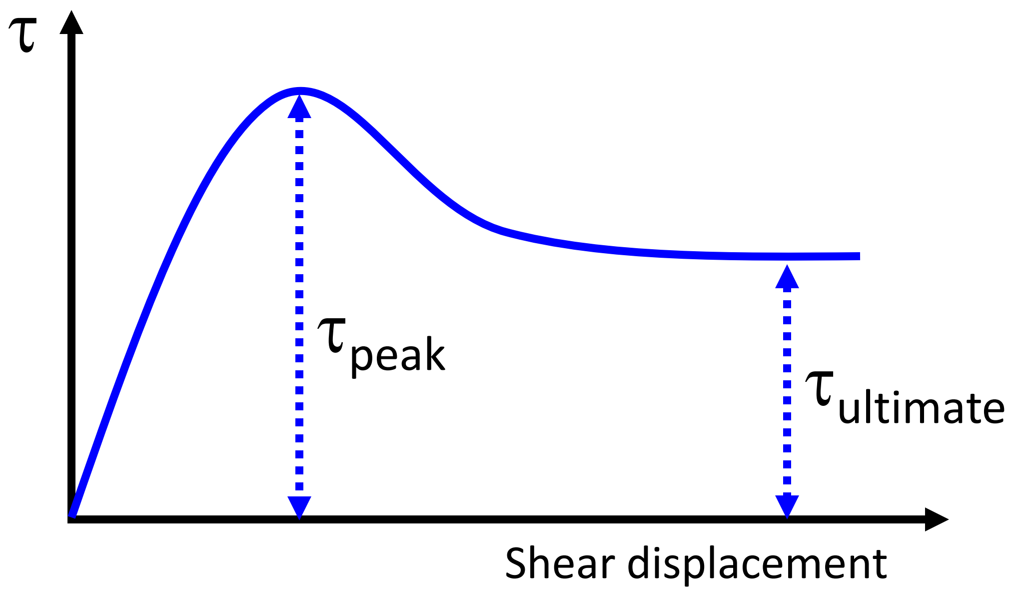
Since the test is very fast, Unconsolidated Undrained (UU) can be expected

Can you derive this ???

Vane shear test



After the initial test, vane can be rapidly rotated through several revolutions until the clay become remoulded



Since the test is very fast, Unconsolidated Undrained (UU) can be expected

$$Sensitivity = \frac{Peak\ Stength}{Ultimate\ Stength}$$

Some important facts on vane shear test

Insertion of vane into soft clays and silts disrupts the natural soil structure around the vane causing reduction of shear strength

The above reduction is partially regained after some time

C_u as determined by vane shear test may be a function of the rate of angular rotation of the vane

Correction for the strength parameters obtained from vane shear test

Bjerrum (1974) has shown that as the plasticity of soils increases, C_u obtained by vane shear tests may give unsafe results for foundation design. Therefore, he proposed the following correction.

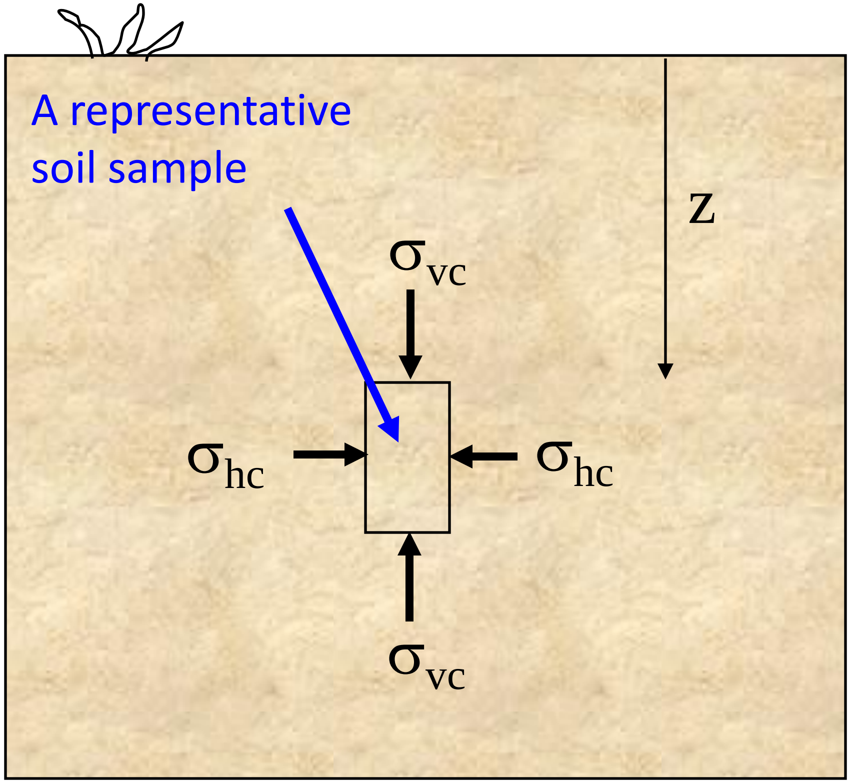
$$C_{u(design)} = \lambda C_{u(vane\ shear)}$$

Where, λ = correction factor = 1.7 – 0.54 log (PI)

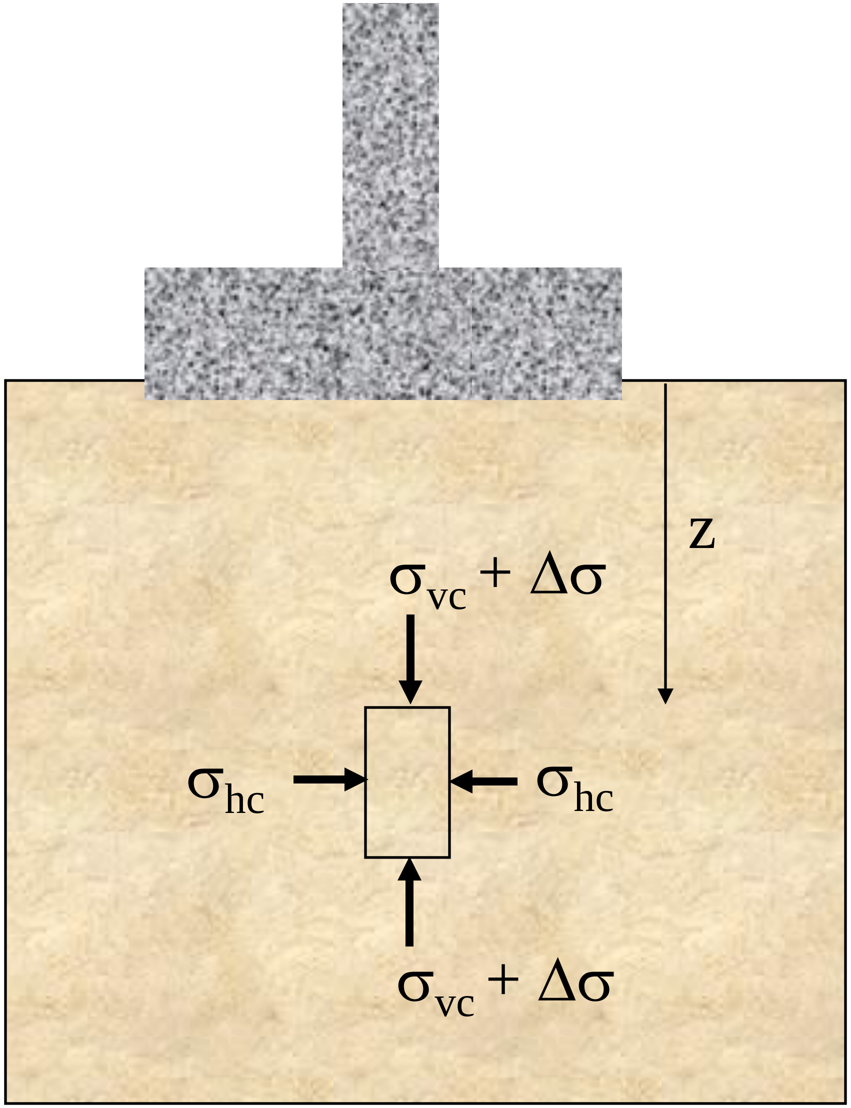
PI = Plasticity Index

Laboratory tests

Field conditions



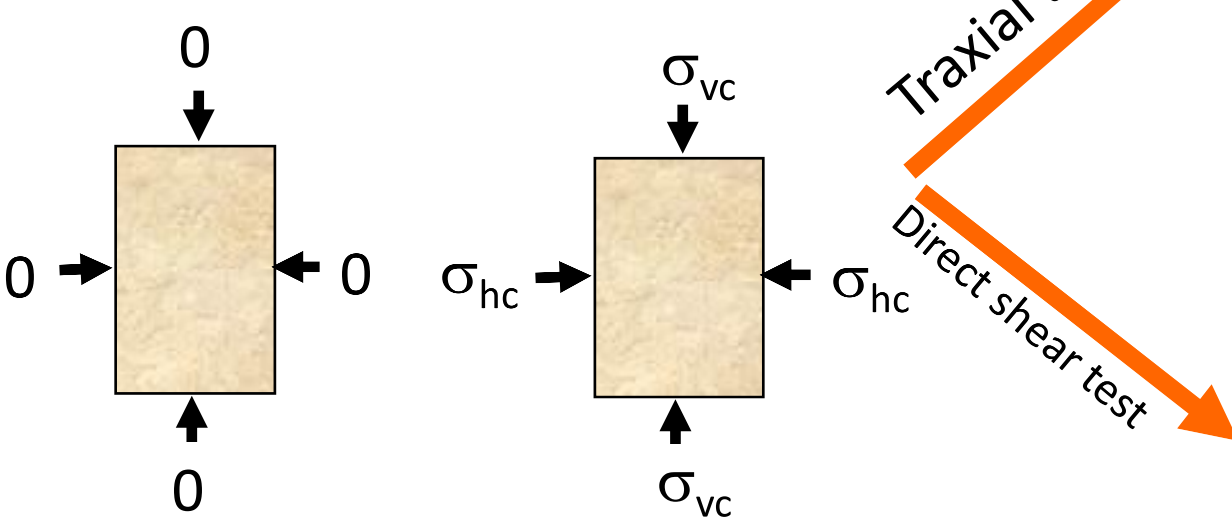
Before construction

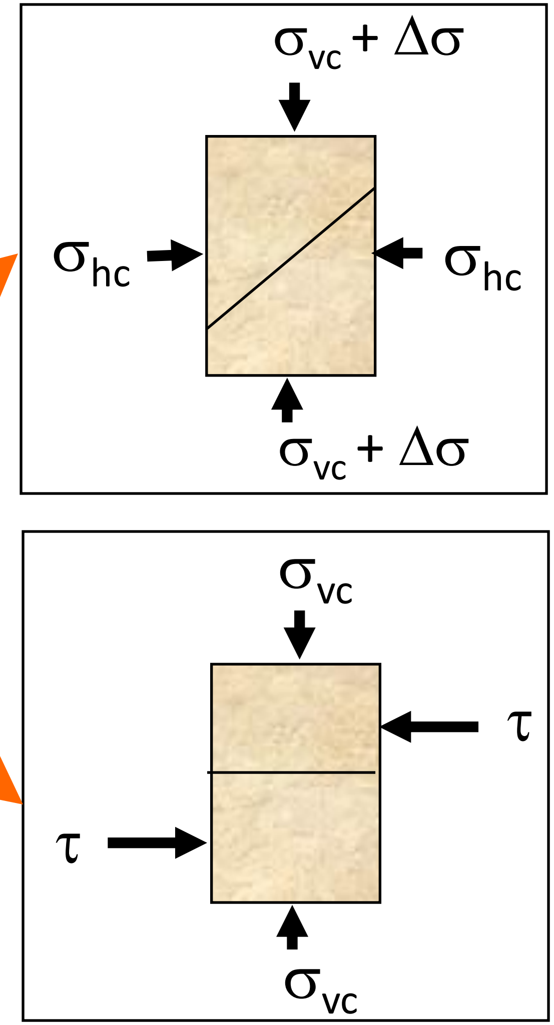


After and during construction

Laboratory tests

Simulating field conditions in the laboratory





Representative soil sample taken from the site

Step 1
Set the specimen in the apparatus and apply the initial stress condition

Step 2
Apply the corresponding field stress conditions

Direct Shear Test

Direct Shear Test Apparatus

Direct shear test

NEED AND SCOPE

In many engineering problems such as

- design of foundation,
- retaining walls,
- slab bridges,
- pipes,
- sheet piling,

The value of the angle of internal friction and cohesion of the soil involved are required for the design.

Direct shear test is used to predict these parameters quickly.

Direct shear test

1. This test is performed to determine the consolidated - drained shear strength of a sandy to silty soil.
2. The shear strength is one of the most important engineering properties of a soil, because it is required whenever a structure is dependent on the soil's shearing resistance.
3. The shear strength is needed for engineering situations such as determining the stability of slopes or cuts, finding the bearing capacity for foundations, and calculating the pressure exerted by a soil on a retaining wall.

Apparatus

1. Direct shear box apparatus
2. Loading frame (motor attached).
3. Dial gauge.
4. Proving ring.
5. Tamper.
6. Straight edge.
7. Balance to weigh upto 200 mg.
8. Aluminum container.
9. Spatula.

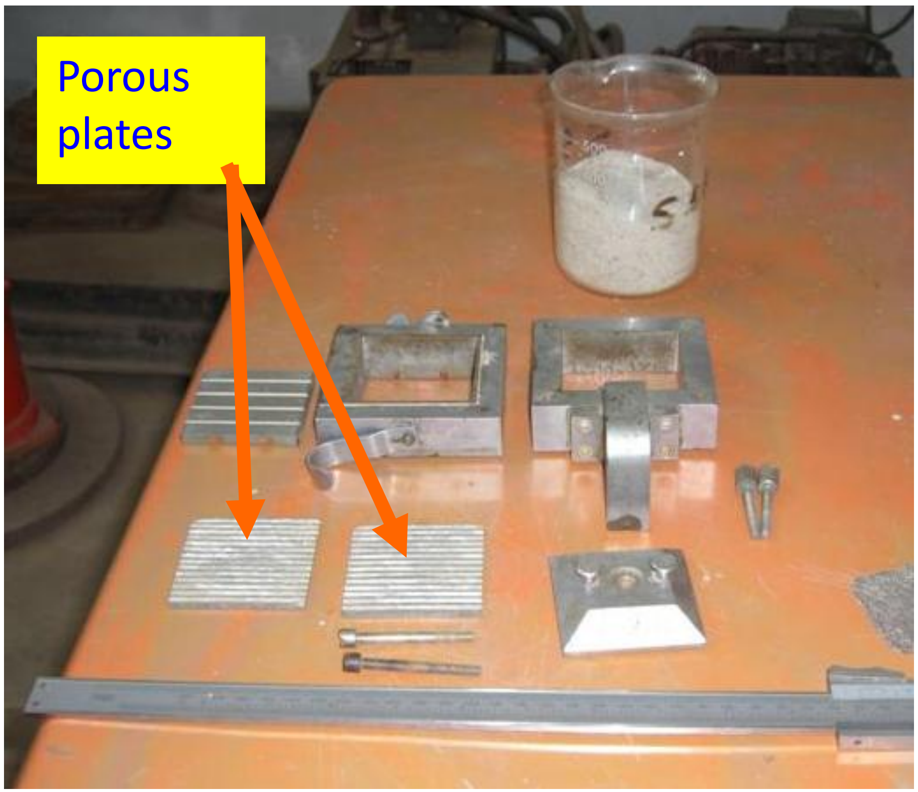
PROCEDURE

- Check the inner dimension of the soil container.
- Put the parts of the soil container together.
- Calculate the volume of the container. Weigh the container.
- Place the soil in smooth layers (approximately 10 mm thick). If a dense sample is desired tamp the soil.
- Weigh the soil container, the difference of these two is the weight of the soil. Calculate the density of the soil.
- Make the surface of the soil plane.
- Put the upper grating on stone and loading block on top of soil.

Direct shear test

Direct shear test is most suitable for <u>consolidated drained</u> tests specially on granular soils (e.g.: sand) or stiff clays

Preparation of a sand specimen



Components of the shear box

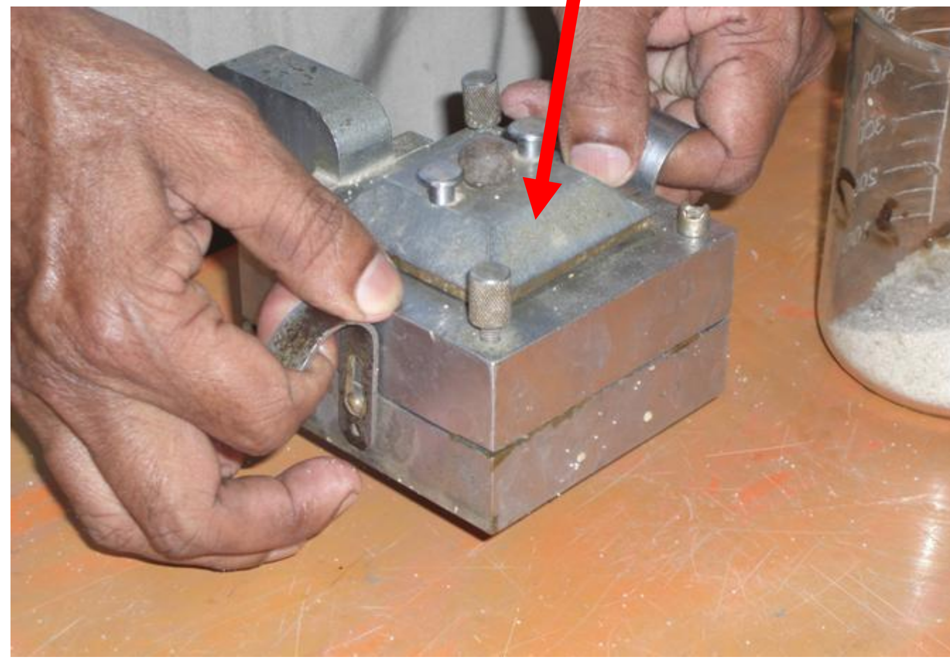
Preparation of a sand specimen

Direct shear test

Preparation of a sand specimen

Leveling the top surface of specimen

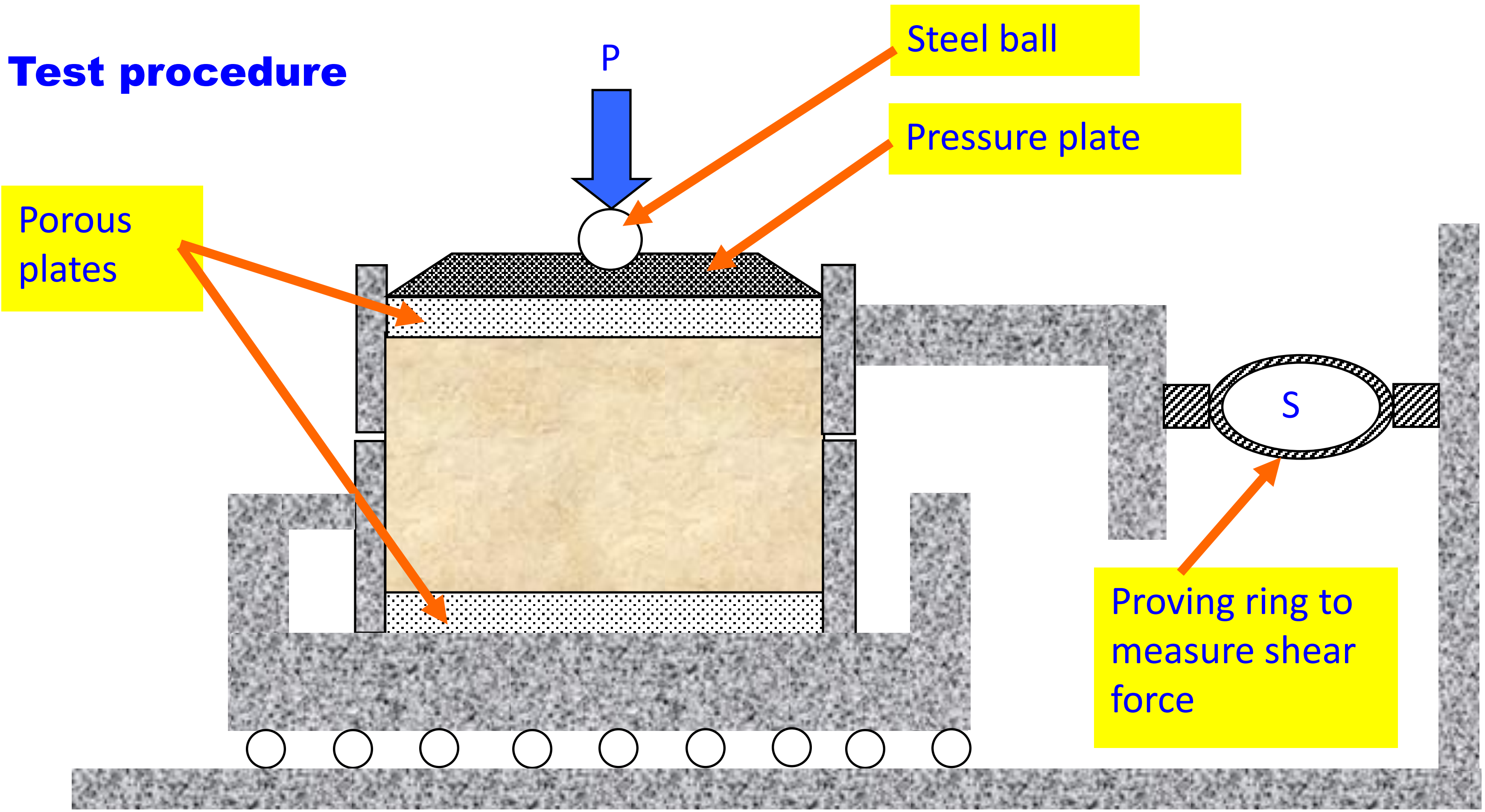
Pressure plate



Specimen preparation completed

Direct shear test

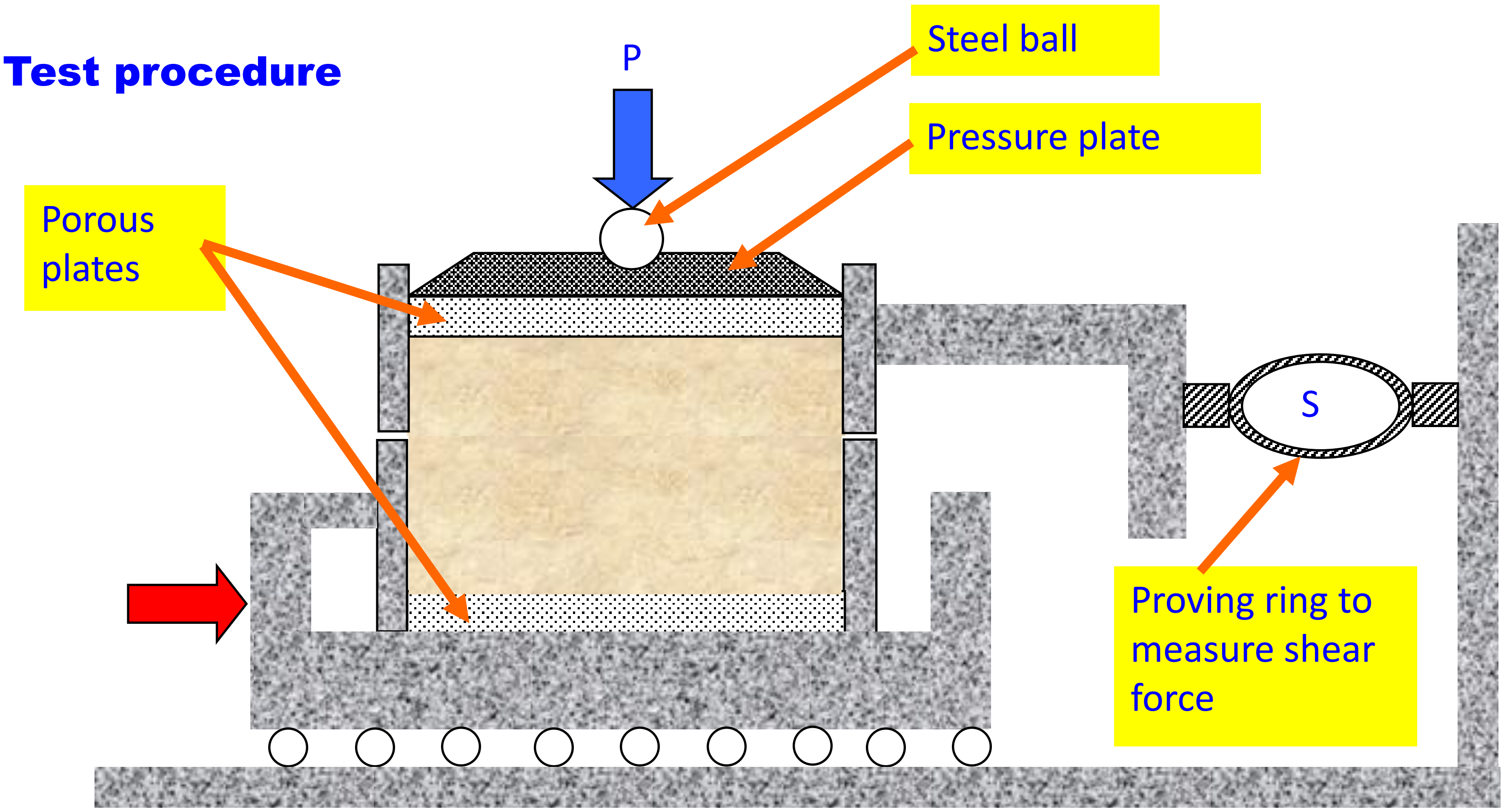
Test procedure



Step 1: Apply a vertical load to the specimen and wait for consolidation

Direct shear test

Test procedure



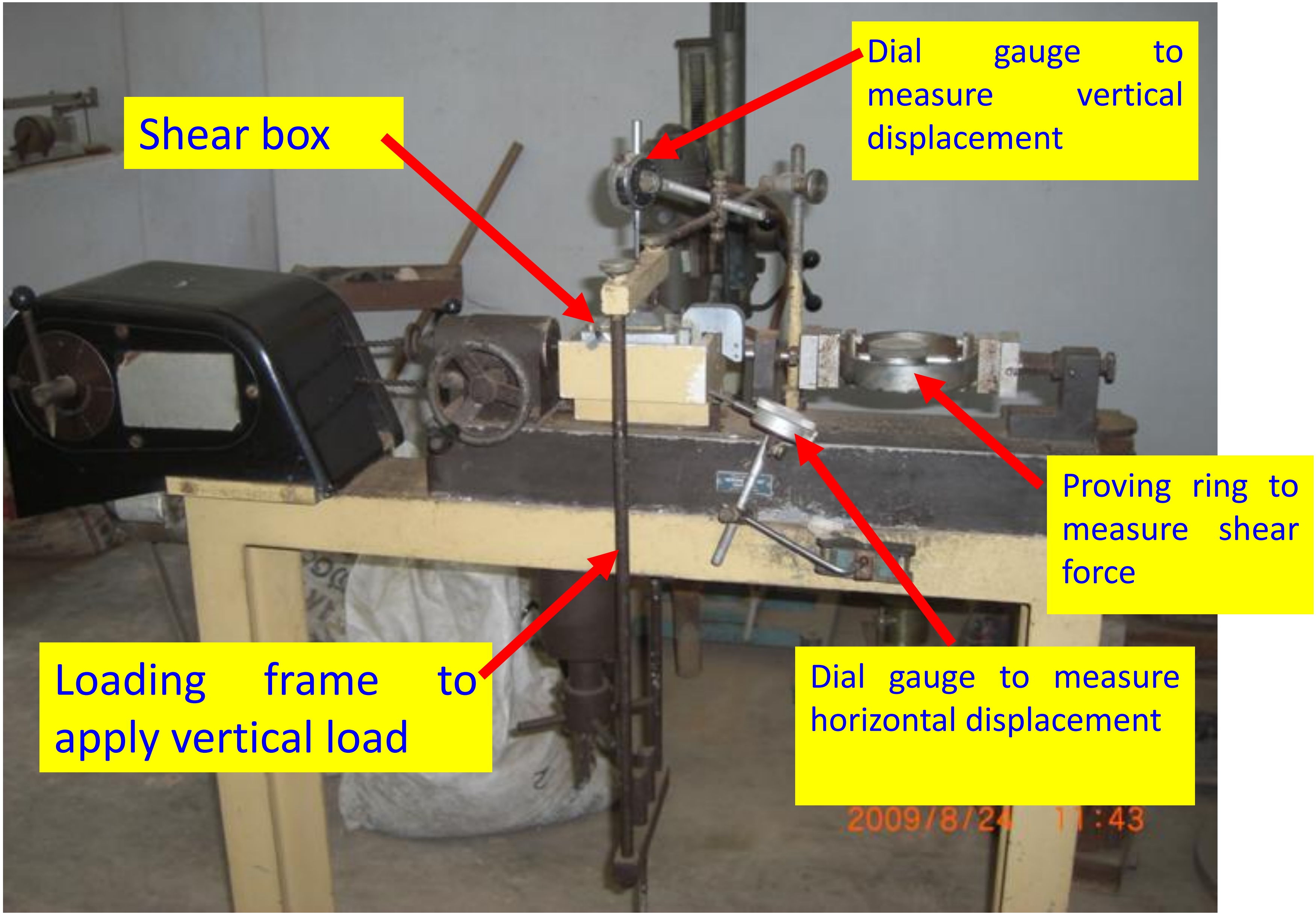
Step 1: Apply a vertical load to the specimen and wait for consolidation

Step 2: Lower box is subjected to a horizontal displacement at a constant rate

PROCEDURE

8. Measure the thickness of soil specimen.
9. Apply the desired normal load.
10. Remove the shear pin.
11. Attach the dial gauge which measures the change of volume.
12. Record the initial reading of the dial gauge and calibration values.
13. Before proceeding to test check all adjustments to see that there is no connection between two parts except sand/soil.
14. Start the motor. Take the reading of the shear force and record the reading.
15. Take volume change readings till failure.
16. Add 5 kg normal stress 0.5 kg/cm2 and continue the experiment till failure
17. Record carefully all the readings. Set the dial gauges zero, before starting the experiment

Direct shear test



Direct shear test

Analysis of test results

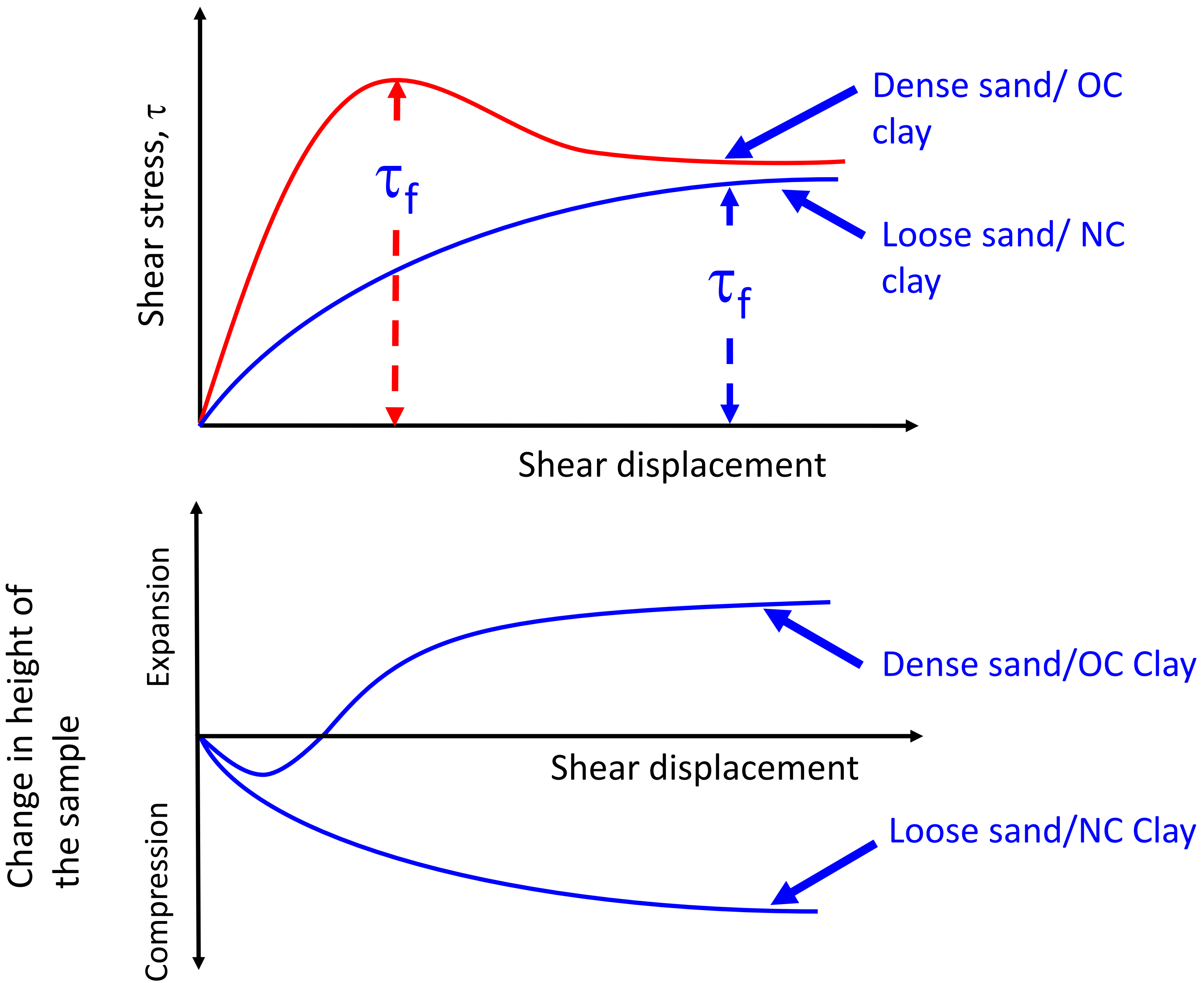
$$\sigma = \text{Normal stress} = \frac{\text{Normal force (P)}}{\text{Area of cross section of the sample}}$$

$$\tau = \text{Shear stress} = \frac{\text{Shear resistance developed at the sliding surface (S)}}{\text{Area of cross section of the sample}}$$

Note: Cross-sectional area of the sample changes with the horizontal displacement

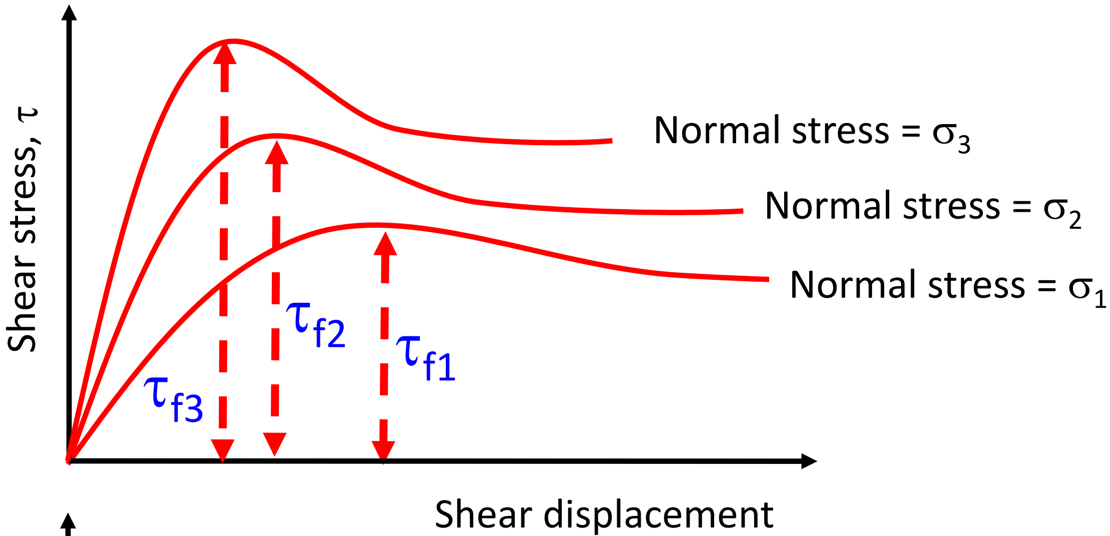
Direct shear tests on sands

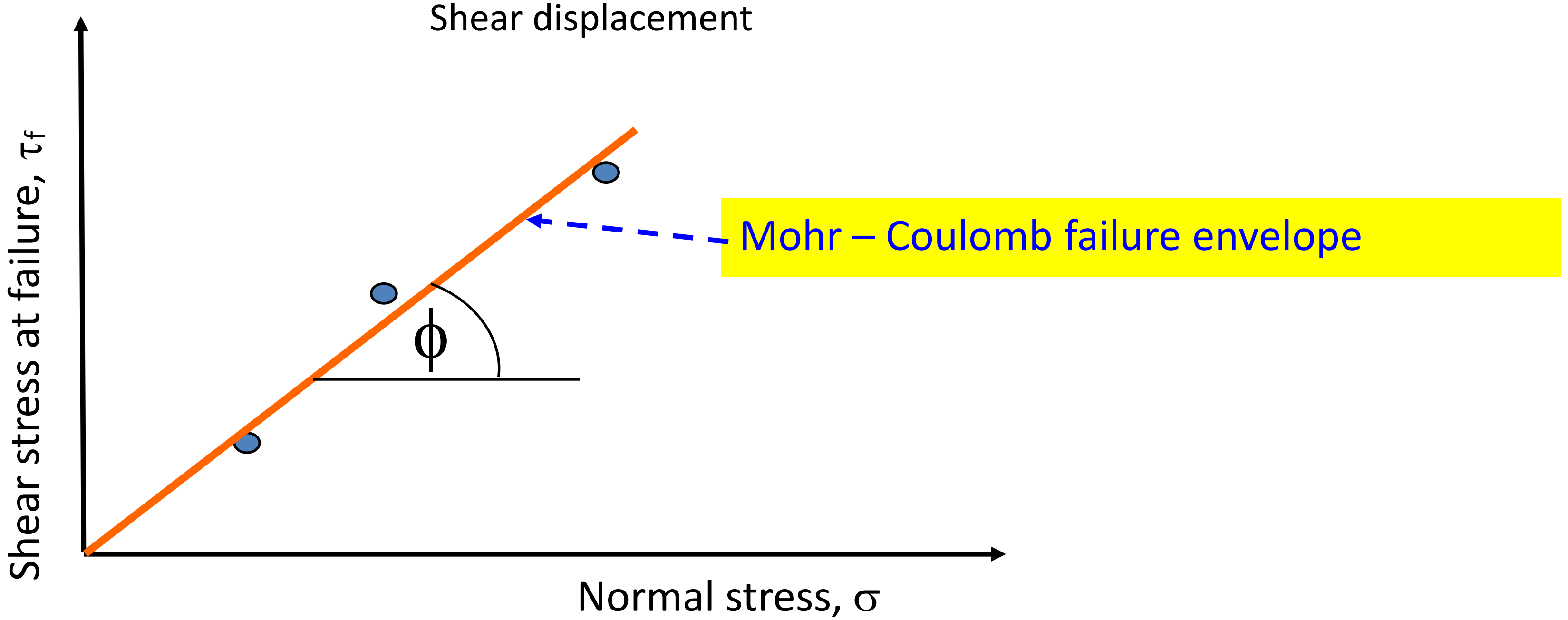
Stress-strain relationship



Direct shear tests on sands

How to determine strength parameters c and ϕ





Direct shear tests on sands

Some important facts on strength parameters c and ϕ of sand

Sand is cohesionless hence c = 0

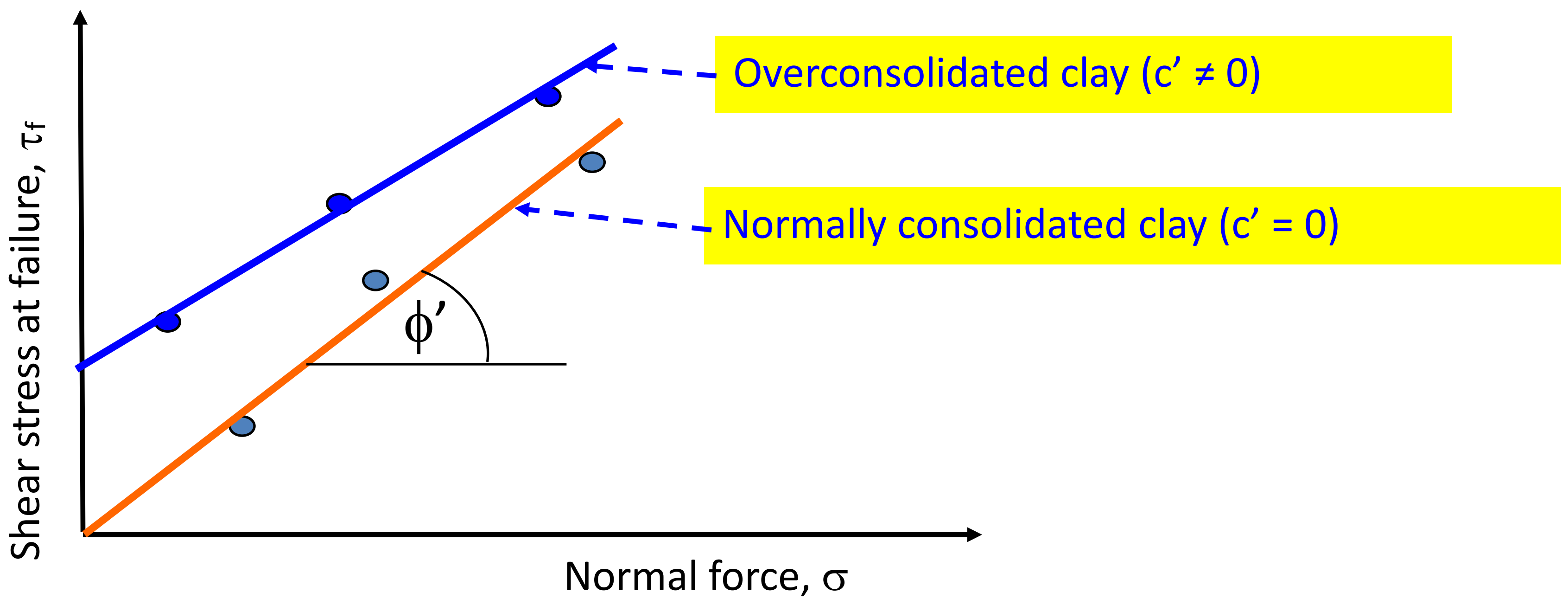
Direct shear tests are drained and pore water pressures are dissipated, hence u = 0

Therefore,
$\phi' = \phi$ and $c' = c = 0$

Direct shear tests on clays

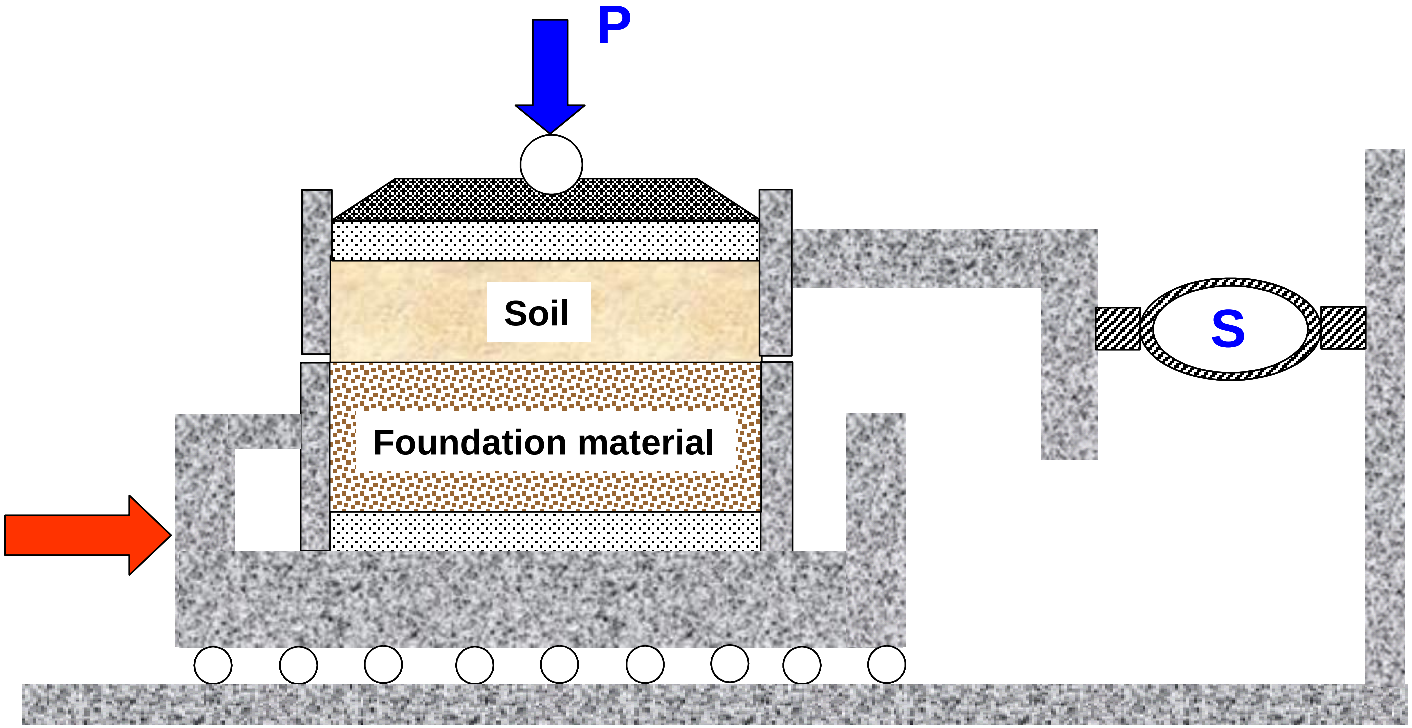
In case of clay, horizontal displacement should be applied at a very slow rate to allow dissipation of pore water pressure (therefore, one test would take several days to finish)

Failure envelopes for clay from drained direct shear tests



Interface tests on direct shear apparatus

In many foundation design problems and retaining wall problems, it is required to determine the angle of internal friction between soil and the structural material (concrete, steel or wood)



$$\tau_f = c_a + \sigma' \tan \delta$$

Where,
c_a = adhesion,
δ = angle of internal friction

Advantages of direct shear apparatus

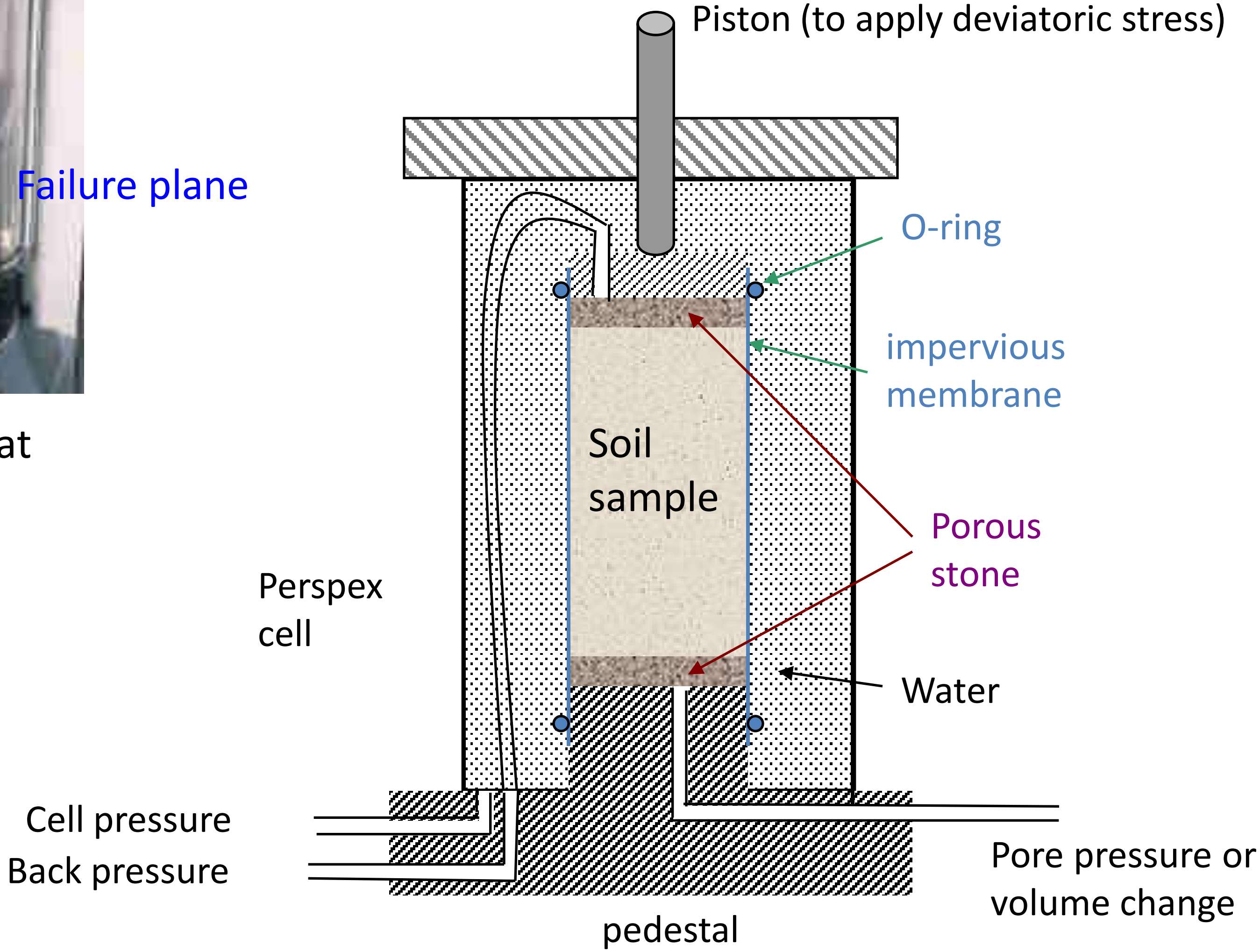
- Due to the smaller thickness of the sample, rapid drainage can be achieved
- Can be used to determine interface strength parameters
- Clay samples can be oriented along the plane of weakness or an identified failure plane

Disadvantages of direct shear apparatus

- Failure occurs along a predetermined failure plane
- Area of the sliding surface changes as the test progresses
- Non-uniform distribution of shear stress along the failure surface

Triaxial Shear Test

Soil sample at failure



ELE
Pressure indicating panel
ELE
DIGITAL TRITEST

Triaxial Shear Test

Specimen preparation (undisturbed sample)

Sampling tubes

Sample extruder

Triaxial Shear Test

Specimen preparation (undisturbed sample)

Edges of the sample are carefully trimmed

Setting up the sample in the triaxial cell

Triaxial Shear Test

Specimen preparation (undisturbed sample)

Sample is covered with a rubber membrane and sealed

Cell is completely filled with water

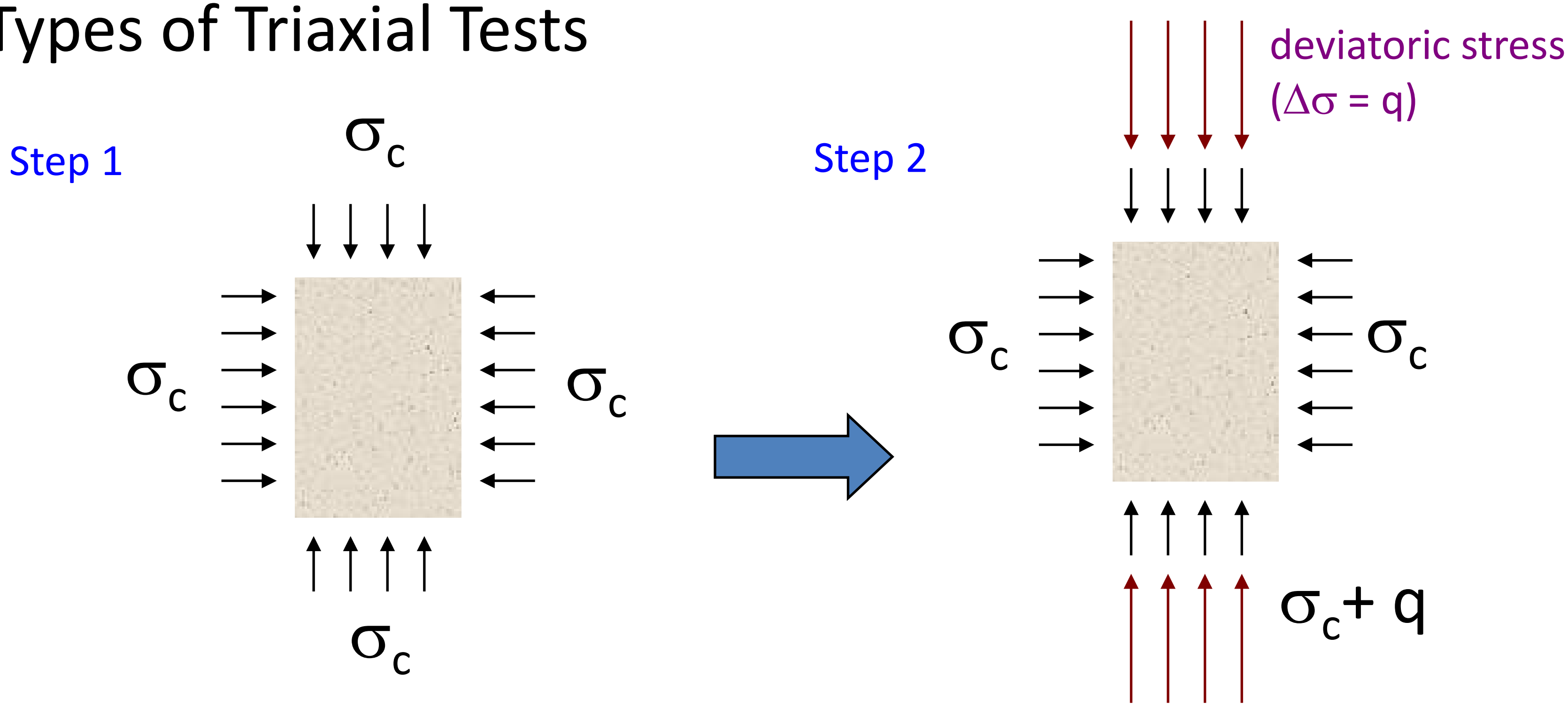
Triaxial Shear Test

Specimen preparation (undisturbed sample)

Proving ring to measure the deviator load

Dial gauge to measure vertical displacement

Types of Triaxial Tests



Under all-around cell pressure σ_c

Shearing (loading)

Is the drainage valve open?

yes → **C**onsolidated sample

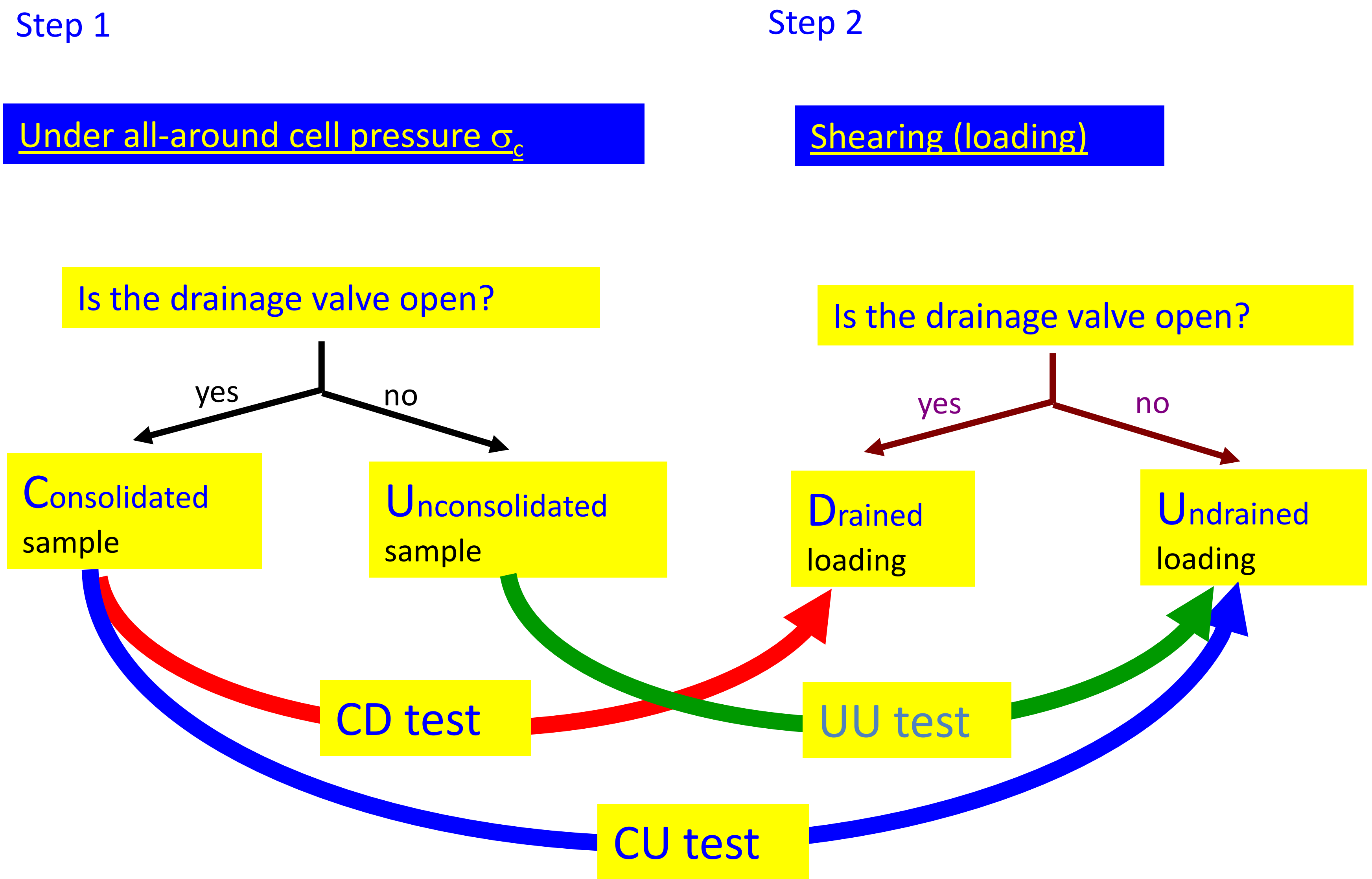
no → **U**nconsolidated sample

Is the drainage valve open?

yes → **D**rained loading

no → **U**ndrained loading

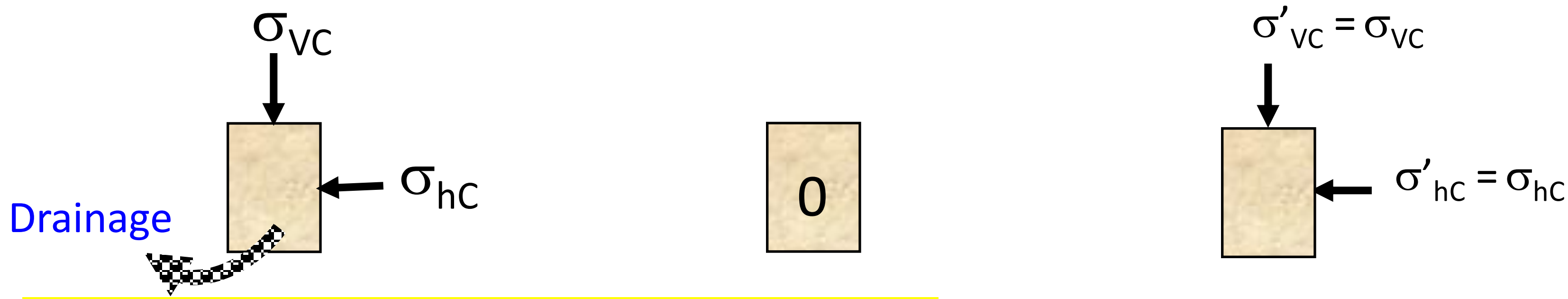
Types of Triaxial Tests



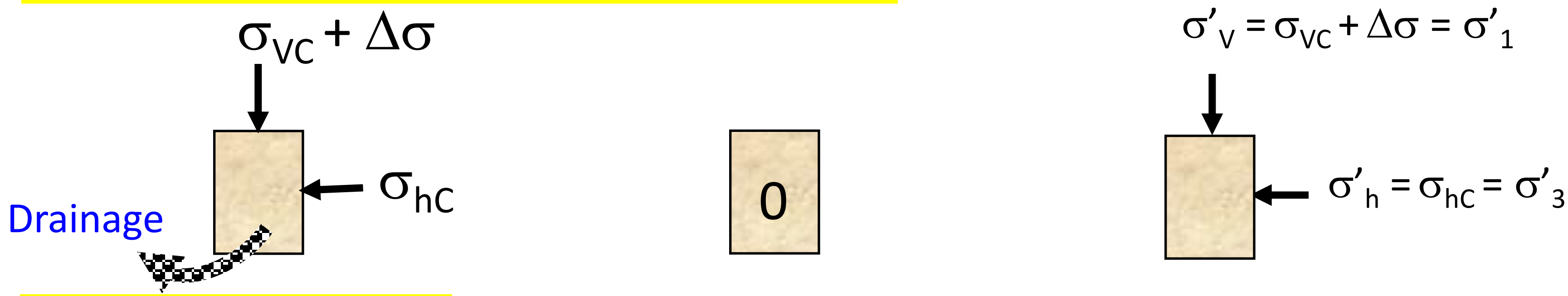
Consolidated- drained test (CD Test)

Total, σ = Neutral, u + Effective, σ'

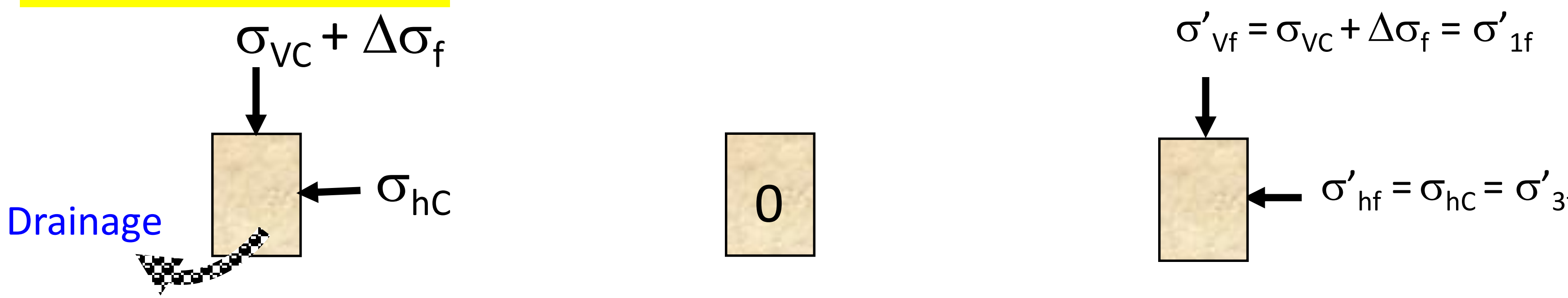
Step 1: At the end of consolidation



Step 2: During axial stress increase

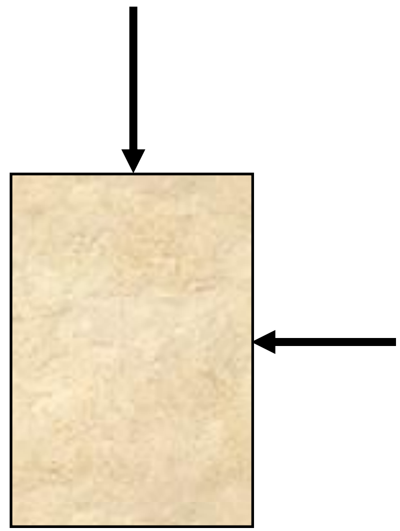


Step 3: At failure



Consolidated- drained test (CD Test)

$$\sigma_1 = \sigma_{VC} + \Delta\sigma$$

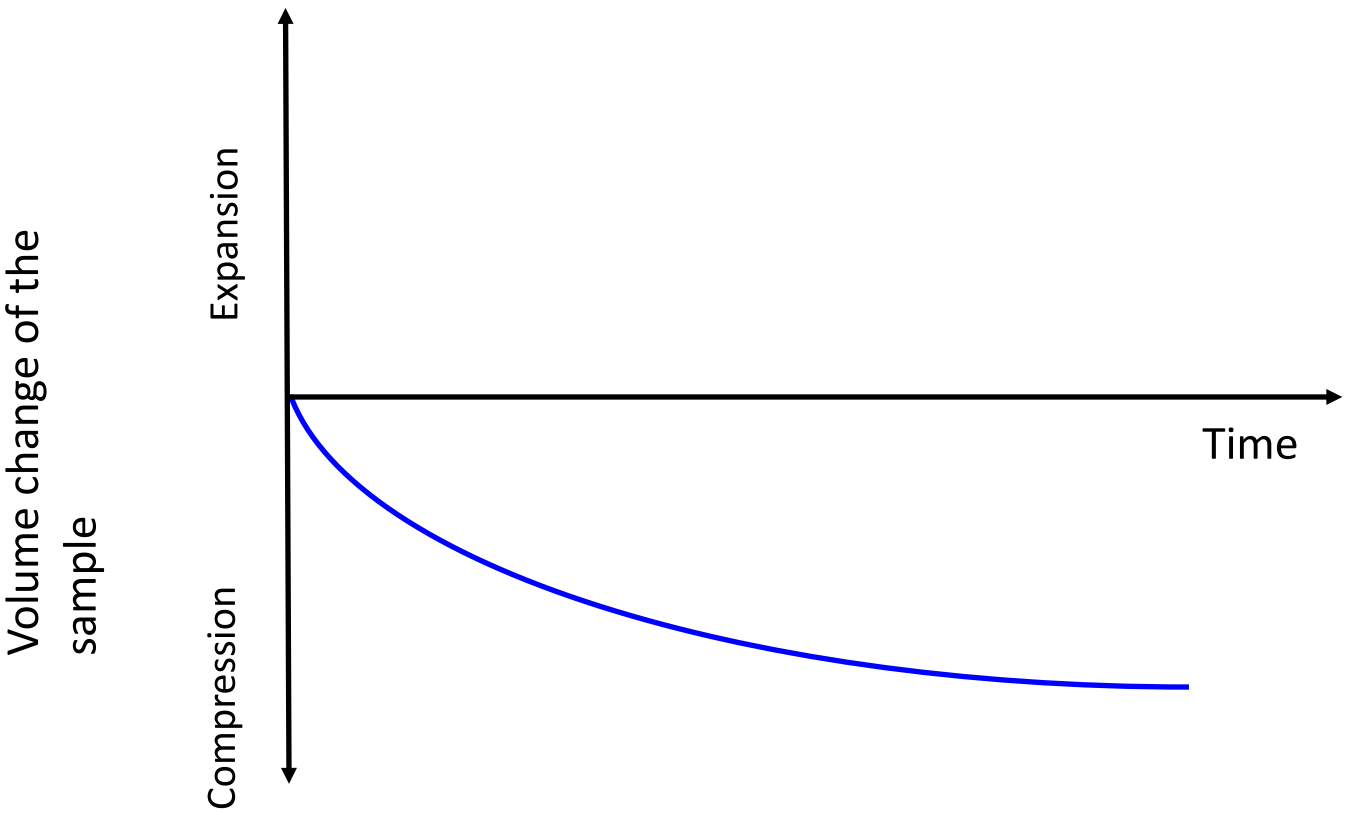


$$\sigma_3 = \sigma_{hC}$$

Deviator stress (q or $\Delta\sigma_d$) = $\sigma_1 - \sigma_3$

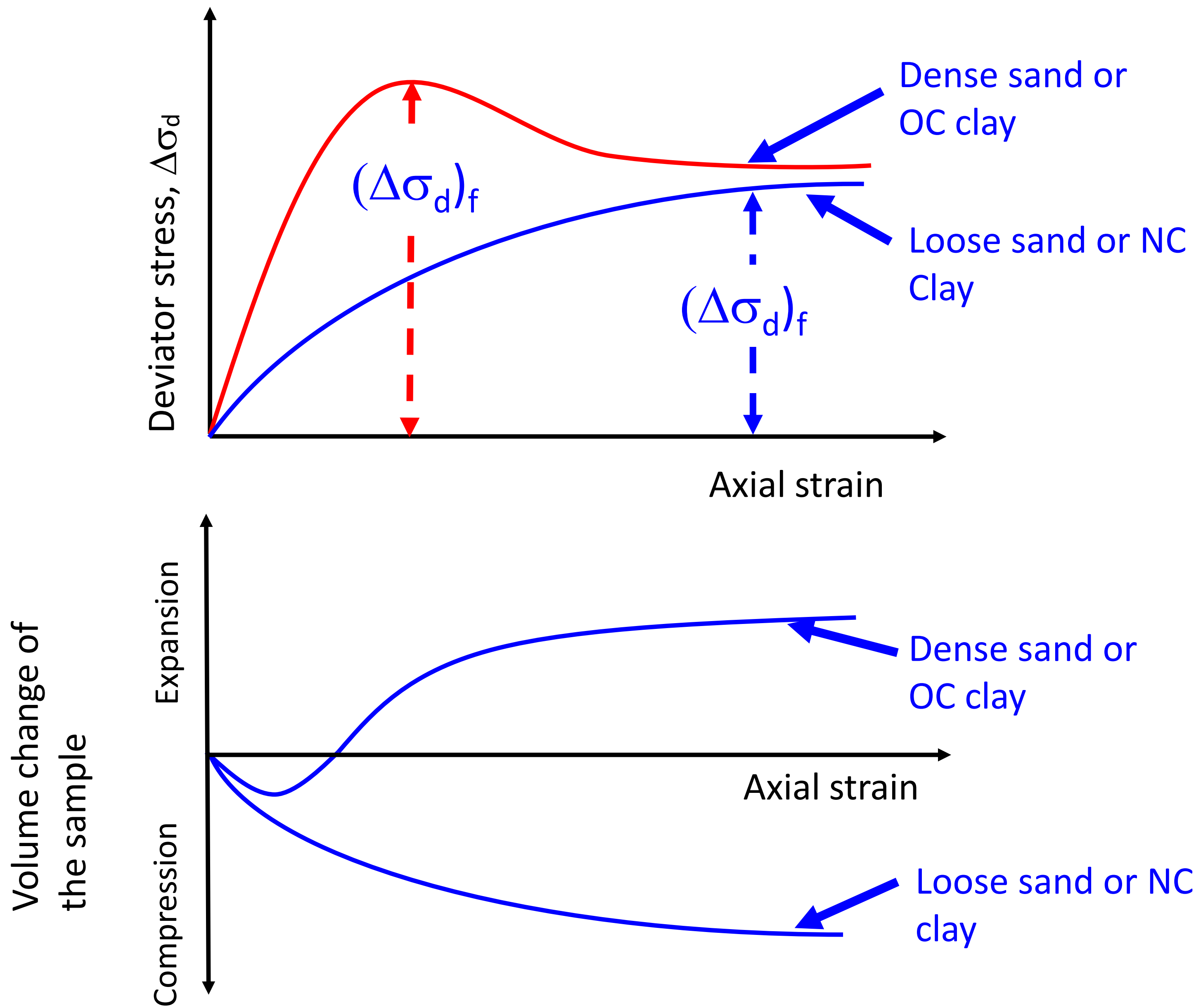
Consolidated- drained test (CD Test)

Volume change of sample during consolidation



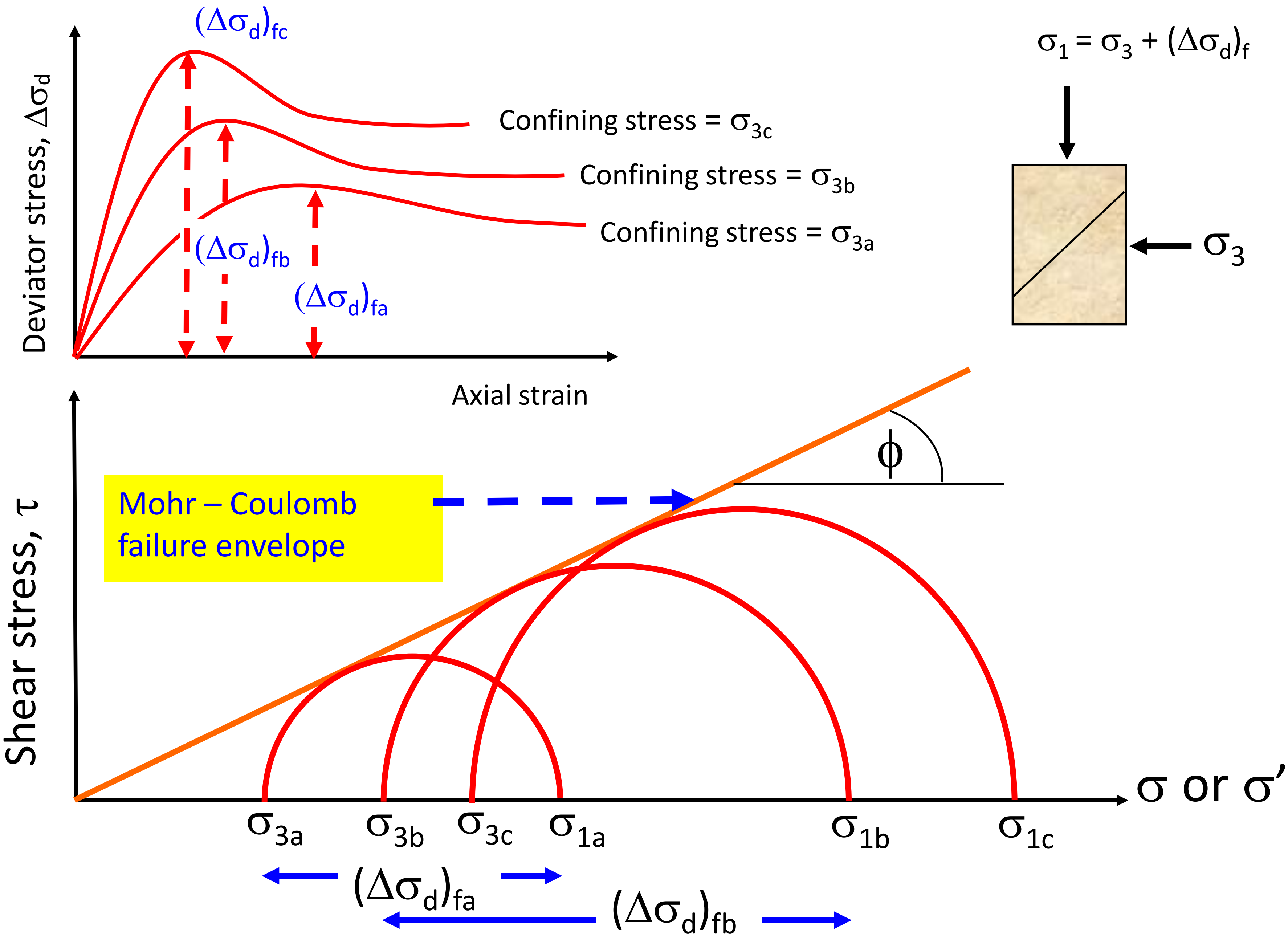
Consolidated- drained test (CD Test)

Stress-strain relationship during shearing



CD tests

How to determine strength parameters c and ϕ



CD tests

Strength parameters c and ϕ obtained from CD tests

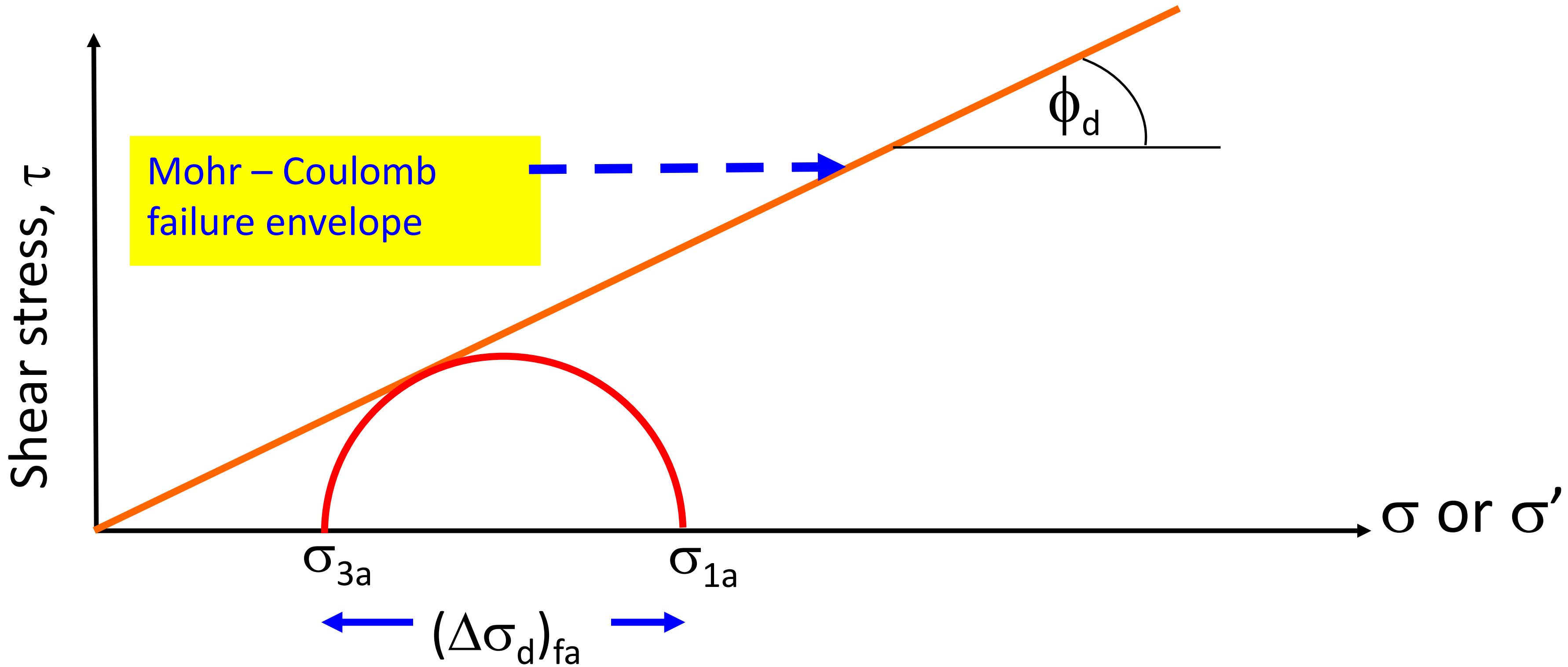
Since u = 0 in CD tests, $\sigma = \sigma'$

Therefore, $c = c'$ and $\phi = \phi'$

c_d and ϕ_d are used to denote them

CD tests   Failure envelopes

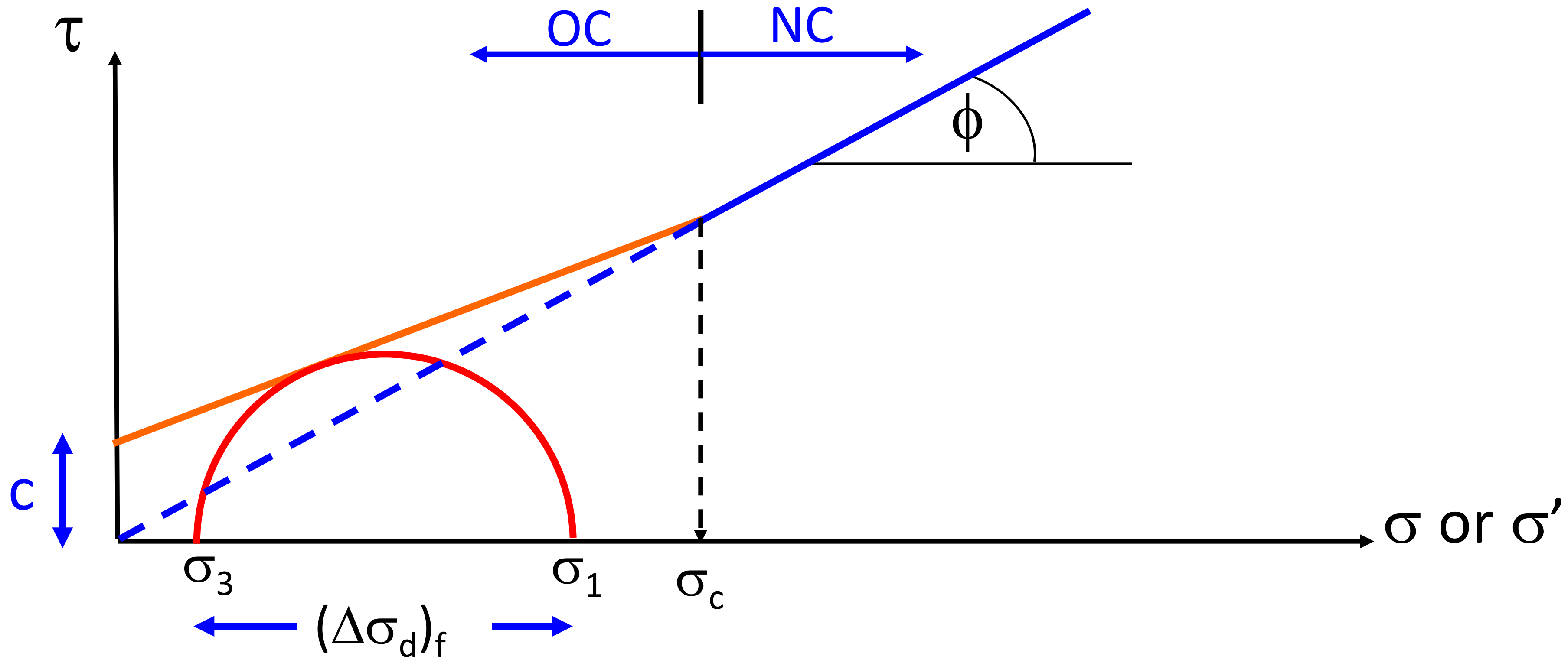
For sand and NC Clay, $c_d = 0$



Therefore, one CD test would be sufficient to determine ϕ_d of sand or NC clay

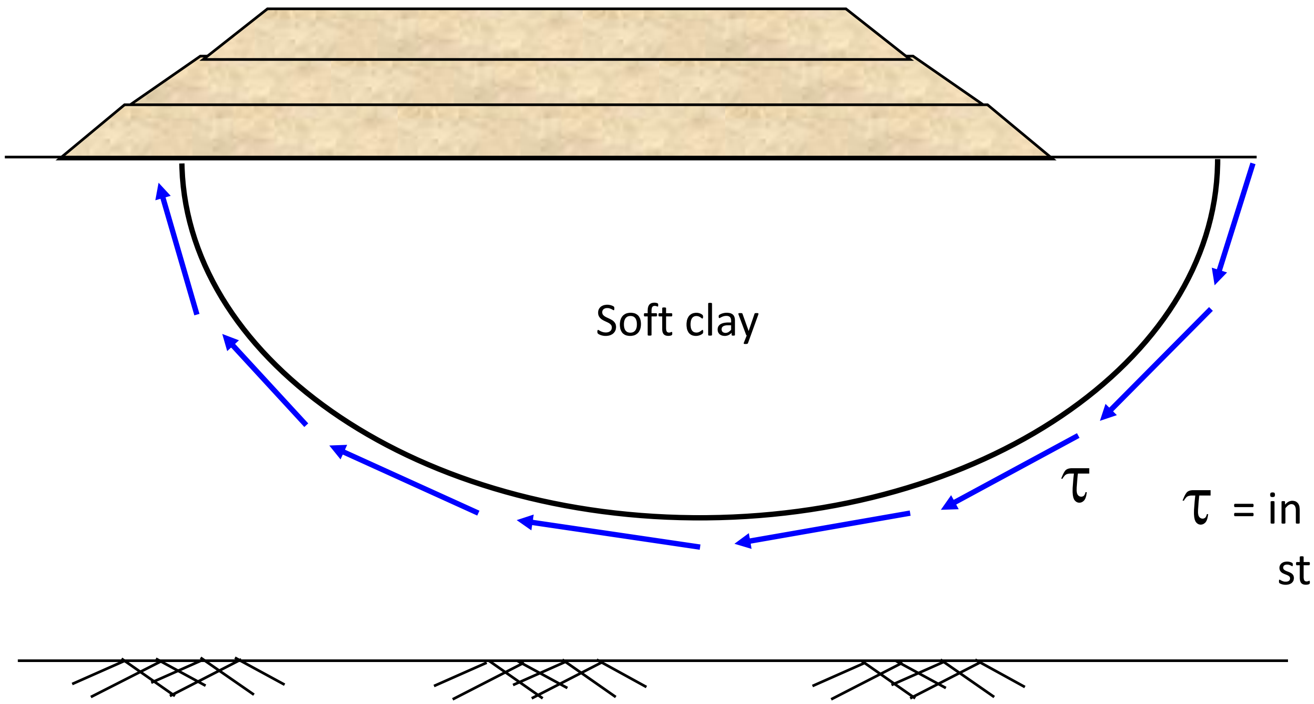
CD tests   Failure envelopes

For OC Clay, $c_d \neq 0$



Some practical applications of CD analysis for clays

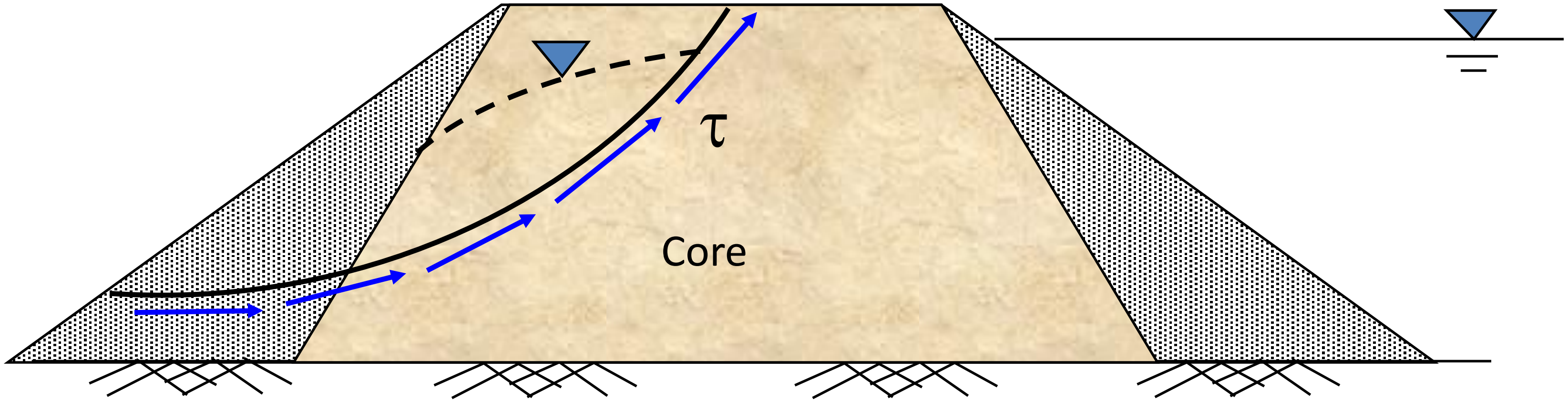
1. Embankment constructed very slowly, in layers over a soft clay deposit



τ = in situ drained shear strength

Some practical applications of CD analysis for clays

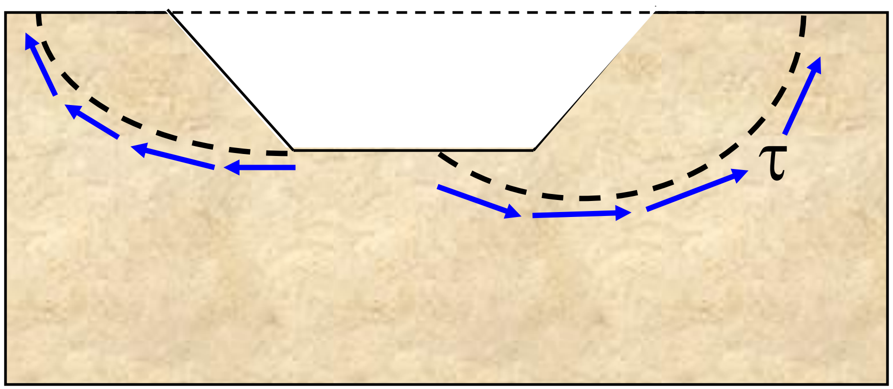
2. Earth dam with steady state seepage

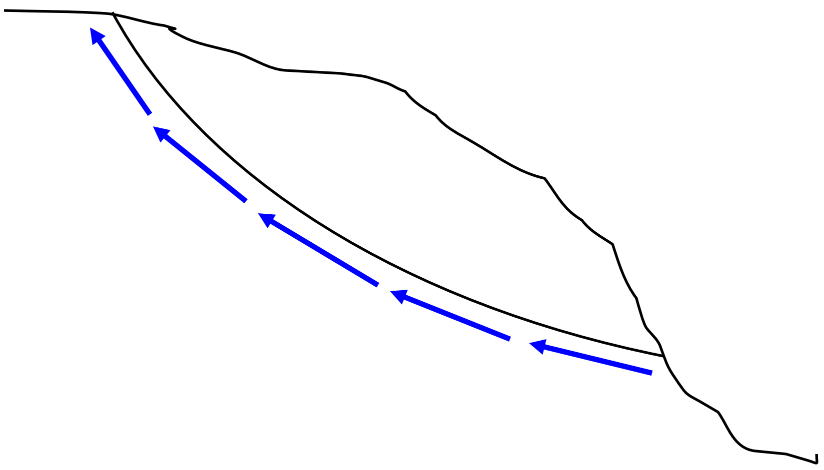


τ = drained shear strength of clay core

Some practical applications of CD analysis for clays

3. Excavation or natural slope in clay





τ = In situ drained shear strength

Note: CD test simulates the <u>long term condition</u> in the field. Thus, c_d and ϕ_d should be used to evaluate the long term behavior of soils

Consolidated- Undrained test (CU Test)

Total, σ = Neutral, u + Effective, σ'

Step 1: At the end of consolidation

σ_{VC}

σ_{hC}

Drainage

0

$\sigma'_{VC} = \sigma_{VC}$

$\sigma'_{hC} = \sigma_{hC}$

Step 2: During axial stress increase

$\sigma_{VC} + \Delta\sigma$

No drainage

σ_{hC}

$\pm\Delta u$

$\sigma'_{V} = \sigma_{VC} + \Delta\sigma \pm \Delta u = \sigma'_{1}$

$\sigma'_{h} = \sigma_{hC} \pm \Delta u = \sigma'_{3}$

Step 3: At failure

$\sigma_{VC} + \Delta\sigma_{f}$

No drainage

σ_{hC}

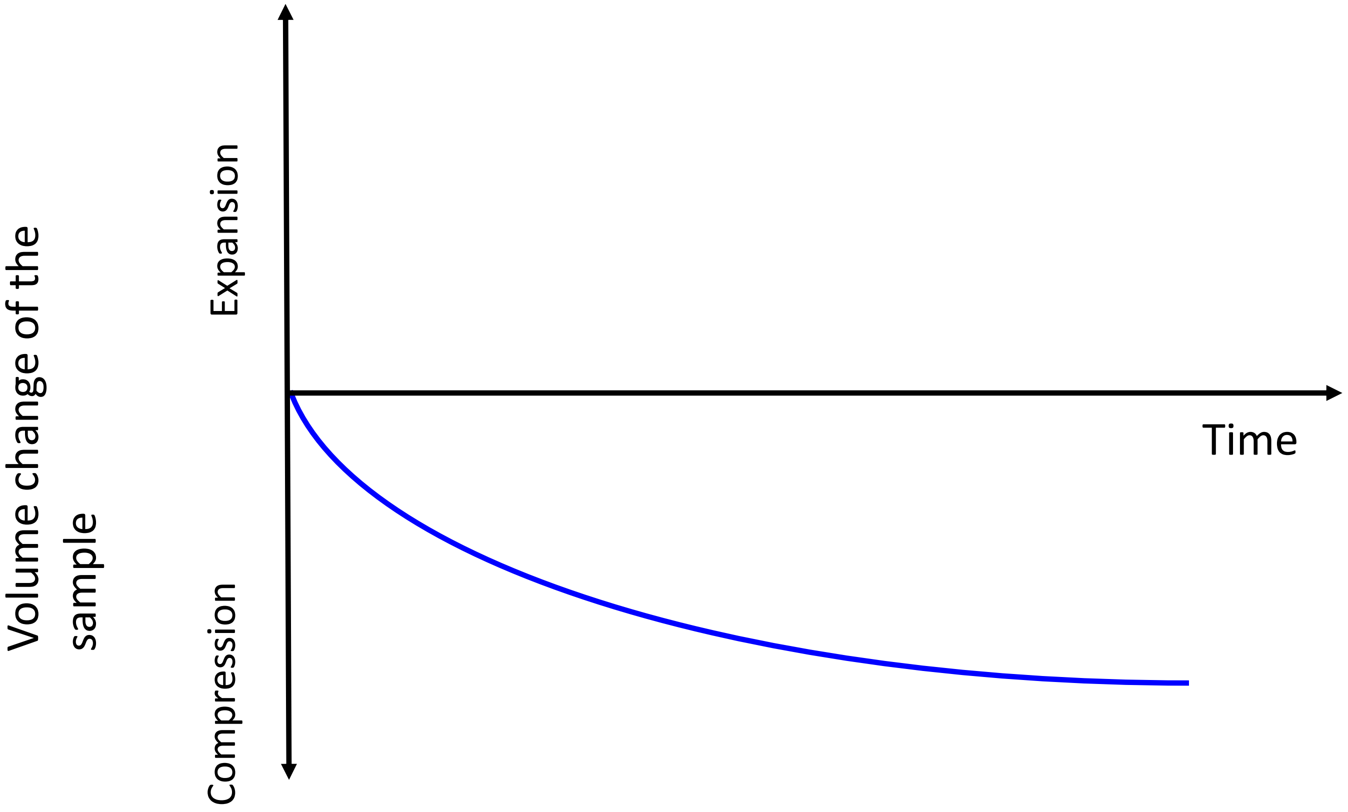
$\pm\Delta u_{f}$

$\sigma'_{Vf} = \sigma_{VC} + \Delta\sigma_{f} \pm \Delta u_{f} = \sigma'_{1f}$

$\sigma'_{hf} = \sigma_{hC} \pm \Delta u_{f} = \sigma'_{3f}$

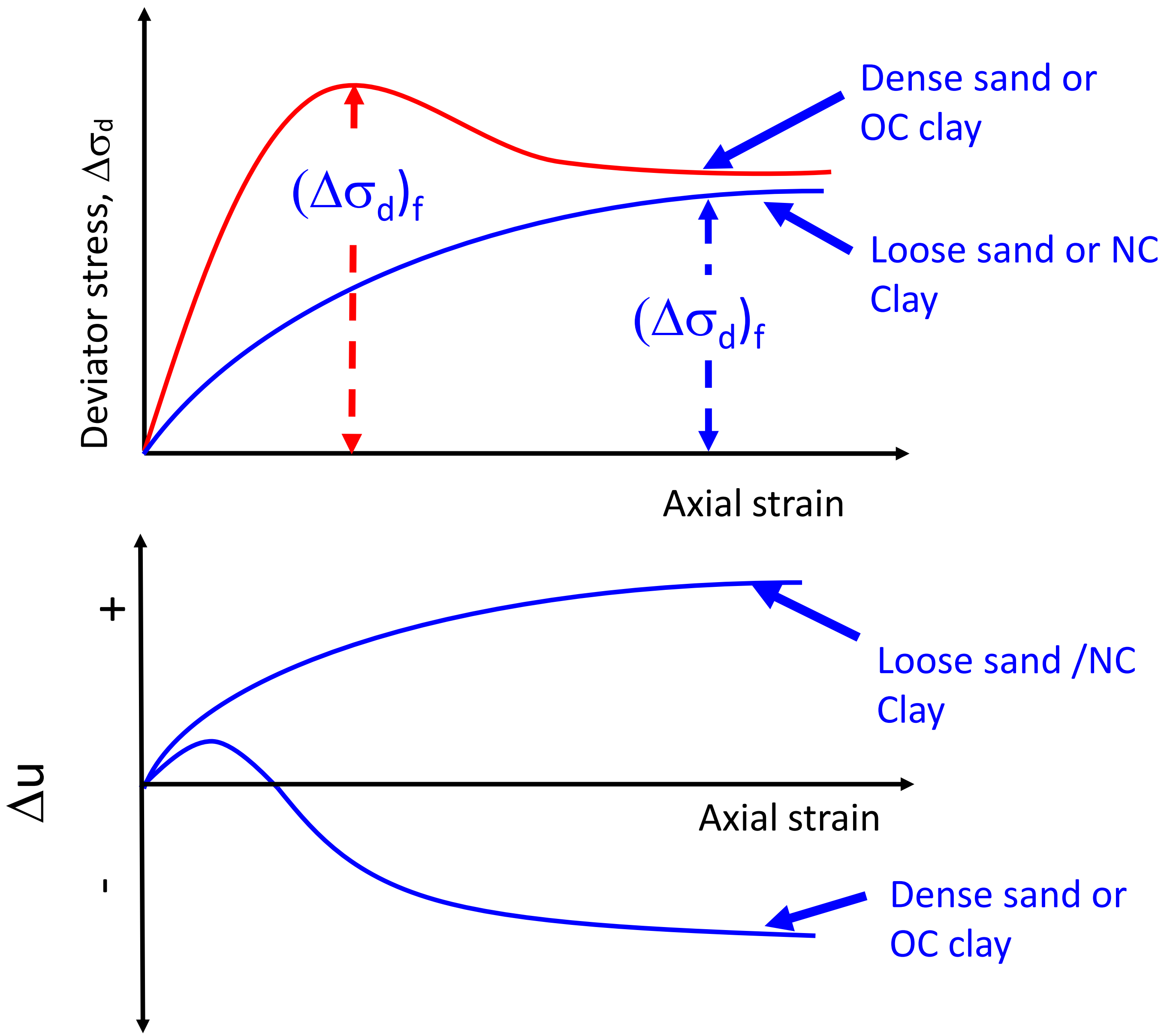
Consolidated- Undrained test (CU Test)

Volume change of sample during consolidation



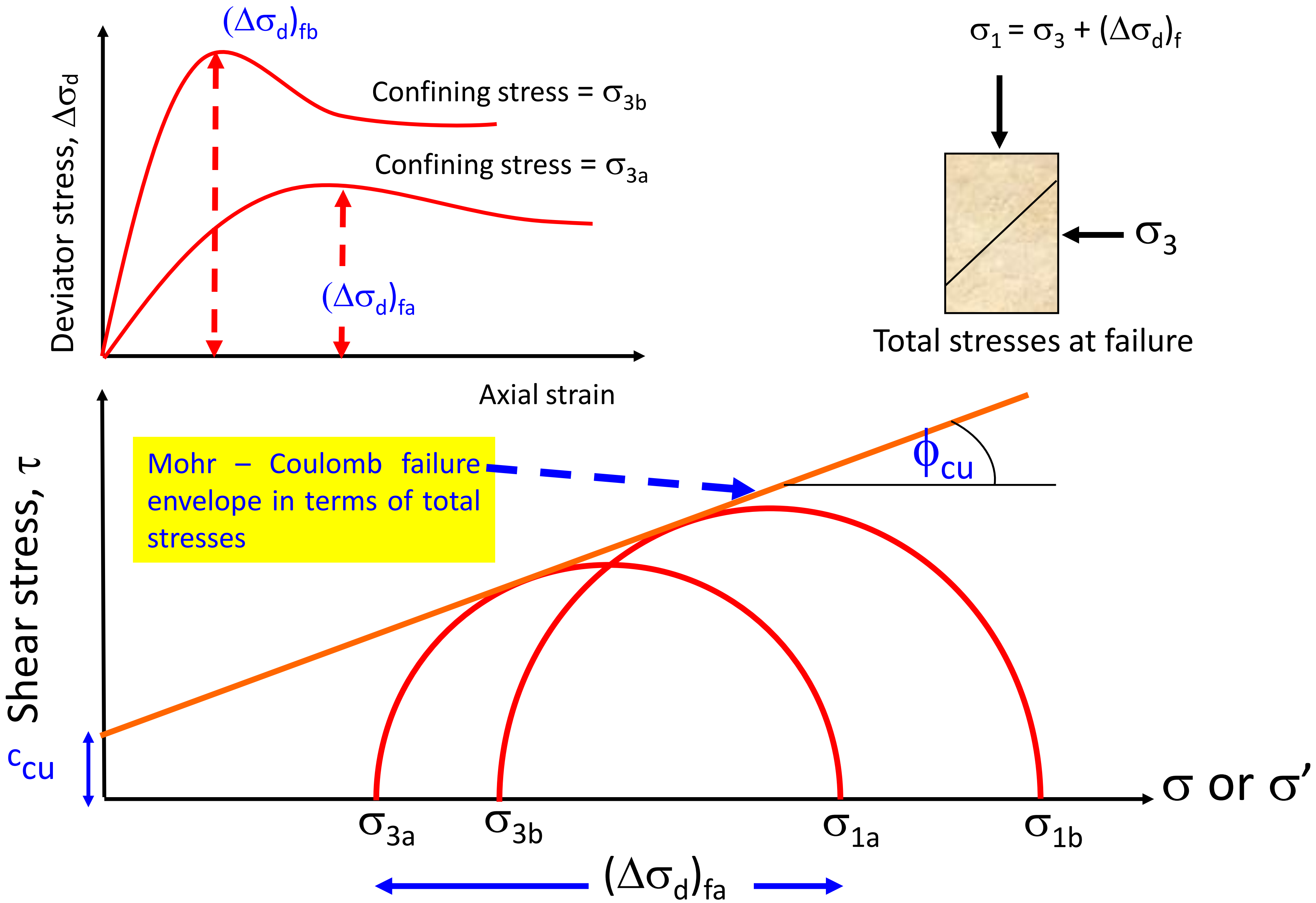
Consolidated- Undrained test (CU Test)

Stress-strain relationship during shearing



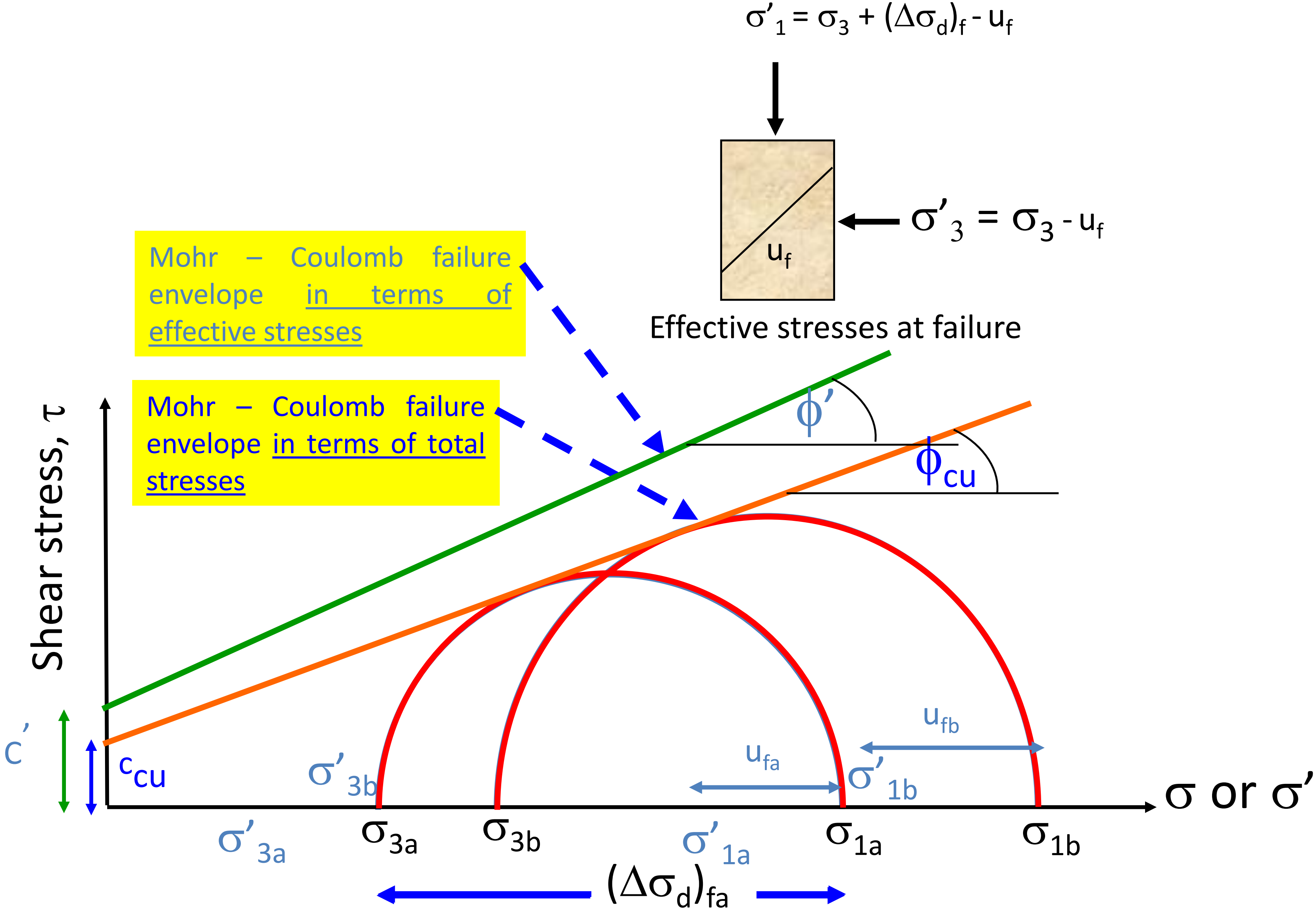
CU tests

How to determine strength parameters c and ϕ



CU tests

How to determine strength parameters c and ϕ



CU tests

Strength parameters c and ϕ obtained from CD tests

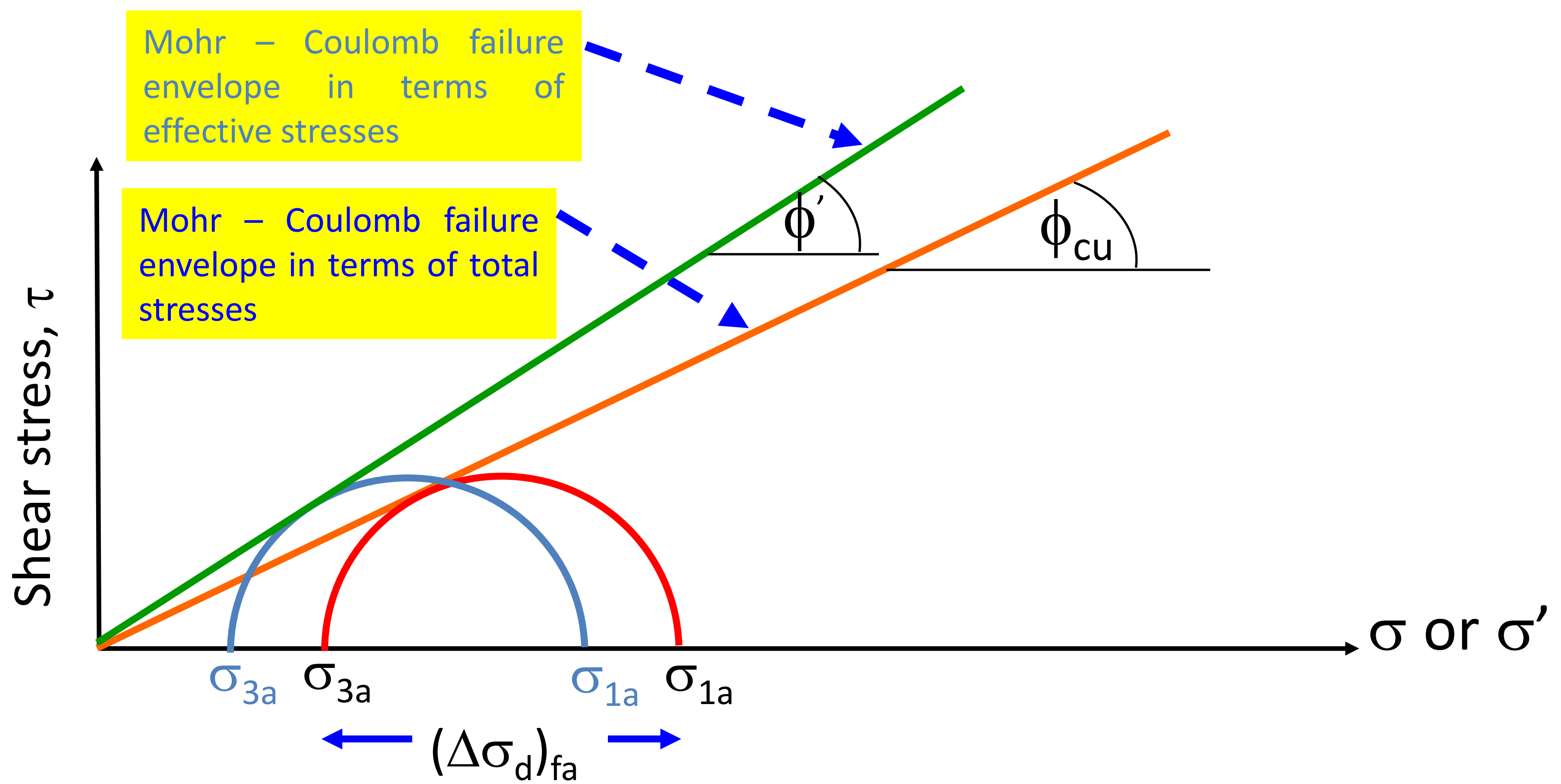
Shear strength parameters in terms of total stresses are c_{cu} and ϕ_{cu}

Shear strength parameters in terms of effective stresses are c' and ϕ'

$c' = c_d$ and $\phi' = \phi_d$

CU tests   Failure envelopes

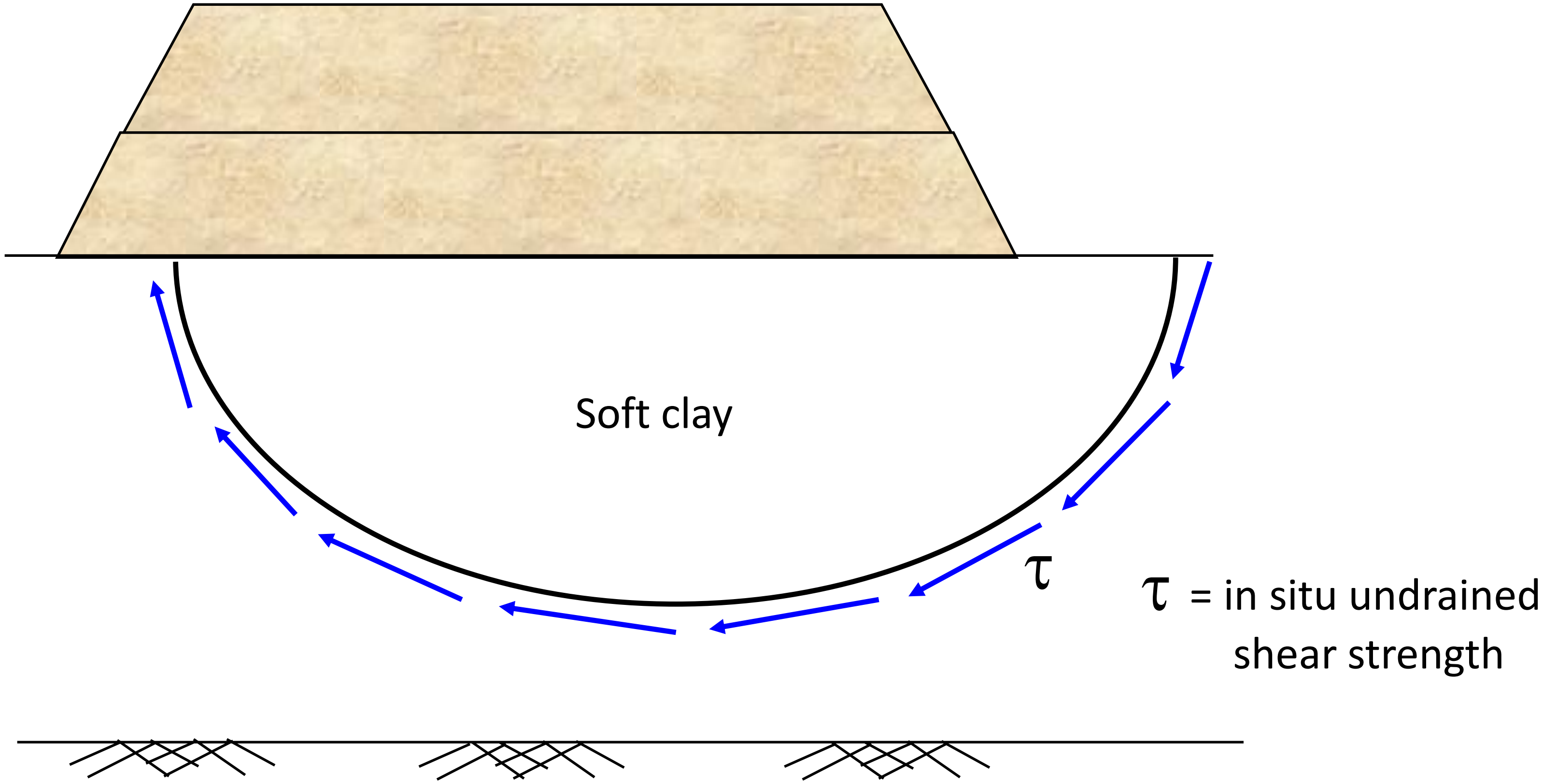
For sand and NC Clay, c_{cu} and $c' = 0$



Therefore, one CU test would be sufficient to determine ϕ_{cu} and $\phi'(= \phi_d)$ of sand or NC clay

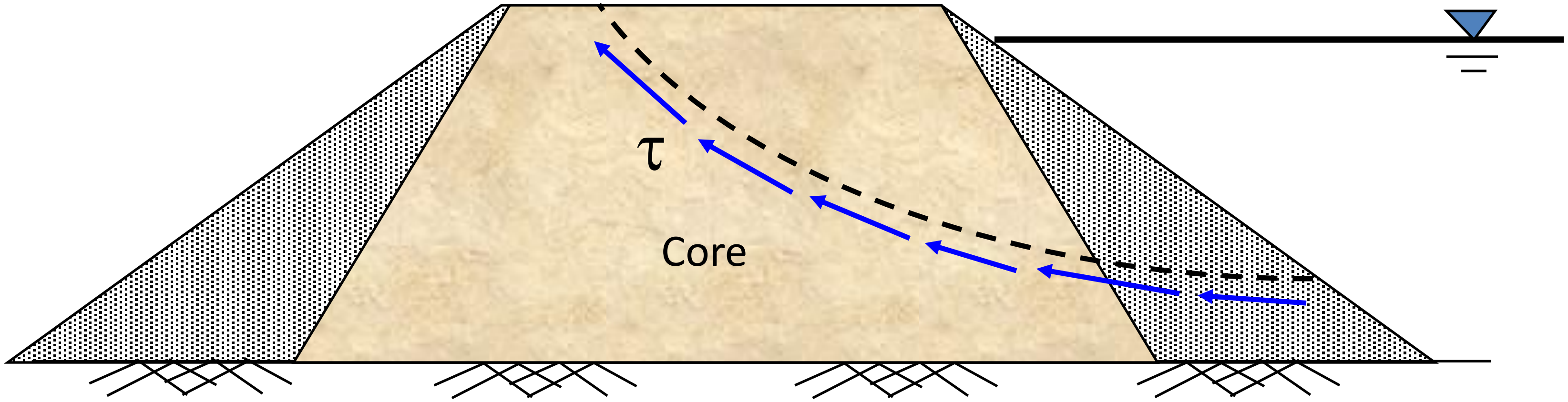
Some practical applications of CU analysis for clays

1. Embankment constructed rapidly over a soft clay deposit



Some practical applications of CU analysis for clays

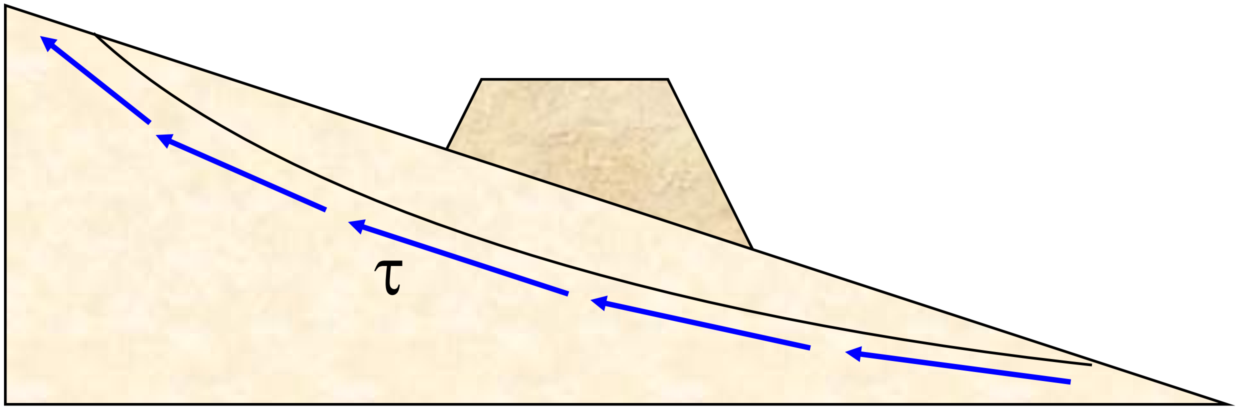
2. Rapid drawdown behind an earth dam



τ = Undrained shear strength of clay core

Some practical applications of CU analysis for clays

3. Rapid construction of an embankment on a natural slope



τ = In situ undrained shear strength

Note: Total stress parameters from CU test (c_{cu} and ϕ_{cu}) can be used for stability problems where,

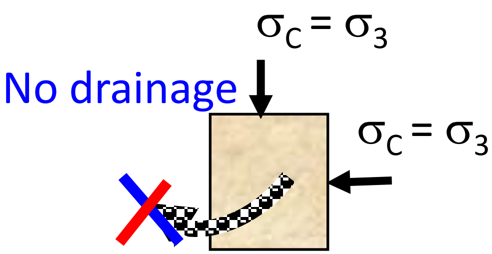
Soil have become fully consolidated and are at equilibrium with the existing stress state; Then for some reason additional stresses are applied quickly with no drainage occurring

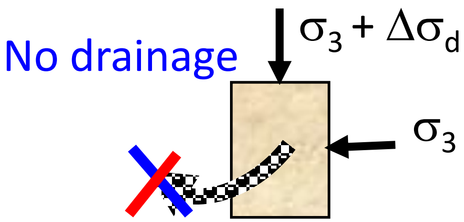
Unconsolidated- Undrained test (UU Test)

Data analysis

Initial specimen condition

Specimen condition during shearing





Initial volume of the sample = $A_0 \times H_0$

Volume of the sample during shearing = $A \times H$

Since the test is conducted under undrained condition,

$A \times H = A_0 \times H_0$

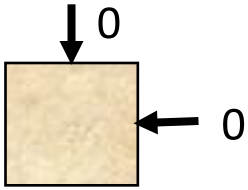
$A \times (H_0 - \Delta H) = A_0 \times H_0$

$A \times (1 - \Delta H/H_0) = A_0$

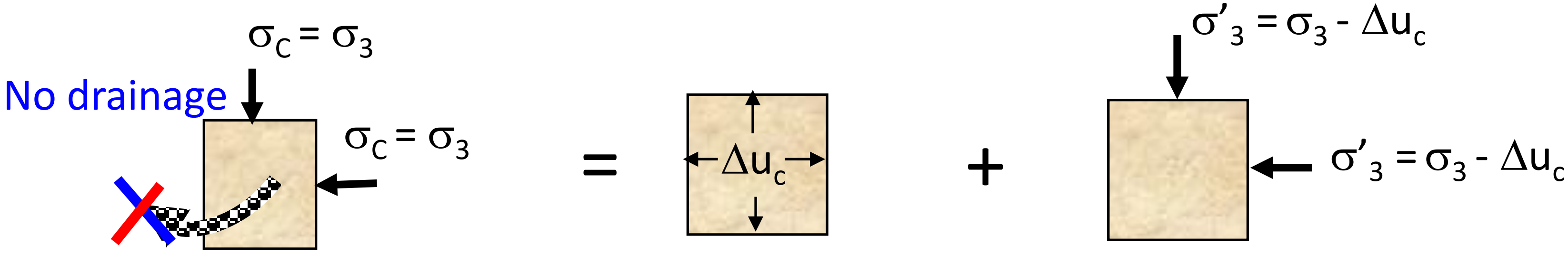
$$A = \frac{A_0}{1 - \varepsilon_z}$$

Unconsolidated- Undrained test (UU Test)

Step 1: Immediately after sampling



Step 2: After application of hydrostatic cell pressure



$$\Delta u_c = B\, \Delta\sigma_3$$

Increase of pwp due to increase of cell pressure

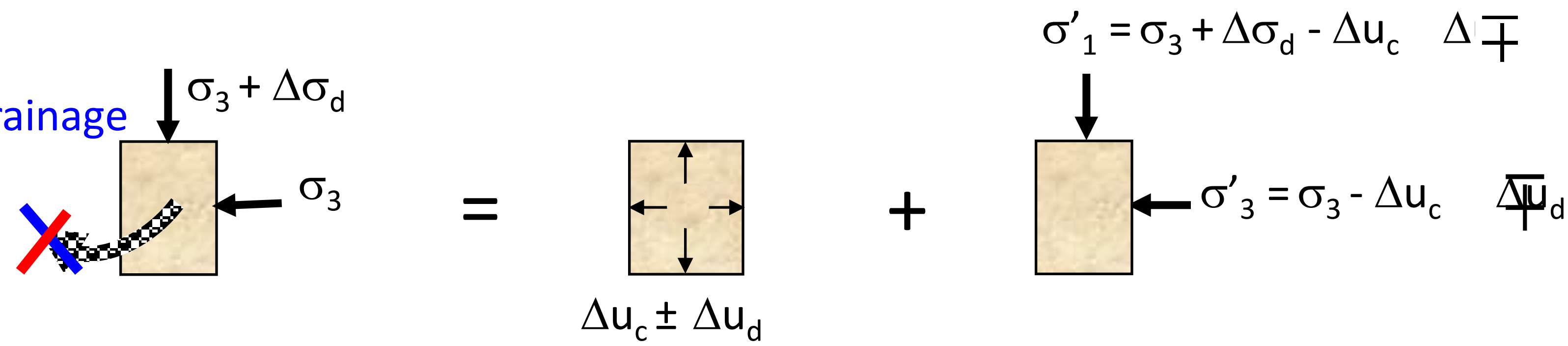
Skempton's pore water pressure parameter, B

Increase of cell pressure

Note: If soil is fully saturated, then B = 1 (hence, $\Delta u_c = \Delta\sigma_3$)

Unconsolidated- Undrained test (UU Test)

Step 3: During application of axial load



$$\Delta u_d = A\Delta\sigma_d$$

Increase of pwp due to increase of deviator stress

Increase of deviator stress

Skempton's pore water pressure parameter, A

Unconsolidated- Undrained test (UU Test)

Combining steps 2 and 3,

$$\Delta u_c = B\,\Delta\sigma_3$$

$$\Delta u_d = A\Delta\sigma_d$$

Total pore water pressure <u>increment</u> at any stage, Δu

$$\Delta u = \Delta u_c + \Delta u_d$$

$$\Delta u = B\,\Delta\sigma_3 + A\Delta\sigma_d$$

$$\Delta u = B\,\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)$$

Skempton's pore water pressure equation

Unconsolidated- Undrained test (UU Test)

Total, σ = Neutral, u + Effective, σ'

Step 1: Immediately after sampling

Total: 0 (vertical), 0 (horizontal)

Neutral: $-u_r$

Effective: $\sigma'_{V0} = u_r$, $\sigma'_{h0} = u_r$

Step 2: After application of hydrostatic cell pressure

No drainage

Total: σ_C (vertical), σ_C (horizontal)

Neutral: $-u_r + \Delta u_c = -u_r + \sigma_c$

$(S_r = 100\% ; B = 1)$

Effective: $\sigma'_{VC} = \sigma_C + u_r - \sigma_C = u_r$, $\sigma'_h = u_r$

Step 3: During application of axial load

No drainage

Total: $\sigma_C + \Delta\sigma$ (vertical), σ_C (horizontal)

Neutral: $-u_r + \sigma_c \pm \Delta u$

Effective: $\sigma'_V = \sigma_C + \Delta\sigma + u_r - \sigma_c \mp \Delta u$

$\sigma'_h = \sigma_C + u_r - \sigma_c \mp \Delta u$

Step 3: At failure

No drainage

Total: $\sigma_C + \Delta\sigma_f$ (vertical), σ_C (horizontal)

Neutral: $-u_r + \sigma_c \pm \Delta u_f$

Effective: $\sigma'_{Vf} = \sigma_C + \Delta\sigma_f + u_r - \sigma_c \mp \Delta u = \sigma'_{1f}$

$\sigma'_{hf} = \sigma_C + u_r - \sigma_c \mp \Delta u = \sigma'_{3f}$

Unconsolidated- Undrained test (UU Test)

Total, σ = Neutral, u + Effective, σ'

Step 3: At failure

No drainage

$\sigma_C + \Delta\sigma_f$

σ_C

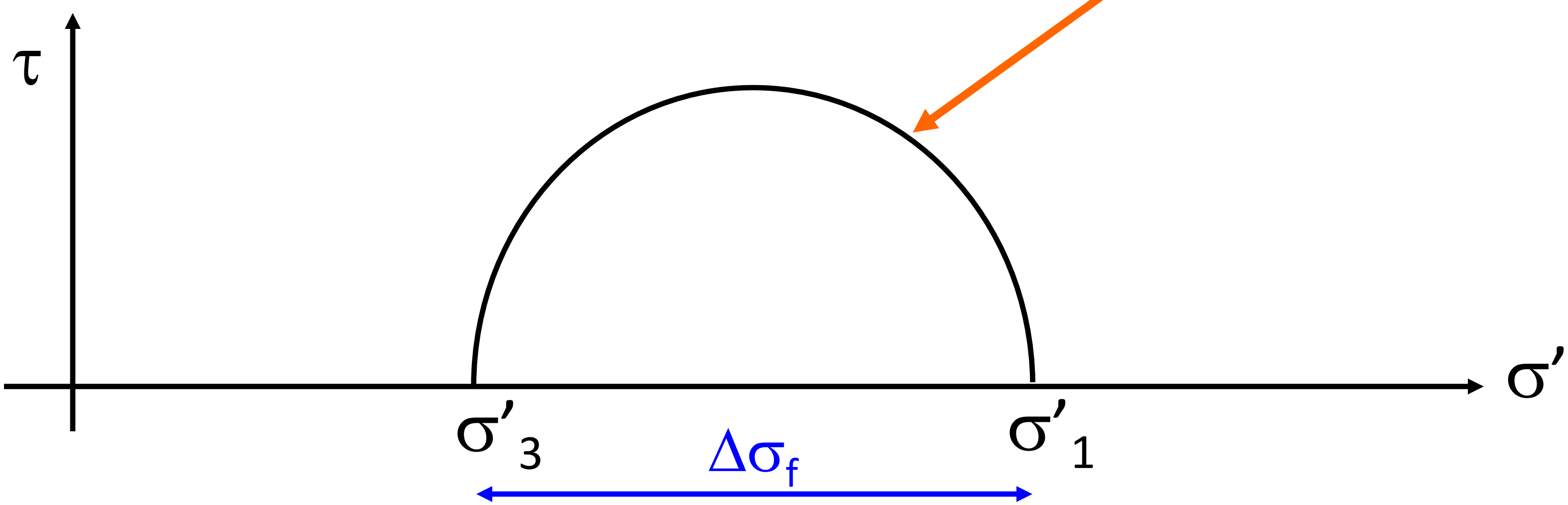
$-u_r + \sigma_c \pm \Delta u_f$

$\sigma'_{Vf} = \cancel{\sigma_c} + \Delta\sigma_f + u_r - \sigma_c \quad \cancel{\Delta u_f} \mp \sigma'_{1f}$

$\sigma'_{hf} = \cancel{\sigma_c} + u_r - \sigma_c \quad \cancel{\Delta u_f} \mp = \sigma'_{3f}$

Mohr circle in terms of effective stresses do not depend on the cell pressure.

Therefore, we get only one Mohr circle in terms of effective stress for different cell pressures



Unconsolidated- Undrained test (UU Test)

Total, σ = Neutral, u + Effective, σ'

Step 3: At failure

No drainage

$\sigma_C + \Delta\sigma_f$

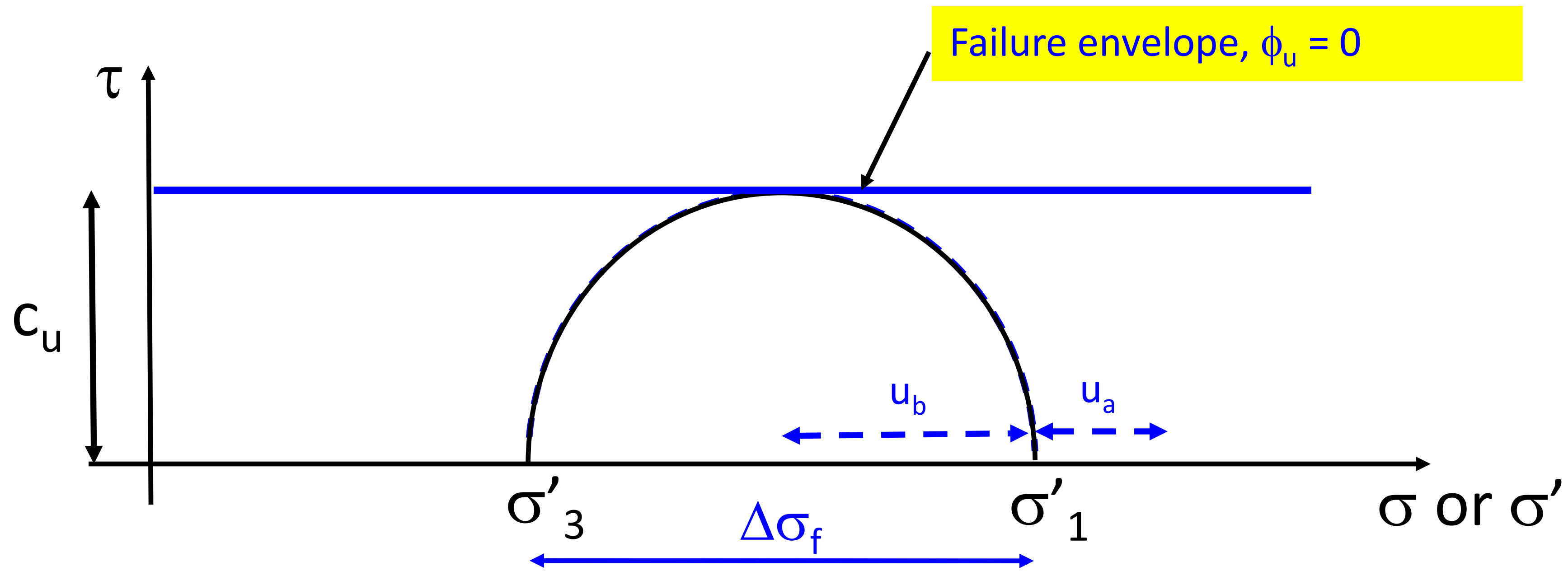
σ_C

$-u_r + \sigma_c \pm \Delta u_f$

$\sigma'_{Vf} = \cancel{\sigma_c} + \Delta\sigma_f + u_r - \sigma_c \quad \cancel{\Delta u} \mp \sigma'_{1f}$

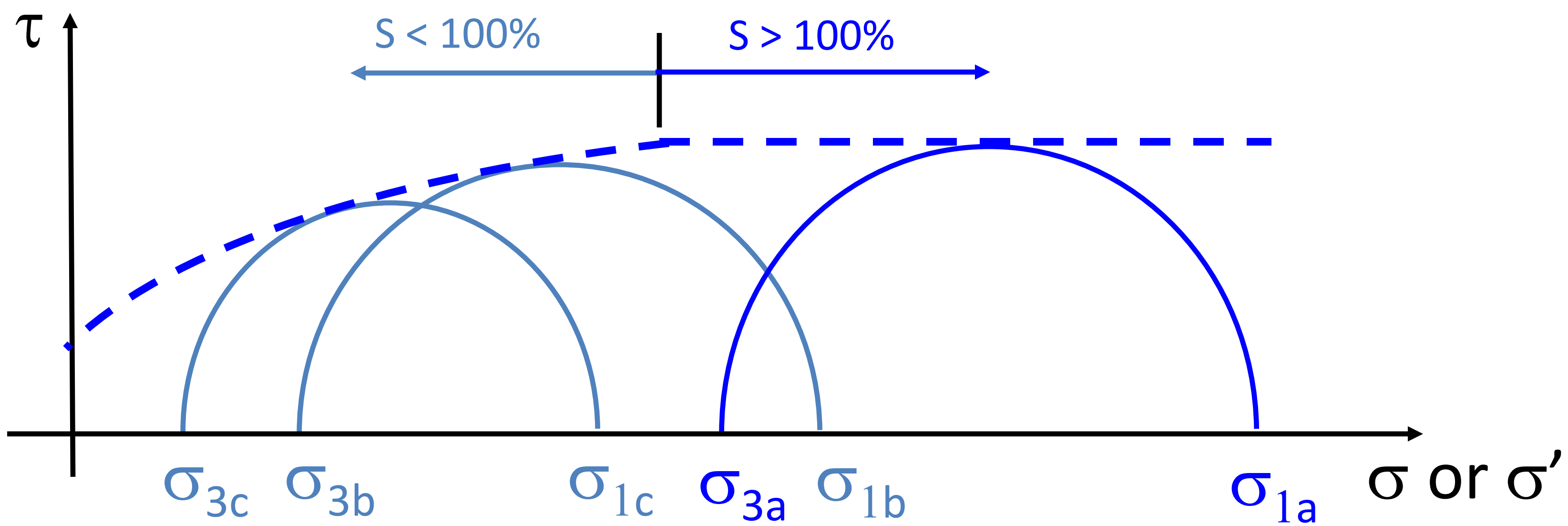
$\sigma'_{hf} = \cancel{\sigma_c} + u_r - \sigma_c \quad \cancel{\Delta u} \mp = \sigma'_{3f}$

Mohr circles in terms of total stresses



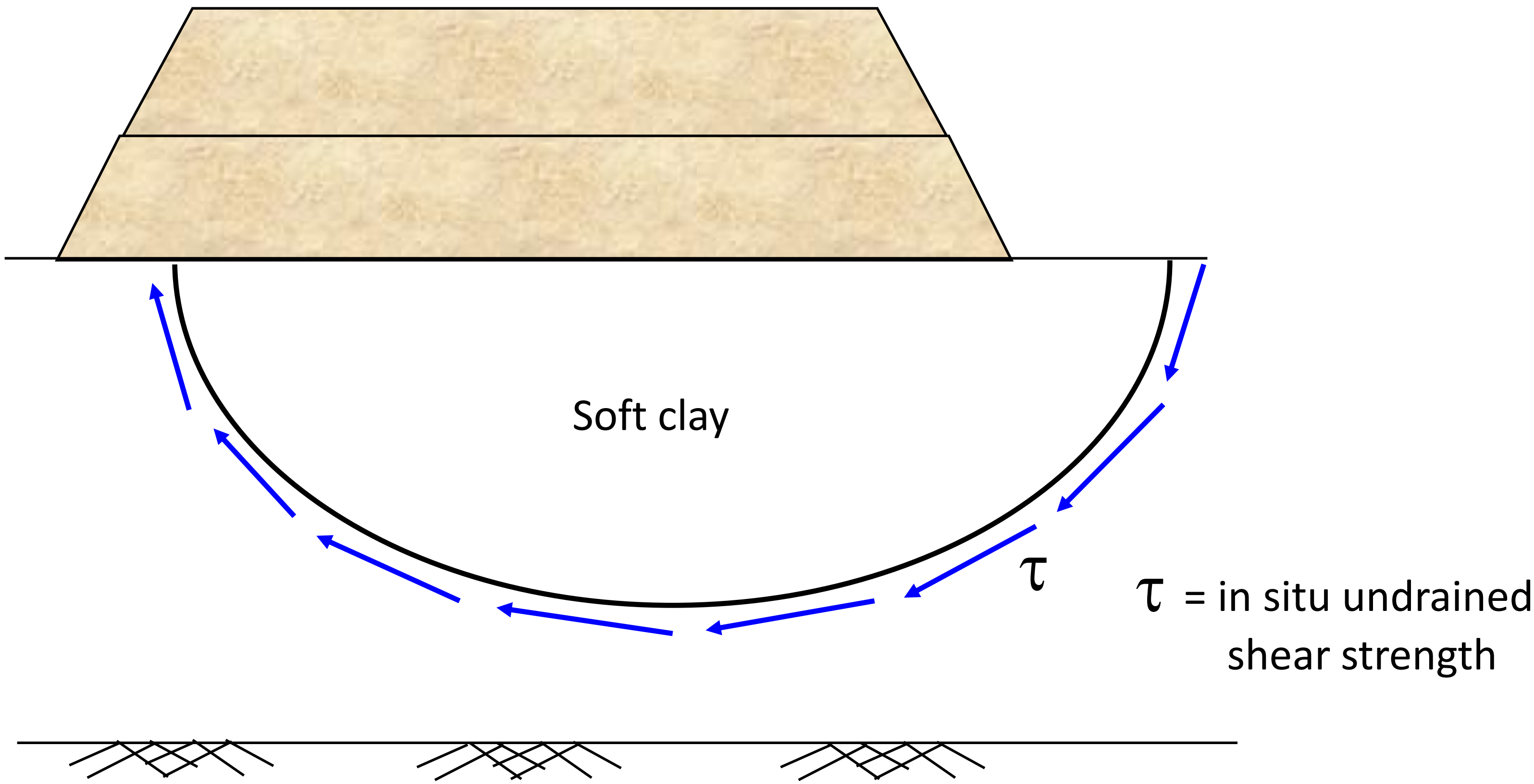
Unconsolidated- Undrained test (UU Test)

Effect of degree of saturation on failure envelope



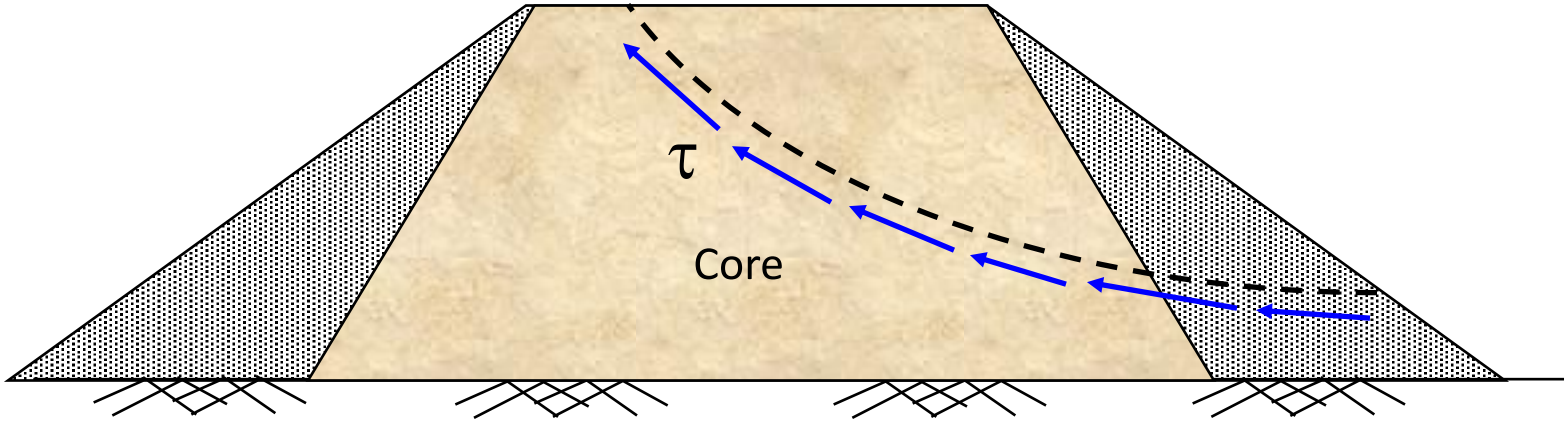
Some practical applications of UU analysis for clays

1. Embankment constructed rapidly over a soft clay deposit



Some practical applications of UU analysis for clays

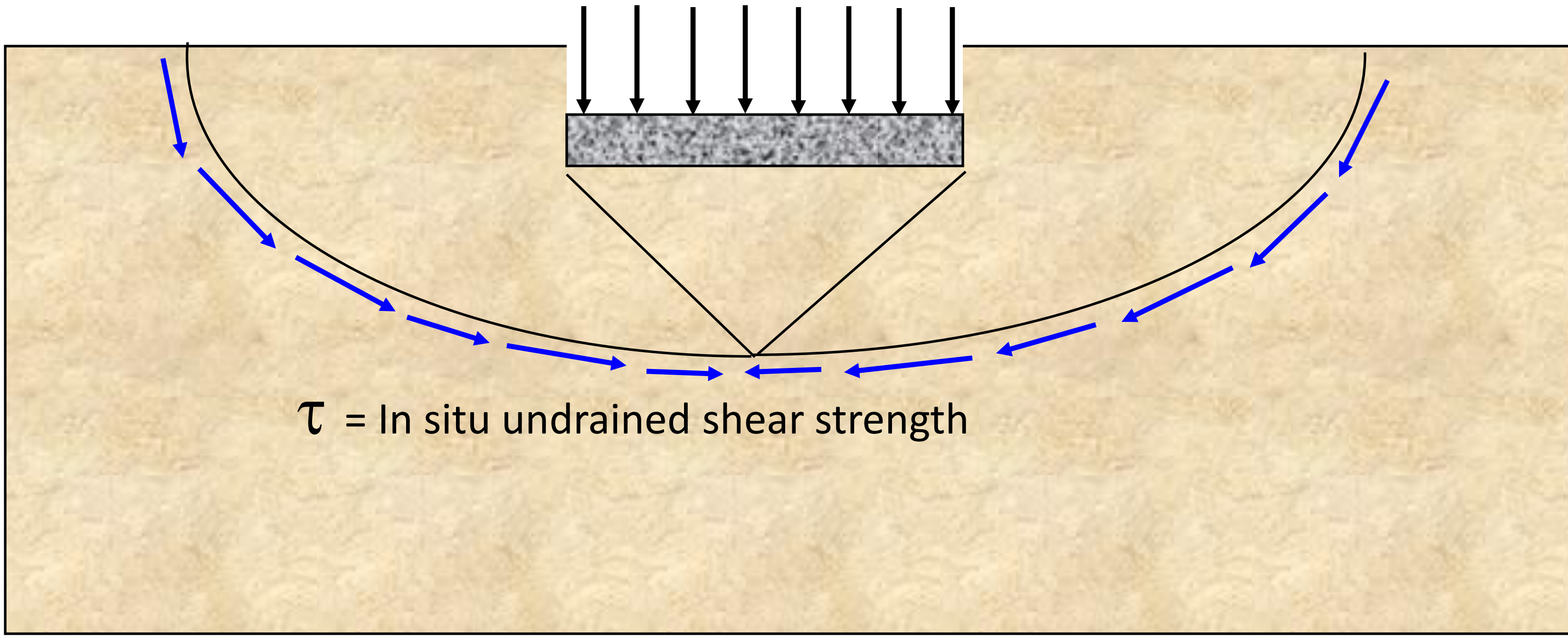
2. Large earth dam constructed rapidly with no change in water content of soft clay



τ = Undrained shear strength of clay core

Some practical applications of UU analysis for clays

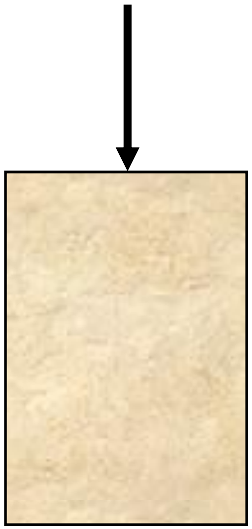
3. Footing placed rapidly on clay deposit



Note: UU test simulates the <u>short term condition</u> in the field. Thus, c_u can be used to analyze the short term behavior of soils

Unconfined Compression Test (UC Test)

$$\sigma_1 = \sigma_{VC} + \Delta\sigma$$

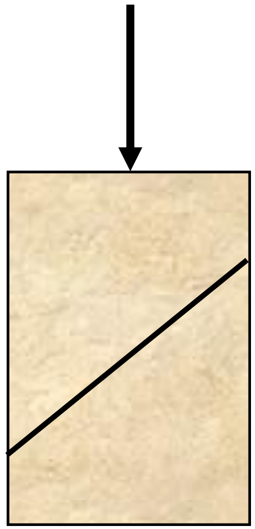


$$\sigma_3 = 0$$

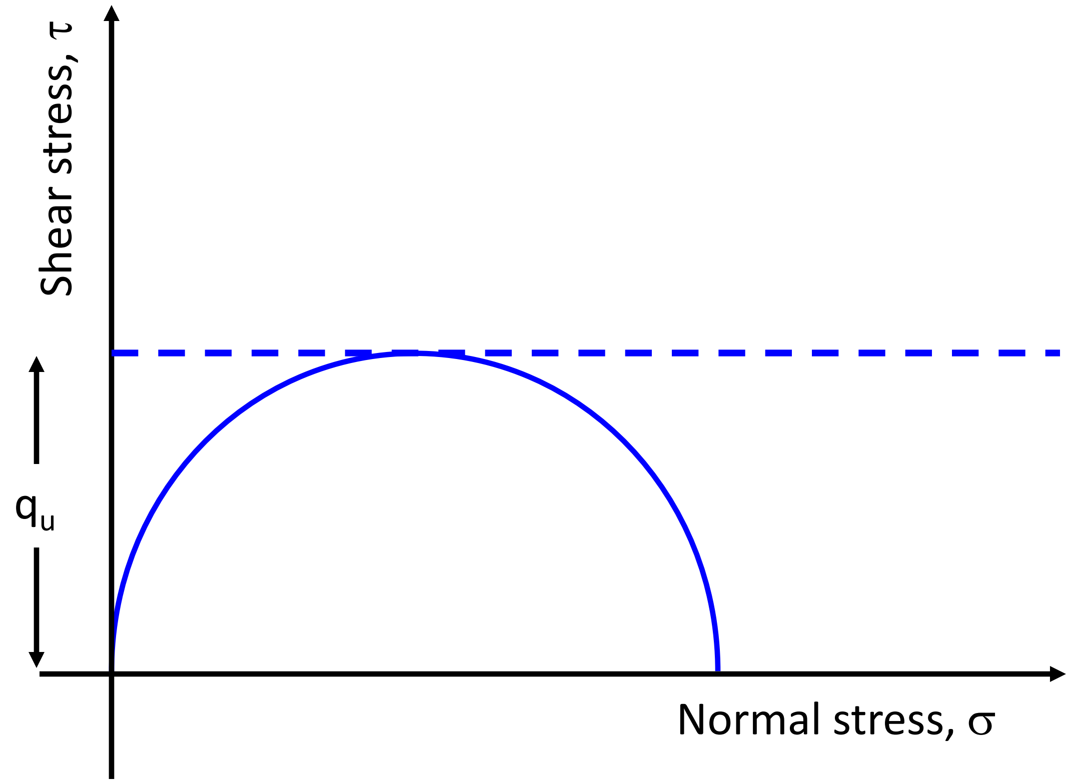
Confining pressure is zero in the UC test

Unconfined Compression Test (UC Test)

$\sigma_1 = \sigma_{VC} + \Delta\sigma f$



$\sigma_3 = 0$



$\tau_f = \sigma_1/2 = q_u/2 = c_u$

THE END