INSTITUTE OF AERONAUTICAL ENGINEERING
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GEOTECHNICAL ENGINEERING

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B. Tech.Course: IV Semester.

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UNIT I

INTRODUCTION AND INDEX PROPERTIES OF SOILS

Types of Soils

(1) **Glacial soils**: formed by transportation and deposition of glaciers.

(2) **Alluvial soils**: transported by running water and deposited along streams.

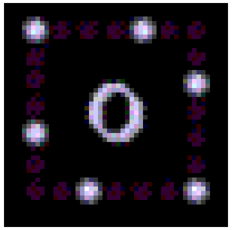
(3) **Lacustrine soils**: formed by deposition in quiet lakes (e.g. soils in Taipei basin).

(4) **Marine soils**: formed by deposition in the seas (Hong Kong).

(5) **Aeolian soils**: transported and deposited by the wind (e.g. soils in the loess plateau, China).

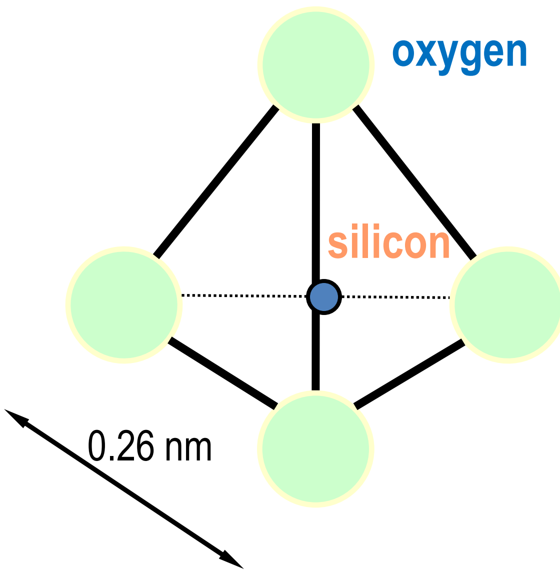
(6) **Colluvial soils**: formed by movement of soil from its original place by gravity, such as during landslide (*Hong Kong*). (from Das, 1998)

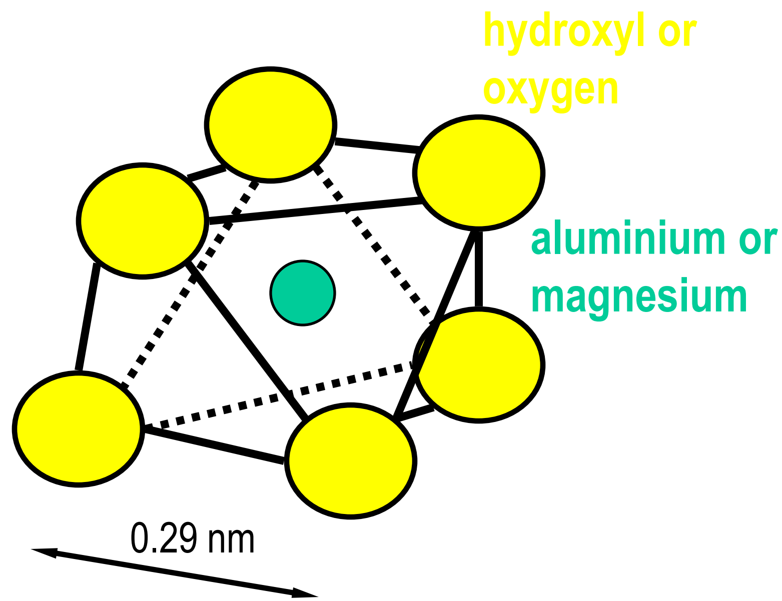
Atomic Structure



Basic Structural Units

Clay minerals are made of two distinct structural units.



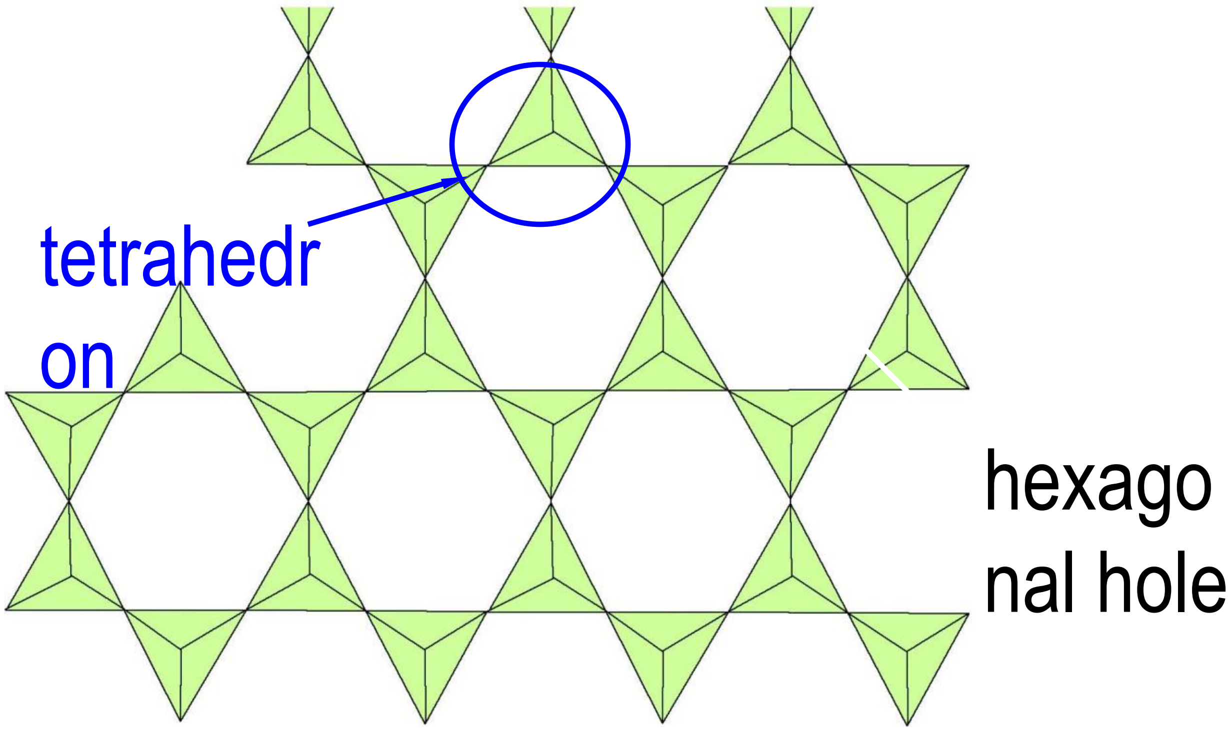


<u>Silicon tetrahedron</u>

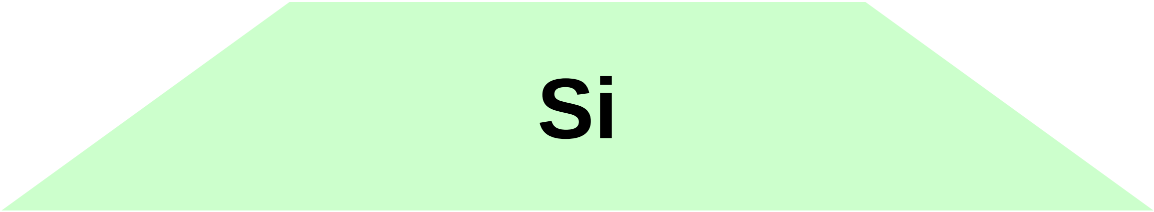
<u>Aluminium Octahedron</u>

Tetrahedral Sheet

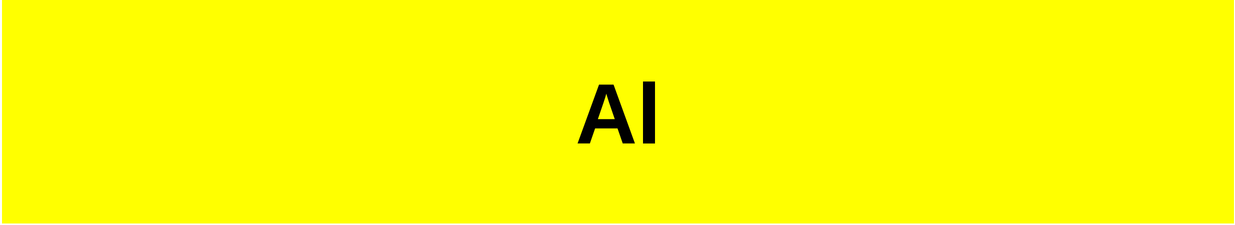
Several tetrahedrons joined together form a tetrahedral sheet.



For simplicity, let's represent silica **tetrahedral sheet** by**:**



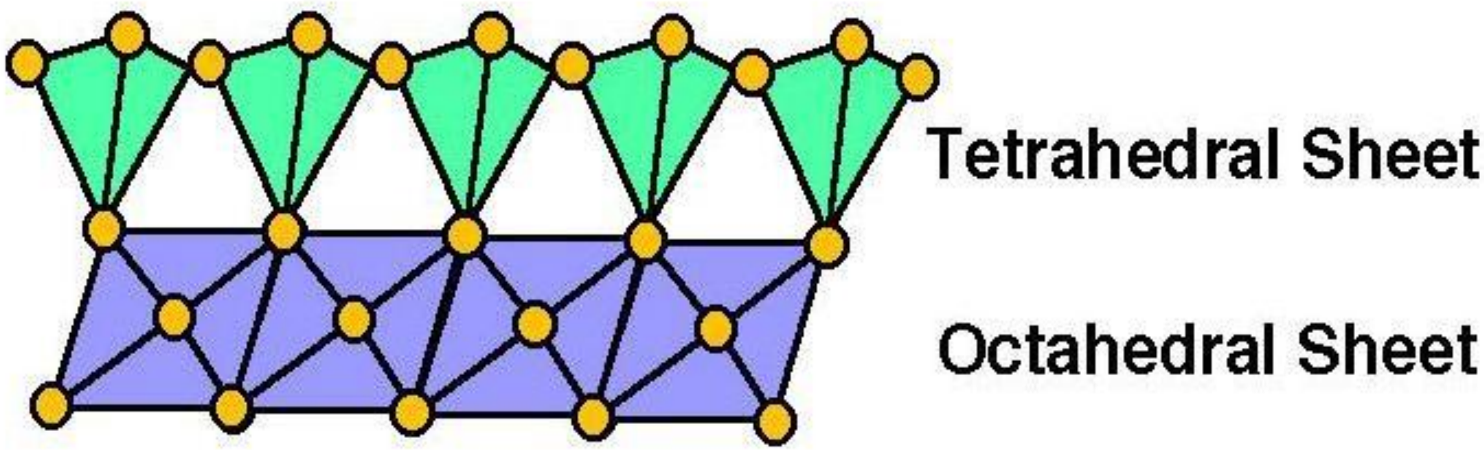
and alumina **octahedral sheet** by:



Different Clay Minerals

Different combinations of tetrahedral and octahedral sheets form different clay minerals:

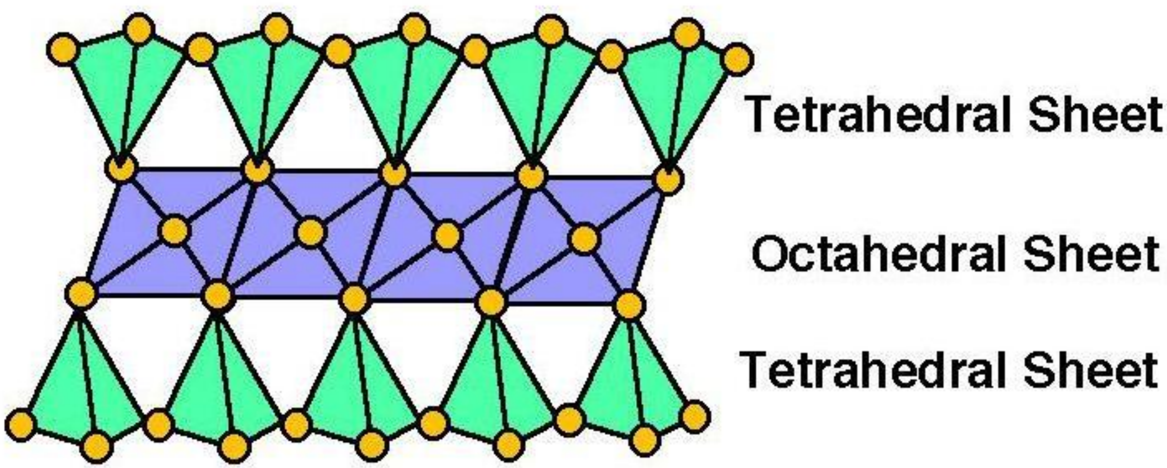
<u>1:1 Clay Mineral </u>(e.g., kaolinite, halloysite):



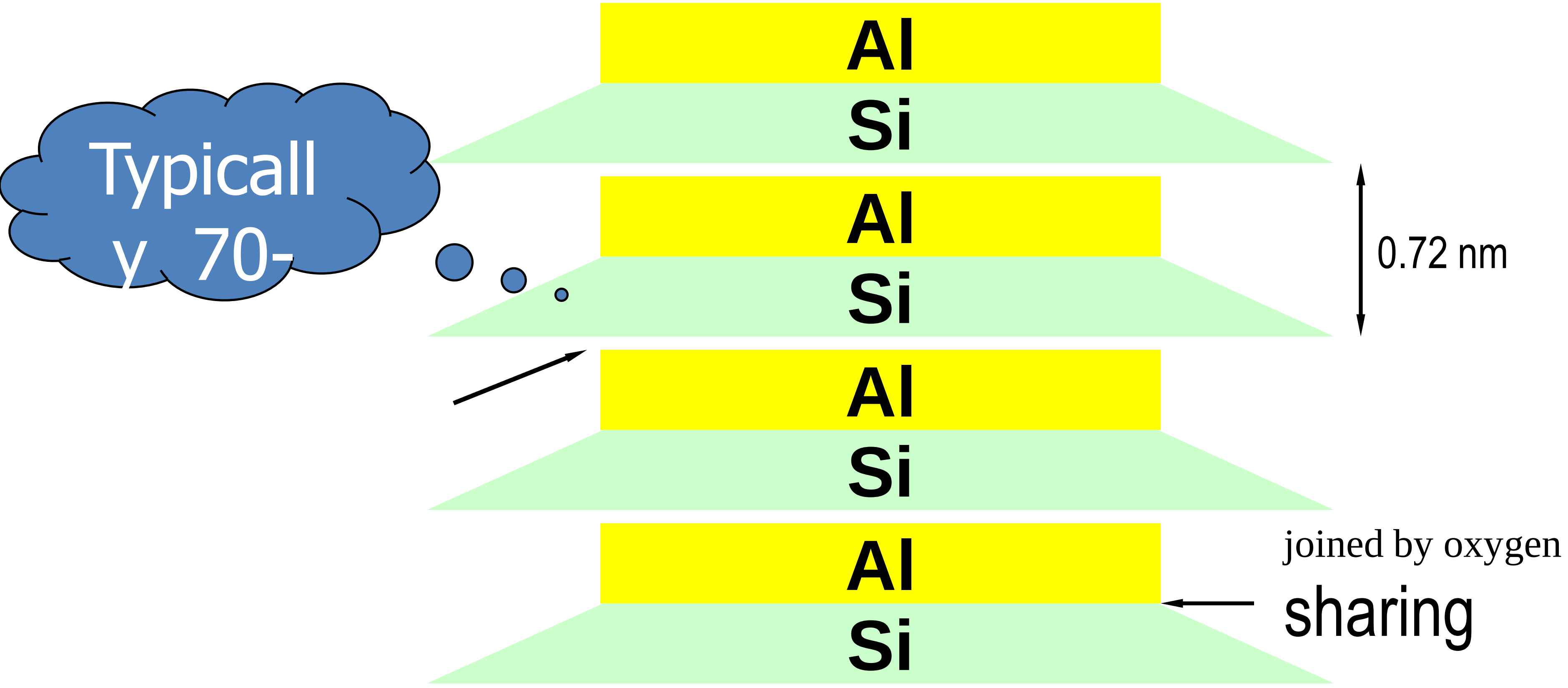
Different Clay Minerals

Different combinations of tetrahedral and octahedral sheets form different clay minerals:

<u>2:1 Clay Mineral</u> (e.g., montm



Kaolinite


Al
Si
Al
Si
Al
Si
Al
Si
Typicall
y 70-
0.72 nm
joined by oxygen
sharing

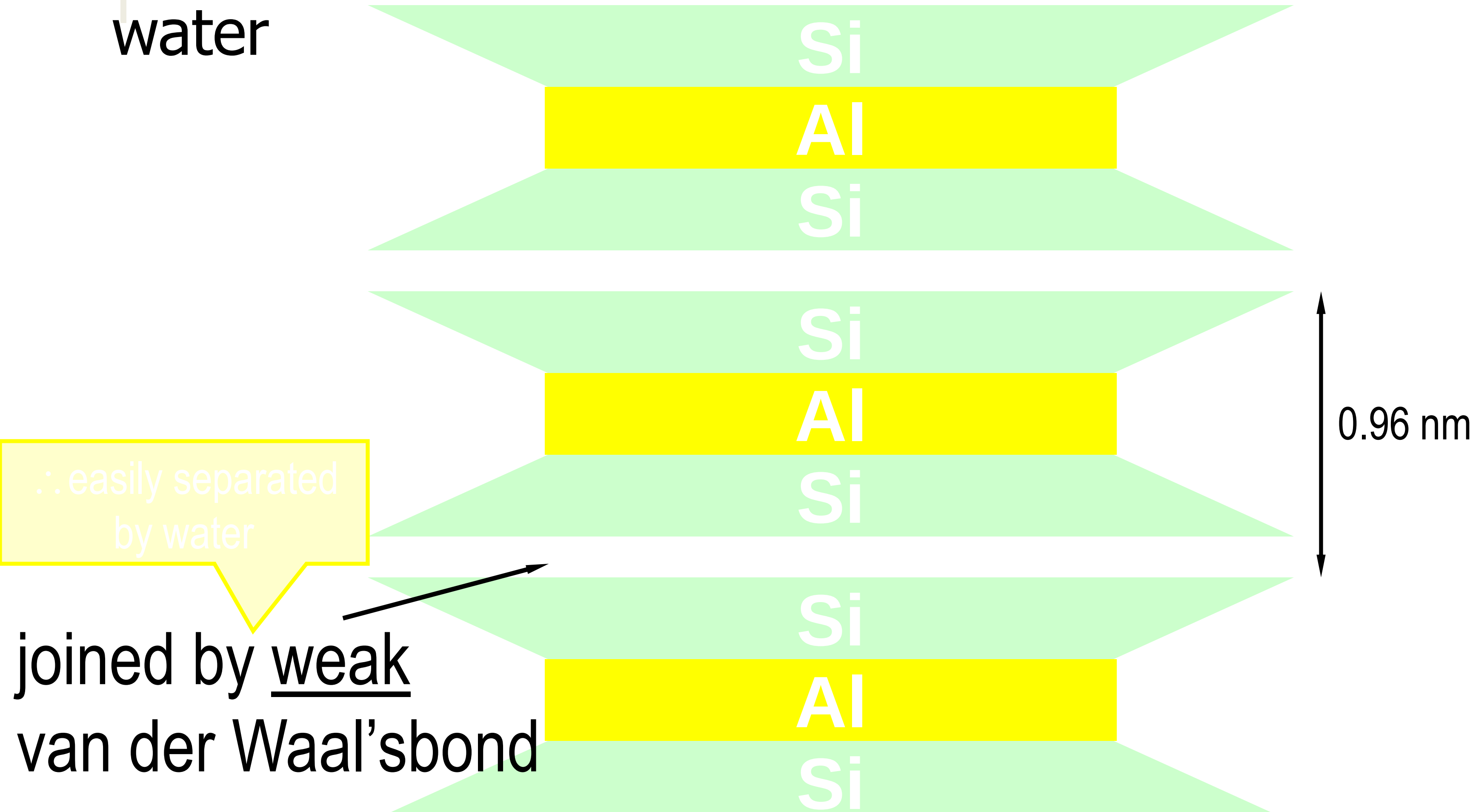
Kaolinite

- used in paints, paper and in pottery and pharmaceutical industries
- $(OH)_8Al_4Si_4O_{10}$

Halloysite

- kaolinite family; hydrated and tubular structure
- $(OH)_8Al_4Si_4O_{10}.4H_2O$

- also called **<u>smectite</u>**; expands on contact with water



- A highly reactive (expansive) clay

swells on contact with water

- $(OH)_4Al_4Si_8O_{20}.nH_2O$

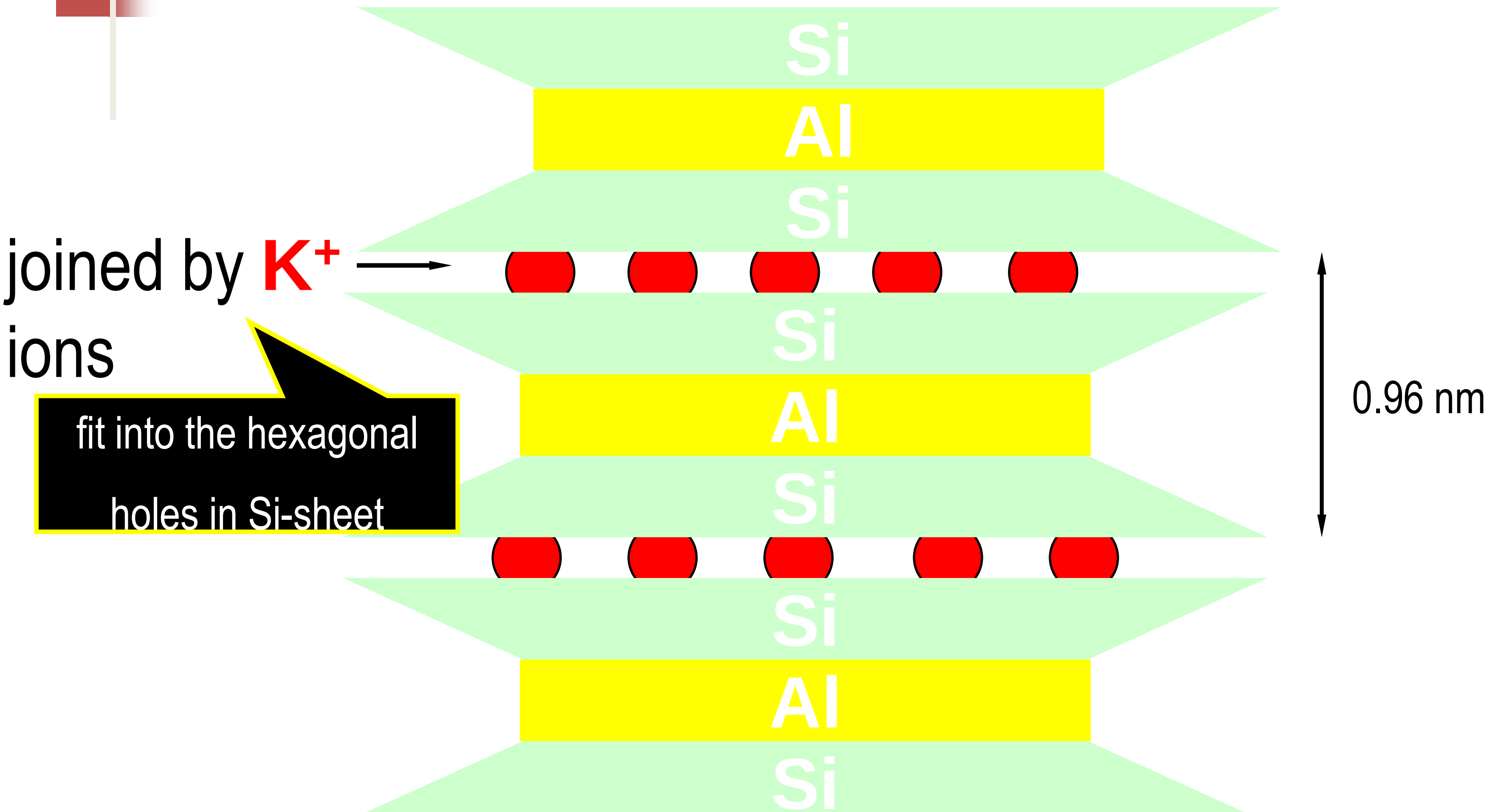
high affinity to

Bentonite

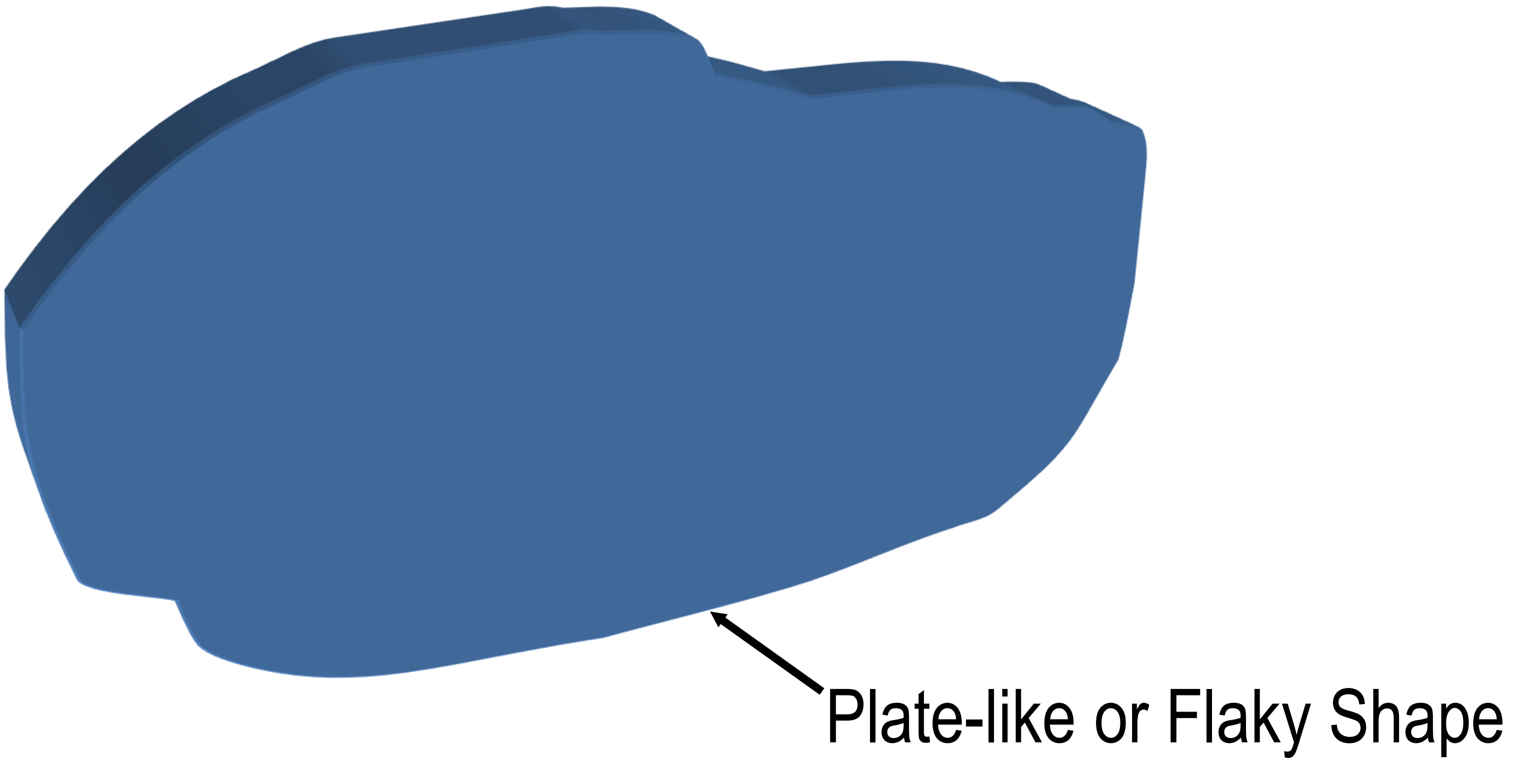
- montmorillonite family

- used as drilling mud, in slurry trench walls, stopping leaks

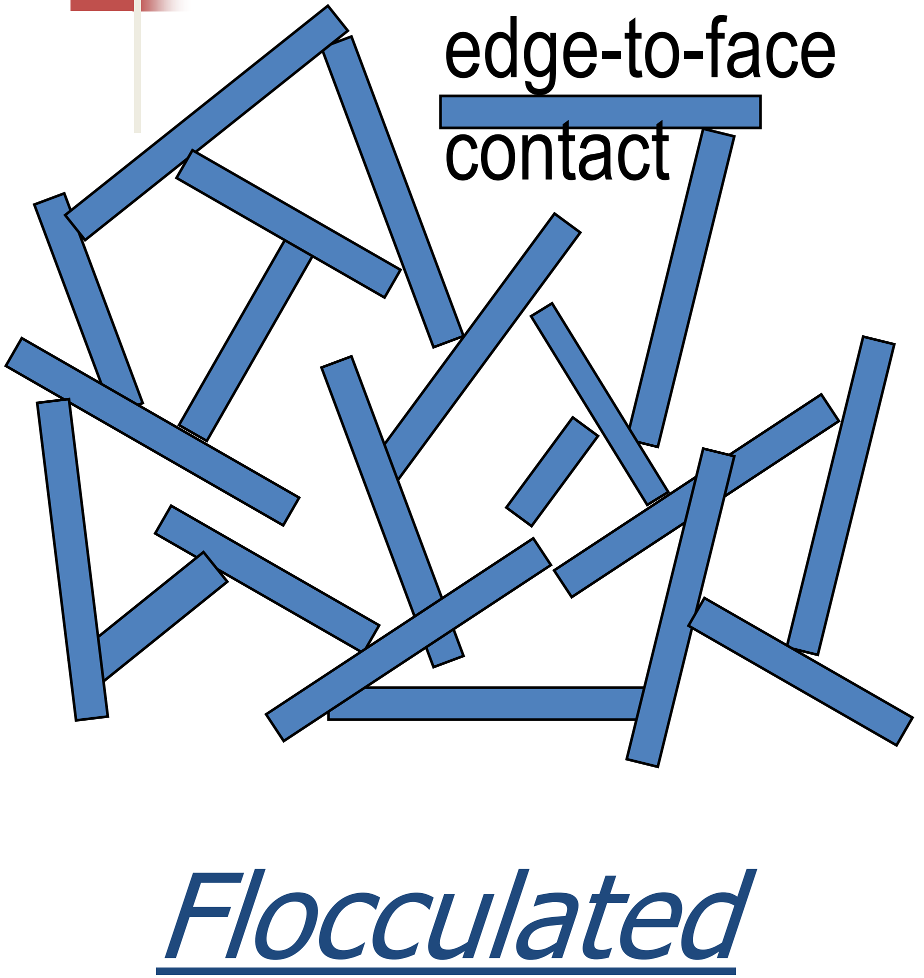
Illite

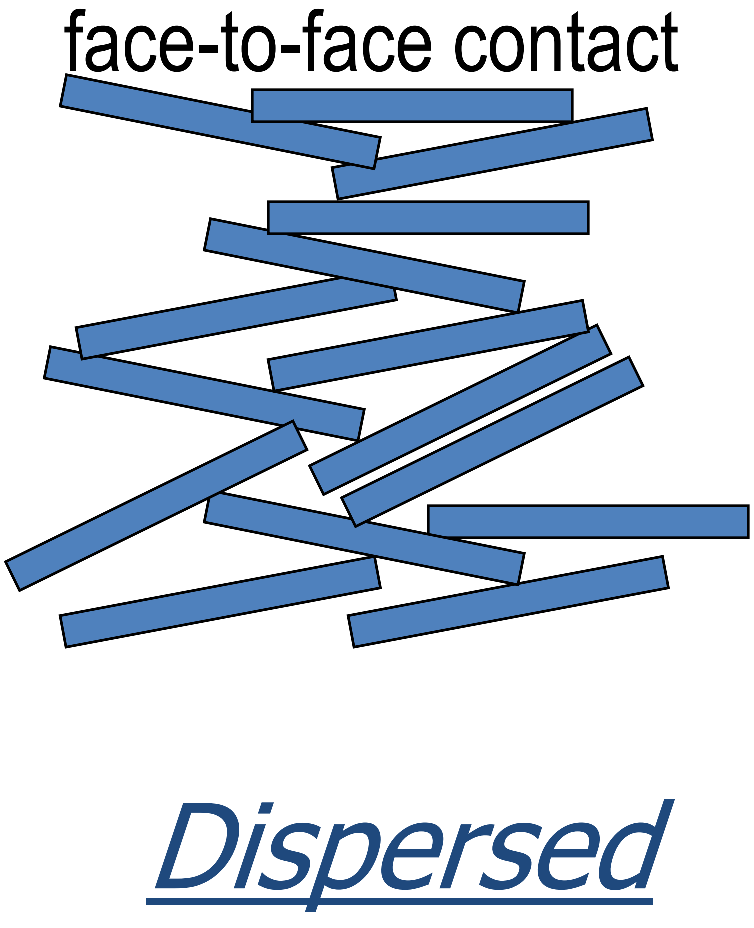

Si
Al
Si
joined by K+ ions
fit into the hexagonal holes in Si-sheet
Si
Al
Si
0.96 nm
Si
Al
Si

A Clay Particle



Clay Fabric


edge-to-face contact
Flocculated


face-to-face contact
Dispersed

Clay Fabric

- Electrochemical environment (i.e., pH, acidity, temperature, cations present in the water) during the time of sedimentation influence clay fabric significantly.

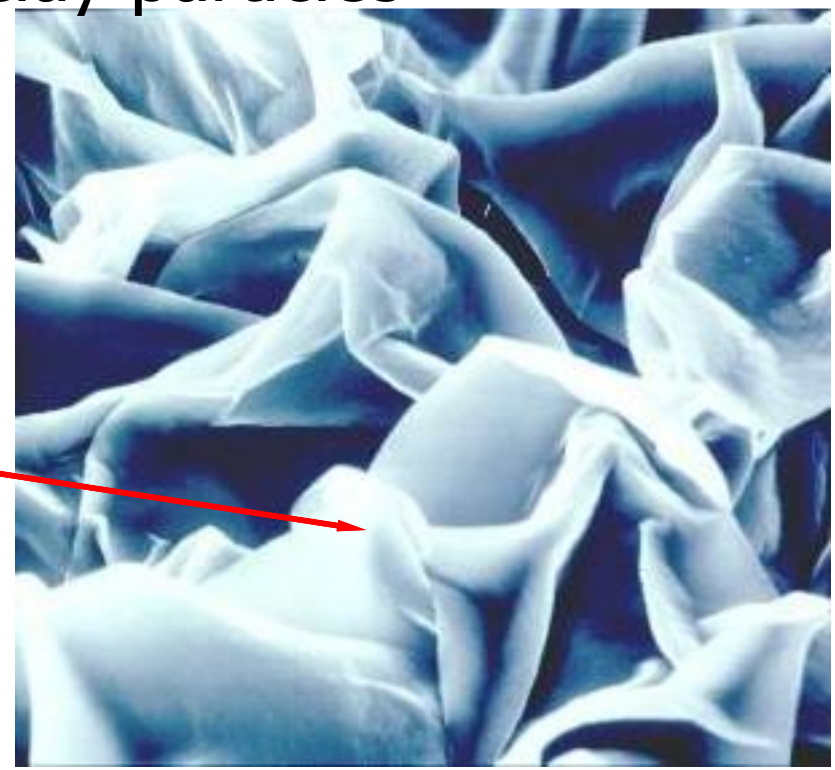
- Clay particles tend to align perpendicular to the load applied on them.

Identifying Clay Minerals

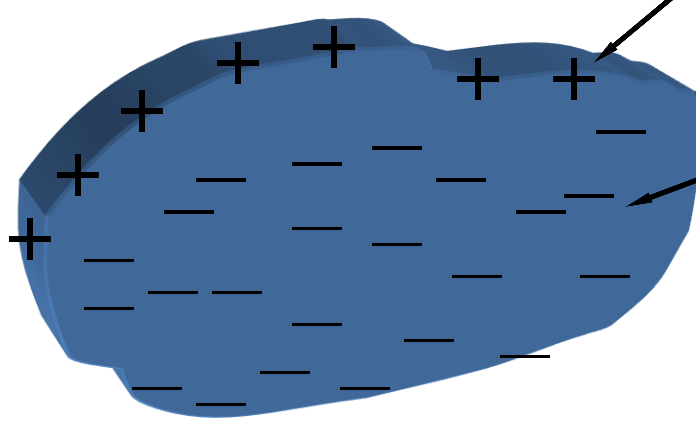
Scanning Electron Microscope

- common technique to <u>see</u> clay particles
- qualitative

plate-like structure



- substitution of Si^{4+} and Al^{3+} by other lower valence (e.g., Mg^{2+}) cations
- results in charge imbalance (net negative)



Clay Particle with Net negative Charge

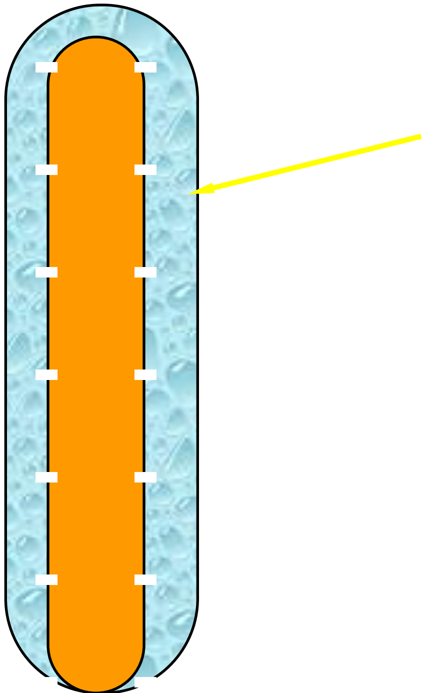
Cation Exchange Capacity (c.e.c)

- capacity to attract cations from the water (i.e., measure of the net negative charge of the clay particle)
- measured in meq/100g (net negative charge per 100 g of clay)
- The replacement power is greater for higher valence and larger cations.

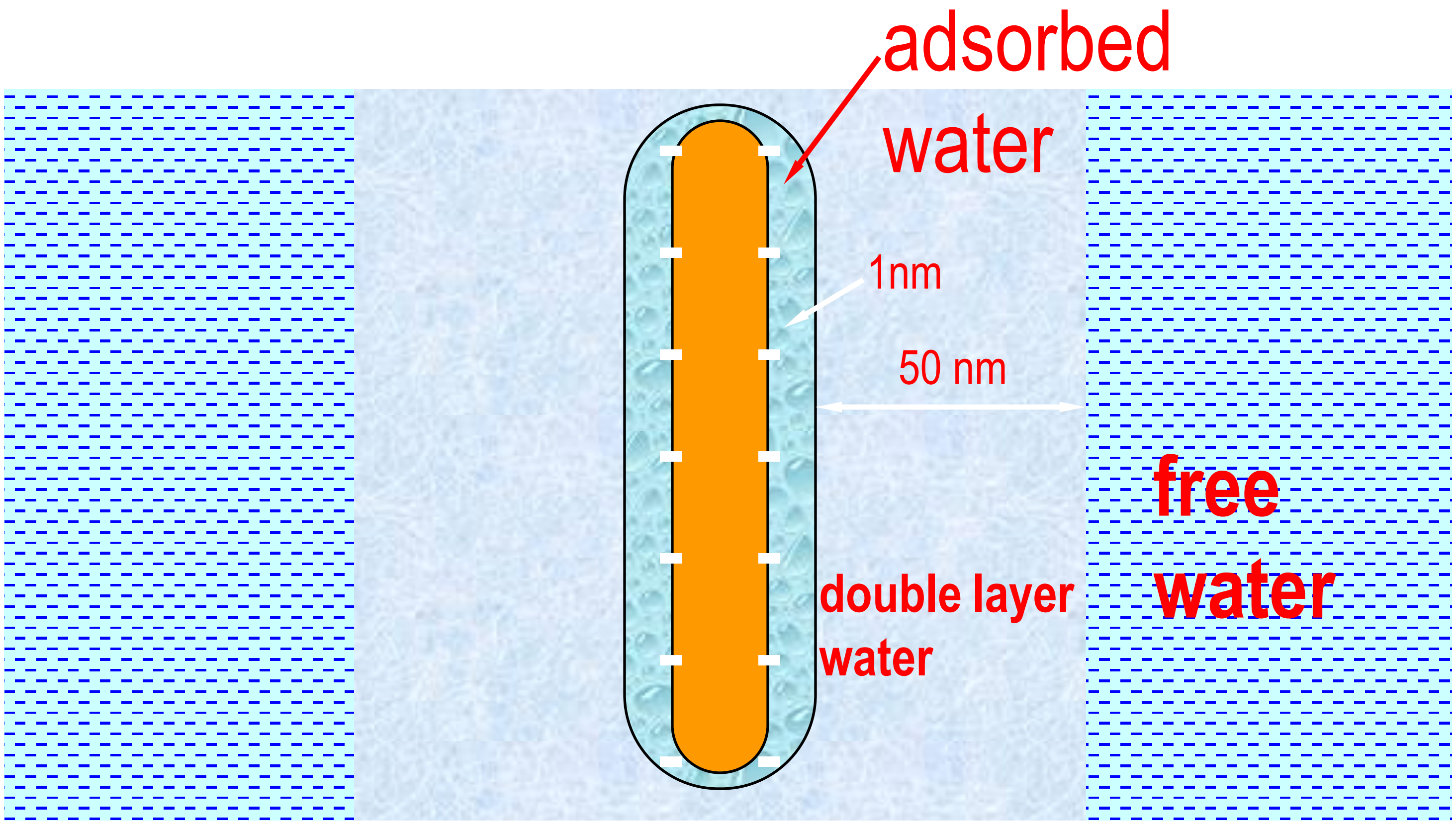
$Al^{3+} > Ca^{2+} > Mg^{2+} >> NH_4^{+} > K^{+} > H^{+} > Na^{+} > Li^{+}$

Adsorbed Water

- A thin layer of water tightly held to particle; like a skin
- 1-4 molecules of water (1 nm) thick
- more viscous than free water



Clay Particle in Water



Summary - Montmorillonite

- Montmorillonites have very high specific surface, cation exchange capacity, and affinity to water. They form reactive clays.

- Montmorillonites have very high liquid limit (100+), plasticity index and activity (1-7).

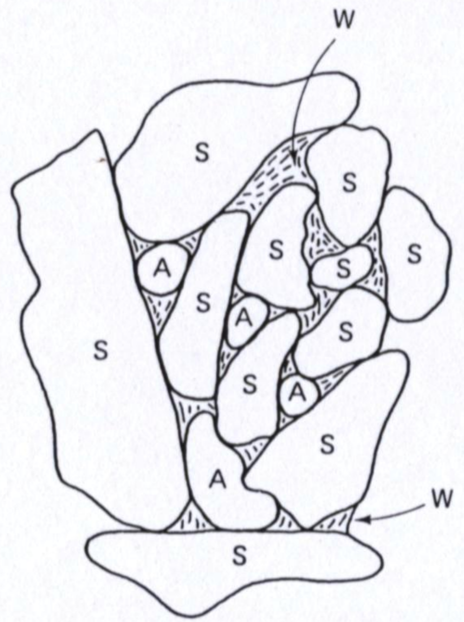
- Bentonite (a form of Montmorillonite) is frequently used as drilling mud.

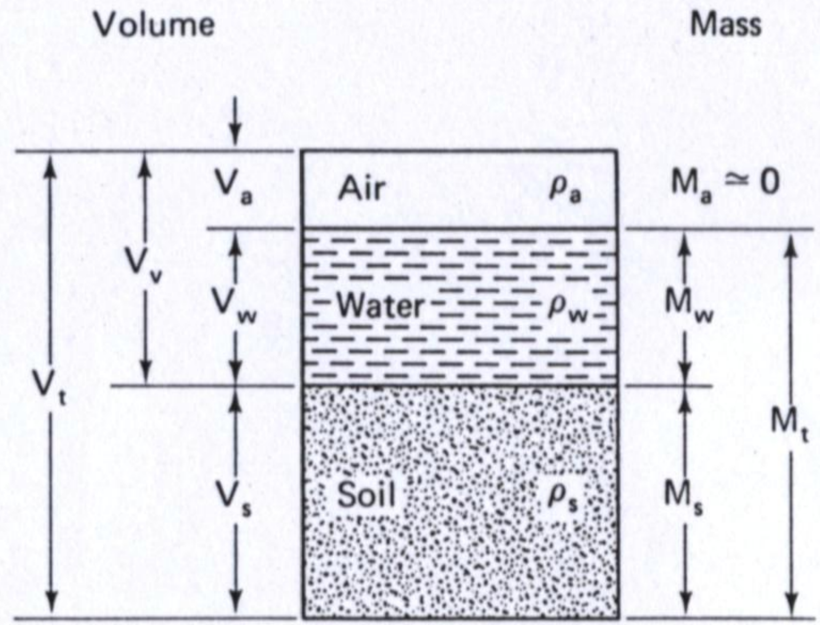
Three Phases in Soils

S : Solid — Soil particle

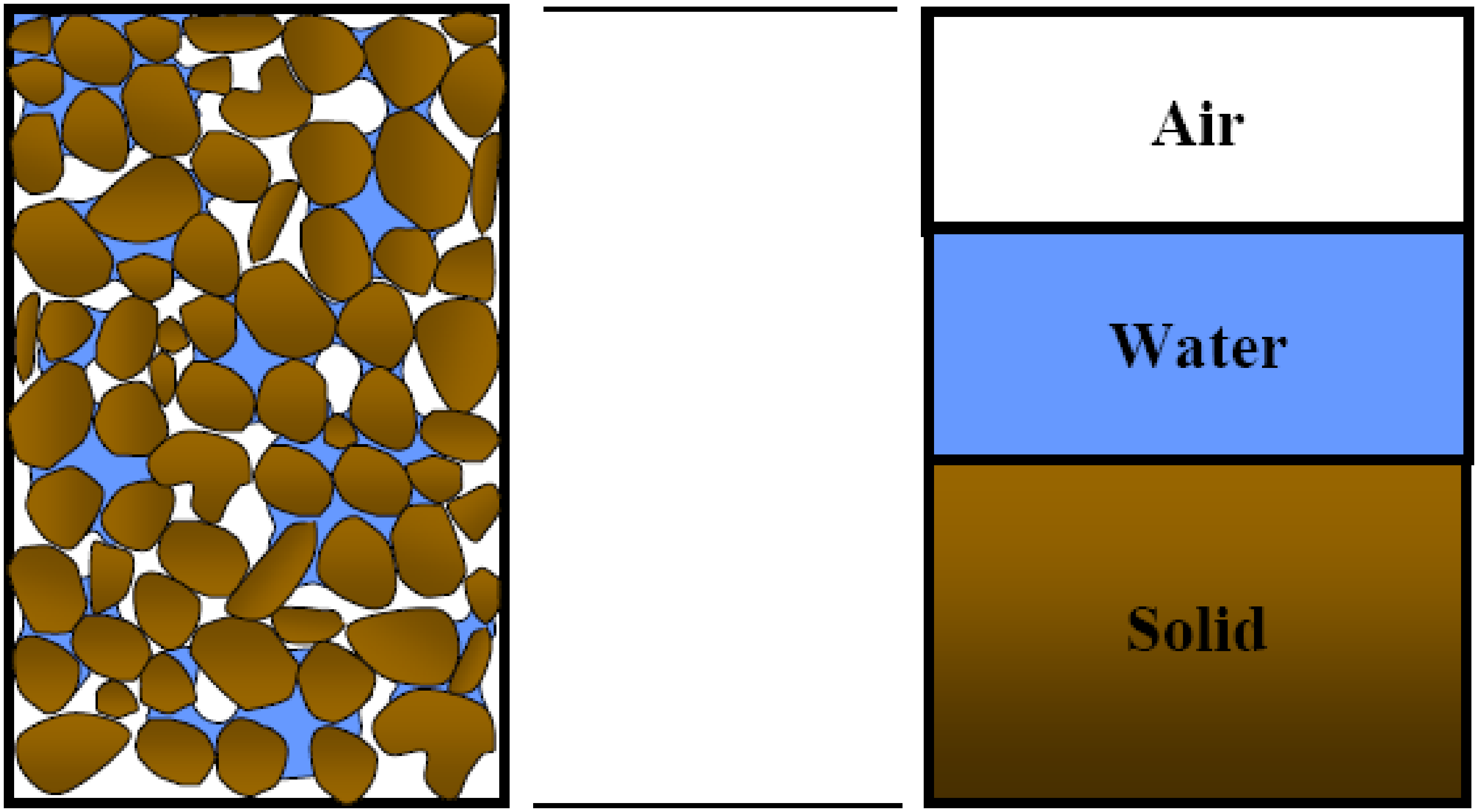
W: Liquid — Water (electrolytes)

A: Air — Air

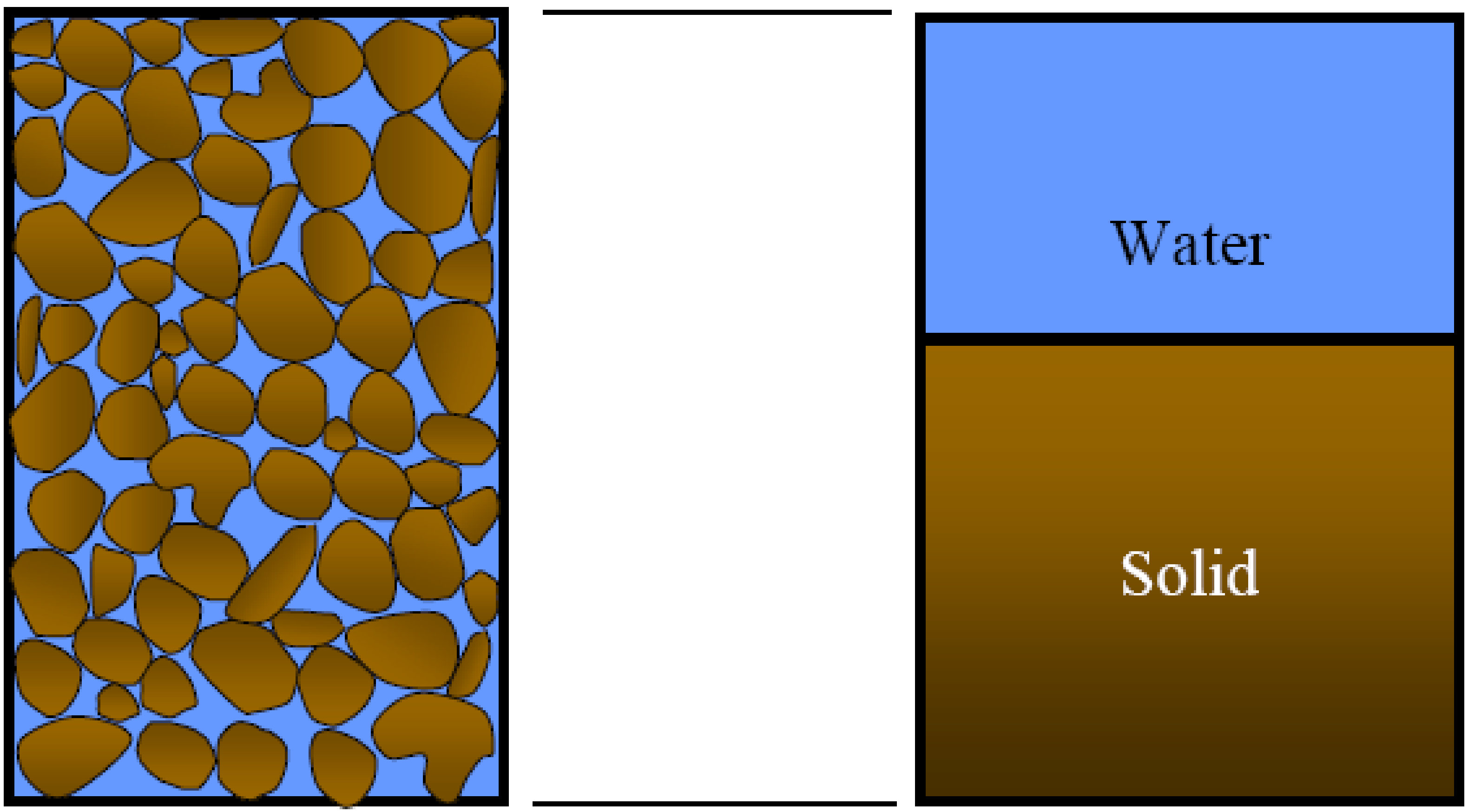




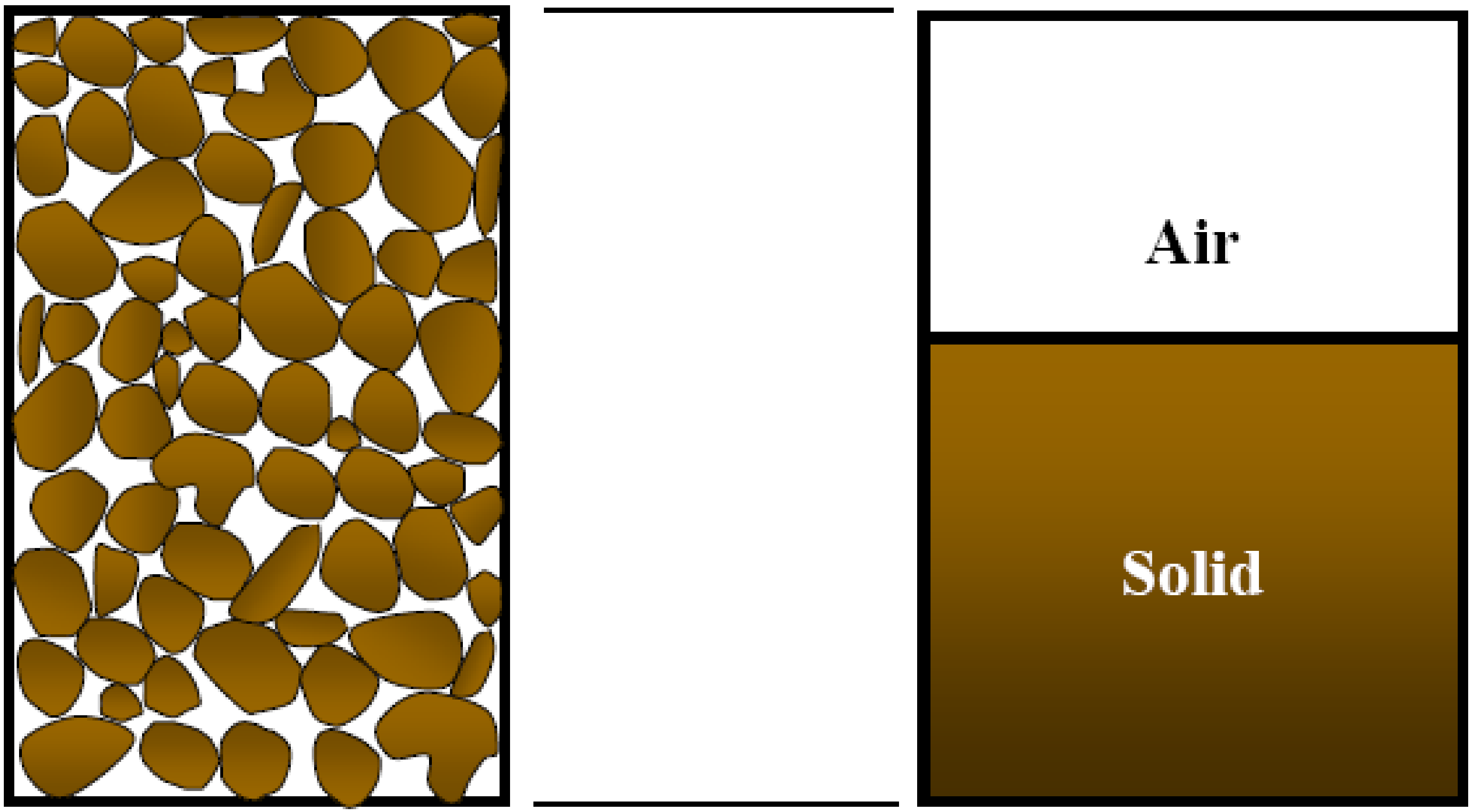
Three Phase Diagram



Fully Saturated Soils (Two phase)

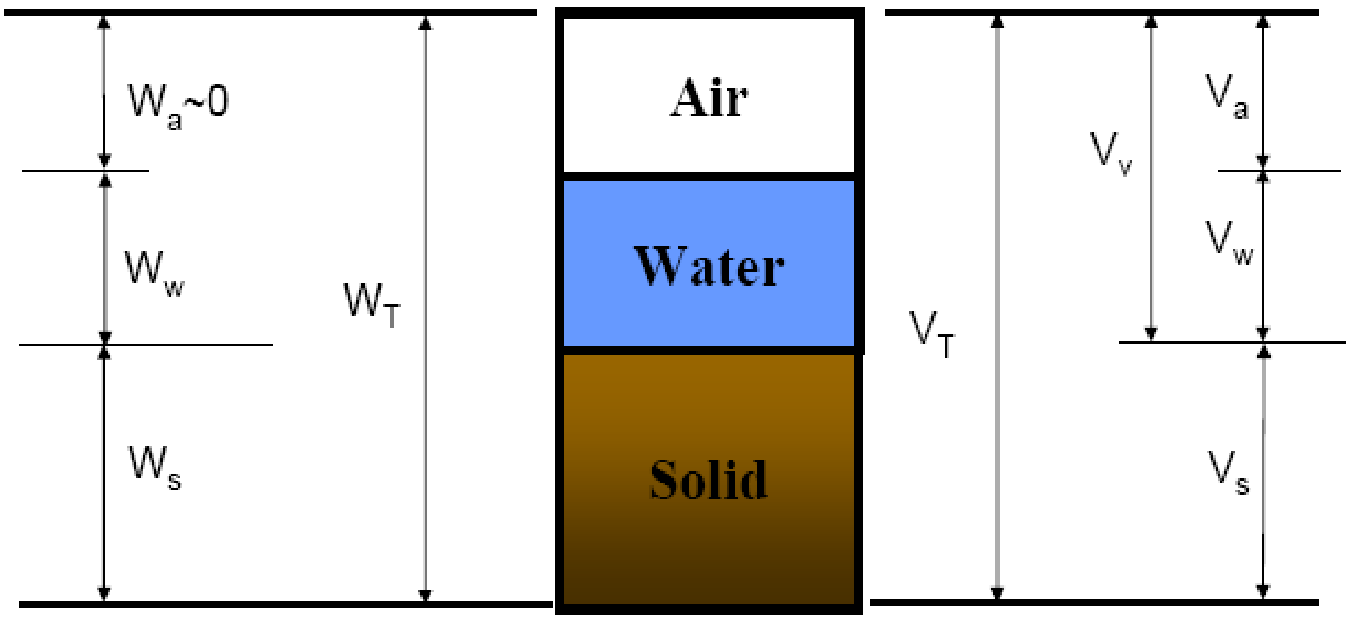


Dry Soils (Two phase) [Oven Dried]



PHASE DIAGRAM

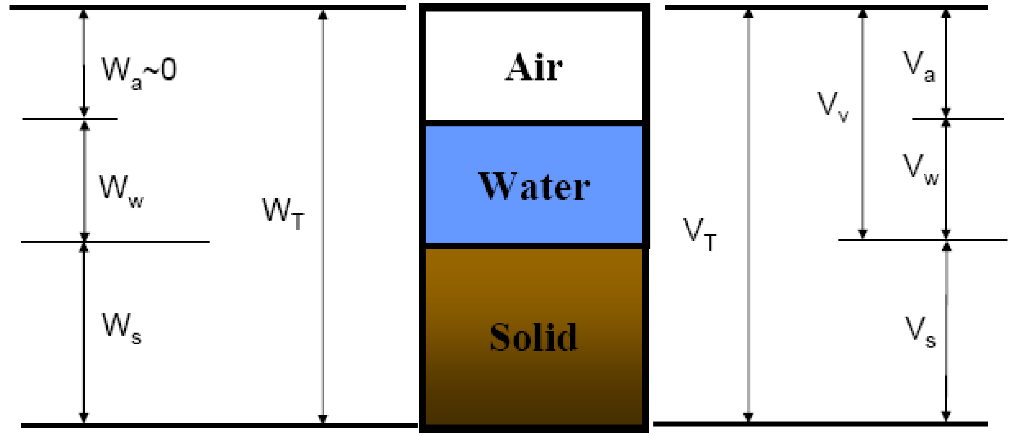
For purpose of study and analysis, it is convenient to represent the soil by a PHASE DIAGRAM, with part of the diagram representing the solid particles, part representing water or liquid, and another part air or other gas.



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

Volumetric Ratios



(1) Void ratio **e**

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

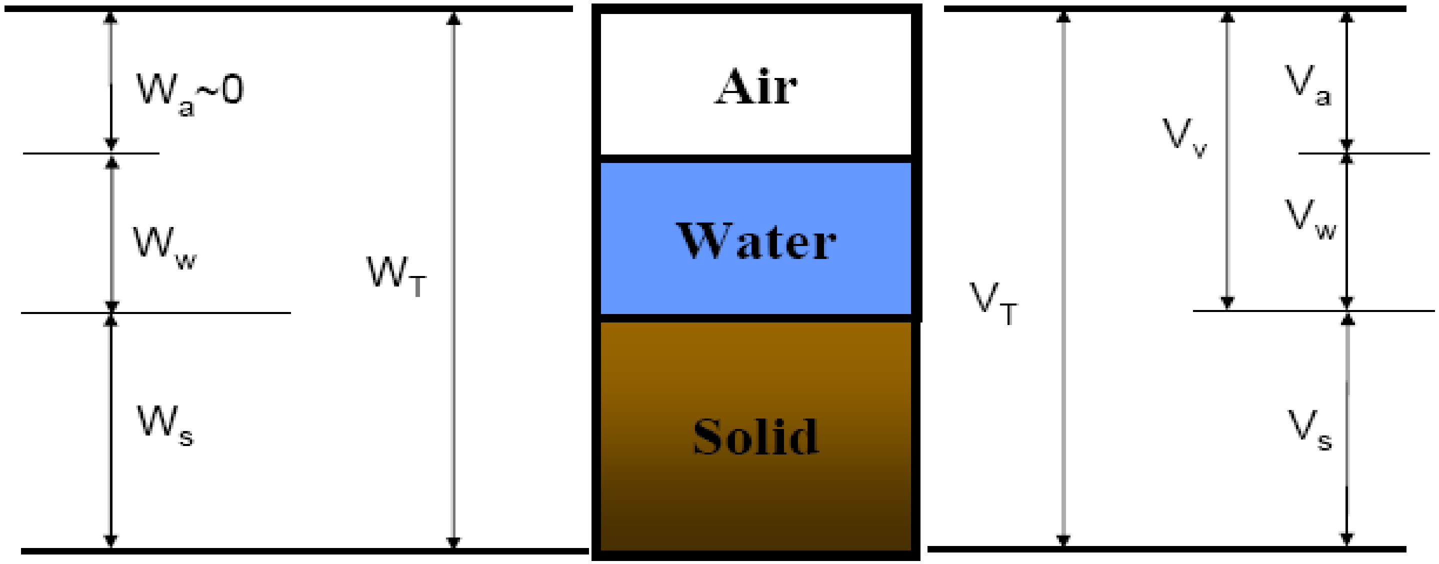
(2) Porosity **n%**

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

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

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

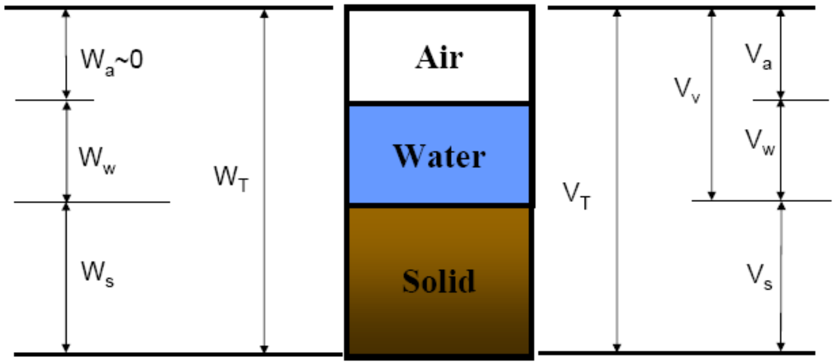
Weight Ratios



(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

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

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

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

(4) Submerged unit weight

$$\gamma' = \gamma_{sat} - \gamma_w$$

Note: 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 weight of solid particles to the weight of an equal volume of distilled water at 4°C

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

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.

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

- $G_{w = 1}$
- $G_{mercury} = 13.6$

- 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

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**

1. Relationship between e and n

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

Also, from Eq. (3.6),

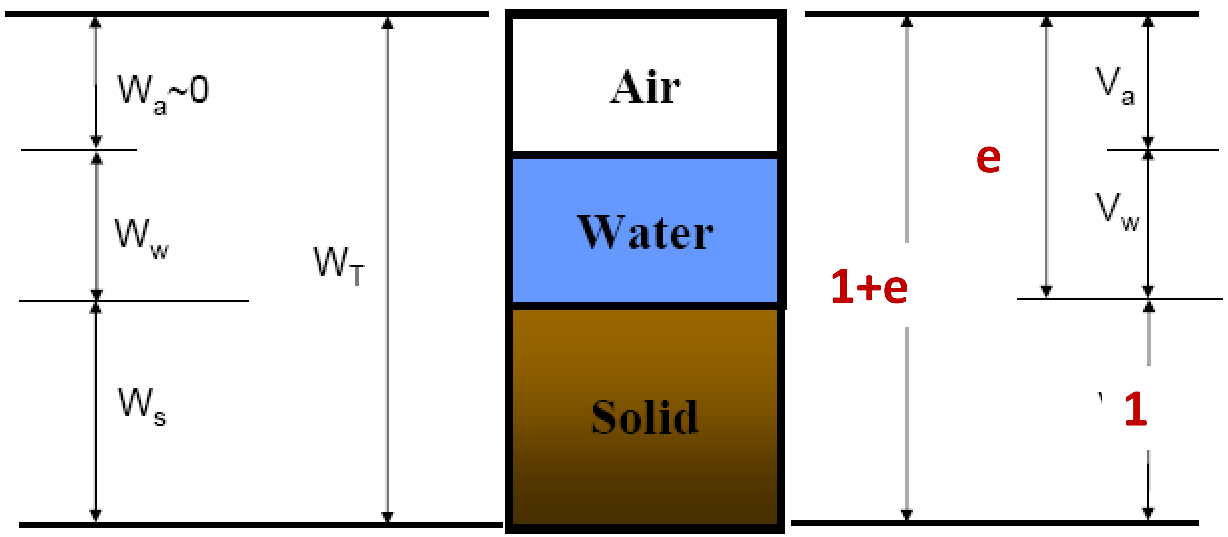
$$n = \frac{e}{1 + e} \quad (3.7)$$

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.**

3. Relationship among γ, e, S and G_s

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

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

- **<u>Notes:</u>**
  - **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.**

Method to solve Phase Problems

Method : Memorize relationships

$$Se = wG_s$$

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

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

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

Example 1

The moist unit weight of a soil is 19.2 kN/m³. Given that G_s 2.69 and $w = 9.8\%$, determine

a. Dry unit weight

b. Void ratio

c. Porosity

d. Degree of saturation

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

Example 2

Field density testing (e.g., sand replacement method) has shown bulk density of a compacted road base to be 2.06 g/cc 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.

<u>Solution</u>:

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

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

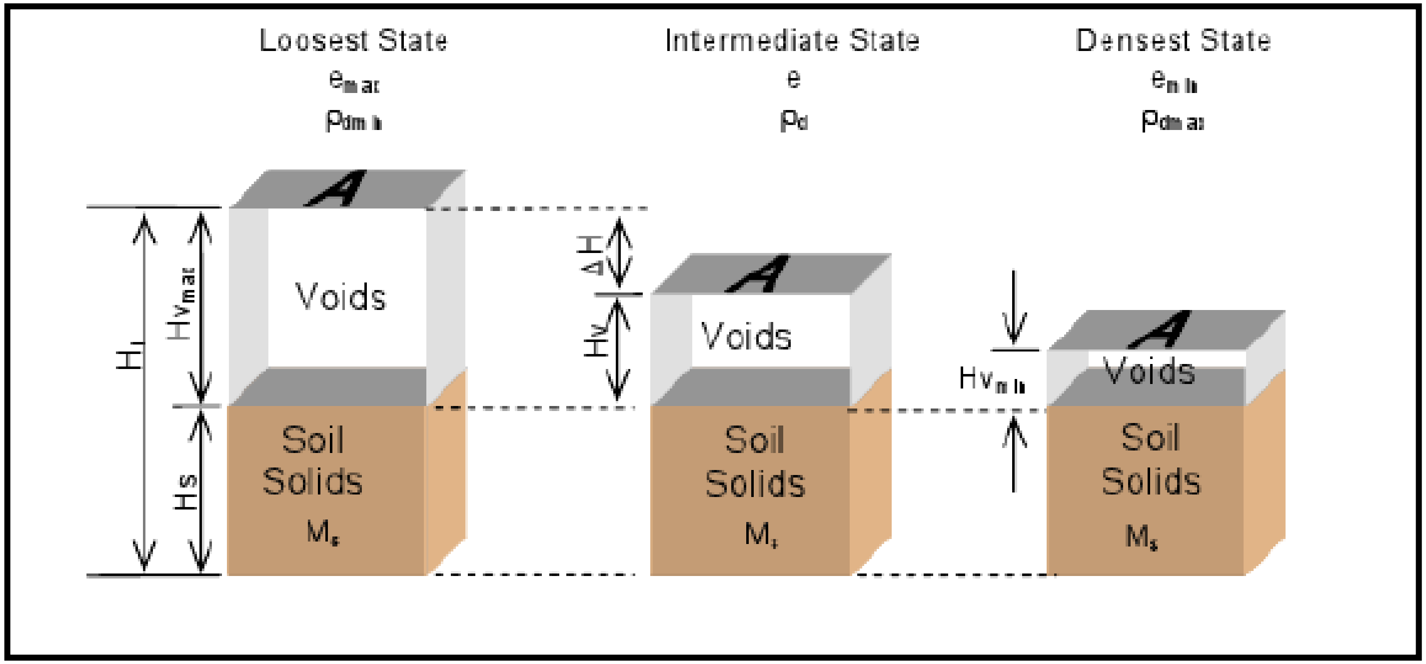
$$\rho_m = \frac{G_s + Se}{1 + e}\rho_w$$

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

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

•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

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

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

• **Remarks**

- The 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.

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

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

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

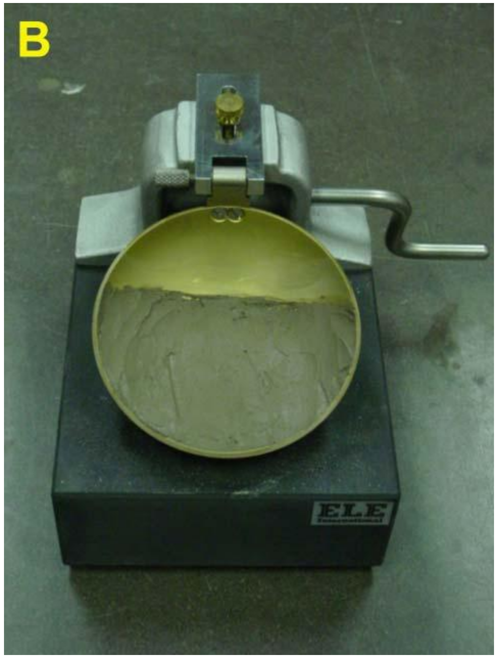
ATTERBERG LIMITS

- **Liquid limit test:**

- A soil is place in the grooving tool which consists of
  brass cup and a hard rubber base.
- A groove is cut at the center of the soil pat using a standard grooving tool.
- The cup is then repeatedly drooped from a height of 10mm until a groove closure of 12.7 mm.
- The soil is then removed and its moisture content is determined.
- The soil is said to be at its liquid limit when exactly 25 drops are required to close the groove for a distance of 12.7 mm (one half of an inch)

APPARATUS


A
Spatula
Liquid limit device
Grooving tool
Soil specimen
Moisture cans


B


C


D
13mm

➢ **Plastic limit test:**

- A soil sample is rolled into threads until it becomes thinner and eventually breaks at 3 mm.
- It is defined as the moisture content in percent at which the soil crumbles when rolled into the threads of 3.0 mm.
- If it is wet, it breaks at a smaller diameter; if it is dry it breaks at a larger diameter.

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 **D_{10}, D_{30} and D_{60}** of this soil using the sieve analysis?
5. Calculate both the **C_u and 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 practical class

Answer all the above questions in your first report.

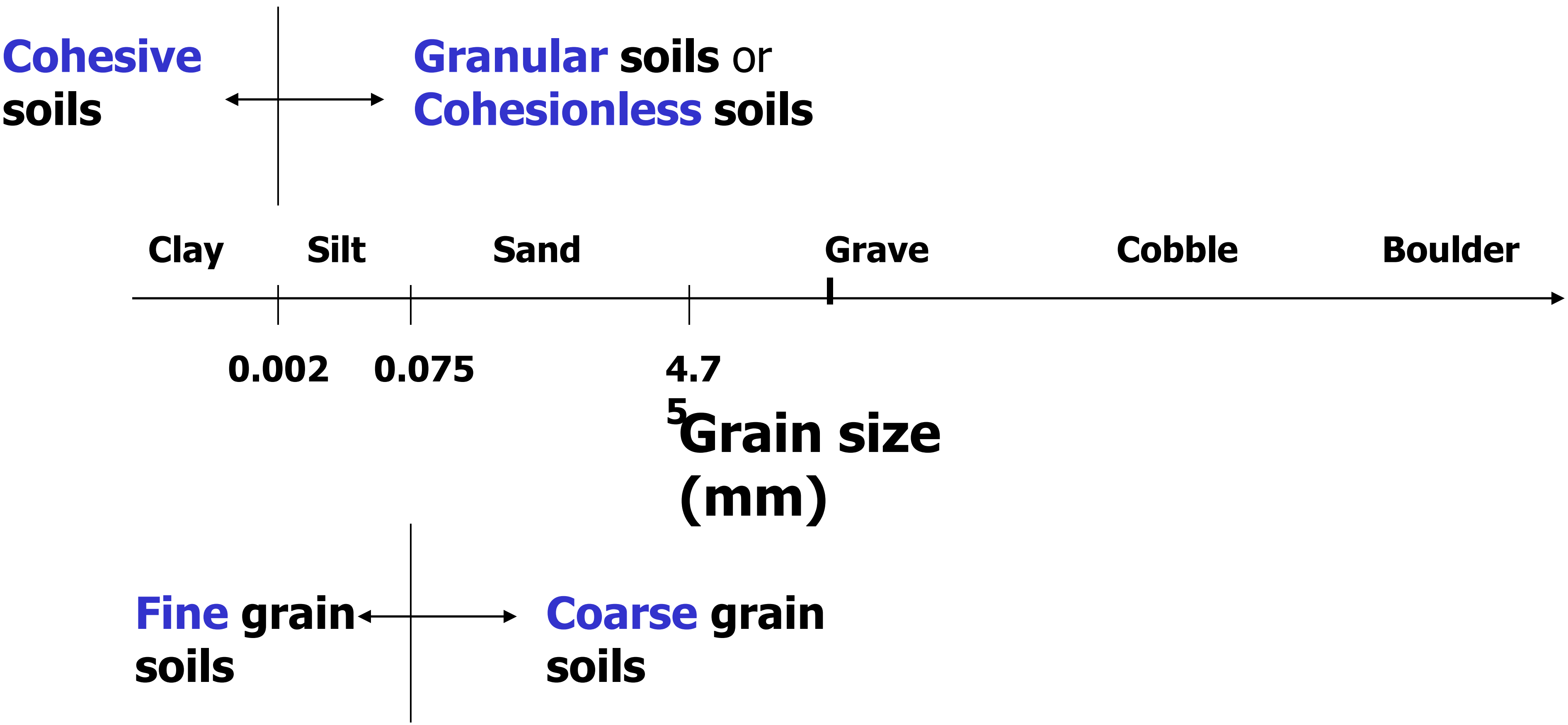
Purpose:

- 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



Significance of GSD:

- To know the relative proportions of different grain sizes.

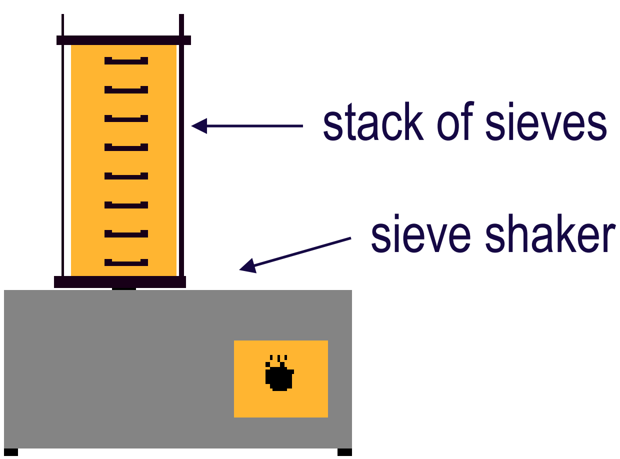
- An important factor influencing the geotechnical characteristics of a **coarse** grain soil.

- Not important in fine grain soils.

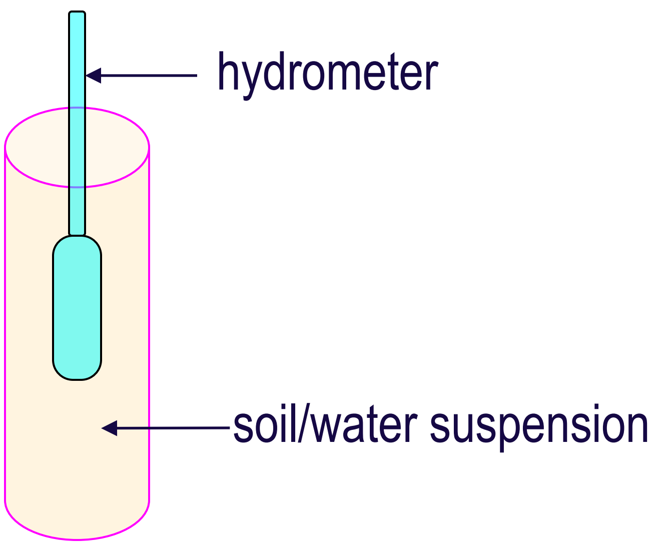
Determination of GSD:

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

- In fine grain soils …... By hydrometer analysis

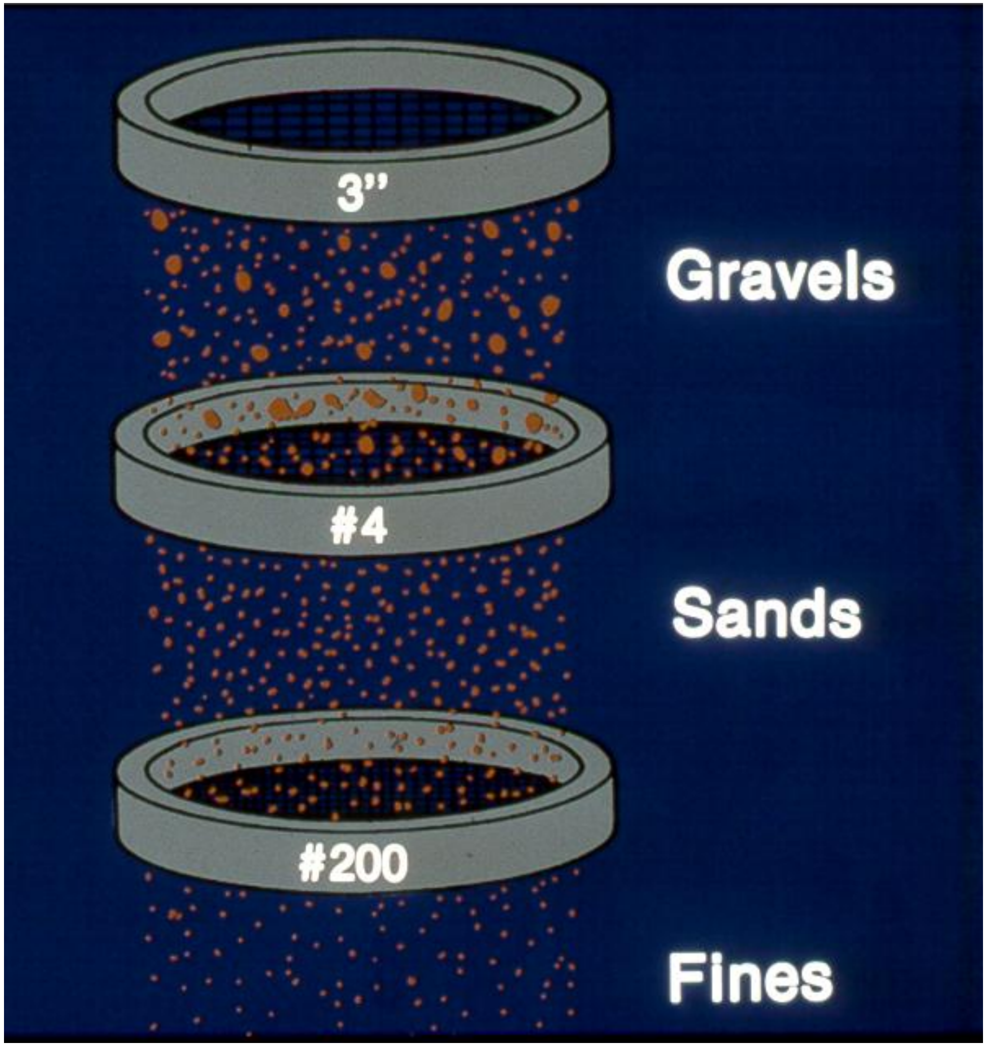


Sieve Analysis



Hydrometer Analysis

Sieve Analyses



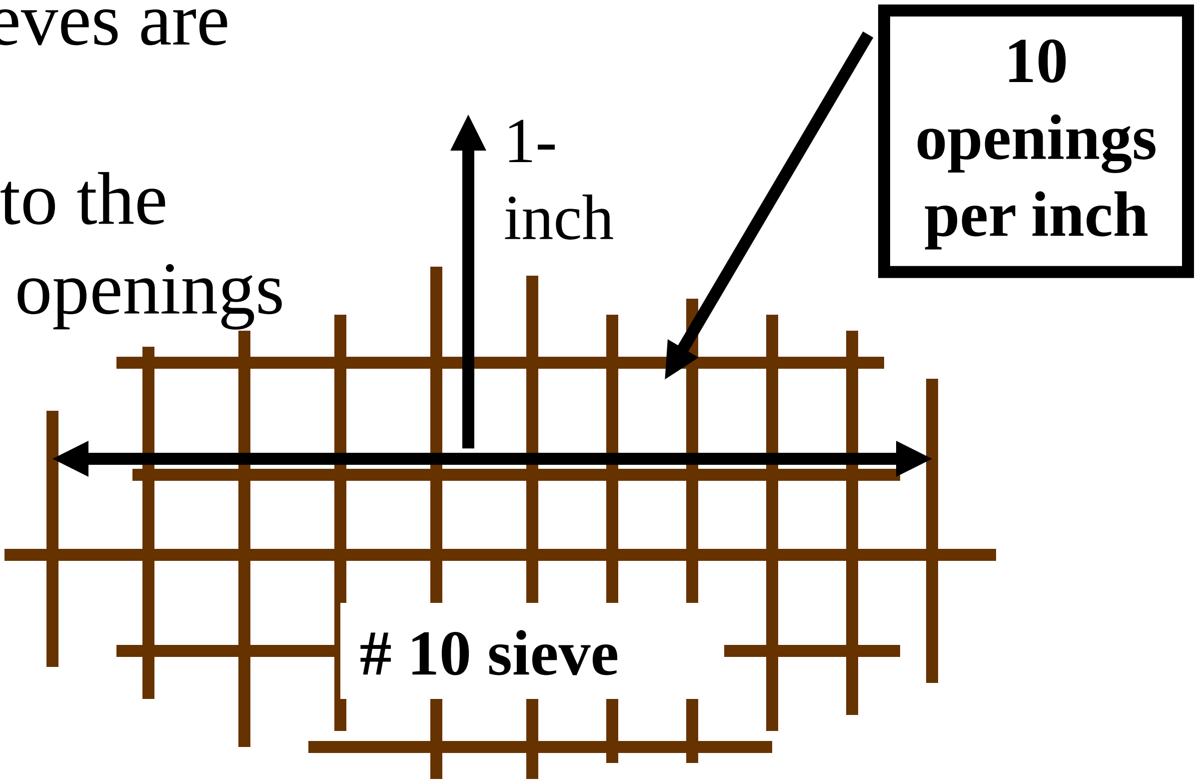
Sieve Analysis

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.

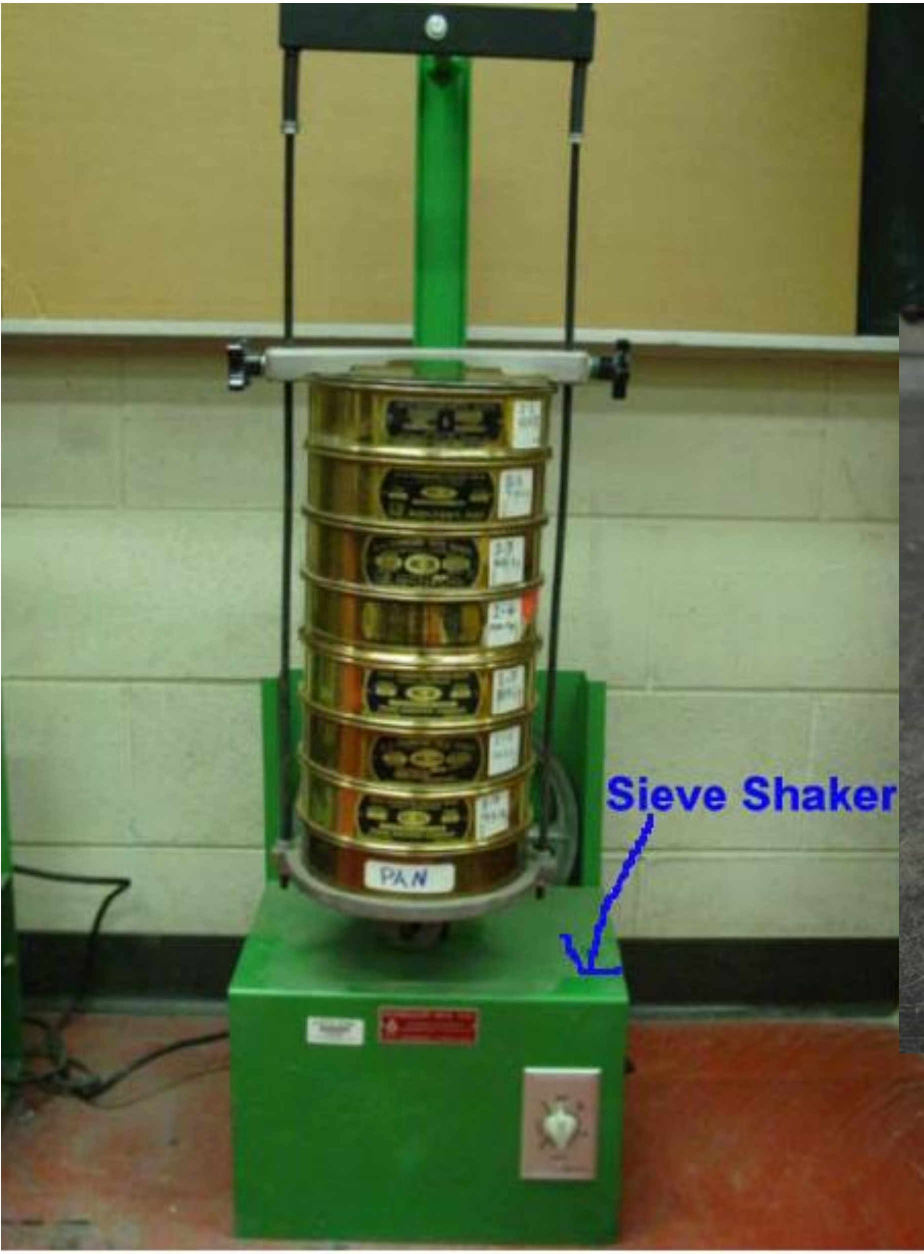
(3) Make sure that all the sieves are clean, and assemble them in the ascending order of sieve numbers (#4 sieve at top and #200 sieve at bottom). Place the pan below #200 sieve. Carefully pour the soil sample into the top sieve and place the cap over it.

(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


Sieve Shaker
PAN


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

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

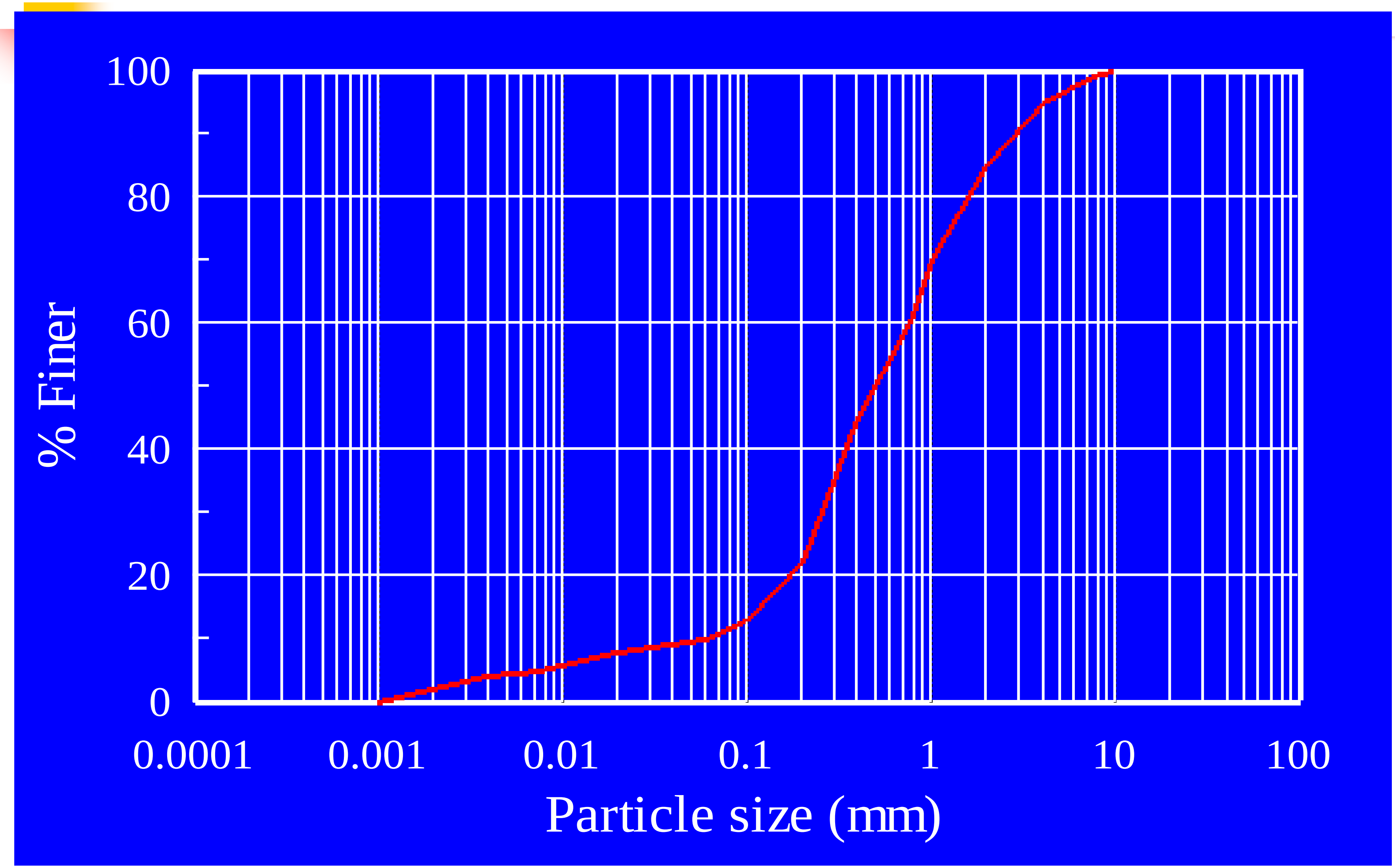
The percent retained is calculated as;

% 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

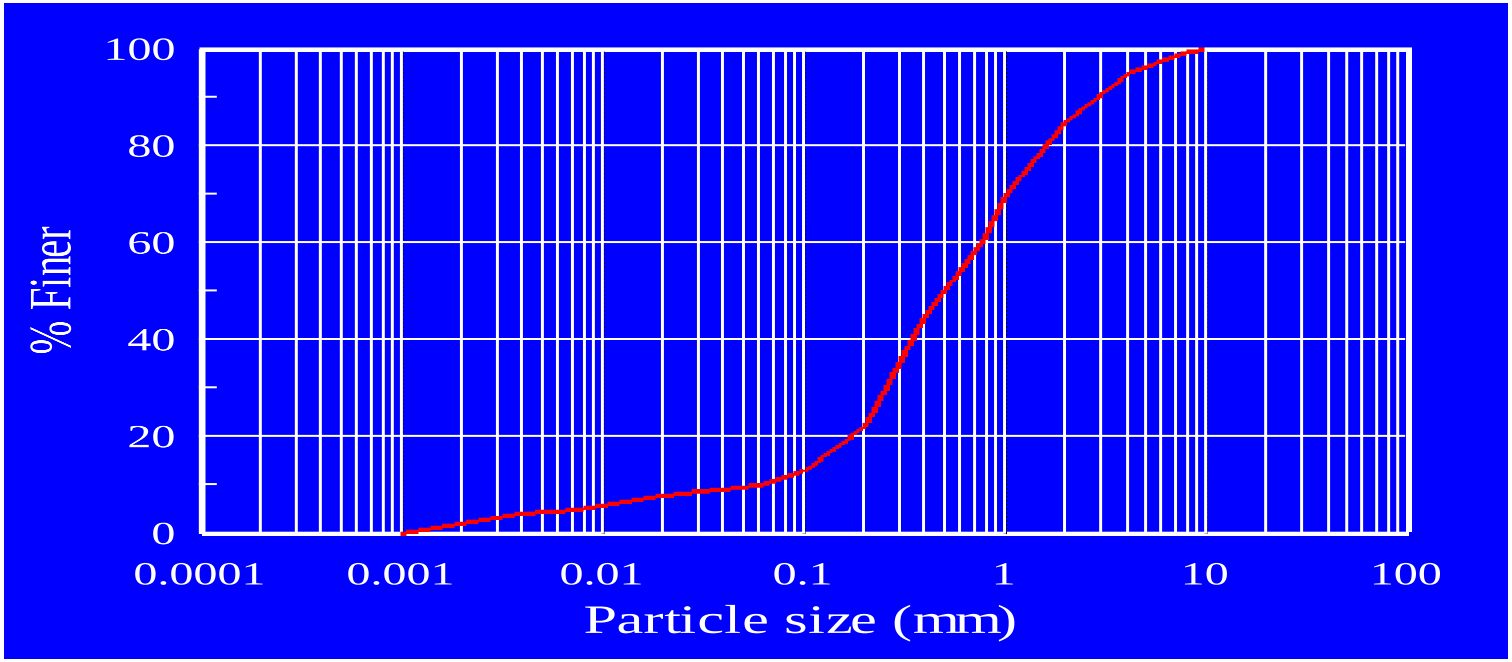
Unified Soil Classification

To determine W or P, calculate C_u and C_c

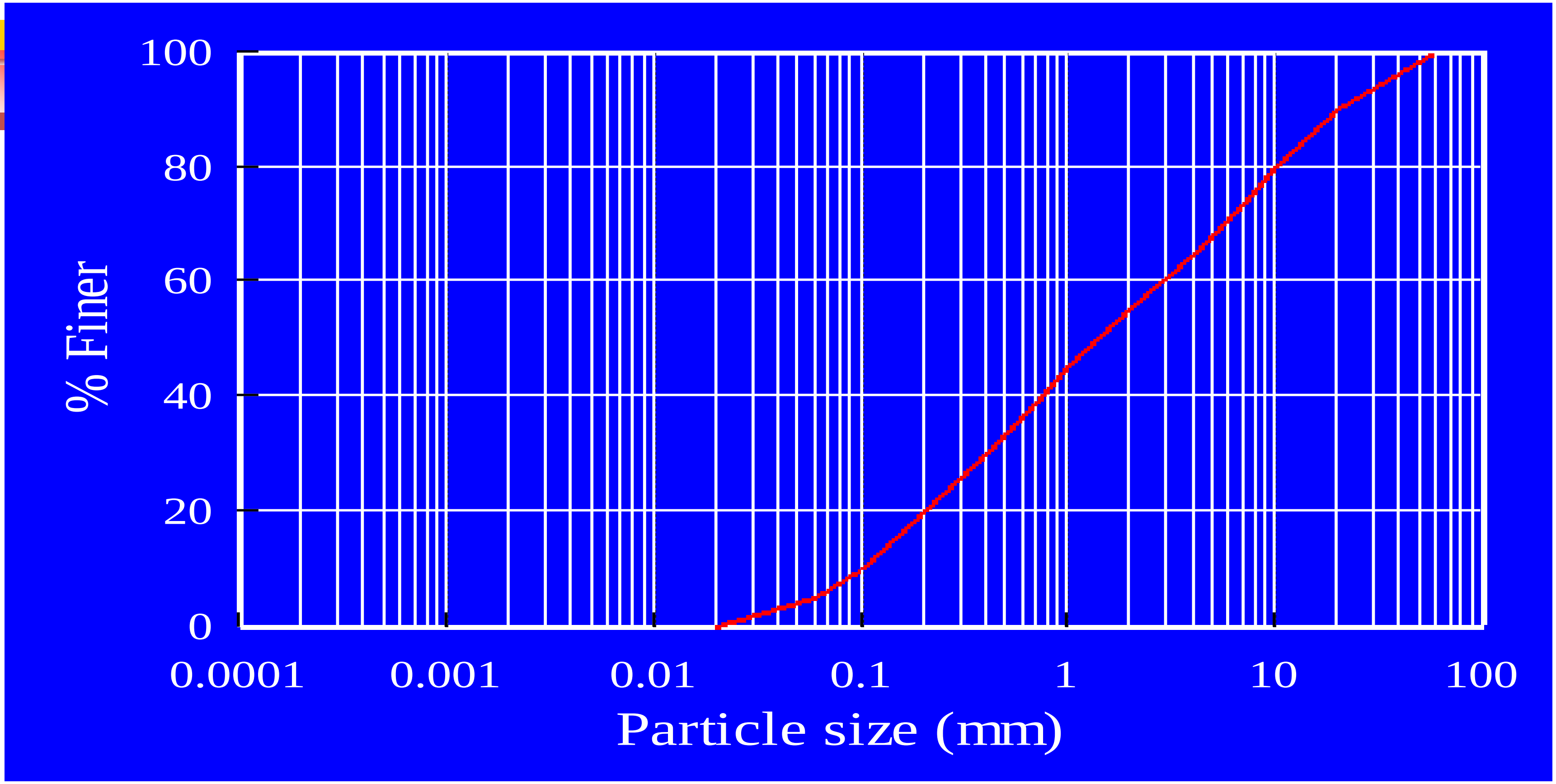
$$C_u = \frac{D_{60}}{D_{10}}$$

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

x% of the soil has particles smaller than D_x

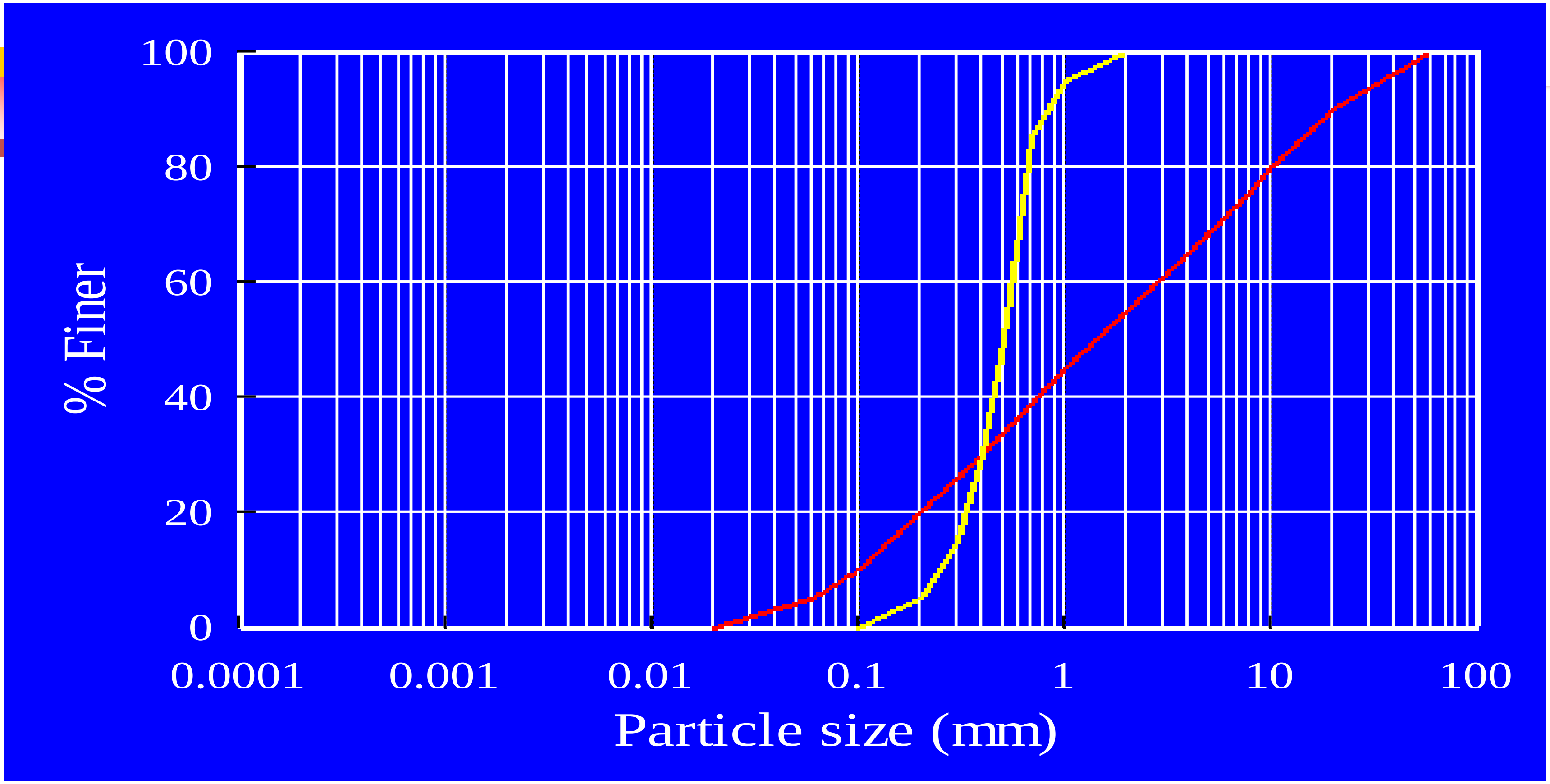


Grading curves



W Well graded

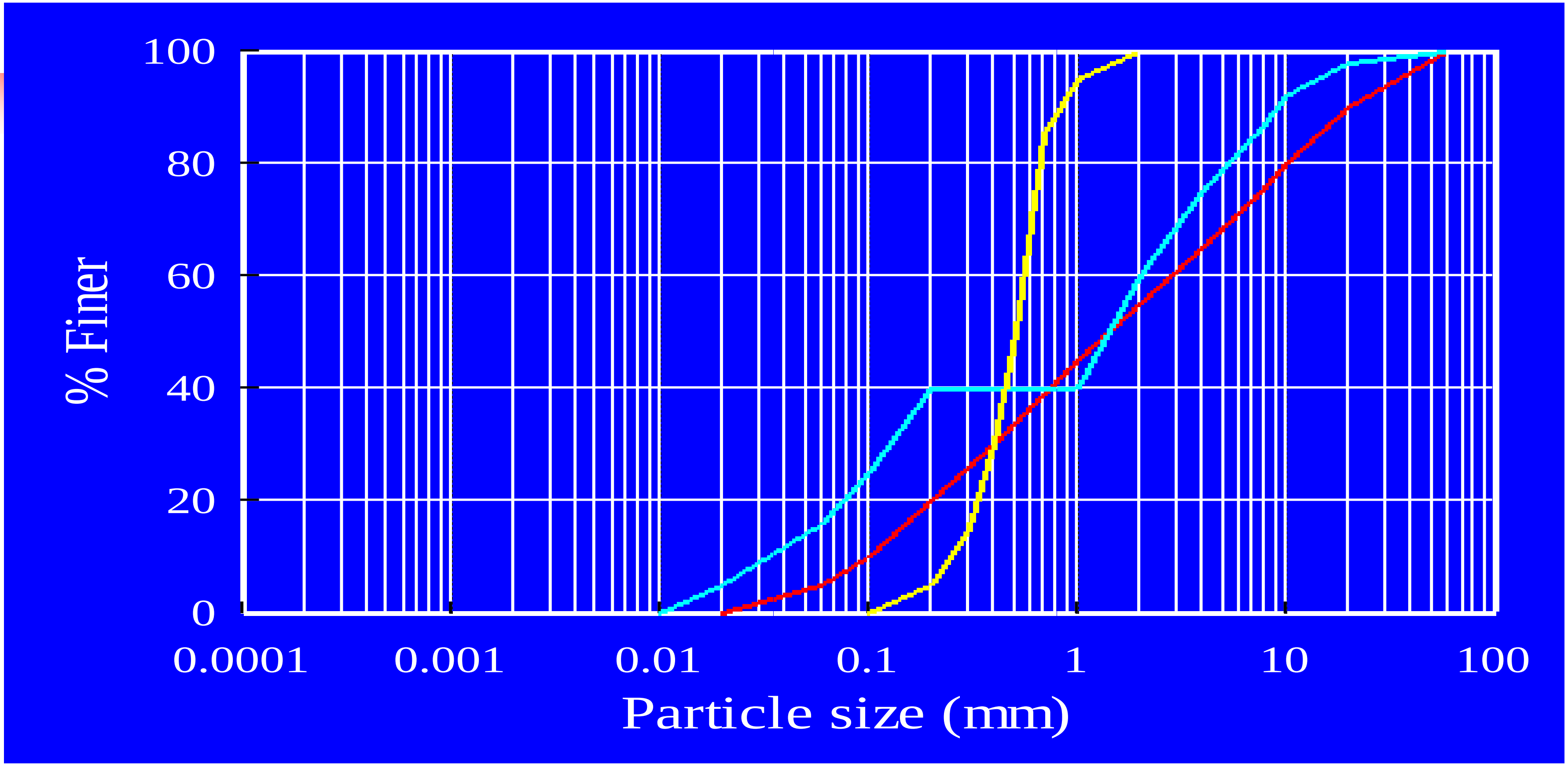
Grading curves



W Well graded

U Uniform

Grading curves

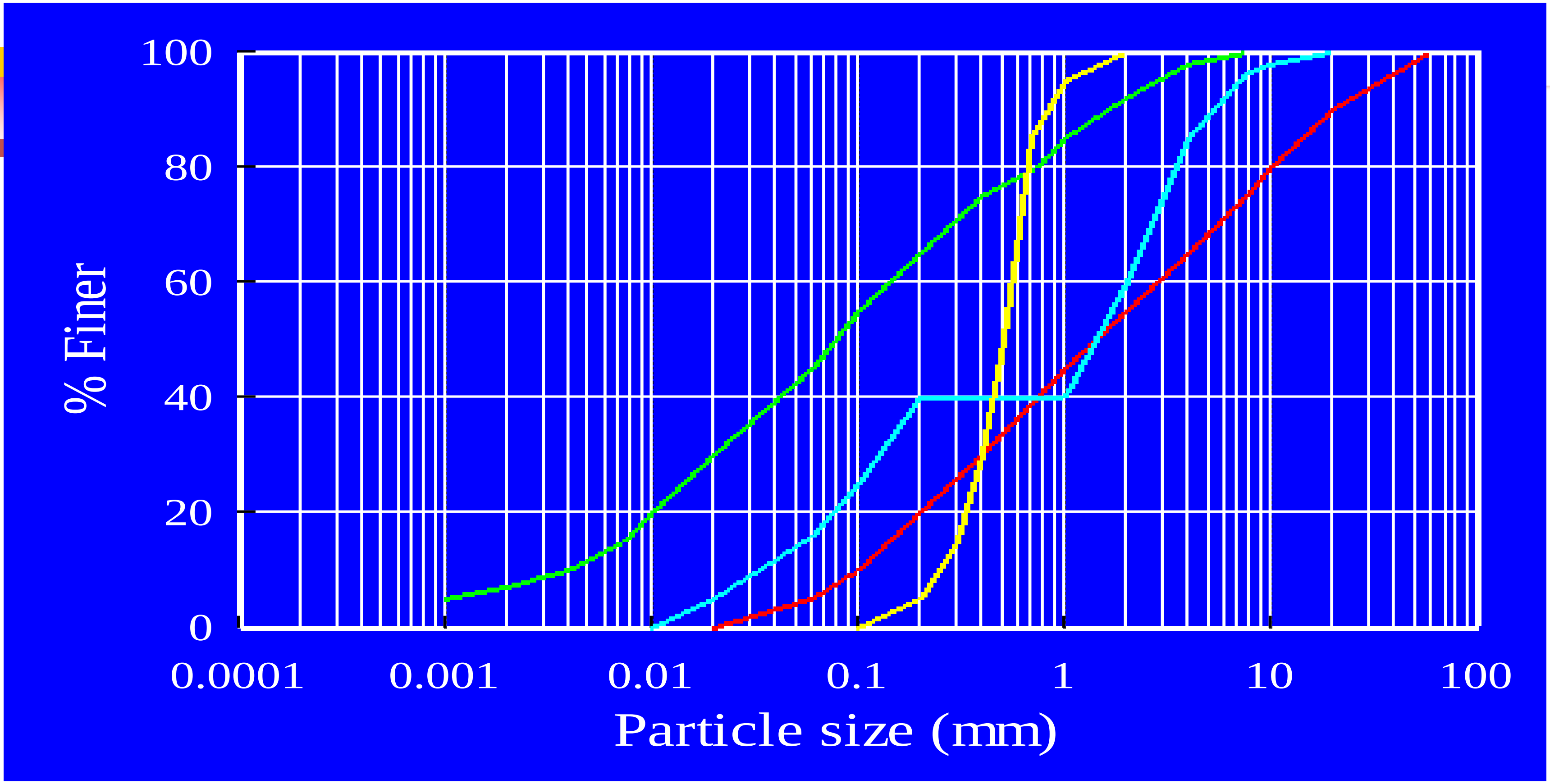


W Well graded

U Uniform

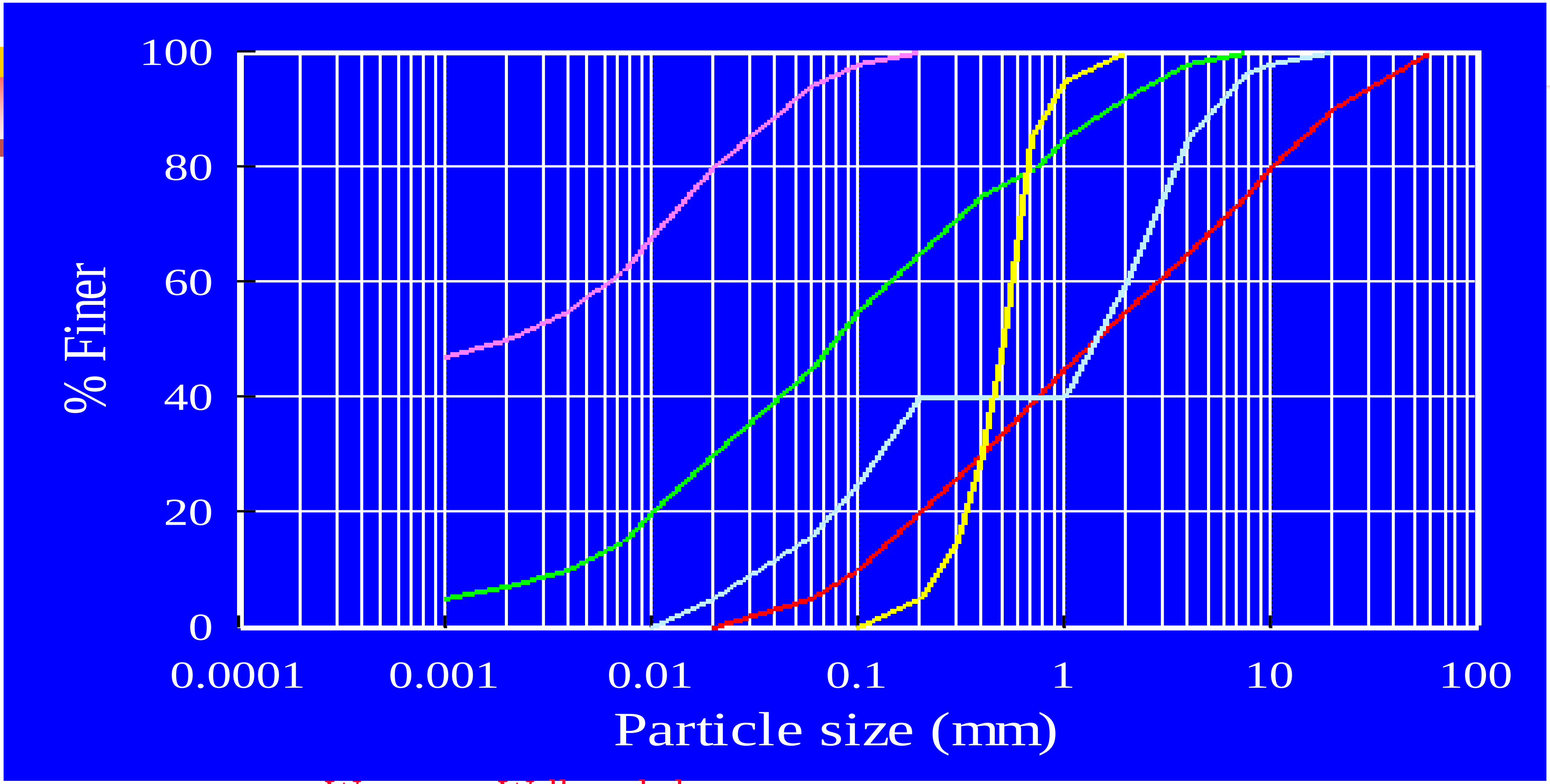
P Poorly graded

Grading curves

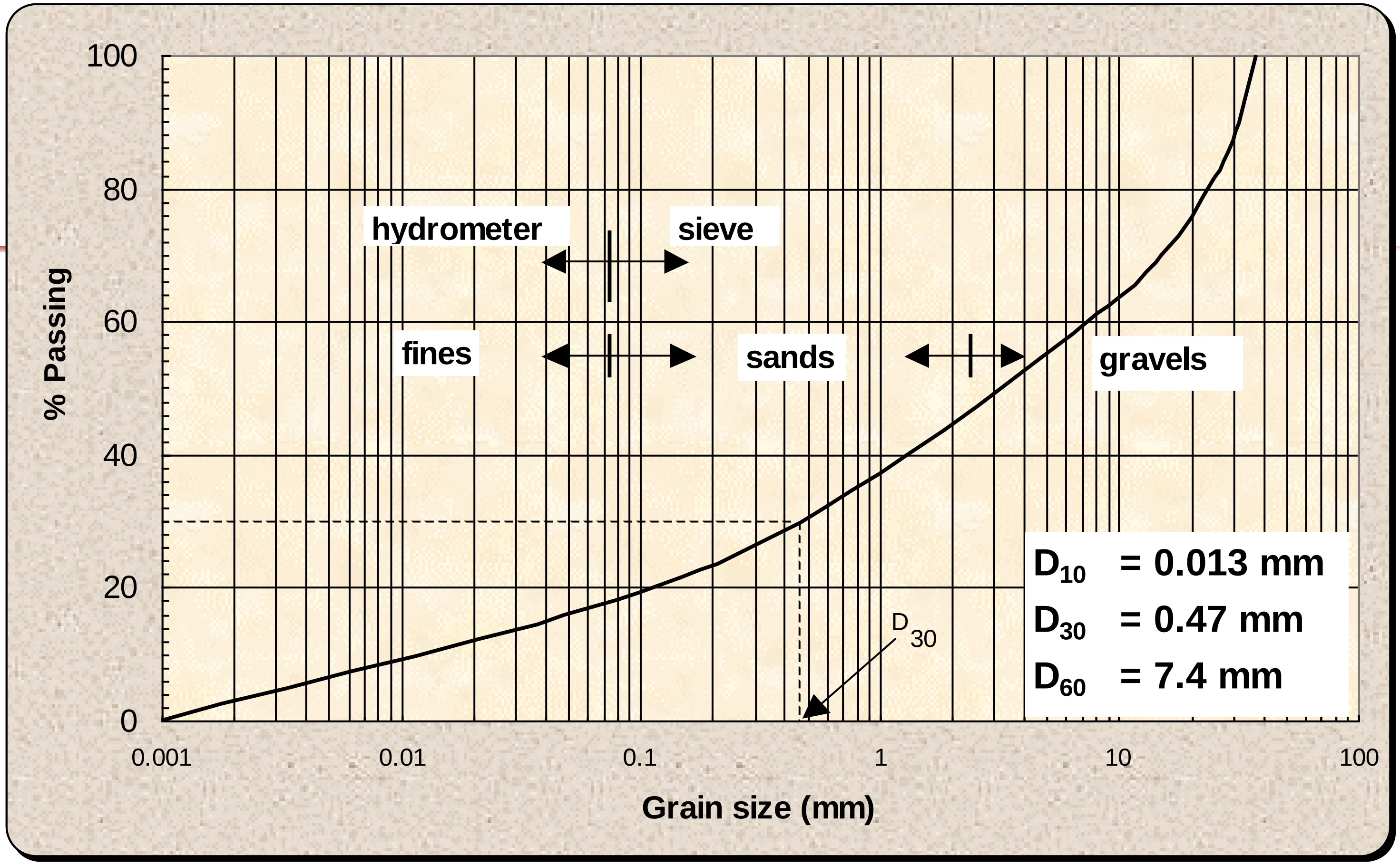


W	Well graded
U	Uniform
P	Poorly graded
C	Well graded with some clay

Grading curves



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



Grain Size Distribution Curve

- can find % of gravels, sands, fines

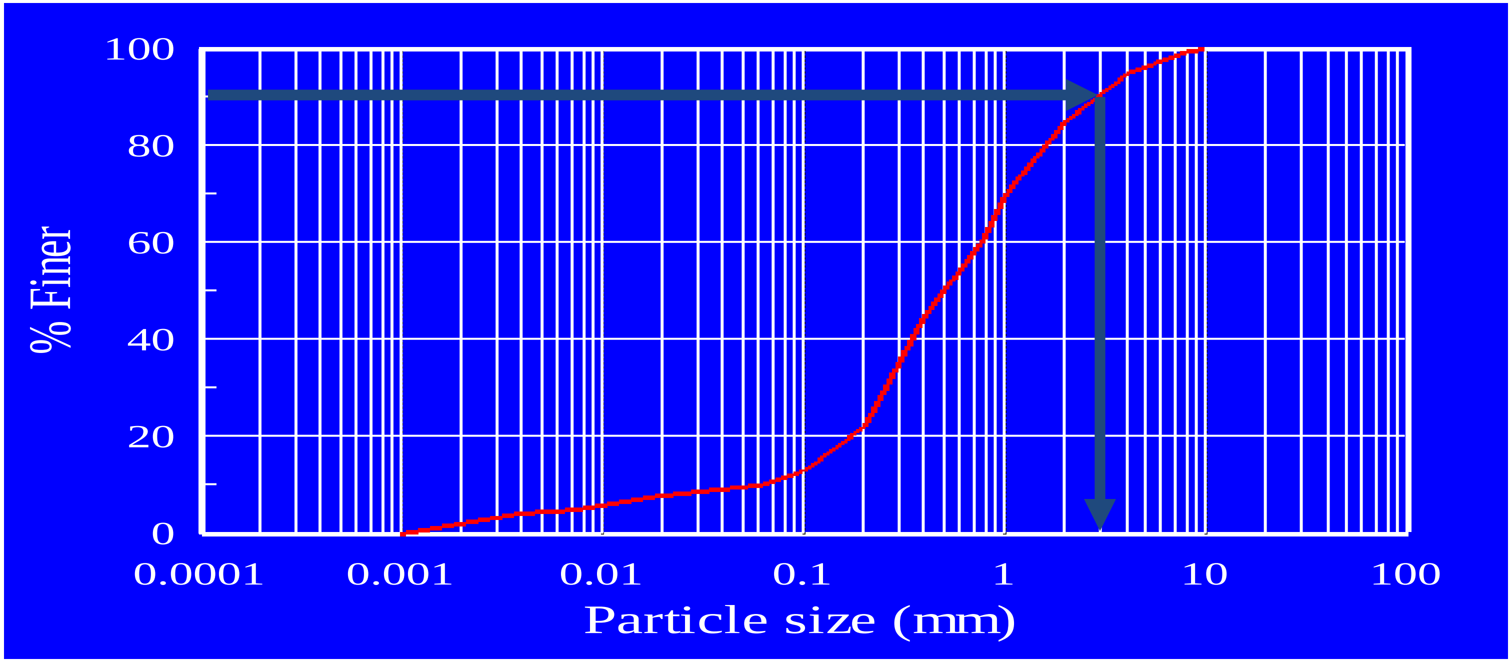
- define D_{10}, D_{30}, D_{60}.. as above.

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

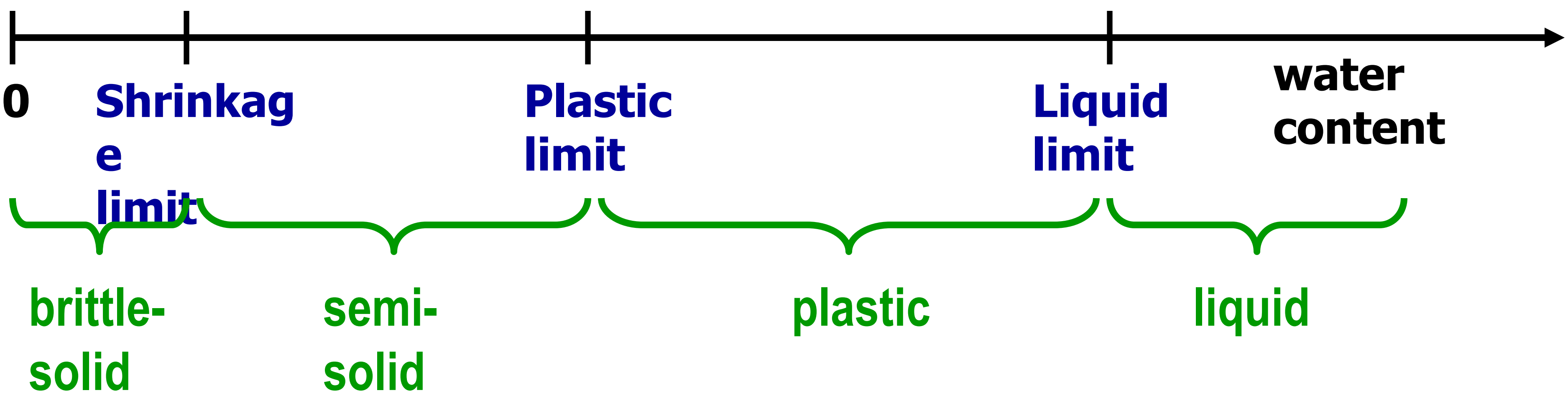
Poorly Graded Soils

Others, including two special cases:

(a) Uniform soils – grains of same size

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

- 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 limits are based on the moisture content of the soil.

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
Liquid limit device
Spatula
Grooving tool
Soil specimen
Moisture cans

Liquid Limit Definition

- The water content at which a soil changes from a plastic consistency to a liquid consistency
- Defined by Laboratory Test concept developed by Atterberg in 1911.

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):

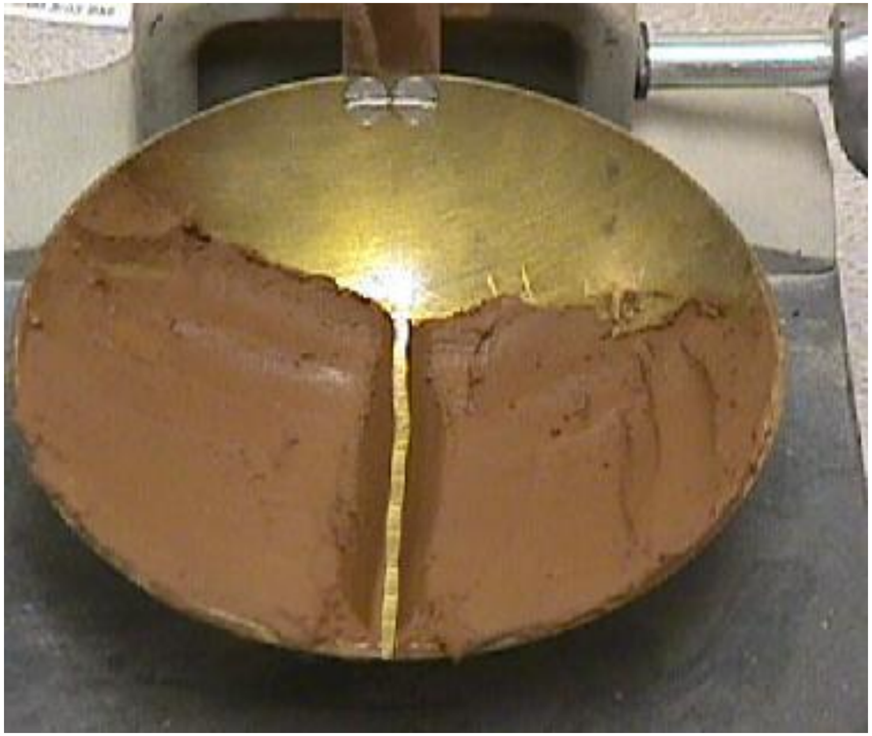
At w<SL, no volume reduction on drying

LL Test Procedure

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

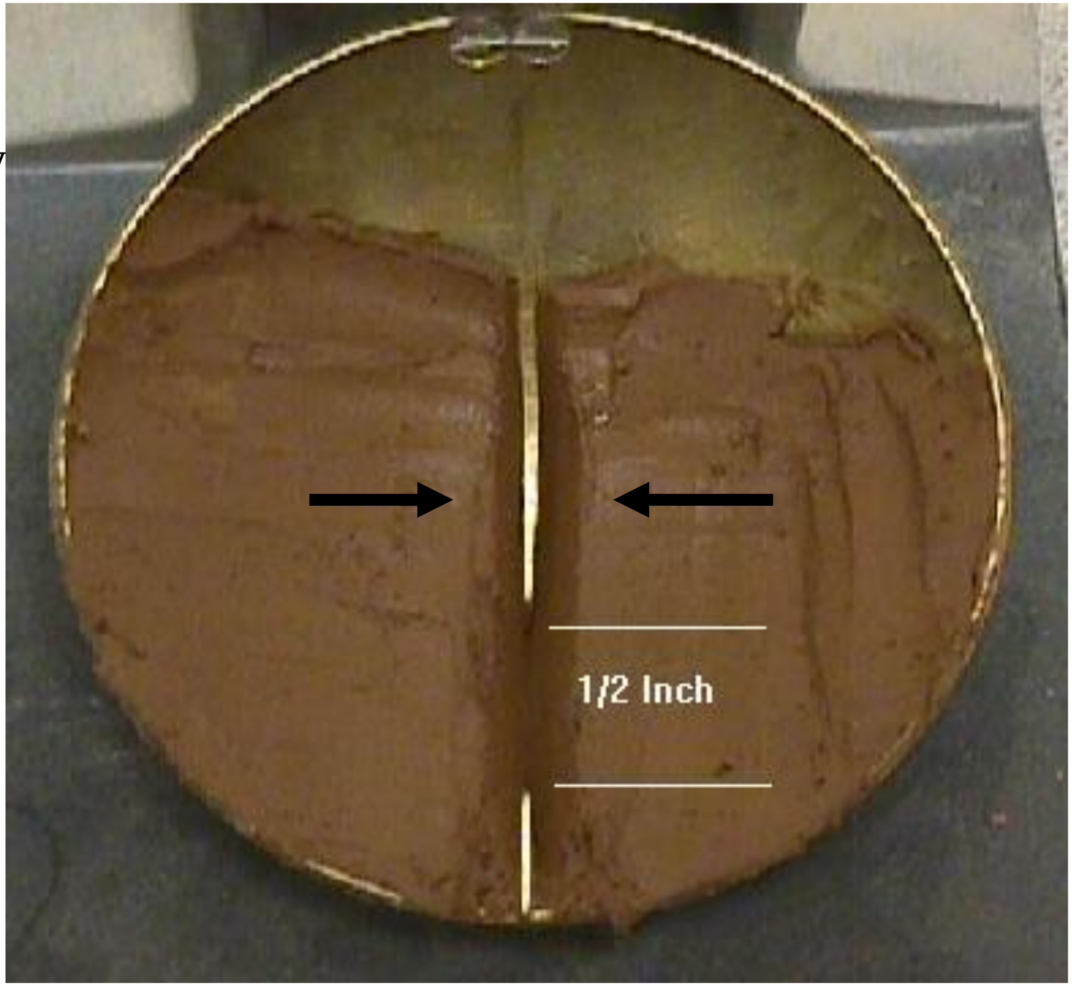
LL Test Procedure

- Cut groove in soil paste with standard grooving tool

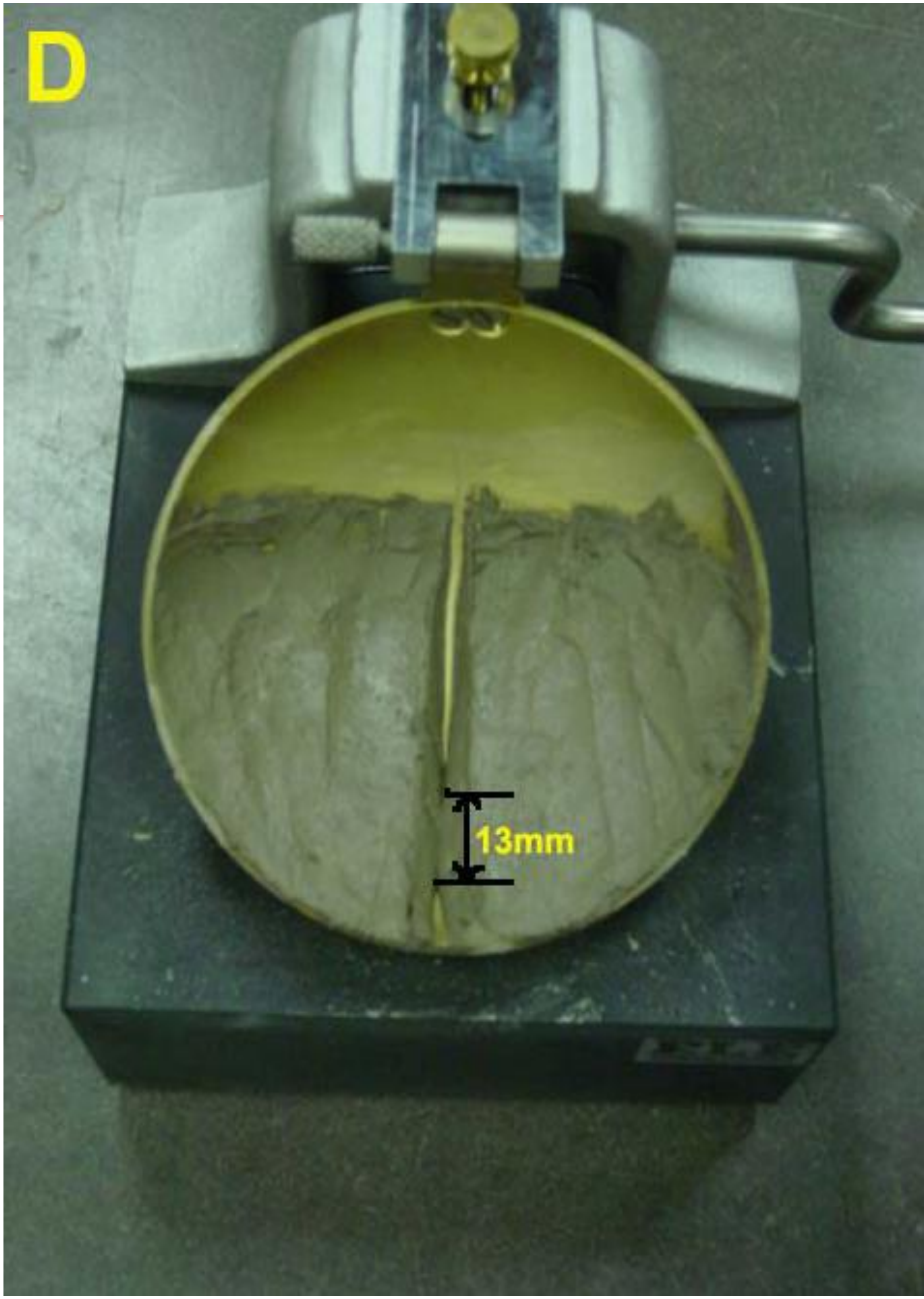


LL Test Procedure

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




C

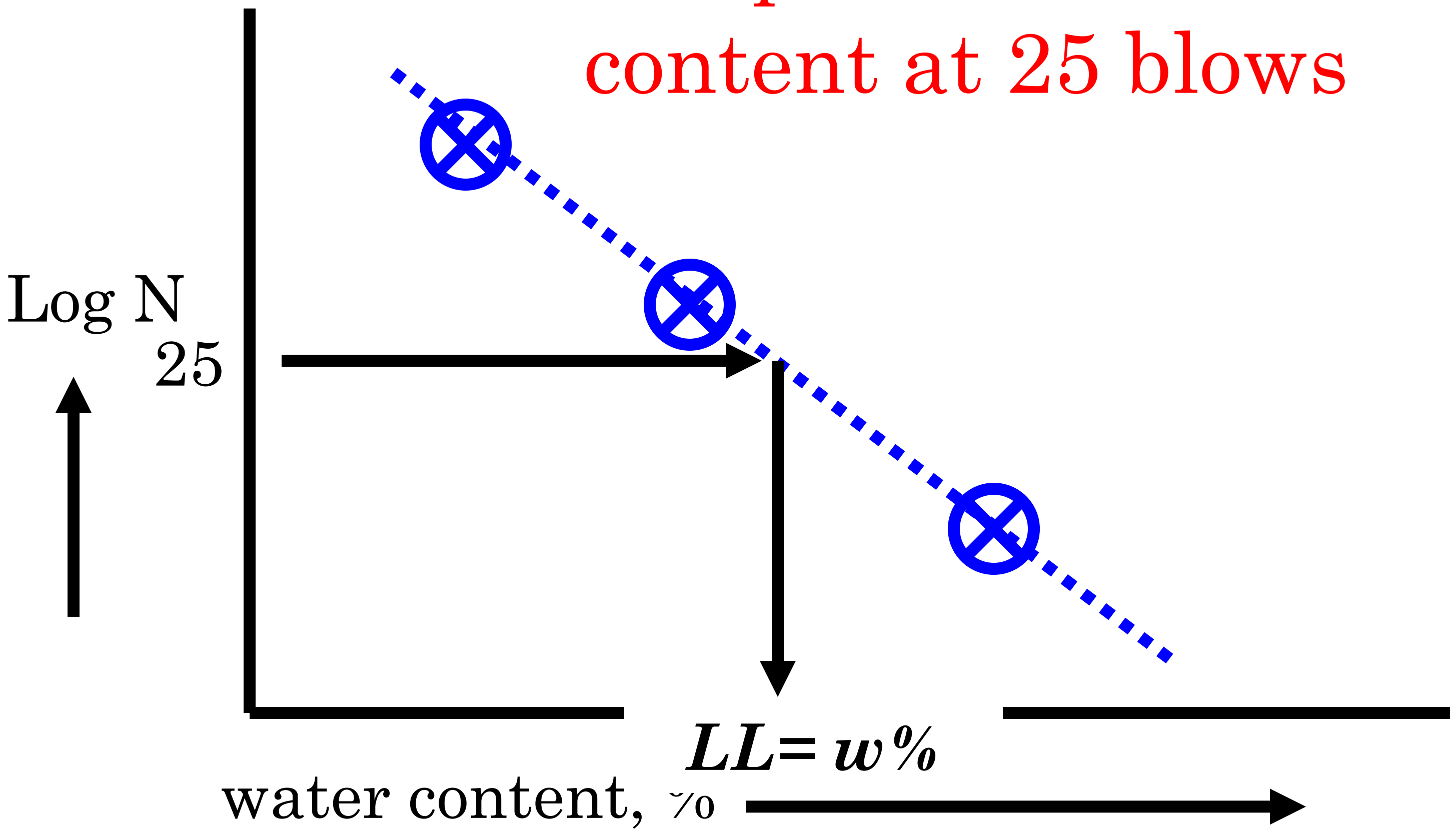

D
13mm

LL Test Procedure

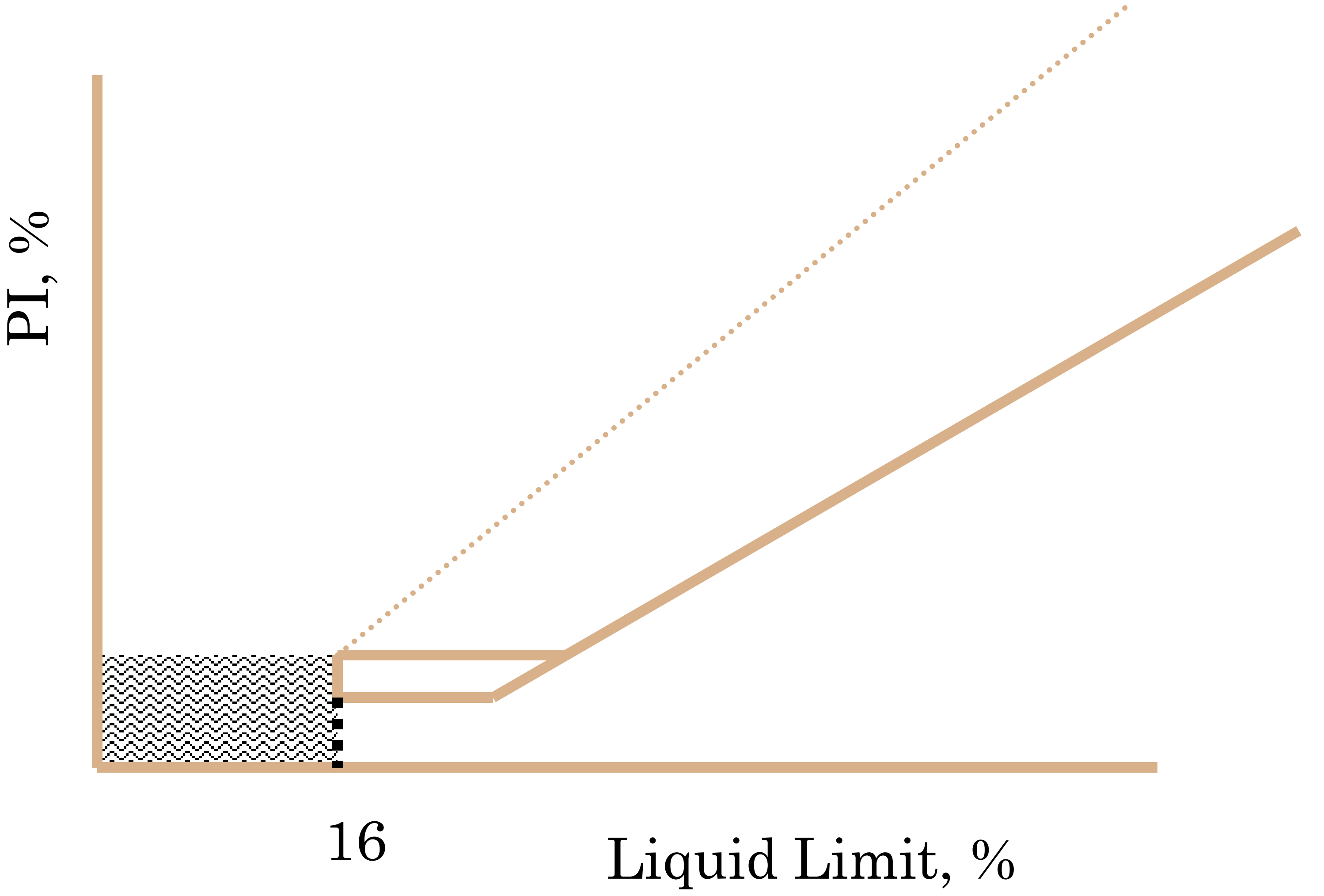
- Perform on 3 to 4 specimens that bracket 25 blows to close groove
- Obtain water content for each test
- Plot water content versus number of blows on semi-log paper

LL Test Results

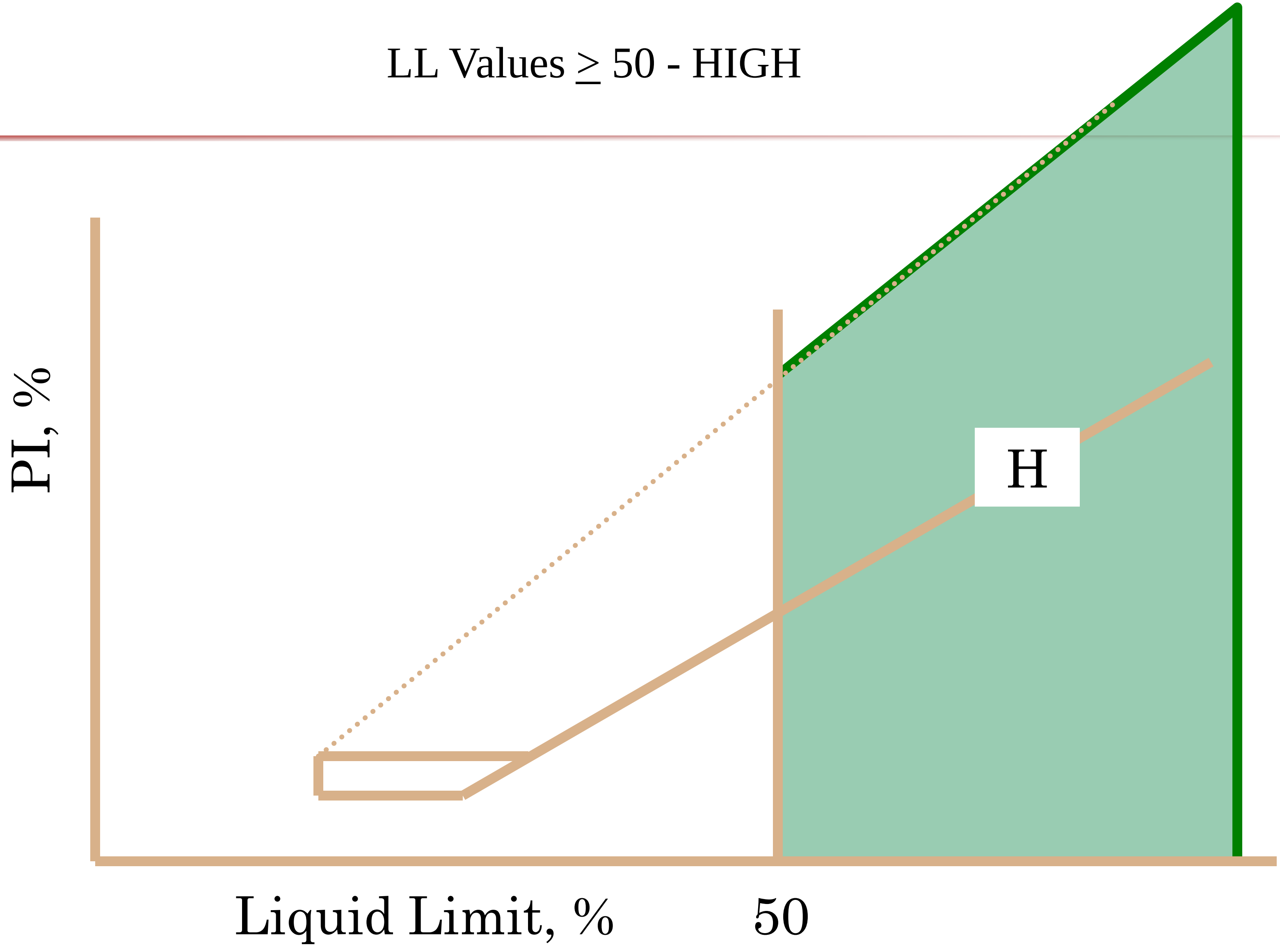
Interpolate LL water content at 25 blows



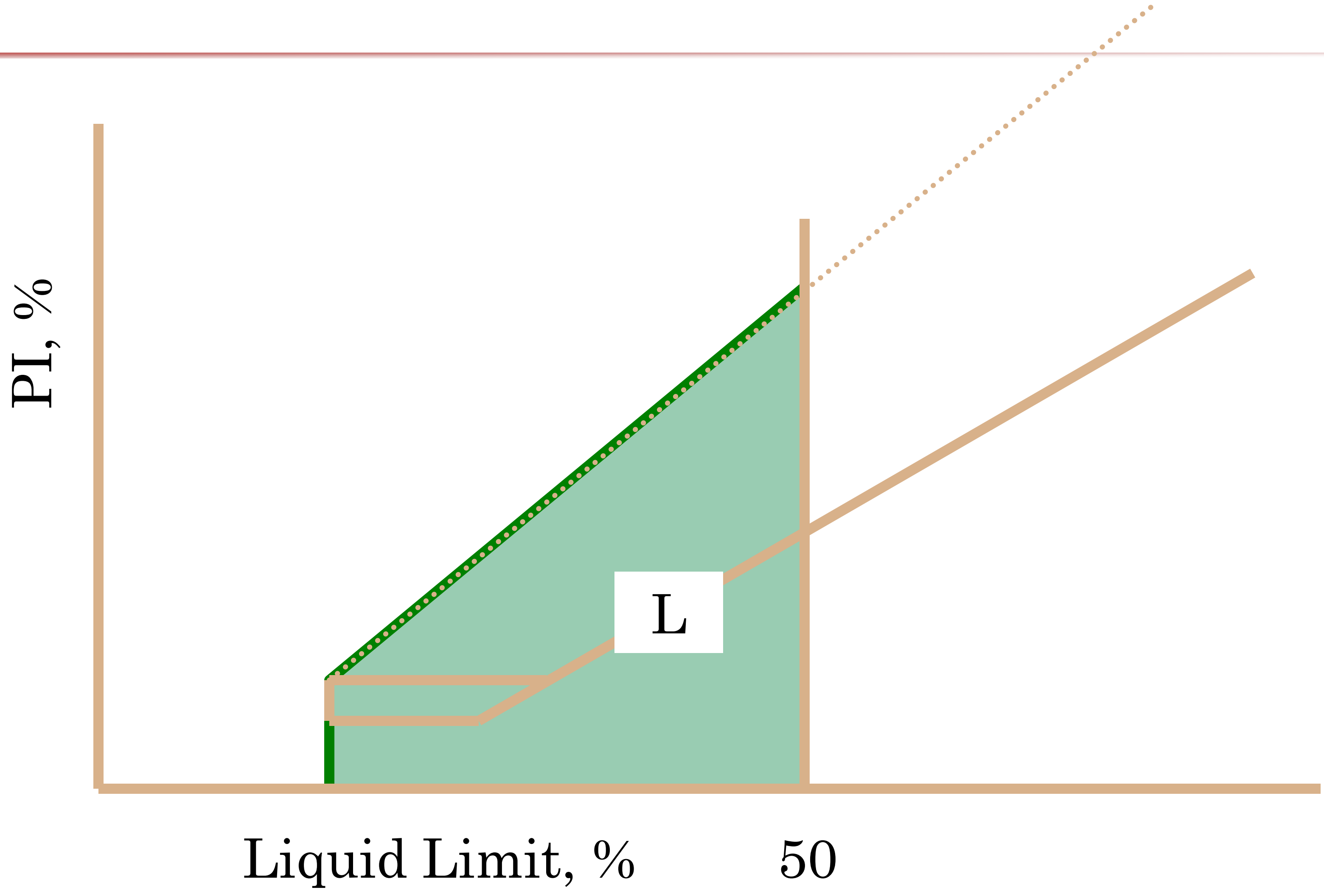
LL Values < 16 % not realistic



LL Values ≥ 50 - HIGH

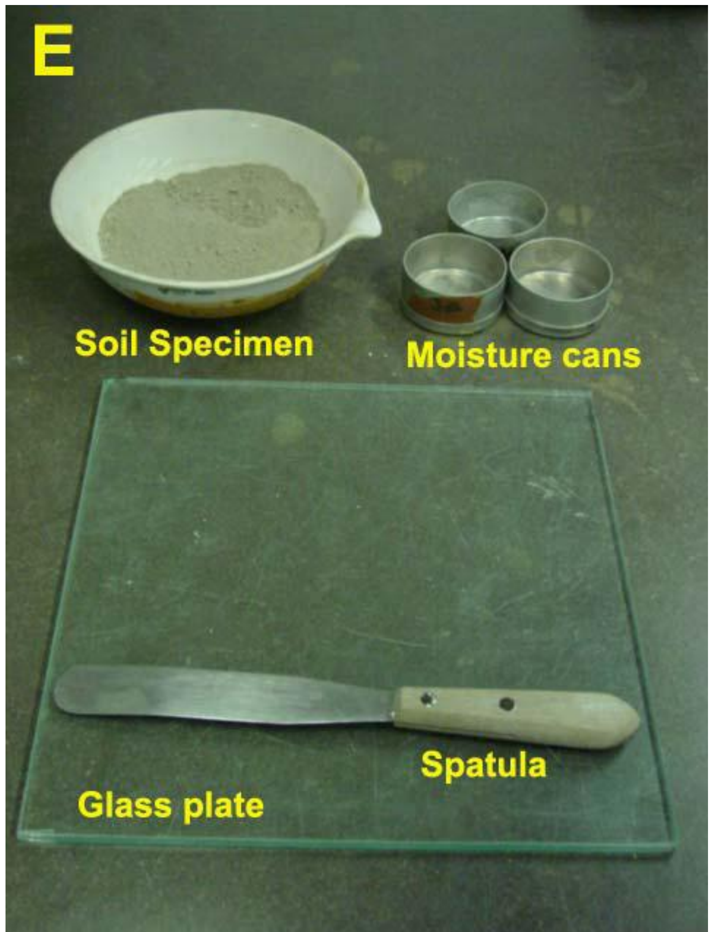


LL Values < 50 - LOW



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

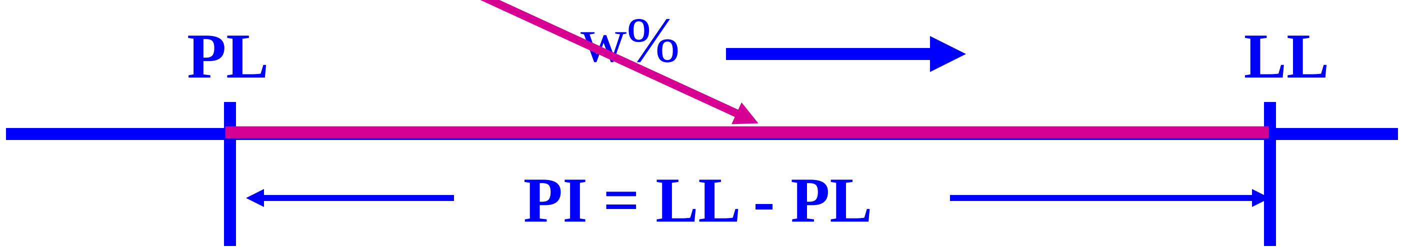

G

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

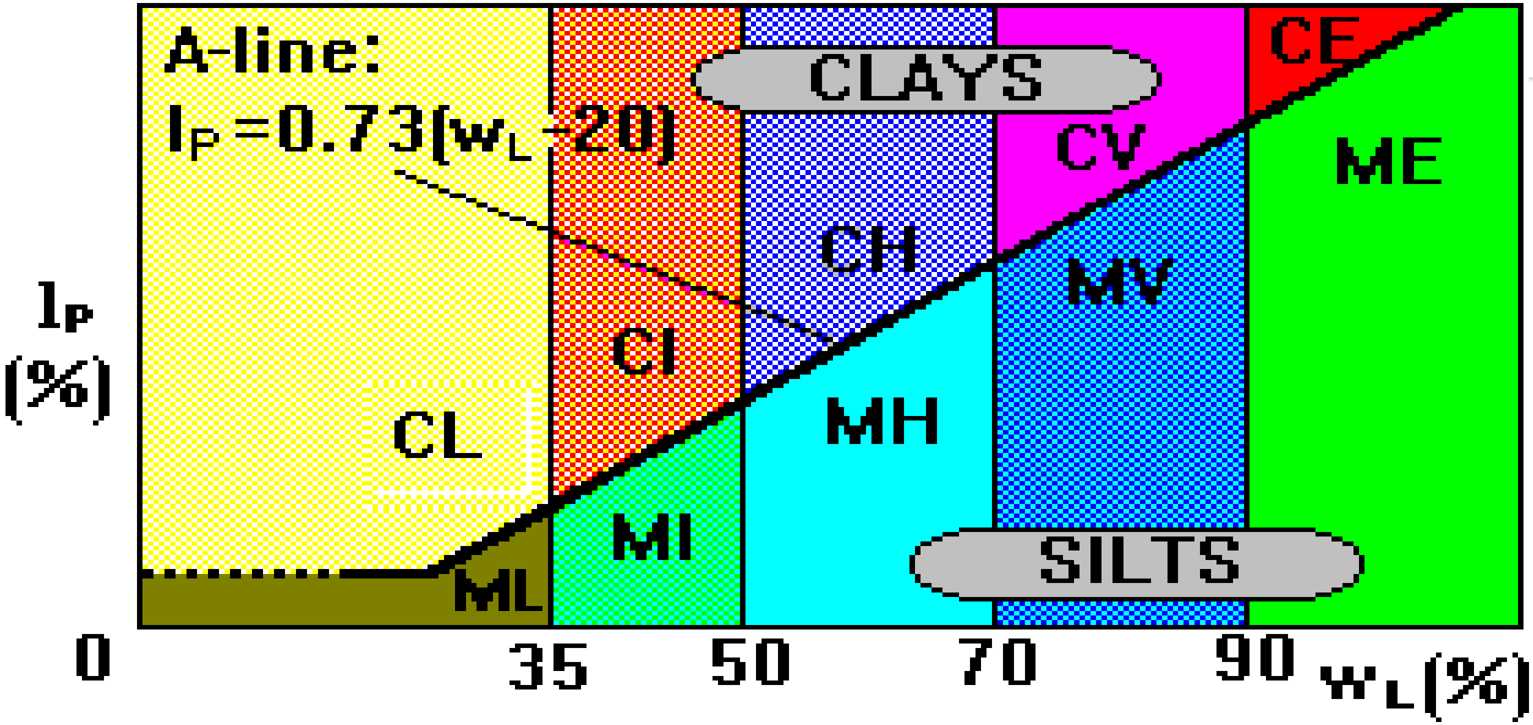
- Plasticity Index is the numerical difference between the Liquid Limit w% and the Plastic Limit w%

Plasticity Index = Liquid Limit – Plastic Limit



plastic (remoldable)

Plasticity Chart

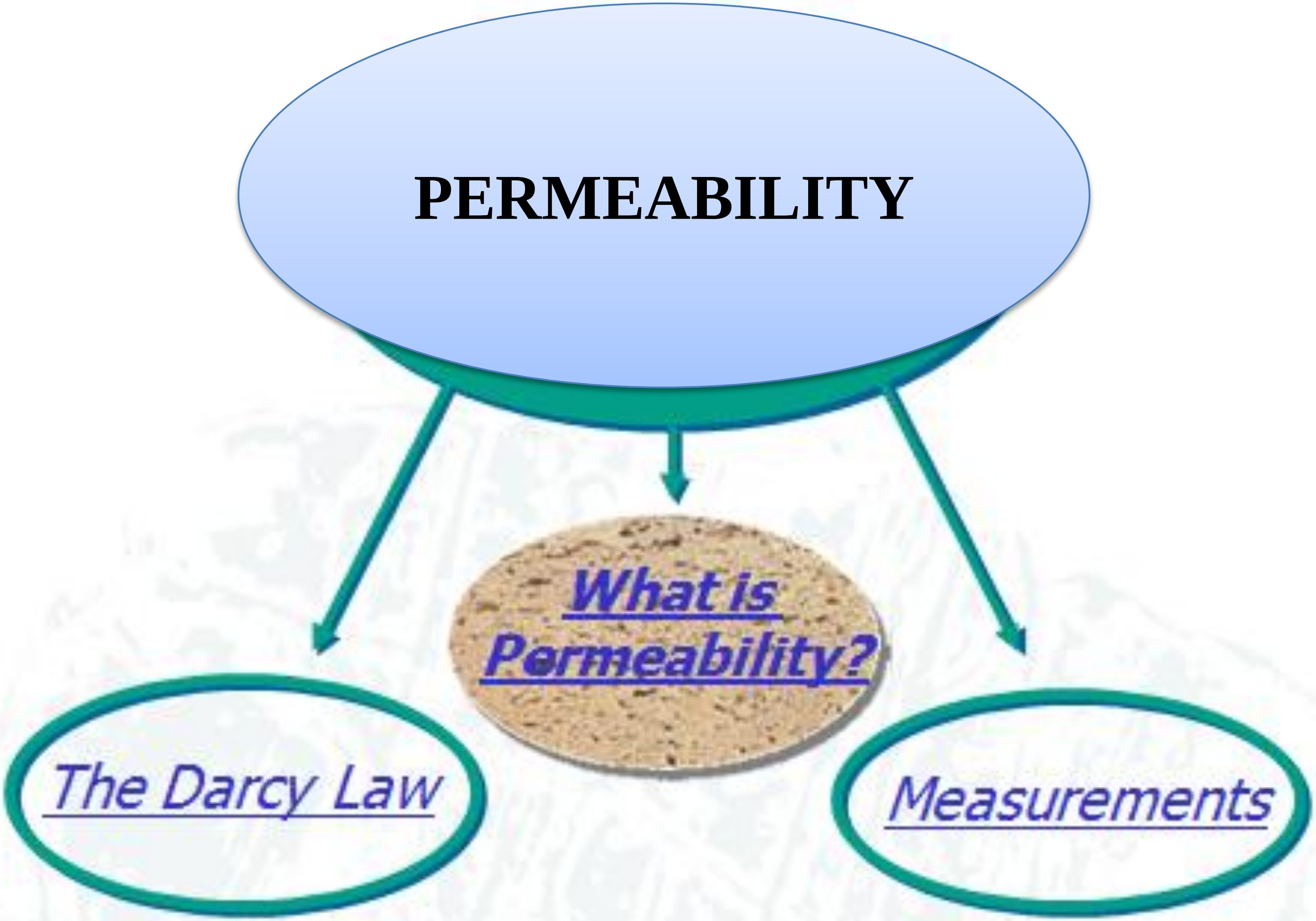


Low plasticity	w_L = < 35%
Intermediate plasticity	w_L = 35 - 50%
High plasticity	w_L = 50 - 70%
Very high plasticity	w_L = 70 - 90%
Extremely high plasticity	w_L = > 90%

UNIT II

PERMEABILITY, EFFECTIVE STRESS AND SEEPAGE THROUGH SOILS

Permeability



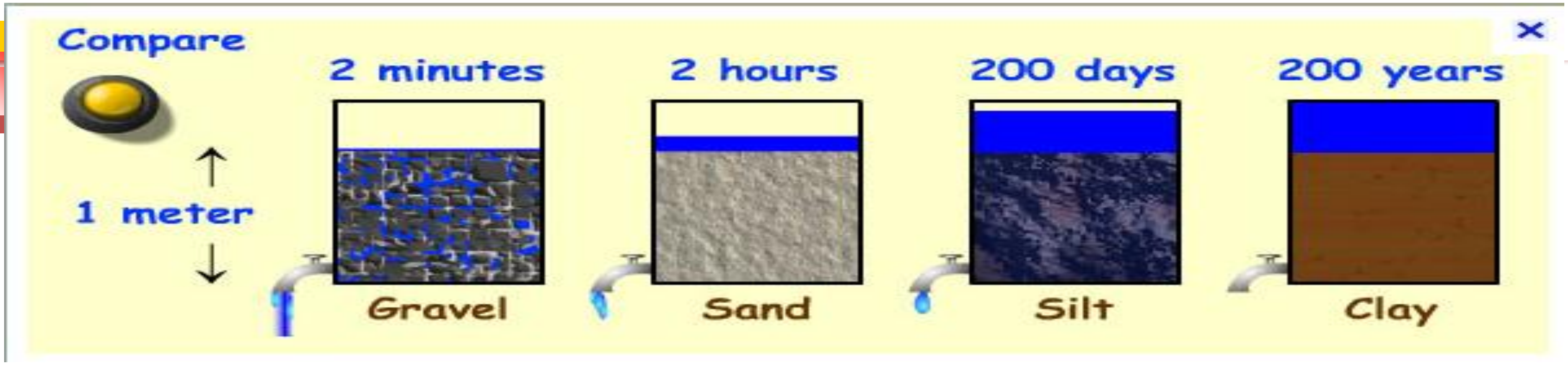
PERMEABILITY
What is Permeability?
The Darcy Law
Measurements

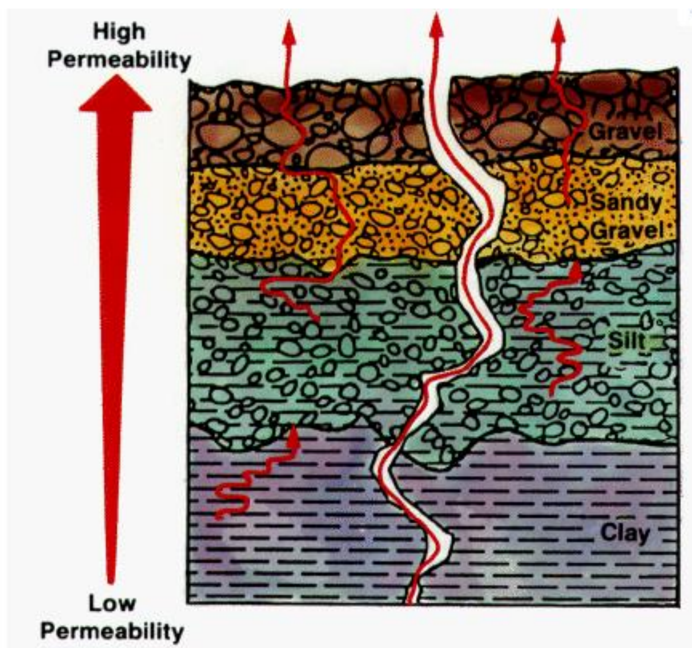
What is **permeability**?

- Property of a soil which permits the flow of water
- Permeability is defined as the property of a porous material which permits the passage or seepage of water through its interconnecting voids.
- It is a very important Engineering property

gravels → highly permeable

stiff clay → least permeable


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


High
Permeability
Low
Permeability
Gravel
Sandy
Gravel
Silt
Clay

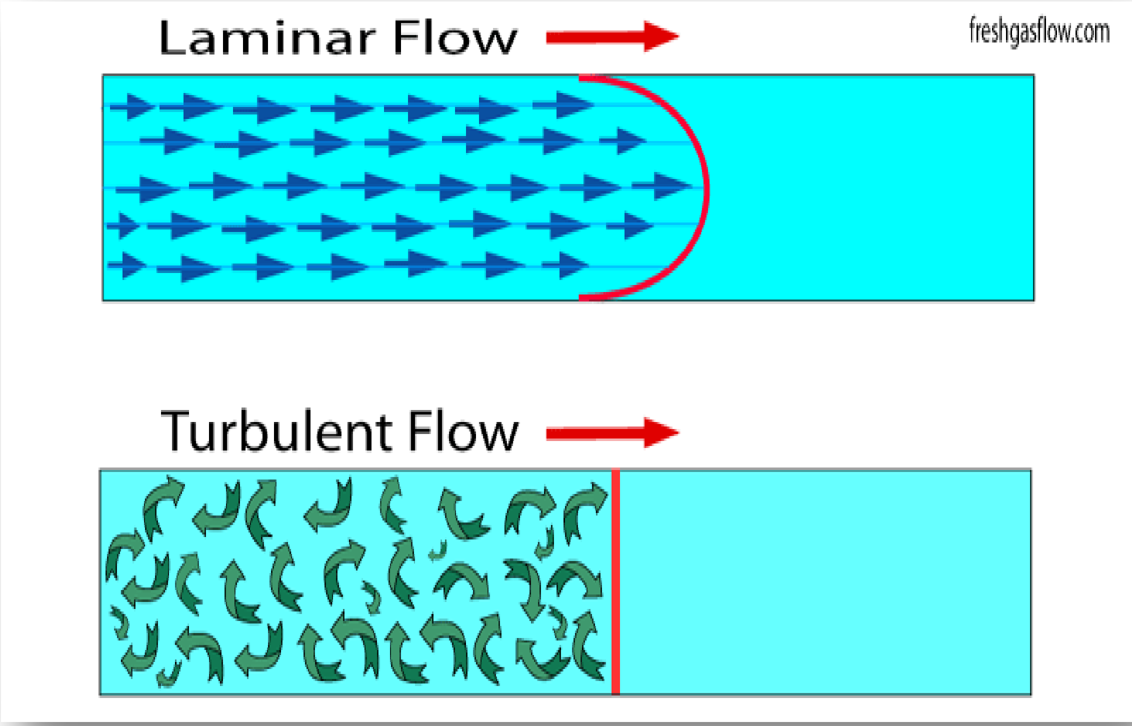
Permeability through soil is important for the following engineering problems:

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

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

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

Paths are irregular and twisting, crossing at random



Coefficient Of Permeability

Depends not only on the properties of soil but also on the properties of water

Absolute permeability

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

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

Henry Darcy (1803-1858), Hydraulic Engineer. His law is a foundation stone for several fields of study

Darcy's Law

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

$$q = vA$$

$$v = ki$$

$$q = kiA$$

Validity of darcy's law - When flow is laminar

Bernouli's Equation:

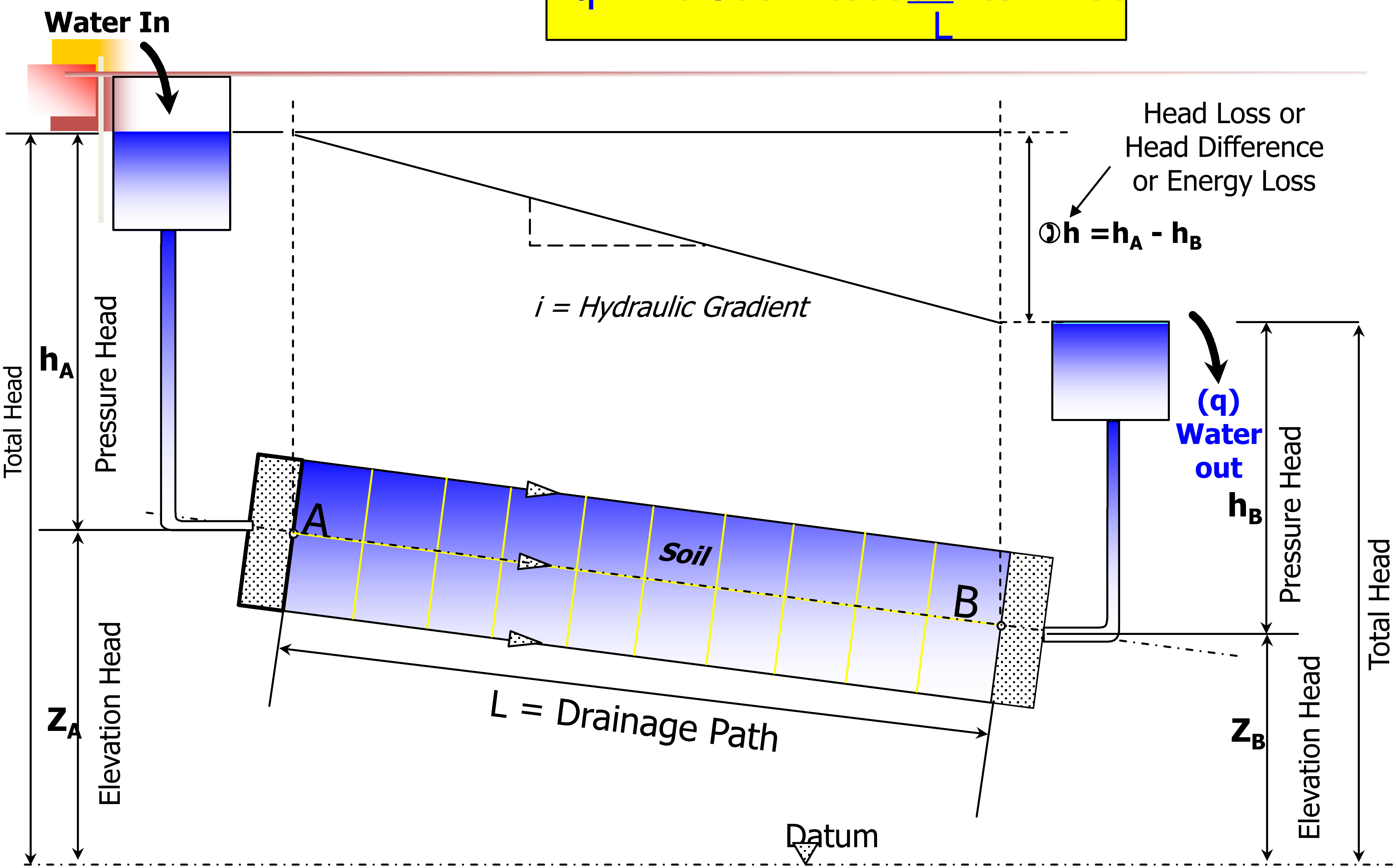
- Total Energy = Elevation Energy + Pressure Energy + Velocity Energy

- Total Head = Elevation Head + Pressure Head + Velocity Head

$$H = z + \frac{v^2}{2g} + \frac{p}{\rho g}$$

- Total head of water in soil engineering problems is equal to the sum of the elevation head and the pressure head

$$q = v \,.\, A = k\, i\, A = k \frac{\Delta h}{L} A$$



Factors Affecting Permeability

- Particle size
- Structure of soil mass
- Shape of particles
- Void ratio
- Properties of water
- Degree of saturation
- Adsorbed water
- Impurities in water

Constant Head Permeability Test

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

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

K = Coefficient of permeability

Q = total quantity of water

t = time

L = Length of the coarse soil

Variable head permeability test

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

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

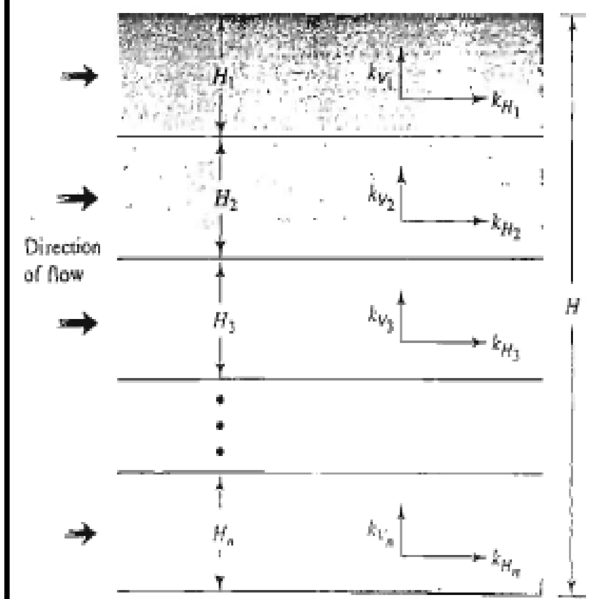
t = time
L = Length of the fine soil
A = cross section area of soil
a= cross section area of tube
K = Coefficient of permeability

Flow parallel to the plans of stratification

$$q = kiA$$

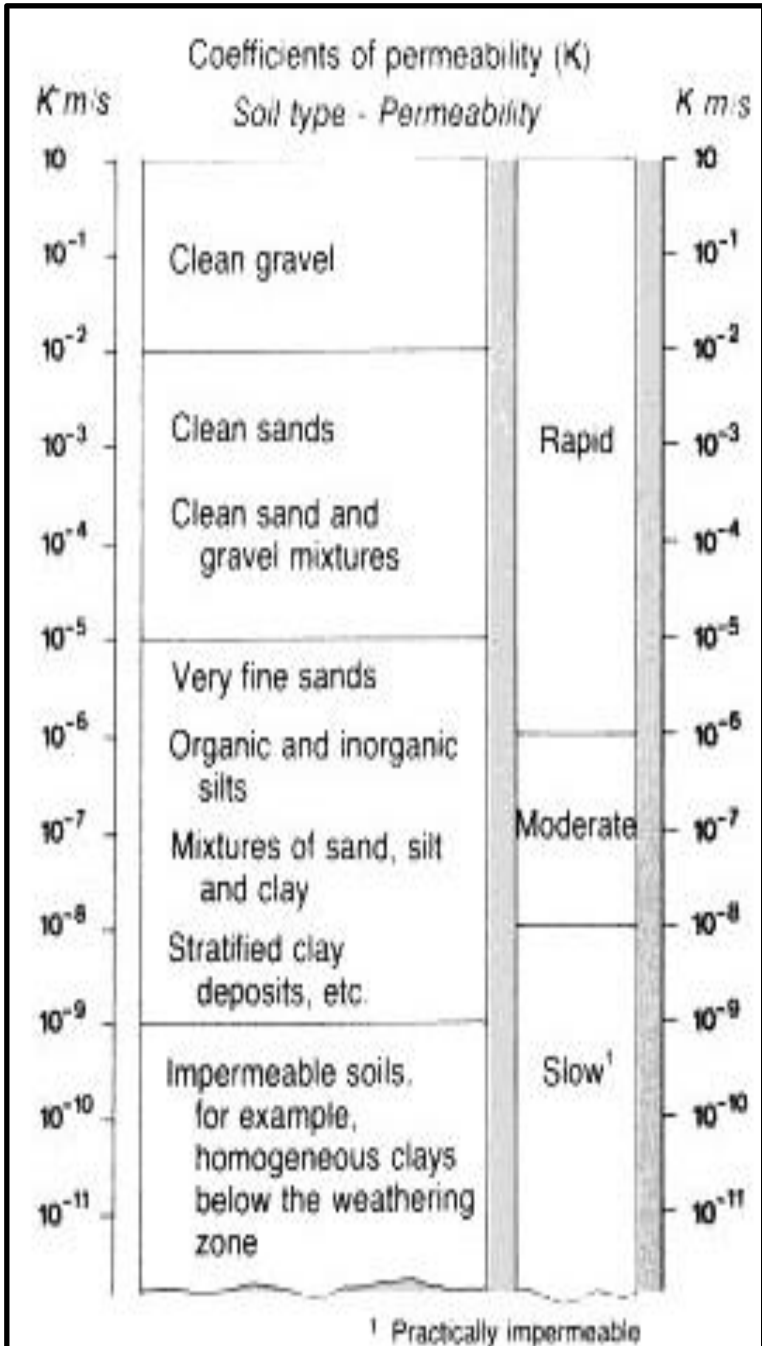
$$q = k_x iH = (k_1 H_1 + k_2 H_2 + \cdots k_n H_n)i$$

$$k_x = \frac{k_1 H_1 + k_2 H_2 + \cdots k_n H_n}{H}$$



Flow normal to the plans of stratification

$$\boxed{k_y = \frac{H}{\dfrac{H_1}{k_1} + \dfrac{H_2}{k_2} + \cdots + \dfrac{H_n}{k_n}}}$$



Soil Permeability Classes

Permeability is commonly measured in terms of the rate of water flow through the soil in a given period of time.

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

UNIT III

STRESS DISTRIBUTION IN SOILS AND COMPACTION

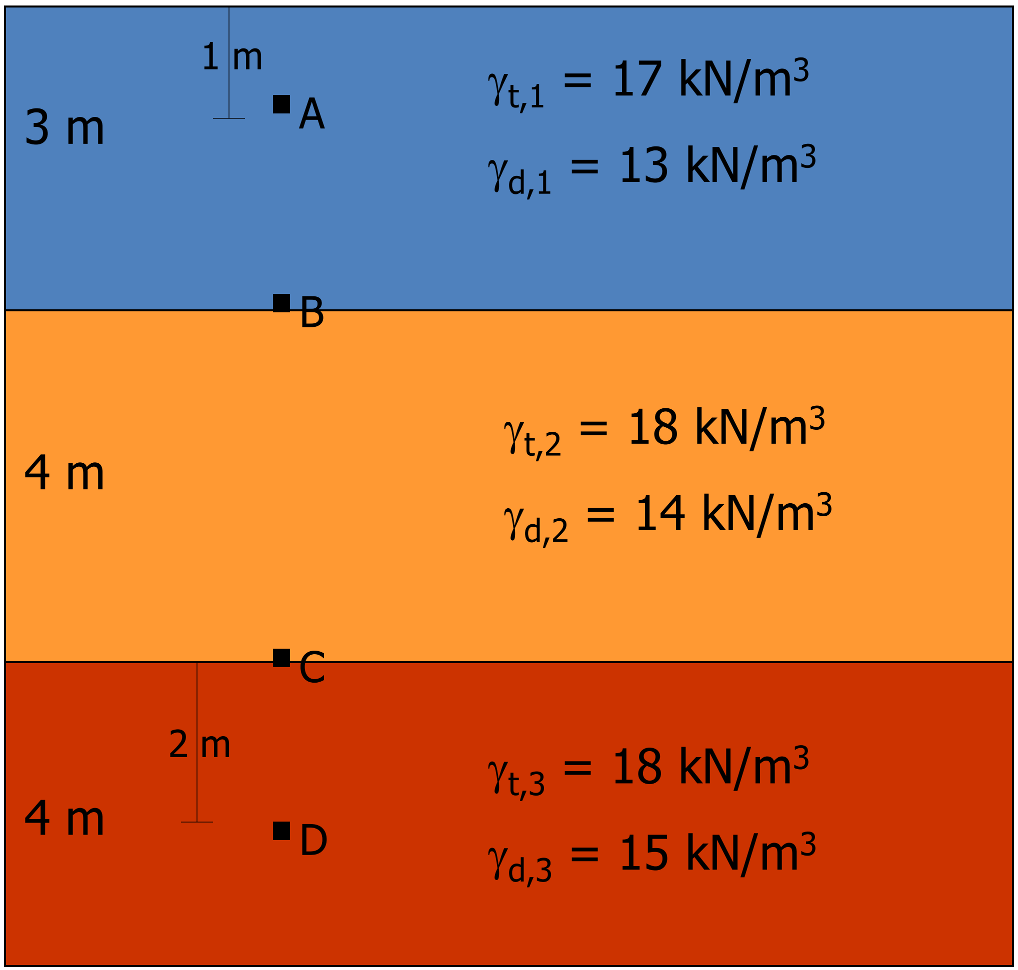
TOTAL STRESS

- Generated by the mass in the soil body, calculated by sum up the unit weight of all the material (soil solids + water) multiflied by soil thickness or depth.
- Denoted as σ, σ_v, Po
- The unit weight of soil is in natural condition and the water influence is ignored.

$$\sigma = \sum \gamma_t . z$$

z = The depth of point

EXAMPLE



$\sigma_A = \gamma_{t,1}$ x 1 m

$= 17$ kN/m^2

$\sigma_B = \gamma_{t,1}$ x 3 m

$= 51$ kN/m^2

$\sigma_C = \gamma_{t,1}$ x 3 m + $\gamma_{t,2}$ x 4 m

$= 123$ kN/m^2

$\sigma_D = \gamma_{t,1}$ x 3 m + $\gamma_{t,2}$ x 4 m + $\gamma_{t,3}$ x 2 m

$= 159$ kN/m^2

EFFECTIVE STRESS

- Defined as soil stress which influenced by water pressure in soil body.
- Published first time by Terzaghi at 1923 base on the experimental result
- Applied to saturated soil and has a relationship with two type of stress i.e.:
  - Total Normal Stress (σ)
  - Pore Water Pressure (u)
- Effective stress formula

$$\boldsymbol{\sigma' = \sigma - u}$$

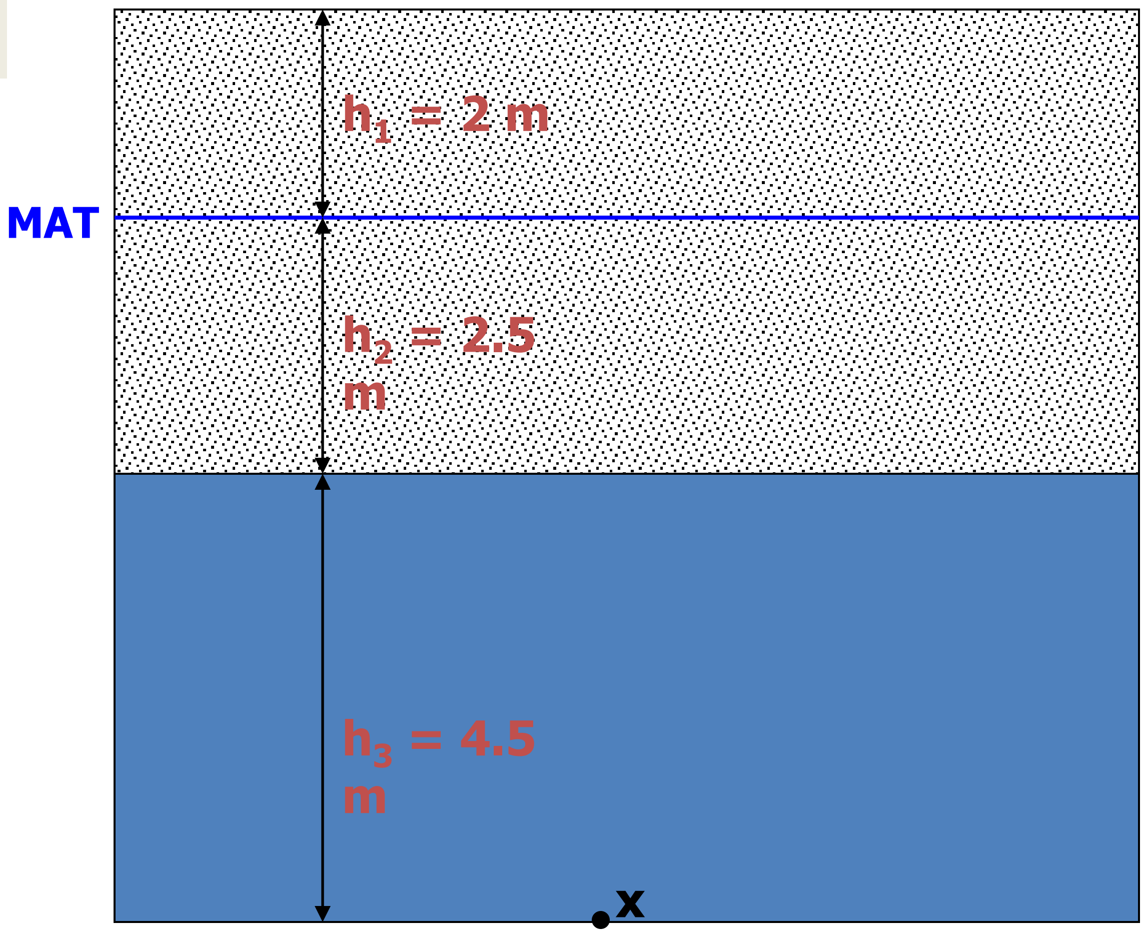
EFFECTIVE STRESS

$$\sigma' = \sigma - u$$

$$\sigma = \gamma_t . z \qquad\qquad u = \gamma_w . z$$

$$\sigma' = (\gamma_t - \gamma_w).z = \gamma' . z$$

EXAMPLE



Sand

$\gamma_t = 18.0 \text{ kN/m}^3$

$\gamma_d = 13.1 \text{ kN/m}^3$

Clay

$\gamma_t = 19.80 \text{ kN/m}^3$

EXAMPLE

- Total Stress

  $\sigma = \gamma_{d,1} \cdot h_1 + \gamma_{t,1} \cdot h_2 + \gamma_{t,2} \cdot h_3$

  $\sigma = 13.1 \cdot 2 + 18 \cdot 2.5 + 19.8 \cdot 4.5$

  $= 160.3\ kN/m^2$

- Pore Water Pressure

  $u = \gamma_w \cdot (h_2+h_3)$

  $u = 10 \cdot 7$

  $= 70\ kN/m^2$

- Effective Stress

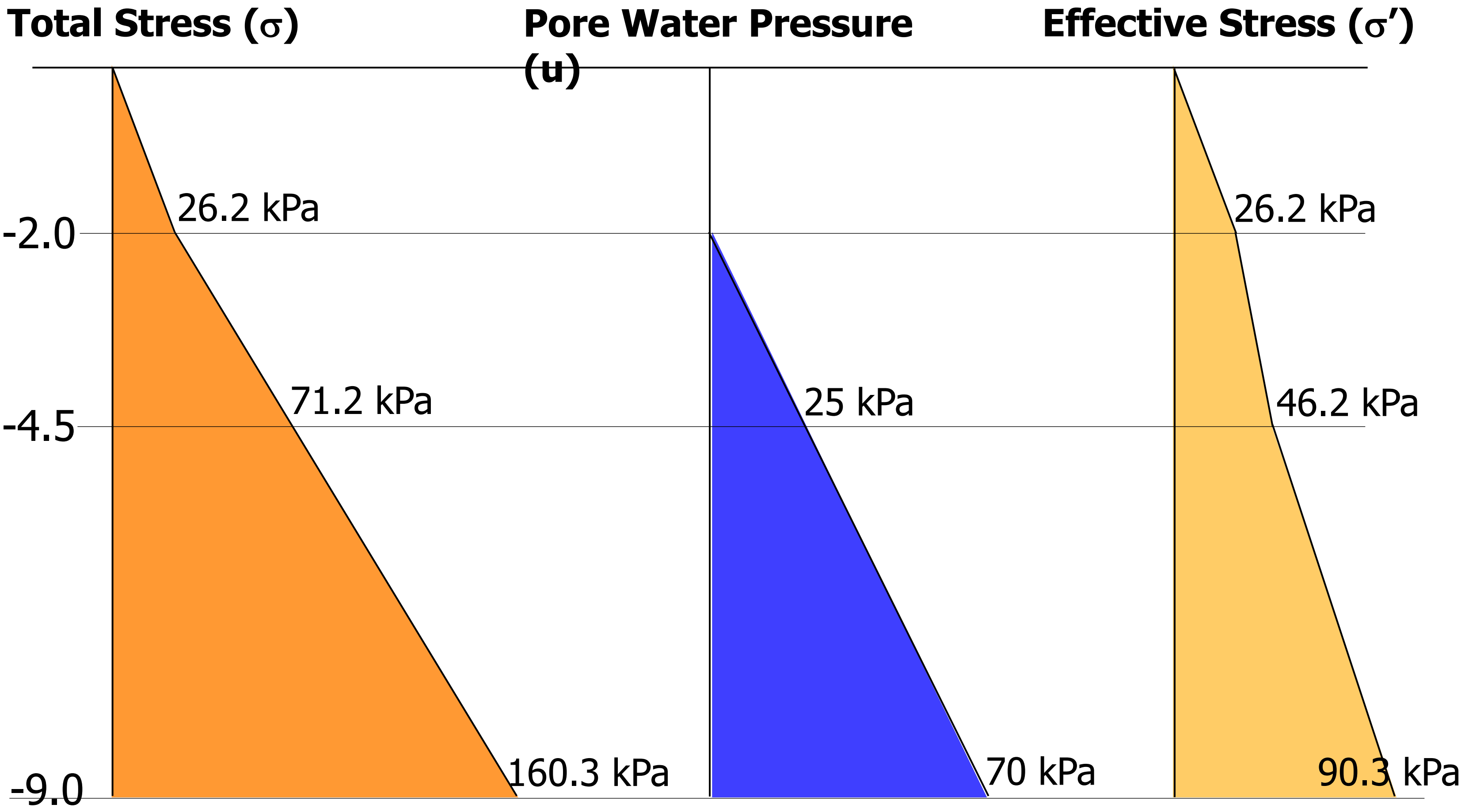
  $\sigma' = \sigma - u = 90.3\ kN/m^2$

$\boldsymbol{\sigma' = \gamma_{d,1} \cdot h_1 + (\gamma_{t,2} - \gamma_w) \cdot h_2 + (\gamma_{t,2} - \gamma_w) \cdot h_3}$

$\boldsymbol{\sigma' = 13.1 \cdot 2 + (18-10) \cdot 2.5 + (19{,}8-10) \cdot 4.5}$

$\boldsymbol{= 90.3\ kN/m^2}$

EXAMPLE

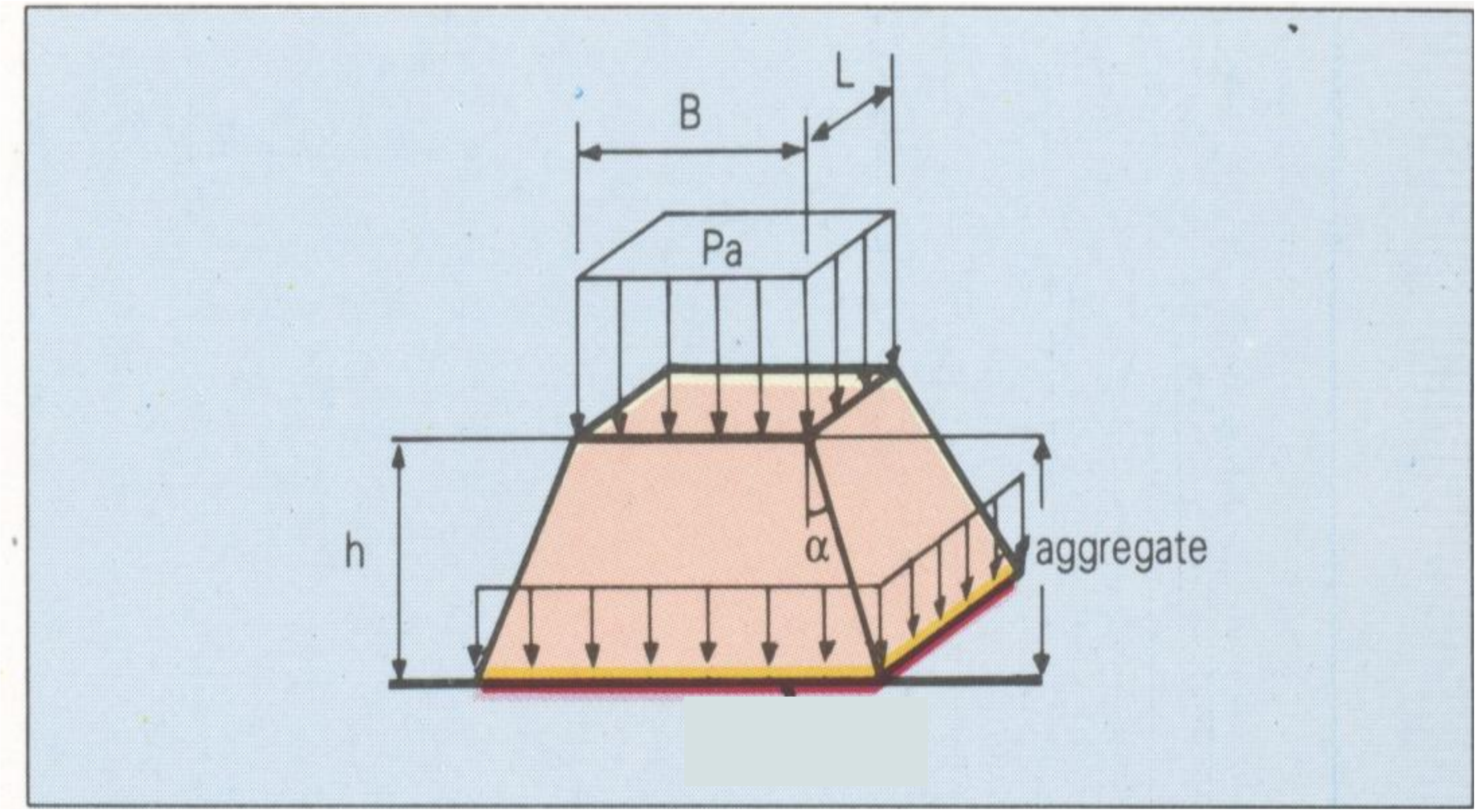


Profile of Vertical Stress

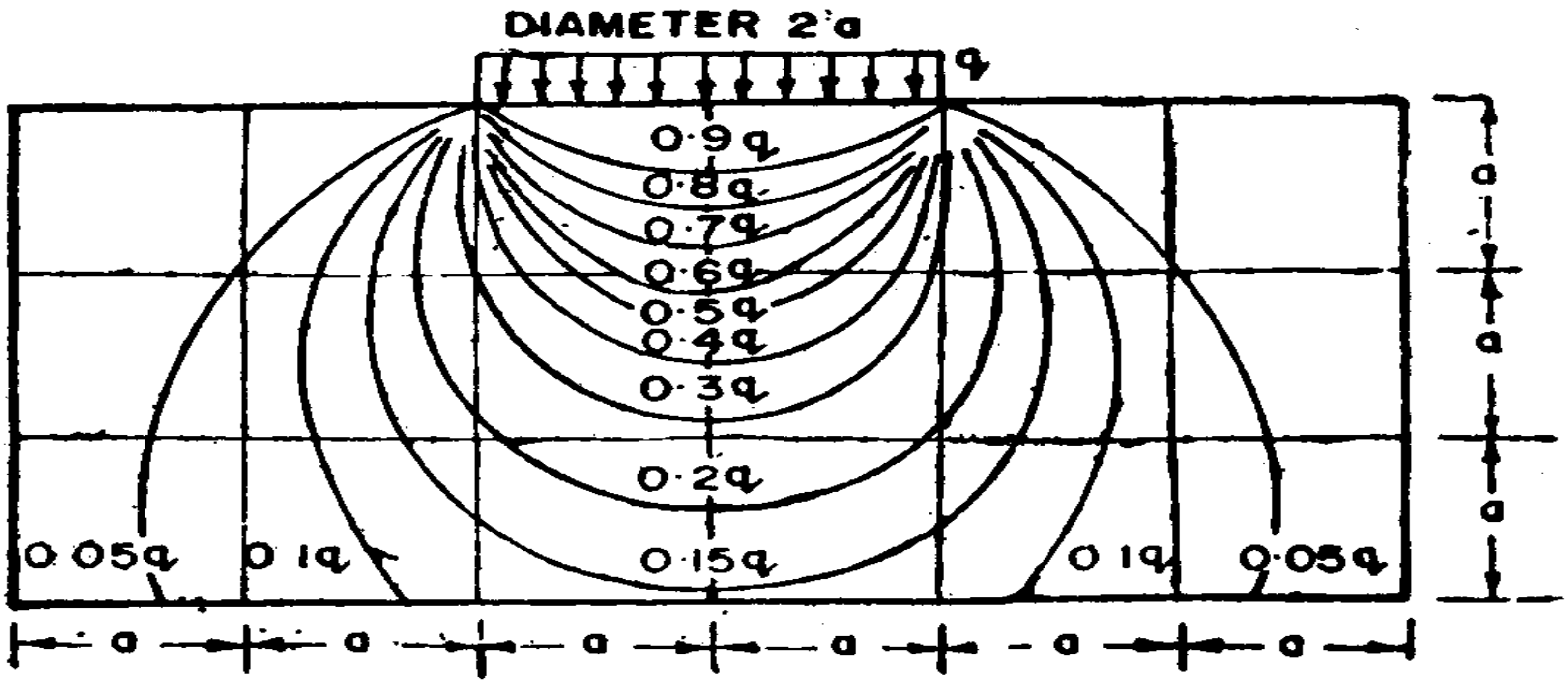
SOIL STRESS CAUSED BY EXTERNAL LOAD

- External Load Types
  - Point Load
  - Line Load
  - Uniform Load

LOAD DISTRIBUTION PATTERN

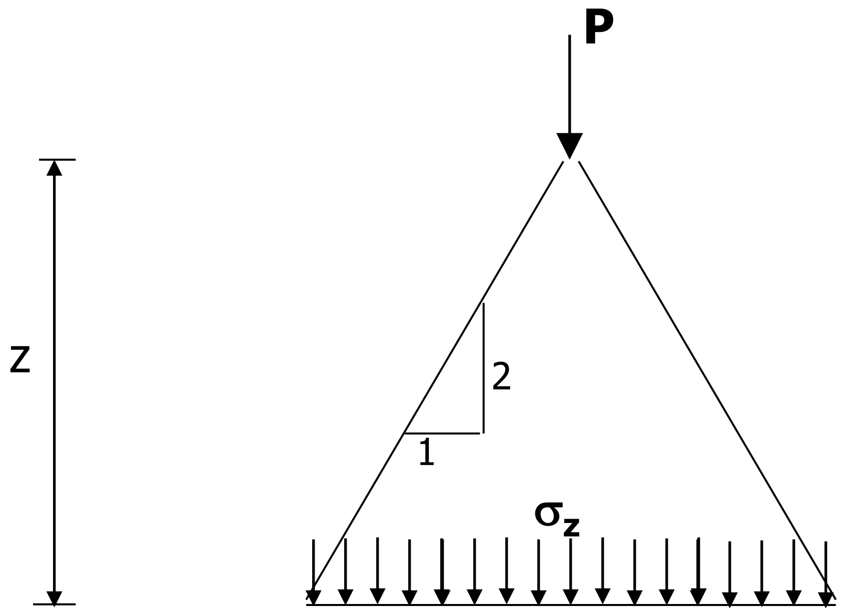


STRESS CONTOUR



STRESS DISTRIBUTION

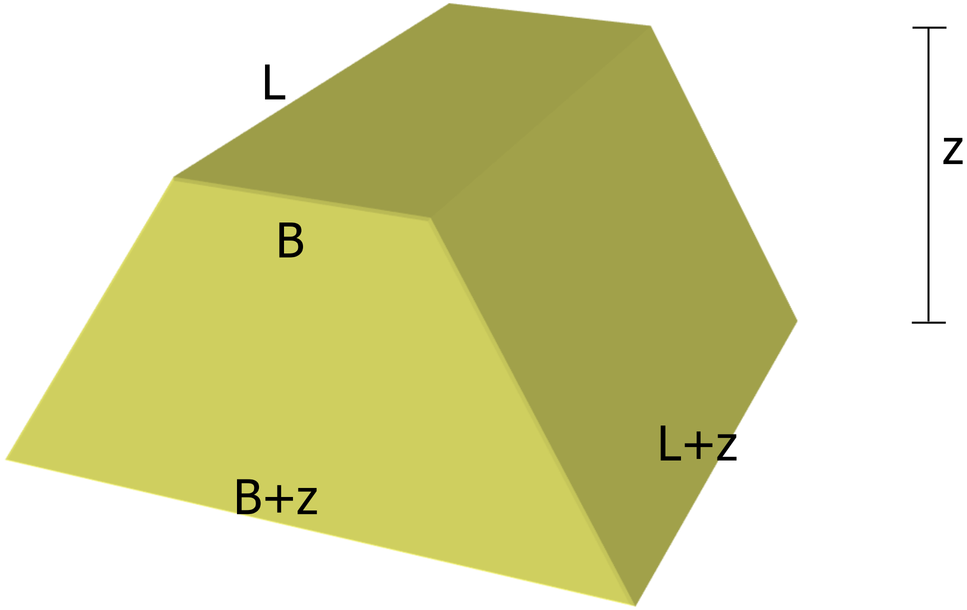
- Point Load



$$\sigma_z = \frac{P}{z^2}$$

STRESS DISTRIBUTION

- Uniform Load



$$\sigma_z = \frac{q.B.L}{(B+z)(L+z)}$$

Stresses beneath point load

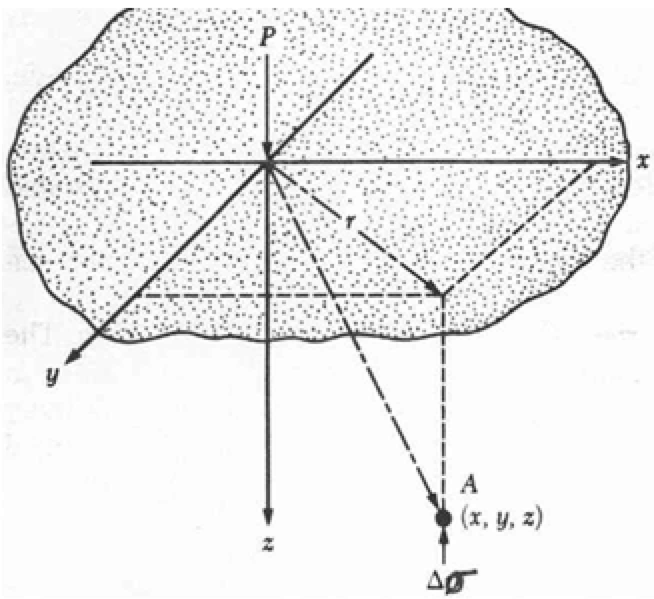
- ***Boussinesq*** published in 1885 a solution for the stresses beneath a point load on the surface of a material which had the following properties:
- Semi-infinite – this means infinite below the surface therefore providing no boundaries of the material apart from the surface
- Homogeneous – the same properties at all locations
- Isotropic –the same properties in all directions
- Elastic –a linear stress-strain relationship.

Vertical Stress Increase with Depth

- Allowable settlement, usually set by building codes, may control the allowable bearing capacity
- The vertical stress increase with depth must be determined to calculate the amount of settlement that a foundation may undergo

Stress due to a Point Load

- In 1885, Boussinesq developed a mathematical relationship for vertical stress increase with depth inside a homogenous, elastic and isotropic material from point loads as follows:

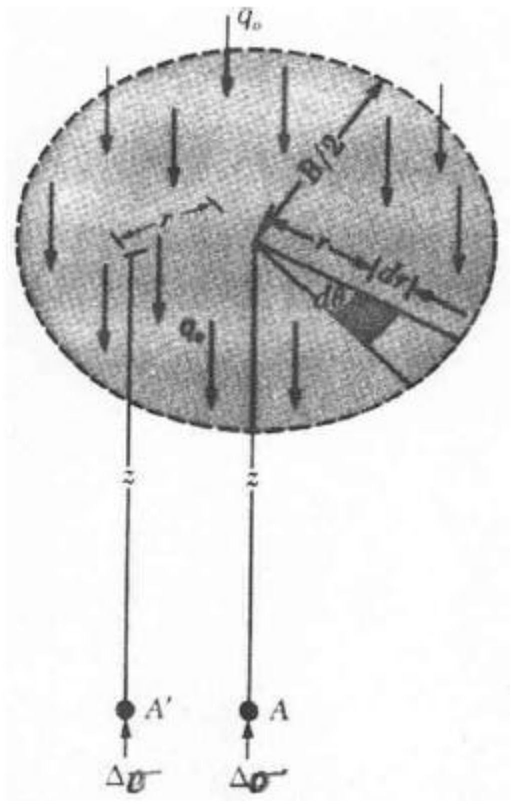


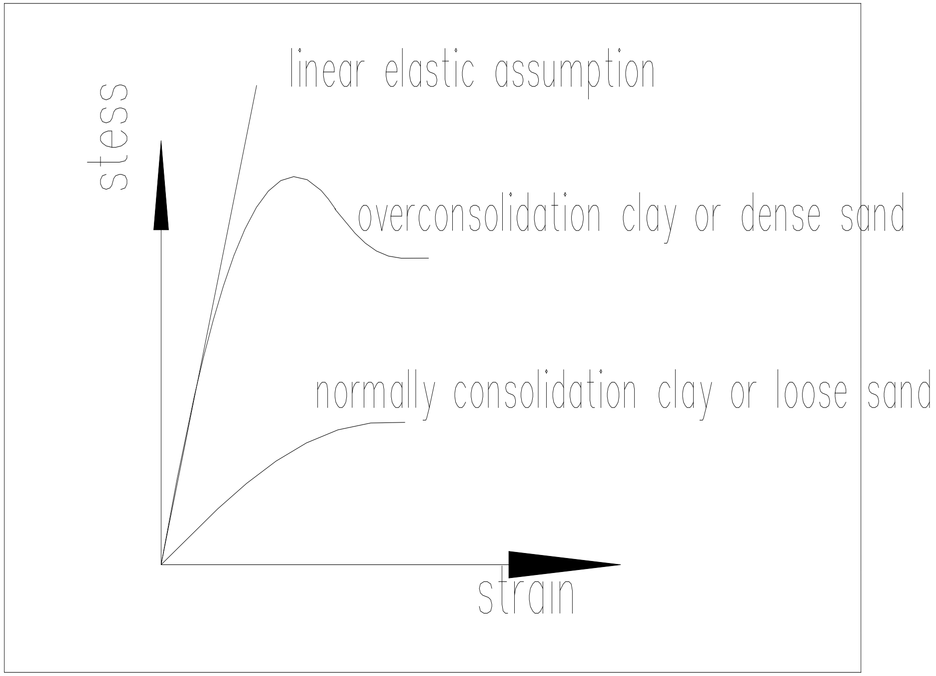
$$\Delta\sigma = \frac{3 \cdot P}{2\pi \cdot z^2 \left[1 + \left(\frac{r}{z}\right)^2\right]^{5/2}}$$

Stress due to a Circular Load

- The Boussinesq Equation as stated above may be used to derive a relationship for stress increase below the center of the footing from a **flexible** circular loaded area:

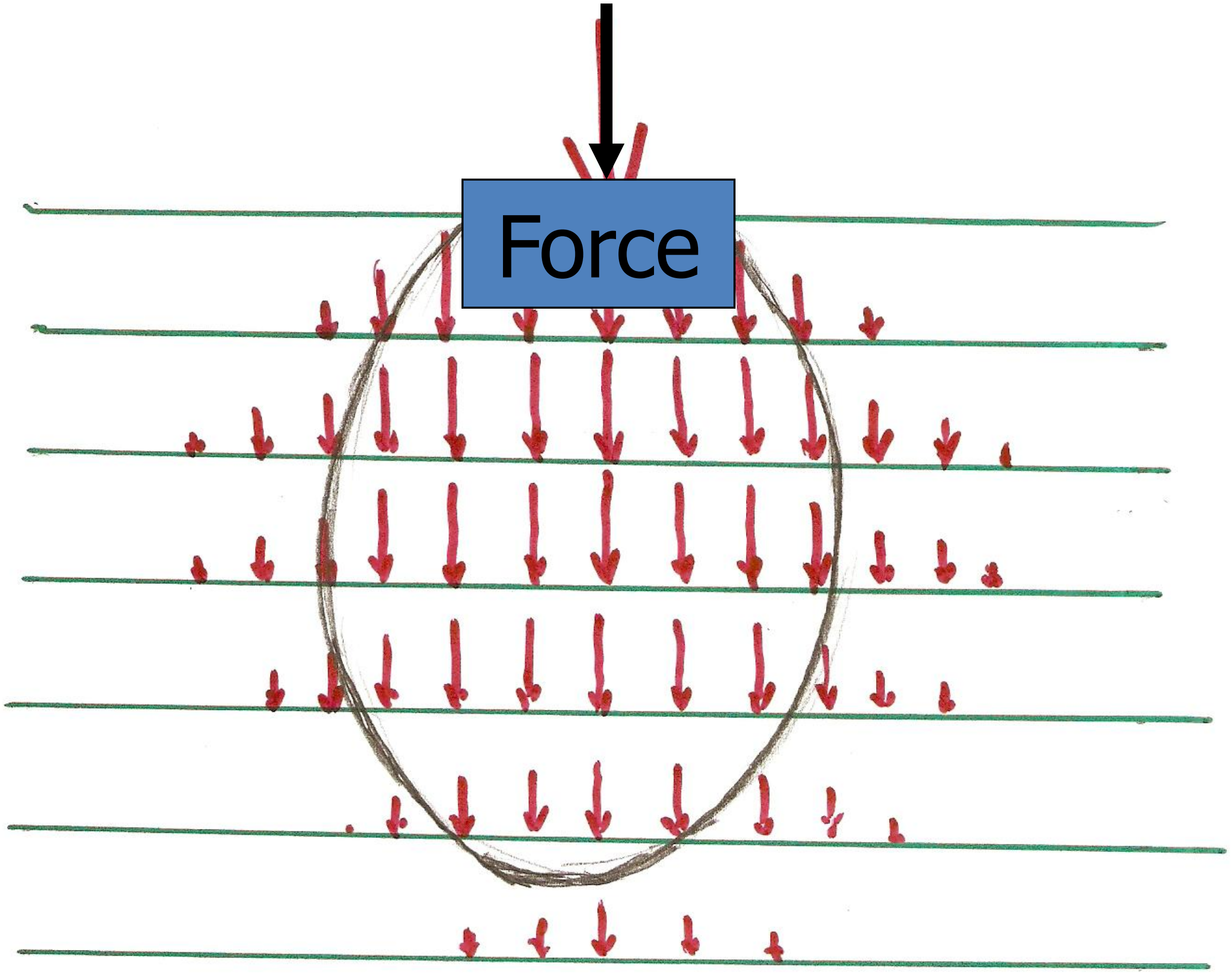
$$\Delta\sigma = q_o\left(1 - \left[1 + \left(\frac{B}{2z}\right)^2\right]^{-\frac{3}{2}}\right)$$





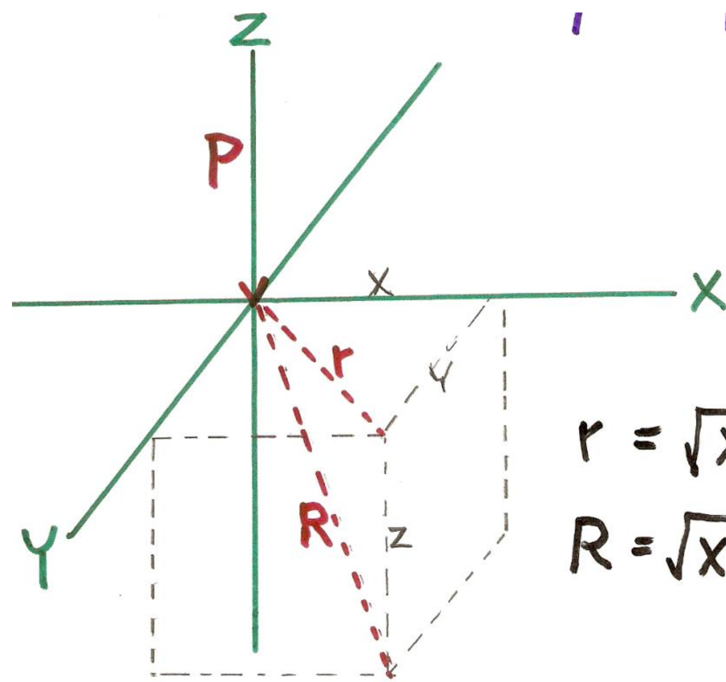
Linear elastic assumption

I. The Bulb of Pressure



II. The Boussinesq Equation

B. The Equation:



$$r = \sqrt{x^2 + y^2}$$

$$R = \sqrt{x^2 + y^2 + z^2}$$

Vertical Stress (σ_z)

$$\sigma_z = \frac{3P}{2\pi} \frac{z^3}{R^5}$$

Horizontal Stress (σ_x)

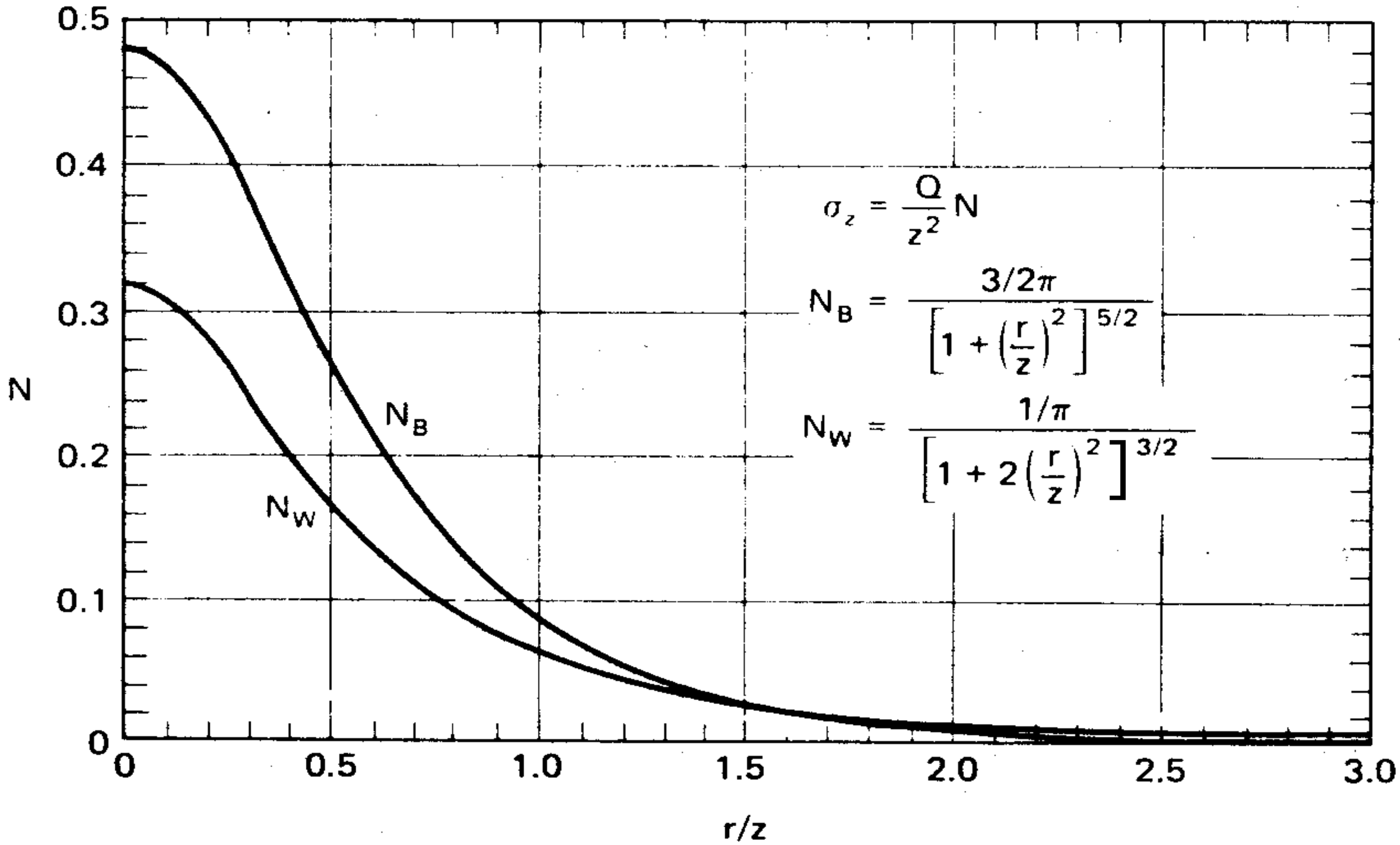
$$\sigma_x = \frac{P}{2\pi}\left[\frac{3x^2 z}{R^5} - (1-2\nu)\left(\frac{x^2-y^2}{Rr^2(R+z)} + \frac{y^2 z}{R^3 r^2}\right)\right]$$

Where ν =0.48 Poisson's Ratio ()

Also an equation for σ_y

BOUSSINESQ METHOD

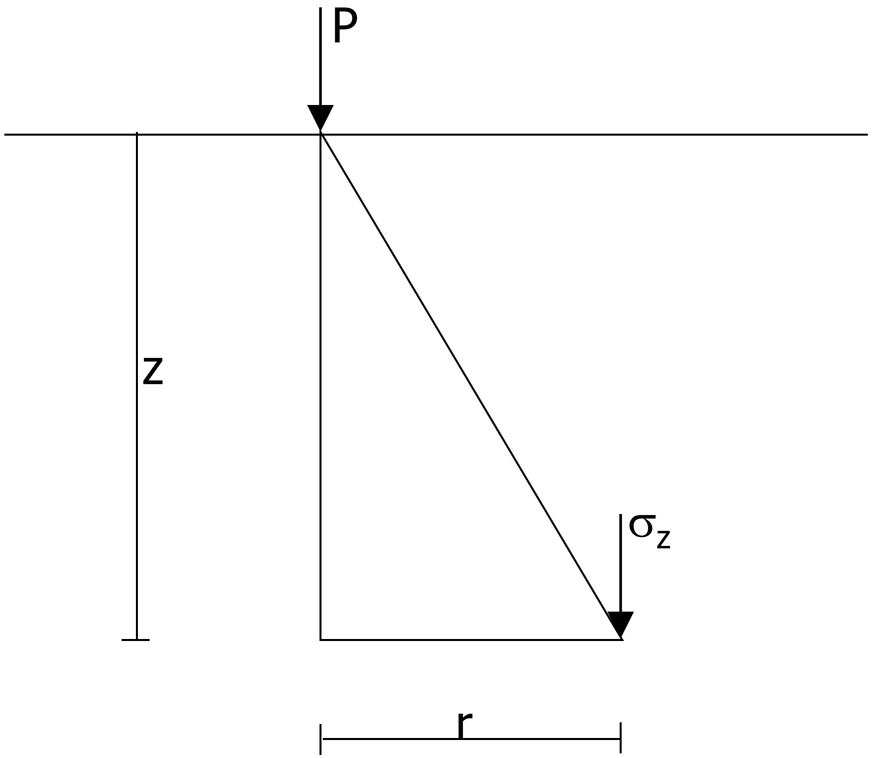
(a)



(b)

BOUSSINESQ METHOD

- Point Load

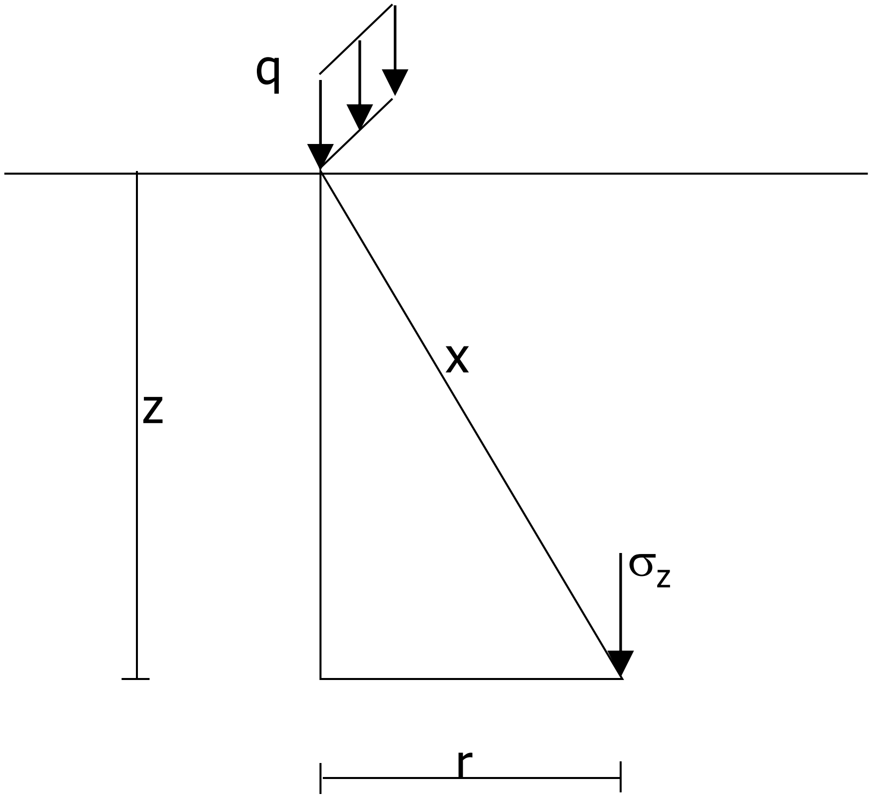


$$\sigma_z = \frac{P\left(3z^3\right)}{2\pi\left(r^2 + z^2\right)^{5/2}}$$

$$\sigma_z = \frac{P}{z^2} N_B$$

BOUSSINESQ METHOD

- Line Load



$$\sigma_z = \frac{2q}{\pi}\frac{z^3}{x^4}$$

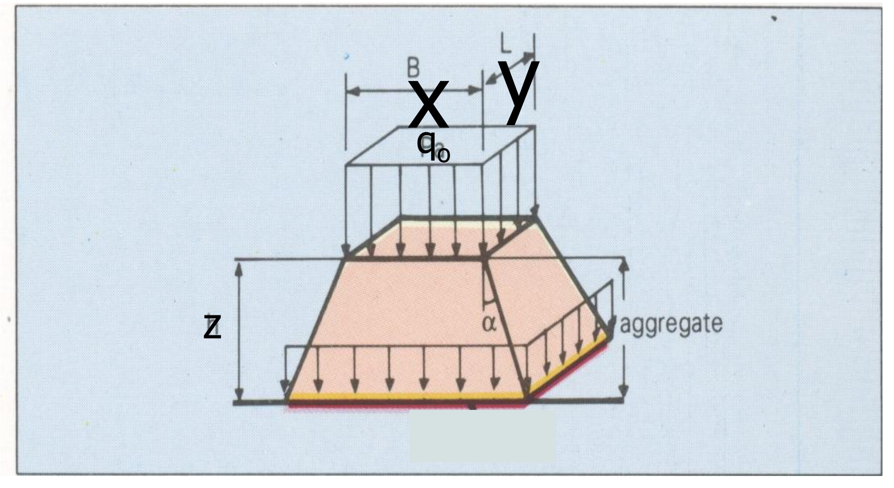
$$x = \sqrt{z^2 + r^2}$$

BOUSSINESQ METHOD

- Uniform Load
  - Square/Rectangular
  - Circular
  - Trapezoidal
  - Triangle

BOUSSINESQ METHOD

- Rectangular



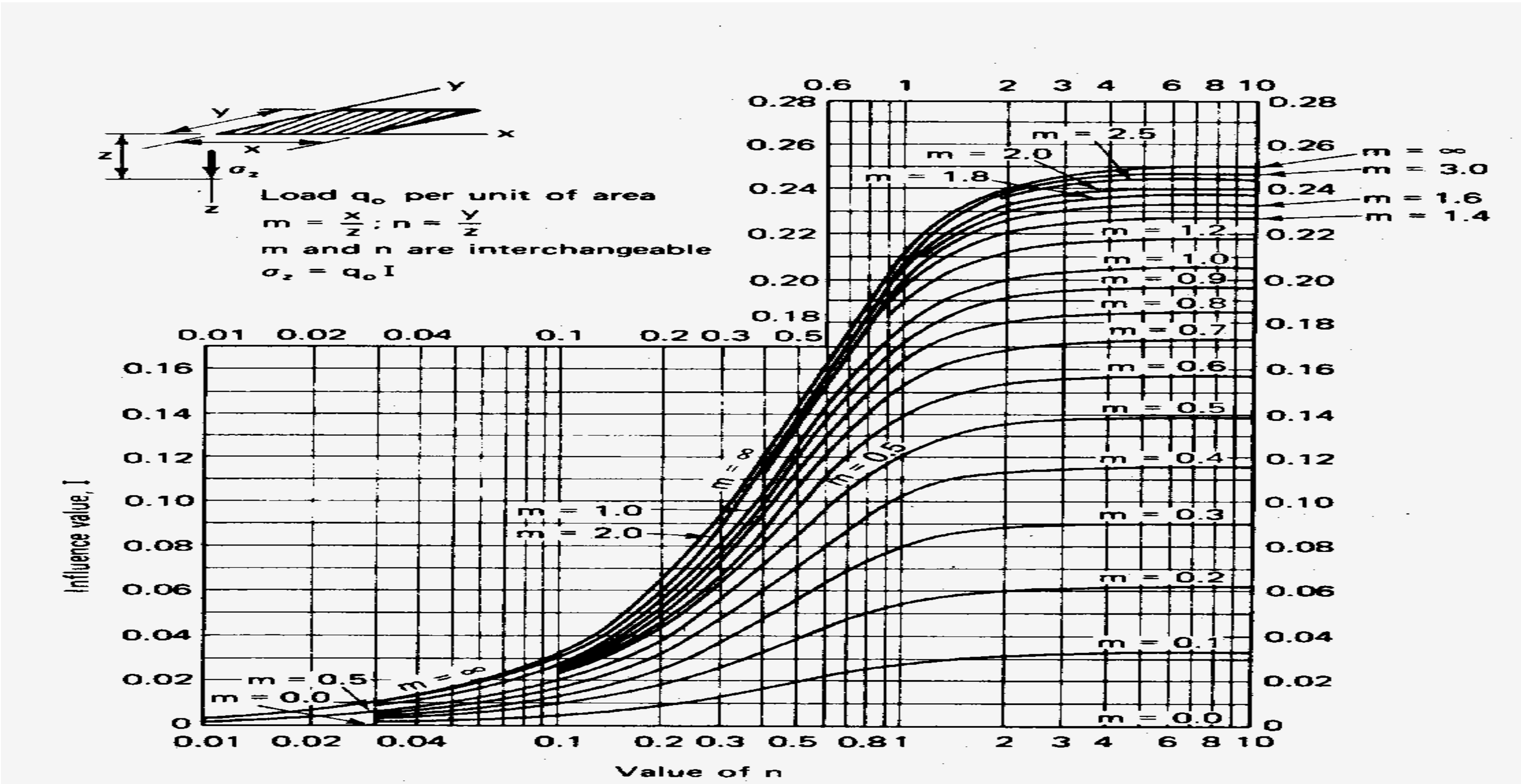
m = x/z

n = y/z

$$\sigma_z = q_o \frac{1}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1+m^2n^2} x \frac{\left(m^2+n^2+2\right)}{\left(m^2+n^2+1\right)} + \tan^{-1}\left(\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1-m^2n^2}\right)\right]$$

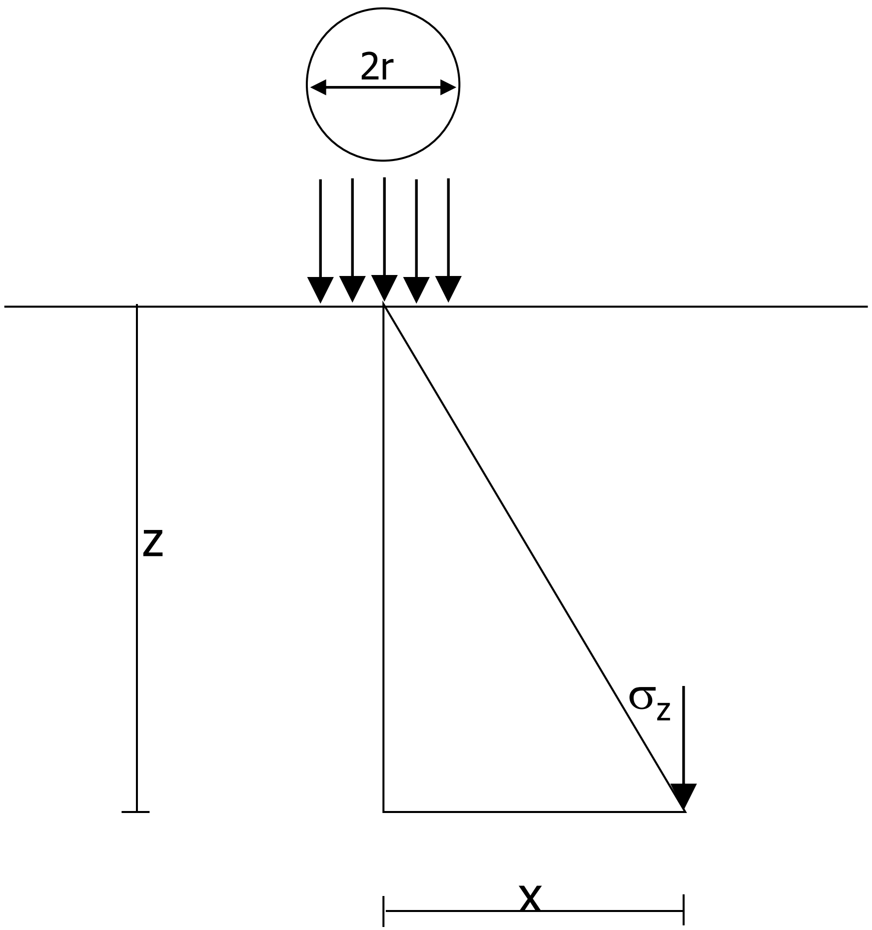
BOUSSINESQ METHOD

- Rectangular



BOUSSINESQ METHOD

- Circular



At the center of circle (X = 0)

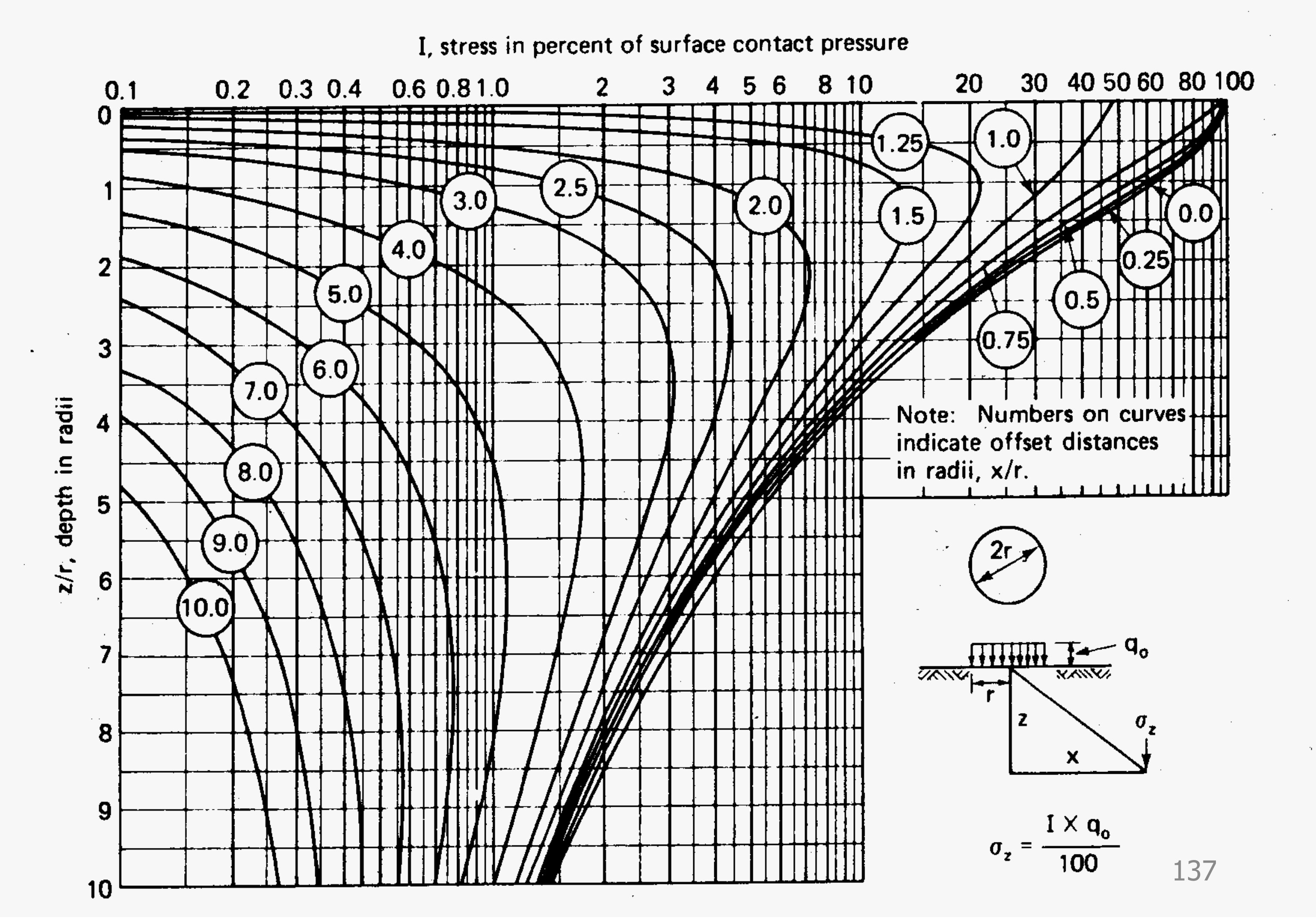
$$\sigma_z = q_o\left\{1-\left[1+\left(\frac{r}{z}\right)^2\right]^{-1,5}\right\}$$

For other positions (X ≠ 0),

Use chart for finding the influence factor

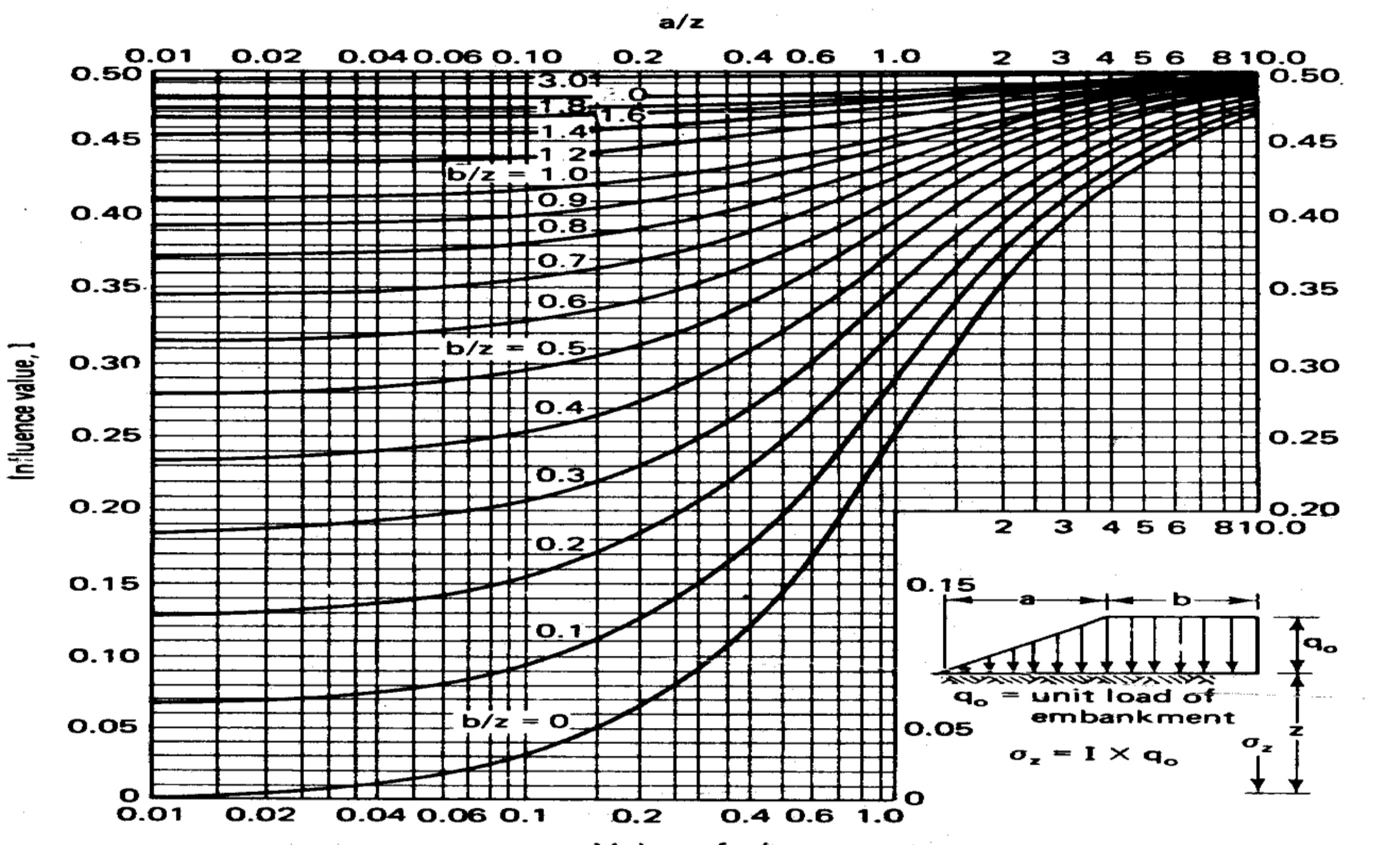
BOUSSINESQ METHOD

- Circu



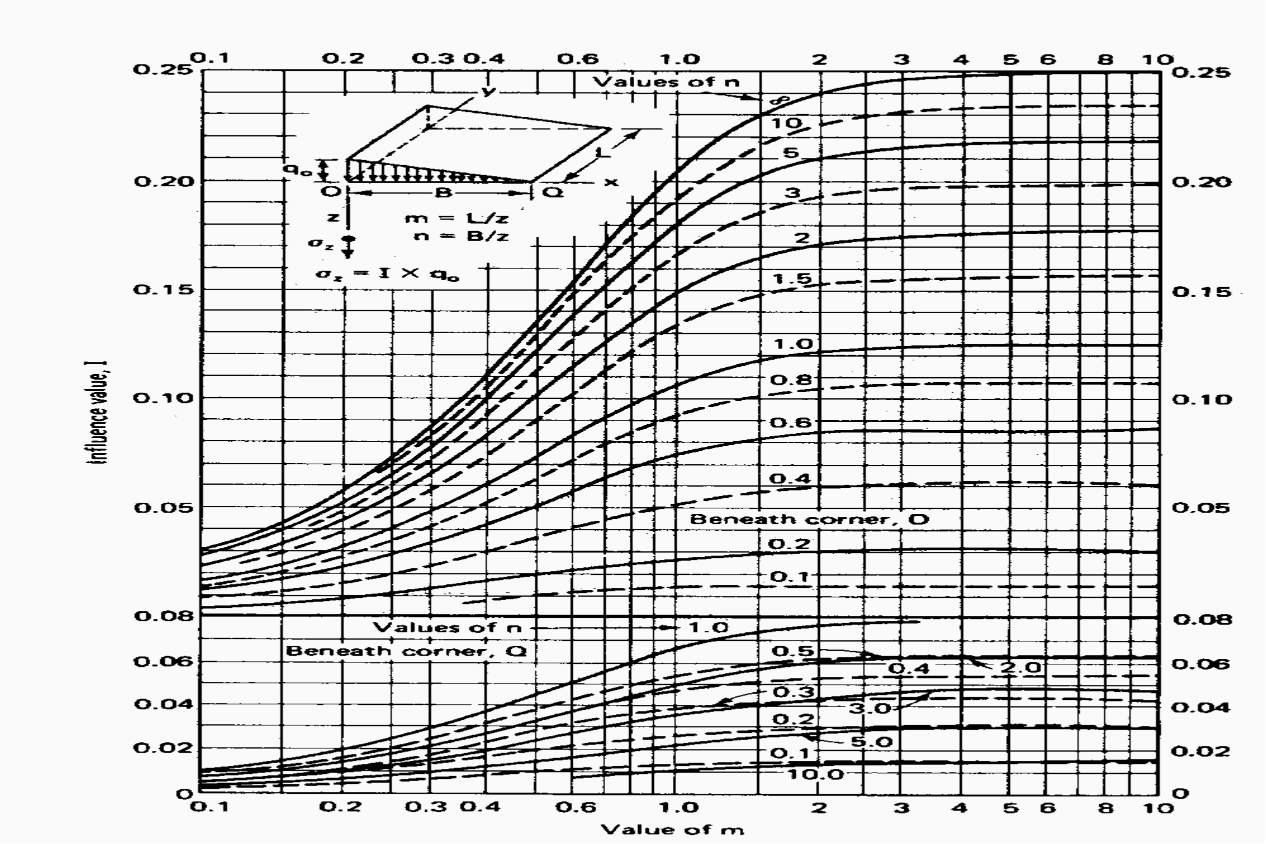
BOUSSINESQ METHOD

- Trapezoidal



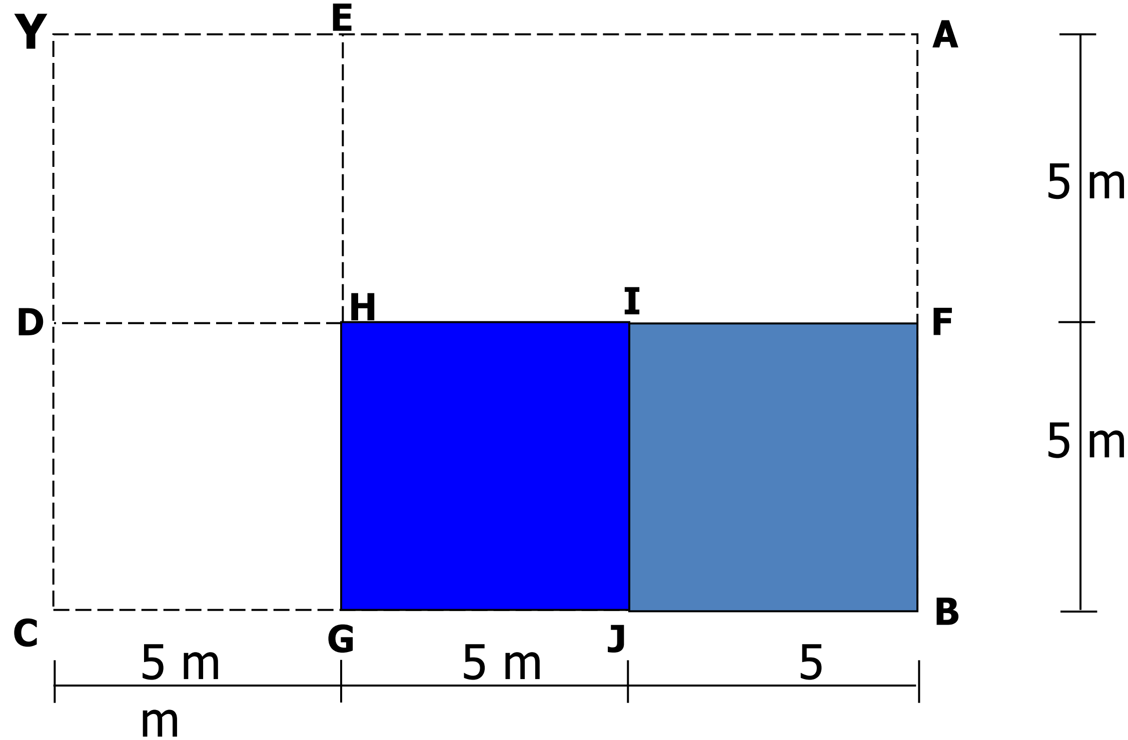
BOUSSINESQ METHOD

- Triangle



EXAMPLE

- The 5 m x 10 m area uniformly loaded with 100 kPa



- Question :
  1. Find the at a depth of 5 m under point Y
  2. Repeat question no.1 if the right half of the 5 x 10 m area were

EXAMPLE

Question 1

Item	Area			
	YABC	**-YAFD**	**-YEGC**	**YEHD**
x	15	15	10	5
y	10	5	5	5
z	5	5	5	5
m = x/z	3	3	2	1
n = y/z	2	1	1	1
I	0.238	0.209	0.206	0.18
σ_z	23.8	- 20.9	-20.6	18.0

σ_z total = 23.8 – 20.9 – 20.6 + 18 = 0.3 kPa

EXAMPLE

Question 2

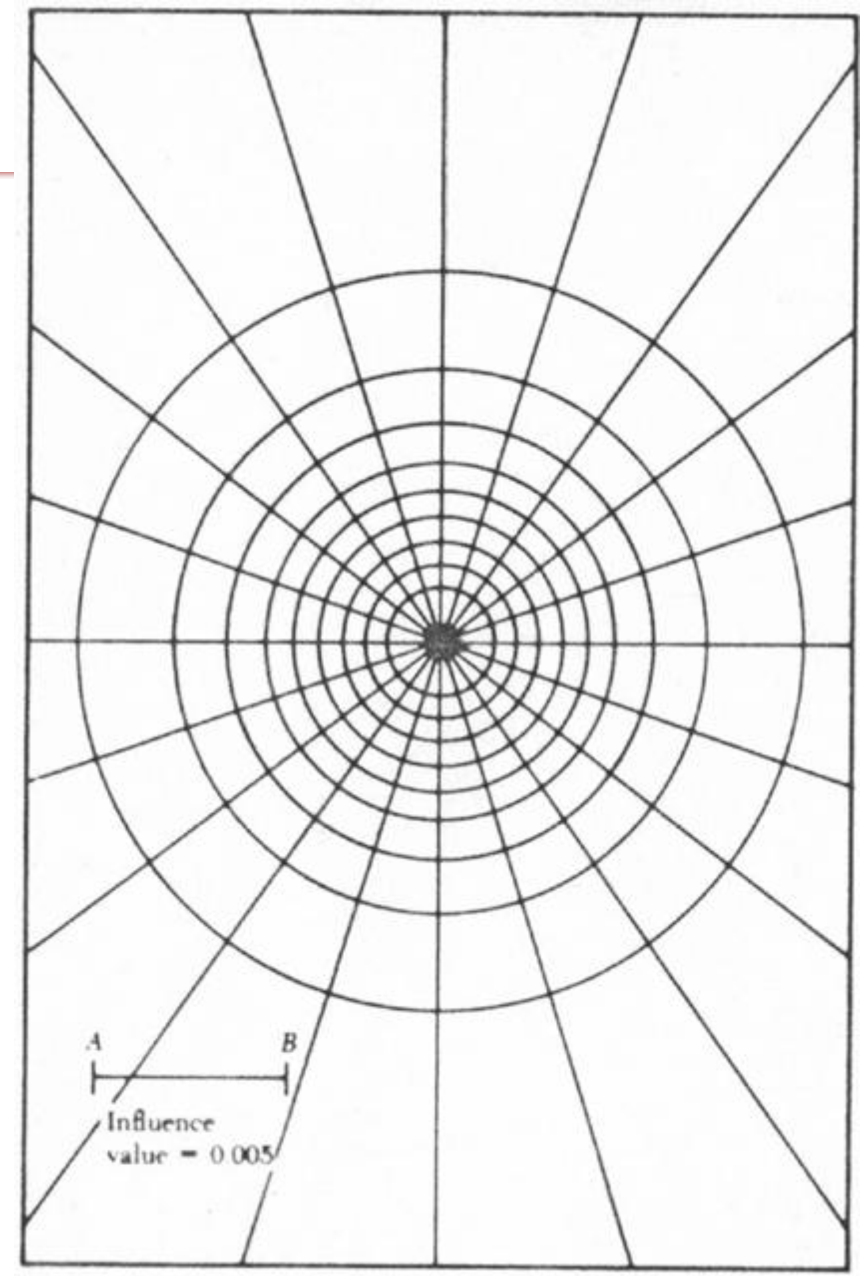
Item	Area			
	Y_{ABC}	$-Y_{AFD}$	$-Y_{EGC}$	Y_{EHD}
x	15	15	10	5
y	10	5	5	5
z	5	5	5	5
m = x/z	3	3	2	1
n = y/z	2	1	1	1
I	0.238	0.209	0.206	0.18
σ_z	47.6	- 41.9	-43.8	38.6

σ_z total = 47.6 – 41.9 – 43.8 + 38.6 = 0.5 kPa

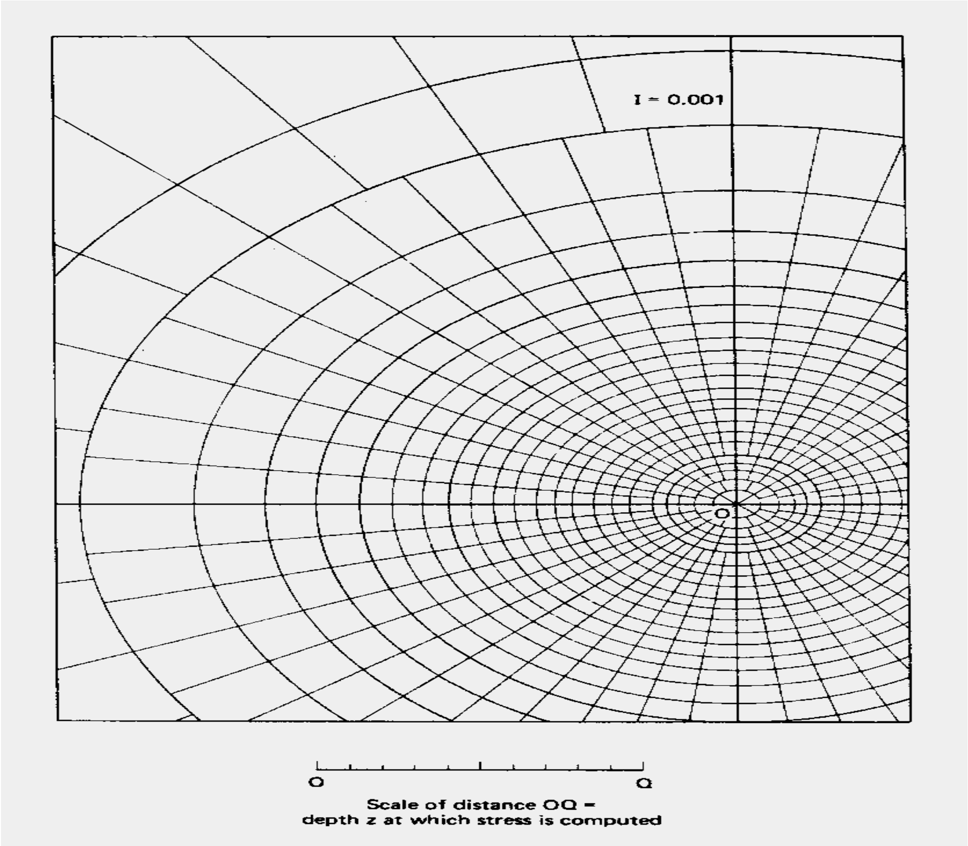
Newmark's Influence Chart

- The Newmark's Influence Chart method consists of concentric circles drawn to scale, each square contributes a fraction of the stress
- In most charts each square contributes 1/200 (or 0.005) units of stress (influence value, IV)
- Follow the 5 steps to determine the stress increase:
  1. Determine the depth, z, where you wish to calculate the stress increase
  2. Adopt a scale of z=AB
  3. Draw the footing to scale and place the point of interest over the center of the chart
  4. Count the number of elements that fall inside the footing, N
  5. Calculate the stress increase as:

$$\Delta\sigma = q_o(IV)\cdot(N)$$



NEWMARK METHOD



$$\sigma_Z = q_o . I . N$$

Where :

q_o = Uniform Load

I = Influence factor

N = No. of blocks

NEWMARK METHOD

- Diagram Drawing

$$\sigma_z = q_o \left\{ 1 - \left[1 + \left(\frac{r}{z} \right)^2 \right]^{-1,5} \right\} \longrightarrow \frac{r}{z} = \left[\left(1 - \frac{\sigma_z}{q_o} \right)^{-2/3} - 1 \right]^{1/2}$$

1. Take σ_z/q_o between 0 and 1, with increment 0.1 or other, then find r/z value
2. Determine the scale of depth and length

   Example : 2.5 cm for 6 m
3. Calculate the radius of each circle by r/z value multiplied with depth (z)
4. Draw the circles with radius at step 3 by considering the scale at step 2

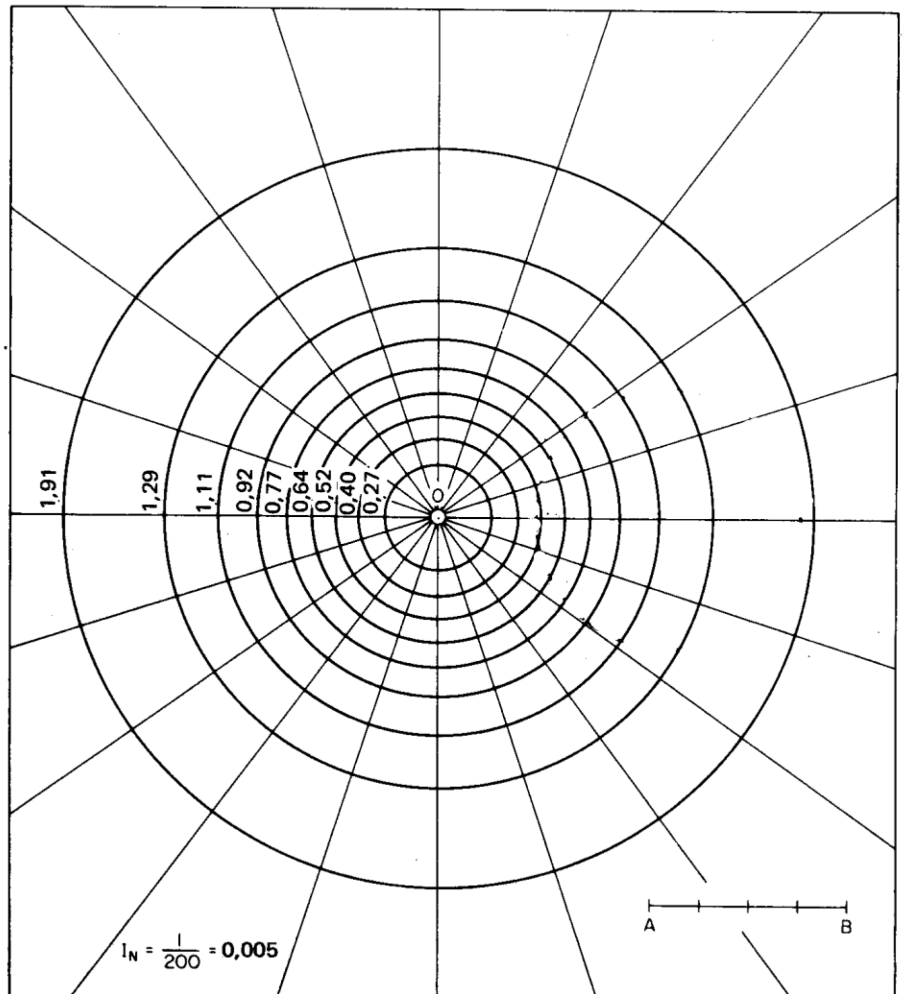
NEWMARK METHOD

- Example, the depth of point (z) = 6 m

σ_z/q_o	r/z	Radius (z=6 m)	Radius at drawing	Operation
0.1	0.27	1.62 m	0.675 cm	1.62/6 x 2.5 cm
0.2	0.40	2.40 m	1 cm	2.4/6 x 2.5 cm
0.3	0.52	3.12 m	1.3 cm	3.12/6 x 2.5 cm
0.4	0.64	3.84 m	1.6 cm	3.84/6 x 2.5 cm

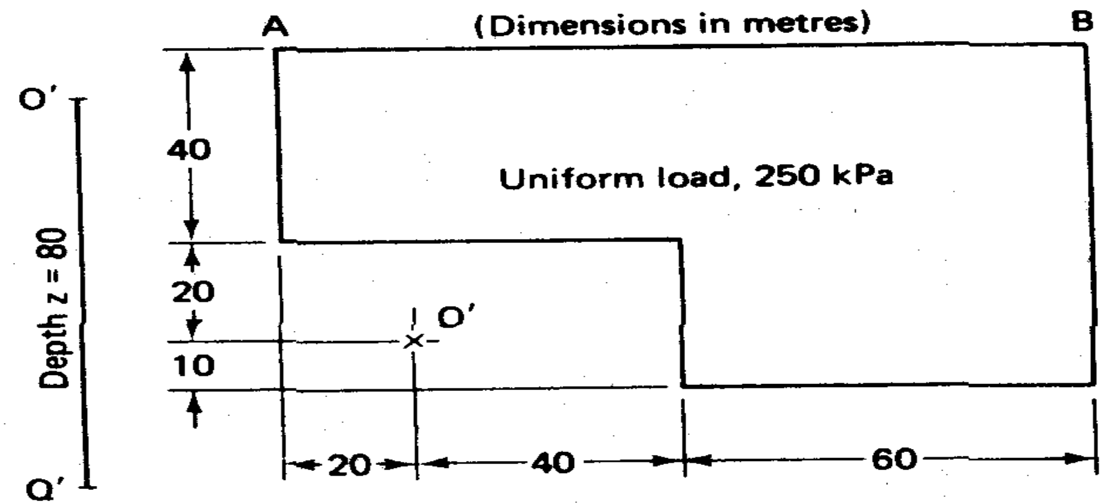
And so on, generally up to $\sigma_z/q_o \approx 1$ because if $\sigma_z/q_o = 1$ we get $r/z = \infty$

NEWMARK METHOD



EXAMPLE

- A uniform load of 250 kPa is applied to the loaded area shown in next figure :



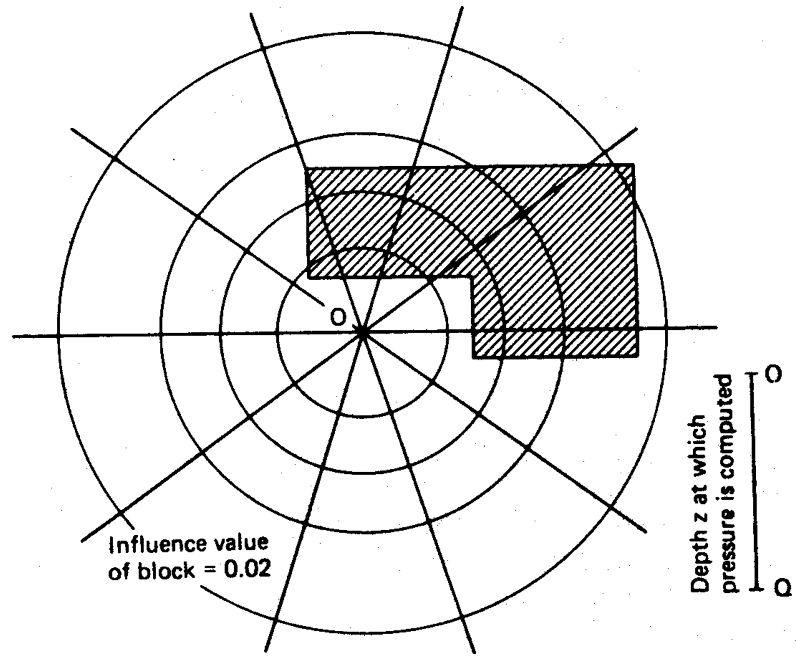
(a)

- Find the stress at a depth of 80 m below the ground surface due to the loaded area under point O'

EXAMPLE

Solution :

- Draw the loaded area such that the length of the line OQ is scaled to 80 m.
- Place point O', the point where the stress is required, over the center of the influence chart
- The number of blocks are counted under the loaded area
- The vertical stress at 80 m is then indicated by : $\sigma_v = q_o \cdot I \cdot N$



(b)

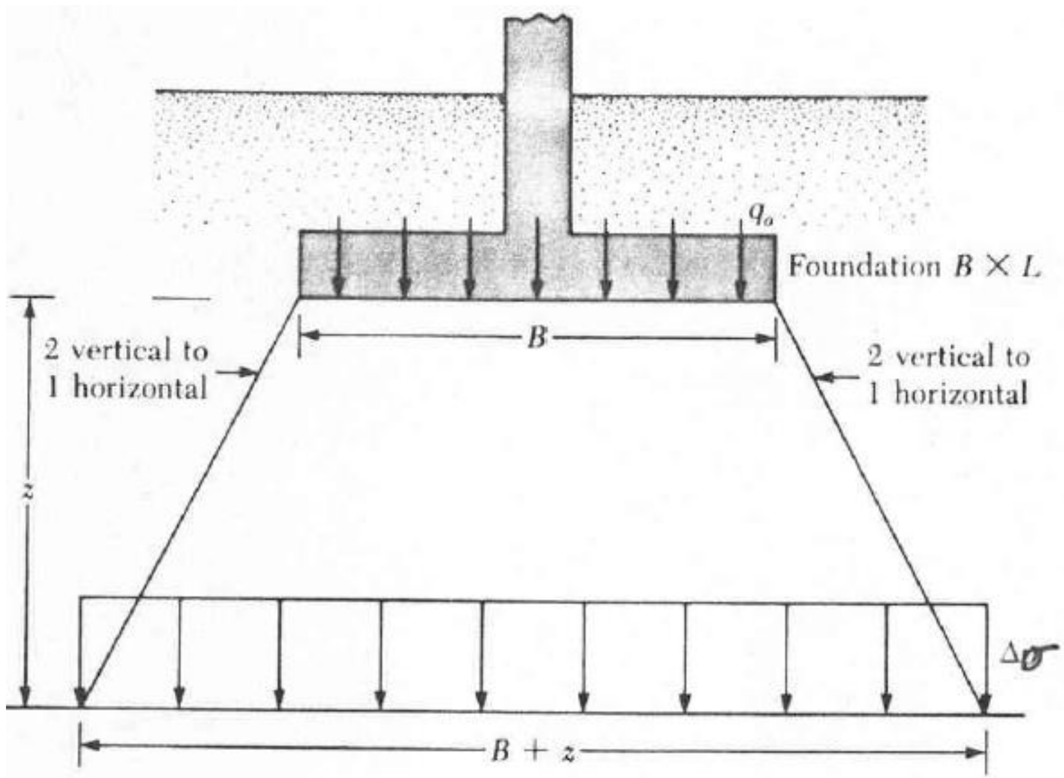
$$\sigma_v = 250 \cdot 0.02 \cdot 8 = 40 \text{ kPa}$$

Simplified Methods

- The 2:1 method is an approximate method of calculating the apparent "dissipation" of stress with depth by averaging the stress increment onto an increasingly bigger loaded area based on 2V:1H.
- This method assumes that the stress increment is constant across the area (B+z)·(L+z) and equals zero outside this area.

- The method employs simple geometry of an increase in stress proportional to a slope of 2 vertical to 1 horizontal
- According to the method, the increase in stress is calculated as follows:

$$\Delta\sigma = \frac{q_o BL}{(B+z)\cdot(L+z)}$$



Westergaard' s Theory of stress distribution

- Westergaard developed a solution to determine distribution of stress due to point load in soils composed of thin layer of granular material that partially prevent lateral deformation of the soil.

Westergaard' s Theory of stress distribution

- **Assumptions:**
  (1) The soil is elastic and semi-infinite.
  (2) Soil is composed of numerous closely spaced horizontal layers of negligible thickness of an infinite rigid material.
  (3) The rigid material permits only the downward deformation of mass in which horizontal deformation is zero.

WESTERGAARD METHOD

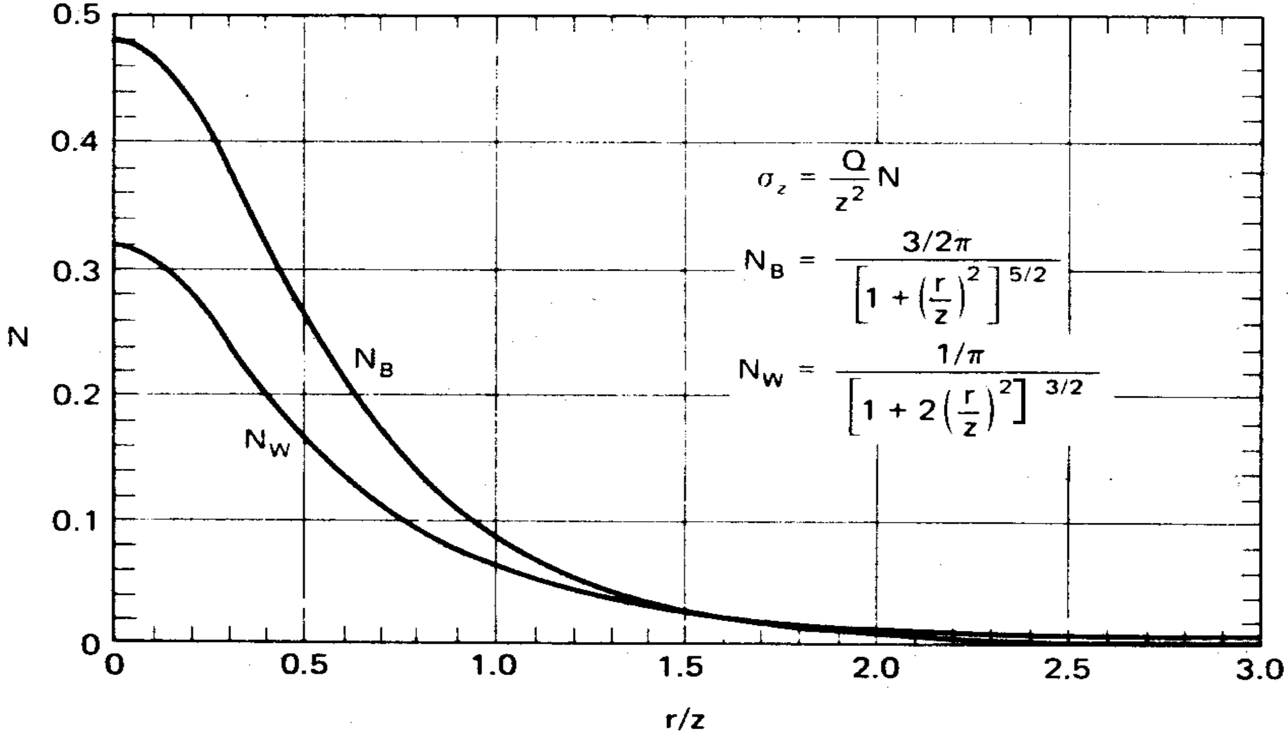
- **Point Load**

$$\sigma_z = \frac{P}{z^2\pi}\frac{1}{\left[1+2\left(\frac{r}{z}\right)^2\right]^{3/2}}$$

$$a = \sqrt{\frac{1-2\nu}{2-2\nu}}$$

$\nu = 0 \longrightarrow$

WESTERGAARD METHOD



(b)

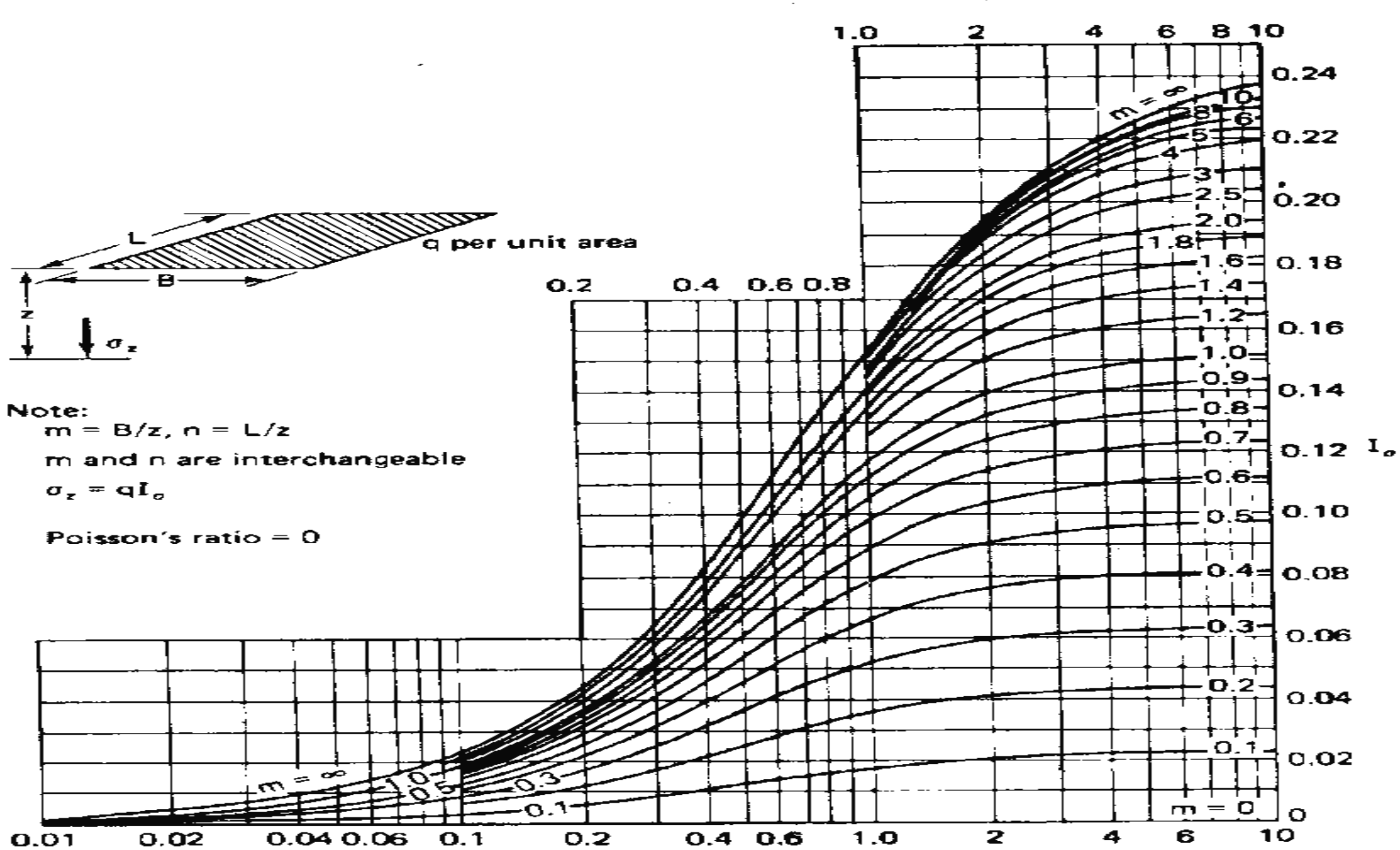
WESTERGAARD METHOD

- **Circular Uniform Load**

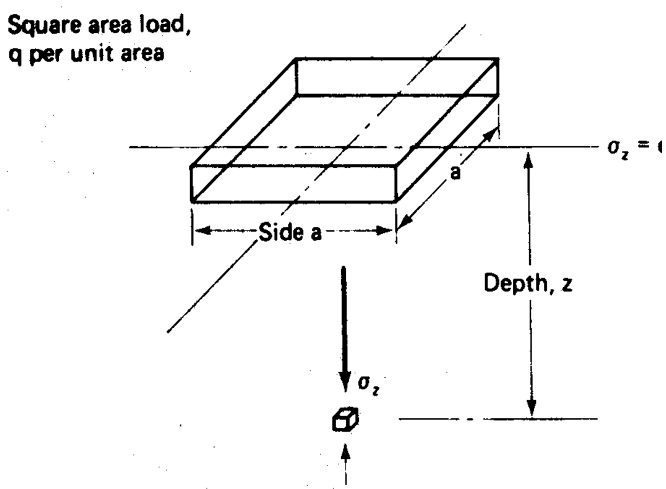
$$\sigma_z = q_o\left(1-\sqrt{\frac{a}{a+\left(r/z\right)^2}}\right)$$

$$a = \sqrt{\frac{1-2\nu}{2-2\nu}}$$

WESTERGAARD METHOD



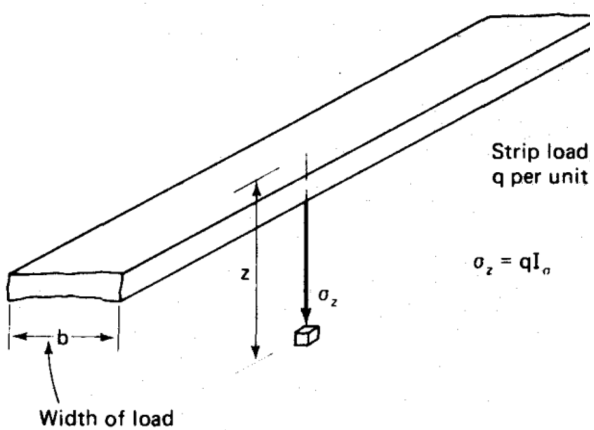
BOUSSINESQ VS WESTERGAARD



a/z	I_σ	
	Boussinesq	Westergaard
∞	1.0000	1.0000
20	0.9992	0.9365
16	0.9984	0.9199
12	0.9968	0.8944
10	0.9944	0.8734
8	0.9892	0.8435
6	0.9756	0.7926
5	0.9604	0.7525
4	0.9300	0.6971
3.6	0.9096	0.6659
3.2	0.8812	0.6309
2.8	0.8408	0.5863
2.4	0.7832	0.5328
2.0	0.7008	0.4647
1.8	0.6476	0.4246
1.6	0.5844	0.3794
1.4	0.5108	0.3291
1.2	0.4276	0.2858
1.0	0.3360	0.2165
0.8	0.2410	0.1560
0.6	0.1494	0.0999
0.4	0.0716	0.0477
0.2	0.0188	0.0127
0	0.0000	0.0000

*After Duncan and Buchignani (1976).

BOUSSINESQ VS WESTERGAARD



b/z	I_σ	
	Boussinesq	Westergaard
∞	1.000	1.000
100	1.000	0.990
10	0.997	0.910
9	0.996	0.901
8	0.994	0.888
7	0.991	0.874
6.5	0.989	0.864
6.0	0.986	0.853
5.5	0.983	0.835
5.0	0.977	0.824
4.5	0.970	0.807
4.0	0.960	0.784
3.5	0.943	0.756
3.0	0.920	0.719
2.5	0.889	0.672
2.0	0.817	0.608
1.5	0.716	0.519
1.2	0.624	0.448
1.0	0.550	0.392
0.8	0.462	0.328
0.5	0.306	0.216
0.2	0.127	0.089
0.1	0.064	0.045
0	0.000	0.000

*After Duncan and Buchignani (1976).

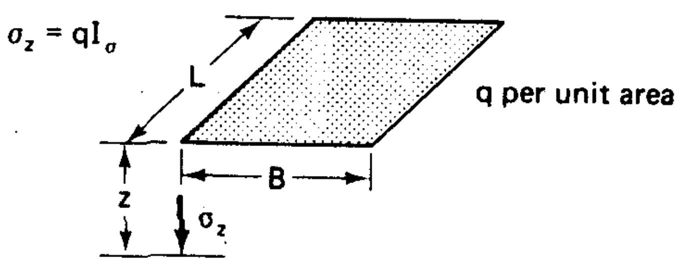
BOUSSINESQ VS WESTERGAARD

Boussinesq Case

	L/z							
B/z	0.1	0.2	0.4	0.6	0.8	1.0	2.0	∞
0.1	0.005	0.009	0.017	0.022	0.026	0.028	0.031	0.032
0.2	0.009	0.018	0.033	0.043	0.050	0.055	0.061	0.062
0.4	0.017	0.033	0.060	0.080	0.093	0.101	0.113	0.115
0.6	0.022	0.043	0.080	0.107	0.125	0.136	0.153	0.156
0.8	0.026	0.050	0.093	0.125	0.146	0.160	0.181	0.185
1.0	0.028	0.055	0.101	0.136	0.160	0.175	0.200	0.205
2.0	0.031	0.061	0.113	0.153	0.181	0.200	0.232	0.240
∞	0.032	0.062	0.115	0.156	0.185	0.205	0.240	0.250

Westergaard Case

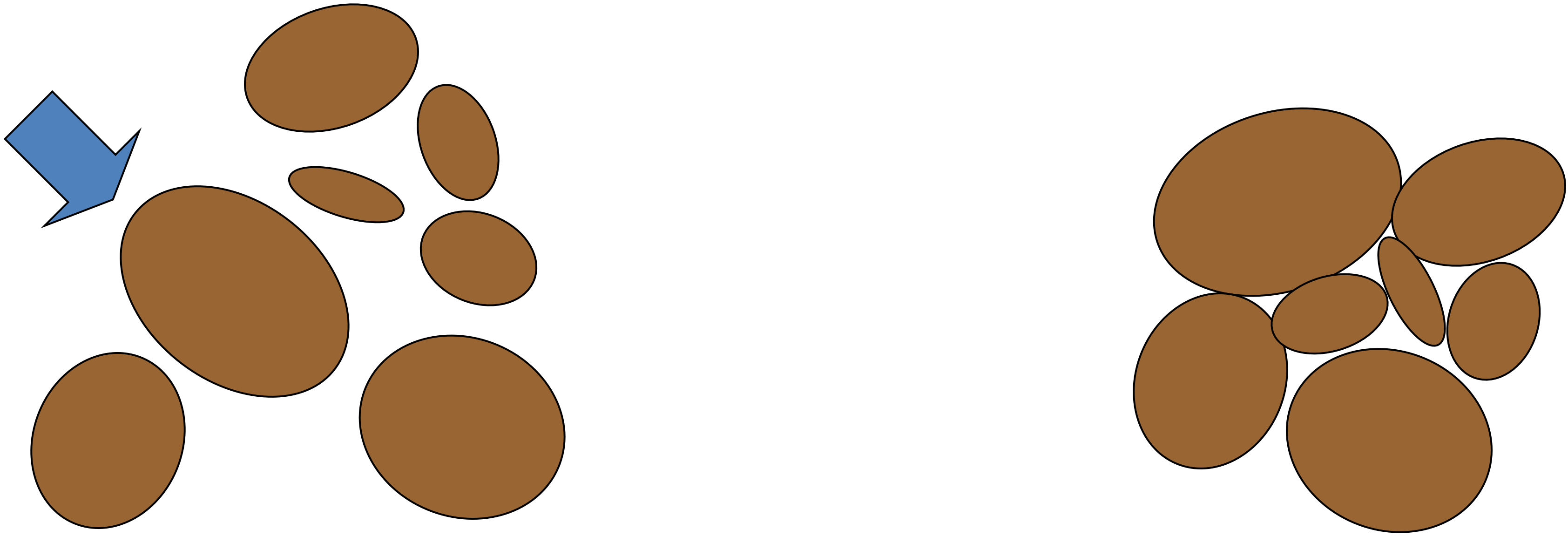
	L/z							
B/z	0.1	0.2	0.4	0.6	0.8	1.0	2.0	∞
0.1	0.003	0.006	0.011	0.014	0.017	0.018	0.021	0.022
			021	0.028	0.033	0.036	0.041	0.044
			039	0.052	0.060	0.066	0.077	0.082
			052	0.069	0.081	0.089	0.104	0.112
			060	0.081	0.095	0.105	0.125	0.135
			066	0.089	0.105	0.116	0.140	0.152
			077	0.104	0.125	0.140	0.174	0.196
			082	0.112	0.135	0.152	0.196	0.250



ignani (1976).

What is compaction?

A simple ground improvement technique, where the soil is densified through external compactive effort.



Laboratory Compaction Test

- to obtain the compaction curve and define the optimum water content and maximum dry density for a specific compactive effort.

hammer

Standard Proctor:

- 25 blows per layer
- 3 layers
- 2.7 kg hammer
- 300 mm drop

1000 ml compaction mould

Modified Proctor:

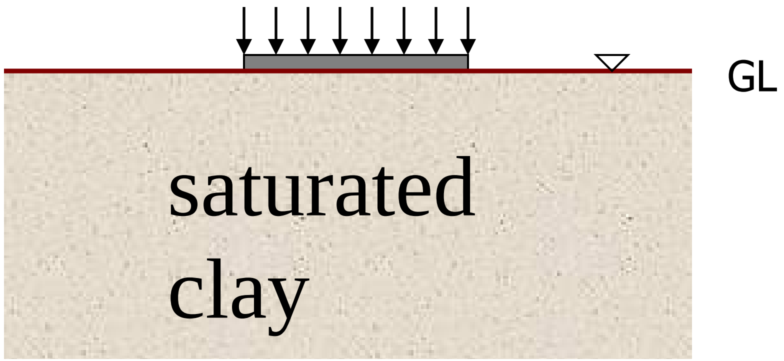
- 4.9 kg hammer
- 450 mm drop
- 5 layers
- 25 blows per layer

Earthmoving Equipment

UNIT IV

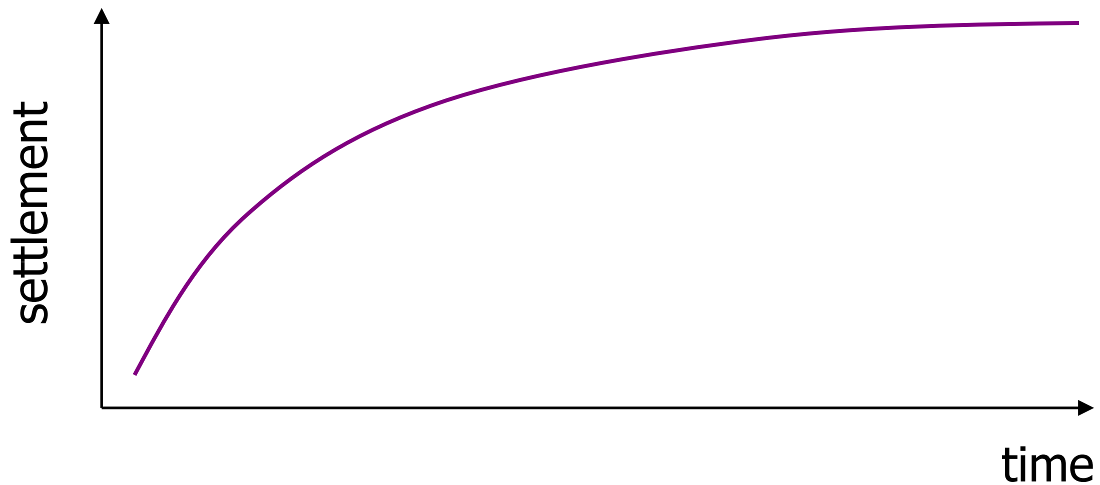
CONSOLIDATION

When a saturated clay is loaded externally,



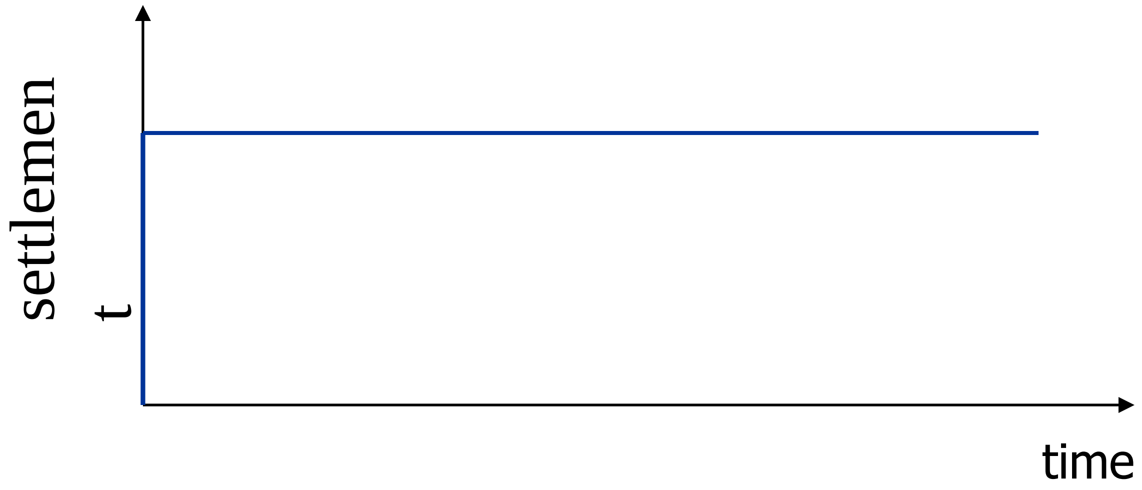
the water is squeezed out of the clay over a long time (due to low permeability of the clay).

This leads to settlements occurring over a long time,



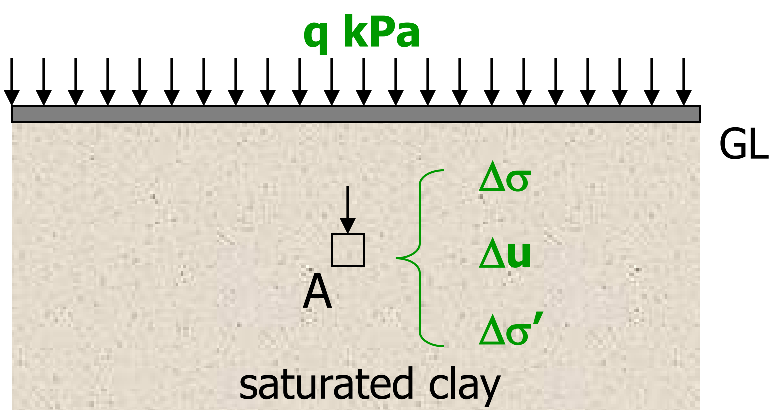
which could be several years.

Granular soils are freely drained, and thus the settlement is instantaneous.



Due to a surcharge q applied at the GL,

the stresses and pore pressures are increased at A.



..and, they vary with time.

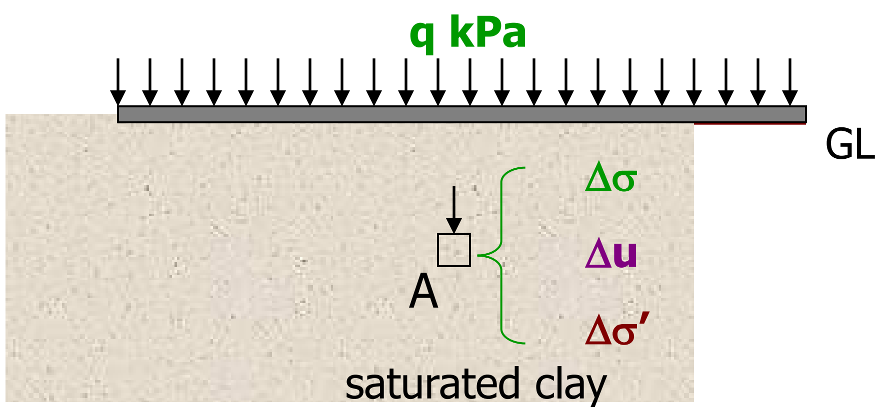
During consolidation…

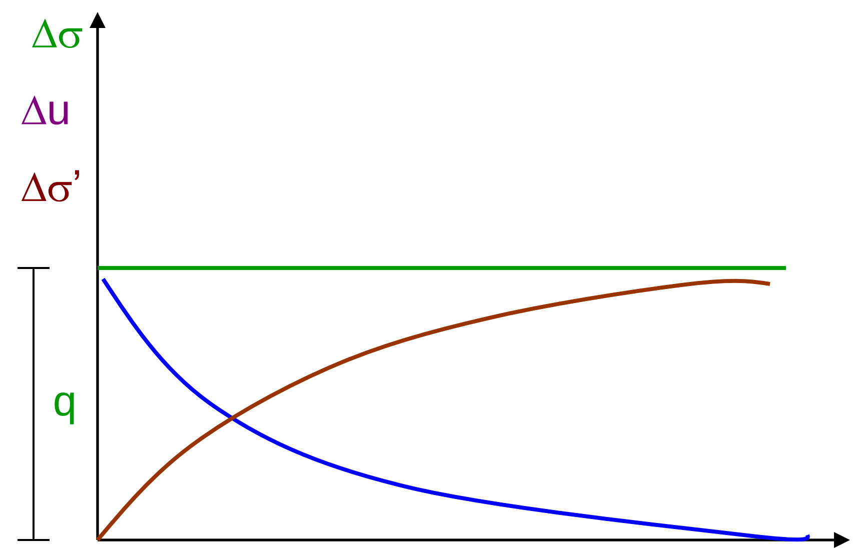
$\Delta\sigma$ remains the same (=q) during consolidation.

Δu decreases (due to drainage)

while $\Delta\sigma'$ intransferring the load from water to the soil.

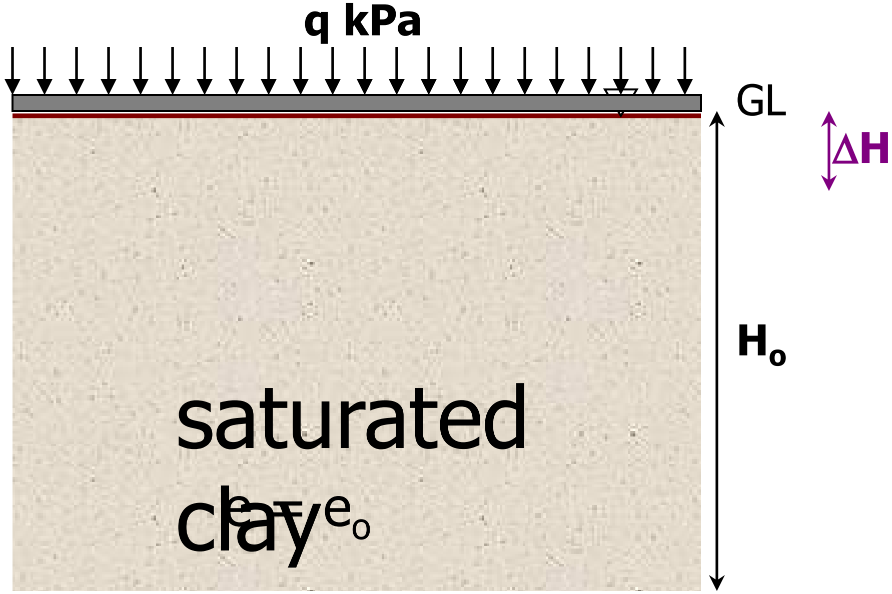
increases,



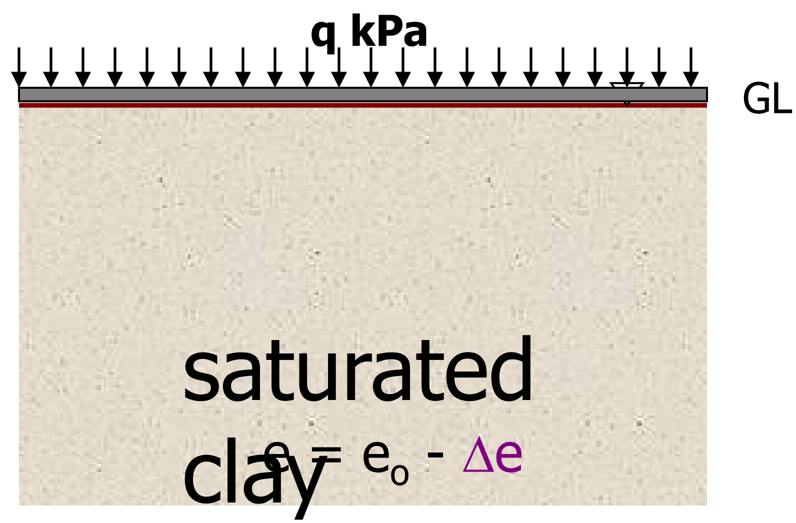


ΔH -Δe Relation

average vertical strain = $\dfrac{\Delta H}{H_o}$

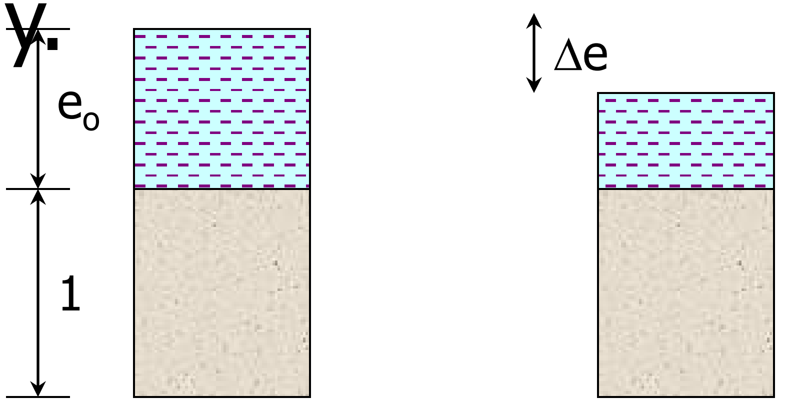


Time = 0$^+$



Time = $\propto$

Consider an element where $V_s = 1$ initially.



Time = 0^+

Time = ∞

$\therefore$ average vertical strain = $\dfrac{\Delta e}{1+e_o}$

ΔH -Δe Relation

Equating the two expressions for average vertical strain,

consolidation settlement

change in void ratio

$$\frac{\Delta H}{H_o} = \frac{\Delta e}{1+e_o}$$

initial thickness of clay layer

initial void ratio

denoted by m_v

Mv is the volumetric strain in a clay element per unit increase in stress

$$= \frac{\text{change in volume}}{\text{original volume}}$$

i.e. ,

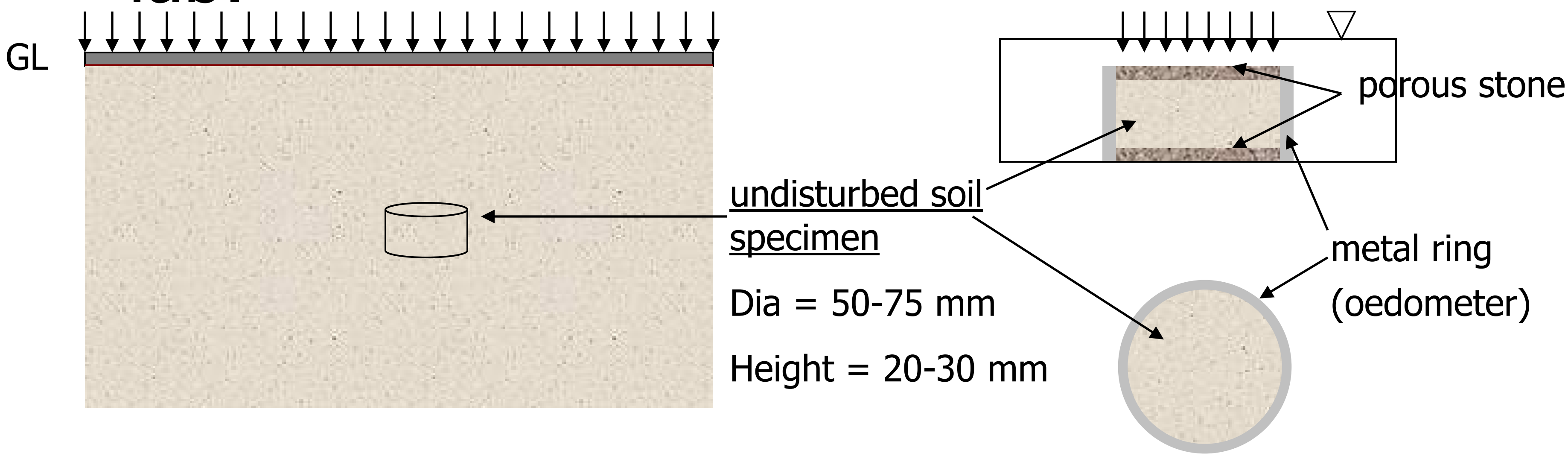
$$m_v = \frac{\Delta V / V}{\Delta \sigma}$$

no units

kPa^{-1} or MPa^{-1}

kPa or MPa

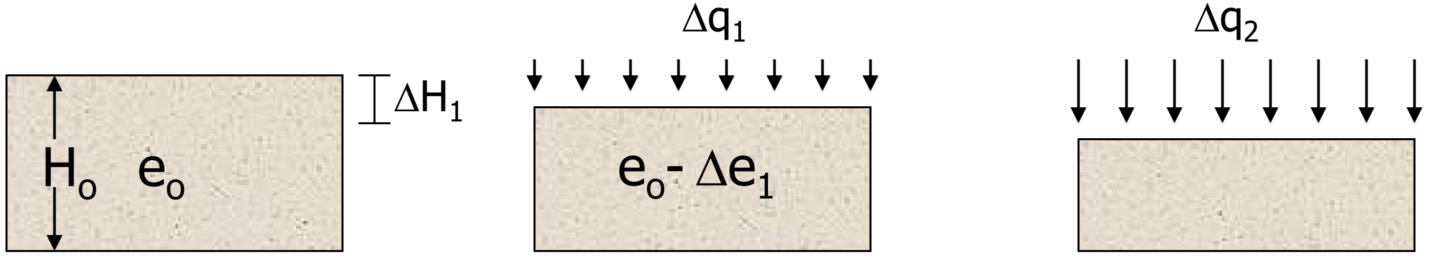
simulation of 1-D field consolidation in lab.



Consolidation Test

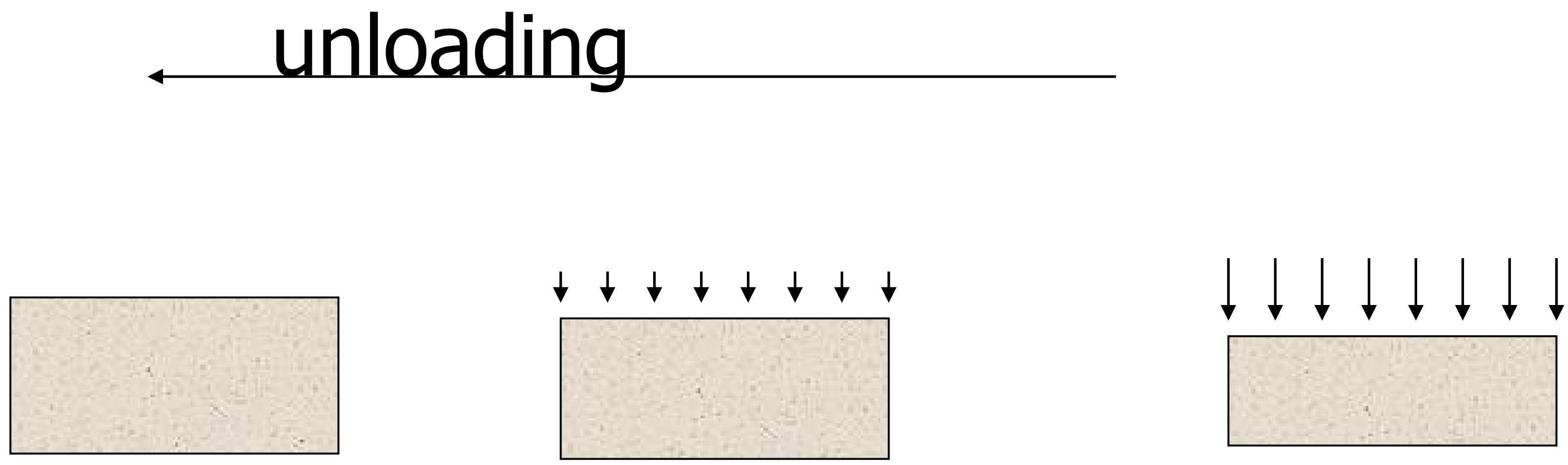
allowing full consolidation before next increment

loading in increments



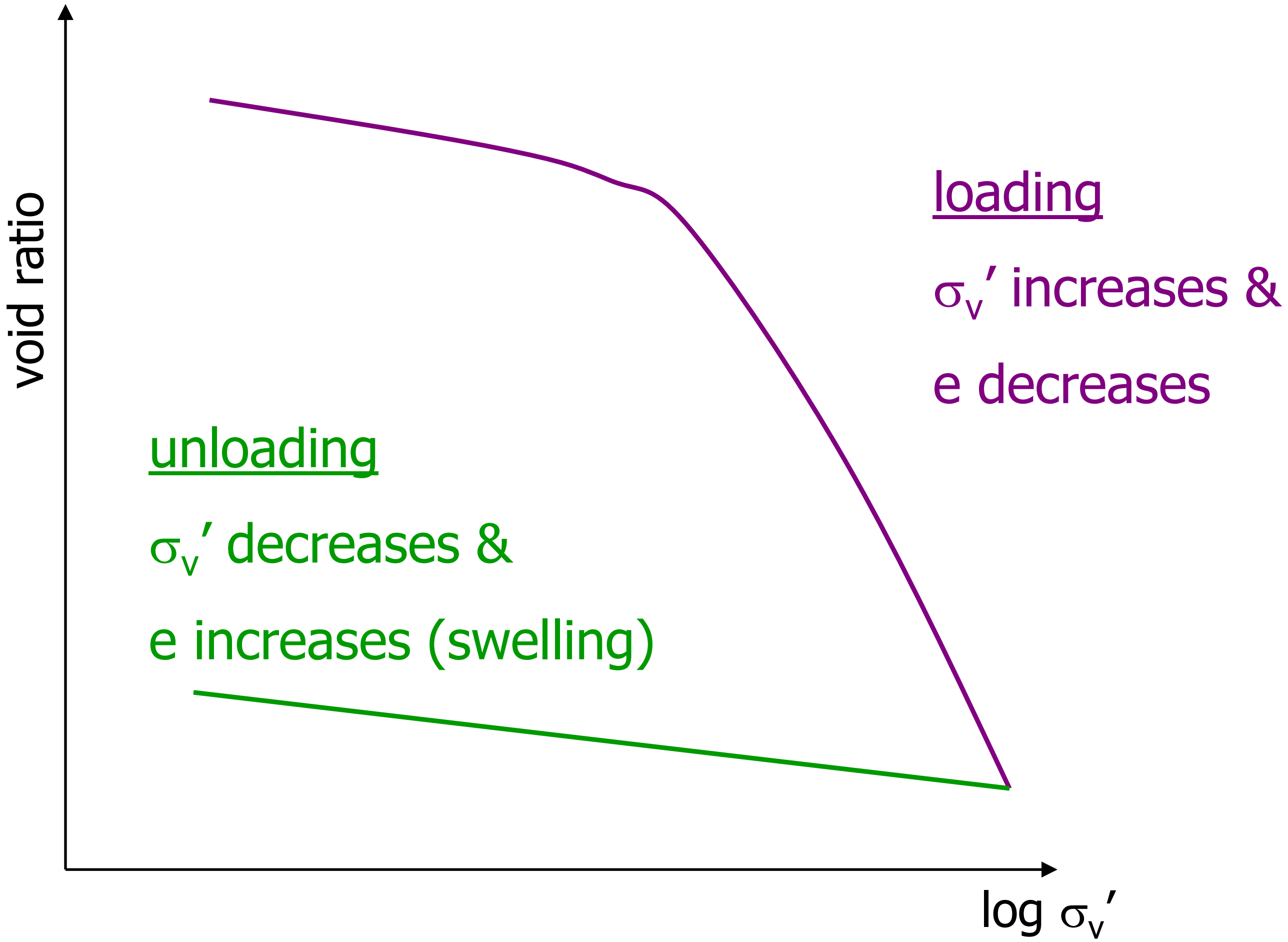
$$\Delta e_1 = \frac{\Delta H_1}{H_o}(1 + e_o)$$

Consolidation Test

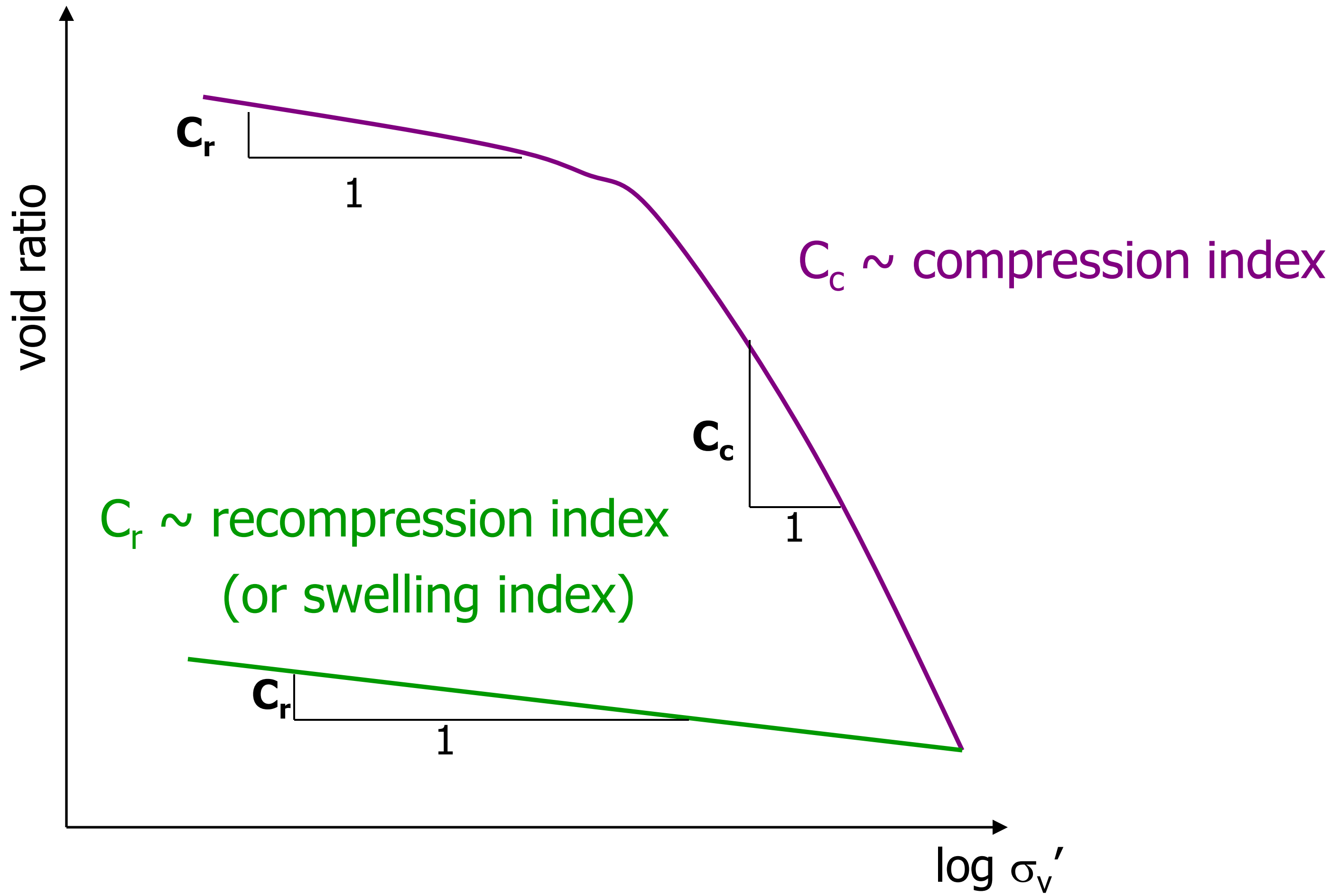


e – log σ_v' plot

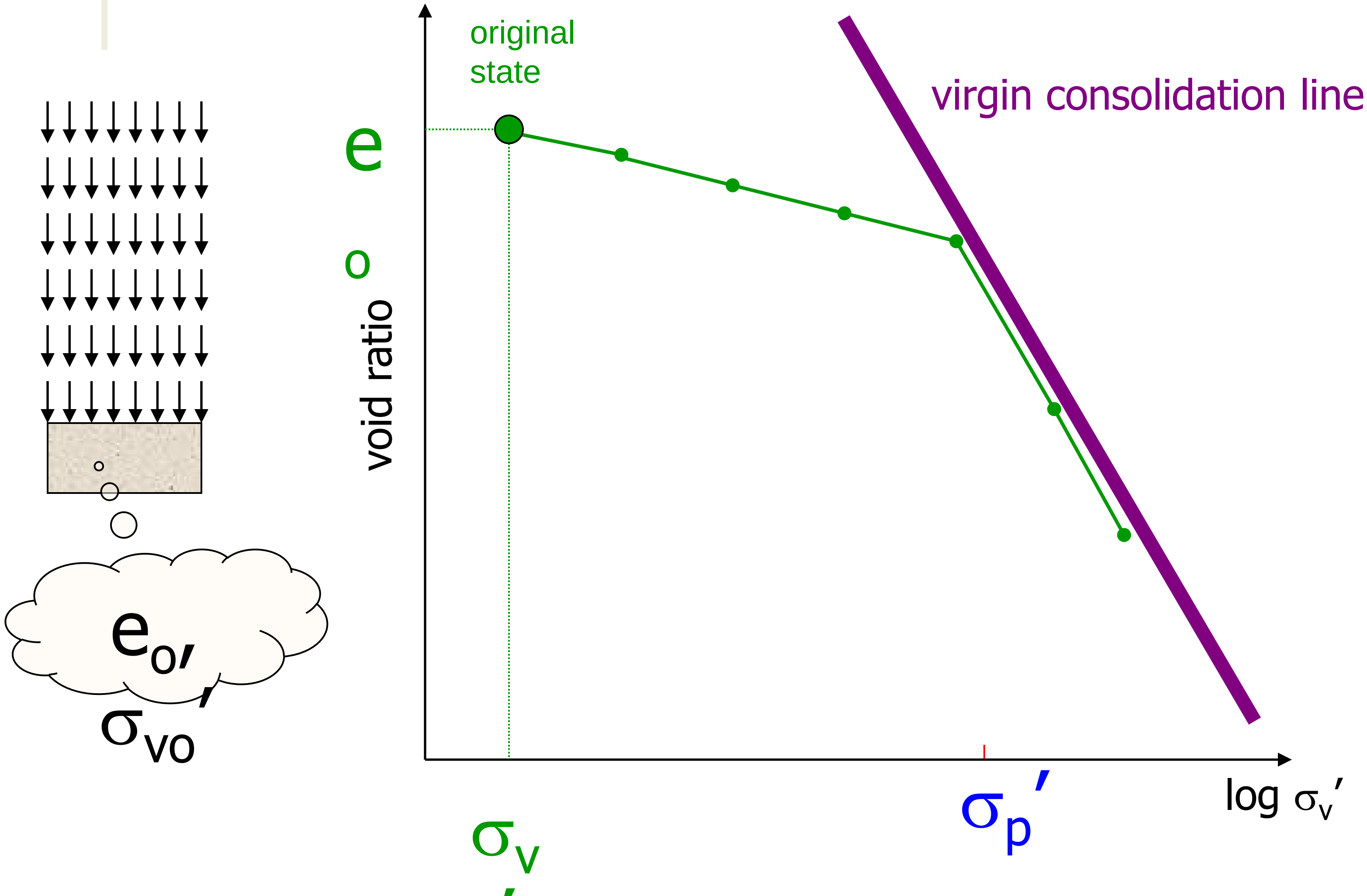
- from the above data



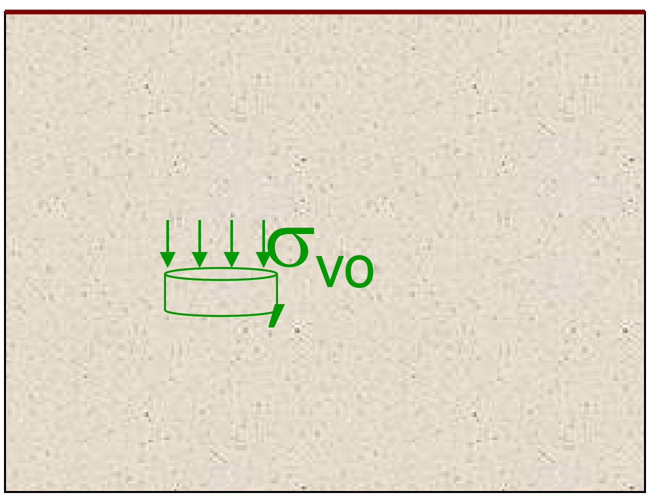
Compression and recompression indices



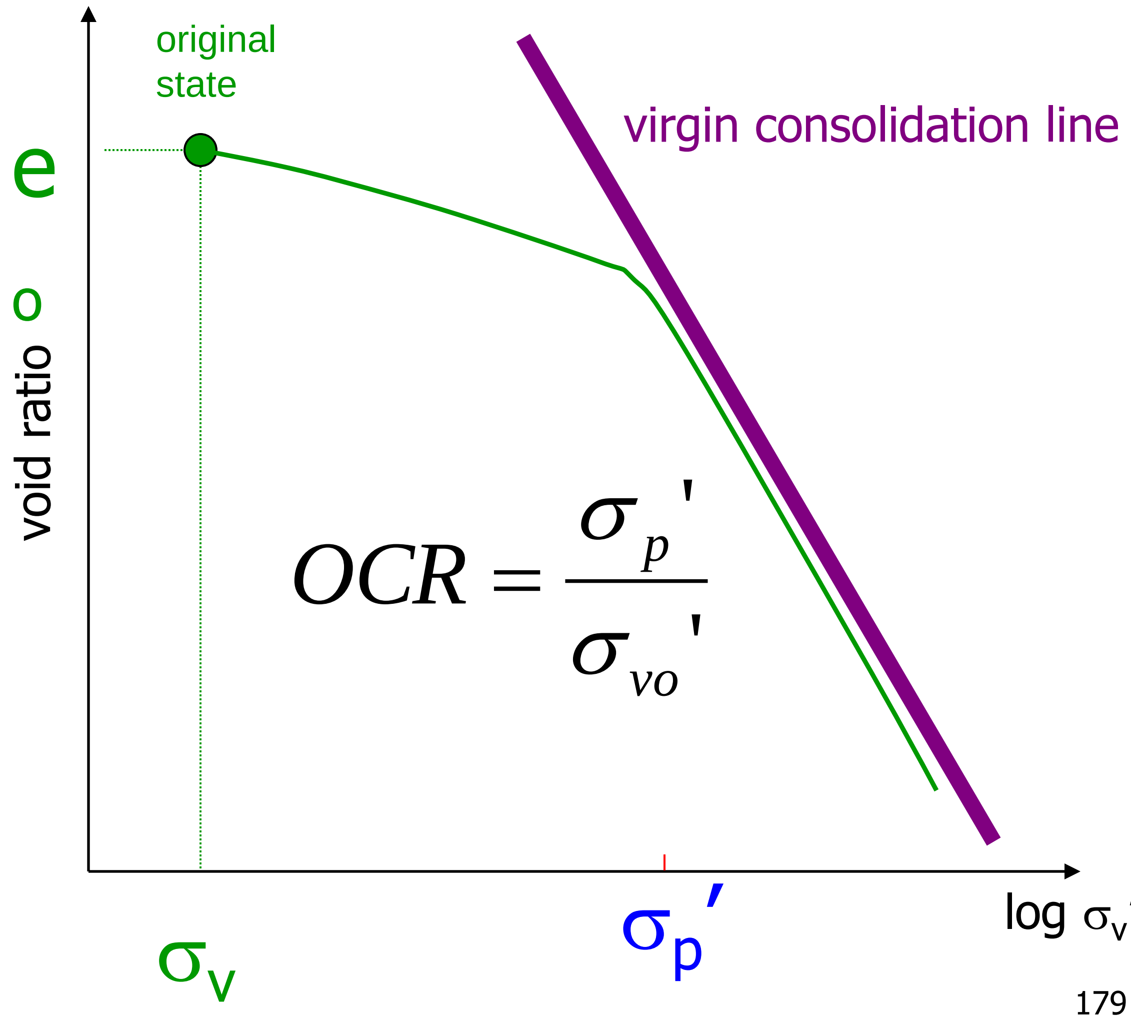
Virgin Consolidation Line



Overconsolidation ratio (OCR)

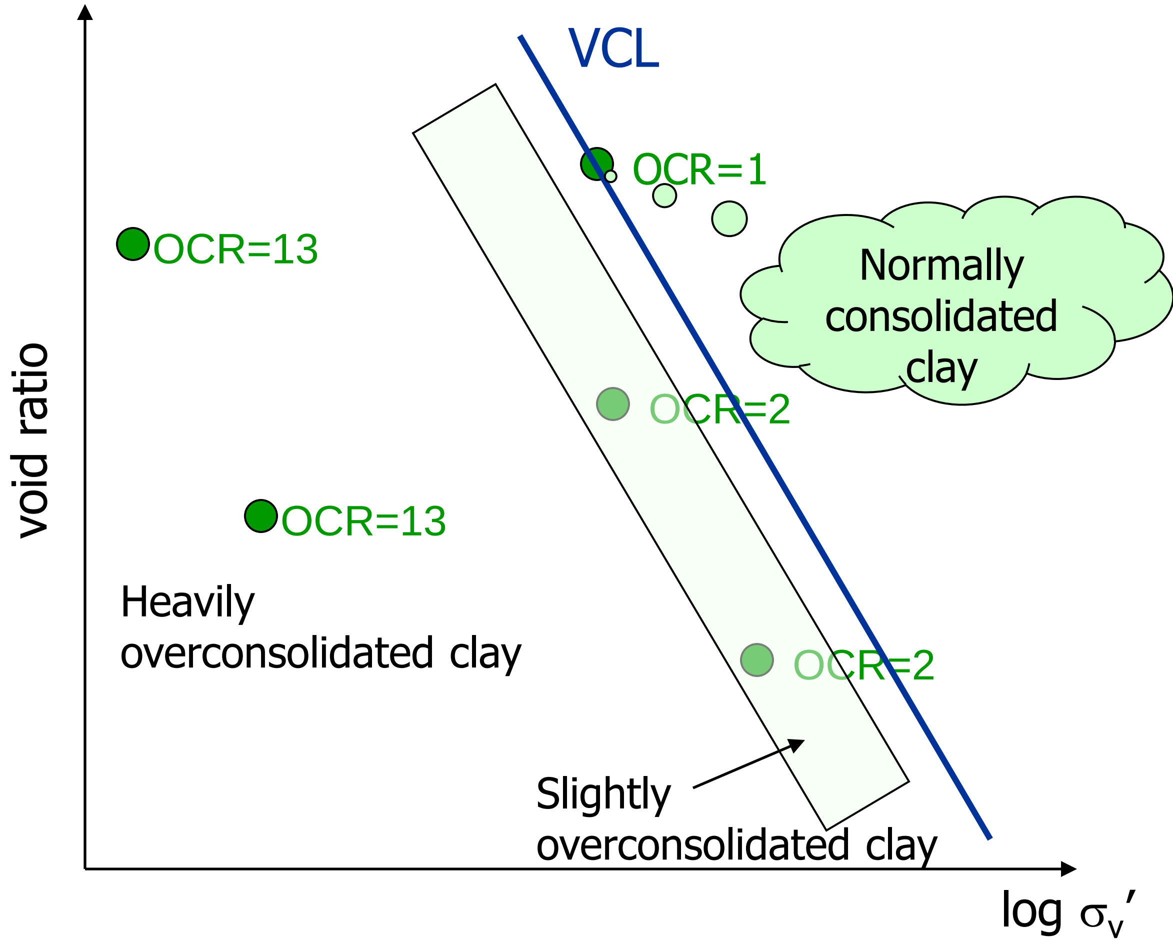


Field

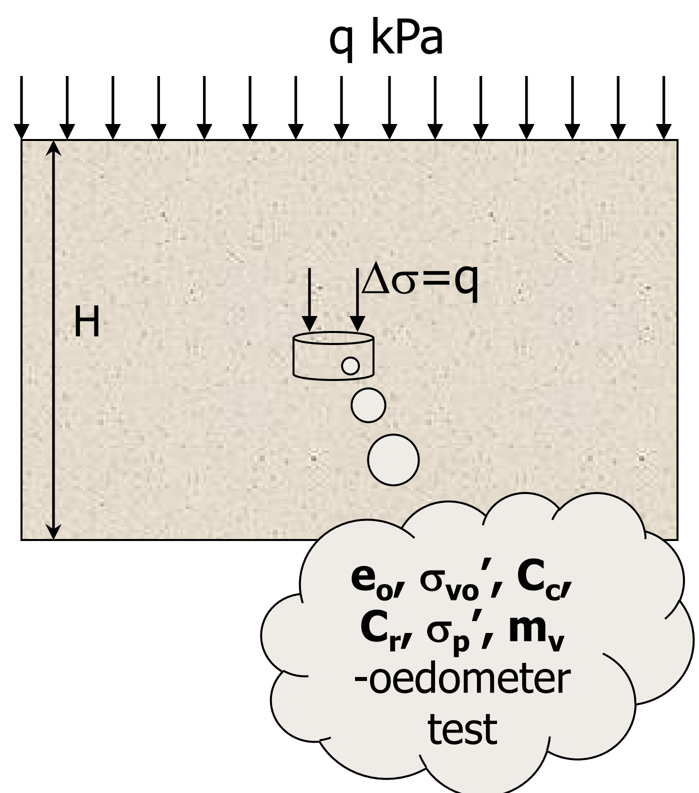


Overconsolidation ratio (OCR)

~current state



Two different ways to estimate the consolidation settlement:



(a) using m_v

$$\text{settlement} = m_v\, \Delta\sigma\, H$$

(b) using e-log σ_v' plot

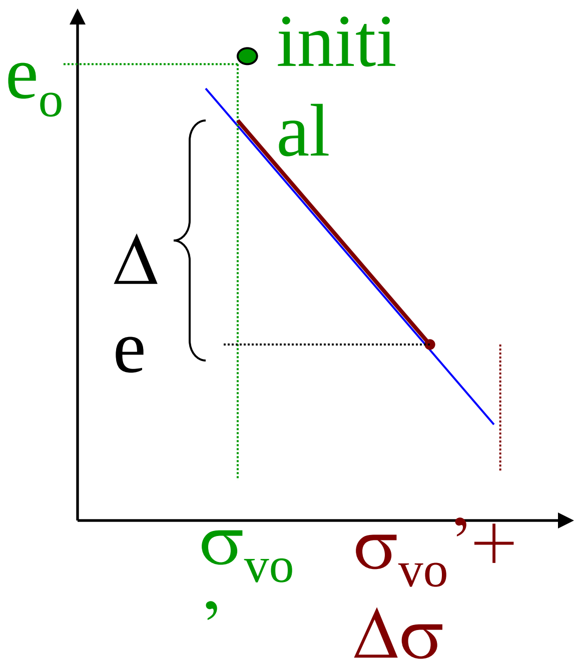
next slide

$$settlement = \frac{\Delta e}{1+e_o} H$$

~ computing Δe using e-log σ_v' plot

If the clay is normally consolidated,

the entire loading path is along the VCL.



$$\Delta e = C_c \log \frac{\sigma_{vo}' + \Delta\sigma'}{\sigma_{vo}'}$$

~ computing Δe using e-log σ_v' plot

If the clay is over consolidated, and remains so by the end of consolidation,

$$\Delta e = C_r \log \frac{\sigma_{vo}' + \Delta\sigma'}{\sigma_{vo}'}$$

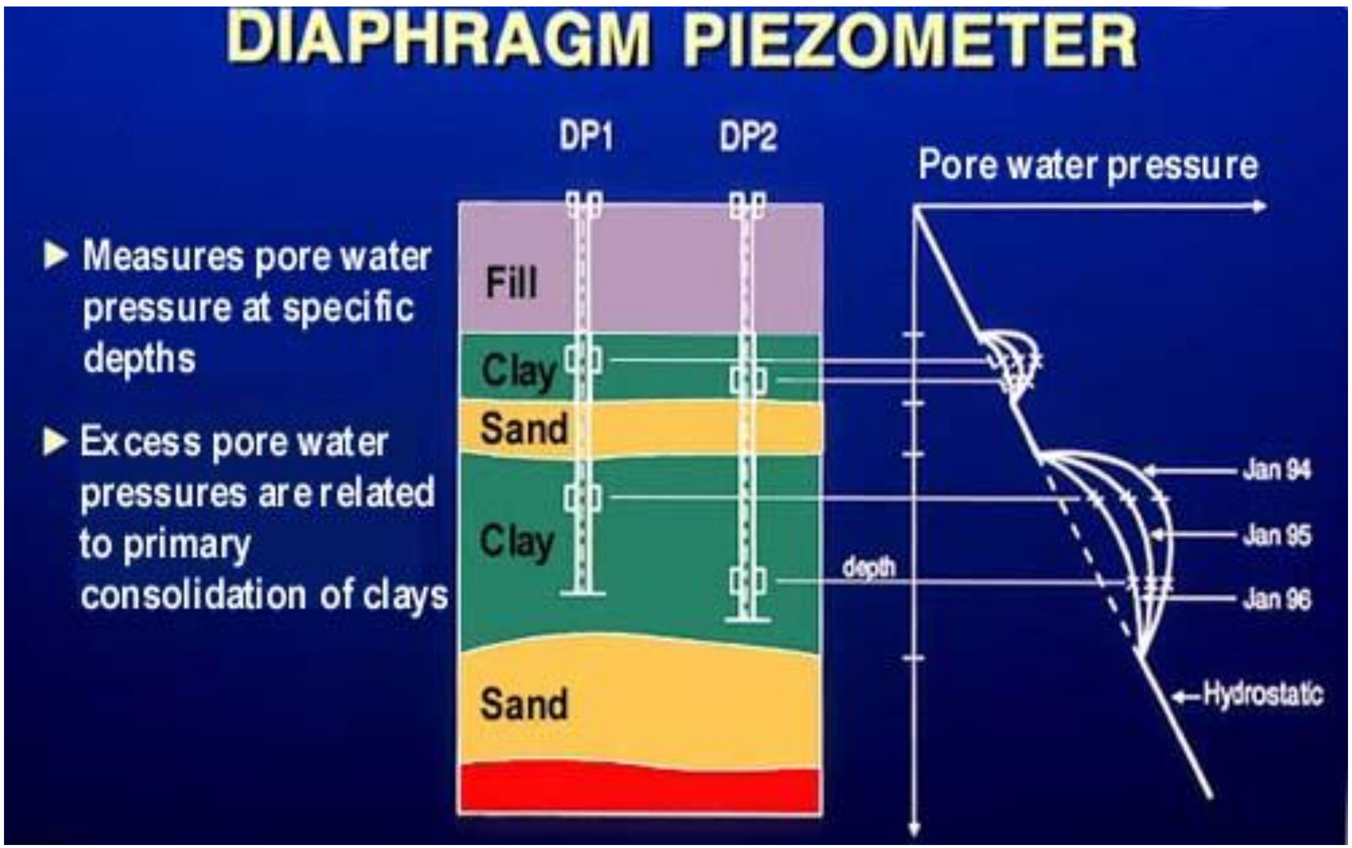
note the use of C_r

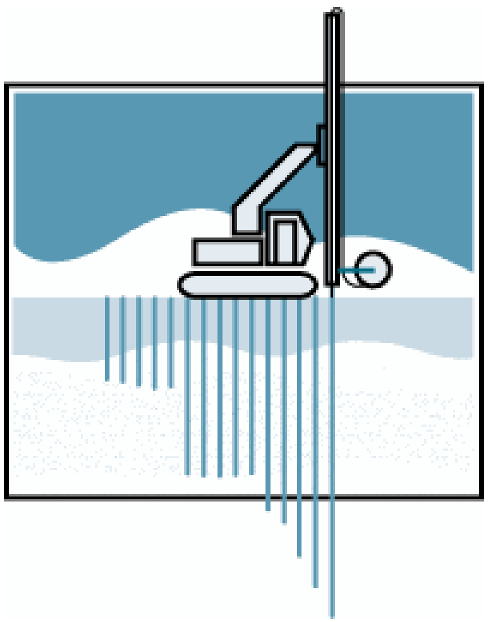
~ computing Δe using e-log σ_v' plot

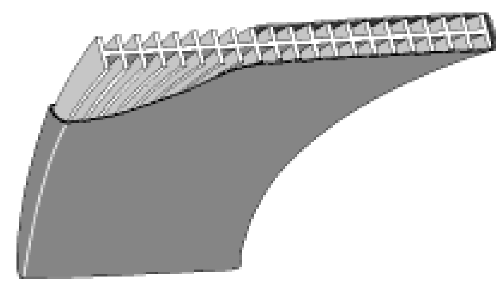
If an <u>overconsolidated</u> clay becomes normally consolidated by the end of consolidation,

$$\Delta e = C_r \log \frac{\sigma_p'}{\sigma_{vo}'} + C_c \log \frac{\sigma_{vo}' + \Delta\sigma'}{\sigma_p'}$$

Piezometers measure pore pressures and thus indicate when the consolidation is over.







Cross section of PVD

Installation

Prefabricated Vertical Drains to Accelerate Consolidation

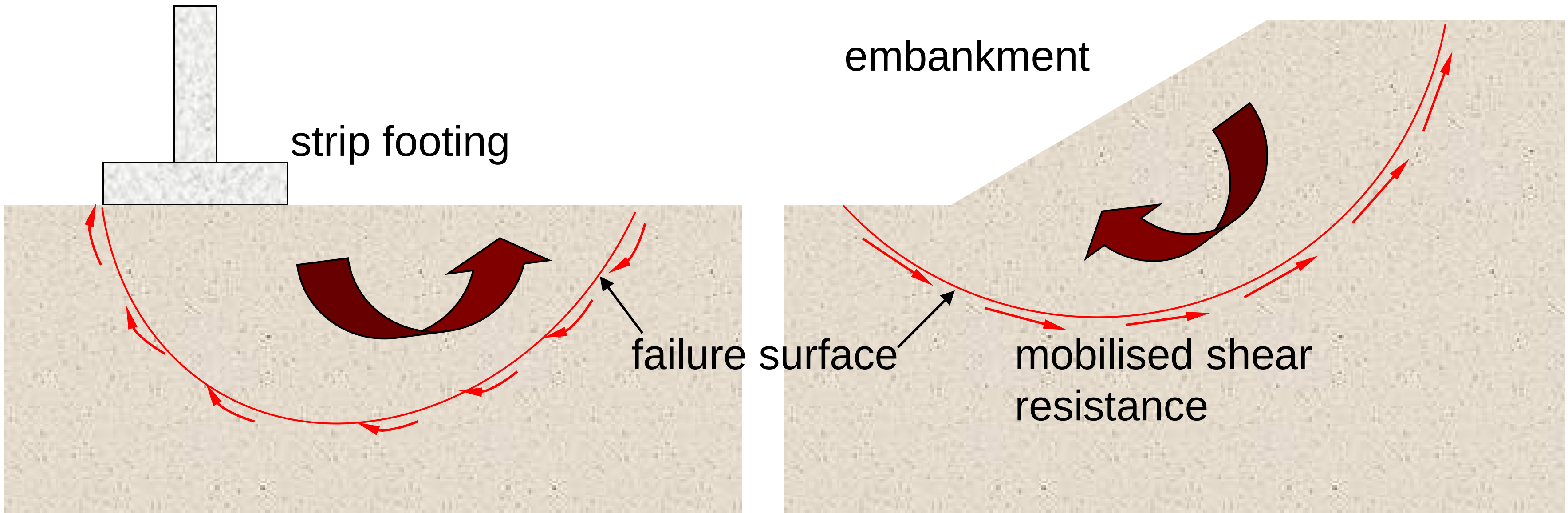
Installation of PVDs

UNIT V

SHEAR STRENGTH OF SOILS

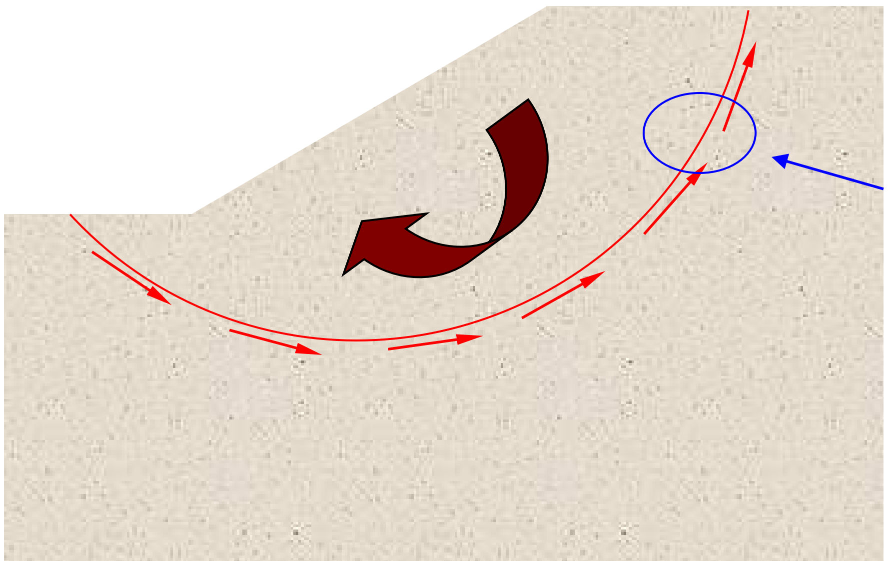
Shear failure

Soils generally fail in shear



At failure, shear stress along the failure surface reaches the shear strength.

Shear failure

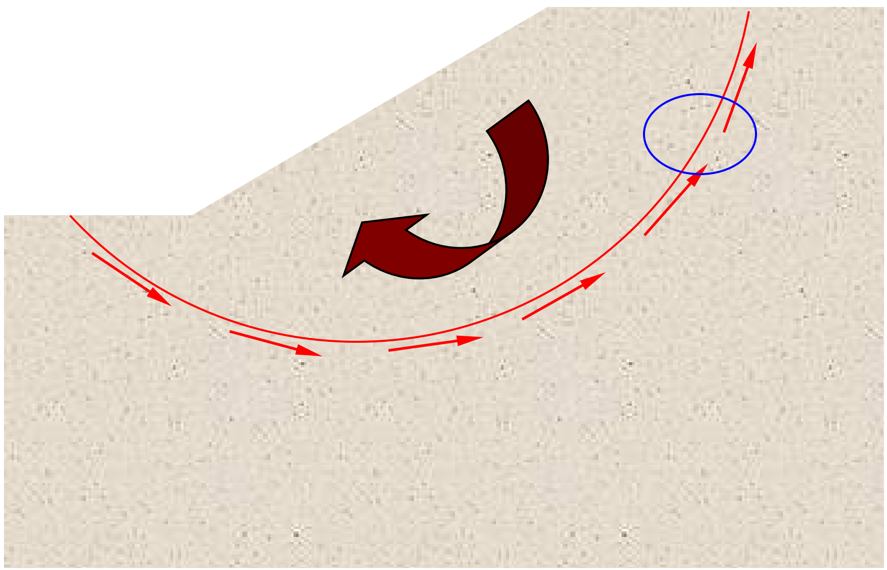


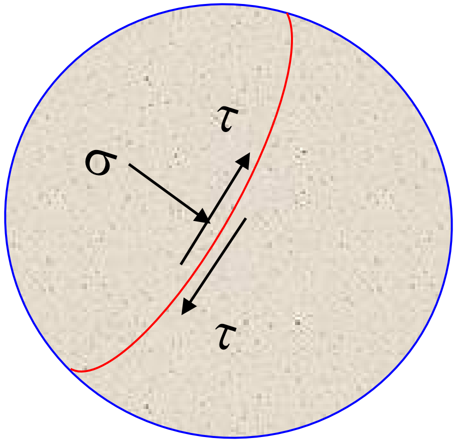
failure surface

The soil grains slide over each other along the failure surface.

No crushing of individual grains.

Shear failure

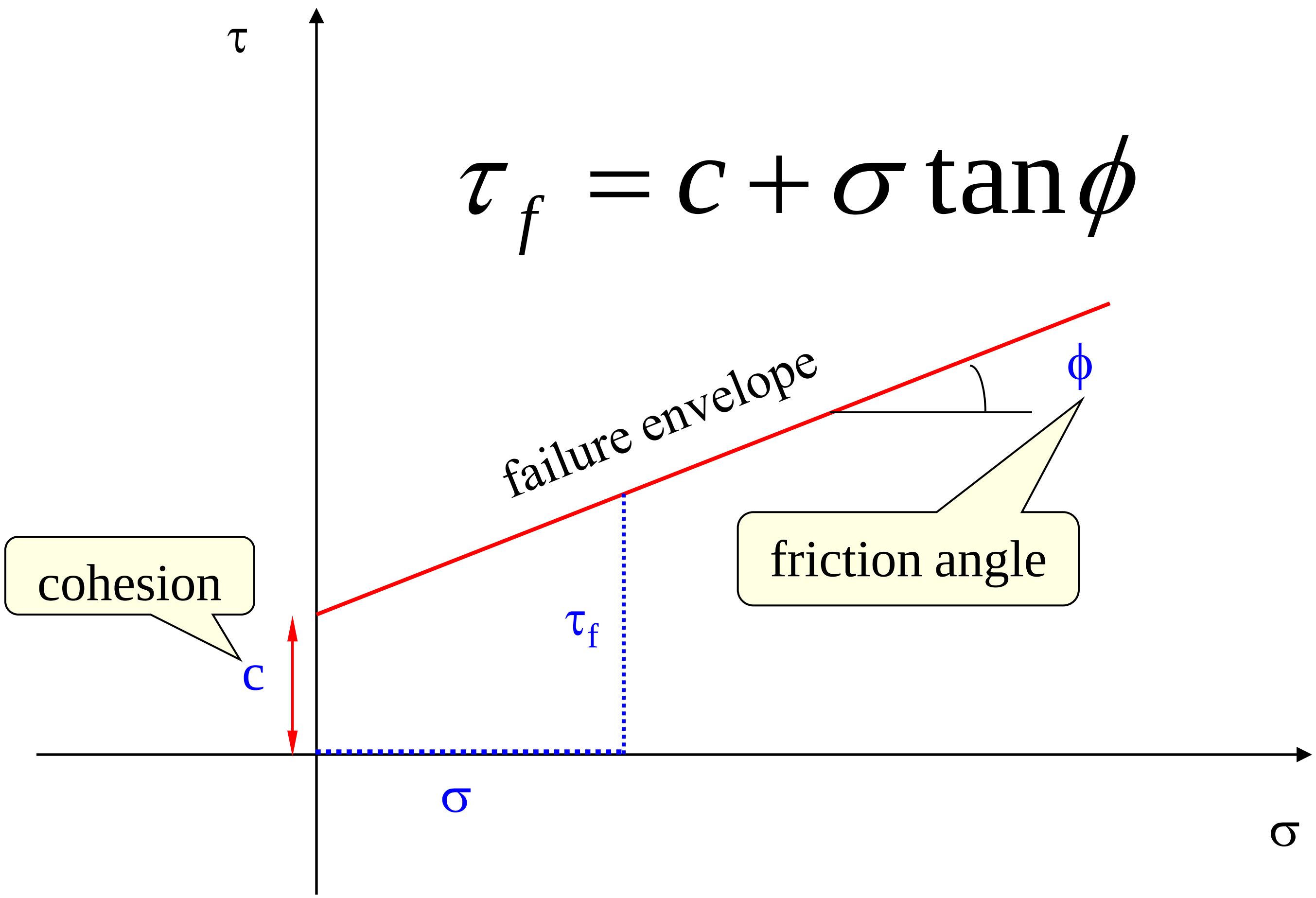




At failure, shear stress along the failure surface (τ) reaches the shear strength (τ_f).

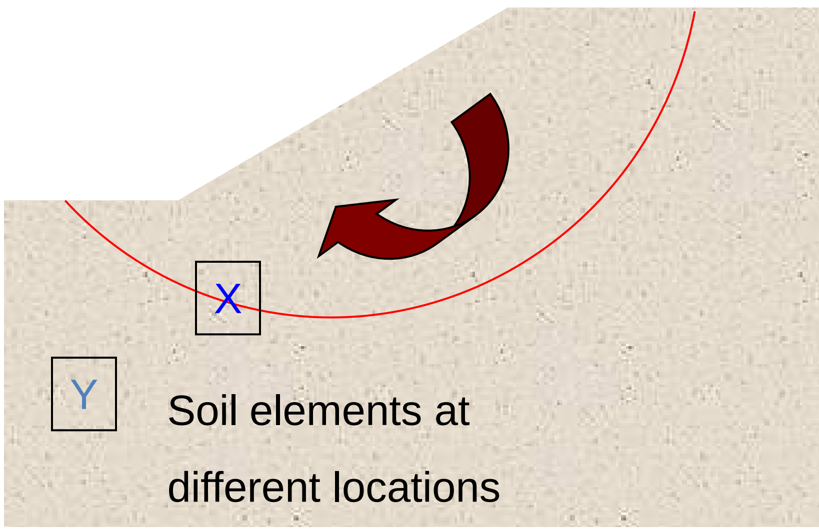
Mohr-Coulomb Failure Criterion

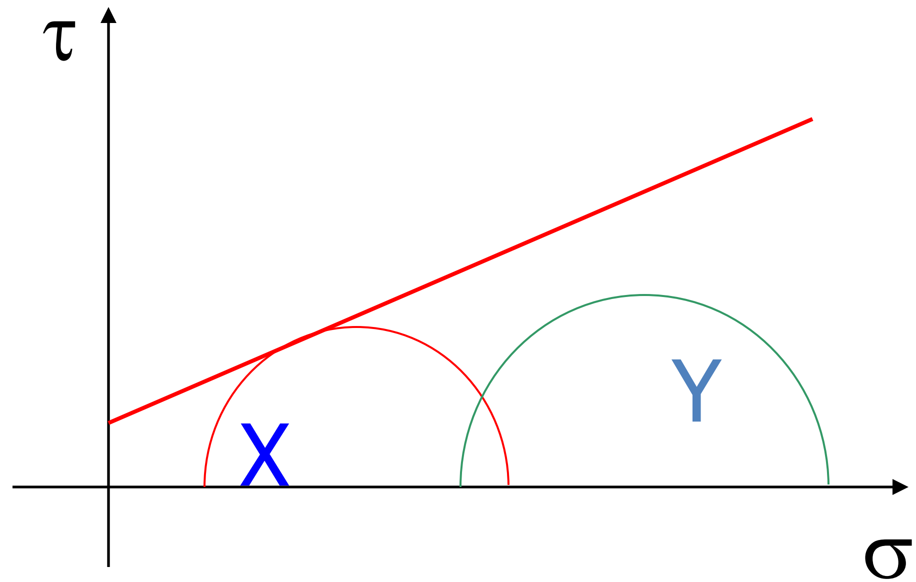
$$\tau_f = c + \sigma \tan\phi$$



τ_f is the maximum shear stress the soil can take without failure, under normal stress of σ.

Mohr Circles & Failure Envelope



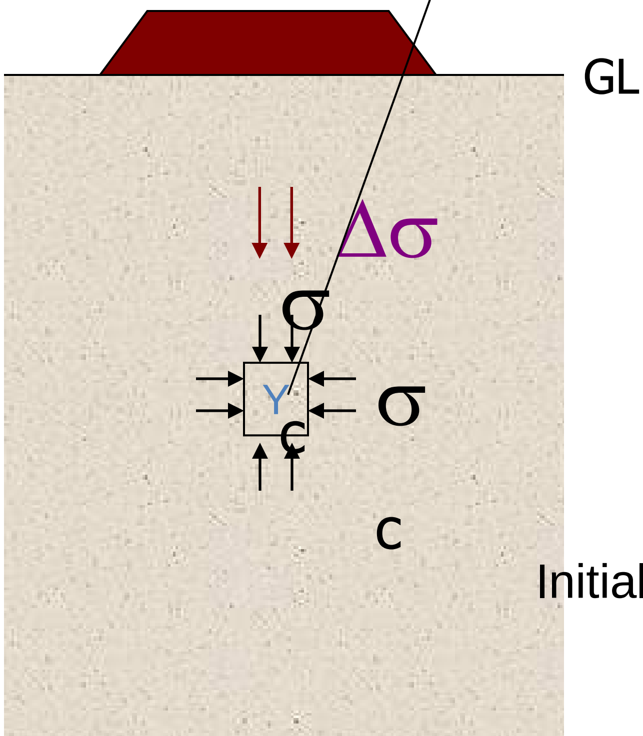


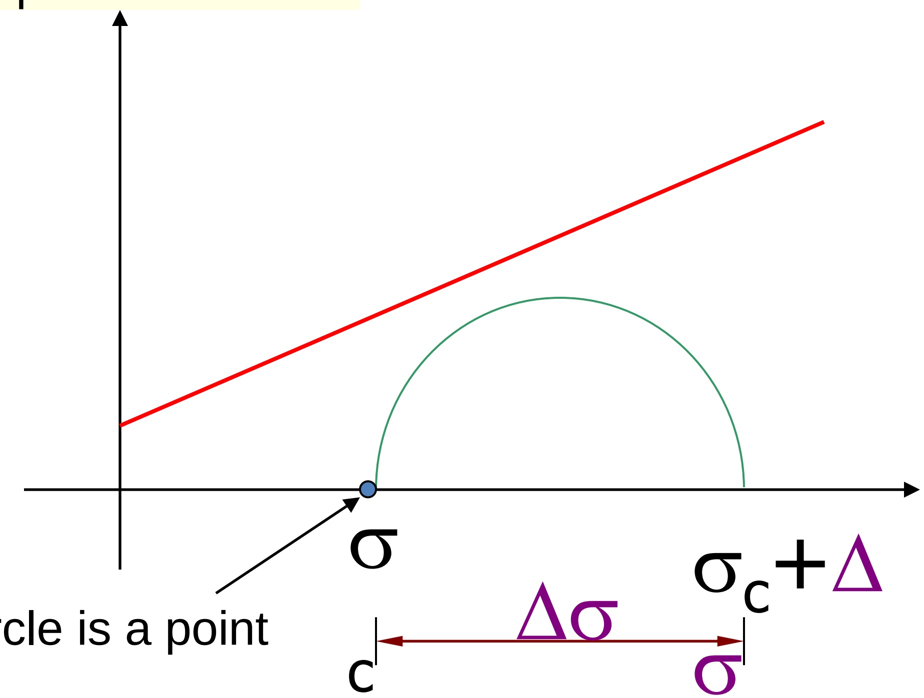
X ~ failure

Y ~ stable

Mohr Circles & Failure Envelope

The soil element does not fail if the Mohr circle is contained within the envelope

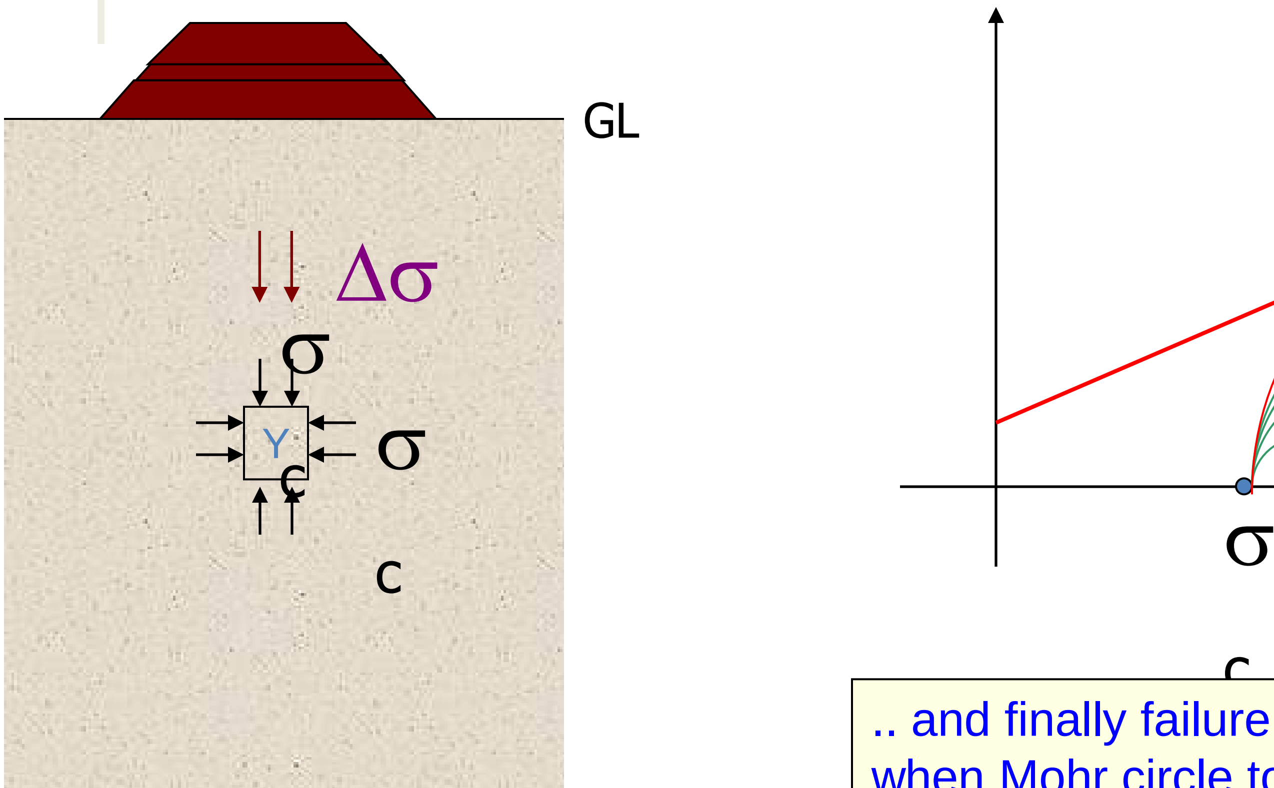




Initially, Mohr circle is a point

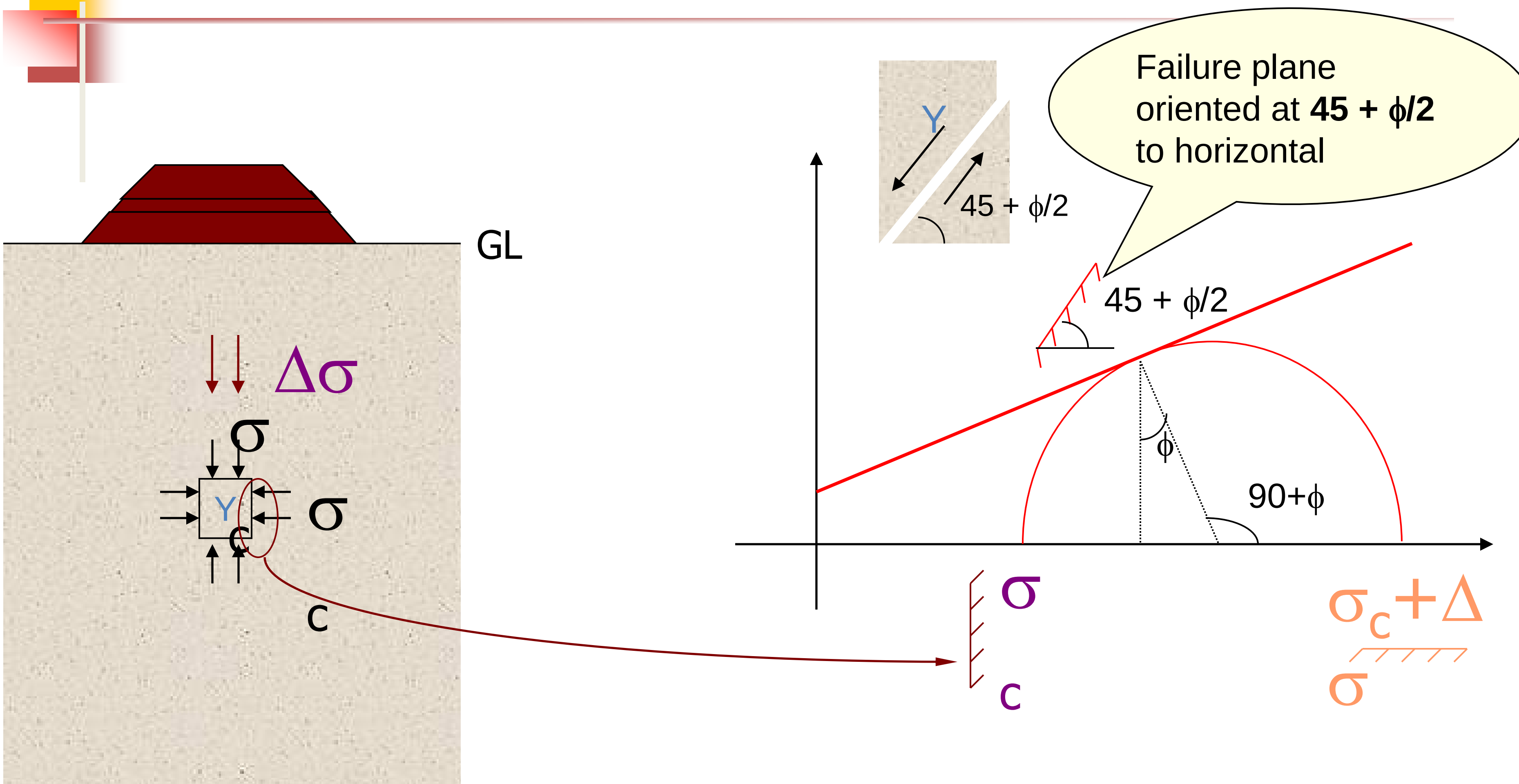
Mohr Circles & Failure Envelope

As loading progresses, Mohr circle becomes larger…

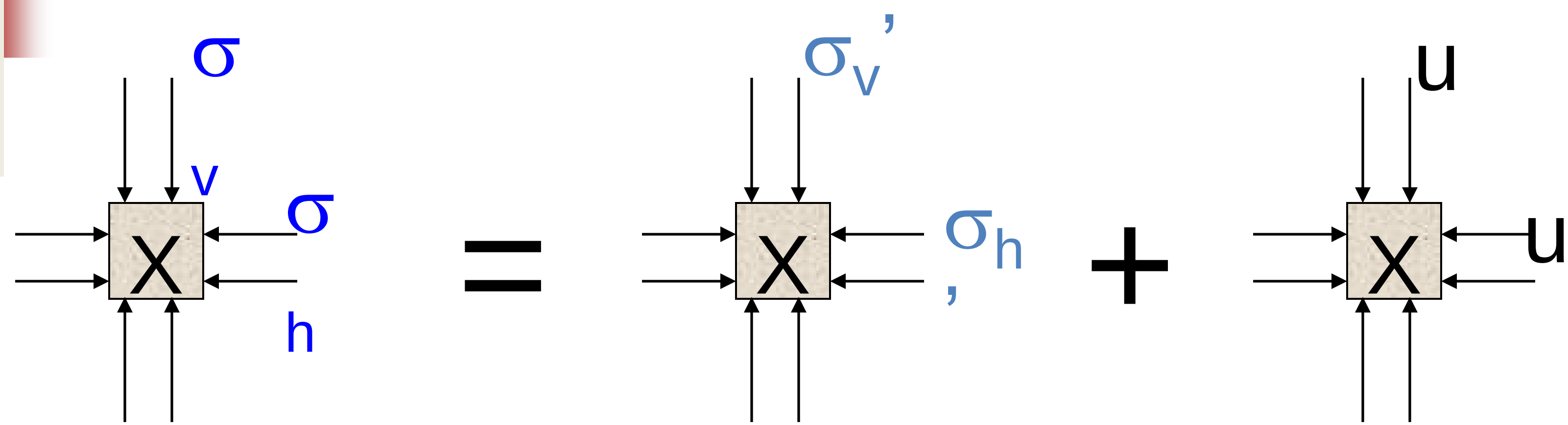


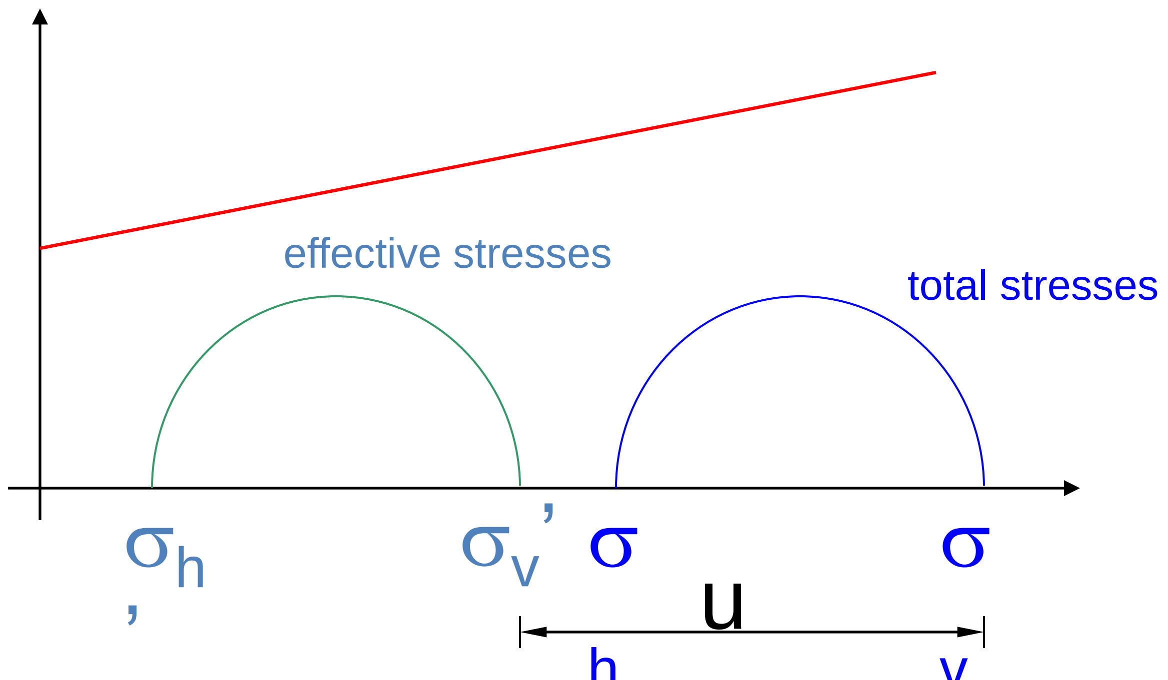
.. and finally failure occurs when Mohr circle touches the envelope

Orientation of Failure Plane



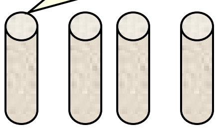
Mohr circles in terms of σ & σ'

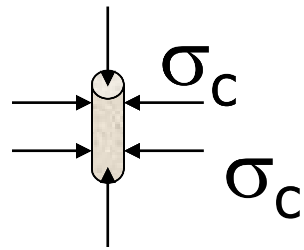




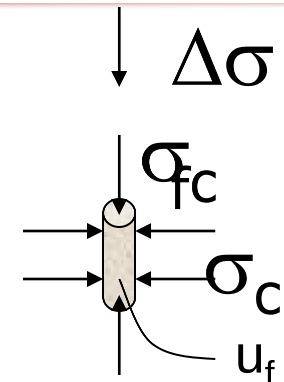
Envelopes in terms of σ & σ'

Identical specimens initially subjected to different isotropic stresses (σ_c) and then loaded axially to failure





Initially…

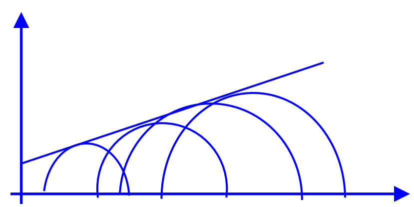


Failure

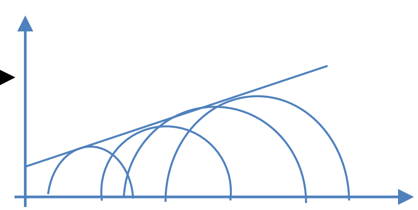
At failure,

$\sigma_3 = \sigma_c$; $\sigma_1 = \sigma_c + \Delta\sigma_f$

$\sigma_3' = \sigma_3 - u_f$; $\sigma_1' = \sigma_1 - u_f$



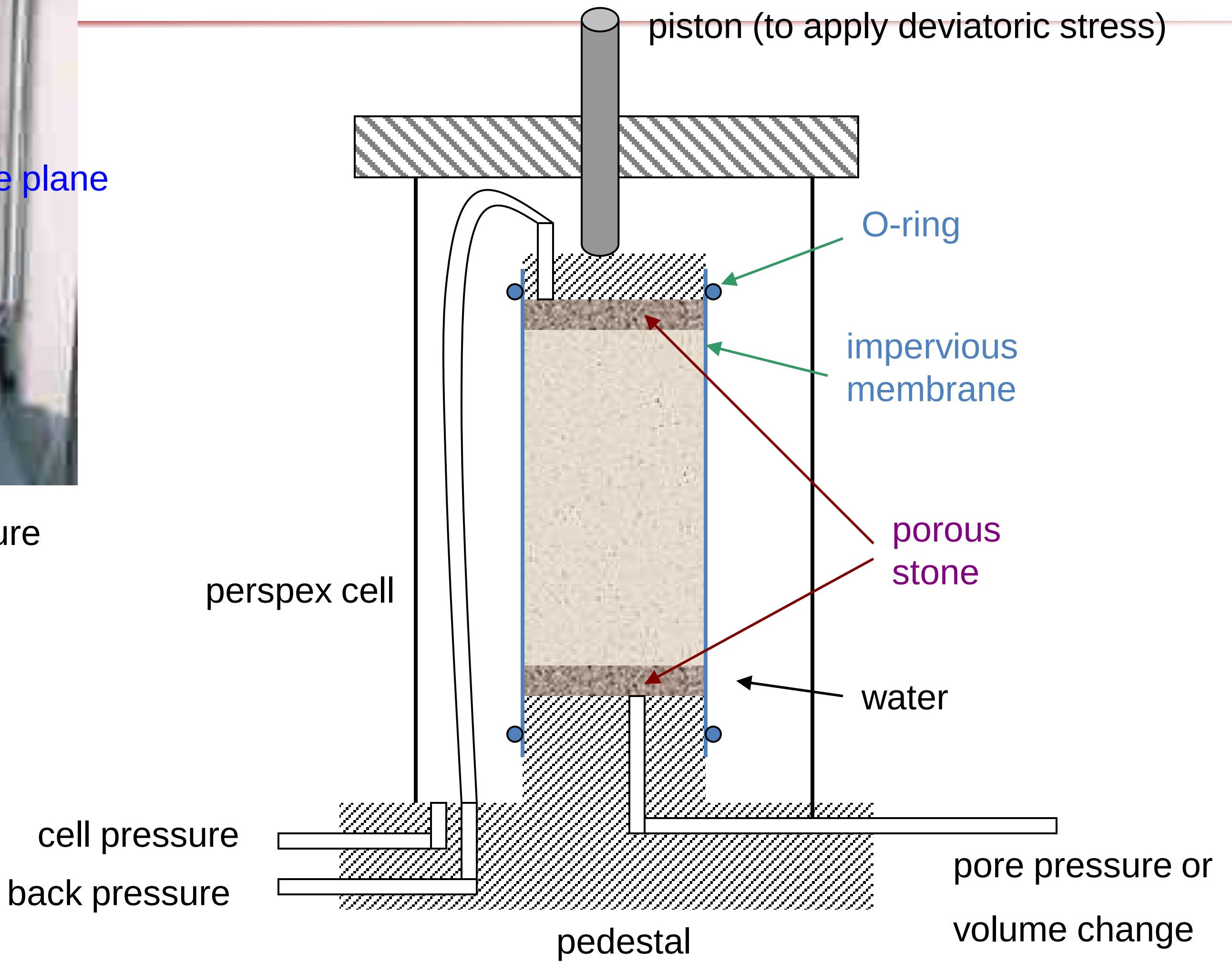
c, φ in terms of σ



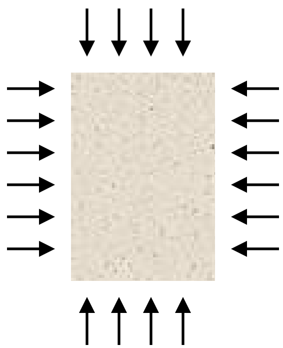
c', φ' in terms of σ'

Triaxial Test Apparatus

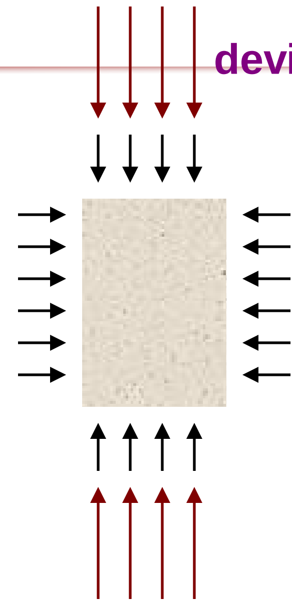
soil sample at failure



Types of Triaxial Tests



Under all-around cell pressure σ_c



Shearing (loading)

Is the drainage valve open?

yes → **Consolidated sample**

no → **Unconsolidated sample**

Is the drainage valve open?

yes → **Drained loading**

no → **Undrained loading**

Types of Triaxial Tests

Depending on whether drainage is allowed or not during

- ❖ initial isotropic cell pressure application, and
- ❖ shearing,

there are three special types of triaxial tests that have practical significances. They are:

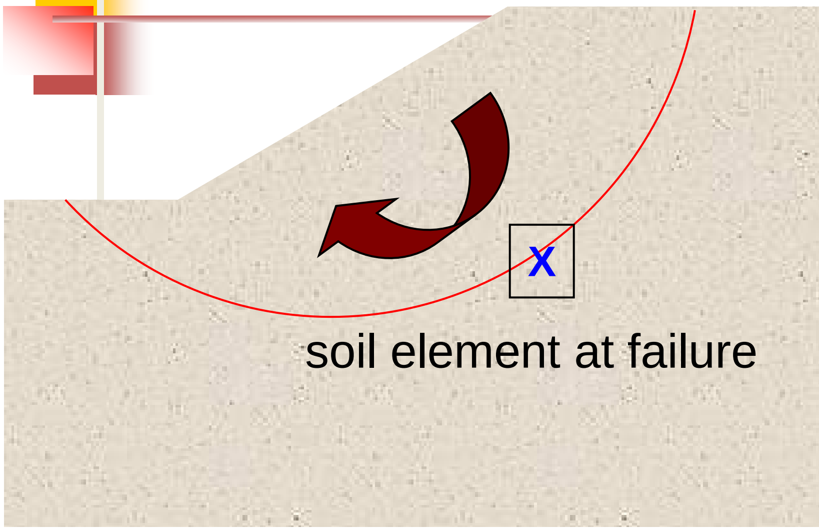
Consolidated Drained (CD) test
Consolidated Undrained (CU) test
Unconsolidated Undrained (UU) test

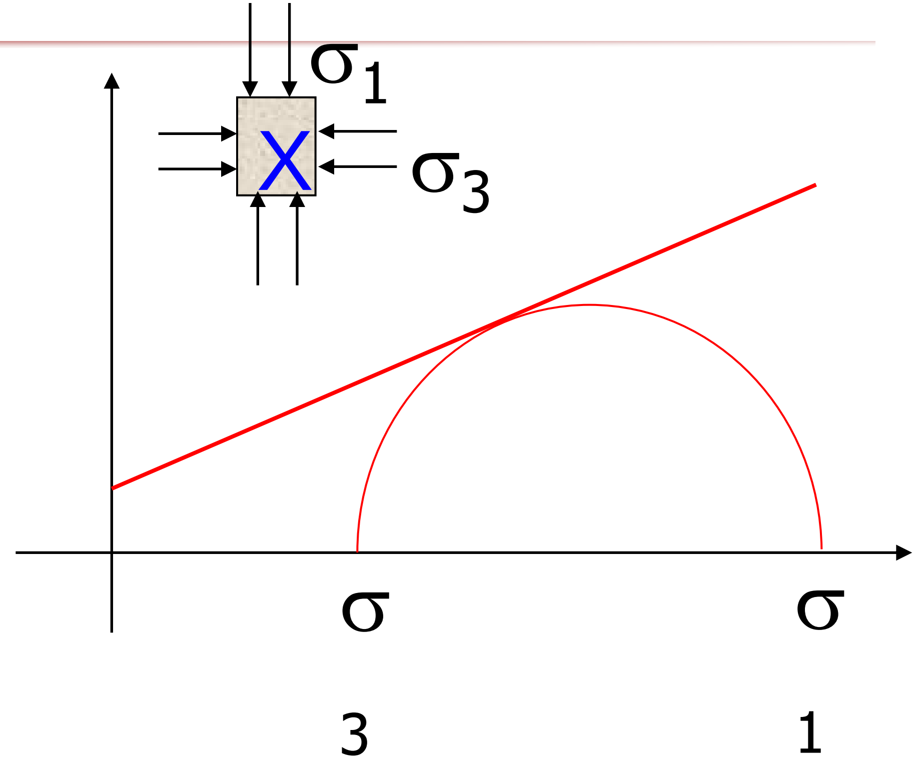
For unconsolidated undrained test, in terms of total stresses, $\phi_u = 0$

Granular soils have no cohesion.
$c = 0$ & $c' = 0$

For normally consolidated clays, $c' = 0$ & $c = 0$.

σ_1- σ_3 Relation at Failure

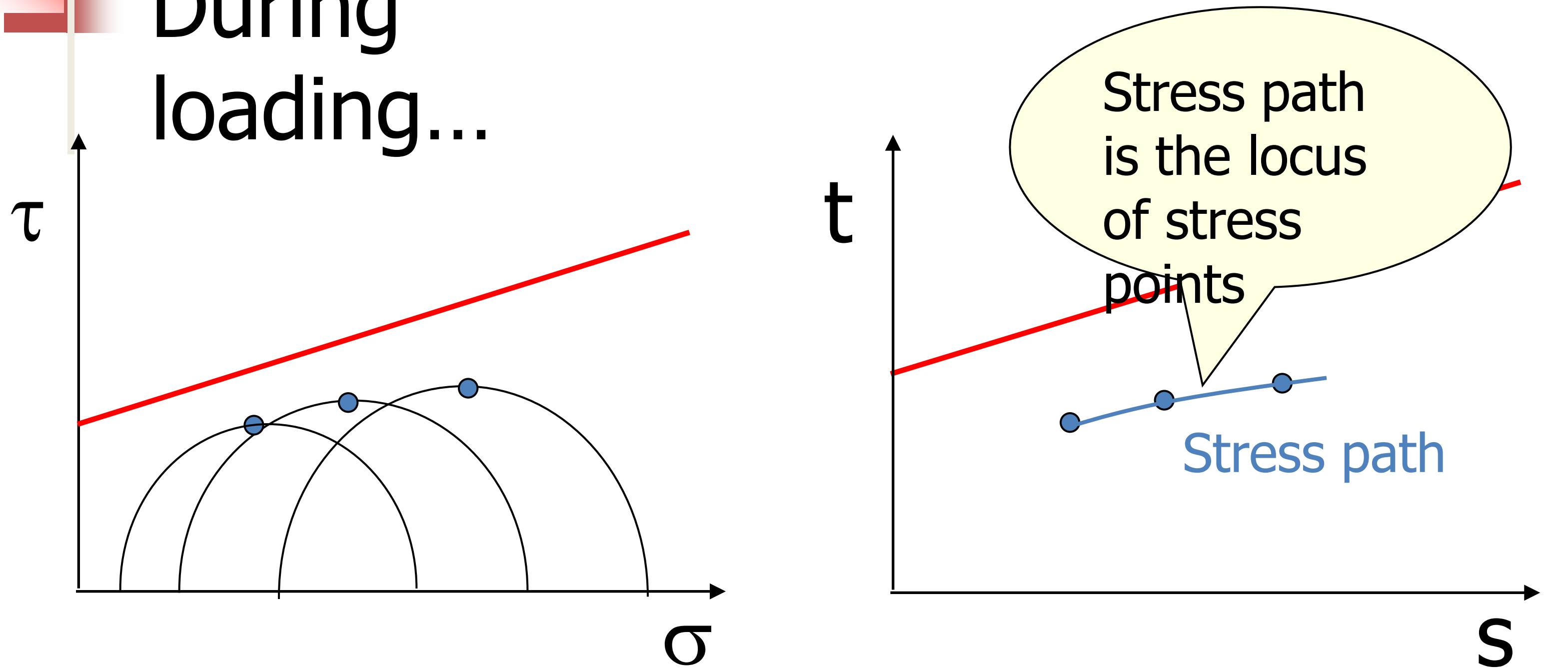




$$\sigma_1 = \sigma_3 \tan^2(45 + \phi/2) + 2c\tan(45 + \phi/2)$$

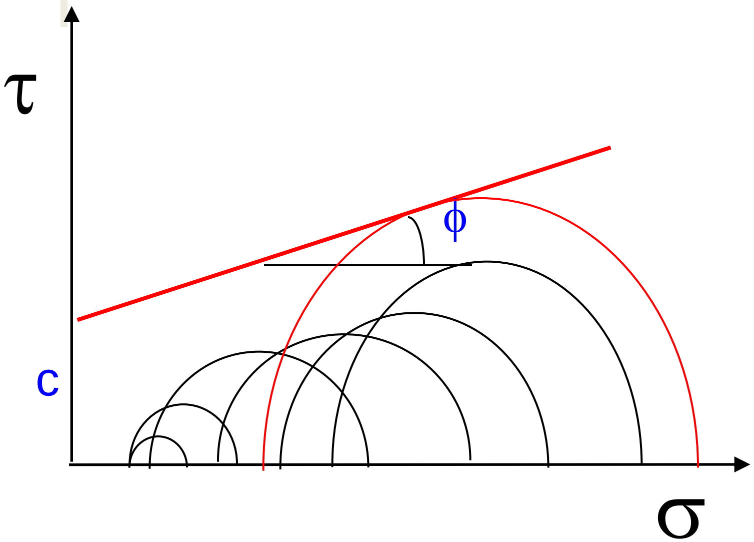
$$\sigma_3 = \sigma_1 \tan^2(45 - \phi/2) - 2c\tan(45 - \phi/2)$$

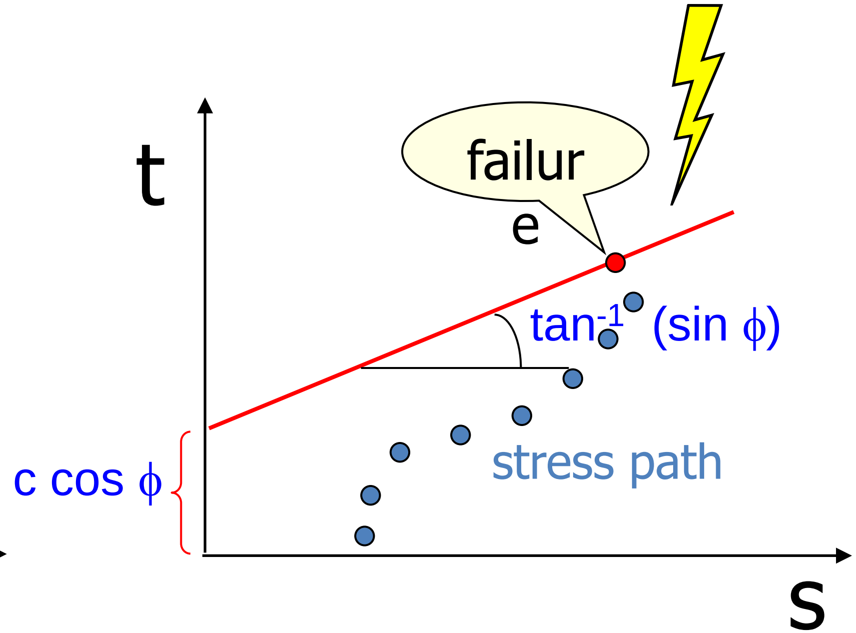
During loading…



Stress path is a convenient way to keep track of the progress in loading with respect to failure envelope.

Failure Envelopes

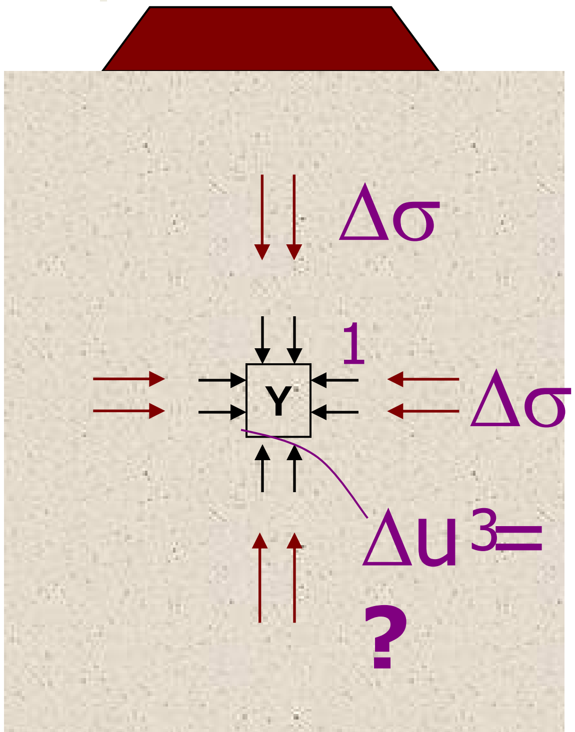




During loading (shearing)....

Pore Pressure Parameters

A simple way to estimate the pore pressure change in undrained loading, in terms of total stress changes ~ after Skempton (1954)



Skempton's pore pressure parameters A and B

$$\Delta u = B\left[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)\right]$$