GEOTECHNICAL ENGINEERING

Unit I

Soil Classification and Compaction

Definition of Soil

It is defined as

unconsolidated material

composed of solid particles

produced by

mechanical and chemical

disintegration of rocks

Soil Mechanics

Soil Mechanics is the

Application of

laws of mechanics and hydraulics

to

Engineering problems

Dealing with soil

Origin of Soil

Soil is formed by weathering of rocks

due to

mechanical disintegration (or) chemical decomposition;

when

the rock surface is exposed to atmosphere

for long duration.

Types of Soil

Soil can be classified into two broad categories:

1. Residual Soil
2. Transported Soil

Residual soils

If the soil deposited at the place of its formation
just near (or) above parent rock,
it is known as residual soil.
The properties of the residual soil resembles
that of the parent rock in general

The depth of residual soils varies
from 5 to 20 m.

(2) Transported soils

Soil transported from the origin called transported soil.

soil has been deposited at a place

away from the place of origin

by various transporting agents

such as air, water and ice or snow

The engineering properties of transported soil at a place of deposition are entirely different from the properties of the parent rock

Most of the soil deposits are transported soil only.

Classification of Transported Soil

Alluvial Soil

Aeolian Soil

Glaciers deposit

Marine Deposit

Colluvial soils

Alluvial Soil

- Running water carries large quantities of soil either in suspension (or) by rolling on a long bed.
- Water erodes hills and deposits soils in the valleys.
- Deposits made in lakes are called as lacustrine deposits
- Deposits made when the flowing water carries soil to ocean is called marine deposits

Aeolian Soil

A type of **soil** that is transported from one place to another by the wind called **Aeolian soil**

It consist primarily of
sand or silt-sized particles.
The particles size of the soil depends upon
the velocity of wind
The finer particles are carried
far away from the origin

Marine deposits:

These are mainly confined on

a long a narrow belt near the coast,

These are thick layers of sand above deep deposits of soft marine clays

These deposits have very low shearing strength and are highly compressible.

It contain a large amount of organic matter.

these are softly and highly plastic

Glaciers Deposits

- when annual snowfall is greater than snow melt

  then Snow accumulates along with soil
  it compresses into ice and begins to flow during summer

- Glaciers carry soil when they flow

  A..Glacial Till

  unconsolidated deposit directly by ice
- containing any proportion of gravels, sands, clay, boulders.,
- B.Glacial Outwash

  stratified deposits formed from melting of ice in the summer
  contains large rocks and boulders ,gravelly, sandy, stratified soils

Colluvial soils

Under the influence of gravity

Solid particles are removed from the mountains top and

get accumulated at the base of the steep slopes.

The soils thus formed are stony and are never stratified.

Different types of soils:

1. Bentonitte:

It is a type of clay with very high percentage of clay mineral montmorilonitte. It results from decomposition of volcanic ash.

2. Clay:

It consists of microscopic and sub microscopic particles The soil size is less than 0.002mm

3. Sand:

I t is a coarse grained soil having particles size between 0.075 to 4.75mm. The particles are visible in eye

4. Silt:

It is fine graded sand particles

size from 0.002 -0.0075mm.

The particles are not visible through eyes.

5. Gravel:

Coarse soil of size from 4.75 to 80mm

6. Cobbles:

These are large size particles in range of size 80 to 300mm

7. Kankar:

Impure form of limestone, it contains calcium carbonate.

8. Loam:

It is a mixture of sand, silt and clay in alternate layers

9. Boulders: These are large size particles of size>300mm

Cohesive soils:

Soil which absorbs water and having particles attraction

such that

it deforms plastically at varying water content are known as cohesive soil

Example: clay sand, plastic silt and clay.

cohesion less soils

The soil composed of bulky grains are cohesive less soils

Its plasticity effects is insignificant

Example: non plastic silt and sand gravel

Phase Relations of Soil

Symbols for Phase Relations of soils

$V \rightarrow$ *Total volume* *(solids + water + air).*

$V_a \rightarrow$ *Volume of air.*

$V_V \rightarrow$ *Volume of voids (water + air).*

$V_S \rightarrow$ *Volume of solids.*

$V_W \rightarrow$ *Volume of water.*

Phase Relations of Soil

Symbols for Phase Relations of soils

$e \longrightarrow$ *Voids ratio.*

G or $G_S \longrightarrow$ *Specific gravity of the solids of a soil.*

$n \longrightarrow$ *Porosity.*

$S \longrightarrow$ *Degree of saturation.*

$w \longrightarrow$ *Water content (also known as the moisture content).*

$W_S \longrightarrow$ *Weight of solids.*

$W_W \longrightarrow$ *Weight of water.*

Phase Relations of Soil

Symbols for Phase Relations of soils

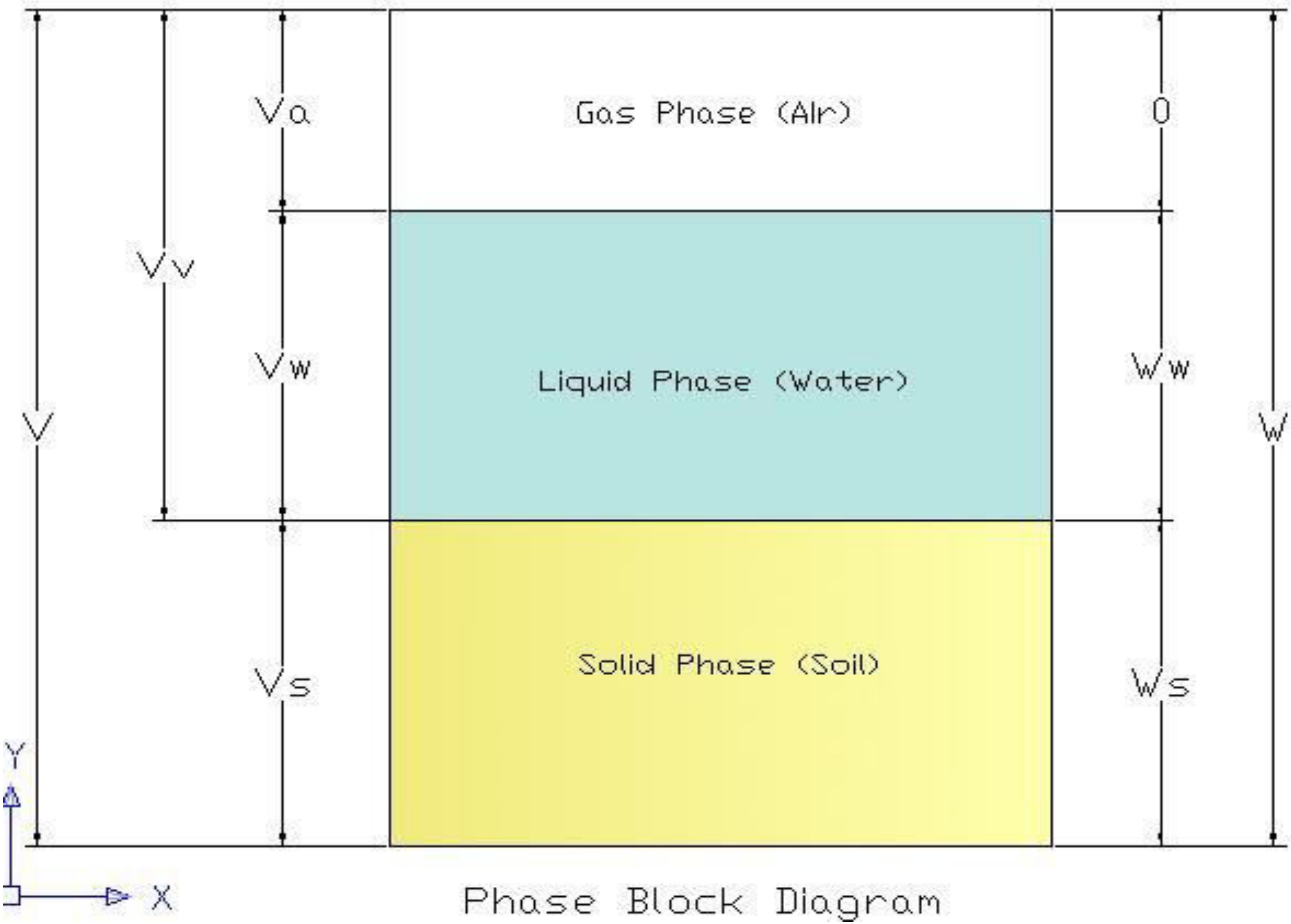
$$\gamma = \gamma_b = Bulk\ Unit\ Weight\ of\ Soil$$

$$\gamma_W = Unit\ Weight\ of\ Water$$

$$\gamma_d = Dry\ Unit\ Weight\ of\ Soil$$

$$\gamma_{Sub} = \gamma' = Submerged\ Unit\ Weight\ of\ Soil$$

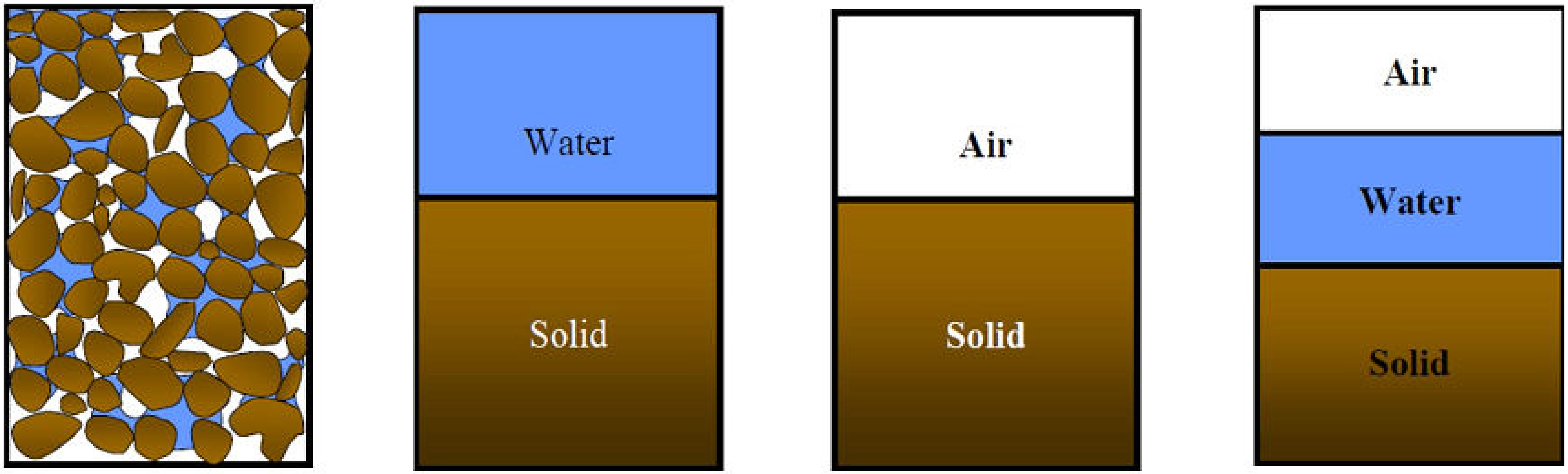
$$\gamma_{Sat} = saturated\ Unit\ Weight\ of\ Soil$$



Phase Block Diagram

Total volume, $V = V_a + V_w + V_v$

Three-phase System



Mineral Skeleton | Fully Saturated | Dry Soil | Partially Saturated

Soils can be classified as
partially saturated (with both air and water present),
fully saturated (no air content)
perfectly dry (no water content).

In a saturated soil or a dry soil,
the three-phase system thus reduces to two phases only, as shown.

Volume Relations

As the amounts of both water and air are variable,

the volume of solids is taken as the reference quantity.

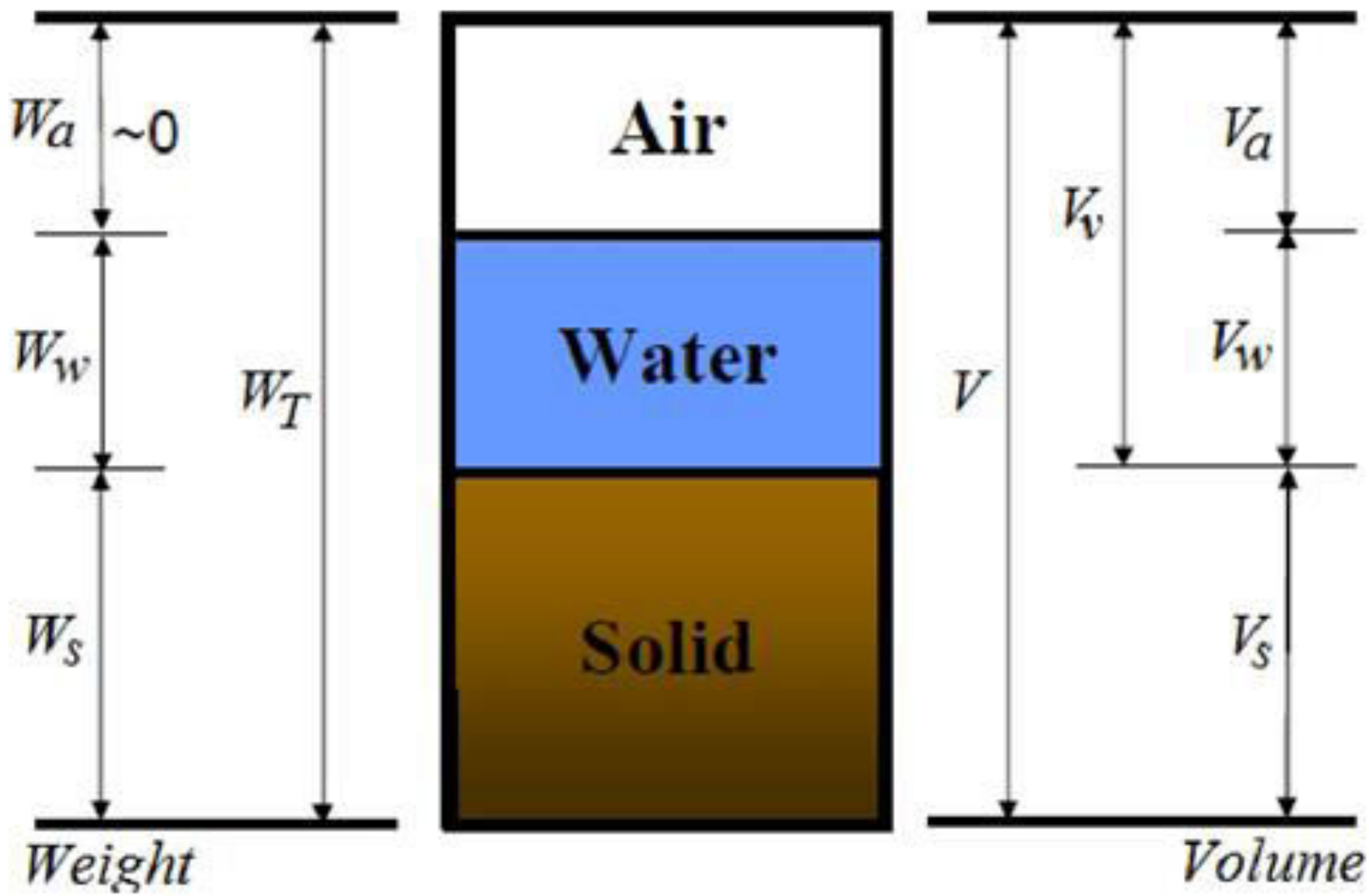
Thus,

several relational volumetric quantities

may be defined.

The following are the **basic volume relations:**

Weight-Volume Relationships



W_a
~0
W_w
W_T
W_s
Weight
Air
Water
Solid
V
V_v
V_a
V_w
V_s
Volume

W_s = weight of soil solid
W_w = weight of water
W_a = weight of air ≈ 0

V_s = volume of soil solid
V_v = volume of voids
V_w = volume of water
V_a = volume of air
V = total volume

$$W = W_s + W_w$$

$$V = V_s + V_v = V_s + V_w + V_a$$

Weight-Volume Relationships

- There are three volumetric ratios that are very useful in geotechnical engineering and these can be determined directly from the phase diagram:

1. ***Void ratio*** (e), defined as the ratio of the volume of voids to the volume of solids:

$$e = \frac{V_v}{V_s}$$

2. ***Porosity*** (n), defined as the ratio of the volume of voids to the total volume:

$$n = \frac{V_v}{V_T}$$

3. ***Degree of saturation*** (S), defined as the ratio of the volume of water to the volume of voids:

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

- (S) is always expressed as *a percentage*. *When S = 0%, the soil is completely dry, and when S = 100%, the soil is fully saturated.*

The relationship between void ratio and porosity can be derived:

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

Void ratio in terms of porosity

Porosity in terms of Void ratio

$$n = \frac{V_v}{V_T} = \frac{V_v}{V_s + V_v} = \frac{\left(\frac{V_v}{V_s}\right)}{1 + \left(\frac{V_v}{V_s}\right)} = \frac{e}{1+e}$$

Weight-Volume Relationships

2. ***Unit weight*** (γ) is the weight of soil per unit volume. There are several commonly used unit weights:

a) ***Total unit weight*** (moisture/wet/bulk unit weight): $\gamma = \frac{W}{V} = \frac{W_s + W_w}{V}$

b) **Dry unit weight** (when saturation S = 0): $\gamma_d = \frac{W_s}{V}$

c) ***Saturated unit weight*** (when saturation S = 100%): $\gamma_{sat} = \frac{W_{sat}}{V}$

d) **Unit weight of soil solids:** $\gamma_s = \frac{W_s}{V_s}$

The moisture unit weight and the dry unit weight ca also be expressed as:

$$\gamma = \frac{W}{V} = \frac{W_s + W_w}{V} = \frac{W_s\left[1 + \left(\frac{W_w}{W_s}\right)\right]}{V} = \frac{W_s(1 + w)}{V} \quad \text{and} \quad \gamma_d = \frac{\gamma}{1 + w}$$

Note: Multiply and Divde by Ws

- Another very useful concept in geotechnical engineering is the ***density*** (equivalent to unit weight) which is expressed as mass per unit volume. There are several commonly used densities:

a) ***Total density*** : $\rho = \dfrac{M}{V}$

b) **Dry *density*** (when saturation S = 0): $\rho_d = \dfrac{M_s}{V}$

c) ***Saturated density*** (when saturation S = 100%): $\rho_{sat} = \dfrac{M_{Sat}}{V}$

d) ***Density of solids***: $\rho_s = \dfrac{M_s}{V_s}$

- The unit weight can be obtained from densities as:

$$\gamma = \rho g$$

$$\gamma_d = \rho_d g$$

M = total mass of the soil sample (kg)

W_s = mass of soil solids in the sample (kg)

V = total volume of the soil (m^3)

Specific Gravity

- The ***Specific Gravity*** (G) which is defined as the ratio of unit weight (or density) of a given material to the unit weight (or density) of water.

a) The specific gravity of a mass of soil (including air, water and solids) is termed as ***bulk specific gravity*** (G_m). It is expressed as:

$$G_m = \frac{\gamma}{\gamma_w} = \frac{\rho g}{\rho_w g} = \frac{\rho}{\rho_w}$$

b) The ***specific gravity of solids*** (G_s), excluding air and water, is expressed by:

$$G_s = \frac{\gamma_s}{\gamma_w} = \frac{\rho_s g}{\rho_w g} = \frac{\rho_s}{\rho_w}$$

- Note that:

$$\gamma_w = \rho_w g = \left(1000\frac{\text{kg}}{\text{m}^3}\right)\left(9.81\frac{\text{m}}{\text{s}^2}\right) = 9810\frac{\text{kg.m}}{\text{m}^3.\text{s}^2} = 9810\frac{\text{N}}{\text{m}^3} = 9.81\frac{\text{kN}}{\text{m}^3}$$

Specific Gravity

- We can use G_s to calculate the density or unit weight of the solid particles:

$$\rho_s = G_s \rho_w \qquad \text{and} \qquad \gamma_s = G_s \gamma_w$$

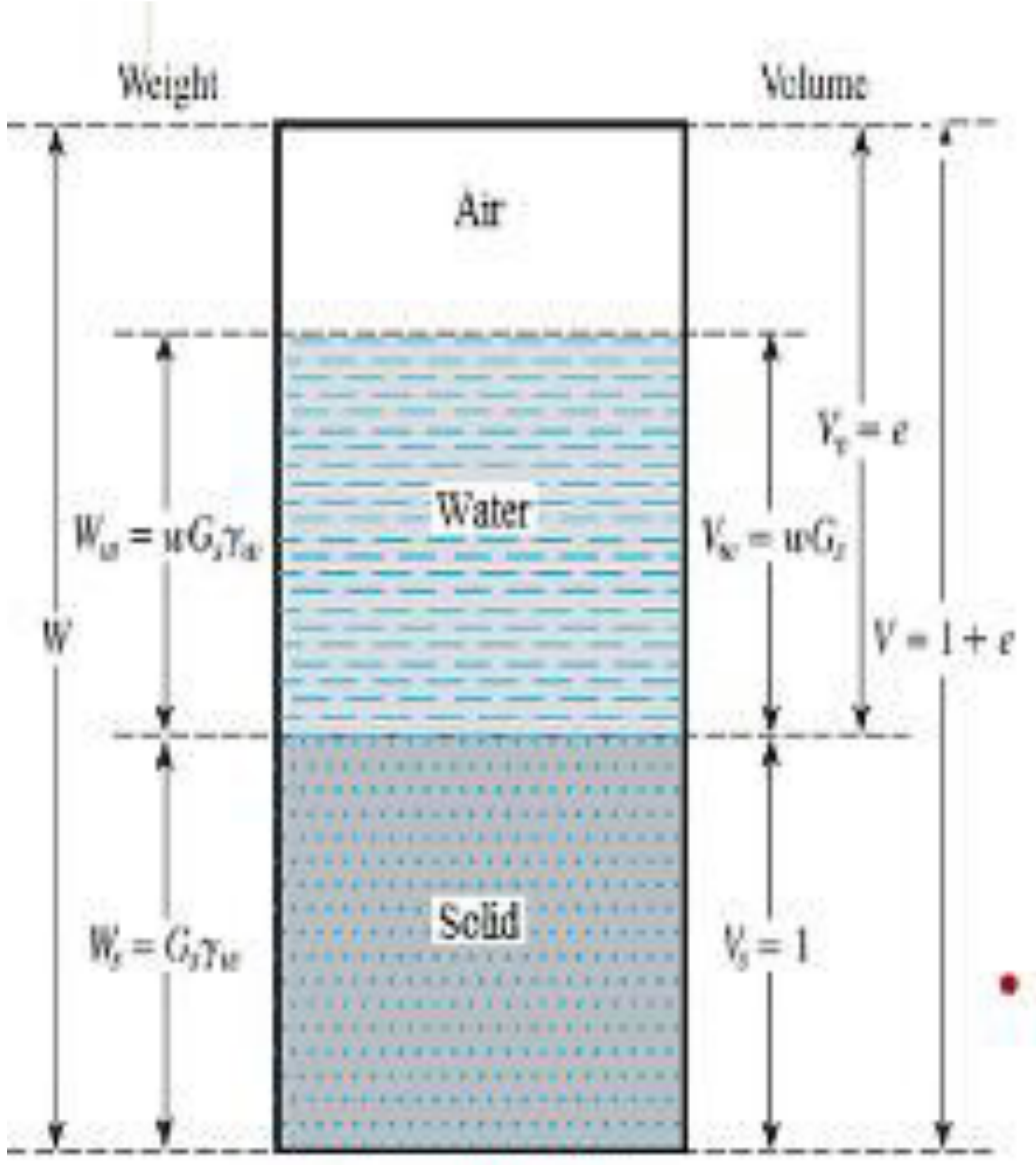
- and hence the volume of the solid particles if the mass or weight is known:

$$V_s = \frac{W_s}{\gamma_s} = \frac{M_s}{\rho_s} = \frac{W_s}{G_s \gamma_w} = \frac{M_s}{G_s \rho_w}$$

- Expected values for G_s:

Type of Soil	G_s
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 among γ, *e*, *w* and G_s



Relationships among γ, e, w and G_s

$$\gamma_d = \frac{W_s}{V} = \frac{G_s \gamma_w}{1+e} \qquad e = \frac{G_s \gamma_w}{\gamma_d} - 1$$

$$V_w = \frac{W_w}{\gamma_w} = \frac{w G_s \gamma_w}{\gamma_w} = w G_s$$

$$S = \frac{V_w}{V_v} = \frac{w G_s}{e} \Rightarrow Se = w G_s$$

When the soil is saturated (S = 100%): S=1

$$\gamma_{sat} = \frac{(G_s + e)\gamma_w}{1+e} \qquad e = w G_s$$

. **Air content (a_c)** is the ratio of the volume of air (V_a) to the volume of voids.

$$a_c = \frac{V_a}{V_v}$$

. **Percentage air voids (n_a)** is the ratio of the volume of air to the total volume.

$$n_a = \frac{V_a}{V} x 100 = n \, x \, a_c$$

Density Index

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

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

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 $e_{\min}$)

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

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

Relative density (%)	Classification
< 15	Very loose
15-35	Loose
35-65	Medium
65-85	Dense
> 85	Very dense

Memorize relationships

$$Se = wG_s$$

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

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

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

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

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

Example 1: A soil has void ratio = 0.72, moisture content = 12% and G_s = 2.72. Determine its

(a) Dry unit weight
(b) Moist unit weight, and the
(c) Amount of water to be added per m^3 to make it saturated.

Use $\gamma_w = 9.81\, kN/m^3$

(a) $\gamma_d = \dfrac{G_s.\gamma_w}{1+e} = \dfrac{2.72 \, x \, 9.81}{1+0.72}$ = 15.51 kN/m^3

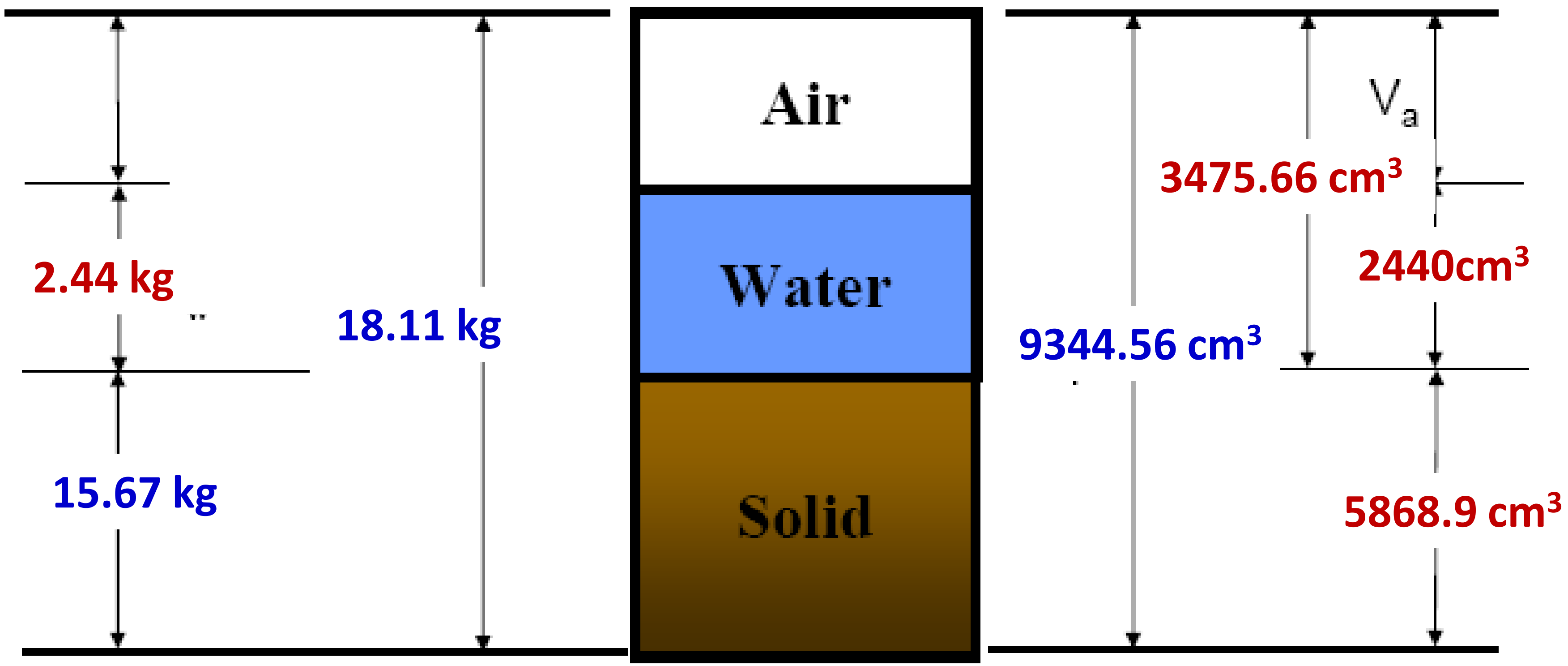
(b) $\gamma = \gamma_d(1+w)$

$= \dfrac{1+0.12}{} \, x$ 15.51 = 17.38 kN/m^3

Amount of water to be added per cubic meter of soil = Dry Unit weight X water content = 15.51 X 0.12 = 1.861 kN

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.



$W = M – Md / Md$

$V = 9344.56$ cc

$M = 18.11$ kg = 18110g

$Md = 15.67$ kg = 15670g

Therefore , $w = 15.57$ %

Moist density -= M / V

Dry Density = Md / V

Dry density = $\rho d = G\rho w /1 +e$

Therefore, $e =\{ G\rho w/ \rho d\} – 1$

We Know, $Se =wG$

Therefore,

$S = wG/e$

Example 1

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

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 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.

<u>Solution</u>:

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

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

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

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

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

SOLVED PROBLEMS

A cylindrical specimen of moist clay has a diameter of 38 mm, height of 76 mm and mass of 174.2grams. After drying in the oven at 105°C for about 24 hours, the mass is reduced to 148.4 grams. Find the dry density, bulk density and water content of the clay. Assuming the specific gravity of the soil grains as 2.71; find the void ratio and the degree of saturation.

Solution:

Volume of the specimen = $V = \pi\,(1.9)^2(7.6) = 86.2\ \text{cm}^3$

Wet Soil Mass = $M_b = 174.2$ g

Dry Soil Mass = $M_s = 148.4$ g

Mass of Water in the Soil = M_b - M_s = 174.2-148.4 = 25.8 g

Dry Density = $\rho_d = M_s / V = 148.4/86.2 = 1.722$ g/cm^3

Bulk Density = $\rho_b = M_b / V = 174.2/86.2 = 2.021$ g/cm^3

Water content = w = Mass of Water / Mass of Dry Soil = $M_w/M_s = 25.8 / 148.4 = 0.174 = 17.4\%$

e = Void ratio =
$[G \rho_w / \rho_d] - 1 = [2.71 X 1 /1.722] =$ **0.574**

$Se = wG; \therefore$

***S*= Degree of saturation** =
wG/e = [0.174X 2.71] / 0.574 = **0.821 or 82.1%**

Field density testing on a soil sample has shown bulk density of a compacted road base to be 2.06 t/m^3 with 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:

Given Specific gravity of soil sample = G = 2.69

Water content = w = 11.6% or = 0.116

Bulk density of road base = ρ_b = 2.06 t /m^3 = **2.06 g / cm^3**

P_d = Dry density of soil = ρ_b / [1+ w] = 2.06 / 1+ 0.116 = **1.846** g / cm^3

e = Void ratio = [G ρ_w / ρ_d] – 1 = [2.69X 1 /1.846] -1 = **0.457**

S = Degree of saturation =

wG /e = [0.116 X 2.69] / 0.457 = **0.6651 = 66.51%**

A soil sample assumed to consist of spherical grains all of same diameter will have max void ratio, when the grains are arranged in a cubical array. Find the void ratio, dry unit weight. Take unit weight of grains 20kN/m^2

Solution:

Let the spherical particle diameter = d

Let the spherical particle diameter = d

No. of spherical grains in the cubical array

$$= \frac{1}{d} \times \frac{1}{d} \times \frac{1}{d} = \frac{1}{d^3}$$

Volume of each spherical grains $= \frac{\pi d^3}{6}$

Volume of a soil solid = Vs $= \frac{1}{d^3} \times \frac{\pi d^3}{6} = \frac{\pi}{6}$

Total volume of cube = $V = 1 \times 1 \times 1 = 1 m^3$

Volume of void = $V_v = V - V_s = 1 - \frac{\pi}{6} = \frac{6 - \pi}{6}$

$$e = \frac{V_v}{V_s} = \frac{6 - \pi}{6} \Big/ \pi/6 = \frac{6 - \pi}{\pi} = 0.91$$

Dry unit weight = $\gamma_d = \frac{W_s}{V} = \frac{V_s \gamma_s}{V}$

Note: $\frac{W_s}{V_s} = \gamma_s$ **OR**

$$W_s = V_s \gamma_s$$

$$\gamma_d = \frac{\pi/6 \times 20}{\mathbf{1}} = 10.47 \text{ kN/ } m^3$$

A soil sample of 5 kg with a natural water content of 3%. How much water to be added to rise the water content to 12%.

Solution:

Given the weight of wet soil mass = M_b = 5 kg

Let the mass of the dry soil = M_s = x kg

Water content = 3% = 0.03 = $[M_b – M_s] /M_s$ = [5-x] / x = [5/x] – 1

[5/x] – 1 = 0.03; 5 /x = 0.03+1 = 1.03

5 = 1.03x; x = 5 /1.03 = 4.854 kg = mass of the dry soil

At w = 12%, M_w = Mass of water = (4.854)(0.12) = 0.582 kg

At w = 3%, M_w = Mass of water = (4.854)(0.03) = 0.146 kg

∴ Amount of water to be added to rise the water content as 12% from 3% = 0.582 - 0.146 kg = 0.436 kg = 436g =436 ml

Example 2: The dry density of a sand with porosity of 0.387 is 1600 kg/m^3. Find the void ratio of the soil and the specific gravity of the soil solids. [Take $\gamma_w = 1000\ kg/m^3$]

n = 0.387

γ_d = 1600 kg/m^3

Solution:

(a) e = $\dfrac{n}{1-n} = \dfrac{0.387}{1-0.387}$ = 0.631

(b) $\gamma_d = \dfrac{G_s.\gamma_w}{1+e}$

$\therefore$ G$_s$ = $\dfrac{(1+e)}{\gamma_w}.\gamma_d = \dfrac{1+0.631}{1000} x\, 1600 = 2.61$

Soil Embankment - Borrow pit type probems

Let volume of Earth fill in the Embankment $= V_1$

Void Ratio of Earth fill in the Embankment $= e_1$

Volume of borrow soil $= V_2$

Void Ratio of borrow soil $= e_2$

We know $e = \frac{V_v}{V_s}$

Add 1 on both side

$$e+1 = \frac{V_v}{V_s} + 1 = \frac{V_v + V_s}{V_s} = \frac{V}{V_s}$$

For soil in the Embankment

$$e_1 + 1 = \frac{V_1}{V_s}$$

$$V_s = \frac{V_1}{e_1 + 1} \quad \text{—— (1)}$$

Mly For soil in the borrow pit

$$V_s = \frac{V_2}{e_2 + 1} \quad \text{—— (2)}$$

V_s is same in both the cases.

$\therefore$ Eqn (1) = Eqn (2)

$$\frac{V_1}{e_1 + 1} = \frac{V_2}{e_2 + 1} \quad \text{(or)} \quad V_1 = \frac{V_2}{e_2 + 1} \cdot (e_1 + 1)$$

$$V_1 = \left(\frac{1 + e_1}{1 + e_2}\right) V_2$$

$$V_2 = \left(\frac{1 + e_2}{1 + e_1}\right) V_1$$

In a Bored Soil – $w = 10\%$. $\rho_b = 1.80\ Mg/m^3$

Embankment – $w = 18\%$ – $\rho_d = 1.85\ Mg/m^3$

Determine how may m^3 of bored soil require to construct an embankment of $1\ m^3$ and Additional water to be added.

Soln

$$(\gamma_d)_b = \frac{\gamma_b}{1+w}$$

$1\ Mg/m^3 = 1 \times 9.81\ kN/m^3$

$1.80 = 17.658\ kN/m^3$

$$= \frac{17.658}{1+0.10} = 16.05\ kN/m^3$$

$(\gamma_d)_e =$ given $1.85\ Mg/m^3 = 1.85 \times 9.81$

$= 18.149\ kN/m^3$

$$\gamma_d = \frac{W_s}{V}$$

$$\gamma_d V = W_s$$

$$\gamma_{d1} V_1 = W_s$$

$$\gamma_{d2} V_2 = W_s$$

$$\gamma_{d1} V_1 = \gamma_{d2} V_2 \Rightarrow \frac{\gamma_{d1}}{\gamma_{d2}} = \frac{V_2}{V_1}$$

$$V_1 = \frac{\gamma_{d2}}{\gamma_{d1}} \times V_2 = \frac{18.149}{16.05} \times 1 = \underline{1.131 m^3}$$

How many cubic meters of soil is to be excavated from a borrow pit in which the void ratio is 0.95 towards to construct a earth fill of 1000 m^3 with void ratio 0.7

$$V_2 = \left(\frac{1+e_2}{1+e_1}\right) V_1 = 1147 m^3$$

In an Earthen Embankment under construction the bulk unit weight is 16.5 kN/m³ at water content of 11%. If the water content is to be raised to 15%, compute the Amount of additional water to be added per m³ of soil. Assume No change in the void ratio

$$\gamma_d = \frac{\gamma_b}{1+w} = \frac{16.55}{1+0.11} = 14.86 \text{ kN/m}^3$$

$$\gamma_d = \frac{W_s}{V} \quad , \quad \text{Take } V = 1. \qquad W_s = \underline{14.86 \text{ kN}}$$

@ 11% Water Content,

Amount of water available: $W_w = w \times W_s$

$= 0.11 \times 14.86 = 1.635$ kN

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

@15% water content $W_w = w \times W_s = 0.15 \times 14.8$
$= 2.239$ kN

Additional water required $= 2.239 - 1.635 = \underline{0.604}$ kN

$\left(\frac{W_w}{V_w} = \gamma_w\right)$ $\therefore$ $V_w = \frac{W_w}{\gamma_w} = \frac{0.604}{9.81} =$

$= 0.06157$ m3 $= \underline{61.57}$ l

Soil from a borrow pit has a bulk unit wt of 18.44 kN/m^3 and water content of 5%. Calculate the amount of water to be added to 1 m^3 of soil to raise the water content to 15%. Assume the void ratio remain constant. What will be then the degree of saturation.

Assume $G = 2.7$

$$\gamma_d = \frac{\gamma_b}{1+w} = \frac{18.44}{1+0.05} = 17.56 \text{ kN/m}^3$$

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

$$\gamma_d = \frac{W_s}{V}$$

For one m^3 soil $V = 1$

$\therefore \gamma_d = W_s = \underline{17.56 \text{ kN}}$

$W_w = w \times W_s = 0.05 \times 17.56 = 0.878\,kN$

$V_w = \frac{W_w}{\gamma_w} = \frac{0.878}{9.81} = 0.0895\,m^3$
$= 89.5\,lit$

II case $w = 15\%$

$\therefore W_w = w \times W_s = 0.15 \times 17.56 = 2.634\,kN$

$V_w = \frac{W_w}{\gamma_w} = \frac{2.634}{9.81} = 0.2685\,m^3$
$= 268.5\,lit$

Addtional Amount of water to be aded

$= 268.5 - 89.5 = 179\,lit$

200,000m^3 of Soil excavated from a borrow pit is being used to build an embankment. The void ratio of the soil at the borrow pit is 1.14 and if the porosity of the compacted soil in the embankment is 40%

then how many cubic meters of embankment can be built using above soil?

Given Data:

Volume of borrow soil = V1 = 200000m^3

Volume of embankment can be constructed = V2 =?

Void ratio of soil in the borrow pit = e1 = 1.14

Void ratio required in the embankment construction = e2 =?

Given the porosity of soil required in the embankment = 40% = 0.4

We know e = n / [1-n]; e2= n2 / [1 –n2] = 0.4 /[1-0.4] = 0.667

We know V2 / V1 = [1+e2] / [1+e1] or V2 = {V1 x [1+ e2]} / [1+e1]

V2 = 200000 x [1+0.667] / [1+1.14] =

V2 = Volume of embankment can be constructed = 155794.4 m^3

Soil Classification

Classification of soil is the

separation of soil into

classes or groups

of each

having

similar characteristics and

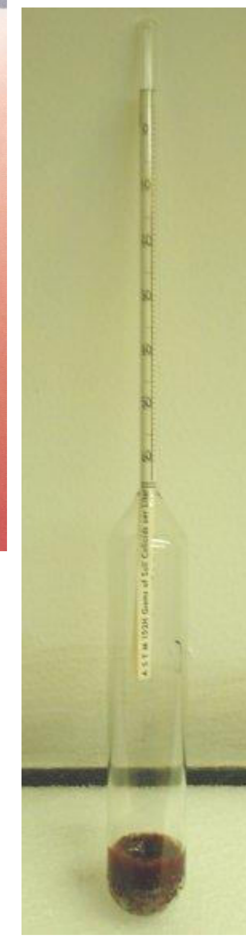
similar behaviour.

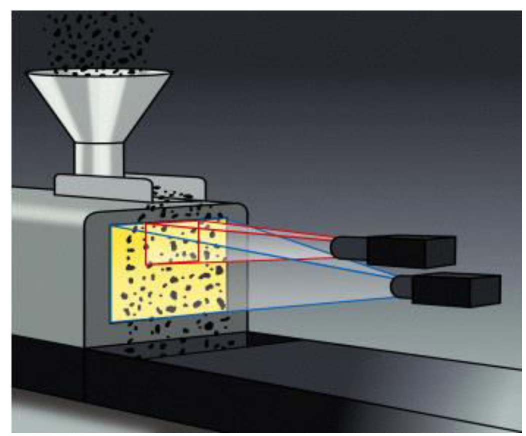
IS CLASSIFICATION OF SOIL

COBBLES	BOULDERS	GRAVEL	SAND	SILT	CLAY
>300	300-80	80-4.75	4.75-0.075	0.075-0.002	<0.002

Methods to Determine Particle Size Distribution

- *Sieving* methods – soil particles ≥ 0.05 mm (sand fraction) we use *Sieving* methods.
- *Sedimentation* methods
  - Pipette
  - Hydrometer
  - X-ray attenuation
- *Particle counting* methods
  - Light, SEM Microscopy
  - Coulter method
- *Laser/Light diffraction* methods





Particle Size Distribution

- **Wet sieving** is carried out for separating fine grains from coarse grains by washing the soil specimen on a 75 micron sieve mesh.

Dry sieve analysis

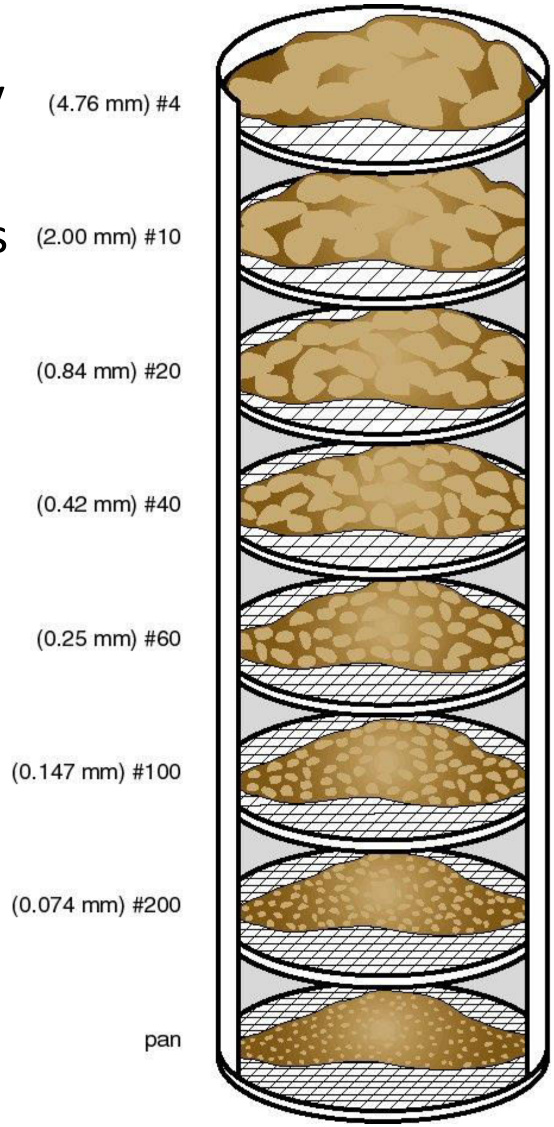
- It is carried out on particles coarser than 75 micron.
- Samples (with fines removed) are dried and shaken through a set of sieves of descending size.
- The weight retained in each sieve is measured.
- The cumulative percentage quantities finer than the sieve sizes (passing each given sieve size) are then determined.
- The resulting data is presented as a distribution curve with
- **grain size** along x-axis (log scale) and
- **percentage passing** along y-axis (arithmetic scale).

Sieving Methods

For particles ≥ 0.05 mm (sand fraction) we apply SIEVING methods.

Results are expressed as particle diameters

Note - particles are rarely spherical, hence these diameters should be regarded as effective diameters based on sieve opening size.



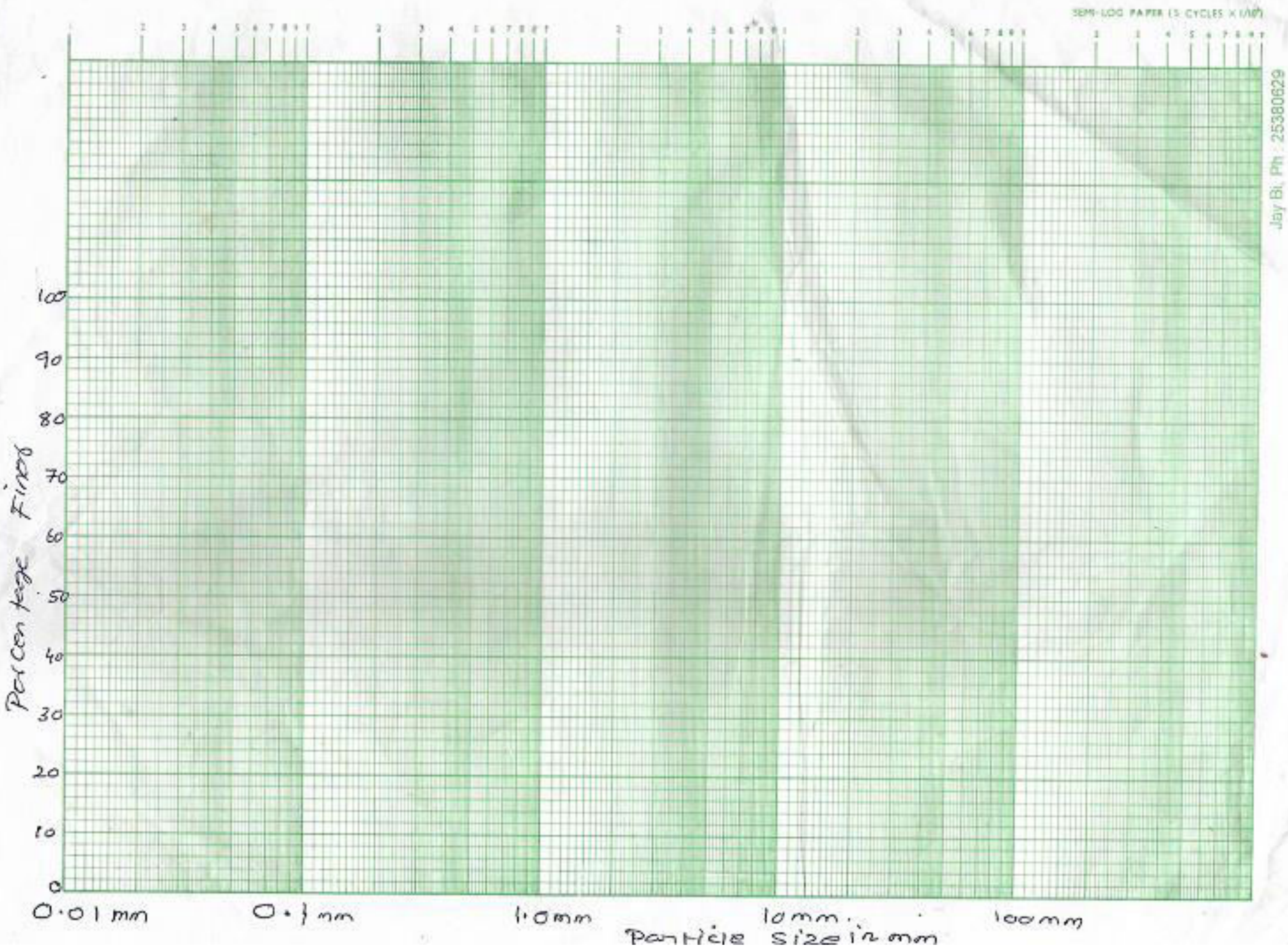
Sieve Shaker

Weight of soil taken for Dry sieve analysis was 500g. The weight retained on different sieve are as follows. Classify the soil.

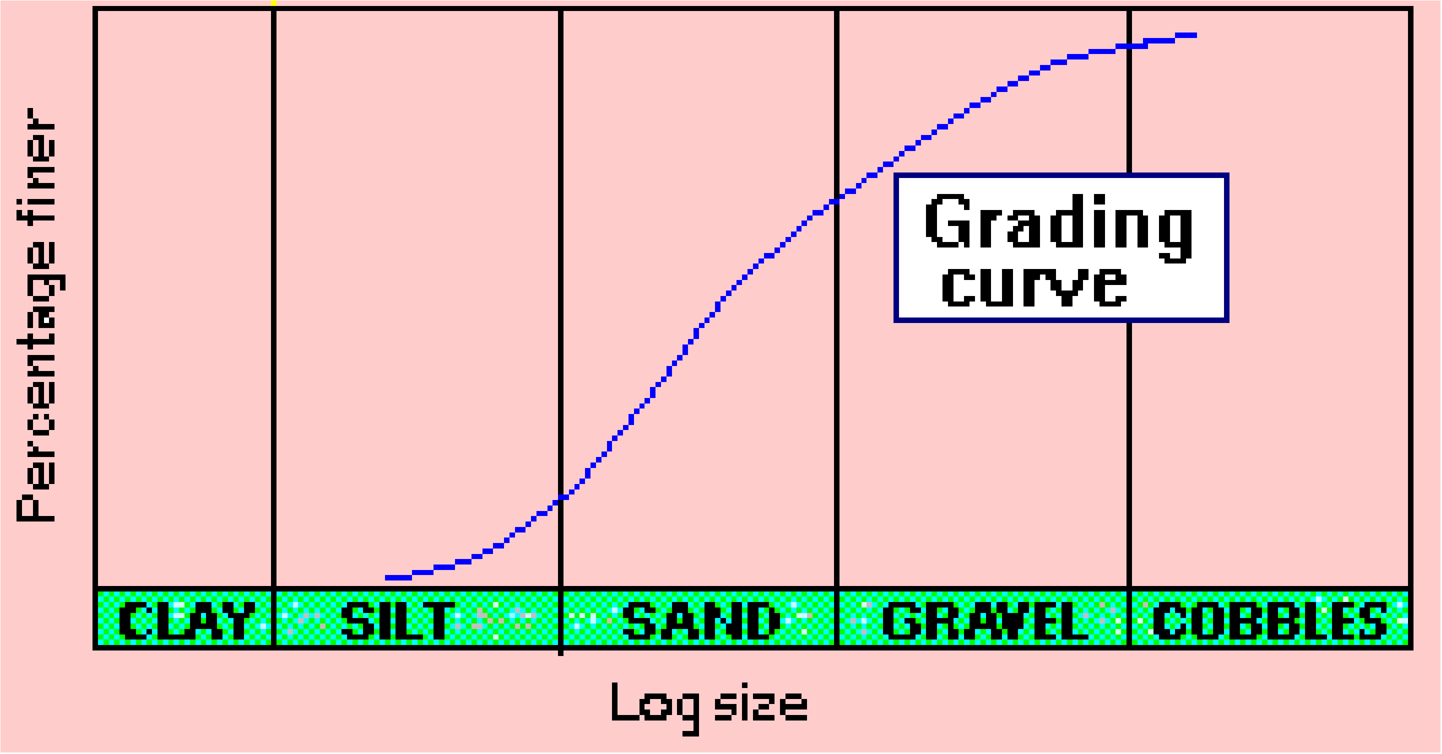
Sieve No IS Designation	Size of particle mm	Weight of soil retained g	% Retained	Cumulative % Retained g	% Finer
4.75 mm	4.75				
2 mm	2				
1 mm	1				
600 micron	0. 600				
425 micron	0.425				
212 micron	0.212				
150 micron	0.150				
75 micron	0.075				

- 5kN of soil sample was taken for sieve analysis. The weight of soil retained on each sieve is as follows. Classify the soil.

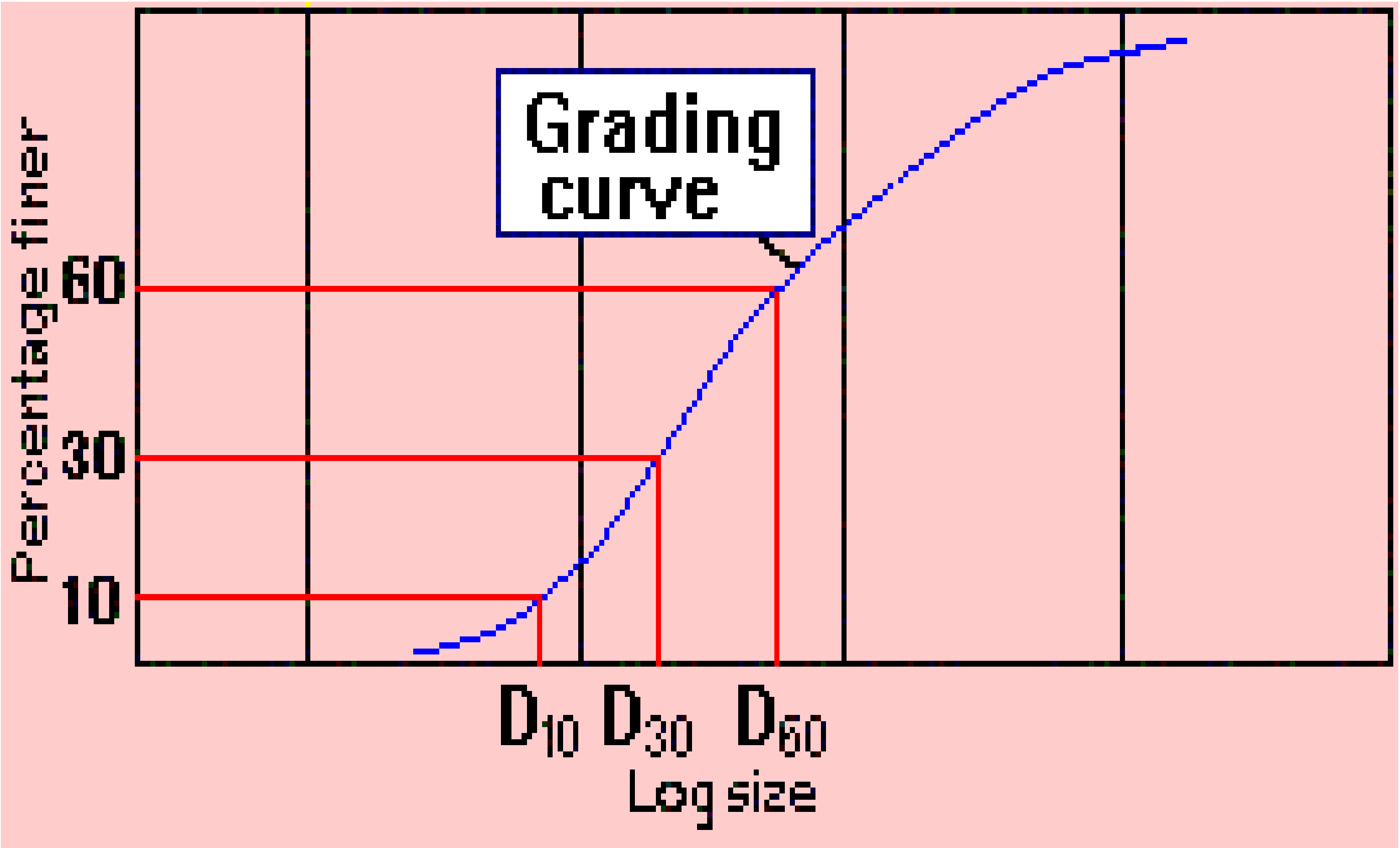
SIEVE SIZE IN MM	4.75	2.36	1.0	425 µ	150 µ	75 µ
WEIGHT RETAINED IN N	12	18	950	933	1338	895


SEMI-LOG PAPER (5 CYCLES X 1/10)
Jay Bi. Ph : 25380629
100
90
80
70
60
50
40
30
20
10
0
Percentage Fines
0·01mm
0·1mm
1·0mm
10mm.
100mm
Particle size in mm

Grain-Size Distribution Curve



Grading Characteristics



The grading characteristics are then determined as follows:

1. Effective size = D_{10}

2. Uniformity coefficient

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

3. Curvature coefficient,

,

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

Both C_u and C_c will be 1 for a single-sized soil.

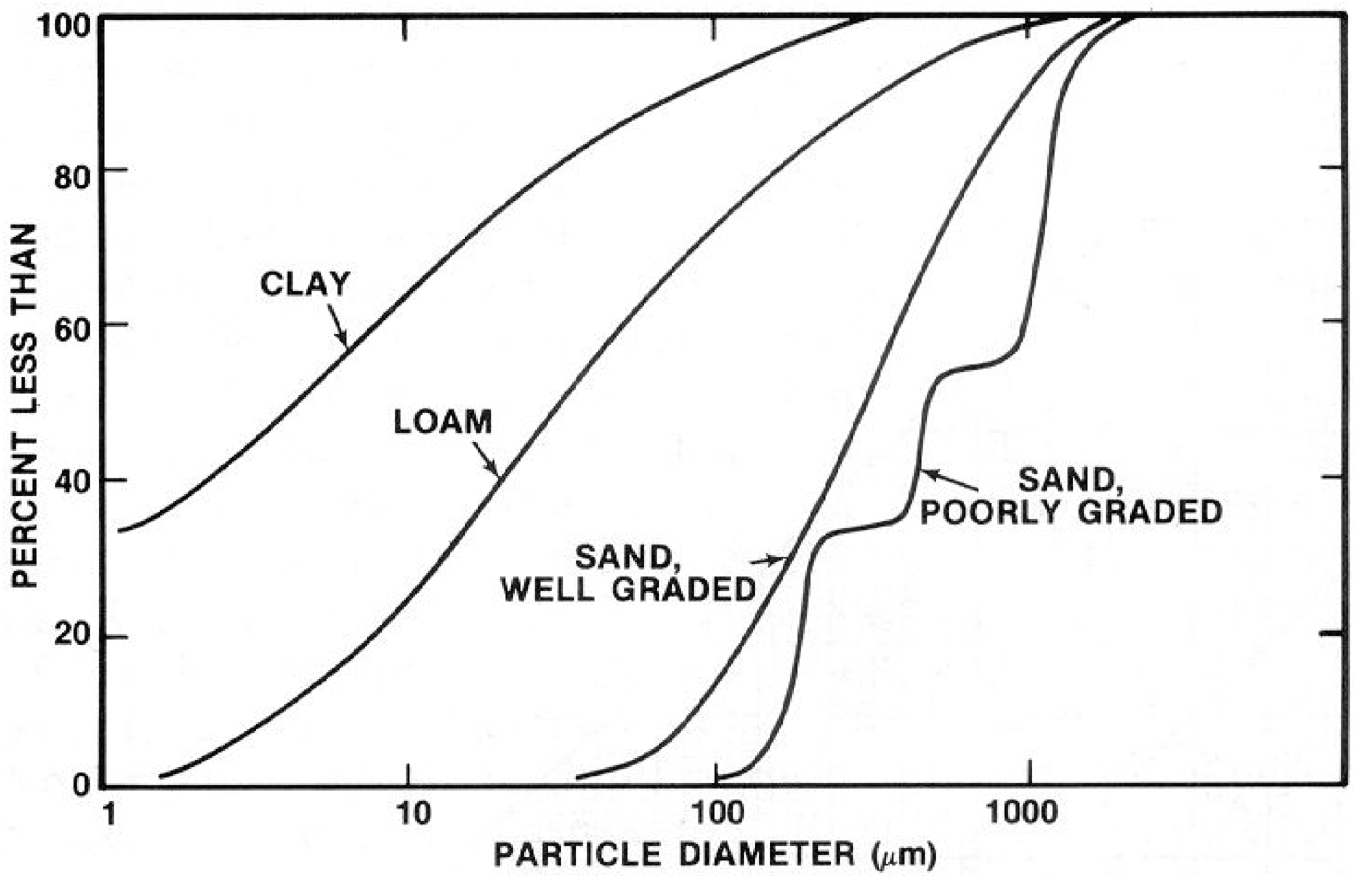
$Cu < 2$ are uniform size as Cc 1-3

$Cu < 4$ are well graded sand as Cc 1-3

$Cu < 6$ are well graded gravel as Cc 1-3

Cu and Cc not meet the above requirements are classified as gap graded

Particle-Size Diagram



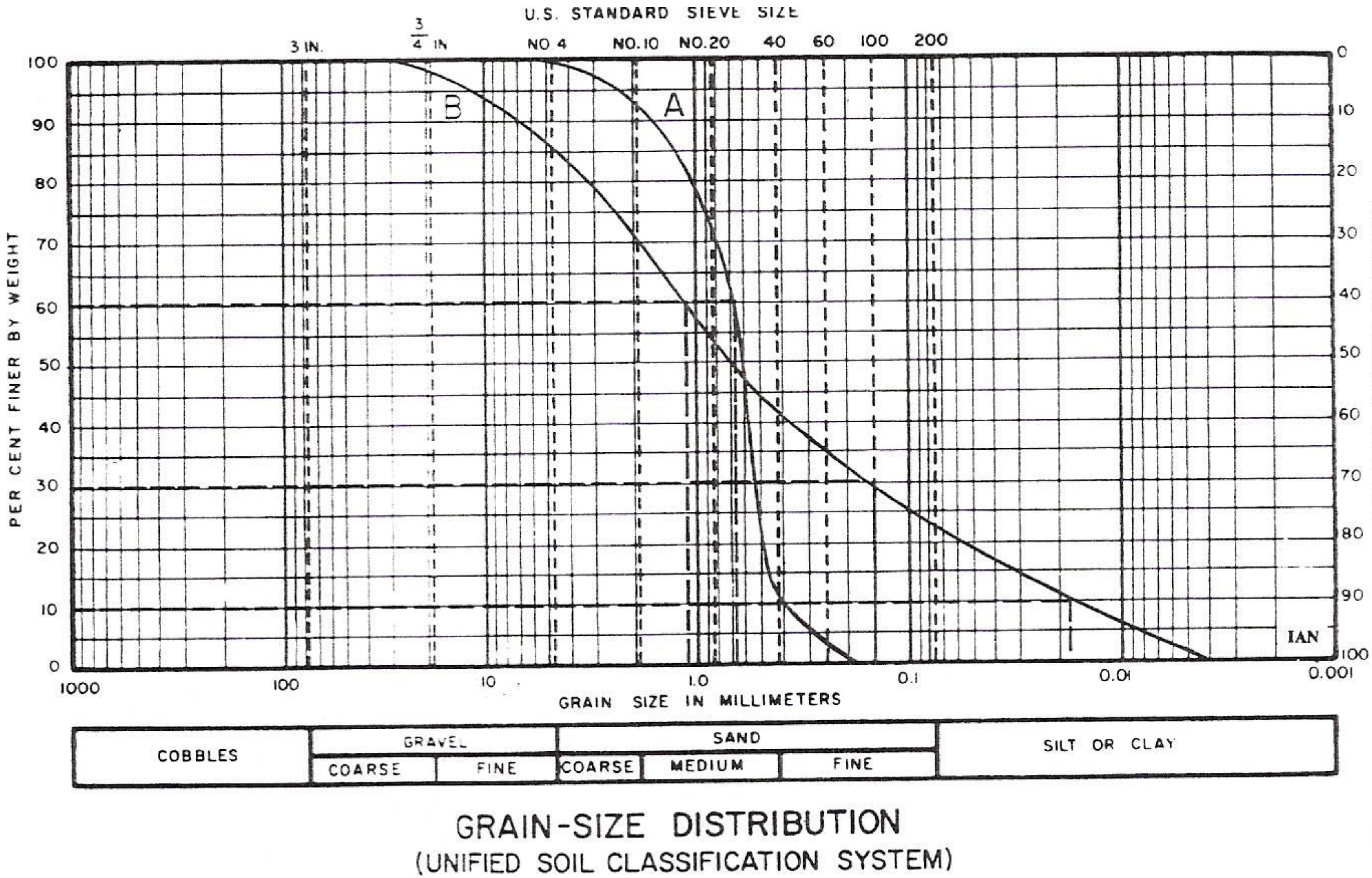
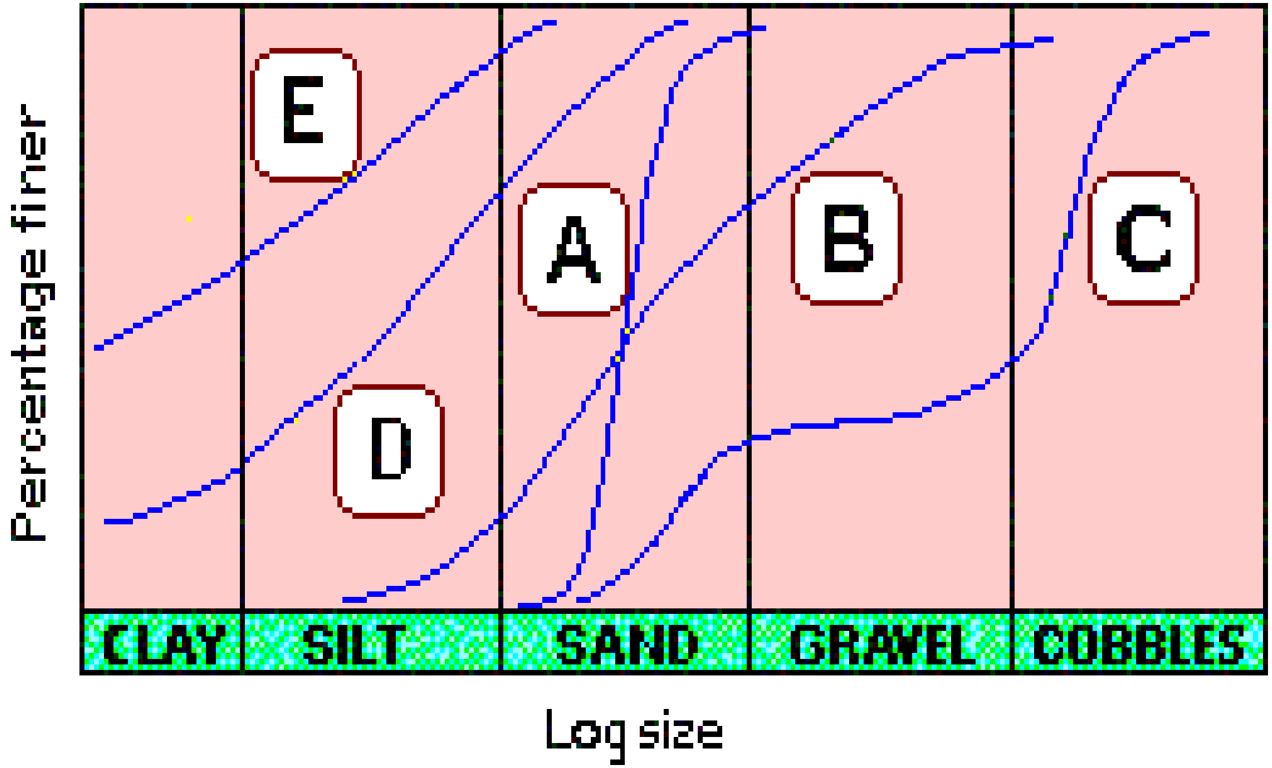


Figure 3-9 Particle-Size Distribution Curves for Two Granular Soil Samples (Worked Example 3-1).



Curve A - a poorly-graded medium SAND

Curve B - a well-graded GRAVEL-SAND (i.e. having equal amounts of gravel and sand)

Curve C - a gap-graded COBBLES-SAND

Curve D - a sandy SILT

Curve E - a silty CLAY (i.e. having little amount of sand)

Consistency

Consistency is a term used to describe the

degree of firmness of fine-grained soils (silt and clay).

The consistency of fine grained soils is

expressed qualitatively by

such terms as
very soft , soft, stiff, very stiff and hard.

Atterberg Limits:

These are water contents at certain limiting or critical stages in soil behavior. These limits are:

Liquid Limit (LL):

The water content, in percent, at the point of transition from plastic to liquid state

Or

The minimum moisture content at which soil begins to behave as a liquid and begins to flow

Plastic Limit (PL):

The water content, in percent, at the point of transition from semisolid to plastic state.

Or

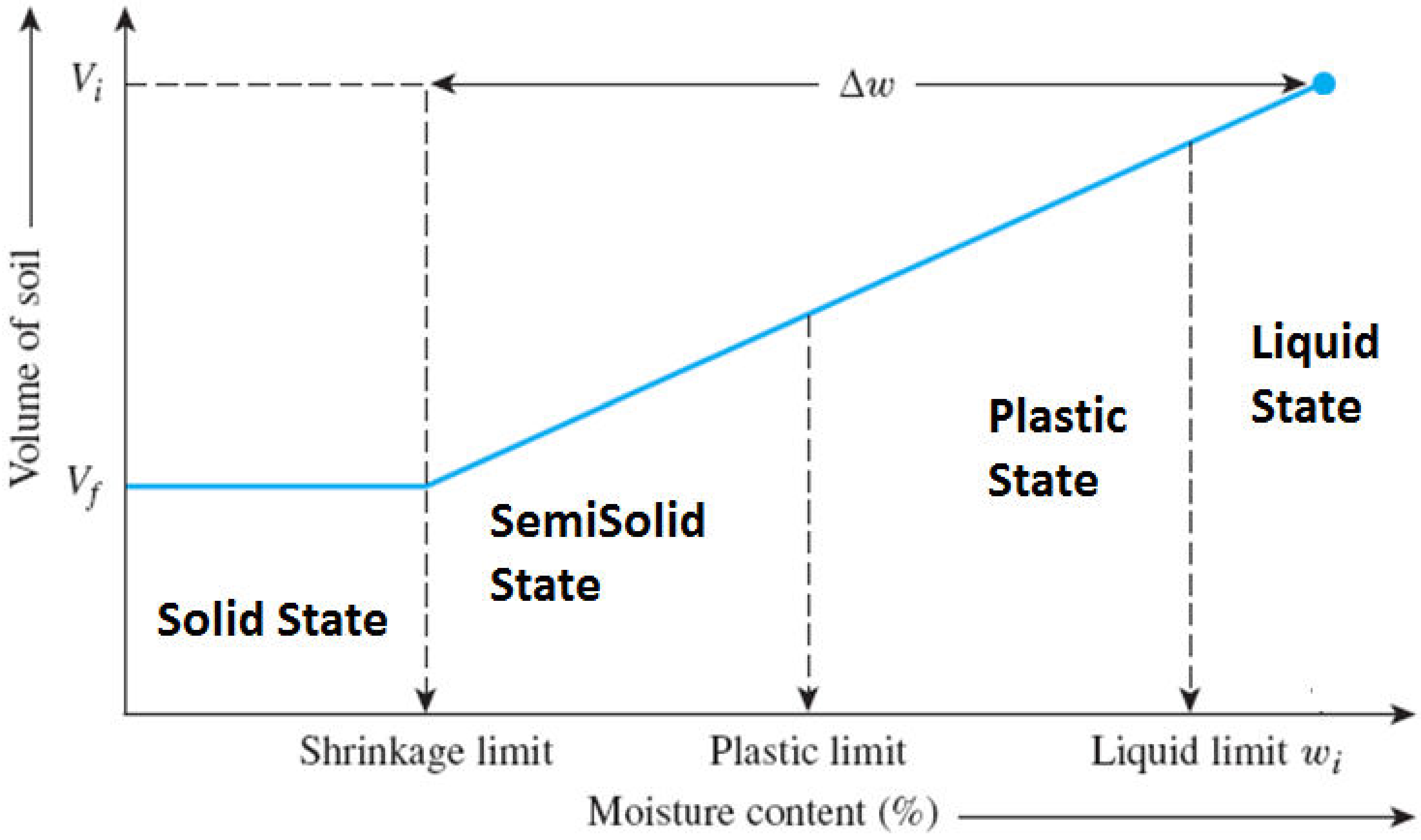
the moisture content at which soil begins to behave as a plastic material

Shrinkage Limit (SL):
The water content, in percent,
at the point of transition
from solid to semisolid state

Or

The moisture content at which
no further volume change occurs
with further reduction in moisture content

STAGES OF CONSISTENCY

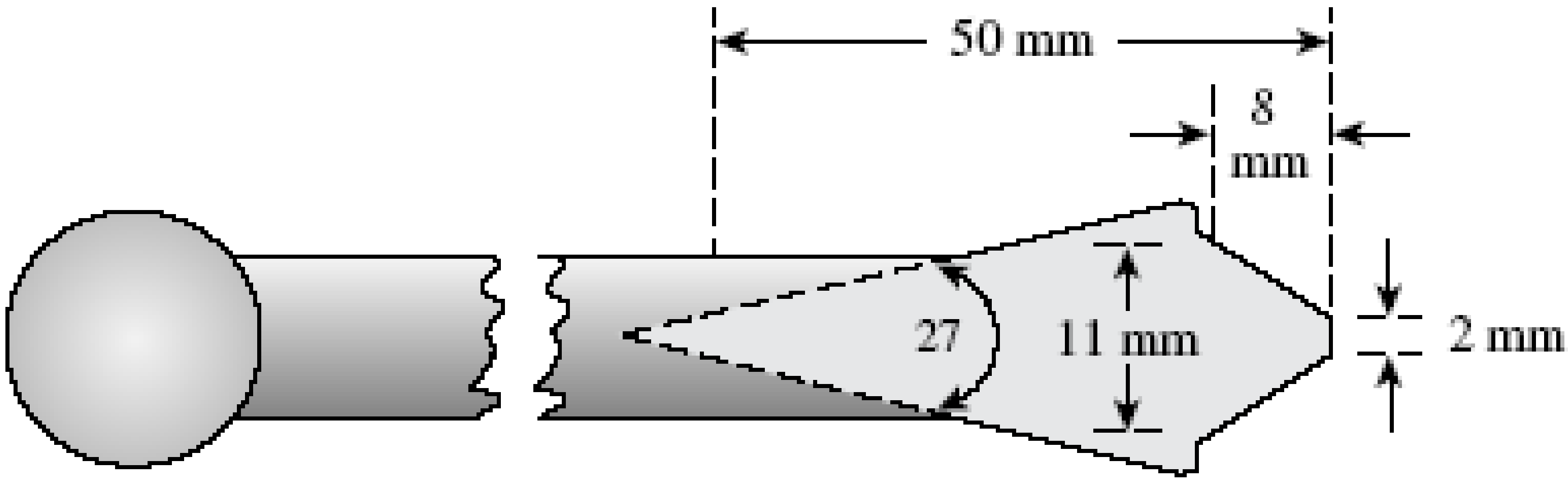


V_i
Δw
Volume of soil
V_f
Liquid State
Plastic State
SemiSolid State
Solid State
Shrinkage limit
Plastic limit
Liquid limit w_i
Moisture content (%)

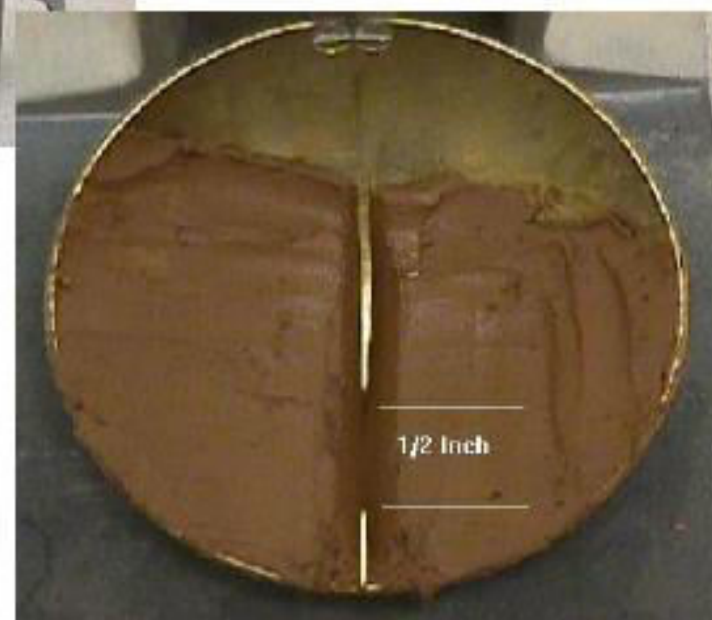
Liquid limit (LL) determination

The water content required
to close a distance of ½ inch
(12.7 mm)
along the bottom of the groove
after 25 blows is
defined as the Liquid Limit.

Casagrande Cup



(b)


1/2 Inch

Liquid Limit Test:

Take about 120g of the given soil sieved through 425 micron sieve and

mix it thoroughly with distilled water to form a uniform paste.

The amount of water to be added shall be such, so as to

require 30 to 35 Blows of the cup

to cause the required closure of the groove.

Place a small amount of soil to the correct depth of the grooving tool, well centred in the cup with respect to the hinge.

Smooth the surface of the soil pat carefully, and

using the grooving tool, cut a clean straight groove that completely separates the soil pat into two parts.

Turn the crank at a rate of about two revolutions per second and

count the blows necessary to close the groove in the soil

for a distance of about 12mm.

Take the water content sample from the closed part of the groove.

Weigh the sample.

Remove the remaining soil from brass cup and
return it to the porcelain dish.

Add a small amount of water to the soil further in the dish and

carefully mix to a consistency to yield
a blow count of between 25 and 30 blows.

Repeat the sequence for two additional tests

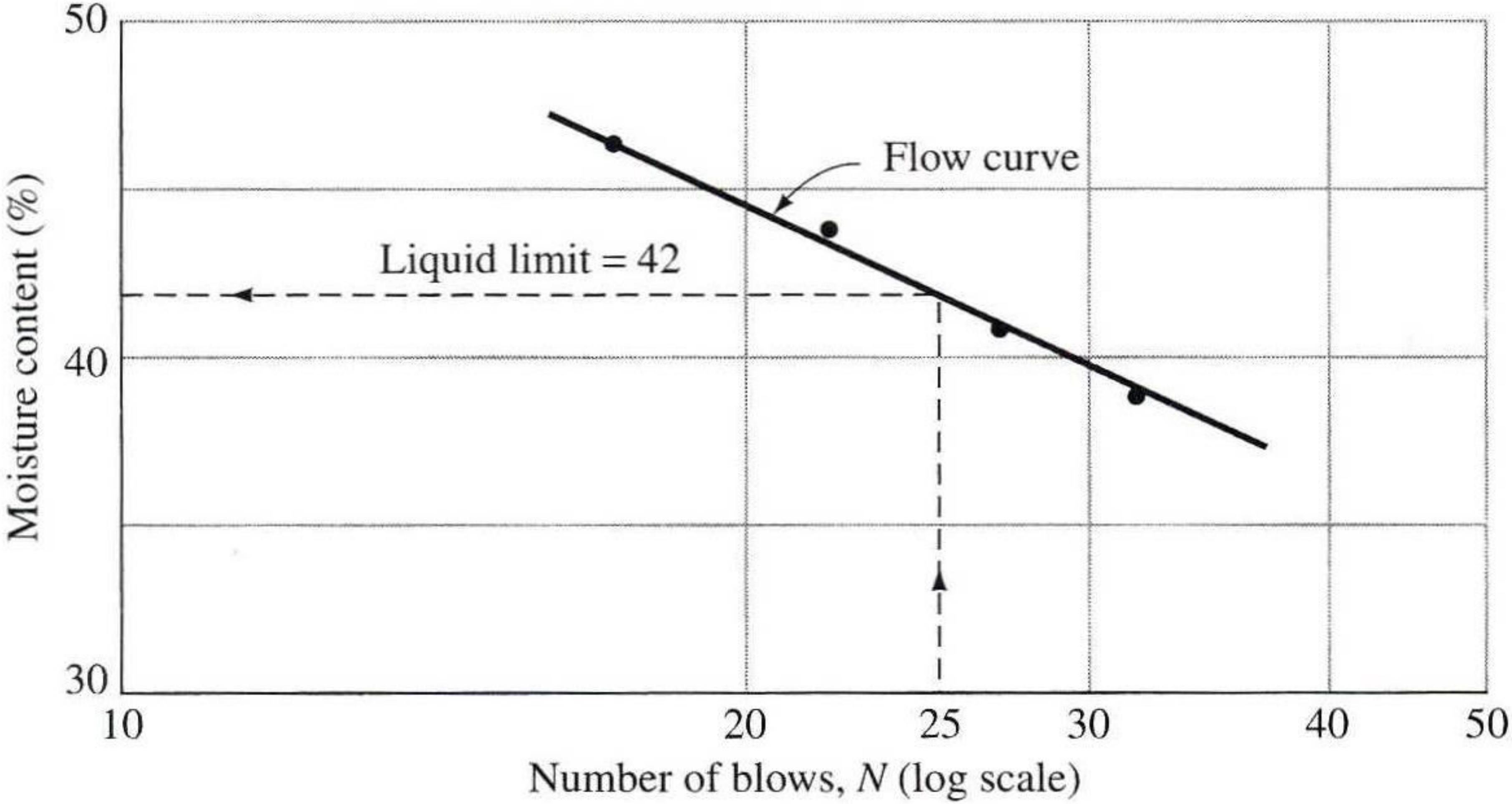
for blow counts of between 20 and 25 and

between 15 and 20, for a total of four test determinations

w % values are plotted against the

logarithm of the number of blows, N.

Liquid Limit - Measurement



Liquid Limit (LL) at N = 25

Plastic Limit - Definition

The moisture content in (%) at which the soil when rolled into threads of 3.2mm (1/8 in) in diameter, will get crumbled or just show cracks

Plastic Limit - Measurement

PL = w% at d 3.2 mm (1/8 in.)

Plastic Limit Test: Procedure

Break about 20g of soil into four peanut-sized samples, using little water.

Roll the peanut of soil on a glass plate until it just crumbles at 3mm
(use a glass or welding rod for comparison if you are unsure of what 3mm is).

Place the crumbled soil in the pre-weighed moisture cup and
put weight of wet crumble samples of different trails.

Repeat this sequence three more times.

place the moisture cup in the oven for 24 hours
Put the weights of dry crumble sample of different trials

PL = Average of w1, w2, w3….

Where w1 plastic limit water content of trail 1 sample =

{[Wet weight – Dry weight] x 100} / Dry weight

Shrinkage Limit (SL) (ASTM D-427, ASTM D-4943)

The shrinkage limit is defined as
the moisture content, in percent, at which
the volume of the soil mass ceases to change.

Mineral	Shrinkage limit
Montmorillonite	8.5–15
Illite	15–17
Kaolinite	25–29

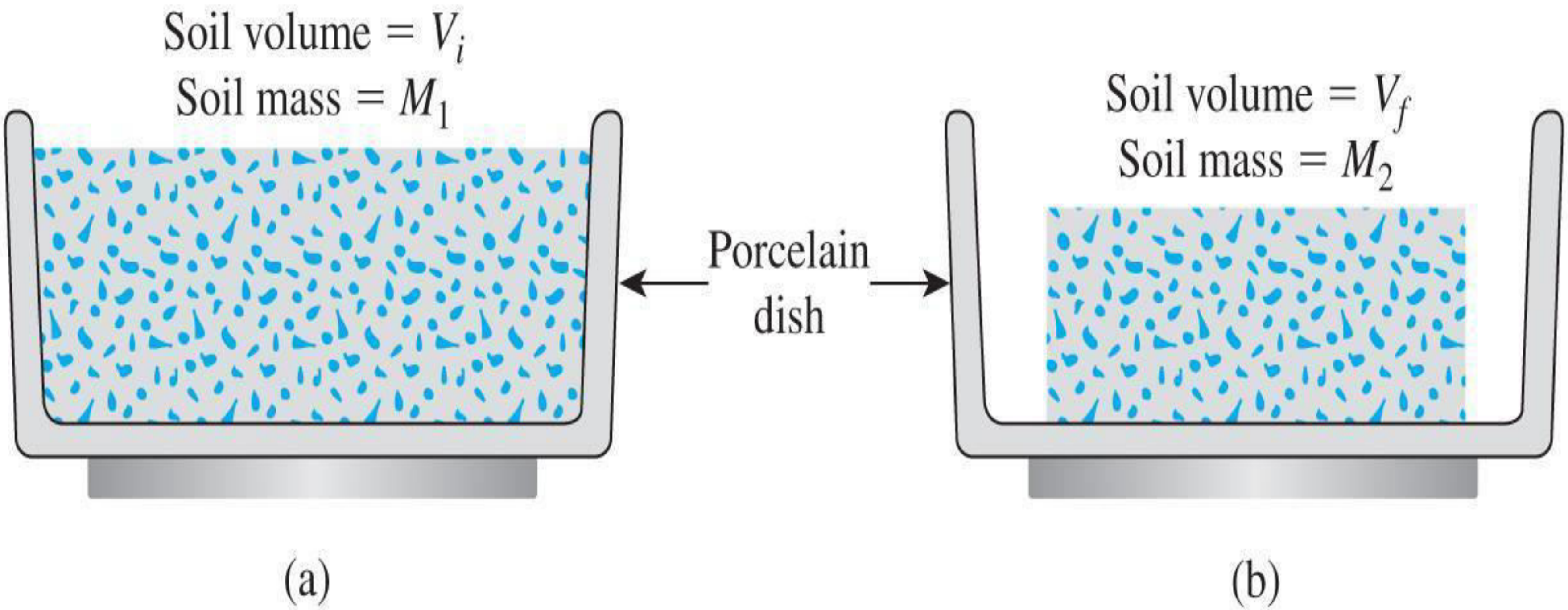


Figure 4.10 Shrinkage limit test: (a) soil pat before drying; (b) soil pat after drying

Shrinkage Limit – Test Procedure

Take about 40g of soil sieved through 425micron sieve and mix with distilled water to make a creamy paste.

Make a water content slightly above liquid limit so that the paste can be placed in the shrinkage dish without air voids.

Coat the inside of the shrinkage dish with a very thin layer of grease before filling with soil

weigh the dish and record the weight.

Fill the dish with wet soil in approximately three layers.

tapping the dish gently each time to exclude air bubbles.

Fill the last layer to slightly overflow and strike off smooth with a straight edge.

Weigh the dish with the wet soil.

Allow the wet soil pat to slightly air-dry and oven dry the pat for 24hrs

Find the dry weight of the pat thereafter.

Find the volume of the shrinkage dish by first filling it with mercury.

Press a flat glass plate down on the mercury surface to remove the excess overflows in the large evaporating dish

Weigh the dish with mercury and compute the volume of dish as
weight of mercury/13.58,

Note: 13.58 g/cm^3 being the unit weight of mercury.

This is also the initial volume of the soil pat.

Determine the volume of the dry soil pat by the mercury displacement method.

Fill the glass cup with mercury.

Remove excess mercury by pressing the glass plate over the top of the cup,

Now Place the dry pat on the surface of mercury in the cup

Gently force the pat into the mercury with the three pronged glass plate.

Collect the overflow mercury in the evaporating dish and weigh it.
Now Volume of dry Pat = weight of overflow mercury/13.8.
This is known as final volume of Dry Pat

Trial No	1	2	3
Container No	9		
Wt. of container, g	67.720		
Wt. of wet sample+ container, g	86.766		
Wt. of dry sample+ container, g	80.903		
Wt. of water, g	5.863		
Wt of dry soil pat, **Wo**, g	13.183		
Water content,% **W**	44.47		
Vol.of container,**V**.cm^3	10.88		
Vol.of dry soil pat, **Vo**,cm^3	6.49		
Shrinkage limit , w_s (%)	11.17		
Shrinkage ratio	2.03		

Shrinkage Limit calculation

$$W_S = W - \frac{(V - V_0)\,\gamma_w}{W_0} \times 100$$

Where W_S	=	Shrinkage limit in percent
W	=	Moisture content of the wet soil pat in percent
V	=	Volume of wet soil pat in cm^3
V_0	=	volume of dry soil pad cm^3
W_0	=	Weight of oven dry soil pat in g

Calculate the shrinkage index (Is) using the following formula

I_s = $W_L - W_S$ Where W_L = Liquid limit of the soil

Calculate Shrinkage ratio (SR) from

$SR = W_0 / V_0 \gamma_w$

Where

W_0 = Weight of oven dry pat in a g and

V_0 = Volume of oven drypat in cm^3

The shrinkage ratio gives an indication of how much volume change may occur with changes in water content.

Colloidal Content. %	Plasticity Index, %	Shrinkage limit. %	Swell Classification
0-15	0-15	>12	low
10-25	10-35	8-18	medium
20-35	20-45	6-12	high
>35	>30	<10	Very high

Shrinkage Index, %	Swell classification
0-20	low
20-30	Medium
30-60	High
>60	Very high

Consistency Indices

1. Plasticity Index (PI)

Difference between Liquid Limit and Plastic Limit

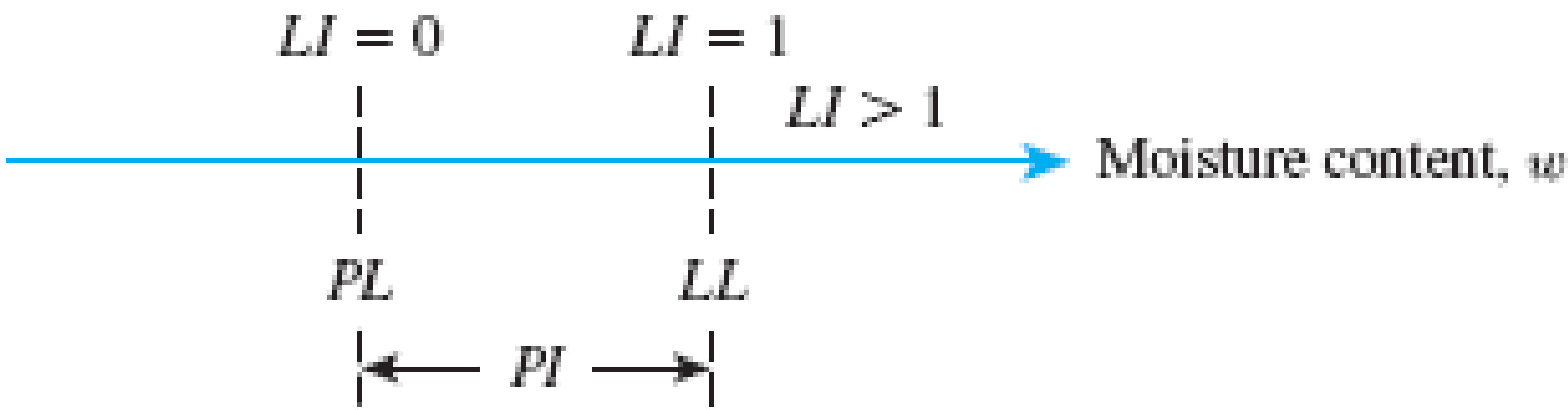
PI = LL - PL

PI	**Description**
0	Nonplastic
1–5	Slightly plastic
5–10	Low plasticity
10–20	Medium plasticity
20–40	High plasticity
>40	Very high plasticity

2. Liquidity Index (LI)

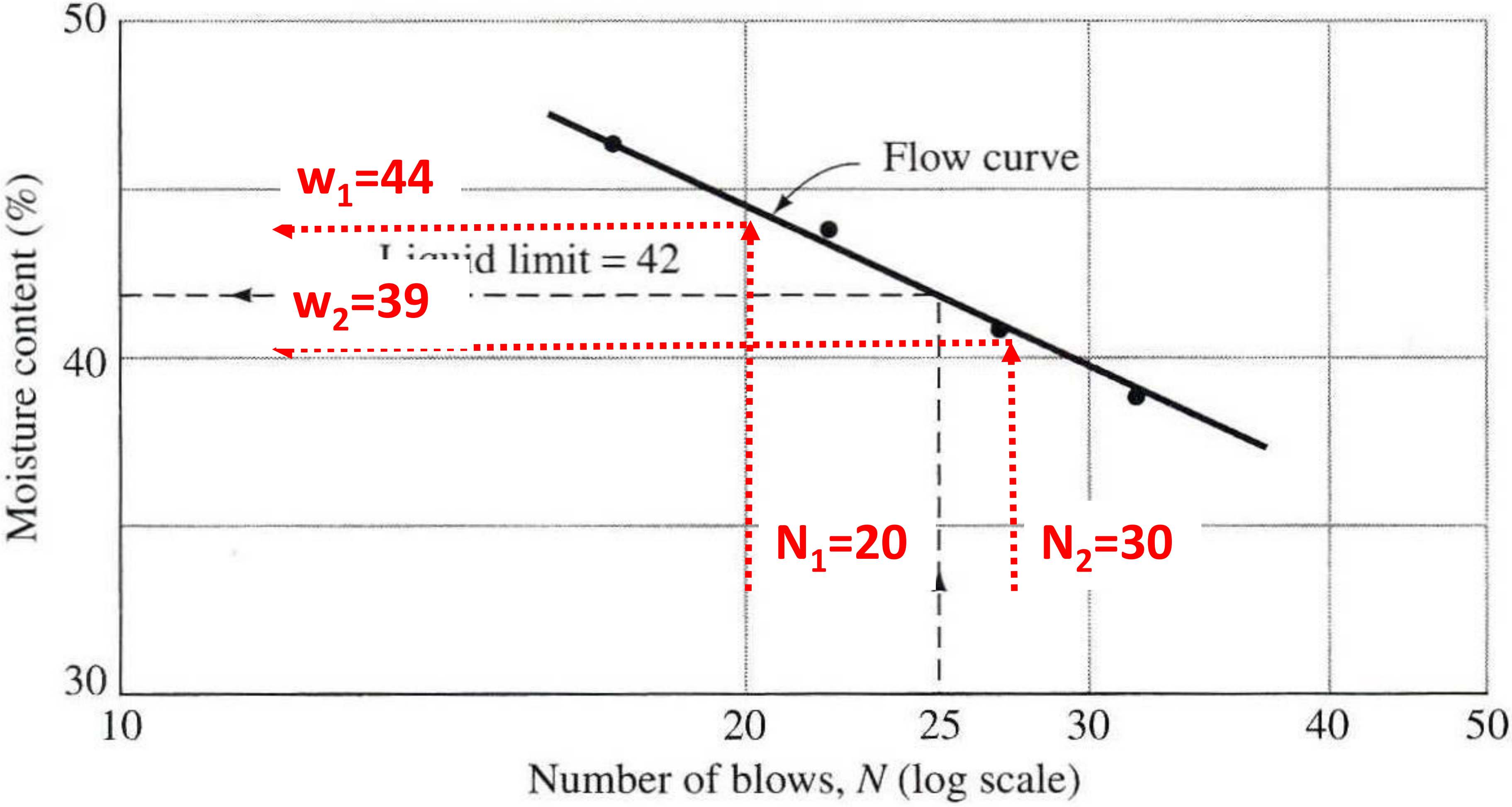
The relative consistency of a cohesive soil in the natural state

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



Liquidity index	Classification
> 1	Liquid
0.75 - 1.00	Very soft
0.50 - 0.75	Soft
0.25 - 0. 50	Medium stiff
0 - 0.25	Stiff
< 0	Semi-solid

3. Flow Index



50
40
30
Moisture content (%)
10
20
25
30
40
50
Number of blows, N (log scale)
Flow curve
Liquid limit = 42
w1=44
w2=39
N1=20
N2=30

Flow index:

Flow index is defined as the slope of the flow curve

Flow Index =

$$I_F = \frac{w_1 - w_2}{\log\left(\frac{N_2}{N_1}\right)}$$

Where N_2, N_1 are the number of blows corresponding to the water content W_1, W_2

4. Consistency Index

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

If *w is equal to the liquid limit, the consistency index* is zero. Again, if *w = PI, then CI = 1.*

5. Toughness index (I_t)

$$(I_t) = (I_P/I_f)$$

6. Activity

Activity is defined as the slope of the line correlating PI and %finer than 2 micrometer and expressed as:

$$A = \frac{PI}{(\%\text{of clay-size fraction, by weight})}$$

Table 4.1 Typical Values of Liquid Limit, Plastic Limit, and Activity of Some Clay Minerals

Mineral	Liquid limit, *LL*	Plastic limit, *PL*	Activity, *A*
Kaolinite	35–100	20–40	0.3–0.5
Illite	60–120	35–60	0.5–1.2
Montmorillonite	100–900	50–100	1.5–7.0

The activity factor gives information on the type and effect of CLAY MINERAL in a soil.

- Clay minerals with KAOLINITE have LOW activity, whereas those soils with MONTMORILLONITE will have a high activity value.

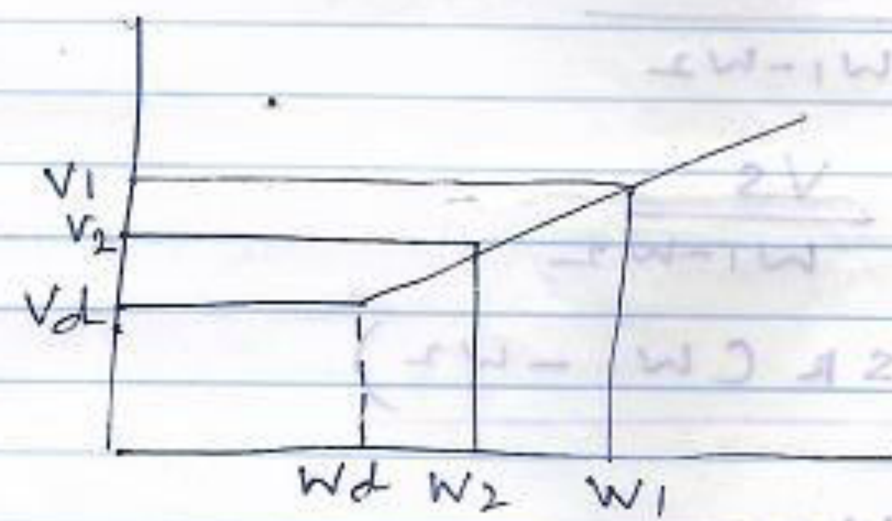
- Activity is used as an index for identifying the swelling potential of clay soils.

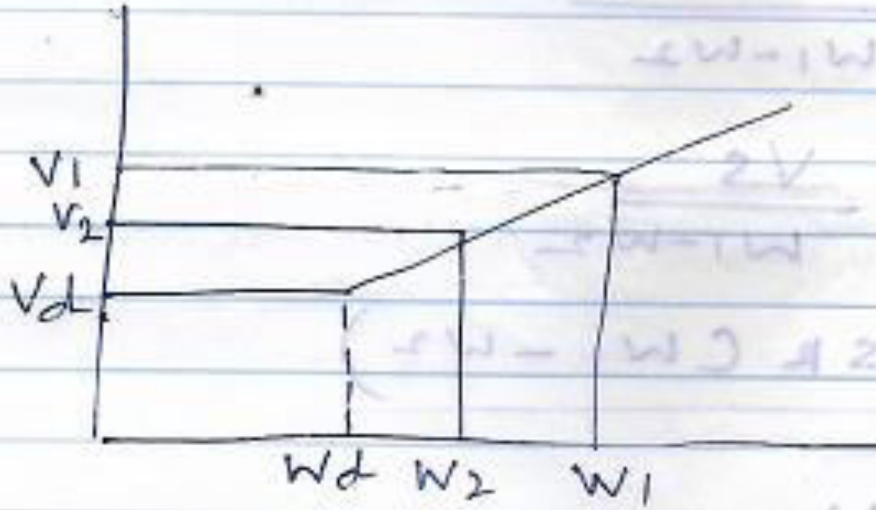
Activity	**Classification**
<0.75	Inactive clays
0.75-1.25	Normal Clays
>1.25	Active Clays

SHRINKAGE RATIO:

When wet soil Mass subjected to Reduction of water content, simeltaneously there is a reduction in the volume of soil mass.

Shrinkage Ratio is defined as Ratio of change in the volume with respect to dry volume to the change in the corresponding water content.





$$SR = \frac{V_1 - V_2}{V_d} \Big/ W_1 - W_2$$

V1 = Initial volume of soil

W1 = Initial water content

V2 = Final volume of soil after evaporation

W2 = Final water content after evaporation

Vd = dry volume of soil

Wd = Water content at shrinkage limit

$W_d = W_s$

Volumetric shrinkage

It is defined as decrease in the volume of soil mass when subjected to evaporation with respect to its dry volume

$$V_S = \frac{V_1 - V_2}{V_d} \times 100$$

$$SR = \frac{\frac{V_1 - V_2}{V_d}}{W_1 - W_2}$$

$$SR = \frac{V_S}{W_1 - W_2}$$

$$V_S = SR(W_1 - W_2)$$

Linear shrinkage

It is defined as decrease in one dimension of a soil mass expressed as a percentage of original dimension.

$$L_s = \left[1 - \left(\frac{100}{V_s + 100}\right)^{1/3}\right] 100$$

Where V_s = Volumetric shrinkage in %.

The following test results were obtained for a fine-grained soil:

$W_L = 48\%$; $W_P = 26\%$

Clay content = **25%**

Silt content = 35%

Sand content = 10%

In situ moisture content = 39% = w

Classify the soil, and determine its activity and liquidity index

Plasticity index, $I_P = W_L - W_P = 48 - 26 = 22\%$

Liquid limit lies between 35% and 50%.

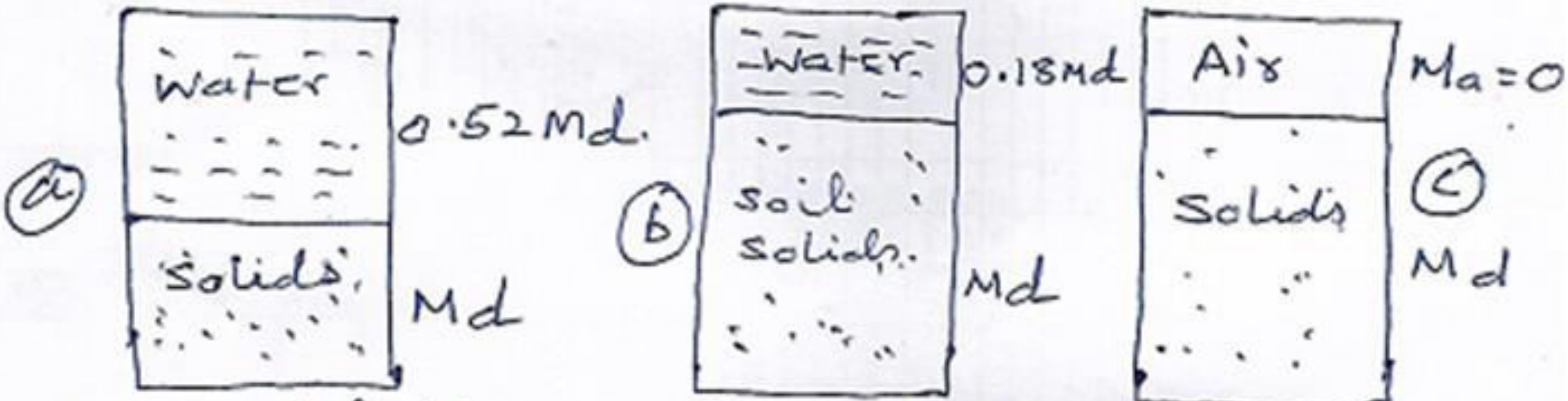
According to the Plasticity Chart, the soil is classified as CI, i.e. clay of intermediate plasticity.

$$\Rightarrow Activity = \frac{I_p}{Clay\ content} = \frac{22}{25} = 0.88$$

Liquidity index, $LI = \dfrac{w - W_p}{I_p} = \dfrac{39-26}{22} = 0.59$

The clay is of normal activity and is of soft consistency.

Given Liquid limit = 52%.
Plastic limit = 30%.
shrinkage limit = 18%.
Volume of shrinks from 39.5 cc to 24.2 cc at shrinkage limit.,
Calculate sp. gravity.



@ Liquid limit   @ shrinkage Limit   @ Dry state

Difference of mass of water in a & b

$= 39.5 - 24.2 = 15.3\,gm.$

$15.3 = (0.52 - 0.18)\,Md.$

$\therefore Md = 45\,gm.$

Volume of water in (b) @ S.L =

0.18 Md $\Rightarrow$ 0.18 × 45 = 8.1 cm³.

Volume of solids = 24.2 − 8.1 = 16.1 cm³.

$$\rho_s = \frac{Md}{Vs} = \frac{45}{16.1} = 2.8 \ g/cc$$

$$G_s = \frac{\gamma_s}{\gamma_w} = \frac{\rho_s}{\rho_w} = \frac{2.8}{1} = \underline{2.8}$$

The Natural Water content of a soil is equal to half of the sum of its liquid limit and plastic limit

Find the relation between liquidity index and consistensy index

Given $w = \frac{w_L + w_P}{2}$ —(1)

$$I_C = \frac{W_L - w}{I_P} \quad -(2)$$

$$I_L = \frac{w - w_P}{I_P} \quad -(3)$$

Sub. (1) in (2) $I_C = \frac{W_L - \frac{w_L + w_P}{2}}{I_P}$

$$I_C = \frac{2w_L - w_L + w_P}{2 I_P'} \quad -(4)$$

$$I_C = \frac{w_L + w_P}{2 I_P} \quad -(4)$$

$$I_L = \frac{w_L - w_p}{I_p} = \frac{\frac{w_L + w_p}{2} - w_p}{I_p}$$

$$I_L = \frac{w_L + w_p - 2w_p}{2I_p} = \frac{w_L - w_p}{2I_p} = \frac{\cancel{I_p}}{2\cancel{I_p}} = \frac{1}{2}$$

$$I_L = \frac{1}{2}$$

$$\frac{I_C}{I_L} = \frac{\;\frac{w_L + w_p}{2I_p}\;}{\frac{1}{2}} = \frac{(w_L + w_p)\cancel{2}}{\cancel{2}I_p} = \frac{w_L + w_p}{I_p}$$

$$I_C = \frac{w_L + w_p}{I_p} \times I_L$$

If consistency Index $= 1$.
What is the state of soil ?

$$I_C = \frac{w_L - w}{I_P}$$

w = Natural w. cont.

$I_P = w_L - w_P$.

$$1 = \frac{w_L - w}{w_L - w_P} \Rightarrow w_L - w_P = w_L - w$$

$$\Rightarrow \cancel{w_L} - \cancel{w_L} + w = w_P$$

$$\Rightarrow w = w_P.$$

i.e Natural water content itself at Plastic limit.

$I_C = 0 \Rightarrow$

$$0 = \frac{w_L - w}{w_L - w_P} \Rightarrow 0 = w_L - w$$

$\Rightarrow w = w_L \Rightarrow$ Natural water content at liquid limit.

If Liquidity Index = ϕ, What is the state of soil ?.

$$I_L = \frac{w - w_p}{I_p} \Rightarrow \frac{w - w_p}{w_L - w_p}$$

$$\phi = \frac{w - w_p}{w_L - w_p} \Rightarrow w_L - w_p = w - w_p$$

$$\Rightarrow w_L - \cancel{w_p} + \cancel{w_p} = w \Rightarrow$$ Natural wat. cont @ Liquid limit st

$$\text{If } I_L = 1, \Rightarrow 0 = \frac{w - w_p}{w_L - w_p} \Rightarrow \cancel{w_L - w_p = w - w_p}$$

$$\Rightarrow 0 = w - w_p \Rightarrow w = w_P$$

Natural water. cont. @ plastic limit.

The Dry density of soil is 18 kN/m^3 and specific gravity of solids is 2.7 and Degree of Saturation is 100%. Find the Moisture required.

$$Se = wG, \quad 1.0 \times e = w \times 2.7$$

$$e = \frac{G \gamma_w}{\gamma_d} - 1 = 0.5 \quad \therefore w = 18.5\%$$

2) If the Liquidity Index of a Soil is Zero, Find consistency Index

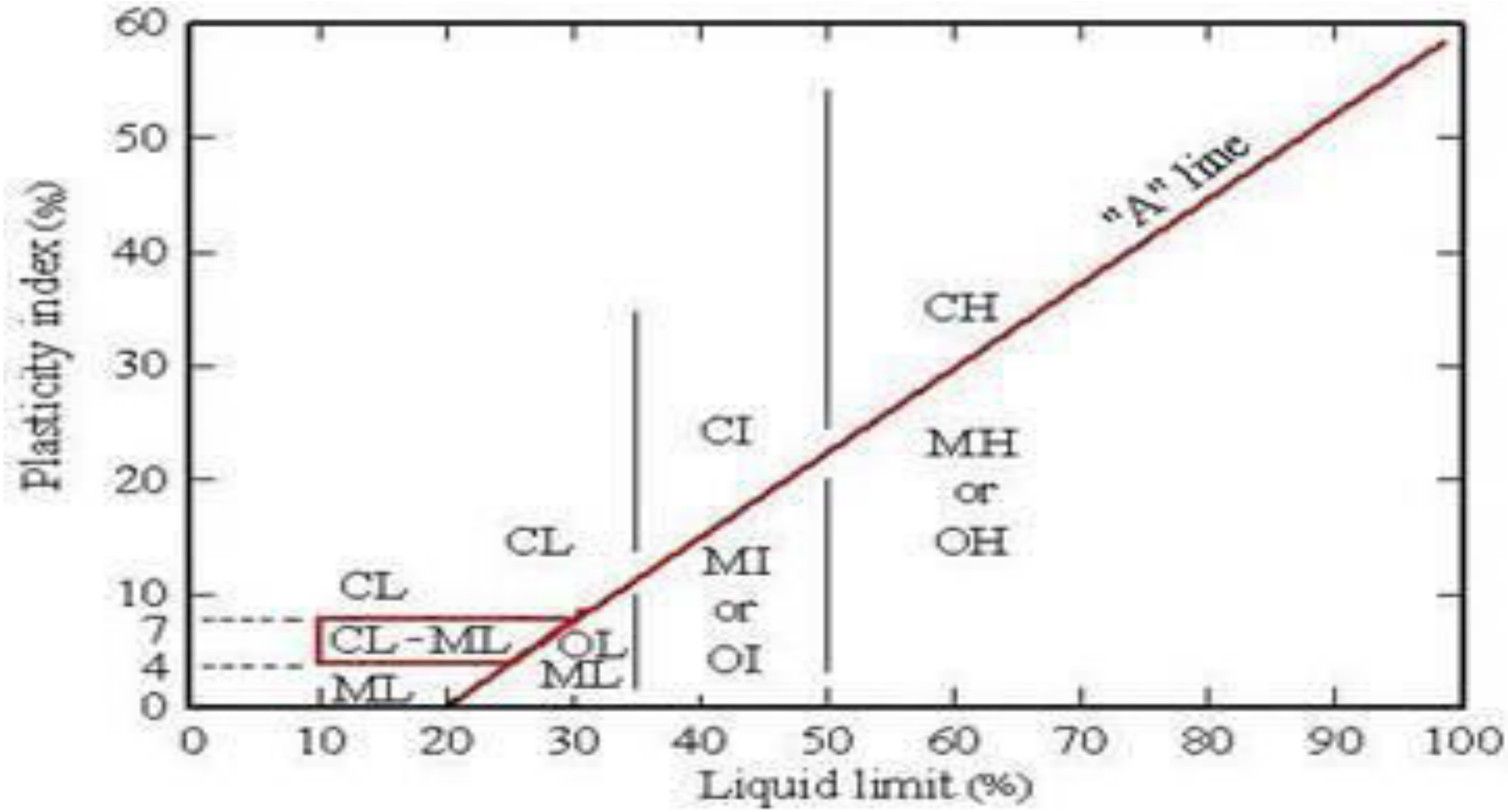
$$\text{Consistency Index} = \frac{W_L - W_n}{W_L - W_P} \quad \text{—(1)}$$

$$\text{Liquidity Index} = \frac{W_n - W_P}{I_P} = 0$$

$$\therefore W_n = W_P \rightarrow \text{sub in Eqn - (1)}$$

We get $\dfrac{W_L - W_P}{W_L - W_P} = 1.$

A Line Chart



60
50
40
30
20
10
7
4
0
Plasticity index (%)
"A" line
CH
CI
MH
or
OH
CL
MI
or
OI
CL
CL-ML
OL
ML
ML
0
10
20
30
40
50
60
70
80
90
100
Liquid limit (%)

Depending on the point in the chart, fine soils are divided into **clays (C)**, **silts (M)**, or **organic soils (O)**. The organic content is expressed as a percentage of the mass of organic matter in a given mass of soil to the mass of the dry soil solids Three divisions of plasticity are also defined as follows.

Low plasticity	$W_L < 35\%$
Intermediate plasticity	$35\% < W_L < 50\%$
High plasticity	$W_L > 50\%$

The **'A' line** and vertical lines at W_L equal to 35% and **50%** separate the soils into various classes.

For example, the combined symbol CH refers to clay of high plasticity.

Group Classification By ISC Method

Table 5.6. Classification of Coarse-grained Soils (ISC System)

Division	*Subdivision*		*Group symbol*	*Typical names*	*Laboratory Criteria*		*Remark*
(1) Coarse-grained soils (More than half of material is larger than 75-micron IS sieve size)	Gravel (G) (more than half of coarse fraction is larger than 4.75 mm IS sieve)	Clean gravels (Fines less than 5%)	(1) GW	Well graded gravels	C_u grater than 4 C_c between 1 and 3		When fines are between 5% to 12%, border line cases requiring dual symbols such as GP—GM, SW—SC, etc.
			(2) GP	Poorly graded gravels	Not meeting all gradation requirements for GW		
		Gravels with appreciable amount of fines (Fines more than 12%)	(3) GM	Silty gravels	Atterberg Limits below A-line or I_p less than 4	Atterberg Limits plotting above A-line with I_p betwen 4 and 7 are border-line cases requiring use of dual symbol GM—GC	
			(4) GC	Clayey gravels	Atterberg limits above A-line and I_p grater than 7		

Table 5.6. Classification of Coarse-grained Soils (ISC System)

<table>
<tr><th>Division</th><th colspan="2">Subdivision</th><th>Group symbol</th><th>Typical names</th><th colspan="2">Laboratory Criteria</th><th>Remark</th></tr>
<tr><td rowspan="4">(1) Coarse-grained soils (More than half of material is larger than 75-micron IS sieve size)</td><td rowspan="4">Sand (S) (More than half of coarse fraction is smaller than 4.75 mm IS sieve)</td><td rowspan="2">Clean sands (Fines less than 5%)</td><td>(5) SW</td><td>Well-graded sands</td><td colspan="2">C_u greeater than 6
C_c between 1 and 3</td><td rowspan="4">When fines are between 5% to 12%, border line cases requiring dual symbols such as GP—GM, SW—SC, etc.</td></tr>
<tr><td>(6) SP</td><td>Poorly-graded sands</td><td colspan="2">Not meeting all gradation requirements for SW</td></tr>
<tr><td rowspan="2">Sands with appreciable amout of fines (Fines more than 12%)</td><td>(7) SM</td><td>silty sands</td><td>Atterberg Limits below A-line or I_p less than 4</td><td rowspan="2">Atteraerg's Limits plotting above A-line with I_p between 4 and 7 are border-line cases requiring use of double symbols SM—SC</td></tr>
<tr><td>(8) SC</td><td>Clayey sands</td><td>Atterberg limits above A line with I_p greater than 7</td></tr>
</table>

Table 5.7. Classification of Fine-grained Soils (ISC System)

<table>
<tr><th>Division</th><th>Subdivision</th><th>Group Symbols</th><th>Typical names</th><th colspan="2">Laboratory Criteria (see Fig 5.6)</th><th>Remarks</th></tr>
<tr><td rowspan="6">2) Fine-grained soils (more than 50% pass 75μ, IS sieve)</td><td rowspan="3">Low-compressibility (L) (Liquid Limit less than 35%)</td><td>(1) ML</td><td>Inorganic silts with none to low plasticity</td><td>Atterberg limits plot below A-line or I_p less than 7</td><td rowspan="2">Atterberg limits pioting above A-line with I_p between 4 to 7 (hatched zone) ML-CL</td><td rowspan="4">(1) Organic and inorganic soils plotted in the same zone in plasticity chart are distinguished by odour and colour or liquid limit test after oven-drying.
A reduction in liquid limit after oven- drying to a value less than three- fourth of the liquid limit before oven- drying is positive identification of organic soils.</td></tr>
<tr><td>(2) CL</td><td>Inorganic clays of low plasticity</td><td>Atterberg limits plot above A-line and I_p greater than 7</td></tr>
<tr><td>(3) OL</td><td>Organic silts of low plasticity</td><td colspan="2">Atterberg limits plot below A-line</td></tr>
<tr><td rowspan="3">Intermediate compressibility (I) (Liquid limit greater than 35% but less than 50%</td><td>(4) MI</td><td>Inorganic silts of medium plasticity</td><td colspan="2">Atterberg limits plot below A-line</td></tr>
<tr><td>(5) CI</td><td>Inorganic clays of medium plasticity</td><td colspan="2">Atterberg limits plot above A-line</td><td rowspan="2">(2) Black cotton soils of India lie along a band partly above the A-line and partly below tha A line</td></tr>
<tr><td>(6) OI</td><td>Orgaic silts of medium plasticity</td><td colspan="2">Atterberg limits plot below A-line</td></tr>
</table>

Table 5.7. Classification of Fine-grained Soils (ISC System)

Division	*Subdivision*	*Group Symbols*	*Typical names*	*Laboratory Criteria (see Fig 5.6)*	*Remarks*
2) Fine-grained soils (more than 50% pass 75μ, IS sieve)	High compressibiliy (*H*) (Liquid limit greater than 50%)	(7) MH	Inorganic silts of high compressibility	Atterberg limits plot below A-line	(1) Organic and inorganic soils plotted in the same zone in plasticity chart are distinguished by odour and colour or liquid limit test after oven-drying. A reduction in liquid limit after oven-drying to a value less than three-fourth of the liquid limit before oven-drying is positive identification
		(8) CH	Inorganic clays of high plasticity	Atterberg limits plot above A-line	
		(9) OH	Organic clays of medium to high plasticity	Atterberg limits plot below A-line	
(3) Highly organic soil		Pt	Peat and other highly organic soils	Readily identified by colour, odour, spongy feel and fibrous texture	

Coarse Grain Soil classification using group symbols is as follows:

Group Symbol	Classification
Coarse soils	
GW	Well-graded GRAVEL
GP	Poorly-graded GRAVEL
GM	Silty GRAVEL
GC	Clayey GRAVEL
SW	Well-graded SAND
SP	Poorly-graded SAND
SM	Silty SAND
SC	Clayey SAND

Fine Grain Soil classification using group symbols is as follows:

Fine soils	
ML	SILT of low plasticity
MI	SILT of intermediate plasticity
MH	SILT of high plasticity
CL	CLAY of low plasticity
CI	CLAY of intermediate plasticity
CH	CLAY of high plasticity
OL	Organic soil of low plasticity
OI	Organic soil of intermediate plasticity
OH	Organic soil of high plasticity
Pt	Peat

Classify the following soils on the basis of Data provided.

Soil	w_L	w_P	% pass 75 μ	% Gravel > 4.75	% Sand 4.75 to 0.075
A	450	50	100	0	0
B	34	20	80	0	20
C	60	30	90	0	10
D	–	Nonplastic	100	0	0
E	35	20	20	60	20
F	–	NP	10	20	20

Soil - A → % passing 75 μ > 50%.
Hence Fine grained soil

$W_L > 450\%$. $W_P = 50\%$. ∴ $I_P > 400$

∴ It falls above A-line and $W_L > 50\%$

∴ Highly compressible clay

∴ The given soil is classified as
In organic clay of high compressible

Soil passing 75μ is > 50%. Hence Fine grained

soil B :-

$W_L = 34\%$. i.e. Less Than 35%. line

$W_P = 20\%$. ∴ $I_P = 34 - 20 = 14$ ~~> 35~~%.

∴ but W_L is very close to 35% line

∴ It is classified as CL - CI

@ $W_L = 34$
A-line pt is
$I_P = W_L - 20$
$= 34 - 20 = 14$
Slightly falling above A-line
∴ Fine
∴ grained

Soil C: - Soil Passing 75 micron sieve = > 50%;
Hence it is fine grained.
Liquid Limit >50%; Hence High Compressible
$I_p = W_L – W_p = 60 – 30 = 30$ and
I_p value on the A Line for the given Liquid Limit = $(w_L - 20) = (60 -20) = 40$
There fore Soil I_p lies below A line
Hence given soil is classified as MH = Silt of High compressible

Soil D

% passing $75\mu > 50\%$ at ∴ Fine grained

It is given Non plastic

∴ It is classified as Non plastic fine grained silt

of Low compressible silt

ML (Ans)

Soil E

% passing 75 μ < 50%.

It is Coarse grain

% ~~gravel~~

% of gravel more than 50%

∴ It is gravel

$I_P = 35 - 20 = 15 > 12\%$.

silt

∴ It is mixture

∴ ~~GM~~

Soil ~~passing~~ 75 μ & 4.75 mm is more than 12%

ie 20%.

$W_L = 35\%$. $I_P = 15$

$I_P > 7$

above A-line

The pt Falls

Hence it is classified GC

Soil F

% passing 75 μ is less than 50%

∴ It is coarse grain soil

% Sand is > 50% ⇒ 70% ∴ It is Sand

It is also given Non plastic

Soil passing 75 μ is 10%. It is bet 5 & 12%

∴ Dual classification SP-SM (NP)

(or) SW-SM (NP)

For correct classification Cu and Cc Values

are required

Soil Compaction

Compaction

The process of bringing the soil particles closer to a dense state by mechanical means.

The voids are reduced by expulsion of air and the soil particles are packed together, thereby increasing its unit weight.

Objectives of Compaction

- Increase the shear strength.
- Increased bearing capacity for foundation support.
- Reduce compressibility and smaller settlement of buildings and lesser deformation of earth structures.
- Reduce permeability, leading to less seepage of water.
- Improve stability and lower damage due to frost action.
- To reduce the degree of shrinkage and formation of cracks on drying.

II. Laboratory Methods for Determining OM and MD

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

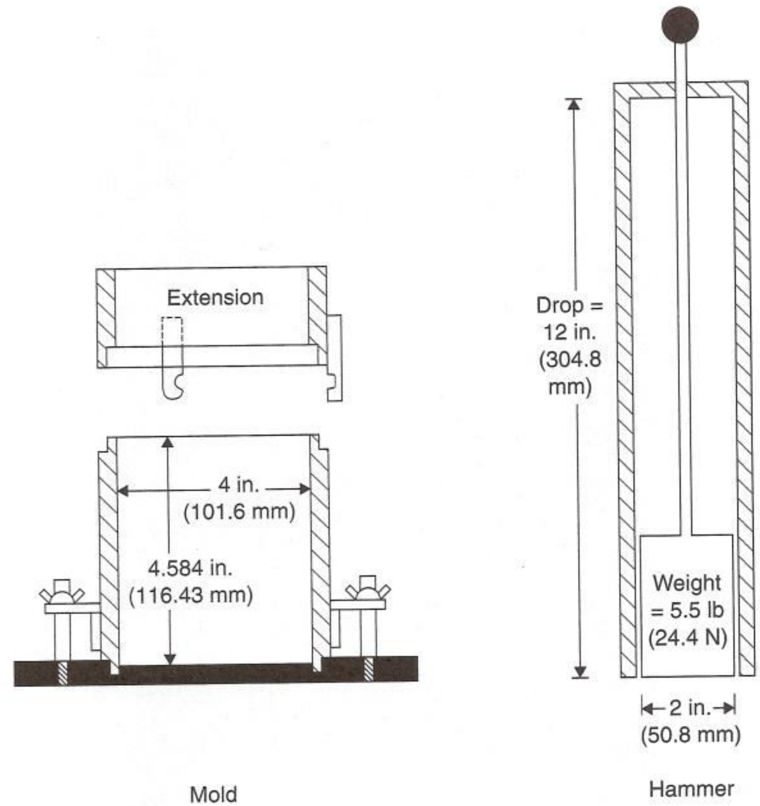


Figure 12.2. Standard Proctor mold and hammer.

Test	Hammer Mass, Kg	Hammer Drop, m	Compactive Energy kJ/m^3
Standard Proctor	2.5 (25 blows per layer, 3 layers)	0.3	590
Modified Proctor	4.5 (25 blows per layer, 5 layers)	0.46	2700

Recommended procedure

Take about 3kg (normal weight) of air dry – material passing through 4.75 mm sieve.

Measure the diameter and height of the mould without collar and find the volume of the mould (V).

Clean the empty mould and weigh it to the nearest gram (W_1).
Grease the inside of the mould lightly.
Fit the mould with the collar on to the base plate and place it on the solid base.

Add enough water to the soil to bring its moisture content to about 6%
Mix thoroughly to ensure uniform distribution of moisture.

Place the moist soil in the proctor mould in three layers of about equal thickness and compact each layer 25 blows with the hammer.
Take care to uniformly distribute the blows

Remove the collar and carefully strike both the top and base of the compacted cylinder of soil with a steel straight

Weigh the mould and cylinder of soil to the nearest gram (W_2)

Eject the cylinder of soil from the mould, split it and take three water content samples, one near the top, one at middle and the other near the bottom and mix it as one sample and Weigh the sample and oven-dry

Break up the sample and mix it with the unused portion.
Add sufficient water to raise the water content by about 2-3 percent,
carefully remix and repeat the experiment until
the peak wet density is followed by two slightly lesser compacted weights

II. The Method

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

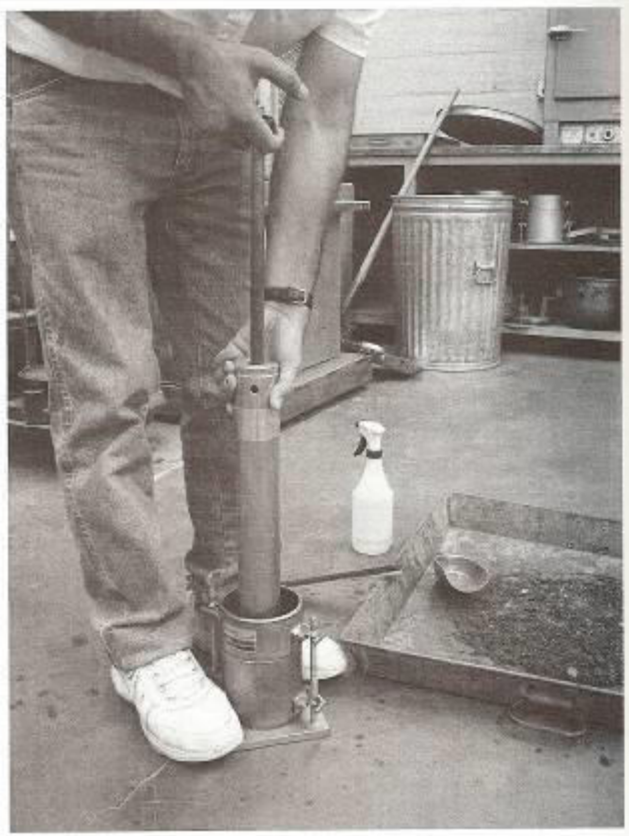
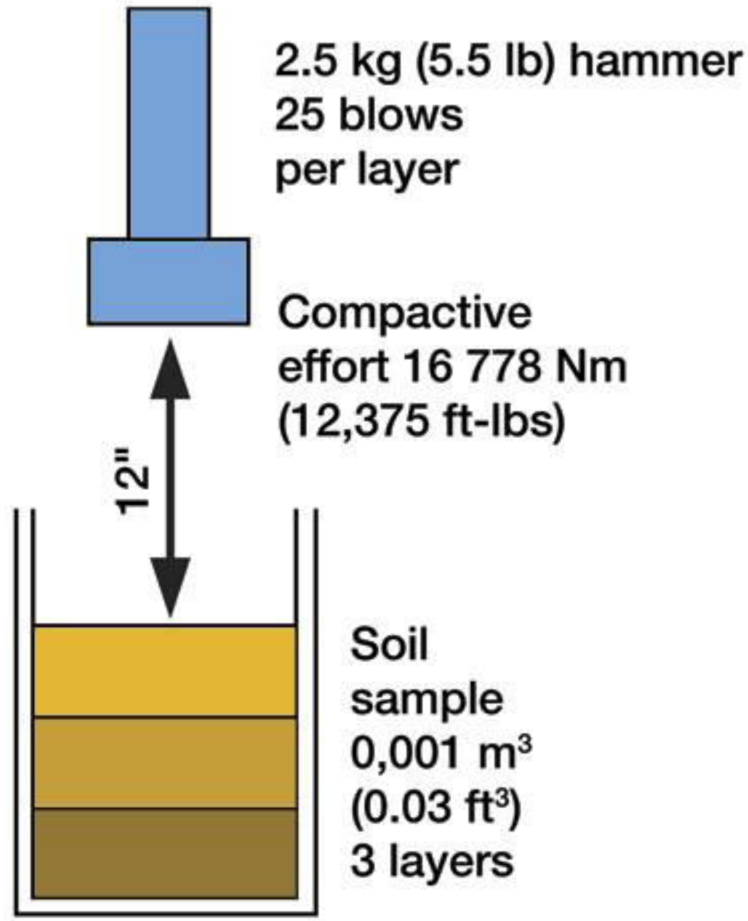


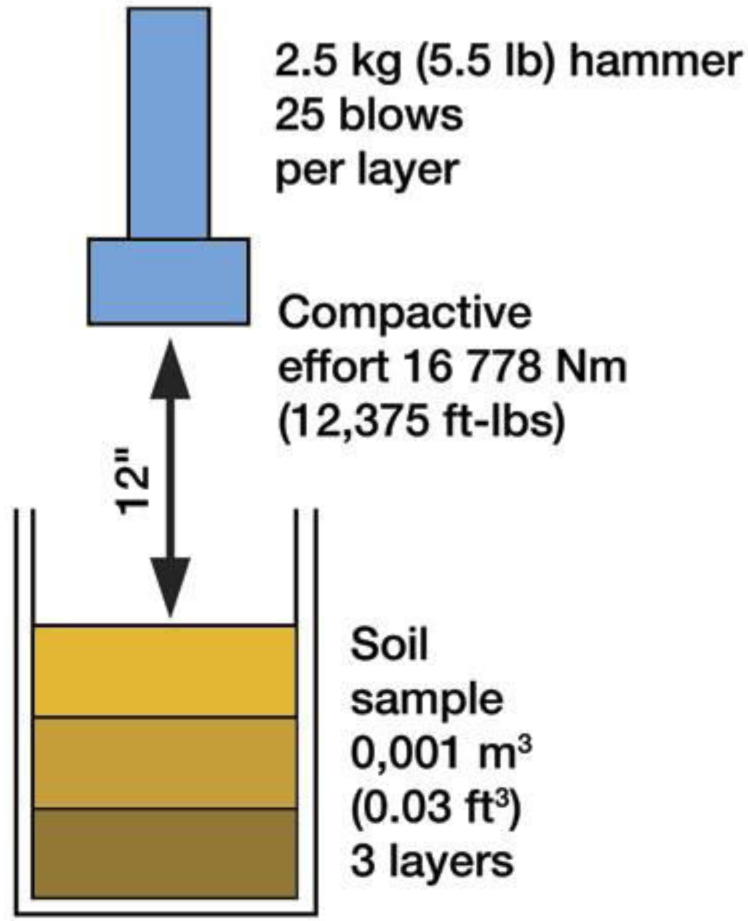
Figure 12.3. Compaction of soil in Proctor mold.



II. The Method

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

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



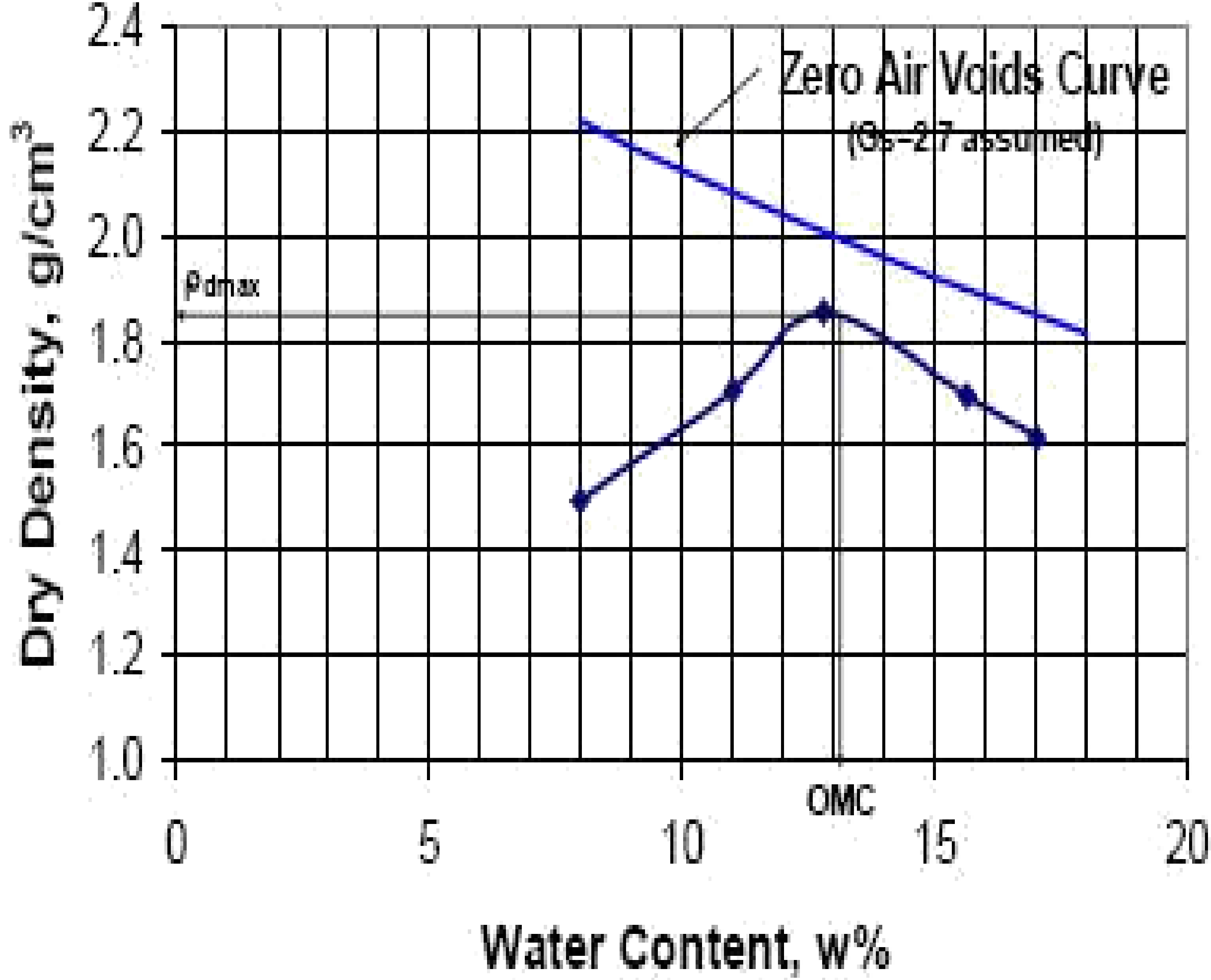
Calculation:

1. Wt. of compacted soil = $W_2 - W_1$ =

2. Wet density, $\gamma_b = (W_2 - W_1) / V$ =

3. Dry density, $\gamma_d = \gamma_b / (1 + w/100)$ =

4. Void ratio, $e = (G_s \gamma_w / \gamma_d) - 1$ =


Zero Air Voids Curve
(Gs=2.7 assumed)
ρdmax
OMC
Dry Density, g/cm3
Water Content, w%
2.4
2.2
2.0
1.8
1.6
1.4
1.2
1.0
0
5
10
15
20

Zero air void curve - the right hand limb of moisture – density curve roughly parallels a line designated as "zero air voids" This line represents the dry density if the entire volume is water and solids. Since compaction is a process for expelling air, the moisture density curves cannot cross this line. Since the line represents a theoretical limit on density at any water content, its position is often shown on moisture-density plots. The zero air-voids density for any moisture content maybe calculated from

$$\gamma Z(av) = G_s \gamma_w / \left(1 + (w\, G_s/100)\right)$$

where $\gamma_{z(av)}$ = dry density at saturation, G_s=the specific gravity of soil particles, γ_w= the unit weight of water and w the moisture content

Computations:

Compute the dry density and make a plot of dry density versus water content. Note the maximum dry density and optimum water content for the type of soil tested and the compactive energy used. On the curve of dry density versus water content plot the zero –air voids curve.

1. Dry density, $\gamma_d = \gamma_b / (1+w/100)$ g/cm^3

   Where γ_b = wet density g/cm^3

   w = Water content

2. Wet density, $\gamma_b = (W_2 - W_1) / V$ g/cm^3

   Where W_2 = Wt. of mould + compacted soil, g

   W_1 = Wt. of mould, g

   V = Volume of the mould.

3. Water content = w = (Wt. of water / wt. of soil) x 100

4 Void ratio, $e = (G_s \gamma_w / \gamma_d) - 1$

Where G_S = Specific gravity of soil

γ_w = Specific gravity of water

5. Zero air voids = $\gamma_{Z(av)} = G_s \gamma_w / 1 + (w\, G_s / 100)$

6. Compactive energy/ unit volume

$$= \frac{\text{(number of blows / layer) X (Number of layers) X Wt. of hammer X Ht. of drops of hammer}}{\text{Volume of the mould}}$$

Soil Compaction in the field

Soil Compaction can be achieved either by static or dynamic loading:

1- Smooth-wheel rollers

2- Sheep foot rollers

3- Rubber-tired rollers

4- Vibratory Rollers

5- Vibro flotation

Field Compaction depends on:

– Weight of roller
– No of passes of roller

Because of the differences between lab and field compaction methods, the maximum dry density in the field may reach 90% to 95%.

Soil Compaction in the Field:

1- Rammers

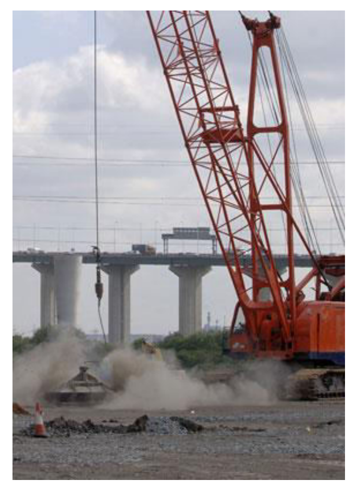
2- Vibratory Plates

3- Smooth Rollers

4- Rubber-Tire

5- Sheep foot Roller

6- Dynamic Compaction



Factors affecting Compaction

1. Physical & chemical properties
2. Moisture content
3. Method of compaction
4. Amount of compactive effort
5. Thickness of layer or “lift” being compacted

Moisture Content and Compaction

The degree of compaction of soil is measured by its unit weight, , and optimum moisture content,

By reducing the air voids, more soil can be added to the block.

When moisture is added to the block water content, is increasing

the soil particles will slip more on each other causing more

reduction in the total volume, which will result in adding more soil

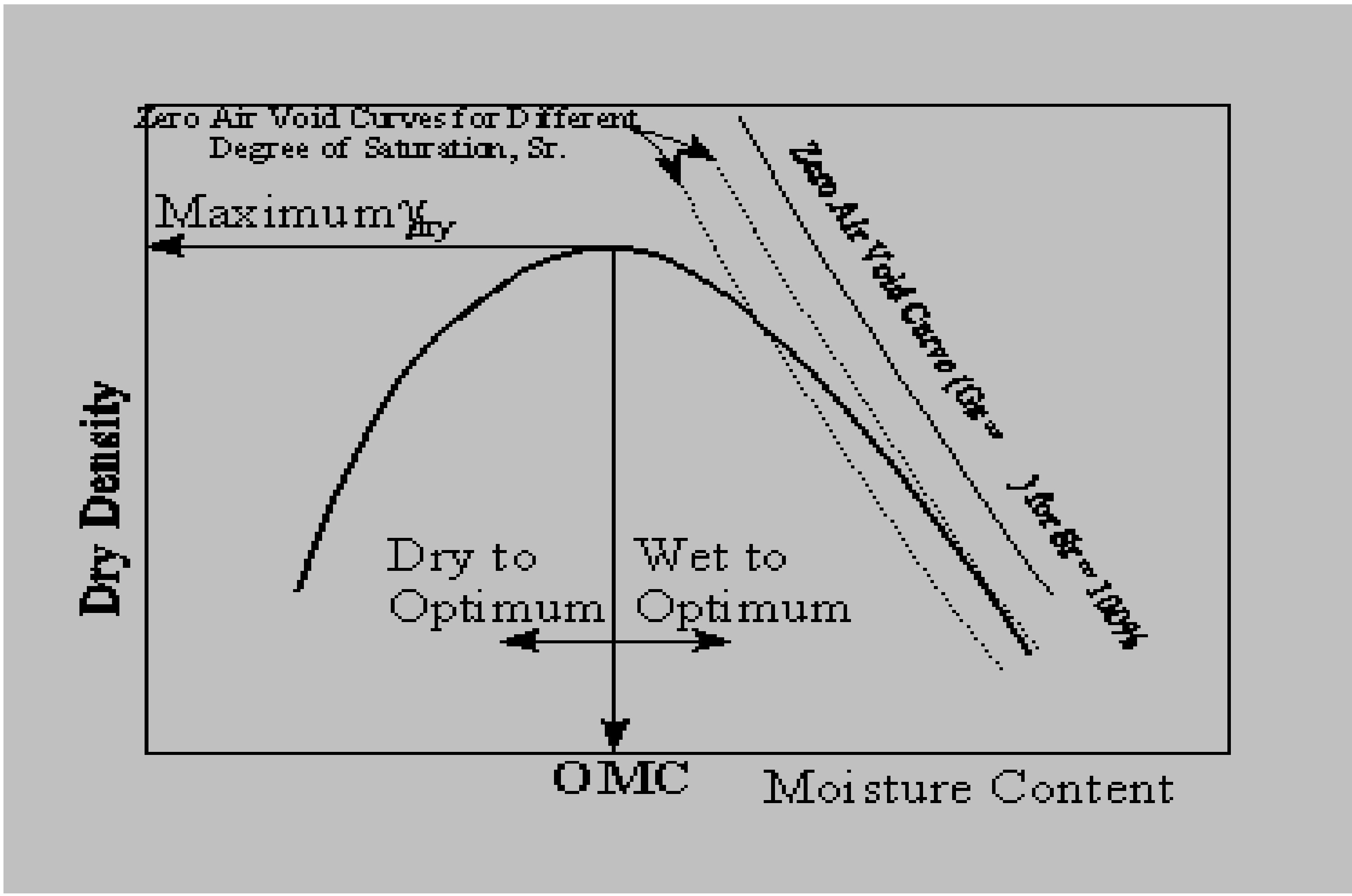
and, hence, the dry density will increase, accordingly.

Increasing water content will increase dry unit weight to certain limit

called Optimum moister Content, (OMC)

After this limit Increasing water will decrease unit weight

Optimum Moisture Content



2. Compaction Efforts.

The increase in amount of compaction (energy applied per unit of volume) results in an increase in the maximum dry density and decrease the OMC.

In laboratory compaction efforts are applied through:

Two Tests are usually performed in the laboratory to determine the maximum dry unit weight and the OMC.

Standard Proctor Test

Modified Proctor Test

In both the cases the compaction energy is given as

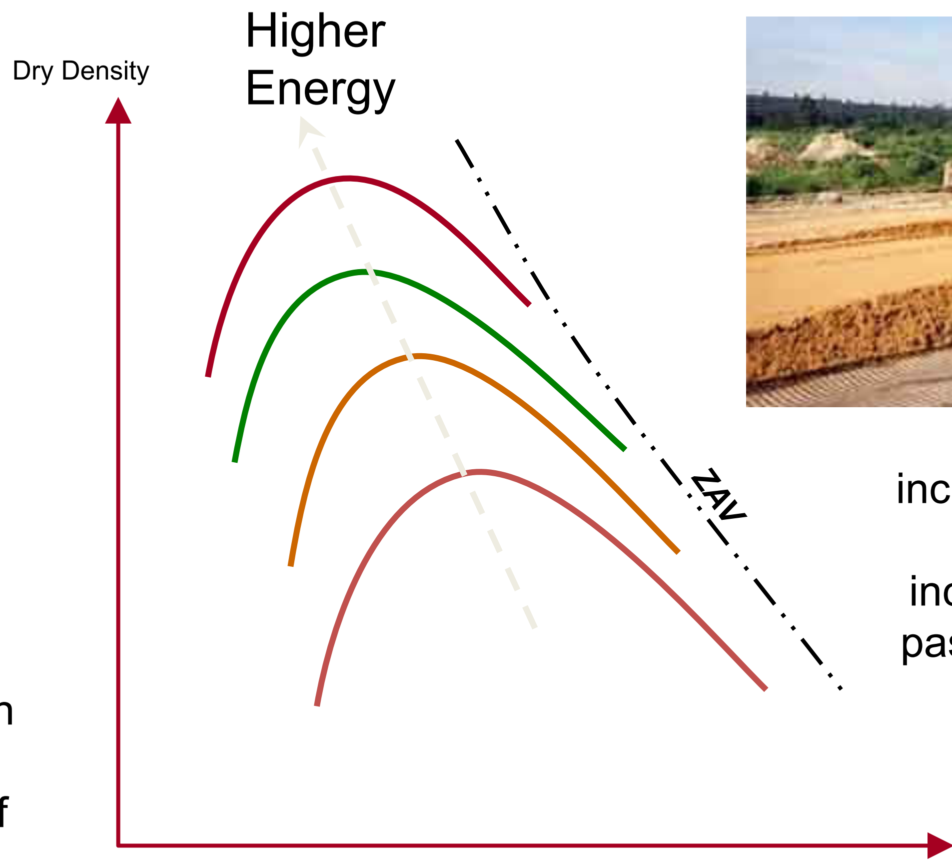
The degree of compaction is not directly proportional to compaction efforts and dry density doesn’t increase indefinitely.

When the soil is initially loose, the compaction increases the dry density, but further compaction beyond certain point doesn’t increase the density.

Effect of Energy on Soil Compaction

Increasing compaction energy → Lower MC and higher dry density

In the lab increasing compaction energy = increasing number of blows



In the field increasing compaction energy = increasing number of passes or reducing lift depth

3. Soil Type:

The following physical properties of soil has direct effect on the compaction efforts:

Shape of particles

Specific gravity of solids.

Amount and type of clay minerals.

Texture of soils

4. Compaction method

Compaction efforts may be provided by:

Kneading (Using punching device)

Dynamic (Rollers)

Static action (Mechanical jacks)

5. Admixtures:

Lime
Cement
Bitumen
Industrial wastes

7. Processing amount

By thorough mixing of moisture in the soil, higher density is achieved.

8. Energy Distribution:

Uniform distribution of compaction loads lead to better compaction and higher dry density.

Exercise Questions

1. The mass of wet soil when compacted in a mould was 19.55 kN. The water content of the soil
was 16%. If the volume of the mould was 0.95 m3. Determine (i) dry unit weight, (ii) Void
ratio, (iii) degree of saturation and (iv) percent air voids. Take G = 2.68

2.Sandy soil in a borrow pit has unit weight of solids as 25.8 kN/m3, water content equal to
11% and bulk unit weight equal to 16.4 kN/m3. How many cubic meter of
compacted fill could be constructed of 3500 m3 of sand excavated from borrow pit, if
required value of porosity in the compacted fill is 30%. Also calculate the change in degree
of saturation.

3.A soil has a bulk unit weight of 20.11 KN/m3 and water content of 15 percent. Calculate the
water content of the soil partially dries to a unit weight of 19.42 KN/m3 and the
voids ratio remains unchanged

4.Explain Standard Proctor Compaction test with neat sketches.

5.Explain all the consistency limits and indices.

6.Explain in detail the procedure for determination of grain size distribution of soil by
sieve analysis

EXERCISE PROBLEMS

A 75 mm (internal diameter) thin walled sampling tube is pushed into the wall of an excavation and a 200 mm long undisturbed specimen, weighing 1740.6 g, was obtained. When dried in the oven, the specimen weighed 1421.2 g. Assuming that the specific gravity of the soil grains is 2.70, find the void ratio, water content, degree of saturation, bulk density and dry density.

(Ans. 0.679, 22.5%, 89.5%, 1.970 g/cm³, 1.608 g/cm³)

3. Find the weight of a 1.2 m^3 rock mass, having a porosity of 1%. Assume that the specific gravity of the rock mineral is 2.69.

(Ans. 31.35 kN)

4. A soil sample has the following characteristics: w = 18.5%, γ_m = 19.6 kN/m^3 and G_s = 2.72. Find the void ratio, degree of saturation and the dry unit weight.

(Ans. 0.613, 82.0%, 16.54 kN/m^3)

5. A one metre thick fill was compacted by a vibrating roller, and there was 30 mm reduction in the fill thickness. The initial void ratio was 0.94. What would be the void ratio after compaction?

(Ans. 0.883)

6. A 75 mm diameter and 20 mm thick cylindrical saturated clay specimen has a mass of 164.1 g. When dried in the oven at 105^{o}C, the mass is reduced to 121.3 g. What is the specific gravity of the soil grains?

(Ans. 2.66)

- 7. A section of canal 200 m long and 10 m wide is being deepened 1 m by means of a dredge. The effluent from the dredging operation is found to have a unit weight of 12.5 kN/m^3. The soil at the bottom of the canal has an in-place unit weight of 18.6 kN/m^3. The specific gravity of the soil grains is 2.69. If the effluent is to be pumped out at the rate of 500 litres/minute, how many operational hours would be required to complete the dredging work?-
- *(Ans. 218 hours*

8. Soil for a compacted earth fill is available from three different borrow sites. At the earth fill, the soil is to be compacted to a void ratio of 0.62 with a finished volume of 150,000 m^3. The in situ void ratios and the costs (soil and transportation) per cubic metre for the three sites are given below. Which site would be economical? Borrow	Void ratio	Cost/m^3
X	0.85	\$7.80
Y	1.1	\$7.50
Z	1.4	\$6.60

THANK YOU

UNIT II

Soil Water,

Permeability and stress distribution

UNIT II SOIL WATER AND PERMEABILITY **9**

Soil water – types – capillary stress – Permeability measurement in the laboratory and in field – factors influencing permeability of soils – Seepage – introduction to flow nets – Simple problems – effective stress concept in soil

Types of Soil water

Free water

Held water

Free Water

It moves freely in the pores of the soil under influence of gravity

It flows from one point to another point when there is a difference of Head(Elevation)

The rate at which the head reducing along the flow passage is called Hydraulic gradient

$$= I = h/L$$

The flow of free water in soil is just like laminar flow through pipes

Held water

It is retained in the pores of soil. It can not move under the influence of gravity

Further Classification of Held Water

Structural Water

Adsorbed water

Capillary water

Types of held water :

- 1. Adsorbed water
  - "HYGROSCOPIC WATER"
  - It is held by electrochemical forces existing on the soil surface
  - Quantity depends upon the colloidal fractions in the soil
  - It is significant in the clay soil and negligible in coarse grained soils

  - Remove by oven drying
  - Not available to plants

2. Structural water

- It is chemically combined water in the crystals of structure of the minerals in the soil
- This water can not be removed without breaking the structure of the minerals
- A high temperature of 300°C is required for removing this water

CAPILLARY WATER

This water held in the pore space lines(interstices) of soil due to capillary forces(Surface Tension)

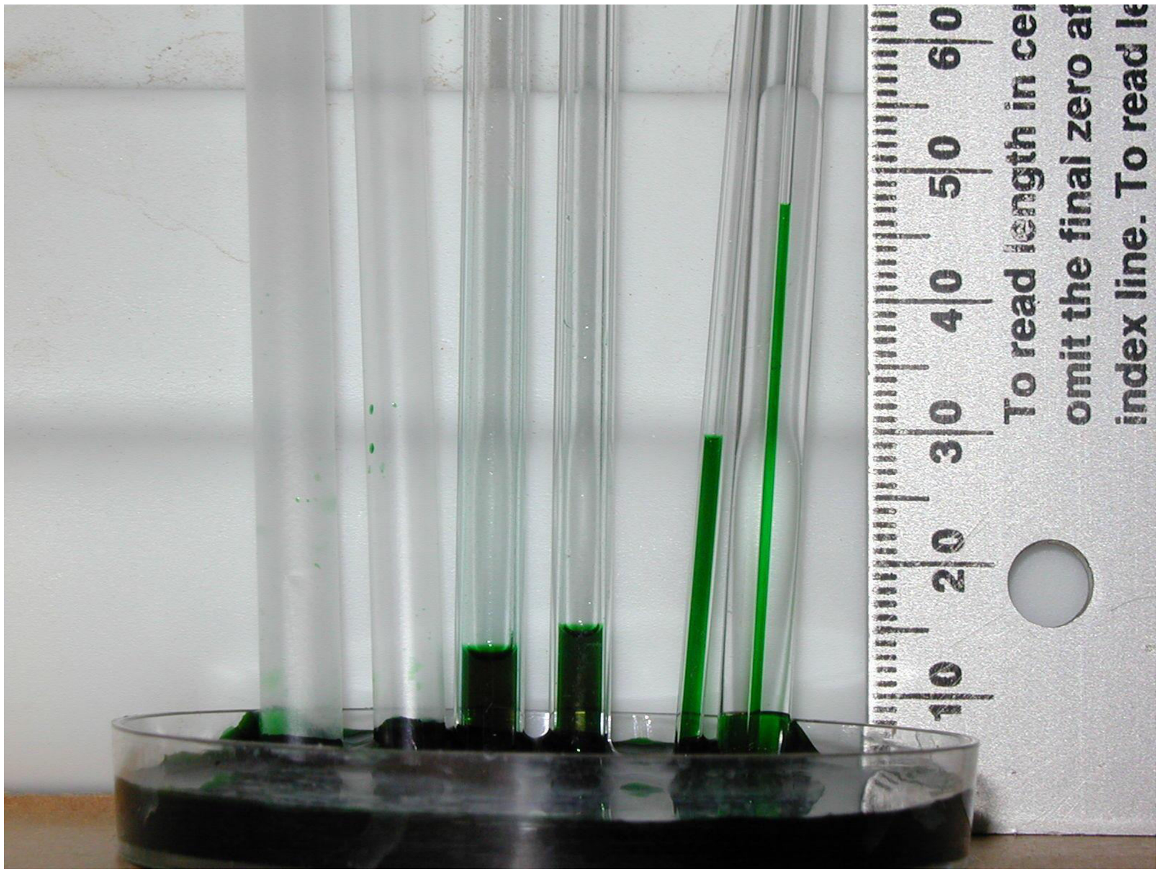
It exists in soil so long as

there is an air- water interface

As soon as the soil submerged in water,

the capillary water become normal

capillarity



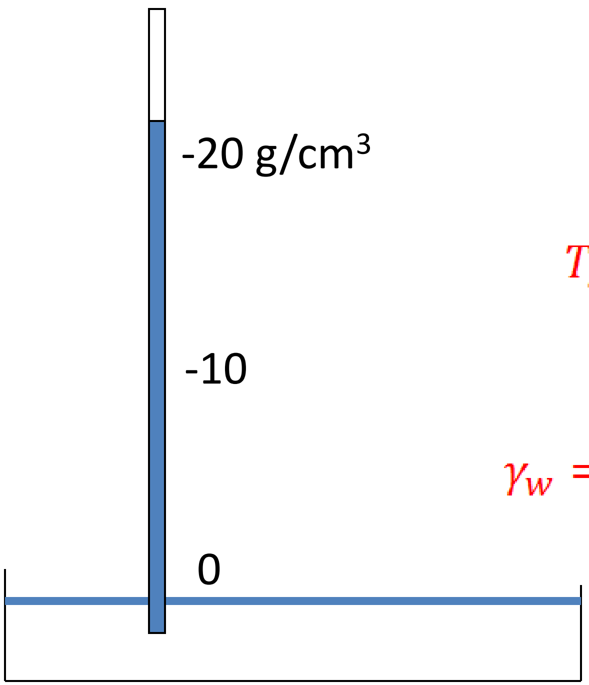
Plastic Glass

Height water will rise in cylinder depends on diameter of tube; due to adhesion of water and tube

Capillary pressure

- Thin tube in open pan water

$$h_C = \frac{4\,T_S \cos\theta}{\gamma_w d}$$

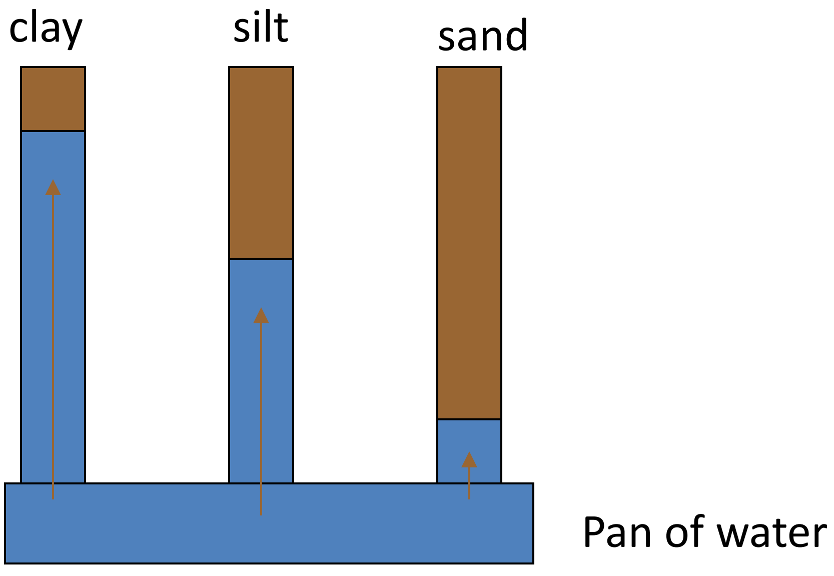


$Where\ h_C = Height\ of\ capillaryCapillary$

$T_{S=Surfce\ tension,\theta=Angle\ between\ surfce\ of\ water\ and\ wall}$

$\gamma_w = Unit\ weight\ of\ water, d = diameter\ of\ porespace$

- the smaller the pore space, the higher capillary water will rise in profile



Seepage Pressure or Seepage Force

Seepage:

Flow of water through a soil under hydraulic gradient

Seepage pressure

When water flowing through soil pores, a viscous friction exerted by water on the soil surface

due to that

An energy transfer is effected between soil and water

The force corresponding to this energy transfer is called seepage force or pressure.

Thus

It is the pressure exerted by water on the soil through which it percolates.

DARCY'S LAW

- The law of flow of water through soil was first studied by Darcy in the Year 1856.

- "for laminar flow through saturated soil mass, the discharge per unit time is proportional to the hydraulic gradient.

$$q = k * I * A$$

$$q/A = k * I$$

$$v = k * I$$

q = discharge per unit time

A = total c/s area of soil mass.

I = hydraulic gradient = h/L

k = darcy's coefficient of permeability

- ✓ $I = h/L = (h1 - h2)/L$
- ✓ $q = k * (h1-h2) * A/L$

Co Efficient of Permeability

Also called hydraulic conductivity

$k = v/i$

Ability of a soil to transmit water or allow water to flow through it

Factors affecting permeability of soils

i. Particle size.
ii. Properties of pore fluid.
iii. Void ratio of soil.
iv. Shape of particles.
v. Structure of soil mass.
vi. Degree of saturation.
vii. Adsorbed water.
viii. Impurities in water.

1) Particle size :-

permeability varies approximately as the square of grain size

$$K=C*(D10)^2$$

k=coefficient of permeability(cm/sec)

D10=effective diameter(cm)

C= constant = between 100 and 150

2)Properties of pore fluid :

The permeability is directly proportional to the unit weight of water and inversely proportional to is viscosity.

3)Void ratio of soil :

The coefficient of Permeability varies as $e^3/(1+e)$.

For a given soil , greater the void ratio , the higher is the value of the coefficient of permeability.

4) Shape of particles :

The permeability of soil depends upon the shape of particles.

5) Structure of soil mass :

stratified soil deposits have greater permeability when flow parallel to the plane of stratification then that perpendicular to this plane.

6) degree of saturation :

If the soil is not fully saturated, it contains air pockets formed due to entrapped air. The presence of air in soils, causes blockage of passage and permeability is reduced.

7) Adsorbed water :

the fine grained soils have a layer of adsorbed water strongly attached to their surface. There by reduce permeability.

8) Impurities in water :

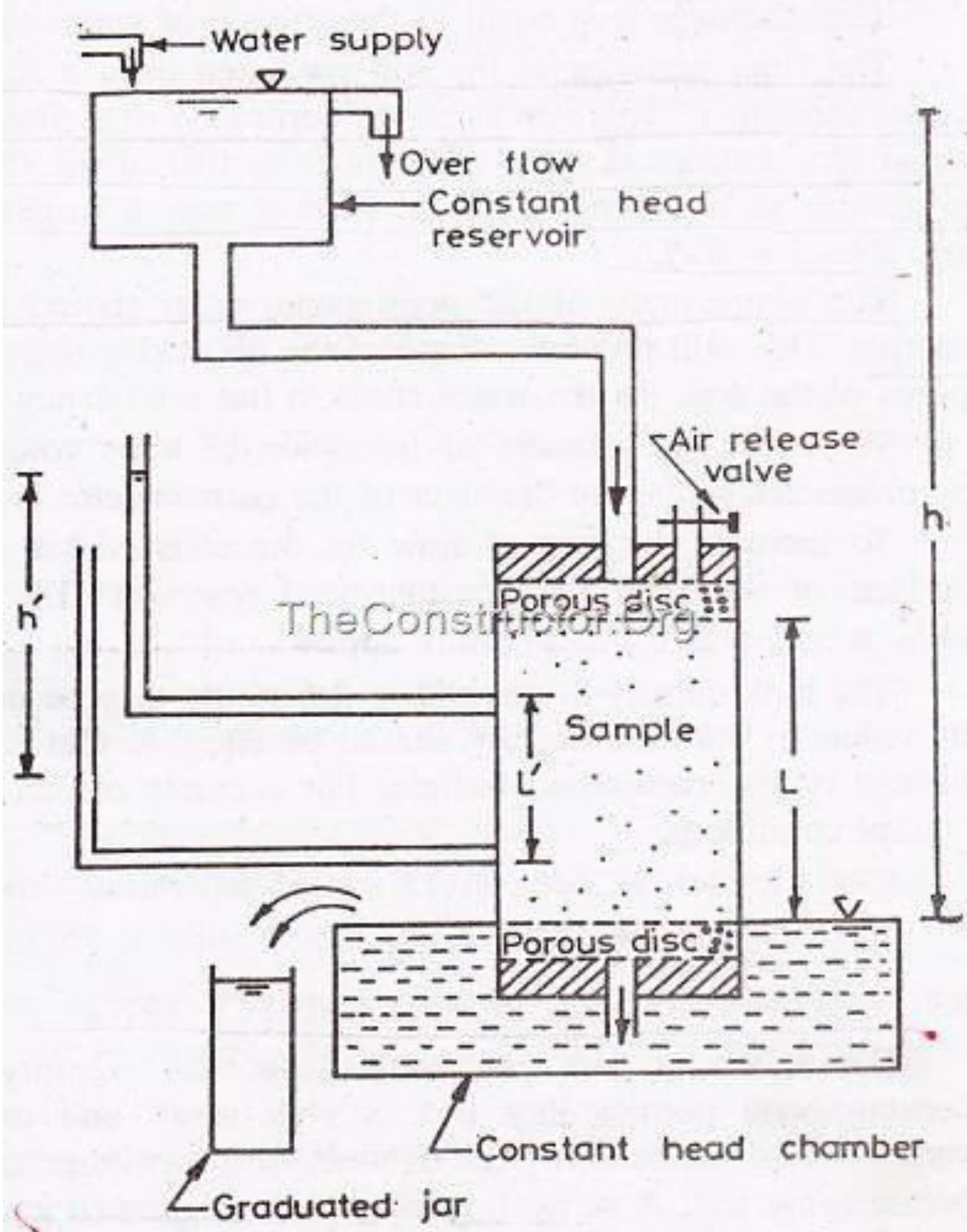
Any foreign matter in water has a tendency to plug the flow passage and reduce the permeability of soils.

DETERMINATION OF COFFICIENT OF PERMEABILITY

1) Laboratory methods
   - A-: constant head permeability test
   - B-: falling head permeability test
2) Field methods
   - A-: pumping out tests
   - B-: pumping in tests
3) Indirect methods
   - A-: computation from the particle size
   - B-: computation from consolidation test.

A) 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


Water supply
Over flow
Constant head reservoir
Air release valve
h
h'
Porous disc
TheConstructor.Org
Sample
L'
L
Porous disc
Constant head chamber
Graduated jar

Equipments :

- Permeability mould, internal diameter = 100mm, effective height =127.3, capacity = 1000 ml,
- Constant head tank.
- Graduated cylinder, stop water, thermo meter.
- Filter paper, vacuum pump.
- Weighing balance, 0.1 gm accuracy.

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

The total head loss (h) 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.

Using a graduated flask, we can collect a volume of water (V) in a period of time (t). From this we can calculate the flow rate

(Q= V/t).

- Apply Darcy's law:

$$Q = Av$$

$$V/t = Akh/L$$

$$\mathbf{k = VL/Aht}$$

where

V = volume of water collected in time = t

h = constant head difference

A = x-sectional area of soil specimen

L = length of soil specimen

B) FALLING HEAD PERMEABILITY TEST

- *All the equipment required for the constant head permeability test.*
- *In addition A stand pipe*
- It is recommended for fine grained soil

Procedure :

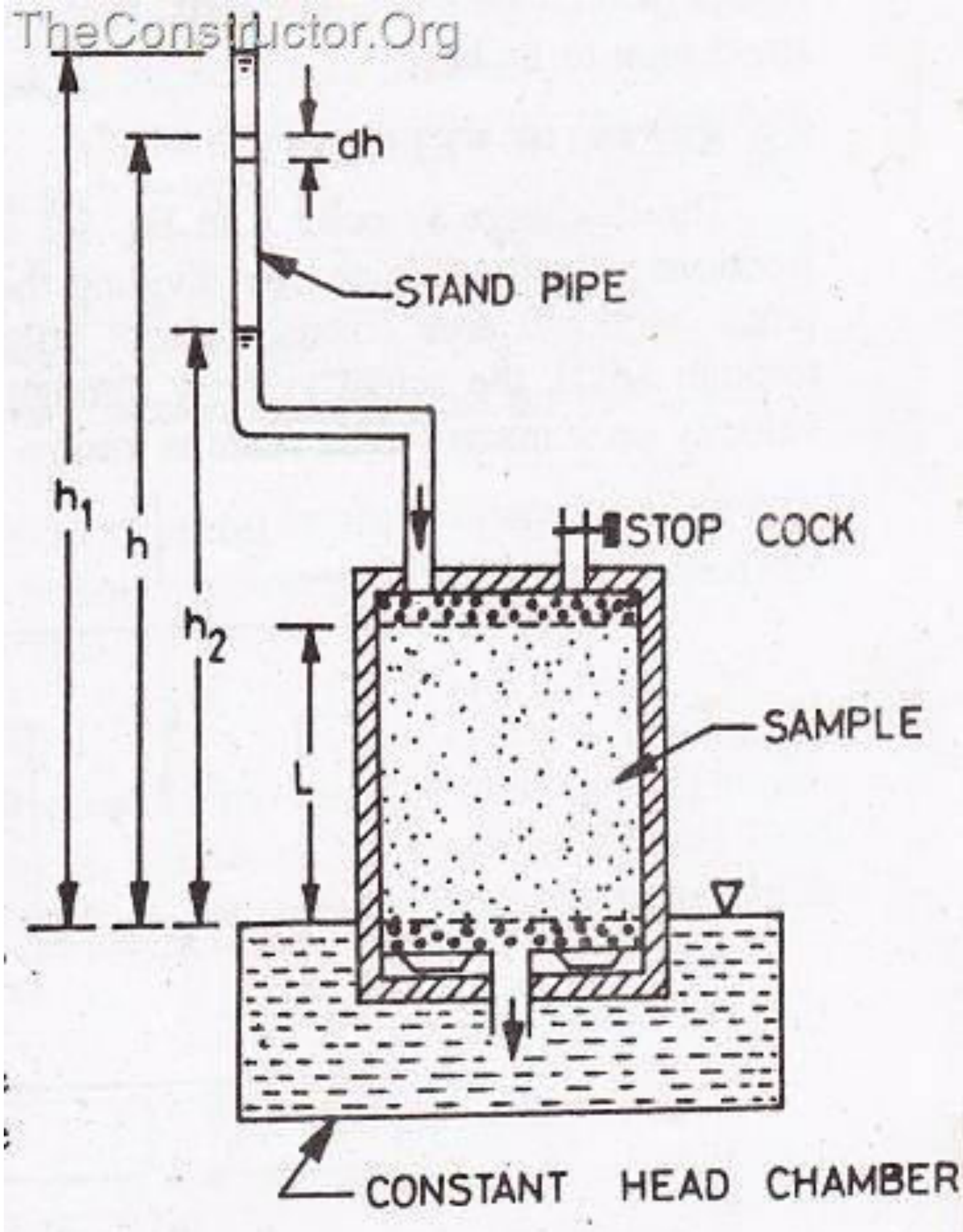
- For a falling head test arrangement the specimen(soil sample) and stand pipe shall be connected through the top inlet..
- The time interval required for the water level to fall from a known initial headh1 to a known final head h2 as measured above the centre of the outlet shall be recorded

The stand-pipe shall be refilled with water and the test repeated till three successive observations give nearly same time interval.

RECORD OF OBSERVATION:

The dimensions of specimen, length L and diameter D, are measured. Area "a" of stand-pipe is recorded.

During the test, observations are made of initial time t_1, final time t_2 initial head h_1 final head h_2 in stand-pipe and are recorded.



TheConstructor.Org
dh
STAND PIPE
STOP COCK
h_1
h
h_2
L
SAMPLE
CONSTANT HEAD CHAMBER

Derivation of Equation

h_1 = head of water in stand pipe at time t_1

h_2 = head of water at time t_2

h = head at any time t

L = Length of specimen

From Darcy's Law;

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

$$= k\left(\frac{h}{L}\right)A \quad \text{——(1)}$$

If Level Drop in stand pipe is dh in time dt;

Then Discharge rate $q = a\left[\frac{-dh}{dt}\right]$ —(2)

Equate (1) & (2)

$$a\left[\frac{-dh}{dt}\right] = k\,\frac{h}{L}\,.\,A$$

$$\frac{-dh}{h} = \frac{kA}{La} \cdot dt$$

on integration

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

$$\int_{h_2}^{h_1} \frac{dh}{h} = \frac{kA}{La} \int_{t_1}^{t_2} dt.$$

$$(t_2 - t_1) = t$$

$$\Rightarrow \quad \text{Log}_e \left[h\right]_{h_2}^{h_1} = \frac{kA}{La} \left[t\right]_{t_1}^{t_2}$$

$$\Rightarrow \quad \text{Loge} \left[h_1 - h_2\right] = \frac{kA}{La} \left[t_2 - t_1\right]$$

$$\Rightarrow \quad k = \frac{aL}{At} \text{Log}_e\left(\frac{h_1}{h_2}\right).$$

Where, a= c/s area of stand pipe (cm2)

- A= c/s area of soil sample
- L= length of soil sample (cm)
- t= time interval to fall head from h1 to h2
- h1= initial head (cm)
- h2= final head (cm)

Seepage Velocity

$$V_s = \frac{q}{A_v} = \frac{A \cdot v}{A_v}$$

$$\therefore V_s = \frac{1}{n} \cdot v = \frac{v}{n}$$

$$\therefore \boxed{V_s = \frac{v}{n}}$$

$$\frac{V_v}{V} = n$$

$$\frac{V}{V_v} = \frac{1}{n}$$

$$\text{IIIly } \frac{A}{A_v} = \frac{1}{n}$$

A constant head permeability test was. carried out on a Sandy soil sample of 160 mm in length and 6000 mm^2 in cross sectional Area. the porosity was 40%. under a constant head of 300 mm, a discharge was found to be $45 \times 10^3 \, mm^3$ in 18 sec. Calculate the coefficient of permeability and evaluate the discharge velocity and seepage velocity. Also Determine the permeability of soil When the porosity is 30%.

$$K = \frac{QL}{AHt} = \frac{45 \times 10^3 \times 160}{6000 \times 300 \times 18} = 0.222 \text{ mm/sec}$$

$$\text{Discharge velocity} = V = \frac{q}{A} = \frac{45 \times 10^3}{18 \times 6000} = 0.4167 \text{ mm/sec}$$

$$\text{seepage velocity} = V_s = \frac{V}{n} = \frac{0.4167}{0.4} = \underline{1.0417} \text{ mm/sec}$$

@ $n = 40\%$.

$$V_s = 1.0417 = \frac{v}{n} = \frac{\cancel{0.4167}\ v}{0.3} \quad \therefore \quad V = 1.0417 \times 0.3 =$$

In a falling head permeameter test, the sample was 18 cm long and 22 cm^2 cross section Area. The Area of stand pipe is 2 cm^2. calculate the time required for the head to drop from 25 cm to 10 cm. The sample is heterogeneous having coefficients of permeability of 3×10^{-4} cm/sec for the first 6 cm tk, 4×10^{-4} cm/s for the second 6 cm tk and 6×10^{-4} cm/s for the last 6 cm tk. Assume the flow is perpendicular to the bedding plane. and ∥el to the bedding. plane.

Case A :- Flow $\perp$er to bedding plane

$$K_{av} = \frac{Z}{Z_1/K_1 + Z_2/K_2 \cdots}$$

$$= \frac{18}{\frac{6}{3\times10^{-4}} + \frac{6}{4\times10^{-4}} + \frac{6}{6\times10^{-4}}}$$

$$= \frac{18}{\frac{6}{10^{-4}}\left[\frac{1}{3+4+6}\right]} = \frac{18\times10^{-4}}{6}$$

$$= \underline{39\times10^{-4}}$$

$$K_{aH} = \frac{k_1 z_1 + k_2 z_2 \cdots}{z}$$

$$= \frac{6 \times (3 \times 10^{-4}) + 6 \times (4 \times 10^{-4}) + 6 \times (6 \times 10^{-4})}{18}$$

$$= \frac{6}{18}\left[3+4+6\right] \times 10^{-4} = \frac{6}{18} \times 13 \times 10^{-4}$$

$$= \frac{13}{3} \times 10^{-4}$$

A cylintrical Mould of dia 7.5 cm Contains 8 cm long sample of Sand When water flows through the Soil under constant head at the rate of 55 cc/min. The Loss of head through the soil is 12.5 cm. Determine the Coefficient of permeability of Sample.

<u>Data given:</u>

$q = 55\,cc/min = 0.92\ cm^3/s.$

$i = \frac{h}{L} = \frac{12.5\ cm}{8\ cm} = 1.56$

$A = \frac{\pi d^2}{4} = \frac{\pi \times (7.5)^2}{4} = 44.18\ cm^2$

As Per Darcy's Law; $q = KiA$

$\therefore\ K = \frac{q}{iA} = \frac{0.92}{1.56 \times 44.18} = 0.033.\ cm$

A soil sample 5 cm in length and 60 cm in cross-sectional area, water percolates through the sample in 10 minutes is 480 ml under a constant head of 40 cm. Weight of oven dried sample is 498 gm and specific gravity of soil = 2.65.

Calculate:

(i) Coefficient of permeability

(ii) Seepage velocity.

Solution : Given :

$$Q = 480 \text{ ml}$$
$$L = 5 \text{ cm}$$
$$A = 60 \text{ cm}^2$$
$$h = 40 \text{ cm}$$
$$W_d = 498 \text{ gm}$$
$$G = 2.65$$
$$t = 10 \times 60 = 600 \text{ secs}$$
$$i = \frac{h}{L} = \frac{40}{5} = 8$$

(i) We know $$K = \frac{Q}{tiA} = \frac{480}{600 \times 8 \times 60} = 1.67 \times 10^{-3} \text{ cm/s} \quad \textbf{Ans.}$$

(ii) Discharge velocity, $$V = \frac{q}{A} = \frac{Q}{tA} = \frac{480}{600 \times 60} = 1.33 \times 10^{-2} \text{ cm/s}$$

Seepage velocity, $$V_s = \frac{V}{n} \qquad \text{...(i)}$$

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

and $$e = \frac{G\gamma_w}{\gamma_d} - 1 \qquad \text{...(ii)}$$

$$\gamma_d = \frac{W_d}{A \times L} = \frac{498}{60 \times 5} = 1.66 \text{ gm/c.c}$$

Putting the value of γ_d in (*ii*) we get

$$e = \frac{2.65 \times 1}{1.66} - 1 = 0.596$$

$\therefore$ $$n = \frac{0.596}{1 + 0.596} = 0.373$$

Putting the value of V and n in (*i*) we get

$$V_s = \frac{1.33 \times 10^{-2}}{0.373} = 3.56 \times 10^{-2} \text{ cm/s}$$ **Ans.**

If during a permeability test on a soil sample with falling head permeameter,

equal time intervals are noted for

drops of head from h_1 and h_2 and

again from h_2 to h_3,

find a relationship between h_1, h_2 and h_3.

$$K = 2.3\frac{aL}{At}\log_{10}\left(\frac{h_1}{h_2}\right) \qquad ...(i)$$

For falling head from h_2 to h_3

$$K = 2.3\frac{aL}{At}\log_{10}\left(\frac{h_2}{h_3}\right) \qquad ...(ii)$$

a, L, A and t are same for both the tests.

From (*i*) and (*ii*) we get

$$2.3\frac{aL}{At}\log_{10}\left(\frac{h_1}{h_2}\right) = 2.3\frac{aL}{At}\log_{10}\left(\frac{h_2}{h_3}\right)$$

$$\log_{10}\left(\frac{h_1}{h_2}\right) = \log_{10}\left(\frac{h_2}{h_3}\right)$$

$$\frac{h_1}{h_2} = \frac{h_2}{h_3}$$

$$h_2^2 = h_1h_3$$

$$\therefore \quad h_2 = \sqrt{h_1h_3} \quad \textbf{Ans.}$$

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Flow Net

Solving the mathematical equation

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial Z^2} = 0$$

Called Laplace equation of two dimensional which governs the flow of water in soils

It is very difficult to solve the above equation,
Hence a graphical flow grit
formed by intersection of
imaginary flow lines and
equipotential lines
to solve the above equation
called **flow net**

Assumptions Needed For Flow Net Construction

Aquifer is homogeneous,
isotropic
saturated
there is no change in head with time

soil and water are incompressible

flow is laminar, and Darcy's Law is valid

All boundary conditions are known.

Flow Net Properties

1. Streamlines Ψ and Equip. lines ϕ are $\perp$.
2. Streamlines Ψ are parallel to flow boundaries.
3. Grids are curvilinear squares, where diagonals cross at right angles.
4. Each stream tube carries the same flow.

Drawing Method:

1. Draw to a convenient scale of the following

The cross sections of the structure, water elevations, and aquifer profiles.

2. Establish boundary conditions and draw one or two flow lines Ψ and equipotential lines Φ near the boundaries.

Method cont…

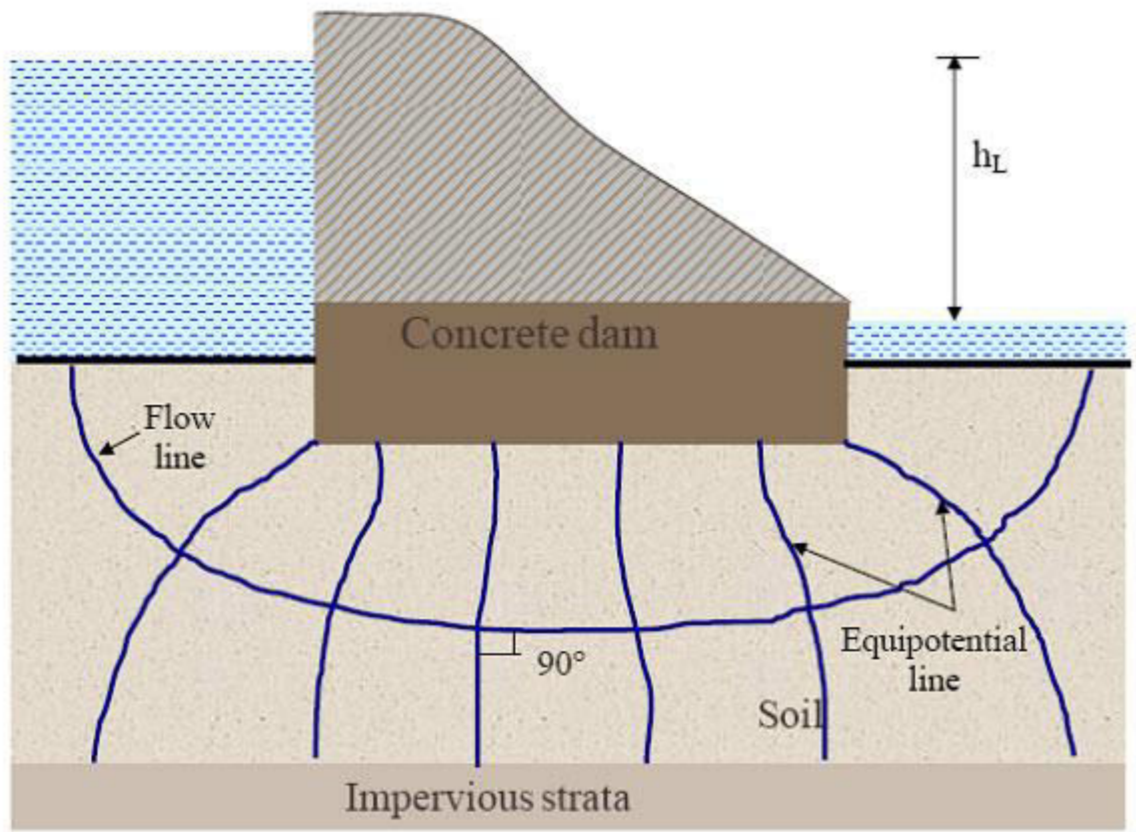
3. Sketch intermediate flow lines and equipotential lines by smooth curves
4. Adhering to right-angle intersections and square grids. Where flow direction is a straight line, flow lines are an equal distance apart and parallel.

5. Continue sketching until a problem develops.
6. Successive trials will result in a reasonably consistent flow net.

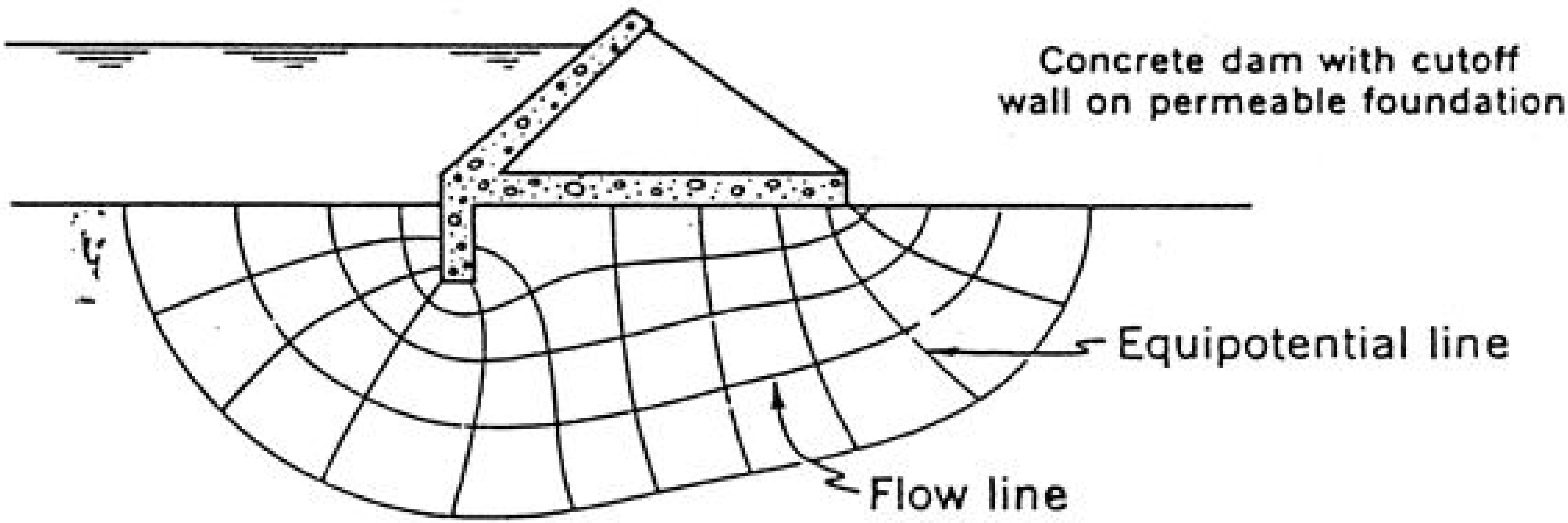
Method cont…

7. In most cases, 5 to 10 flow lines are usually sufficient.
8. Depending on the no. of flow lines selected,
   the number of equipotential lines will automatically be fixed by geometry and grid layout.

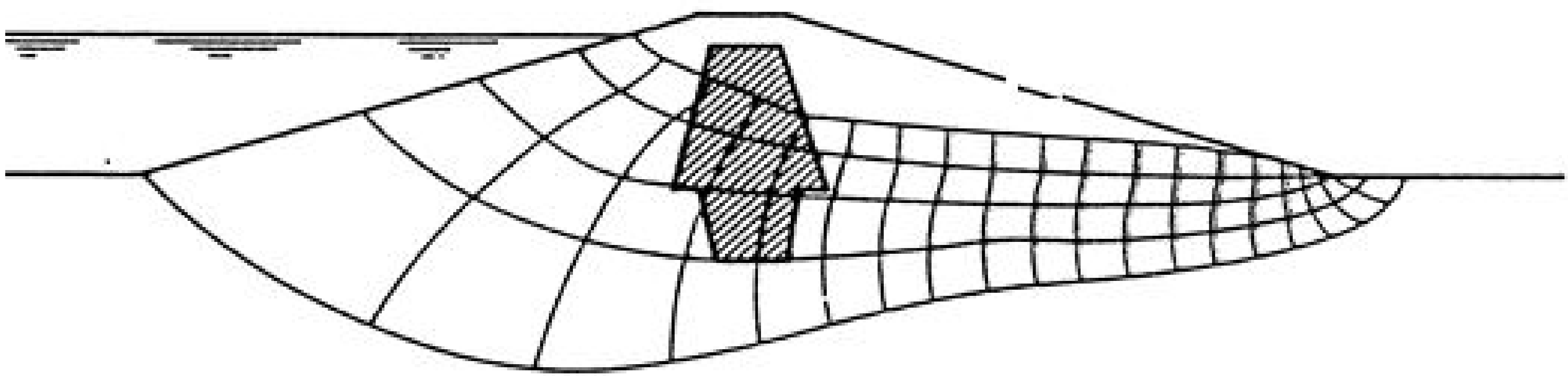
Typical Flow Net

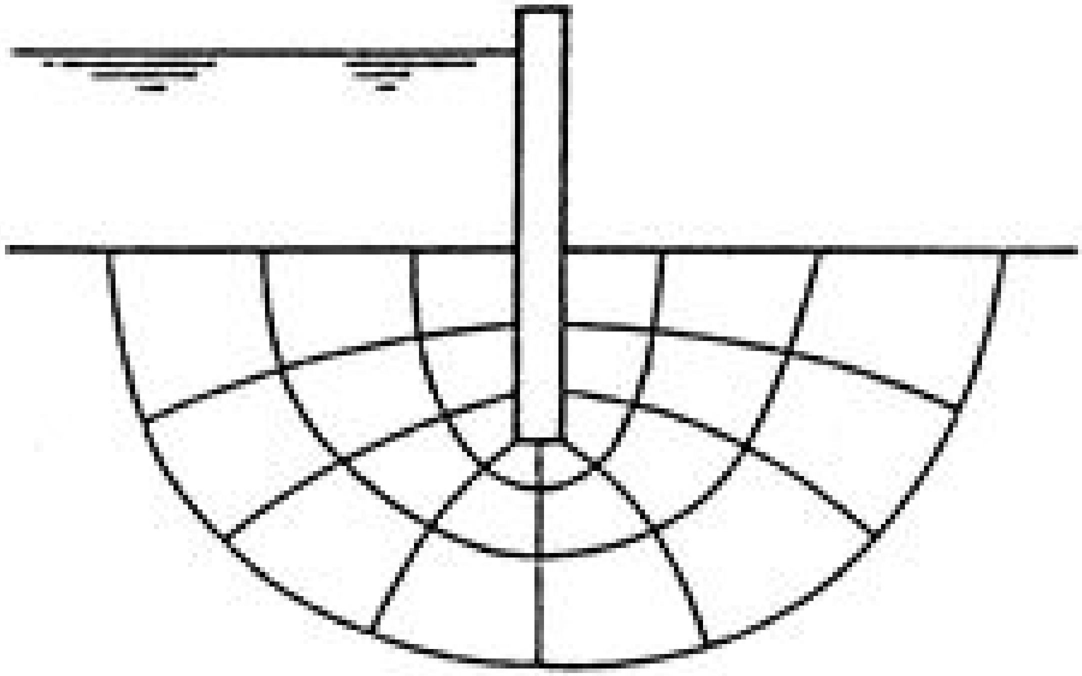


Typical Flow net example



Earth dam with relatively impermeable core on permeable foundation





Sheet piling

Determination of Total Discharge through a complete Flow Net

Total Discharge through a complete FlowNet → 'q' :

$q = \Delta q_1 + \Delta q_2 + \Delta q_3 + \cdots = \Sigma \Delta q$

Since $\Delta q_1 = \Delta q_2 = \Delta q$.

$\therefore q = \Sigma \Delta q = N_f \Delta q$ --5

$\Delta q = k \cdot \Delta h \cdot \frac{x}{l}$, $\frac{x}{l} = 1$, $\Delta h = \frac{H}{N_d}$ $\frac{\text{Total head loss}}{\text{No. of potential drop line}}$

$\therefore \Delta q = K \cdot \frac{H}{N_d}$ — (6)

Substitute eqn - (6) in (5)

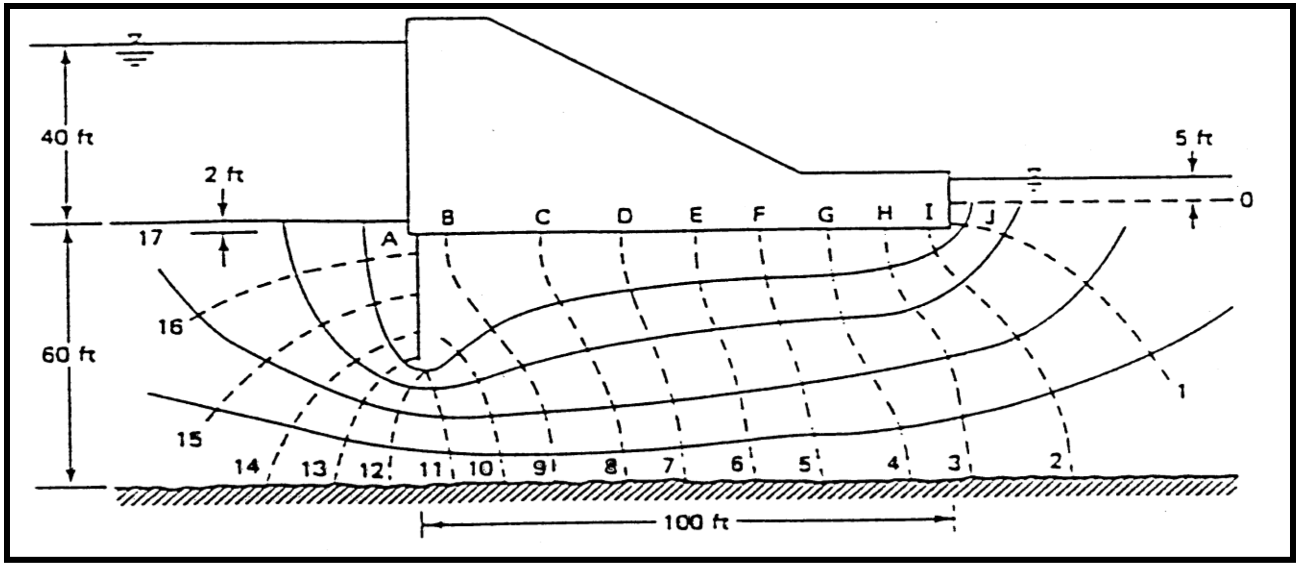
$q = N_f \left[K \frac{H}{N_d} \right] = \boxed{\frac{N_f}{N_d} K H}$

Flow Nets: an example

- A dam is constructed on a permeable stratum underlain by an impermeable rock. A row of sheet pile is installed at the upstream face. If the permeable soil has a hydraulic conductivity of 150 ft/day, determine the rate of flow or seepage under the dam.

Flow Nets: an example

The flow net is drawn with: m = 5   n = 17



Flow Nets: the solution

- Solve for the flow per unit width:

$$q = (m/n)\,K\,h$$

$$= (5/17)(150)(35)$$

$$= 1544 \text{ ft}^3\text{/day per ft}$$

The section through a dam in shown in Fig. 2.9. Determine the quantity of seepage under the dam and plot the distribution of uplift pressure on the base of the dam. The coefficient of permeability of the foundation soil is $2{\cdot}5 \times 10^{-5}$ m/s.

The flow net is shown in the figure. The downstream water level is selected as datum. Between the upstream and downstream equipotentials the total head loss is 4·00 m. In the flow net there are 4·7 flow channels and 15 equipotential drops. The seepage is given by:

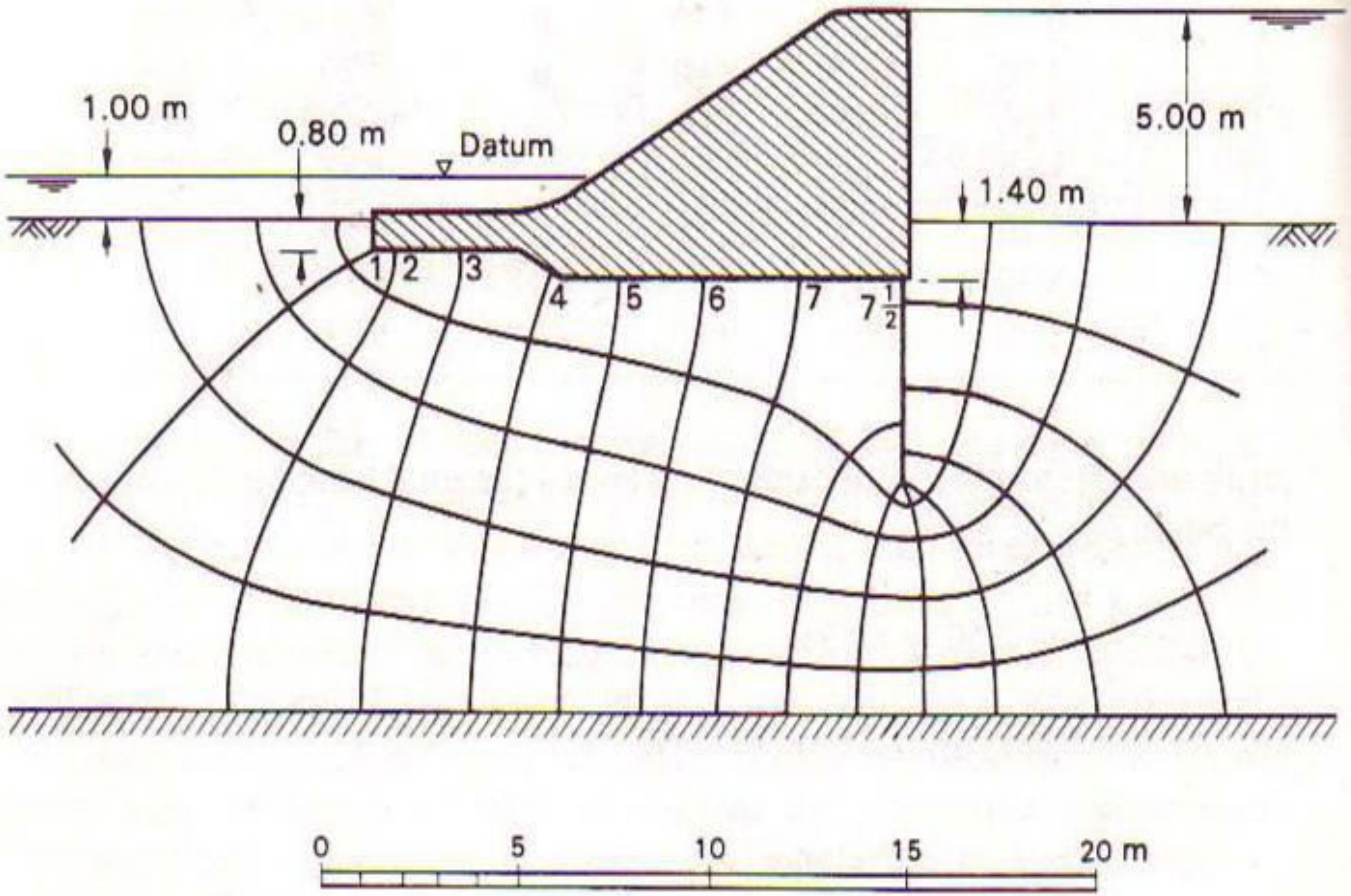


Fig. 2.9

$$q = kh\frac{N_f}{N_d} = 2{\cdot}5 \times 10^{-5} \times 4{\cdot}00 \times \frac{4{\cdot}7}{15}$$

$$= 3{\cdot}1 \times 10^{-5}\ \mathrm{m^3/s}\ \text{(per m)}$$

Quick Sand Condition

The vertical Effective pressure may be increase (or) decrease due to seepage pressure depending of the direction of flow.

<u>at submerged condition</u>;

$$\sigma' = Z\gamma' \pm P_s$$

Where $P_s = i Z \gamma_w$

$$\therefore \sigma' = Z\gamma' \pm i Z \gamma_w$$

When Flow upward ($\uparrow$) $\rightarrow$ $-$ve sign

Flow Downward ($\downarrow$) $\rightarrow$ $+$Ve sign

Whenever Water table rises due to heavy flooding, Flow takes place in an upward direction and therefore, effective pressure get reduced due to increase in seepage pressure (P_s).
at one stage when P_s = pressure due to submerged soil. Then $\sigma' = 0$
i.e effective stress become zero.

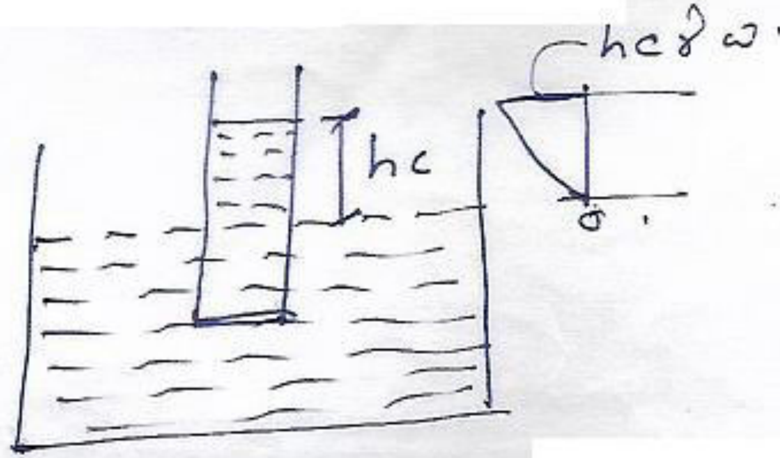
In this stage, cohesion less soils losses full of its shear strength because $S = \sigma' \tan\phi$ for ϕ-soils.
Due to that soil solid particles have a tendency to move up in the direction of Flow
This phenomenon of lifting of soil solid particles called Quick sand condition (or) Boiled condition

Suction Pressure or Capillary Pressure

The pressure deficiency (or) reduction (or) –ve pressure in the pore water (or) the pressure below atmospheric by which water retained in the soil mass in the Zone of aeration.

The Max value of suction pressure is equal to $h_c \gamma_w$ and it decreases linearly from Max value to Zero value at the free water surface

pF Value



The common Logaritham of this height of capillary rise 'hc' in cm (or) the pressure in g/cm^2 is known as pF value

Thus pF value $= Log_{10}(hc)$

① The pF value of a given Soil equal to 2. Determine the Height of Capillary rise

$$pF = \log_{10}(h_c)$$

$$\therefore h_c = \log_{10}^{-1}(pF)$$

$$= \log_{10}^{-1}[2] = \underline{100\,cm}$$

② $P_F = 1.92$, $h_c = ?$ Ans:- 83.18 cm

Capillary Potential = ? Ans: 8.16 kN/m²

Hint → Capillary potential = $P_s = h_c \gamma_w$

Critical Hydraulic Gradient

Critical Hydraulic gradient = $i_c = \frac{G-1}{1+e}$

Problem

Given $G = 2.7$ $i_c = 1$, $n = ?$

Soln

$i_c = \frac{G-1}{1+e} \Rightarrow 1.0 = \frac{2.7-1}{1+e} \Rightarrow e = (2.7-1)-1 = \underline{0.7}$

We know n = porosity = $\frac{e}{1+e} = \frac{0.7}{1+0.7} = \underline{0.412}$

Effective stress due to self weight of soil

Effective Stress Equation.

The effective stress σ'

may be calculated by
a simple subtraction of the pore pressure from the total stress:

where σ is the total stress and u is the pore pressure.

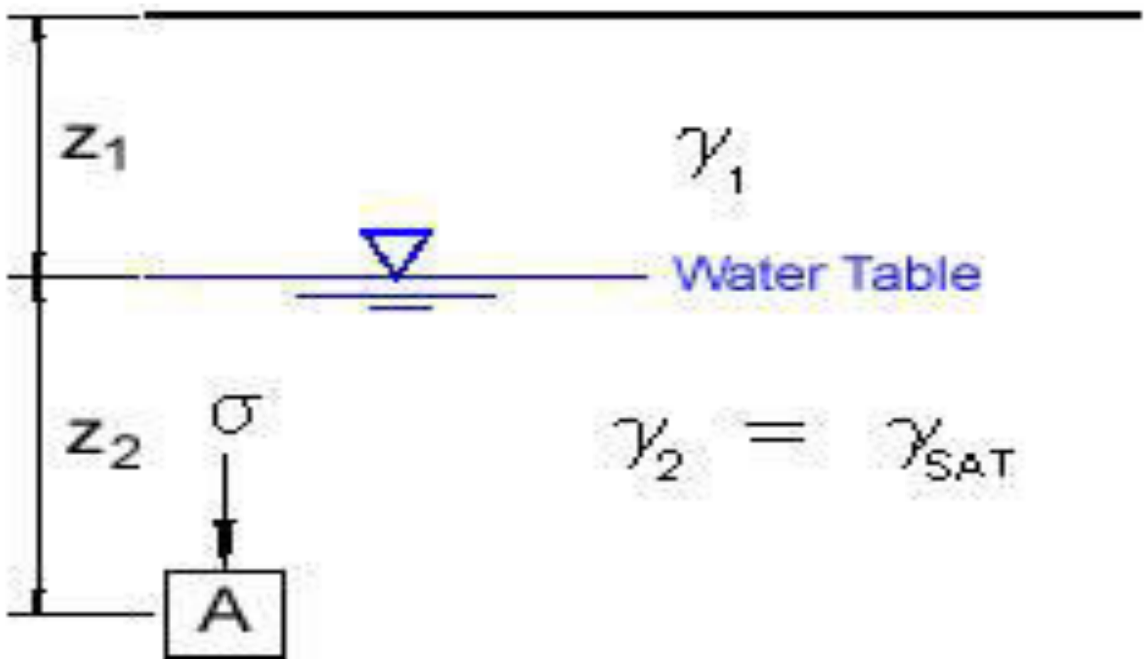
$$\sigma = \sigma' + u$$

σ = Total Stress

u = Neutral Stress

σ' = Effective Stress

Effective stress calculation



$$\sigma = \text{Total Stress} = z_1\gamma_1 + z_2\gamma_2$$

$$u = \text{Neutral Stress} = z_2\gamma_w$$

$$\sigma' = \text{Effective Stress} = \sigma - u$$

$$\sigma' = z_1\gamma_1 + z_2\gamma_2 - z_2\gamma_w$$

$$\sigma' = z_1\gamma_1 + z_2(\gamma_2 - \gamma_w)$$

Typical Stress Profile

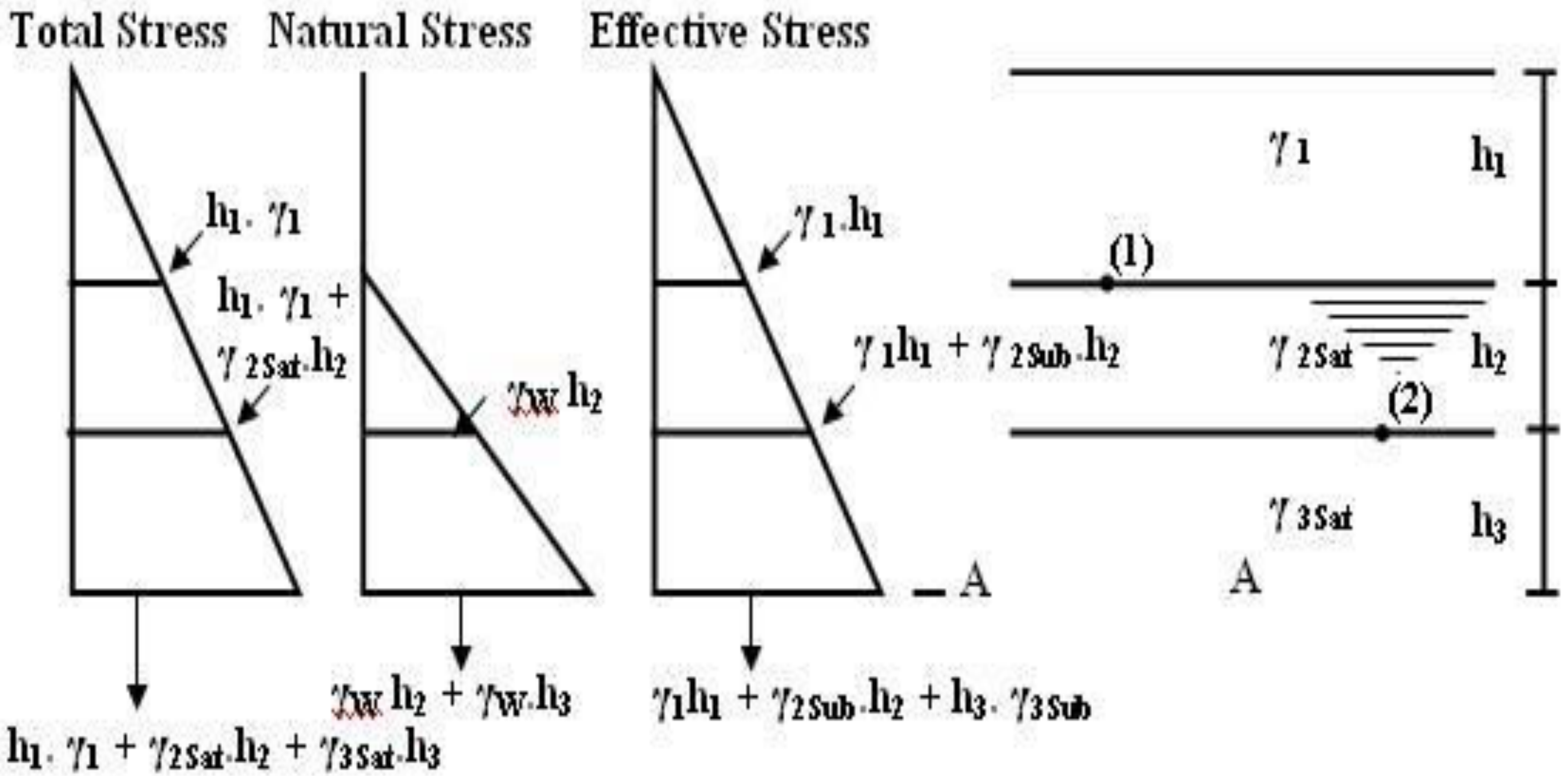
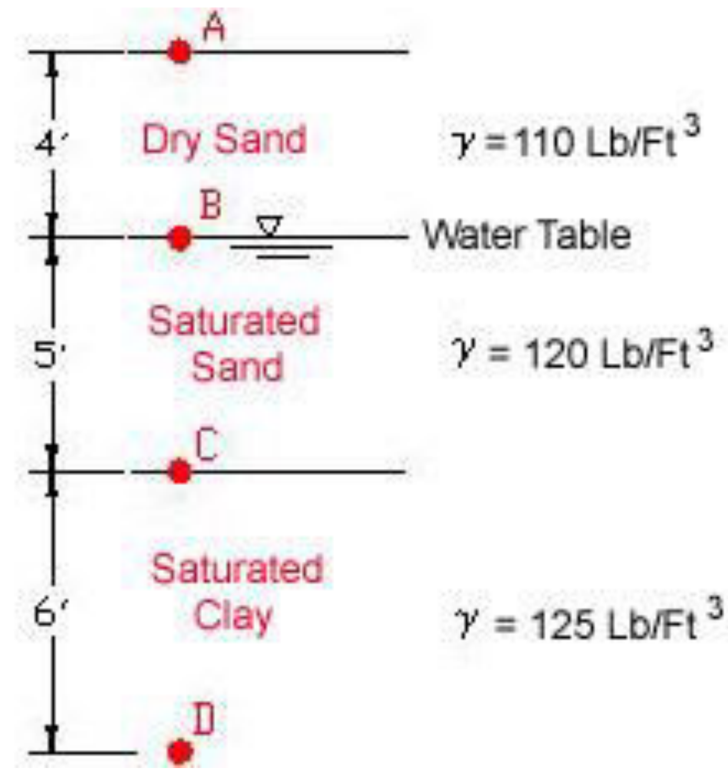


Fig. (6): Stress Profile

Determine the stresses at points A, B, C, and D in the soil profile



Unit weight of water : 62.4 Lb/Ft^3

Point	Total Stress	Neutral Stress	Effective Stress
A	0	0	0
B	440	0	440
C	1040	312	728
D	1790	686	1104

calculations

Point D

$$\sigma = 4(110) + 5(120) + 6(125) = 1790\,\frac{lb}{ft^2}$$

$$u = 11(62.4) = 686\,\frac{lb}{ft^2}$$

$$\sigma' = \sigma - u = 1104\,\frac{lb}{ft^2}$$

Point C

$$\sigma = 4(110) + 5(120) = 1040\,\frac{lb}{ft^2}$$

$$u = 5(62.4) = 312\,\frac{lb}{ft^2}$$

$$\sigma' = \sigma - u = 728\,\frac{lb}{ft^2}$$

Point B

$$\sigma = 4\,ft(110\,\frac{lb}{ft^3}) = 440\,\frac{lb}{ft^2}$$

$$u = 0$$

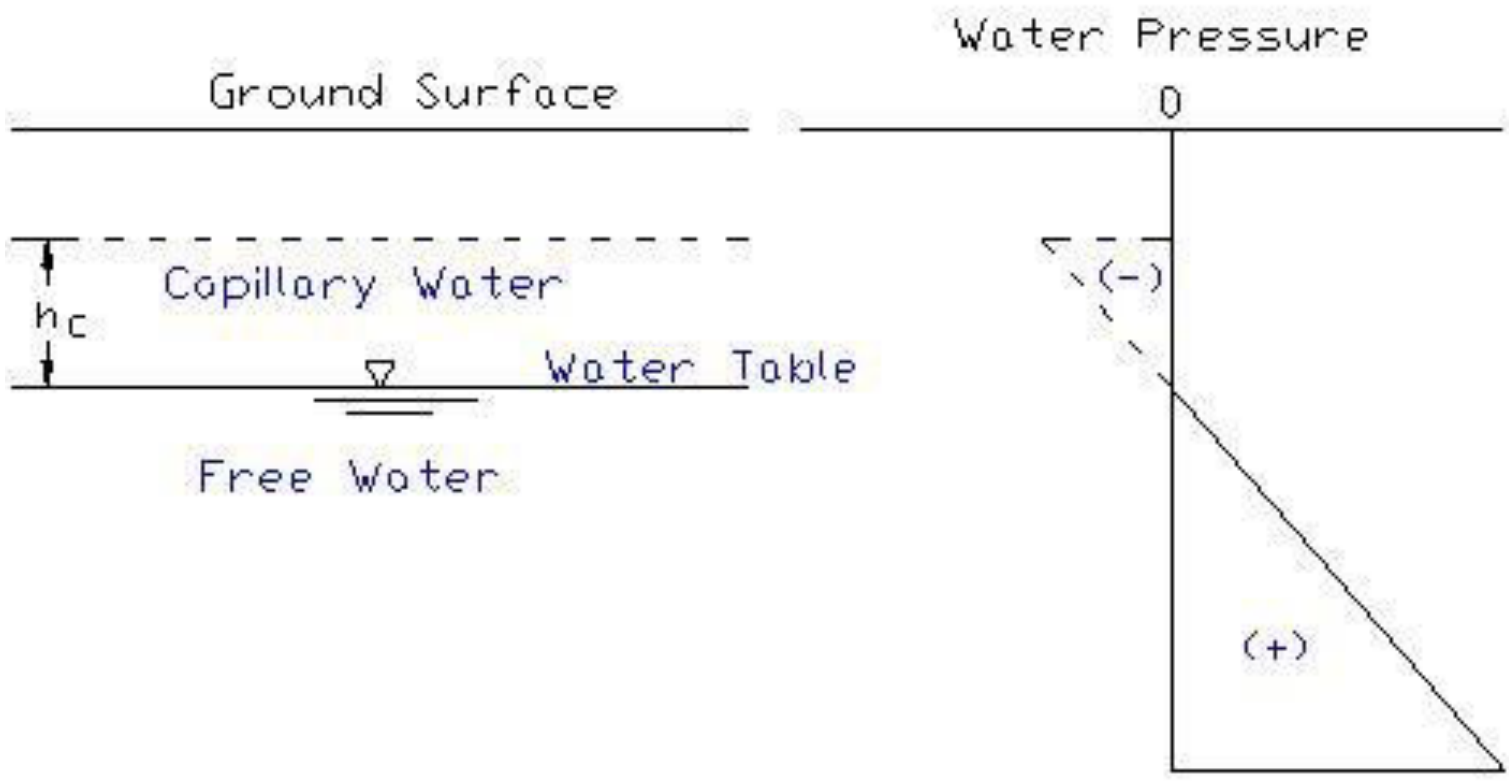
$$\sigma' = \sigma - u = 440\,\frac{lb}{ft^2}$$

Capillary Rice

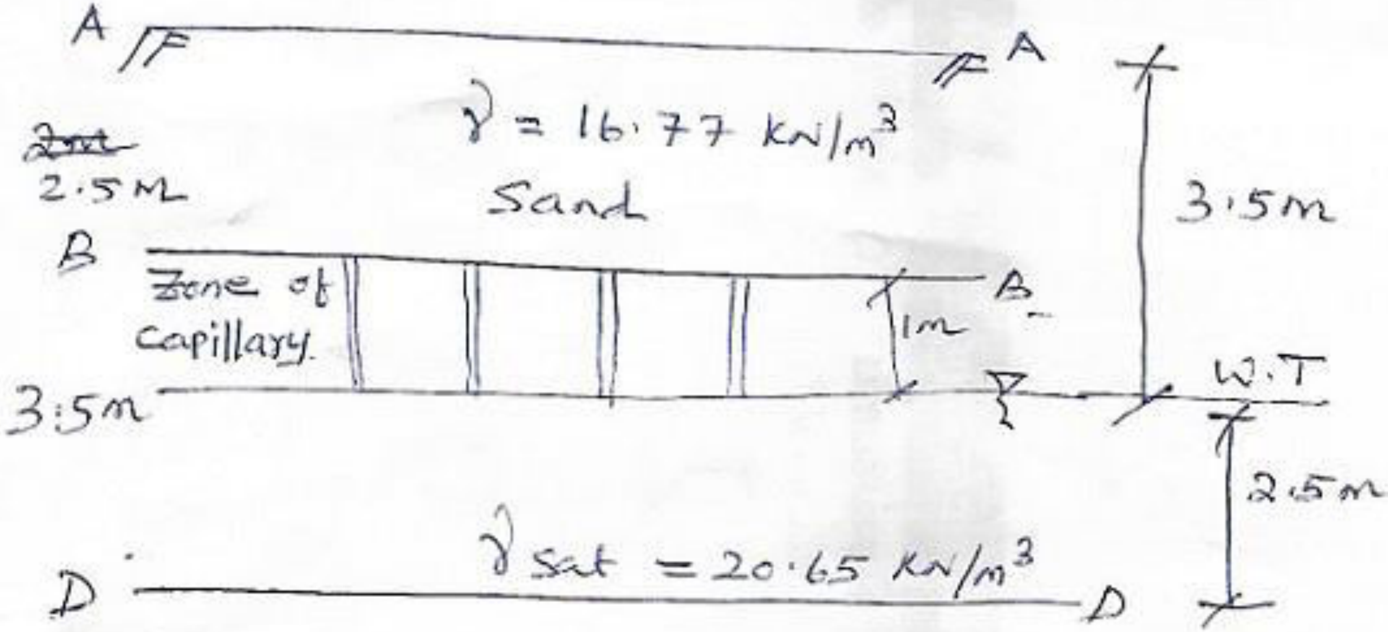
Above the water table up to the height of capillary rise, the Zone called capillary zone and the water is called capillary water.

The water pressure is negative (less than atmospheric) in the capillary zone.

These observations are illustrated in the figures below.



A sand deposit consists of two Layers.
The top layer is 2.5 m thick ($\gamma = 16.77\ kN/m^3$)
The bottom layer is 3.5 m thick ($\gamma_{sat} = 20.65\ kN/m^3$)
The water table is at a depth of 3.5 m
from the ground surface. The zone
of capillary is 1 m above water table.
Draw the diagram of Total stress,
neutral stress and effective stress variation



<u>stress calculations</u>

@ AA ; $\sigma = u = \bar{\sigma} = 0$

@ BB ; Top Layer.

$\sigma = 16.77 \times 2.5 = 41.93\ kN/m^2$

$u = 0, \quad \bar{\sigma} = 41.93\ kN/m^2$

@ BB ; Bottom layer.

$\sigma = 16.77 \times 2.5 = 41.93\ kN/m^2$

$u = -1 \times 9.81 = -9.81\ kN/m^2$

$\therefore \bar{\sigma} = 16.77 \times 2.5 - (-9.81) = 51.74\ kN/m^2$

@ CC — @ water table

$\sigma = 16.77 \times 2.5 + 20.65 \times 1.0 = 62.58\ kN/m^2$

$u = 0$

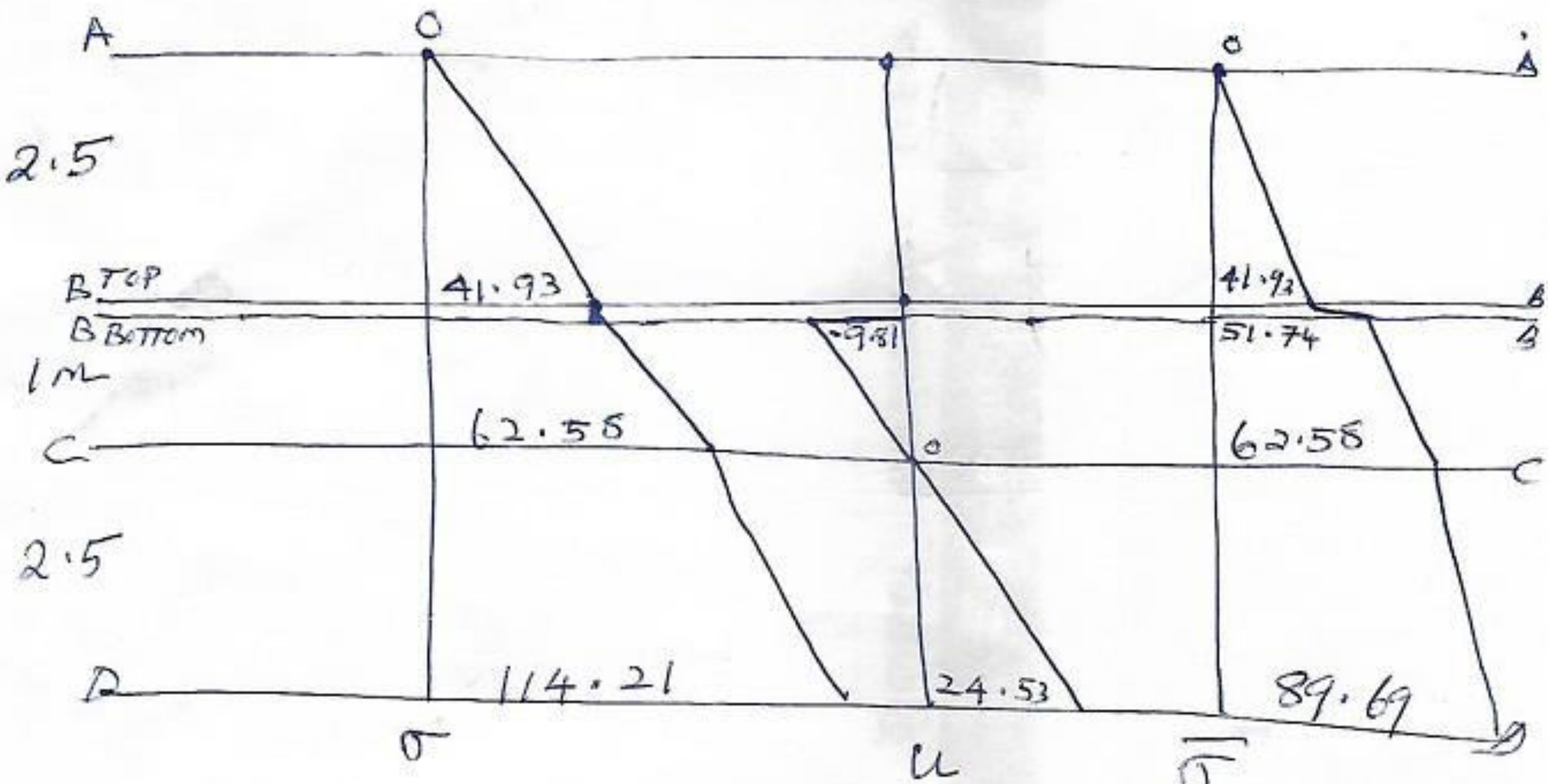
$\bar{\sigma} = 62.58\ kN/m^2$

@ DD - Bottom most

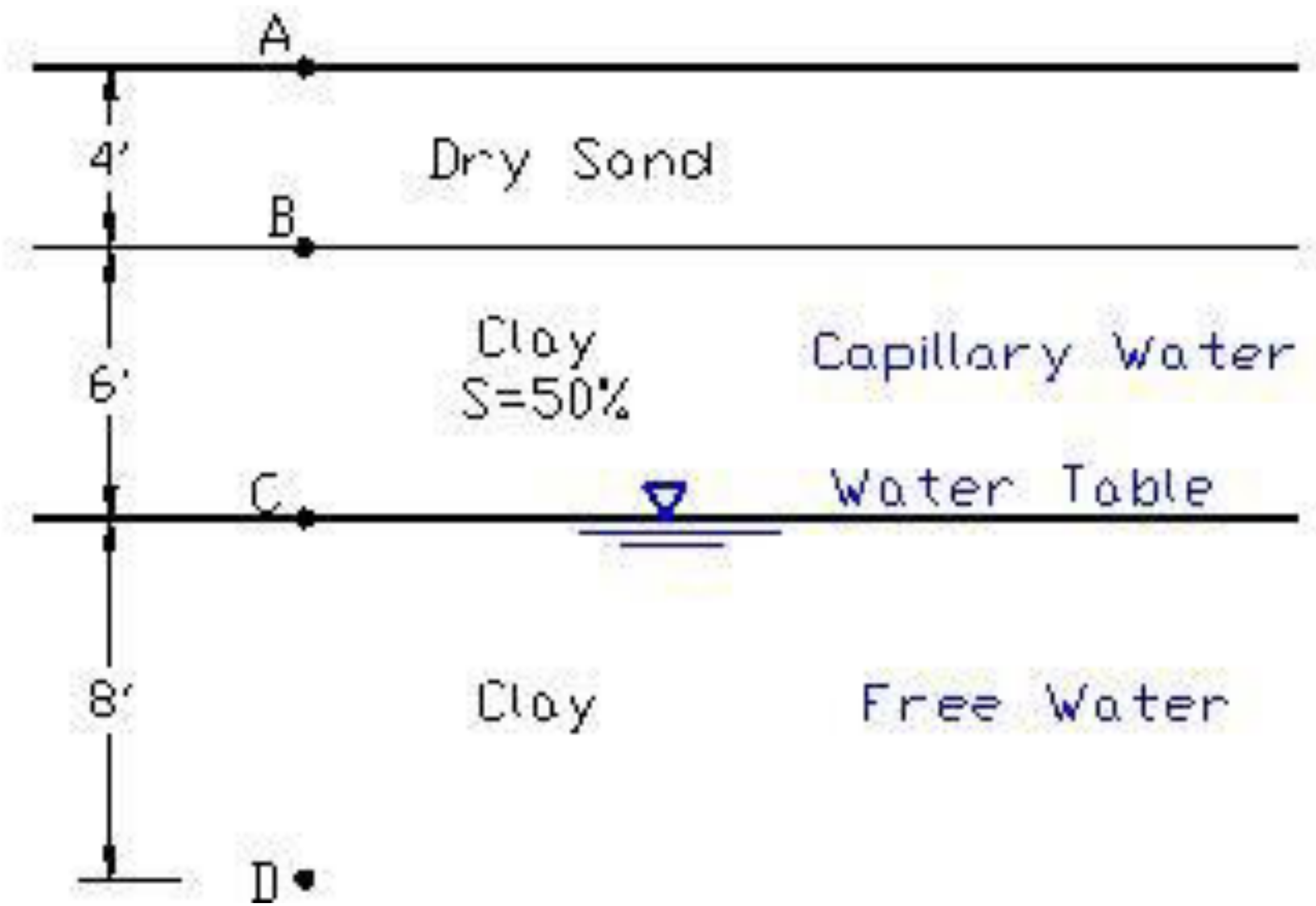
$\sigma = 16.77 \times 2.5 + 20.65 \times 3.5 = 114.21\ kN/m^2$

$u = 9.81 \times 2.5 = 24.53\ kN/m^2$

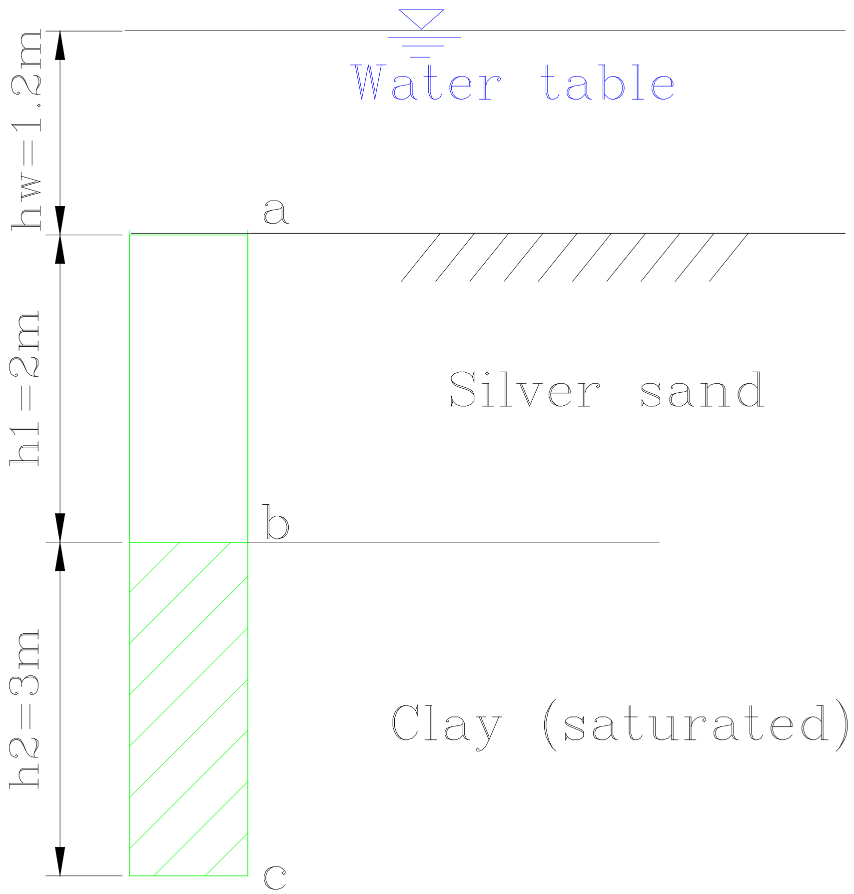
$\bar{\sigma} = \sigma - u = 114.21 - 24.53 = 89.69\ kN/m^2$


A
O
o
A
2·5
B TOP
41·93
41·93
B
B BOTTOM
9·81
51·74
B
1m
62·58
62·58
C
o
C
2·5
D
114·21
24·53
89·69
D
σ
u
σ̄

A soil profile is shown with capillary water and free water. The problem is to determine the pore water pressures at point A, B, C, and D.



The stratum's conditions and the related physical characteristics parameters of a foundation are shown in Fig below. Calculate the stress due to self-weight at a,b,c. Draw the stress distribution.



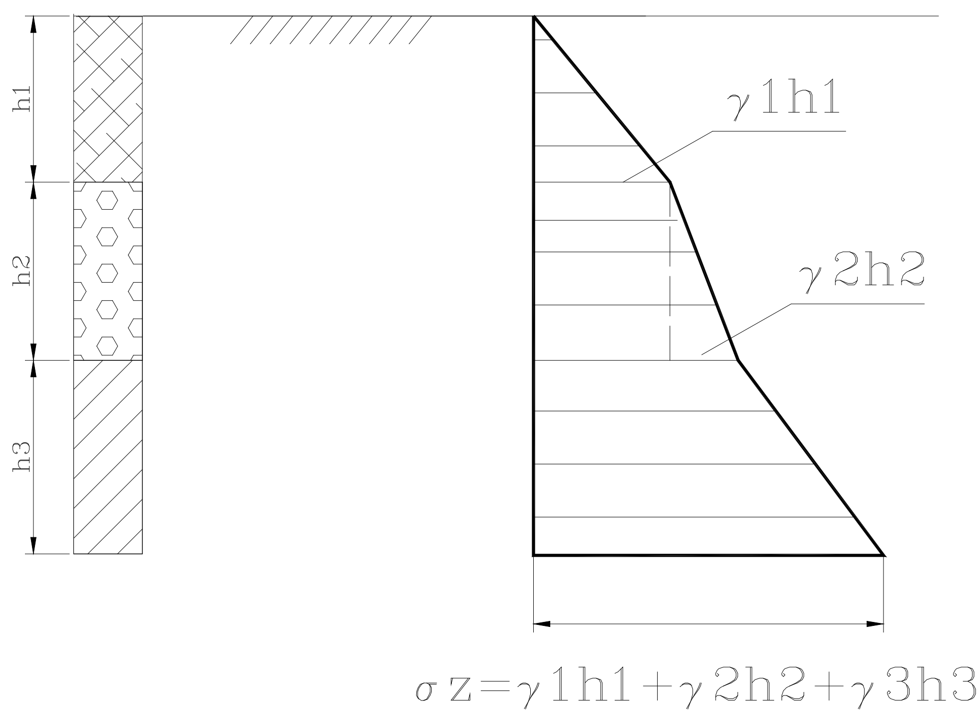
$w=15.6\%$
$e=0.57$

$\gamma_s=26.6kN/m^3$

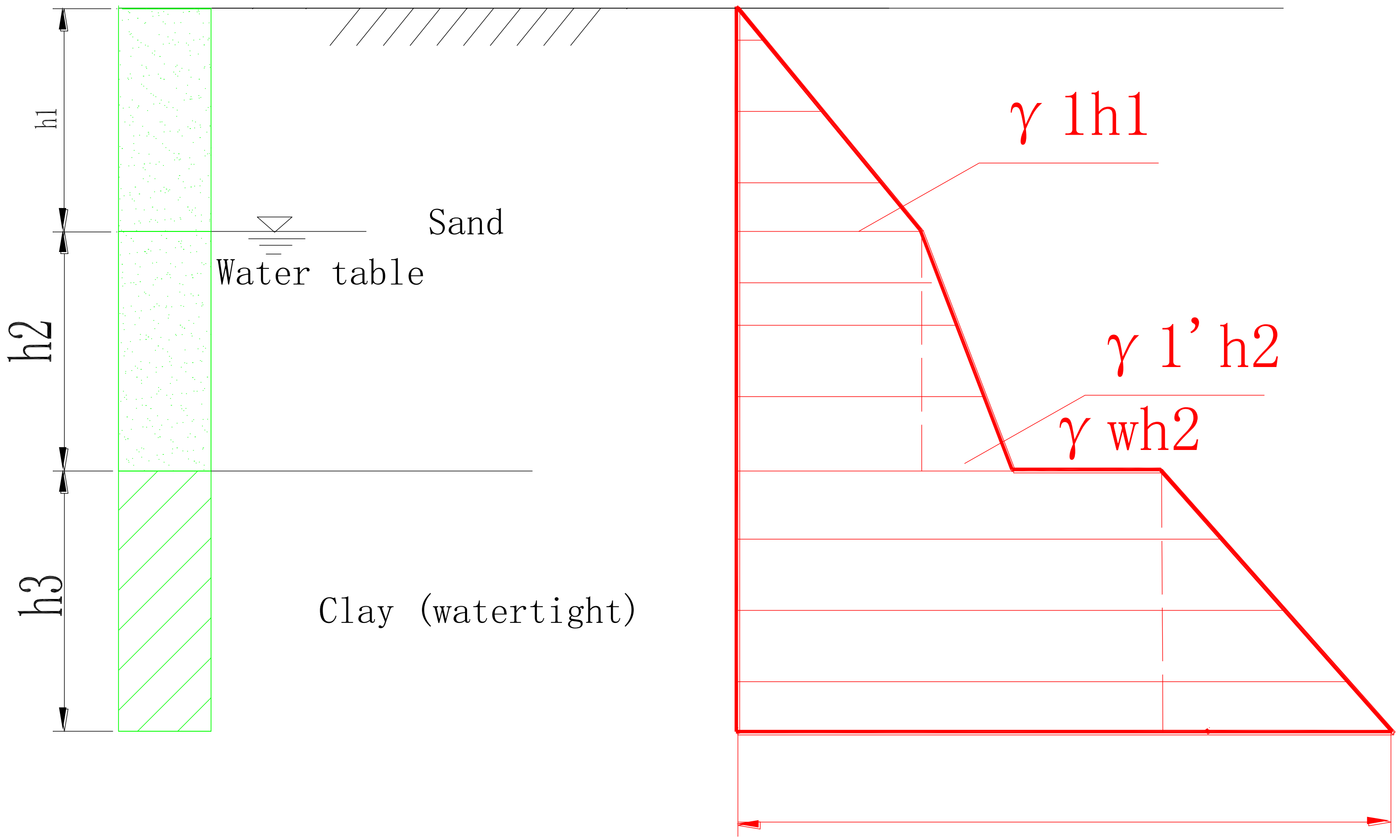
$w=22\%$ $w_L=32\%$
$w_p=23\%$

$\gamma_s=27.3kN/m^3$

Many Layers of soil, the vertical stress due to self-weight of soil is given as following.

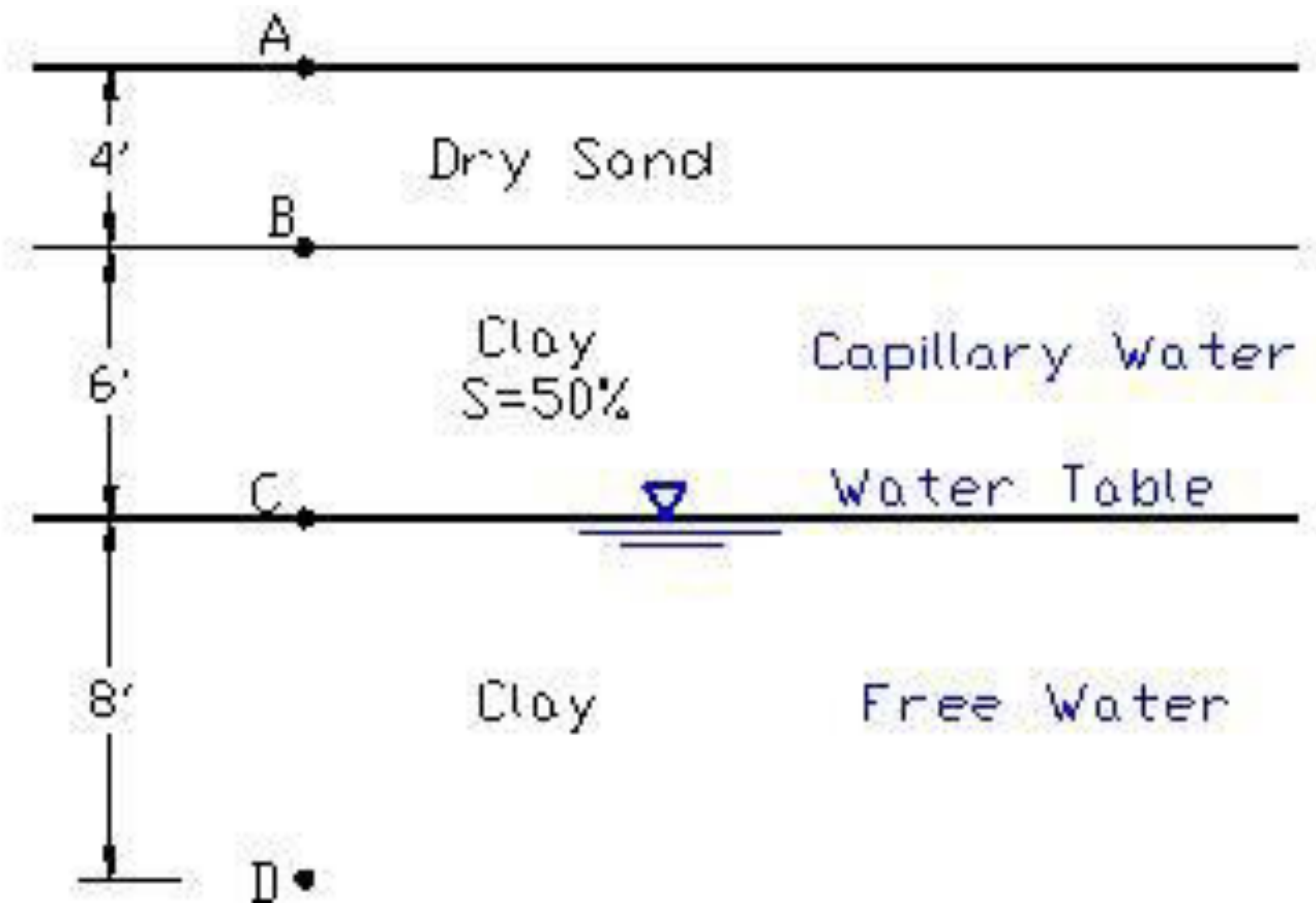


Point of Stress under water Table



$$\sigma z = \gamma 1h1 + \gamma 1' h2 + \gamma wh2 + \gamma sat3h3$$

A soil profile is shown with capillary water and free water. The problem is to determine the pore water pressures at point A, B, C, and D.



Point A – Dry

$$u_A = 0$$

Point B – Capillary Water

$$u_B = (-)\frac{50}{100}(62.4\,\frac{lb}{ft^3})(6\,ft) = -374\,\frac{lb}{ft^2}$$

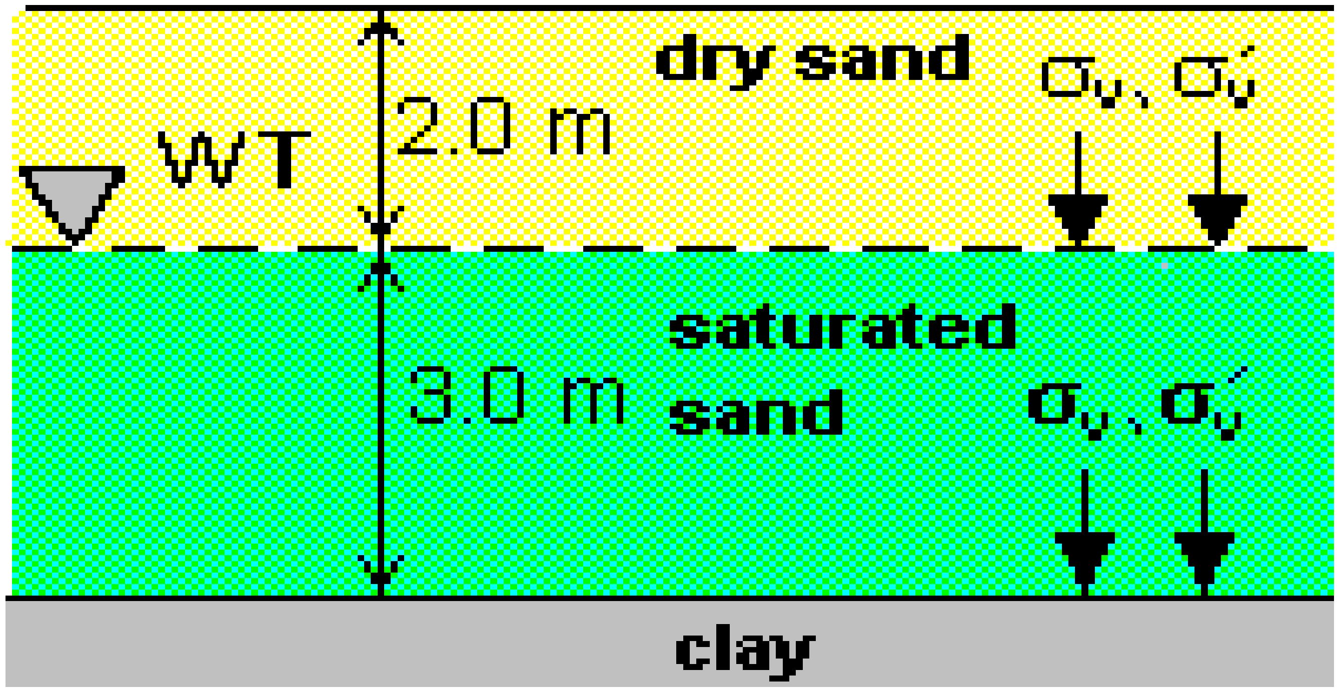
Point C – Water Table

$$u_C = 0$$

Point D – Free Water

$$u_D = 8\,ft(62.4\,\frac{lb}{ft^3}) = 499\,\frac{lb}{ft^2}$$

Calculating vertical stress



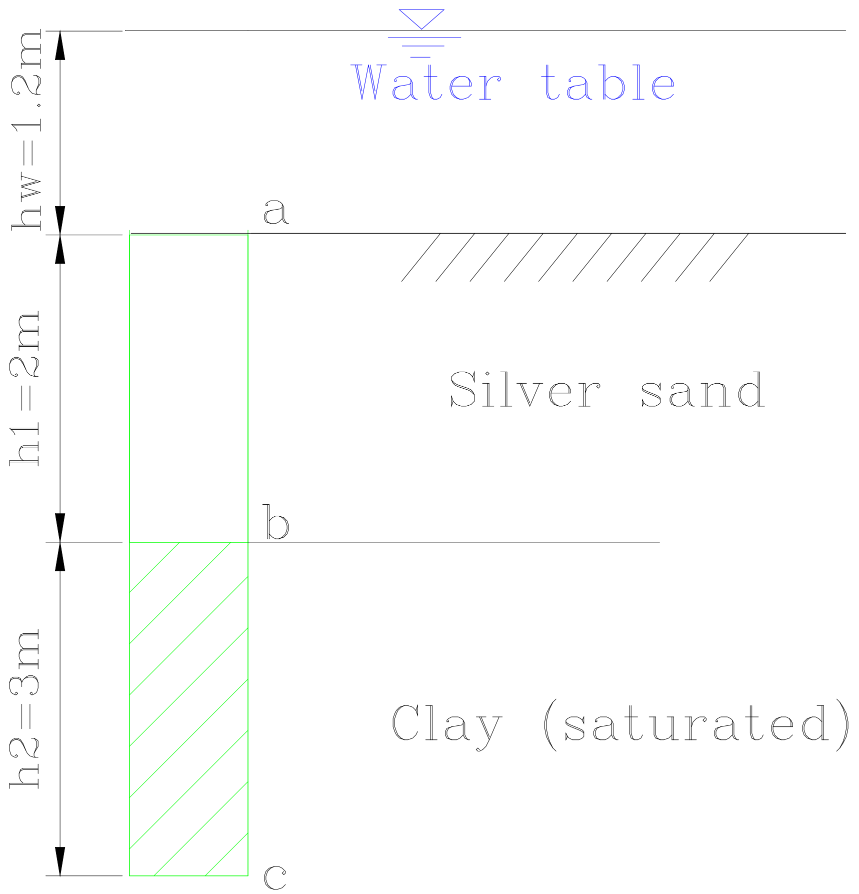
(a) At the top of saturated sand (z = 2.0 m)

Vertical total stress	$\sigma_v = 16.0 \times 2.0$	**= 32.0 kPa**
Pore pressure	$u = 0$	
Vertical effective stress	$\sigma'_v = \sigma_v - u$	**= 32.0 kPa**

(b) At the top of the clay (z = 5.0 m)

Vertical total stress	$\sigma_v = 32.0 + 20.0 \times 3.0$	**= 92.0 kPa**
Pore pressure	$u = 9.81 \times 3.0$	= 29.4 kPa
Vertical effective stress	$\sigma'_v = \sigma_v - u = 92.0 - 29.4$	**= 62.6 kPa**

The stratum's conditions and the related physical characteristics parameters of a foundation are shown in Fig below. Calculate the stress due to self-weight at a,b,c. Draw the stress distribution.



w=15.6%
e=0.57
γ_s=26.6kN/m^3

w=22% w_L=32%
w_p=23%
γ_s=27.3kN/m^3

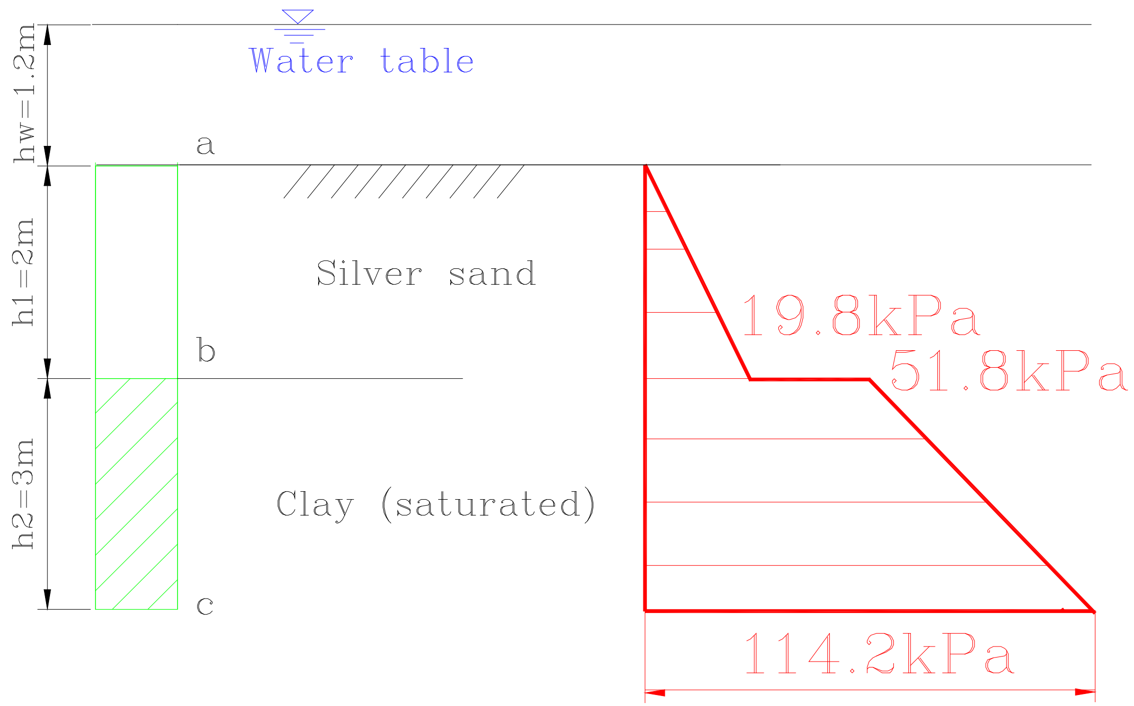
a $\sigma_z=0$

b $\sigma_{z(upper)}=\gamma'_1 h_1=9.9\times 2=19.8kPa$

$\sigma_{z(Down)}=\gamma'_1 h_1+\gamma_w(h_1+h_w)=9.9\times 2+10\times(2+1.2)=51.8kPa$

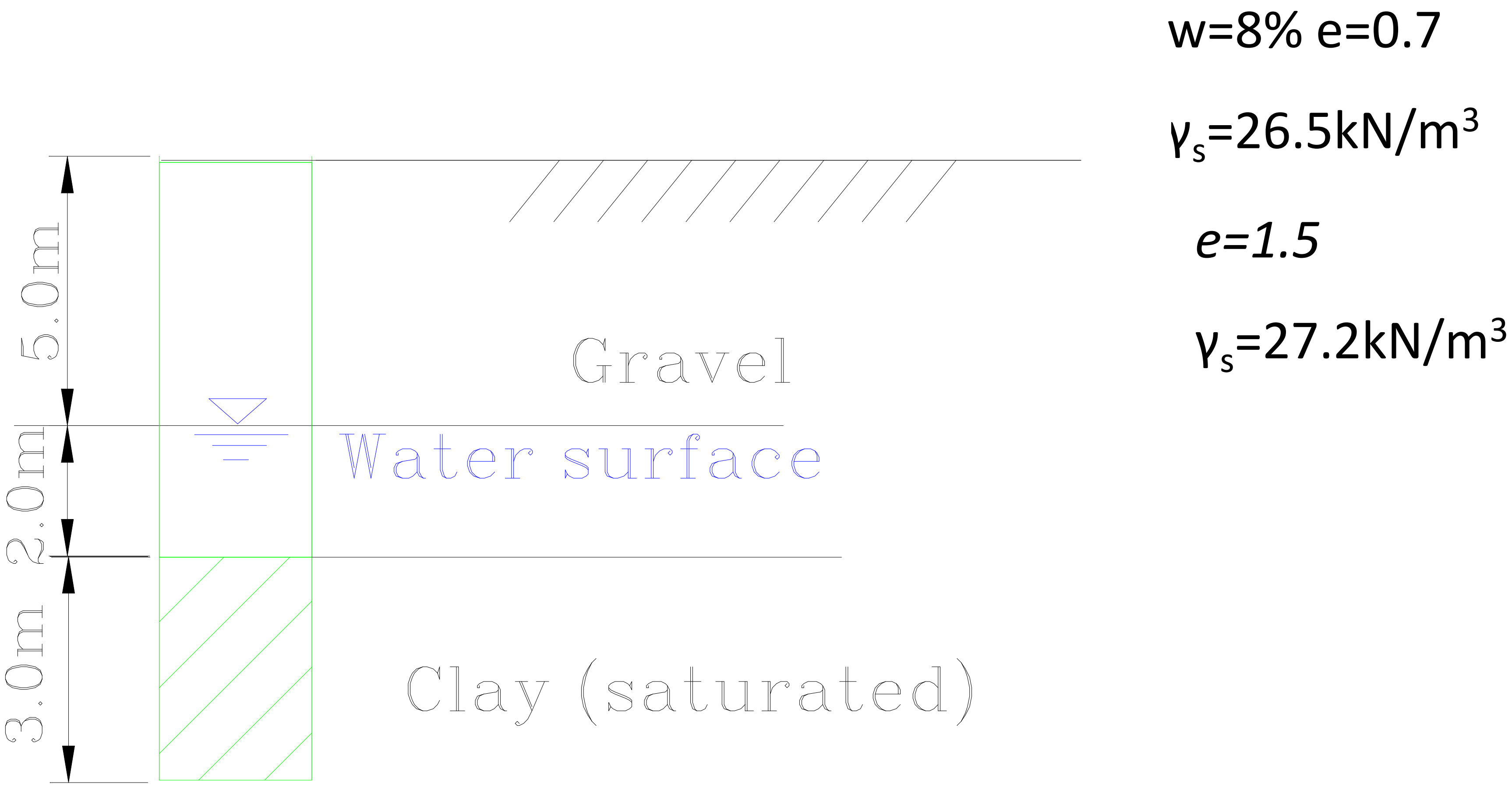
c $\sigma_z=\gamma'_1 h_1+\gamma_w(h_1+h_w)+\gamma_{sat2} h_2$

$= 9.9\times 2+10\times(2+1.2)+20.8\times 3=114.2kPa$



The stratum's conditions and the related physical characteristics parameters of a foundation are shown in Fig below. Calculate the stress due to self-weight at 10m depth. Draw the stress distribution.

Note: For saturated clay, both cases (watertight and non-watertight) need to consider.



2. Field permeability tests

1) Pumping-out tests:-

   A-: for unconfined aquifer

   B-: for confined aquifer

2) Pumping –in tests:-

   A-: open end tests

   B-: single packer tests

   C-: double packer tests

1) Pumping-out Tests

- For large engineering projects, it is the usual practice to measure the permeability of soils of entire aquifer by pumping out tests.
- This method is very useful for a homogeneous coarse
- The pumping out tests are very costly.

Definitions of Technical Terms related to permeability

1. Aquifer
2. Aquiclude
3. Aquitard
4. Aquifuge
5. Unconfined aquifer
6. Confined aquifer

Important terminologies related to Aquifer

Specific Capacity :-

It is a Measure about how much water can be withdrawn from the Well over a period of time.

Unit :- M^3/hr (or) M^3/day.

Static Water level

It is the Level of water in the Well When no water is being taken out.

Dynamic Water level

It is the Level of water during water being drawn from the well. Due to which a cone of depression occurs during Pumping when water flows from all directions towards the pump.

Aquiclude

A water-bearing strata that is in capable of transmitting water.

Aquifer

A water-bearing strata that is capable of transmitting significant quantities of water.

Aquitard

A water-bearing strata that is capable of transmitting very less quantities of water.

Draw Down

The amount of water level decline in a well due to pumping.

Confined Aquifer

An aquifer sanwitched between two Impermeable Layers.

Un confined Aquifer

An aquifer whose Top boundary is water table itself and bottom boundary is impermeable layer

Transmissivity (T)

The product of Aquifer thickness and Permeability $\Rightarrow$ $\underline{T = Kb}$

Well Efficiency

It is the ratio between theoretical drawdown and the actual drawdown Measured in the well expressed as:-

$$\text{Well Efficiency} = \frac{S_t}{S_a} \, \%$$

A well efficiency of 70% (or) More is usually acceptable.

Aquifuge

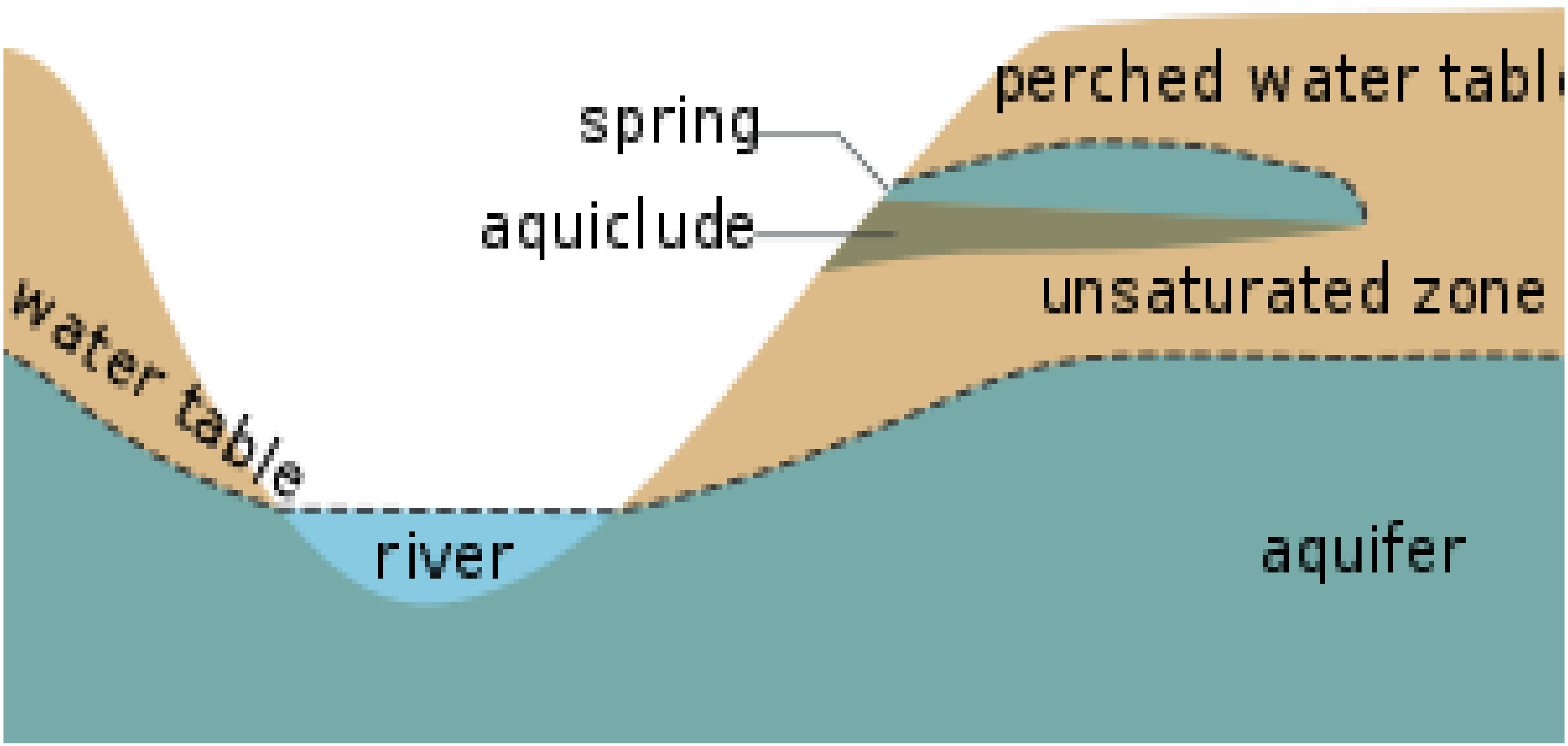
An impermeable body of rock which contains

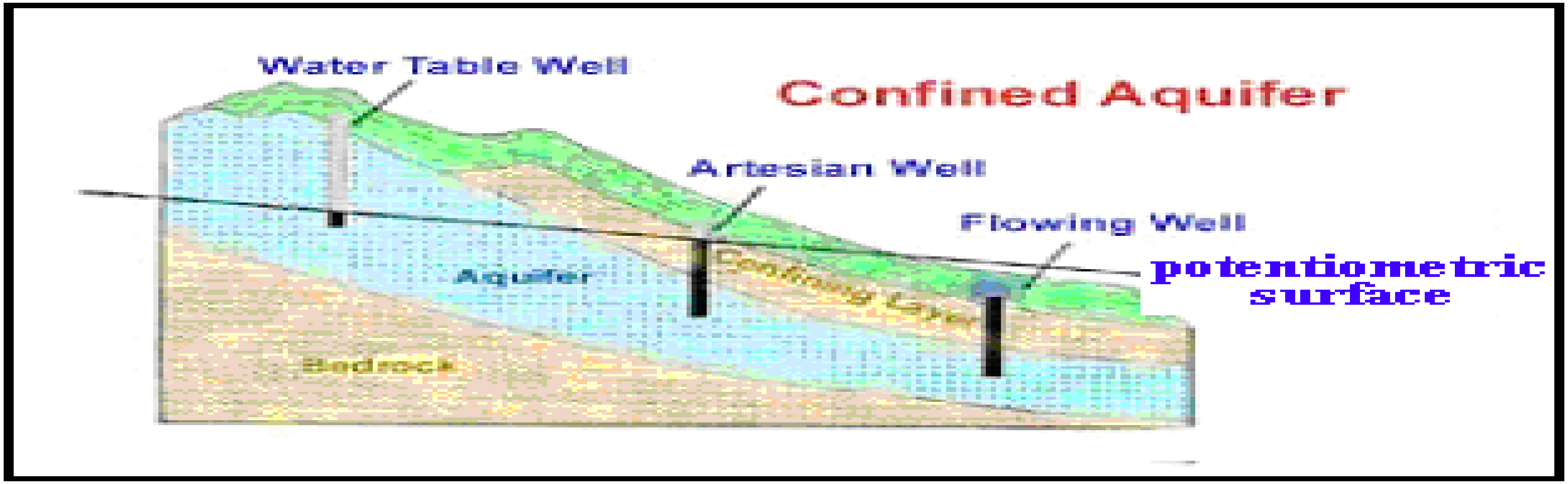
no interconnected pore spaces(interstices)

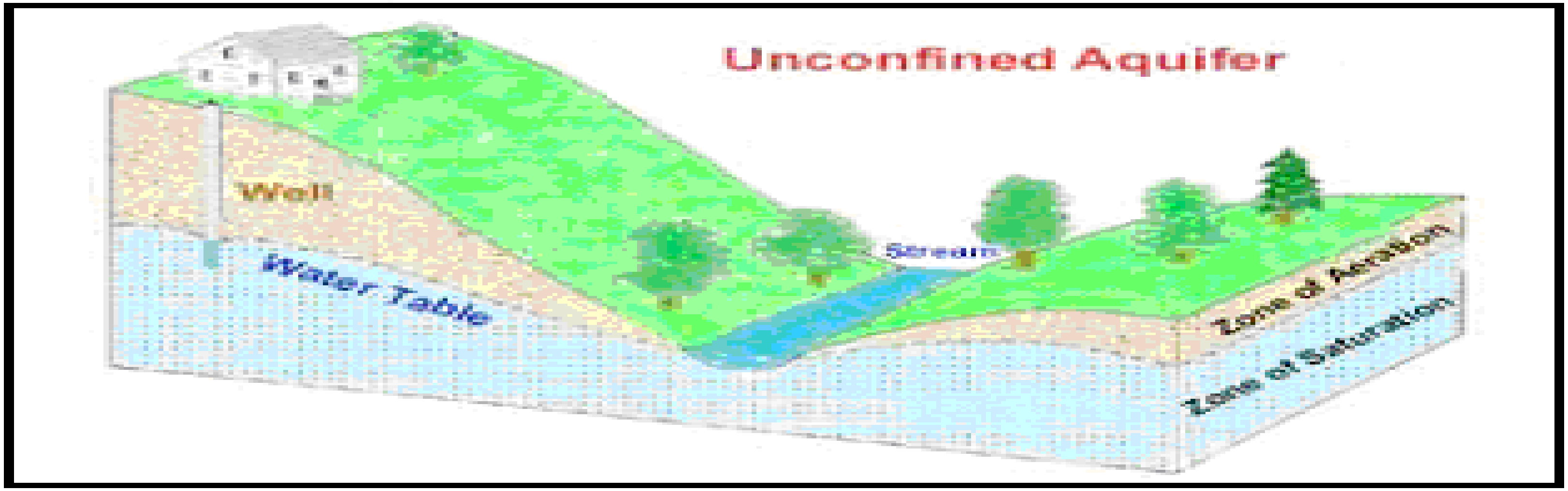
and therefore

neither absorbs nor transmits water.

Typical View of Perched Aquifer




Water Table Well
Confined Aquifer
Artesian Well
Flowing Well
potentiometric surface
Aquifer
Confining Layer
Bedrock


Unconfined Aquifer
Well
Water Table
Stream
Zone of Aeration
Zone of Saturation

2) Pumping –in tests

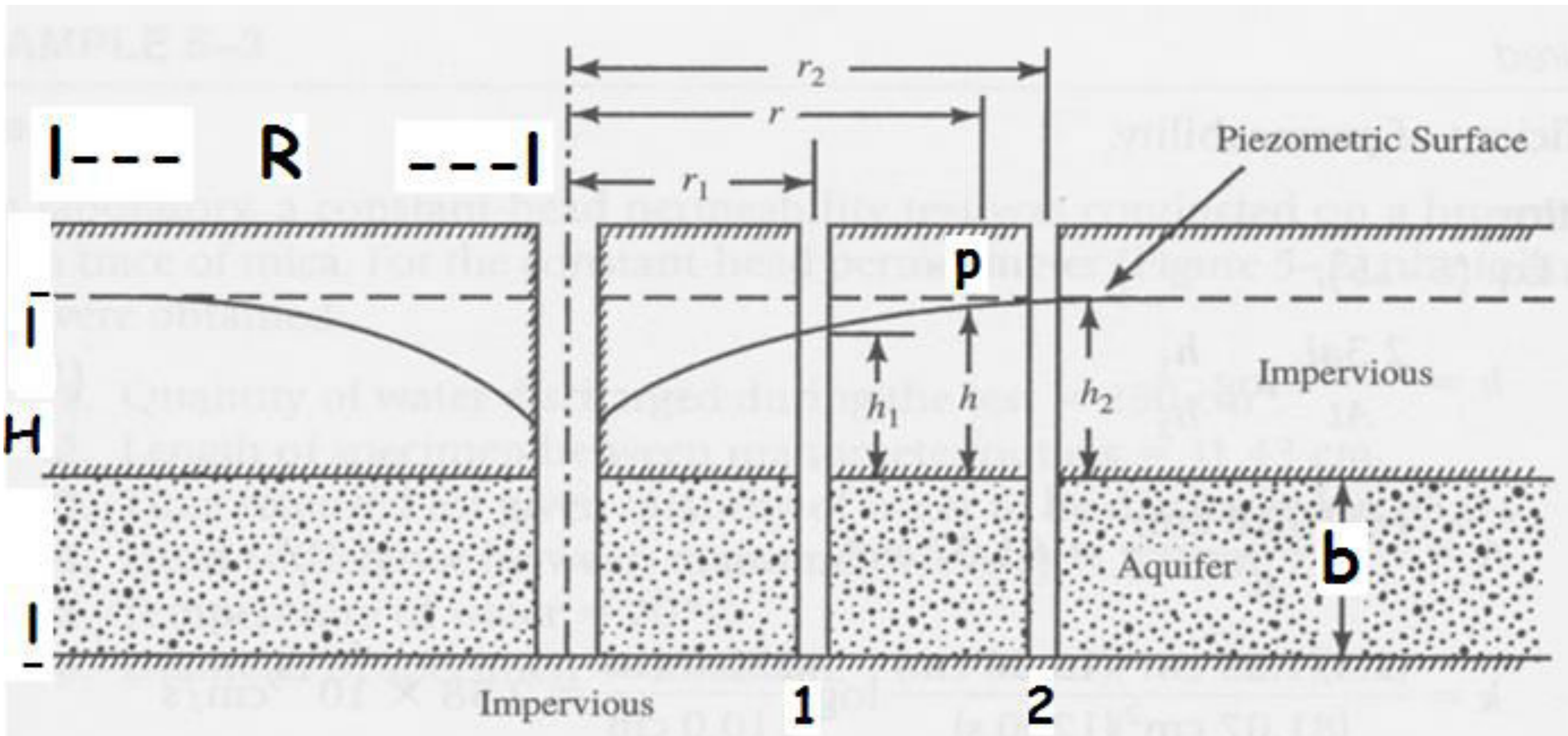
Pumping in tests are conducted to determine
The coefficient of permeability of
an individual stratum thorough which a hole is drilled.

The tests are more economical than pumping out tests

The water pumped in should be clean.
impurities such as silt, clay or any other foreign matter may cause plugging of the flow passages.

Confined flow pumping test

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



Confined Aquifer:

As per Darcy's Law;

$$q = K i A$$

$$q = K \cdot \left[\frac{dz}{dr} \right] \cdot 2\pi r b$$

i at point p $= \frac{dz}{dr} = \frac{dh}{dr}$

on Integration

$$\Rightarrow \int_{r_1}^{r_2} \frac{dr}{r} = \int_{h_1}^{h_2} \frac{2\pi K b}{q} \cdot dh.$$

$$\Rightarrow \int_{r_1}^{r_2} \frac{dr}{r} = \frac{2\pi K b}{q} \int_{h_1}^{h_2} dh$$

$$\Rightarrow \quad \log_e (r)_{r_1}^{r_2} = \frac{2\pi K b}{q} [h]_{h_1}^{h_2}$$

$$\Rightarrow \quad \log_e (r_2 - r_1] = \frac{2\pi K b}{q} [h_2 - h_1]$$

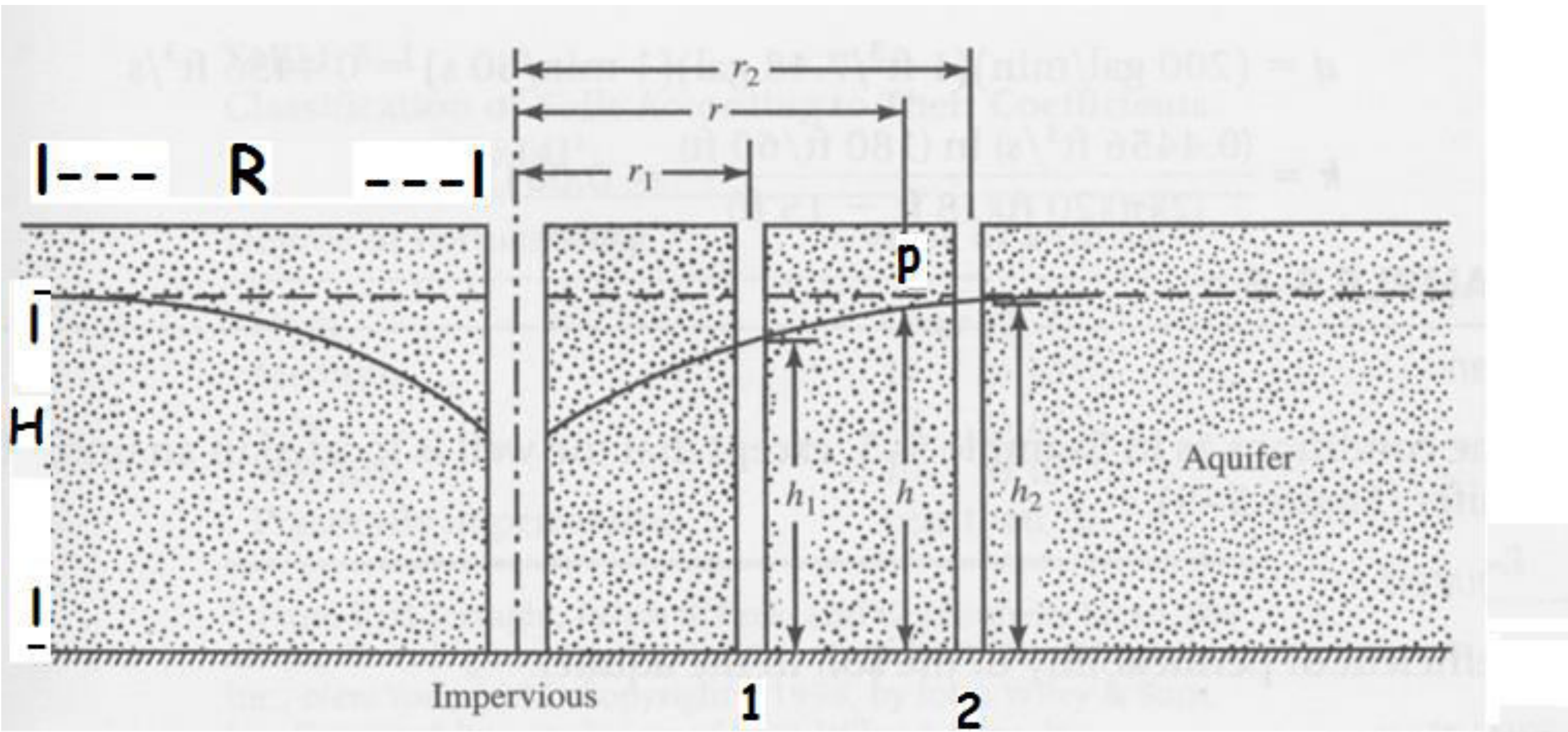
$$\therefore \quad K = \frac{q \log_e \left[\frac{r_2}{r_1}\right]}{2\pi b [h_2 - h_1]}$$

If R and r_w and H and h_w values given;

Then $K = \dfrac{q \log_e \left[\dfrac{R}{r_w}\right]}{2\pi b [H - h_w]}$ ✓

Unconfined flow pumping test

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



Un Confined Aquifer

As per Darcy's Law; $q = kiA$.

$$q = k\left[\frac{dh}{dr}\right] \cdot 2\pi r \cdot h.$$

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

on Integration;

$$\Rightarrow \int_{r1}^{r2} \frac{dr}{r} = \frac{2\pi k}{q} \int_{h1}^{h2} h \cdot dh$$

$$\Rightarrow \log_e\left[\frac{r2}{r1}\right] = \frac{2\pi k}{q}\left[\frac{h_2^2 - h_1^2}{2}\right]$$

$$\Rightarrow \quad K = \frac{q}{\pi\left[h_2^2 - h_1^2\right]} \cdot \log_e\left[\frac{r_2}{r_1}\right]$$

Note: $h_1 = H - S_1$

$h_2 = H - S_2$

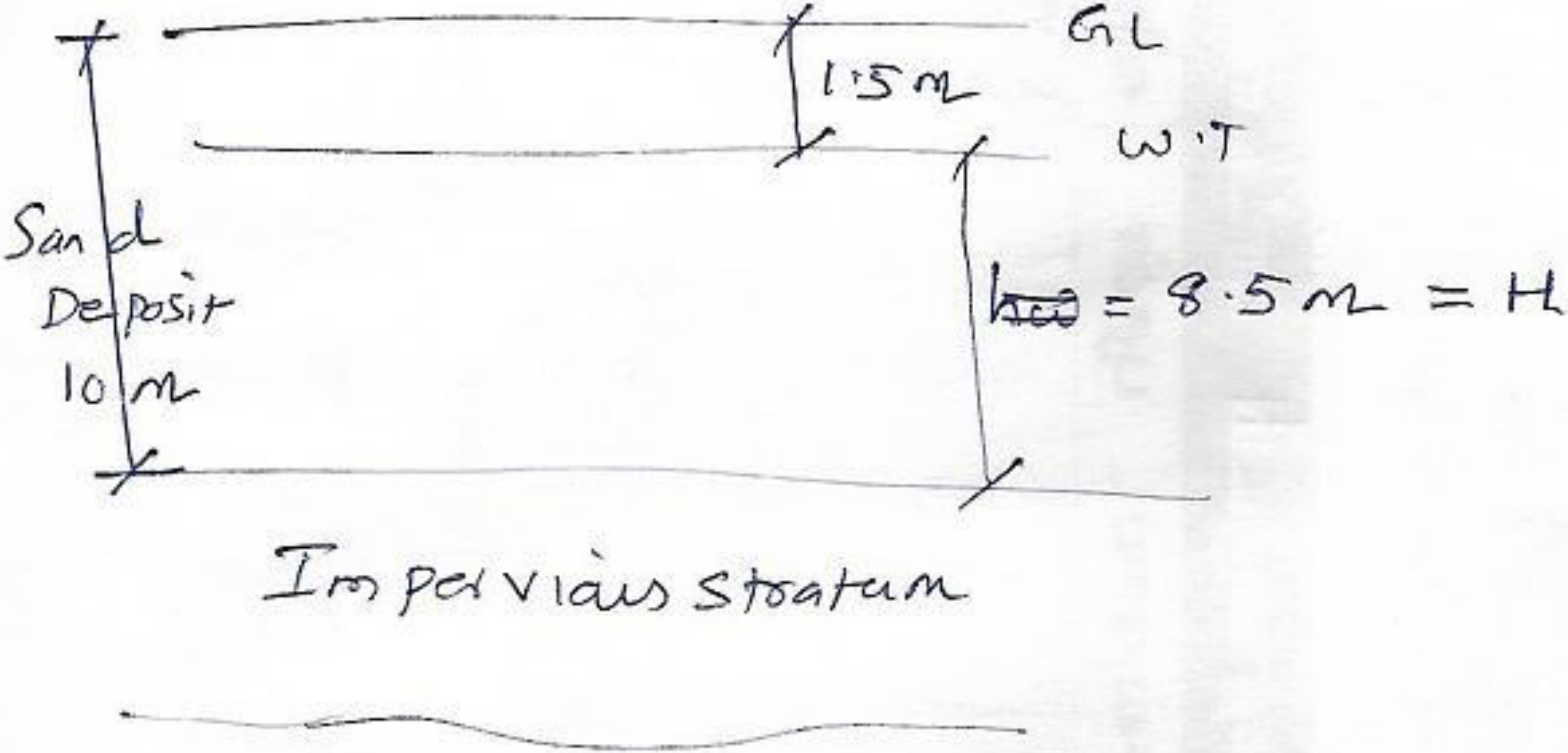
Where S_1 and S_2 are Drawdown values in Testing wells one & Two respectively

Determine co.eff. of permeability of a confined Aquifer 5m tk, which gives a steady discharge of 20 lps through a ~~open~~ well of 0.3 m radius. The height of water in the well was 10 m above the base before pumping and droped to 8 m after pumping. The Radius of influence was 300 m.

$$K = \frac{q \cdot \log_e \left[\frac{R}{r_w} \right]}{2\pi b \left(H - h_w \right)} = 0.0022 \ m/s$$

$$= \underline{2.2 \ mm/s}$$

A Sandy Layer 10 m thick over lies an impervious stratum. The Water table is in the sandy Layer at a depth of 1.5 m below the ground Surface. Water is pumped out from a well at the rate of 100 lit per second and the draw down of the water table at radial distances of 3.0 m and 25 m are 3.0 m and 0.5 m respectively. Determine the coefficient of Permeability.



GL
1.5 m
W.T
Sand Deposit 10 m
h = 8.5 m = H
Impervious stratum

Given: $H = 8.5\,m$

$S_1 = 3.0\,m$

$S_2 = 0.5\,m$

$\therefore h_1 = H - S_1 = 8.5 - 3.0 = 5.5\,m$

$h_2 = H - S_2 = 8.5 - 0.5 = 8.0\,m$

Also given $r_1 = 3.0\,m$; $r_2 = 25\,m$

$q = 100\,l/s = \frac{100}{1000}\,\frac{m^3}{s} = 0.1\,m^3/s$

$$\therefore K = \frac{0.1}{\pi\left[8^2 - (5.5)^2\right]} \cdot \log_e\left[\frac{25}{3}\right]$$

$$= 0.002\,m/s = 2\,mm/s.$$

Determine coefficient of permeability of a confined aquifer 5 m thick which gives a steady discharge of 20 l/s through a well of 0.3 m radius. The height of water in the well which was 10 m above the base before pumping and droped to 8 m. Take radius of influence as 300 m.

Given

$R = 300\,m$

$r_w = 0.3\,m$

$h_w = 8\,m$

$H = 10\,m$

$b = 5\,m$

$q = 20\,l/s = 0.2\,m^3/s.$

$$\therefore K = \frac{q \cdot \log_e \left[\frac{R}{r_w} \right]}{2\pi b \cdot [H - h_w]}$$

$$= \frac{0 \cdot 2 \times \log_e \left[\frac{300}{0 \cdot 3} \right]}{2 \times \pi \times 5 \times [10 - 8]} = 0.0022 \text{ m/s}$$

$$= 2.2 \text{ mm/s}$$

Permeability of Stratified Soils:

When a soil profile consists of a number of strata having different permeability,

the equivalent or average permeability of the soil is different for

Flow of water is parallel or

normal to the plane of stratification.

Flow parallel to plane of stratification

In this case

The hydraulic gradient in each layer is the same and

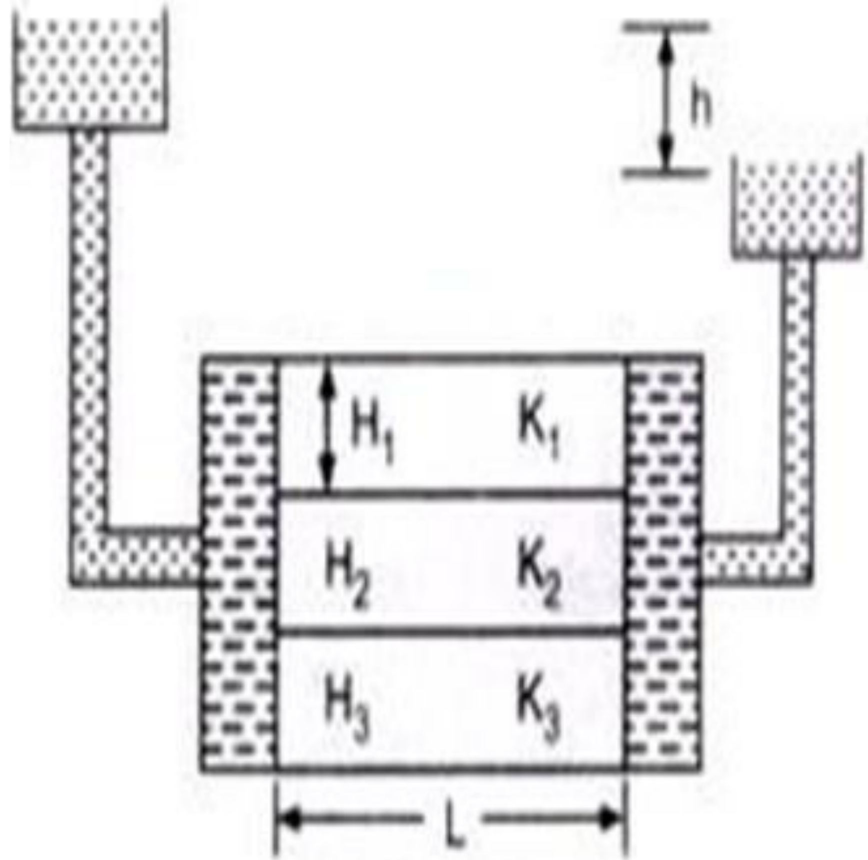
The total flow rate is the sum of flow rates in all the layers.

$$q = q_1 + q_2 + q_3$$

$$K_x iH = K_1 iH_1 + K_2 iH_2 + K_3 iH_3$$

$$K_x H = K_1 H_1 + K_2 H_2 + K_3 H_3$$

$$K_x = \frac{K_1 H_1 + K_2 H_2 + K_3 H_3}{H}$$



Where K_x = Equivalent or average permeability in direction parallel to the plane of stratification – X direction

Flow perpendicular or normal to plane of stratification

In this case

The flow rate must be same in all layers for steady flow, and

as the flow area ‘A’ is constant,

The flow velocity across layer is also the same

But

The total head loss equal to Sum of head loss in all layers

The total head loss 'h' is equal to the sum of the losses in the three layer.

$$h = h_1 + h_2 + h_3$$

Note: H = L in this case

$i = h/L = h/H$ or $h = iH$

$$iH = i_1 H_1 + i_2 H_2 + i_3 H_3$$

$V = K/i$ or $i = V/K$

$$\frac{vH}{K_z} = \frac{v_1}{K_1}H_1 + \frac{v_2}{K_2}H_2 + \frac{v_3}{K_3}H_3$$

As $v = v_1 = v_2 = v_3$

We can write.

$$\frac{H}{K_z} = \frac{H_1}{K_1} + \frac{H_2}{K_2} + \frac{H_3}{K_3} \quad \text{or} \quad K_z = \frac{H}{\frac{H_1}{K_1} + \frac{H_2}{K_2} + \frac{H_3}{K_3}}$$

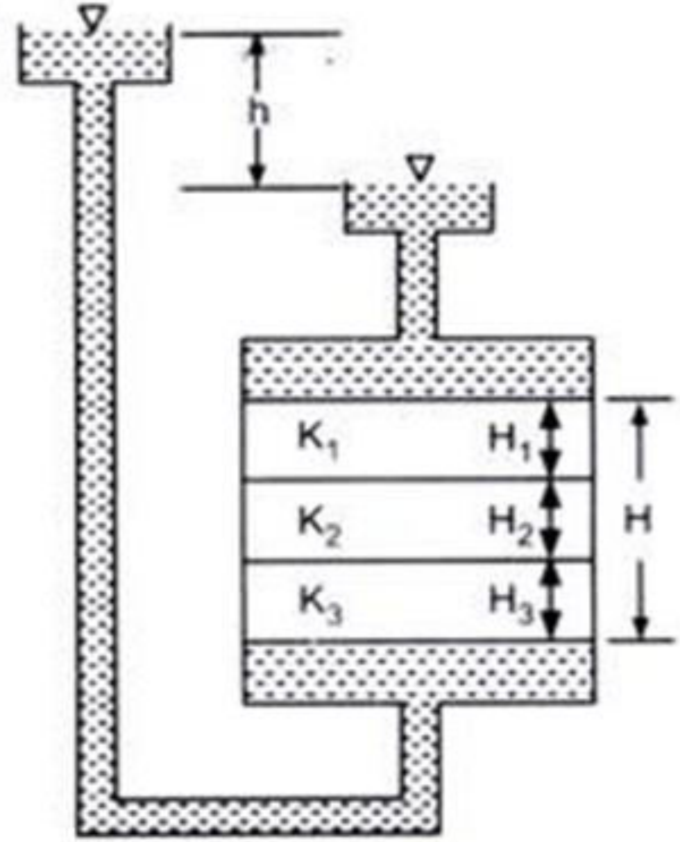


FIG. 4.6

Where K_z = equivalent permeability for flow normal to the layers.

So the equivalent permeability for flow parallel to the strata is always

greater than that for flow normal to the strata

i.e., K_x is always greater than K_z.

A sand deposit is made up of three horizontal layers of equal thickness. The permeability of the top and bottom layers is 2×10^{-4} cm/s and that of middle layer is 3.2×10^{-2} cm/s. Find the equivalent permeability in the horizontal and vertical direction and their ratio.

Given $H_1 = H_3$

$K_1 = K_3 = 2 \times 10^{-4}$ cm/s

$K_2 = 3.2 \times 10^{-2}$ cm/s

$$K_x = \frac{K_1H_1 + K_2H_2 + K_3H_3}{H}$$

$$= \frac{2 \times 10^{-4} \times H_1 + 3.2 \times 10^{-2} \times H_1 + 2 \times 10^{-4} \times H_1}{3H_1}$$

$$[\because H_1 = H_2 = H_3]$$

$K_x = 1.08 \times 10^{-2}$ cm/s **Ans**

$$K_z = \frac{H}{\frac{H_1}{K_1} + \frac{H_2}{K_2} + \frac{H_3}{K_3}}$$

$$= \frac{3H_1}{\frac{H_1}{2 \times 10^{-4}} + \frac{H_1}{3.2 \times 10^{-2}} + \frac{H_1}{2 \times 10^{-4}}}$$

$= 2.99 \times 10^{-4}$ cm/s

$\frac{K_x}{K_z} = 36.1$ **Ans.**

Determine the av. coeff. of permeability in the horizontal and vertical direction for a deposit consist of three Layers of thickness 5m, 1m and 2.5m and having coeff of permeability 3×10^{-2} mm/s, 3×10^{-5} mm/sec and 4×10^{-2} mm/sec respectively.

$$H = 5+1+2.5 = 8.5m$$

$$K_{aH} = \frac{k_1 H_1 + k_2 H_2 + k_3 H_3}{H}$$

$$= \frac{3\times10^{-2}\times5\times10^{3} + 3\times10^{-5}\times1\times10^{3} + 4\times10^{-2}\times2.5\times10^{3}}{8.5\times10^{3}}$$

$$= \frac{150 + 0.03 + 100}{8500} = \underline{0.0294} \text{ mm/sec}$$

$$K_{av} = \frac{H}{H_1/K_1 + \frac{H_2}{K_2} + \frac{H_3}{K_3}}$$

$$K_{av} = \frac{8500}{\frac{5\times10^3}{3\times10^{-2}} + \frac{1\times10^3}{3\times10^{-5}} + \frac{2.5\times10^3}{4\times10^{-2}}}$$

$$= \frac{8500}{0.166\times10^5 + 0.333\times10^8 + 6.125\times10^4}$$

$$= \frac{8.500}{10^4\left[1.66 + 3330 + 6.125\right]}$$

$$= \frac{0.85}{3337.785} = \underline{0.253\times10^{-3}\ \text{mm/s}}$$

What will be the ratio of average permeability in the horizontal direction to the vertical direction for a soil deposit consisting of three Layers. The thickness and permeability of second layer are twice that of First Layer and those of third is twise that of second.

$$K_{aH} = \frac{K_1 H_1 + K_2 H_2 + \cdots}{H}$$

$$= \frac{K H + 2K\, 2H + 4K\, 4H}{7H} = \frac{K\cancel{H}}{\cancel{H}}\left[\frac{21}{7}\right] = 3K$$

$$K_{av} = \frac{H}{\frac{H_1}{K_1} + \frac{H_2}{K_2} + \cdots}$$

$$= \frac{7H}{\frac{H}{K} + \frac{2H}{2K} + \frac{4H}{4K}} = \frac{H}{\frac{H}{K}} \frac{7}{(1+1+1)}$$

$$\Rightarrow \frac{\cancel{H}}{1} \times \frac{K}{\cancel{H}} \times \frac{7}{3} = \frac{7K}{3}$$

$$\text{Ratio of } \frac{K_{aH}}{K_{av}} = \frac{3K}{\frac{7K}{3}} = \frac{3\cancel{K}}{1} \times \frac{3}{7\cancel{K}}$$

$$= 9/7$$

THANK YOU

UNIT III

EFFECTIVE STRESS DUE TO APPLIED LOADS
&
CONSOLIDATION

VERTICAL STRESS INCREASES IN SOIL DUE TO APPLIED LOADS

Types of Loading

1) Point Loads – Electric Post ,Column ,etc.

2) Lines Loads – Boundary wall , Centre wall ,etc.

3) Strip Loads – Rail way track loading ,Strip foundation ,etc

.

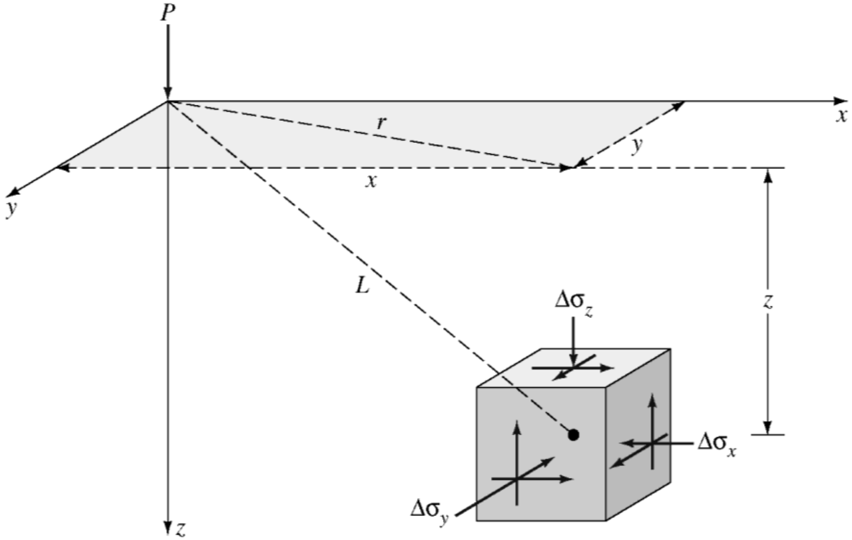
4) Rectangular Loads – Raft or Rectangular footing, etc.

5) Circular Loads - Water tank

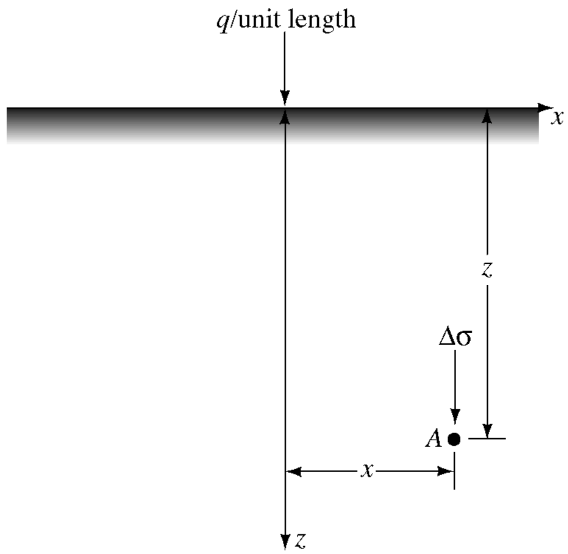
Vertical Stress Increases in Soil

Types of Loading

Point Loads (P)



Line Loads (q/unit length)



Examples:

Electric post, column

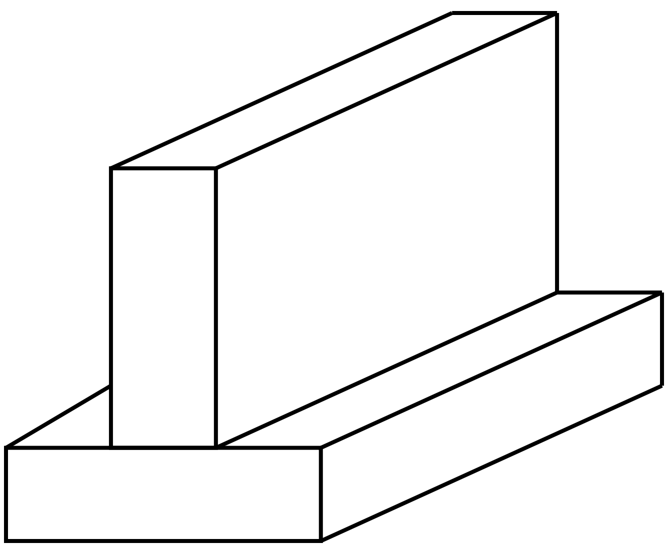
Examples:

- Rail way track

Vertical Stress Increases in Soil

Types of Loading

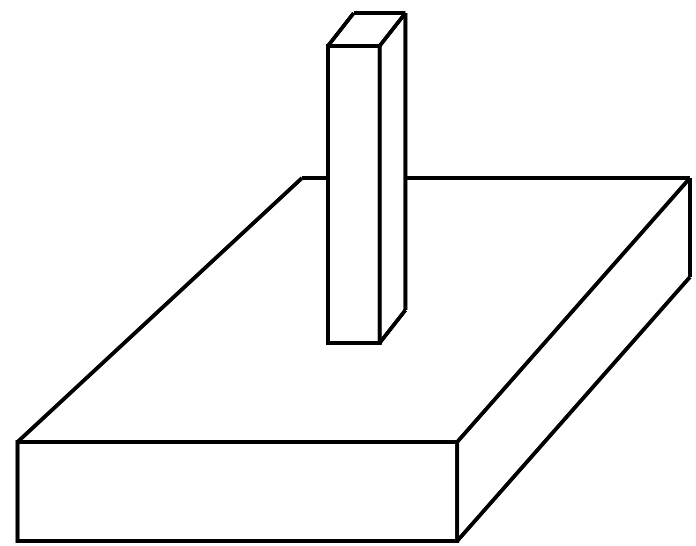
Strip Loads (q)



Examples:

- Exterior Wall Foundations

Area Loads (q)



Examples:

- Column Footings

BOUSSINESQ THEORY

Assumptions:

1. The soil mass is an elastic medium (elasticity is constant)
2. The soil is homogeneous.
3. The soil is isotropic.
4. The soil mass is semi-infinite.
5. Self weight of soil is neglected.
6. The soil is initially stress free.
7. The change in volume of soil is neglected.

Effective stress due to point load

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Das Bildelement mit der Beziehungs-ID rId2 wurde in der Datei nicht gefunden.

OR

Das Bildelement mit der Beziehungs-ID rId3 wurde in der Datei nicht gefunden.

Where,

Das Bildelement mit der Beziehungs-ID rId4 wurde in der Datei nicht gefunden.

= Boussinesq influence coefficient for the vertical stress.

(BOUSSINESQ Influence Coefficient Values)

r/z	I_B		r/z	I_B
0	0.4775		0.9	0.1083
0.1	0.4657		1.0	0.0844
0.2	0.4329	B	1.5	0.0251
0.3	0.3849		1.75	0.0144
0.4	0.3295		2.0	0.0085
0.5	0.2733		2.5	0.0034
0.6	0.2214		3.0	0.0015
0.7	0.1762		4.0	0.0004
0.8	0.1386		5.0	0.00014

EFFECTIVE STRESS DUE TOCIRCULAR LOADING

RECTANGULAR LOADING

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Das Bildelement mit der Beziehungs-ID rId3 wurde in der Datei nicht gefunden.

2V:1H DISTRIBUTION METHOD

Das Bildelement mit der Beziehungs-ID rId2 wurde in der Datei nicht gefunden.

$Q = q \times$ Area of Foundation

Where:

Q = Applied Foundation Load

B = Foundation Width

L = Foundation Length

Das Bildelement mit der Beziehungs-ID rId2 wurde in der Datei nicht gefunden.

Fig.3.17 Equivalent point- load method

Das Bildelement mit der Beziehungs-ID rId2 wurde in der Datei nicht gefunden.

Das Bildelement mit der Beziehungs-ID rId3 wurde in der Datei nicht gefunden.

- Uniformly Distributed Load q can be divide in to More umber of square small area and calculate concentrated load as consideration.

$$\sigma_z = I_B \cdot \frac{Q}{Z^2}$$

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Das Bildelement mit der Beziehungs-ID rId3 wurde in der Datei nicht gefunden.

Das Bildelement mit der Beziehungs-ID rId2 wurde in der Datei nicht gefunden.

Das Bildelement mit der Beziehungs-ID rId2 wurde in der Datei nicht gefunden.

Das Bildelement mit der Beziehungs-ID rId3 wurde in der Datei nicht gefunden.

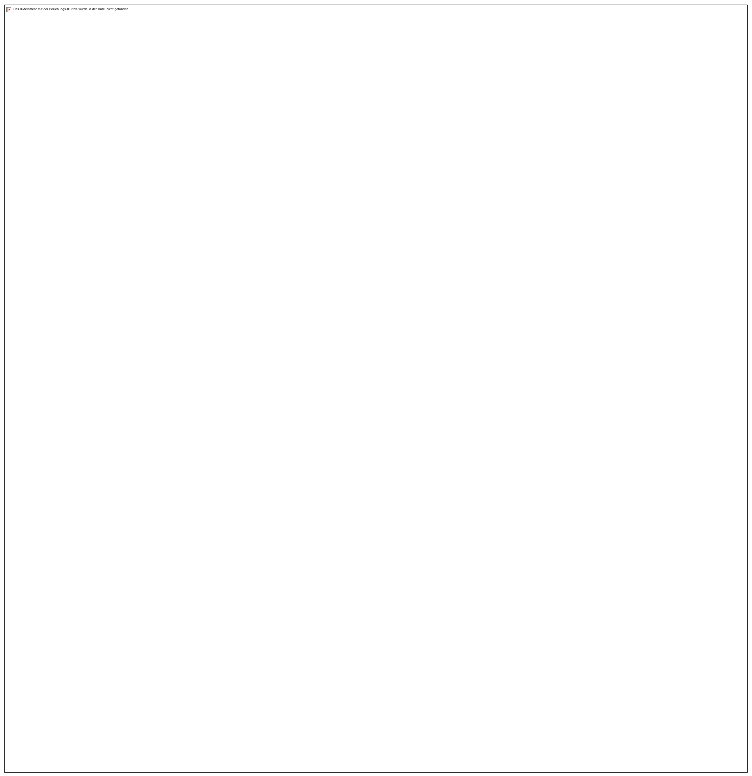
For group (III):

$$\frac{r}{z} = \frac{\sqrt{1^2+1^2}}{6} = \frac{\sqrt{2}}{6} = 0.236$$

$$I_p = 0.4$$

Das Bildelement mit der Beziehungs-ID rId2 wurde in der Datei nicht gefunden.

VERTICAL STRESS INCREASE (σ_z) IN SOIL DUE TO LINE LOADING



Line Load over the Surface of a Semi-infinite Soil Mass

Figure

$$\Delta\sigma = \frac{2qz^3}{\pi(x^2 + z^2)^2}$$

or

Dimensionless Form

$$\frac{\Delta\sigma}{(q/z)} = \frac{2}{\pi\left[\left(\frac{x}{z}\right)^2 + 1\right]^2}$$

Where:

- $\Delta\sigma$ = Change in Vertical Stress
- q = Load per Unit Length
- z = Depth
- x = Distance from Line Load

VERTICAL STRESS INCREASE (σ_z) IN SOIL DUE TO STRIP LOADING

Where:

$\Delta\sigma$ = Change in Vertical Stress
q = Load per Unit Area
z = Depth
x = Distance from Line Load

Angles measured in counter-clockwise direction are taken as positive

ISOBAR-Pressure Bulb

- An *isobar is a curve which connects all points of equal stress below the ground surface.*

- Since these isobars form closed contour and resemble the form of a bulb, they are also termed *as pressure bulb.*

Das Bildelement mit der Beziehungs-ID rId2 wurde in der Datei nicht gefunden.

Example of Pressure bulb.

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NEWMARK’S INFLUENCE CHART

Das Bildelement mit der Beziehungs-ID rId2 wurde in der Datei nicht gefunden.

The Newmark's Influence Chart is useful for the determination of vertical stress(σ)

at any point below the uniformly loaded area of any shape.

This method is based on the concept that

each area unit sector causes

the equal vertical stress at the centre of the circle.

NEWMARK'S INFLUENCE CHARTS

- Chart consisting of10 number of circles divided in to 24 sectors of each 15degree.

- By that entire chart is divided in to 240 parts.
- Therefore Influence co efficient =

Das Bildelement mit der Beziehungs-ID rId2 wurde in der Datei nicht gefunden.

- New mark chart drawn to a scale of 1: 100 and by assuming Depth Z = 5m

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Das Bildelement mit der Beziehungs-ID rId2 wurde in der Datei nicht gefunden.

Das Bildelement mit der Beziehungs-ID rId2 wurde in der Datei nicht gefunden.

Circular No.	σ_z/q	r/z	r
1	0.1	0.27	1.35
2	0.2	0.4	2.0
3	0.3	0.518	2.5
4	0.4	0.637	3.2
5	0.5	0.766	3.85
6	0.6	0.918	4.6
7	0.7	1.11	5.55
8	0.8	1.387	6.95
9	0.9	1.908	9.55
10	0.95		
	1.0	∞	∞

- .When using the diagram, the foundation plan view is drawn to the scale 1cm = 5m
- It is placed over the influence diagram so that the point where the stress is to be calculated lies directly under the centre of the influence diagram
- the number of subdivisions of the influence diagram that are covered by the foundations are counted.
- Note: More than ¾ part covered taken as Full part
- Less than ¼ part covered is just ignored
- More than ¼ part and less than ¾ part covered are treated as Half part

The vertical stress at the point is then:

the multiple of the influence factor,
the number of subdivisions covered,
and the average applied pressure.

- The induced vertical pressure at any other position at the same depth is found in the same manner just by shifting the position of the foundation plan on the influence diagram and counting covered subdivisions again.
- If the depth is more than 5m say 7 m, then loaded plan view should be drawn to a scale 1cm = 5/7 m – small size, covers less number of parts, vertical stress is less.
- If the depth is less than 5m say 3 m, then loaded plan view should be drawn to a scale 1cm = 5/3 m – Larger size covers more number of parts, vertical stress is more.

Contact pressure

The upward pressure due to soil
on the underside of the footing is
Termed as contact pressure.

In the derivations of vertical stress below the loaded
areas
using Boussinesq' s theory or Westergaard' s theory,

it has been assumed that
the footing is flexible and the contact pressure
distribution is uniform and equal to `q' .
But Actual footings are not flexible as assumed.

The actual distribution of the contact pressure depends
on a number of factors.

a. Contact pressure diagram on sand

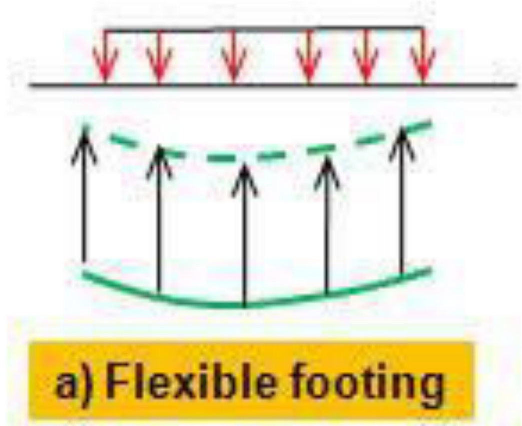
when the footing is flexible,

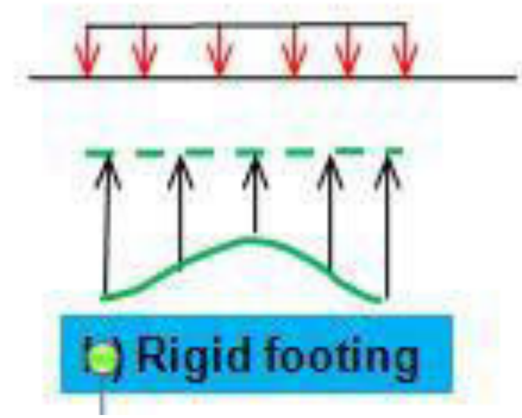
the edges undergo a large settlement than at centre. The soil at centre is confined and therefore has a high modulus of elasticity and deflects less for the same contact pressure.

When the footing is rigid

the settlement is uniform. The contact pressure is parabolic with zero intensity at the edge and maximum at the centre.

B . Contact pressure diagram on saturated clay





When the footing is flexible,

it deforms into the shape of a bowel,

with the maximum deflection at the centre.

When the footing is Rigid,

The contact pressure distribution is

minimum at the centre and the maximum at the edges.

Consolidation

Consolidation

The process involving
a gradual compression occurring simultaneously
with expulsion of water from the soil mass and
with a gradual transfer of the applied pressure
from the pore water to the solid particles
is called consolidation.

The process opposite to consolidation is called swelling

which involves an increase in the water content
due to an increase in the volume of the voids.

Factors Affecting Consolidation

1. External static loads from structures.

2. Self-weight of the soil such as recently placed fills.

3. Lowering of the ground water table.

4. Desiccation (Draught).

Components of Settlement

The total compression of a saturated clay strata under excess effective pressure may be considered as the sum of

1. Immediate compression,
2. Primary consolidation, and
3. Secondary compression.

Initial Consolidation

Sudden small volume reduction due to expulsion of air from the voids

referred as initial consolidation

Primary Consolidation

Volume reduction in the soil due to expulsion of water from the soil over a long time due to static application of load is referred as primary consolidation.

Secondary consolidation

This compression starts after the primary consolidation ceases,

that is after the excess pore water pressure approaches zero

due to

Re-orientation or re- arrangement of of solid particles

Difference between Compaction and Consolidation

***compressibility* (compaction or Consolidation)**

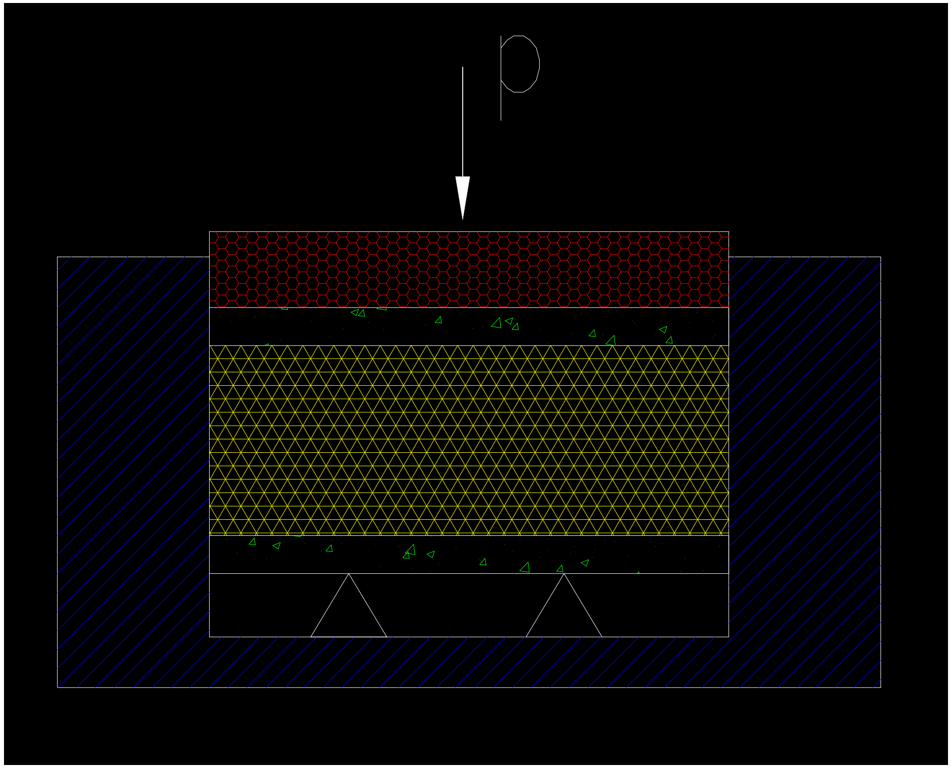
The process of volume change in a soil when subjected to pressure

consolidation

The process of rate of volume change over a time period due to static loading

compaction

Expulsion of air from a soil by applying Dynamic Loading and thereby sudden volume change.



Consolidation test

<u>oedometer</u>

Assumption:

- Load distribution-uniform
- Stress distribution(in different height)-the same
- Lateral deformation-0
- The area of the sample section-unchangeable
- Solid soil-uncompressible

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oedometer

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Das Bildelement mit der Beziehungs-ID rId2 wurde in der Datei nicht gefunden.

Consolidometer

Testing Methodology

The soil sample is contained in the brass ring between two porous stones about 1.25 cm thick.

by means of the porous stones water has free access to and from both surfaces of the specimen.

The compressive load is applied to the specimen through a piston,

either by means of a hanger and dead weights or by a system of levers.

The compression is measured on a dial gauge.

At the bottom of the soil sample the water expelled from the soil flows through the filter stone into the water container.

At the top, a well-jacket filled with water is placed around the stone in order to

prevent excessive evaporation from the sample during the test.

Water from the sample also flows into the jacket through the upper filter stone.

The soil sample is kept submerged in a saturated condition during the test.

Loads are applied in steps in such a way that

the successive load intensity, *p, is twice the* preceding one.

The load intensities commonly used being 25, 50,100,200,400, 800 and 1600 kN/m^2.

Each load is allowed to stand until compression has practically ceased (no longer than 24 hours).

The dial readings are taken at elapsed times of 1/4, 1/2, *1,2,4, 8,*15, 30, 60, 120, 240, 480 and 1440 minutes from the time the new increment of load is put on the sample.

After the greatest load required for the test has been applied to the soil sample,

the load is removed in decrements to provide data for plotting the expansion curve of the soil in order to

learn its elastic properties and magnitudes of plastic or permanent deformations.

The following data should also be obtained:

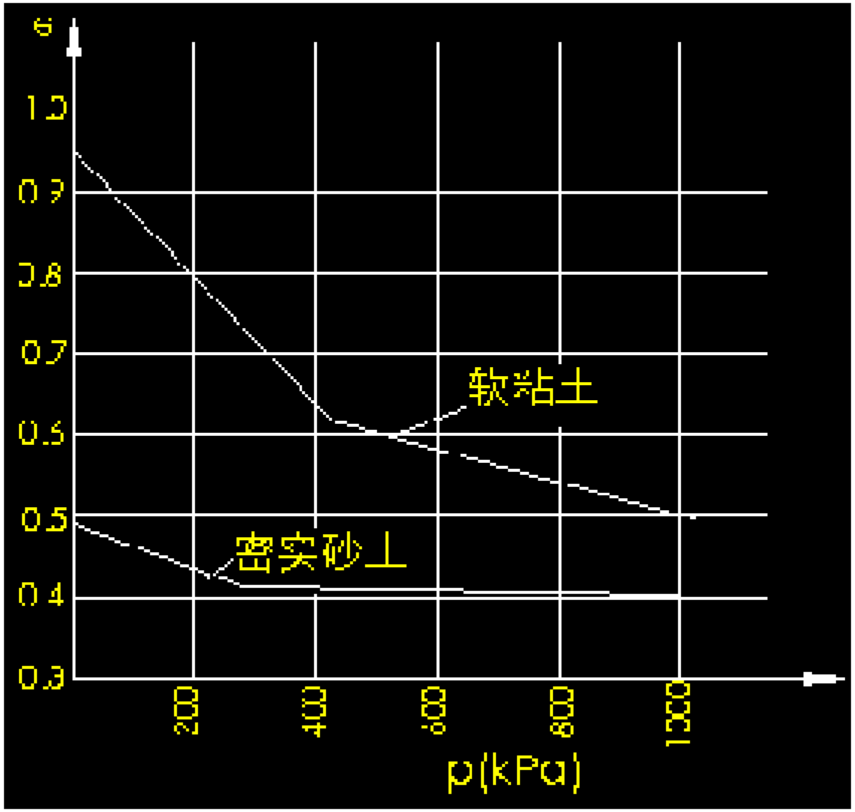
Moisture content and weight of the soil sample before the commencement of the test.

Moisture content and weight of the sample after completion of the test.

The specific gravity of the solids.

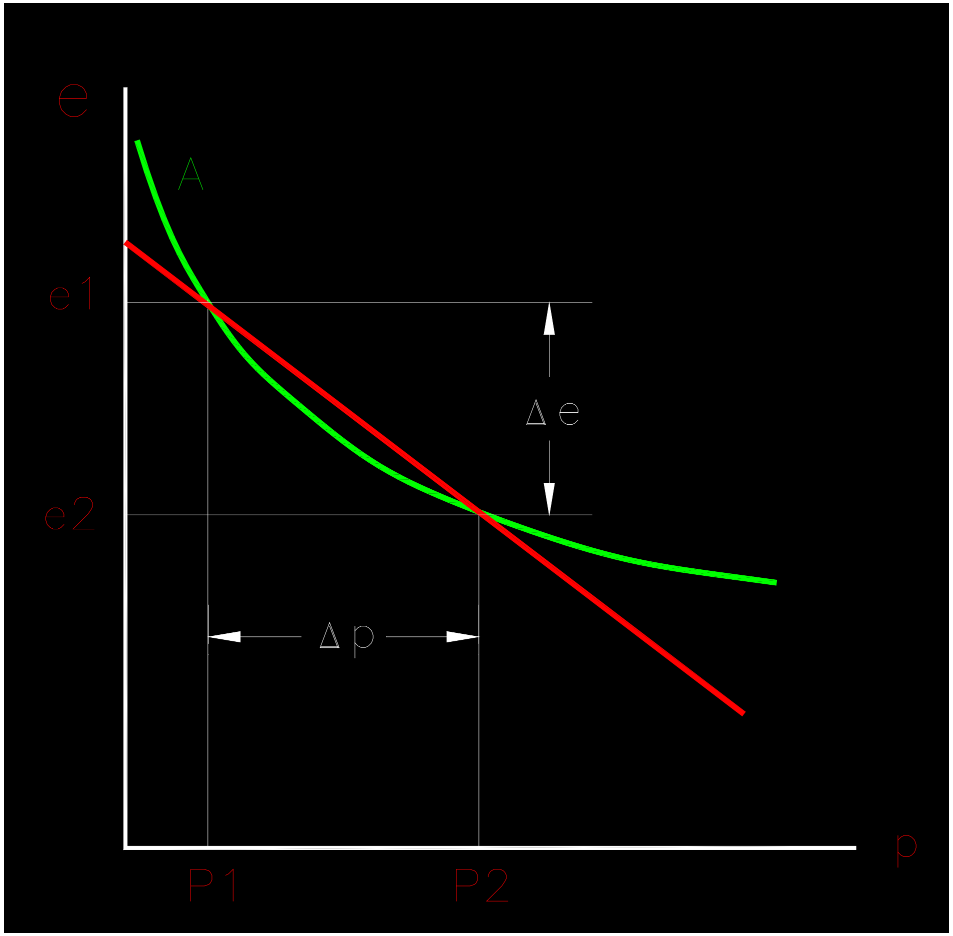
The temperature of the room where the test is conducted

e Log P curve



3 compression Coefficient

$$a_v = \frac{e_1 - e_2}{p_2 - p_1} = -\frac{de}{dp}, \qquad MPa^{-1}, \quad kPa^{-1}$$



$a_v \quad < \quad 0.1\text{Mpa}^{-1}$, Low compressibility

$0.1 \leq a_v \quad < 0.5\text{Mpa}^{-1}$, Middle compressibility

$a_v \quad \geq 0.5\text{Mpa}^{-1}$, High compressibility

4 compression index Cc

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

$$m_v = \frac{a_v}{1+e_0} \textit{coefficient of volume compressibility}$$

over consolidation ratio ***OCR***

It is Defined as

Previous Maximum Effective stress / present Maximum Effective Stress

For a normally consolidated clay

the present effective stress is also the previous maximum so OCR=1.

for a heavily over consolidated clay
OCR may be 4 or more

therefore this type of soil has been subjected to a much greater stress in the past

Settlement of Foundation

$$s = \frac{e_0 - e_1}{1 + e_0} \cdot h = \frac{a_v \cdot \Delta p}{1 + e_0} \cdot h = m_v \cdot \Delta p \cdot h$$

Terzaghi One Dimensional Consolidation Theory

Dr. Karl Terzaghi gave the theory of one dimensional consolidation based on the following assumptions:

1. Soil is completely saturated.
2. Soil & water are virtually incompressible.
3. The compression is one- dimensional.
4. Darcy's Law is valid.
5. Soil permeability is constant.
6. The coefficient of volume compressibility (m_v) is assumed to be constant
7. No secondary compression or creep occurs
8. The total stress on the element is assumed to remain constant.

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. Time factor

$$T = \frac{c_v \cdot t}{H^2}$$

Where average degree of consolidation = (U)

Taylor's Square Root of Time Fitting Method

From the oedometer test the dial reading (settlement) corresponding to a particular time is measured.

From the measured data, dial reading vs $\sqrt {Time}$ graph can be drawn as shown in Figure .

A straight line can be drawn passing through the points on initial straight portion of the curve .

The intersection point between the straight line and the dial reading axis is denoted as R_0 which is corrected zero reading i.e U = 0%.

Starting from $R_{0,}$ draw another straight line such that its abscissa is 1.15 times the abscissa of first straight line.

The intersection point between the second straight line and experimental curve represents the R_{90} and corresponding $\sqrt {{t_{90}}}$ is determined.

Thus, the time required (t_{90}) for 90% consolidation is calculated.

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Das Bildelement mit der Beziehungs-ID rId3 wurde in der Datei nicht gefunden.

Casagrande's Logarithm of Time Fitting Method

Select two points t_1 and t_2 in initial part of the curve such that $t_2 = 4\ t_1$.

The points corresponding to the chosen times are marked on the curve.

The vertical distance (z) between the two points on the curve is measured.

Select another point R_0 such that the vertical distance between that point and point on the curve corresponding to the t_1 time is also z.

R_0 is corrected zero reading i.e U = 0%.

Determine the U=100% line by drawing two tangents form the straight portion of the curve .

Once U=0% and 100% lines are identified, U= 50% line is also determined by choosing the middle point between the U=0% and 100% lines.

Time (t_{50}) corresponding to the 50% degree of consolidation is determined from the curve.

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$$C_V = \frac{T_V \, H^2}{t_{50}}$$

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Das Bildelement mit der Beziehungs-ID rId2 wurde in der Datei nicht gefunden.

- A recently completed fill was 32.8 ft thick and its initial average void ratio was 1.0. The fill was loaded on the surface by constructing an embankment covering a large area of the fill. Some months after the embankment was constructed, measurements of the fill indicated an average void ratio of 0.8. Estimate the compression of the fill.

- A soil sample has a compression index of 0.3. If the void ratio *e at a stress of 2940 Ib/ft2 is 0.5,* compute (i) the void ratio if the stress is increased to 4200 Ib/ft2, and (ii) the settlement of a soil stratum 13 ft thick.

THANK YOU

Shear Strength of Soils

UNIT IV

Strength of different materials

Steel

Tensile strength - significant

Concrete

Compressive strength - significant

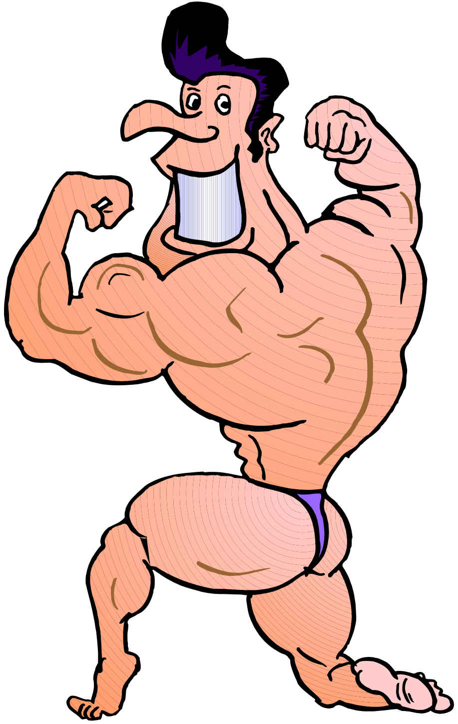
Soil

Shear strength – significant

Presence of pore water

Complex behavior

SHEAR STRENGTH OF SOIL



Definition -1

The maximum shear stress
That
the soil can sustain
just before failure,
under normal stress of σ
known as
Shear strength τ_f

Definition -2

the shear strength of any material

is

the load per unit area

or

pressure that

it can withstand

before

undergoing shearing failure

Definition -3

Shear strength

is a

soils' ability

to resist

sliding of solid particles

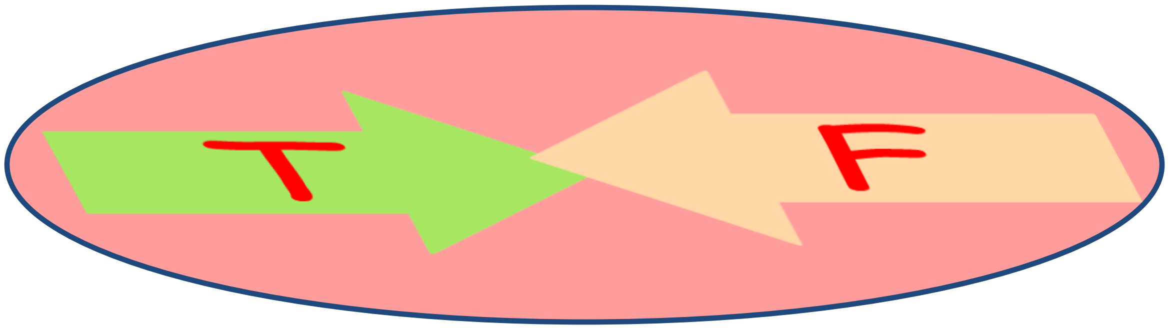
along

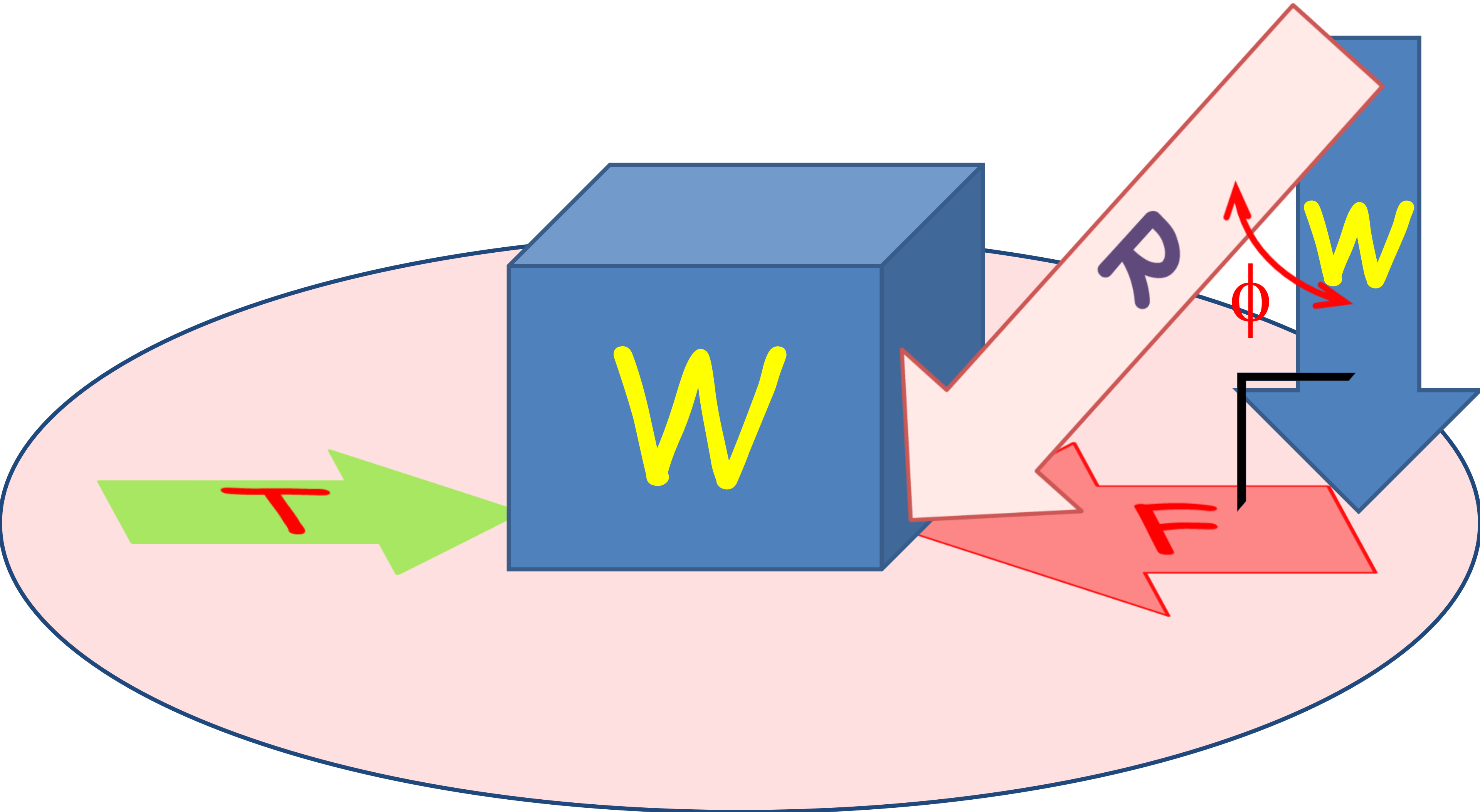
internal surfaces

within the soil mass

The Shear Force 'F' that acts on the failure plane
is applied on the
soil element of weight W

resisted by
the strength of the material T





W
R
W
φ
T
F

To overcome the friction force F

an internal reaction force T caused

gives

the resultant vector, R

which

acts at an angle of ϕ

with respect to

the normal to the plane.

Angle ϕ is known as the coefficient of friction.

Principal planes and principal stresses

Infinite number of planes passing through
a point in a soil mass
Among them
There are
Three mutually perpendicular planes
on which
shear stress is zero
called principal planes.

Only normal stresses that acts on these planes
are
called principal stresses.

The largest principal stress is called major principal stress (σ_1),

The lowest principal stress is called minor principal stress (σ_3)

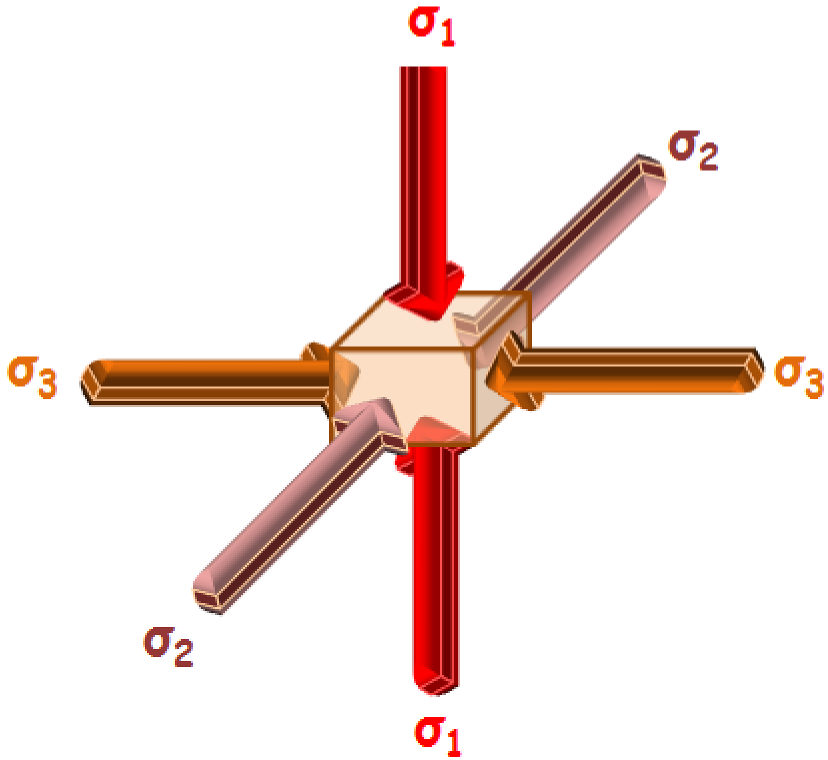
The third stress is called intermediate stress (σ_2).

The corresponding planes are called

major, minor and intermediate plane, respectively.

The critical stress values or failure stresses generally occur on the plane normal to the intermediate plane.

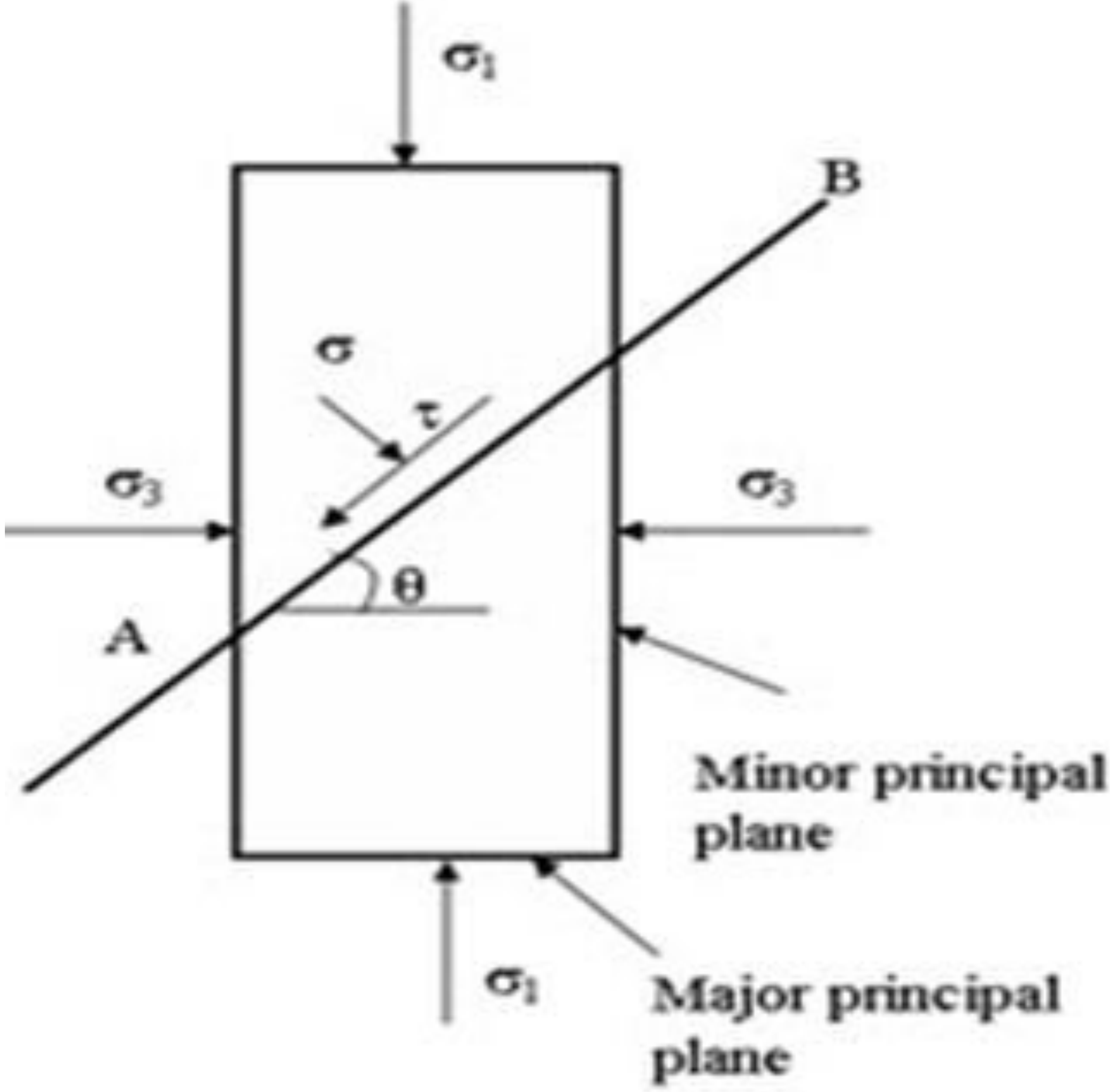
Thus, only σ_1 and σ_3 are considered.



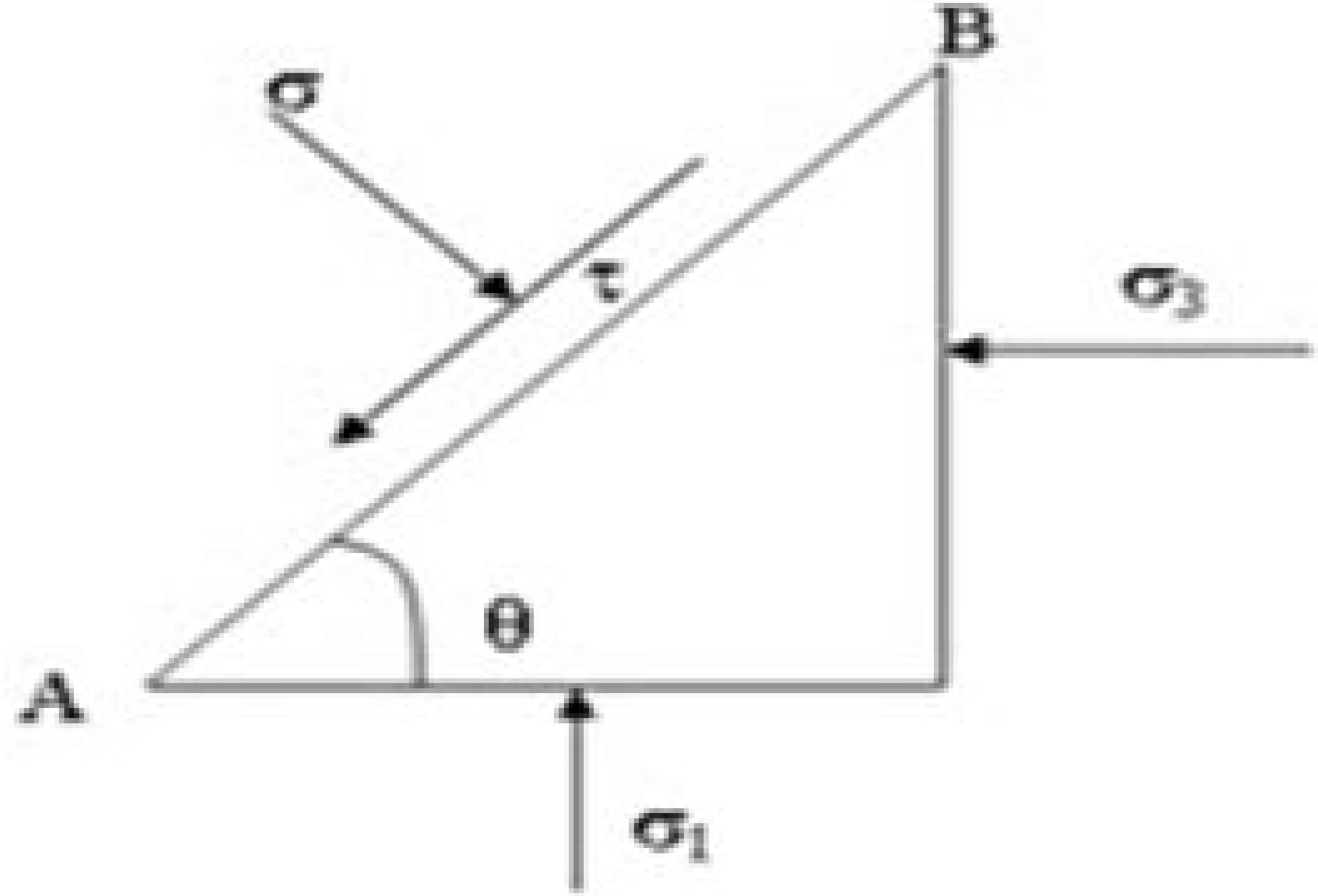
Due to Normal Loading
The soil element squeezed vertically will tend to bulge horizontally

Due to which the soil reacts with confining pressures σ_2 and σ_3 in the other principal directions.

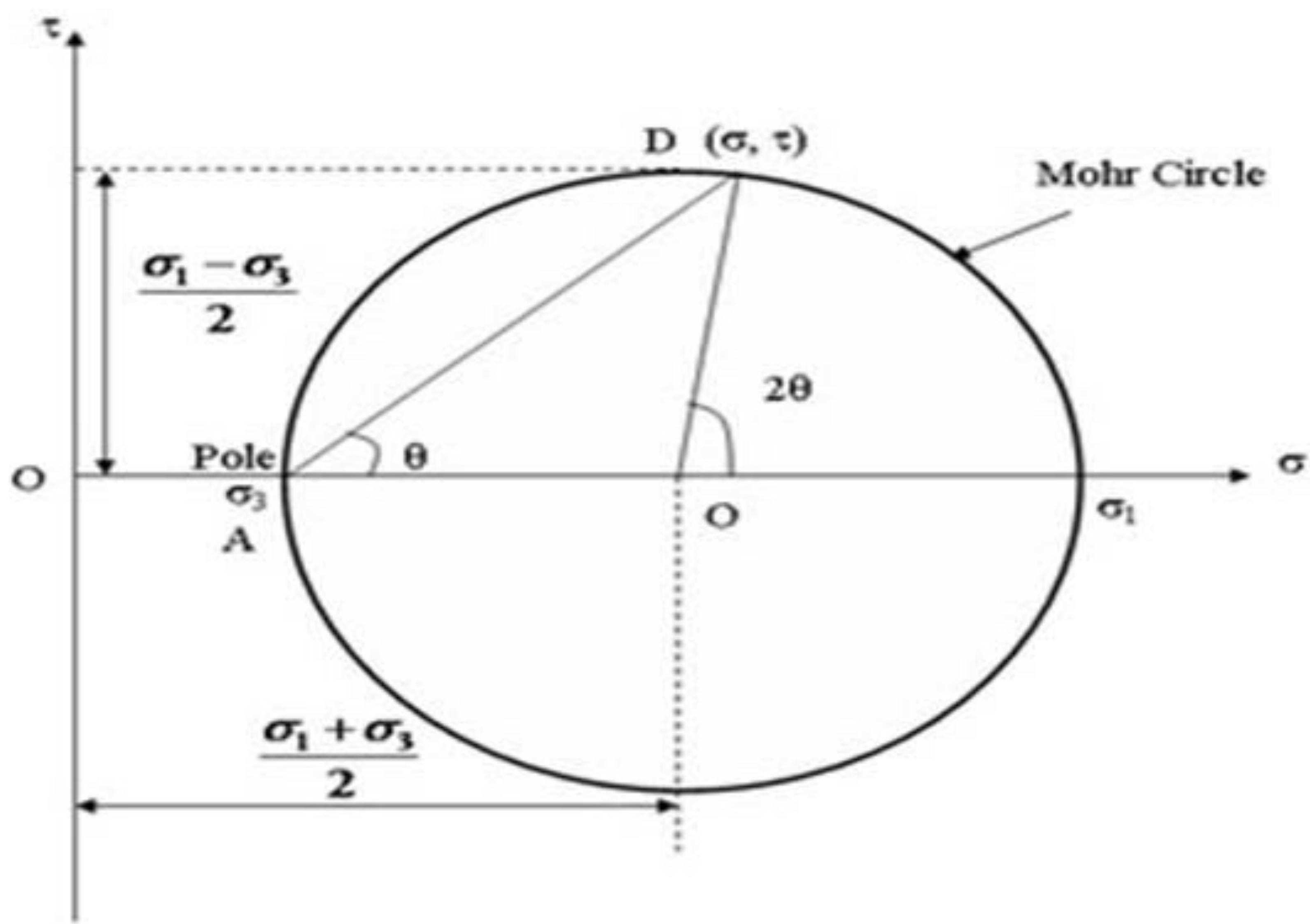
Since we assume the soil is isotropic, the confining lateral pressure will be the same in all directions and so $\sigma_2 = \sigma_3$



σ_1
B
σ
τ
σ_3
σ_3
θ
A
Minor principal plane
σ_1
Major principal plane



B
σ
τ
σ3
θ
A
σ1



τ
D (σ, τ)
Mohr Circle
$\frac{\sigma_1 - \sigma_3}{2}$
2θ
Pole
θ
O
σ
σ_3
A
O
σ_1
$\frac{\sigma_1 + \sigma_3}{2}$

Factors Influencing Shear Strength

***soil composition*:**

Mineralogy

grain size and grain size distribution

shape of particles

pore fluid type and content

type of ions in pore fluid.

Initial state:

State can be describe by terms such as:

Loose or dense

over-consolidated or normally consolidated

stiff or soft

***Structure*:**

Refers to the arrangement of particles within the soil mass;

the manner in which the particles are packed or distributed.

Such as

Flocculated or dispersed

Loading conditions:

Type of loading
(static or dynamic)

Time history:
monotonic or cyclic.

Magnitude of loading

Rate of Loading

Examples of Shear failure

Would you like this to happen?

The failure occurs because shear stress applied exceeded the shear strength of the soil.

We need to determine the soil's shear strength and design the slope such that

The shear stress imposed is not greater than the shear strength of the soil.

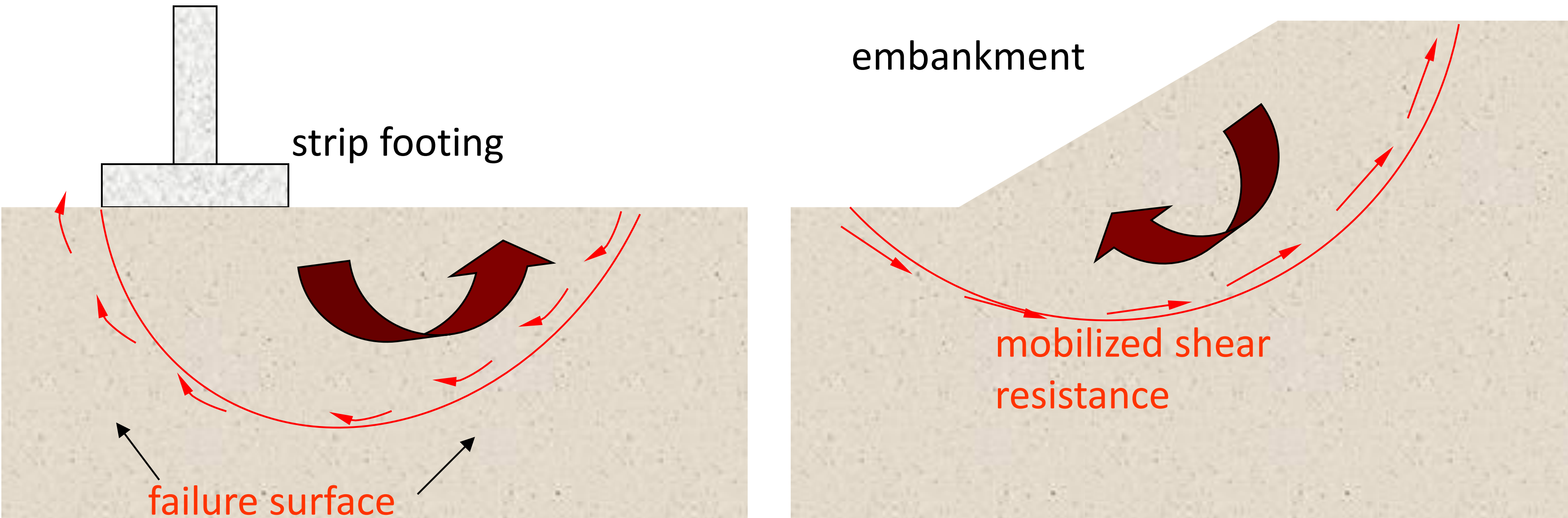
Shear failure of soils - Embankment

Embankment Failure

Shear failure of soils – Retaining wall

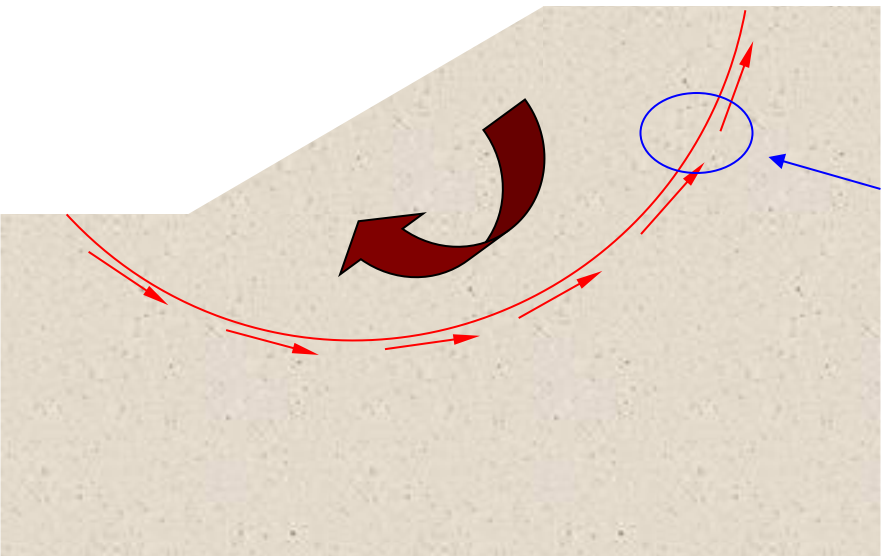
Shear failure

Soils generally fail in shear



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

Shear failure

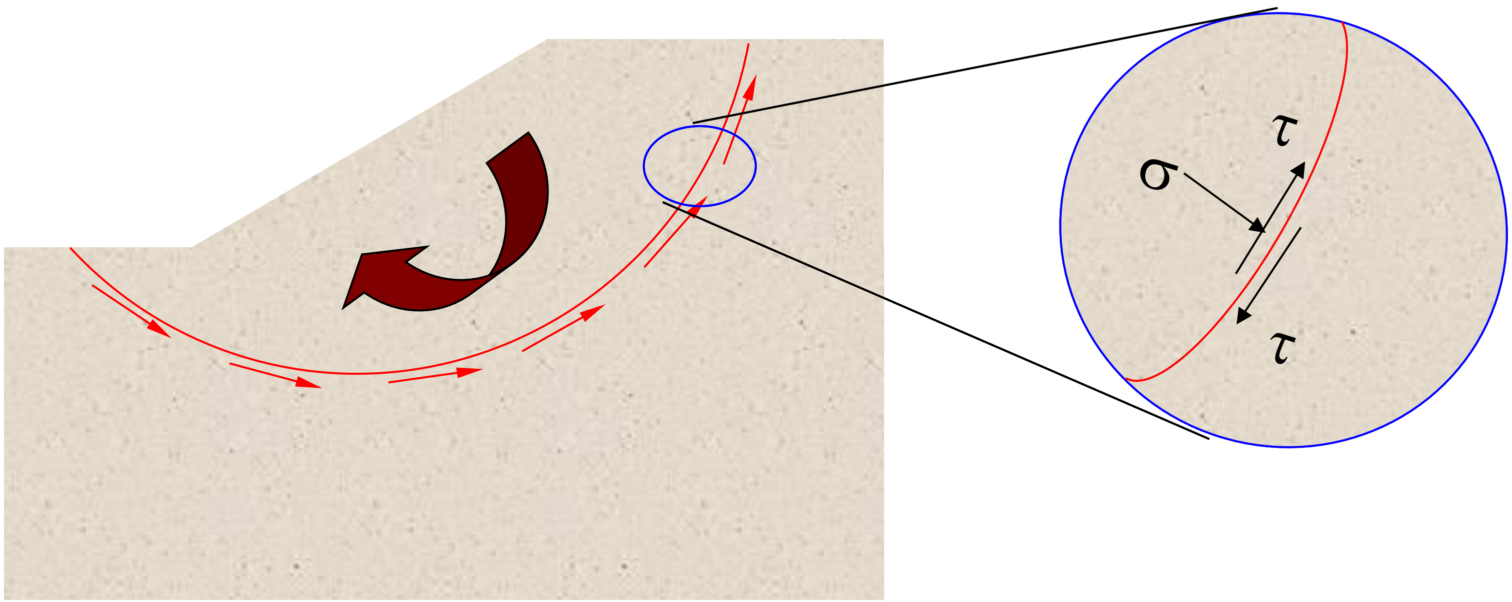


failure surface

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

No crushing of individual grains.

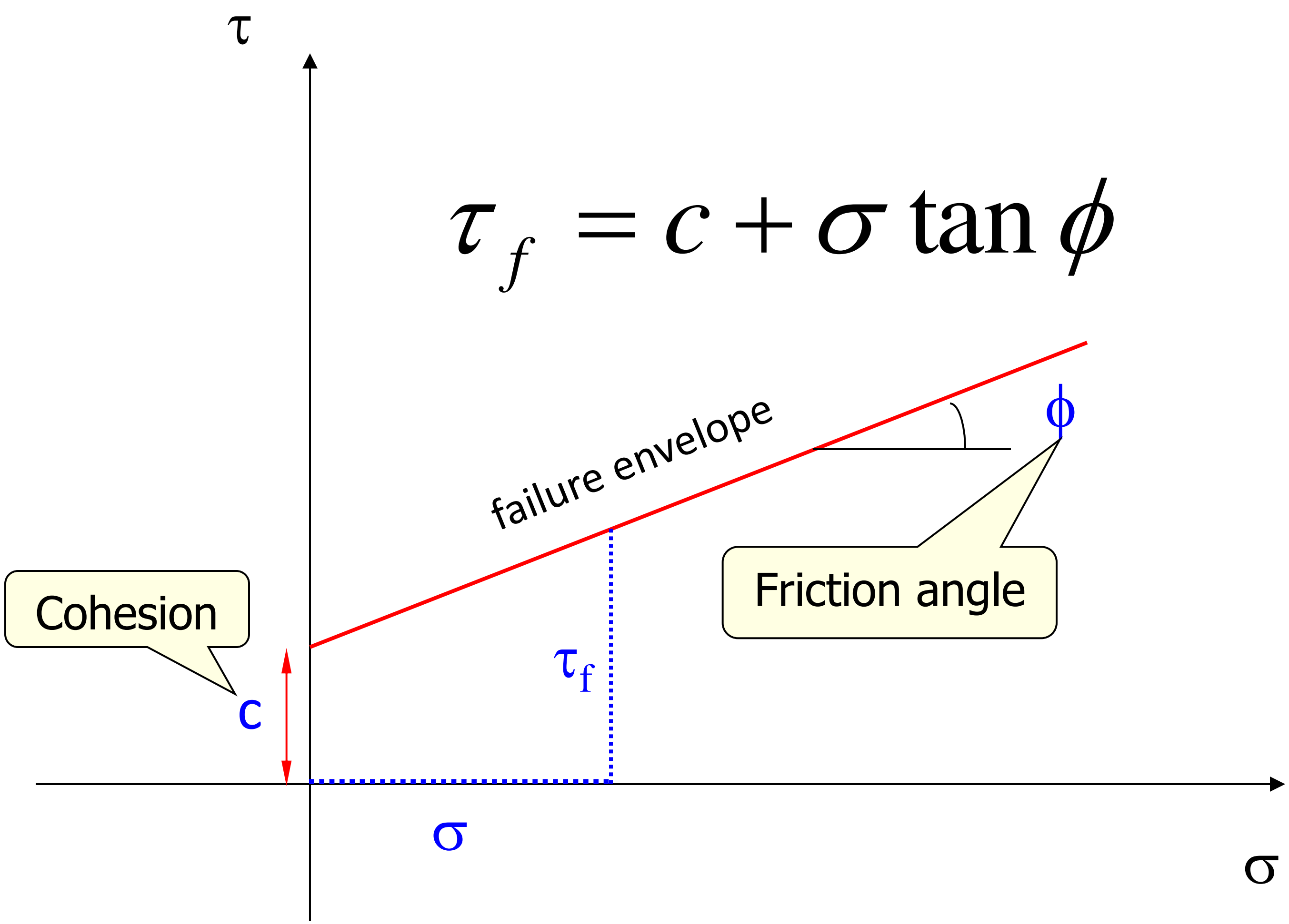
Shear failure mechanism



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

Mohr-Coulomb Failure Criterion

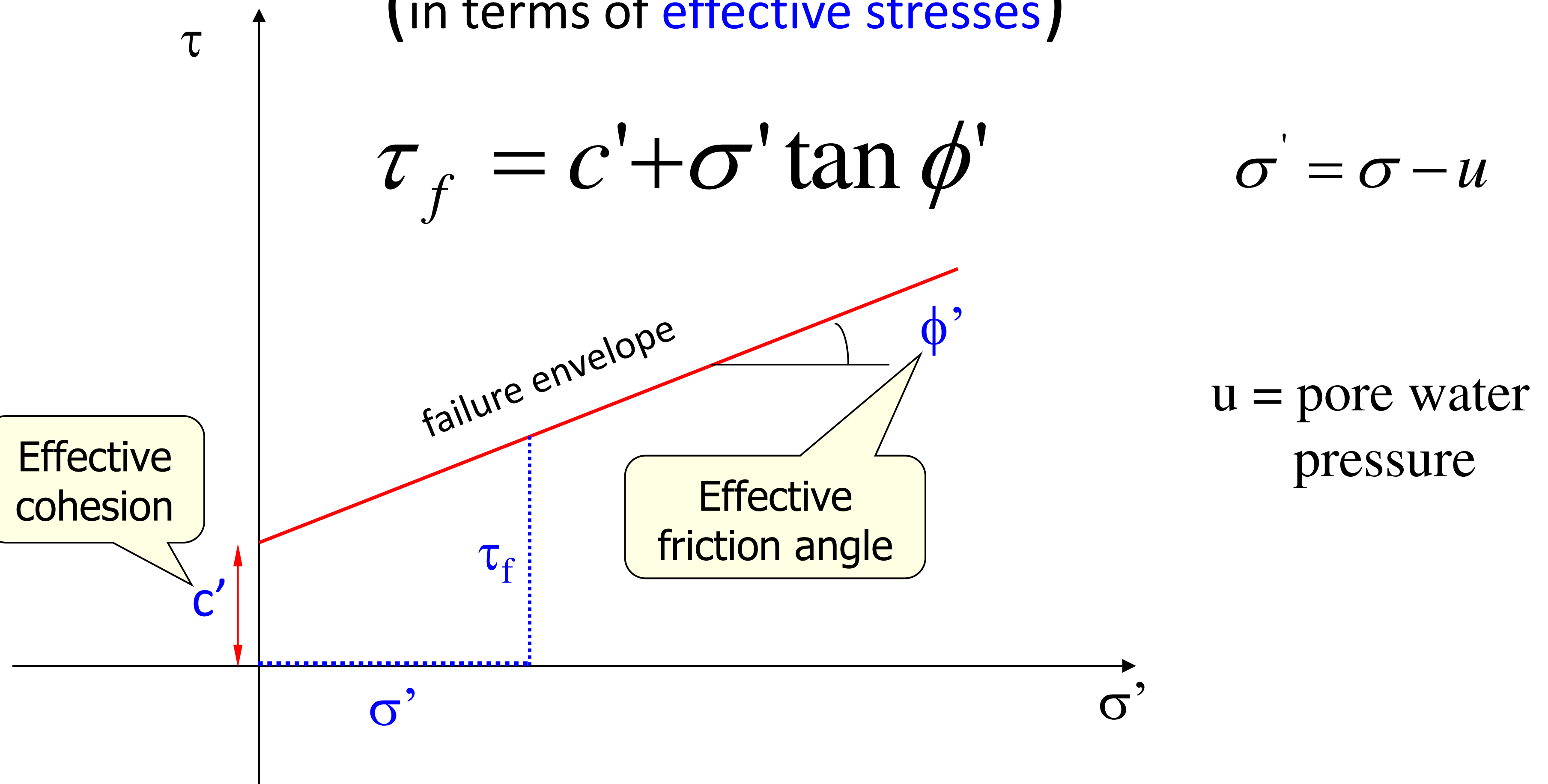
(in terms of total stresses)



c and ϕ are measures of shear strength.

Mohr-Coulomb Failure Criterion

(in terms of effective stresses)

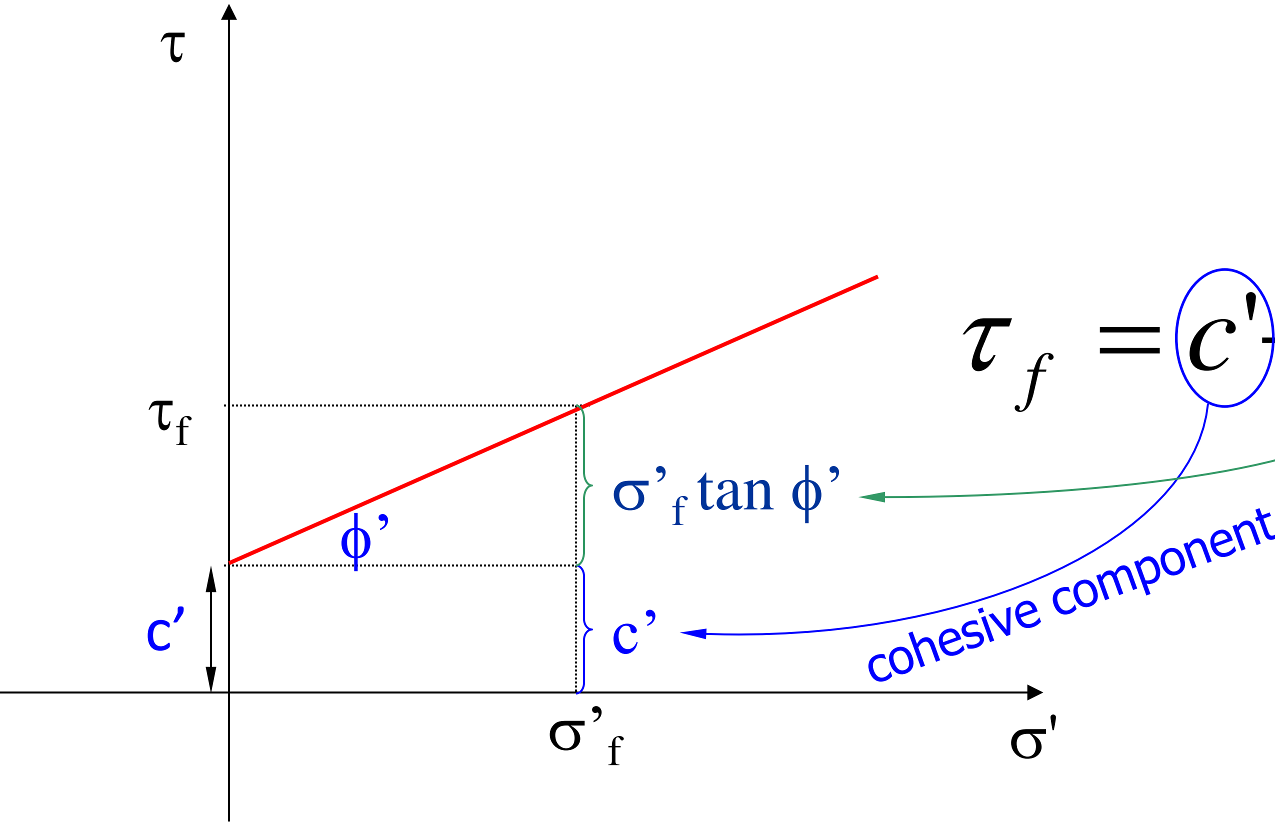


τ_f is the maximum shear stress the soil can take without failure, under normal stress of σ.

C' and ϕ' are measures of shear strength.

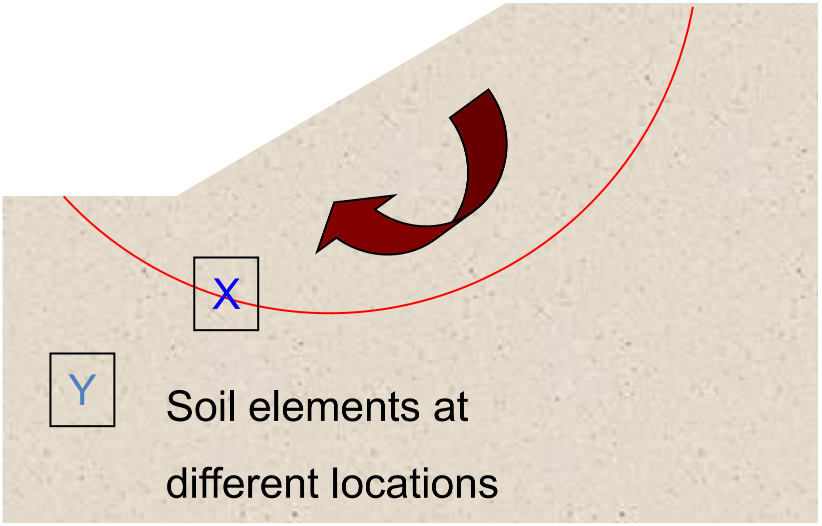
Mohr-Coulomb Failure Criterion

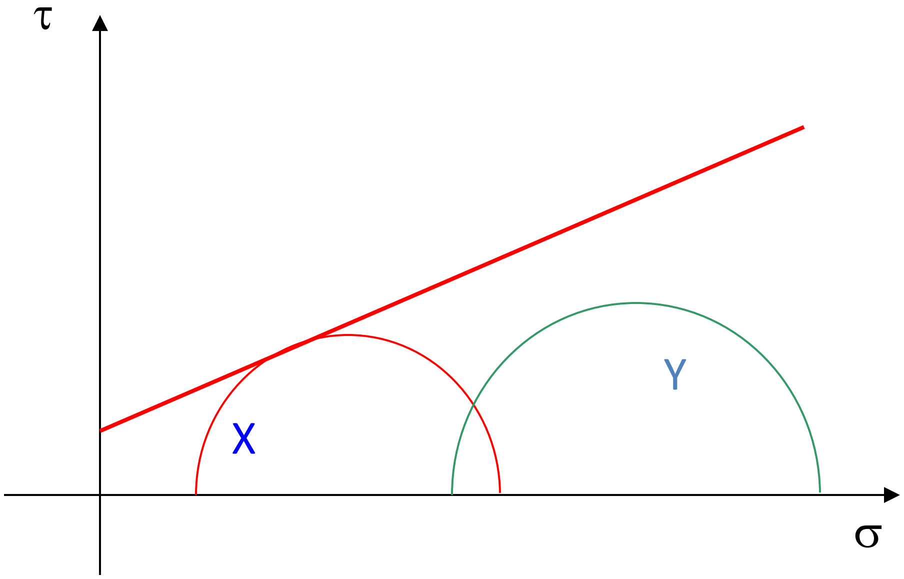
Shear strength consists of two components: cohesive and frictional.



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

Mohr Circles & Failure Envelope



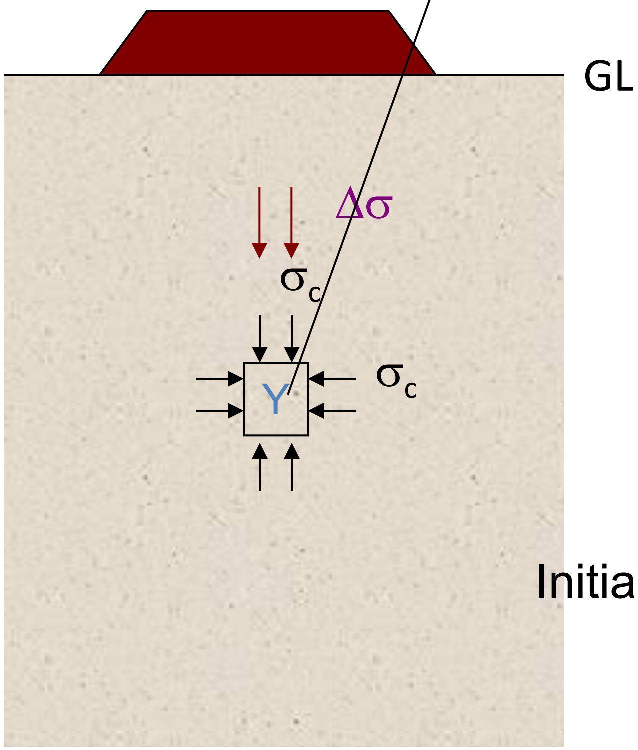


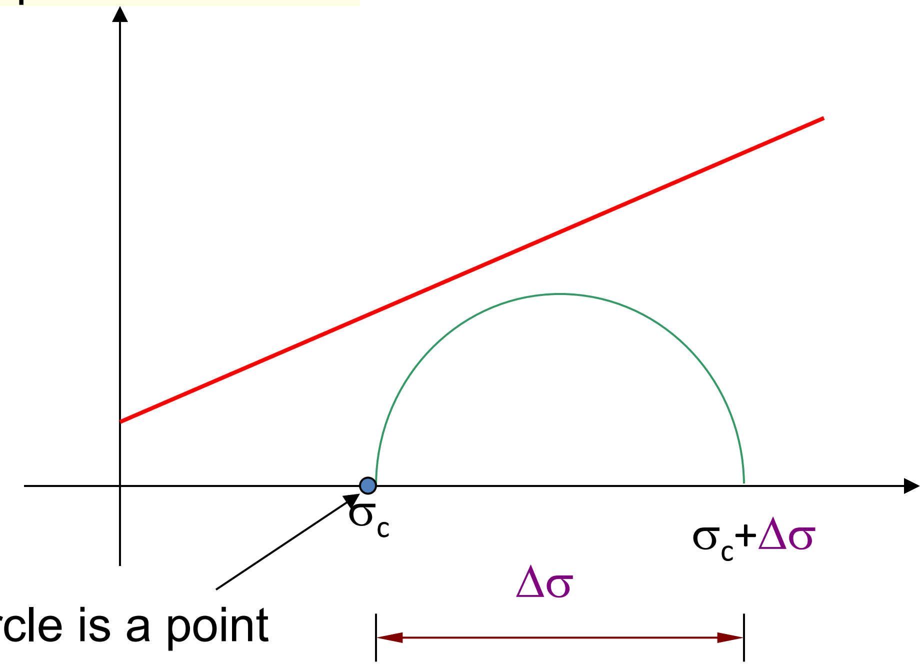
X ~ failure

Y ~ stable

Mohr Circles & Failure Envelope

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

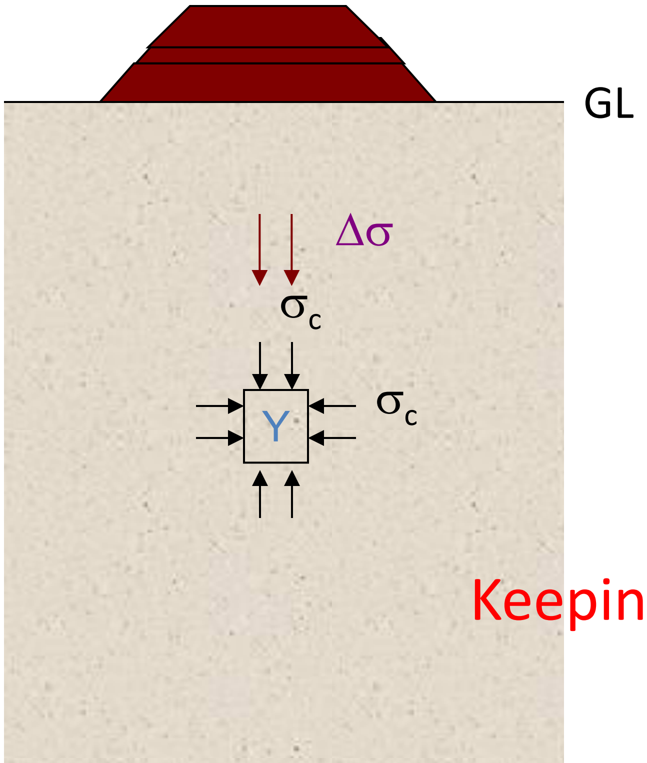


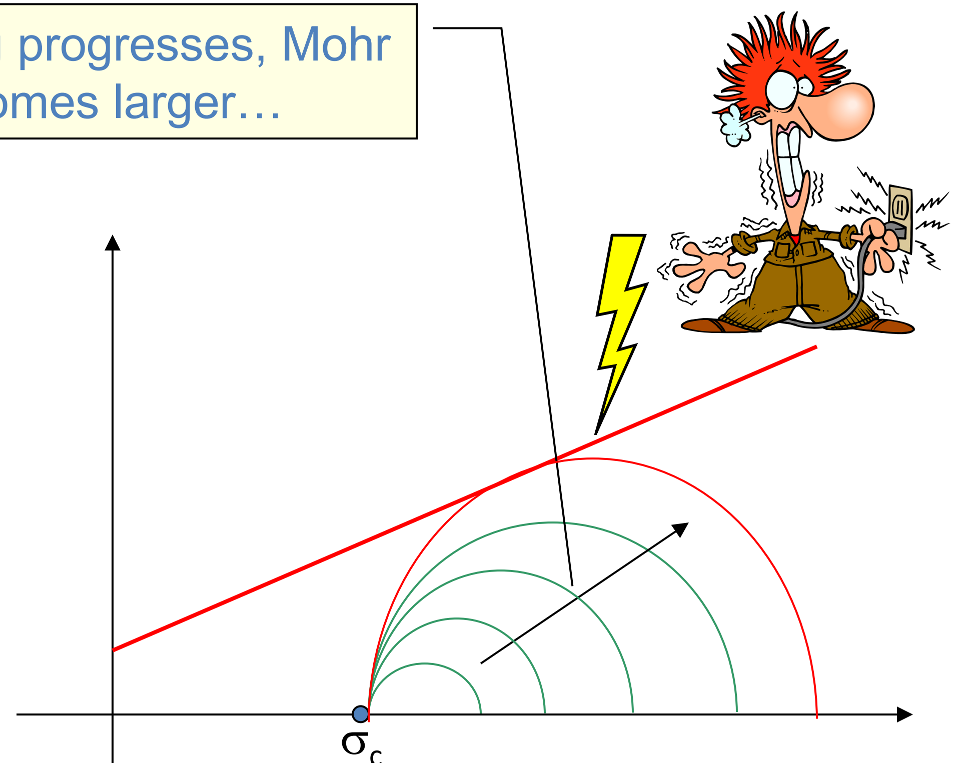


Initially, Mohr circle is a point

Mohr Circles & Failure Envelope

As loading progresses, Mohr circle becomes larger…

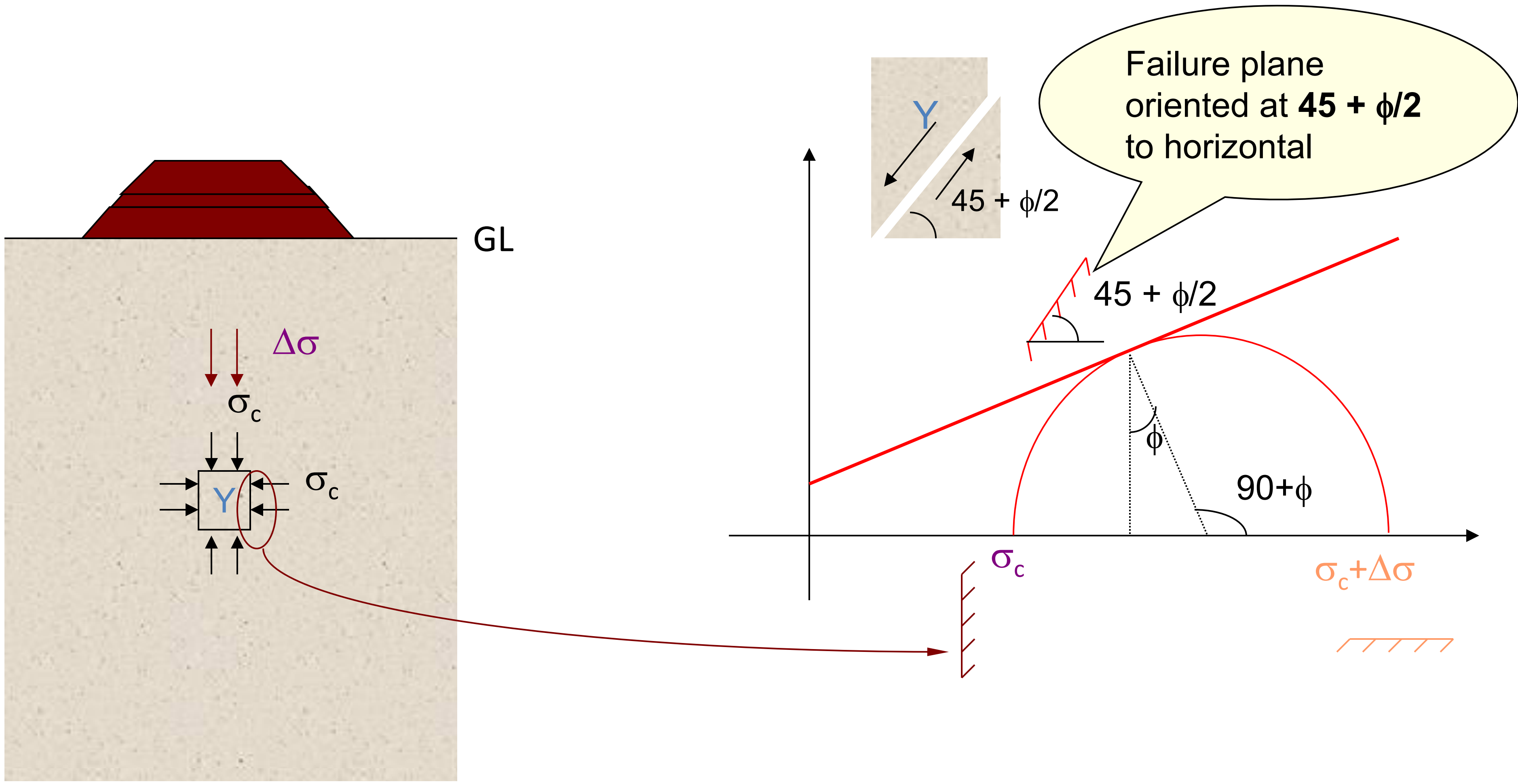




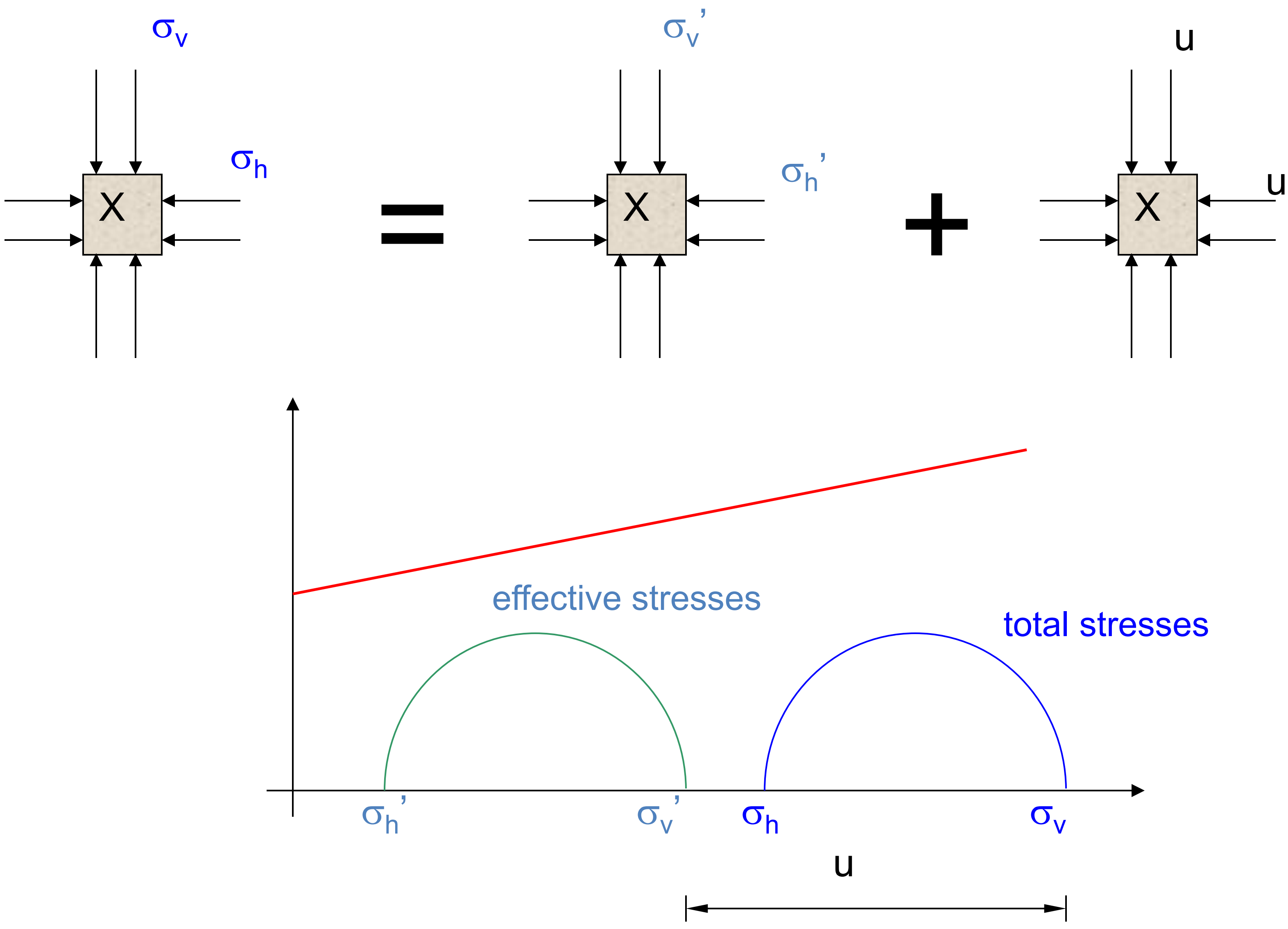
Keeping σ_3 constant, if vertical stress (σ_1) increases the Mohr Circle becomes larger and

finally it will touch the failure envelop and failure will take place

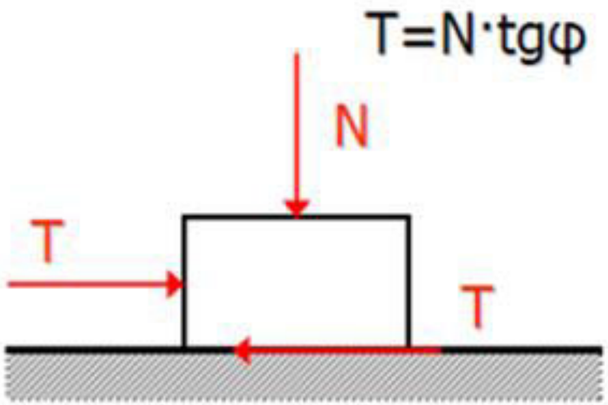
Orientation of Failure Plane



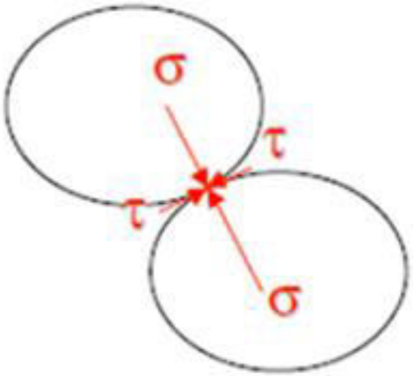
Mohr circles in terms of σ & σ'

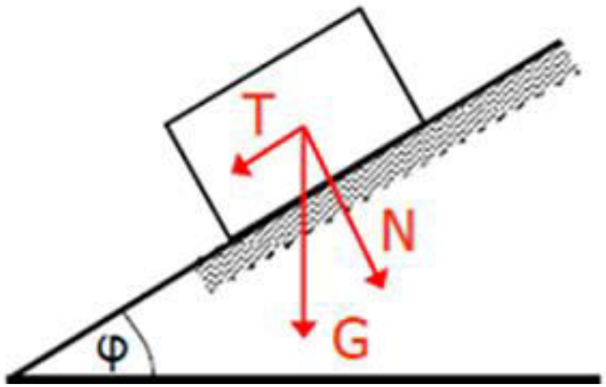


Shear Strength of Granular soils



$\tau = \sigma \cdot \tan\varphi$





For soils this φ angle is called:

angle of internal friction or

friction angle

Angle of repose = φ

Shear Strength of Fine Grained Soils

Their strength is, apart from from friction, due to internal forces holding the particles together

This propertiy is called cohesion, and soils possessing it are cohesive soils

Coulomb's law extended to cohesive soils:

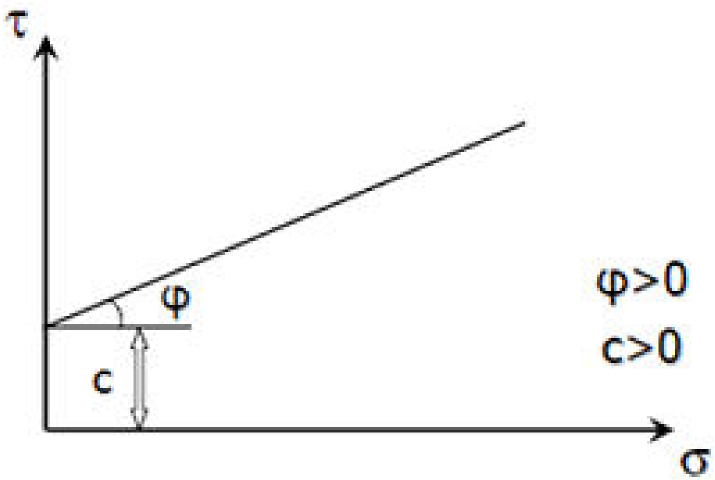
$$\tau = \sigma \cdot \tan\varphi + c$$

In case of saturated soils this can be expressed as:

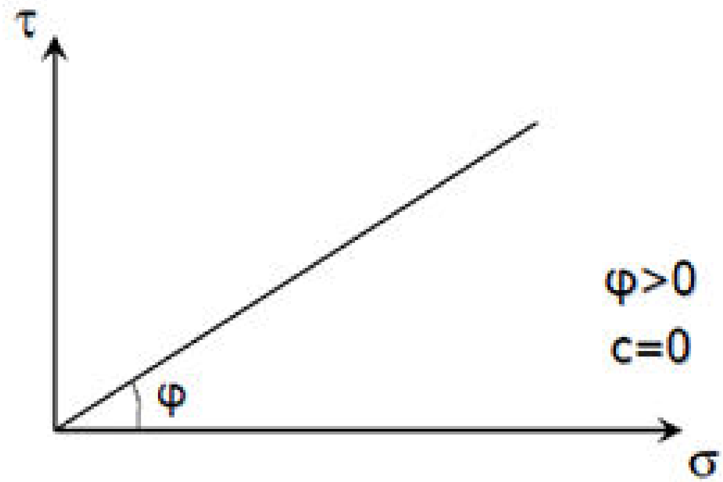
$$\tau = (\sigma - u) \cdot \tan\varphi + c$$

Graphical represantation of Mohr Coulomb failure criteria

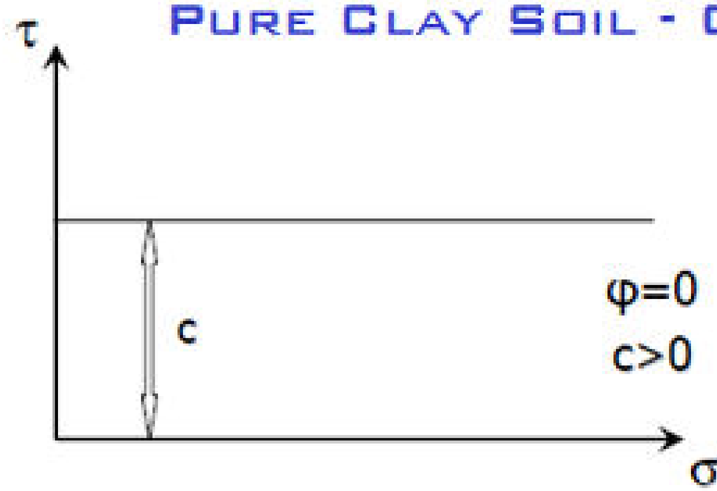
GENERAL CASE C - φ soils



GRANULAR SOIL - φ soils



PURE CLAY SOIL - C SOIL



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. Un confined Compression (UCC) Test
4. Vane Shear Test

Field tests

1. Torvane
2. Pocket penetrometer
3. Fall cone
4. Pressuremeter
5. Static cone penetrometer
6. Standard penetration test
7. Field Vane Shear Test

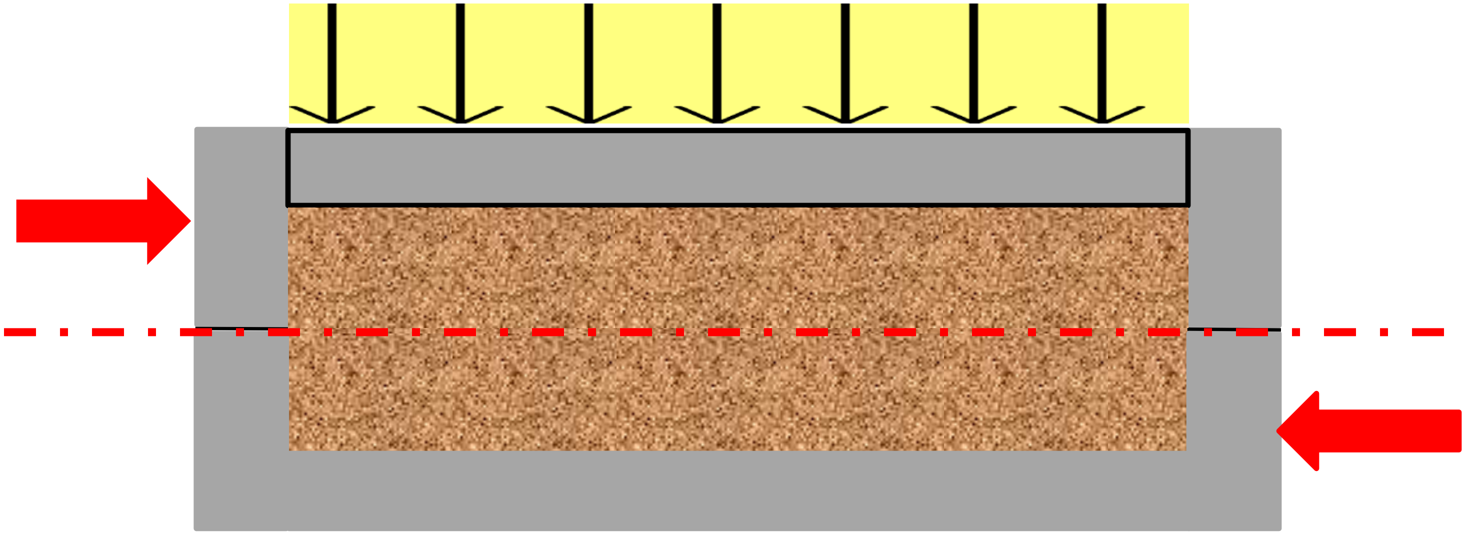
DIRECT SHEAR TEST

Can be performed on all types of soil, moist or dry.

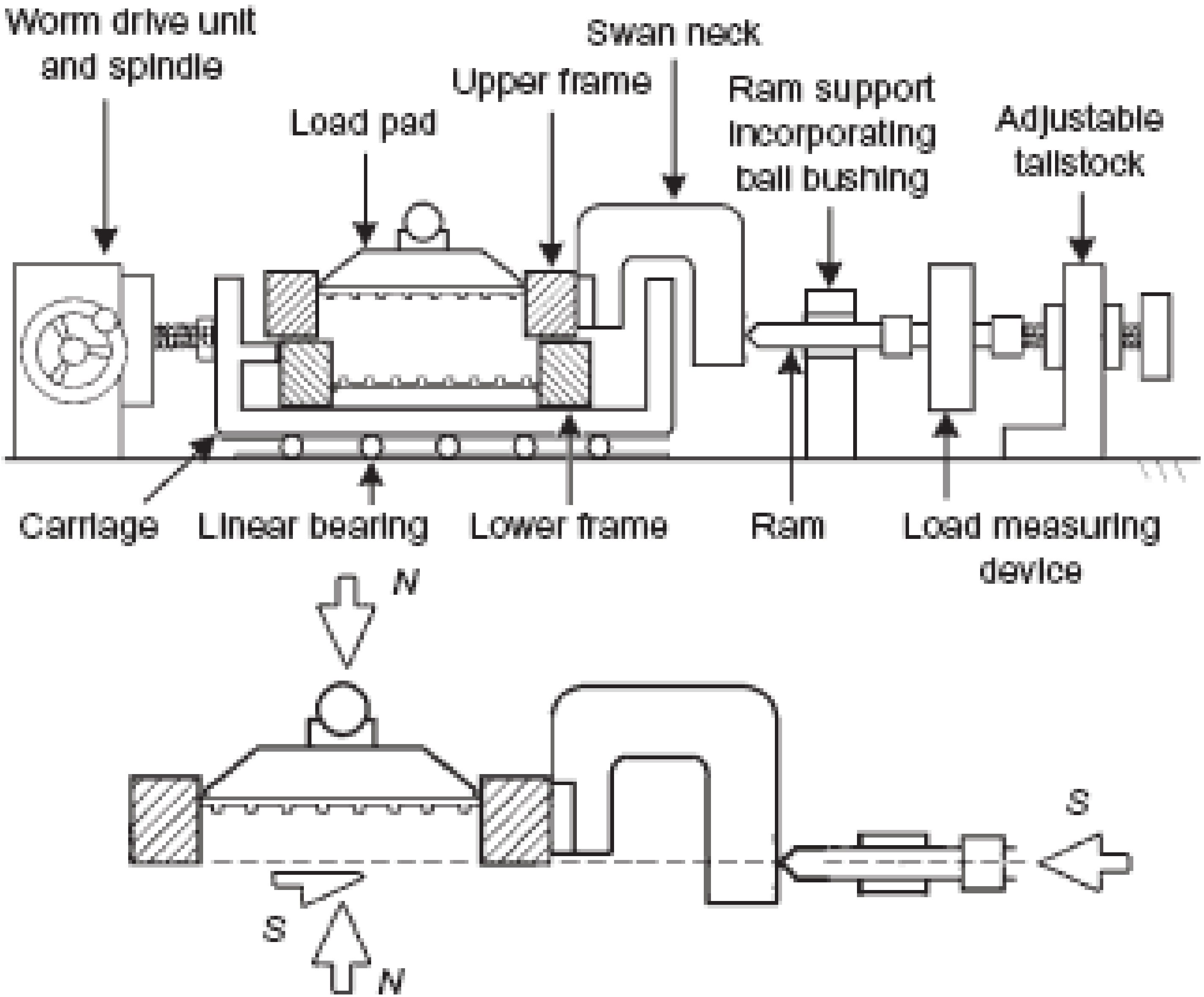
Measures shear stress at failure on failure plane

for various normal stresses.

DIRECT SHEAR TEST



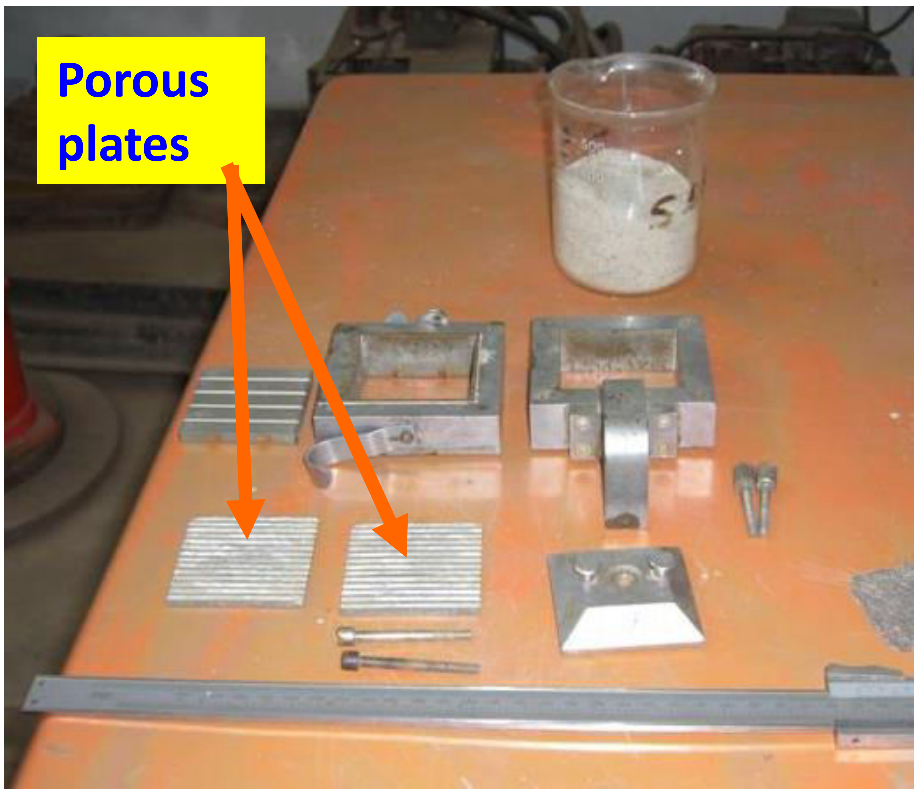
Direct shear test



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



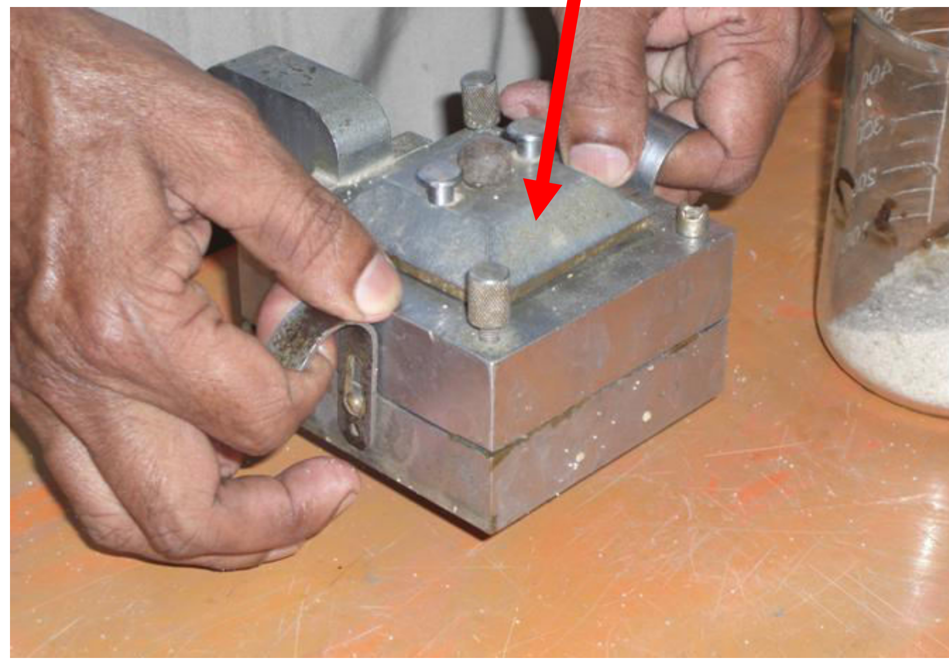
Components of the shear box

Preparation of a sand specimen

Direct shear test

Preparation of a sand specimen

Pressure plate

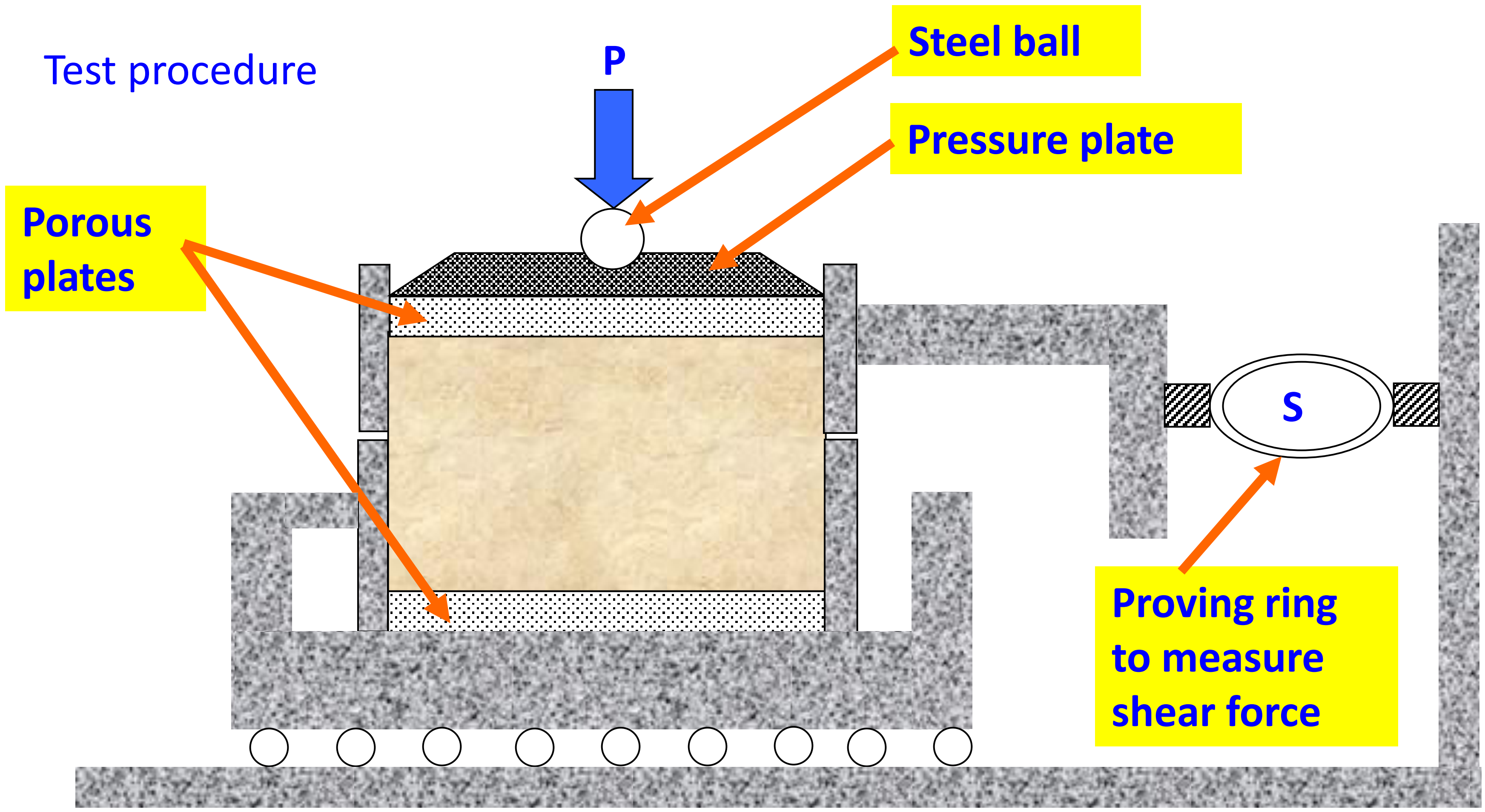


Leveling the top surface of specimen

Specimen preparation completed

Direct shear test

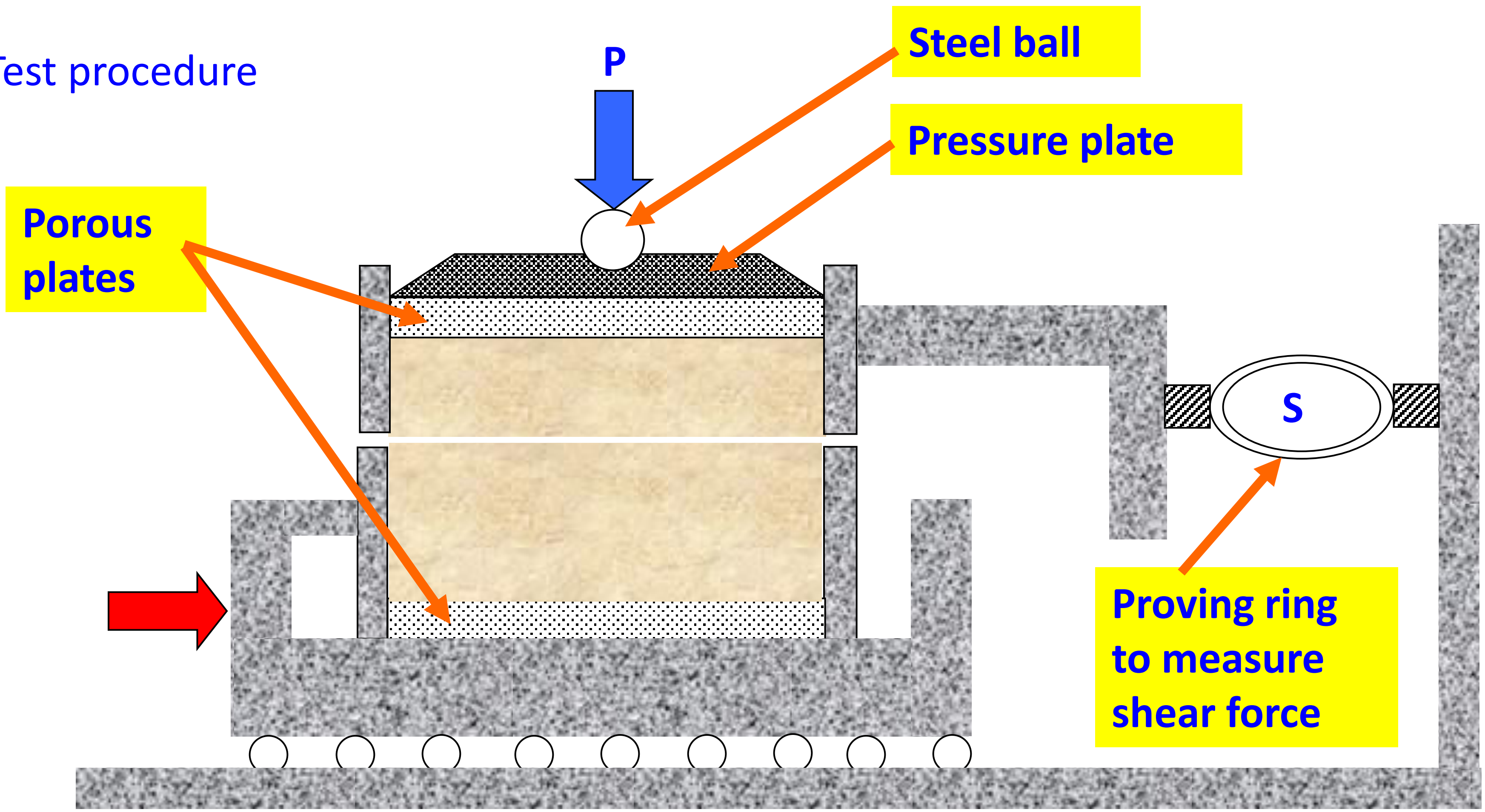
Test procedure



Step 1: Apply a vertical load to the specimen and wait for consolidation

Direct shear test

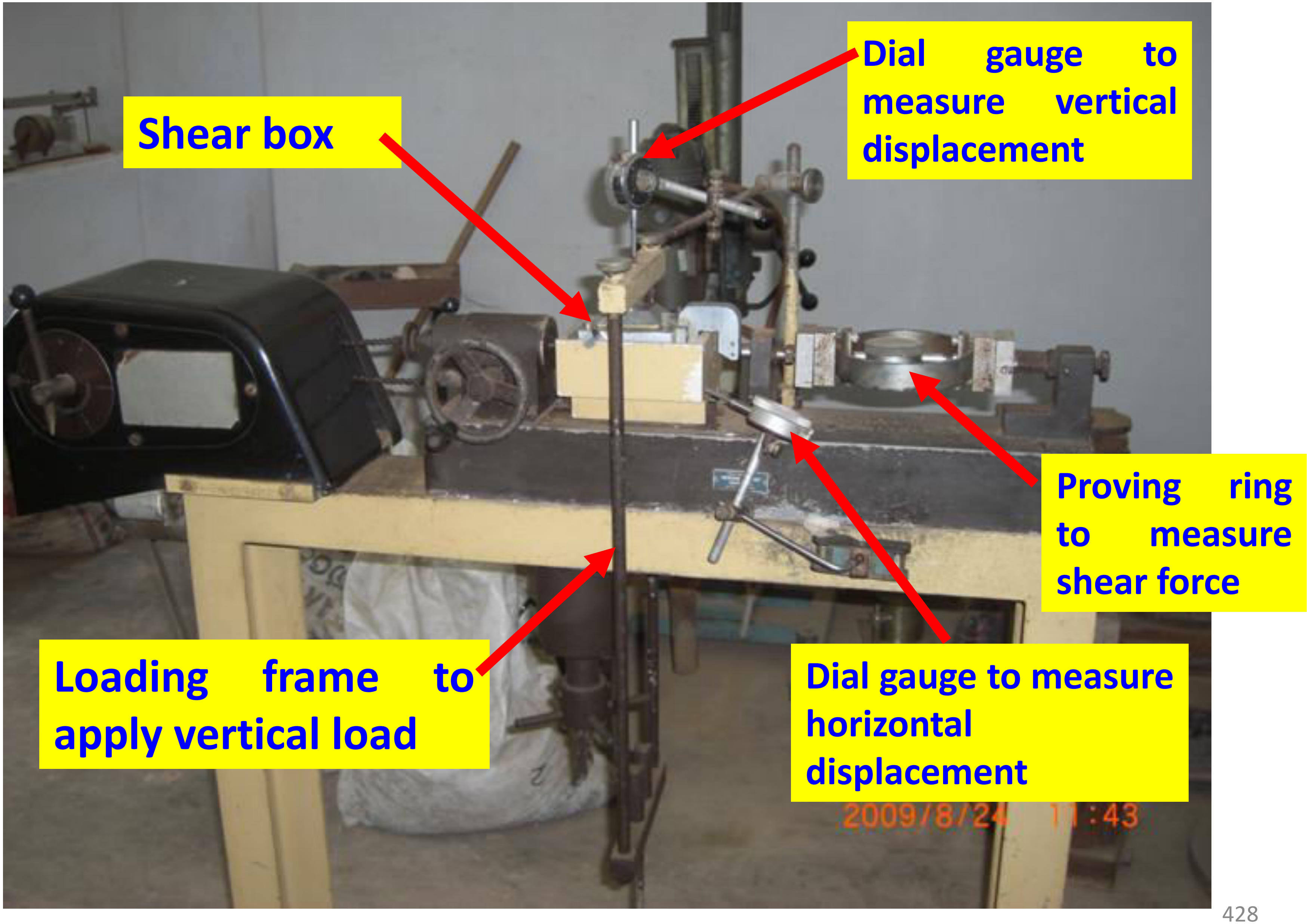
Test procedure



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

Direct shear test



A shear box has three parts:

a top extension, Bottom base and a normal load piston

The prepared soil sample is placed in the box

A normal (90° to the horizontal) load is applied to the soil

Then the top and base are pushed in opposite directions

Due to this operation, failure occur on a horizontal plane between the top and base

The horizontal force is increased until the sample shears in two halves

The procedure is repeated two more times using successively heavier normal loads.

The inside dimensions of the shear box are 60 mm by 60 mm

This means the failure plane has an area of 3600 mm^2.

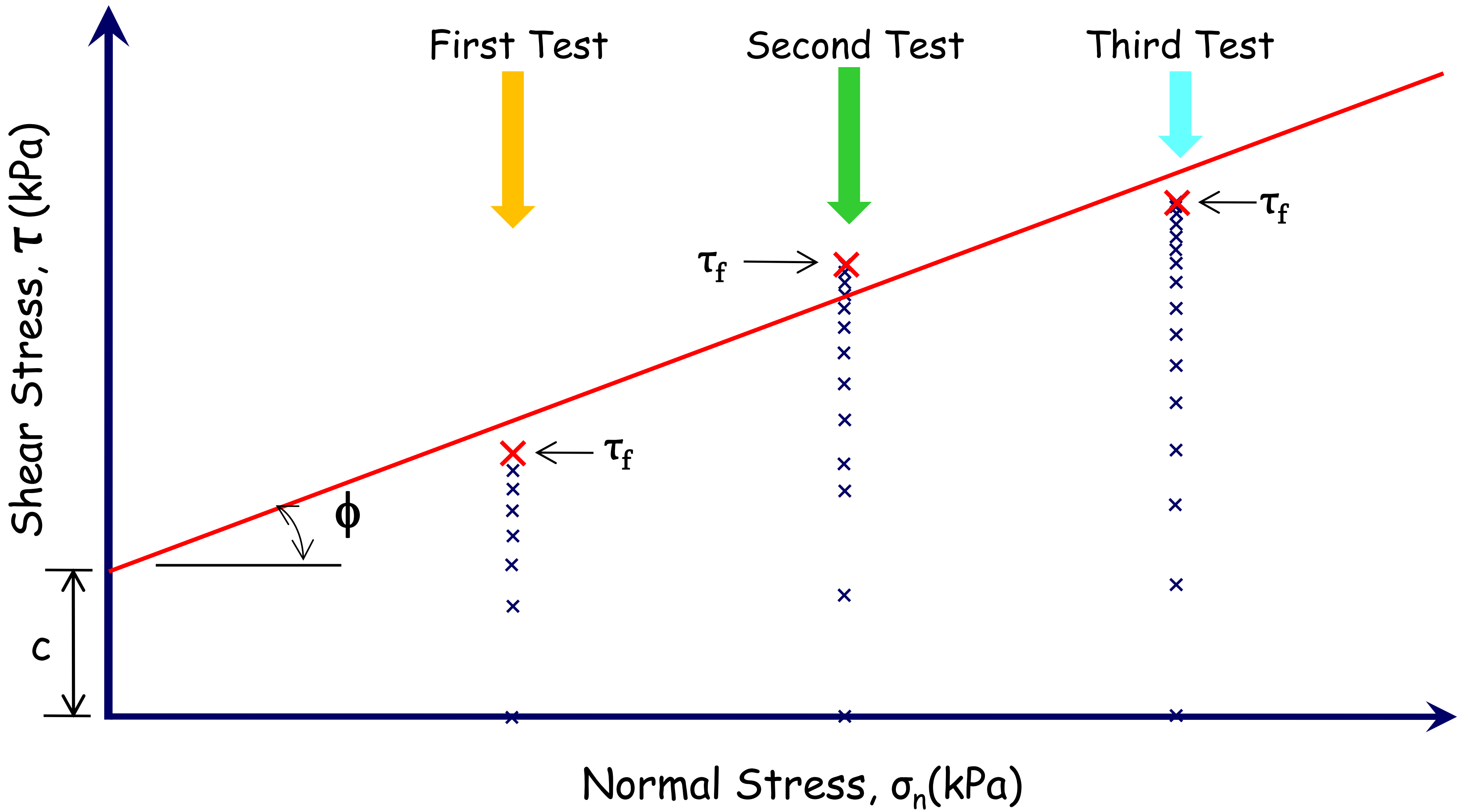
The shear force at failure (maximum) and normal load, are divided by

this plane area to find

the shear stress and the normal stress at failure in MPa.

The shear force required to shear the sample increases in proportion to the normal load.

The shear strength of the soil therefore is not constant but changes with the confining pressure



First Test
Second Test
Third Test
τ_f
τ_f
τ_f
ϕ
c
Shear Stress, τ (kPa)
Normal Stress, σ_n(kPa)

Plotting the shear stress versus normal stress

Fitting a best fit line through these points:

The slope angle of this line is
the angle of internal friction, ϕ of the soil.

The τ axis intercept is
the apparent cohesion, c of the soil :

Direct shear test

Analysis of test results

$$\sigma = \text{Normal stress} = \frac{\text{Normal force (P)}}{\text{Area of cross section of the sample}}$$

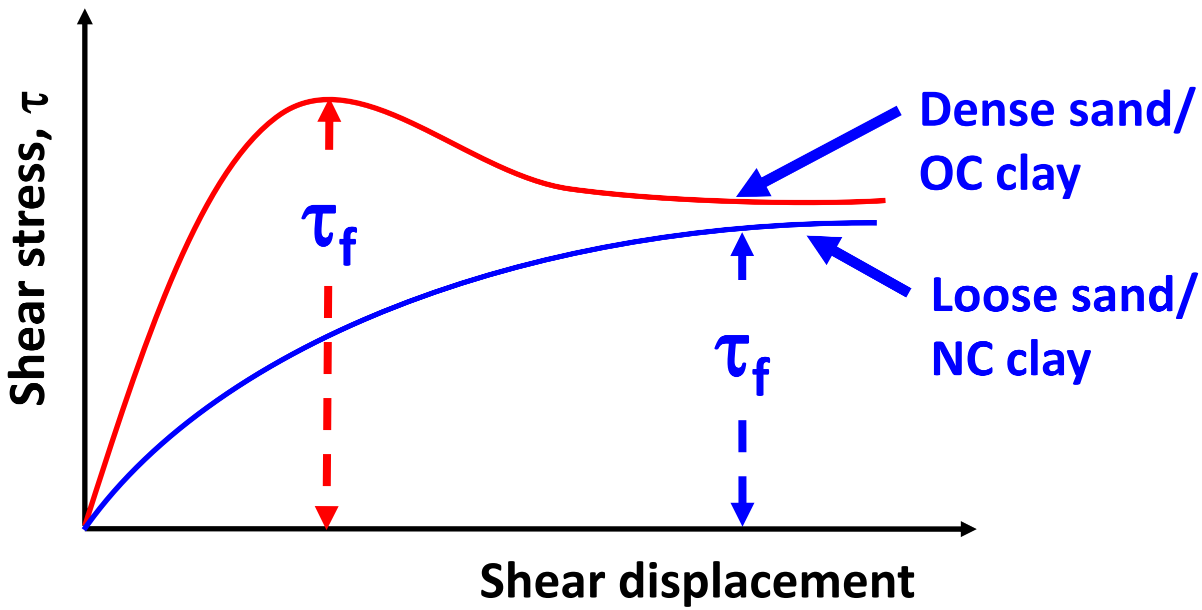
$$\tau = \text{Shear stress} = \frac{\text{Shear resistance developed at the sliding surface (S)}}{\text{Area of cross section of the sample}}$$

we have an estimate of Coulomb's failure envelope

The equation of Coulomb's failure envelope:

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

Direct shear tests on sands Stress-strain relationship



Direct shear tests on sands

Some important facts on strength parameters c and ϕ of sand

Sand is cohesionless hence $c = 0$

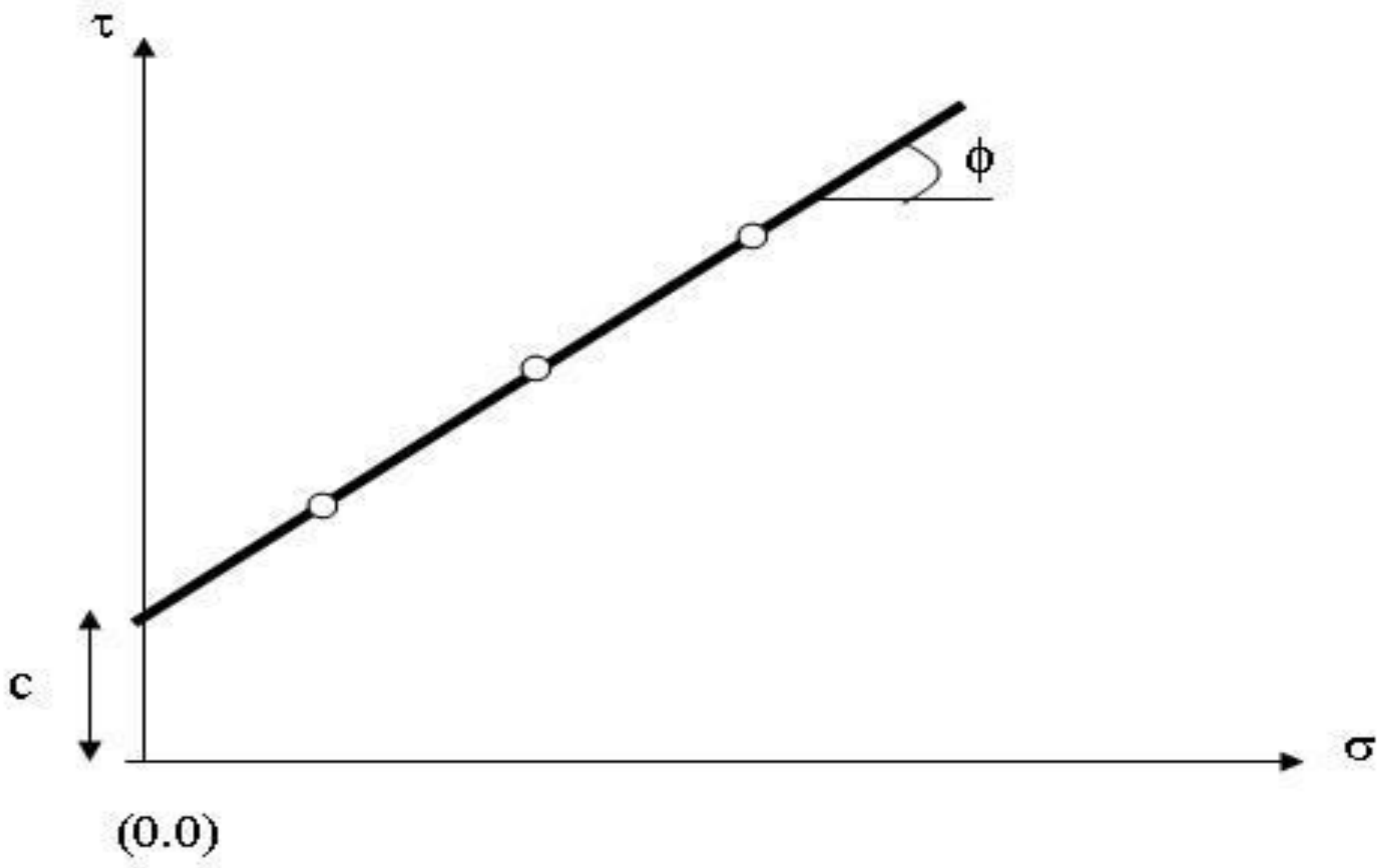
Direct shear tests are drained and pore water pressures are dissipated, hence $u = 0$

Therefore,

$\phi' = \phi$ and $c' = c = 0$

A saturated compacted gravel was tested in a large shear box, 300 mm x 300 mm in plan. What properties of the gravel can be deduced from the following results?

Normal load (N)	Peak Shear Load (N)
4500	4500
9200	7890
1380	11200


τ
φ
c
σ
(0.0)

Advantages and Disadvantages of Direct Shear test

Advantages

Simplest test

most economical for sandy soil

Quick test ie Time consumed is less

Disadvantages

Soil not allowed to fail along the weakest plane.

Shear stress distribution is not uniform.

TRIAXIAL COMPRESSION TEST

Can be performed on all types of soil, moist or dry and

can consolidate sample to in situ conditions by tracking pore water pressures.

Measures vertical stress applied to soil sample and confining pressure.

Shear stress on failure plane must be calculated from principal stresses.

TRIAXIAL COMPRESSION TEST

The usual sizes of the samples are: 76 mm (length) x 38 mm (diameter)

or

100 mm (length) x 50 mm (diameter).

Thus,

the length/diameter ratio of the cylindrical sample is 2.

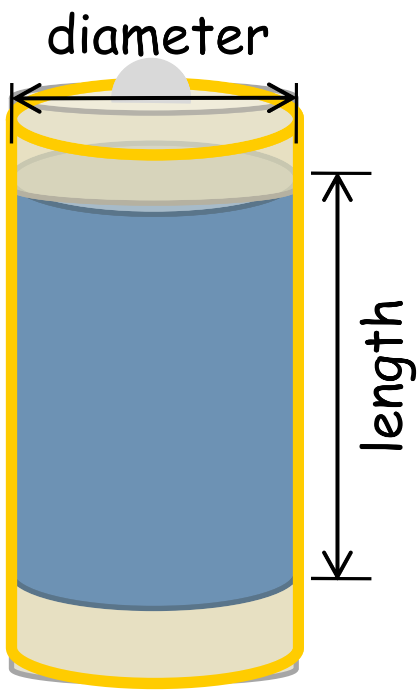
This test is suitable for both sand and clay.

TRIAXIAL COMPRESSION TEST

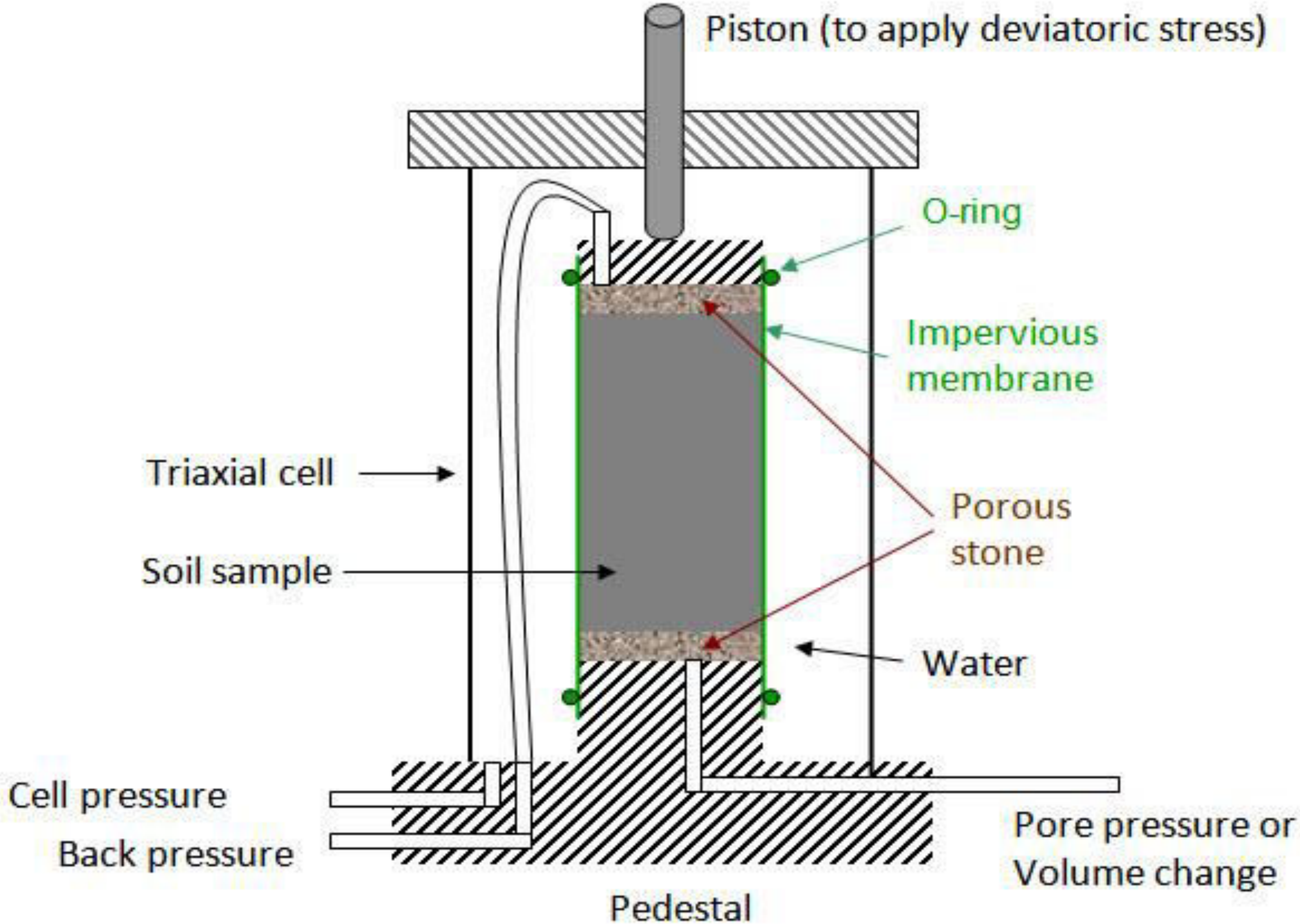
Cylindrical specimens are prepared from sampled soil

Specimens are weighed and dimensions measured first

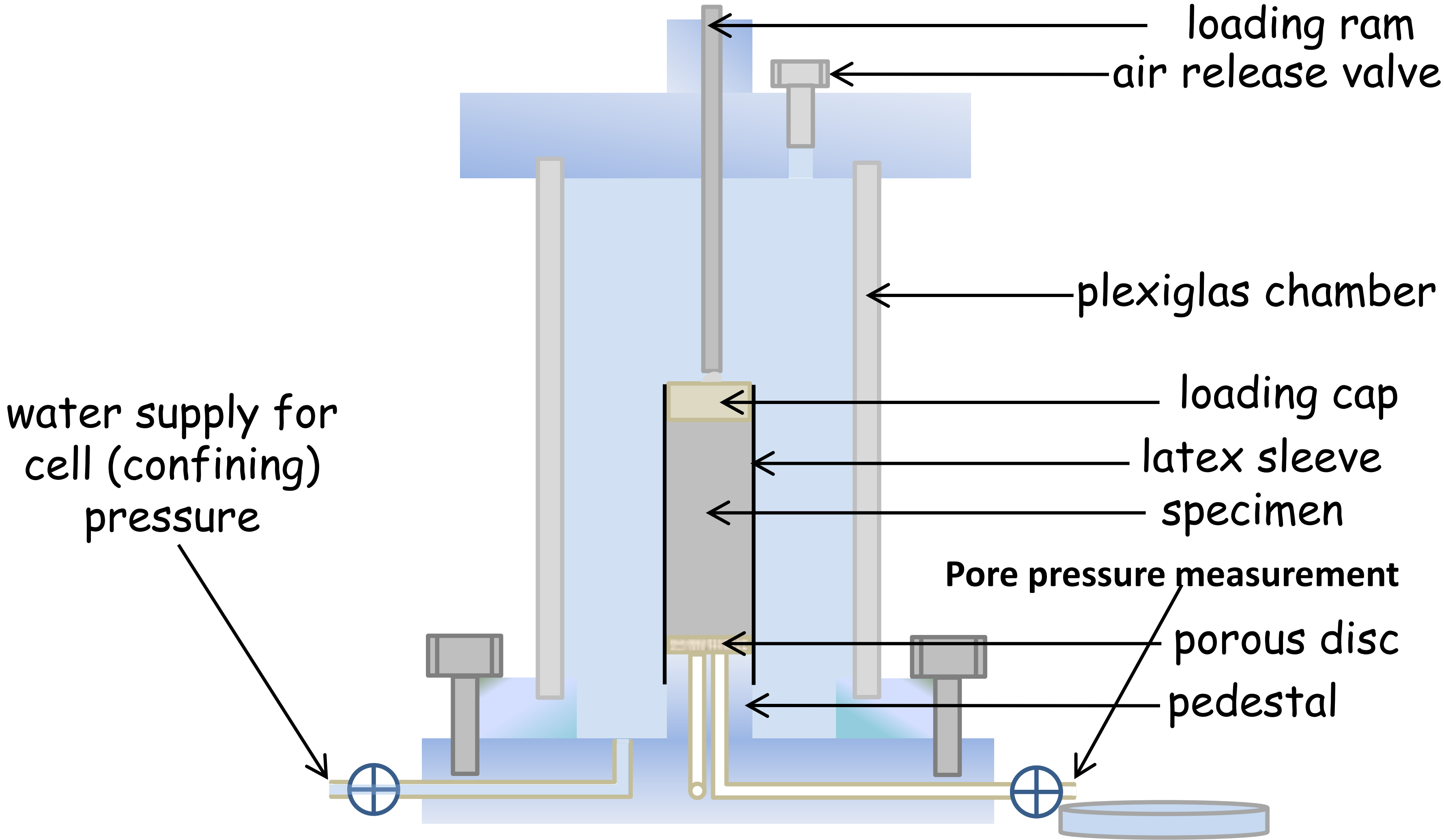
The specimen is then placed in a plexi glass chamber



The specimen is mounted between 2 platens and then inserted into a latex sleeve.

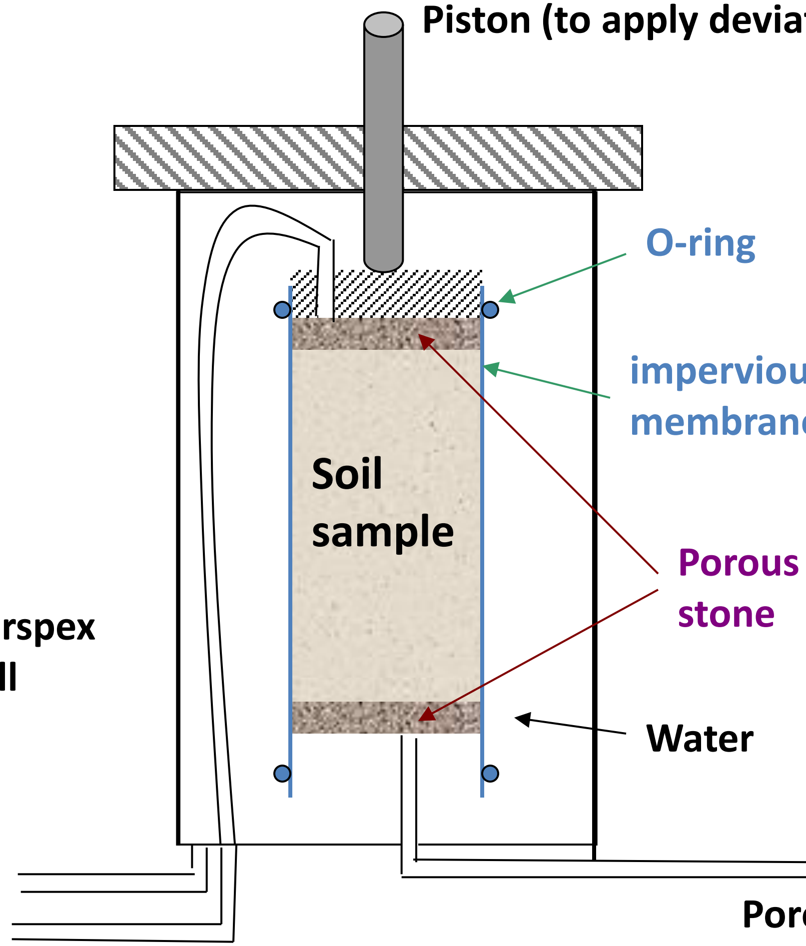

Piston (to apply deviatoric stress)
O-ring
Impervious membrane
Triaxial cell
Porous stone
Soil sample
Water
Cell pressure
Back pressure
Pore pressure or Volume change
Pedestal

TRIAXIAL COMPRESSION TEST



Triaxial Shear Test

Soil sample at failure



Triaxial Shear Test

Specimen preparation (undisturbed sample)

Sampling tubes

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

The specimen is mounted on the pedestal of the chamber

Then the chamber is placed on the base and locked into place.

The assembly is then mounted on the compression testing machine.

For a drained test the drain valve is opened and pore water collected

For an un-drained test, the drain valve is closed.

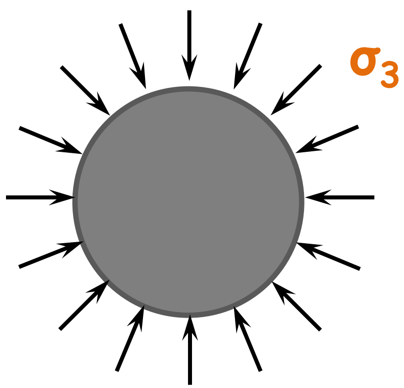
Water is forced into the cell
with the supply valve open as well as the air release valve

Once the cell is filled with water,
the air release valve is closed and

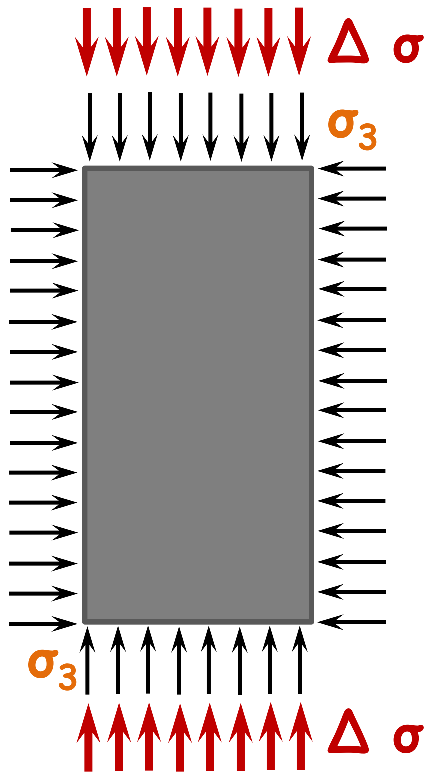
the cell pressure is increased to
the desired value for the test.

TRIAXIAL COMPRESSION TEST

The effect of the cell pressure on the specimen is illustrated below:



Plan View of Specimen



Side View of Specimen

$$\sigma_1 = \Delta\sigma + \sigma_3$$

The cell pressure, σ_3, is also known as the Minor Principal Stress.

Then a vertical axial load is applied to the loading ram creating compressive stresses or the deviator stress $\Delta\sigma$:

The Major Principal Stress, σ_1, is the combination of the deviator stress and cell pressure:

find τ_f and σ_f from σ_1 and σ_3 by preparing a Mohr's circle!

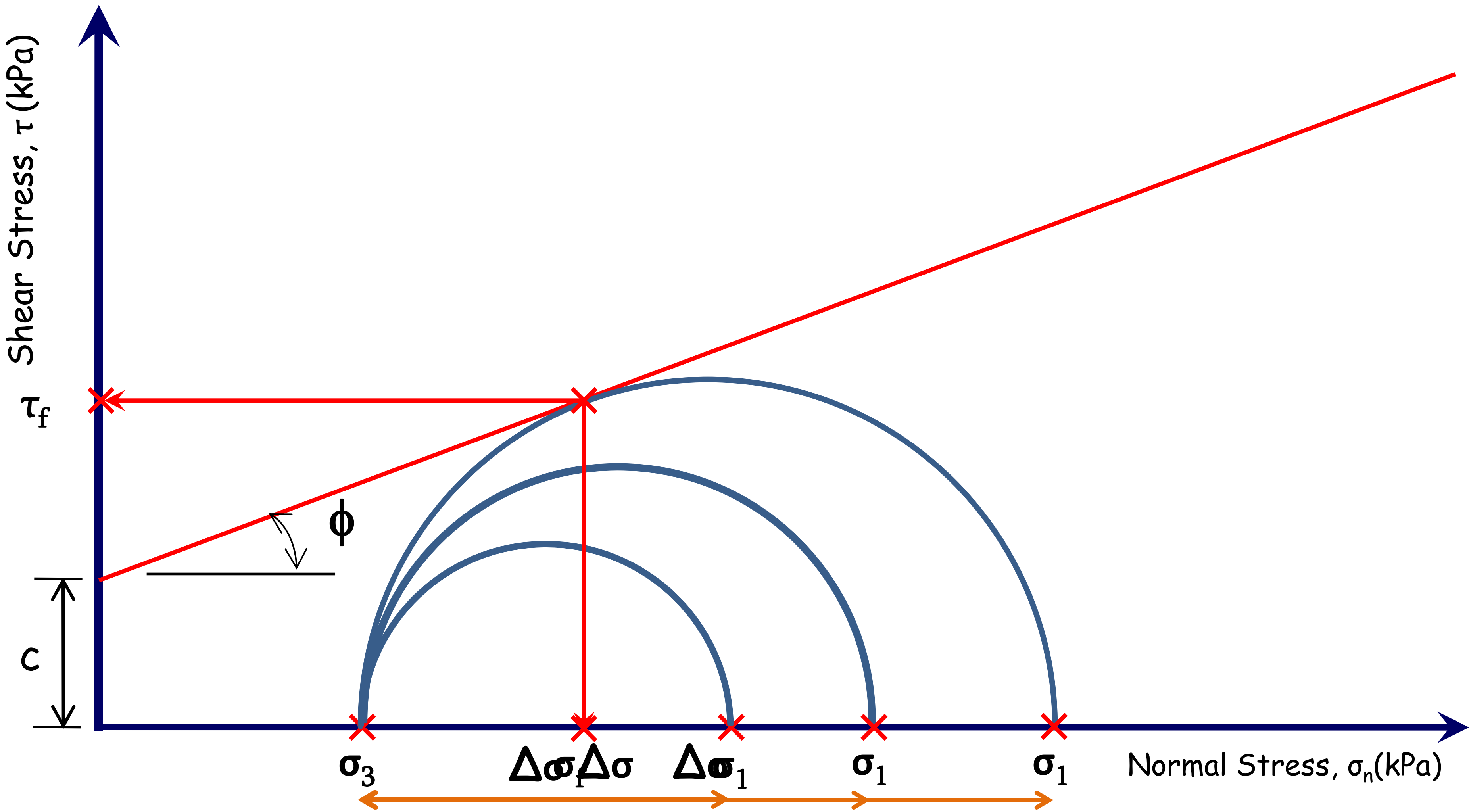
plot σ_1 and σ_3 on the σ_n axis

During the test, this circle starts as one point at σ_3 and then grows to the right as axial stress, $\Delta\sigma$ increases but σ_3 remains constant.

Ultimately, the test ends when shear failure occurs and the circle has become tangent to the failure envelope.

The point of tangency of the circle and failure envelope defines the shear strength, τ_f and normal stress, σ_f.

TRIAXIAL COMPRESSION TEST



But how do we find the failure envelope from a triaxial compression test?

Geometrically, you need at least two circles in order to define a line tangent to both.

This means that you need to perform the test at least twice on the same material but at different cell pressures.

But how can you be sure one of them isn't bogus?

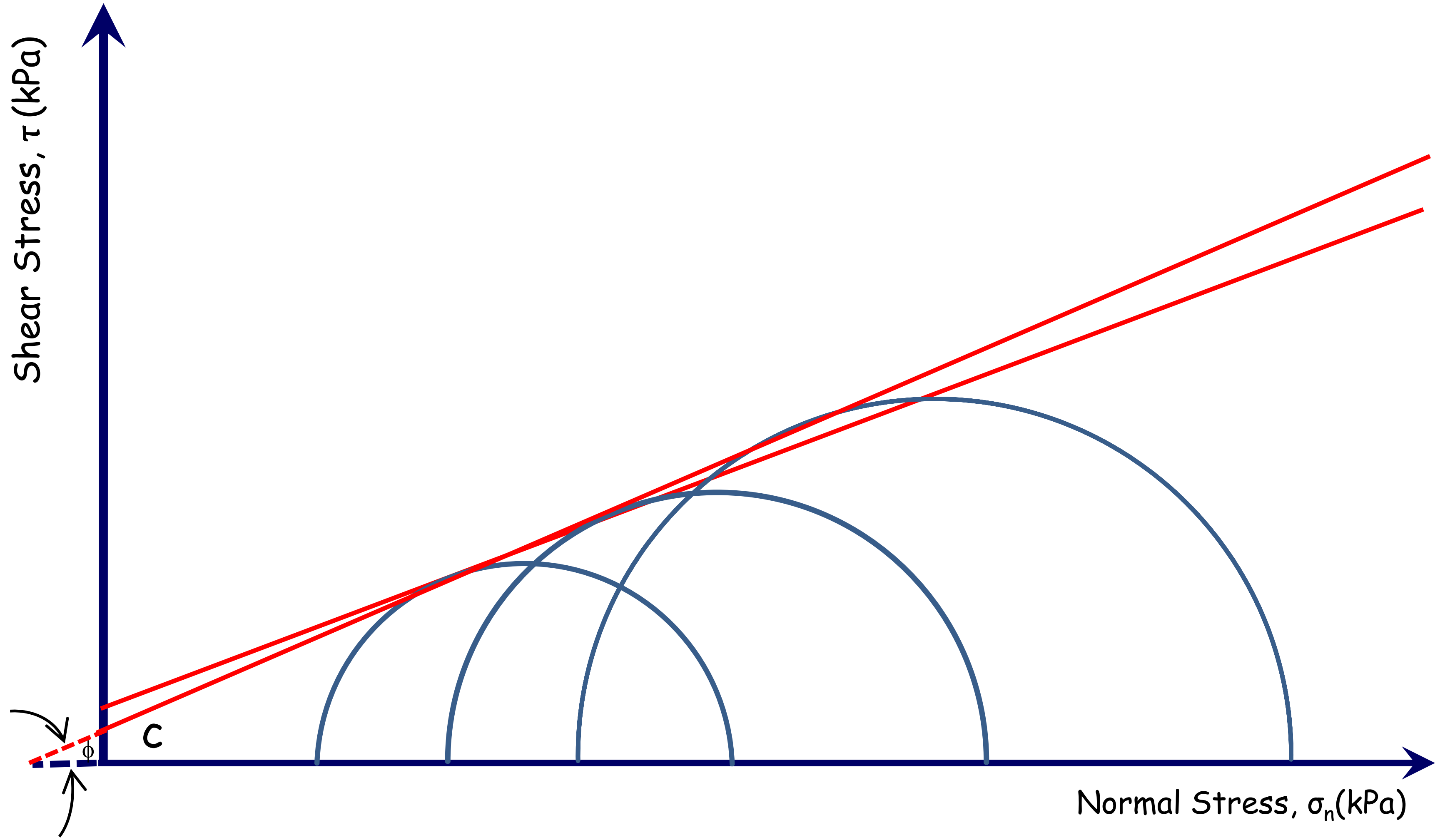
A third test at yet another cell pressure would help to confirm the validity of the failure envelope.

As with most lab measurements, the ideal (one line tangent to all three circles) is difficult to achieve.

Therefore

a best fit is made as long as one circle is not out to compared to the others.

TRIAXIAL COMPRESSION TEST



Instead of doing this graphically, we can use trigonometry to find equations for τ_f and σ_f using the angle of the failure plane, Θ and the values of σ_1 and σ_3

The Centre of the Mohr's Circle, C is then:

$$C = \frac{\sigma_1 + \sigma_3}{2}$$

Remember the deviator stress, $\Delta\sigma = \sigma_1 - \sigma_3$, which is the diameter of the Mohr's Circle

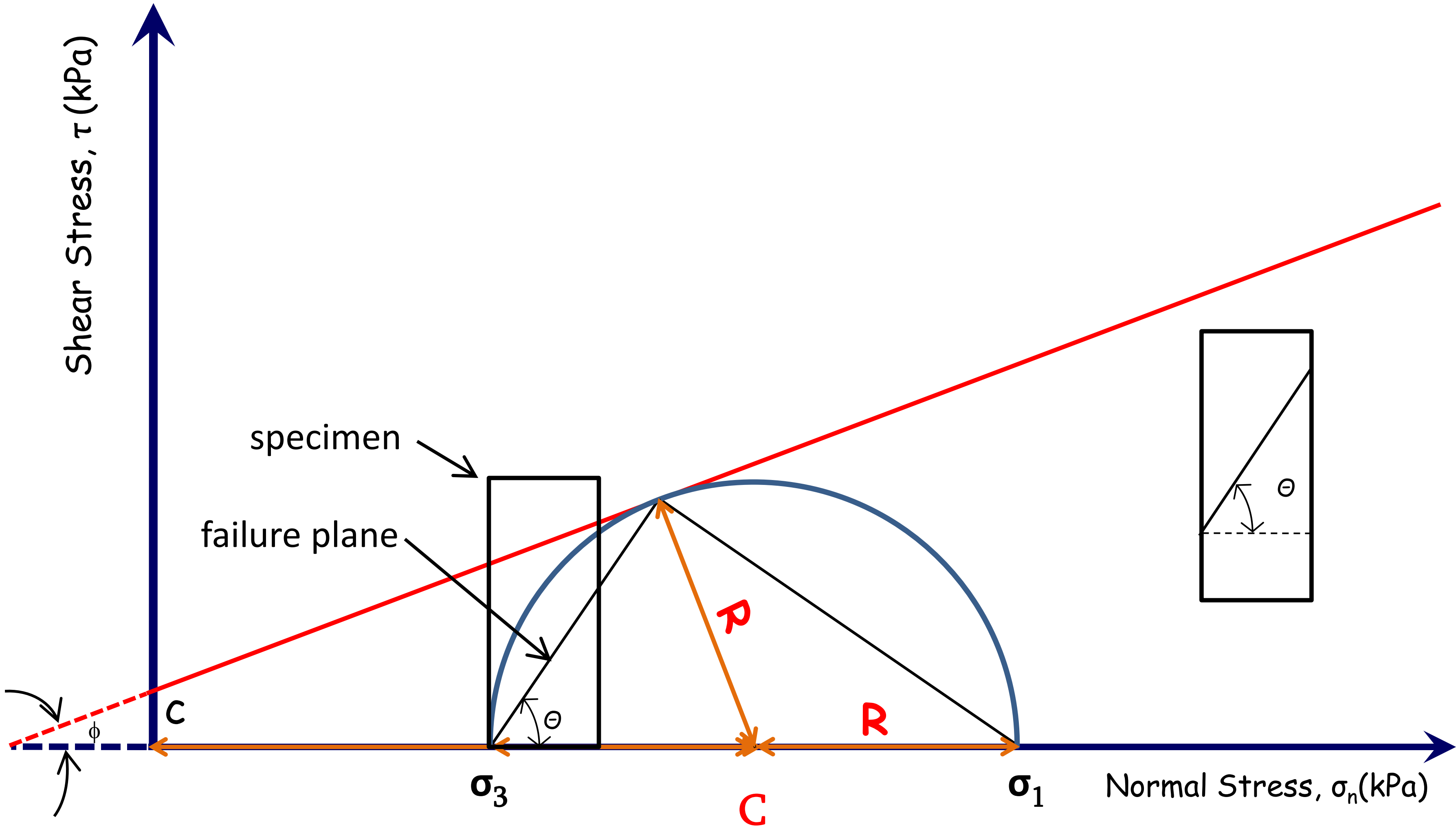
So the radius of the Mohr's Circle, R is half the diameter or:

$$R = \frac{\sigma_1 - \sigma_3}{2}$$

then for each test, the shear strength, τ_f and normal stress, σ_f can be found.

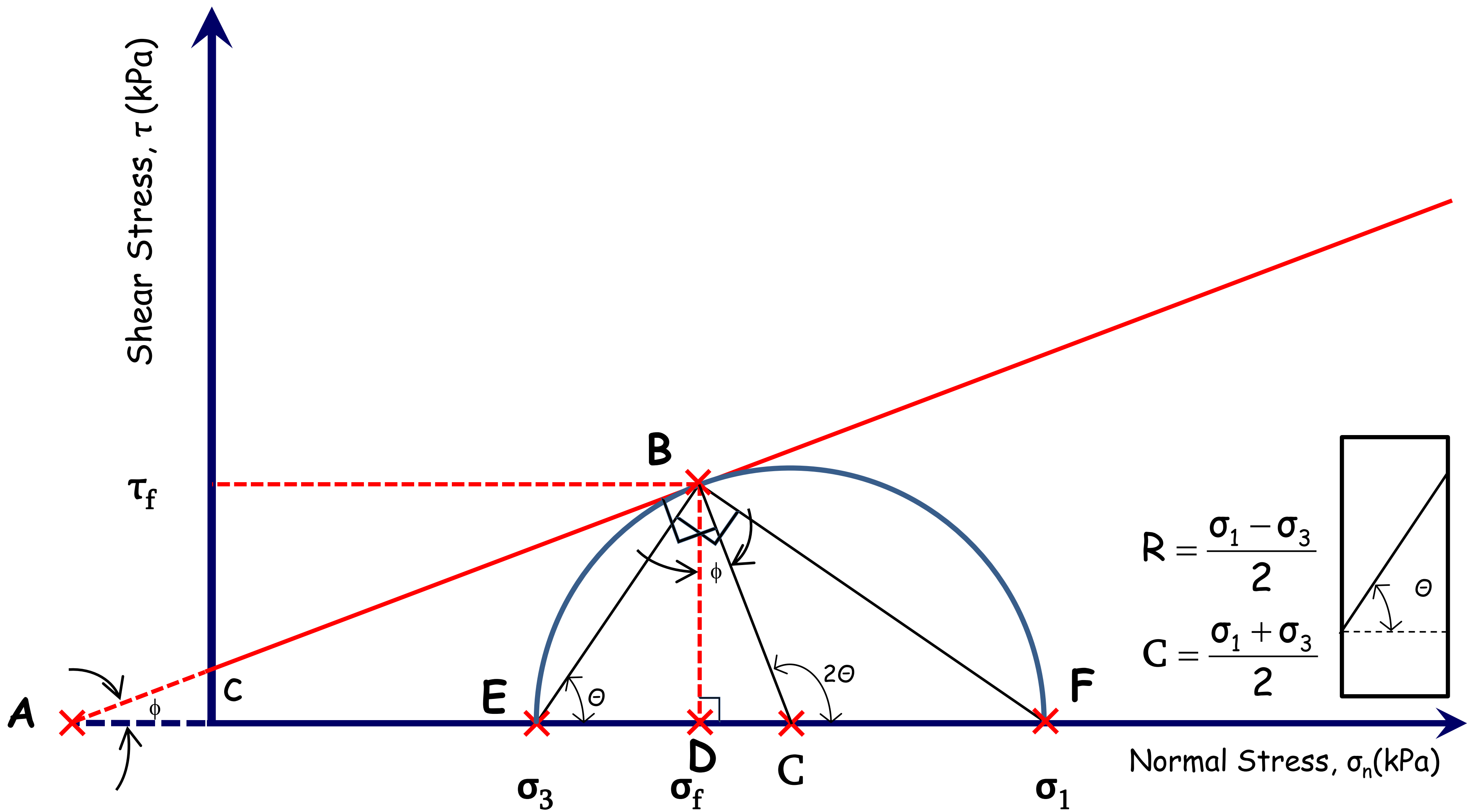
Once we have the shear strength parameters, ϕ and c defining the failure envelope,

TRIAXIAL COMPRESSION TEST



TRIAXIAL COMPRESSION TEST

To follow the trig we label the vertices:



$$\theta = 45^\circ - \frac{\phi}{2}$$

using Θ and the σ_1 & σ_3 values for each trial, τ_f and σ_f can be found for each trial

$$\sigma_f = \frac{1}{2}(\sigma_1 + \sigma_3) + \frac{1}{2}(\sigma_1 - \sigma_3)\cos 2\theta \quad \text{(Eqn. 4.4)}$$

$$\tau_f = R\sin(180^\circ - 2\theta) = \frac{1}{2}(\sigma_1 - \sigma_3)\sin 2\theta$$

UNDRAINED TEST

[when the pore water is not allowed to drain]

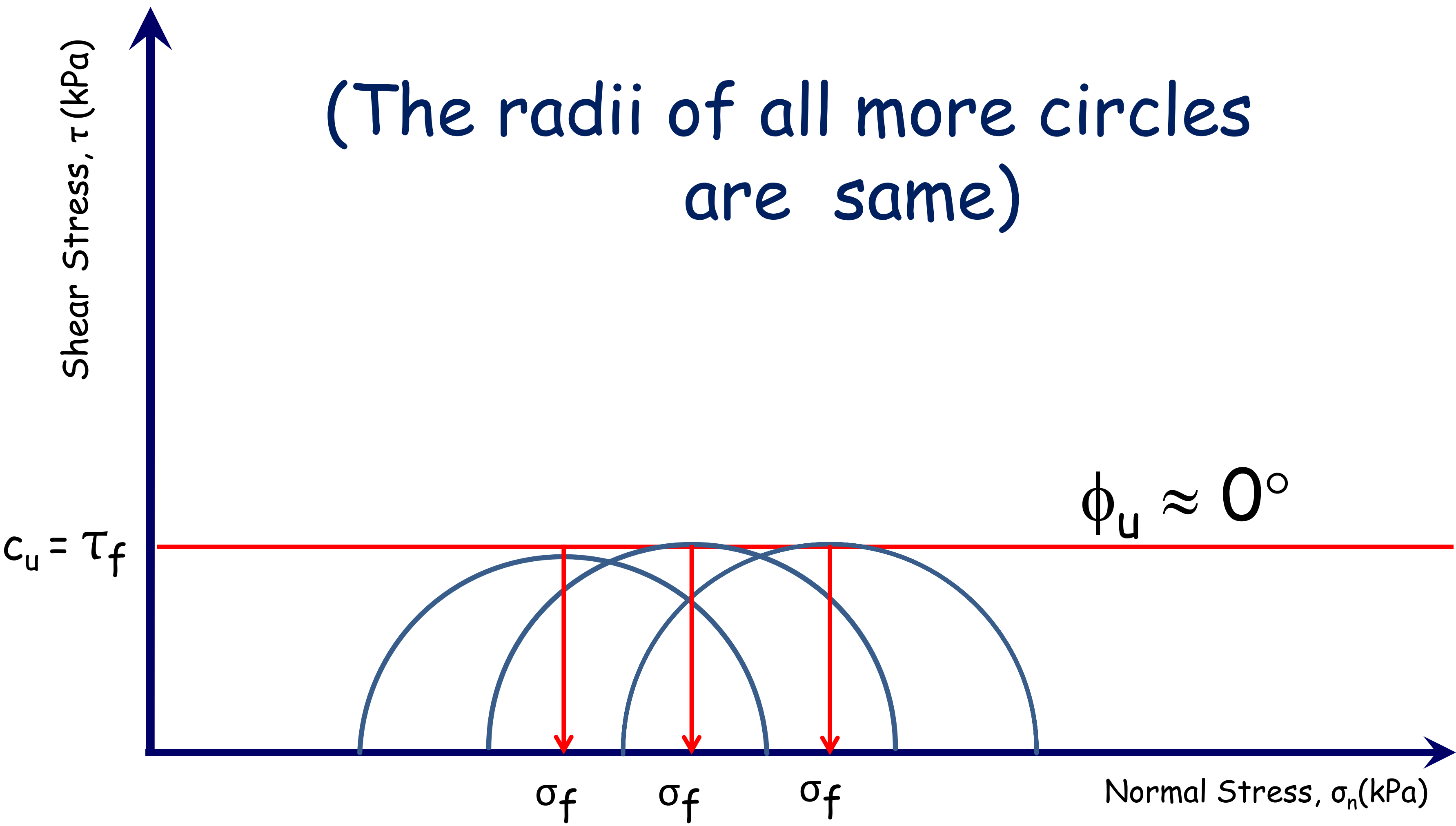
As the external pressure increases,

the internal pore water pressure increases

Therefore, the failure envelope is

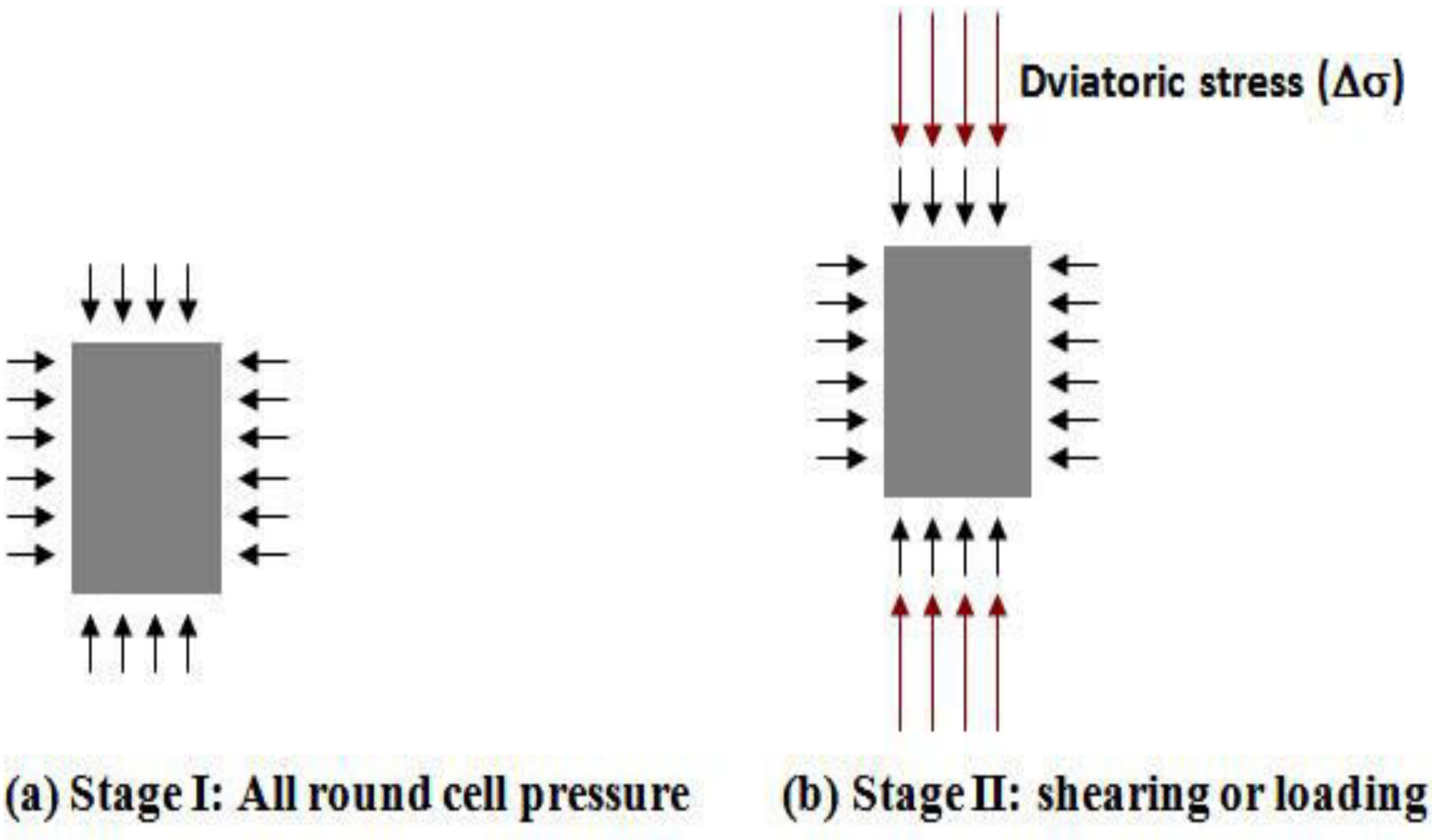
typically a horizontal line and $\phi_u = 0°$.

Typically,
the deviator stress at failure is
fairly constant
for each different cell pressure.
And,
the apparent cohesion, c_u
will be the same
for each trial and
equal to the shear strength, τ_f
The normal stress, σ_f for each trial
will then be $\sigma_3 + c_u$



Shear Stress, τ (kPa)
(The radii of all more circles are same)
$\phi_u \approx 0°$
$c_u = \tau_f$
σ_f
σ_f
σ_f
Normal Stress, σ_n(kPa)

Stages of Tri – Axial Shear Test



Tri-axial tests are conducted in two stages

Stage I: under all round cell pressure (σ_3) Application)

The all round cell pressure is applied by using water inside the tri-axial cell.

if drainage is allowed

the consolidation takes place in the sample then

it is called consolidated sample.

if drainage is not allowed then

the sample is called unconsolidated sample

Stage II: under shearing or loading Deviator stress, $\Delta\sigma_d$ Application

if drainage is allowed then

loading is called drained loading.

if drainage is not allowed then

the loading is called un-drained loading

The Consolidation and drainage in the sample is controlled by closing or opening the drainage valve.

Types of Tri-axial Tests

Depending on

Whether consolidation and drainage is allowed or not

during both stages

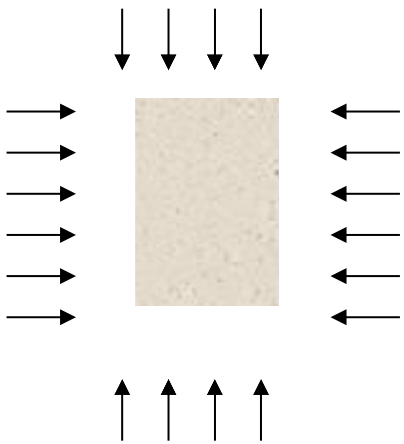
three types of tri-axial tests are conducted:

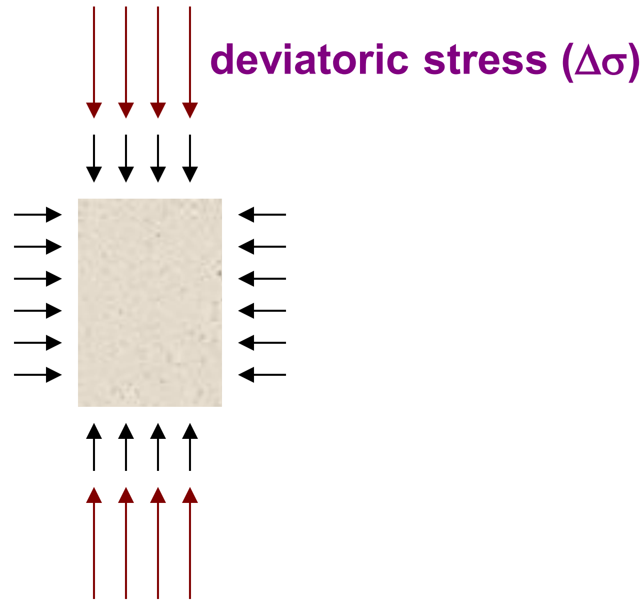
(i) Unconsolidated Un-drained (UU) test

(ii) Consolidated Un-drained (CU) test

(iii) Consolidated Drained (CD) test

Types of Tri axial Tests





Under all-around cell pressure σ_c

Is the drainage valve open?

- **yes** → **Consolidated** sample
- **no** → **Unconsolidated** sample

Shearing (loading)

Is the drainage valve open?

- **yes** → **Drained** loading
- **no** → **Undrained** loading

Consolidated Drained (CD) Test

❖ **no excess pore pressure throughout the test $\Delta u = 0$**

❖ **very slow shearing to avoid build-up of pore pressure**

Can be days!

∴ not desirable

❖ **c′ and ϕ′ are determined**

Test results are used for analysing long term stability of the structure

Consolidated Un drained (CU) Test

- **pore pressure develops during deviator stress application** **and it is also measured**

- **c′ and ϕ′ are determined**

- **faster than CD**

- **($\therefore$ preferred way to find c′ and ϕ′)**

But

slower than the UU

Unconsolidated Un drained (UU) Test

- **pore pressure develops during load application**

Not measured
$\therefore \sigma'$ unknown

Δu= **0**; i.e., failure envelope is horizontal

- analysed in terms of σ ➔ gives c_u and ϕ_u

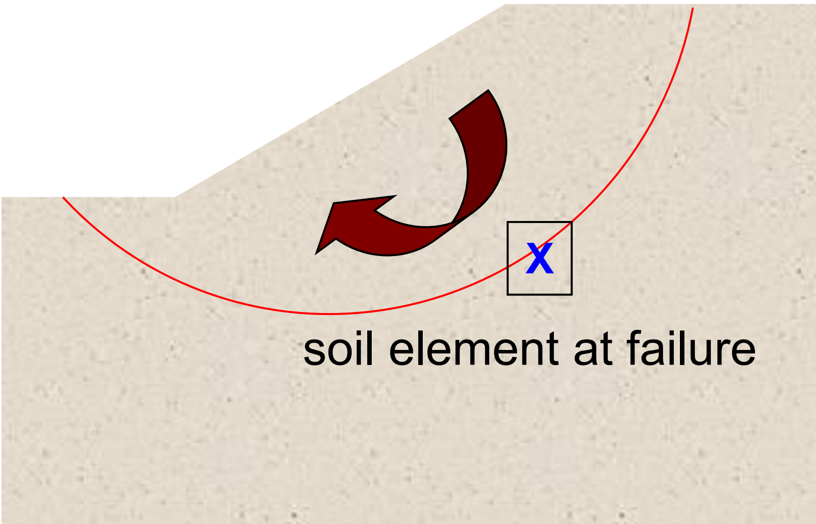
- **very quick test**

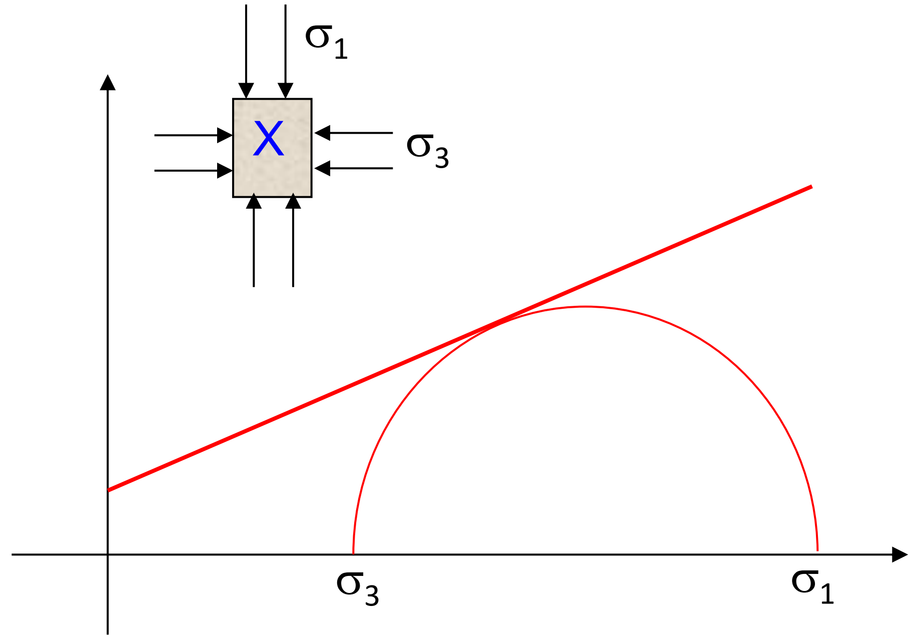
Use c_u and ϕ_u for analysing

undrained situations

(e.g., short term stability,)

σ_1 to σ_3 Relation at Failure





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

Stress path

Stress path is a curve used
to represent
the successive states of stress
in a test specimen of soil
during loading or unloading.

Series of Mohr circles drawn to
represent
the successive states of stress
but it is difficult to
represent number of circles in one diagram.

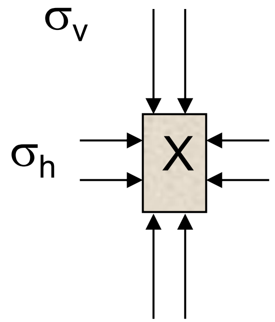
Figure shows number of Mohr circles
by keeping σ_3 and
increasing σ_1
on σ - t plane.

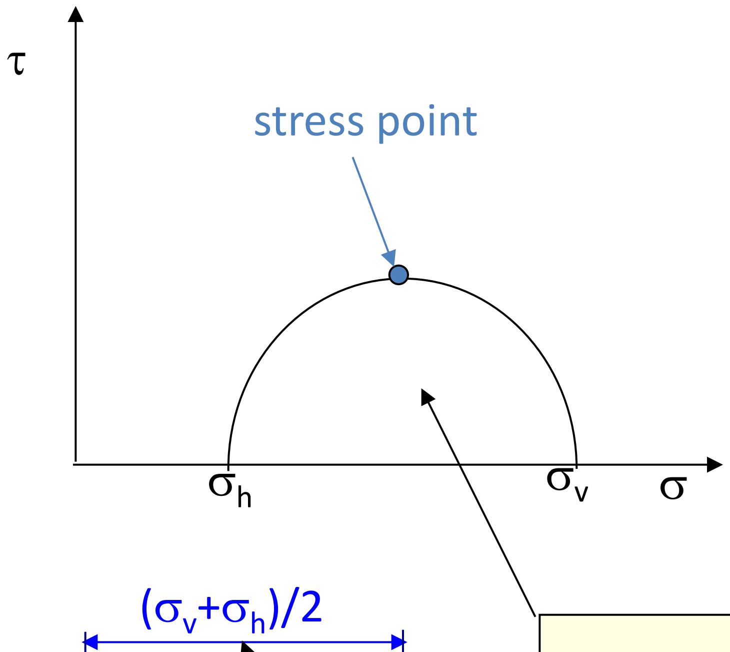
The successive states of stress can be represented by a series of stress points and

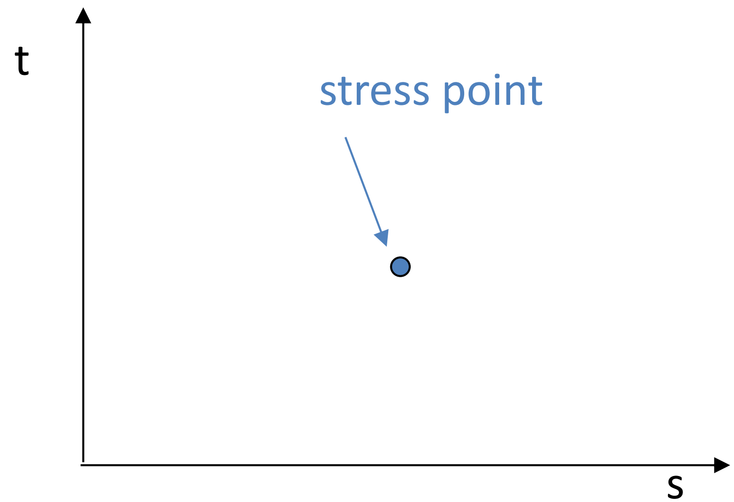
a locus of these points
(in the form of straight or curve) is obtained.
The locus is called stress path.

The stress points on σ - t plane can be transferred to p-q plane

Stress Point

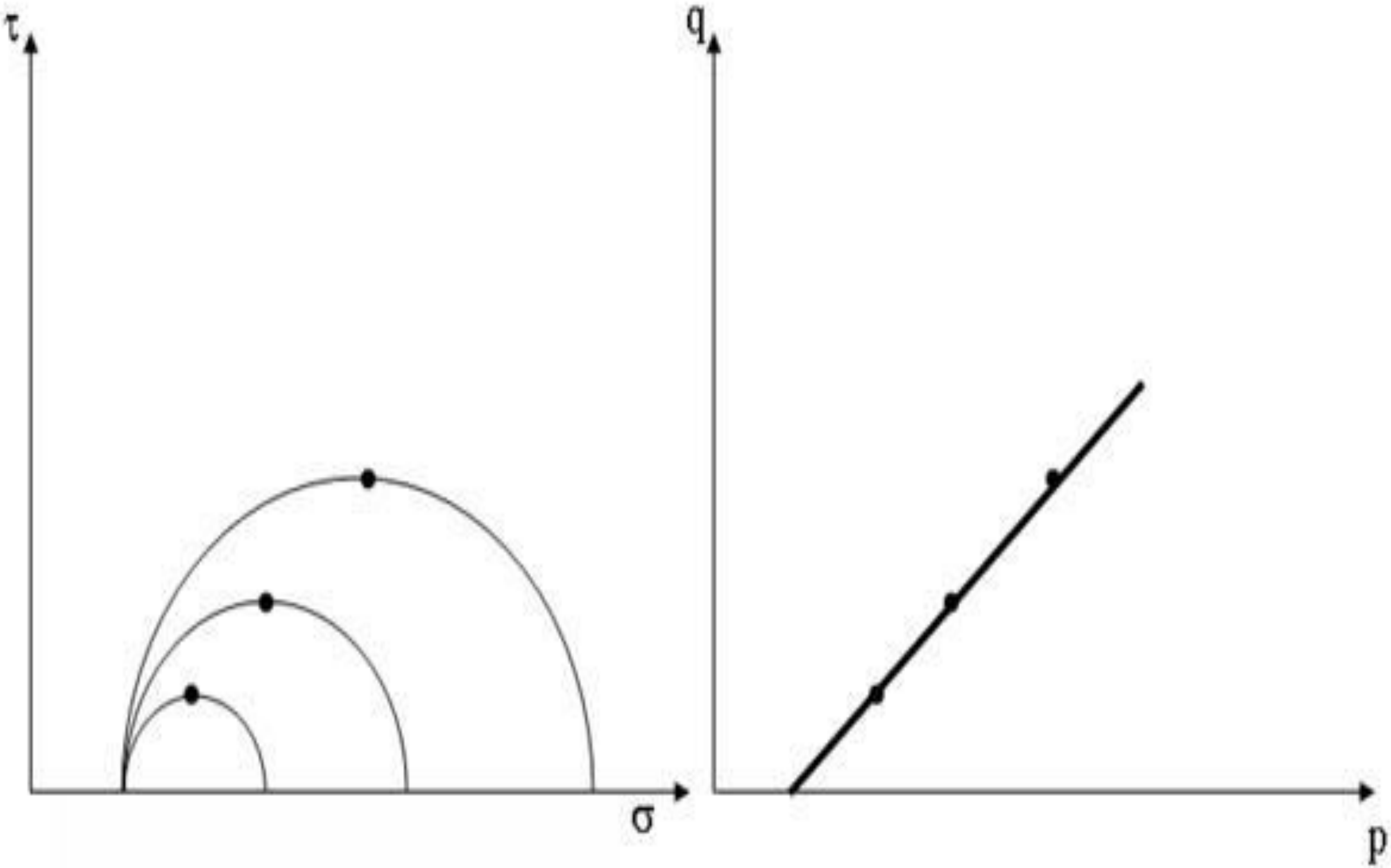






$$t = \frac{\sigma_v - \sigma_h}{2}$$

$$s = \frac{\sigma_v + \sigma_h}{2}$$


τ
σ
q
p

Types of Stress Path

(a) Total stress path (TSP)

(b) Effective stress path (ESP)

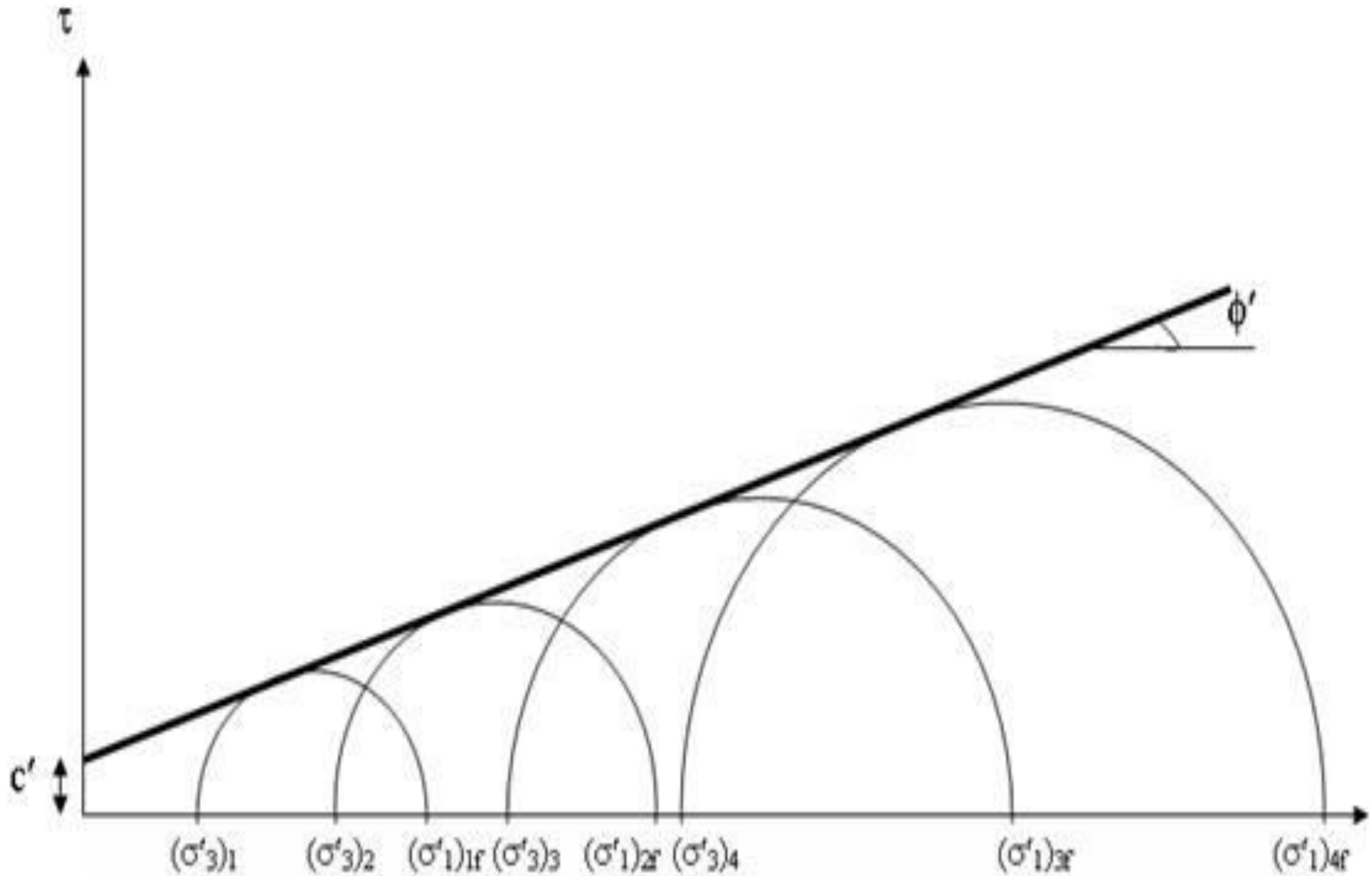
(c) Stress path of

total stress minus

static pore water pressure (TSSP)

1. Un-drained tri- axial tests with pore pressure measurement have been performed on three samples of a particular soil, after consolidation to different cell pressures. What information (strength parameters) can be obtained from the results given below?

Cell pressure in kPa	Deviator Stress in kPa	Pore Pressure in kPa
24	31	12
48	76	18
72	104	30


τ
φ′
c′
(σ′3)1
(σ′3)2
(σ′1)1f
(σ′3)3
(σ′1)2f
(σ′3)4
(σ′1)3f
(σ′1)4f

2. A soil has an apparent cohesion c' = 5 kPa and an angle of friction ϕ' = 35◦ .

A sample of this soil is consolidated in a triaxial cell by applying a cell pressure σ3 = 70 kPa.

The sample is then failed by increasing the axial stress under undrained conditions (σ3 remains constant). Calculate the axial stress at failure if the pore pressure at failure u = 20 kPa.

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

UNCONFINED COMPRESSION TEST

The test is suitable for saturated clay ($\varnothing_u = 0$).

The test is conducted under zero cell pressure. Thus,

it a special case of tri-axial test with $\sigma_3 = 0$

A cylindrical specimen is

subjected to axial stress until failure.

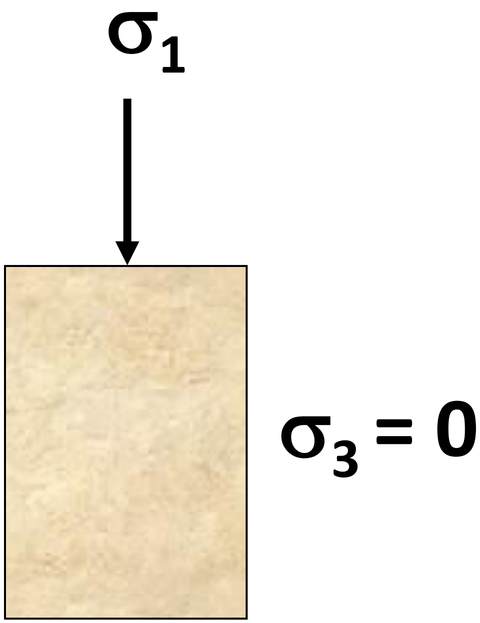
the subscript u is used as

the test is un-drained test.

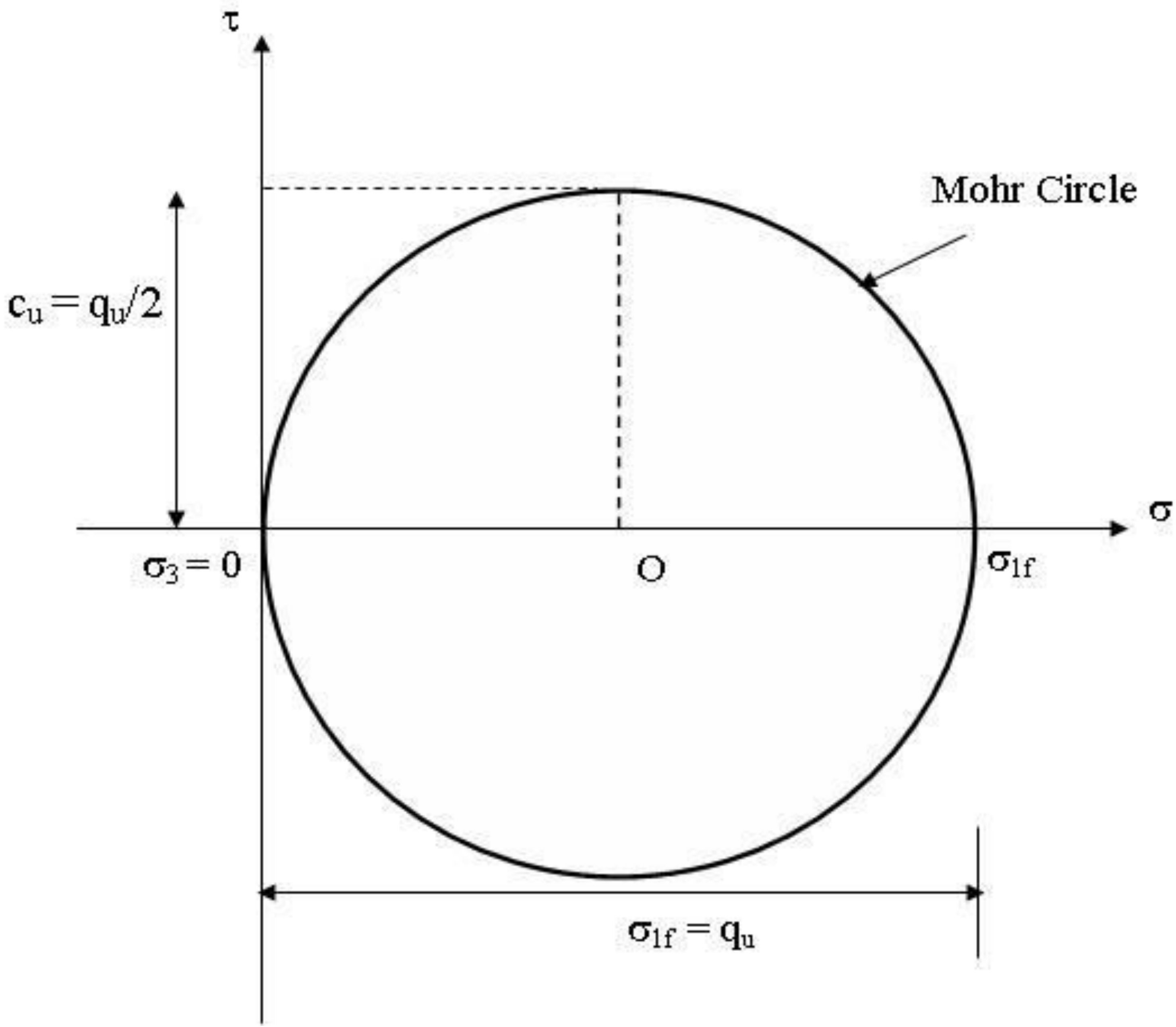
The major principle stress (σ_1) is
equal to the

unconfined compressive strength of the soil (q_u).

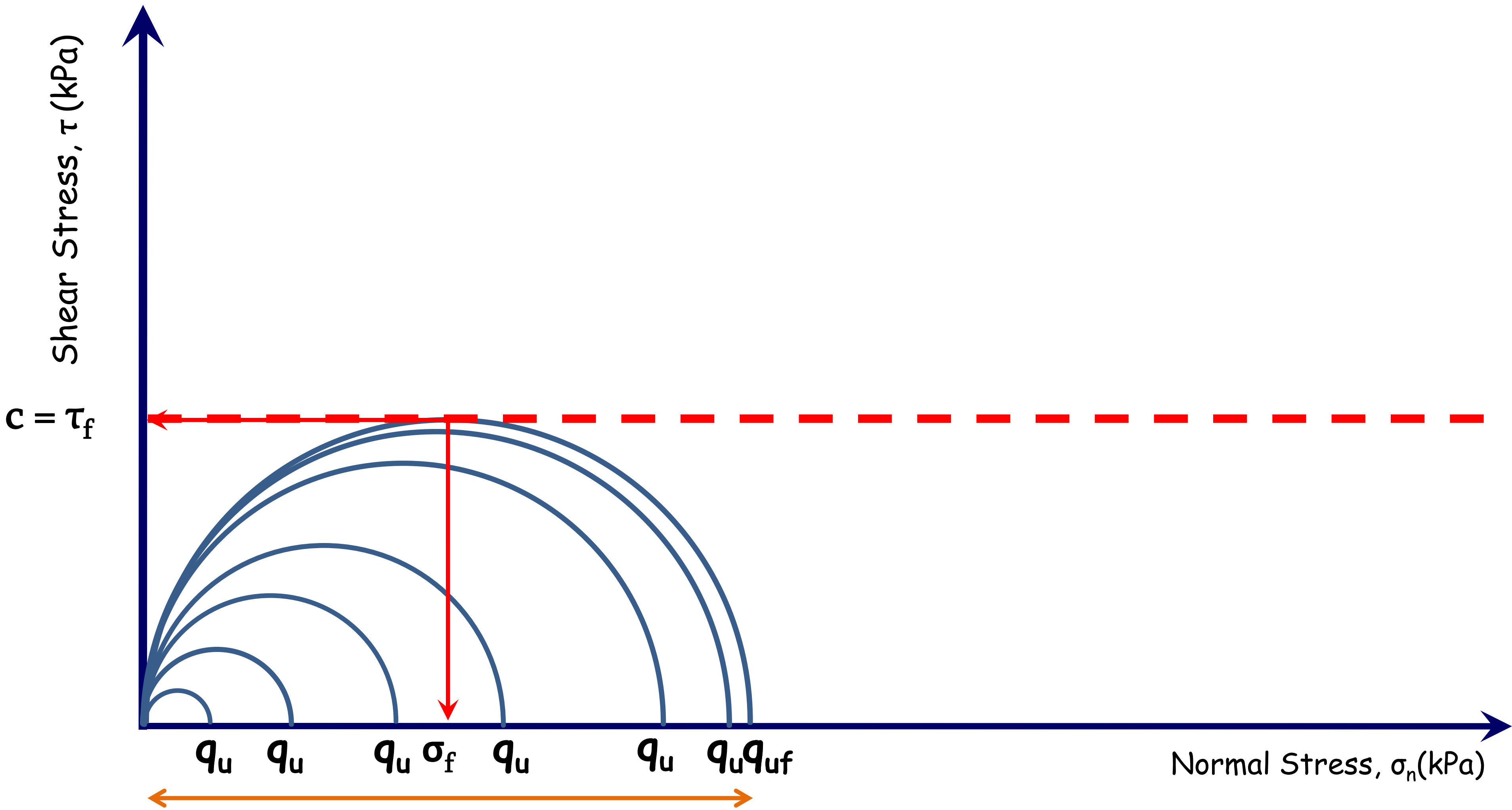
Unconfined Compression Test (UC Test)



Confining pressure is zero in the UC test



τ
Mohr Circle
$c_u = q_u/2$
σ
$\sigma_3 = 0$
O
σ_{1f}
$\sigma_{1f} = q_u$



Shear Stress, τ (kPa)
c = τf
qu
qu
qu
σf
qu
qu
qu
quf
Normal Stress, σn(kPa)

The Un-drained Cohesion can be determined as

$$c_u = \frac{q_u}{2}$$

For determining the unconfined compressive strength of the soil (q_u)

The applied load at failure divided by
the cross-sectional area of the sample

The cross-section of the soil sample at failure load (A_f) is determined as:

$$A_f = \frac{A_0}{1-\varepsilon}$$

where A_0 is the initial cross-sectional area of the sample and ε Is the axial strain in the sample. The strain in the sample can be determined as: ε

Change in length/ Original Length

Vane Shear Test

Vane shear test is a useful method of measuring the shear strength of clay.

It is a cheaper and quicker method.

The laboratory vane shear test is useful for

soils of low shear strength (less than 0.3 kg/cm^2) for which tri-axial or unconfined tests can not be performed.

The test gives the un-drained strength of the soil.

The undisturbed and remoulded strength obtained are useful for evaluating the sensitivity of soil.

EXPERIMENTAL PROCEDURE

Prepare two or three specimens of

the soil sample of dimensions of

at least 37.5 mm diameter and 75 mm length in specimen

(L/D ratio 2 or 3).

Mount the specimen container with the specimen

on the base of the
vane shear apparatus.

Gently lower the shear vanes into the specimen
to their full length
without disturbing the soil specimen.

The top of the vanes should be
at least 10 mm below
the top of the specimen.

Rotate the vanes at an uniform rate say $0.1^{o}/s$
until the specimen fails.

Note the reading of the angle of twist.
Find the value of blade height in cm.
Find the value of blade width in cm.

Calculations

Shear strength, $$S = \frac{T}{\pi\left[\frac{D^2H}{2} + D^3\right]}$$

Where S = shear strength of soil in kg/cm2

T = torque in cm kg

D = overall diameter of vane in cm

H = Overall Height of vane in cm

Soil Liquefaction

soil liquefaction occurs

when
the effective stress of soil is

reduced to essentially zero,

which corresponds to
a complete loss of shear strength.

Soil Liquefaction
may be initiated by
either monotonic loading

(e.g. single sudden occurrence of a change in stress –
examples include
an increase in load on an embankment

or
sudden loss of toe support) or cyclic loading

(e.g. repeated change in stress condition –
examples include
wave loading or earthquake shaking).

cyclic mobility

The term 'cyclic mobility' refers to

the mechanism of progressive reduction of effective stress
due to cyclic loading.

In case of a dense soils,

on reaching a state of zero effective stress,

such soils immediately dilate and regain strength known as cyclic mobility

Causes of Liquefaction

Occurrence of liquefaction is
the result of

rapid load application and
break down of the loose and saturated sand and

the loosely-packed individual soil particles

tries to move
into

a denser configuration.

However,

there is not enough time for the pore-water of the soil to be squeezed out in case of earthquake.

Instead,

the water is trapped and prevents the soil particles from moving closer together.

Thus,

there is an increase in water pressure which reduces the contact forces between the individual soil particles causing

softening and weakening of soil deposit.

In extreme conditions,

the soil particles may lose contact with each other due to the increased pore-water pressure.

In such cases,
the soil will have very little strength, and will behave more like a liquid than a solid –

hence, the process is called "liquefaction".

Effects of Soil Liquefaction

The pressure generated during large earthquakes

with many cycles of shaking can cause

the liquefied sand and excess water to force its way to the

ground surface from several metres below the ground.

This is often observed as

"sand boils" also called "sand blows" or "sand volcanoes"

Other common Effect of Soil Liquefaction

The other common observation is

land instability –

cracking and movement of the ground down slope or

towards unsupported margins of rivers, streams, or the coast.

The failure of ground in this manner is called

'lateral spreading', and may occur

on very shallow slopes of angles of only 1 or 2 degrees to horizontal.

Examples of Liquefaction Effects

Positive Aspect of Soil Liquefaction

One positive aspect of soil liquefaction is

the tendency for the effects of earthquake shaking to be significantly

damped (reduced)

for the remainder of the earthquake.

This is because liquids do not support a shear stress and so

once the soil liquefies due to shaking, subsequent

earthquake shaking
(transferred through ground shear waves) is not

transferred to buildings at the ground surface.

Skempton Pore Pressure Parameters

In un-drained tests,

the general expression relating total pore water pressure developed and

changes in applied stresses for both the stages is:

$$\Delta u = \Delta u_1 + \Delta u_2$$

$$= B.\Delta\sigma_3 + B.A\,(\Delta\sigma_1 - \Delta\sigma_3)$$

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

where Δu_1 = pore water pressure developed in the first stage during application of confining stress $\Delta\sigma_3$,

Δu_2 = pore water pressure developed in the second stage during application of deviator stress ($\Delta\sigma_1 - \Delta\sigma_3$), and

B and **A** are Skempton's pore water pressure parameters.

Parameter **B** is a function of the degree of saturation of the soil

(= 1 for saturated soils, and = 0 for dry soils).

Parameter **A** is also not constant, and it varies with the over-consolidaton ratio of the soil and also with the magnitude of deviator stress.

The value of **A** at failure is necessary in plotting the effective stress Mohr circles.

Consider the behaviour of saturated soil samples in un-drained tri-axial tests.

In the first stage, increasing the cell pressure without allowing drainage has the effect of increasing the pore water pressure by the same amount.
Thus, there is no change in the effective stress.

During the second shearing stage, the change in pore water pressure can be either positive or negative.

For **UU tests** on saturated soils, pore water pressure is not dissipated in both the stages (i.e., $\Delta u = \Delta u_1 + \Delta u_2$).

For **CU tests** on saturated soils, pore water pressure is not dissipated in the second stage only (i.e., $\Delta u = \Delta u_2$).

THANK YOU

UNIT V

SOIL SLOPE STABILITY

Syllabus

Slope failure mechanisms – Types –

infinite slopes –

finite slopes –

Total stress analysis for saturated clay &C-$^{\phi}$ soil

Finite Slope Analysis

Method of Slices–

Friction circle method –

Use of stability number –

slope protection measures.

Slope

A slope is an inclined boundary surface
between
air and the body of an earthwork
such as
highways, cut or fill,
railway cut or fill,
earth dams,
Hill side surface

Causes of Slope failure

a) **Increased unit weight of soil by wetting**

b) Added external loads (moving loads, buildings etc)

c) Steepened slopes either by excavation or by erosion

d) Shock loads

e) Vibration and earthquakes

f) Increase in moisture content

g) Freezing and thawing action

h) Increase in pore pressure

i) Loss of cementing pressure

Objectives of slope Stability

Determination of the

potential failure surface

forces tending to cause slip.

forces tending to stabilize the mass of earth.

Assumptions for Slope Stability Analysis

Problems are two dimensional

Coulomb's theory can be used to compute shear strength

shear strength is assumed as uniform along the slip surface.

The flow net in case of seepage can be drawn and seepage forces evaluated.

Types of Slopes

Infinite Slope -Natural – Ex - Hill slope

[Slopes extending to infinity do not exists in nature.

For all practical purposes

any slope of great extent with

soil conditions essentially same for

all identical depth below

the earth surface are

known as infinite slopes.]

Therefore

The slip surface will be the

plane parallel

to the surface of slope.

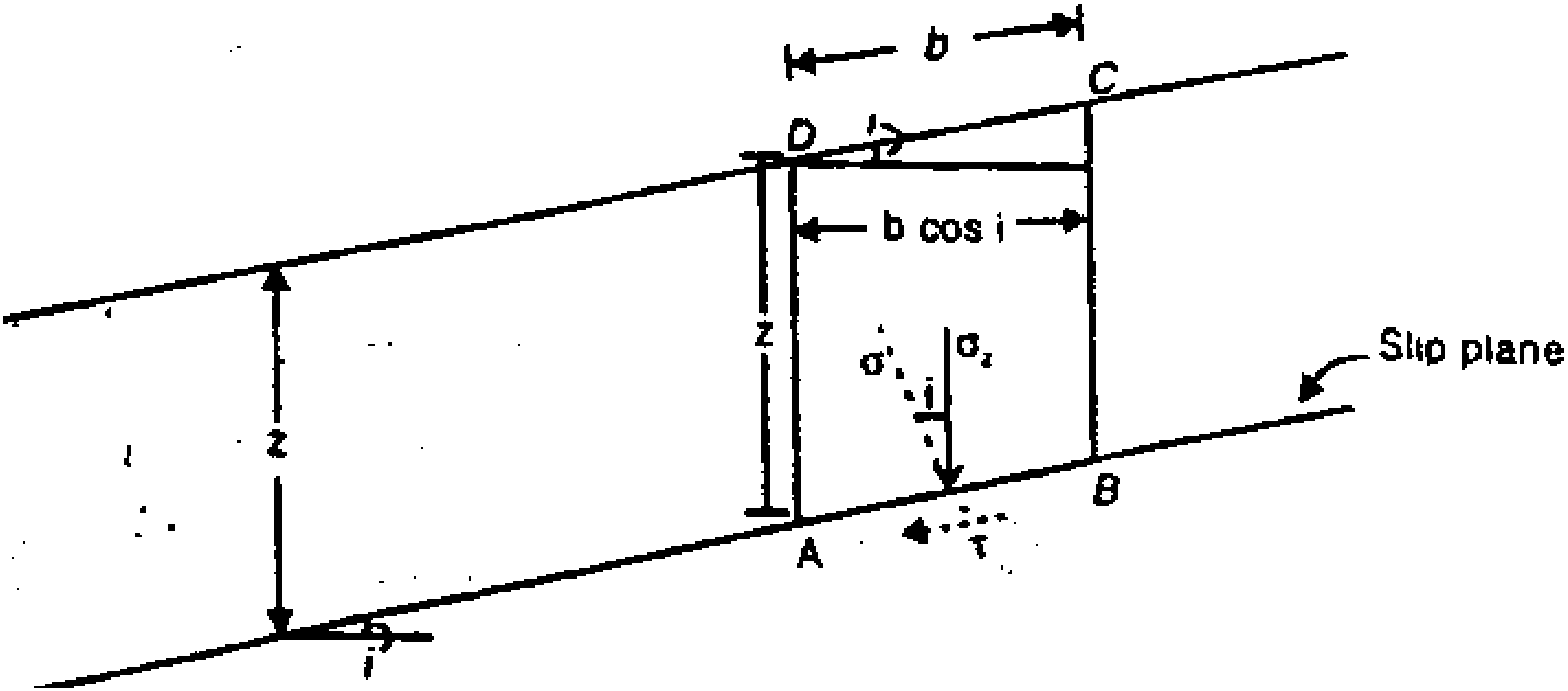
FACTOR OF SAFETY

- Factor of safety of a slope is defined as the ratio of average shear strength τ_f of a soil to the average shear stress τ_d developed along the potential failure surface.

$$FS = \frac{\tau_f}{\tau_d}$$

- FS = Factor of safety
- τ_f = *average shear strength of the soil*
- τ_d = average shear stress developed along the potential surface.

Stability of Infinite slope



Let a slip plane be at depth z below the surface of slope, with i as the angle of slope.

We consider a prism of soil ABCD with inclined width b, depth z and of

unit thickness perpendicular to the plane of paper, as shown in figure.

Vertical stress on plane AB,

$$\sigma_z = \frac{\text{Weight of prism ABCD}}{\text{Area of plane AB}} = \frac{(z\, b \cos i)\gamma}{b} = \gamma z \cos i \qquad \text{.............(i)}$$

$\therefore$

Normal stress on slip plane, $\sigma = \sigma_z \cos i = \gamma z \cos^2 i$ ....(ii)

Shear stress on slip plane, $\tau = \sigma_2 \sin i = \gamma z \cos i \sin i$ ....(iii)

Case (i): Slope of cohesionless soil ($c = 0$)

Factor of safety against sliding

$$F = \frac{\tau_f}{\tau} = \frac{c + \sigma \tan\phi}{\tau} = \frac{\gamma z \cos^2 i \tan\phi}{\gamma z \cos i \sin i} = \frac{\tan\phi}{\tan i} \qquad \text{..............(iv)}$$

We observe that for a slope of cohesionless soil

the factor of safety against sliding is independent of depth z.

The slope will be stable as long as

the angle of slope i is less than or equal to angle of shearing resistance ϕ of soil.

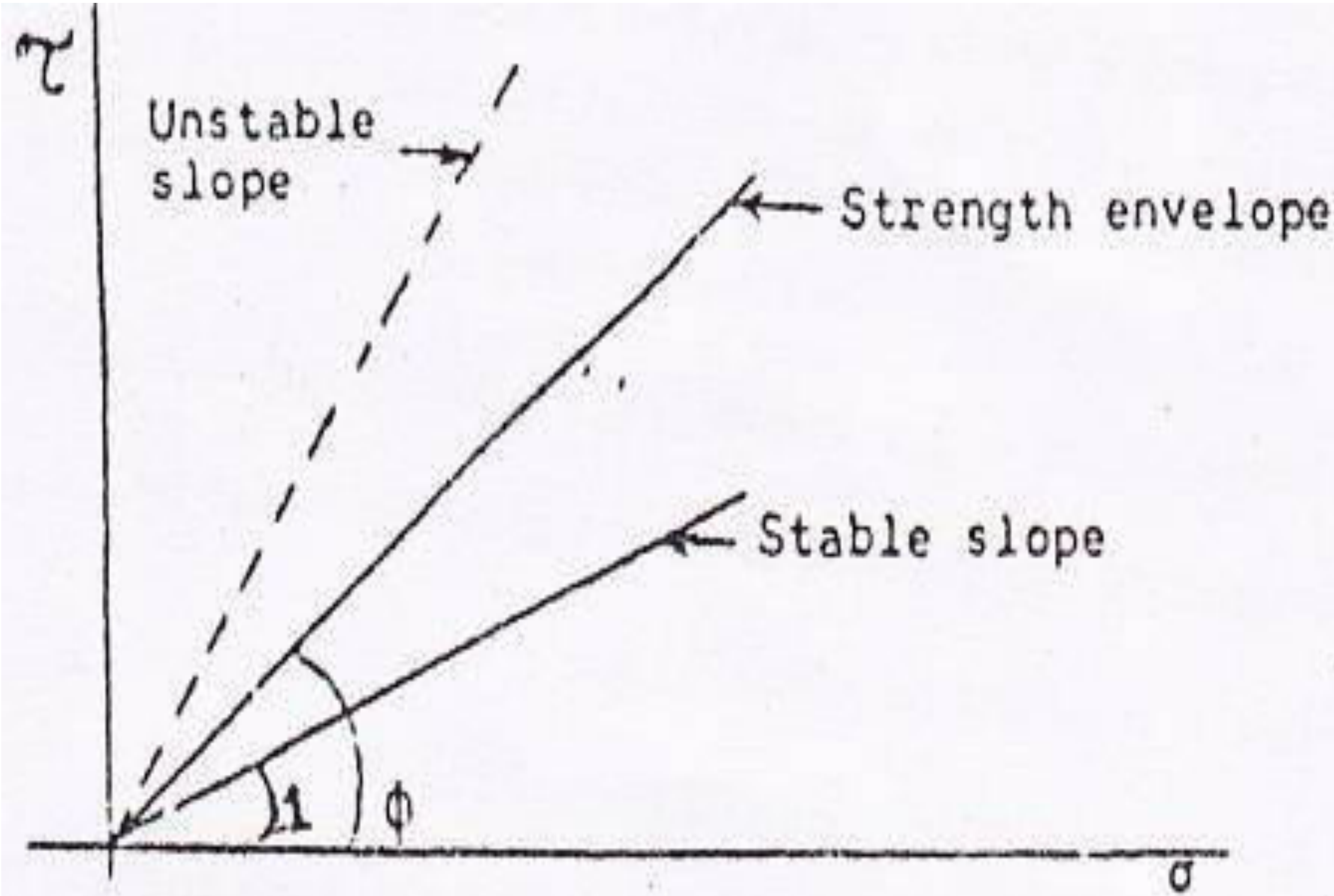


Fig.1- Failure condition for an infinite slope of cohesion less soil

Case (ii): Cohesive soil (c - ϕ soil)

Factor of safety against sliding,

$$F = \frac{\tau_f}{\tau} = \frac{c + \sigma \tan\phi}{\tau} = \frac{c + \gamma z \cos^2 i \tan\phi}{\gamma z \cos i \sin i} \quad \text{..............(vi)}$$

In this case we observe that the factor of safety against sliding depends on depth z.

For a given slope angle $i < \phi$, the slope will be stable up to

certain depth z_c called the critical depth.

Substituting $z = z_c$ and $F = 1$ in equation (vi) we get

$$1 = \frac{c + \gamma\, z_c \cos^2 i \tan\phi}{\gamma\, z_c \cos i \sin i}$$

$$\gamma\, z_c \cos i \sin i = c + \gamma\, z_c \cos^2 i \tan\phi$$

$$z_c = \frac{c}{\gamma\,(\cos i \sin i - \cos^2 i \tan\phi)}$$

Further $\frac{c}{\gamma z_c} = (\cos i \sin i - \cos^2 i \tan\phi)$

The quantity $\frac{c}{\gamma z_c}$ is dimensionless,

referred to as stability number and denoted by S_n.

We note that it is a function of i and ϕ only.

For $i < \phi$ the slope will always be stable irrespective of depth z.

If the factor of safety Fc is applied in cohesion,

the mobilized cohesion at depth H, given by

$$F_c = \frac{C}{C_m}$$

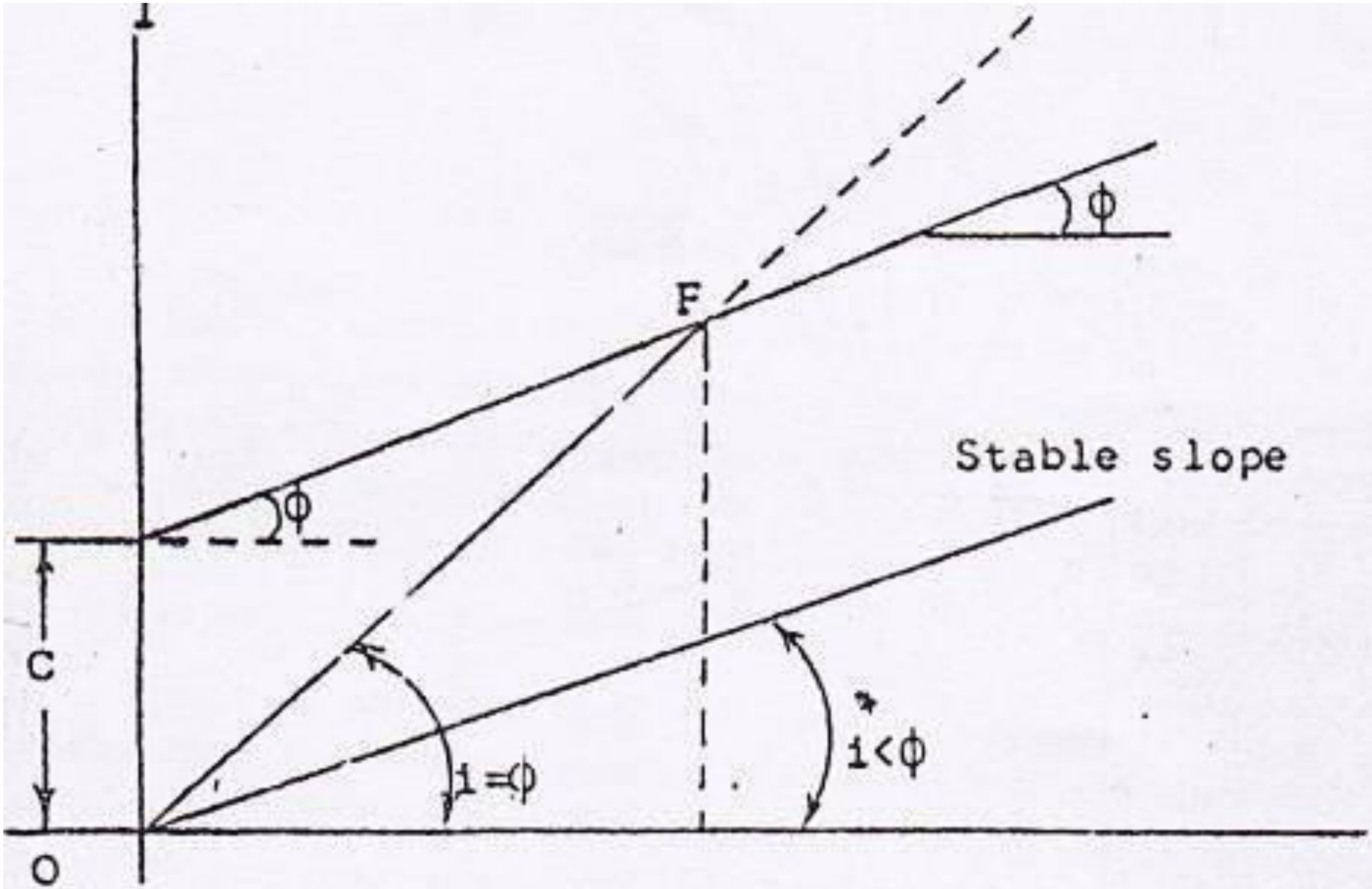
Then the depth H calculated by using mobilized cohesion Cm will not be critical.

the factor of safety against height also represents

the factor of safety with respect to cohesion Fc.

Fc is given by

$$F_c = \frac{H_c}{H}$$



F
ϕ
ϕ
Stable slope
C
O
i = ϕ
i<ϕ

A slope of very large extent of soil with properties $c' = 0$ and $\phi' = 32°$ is likely to be subjected to seepage parallel to the slope with water level at the surface. Determine the maximum angle of slope for a factor of safety of 1.5 treating it as an infinite slope. For this angle of slope what will be the factor of safety if the water level were to come down well below the surface? The saturated unit weight of soil is 20 kN/m^3.

Solution:

$$c' = 0 \qquad \phi' = 32^\circ \qquad \gamma_{sat} = 20\ \text{kN/m}^3$$

Case (i) When water level is at surface and seepage occurs parallel to surface

Factor of safety, $$F = \left(\frac{\gamma'}{\gamma_{sat}}\right)\left(\frac{\tan\phi'}{\tan i}\right)$$

$$\therefore \qquad \tan i = \frac{(20-9.81)}{20} \cdot \frac{\tan 32^\circ}{1.5} = 0.2122$$

Slope angle, i = 12°

Case (ii) When the water table goes well below the surface,

$$F = \frac{\tan\phi'}{\tan i} = \frac{\tan 32^\circ}{\tan 12^\circ} = 2.94$$

Example: A slope 1 in 2 with a height of 8 m has the following soil properties:

$c = 28\ kN/m^2$, $\phi = 10°$ $\gamma = 18\ kN/m^3$

Calculate (i) factor of safety with respect to cohesion an d(ii) critical height of slope.

Solution:

(i) If i is the slope angle, we have

$$\tan i = \frac{1}{2}$$

$$\therefore \quad i = \tan^{-1}\frac{1}{2} = 26.6^\circ$$

From Taylor stability chart, for I = 26.6° and $\phi = 10°$, $S_n = 0.064$

$$S_n = \frac{c}{F_c \gamma H}$$

$$\therefore \quad F_c = \frac{c}{S_n \gamma H} = \frac{28}{(0.064)(18)(8)} = 3.04$$

(ii) $$F_c = \frac{H_c}{H}$$

$$\therefore \quad H_c = F_c . H = (3.04)(8) = 24.32 \text{ m}$$

Example: A slope is to be laid at an angle of 30° with the horizontal. Find safe height of slope for a factor of safety of 1.5 if the soil properties are:

$c = 15\ kN/m^2$ $\phi = 22°$ $\gamma = 18\ kN/m^3$

Solution:

Since factor of safety given in the problem is with respect to shear strength, it is applicable to both c and ϕ.

Mobilized frictional angle, ϕ_m is given by

$$F_\phi = \frac{\tan\phi}{\tan\phi_m} \quad \therefore\ \phi_m = \tan^{-1}\left(\frac{\tan 22^\circ}{1.5}\right) = 15^\circ$$

From Taylor stability chart,

For $i = 30^\circ$ and $\phi_m = 15^\circ$, $S_n = 0.046$

We have $$S_n = \frac{c}{F_\gamma H}$$

$$\therefore \quad H = \frac{c}{S_n.F.\gamma} = \frac{15}{(0.046)(1.5)(18)} = 12.1\text{m}$$

Example: A 5 m deep canal has side slopes of 1:1. The properties of soil are c_u = 20 kN/m^2, ϕ_u = 10°, e = 0.8 and G = 2.8. If Taylor's stability number is 0.108, determine the factor of safety with respect to cohesion, when the canal runs full. Also find the same in case of sudden drawdown, if Taylor's stability number for this condition is 0.137.

Solution:

$$c_u = 20\ \text{kN/m}^2, \qquad \phi_u = 10°, \qquad G = 2.8$$

$$\gamma_{sat} = \frac{(G+e)\gamma_m}{1+e} = \frac{(2.8+0.8)9.81}{1+0.8} = 19.62\ \text{kN/m}^3$$

$$\gamma' = \gamma_{sat} - \gamma_w = 19.62 - 9.81 = 9.81\ \text{kN/m}^3$$

Case (i) when canal runs full the side slopes are merged.

$$S_n = \frac{c}{F_c \gamma' H}$$

$$F_c = \frac{c}{S_n \gamma' H} = \frac{20}{(0.108)(9.81)(5)} = 3.8$$

Case (ii) Sudden drawdown condition, $S_n = 0.137$

A canal with a depth of 5m has banks with slope 1:1. The properties of soil are $c = 20\ kN/m^2$ $\phi = 15^\circ$ $e = 0.7$ $G = 2.6$

Calculate factor of safety with respect to cohesion (i) when canal runs full and (ii) it is suddenly and completely emptied.

Solution:

Case (i) when canal runs full, the side slopes are submerged:

$$\gamma_{sat} = \frac{(G+e)\gamma_w}{1+e} = \frac{(2.6+0.7)(9.81)}{1+0.7} = 19.04\,kN/m^3$$

$$\gamma' = \gamma_{sat} - \gamma_w = 19.04 - 9.81 = 9.23\,kN/m^3$$

From Taylor stability chart,

For $i = 45°$ and $\phi_w = 7.3°$, $S_n = 0.122$

$$S_n = \frac{c}{F_c \cdot \gamma_{sat} . H}$$

$$F_c = \frac{c}{S_n \cdot \gamma_{sat} H} = \frac{20}{(0.122)(19.04)(5)} = 1.72$$

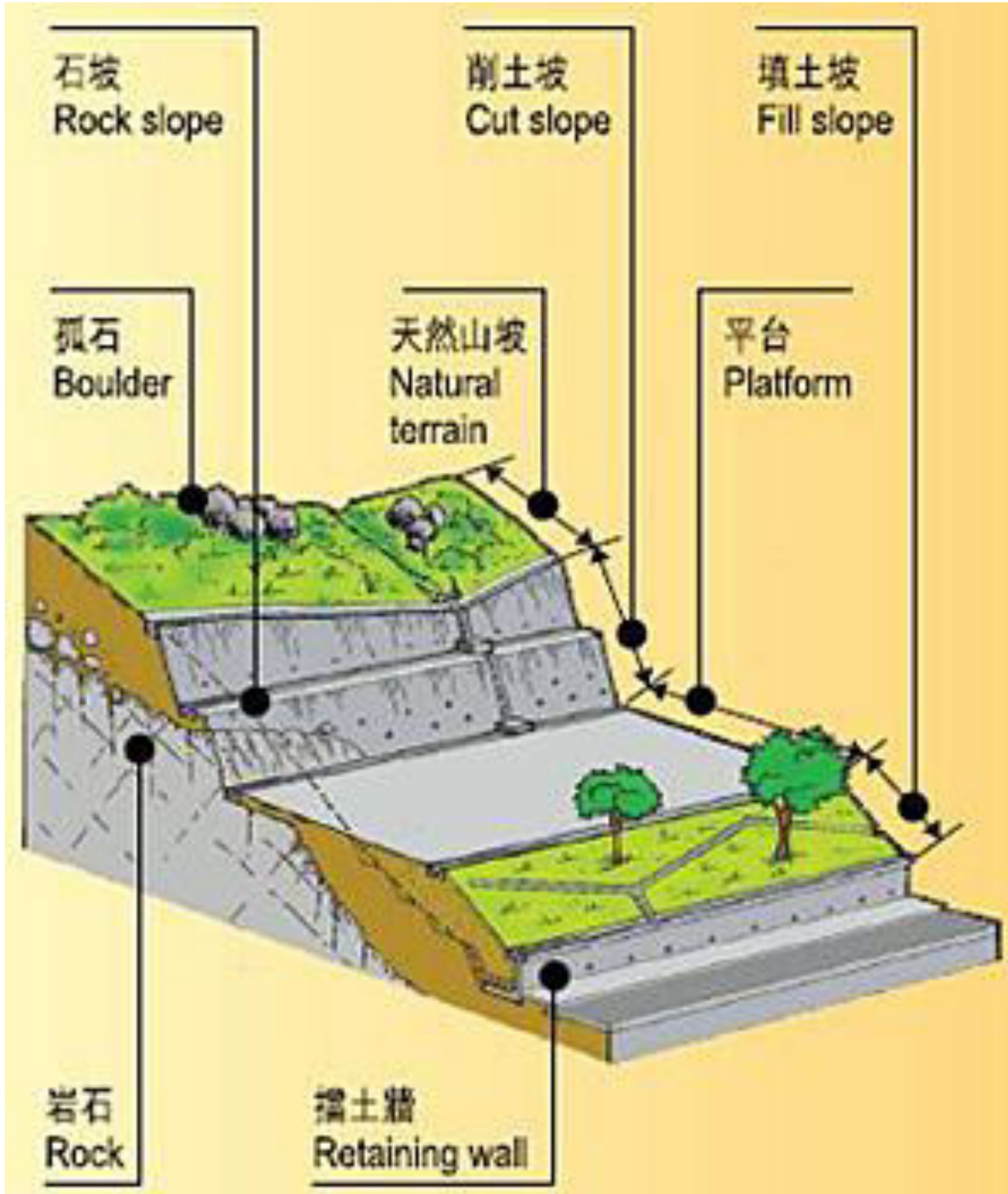
Finite slopes

These are Man made Slopes

A finite slope is one with
a base, top surface and the height being limited.

The slopes of
earth dams,
Railways and High ways embankments,
are examples of finite slopes.
The slope length depends on
the height of the dam or embankment.

Man-made slope


石坡
Rock slope
削土坡
Cut slope
填土坡
Fill slope
孤石
Boulder
天然山坡
Natural
terrain
平台
Platform
岩石
Rock
擋土牆
Retaining wall

Stability analysis of finite slopes

Failure of finite slopes occurs along

a curved surface.

In stability analysis of finite slopes,

the real surface of rupture is replaced by

an arc of a circle.

Types of Slope failure

Rotational failure

movement results from forces that

cause a rotation about

a point above the centre of gravity of the unit.

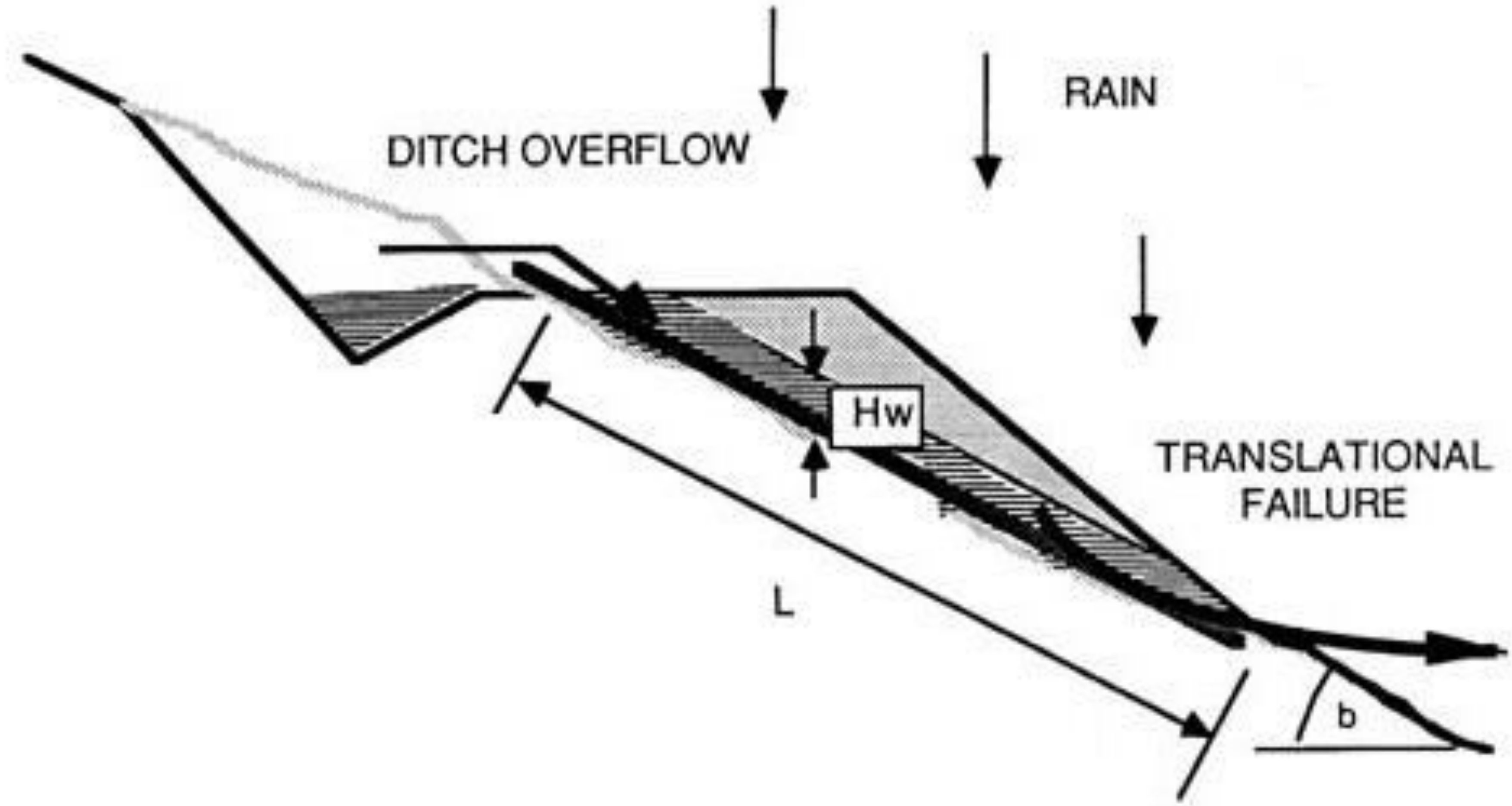
The surface of rupture concaves upwards.

Translational Failure

Where sliding occurs along

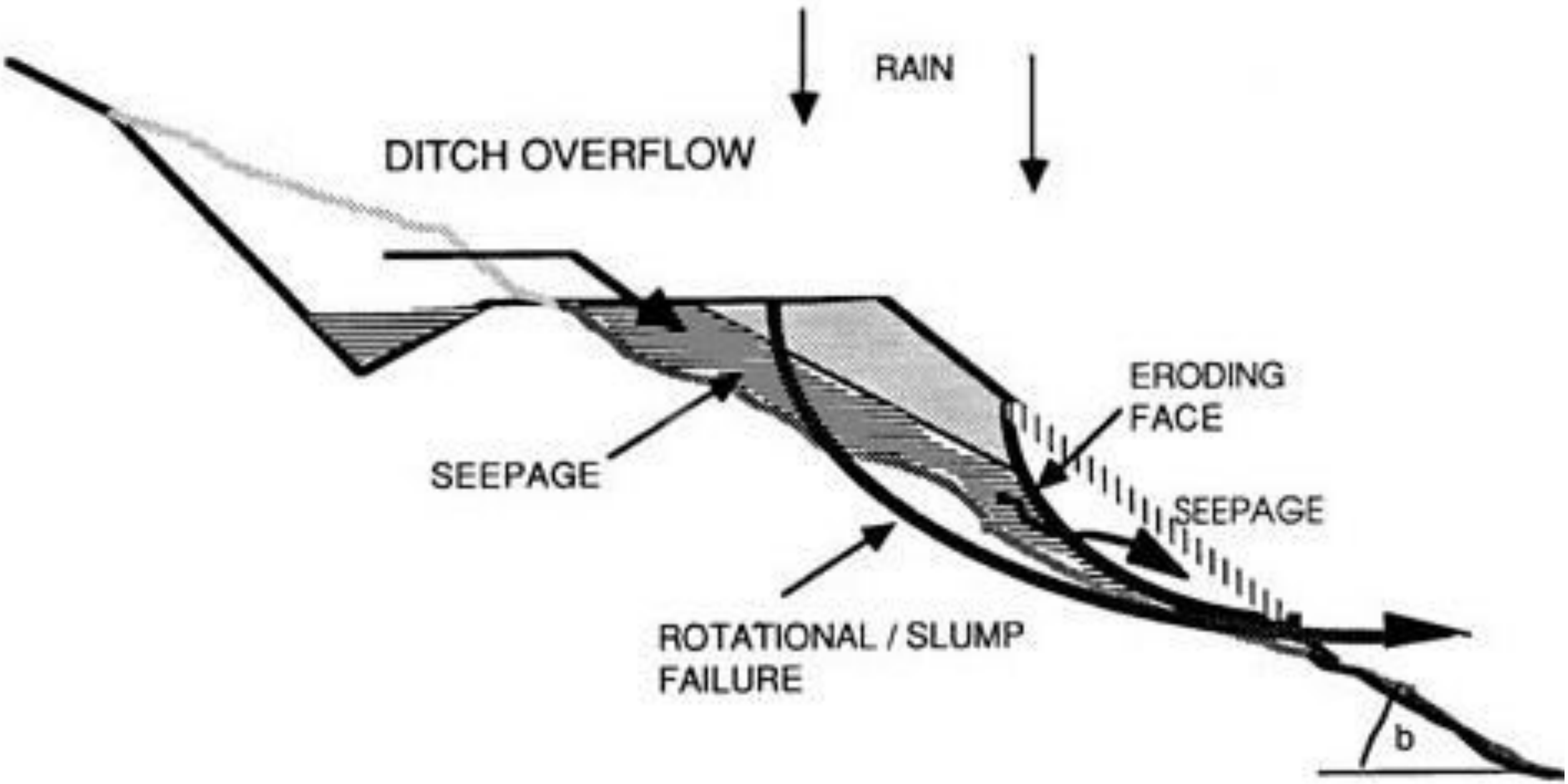
more or less planar or gently undulatory surfaces

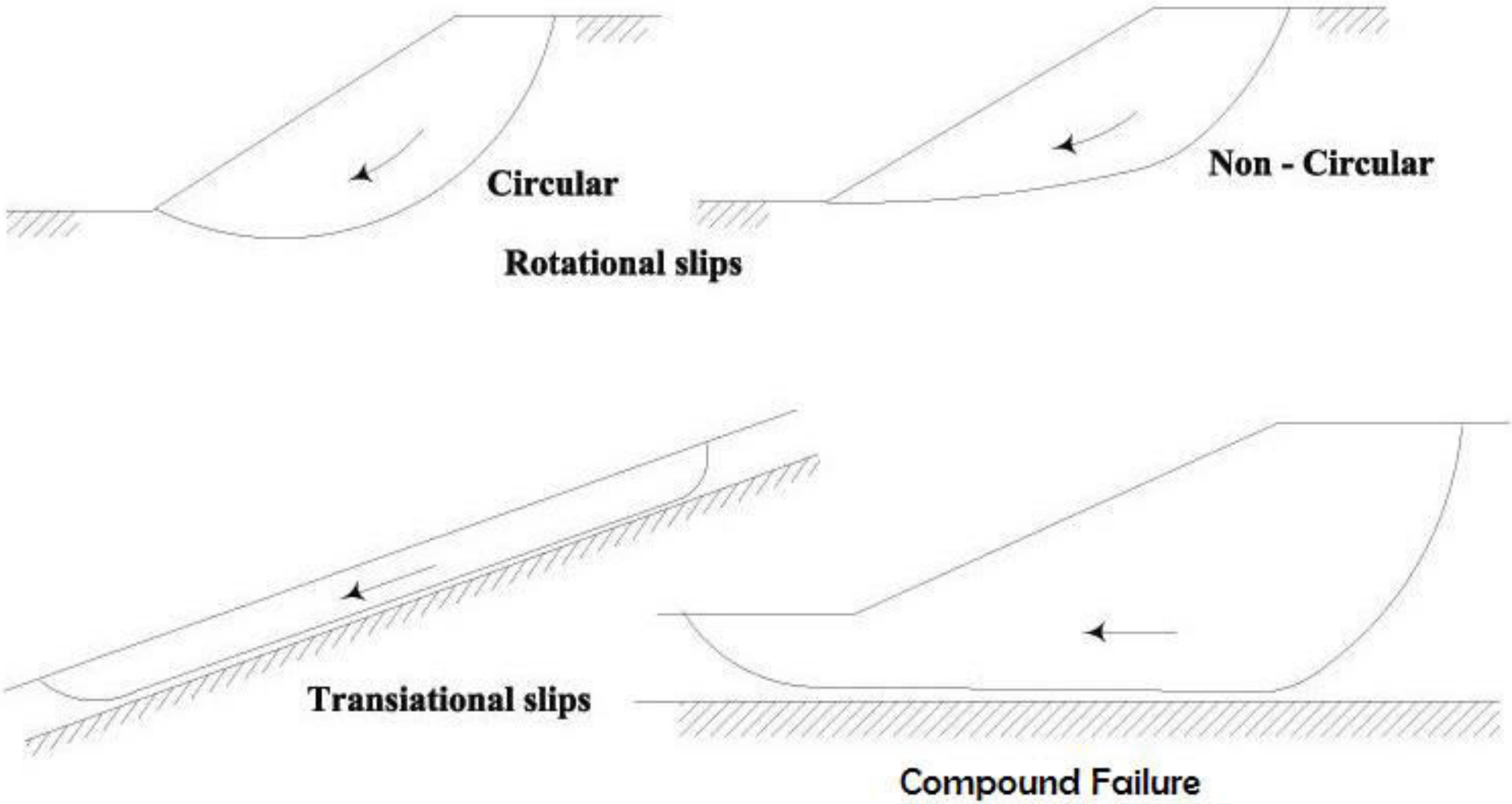
Parallel to the slope

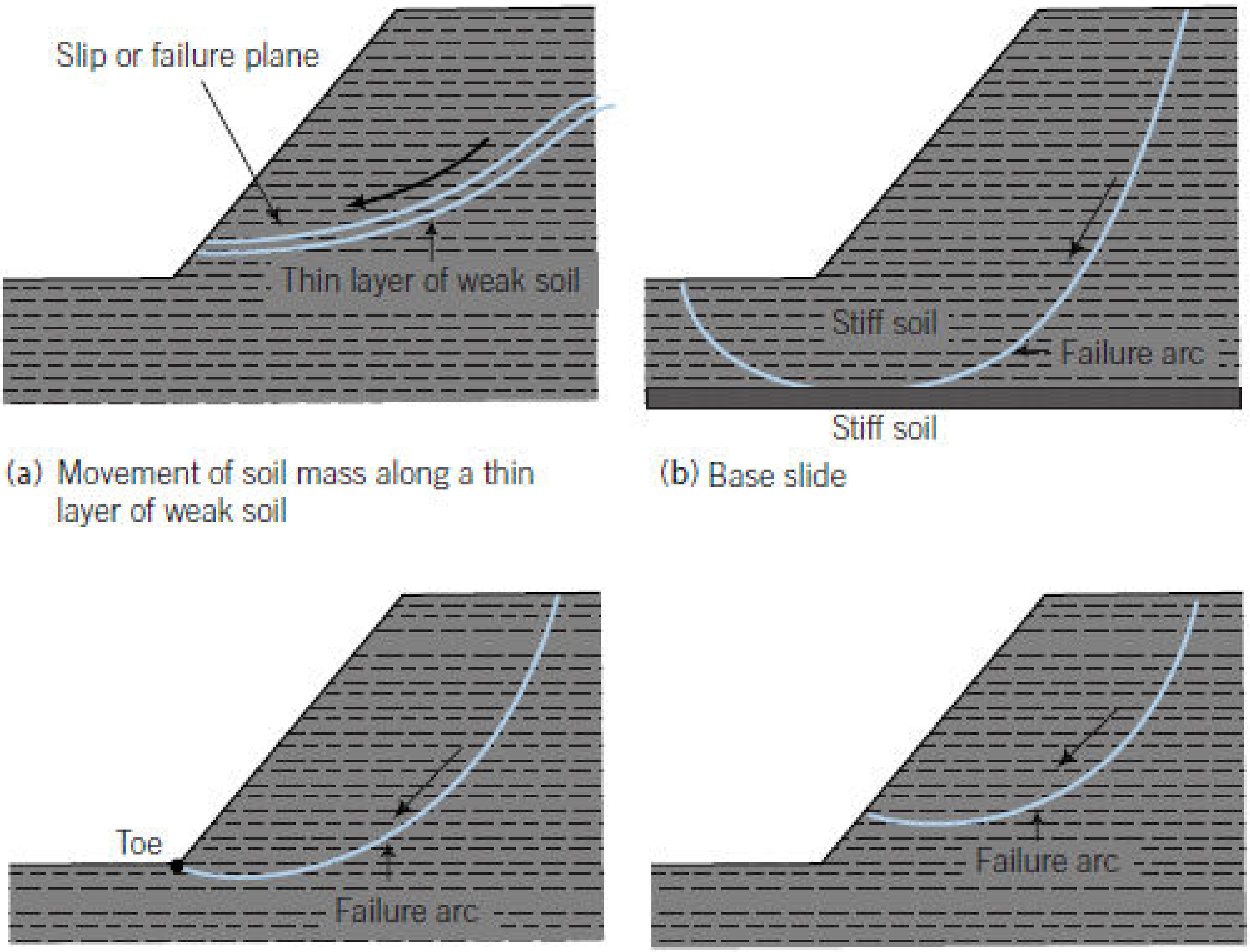


Translational failure

Rotational failure




Circular
Non - Circular
Rotational slips
Transiational slips
Compound Failure



(a) Movement of soil mass along a thin layer of weak soil

(b) Base slide

(c) Toe slide

(d) Slope slide

Types of Rotational Slope Failure

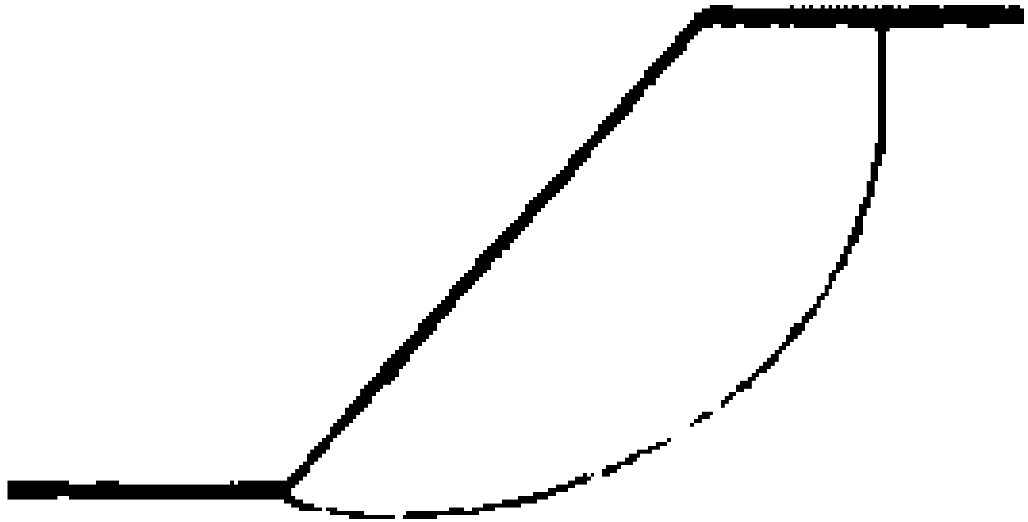
(i) Face failure

The failure surface passing through

the face of the slope or above the toe of slope is known as

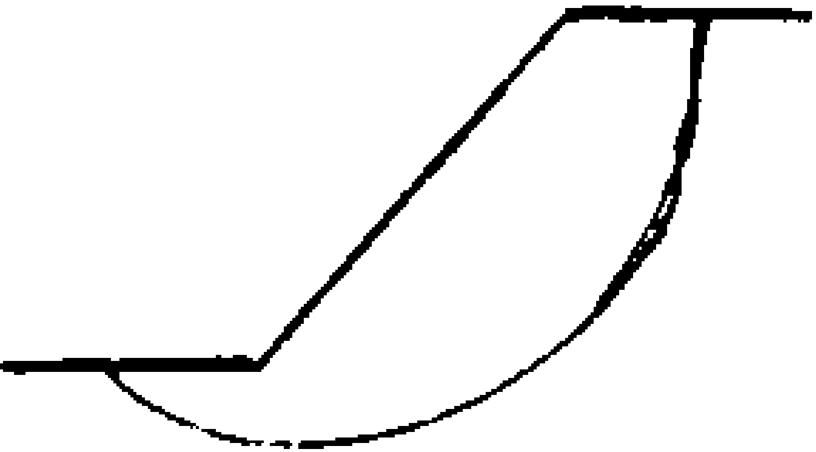
slope failure or face failure

Toe failure



(ii) Toe failure

The failure surface passing through the toe of the slope is known as toe failure.



(iii) Base failure

The rupture is deep seated and passes through

the embankment supporting soil (Base)

below the toe of the slope is

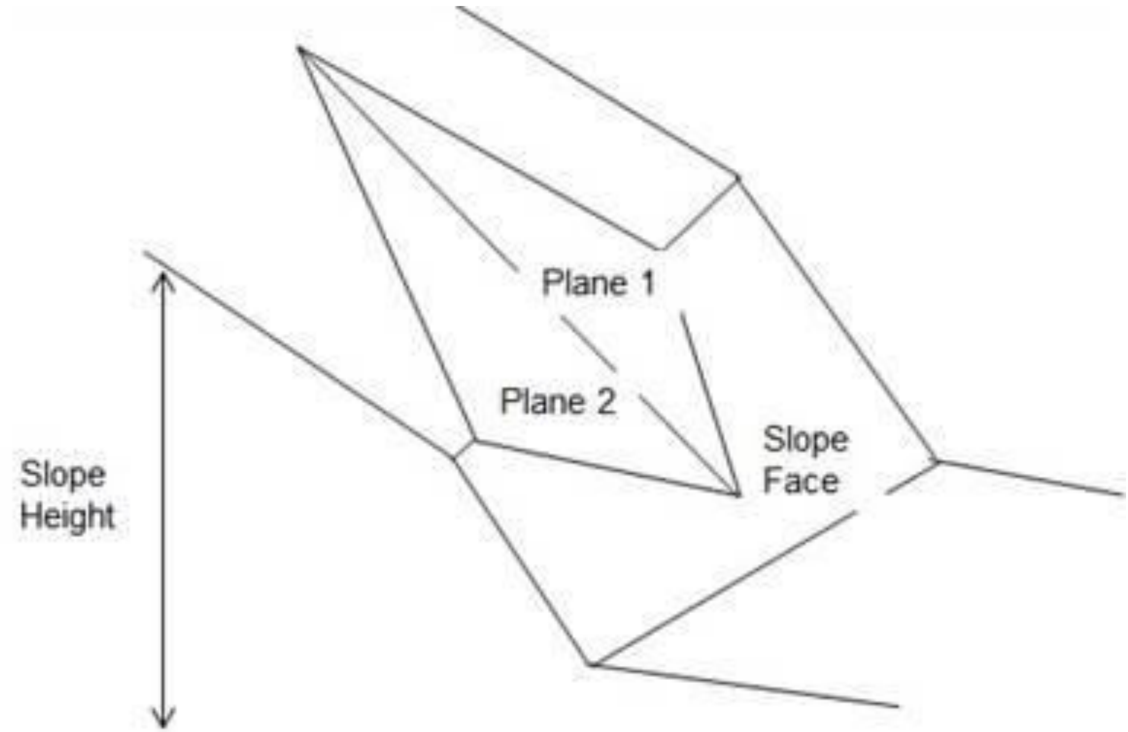
known as base failure.

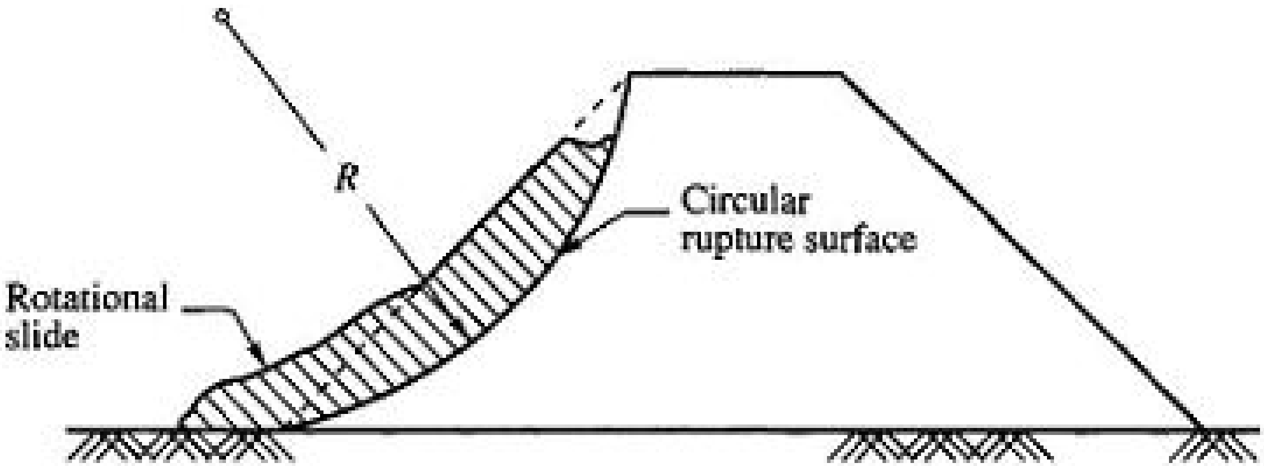
The base failure generally occurs particularly when

the soil beneath the embankment is
softer and more plastic than
the slope forming soil itself.

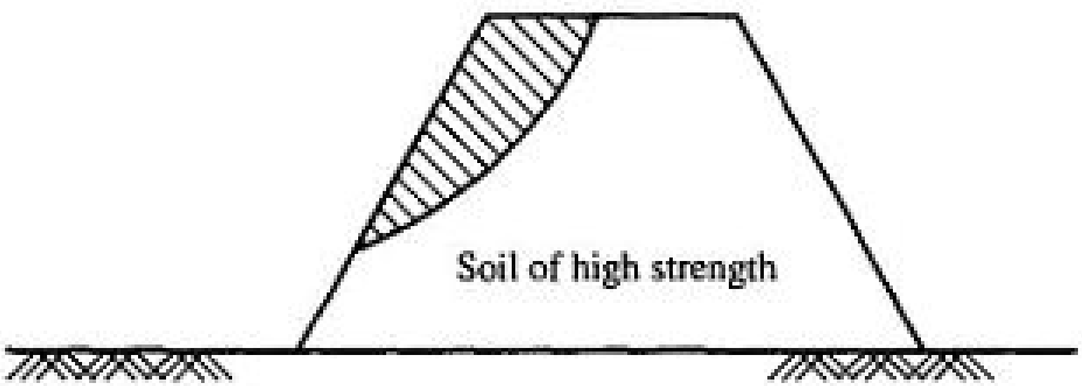
Wedge failure

Wedge failure results when rock mass slides along two intersecting discontinuities thus forming a wedge-shaped block

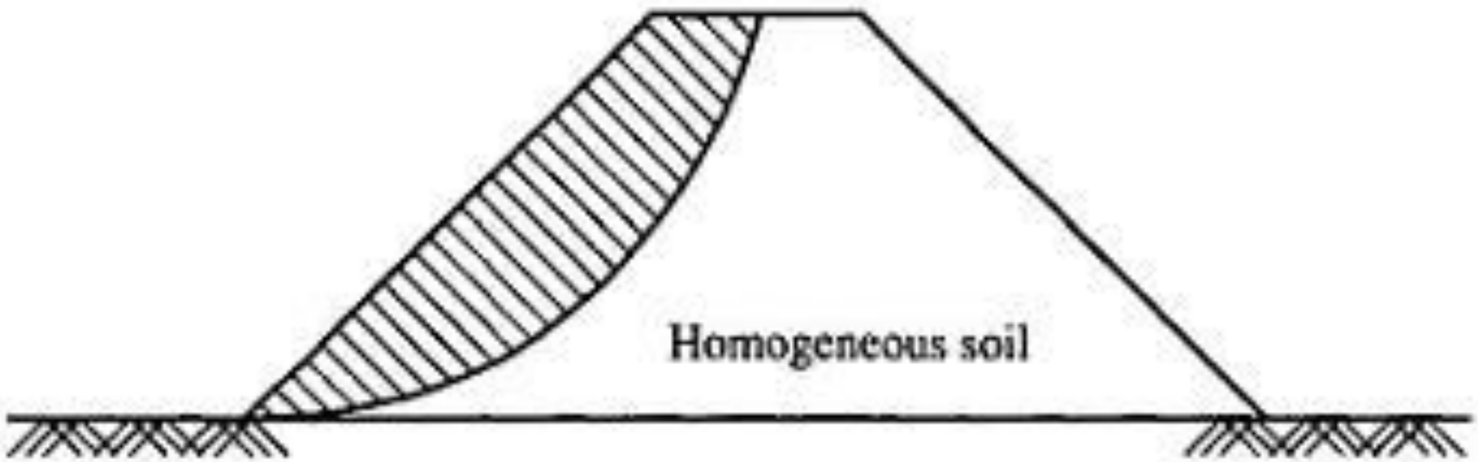




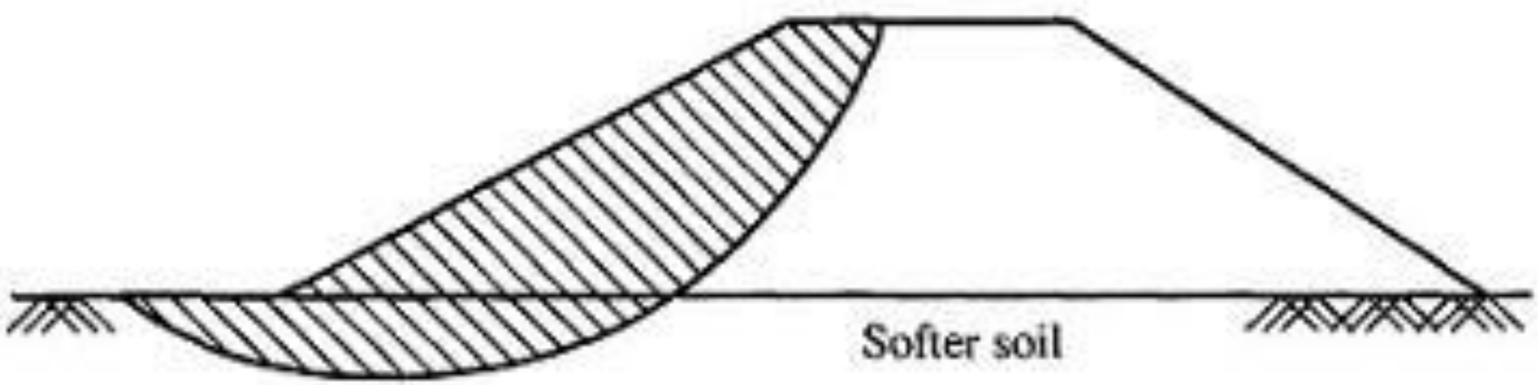
(a) Rotational slide



(b) Slope failure



(c) Toe failure



(d) Base failure

methods of stability analysis of finite slopes

Ordinary Method of Slices(Fellenius method)

Friction circle method

By use of Taylor stability number

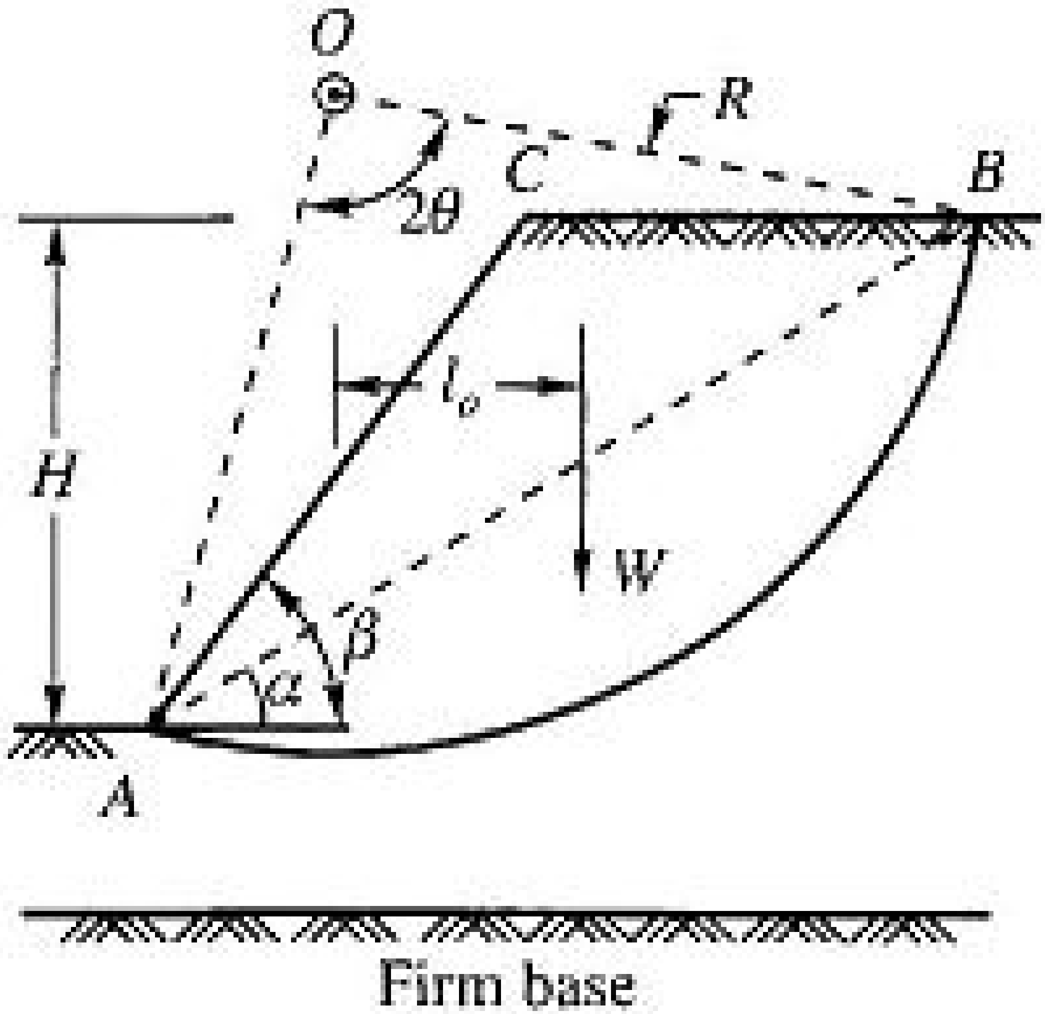
Bishop’s method.

FAILURE UNDER UNDRAINED CONDITIONS ($\phi_u = 0$)

A fully saturated clay slope may fail under undrained conditions ($\phi_u = 0$) immediately after construction. The stability analysis is based on the assumption that the soil is homogeneous and the potential failure surface is a circular arc. Two types of failures considered are

1. Slope failure
2. Base failure

The undrained shear strength c_u of soil is assumed to be constant with depth. A trial failure circular surface AB with center at O and radius R is shown in Fig. for a toe failure. The slope AC and the chord AB make angles β and α with the horizontal respectively.



W is the weight per unit length of the soil lying above the trial surface acting through the center of gravity of the mass.

l_o is the lever arm,

L_a is the length of the arc,

L_c the length of the chord AB and

c_m the mobilized cohesion

β = Angle of slope with respect to horizontal

α = Angle of Chord line with respect to horizontal

for any assumed surface of failure.

We may express the factor of safety F_s as

$$F_s = \frac{c_u}{c_m}$$

For equilibrium of the soil mass

lying above the assumed failure surface,

we may write

resisting moment M_r = actuating moment M_a

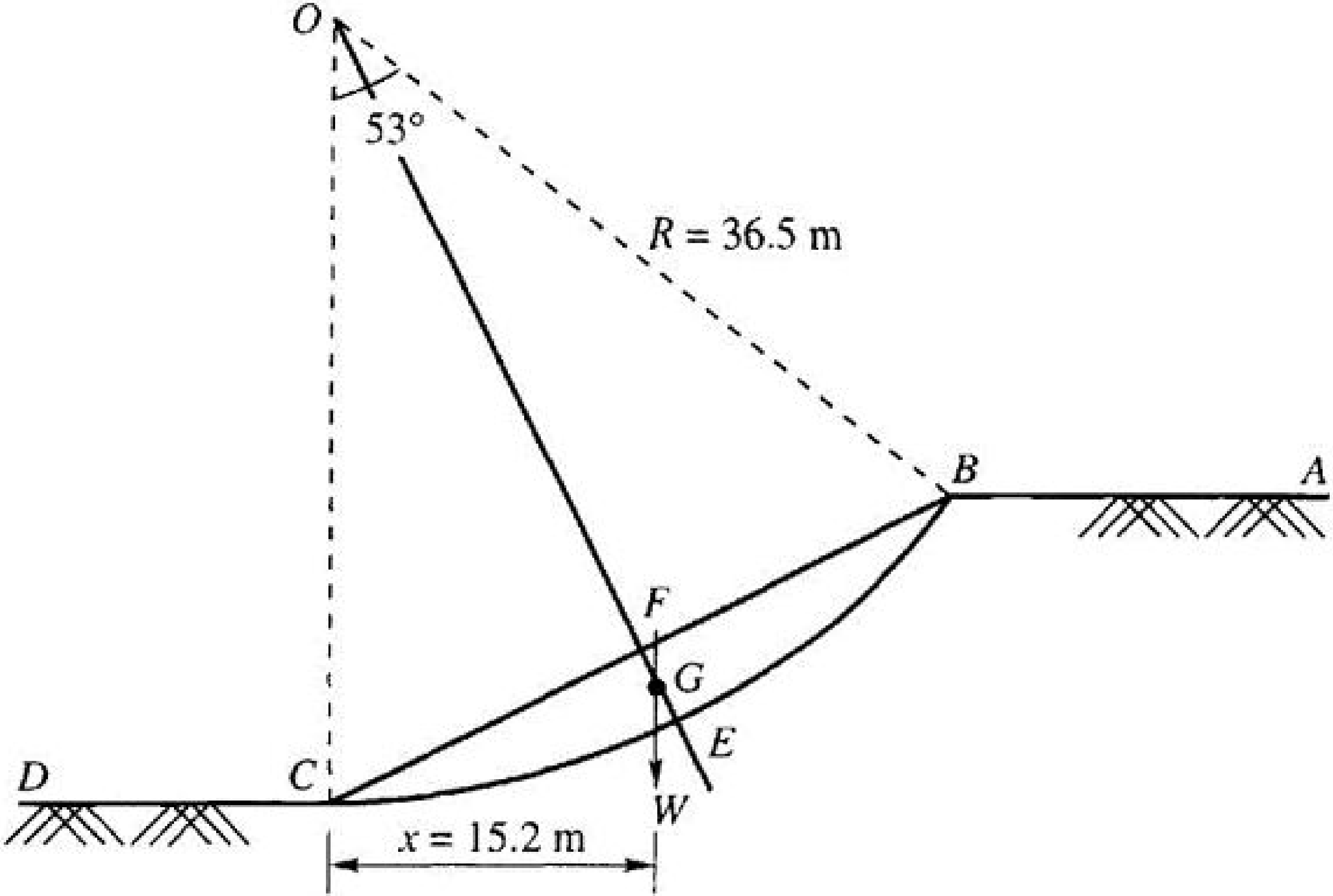
The resisting moment $M_r = L_a c_m R$

Actuating moment, $M_a = W l_o$

Equation for the mobilized c_m is

$$c_m = \frac{W l_o}{L_a R}$$

Calculate the factor of safety against shear failure along the slip circle shown in Fig.
Assume cohesion = 40 kN/m^2, angle of internal friction = zero and the total unit weight of soil = 20.0 kN/m^3.



Solution

Draw the given slope *ABCD* as shown in Fig.

To locate the center of rotation

extend the bisector of line *BC* to cut the vertical line drawn from *C* at point *O*.

With *O* as center and *OC* as radius,

draw the desired slip circle.

Radius $OC = R = 36.5$ m, Area $BECFB = \frac{2}{3} \times EF \times BC$

$$= \frac{2}{3} \times 4 \times 32.5 = 86.7 \text{ m}^2$$

Therefore $\quad W = 86.7 \times 1 \times 20 = 1734$ kN

W acts through point G which may be taken as the middle of FE.

From the figure we have, $x = 15.2$ m, and $\theta = 53^\circ$

$$\text{Length of arc } BEC = R\theta = 36.5 \times 53^\circ \times \frac{3.14}{180} = 33.8 \text{ m}$$

$$F_s = \frac{\text{length of arc} \times \text{cohesion} \times \text{radius}}{Wx} = \frac{33.8 \times 40 \times 36.5}{1734 \times 15.2} = 1.87$$

FRICTION-CIRCLE METHOD

This method is very useful for the stability analysis of homogeneous soils.

In this method the slip circle is assumed to be an arc of the circle.

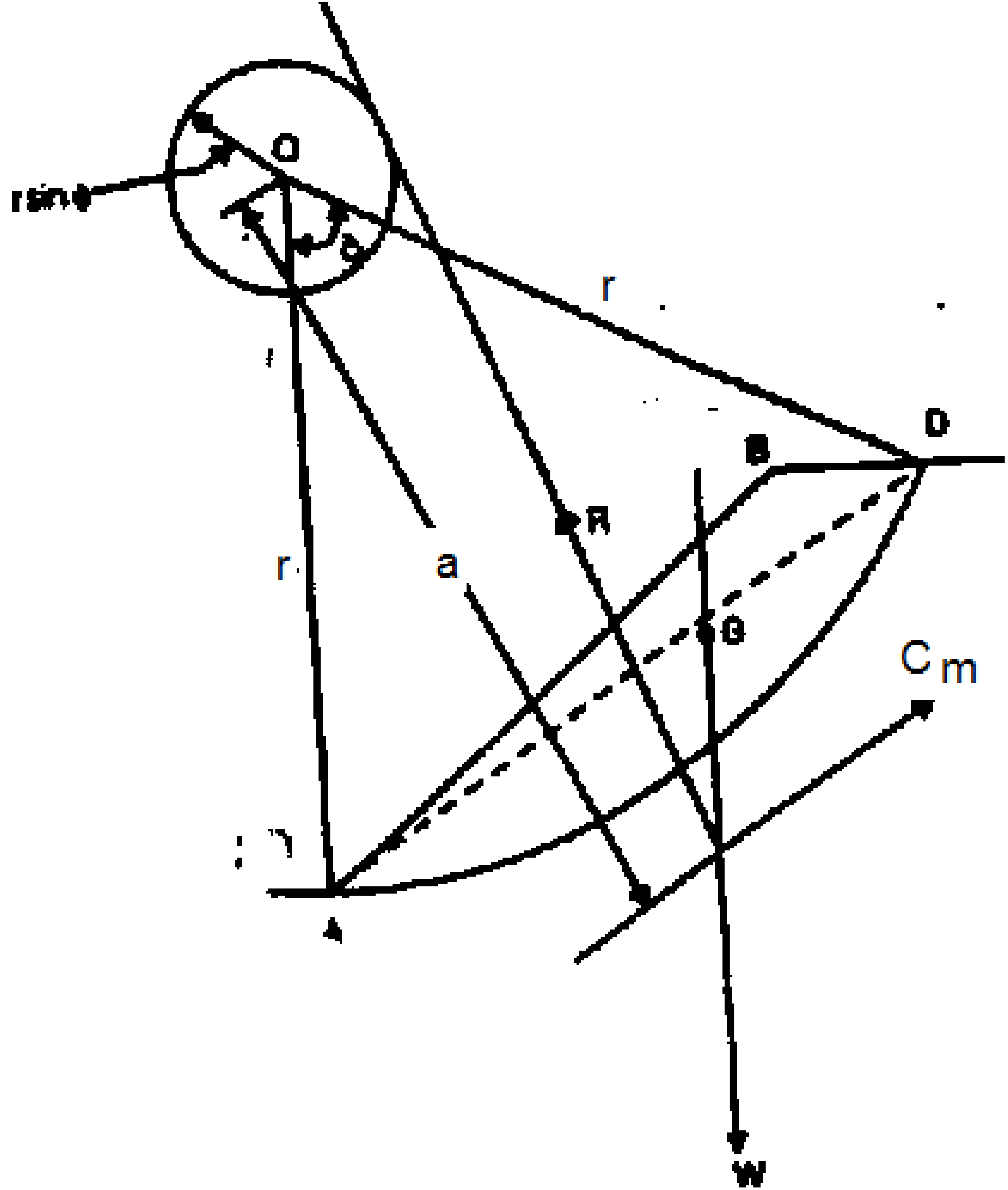
With the centre **O** a small circle is drawn with a radius of $\mathbf{r\sin\varphi_m}$ known as friction circle.

Where, φ_m = **Mobilized friction**

It is obtained from the following equation.

By assuming the value of $\mathbf{F\varphi}$

$$F\varphi = \frac{\tan\varphi}{\tan\varphi_m}$$



O
r sin θ
r
r
a
R
B
D
G
C m
A
W

1. With centre O and radius r, the slip circle AD is constructed.

The friction circle is drawn with centre O and radius Kr sinϕ.

K is taken as 1 unless otherwise given.

A vertical line is drawn through centroid of section ABDA,

to get the line of action of weight W.

3. A line drawn parallel to chord AD and at distance 'a'

to get the line of action of resultant cohesive force c_m

$$a = r.\frac{\hat{L}}{L} \text{ from O,}$$

$\hat{L}$ is the length of arc AD.

$$\hat{L} = \frac{\pi r \hat{\delta}}{180}.$$

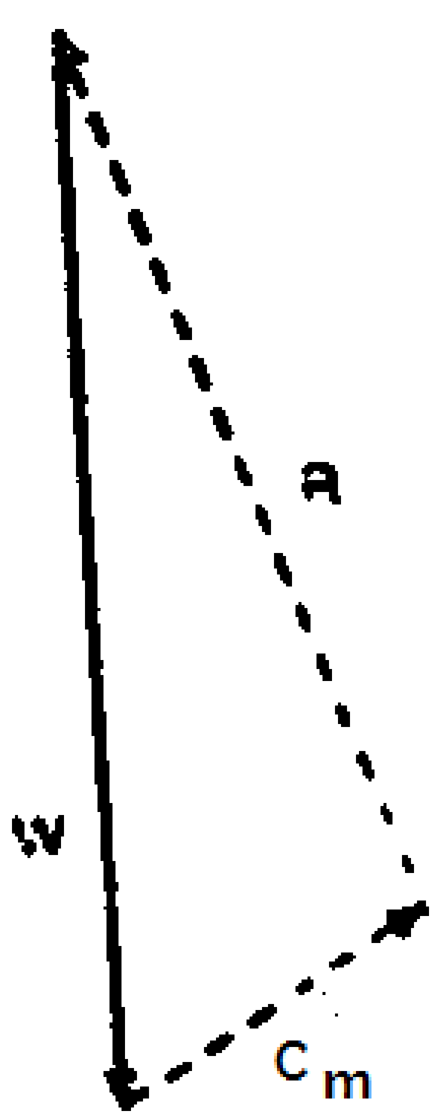
The length of chord AD = L is obtained by measurement.

4 Through the
point of intersection of the
lines of action of forces W and $c_m \hat{L}$,

a line is draw tangential to the friction circle,

to get the line of action of
resultant reaction R.

5. The weight W of the sliding soil mass ABDA is computed and plotted to scale as shown in figure. Through the ends of the vector representing W, lines are drawn parallel to the lines of action of forces $c_m \hat{L}$ and R to complete the triangle of forces.


c m

6 . The value of $c_m \hat{L}$ is obtained from the force triangle and divided by value of $\hat{L}$ to obtain the value of mobilized cohesion c_m.

7 The factor of safety with respect to cohesion.

Fc is given by, $F_c = \frac{c}{c_m}$

Where c = ultimate cohesion.

If F_C is not equal to the assumed value of F_ϕ than

the procedure is repeated for different assumed values of F_ϕ and

the slip surface which gives the minimum factor of safety is

the most critical circle.

Note:

For pure cohesive soils $\mathbf{F_{\phi}}$ is assumed to be unity and

the factor of safety with respect to cohesion only determined.

In the case of submerged slope the procedure is same but

submerged unit weight should be used in the place of

dry unit weight when calculating the weight of the failure wedge.

Problem

A slip surface with a radius of 22 m in a slope with a height of 14 m and

an angle of inclination of 45°. If the mobilized friction is 15°,

unit weight of soil is 18 kN /m^3 and cohesive strength of soil is 40 kN / m^2,

determine the factor of safety with respect to cohesion

using friction circle method..

Take the weight of slip surface is 1500 kN and

the angle subtended by the slip surface is 64°.

$R \sin \varphi_m = 22 \sin 15° = 5.7$ m.

The friction circle is drawn with a radius of 5.7 m

$$a = \frac{r\,L_a}{L_c} = \frac{22 \times 2\pi \times 22 \times (64/360)}{2 \times 22 \times \sin(64/2)} = 23.20 \text{ m}$$

A line parallel to chord AB @ a distance of 23.20m from

centre of friction circle is drawn.

A vertical line is dawn from the point of center of gravity of

the failure surface which intersects

the cohesive force line at a point **P**.

The line of reaction **R** should be drawn such that

it should tangent to the friction circle and

it should pass through the point **P**

Now draw a force triangle with sides of lines

which are parallel to **R, C** and **W** as shown below.

From the force triangle Value of **C** measured as 600 kN.

$$\mathbf{C_m = \frac{C}{L_C} = \frac{600}{22.32} = 25.7\ kN / m^2}$$

$$\mathbf{F_C = \text{Factor of safety against cohesion} = \frac{40}{25.7} = \underline{1.56}}$$

Example 13.2

Calculate the factor of safety for the slip circle

shown in Figure for a cutting in a

purely cohesive soil with $\phi_u = 0$,

$c_u = 30\ kN/m^2$ and $\gamma = 20.5\ kN/m^3$

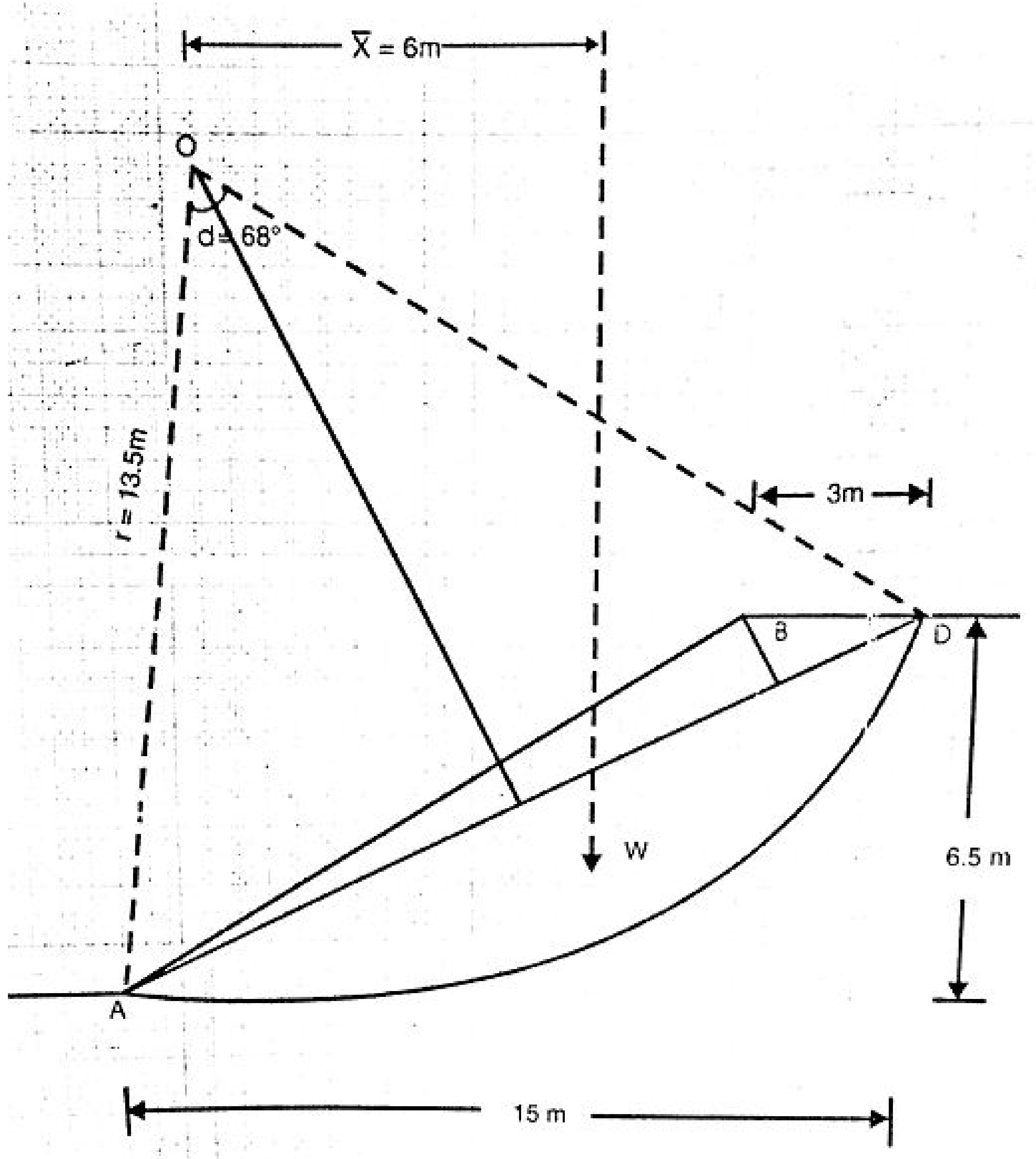
Solution:

$c_u = 30\ kN/m^2$ and $\gamma = 20.5\ kN/m^3$

From plot, $\delta = 68°$.

Length of are AD,

$$\hat{L} = \frac{\pi r \delta}{180} = \frac{\pi\,(13.5)\,(68)}{180} = 16.02m$$



X̄ = 6m
O
d = 68°
r = 13.5m
3m
B
D
W
6.5 m
A
15 m

Area of section ABDA =

area of sector OAD - area of triangle OAD + area of triangle ABD

$$= \frac{1}{2}(13.5)^2\left(68\times\frac{\pi}{180}\right) - \frac{1}{2}(15)(11.1) + \frac{1}{2}(15)(1.2)$$

$= 33.89\ \mathrm{m}^2$

Weight of sliding soil mass ABDA = 33.89×20.5 =694.75 kN

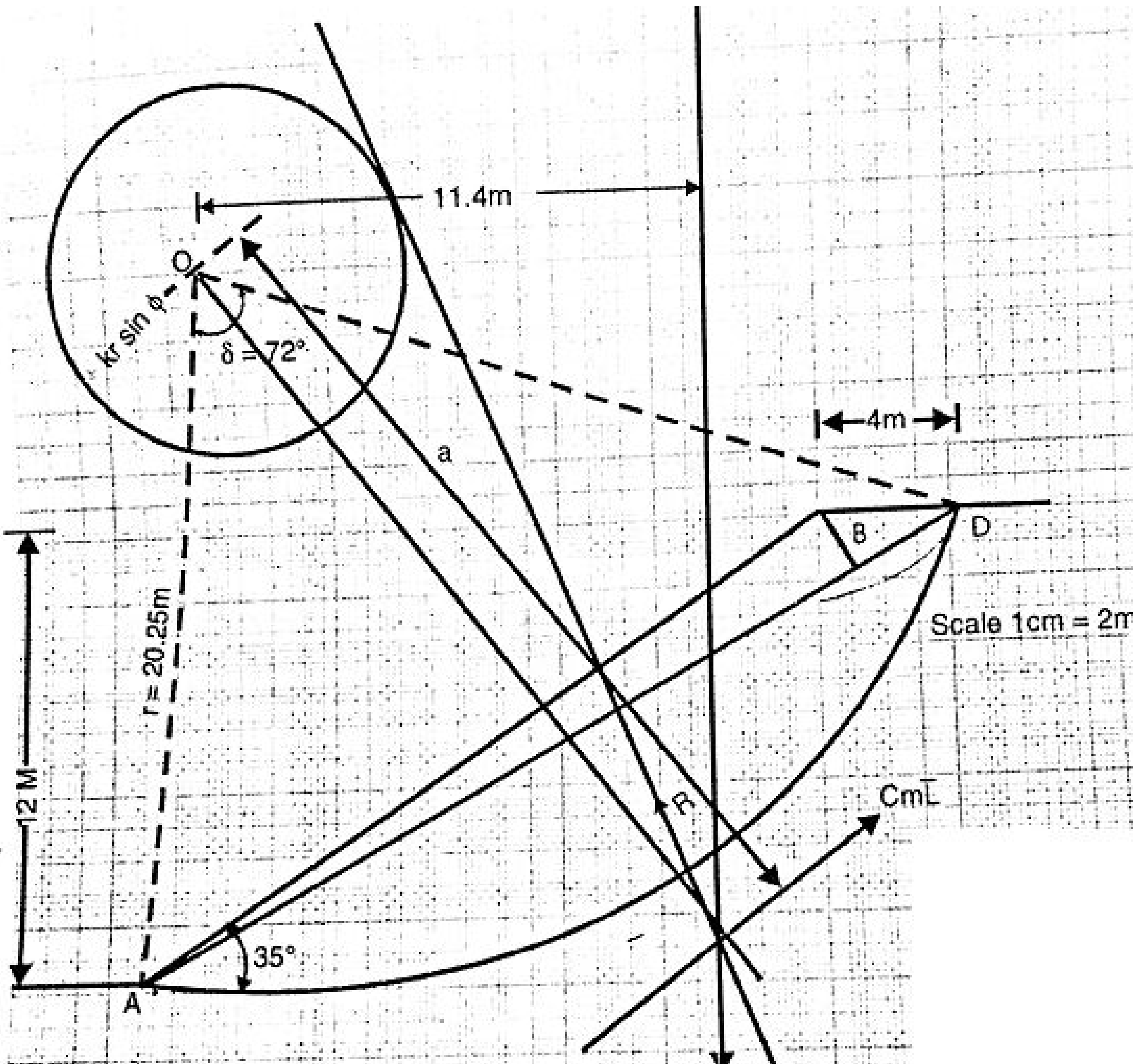
Factor of safety against sliding,

$$F = \frac{M_R}{M_D} = \frac{c\hat{L}r}{W\bar{x}} = \frac{(30)(16.02)(13.5)}{(694.75)(6)} = 1.56$$

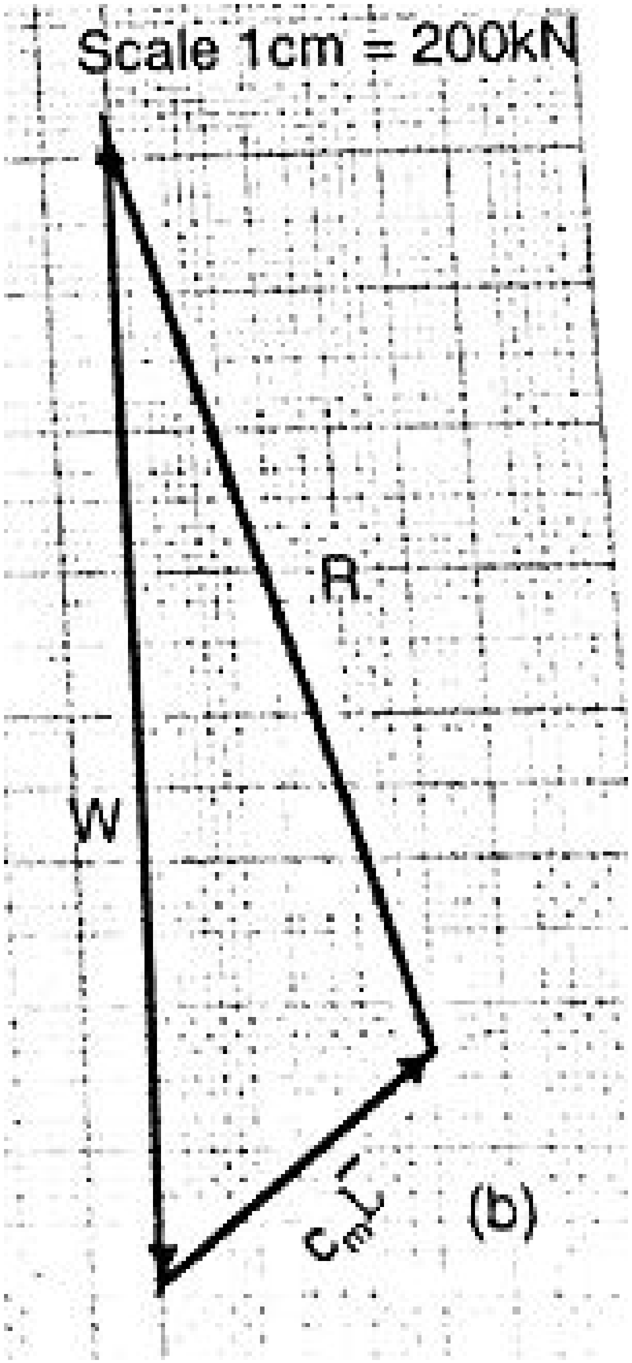
Example 13.3

Determine the factor of safety against sliding for the slip circle shown in the figure

The properties of soil are $c = 30\ kN/m^2$, $\phi = 15^\circ$ and $\gamma = 21\ kN/m^3$.



(a)



Scale 1cm = 200kN
R
W
$c_m\bar{L}$
(b)

Solution :

$c = 30 \text{ kN/m}^2, \phi = 15°,$

$\gamma = 21 \text{ kN/m}^3$

Radius of friction circle = Kr

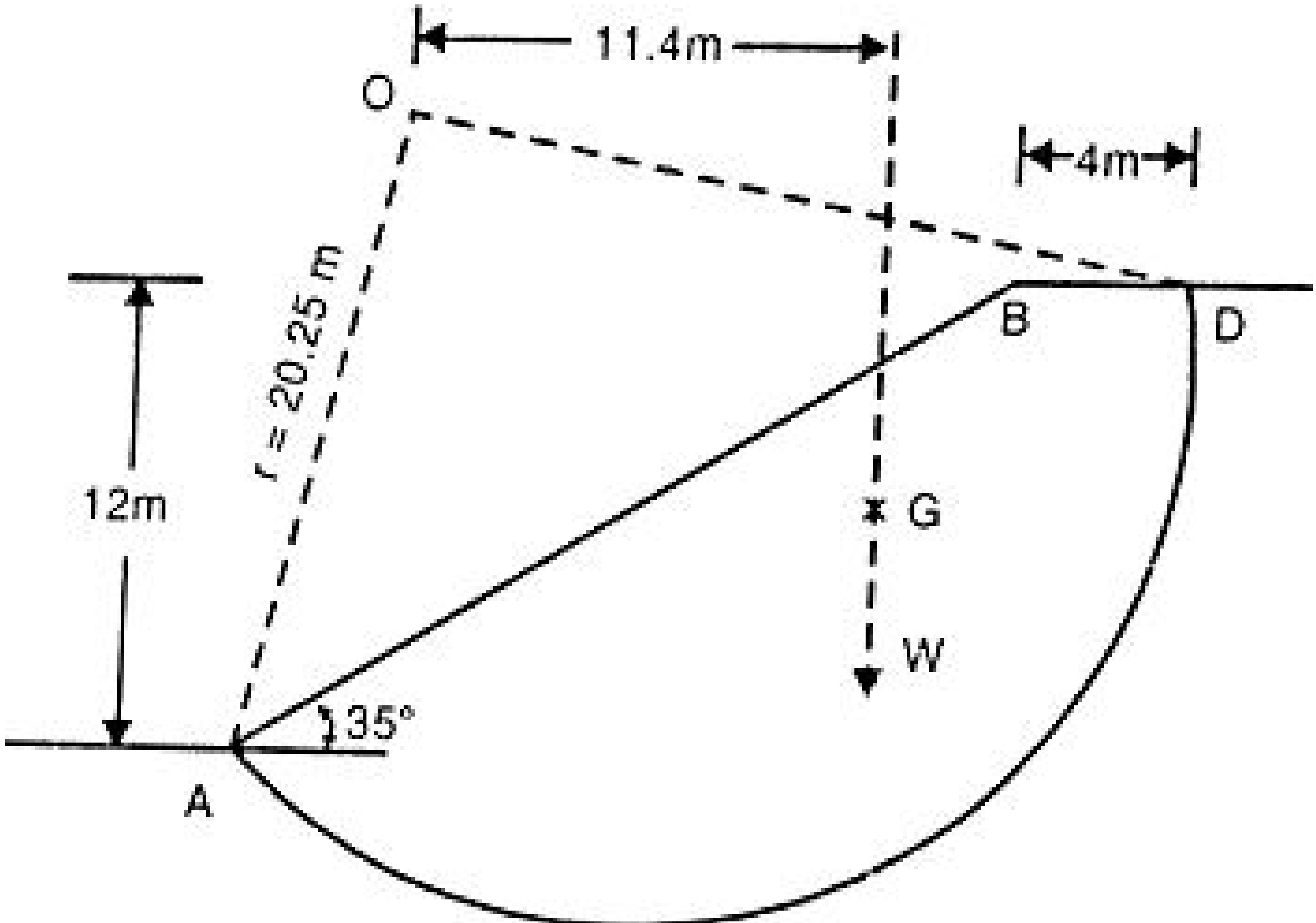
$\sin\phi = (1)(20.25) \text{ Sin } 15° = 5.25\text{m}$

Length of chord AD,

$\bar{L} = 23.6 \text{ m}$

Length of arc AD,

$$\hat{L} = \frac{\pi r \delta}{180} = \frac{\pi(20.25)(72)}{180} = 25.45\,\text{m}$$



11.4m
O
4m
r = 20.25 m
12m
B
D
G
W
35°
A

Area of section ABDA = area of sector AOD - area of triangle AOD + area of triangle ABD

$$= \frac{1}{2}(20.25)^2\left(\frac{72\times\pi}{180}\right) - \frac{1}{2}(23.6)(16.6) + \frac{1}{2}(23.6)(2) = 85.36\,\text{m}$$

Weight of sliding soil mass ABDA, W = $85.36 \times 21 = 1792.6$ kN (Per meter length perpendicular to plane of fig)

From force triangle [Fig (*b*)], $c_m\,\bar{L} = 2.8 \times 200 = 560$ kN

$\therefore$

$$c_m = \frac{c_m\,\bar{L}}{\bar{L}} = \frac{560}{23.6} = 23.73\ \text{kN/m}^2$$

$$F_c = \frac{c}{c_m} = \frac{30}{23.73} = 1.26$$

Method of Slices

Assume some failure surface

Divide failure surface into smaller elements (slices)

Curved bottom of each slice approximated as chord
More slices = more refined solution(10-40 slices sufficient)

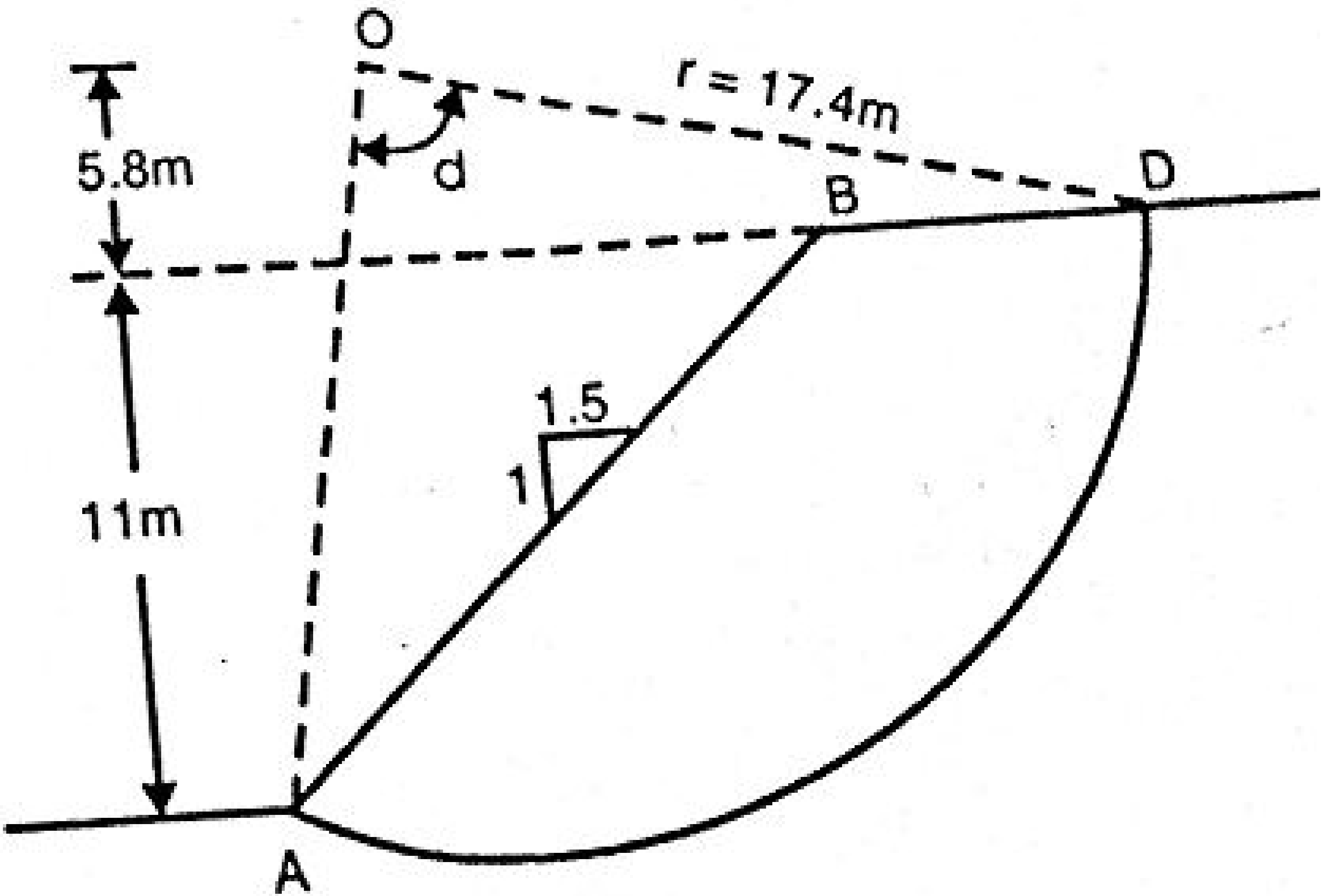
Calculate factor of safety for each slice and

overall factor of safety

Find lowest FS for different failure surfaces

Example 13.1

Determine the factor of safety against sliding for the slip surface shown in the accompanying sketch. The properties of soil are $c = 15\ kN/m^2$, $\phi = 32°$ and $\gamma = 20\ kN/m^3$. Use Swedish method of slices.



O
r = 17.4m
5.8m
d
D
B
1.5
1
11m
A

Solution

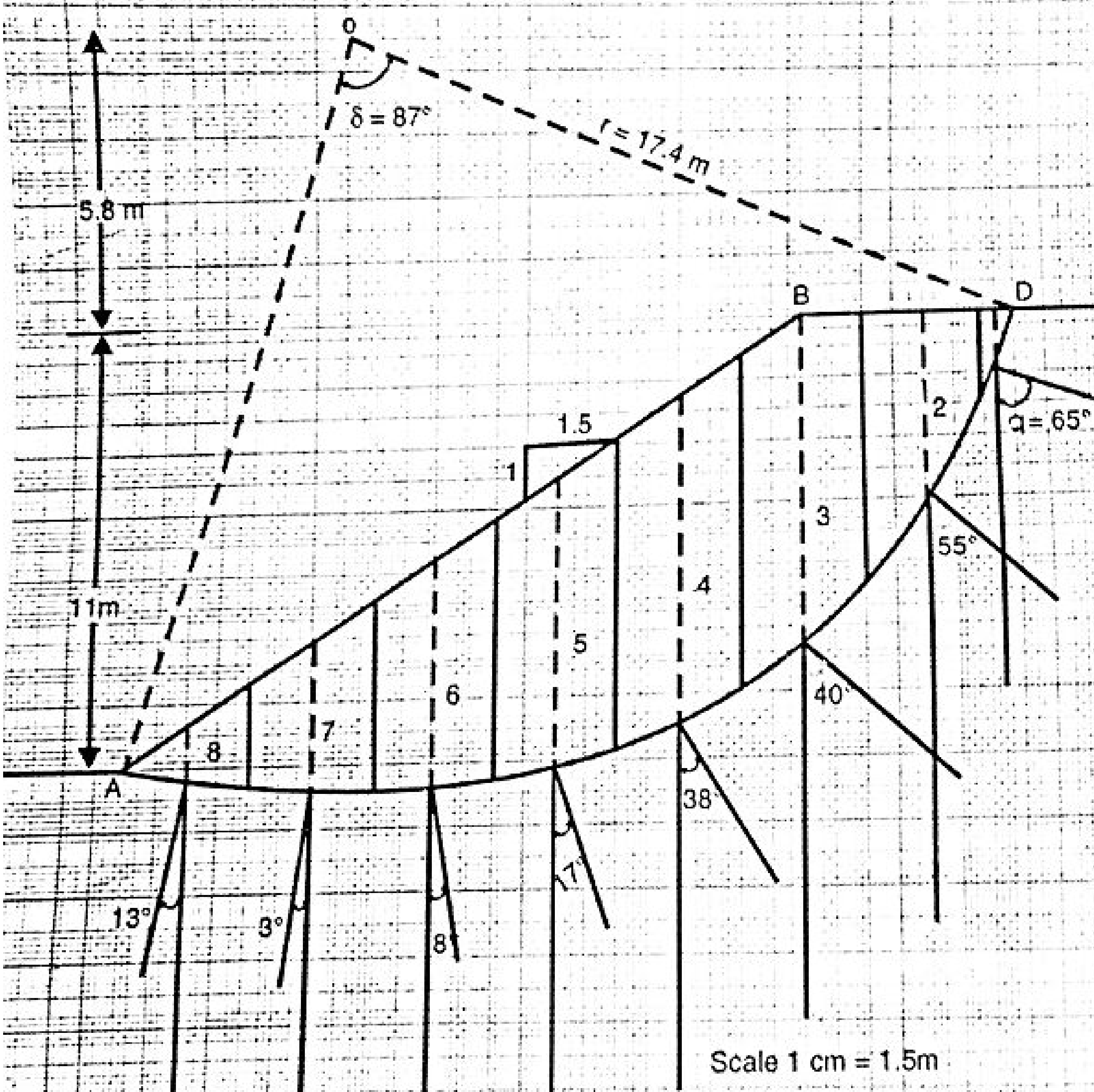
$c = 15\ \text{kN/m}^2$ $\qquad\qquad$ $\phi = 32°$

$\gamma = 20\ \text{kN/m}^3$

From plot, $\delta = 87°$

Length of are AD,

$$\hat{L} = \frac{\pi r \delta}{180°} = \frac{\pi(17.4)(87)}{180} = 26.42m$$



O
δ = 87°
r = 17.4 m
5.8 m
11m
B
D
1.5
1
2
q = 65°
3
55°
4
5
40°
6
7
8
38°
A
17°
13°
3°
8°
Scale 1 cm = 1.5m

Slice No.	Mid-ordinate (m)	Width (m)	Wt. of Slice W (kN)	θ (degrees)	N = W Cos θ (kN)	T = W Sin θ (kN)
1	1.35	0.75	20.25	65	8.56	18.35
2	4.35	3.0	261.00	55	149.70	213.80
3	7.65	3.0	459.00	4.	351.61	295.04
4	7.50	3.0	450.00	38	354.60	277.05
5	6.90	3.0	414.00	17	395.91	121.04
6	5.55	3.0	333.00	8	329.76	46.34
7	3.90	3.0	234.00	3	233.68	– 12.25
8	1.35	3.0	81.00	13	78.92	– 18.22
					ΣN = 1902.74	ΣT = 941.15

Factor of safety against sliding,

$$F = \frac{c\hat{L} + \sum N \tan \phi}{\sum T}$$

$$= \frac{(15)(26.42) + (1902.74) \tan 32^\circ}{941.15} = 1.68$$

Example 13.4

Stability analysis by Swedish method of slices gave following values per running metre for a 10m high embankment.

- (*i*) Total shearing fore = 480 kN
- (*ii*) Total normal force = 1950 kN
- (*iii*) Total neutral force = 250 kN
- (*iv*) Length of arc = 22 m

If the properties of soil arc $c = 24\ \text{kN/m}^2$ and $\phi = 6°$, calculate the factor of safety with respect to shear strength.

Solution :

$$\Sigma T = 480 \text{ kN}$$

$$\Sigma N = 1950 \text{ kN}$$

$$\Sigma U = 250 \text{ kN}$$

$$\text{Length of arc, } \hat{L} = 22 \text{ m}$$

$$c = 24 \text{ kN/m}^2$$

$$\phi = 6°$$

$$F = \frac{c\,\hat{L} + \Sigma\,(N - U)\tan\phi}{\Sigma T}$$

$$= \frac{(24)\,(22) + (1950 - 250)\,\tan 6°}{480} = 1.47$$

Different methods of slope protection:

Unloading:

Decrease the shear stress by unload it

- by reducing the slope height (or)
- Increase the slope ratio (or) using light weight fill.

Buttressing:

The usual construction procedure is to

over excavate the proposed cut slope,

then bring it back to design grades

using high – quality light weight fill.
(with high, c & ϕ)

Structural Stabilization:

Retaining walls

These are structural member that maintain the adjacent ground surface at the different elevations

Tie back Anchors:

Steel rods are inserted into grouted holes that extend well beyond the critical failure surface.

Drainage

Slope stability can be improved by

draining water

both from

surface and sub Surface.

-

Surface drainage

providing appropriate grades
so as to

surface water to flow
away from the slope
instant of towards it

Subsurface Drainage

special pipe with holes buried in the ground to

collect water at and carry to safe location.

a. **Wells:**

These are vertical holes drilled into the ground and equipped with pumps to remove the water.

b. **Horizontal Drains:**

Horizontal holes are drilled from the slope face and are slightly inclined upward

Reinforcement:

installing synthetic reinforcement

such as

thin steel strips, special grids and geo-textiles.

Vegetation:

Planting appropriate vegetation

on the surface of slope

provide

erosion protection,

reinforcement of the soil

and aesthetic value

Densification by use of Explosives

Vibro -flotation helps to

increase the shear strength of cohesion less soils and thus

increase stability.

Consolidation

surcharging,

Electro osmosis process etc

helps to increase

the stability of slope in cohesive soils.

Grouting and injection:

cement and other chemicals are applied

Sheet piles

It can be installed to provide lateral support

Thank you