

Geotechnical Engineering II

PCC-CE304

Module 1:*Consolidation of Soil* - Introduction, comparison between compaction and consolidation, initial, primary & secondary consolidation, spring analogy for primary consolidation, interpretation of consolidation test results, Terzaghi's theory of consolidation, final settlement of soil deposits, computation of consolidation settlement and secondary consolidation.

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

- Understand the basic mechanism of consolidation of soil;
- Determine various consolidation parameters of soil through laboratory test; □ Evaluate ground settlements against time.

Module 2:*Shear Strength* - Mohr circle and its characteristics, principal planes, relation between major and minor principal stresses, Mohr-Coulomb theory, types of shear tests: direct shear test, merits of direct shear test, triaxial compression tests, test behaviour of UU, CU and CD tests, pore-pressure measurement, computation of effective shear strength parameters. unconfined compression test, vane shear test On completion of this module, the student must be able to:

- Determine graphically and analytically the stress state in any plane of the soil mass; □ Perform various shear strength tests and appreciate the different field conditions which they simulate;
- Understand the significance of shear strength parameters in various geotechnical analyses;
- Evaluate the stiffness of soil using shear strength parameters

Module 3:*Stability of Slopes* - Introduction, types of slopes and their failure mechanisms, factor of safety, analysis of finite and infinite slopes, wedge failure Swedish circle method, friction circle method, stability numbers and charts. On completion of this module, the student must be able to:

- Differentiate various modes of slope failure;
- Evaluate factor of safety of infinite slopes based on different ground conditions; □
Understand various methods for computation of factor of safety for finite slopes.

Module 4: Soil Exploration- Introduction, methods of site exploration and soil investigation, methods of boring, soil samplers, sampling procedures, trial pits, borings, penetrometer tests, analysis of borehole logs, geophysical and advance soil exploration methods.

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

- Specify a strategy for site investigation to identify the soil deposits and determine the depth and spatial extent within the ground;
- Understand various site investigation techniques and their in-situ applications; □ Prepare a soil investigation report based on borehole log data and various in-situ tests like SPT, CPT, etc.

Module 5 Application of soil mechanics to determine earth pressures, analysis of retaining walls, cuts & excavations and sheet piles, stability of slopes, instrumentation.

Text/Reference Books:

9. Soil Mechanics by Craig R.F., Chapman & Hall
10. Fundamentals of Soil Engineering by Taylor, John Wiley & Sons
11. An Introduction to Geotechnical Engineering, by Holtz R.D. and Kovacs, W.D., Prentice Hall, NJ
12. Principles of Geotechnical Engineering, by Braja M. Das, Cengage Learning
13. Principles of Foundation Engineering, by Braja M. Das, Cengage Learning
14. Essentials of Soil Mechanics and Foundations: Basic Geotechnics by David F. McCarthy
15. Soil Mechanics in Engineering Practice by Karl Terzaghi, Ralph B. Peck, and Gholamreza Mesri.
16. Geotechnical Engineering: Principles and Practices of Soil Mechanics and Foundation Engineering (Civil and Environmental Engineering) by V.N.S. Murthy

Course outcome

- Understand the compaction and consolidation
- Calculate settlement due to consolidation.
- Evaluate the shear strength parameters of soil through different laboratory methods
- Apply the concepts of soil mechanics in stability of slopes
- Analyze earth pressure, retaining walls. Cuts and excavations and sheet piles.
- Apply different technique in soil exploration.

Review of Geotechnical Engineering-I

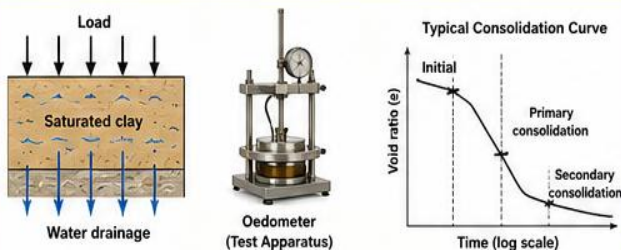
- Index Properties-
 - particle size distribution and Plasticity characteristics
- Engineering Properties
 - Shear strength of soil
 - Permeability of soil
 - Compressibility of soil- Compaction and consolidation

Geotechnical Engineering-II

Lecture 1

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

1 CONSOLIDATION OF SOIL



KEY TOPICS

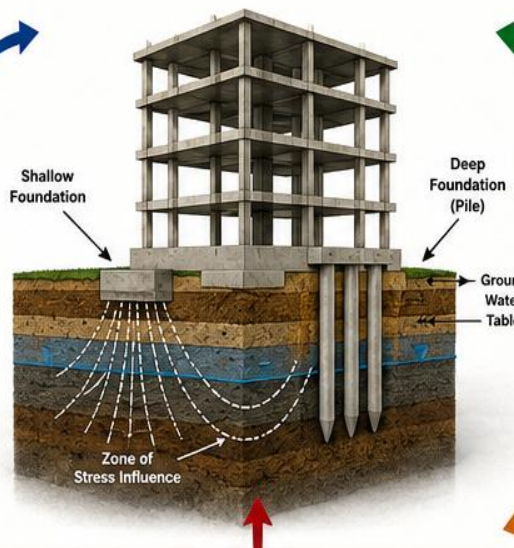
- Compaction vs Consolidation
- Initial, Primary & Secondary Consolidation
- Spring Analogy for Primary Consolidation
- Terzaghi's Theory of Consolidation
- Consolidation Test (Oedometer Test)
- Final Settlement Computation

LEARNING OUTCOMES

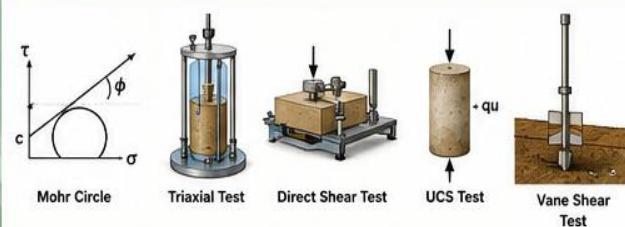
- ✓ Understand basic mechanism of consolidation
- ✓ Determine consolidation parameters from laboratory test
- ✓ Evaluate ground settlements against time

GEOTECHNICAL ENGINEERING – II

CONSOLIDATION • SHEAR STRENGTH • SLOPE STABILITY
SOIL EXPLORATION • GEOTECHNICAL APPLICATIONS



2 SHEAR STRENGTH



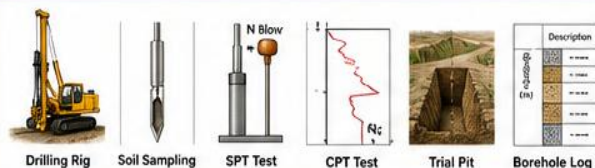
KEY TOPICS

- Principal Planes & Principal Stresses
- Mohr Circle and its Characteristics
- Mohr-Coulomb Failure Theory
- Direct Shear Test – Merits & Limitations
- Triaxial Compression Tests (UU, CU, CD)
- Pore Water Pressure Measurement
- Unconfined Compression (UC) Test
- Vane Shear Test

LEARNING OUTCOMES

- ✓ Determine stress state in any plane of soil mass
- ✓ Perform shear strength tests and interpret results
- ✓ Evaluate shear strength parameters (c , ϕ , c_u)
- ✓ Assess soil stiffness/strength for geotechnical design

4 SOIL EXPLORATION



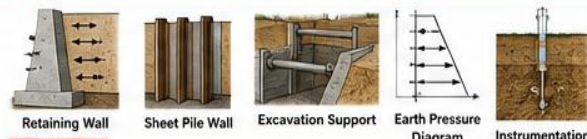
KEY TOPICS

- Methods of Site Exploration
- Boring Methods – Rotary, Wash, Percussion
- Soil Sampling – Disturbed & Undisturbed
- Trial Pits & Test Pits
- Penetrometer Tests – SPT, CPT
- Borehole Logs – Preparation & Interpretation
- Geophysical & Advanced Techniques

LEARNING OUTCOMES

- ✓ Plan site investigation strategy
- ✓ Identify soil strata and groundwater conditions
- ✓ Interpret borehole logs and in-situ test results
- ✓ Prepare soil investigation report

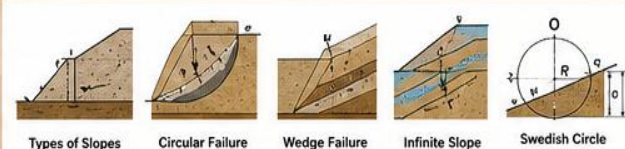
5 GEOTECHNICAL APPLICATIONS



KEY TOPICS

- Earth Pressure – At Rest, Active & Passive
- Analysis & Design of Retaining Walls
- Cuts & Excavations – Support Systems
- Sheet Piles – Design & Analysis
- Stability of Slopes
- Instrumentation & Monitoring in Geotechnical Works
- Practical Applications in Geotechnical Engineering

3 STABILITY OF SLOPES



KEY TOPICS

- Types of Slopes – Infinite & Finite
- Failure Mechanisms – Planar, Circular, Wedge, Toppling
- Factor of Safety – Concept & Importance
- Infinite Slope Analysis
- Finite Slope Analysis – Swedish Circle Method
- Friction Circle Method
- Stability Numbers & Charts

LEARNING OUTCOMES

- ✓ Differentiate various modes of slope failure
- ✓ Compute factor of safety for infinite slopes
- ✓ Determine factor of safety for finite slopes using Swedish circle method
- ✓ Use stability charts and design safe slopes

SOFTWARE & DESIGN APPLICATIONS



KEY REFERENCE BOOKS

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



OVERALL COURSE OUTCOMES



Analyze soil behavior under various loading conditions



Interpret laboratory and field test results



Design and evaluate geotechnical structures safely and economically



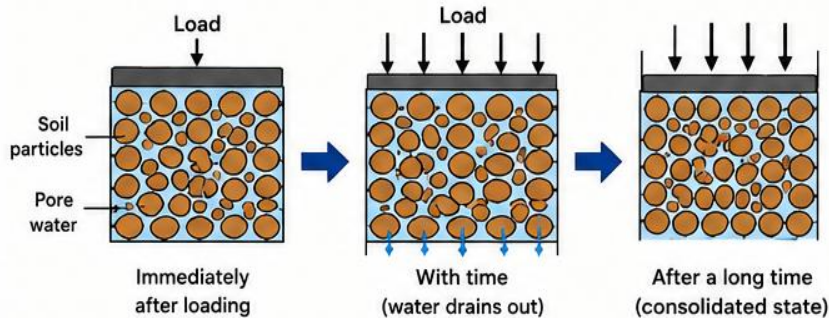
Apply geotechnical principles to real-world engineering problems

CONSOLIDATION OF SOIL

Consolidation is the **time-dependent** decrease in volume of a saturated soil due to the **expulsion of water** from its voids under **sustained loading**.

WHAT IS CONSOLIDATION?

When a load is applied to a saturated soil, the load is initially carried by the pore water. With time, water drains out of the voids and the soil particles come closer, causing a reduction in volume and settlement.



Result: Reduction in volume and settlement of the soil mass.

FACTORS AFFECTING CONSOLIDATION



Soil Type and Structure: Finer soils (clays and silts) consolidate more than coarser soils (sands and gravels).



Permeability of Soil: Lower permeability results in slower drainage of water and slower consolidation.



Thickness of Compressible Layer: Greater thickness increases the drainage path and time required for consolidation.



Magnitude of Applied Load: Higher load increases the amount of consolidation settlement.



Time Duration: Consolidation is a time-dependent process; with more time, more settlement occurs.



Drainage Conditions: Better drainage (e.g., sand drains, vertical drains) accelerates consolidation.

IMPORTANCE OF CONSOLIDATION



Settlement Prediction: Consolidation helps in estimating total and differential settlement of foundations, embankments, and other structures.



Safe and Economical Design: Ensures safe design of foundations and structures by considering time rate of settlement.



Time Factor: Consolidation is a time-dependent process; it is critical in projects where time rate of settlement is an important consideration.



Stability of Structures: Excessive consolidation settlement can lead to tilting, distress, and even failure of structures.



Embankments and Dams: Essential in assessing settlement and stability of earth structures such as embankments and dams.

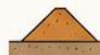


Ground Improvement Evaluation: Helps in evaluating the effectiveness of soil improvement and preloading techniques.

APPLICATIONS OF CONSOLIDATION



Foundation Design: Estimation of settlement of shallow and deep foundations on clayey soils.



Embankments: Predicting settlement and time rate of settlement of highways, railways, and earth embankments.



Dams and Reservoirs: Assessment of settlement of foundations and abutments under water pressure.



Multi-storied Buildings: Evaluation of long-term settlement to ensure serviceability and safety.



Bridges and Culverts: Designing foundations and approaches considering consolidation settlement.



Ground Improvement Projects: Assessment of performance of preloading, surcharge and vertical drains for accelerating consolidation.

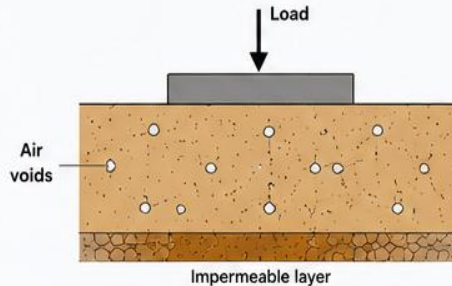
TYPES OF SETTLEMENT IN SOILS

When a load is applied to a foundation resting on soil, the total settlement that occurs with time is the sum of the following three components:

$$\text{Total Settlement} = \text{Immediate Settlement} + \text{Settlement due to Primary Consolidation} + \text{Settlement due to Secondary Consolidation}$$

1. IMMEDIATE SETTLEMENT (S_i)

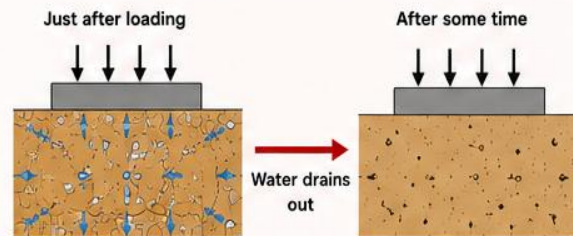
- It occurs immediately after the application of load.
- Caused by:
 - Compression of soil grains
 - Expulsion of air from the voids
 - No significant drainage of water



Very small in magnitude and occurs almost instantaneously (within seconds to minutes).

2. SETTLEMENT DUE TO PRIMARY CONSOLIDATION (S_c)

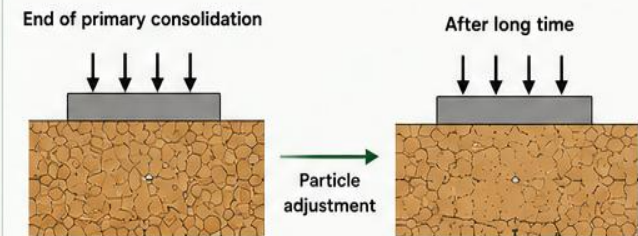
- It is the time-dependent settlement that occurs due to the expulsion of water from the voids.
- Caused by:
 - Dissipation of excess pore water pressure
 - Volume reduction of soil with constant mass
 - Occurs in saturated fine-grained soils (clays and silts)



It is the major component of settlement in saturated fine-grained soils and occurs over months to years.

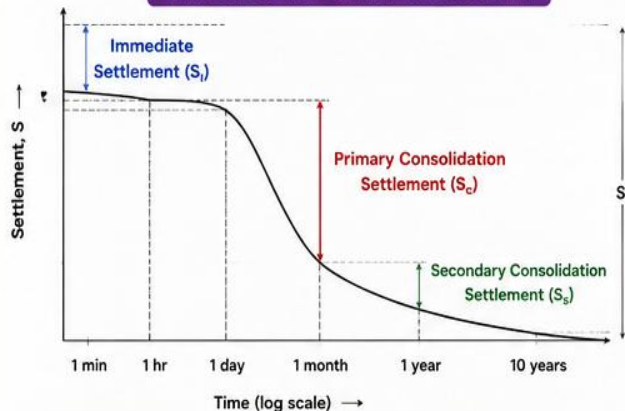
3. SETTLEMENT DUE TO SECONDARY CONSOLIDATION (S_s)

- It is the time-dependent settlement that occurs after the completion of primary consolidation.
- Caused by:
 - Adjustment or rearrangement of soil particles
 - Creep or plastic deformation of soil
 - Occurs at essentially constant effective stress



It is usually small compared to primary settlement but continues for a long period (years to decades).

TYPICAL SETTLEMENT-TIME CURVE



S_i = Immediate settlement (occurs instantly)

S_c = Settlement due to primary consolidation (time-dependent)

S_s = Settlement due to secondary consolidation (long-term)

S_t = Total settlement at any time t ($S_t = S_i + S_c + S_s$)

SUMMARY COMPARISON

Type of Settlement	Time of Occurrence	Cause	Drainage Involved?	Magnitude	Duration
Immediate Settlement (S_i)	Immediately after loading	Compression of soil grains and expulsion of air	No	Very small	Seconds to minutes
Primary Consolidation Settlement (S_c)	After loading and continues with time	Expulsion of pore water and volume reduction	Yes	Major portion (in clays/silts)	Months to several years
Secondary Consolidation Settlement (S_s)	After completion of primary consolidation	Adjustment/rearrangement of particles (creep)	No	Small	Years to decades

KEY POINTS

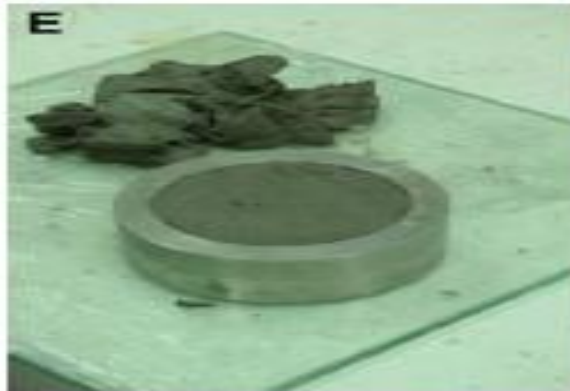
- Immediate settlement occurs instantly and is small.
- Primary consolidation is the major time-dependent settlement due to drainage of water.
- Secondary consolidation occurs after primary consolidation and continues for a long time.
- Total settlement is the sum of all three components at any time.

Geotechnical Engineering-II

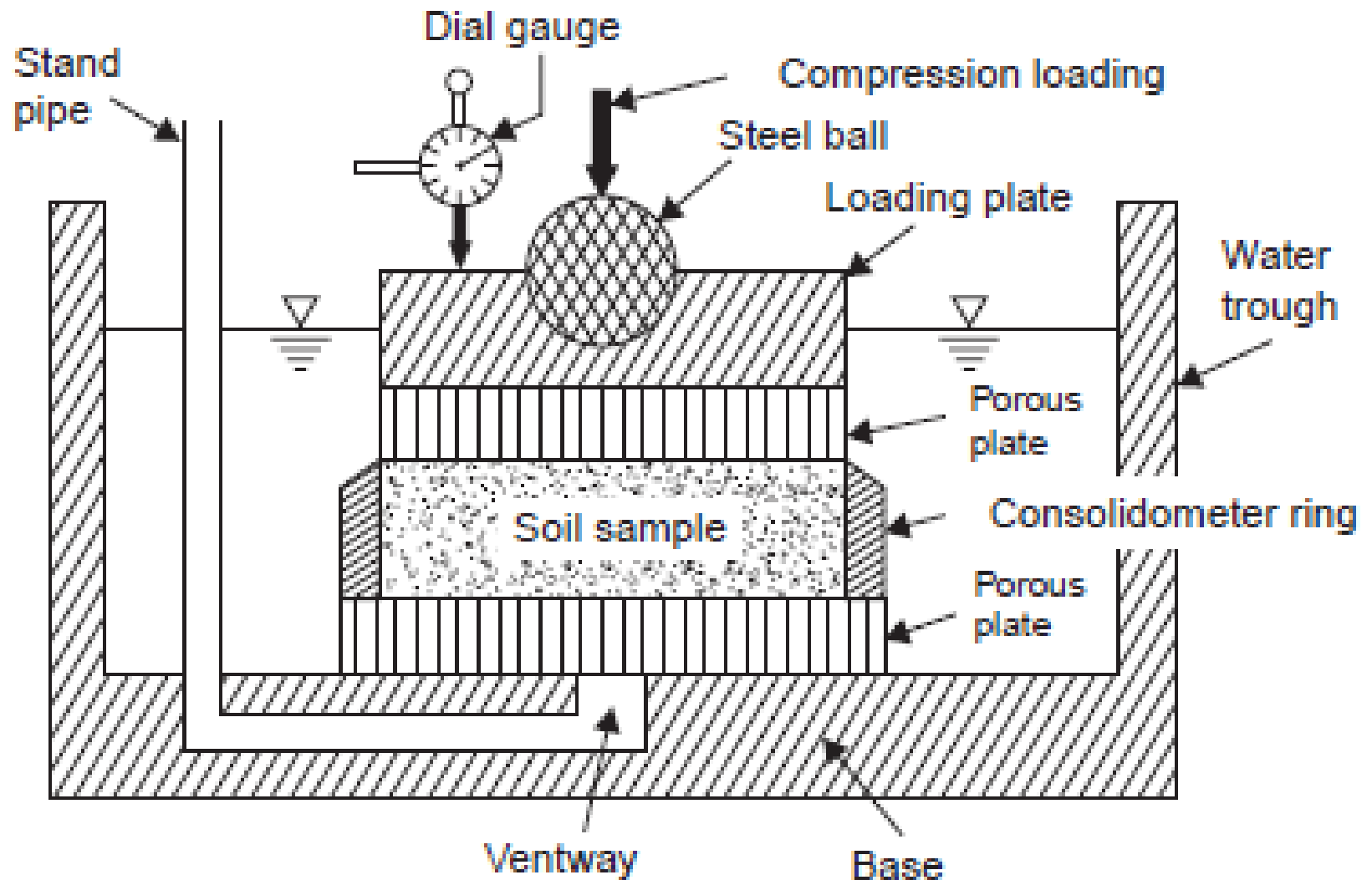
Lecture 2,3,4

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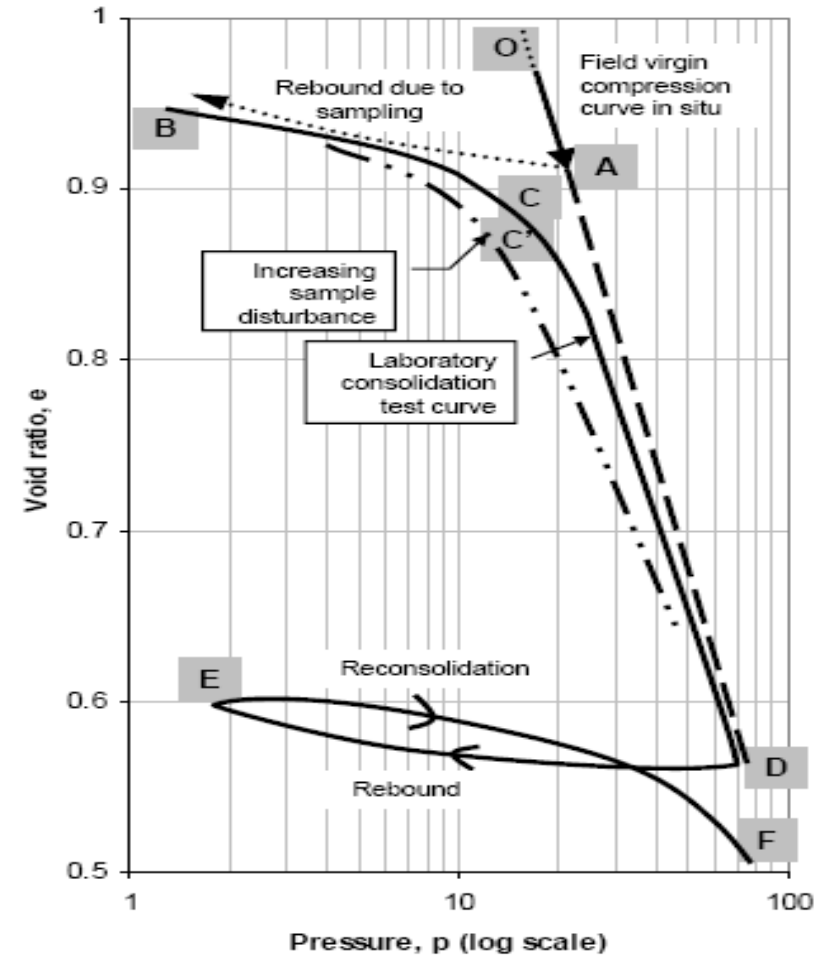
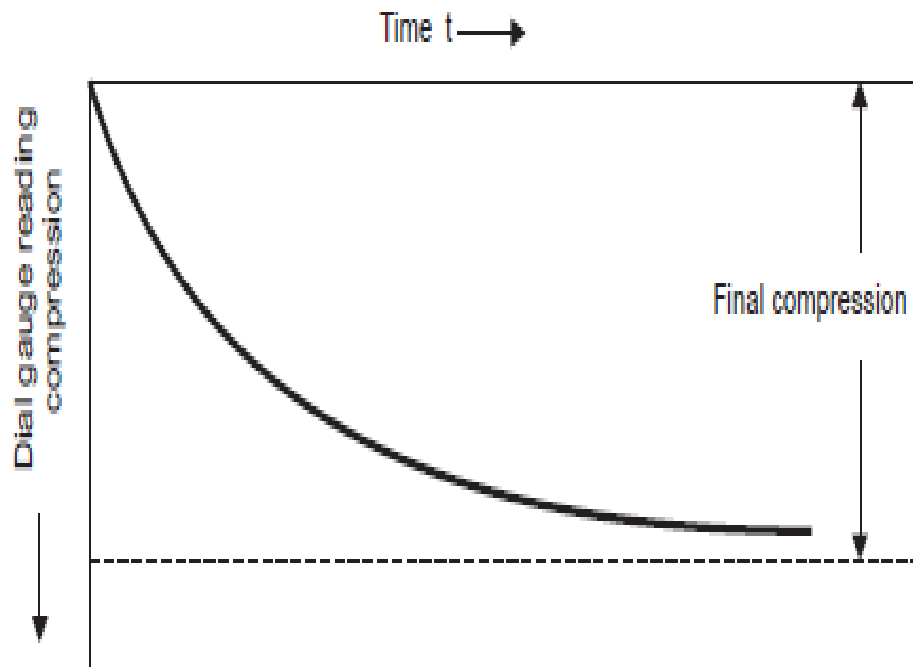
Oedometer Test



Oedometer Test



Oedometer Test Results



Normal Consolidated and Overconsolidated soil

- When soil is loaded to a stress level greater than it has ever experienced in the past, the soil structure is no longer able to sustain the increased load and starts to break down.
- **Preconsolidation Pressure** - σ'_p : Maximum pressure experienced by soil in the past
- **Over-consolidated soil:** $\sigma'_p > \sigma'_o$
 - Previously subjected to higher stress due to erosion, excavation, glacier, surcharge removal, etc.
 - Higher stiffness, higher shear strength and lower compressibility.
- **Normal Consolidated soil:** $\sigma'_p = \sigma'_o$
 - Never subjected to higher effective stress.
 - High compressibility and large settlement

Normal Consolidated and Overconsolidated soil

Preconsolidation Pressure (σ'_p)

- Maximum effective vertical stress experienced in geological history.
- Separates recompression and virgin compression zones.
- Determined from e - $\log \sigma'$ curve

Overconsolidation Ratio (OCR)

$$\text{OCR} = \sigma'_p / \sigma'_o$$

Interpretation:

- $\text{OCR} = 1$: Normally Consolidated
- $\text{OCR} > 1$: Overconsolidated
- $\text{OCR} < 1$: Indicates incorrect data for natural soils.

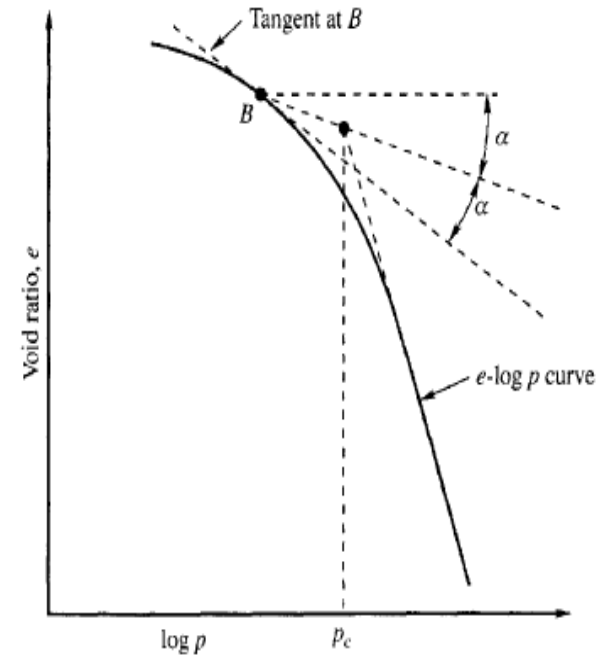
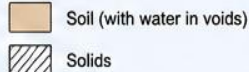
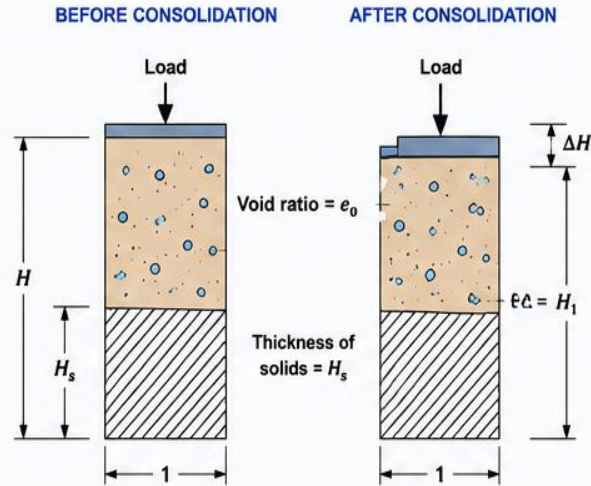


Figure 7.8 Method of determining p_c by Casagrande method

CONSOLIDATION SETTLEMENT CALCULATION AND EMPIRICAL COMPRESSION INDEX CORRELATIONS

1. CONSOLIDATION SETTLEMENT CONCEPT

Reduction in void ratio causes reduction in layer thickness, producing consolidation settlement.



2. DERIVATION OF SETTLEMENT EQUATION

$$\frac{\Delta H}{H} = \frac{\Delta V}{V}$$

For a unit area of clay stratum,

$$V = H, \quad V_1 = H_1$$

$$\begin{aligned} \Delta V &= (H - H_1) = H_s(1 + e_0) - H_s(1 + e_1) \\ &= H_s(e_0 - e_1) \end{aligned}$$

$$\frac{\Delta V}{V} = \frac{H_s(e_0 - e_1)}{H_s(1 + e_0)} = \frac{e_0 - e_1}{1 + e_0} = \frac{\Delta e}{1 + e_0}$$

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

(a) For Normally Consolidated Clay

$$S_c = \frac{C_c H}{1 + e_0} \log \frac{\sigma'_0 + \Delta \sigma}{\sigma'_0}$$

C_c = Compression index
 H = Thickness of compressible layer
 e_0 = Initial void ratio
 σ'_0 = Initial effective overburden pressure
 $\Delta \sigma$ = Increase in effective stress

(b) Alternative Equation

$$S_c = \frac{a_v H}{1 + e_0} \Delta \sigma = m_v H \Delta \sigma$$

a_v = Coefficient of volume compressibility
 m_v = Coefficient of volume compressibility
 $(m_v = a_v / (1 + e_0))$

3. EMPIRICAL CORRELATIONS FOR COMPRESSION INDEX C_c

Researcher	Correlation for C_c	Where
Skempton (1944)	$C_c = 0.007 (w_L - 10)$	w_L = Liquid limit (%)
Terzaghi & Peck (1948)	$C_c = 0.009 (w_L - 10)$	w_L = Liquid limit (%)
Azzouz et al. (1976)	$C_c = 0.37 (e_0 + 0.003 w_L + 0.0004 w_n - 0.34)$	e_0 = Initial void ratio w_L = Liquid limit (%) w_n = Natural water content (%)
Organic Soil (Azzouz et al.)	$C_c = 0.115 w_n$	w_n = Natural water content (%)
Hough (1957)	$C_c = 0.30 (e_0 - 0.27)$	e_0 = Initial void ratio

Note:

These equations provide approximate values of C_c and should be used for preliminary estimates. Laboratory consolidation test results are always preferred whenever available.

ENGINEERING NOTES



Settlement increases with:

- Higher Compression Index (C_c)
- Greater thickness of compressible layer (H)
- Higher increase in effective stress ($\Delta \sigma$)



Settlement decreases with:

- Higher initial void ratio (e_0)
- Overconsolidation ($OCR > 1$)
- Lower increase in effective stress



Use laboratory consolidation

test to obtain accurate values of C_c , e_0 , and preconsolidation pressure (σ'_p).

EXAMPLE

Given:

- Thickness of layer, $H = 5.0$ m
- Compression index, $C_c = 0.28$
- Initial void ratio, $e_0 = 0.85$
- Initial effective stress, $\sigma'_0 = 100$ kPa
- Increase in effective stress, $\Delta \sigma = 80$ kPa

Calculate settlement (S_c)

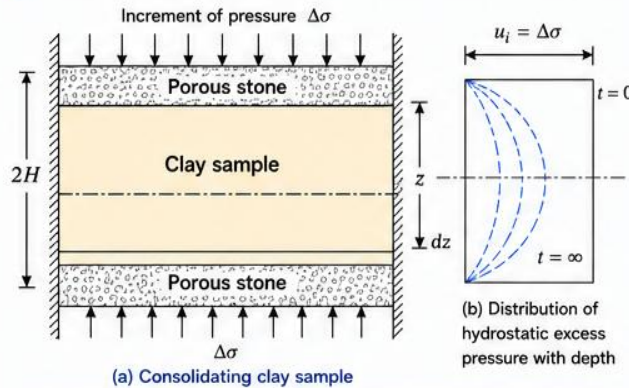
$$\begin{aligned} S_c &= \frac{C_c H}{1 + e_0} \log \frac{\sigma'_0 + \Delta \sigma}{\sigma'_0} \\ &= \frac{0.28 \times 5.0}{1 + 0.85} \log \frac{100 + 80}{100} \\ &= \frac{1.40}{1.85} \log (1.80) \\ &= 0.7568 \times 0.2553 \\ &= 0.193 \text{ m} = 193 \text{ mm} \end{aligned}$$

Therefore,
Settlement
 $S_c = 0.193 \text{ m}$
 $= 193 \text{ mm}$

ONE-DIMENSIONAL CONSOLIDATION THEORY OF TERZAGHI

1. ASSUMPTIONS OF TERZAGHI THEORY

- 1 The voids of the soil are completely filled with water.
- 2 Both water and solid constituents are incompressible.
- 3 Darcy's law is strictly valid.
- 4 The coefficient of permeability (k) is constant.
- 5 The time lag of consolidation is due entirely to the low permeability of the soil.
- 6 The clay is laterally confined.



Clay layer of thickness $2H$ is confined laterally and allowed to drain only at the top and bottom surfaces.

2. CONSOLIDATION EQUATION

Rearranging,

$$\frac{\partial u}{\partial t} = \frac{k}{\gamma_w m_v} \frac{\partial^2 u}{\partial z^2} = c_v \frac{\partial^2 u}{\partial z^2}$$

Where,

- u = Excess pore water pressure
 t = Time
 z = Depth
 k = Coefficient of permeability
 γ_w = Unit weight of water
 m_v = Coefficient of volume compressibility
 c_v = Coefficient of consolidation
 $\left(c_v = \frac{k}{\gamma_w m_v} \right)$

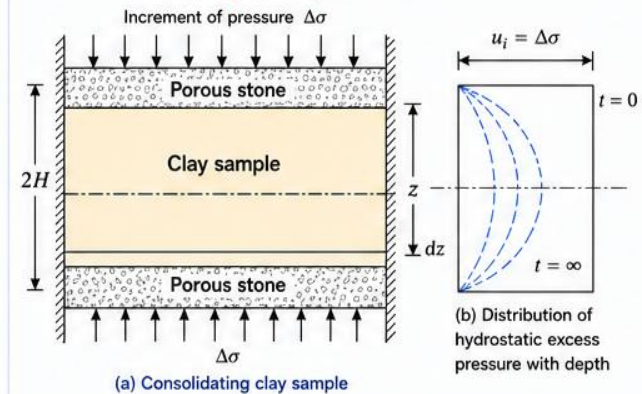
➤ **Boundary conditions** are the conditions of drainage at the surfaces of the consolidating layer.

➤ **Initial conditions** are the effective stress and pore water conditions at the beginning of loading.

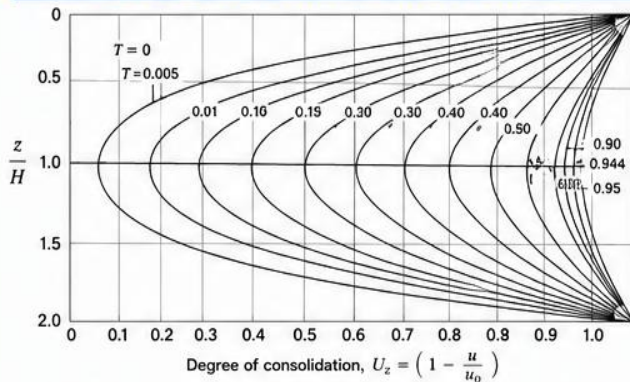
3. BOUNDARY CONDITIONS

The following are the boundary conditions:

- 1 There is drainage at the top of the sample:
At $z = 0$, $u = 0$, for all t .
- 2 There is drainage at the bottom of the sample:
At $z = 2H$, $u = 0$, for all t .
- 3 The initial hydrostatic excess pressure u_i is equal to the pressure increment, $\Delta\sigma$:
 $u = u_i = \Delta\sigma$, at $t = 0$.



4. SOLUTION OF CONSOLIDATION EQUATION



This chart gives the variation of degree of consolidation U_z with depth ratio z/H for different time factor T_v .

5. DIMENSIONLESS FACTORS

$$U_z = \frac{u_i - u}{u_i} = \left(1 - \frac{u}{u_i} \right)$$

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

Where,

- U_z = Dimensionless excess pore water pressure
 u_i = Initial excess pore water pressure = $\Delta\sigma$
 u = Excess pore water pressure at time t and depth z
 T = Time factor
 c_v = Coefficient of consolidation
 t = Time
 H = Drainage path ($= H$ for single drainage, $= H/2$ for double drainage)

6. TIME RATE OF CONSOLIDATION

$$U_t = \frac{S_t}{S}$$

Where, U_t = average degree of consolidation (%)

S_t = settlement of the layer at time t

S = ultimate settlement of the layer due to primary consolidation

Time factor, T_v

For $U_t = 0$ to 60%,

$$T_v = \frac{\pi}{4} \left(\frac{U_t \%}{100} \right)^2$$

For $U_t > 60\%$,

$$T_v = 1.781 - 0.933 \log(100 - U_t \%)$$

Also,

$$T_v = \frac{c_v t}{H_{dr}^2}$$

$$\text{or } t = \frac{T_v H_{dr}^2}{c_v}$$

H_{dr} = average longest drainage path during consolidation

ONE-DIMENSIONAL CONSOLIDATION THEORY OF TERZAGHI

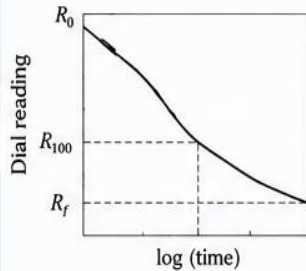
DETERMINATION OF COEFFICIENT OF CONSOLIDATION (c_v)

The coefficient of consolidation (c_v) is determined from consolidation test data using either Casagrande logarithm of time fitting method or Taylor square root of time fitting method.

(A) CASAGRANDE LOGARITHM OF TIME FITTING METHOD

1. Plot the data

Plot dial reading vs log(time) from the consolidation test data.

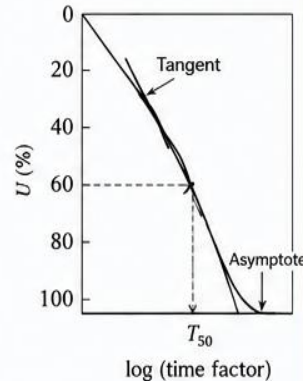


2. Determine T_{50}

Locate 50% consolidation ($R_{50} = \frac{R_0 + R_f}{2}$).

Draw tangent at the point of 50% consolidation and extend to intersect the baseline.

Read the time factor T_{50} corresponding to this intersection.



3. Calculate c_v

For $U = 50\%$, $T_{50} = 0.197$
(From Terzaghi's theory)
For double drainage:

$$c_v = \frac{T_{50} H_{dr}^2}{t_{50}} = \frac{0.197 H_{dr}^2}{t_{50}}$$

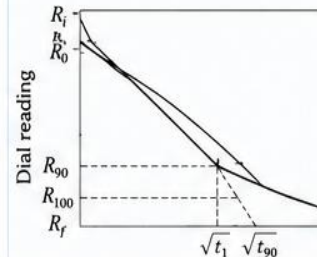
where,

H_{dr} = drainage path ($= H/2$ for double drainage)
 t_{50} = time corresponding to 50% consolidation
(from the graph)

(B) TAYLOR SQUARE ROOT OF TIME FITTING METHOD

1. Plot the data

Plot dial reading vs square root of time ($\sqrt{t}$) from the consolidation test data.



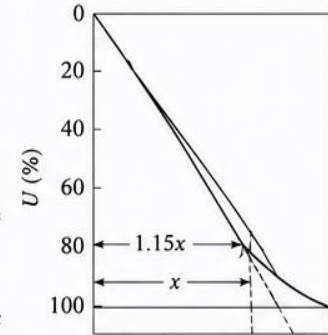
2. Determine T_{90}

Draw a straight line to represent the initial portion of the curve.

From the curve, locate 90% consolidation (R_{90}).

From this point, draw a horizontal line to cut the straight line.

Measure the distance x between the two lines.



3. Calculate c_v

For $U = 90\%$, $T_{90} = 0.848$
(From Terzaghi's theory)
For double drainage:

$$c_v = \frac{T_{90} H_{dr}^2}{t_{90}} = \frac{0.848 H_{dr}^2}{t_{90}}$$

where,

H_{dr} = average drainage path ($= H/2$ for double drainage)
 t_{90} = time corresponding to 90% consolidation (from the graph)

METHODOLOGY SUMMARY

1

Perform consolidation test and record dial readings at different times.

2

Plot the data using either method:
(a) Dial reading vs log(time) or
(b) Dial reading vs $\sqrt{t}$

3

From the graph, determine the time factor T (T_{50} or T_{90}) using the appropriate construction.

4

Read the corresponding time t for the determined consolidation level.

5

Calculate c_v using
$$c_v = \frac{T H_{dr}^2}{t}$$

(Use $H/2$ for double drainage)

KEY POINTS



Casagrande method uses 50% consolidation point and log(time) plot.



Taylor method uses 90% consolidation point and $\sqrt{t}$ plot.



Time factors are theoretical constants: $T_{50} = 0.197$, $T_{90} = 0.848$.



c_v depends on drainage path, hence use $H/2$ for double drainage.



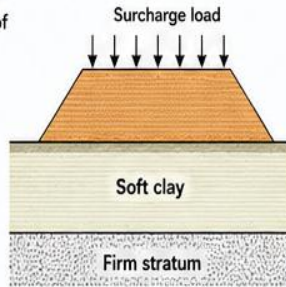
Accurate plotting and correct time factor evaluation are essential for reliable determination of c_v .

METHODS TO ACCELERATE CONSOLIDATION IN THE FIELD

Consolidation is a time-dependent process. In the field, the rate of consolidation can be increased by reducing the drainage path, increasing the permeability or applying surcharge to increase the driving force.

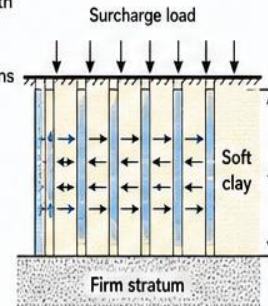
1. PRELOADING (SURCHARGING)

- Additional load greater than the final design load is applied in the form of embankment.
- Increases the effective stress and driving force, thus speeding up consolidation.



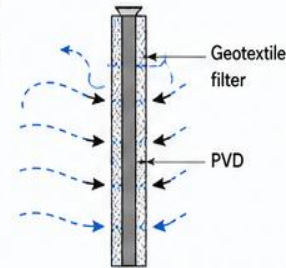
2. VERTICAL DRAINS (SAND DRAINS / PVDs)

- Vertical drains reduce the drainage path from $H/2$ to a smaller value.
- Pore water escapes laterally to the drains and then vertically.
- Greatly reduces the time required for consolidation.



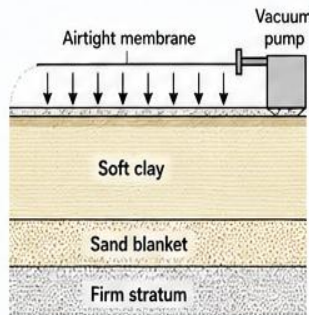
3. PREFABRICATED VERTICAL DRAINS (PVDs)

- Manufactured drains (wick drains) wrapped in a filter are installed in the ground.
- Provide high discharge capacity and long-term performance.
- Installed by mandrel or static/ vibratory methods.



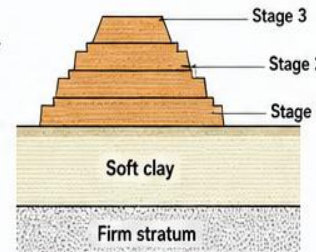
4. VACUUM CONSOLIDATION

- Vacuum pressure is applied at the ground surface through an airtight membrane.
- Reduces pore water pressure throughout the soil mass.
- Effective stress increases, thus accelerating consolidation.



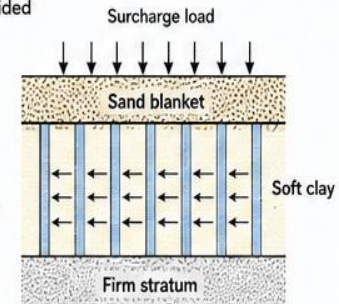
5. STAGED CONSTRUCTION

- The structure is constructed in stages with time gaps.
- Allows consolidation to occur between stages.
- Reduces total and differential settlement.



6. DRAINAGE BLANKET / SAND BLANKET

- A permeable sand layer is provided on the top.
- Increases the rate of lateral drainage of pore water to vertical drains or to the edges.
- Also acts as a working platform for construction.



SUMMARY OF BENEFITS



Reduces time required for consolidation significantly.



Reduces total and differential settlement of structures.



Improves stability during and after construction.



Allows early construction of overlying structures or reduces project time.



Enhances overall performance and serviceability.



Note: Selection of method depends on site conditions, soil type, required rate of consolidation, economy and construction feasibility.

IMMEDIATE (ELASTIC / DISTORTION) SETTLEMENT

Immediate settlement (S_i) is the initial elastic compression of soil that occurs immediately after the application of load, before primary consolidation takes place.

1. TOTAL SETTLEMENT

$$\text{Total Settlement : } S_t = S_i + S_c + S_s$$

S_i : Immediate (distortion) settlement (initial compression)

S_c : Primary consolidation settlement

S_s : Secondary compression settlement



S_i is estimated using elastic theory assuming the soil behaves like an elastic medium under short-term loading.

2. FORMULAE FOR IMMEDIATE SETTLEMENT

◆ For cohesive soils (flexible foundation)

$$S_i = q B \cdot \frac{(1 - \nu^2)}{E_s} \cdot I_s$$

q = Net load intensity on foundation (kN/m²)

B = Width of foundation (m)

ν = Poisson's ratio of soil

E_s = Elastic modulus of soil (kN/m²)

I_s = Influence factor (depends on L/B and shape)

◆ For rigid foundation

$$S_i = q B \cdot \frac{(1 - \nu^2)}{E_s} \cdot I_s$$

3. INFLUENCE FACTOR (I_s) TABLES

(a) Flexible Foundation (Strip footing)

L/B	1	2	3	4	5
I_s	0.56	0.76	0.88	0.96	1.00

(b) Rigid Foundation

L/B	Circle	1	2	5	10
Influence factor (I_s)	0.73	0.82	1.00	1.22	1.26



For perfectly flexible square footing, settlement at centre = **2 × settlement at corner**.

4. RANGE OF MATERIAL PARAMETERS

Typical range of elastic parameters for different soils

Type of Soil	E_s (MN/m ²)	Poisson's ratio (ν)
Loose sand	10.5 – 24	0.2 – 0.4
Medium dense sand	17.25 – 27.6	0.25 – 0.4
Dense sand	34.5 – 55.2	0.3 – 0.45
Silty sand	10.35 – 17.25	0.2 – 0.4
Sand and gravel	69 – 172.5	0.15 – 0.35
Soft clay	4.1 – 20.7	0.2 – 0.5
Stiff clay	41.4 – 96.6	0.2 – 0.5

5. APPLICATION



An earthen embankment may be taken as a flexible foundation and the above formula may be used to determine S_i below such construction.

6. KEY POINTS

- Immediate settlement occurs instantly with load application.
- Depends on q , B , E_s , ν , and influence factor I_s .
- I_s accounts for foundation shape and L/B ratio.
- Accurate estimation requires reliable E_s and ν values.

7. METHODOLOGY (SUMMARY)

- 1 Determine net load intensity, q , and foundation width, B .
- 2 Identify foundation type (flexible or rigid) and L/B ratio.
- 3 Obtain influence factor I_s from the appropriate table.
- 4 Determine E_s and ν from site data or typical ranges.
- 5 Compute immediate settlement:

$$S_i = q B \cdot \frac{(1 - \nu^2)}{E_s} \cdot I_s$$



Objective: To estimate the initial elastic compression of soil for safe and serviceable foundation design.

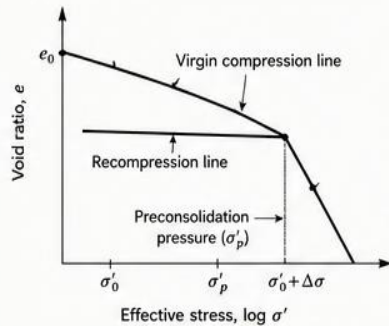
PRIMARY CONSOLIDATION SETTLEMENT (S_c)

Primary consolidation settlement (S_c) is the time-dependent settlement that occurs due to expulsion of pore water from the soil voids under sustained loading.

1. APPLICABLE TO

- Saturated cohesive soils (clays and silts).
- Settlement occurs as pore water pressure dissipates.

3. ILLUSTRATION ($e - \log \sigma'$ CURVE)



4. KEY POINTS

- Represents the major portion of total settlement in clayey soils.
- Time required depends on drainage path and coefficient of consolidation (c_v).
- Obtained from oedometer test using $e - \log \sigma'$ curve parameters.

5. TYPICAL RANGE OF PARAMETERS

Soil Type	C_c	C_r	e_0
Soft clay	0.2 – 0.6	0.02 – 0.1	0.8 – 1.5
Medium clay	0.15 – 0.4	0.015 – 0.06	0.6 – 1.2
Stiff clay	0.05 – 0.2	0.005 – 0.03	0.4 – 0.8

2. FORMULA

◆ For normally consolidated soil

$$S_c = \frac{C_c H}{1 + e_0} \log_{10} \left(\frac{\sigma'_0 + \Delta\sigma}{\sigma'_0} \right)$$

◆ For over-consolidated soil

If final effective stress ($\sigma'_0 + \Delta\sigma$) is less than preconsolidation stress (σ'_p):

$$S_c = \frac{C_r H}{1 + e_0} \log_{10} \left(\frac{\sigma'_0 + \Delta\sigma}{\sigma'_0} \right)$$

If ($\sigma'_0 + \Delta\sigma$) is greater than σ'_p :

$$S_c = \frac{C_r H}{1 + e_0} \log_{10} \left(\frac{\sigma'_p}{\sigma'_0} \right) + \frac{C_c H}{1 + e_0} \log_{10} \left(\frac{\sigma'_0 + \Delta\sigma}{\sigma'_p} \right)$$

where,

- C_c = Compression index
- C_r = Recompression index
- e_0 = Initial void ratio
- H = Thickness of compressible layer (m)
- σ'_0 = Initial effective overburden stress (kN/m^2)
- $\Delta\sigma$ = Increase in vertical stress due to load (kN/m^2)
- σ'_p = Preconsolidation pressure (kN/m^2)

SECONDARY CONSOLIDATION SETTLEMENT (S_s)

Secondary consolidation settlement (S_s) is the time-dependent settlement that occurs after primary consolidation is complete, due to continued adjustment (creep) of soil particles.

1. APPLICABLE TO

- All fine-grained soils (clays and silts) after primary consolidation is complete.
- Caused by plastic adjustment of soil structure.

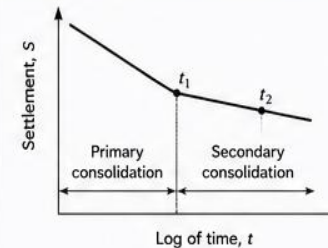
2. FORMULA (LOG TIME METHOD)

$$S_s = C_a H \log_{10} \left(\frac{t_2}{t_1} \right)$$

where,

- C_a = Secondary compression index
- H = Thickness of compressible layer (m)
- t_1 = Time at end of primary consolidation (years)
- t_2 = Time at which settlement is required (years)

3. SETTLEMENT VS LOG TIME CURVE



4. DETERMINATION OF C_a

- From the linear portion of secondary consolidation curve (S vs $\log t$).
- C_a = Slope of the straight line

$$= \frac{\Delta S}{H \Delta(\log t)}$$

5. TYPICAL RANGE OF C_a

Soil Type	C_a
Soft clay	0.005 – 0.02
Medium clay	0.003 – 0.01
Stiff clay	0.001 – 0.005

6. KEY POINTS

- S_s increases logarithmically with time.
- Though smaller than S_c , it may become significant over long periods.
- Important in long-term settlement evaluation of structures.

7. TOTAL SETTLEMENT

Total Settlement,

$$S_t = S_i + S_c + S_s$$

- S_i = Immediate (elastic) settlement
- S_c = Primary consolidation settlement
- S_s = Secondary consolidation settlement



Note: S_c can be determined from oedometer (consolidation) test results.



Note: Secondary consolidation depends on soil type, stress level, and duration of loading.

Thank You

Shear Strength of Soils



01 1X15 SOIL MECHANICS & ROCK MECHANICS

L-T-P : 3-0-3

Credit : 5

Theory :

1. Shear Strength of Soil : Engineering use of shear strength, Direct and triaxial shear tests, Mohr-Coulomb strength criterion, drained, consolidated undrained and undrained tests, strength of loose and dense sands, NC and OC soils, dilation, pore pressure and Skempton's pore pressure coefficients. **Lecture : 7**

2. Earth pressure theories & Retaining Walls : Limit equilibrium method, effect of wall movement on earth pressure, pressure at rest, Rankine state of plastic equilibrium, Coulomb's theory, Rebhann and Culmann's graphical methods. Sheet piles – Types and uses of sheet piles, Analysis of Cantilever and anchored sheet piles in cohesionless and cohesive soil, Rowe's theory of moment reduction. **Lecture: 5**

3. Stability of slopes : Limit equilibrium methods, methods of slices, simplified Bishop's method and friction circle method, factors of safety, stability under conditions of submergence, drawdown and steady seepage, location of critical arc, stability number, chart. **Lecture : 9**

4. Engineering properties of rocks : engineering classification of rocks, Surface and sub-surface investigation in rock including geophysical studies. **Lecture : 7**

5. Weathering of rocks : discontinuities, field and laboratory testing of rocks and rock masses, Stress- strain characteristics, Deformability of rocks, Friction and Shear strength, Slope stability, effect of water. **Lecture : 8**

6. Introduction to analysis and design of tunnels, blasting, bolting, tunneling techniques, application. **Lecture : 6**

Text Books :

1. A Text Book of Soil Mechanics and Foundation Engineering, Revised and enlarged 4th Edition 1993 by V.N.S. Murthy, Saikripa Technical Consultants, Bangalore.
2. Basic and Applied Soil Mechanics by Gopal Ranjan and A.S.R.Rao, Wiley Eastern Ltd., New Delhi.
3. Soil Mechanics and Foundation Engineering by K.R.Arora, Standard Pub. and Dist., Delhi., 1992.
4. Introduction to Rock Mechanics by R.E. Goodman, John Wiley and Sons, New York.
5. Rock Mechanics and Engineering by Jacger, Charles, Cambridge University Press, London.

Reference Books :

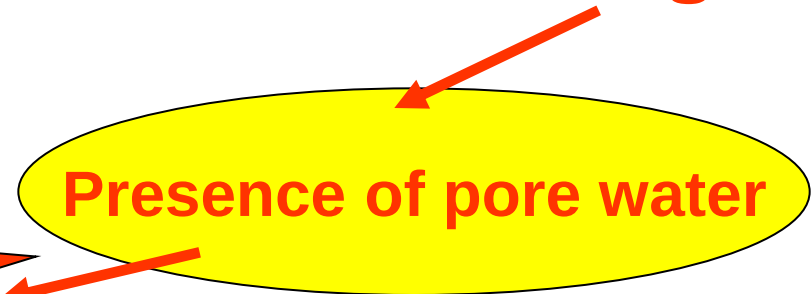
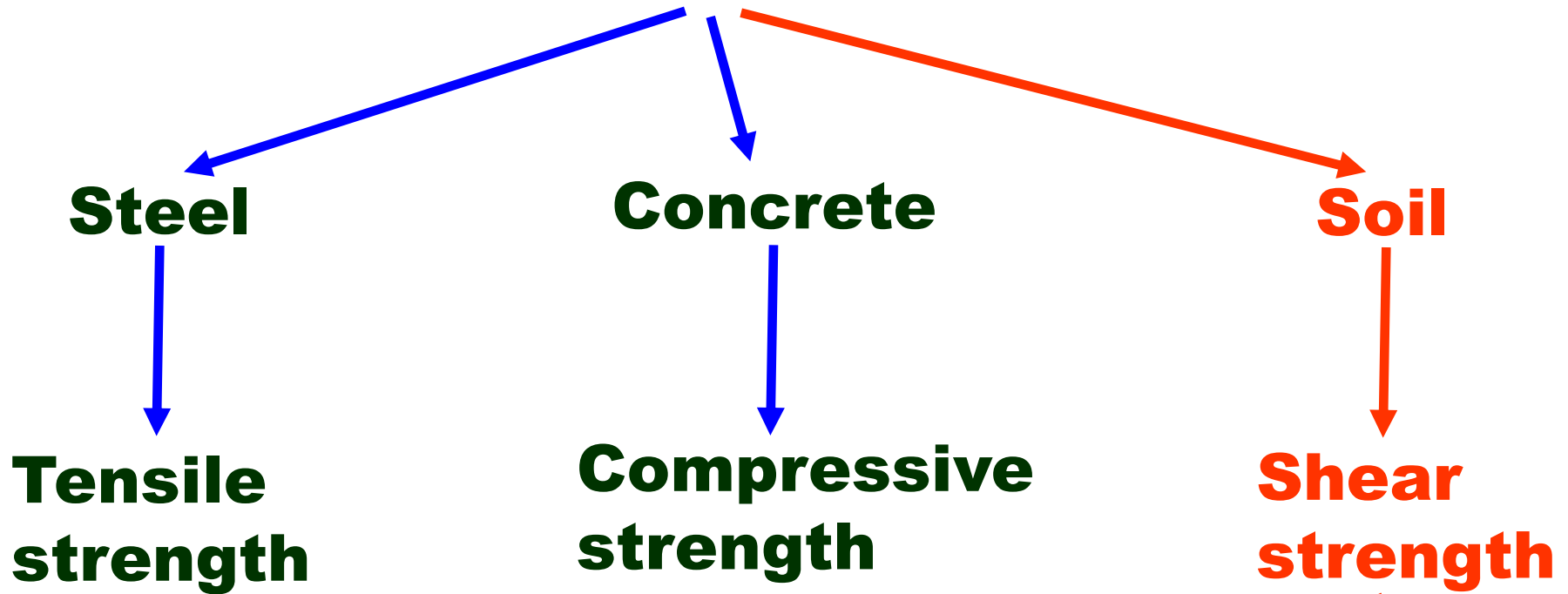
- (1) Soil Mechanics in Engineering Practice by Terzaghi and Peck, John Wiley and Sons Inc. New York, 1967
- (2) Soil Mechanics by Lamb and Whitman, Wiley Eastern Pvt. Ltd., New Delhi, 1969.
- (3) Fundamentals of Soil Mechanics by Taylor, John Wiley and Sons Inc New York, 1948.
- (4) Tunnels : Planning, Design, Construction by Megaw, T.M. and J.V. Bartlett. International Edition, Ellis Horwood Limited, John Wiley and Sons, New York.

Practical : Direct Shear Test, Vane Shear Test, Unconfined Shear Test, Triaxial Shear Test, Brazilian test for tensile strength of rocks, Bending Test for tensile strength of rocks, Uniaxial Compressive Strength Test.

Course outcomes

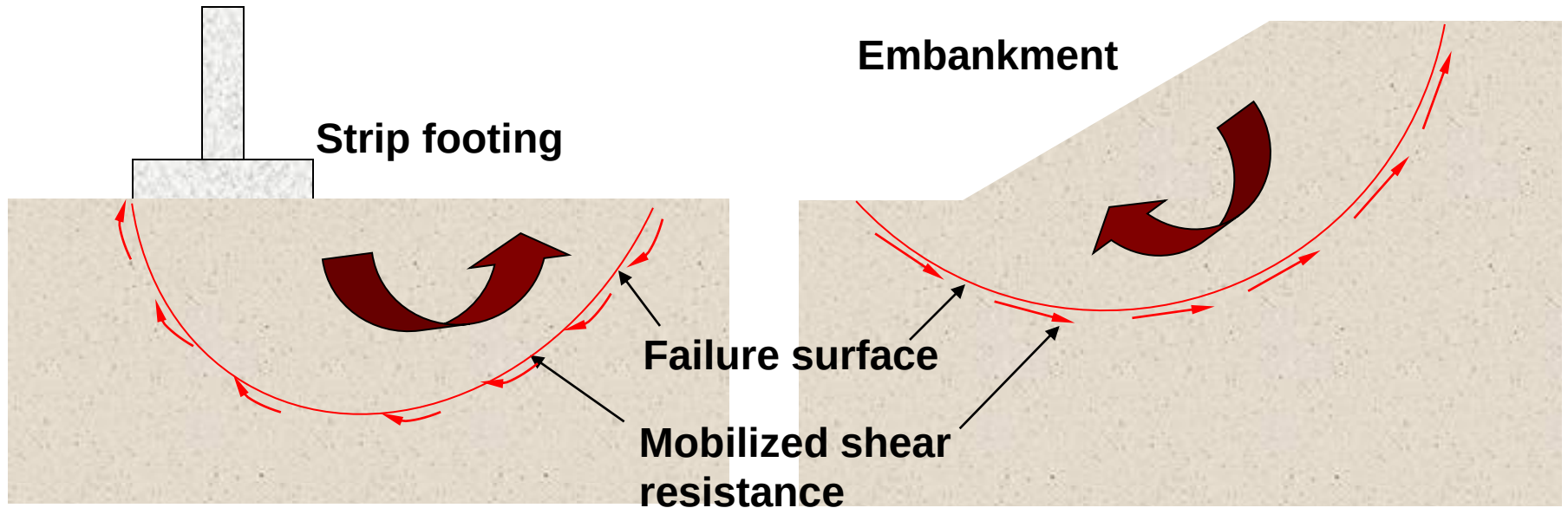
Course outcome	
CO1	Understanding of the shear strength behavior of soil and different laboratory methods for determination of the strength of soil
CO2	Application of the principles and basic of soil mechanics in the analysis of slope stability and retaining wall
CO3	Understanding and evaluation of the basic and engineering behavior of the rock
CO4	Application of the principles and basic of rock mechanics in the analysis of slope stability and tunnelling

Strength of different materials



Shear failure of soils

Soils generally fail in shear



At failure, shear stress along the failure surface (mobilized shear resistance) reaches the shear strength.

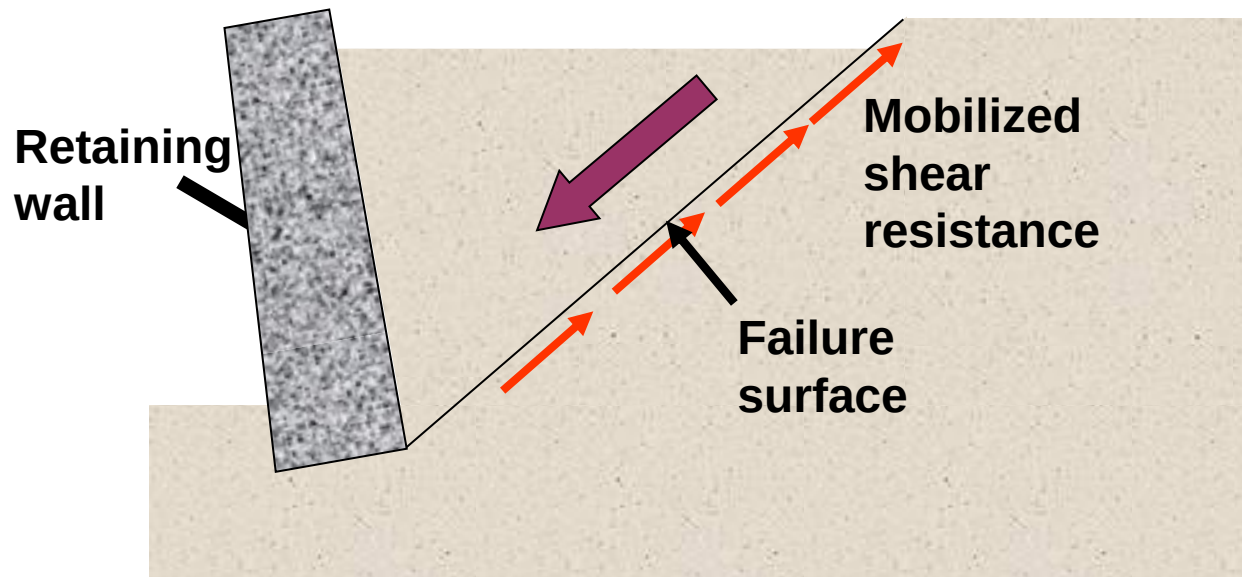
Shear failure of soils

Soils generally fail in shear



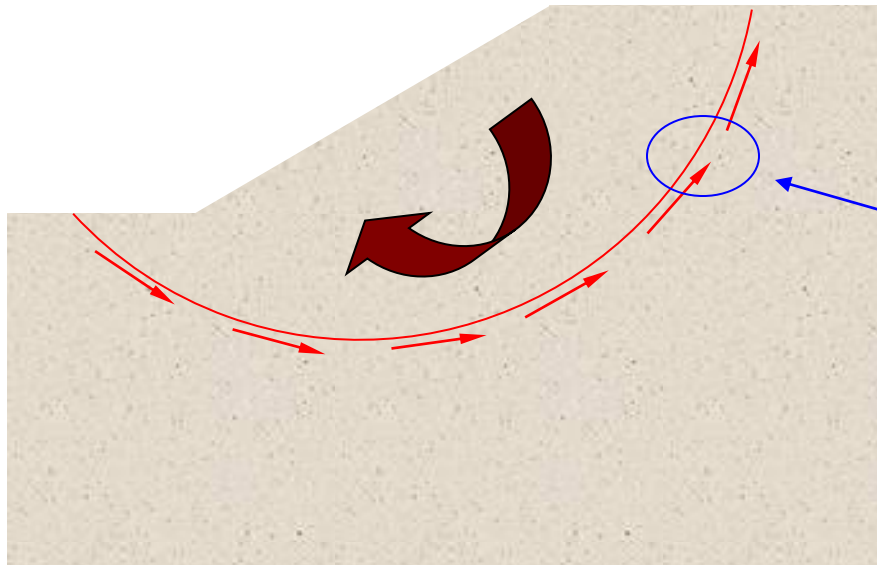
Shear failure of soils

Soils generally fail in shear



At failure, shear stress along the failure surface (mobilized shear resistance) reaches the shear strength.

Shear failure mechanism



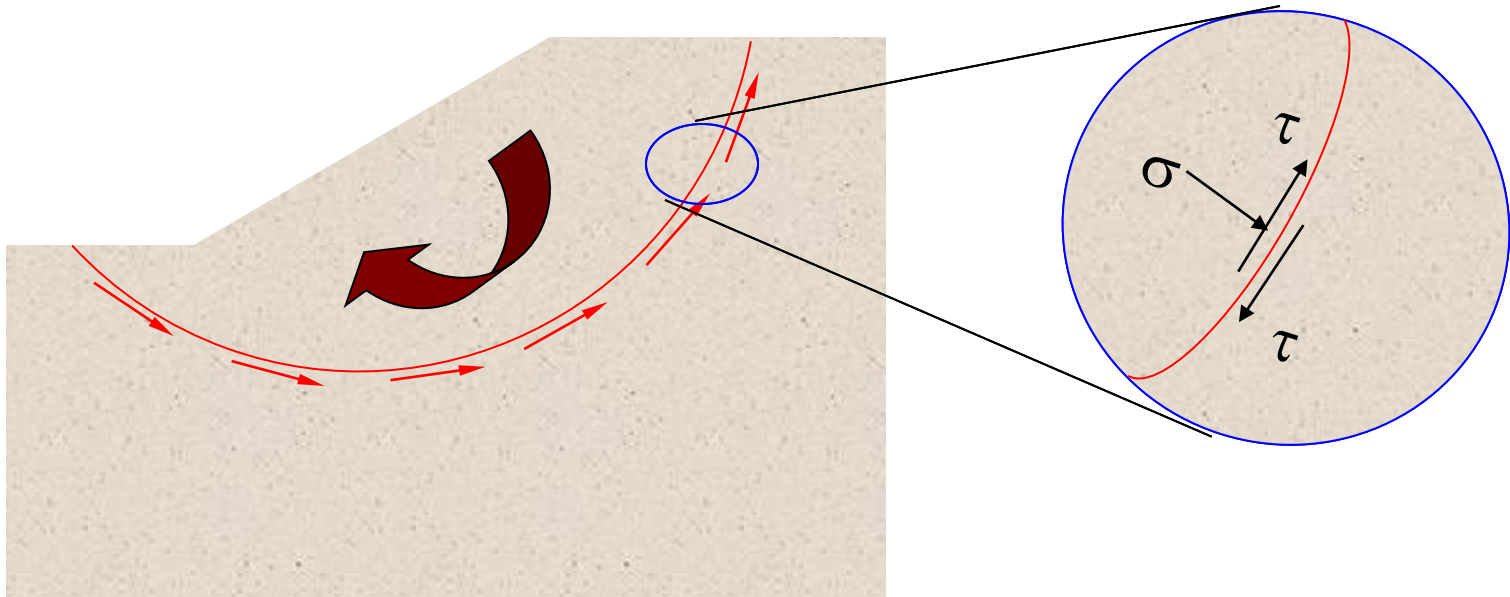
failure surface

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

No crushing of individual grains.



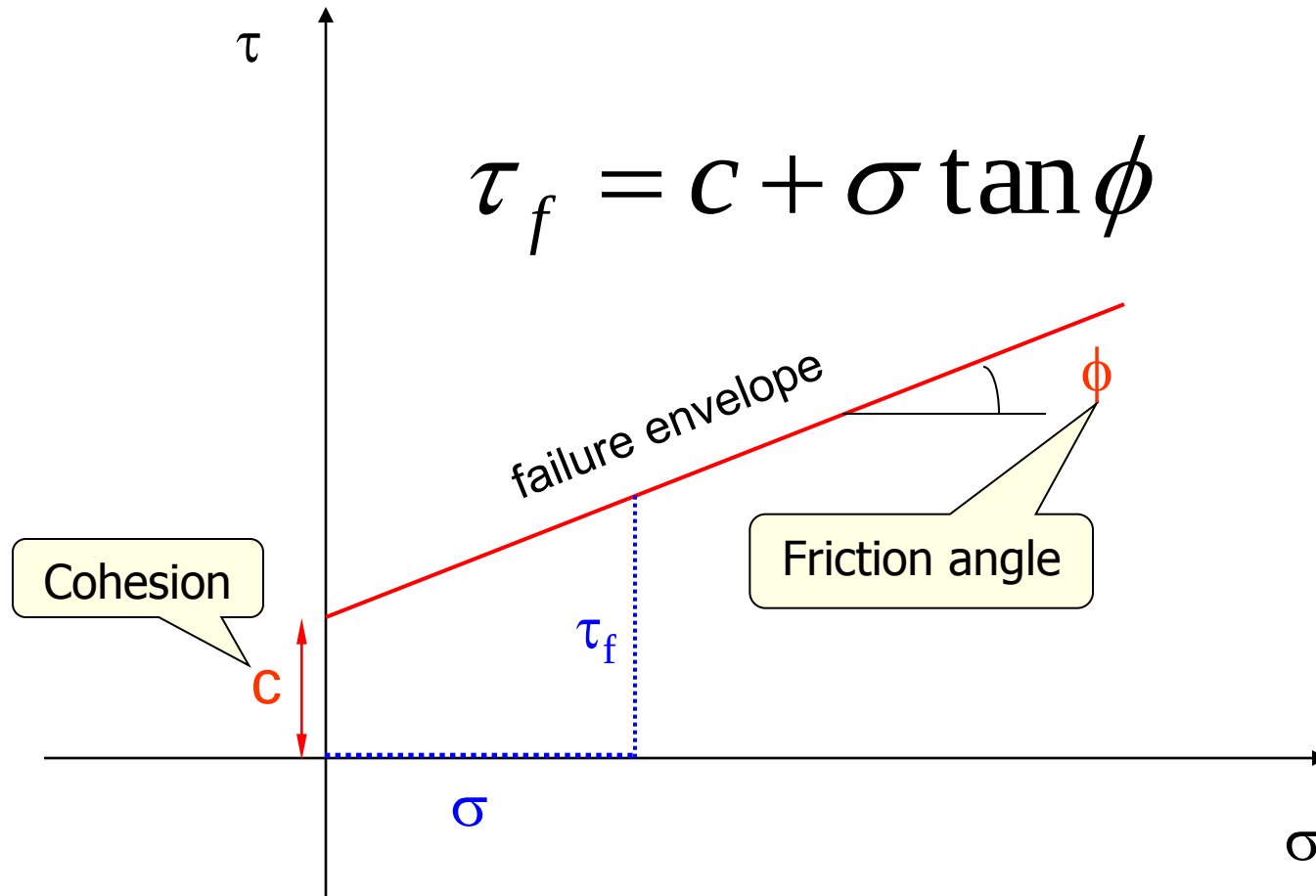
Shear failure mechanism



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

Mohr-Coulomb Failure Criterion

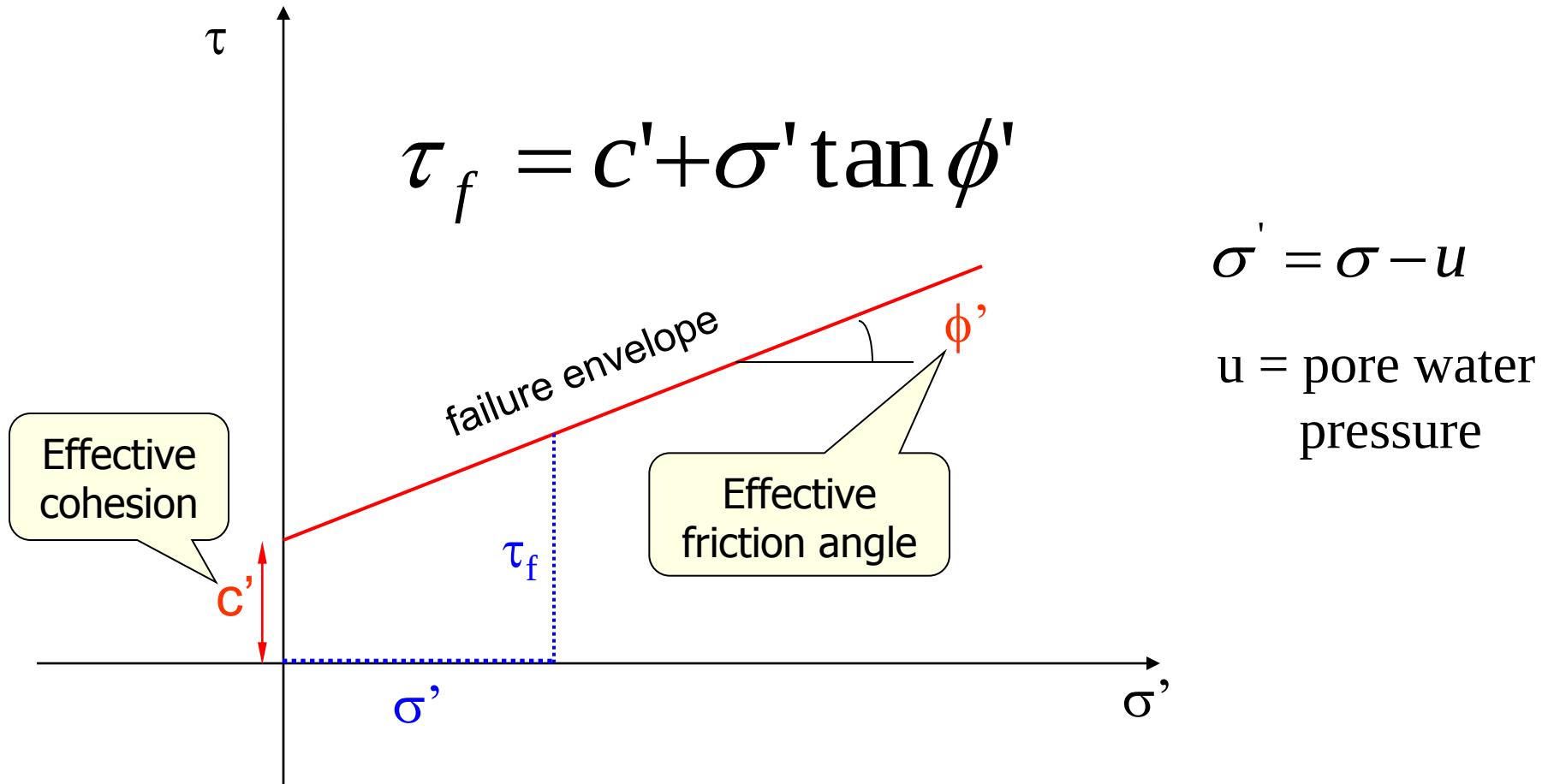
(in terms of **total stresses**)



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

Mohr-Coulomb Failure Criterion

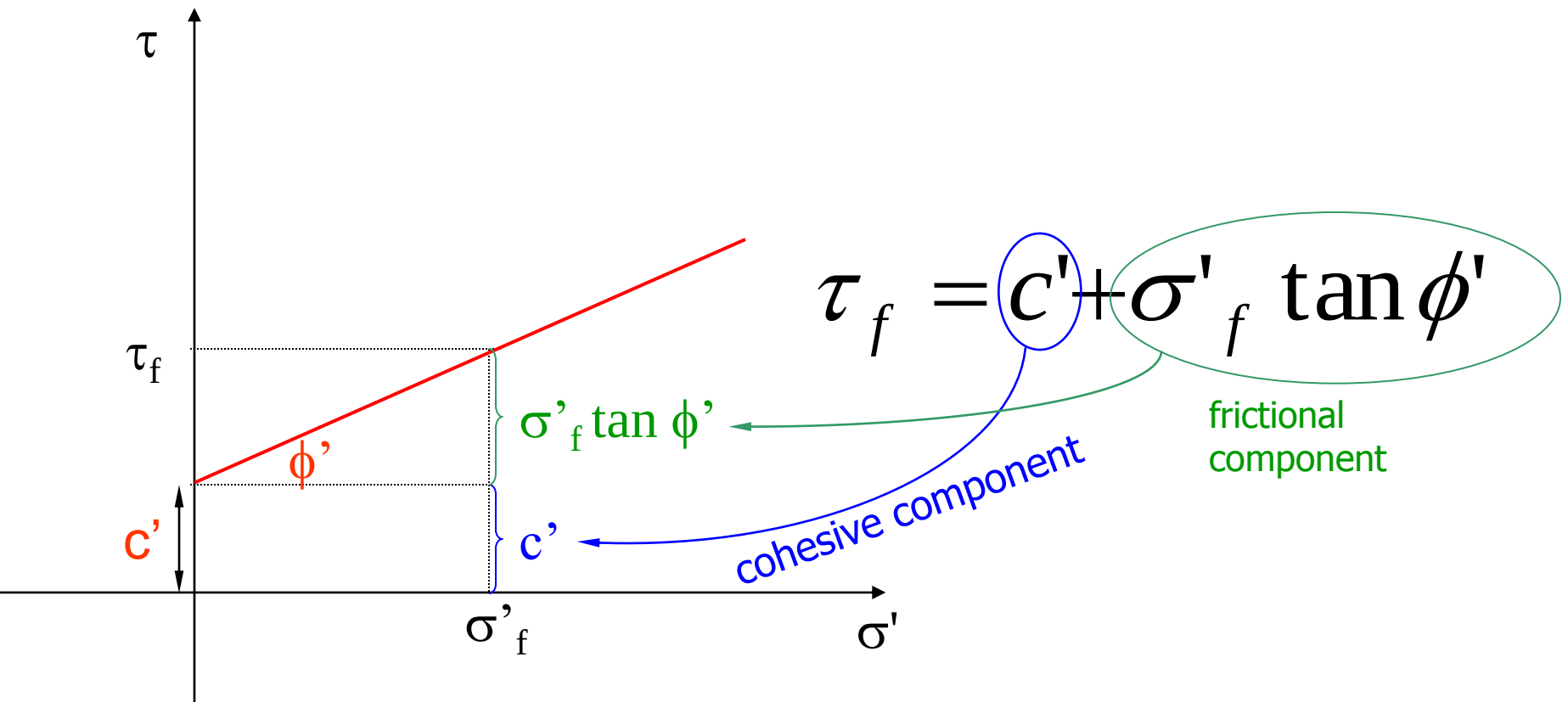
(in terms of **effective stresses**)



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

Mohr-Coulomb Failure Criterion

Shear strength consists of two components: **cohesive** and **frictional**.

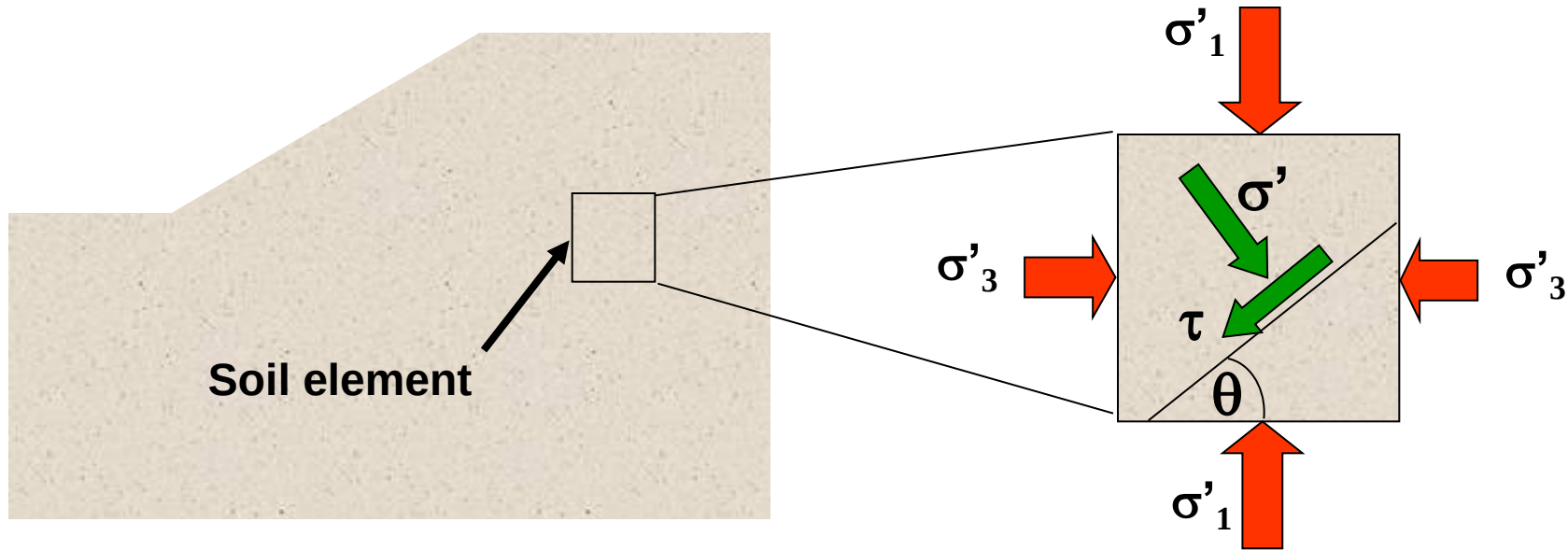




c and ϕ are measures of shear strength.

Higher the values, higher the shear strength.

Mohr Circle of stress



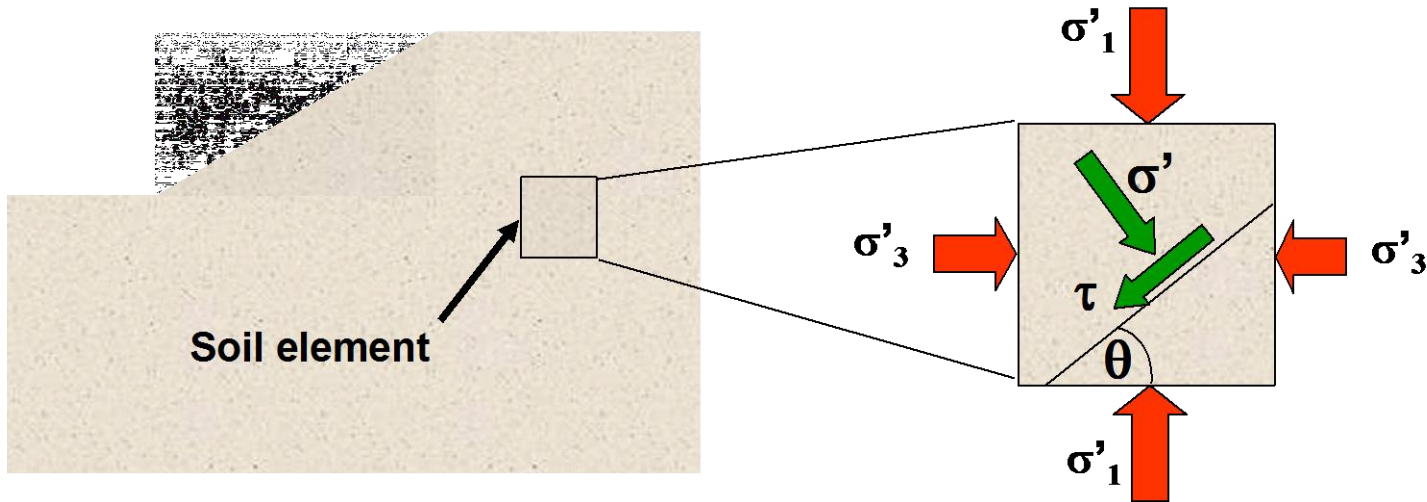
Resolving forces in σ and τ directions,

$$\tau = \frac{\sigma'_1 - \sigma'_3}{2} \sin 2\theta$$

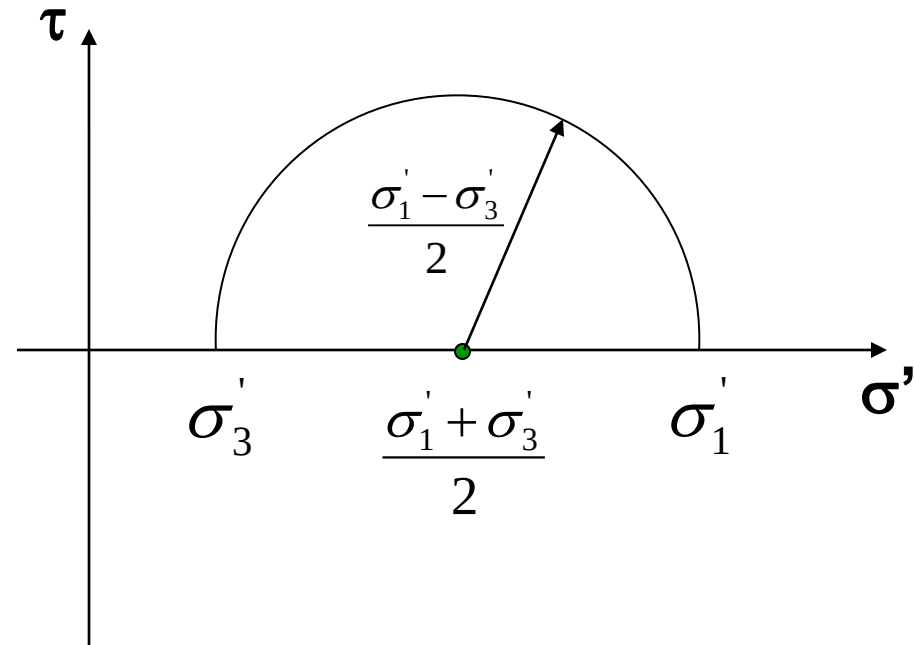
$$\sigma' = \frac{\sigma'_1 + \sigma'_3}{2} + \frac{\sigma'_1 - \sigma'_3}{2} \cos 2\theta$$

$$\tau^2 + \left(\sigma' - \frac{\sigma'_1 + \sigma'_3}{2} \right)^2 = \left(\frac{\sigma'_1 - \sigma'_3}{2} \right)^2$$

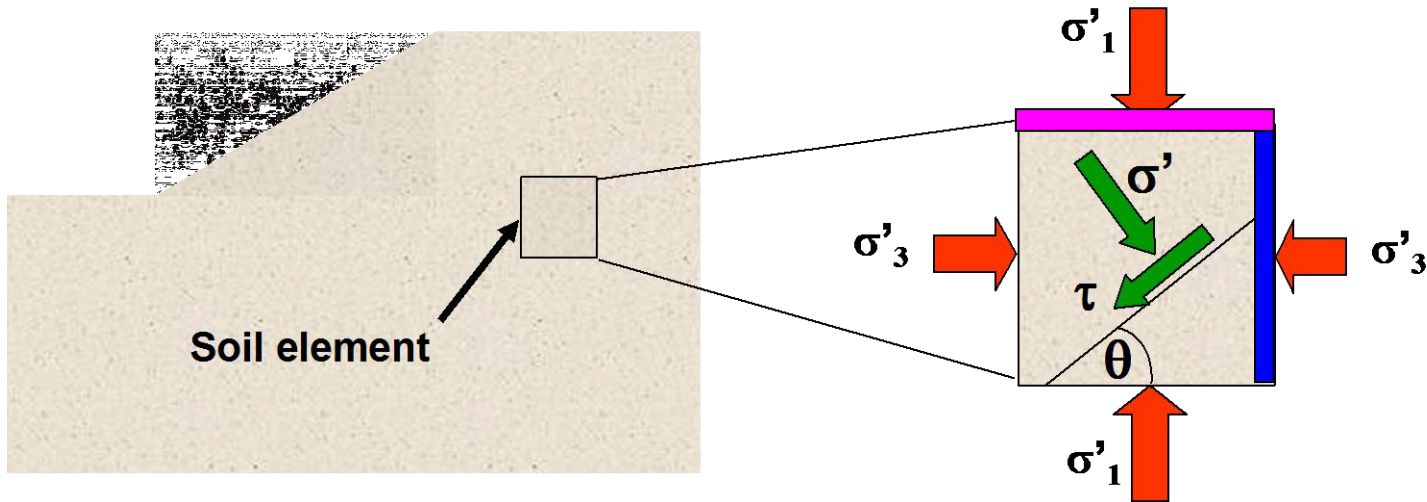
Mohr Circle of stress



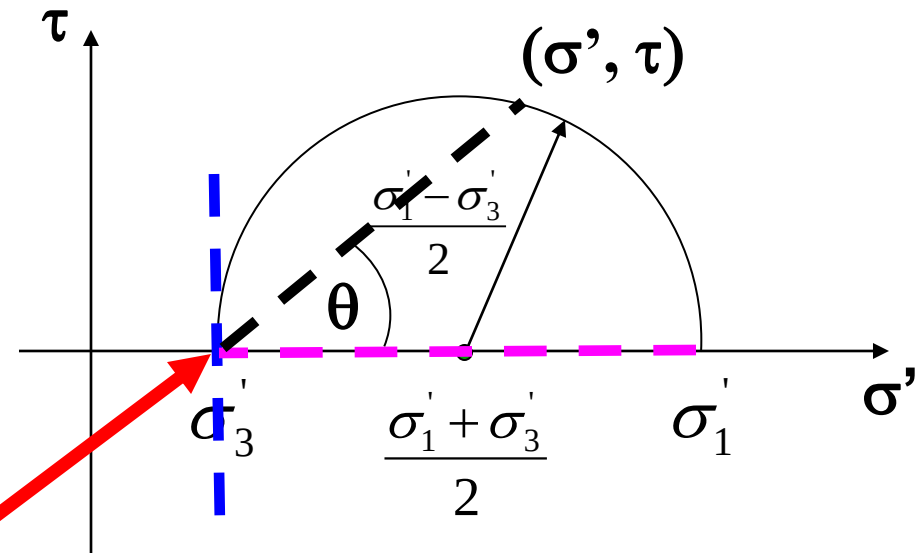
$$\tau^2 + \left(\sigma' - \frac{\sigma'_1 + \sigma'_3}{2} \right)^2 = \left(\frac{\sigma'_1 - \sigma'_3}{2} \right)^2$$



Mohr Circle of stress

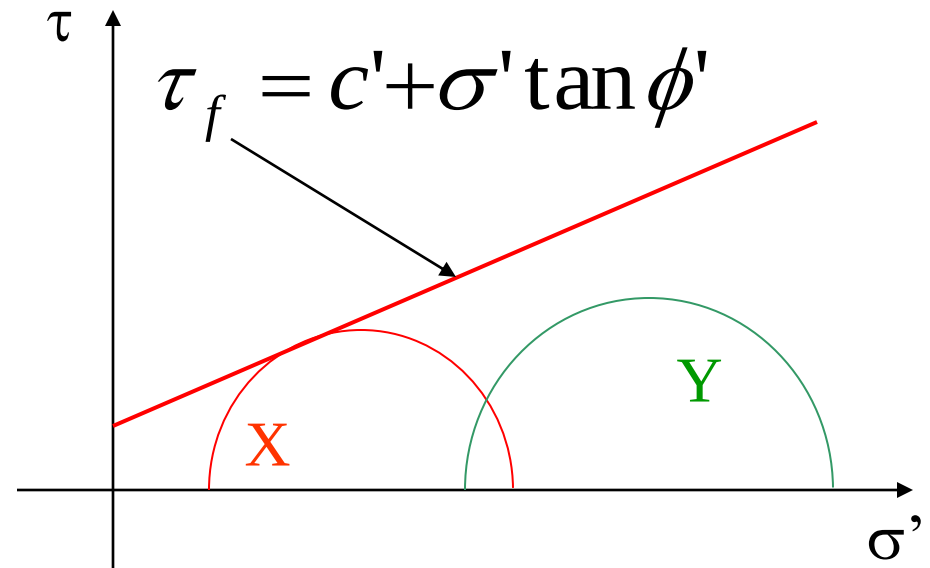
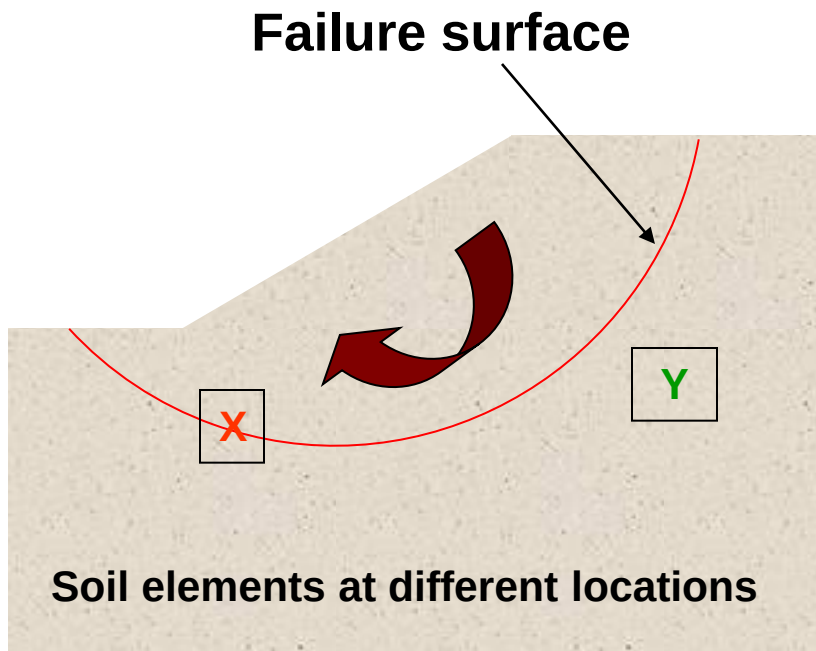


$$\tau^2 + \left(\sigma' - \frac{\sigma'_1 + \sigma'_3}{2} \right)^2 = \left(\frac{\sigma'_1 - \sigma'_3}{2} \right)^2$$



$P_D = \text{Pole w.r.t. plane}$

Mohr Circles & Failure Envelope

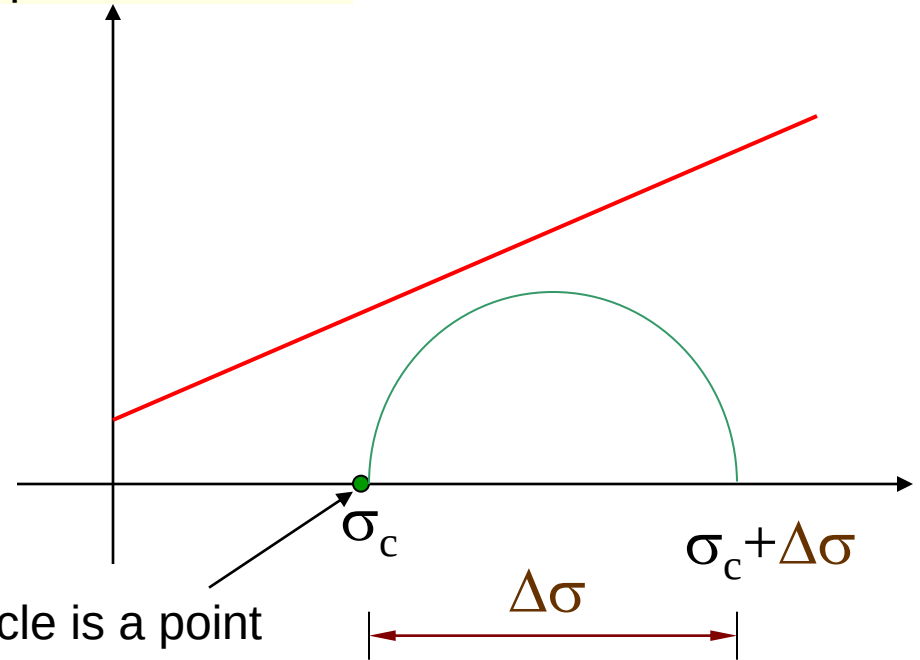
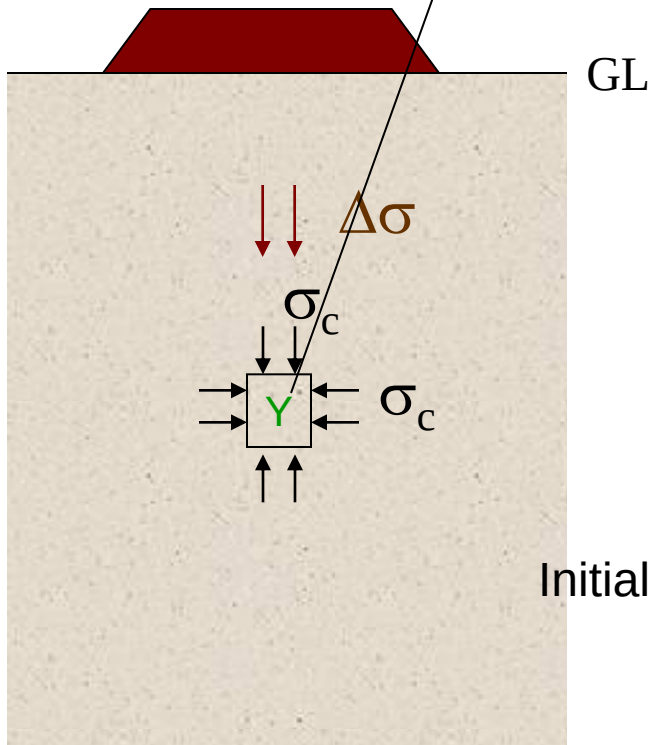


Y $\sim$ stable

X $\sim$ failure

Mohr Circles & Failure Envelope

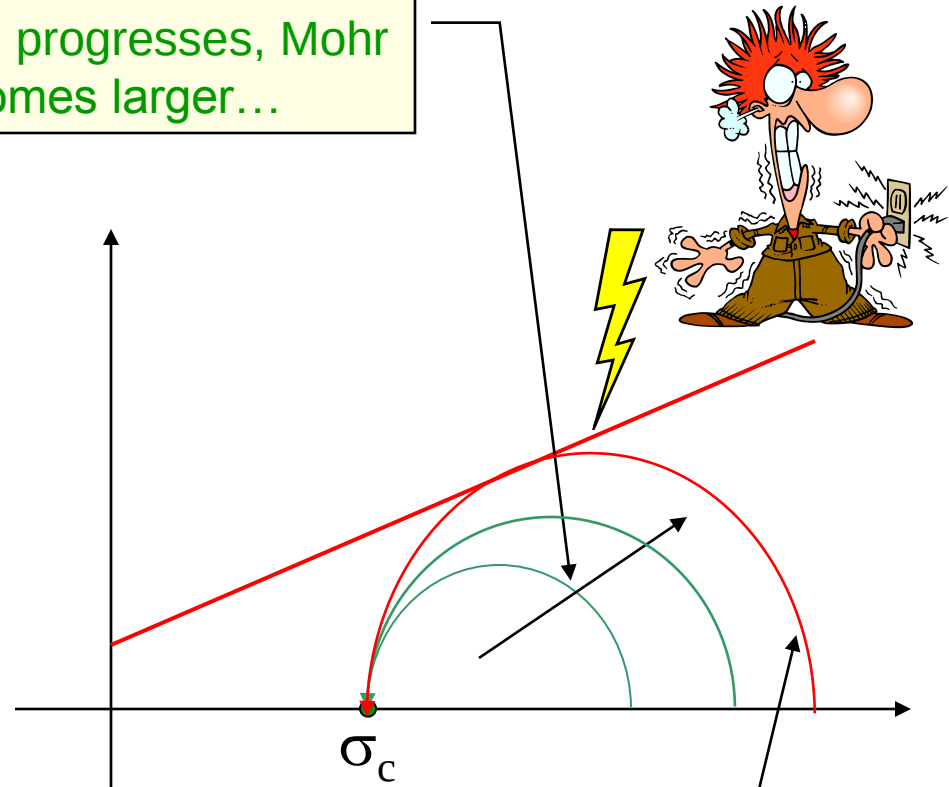
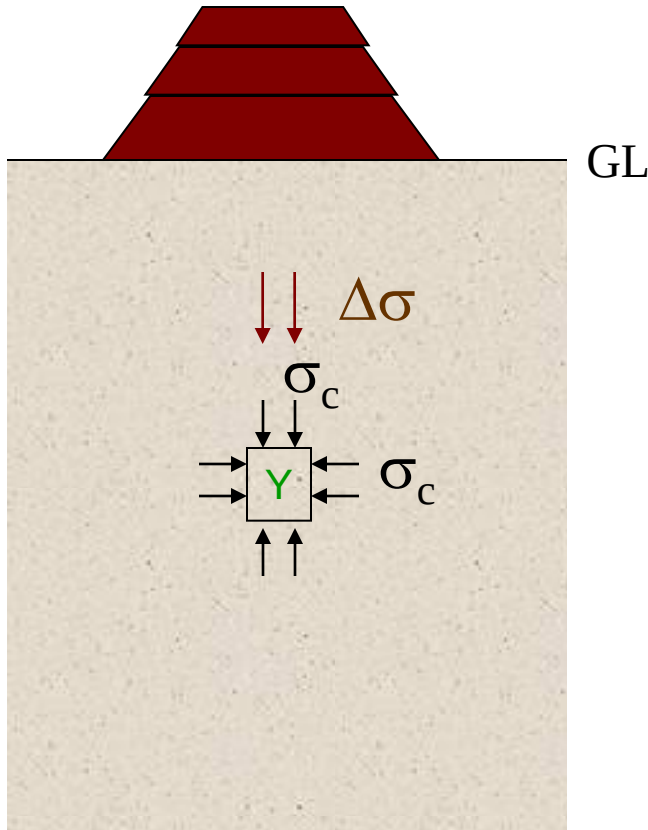
The soil element does not fail if the Mohr circle is contained within the envelope



Initially, Mohr circle is a point

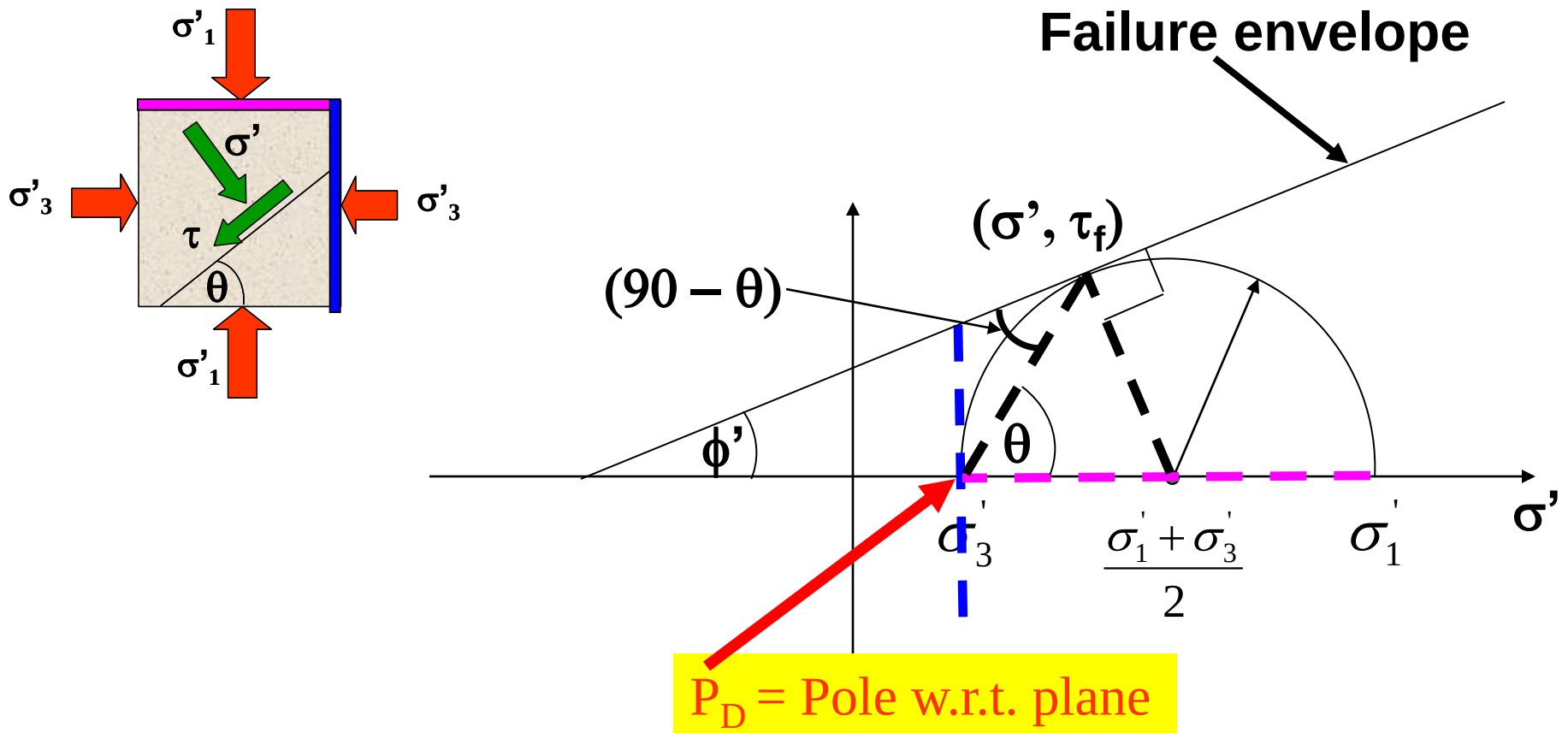
Mohr Circles & Failure Envelope

As loading progresses, Mohr circle becomes larger...



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

Orientation of Failure Plane

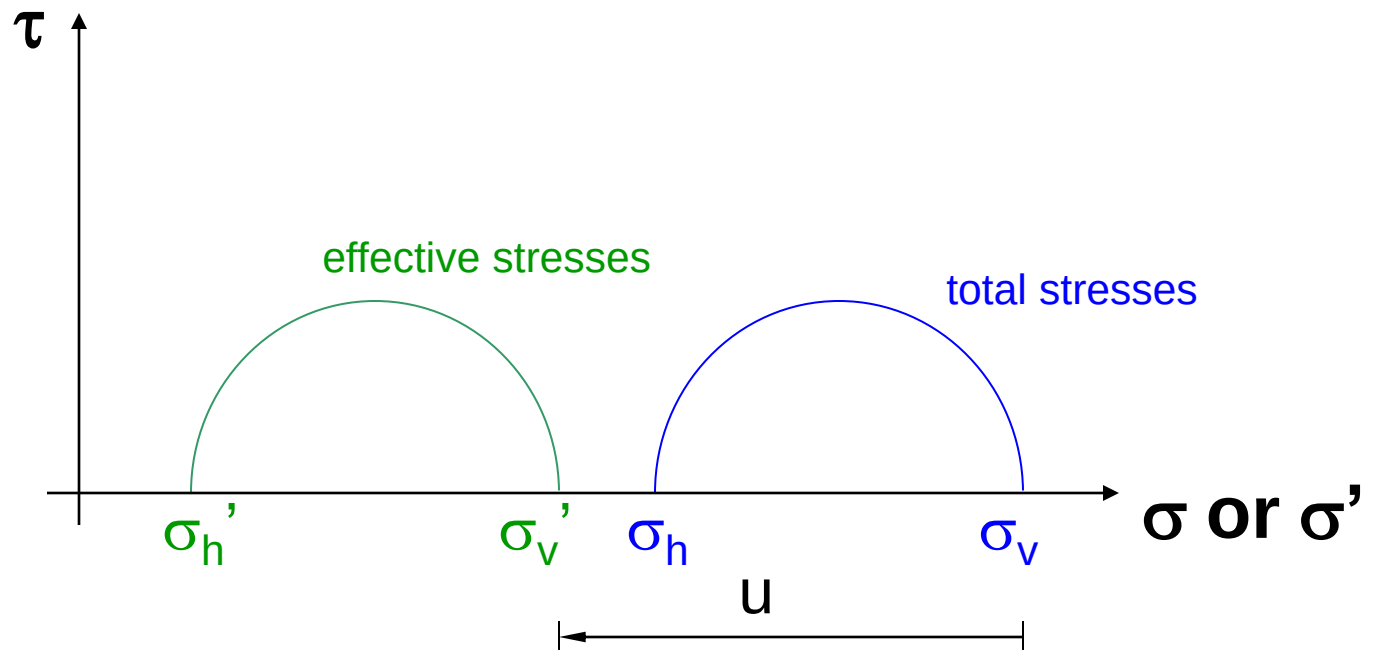
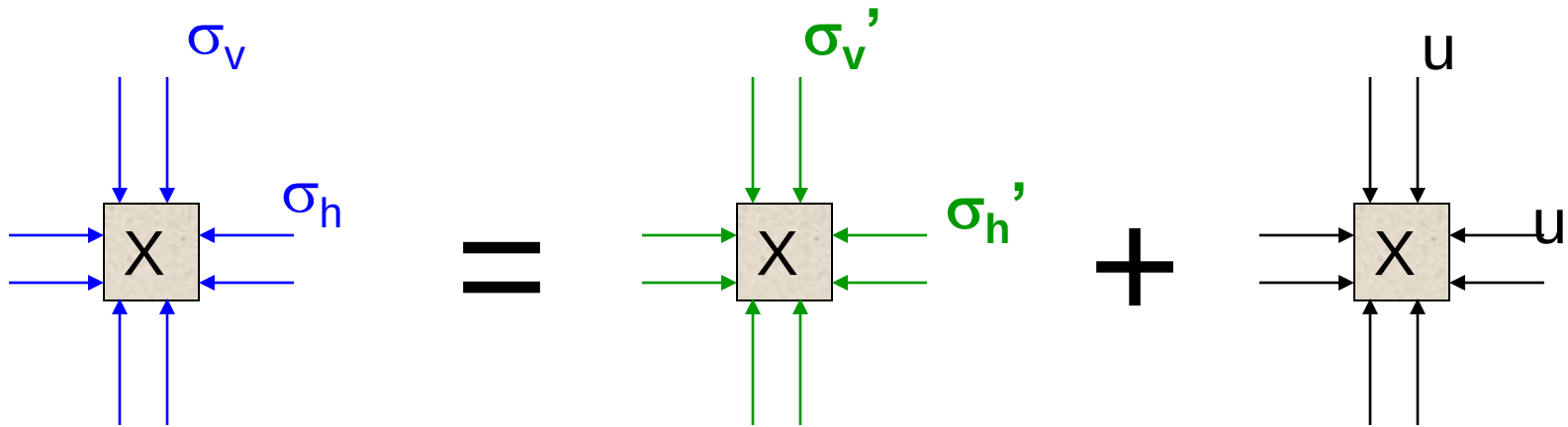


Therefore,

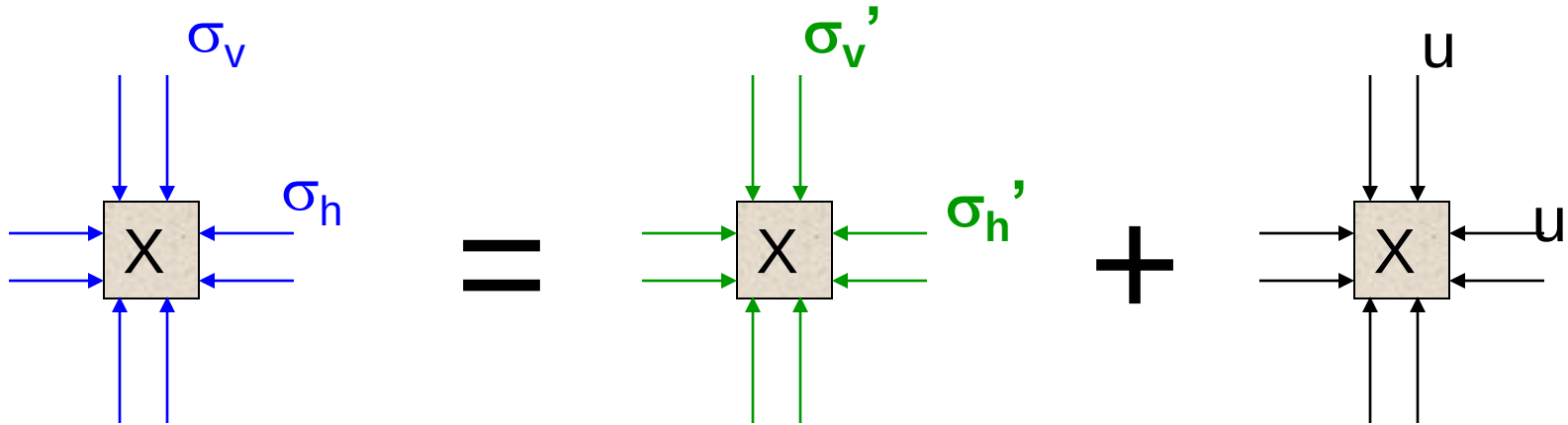
$$90 - \theta + \phi' = \theta$$

$$\theta = 45 + \phi'/2$$

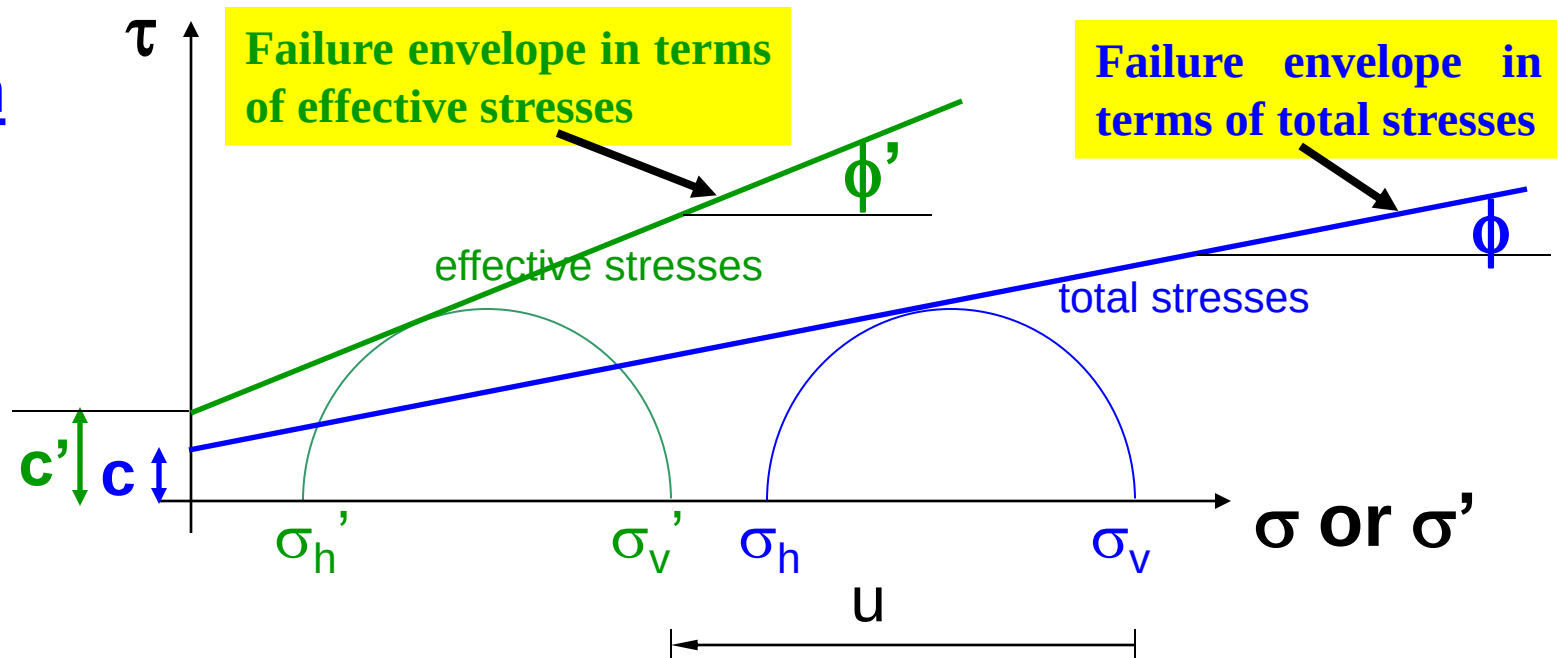
Mohr circles in terms of **total** & **effective** stresses



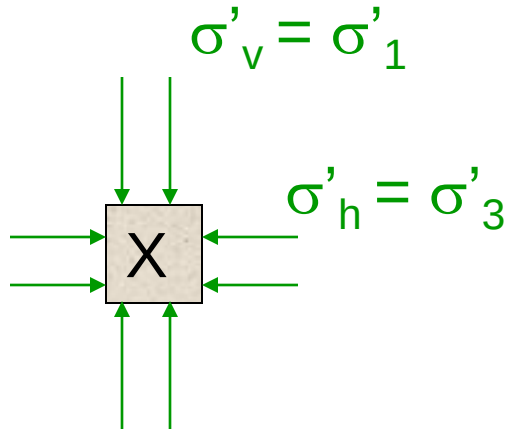
Failure envelopes in terms of **total** & **effective** stresses



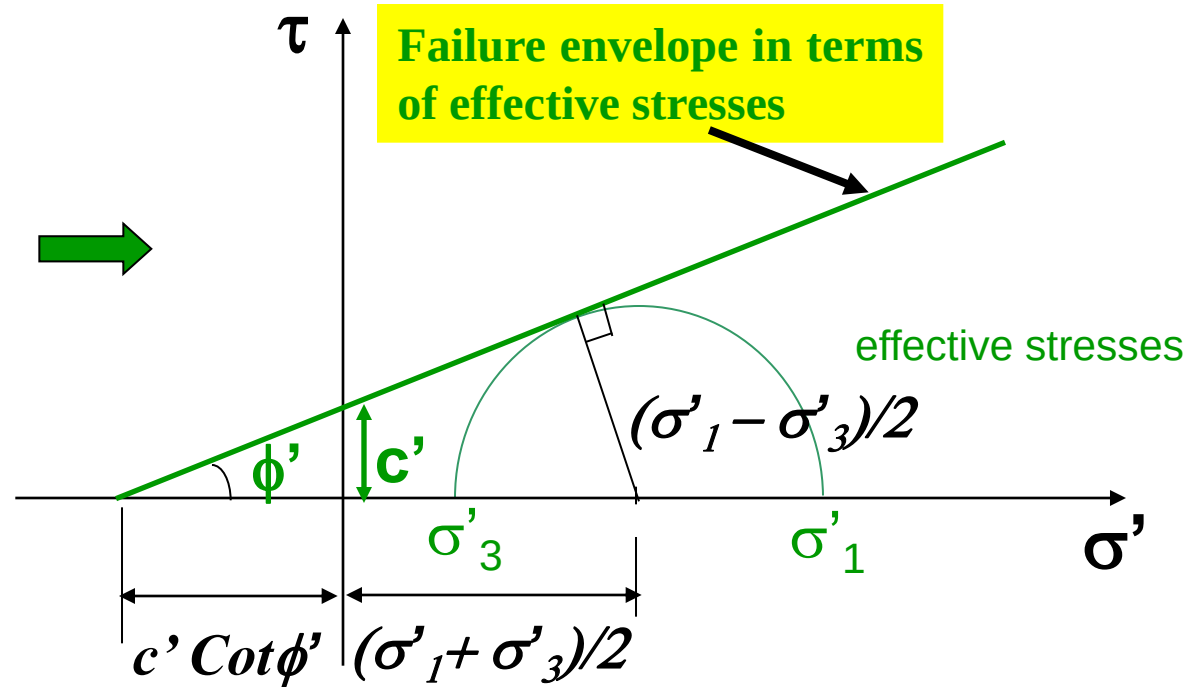
If X is on failure



Mohr Coulomb failure criterion with Mohr circle of stress



X is on failure



Therefore,

$$\left[c' \cot \phi' + \left(\frac{\sigma'_1 + \sigma'_3}{2} \right) \right] \sin \phi' = \left(\frac{\sigma'_1 - \sigma'_3}{2} \right)$$

Mohr Coulomb failure criterion with Mohr circle of stress

$$\left[c' \cot \phi' + \left(\frac{\sigma_1' + \sigma_3'}{2} \right) \right] \sin \phi' = \left(\frac{\sigma_1' - \sigma_3'}{2} \right)$$

$$(\sigma_1' - \sigma_3') = (\sigma_1' + \sigma_3') \sin \phi' + 2c' \cos \phi'$$

$$\sigma_1' (1 - \sin \phi') = \sigma_3' (1 + \sin \phi') + 2c' \cos \phi'$$

$$\sigma_1' = \sigma_3' \left(\frac{1 + \sin \phi'}{1 - \sin \phi'} \right) + 2c' \left(\frac{\cos \phi'}{1 - \sin \phi'} \right)$$

$$\sigma_1' = \sigma_3' \tan^2 \left(45 + \frac{\phi'}{2} \right) + 2c' \tan \left(45 + \frac{\phi'}{2} \right)$$

Determination of shear strength parameters of soils (c , ϕ or c' , ϕ')

```
graph TD; A[Determination of shear strength parameters of soils (c, φ or c', φ')] --> B[Laboratory tests on specimens taken from representative undisturbed samples]; A --> C[Field tests]; B --> D[Most common laboratory tests to determine the shear strength parameters are, 1. Direct shear test 2. Triaxial shear test]; C --> E[1. Vane shear test 2. Torvane 3. Pocket penetrometer 4. Fall cone 5. Pressuremeter 6. Static cone penetrometer 7. Standard penetration test];
```

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**

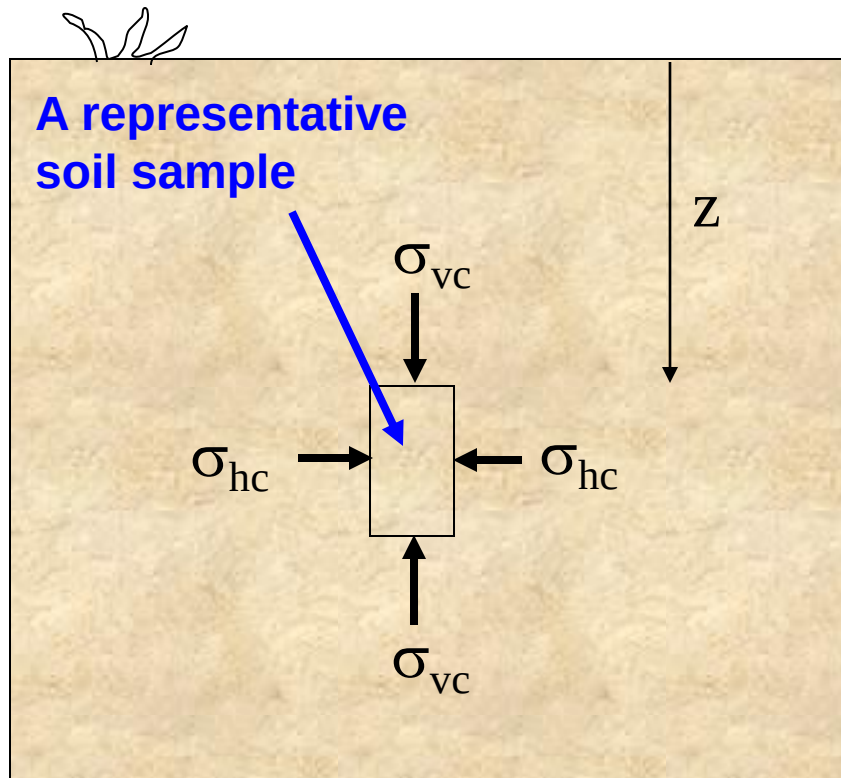
Other laboratory tests include,
Direct simple shear test, torsional ring shear test, plane strain triaxial test, laboratory vane shear test, laboratory fall cone test

Field tests

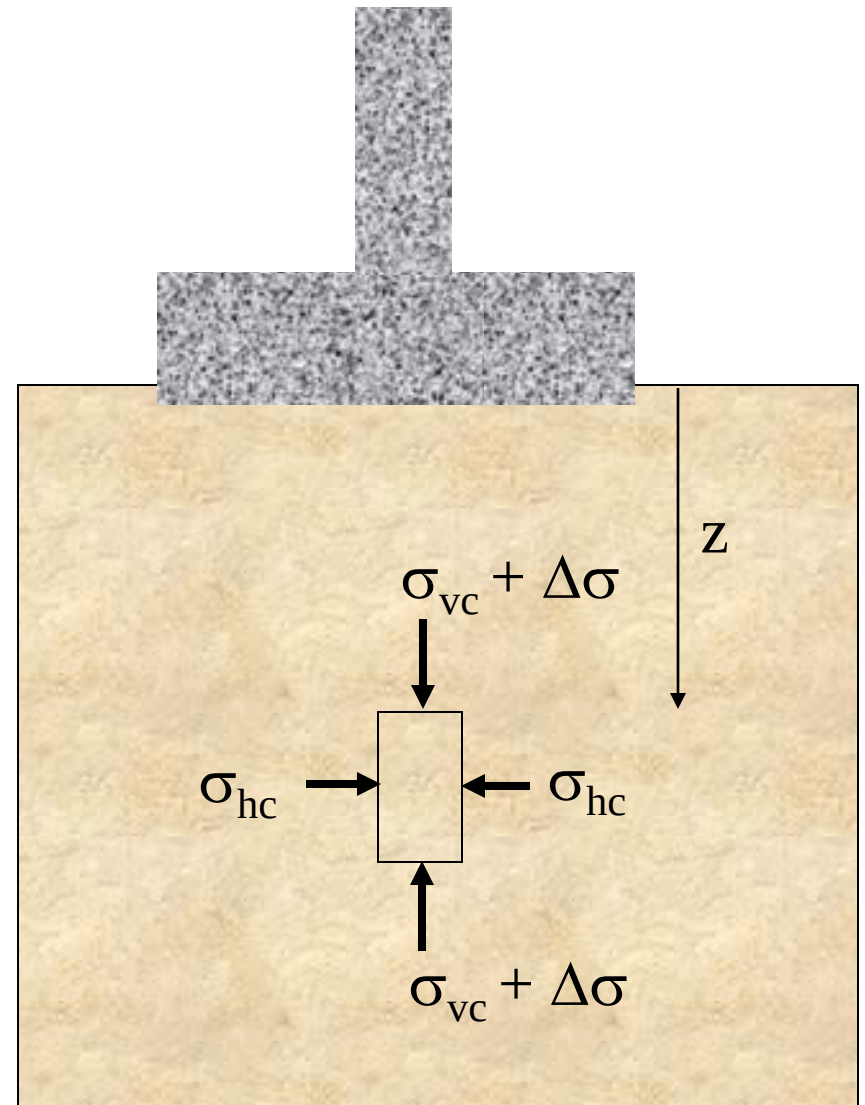
- 1. Vane shear test**
- 2. Torvane**
- 3. Pocket penetrometer**
- 4. Fall cone**
- 5. Pressuremeter**
- 6. Static cone penetrometer**
- 7. Standard penetration test**

Laboratory tests

Field conditions



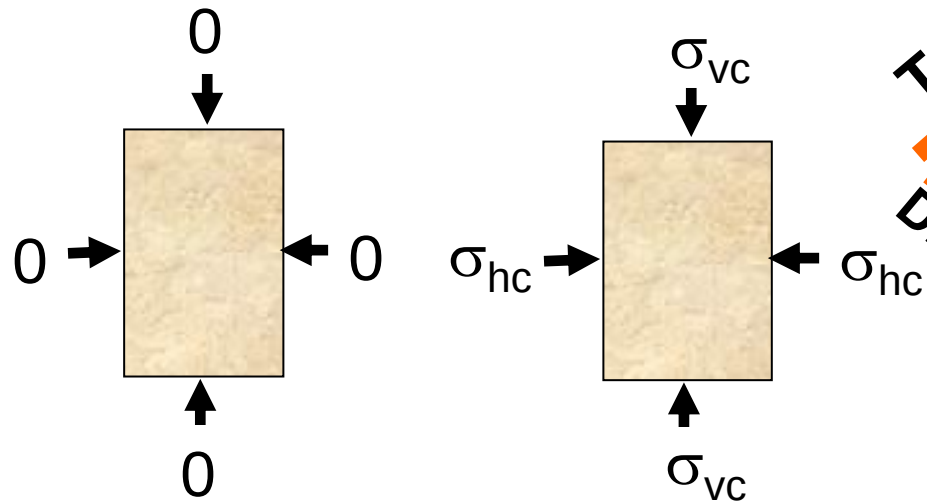
Before construction



After and during construction

Laboratory tests

Simulating field conditions in the laboratory



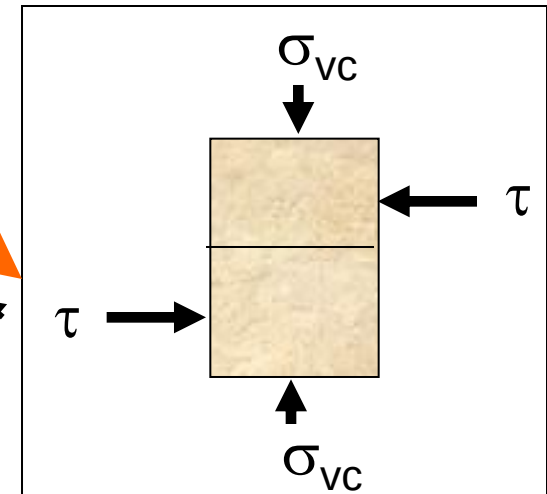
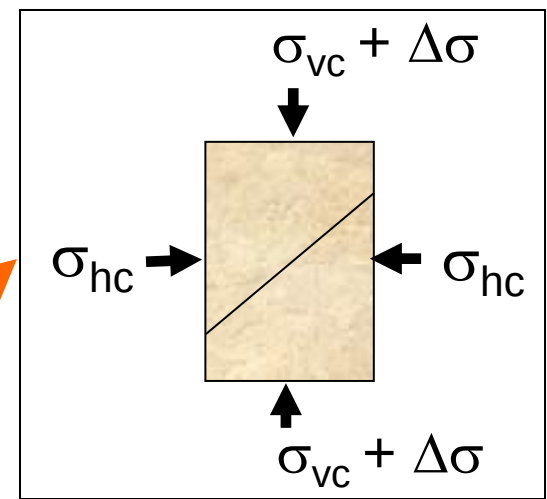
Representative soil sample taken from the site

Step 1

Set the specimen in the apparatus and apply the initial stress condition

Triaxial test

Direct shear test

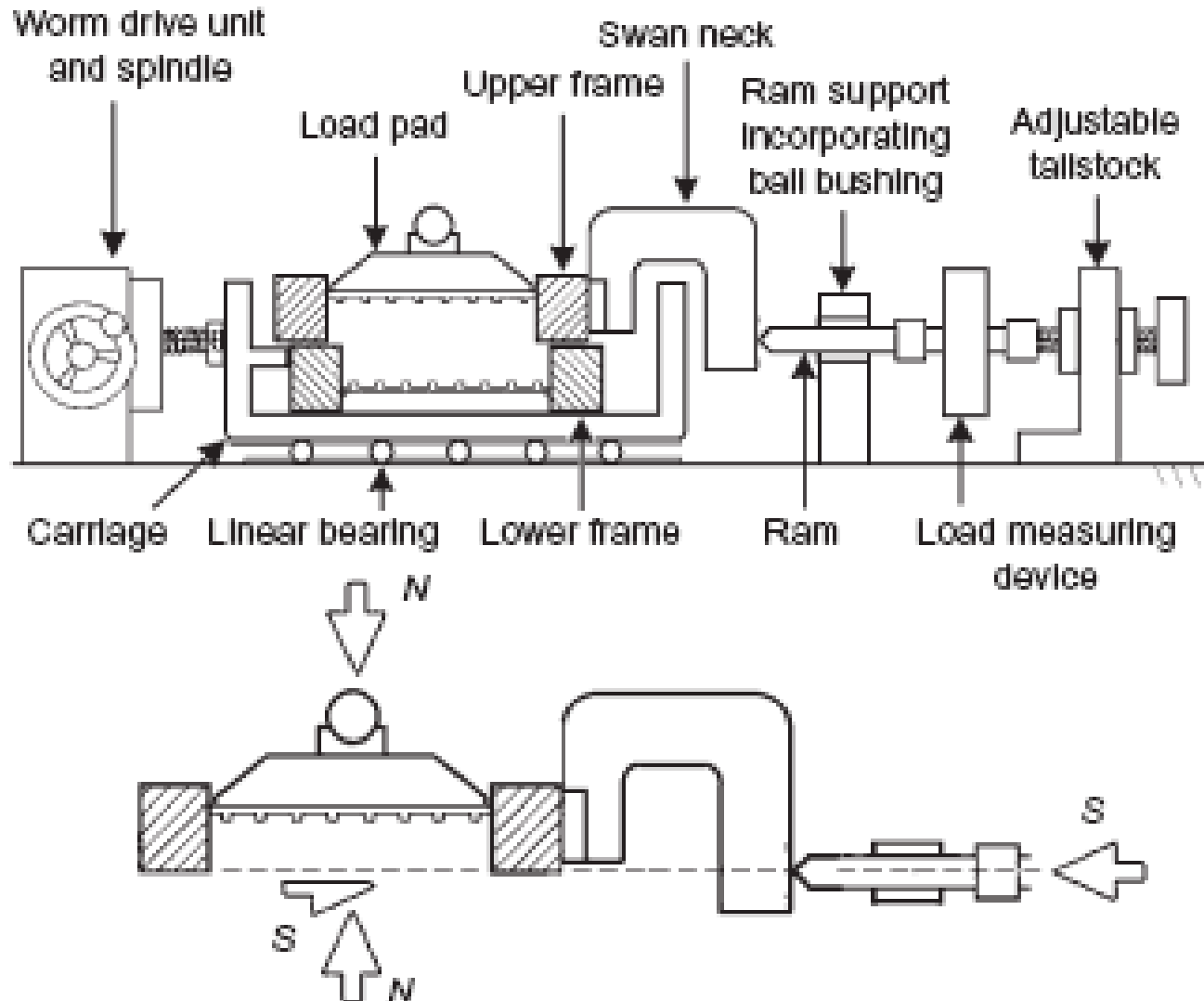


Step 2

Apply the corresponding field stress conditions

Direct shear test

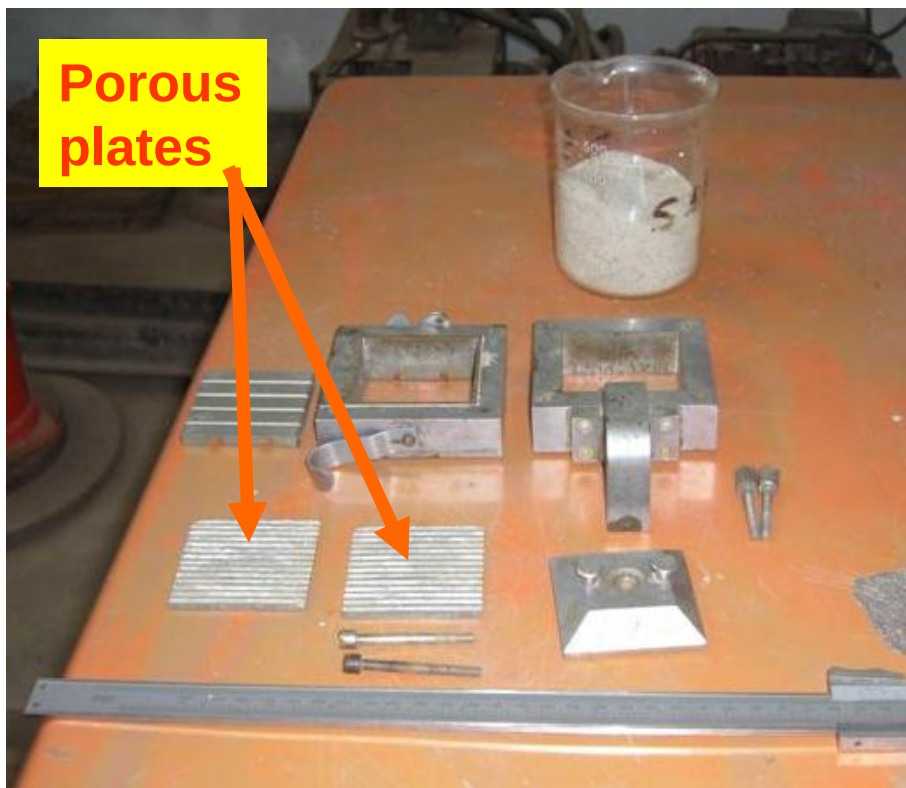
Schematic diagram of the direct shear apparatus



Direct shear test

Direct shear test is most suitable for consolidated drained tests specially on granular soils (e.g.: sand) or stiff clays

Preparation of a sand specimen



Components of the shear box



Preparation of a sand specimen

Direct shear test

Preparation of a sand specimen



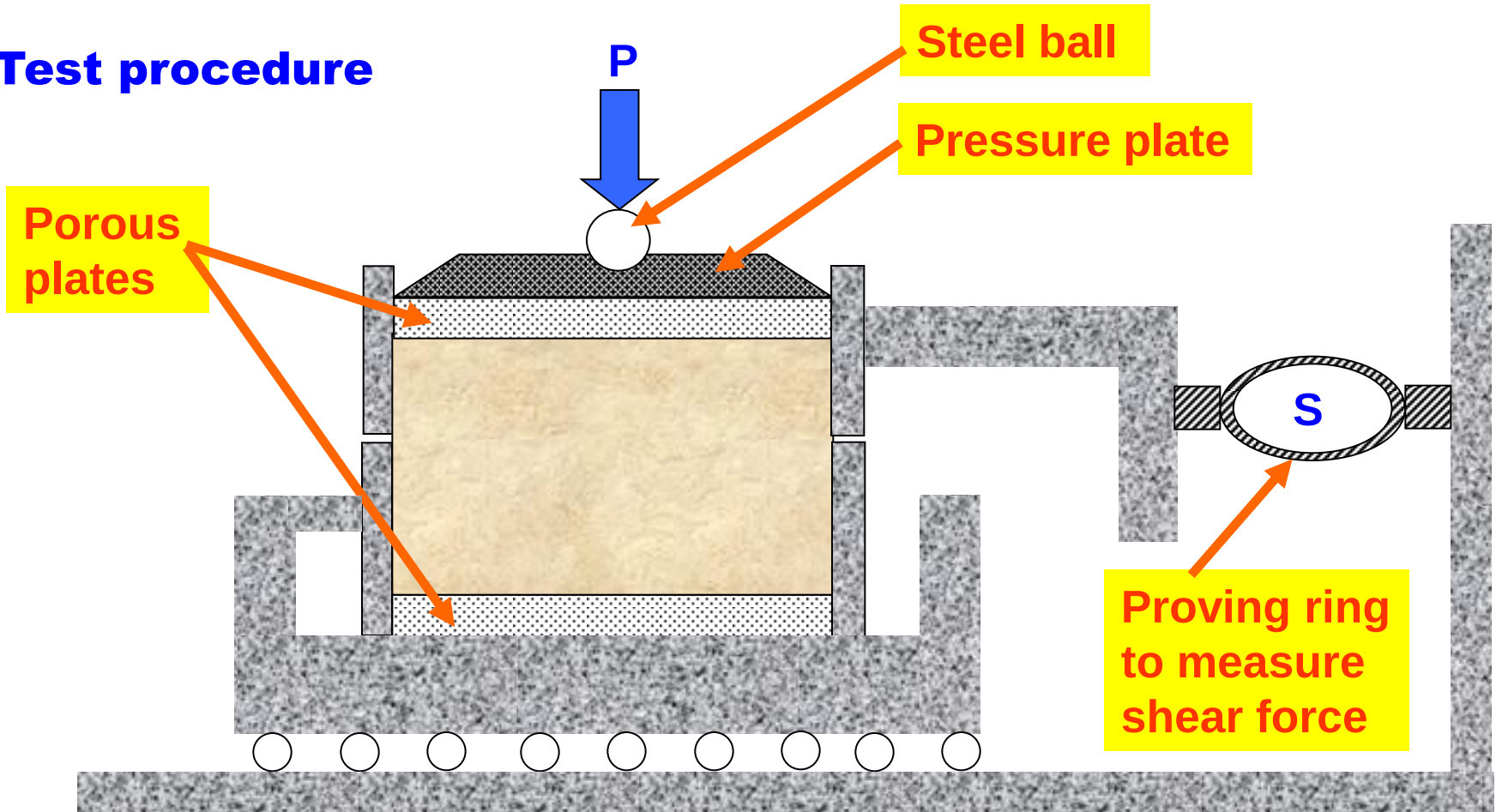
Leveling the top surface
of specimen



Specimen preparation
completed

Direct shear test

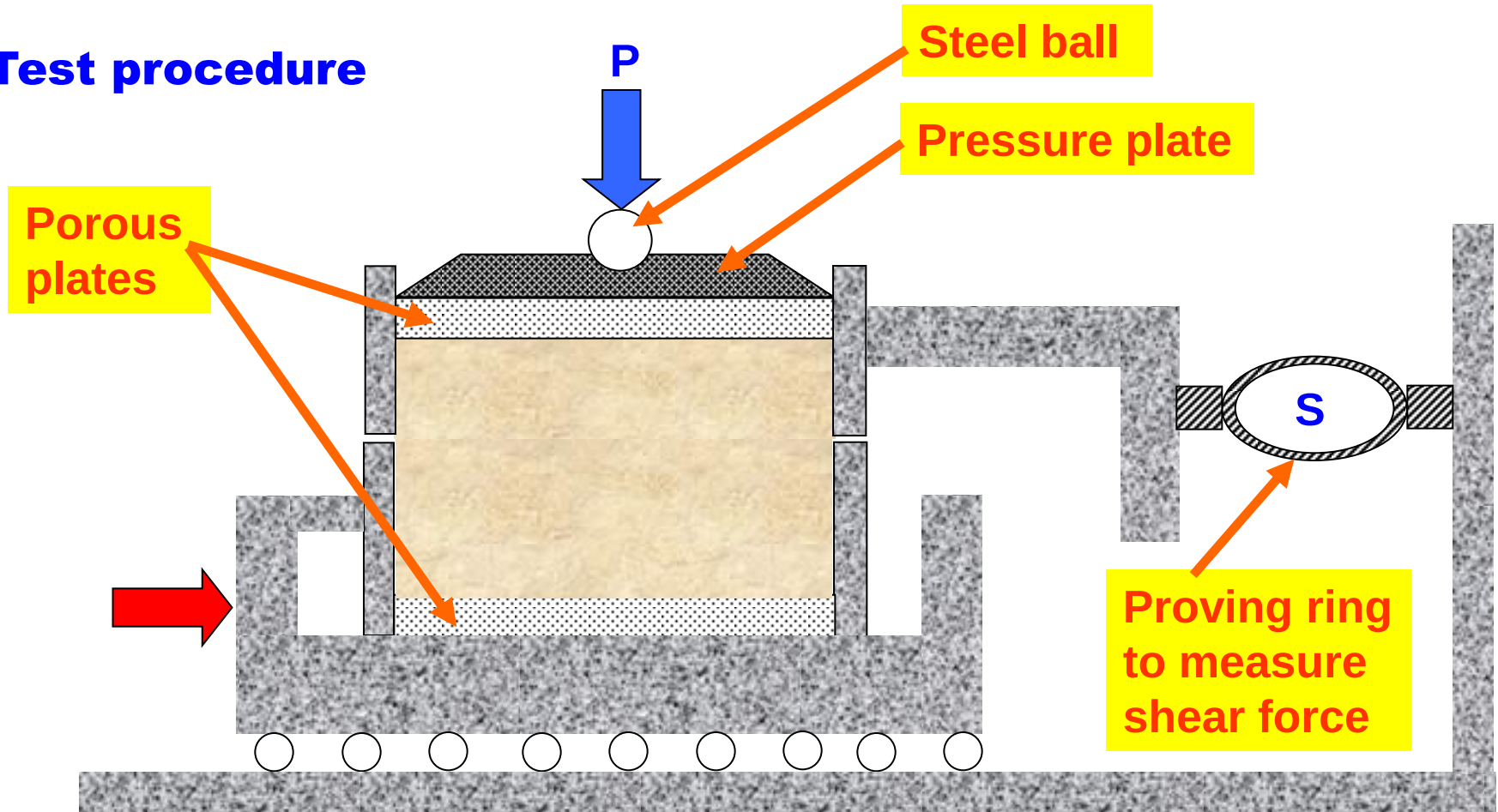
Test procedure



Step 1: Apply a vertical load to the specimen and wait for consolidation

Direct shear test

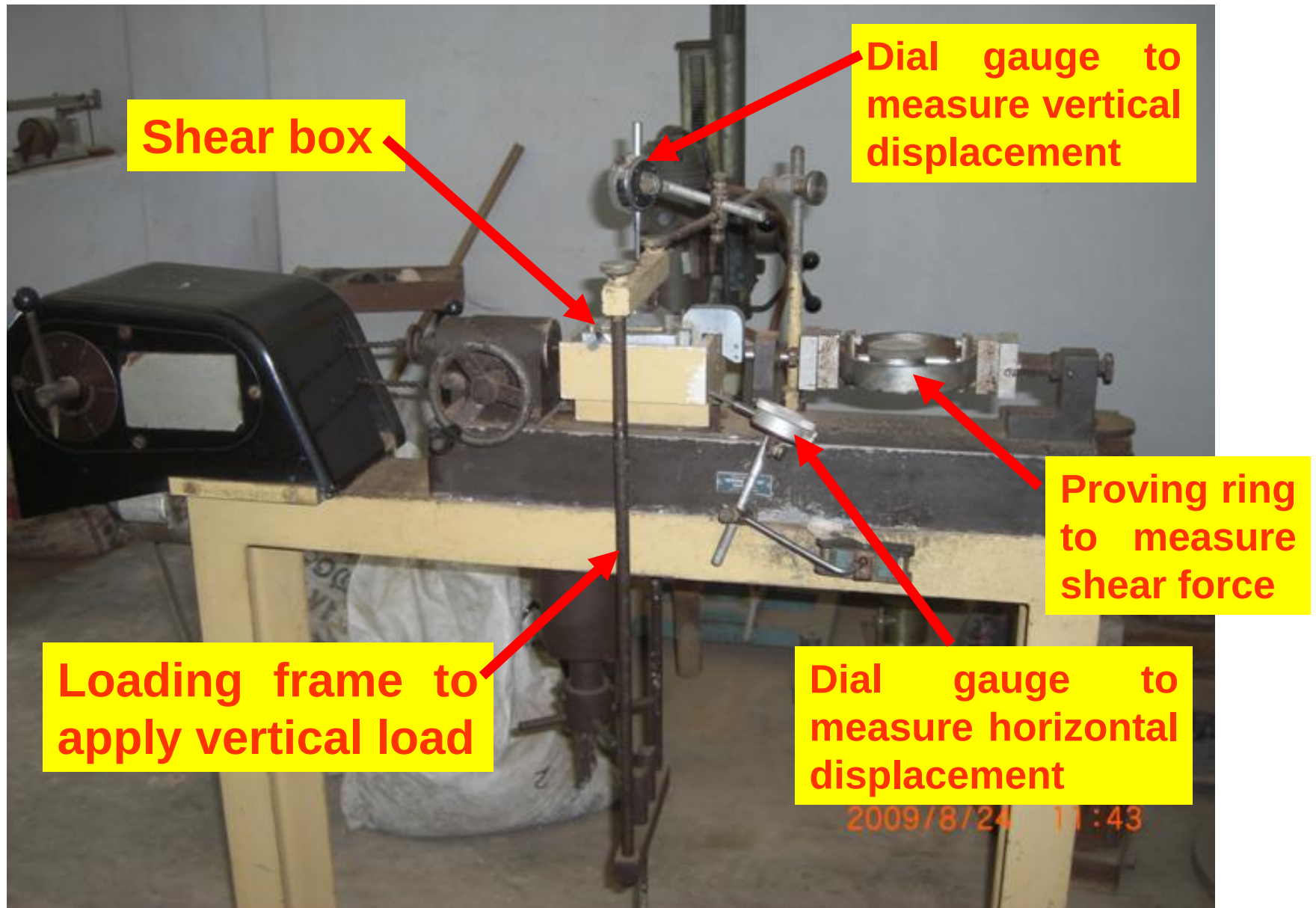
Test procedure



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



Direct shear test

Analysis of test results

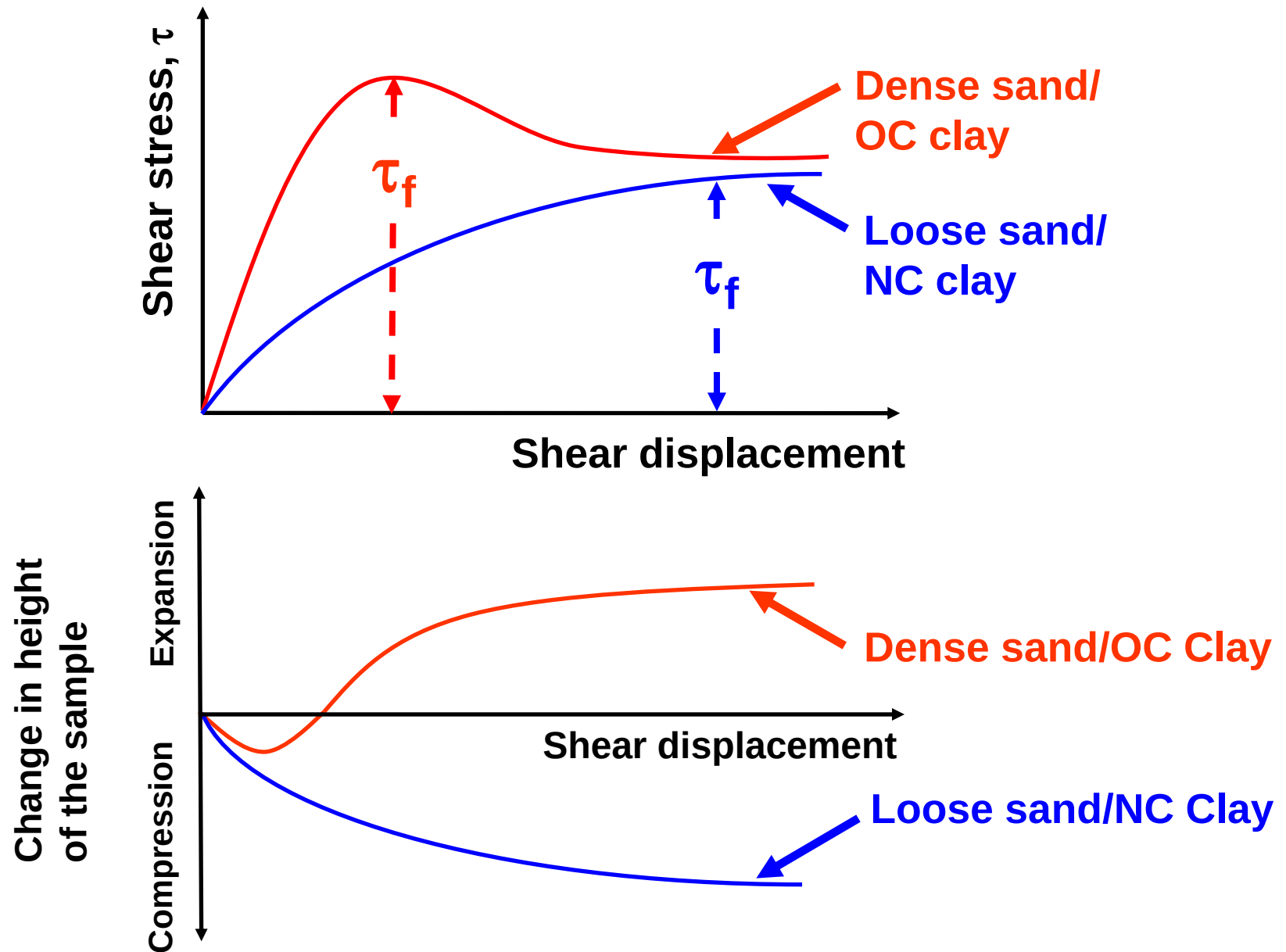
$$\sigma = \text{Normal stress} = \frac{\text{Normal force (P)}}{\text{Area of cross section of the sample}}$$

$$\tau = \text{Shear stress} = \frac{\text{Shear resistance developed at the sliding surface (S)}}{\text{Area of cross section of the sample}}$$

Note: Cross-sectional area of the sample changes with the horizontal displacement

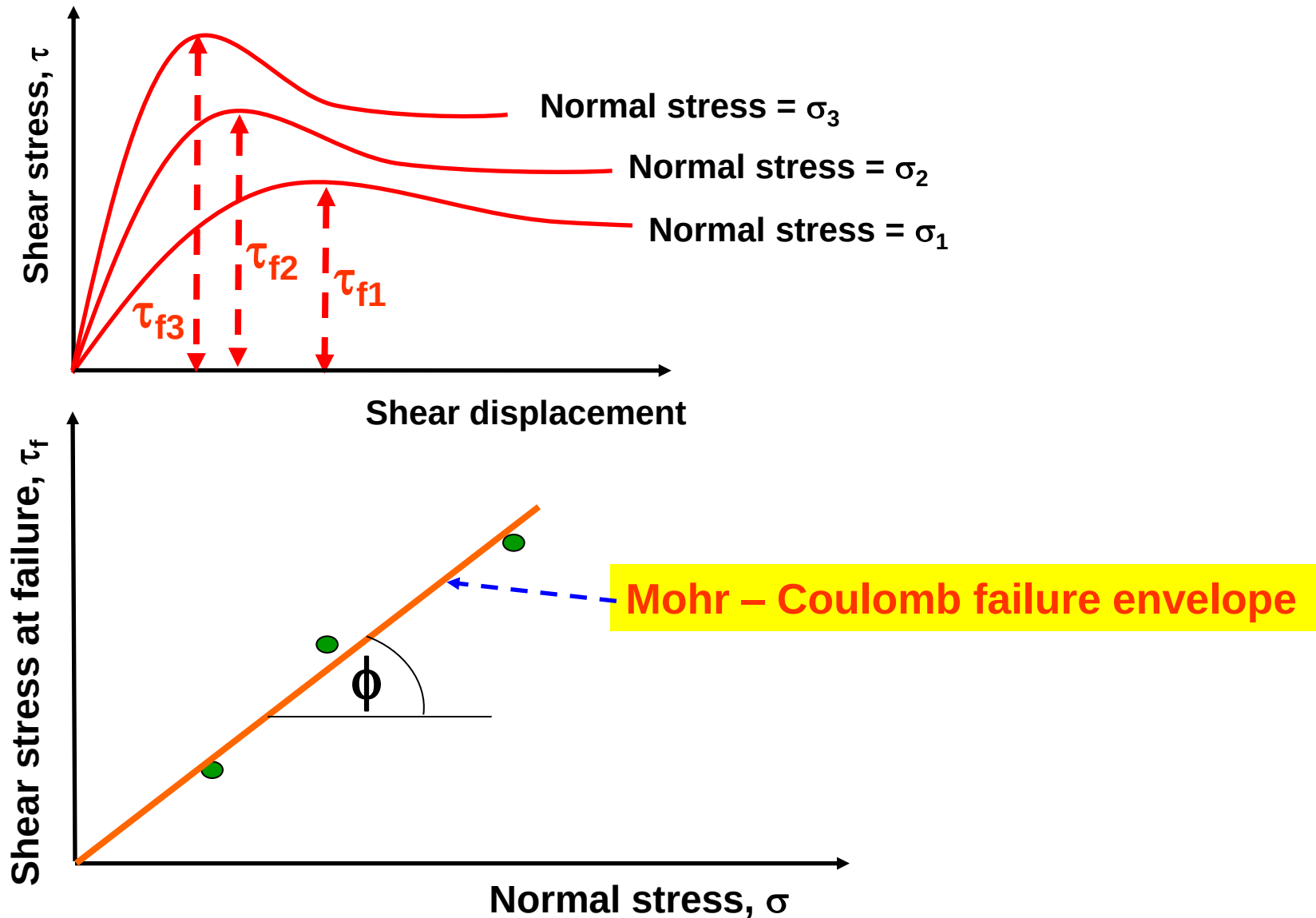
Direct shear tests on sands

Stress-strain relationship



Direct shear tests on sands

How to determine strength parameters c and ϕ



Direct shear tests on sands

Some important facts on strength parameters c and ϕ of sand

Sand is cohesionless
hence $c = 0$

Direct shear tests are
drained and pore water
pressures are
dissipated, hence $u = 0$

Therefore,

$\phi' = \phi$ and $c' = c = 0$



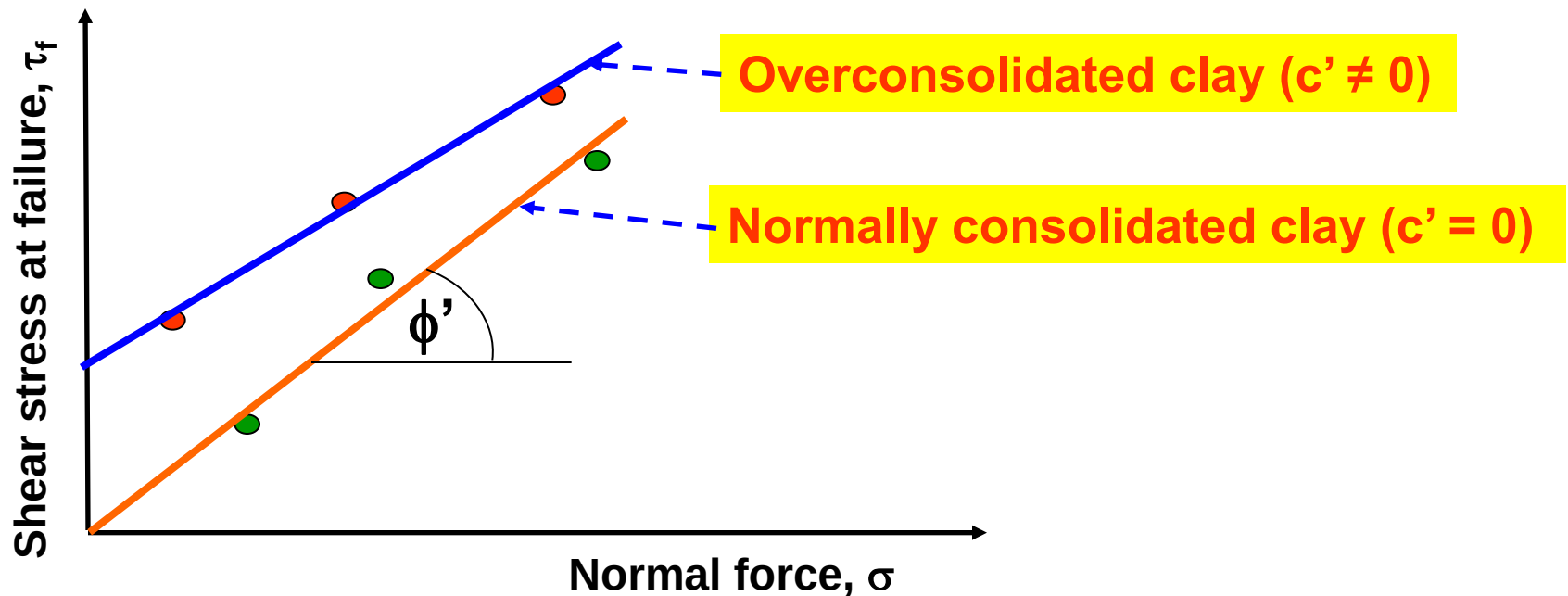
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A GENTLE REMINDER ...

Direct shear tests on clays

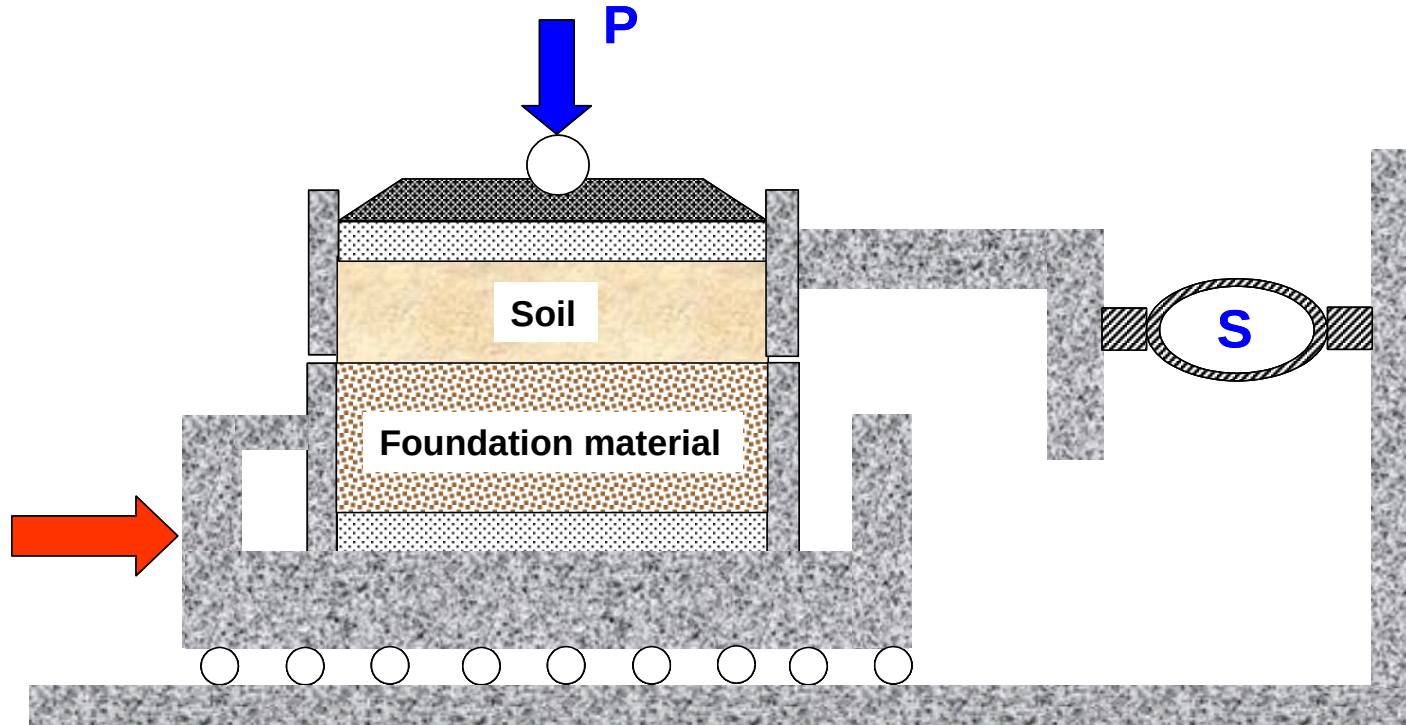
In case of clay, horizontal displacement should be applied at a very slow rate to allow dissipation of pore water pressure (therefore, one test would take several days to finish)

Failure envelopes for clay from drained direct shear tests



Interface tests on direct shear apparatus

In many foundation design problems and retaining wall problems, it is required to determine the angle of internal friction between soil and the structural material (concrete, steel or wood)



$$\tau_f = c_a + \sigma' \tan \delta$$

Where,

c_a = adhesion,

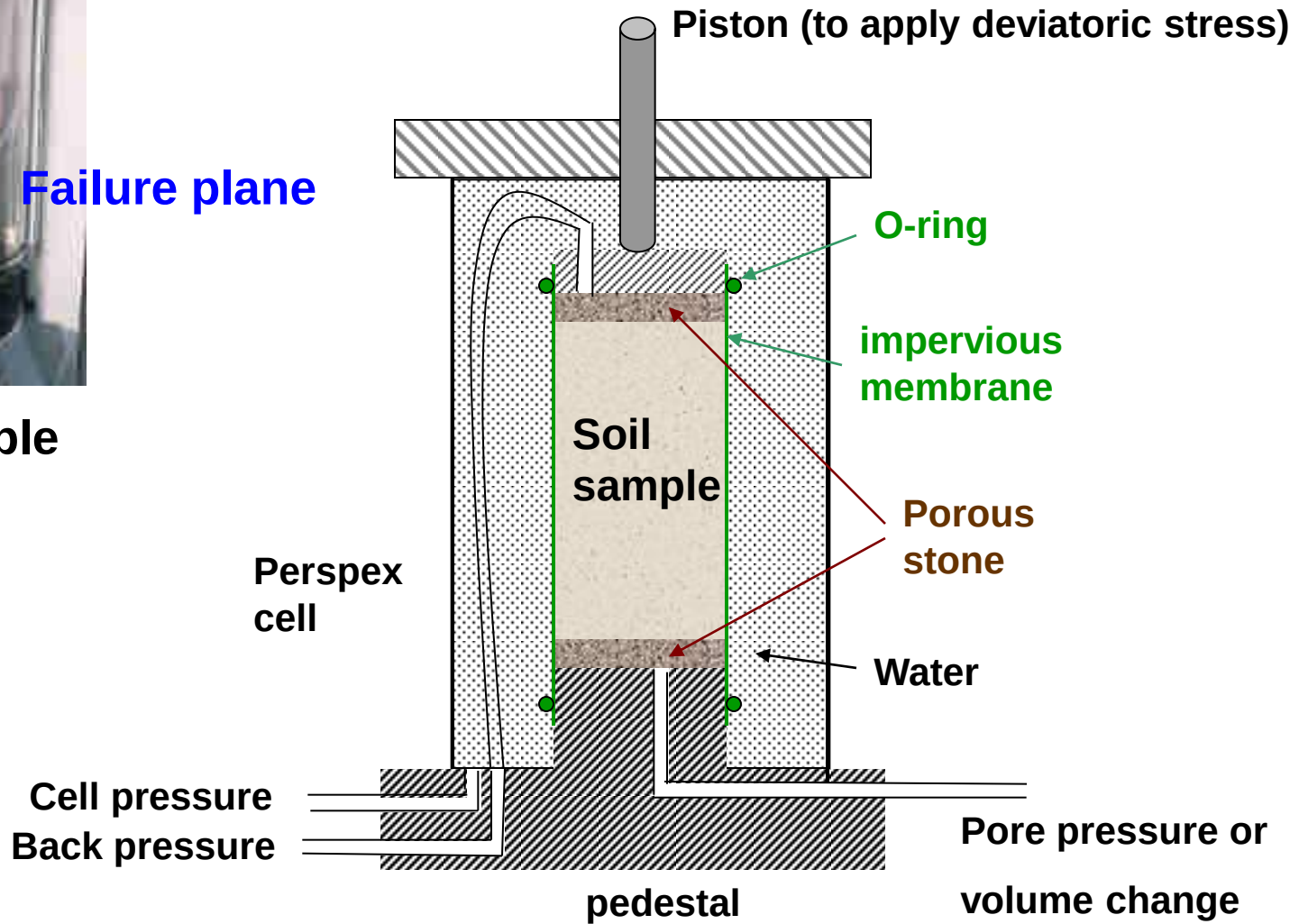
δ = angle of internal friction

Triaxial Shear Test



Failure plane

Soil sample
at failure



Triaxial Shear Test

Specimen preparation (undisturbed sample)



Sampling tubes



Sample extruder

Triaxial Shear Test

Specimen preparation (undisturbed sample)



Edges of the sample
are carefully trimmed



Setting up the sample
in the triaxial cell

Triaxial Shear Test

Specimen preparation (undisturbed sample)



Sample is covered with a rubber membrane and sealed



Cell is completely filled with water

Triaxial Shear Test

Specimen preparation (undisturbed sample)

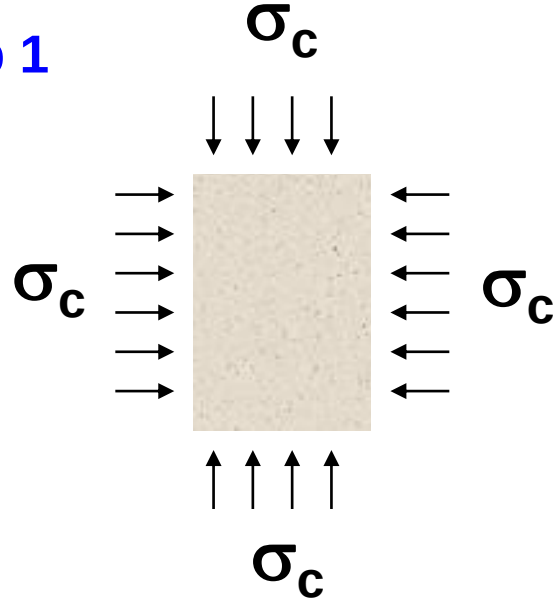


Proving ring to measure the deviator load

Dial gauge to measure vertical displacement

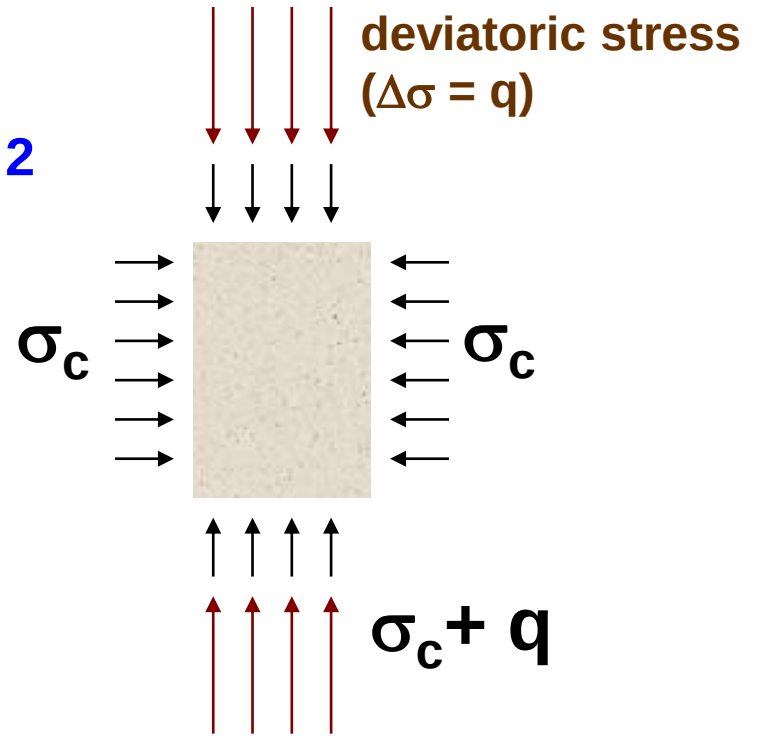
Types of Triaxial Tests

Step 1



Under all-around cell pressure σ_c

Step 2



Shearing (loading)

Is the drainage valve open?

yes

no

Consolidated
sample

Unconsolidated
sample

Is the drainage valve open?

yes

no

Drain
loading

Undrain
loading

Types of Triaxial Tests

Step 1

Under all-around cell pressure σ_c

Is the drainage valve open?

yes

no

Consolidated
sample

Unconsolidated
sample

CD test

Step 2

Shearing (loading)

Is the drainage valve open?

yes

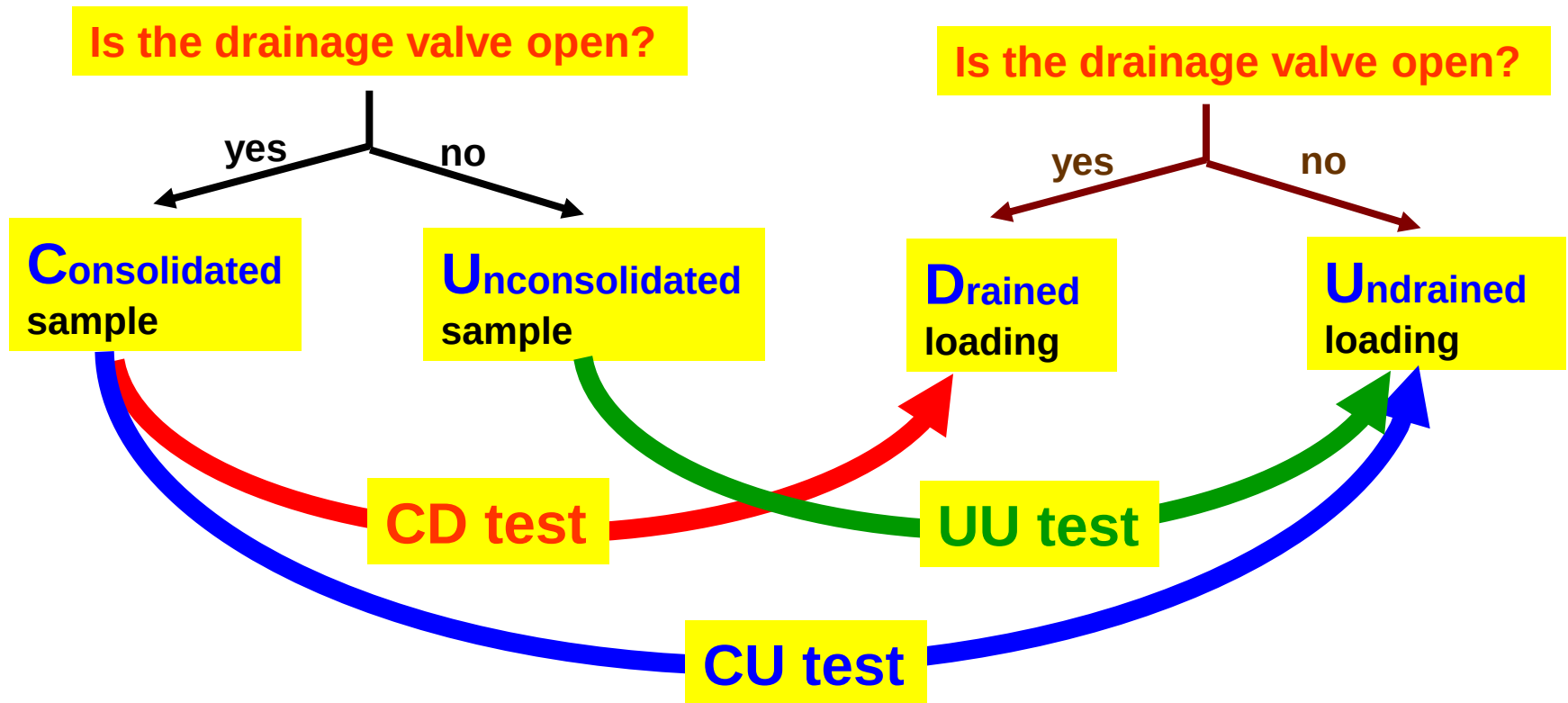
no

Drained
loading

Undrained
loading

UU test

CU test



Consolidated- drained test (CD Test)

Total, σ

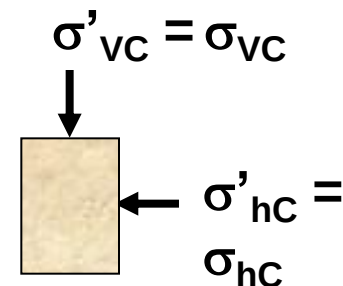
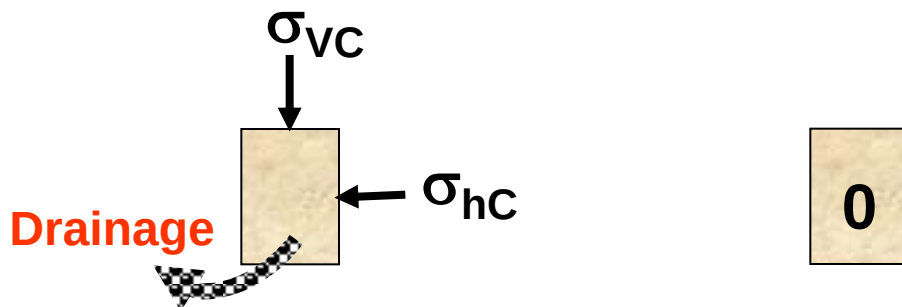
=

Neutral, u

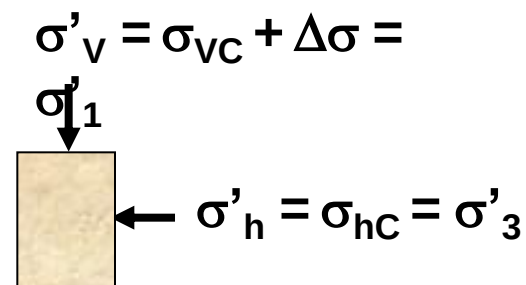
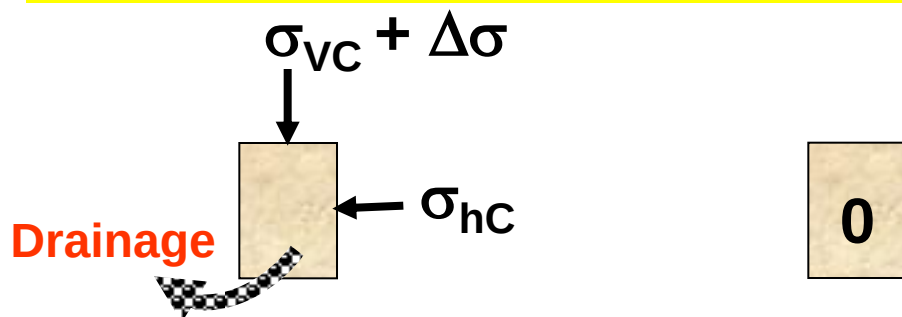
+

Effective, σ'

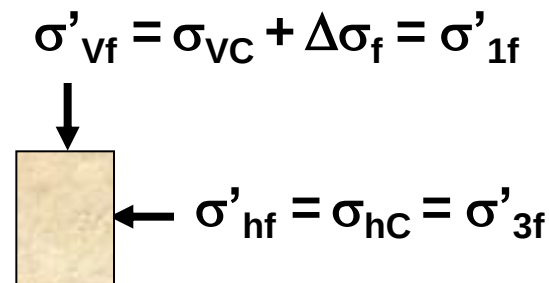
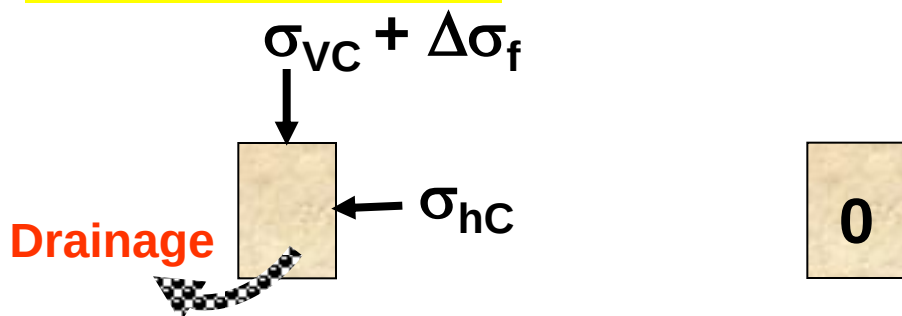
Step 1: At the end of consolidation



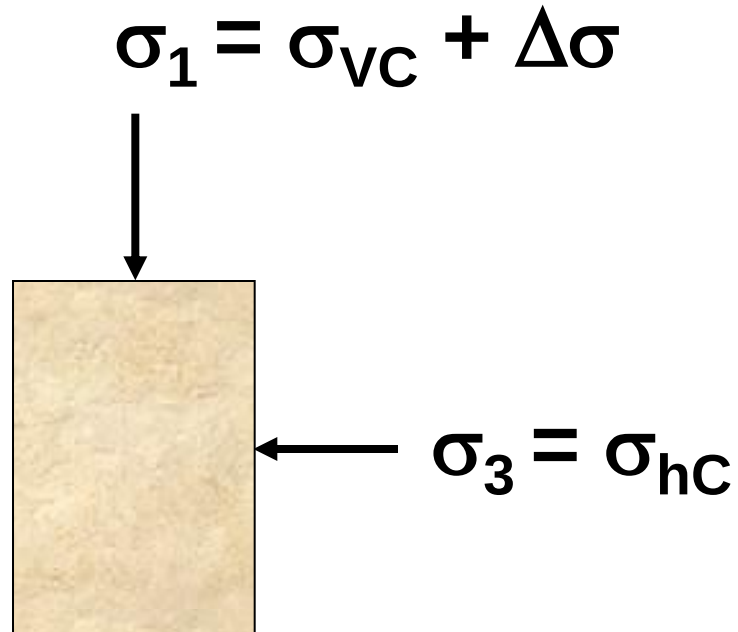
Step 2: During axial stress increase



Step 3: At failure



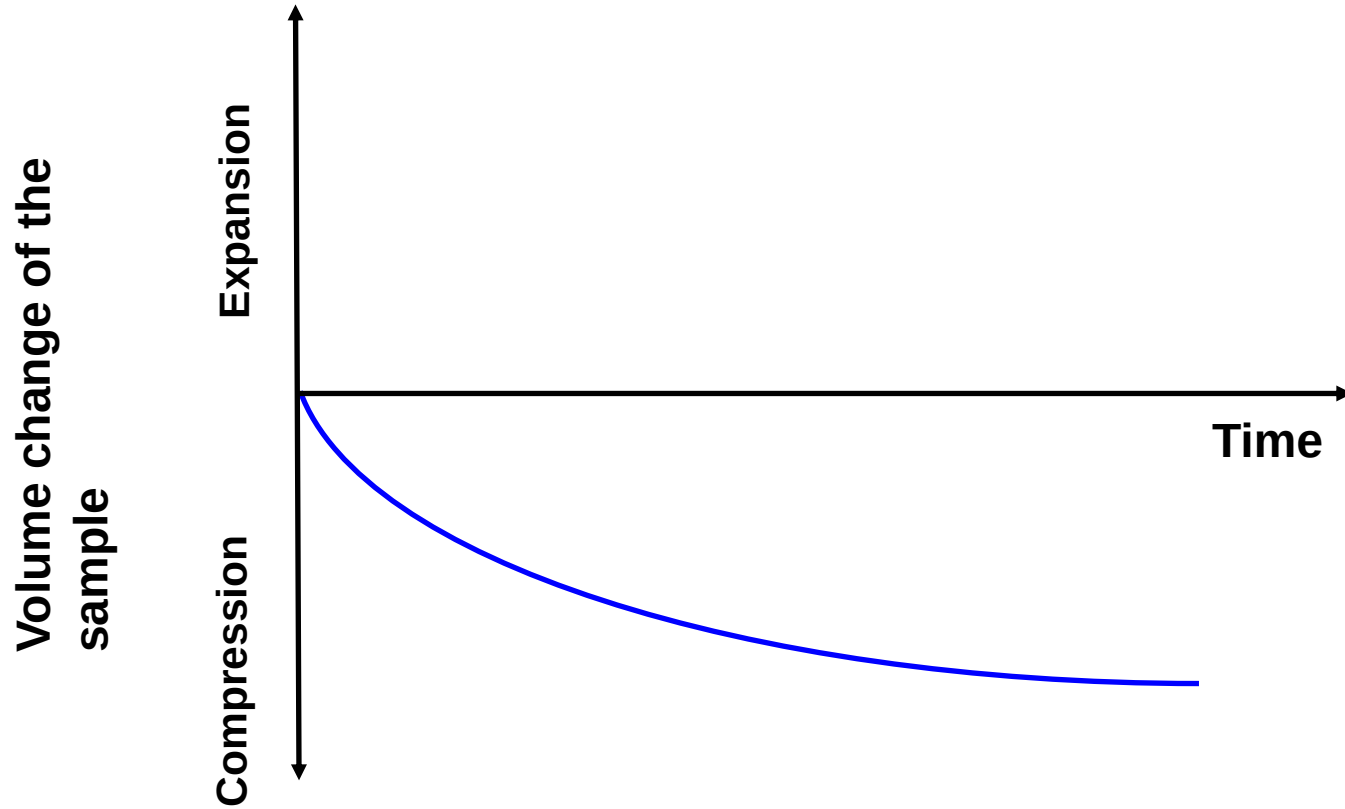
Consolidated- drained test (CD Test)



Deviator stress (q or $\Delta\sigma_d$) = $\sigma_1 - \sigma_3$

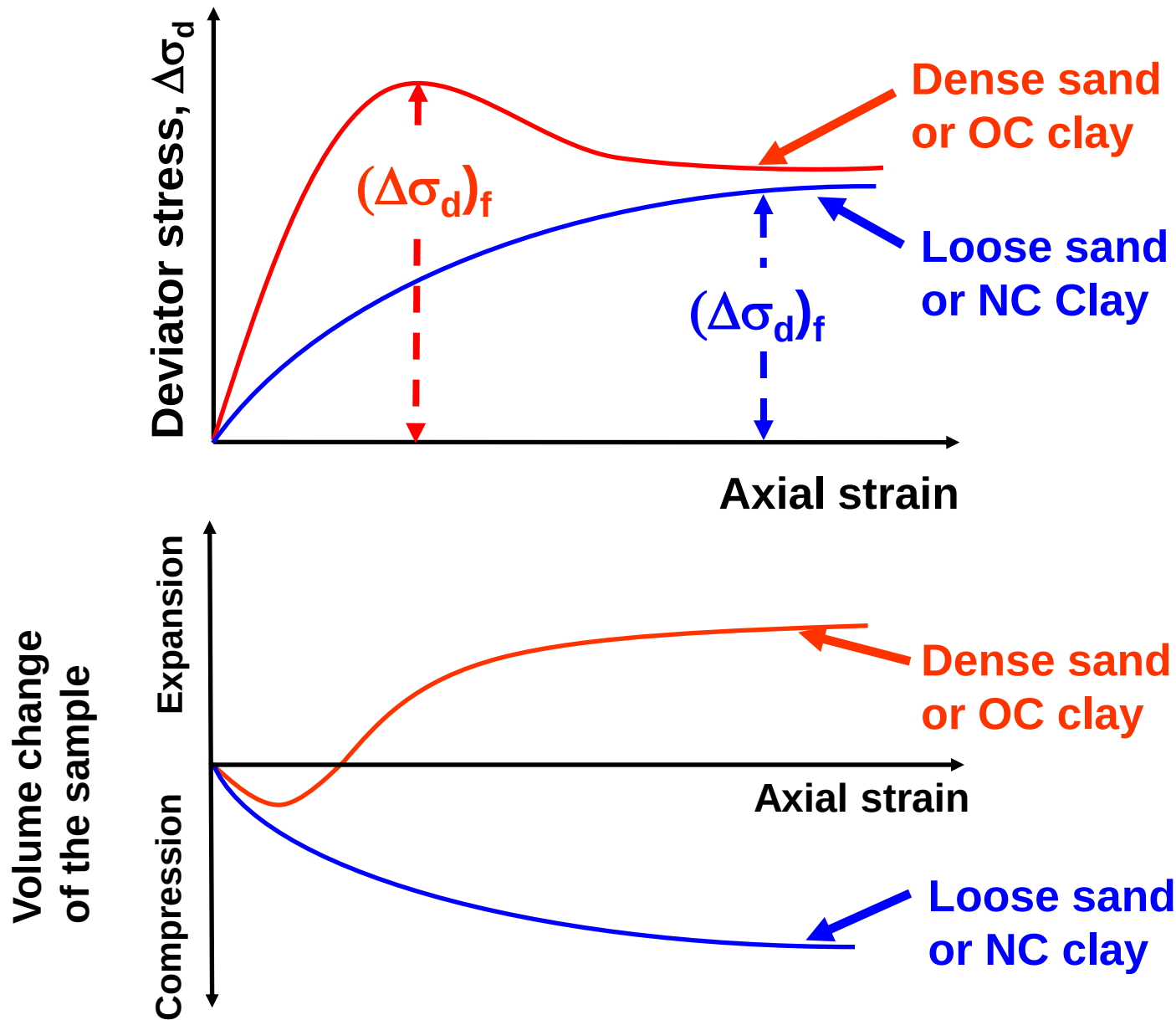
Consolidated- drained test (CD Test)

Volume change of sample during consolidation

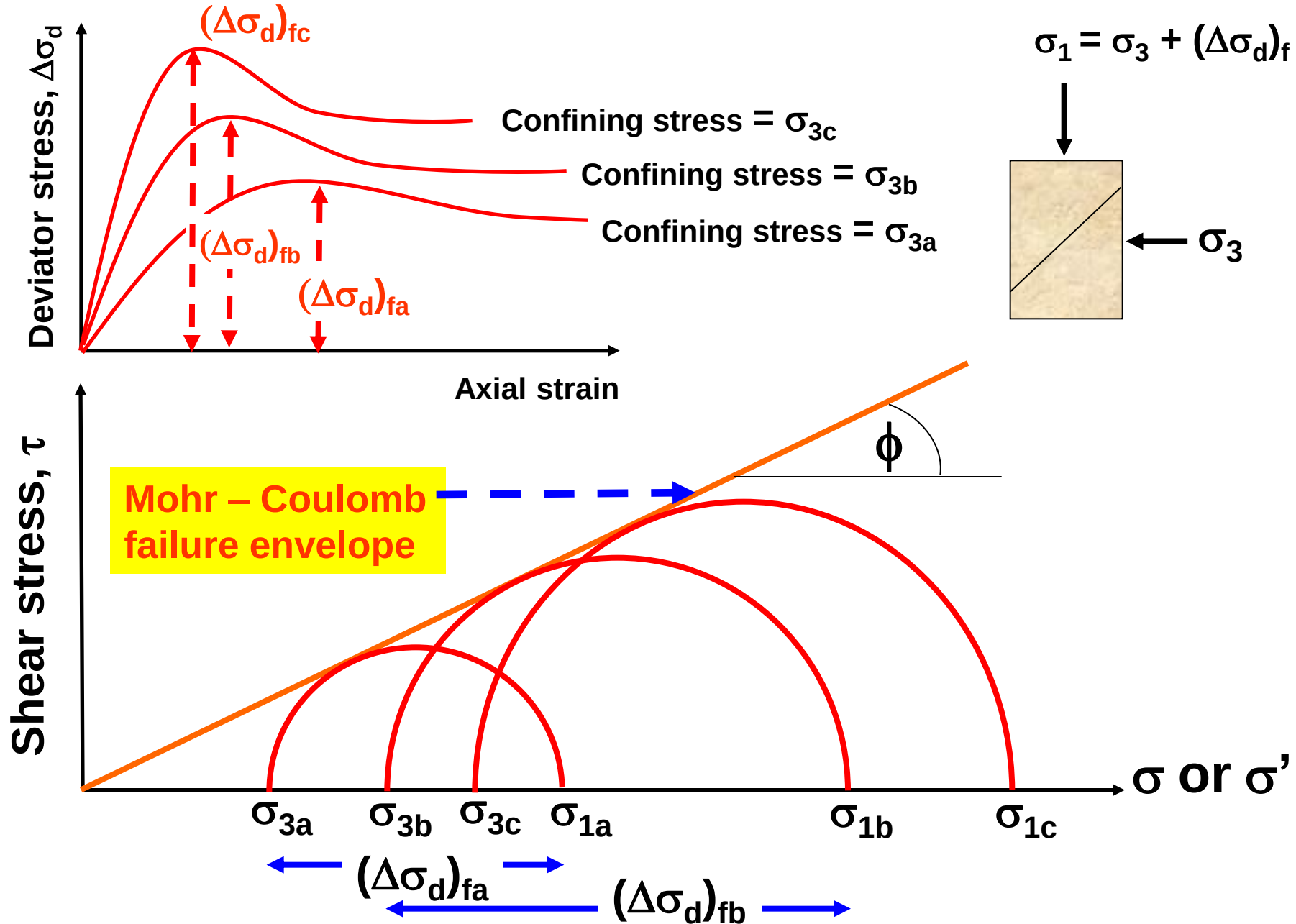


Consolidated- drained test (CD Test)

Stress-strain relationship during shearing



CD tests How to determine strength parameters c and ϕ



CD tests

Strength parameters c and ϕ obtained from CD tests

Since $u = 0$ in CD tests, $\sigma = \sigma'$

Therefore, $c = c'$ and $\phi = \phi'$

c_d and ϕ_d are used to denote them

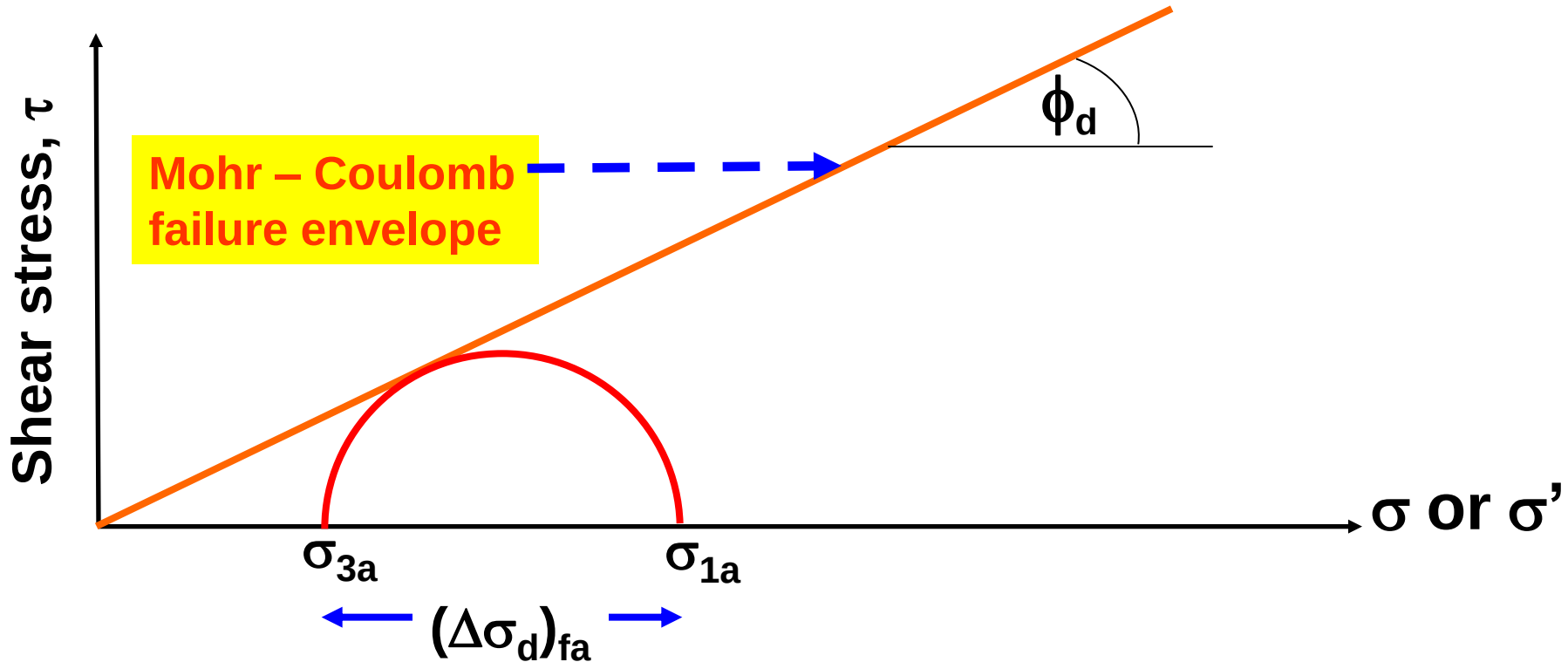


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A GENTLE REMINDER ...

CD tests Failure envelopes

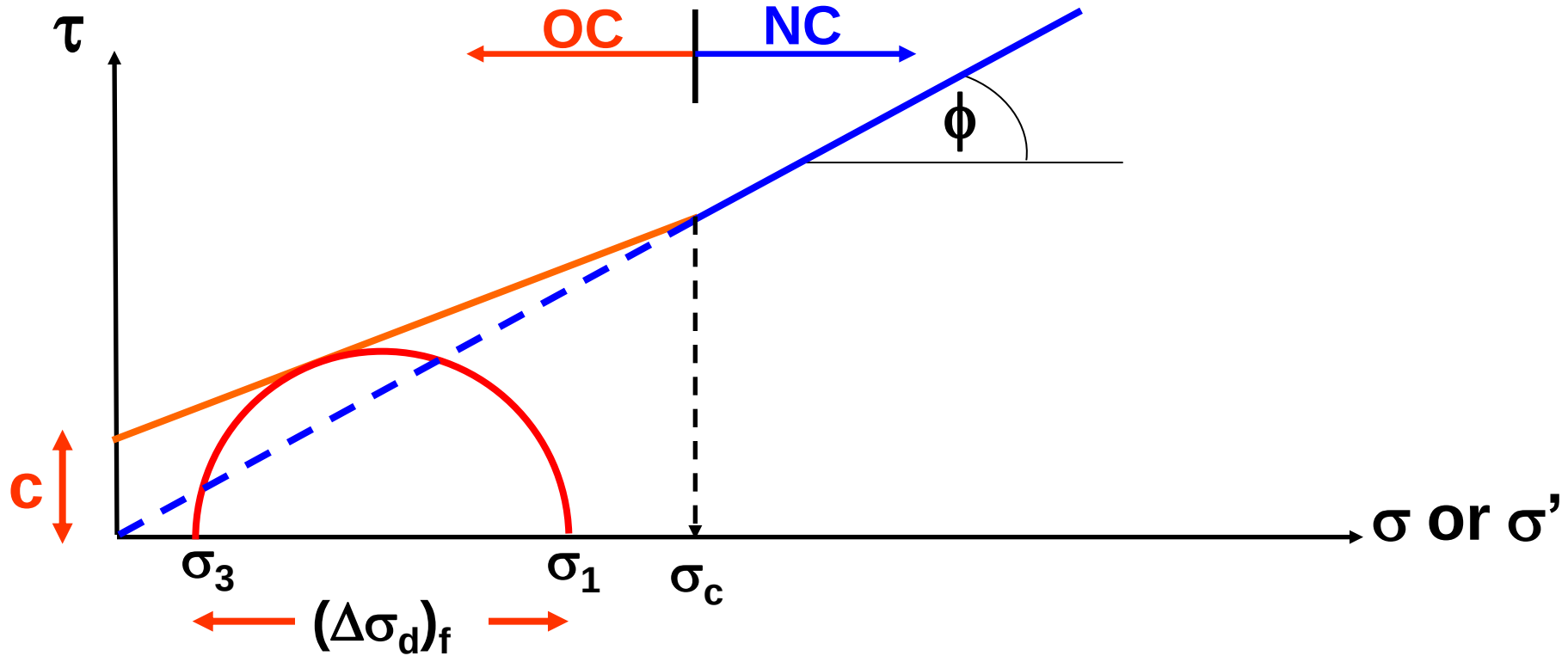
For sand and NC Clay, $c_d = 0$



Therefore, one CD test would be sufficient to determine ϕ_d of sand or NC clay

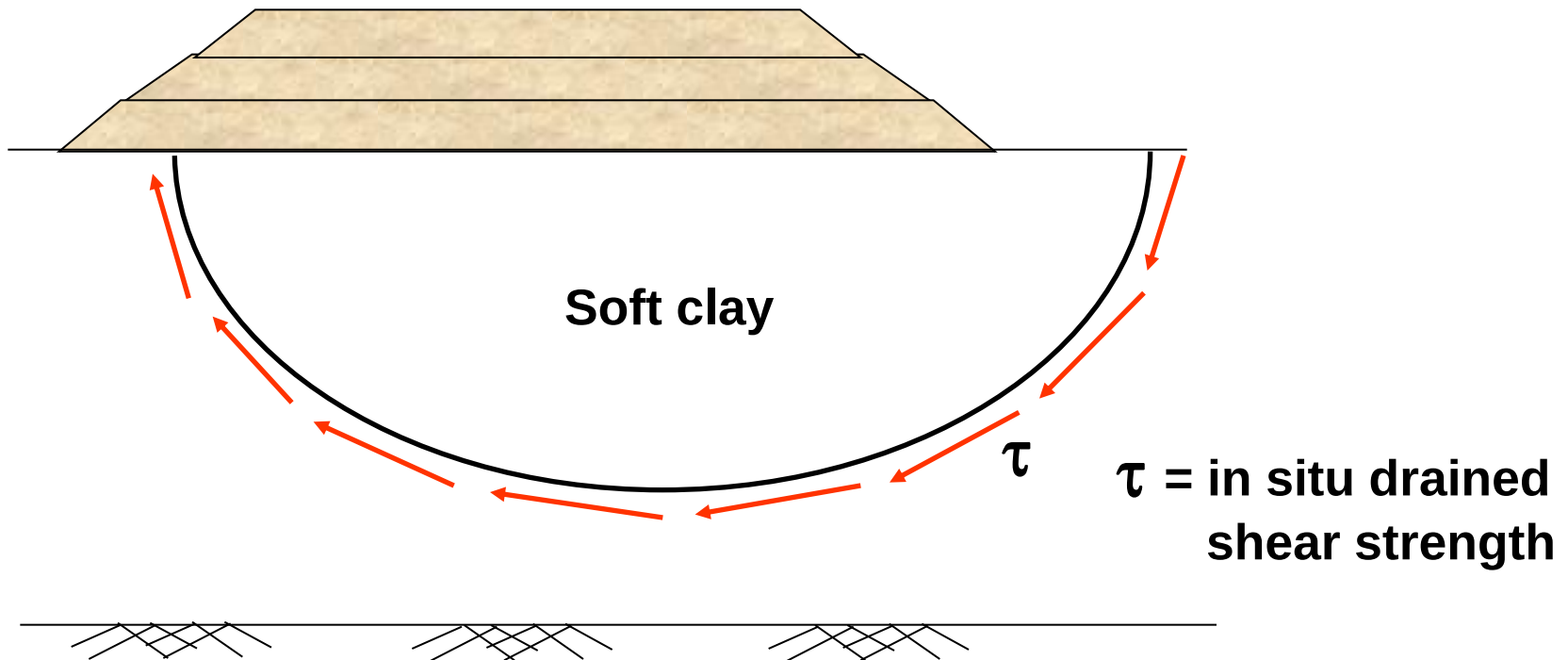
CD tests Failure envelopes

For OC Clay, $c_d \neq 0$



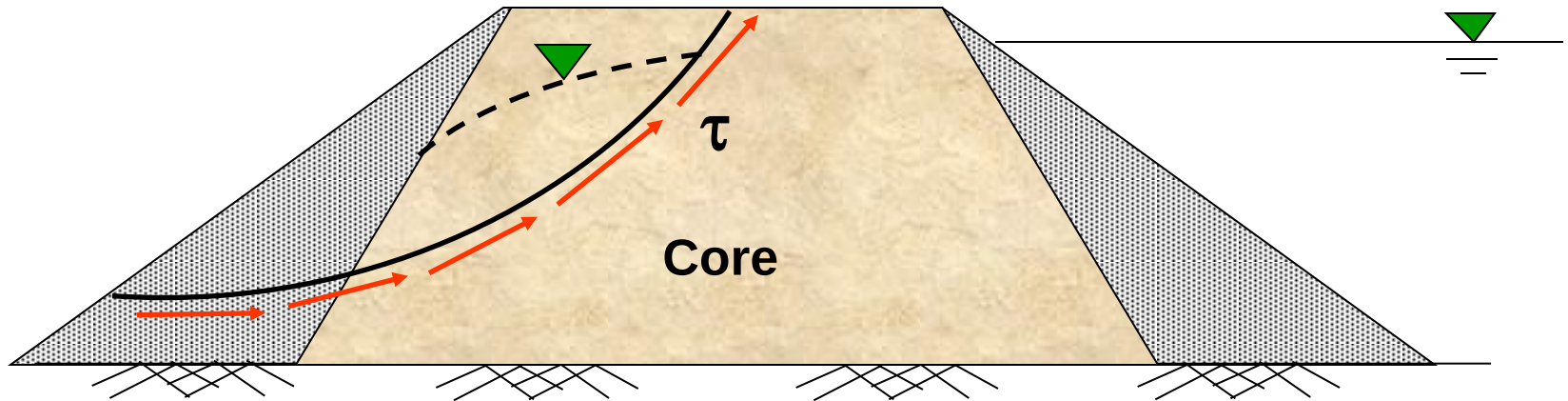
Some practical applications of CD analysis for clays

1. Embankment constructed very slowly, in layers over a soft clay deposit



Some practical applications of CD analysis for clays

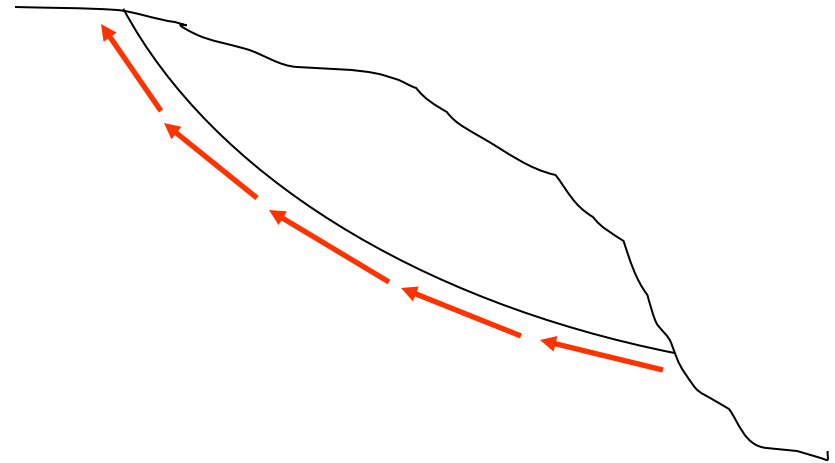
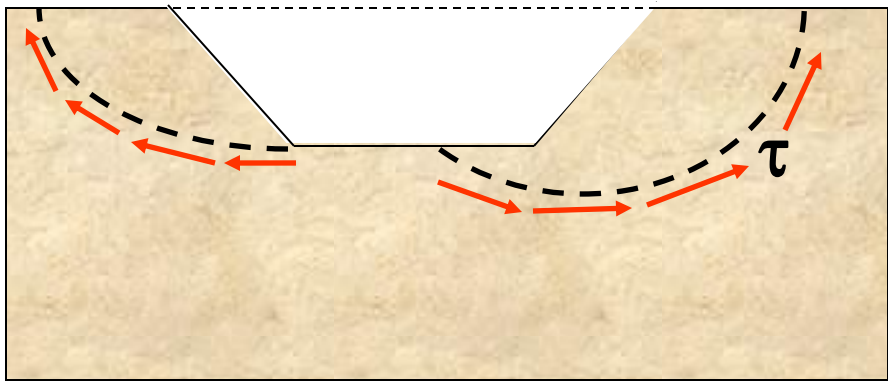
2. Earth dam with steady state seepage



τ = drained shear
strength of clay core

Some practical applications of CD analysis for clays

3. Excavation or natural slope in clay



τ = In situ drained shear strength

Note: CD test simulates the long term condition in the field. Thus, c_d and ϕ_d should be used to evaluate the long term behavior of soils

Consolidated- Undrained test (CU Test)

Total, σ

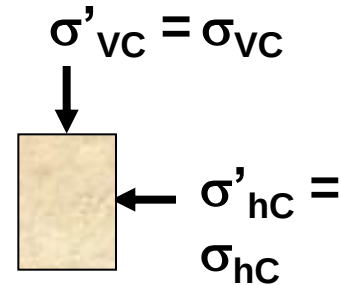
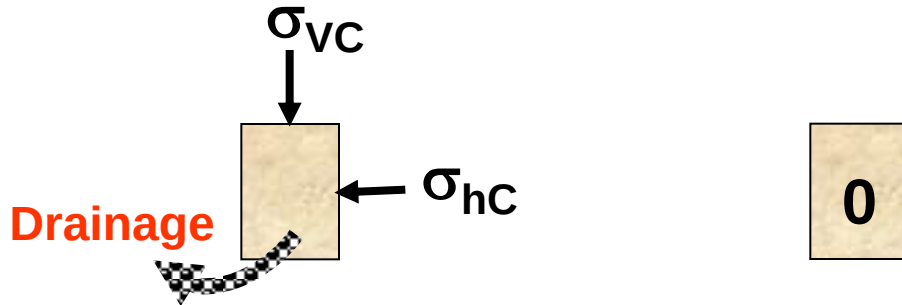
=

Neutral, u

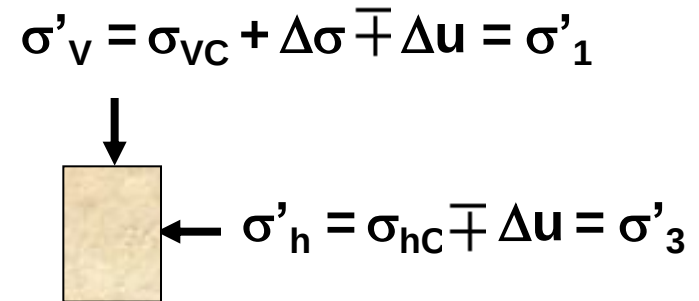
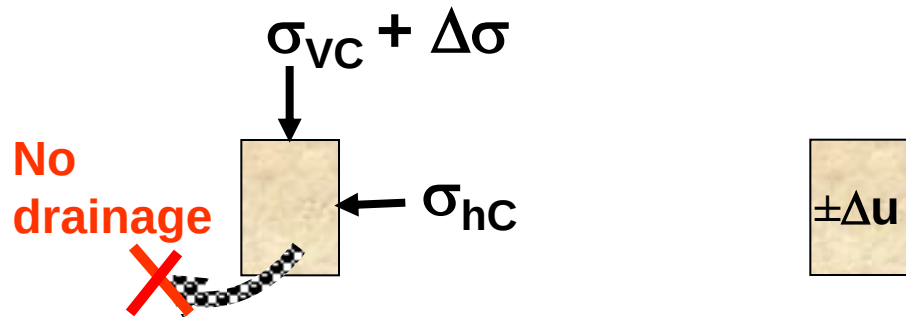
+

Effective, σ'

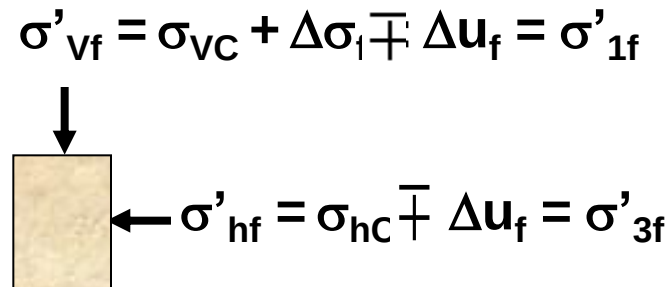
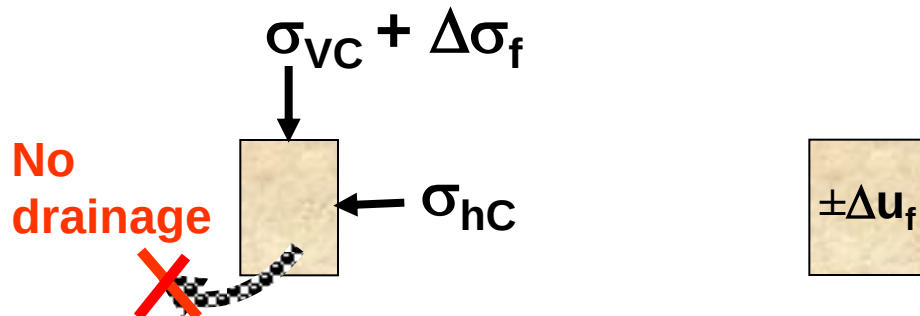
Step 1: At the end of consolidation



Step 2: During axial stress increase

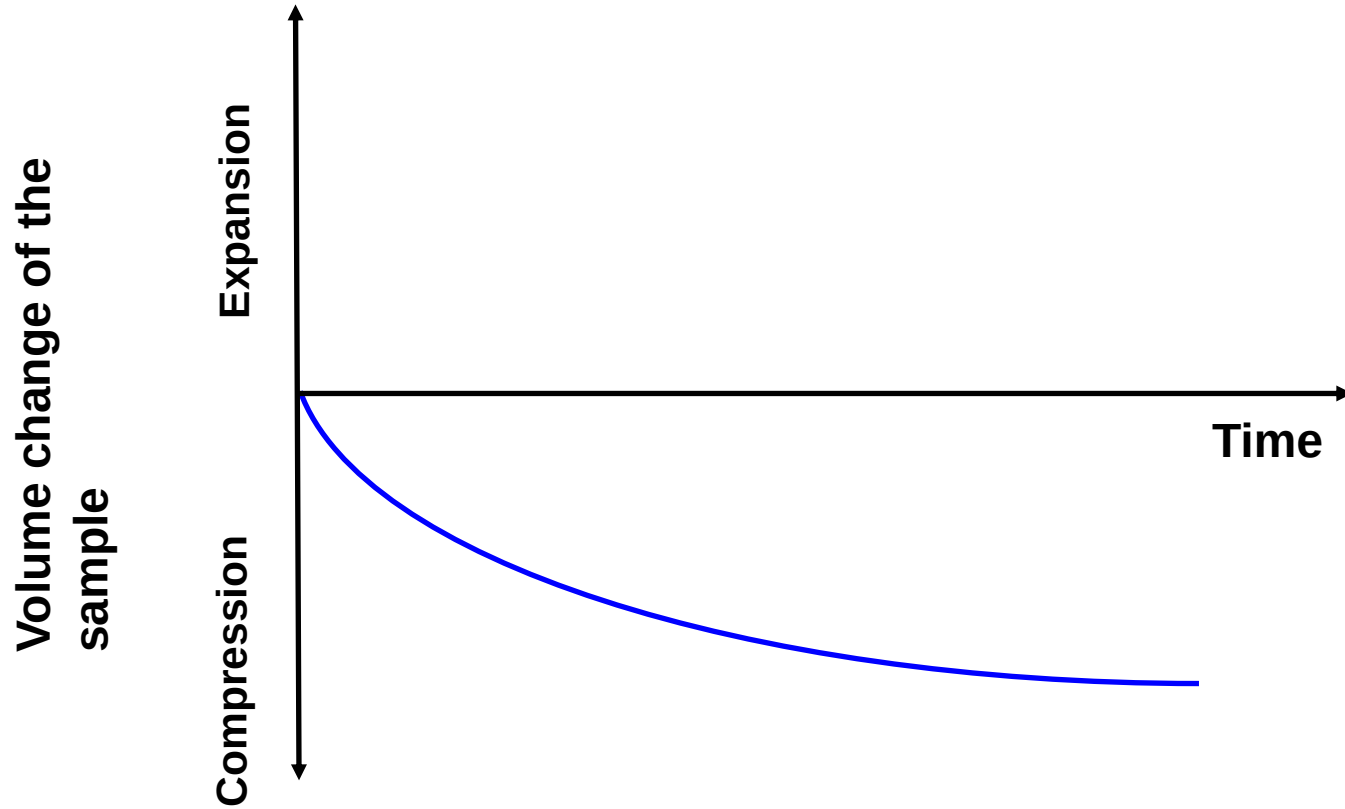


Step 3: At failure



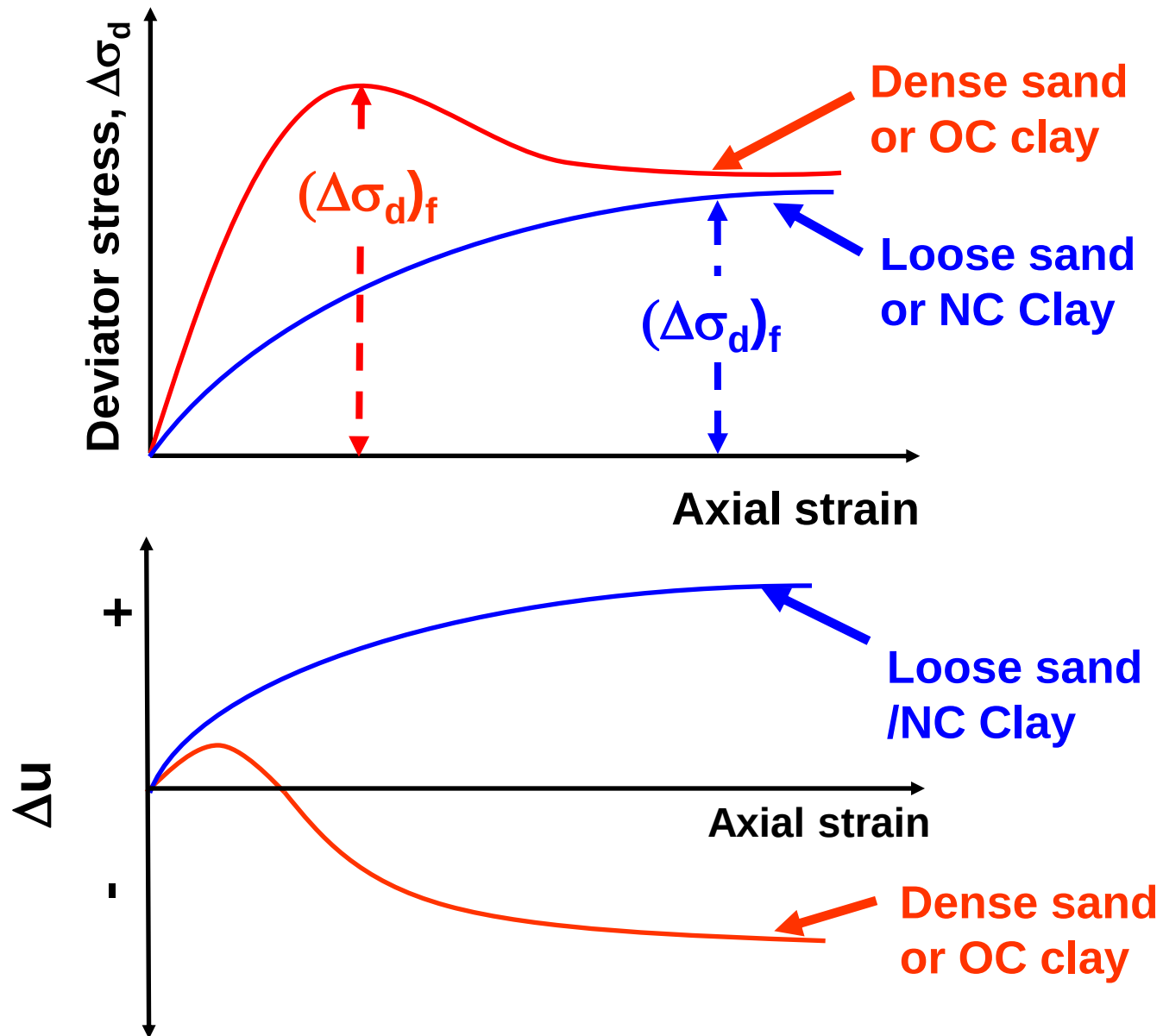
Consolidated- Undrained test (CU Test)

Volume change of sample during consolidation



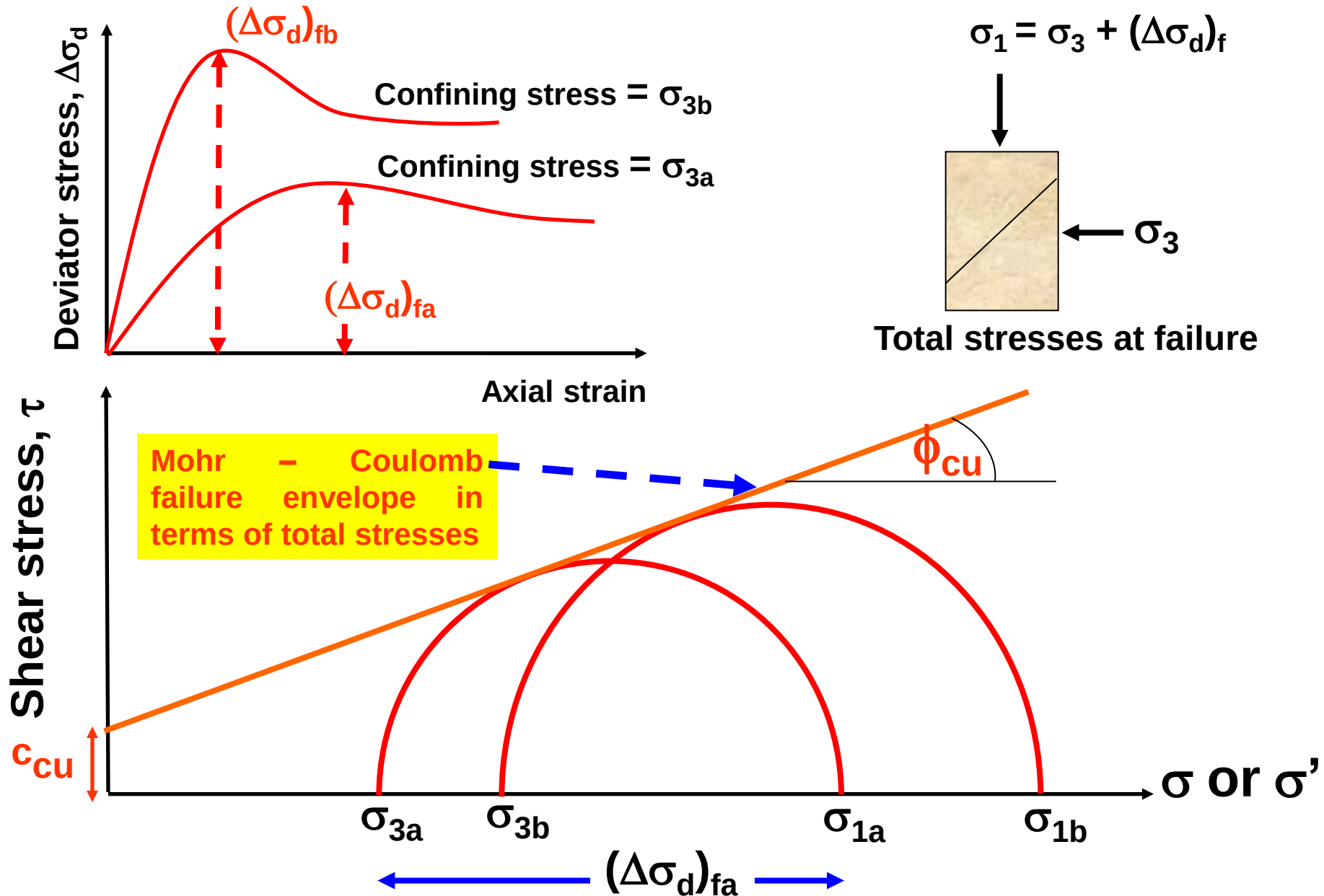
Consolidated- Undrained test (CU Test)

Stress-strain relationship during shearing

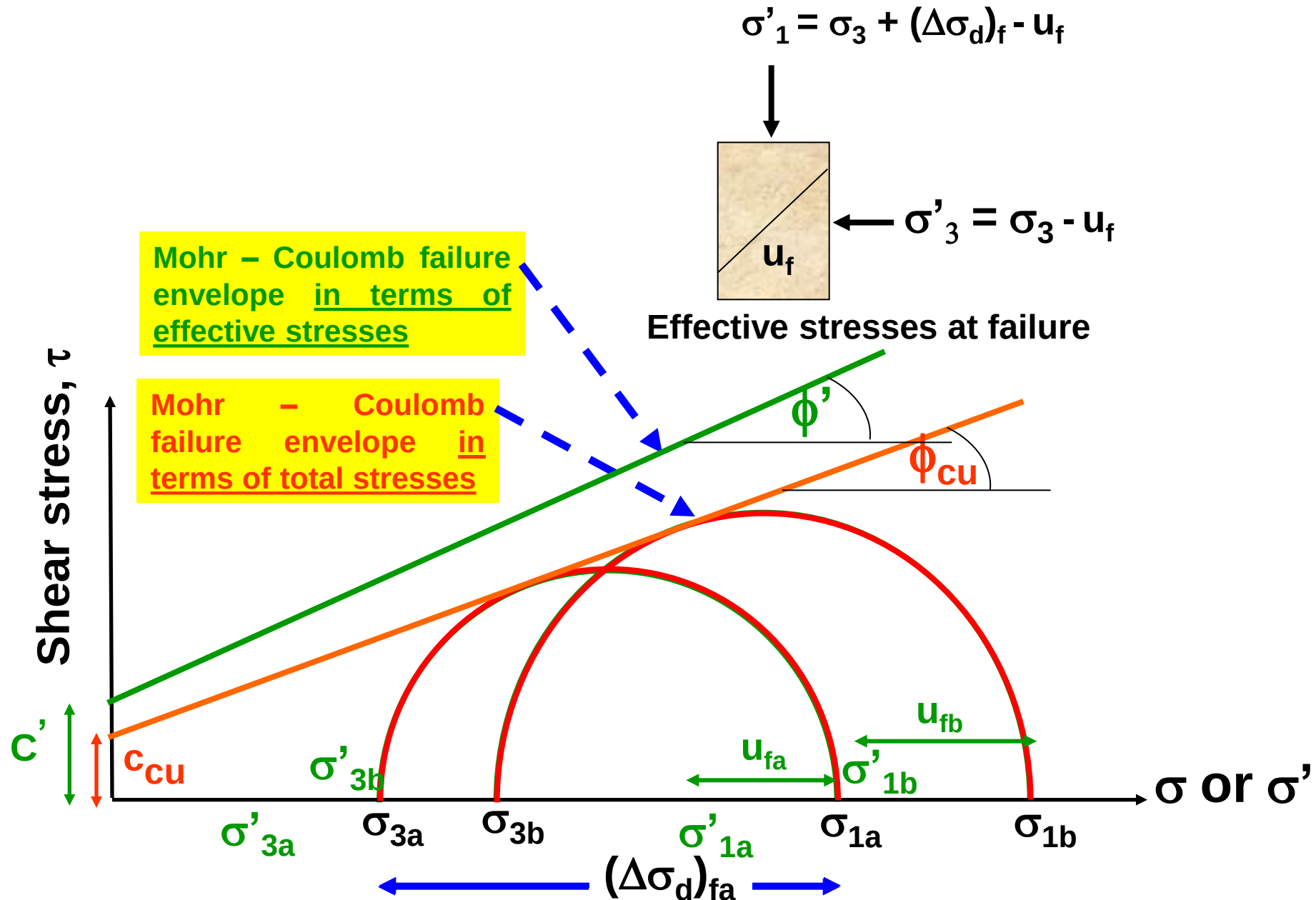


CU tests

How to determine strength parameters c and ϕ



CU tests How to determine strength parameters c and ϕ



CU tests

Strength parameters c and ϕ obtained from CD tests

Shear strength parameters in terms of total stresses are c_{cu} and ϕ_{cu}

Shear strength parameters in terms of effective stresses are c' and ϕ'

$c' = c_d$ and $\phi' = \phi_d$

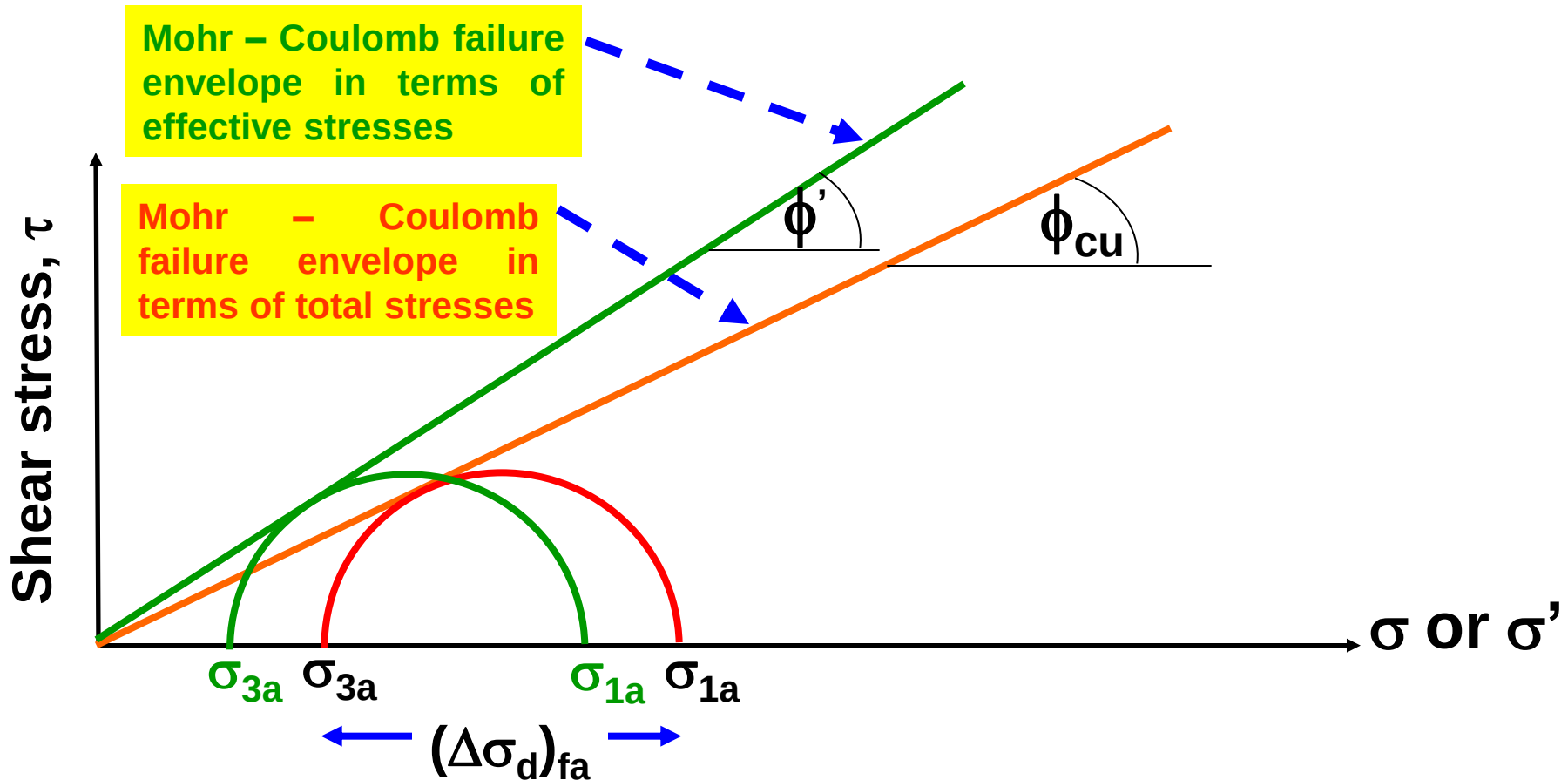


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A GENTLE REMINDER ...

CU tests Failure envelopes

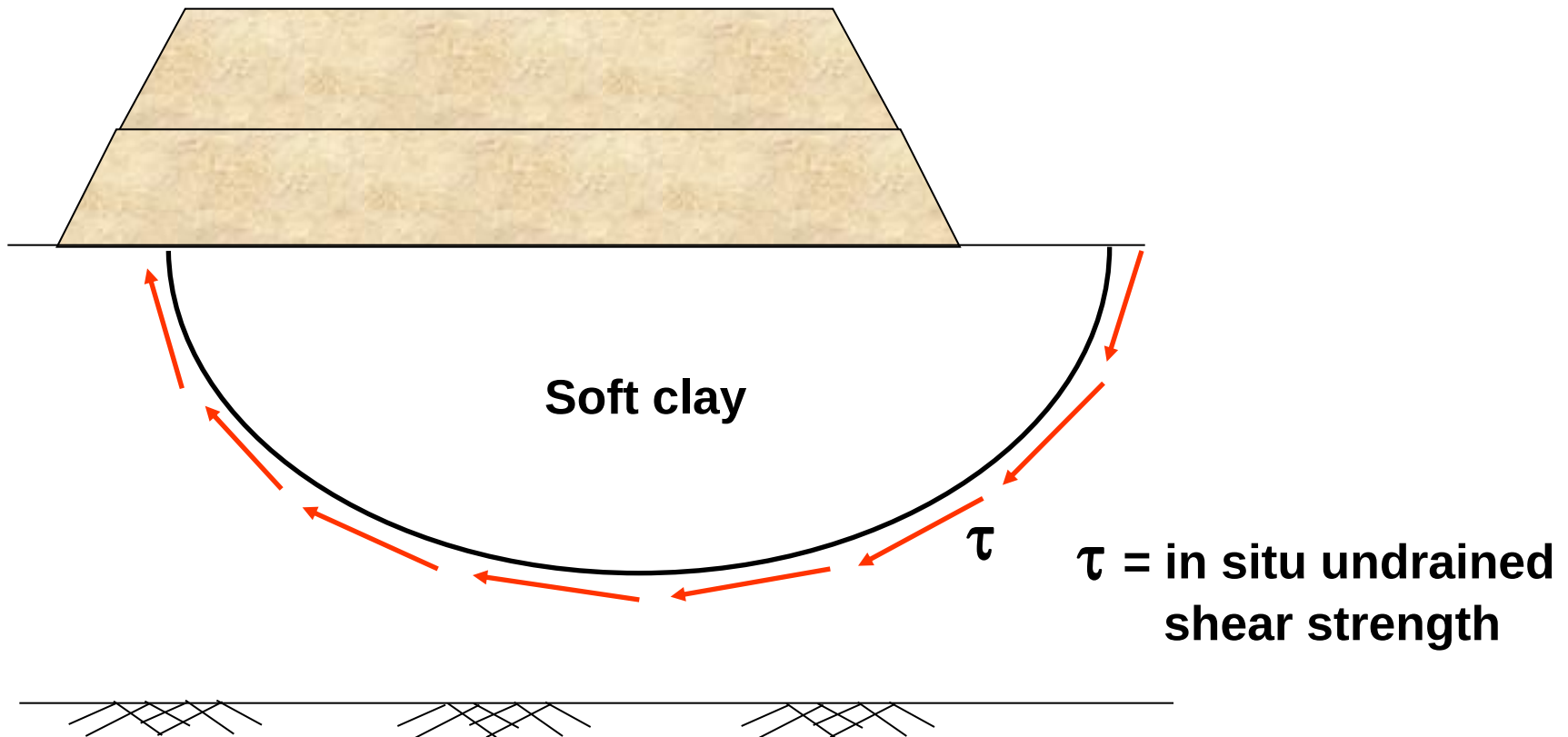
For sand and NC Clay, c_{cu} and $c' = 0$



Therefore, one CU test would be sufficient to determine ϕ_{cu} and $\phi' (= \phi_d)$ of sand or NC clay

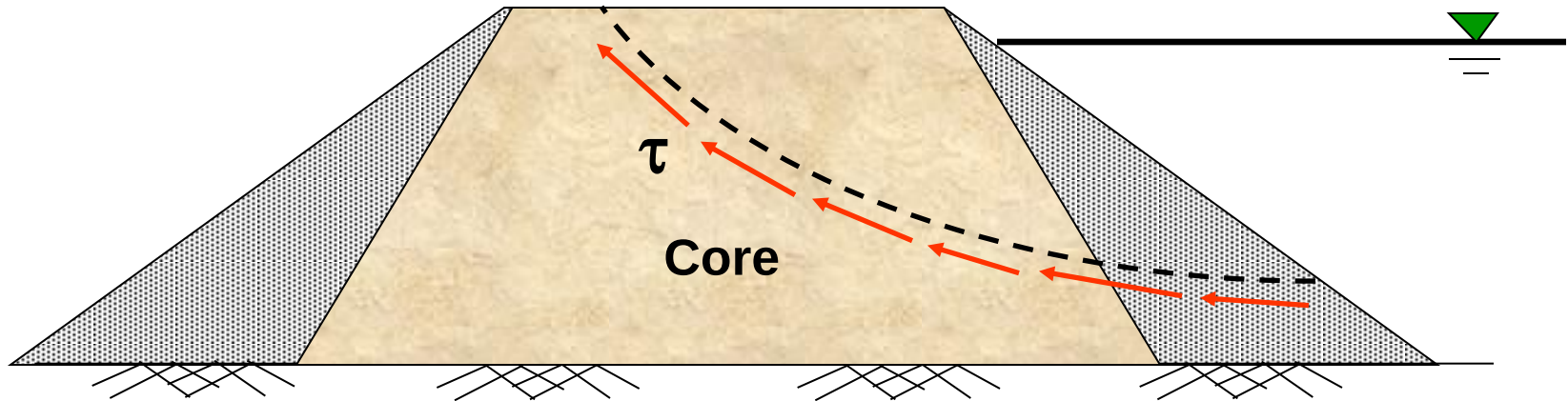
Some practical applications of CU analysis for clays

1. Embankment constructed rapidly over a soft clay deposit



Some practical applications of CU analysis for clays

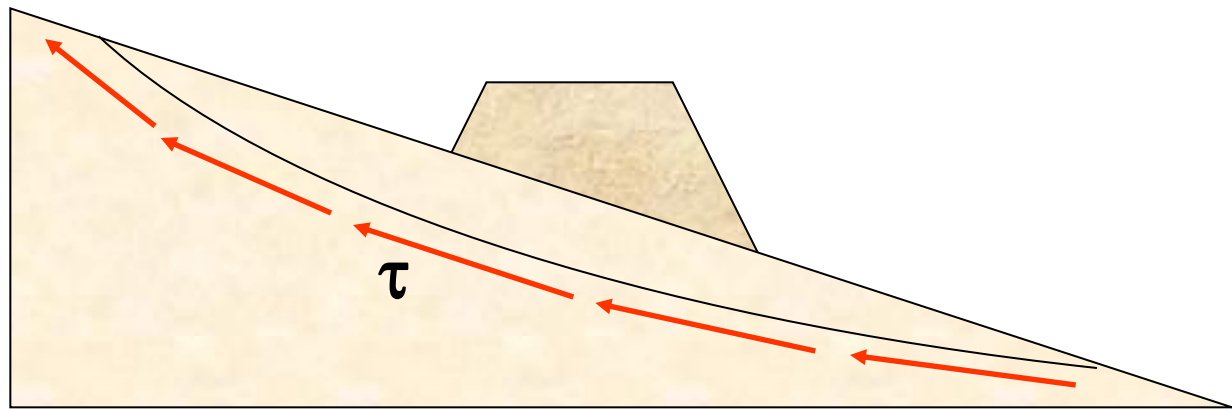
2. Rapid drawdown behind an earth dam



τ = Undrained shear strength of clay core

Some practical applications of CU analysis for clays

3. Rapid construction of an embankment on a natural slope



τ = In situ undrained shear strength

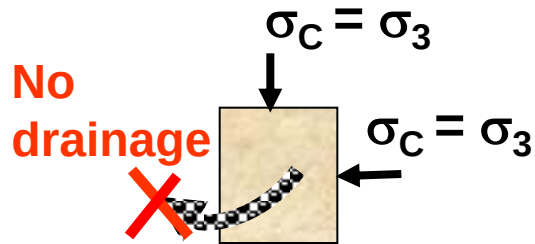
Note: Total stress parameters from CU test (c_{cu} and ϕ_{cu}) can be used for stability problems where,

Soil have become fully consolidated and are at equilibrium with the existing stress state; Then for some reason additional stresses are applied quickly with no drainage occurring

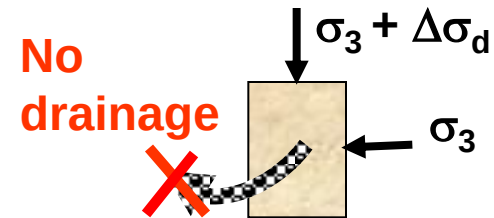
Unconsolidated- Undrained test (UU Test)

Data analysis

Initial specimen condition



Specimen condition during shearing



Initial volume of the sample = $A_0 \times H_0$

Volume of the sample during shearing = $A \times H$

Since the test is conducted under undrained condition,

$$A \times H = A_0 \times H_0$$

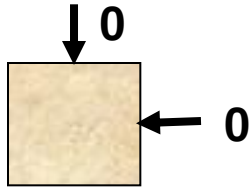
$$A \times (H_0 - \Delta H) = A_0 \times H_0$$

$$A \times (1 - \Delta H/H_0) = A_0$$

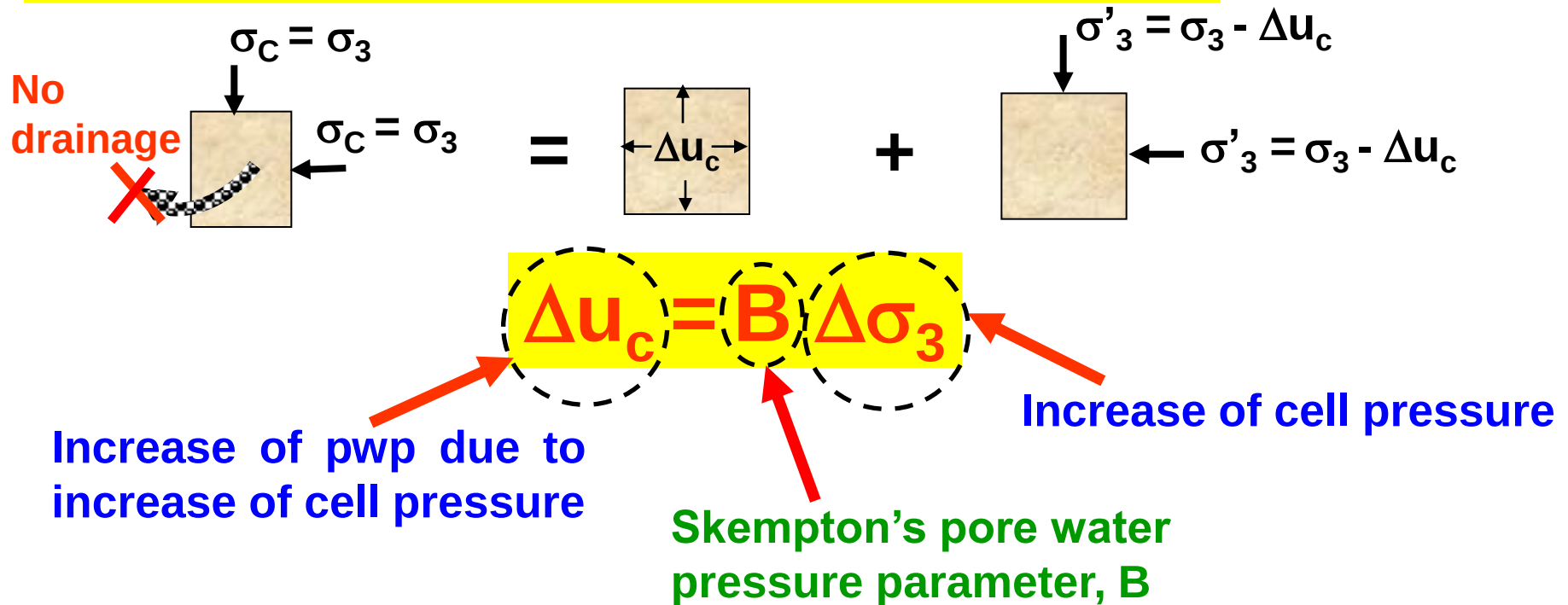
$$A = \frac{A_0}{1 - \varepsilon_z}$$

Unconsolidated- Undrained test (UU Test)

Step 1: Immediately after sampling



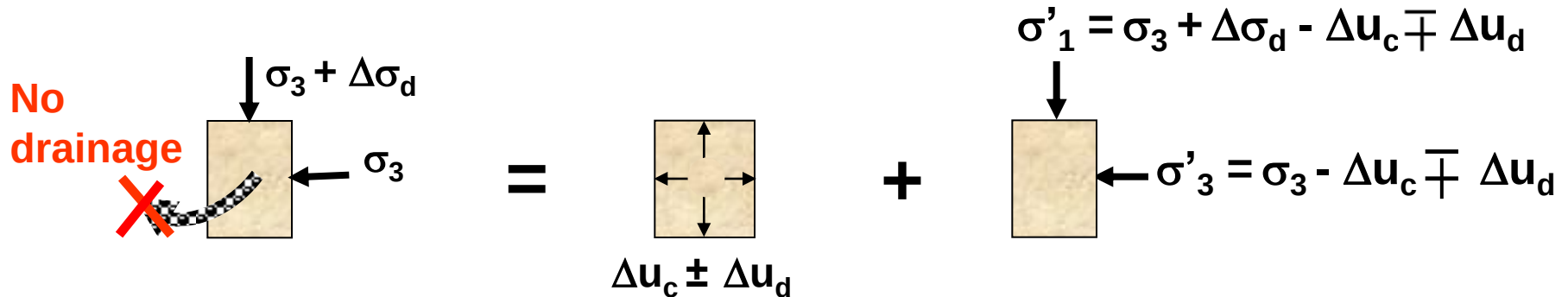
Step 2: After application of hydrostatic cell pressure



Note: If soil is fully saturated, then $B = 1$ (hence, $\Delta u_c = \Delta \sigma_3$)

Unconsolidated- Undrained test (UU Test)

Step 3: During application of axial load



$$\Delta u_d = AB \Delta\sigma_d$$

Increase of pwp due to increase of deviator stress

Increase of deviator stress

Skempton's pore water pressure parameter, A

Unconsolidated- Undrained test (UU Test)

Combining steps 2 and 3,


$$\Delta u_c = B \Delta \sigma_3$$


$$\Delta u_d = AB \Delta \sigma_d$$

Total pore water pressure increment at any stage, Δu

$$\Delta u = \Delta u_c + \Delta u_d$$

$$\Delta u = B [\Delta \sigma_3 + A \Delta \sigma_d]$$

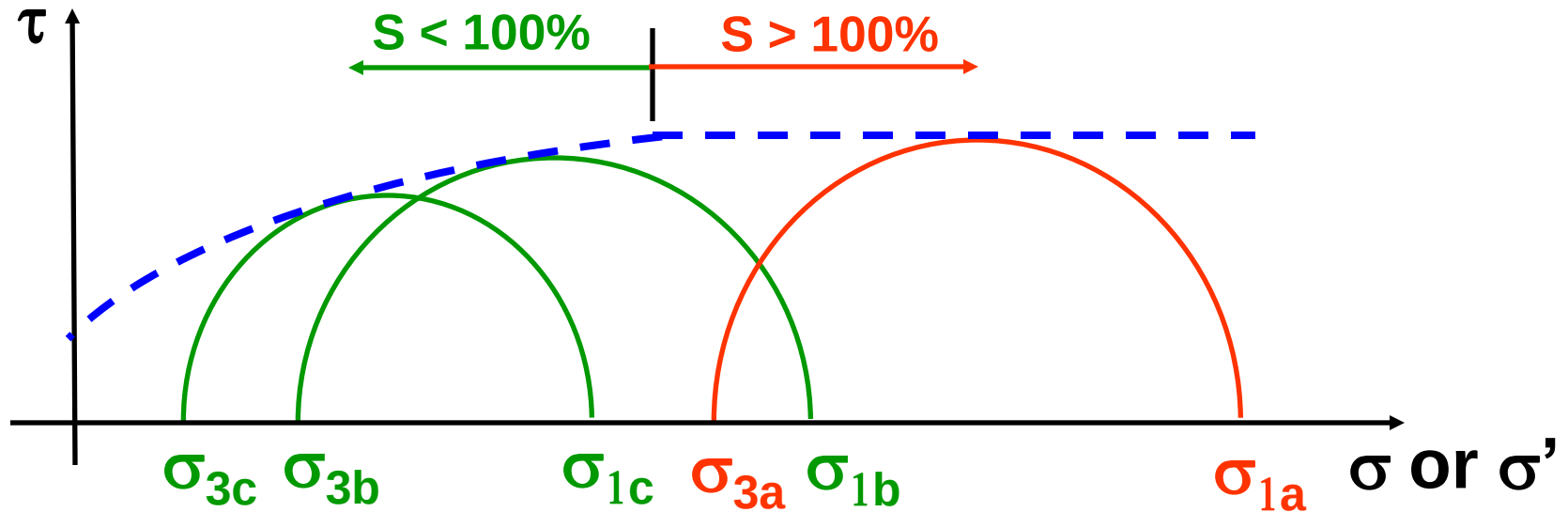
$$\Delta u = B [\Delta \sigma_3 + A(\Delta \sigma_1 - \Delta \sigma_3)]$$

Skempton's pore
water pressure
equation



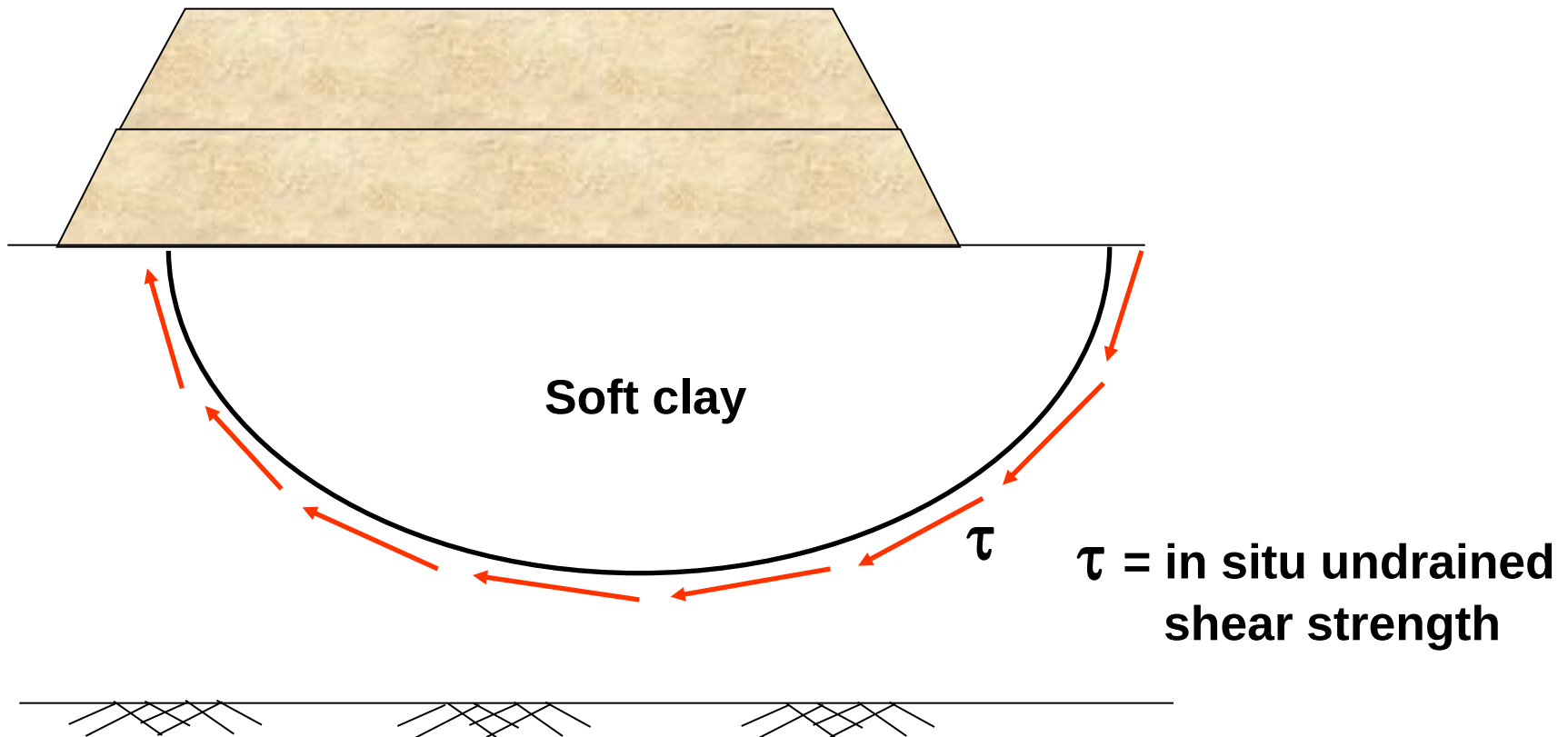
Unconsolidated- Undrained test (UU Test)

Effect of degree of saturation on failure envelope



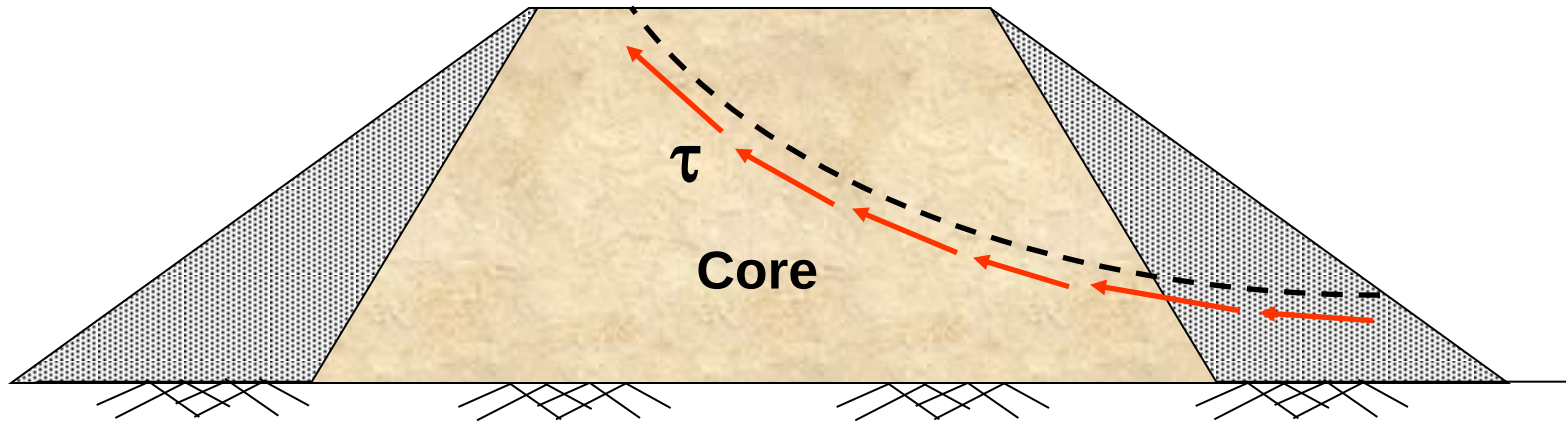
Some practical applications of UU analysis for clays

1. Embankment constructed rapidly over a soft clay deposit



Some practical applications of UU analysis for clays

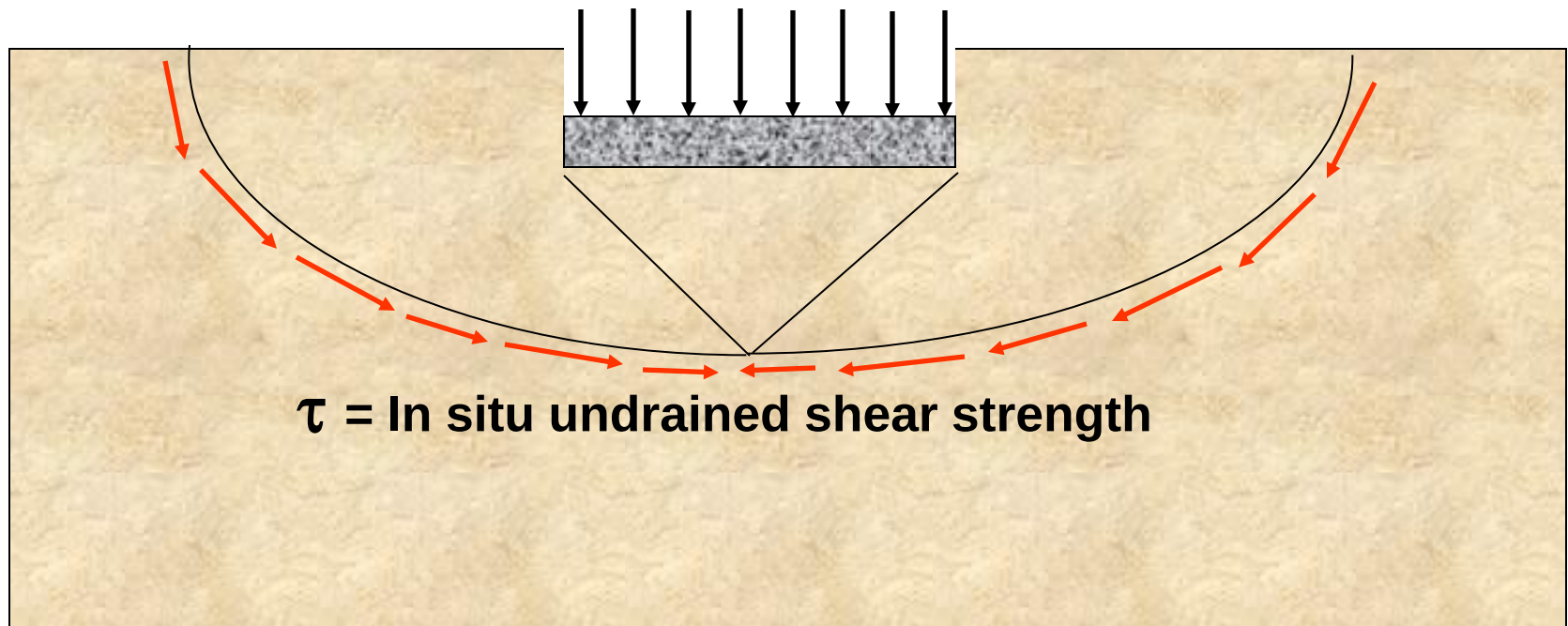
2. Large earth dam constructed rapidly with no change in water content of soft clay



τ = Undrained shear strength of clay core

Some practical applications of UU analysis for clays

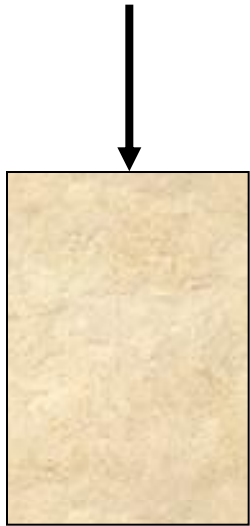
3. Footing placed rapidly on clay deposit



Note: UU test simulates the short term condition in the field. Thus, c_u can be used to analyze the short term behavior of soils

Unconfined Compression Test (UC Test)

$$\sigma_1 = \sigma_{vc} + \Delta\sigma$$

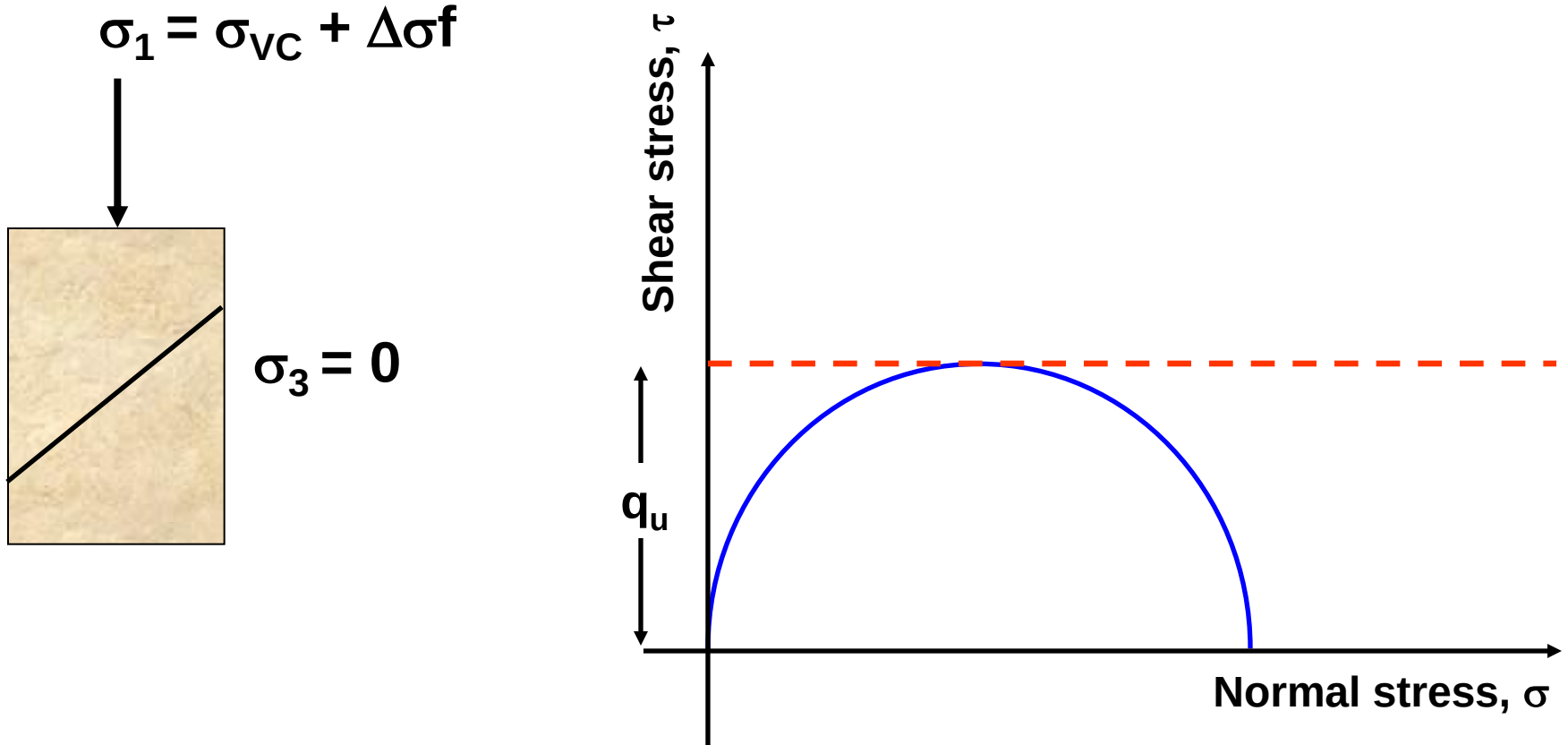


$$\sigma_3 = 0$$



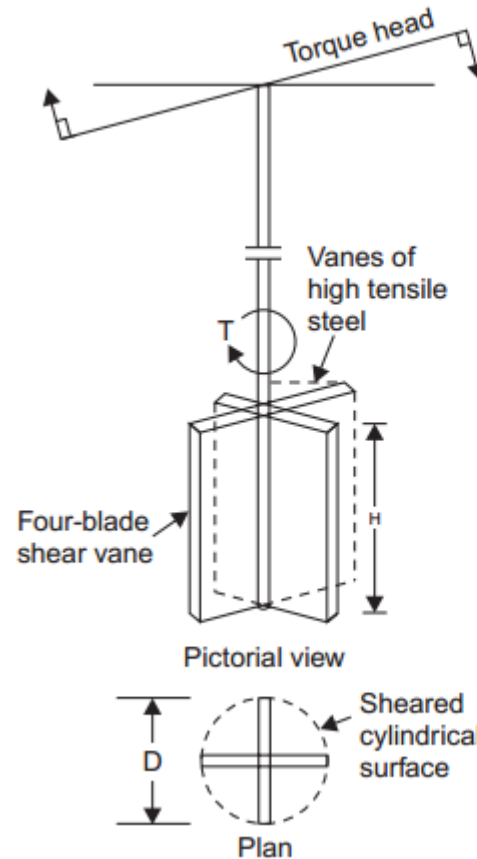
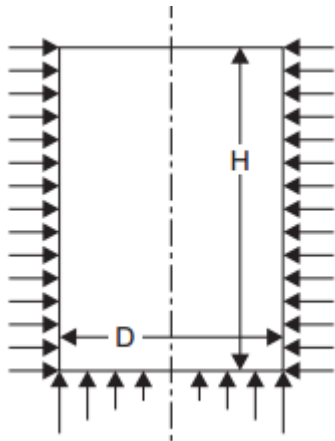
Confining pressure is zero in the UC test

Unconfined Compression Test (UC Test)



$$\tau_f = \sigma_1/2 = q_u/2 = c_u$$

Vane shear test



Laboratory vane:

$H = 20 \text{ mm}$

$D = 12 \text{ mm}$

$t = 0.5 \text{ to } 1 \text{ mm}$

Field vane:

$H = 10 \text{ to } 20 \text{ cm}$

$D = 5 \text{ to } 10 \text{ cm}$

$t = 2.5 \text{ cm}$

Derivation of Formula. In the derivation of the formula, it is assumed that the shear strength (s) of the soil is constant on the cylindrical sheared surface and at the top and bottom faces of the sheared cylinder. The torque applied (T) must be equal to the sum of the resisting torque at the sides (T_1) and that at the top and bottom (T_2). Thus,

$$T = T_1 + T_2 \quad \dots(a)$$

The resisting torque on the sides is equal to the resisting force developed on the cylindrical surface multiplied by the radial distance. Thus,

$$T_1 = (s\pi DH) \times D/2 \quad \dots(b)$$

The resisting torque T_2 due to the resisting forces at the top and bottom of the sheared cylinder can be determined by the integration of the torque developed on a circular ring of radius r and width dr [Fig. 13.30 (b)]. Thus,

$$T_2 = 2 \int_0^{D/2} [s (2\pi r) dr] r = 4\pi s \left[\frac{r^3}{3} \right]_0^{D/2}$$

or

$$T_2 = \pi s \frac{D^3}{6} \quad \dots(c)$$

From Eqs. (a), (b) and (c), $T = \pi s [D^2 H/2 + D^3/6]$

or

$$s = \frac{T}{\pi (D^2 H/2 + D^3/6)} \quad \dots(13.27)$$

For example, if $D = 1.2$ cm, and $H = 2.4$ cm, $s = 0.158 T$
 where T is in N-cm and s in N/cm^2 .

Eq. 13.27 is modified if the top of the vane is above the soil surface and the depth of the vane inside the sample is H_1 . In such a case.

$$s = \frac{T}{\pi (D^2 H_1/2 + D^3/12)} \quad \dots(13.28)$$

The shear strength of the soil under undrained conditions is equal to the apparent cohesion c_u .

THE END

Soil Exploration

Topics Covered

- Introduction
- Methods of investigation
- Methods of boring
- Soil samplers and sampling
- Location and number of pits and borings
- Different tests for soil exploration

Introduction

- Soil exploration is a part of site investigation.
- Site investigation, in general deals with determining in general, the suitability of the site for the proposed construction.

Site Investigation



Introduction (Cont'd)...

WHAT?

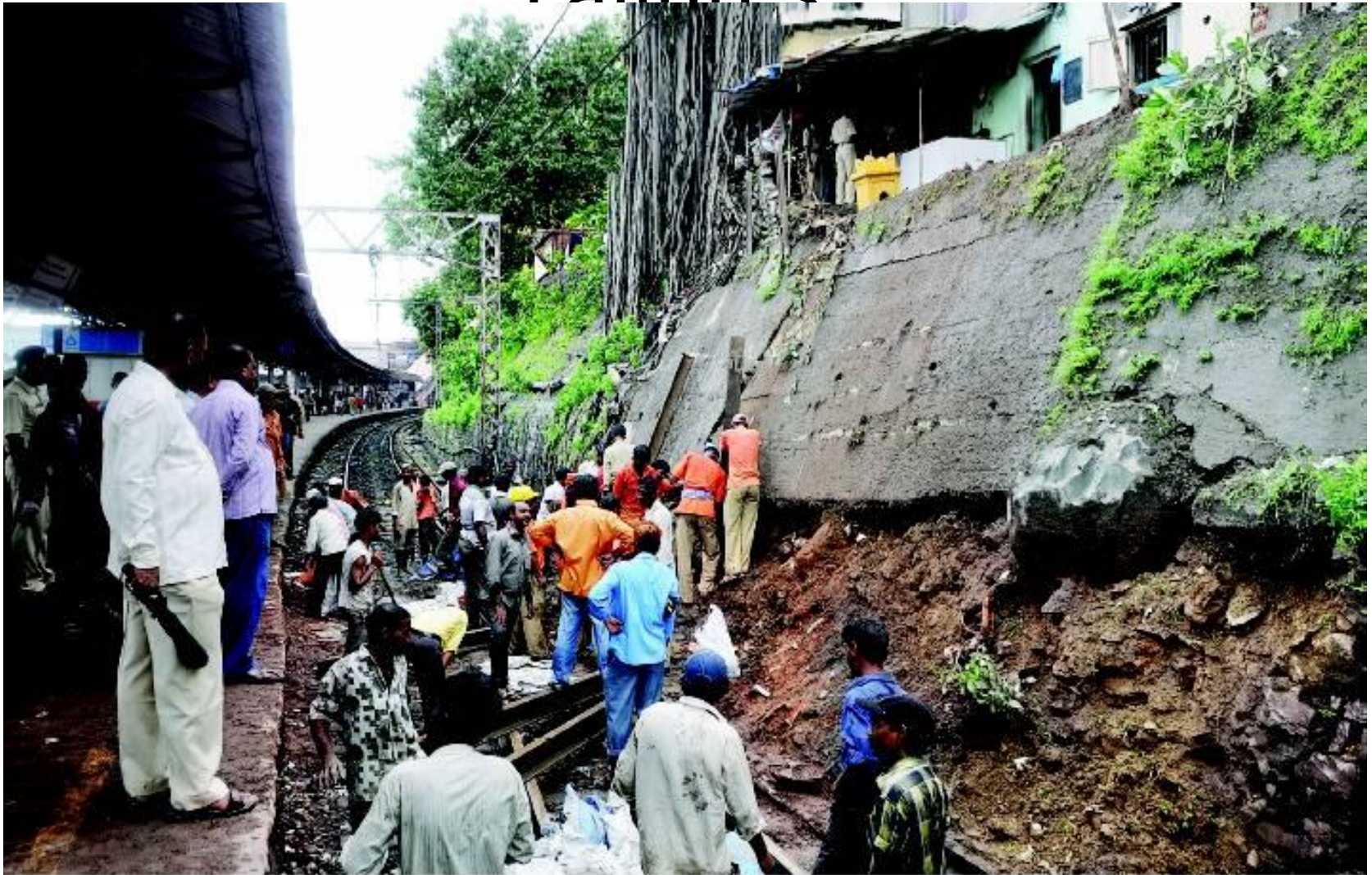
- Attempt at understanding the subsurface conditions such as:
 - Soil and rock profile
 - Geological features of the region
 - Position and variation of ground water table
 - Physical properties of soil and rock
 - Contamination, if any
 - General data of adjacent structures, hydrological data, topography, soil maps, seismicity, etc.
 - Engineering properties

Introduction (Cont'd)...

WHY?

- To determine the type of foundation required for the proposed project at the site, i.e. shallow foundation or deep foundation.
- To make recommendations regarding the safe bearing capacity or pile load capacity.
- Ultimately, it is the subsoil that provides the ultimate support for the structures.

Failures



Leaning Tower and Sinkhole



Introduction (Cont'd)...

HOW?

- The three important aspect are planning, execution and report writing.
- Planning
 - To minimize cost of explorations and yet give reliable data.
 - Decide on quantity and quality depending on type, size and importance of project and whether investigation is preliminary or detailed.

Introduction (Cont'd)...

- Execution:
 - Collection of disturbed and/or undisturbed samples of subsurface strata from field.
 - Conducting in-situ tests of subsurface material and obtaining properties directly or indirectly.
 - Study of ground water conditions and collection of sample for chemical analysis.
 - Geophysical exploration, if necessary.
 - Laboratory testing on samples



Introduction (Cont'd)...

- Report writing:
 - Description of site conditions – topographic features, hydraulic conditions, existing structures, etc. supplemented by plans/drawings.
 - Description of nature, type and importance of proposed construction
 - Description of field and lab tests carried out.
 - Analysis and discussion of data collected
 - Preparation of charts, tables, graphs, etc.
 - Calculations performed
 - Recommendations

Introduction (Cont'd)...

A complete site investigation will consist of:

- Preliminary work

- Collecting general information and already existing data such as study of geologic , seismic maps, etc. at or near site.
- Study site history – if previously used as quarry, agricultural land, industrial unit, etc.

- Site Reconnaissance: Actual site inspection.

- To judge general suitability
- Decide exploration techniques

Introduction (Cont'd)...

- Exploration
 - Preliminary Investigations: Exploratory borings or shallow test pits, representative sampling, geophysical investigations, etc
 - Detailed Investigations: Deep boreholes, extensive sampling, in-situ testing, lab testing, etc.
 - Depth and spacing: In general, depth of investigation should be such that any/all strata that are likely to experience settlement or failure due to loading. Spacing depends upon degree of variation of surface topography and subsurface strata in horizontal direction.

Soil Exploration

Methods of Investigation

- Test pits:
 - Permits visual inspection of subsurface conditions in natural state.
 - Max. depth limited to 18 - 20 feet.
 - Especially useful for gravelly soil where boreholes may be difficult.
 - Sampling/testing done on exposed surfaces.



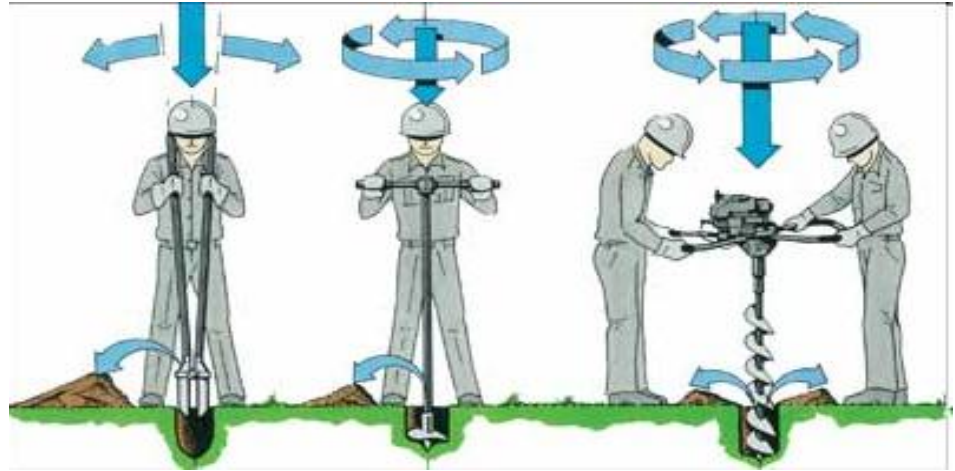
Methods of Boring

- Auger Boring
- Wash Boring
- Percussion Boring
- Rotatory Boring

Auger Borings:

- Simplest method of exploration and sampling.
- Power driven or hand operated.
- Max. depth 10 m
- Suitable in all soils above GWT but only in cohesive soil below GWT
- Hollow stem augers used for sampling or conducting Standard Penetration Tests.

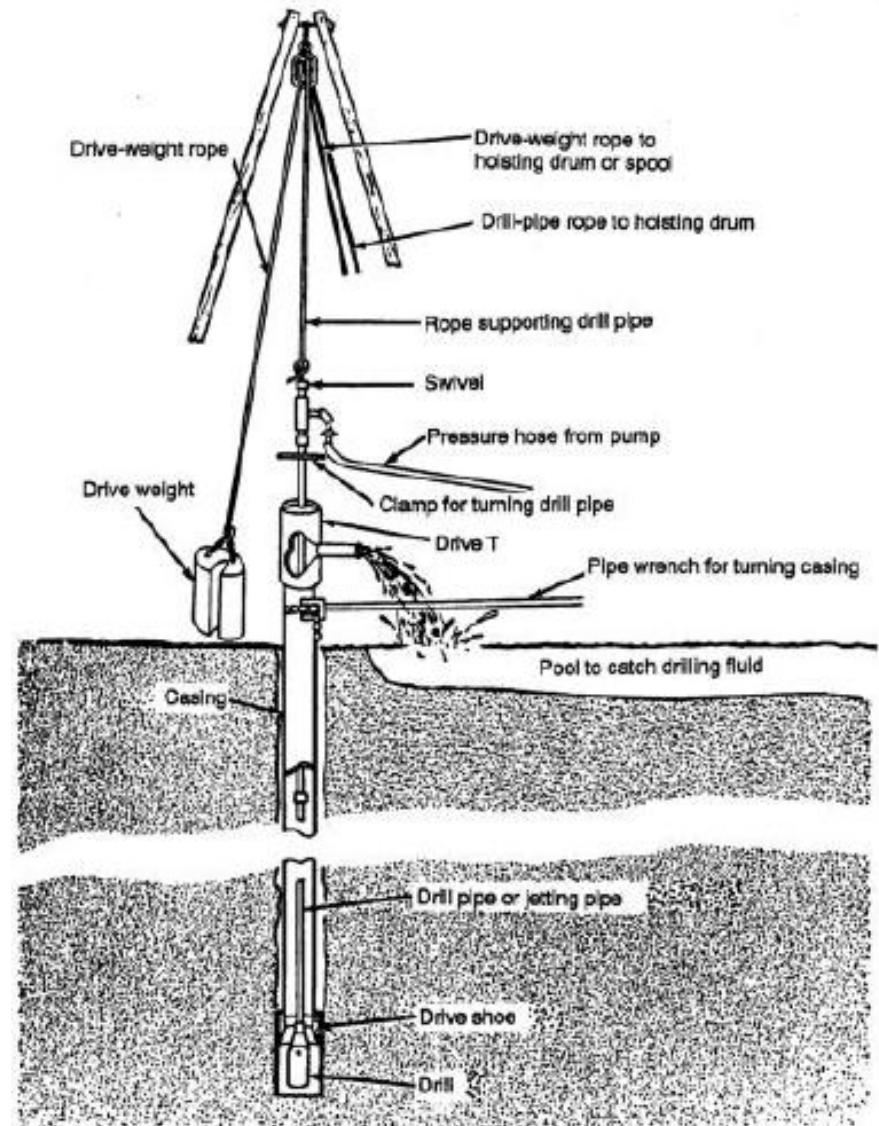
Hand operated augers



Power driven augers

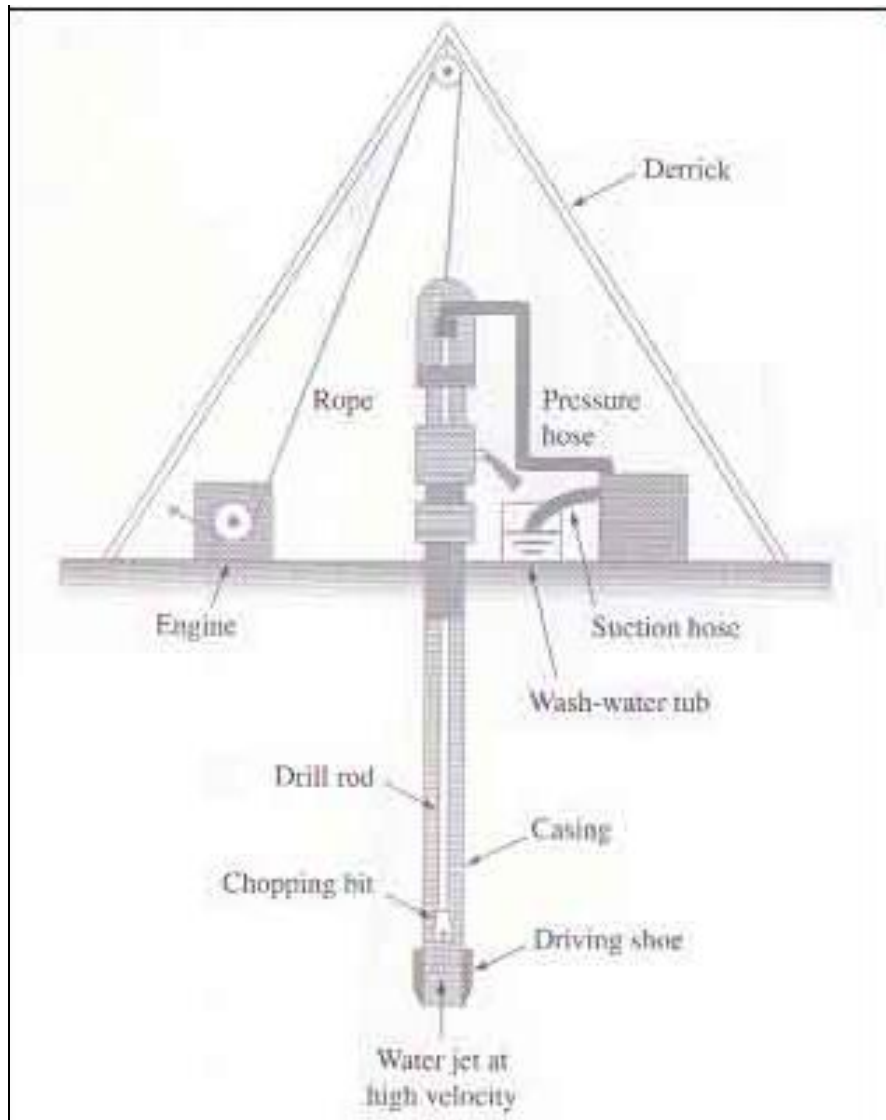
Shell and Auger Method

- Consist of sand bailer which have heavy weight with cutting edge.
- Bailer raised and let in fall in bore hole.



Wash Boring:

- A casing is driven with a drop hammer. A hollow drill rod with chopping bit is inserted inside the casing.
- Soil is loosened and removed from the borehole using water or a drilling mud jetted under pressure.
- The water is jetted in the hole through the bottom of a wash pipe and leaves the hole along with the loose soil, from the annular space between the hole and wash pipe.
- The water reaches the ground level where the soil in suspension is allowed to settle and mud is re-circulated.



Schematic for wash boring



Rotatory drilling

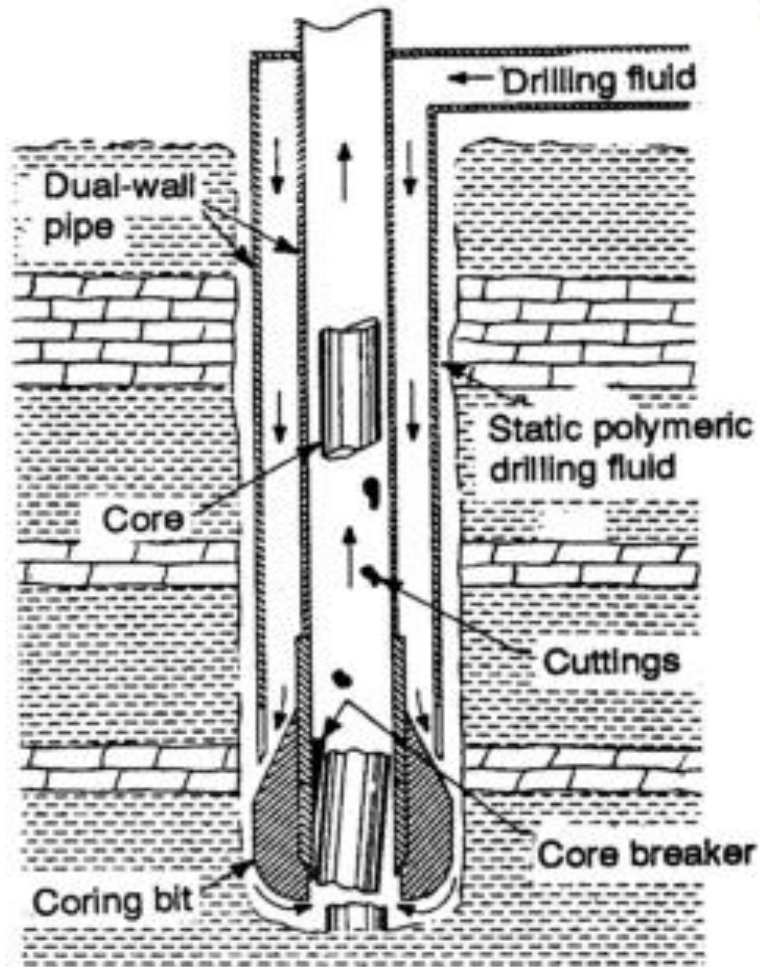
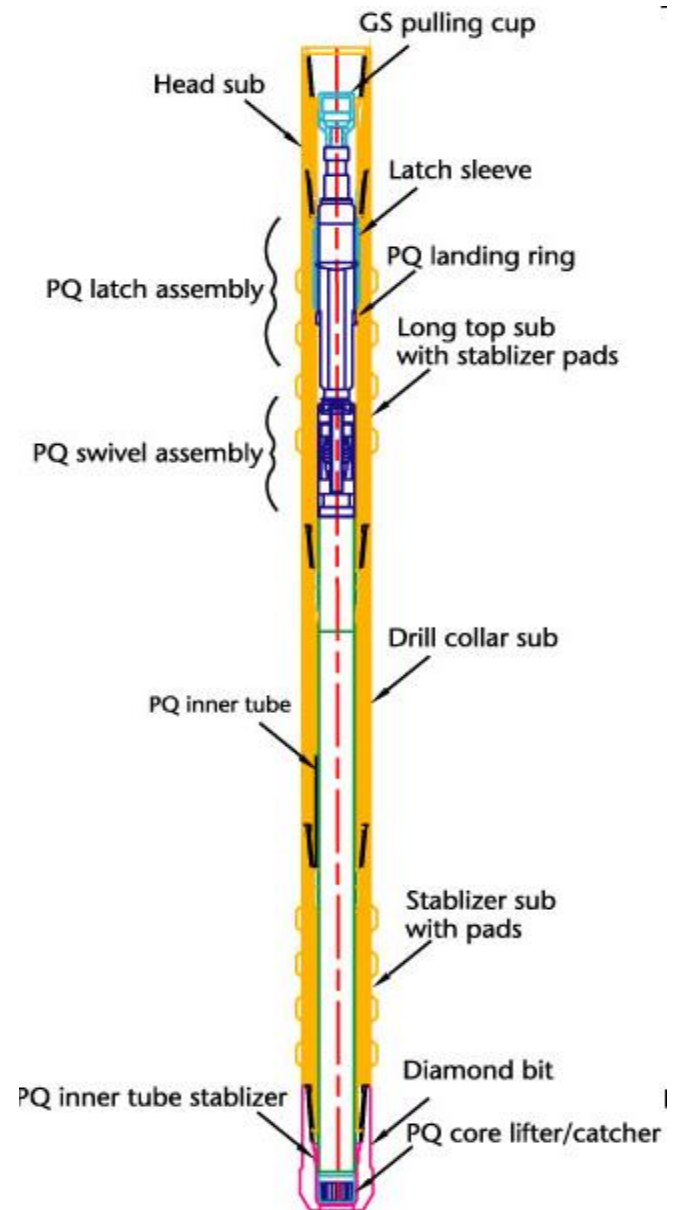


Figure 11-1. Core-drilling equipment





Diamond Drill Bit

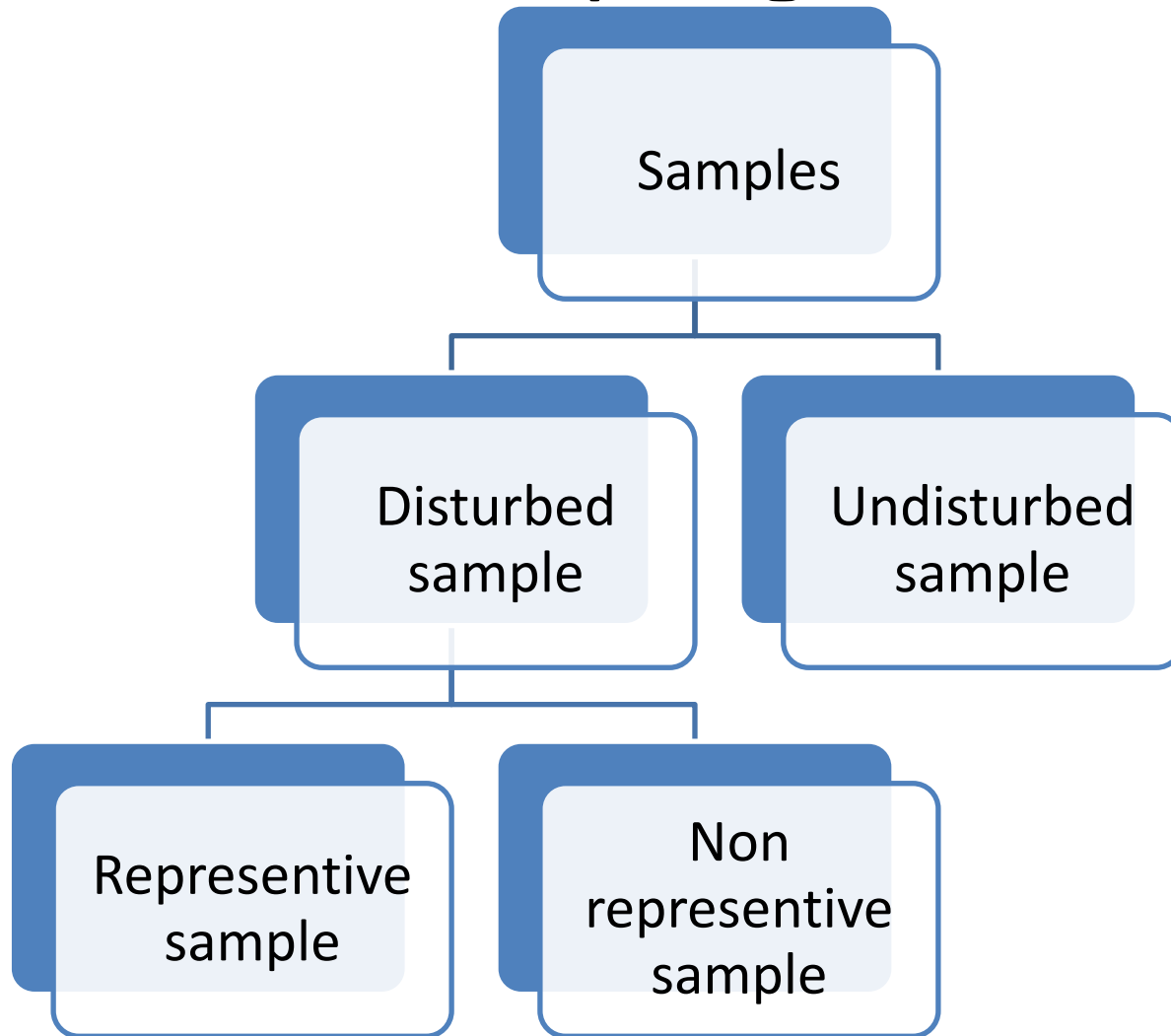
Tricone drill bit —————>



Thank You

Soil exploration Sampling

Sampling



Cont...

- Extent of disturbance depends upon:
 - Cutting edge
 - Inside wall friction
 - Non return valve

Cont

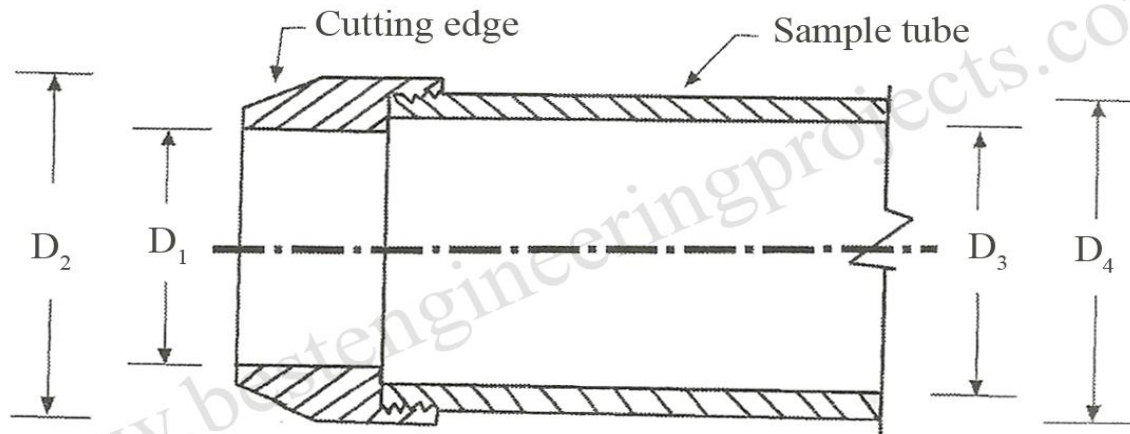


Fig 1 A Soil Sampler

- Inside clearance (C_i) = $(D_3 - D_1) / D_1$
- Out side clearance (C_o) = $(D_2 - D_4) / D_4$
- Area ratio (Ar) = $\frac{D_2^2 - D_1^2}{D_1^2}$

Cont...

- Inside clearance = 1-3 % to reduce friction between soil and sampler
- Outside clearance is to reduce the friction when sampler is driven for withdrawn
- Area ratio should be kept as low as possible, less than 20% for stiff formation and 10 % for soft and sensitive clays.

Cont...

- Recovery ratio = L_r = Recovery length of sample/ penetration length of sampler
 - $L_r = 1$ ----- good recovery
 - $L_r < 1$ ----- soil is compressed
 - $L_r > 1$ ----- soil has swelled

Sample required in laboratory

Type of test	Type of sample required
Natural moisture content	Representative
Density	Undisturbed
Specific gravity	Representative or undisturbed
Atterberg limits	Representative or undisturbed
Hydraulic conductive	Undisturbed
Consolidation parameters	Undisturbed
Shear strength	undisturbed

Depth of exploration

- Significant depth of exploration → depth likely to undergo settlement
- It depends upon type of structure, size, shape of soil, loaded area, intensity of loading, soil profile, soil properties
- Boring should be carried out to such a depth that the net increase in the soil stress due to the load structure is less than 10 % of the average contact pressure.

IS 1892-1979 recommendations

Type of foundation	Depth of exploration
Isolated spread footing or raft	1.5 x width
Adjacent footing with clear $A < 2 B$ (Width)	1.5 x L
$A \geq 4B$ $A = 2B - 4B$ $A \leq 2B$	1.5B 3B 4.5B
Pile and well foundation	1.5 times the width of structure from the bearing level
Road cuts and fill	Bottom width of the cut or 2 m below the GL or equal to height of fill which ever is greater

Lateral extent of exploration

- The lateral extent of exploration and spacing of bore holes depends mainly on the variation of the strata.
- For small and less important buildings, even bore hole or a trial pit in the centre may suffice.
- For compact buildings, covering an area about 0.4 hectares, there should be at least 5 bores, one at the centre and four near the corners.
- The spacing of bore holes is generally kept 10-30 m, depending upon the variation in the subsurface conditions and loading.
- For highways subsurface explorations are usually carried out along the proposed centre line at distance of 150 m to 300 m.

Tests for determination of properties of soil

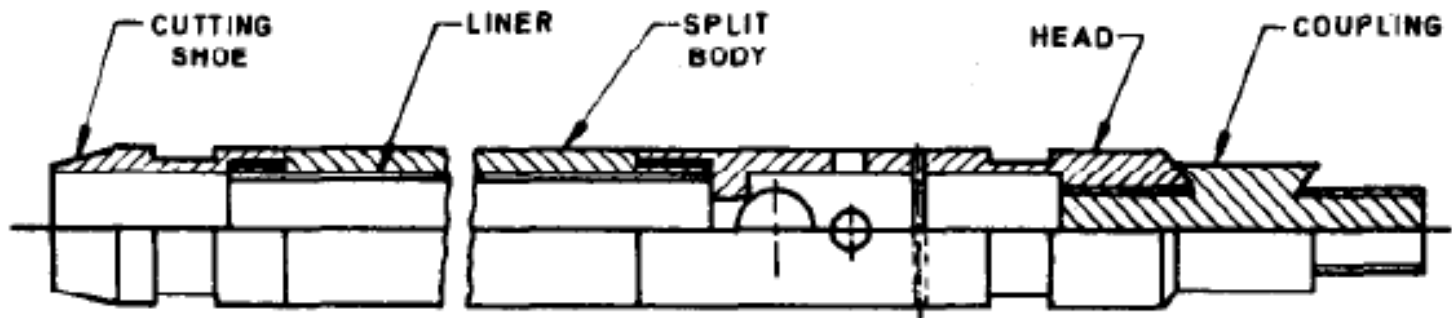
- **Field test:**
 - Soil is undisturbed.
 - Useful for cohesionless soil
 1. Penetration test
 2. Plate load test
 3. Vane shear test
 4. Pressure meter
 5. Geophysical method
- Laboratory test
 - Disturbed sample
 - Useful for cohesive soil
 1. UCS
 2. Triaxial test
 3. Consolidation test

Soil Exploration

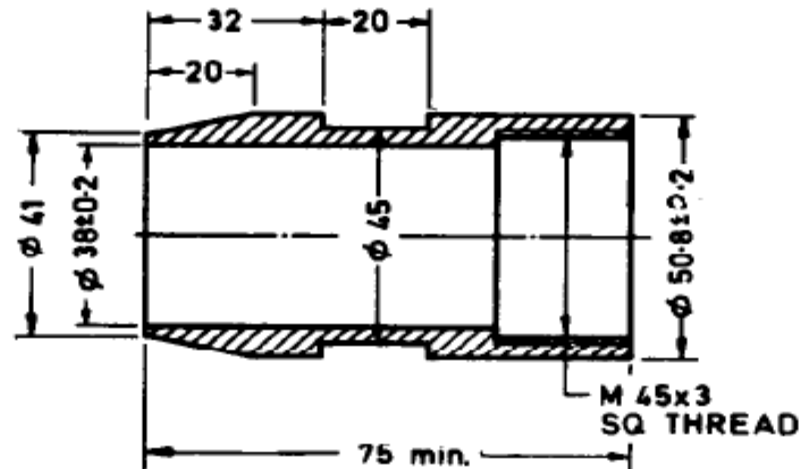
Standard Penetration Test

Standard penetration test (SPT)

- Useful for cohesionless soil
- Sample can be obtained from desired depth
- Consists of assembly of split spoon sampler

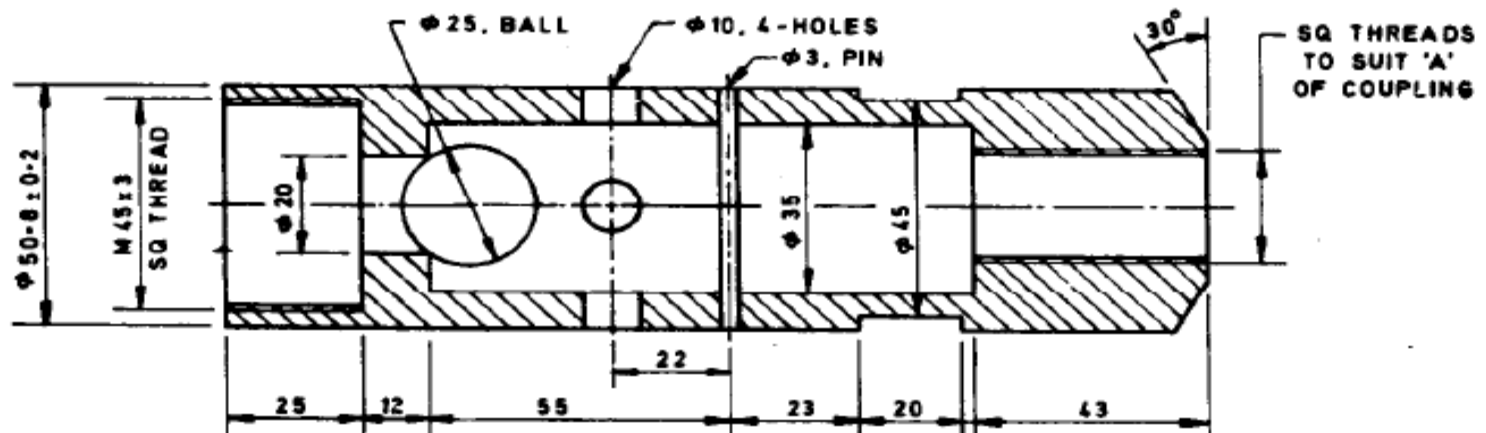


Cont...



All dimensions in millimetres.

FIG. 2. CUTTING SHOE



Steps

- Advanced bore hole to desired depth
- SPT sampler attached to standard drill rod is lowered into hole
- Hammer should have weight of 65 kg and fall should be 750 mm
- no. of blows required to cause 15 cm of successive penetration should recorded

penetration (cm)

No. of blows

0-15

N_1 → seating drive

15-30

N_2 Penetration resistance = $N_2 + N_3$

30-45

N_3

- After 45 cm penetration split spoon sampler is taken out sample is taken out

Condition for refusal

- 50 or more blows are required for 15 cm penetration
- 100 blows or more for 30 cm of penetration
- 10 successive blow produce no penetration
- Refusal may be due to stone also

Cont...

- The energy used during penetration:
 - To do work in shearing 30 cm of soil
 - To do work in compression of rods
 - losses

Factor affecting N value

A. Depth:

- At greater depth, overburden will be large. So confining pressure will be large.
- Major principle stress at failure will be large for given angle of shearing or same soil
- Observed value of N will be large
- For same soil strength is different for different depth
- For saturated soil stress is not function of angle of friction. So it doesn't depends on confining pressure.
- Correction for depth is not required for clay.

Cont...

B. Drainage condition:

- Rate of dissipation of pore pressure is high- drained condition
- Other wise undrained condition and strength will be governed by effective stress
- Observed N value will be affected by effective stress. (undrained condition)

C. Angle of shearing resistance and relative density:

- Angle of friction is high for high relative density of soil i.e. For dense soil
- Value of N is higher for large angle of friction.

SPT correction

- **Correction for overburden:**
- **To eliminate effect of overburden pressure it has been standardized as 1 kg/cm^2**
- **IF effective overburden pressure $> 1 \text{ kg/cm}^2$, observed N is higher than actual N. Correction will be -ve.**
- **IF effective overburden pressure $< 1 \text{ kg/cm}^2$, observed N is lower than actual N. Correction will be +ve.**

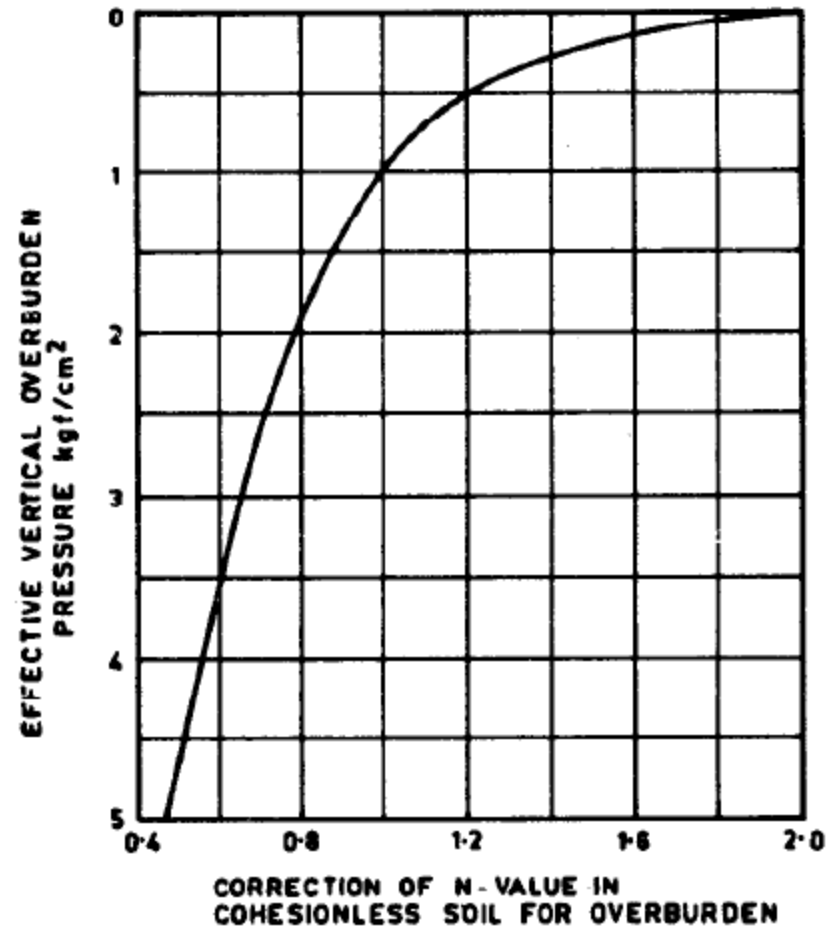
Cont...

- **Sandy soil**

- $N' = C_N N_{\text{obs}}$
- $C_N = 0.77 \log_{10}(2000/p')$ for $p' > 25 \text{ kPa}$

where p' is effective overburden pressure

- For $p < 25 \text{ kPa}$ use chart



Cont...

- **Correction for dilatancy (drainage)**
- only for fine sands and silts, saturated soil and for $N > 15$, because they are low permeable, voids are filled with water and dense
- In dense soil negative pore water generate during shearing so effective stress increase.
- This correction is not for clay.
- $N'' = 15 + (N' - 15)/2$

Summary

- Get observed N_{obs} for depth
- If soil is granular apply overburden correction otherwise no correction.
- Again if soil is fine sand or silt. It is saturated and $N' > 15$ apply dilatancy correction

Use of SPT result

N value	Angle of friction	Dr	Description
<4	25-32	0-15	Very loose
4-10	27-35	15-35	Loose
10-30	30-40	35-65	Medium
30-50	35-45	65-85	Dense
>50	>45	85-100	Very dense

Cont...

N-value	UCS	Consistency
0-2	<25	Very soft
2-4	25-50	Soft
4-8	50-100	Medium stiff
8-16	100-200	Very stiff
16-32	200-400	Hard

Example

- N is at the middle of soil layer. Three value of first, second and third 15 cm. Compute corrected N at the middle of each layer.

Clay, $\gamma = 18 \text{ kN/m}^3$, $N = 10, 11, 12$ 4 m

Clay, $\gamma = 20 \text{ kN/m}^3$, $N = 8, 9, 10$ 4 m

Fine sand, $\gamma = 18 \text{ kN/m}^3$, $N = 14, 15, 16$ 4 m

Coarse sand, $\gamma = 17 \text{ kN/m}^3$, $N = 18, 20, 22$ 4 m

Thank You

SOIL EXPLORATION

Dynamic Cone penetration Test

Dynamic cone penetration test (DCPT)

IS 4968-Part II-1976

- Mainly to supplement of the SPT or gives rough idea about the subsoil strata.
- Quick test give lesser information, but in many situation it is only possible test eg.: gravel, kankar
- The blows required to cause successively 15 cm penetration are continuously corrected.
- Diameter of cone = 50 mm (without slurry)
65 mm (with bentonite slurry)
- No. of blows for 30 cm is taken as $\rightarrow N_{cd}$

Refusal condition

- 50 or more blows are required for 15 cm penetration
- 100 blows for 30 cm penetration
- 10 successive blows produce no penetration
-

Interpretation

- Generally used to supplement SPT and not directly used for design
- Ncd is a measure of energy required for 30 cm penetration
- Large Ncd hard strata, small Ncd soft stratum.
- The values should also increase gradually with depth due to overburden
- Sudden change in Ncd values may be change in stratum.

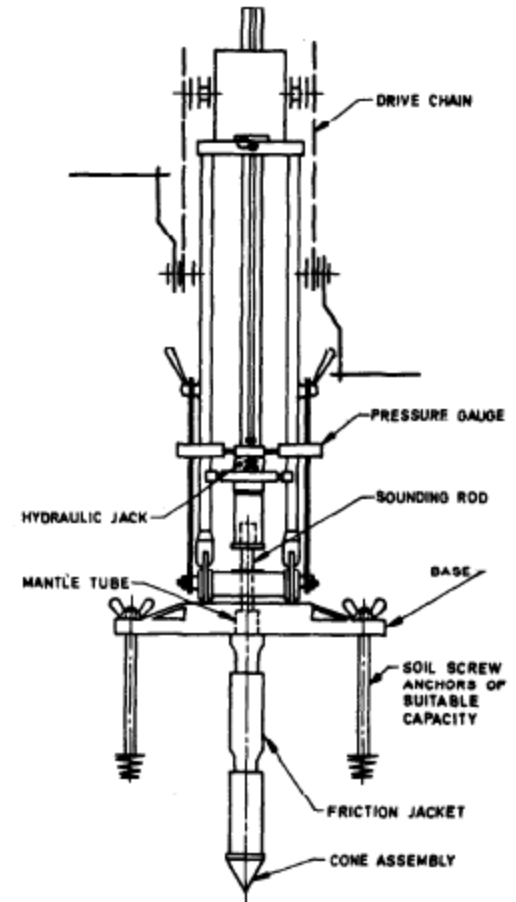
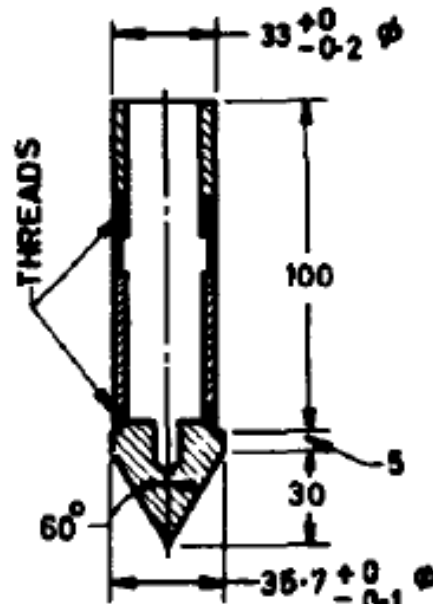
Advantages of DCPT

- It is a quick test and cover large area economically.
- It helps in identifying uniformity or variability of the sub-strata.
- It can be used to identify the soft pockets
- Rock strata can be located
- It can be used as starting point of study.
- Approximately relationship
- For 50 mm Ncd = 1.5 N → upto 3m
= 1.75 N → 3-6 m
= 2N → >6m
- CBRT, Roorkee for 65 mm
 - Ncd = 1.5 N for depth < 4m
= 1.75 N for depth 4-9 m
= 2N for depth >9 m

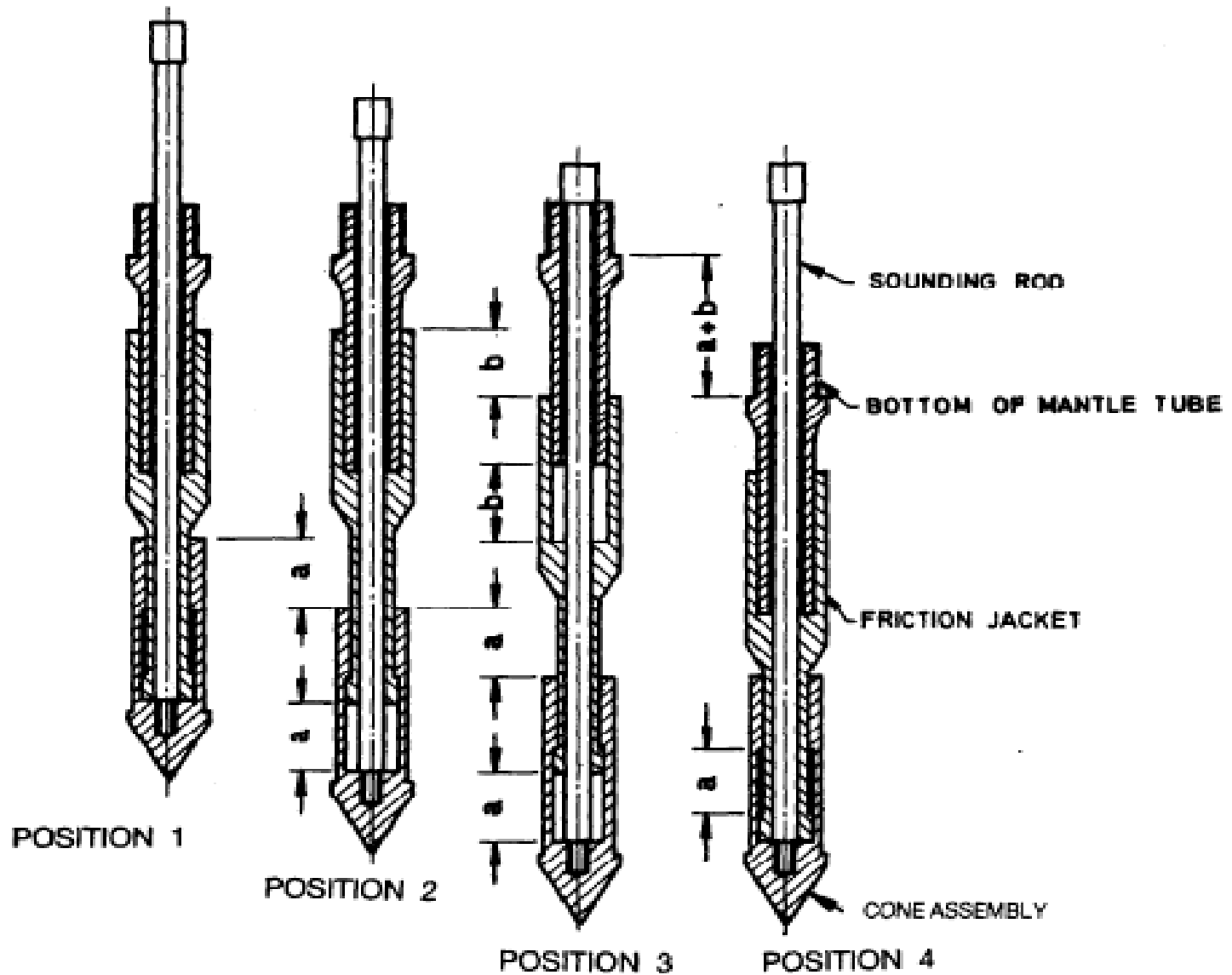
Static cone penetration test (SCPT)

IS 4968-part 3

- Being used for soft clay, silts and fine to medium sand deposits
- Was developed in Holland and is also known as Dutch cone test
- No-bore hole is required.



Operation



Cont...

- Tip, point or cone resistance $q_c = Q_c / A_c$ at 40 mm displacement
- $Q_f = A_f f_s \rightarrow$ force required to push friction jacket
 $A_f \rightarrow$ area of friction jacket
 $f_s \rightarrow$ area of friction jacket
- $Q = Q_f + Q_c$

Cont...

- **Advantages:**

- Continuous record of variation of cone resistance and friction resistance with depth is obtained
- Friction ratio = friction resistance/ cone resistance
- Unlike SPT this test measure static resistance of soil

Limitation:

- No sample of soil can be obtained
- The test is not suitable for gravels and very dense sand
- Equipment is heavy and may not be suitable for tough sites like hills or if no space is available

Correlation between q_c and N

Cohesionless soil

Type of soil	q_c / N
Sandy gravel and gravel	8-10
Coarse sand	5-10
Clean, fine to medium sand and slightly silty sand	3-4
Silts, sandy silts, slightly cohesive silt, sand mixtures	2

cont...

- **Cohesive soil**

- $q_c = N_k C_u + \sigma$

- where,

- $N_k \rightarrow$ cone factor

- $C_u \rightarrow$ undrained shear strength

- $\sigma \rightarrow$ total overburden pressure (not effective)

For NC soil, $N_k \rightarrow 11-19$

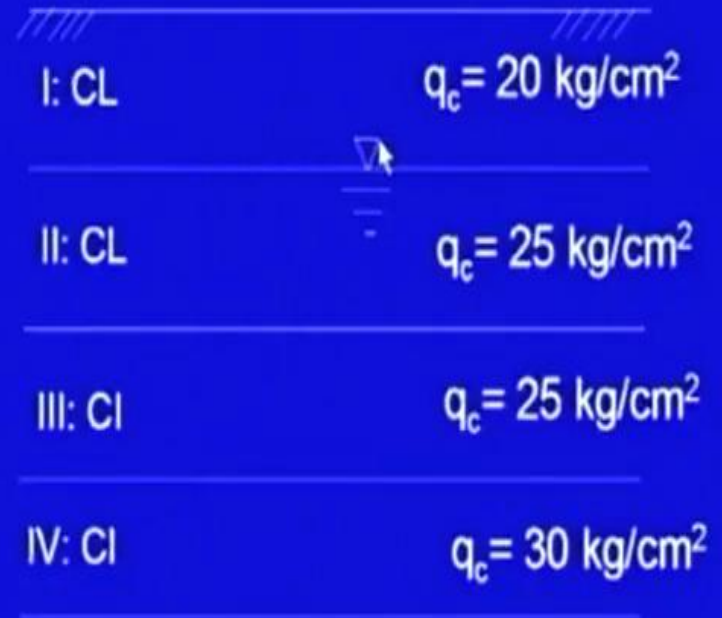
OC clays, $N_k \rightarrow 15-20$ for shallow depths

12-18 for large depths

Find C_u at mid points of layers

Layer I	Thickness (m)	γ (kN/ m ³)
I	4	18
II	4	20
III	4	18
IV	4	17

Example



SOIL EXPLORATION

Geophysical Method

GEOPHYSICAL METHOD



Although boring and test pits provide definite results but they are time consuming and expensive.



Subsurface conditions are known only at the bore or test pit location.



The subsurface conditions between the boring need to be interpolated or estimated.



Geophysical methods are more quick and cheaper.



They provide thorough coverage of the entire area.



The results of Geophysical testing however are less definitive and require subjective interpretation.



Therefore both methods are important. In case geophysical testing is major in scope, few borings and sampling will be required for accurate determination of soil properties.

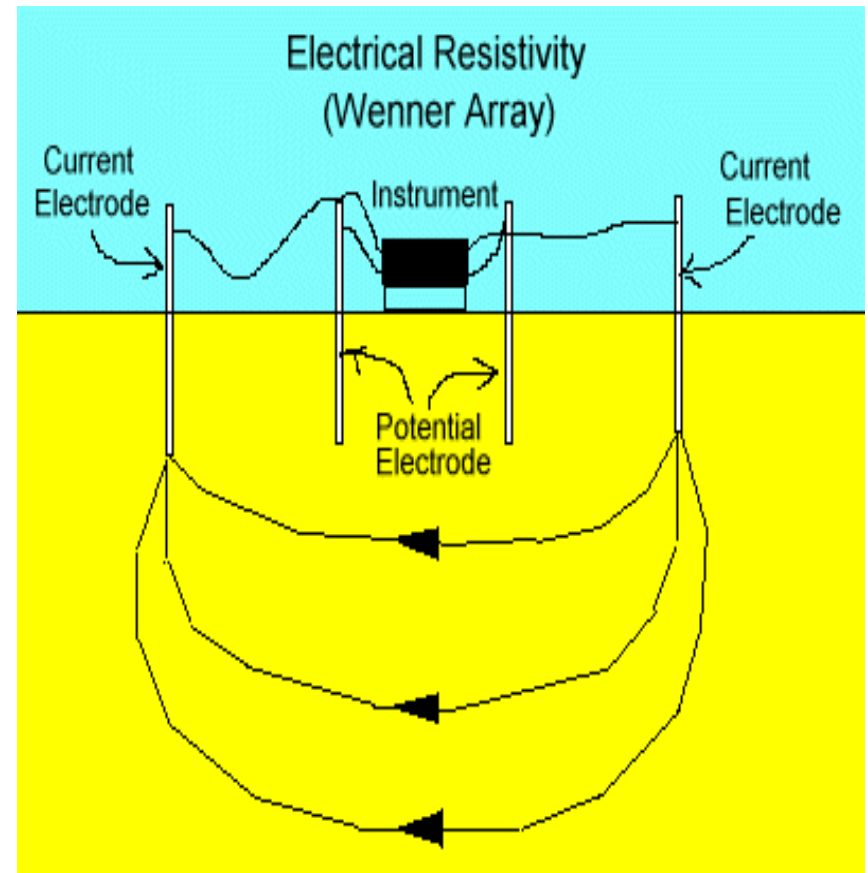


If boring is major in scope then few geophysical lines will be required to know the conditions in-between the borings.

Geophysical Techniques Indirect Methods

Advantages

- **Non-Destructive**
- **Cost Effective**
- **Provides Preliminary or Supplemental Information**



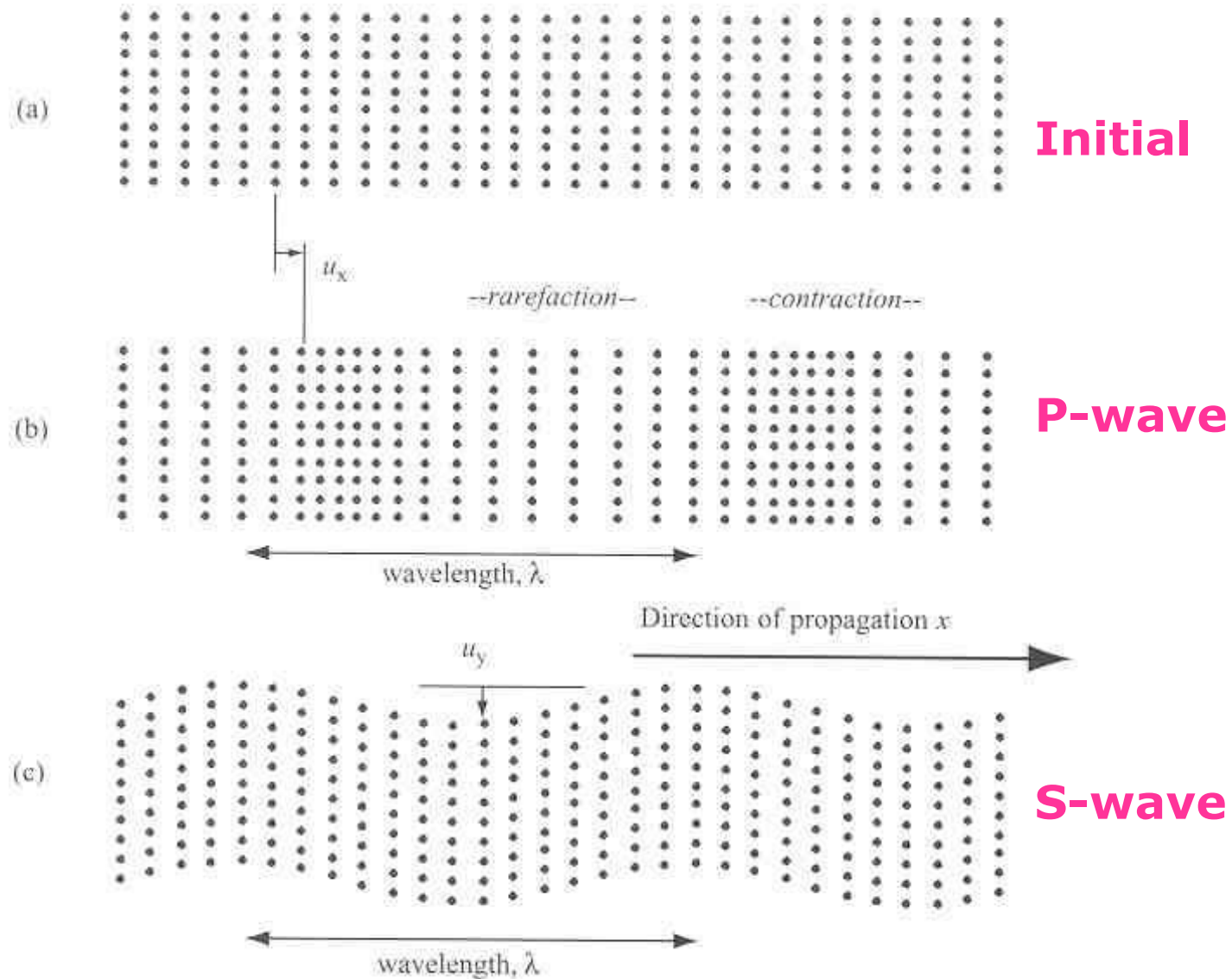
Geophysical Methods

- Mechanical Wave Measurements
 - Crosshole Tests (CHT)
 - Downhole Tests (DHT)
 - Spectral Analysis of Surface Waves
 - Seismic Refraction
 - Suspension Logging
- Electromagnetic Wave Techniques
 - Ground Penetrating Radar (GPR)
 - Electromagnetic Conductivity (EM)
 - Surface Resistivity (SR)
 - Magnetometer Surveys (MT)

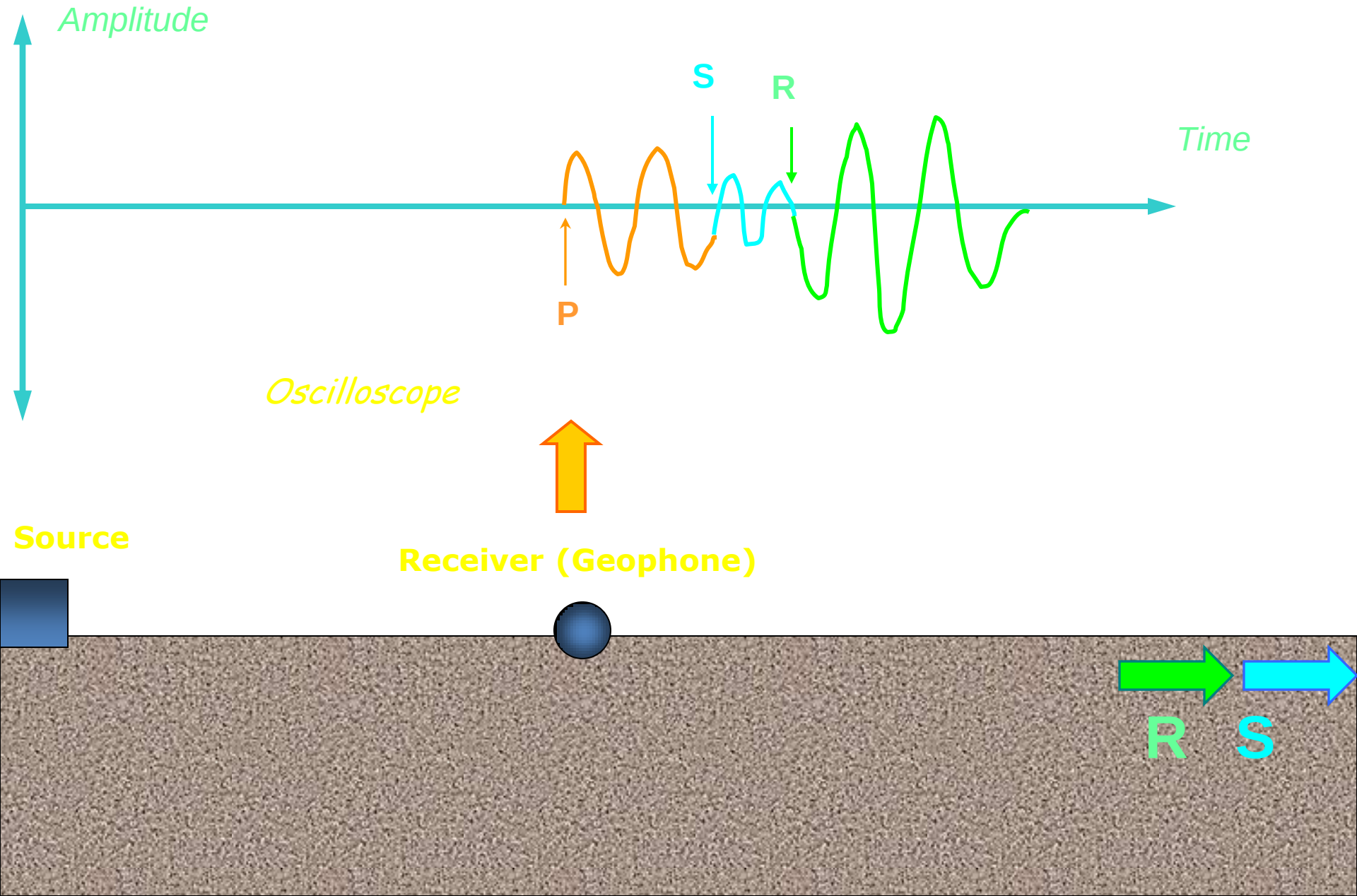
Mechanical Wave Geophysics

- Compression (P-) wave is fastest wave; easy to generate.
- Shear (S-) wave is second fastest wave. Is directional and polarized. Most fundamental wave to geotechnique.
- Rayleigh (R-) or surface wave is very close to S-wave velocity (90 to 94%). Hybrid P-S wave at ground surface boundary.
- Love (L-) wave interface boundary effect

Mechanical Body Waves

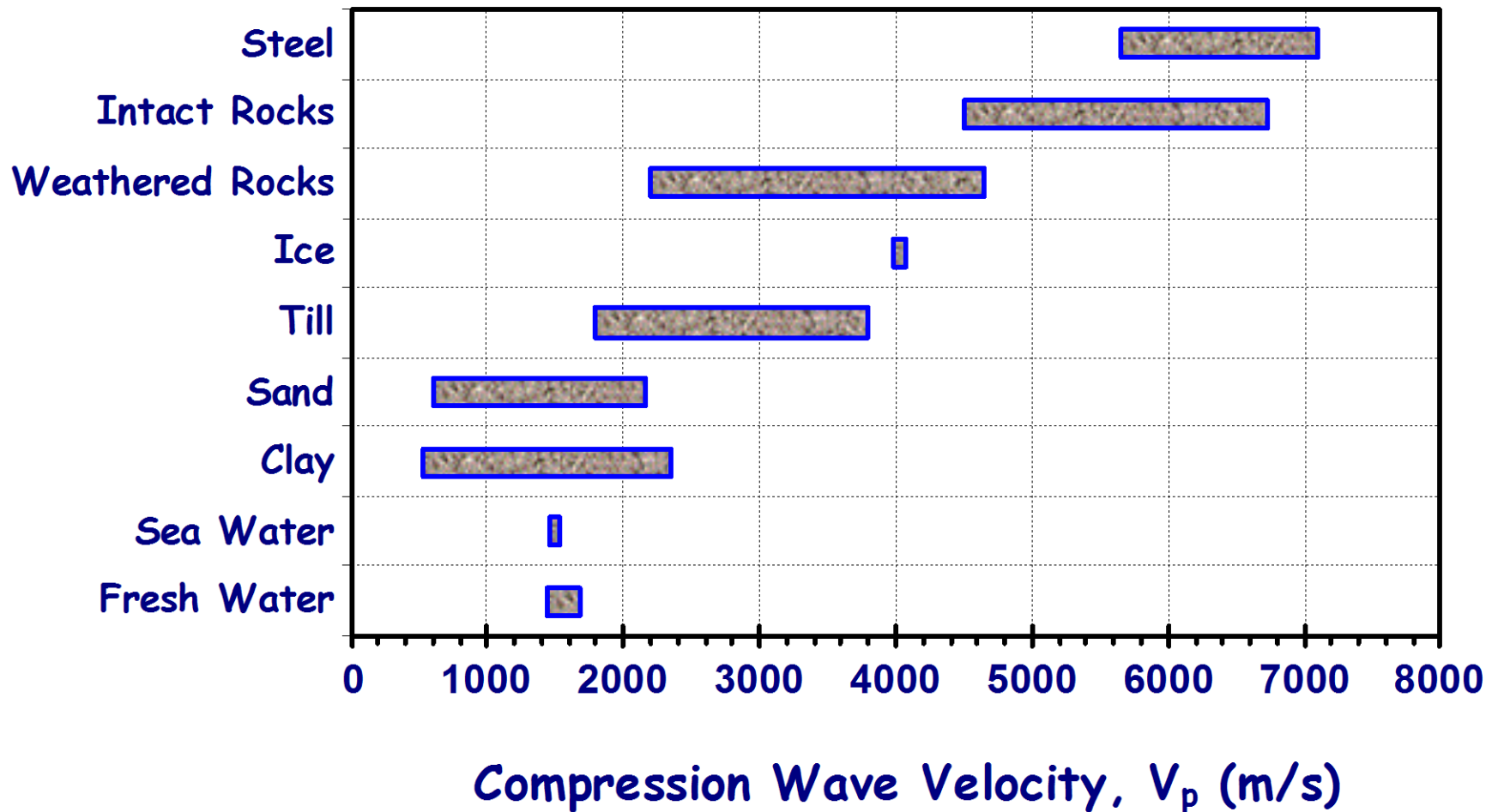


Mechanical Body Waves



Mechanical Waves (Compression)

P - Wave Velocities



Geophysical Equipment



Seismograph



Spectrum Analyzer



Portable Analyzer



Velocity Recorder

Seismic method

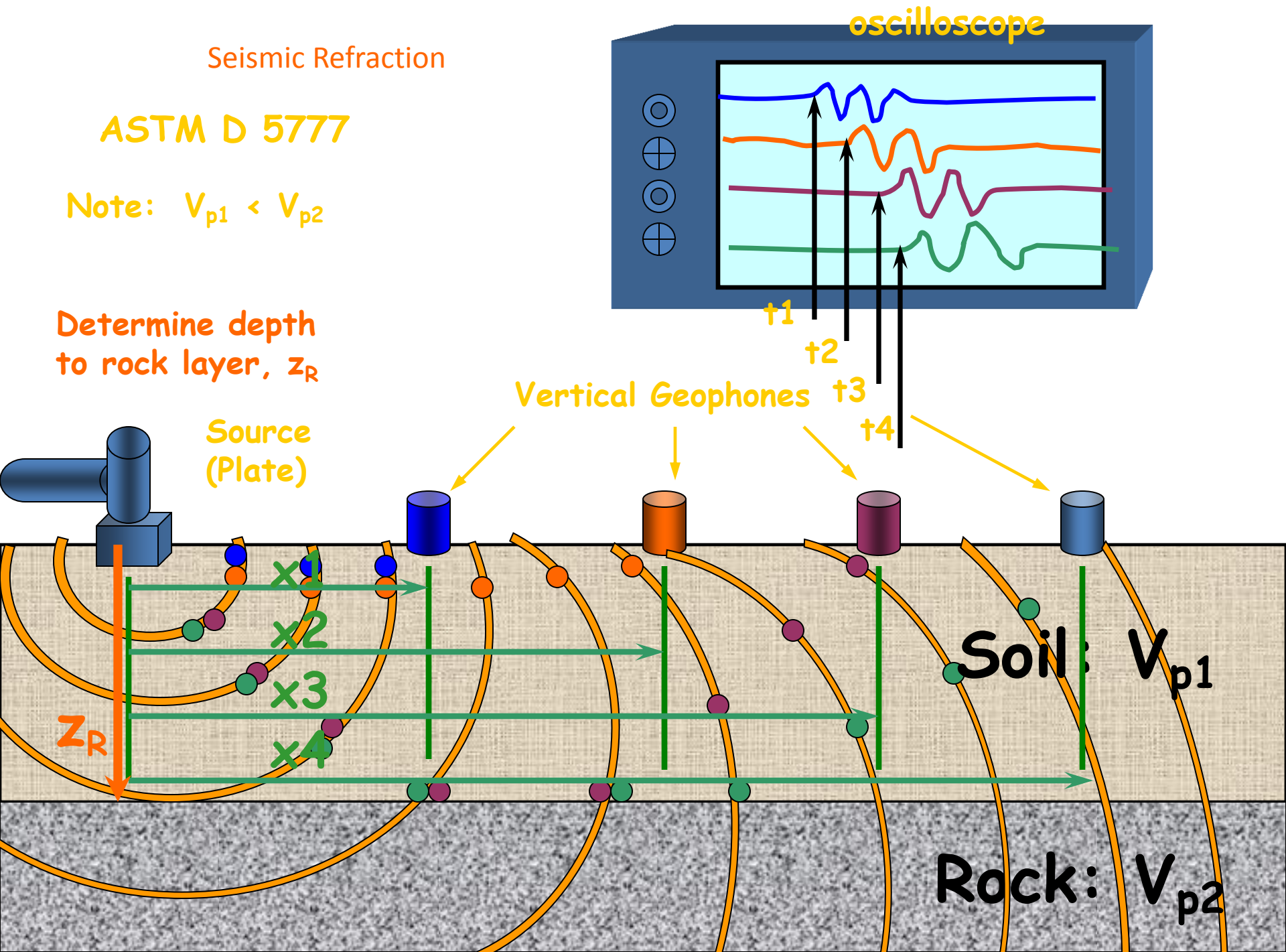
- Principle:
 1. Seismic waves are of different velocity in different rocks/soil.
 2. When they cross the boundary of different soil or rock types, the waves are refracted.

Seismic Refraction

ASTM D 5777

Note: $V_{p1} < V_{p2}$

Determine depth
to rock layer, z_R

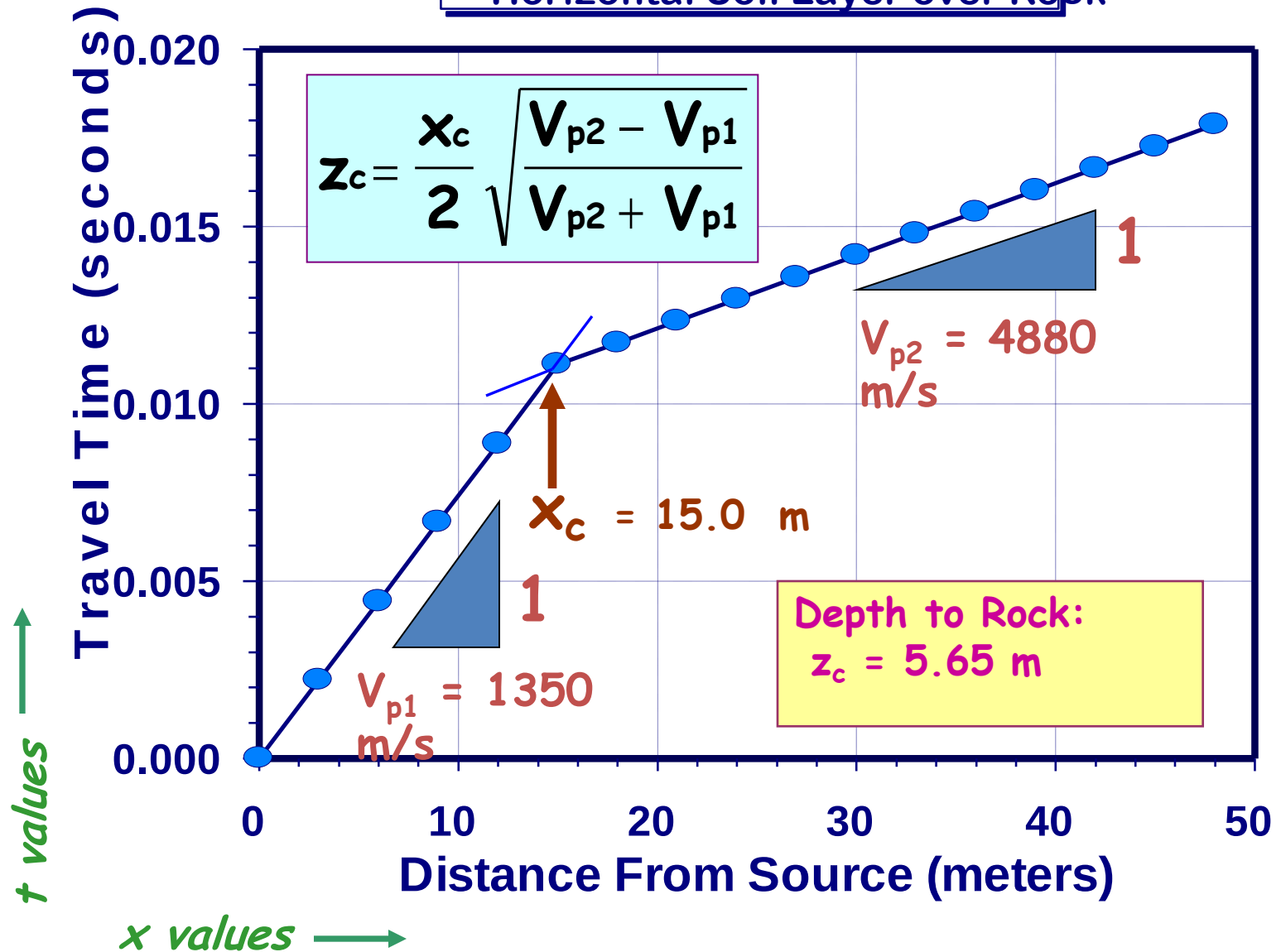


Steps

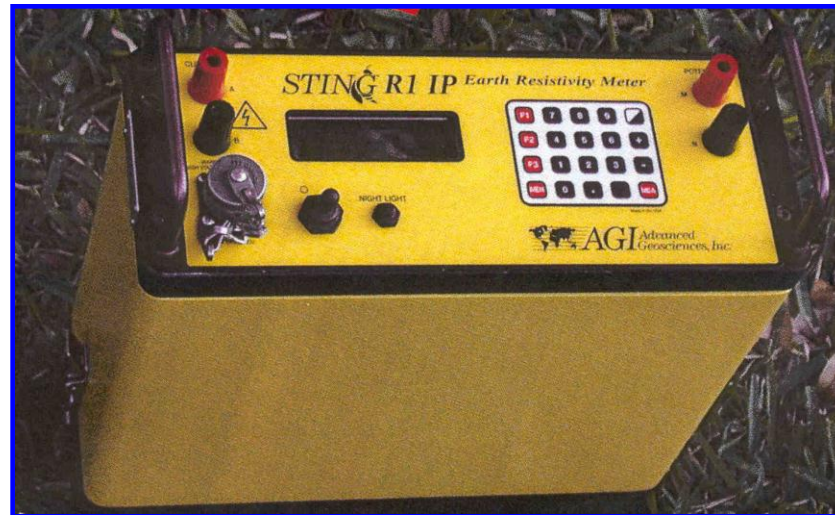
- Waves are generated by inducing impact.
- The wave so generated are recorded by device called a geophone which records the time of travels
- Geophones are placed at fixed distance from the source
- The direct (primary) waves travel direct along the ground
- The other waves go into the ground and get reflected if they pass into a stratum of different seismic velocity.
- If the underlying layer is denser, the refracted waves travel much faster.
- If the distance between the geophone and source is large the refracted wave reaches the geophones much earlier.
- If the geophone is placed very close to the source the direct waves reach earlier.

Seismic Refraction

Horizontal Soil Layer over Rock



Electrical Resistivity Measurements



Geophysical method

SOIL EXPLORATION

GEOPHYSICAL METHOD



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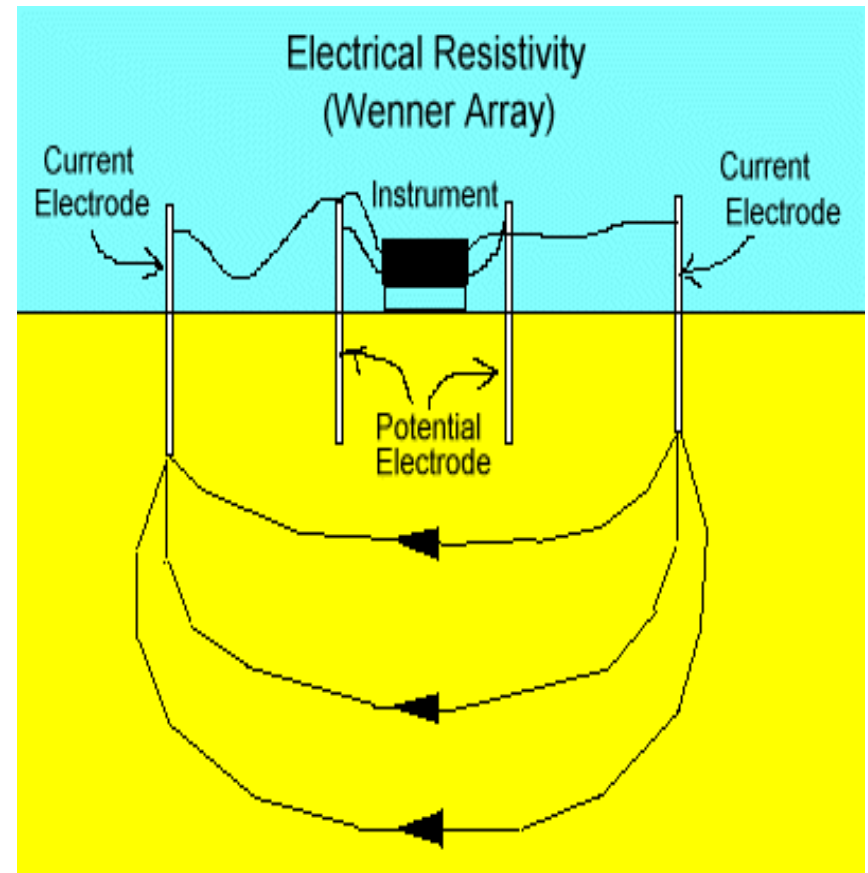


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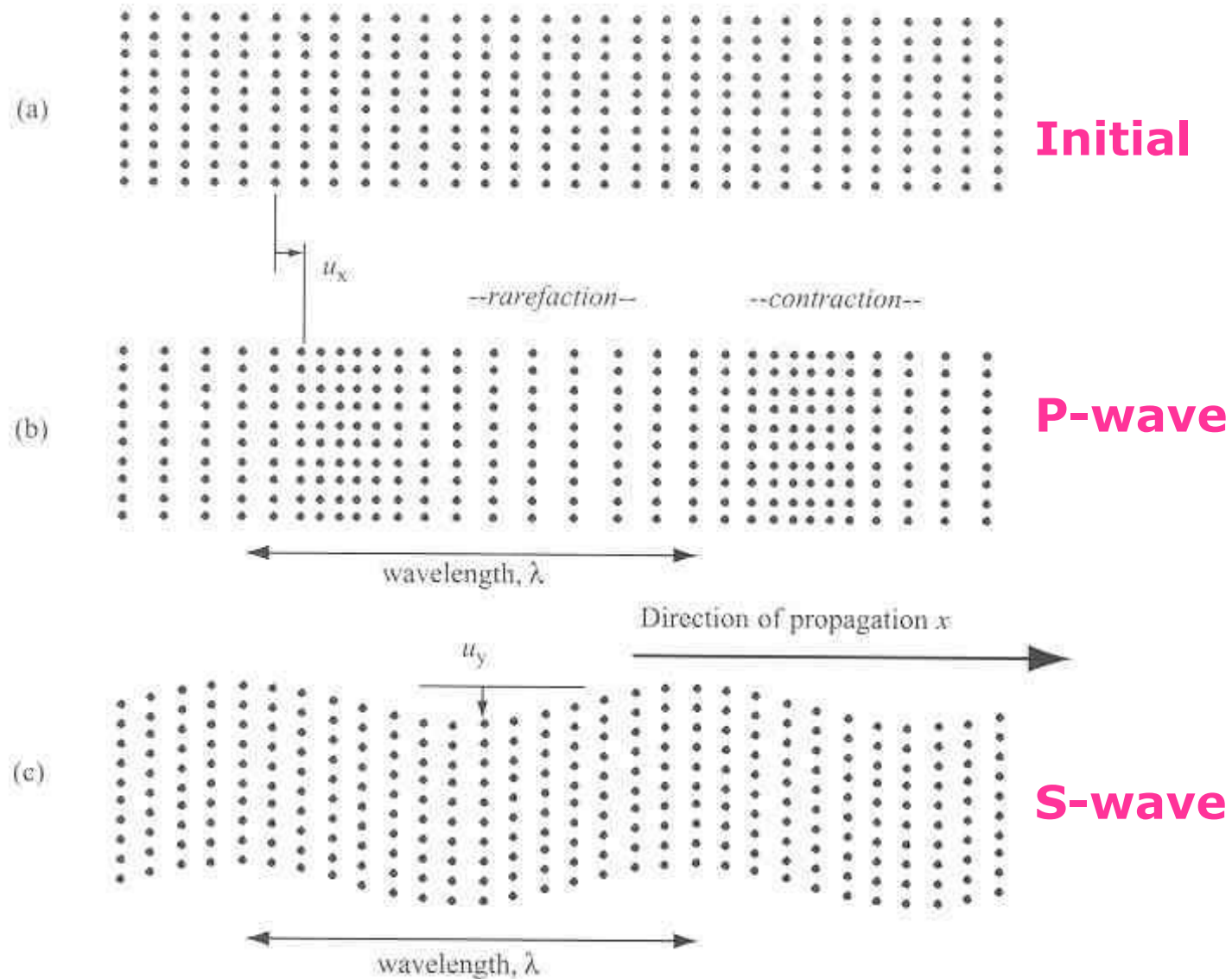
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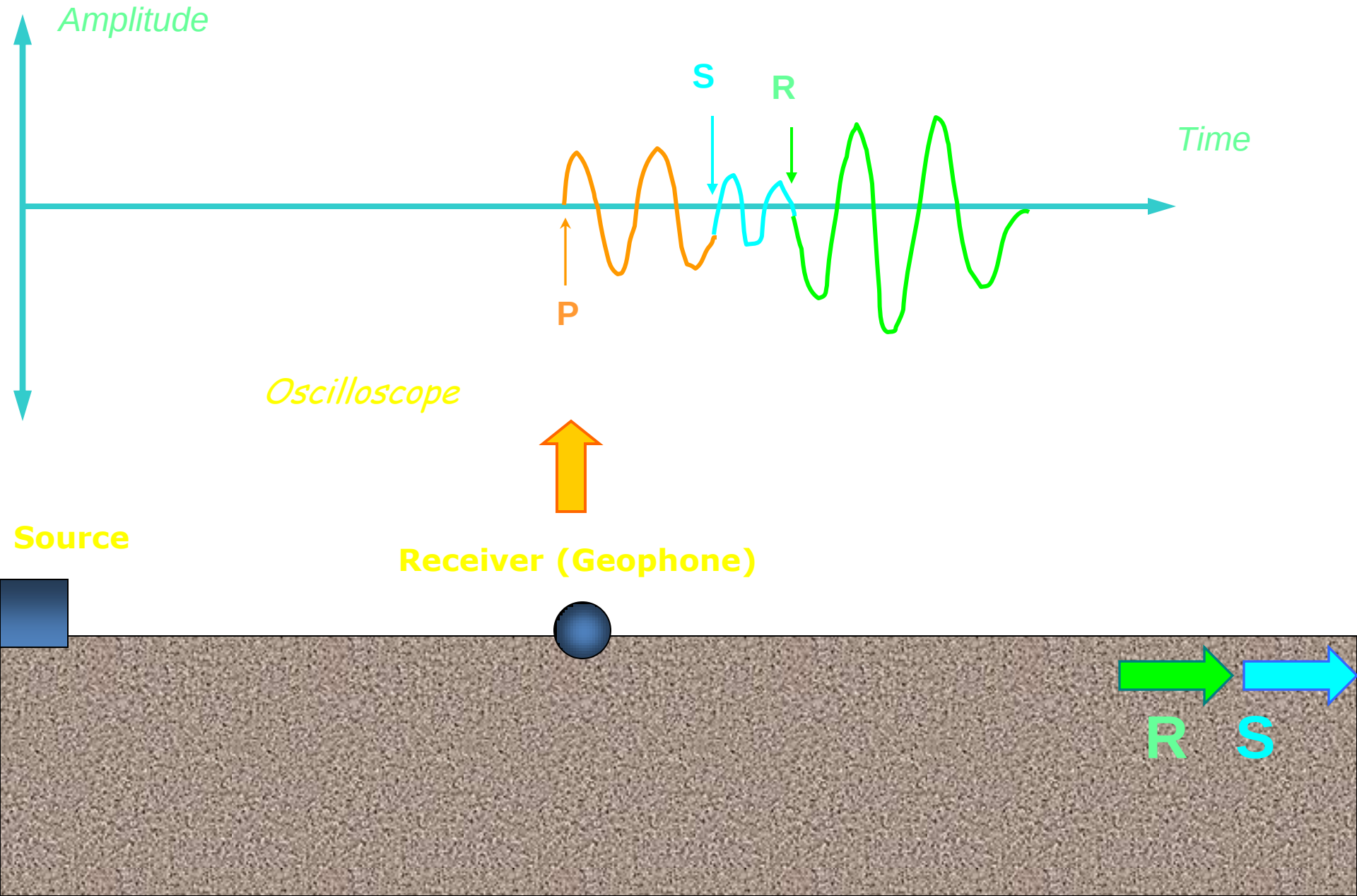
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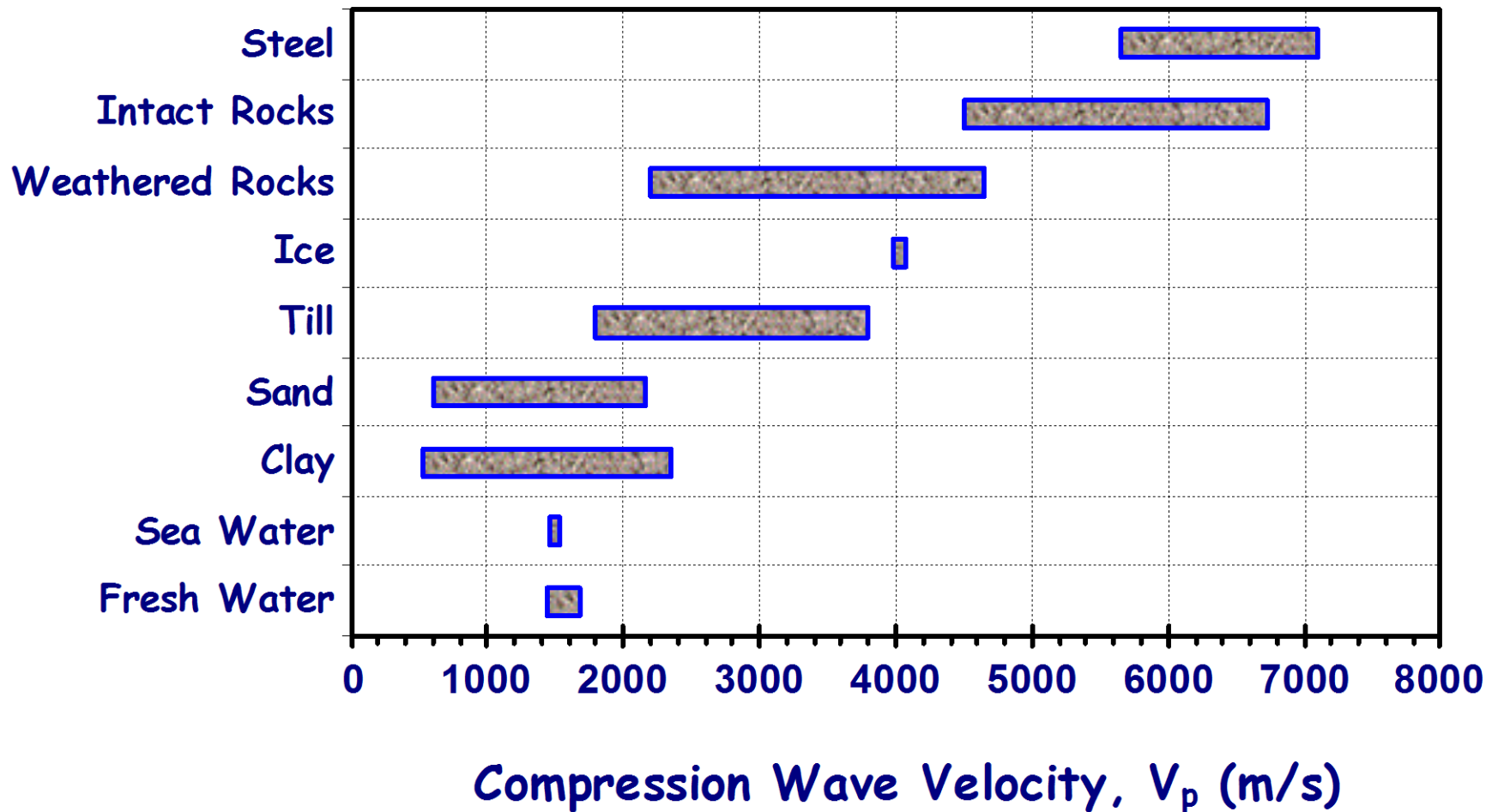


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Mechanical Waves (Compression)

P - Wave Velocities



Geophysical Equipment



Seismograph



Spectrum Analyzer



Portable Analyzer



Velocity Recorder

Seismic method

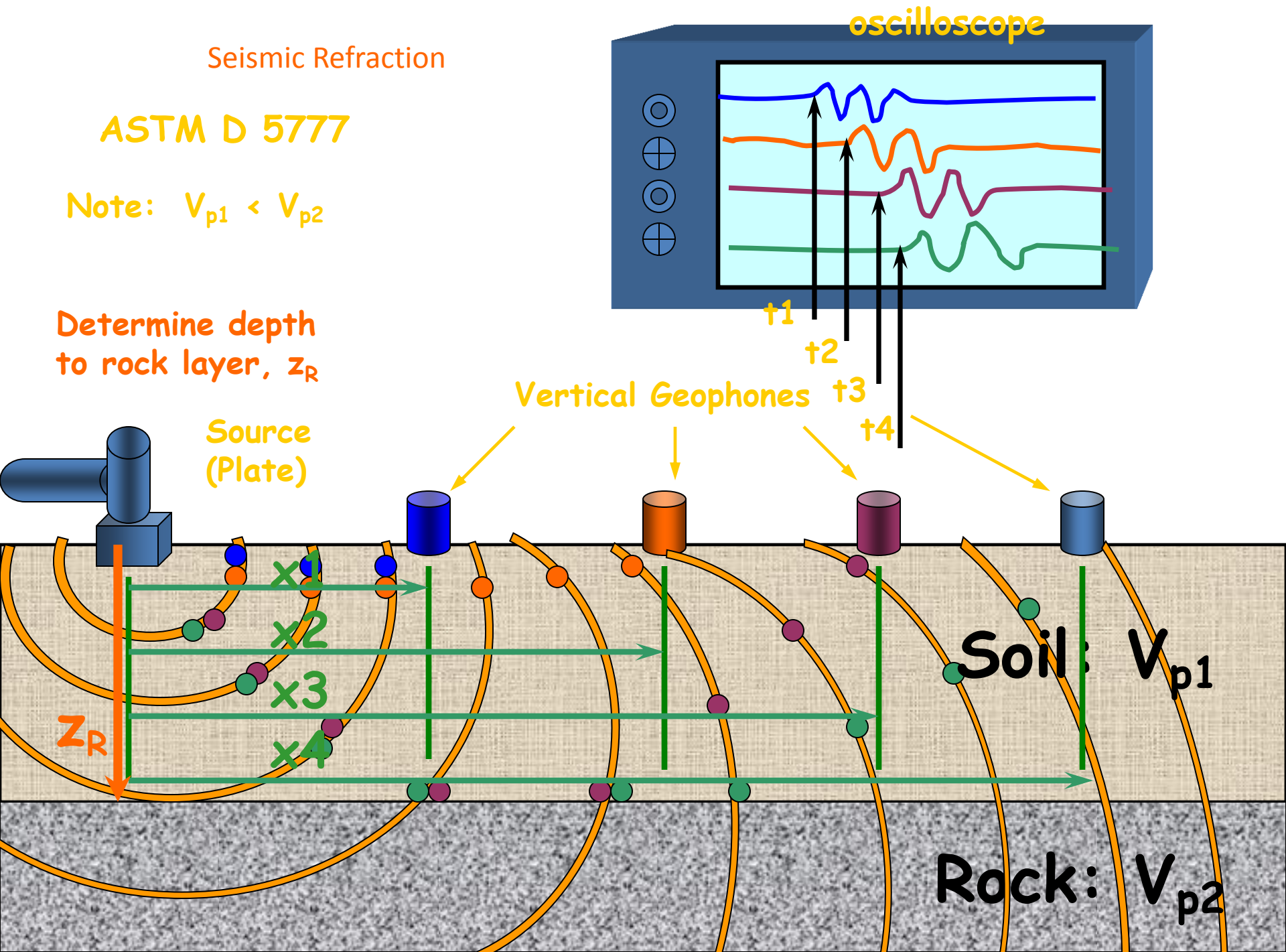
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ASTM D 5777

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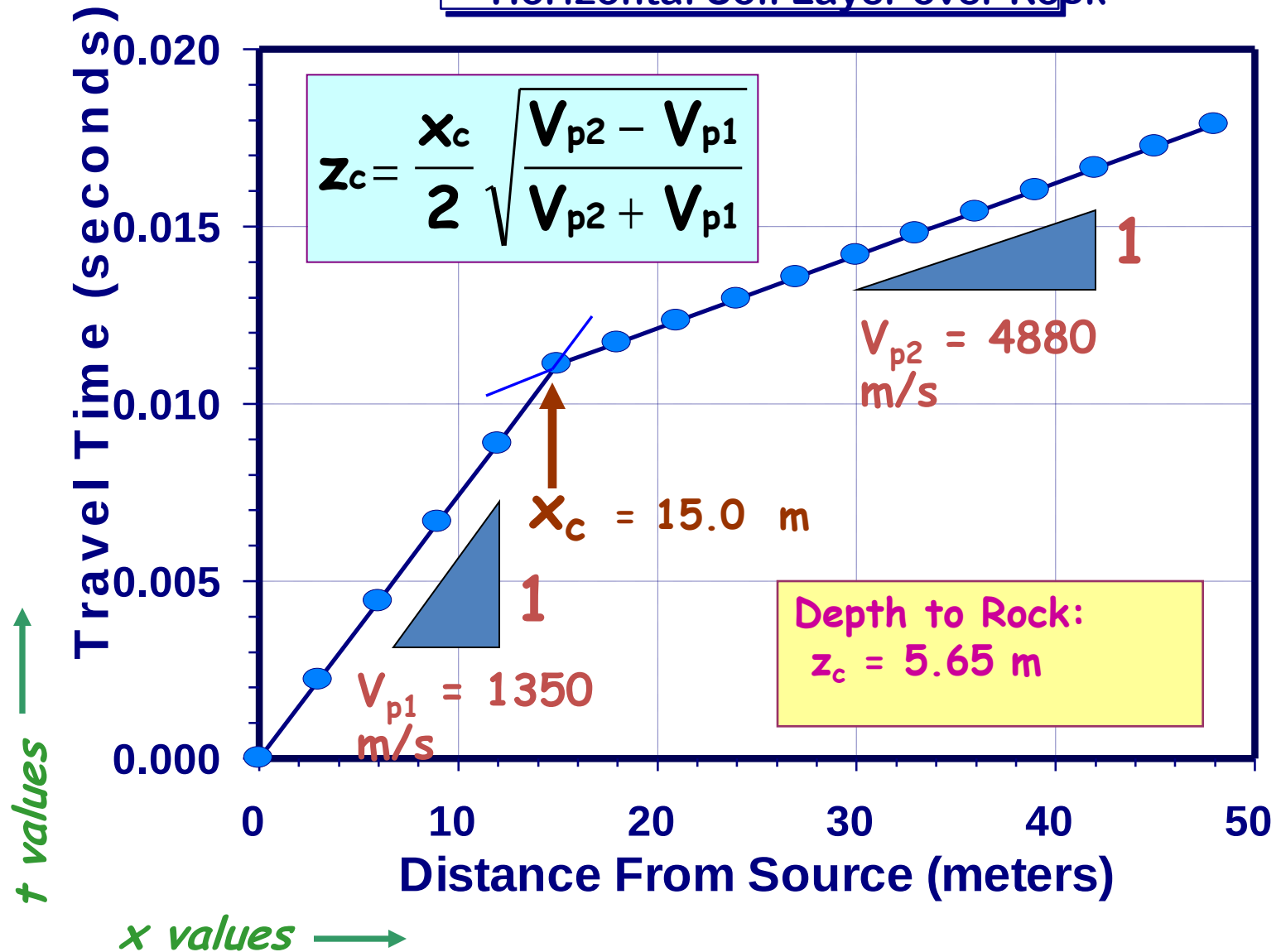


Steps

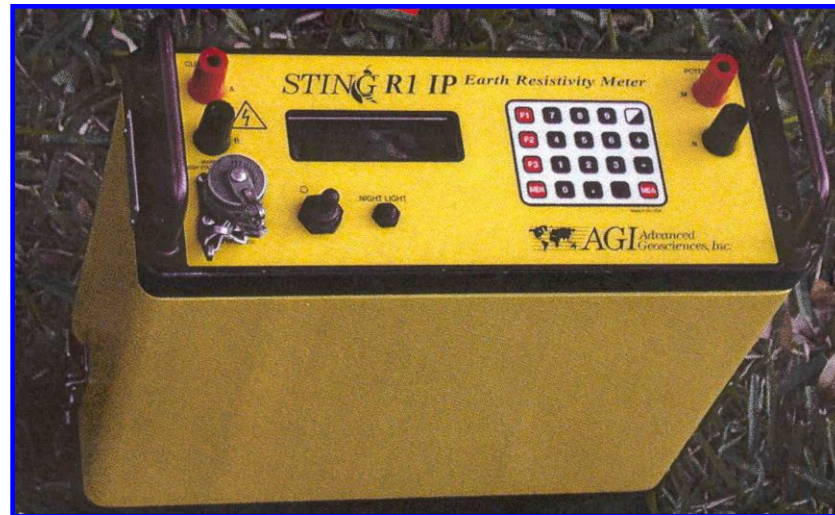
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Horizontal Soil Layer over Rock



Electrical Resistivity Measurements



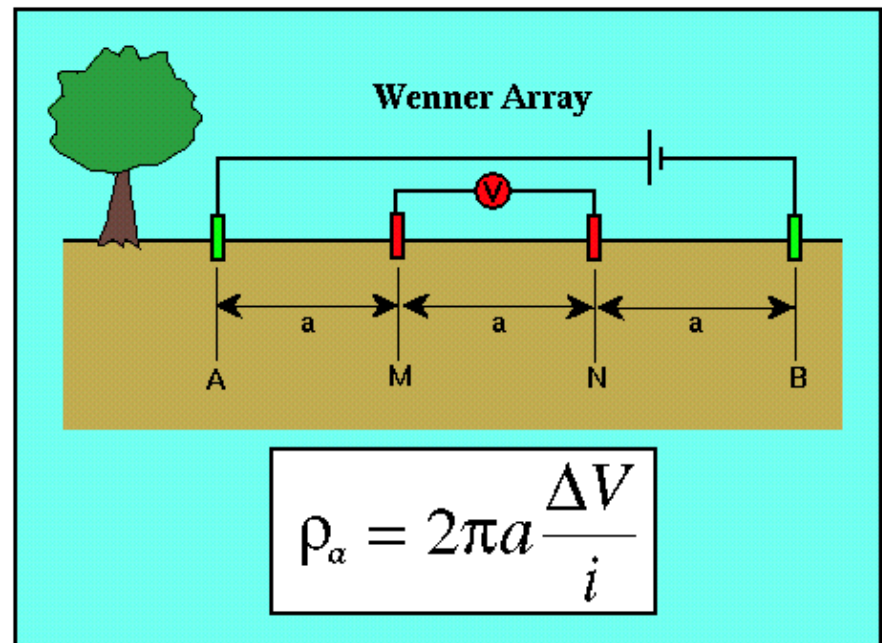
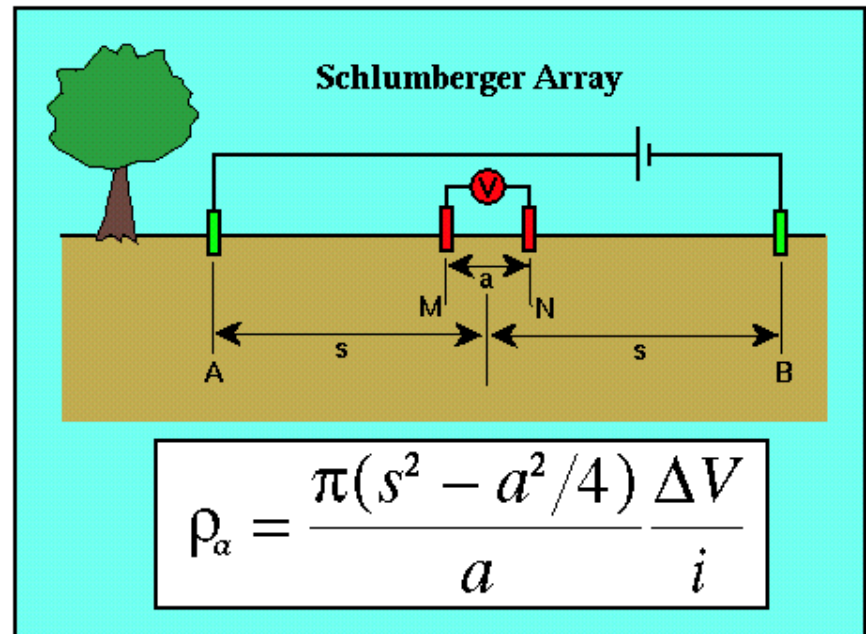
Electrical Resistivity (ER) Surveys

- Resistivity ρ_R (ohm-m) is an electrical property. It is the reciprocal of conductivity
- Arrays of electrodes used to measure changes in potential.
- Evaluate changes in soil types and variations in pore fluids
- Used to map faults, karst features (caves, sinkholes), stratigraphy, contaminant plumes.

Electrical Resistivity Measurements

What will be gained by changing electrode spacing?

Depth of ER survey:
i.e., greater spacing
influences deeper



Types of methods

1. **Electrical profiling method:**

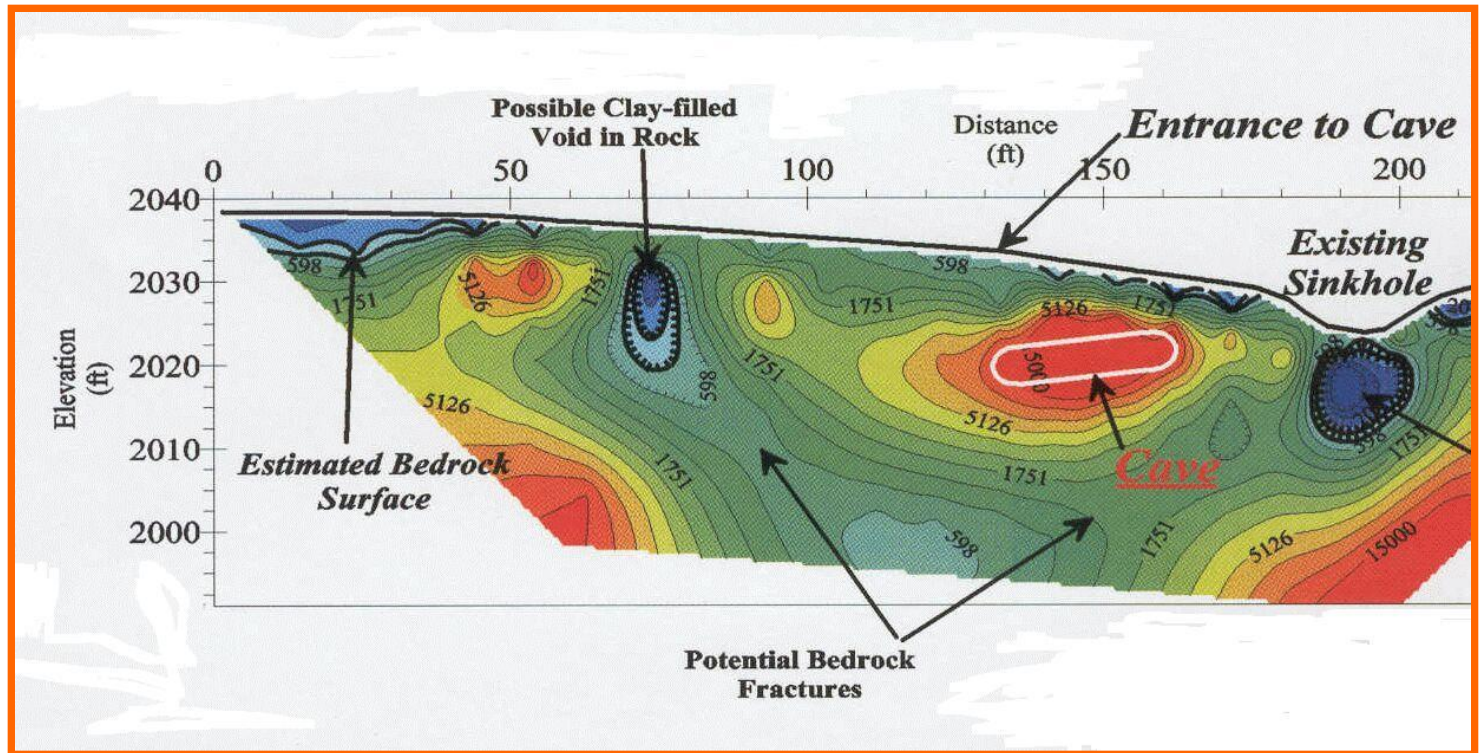
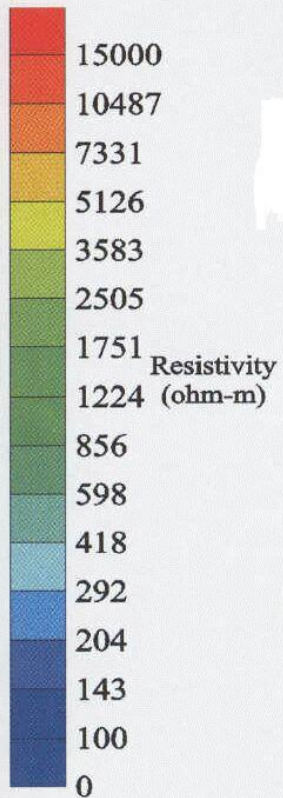
- Also known as resistivity mapping method.
- Distance of the electrodes will be fixed.
- Electrode are moved as group, across the area.

2. **Electrical sounding method**

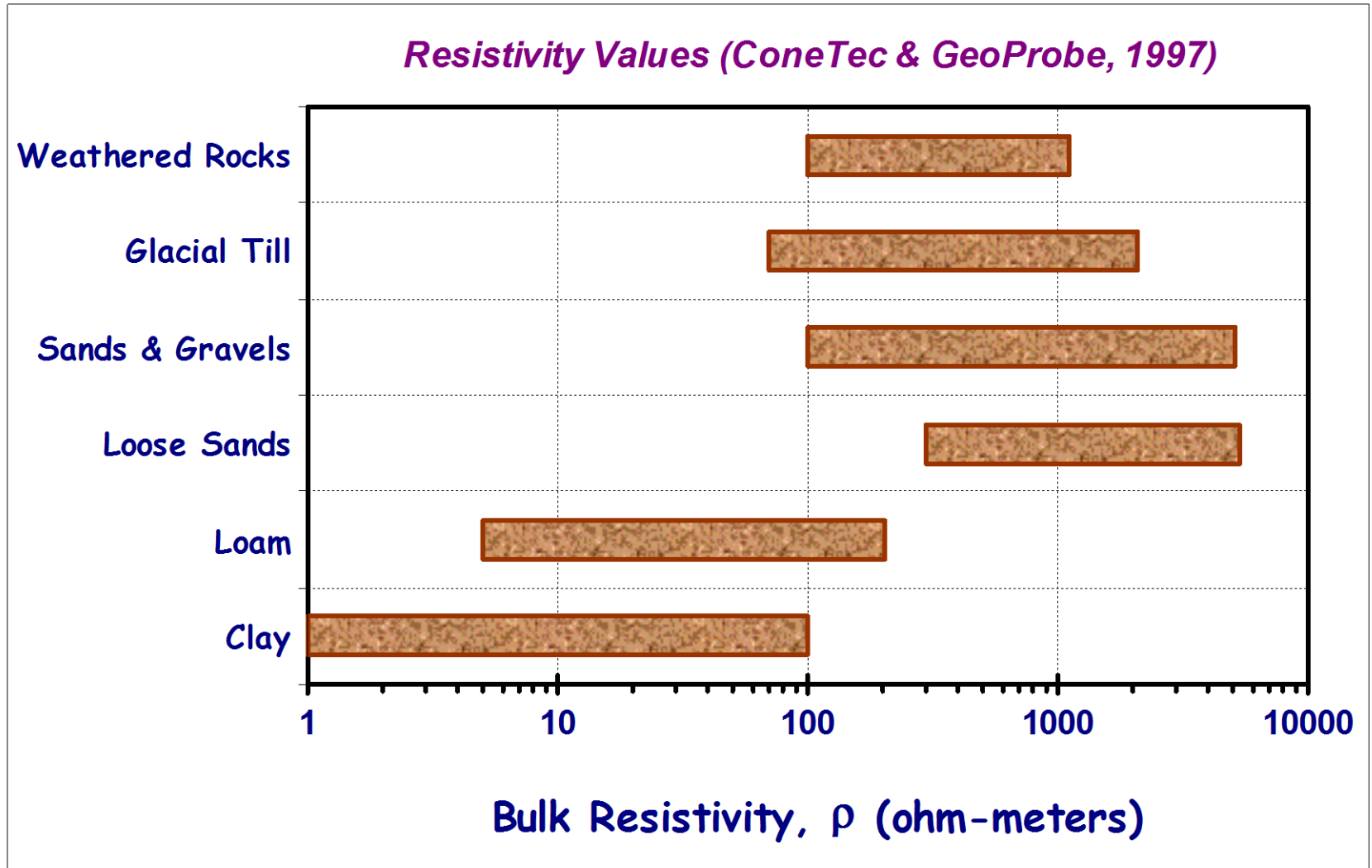
- In this method electrodes is expanded about a fixed location
- The spacing is gradually increasing up to the depth of exploration.

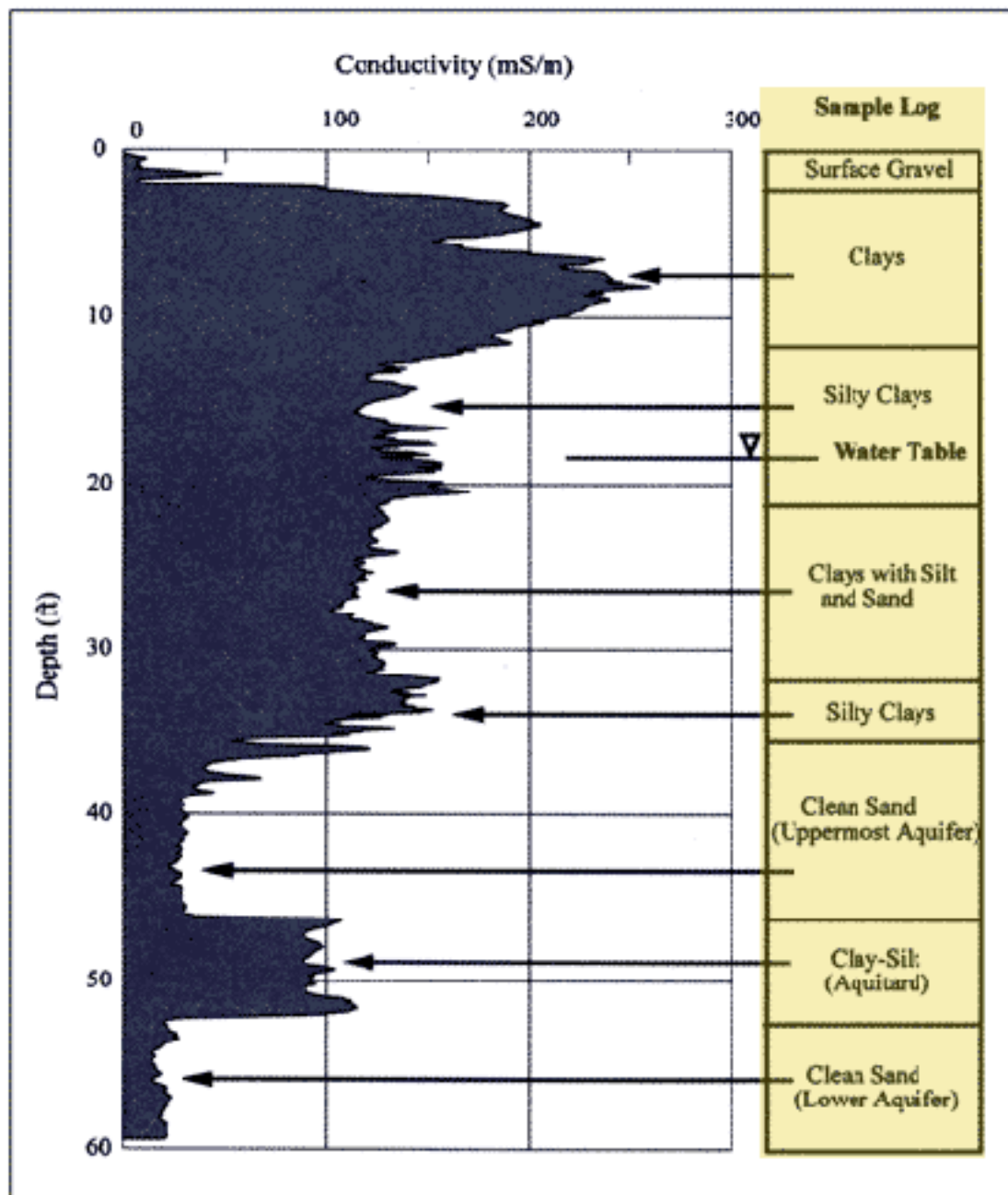
Electrical Resistivity Measurements

Legend



Electrical Resistivity Measurements





Sub-soil investigation report

- Report should contain the data obtained from bore holes site observations and laboratory results.
- All relevant data for the bore hole is recorded in boring log.
- A soil exploration report generally consists of following:
 1. Introduction, which gives the scope of the investigation.
 2. Description of proposed structured, the location and the geological condition at the site.
 3. Detail of the field exploration programme, indicating the number of borings, their location and depths.
 4. Details of the methods of exploration
 5. General description of sub-soil conditions as obtained from in-situ tests, such as SPT, cone test
 6. Details of the laboratory test conducted on the soil samples obtained and the results obtained.

Cont..

7. Depth of the ground water table and the change in water levels
8. Discussion of the results
9. Recommendation about the allowable bearing pressure the type of foundation or structure.
10. Conclusion. The main-findings of investigation should clearly stated. It should be briefly but should mention the main points.

Boring Logs



ACME Consulting Limited
44 Canadian Oaks Drive
Whitby, Ontario

Log of Borehole:

Project No.:

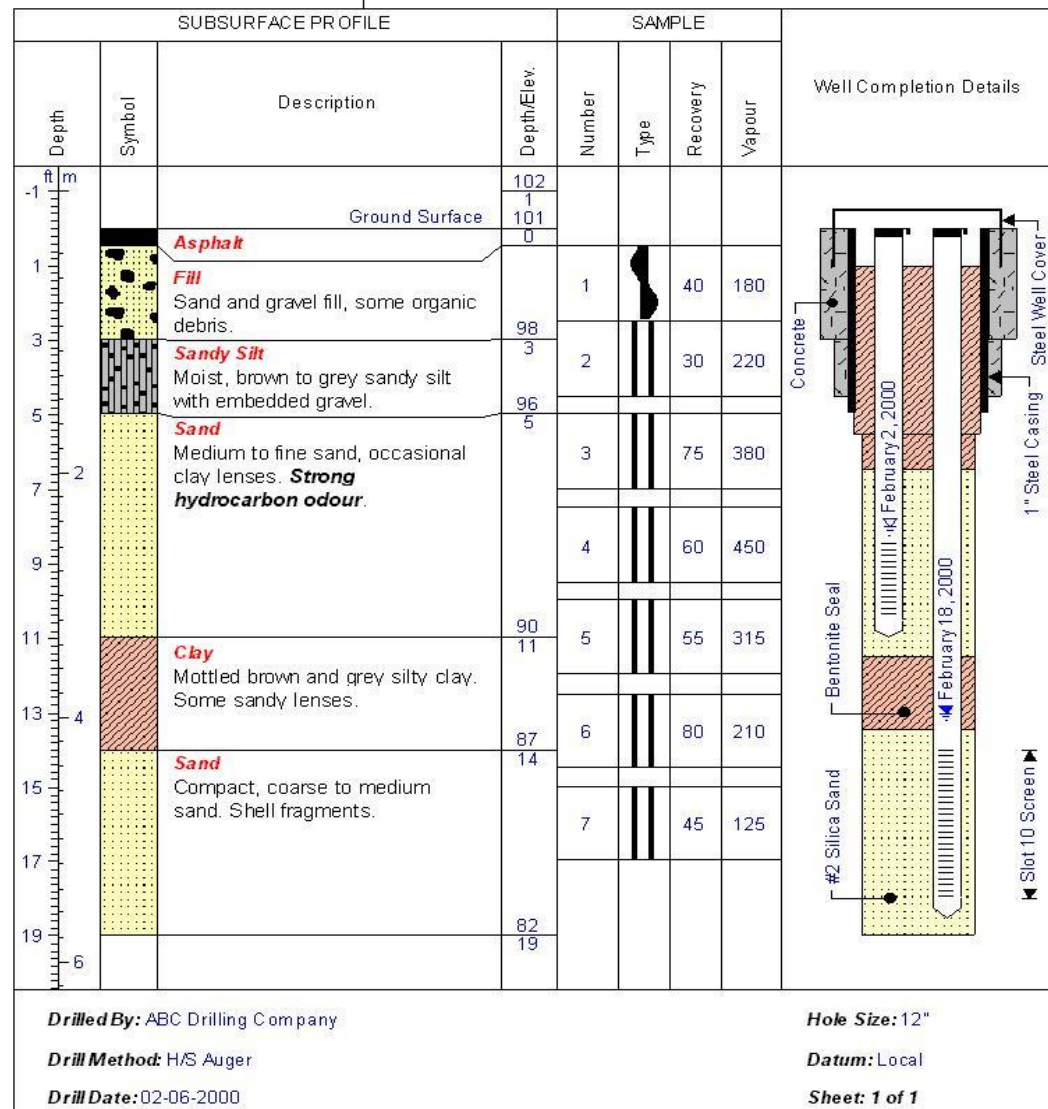
Project: Port Sidney Oil Terminal

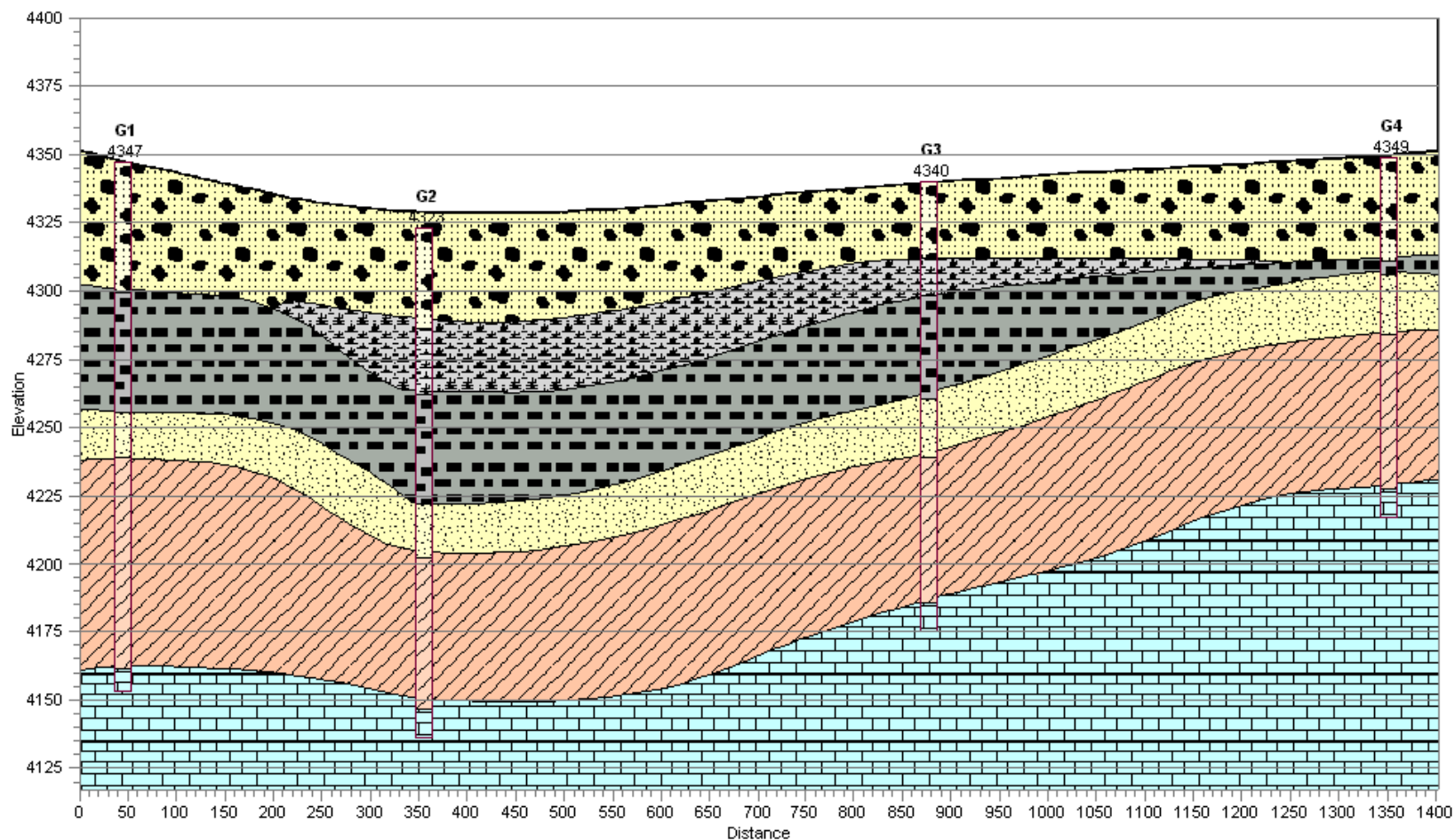
Client: Inter-Island Gas

Location: Port Sidney

Enclosure: 1

Project Manager: M. Fraser





GAEA Technologies
44 Canadian Oaks Dr.
Whitby, ON L1N 6W8
CANADA

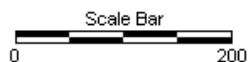
Project: Tutorial

Project Number: 2000-001

Location: Someplace, somewhere

Drawn By: M. Fraser

Date: 11/02/2000



Legend



Till



Coal



Clay



Peat



Sand



Limestone

Plan View



EARTH PRESSURE THEORY

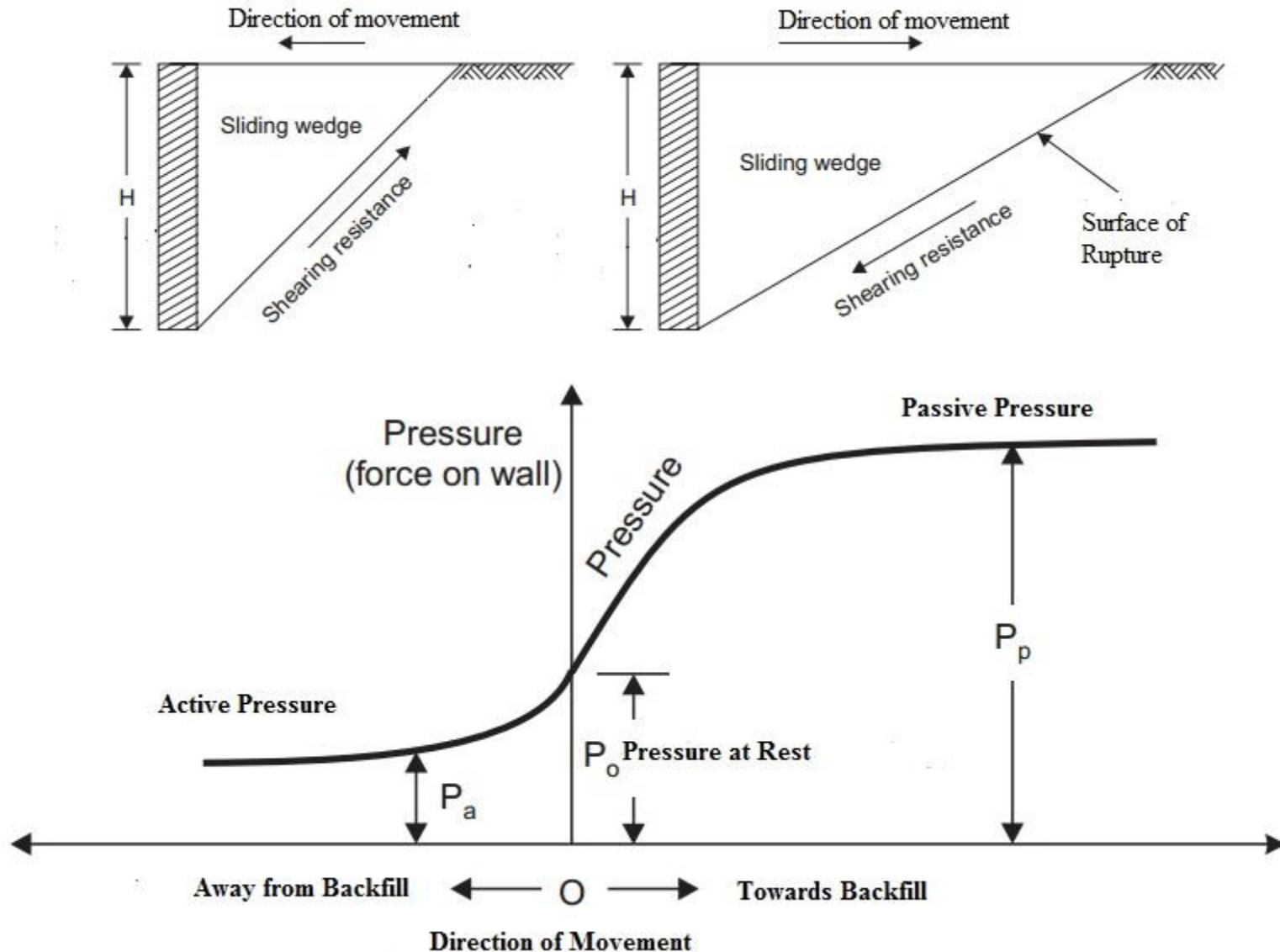
OUTLINE

- **Earth Pressure Theory**
- **Analysis of retaining walls**
- **Analysis of cut and excavation**
- **Analysis of sheet piles**

LATERAL EARTH PRESSURE

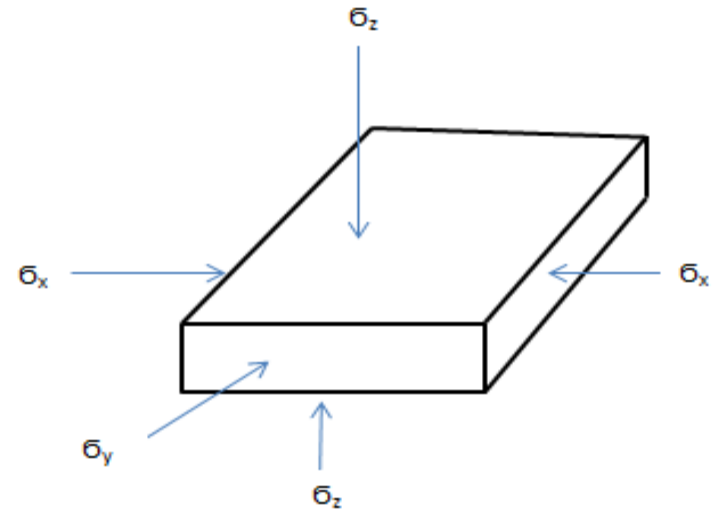
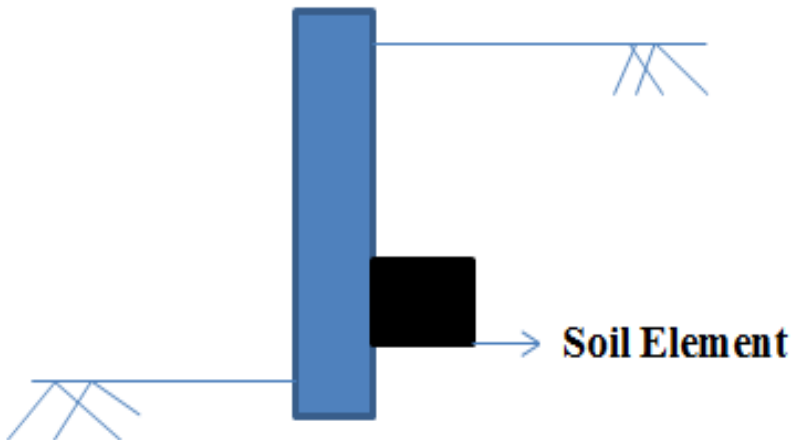
- Lateral earth pressure is the force exerted by the soil mass upon an earth-retaining structure, such as a retaining wall.
- Following are three different types of lateral earth pressure:
 - **Earth Pressure at Rest**
 - **Active earth pressure**
 - **Passive earth pressure**

LATERAL EARTH PRESSURE



EARTH PRESSURE AT REST

- Earth pressure at rest can be evaluated using theory of elasticity.
- Assumptions;
 - Soil is homogenous, isotropic, and semi infinite.
 - Elastic modulus and Poisson ratio is constant.



EARTH PRESSURE AT REST

$$\varepsilon_x = \frac{1}{E} [\sigma_x - \mu(\sigma_z + \sigma_y)] = 0$$

$$\sigma_y = [\sigma_x]$$

$$\sigma_x = \frac{\mu}{1 - \mu} \sigma_z$$

$$K_o = \frac{\mu}{1 - \mu}$$

EARTH PRESSURE AT REST

- *Emperical Relationship for K_o*

- *Jaky 1944,*

$$K_o = (1 - \sin\phi)$$

- *Fraser 1957*

$$K_o = 0.9(1 - \sin\phi)$$

- *Kenney 1959*

$$K_o = 0.19 + 0.233 \log I_p$$

- *For OC soil*

$$K_{o(OC)} = K_{o(NC)} \sqrt{OCR}$$

ACTIVE and PASSIVE EARTH PRESSURE

- Wall movement of around 0.2% for dense soil and 0.5% for loose sand required for active earth pressure.
- Following are two classical theories of earth pressure:
 - Rankine's theory (1857)
 - Coulomb's theory (1776)

Rankine's Theory

- Rankine' theory consider plastic equilibrium i.e. shear failure takes place at every point.
- Assumption
 - Soil is semi-infinite, homogeneous-isotropic, dry and cohesionless.
 - Soil is in state of plastic condition at the time of active and passive pressure generation.
 - The backfill soil is horizontal.
 - Back of wall is vertical and smooth
 - Rupture surface is a planar surface which is obtained by considering the plastic equilibrium of soil

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ACTIVE EARTH PRESSURE

- Lateral earth pressure is the force exerted by the soil mass upon an earth-retaining structure, such as a retaining wall.
- Following are three different types of lateral earth pressure:
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 - Active earth pressure
 - Passive earth pressure

RETAINING WALL AND SHEET PILES

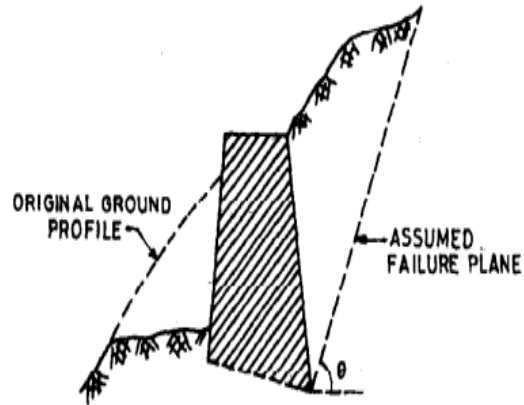
Retaining Structures

- A structure which is used to hold back a soil mass is called a retaining structures.
- **Types of Retaining structures:**
 - Retaining wall- Gravity type, cantilever type, counterfort type
 - Bracing in cuts
 - Abutment of a bridge
 - Sheet Pile/Anchored sheet piles

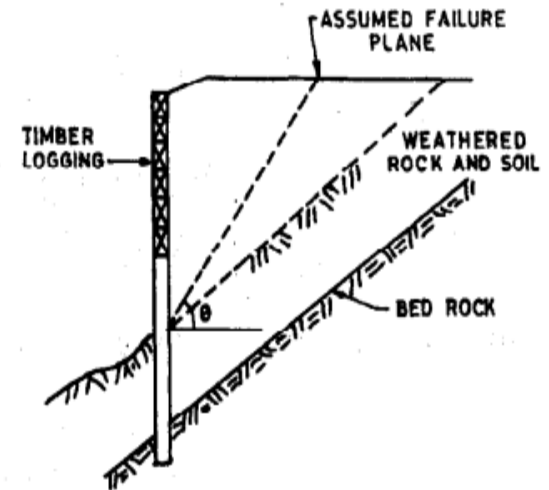
Standard related to Retaining wall

- **IS 14458- Retaining wall for Hill area Guidelines**
 - Part 1 Selection of type of wall,
 - Part 3 Construction of dry stone walls,
 - Part 4 Construction of banded dry stone walls,
 - Part 5 Construction of cement stone walls,
 - Part 6 Construction of gabion walls,
 - Part 7 Construction of RCC crib walls,
 - Part 8 Construction of timber crib walls,
 - Part 9 Design of RCC cantilever wall/buttressed walls/L-type walls, and
 - Part 10 Design and construction of reinforced earth retaining walls.

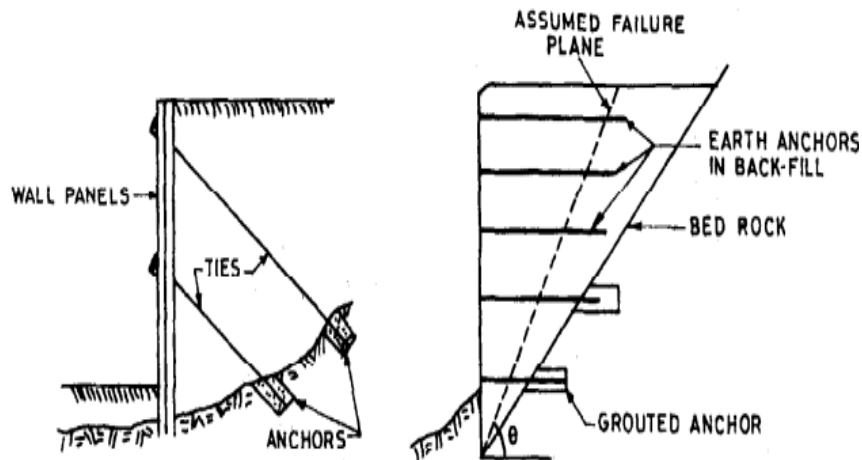
Types of Retaining wall



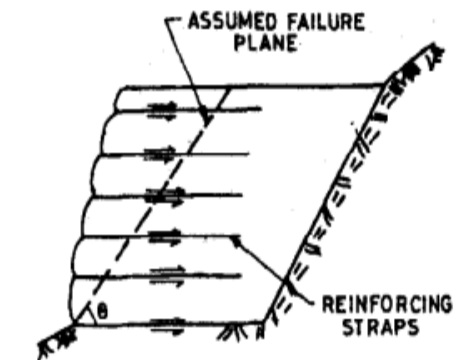
1(a) GRAVITY WALL



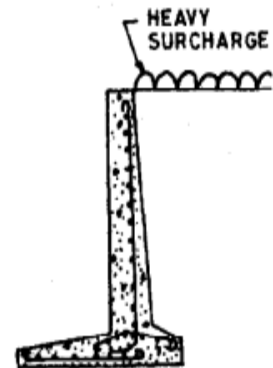
1(c) DRIVEN CANTILEVER WALL



1(b) TIE BACK WALL



1(d) REINFORCED EARTH WALL



1(e) RCC WALLS

Analysis of Retaining wall

- **Forces to be considered**
 - Self weight of the retaining structure
 - Live and imposed loads
 - Earth pressure acting on the wall
 - Water pressure due to water table/subsurface seepage
 - Water pressure due to water table on toe side
 - Seismic forces
 - Special loads

Analysis of Retaining wall

- **Requirements to be considered**
 - No sliding
 - No overturning
 - No bearing capacity failure
 - No tension

Analysis of Retaining wall

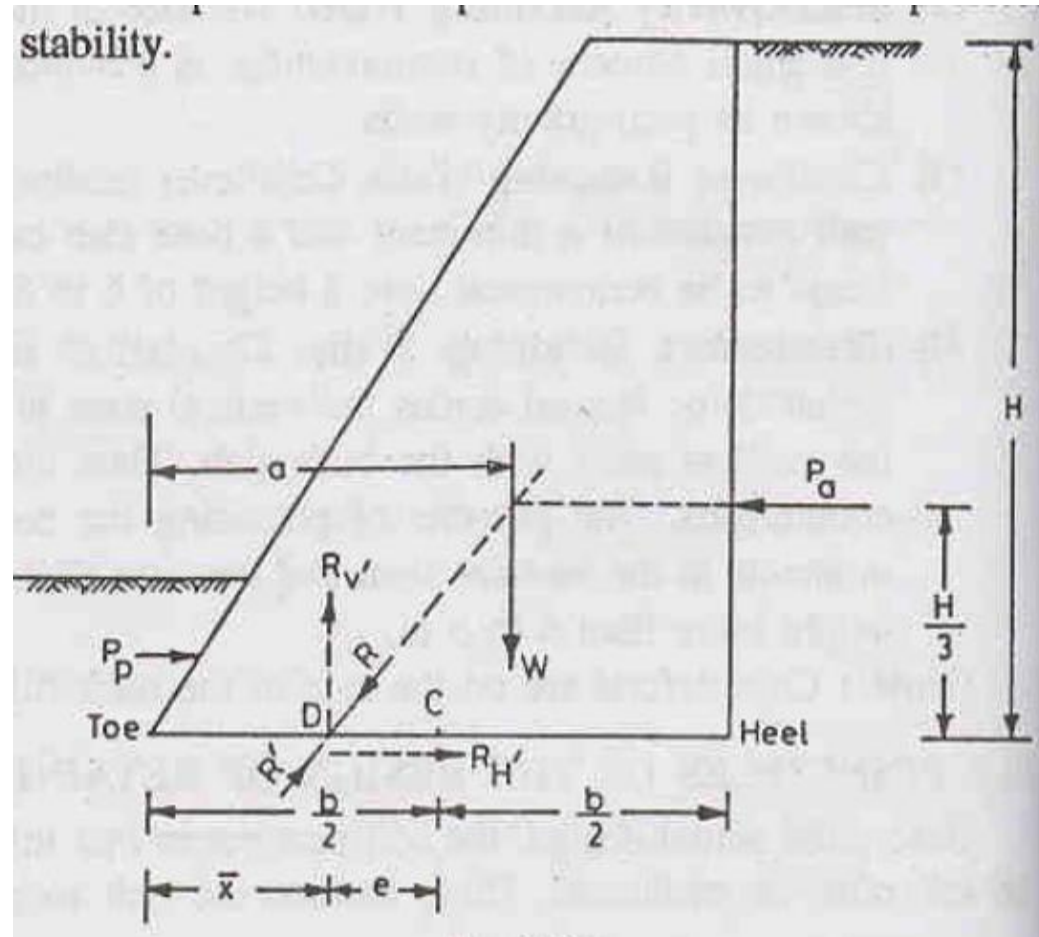
$$R_V' = W, \quad \text{and} \quad R_H' = P_a$$

$$R_V' \times \bar{x} = W \times a - P_a(H/3)$$

$$\bar{x} = \frac{W \times a - P_a \times H/3}{R_V'}$$

where $\bar{x}$ is the distance of the point D from the toe.

Thus, eccentricity, $e = b/2 - \bar{x}$



Analysis of Retaining wall

No sliding

$$\mu R_V > R_H$$

where R_V and R_H are vertical and horizontal components of R , respectively. The factor of safety against sliding is given by

$$F_s = \frac{\mu R_V}{R_H} \quad \dots(20.3)$$

where μ = coefficient of friction between the base of the wall and the soil (= $\tan \delta$).

A minimum factor of safety of 1.5 against sliding is generally recommended.

No Overturning

$$F_0 = \frac{\Sigma M_R}{\Sigma M_O} \quad \dots(20.4)$$

where ΣM_R = sum of resisting moment about toe,
and ΣM_O = sum of overturning moment about toe.

In Fig. 20.2,

$$F_0 = \frac{W \times a}{P_a \times H/3} \quad \dots(20.5)$$

The factor of safety against overturning is usually kept between 1.5 to 2.0.

Analysis of Retaining wall

No bearing capacity

The pressure distribution at the base is assumed to be linear. The maximum pressure is given by

$$p_{\max} = \frac{R_V}{b} (1 + 6e/b) \quad \dots(20.6)$$

The factor of safety against bearing failure is given by

$$F_b = \frac{q_{na}}{p_{\max}} \quad \dots(20.7)$$

where q_{na} = allowable bearing pressure.

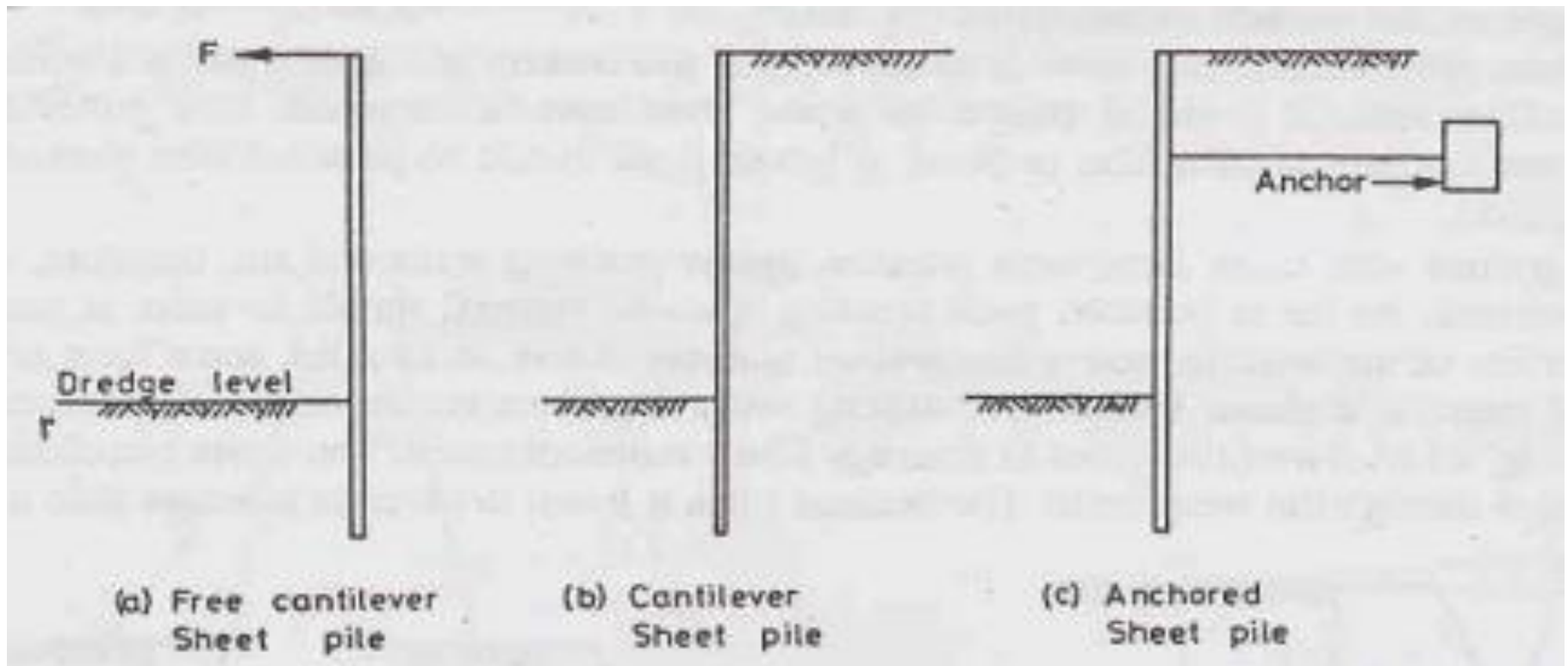
A factor of safety of 3 is usually specified, provided the settlement is also within the allowable limit.

No Tension

To avoid tension the $e < b/6$

SHEET PILES

- A sheet pile wall is an earth and water retaining structure which behaves as fixed cantilevers in resisting lateral earth pressure



FREE CANTILEVER SHEET PILES

For equilibrium, the moment of all the forces about O must be zero, i.e.

$$M_0 = F(h + d) - \left[\frac{1}{2} \gamma d (K_p - K_a) d \right] \times \frac{d}{3} = 0$$

where F is the horizontal force, h is the height of wall above the dredge level, d is the depth of embedment.

Eq. 20.20 can be solved for d . The actual depth to be provided is generally taken as 1.2 d .

The point of the maximum bending in the sheet pile can be determined as under.

The bending moment at depth x below the dredge level is given by

$$M_x = F(h + x) - \frac{\gamma x^3}{6} (K_p - K_a)$$

For maximum bending ($M_{\max}$), $\frac{d(M_x)}{dx} = 0$

or

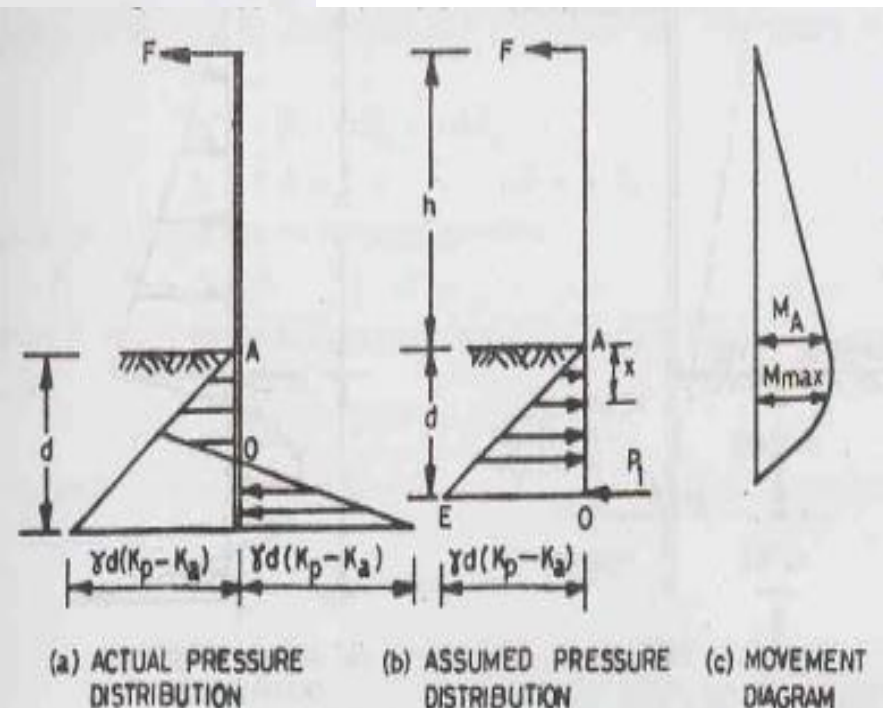
$$F - \frac{\gamma (K_p - K_a)}{6} (3x^2) = 0$$

or

$$x = \sqrt{\frac{2F}{\gamma (K_p - K_a)}}$$

The section modulus of the sheet pile can then be determined as

$$S = \frac{M_{\max}}{\sigma_a}$$



CANTILEVER SHEET PILES IN COHESIONLESS SOIL

The depth a of point O_1 of zero pressure is given by

$$p_1 - \gamma a (K_p - K_a) = 0$$

or

$$a = \frac{p_1}{\gamma (K_p - K_a)}$$

Let the total active pressure above point O_1 be P_1 acting at a height of $\bar{Z}_1$ above O_1 .

The passive pressure is given by the diagram O_1EO . The passive pressure intensity at the bottom tip A can be expressed as

$$p_2 = \gamma(K_p - K_a)(d - a) = \gamma(K_p - K_a)b$$

where $b = d - a$, in which d is the depth of point A below the dredge level.

The passive pressure is indicated by the diagram OAF on the back side. The intensity of pressure at the tip A is given by

$$p_3 = \gamma(h + d)K_p - \gamma dK_a$$

or

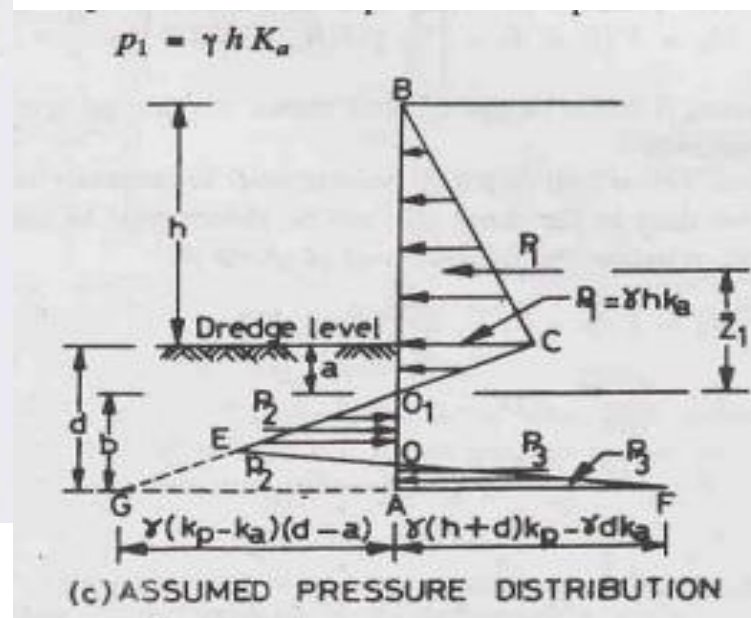
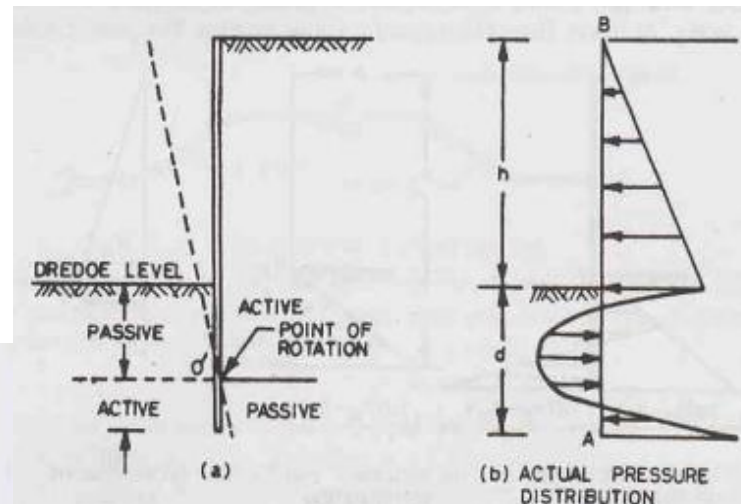
$$p_3 = \gamma(h + b + a)K_p - \gamma(b + a)K_a$$

From the equation of equilibrium in the horizontal direction,

$$P_1 + P_3 - P_2 = 0$$

The total pressure P_3 and P_2 can be expressed in terms of p_3 and p_2 as follows:

$$P_1 + \frac{1}{2}m(p_2 + p_3) - \frac{1}{2}p_2b = 0 \quad \dots(20.25)$$



CANTILEVER SHEET PILES IN COHESIONLESS SOIL

In Eq. 20.25, the equivalence of areas has been taken as shown in Fig. 20.19. The height of the point E above the tip A is taken as m .

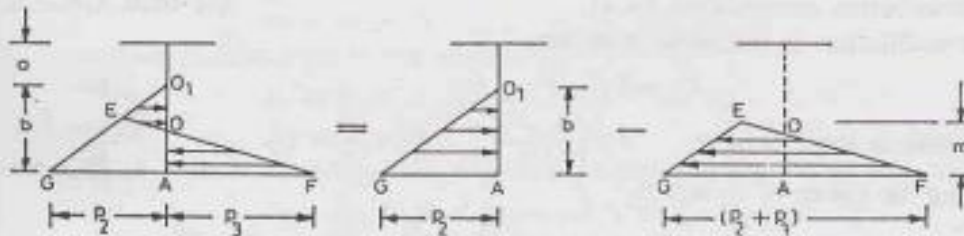


Fig. 20.19.

From Eq. 20.25,

$$m = \frac{1/2 p_2 b - P_1}{1/2 (p_2 + p_3)} = \frac{p_2 b - 2 P_1}{p_2 + p_3} \quad \dots(20.26)$$

Taking moments of all the forces about A ,

$$P_1 (b + \bar{Z}_1) - \frac{1}{2} p_2 b (b/3) + \frac{1}{2} m (p_2 + p_3) \times \frac{m}{3} = 0$$

Substituting the value of m from Eq. 20.26,

$$P_1 (b + \bar{Z}_1) - \frac{p_2 b^2}{6} + \frac{(p_2 + p_3)}{6} \left[\frac{p_2 b - 2 P_1}{p_2 + p_3} \right]^2 = 0 \quad \dots(20.27)$$

Eq. 20.27 can be written as

$$b^4 + C_1 b^3 - C_2 b^2 - C_3 b - C_4 = 0 \quad \dots(20.28)$$

where

$$C_1 = \frac{P_4}{\gamma (K_p - K_a)} \quad ; \quad C_2 = \frac{8 P_1}{\gamma (K_p - K_a)}$$

$$C_3 = \frac{6 P_1 [2 \gamma (K_p - K_a) \bar{Z}_1 + p_4]}{[\gamma (K_p - K_a)]^2} \quad ; \quad C_4 = \frac{P_1 [6 \bar{Z}_1 p_4 + 4 P_1]}{[\gamma (K_p - K_a)]^2}$$

in which

$$p_4 = \gamma h K_p + \gamma a (K_p - K_a) \quad \dots(20.29)$$

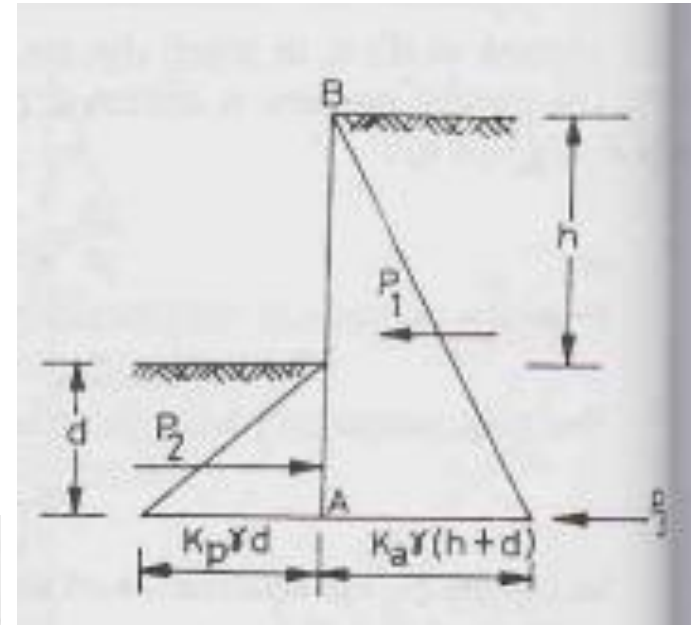
Eq. 20.28 is solved by trial and error to determine b . The value of d is equal to $(b + a)$. The depth d is for a factor of safety of unity. The required depth (D) is usually taken as $1.2 d$ to $1.4 d$. Thus

$$D = 1.2 d \text{ to } 1.4 d \quad \dots(20.30)$$

This gives a factor of safety of about 1.50 to 2.0.

CANTILEVER SHEET PILES IN COHESIONLESS SOIL

- Approximate analysis



From the equilibrium in horizontal direction,

$$P_1 - P_2 + P_3 = 0$$

Taking moments about point A, $P_1 \left(\frac{h + d}{3} \right) - P_2 \times \frac{d}{3} = 0$

Substituting the values of P_1 and P_2 ,

$$\frac{1}{2} K_a \gamma (h + d)^2 \frac{(h + d)}{3} - \frac{1}{2} K_p \gamma d^2 \times \frac{d}{3} = 0$$

or $(K_p - K_a) d^3 - 3h K_a d^2 - 3h^2 K_a d - K_a h^3 = 0$

Eq. 20.31 is solved by trial and error for d .

The value of d so obtained is usually increased by 20 to 40%. Thus

$$D = 1.2 d \text{ to } 1.4 d.$$

CANTILEVER SHEET PILES IN COHESIVE SOIL

The pressure p_1 at the dredge line on the back side is given by

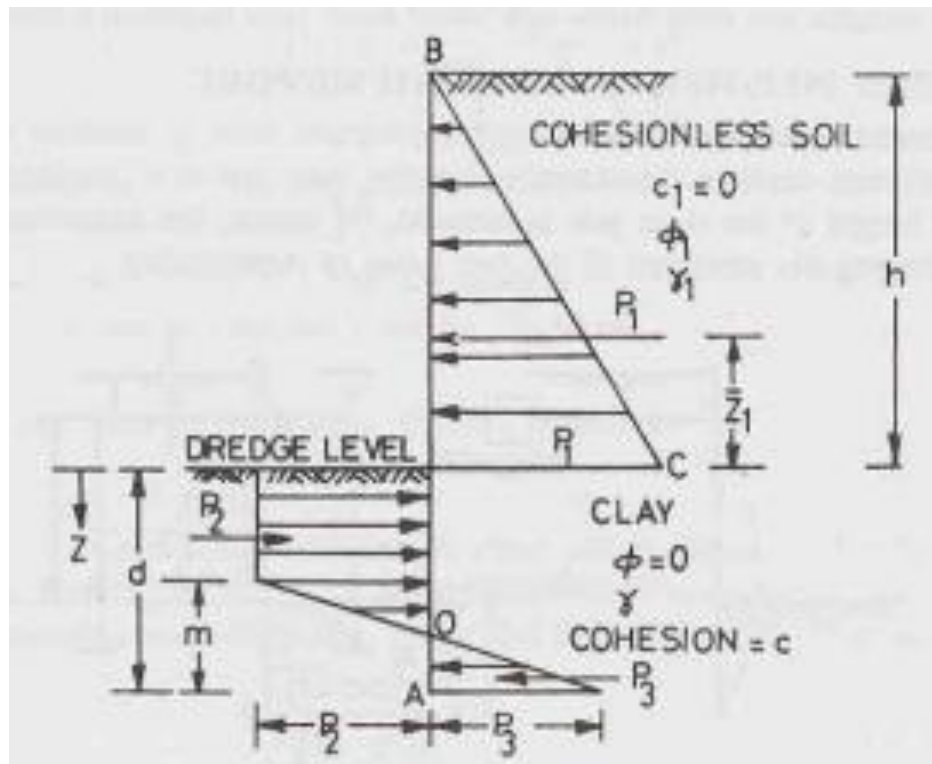
$$p_1 = \gamma_1 h K_{a1}$$

Below the dredge level but above the point of rotation O , the passive pressure acts from left to right and the active pressure acts from right to left. Therefore, the pressure at depth Z below the dredge level is given by

$$p_2 = p_p - p_a$$

or

$$p_2 = (K_p \gamma Z + 2c \sqrt{K_p}) - [K_a \gamma (Z + h) - 2c \sqrt{K_a}]$$



CANTILEVER SHEET PILES IN COHESIVE SOIL

For $\phi = 0$, we have $K_p = K_a = 1.0$.

Therefore, $p_2 = 4c - \gamma h$

Likewise, the pressure p_3 from right to left is given by

or $p_3 = K_p (h + d) \gamma + 2c \sqrt{K_p} - [K_a \gamma d - 2c \sqrt{K_a}]$

For $\phi = 0$, we have $p_3 = 4c + \gamma h$

From equilibrium in the horizontal direction, considering equivalent areas as in Fig. 20.19,

$$P_1 - [p_2 \times d] + [p_2 + p_3] \times m/2 = 0$$

or $P_1 - (4c - \gamma h) d + [8c] \times m/2 = 0$

or $m = \frac{(4c - \gamma h) d - P_1}{4c} \dots(20.32)$

Taking moments of all the forces about A,

$$P_1 (\bar{Z}_1 + d) - [4c - \gamma h] \times d \times \frac{d}{2} + \frac{1}{2} \times (8c) \times m \times \frac{m}{3} = 0 \dots(20.23)$$

Substituting the value of m from Eq. 20.32,

$$P_1 (\bar{Z}_1 + d) - (4c - \gamma h) \frac{d^2}{2} + \frac{4}{3} c \left[\frac{(4c - \gamma h) d - P_1}{4c} \right]^2 = 0 \dots(20.34)$$

The above equation can be written as

$$d^2 (4c - \gamma h) - 2P_1 d - \frac{P_1 (12c \bar{Z}_1 + P_1)}{2c + \gamma h} = 0 \dots(20.35)$$

Eq. 20.35 can be solved for d . The actual depth D is kept 40% to 60% more. Thus

$$D = 1.4 d \text{ to } 1.6 d$$

ANCHORED SHEET PILE WITH FREE-EARTH SUPPORT

(a) Cohesionless Soils.

Fig. 20.22 (a) shows the forces acting on the pile, assuming that the material above and below the dredge level is cohesionless.

From equilibrium, $T + P_2 - P_1 = 0$

where T is the tensile force in anchor.

The depth a to the point of zero pressure can be determined as under.

$$\gamma K_a (h + a) - \gamma K_p a = 0$$

$$\text{or} \quad a \gamma (K_p - K_a) = \gamma K_a h$$

$$\text{or} \quad a = \frac{h K_a}{(K_p - K_a)}$$

$$\text{Therefore,} \quad P_2 = \frac{1}{2} p_2 b = \frac{1}{2} \gamma (K_p - K_a) b^2$$

$$\text{where} \quad p_2 = \gamma (K_p - K_a) b$$

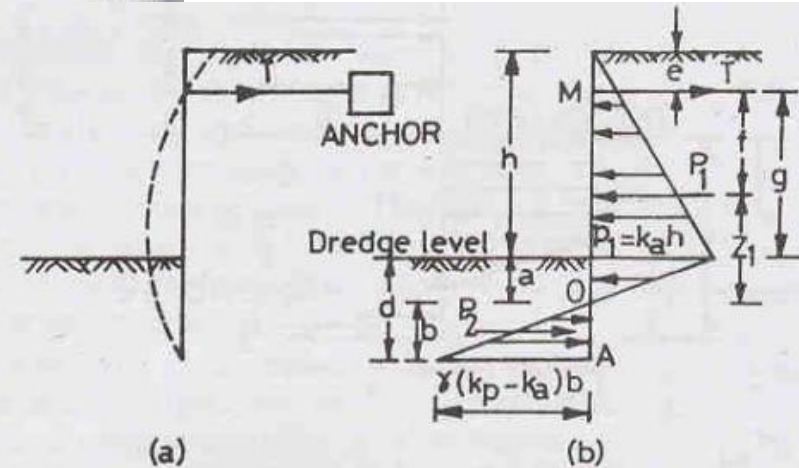
Taking moments of all the forces about anchor point M ,

$$P_1 (a + h - e - \bar{Z}_1) - P_2 (h - e + a + 2b/3) = 0$$

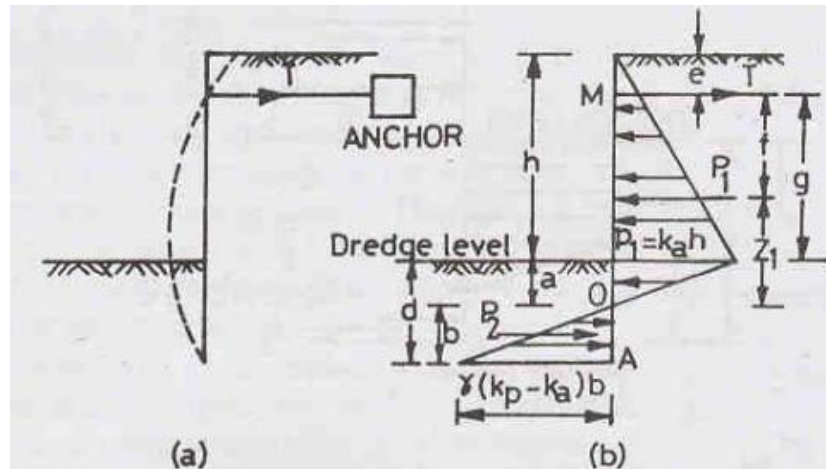
The distance $\bar{Z}_1$ is determined as in the case of cantilever piles.

Substituting the value of P_2 from Eq. (a) in Eq. 20.38 (a),

$$P_1 (a + h - e - \bar{Z}_1) - \gamma (K_p - K_a) b \times b/2 (h - e + a + 2b/3) = 0 \quad \dots [20.38(b)]$$



ANCHORED SHEET PILE WITH FREE-EARTH SUPPORT



The above equation can be written as

$$b^3 (K_p - K_a) \gamma / 3 + b^2 (K_p - K_a) \gamma / 2 (g + a) - P_1 f = 0$$

or

$$b^3 + 1.5b^2 (g + a) - \frac{3 P_1 f}{\gamma (K_p - K_a)} = 0$$

where

$$f = a + h - e - \bar{Z}_1 \quad \text{and} \quad g = h - e$$

Eq. 20.40 can be solved for b . Then d is determined as

$$d = b + a$$

The actual depth D is taken equal to 1.2 to 1.4 times d .

The force in anchor rod can be obtained from Eq. 20.36 as,

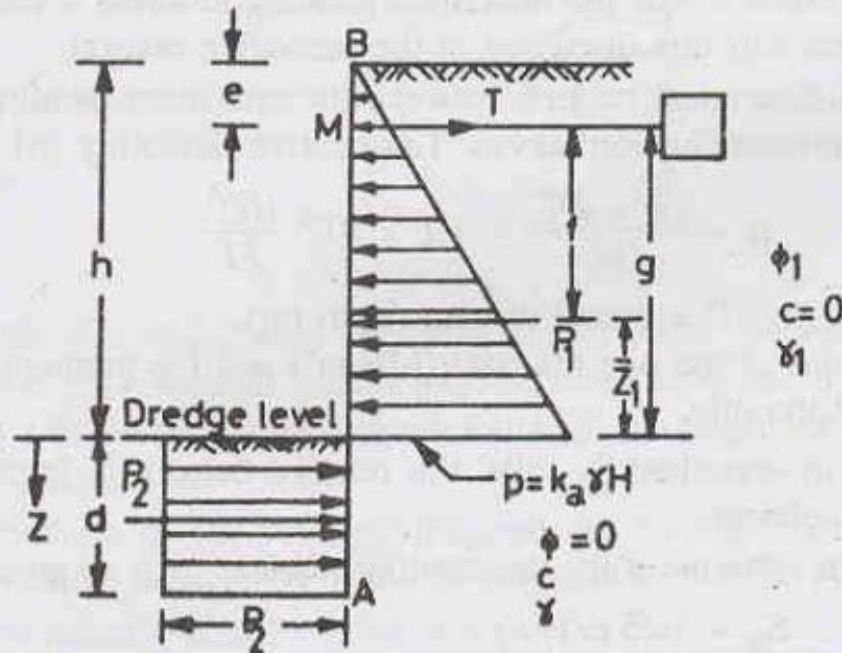
$$T = P_1 - P_2$$

The values of P_1 and P_2 are obtained from pressure diagrams.

ANCHORED SHEET PILE WITH FREE-EARTH SUPPORT

(b) Cohesive Soils

Let us now consider the case when the anchored sheet pile is driven in clay ($\phi = 0$), but has the backfill of cohesionless, granular material (Fig. 20.23). The pressure distribution above the dredge line is the same as that in the case of cohesionless soils. However, below the dredge line, the pressure is given by



ANCHORED SHEET PILE WITH FREE-EARTH SUPPORT

For $\phi = 0$,

$$p_2 = (K_p \gamma Z + 2c \sqrt{K_p}) - [K_a(Z + h) \gamma - 2c \sqrt{K_a}]$$

$$K_p = K_a = 1.0. \text{ Therefore}$$

$$p_2 = 2c + 2c - \gamma h = 4c - \gamma h$$

From equilibrium of forces,

$$P_1 - P_2 = T$$

or

$$P_1 - p_2 \times d = T$$

Taking moments of all forces about M ,

$$P_1 \times f - p_2 d (g + d/2) = 0$$

Substituting $p_2 = 4c - \gamma h$,

$$P_1 \times f - (4c - \gamma h) d (g + d/2) = 0$$

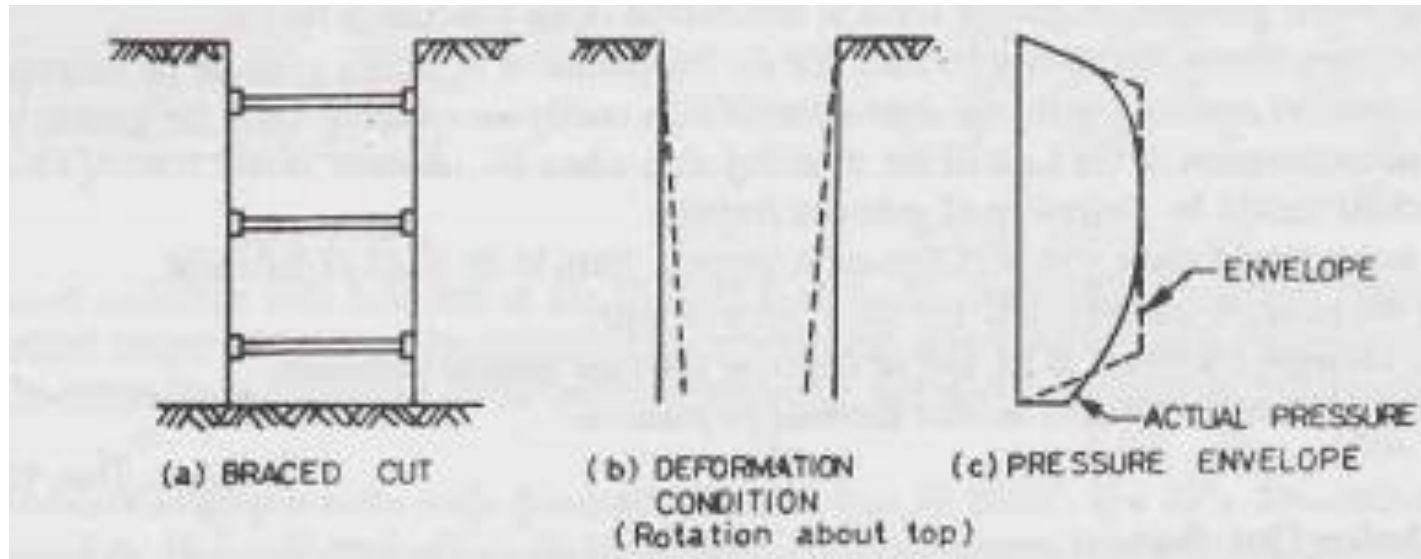
or

$$d^2 + 2gd - \frac{2P_1 f}{4c - \gamma H}$$

can be solved for d . The actual depth (D) provided is 20 to 40% more than d .

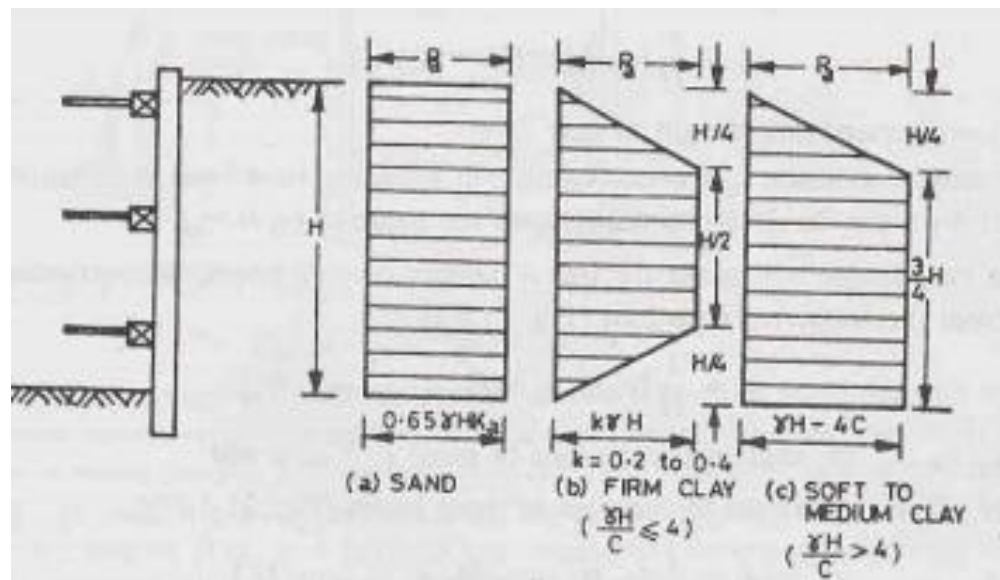
Braced cuts

- Deep excavation with vertical sides require lateral supports to prevent cave-in of the earth and to protect the adjacent areas against ground subsidence and lateral movement of the subsoil



Lateral earth pressure on sheetings

- Deep excavation with vertical sides require lateral supports to prevent cave-in of the earth and to protect the adjacent areas against ground subsidence and lateral movement of the subsoil



Non-uniform soils

1. When the braced cut passes through layers of both sand and clay [Fig. 21.4 (a)], an equivalent value of cohesion c_e ($\phi = 0$ concept) is determined using the following equation suggested by Peck (1943).

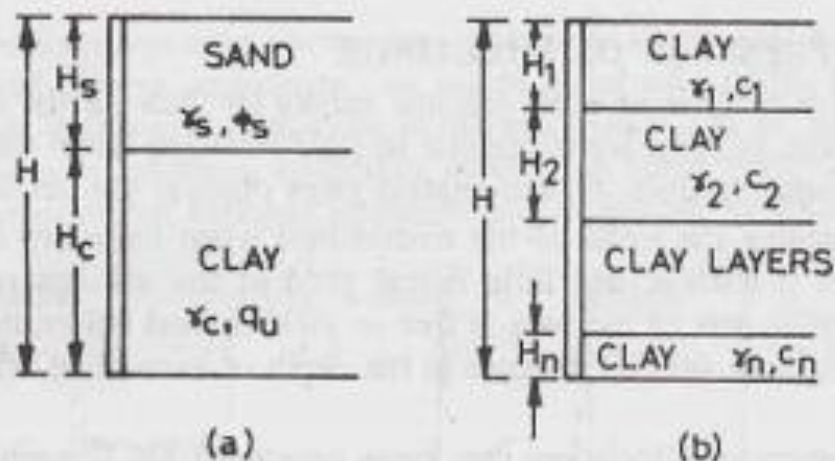


Fig. 21.4.

$$c_e = \frac{1}{2H} [\gamma_s K_s H_s^2 \tan \phi_s + (H - H_s) n' q_u] \quad \dots(21.2)$$

where H = total height of the cut, γ_s = unit weight of sand, H_s = height of the sand layer, K_s = a lateral earth pressure coefficient (≈ 1), ϕ_s = angle of friction of sand, q_u = unconfined compressive strength of clay, n' = a coefficient of progressive failure (average value 0.75).

The equivalent unit weight γ_e of the layers is determined from the following equation:

$$\gamma_e = \frac{1}{H} [\gamma_s H_s + (H - H_s) \gamma_c] \quad \dots(21.3)$$

where γ_c = saturated unit weight of clay layer.

Once the equivalent cohesion c_e and equivalent unit weight γ_e have been determined, the diagram in Fig. 21.3 (b) or Fig. 21.3 (c) can be used depending upon the value of $(\gamma_e H / c_e)$.

2. When the braced cut passes through a number of clay layers, the equivalent values of c_e and γ_e are determined from the following equations [Fig. 21.4 (b)].

$$c_e = \frac{1}{H} [c_1 H_1 + c_2 H_2 + \dots + c_n H_n] \quad \dots(21.4)$$

where $c_1, c_2 \dots c_n$ are undrained cohesions of layer 1, 2 ... n and

$H_1, H_2 \dots H_n$ are the thicknesses of these layers [Fig. 21.4 (b)].

Likewise,
$$\gamma_e = \frac{1}{H} [\gamma_1 H_1 + \gamma_2 H_2 + \dots + \gamma_n H_n] \quad \dots[21.5]$$

Types of sheeting and bracing systems

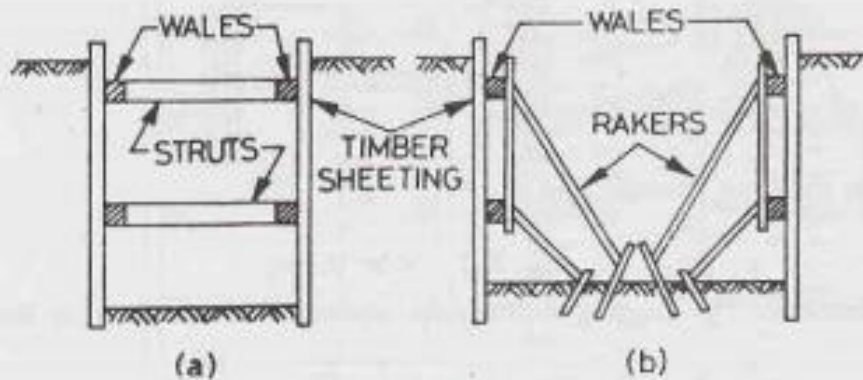


Fig. 21.5. Vertical Timber Sheeting.

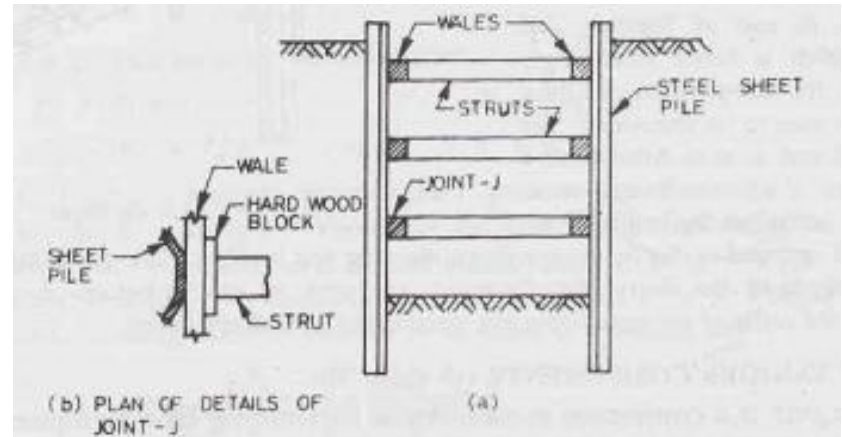


Fig. 21.6. Steel Sheet Piles.

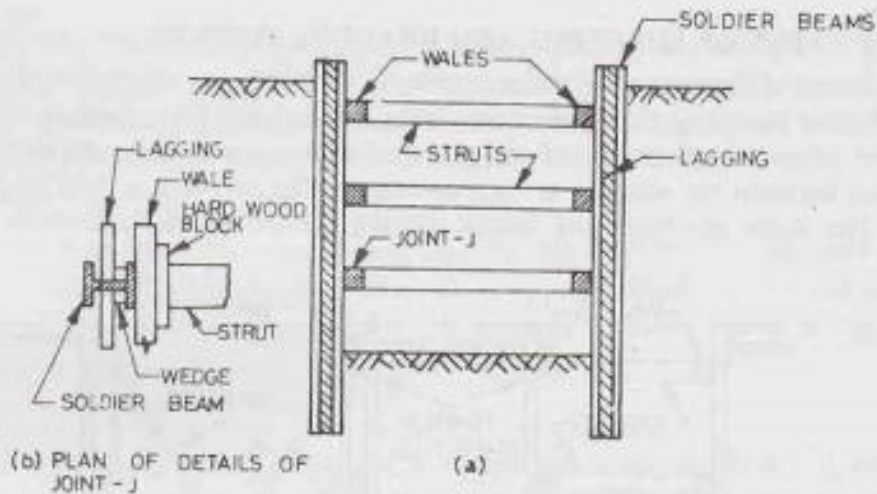


Fig. 21.7. Soldier Beams.

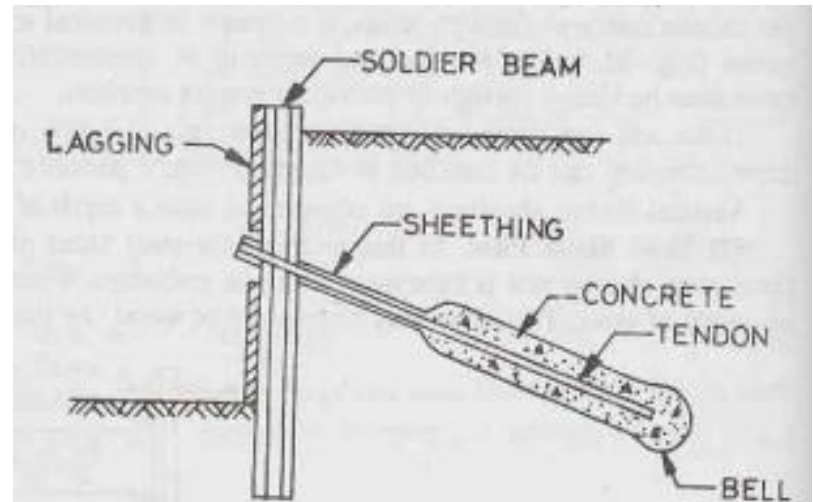


Fig. 21.8. Tie Back.

Practice problem

Illustrative Example 20.1. Check the stability of the gravity retaining wall shown in Fig. E-20.1. Take allowable soil pressure equal to 600 kN/m^2 . Use Coulomb's theory.

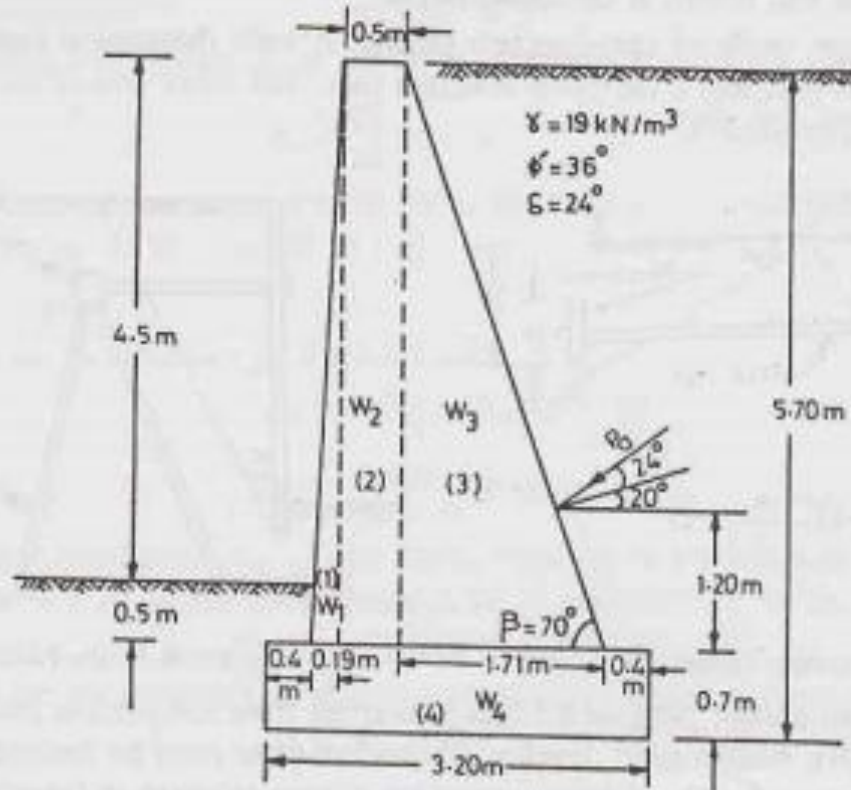


Fig. E-20.1.

$$K_a = \frac{\sin^2(\beta + \phi')}{\sin^2 \beta \sin(\beta - \delta) \left[1 + \sqrt{\frac{\sin(\phi' + \delta) \sin(\phi' - i)}{\sin(\beta - \delta) \sin(\beta + i)}} \right]^2}$$

$$\text{or } K_a = \frac{\sin^2(70^\circ + 36^\circ)}{\sin^2 70^\circ \sin(70^\circ - 24^\circ) \left[1 + \sqrt{\frac{\sin(36^\circ + 24^\circ) \sin(36^\circ - 0^\circ)}{\sin(70^\circ - 24^\circ) \sin(70^\circ + 0^\circ)}} \right]^2}$$

$$\text{or } K_a = \frac{0.924}{0.883 \times 0.719 \left[1 + \sqrt{\frac{0.866 \times 0.588}{0.719 \times 0.940}} \right]^2} = 0.417$$

$$\text{Therefore, } P_a = \frac{1}{2} \times 0.417 \times 19 \times (5.7)^2 = 128.7 \text{ kN}$$

The total pressure acts inclined at 24° to the normal.

$$\text{Horizontal component, } P_h = P_a \cos(20^\circ + 24^\circ) = 92.6 \text{ kN}$$

$$\text{Vertical component, } P_v = P_a \sin(20^\circ + 24^\circ) = 89.4 \text{ kN}$$

Calculations are shown in the tabular form. The moment are taken about toe. The clockwise moments are taken as positive. The unit weight of concrete is taken as 24 kN/m^3 .

S.No.	Description	Forces (kN)		Lever arm	Moments about toe (kN-m)	
		Vertical (kN)	Horizontal (kN)		Clockwise	Counter Clockwise
1.	Weight, W_1 $= 1/2 \times 5 \times 0.19 \times 24$	11.4		0.53 m	6.0	
2.	Weight, W_2 $= 5 \times 0.5 \times 24$	60		0.84 m	50.4	
3.	Weight, W_3 $= 1/2 \times 5 \times 1.71 \times 24$	102.6		1.66 m	170.3	
4.	Weight, W_4 $= 3.2 \times 0.7 \times 24$	53.8		1.60 m	86.1	
5.	Vertical component of $P = P_{av}$	89.4		2.39 m	213.6	
6.	Horizontal component of $P = P_{ah}$		92.6	1.90 m		175.9
		$\Sigma 317.2$	$\Sigma 92.6$		$\Sigma 526.4$	$\Sigma 175.9$

$$\Sigma M = 350.5 \text{ kN-m}$$

Neglecting passive resistance, the factor of safety against sliding is given by Eq. 20.3 as

$$F_s = \frac{\mu R_v}{R_H} = \frac{\tan 24^\circ \times 317.2}{92.6} = 1.53 \text{ (safe)}$$

The factor of safety against overturning is obtained from Eq. 20.4,

$$F_o = \frac{\Sigma M_R}{\Sigma M_o} = \frac{526.4}{175.9} = 2.99 \text{ (safe)}$$

From Eq. 20.17,

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{350.5}{317.2} = 1.10 \text{ m}$$

From Eq. 20.18,

$$e = b/2 - \bar{x} = 1.60 - 1.10 = 0.50 \text{ m}$$

As $e < b/6$, there is no tension.

The pressure at the base are determined from Eq. 20.19.

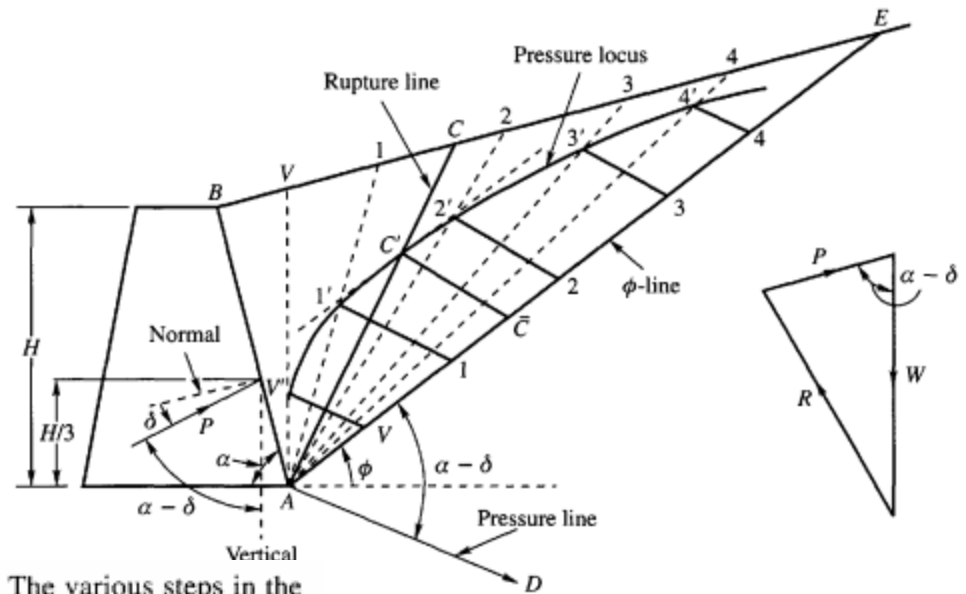
$$p_{\max} = \frac{\Sigma V}{b} \left(1 + \frac{6e}{b} \right) = \frac{317.2}{3.20} \left(1 + \frac{6 \times 0.5}{3.2} \right) = 192 \text{ kN/m}^2$$

$$p_{\min} = \frac{\Sigma V}{b} \left(1 - \frac{6e}{b} \right) = \frac{317.2}{3.20} \left(1 - \frac{6 \times 0.5}{3.2} \right) = 6.2 \text{ kN/m}^2$$

The factor of safety against bearing capacity failure is given by Eq. 20.7 as

$$F_b = \frac{q_{\text{net}}}{p_{\max}} = \frac{600}{192} = 3.1 \text{ (safe)}$$

Active earth pressure by Culmann's



In Fig. 11.19(a) AB is the retaining wall drawn to a suitable scale. The various steps in the construction of the pressure locus are:

1. Draw ϕ -line AE at an angle ϕ to the horizontal.
2. Lay off on AE distances, AV, A1, A2, A3, etc. to a suitable scale to represent the weights of wedges ABV, AB1, AB2, AB3, etc. respectively.
3. Draw lines parallel to AD from points V, 1, 2, 3 to intersect assumed rupture lines AV, A1, A2, A3 at points V', 1', 2', 3', etc. respectively.
4. Join points V', 1', 2', 3', etc. by a smooth curve which is the pressure locus.
5. Select point C' on the pressure locus such that the tangent to the curve at this point is parallel to the ϕ -line AE.
6. Draw C'C parallel to the pressure line AD. The magnitude of C'C in its natural units gives the active pressure P_a .
7. Join AC' and produce to meet the surface of the backfill at C. AC is the rupture line.

SLOPE STABILITY

Slopes

- Slopes are typically categorized in two types: natural and artificially-made slopes.
- Natural slopes are formed due to physical processes that include plate tectonics and weathering/erosion of rock masses that result in material deposition.
- Artificially-made slopes are established to facilitate infrastructure projects, ex., embankments, earth dams, road cuttings etc.



Types of Slope failure

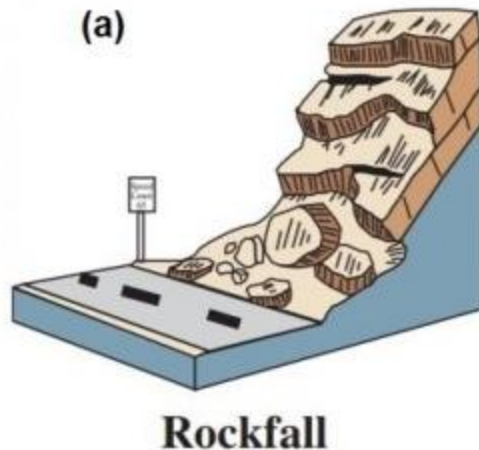
As per Varnes (1978) five types of landslide movement can be observed.

- Falls
- Topples
- Slides
- Lateral spreads
- Flows

Types of Slope failure

- **Falls**

Falls are downward movements that progress rapidly and may not be preceded by initial movements or warnings. They occur when a rocky mass is detached from a slope along a discontinuity plane that can be associated with fractures, joints or bedding.

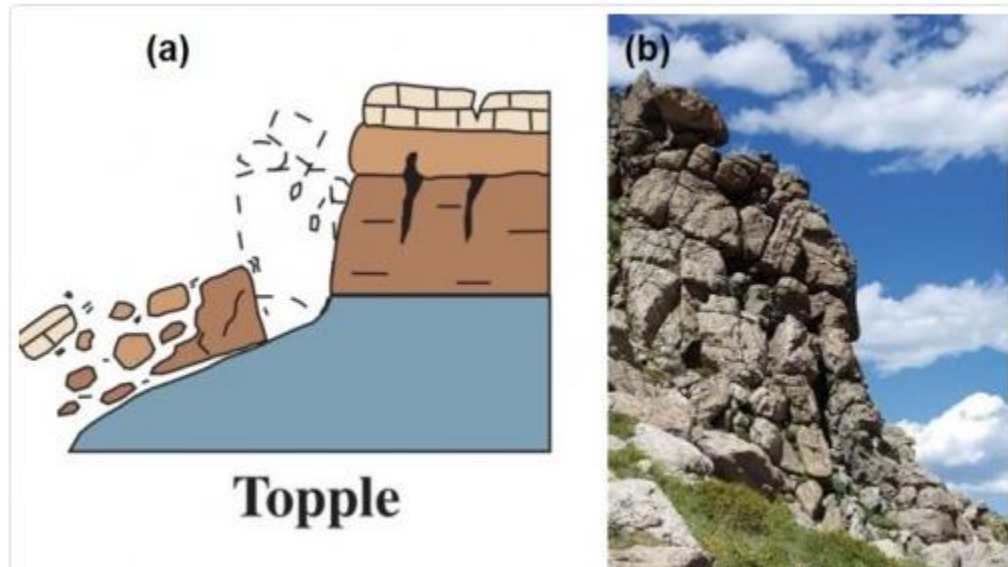


Types of Slope failure

- **Topples**

Topples are also failures that occur in rocky materials and resemble falls. However, this type of failure is associated with a rotational movement that occurs around a certain point located in a relatively low position. Topples are controlled by a combined action of gravitational forces that induce a bending moment and external forces (e.g., weathering, water pressure, freeze-thaw cycles).

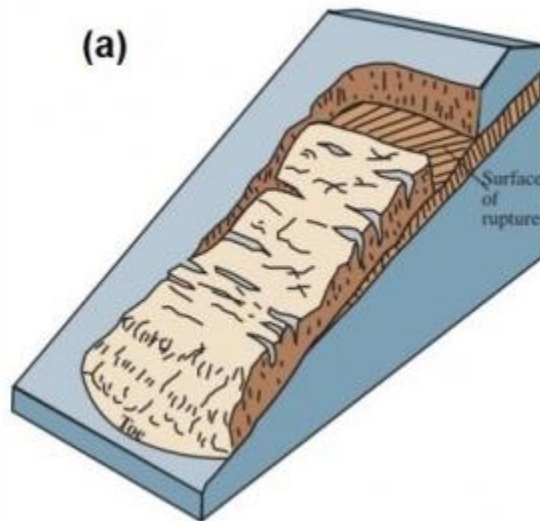
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Types of Slope failure

Slides

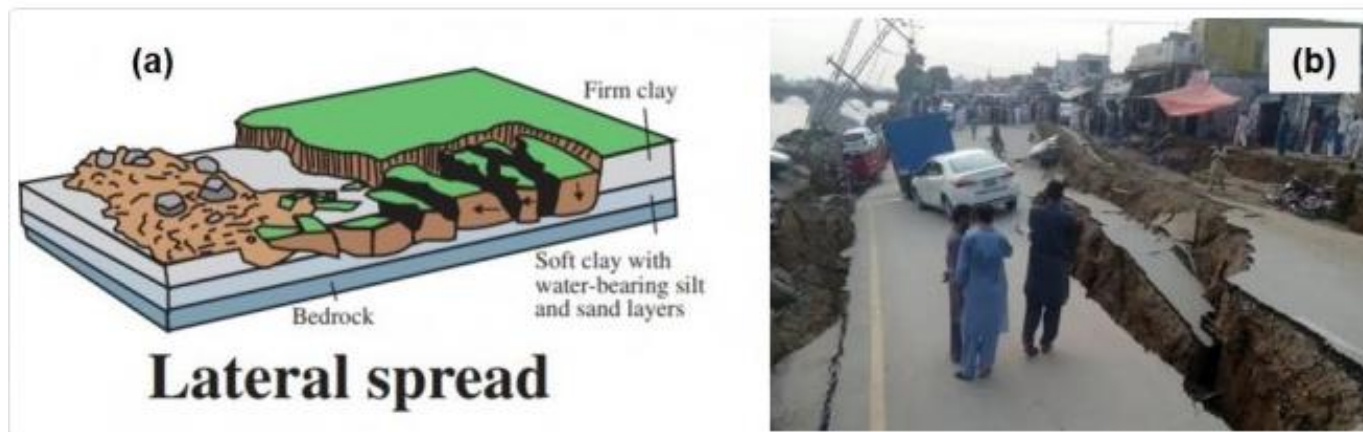
- A slide occurs when the shear stress applied along a surface overcomes its shear strength. The failure may propagate progressively from a local failure initiation.
- There are two types of slides: **rotational slide** and **translational slide**.



Types of Slope failure

Lateral spreads

- Lateral spreads are deformational phenomena caused by liquefaction, the process during which a saturated soil (usually sands) experiences loss of strength after a sudden change in its initial stress conditions.



Types of Slope failure

Flows

There are certain landslide phenomena that take the form of slow or fast-moving flows. In rocks, there are types of slow movements that result in folding or bending. Due to the fact that these displacements resemble viscous fluids, they can be characterized as rock flows.



Factor of safety for slope

- The **Factor of Safety (FoS or FS)** is defined as the ratio between the aforementioned two components, as

$$FoS = \frac{\textit{Resistance Forces}}{\textit{Driving Forces}}$$

Types of slope stability methods

- Slope stability analysis is widely used in soil mechanics and geotechnical engineering through many techniques such as
 - empirical approaches (ex., Q-slope, SMR),
 - limit equilibrium methods,
 - finite element or finite difference methods
 - discrete element analysis.

Limit Equilibrium Method

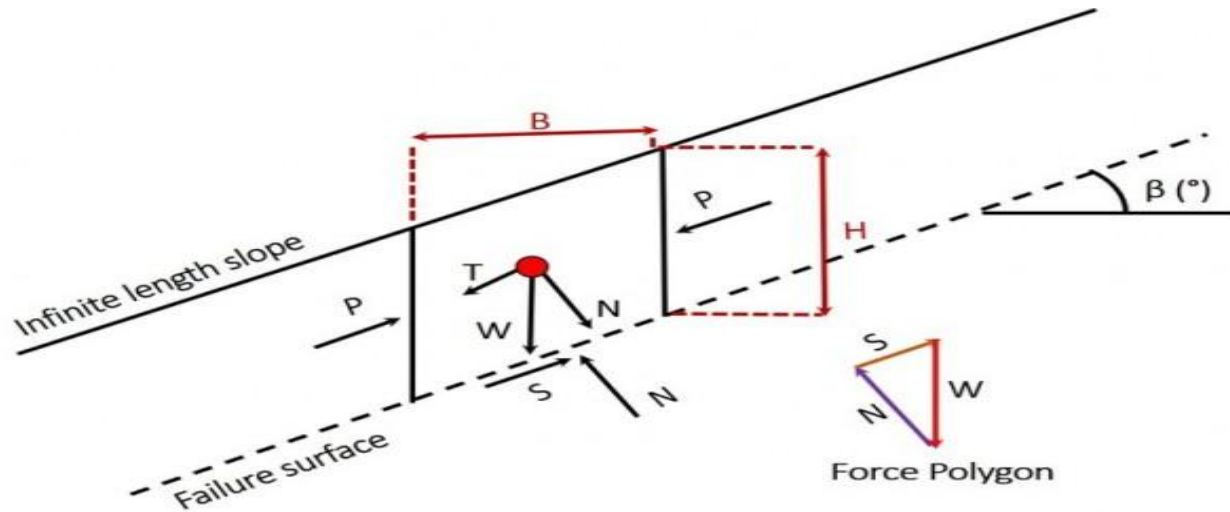
- The most widely used, practical approach for both 2-D and 3-D slope is the **Limit Equilibrium Method (LEM)**.
- The LEM method is assessing the stability of a slope by computing its FoS.
- LEM differs from kinematic analyses since it investigates whether the slope will fail or not but it cannot predict the magnitude or the velocity of landslide deformations.
- It is an adequate method for most slope stability problems but it may be insufficient in certain cases (e.g., in progressive failures, liquefaction or creep) in which more advanced numerical techniques are required.

Limit Equilibrium Method

To employ the LEM, the following procedure is followed:

- *Definition of the failure surface*
- *Derivation of driving and resistance forces*
- *Calculation of the Factor of Safety*

Stability of Infinite slope



- Cohesionless soil

The weight of the analyzed section with height H and width B is calculated as:

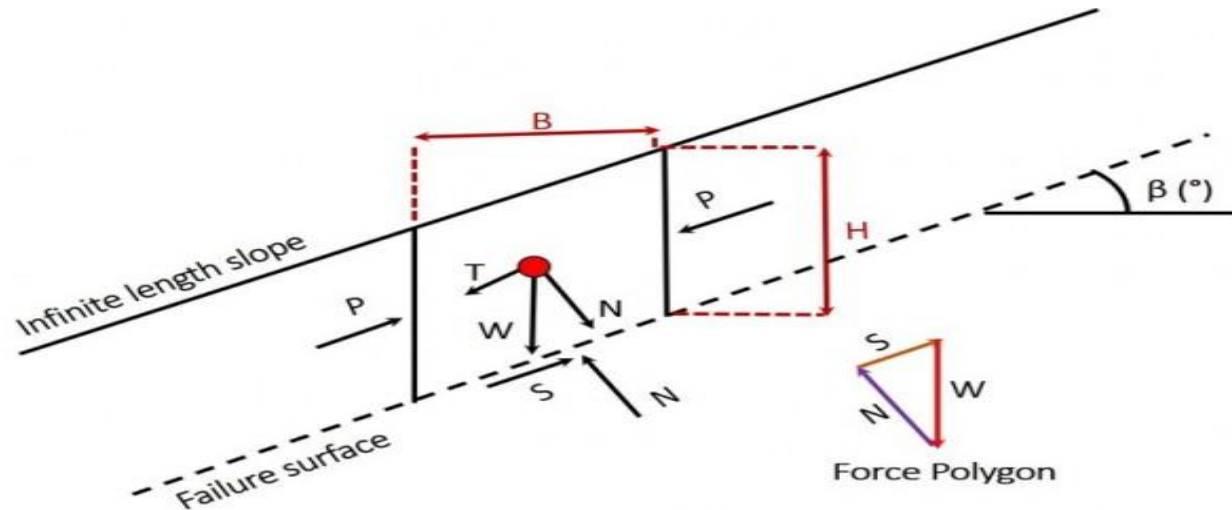
$$W = \gamma_{dry} * B * H \left(\frac{kN}{m} \right) \quad [1]$$

where γ refers to the unit weight (kN/m^3) of the soil. Given the inclination of the slope (β), the weight is analyzed into two components, as shown in **Figure 1**.

$$N = W \cos(\beta) \quad [2]$$

$$T = W \sin(\beta) \quad [3]$$

Stability of Infinite slope



- Cohesionless soil

The vertical reaction acting on the section from the underlying soil is equal to N , while the maximum shear stress that can be developed along the failure plane is:

$$S = N \tan(\varphi) \quad [4]$$

where φ is the friction angle of the soil.

The driving force of the slope movement is the component of the weight parallel to the potential failure surface (T), while the resisting force is the shear strength of the ground (S). Hence:

$$FoS = \frac{S}{T} = \frac{N \tan(\varphi)}{W \sin(\beta)} \xrightarrow{\text{Eq. 4}} FoS = \frac{W \cos(\beta) \tan(\varphi)}{W \sin(\beta)} = \frac{\tan(\varphi)}{\tan(\beta)} \quad [5]$$

Stability of Infinite slope

At failure,

$$s = c + \sigma_n \tan \phi$$

$$s = \tau_f = c + \sigma_{nf} \tan \phi$$

But

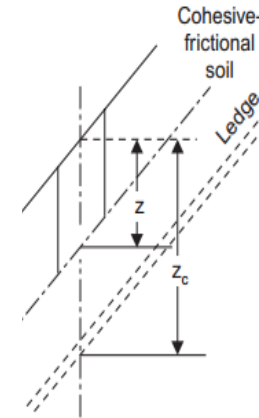
$$\sigma_{nf} = \gamma \cdot z_c \cdot \cos^2 \beta$$

$$\tau_f = \gamma z_c \cdot \sin \beta \cos \beta, \text{ from Eqs. 9.2 and 9.3.}$$

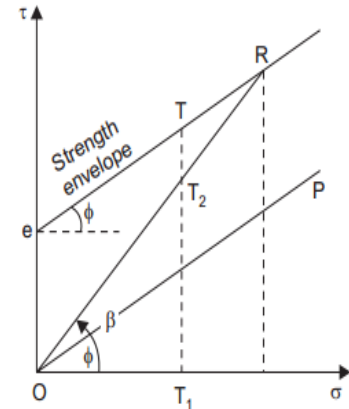
$$\therefore \gamma z_c \cdot \sin \beta \cos \beta = c + \gamma z_c \cdot \cos^2 \beta \cdot \tan \phi$$

$$z_c \gamma \cos \beta (\sin \beta - \cos \beta \tan \phi) = c$$

$$\therefore z_c = (c/\gamma) \cdot 1/[\cos^2 \beta (\tan \beta - \tan \phi)]$$



(a) Infinite slope in cohesive-frictional soil-critical depth



(b) Relation between strength envelope and angle of slope

$$\frac{c}{\gamma z_c} = \cos^2 \beta (\tan \beta - \tan \phi)$$

The quantity $\frac{c}{\gamma z_c}$ is called the stability number S_n .

For any depth z less than z_c , the factor of safety

$$F = \frac{\text{Shearing strength}}{\text{Shearing stress}}$$

$$F = \frac{c + \gamma z \cos^2 \beta \cdot \tan \phi}{\gamma z \cos \beta \cdot \sin \beta}$$

$\therefore$

$$F_c = \frac{c}{c_m}, \text{ where } c_m = \text{mobilised cohesion, at depth } z,$$

$$S_n = c/\gamma z_c = c_m/\gamma z = c/F_c \cdot \gamma z = \cos^2 \beta (\tan \beta - \tan \phi)$$

Cohesive soil

Stability of Infinite slope

Normal stress

$$\sigma'_n = (\gamma_{sat} - \gamma_w)H \cos^2 \beta = \gamma_b H \cos^2 \beta$$

the shearing stress

$$\tau = \gamma_{sat} H \sin \beta \cos \beta$$

Now substituting Eqs (10.11a) and (10.11b) into equation

$$\tau = c'_m + \sigma'_n \tan \phi'_m$$

and simplifying, the stability expression obtained is

$$\frac{c'_m}{\gamma_{sat} H} = \cos^2 \beta \tan \beta - \frac{\gamma_b}{\gamma_{sat}} \tan \phi'_m \quad (10.12)$$

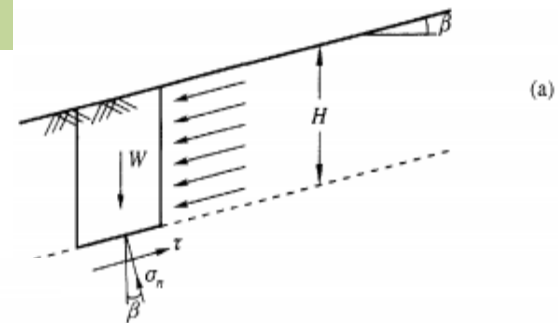
As before, if the factor of safety with respect to friction is unity, the stability number which represents the cohesion may be written as

$$N_s = \frac{c'}{F_c \gamma_{sat} H} = \frac{c'}{\gamma_{sat} H_c} = \cos^2 \beta \tan \beta - \frac{\gamma_b}{\gamma_{sat}} \tan \phi' \quad (10.13)$$

If the slope is completely submerged, and if there is no seepage as in Fig. 10.4(b), then Eq. (10.13) becomes

$$N_s = \frac{c'}{F_c \gamma_b H} = \frac{c'}{\gamma_b H_c} = \cos^2 \beta (\tan \beta - \tan \phi') \quad (10.14)$$

where γ_b = submerged unit weight of the soil.



(10.11b)

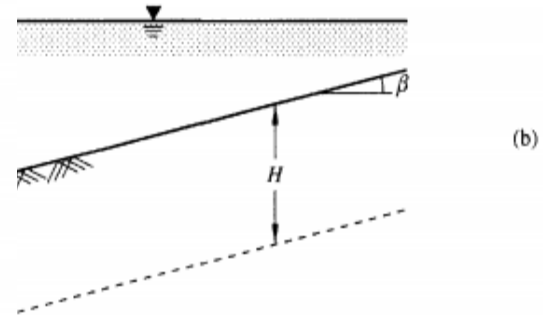


Fig. 10.4 Stability of infinite slope (a) with seepage flow through the entire mass, and (b) with completely submerged slope.

Problem 1

- Find the factor of safety of a slope of infinite extent having a slope of 25 degree. The slope is of cohesionless soil with angle of friction $=30$ degree.
- If it is made of clay having $C'=30$ kN/m², $\phi = 20$ degree, $e= 0.65$ and $G = 2.7$ and under the following conditions (i) when soil is dry (ii) when water seeps parallel to the surface of the slope (iii) when the slope is submerged.

For $e = 0.65$ and $G_s = 2.7$

$$\gamma_d = \frac{2.7 \times 9.81}{1 + 0.65} = 16.05 \text{ kN/m}^3, \quad \gamma_{\text{sat}} = \frac{(2.7 + 0.65) \times 9.81}{1 + 0.65} = 19.9 \text{ kN/m}^3,$$

$$\gamma_b = 10.09 \text{ kN/m}^3$$

(i) For dry soil the stability number N_s is

$$N_s = \frac{c'}{\gamma_d H_c} = \cos^2 \beta (\tan \beta - \tan \phi') \quad \text{when } F_\phi = 1$$

$$= (\cos 25^\circ)^2 (\tan 25^\circ - \tan 20^\circ) = 0.084.$$

$$\text{Therefore, the critical height } H_c = \frac{c'}{\gamma_d \times N_s} = \frac{30}{16.05 \times 0.084} = 22.25 \text{ m}$$

(ii) For seepage parallel to the surface of the slope [Eq. (10.13)]

$$N_s = \frac{c'}{\gamma_t H_c} = \cos^2 25^\circ \tan 25^\circ - \frac{10.09}{19.9} \tan 20^\circ = 0.2315$$

$$H_c = \frac{c'}{\gamma_t N_s} = \frac{30}{19.9 \times 0.2315} = 6.51 \text{ m}$$

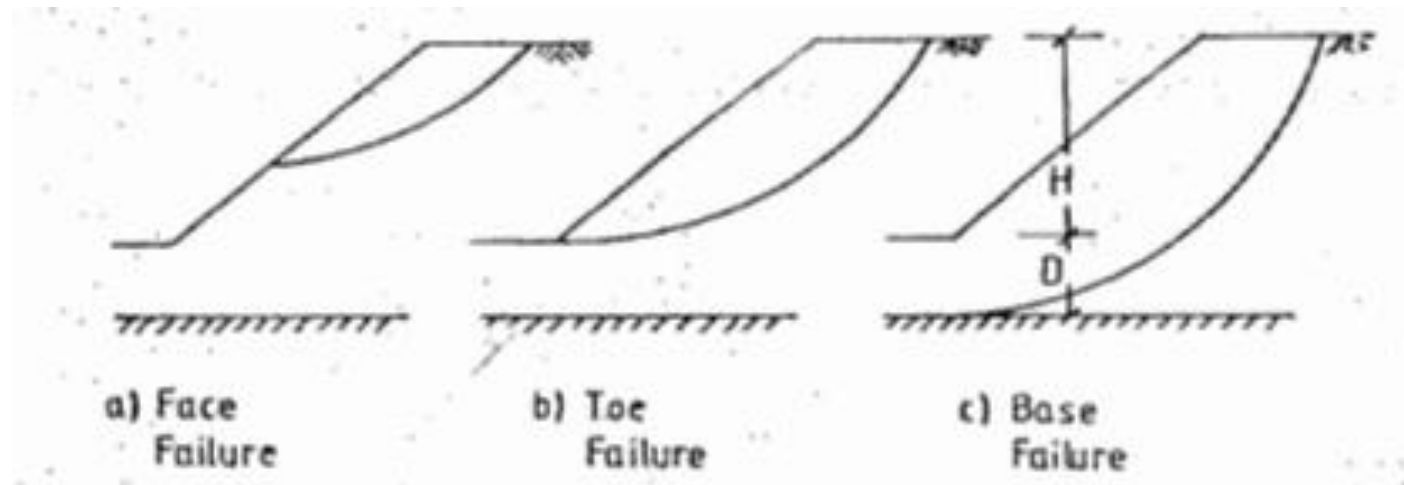
(iii) For the submerged slope [Eq. (10.14)]

$$N_s = \cos^2 25^\circ (\tan 25^\circ - \tan 20^\circ) = 0.084$$

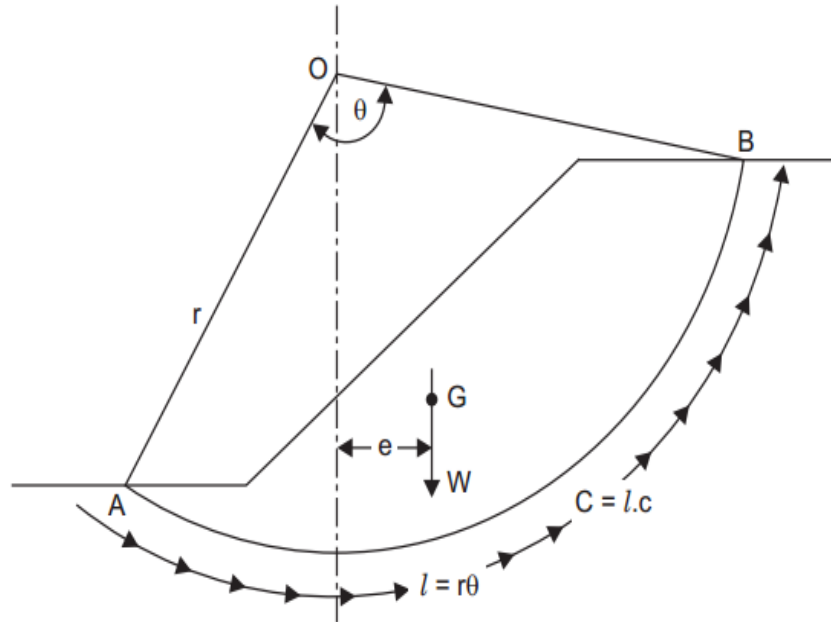
$$H_c = \frac{c'}{\gamma_b N_s} = \frac{30}{10.09 \times 0.084} = 35.4 \text{ m}$$

Stability of finite slope

- A 'finite slope', as has already been defined, is one with a base and a top surface, the height being limited.
- The inclined faces of earth dams, embankments, excavations and the like are all finite slopes.



Total stress analysis for a purely cohesive



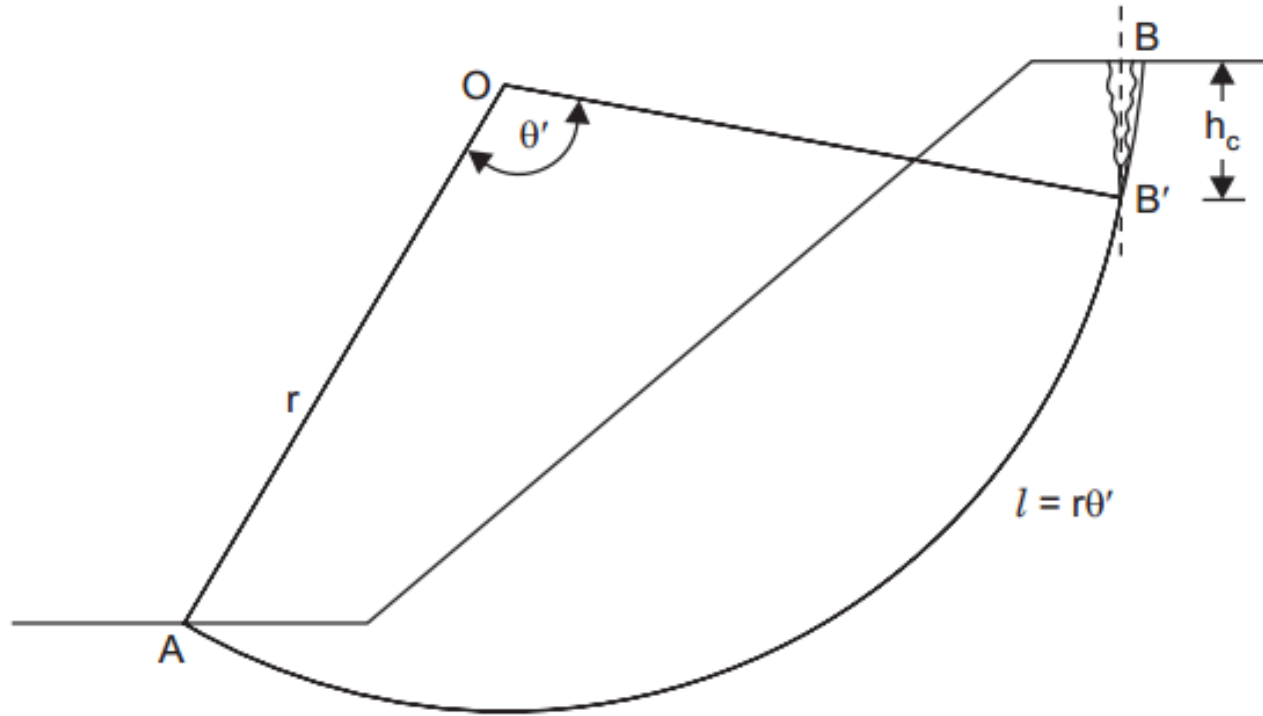
Let W be the weight of the soil within the slip surface and G the position of its centre of gravity. The shearing strength of the soil is c , since $\phi = 0^\circ$.

Taking moments about O , the centre of rotation:

$W.e. = c.l.r = c.r.\theta.r = cr^2.\theta$, for equilibrium (incipient failure).

Factor of safety,
$$F = \frac{\text{Restraining moment}}{\text{Sliding moment}} = \frac{c.r^2.\theta}{W.e} \quad \dots(\text{Eq. 9.17})$$

Total stress analysis for a purely cohesive



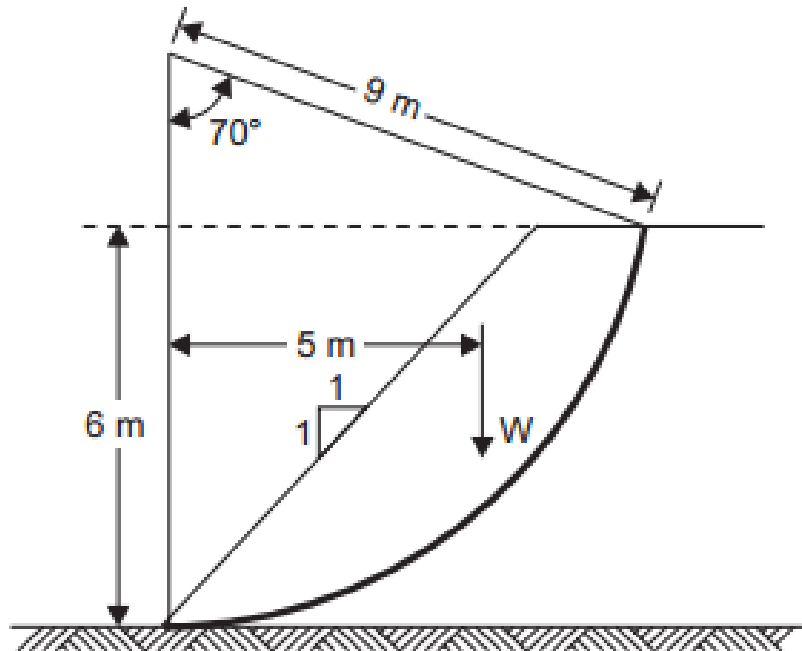
The depth of tension crack is given by:

$$h_c = 2c/\gamma$$

Tension crack reduces the resistance length to AB' so θ will change in θ'

Problem 2

Fig. shows the details of an embankment made of cohesive soil with $\phi = 0$ and $c = 30 \text{ kN/m}^2$. The unit weight of the soil is 18.9 kN/m^3 . Determine the factor of safety against sliding along the trial circle shown. The weight of the sliding mass is 360 kN acting at an eccentricity of 5.0 m from the centre of rotation. Assume that no tension crack develops. The central angle is 70°



Sliding moment = $360 \times 5 = 1800$ kNm

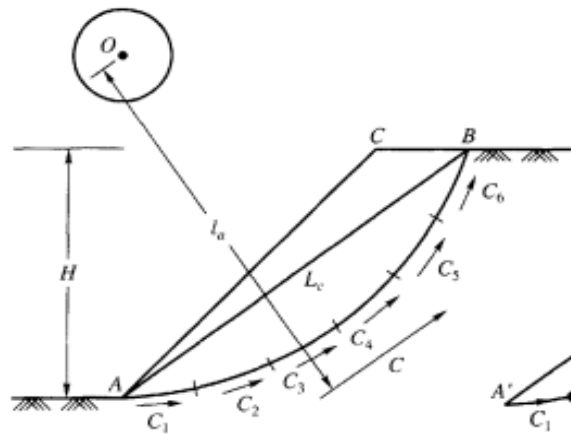
Restoring moment

$$= c. r^2 \theta = 30 \times 9^2 \times \frac{70}{180} \times \pi$$
$$= 2970 \text{ kNm}$$

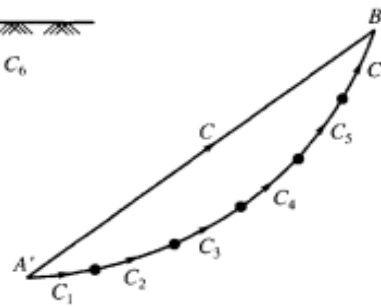
Factor of safety against sliding,

$$F = 2970/1800 = \mathbf{1.65}.$$

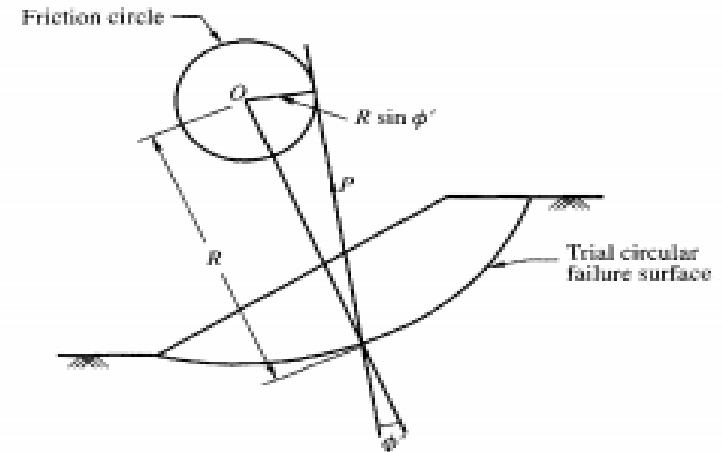
Friction-circle Method



(a) Cohesive forces on a trial arc



(b) Polygon of forces



- Useful for stability analysis of slopes made of homogeneous soils.
- The surface is assumed to be an arc of a circle.

The forces considered in the analysis are

- 1. The total weight W of the mass above the trial circle acting through the center of mass. The Center of mass may be determined by anyone of the known methods.
- 2. The resultant boundary neutral force U . The vector U may be determined by a graphical Method from flow net construction.
- 3. The resultant intergranular force, P , acting on the boundary.
- 4. The resultant cohesive force C .

Problem 3

- An embankment is inclined at an angle of 35° and its height is 15 m. The angle of shearing resistance is 15° and the cohesion intercept is 200 kN/m². The unit weight of soil is 18.0 kN/m³. If Taylor's stability number is 0.06, find the factor of safety with respect to cohesion

Taylor's stability Number $N = 0.06$

Since
$$N = \frac{c_m}{\gamma H}$$

$$\therefore 0.06 = \frac{c_m}{18 \times 15}$$

$\therefore$ Mobilised cohesion,

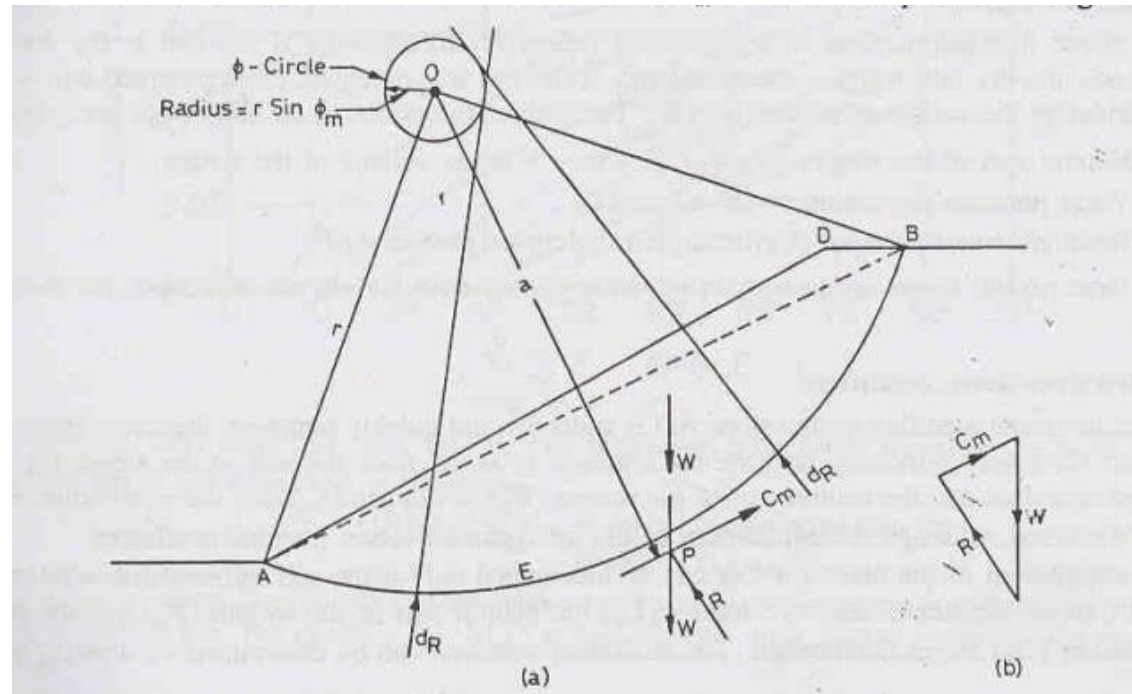
$$\begin{aligned} c_m &= 0.06 \times 18 \times 15 \text{ kN/m}^2 \\ &= 16.2 \text{ kN/m}^2 \end{aligned}$$

Cohesive strength $c = 200 \text{ kN/m}^2$

$\therefore$ Factor of safety with respect to cohesion:

$$F_c = \frac{c}{c_m} = \frac{200}{16.2} \approx \mathbf{12.3}.$$

Friction-circle Method



The cohesive force C_m is equal to $c_m L_a$ where c_m is the mobilised cohesion and L_a is the length of the circular surface arc. It is convenient to replace this force acting along the arc by an equivalent force C acting along a line. The force along arc AEB is also equal in magnitude to the force $c_m \times L_c$ where L_c is the length of the chord AB. The line of action of this force can be determined by taking moments of the actual force and the equivalent force about O.

$$(L_c \times c_m) \times a = (c_m \times L_a) \times r$$

or

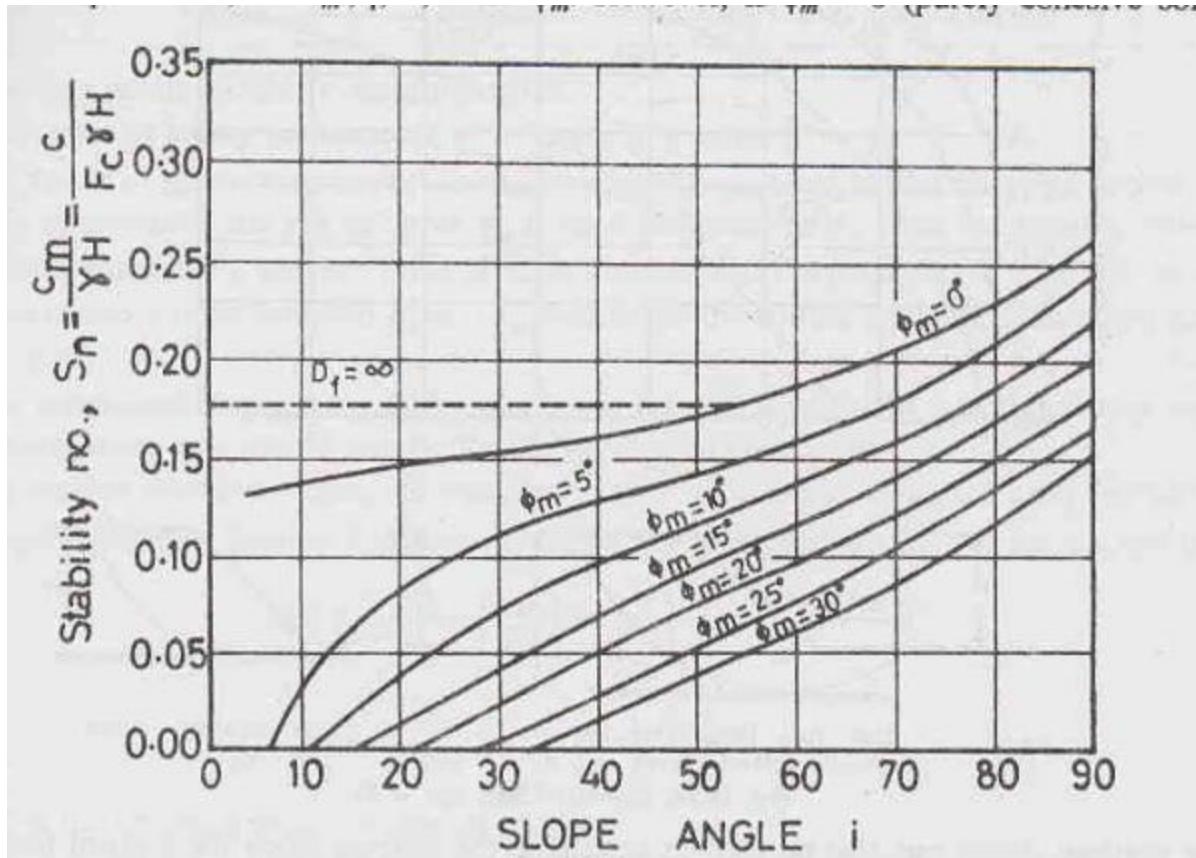
$$a = r \frac{L_a}{L_c}$$

...(18.24)

Obviously, the distance a is greater than r, as $L_a > L_c$.

Stability chart method

- Reciprocal of the stability number is known as stability factor.
- Valid for homogeneous soils.

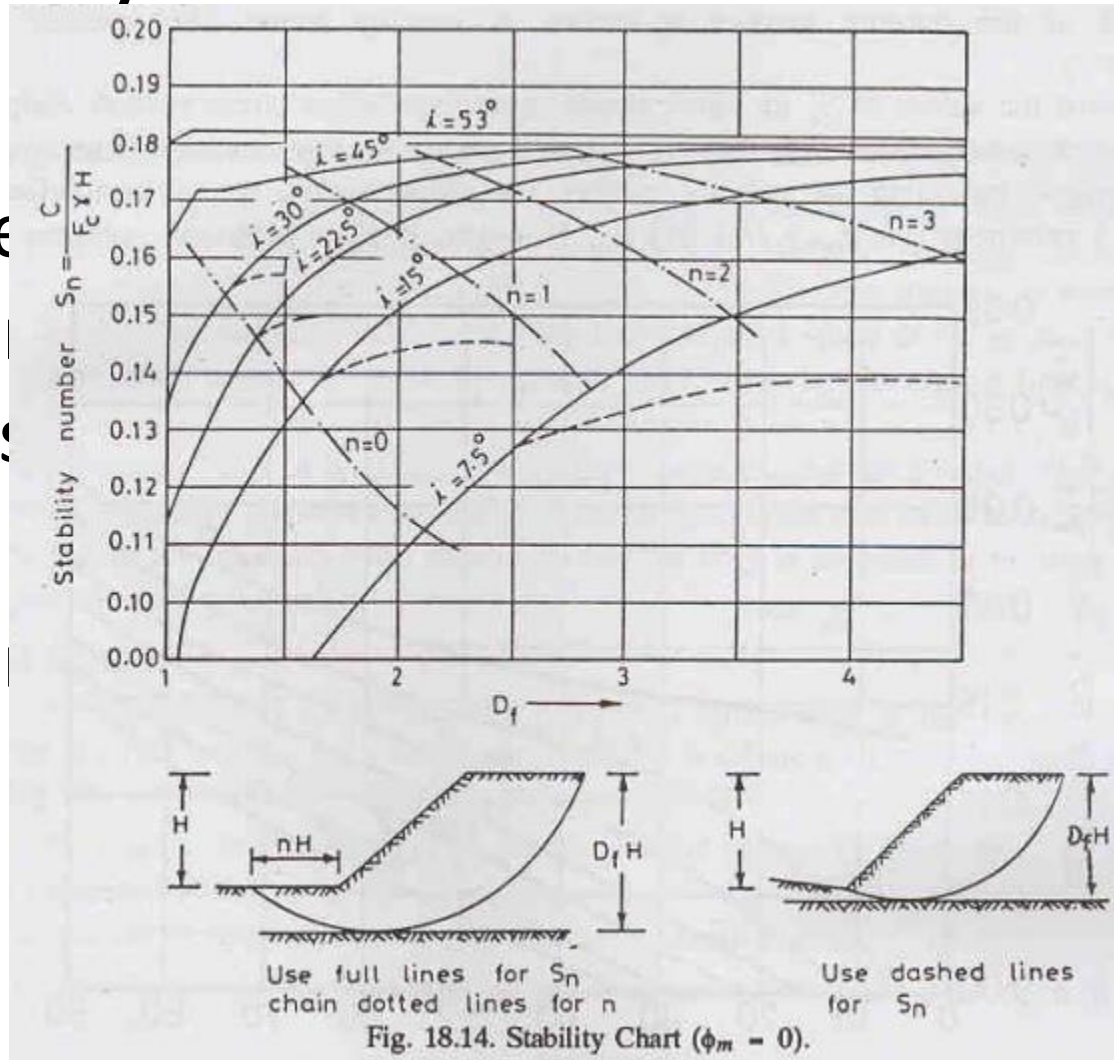


The stability number (S_n), as defined in Sect. 18.8, is given by

$$S_n = \frac{c_m}{\gamma H} = \frac{c}{F_c \gamma H}$$

Stability chart method

- Reciprocal of the stability number is known as stability factor.
- Valid for homogeneous soils.



$$D_f = \frac{\text{Depth of hard stratum below the top of the slope}}{\text{Height of slope}}$$

Fig. E-18.10.

Solution. $r \sin \phi = 22 \times \sin 15^\circ = 5.7 \text{ m.}$

The friction circle is drawn with a radius of 5.7 m.

From Eq. 18.24,
$$a = r \times \frac{L_a}{L_c} = 22 \times \frac{(2\pi \times 22/360) \times 64}{2 \times 22 \times \sin 32^\circ} = 23.20 \text{ m}$$

A line is drawn parallel to the chord at a distance of 23.20 m. The total cohesive force passes through P which is at the intersection of the vertical line through W . The reaction R also passes through P and is tangential to the friction circle.

The figure also shows the force triangle,

From the force triangle, $C = 600 \text{ kN}$

$$c_m = \frac{C}{L_c} = \frac{600}{23.32} = 25.7 \text{ kN/m}^2$$

From Eq. 18.5,
$$F_c = \frac{c}{c_m} = \frac{40}{25.7} = 1.56$$

Problem

Example 9.6: A cutting is to be made in clay for which the cohesion is 35 kN/m^2 and $\phi = 0^\circ$. The density of the soil is 20 kN/m^3 . Find the maximum depth for a cutting of side slope $1 \frac{1}{2}$ to 1 if the factor of safety is to be 1.5. Take the stability number for a $1 \frac{1}{2}$ to 1 slope and $\phi = 0^\circ$ as 0.17.

$$\begin{aligned}
 c &= 35 \text{ kN/m}^2 & \phi &= 0^\circ \\
 \gamma &= 20 \text{ kN/m}^3 & N &= 0.17 \\
 F_c &= 1.5
 \end{aligned}$$

$$\therefore c_m = c/F_c = 35/1.5 = \frac{70}{3} \text{ kN/m}^2$$

$$\text{But } N = \frac{c_m}{\gamma H}$$

$$\therefore 0.17 = \frac{c_m \times 100}{20 \times H} = \frac{70 \times 100}{3 \times 20 \times H}$$

$$\begin{aligned}
 \therefore H &= \frac{70 \times 100}{60 \times 0.17} \text{ cm} \\
 &= \mathbf{6.86 \text{ m.}}
 \end{aligned}$$

Problem

Example 9.7: At cut 9 m deep is to be made in a clay with a unit weight of 18 kN/m^3 and a cohesion of 27 kN/m^2 . A hard stratum exists at a depth of 18 m below the ground surface. Determine from Taylor's charts if a 30° slope is safe. If a factor of safety of 1.50 is desired, what is a safe angle of slope ?

Depth factor $D = 18/9 = 2$

From Taylor's charts,

$$\text{for } D = 2; \beta = 30^\circ \\ N = 0.172$$

$$0.172 = \frac{c_m}{18 \times 9}$$

$$c_m = 0.172 \times 18 \times 9 = 27.86 \text{ kN/m}^2$$

$$c = 27 \text{ kN/m}^2$$

$$F_c = c/c_m = 27/27.86 = 0.97$$

The proposed slope is therefore not safe.

$$\text{For } F_c = 1.50$$

$$c_m = c/F_c = 27/1.5 = 18 \text{ kN/m}^2$$

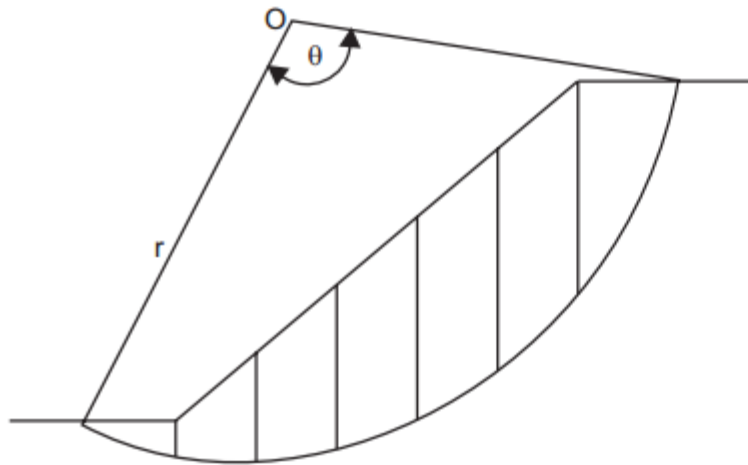
$$N = 18/18 \times 9 = 1/9 = 0.11$$

$$\text{For } D = 2.0, \text{ and } N = 0.11$$

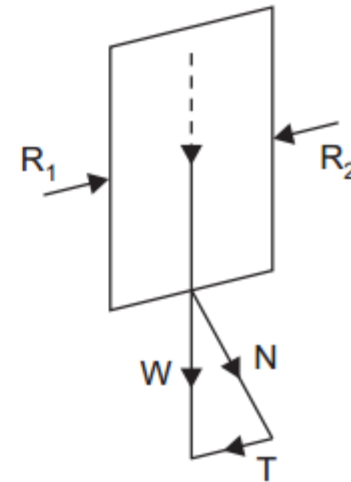
from Taylor's charts,

we have $\beta = 8^\circ$

The Swedish Method of slices for a cohesive-frictional soil



(a) Slip circle divided into slices



(b) Forces on a typical slice

Taking moment about the centre of rotation,

Sliding moment = $r \sum T$ (reckoned positive if clockwise)

Restoring moment = $r (cr\theta + \sum N \tan \phi)$ (reckoned positive if counterclockwise)

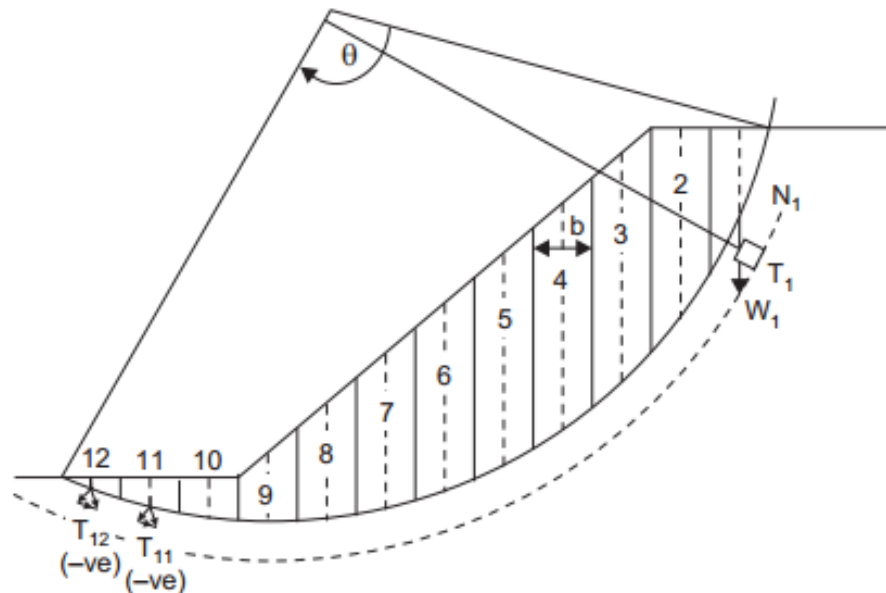
$$\therefore \text{Factor of Safety } F = \frac{(cr\theta + \sum N \tan \phi)}{\sum T} \quad \dots(\text{Eq. 9.19})$$

The effect of a tension crack can be allowed for as in the previous case. But the depth of the crack in this case is given by:

$$h_c = (2c/\gamma) \cdot \tan (45^\circ + \phi/2) \quad \dots(\text{Eq. 9.20})$$

The Swedish Method of slices for a cohesive-frictional soil

<i>Slice no.</i>	<i>Area m²</i>	<i>Weight W (kN)</i>	<i>Normal component N (kN)</i>	<i>Tangential components T (kN)</i>
1.				
2.				
3.				
.				
.				
.				
			Sum, $\Sigma N = \dots \text{ kN}$	$\Sigma T = \dots \text{ kN}$



References

- Varnes, D.J. (1978). Slope movement types and processes. In: Special Report 176: Landslides: Analysis and Control (Eds: Schuster, R. L. & Krizek, R. J.). Transportation and Road Research Board, National Academy of Science, Washington D. C., 11-33.