University of Global Village (UGV), Barishal

Geotechnical Engineering-II

Content of Theory Course

Prepared By
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Program: B.Sc. in CE

BASIC COURSE INFORMATION

Course Title	Geotechnical Engineering-II
Course Code	CE 0732-4101
Credits	04
CIE Marks	120
SEE Marks	80
Exam Hours	2 hours (Mid Exam) 3 hours (Semester Final Exam)
Level	7th Semester

Geotechnical Engineering-II

Course Code: CE 0732-4101 | **CREDIT:04**

Course Teacher: Rubieyat Bin Ali | **TOTAL MARKS:200**

Mid Exam Hours 2 | **CIE MARKS: 120**

Semester End Exam Hours 3 | **SEE MARKS: 80**

Course Learning Outcomes (CLOs): After completing this course successfully, the students will be able to-

CLO 1	**Analyze** and predict soil behavior under various loading conditions, including consolidation, shear strength, and settlement.
CLO 2	**Demonstrate** the ability to apply geotechnical principles to solve real-world engineering problems related to foundations, slope stability, and earth retaining structures.
CLO 3	**Develop** proficiency in conducting laboratory tests to characterize soil properties and in performing field investigations to assess subsurface conditions.
CLO 4	**Designing** and analyzing shallow and deep foundations, retaining walls, and slope stabilization measures considering safety, sustainability, and economic factors.
CLO 5	**Effectively generate** design solutions through technical reports, presentations, and graphical representations.

SL	Content of Course	Hrs	CLOs
1	Different types of Settlement, Spring analogy of consolidation, e-log p graph, co-efficient of compressibility/compression, normally and over consolidated soil, Virgin curve and one-dimensional consolidation, Degree of consolidation and time factor, Methods of determining preconsolidation pressure, Settlement analysis	8	CLO1
2	Details about Standard penetration tests, Shear strength properties of soil, Stress-controlled direct shear test, Strain-controlled direct shear test, Tri-axial shear Test (UU test procedure and equipment's, CU and CD test procedure), Stress-strain behavior of clay soil	8	CLO2, CLO3
3	Vane Shear Test, Definition and objectives of foundation, Shallow and deep foundation, Isolated, combined and strip footing, Raft or mat foundation, Deep foundation, Rankin's theory of shallow foundation, Soil-foundation failure, Ultimate bearing capacity of shallow foundation, Water table corrections, Terzaghi's bearing capacity equation	9	CLO3, CLO4
4	Skin friction pile, End-bearing pile, Negative skin friction, Group pile, Ultimate bearing capacity of pile foundation, Pile load test, Slope stability methods, Full soil report analysis	9	CLO4, CLO5

Text Book:

1. "Soil Mechanics and Foundations" by Punmia, Dr. B. C.; Jain, Ashok Kumar and Jain, A. K.
2. "Soil Mechanics and Foundation Engineering" by Dr. K.R. Arora
3. "Principles of Foundation Engineering" by Braja M. Das.

ASSESSMENT PATTERN
CIE- Continuous Internal Evaluation (120 Marks)
SEE- Semester End Examination (80 Marks)

Bloom's Category	Tests
Remember	**10**
Understand	**15**
Apply	**15**
Analyze	**15**
Evaluate	**15**
Create	**10**

Bloom's Category Marks (out of 120)	Tests (min 4) (60)	Assignments (min 2) (20)	Quizzes (20)	External Participation in Curricular/Co-Curricular Activities (20)
Remember	10		10	Attendance/Field Visit/Presentation /Project Exhibition (20)
Understand	10		10	
Apply	10			
Analyze	10			
Evaluate	10			
Create	10	20		

Course plan specifying content, CLOs, teaching learning and assessment strategy mapped with CLOs

Week	Topic	Teaching-Learning Strategy	Assessment Strategy	Corresponding CLOs
1	Soil Consolidation I (Different types of Settlement, Spring analogy of consolidation, e-log p graph, co-efficient of compressibility/compression, Virgin curve)	Lecture, discussion, group work	Quiz, Written Exam	CLO1
2	Soil Consolidation II (normally and over consolidated soil, one-dimensional consolidation)	Oral Presentation, debate	Assignment, Written, Quiz	CLO1
3	Soil Consolidation III (Degree of consolidation and time factor)	Video lecture, Field visit	Report writing, Demonstration	CLO1
4	Soil Consolidation IV (Methods of determining preconsolidation pressure, Settlement analysis)	Lecture	Viva, Quiz	CLO1, CLO3
5	SPT Test	Project exhibition	Project, Field visit	CLO1
6	Direct Shear Test (Stress/Strain Controlled)	Discussion, Video Presentation	Quiz, Written Exam	CLO1
7	Tri-axial Shear Test (UU, CU and CD)	Case-based Learning, Demonstration	Assignment, Written, Quiz	CLO1, CLO4
8	Unconfined Compression Test	Lecture, discussion, group work	Report writing, Demonstration	CLO3, CLO4
9	Vane Shear Test, Definition and objectives of foundation	Oral Presentation, debate	Viva, Quiz	CLO2
10	Shallow Foundation I	Video lecture	Project, Field visit	CLO2

Course plan specifying content, CLOs, teaching learning and assessment strategy mapped with CLOs

Week	Topic	Teaching-Learning Strategy	Assessment Strategy	Corresponding CLOs
11	Shallow Foundation II	Lecture	Quiz, Written Exam	CLO3, CLO5
12	Pile Foundation I	Project exhibition	Assignment, Written, Quiz	CLO3, CLO3
13	Pile Foundation II	Discussion, Video Presentation	Report writing, Demonstration	CLO1
14	Pile load Test and causes for unstable slopes	Case-based Learning, Demonstration	Viva, Quiz	CLO1
15	Slopes stability techniques and soil report analysis	Lecture, discussion, group work	Project, Field visit	CLO3, CLO5
16	Final soil report analysis	Oral Presentation, debate	Quiz, Written Exam	CLO3, CLO4
17	Practice and Exercise, Practice and Exercise, Practice and Exercise	Video lecture	Assignment, Written, Quiz	CLO3, CLO5

Soil Consolidation

Week 1

Pages 09-51

TERMINOLOGY

1.1 COMPRESSIBILITY :

It is a process of decrease in volume of soil due to increase in load on the soil. The compressibility of soil may be due to:

1.1.A : Expulsion of pore air; **Compaction.**

1.1.B: Expulsion of pore water; **1st Degree Consolidation.**

1.1.C: Compression of water and solid molecules; **Negligible.**

1.1.D: Plastic readjustment of soil molecules to achieve more stable position; **2nd Degree Consolidation.**

COMPRESSIBILITY OF SOIL

COMPACTION

1. INSTANTANEOUS
2. DUE TO EXPULSION OF PORE AIR

CONSOLIDATION

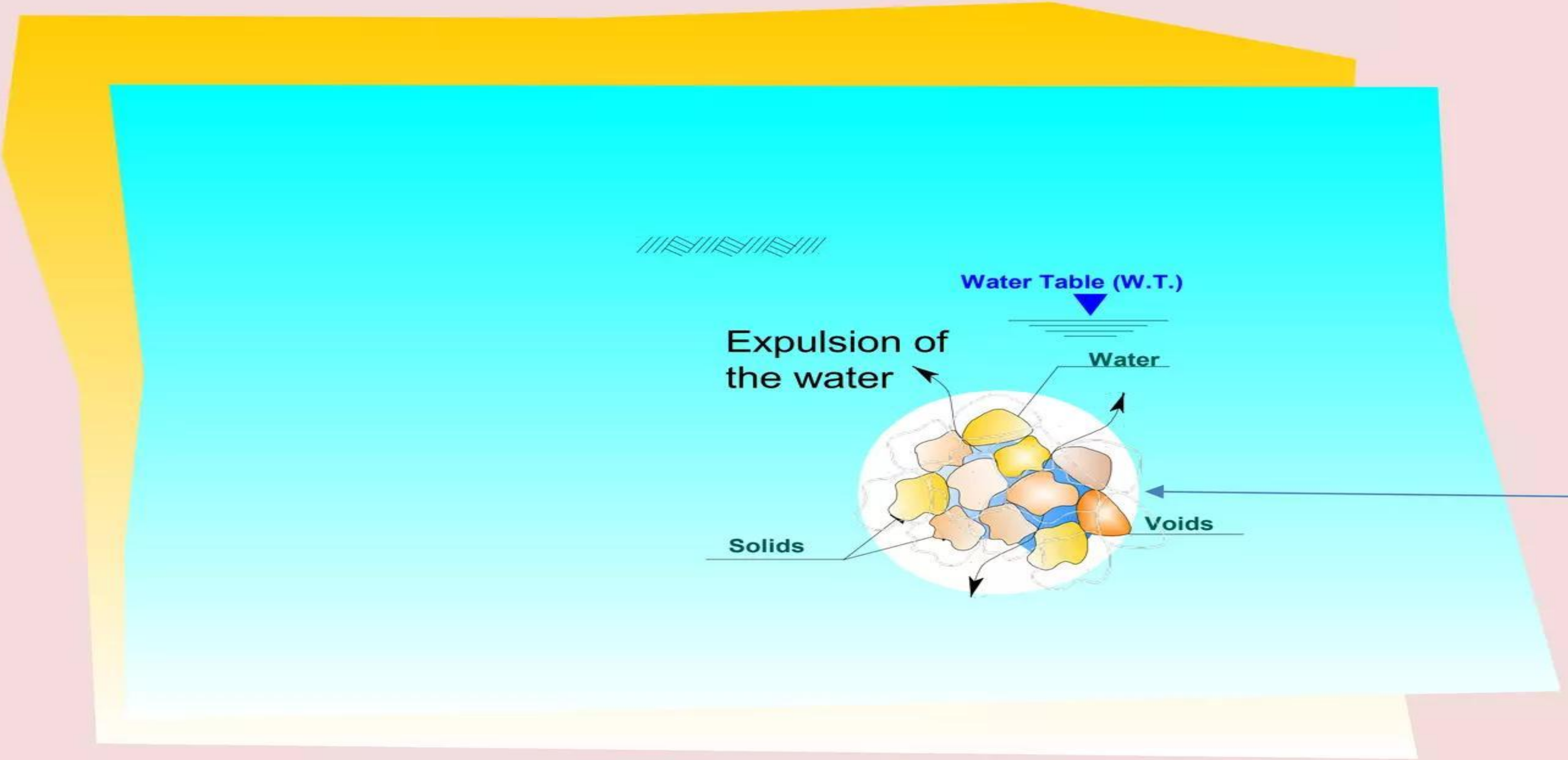
PRIMARY CONSOLIDATION

1. TIME DEPENDENT
2. DUE TO EXPULSION OF PORE WATER

SECONDARY CONSOLIDATION

1. TIME DEPENDENT
2. VERY SLOW; TAKE YEARS
3. DUE TO PLASTIC READJUSTMENT OF SOIL MOLECULES

CONSOLIDATION



WHEN THE WATER IN THE VOIDS STARTS TO FLOW OUT OF THE SOIL MATRIX DUE TO CONSOLIDATION OF THE CLAY LAYER. CONSEQUENTLY, THE EXCESS PORE WATER PRESSURE (ΔU) WILL REDUCE, AND THE VOID RATIO (E) OF THE SOIL MATRIX WILL REDUCE TOO.

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

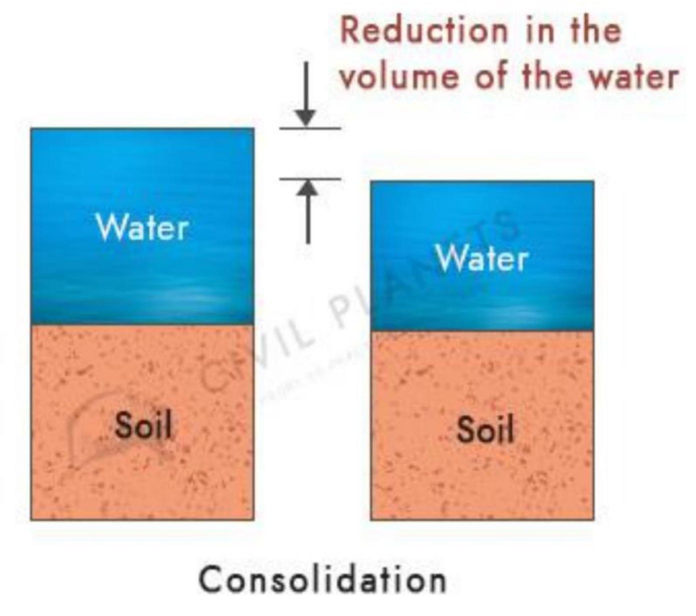
Compaction

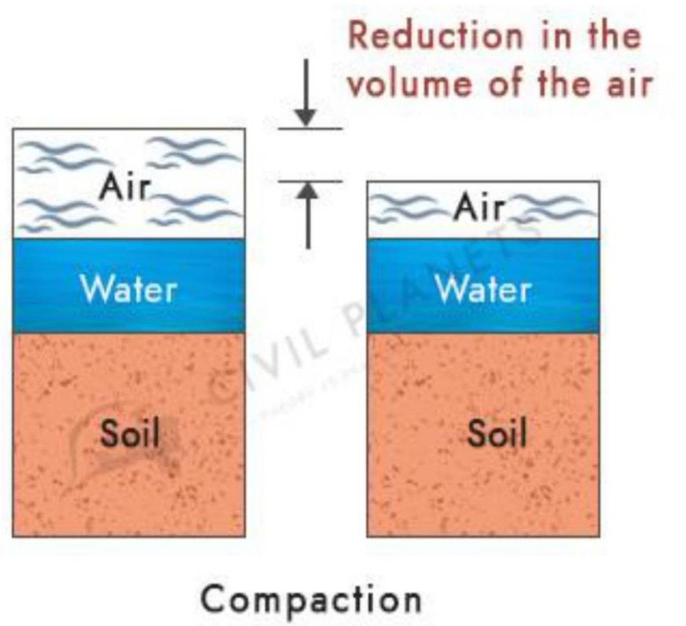
In geotechnical engineering, soil compaction is the process in which stress applied to a soil causes densification as air is displaced from the pores between the soil grains.

At low water content, the soil is stiff and offers more resistance to compaction. As the water content is increased, the soil particles get lubricated. The soil mass becomes more workable and the particles have closer packing.

Consolidation

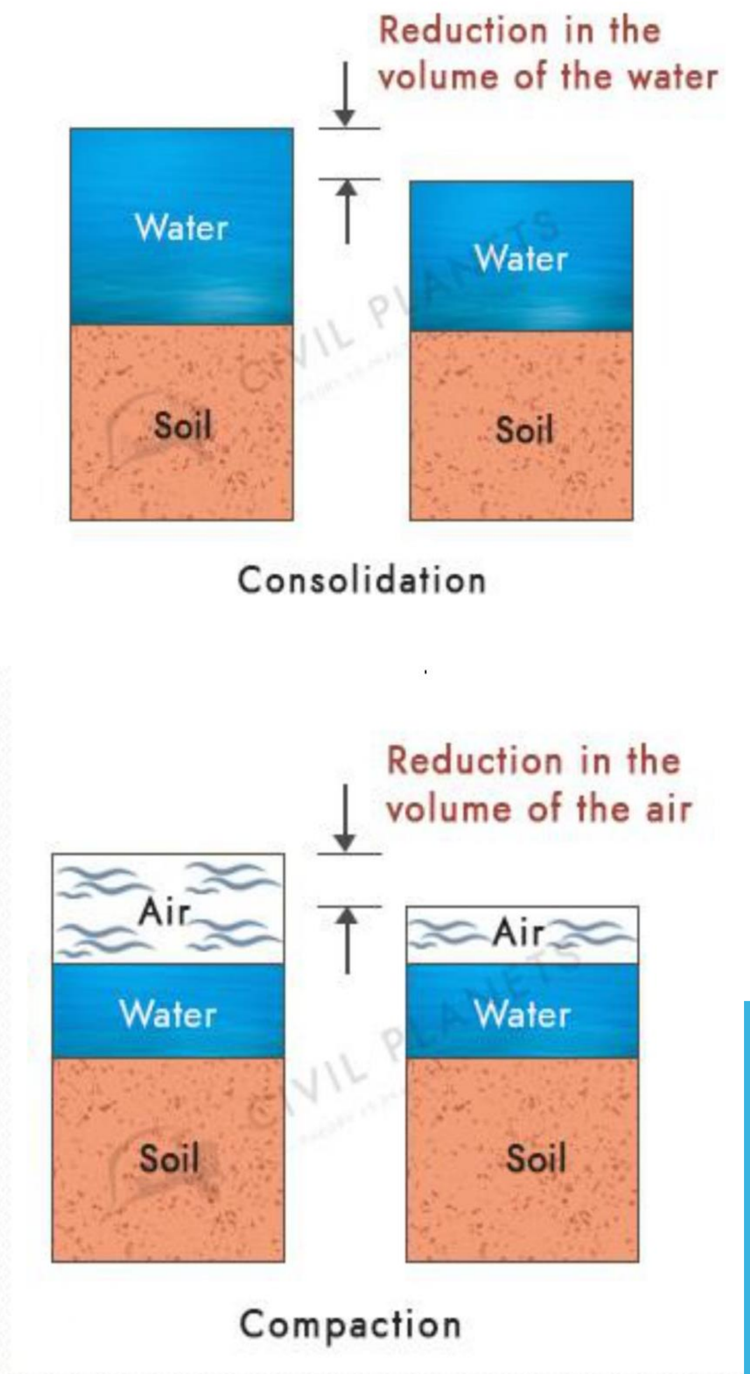
According to the "father of soil mechanics", Karl von Terzaghi, consolidation is "any process which involves a decrease in water content of saturated soil without replacement of water by air".





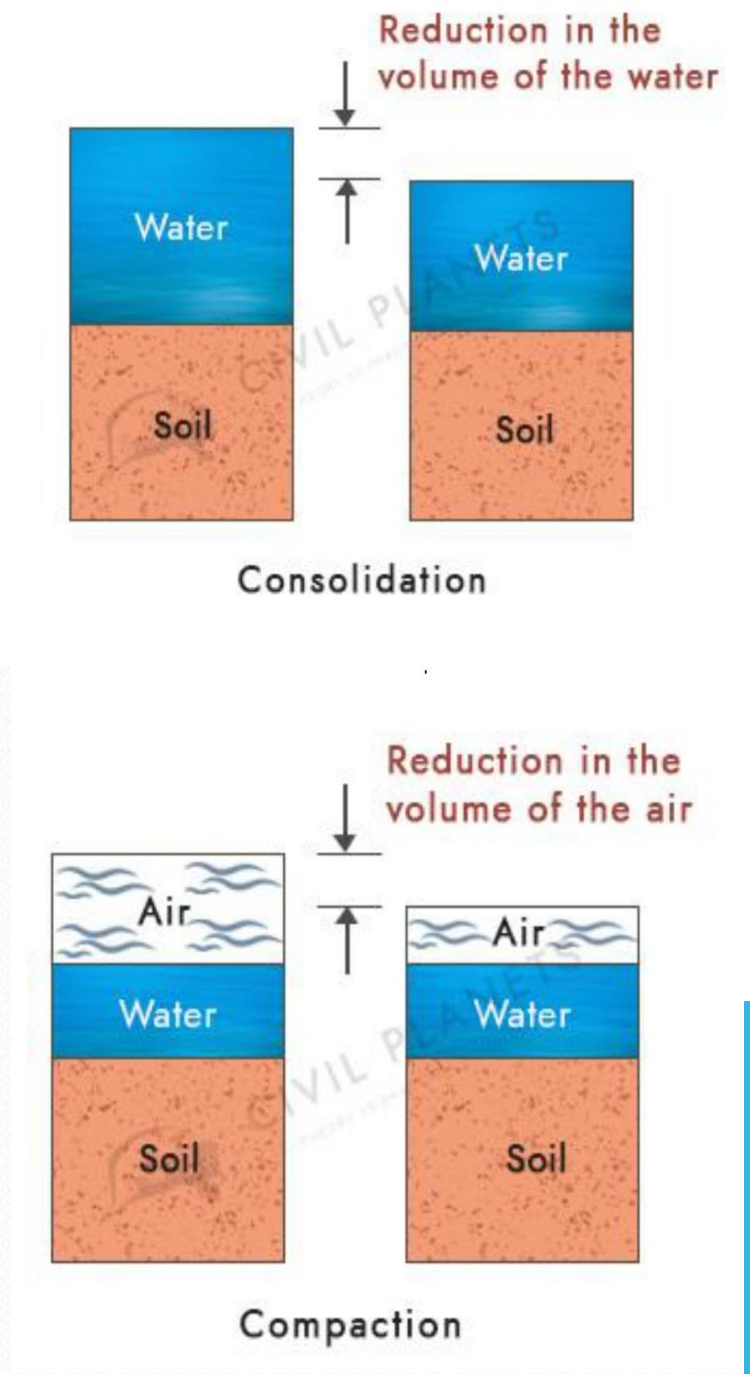
COMPARISON BETWEEN COMPACTION AND CONSOLIDATION

Compaction	Consolidation
1. Compaction is the process by which solid soil particles are packed more closely together by mechanical means. 2. It is achieved through reduction of air voids. 3. It is a rapid process. 4. It is an artificial process. 5. Proper compaction of soil is achieved at optimum moisture content.	1. Consolidation is the process by which soil particles are packed more closely together under the application of static loading 2. It is achieved through gradual drainage of water from soil pores. 3. It is a gradual process. In some soils it takes many years. 4. It is a natural process. 5. Consolidation is strictly applicable for saturated or nearly saturated clays or soils

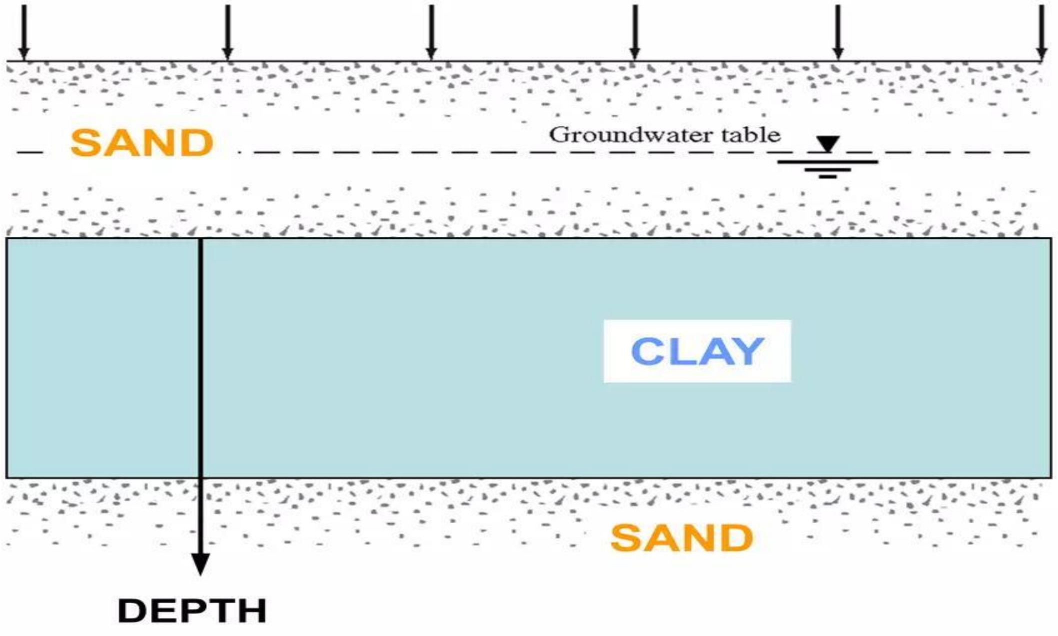


DIFFERENCE BETWEEN COMPACTION AND CONSOLIDATION

COMPACTION	CONSOLIDATION
Compaction process is instantaneous process.	Consolidation is a time dependent process.
Soil is partially saturated.	In consolidation, we assume fully saturated soil.
Thickness of layer decreases due to expulsion of air from the voids.	Thickness of layer decreases due to dissipation of pore water pressure.
Loading is impact loading.	Loading is gradual loading.
Void ratio decreases.	Void ratio decreases.



FUNDAMENTALS OF CONSOLIDATION



after Figure 7.1a. Das FGE (2005).

CONSOLIDATION:
Volume change in saturated soils caused by the expulsion of pore water from loading.

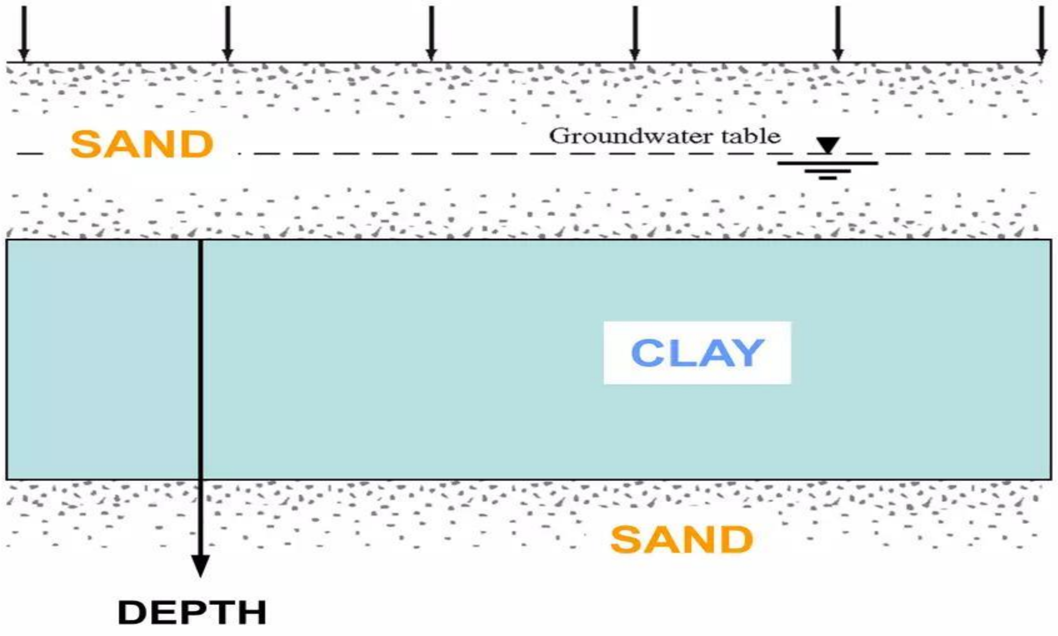
Saturated Soils:

Δσ causes *u* to increase immediately

Sands: Pore pressure increase dissipates rapidly due to high permeability.

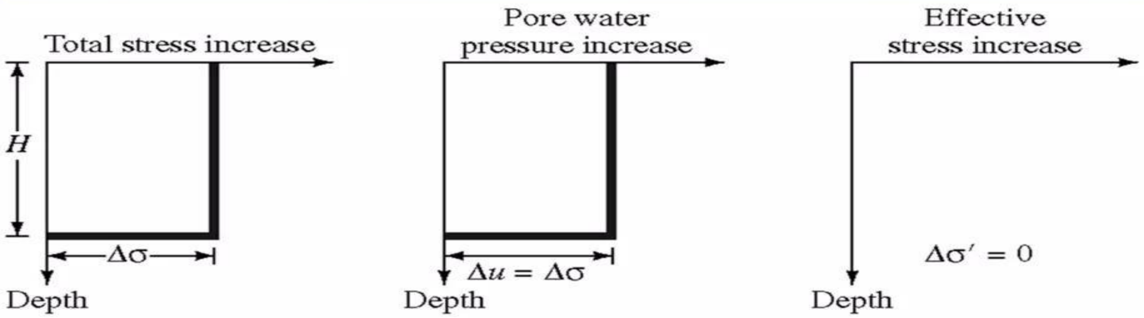
Clays: Pore Pressure dissipates slowly due to low permeability.

FUNDAMENTALS OF CONSOLIDATION



after Figure 7.1a. Das FGE (2005).

At Time of Initial Loading (t = 0)

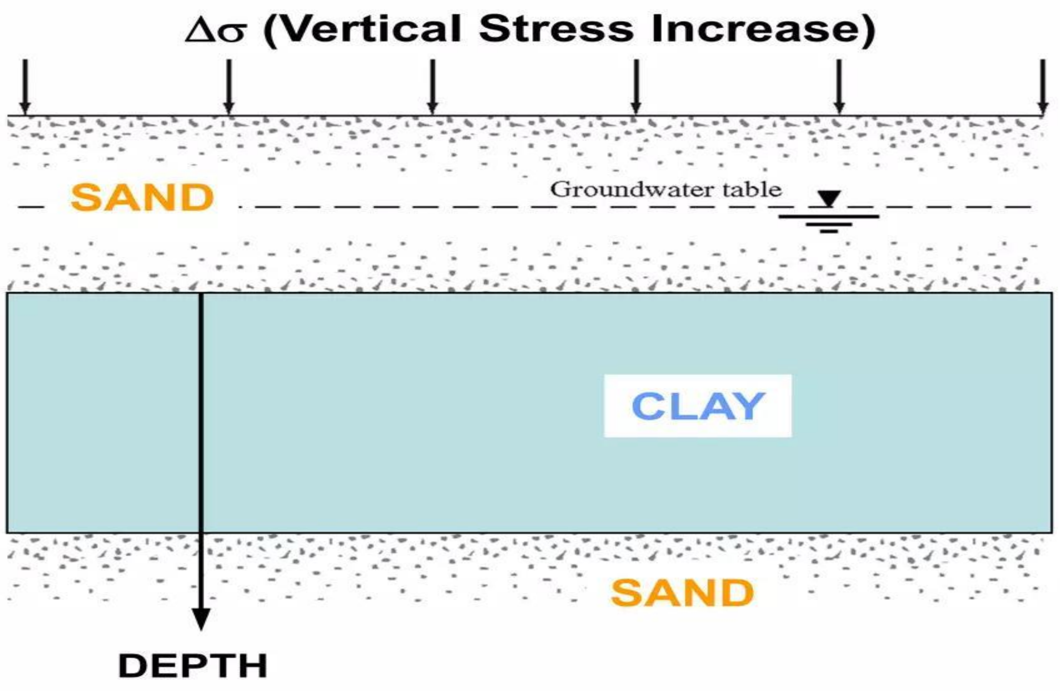


Variation in Total, Pore water, and Effective Stresses in Clay Layer
Figure 7.1b. Das FGE (2005)

Pore water takes initial change in vertical loading ($\Delta\sigma = \Delta u$) since water is incompressible

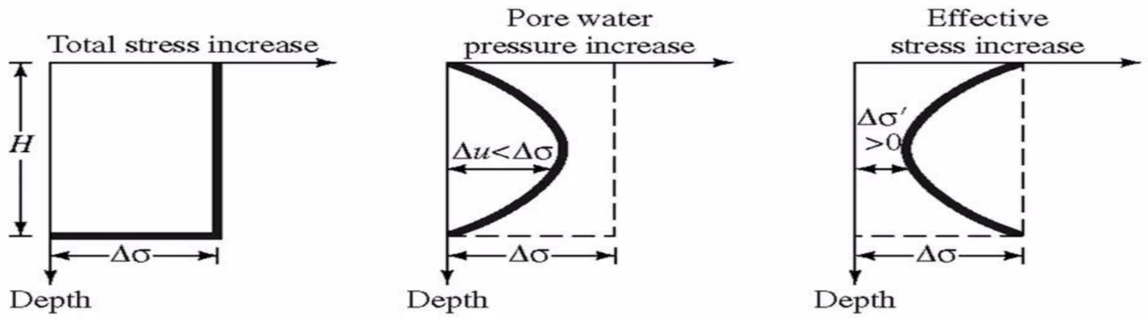
Soil skeleton does not see initial loading

FUNDAMENTALS OF CONSOLIDATION



after Figure 7.1a. Das FGE (2005).

Between time t = 0 to t = ∞

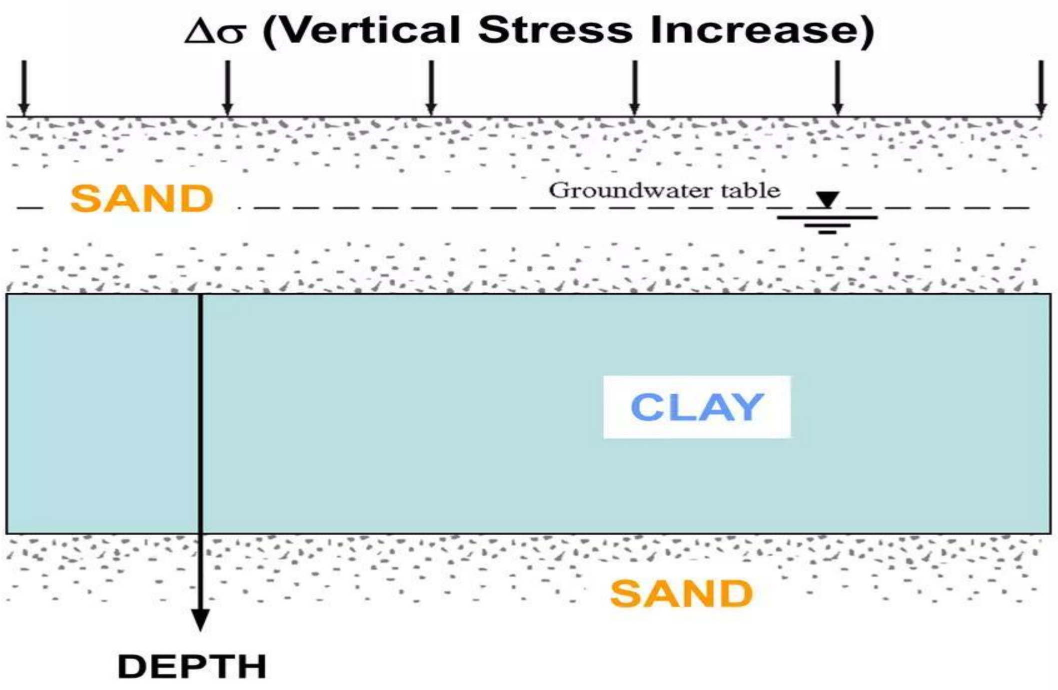


Variation in Total, Pore water, and Effective Stresses in Clay Layer
Figure 7.1c. Das FGE (2005)

Pore water increase due to initial loading dissipates

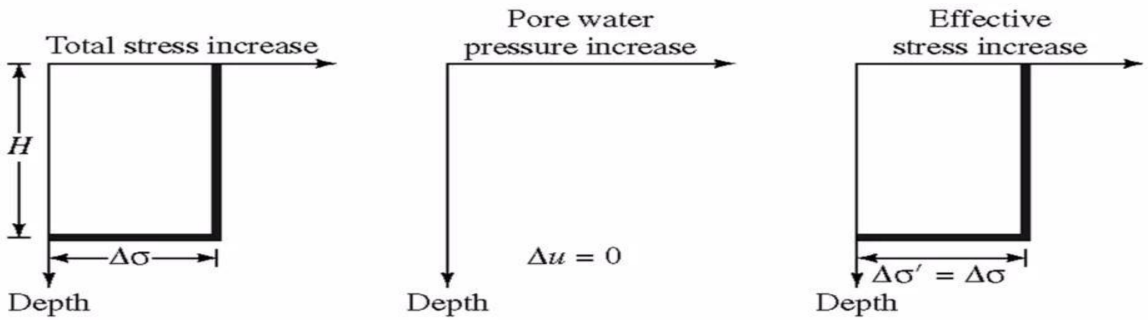
Soil skeleton takes loading as pore pressure decreases

FUNDAMENTALS OF CONSOLIDATION



after Figure 7.1a. Das FGE (2005).

At time t = ∞



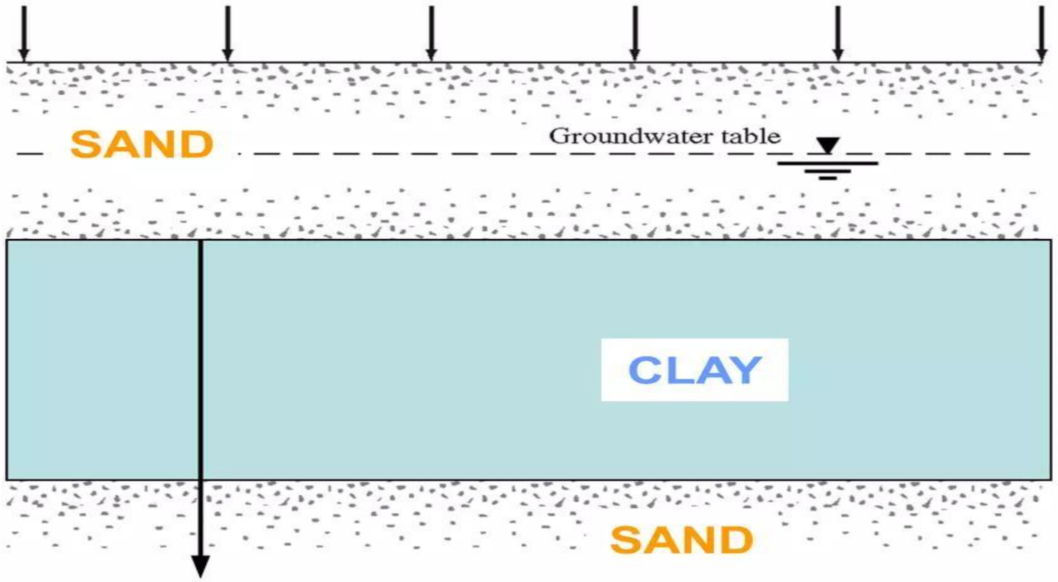
Variation in Total, Pore water, and Effective Stresses in Clay Layer
Figure 7.1e. Das FGE (2005)

Pore water increase due to initial loading completely dissipated ($\Delta u = 0$)

Soil skeleton has taken loading. Effective stress increase now equals vertical stress increase ($\Delta\sigma = \Delta'\sigma$)

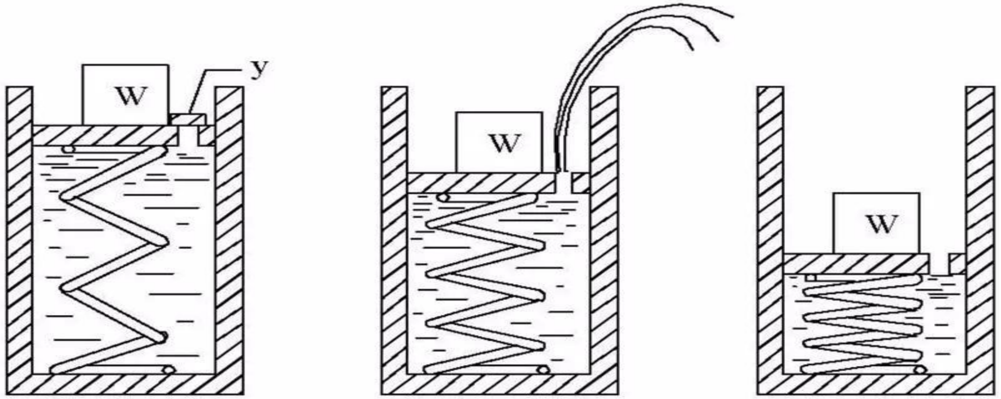
FUNDAMENTALS OF CONSOLIDATION

Δσ (Vertical Stress Increase)



after Figure 7.1a. Das FGE (2005).

THE SPRING ANALOGY



(a)
Initial Loading

Water takes load

Soil (i.e. spring) has no load

(b)
Dissipation of Excess Water Pressure

Water dissipating

Soil starts to

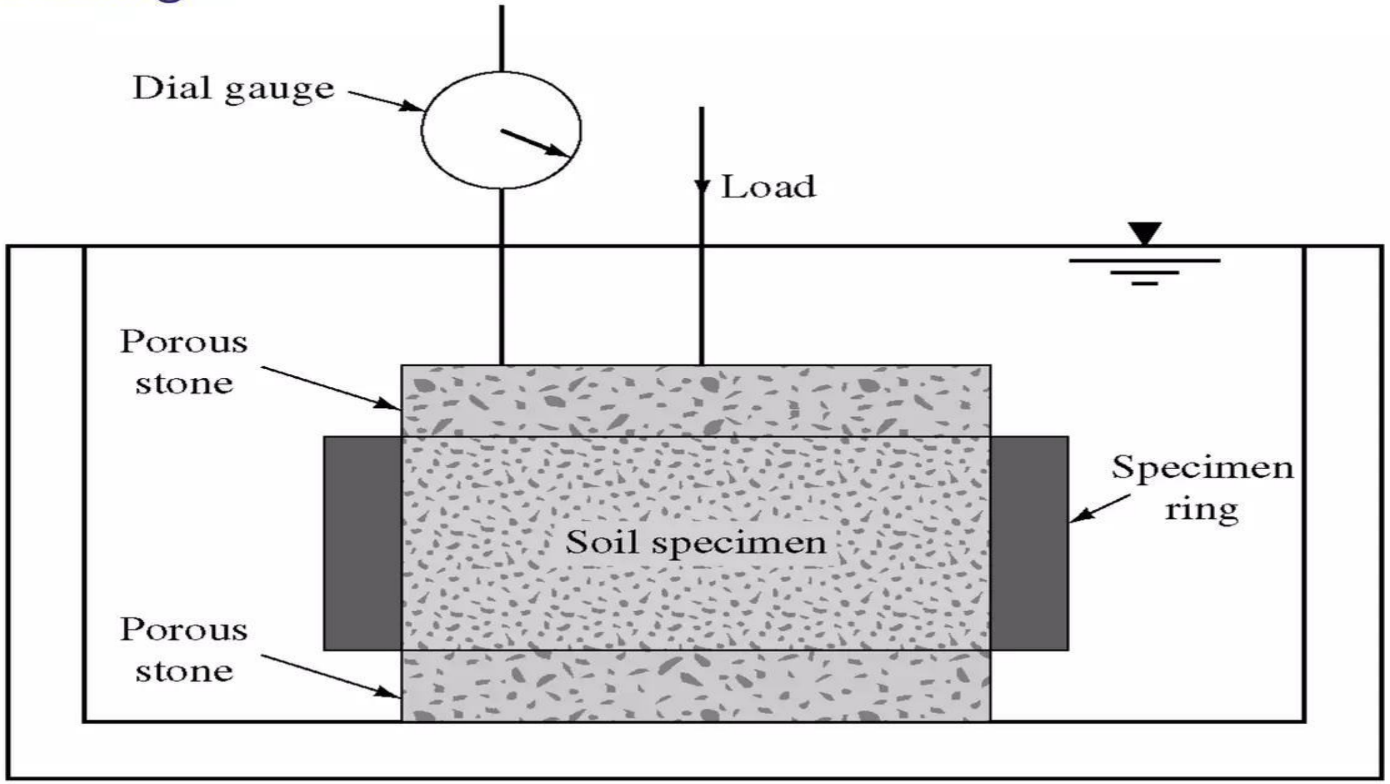
(c)
Final Loading

Water dissipated

Soil has load

ONE DIMENSIONAL (1D) CONSOLIDATION TEST

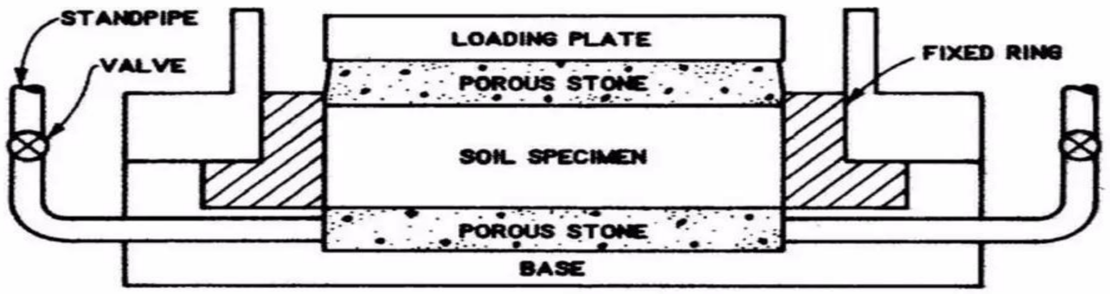
D2435-11 Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading



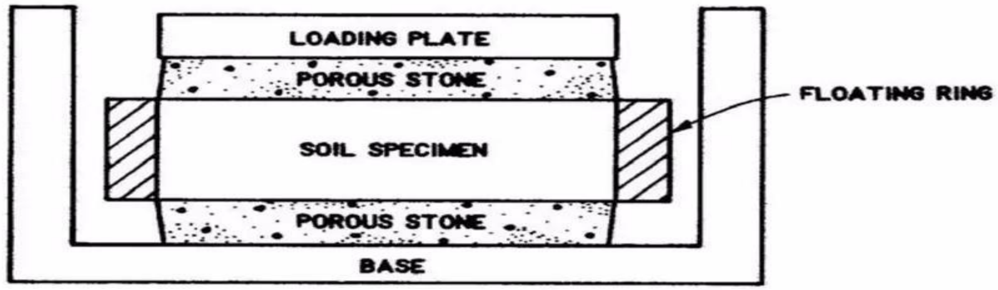
Consolidometer

Figure 7.2. Das FGE (2005)

1D Consolidation Test Equipment



a. FIXED-RING CONSOLIDOMETER Or Oedometer



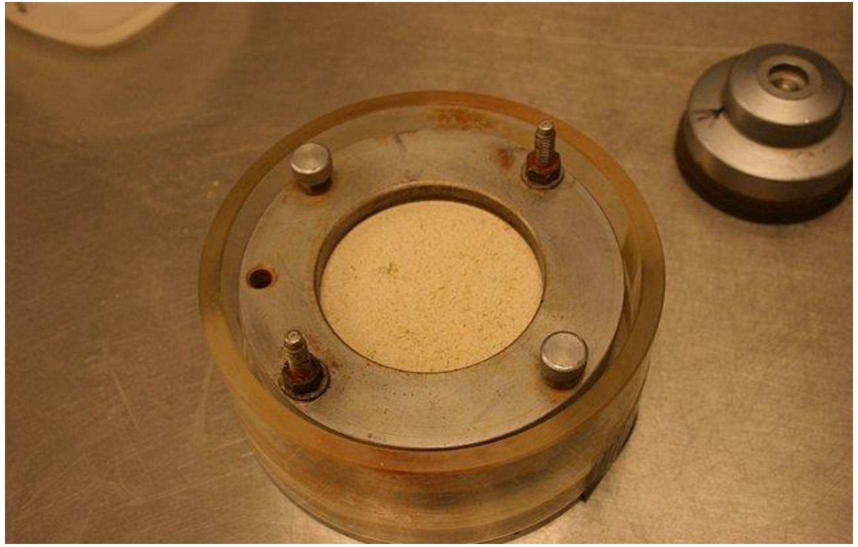
b. FLOATING-RING CONSOLIDOMETER

Figure E-1 USACE EM1110-1-1904.

ShearTrac II DSS Equipment
(Courtesy of Geocomp Corporation)

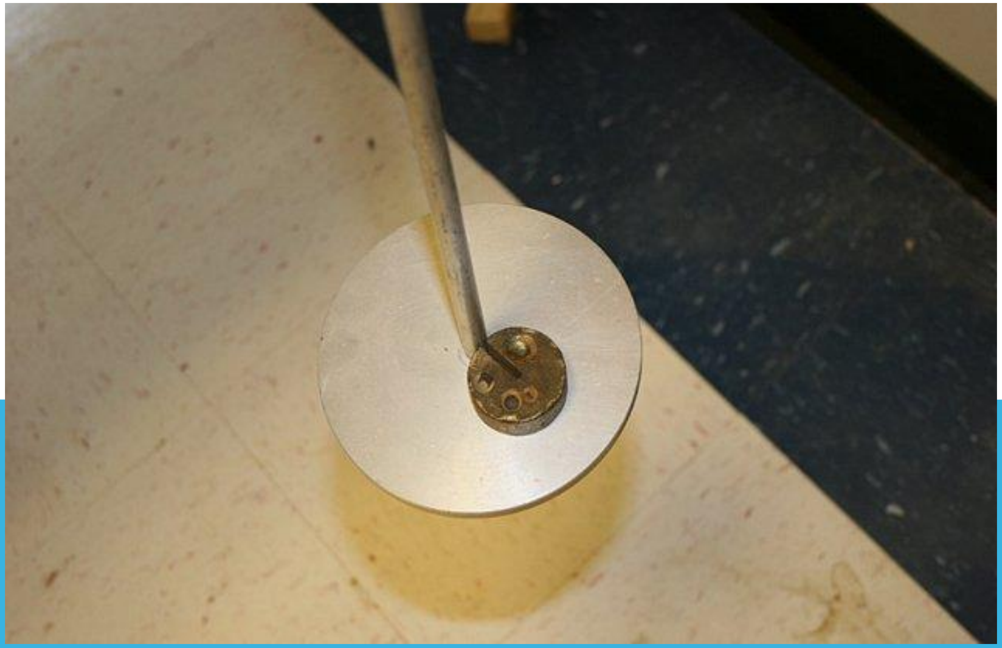
Consolidometer

Soil sample



Assemble of cell

Soil loading preparation



Seating Load

Porous stones are used in consolidation tests to support soil samples and allow water to flow through. The stones can be wet or dry, depending on the type of soil being tested.

Explanation

Wet stones

Wet stones are used when testing softer clays. The stone may be covered with wet filter paper.

Dry stones

Dry stones are used when testing soils that are sensitive to moisture, such as soils that swell or collapse.

How porous stones are used

The bottom porous stone is placed on the base of the consolidation cell.

The soil sample is placed on the bottom porous stone.

The upper porous stone is placed on top of the soil sample.

The loading cap is placed on top of the upper porous stone.

The consolidation cell is filled with water.

The specimen is allowed to reach equilibrium for 24 hours.

Purpose of the consolidation test

The consolidation test is a laboratory test that measures how a soil sample deforms when a vertical load is applied. The test results are used to determine the compressibility of the soil and the rate at which it consolidates

Initial Reading

Next load increment

Consolidation Test Procedure (Click this)

1D Consolidation Testing

Load Increment Data

THREE STAGES

Stage I: Initial Compression
Primarily caused by preloading.

Stage II: Primary Consolidation
Excess pore water pressure dissipation and corresponding soil volume change.

Stage III: Secondary Consolidation
Occurs after excess pore water pressure dissipation. Due to plastic deformation/ readjustment of soil particles.

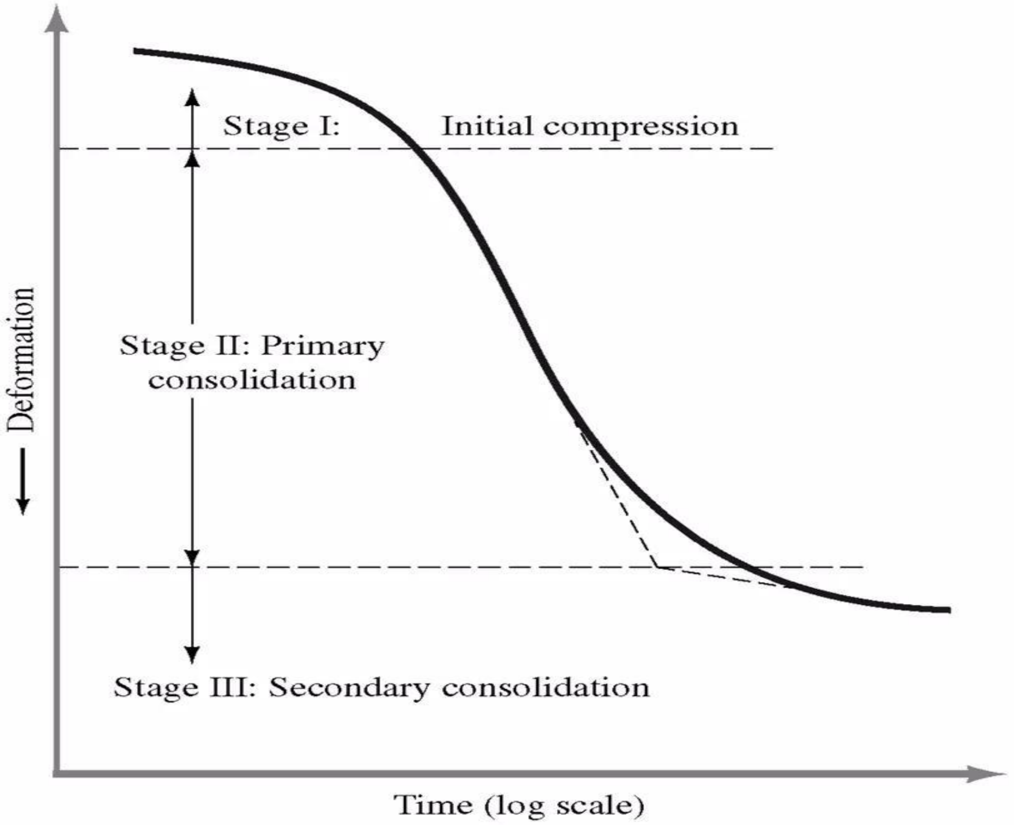


Figure 7.4. Das FGE (2005).

VOID RATIO-PRESSURE PLOTS

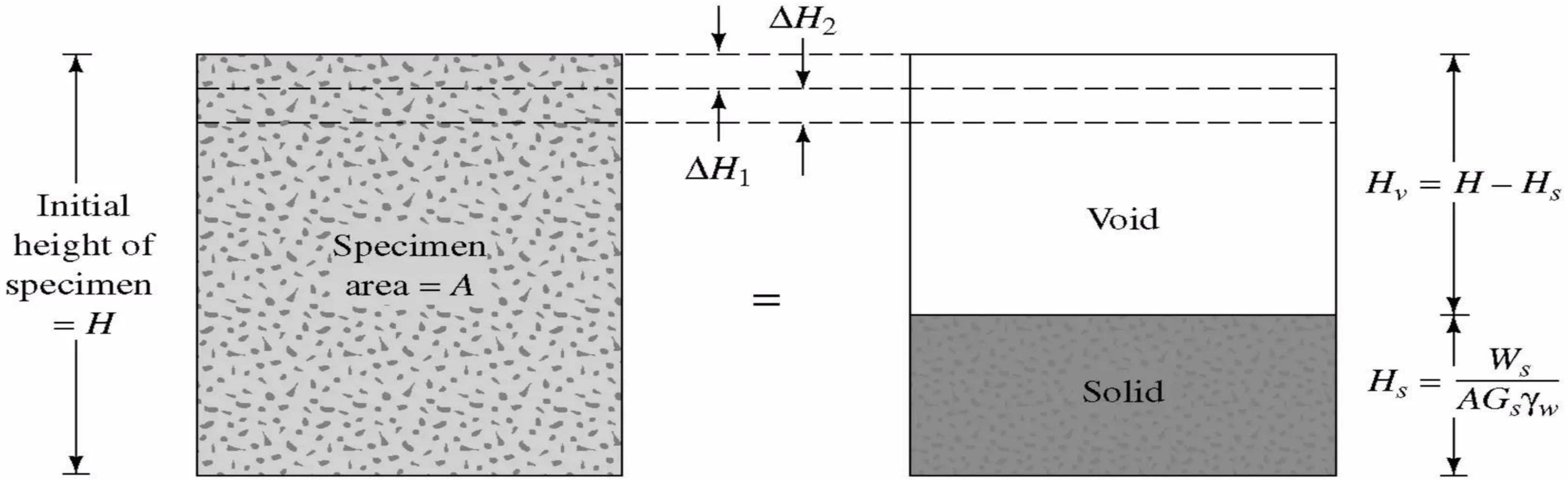


Figure 7.5. Das FGE (2005)

Initial Void Ratio (e_o): $e_o = \frac{V_v}{V_s} = \frac{H_v A}{H_s A} = \frac{H_v}{H_s}$

VOID RATIO-PRESSURE PLOTS

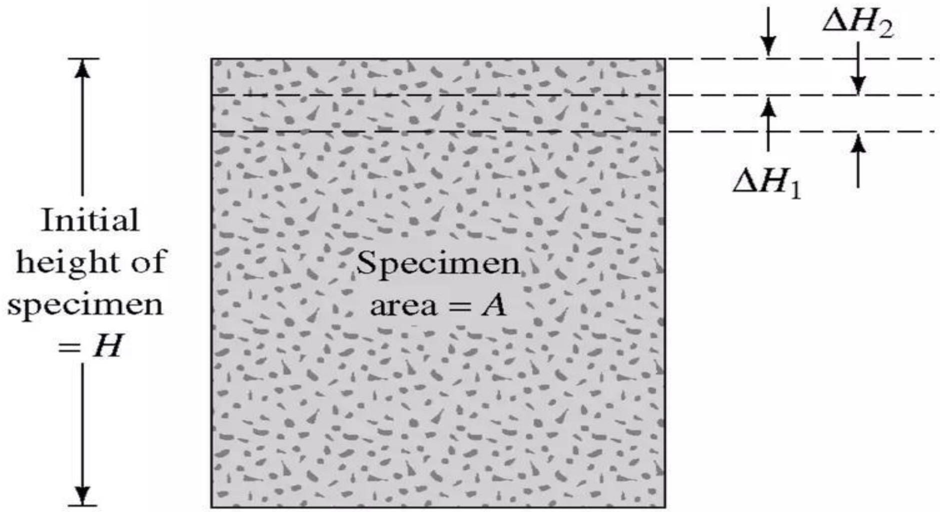


Figure 7.5. Das FGE (2005)

Change in Void Ratio due to 1st Loading (Δe_1):

$$\Delta e_1 = \frac{\Delta H_1}{H_s}$$

New Void Ratio after 1st Loading:

$$e_1 = e_o - \Delta e_1$$

@ End of Load 1

$$\sigma' = \sigma_1 = \sigma_1'$$

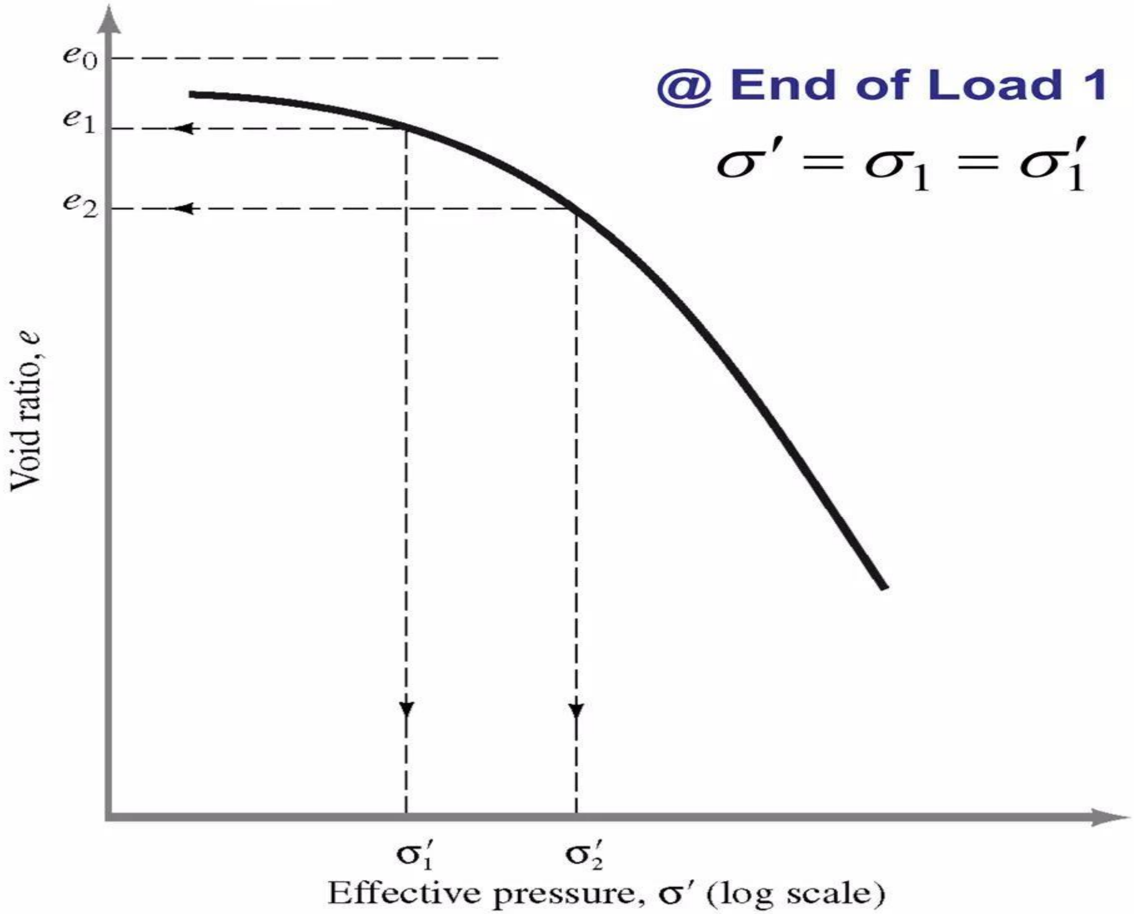


Figure 7.6. Das FGE (2005)

VOID RATIO-PRESSURE PLOTS

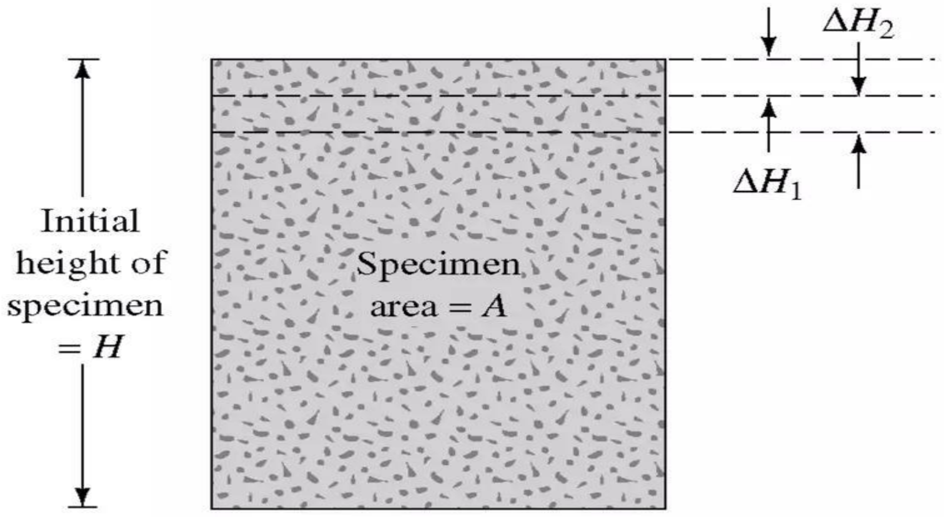


Figure 7.5. Das FGE (2005)

Change in Void Ratio due to 2nd Loading (Δe_2):

$$\Delta e_1 = \frac{\Delta H_2}{H_s}$$

New Void Ratio after 2nd Loading:

$$e_2 = e_1 - \frac{\Delta H_2}{H_s}$$

@ End of Load 2

$$\sigma' = \sigma_2 = \sigma_2'$$

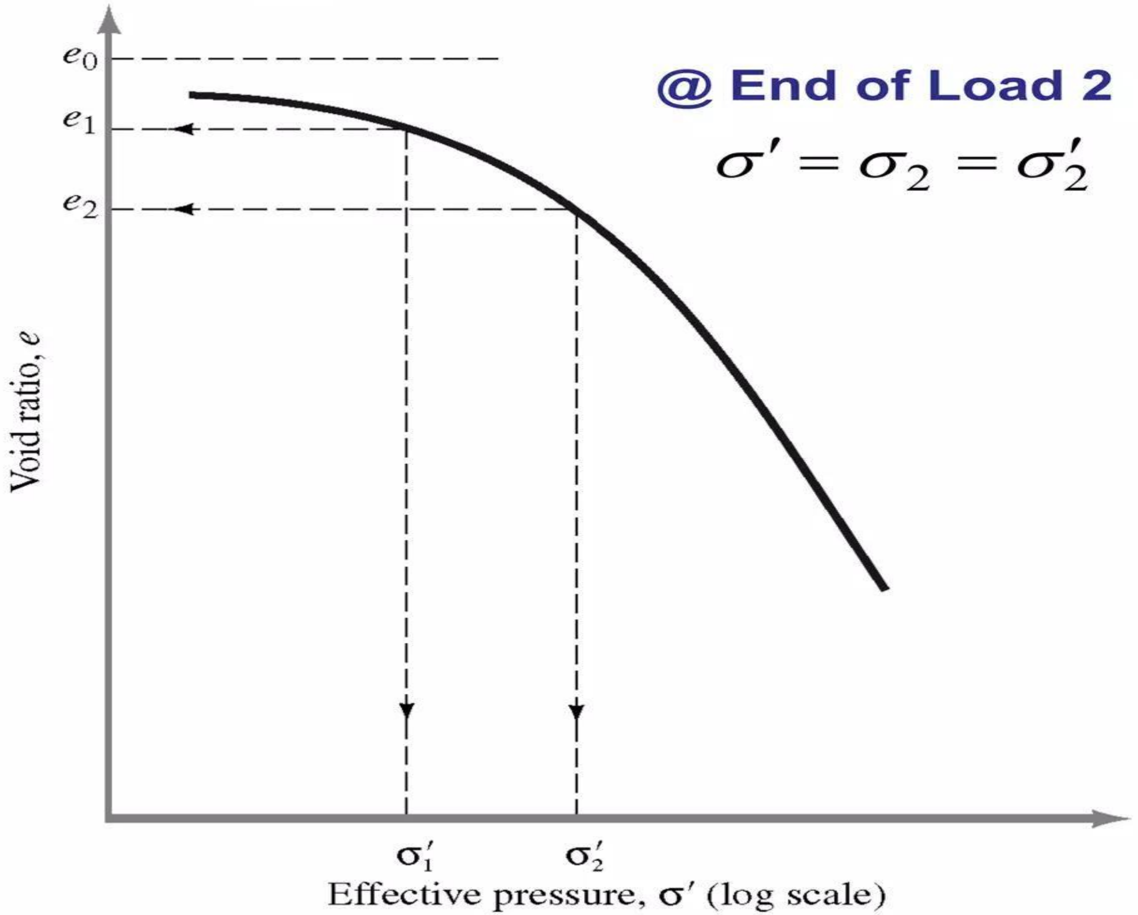


Figure 7.6. Das FGE (2005)

VOID RATIO-PRESSURE PLOTS

Final e – log σ´ plots consist of results of numerous load & unload increments

Two Definitions of Clays based on Stress History:

Normally Consolidated (NC):
The present overburden pressure (a.k.a. effective in-situ stress) is the most the soil has ever seen.

Overconsolidated Clay (OC):
The present overburden pressure is less than the soil has experienced in the past. The maximum effective past pressure is called the preconsolidation pressure (σ'_c) or ***Maximum Past Pressure*** (σ'_{vm})

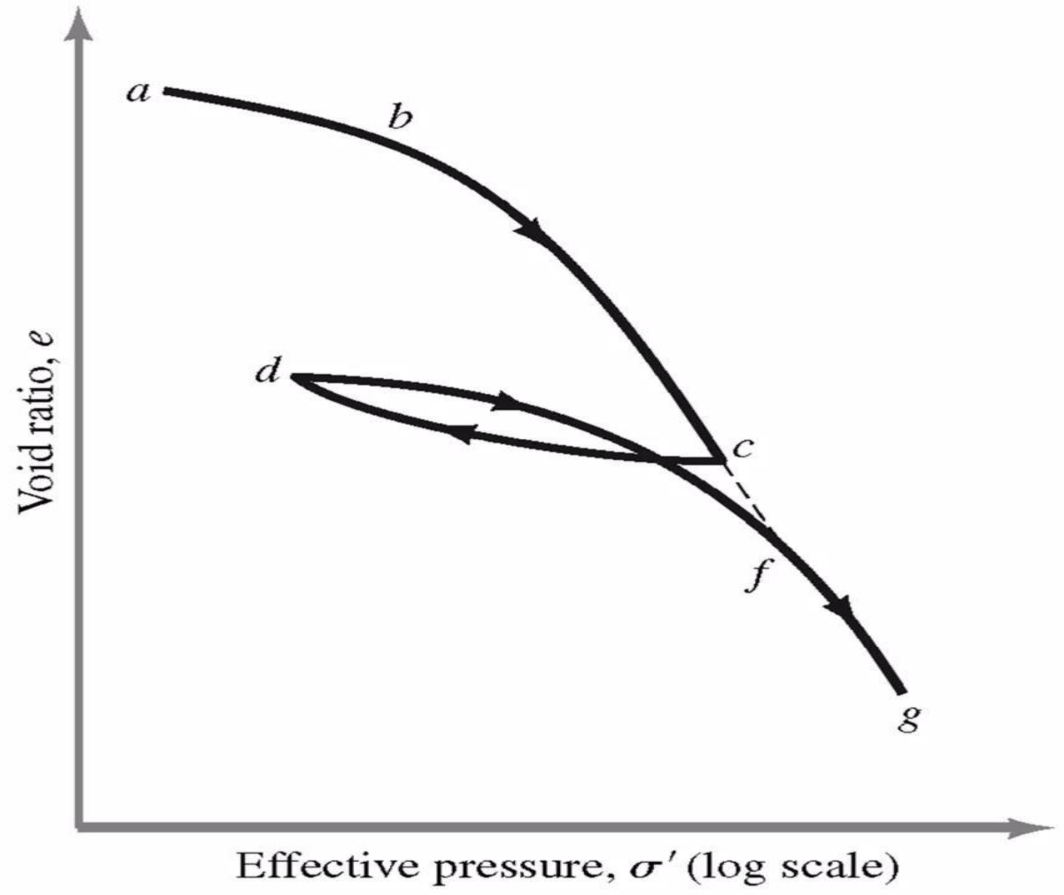
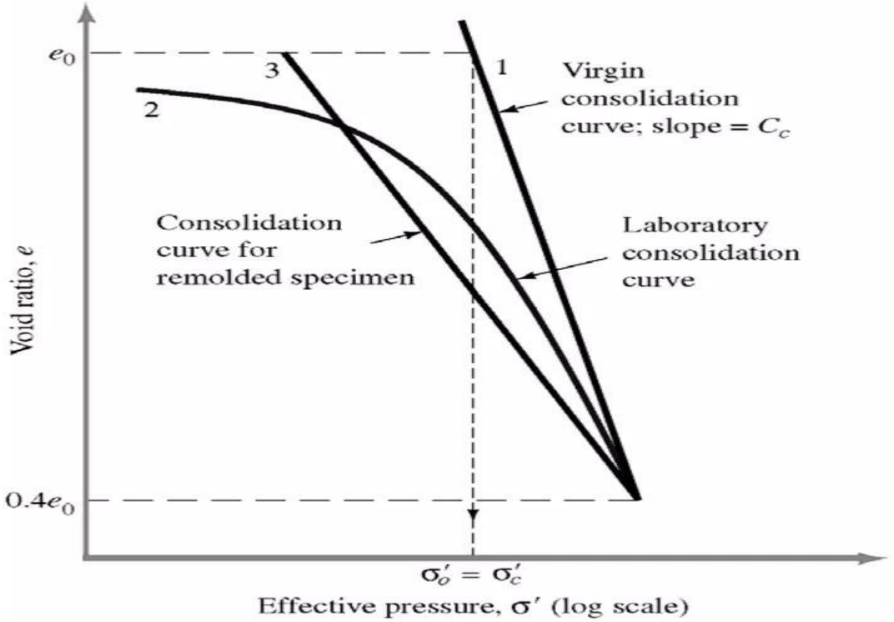


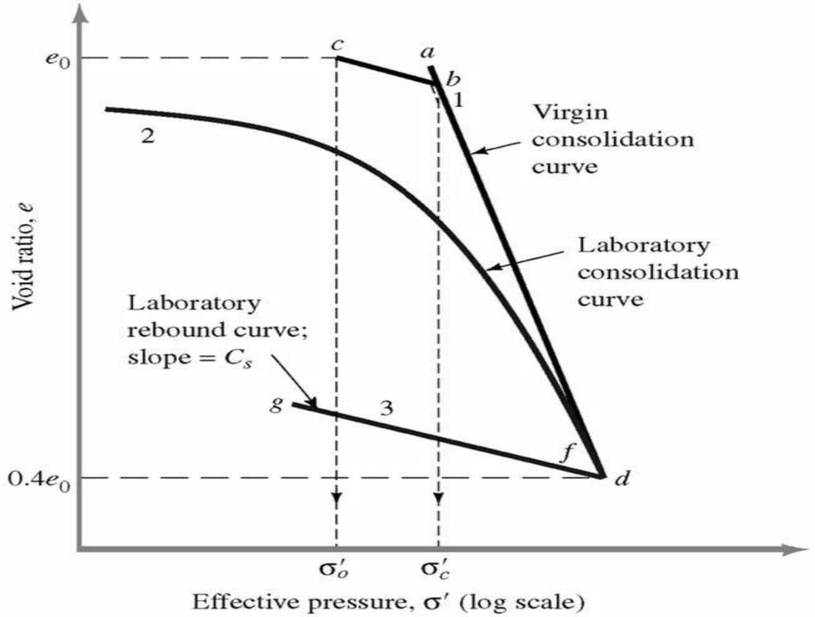
Figure 7.7. Das FGE (2005).

EFFECTS OF SAMPLE DISTURBANCE

NC and OC soils of low to medium sensitivity will experience disturbance due to remolding. This changes the consolidation characteristics of the 1D consolidation tests.



NC Clays - Figure 7.9. Das FGE (2005)



OC Clays - Figure 7.10. Das FGE (2005)

Virgin Compression Curve – Consolidation Curve Insitu (i.e. w/o disturbance)

Sensitivity (S_t) $S_t = \frac{q_{u(undisturbed)}}{q_{u(remolded)}}$

Where q_u = Unconfined Compressive Strength

EFFECTS OF SAMPLE DISTURBANCE

Reconstruction of Virgin Consolidation Curves (EM 1110-1-1904)

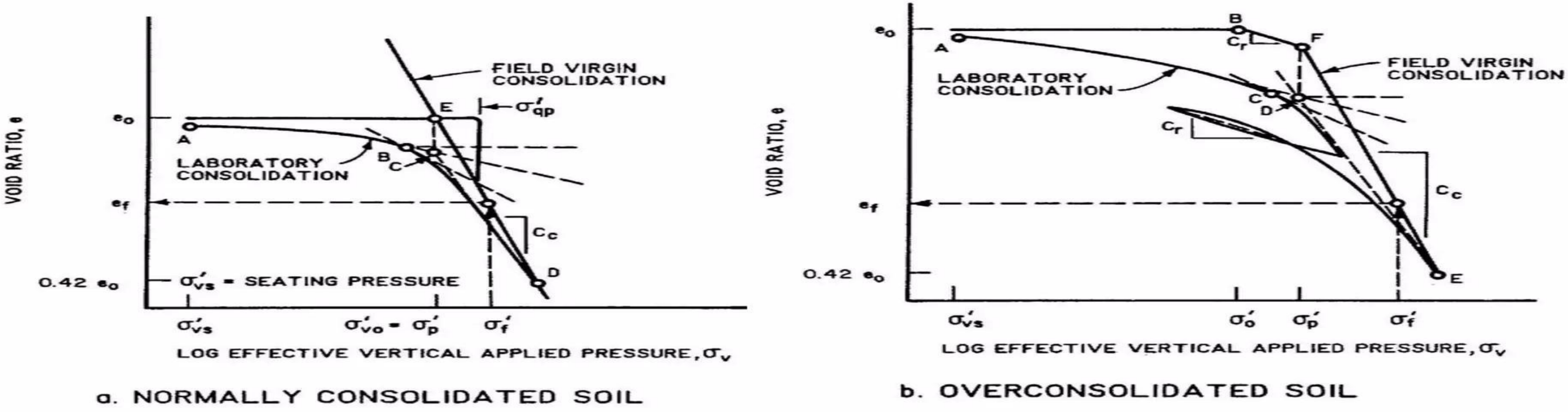


Figure 3-12. EM 1110-1-1904 Settlement Analysis.

EFFECTS OF SAMPLE DISTURBANCE

Reconstruction of Virgin Consolidation Curves (EM 1110-1-1904)

Table 3-6. EM 1110-1-1904 Settlement Analysis.

a. Normally Consolidated Soil (Figure 3-12a)

Step	Description
1	Plot point B at the point of maximum radius of curvature of the laboratory consolidation curve.
2	Plot point C by the Casagrande construction procedure: (1) Draw a horizontal line from B ; (2) Draw a line tangent to the laboratory consolidation curve through B ; and (3) Draw the bisector between horizontal and tangent lines. Point C is the intersection of the straight portion of the laboratory curve with the bisector. Point C indicates the maximum past pressure σ'_p .
3	Plot point E at the intersection e_o and σ'_p . e_o is given as the initial void ratio prior to testing in the consolidometer and σ'_p is found from step 2.
4	Plot point D at the intersection of the laboratory virgin consolidation curve with void ratio $e = 0.42e_o$.
5	The field virgin consolidation curve is the straight line determined by points E and D.

EFFECTS OF SAMPLE DISTURBANCE

Reconstruction of Virgin Consolidation Curves (EM 1110-1-1904)

Table 3-6. EM 1110-1-1904 Settlement Analysis.

b. Overconsolidated Soil (Figure 3-12b)

Step	Description
1	Plot point B at the intersection of the given e_o and the initial estimated in situ effective overburden pressure σ'_o .
2	Draw a line through B parallel to the mean slope C_r of the rebound laboratory curve.
3	Plot point D using step 2 in Table 3-6a above for normally consolidated soil.
4	Plot point F by extending a vertical line through D up through the intersection of the line of slope C_r extending through B.
5	Plot point E at the intersection of the laboratory virgin consolidation curve with void ratio $e = 0.42e_o$.
6	The field virgin consolidation curve is the straight line through points F and E.

Settlement from 1D Primary Consolidation

At End of Primary Consolidation $\Delta\sigma = \Delta\sigma'$

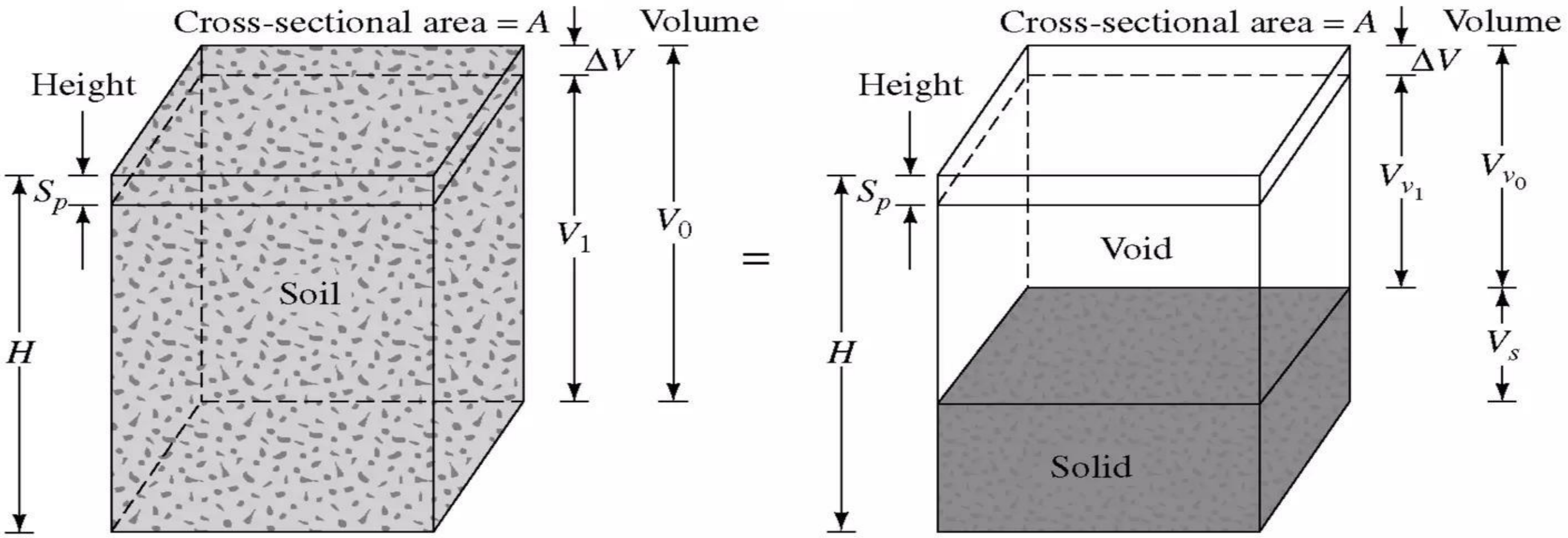


Figure 7.11. Das FGE (2005)

$$\Delta V = V_o - V_1 = HA - (H - S_p)A = S_p A$$

Where:

V = Volume, V_o = Initial Volume, V_1 = Final Volume, S_p = Primary Settlement

SETTLEMENT FROM 1D PRIMARY CONSOLIDATION

At End of Primary Consolidation $\Delta\sigma = \Delta\sigma'$

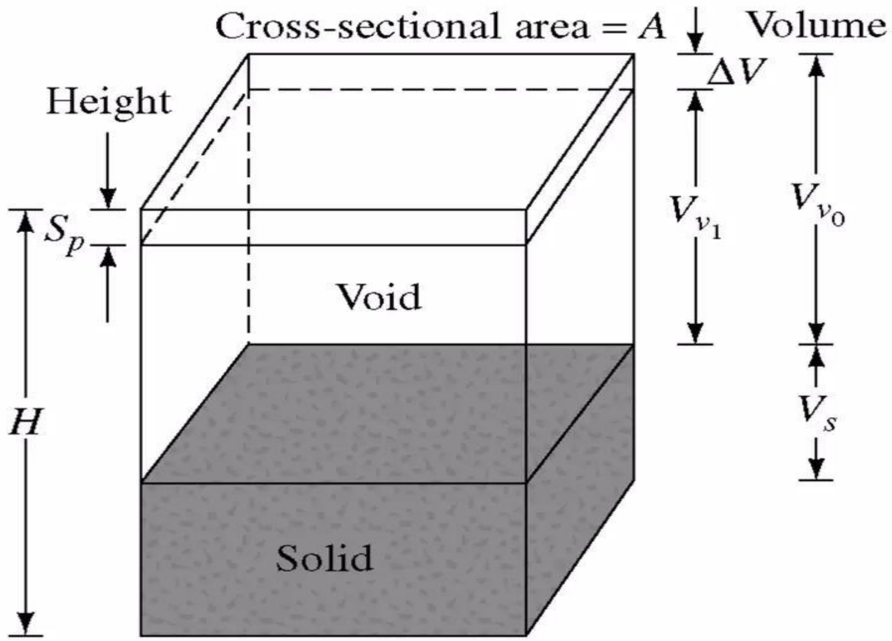


Figure 7.11. Das FGE (2005).

$$\Delta V = S_p A = V_{vo} - V_{v1} = \Delta V_v$$

Where:
V_{vo} = Initial Void Volume, V_{v1} = Final Void Volume

$$\Delta V_v = \Delta e V_s$$

Where:
Δe = Change in Void Ratio

$$V_s = \frac{V_o}{1+e_o} = \frac{AH}{1+e_o}$$

Where:
e_0 = Initial Void Ratio

SETTLEMENT FROM 1D PRIMARY CONSOLIDAITON

At End of Primary Consolidation Δσ = Δσ ´

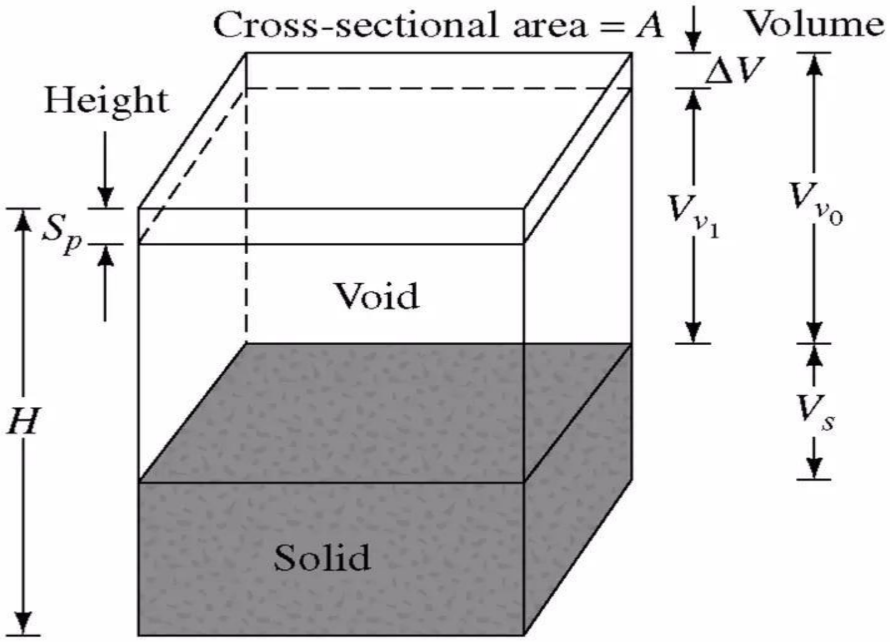


Figure 7.11. Das FGE (2005).

Therefore:

$$\Delta V = S_p A = \Delta e V_s = \frac{AH}{1+e_o}\Delta e$$

or

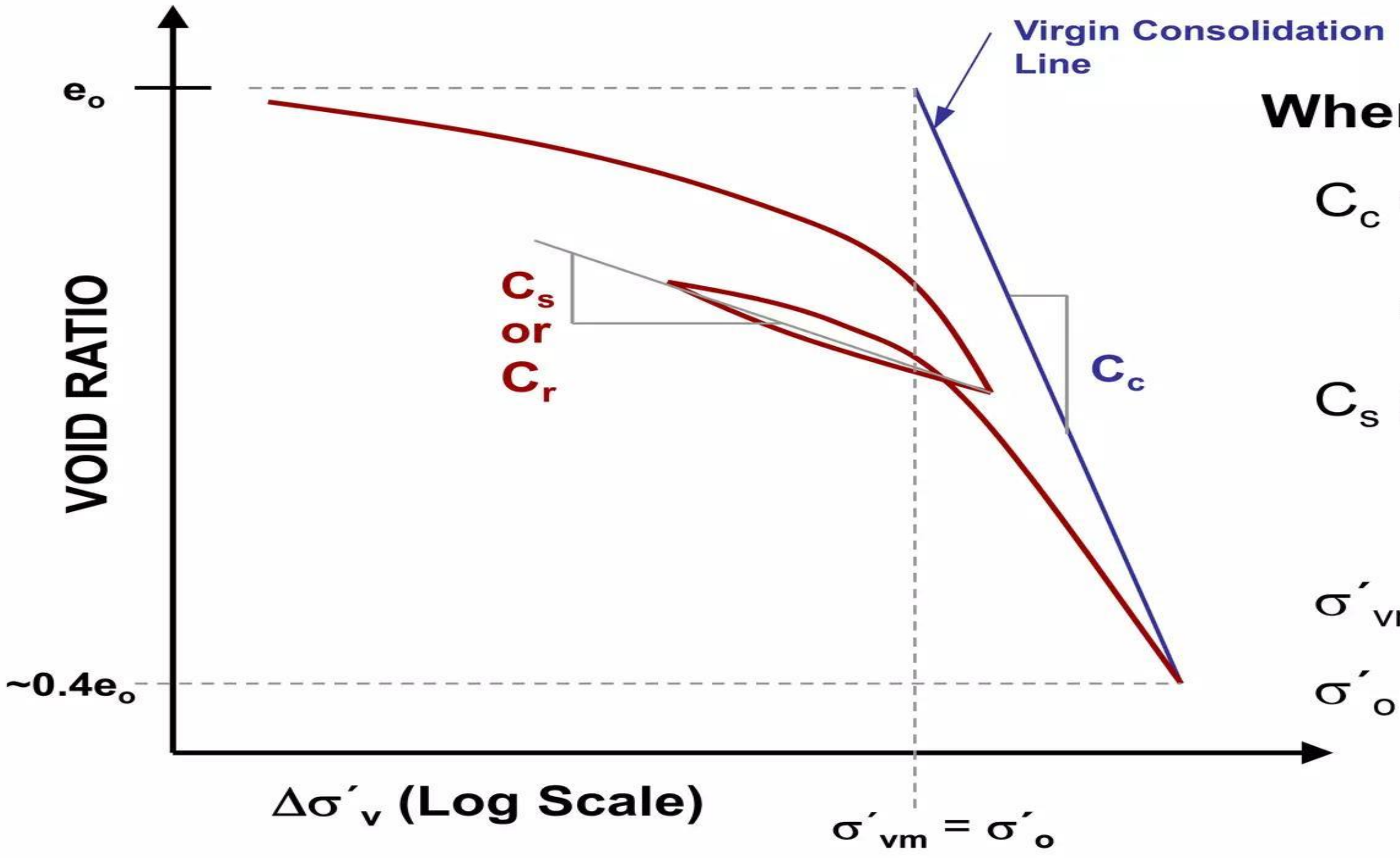
$$S_p = H\frac{\Delta e}{1+e_o}$$

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

Where:

ε_v = Vertical Strain

Settlement from 1D Primary Consolidation



NC Clay

Where:

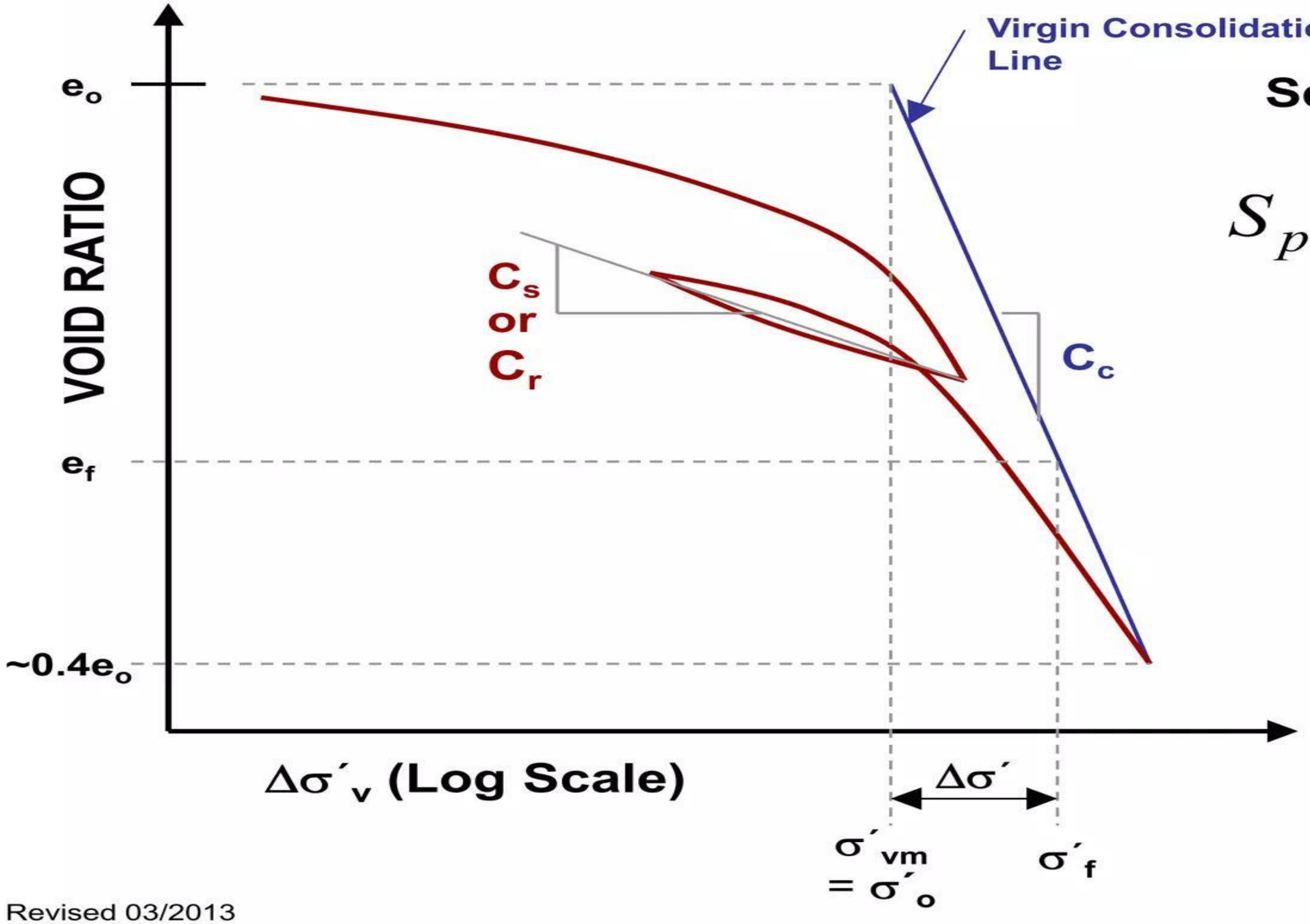
C_c = Slope of Field Virgin Consolidation Curve = **Compression Index**

C_s (or C_r) = Slope of Rebound Curve = **Swell Index**

σ'_{vm} = Maximum Past Pressure

σ'_o = Initial Vertical Effective Stress

Settlement from 1D Primary Consolidation



NC Clay

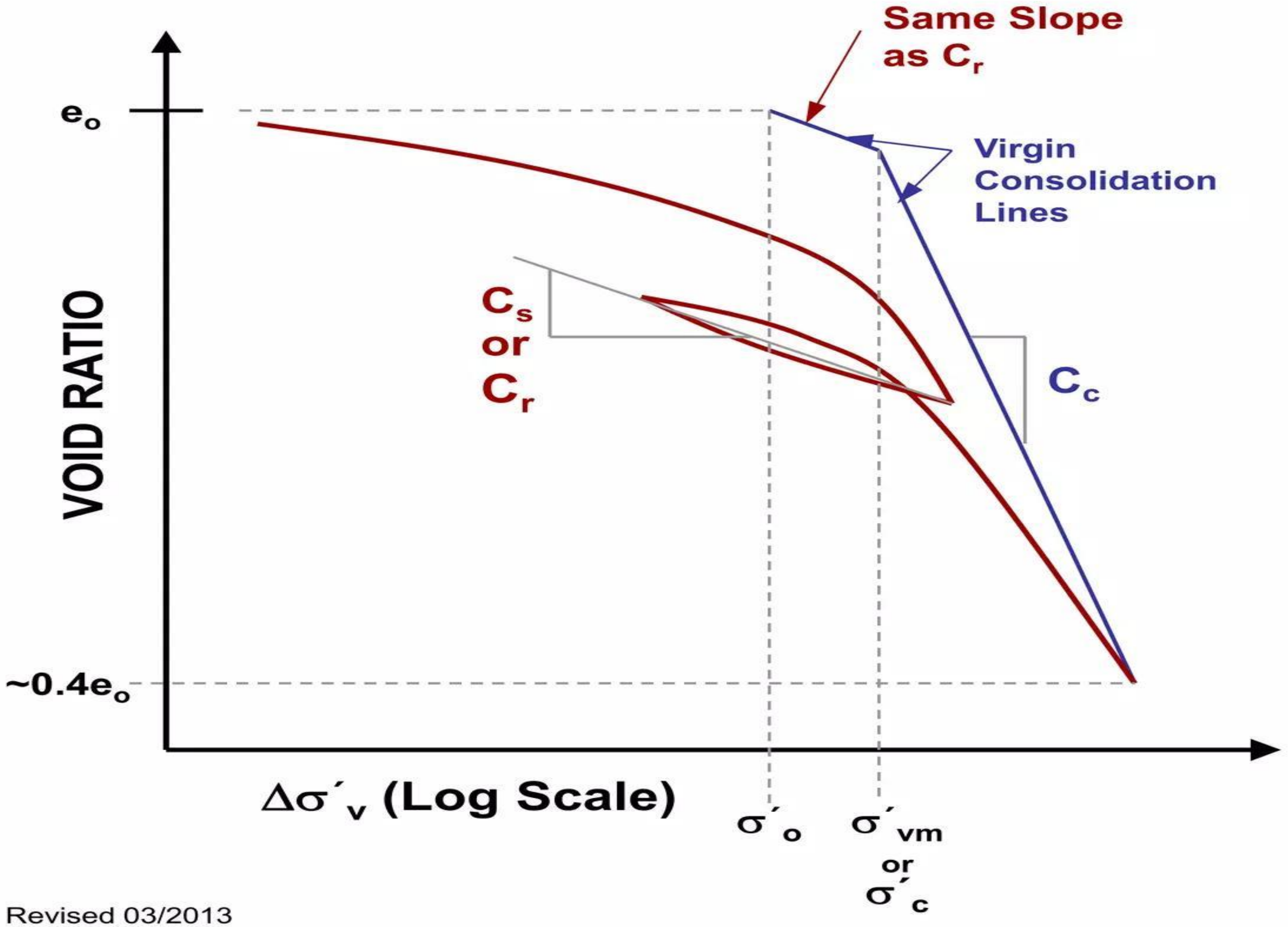
Settlement (S_p) using Void Ratio

$$S_p = \frac{C_c H}{1+e_0}\log\left(\frac{\sigma_o' + \Delta\sigma'}{\sigma_o'}\right)$$

Where:

- S_p = Settlement
- H = Height of Soil Layer
- σ'_{vm} = Final Vertical Effective Stress = σ_o' - Current Vertical Effective Stress
- $\Delta\sigma'$ = Change in Vertical Effective Stress
- σ_f' = Final Vertical Effective Stress
- e_f = Final Void Ratio

SETTLEMENT FROM 1D PRIMARY CONSOLIDATION



OC Clay

Where:

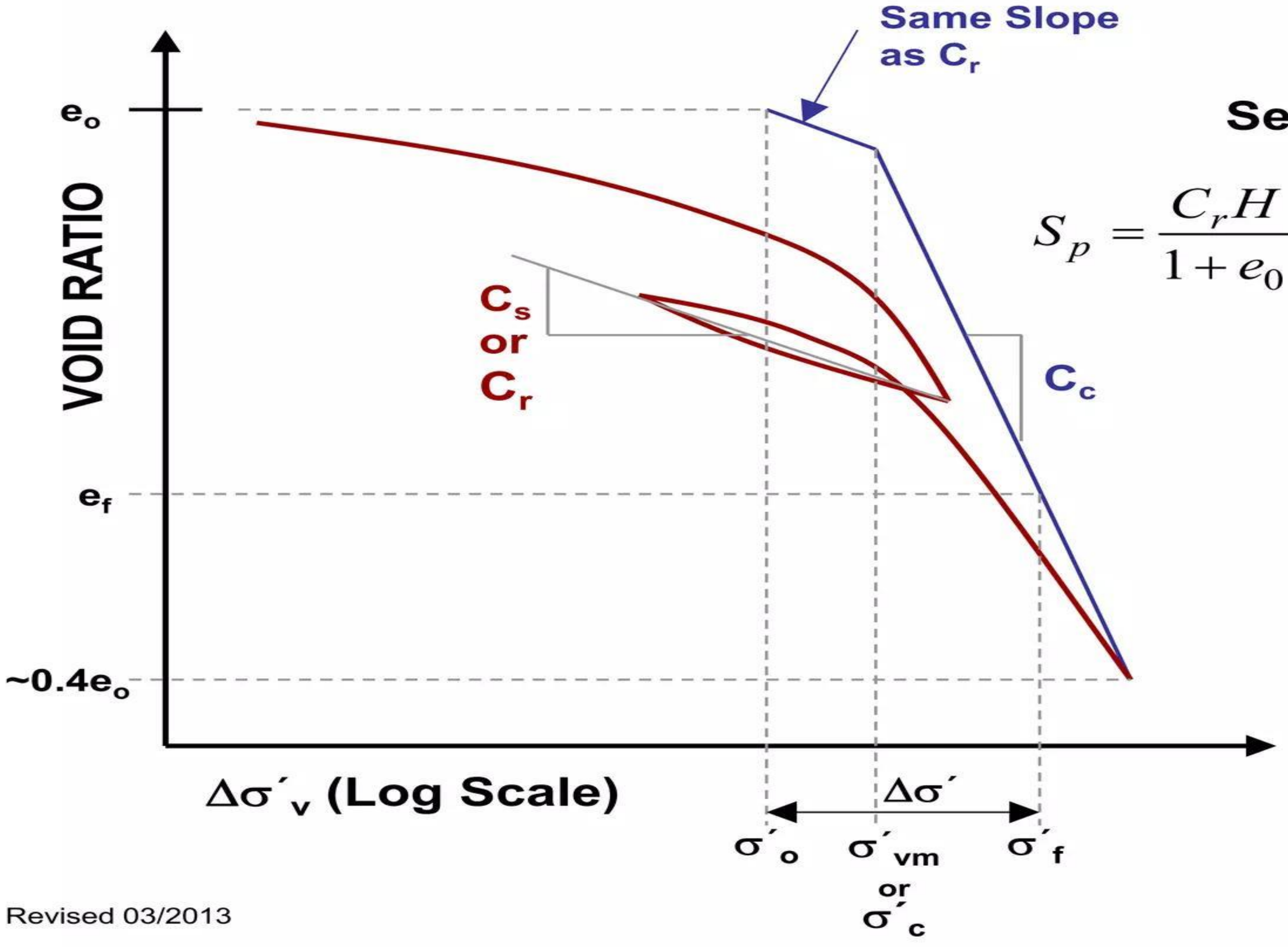
C_c = Slope of Field Virgin Consolidation Curve = **Compression Index**

C_s (or C_r) = Slope of Rebound Curve = **Swell Index**

σ'_{vm} = Maximum Past Pressure

σ'_o = Initial Vertical Effective Stress

SETTLEMENT FROM 1D PRIMARY CONSOLIDATION



OC Clay

Settlement (S_p) using Void Ratio

$$S_p = \frac{C_r H}{1+e_0}\log\left(\frac{\sigma'_{vm}}{\sigma'_o}\right) + \frac{C_c H}{1+e_0}\log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_o}\right)$$

Where:

S_p = Settlement
H = Height of Soil Layer
$\Delta\sigma'$ = Change in Vertical Effective Stress
σ_o' = Initial Vertical Effective Stress
σ_f' = Final Vertical Effective Stress
e_f = Final Void Ratio

SETTLEMENT FROM 1D PRIMARY CONSOLIDATION

Compression Index (C_c) Estimates from Other Laboratory Tests

Soil	C_c Equation	Reference
Undisturbed Clays	$C_c = 0.009(LL - 10)$	Terzaghi & Peck (1967)
Disturbed Clays	$C_c = 0.007(LL - 10)$	
Organic Soils, Peat	$C_c = 0.0115 W_n$	EM 1110-1-1904
Clays	$C_c = 1.15(e_o - 0.35)$	
	$C_c = 0.012 W_n$	
	$C_c = 0.01(LL - 13)$	
Varved Clays	$C_c = (1 + e_o) - [0.1 + 0.006(W_n - 25)]$	
Uniform Silts	$C_c = 0.20$	

SETTLEMENT FROM 1D PRIMARY CONSOLIDATION

Compression Index (C_c) Estimates from Other Laboratory Tests

Soil	C_c Equation	Reference
Clays	$C_c = 0.141 G_s^{1.2} \left(\frac{1+e_o}{G_s} \right)^{2.38}$	Rendon-Herrero (1983)
Clays	$C_c = 0.2343 \left[\frac{LL}{100} \right] G_s$	Nagaraj & Murty (1985)

Where:

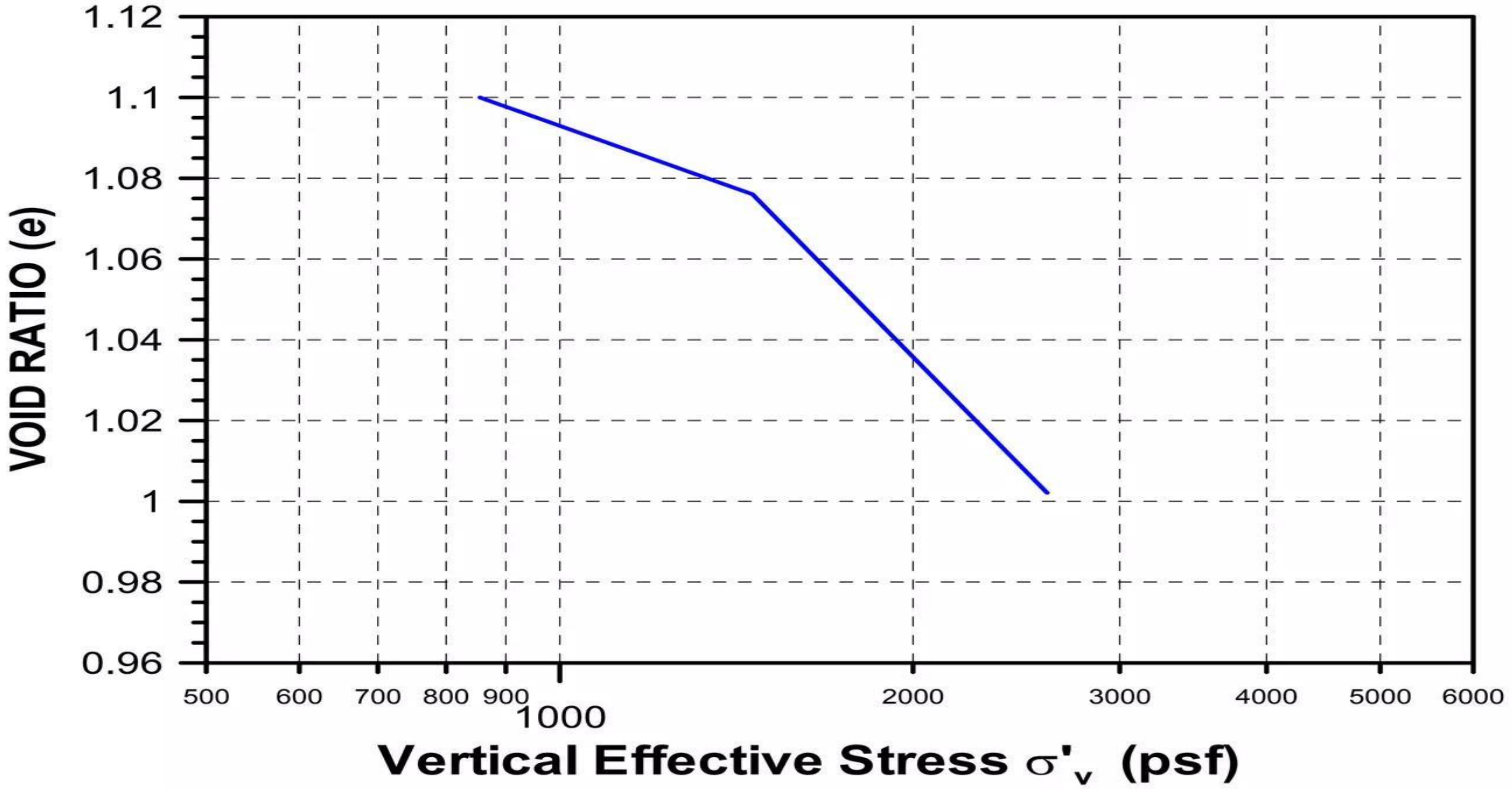
G_s = Specific Gravity of Solids
LL = Liquid Limit (in %)
W_n = Natural Water Content
e_o = Initial Void Ratio

SETTLEMENT FROM 1D PRIMARY CONSOLIDATION

Compression Index (C_c) Estimates from Other Laboratory Tests

Soil	C_c Equation	Reference
Clays	$C_c = 0.141 G_s^{1.2}\left(\frac{1+e_o}{G_s}\right)^{2.38}$	Rendon-Herrero (1983)
Clays	$C_c = 0.2343\left[\frac{LL}{100}\right]G_s$	Nagaraj & Murty (1985)

Example: Settlement from Virgin Consolidation Curves



GIVEN:

OC CH layer

$\sigma_o' = 855$ psf

$\sigma_{vm}' = 1460$ psf

$\Delta\sigma' = 1005$ psf

$e_o = 1.1$

Height of CH Layer = 10 ft

Figure 1. Example of Virgin Consolidation Curves.

Example: Settlement from Virgin Consolidation Curves

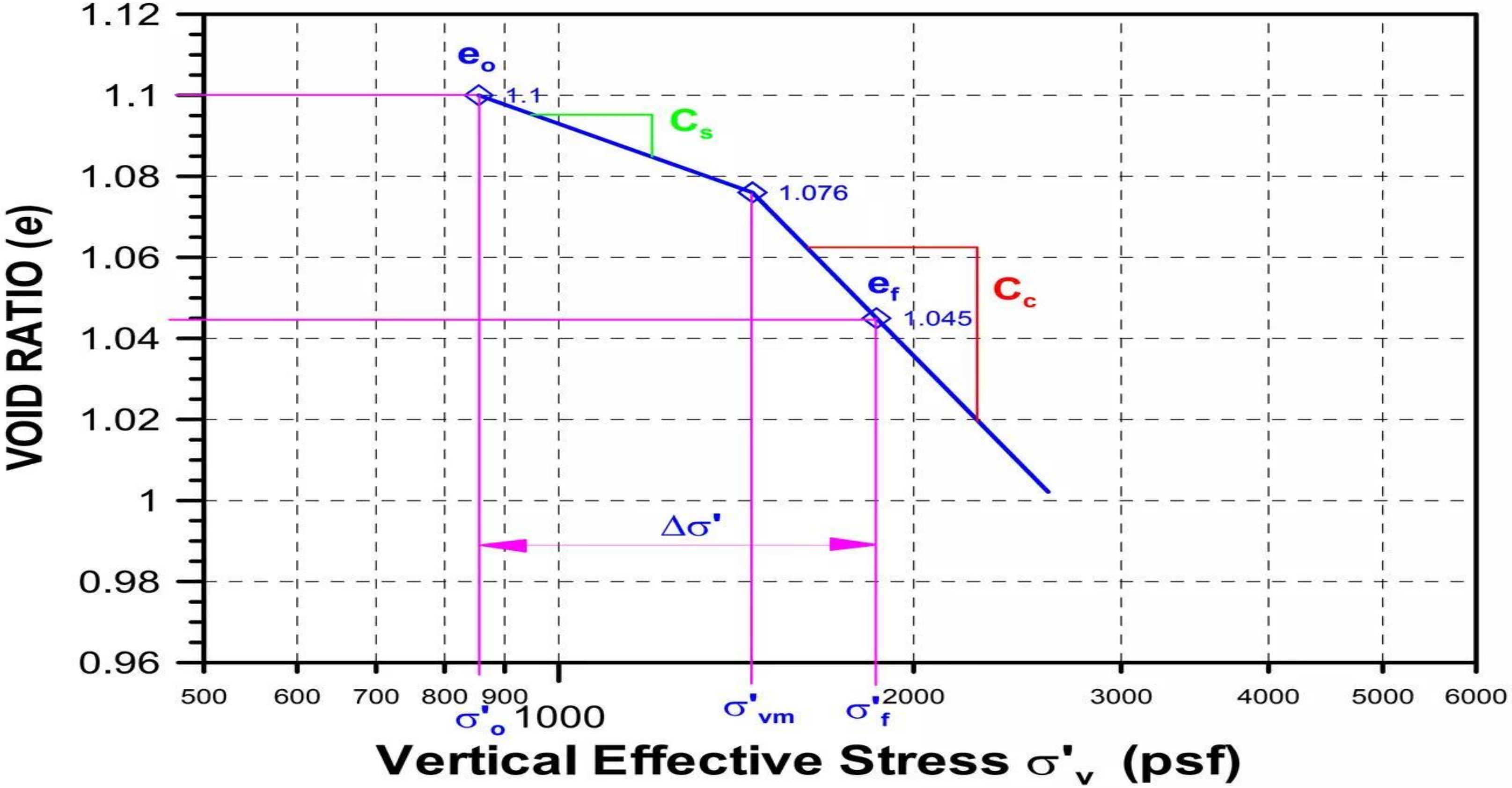


Figure 1. Example of Virgin Consolidation Curves.

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

$$\Delta e = 1.1 - 1.045 = 0.055$$

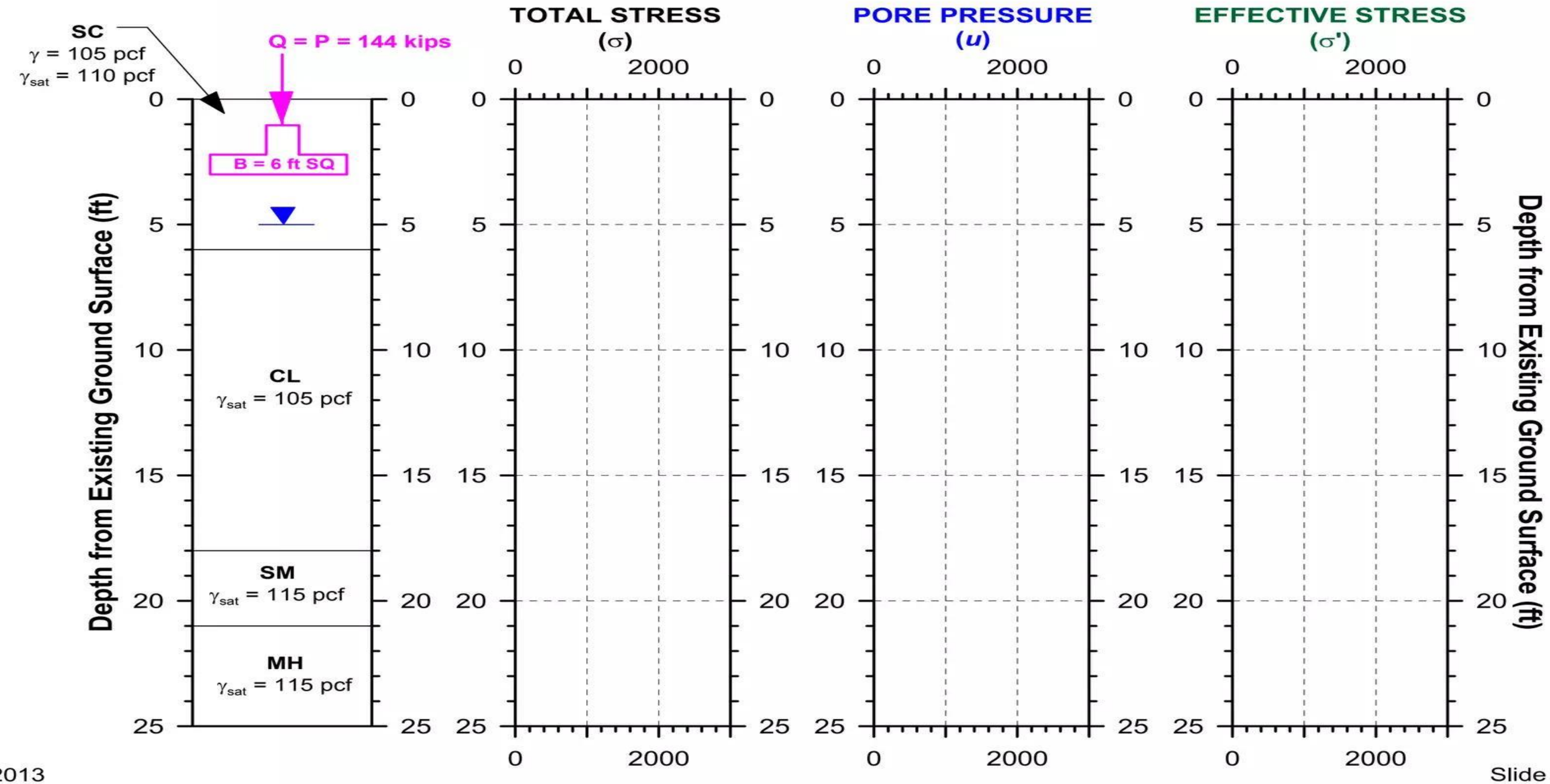
$$e_o = 1.1$$

$$S_p = (10\,ft)\left(\frac{0.055}{1+1.1}\right)$$

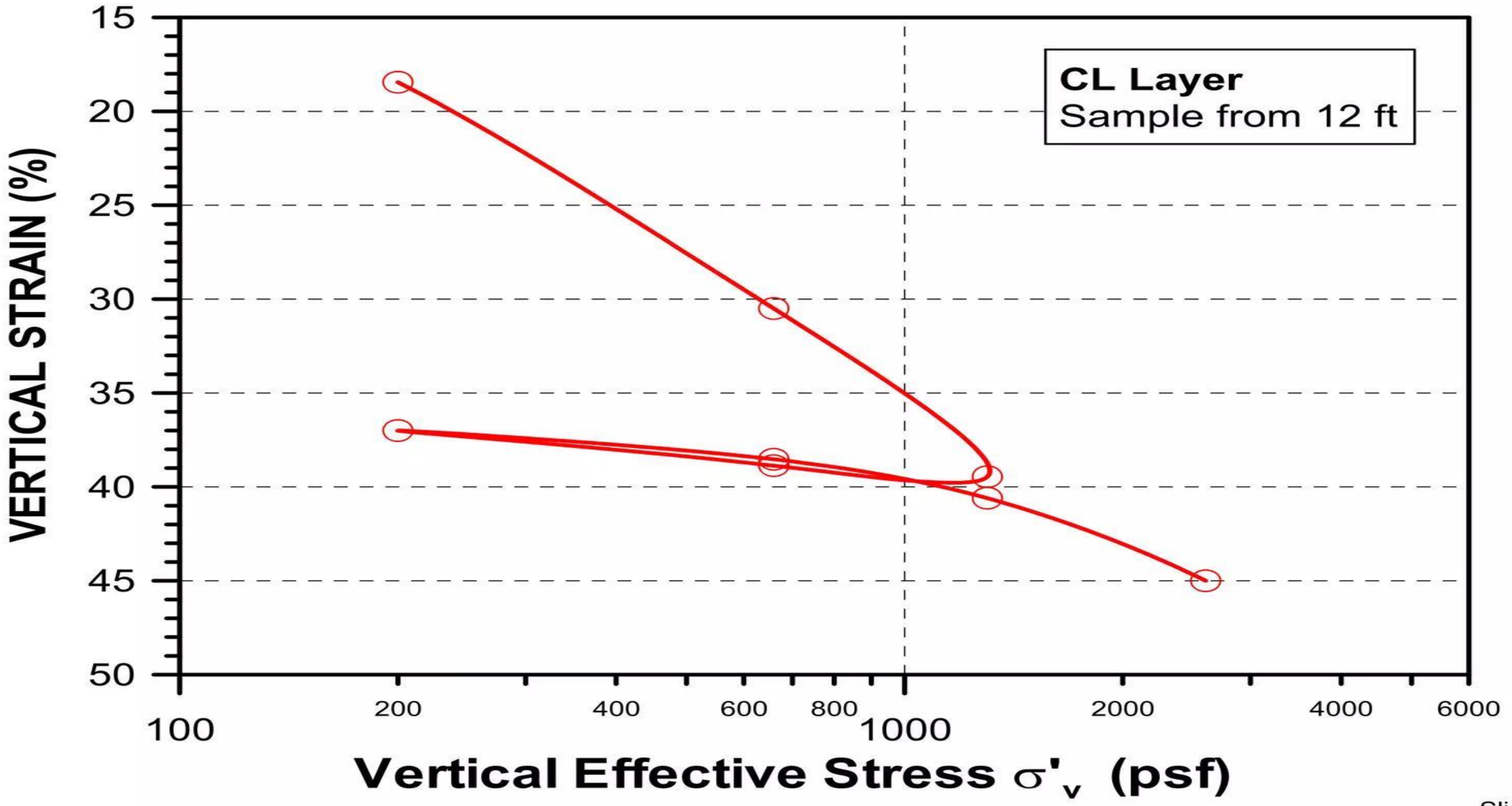
$$S_p = 0.262\,ft$$

$$S_p = 3.14 in = 3\frac{1}{4} in$$

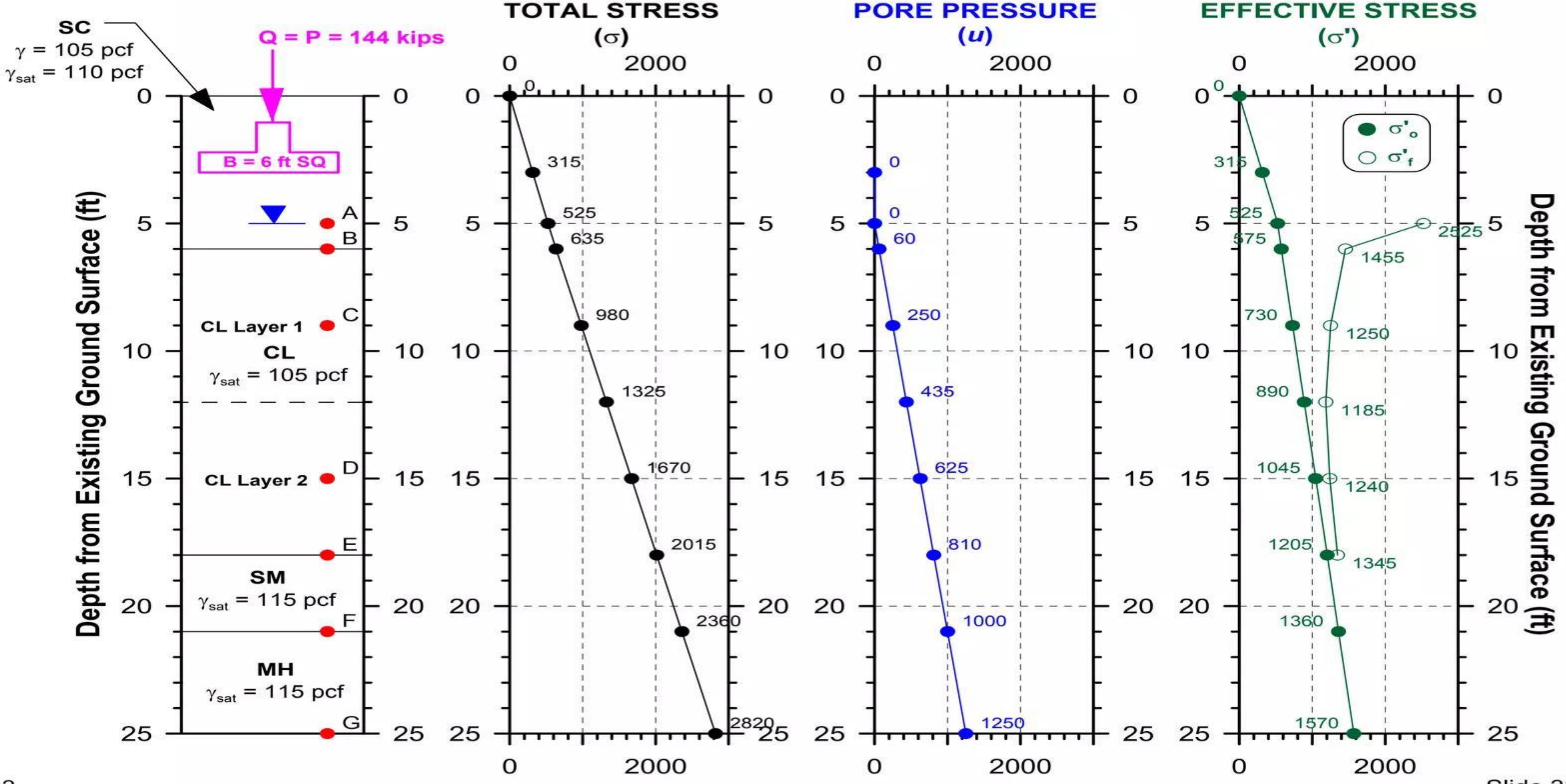
EXAMPLE: SETTLEMENT FROM 1D TEST STRAIN RESULTS



Example: Settlement from 1D Test Strain Results



EXAMPLE: SETTLEMENT FROM 1D TEST STRAIN RESULTS



EXAMPLE: SETTLEMENT FROM 1D TEST STRAIN RESULTS

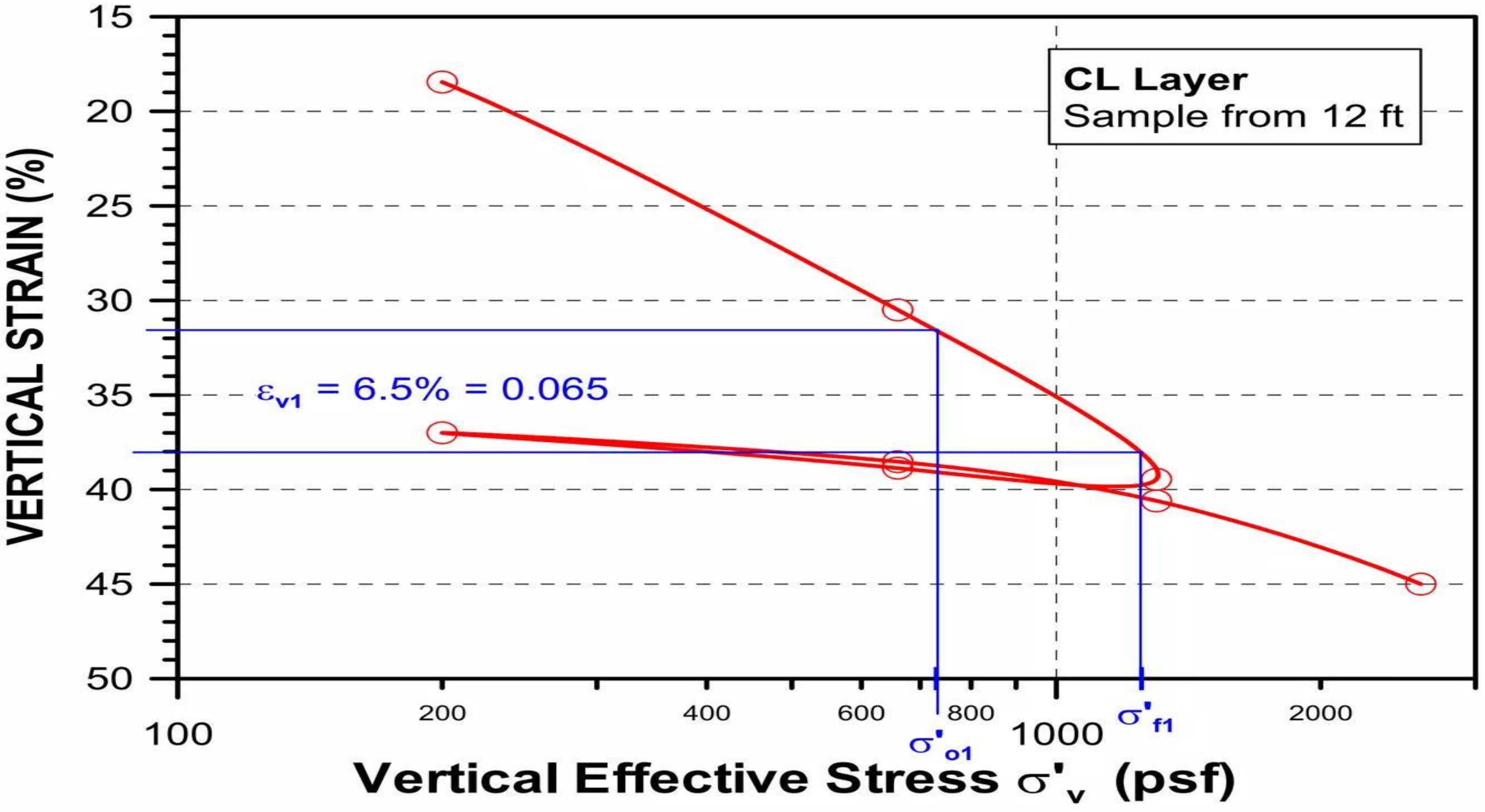
CL Layer 1

$$S_{p_1} = H_1 \varepsilon_{v1}$$

$$S_{p_1} = (6\,ft)(0.065)$$

$$S_{p_1} = 0.39\,ft$$

$$S_{p_1} = 4.7in$$



EXAMPLE: SETTLEMENT FROM 1D TEST STRAIN RESULTS

CL Layer 2

$$S_{p2} = H_2 \varepsilon_{v2}$$

$$S_{p2} = (6\,ft)(0.024)$$

$$S_{p2} = 0.14\,ft$$

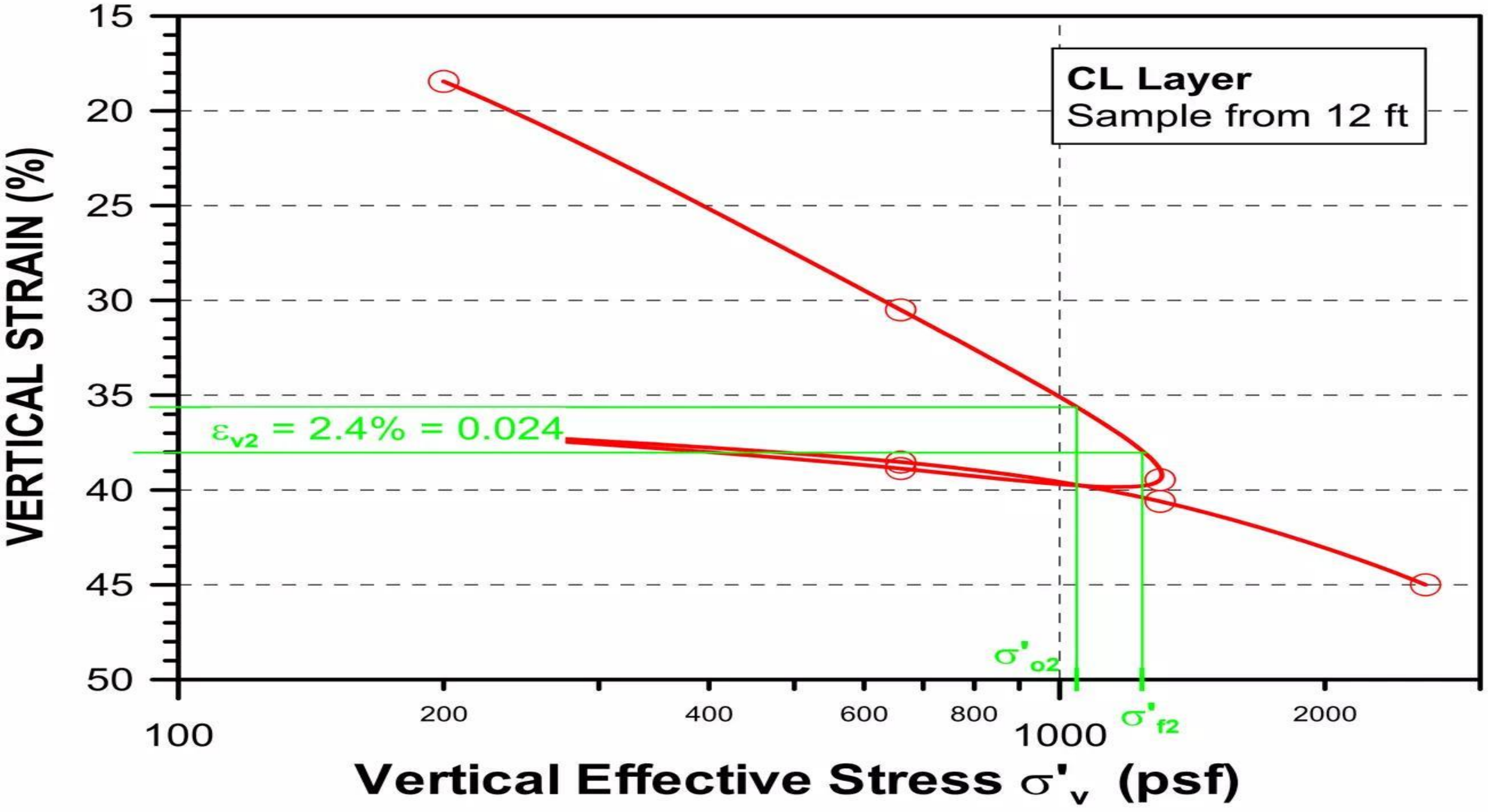
$$S_{p2} = 1.7in$$

Total Settlement

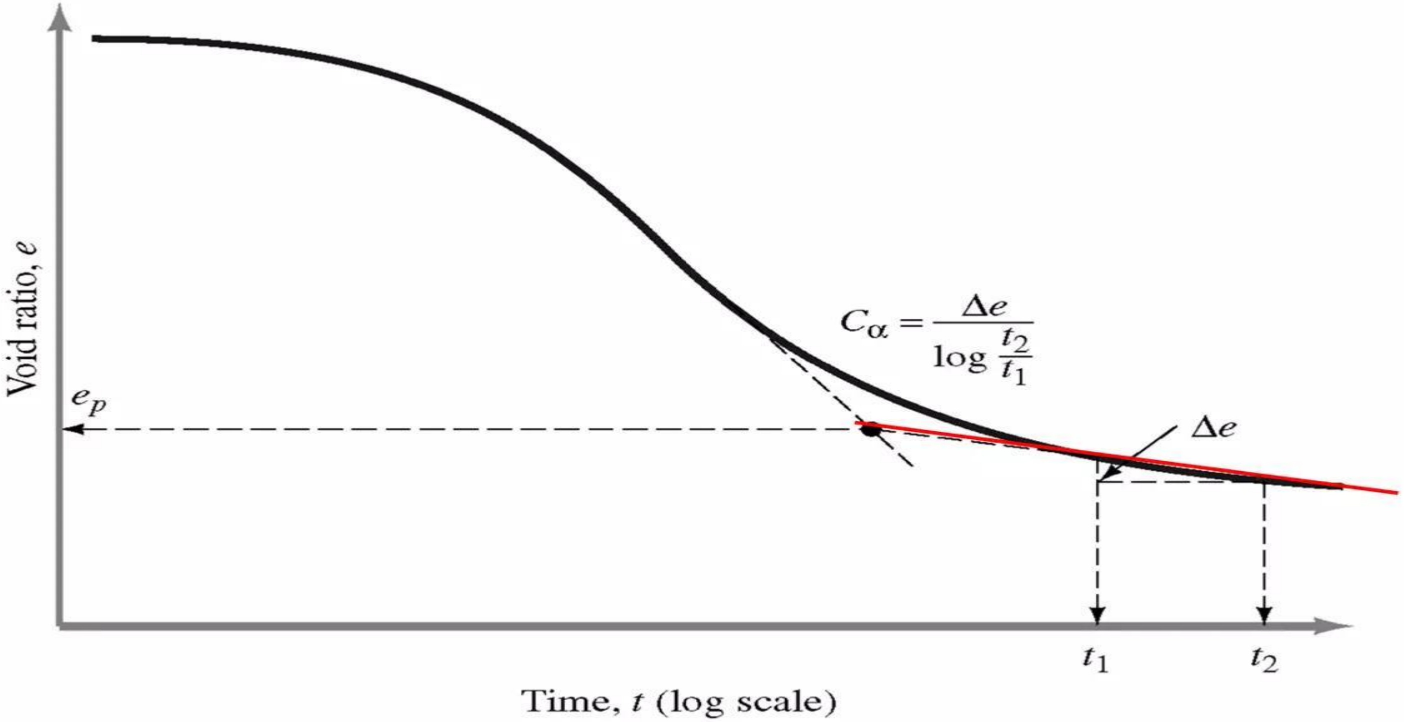
$$S_{Ptotal} = S_{p1} + S_{p2}$$

$$S_{Ptotal} = 6.4in$$

$$S_{Ptotal} = 6\frac{1}{2}in$$



Settlement from Secondary Consolidation



Results of 1D Consolidation Test @ One Load Increment

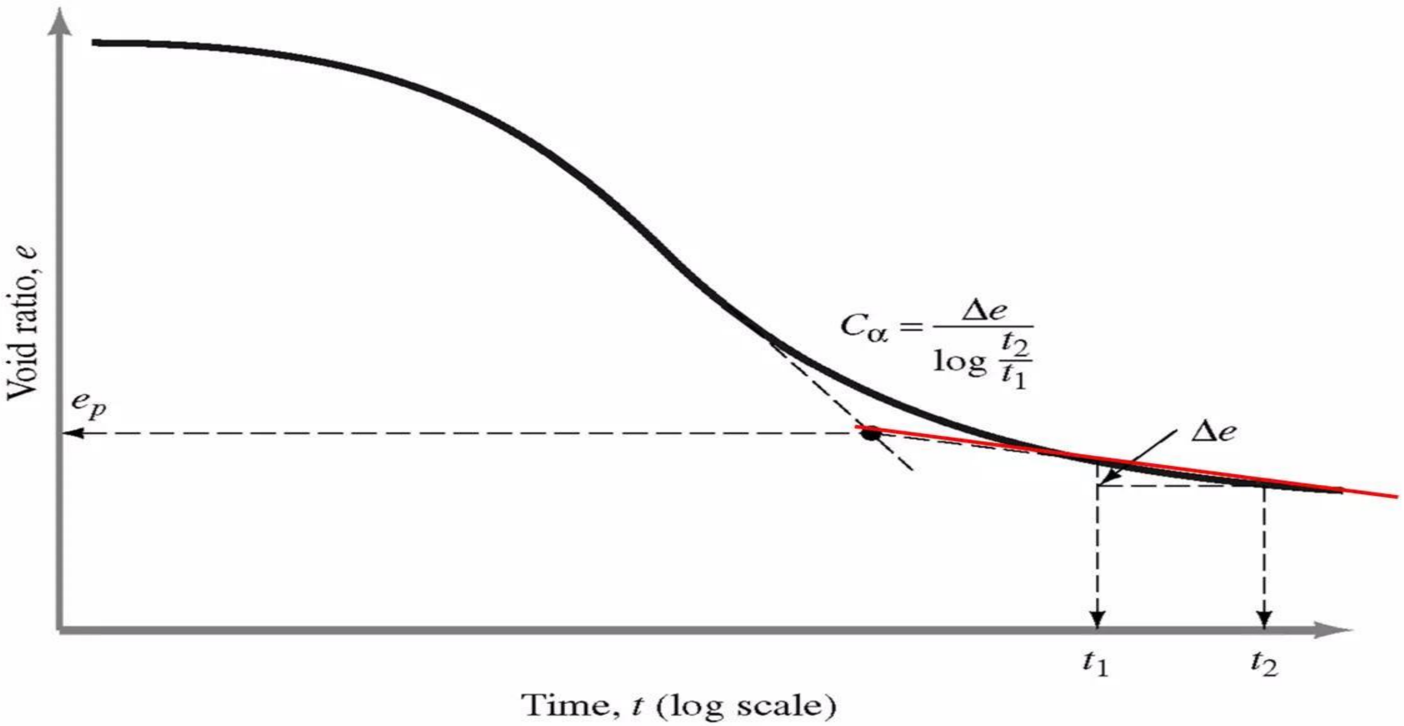
Figure 7.15. Das FGE (2005).

$$C_\alpha = \frac{\Delta e}{\log t_2 - \log t_1}$$

Where:

C_α = Secondary Compression Index
Δe = Change in Void Ratio
t = Time

SETTLEMENT FROM SECONDARY CONSOLIDATION



Results of 1D Consolidation Test @ One Load Increment

Figure 7.15. Das FGE (2005).

$$S_s = C'_\alpha H \log\left(\frac{t_2}{t_1}\right)$$

$$C'_\alpha = \frac{C_\alpha}{1 + e_p}$$

Where:

H = Height of Soil Layer
e_p = Void Ratio @ End of Primary Consolidation
t = Time

Soil Consolidation

Week 2

Pages 53-57

NORMALLY CONSOLIDATED SOIL:

- Which are loaded 1st time in the history to the present applied effective stress.
- Such soils are more compressible representing high settlement of loading.

OVER CONSOLIDATED SOIL:

- Which are loaded in the past history.
- Past applied effective stress is more than present applied effective stress.
- Such soils are less compressible and more stable.

Destroyed building loads, eroded loads, Malted glacier and Increased/decreased ground water are considered some preconsolidated pressure.

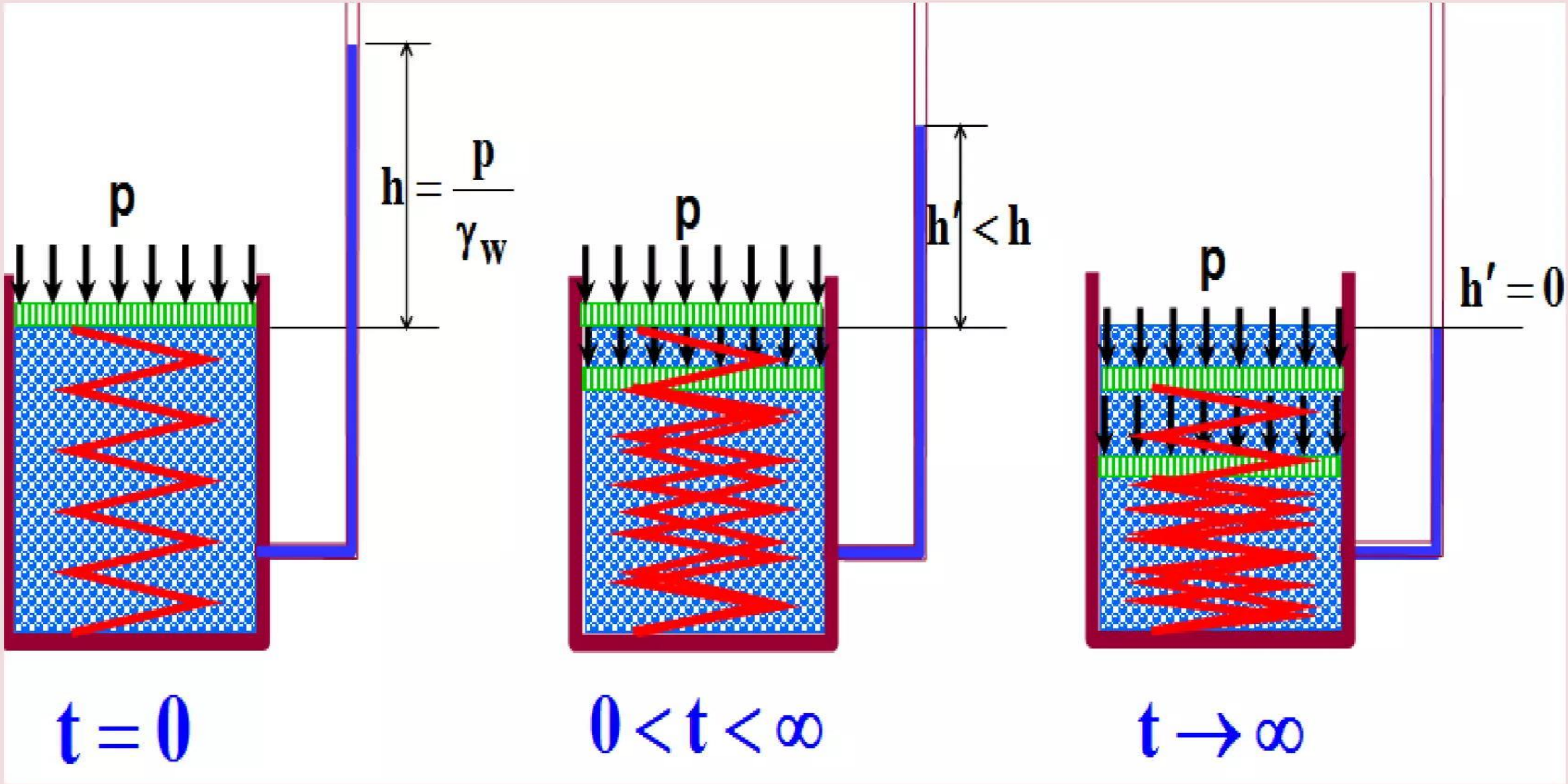
OVER CONSOLIDATION RATIO (OCR) :

$$\text{OCR} = \frac{\textit{Maximum applied effective stress in the past}}{\textit{present applied effective stress}}$$

1. For Over Consolidated Soil OCR > 1
2. For Heavily Over Consolidated Soil OCR > > 1
3. For Normally Consolidated Soil OCR ≤1

Maximum value of OCR for normally consolidated soil is 1.

TERZAGHI'S THEORY OF ONE-DIMENSIONAL CONSOLIDATION



The assumptions made in the theory are:

1. The soil is homogeneous and fully saturated.

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

3. The solid particles and water are incompressible.

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

5. Strains are small.

6. Darcy's law is valid at all hydraulic gradients.

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

The total stress increment

↓

soil skeleton

↓

increasing effective stress

↓

the excess pore water pressure decreases

$$\frac{\partial u}{\partial t} = \frac{k\left(1+e_1\right)}{\gamma_w a}\frac{\partial^2 u}{\partial z^2}$$

$$where, c_v = \frac{k(1+e_1)}{a\gamma_\omega}$$

- $\frac{\partial u}{\partial t}$ = Rate of change of pore pressure
- c_v = Co-efficient of consolidation
- $\frac{\partial^2 u}{\partial t^2}$ = Rate of change of pore pressure with depth
- u = Excess pore water

Soil Consolidation

Week 3

Pages 59-100

DEGREE OF CONSOLIDATION

1. Degree of consolidation represents percentage completion of 1st degree at any instant of time.

2. Theoretically consolidation is completed in infinite time. However practically it completes in certain finite time.

3. DOC can be computed in following three forms:

   a. When settlement is given.

   b. when void ratio is given.

   c. when excess pore pressure is given.

WHEN SETTLEMENT IS GIVEN:

△ H is ultimate consolidation settlement when degree of consolidation 100%.

△h is the settlement of soil after time "t".

If degree of consolidation is "u", then

$$\% \ U = \frac{\triangle h}{\triangle H} * 100$$

(The ultimate consolidation settlement does not depend on drainage condition while settlement at time "t" depends on drainage condition i.e one way or two way drainage condition).

WHEN VOID RATIO IS GIVEN

If e_0 = void ratio at the centre of soil at the beginning of consolidation.

e_{100} = void ratio at the centre of soil after completion of consolidation.

e = void ratio after time "t" at the centre of soil then

Degree of Consolidation, $\%U = \frac{e_0 - e}{e_0 - e_{100}} * 100$

1. At $t = 0$, $e = e_0$, $U = 0$
2. At $t = \infty$, $e = e_{100}$, $U = 100\%$.

WHEN EXCESS PORE PRESSURE IS GIVEN

If U_i is initial excess pore pressure at the beginning of test (t=0) and

U is the excess pore pressure after the time "t"

Then

Degree of Consolidation, $\%U = \frac{Ui-U}{Ui} * 100$

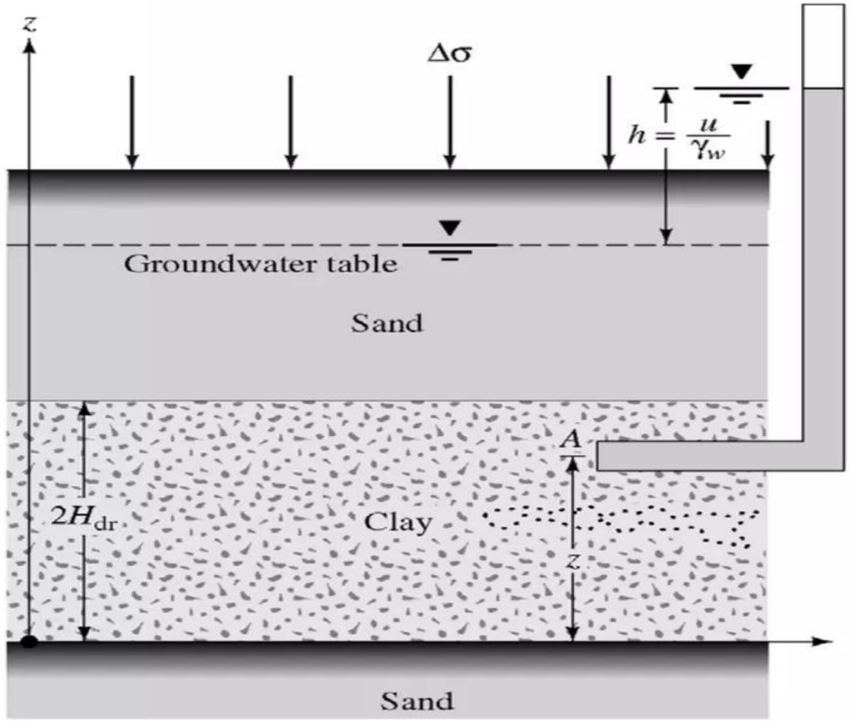
1. At $t = 0$, $U = U_i$, $U\% = 0\%$
2. At $t = \infty$, $U = 0$, $U\% = 100\%$

TIME FACTOR

Time factor is the parameter which relates to degree of consolidation and time required for that consolidation. It depends on the following:

1. Degree of Consolidation
2. Length of Drainage path
3. Time of Consolidation
4. Co- efficient of Consolidation

Time Rate of Consolidation



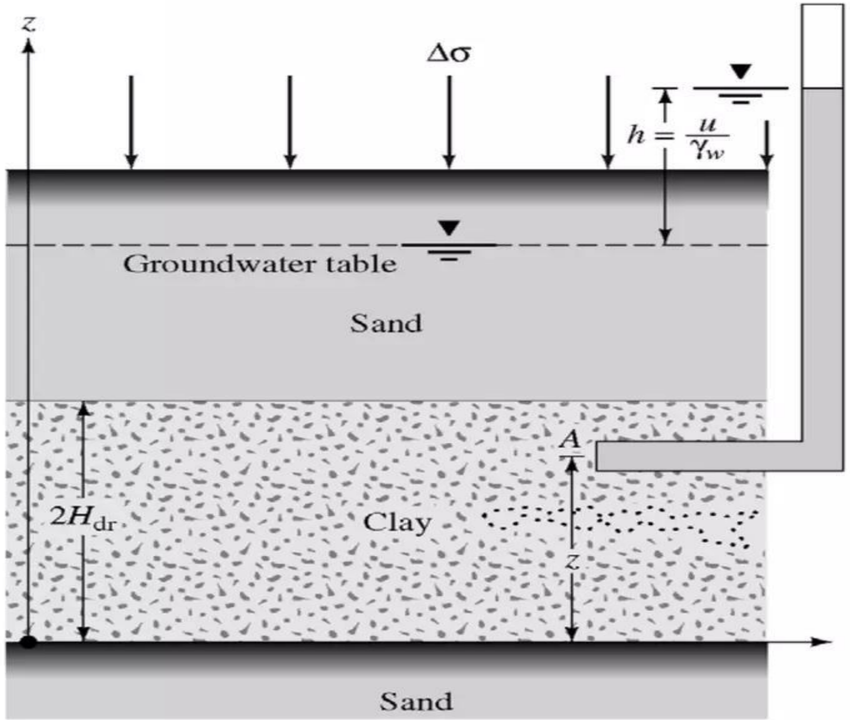
Clay Layer Undergoing Consolidation
Figure 7.17a. Das FGE (2005).

Theory of 1D Consolidation (Terzaghi, 1925)

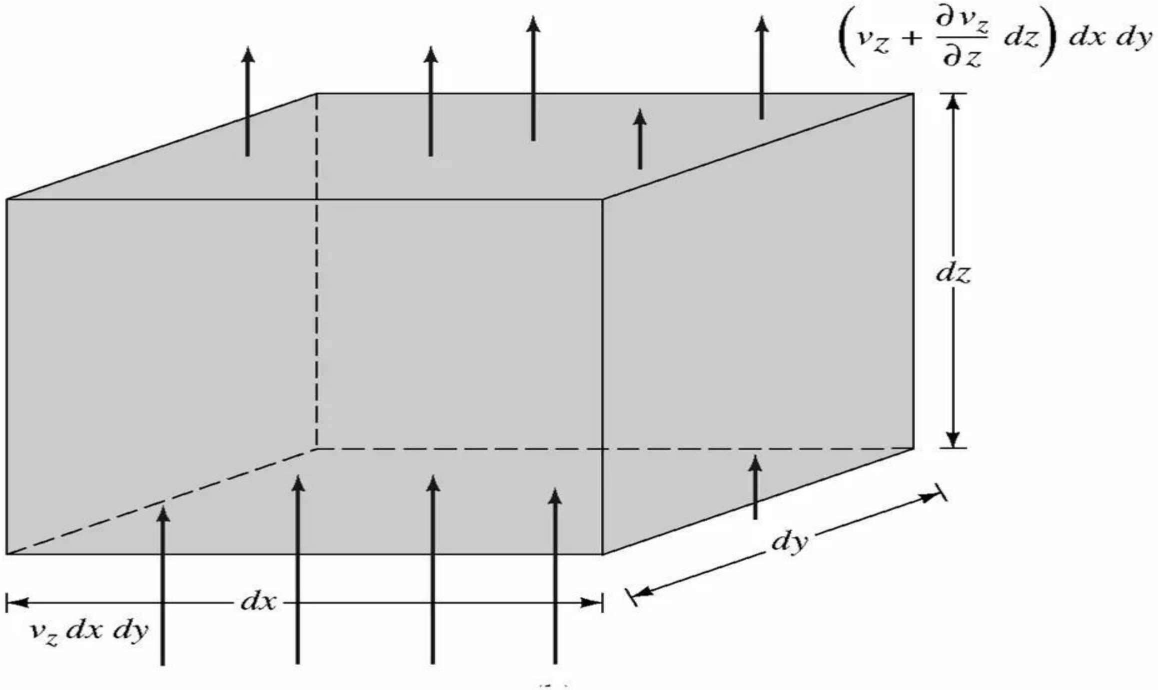
Assumptions:

1. The clay-water system is homogenous.
2. Saturation is complete (S = 100%).
3. Compressibility of water is negligible.
4. Compressibility of soil grains is negligible (but soil particles rearrange).
5. Flow of water is in one direction only.
6. Darcy's Law is Valid.

Time Rate of Consolidation

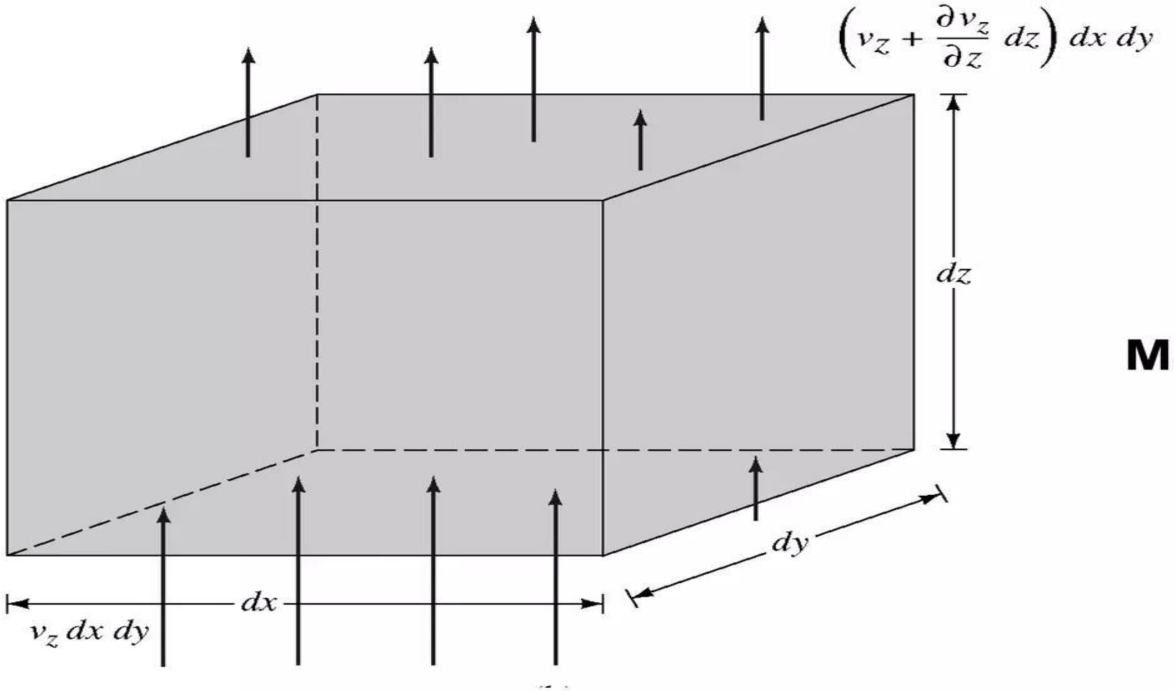


Clay Layer Undergoing Consolidation
Figure 7.17a. Das FGE (2005).



Flow of Water @ Point A
Figure 7.17b. Das FGE (2005).

TIME RATE OF CONSOLIDATION



Flow of Water @ Point A
Figure 7.17b. Das FGE (2005).

(Rate of Water Outflow) –

(Rate of Water Inflow) =

(Rate of Volume Changes)

Mathematical Equation:

$$\left(v_z + \frac{\partial v_z}{\partial z} dz\right) dxdy - v_z dxdy = \frac{\partial V}{dt}$$

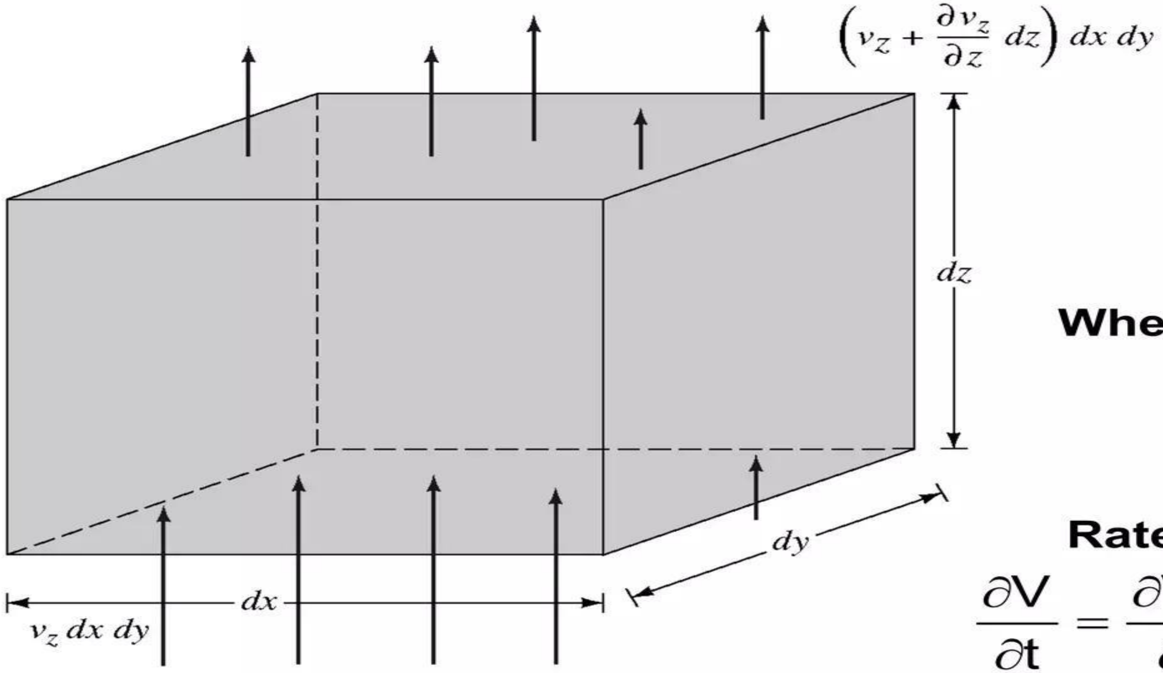
or

$$\frac{\partial v_z}{\partial z} dxdydz = \frac{\partial V}{dt}$$

Where:

V = Volume of Soil Element
v_z = Velocity of flow in z direction

TIME RATE OF CONSOLIDATION



Flow of Water @ Point A
Figure 7.17b. Das FGE (2005).

$$\frac{\partial v_z}{\partial z}dxdydz = \frac{\partial V}{\partial t}$$

Using Darcy's Law (*v* = *ki*)

$$v_z = ki = -k\frac{\partial h}{\partial z} = -\frac{k}{\gamma_w}\frac{\partial u}{\partial z}$$

Where *u* = excess pore pressure. From algebra:

$$-\frac{k}{\gamma_w}\frac{\partial^2 u}{\partial z^2} = \frac{1}{dxdydz}\frac{\partial V}{\partial t}$$

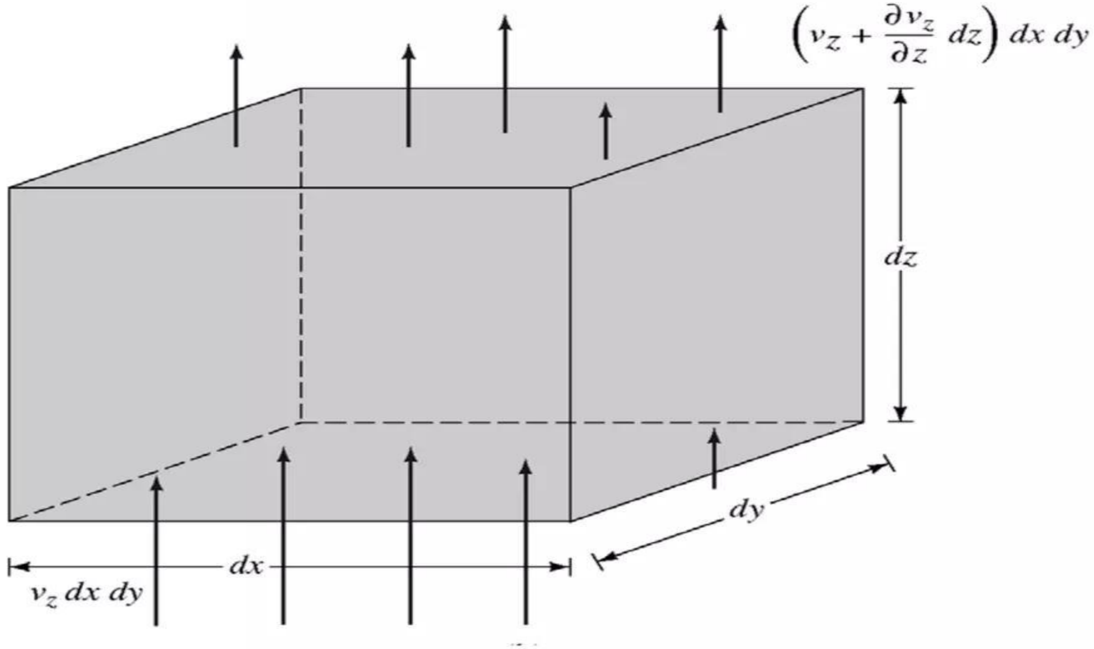
Rate of change in *V* = Rate of Change in V_v

$$\frac{\partial V}{\partial t} = \frac{\partial V_v}{\partial t} = \frac{\partial(V_s + eV_s)}{\partial t} = \frac{\partial V_s}{\partial t} + V_s\frac{\partial e}{\partial t} + e\frac{\partial V_s}{\partial t}$$

Where:

V_s = Volume of Solids
v_v = Volume of Voids

TIME RATE OF CONSOLIDATION



Flow of Water @ Point A

Figure 7.17b. Das FGE (2005).

From Previous Slide

$$\frac{\partial V}{\partial t} = \frac{\partial V_v}{\partial t} = \frac{\partial (V_s + eV_s)}{\partial t} = \frac{\partial V_s}{\partial t} + V_s \frac{\partial e}{\partial t} + e \frac{\partial V_s}{\partial t}$$

Assuming soil solids are incompressible

$$\frac{\partial V_s}{\partial t} = 0$$

and

$$V_s = \frac{V}{1+e_o} = \frac{dxdydz}{1+e_o}$$

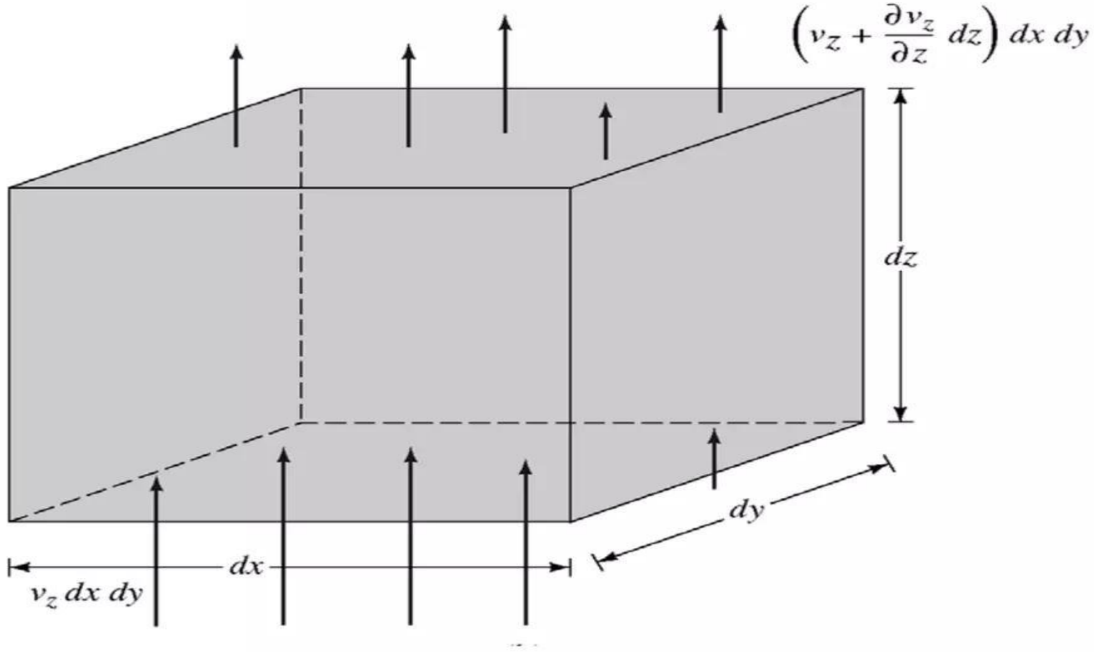
e_o = Initial Void Ratio. Substituting:

$$\frac{\partial V}{\partial t} = \frac{dxdydz}{1+e_o} \frac{\partial e}{\partial t}$$

Combining equations:

$$-\frac{k}{\gamma_w} \frac{\partial^2 u}{\partial z^2} = \frac{1}{1+e_o} \frac{\partial e}{\partial t}$$

TIME RATE OF CONSOLIDATION



Flow of Water @ Point A
Figure 7.17b. Das FGE (2005).

From Previous Slide $-\frac{k}{\gamma_w}\frac{\partial^2 u}{\partial z^2} = \frac{1}{1+e_o}\frac{\partial e}{\partial t}$

The change in void ratio is caused by the increase in effective stress. Assuming linear relationship between the two:

$$\partial e = a_v \partial(\Delta\sigma') = -a_v \partial u$$

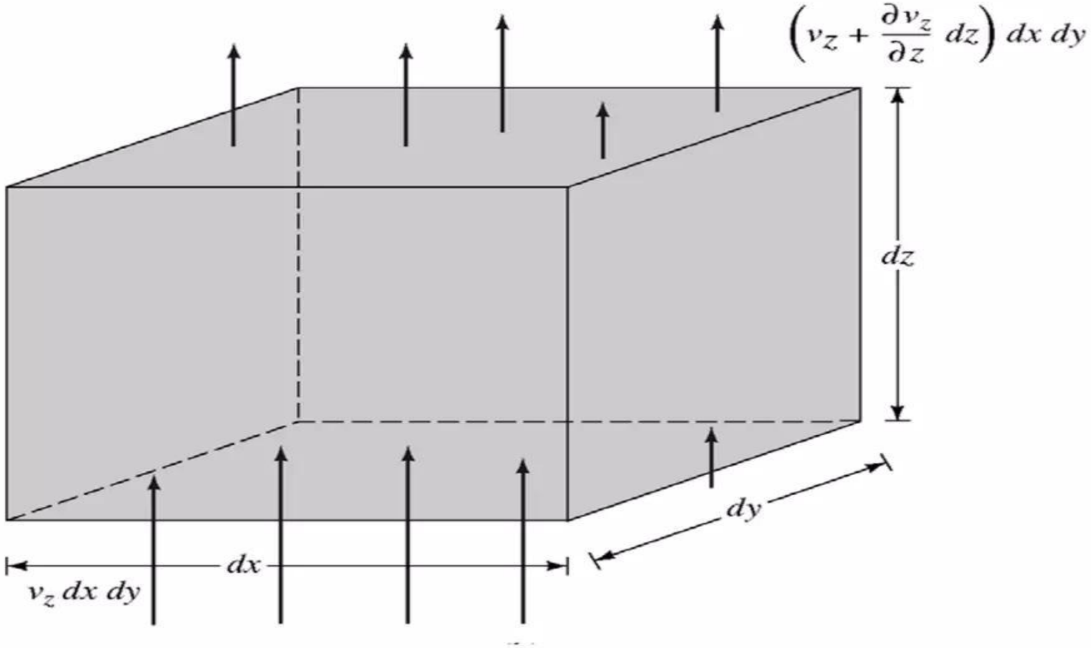
a_v = Coefficient of Compressibility. Can be considered constant over narrow pressure increases. Combining equations:

$$-\frac{k}{\gamma_w}\frac{\partial^2 u}{\partial z^2} = -\frac{a_v}{1+e_o}\frac{\partial u}{\partial t} = -m_v\frac{\partial u}{\partial t}$$

m_v = Coefficient of Volume Compressibility.

$$m_v = \frac{a_v}{1+e_o}$$

TIME RATE OF CONSOLIDATION



Flow of Water @ Point A

Figure 7.17b. Das FGE (2005).

From Previous Slide

$$-\frac{k}{\gamma_w}\frac{\partial^2 u}{\partial z^2} = -\frac{a_v}{1+e_o}\frac{\partial u}{\partial t} = -m_v\frac{\partial u}{\partial t}$$

a_v = Coefficient of Compressibility.
m_v = Coefficient of Volume Compressibility.

$$m_v = \frac{a_v}{1+e_o}$$

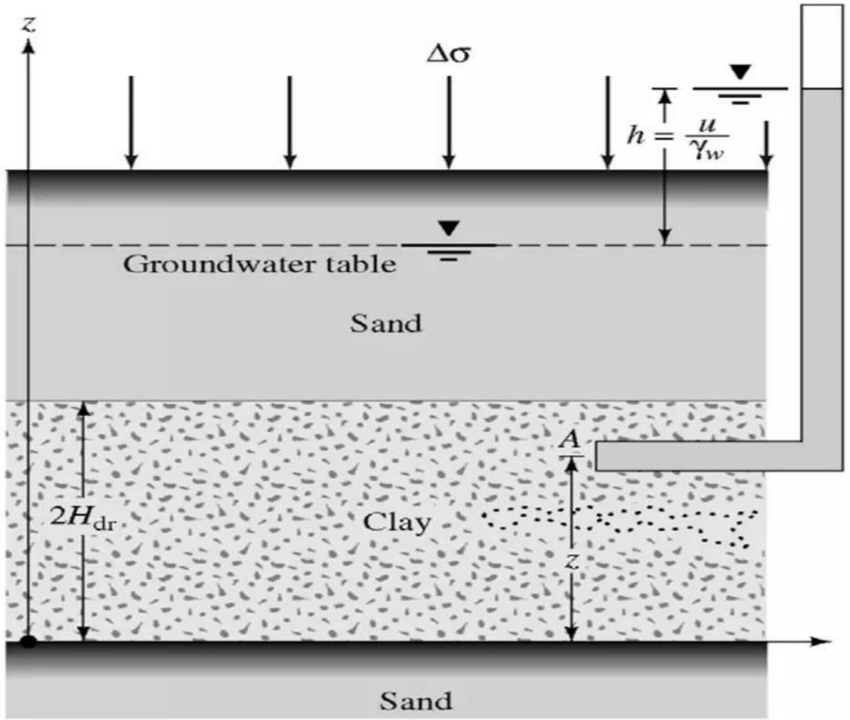
Rearranging Equations:

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

Where c_v = Coefficient of Consolidation.

$$c_v = \frac{k}{(\gamma_w m_v)}$$

TIME RATE OF CONSOLIDATION



Clay Layer Undergoing Consolidation
Figure 7.17a. Das FGE (2005).

Basic Differential Equation of 1D Consolidation Theory

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

Can be solved with the following boundary conditions:

$$z = 0, u = 0$$

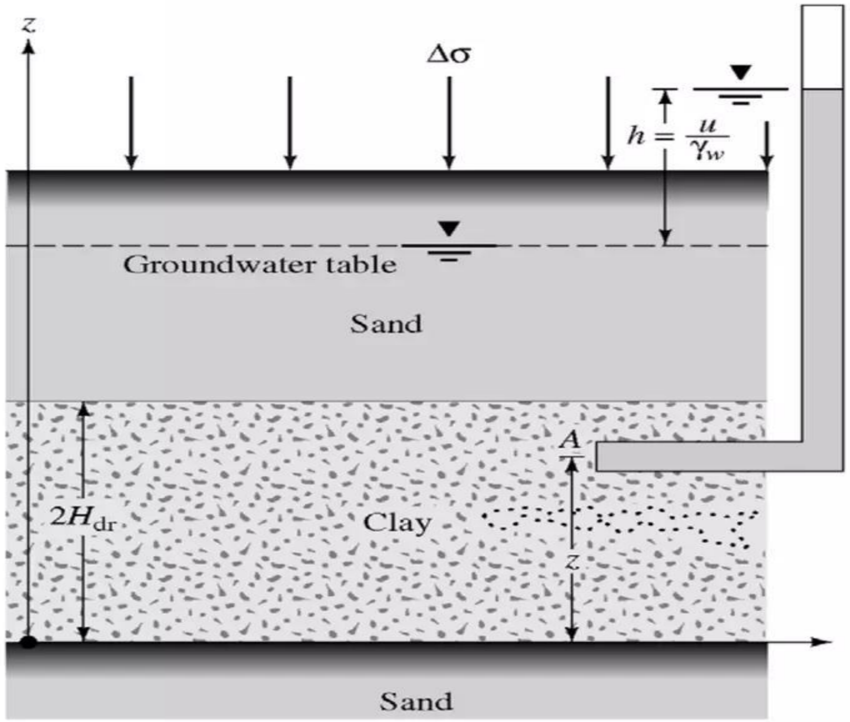
$$z = 2H_{dr}, u = 0$$

$$t = 0, u = u_0$$

The solution yields

$$u = \sum_{m=0}^{m=\infty} \left[\frac{2u_o}{M} \sin\left(\frac{Mz}{H_{dr}} \right) \right] e^{-M^2 T_v}$$

TIME RATE OF CONSOLIDATION



Clay Layer Undergoing Consolidation

Figure 7.17a. Das FGE (2005).

From Previous Slide

$$u = \sum_{m=0}^{m=\infty}\left[\frac{2u_o}{M}\sin\left(\frac{Mz}{H_{dr}}\right)\right]e^{-M^2T_v}$$

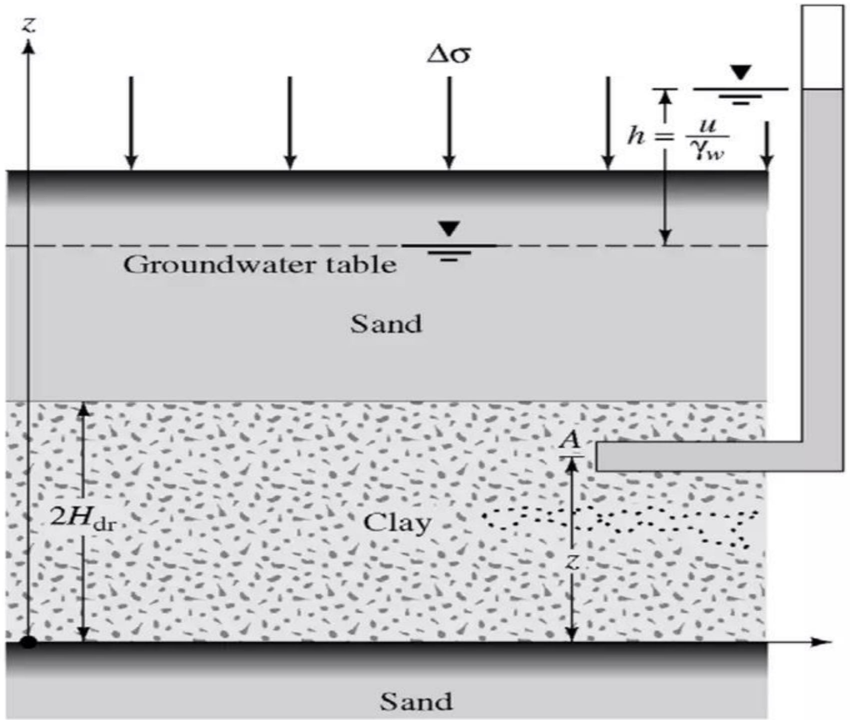
Where:

$$M = \frac{\pi}{2}(2m+1)$$

u_o = Initial excess pore water pressure

$$T_v = \frac{c_v t}{H^2_{dr}} = \text{TIME FACTOR}$$

TIME RATE OF CONSOLIDATION



Clay Layer Undergoing Consolidation

Figure 7.17a. Das FGE (2005).

Because consolidation progress by dissipation of excess pore pressure, the degree of consolidation (U_z) at a distance z at any time t is:

$$U_z = \frac{u_o - u_z}{u_o} = 1 - \frac{u_z}{u_o}$$

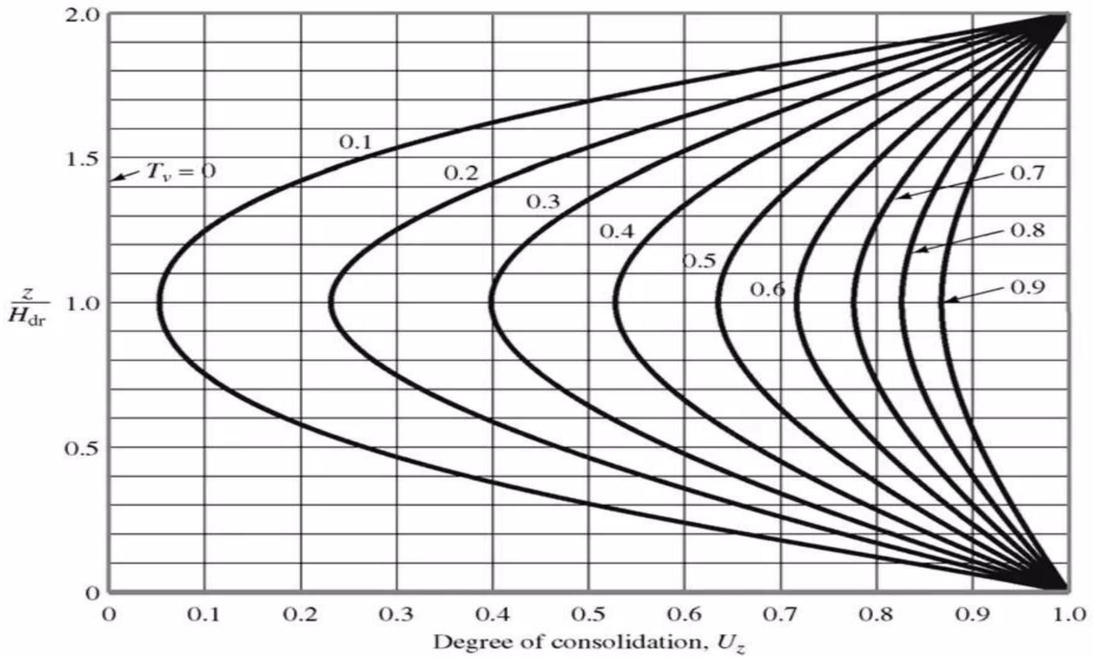


Figure 7.18. Das FGE (2006).

Time Rate of Consolidation

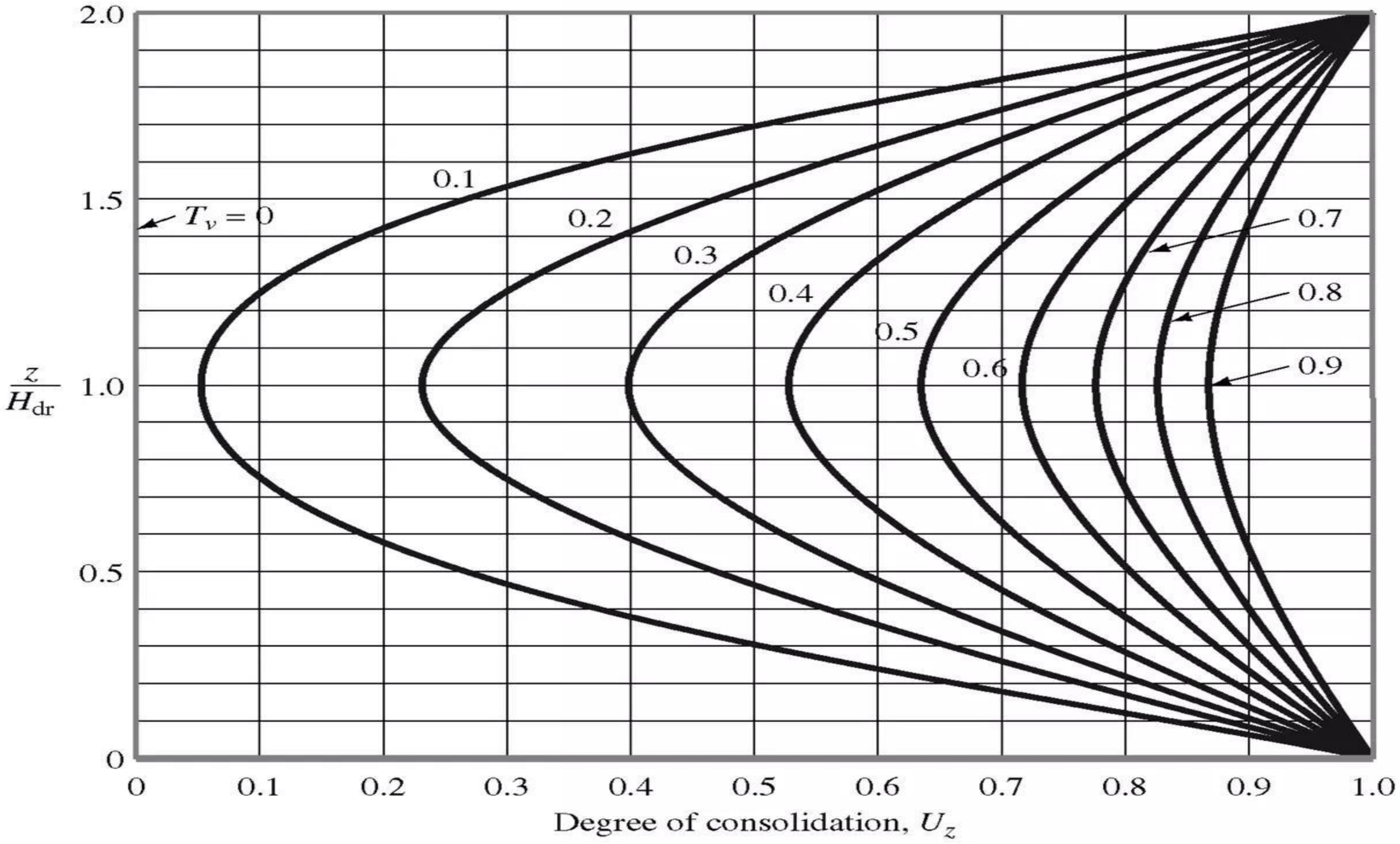
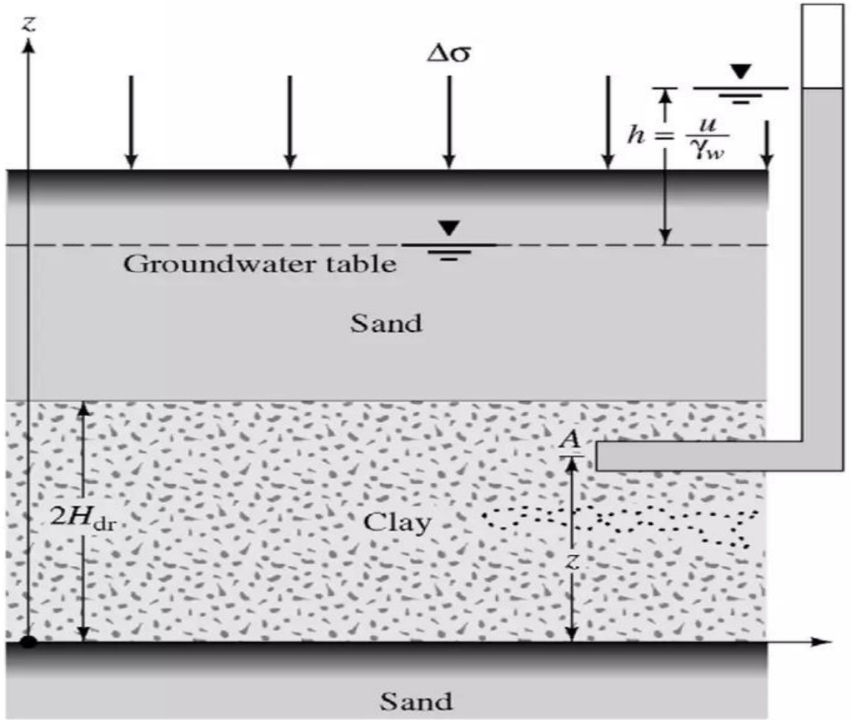


Figure 7.18. Das FGE (2006).

TIME RATE OF CONSOLIDATION



Clay Layer Undergoing Consolidation
Figure 7.17a. Das FGE (2005).

Average degree of consolidation (U) for the entire depth of the clay layer at any time t is:

$$U = \frac{S_t}{S_p} = 1 - \frac{\left(\frac{1}{2H_{dr}}\right)\int_0^{2H_{dr}} u_z dz}{u_o}$$

Where:

U = Average degree of Consolidation

S_t = Settlement of layer at time t

S_p = Settlement of Layer from Primary Consolidation

Substituting *U* for *u*

$$U = 1 - \sum_{m=0}^{m=\infty} \frac{2}{M^2} e^{-M^2 T_v}$$

***U* can be approximated by the following relationships:**

For $U = 0\%$ to 60%, $T_v = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2$

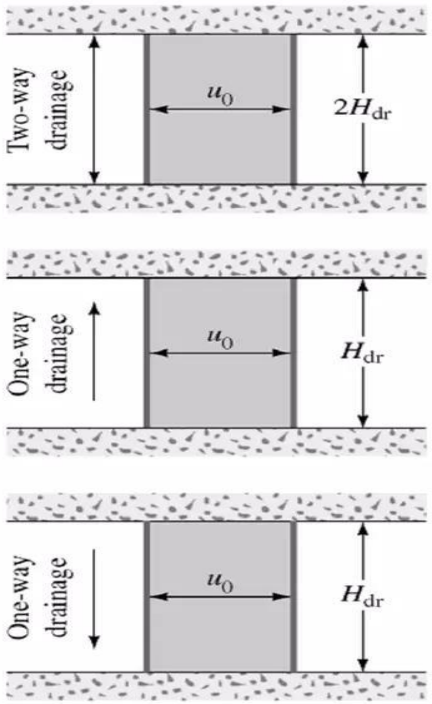
For $U > 60\%$, $T_v = 1.781 - 0.933\log(100 - U\%)$

Time Rate of Consolidation

Variation of T_v with U
Table 7.1 Das PGE (2006).

U (%)	T_v	U (%)	T_v	U (%)	T_v
0	0	34	0.0907	68	0.377
1	0.00008	35	0.0962	69	0.390
2	0.0003	36	0.102	70	0.403
3	0.00071	37	0.107	71	0.417
4	0.00126	38	0.113	72	0.431
5	0.00196	39	0.119	73	0.446
6	0.00283	40	0.126	74	0.461
7	0.00385	41	0.132	75	0.477
8	0.00502	42	0.138	76	0.493
9	0.00636	43	0.145	77	0.511
10	0.00785	44	0.152	78	0.529
11	0.0095	45	0.159	79	0.547
12	0.0113	46	0.166	80	0.567
13	0.0133	47	0.173	81	0.588
14	0.0154	48	0.181	82	0.610
15	0.0177	49	0.188	83	0.633
16	0.0201	50	0.197	84	0.658
17	0.0227	51	0.204	85	0.684
18	0.0254	52	0.212	86	0.712
19	0.0283	53	0.221	87	0.742
20	0.0314	54	0.230	88	0.774
21	0.0346	55	0.239	89	0.809
22	0.0380	56	0.248	90	0.848
23	0.0415	57	0.257	91	0.891
24	0.0452	58	0.267	92	0.938
25	0.0491	59	0.276	93	0.993
26	0.0531	60	0.286	94	1.055
27	0.0572	61	0.297	95	1.129
28	0.0615	62	0.307	96	1.219
29	0.0660	63	0.318	97	1.336
30	0.0707	64	0.329	98	1.500
31	0.0754	65	0.304	99	1.781
32	0.0803	66	0.352	100	∞
33	0.0855	67	0.364		

* u_0 constant with depth.



Different types of drainage with u_0 constant

Revised 03/2013

TIME RATE OF CONSOLIDATION

Difference between Average Degree of Consolidation and Midplane Degree of Consolidation

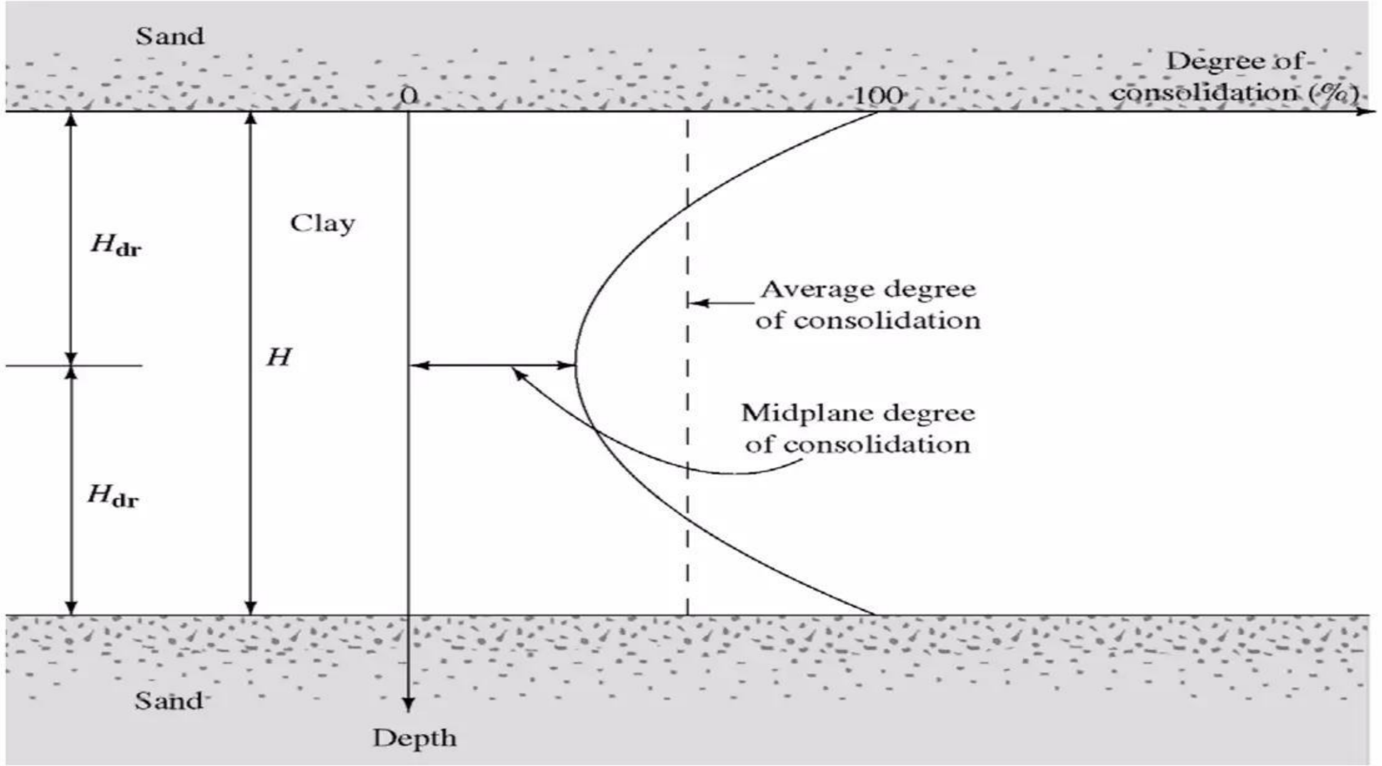
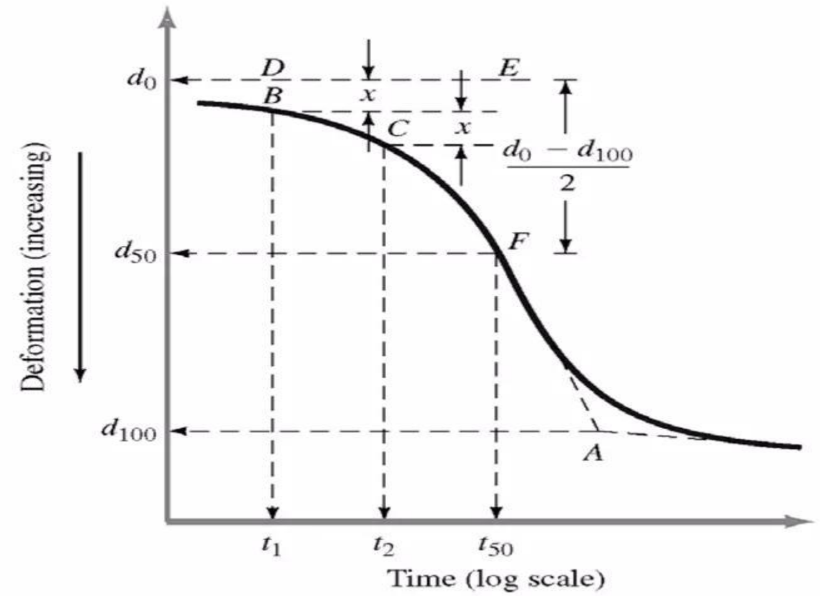


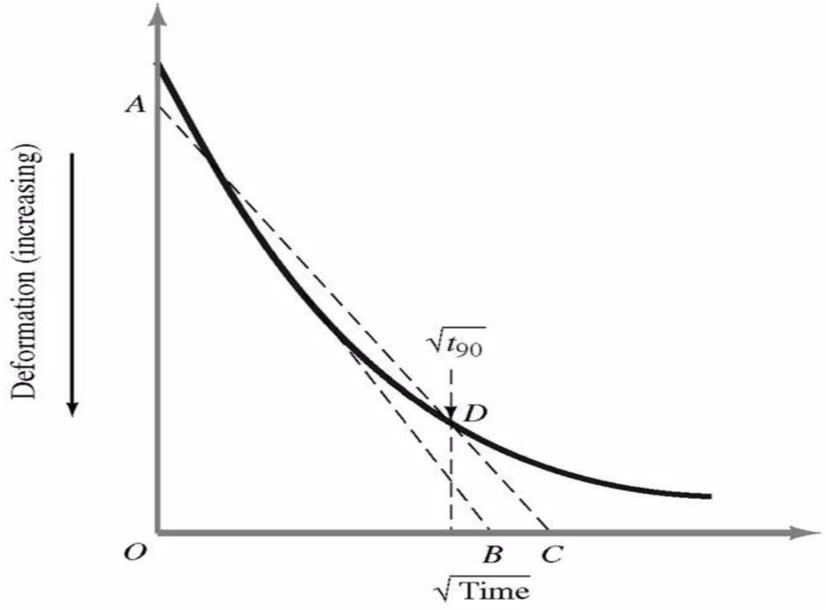
Figure 7.28. Das FGE (2006).

COEFFICIENT OF CONSOLIDATION (c_v)

- Generally decreases as Liquid Limit (LL) increases.
- Determined from 1D Consolidation Test Lab per Load Increment.



Logarithm of Time Method
(Casagrande and Fadum, 1940)
Figure 7.19 Das FGE (2006).



Square Root of Time Method
(Taylor, 1942)
Figure 7.20 Das FGE (2006).

COEFFICIENT OF CONSOLIDATION (c_v)

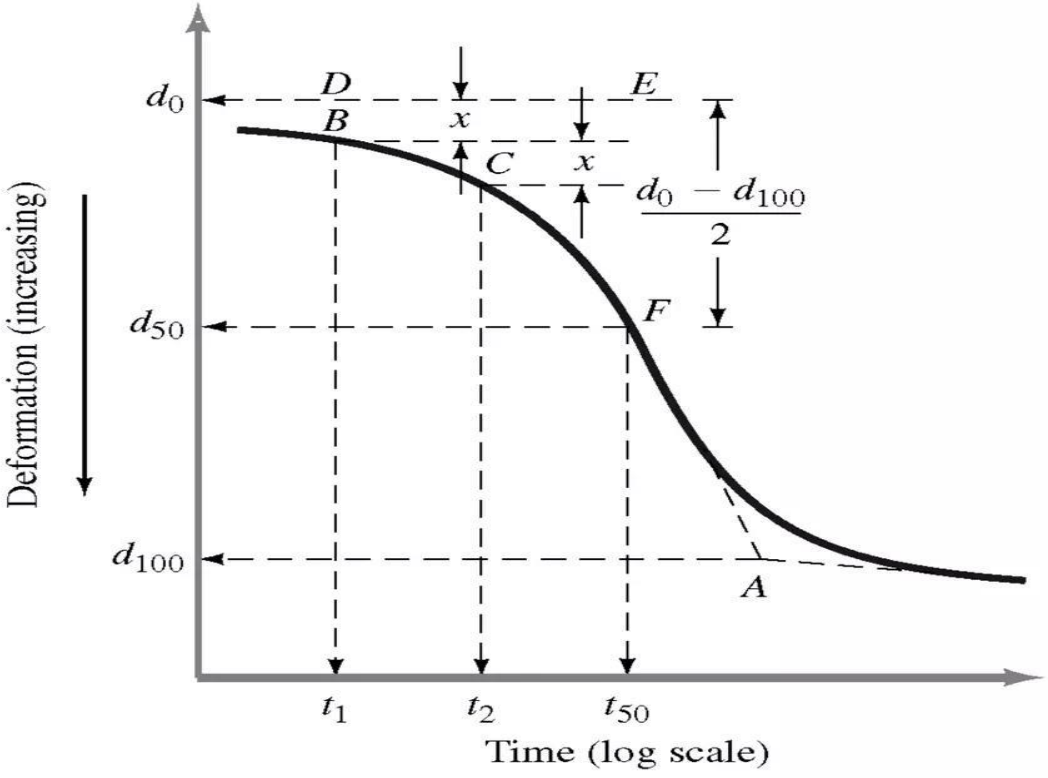


Figure 7.19. Das FGE (2006).

Logarithm of Time Method

1. Extend the straight line portion of primary and secondary consolidations to interest at Point A. Point A represents d_{100} (Deformation at 100% primary consolidation).
2. The initial curved portion of the deformation plot versus log t is approximated to be a parabola on a natural scale. Select times t_1 and t_2 on the curved portion such that $t_2 = 4t_1$. Let the difference of the specimen deformation between ($t_2 - t_1$) be equal to x.
3. Draw a line horizontal to *DE* such that the vertical distance *BD* is equal to x. The deformation corresponding to the line *DE* is d_0 (Deformation at 0% primary consolidation).

COEFFICIENT OF CONSOLIDATION (c_v)

Logarithm of Time Method

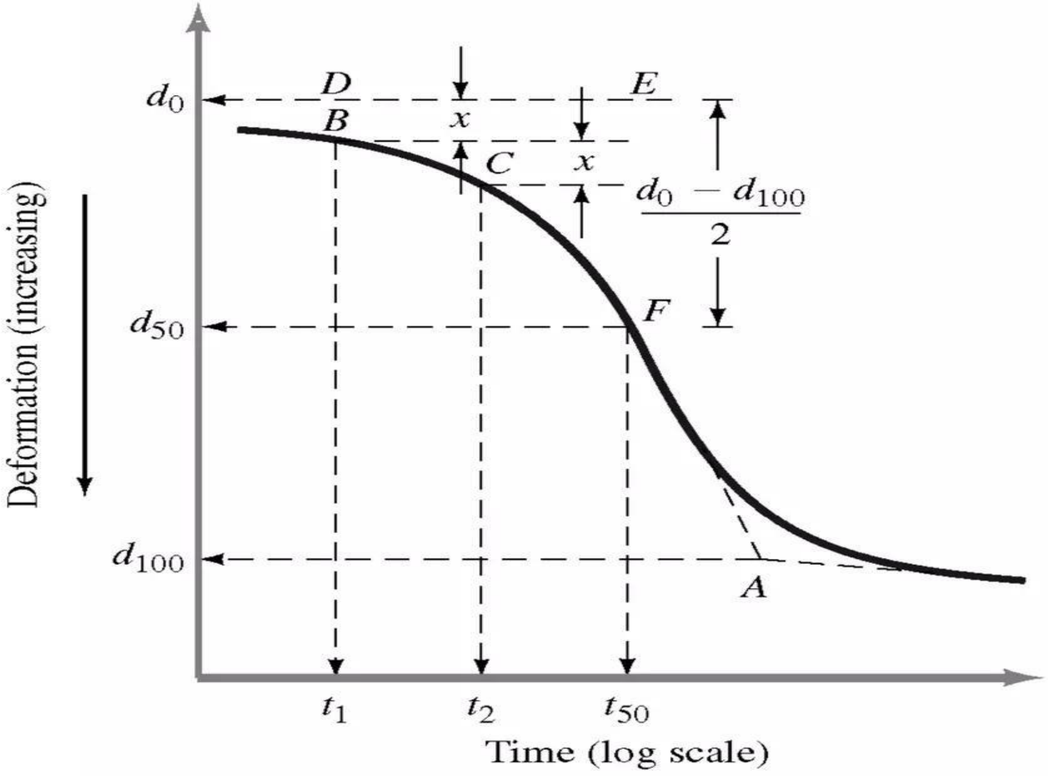


Figure 7.19. Das FGE (2006).

4. The ordinate of Point *F* on the consolidation curve represents the deformation at 50% primary consolidation (d_{50}).

5. For 50% average degree of consolidation (U = 50%), T_v = 0.197 (see Table 7.1, Das FGE 2006).

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

or

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

Where:

H_{dr} = Average longest drain path during consolidation.

COEFFICIENT OF CONSOLIDATION (c_v)

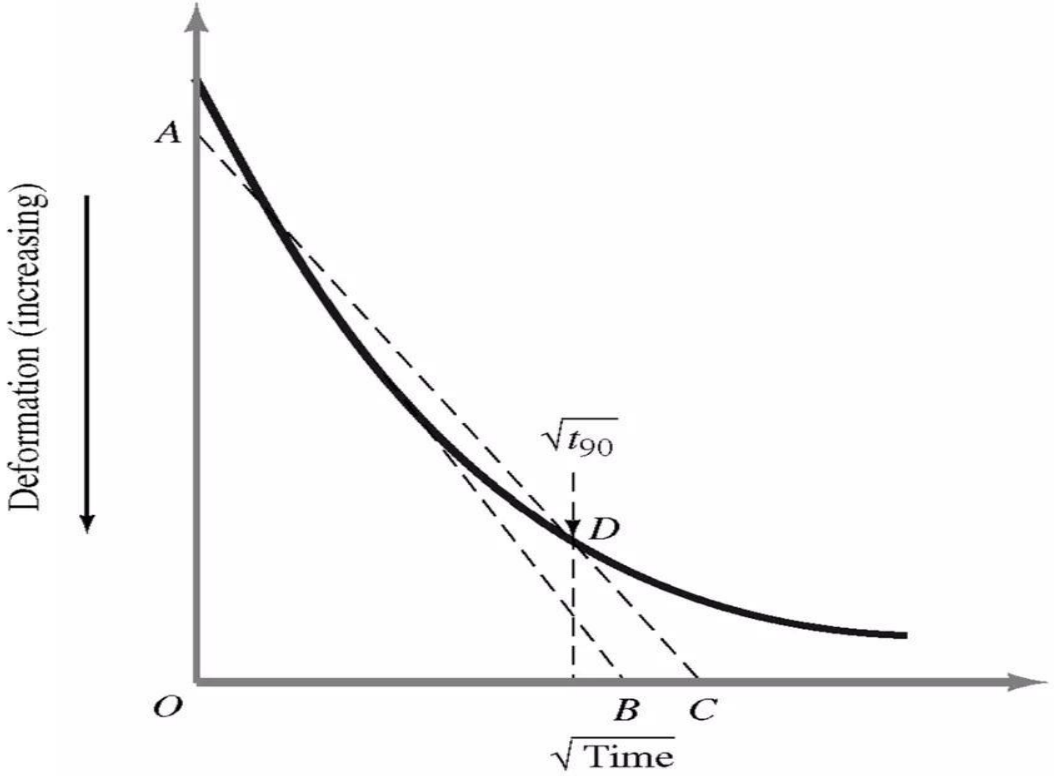


Figure 7.20. Das FGE (2006).

Square Root of Time Method

1. Draw a line *AB* trough the early portion of the curve.
2. Draw a line *AC* such that *OC* = 1.15*OB*. The time value for Point *D* (i.e. the intersection of line *AC* and the data) is the square root of time for t_{90} (i.e. the time to 90% primary consolidation).
3. For 90% consolidation, T_v = 0.848 (see Table 7.1, Das FGE 2006).

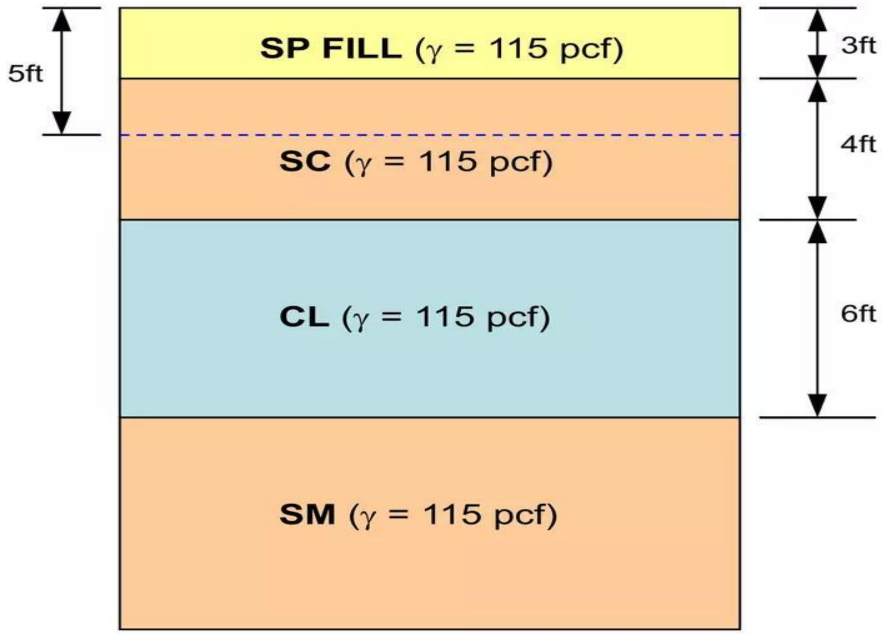
$$T_{90} = 0.848 = \frac{c_v t_{90}}{H_{dr}^2}$$

or

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

COEFFICIENT OF CONSOLIDATION (c_v)

Example



GIVEN: Soil Profile (NTS).
2 way drainage.

REQUIRED: Determine the following:

a. The change in pore pressure in the CL layer immediately after the application of the 3 ft of SP Fill.

b. The degree of consolidation in the middle of the clay layer when the excess pore pressure (u_e) is 170 psf.

c. How high would the water in a piezometer located in the middle of the layer rise above the GWT when u_e = 170 psf?

d. If c_v = 0.000004 ft²/sec, how long would it take to get to 25% average degree of consolidation? To U = 50%? To U = 99%?

PRECOMPRESSION – GENERAL CONSIDERATIONS

PRECOMPRESSION: Loading an area prior to placement of the planned structural loading to limit post-construction settlement. Also known as **Surcharging.**

Settlement caused by structural loading (S_p):

$$S_p = \frac{C_c H}{1+e_0} \log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_o}\right)$$

Settlement caused by structural loading and surcharging (S'_p or S_{p+f}):

$$S'_p = S_{p+f} = \frac{C_c H}{1+e_0} \log\left(\frac{\sigma'_o + \left[\Delta\sigma' + \Delta\sigma_f\right]}{\sigma'_o}\right)$$

Where:

$\Delta\sigma_f$ = Change in vertical stress due to Fill added.

PRECOMPRESSION – GENERAL CONSIDERATIONS

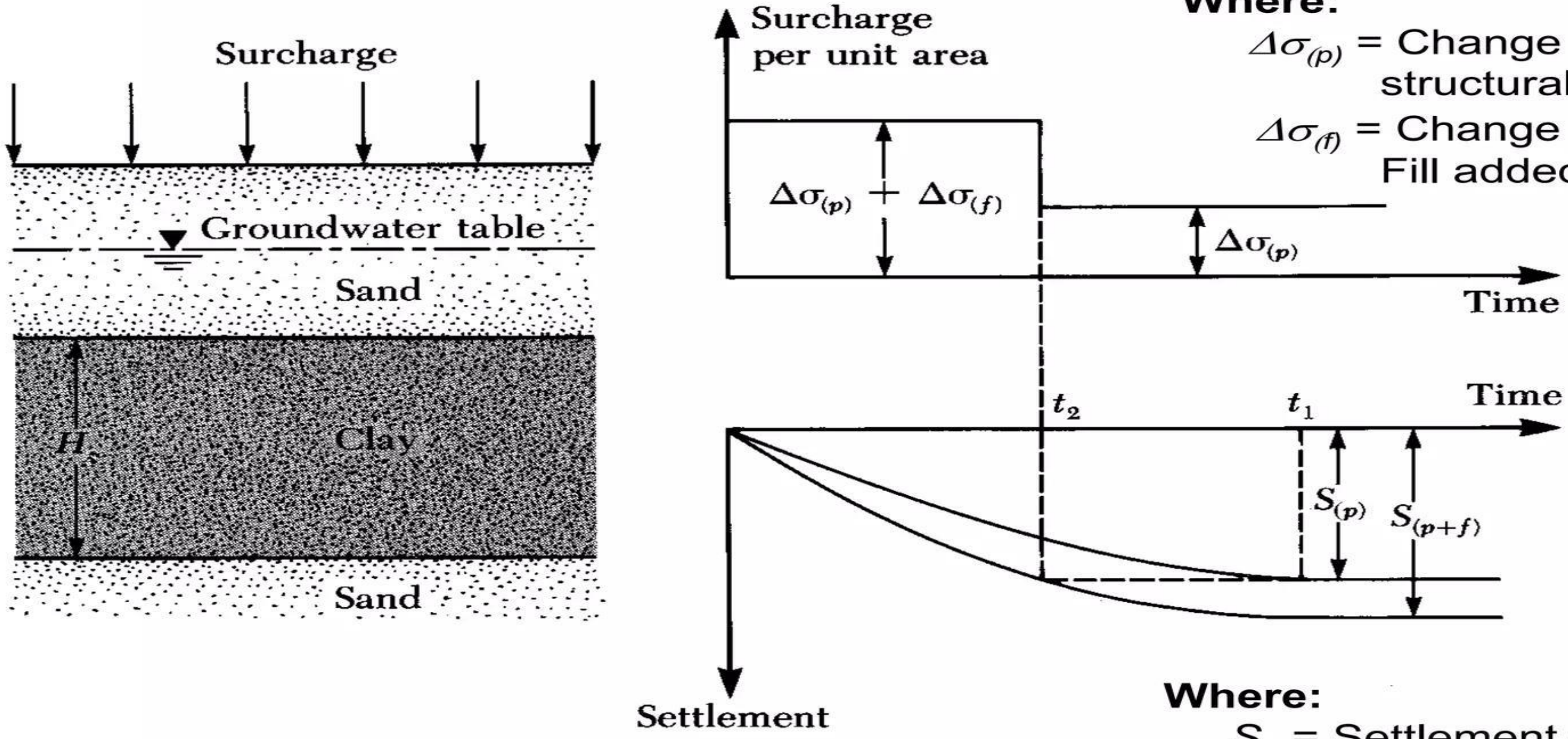


Figure 7.26. Das FGE (2006).

Where:

$\Delta\sigma_{(p)}$ = Change in vertical stress due to structural load.

$\Delta\sigma_{(f)}$ = Change in vertical stress due to Fill added.

Where:

S_p = Settlement due to structural load.

S_{p+f} = Settlement due to structural load and Fill.

PRECOMPRESSION – PLANNING

Mathematical Equations

$$U = \frac{S_p}{S'_p}$$

Definition of average Degree of Consolidation U

$$U = \frac{\log\left[\dfrac{\sigma'_o + \Delta\sigma'_{(p)}}{\sigma'_o}\right]}{\log\left[\dfrac{\sigma'_o + \Delta\sigma'_{(p)} + \Delta\sigma'_{(f)}}{\sigma'_o}\right]}$$

Substitution

$$U = \frac{\log\left[1 + \dfrac{\Delta\sigma'_{(p)}}{\sigma'_o}\right]}{\log\left\{1 + \dfrac{\Delta\sigma'_{(p)}}{\sigma'_o}\left[1 + \dfrac{\Delta\sigma'_{(f)}}{\Delta\sigma'_{(p)}}\right]\right\}}$$

Re-arranging (Eqn 7.56 Das FGE 2006)

Place in graphical form for design use (Figure 7.27 Das FGE 2006)

Precompression – Planning

Where:

- $\Delta\sigma_{(f)}$ = Change in vertical stress due to Fill added.
- $\Delta\sigma_{(p)}$ = Change in vertical stress due to Structural Loading.
- σ'_o = Initial vertical effective stress.

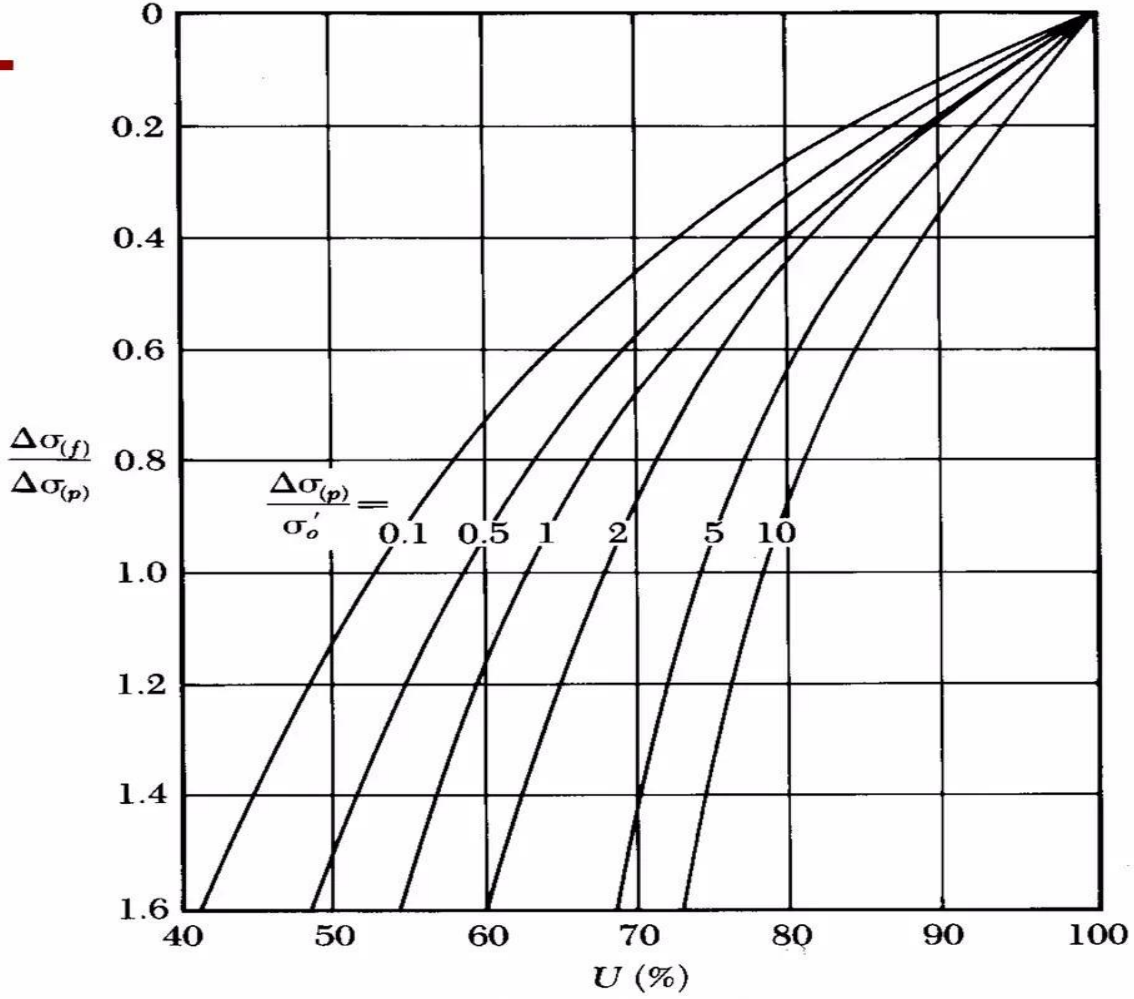


Figure 7.27. Das FGE (2006).

PRECOMPRESSION – PLANNING

STEPS:

1. Calculate primary consolidation settlement from planned loading (S_p).
2. Calculate primary consolidation settlement from planned loading plus surcharge (S_{p+f}).
3. Calculate average degree of consolidation U. Note $U = S_p/S_{p+f}$. Can also use Figure 7.27 or Eqn 7.56 (Das FGE 2006).
1. Find T_v from calculated U. To find time to when surcharge loading should be removed (i.e. t_2):

$$t_2 = \frac{T_v H_{dr}^2}{c_v}$$

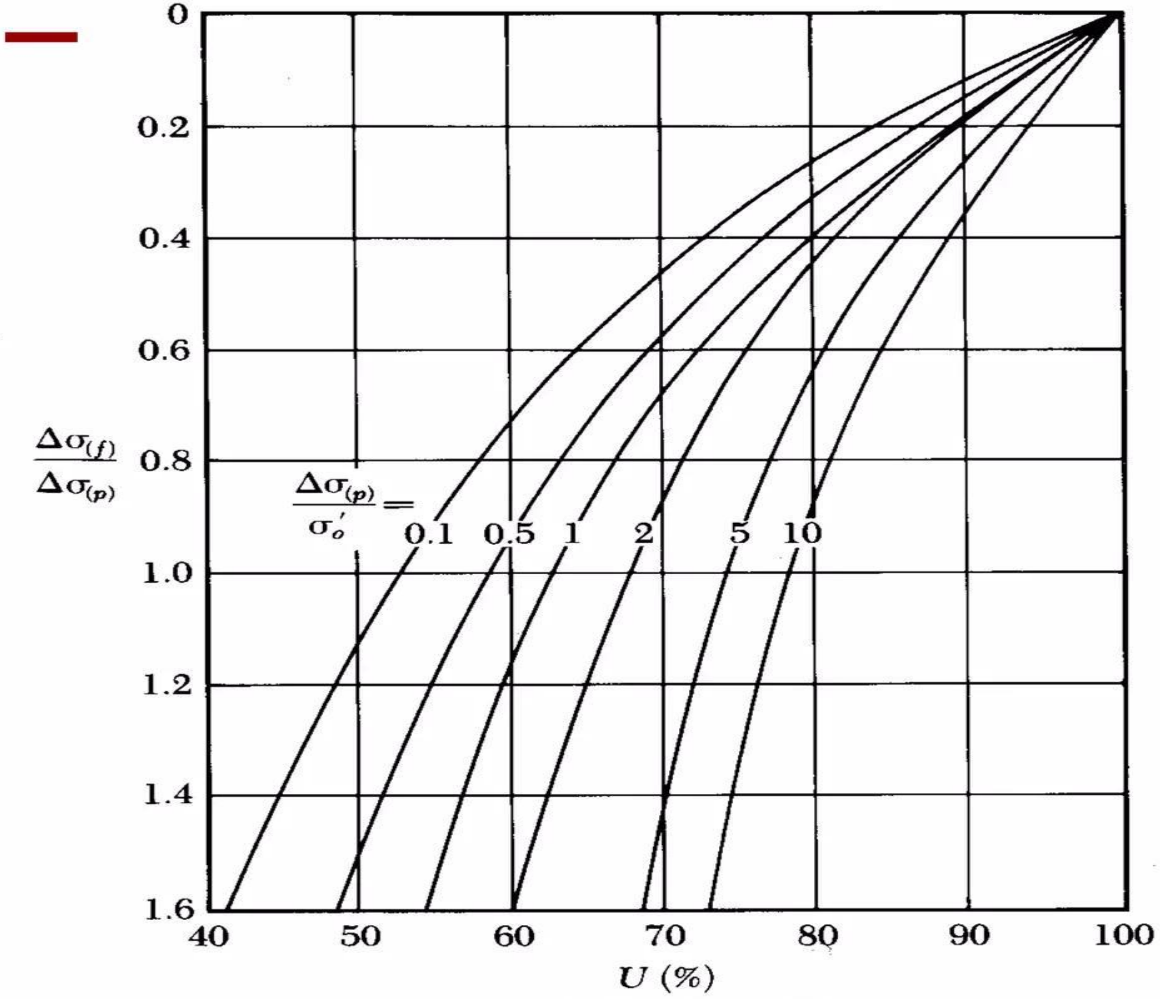


Figure 7.27. Das FGE (2006).

TIME RATE OF CONSOLIDATION

Difference between Average Degree of Consolidation and Midplane Degree of Consolidation

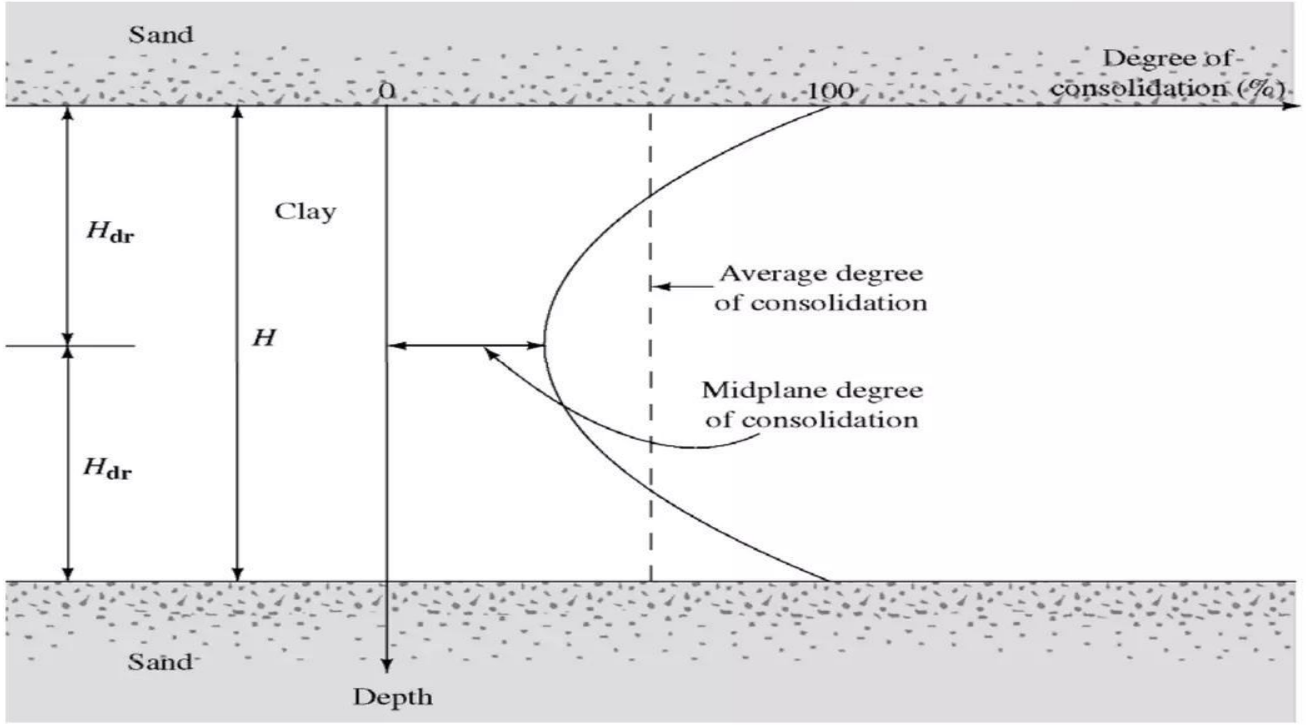


Figure 7.28. Das FGE (2006).

Removal of Surcharge may still cause net settlement
(swelling near drainage layers, settlement @ middle)

Conservative Approach:
Assume U is the midplane degree of consolidation.

TIME RATE OF CONSOLIDATION

Midplane Degree of Consolidation

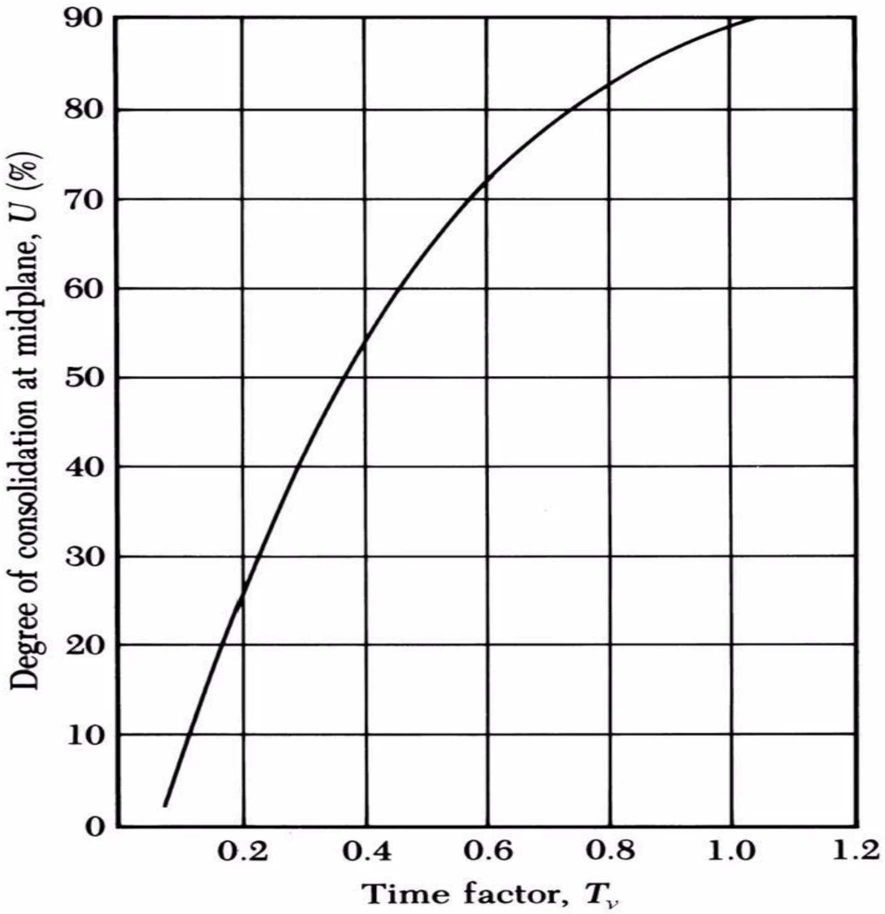
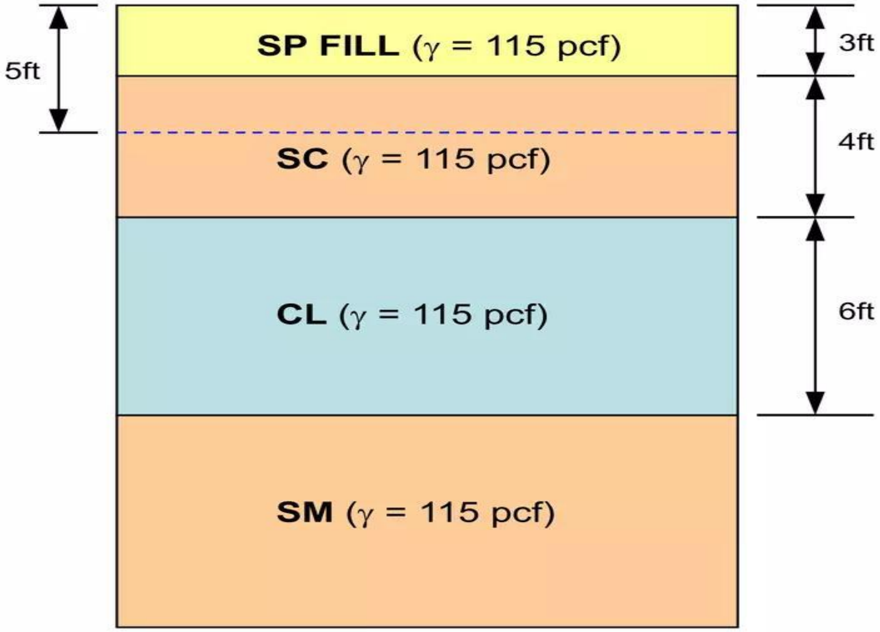


Figure 7.29. Das FGE (2006).

SURCHARGING EXAMPLE



GIVEN: Soil Profile (NTS).
2 way drainage.

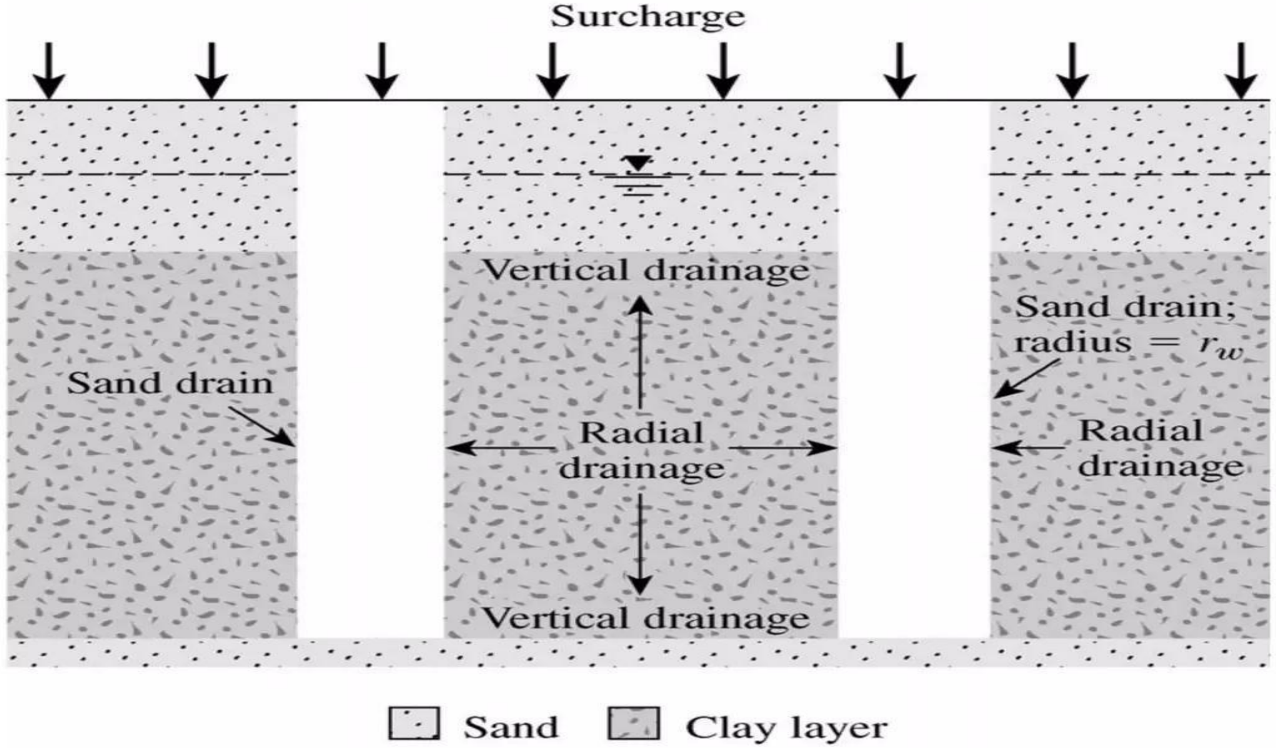
REQUIRED: Determine the following:

a. If c_v = 0.000004 ft²/sec, how long would it take to get to 99% average degree of consolidation?

b. If a surcharge of 4 ft of fill was placed in addition to the 3 ft of fill planned, when would you be able to remove the surcharge? Use the same value for c_v given in a.

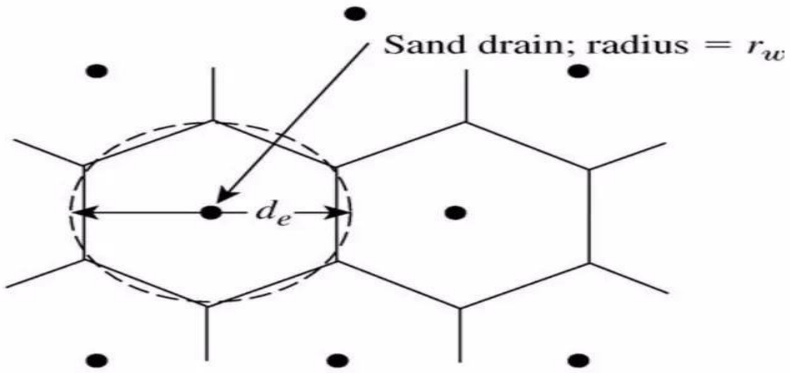
Ground Modification for Consolidation

Sand Drains



Section View
Figure 10.38. Das PGE (2006).

r_w = Sand Drain Radius
d_e = Effective Diameter



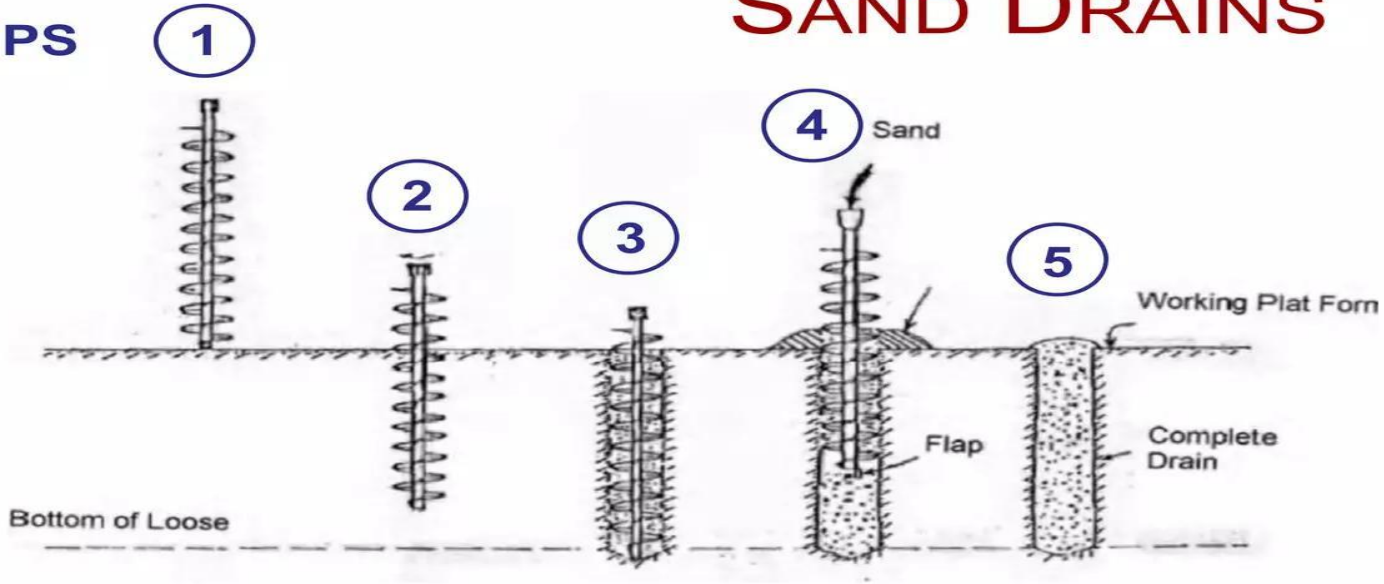
Plan View – Triangular Spacing
Figure 10.38. Das PGE (2006)

Reduction Drainage Path = Reduction in Drainage Time

Ground Modification for Consolidation

Sand Drains

STEPS



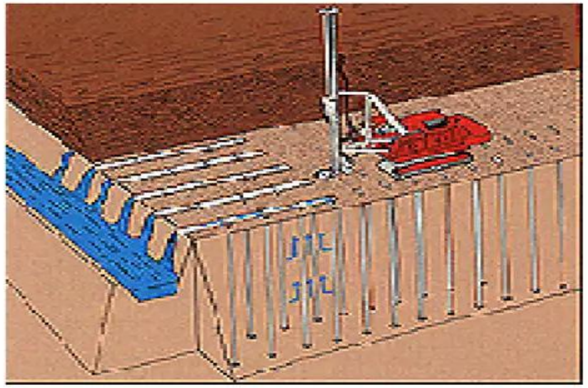
1. Place auger at drain location.
2. Screw auger to selected depth.
3. Rotate auger at selected depth to remove soil.
4. Inject sand while auger is extracted.
5. Complete sand drain to working platform level.

Sand Drain Installation: Auger Method
(Kirmani, 2004)

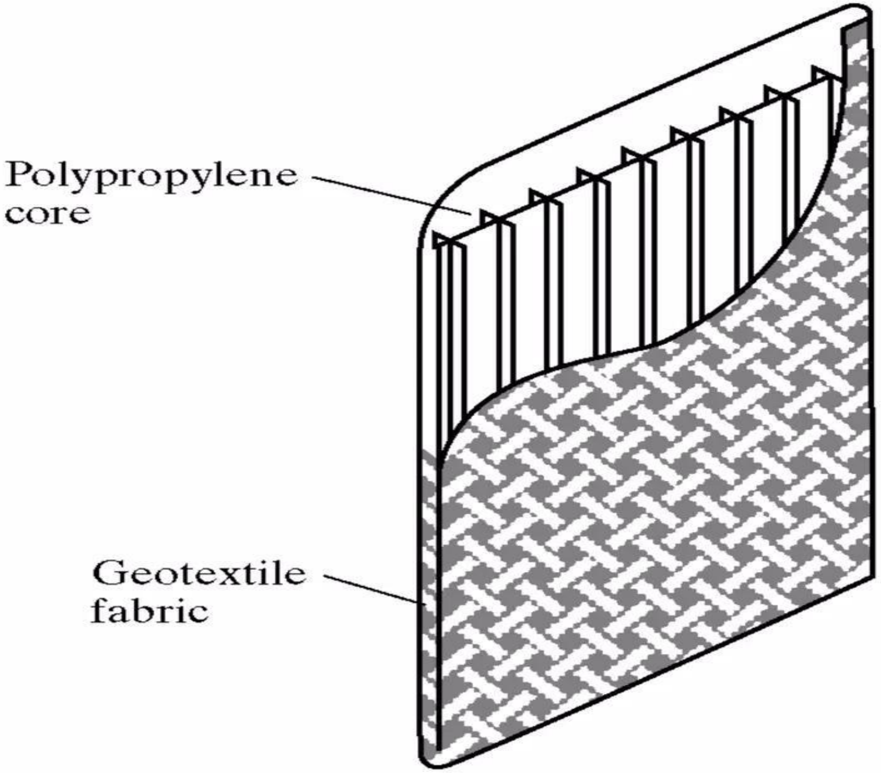
Figure 10.39. Das PGE (2006).

Ground Modification for Consolidation

Prefabricated Vertical Drains (PVD's) (a.k.a. Wick Drains)



Conceptual Concept
Courtesy of www.americanwick.com



Courtesy of
www.americandrainagesystems.com

Figure 7.31. Das FGE (2006).

Ground Modification for Consolidation

Prefabricated Vertical Drains (PVD's) (a.k.a. Wick Drains)

Courtesy of www.americandrainagesystems.com

Courtesy of www.nilex.com

Courtesy of www.nilex.com

Ground Modification for Consolidation

Radial Consolidation

U_r = Average Degree of Radial Consolidation

$$U_r = 1 - \exp\left(\frac{-8T_r}{m}\right)$$ **Barron (1948)**

$$m = \left(\frac{n^2}{n^2 - 1}\right)\ln(n) - \frac{3n^2 - 1}{4n^2}$$

$$n = \frac{d_e}{2r_w}$$

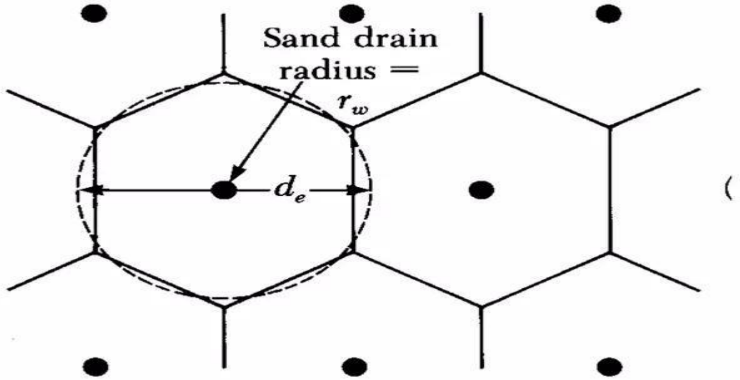
d_e = Effective Diameter
r_w = Sand Drain Radius

$$T_r = \frac{c_{vr} t}{d_e^2}$$

c_{vr} = Coefficient of Radial Consolidation
T_r = Time Factor for Radial Consolidation

$$c_{vr} = \frac{k_h}{\left[\frac{\Delta e}{\Delta\sigma'(1+e_o)}\right]\gamma_w}$$

k_h = Coefficient of Horizontal Permeability
T_r = Time Factor for Radial Consolidation
e_o = Initial Void Ratio



Plan View – Sand Drain Triangular Spacing
Figure 7.30. Das FGE (2006).

Time Rate of Radial Consolidation

Variation of T_r with U - Table 7.3 Das PGE (2006).

Degree of consolidation, U_r (%)	Time factor, T_r, for values of *n*				
	5	10	15	20	25
0	0	0	0	0	0
1	0.0012	0.0020	0.0025	0.0028	0.0031
2	0.0024	0.0040	0.0050	0.0057	0.0063
3	0.0036	0.0060	0.0075	0.0086	0.0094
4	0.0048	0.0081	0.0101	0.0115	0.0126
5	0.0060	0.0101	0.0126	0.0145	0.0159
6	0.0072	0.0122	0.0153	0.0174	0.0191
7	0.0085	0.0143	0.0179	0.0205	0.0225
8	0.0098	0.0165	0.0206	0.0235	0.0258
9	0.0110	0.0186	0.0232	0.0266	0.0292
10	0.0123	0.0208	0.0260	0.0297	0.0326
11	0.0136	0.0230	0.0287	0.0328	0.0360
12	0.0150	0.0252	0.0315	0.0360	0.0395
13	0.0163	0.0275	0.0343	0.0392	0.0431
14	0.0177	0.0298	0.0372	0.0425	0.0467
15	0.0190	0.0321	0.0401	0.0458	0.0503
16	0.0204	0.0344	0.0430	0.0491	0.0539
17	0.0218	0.0368	0.0459	0.0525	0.0576
18	0.0232	0.0392	0.0489	0.0559	0.0614
19	0.0247	0.0416	0.0519	0.0594	0.0652
20	0.0261	0.0440	0.0550	0.0629	0.0690
21	0.0276	0.0465	0.0581	0.0664	0.0729
22	0.0291	0.0490	0.0612	0.0700	0.0769
23	0.0306	0.0516	0.0644	0.0736	0.0808
24	0.0321	0.0541	0.0676	0.0773	0.0849
25	0.0337	0.0568	0.0709	0.0811	0.0890
26	0.0353	0.0594	0.0742	0.0848	0.0931
27	0.0368	0.0621	0.0776	0.0887	0.0973
28	0.0385	0.0648	0.0810	0.0926	0.1016
29	0.0401	0.0676	0.0844	0.0965	0.1059
30	0.0418	0.0704	0.0879	0.1005	0.1103
31	0.0434	0.0732	0.0914	0.1045	0.1148
32	0.0452	0.0761	0.0950	0.1087	0.1193
33	0.0469	0.0790	0.0987	0.1128	0.1239
34	0.0486	0.0820	0.1024	0.1171	0.1285
35	0.0504	0.0850	0.1062	0.1214	0.1332
36	0.0522	0.0881	0.1100	0.1257	0.1380
37	0.0541	0.0912	0.1139	0.1302	0.1429
38	0.0560	0.0943	0.1178	0.1347	0.1479
39	0.0579	0.0975	0.1218	0.1393	0.1529
40	0.0598	0.1008	0.1259	0.1439	0.1580
41	0.0618	0.1041	0.1300	0.1487	0.1632
42	0.0638	0.1075	0.1342	0.1535	0.1685
43	0.0658	0.1109	0.1385	0.1584	0.1739
44	0.0679	0.1144	0.1429	0.1634	0.1793
45	0.0700	0.1180	0.1473	0.1684	0.1849
46	0.0721	0.1216	0.1518	0.1736	0.1906
47	0.0743	0.1253	0.1564	0.1789	0.1964
48	0.0766	0.1290	0.1611	0.1842	0.2023
49	0.0788	0.1329	0.1659	0.1897	0.2083
50	0.0811	0.1368	0.1708	0.1953	0.2144
51	0.0835	0.1407	0.1758	0.2020	0.2206
52	0.0859	0.1448	0.1809	0.2068	0.2270
53	0.0884	0.1490	0.1860	0.2127	0.2335
54	0.0909	0.1532	0.1913	0.2188	0.2402
55	0.0935	0.1575	0.1968	0.2250	0.2470
56	0.0961	0.1620	0.2023	0.2313	0.2539
57	0.0988	0.1665	0.2080	0.2378	0.2610
58	0.1016	0.1712	0.2138	0.2444	0.2683
59	0.1044	0.1759	0.2197	0.2512	0.2758
60	0.1073	0.1808	0.2258	0.2582	0.2834
61	0.1102	0.1858	0.2320	0.2653	0.2912
62	0.1133	0.1909	0.2384	0.2726	0.2993
63	0.1164	0.1962	0.2450	0.2801	0.3075
64	0.1196	0.2016	0.2517	0.2878	0.3160
65	0.1229	0.2071	0.2587	0.2958	0.3247
66	0.1263	0.2128	0.2658	0.3039	0.3337
67	0.1298	0.2187	0.2732	0.3124	0.3429
68	0.1334	0.2248	0.2808	0.3210	0.3524
69	0.1371	0.2311	0.2886	0.3300	0.3623
70	0.1409	0.2375	0.2967	0.3392	0.3724
71	0.1449	0.2442	0.3050	0.3488	0.3829
72	0.1490	0.2512	0.3134	0.3586	0.3937
73	0.1533	0.2583	0.3226	0.3689	0.4050
74	0.1577	0.2658	0.3319	0.3795	0.4167
75	0.1623	0.2735	0.3416	0.3906	0.4288
76	0.1671	0.2816	0.3517	0.4021	0.4414
77	0.1720	0.2900	0.3621	0.4141	0.4546
78	0.1773	0.2988	0.3731	0.4266	0.4683
79	0.1827	0.3079	0.3846	0.4397	0.4827
80	0.1884	0.3175	0.3966	0.4534	0.4978
81	0.1944	0.3277	0.4090	0.4679	0.5137
82	0.2007	0.3383	0.4225	0.4831	0.5304
83	0.2074	0.3496	0.4366	0.4922	0.5481
84	0.2146	0.3616	0.4516	0.5163	0.5668

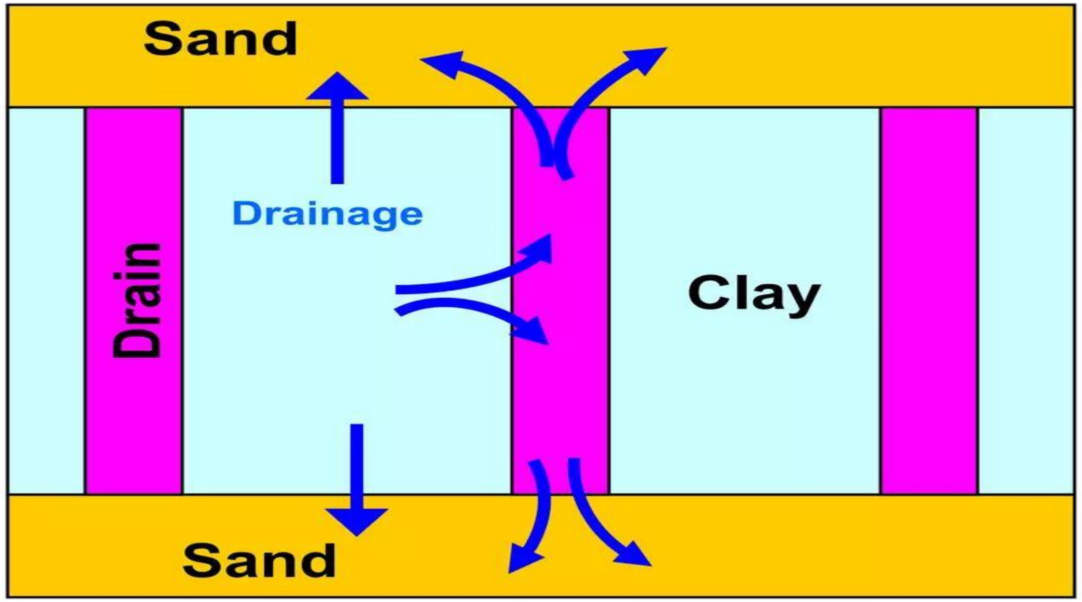
TIME RATE OF RADIAL CONSOLIDATION

Variation of T_r with U - Table 7.3 Das PGE (2006).

Degree of consolidation, U_r (%)	Time factor, T_r, for values of n				
	5	10	15	20	25
85	0.2221	0.3743	0.4675	0.5345	0.5868
86	0.2302	0.3879	0.4845	0.5539	0.6081
87	0.2388	0.4025	0.5027	0.5748	0.6311
88	0.2482	0.4183	0.5225	0.5974	0.6558
89	0.2584	0.4355	0.5439	0.6219	0.6827
90	0.2696	0.4543	0.5674	0.6487	0.7122
91	0.2819	0.4751	0.5933	0.6784	0.7448
92	0.2957	0.4983	0.6224	0.7116	0.7812
93	0.3113	0.5247	0.6553	0.7492	0.8225
94	0.3293	0.5551	0.6932	0.7927	0.8702
95	0.3507	0.5910	0.7382	0.8440	0.9266
96	0.3768	0.6351	0.7932	0.9069	0.9956
97	0.4105	0.6918	0.8640	0.9879	1.0846
98	0.4580	0.7718	0.9640	1.1022	1.2100
99	0.5391	0.9086	1.1347	1.2974	1.4244

Ground Modification for Consolidation

Average Degree of Consolidation due to Vertical & Radial Drainage



Vertical and Radial Drainage
Courtesy of www.nhi.fhwa.dot.gov

$$U_{v,r} = 1 - (1 - U_r)(1 - U_v)$$

Where:

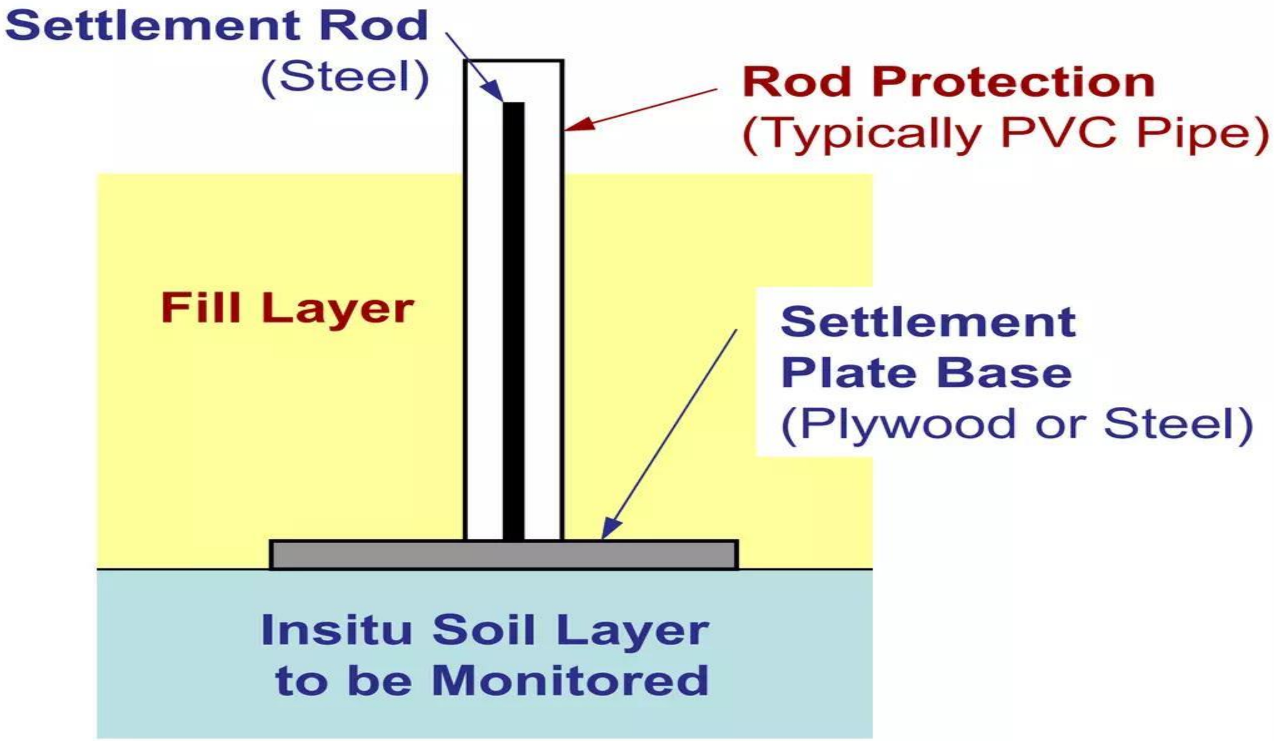
$U_{v,r}$ = Average Degree of Consolidation due to Vertical & Radial Drainage

U_v = Average Degree of Consolidation due to Vertical Drainage

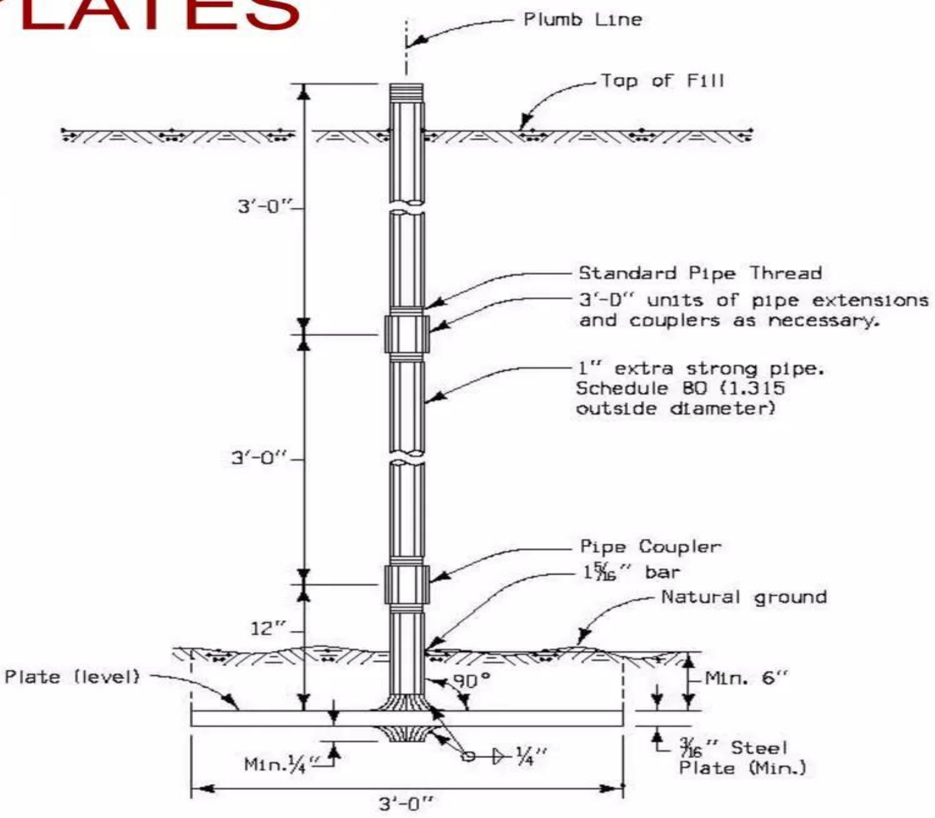
U_r = Average Degree of Consolidation due to Radial Drainage

Consolidation Monitoring

Settlement Plates

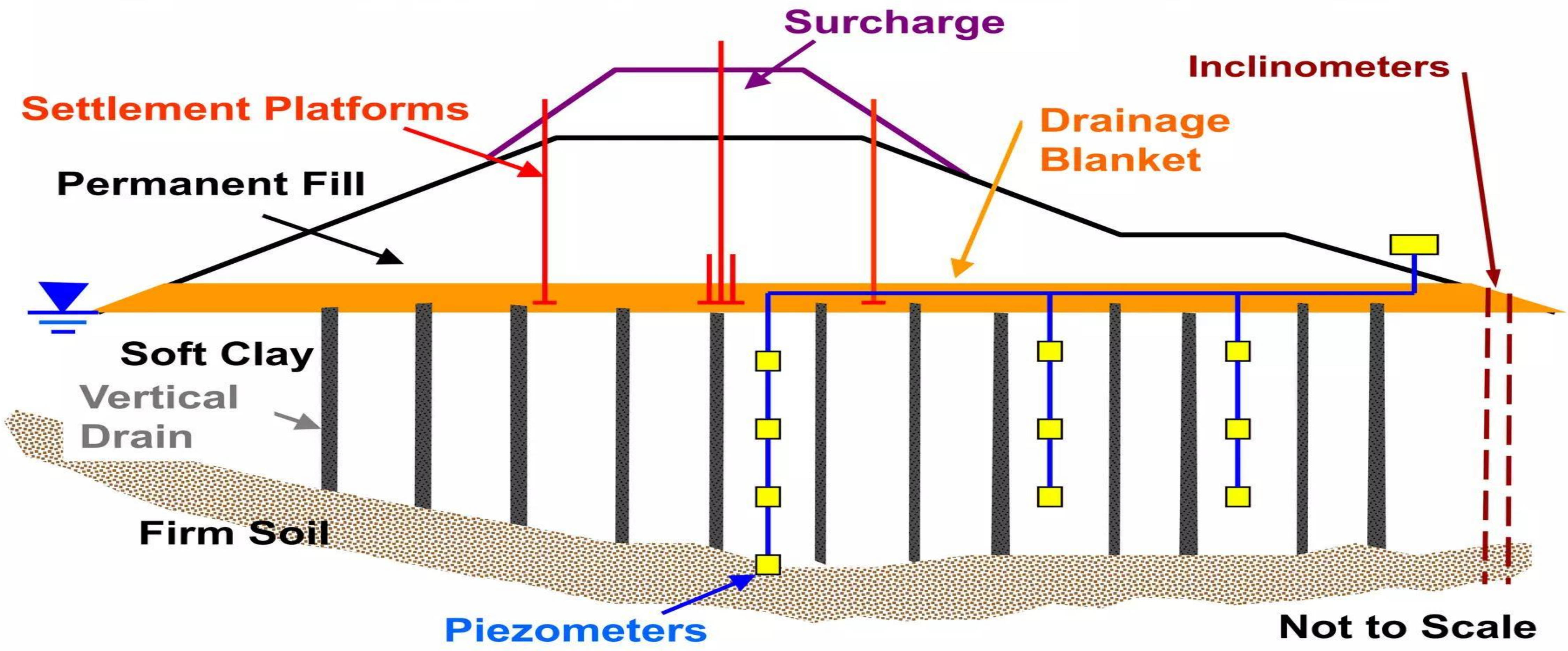


General Concept



Standard Plan Detail
(Courtesy of Iowa DOT)

SURCHARGING INSTRUMENTATION EXAMPLE



Courtesy of www.nhi.fhwa.dot.gov

Soil Consolidation

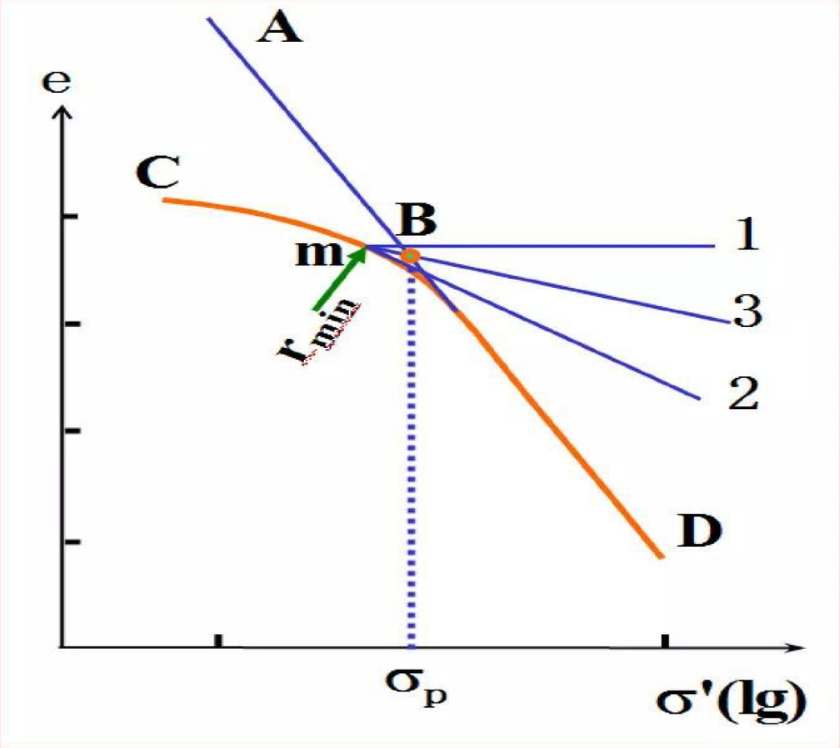
Week 4

Pages 102-104

PRECONSOLIDATION PRESSURE

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

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



How to obtain the preconsolidation pressure:

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

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

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

4 The vertical through intersection point of the bisector and CB produced gives the approximate value of the preconsolidation pressure.

Determination of Maximum Past Pressure (σ'_c or σ'_{vm})

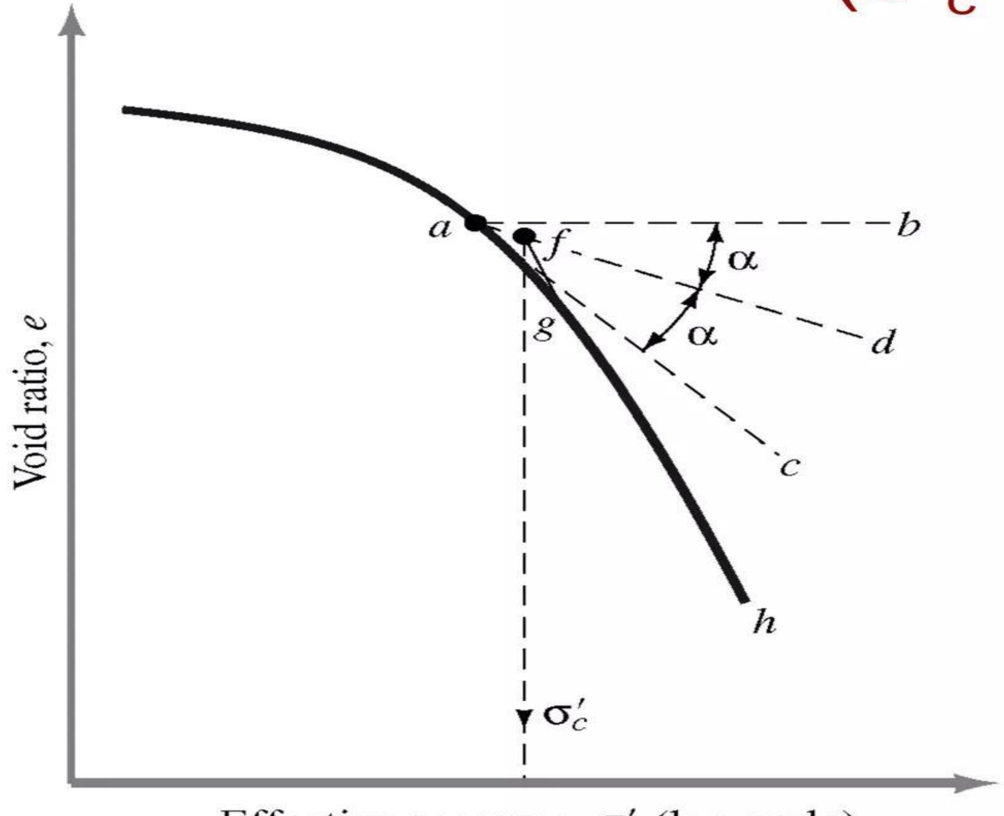


Figure 7.8. Das FGE (2005).

Graphical Method (Casagrande, 1936)

1. Visually identify point of minimum radius of curvature on *e*-log σ' curve (i.e. Point *a*).
2. Draw horizontal line from Point *a* (i.e. Line *ab*).
3. Draw Line *ac* tangent to Point *a*.
4. Draw Line ad bisecting Angle *bac*.
5. Project the straight line portion of *gh* on *e*-log σ' curve to intersect Line *ad*. This intersection (Point *f*) is the maximum past pressure (a.k.a. preconsolidation pressure).

OVERCONSOLIDAITON RATIO (OCR)

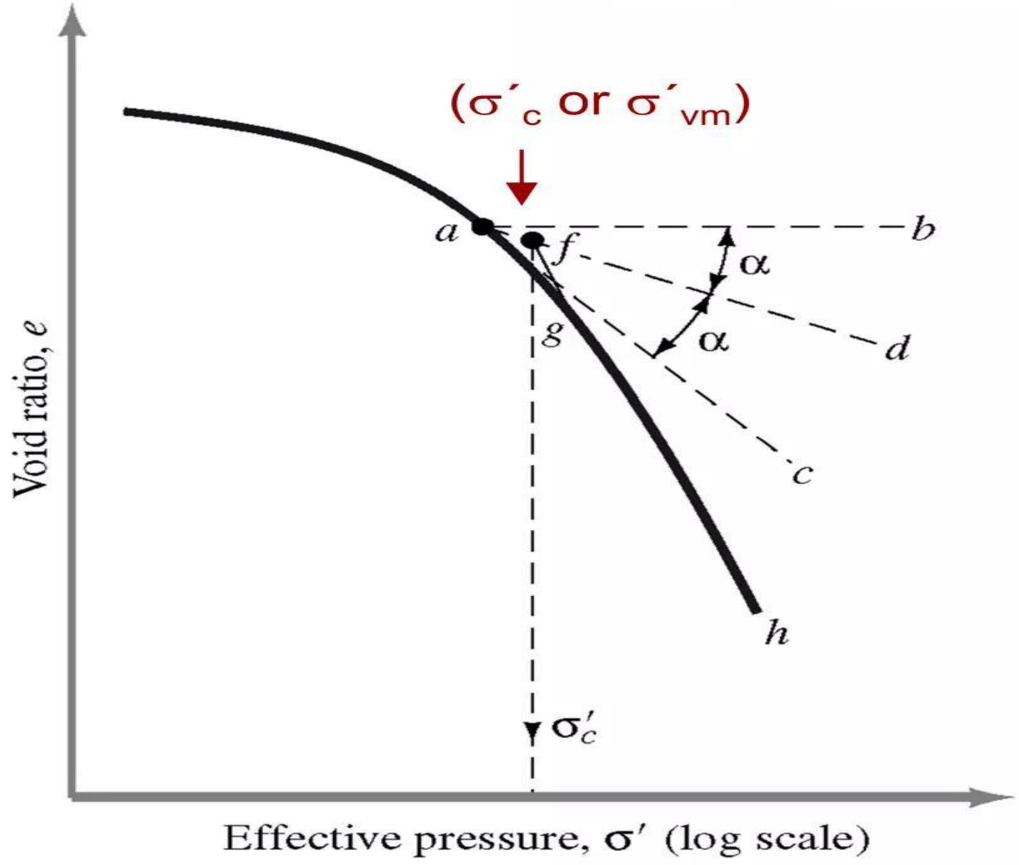


Figure 7.8. Das FGE (2005).

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

Where:

σ'_c (a.k.a. σ'_{vm}) = Preconsolidation Pressure (a.k.a Maximum Past Pressure).

σ' = Present Effective Vertical Stress

General Guidelines:
NC Soils: $1 \leq OCR \leq 2$
OC Soils : $OCR > 2$

Possible Causes of OC Soils:
Preloading (thick sediments, glacial ice); fluctuations of GWT, underdraining, light ice/snow loads, desiccation above GWT, secondary compression.

Click The Link

For Settlement Analysis Math, Please see chapter 10

Standard Penetration Test

Week 5

Pages 107-134

INTRODUCTION

- The **Standard Penetration test** (**SPT**) is a common in situ **testing** method used to determine the geotechnical engineering properties of subsurface soils. It is a simple and inexpensive **test** to estimate the relative density of soils and approximate shear strength parameters.

USES SPT TEST

These can be used for identification test like specific gravity, grain size distribution, Atterberg limit, compaction etc.

USEFUL IN FINDING OUT

- Relative density of cohesion less soils.
- Angle of shearing resistance of cohesion less soils.
- Unconfined compressive strength of cohesive soils.

INSTRUMENTS

1) DRILLING EQUIPMENT FOR BOREHOLES:

- Any drilling equipment is acceptable that provides a reasonably clean hole, which is at least 5 mm larger than the sampler or sampling rods, and less than 170 mm diameter.

INSTRUMENTS

2)SPLIT – SPOON SAMPLER

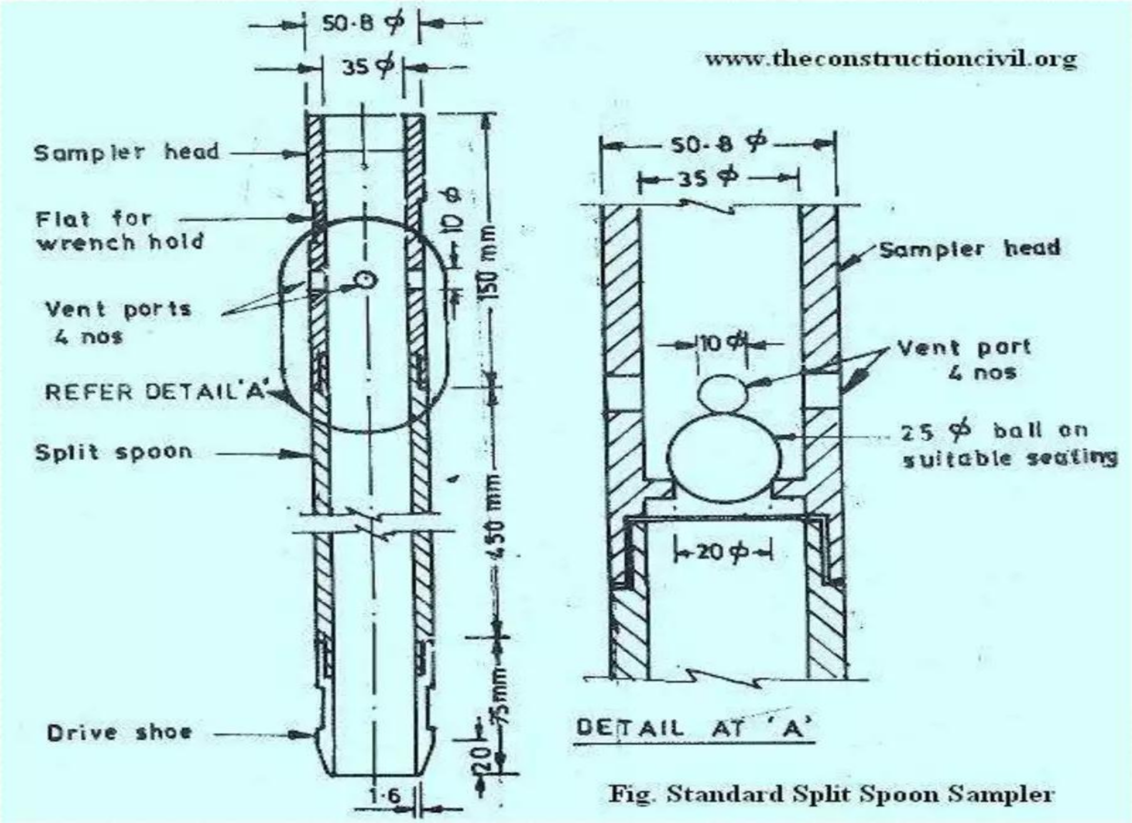


Fig. Standard Split Spoon Sampler

- It is a sampler for obtaining a disturbed sample of soil and consists of-
- **Driving shoe**: made of tool-steel about 75 mm long
- **Steel tube**: 450 mm long, split longitudinally in two halves
- **Coupling**: 150 mm long, provided at the top
- Check valve
- **4 venting ports**: 10 mm diameter.

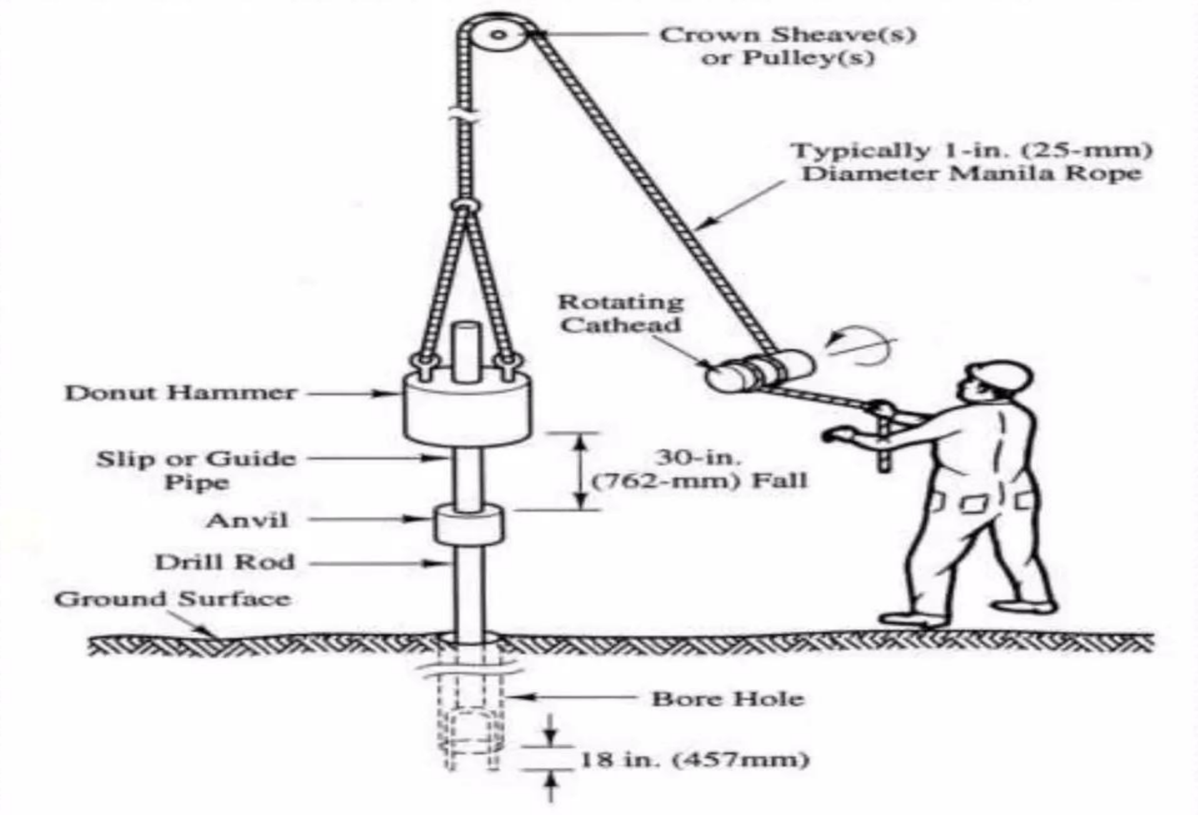
INSTRUMENTS

3) DRIVE – WEIGHT ASSEMBLY

- Hammer of 63.5 kg
- A driving head(anvil)
- A guide permitting a free fall of 0.76 m and over lift capability of at least 100 mm.

INSTRUMENTS

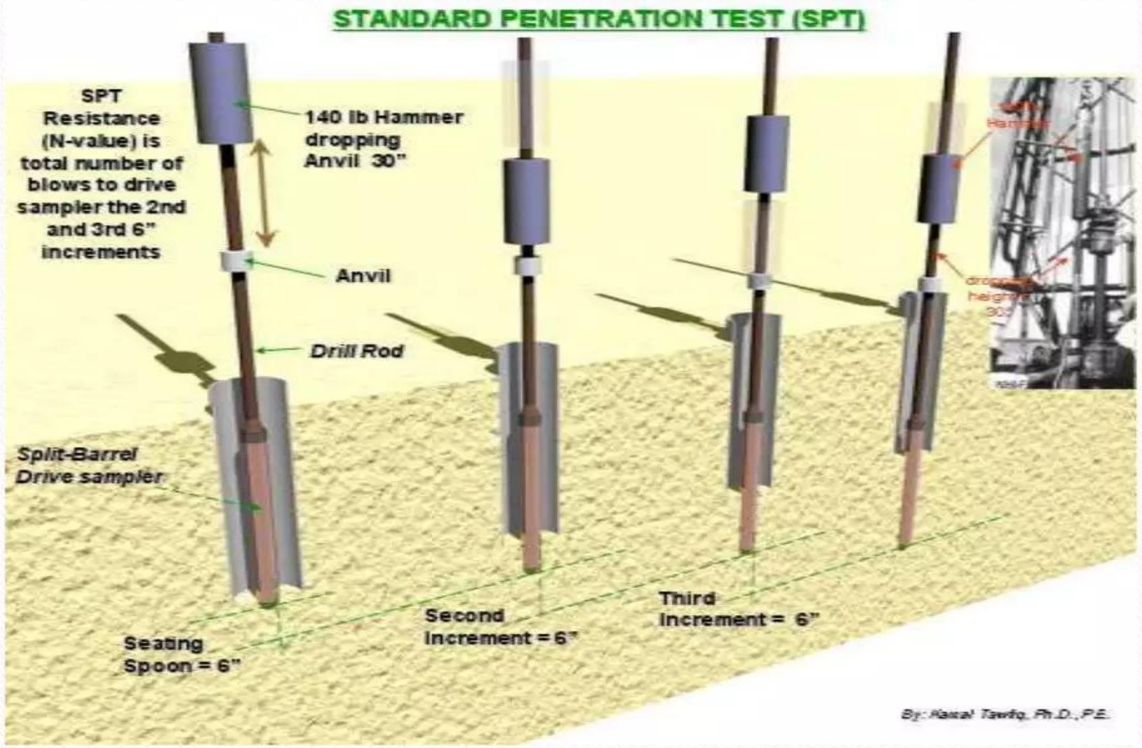
4) CATHEAD



- Operating at approximately 100 rpm.
- Equipped with suitable rope and overhead sheave for lifting drive-weight.

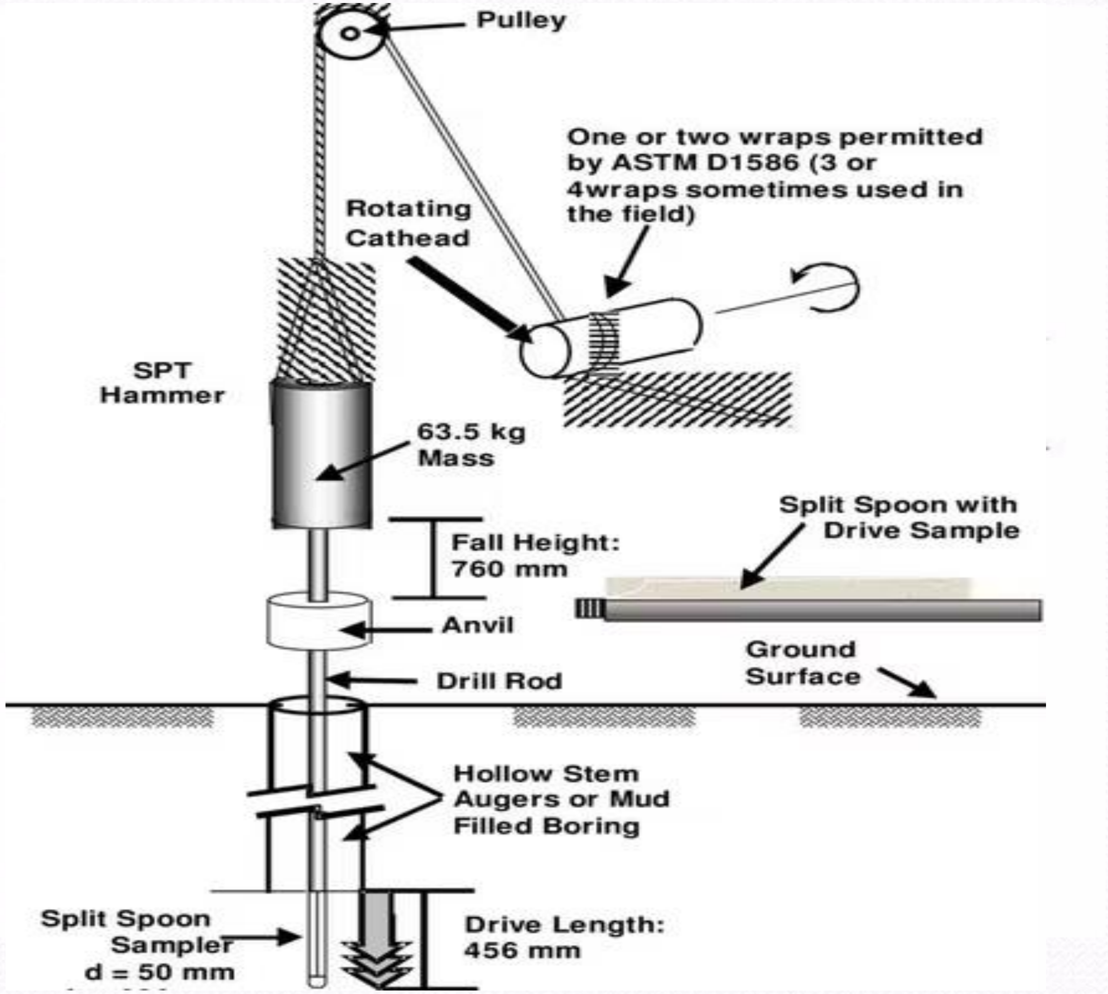
INSTRUMENTS

5)HAMMER



- **A)SEFETY HAMMER**
- Closed system
- Delivers approximately 60% of the maximum free fall energy
- Highly variable energy transfer

INSTRUMENTS

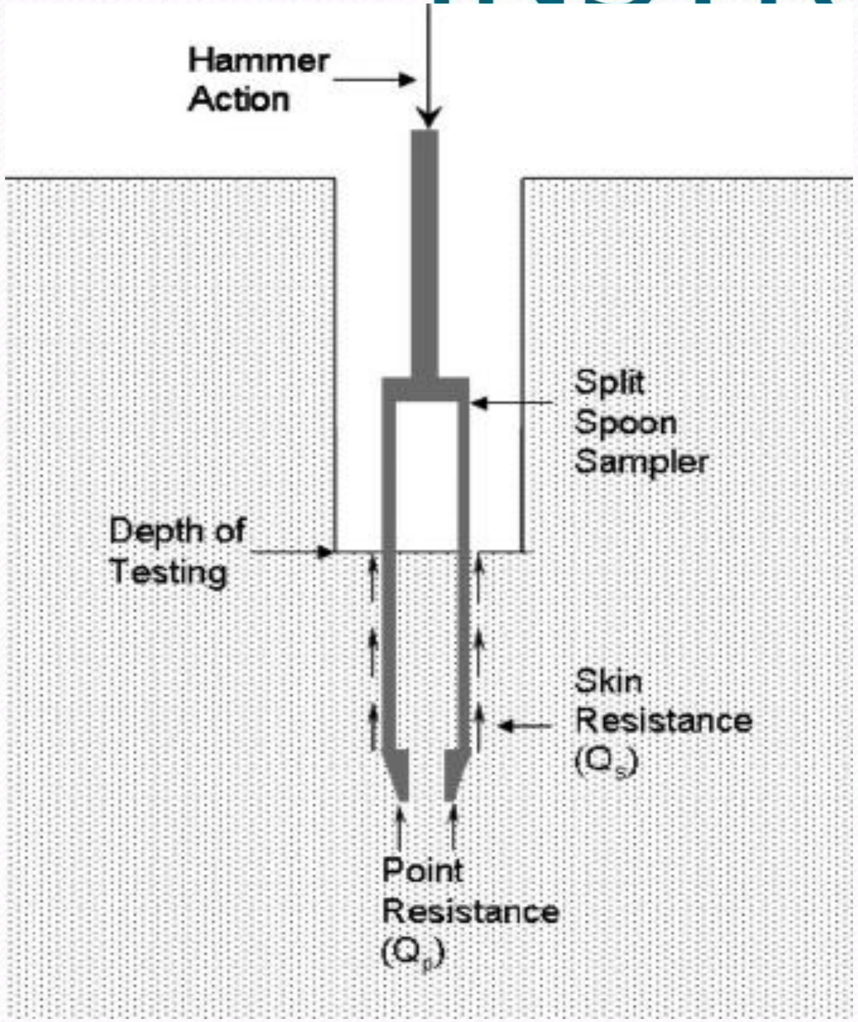


- **B) DONUT HAMMER**
- Open system
- Delivers approximately

45% of the maximum free fall energy

Highly variable energy transfer.

INSTRUMENTS



- **C) AUTOMATIC HAMMER**
- Safest system
- Delivers approximately 95-100% of the maximum free fall energy
- Consistent and effective energy transfer
- Increased production.

PROCEDURE

1)DRILLING OF BOREHOLE

- Drill the borehole to the desired sampling depth and clean out all disturbed material.
- The equipment used shall provide a clean borehole, 100 to 150 mm in diameter, for insertion of the sampler to ensure that the penetration test is performed on undisturbed soil.
- Casing shall be used when drilling in sand, soft clay or other soils in which the sides of borehole are likely to cave in.

PROCEDURE

2) DRIVING THE CASING

- Where casing is used, it shall not be driven below the level at which the test is made or soil sample is taken.
- In the case of cohesion less soils which cannot stand without casing, the advancement of the casing pipe should be such that it does not disturb the soil to be tested or sampled; the casing shall preferably be advanced by slowly turning the casing rather then by driving may alter the density of such deposits immediately below the bottom of the borehole.

PROCEDURE

3) ASSEMBLING EQUIPMENT

- Attach the split-spoon sampler to the drill rod and lower into the hole until it is sitting on the undisturbed material.
- Attach the drive weight assembly.
- Lift the 63.5 kg hammer approximately 0.76 m and allow it to fall on the anvil delivering one seating blow.
- Mark the drill rod in 3 successive .15 m increments to observe penetration.

PROCEDURE

4) PENETRATION TESTING

- Raise and drop the hammer 0.76 m successively by means of the rope and cathead, using no more than two and one forth wraps around the cathead.
- The hammer should be operated between 40 and 60 blows per minute and should drop freely.
- Record the number of blows for each .15 m of the penetration.
- The first .15 m increment is the "seating" drive.

PROCEDURE

- The sum of the blows for second and third increment of 0.15 m penetration is termed “penetration resistance or N-value”.
- If the split spoon sampler is driven less then 45 cm(total), then the penetration resistance shall be for the last 30 cm of penetration (if less than 30 cm is penetrated, the logs should state the number of blows and the depth penetrated).
- If the no. of blows for 15 cm drive exceeds 50, it is taken as a refusal and the test is discontinued.

PROCEDURE

- Tests shall be made at every change in stratum or at intervals of not more than 1-5 m whichever is less. Tests may be made at lesser intervals if specified or considered necessary.
- The intervals be increased to 3 m if in between vane shear test is performed.
- The entire sampler may sometimes sink under its own weight when very soft sub-soil stratum is encountered.
- Under such conditions, it may not be necessary to give any blow to the split spoon sampler and SPT value should be indicated as zero.

PROCEDURE

4)HANDLING SAMPLE

- Bring the sampler to the surface and open it. Remove any obvious contamination from the ends or sides and drain excess water. Carefully scrape or slice along one side to expose fresh material and any stratification.
- Record the length, composition, color, stratification and condition of sample.
- Remove sample and wrap it or seal in a plastic bag to retain moisture. If the sample can be removed relatively intact, wrap it in several layers of plastic and seal ends with tape.

CORRECTION

- No correction for cohesive soils.
- 1)correction for overburden pressure
- 2)correction for dilatancy

CORRECTOIN FOR OVERBURDEN PRESSURE

- Because of confining pressure, the N values at shallow depths are under-estimated and those at larger depths are over estimated.
- Correction SPT, $N^{-1} = C_N N$
- CN=correction due to overburden.

$$C_N = 0.77 \log \frac{2000}{\partial_0^{'}}$$

- Valid for
- Effective overburden stress in $\partial_0^{'} \geq 25 KN/m^2$

CORRECTION FOR DILATANCY

- Dilatancy correction should be applied when N' obtained after applying overburden pressure correction exceeds 15 in saturated fine sands and silts.
- $$N'' = 15 + 1/2(N' - 15)(when N' > 15)$$
  $$N'' = N' (when N' < 15)$$
- N'' is the final corrected SPT value to be used in design, N' is the SPT value after applying overburden pressure correction.

- N'>15 is an indication of dense sand, in such soil, blows of drop hammer will cause increase in shear resistance (due to negative express pore water pressure). This results in an SPT value higher than the actual one.
- In addition, correction for hammer energy or hammer efficiency may be applied as per requirement. however IS:2131(1981) is silent on this issue.

Correction factor	Equipment variable	correction
Overburden pressure(CN)		For $\sqrt{\frac{100}{\partial'_o}} \leq 1.7$ Cohesion Less soil
Energy ratio(CE)	Donut hammer	0.5-1.0
	Safety hammer	0.7-1.2 CE=E%/60
	Automatic trip Donut hammer type	0.8-1.3
Borehole diameter	65mm-115mm	1
	150mm	1.05
	200mm	1.15
Rod length	<3m	0.75
	3m-4m	0.8
	4m-6m	0.85
	6m-10m	0.95
	10m-30m	1.0
sampler	Standard sampler	1.0
	Sampler without liner	1.1-1.3

- Corrected SPT value

$$N_{1,60} = N * C_N * C_E * C_B * C_R * C_S$$

- SPT below corrected against 1 Atm and 60% hammer efficiency

N

EMPERICAL CORELATIONS WITH SPT VALUE

- SPT is not considered to be safe very precise and reliable method of soil investigation. Despite of this the N value gives useful information regarding the compaction of cohesionless soil and consistency of cohesive soil.

CORRECTION OF N VALUE WITH PROPERTIES OF GRANULAR SOIL (RELATIVE DENSITY)

SPT blow count (N)	compactness	Relative Density	Angle of internal fraction
0-4	Very loose	0-15%	Less than 28 degree
4-10	Loose	15-35%	28-35 degree
10-30	medium	35-65%	30-36 drgree
30-50	dense	65-85%	36-41 degree
Greater than 50	very	Greater than 85%	Greater than 41 degree

$$\phi = 0.36N + 27$$

N value and properties of saturated cohesive soil

consistency	Clay type	SPT below count(N)	UCS(KN/m2)	remarks
Very soft		0.2	Less than 25	Squishes between fingers when squeezed
soft	NC clay	3-5	25-50	Very soft clay deformed by squeezing
Medium		6.9	50-100	Can be deformed dry squeezing with some effort

consistency	Clay type	SPT below count(N)	UCS(KN/M2)	REMARKS
stiff		10-16	100-200	Hard to deform by squeezing
Very stiff	OC clay	17-30	200-400	Very hard to deform by squeezing
hand		More than 30	More than 400	Nearly impossible to deform by squeezing

USEFULNESS AND LIMITATION OF SPT

advantage

- 1)SPT is relatively quick, to perform and inexpensive
- Able to penetrate dense layers, gravel etc.
- Enables to collect representative samples.
- Highly useful to get qualitative soil properties from Imperial correlations.
- Persons having experience in SPT are easily available.

disadvantage

- Representative samples collected in SPT can not be used in shear strength, consolidation and permeability test.
- Unlike CPT, the soil profile cannot be detected continuously
- The results are not very precise and highly reliable.
- Results are susceptible to errors if there is any wear and tear of the cutting shoe, improper height of fall improper alignment etc.

Shear Strength Parameter (Direct Shear Test)

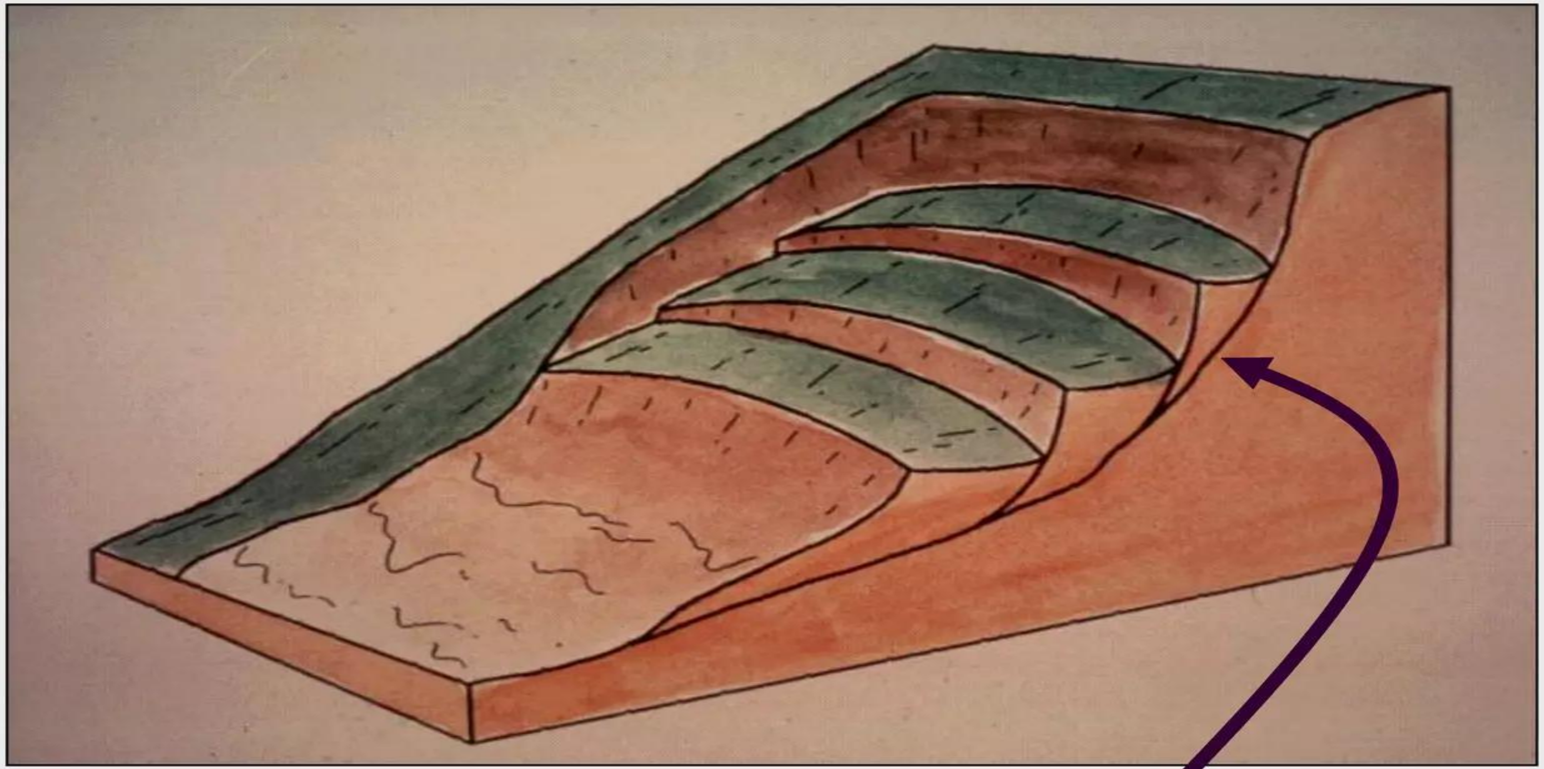
Week 6

Pages 136-164

Shear Strength

- The strength of a material is the greatest stress it can sustain
- The safety of any geotechnical structure is dependent on the strength of the soil
- If the soil fails, the structure founded on it can collapse

Slope Failure in Soils



Failure due to inadequate strength at shear interface

Civi

Shear Failure in Soils

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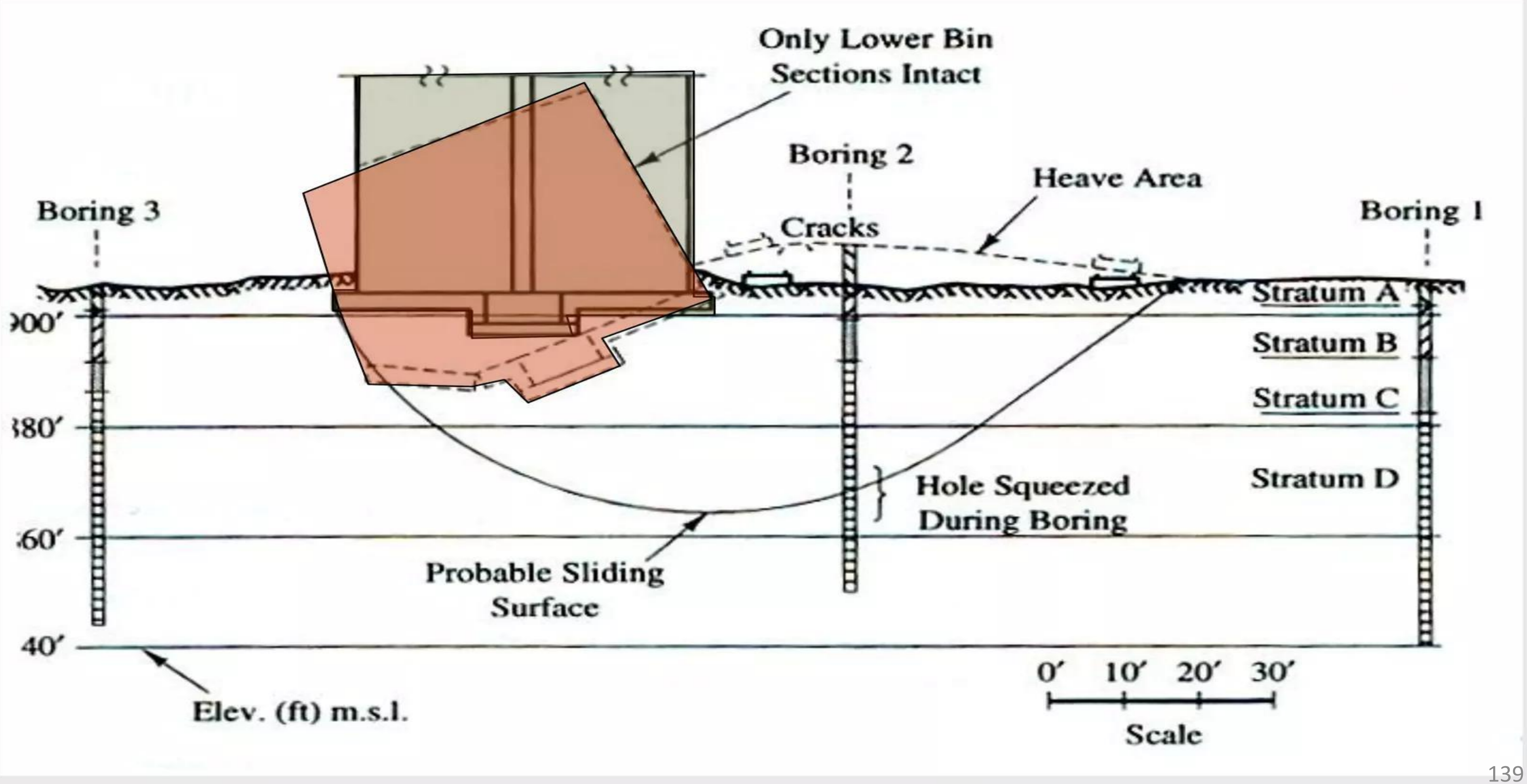
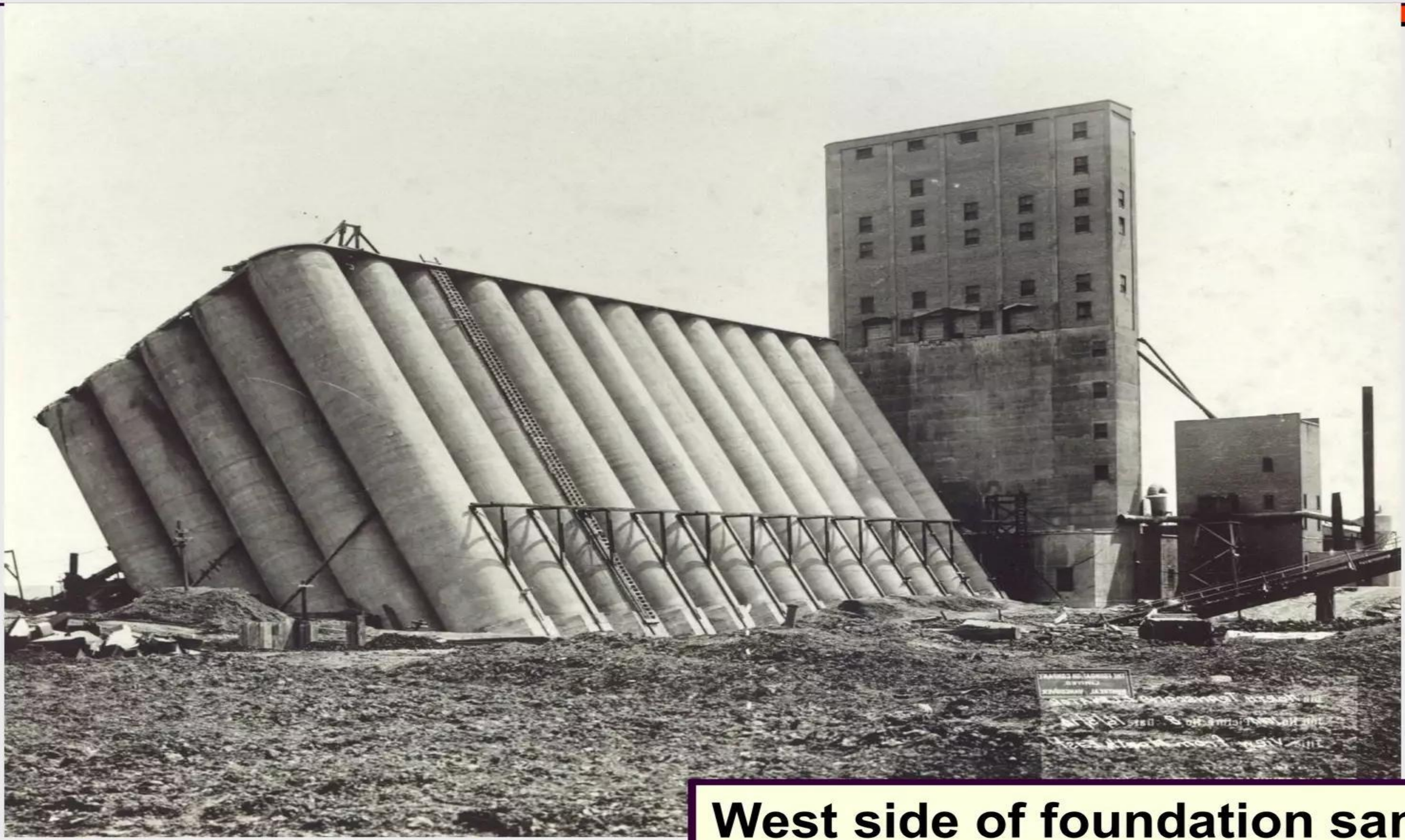
Bearing Capacity Failure



Transcosna Grain Elevator Canada (Oct. 18, 1913)



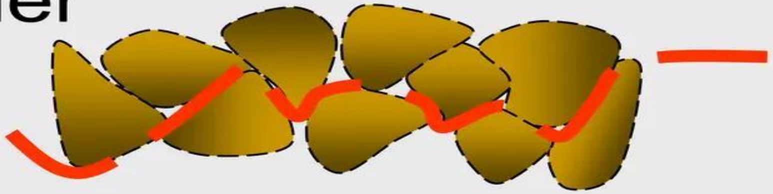
West side of foundation sank 24-ft

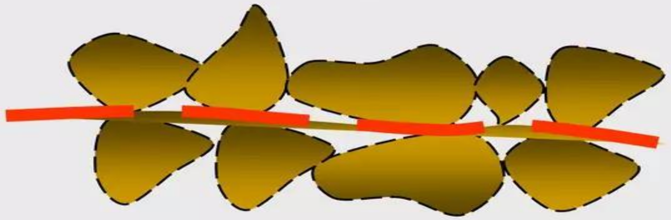
Significance of Shear Strength

- Engineers must understand the nature of shearing resistance in order to analyze soil stability problems such as;
  - Bearing capacity
  - Slope stability
  - Lateral earth pressure on earth-retaining structures
  - Pavement

Shear Strength in Soils

- The shear strength of a soil is its resistance to shearing stresses.
- It is a measure of the soil resistance to deformation by continuous displacement of its individual soil particles
- Shear strength in soils depends primarily on interactions between particles
- Shear failure occurs when the stresses between the particles are such that they slide or roll past each other





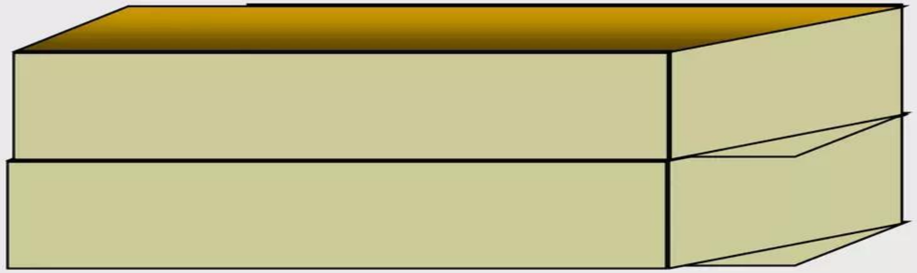
Shear Strength in Soils (cont.)

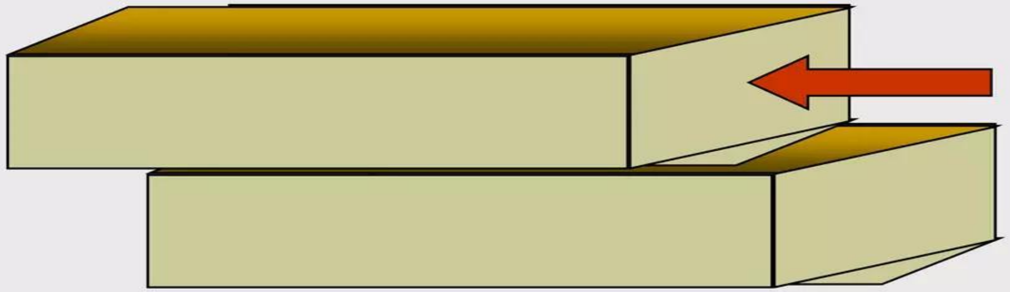
- Soil derives its shear strength from two sources:
  - Cohesion between particles (stress independent component)
    - Cementation between sand grains
    - Electrostatic attraction between clay particles
  - Frictional resistance between particles (stress dependent component)

Shear Strength of Soils; Cohesion

- Cohesion (C), is a measure of the forces that cement particles of soils

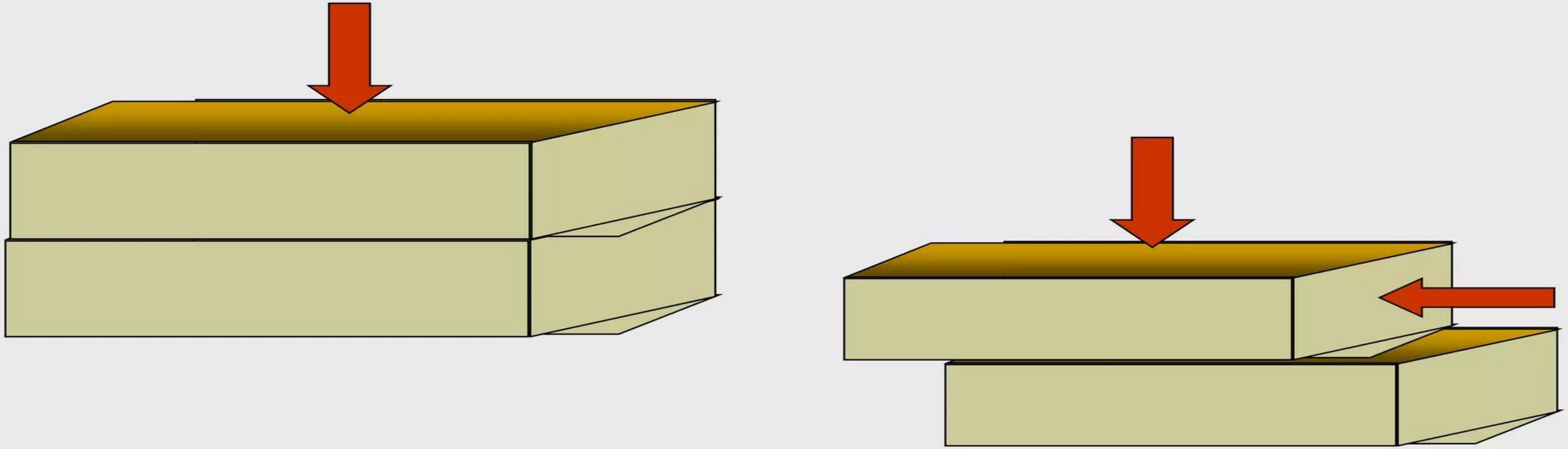
- Dry sand with no cementation
- Dry sand with some cementation
- Soft clay
- Stiff clay





Shear Strength of Soils; Internal Friction

- Internal Friction angle (ϕ), is the measure of the shear strength of soils due to friction



Mohr-Coulomb Failure Criteria

- This theory states that a material fails because of a critical combination of normal stress and shear stress, and not from their either maximum normal or shear stress alone.
- The relationship between normal stress and shear is given as

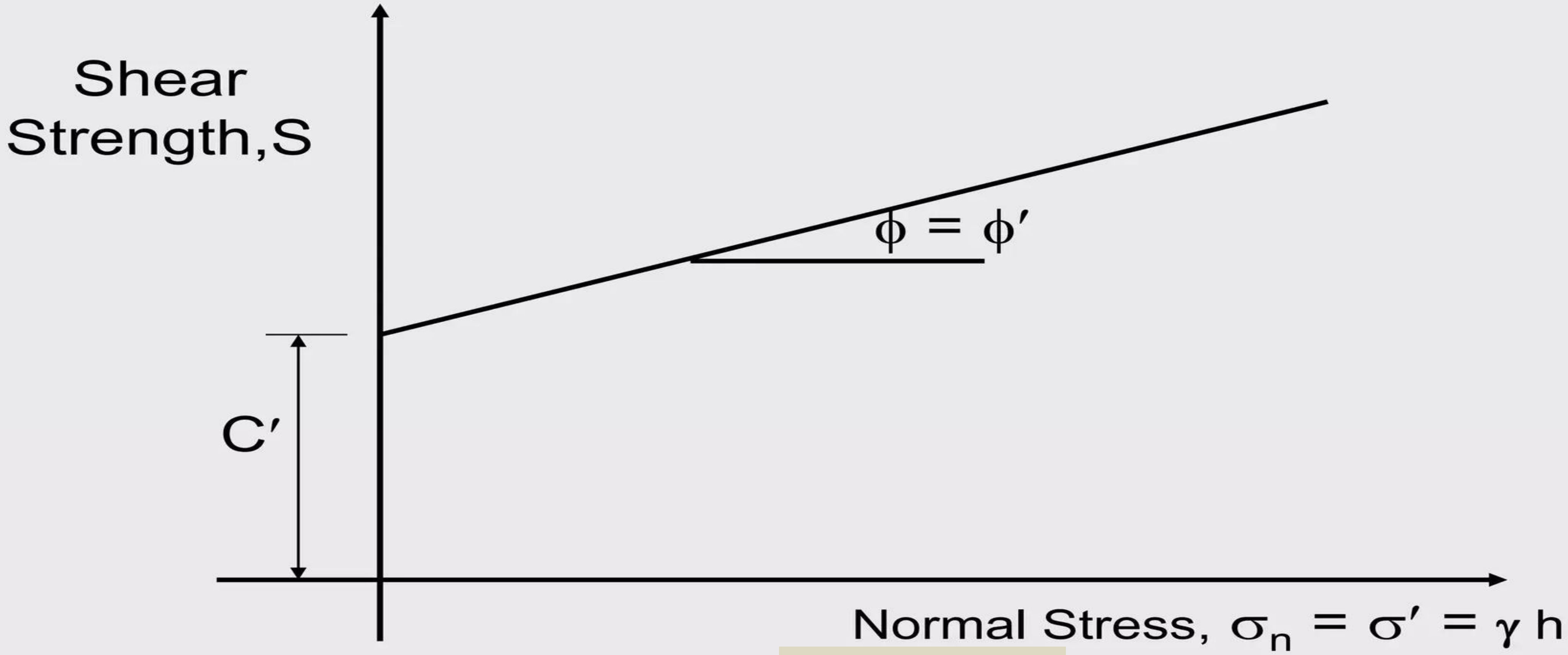
$$s = c' + \sigma' \tan \phi'$$

s = shear strength

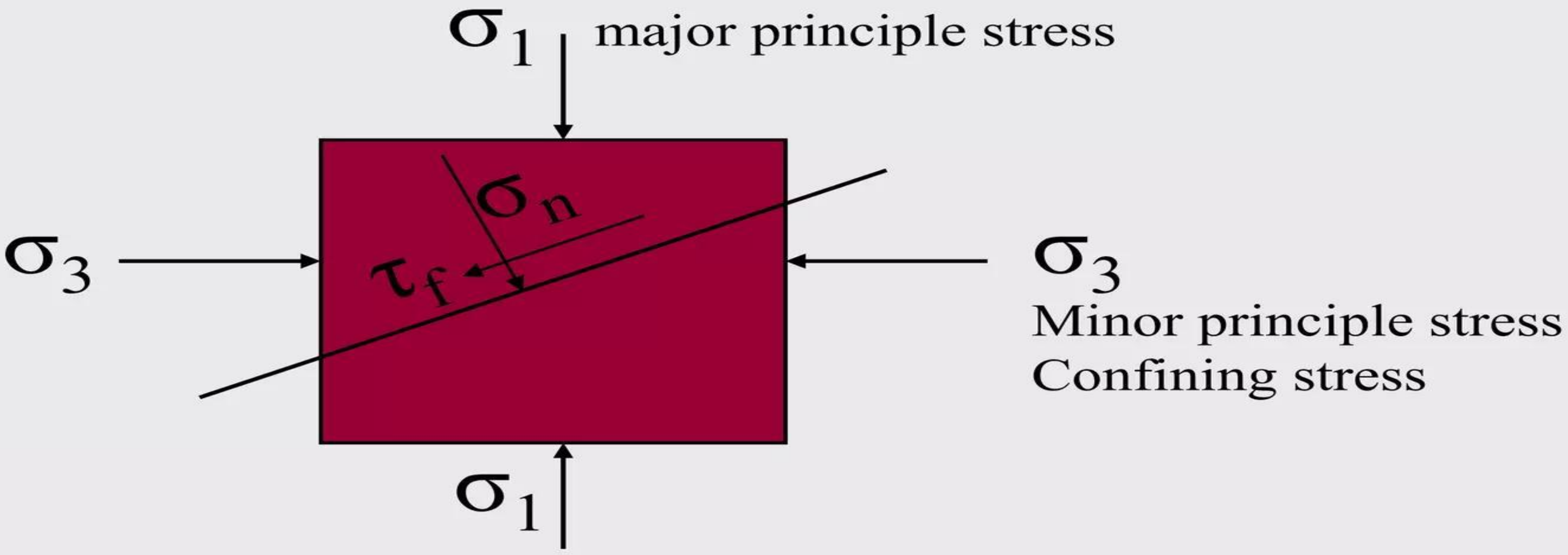
c' = cohesion

ϕ' = angle of internal friction

Mohr-Coulomb Failure Criterion



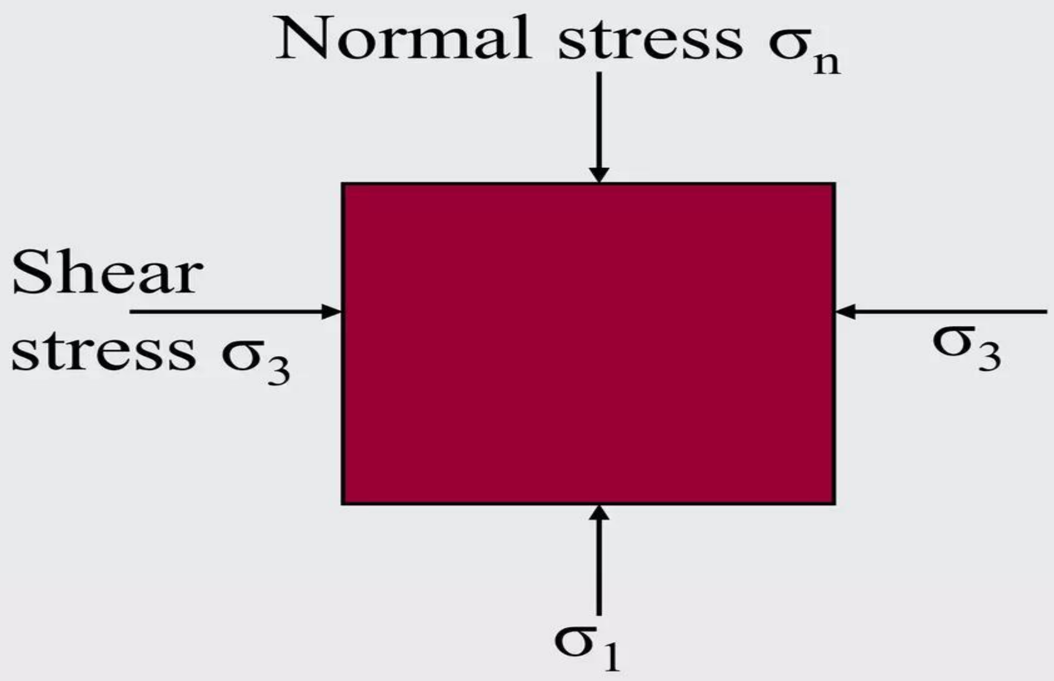
General State of Stress



State of Stresses in Soils

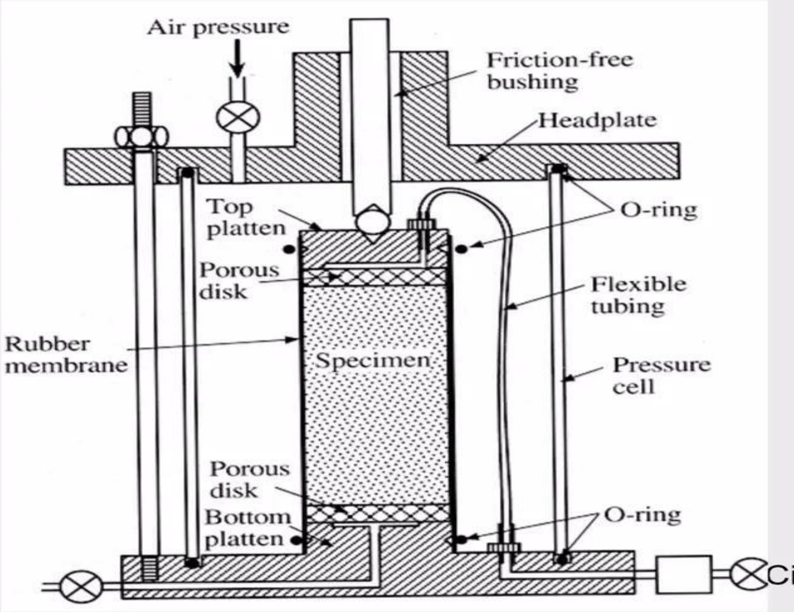
Consider the following situation:

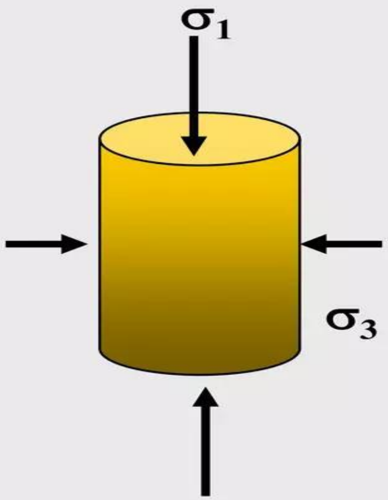
- A normal stress is applied vertically and held constant
- A shear stress is then applied until failure

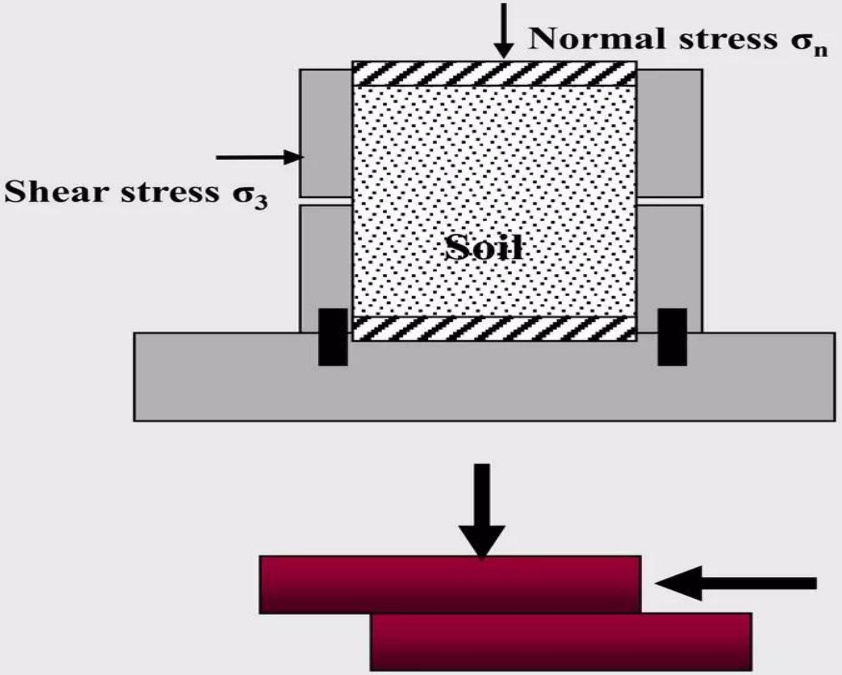


Determination of Shear Strength Parameters

- The shear strength parameters of a soil are determined in the lab primarily with two types of tests;
  - Direct Shear Test
  - Triaxial Shear Test





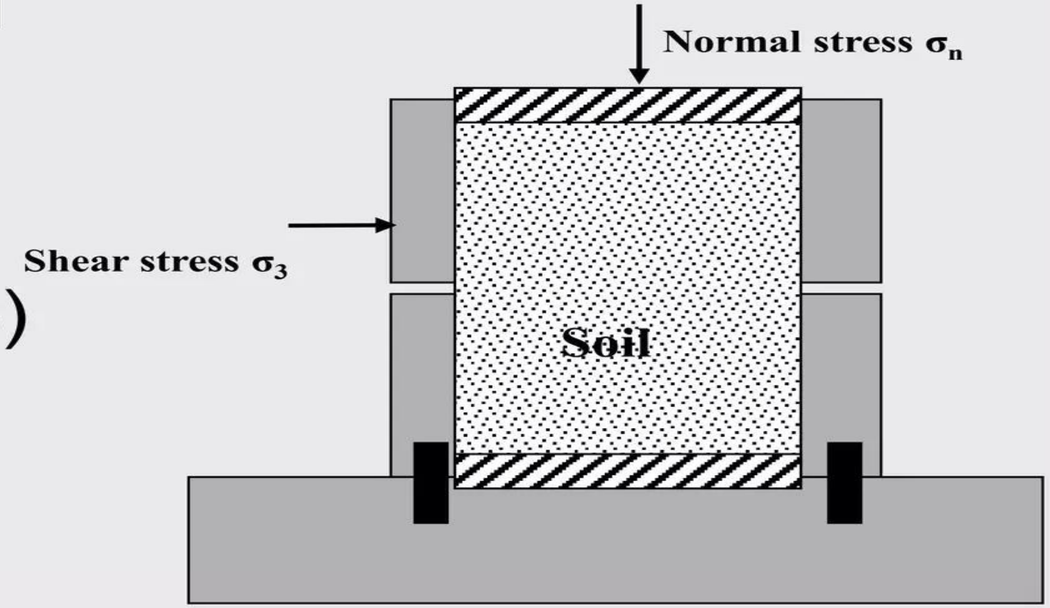


Direct Shear Test

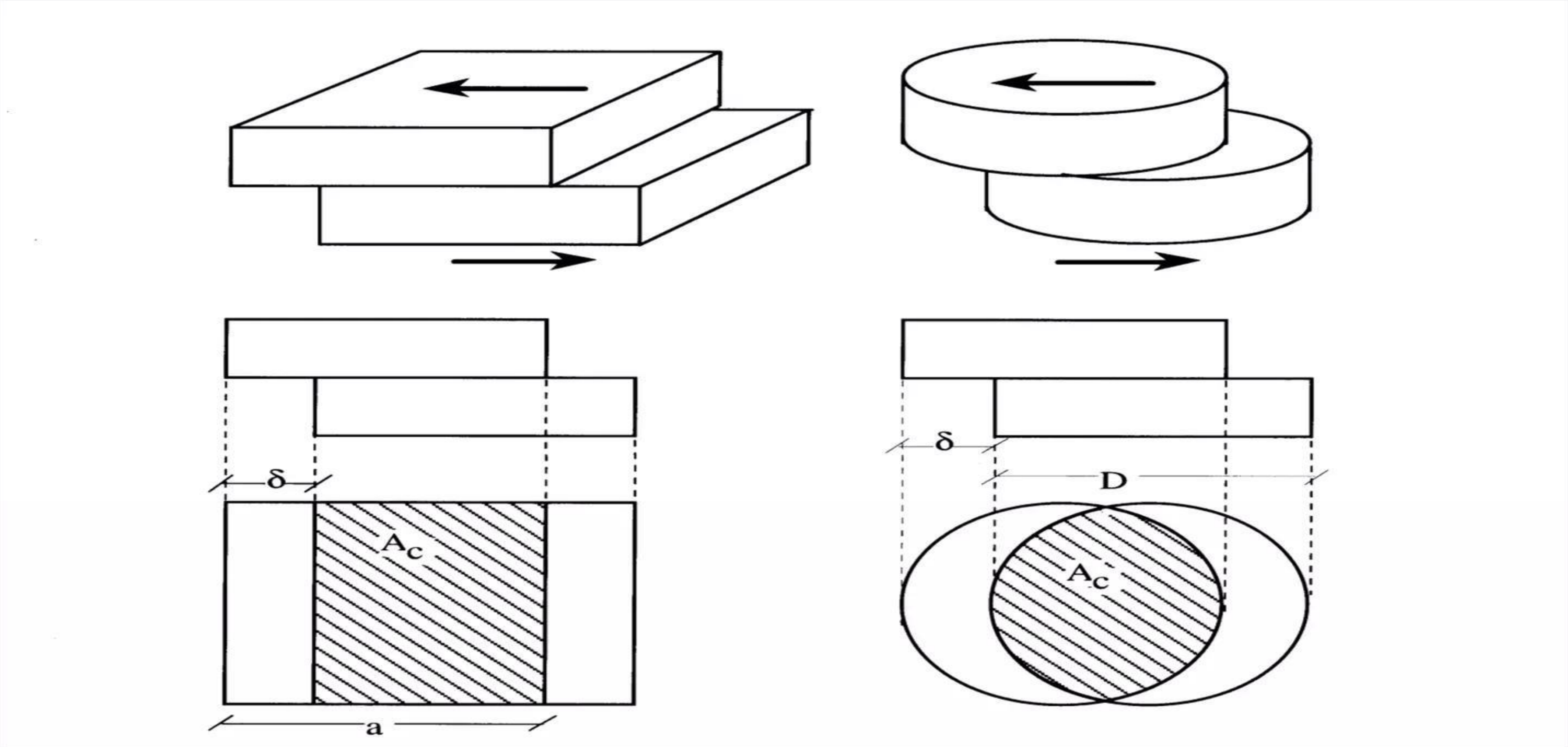
- Direct shear test is Quick and Inexpensive
- Shortcoming is that it fails the soil on a designated plane which may not be the weakest one
- Used to determine the shear strength of both cohesive as well as non-cohesive soils
- ASTM D 3080

Direct Shear Test (cont.)

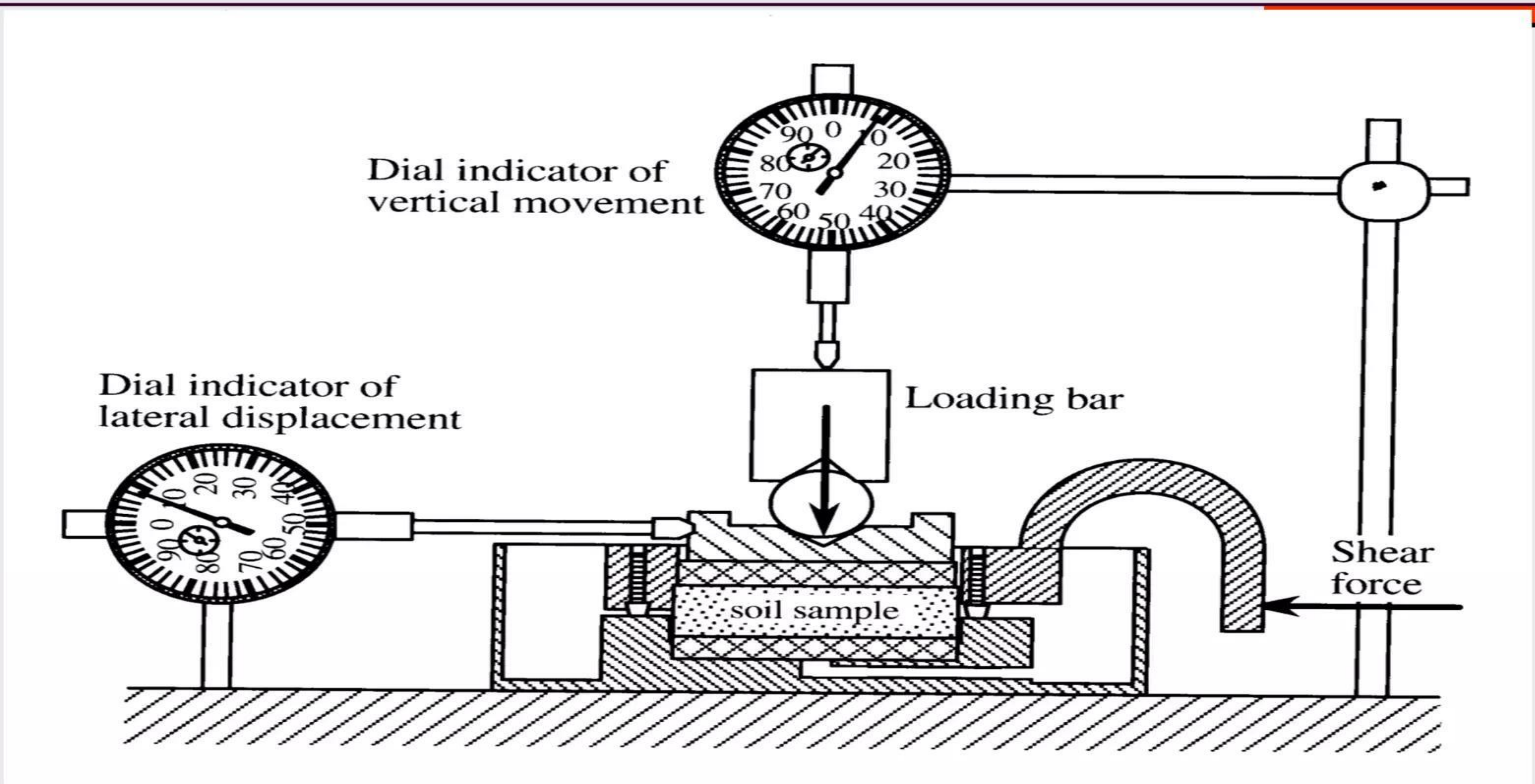
- The test equipment consists of a metal box in which the soil specimen is placed
- The box is split horizontally into two halves
- Vertical force (normal stress) is applied through a metal platen
- Shear force is applied by moving one half of the box relative to the other to cause failure in the soil specimen



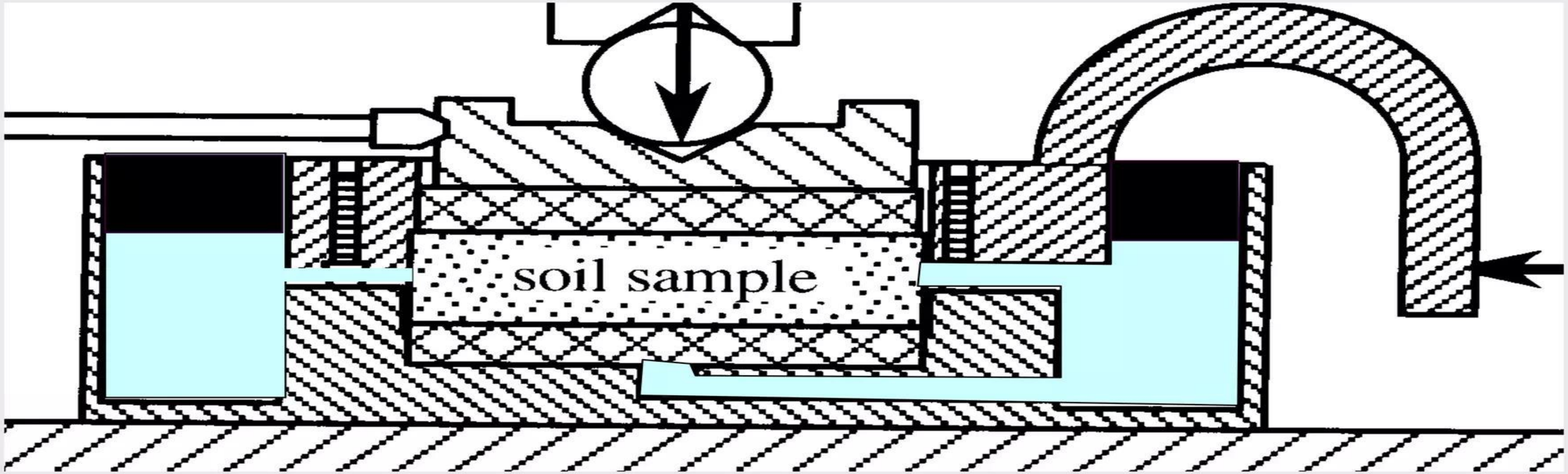
Direct Shear Test



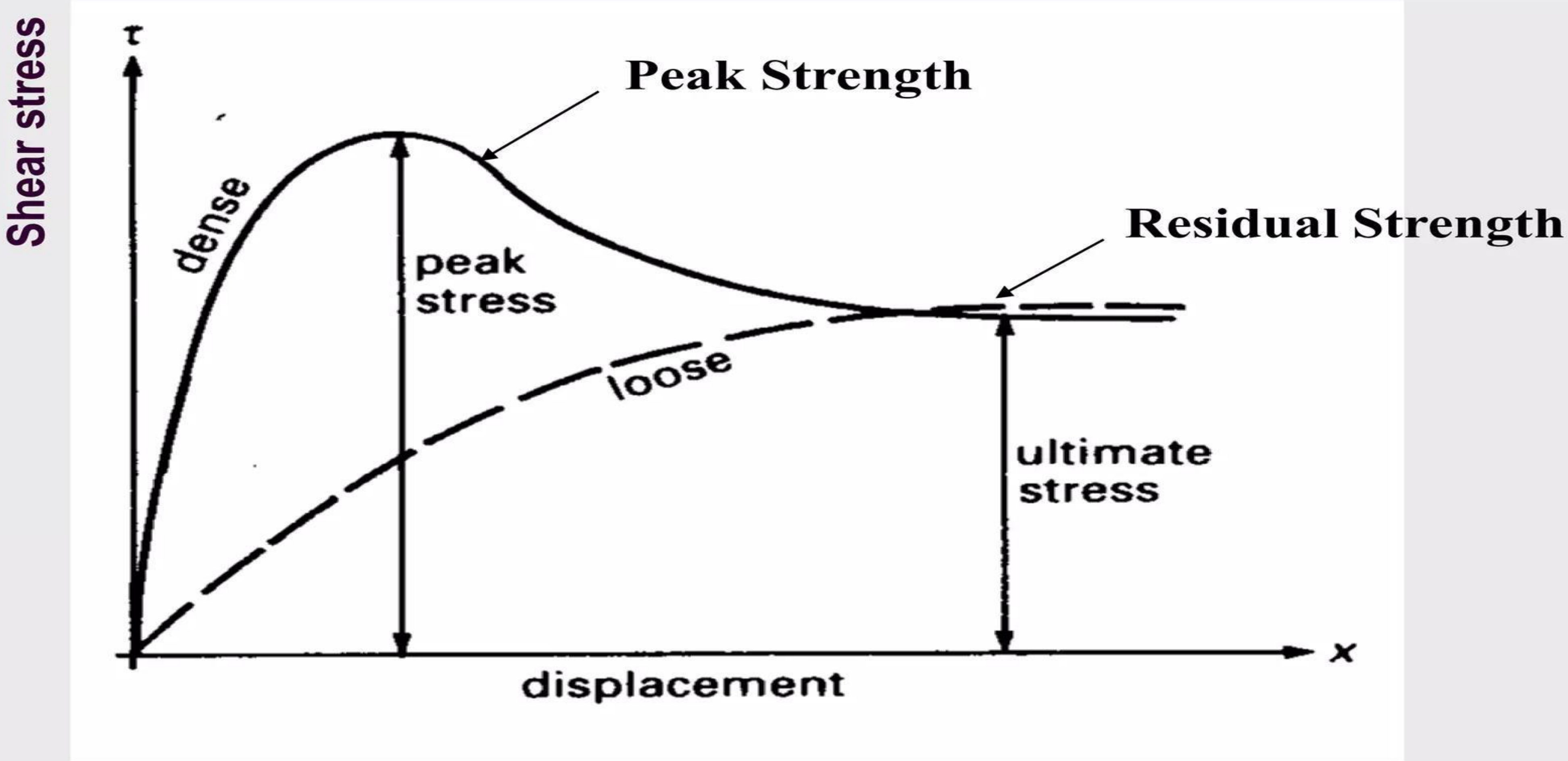
Direct Shear Test



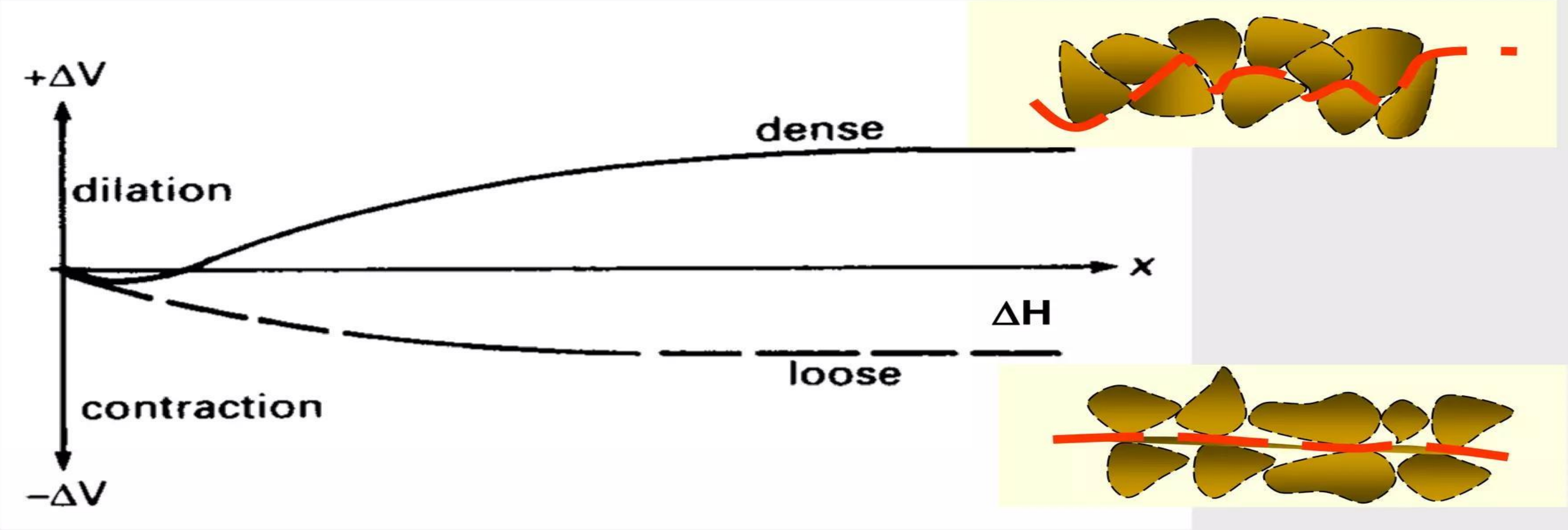
Direct Shear Test



Direct Shear Test Data



Direct Shear Test Data
Volume change



Direct Shear Test (Procedure)

1. Measure inner side or diameter of shear box and find the area
2. Make sure top and bottom halves of shear box are in contact and fixed together.
3. Weigh out 150 g of sand.
4. Place the soil in three layers in the mold using the funnel. Compact the soil with 20 blows per layer.
5. Place cover on top of sand
6. Place shear box in machine.
7. Apply normal force. The weights to use for the three runs are 2 kg, 4 kg, and 6 kg if the load is applied through a lever arm, or 10 kg, 20 kg, and 30 kg, if the load is applied directly.

Note: Lever arm loading ratio 1:10 (2kg weight = 20 kg)

Direct Shear Test (Procedure)

8. Start the motor with selected speed (0.1 in/min) so that the rate of shearing is at a selected constant rate
9. Take the horizontal displacement gauge, vertical displacement gage and shear load gage readings. Record the readings on the data sheet.
10. Continue taking readings until the horizontal shear load peaks and then falls, or the horizontal displacement reaches 15% of the diameter.

Direct Shear Test Data

Displacement rate: ______
Normal stress: 2.27 psi

Horizontal Dial Reading (0.001 in)	Horizontal Displacement (in)	Load Dial Reading	Horizontal Shear Force (lb)	Shear Stress (psi)
0	0	0	0	0
10	0.01	4	5.142	1.064
19	0.019	4.3	5.231	1.082
29	0.029	4.8	5.379	1.113
36	0.036	5	5.439	1.126
44	0.044	7	6.033	1.248

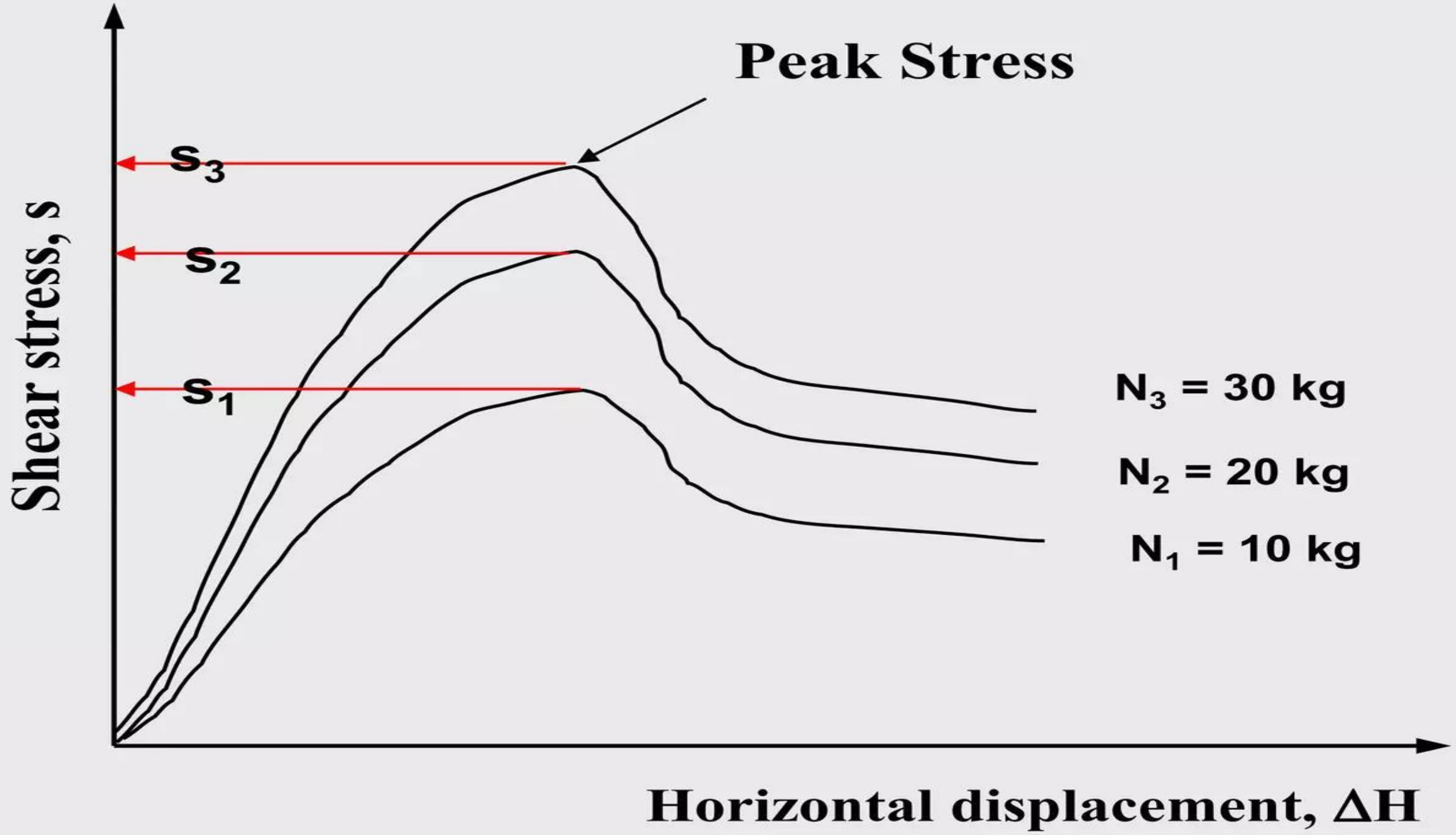
Calculations

1. Determine the dry unit weight, γ_d

2. Calculate the void ratio, e
$$e = \frac{Gs\gamma_w}{\gamma_d} - 1$$

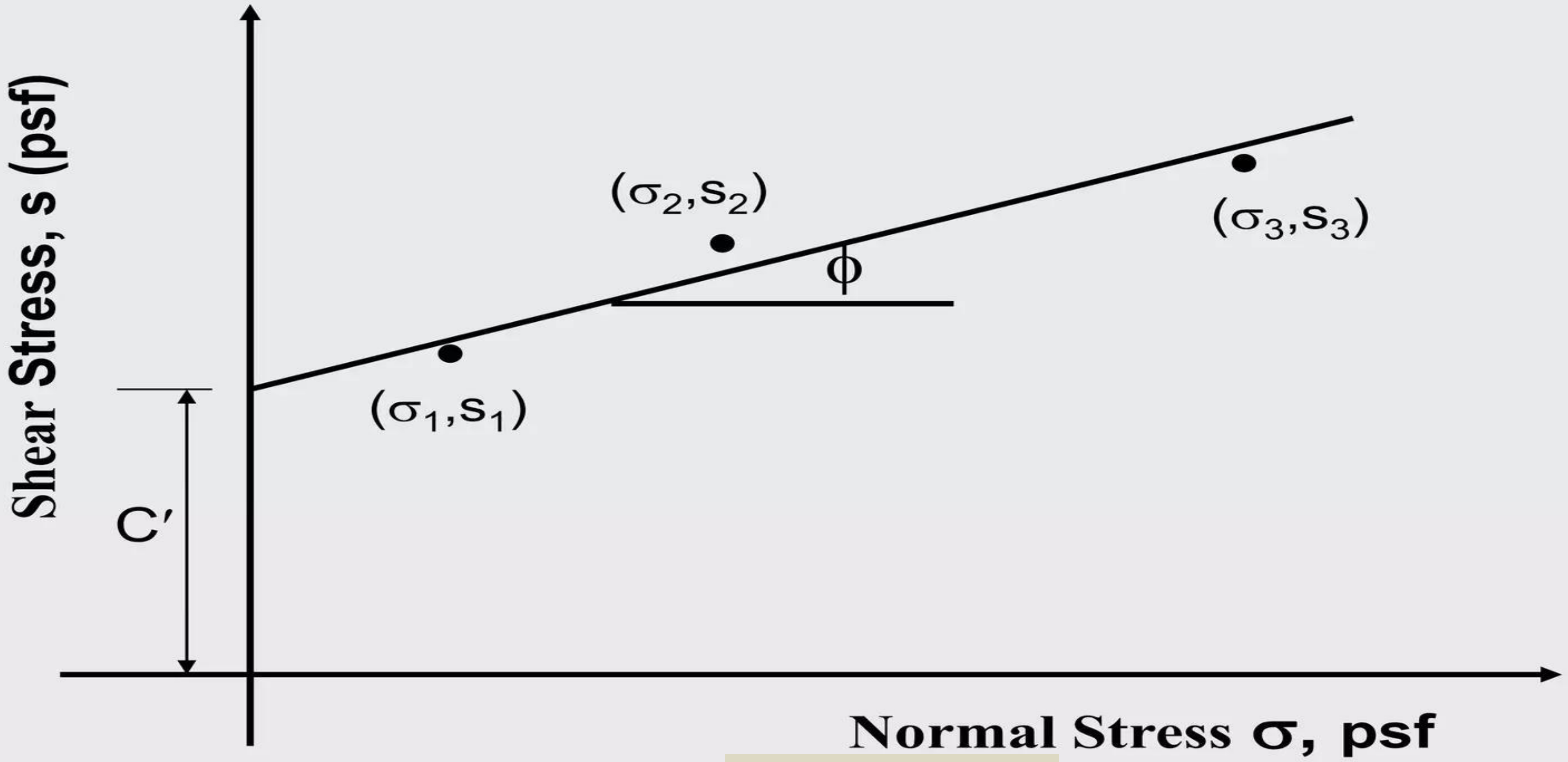
3. Calculate the normal stress & shear stress
$$\sigma = \frac{N}{A} \quad ; \quad \tau = \frac{V}{A}$$

Figures

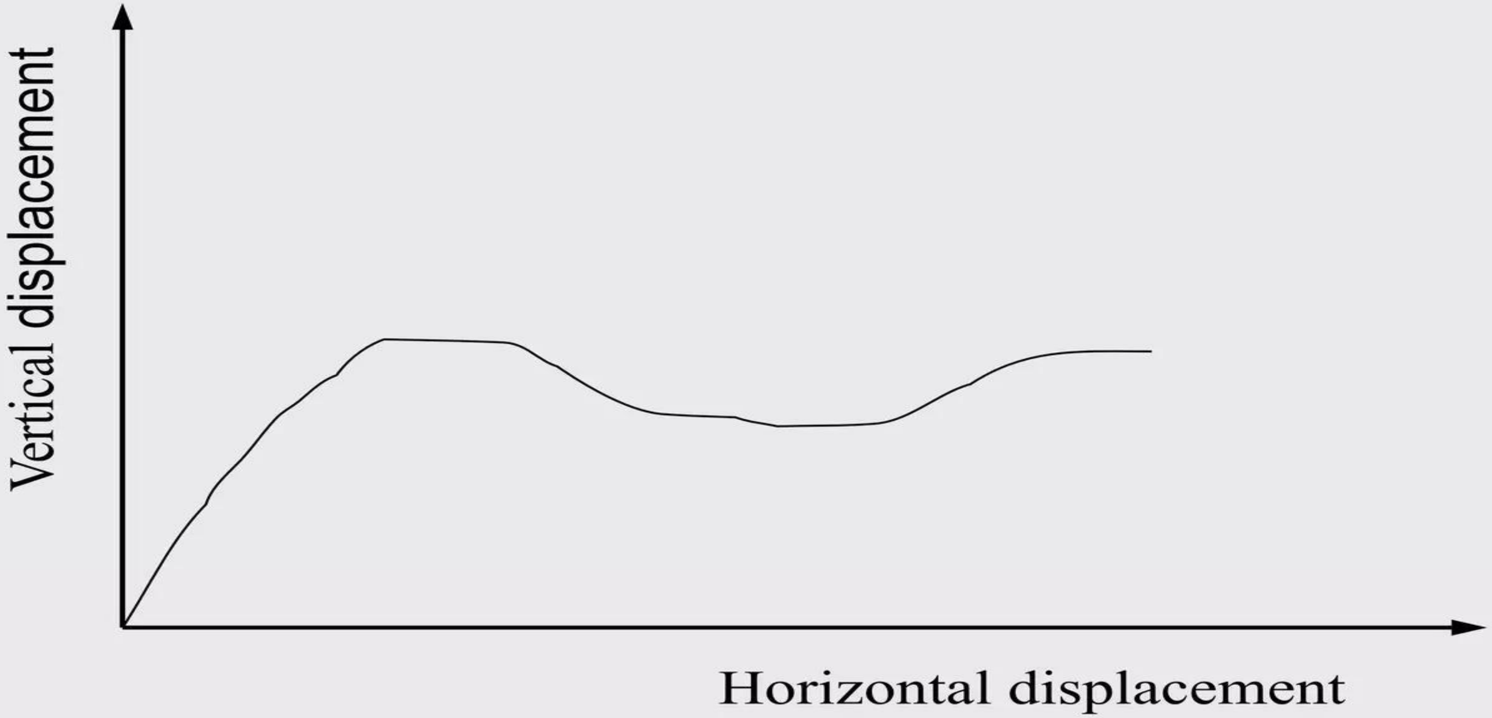


Peak Stress
s_3
s_2
s_1
Shear stress, s
N_3 = 30 kg
N_2 = 20 kg
N_1 = 10 kg
Horizontal displacement, ΔH

Figures (cont)



Figures (cont)



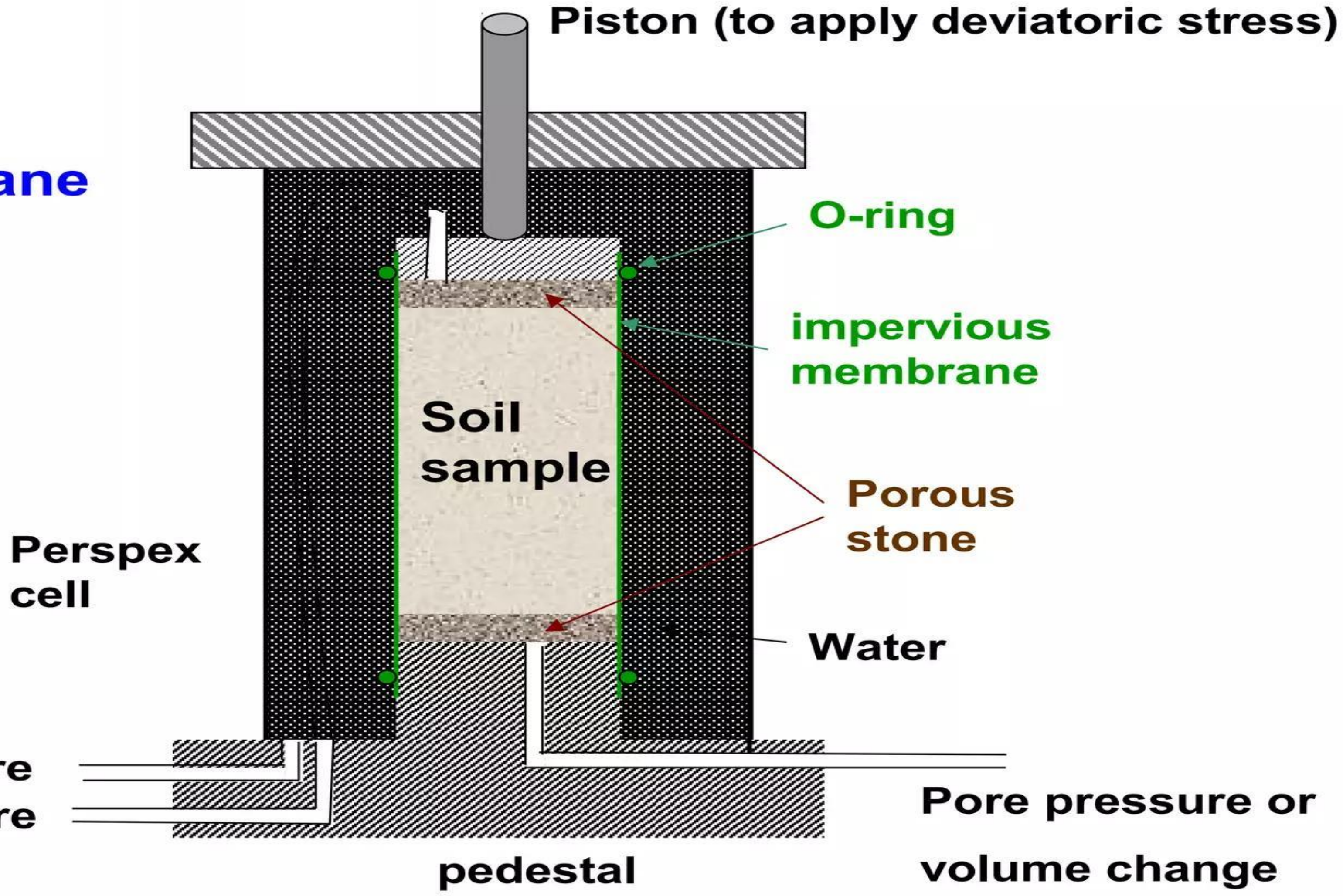
Shear Strength Parameter (Tri-axial Shear Test)

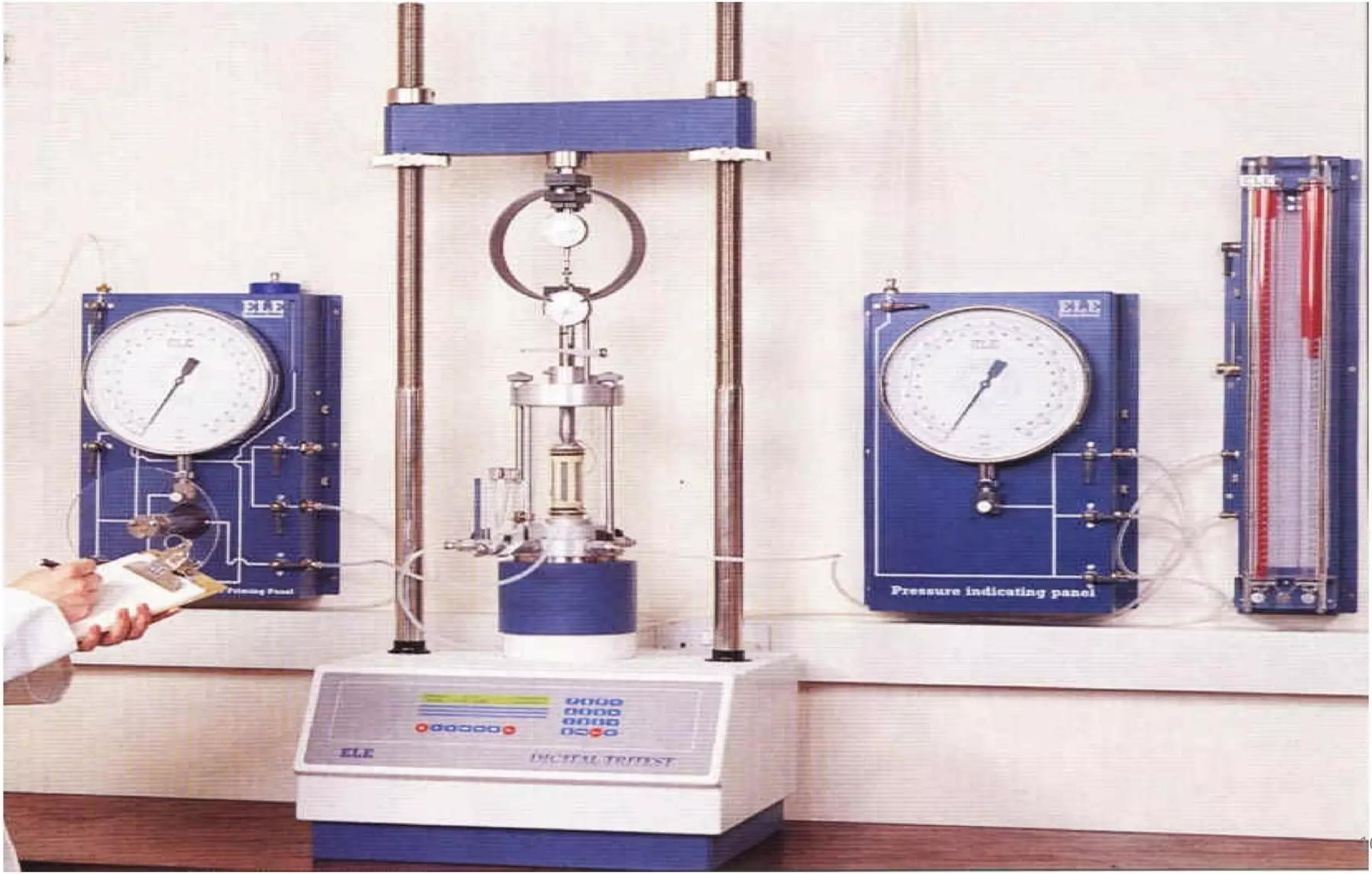
Week 7

Pages 165-193

Triaxial Shear Test

Soil sample at failure




ELE
ELE
Pressure indicating panel
ELE
DIGITAL TRITEST

Triaxial Shear Test

Specimen preparation (undisturbed sample)

Sampling tubes

Sample extruder

Triaxial Shear Test

Specimen preparation (undisturbed sample)

Edges of the sample are carefully trimmed

Setting up the sample in the triaxial cell

Triaxial Shear Test

Specimen preparation (undisturbed sample)

Sample is covered with a rubber membrane and sealed

Cell is completely filled with water

Triaxial Shear Test

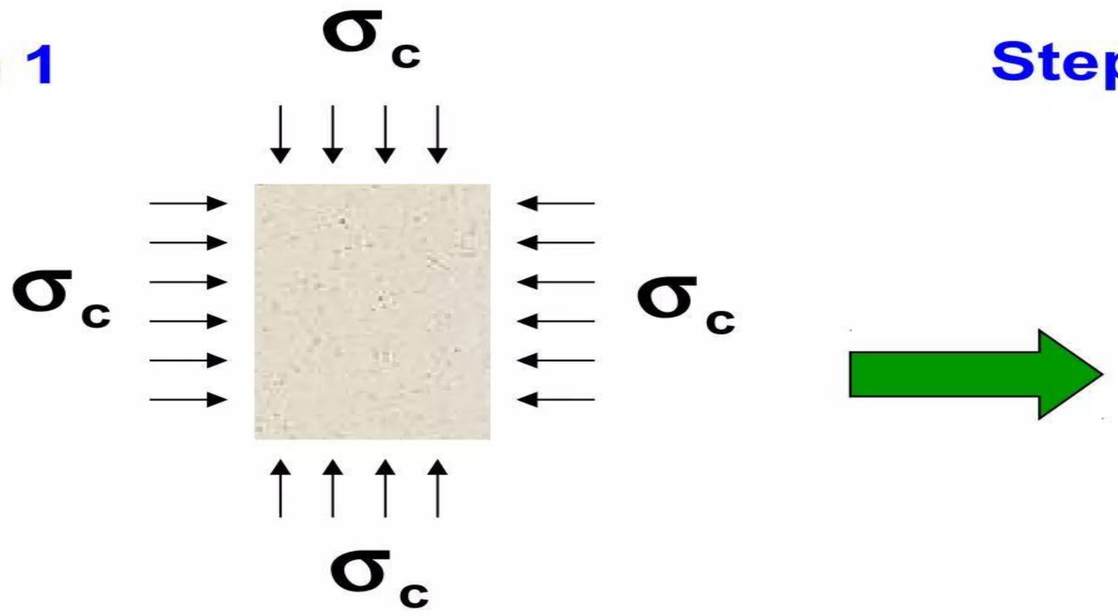
Specimen preparation (undisturbed sample)

Proving ring to measure the deviator load

Dial gauge to measure vertical displacement

Types of Triaxial Tests

Step 1

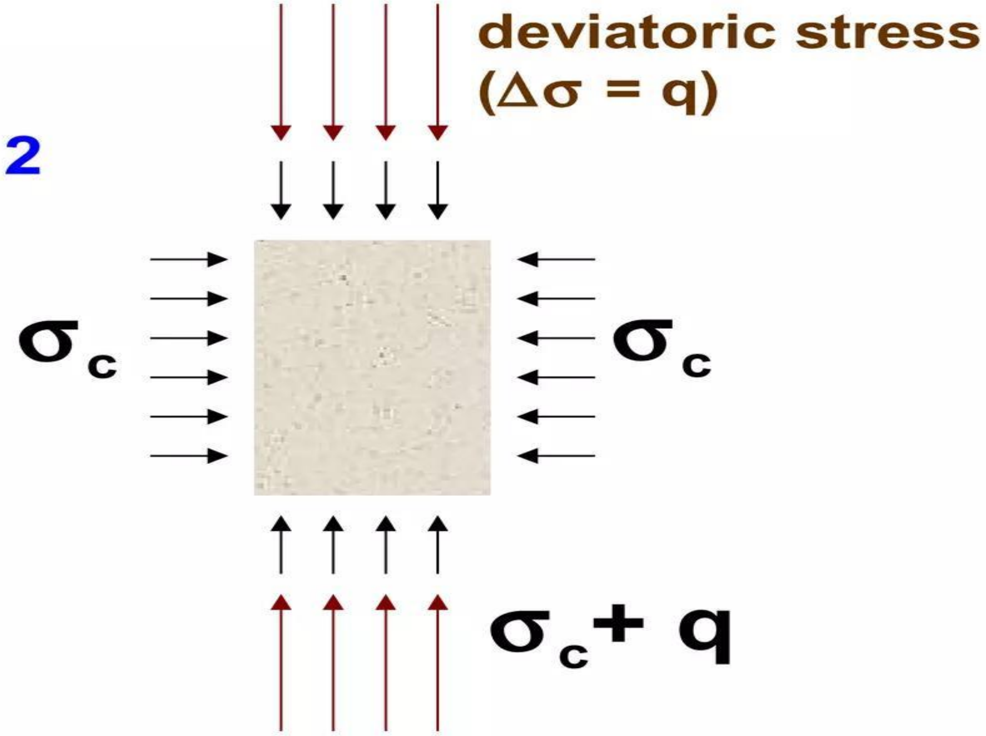


Under all-around cell pressure σ_c

Is the drainage valve open?

- yes → **Consolidated sample**
- no → **Unconsolidated sample**

Step 2

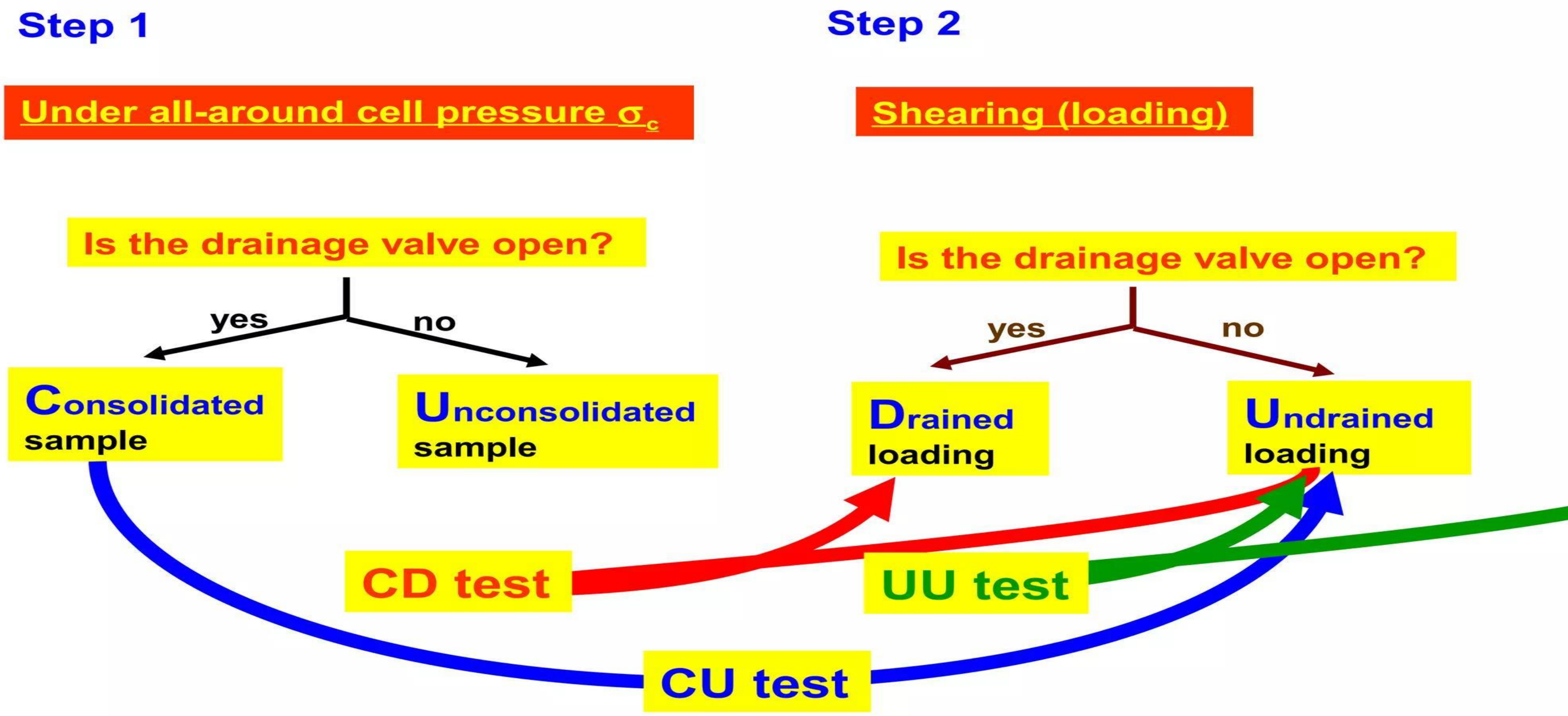


Shearing (loading)

Is the drainage valve open?

- yes → **Drained loading**
- no → **Undrained loading**

Types of Triaxial Tests



Consolidated- drained test (CD Test)

Total, σ = **Neutral, u** + **Effective, σ'**

Step 1: At the end of consolidation

σ_{VC}

σ_{hC}

Drainage

0

$\sigma'_{VC} = \sigma_{VC}$

$\sigma'_{hC} = \sigma_{hC}$

Step 2: During axial stress increase

$\sigma_{VC} + \Delta\sigma$

σ_{hC}

Drainage

0

$\sigma'_{V} = \sigma_{VC} + \Delta\sigma = \sigma'_{1}$

$\sigma'_{h} = \sigma_{hC} = \sigma'_{3}$

Step 3: At failure

$\sigma_{VC} + \Delta\sigma_{f}$

σ_{hC}

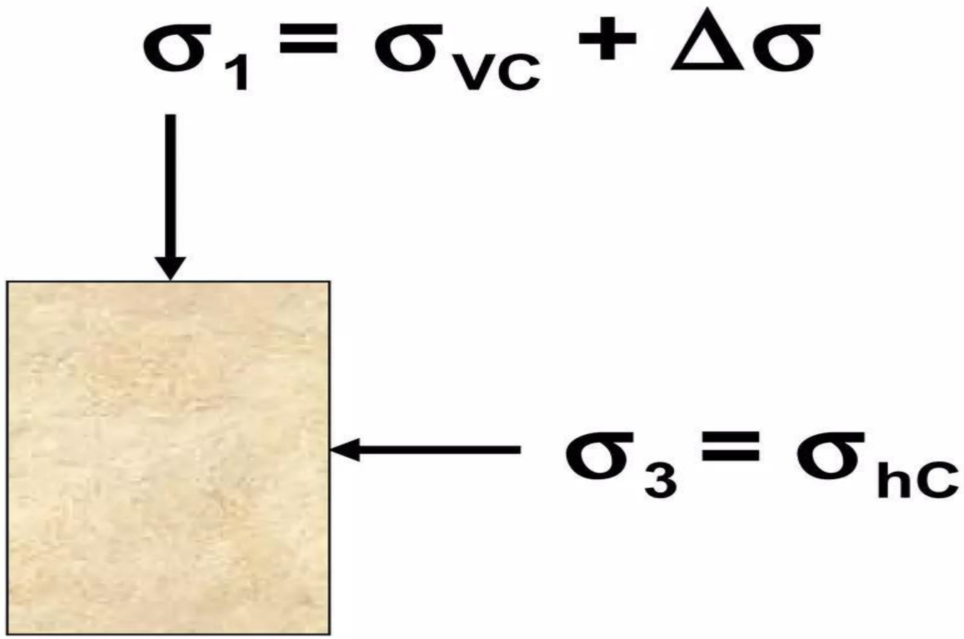
Drainage

0

$\sigma'_{Vf} = \sigma_{VC} + \Delta\sigma_{f} = \sigma'_{1f}$

$\sigma'_{hf} = \sigma_{hC} = \sigma'_{3f}$

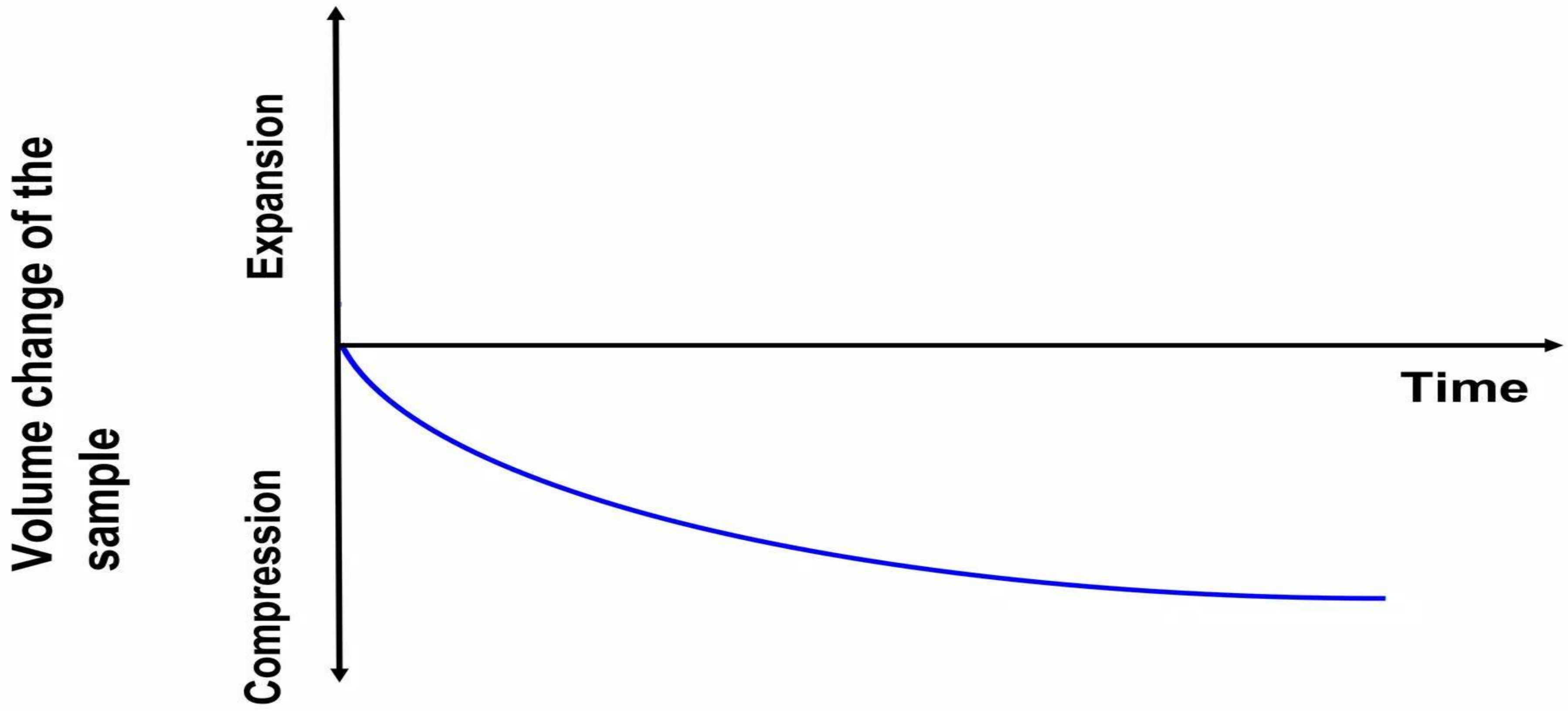
Consolidated- drained test (CD Test)



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

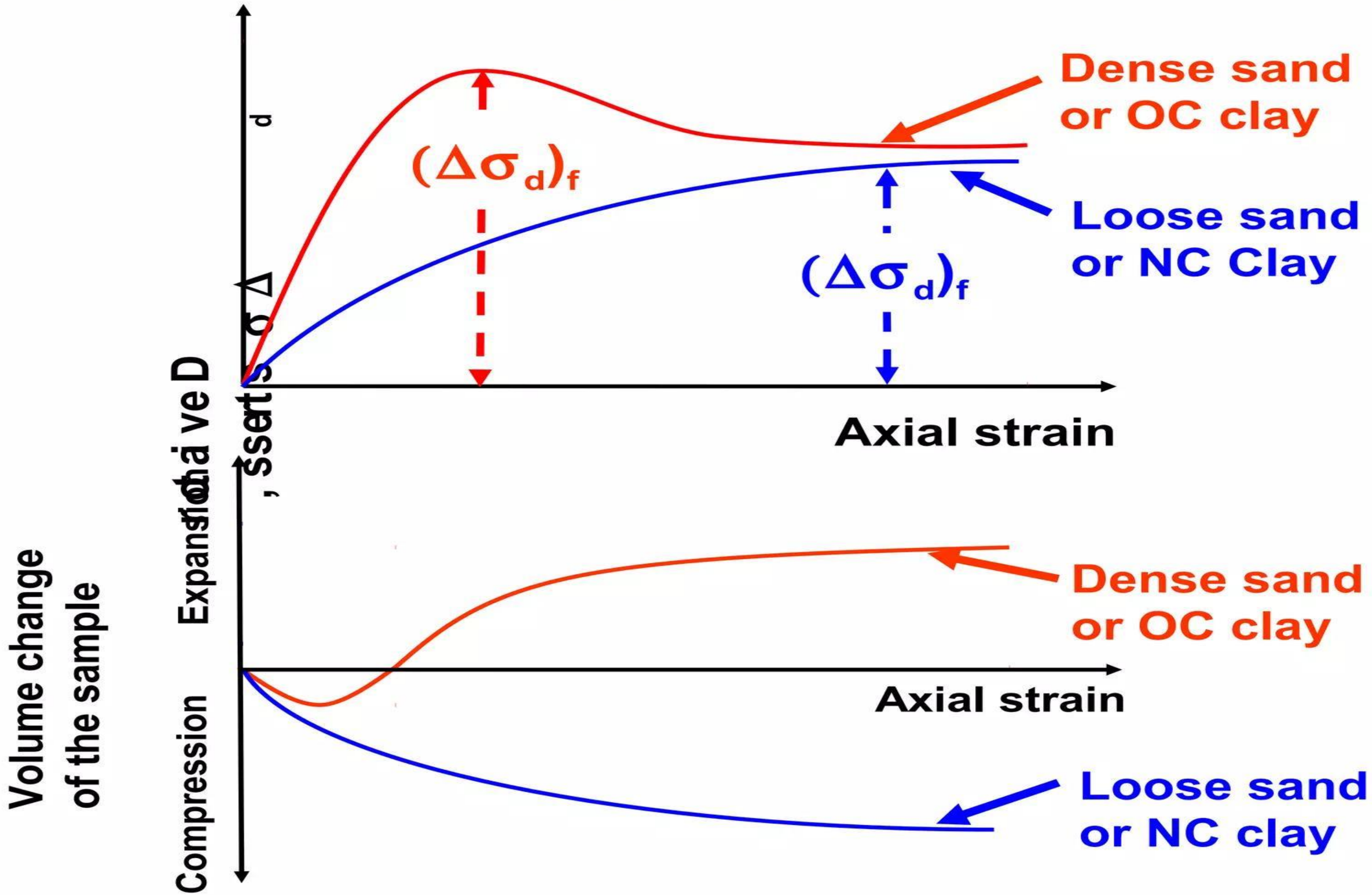
Consolidated- drained test (CD Test)

Volume change of sample during consolidation



Consolidated- drained test (CD Test)

Stress-strain relationship during shearing



Tri-axial compression

- Tri-axial compression or tri-axial shear test is a common method to measure the mechanical properties of many deformable solids, especially soil (e.g., sand, clay) and rock, and other granular materials or powders.
- In this test, soil sample is subjected to stress, such as the stress resulted in one direction will be different in perpendicular direction.
- The material properties of soil like shear resistance, cohesion and the stress is determined from this test.
- The test is most widely used and is suitable for all types of soil.

Tri-axial shear test apparatus

- The important apparatus for tri-axial shear test are:

i. Tri-axial testing machine complete with tri-axial cell
ii. Equipment for loading
iii. Equipment to measure load and deformation:

- Proving is used to measure load application by the piston. A dial gauge is use to measure the deformation of specimen.

Tri-axial shear test apparatus

i. Tri-axial testing machine complete with tri-axial cell:

- This unit have a provision to insert a cylindrical soil specimen that is sealed by mean of rubber membrane to resist the entry of literal fluid.
- Radical fluid pressure and the vertical stress is applied by means of a piston arrangement.
- The unit also have the provision to prevent the drainage of specimen.
- The fluid pressure in a cell can not be measured by mean of pressure gauge.

Tri-axial shear test procedure

- The specimen can be prepared either re-moulded or undisturbed.
- Undisturbed soil can bed tested on soils that have sufficient cohesion.
- In order to make re-moulded soil is collected and compacted property.
- Care is taken while preparing the cohesion less soils.
- The trial test be conducted in different variations. The most commonly employed types are:

i. Unconsolidated un-drained test (UU)

ii. Consolidated un-drained test (CU)

iii. Consolidated drained test (CD)

Tri-axial shear test procedure

i. Unconsolidated un-drained test (UU)

- As the name tells, the soil sample is subjected to cell pressure with no provision of drainage.
- Here the cell pressure is maintained to a constant value and the applied deviator stress is increased till the sample falls.
- This is called quick test.

Tri-axial shear test procedure

ii. Consolidated un-drained test (CU)

- Here during application of cell pressure on the sample, drainage is permitted.
- And the deviator stress is applied keeping the cell pressure constant and no provision of further drainage.

Tri-axial shear test procedure

iii. Consolidated drained test (CD)

- This test is also called as drained or slow test.
- Here the deviator stress is increased by allowing the drainage to happen as it was cell pressure is also kept constant.
- Here the loading is applied slowly so that excess pore pressure is not developed with in the sample.

Tri-axial shear test procedure

- To prepared specimen in enveloped in the membrane and positioned in the tri-axial cell.
- To this, the desired lateral pressure is applied.
- Till the specimen fails, the literal pressure is applied.
- The vertical deformation and the load reading are recorded.
- The main objective of the test is to determine the values of cohesion angle and internal friction.
- To determined these values, three different lateral pressure values have to been tested on the sample.

Calculation

- The specimen is subjected to all around lateral pressure ($\sigma 3$).
- The deviatric stress applied be σd.
- Then total vertical stress $\sigma 1$.

$$\sigma 1 = \sigma d + \sigma 3$$

- A Mohr's circle is drawn by plotting $\sigma 1$ and $\sigma 3$ in x-axis and the shear stress is the y-axis.
- Mohr's rupture is envelope is obtained by drawing the tangent to the circles obtained.
- The tangent intercept at the y-axis. The y tangent will intercept will give the value of cohesion (C).

Calculation

- The slope of failure plane or the tangent line give the angle of internal friction of the soil (Ø).
- The loading can increase the cross section of the soil specimen.
- This will required a correction for the deviatric stress σd.
- Here the correction is applied by assuming the volume of the specimen remain constant and the specimen remain constant and the area is veried.

Calculation

- The corrected deviatric stress is

$$\boldsymbol{\sigma d = (P1/Ao) \times \{1 - (L1/Lo)\}}$$

P1 = applied load, Ao = original area of cross section

Lo = specimen original length, L1 = deformation of specimen

- The shear resistance of sample is given by:

$$\boldsymbol{\tau f = C + \sigma 1 \tan Ø}$$

Pore water pressure

- Pore water pressure must be measured under condition of no either out of or into specimen, otherwise the correct pressure get modified.
- It is possible to measure pore water pressure at one end of the specimen while drainage is taken place at the other end.
- The no flow condition is maintained by the use of null indicator, essentially a U-tube partly with mercury.

Advantages of tri-axial test

- Stress distribution on the failure plane is uniform.
- The specimen is free to fall on the weakest plane.
- There is complete control over the drainage.
- Pore pressure changes are the volumetric changes can be measured directly.
- The stress of stress at all intermediate stages up to failure is known.
- The Mohr's cycle can drawn at any stage of shear.
- This is suitable for accurate research work and the apparatus adaptable to special requirements such as extension test and for different stress path.

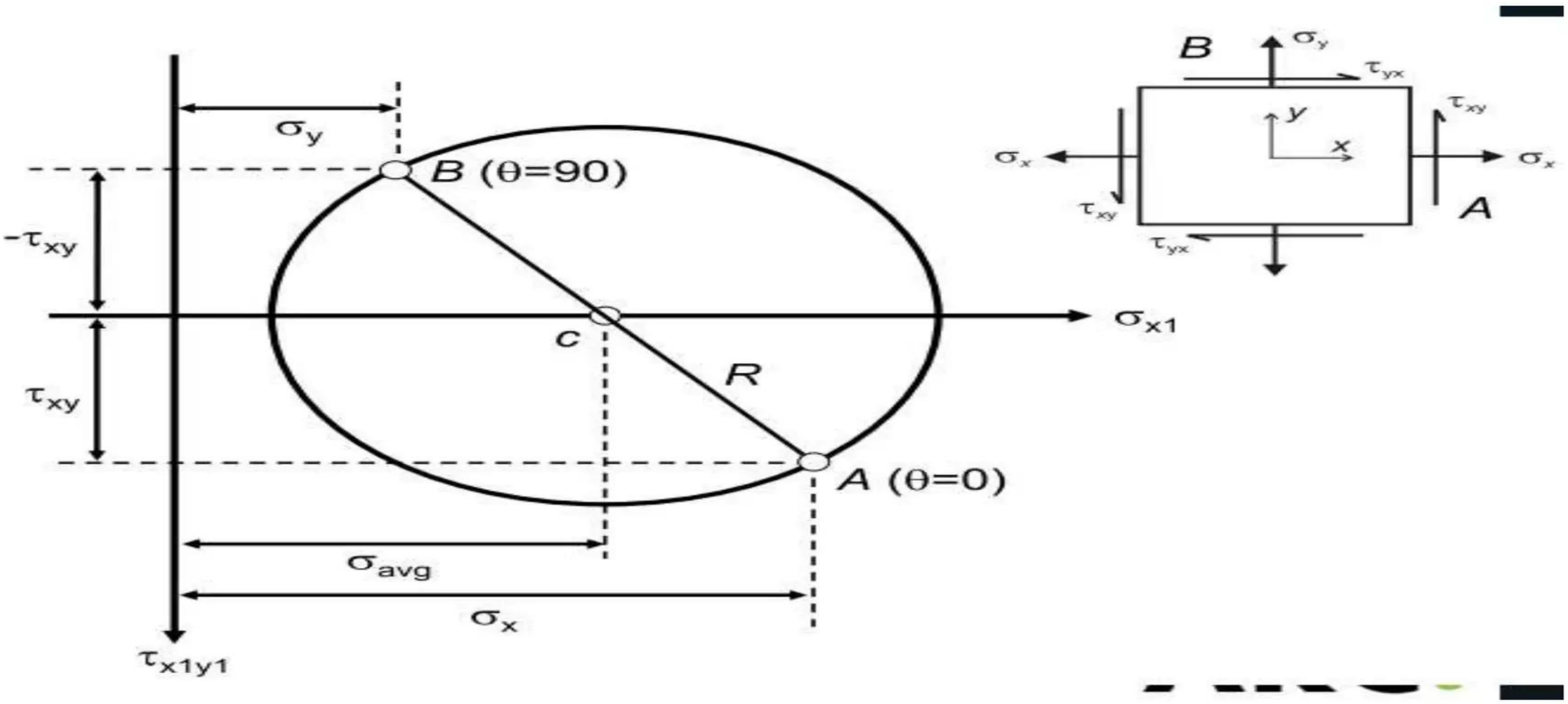
Disadvantages of tri-axial test

- The apparatus is elaborate, bulky and costly.
- The drain test take place a longer period in comparison with direct shear test.
- It is not possible to determine the cross sectional area of specimen at larger strains, at the assumption that the cylinder remains cylindrical does not hold good.
- The strain condition in the specimen are not uniform due to frictional restraint produced by loading cap and the pedestal disc. This lead the formation of dead zones at each end of the specimen.
- The consolidation of the specimen in the test is isotropic, where as in the field as inisotropic.

Mohr’s Circle

- Mohr's circle if often used in calculations relating to mechanical engineering for material strength , geotechnical engineering for strength of soil and structural engineering for strength of built structures relatively.
- It is calculating stresses in many planes by reducing them to vertical and horizontal components.
- Mohr's circle can also be used to find principals planes and principal stresses in a graphical representation.

Constructing Mohr circle procedure



Shear Strength Parameter (Unconfined Compression Test)

Week 8

Pages 195-206

❖ Introduction:-

Unconfined Compression Test is a special type of Unconsolidated – Undrained (UU) test that is commonly used for clay specimens. It is special case of a triaxial compression test.

In this test the confining pressure (σ_3) is 0. In this, cylindrical soil specimen (with height to diameter ratio of 2 to 2.5) is loaded axially by a compressive force until failure takes place. No rubber membrane is necessary to encase the specimen. The vertical compressive stress is the major principal stress (σ_1) and the other two principal stresses are zero.

This test may be conducted on undisturbed or remoulded cohesive soils. It cannot be conducted on coarse-grained soils such as sands and gravels as these cannot stand without lateral support. Also the test is essentially a quick or Undrained one because it is assumed that there is no loss of moisture during the test, which is performed fairly fast.

Required apparatus for Unconfined compression test:-

- Compression Machine
- Proving Ring of capacity 500 N & 1000 N (with least count 1.0 & 0.2 resp.)
- Dial gauge of least count of 0.01 mm.
- Split mould of internal dia. 38 mm & length 76mm.
- Sampling tube of internal dia. 38mm & length 200mm.
- Balance of accuracy 0.1 gm.
- Sample extractor
- Stop watch
- Scale
- Knife
- Grease / oil

❖ Parts of Compression Machine-

COMPRESSION MACHINE

➢ The main question is how to perform unconfined compression test ????

Procedure:-

→ Push the sampling tube into the sample, remove the sampling tube along with the soil.
→ Saturated the soil sample in sampling tube.
→ Coat the inside of the split mould with a thin layer of grease/oil to prevent adhesion of the soil.
→ Extrude the specimen from the sampling tube to the split mould with the help of sample extractor and knife.
→ Trim the two ends of the mould.
→ Weight the soil sample and mould.
→ Remove the sample from the mould by spliting it in two parts.
→ Measure the length and dia. of the specimen.
→ Place the specimen on the bottom plate of the compression machine.
→ Raise the bottom plate of the machine to make contact of the specimen with the upper plate.

→ Adjust the strain dial gauge and proving ring dial gauge to read zero.

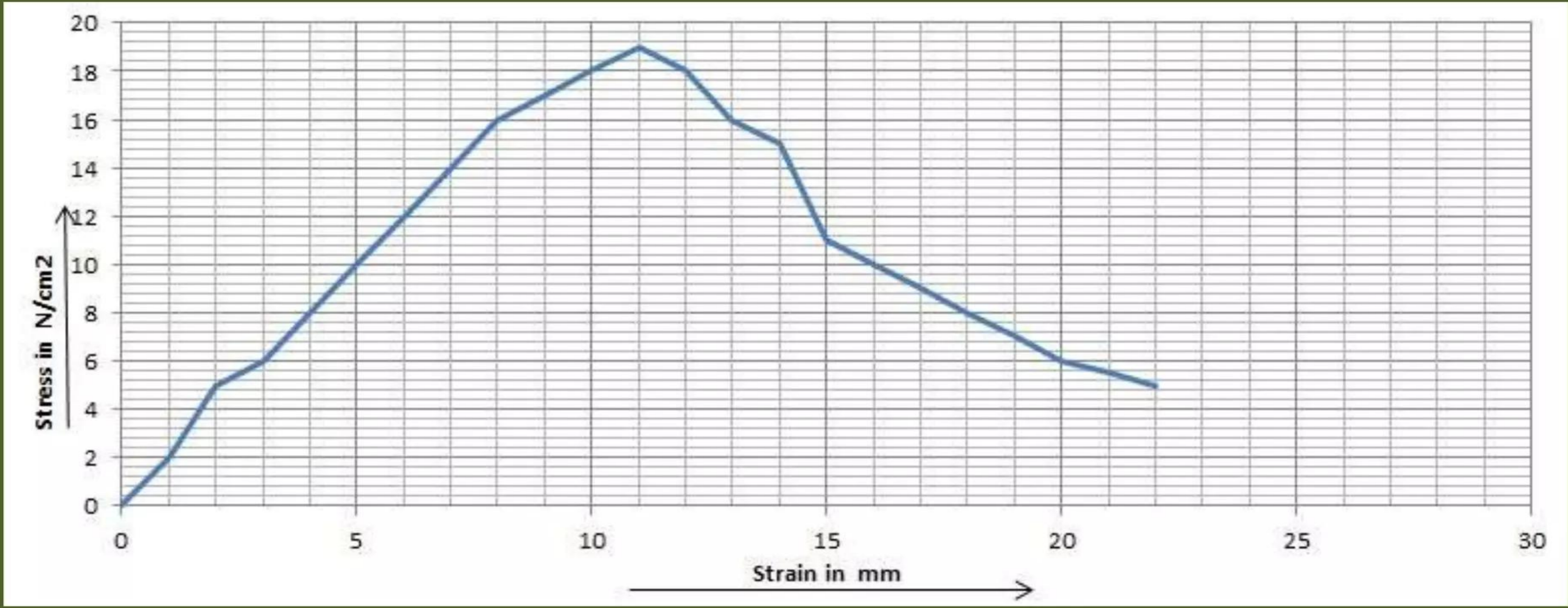
→ Apply the compression load by raising the bottom plate of the machine to produce axial strain at a rate of $\frac{1}{2}$ to 2% per minute.

→ Record the strain and proving ring dial gauges readings every 30 seconds.

→ Compress the specimen till it fails or 20% vertical deformation is reached, whichever is earlier.

→ Note the least count of strain gauge and load dial gauge.

- From the recorded value of strain and proving ring dial gauge reading(after every 30 sec. interval, before failure), we will draw a stress stain curve



Due to this test we can check the various parameters of the soil, like

- Unconfined compressive strength
- Sensitivity of soil
- Shear parameters of the soil, etc.

❑ The Unconfined Compressive Strength (qu) is defined as the ratio of failure load to the cross sectional area of the soil sample, if it is not subjected to any lateral pressure.

$$q_u = \frac{P}{A_c}$$

Where:-

q_u = Unconfined Compressive Strength
P = Failure Load
A_C = Corrected Area at failure.

Now :- $A_c = \frac{A_o}{1-e}$

A_o = Initial Area
e = Strain

Again, $e = \frac{\Delta L}{L_o}$

ΔL = Change in length
L_O = Initial Length of the sample

Water content of the soil is assumed to remain constant during the duration of the test which generally takes only a few minutes.

Relationship between consistency of clays and *qu*

q_u, kN/m2	Consistency
<25	Very soft
25-50	Soft
50-100	Medium
100-200	Stiff
200-400	Very stiff
>400	Hard

- Sensitivity (S_t), is defined is the ratio unconfined compressive strength of undisturbed soil sample to the unconfined compressive strength of remoulded soil sample at constant moisture content.

$$\text{Sensitivity} = \frac{\text{Unconfined compressive strength of Undisturbed soil sample}}{\text{Unconfined compressive strength of Remoulded soil sample}}$$

Soil classification on the basis of sensitivity

Sensitivity S_t	Nature of clay
1	Insensitive clays
1-2	Low-sensitive clays
2-4	Medium sensitive clays
4-8	Sensitive clays
8-16	Extra-sensitive clays
>16	Quick clays

- ❑ Cohesion of the soil sample may be calculated by using the following relations

$$\sigma_1 = \sigma_3 \, tan^2 \, \alpha + 2C \, tan. \alpha$$

Where:- σ_1 = Major principal stress at failure
σ_3 = Minor principal stress at failure
α = Failure angle with major principal plane

$$\alpha = 45 + \frac{\emptyset}{2}$$

Where:- Ø = Angle of internal friction

In unconfined compression test, σ_3 = 0
σ_1 = q_u

Hence,

$$q_u = 2C \tan\left(45 + \frac{\varnothing}{2}\right)$$

∴

$$C = \frac{q_u}{2\tan(45 + \frac{\varnothing}{2})}$$

If the soil sample is fully saturated and no drainage is allowed, then $\varnothing = 0$,

∴

$$C = \frac{q_u}{2}$$

- <u>Shear Strength</u> of the soil is estimated from <u>coulomb's equation</u> :

$$\tau f = C + \sigma_{ef} \tan \varnothing$$

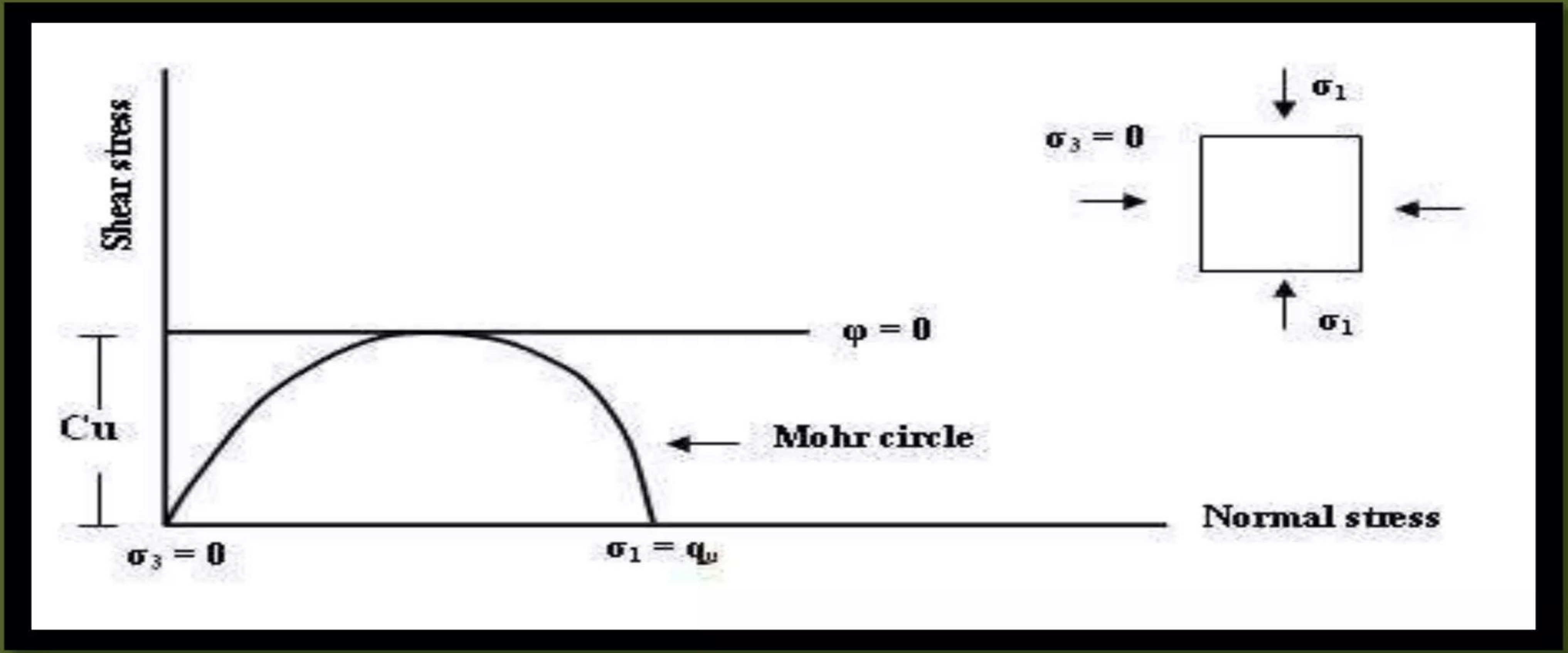
Where:- τf = Shear strength

σ_{ef} = Effective normal stress

If $\varnothing = 0$, then

$$\tau f = C$$

The Mohr circle can be draw for stress conditions at failure. As the minor principal stress is zero, the Mohr circle passes through the origin .The failure envelop is horizontal. The cohesion intercept is equal to the radius of the circle.



Mohr Circle for Unconfined Compression Test

Vane Shear Test

Week 9

Pages 208-216

Purpose

- To find the undrained shear strength of soils (e.g soft clays)

Drained and Undrained Conditions

Drained Condition

- occurs when there is no change in pore water pressure due to external loading.
- the pore water can drain out of the soil easily, causing volumetric strains in the soil.

Undrained condition

- occurs when the pore water is unable to drain out of the soil.
- the rate of loading is much quicker than the rate at which the pore water is able to drain out of the soil.
- The tendency of soil to change volume is suppressed

Apparatus

Vertical steel rod having four thin stainless blades(vanes) fixed at its bottom

(IS : 2720—1980 recommends

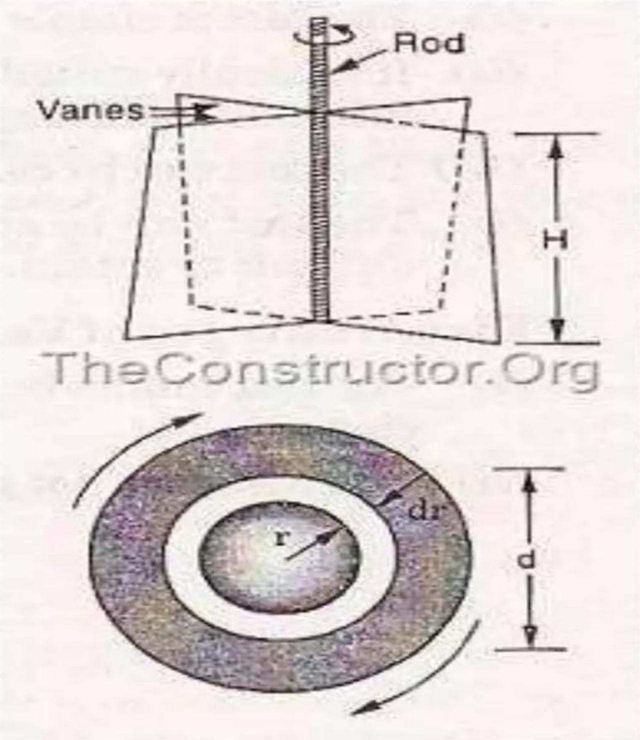
H should be 2 D

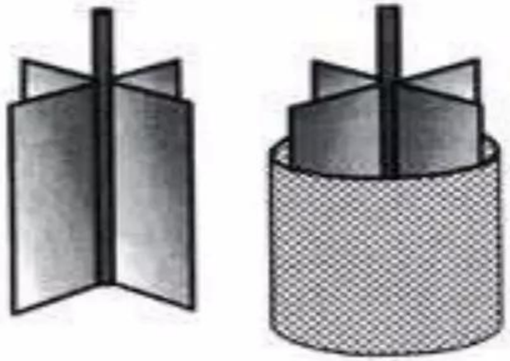
where H = height of the vane

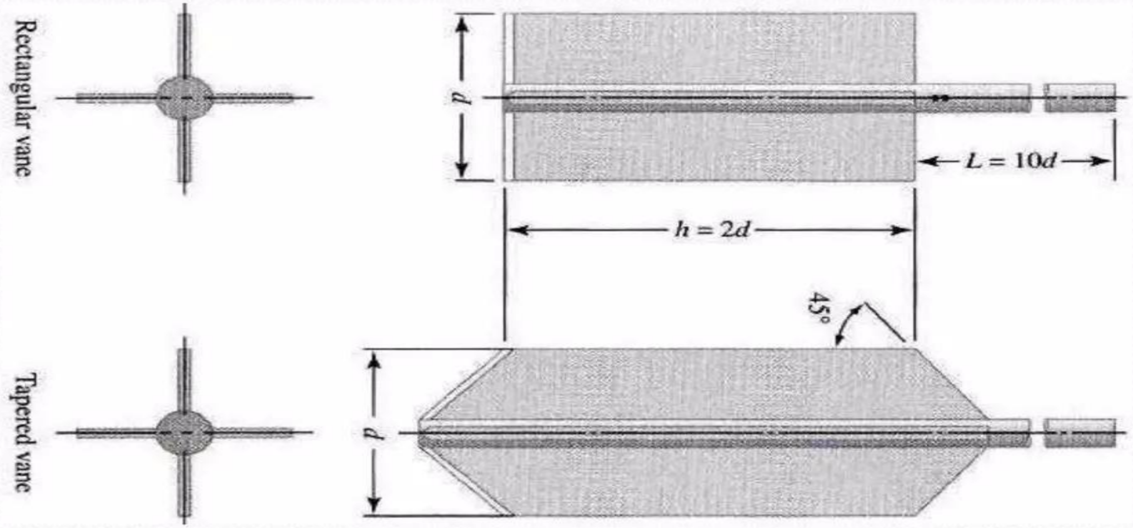
D = overall diameter)

Recommended D = 2.5 mm

length of rod = 60mm


Rod
Vanes
H
TheConstructor.Org
r
dr
d




Rectangular vane
d
L = 10d
h = 2d
45°
Tapered vane
d

Equipment

1Vane shear apparatus.

2.Specimen

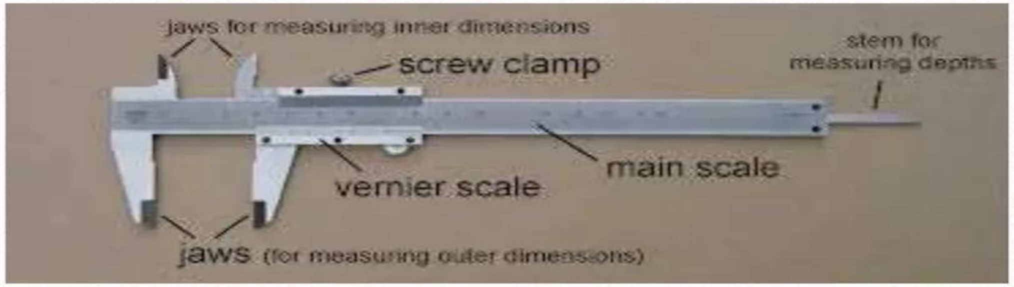
Size :D =37.5mm

H = 75 mm

(L/D ratio 2 or 3)

3.Specimen container.

4.Callipers



Experimental Procedure

- Prepare 2-3 specimens of soil sample
- Mount the specimen container with the specimen on the base of the vane shear apparatus
- Gently lower the shear vanes into the specimen to their full length without disturbing the soil specimen .The top of the vanes should be atleast *10 mm* below the top of the specimen. Note the readings of the angle of twist.
- Rotate the vanes at an uniform rate say $0.1^{o}/s$ by suitable operating the torque application handle until the specimen fails.
- Note the final reading of the angle of twist.

Calculations

- Shear strength in N/ cm2

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

Where T = Torque in N- cm

D= overall diameter of vane in cm

H = height of vane in cm

T is given by

$$\frac{\text{Spring constant}}{(180\text{x difference bet initial and final reading})}$$

Sensitivity of soils

- Sensitivity of soils can also be determine by vane shear test
- Procedure: After initial test, the vane is rotated rapidly through several revolutions such that soil becomes remoulded. The test is repeated on remoulded soil and the shear strength in moulded state is determine

$$Sensitivity(St) = \frac{(s)undisturbed}{(s)remoulded}$$

General Remarks

- This test is useful when the soil is soft and its water content is nearer to liquid limit.
- Ideally suited for determination of in-situ undrained shear strength of non- fissured fully saturated clay

Bearing Capacity (Shallow Foundation)

Week 11-12

Pages 217-249

Foundations

- It is defined as that part of the structure that connects and transmits the load from the structure to the ground soil.
- The solid ground on which the foundation rests is termed as the foundation bed.
- The foundation transmits the load of the structure and it’s self-weight to the soil such that the ultimate bearing capacity of the soil is not exceeded and the settlement is tolerable.
- They are mainly 2 types- **deep and shallow foundations**.

Every structures are provide with foundation at the base to fulfill the following objectives and purposes:

- To distribute the load of the structure over a large bearing area.
- To load the bearing surface at uniform rate so as to avoid unequal settlement.
- To prevent the lateral movement of the supporting material.
- To increase the stability of the structure as a whole

Shallow Foundation

- They are those foundations in which the depth at which the foundation is placed is less than the width of the foundation (D < B).
- They are generally termed as spread footing as they transmit the load of the super structure laterally into the ground.

Classification

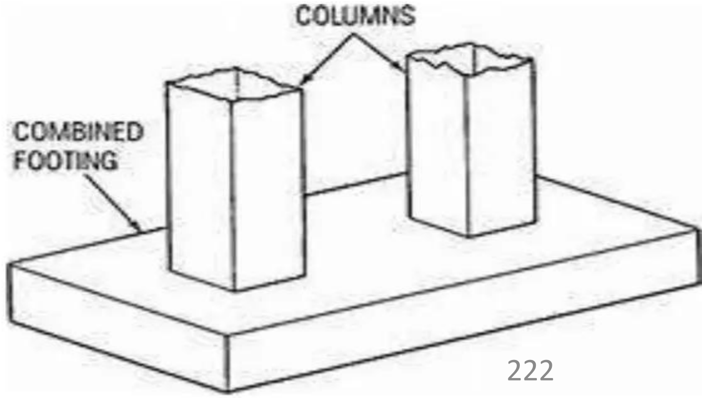
1. Isolated Footing
2. Combined Footing
3. Cantilever (Strap) Footing
4. Mat (Raft) Foundation

Individual footings

- These are used when the load of the building is carried by columns.
- Each column will have its own footing. The footing is just a square or rectangular pad of concrete on which the column sits.

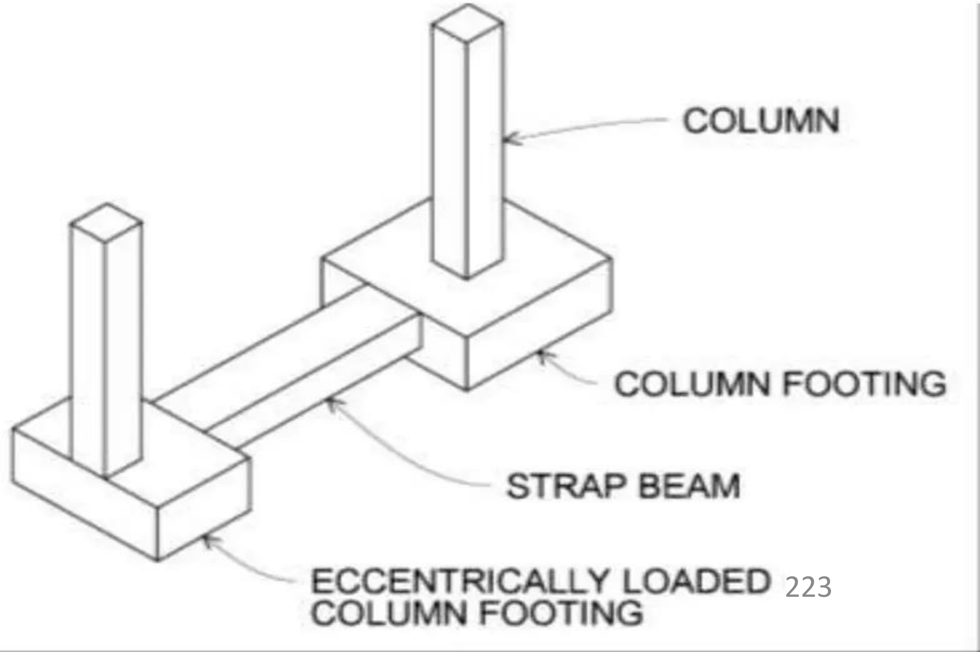
Combined Footing

- They are those foundations that are made common for two or more columns in a row. It is used when the footing for a column may extend beyond the property line.
- It is also suitable when the two columns are closely spaced and the soil on which the structure resist is of low bearing capacity.
- It may be rectangular or trapezoidal in shape.



Strap Footing

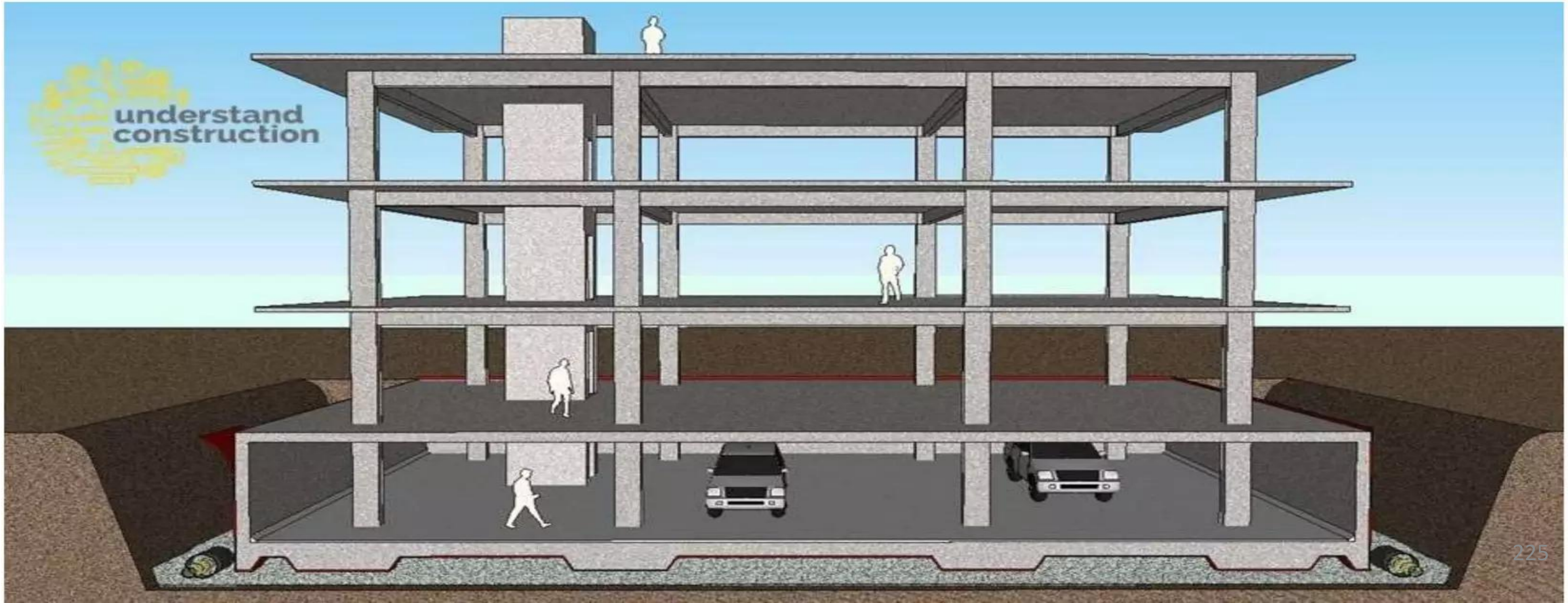
- When an edge footing cannot be extended beyond the property line the edge footing is linked up with the other interior footing by means of a strap beam. Such footings are called as strap footing.
- It is also know as **cantilever footing.**



Raft or Mat Foundations

- This type of foundation spread across the entire area of the building to support heavy structural loads.
- They are used when the loads from structure on columns and walls are very high.
- They prevent differential settlement of individual footings.
- It is suitable for expansive soils whose bearing capacity is less.
- It is economical when one-half area of the structure is covered with individual footings and wall footings are provided.

•This cannot be used where the groundwater table is above the bearing surface of the soil since it may result in liquefaction and scouring.



Deep Foundation

- This are those foundations in which the depth of the foundation is greater than its width (D>B).
- The D/B ratio is usually 4-5 for deep foundation.
- They transmits the load of the superstructure vertically to the rock strata lying deep.
- They are used when the shallow foundation cannot support the load of the structure.

Classification of Deep Foundation

1. Pile Foundation
2. Pier Foundation
3. Well (Caissons) Foundation

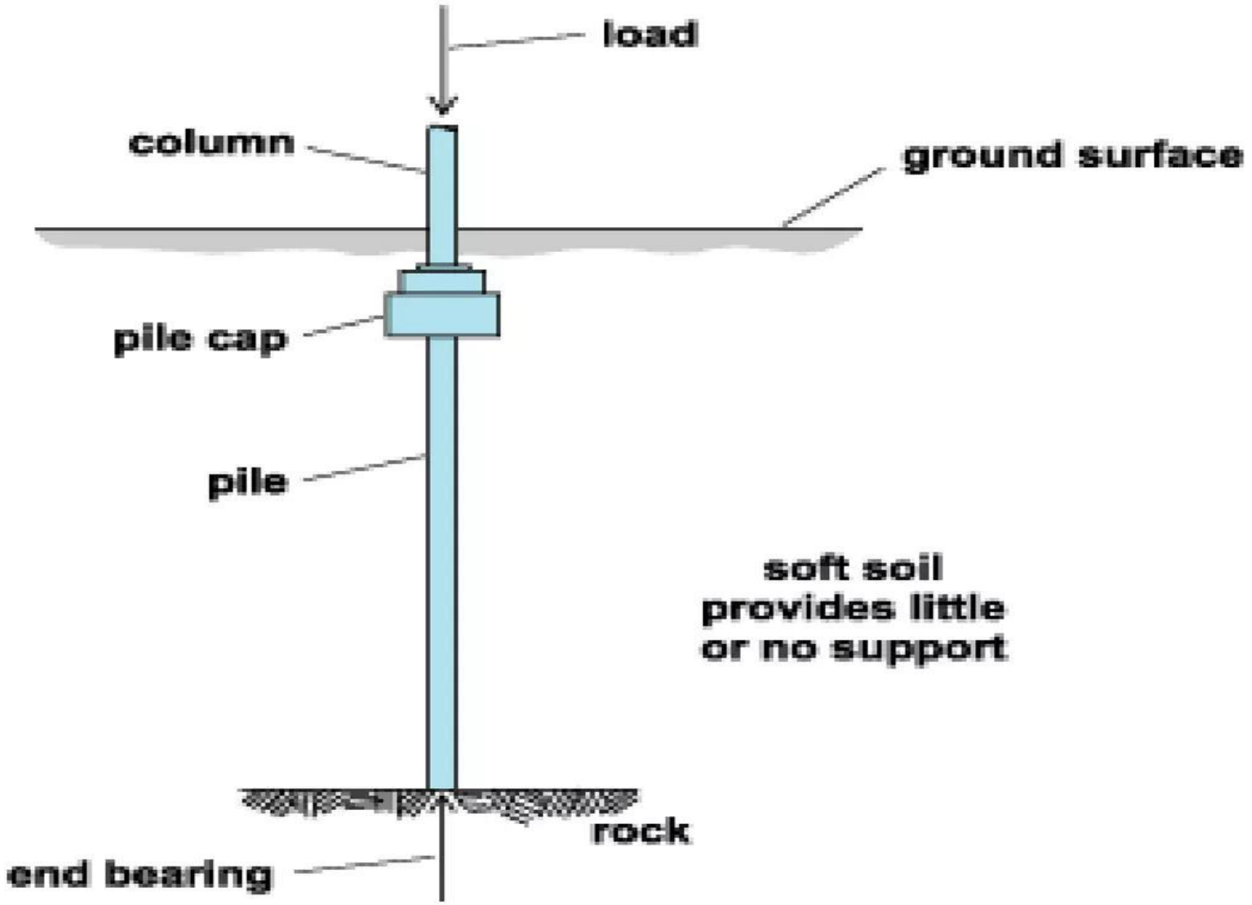
Pile Foundation

- Pile is a slender member with small area of cross-section relative to its length.
- They can transfer load either by friction or by bearing.

Pile foundation are used when:

- The load is to be transferred to stronger or less compressible stratum, preferably rock.
- The granular soils need to be compacted.
- The horizontal and the inclined forces need to be carried from the bridge abutments

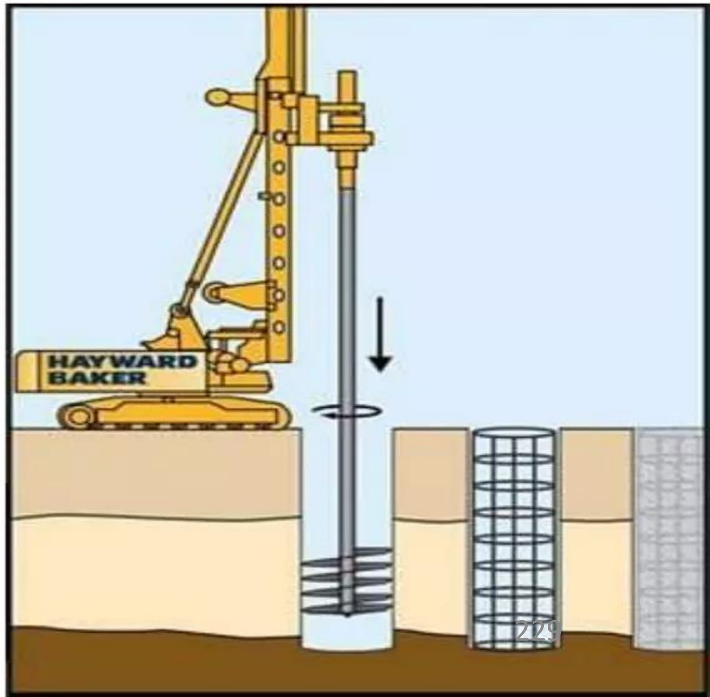
  and the retaining walls.


load
column
ground surface
pile cap
pile
soft soil
provides little
or no support
rock
end bearing

Drilled Shafts or Caisson Foundation

- It is a type of deep foundation and has action similar to pile foundations but are high capacity cast-in-situ foundations.
- It resists loads from structure through shaft resistance, toe resistance and / or combination of both of these.
- The construction of drilled shafts or

caissons are done using an auger.



Pier Foundation

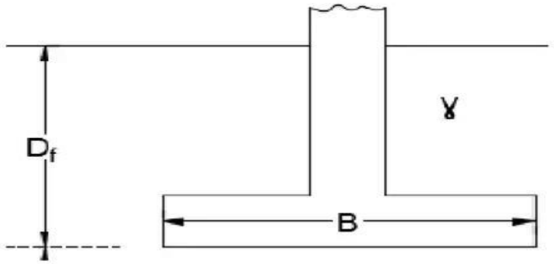
- They are underground cylindrical structural member that support heavier load of the structure which shallow foundations cannot resist.
- They can only transfer load by bearing.
- Pier foundation are shallower in depth than the pile foundation.
- Pier foundation are used when:
- The top strata is a decomposed rock underlying as sound rock strata.
- The soil is a stiff clay that occurs large resistance for driving the bearing pile.

Introduction

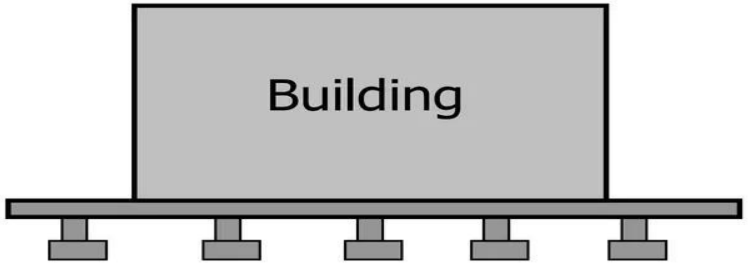
Foundations are structural elements, which are designed to transfer building loads safely to the soil. They must satisfy the following two design criteria:

1. **Bearing capacity :** There must be no shear failure within the soil.
2. **Settlement :** The settlement must be within tolerable limits.

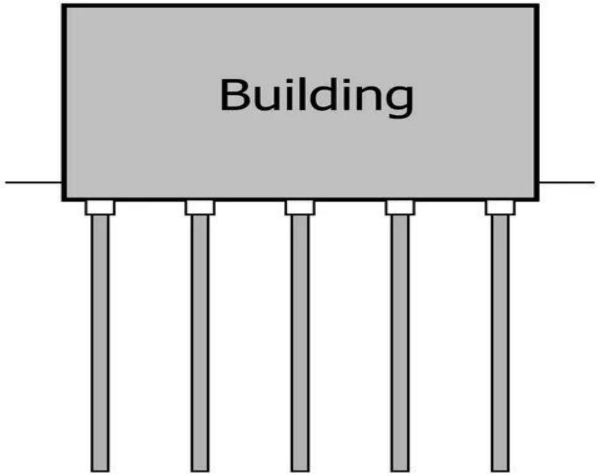
Foundations are divided in to two types based on depth of embedded to width ratio $\left(D/B\right)$ as:



1. **Shallow Foundations:** $\left(\frac{D}{B} =< 4\right)$ such as spread ,strip, continuous ,combined and raft (mat) foundation.
2. **Deep Foundations:** $\left(\frac{D}{B} > 4\right)$ such as piles and drilled shifts.



Shallow Foundations

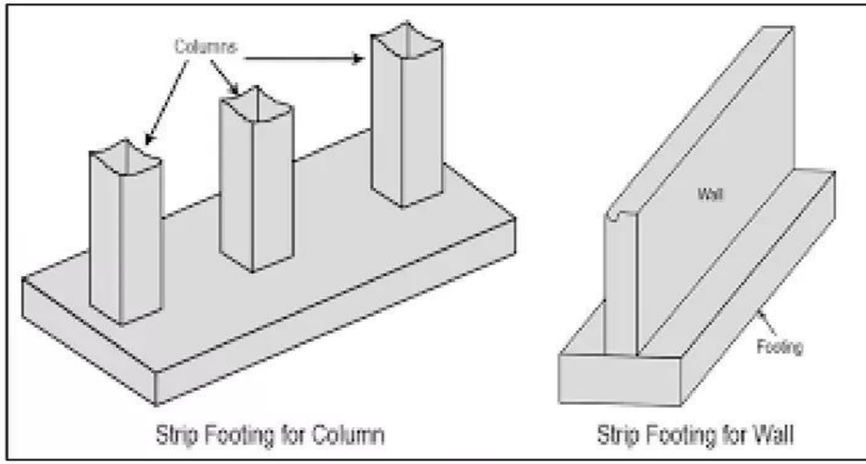


Deep Foundations

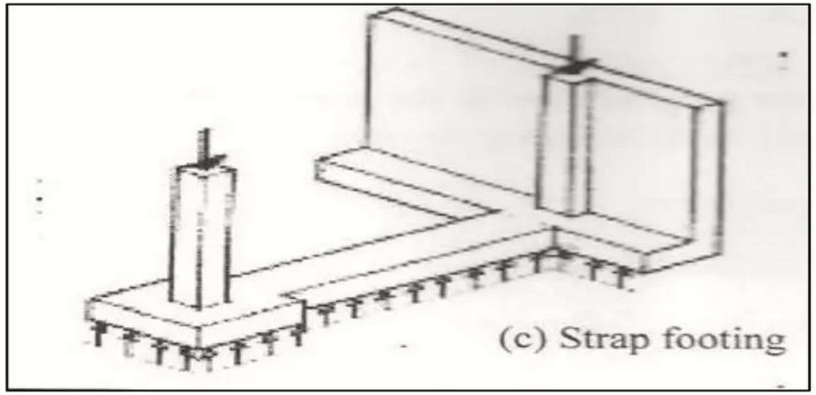
Shallow Foundations: $\left(\frac{D}{B} =< 4\right)$

Spread footing

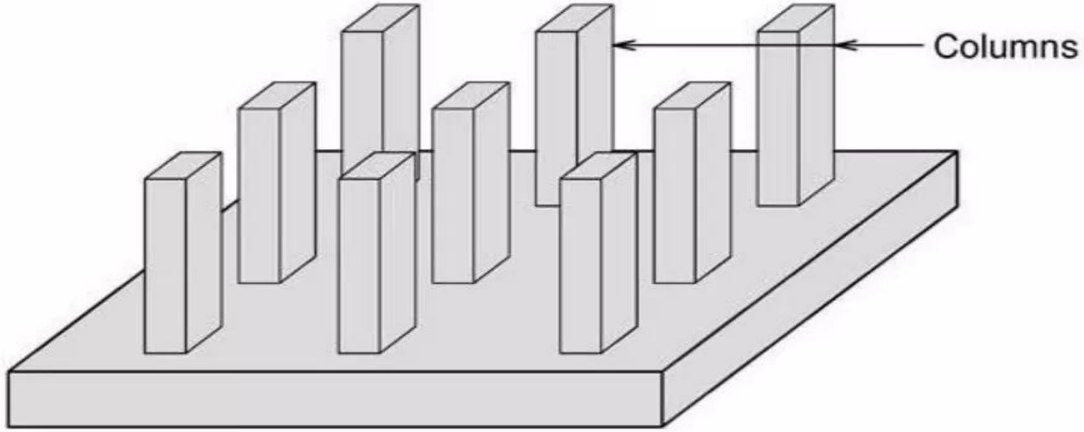
Combined footing



Strip footing



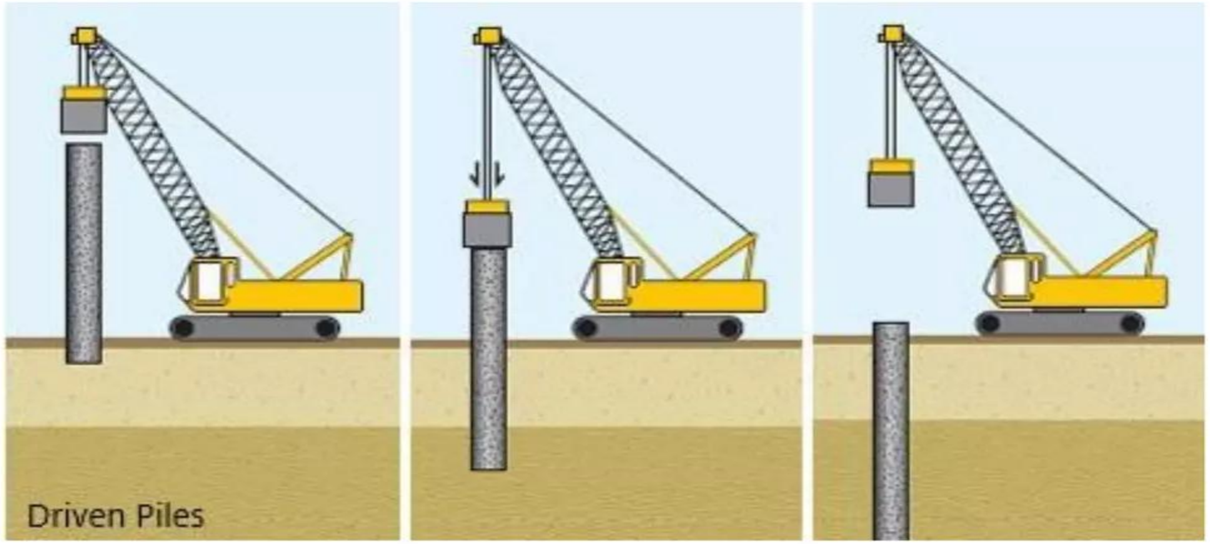
Strap footing



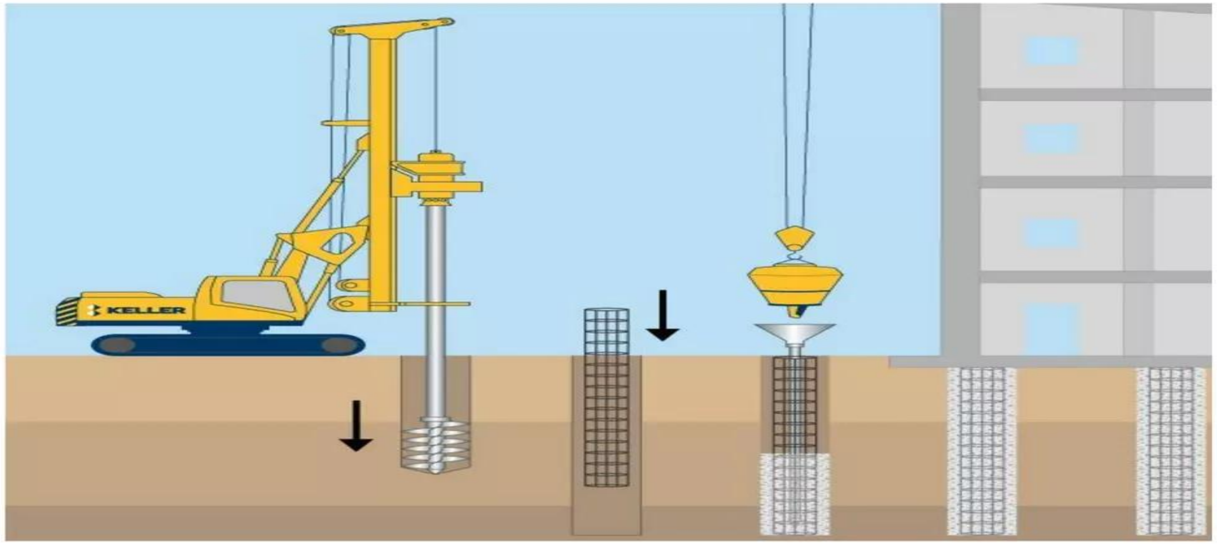
Mat (Raft) foundation

Deep Foundations: $\left(\frac{D}{B} > 4\right)$

Driven piles



Bored (drilled shafts) piles



<u>Bearing Capacity of soil :-</u>

(a) Total Overburden Pressure q_o: Is the intensity of total overburden pressure due to the weight of both soil and water at the base level of the foundation. $q_o = \gamma D_f$

(b) Effective Overburden Pressure q_o': Is the intensity of effective overburden pressure due to the weight of soil only at the base level of the foundation. $q_o' = \gamma' D_f$

(c) Ultimate Bearing Capacity (q_{ult}) : Is the maximum bearing capacity of soil at which the soil fails by shear.

(d) The Net Ultimate Bearing Capacity (q_{nu}): Is the bearing capacity in excess of the effective overburden pressure, expressed: $q_{nu} = q_{ult} - q_o'$

(e) Allowable Bearing Capacity (q_{all}) : The maximum allowable net load intensity on the soil allowing for both shear and settlement and it is simply the ultimate bearing capacity divided by the a factor of safety.

$$q_{all} = \frac{q_{ult}}{F.S}$$

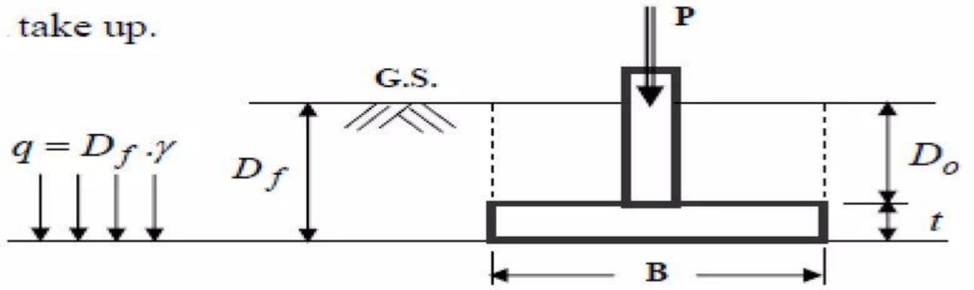
(f) Net Allowable Bearing Pressure, q_{na} is expressed as $q_{na} = \frac{q_{ult}}{F.S} = \frac{q_{ult} - q_o'}{F.S}$

(g) Safe Bearing Pressure, q_s :is defined as the net safe bearing pressure which produces a settlement of the foundation which does not exceed a permissible limit.

Note: In the design of foundations, one has to use the least of the two values of q_{na} and q_s.

(h) Gross Bearing Capacity (q_{gross}): It is the total unit pressure at the base of footing which the soil can take up.

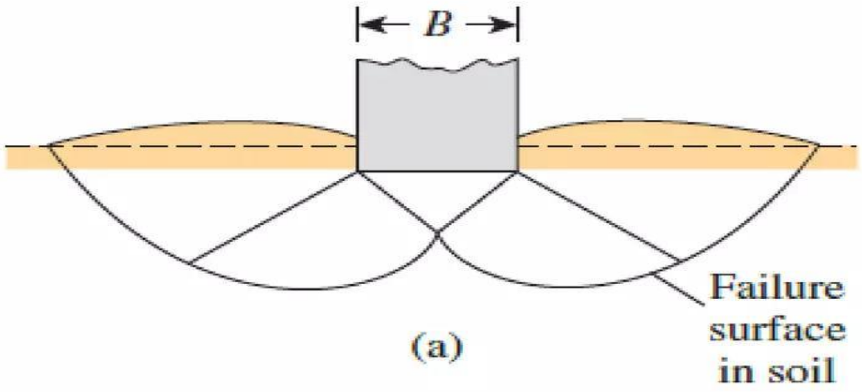
$$q_{gross} = (P + \gamma_s . D_o . B.L + \gamma_c . t.B.L)/B.L = \frac{P}{B.L} + \gamma_s . D_o + \gamma_c . t .$$



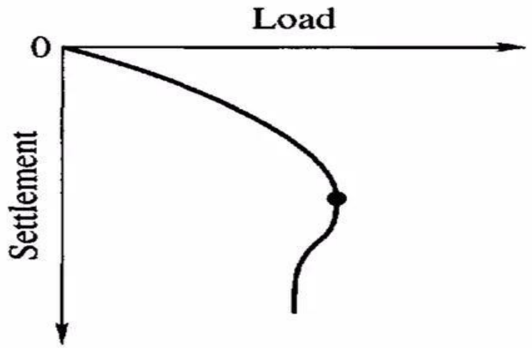
Types of Shear Failure:

Shear Failure: Also called "Bearing capacity failure" and it's occur when the shear stresses in the soil exceed the shear strength of the soil.

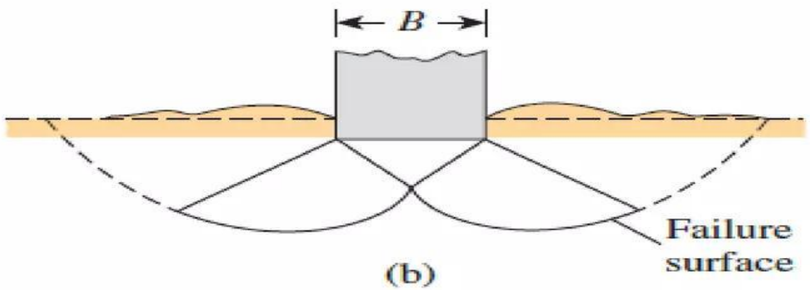
1. General Shear Failure :
Occurs over dense to very dense cohesion less soils and very stiff to hard cohesive soil.



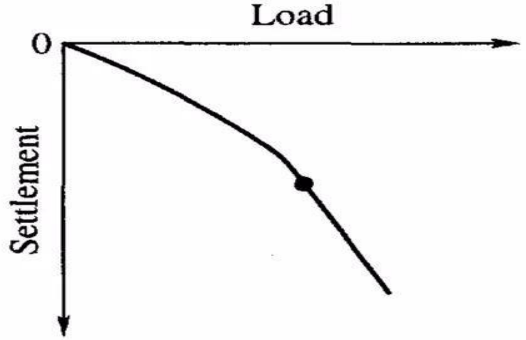
(a)



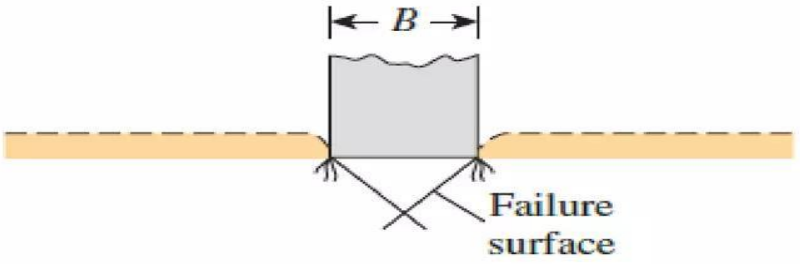
2. Local Shear Failure :
Occurs over medium to dense cohesion less soils and medium to stiff cohesive soil.



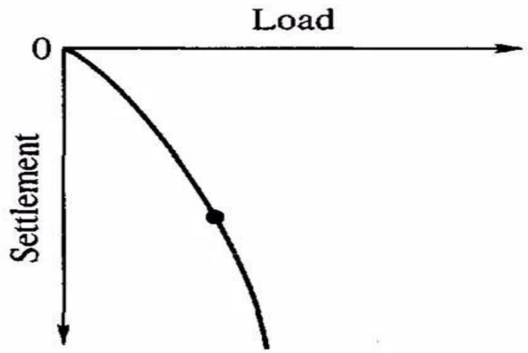
(b)



3. Punching Shear Failure :
Occurs loose to very loose cohesion less soils and soft to very soft cohesive soil.



(c)



Determination of the Ultimate Bearing Capacity of Soil:

1. Terzaghi's Bearing Capacity Theory: Terzaghi was the first to present a comprehensive theory for evaluation of the ultimate bearing capacity of rough shallow foundation. **This theory is based on the following assumptions:**

1. Soil beneath the foundation is a homogeneous , isotropic and infinite half – space.
2. Depth of embedment not more than width $(D_f \leq B)$.
3. The load is concentric and vertical.
4. Foundation has a horizontal base on a level ground surface.
5. The failure mode is General Shear Failure .
6. The foundation is strip. ($B/L > 5$).

$$q_u = c' Nc + q' Nq + 0.5 B \gamma' N\gamma$$ **Terzaghi's Bearing Capacity Equations for strip footing**

q_u =Ultimate bearing capacity of the **underlying** soil (KN/m^2)
c' =Cohesion of Soil beneath foundation (KN/m^2)
q' =**Efeective** stress at the bottom of the foundation $= \gamma' D_f$ (KN/m^2)
B: least width (or diameter) of footing.
γ' :Unit weight of the soil
Nc,Nq,Nγ=Bearing capacity factors (nondimensional)and are functions **only** of the **underlying** soil friction angle,ϕ,→→ The variations of bearing capacity factors and underlying soil friction angle are given in **(Table.1) for general shear failure.**

Table.1 Terzaghi's Bearing Capacity Factors

ϕ'	N_c	N_q	N_γ[a]	ϕ'	N_c	N_q	N_γ[a]
0	5.70	1.00	0.00	26	27.09	14.21	9.84
1	6.00	1.10	0.01	27	29.24	15.90	11.60
2	6.30	1.22	0.04	28	31.61	17.81	13.70
3	6.62	1.35	0.06	29	34.24	19.98	16.18
4	6.97	1.49	0.10	30	37.16	22.46	19.13
5	7.34	1.64	0.14	31	40.41	25.28	22.65
6	7.73	1.81	0.20	32	44.04	28.52	26.87
7	8.15	2.00	0.27	33	48.09	32.23	31.94
8	8.60	2.21	0.35	34	52.64	36.50	38.04
9	9.09	2.44	0.44	35	57.75	41.44	45.41
10	9.61	2.69	0.56	36	63.53	47.16	54.36
11	10.16	2.98	0.69	37	70.01	53.80	65.27
12	10.76	3.29	0.85	38	77.50	61.55	78.61
13	11.41	3.63	1.04	39	85.97	70.61	95.03
14	12.11	4.02	1.26	40	95.66	81.27	115.31
15	12.86	4.45	1.52	41	106.81	93.85	140.51
16	13.68	4.92	1.82	42	119.67	108.75	171.99
17	14.60	5.45	2.18	43	134.58	126.50	211.56
18	15.12	6.04	2.59	44	151.95	147.74	261.60
19	16.56	6.70	3.07	45	172.28	173.28	325.34
20	17.69	7.44	3.64	46	196.22	204.19	407.11
21	18.92	8.26	4.31	47	224.55	241.80	512.84
22	20.27	9.19	5.09	48	258.28	287.85	650.67
23	21.75	10.23	6.00	49	298.71	344.63	831.99
24	23.36	11.40	7.08	50	347.50	415.14	1072.80
25	25.13	12.72	8.34				

[a]From Kumbhojkar (1993)

Effect of the footing shape for Terzaghi equation

To estimate the ultimate bearing capacity of **square, circular and rectangular foundations**, Terzaghi equation may be respectively modified to $\mathbf{q_u = c' N_c S_c + q' N_q S_q + 0.5 B \gamma' N_\gamma S_\gamma}$

Where (S) is shape factor

Footing Shape	B/L	S_c	S_q	S_γ	**Bearing Capacity Equations**
Strip	0	1	1	1	$\mathbf{q_u = c' N_c + q' N_q + 0.5 B \gamma' N_\gamma}$
Circular	--	1.3	1	0.6	$\mathbf{q_u = 1.3 c' N_c + q' N_q + 0.3 D \gamma' N_\gamma}$
Square	1	1.3	1	0.8	$\mathbf{q_u = 1.3 c' N_c + q' N_q + 0.4 B \gamma' N_\gamma}$
Rectangular	$> 0\ and\ 1 <$	$1 + 0.3\frac{B}{L}$	1	$1 - 0.2\frac{B}{L}$	$\mathbf{q_u = (1 + 0.3\frac{B}{L}) c' N_c + q' N_q + (1 - 0.2\frac{B}{L}) 0.5 B \gamma' N_\gamma}$

Effect of the type of failure for Terzaghi equation

For **local shear failure** Terzaghi suggested using of modified cohesion and angle of internal friction ($\bar{c}$ and $\bar{\emptyset}$) as follows:

$$\bar{c} = \frac{2}{3} C \qquad\qquad tan\bar{\emptyset} = \frac{2}{3} tan\emptyset$$

Effect of Water Table on Bearing Capacity:

Terzaghi and Meyerhof equations give the ultimate bearing capacity based on the assumption that the water table is located well below the foundation. However, if the water table is close to the foundation, the bearing capacity will decreases due to the effect of water table, some modification of the bearing capacity equations (Terzaghi and Meyerhof) will be necessary.

The values which will be modified are:

1. (**q** for soil above the foundation) in the second term of equations.
2. (**γ** for the underlying soil) in the third (last) term of equations .

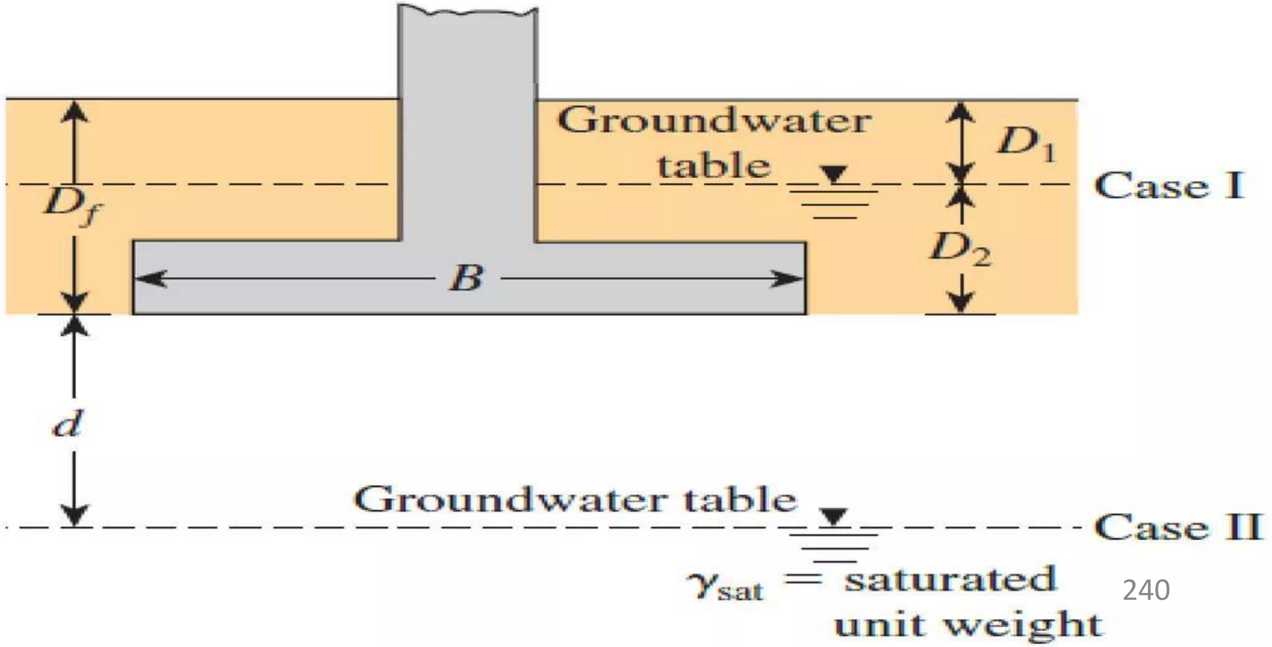
Case I. The water table is located so that $0 \le D1 \le Df$ as shown in the following figure:

- For the soil above the foundation) The factor ,**q**, (second term) $\mathbf{q = D1 \times \gamma d + D2 \times (\gamma sat - \gamma w)}$
- For the soil under the foundation) The factor ,**γ**, (third term) $\mathbf{\gamma' = \gamma sat - \gamma w}$

Case II. The water table is located so that $0 \le d < B$ as shown in the following figure:

- The factor ,q, (second term)= $\mathbf{q = Df \times \gamma d}$
- The factor ,γ, (third term)= $\bar{\gamma} = \gamma' + \frac{d}{B}(\gamma d - \gamma')$

Case III. The water table is located so that $d \ge B$, in this case the water table is assumed have no effect on the ultimate bearing capacity.



Net Ultimate Bearing Capacity and Safety Factor

The net ultimate bearing capacity q_{nu} is defined as the pressure at the base level of the foundation in excess of the effective overburden pressure $q'_o = \gamma' D_f$

The net q_{nu} for a strip footing is

$$q_{nu} = q_{ult} - q'_o = q_{ult} - \gamma' D_f = c' Nc + \gamma' D_f' (Nq-1) + 0.5B\gamma' N\gamma$$

Similar expressions can be written for square, circular, and rectangular foundations and also for local shear failure conditions.

Allowable Bearing Pressure

The gross allowable bearing pressure is $q_{all} = \frac{q_{ult}}{F.S}$

In the same way the net allowable bearing pressure q_{na} is $q_{na} = \frac{q_{ult}}{F.S} = \frac{q_{ult} - q'_o}{F.S}$

where *Fs* = factor of safety which is normally assumed as equal to 3.

Example (1): Determine the allowable bearing capacity of a strip footing shown below using Terzaghi Equations if c = 0, φ = 30° , Df = 1.0 m , B = 1.0m , γ soil = 19 kN/m^3, the water table is at ground surface, and SF=3.

Solution:- $q_u = c' N_c S_c + q' N_q S_q + 0.5 B \gamma' N_\gamma S_\gamma$

Shape factors: from table , for strip footing $S_c = S_q = S_\gamma = 1.0$

Bearing capacity factors: from table (1), for φ = 30°, Nq = 22.46,..Nγ = 19.13

$q_u = c' N_c S_c + q' N_q S_q + 0.5 B \gamma' N_\gamma S_\gamma$ =0+1x1.0 (19-10)x22.46+0.5x1(19-10)19.13x1.0 = **288.225** KN/m^2

$q_{all} = \frac{q_{ult}}{F.S} = \frac{288.225}{3} = 96.075$ KN/m^2

Example (2):

A 2.0 m wide strip foundation is placed at a depth of 1.5 m within a sandy clay, where $c' = 10$ kN/m^2, $\phi' = 26°$, and $\gamma = 19.0$ kN/m^3. Determine the maximum wall load that can be allowed on the foundation with a factor of safety of 3, assuming general shear failure. Use gross values.

Solution:- $q_u = c' N_c + q' N_q + 0.5 B \gamma' N_\gamma$

From Table 1, $N_c = 27.09$, $N_q = 14.21$, and $N_\gamma = 9.84$. Thus,

$$q_u = (10)(27.09) + (19.0 \times 1.5)(14.21) + (0.5)(19.0)(2.0)(9.84) = 862.8 \text{ kN/m}^2$$

$$q_{all} = \frac{q_u}{FS} = \frac{862.8}{3} = 287.6 \text{ kN/m}^2$$

Therefore, the maximum allowable load $Q = 287.6 \times 2 =$ **575 kN/m**.

Example (3):

A design requires placing a square foundation at 1.0 m depth to carry a column load of 1500 kN. The soil properties are: $c' = 15\ \text{kN/m}^2$, $\phi' = 24°$, and $\gamma = 18.5\ \text{kN/m}^3$. What should be the width B of the foundation?

SOLUTION

From **B.C** .Eq.

$$q_u = 1.3c'N_c + qN_q + 0.4\gamma BN_\gamma$$

From Table $N_c = 23.36$, $N_q = 11.40$, and $N_\gamma = 7.08$.

$$q_u = (1.3)(15)(23.36) + (18.5 \times 1.0)(11.40) + (0.4)(18.5)(B)(7.08)$$
$$= 52.4\,B + 666.4\ \text{kN/m}^2$$

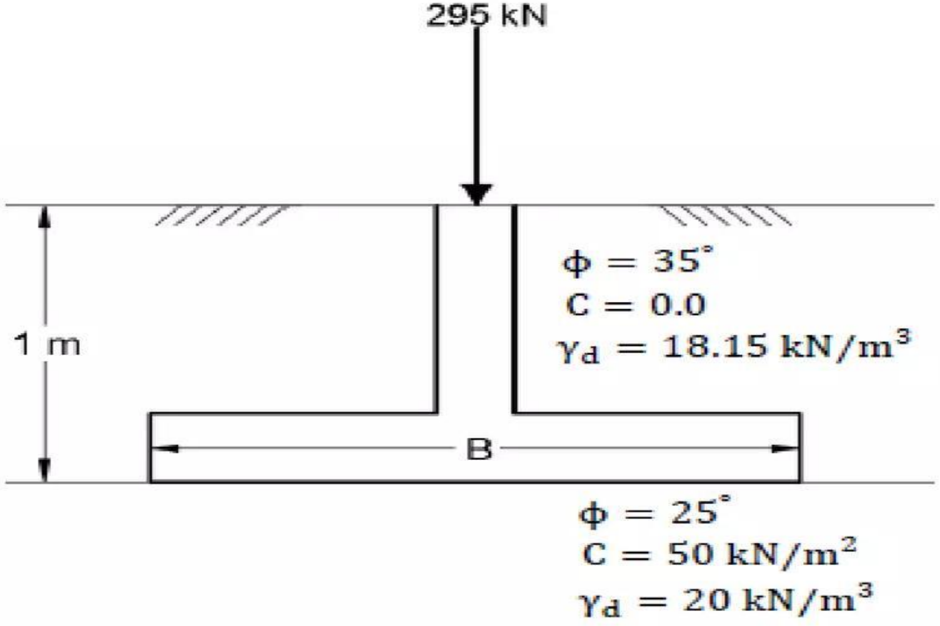
$$q_{\text{all}} = \frac{q_u}{\text{FS}} = \frac{52.4B + 666.4}{3} = 17.5B + 222.1$$

The applied pressure to the ground is $\frac{1500}{B^2}$ kN/m^2. Therefore, $\frac{1500}{B^2} = 17.5B + 222.1$.

By trial and error (or use of a graphics calculator), B = **2.4 m**. **=2.5 m**

Example (4):

Determine the size of square footing to carry net allowable load of 295 KN. FS=3. Use Terzaghi equation assuming general shear failure.



$Q_{all,net} = 295\ kN$ and we know $q_{all,net} = \frac{Q_{all,net}}{Area} \rightarrow q_{all,net} = \frac{295}{B^2}$

Also, $q_{all,net} = \frac{q_u - q}{FS}$

$q(\text{effective stress}) = \gamma \times D_f = 18.15 \times 1 = 18.15\ kN/m^2$, $FS = 3$

$$\rightarrow \frac{295}{B^2} = \frac{q_u - 18.15}{3} \rightarrow q_u = \frac{885}{B^2} + 18.15 \rightarrow\rightarrow (1)$$

$q_u = 1.3cN_c + qN_q + 0.4B\gamma N_\gamma$

$c = 50\ kN/m^2$

$q(\text{effective stress}) = 18.15\ kN/m^2$

$\gamma = 20\ kN/m^3$ (for underlying soil)

For $\phi = 25^{\circ} \rightarrow N_c = 25.13,\ N_q = 12.72,\ N_\gamma = 8.34$ (**Table 3.1**)

Substitute from all above factor in Terzaghi equation:

$q_u = 1.3 \times 50 \times 25.13 + 18.15 \times 12.72 + 0.4 \times B \times 20 \times 8.34$

$\rightarrow q_u = 1864.318 + 66.72B$

Substitute from Eq. (1):

$\frac{885}{B^2} + 18.15 = 1864.318 + 66.72B$

Multiply both side by B^2:

$66.72\ B^3 + 1846.168B^2 - 885 = 0.0$

$\rightarrow B = 0.68\ m$ ✓.

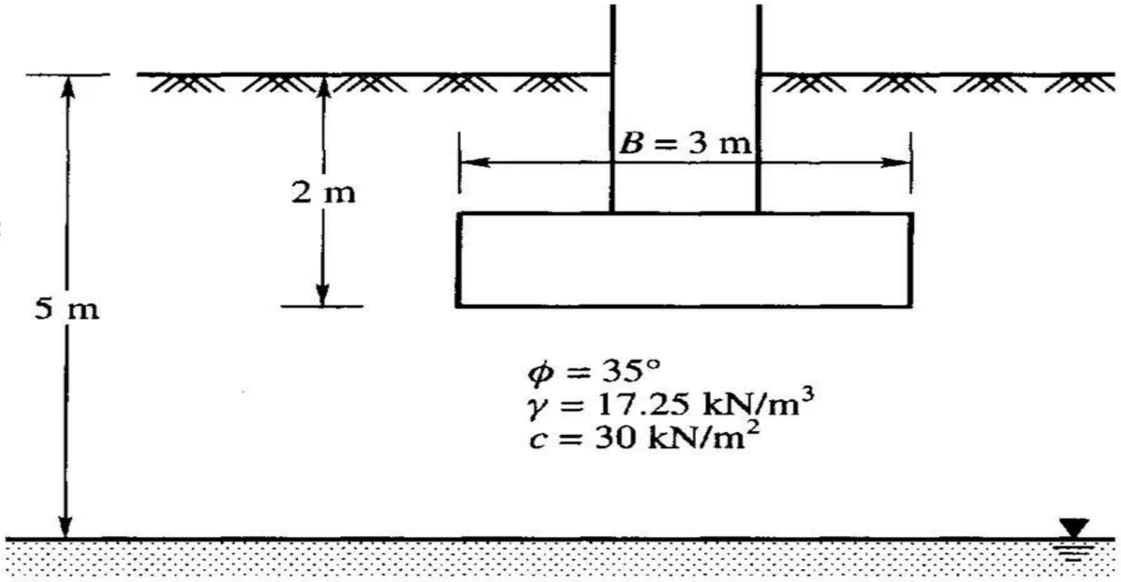
Example (5):

A strip footing of width 3 m is founded at a depth of 2 m below the ground surface in a $(c-\phi)$ soil having a cohesion $c = 30$ kN/m^2 and angle of shearing resistance $\phi = 35°$. The water table is at a depth of 5 m below ground level. The moist weight of soil above the water table is 17.25 kN/m^3. Determine

(a) the ultimate bearing capacity of the soil,

(b) the net bearing capacity,

(c) the net allowable bearing pressure and the load/m for a factor of safety of 3. Use the general shear failure



Solution:-

For $\phi = 35°$, $N_c = 57.75$, $N_q = 41.44$, and $N_\gamma = 45.41$

From B.C. Eq.

a- $q_u = cN_c + \gamma D_f N_q + \frac{1}{2}\gamma B N_\gamma$

$= 30 \times 57.75 + 17.25 \times 2 \times 41.44 + \frac{1}{2} \times 17.25 \times 3 \times 45.41 = 4337 \quad kN/m^2$

b- $q_{nu} = q_u - \gamma D_f = 4259 - 17.25 \times 2 \approx 4032.66 \quad kN/m^2$

c- $q_{na} = \frac{q_{nu}}{F_s} = \frac{4032.66}{3} \approx 1434 \quad kN/m^2$

$Q_a = q_{na} B = 1434 \times 3 = 4302 \quad kN/m$

Example 6.

If the soil in Ex. 5 fails by local shear failure, determine the net safe bearing pressure. All the other data given in Ex. 5 remain the same.

Solution

For local shear failure: $\bar{\phi} = \tan^{-1} 0.67 \tan 35^\circ = 25^\circ$

$\bar{c} = 0.67c = 0.67 \times 30 = 20 \text{ kN/m}^2$

From Table 1, for $\bar{\phi} = 25^\circ$, $\bar{N}_c = 25.13$ $\bar{N}_q = 12.72$ $\bar{N}_\gamma = 8.34$

Now from Eq. of B.C.

$$q_u = 20 \times 25.13 + 17.25 \times 2 \times 12.72 + \frac{1}{2} \times 17.25 \times 3 \times 8.34 = 1157.23 \text{ kN/m}^2$$

$$q_{nu} = 1157.23 - 17.25 \times 2 = 1122.7 \text{ kN/m}^2$$

$$q_{na} = \frac{1122.7}{3} = 374.2 \text{ kN/m}^2$$

$$Q_a = 374.2 \times 3 = 1122.7 \text{ kN/m}$$

__**H.W.1:**__

If the water table in Ex. 5 rises to the ground level, determine the net safe bearing pressure of the footing. All the other data given in Ex. 5 remain the same. Assume the saturated unit weight of the soil $\gamma_{sat} = 18.5 \text{ kN/m}^3$.

H.W.2:

If the water table in Ex. 5 occupies any of the positions (a) 1.25 m below ground level or (b) 1.25 m below the base level of the foundation, what will be the net safe bearing pressure?

Assume γ_{sat} = 19 kN/m^3, γ(above WT) = 17 kN/m^3. All the other data remain the same as given in Ex. 5

H.W.3:

A square footing fails by general shear in a cohesionless soil under an ultimate load of Q_{ult} = 7500 kN .The footing is placed at a depth of 2.m below ground level. Given ϕ = 35°, γ_d = 16 kN/m^3 and γ_{sat} = 18.5 kN/m^3 determine the size of the footing if the water table is at 3 m below the ground surfacs.

2. Skempton's Bearing Capacity:

For saturated clay soils($\varphi = 0$) , **Skempton** (1951) proposed the following equation for a strip foundation

$$q_u = c_u N_c + q$$

- c_u: Undrain cohesion
- **Nc:** Bearing capacity factor for strip and square (or circular) foundations as a function of the D/B ratio are given in Fig. below.
- q :Total stress (In all cases, whether the soil is dry or saturated) ($q = \gamma_t D_f$).

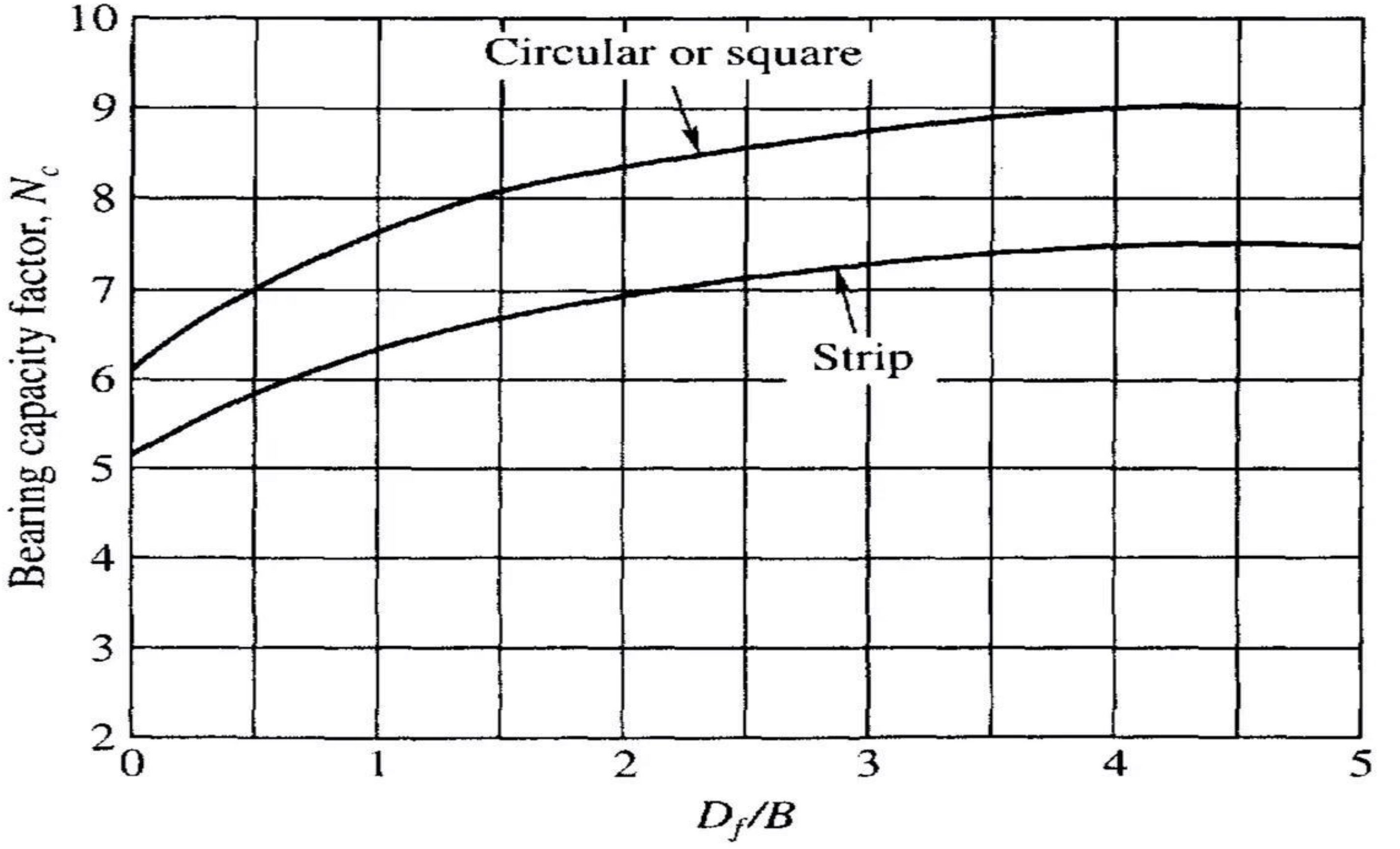


Fig. Skempton's bearing capacity factor *Nc* for clay soils

- ***For Rectangular Footing***

$$N_{C\,(Rectangular)} = \left[0.84 + 0.16\frac{B}{L}\right] * N_{C(square)}$$

Example 7.

A rectangular footing of (6×15) m is founded at a depth of 4.5 m in a cohesive soil having the following properties: $\gamma = 18\ kN/m^3$, $c_u = 40\ kN/m^2$

Determine the allowable bearing capacity if FS is 2.5.

Solution: $q_u = c_u Nc + q$, $N_{C(square)}\left(\frac{Df}{B} = \frac{4.5}{6} = 0.75\right) = 7.3\ from\ fig.$

$$N_{C\ (Rectangular)} = \left[0.84 + 0.16\frac{6}{15}\right]7.3 = 6.5992$$

$$q_u = c_u Nc + q = 40*6.5992 + 18*4.5 = 344.968\ kpa$$

$$q_a = \frac{q_u}{F.S} = \frac{344.968}{2.5} = 137.9872\ KPa$$

H.W.4:

A (1.25×1.25) m footing is located 1.25 m below the ground surface of saturated clay. It is subjected to a total load of 500 kN. Find the length of a rectangular footing of width 2 m placed at a depth of 3 m to carry a total load of 200 Ton in the same soil. Use $\gamma = 20\ kN/m^3$ and FS = 3 for both cases.

Bearing Capacity (Deep Foundation)

Week 12-13

Pages 251-271

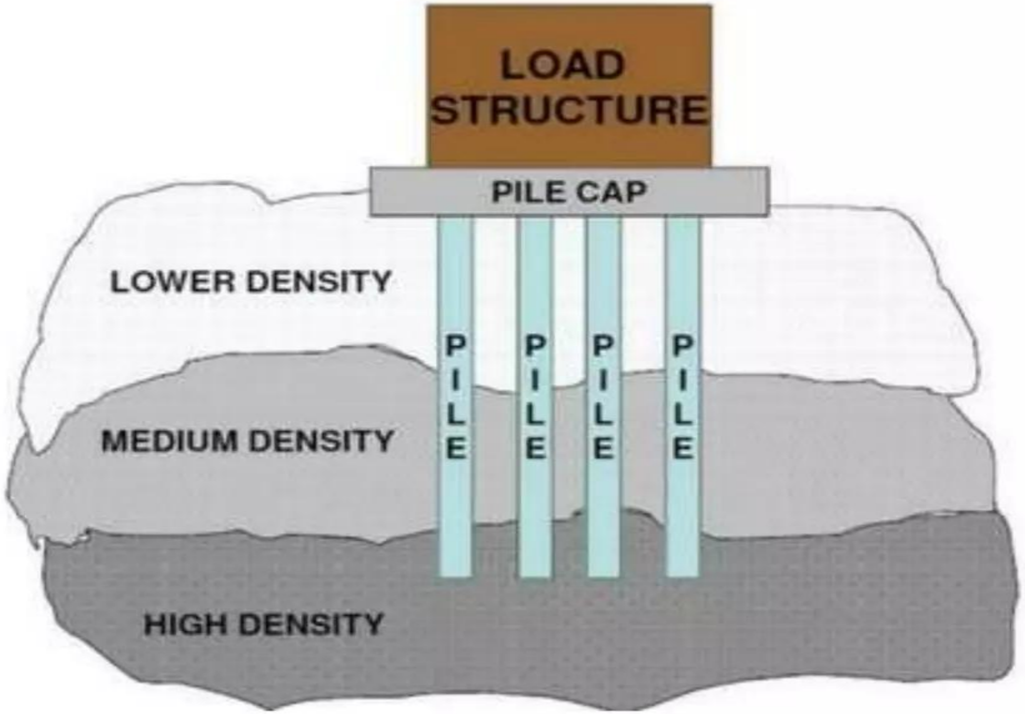
Contains

- Objective
- Type Of Foundation
- Pile Foundation
- Load Carrying Capacity
- Static Method
- Allowable Load
- Negative Skin Friction
- Advantage
- Disadvantage

Objective

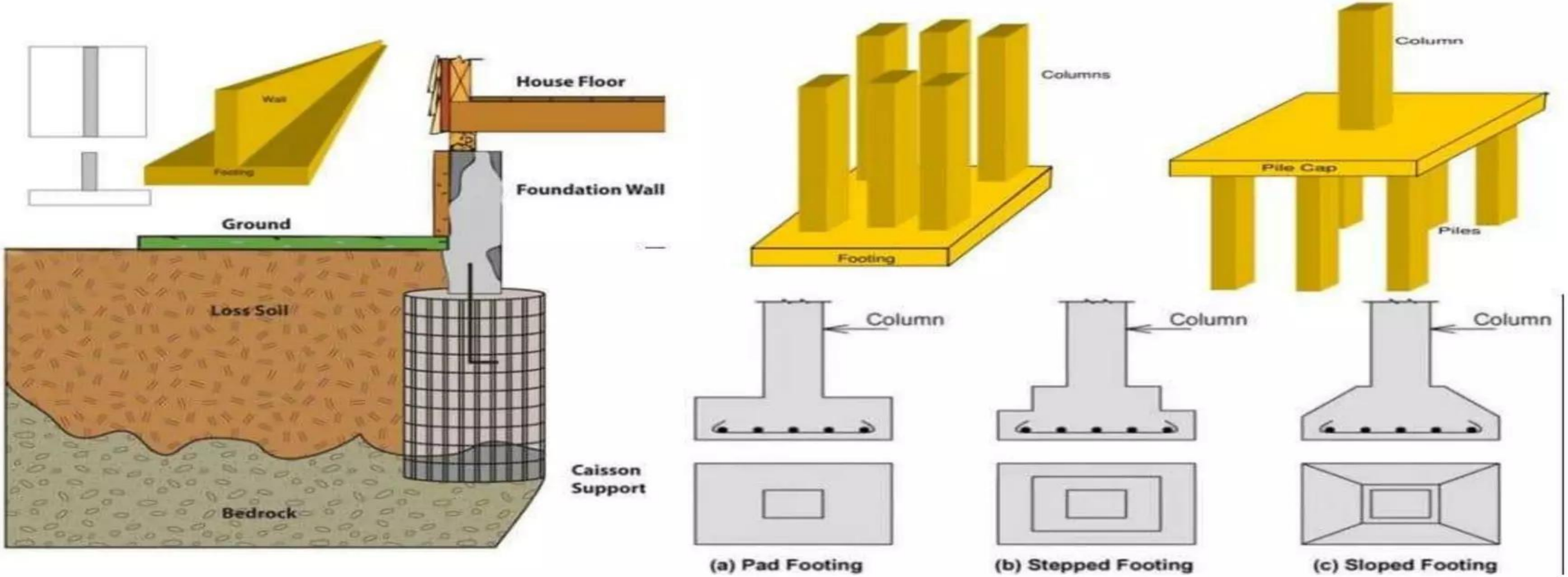
1) To understand static method to determine ultimate pile capacity

2) Evaluate the ultimate load carrying capacity of a pile

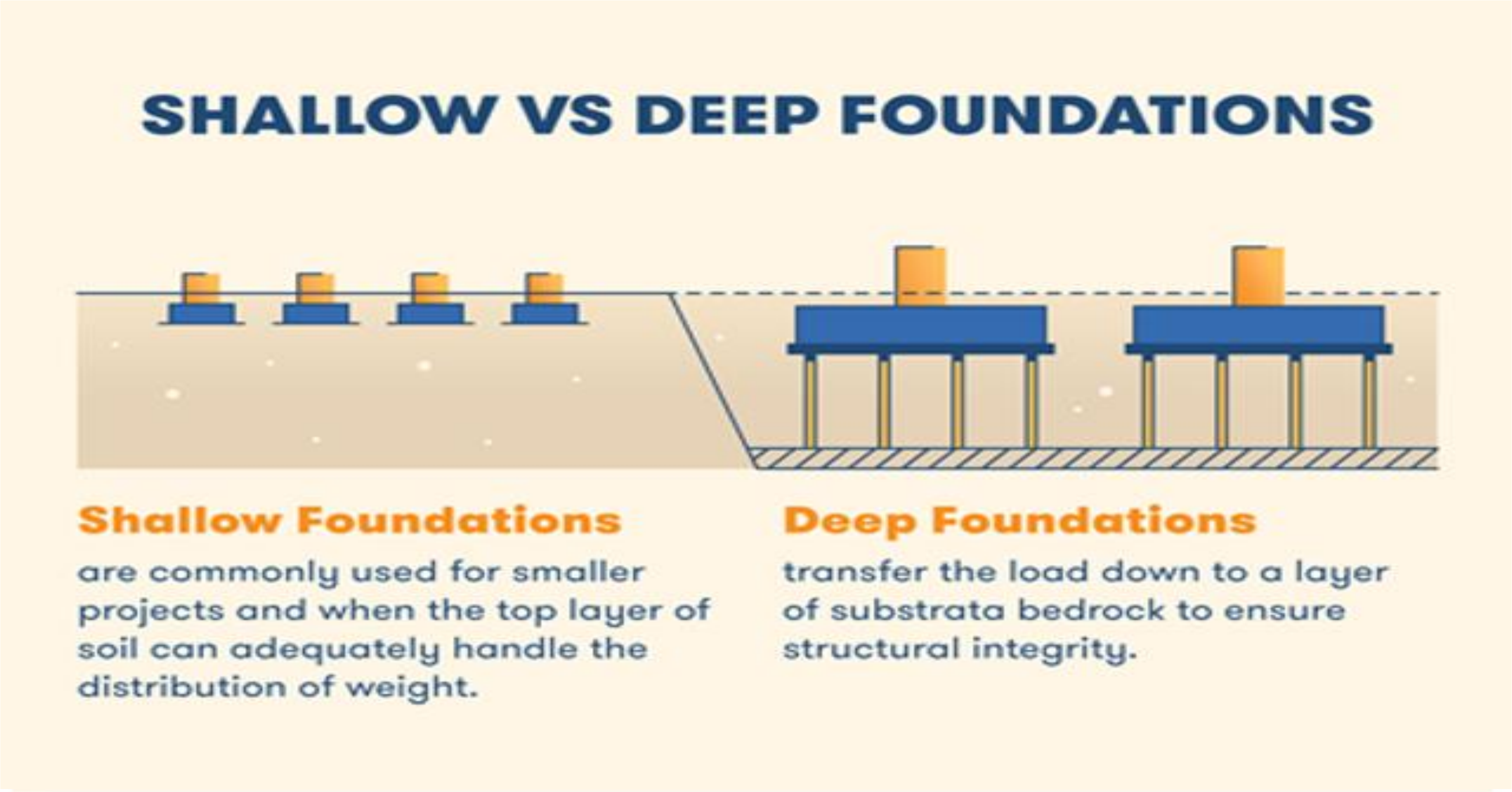


Foundation

Foundations provide support to the structure, transfers the loads from the structure to the soil.

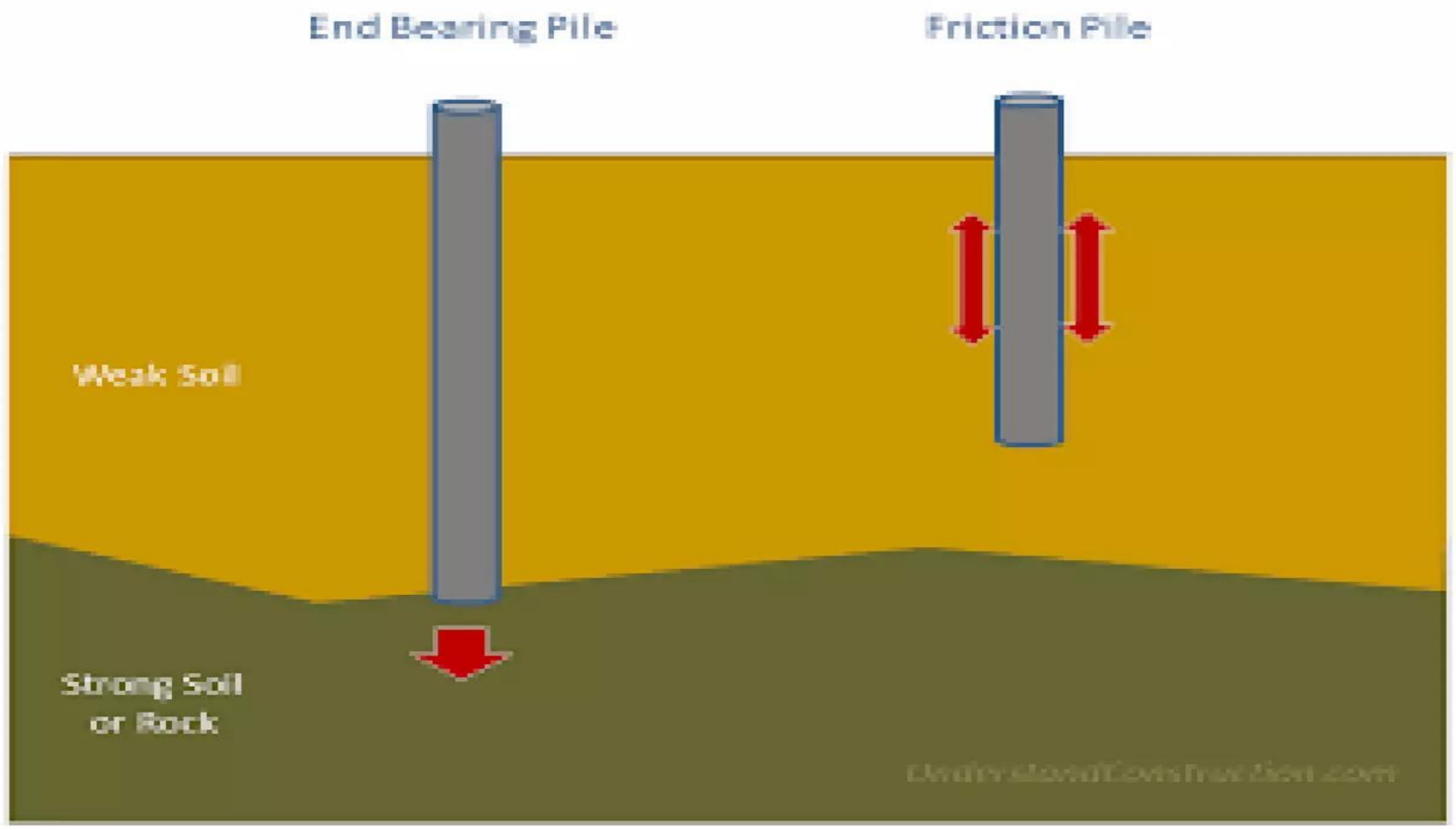


Type of foundation



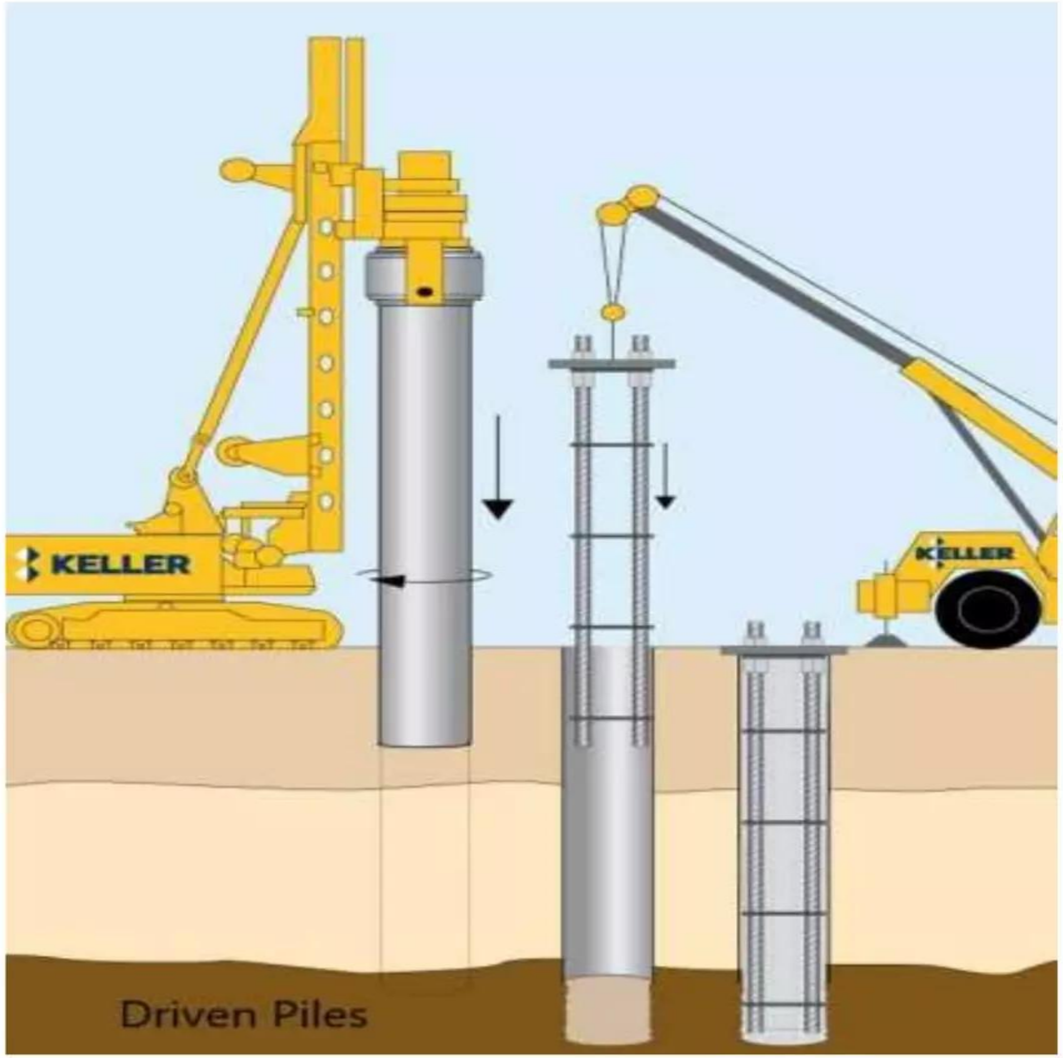
Pile foundation

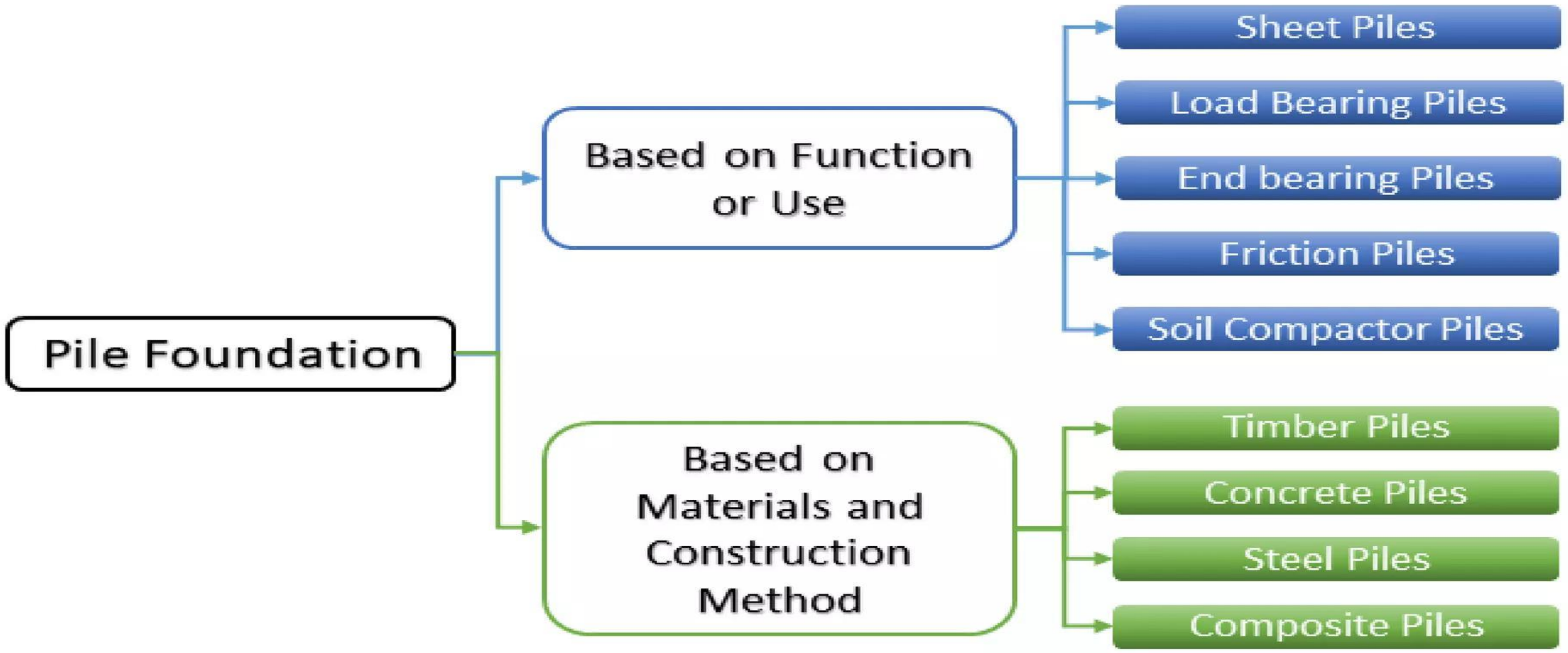
Pile foundation, a kind of deep foundation, is actually a slender column or long cylinder which transfer the load at desired depth either by end bearing or skin friction.



When to Use Pile Foundation

1. When the groundwater table is high.
2. When the soil at shallow depth is compressible.
3. When there is the possibility of scouring, due to its location near the river bed or seashore, etc.
4. When there is a canal or deep drainage systems near the structure.
5. When soil excavation is not possible up to the desired depth due to poor soil condition.
6. When it becomes impossible to keep the foundation trenches dry by pumping or by any other measure due to heavy inflow of seepage.




Pile Foundation
Based on Function or Use
Sheet Piles
Load Bearing Piles
End bearing Piles
Friction Piles
Soil Compactor Piles
Based on Materials and Construction Method
Timber Piles
Concrete Piles
Steel Piles
Composite Piles

Load carrying capacity

Pile load carrying capacity depends on the various factors including

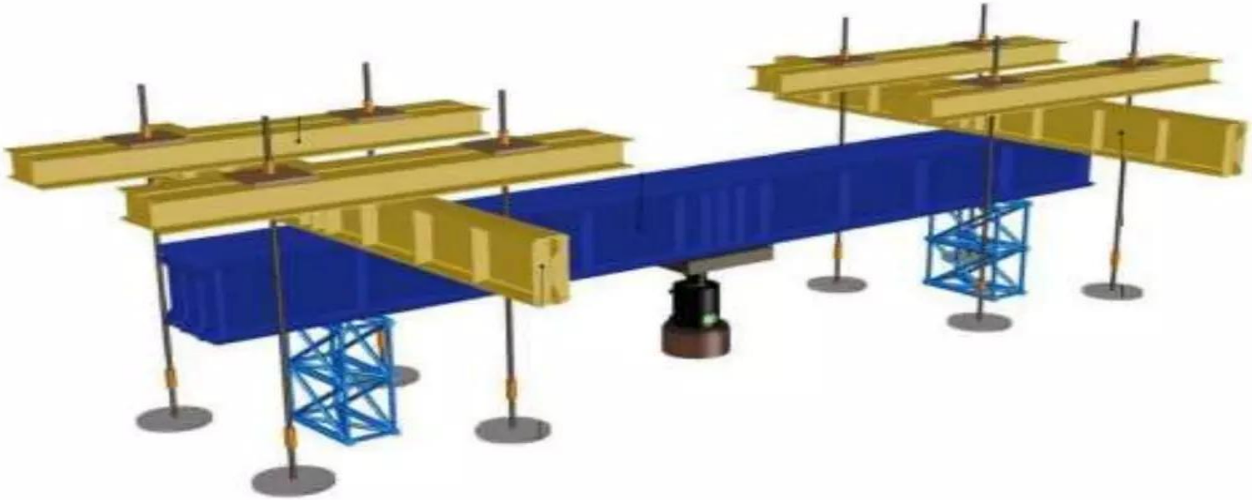
1) Pile characteristics such as pile length , cross section and shape
2) Soil configuration , short & long-term soil properties
3) Pile installation method

Load carrying capacity

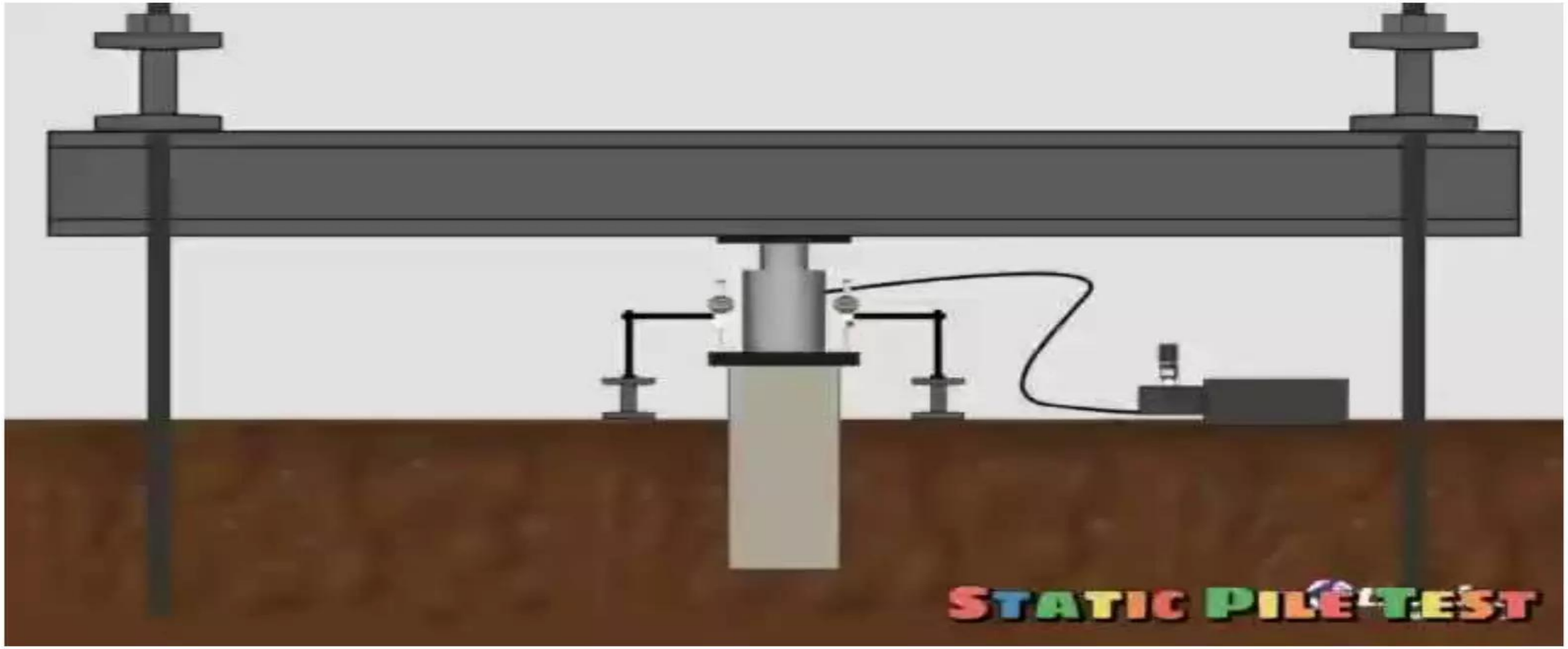
1) Static method
2) Dynamic formulas
3) In-situ penetration test
4) Pile load test

What is static method?

Static capacity is the sum of the soil/rock resistances along the pile shaft and at the pile toe. Static analyses are performed to determine ultimate pile capacity and the pile group response to applied loads.

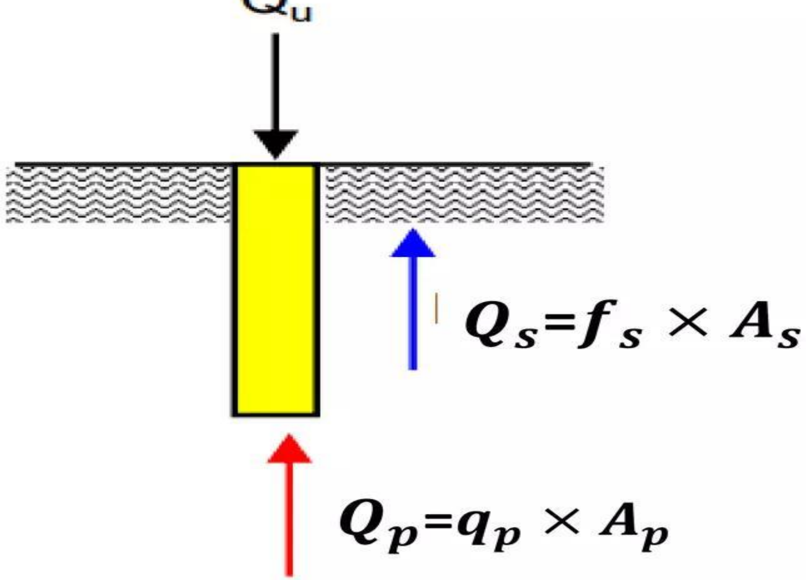


Steps of static method



The ultimate load bearing capacity

$$Q_u = Q_p + Q_s$$



When $f_s = 0$

$$Q_u = Q_p$$

When hard layer is not found .

$$Q_u = Q_s$$

Here , $\boldsymbol{q_p}$= ultimate bearing capacity of the soil at the pile tip

$\boldsymbol{A_p}$= area of the pile tip

$\boldsymbol{f_s}$=average unit skin friction

$\boldsymbol{A_s}$=effective surface area of the pile in contact with the soil

Static formula for cohesive soil

- For the pile in cohesion soil , point bearing is generally neglected for individual pile

Action , since it is negligible as compared to frictional resistance.
When hard layer is not found .

$$Q_u = Q_s$$

Methods of determination of Q_s

$$f_s = K\bar{\sigma}_v \tan\delta$$

K = earth pressure cofficient

$\bar{\sigma}_v$= effective vertical pressure at that depth

$\tan\delta$=cofficient of friction between sand and pile material

Tomilson (1975) gave the value of δ & K

Pile Material	δ	K (loose sand)	K (dense sand)
Steel	20°	0.50	1.0
Concrete	0.75φ	1.0	2.0
Timber	0.67φ	1.5	4.0

Methods of determination of Q_s

As stated earlier , the effective vertical pressure ($\overline{\sigma}_v$) increases with the depth only upto the critical depth. Below the critical depth, the value of $\overline{\sigma}_v$ remains constant.

So

$$Q_s = \sum_{i=1}^{n} K(\overline{\sigma}_v)_i \tan\delta (A_s)_i$$

Where , n= number of layers in which the pile is installed

$(\overline{\sigma}_v)_i$=effective normal stress in ith layer

$(A_s)_i$= surface area of the pile in ith layer

Now assuming linear variation of σ_v :

$$Q_s = \sum_{i=1}^{n} k \tan\delta (\text{area of } \overline{\sigma}_v \text{ diagram}) \times \text{pile perimeter}$$

Static formula for non- cohesive soil

- Most of the static analysis method in cohesionless soils use the soil friction angle

Determined from laboratory tests or SPT-N values.

When $f_s = 0$

$$Q_u = Q_p$$

Methods for determination of Q_p :

$$q_p = \bar{q}N_q + 0.4\gamma B N_\gamma$$

Where $\bar{q}$= effective vertical pressure at the pile tip

B=pile tip width (or diameter)

γ= unit weight of the soil

N_q& N_γ= bearing capacity factor for deep foundation

So $$\boldsymbol{Q_p = q_p \times A_p}$$

Allowable Load

The allowable load (Q_{all}) is obtained from the ultimate load (Q_u) from the relation

$$Q_{all} = {}^{Q_u}/_{FS}$$

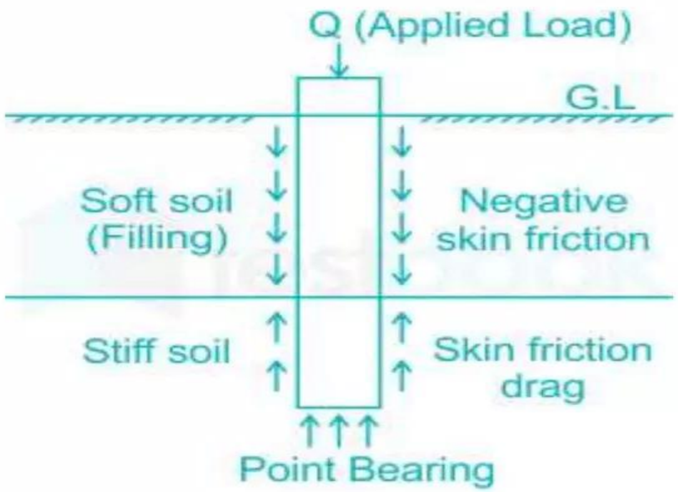
Where FS is the factor of safety. FS generally varies between 2.5 and 4, depending upon the uncertainties involved in the computation of the ultimate load.

Negative skin Friction

Negative skin fraction develops when a soft or loose soil surrounding the pile settles after the pile has been installed. The negative skin friction occurs in the soil zone which moves downward relative to the pile.

Then the net ultimate load carrying capacity of the pile is given by the equation:

$$\acute{Q}_u = Q_u - Q_{nsf}$$



Advantages:

- It is the slower, more precise process.
- static load tests are considered to provide the most accurate results when measuring pile bearing capacities and settlement or uplift of the pile after driving.
- The result can be used to confidently verify calculations used in the design of the project's deep foundations and to refine design parameters and assumptions.
- it can be carried out in all soil conditions and on all pile types and if necessary, tension and lateral testing is also possible.

Disadvantages

- The static method cannot be used when the soil test report has to be submitted within a short period of time.

- This method is very costly

Conclusion

Piles are often used because adequate bearing capacity cannot be found at shallow enough depths to support the structural loads. It is important to understand that piles get support from both end bearing and skin friction.

Pile Load Test

Week 14

Pages 272-290

OBJECTIVE

- This test is used to determine the "LOAD CARRYING CAPACITY" of a pile.
- This test gives us the value of

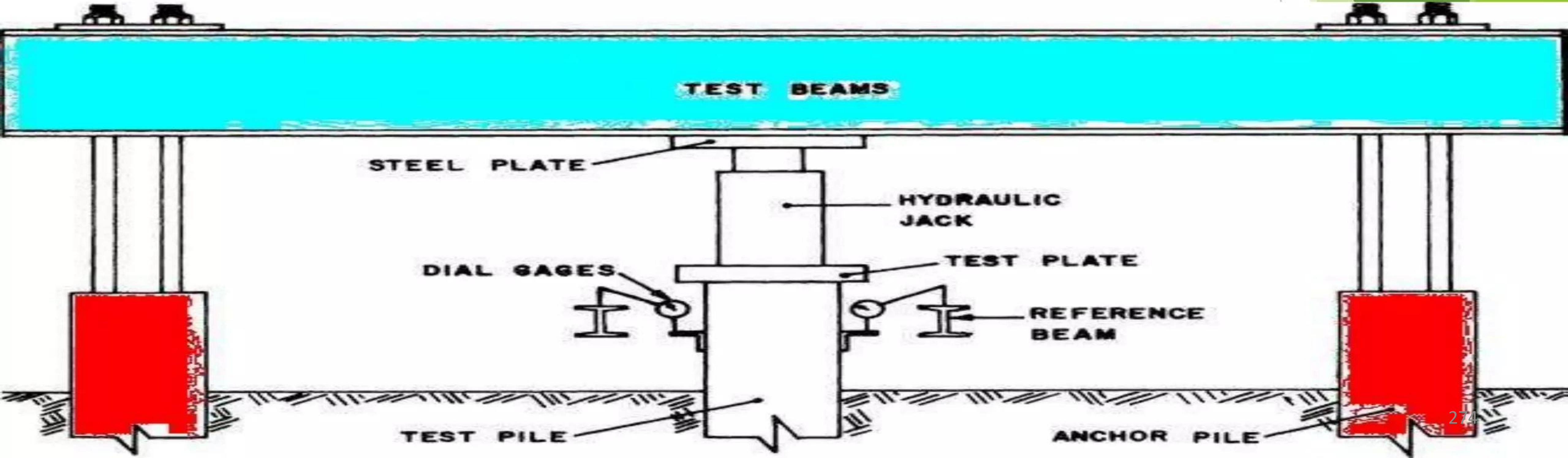
  -ULTIMATE LOAD VALUE

  -SAFE LOAD VALUE

  -SETTLEMENT UNDER DIFFERENT VALUES OF LOADS

EQUIPMENTS

- Anchor Girder or Reaction Girders
- Hydraulic Jack
- Test Pile
- Anchor Pile
- Dial Gauges
- Reaction Truss (in case of truss loading)

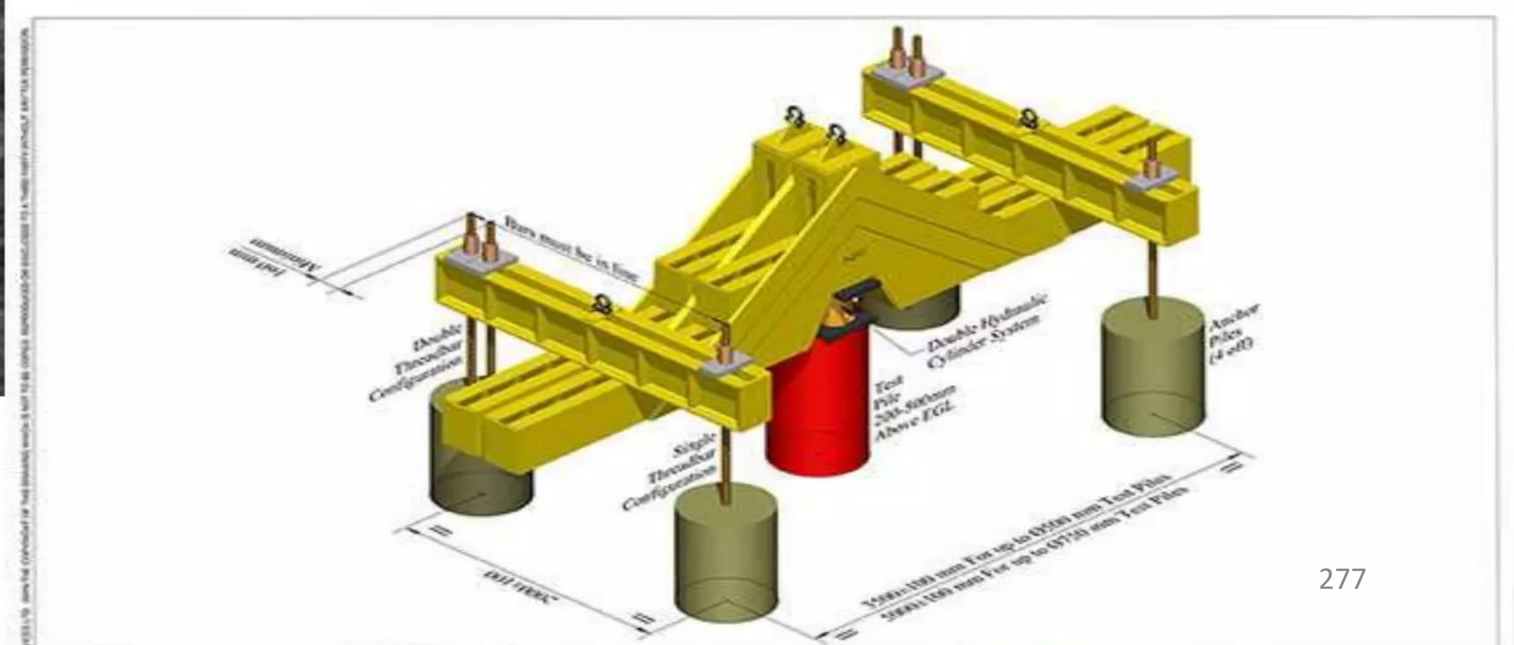


IS 2911:Part 4(2013)

PROCEDURE:-

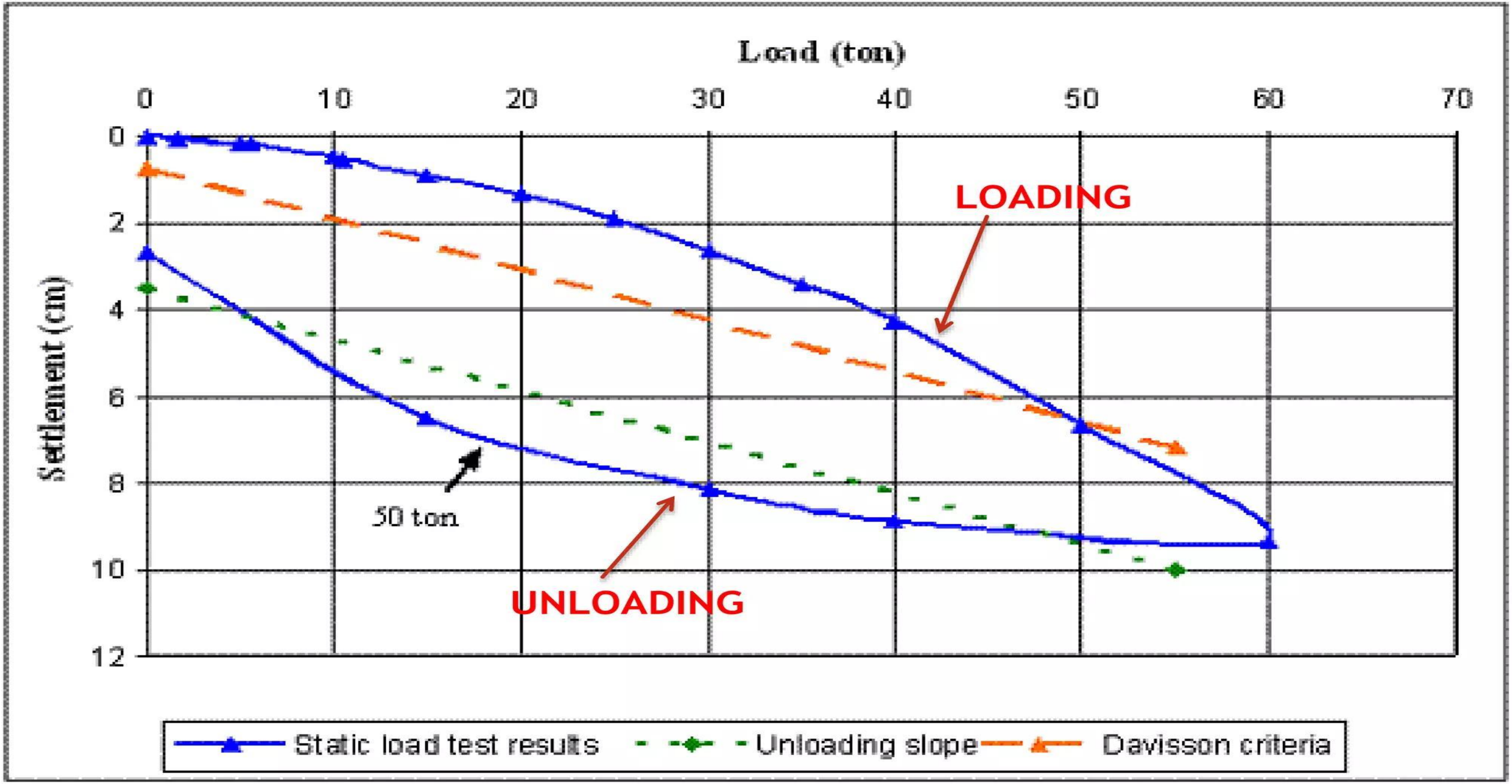
- The set-up consists of two anchor piles provided with an anchor girder or reaction girder at their top.
- The test pile is installed between the anchor piles as like foundation pile is installed. The test pile should be at lest **3B or 2.5m** clear from the anchor pile.
- The test is conducted after a rest period of 3 days after the installation in sandy soils and period of one month in silts and soft clays.
- The load is applied through a hydraulic jack resting on the reaction girder or Truss. The measurement of pile movement are taken with respect to a fixed reference mark.
- The load is applied in equal increment of about **20%** of the allowable load.

By Reaction Girder or Anchor Girder method



By Truss Loading Method

- Settlement should be recorded with **3 dial gauges**.
- Each stage of the loading is maintained till the rate of movement of the pile top is not more than **0.1mm per hour** in sandy soils and **0.02mm per hour** in case of clayey soils as maximum of two hours.
- Under each load increment, settlements are observed at **0.5, 1, 2, 4, 8, 12, 16, 20, 60 minutes**.
- The loading should be continued up to twice the safe load or the load at which the total settlement reaches a specified value.
- The load is removed in the same decrements at **1 hour** interval & the final rebound recorded **24 hours** after the entire load has been removed.
- Plot a graph of **Load→Settlement** and make a curve for loading as well as unloading obtained from a pile load test.



1 cm = 0.39 in.
1 ton (metric) = 2.2 kips

Calculations

- Figure shows a typical Load-Settlement curve for loading as well as unloading from a pile load test.
- For any given load, the **net pile settlement (S_n)** is given by,

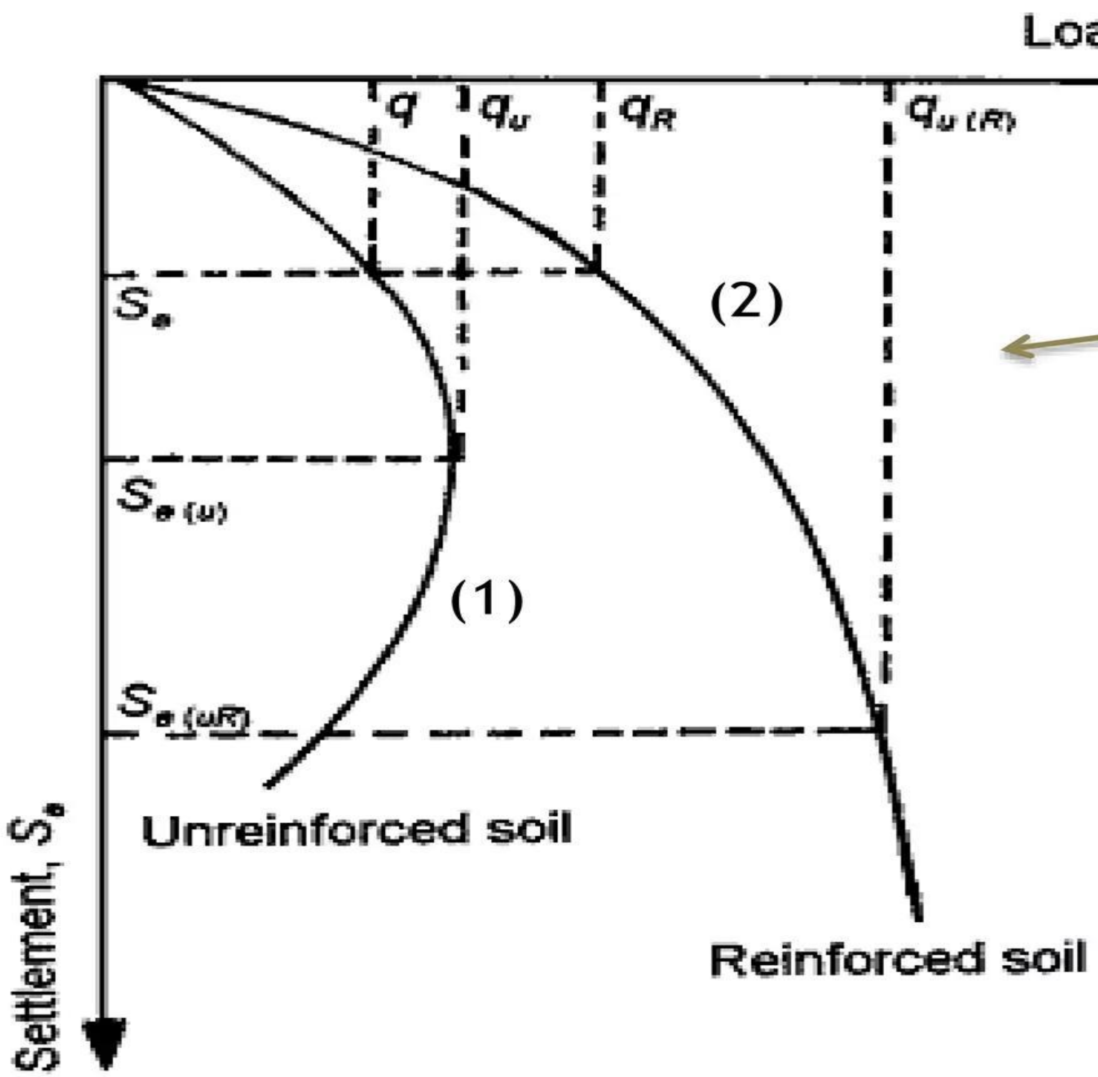
$$S_n = S_t - S_e$$

- Where ,

  S_t = Total settlement (gross settlement)

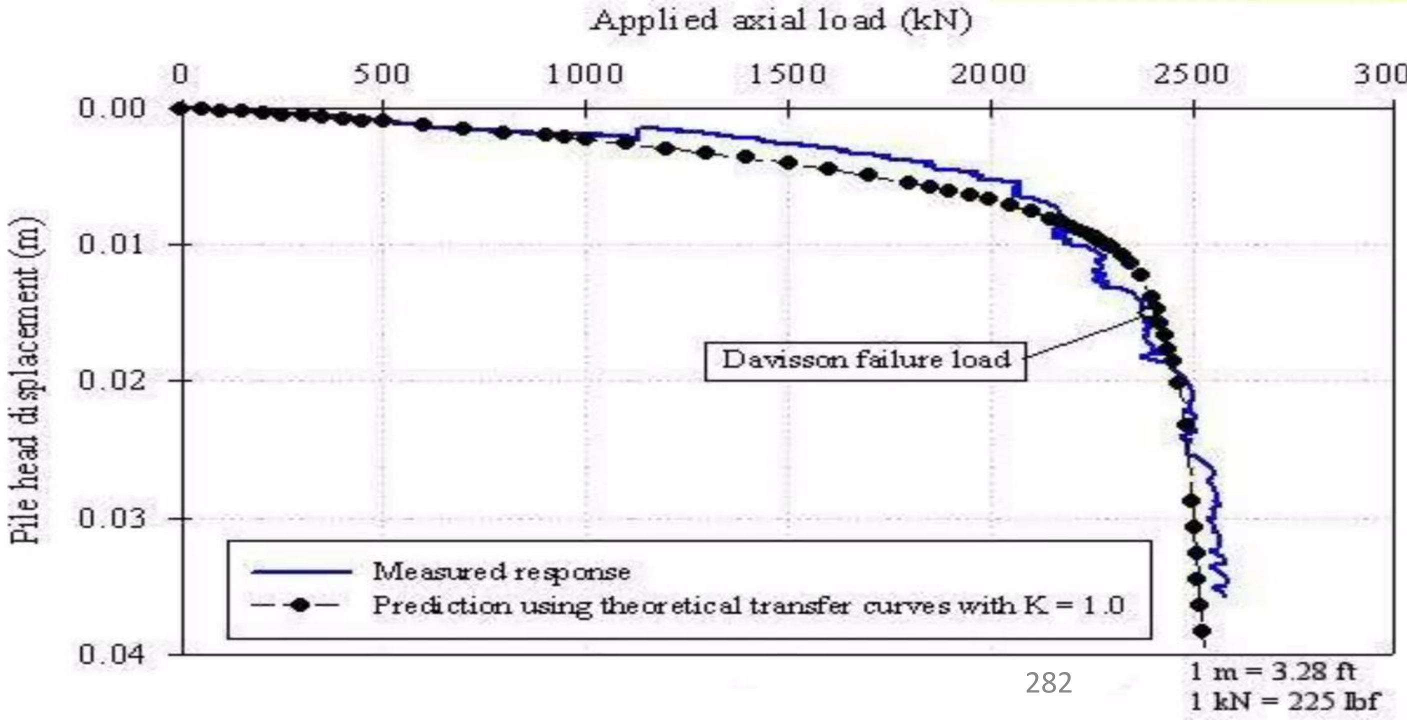
  S_e = Elastic settlement (rebound)

- Now, we will draw Net Load→Settlement curve.



Load/area
q
q_u
q_R
$q_{u(R)}$
S_e
$S_{e(u)}$
$S_{e(uR)}$
(1)
(2)
Unreinforced soil
Reinforced soil
Settlement, S_e

Fig.(A)



Applied axial load (kN)
0
500
1000
1500
2000
2500
0.00
0.01
0.02
0.03
0.04
Pile head displacement (m)
Davisson failure load
Measured response
Prediction using theoretical transfer curves with K = 1.0
1 m = 3.28 ft
1 kN = 225 lbf

Fig.(B)

RESULTS

- At the **ULTIMATE LOAD (Q_u)**, the load-settlement curve becomes either linear as curve (2)
- Or There is a sharp break as in the curve (1), as shown in Fig.(A).
- The safe load is usually taken as **one-half** of the **Ultimate load.**
- According to **IS:2911**,

1. the safe load is taken as **one-half** of the load at which the total settlement is equal to **10%** of the pile diameter or
2. two-third of the final load at which the total settlement is **12mm**, whichever is less.
3. Two-third of the final load which causes a net settlement(residual settlement after removal of load) of 6 mm.

INTRODUCTION :

Plate Load Test is a field test for determining the **ultimate bearing capacity of soil** and the likely settlement under a given load

Total value of load on plate divided by the area of the steel plate gives the value of ultimate bearing capacity

Its subdivided into following steps :

* Test setup
* Testing procedure
* Interpretation of results
* Limitation of the test

Testing procedure:

- The test can be done by two methods .they are :

  *gravity method   *truss method

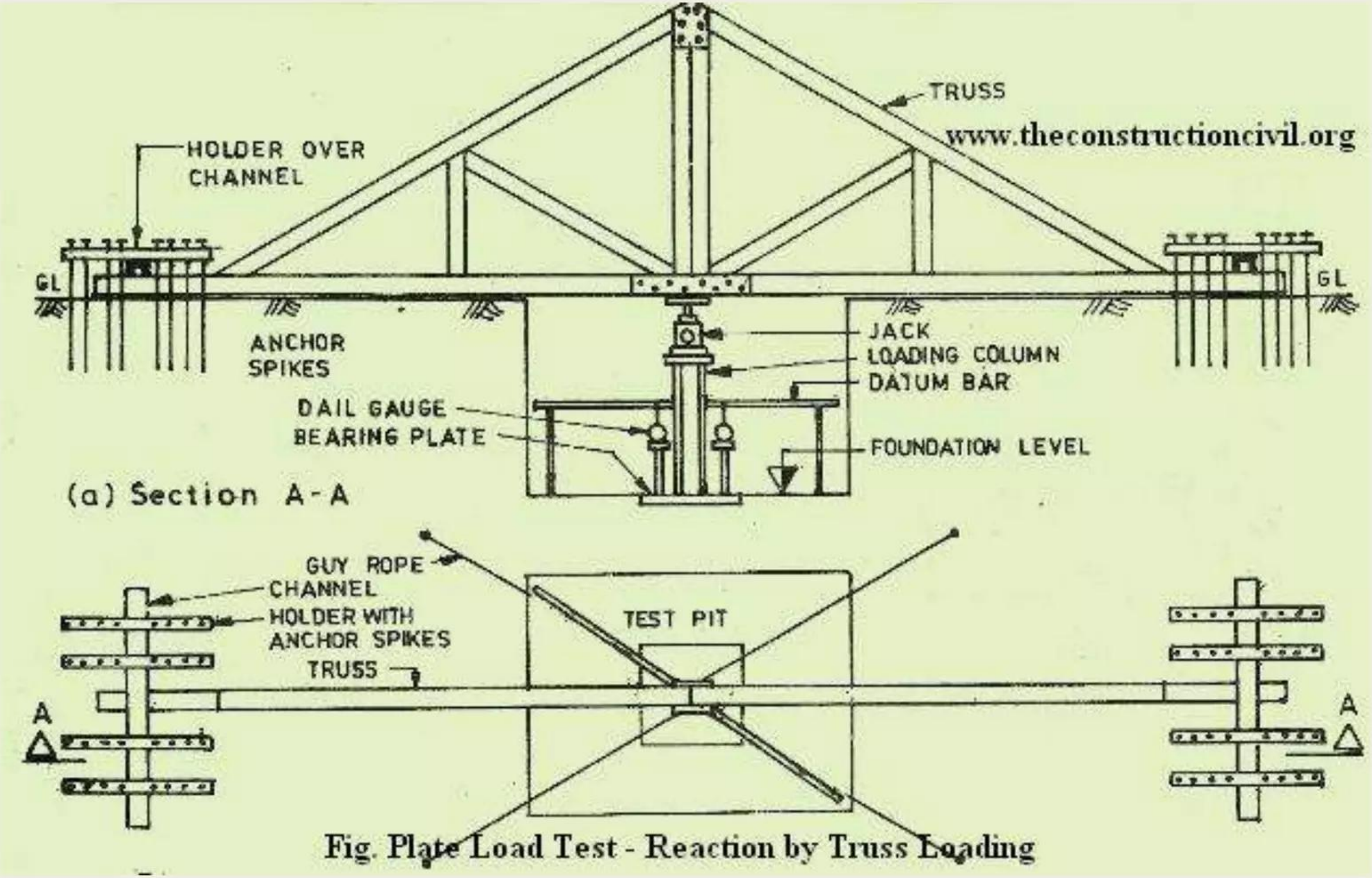
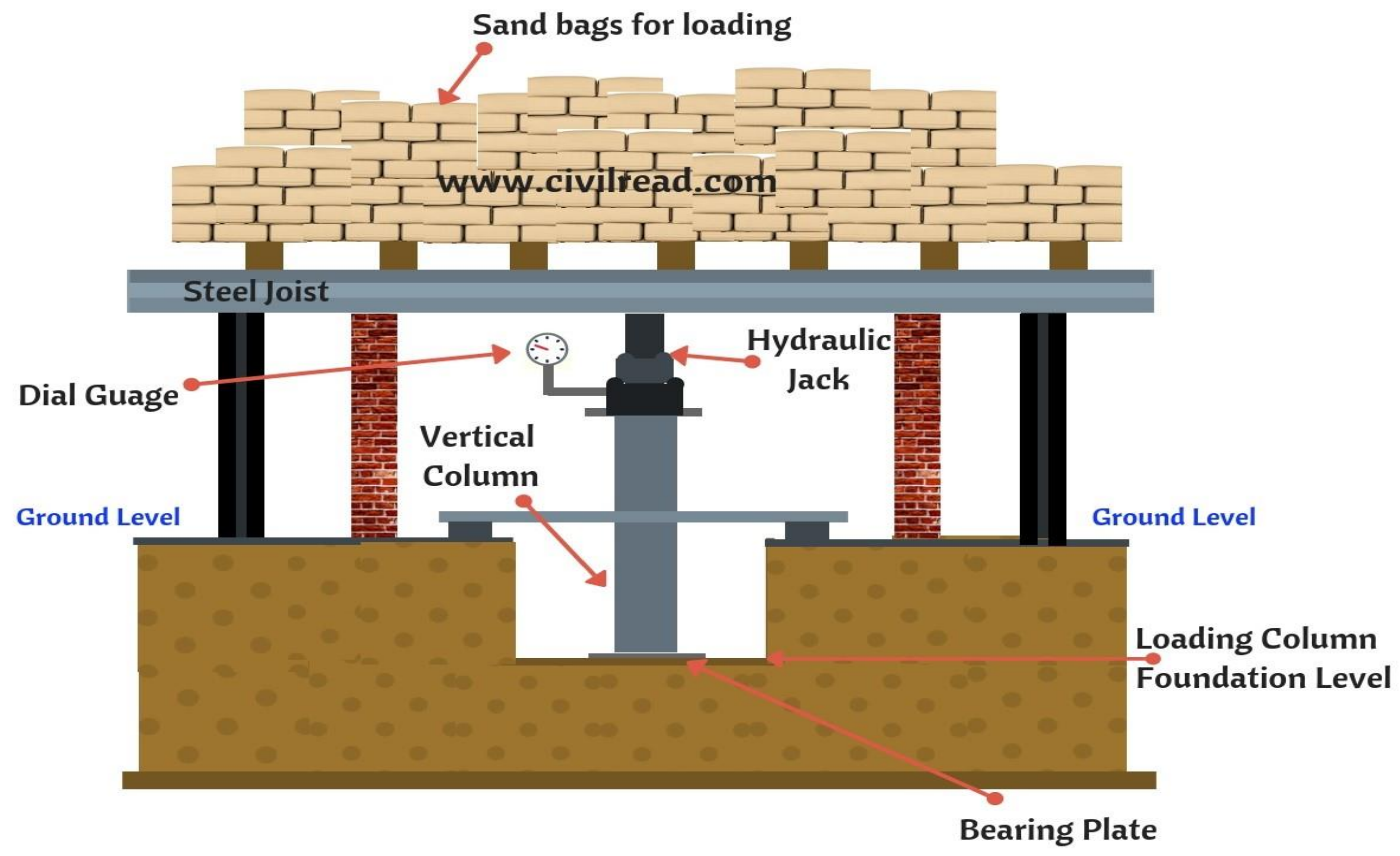


Fig. Plate Load Test - Reaction by Truss Loading



Gravity Loading – Plate Load Test

Interpretation of results:

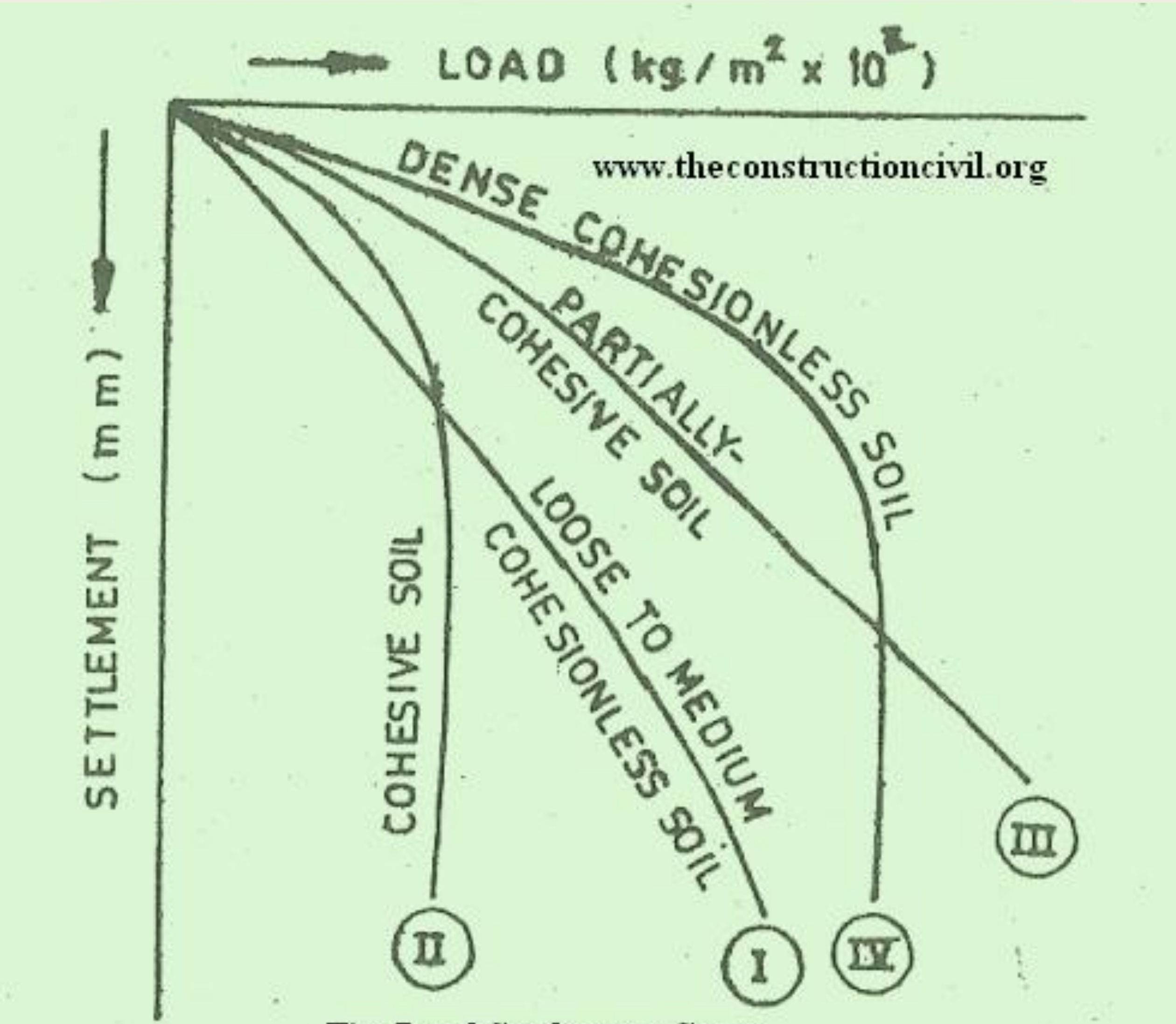


Fig. Load Settlement Curve

Limitations of plate load test :

- The test results reflect only the character of the soil located within a depth of less than twice the width of bearing plate
- The **Plate Load Test** being of short duration, does not give the ultimate settlements particularly in case of cohesive soils.
- For clayey soils the bearing capacity (from shear consideration) for a large foundation, is almost same as that for the smaller test plate.
- In dense sandy soils the bearing apacity increases with the size of the foundation and hence the test with smaller size test plate tends to give conservative values in dense sandy soils.

Slope Stability

Week 15

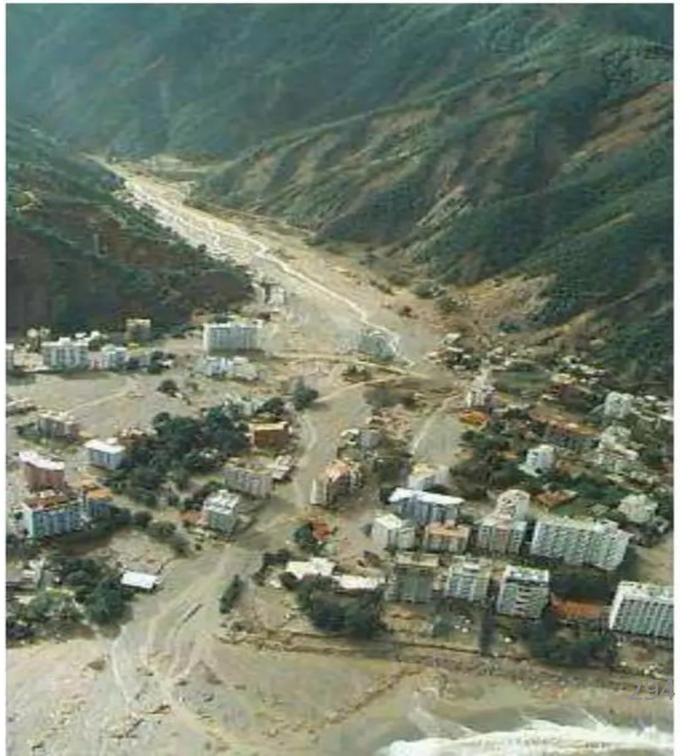
Pages 292-316

Introduction

- Why need to understand the slope failures?

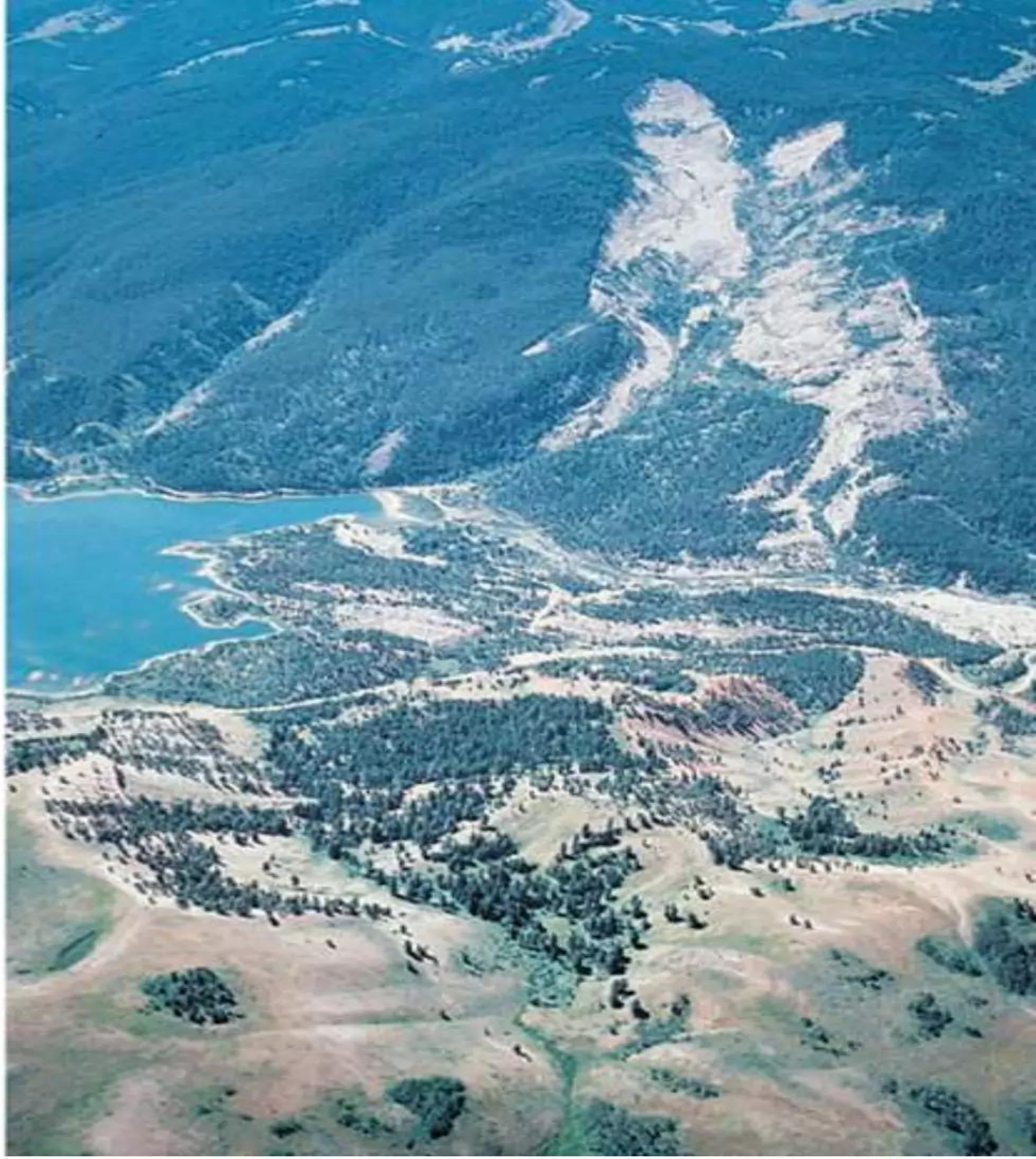
Sikkim, Gantok



Himalayan Region

50 million cubic yards

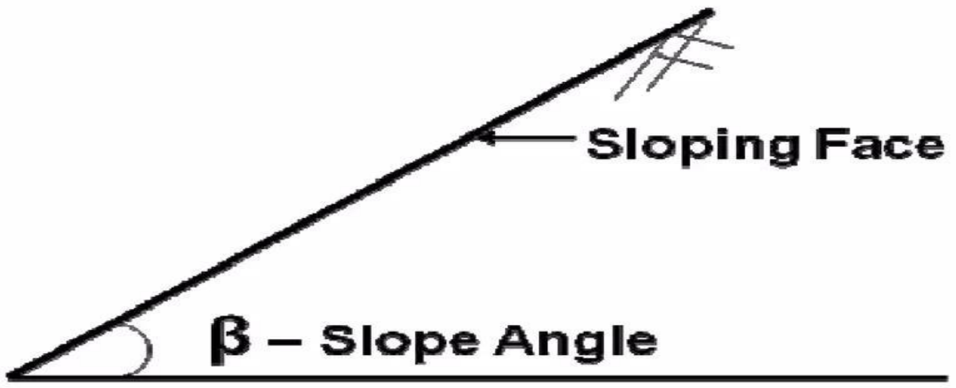


Indian Overall Topography

Slopes and its necessity

- An exposed ground surface that stands at an angle (b) with the horizontal is called slope.

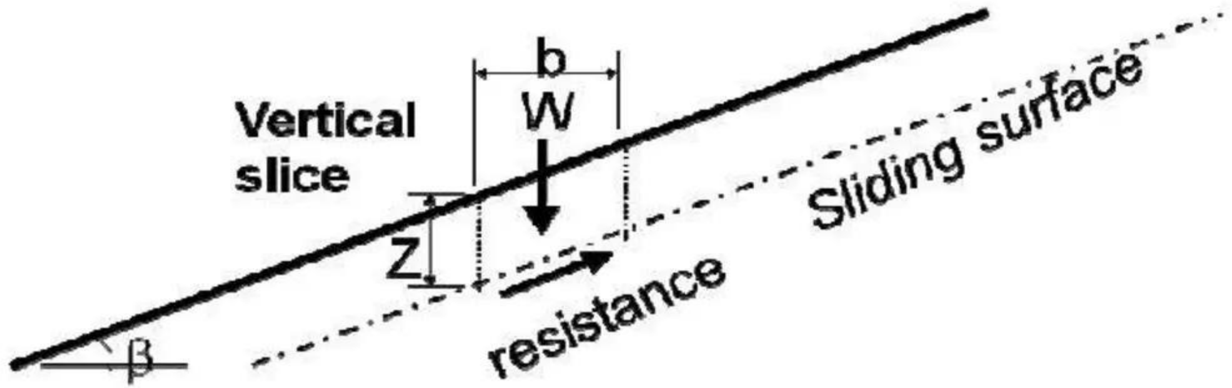
- construction of highway and railway
- Embankments
- earth dams
- levees and canals.



Types of Slopes

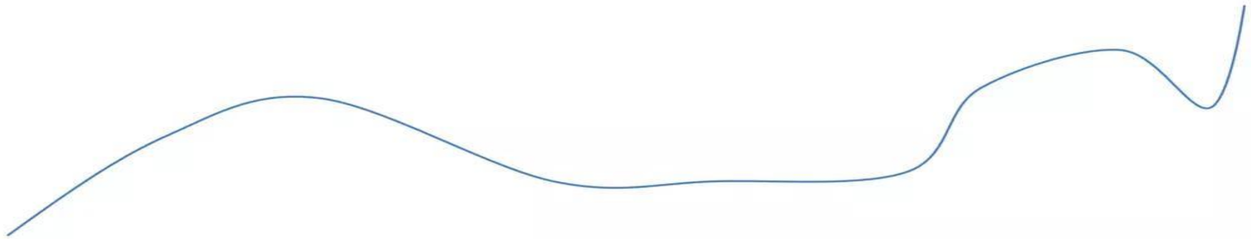
Infinite Slopes

- They have dimensions that extend over great distances and the soil mass is inclined to the horizontal.



Finite Slopes

- A finite slope is one with a base and top surface, the height being limited.
- The inclined faces of earth dams, embankments and excavation and the like are all finite slopes.



Types of slopes

- Natural : Formation due to geological features of the earth
- man made: Construction activity like cutting, filling etc
-

Why to understand slopes?

- Failure of natural slopes (landslides) and man-made slopes has resulted in much death and destruction.

- Civil Engineers are expected to check the safety of natural and slopes of excavation.

- Slope stability analysis consists of determining and comparing the shear stress developed along the potential rupture surface with the shear strength of the soil.

- Attention has to be paid to geology, surface drainage, groundwater, and the shear strength of soils in assessing slope stability.

Slope Failure Triggering Mechanisms

- Intense Rain-Fall
- Water-Level Change
- Seepage Water Flow
- Volcanic Eruption
- Earthquake Shaking
- Human activity

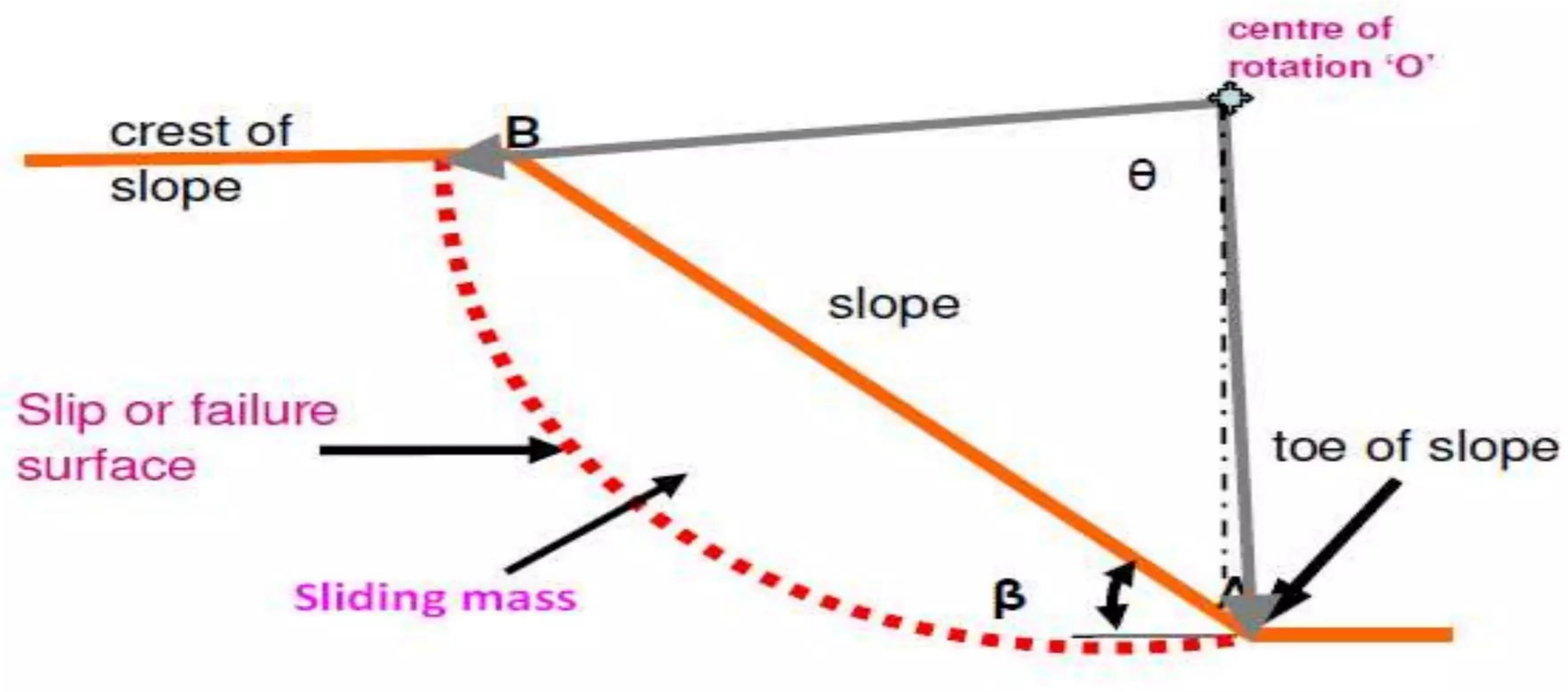
Causes of Slope failure

- **Erosion: The wind and flowing water causes erosion of top surface of** slope and makes the slope steep and thereby increase the tangential component of driving force.
- **Steady Seepage: Seepage forces in the sloping direction add to gravity** forces and make the slope susceptible to instability. The pore water pressure decrease the shear strength. This condition is critical for the downstream slope.
- **Sudden Drawdown: in this case there is reversal in the direction flow and** results in instability of side slope. Due to sudden drawdown the shear stresses are more due to saturated unit weight while the shearing resistance decreases due to pore water pressure that does not dissipate quickly.
- **Rainfall: Long periods of rainfall saturate, soften, and erode soils. Water** enters into existing cracks and may weaken underlying soil layers, leading to failure, for example, mud slides.

Causes of Slope failure ....

- **Earthquakes: They induce dynamic shear forces. In addition there is** sudden buildup of pore water pressure that reduces available shear strength.
- **External Loading: Additional loads placed on top of the slope increases** the gravitational forces that may cause the slope to fail.
- **Construction activities at the toe of the slope: Excavation at the bottom** of the sloping surface will make the slopes steep and there by increase the gravitational forces which may result in slope failure

Definition of Key Terms



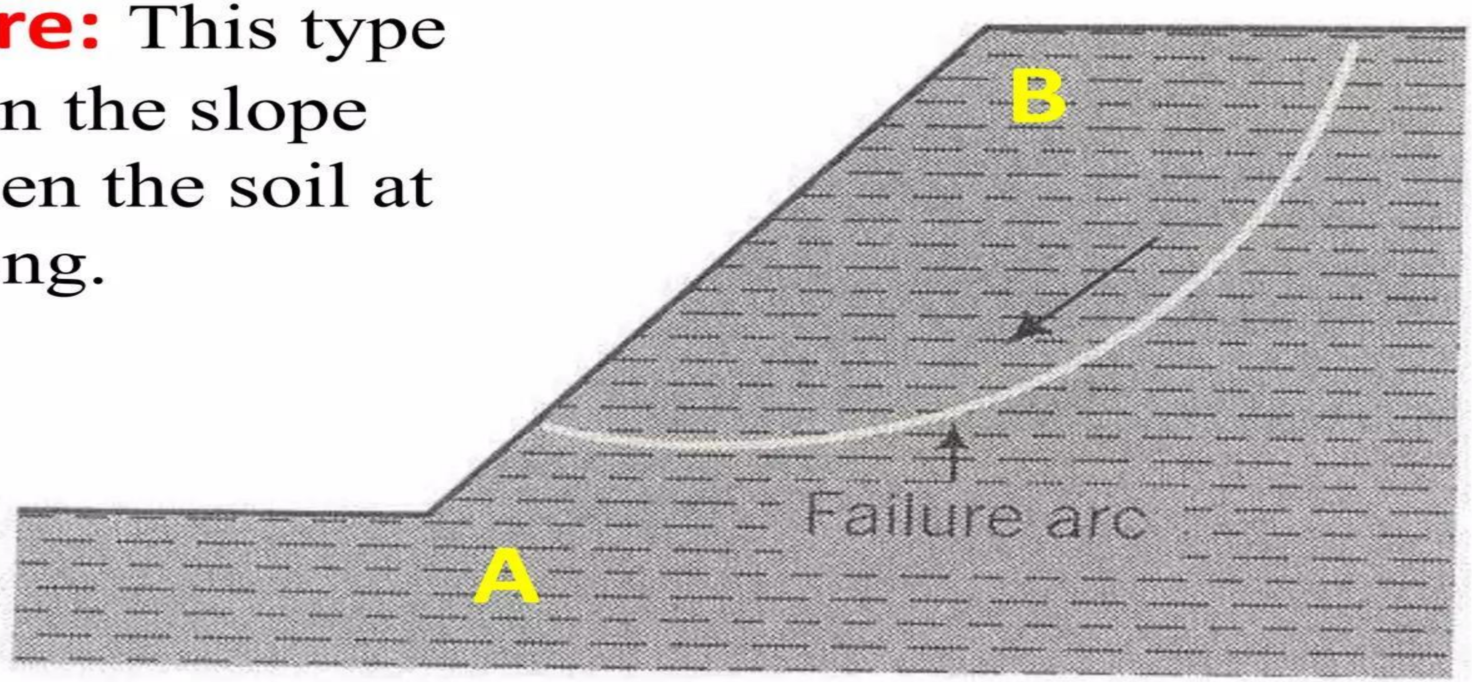
- **Slip or failure zone**: It is a thin zone of soil that reaches the critical state or residual state and results in movement of the upper soil mass.
- **Slip plane or failure plane or slip surface or failure surface:** It is the surface of sliding.
- **Sliding mass:** It is the mass of soil within the slip plane and the ground surface.
- **Slope angle :** It is the angle of inclination of a slope to the horizontal.
- The slope angle is sometimes referred to as a ratio, for example, 2:1 (horizontal: vertical).

Types of failure

- Broadly slope failures are classified into 3 types as
  1. Face (Slope) failure
  2. Toe failure
  3. Base failure

Face (Slope) Failure:

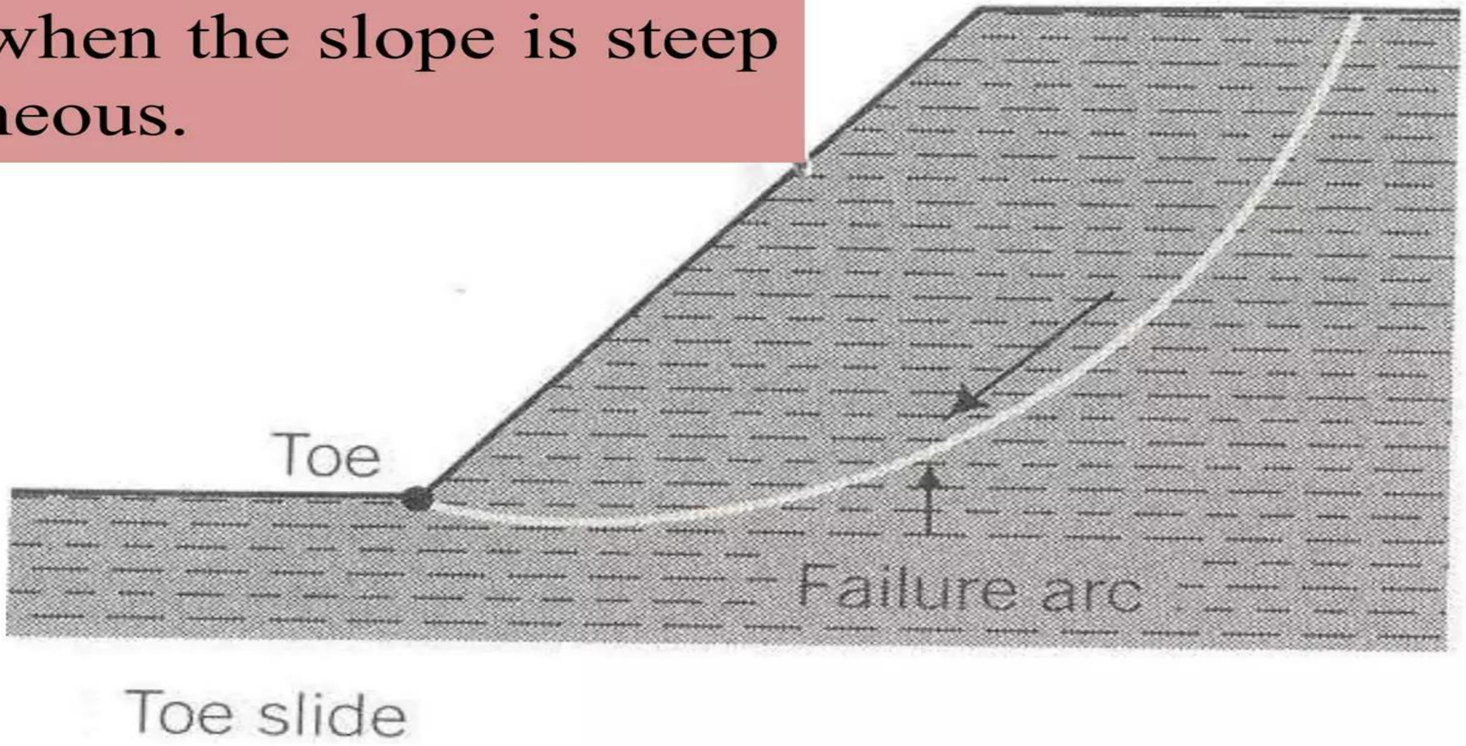
Face (Slope) Failure: This type of failure occurs when the slope angle is large and when the soil at the toe portion is strong.



Slope slide

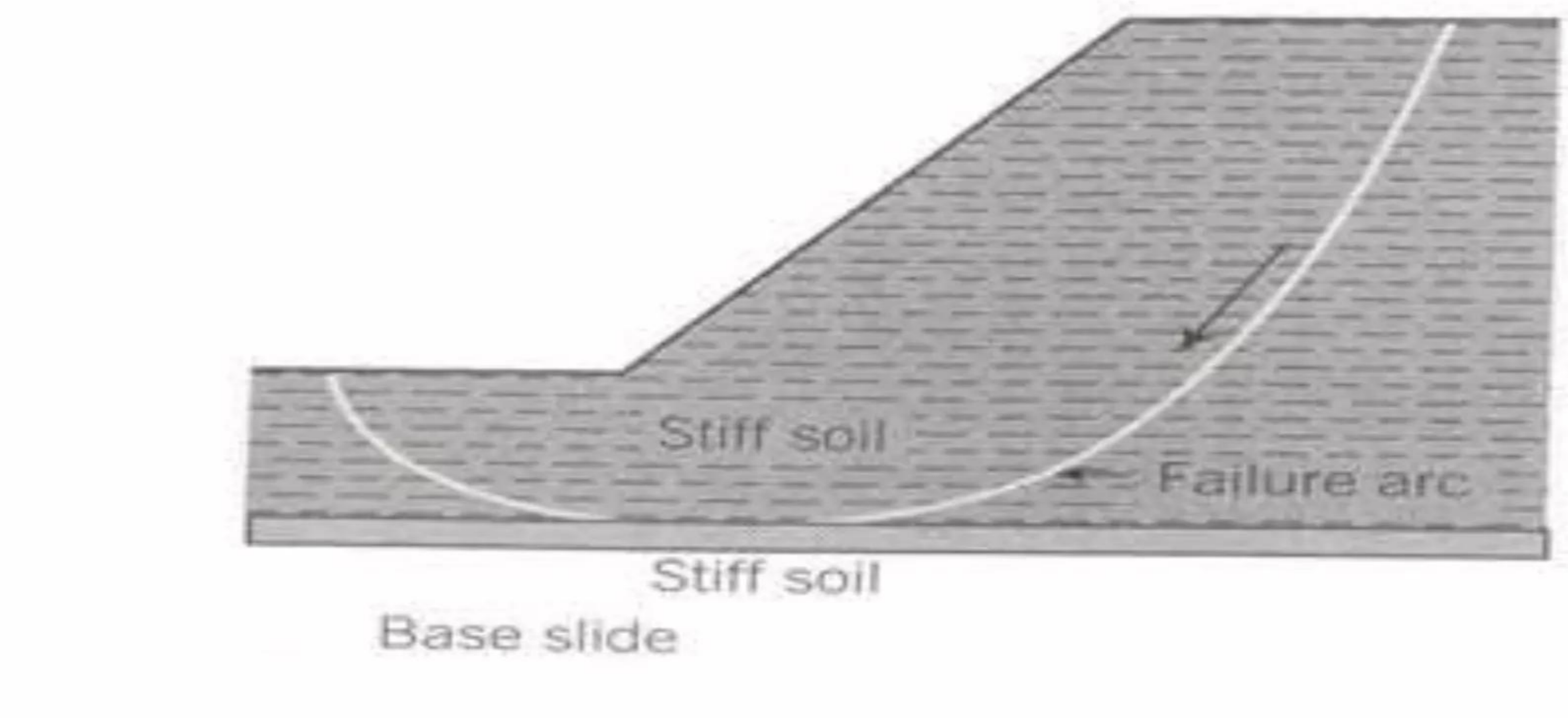
Toe Failure:

In this case the failure surface passes through the toe.
This occurs when the slope is steep and homogeneous.



Base Failure:

In this case the failure surface passes below the toe. This generally occurs when the soil below the toe is relatively weak and soft.



Analysis of slopes : Factor of safety

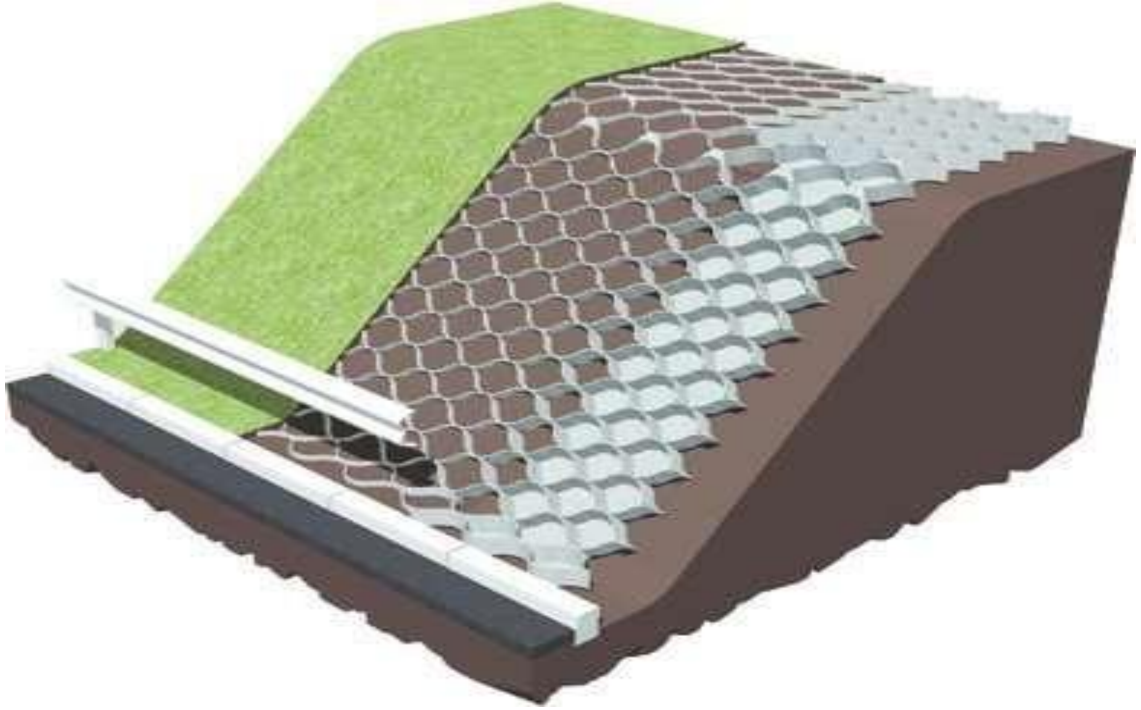
- Factor of safety of a slope is defined as the ratio of average shear strength (tf) of a soil to the average shear stress (td) developed along the potential failure surface.

$$FS = \frac{\tau_f}{\tau_d}$$

ALTERNATIVES TO RETAINING WALLS

CELLULAR CONFINEMENT

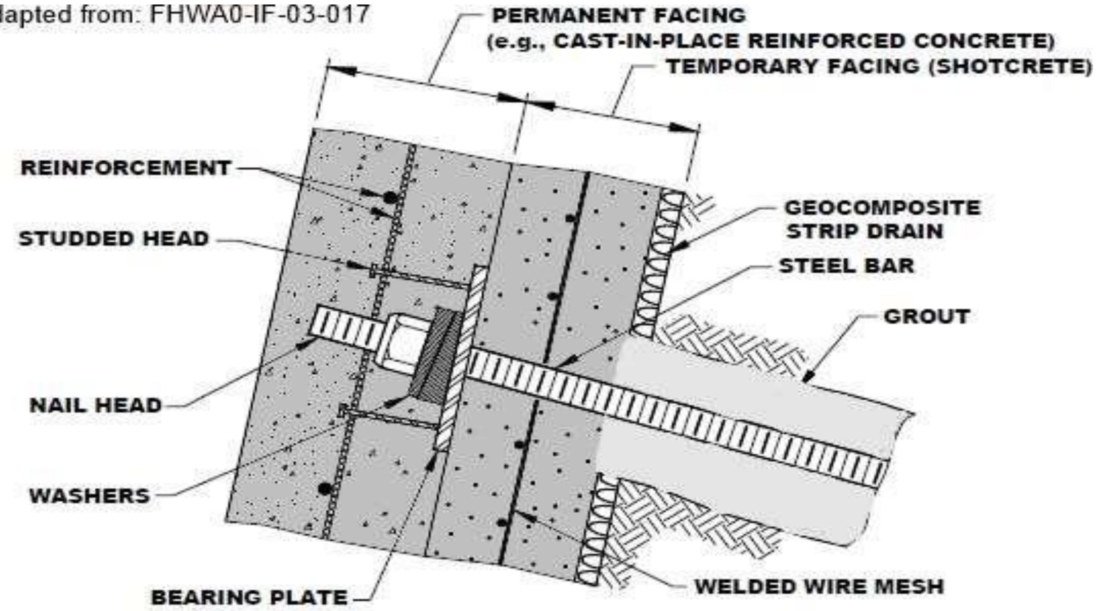
- Cellular confinement systems have become increasingly popular for earth retention applications.
- They can be constructed as a gravity wall or a "geogrid" wall which consists of vertical layers of geocells with geogrid reinforcement installed behind the face of the wall every few layers of the geocell depending on design.



- Cellular confinement systems (geocells) are also used for steep earth stabilization in gravity and reinforced retaining walls with geogrids.
- Geocell retaining walls are structurally stable under self- weight and externally imposed loads, while the flexibility of the structure offers very high seismic resistance.
- The outer fascia cells of the wall can be planted with vegetation to create a green wall.

- Soil nailing is a technique in which soil slopes, excavations or retaining walls are reinforced by the insertion of relatively slender elements - normally steel reinforcing bars.
- The bars are usually installed into a pre-drilled hole and then grouted into place or drilled and grouted simultaneously.
- They are usually installed un-tensioned at a slight downward inclination.
- A rigid or flexible facing (often sprayed concrete) or isolated soil nail heads may be used at the surface.

Adapted from: FHWA0-IF-03-017



GABION MESHES

- This type of soil strengthening, often also used without an outside wall, consists of wire mesh "boxes, which are filled with roughly cut stone or other material.
- The mesh cages reduce some internal movement and forces, and also reduce erosive forces.
- Gabion walls are free- draining retaining structures and as such are often built in locations where ground water is present.

BUTTRESS WALL

- It is similar to counterfort wall, except that the transverse stem supports, Called buttress, are located in the front side, interconnecting the stem with the toe slab(and not with heel slab, as with counterforts)

- Although the buttresses are structurally more efficient (and more economical) counterforts, the counterfort wall is generally preferred to the buttress wall as it provides free usable space (and better aesthetics)in front of the wall.

Soil Report

Week 16

Click The Link (pdf)

Soil Report

D:\UGV\Soil Test Report (2).pdf

Review and Practice Class

Week 17

Any Questions?

Thank you for your kind attention

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