University of Global Village (UGV), Barishal

Materials Testing III (Soil)

Content of Laboratory Course

Program: B.Sc. in CE

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BASIC COURSE INFORMATION

Course Title	Materials Testing III (Soil)
Course Code	CE 0732-4102
Credits	01
CIE Marks	30
SEE Marks	20
Exam Hours	2 hours (Semester Final Exam)
Level	6^{th} Semester

Materials Testing III (Soil)

COURSE CODE: CE 0732-4102 | **CREDIT:01**

TOTAL MARKS:50

CIE MARKS: 30

Semester End Exam Hours 2 | **SEE MARKS: 20**

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	**Designing** and **analyzing** shallow and deep foundations, retaining walls, and slope stabilization measures considering safety, sustainability, and economic factors
CLO 4	Effectively **communicate** their findings and design solutions through technical reports, presentations, and graphical representations.

SL	Content of Course	Hrs	CLOs
1	Standard Penetration Test (SPT)	5	CLO1
2	Direct Shear Test	5	CLO3
3	Unconfined Compression Test	10	CLO2, CLO4
4	Tri-axial Compression Test (Unconsolidated Undrained)	10	CLO1, CLO3
5	Tri-axial Compression Test (Consolidated Undrained)	10	CLO1
6	Tri-axial Compression Test (Consolidated Drained)	10	CLO3
7	Consolidation Basic/Settlement Analysis	10	CLO1
8	Soil Report Study	10	CLO1
9	Lab Test, Viva, Quiz, Overall Assessment, Skill Development Test (Competency)	15	CLO3

Text Book:

1. Principles of Geotechnical Engineering (8th Ed.)-Braja M. Das & Khaled Sobhan.
2. Foundation Engineering (2nd Ed.)-R. B. Peck, W. E. Hanson & T. H. Thornburn.
3. An Introduction to Geotechnical Engineering (2nd Ed.) - R. D. Holtz & William D. Kovacs.
4. Geotechnical Engineering – Principles and Practices (2nd Ed.) - D. P. Coduto.
5. Geotechnical Engg. (2010) – A practical problem-solving approach - N. Siv. and B. M. Das.
6. Soil Mechanics in Engineering Practice (3rd Ed.) - Terzaghi, Peck & Mesri.
7. Craigs Soil Mechanics - R. F. Craig & R. F. Pink.
8. Engineering Soil Mechanics - Jan J. Tuma& M. Abdel-Hady.
9. Elements of Soil Mechanics - Geoffrey Nesbitt Smith.

ASSESSMENT PATTERN

CIE- Continuous Internal Evaluation (30 Marks)

SEE- Semester End Examination (20 Marks)

SEE- Semester End Examination (40 Marks) (should be converted in actual marks (20))

Bloom's Category	**Tests**
Remember	**05**
Understand	**07**
Apply	**08**
Analyze	**07**
Evaluate	**08**
Create	**05**

CIE- Continuous Internal Evaluation (100 Marks) (should be converted in actual marks (30))

Bloom's Category Marks (out of 100)	Lab Final (30)	Lab Report (10)	**Continuous lab performance** (30)	Presentation & Viva (10)	External Participation in Curricular/**Final Project Exhibition** (10)
Remember/**Imitation**	05		05	02	Attendance 10
Understand/**manipulation**	05	05	05	03	
Apply/**Precision**	05		05		
Analyze/**Articulation**	05		05		
Evaluate/**Naturalisation**	05	05	05		
Create	05		05	05	

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

Week	**Topic**	**Teaching-Learning Strategy**	**Assessment Strategy**	**Correspondin g CLOs**
1-2	Standard Penetration Test (SPT)	Lecture, discussion, Experiment	Quiz, Lab Test	CLO1
3-4	Direct Shear Test	Oral Presentation, Project Exhibition	Lab Report Assessment, viva	CLO3
5	Tri-axial Compression Test (Unconsolidated Undrained)	Presentation, Field visit	Skill Development Test	CLO2, CLO4
6	Tri-axial Compression Test (Consolidated Undrained)	Lecture, discussion, Experiment, Demonstration	Quiz, Lab Test	CLO1, CLO3
7	Tri-axial Compression Test (Consolidated Drained)	Oral Presentation, Project Exhibition	Lab Report Assessment, viva	CLO1
8	Unconfined Compression Test	Presentation, Field visit	Skill Development Test	CLO3
9-10	Consolidation Basic/Settlement Analysis	Lecture, discussion, Experiment	Quiz, Lab Test	CLO2, CLO4
11-13	Soil Report Study	Presentation, Field visit	Skill Development Test	CLO3
14-17	Lab Test, Viva, Quiz, Overall Assessment, Skill Development Test (Competency)	Lecture, discussion, Experiment	Quiz, Lab Test	CLO2, CLO4

Standard Penetration Test

Week 1-2

Pages 8-35

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

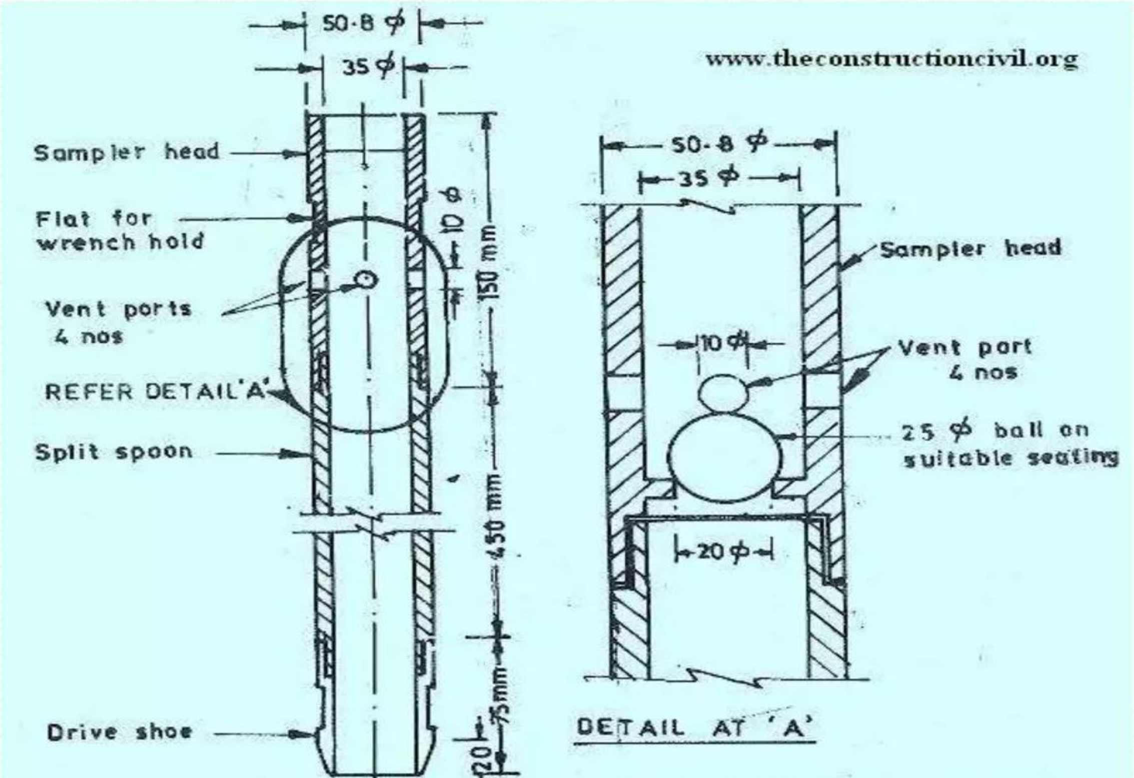


Fig. Standard Split Spoon Sampler

- 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.

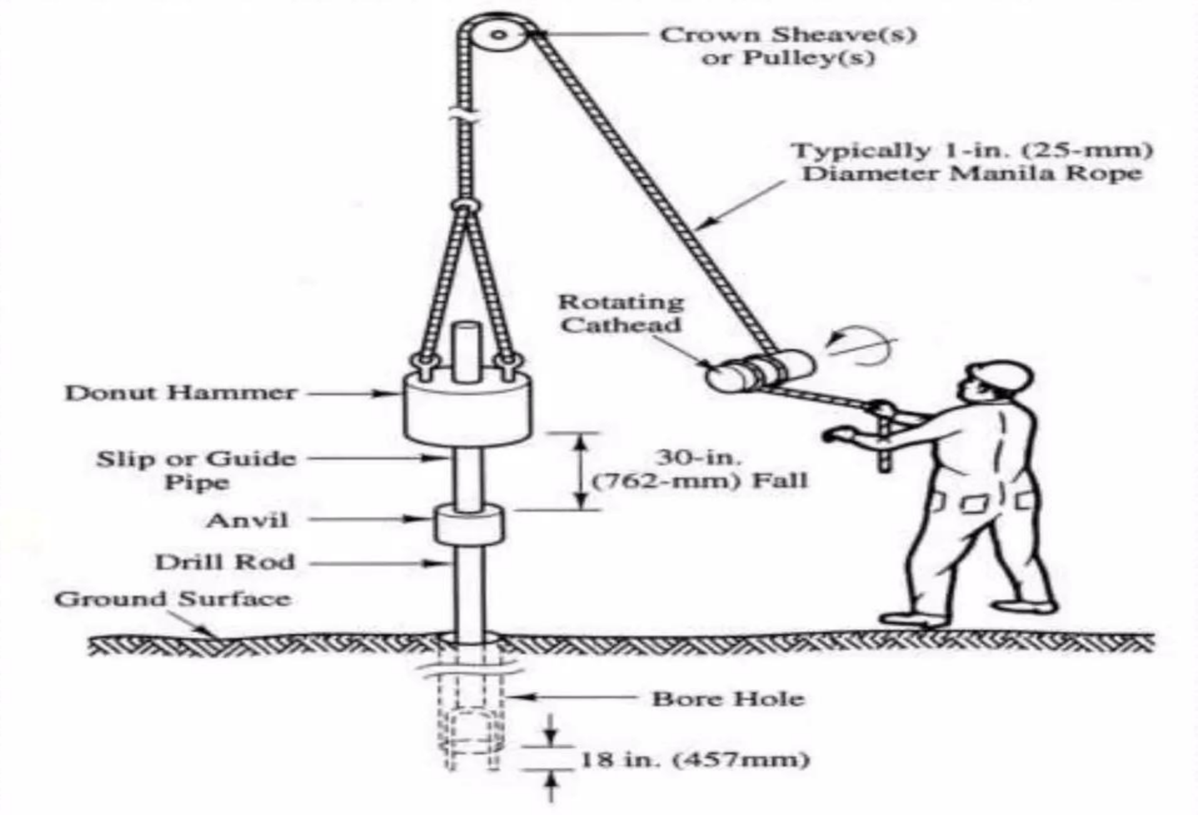
INSTRUMENTS

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

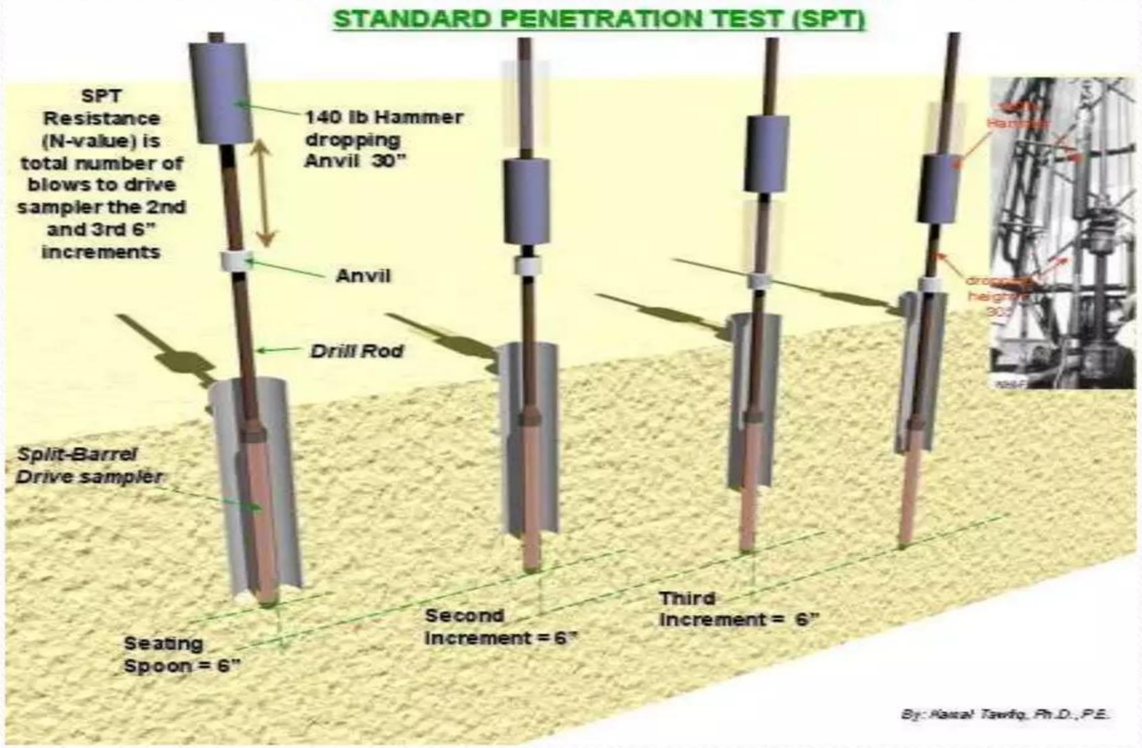


Operating at approximately 100 rpm.

Equipped with suitable rope and overhead sheave for lifting drive-weight.

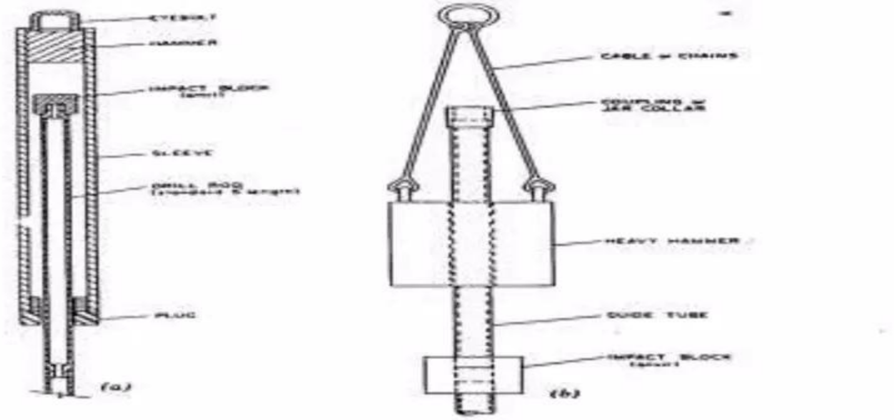
INSTRUMENTS

5)HAMMER

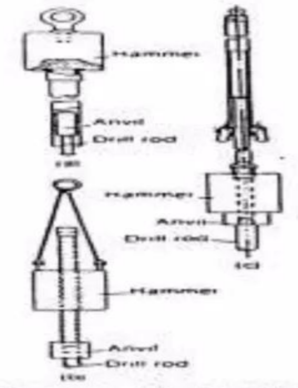


- **A)SEFETY HAMMER**
- Closed system
- Delivers approximately 60% of the maximum free fall energy
- Highly variable energy transfer

INSTRUMENTS



.—Configuration of SPT Hammers (Adopted from Ref. 48): (a) Safety Hammer; (b) Donut Hammer



SPT hammers: (a) old standard; (b) donut; (c) trip

- **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.

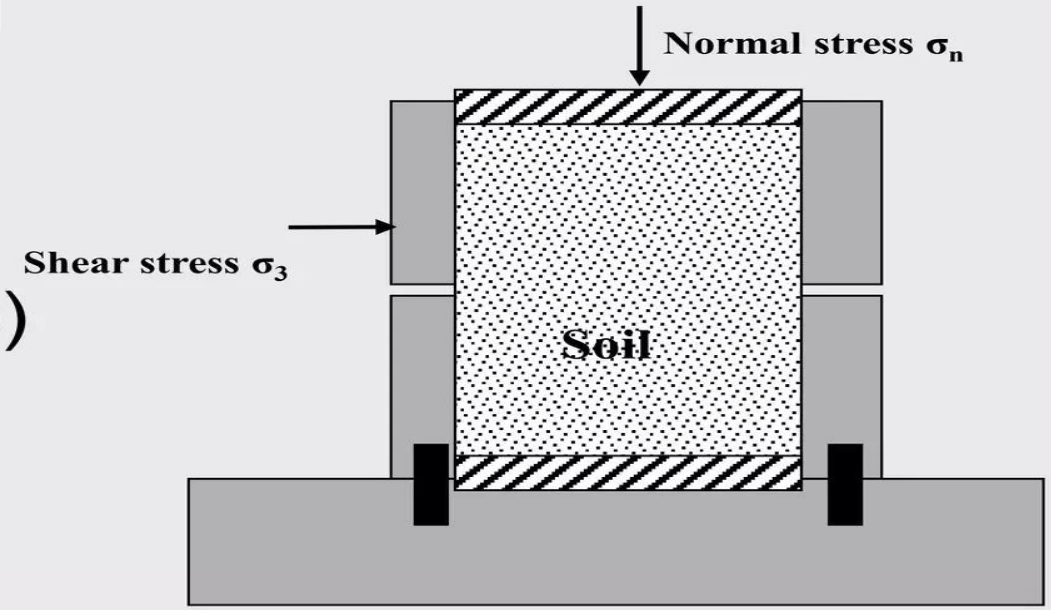
Direct shear Test

Week 3-4

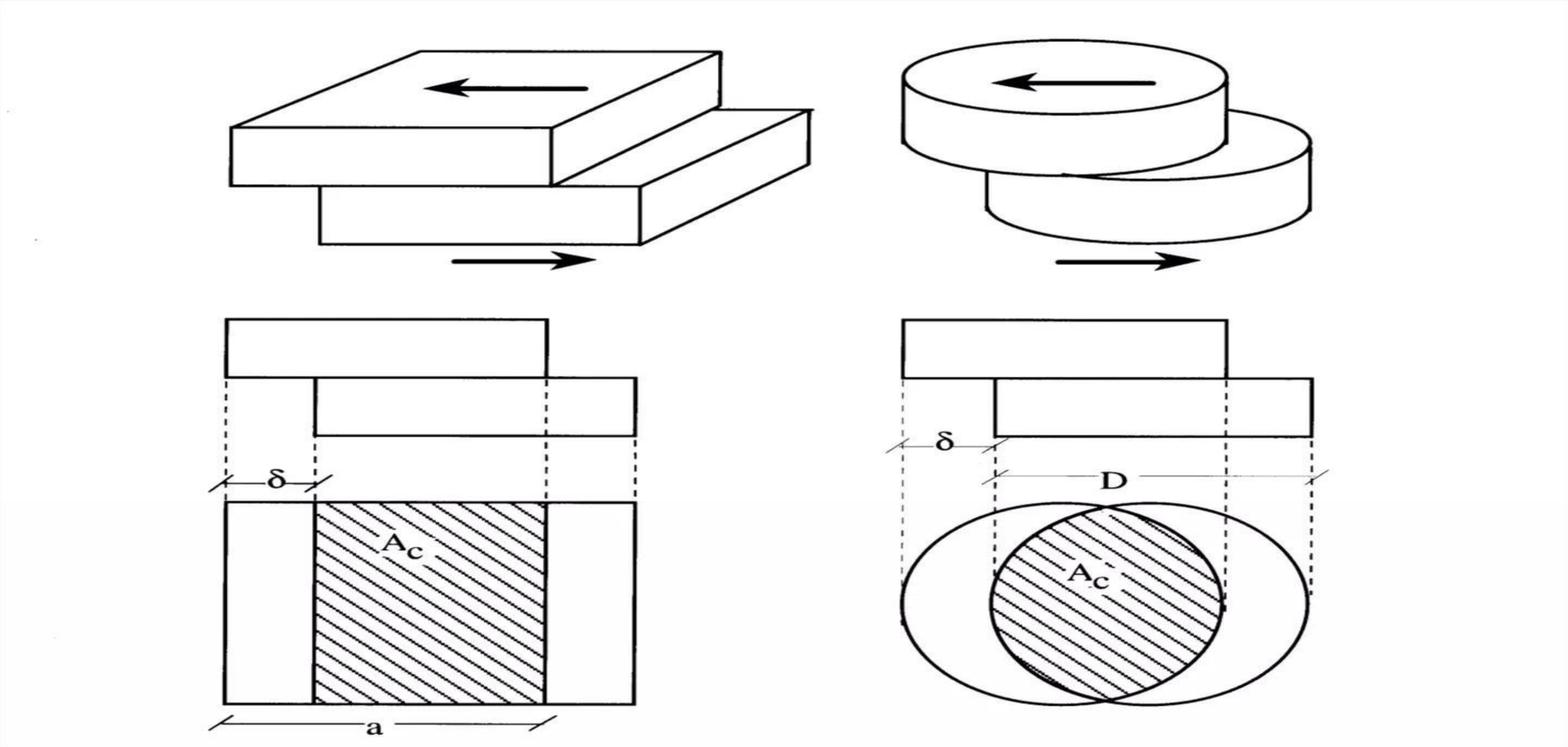
Pages 37-48

Direct Shear Test (cont.)

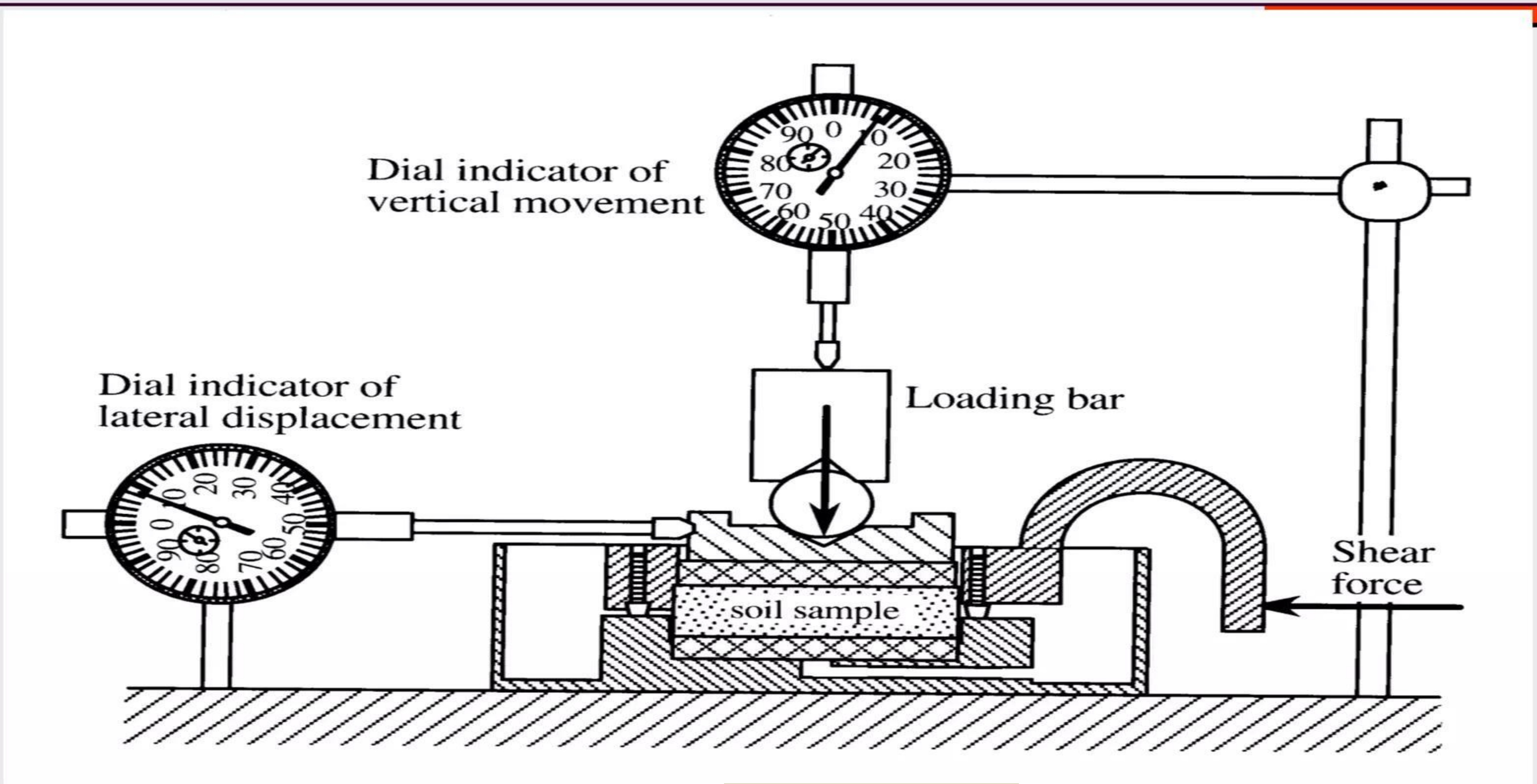
- The test equipment consists of a metal box in which the soil specimen is placed
- The box is split horizontally into two halves
- Vertical force (normal stress) is applied through a metal platen
- Shear force is applied by moving one half of the box relative to the other to cause failure in the soil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



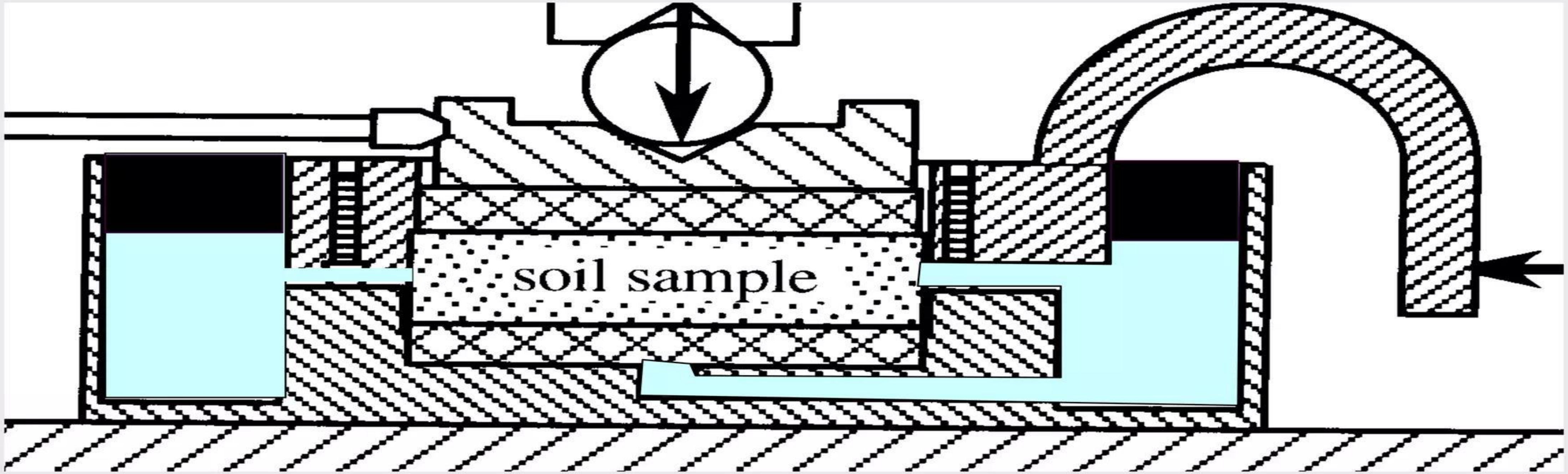
Direct Shear Test



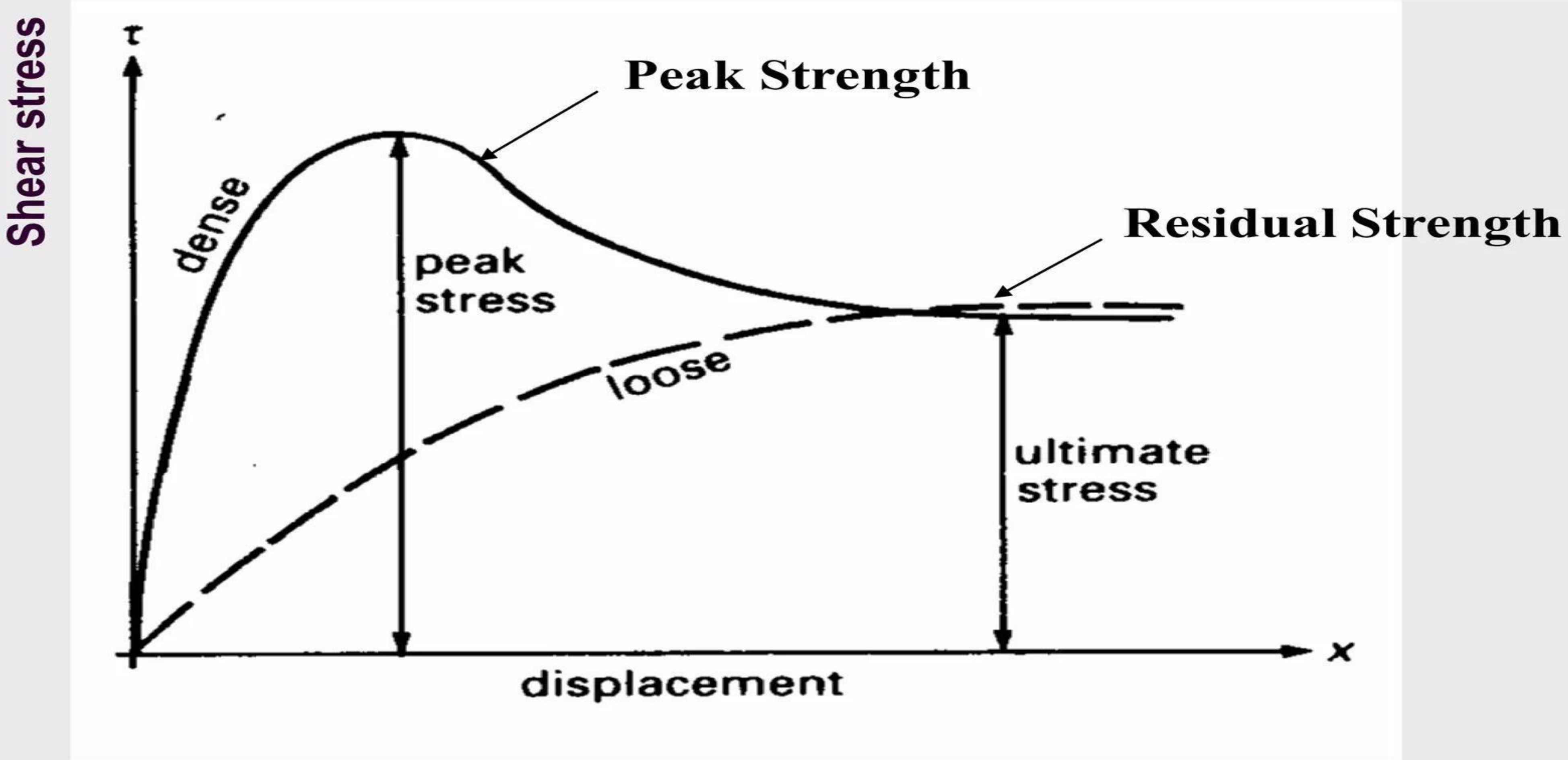
Direct Shear Test



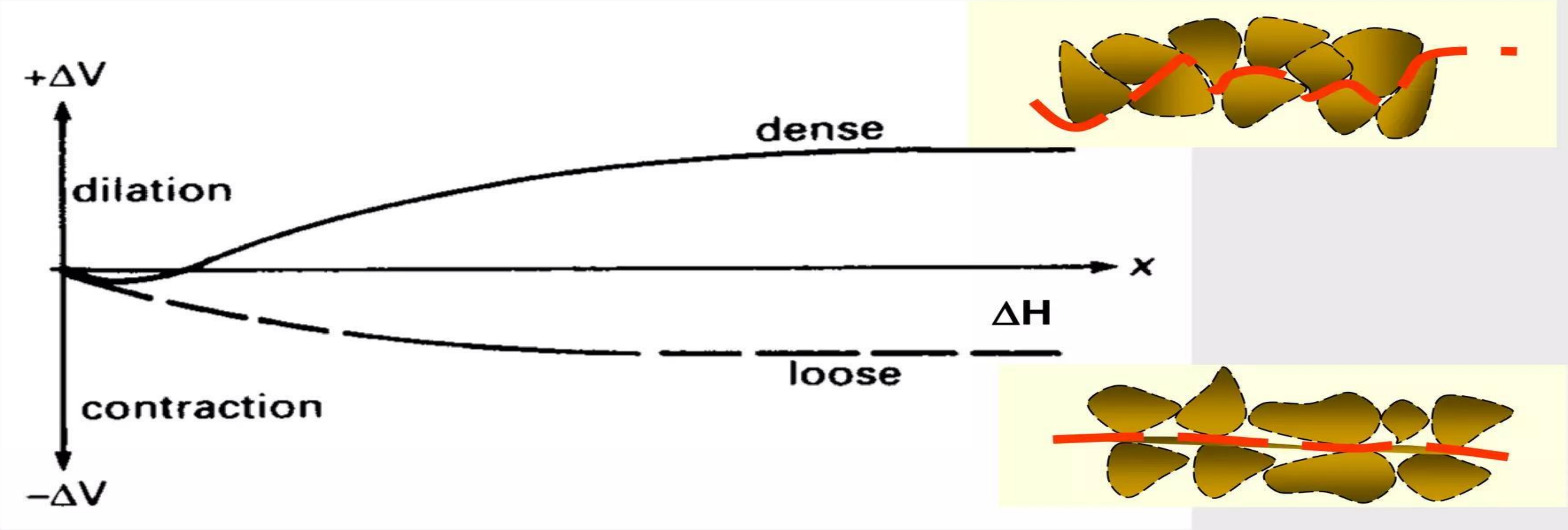
Direct Shear Test



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

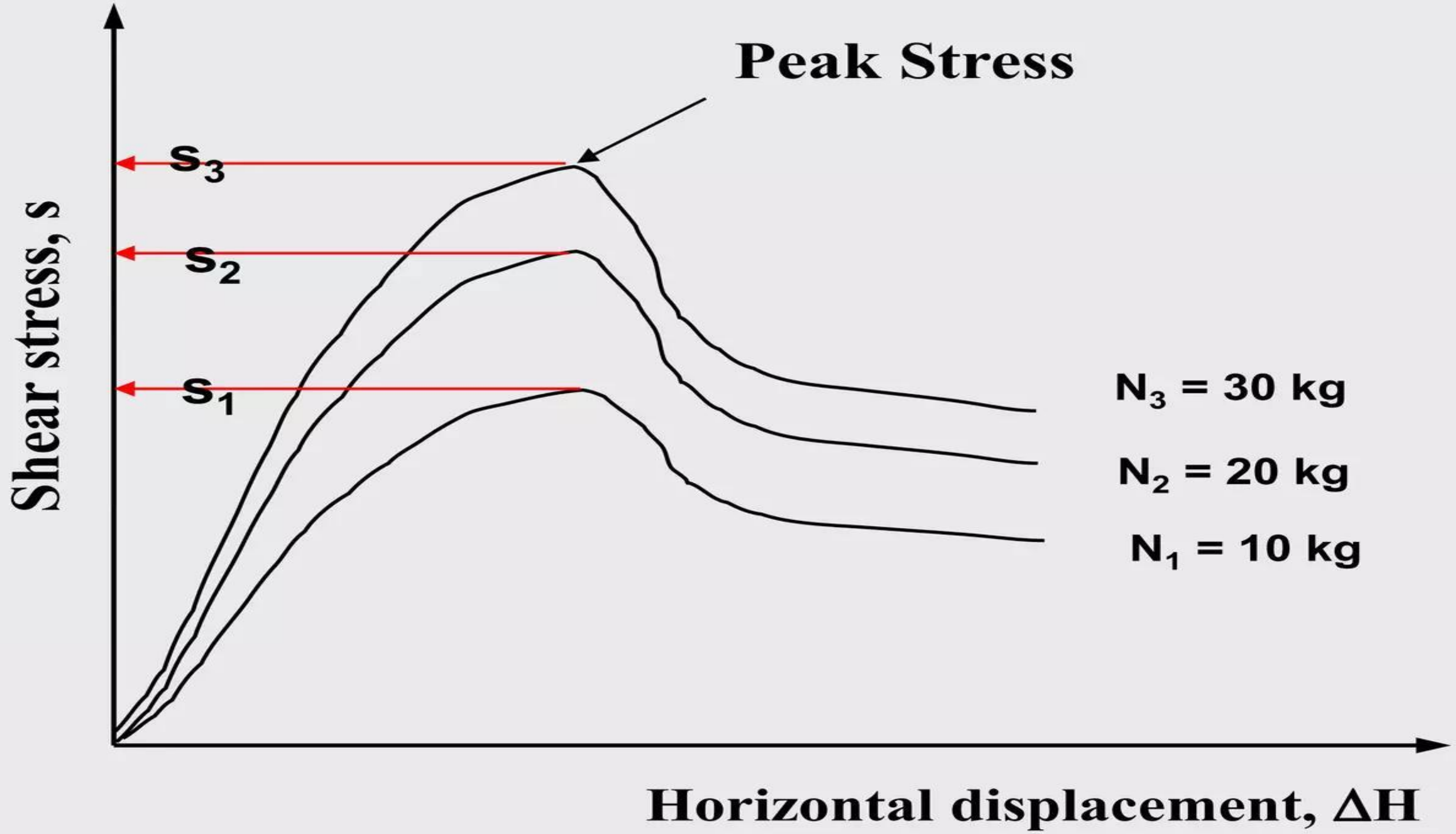
Calculations

1. Determine the dry unit weight, γ_d

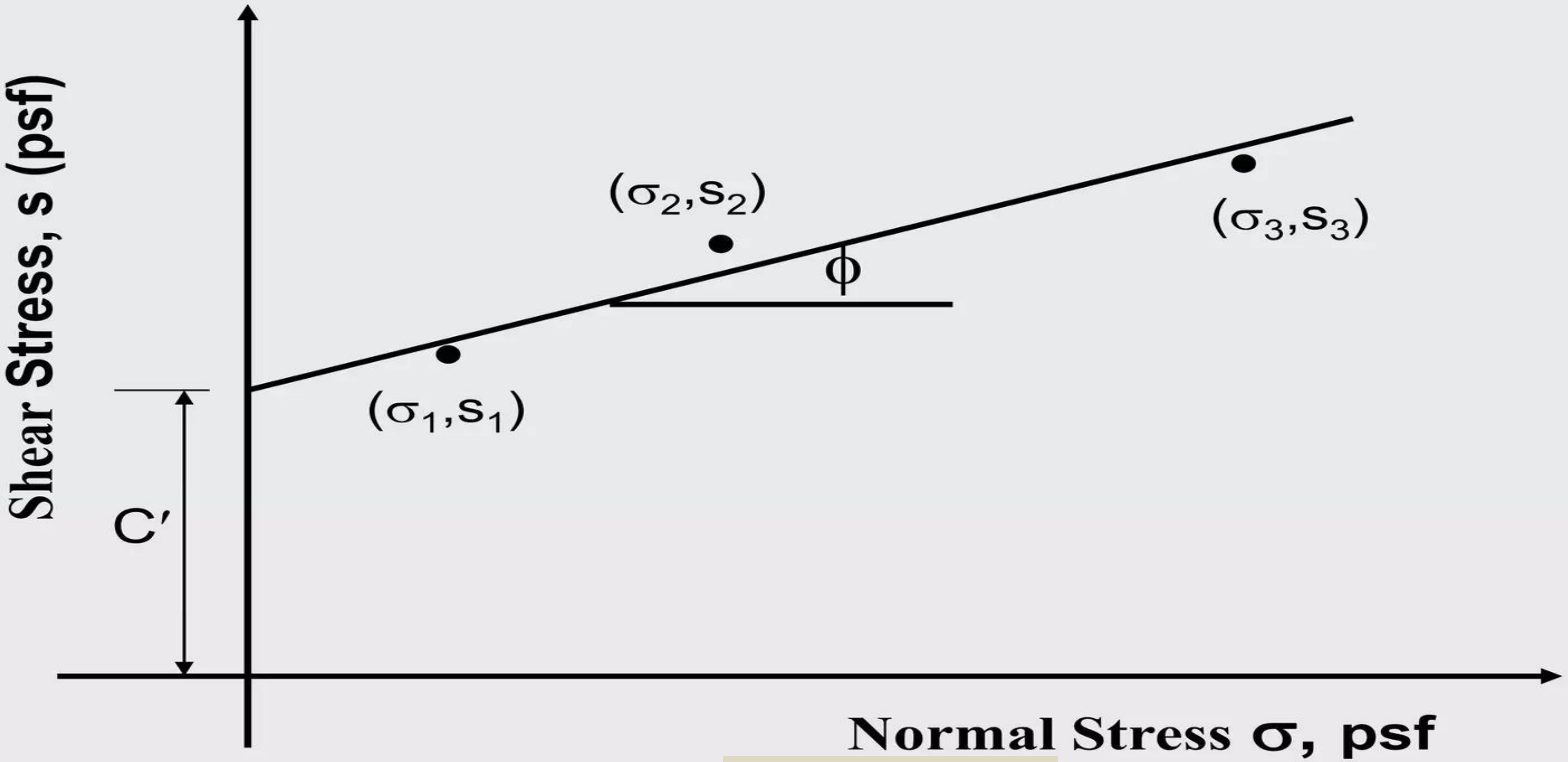
2. Calculate the void ratio, e
$$e = \frac{Gs\gamma_w}{\gamma_d} - 1$$

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)



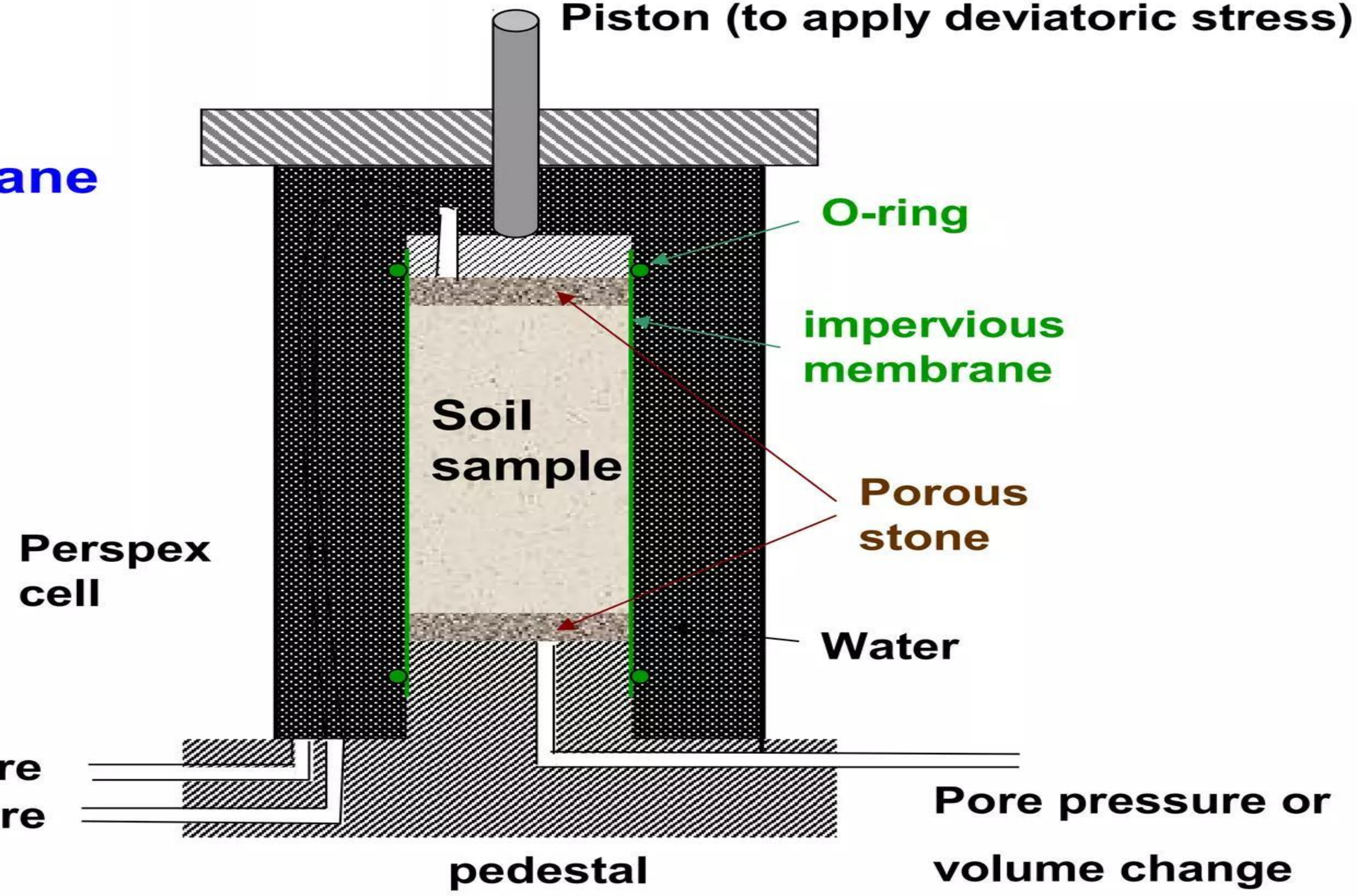
Tri-axial Shear Test

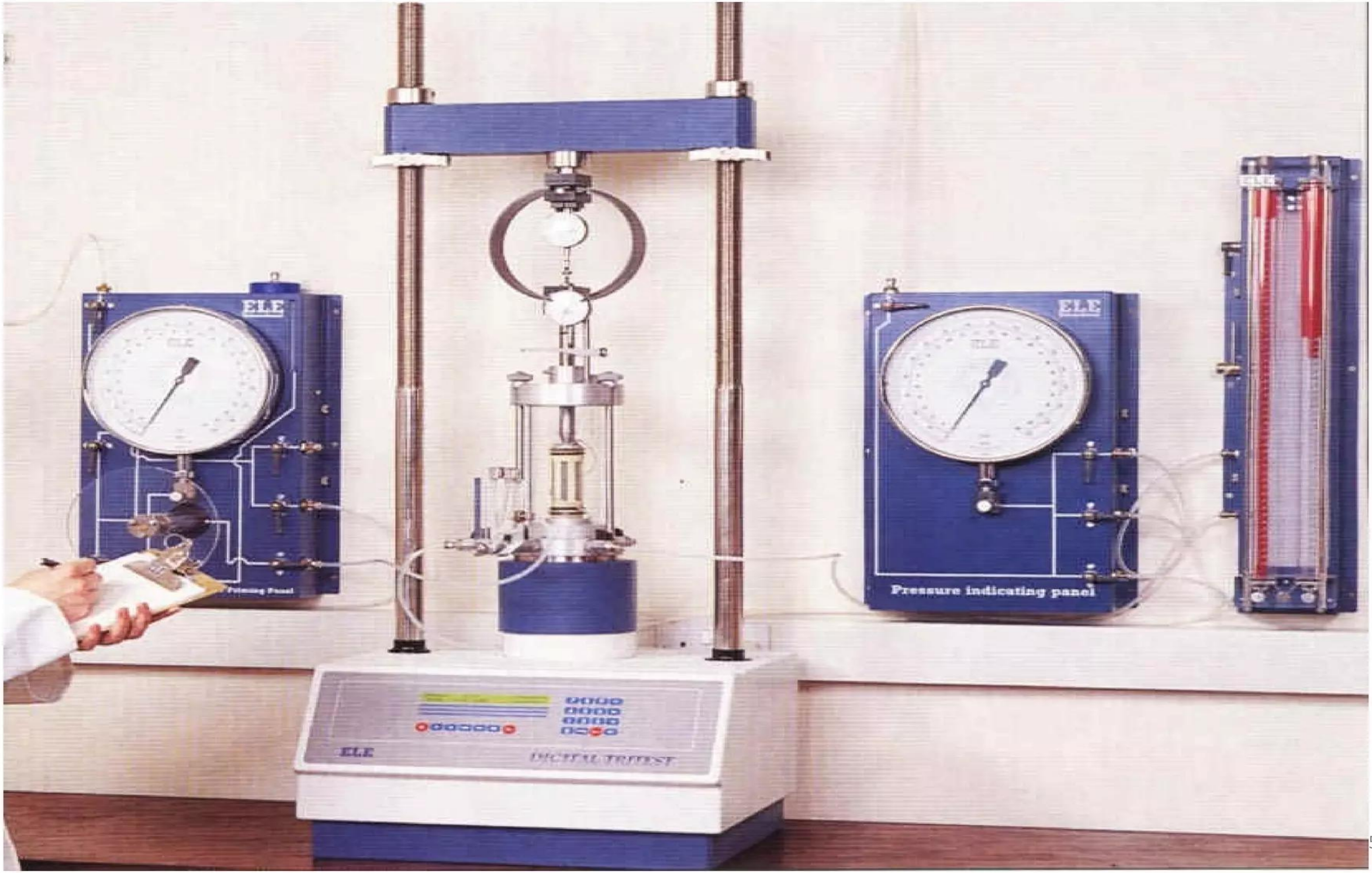
Week 5-7

Pages 50-76

Triaxial Shear Test

Soil sample
at failure




ELE
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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

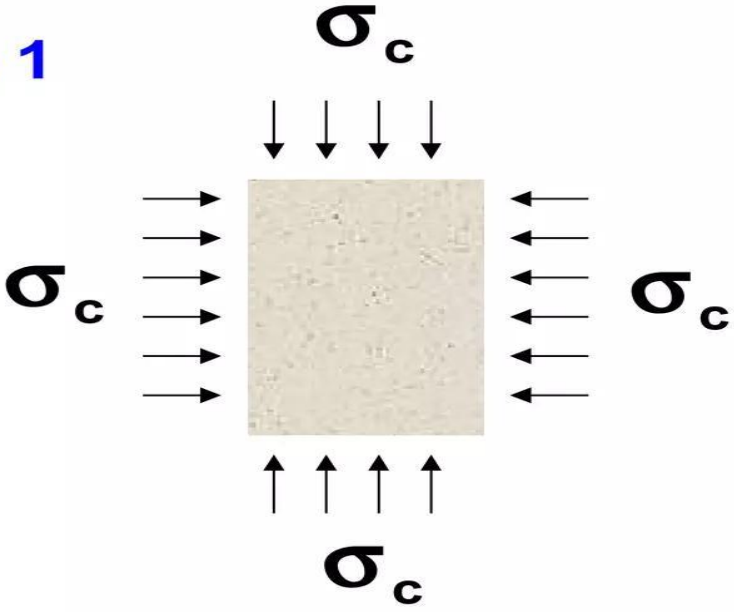
Specimen preparation (undisturbed sample)

Proving ring to measure the deviator load

Dial gauge to measure vertical displacement

Types of Triaxial Tests

Step 1



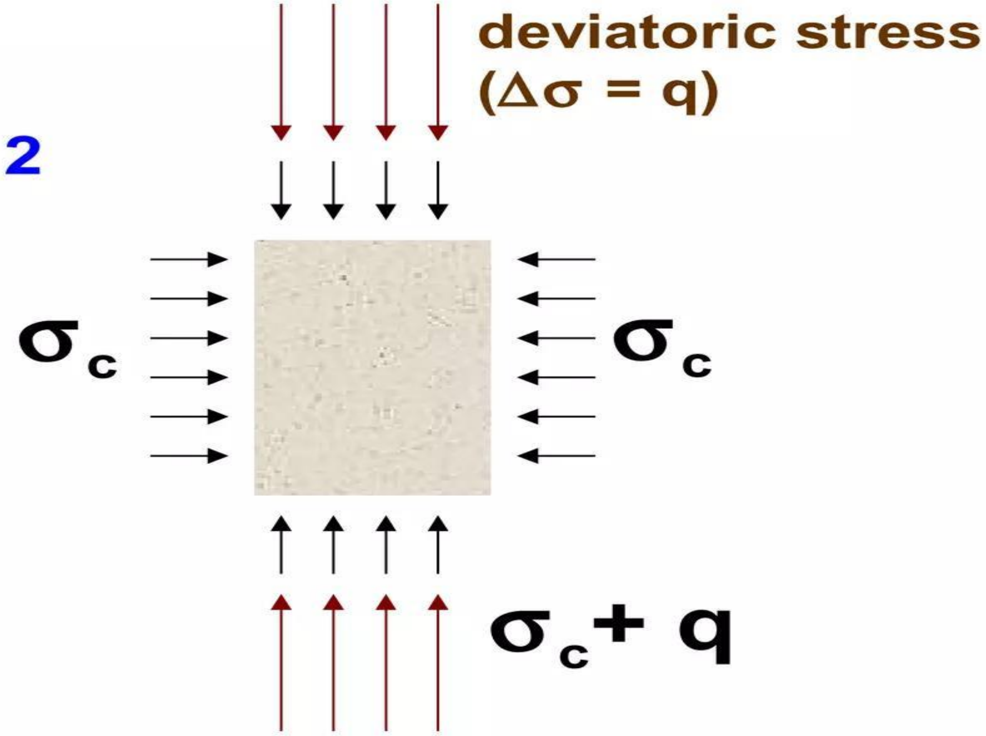
Under all-around cell pressure σ_c

Is the drainage valve open?

yes → **Consolidated sample**

no → **Unconsolidated sample**

Step 2



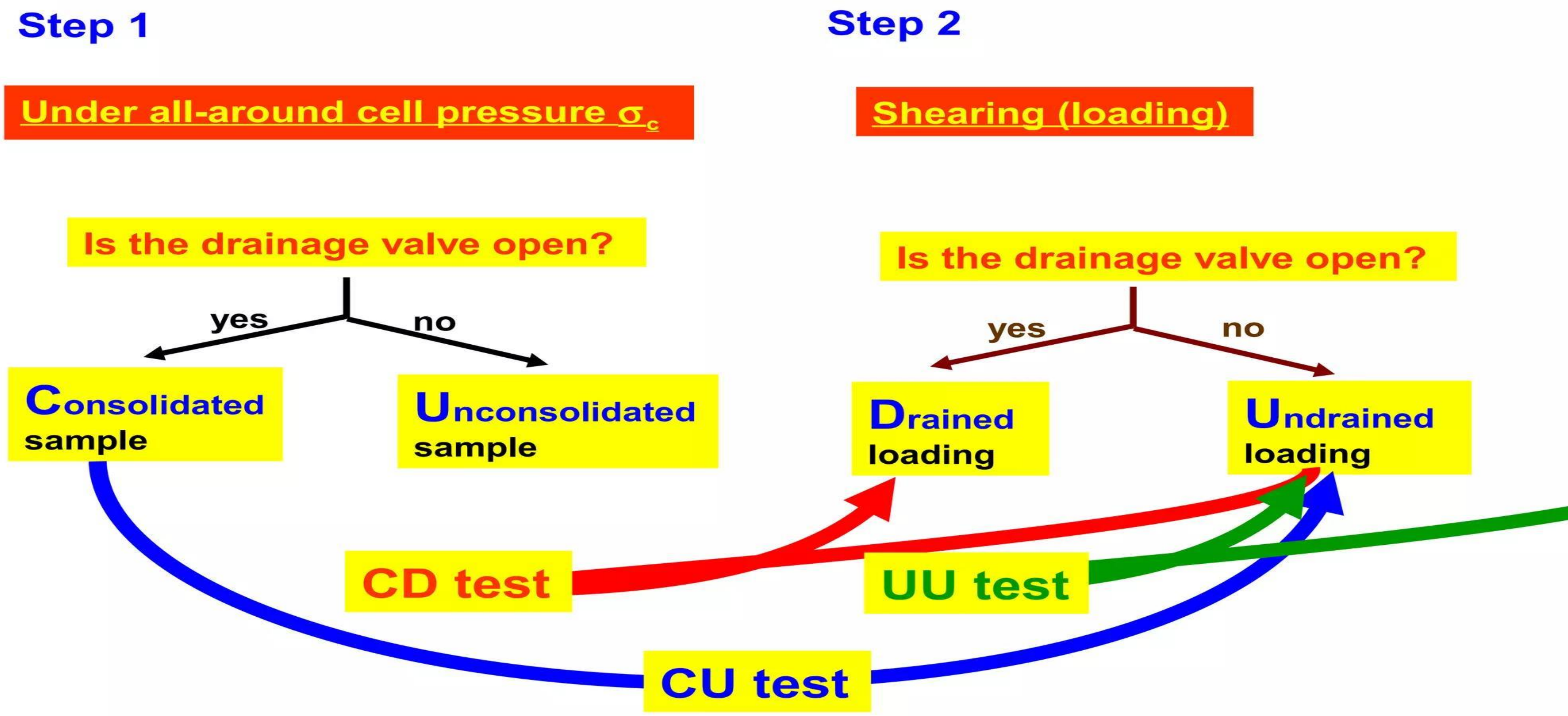
Shearing (loading)

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

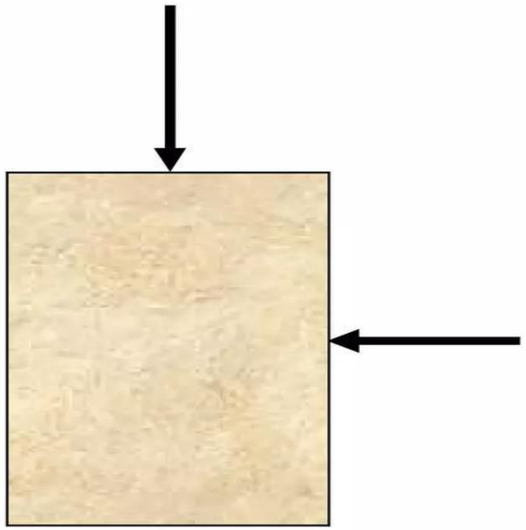
0

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

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

Consolidated- drained test (CD Test)

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

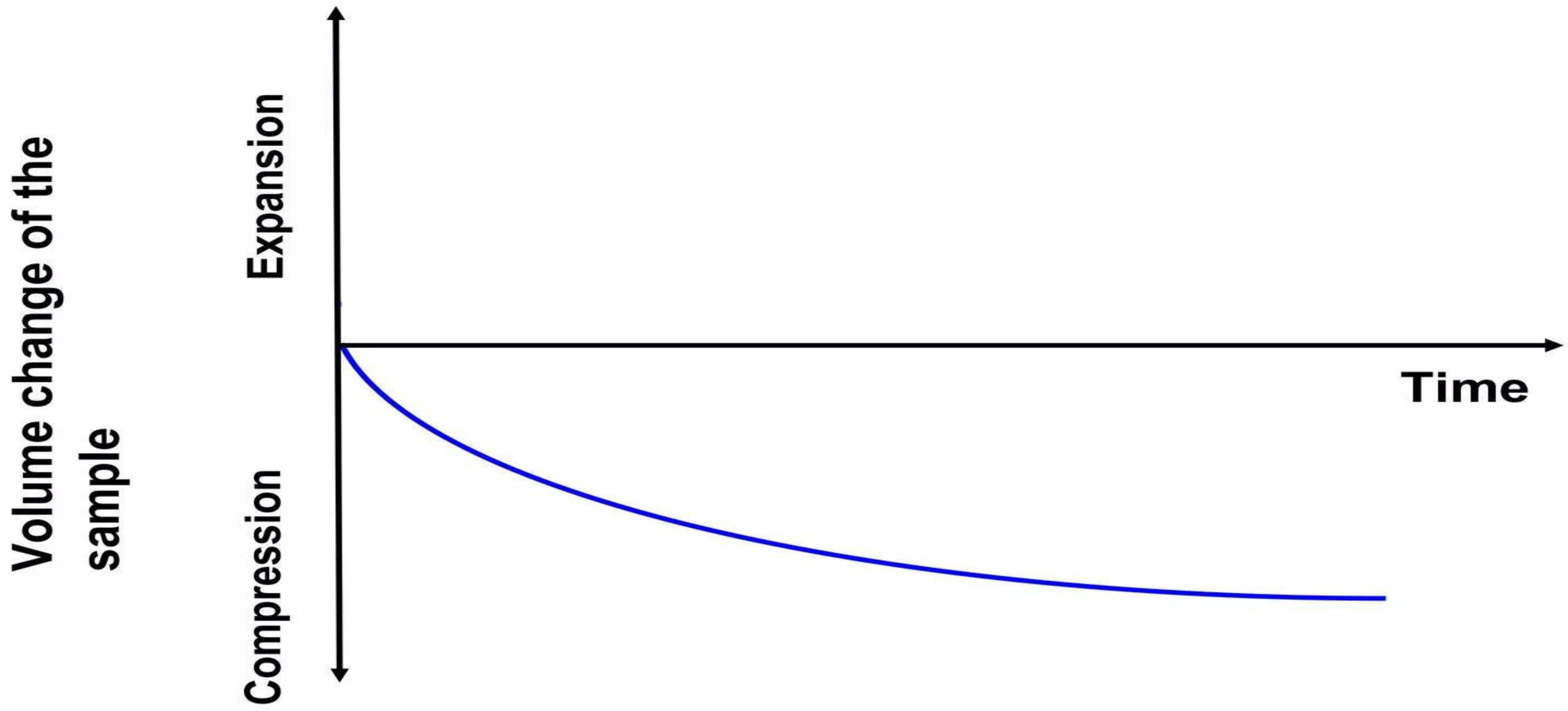


$$\sigma_3 = \sigma_{hC}$$

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

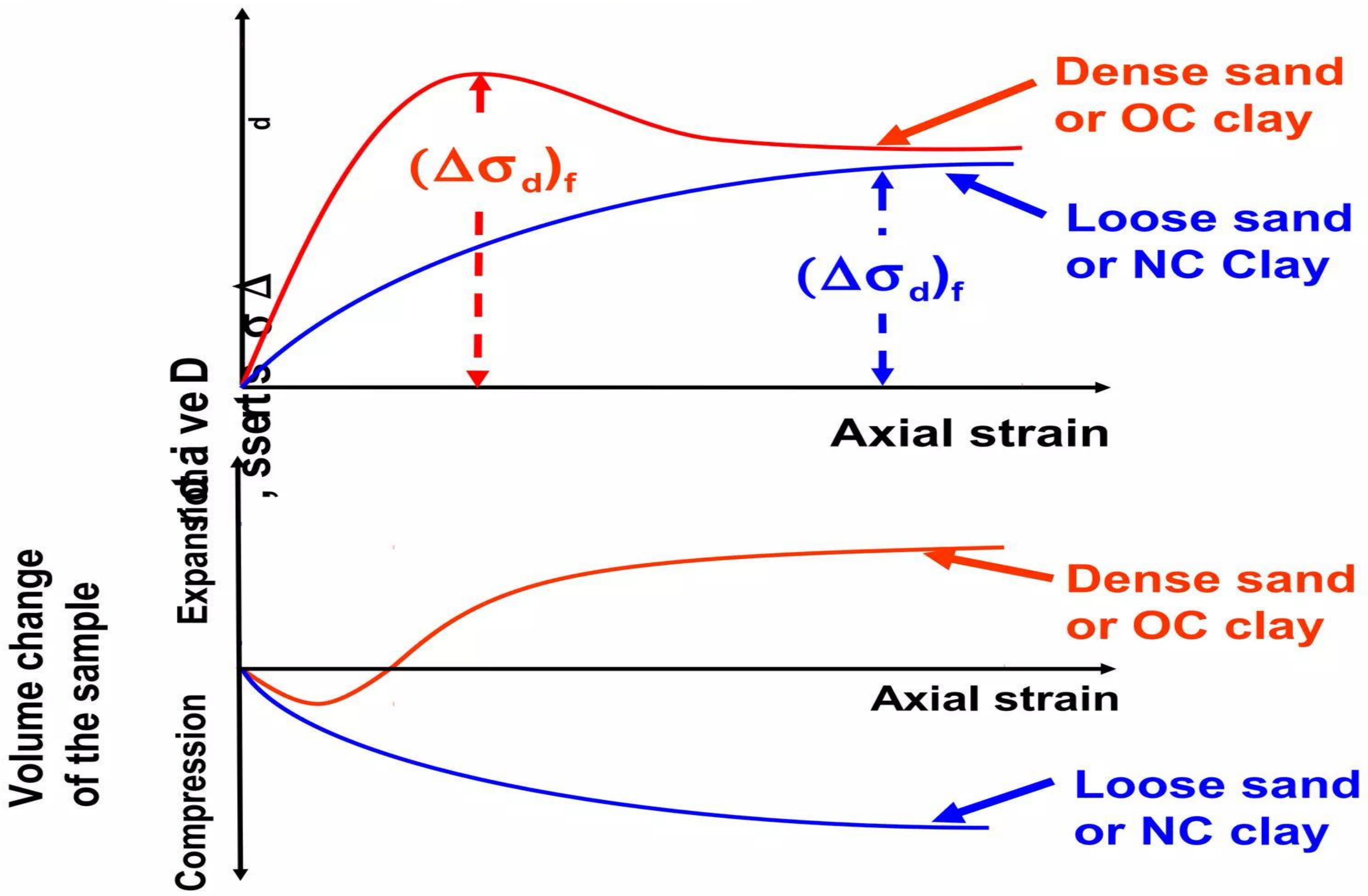
Consolidated- drained test (CD Test)

Volume change of sample during consolidation



Consolidated- drained test (CD Test)

Stress-strain relationship during shearing



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:

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

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.

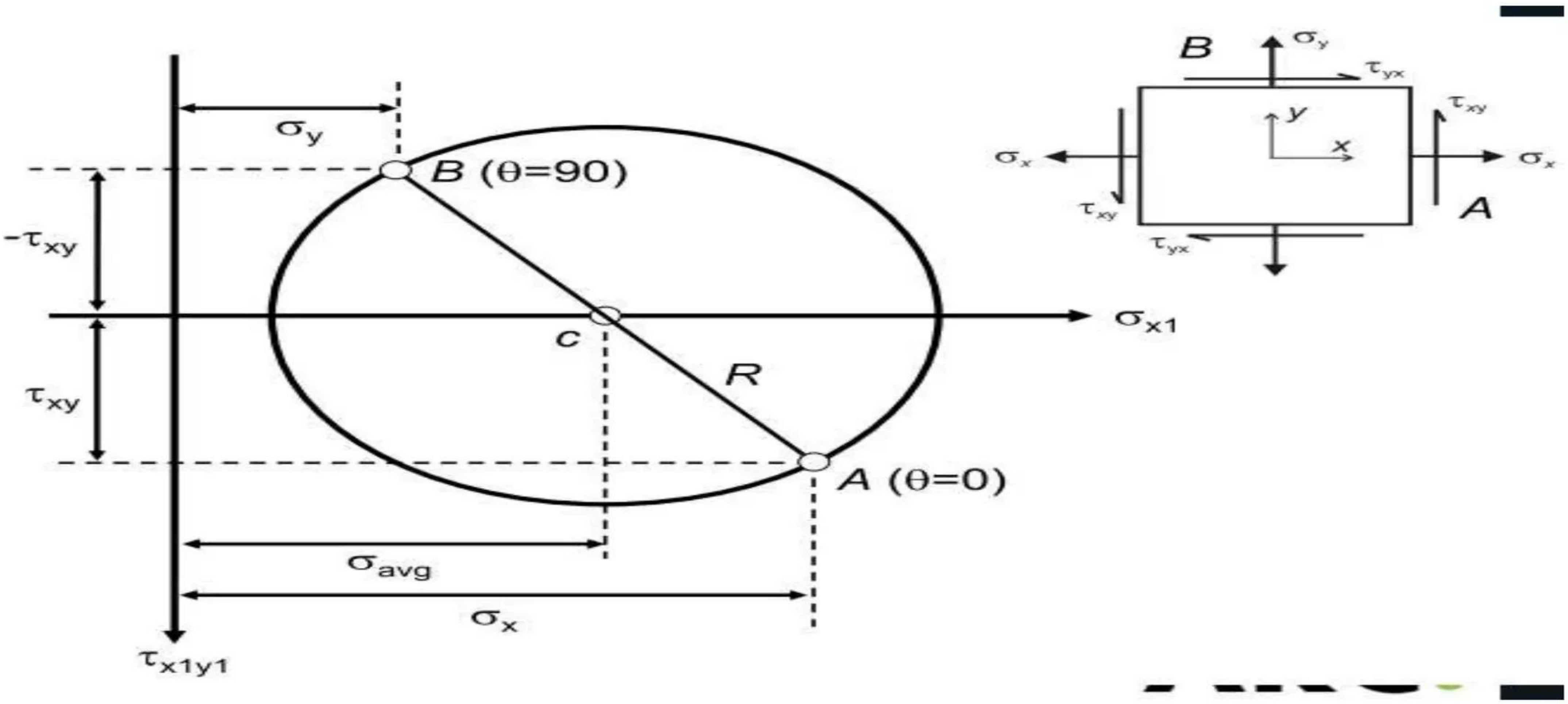
Disadvantages of tri-axial test

- 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



Unconfined Compression Test

Week 8

Pages 78-89

❖ 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

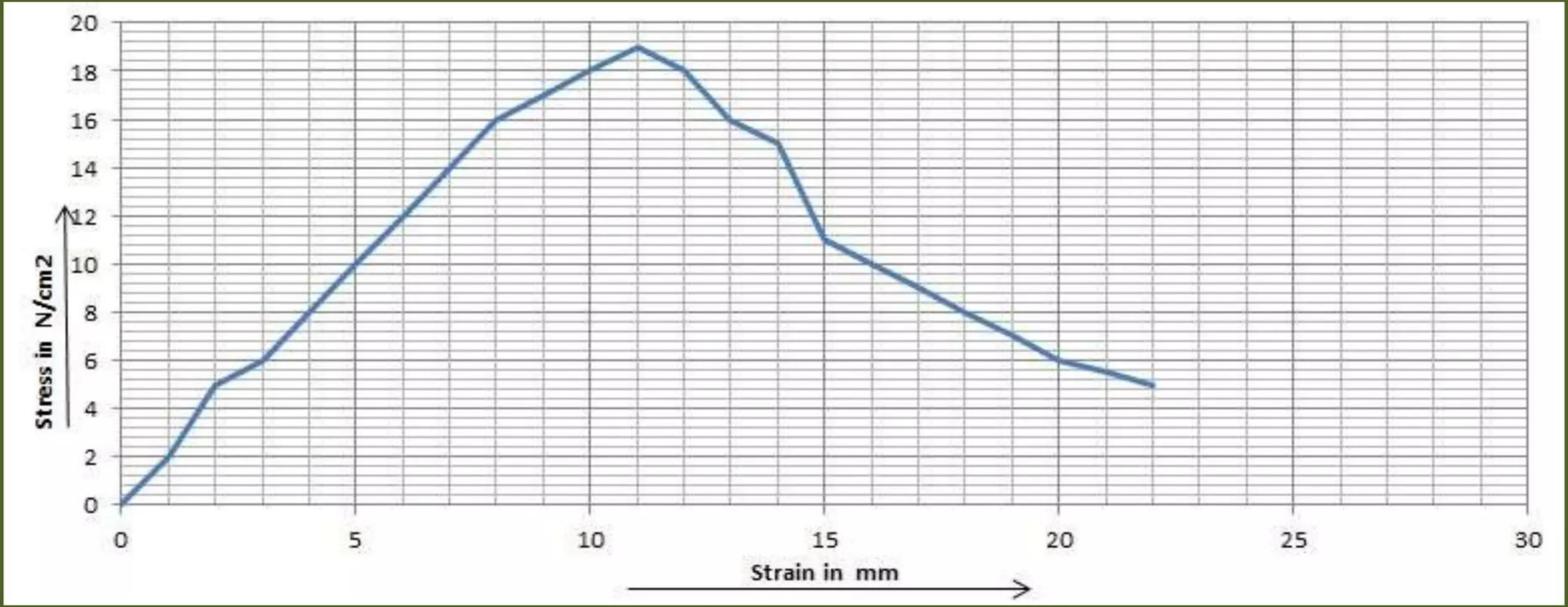
➢ The main question is how to perform unconfined compression test ????

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:- $\emptyset$ = Angle of internal friction

In unconfined compression test, $\sigma_3 = 0$
$\sigma_1 = q_u$

Hence,

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

$\therefore$

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

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

$\therefore$

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

- Shear Strength of the soil is estimated from coulomb's equation :

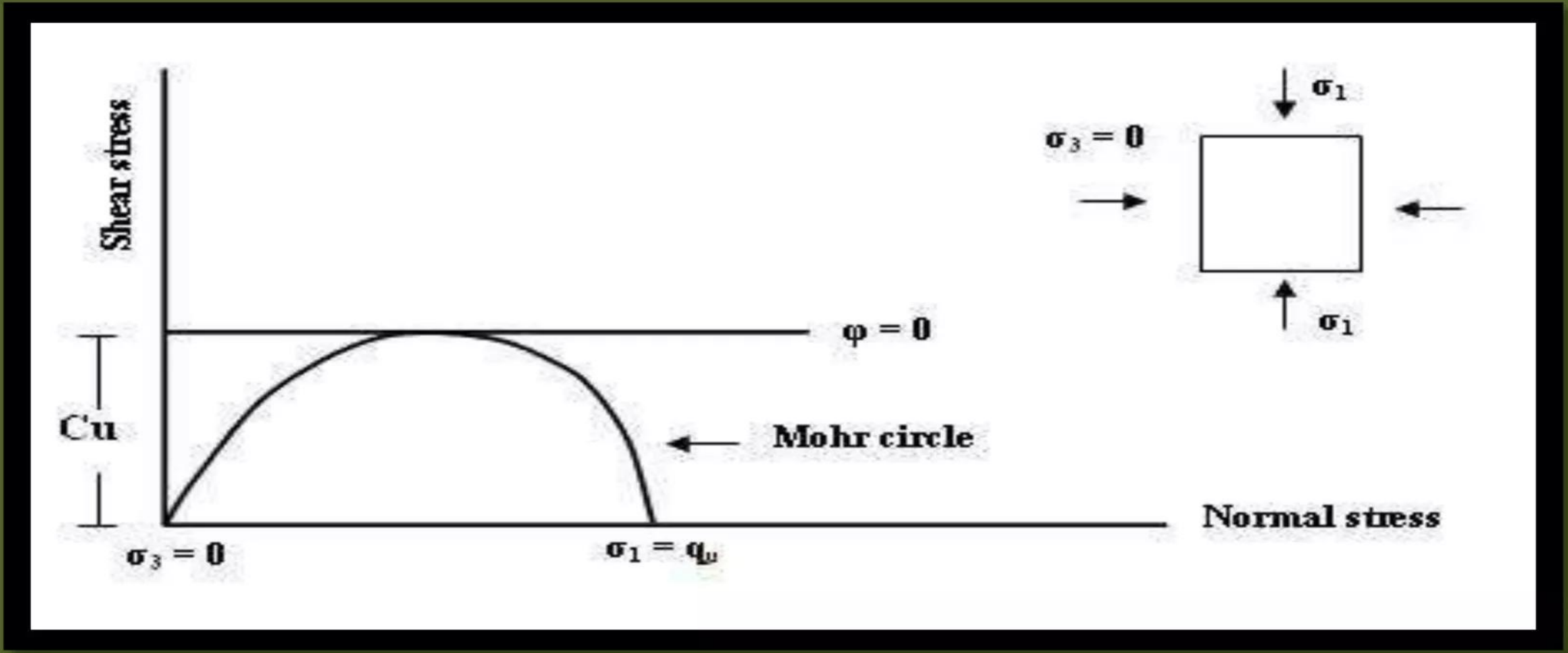
$$\tau f = C + \sigma_{ef} \tan \emptyset$$

Where:- τf = Shear strength
σ_{ef} = Effective normal stress

If Ø = 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.



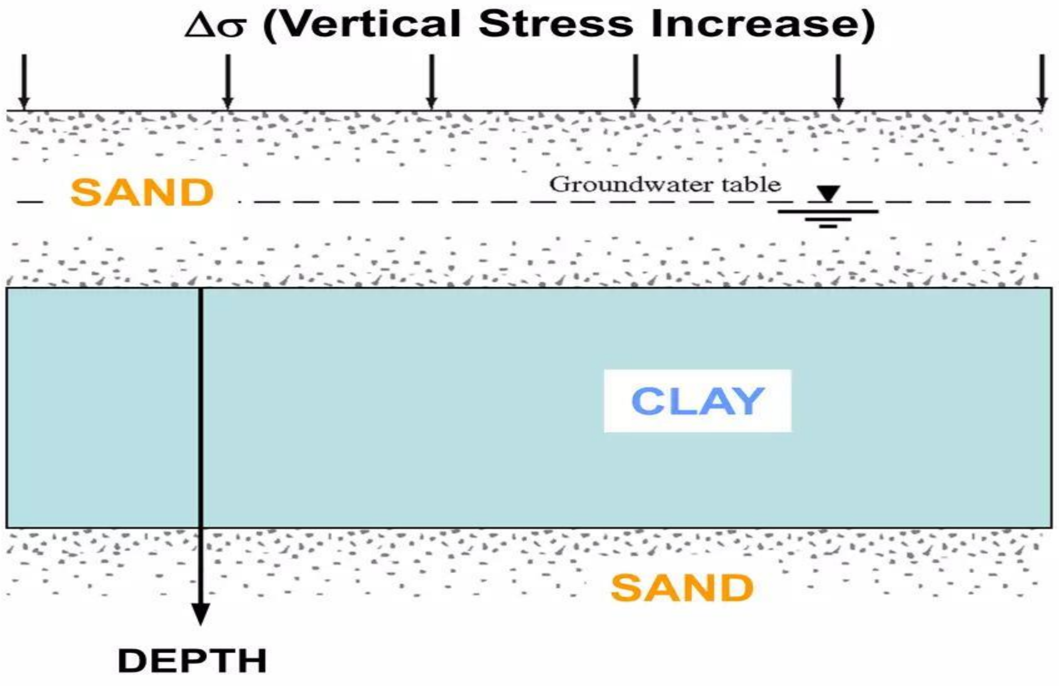
Mohr Circle for Unconfined Compression Test

Consolidation Basic/ Settlement Formula

Week 9-10

Pages 91-165

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.

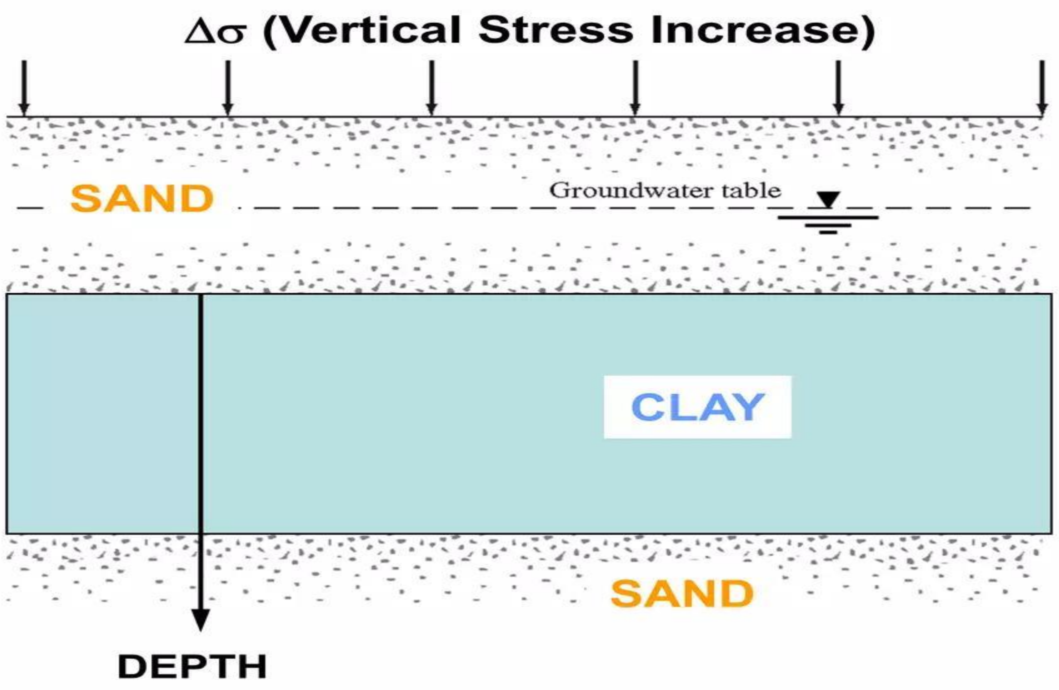
Saturated Soils:

$\Delta\sigma$ **causes** ***u*** **to increase immediately**

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

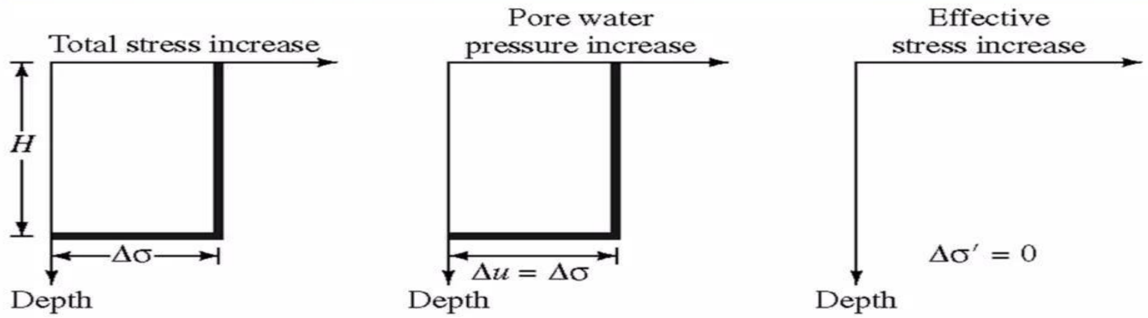
Clays: Pore Pressure dissipates slowly due to low permeability.

FUNDAMENTALS OF CONSOLIDATION



after Figure 7.1a. Das FGE (2005).

At Time of Initial Loading (t = 0)

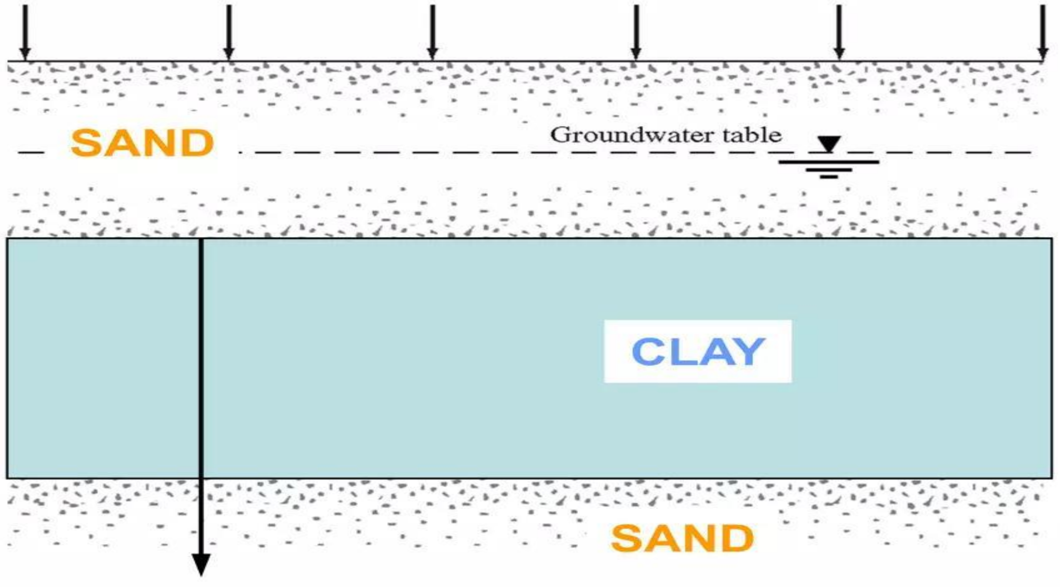


Variation in Total, Pore water, and Effective Stresses in Clay Layer
Figure 7.1b. Das FGE (2005)

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

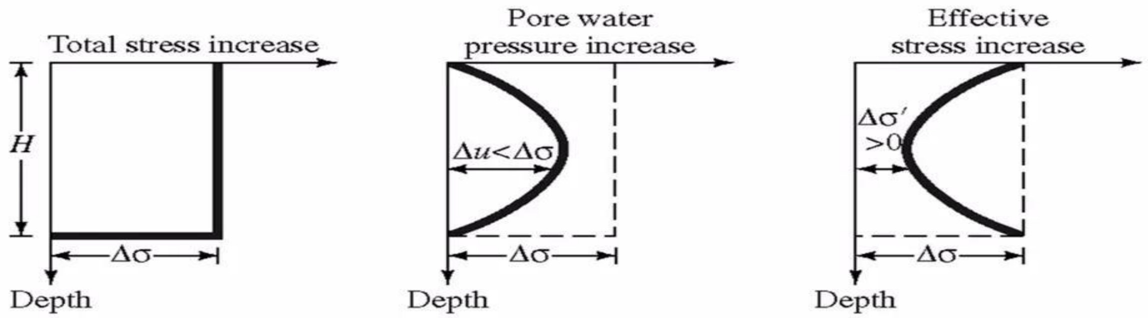
Soil skeleton does not see initial loading

Fundamentals of Consolidation



after Figure 7.1a. Das FGE (2005).

Between time t = 0 to t = ∞

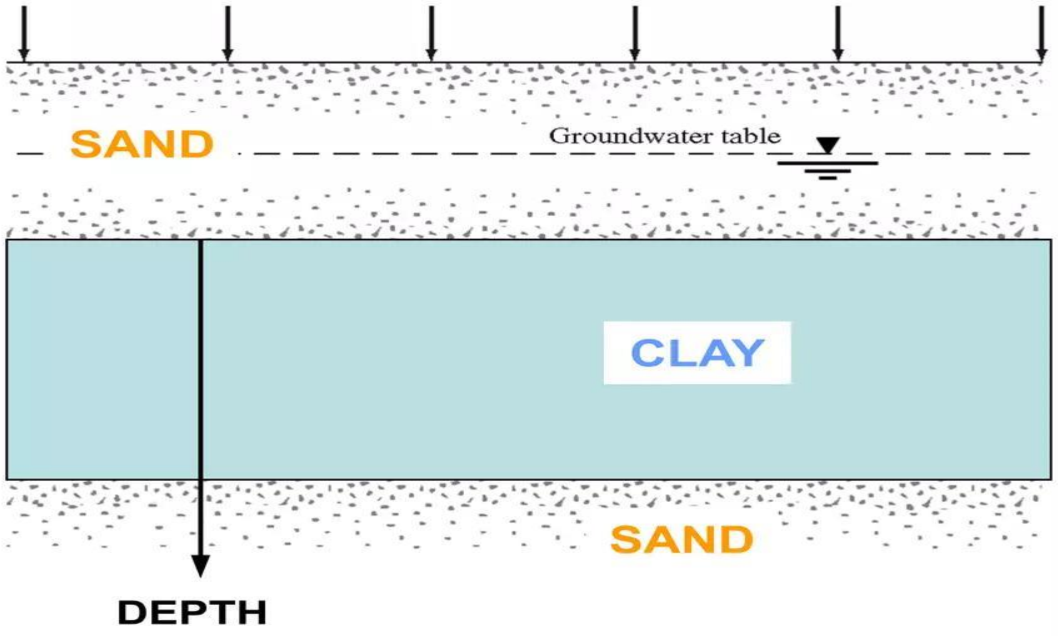


Variation in Total, Pore water, and Effective Stresses in Clay Layer
Figure 7.1c. Das FGE (2005)

Pore water increase due to initial loading dissipates

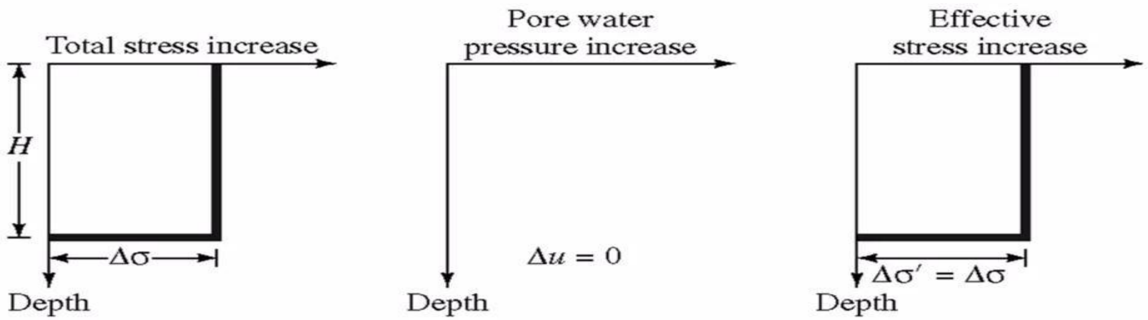
Soil skeleton takes loading as pore pressure decreases

FUNDAMENTALS OF CONSOLIDATION



after Figure 7.1a. Das FGE (2005).

At time t = ∞



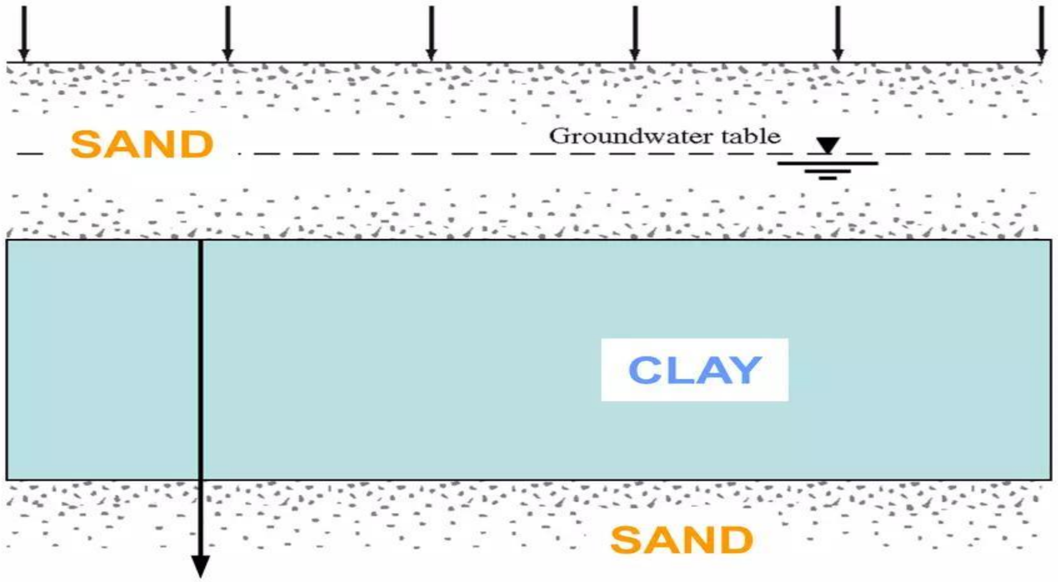
Variation in Total, Pore water, and Effective Stresses in Clay Layer
Figure 7.1e. Das FGE (2005)

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 ($\Delta\sigma = \Delta'\sigma$)

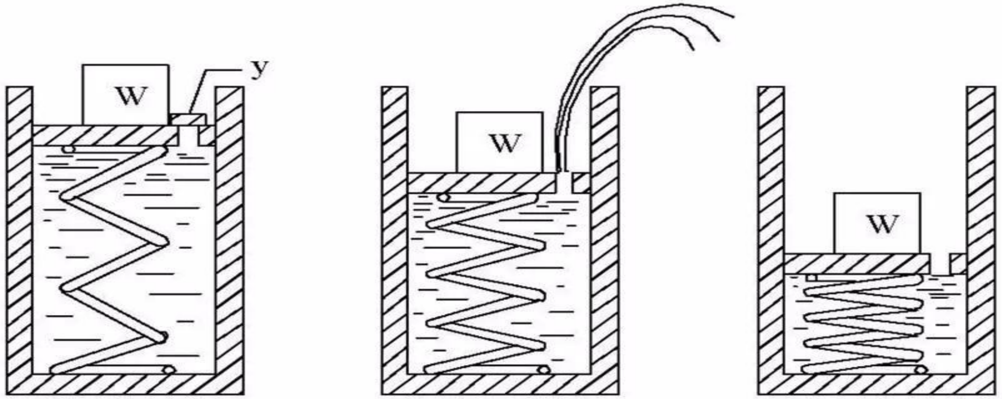
FUNDAMENTALS OF CONSOLIDATION

$\Delta\sigma$ (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

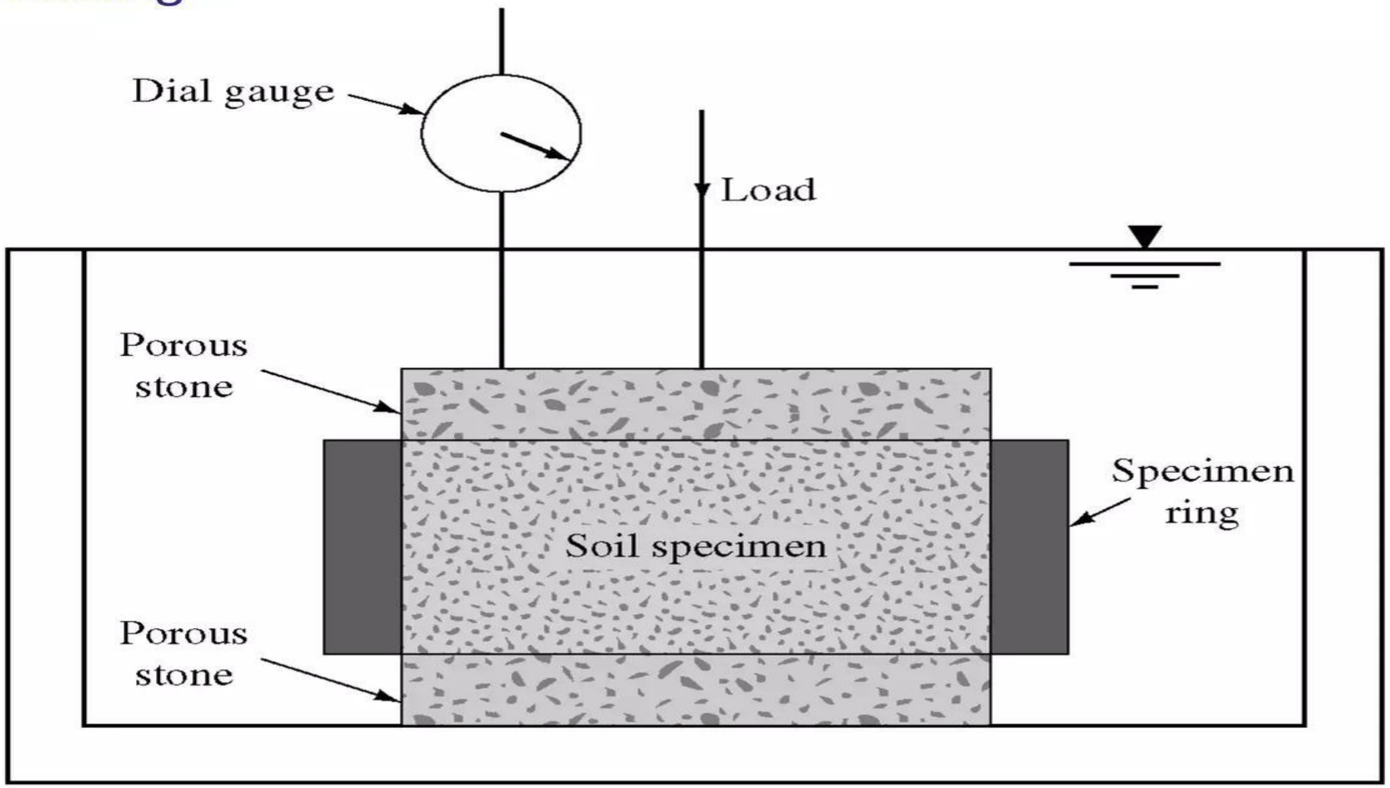
(c) Final Loading

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

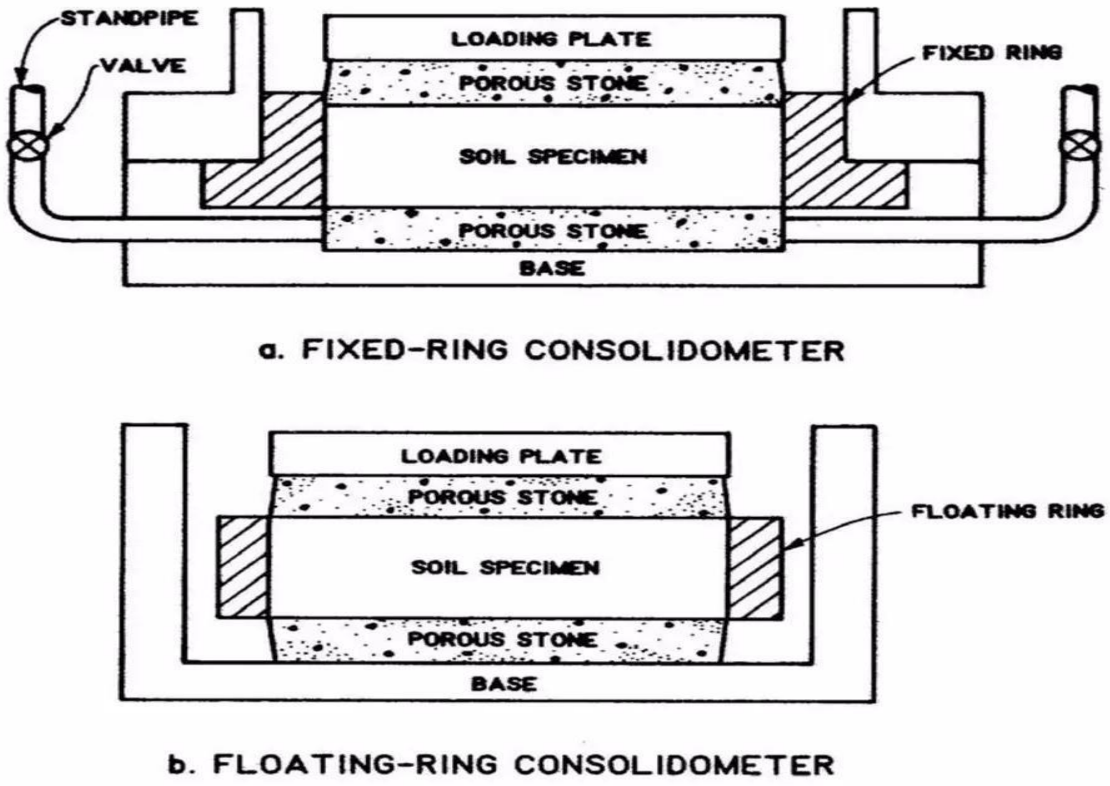


Figure E-1 USACE EM1110-1-1904.

ShearTrac II DSS Equipment
(Courtesy of Geocomp Corporation)

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.

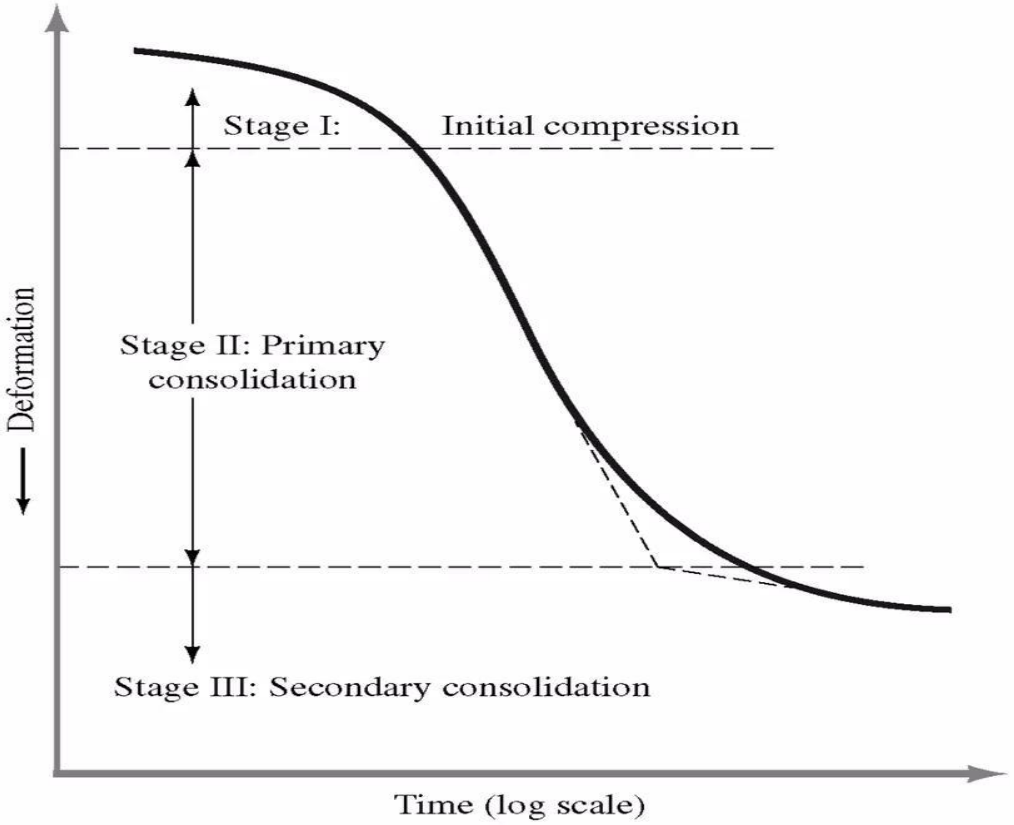


Figure 7.4. Das FGE (2005).

Void Ratio-Pressure Plots

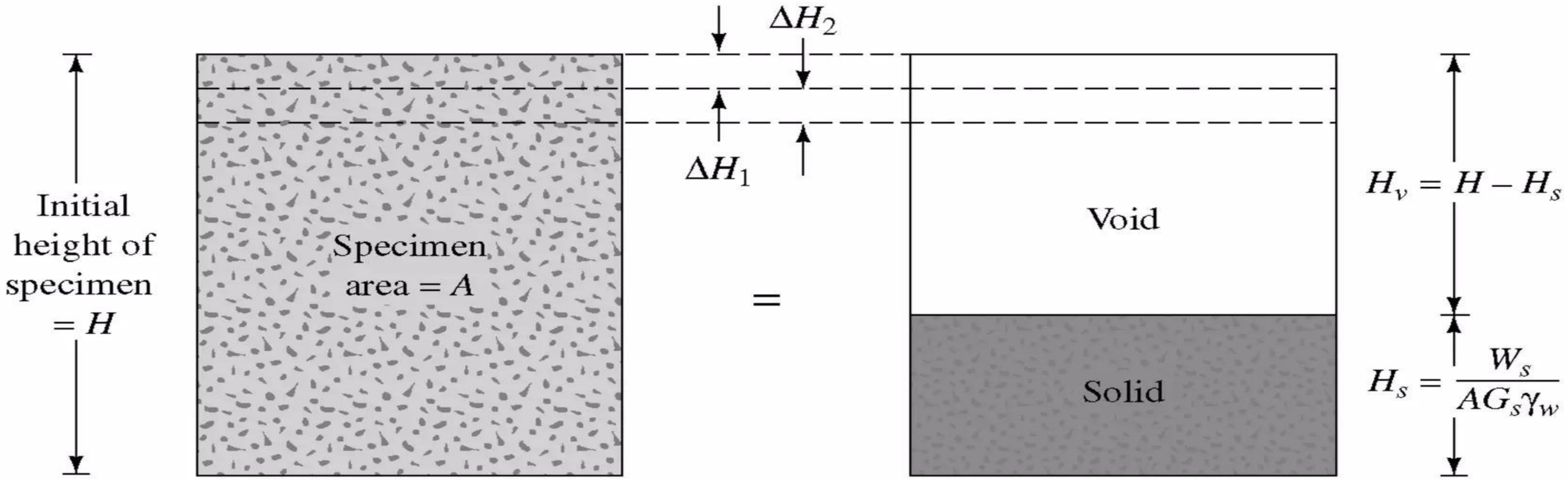


Figure 7.5. Das FGE (2005)

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

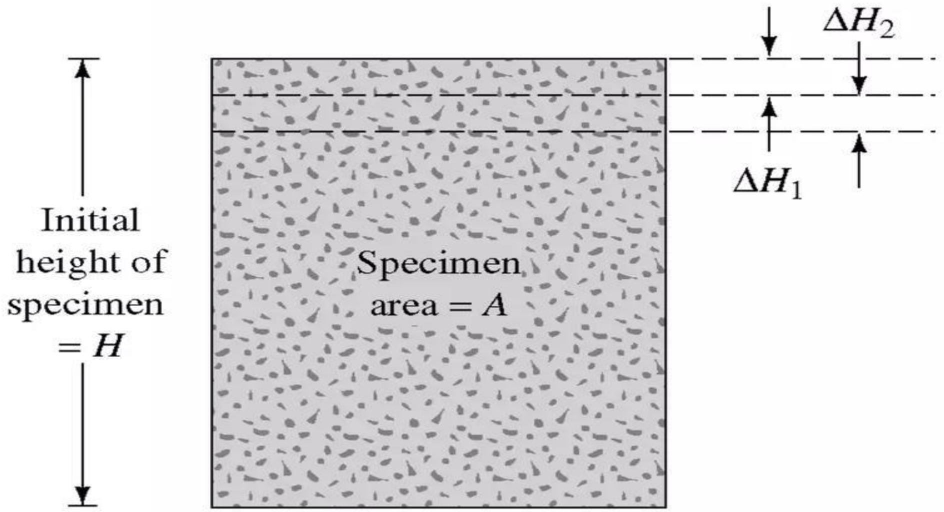


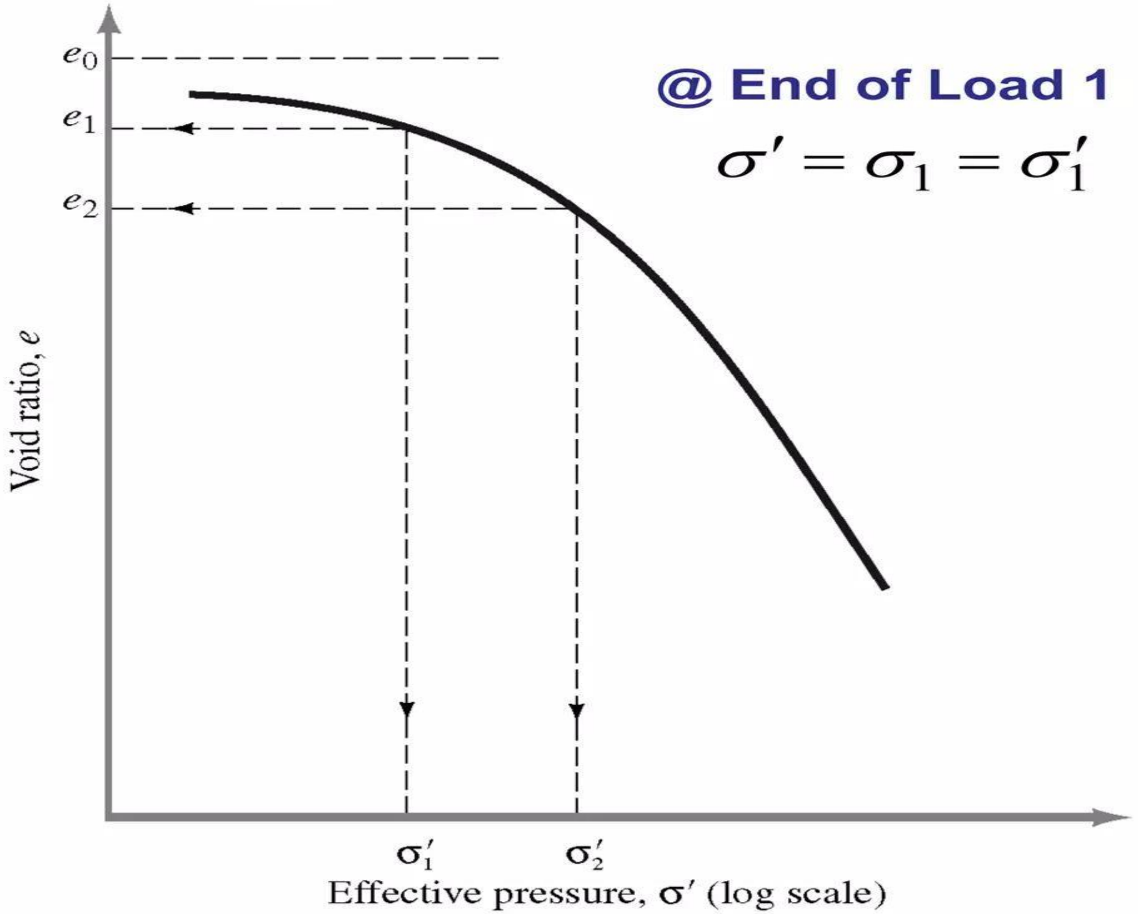
Figure 7.5. Das FGE (2005)

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

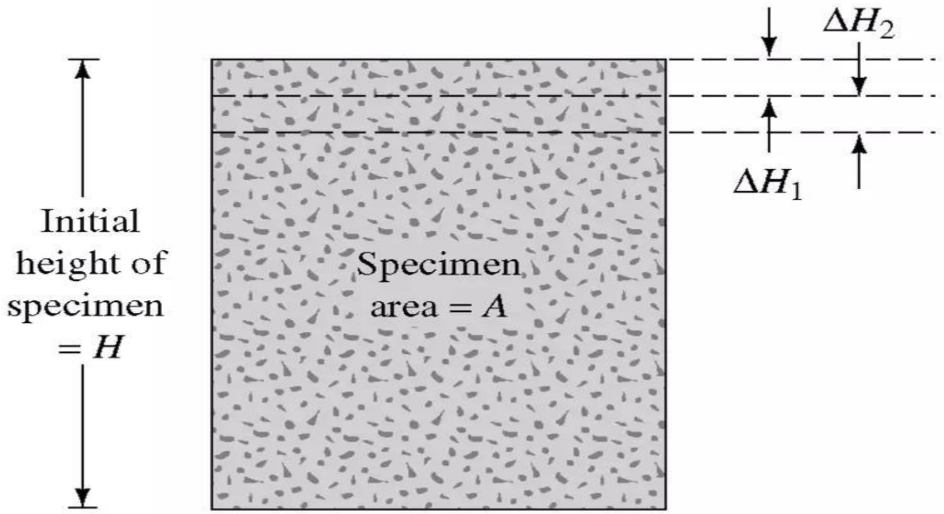


Figure 7.5. Das FGE (2005)

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

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

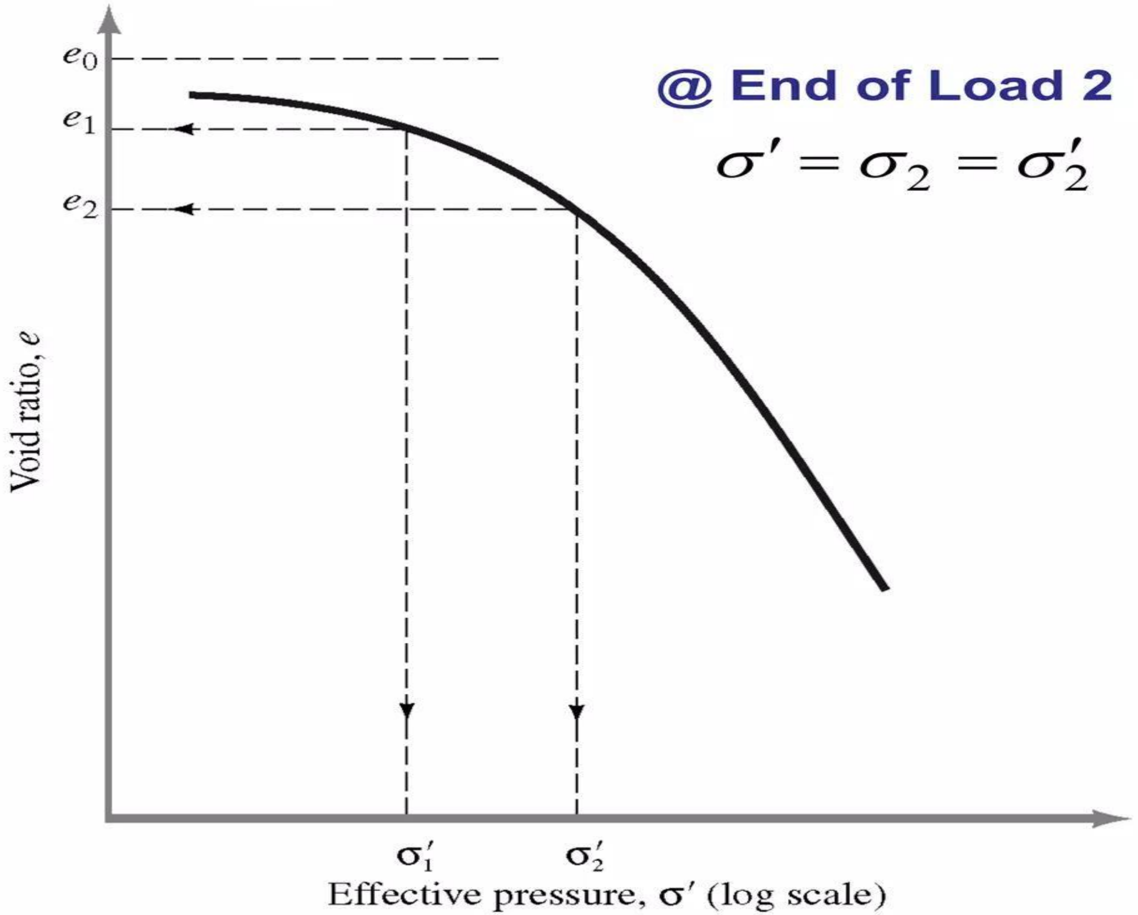


Figure 7.6. Das FGE (2005)

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})

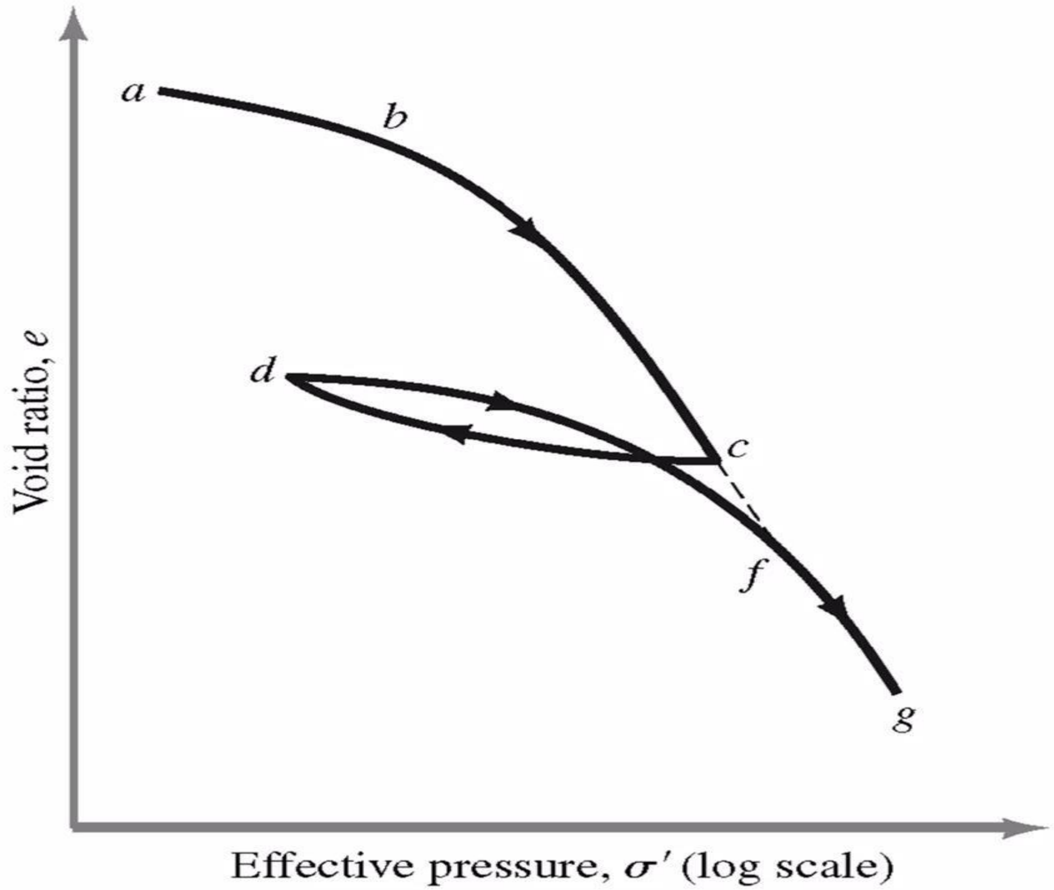


Figure 7.7. Das FGE (2005).

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

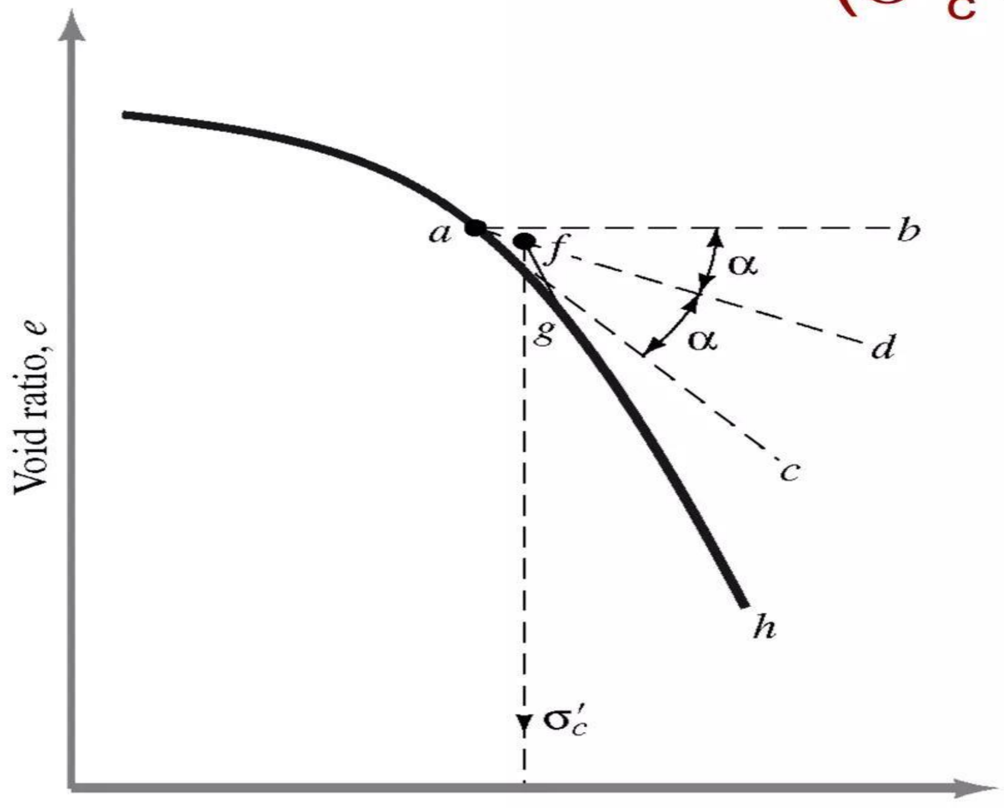


Figure 7.8. Das FGE (2005).

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)

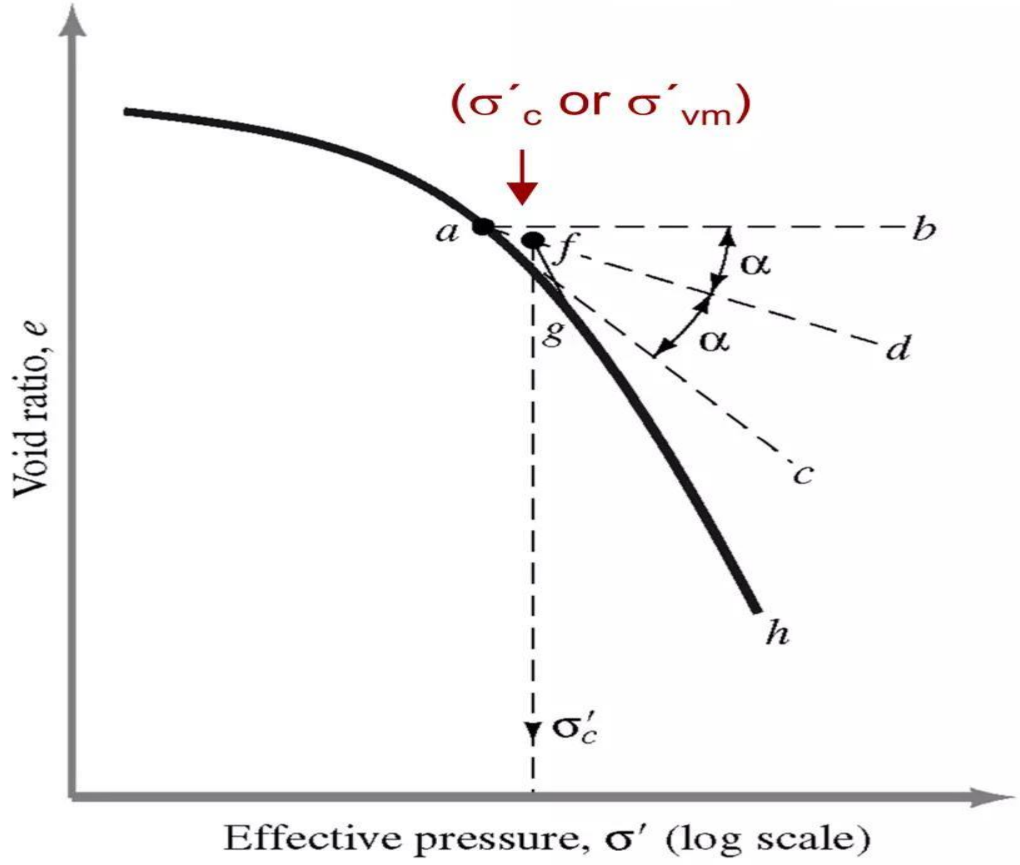


Figure 7.8. Das FGE (2005).

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

Where:

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

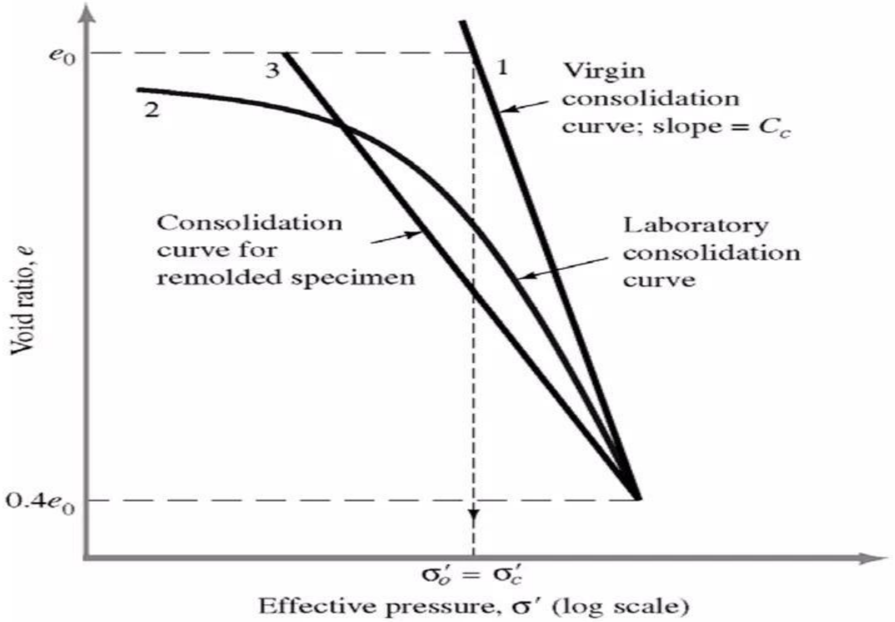
σ' = Present Effective Vertical Stress

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

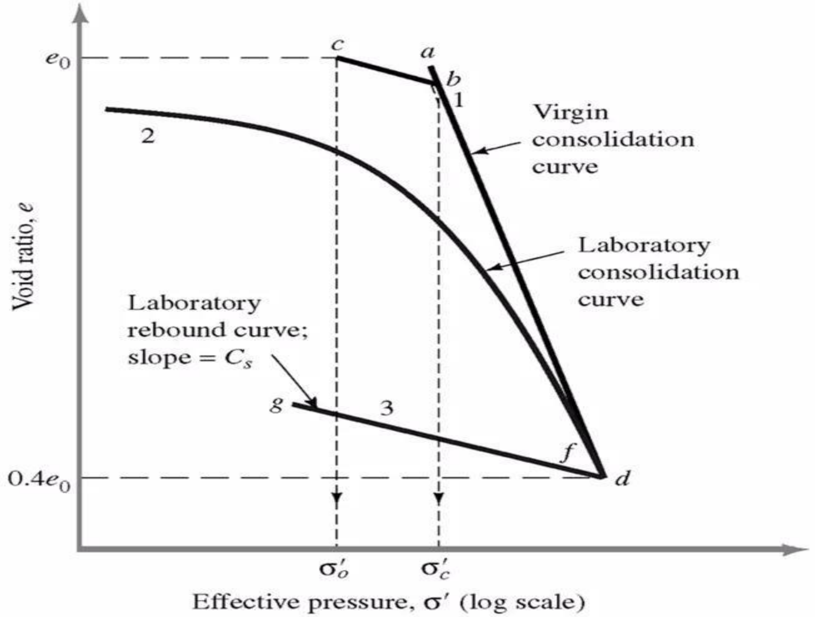
Possible Causes of OC Soils:
Preloading (thick sediments, glacial ice); fluctuations of GWT, underdraining, light ice/snow loads, desiccation above GWT, secondary compression.

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

Where q_u = Unconfined Compressive Strength

EFFECTS OF SAMPLE DISTURBANCE

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

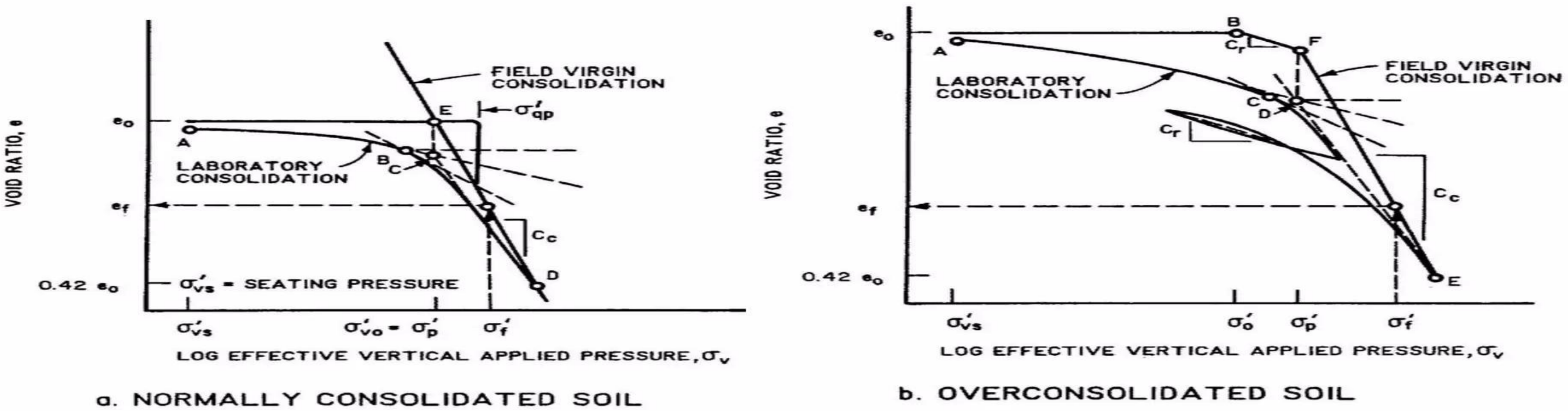


Figure 3-12. EM 1110-1-1904 Settlement Analysis.

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 consolidaation 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 $\Delta\sigma = \Delta\sigma'$

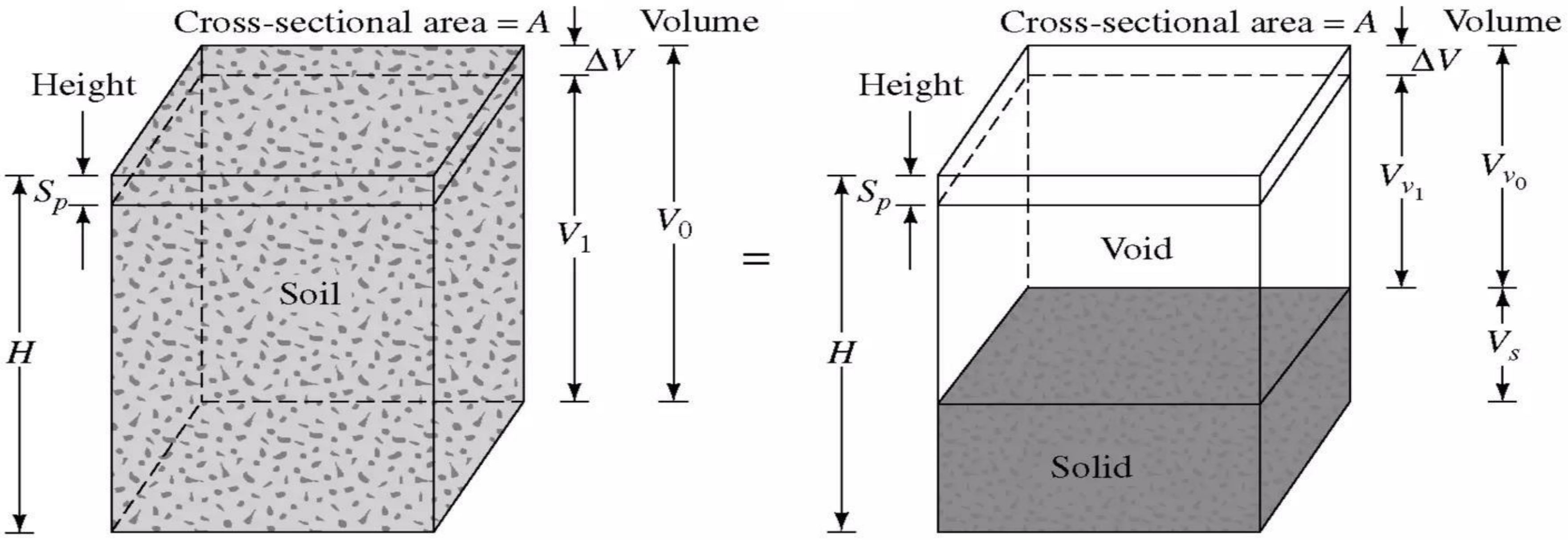


Figure 7.11. Das FGE (2005)

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

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 $\Delta\sigma = \Delta\sigma'$

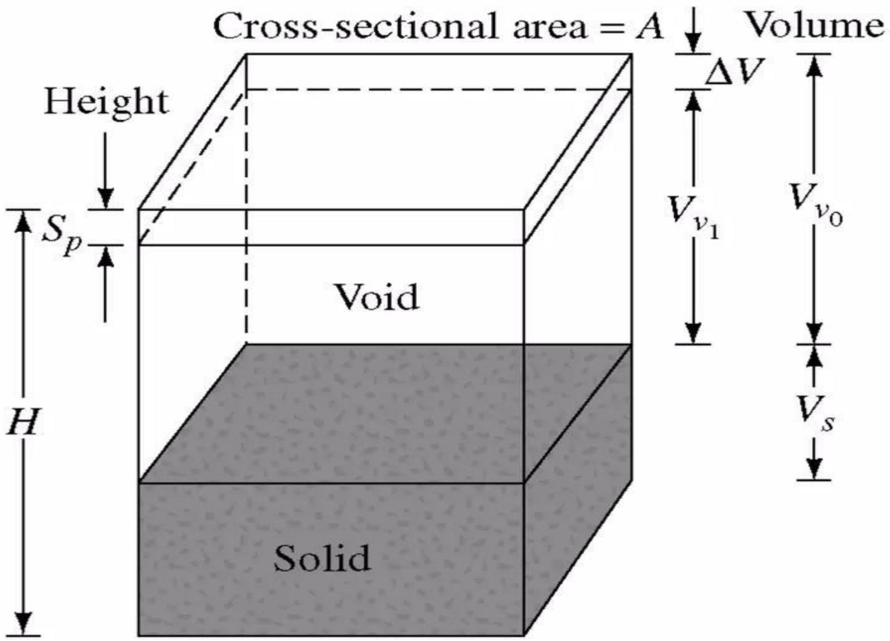


Figure 7.11. Das FGE (2005).

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

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 $\Delta\sigma = \Delta\sigma'$

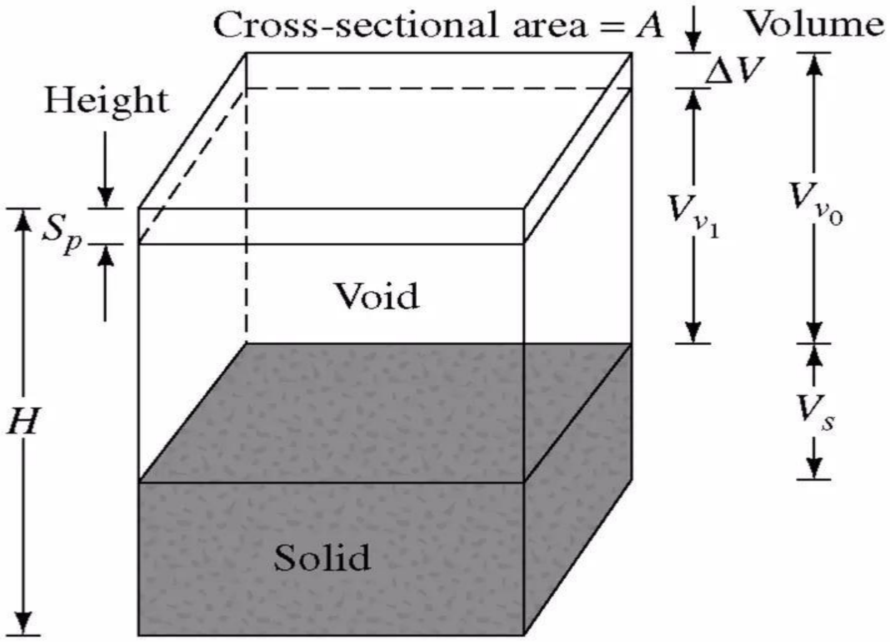


Figure 7.11. Das FGE (2005).

Therefore:

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

or

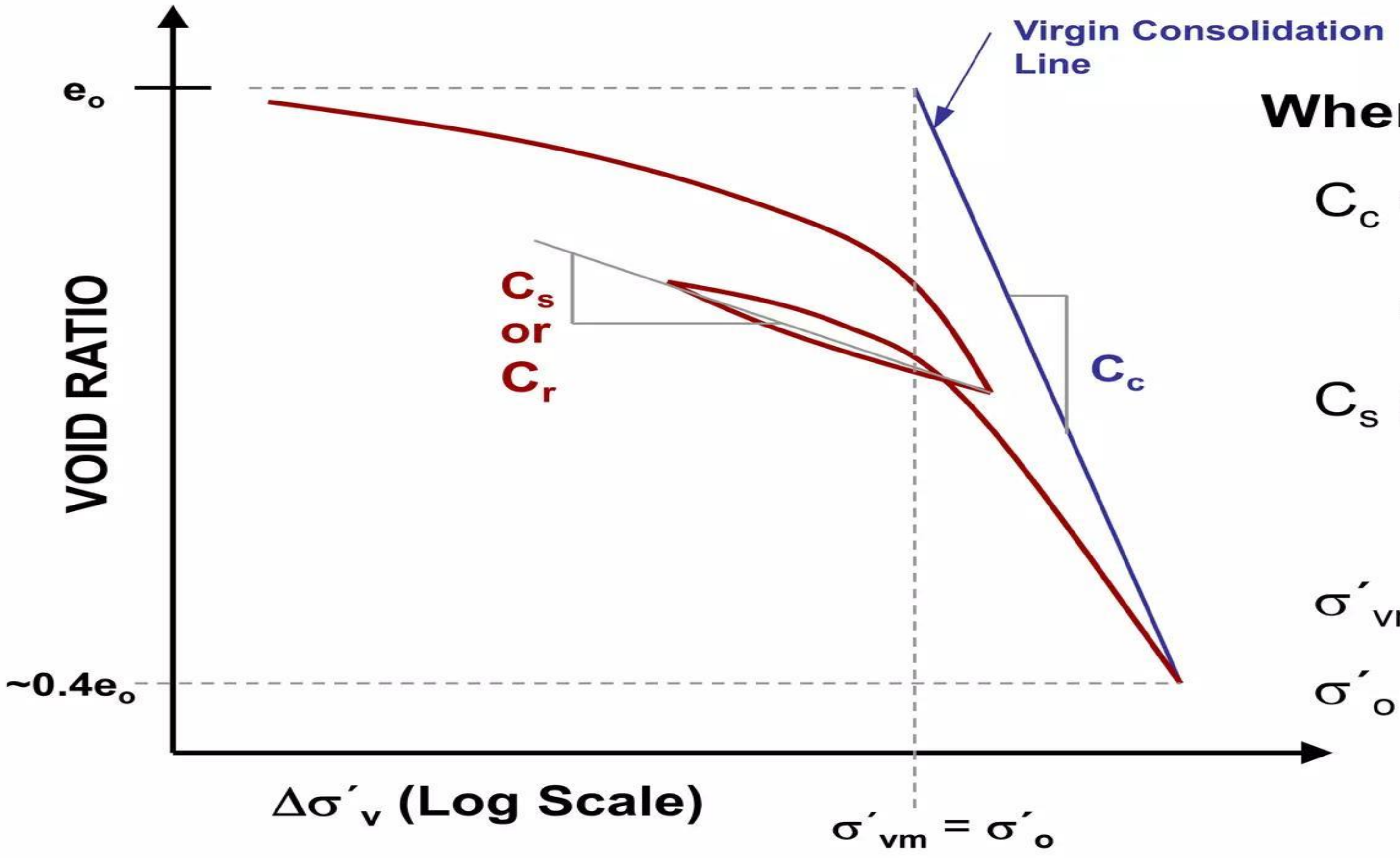
$$S_p = H\frac{\Delta e}{1+e_o}$$

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

Where:

ε_v = Vertical Strain

SETTLEMENT FROM 1D PRIMARY CONSOLIDATION



NC Clay

Where:

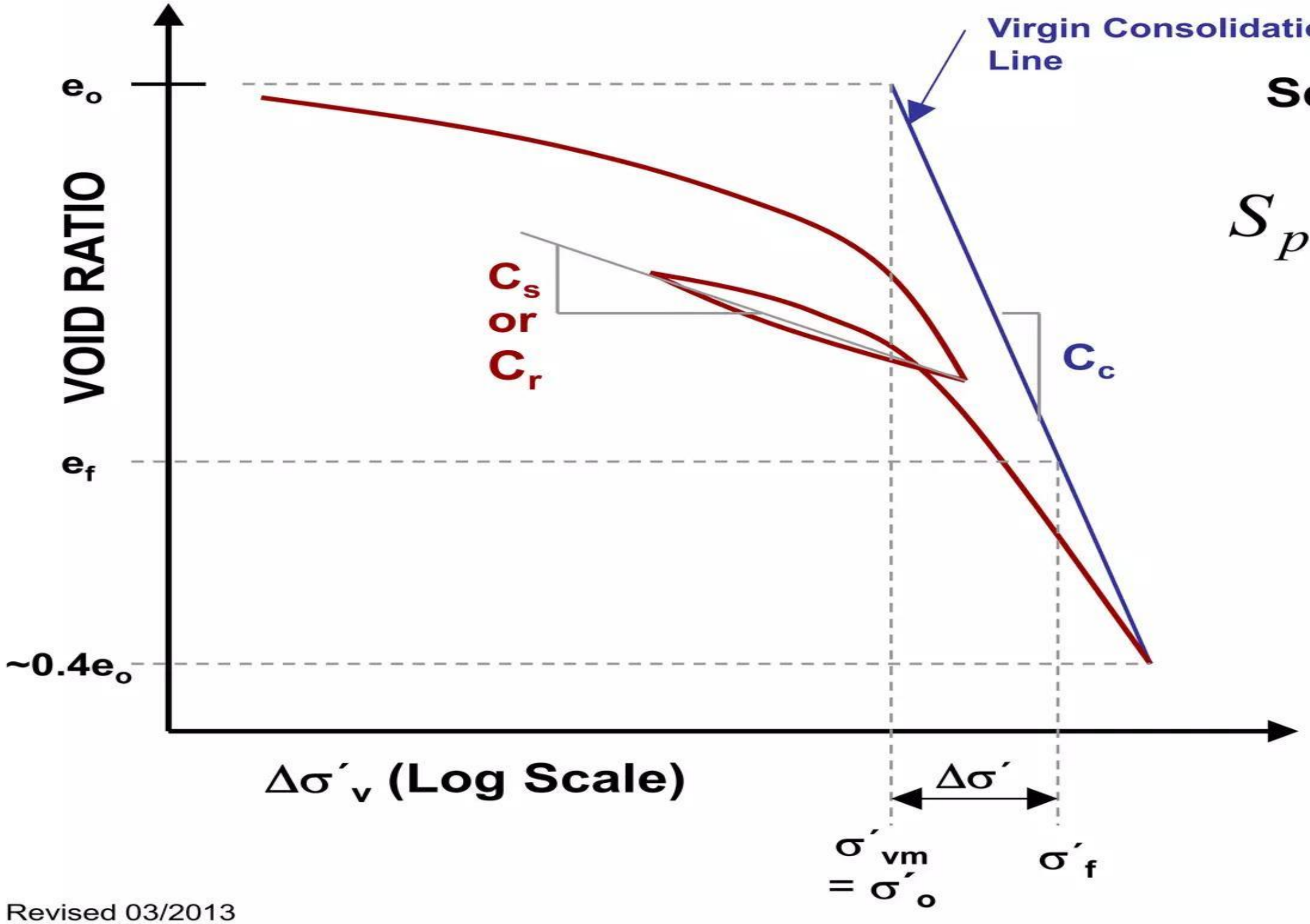
C_c = Slope of Field Virgin Consolidation Curve
= **Compression Index**

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

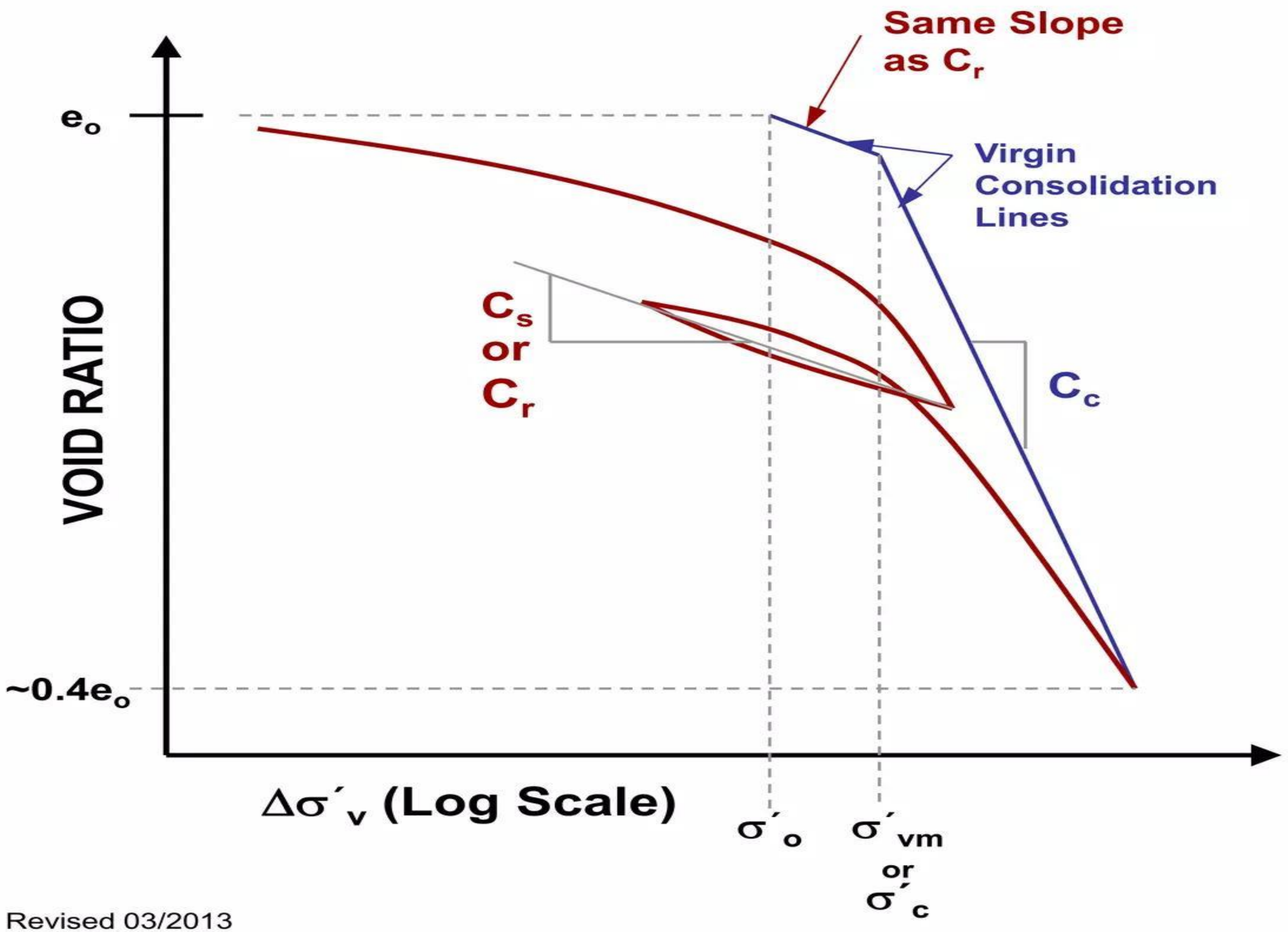
Settlement (S_p) using Void Ratio

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

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:

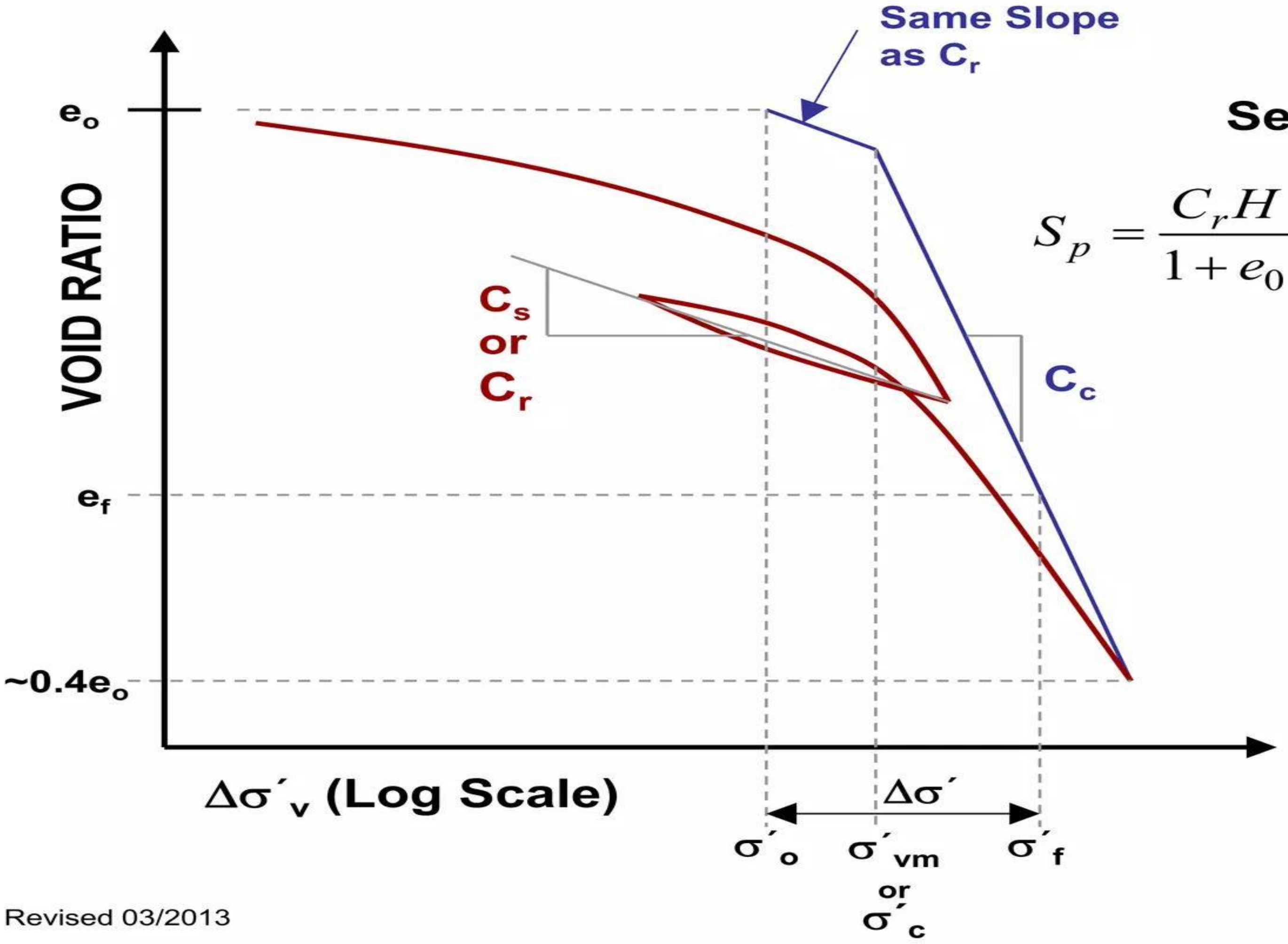
C_c = Slope of Field Virgin Consolidation Curve = **Compression Index**

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

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

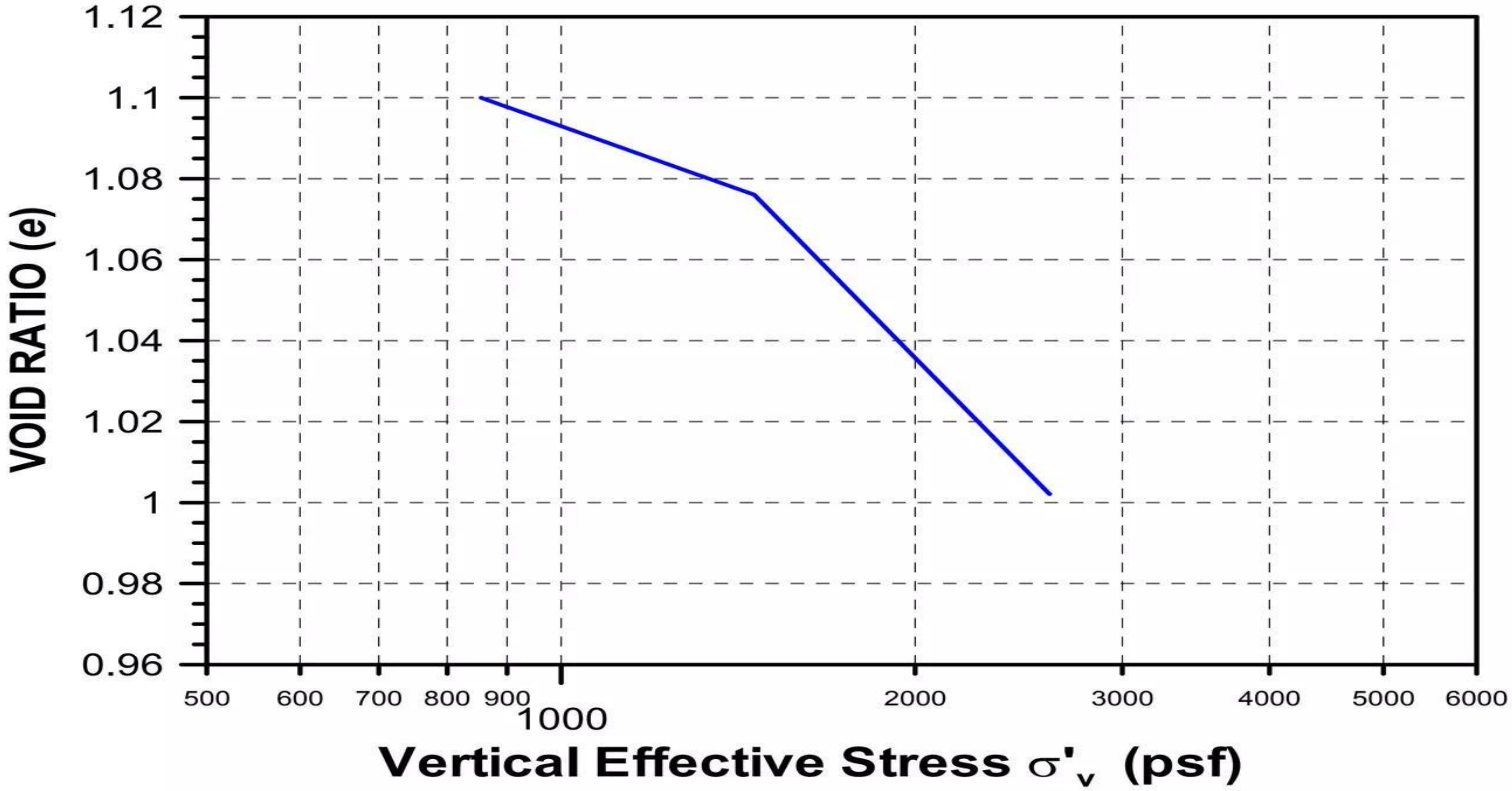


Figure 1. Example of Virgin Consolidation Curves.

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

Example: Settlement from Virgin Consolidation Curves

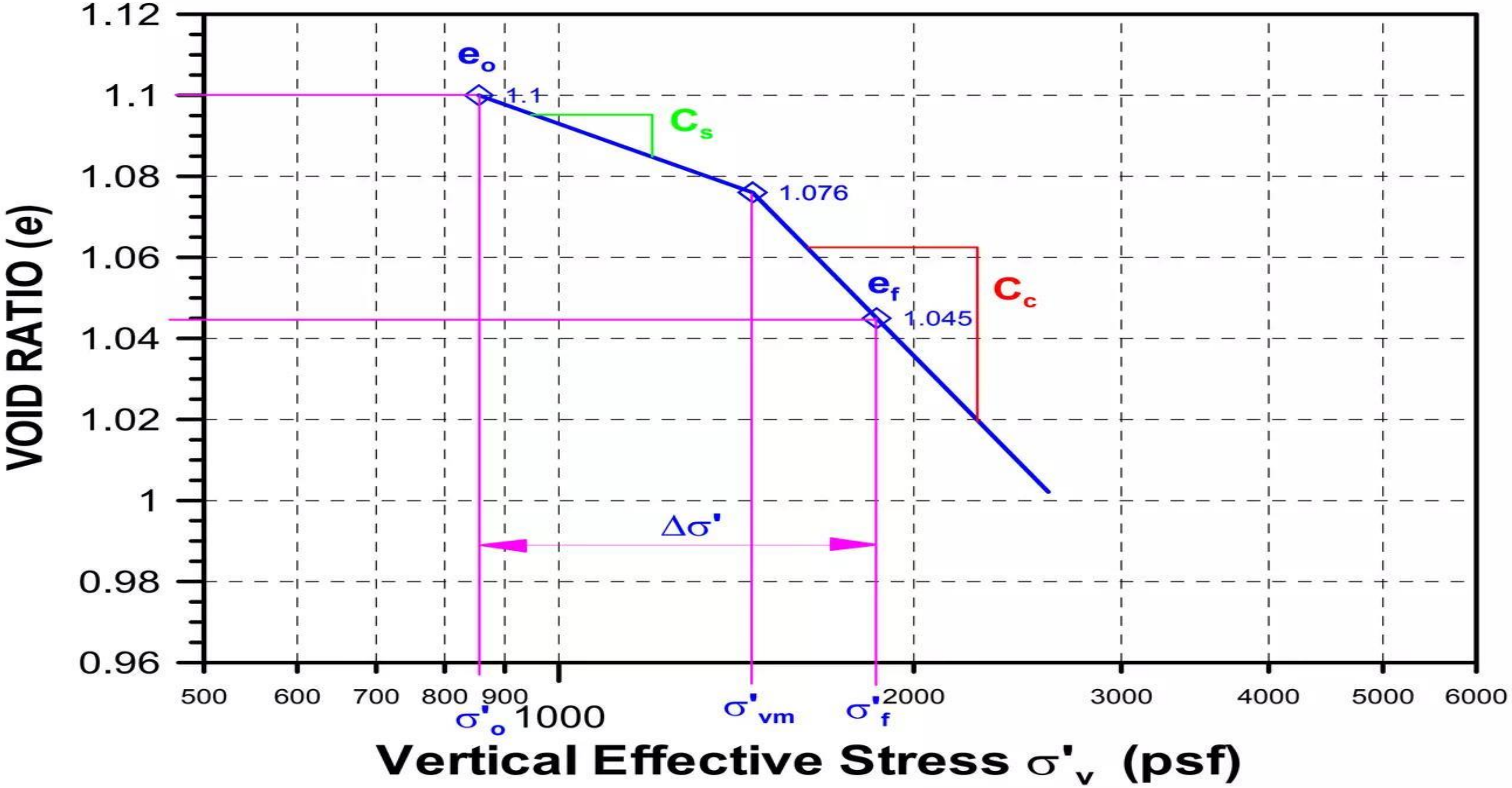


Figure 1. Example of Virgin Consolidation Curves.

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

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

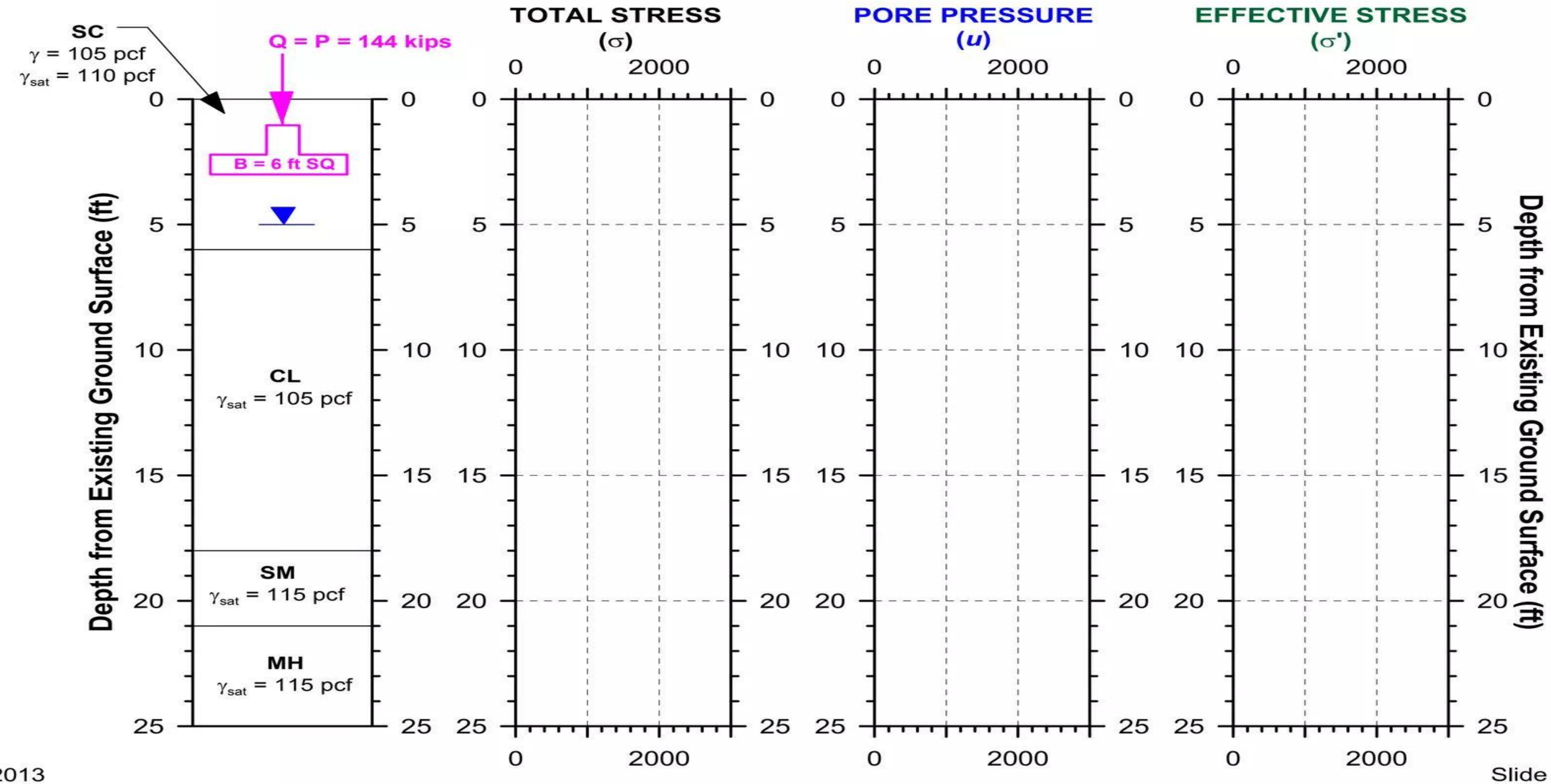
$$e_o = 1.1$$

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

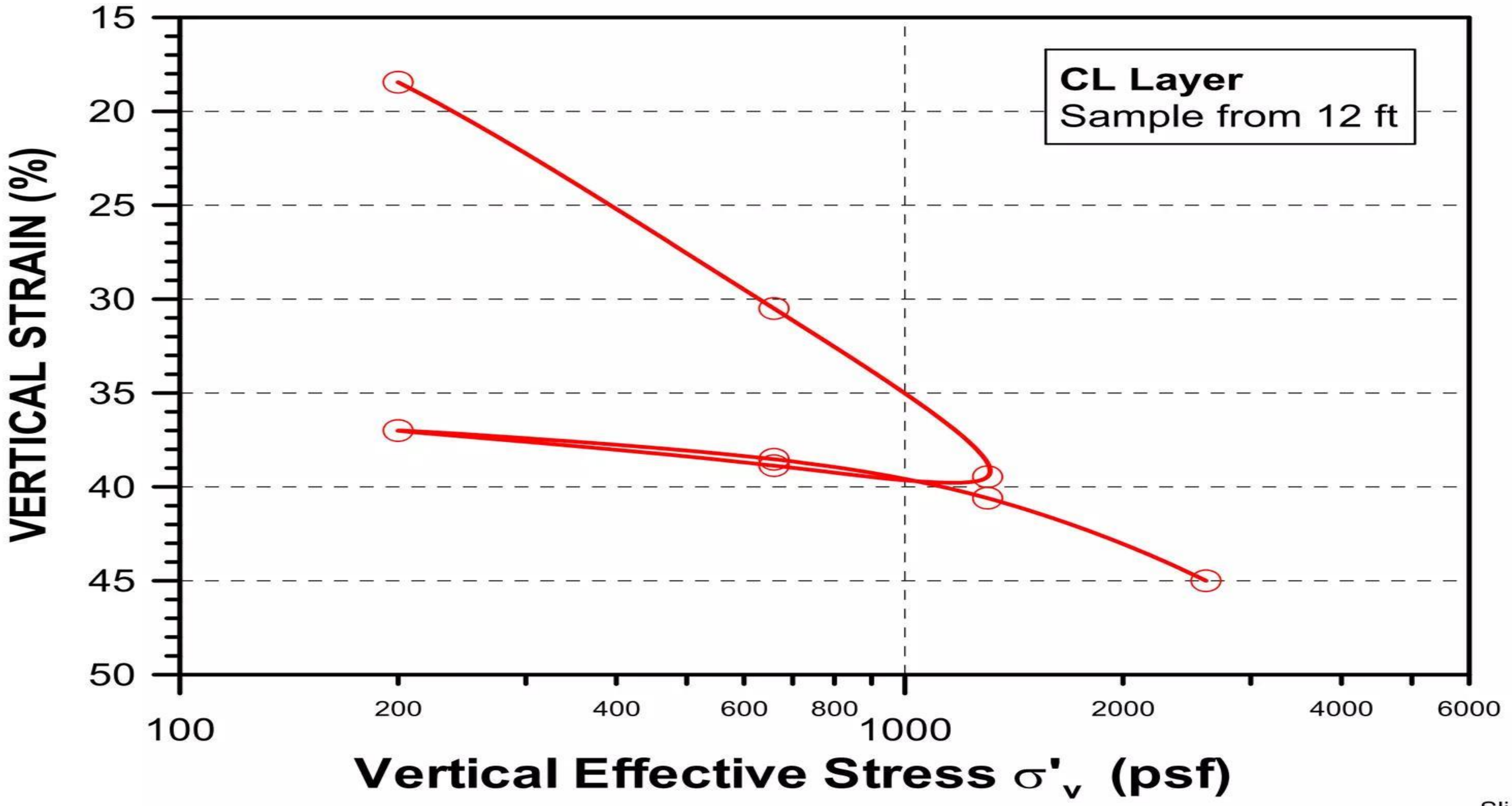
$$S_p = 0.262\,ft$$

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

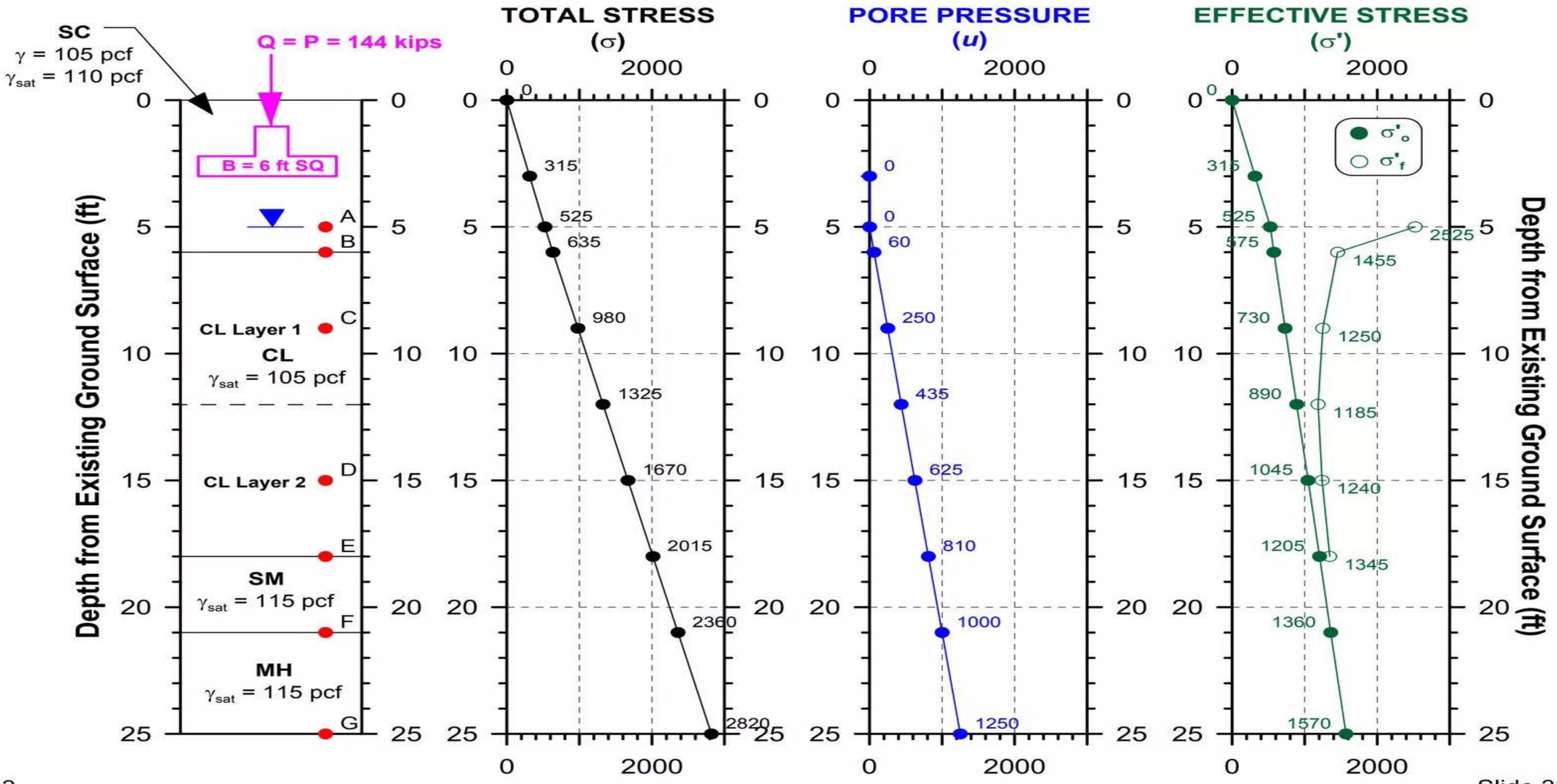
Example: Settlement from 1D Test Strain Results



Example: Settlement from 1D Test Strain Results



EXAMPLE: SETTLEMENT FROM 1D TEST STRAIN RESULTS



Example: Settlement from 1D Test Strain Results

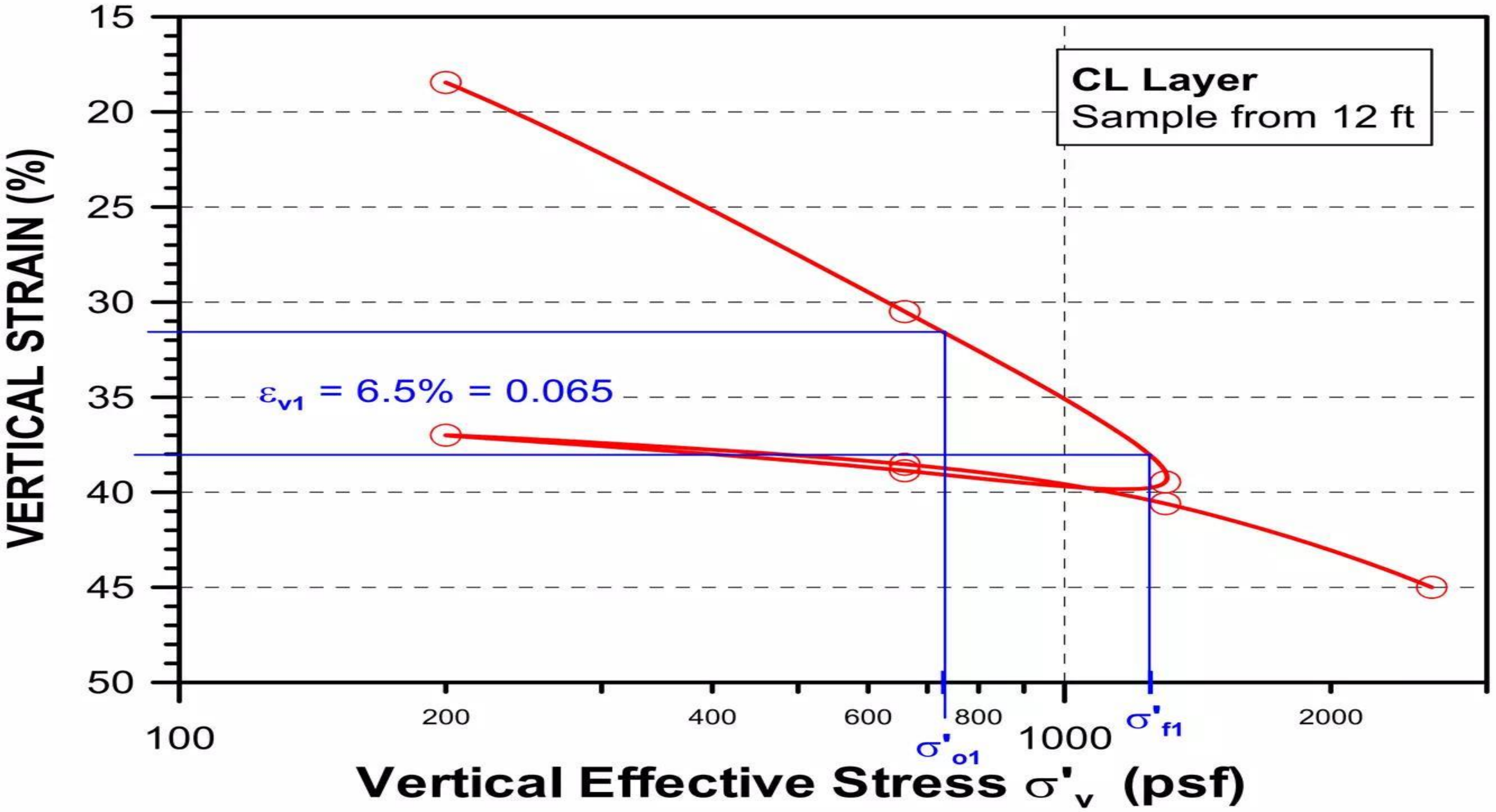
CL Layer 1

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

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

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

$$S_{p_1} = 4.7in$$



Example: Settlement from 1D Test Strain Results

CL Layer 2

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

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

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

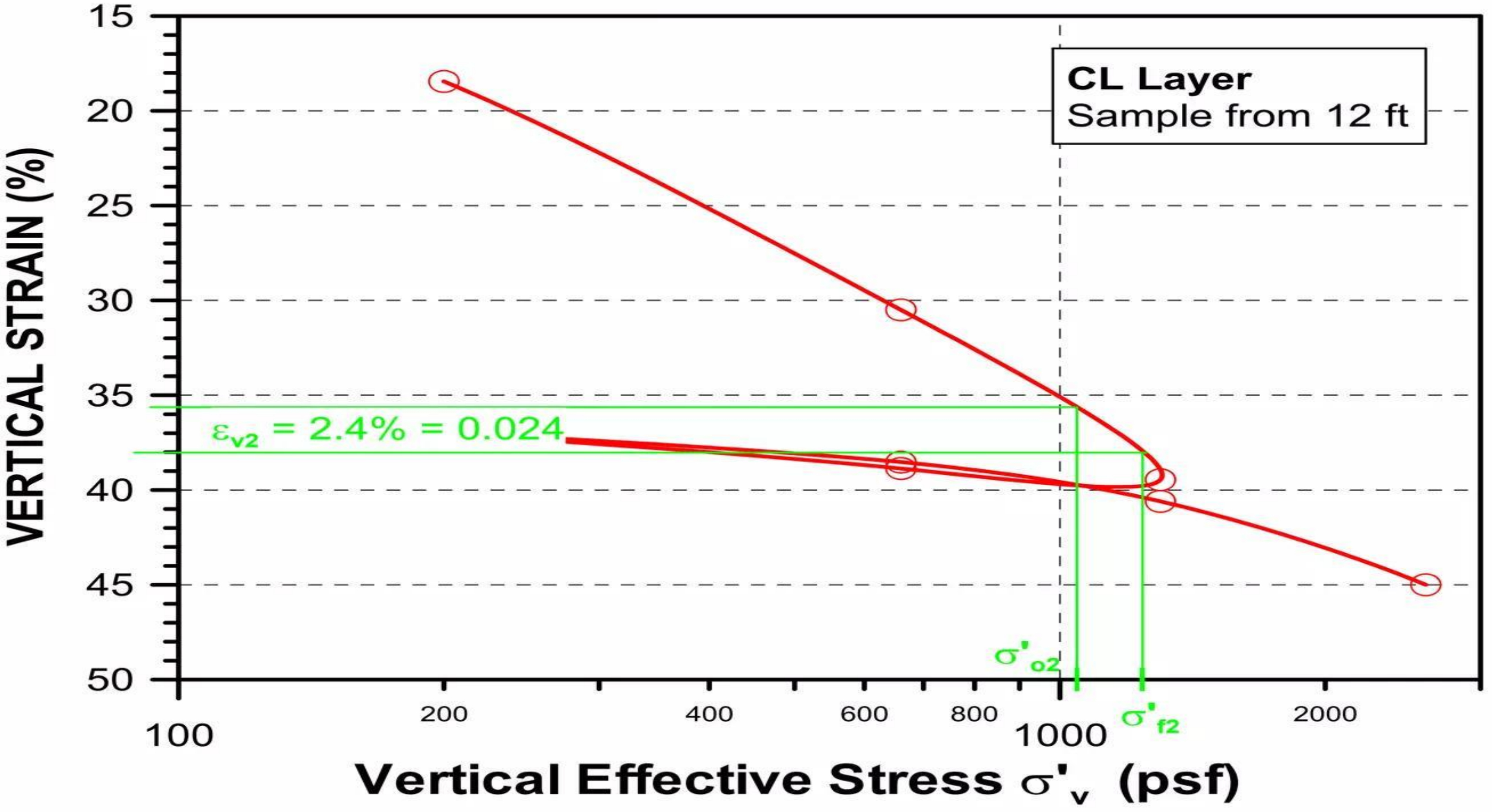
$$S_{p2} = 1.7\,in$$

Total Settlement

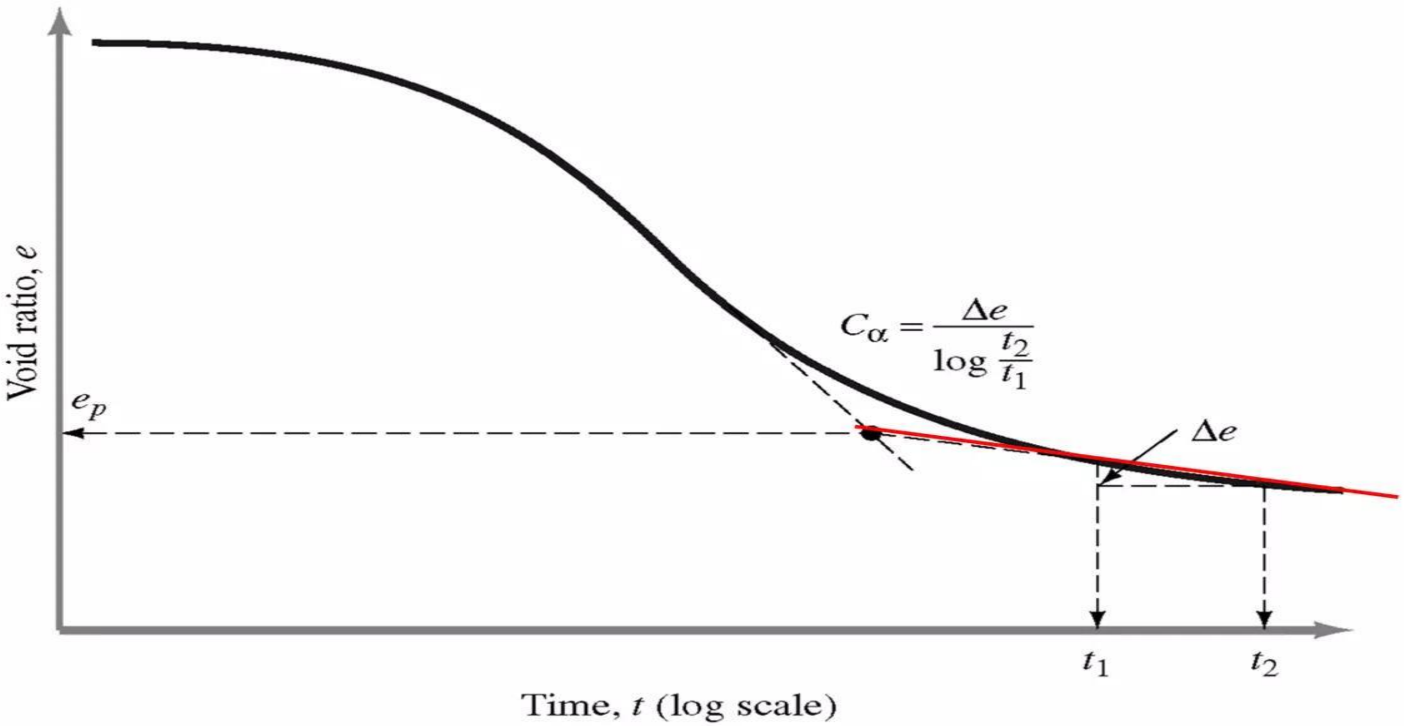
$$S_{Ptotal} = S_{p1} + S_{p2}$$

$$S_{Ptotal} = 6.4\,in$$

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



Settlement from Secondary Consolidation



Results of 1D Consolidation Test @ One Load Increment

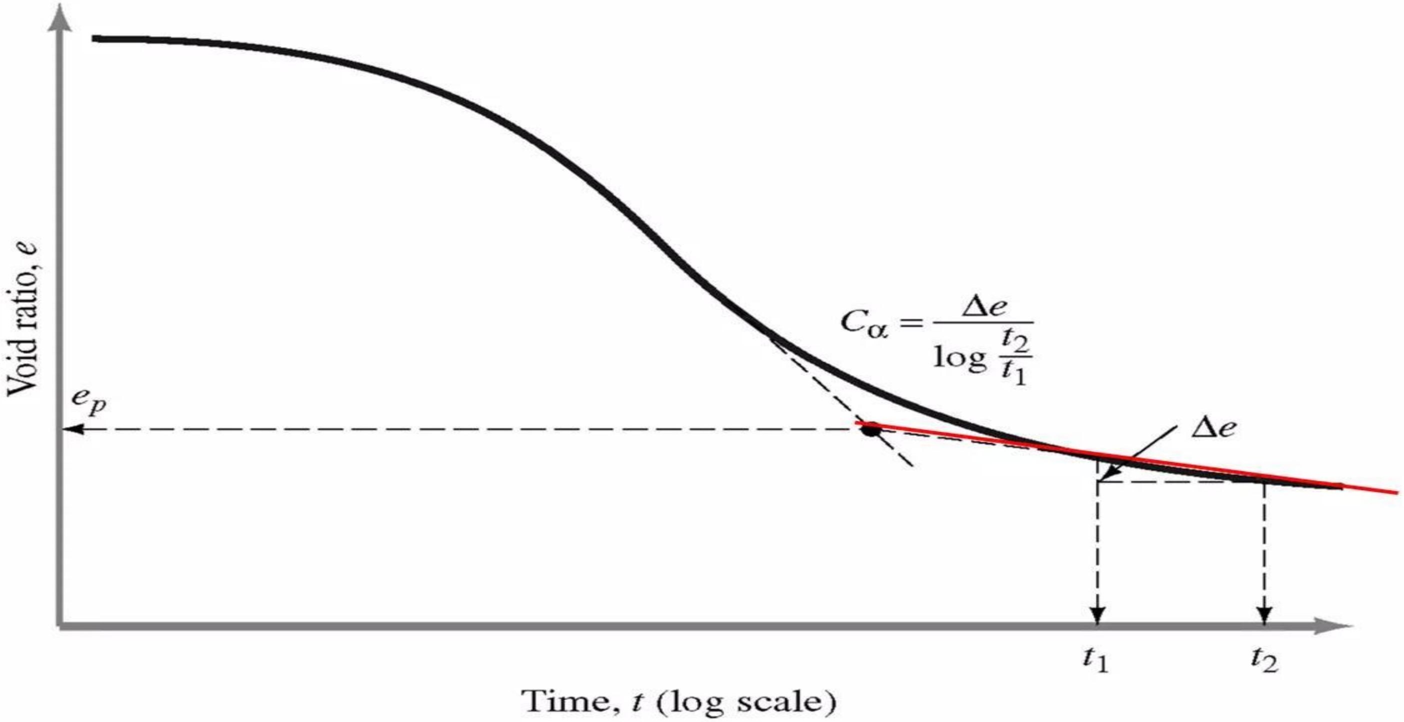
Figure 7.15. Das FGE (2005).

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

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).

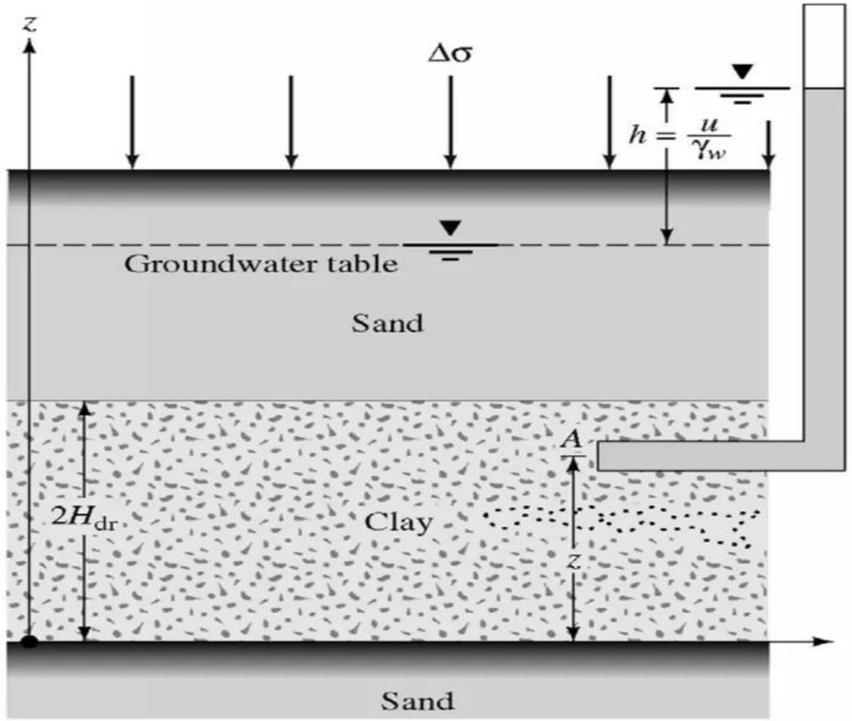
$$S_s = C'_\alpha H \log\left(\frac{t_2}{t_1}\right)$$

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

Where:

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

TIME RATE OF CONSOLIDATION



Clay Layer Undergoing Consolidation

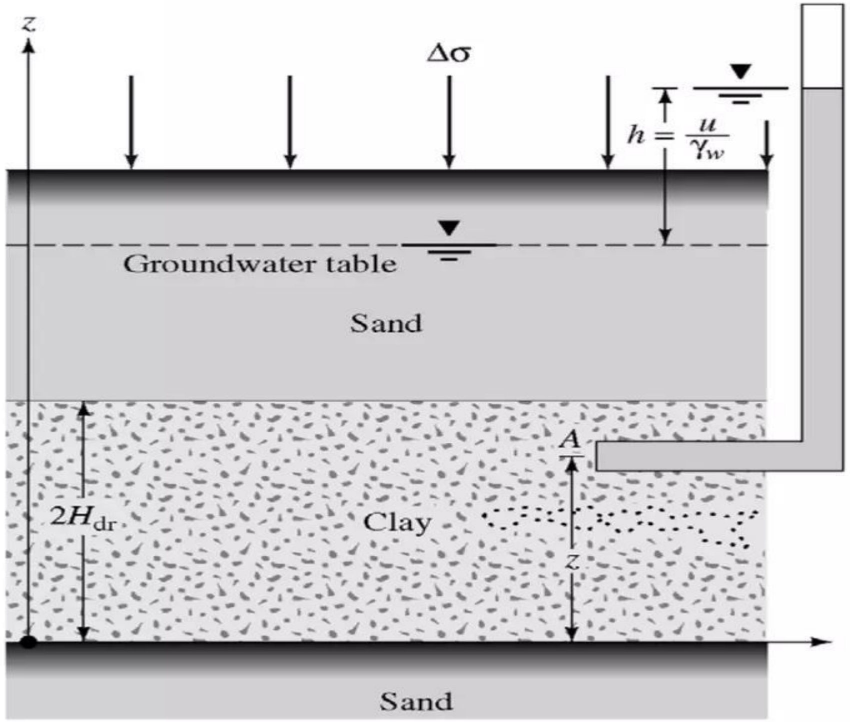
Figure 7.17a. Das FGE (2005).

Theory of 1D Consolidation (Terzaghi, 1925)

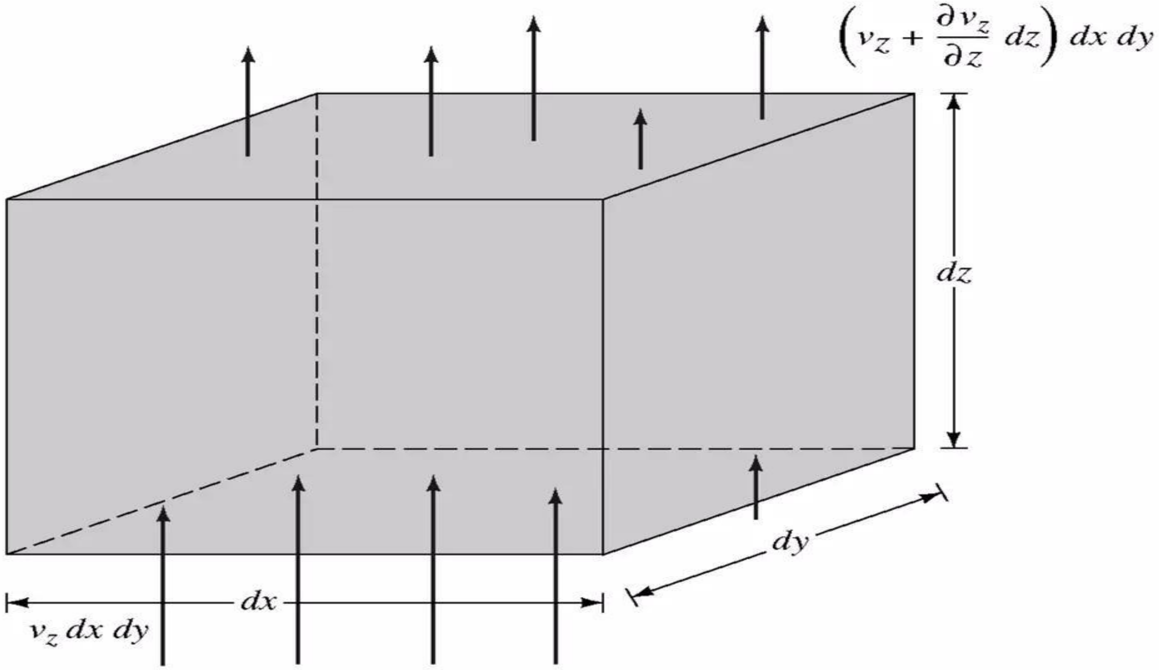
Assumptions:

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

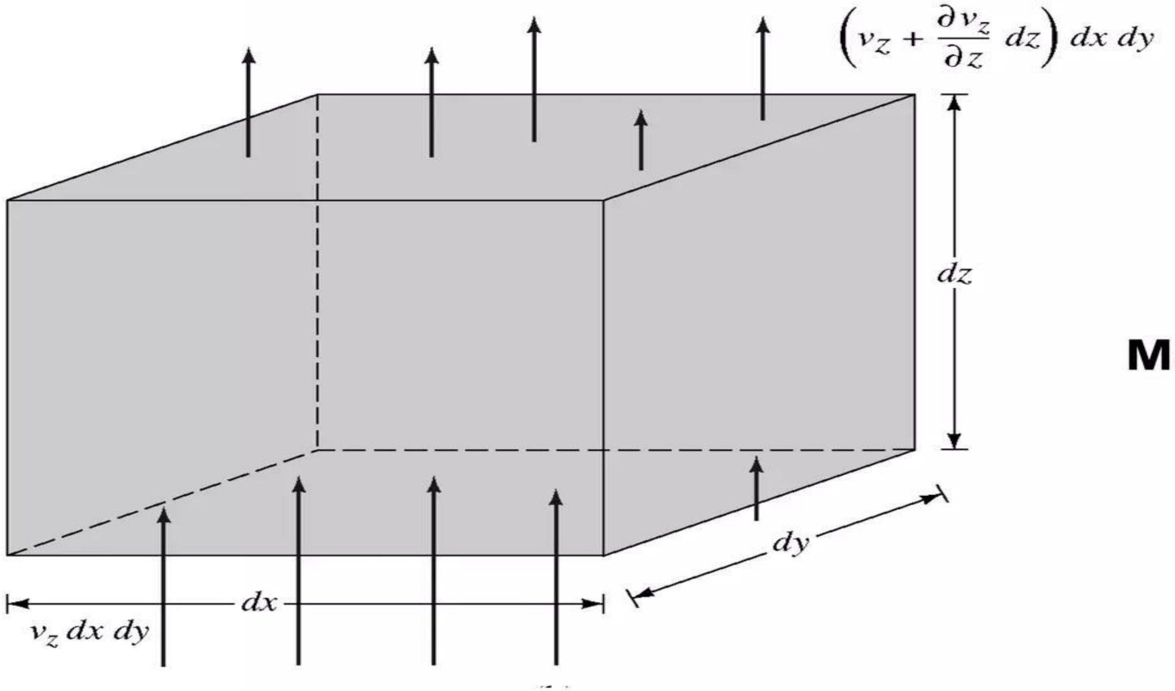


Clay Layer Undergoing Consolidation
Figure 7.17a. Das FGE (2005).



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:

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

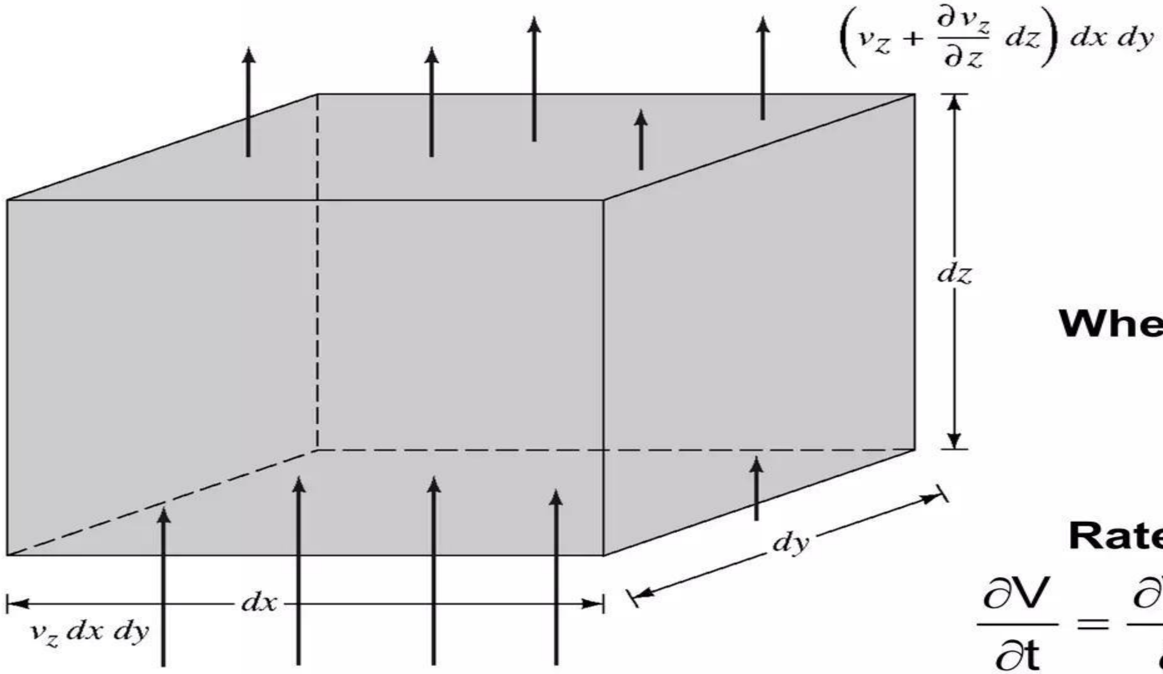
or

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

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:

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

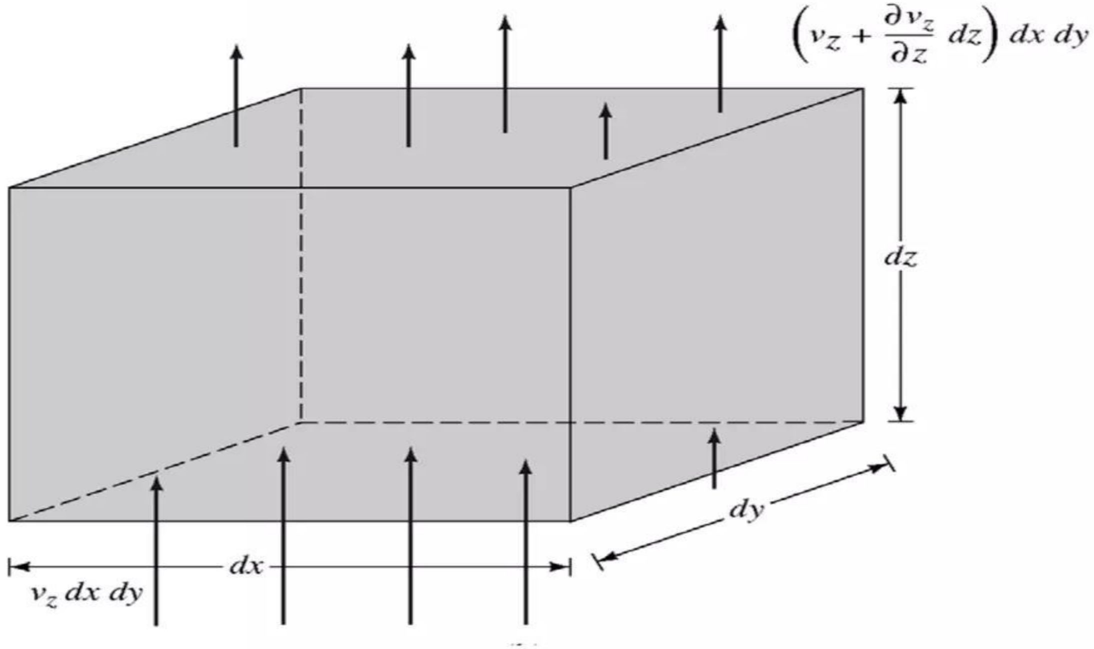
Rate of change in *V* = Rate of Change in V_v

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

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

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

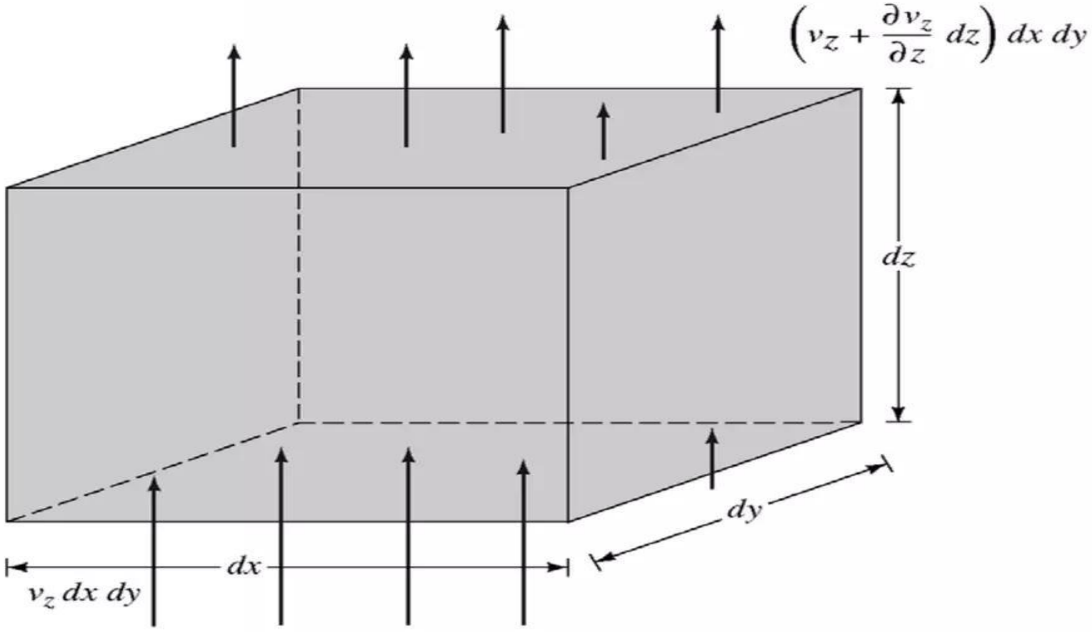
e_o = Initial Void Ratio. Substituting:

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

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:

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

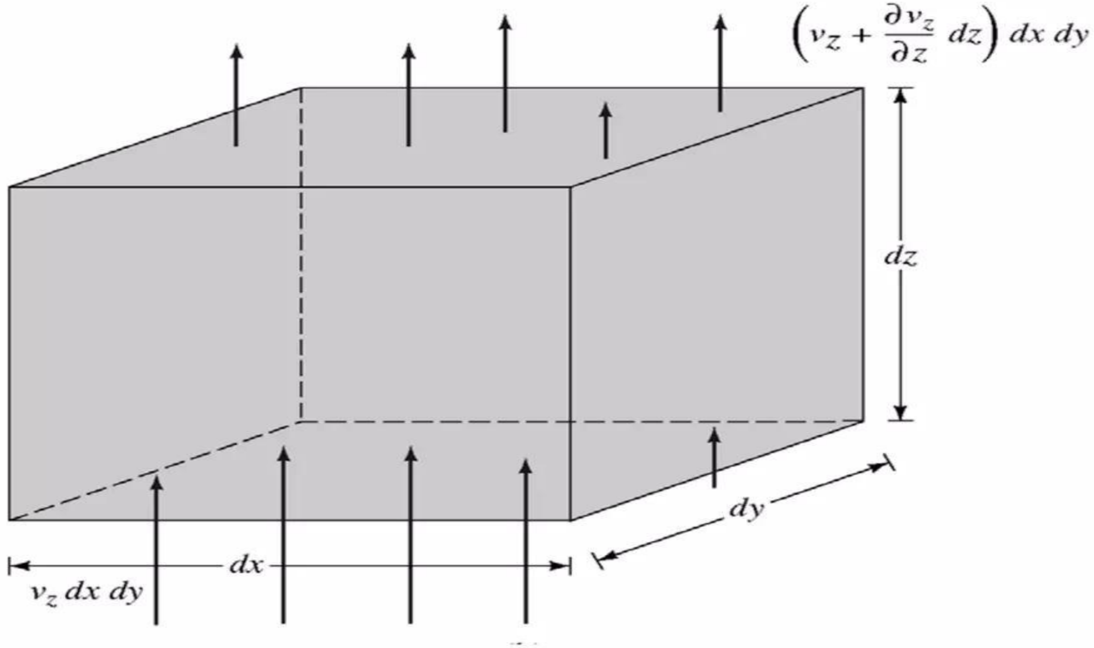
a_v = Coefficient of Compressibility. Can be considered constant over narrow pressure increases. Combining equations:

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

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.

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

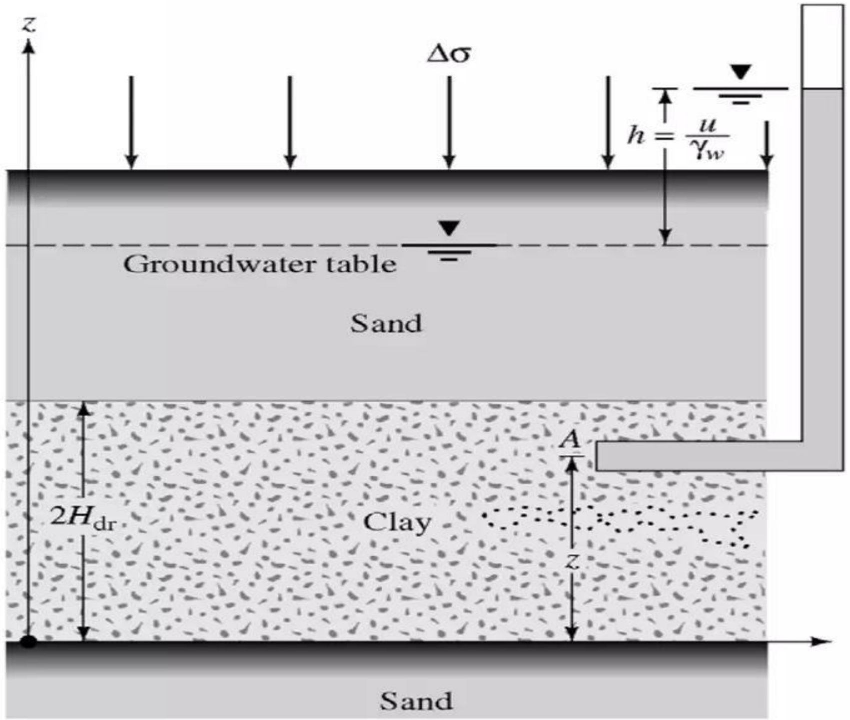
Rearranging Equations:

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

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:

$$z = 0, u = 0$$

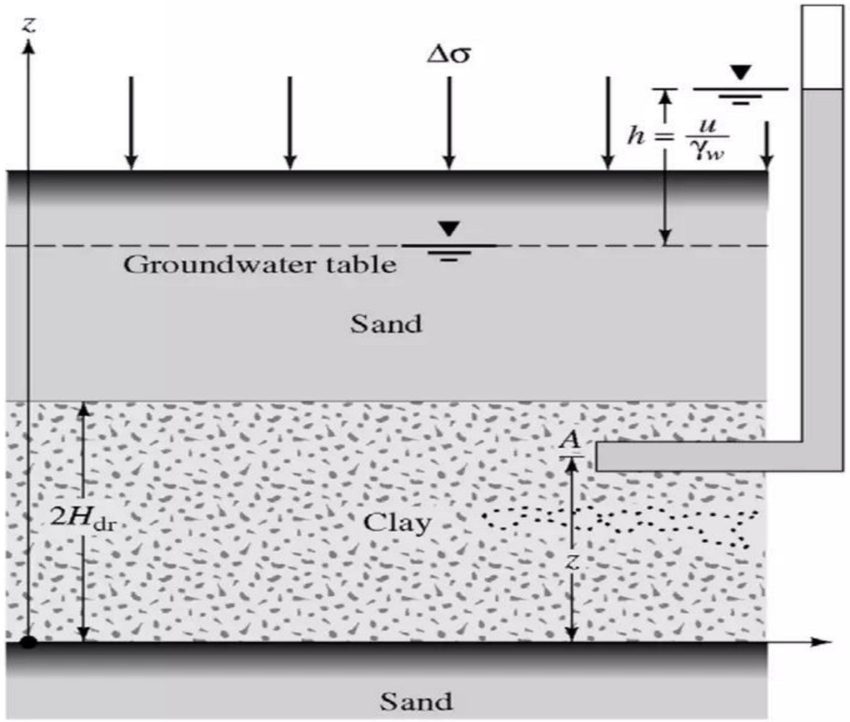
$$z = 2H_{dr}, u = 0$$

$$t = 0, u = u_0$$

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

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

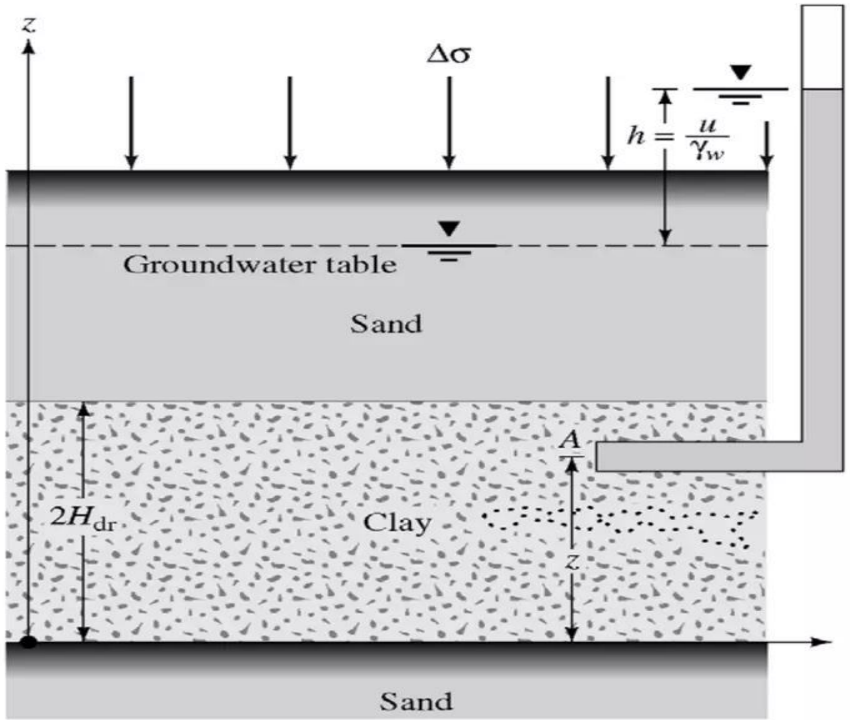
Where:

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

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:

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

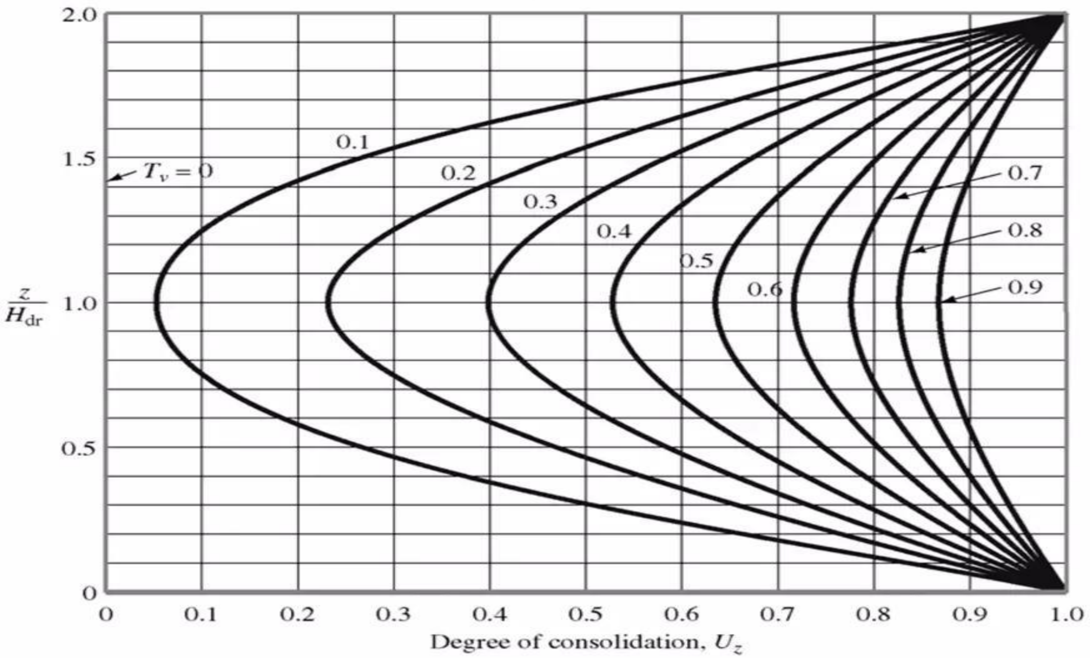


Figure 7.18. Das FGE (2006).

TIME RATE OF CONSOLIDATION

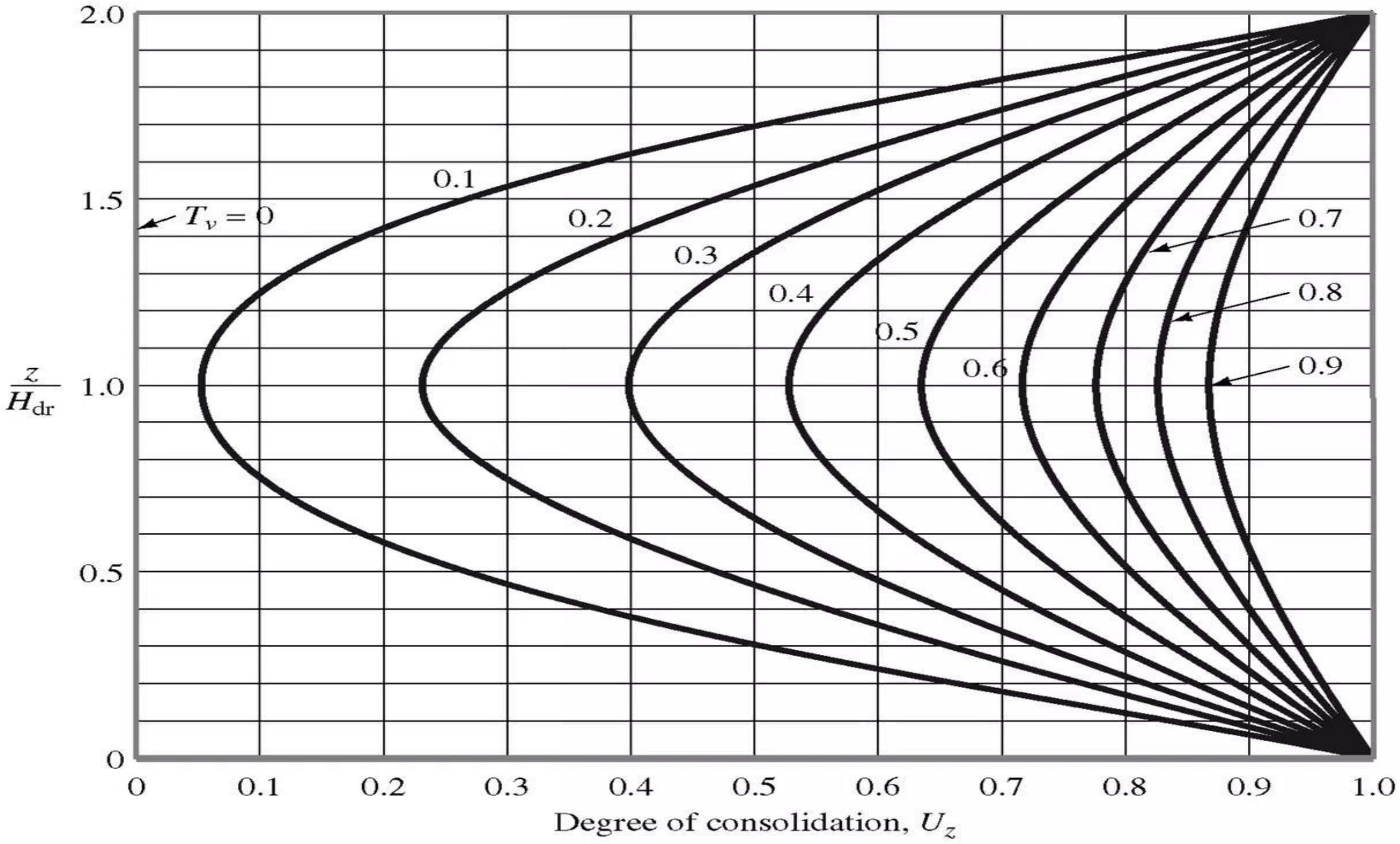
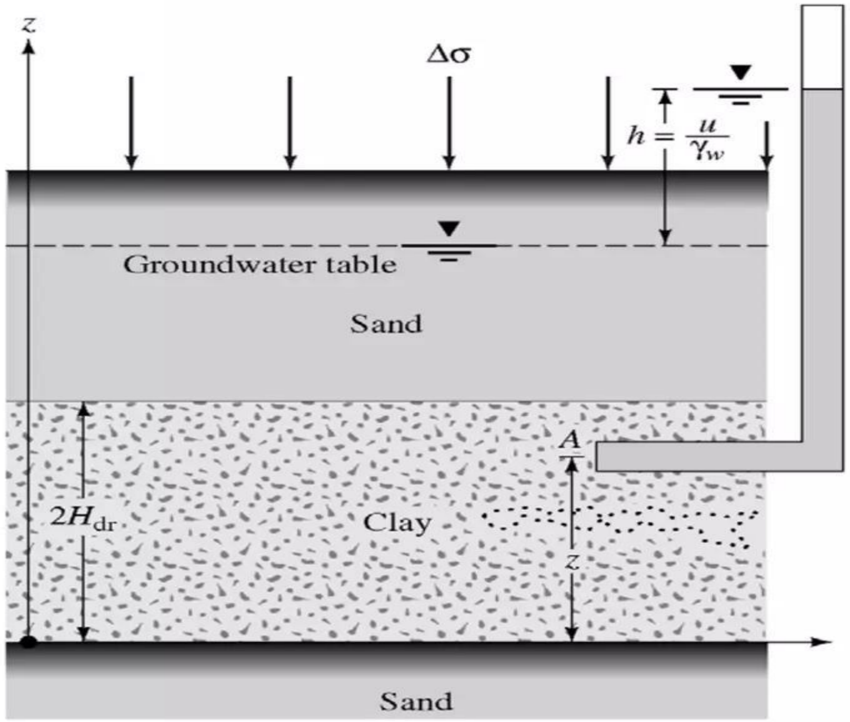


Figure 7.18. Das FGE (2006).

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

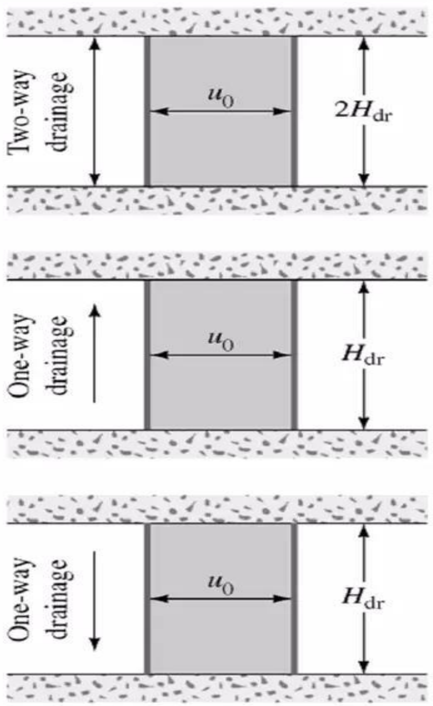
For $U > 60\%$, $T_v = 1.781 - 0.933\log(100 - U\%)$

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

TIME RATE OF CONSOLIDATION

Difference between Average Degree of Consolidation and Midplane Degree of Consolidation

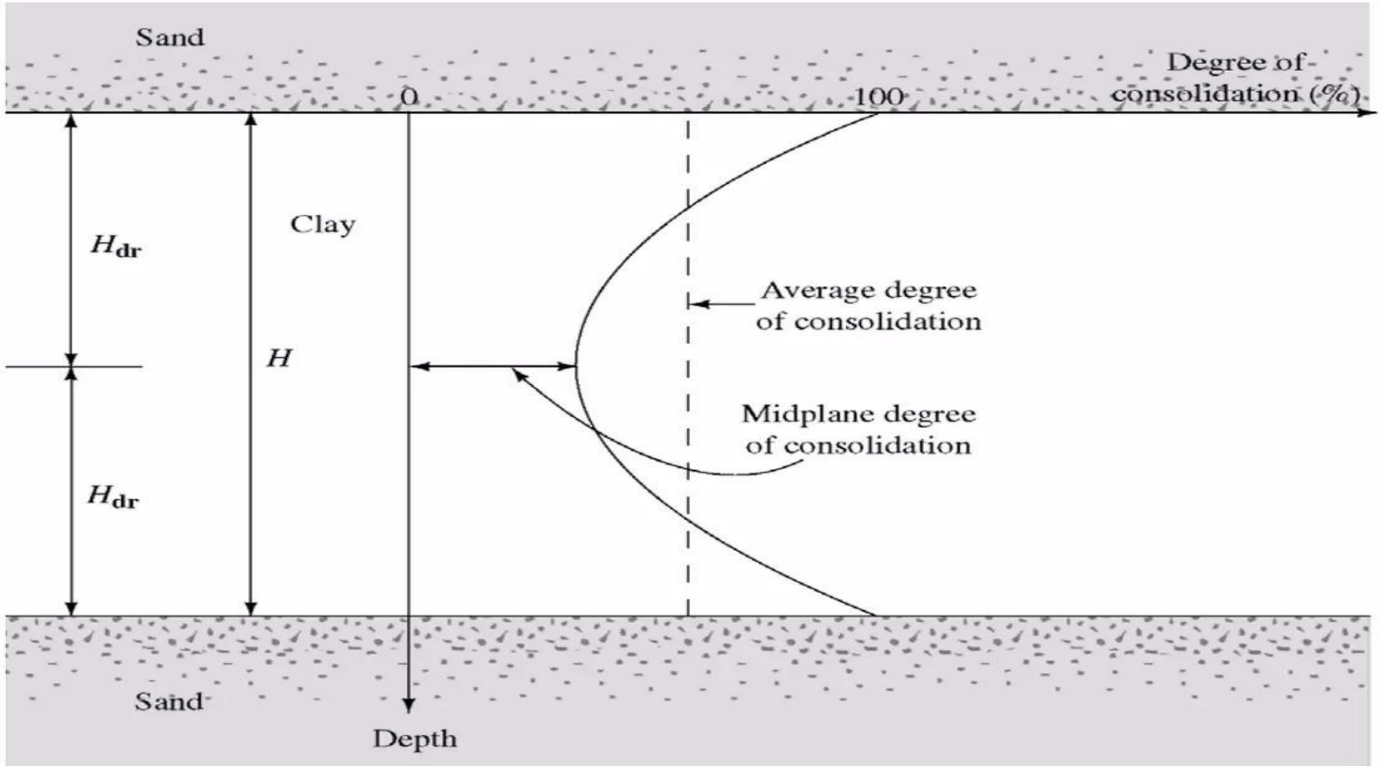
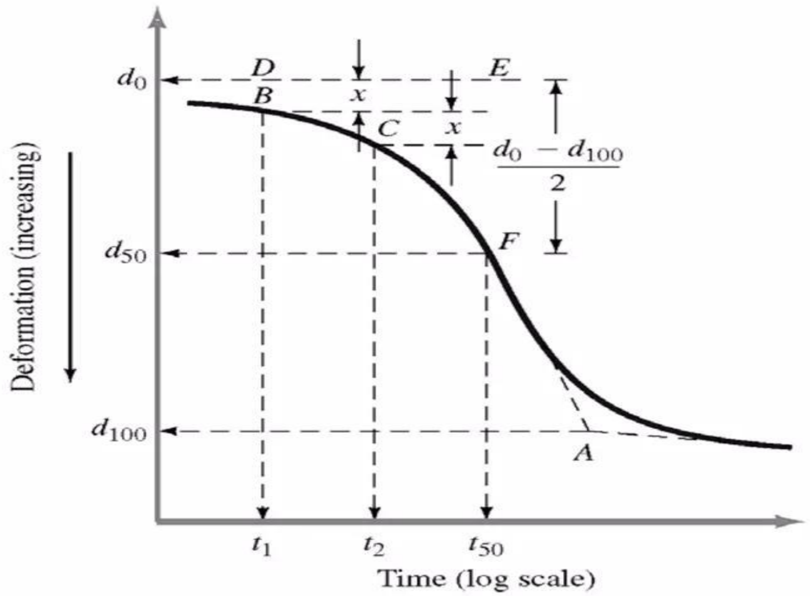


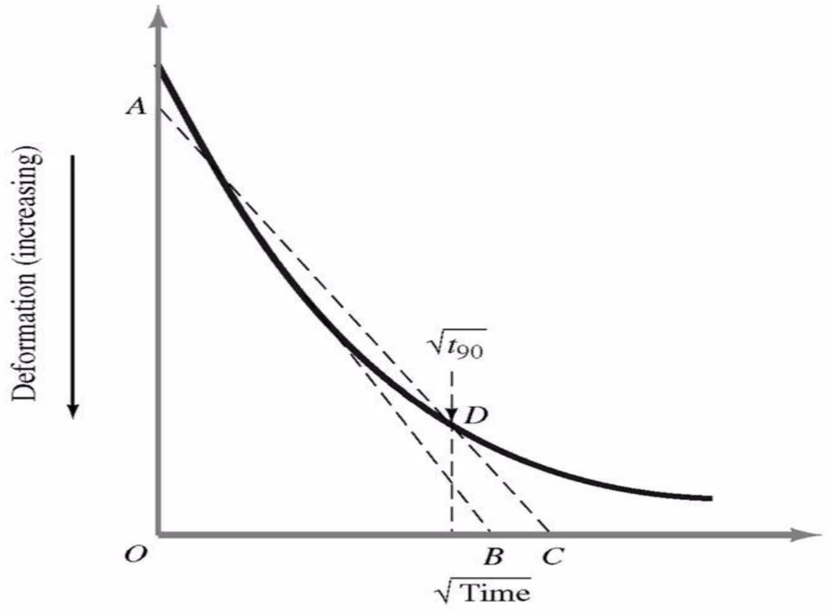
Figure 7.28. Das FGE (2006).

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)

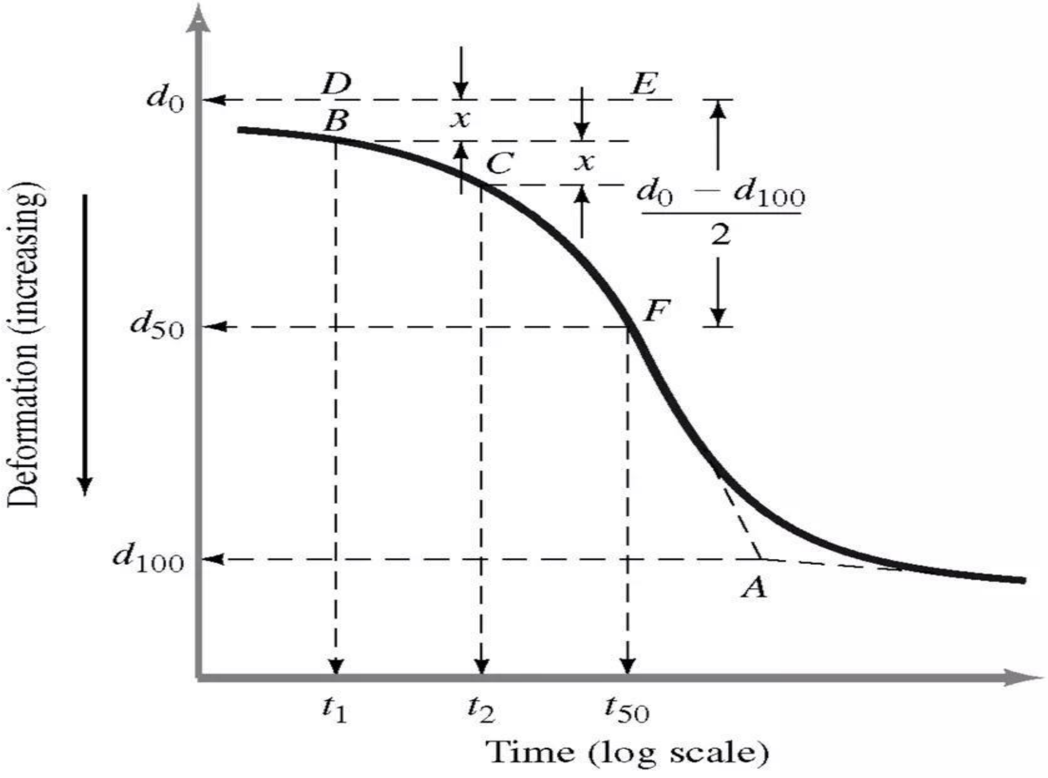


Figure 7.19. Das FGE (2006).

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

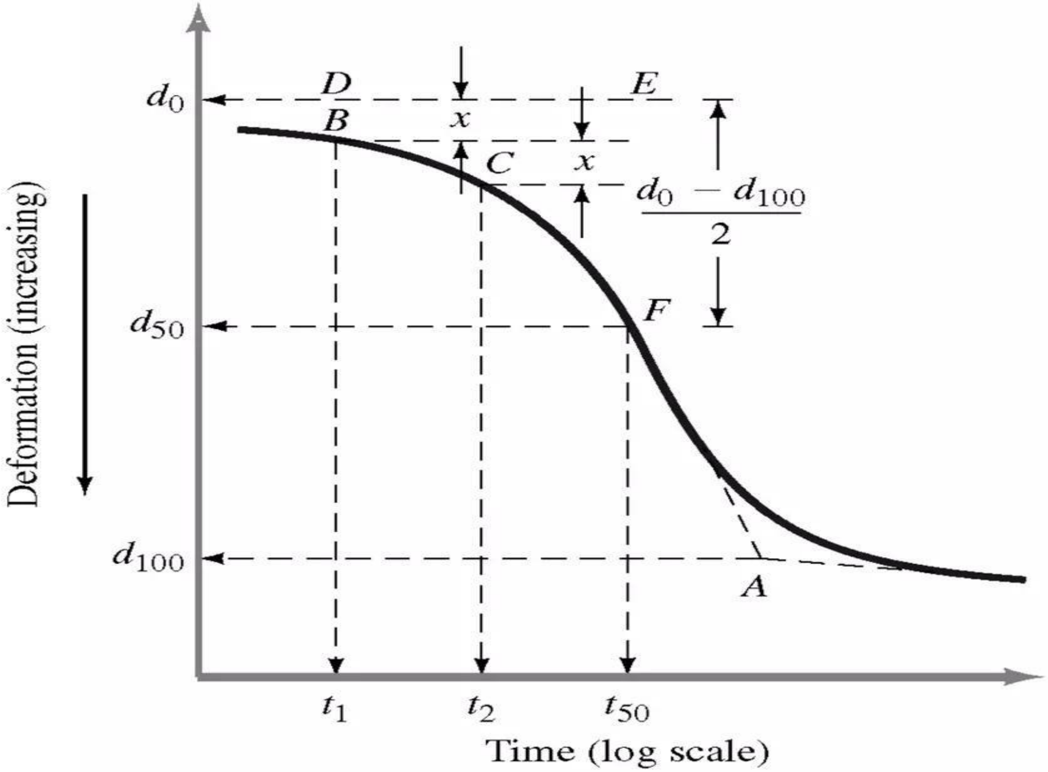


Figure 7.19. Das FGE (2006).

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)

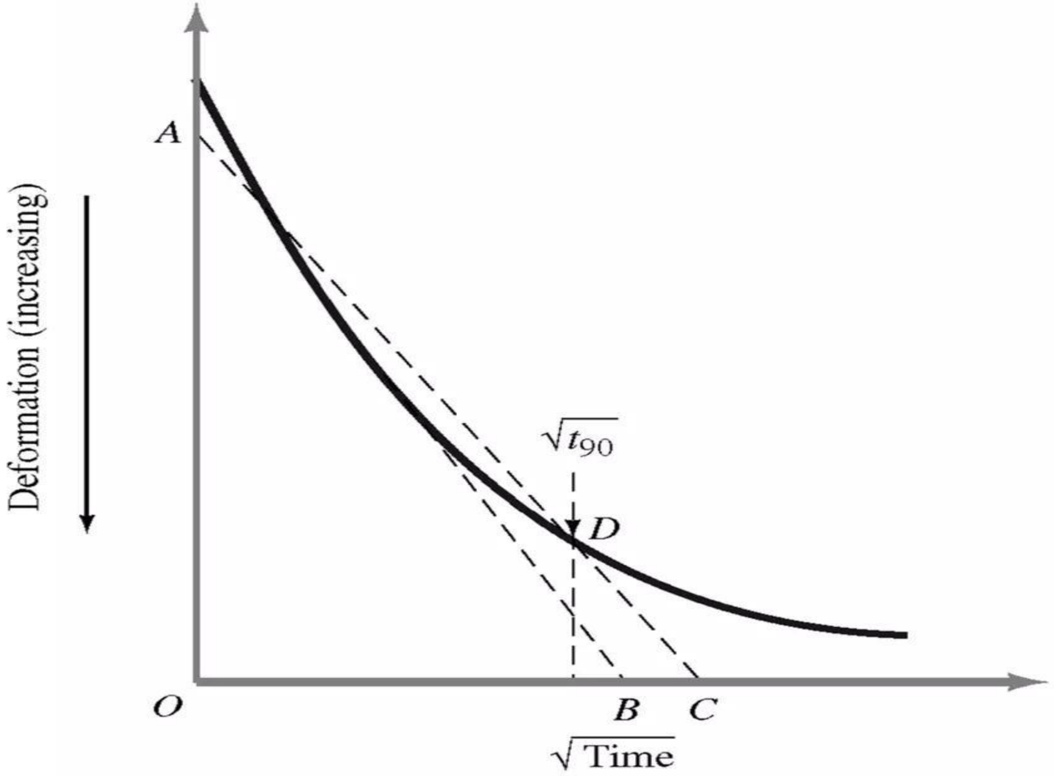


Figure 7.20. Das FGE (2006).

Square Root of Time Method

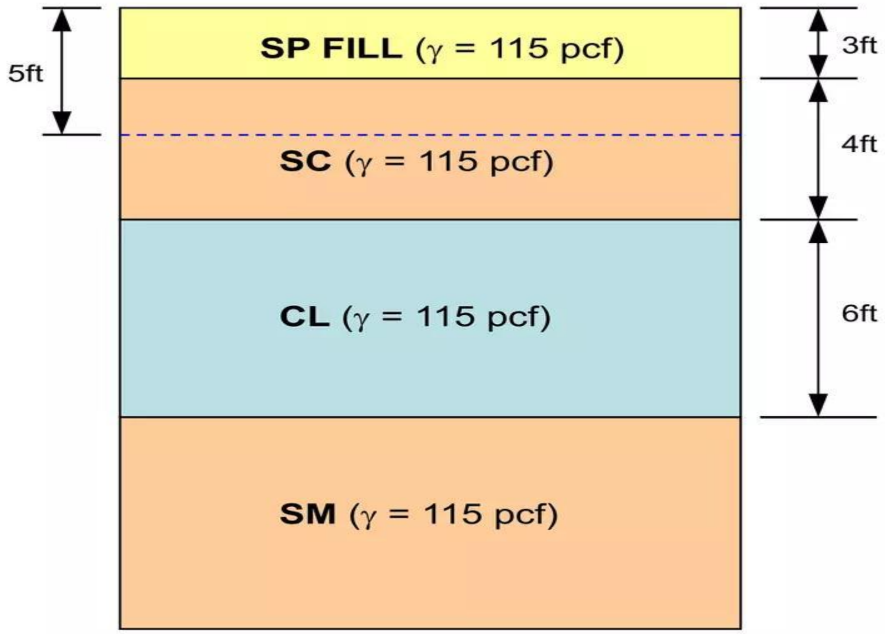
1. Draw a line *AB* trough the early portion of the curve.
2. Draw a line *AC* such that *OC* = 1.15*OB*. The time value for Point *D* (i.e. the intersection of line *AC* and the data) is the square root of time for t_{90} (i.e. the time to 90% primary consolidation).
3. For 90% consolidation, T_v =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

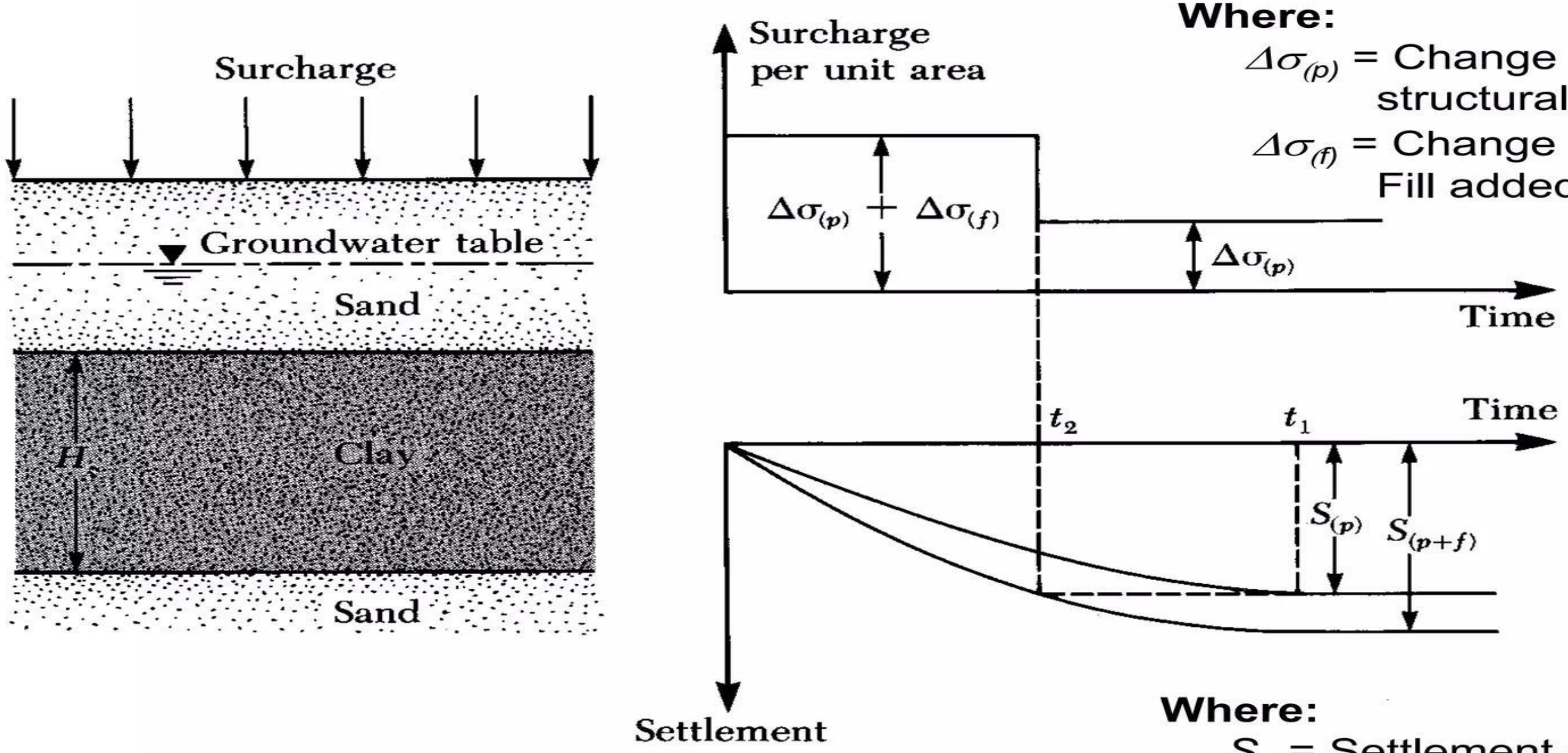


Figure 7.26. Das FGE (2006).

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.

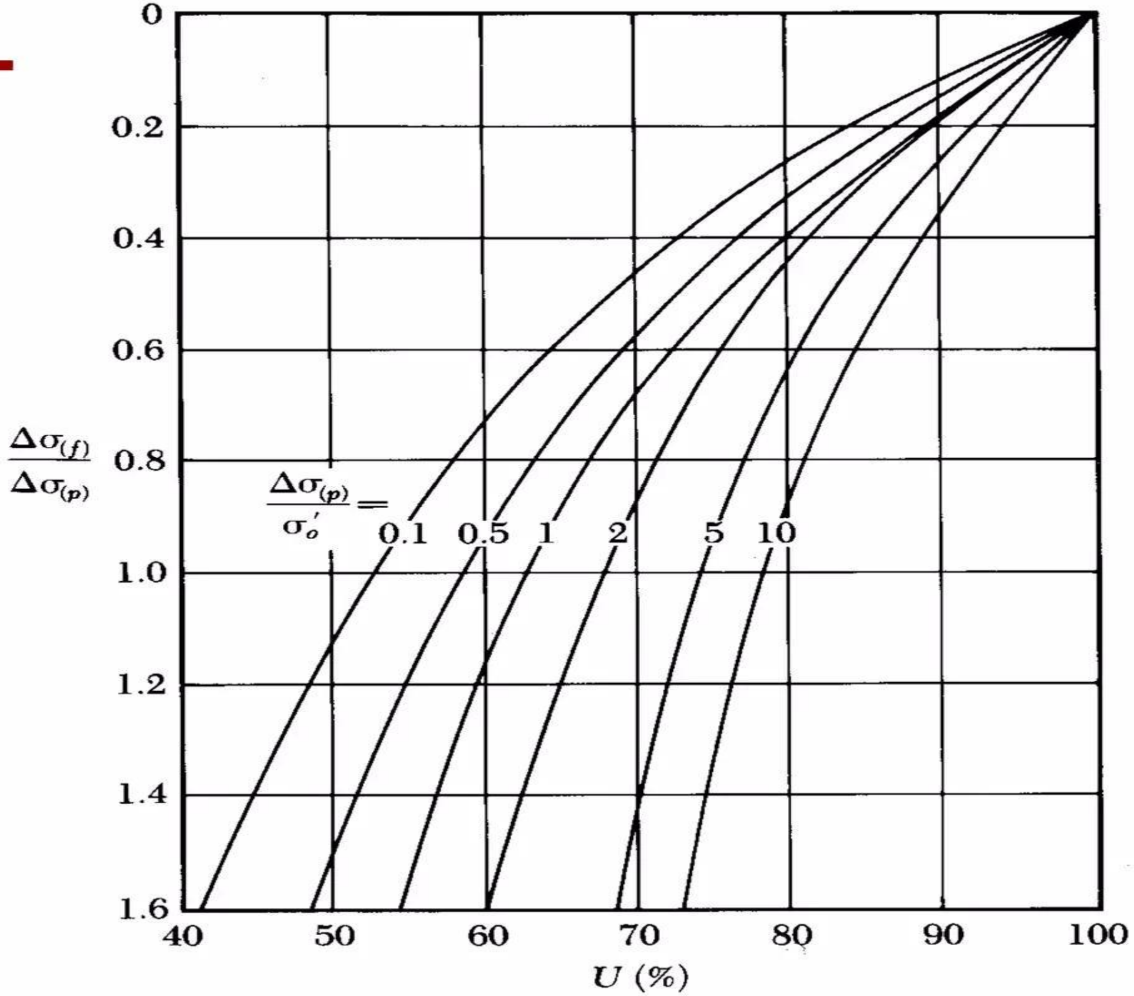


Figure 7.27. Das FGE (2006).

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):

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

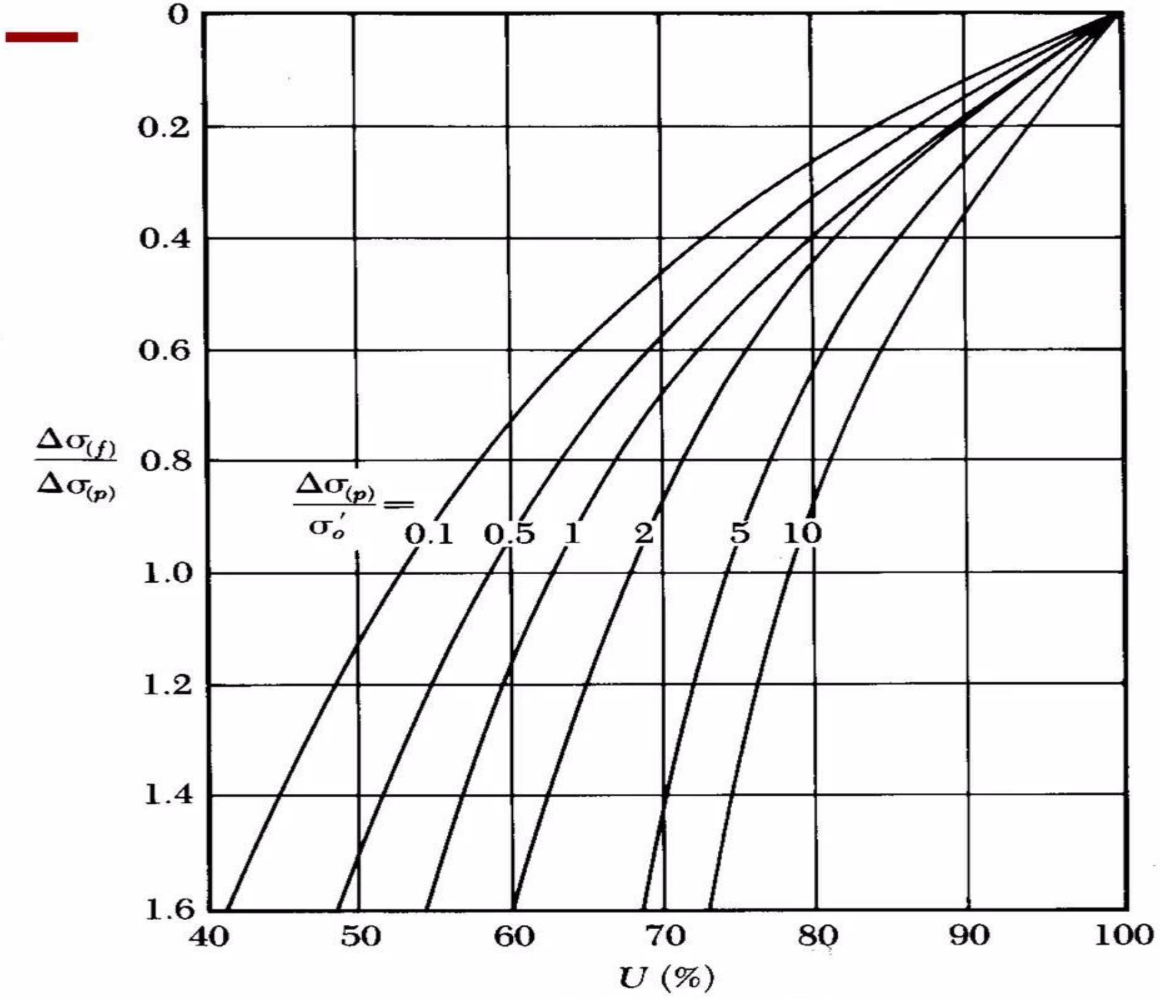


Figure 7.27. Das FGE (2006).

TIME RATE OF CONSOLIDATION

Difference between Average Degree of Consolidation and Midplane Degree of Consolidation

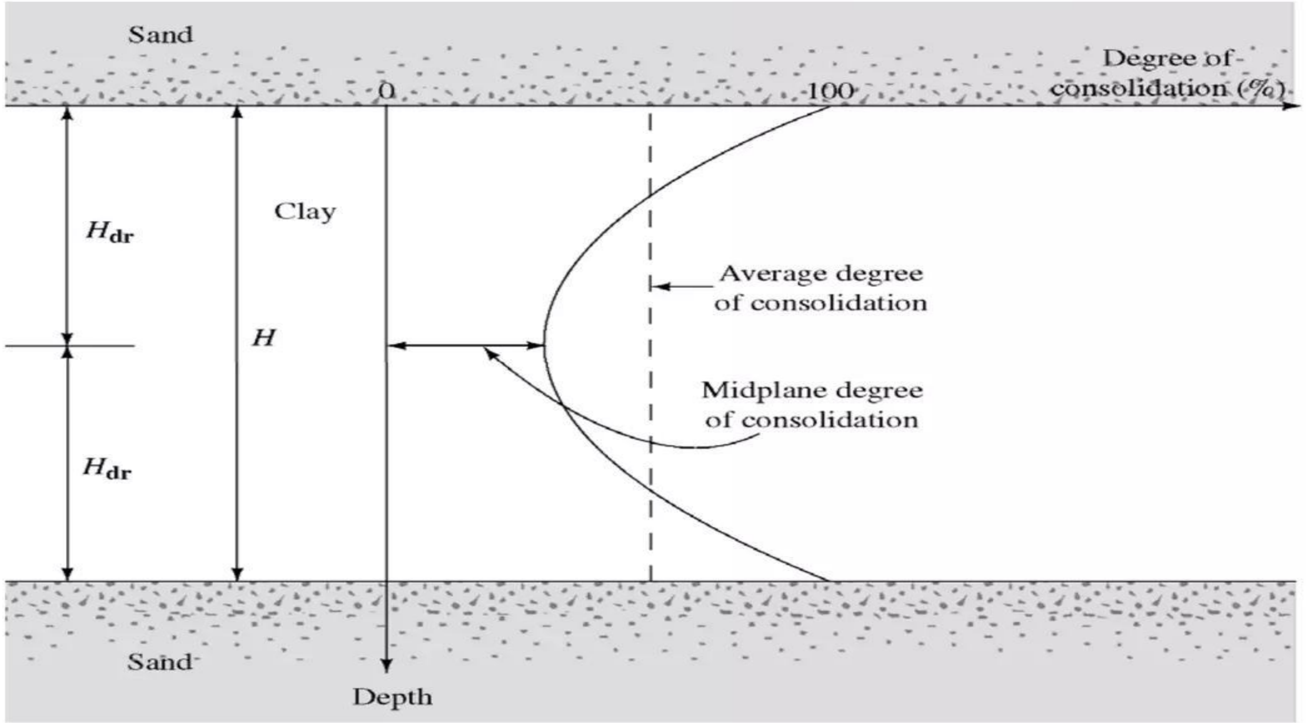


Figure 7.28. Das FGE (2006).

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

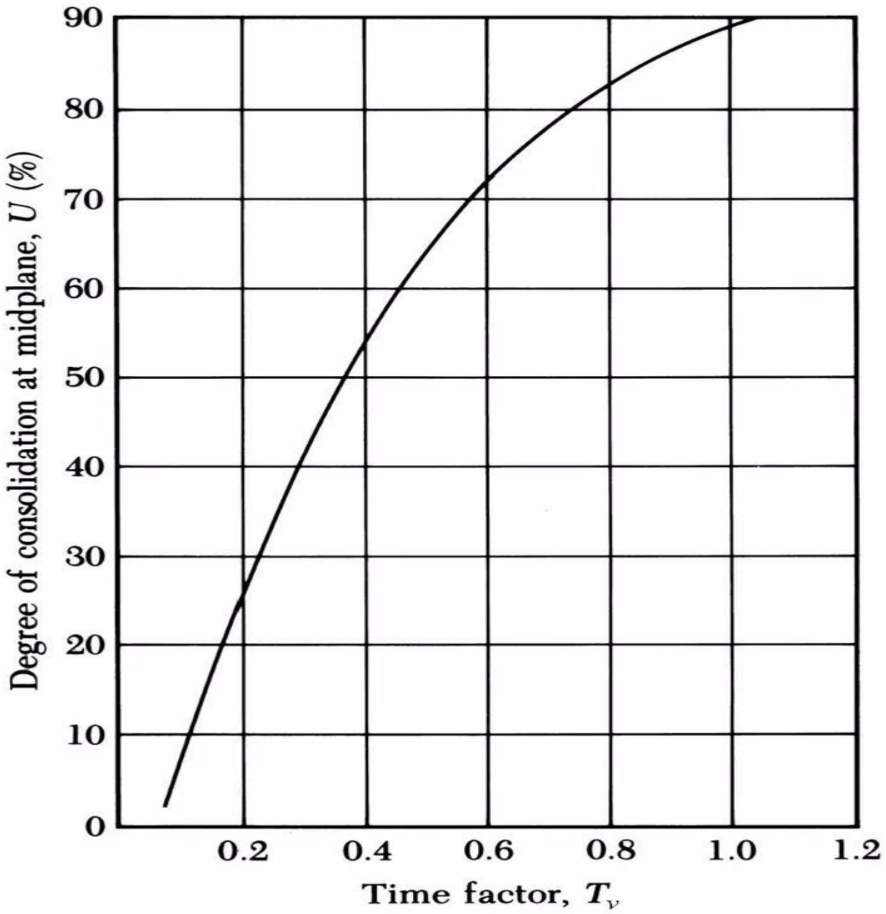
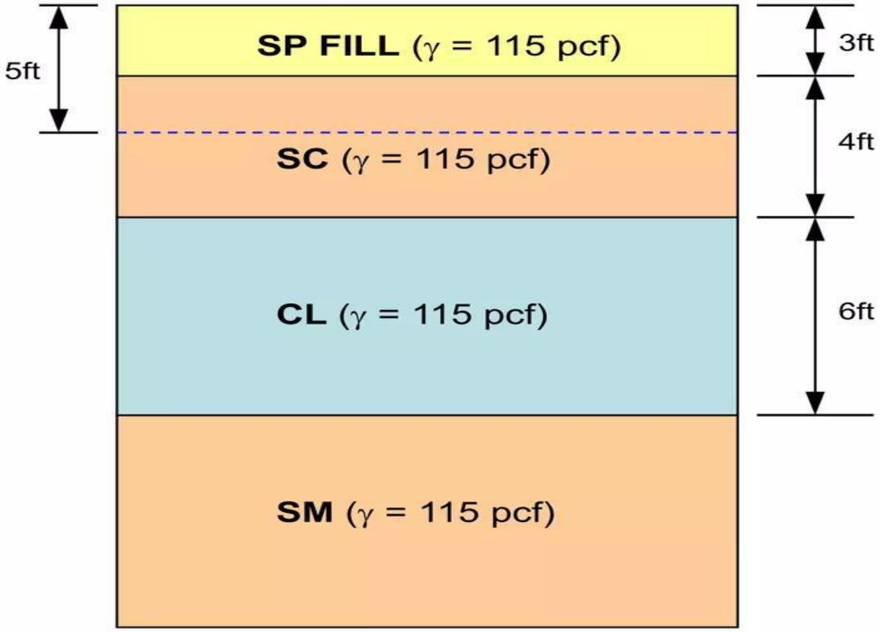


Figure 7.29. Das FGE (2006).

SURCHARGING EXAMPLE



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

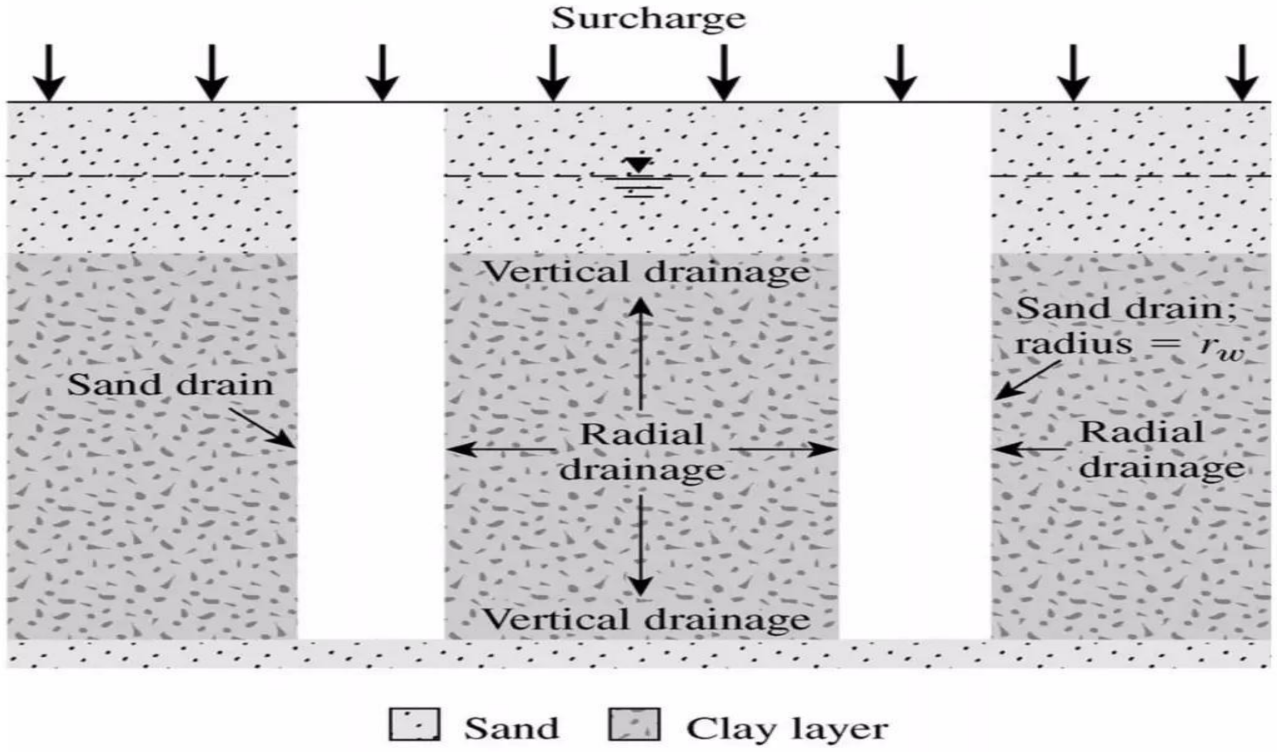
REQUIRED: Determine the following:

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

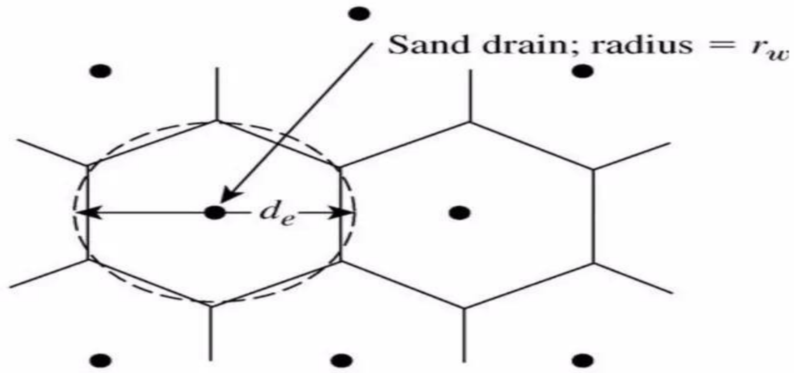
Sand Drains



Section View

Figure 10.38. Das PGE (2006).

r_w = Sand Drain Radius
d_e = Effective Diameter



Plan View – Triangular Spacing

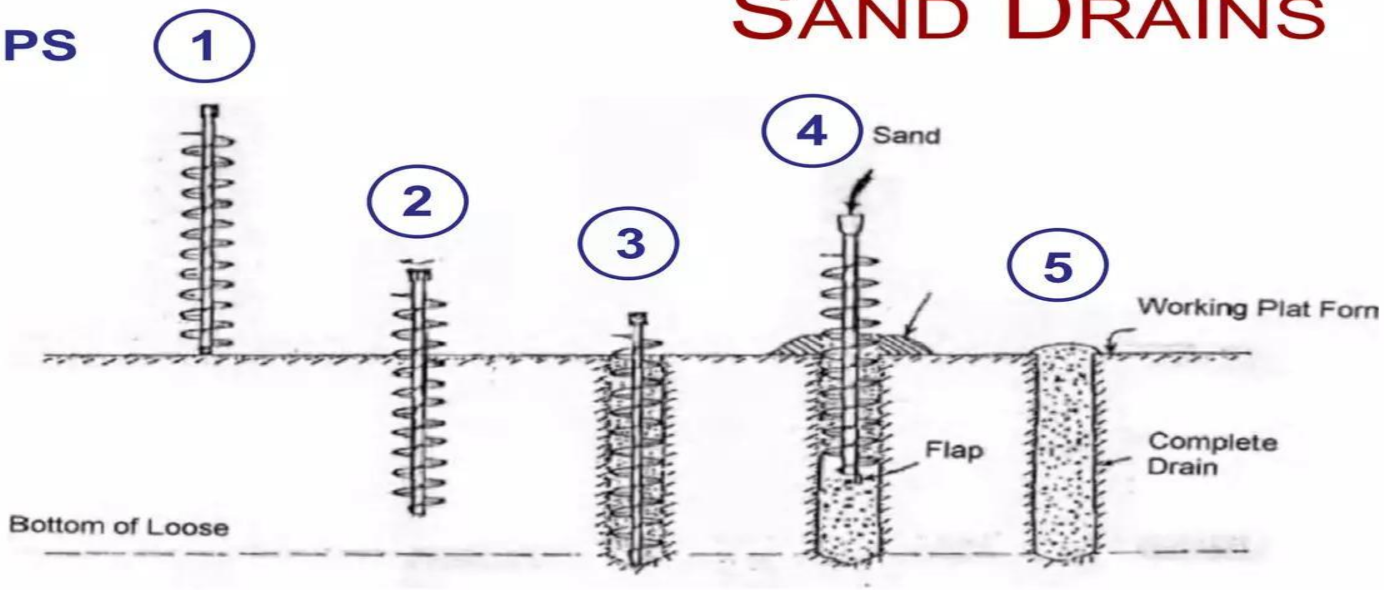
Figure 10.38. Das PGE (2006)

Reduction Drainage Path = Reduction in Drainage Time

Ground Modification for Consolidation

Sand Drains

STEPS



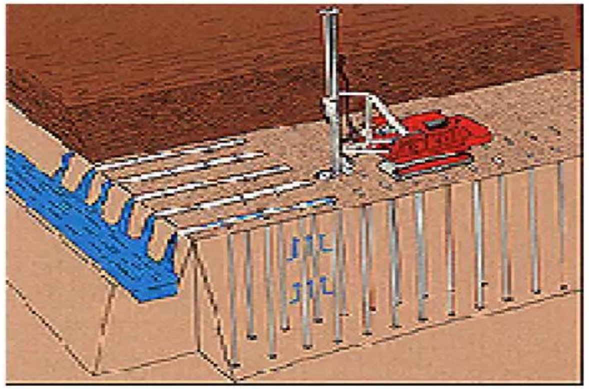
1. Place auger at drain location.
2. Screw auger to selected depth.
3. Rotate auger at selected depth to remove soil.
4. Inject sand while auger is extracted.
5. Complete sand drain to working platform level.

Sand Drain Installation: Auger Method
(Kirmani, 2004)

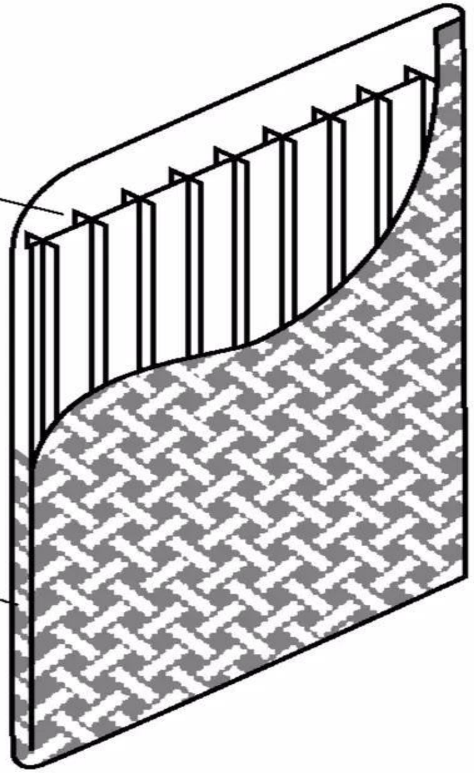
Figure 10.39. Das PGE (2006).

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)

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

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

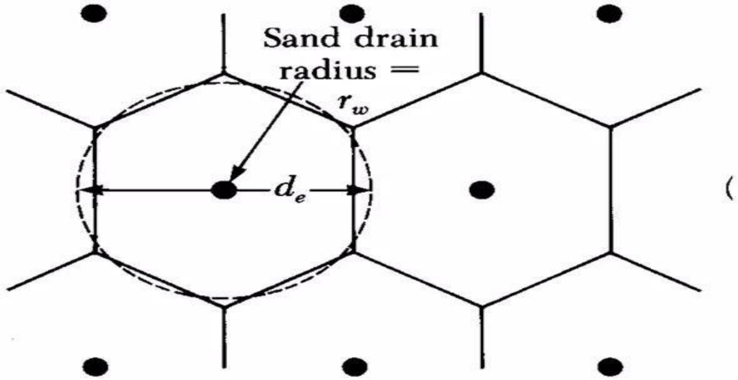
d_e = Effective Diameter
r_w = Sand Drain Radius

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

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

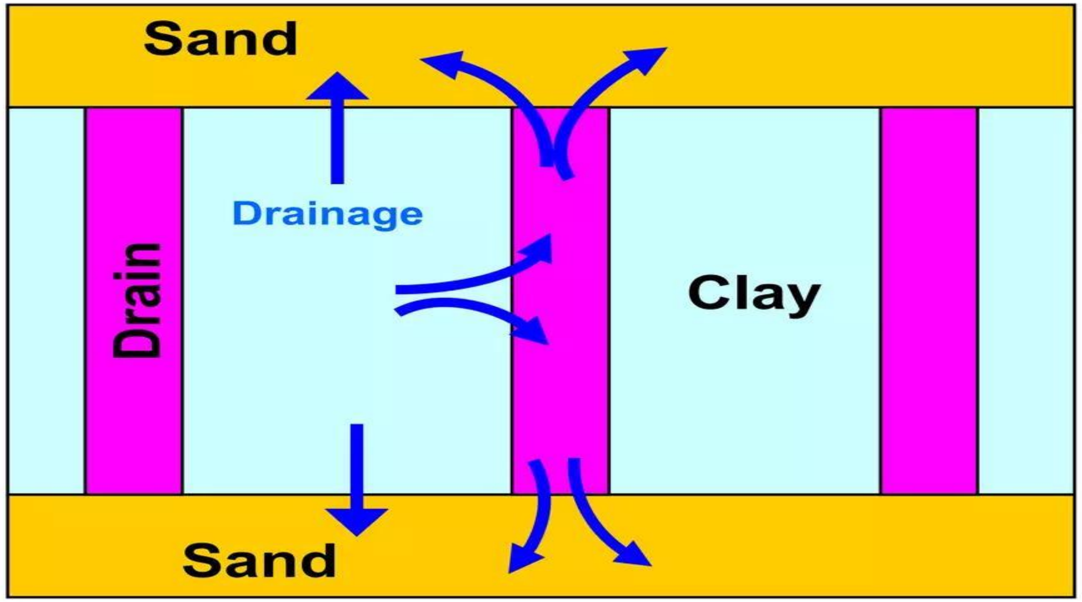
Time Rate of Radial Consolidation

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:

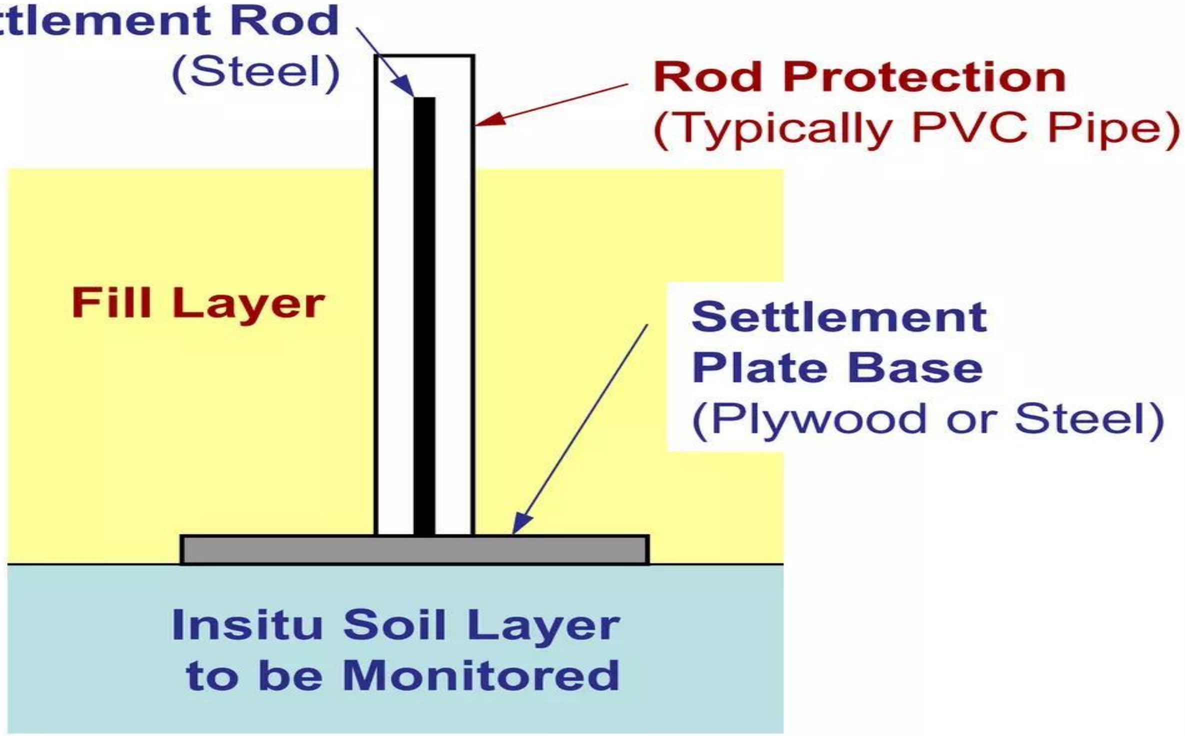
$U_{v,r}$ = Average Degree of Consolidation due to Vertical & Radial Drainage

U_v = Average Degree of Consolidation due to Vertical Drainage

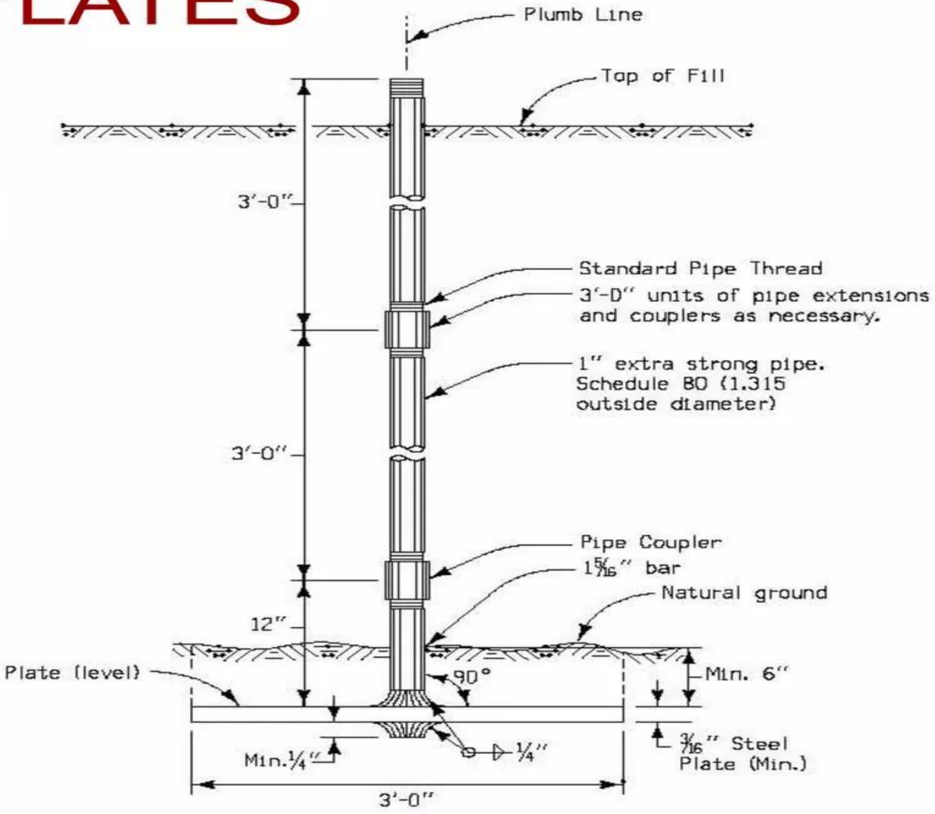
U_r = Average Degree of Consolidation due to Radial Drainage

Consolidation Monitoring

Settlement Plates

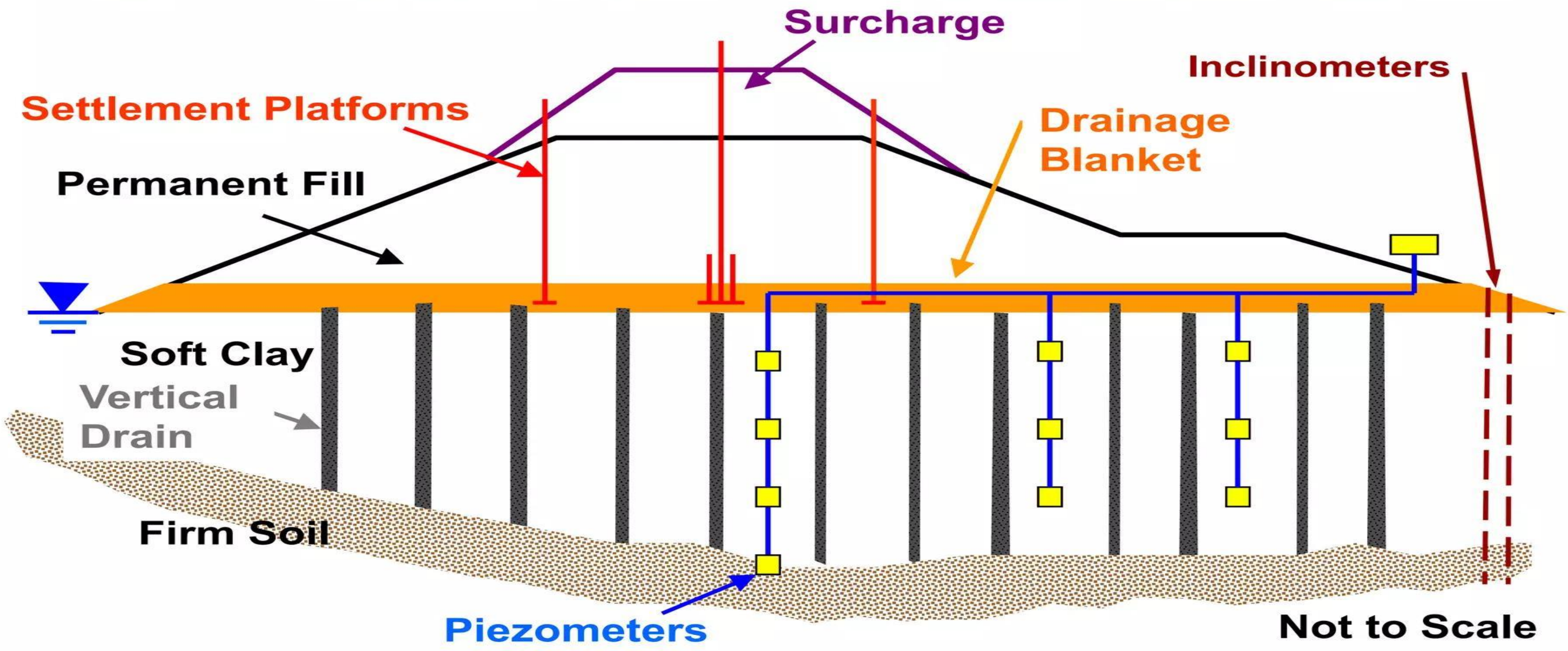


General Concept



Standard Plan Detail
(Courtesy of Iowa DOT)

SURCHARGING INSTRUMENTATION EXAMPLE



Courtesy of www.nhi.fhwa.dot.gov

Click The Link

For Settlement Analysis Math, Please see chapter 10

Soil Report Study

Week 11-13

Click The Link (pdf)

Soil Report

PDF

Soil Test Report (2).pdf

Click The Link (pdf)

[Geotechnical Engineering II Manual]

[All tests easy representation pdf]

Review and Problem Solving Class

Week 14-15

Viva, practical exams

Week 16-17

Any Questions?

Thank you for your kind attention

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