CE 344
Geotechnical Engineering Sessional-I
(Lab Manual)

Department of Civil Engineering
Ahsanullah University of Science and Technology

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Preface

Geotechnical Engineering is the specialty of Civil Engineering which deals with the property and behavior of soil and rock in engineering purposes. To obtain different properties of soil, laboratory tests are performed on collected disturbed and undisturbed soil samples, while field tests are performed on sub-soil at in-situ condition following mainly standard ASTM methods. In Bangladesh mainly wash boring method is adopted to make holes which are known as Bore Holes (BH) and from these holes, disturbed samples are collected from different layers by mainly split spoon sampler as well as undisturbed samples are collected by thin-walled tube/Shelby tube during soil exploration/investigation programme at the site. This Lab manual mainly deals with the common and standard laboratory tests on different types of soil. Field identification tests of soil and laboratory tests like- grain size analysis by sieve and hydrometer tests, specific gravity test, moisture content determination test, organic content determination test, Atterberg limits test, compaction test, relative density test and direct shear test may be performed on collected disturbed soil samples, whereas permeability test, unconfined compression test, consolidation test will be performed on collected undisturbed soil samples according to ASTM (American Standards for Testing Materials) methods.

Prepared by
Department of Civil Engineering
Ahsanullah University of Science and Technology

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Experiment No. 1

FIELD IDENTIFICATION TEST

Standard practice for Description and Identification of Soils (Visual-Manual procedure)

Scope of the test

- The practice covers the procedures for the description of soils for engineering use.
- The identification is based on Visual- examination and manual test. It must be clearly stated in reporting identification that it is based on visual-manual procedure.

Standard Reference

ASTM D2488-17 - Standard Practice for Description and Identification of Soils (Visual - Manual Procedure)

Significance

The first step in any geotechnical engineering project is to identify and describe the subsoil condition. For example, as soon as a ground is identified as gravel, engineer can immediately form some ideas on the nature of problems that might be encountered in a tunneling project. In contrast, a soft clay ground is expected to lead to other types of design and construction considerations. Therefore, it is useful to have a systematic procedure for identification of soils even in the planning stages of a project. Soils can be classified into two general categories: (1) coarse grained soils and (2) fine grained soils. Usually coarse-grained soils are sand, gravel, cobble and boulder, while fine-grained soils are silt and clay. Procedures for visually identification of fine grained soils are described in the following sections.

Apparatus

- Spatula
- Wooden hammer
- Beaker
- Glass rod / Stirrer
- 1/8 inch dia steel rod

Identification of Peat

A sample composed primarily of vegetable tissue in various stages of decomposition that has a fibrous to amorphous texture, usually a dark brown to black color, and organic odor shall be designated as a highly organic soil and shall be identified as peat, PT.

Figure 1.1: Peat Soil

Preliminary identification

Soils can be classified into two general categories: (1) coarse grained soils and (2) fine grained soils. Examples of coarse-grained soils are gravels and sands. Examples of fine-grained soils are silts and clays. Procedures for visually identifying these two general types of soils are described in the following sections.

- The soil is fine grained if it contains 50% or more fines.
- The soil is coarse grained if it contains less than 50% fines.

(1) Identify the color (e.g. brown, gray, brownish gray), odor (if any) and texture (coarse or fine-grained) of soil.

(2) Identify the major soil constituent (>50% by weight) using Table 1.1 as coarse gravel, fine gravel, coarse sand, medium sand, fine sand, or fines.

(3) Estimate percentages of all other soil constituents using Table 1.1 and the following terms:
Trace - 0 to 5% by weight
Few – 5 to 10 %
Little - 15 to 25%
Some - 30 to 45%
Mostly - 50 to 100%
(Examples: trace fine gravel, little silt, some clay)

Table 1.1: Grain size distributions (Engineering Properties of Soils Based on Laboratory Testing Prof. Krishna Reddy, UIC)

Soil Constituent	Size Limits	Familiar Example
Boulder	12 in. (305 mm) or more	Larger than basketball
Cobbles	3 in (76 mm) -12 in (305 mm)	Grapefruit
Coarse Gravel	¾ in. (19 mm) – 3 in. (76 mm)	Orange or Lemon
Fine Gravel	4.75 mm (No.4 Sieve) – ¾ in. (19 mm)	Grape or Pea
Coarse Sand	2 mm (No.10 Sieve) – 4.75 mm (No. 4 Sieve)	Rocksalt
Medium Sand	0.42 mm (No. 40 Sieve) – 2 mm (No. 10 Sieve)	Sugar, table salt
Fine Sand*	0.075 mm (No. 200 Sieve) – 0.42 mm (No. 40 Sieve)	Powdered Sugar
Fines	Less than 0.0075 mm (No. 200 Sieve)	-

*Particles finer than fine sand cannot be discerned with the naked eye at a distance of 8 in (20 cm).

(4) If the major soil constituent is sand or gravel: Identify particle distribution. Describe as well graded or poorly graded. Well-graded soil consists of particle sizes over a wide range. Poorly graded soil consists of particles which are all about the same size. Identify particle shape (angular, subangular, rounded, subrounded) using Figure 1.2 and Table 1.2.

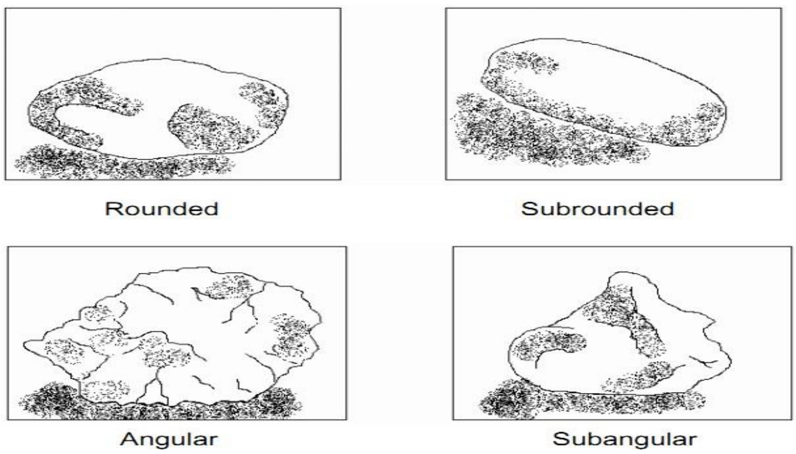


Figure 1.2 Shapes of coarse-grained soil particles (Engineering Properties of Soils Based on Laboratory Testing Prof. Krishna Reddy, UIC)

Table 1.2: Criteria for describing shape of coarse-grained soil particle (Engineering Properties of SoilsBased on Laboratory Testing Prof. Krishna Reddy, UIC)

Description	Criteria
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces.
Subangular	Particles are similar to angular description, but have rounded edges.
Subrounded	Particles have nearly plane sides, but have well-rounded corners and edges.
Rounded	Particles have smoothly curved sides and no edges.

(5) According to the major soil constituents are, perform the following tests:

Procedure for Identifying Coarse-grained Soils

Gravel, sand and Fines

(1) The percentages of the following particle fractions are estimated

(i) Gravel fraction (75 mm – 4.75 mm or approximately 5 mm),
(ii) Sand fraction (4.75 or 5 mm – 75 micron), and
(iii) Soil fines, i.e., silt and clay fraction (smaller than 75 micron).

According to USCS followed by USBR (USA), Army Corps of Engineers ASTM and ASCE:
Range of silt size is 0.002 mm to 0.074 mm and clay particles are smaller than 0.002 mm. (After S.K. Garg – Soil Mechanics and Foundation Engineering, Sixth Revised Edn. 2005, p. 17)

(2) If the gravel fraction is greater than sand fraction, identify the soil as "gravel" (G). The gravel fraction may be further divided into

(i) coarse gravel (75 mm-20 mm) and
(ii) fine gravel (20 mm-4.75 mm).

(3) If the gravel fraction is equal to or less than sand fraction, identify the soil as "sand" (S). The sand fraction may be further divided into

(i) Coarse sand (4.75 mm – 2 mm),
(ii) Medium sand (2 mm – 425 micron), and
(iii) Fine sand (425 micron – 75 micron)

(4) Identify the soil further as —clean gravel‖ or "clean sand" if the percentage of fines is estimated to be less than 5 percent. Identify it as "gravel with fines" or "sand with fines" if the percentage of fines is estimated to be more than 15%.

(5) Classify the "clean gravels" or "clean sand" as follows:
Identify the soil as well graded gravel (GW), or as a well graded sand (SW) if there is good representation of all particle sizes.
Identify the soil as poorly graded gravel (GP), or as a poorly graded sand (SP) if it contains predominantly of one size (uniformly graded) or it has a wide range of sizes with some intermediate size(s) obviously missing (gap graded).

(6) Classify "gravel with fines" or "sand with fines" as follows:
(a) If the other course grained constituent is less than 15% then:
Silty gravel (GM) or silty sand (SM); if the fines have little or no plasticity, or
Clayey gravel (GC) or clayey sand (SC), if the fines are of low to medium or high plasticity.
(b) If the other course grain constituent is greater than 15%, then the group name shall be a combination from the two columns below:

Silty gravel	With sand
Clayey gravel	
Silty sand	With gravel
Clayey sand	

(7) If the percentage of fine is in between 5~15% then the group name shall be a combination from the two columns below:

Well graded gravel	With clay
Poorly graded gravel	With silt
Well graded sand	
Poorly graded sand	

Boundary classifications: Assume the coarser soil first, when there is a choice, complete the classification and assign the appropriate symbol.

Then beginning where the choice was made, assume the finer soil, complete the classification and assign the second group symbol. The examples are as follows: GW-GP, GM-GC, GW-GM, GW-GC SW-SP, SM-SC, SW-SM, SW-SC, GW-SW, GP-SP, GM-SM, GC-SC, SM-ML, SC-CL, etc.

Field Identification of Fine-grained Soils (Silts and Clays)

Select a representative sample of the material for examination. Silt and clay particles are not visible to naked eyes. The amount of silt and clay in a soil sample can be identified by the following field tests:

- Dispersion Test
- Dry strength Test
- Dilatancy Test/ Shaking Test/ Water Mobility Test
- Plasticity Test/Toughness Test

1. <u>Dispersion Test</u>:

- A small quantity of the collected soil sample is dispersed or mixed with water in a glass cylinder or test tube or beaker and then allowed to settle.
- Silt particles usually settle in 15 to 60 min. whereas clay particles will remain in suspension for at least several hours or may remain even for several days. If some sand particles are present in sample, then it settles in 30 to 60 sec.

Figure 1.3: Dispersion Test (https://civil-engg-world.blogspot.com)

2. <u>Dry Strength Test</u>:

- At first the soil samples are molded. Then prepare four or five pats from this molded sample, about 25 mm in diameter and 6 mm in thickness.
- Dry all the pats completely (either naturally or in an oven).
- Measure its resistance to crushing between the fingers. This resistance, called the dry strengthor crushing strength, is a measure of the plasticity of the soil.
- Crushing of dry clay lumps is relatively difficult, whereas silt lumps break quite easily.
- Test the strength of the dry pats or lumps by crushing between the fingers and note the strength in accordance with the criteria in Table 1.3.

Figure 1.4: Dry Strength Test

Table 1.3: Dry Strength Test (Engineering Properties of Soils Based on Laboratory Testing Prof. Krishna Reddy, UIC)

Description	Criteria
None	The dry specimen crumbles into powder with mere pressure of handling
Low	The dry specimen crumbles into powder with some finger pressure
Medium	The dry specimen break into pieces or crumbles with considerable finger pressure
High	The specimen cannot be broken with finger pressure. Specimen will break into pieces between thumb and hand surface.
Very High	The specimen cannot be broken between the thumb and hard surface.

The presence of high-strength water soluble cementing materials, such as calcium carbonates, may cause exceptionally high dry strength. The presence of calcium carbonate can usually be deleted from the intensity of the reaction with dilute hydrochloric acid.

3. <u>Dilatancy Test</u>:
From the specimen select enough material to mold into a ball about 0.5" in diameter. Mold the material, adding water if necessary, until it has a soft, but not sticky, consistency. Smooth the soil ball in the palm of one hand with the blade of a knife or small spatula. Shake horizontally, striking the side of the hand vigorously against the other hand several times. Note the reaction of water appearing on the surface of the soil. Squeeze the sample by closing the hand or punching the soil between the fingers, and note the reaction in accordance with the criteria in Table 1.4.

Table 1.4: Dilatancy Test (Engineering Properties of Soils Based on Laboratory Testing Prof. Krishna Reddy, UIC)

Description	Criteria
None	No visible change in specimen
Slow	Water appears slowly on the surface of the specimen during shaking and does not disappear or disappear slowly upon squeezing.
Rapid	Water appears quickly on the surface of the specimen during shaking and disappears quickly upon squeezing.

4. <u>Toughness or plasticity Test</u>:

- Following the completion of the dilatancy test, the test specimen is shaped into an elongated pat and rolled by hand on a smooth surface or between the palms into a thread about 1/8 in, in diameter. If the sample is too wet to roll easily, it should be spread into a thin layer and allowed to lose some water by evaporation.
- Fold the sample threads and reroll repeatedly until the thread crumbles at a diameter of about 1/8 in. The thread will crumbles at a diameter of 1/8 in, when the soil is near the plastic limit. Note the pressure required to roll the thread near the plastic limit.
- Also note the strength of the thread. After the thread crumbles, note the toughness of the materials during kneading.

➢ In a word, it is expressed as: clay can be rolled out into small threads (about 1/8in. thickness), whereas silt is much more difficult to roll into small threads and generally requires more water.

Figure 1.5: Toughness and plasticity Test

Table-1.5: Toughness Test of plastic thread (Engineering Properties of Soils Based on Laboratory Testing Prof. Krishna Reddy, UIC)

Description	Criteria
Low	Only slight pressure is required to roll the thread near the plastic limit. The thread and the lump are weak and soft
Medium	Medium pressure is required to roll the thread near the plastic limit. The thread and the lump have medium stiffness.
High	Considerable pressure is required to roll the thread near the plastic limit. The thread and the lump have very high stiffness.

Table 1.6: Plasticity Test (Engineering Properties of Soils Based on Laboratory Testing Prof. Krishna Reddy, UIC)

Description	Criteria
Nonplastic	A 1/8 in, thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easily to roll and not much time is required to reach the plastic limit. The thread cannot be rolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

Combining all four field test result:

Table 1.7: Field Identification of Fine Grained Soil (Engineering Properties of Soils Based on Laboratory Testing Prof. Krishna Reddy, UIC)

Typical Name	Dry strength	Dilatancy reaction	Toughness of plastic thread	Time to settle in dispersion test
Sandy silt	None to very high	Rapid	Low	30sec to 60 min
Silt	Very low to low	Rapid	Low	15 to 60 min
Clayey silt	Low to medium	Rapid to slow	Medium	15 to several hrs
Sandy clay	Low to high	Slow to none	Medium	30 sec to several hrs
Silty clay	Medium to high	Slow to none	Medium	15 min to several hrs
Clay	High to very high	None	High	Several hours to days
Organic silt	Low to medium	Slow	Low	15 min to several hrs
Organic clay	Medium to very high	None	High	Several hours to days

Table 1.8: Identification of inorganic fine-grained soil (Engineering Properties of Soils Basedon Laboratory Testing Prof. Krishna Reddy, UIC)

Soil Symbol	Dry Strength	Dilatancy	Toughness
ML	None or Low	Slow to Rapid	Low or thread cannot be formed
CL	Medium to High	None to Slow	Medium
MH	Low to Medium	None to Slow	Low to Medium
CH	High to Very High	None	High

Note: ML = Silt; CL = Lean Clay (low plasticity clay); MH = Elastic Soil; CH = Fat Clay (high plasticity clay). The terms 'lean' and 'fat' may not be used in certain geographic regions (midwest).

Table 1.9: Criteria for describing soil-moisture condition (Engineering Properties of SoilsBased on Laboratory Testing Prof. Krishna Reddy, UIC)

Description	Criteria
Dry	Soil is dry to the touch, dusty, a clear absence of moisture
Moist	Soil is damp, slight moisture; soil may begin to retain molded form
Wet	Soil is clearly wet; water is visible when sample is squeezed
Saturated	Water is easily visible and drains freely from the sample

One moistens a spot on the soil and rubs a finger on it. If the rubbed spot appears smooth, the material is clay, but if it appears scratched, it is silt or silty.

DATA SHEET

Experiment Name : **Field Identification of Soil**
Experiment Date :

Student's Name :
Student's ID :
Year/ Semester :
Section/ Group :

1. Color :
2. Odor :
3. Texture :
4. Major soil constituent :
5. Minor soil constituents :

6. For coarse-grained soils:

Gradation :
Particle Shape :

7. For fine-grained soils:

Name of Test	Observation	Probable soil Type (Considering individual tests)	Conclusion on soil type considering all the four field tests
Dry strength			
Dilatancy reaction			
Toughness of plastic thread			
Time to settle in dispersion test			

8. Moisture Condition:

Classification :
Course Teacher :
Designation :

Signature

Experiment No. 2

SPECIFIC GRAVITY TEST

Introduction

In general the specific gravity G_s of a material represents the ratio of the mass of a given volume of that material at a temperature to the mass of an equal volume of distilled water at the same temperature.

$$G_s\ (\text{at T°C}) = \frac{Wt.of\ a\ Given\ Volume\ of\ Material\ at\ \text{T°C}}{Wt.of\ Same\ Volume\ of\ Water\ at\ \text{T°C}}$$

However, for geotechnical Engineering purpose, the Sp. Gr. of soils are usually reported for water temperature at 20°C.

$$G_s\ (\text{at } 20^{\circ}\text{C}) = \frac{Wt.of\ a\ Given\ Volume\ of\ Material\ at\ \text{T°C}}{Wt.of\ Same\ Volume\ of\ Water\ at 20^{\circ}C}$$

From this basic definition, specific gravity can be obtained from different formulae such as :

$$G_s\ (\text{at } 20^{\circ}\text{C}) = \frac{Wt.of\ unit\ volume\ of\ soil\ at\ \text{T°C}}{Wt.of\ unit\ volume\ of\ water\ at 20^{\circ}C}$$

If the specific gravity of soil for water temperature T0C is known then it can be converted to the specific gravity for water temperature 20°C as follows:

$$G_s\ (\text{at } 20^{\circ}\text{C}) = \frac{Wt.of\ unit\ volume\ of\ soil\ at\ \text{T°C}}{Wt.of\ unit\ volume\ of\ water\ at 20^{\circ}C}$$

$$= \frac{Wt.of\ unit\ volume\ of\ soil\ at\ \text{T°C}}{Wt.of\ unit\ volume\ of\ water\ at\ \text{T°C}} \text{ x } \frac{Wt.of\ unit\ volume\ of\ water\ at\ \text{T°C}}{Wt.of\ unit\ volume\ of\ water\ at 20^{\circ}C}$$

$$= G_s\ (\text{at T °C}) \text{ x } \frac{\gamma_{W\ at\ T°C}}{\gamma_W\ at\ 20°C}$$

$$= G_s\ (\text{at T °C}) \text{ x } \frac{G_{W\ at\ T°C}}{G_W\ at\ 20°C}$$

It is usually determined by the principle of displacement of water by soil in a density bottle, volumetric flask, pycnometer or a gas jar.

Table 2.1 Typical Values of Specific Gravity or Soil

Soil Type	**Specific Gravity**
Sand	2.65 – 2.68
Silty sand	2.67 – 2.70
Inorganic silt	2.62 – 2.68
Inorganic clay	2.68 – 2.80
Organic clay	2.58 – 2.65
Soils with mica or iron	2.75 3.00
Organic soils	May be < 2.0 (2.2 – 2.64)
Soil with micas or iron	2.75 – 3.00

Table 2.2 Specific Gravity of water at different temperature (T.W. Lambe, Soil Testing for Engineers)

0^{o}C	0	1	2	3	4	5	6	7	8	9
	0.9999	0.9999	1.0000	1.0000	1.0000	1.0000	1.0000	0.9999	0.9999	0.9998
10	0.9997	0.9996	0.9995	0.9994	0.9993	0.9991	0.9990	0.9988	0.9986	0.9984
20	0.9982	0.9980	0.9978	0.9976	0.9973	0.9971	0.9968	0.9965	0.9963	0.9960
30	0.9957	0.9954	0.9951	0.9947	0.9944	0.9941	0.9937	0.9934	0.9930	0.9926
40	0.9922	0.9919	0.9915	0.9911	0.9907	0.9902	0.9898	0.9894	0.9890	0.9885
50	0.9881	0.9876	0.9872	0.9867	0.9862	0.9857	0.9852	0.9848	0.9842	0.9838
60	0.9832	0.9827	0.9822	0.9817	0.9811	0.9806	0.9800	0.9795	0.9789	0.9784
70	0.9778	0.9772	0.9767	0.9761	0.9755	0.9749	0.9743	0.9737	0.9731	0.9724
80	0.9718	0.9712	0.9706	0.9699	0.9693	0.9686	0.9680	0.9673	0.9667	0.9660
90	0.9653	0.9647	0.9640	0.9633	0.9626	0.9619	0.9612	0.9605	0.9598	0.9591

Scope of the test

This test is performed to determine the specific gravity of soil by using a pycnometer. Specific gravity is the ratio of unit weight of soil at a stated temperature to the unit weight of same volume of gas-free distilled water at a stated temperature.

Standard reference

ASTM D854 – Standard Test for Specific Gravity of Soil Solids by Water Pycnometer.

Equipment

- Pycnometer (volumetric bottle)
- Balance (0.01g sensitivity), Distilled water, Drying oven, Desiccator,
- Vacuum source (optional)
- Funnel, Thermometer, Pipette or medicine dropper, Spoon etc.

Significance

- Specific gravity of a soil is used in the phase relationship.
- Soil density calculation.

<u>Test Procedure</u>

A. Pycnometer Calibration (Theoretical Procedure)

Points for the calibration curve can be obtained by substituting different temperatures in the following equations:

$W_2 = W_B + V_B (1 + \Delta T.\ \varepsilon) (\gamma_T - \gamma_a)$

W_2 = weight of the bottle + water

W_B = weight of clean and dry bottle

V_B = calibrated volume of bottle at T_c

$\Delta T = T - T_c$

T = Temp. in ^{0}C at which W_2 is desired

T_c = calibration temp. of bottle (usually 20°C)

ε = thermal coefficient of cubical expansion for pyrex glass = 0.100 x 10^{-4} per ^{0}C

γ_T = unit wt. of water at T°C (See Table A-2, Soil Testing - Lambe, p-147)

γ_a = unit wt. of air at T and atmospheric pressure = average value, 0.0012 g/cc)

B. Specific Gravity Determination

(i) For Cohesionless Soil

(1) Put about 150g of oven-dry soil into pycnometer (without losing any soil grains) which is already half full of deaired distilled water.

(2) Remove all of the air which is entrapped in the soil by 10 min., accompany the boiling with continuous agitation.

(3) Cool the bottle and suspension to some temperature within the range of the calibration curve for the bottle.

(4) Add water to bring the bottom of the meniscus to the calibration mark.

(5) Dry the outside of the bottle and the inside of the neck above the meniscus.

(6) Weigh the bottle with water and soil in it to 0.01g.

(7) After checking to be sure that the contents of the bottle are at a uniform temperature, record the temp.

(ii) For Cohesive Soil

(1) Work a sample of the soil to be tested into a smooth paste by mixing it with distilled water. The sample used should contain approximately 50g in dry weight.

(2) Pour the paste into a calibrated pycnometer

(3) Remove the entrapped air, cool, and obtain the weight and temperature as was done in steps 2-7 of the procedure for the cohesionless soil.

(4) Pour the entire mixture of soil and water into a large evaporating dish of known weight.

(5) Dry the soil in the oven, cool, and weigh. From this the dry weight of the soil grains can be obtained.

Alternative Procedure:

(1) Determine and record the weight of the empty clean and dry pycnometer.

(2) Place 50g of a dry soil sample (passed through the sieve No. 10) in the pycnometer. Determine and record the weight of the pycnometer containing the dry soil.

(3) Add distilled water to fill about half to three-fourth of the pycnometer. Soak the sample for 10 minutes.

(4) Apply a partial vacuum to the contents for 10 minutes, to remove the entrapped air.

(5) Stop the vacuum and carefully remove the vacuum line from pycnometer.

(6) Fill the pycnometer with distilled (water to the mark), clean the exterior surface of the pycnometer with a clean, dry cloth. Determine the weight of the pycnometer and contents.

(7) Empty the pycnometer and clean it. Then fill it with distilled water only (to the mark). Clean the exterior surface of the pycnometer with a clean, dry cloth. Determine the weight of the pycnometer and distilled water

(8) Empty the pycnometer and clean it.

Sample Calculation

The specific gravity of the soil, $G_s = \frac{W_s\, G_T}{W_s - W_1 + W_2}$

where G_T = specific gravity of distilled water at temperature T (from Table)

W_s = dry weight of soil

W_1 = weight of pycnometer + soil + water

W_2 = weight of pycnometer + water (from calibration curve)

For Alternative Procedure: $G_s = \frac{(M2-M1)}{(M4-M1)-(M3-M2)} G_T$

Here,
M_1=Weight of Pycnometer
M_2=Weight of Pycnometer+Soil
M_3=Weight of Pycnometer+Soil+Water
M_4=Weight of Water + Pycnometer
G_T=Specific gravity of distilled water. (If normal tap water is used)
(You will get value of G_T from Soil Testing for Engineers-Lambe: Appendix A-Table A2)

Figure 2.1: (a) Weight a blank pycnometer (b) Fill the pycnometer with 50gm soil and take weight (c) Fill the pycnometer with water up to mark along with soil (d) Use vacuumed pump to remove voids (e) taking weight of soil and water with pycnometer (f) Fill the pycnometer with only water up to mark and take weight

DATA SHEET

Experiment Name : **Specific Gravity Determination of Soil**
Experiment Date :

Student Name :
Student's ID :
Year/ Semester :
Section/ Group :

Sample Description :
Data Table :

Specimen number	1	2
Pycnometer bottle number		
Weight of Pycnometer, M_1		
Weight of Pycnometer + Soil, M_2		
Weight of Pycnometer + Soil+Water, M_3		
Weight of Pycnometer + Water, M_4		
Specific gravity of distilled water, G_T		
Specific gravity of the Soil, G_S		

Sample Calculation :

$$G_s = \frac{(M2-M1)}{(M4-M1)-(M3-M2)} G_T$$

Result:
Specific Gravity of the soil is :

Course Teacher :
Designation :

Signature

Experiment No. 3
GRAIN SIZE ANALYSIS
(SIEVE ANALYSIS)

Introduction

Sieve analysis is performed to determine the gradation (or grain size distribution) of different particle sizes of coarse grained soil (sand) or coarser portion of a soil containing both coarse (sand & silt) and fine (silt & clay) particles. A sieve analysis consists of shaking the soil through a stack of wire screens with openings of known sizes; the definition of particle diameter for a sieve test is therefore the side dimension of a square hole. It should be noted that sieve analysis does not give any idea about the shape of the particles.

Sieves are made of woven wires with square openings. Table 4 gives a list of U.S. standard sieve numbers with the corresponding size of openings. For all practical purposes #200 sieve is the sieve with the smallest opening that should be used for the test. The sieves that are commonly used in the test have diameter of 8 inch (203mm).

Table 3.1 Sieve no. and openings (Principles of Geotechnical Engineering, B.M. Das)

Sieve no.	Opening	Sieve no.	Opening
4	4.75	**35**	0.50
5	4.00	**40**	0.425
6	3.35	**45**	0.355
7	2.80	**50**	0.30
8	2.36	**60**	0.25
10	2.00	**70**	0.212
12	1.70	**80**	0.180
14	1.40	**100**	0.150
16	1.18	**120**	0.125
18	1.00	**140**	0.106
20	0.85	**200**	0.075
25	0.71	**270**	0.053
30	0.60	**400**	0.038

Actually for the purpose of gradation of soils there may be one of the following three tests:

1. Sieve analysis – Applicable when the soil contains mainly sand and larger particles.
2. Hydrometer analysis – Applicable when the soil is fine grained or silt and clay particles.
3. Combined analysis i.e. both sieve and hydrometer analysis – Applicable when the soil contains sizable portion of both fine and coarse fraction.

From sieve analysis, information about grain size distribution is obtained. Besides the gradation curve, other parameters namely uniformity coefficient C_u, coefficient of curvature, C_c are determined and all these information are used for classification.

It is pointed here that FM (Fineness Modulus) is determined from sieve analysis done on fine and coarse aggregates (as you have done in concrete laboratory). However, FM has almost no meaning and use in geotechnical interpretation and understanding of soil behaviour as a foundation material.

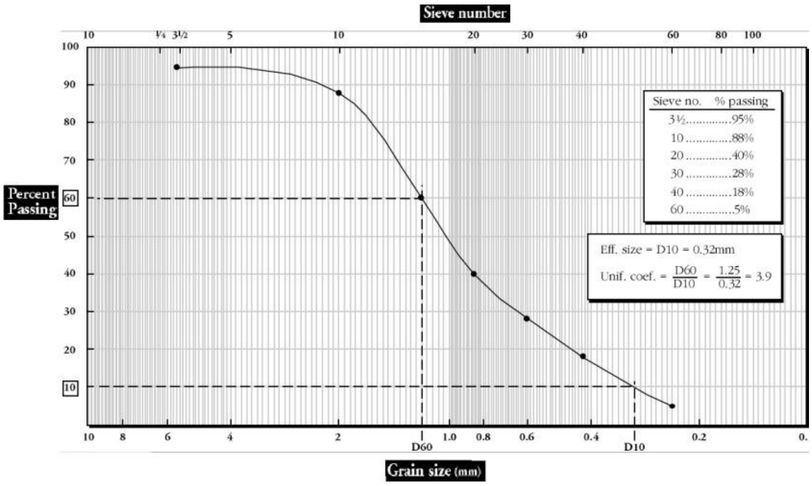


Figure 3.1: Gradation Curve (www.aboutcivil.org/Sieve-analysis-and-soil-classification.html)

Scope of the test

This test is performed to determine the percentage of different grain sizes contained within a soil. The mechanical or sieve analysis is performed to determine the distribution of the coarser, larger-sized particles, and the hydrometer method is used to determine the distribution of the finer particles.

Significance

The distribution of different grain sizes affects the engineering properties of soil. Grain size analysis provides the grain size distribution and it is required in classifying the soil.

Standard Reference

ASTM D6913-04 - Standard Test Method for particle Size Analysis of Soils

Apparatus

- ASTM Sieve #4, #8, #16, #30, #50, #100, #200
- Pan
- Lead
- Brush
- Container
- Spoon
- Bowel
- Balance
- Sieve shaker

Test Procedure

(1) Write down the weight of each sieve as well as the bottom pan to be used in the analysis.

(2) Record the weight of the given dry soil sample.

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

(4) Place the sieve stack in the mechanical shaker and shake for 10 minutes.

(5) Remove the stack from the shaker and carefully weigh and record the weight of each sieve with its retained soil. In addition, remember to weigh and record the weight of the bottom pan with its retained fine soil.

Data Analysis

(1) Obtain the mass of soil retained on each sieve by subtracting the weight of the empty sieve from the mass of the sieve + retained soil, and record this mass as the weight retained on the data sheet. The sum of these retained masses should be approximately equals the initial mass of the soil sample. A loss of more than two percent is unsatisfactory.

(2) Calculate the percent retained on each sieve by dividing the weight retained on each sieve by the original sample mass.

(3) Calculate the percent passing (or percent finer) by starting with 100 percent and subtracting the percent retained on each sieve as a cumulative procedure.

(4) Make a semi logarithmic plot of grain size vs. percent finer.

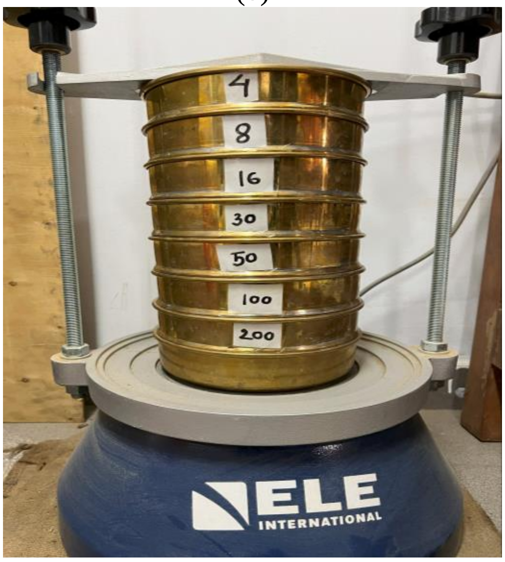
(5) Compute Uniformity coefficient, C_u and Coefficient of gradation, C_z for the soil.

$$C_u=\frac{D_{60}}{D_{10}} \quad \text{and } C_z=\frac{D_{30}^2}{D_{60}D_{10}}$$

(a)

(b)

(c)



(d)

(e)

Figure 3.1: (a) Clean all sieves before test. (b) Weight all sieves (c) Serially set all sieves according to their opening size. (d) After pouring the soil place it to sieve shaker and shake 10 min. (e) Weight all sieve with contained soil

Data Sheet

Experiment Name : **Grain Size Analysis Test by Sieve Analysis**
Experiment Date :

Student's Name :
Student's ID :
Year/ Semester :
Section/ Group :

Visual Classification :

Data Table :

Sieve No.	Sieve opening (mm)	Wt. of container (gm)	Wt. of container + soil (gm)	Wt. of soil retained (gm)	Percent of soil retained	Cumulative percent retained	Percent finer
4	4.76						
8	2.380						
16	1.190						
30	0.590						
50	0.287						
100	0.149						
200	0.074						
Pan							
			Total weight=				

From Grain Size Distribution Curve:

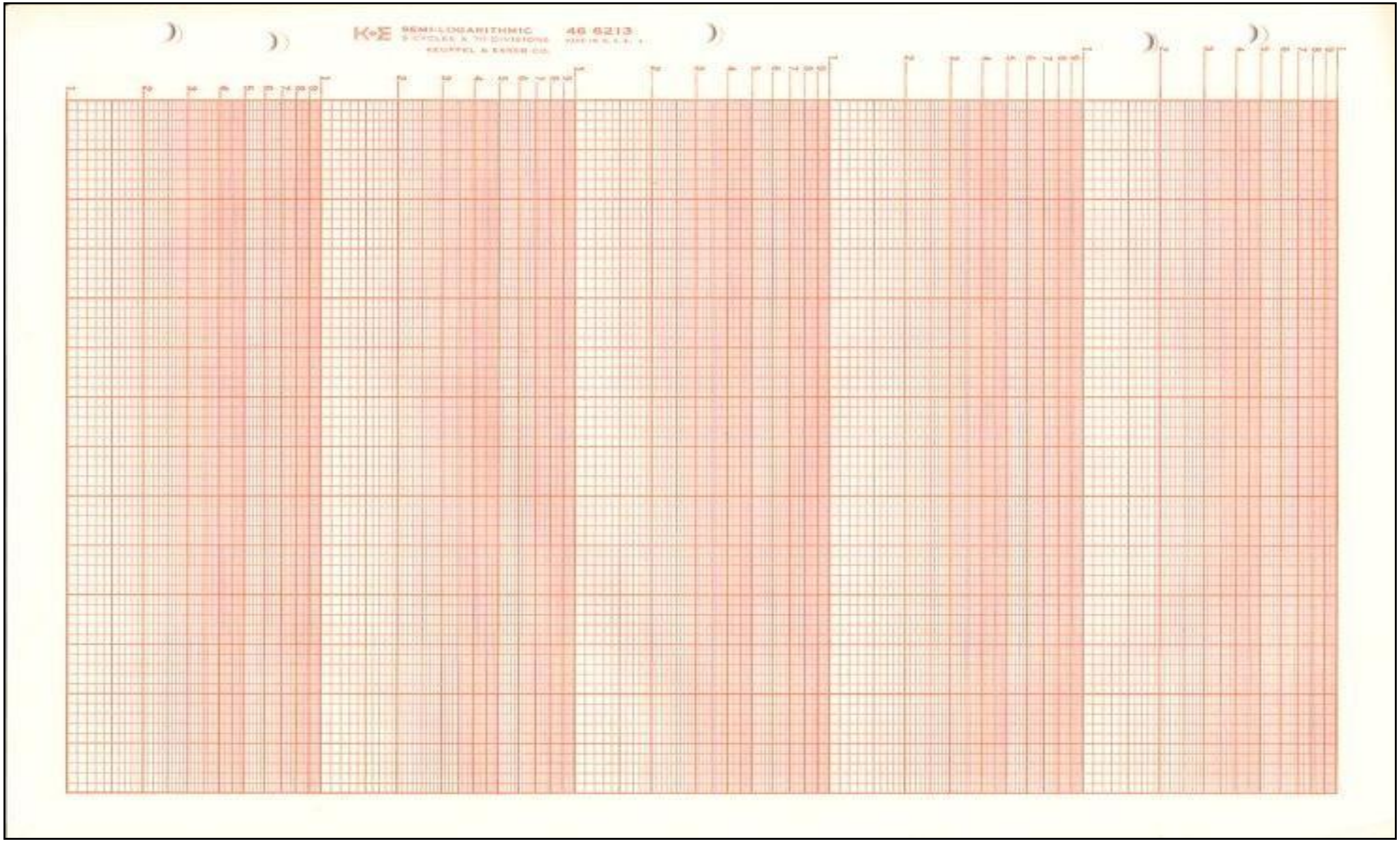
% Gravel :		D10 :	mm	Cu	:
% Sand :		D30 :	mm	C_Z	:
% Fines :		D60 :	mm		

Unified Classification of Soil :

Course Teacher :
Designation :

Signature

Grain size distribution curve



Experiment No. 4
PARTICLE SIZE ANALYSIS BY HYDROMETER

Introduction

Grain size distribution of soil which contain significant amount of finer particles (silt and clay) cannot be done by sieve analysis. Hydrometer analysis is required to determine the grain size distribution of the finer portion. For many natural soils we require both sieve analysis and hydrometer analysis to obtain the complete gradation of the coarse and fine fraction. In the sieve analysis test, you have used #200 sieve (opening 0.074 mm) as the finest sieve. Now you will perform hydrometer analysis on a fine grained soil that passes #200 sieve.

According to ASTM D7928-21, when combined analysis is required, the sample is to be divided into two parts. Sieve analysis is to be done on the coarser portion and hydrometer analysis is to be done into finer portion. Division of the sample into two portion is to be done by either of #4 (4.75mm), #10 (2.00 mm), #40 (0.425mm) or #200 (0.074 mm) sieve depending on the sp.gr. of particles. However, for our natural soils separation on #200 sieve will be sufficient.

Determination particle diameter

Determination of particle diameter using hydrometer is based on Stoke's Law which states that the the terminal velocity (v) of a freely falling sphere through a medium is proportional to the square of the diameter (D) of the particle i.e. $v \propto D^2$. From this we obtain

$$v = \frac{\gamma_s - \gamma_w}{18\eta} D^2$$

where v = velocity (cm/s)

γ_s = Unit wt. of soil particles (gm/cc)

γ_w = Unit wt. of water (gm/cc)

η= Viscosity of water ($\frac{gm.s}{cm^2}$)

D = Diameter of particle (cm)

Therefore, if the velocity of fall v is known, then diameter of the particle can be calculated as

$$D = \sqrt{\frac{18\eta}{\gamma_s - \gamma_w}} . \sqrt{v}$$

To understand the details of hydrometer analysis it is necessary to have an idea about the apparatus ―hydrometer‖. There are two types of ASTM soil hydrometers. One is designated ASTM 151H and the other is designated ASTM 152H. The readings obtained from these two hydrometers have different meanings. Both can be used for hydrometer analysis. But, calculation procedures for these two are different. Here we have described the procedure for ASTM 152H hydrometer that you will use.

The hydrometer, made of glass, has a stem and a bulb. When it is inserted into a liquid, it floats or submerges in an upright position with its bulb on the downside. The stem of the hydrometer has graduation. ASTM 152H hydrometers are calibrated such that when inserted in a soil water suspension the reading on the hydrometer stem indicates grams of soil that are in 1000cc suspension of the density at the center of the bulb. Thus, when the hydrometer reading is zero, it indicates pure water and when the reading is 30 it means that there are 30 gms of soil in 1000cc soil suspension. Usually there are readings from 0 to 60 on a hydrometer stem. It can be easily conceived that as the density of suspension increases, more of the stem will come out of the suspension, and so the hydrometer reading will be increased. If we prepare a soil suspension in a jar and take hydrometer readings at different time intervals from the instant when the suspension is not disturbed by stirring or any other means, then successive hydrometer readings will decrease because particles of soil will start to fall and finally settle at the bottom of the jar resulting in reduced density of the suspension. Now, suppose at any time interval t, the distance from the c.g. (center of gravity) of the bulb to the surface of suspension is L. Because the location of c.g. of the hydrometer bulb is related to the sp.gr. of the suspension around it, L may be considered to be the distance of fall of a particle in time t. Therefore, velocity of the particle will be v=L/t.

Therefore,

$$D = \sqrt{\frac{18\eta}{\gamma_s - \gamma_w}} . \sqrt{\frac{L}{t}}$$

If we want to put t in minute, L in cm, ή in poise (1 gm-sec per square cm =g poise =980.7 poise) and want to obtain D in mm then the above equation should be modified as

$$D\,(mm) = \sqrt{\frac{18\eta}{980.7 * (\gamma_s - \gamma_w)}} . \sqrt{\frac{L}{t * 60}} * 10$$

which upon simplification becomes

$$D\,(mm) = \sqrt{\frac{30\eta(poise)}{980.7 * [\gamma_s(\frac{gm}{cm^3}) - \gamma_w(\frac{gm}{cm^3})]}} . \sqrt{\frac{L(cm)}{t(min)}}$$

$$\text{or } D\,(mm) = \sqrt{\frac{30\eta(poise)}{980.7 * (G_s - 1)\gamma_w(\frac{gm}{cm^3})}} . \sqrt{\frac{L(cm)}{t(min)}}$$

The above equation is sometimes written as

$$D = K . \sqrt{\frac{L}{t}}$$

where $K = \int (T, G_s, \eta)$

For example for T=20^{0}C, γ_w = 0.9971, G_s =2.65,

However, when we insert the hydrometer in a soil-water suspension we do not measure L directly. Instead, if we measure the distance L1 and L2 from the c.g. of the bulb corresponding to two hydrometer readings R1 and R2 then we can calculate L easily for any hydrometer reading R from the equation (see fig.1)

$$L = \frac{L_b}{2} + L_1 - \frac{L_1 - L_2}{R_2 - R_1} . (R_L - R_1) \qquad (1)$$

Here we need to notice that as we insert the hydrometer in the jar containing the soil- water hydrometer bulb by half of this i.e. 2V. Ab$_j$ from the level where it would be if this rise have not occurred (see Fig 4.1d); V_b =Volume of the hydrometer bulb and A_j = Cross-sectional Area of the hydrometer jar. Thus equation 1 need to be modified as

$$L = \frac{L_b}{2} + L - \frac{L_1 - L_2}{R_2 - R_1}(R_L - R_1) - \frac{V_b}{2A_j} \qquad (2)$$

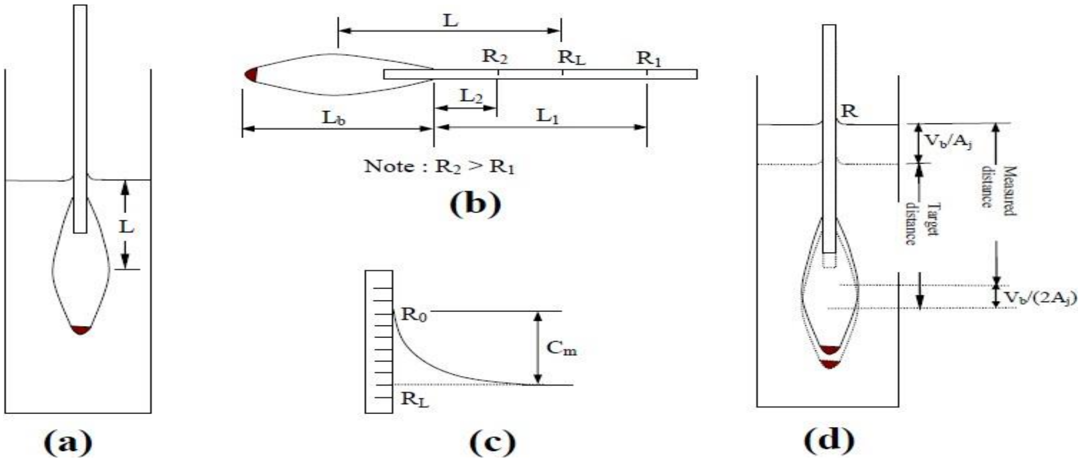


Figure 4.1 Schematic diagrams showing hydrometer readings and corrections.

Furthermore, in the soil-water suspension we can read the upper meniscus, whereas we need the reading at the lower meniscus. So, if R0 be the observed reading, then to put into equation 2, RL should be calculated as (see Fig. 4.1c)

$$R_L = R_0 \pm C_m \quad (3)$$

For ASTM 152H hydrometers for R_1=0 the distance L_1=10.5 cm and for R_2=50, the distance L_2 =2.3 cm, L_b=14 cm, V_b=67 cm^3. Cross-sectional area of hydrometer jar is 27.8 cm^2. For these values, Equation 2 becomes

$$L = 16.29 + 0.164 * R_L \text{ , (L in cm)} \quad (3a)$$

Equation 3a is sometimes presented in tabular form as in Table 1 wherefrom values of L can be obtained for any hydrometer reading.

Table 4.1: Values of Effective depth RL for hydrometer for which (L_1=10.5 cm, L_2=2.3cm, R_1=0, R_2=50, L_b=14.0 cm, V_b= 67 cm^3, A_j=27.8 cm^3)

Hydrometer reading, R_L	L	Hydrometer reading, R_L	L	Hydrometer reading, R_L	L	Hydrometer reading, R_L	L
0	16.3	15	13.8	30	11.4	45	8.9
1	16.1	16	13.7	31	11.2	46	8.8
2	16.0	17	13.5	32	11.0	47	8.6
3	15.8	18	13.3	33	10.9	48	8.4
4	15.6	19	13.2	34	10.7	49	8.3
5	15.5	20	13.0	35	10.6	50	8.1
6	15.3	21	12.9	36	10.4	51	7.9
7	15.1	22	12.7	37	10.2	52	7.8
8	15.0	23	12.5	38	10.1	53	7.6
9	14.8	24	12.4	39	9.9	54	7.4
10	14.7	25	12.2	40	9.7	55	7.3
11	14.5	26	12.0	41	9.6	56	7.1
12	14.3	27	11.9	42	9.4	57	6.9
13	14.2	28	11.7	43	9.2	58	6.8
14	14.0	29	11.5	44	9.1	59	6.6

Determination of percent finer

It has been mentioned earlier that ASTM 152H hydrometers are calibrated such that for soil of sp.gr of 2.65, the hydrometer reading after an elapsed time t, indicates the grams of soil in suspension at that instant. This means that if R_c be the hydrometer reading, then (corresponding to the diameter determined using this reading)

$$\text{Percent finer, N} = \frac{R_c}{W} * 100 \qquad (4)$$

where W_s = Weight (in gms) of soil used to prepare the suspension and R is obtained from R_0 after applying some corrections.

Because the sp.gr. of a soil may vary from 2.65 a correction factor is introduced in equation 4. Thus,

$$\text{Percent finer, N} = a * \frac{R_c}{W} * 100 \qquad (5)$$

Where 'a' is the correction factor introduced to accommodate the variation of specific gravity from 2.65. The correction factor a can be computed as

$$a = \frac{G_s * 1.65}{(G_s - 1) * 2.65} \qquad (6)$$

Values of 'a' for different specific gravity of soil particles may also be obtained from Table 4.

The reading R_c to be used in equation 5 should be obtained from the observed hydrometer reading R_o after applying the following corrections

(a) Correction for meniscus (C_m). This correction is always additive.
(b) Zero correction (C_z). This correction is introduced because a deflocculating agent is used in the preparation of the soil water suspension, which increases the hydrometer reading. This correction is always subtractive.
(c) Temperature correction (C_T). ASTM 152H hydrometers are calibrated at 20^0C. So if the test temperature is different from soil water suspension a correction is needed. The values for temperature correction are given in Table 5.

$$R_c = R_o + C_m - C_z \pm C_T \qquad (7)$$

Now consider that hydrometer analysis is performed on a soil for which percent of material finer for #200 sieve is P. Then with respect to the original soil

$$N' = N * \frac{P}{100}$$

Scope of the test

This test is performed to determine the percentage of different particle sizes contained within a soil. The hydrometer method is used to determine the distribution of the finer particles.

Apparatus

- Sedimentation Cylinder
- Hydrometer
- Hydrometer Jar bath
- Dispersive agent, Sodium hexa metaphosphate ($NaPO_3$)
- Thermometer

Figure 4.2 Hydrometer Test Apparatus

Test Procedure

(1) Take the fine soil from the bottom pan of the sieve set, place it into a beaker, and add 125 mL of the dispersing agent (sodium hexa metaphosphate (40 g/L)) solution. Stir the mixture until the soil is thoroughly wet. Let the soil soak for at least ten minutes.

(2) While the soil is soaking, add 125mL of dispersing agent into the control cylinder and fill it with distilled water to the mark. Take the reading at the top of the meniscus formed by the hydrometer stem and the control solution. A reading less than zero is recorded as a negative (-) correction and a reading between zero and sixty is recorded as a positive (+) correction. This reading is called the **zero correction**. The **meniscus correction** is the difference between the top of the meniscus and the level of the solution in the control jar (Usually about +1). Shake the control cylinder in such a way that the contents are mixed thoroughly. Insert the hydrometer and thermometer into the control cylinder and note the zero correction and temperature respectively.

(3) Transfer the soil slurry into a mixer by adding more distilled water, if necessary, until mixing cup is at least half full. Then mix the solution for a period of two minutes.

(4) Immediately transfer the soil slurry into the empty sedimentation cylinder. Add distilled water up to the mark.

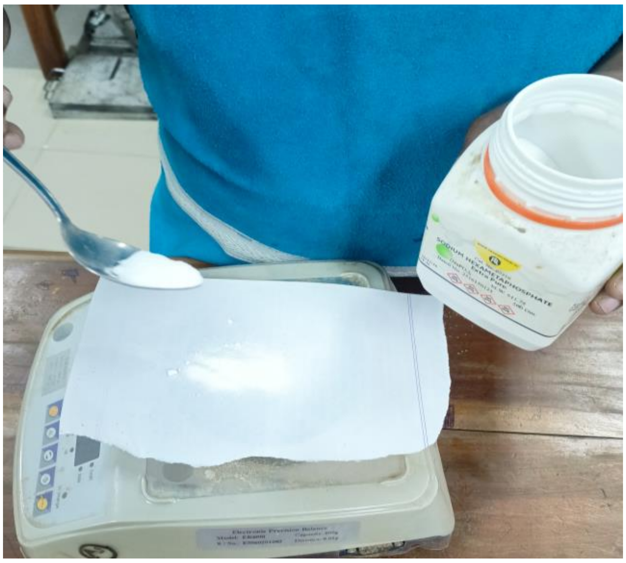
(5) Cover the open end of the cylinder with a stopper and secure it with the palm of your hand. Then turn the cylinder upside down and back upright for a period of one minute. (The cylinder should be inverted approximately 30 times during the minute.)

(6) Set the cylinder down and record the time. Remove the stopper from the cylinder. After an elapsed time of one minute and forty seconds, very slowly and carefully insert the hydrometer for the first reading. (Note: It should take about ten seconds to insert or remove the hydrometer to minimize any disturbance, and the release of the hydrometer should be made as close to the reading depth as possible to avoid excessive bobbing).

(7) The reading is taken by observing the top of the meniscus formed by the suspension and the hydrometer stem. The hydrometer is removed slowly and placed back into the control cylinder. Very gently spin it in control cylinder to remove any particles that may have adhered.

(8) Take hydrometer readings after elapsed time of 2 and 5, 8, 15, 30, 60 minutes and 24 hours.

(a)



(b)

(c)

(d)

(e)

(f)

Figure 4.2: (a) Take the zero correction and meniscus correction (b) Weight the dispersion agent and make a water-agent solution. (c) Take powdered soil and mix with water-agent solution in a pan. (d) Pour the mixture atcylinder carefully (e) Shake very carefully (f) Place the hydrometer at the cylinder and take reading carefully

Table 4.2: Values of k for Use in Equation for Computing Diameter of Particle in Hydrometer Analysis (Engineering Properties of Soils Based on Laboratory Testing Prof. Krishna Reddy, UIC)

Temperature °C	Specific Gravity of Soil Particles								
	2.45	2.50	2.55	2.60	2.65	2.70	2.75	2.80	2.85
16	0.01510	0.01505	0.01481	0.01457	0.01435	0.01414	0.0394	0.01374	0.01356
17	0.01511	0.01486	0.01462	0.01439	0.01417	0.01396	0.01376	0.01356	0.01338
18	0.01492	0.01467	0.01443	0.01421	0.01399	0.01378	0.01359	0.01339	0.01321
19	0.01474	0.01449	0.01425	0.01403	0.01382	0.01361	0.01342	0.01323	0.01305
20	0.01456	0.01431	0.01408	0.01386	0.01365	0.01344	0.01325	0.01307	0.01289
21	0.01438	0.01414	0.01391	0.01369	0.01348	0.01328	0.01309	0.01291	0.01273
22	0.01421	0.01397	0.01374	0.01353	0.01332	0.01312	0.01294	0.01276	0.01258
23	0.01404	0.01381	0.01358	0.01337	0.01317	0.01297	0.01279	0.01261	0.01243
24	0.01388	0.01365	0.01342	0.01321	0.01301	0.01282	0.01264	0.01246	0.01229
25	0.01372	0.01349	0.01327	0.01306	0.01286	0.01267	0.01249	0.01232	0.01215
26	0.01357	0.01334	0.01312	0.01291	0.01272	0.01253	0.01235	0.01218	0.01201
27	0.01342	0.01319	0.01297	0.01277	0.01258	0.01239	0.01221	0.01204	0.01188
28	0.01327	0.01304	0.01283	0.01264	0.01244	0.01255	0.01208	0.01191	0.01175
29	0.01312	0.01290	0.01269	0.01269	0.01230	0.01212	0.01195	0.01178	0.01162
30	0.01298	0.01276	0.01256	0.01236	0.01217	0.01199	0.01182	0.01165	0.01149

Table 4.3: Temperature Correction Factors C_T (Engineering Properties of Soils Based on Laboratory Testing Prof. Krishna Reddy, UIC)

Temperature °C	factor C_T
15	1.10
16	-0.90
17	-0.70
18	-0.50
19	-0.30
20	0.00
21	+0.20
22	+0.40
23	+0.70
24	+1.00
25	+1.30
26	+1.65
27	+2.00
28	+2.50
29	+3.05
30	+3.80

Table 4.4: Correction Factors a for Unit Weight of Solids (Engineering Properties of Soils Based on Laboratory Testing Prof. Krishna Reddy, UIC)

Unit Weight of Soil Solids, g/cm³	Correction factor a
2.85	0.96
2.80	0.97
2.75	0.98
2.70	0.99
2.65	1.00
2.60	1.01
2.55	1.02
2.50	1.04

Table 4.5: Properties of distilled water

Temp., °C	Unit weight of water, g/cm³	Viscosity of water, poise*
4	1.00000	0.01567
16	0.99897	0.01111
17	0.99880	0.01083
18	0.99862	0.01056
19	0.99844	0.01030
20	0.99823	0.01005
21	0.99802	0.00981
22	0.99780	0.00958
23	0.99757	0.00936
24	0.99733	0.00914
25	0.99708	0.00894
26	0.99682	0.00874
27	0.99655	0.00855
28	0.99627	0.00836
29	0.99598	0.00818
30	0.99568	0.00801

$$^{*}\text{Poise} = \frac{\text{dyne} \cdot \text{s}}{\text{cm}^2} = \frac{\text{g}}{\text{cm} \cdot \text{s}}$$

Table 4.6: Value of L (effective depth) for use in Stokes diameters of particles for ASTM soil hydrometer 152H

Original hydrometer reading (corrected for meniscus only)	Effective depth L, cm	Original hydrometer reading (corrected for meniscus only)	Effective depth L, cm	Original hydrometer reading (corrected for meniscus only)	Effective depth L, cm
0	16.3	21	12.9	42	9.4
1	16.1	22	12.7	43	9.2
2	16.0	23	12.5	44	9.1
3	15.8	24	12.4	45	8.9
4	15.6	25	12.2	46	8.8
5	15.5	26	12.0	47	8.6
6	15.3	27	11.9	48	8.4
7	15.2	28	11.7	49	8.3
8	15.0	29	11.5	50	8.1
9	14.8	30	11.4	51	7.9
10	14.7	31	11.2	52	7.8
11	14.5	32	11.1	53	7.6
12	14.3	33	10.9	54	7.4
13	14.2	34	10.7	55	7.3
14	14.0	35	10.5	56	7.1
15	13.8	36	10.4	57	7.0
16	13.7	37	10.2	58	6.8
17	13.5	38	10.1	59	6.6
18	13.3	39	9.9	60	6.5
19	13.2	40	9.7		
20	13.0	41	9.6		

Data Sheet

Experiment Name : **Particle Size Analysis by Hydrometer**
Experiment Date :

Student's Name :
Student's ID :
Year/ Semester :
Section/ Group :

Visual Classification :

Hydrometer Model :
Specific Gravity of Soil :
Dispersing Agent : $NaPO_3$
Weight of Soil Sample, M_S : 50gm

Zero Correction :
Meniscus Correction :
Room Temperature :
Temperature Correction, C_T :

Data Table :

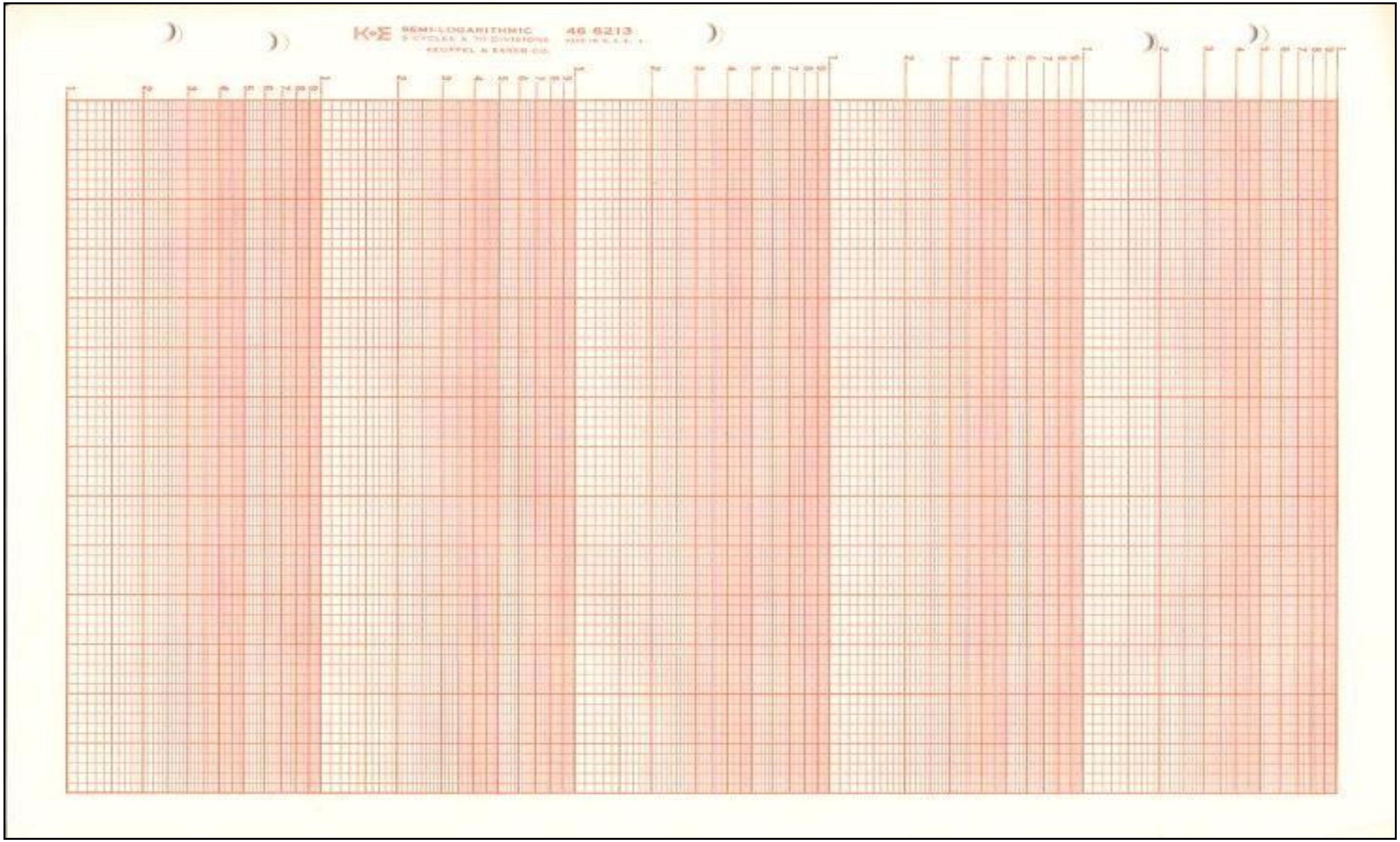
Date	Clock time	Elapsed time, min	Temp °C	Observed Hydrometer reading R_a	Hyd. Corr. Only for meniscus R_L	Effective depth L cm	Unit wt. of water γ_w	Viscosity of water η poise	Grain diameter D mm	Temperature correction C_t	Corrected Hyd. Reading. R_c	% Finer (% of portion finer than #200 sieve only) N	% Finer (combined) N_c
1	2	3	4	5	6	7	8	9	10	11	12	13	14

[Correction factor, a = $\frac{1.65G_S}{2.65(G_S-1)}$] [From Stokes's law, D= $\sqrt{\frac{30n}{(G_S-1)p_w}}\sqrt{\frac{L}{t}}$=K$\sqrt{\frac{L}{t}}$, here, n= Viscosity]

Course Teacher :
Designation :

Signature

Particle size distribution curve



Combined Analysis of Expt. 3 and Expt. 4

For mixture of coarse-grained soil and fine-grained soil the following combined analysis was done:

(1) Oven dry the collected disturbed sample and then break up all lumps with the grinding machine or mechanically by hammer and fingers.

(2) Run a sieve analysis as above. Then wash the soil retained on the No. 200 sieve.

(3) Weigh out to 0.01g about 50g of the dry soil retained in the pan from the sieve analysis.

(4) Run a hydrometer test on the dry soil following above procedure.

(5) Compute the particle size and percent finer for the two parts of the combined analysis as shown in sieve analysis and hydrometer analysis. The weight of dry soil, W_s to be used in computing the sieve analysis should be the total sample.

(6) The corrected percentage, N′ is found as follows:

$N' = N.\frac{W_1}{W_s}$ = N. % finer than No. 200 sieve

where N = percentage finer that was computed in Expt. No. 4.

W_1 = weight of dry soil passing No. 200 sieve.

W_s = total weight of dry soil used for sieve analysis computation.

Experiment No. 5

ATTERBERG LIMITS TESTS

Introduction

As we know, fine grained soils such as clay and plastic silts have a relatively large specific surface and therefore, surface forces dominate in their characteristics. A fine-grained soil can exist in any of several states: which state depends on the amount of water in the soil system. When water is added to such dry soil, each particle is covered with a film of adsorbed water. If the addition of water is continued, the thickness of the water film on a particle increases. Increasing the thickness of the water films permits the particles to slide past one another more easily. The behavior of the soil, therefore, is related to the amount of water in the system.

In the early 1900's Swedish soil scientist, A. Atterberg, identified different states in cohesive soils depending on the amount of water in the system. He was working in the ceramics industry and at that time they had several simple tests to describe the plasticity of clay, which was important, both in molding clay into bricks, for example, and to avoid shrinkage and cracking when fired. After many experiments, Atterberg came to the realization that at least two parameters were required to define plasticity of clays – the upper and lower limits of plasticity. He was able to define several limits of consistency or behavior and he developed simple laboratory tests to define these limits. They are:

- Upper limit of viscous flow.
- Liquid limit – lower limit of viscous flow.
- Sticky limit – clay loses its adhesion to a metal blade.
- Cohesion limit – grains cease to cohere to each other.
- Plastic limit – lower limit of the plastic state.
- Shrinkage limits – lower limit of volume change.

He also defined the plasticity index, which is the range of water content where the soil is plastic, and he was the first to suggest that it could be used for soil classification. Later on, in the late 1920 Terzaghi and A. Casagrande (1932b), working for the U.S. Bureau of Public Roads, standardized the Atterberg limits so that they could be readily used for soils classification purposes. In present geotechnical engineering practice we usually use the **Liquid Limit** (LL or w_L), the **Plastic Limit** (PL or w_p), and sometimes the **Shrinkage Limit** (SL or w_s). The sticky and the cohesion limits are more useful in ceramics and agriculture.

Since the Atterberg limits are water contents where the soil behavior changes from one phase to another phase, we can show these limits on a water content continuum. Also shown are the types of soil behavior for given ranges of water contents. As the water content increases, the state of the soil changes from a brittle solid to a plastic solid and then to a viscous liquid.

Atterberg's original consistency limit tests were rather arbitrary and not easily reproducible, especially by inexperienced operators. As mentioned, Casagranade (1932b, 1958) worked to standardize the tests, and he developed the liquid limit device (Fig. 3.2) so that the test became more operator-independent. He defined the LL as that water content at which a standard-groove cut in the remolded soil sample by a grooving tool (Figs. 3.2 a, b) will close over a distance of 13 mm (1/2 in) at 25 blows of the LL cup falling 10 mm on a hard rubber or micarta plastic base (Fig. 3.2 c). In practice, it is difficult to mix the soil so that the groove closure occurs at exactly 25 blows, but Casagrande found that if you plot the water contents of tests where you get closure at other blow counts versus the logarithm of the number of blows, you get a straight line called the flow curve. Where the flow curve crosses 25 blows, that water content is defined as the liquid limit.

The plastic limit test is somewhat more arbitrary, and it requires some practice to get consistent and reproducible results. The PL is defined as the water content at which a thread of soil just crumbles when it is carefully rolled out to a diameter of 3 mm. It should break up into segments about 3 to 10 mm long. If the threads can be rolled to a smaller diameter, then the soil is too wet (above the PL); if it crumbles before you reach 3mm in diameter, then you are past the PL.

The limits are determined on the portion of soil finer than #40 sieve.

Shrinkage Limit (SL) is defined as the moisture content at which no further volume change occurs with further reduction in moisture content. (SL represents the amount of water required to fully saturate the soil (100% saturation)). It is needed in producing bricks and ceramics.

Scope of the test
This test is performed to determine the plastic and liquid limits of a fine-grained soil.

Standard Reference
ASTM D 4318-17 - Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.

Significance
The Swedish soil scientist Albert Atterberg originally defined seven –limits of consistency to classify fine-grained soils, but in current engineering practice only two of the limits, the liquid and plastic limits, are commonly used. (A third limit, called the shrinkage limit, is used occasionally.) The Atterberg limits are based on the moisture content of the soil. The plastic limit is the moisture content that defines where the soil changes from a semi-solid to a plastic (flexible) state. The liquid limit is the moisture content that defines where the soil changes from a plastic to a viscous fluid state. The shrinkage limit is the moisture content that defines where the soil volume will not reduce further if the moisture content is reduced. A wide variety of soil engineering properties have been correlated to the liquid and plastic limits, and these Atterberg limits are also used to classify a fine-grained soil according to the Unified Soil Classification system or AASHTO system.

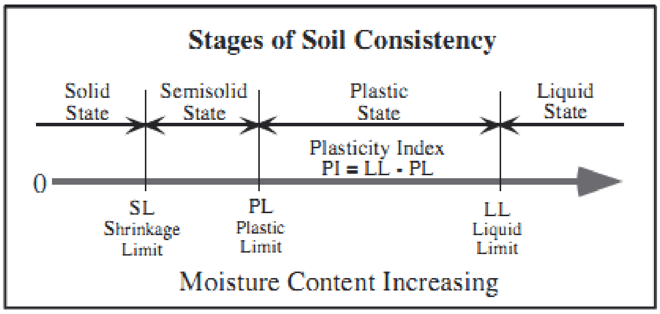


Figure 5.1: Atterberg Limits (Principles of Geotechnical Engineering, B. M. Das)

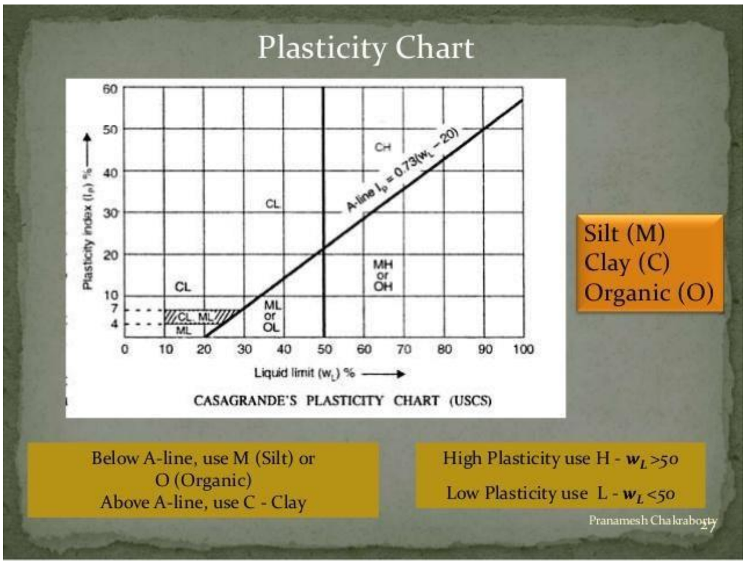


Figure 5.2: Modified Plasticity chart (Principles of Geotechnical Engineering, B. M. Das)

Equipment

- Liquid limit device
- Porcelain (evaporating) dish
- Flat grooving tool with gage,
- Moisture cans
- Balance
- Glass plate
- Spatula
- Wash bottle filled with distilled water
- Drying oven set at 105°C

Figure 5.3: Atterberg Limit Test Apparatus

Test Procedure

Liquid Limit

(1) Take roughly 3/4 of the soil and place it into the porcelain dish. Assume that the soil was previously passed though a No. 40 sieve, air-dried, and then pulverized. Thoroughly mix the soil with a small amount of distilled water until it appears as a smooth uniform paste. Cover the dish with cellophane to prevent moisture from escaping.

(2) Weigh four of the empty moisture cans with their lids, and record the respective weights and can numbers on the data sheet.

(3) Adjust the liquid limit apparatus by checking the height of drop of the cup. The point on the cup that comes in contact with the base should rise to a height of 10 mm. The block on the end of the grooving tool is10 mm high and should be used as a gage. Practice using the cup and determine the correct rate to rotate the crank so that the cup drops approximately two times per second.

(4) Place a portion of the previously mixed soil into the cup of the liquid limit apparatus at the point where the cup rests on the base. Squeeze the soil down to eliminate air pockets and spread it into the cup to a depth of about 10 mm at its deepest point. The soil pat should form an approximately horizontal surface.

(5) Use the grooving tool carefully cut a clean straight groove down the center of the cup. The tool should remain perpendicular to the surface of the cup as groove is being made. Use extreme care to prevent sliding the soil relative to the surface of the cup.

(6) Make sure that the base of the apparatus below the cup and the underside of the cup is clean of soil. Turn the crank of the apparatus at a rate of approximately two drops per second and count the number of drops, N, it takes to make the two halves of the soil pat come into contact at the bottom of the groove along a distance of **13 mm** (1/2 in.). If the number of drops exceeds 50, then go directly to step eight and do not record the number of drops, otherwise, record the number of drops on the data sheet.

(7) Take a sample, using the spatula, from edge to edge of the soil pat. The sample should include the soil on both sides of where the groove came into contact. Place the soil into a moisture can cover it. Immediately weigh the moisture can containing the soil, record it's mass, remove the lid, and place the can into the oven. Leave the moisture can in the oven for at least 16 hours. Place the soil remaining in the cup into the porcelain dish. Clean and dry the cup on the apparatus and the grooving tool.

(8) Remix the entire soil specimen in the porcelain dish. Add a small amount of distilled water to increase the water content so that the number of drops required closing the groove decrease.

(9) Repeat steps six, seven, and eight for at least two additional trials producing successively lower numbers of drops to close the groove. One of the trials shall be for a closure requiring 25 to 35 drops, one for closure between 20 and 30 drops, and one trial for a closure requiring 15 to 25 drops. Determine the water content from each trial by using the same method used in the first laboratory. Remember to use the same balance for all weighing.

Plastic Limit

(1) Weigh the remaining empty moisture cans with their lids, and record the respective weights and can numbers on the data sheet.

(2) Take the remaining 1/4 of the original soil sample and add distilled water until the soil is at a consistency where it can be rolled without sticking to the hands.

(3) Form the soil into an ellipsoidal mass. Roll the mass between the palm or the fingers and the glass plate. Use sufficient pressure to roll the mass into a thread of uniform diameter by using about 90 strokes per minute. (A stroke is one complete motion of the hand forward and back to the starting position.) The thread shall be deformed so that its diameter reaches 3.2 mm (1/8 in.), taking no more than two minutes.

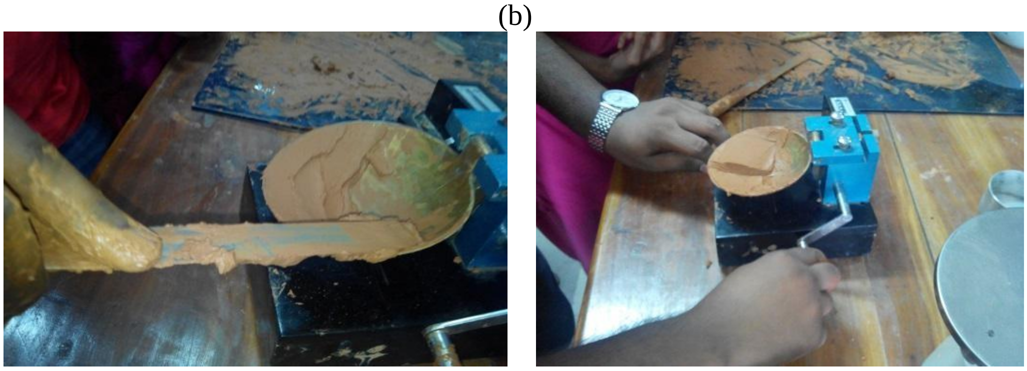
(4) When the diameter of the thread reaches the correct diameter, break the thread into several pieces. Knead and reform the pieces into ellipsoidal masses and re-roll them. Continue this alternate rolling, gathering together, kneading and re-rolling until the thread crumbles under the pressure required for rolling and can no longer be rolled into a 3.2 mm diameter thread.

(5) Gather the portions of the crumbled thread together and place the soil into moisture can, and then cover it. If the can does not contain at least 6 grams of soil, add soil to the can from the next trial. Immediately weigh the moisture can containing the soil, record it's mass, remove the lid, and place the can into the oven. Leave the moisture can in the oven for at least 16 hours.

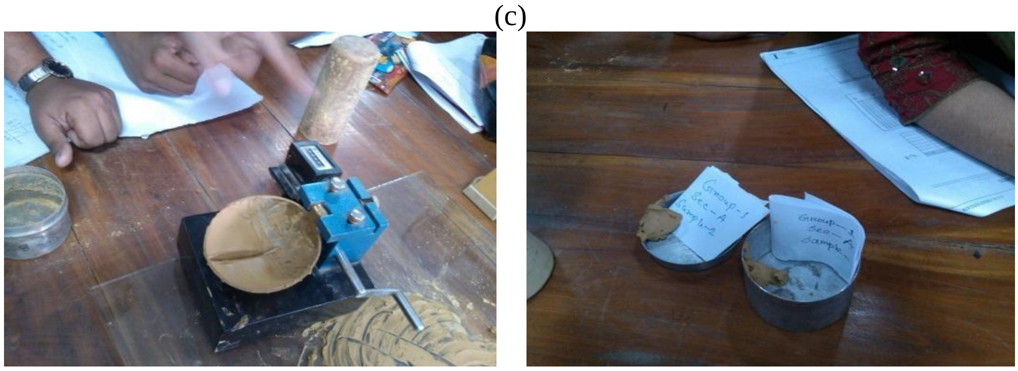
(6) Repeat steps three, four, and five at least two more times. Determine the water content from each trial by using the same method used in the first laboratory. Remember to use the same balance for all weighing.

(a)

(b)



(c)



(d)

Figure 5.4: (a) Grind the soil sample into pieces (b) Mix considerable amount of water with the soil and make a paste (c) Using the spatula place soil sample to the liquid limit device, cut the soil sample using grooving tools and give anti clockwise blow. (d) After come in contact the soil take samples, weight and dry them.

Figure 5.5: (a) Ellipsoidal Soil Mass (b) Soil Rolling (c) Soil Gets Crumbled at 1/8 inch (B.Sc. Eng. Thesis, Zakia Tasnim)

DATA SHEETS

Experiment Name : **Atterberg Limits Tests**
Experiment Date :

Student's Name :
Student's ID :
Year/ Semester :
Section/ Group :

Visual Classification :

Liquid Limit Determination :

Sample No.	**1**	**2**	**3**	**4**
Can no.				
Can weight				
Can + wet soil				
Can + dry soil				
Weight of water				
Water content w%				
No. of drop				

Liquid Limit for 25 blows from graph:

Plastic Limit Determination:

Sample No.	**1**	**2**	**3**	**4**
Can no.				
Can weight				
Can + wet soil				
Can + dry soil				
Weight of water				
Water content w%				

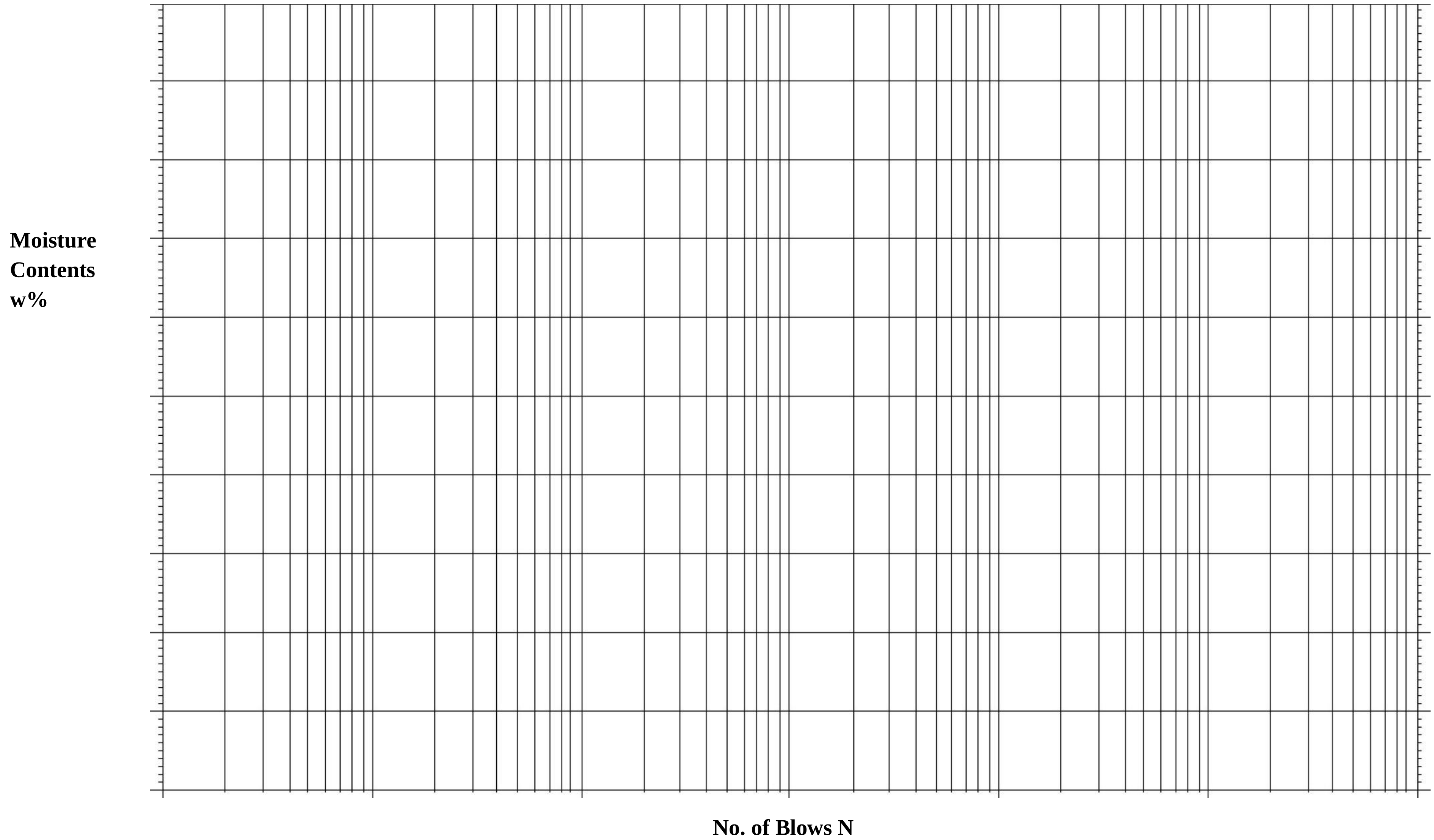
Plastic Limit (PL) = Average w %:

Final Results:
Liquid Limit :
Plastic Limit :
Plasticity Index :

Course Teacher :
Designation :

Signature

Liquid Limit Graph



Experiment No. 6 (a)
CONSTANT HEAD PERMIABILITY TEST
Experiment No. 6 (b)
FALLING HEAD PERMIABILITY TEST

Introduction

Permeability is a measure of the ease in which water can flow through a soil volume. It is one of the most important geotechnical parameters. However, it is probably the most difficult parameter to determine. In large part, it controls the strength and deformation behavior of soils. It directly affects the following:

- Quantity of water that will flow toward an excavation
- Design of cutoffs beneath dams on permeable foundations
- Design of the clay layer for a landfill liner.

Application

- Estimation of quantity of underground seepage water under various hydraulic conditions
- Quantification of water during pumping for underground construction
- Stability analysis of slopes, earth dams, and earth retaining structures
- Design of landfill liner.

Scope of the test

The purpose of this test is to determine the permeability (hydraulic conductivity) of a sandy soil by the constant head test method. There are two general types of permeability test methods that are routinely performed in the laboratory:

(1) The constant head test method, and

(2) The falling head test method.

The constant head test method is used for permeable soils ($k>10^{-4}$cm/s) and the falling head test is mainly used for less permeable soils ($k<10^{-4}$cm/s).

Standard Reference

ASTM D 2434-22 - Standard Test Method for Permeability of Granular Soils (Constant Head) (Note:The Falling Head Test Method is not standardized)

Significance

Permeability (or hydraulic conductivity) refers to the ease with which water can flow through a soil. This property is necessary for the calculation of seepage through earth dams or under sheet pile walls, the calculation of the seepage rate from waste storage facilities (landfills, ponds, etc.), and the calculation of the rate of settlement of clayey soil deposits.

Permeability depends on a number of factors-

- The size of the soil grains
- The properties of the pore fluid
- The void ratio of the soil
- The shape and arrangement of pores
- The degree of saturation

Equipment

- Permeameter
- Tamper
- Balance
- Scoop
- Cylinders
- Watch (or Stopwatch),
- Thermometer
- Filter paper.

Figure 6.1 Permeability Apparatus

Test Procedure:
(a) Constant Head Test:

(1) Measure the initial mass of the pan along with the dry soil (M1).

(2) Remove the cap and upper chamber of the permeameter by unscrewing the knurled cap nuts and lifting them off the tie rods. Measure the inside diameter of upper and lower chambers. Calculate the average inside diameter of the permeameter (D).

(3) Place one porous stone on the inner support ring in the base of the chamber then place a filter paper on top of the porous stone

(4) Mix the soil with a sufficient quantity of distilled water to prevent the segregation of particle sizes during placement into the permeameter. Enough water should be added so that the mixture may flow freely

(5) Using a scoop, pour the prepared soil into the lower chamber using a circular motion to fill it to a depth of 1.5 cm. A uniform layer should be formed.

(6) Use the tamping device to compact the layer of soil. Use approximately ten rams of the tamper per layer and provide uniform coverage of the soil surface. Repeat the compaction procedure until the soil is within 2 cm. of the top of the lower chamber section

(7) Replace the upper chamber section, and don't forget the rubber gasket that goes between the chamber sections. Be careful not to disturb the soil that has already been compacted. Continue the placement operation until the level of the soil is about 2 cm. below the rim of the upper chamber. Level the top surface of the soil and place a filter paper and then the upper porous stone on it

(8) Place the compression spring on the porous stone and replace the chamber cap and its sealing gasket. Secure the cap firmly with the cap nuts.

(9) Measure the sample length at four locations around the circumference of the permeameter and compute the average length. Record it as the sample length.

(10) Keep the pan with remaining soil in the drying oven.

(11) Adjust the level of the funnel to allow the constant water level in it to remain a few inches above the top of the soil.

(12) Connect the flexible tube from the tail of the funnel to the bottom outlet of the permeameter and keep the valves on the top of the permeameter open.

(13) Place tubing from the top outlet to the sink to collect any water that may come out.

(14) Open the bottom valve and allow the water to flow into the permeameter.

(15) As soon as the water begins to flow out of the top control (desiring) valve, close the control valve, letting water flow out of the outlet for some time.

(16) Close the bottom outlet valve and disconnect the tubing at the bottom. Connect the funnel tubing to the top side port.

(17) Open the bottom outlet valve and raise the funnel to a convenient height to get a reasonable steady flow of water.

(18) Allow adequate time for the flow pattern to stabilize.

(19) Measure the time it takes to fill a volume of 750 - 1000 mL using the graduated cylinder, and then measure the temperature of the water. Repeat this process three times and compute the average time, average volume, and average temperature. Record the values as t, Q, and T, respectively.

(20) Measure the vertical distance between the funnel head level and the chamber outflow level, and record the distance as h.

(21) Repeat step 17 and 18 with different vertical distances.

(22) Remove the pan from the drying oven and measure the final mass of the pan along with the dry soil (M2).

(a) Falling Head Test:

Calculation

1. Calculate the permeability, using the following equation:

$$K_T = \frac{QL}{Ath}$$

Where:
K_T = coefficient of permeability at temperature T, cm/sec.
L = length of specimen in centimeters
t = time for discharge in seconds
Q = volume of discharge in cm^3 (assume 1 mL = 1 cm^3)
A = cross-sectional area of permeameter ($= \frac{\pi D^2}{4}$, D= insidediameter of the permeameter)
h = hydraulic head difference across length L, in cm of water; or it is equal to the vertical distance between the constant funnel head level and the chamber overflow level.

2. The viscosity of the water changes with temperature. As temperatureincreases viscosity decreases and the permeability increases. The coefficient of permeability is standardized at 20°C, and the permeability at any temperature T is related to K20 by the following

$$K_{20} = K_T \frac{\eta_T}{\eta_{20}}$$

Where:

η_T and η_{20} are the viscosities at the temperature T of the test and at 20°C, respectively. From Table below obtain the viscosities and compute K20.

Temperature $^\circ C$	Density (g/cm^3)	Viscosity (Poise*)
4	1.00000	0.01567
16	0.99897	0.01111
17	0.99880	0.01083
18	0.99862	0.01056
19	0.99844	0.01030
20	0.99823	0.01005
21	0.99802	0.00981
22	0.99780	0.00958
23	0.99757	0.00936
24	0.99733	0.00914
25	0.99708	0.00894
26	0.99682	0.00874
27	0.99655	0.00855
28	0.99627	0.00836
29	0. 99598	0.00818
30	0. 99568	0.00801

$$\text{*Poise} = \frac{\text{dyne}\cdot\text{s}}{\text{cm}^2} = \frac{\text{g}}{\text{cm}\cdot\text{s}}$$

3. Compute the dry density (ρ_d) of soil

$$\rho_d = \frac{M}{V} = \frac{M1 - M2}{LA}$$

DATA SHEET

Experiment Name : **Content Head Permeability Test**
Experiment Date :

Student's Name :
Student's ID :
Year/ Semester :
Section/ Group :

Visual Classification :
Initial Dry Mass of Soil + Pan (M_1) : g
Length of Soil Specimen, L : cm
Diameter of the Soil Specimen (Permeameter), D : cm
Final Dry Mass of Soil + Pan (M_2) : g
Dry Mass of Soil Specimen (M) : g
Volume of Soil Specimen (V) : cm^3
Dry Density of Soil (ρ_d) : g/cm^3

Trial No.	Constant Head, h (cm)	Elapsed Time, t (sec)	Outflow Volume, Q (cm^3)	Water Temp., T (^0C)	K_T	K_{20}
1						
2						
3						
4						

Average K_{20} : cm/sec

Result:
Permeability of the soil is:

Course Teacher :
Designation :

Signature

Experiment No. 7

RELATIVE DENSITY TEST

Introduction

Relative Density principles apply to compaction of relatively clean, coarse-grained soils. Relatively clean, coarse-grained soil usually means less than 15 % fines. Compaction Tests are not commonly performed on soils with 15 % or fewer fines. Small percentage of fines means soilscannot easily hold water to examine range of water and effect on dry density.
Relative density can be found by the equation given below:

$$Dr(\%) = \frac{e_{max} - e_{measured}}{e_{max} - e_{min}} \times 100$$

According to the relative density values, soil can be classified using the chart given below:

Relative Density	State of compaction
0-15	Very loose
15-50	Loose
50-70	Medium dense
70-85	Dense
85-100	Very dense

Scope of the test

This test is performed to determine the relative density of cohesionless, free-draining soils using a vibrating table. The relative density of a soil is the ratio, expressed as a percentage, of the difference between the maximum index void ratio and the field void ratio of a cohesionless, free-draining soil; to the difference between its maximum and minimum index void ratios.

Standard References

ASTM D4254-16– Standard Test Methods for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density.
ASTM D4253-16– Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table.

Significance

Relative density and percent compaction are commonly used for evaluating the state of compactness of a given soil mass. The engineering properties, such as shear strength, compressibility, and permeability, of a given soil depend on the level of compaction.

Equipment

- Vibrating Table,
- Mold Assembly consisting of standard mold,
- guide sleeves,
- surcharge base-plate,
- surcharge weights,
- surcharge base-plate handle,
- dial-indicator gage,
- Balance,
- Scoop,
- Straightedge.

Test Procedure:

Minimum Density Index:

(1) Fill the mold with the soil (approximately 0.5 inch to 1 inch above the top of the mold) as loosely as possible by pouring the soil using a scoop or pouring device (funnel). Spiraling motion should be just sufficient to minimize particle segregation.

(2) Trim off the excess soil level with the top by carefully trimming the soil surface with a straightedge.

(3) After filling the mold, excess soil is carefully screed off. Knowing the weight of soil in the mold (M_{s1}), the dry density is easily computed.

Maximum Density Index:

(1) Again fill the mold with soil (do not use the same soil used in step 1) and level the surface of the soil by using a scoop or pouring device (funnel) in order to minimize the soil segregation. The sides of the mold may be struck a few times using a metal bar or rubber hammer to settle the soil so that the surcharge base-plate can be easily placed into position and there is no surge of air from the mold when vibration is initiated.

(2) Place the surcharge base plate on the surface of the soil and twist it slightly several times so that it is placed firmly and uniformly in contact with the surface of the soil. Remove the surcharge base-plate handle.

(3) Attach the mold to the vibrating table.

(4) Vibrate the mold assembly and soil specimen for 10 min.

(5) Determine and record the mass of the mold and soil (M_{S2}).

(6) Determine the decreased height, H′ of the soil to compute the decreased volume of the soil.

(7) Empty the mold and determine the weight of the mold.

(8) Determine and record the dimensions of the mold (i.e., diameter, D and height, H) in order to calculate the calibrated volume of the mold, V_c.

Analysis

(1) Calculate the minimum density index (ρ_{dmin}) as follows:

$$\rho_{d,min} = \frac{M_{s1}}{V}$$

where

Ms1= mass of tested-dry soil

= Mass of mold with soil placed loose – mass of mold

V = Calibrated volume of the mold

(2) Calculate the maximum density index (ρ_{dmax}) as follows:

$$\rho_{dmax} = \frac{M_{S2}}{V}$$

Where

M_{s2} = mass of tested-dry soil

= Mass of mold with soil after vibration – Mass of mold

V = Volume of tested-dry soil

(3) Calculate the maximum and the minimum-index void ratios as follows (use Gs value determined from Experiment 4; ρw=1 g/cm3):

$$e_{min} = \frac{\rho_w G_s}{\rho_{dmax}} - 1 \qquad e_{max} = \frac{\rho_w G_s}{\rho_{dmin}} - 1 \qquad e = \frac{\rho_s}{\rho_d} - 1$$

(4) Calculate the relative densıty as follows:

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

Calculate the void ratio of the natural state of the soil based on ρ_d and $\rho_s = G_S * \rho_w$ (G_s determined from Experiment 2) as follows:

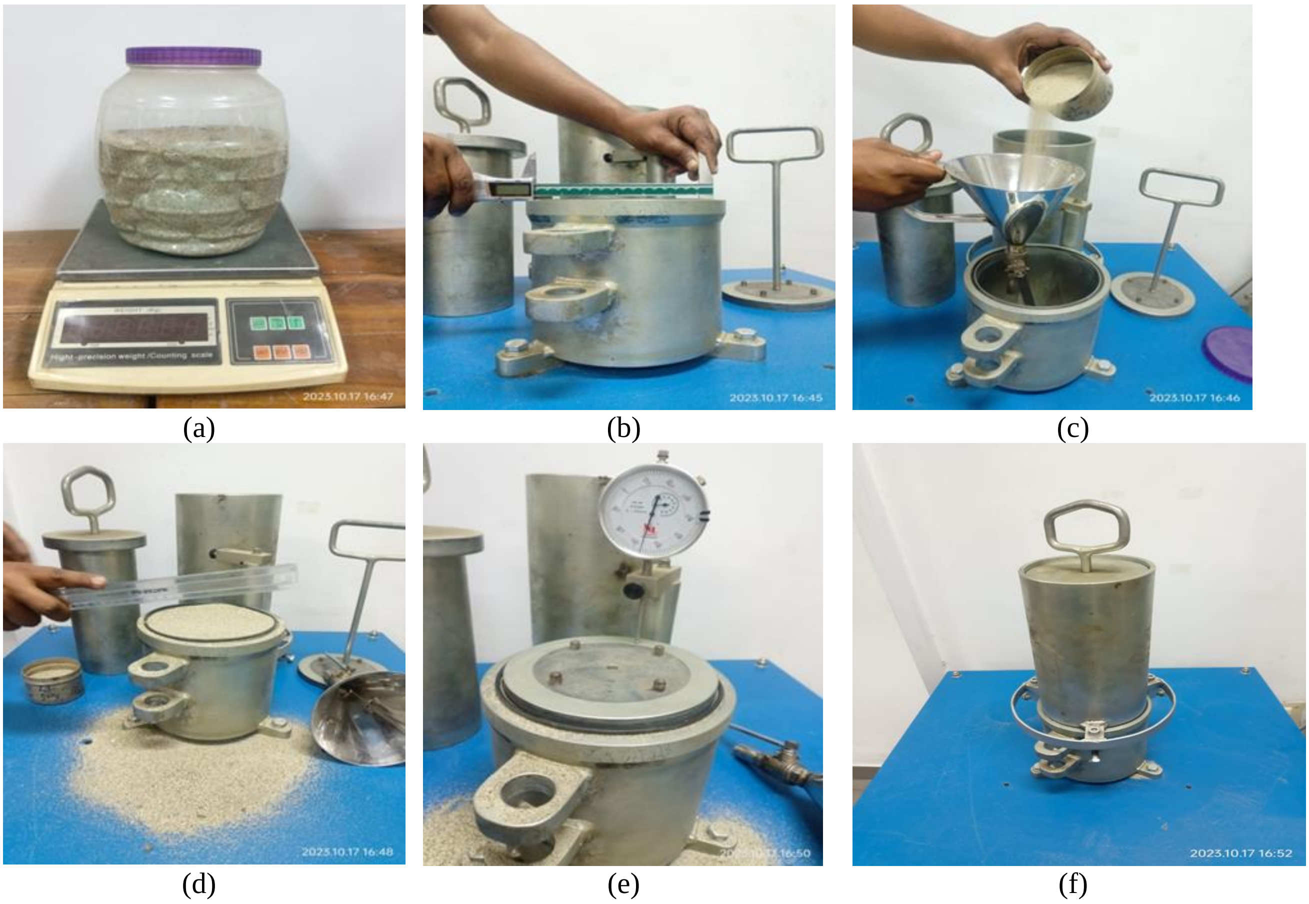


Figure 7.1: Relative density test, (a) Measuring of soil before pouring, (b) Dimension of mold, (c) Pouring of soil at loosest state, (d) leveling of soil, (e) Initial reading and (f) Loaded sample before vibration

DATA SHEET

Experiment Name : **Relative Density Test**
Experiment Date :

Student's Name :
Student's ID :
Year/ Semester :
Section/ Group :

Visual Classification :

Mass of empty mold :
Diameter of empty mold :
Height of empty mold :
Mass of mold and soil (M_1) :
Mass of mold and soil (M_2) :

Calculations:

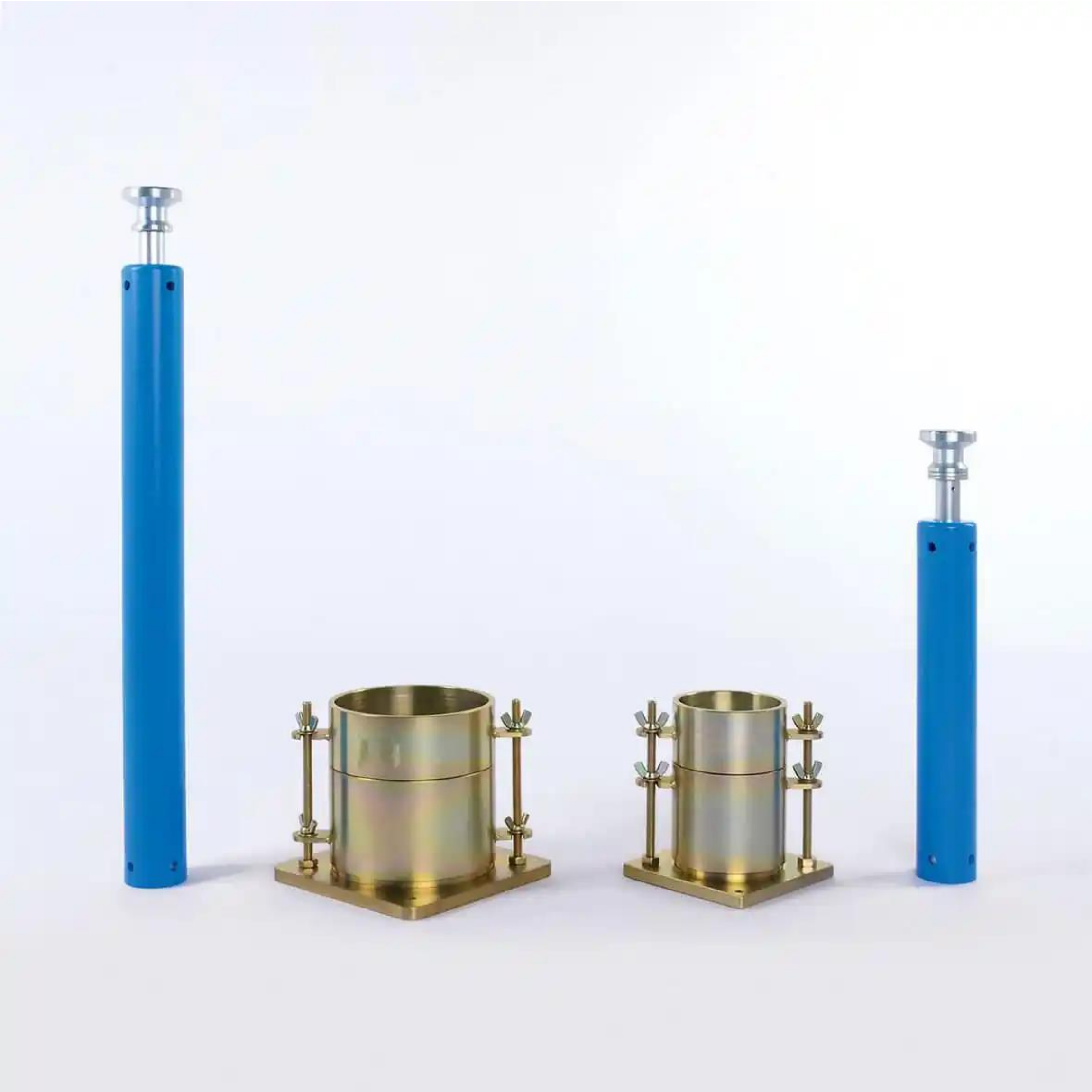
Result:
Relative Density of the soil is:

Course Teacher :
Designation :

Signature

Experiment No. 8

COMPACTION TEST



Introduction

This laboratory test is performed to determine the relationship between the moisture content and the dry density of a soil for a specified compactive effort. The compactive effort is the amount of mechanical energy that is applied to the soil mass. Several different methods are used to compact soil in the field, and some examples include tamping, kneading, vibration, and static load compaction. This laboratory will employ the tamping or impact compaction method using the type of equipment and methodology developed by R. R. Proctor in 1933, therefore, the test is also known as the Proctor test.

Two types of compaction tests are routinely performed: (1) The Standard Proctor Test, and (2) The Modified Proctor Test. Each of these tests can be performed in three different methods as outlined in the attached Table 1. In the Standard Proctor Test, the soil is compacted by a 5.5 lb hammer falling a distance of one foot into a soil filled mold. The mold is filled with three equal layers of soil, and each layer is subjected to 25 drops of the hammer. The Modified Proctor Test is identical to the Standard Proctor Test except it employs, a 10 lb hammer falling a distance of 18 inches, and uses five equal layers of soil instead of three. There are two types of compaction molds used for testing. The smaller type is 4 inches in diameter and has a volume of about 1/30 ft3(944 cm3), and the larger type is 6 inches in diameter and has a volume of about 1/13.333 ft3(2123 cm3). If the larger mold is used each soil layer must receive 56 blows instead of 25 (See Table).

Table 10.1 Alternative Proctor Test Methods

	Standard Proctor ASTM 698			Modified Proctor ASTM 1557		
	Method A	Method B	Method C	Method A	Method B	Method C
Material	≤20% Retained on No.4 Sieve	>20% Retained on No.4 ≤20% Retained on 3/8" Sieve	>20% Retained on No.3/8" <30% Retained on 3/4" Sieve	≤20% Retained on No.4 Sieve	>20% Retained on No.4 ≤20% Retained on 3/8" Sieve	>20% Retained on No.3/8" <30% Retained on 3/4" Sieve
For test sample, use soil passing	Sieve No.4	3/8" Sieve	¾" Sieve	Sieve No.4	3/8" Sieve	¾" Sieve
Mold	4" DIA	4" DIA	6" DIA	4" DIA	4" DIA	6" DIA
No. of Layers	3	3	3	5	5	5
No. of blows/layer	25	25	56	25	25	56

Note: Volume of 4″ diameter mold = 944 cm^3, Volume of 6″ diameter mold = 2123 cm^3(verify these values prior to testing)

Standard Reference

ASTM D698 - 12- Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbs/ft^3 (600 kN-m/m^3))

ASTM D1557 - 12 - Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbs/ft^3(2,700 kN-m/m^3))

Significance

Mechanical compaction is one of the most common and cost effective means of stabilizing soils. An extremely important task of geotechnical engineers is the performance and analysis of field control tests to assure that compacted fills are meeting the prescribed design specifications. Design specifications usually state the required density (as a percentage of the ―maximum‖ density measured in a standard laboratory test), and the water content. In general, most engineering properties, such as the strength, stiffness, resistance to shrinkage, and imperviousness of the soil, will improve by increasing the soil density.

The optimum water content is the water content that results in the greatest density for a specified compactive effort. Compacting at water contents higher than (wet of) the optimum water content results in a relatively dispersed soil structure (parallel particle orientations) that is weaker, more ductile, less pervious, softer, more susceptible to shrinking, and less susceptible to swelling than soil compacted dry of optimum to the same density. The soil compacted lower than (dry of) the optimum water content typically results in a flocculated soil structure (random particle orientations) that has the opposite characteristics of the soil compacted wet of the optimum water content to the same density.

Equipment

Molds, Manual rammer, Extruder, Balance, Drying oven, Mixing pan, Trowel, #4 sieve, Moisture cans, Graduated cylinder, Straight Edge.

Test Procedure

(1) Depending on the type of mold you are using obtain a sufficient quantity of air-dried soil in large mixing pan. For the 4-inch mold take approximately 10 lbs, and for the 6-inch mold take roughly 15 lbs. Pulverize the soil and run it through the # 4 sieve.

(2) Determine the weight of the soil sample as well as the weight of the compaction mold with its base (without the collar) by using the balance and record the weights.

(3) Compute the amount of initial water to add by the following method:

(a) Assume water content for the first test to be 8 percent.

(b) Compute water to add from the following equation:

$$\text{water to add (in ml)} = \frac{(\text{soil mass in grams})8}{100}$$

Where, ―water to add‖ and the ―soil mass‖ are in grams. Remember that a gram of water is equal to approximately one milliliter of water.

(4) Measure out the water, add it to the soil, and then mix it thoroughly into the soil using the trowel until the soil gets a uniform color.

(5) Assemble the compaction mold to the base, place some soil in the mold and compact the soil in the number of equal layers specified by the type of compaction method employed. The number of drops of the rammer per layer is also dependent upon the type of mold used (See Table). The drops should be applied at a uniform rate not exceeding around 1.5 seconds per drop, and the rammer should provide uniform coverage of the specimen surface. Try to avoid rebound of the rammer from the top of the guide sleeve.

(6) The soil should completely fill the cylinder and the last compacted layer must extend slightly above the collar joint. If the soil is below the collar joint at the completion of the drops, the test point must be repeated. (Note: For the last layer, watch carefully, and add more soil after about 10 drops if it appears that the soil will be compacted below the collar joint.)

(7) Carefully remove the collar and trim off the compacted soil so that it is completely even with the top of the mold using the trowel. Replace small bits of soil that may fall out during the trimming process.

(8) Weigh the compacted soil while it's in the mold and to the base, and record the mass. Determine the wet mass of the soil by subtracting the weight of the mold and base.

(9) Remove the soil from the mold using a mechanical extruder and take soil moisture content samples from the top and bottom of the specimen. Fill the moisture cans with soil and determine the water content.

(10) Place the soil specimen in the large tray and break up the soil until it appears visually as if it will pass through the # 4 sieve, add 2 percent more water based on the original sample mass, and re-mix as in step 4. Repeat steps 5 through 9 until, based on wet mass, a peak value is reached followed by two slightly lesser compacted soil masses.

Analysis

(1) Calculate the moisture content of each compacted soil specimen by using the average of the two water contents.

(2) Compute the wet density in grams per cm3 of the compacted soil sample by dividing the wet mass by the volume of the mold used.

(3) Compute the dry density using the wet density and the water content determined in step 1. Use the following formula:

$$\rho_d = \frac{\rho}{1+w}$$

where: w = moisture content in percent divided by 100, and ρ = wet density in grams per cm3 = (M/V).

(4) Plot the dry density values on the y-axis and the moisture contents on the x-axis. Draw a smooth curve connecting the plotted points.

(5) On the same graph draw a curve of complete saturation or **–zero air voids curve‖**. The values of dry density and corresponding moisture contents for plotting the curve can be computed from the following equation:

$$\rho_d = \frac{\rho_w}{\left(\frac{w}{100} + \frac{1}{Gs}\right)}$$

Where: ρ_d = dry density of soil grams per cm^3, Gs = specific gravity of the soil being tested, ρ_w = density of water in grams per cm^3 (approximately $1g/cm^3$), w_{sat} = moisture content in percent for complete saturation.

(6) Identify and report the optimum moisture content and the maximum dry density.

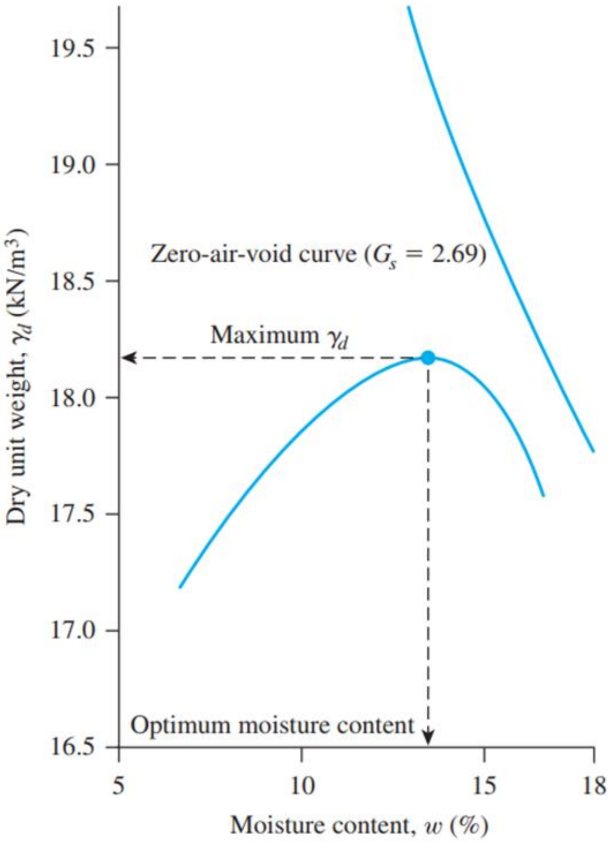


Figure 8.1 : Dry density vs moisture content (Das, 2010)

(a)

(b)

(c)

(d)

Figure 8.2 (a) Take the soil sample in a tray and break them into pieces and mix certain amount of water (b) Fill the 1/3 of the mold with soil and compact them using compaction hammer (c) remove the collar and the extra portion of soil (d) Weight the mold with soil and remove the soil.

DATA SHEET

Experiment Name : **Proctor Compaction Test**
Experiment Date :

Student's Name :
Student's ID :
Year/ Semester :
Section/ Group :

Visual Classification :

Test Method :
Diameter of mold :
Height of mold :
Volume of mold, V :
Massof mold, M :

Water Content Determination

Sample no.	1	2	3	4	5
Moisture can no.					
Mass of empty clean can					
Mass of can + wet soil					
Mass of can + dry soil					
Mass of soil solid					
Mass of pore water					
Water content w%					

Density Determination

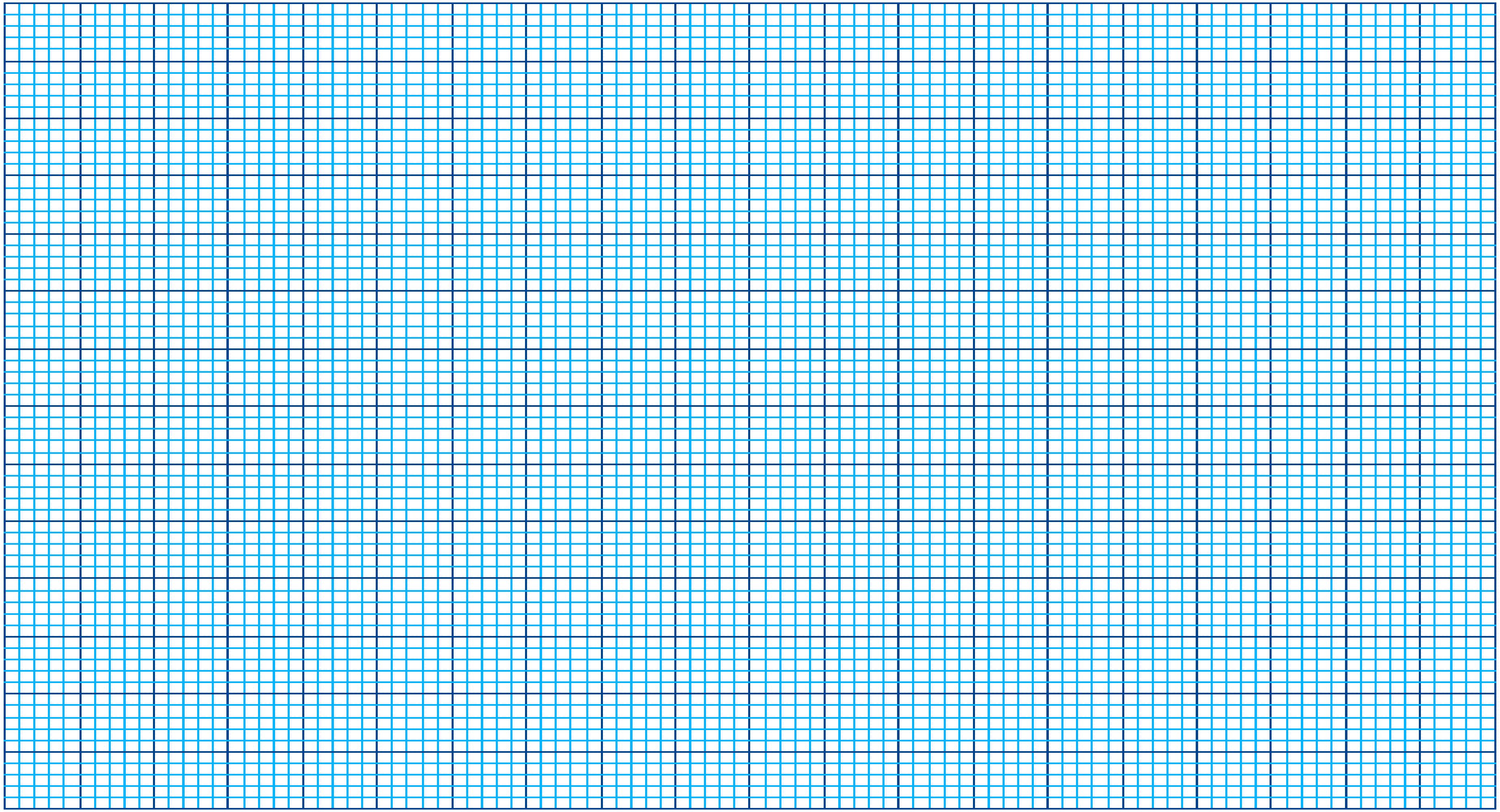
Compacted soil sample no.	1	2	3	4	5
Water content w%					
Mass of compacted soil and mold (gm)					
Mass of wet soil (gm)					
Wet density, P = (M/V)					
Dry density, $P_d = [P/(1+w)]$					
Dry density (S =100%)					

Result:
Optimum Moisture Content: %
Maximum Dry Density: g/cm3

Course Teacher :
Designation :

Signature

Fig. Dry Density vs Moisture Content Content



Experiment No. 9

DIRECT SHEAR TEST

Scope of the test
This test is performed to determine the consolidated-drained shear strength of a sandy to silty soil. The shear strength is one of the most important engineering properties of a soil, because it is required whenever a structure is dependent on the soil's shearing resistance. The shear strength is needed for engineering situations such as determining the stability of slopes or cuts, finding the bearing capacity for foundations, and calculating the pressure exerted by a soil on a retaining wall.

Standard Reference
ASTM D3080-04 - Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions

Significance
The direct shear test is one of the oldest strength tests for soils. In this laboratory, a direct shear device will be used to determine the shear strength of a cohesionless soil (i.e. angle of internal friction (ϕ). From the plot of the shear stress versus the horizontal displacement, the maximum shear stress is obtained for a specific vertical confining stress. After the experiment is run several times for various vertical-confining stresses, a plot of the maxi mum shear stresses versus the vertical (normal) confining stresses for each of the tests is produced. From the plot a straight-line approximation of the Mohr-Coulomb failure envelope curve can be drawn, f may be determined, and, for cohesionless soils (c = 0), the shear strength can be computed from the following equation:

$$\tau = \delta \tan\phi$$

Equipment
Direct shear device, Load and deformation dial gauges, Balance.

Figure 9.1 Direct Shear Test device of the laboratory

Test Procedure
(1) Weigh the initial mass of soil in the pan.

(2) Measure the diameter and height of the shear box. Compute 15% of the diameter in millimeters.

(3) Carefully assemble the shear box and place it in the direct shear device. Then place a porous stone and a filter paper in the shear box.

(4) Place the sand into the shear box and level off the top. Place a filter paper, a porous stone, and a top plate (with ball) on top of the sand.

(5) Remove the large alignment screws from the shear box! Open the gap between the shear box halves to approximately 0.025 in. using the gap screws, and then back out the gap screws.

(6) Weigh the pan of soil again and compute the mass of soil used.

(7) Complete the assembly of the direct shear device and initialize the three gauges (Horizontal displacement gage, vertical displacement gage and shear load gage) to zero.

(8) Set the vertical load (or pressure) to a predetermined value, and then close bleeder valve and apply the load to the soil specimen by raising the toggles witch.

(9) Start the motor with selected speed so that the rate of shearing is at as elected constant rate, and take the horizontal displacement gauge, vertical displacement gage and shear load gage readings. Record the readings on the data sheet. (Note: Record the vertical displacement gage readings, if needed).

(10) Continue taking readings until the horizontal shear load peaks and then falls, or the horizontal displacement reaches 15% of the diameter.

Analysis

(1) Calculate the density of the soil sample from the mass of soil and volume of the shear box.

(2) Convert the dial readings to the appropriate length and load units and enter the values on the data sheet in the correct locations. Compute the sample area A, and the vertical (Normal) stress S_n.

$$S_n = (F/A)$$

Where: F = normal vertical force, and S_n = normal vertical stress

(3) Calculate shear stress (τ) using

$$\tau = (F/A)$$

Where F = shear force measured with shear load gage

(4) Plot the shear stress (τ) versus shear displacement.

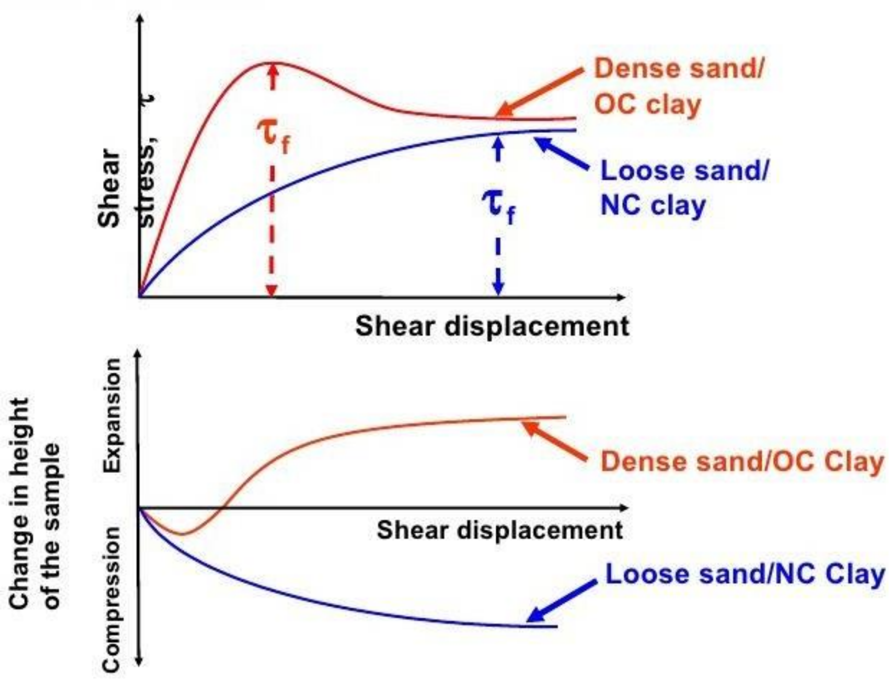


Figure 9.2 Stress vs displacement pattern (https://www.slideshare.net/xakikazmi/strength-nalin)

(5) Calculate the maximum shear stress for each test.

(6) Plot the value of the maximum shear stress versus the corresponding vertical stress for each test, and determine the angle of internal friction (ϕ) from the slope of the approximated Mohr-Coulomb failure envelope.

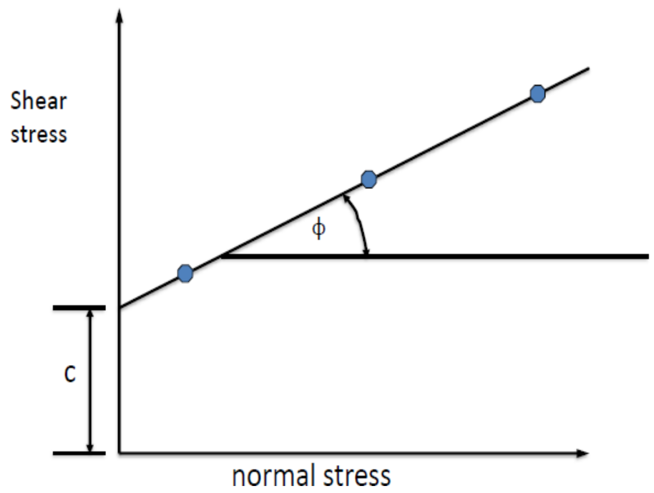
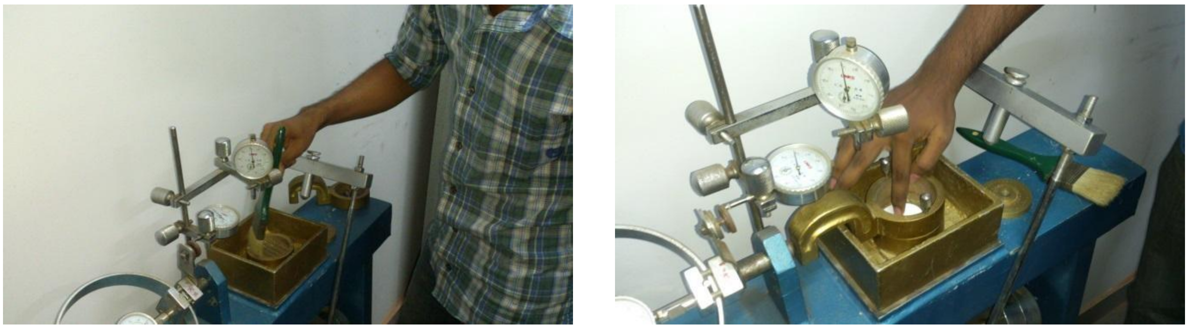


Figure 9.3 Stress-strain relationship of sand



(a)

(b)

(c)

(d)

Figure 9.4 (a) Clean the shear box and re-place it (b) Fill the shear box with soil and slightly compact it (c) Close the shear box in such way that the vertical load will act just perpendicular and uniformly (d) Put the vertical load and switch on the shear load and take reading carefully.

DATA SHEET

Experiment Name : **Direct Shear Test**
Experiment Date :

Student's Name :
Student's ID :
Year/ Semester :
Section/ Group :

Visual Classification :

Shear Box inside Diameter :
Area (A) :
Shear Box Height :
Soil Volume :

Normal stress vs maximum shear stress curve:

Normal Stress **Σ**	Maximum shear stress τ

Result
Angle of internal friction:

Course Teacher :
Designation :

Signature

Data Table for Sample-1

Normal force : lb
Normal Stress : psi
Shear stress :

Elapsed time (min)	Shear displacement dial reading	Shear displacement (in)	Load dial reading	Shear force (lb)	Shear stress (psi)
0.25					
0.5					
1					
1.25					
1.5					
2					
2.25					
2.5					
3					
3.25					
3.5					
4					
4.25					
4.5					
5					
5.25					
5.5					
6					
6.25					
6.5					
7					
7.25					
7.5					
8					
8.25					
8.5					
9					
9.25					
9.5					
10					
10.25					
10.5					
11					
11.25					
11.5					
12					
12.25					
12.5					
13					
13.25					
13.5					
14					
14.25					
14.5					
15					

Data Table for Sample-2

Normal force : lb
Normal Stress : psi
Shear stress :

Elapsed time (min)	Shear displacement dial reading	Shear displacement (in)	Load dial reading	Shear force (lb)	Shear stress (psi)
0.25					
0.5					
1					
1.25					
1.5					
2					
2.25					
2.5					
3					
3.25					
3.5					
4					
4.25					
4.5					
5					
5.25					
5.5					
6					
6.25					
6.5					
7					
7.25					
7.5					
8					
8.25					
8.5					
9					
9.25					
9.5					
10					
10.25					
10.5					
11					
11.25					
11.5					
12					
12.25					
12.5					
13					
13.25					
13.5					
14					
14.25					
14.5					
15					

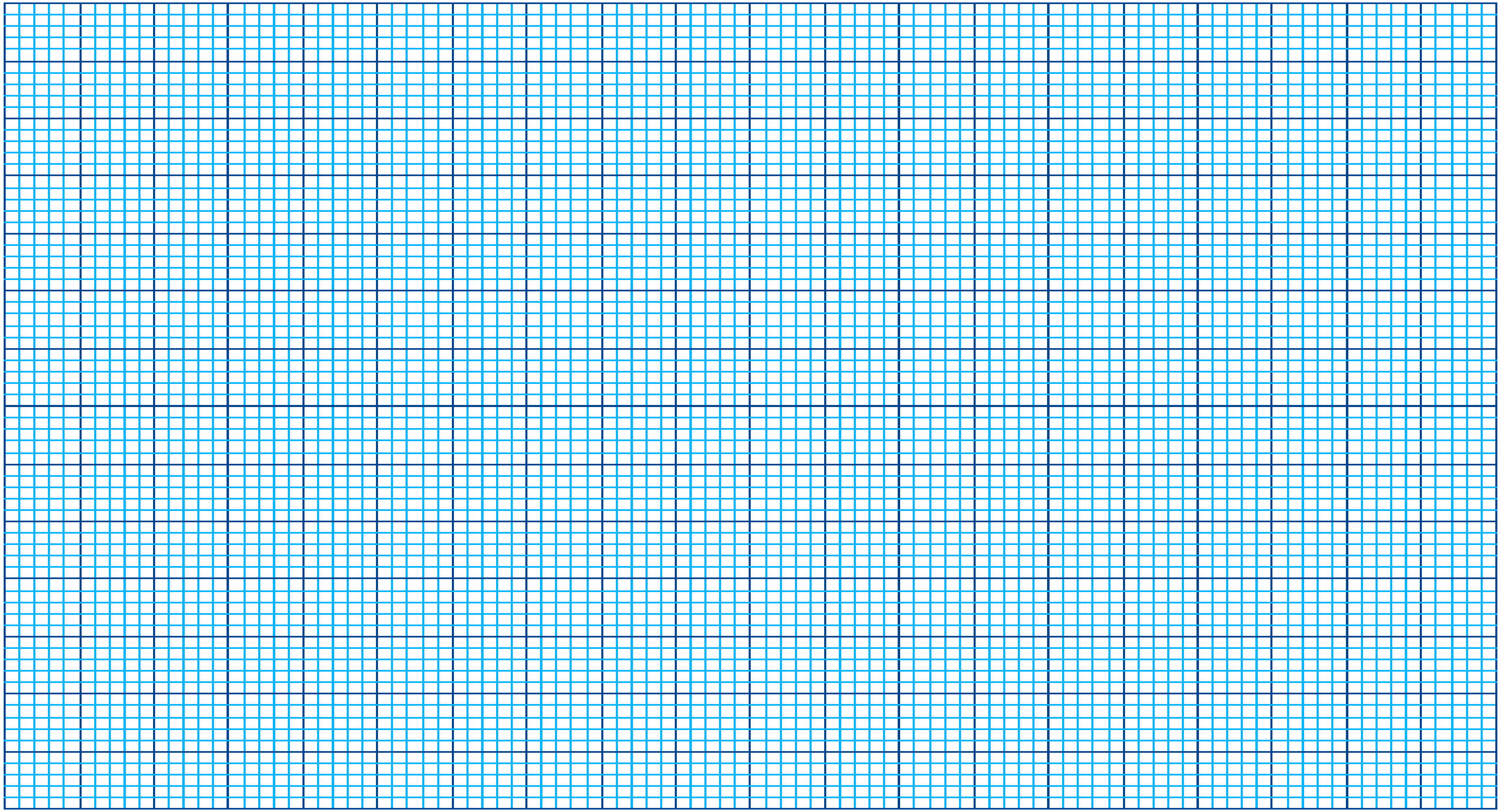
Data Table for Sample-3

Normal force : lb
Normal Stress : psi
Shear stress :

Elapsed time (min)	Shear displacement dial reading	Shear displacement (in)	Load dial reading	Shear force (lb)	Shear stress (psi)
0.25					
0.5					
1					
1.25					
1.5					
2					
2.25					
2.5					
3					
3.25					
3.5					
4					
4.25					
4.5					
5					
5.25					
5.5					
6					
6.25					
6.5					
7					
7.25					
7.5					
8					
8.25					
8.5					
9					
9.25					
9.5					
10					
10.25					
10.5					
11					
11.25					
11.5					
12					
12.25					
12.5					
13					
13.25					
13.5					
14					
14.25					
14.5					
15					

Shear Stress vs Shear Displacement Graph

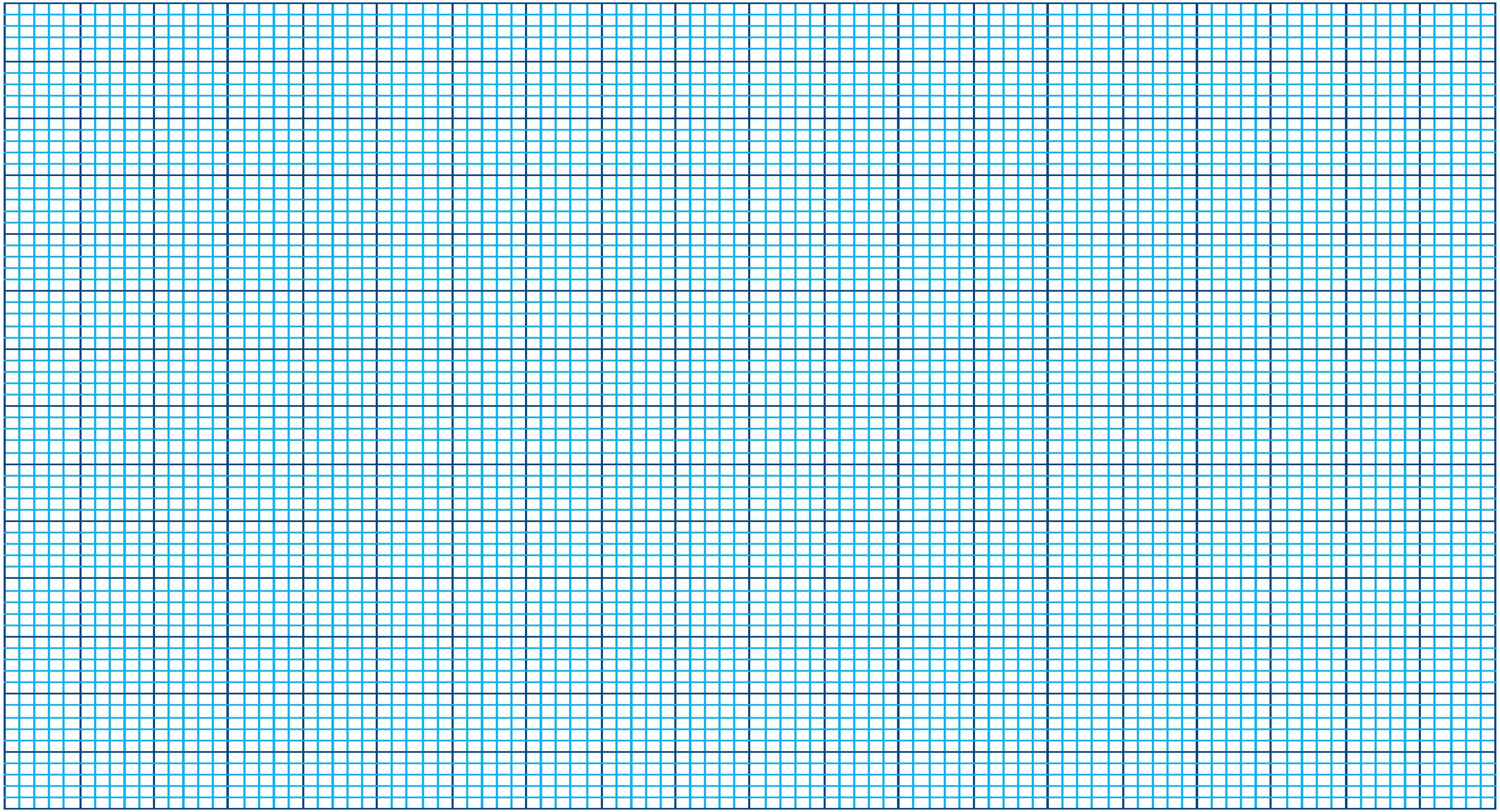
Horizontal Shear Stress (psi)



Shear Displacement (in)

Maximum Shear Stress vs Normal Stress Graph

Shear Stress (psi)



Normal Stress (psi)

Experiment No. 10
UNCONFINED COMPRESSION TEST

Scope of the test

The primary purpose of this test is to determine the unconfined compressive strength, which is then used to calculate the unconsolidated undrained shear strength of the clay under unconfined conditions. According to the ASTM standard, the unconfined compressive strength (q_u) is defined as the compressive stress at which an unconfined cylindrical specimen of soil will fail in a simple compression test. In addition, in this test method, the unconfined compressive strength is taken as the maximum load attained per unit area, or the load per unit area at 15% axial strain, whichever occurs first during the performance of a test.

Standard Reference

ASTM D2166-06 - Standard Test Method for Unconfined Compressive Strength of Cohesive Soil

Significance

For soils, the undrained shear strength (s_u) is necessary for the determination of the bearing capacity of foundations, dams, etc. The undrained shear strength (s_u) of clays is commonly determined from an unconfined compression test. The undrained shear strength (s_u) of a cohesive soil is equal to one-half the unconfined compressive strength (q_u) when the soil is under the f = 0 condition (ϕ = the angle of internal friction). The most critical condition for the soil usually occurs immediately after construction, which represents undrained conditions, when the undrained shear strength is basically equal to the cohesion (c). This is expressed as:

$$s_u = c = \frac{q_u}{2}$$

Then, as time passes, the pore water in the soil slowly dissipates, and the intergranular stress increases, so that the drained shear strength (s), given by $s = c + \sigma \tan\phi$, must be used. Where σ'= intergranular pressure acting perpendicular to the shear plane; and $\sigma' = (\sigma - u)$, (σ = total pressure, and u = pore water pressure; c' and σ' are drained shear strength parameters.

Equipment

- Compression device
- Load and deformation dial gauges
- Sample trimming equipment
- Balance
- Moisture can.

Test Procedure

(1) Extrude the soil sample from Shelby tube sampler. Cut a soil specimen so that the ratio (L/d) is approximately between 2 and 2.5. Where L and d are the length and diameter of soil specimen, respectively.

(2) Measure the exact diameter of the top of the specimen at three locations 120° apart, and then make the same measurements on the bottom of the specimen. Average the measurements and record the average as the diameter on the data sheet.

(3) Measure the exact length of the specimen at three locations 120° apart, and then average the measurements and record the average as the length on the data sheet.

(4) Weigh the sample and record the mass on the data sheet.

(5) Calculate the deformation (DL) corresponding to 15% strain (e).

$$\text{Strain (e)} = \frac{\Delta L}{L_o}$$

Where L_o = Original specimen length (as measured in step 3).

(6) Carefully place the specimen in the compression device and center it on the bottom plate. Adjust the device so that the upper plate just makes contact with the specimen and set the load and deformation dials to zero.

(7) Apply the load so that the device produces an axial strain at a rate of 0.5% to 2.0% per minute, and then record the load and deformation dial readings on the data sheet at every 20 to 50 divisions on deformation the dial.

(8) Keep applying the load until (1) the load (load dial) decreases on the specimen significantly, (2) the load holds constant for at least four deformation dial readings, or (3) the deformation is significantly past the 15% strain that was determined in step 5.

(9) Draw a sketch to depict the sample failure.

(10) Remove the sample from the compression device and obtain a sample for water content determination. Determine the water content as in Experiment 1.

Analysis

(1) Convert the dial readings to the appropriate load and length units, and enter these values on the data sheet in the deformation and total load columns. (Confirm that the conversion is done correctly, particularly proving dial gage readings conversion into load)

(2) Compute the sample cross-sectional area

$$A_0 = \frac{\pi}{4} \times (d)^2$$

(3) Compute the strain,

$$e = \frac{?L}{L_0}$$

(4) Compute the corrected area,

$$A' = \frac{A_0}{1-e}$$

(5) Using A', compute the specimen stress,

$$s_c = \frac{P}{A'}$$

(Be careful with unit conversions and use constant units).

(6) Compute the water content, w%

(7) Plot the stress versus strain. Show q_u as the peak stress (or at 15% strain) of the test. Be sure that the strain is plotted on the abscissa.

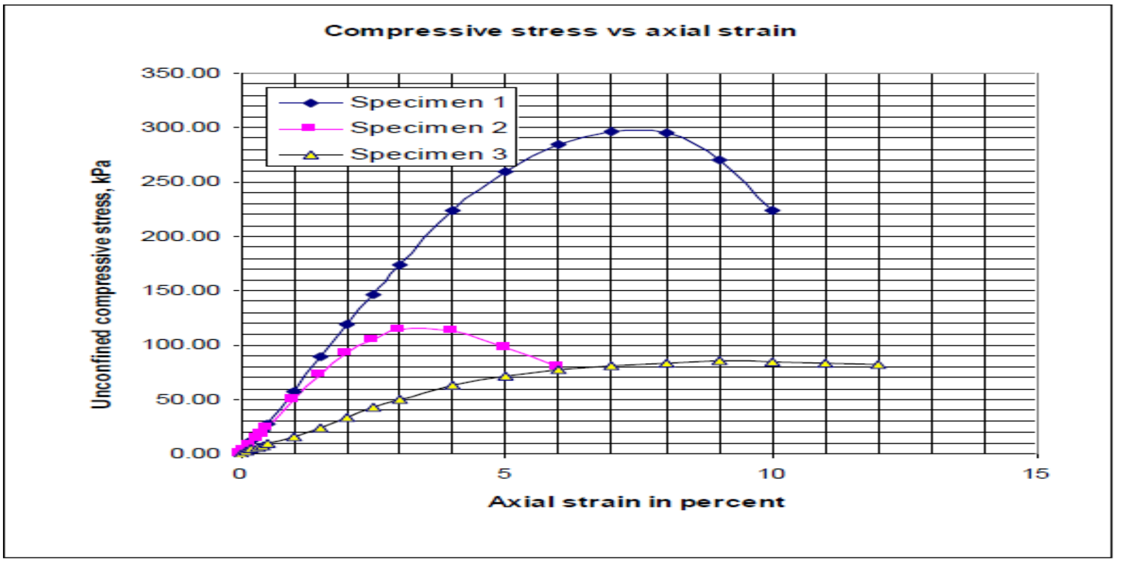


Figure 10.1: Compressive stress vs axial strain(http://www.iitgn.ac.in/research/stl)

(8) Draw Mohr's circle using q_u from the last step and show the undrained shear strength, $s_u = c$ (or cohesion) $= q_u/2$.

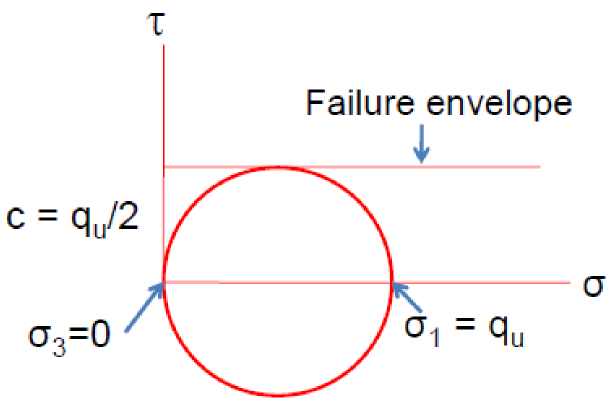


Figure 10.2: Mohr's Circle (http://www.geosci.usyd.edu.au/users/prey/Teaching/Geol-3101/EReport03/GroupA/Report2/Kathy_E_Report.html)

(a)

(b)

Figure 10.3 (a) Cut the soil sample in a uniform shape (b) Place the sample at the apparatus, switch on theapparatus and take reading.

DATA SHEET

Experiment Name : **Unconfined Compression Test**
Experiment Date :

Student Name :
Student's ID :
Year/ Semester :
Section/ Group :

Visual Classification :

Sample data:
Initial Diameter (d) :
Initial Length (L_0) :
Initial Area (A_0) :

Result:
Unconfined compressive strength (q_u) :
Cohesion (c) :

Course Teacher :
Designation :

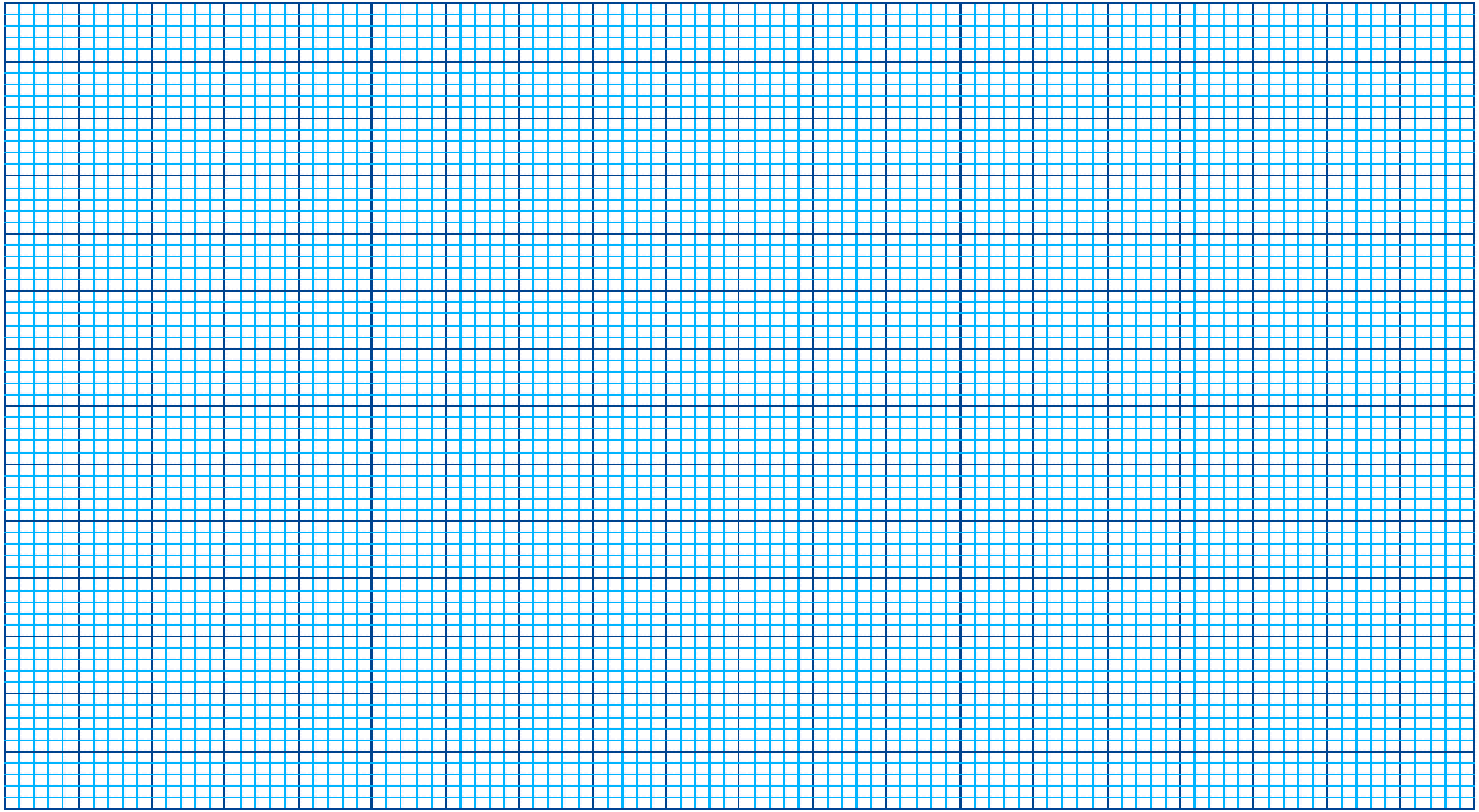
Signature

Unconfined Compression Test Data

Deformation reading		Load reading		Strain (ε)	% Strain	Corrected Area, A = $\frac{A_0}{(1-\varepsilon)}$	Stress (kPa) σ= P/A
Dial Reading	**Sample deformation**	**Dial reading p′ (Proving ring)**	**Load, (lb or kN)**				
20							
40							
60							
80							
100							
120							
140							
160							
180							
200							
220							
240							
260							
280							
300							
320							
340							
360							
380							
400							
420							
440							
460							
480							
500							
520							
540							
560							
580							
600							
620							
640							
660							
680							
700							
720							
740							
760							
780							
800							
820							
840							
860							
880							
900							
920							
940							
960							
980							
1000							

Stress-Strain curve

Axial Stress (kPa)



Axial Strain (%)

Experiment No. 11

CONSOLIDATION TEST

Scope of the test

This test is performed to determine the magnitude and rate of volume decrease that a laterally confined soil specimen undergoes when subjected to different vertical pressures. From the measured data, the consolidation curve (pressure-void ratio relationship) can be plotted. This data is useful in determining the compression index, the recompression index and the preconsolidation pressure (or maximum past pressure) of the soil. In addition, the data obtained can also be used to determine the coefficient of consolidation and the coefficient of secondary compression of the soil.

Standard Reference

ASTM D2435-04 - Standard Test Method for One-Dimensional Consolidation Properties of Soils.

Significance

The consolidation properties determined from the consolidation test are used to estimate the magnitude and the rate of both primary and secondary consolidation settlement of a structure or an earth fill. Estimates of this type are of key importance in the design of engineered structures and the evaluation of their performance.

Equipment

- Consolidation device (including ring, porous stones and load plate)
- Dial gauge (0.0001 inch = 1.0 on dial)
- Sample trimming device
- Glass plate
- Metal straight edge
- Clock
- Moisture can
- Filter paper

Figure 11.1 Consolidation Apparatus of the laboratory

Test Procedure

(1) Weigh the empty consolidation ring together with glass plate.

(2) Measure the height (h) of the ring and its inside diameter (d).

(3) Extrude the soil sample from the sampler, generally thin-walled Shelby tube. Determine the initial moisture content and the specific gravity of the soil, respectively.

(4) Cut approximately a three-inch long sample. Place the sample on the consolidation ring and cut the sides of the sample to be approximately the same as the outside diameter of the ring. Rotate the ring and pare off the excess soil by means of the cutting tool so that the sample is reduced to the same inside diameter of the ring. It is important to keep the cutting tool in the correct horizontal position during this process.

(5) As the trimming progresses, press the sample gently into the ring and continue until the sample protrudes a short distance through the bottom of the ring. Be careful throughout the trimming process to insure that there is no void space between the sample and the ring.

(6) Turn the ring over carefully and remove the portion of the soil protruding above the ring. Using the metal straight edge, cut the soil surface flush with the surface of the ring. Remove the final portion with extreme care.

(7) Place the previously weighed Saran-covered glass plate on the freshly cut surface, turn the ring over again, and carefully cut the other end in a similar manner.

(8) Weigh the specimen plus ring plus glass plate.

(9) Carefully remove the ring with specimen from the Saran-covered glass plate and peel the Saran from the specimen surface. Center the porous stones that have been soaking, on the top and bottom surfaces of the test specimen. Place the filter papers between porous stones and soil specimen. Press very lightly to make sure that the stones adhere to the sample.

(10) Being careful to prevent movement of the ring and porous stones, place the load plate centrally on the upper porous stone and adjust the loading device.

(11) Adjust the dial gauge to a zero reading.

(12) Apply loads on the load plate. First start by 25 kPa then after 1 day 50 and follow loadings according to data sheet.

(13) Record the consolidation dial readings at the elapsed times given on the data sheet.

(14) At the last elapsed time reading, record the final consolidation dial reading and time, release the load, and quickly disassemble the consolidation device and remove the specimen. Quickly but carefully blot the surfaces dry with paper toweling. (The specimen will tend to absorb water after the load is released.)

(15) Place the specimen and ring on the Saran-covered glass plate and, once again, weigh them together.

(16) Weigh an empty large moisture can and lid.

(17) Carefully remove the specimen from the consolidation ring, being sure not to lose too much soil, and place the specimen in the previously weighed moisture can. Place the moisture can containing the specimen in the oven and let it dry for 12 to 18 hours.

(18) Weigh the dry specimen in the moisture can.

Analysis

Determination of pre consolidation pressure, p_o

(1) By visual observation, establish point a, at which the e–log p plot has a minimum radius of curvature.
(2) Draw a horizontal line ab.
(3) Draw the line ac tangent at a.
(4) Draw the line ad, which is the bisector of the angle bac.
(5) Project the straight-line portion gh of e–logp plot back to intersect line ad at f. The abscissa of point f is the preconsolidation pressure, pc.

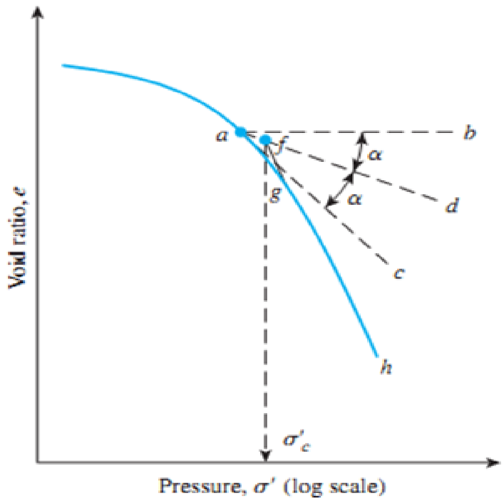


Figure 11.2: Determination of pre-consolidation pressure (Das, 2010)

Determination of t_{50} from deformation vs time curve

(1) By visual observation, establish point 1, at which curve start falling. Take the time t_1 from the point
(2) Take another point 2 on curve from $t_2 = \frac{t_1}{4}$
(3) Draw the horizontal line at a distance from point 2 just equal to the distance of point 1 & 2. Record the deformation reading as d_0.
(4) Take another point d_{100} as like figure.
(5) Take the point t_{50} from $d_{50} = \frac{d_0 + d_{100}}{2}$

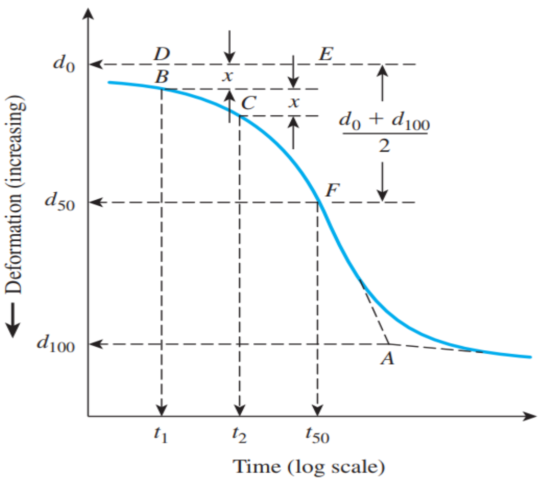


Figure 11.3: Logarithm-of-time method

Determination of t_{90} (from deformation) vs $\sqrt{time}$ curve

(1) Extend the straight portion of the curve and note point t_1.

(2) Take another point t_2=1.15 t_1 and draw a straight line joining t_2 and the first point of the curve.

(3) Take point d_{90} and t_{90} from the intersection point of the curve and the straight line from t_2.

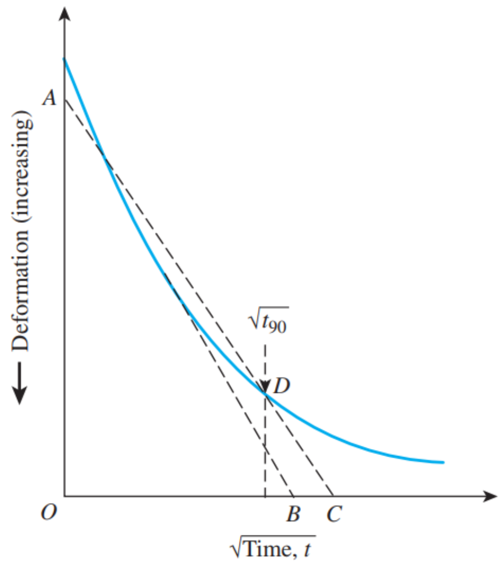


Figure 11.4: Square-root-of-time fitting method

DATA SHEET

Experiment Name : **Consolidation Test**
Experiment Date :

Student Name :
Student's ID :
Year/ Semester :
Section/ Group :

Visual Classification :

Inside diameter of the soil sample :
Initial height of soil sample, H_i :
Area of soil sample, A :
Mass of soil sample, W_S :
Specific gravity of The Soil, G_S :

Consolidation Test Data

Date	**Time**	**Load kPa**	**Elapsed Time**	**Dial Reading**	**Remarks**		**Date**	**Time**	**Load kPa**	**Elapsed Time**	**Dial Reading**	**Remarks**
		25	0						100	0		
			¼							¼		
			½							½		
			1							1		
			2							2		
			4							4		
			8							8		
			15							15		
			30							30		
			60							60		
			120							120		
			240							240		
			480							480		
			1440							1440		
		50	0						200	0		
			¼							¼		
			½							½		
			1							1		
			2							2		
			4							4		
			8							8		
			15							15		
			30							30		
			60							60		
			120							120		
			240							240		
			480							480		
			1440							1440		

Date	Time	Load kPa	Elapsed Time	Dial Reading	Remarks		Date	Time	Load kPa	Elapsed Time	Dial Reading	Remarks
		400	0						1600	0		
			¼							¼		
			½							½		
			1							1		
			2							2		
			4							4		
			8							8		
			15							15		
			30							30		
			60							60		
			120							120		
			240							240		
			480							480		
		800	0									
			¼									
			½									
			1									
			2									
			4									
			8									
			15									
			30									
			60									
			120									
			240									
			480									
			1440									
			Unloading							**Unloading**		
		400	0						200	0		
			¼							¼		
			½							½		
			1							1		
			2							2		
			4							4		
			8							8		
			15							15		
			30							30		
			60							60		
			120							120		
			240							240		
			480							480		
			1440							1440		

Plotting e-log p curve

Initial height of sample, H_i :

Area of soil sample, $A = \frac{\pi d^2}{4}$:

Height of solid, $H_S = \frac{W_S}{G_S \gamma_w A}$:

Height of void, $H_v = (H_i - H_S)$:

Initial Void ratio, $e_o = \frac{HV}{H_S}$:

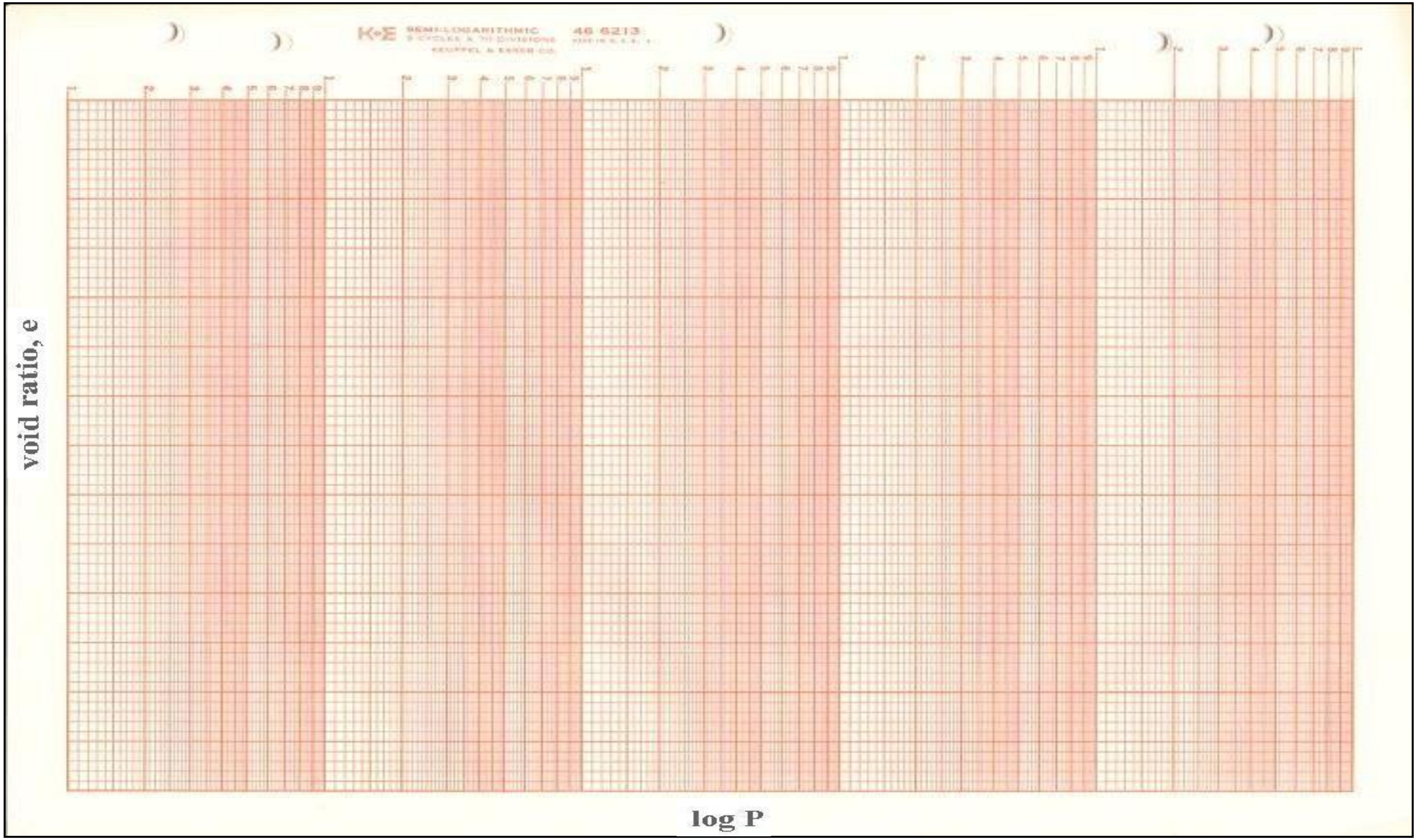
Calculating void ratio for any load

Height of void, H_v = (H_i-change in deformation dial reading x factor-H_S)

Void ratio, $e = \frac{HV}{H_S}$

Load, P (kPa)	Height of void, H_v	Void ratio, e	Log P
25			
50			
100			
200			
400			
800			
1600			
Unloading			
400			
200			

e logP Curve



Determination of coefficient of consolidation, C_V for 50% consolidation:

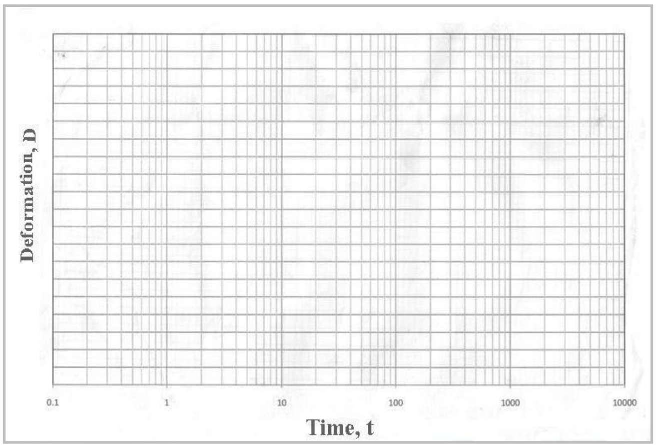
$$C_V = \frac{0.197\ XH_{dr}^2}{t_{50}}$$

Pressure Kpa	D_0 (from graph)	D_{100} (from graph)	$D_{50} = \frac{D_0 + D_{100}}{2}$	t_{50} (from graph for D_{50})	Height of drainage $H_{dr} = H_i - (D_{50} -$ initial reading) x factor	$C_V = \frac{0.197\ xH_{dr}^2}{t_{50}}$
25						
50						
100						
200						
400						
800						
1600						

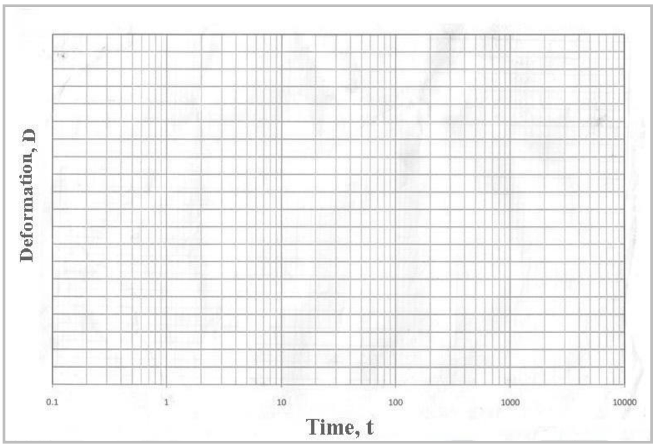
* For double drainage H_{dr} will be divided by 2.

Time vs deformation curve

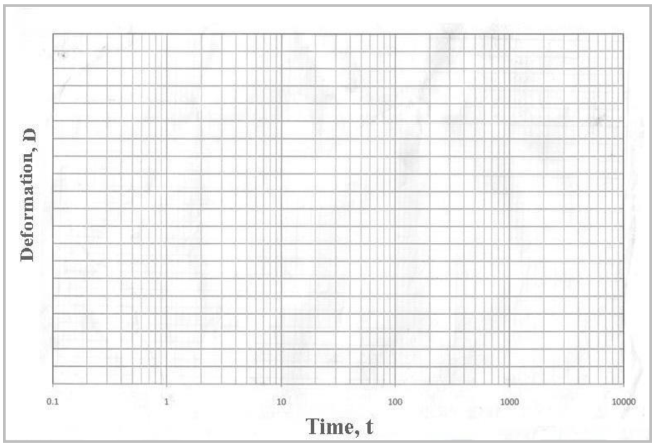
For 25 kPa



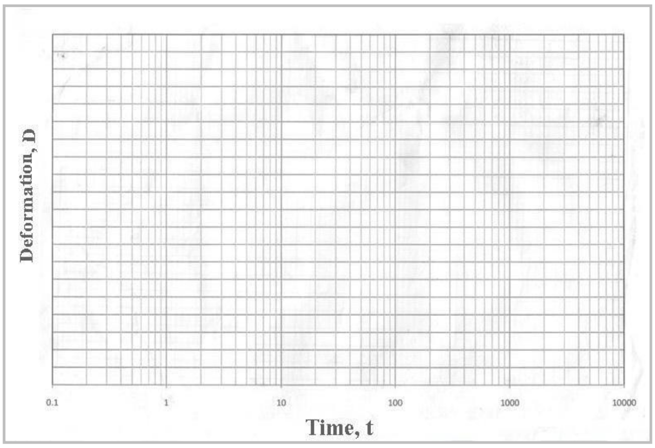
For 50 kPa



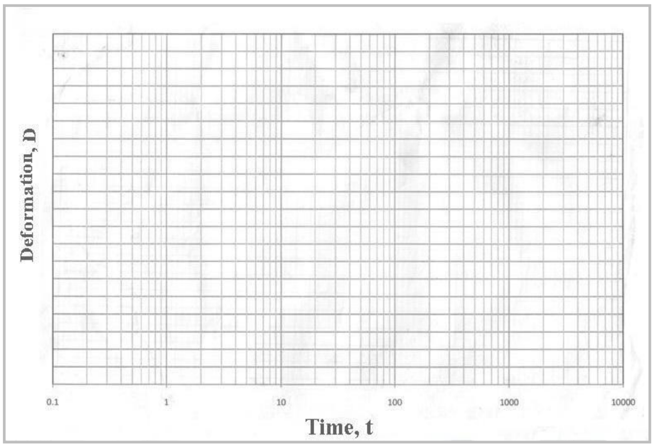
For 100 kPa



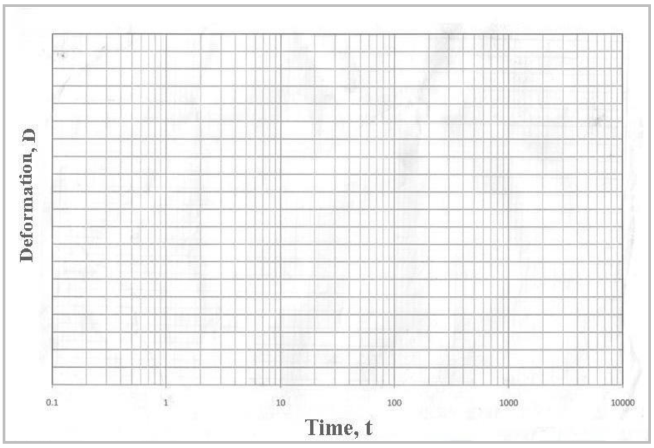
For 200 kPa



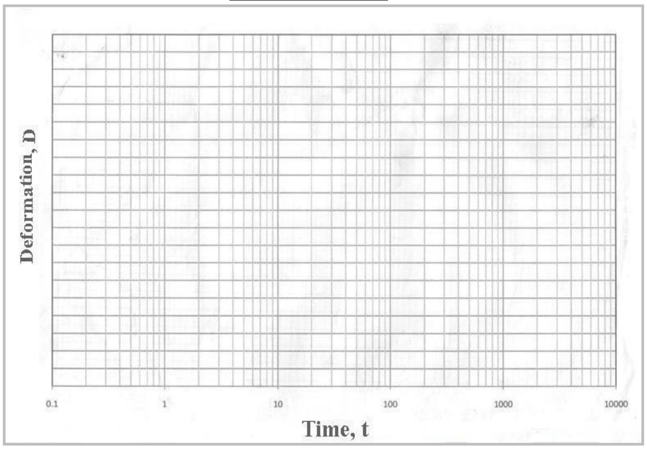
For 400 kPa



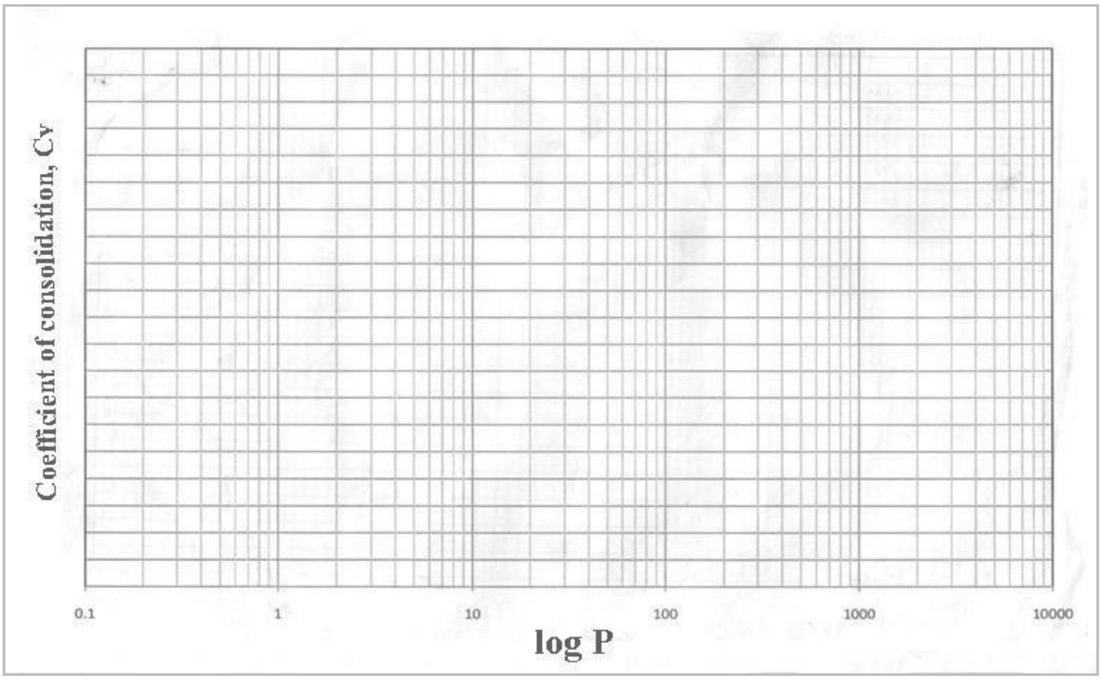
For 800 kPa



For 1600 kPa



C_V vs logP curve:



Determination of coefficient of consolidation, C_V for 90% consolidation:

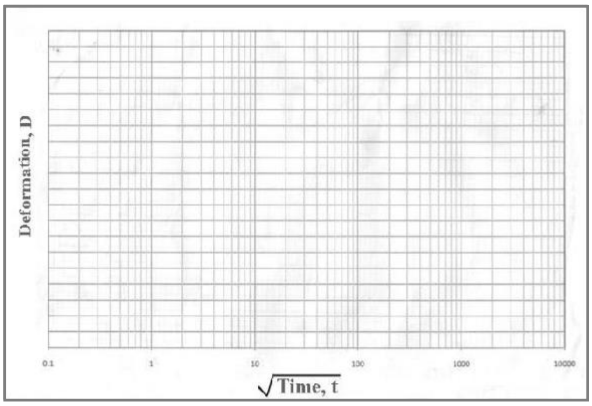
$$C_V = \frac{0.848\ X H_{dr}^2}{t_{90}}$$

Pressure Kpa	**D_{90} (from graph)**	**t_{90} (from graph for D_{90})**	**Height of drainage $H_{dr} = H_i$-(D_{90} – initial reading) x factor**	$C_V = \frac{0.848\ X H^2_d}{t_{50}}$
25				
50				
100				
200				
400				
800				
1600				

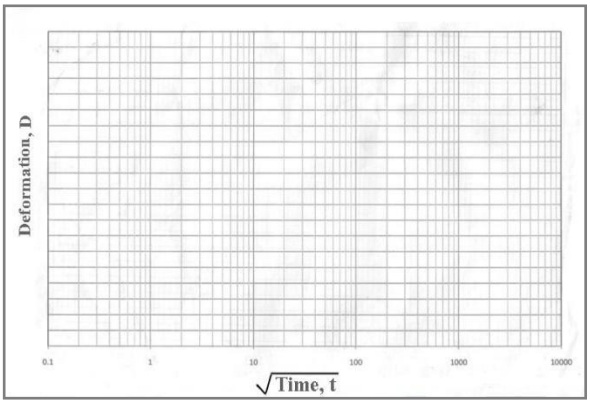
* For double drainage H_{dr} will be divided by 2.

Time vs deformation curve:

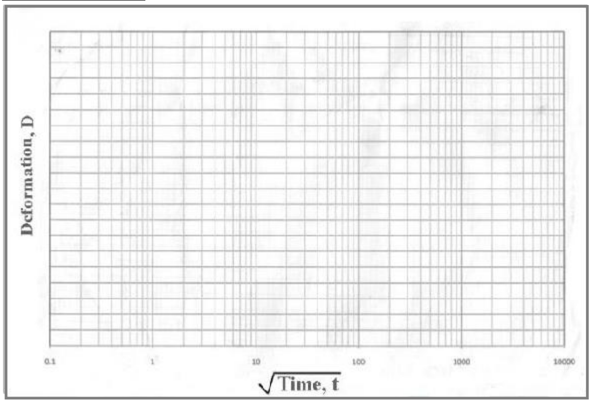
For 25 kPa



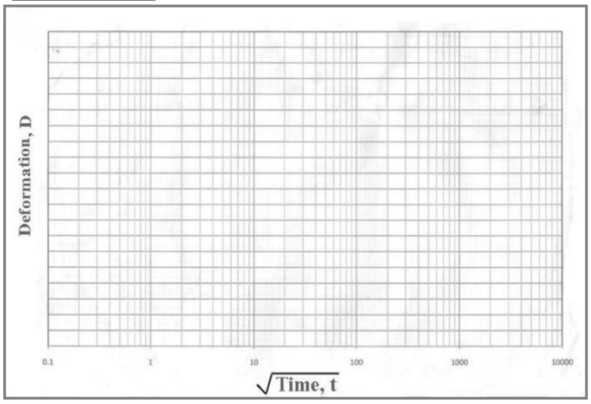
For 50 kPa



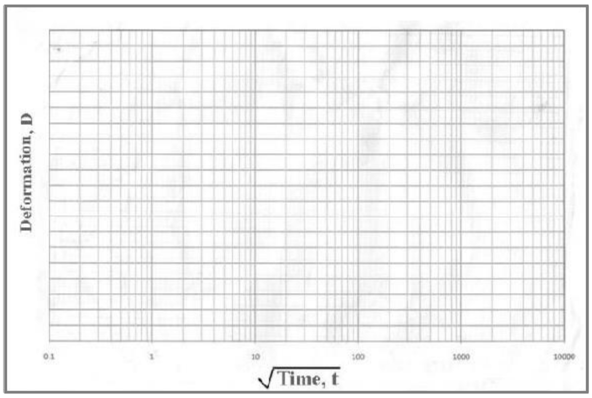
For 100 kPa



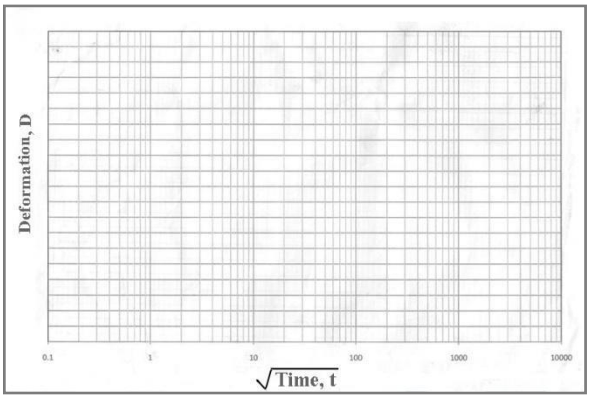
For 200 kPa



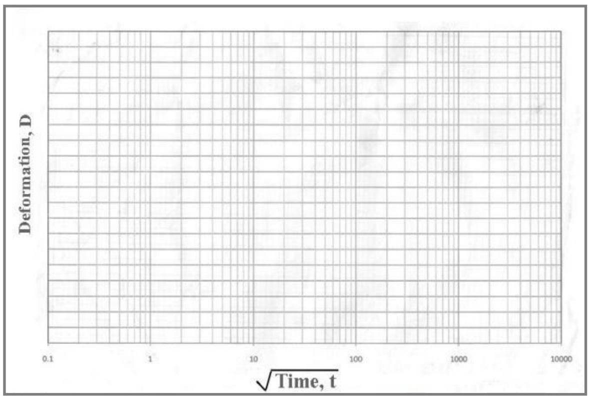
For 400 kPa



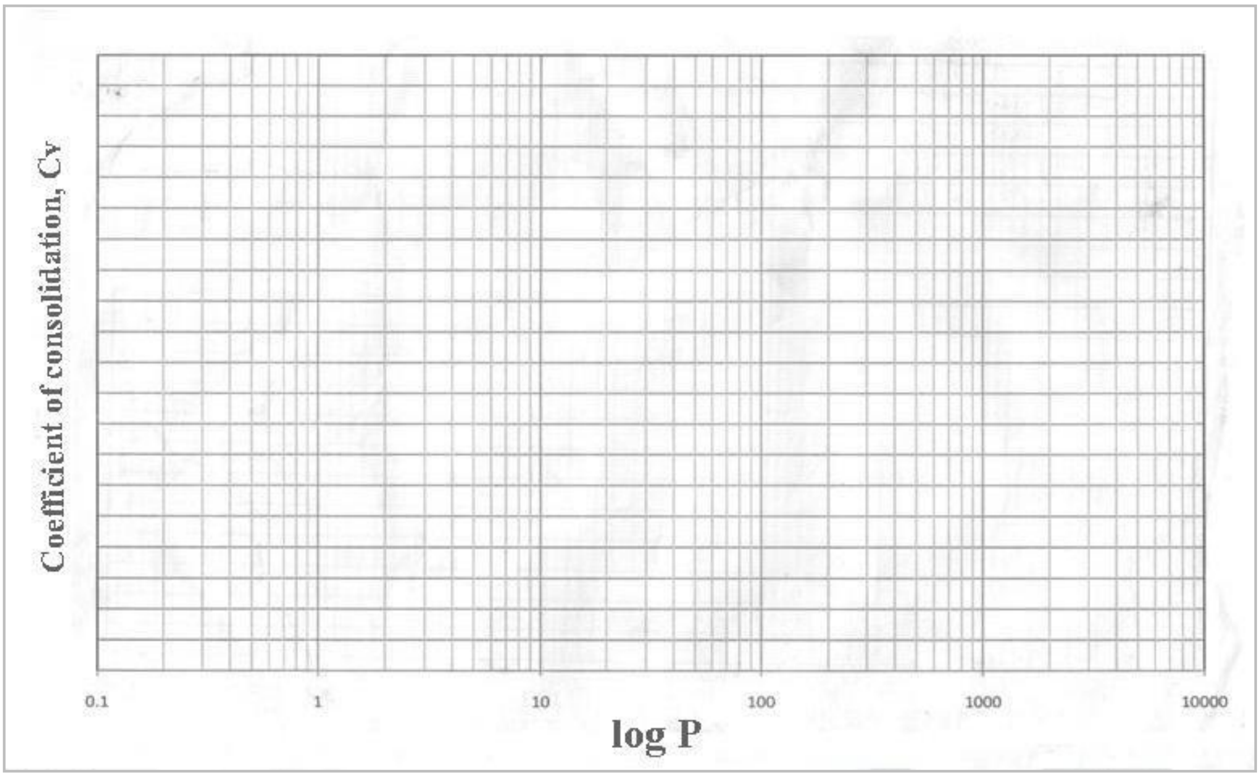
For 800 kPa



For 1600 kPa



C_V vs logP curve:



Final Results:

Compression Index (C_c) :
Recompression Index (C_r) :
Preconsolidation pressure (P_c) :
Coefficient of consolidation (C_v) for 50% consolidation:________to______in^2/min
Coefficient of consolidation (C_v) for 90% consolidation:_______to_______in^2/min

Course Teacher :
Designation :

Signature

APPENDIX

Appendix 1

Common Uses Engineering Properties of Soil

Cohesionless soil

	Very loose	Loose	Medium	Dense	Very dense
Relative density	< 15%	15%-35%	35%-65%	65%-85%	85%-100%
Internal friction angle ϕ (deg)	< 28	28-30	30-36	36-40	> 40
Dry unit weight (lb/ft^3)	< 90	90-100	100-110	110-120	> 120
Moist unit weight (lb/ft^3)	< 100	100-120	120-130	130-140	> 140
SPT Value	0-4	4-10	10-30	30-50	> 50

Cohesive Soil

	Very soft	Soft	Medium	Stiff	Very Stiff	Hard
Moist unit weight (lb/ft^3)	< 100	100	115	120	125	> 130
Unconfined compressive strength (lb/ft^2)	< 500	500-1000	100-2000	2000-4000	4000-8000	> 8000
SPT Value	≤ 2	2-4	4-8	8-16	16-32	> 32
Soil bearing capacity (lb/ft^2)	< 1000	1000-2000	2000-4000	4000-8000	8000-16000	> 16000

Appendix 2
Different Consistencies of soils

Typical Values of Specific Gravity of Soil	
Soil Type	Specific Gravity
Sand	2.65-2.67
Silty sand	2.67-2.70
Inorganic clay	2.70-2.80
Soils with mica or iron	2.75-3.00
Organic Soils	Qiute variable, as low as 2.0

Relative Density	State of compaction
0-15	Very loose
15-50	Loose
50-70	Medium dense
70-85	Dense
85-100	Very dense

Soil	Liquid limit	Plastic limit
Silt clay mixture	25-40	20-30
Kaolinite clay	40-70	20-40
Montmorillonite clay	300-600	100-200

Plasticity Index, I_P	consistency
0-3	Nonplastic
4-6	Slightly plastic
7-15	Moderately plastic
16-35	Plastic
Over 35	Highly plastic

Soil Type	Angle of internal friction, ϕ	
	Loose	Dense
Nonplastic silt	27-30	30-34
Silty sand	27-33	30-35
Uniform sand	28	34
Well graded sand	33	45
Sandy gravel	35	50

Consistency	Unconfined compressive strength, q_u(KN/m^2)
Very soft	<25
Soft	25-50
Medium	50-100
Stiff	100-200
Very stiff	200-400
Hard	400-800

Organic content	Type
<5%	Inorganic
6-20%	Organic silts or clays
21-74%	Silty or clayey organic
>75%	Peat

Degree of Permeability	Coefficient of Permeability K (cm/sec)
High	Over 10^{-1}
Medium	10^{-1} to 10^{-3}
Low	10^{-3} to 10^{-5}
Very Low	10^{-5} to 10^{-7}
Practically Impermeable	Less than 10^{-7}

Appendix 3
Lab Report Format

1. All students must have a same colored printed **cover page**. The design of cover page is provided with the lab manual. Students have to compose only the course teacher's name and designation ant their information.

2. An **index** is provided. It should be printed and set after the cover page. Table may be fill up by pen during each submission after test.

3. Each report must have a common printed **top page**. Only the experiment name and no. and the date may be filled up by pen. A top page design is provided.

4. **A4 papers** have to be used for preparing the lab report. Writing should be done with **pen**. Pencil may be used for any kind of sketch.

5. In each experiment of the lab report the following points must have to be present: **Objective, Standard Reference, Significance, Equipment, Procedure, Data Table (signed), Sample Calculation, Result** and **Discussion**.

CE 344
Geotechnical Engineering Sessional-I
(Lab Report)

Submitted to Course Teacher(s)
Name …………………………..
Designation ……………………
&
Name ………………………….
Designation ……………………

Submitted by the Student
Name ……………….……..
Year/ Semester………….
Student's ID..………….
Group ………………….

INDEX

Expt. No.	Test Name	Date of Performance	Date of Submission	Signature	Comments	Page No.

CE 344
Geotechnical Engineering Sessional-I
(Lab Report)

Experiment No. :
Experiment Name:

Date of Performance:
Date of Submission:

Submitted to Course Teacher(s)
Name …………………………..
Designation ……………………
&
Name ………………………….
Designation ……………………

Submitted by the Student
Name ……………..……..
Year/ Semester………….
Student’s ID..………….
Group ………………….

Appendix 4
Lab Instructions

1. All students must have to be present at laboratory just in time.
2. All students must have to submit the lab report just after the entrance and before the class start.
3. Lab reports have to be submitted serially according to Student's ID.
4. Students have to complete the data sheet in class and complete sample calculations and graphs in class and take sign from the course teacher. (In some experiment which require more times, data sheet should be completed as possible in class time.)
5. Students should be very careful about any test. They should conduct the tests by taking maximum care of the equipment during test.

Relevant Experiment

MOISTURE CONTENT DETERMINATION

Scope of the test

This test is performed to determine the water (moisture) content of soils. The water content is the ratio, expressed as a percentage, of the mass of ―pore‖ or ―free‖ water in a given mass of soil to the mass of the dry soil solids.

Standard Reference

ASTM D 2216-19 - Standard Test Method for Laboratory Determination of Water (Moisture) Content ofSoil, Rock, and Soil-Aggregate Mixtures.

Significance

For many soils, the water content may be an extremely important index used for establishing the relationship between the way a soil behaves and its properties. The consistency of a fine-grained soil largely depends on its water content. The water content is also used in expressing the phase relationships of air, water, and solids in a given volume of soil.

Equipment

- Drying oven
- Balance
- Moisture can
- Gloves
- Spatula.

Test Procedure

(1) Record the moisture can and lid number. Determine and record the mass of empty, clean, and dry moisture can with its lid (M_1)

(2) Place the moist soil in the moisture can and secure the lid. Determine and record the mass of the moisture can (now containing the moist soil) with the lid (M_2).

(3) Remove the lid and place the moisture can (containing the moist soil) in the drying oven that is set at 105 °C. Leave it in the oven overnight.

(4) Remove the moisture can. Carefully but securely, replace the lid on the moisture can using gloves, and allow it to cool to room temperature. Determine and record the mass of the moisture can and lid (containing the dry soil) (M_3).

(5) Empty the moisture can and clean the can and lid.

Data Analysis

(1) Determine the mass of soil solids.

$$\mathbf{M_S = M_3 - M_1}$$

(2) Determine the mass of pore water.

$$\mathbf{M_W = M_3 - M_2}$$

(3) Determine the water content.

$$W = \frac{M_W}{M_S} \, x \, 100\%$$

DATA SHEET

Experiment Name : **Moisture Content determination of Soil**
Experiment Date :

Student's Name :
Student's ID :
Year/ Semester :
Section/ Group :

Sample Description :

Specimen number	1	2
Moisture and lid number		
M_1 = Mass of empty, clean can + lid (gm)		
M_2 = Mass of can, lid and moist soil (gm)		
M_3 = Mass of can, lid and dry soil (gm)		
M_S = Mass of soil solids (gm)		
M_W = Mass of pore water (gm)		
w = Water content %		

Sample Calculation:

Result:
Moisture content of the soil is:

Course Teacher :
Designation :

Signature

ORGANIC CONTENT DETERMINATION TEST

Scope of the test

This test is performed to determine the organic content of soils. The organic content is the ratio, expressed as a percentage, of the mass of organic matter in a given soil mass to the mass of the dry soil solids.

Standard Reference

ASTM D 2974-20 – Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and OrganicSoils.

Significance

Organic matter influences many of the physical, chemical and biological properties of soils. Some of the properties influenced by organic matter include soil structure, soil compressibility and shear strength. In addition, it also affects the water holding capacity, nutrient contributions, biological activity, and water and air infiltration rates.

Equipment

- Muffle furnace,
- Balance,
- Porcelain dish,
- Spatula,
- Tongs

Test Procedure

(1) Determine and record the mass of an empty, clean, and dry porcelain dish (MP).

(2) Place a part of or the entire oven-dried test specimen from the moisture content experiment (Expt.1) in the porcelain dish and determine and record the mass of the dish and soil specimen (MPDS).

(3) Place the dish in a muffle furnace. Gradually increase the temperature in the furnace to 440°C. Leave the specimen in the furnace overnight.

(4) Remove carefully the porcelain dish using the tongs (the dish is very hot), and allow it to cool to room temperature. Determine and record the mass of the dish containing the ash (burned soil) (MPA).

(5) Empty the dish and clean it.

Data Analysis

(1) Determine the mass of the dry soil.

$M_D = M_{PDS} - M_P$

(2) Determine the mass of the ashed (burned) soil.

$M_A = M_{PA} - M_P$

(3) Determine the mass of organic matter

$M_O = M_D - M_A$

(4) Determine the organic matter (content).

$$OM = \frac{M_O}{M_D} x100$$

Data Sheet

Experiment Name : **Organic Content determination of Soil**
Experiment Date :

Student's Name :
Student's ID :
Year/ Semester :
Section/ Group :

Sample Description :

Date Tested:
Tested By:
Project Name:
Sample Number:
Sample Description:

Specimen number	1	2
Porcelain dish number		
M_P = Mass of empty, clean porcelain dish (grams)		
M_{PDS} = Mass of dish and dry soil (grams)		
M_{PA} = Mass of the dish and ash (Burned soil) (grams)		
M_D = Mass of the dry soil (grams)		
M_A = Mass of the ash (Burned soil) (grams)		
M_O = Mass of organic matter (grams)		
OM = Organic matter, %		

Course Teacher :
Designation :

Shrinkage Limit Determination

Standard Reference

ASTM D 4943-18 - Standard Test Method for Shrinkage Factors of Soils by the Mercury Method.

Procedure in Brief:

(1) A sample of fine-grained soil is thoroughly remolded with water to approximate the liquid limit consistency.
(2) The saturated paste is placed into a container of known volume and slowly dried.
(3) The final mass and volume of the solid soil pat are determined.
(4) These measurements are used to compute the SL.

Procedure in Detail (Volume of Wet Soil Pat):

(1) Make the soil paste of liquid consistency
(2) Coat the inside of the shrinkage dish with a thin layer of petroleum jelly, silicone grease, or similar lubricant to prevent the adhesion of the soil to the dish.
(3) Determine and record the mass in grams of the empty dish, M_T.
(4) Place the shrinkage dish in the shallow pan in order to catch any mercury overflow.
(5) Fill the shrinkage dish to overflowing with mercury.
(6) Remove the excess mercury by pressing the glass plate firmly over the top of theshrinkage dish.
(7) Determine the volume of mercury held in the shrinkage dish either by means of the glass graduate or by dividing the measured mass of mercury by the mass density of mercury (equal to 13.55 Mg/m^3).
(8) Record this volume in cubic centimeters of the wet soil pat, *V*.

Procedure in Detail (Volume of Dry Soil Pat)

(1) Place the glass cup in the shallow pan in order to catch any mercury overflow.
(2) Fill the glass cup to overflowing with mercury.
(3) Remove the excess mercury by pressing the glass plate with the three prongs firmly over the top of the cup.
(4) Place the evaporating dish in the shallow pan in order to catch any mercury overflow. Place the cup filled with mercury in the evaporating dish and rest the soil pat on the surface of the mercury (it will float).
(5) Using the glass plate with the three prongs gently press the pat under the mercury and press the plate firmly over the top of the cup to expel any excess mercury. Observe that there is no air trapped between the plate and mercury,
(6) Measure the volume of the mercury displaced into the evaporating dish either by means of the glass graduate or by dividing the measured mass of mercury by the mass density ofmercury.
(7) Record the volume in cubic centimeters of the dry soil pat, V_o.

Analysis to Determine Liquid Limit

(1) Calculate the water content of each of the liquid limit moisture cans after they have been in the oven for at least 16 hours.
(2) Plot the number of drops, N, (on the log scale) versus the water content (w). Draw the best-fit straight line through the plotted points and determine the liquid limit (LL) as the water content at 25 drops.

Analysis to Determine Plastic Limit

(1) Calculate the water content of each of the plastic limit moisture cans after they have been in the oven for at least 16 hours.
(2) Compute the average of the water contents to determine the plastic limit, PL. Check to see if the difference between the water contents is greater than the acceptable range of two results (2.6 %).
(3) Calculate the plasticity index, PI=LL-PL. Report the liquid limit, plastic limit, and plasticity index to the nearest whole number, omitting the percent designation.

Analysis to Determine Shrinkage Limit

(1) Calculate the water content of each of the liquid limit moisture cans after they have been in the oven for at least 16 hours.

$$w = \frac{M - Mo}{Mo} \times 100$$

(2) Compute Shrinkage Limit by the following equation:

$$SL = w - \frac{(V - Vo)\rho w}{Mo} \times 100$$

REFERENCES

- ASTM D1557-12 - Standard Test Methods for Laboratory Compaction Characteristics of Soil UsingModified Effort (56,000 ft-lbs/ft3(2,700 KN-m/m3))
- ASTM D2166 - 06- Standard Test Method for Unconfined Compressive Strength of Cohesive Soil ASTM D 2216 - Standard Test Method for Laboratory Determination of Water (Moisture) Content ofSoil, Rock, and Soil-Aggregate Mixtures.
- ASTM D2434 - 22 - Standard Test Method for Permeability of Granular Soils (Constant Head) (Note:The Falling Head Test Method is not standardized)
- ASTM D2435 -04 - Standard Test Method for One-Dimensional Consolidation Properties of Soils. ASTM D 2488 - Standard Practice for Description and Identification of Soils (Visual - Manual Procedure)
- ASTM D2974 - 20 - Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and OrganicSoils.
- ASTM D6913 - 04 - Standard Test Method for Particle-Size Analysis of Soils.
- ASTM D4943 - 18 - Standard Test Method for Shrinkage Factors of Soils by the Mercury Method.
- ASTM D4318 - 17 - Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.
- ASTM D698 - 12 - Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbs/ft3 (600 KN-m/m3))
- ASTM D 854 - Standard Test for Specific Gravity of Soil Solids by Water Pycnometer.
- B.M. Das, Principles of Geotechnical Engineering, 7th Edition.
- Engineering Properties of Soils Based on Laboratory Testing, Prof. Krishna Reddy, UIC**.**
- http://www.iitgn.ac.in/research/stl/uutriaxial.php/02-01-2018
- https://civil-engg-world.blogspot.com/2014/05/rolling-test-dispersion-test-Differentiating-Silt-Clay-Soil.html/02-01-2018.
- https://www.slideshare.net/faize143/review-of-soil-mechanics/02-01-2018.
- https://www.slideshare.net/xakikazmi/strength-nalin/02-01-2018
- IIT Gandhinagar Soil Testing Lab Manual