Engr. Yasser M. S. Almadhoun

SOIL

MECHANICS

LABORATORY MANUAL

Civil Engineering

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Contents

Acronyms

ASCE	American society of civil engineers
ASTM	American society for testing and materials
BS	British standards
AASHTO	American association of state highway and transportation officials
WC	Water (Moisture) content
SG	Specific gravity
UW	Unit weight
LL	Liquid limit
PL	Plastic limit
SL	Shrinkage limit
PI	Plasticity index
K	Coefficient of permeability (Hydraulic conductivity)
$\emptyset$	Angle of internal friction
C	Cohesion

Preface

The importance of a standardized and consistent procedure in soil mechanics in order to establish quality cannot be over emphasized.

This manual describes the proper procedure for laboratory testing of soil mechanics to determine their physical properties, which are an integral part in the design and construction of structural foundations. Therefore, learning to perform laboratory tests of soils plays an important role in the geotechnical engineering profession. The tests involved in this manual are performed in accordance with the relevant standards, i.e. ASTM (American Society for Testing and Materials) and BS (British Standard Methods of Test for Soils for Civil Engineering Purposes).

Preparation of this manual has been a personal endeavour under my work as a teaching assistant at the Islamic University of Gaza in Palestine during the spring semester of the academic year 2016/2017 to support as a teaching material of soil mechanics laboratory to the students who are engaged in civil engineering in their 3rd level of studying bachelor of science programme.

For each laboratory test procedure described onto this PDF deck, a worked example of measurements, computation, results and conclusion is included. Moreover, a blank laboratory data sheet for each test is provided at the end of the manual for student use in the laboratory, as well as in reporting. For this manual, Microsoft Excel templates have been requested to develop and adopted to use for working with this popular spreadsheet programme.

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February 2017

Report Preparation

A laboratory testing report shall include of the following sections:

A. Front Page

Any front page for a laboratory testing report shall include basic information about the name of the class being attended, student name and number, name of the test and date of testing, name of lecturer and date of submission, etc.

B. Report Contents

a. Table of Contents

It is a list, usually headed simply contents, found on a page before the start of a written work, of its chapter or section titles or brief descriptions with their commencing page numbers. However, a table of contents shall be introduced in the beginning of the report document.

b. Introduction

An introductory paragraph shall be written in the beginning of each report as a glimpse on the experiment that the student is going to report about.

c. Definition

A brief theory about the test the student is going to report about shall be given in the beginning of a report in addition to a clear statement of the exact meaning of the test title. The student shall use his own knowledge and words acquired in the class and laboratory.

d. Significance

The importance and uses of the test being under consideration shall be discussed at a glance.

e. Purpose

A brief statement of the purpose of the test that is being considered shall be indicated.

f. Standard Reference

The adopted slandered references accorded in testing shall be highlighted.

g. Equipment and Materials

Special equipment used in laboratory testing should be briefly described in points with supporting photographs in accompany with those materials used to perform the test.

h. Procedure

There are some points that should be taken into account when writing the procedure for a test:

a. The testing procedure shall be described in points.
b. You must describe the actual procedure that were done in the laboratory, nothing less nothing over.
c. Do not copy procedure from the manual or standard codes. Hence, this would account for a cookie, turning a blind eye to excuses, if any.

i. Measurements

All data for a test should be written in a tabular form including measured data in the laboratory and those assumed, if any. Observations relating to the behaviour of materials should be pointed out.

j. Computation

All equations and formulas used in a test should be obviously indicated and all supporting calculations should be shown in detailed steps. Answers should be properly checked logically, neatly and legibly.

k. Results and Discussion

Results of any test should be summarized and compared in a tabular or graphical format. A brief discussion, thereafter, should be included in which

attention is drawn to the silent facts shown by the tables and diagrams. The test results should be compared with the standard values.

l. Conclusion

In a conclusion paragraph, students summarize what they have written about in their report. They need to think about the main point that they want to get across and be sure it is included. In addition, they need to comment on their results obtained and the circumstances in which their experiment was performed.

C. Appendices
a. Appendix A: Laboratory Data Sheet

The laboratory data sheet, used in each testing, shall be supported in the end of each experiment, in a distinct sheet, namely appendix A. Each data sheet shall include the following sections:

a. General information.
b. Test details.
c. Measurements and computations.
d. Computation space.

b. Appendix B: Grouping Students Names

Every student shall write a list, in a distinct appendix sheet, namely appendix B, for names of the students, whom he has engaged with them in the same group to perform the test.

TEST NO. 01

MEASUREMENT OF WATER CONTENT

(ASTM D2216)

Introduction

This test method covers the measurement of water content of soil, by measuring the weight of water removed by drying a sample of moist soil to reach solid particles. The water content is used to assess the consistency of soil and to establish relationships between volumes and weights of soil ingredients (solids, water and air). This experiment is most important in case of Clay because of its expansion and shrinkage in addition to having high moisture contents and small voids between particles.

Definition

Water content may be defined as the quantity of water contained in a soil. It is commonly known as moisture content. However, percent water content of a soil is the ratio of the weight of free water present either on the soil surface or in the voids between soil particles to the weight of dry solids of that soil.
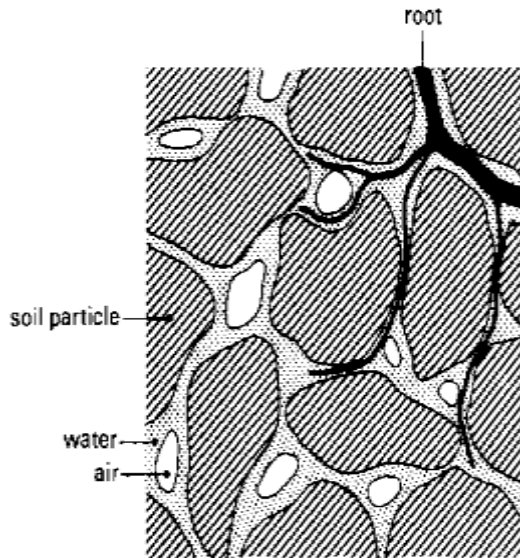
Figure 1.01: Soil components.   Figure 1.02: Model for soil.
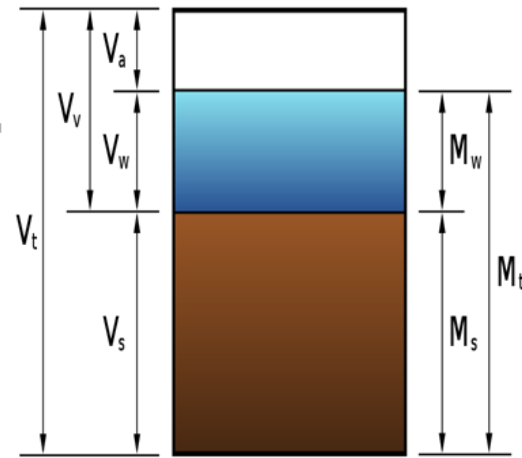
Significance

- For many soils, the water content may be an extremely important index used for establishing a 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.

Purpose

Measurement of the water (moisture) content of soils. The water content is the ratio, expressed as a percentage, of the mass of pore (free) water in a given mass of soil to the mass of the dry soil solids.

Standard Reference

ASTM D2216 – Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock and Soil-Aggregate Mixtures.

Equipment and Materials

(1) Sensitive balance with an accuracy of 0.01 gram.
(2) Drying oven controlled to 105 - 110 ˚C.
(3) Metal container (can).
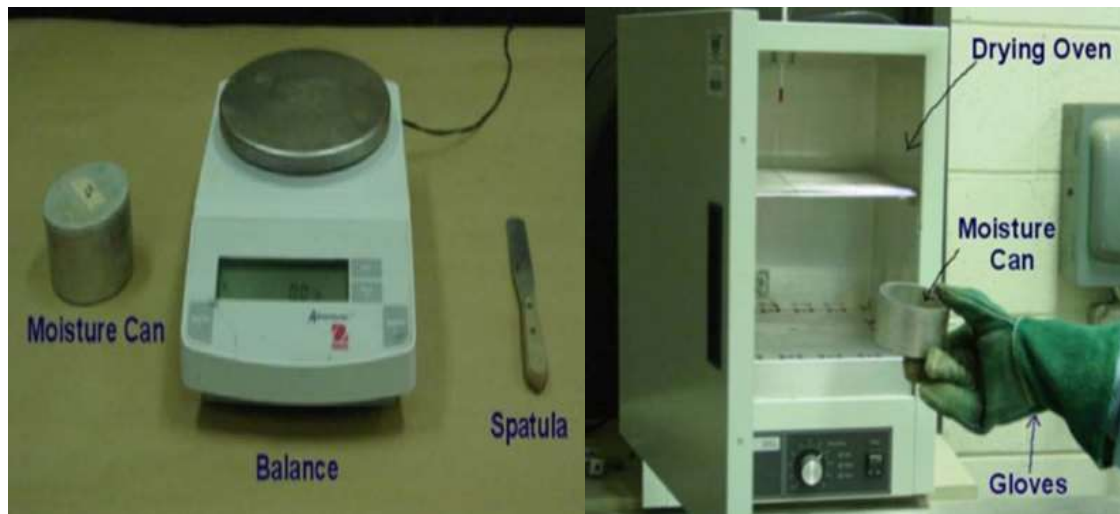(4) Spatula and gloves.
(5) Plastic stack.

Figure 1.03: Moisture can, balance and spatula.   Figure 1.04: Drying oven and gloves.

Procedure

(1) Record the moisture can and lid number. Weigh and record the mass of the can with its lid when it is empty, clean and dry with its lid (M_c).

(2) Place a sample of moist soil in the moisture can and secure the lid. Weigh and record the mass of the moisture can containing the moist soil sample and the lid over it (M_{wc}).

(3) Remove the lid and place the moisture can containing the moist soil sample in a drying oven set at 105 ± 5 ˚C. Leave it in the oven overnight (16 hours).

(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. Weigh and record the mass of the moisture can containing the dry soil and the lid over it (M_{dc}).

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

(6) Repeat the previous procedural steps several times for different samples of the moist soil then get the average water content for the soil being tested.

Note: In case of soils containing significant amount of gypsum or organic materials, keep the oven temperature at not more than 80 ˚C, for longer period in order to avoid oxidation and loss of water of crystallization.

Data Analysis

After going over the previous section of procedure adopted for this test, calculate the water content using the following equation:

$$\%wc = \frac{M_w}{M_s} \times 100 = \frac{M_{wc} - M_{dc}}{M_{dc} - M_c} \times 100$$

Measurements

There just one specimen of soil was tested and Measurements of the testing are shown as given as under:

Table 1.01: Testing measurements.

Item	Data for Sample 01
No. = Moisture can and lid number	Q301
M_c = Mass of empty, clean can with the lid (gram)	103.38
M_{wc} = Mass of can, lid, and moist soil (gram)	335.45
M_{dc} = Mass of can, lid, and dry soil (gram)	309.70
M_s = Mass of soil solids (gram)	…
M_w = Mass of pore water (gram)	…
%wc = Percent water content	…

* The sign (…) means that the cell needs to be evaluated, for more details go to the computation section.

Computation

$$M_s = M_{dc} - M_c = 309.70 - 103.38 = 206.32 \text{ gm}$$

$$M_w = M_{wc} - M_{dc} = 335.45 - 309.70 = 25.75 \text{ gm}$$

$$\%wc = \frac{M_w}{M_s} \times 100 = \frac{M_{wc} - M_{dc}}{M_{dc} - M_c} \times 100$$

$$\%wc = \frac{335.45 - 309.70}{309.70 - 103.38} \times 100 = 8.3145 \ \%$$

Consequently, final results of the testing are shown as given as under:

Table 1.02: Testing results.

Item	Data for Sample 01
No. = Moisture can and lid number	Q301
M_c = Mass of empty, clean can with the lid (gram)	103.38
M_{mc} = Mass of can, lid, and moist soil (gram)	335.45
M_{dc} = Mass of can, lid, and dry soil (gram)	309.70
M_s = Mass of soil solids (gram)	206.32
M_w = Mass of pore water (gram)	025.75
%wc = Percent water content	8.3145

Results and Discussion

After all calculations have been evaluated once, the results were as follows:

%wc = 8.3145 %

Conclusion

- This experiment is of importance for measuring water content present in soils, in particular, for clay soils, since it gives significant values in case of a clayey soil, for instance:
    - Expansion (Swelling) and shrinkage.
    - High water contents.
    - Small voids between solid particles.
- When soil contains significant amount of gypsum or organic materials, it is recommended to keep the oven temperature at 60 - 80 ˚C, for longer period; to avoid oxidation and loss of water of crystallization due to burning of organic matters and thus this lead to varying the weight of solid particles.

TEST NO. 02

MEASUREMENT OF SPECIFIC GRAVITY

(ASTM D854)

Introduction

This test method covers the measurement of specific gravity of soil solids that pass the 4.75-mm (No. 4) sieve by water pycnometer. When soil contains particles larger than the 4.75-mm sieve, the test method described in ASTM C127 shall be used for the soil solids retained on the 4.75-mm sieve and this test method (ASTM D854) shall be used for the soil solids passing the 4.75-mm sieve, by means of a water pycnometer. Soil solids for these test methods do not include solids which can be altered by these methods, contaminated with a substance that prohibits the use of these methods or are highly organic soil solids, such as fibrous matter which floats in water. The specific gravity of the soil solids at the test temperature shall be calculated from the density of soil solids and the density of water at the test temperature.

Definition

Specific gravity of a soil is defined as the ratio of the unit weight (density) of a given soil solids to the unit weight (density) of water. Therefore:

$$G_s = \frac{\rho_{solid}}{\rho_{water}} = \frac{m_s/V_s}{m_w/V_w}$$

$$\text{If} \quad V_s = V_w \quad \text{then} \quad G_s = \frac{m_s}{m_w}$$

The procedure employs Archimedes' principle which is 'a body submerged in water will displace a volume of water equal to its own volume'.
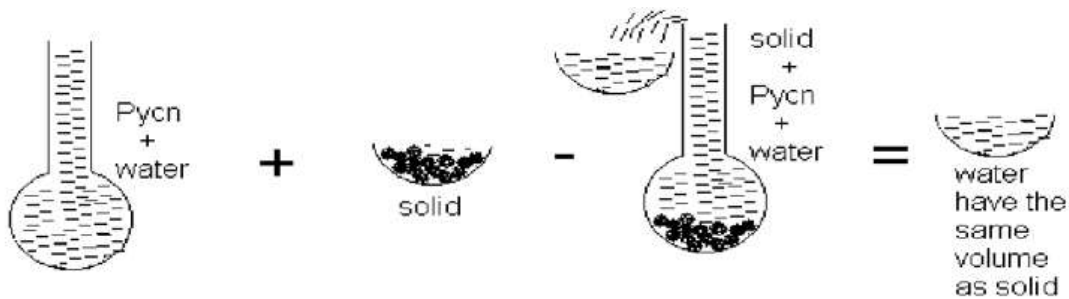

Figure 2.01: Specific gravity measurement.

Most of the values of specific gravity of minerals found in soils fall within a range of 2.6 to 2.9. However, Figure 2.02 shows the specific gravity of some common soils found in nature.

Table 2.01: Specific gravity of common soils.

Soil	Specific Gravity, G_s
Sand	2.65 - 2.67
Silty sand	2.67 - 2.70
Inorganic clay	2.70 - 2.80
Soils with mica and iron	2.75 - 3.00
Clay and silty clay	2.67 - 2.90
Organic soil	< 2.00

Significance

- The specific gravity of soil solids is used in calculating the properties of soils, such as void ratio (e) and degree of saturation (S).
- The specific gravity of soil solids is used to calculate the density of the soil solids. This is done by multiplying its specific gravity by the density of water at proper temperature.
- The term 'soil solids' is typically assumed to mean naturally occurring mineral particles or soil like particles that are not readily soluble in water. Therefore, the specific gravity of soil solids containing extraneous matter, such as cement, lime and the like, water-soluble matter, such as sodium chloride and soils containing matter with a specific gravity less than one, typically require special treatment or a qualified definition of their specific gravity.

Purpose

Measurement of the specific gravity of soil using a pycnometer. Specific gravity is the ratio of the mass of unit volume of soil at a stated temperature to the mass of the 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 and Materials

(1) Pycnometer.
(2) Sieve of 4.75-mm opening (No. 4 sieve).
(3) Vacuum pump, which is used to employ suction in pycnometers.
(4) Balance sensitive to an accuracy of 0.01 gm, equipped with holder for weighing samples in water.
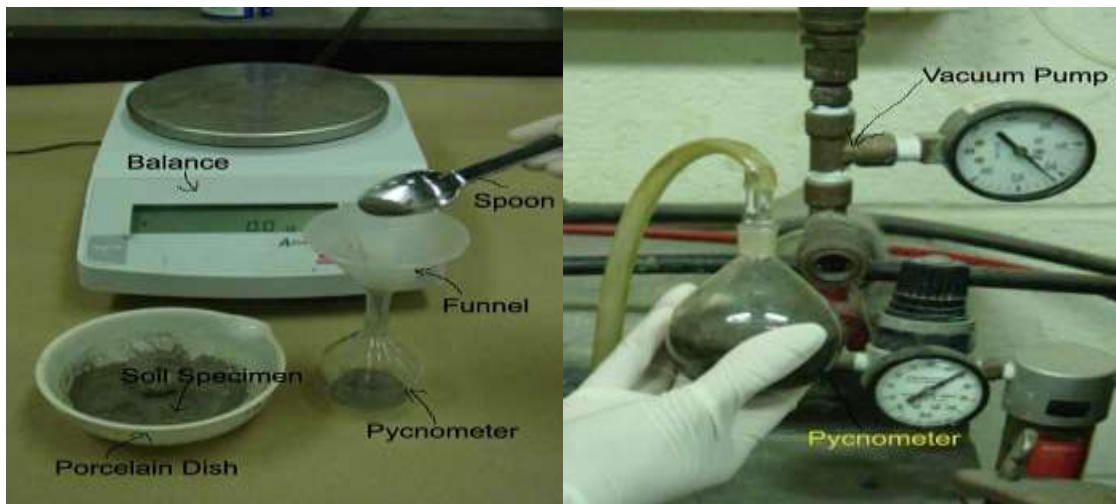(5) Funnel.
(6) Spoon.

Figure 2.02: Balance, Pycnometer and spoon.   Figure 2.03: Vacuum pump.

Procedure

(1) Secure a representative sample of soil, then wash and dry it using an oven at $105° \pm 5°$ C.
(2) Agitate the sample on 4.75-mm sieve (No. 4 sieve) and then weigh the passing soil (M_s).

(3) Fill the pycnometer with water to its capacity and then weigh it (M_{pw}).

(4) Discharge the water from pycnometer and dry it, then place the sample in the pycnometer and fill it partially with water (1/2, 3/4 of the pycnometer).

(5) Apply a partial vacuum to the contents of the pycnometer for 10 minutes, to remove the air entrapped, then stop the vacuum and carefully remove the vacuum line from the pycnometer. Fill the pycnometer to its full capacity, then clean the exterior surface of the pycnometer with a clean, dry cloth and weigh it (M_{pws}).

Data Analysis

After going over the previous section of procedure adopted for this test, calculate the specific gravity of soil using the following equation:

$$G_s = \frac{M_s}{(M_s + M_{pw}) - M_{pws}}$$

Measurements

Measurements of the testing are shown as given as under:

Table 2.02: Testing measurements.

Item	Data
M_p = Mass of empty, clean pycnometer (gram)	-
M_s = Mass of oven-dry soil / solids (gram)	0150.21
M_{pw} = Mass of pycnometer filled with water (gram)	1787.80
M_{pws} = Mass of pycnometer containing soil and water (gram)	1881.71
G_s = Specific gravity	…

* The sign (-) means that the cell has been omitted.

* The sign (…) means that the cell needs to be evaluated, for more details go to the computation section.

Computation

$$G_s = \frac{M_s}{(M_s + M_{pw}) - M_{pws}} = \frac{150.21}{150.21 + 1787.80 - 1881.71} = 2.668$$

Consequently, final results of the testing are shown as given as under:

Table 2.03: Testing results.

Item	Data
M_p = Mass of empty, clean pycnometer (gram)	-
M_s = Mass of oven-dry soil / solids (gram)	0150.21
M_{pw} = Mass of pycnometer filled with water (gram)	1787.80
M_{pws} = Mass of pycnometer containing soil and water (gram)	1881.71
G_s = Specific gravity	002.668

* The sign (-) means that the cell has been omitted.

Results and Discussion

After all calculations have been evaluated once, the results were as follows:

$G_s = 2.668$

Conclusion

- This experiment is very important since specific gravity of soil is used in the phase relationship of air, water and solids in a given volume of soil.
- The results in this experiment are convincing to an extent since they lie within the expected range of the common values ($2.65 \leq 2.668 \leq 2.7$) and the value itself indicates that the soil sample is sand.
- More accuracy can be achieved by increasing the diameter of the pycnometer used. This is due to the decreasing of surface tension effects.

TEST NO. 03

MEASUREMENT OF UNIT WEIGHT

(ASTM D7263)

Introduction

This test method covers the measurement of unit weight of soil specimens. Measurement of soil bulk densities (unit weight) is difficult if there are gravel, stones or other materials present in the soil profile. Of the many methods in hand, the following experiment describes the wax clod method, which is best suited for samples which are not easily deformed or disaggregated. However, unit weight is usually determined in laboratory by measuring the weight and volume of a relatively undisturbed soil sample obtained from a brass ring. Measuring unit weight of the soil in a field may consist of a sand cone test, rubber balloon or nuclear densitometer.

Definition

Unit weight of a soil mass is defined as the ratio of the total weight of soil to the total volume of soil. It is also known as bulk density. However, there are some definitions that a laboratory technician needs to know, these are porosity (n) and void ratio (e). Porosity of a soil is defined as the ratio of the volume of voids contained in the weight of a given soil mass to the total volume of that soil, whereas void ratio of a soil is defined as the ratio of voids volume entrapped between soil grains to the volume of soil solids.

Figure 3.01: Soil sample.
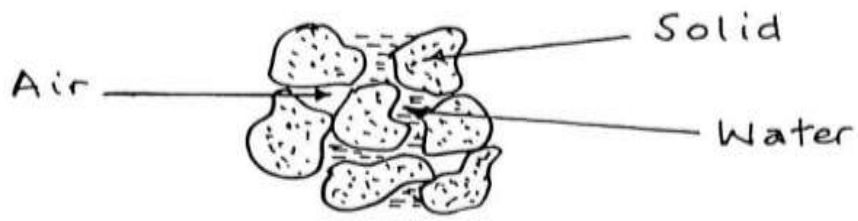
Figure 3.02: Bulk weight of a soil sample.

A technique is offered for estimating the soil bulk density in thin layers (1.0 cm) in loose, non-uniform soils with low moisture levels. The technique consists of the removal of soil in shallow layers. As each layer is removed, the hole is filled with a molten paraffin wax to obtain a casting of the excavated volume. Measured bulk densities values using this procedure compare well to results obtained with other techniques.

The technique involves the removal of soil in shallow layers. As each layer is removed, the hole is filled to the original soil level of the layer with a molten paraffin wax. After the wax has solidified, the wax casting is removed and weighed. The

volume of the wax is then calculated based on the wax specific density. The soil removed from the hole is dried and weighed.

Significance

- Unit weight is of important case in weight-volume relationships, such as porosity, void ratio, etc.
- Unit weight is an important soil characteristic which affects many biological and physical processes, such as root penetrations and water infiltration.
- Dry density, as defined as 'density of soil or rock' and 'bulk density' by soil scientists, can be used to convert the water fraction of soil from a mass basis to a volume basis and vice versa. When particle density, that is, specific gravity is also known, dry density can be used to calculate porosity and void ratio.
- Dry density measurements are also useful for determining degree of soil compaction. Since moisture content is variable, moist soil density provides little useful information except to estimate the weight of soil per unit volume.

Purpose

Measurement of the unit weight of soil using a paraffin wax. Unit weight of soil is the bulk weight (i.e. the soil weight is bulk, not dry soil) of soil sample per unit volume. Air and/or water occupy the spaces (voids and/or pores) between the soil grains which are contained in the weight. this is why called bulk weight.

Standard Reference

ASTM D7263 – Standard Test Methods for Laboratory Determination of Density (Unit Weight) of Soil Specimens.

Equipment and Materials

(1) Balance sensitive to an accuracy of 0.01 gm.

(2) Paraffin wax (of known density, $\rho = 0.9$ g/cm^3).
(3) Melting pan.
(4) Steel straightedge.
(5) Container (can).

Figure 2.03: Sensitive balance.   **Figure 2.04: Steel straightedges.**

Figure 3.05: Paraffin wax.   **Figure 3.06: Container (can).**

Procedure

(1) Secure a sample of soil from the field, where soil is to be tested.
Note: This test differs from previous tests since the soil mass is bulk, not dry soil, which means the moisture content is contained within the weight of sample.

(2) Trim a sample to the appropriate size by means of a straightedge; usually about a rounded cube of about 70 mm in diameter. Specimens should be reasonably smooth and rounded.

(3) Weigh the sample prepared in air (M_s).

(4) Immerse the sample in melted wax and then weigh it while it is covered with wax in air ($M_{sw/air}$).

(5) Submerge the sample covered with wax in water and then weigh it ($M_{sw/water}$).

Data Analysis

After going over the previous section of procedure adopted for this test, calculate the unit weight of soil using the following equations:

$$\gamma = \frac{W_s}{V_s}$$

$$W_s = M_s \times \text{gravity, g}$$

$$V_s = V_{sw} - V_w$$

$$V_{sw} = M_{sw/air} - M_{sw/water}$$

$$V_w = \frac{M_w}{\rho_w}$$

$$M_w = M_{sw/air} - M_s$$

ORIGINAL WAX CASTING   WAX CASTING WITH BEAD   SHAVED WAX BEAD CASTING

Figure 3.07: Schematic diagram of wax casting.

Measurements

Measurements of the testing are shown as given as under:

Table 3.01: Testing measurements.

Item	Data for Sample 01
M_s = Bulk mass of soil sample (gram)	253.20
$M_{sw/air}$ = Mass of soil sampled covered with wax in air (gram)	263.80
$M_{sw/water}$ = Mass of soil sampled covered with wax in water (gram)	127.90
ρ_w = Density of paraffin wax (gram/cubic centimetre)	000.90
W_s = Bulk weight of soil sample (newton)	…
M_w = Mass of paraffin wax (gram)	…
V_w = Volume of paraffin wax (cubic metre)	…
V_s = Volume of soil sample (cubic metre)	…
V_{sw} = Volume of soil covered with wax (cubic metre)	…
γ = Soil unit weight (kilonewton per cubic metre)	…

* The sign (…) means that the cell needs to be evaluated, for more details go to the computation section.

Computation

$$W_s = M_s \times g = \frac{253.20}{1000} \times 9.81 = 2.4839 \text{ N}$$

$$V_{sw} = M_{sw/air} - M_{sw/water} = 263.80 - 127.90 = 135.90 \text{ cm}^3$$

$$M_w = M_{sw/air} - M_s = 263.80 - 253.20 = 10.60 \text{ gm}$$

$$V_w = \frac{M_w}{\rho_w} = \frac{10.60}{0.90} = 11.78 \text{ cm}^3$$

$$V_s = V_{sw} - V_w = 135.90 - 11.78 = 124.12 \text{ cm}^3$$

$$\gamma = \frac{W_w}{V_s} = \frac{2.4839 \text{ (N)} \times \frac{1 \text{ (kN)}}{1000 \text{ (N)}}}{124.12 \text{ (cm}^3) \times \frac{1 \text{ (m}^3)}{100^3 \text{ (cm}^3)}} = 20.012 \text{ kN/m}^3$$

Consequently, final results of the testing are shown as given as under:

Table 3.02: Testing results.

Item	Data for Sample 01
M_s = Bulk mass of soil sample (gram)	253.20
$M_{sw/air}$ = Mass of soil sampled covered with wax in air (gram)	263.80
$M_{sw/water}$ = Mass of soil sampled covered with wax in water (gram)	127.90
W_s = Bulk weight of soil sample (newton)	2.4839
M_w = Mass of paraffin wax (gram)	010.60
ρ_w = Density of paraffin wax (gram/cubic centimetre)	000.90
V_s = Volume of soil sample (cubic metre)	124.12
V_w = Volume of paraffin wax (cubic metre)	011.78
V_{sw} = Volume of soil covered with wax (cubic metre)	135.90
γ = Soil unit weight (kilonewton per cubic metre)	20.012

Results and Discussion

After all calculations have been evaluated once, the results were as follows:

$$\gamma = 20.012 \text{ kN/m}^3$$

Conclusion

- The result in this experiment is convincing to an extent since obtaining such unit weight indicates that soil is heavy.
- The wax casting technique is an effective way of obtaining a measure of soil bulk density in loose and dry soils where other techniques are limited or ineffective.
- The wax technique is also suitable for obtaining bulk density values at micro locations within a soil profile. This is important in applications, such as evaluating the effect of soil layering on water infiltrations or plant root penetration.

- This experiment is very important since the unit weight of soil is used in the phase relationship of air, water and solids in a given volume of soil.

TEST NO. 04

MECHANICAL SIEVE ANALYSIS

(ASTM D422)

Introduction

This test method covers the quantitative determination of the distribution of particle sizes in soils. Mechanical analysis is the determination of the size range of particles present in a soil, expressed as a percentage of the total dry weight. Two methods generally are used to find the particle-size distribution of soil: (1) the distribution of particle sizes larger than 75 µm (0.075 mm or retained on No. 200 sieve) is determined by sieving, while (2) the distribution of particle sizes smaller than 75 µm (0.075 mm or passing on No. 200 sieve) is determined by elutriation (sedimentation technique), using a hydrometer to secure the necessary data. However, the mechanical sieve analysis is performed in this experiment in order to determine the distribution of the coarser, larger-sized particles.

Definition

Sieve analysis consists of shaking a soil sample through a set of sieves that have progressively smaller openings and the process of shaking, for overall, is called sieving.

Sieves are named in two different ways, which come as follows:

(1) Opening size (longitude of sieve opening).
(2) Number of squares are there in a one inch.

U.S. standard sieve numbers and the sizes of openings are given in Table 4.01.

Table 4.01: U.S. standard sieves sizes.

Sieve No.	Opening (mm)	Sieve No.	Opening (mm)
4	4.75	35	0.500
5	4.00	40	0.425
6	3.35	50	0.355
7	2.80	60	0.250
8	2.36	70	0.212
10	2.00	80	0.180
12	1.70	100	0.150
14	1.40	120	0.125
16	1.18	140	0.106
18	1.00	170	0.090
20	0.85	200	0.075
25	0.71	270	0.053
30	0.60		

Particle-size distribution of soils may be one of the following three different forms:

(1) Well graded soil.
(2) Gap-graded soil.
(3) Poorly graded soil.
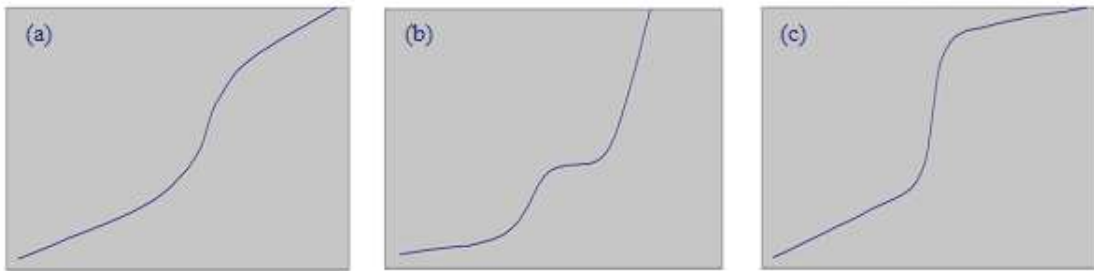
Figure 4.01: (a) Well graded soil; (b) Gap-graded soil; (c) Poorly graded soil.

Once the percent finer for each sieve is calculated, the calculations are plotted on semi-logarithmic graph paper with percent finer as the ordinate (arithmetic scale) and sieve opening size as the abscissa (logarithmic scale). This plot is referred to as the particle-size distribution curve.

Significance

- The distribution of different grain sizes affects soil engineering properties.
- Grain-size analysis provides the grain-size distribution curve, which is required in soil classification. The grain-size analysis is, widely, used in different specifications of classification of soils, for instance, USCS and AASHTO.
- The data obtained from grain-size distribution curves are used in the design of filters (filter materials) for earth dams, as well, to determine the suitability of a soil for road construction, air field, etc.
- Information obtained from grain-size analysis can be used to predict soil water movement, although, permeability tests are more generally used.

Purpose

Determination of the percentage of different grain sizes contained within a soil. The mechanical sieve analysis is performed to determine the distribution of the coarser, larger-sized particles.

Standard Reference

ASTM D422 – Standard Test Method for Particle-Size Analysis of Soils.

Equipment and Materials

(1) Stack of sieves including a pan at the bottom and cover (cap) at the top.
(2) Sieves used are: No. 4, 10, 16, 30, 40, 50, 100 and 200.
(3) Balance with an accuracy to 0.01 gram.
(4) Rubber pestle and mortar (for crushing the soil if lumped or conglomerated).
(5) Mechanical sieve shaker, if any (shaking by hands could be employed).
(6) Drying oven.

Figure 4.02: Sensitive balance.   **Figure 4.03: Set of sieves.**   **Figure 4.04: Mechanical shaker.**

Procedure

(1) Prepare a representative sample of soil from the field, where soil is to be tested.
(2) If soil particles are lumped or conglomerated, the laboratory technician should crush the lumped, not the particles using a pestle and mortar.
(3) Accurately, measure the mass of soil sample in grams (m_t).
(4) Prepare a stack of sieves. Sieves having larger opening sizes (i.e. lower numbers) are placed above the ones having smaller opening sizes (i.e. higher numbers). The very last sieve is No. 200 sieve and a pan is placed under it to collect the portion of soil passing No. 200 sieve (No. 4 and No. 200 sieves should be always included).

(5) Carefully, pour the soil sample onto the top sieve and place the cap over it.

(6) Place the stack of sieves in the mechanical shaker and shake for 10 minutes (shaking by hands can be appropriate).

(7) Remove the stack of sieves from the shaker and, carefully, weigh and record the weight of each sieve containing its retained soil. At the end, weigh and record the weight of the bottom pan containing 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 weight of the sieve containing its retained soil and record this mass as the weight retained onto the data sheet supported. The sum of these retained masses should be approximately equals the initial mass of the soil sample used in the experiment. 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 (finer) by starting with 100 percent and subtracting the percent retained on each sieve as a cumulative procedure.

Measurements

Measurements of the testing are shown as given as under:

Total weight of the sample (m_t) = 67.60 gram.

Table 4.02: Testing measurements.

Sieve No.	Opening (mm)	Weight Retained (gm)	Cumulative Retained (gm)
4	4.750	00.00	00.00
10	2.000	00.30	00.30
16	1.180	00.30	00.60
30	0.600	00.60	01.20
40	0.425	02.10	03.30
50	0.300	05.20	08.50
100	0.150	10.50	19.00
200	0.075	05.50	24.50
Pan	0.000	00.00	24.50

Computation

m_t = 67.60 gm

Table 4.03: Testing results.

Sieve No.	Opening (mm)	Weight Retained (gm)	Cumulative Weight Retained (gm)	Percent Cumulative Weight Retained	Percent Passing (Finer)
4	4.750	00.00	00.00	00.00	100.00
10	2.000	00.30	00.30	00.44	099.56
16	1.180	00.30	00.60	00.89	099.11
30	0.600	00.60	01.20	01.78	098.22
40	0.425	02.10	03.30	04.88	095.12
50	0.300	05.20	08.50	12.57	087.43
100	0.150	10.50	19.00	28.11	071.89
200	0.075	05.50	24.50	36.24	063.76

Worked example: For sieve No. 40 (0.425-mm size)

$$\text{Percent Cumulative Retained} = \frac{3.30}{67.60} \times 100 = 4.88 \%$$

$$\text{Percent Cumulative Passing} = 100 - 4.88 = 95.12 \%$$

Results and Discussion

Results of this test and discussion, thereafter, are combined, together, with those of the mechanical hydrometer analysis and, hence, developing a complete particle-size distribution curve.

Conclusion

Conclusion and comments made on the experiment are included with those of the mechanical hydrometer analysis.

TEST NO. 05

MECHANICAL HYDROMETER ANALYSIS

(ASTM D2216)

Introduction

This test method covers the quantitative determination of the distribution of particle sizes in soils. This test is performed to determine the percentage of different grain sizes contained within a soil. The hydrometer method is used to determine the distribution of the finer particles. The hydrometer analysis is the procedure generally adopted for determining the particle-size distribution in a soil for the fraction that is finer than No. 200 sieve (0.075-mm) size. The lower limit of the particle size determined by this procedure is about 0.001 mm.

Definition

In hydrometer analysis, a soil specimen is dispersed in water. In a dispersed state in water, the soil particles will settle, individually. It is assumed that, the soil particles are spheres and the velocity of the particles can be given by Stokes' law.

Stokes' law

A sphere falling freely through a liquid of infinite extent will accelerate, rapidly, to a certain maximum velocity and will continue at that velocity, as long as conditions remain the same. The relationship of the terminal velocity to the physical properties of the sphere and the liquid are expressed by Stokes' equation as follows:

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

or

$$v = \frac{\rho_s - \rho_w}{18\eta} D^2$$

$$D = \sqrt{\frac{18\,\eta\,v}{\rho_s - \rho_w}} = \sqrt{\frac{18\,\eta}{\rho_s - \rho_w}}\sqrt{\frac{L}{T}}$$

$$\rho_s = G_s \rho_w$$

$$D = \sqrt{\frac{18\,\eta}{G_s\rho_w - \rho_w}}\sqrt{\frac{L}{T}}$$

$$D = \sqrt{\frac{18\,\eta}{\rho_w(G_s - 1)}}\sqrt{\frac{L}{T}}$$

$$cm = \frac{mm}{10} = \sqrt{\frac{18\left(\frac{gm.\,sec}{cm^2}\right)}{\frac{gm}{cm^3}}}\sqrt{\frac{cm}{min \times 60}}$$

Where:

v = velocity of sedimintation.

γ_s/ρ_s = densitys of soil solids.

γ_w/ρ_w = densitys of water.

η = viscosity of water.

D = diameter of soil particles.

Stokes' law is applicable to spheres varying from 0.02 mm to 0.0002 mm in diameter. It can be applied to soil particles falling through water.

In the test procedure described here, the ASTM 152H type hydrometer will be used (see Figure 5.01). If a hydrometer is suspended in water in which soil is dispersed (see Figure 5.02), it will measure the specific gravity of the soil-water suspension at a depth L. The depth L is called effective depth. So, at a time t minutes from the beginning of the test, the soil particles that settle beyond the zone of measurement (i.e. beyond the effective depth L) will have a diameter given by:

$$D \text{ (mm)} = K \sqrt{\frac{L \text{ (cm)}}{T \text{ (min)}}}$$
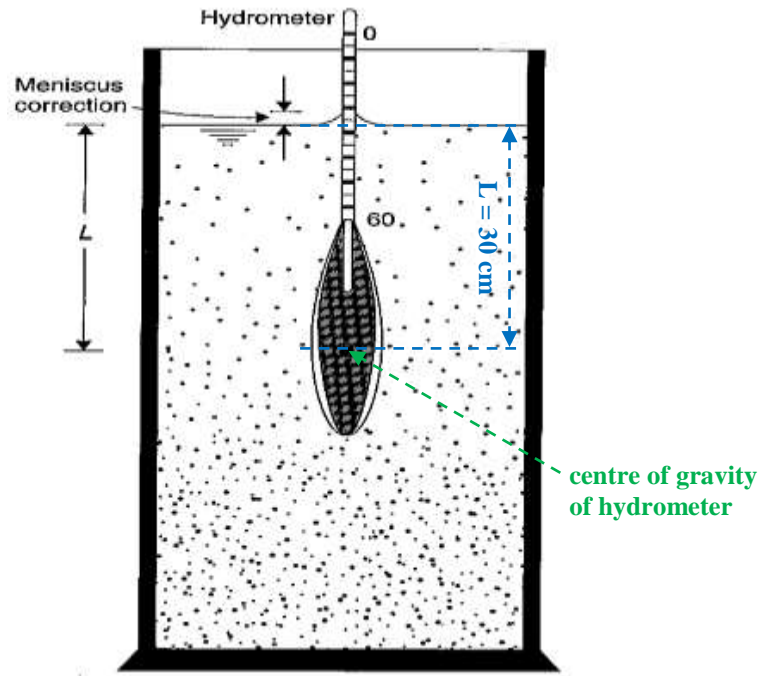
Figure 5.01: Hydrometer suspended in water in which soil is dispersed.
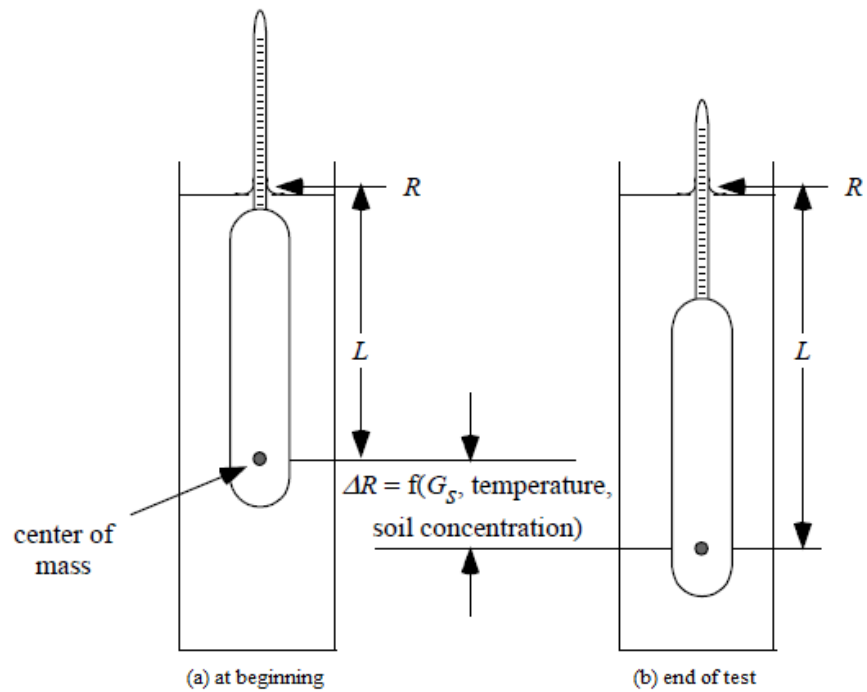
Figure 5.02: Graduate and hydrometer: (a) at beginning; (b) end of test.

Significance

- The resulting grain-size distribution curve can be used to characterize the soil and used to reject or accept the material for engineering applications.
- The practical applications of the particle-size analysis and geotechnical engineering soil description, quantitative soil classification and correlations to permeability based on the Hazen's equation.

Purpose

Determination of the percentage of different grain sizes in a soil that consists of particles sizes smaller than No. 200 (0.075-mm) sieve, which are silt and clay.

Standard Reference

ASTM D422 – Standard Test Method for Particle-Size Analysis of Soils.

Equipment and Materials

(1) Balance sensitive to an accuracy of 0.01 gm.
(2) Soil hydrometer (ASTM 152H hydrometer type is preferred).
(3) Control cylinder.
(4) Sedimentation cylinder.
(5) No. 200 sieve.
(6) Thermometer with a clamp (0.5 increments in Celsius).
(7) Beaker.
(8) Mixer.
(9) Dispersing agent (Calgon 'sodium hexa-metaphosphate ($NaPO_3$)').
(10) Stopwatch (Interval timer).

Figure 5.03: Shaker.   **Figure 5.04: 152H Hydrometer.**   **Figure 5.05: Thermometer.**

Figure 5.06: Control cylinder and sedimentation cylinder.

Figure 5.07: Meniscus correction to the actual hydrometer reading.
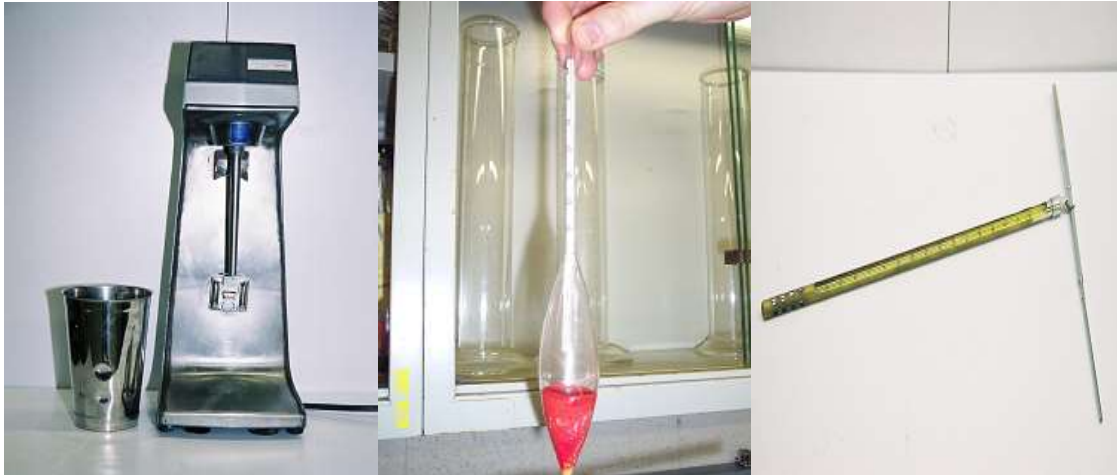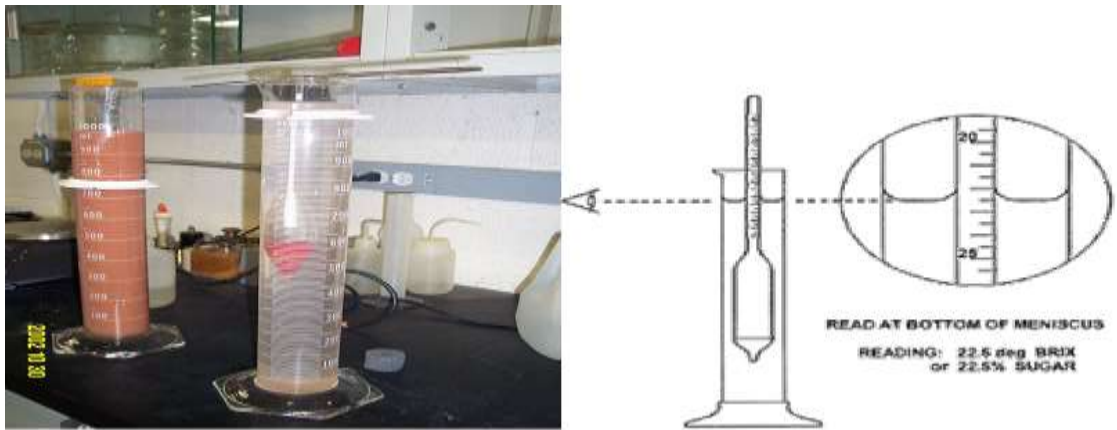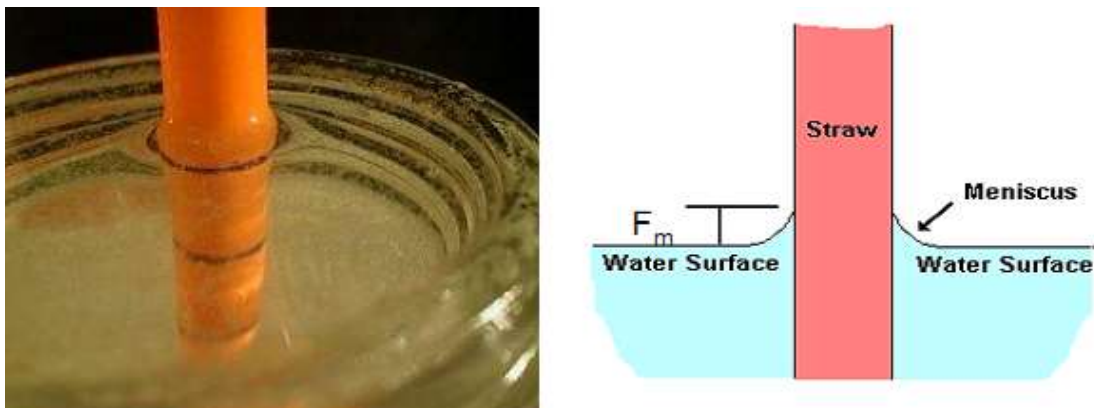
Procedure

(1) Take 50-gm mass from the fine soil that passed No. 200 sieve.

(2) Prepare two graduated cylinders (vessels), put in each one 125 ml of water and 5 gm (4% of 125 ml of water) of a dispersing agent, namely Calgon 'sodium hexa-metaphosphate' and stir the mixture.

(3) Put the 50-gm fine soil in one of the two cylinders and stir the mixture, let the soil soak for one hour.

(4) Transfer water and Calgon from the second cylinder into the control jar. Then add the distilled water up to the (1000-ml) mark and then 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 a way such that the contents are mixed, thoroughly. Insert the hydrometer and thermometer into the control cylinder and note the zero correction and temperature, respectively.

(5) Transfer the soil slurry from the first cylinder 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 3 - 5 minutes.

(6) Immediately, transfer the soil slurry into the empty sedimentation jar. Add distilled water up to the (1000-ml) mark.

(7) 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.

(8) 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 hydrometer should be made as close to the reading depth as possible in order to avoid excessive bobbing.

(9) The reading is taken by observing the top of the meniscus formed by the suspension and 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.

(10) Take the hydrometer and temperature readings at times 1, 2, 3, 4 minutes, if there is a difference of 1 between these readings and the previous four readings, continue to take readings for 8, 16, 30, 60 minutes and then for 2, 4, 8, 16, 24 hours.

Data Analysis

(1) Apply meniscus correction to the actual hydrometer reading, which is usually done by adding a unit to the original hydrometer reading.

$$(R_{corrected})_{meniscus} = R_{actual} + 1.0$$

(2) From Table 5.01, obtain the effective hydrometer depth L in cm (for meniscus corrected reading) for known specific gravity of soil. If not known, assume the value of 2.65 for the specific gravity for laboratory testing purposes and obtain the value of K from Table 5.02.

(3) Calculate the equivalent particle diameter using the following equation:

$$D \text{ (mm)} = K\sqrt{\frac{L \text{ (cm)}}{T \text{ (min)}}}$$

(4) Determine the temperature correction (C_T) from Table 5.03.

(5) Determine correction factor a from Table 5.04 using the value of specific gravity.

(6) Calculate the corrected hydrometer reading as following:

$$R_c = R_{actual} - \text{Zero Correction} + C_T$$

(7) Calculate the percent finer as following:

$$P = \frac{R_c \times a}{W_s} \times 100$$

Where:

W_s = weight of the soil sample in grams.

(8) Adjusted percent finer as following:

$$P_A = \frac{P \times F_{200}}{100}$$

Where:

F_{200} = percent finer then No. 200 sieve.

(9) Plot the particle size D versus the corresponding adjusted percent finer on a semi-logarithmic graph; to develop the ogive (S-shaped curve).

Table 5.01: Variation of L with Hydrometer reading ─ ASTM 152H Hydrometer.

Hydrometer Reading, R	L (cm)	Hydrometer Reading, R	L (cm)
0	16.3	31	11.2
1	16.1	32	11.1
2	16.0	33	10.9
3	15.8	34	10.7
4	15.6	35	10.6
5	15.5	36	10.4
6	15.3	37	10.2
7	15.2	38	10.1
8	15.0	39	09.9
9	14.8	40	09.7
10	14.7	41	09.6
11	14.5	42	09.4
12	14.3	43	09.2
13	14.2	44	09.1
14	14.0	45	08.9
15	13.8	46	08.8
16	13.7	47	08.6
17	13.5	48	08.4
18	13.3	49	08.3
19	13.2	50	08.1
20	13.0	51	07.9
21	12.9	52	07.8
22	12.7	53	07.6
23	12.5	54	07.4
24	12.4	55	07.3
25	12.2	56	07.1
26	12.0	57	07.0
27	11.9	58	06.8
28	11.7	59	06.6
29	11.5	60	06.5
30	11.4		

Table 5.02: Variation of K with the test temperature and the specific gravity of soil solids.

Temperature, T (°C)	Specific Gravity of Soil Solids, G_s							
	2.50	2.55	2.60	2.65	2.70	2.75	2.80	2.85
16	0.0151	0.0148	0.0146	0.0144	0.0141	0.0139	0.0139	0.0136
17	0.0149	0.0100	0.0100	0.0100	0.0100	0.0100	0.0100	0.0100
18	0.0148	0.0144	0.0142	0.0140	0.0138	0.0136	0.0134	0.0132
19	0.0145	0.0143	0.0140	0.0138	0.0136	0.0134	0.0132	0.0131
20	0.0143	0.0141	0.0139	0.0137	0.0134	0.0133	0.0131	0.0129
21	0.0141	0.0139	0.0137	0.0135	0.0133	0.0131	0.0129	0.0127
22	0.0140	0.0137	0.0135	0.0133	0.0131	0.0129	0.0128	0.0126
23	0.0138	0.0136	0.0134	0.0132	0.0130	0.0128	0.0126	0.0124
24	0.0137	0.0134	0.0132	0.013	0.0128	0.0126	0.0125	0.0123
25	0.0135	0.0133	0.0131	0.0129	0.0127	0.0125	0.0123	0.0122
26	0.0133	0.0131	0.0129	0.0127	0.0125	0.0124	0.0122	0.0120
27	0.0132	0.0130	0.0128	0.0126	0.0124	0.0122	0.0120	0.0119
28	0.0130	0.0128	0.0126	0.0124	0.0123	0.0121	0.0119	0.0117
29	0.0129	0.0127	0.0125	0.0123	0.0121	0.0120	0.0118	0.0116
30	0.0128	0.0126	0.0124	0.0122	0.0120	0.0118	0.0117	0.0115

Table 5.03: Variation of C_T with the test temperature.

Temperature, T (°C)	Factor, C_T
15	+1.1
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 5.04: Variation of correction factor a with unit weight of soil solids.

Unit Weight of Soil Solids (gm/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
2.85	0.96

Measurements

Measurements of the testing are shown as given as under:

Table 5.05: Testing measurements and results.

Item	Data
M_s = Mass of soil sample (gram)	50.0
G_s = Specific gravity of soil solids	2.60
Dispersing Agent	Sodium Hex-metaphosphate (Calgon)
Zero correction	+8.0
Meniscus correction	+1.0

Table 5.06: Testing measurements and results.

Date	Time	Elapsed Time (min)	Temperature (°C)	Hydrometer Reading, Ra
23.02.2014	4:22 PM	0.000	20	-
23.02.2014	4:23 PM	1.000	20	47
23.02.2014	4:24 PM	2.000	21	42
23.02.2014	4:26 PM	4.000	20	37
23.02.2014	4:30 PM	8.000	20	32
23.02.2014	4:38 PM	16.00	20	29
23.02.2014	4:52 PM	30.00	21	26
23.02.2014	5:22 PM	60.00	20	24
23.02.2014	6:52 PM	150.0	22	21
24.02.2014	4:22 PM	1440	20	20
25.02.2014	4:22 AM	2160	21	19

Computation

Table 5.07: Testing calculations and results.

Elapsed Time (min)	Temperature (°C)	Hydrometer Reading, R_{actual}	Hydrometer Corrected for Meniscus = $R_{actual} + 1.0$	L from Table 5.01 (cm)	K from Table 5.02 = f (G_s, T°)	$D(mm) = K\sqrt{\dfrac{L\,(cm)}{T\,(min)}}$	C_T from Table 5.03 = f (T°)	$R_C = R_{actual} -$ Zero Correction $+ C_T$	a from Table 5.04 = $f(G_s)$	Percent Finer (%P)	Adjusted Percent Finer (%P_A)
0.000	20	-	-	-	0.0144	-	0.0	-	1.01	-	-
1.000	20	47	48	08.4	0.0142	0.04116	0.0	39.0	1.01	0.78780	0.50228
2.000	21	42	43	09.2	0.0137	0.02938	0.2	34.2	1.01	0.69084	0.44046
4.000	20	37	38	10.1	0.0144	0.02288	0.0	29.0	1.01	0.58580	0.37349
8.000	20	32	33	10.9	0.0144	0.01681	0.0	24.0	1.01	0.48480	0.30910
16.00	20	29	30	11.4	0.0144	0.01215	0.0	21.0	1.01	0.42420	0.27046
30.00	21	26	27	11.9	0.0137	0.00863	0.2	18.2	1.01	0.36764	0.23440
60.00	20	24	25	12.2	0.0144	0.00649	0.0	16.0	1.01	0.32320	0.20606
150.0	22	21	22	12.7	0.0135	0.00393	0.4	13.4	1.01	0.27068	0.17258
1440	20	20	21	12.9	0.0144	0.00136	0.0	12.0	1.01	0.24240	0.15455
2160	21	19	20	13.0	0.0137	0.00106	0.2	11.2	1.01	0.22624	0.14424

Charts and Analysis

Table 5.08: Testing measurements and results.

Sieve Opening (mm)	Percent Passing (Finer)
4.75000	100.0000
2.00000	99.56000
1.18000	99.11000
0.60000	98.22000
0.42500	95.12000
0.30000	87.43000
0.15000	71.89000
0.07500	63.76000
0.04116	00.50228
0.02938	00.44046
0.02288	00.37349
0.01681	00.30910
0.01215	00.27046
0.00863	00.23440
0.00649	00.20606
0.00393	00.17258
0.00136	00.15455
0.00106	00.14424

Grain-Size Distribution Ogive
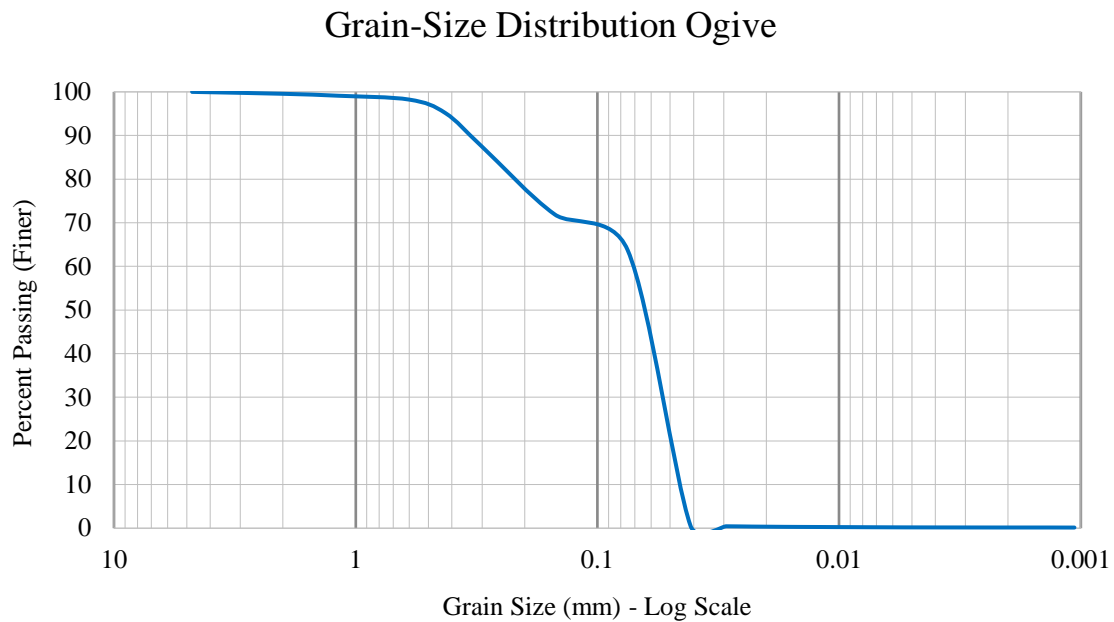
Figure 5.08: Grain-size distribution Ogive.

From the graph drawn above and according to USCS specification:

$\%Gravels = \%passing\ (76.2\ mm) - \%passing\ (4.75\ mm)$

$$= 100 - 100 = 0.00\ \%$$

$\%Sands = \%passing\ (4.75\ mm) - \%passing\ (0.075\ mm)$

$$= 100 - 63.79 = 36.24\ \%$$

$\%Fines = 100 - \%Gravels - \%Sands$

$$= 100 - 0.00 - 36.24 = 63.76\ \%$$

Further, the effective sizes are:

$D_{10} = 0.045$

$D_{30} = 0.055$

$$D_{60} = 0.07$$

Where:

D_{10} = the corresponding diameter for 10% passing.

D_{30} = the corresponding diameter for 30% passing.

D_{60} = the corresponding diameter for 60% passing.

Furthermore, the following parameters are significant:

$$C_u = \frac{D_{60}}{D_{10}} = \frac{0.07}{0.045} = 1.56$$

Where:

C_u = uniformity coefficient.

$$C_z = \frac{D_{30}^2}{D_{10} \times D_{60}} = \frac{(0.055)^2}{0.045 \times 0.07} = 0.96$$

Where:

C_z = coefficient of gradation.

Results and Discussion

After all calculations have been evaluated once, the results were as follows:

Constituting gradients:

%Gravels = 0.00 %

%Sands = 36.24 %

% Fines = 63.76 %

Effective sizes:

$$D_{10} = 0.045$$

$D_{30} = 0.055$

$D_{60} = 0.07$

Significant parameters:

$C_u = 1.56$

$C_z = 0.96$

Conclusion

- The total weight of the sample was 67.60 gram and the weight of the retained on the sieve No. 200 after washing the sample on it was 24.50 gram.
- The fine material available (passed from No. 200 sieve) accounts for 63.76% and granular material available (retained on No. 200 sieve) accounts for 36.24%.
- There are no gravels in the sample since percent gravel is equal to zero.
- The soil is fine-grained soil hence materials passed from No. 200 sieve is 63.76% which is greater than 50% since as per USCS if $F_{200} \geq 50\%$ then soil is fine-grained.

TEST NO. 06

MEASUREMENT OF CONSISTENCY LIMITS

(ASTM D4318)

Introduction

These test methods cover the determination of the consistency limits (e.g. liquid, plastic and shrinkage limits) and the plasticity index of soils. a Swedish scientist named Atterberg developed a method to describe the consistency of fine-grained soils with varying moisture. The Atterberg limits are a basic measure of the critical water contents of a fine-grained soil that passed No. 40 (0.425-mm) sieve. When clay minerals are present in fine-grained soil, soil can be remoulded in the presence of some moisture without crumbling. This cohesive nature is caused by the adsorbed water surrounding the clay particles. In the early 1900s, a Swedish scientist named Atterberg developed a method to describe the consistency of fine-grained soils with varying moisture contents.

Definition

At a very low moisture content, soil behaves more like a solid. When the moisture content is very high, the soil and water may flow like a liquid. Hence, on an arbitrary basis, depending on the moisture content, the behaviour of soil can be divided into four basic states:

(1) Solid.
(2) Semisolid.
(3) Plastic.
(4) Liquid.

The liquid and plastic limit are water contents at which the mechanical properties of soil changes. They are applicable to fine-grained soils and are performed on soil fractions that pass the No. 40 (0.425-mm) sieve. Plastic limit (PL) and liquid limit (LL) are depicted in Figure 6.01. The difference between the PL and the LL is defined as the plasticity index (PI).

In Figure 6.01, the volume of fine-grained soil increases with increasing water content (wc). This indicates that plasticity index (PI) is an indicator of the swell potential of a cohesive soil. Certain clay minerals, including bentonite, montmorillonite and smectite, have a high cation exchange capacity, so their ability to hold water molecules and electrically bind them to their surface is greater. Therefore, they can exist in a plastic state over a relatively wide range of wc and soil volume and have a high swell potential.

A third value called the shrinkage limit (SL) is also depicted in Figure 6.01. Shrinkage limit is the water content at which the volume of soil begins to change as a result of a change in wc. The three parameters (SL, PL and LL) are collectively referred to as the Atterberg limits. Shrinkage limit is measured using a separate standard which is ASTM D427. However, shrinkage limit is not commonly specified in earthwork construction and laboratory shrinkage limit testing includes the handling of mercury, which is not desirable for health and safety purposes. Therefore, the scope of this laboratory includes only plastic limit and liquid limit testing.
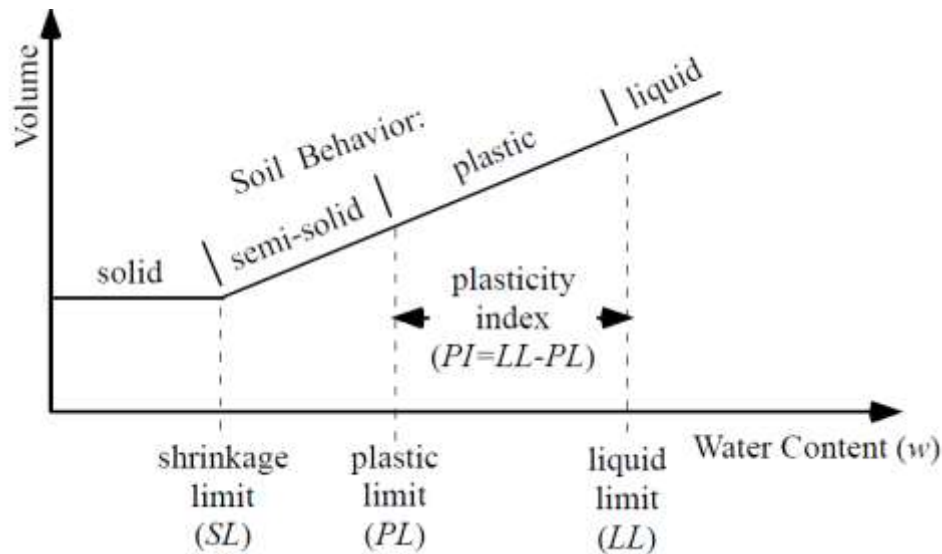
Figure 6.01: Relationship between volume and water content in fine-grained soil.

Repeatedly, the moisture content, in percent, at which the transition from solid to semisolid state takes place, is defined as the shrinkage limit. The moisture content at the point of transition from semisolid to plastic state is the plastic limit, and from plastic to liquid state is the liquid limit. These parameters are also known as Atterberg limits. This experiment discusses the procedure for determining the Atterberg limits.
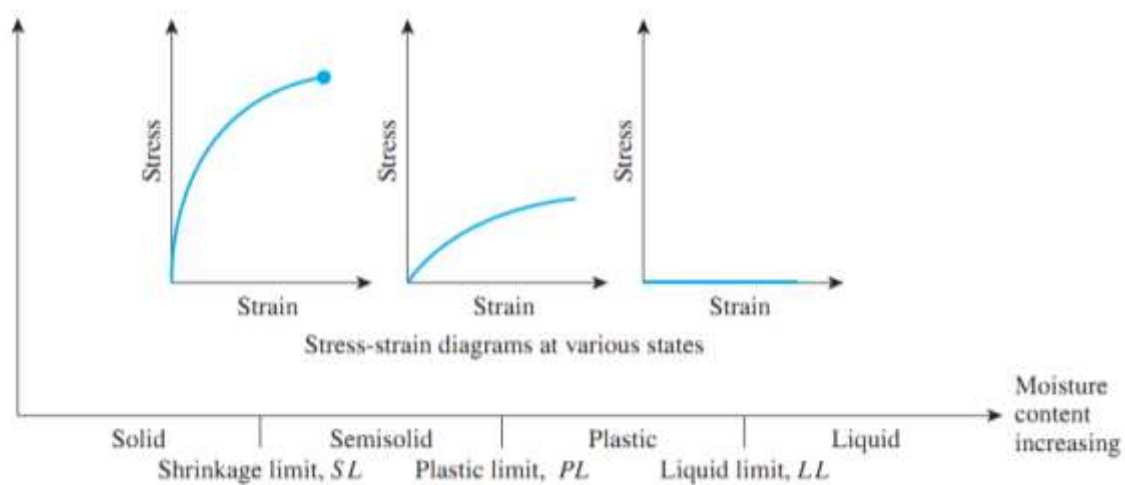
Figure 6.02: Atterberg limits.

Two main types of test are specified: (a) Casagrande apparatus method, which has been used for many years as a basis for soil classification and (b) Cone penetrometer method, which is more satisfactory, this method is based on the measurement of penetration into the soil of a standardized cone of specified mass. At the liquid limit the cone penetration is 20 mm.

Atterberg limits: The word Atterberg is related to the Swedish scientist Albert Atterberg and the limits are to describe the consistency of fine grained soils with arbitrary varying of water content.

Liquid Limit (LL) 'Theoretical': The moisture content in percent at the point of transition from plastic state to liquid state.

Liquid Limit (LL) 'Laboratory as per BS': The moisture content at which a standard cone with angle 30° and weight of 0.78 N (79.5 gm) will penetrate a distance d = 20 mm in 5 seconds when allowed to drop from a position of point contact from a single test.
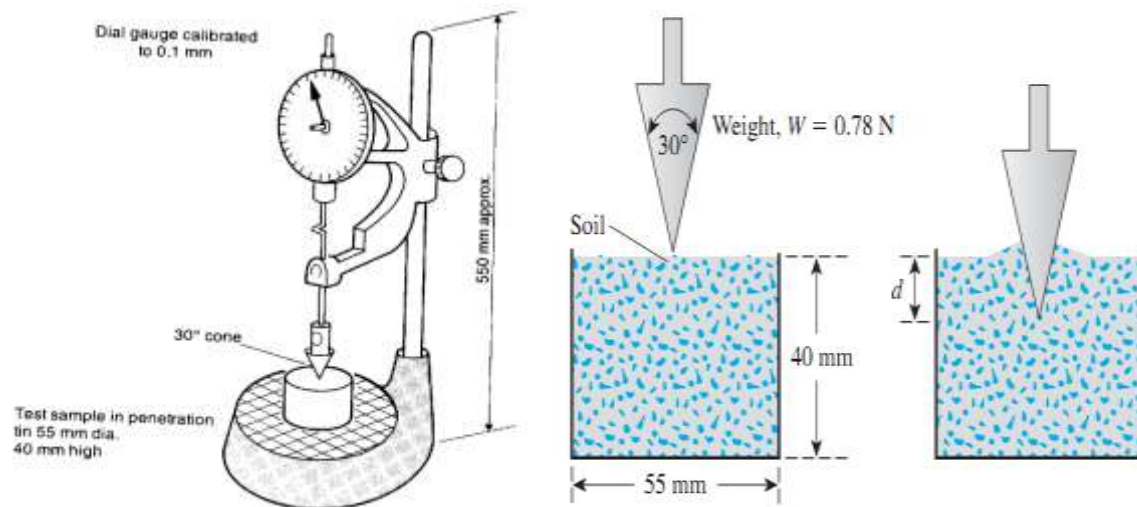
Figure 6.03: Fall cone test.

Plastic Limit (PL) 'Theoretical': The moisture content in percent at the point of transition from semisolid state to plastic state.

Plastic Limit (PL) 'Laboratory according to ASTM': The moisture content in percent, at which the soil crumbles when rolled into threads of 3.175 mm (1/8 in.) in diameter.

Plasticity index (PI): The difference between the liquid limit and the plastic limit of a soil.

$$PI = LL - PL$$

Note: The plasticity index is important in classifying fine-grained soils.

Shrinkage Limit (SL): The moisture content in percent at the point of transition from solid state to semisolid state without changing in soil volume.

$$SL = wc_i(\%) - \Delta wc(\%)$$

$$SL = \left(\frac{M_1 - M_2}{M_2}\right)(100) - \left(\frac{V_i - V_f}{M_2}\right)(\rho_w)(100)$$
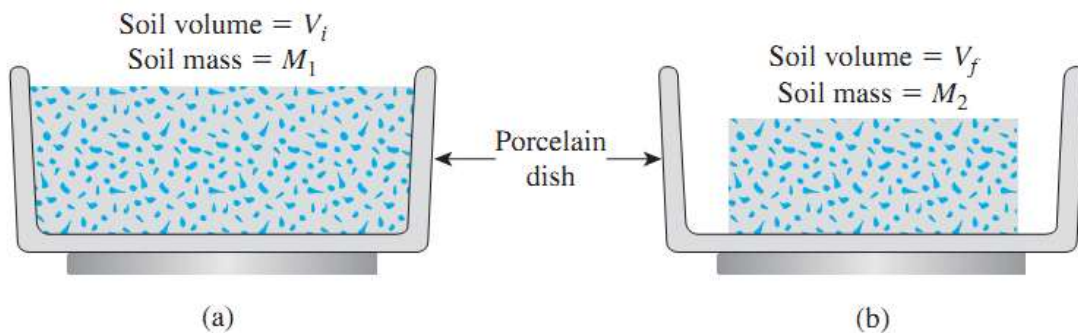
Figure 6.04: Shrinkage limit test: (a) soil pat before drying; (b) soil pat after drying.

Note: SL test is not performed in this experiment.

Significance

- Liquid limit and plasticity index are required parameters for classification of soils. Different soils have varying liquid limits. Also, one must use the plastic limit to determine its plasticity index.

- Liquid limit, plastic limit, and shrinkage limit tests of fine-grained soil are indicators of the nature of its plasticity.

Purpose

Determination of the liquid and plastic limit of a soil and as a result of the test to determine the plasticity index as well.

Standard Reference

ASTM D4318 – Standard Test Methods for Liquid Limit, Plastic Limit and Plasticity Index of Soils.

ASTM D4943 – Standard Test Method for Shrinkage Factors of Soils by the Wax Method.

BS 1377 : Part 2 : 1990 – This British Standard, a Part of the BS 1377 series, specifies methods of test for the classification of soil and for the determination of basic physical properties.

Equipment and Materials

Liquid Limit Test

(1) Penetrometer apparatus (a cone of apex angle 30 and mass of 0.78 N $\cong$ 79.5 gm).
(2) Metal cups (of 55-mm width and 40-mm depth).
(3) Moisture content apparatus (oven dry, sensitive balance and cans).
(4) Metal straight edge.
(5) Two palette knives.

Figure 6.05: Penetrometer apparatus.   **Figure 6.06: Palette knives.**

Figure 5.06: Moisture content apparatus.   **Figure 5.07: Sensitive balance.**

Plastic Limit Test

(1) A flat glass plate on which soil is to be mixed and rolled.

(2) Two palette knives.

(3) Apparatus for moisture content determination (oven dry, sensitive balance and cans).

Figure 6.09: Palette knives (two knives).   Figure 6.10: A flat glass plate.

Procedure

Liquid Limit Testing

(1) Take a sample of 200 gm from material passing the 0.425-mm (No. 40) sieve.

(2) Place the sample on the porcelain mixing dish and mix thoroughly with water (50 - 70 ml) until the mass becomes a thick homogenous paste (uniform in colour).

(3) Push the mixed soil in the metal cup with a palette knife and take care not to trap air. The excess soil shall be struck off in order to give a smooth surface.

(4) Place the cup under the penetrometer and lower (bring down) the cone so that it just touches the soil surface. Thereafter, note the dial gauge readings (initial reading).

(5) Release the cone for a period of 5 seconds and note again the dial gauge reading. The difference between the two readings shall be recorded as a cone penetration.

(6) If necessary, add water so that the first reading is approximately 15 mm.

(7) Take a representative of sample (about 10 gm) from the cup and put it on the dish. Weigh it and dry the soil in an oven for one day and weigh it again.

(8) Take the rest of the soil back in the porcelain mixing dish, add water and mix again thoroughly (completely). Put the soil in a clean cup.

(9) Repeat the described operation at least three times using the same sample to which further increments of water are added. The amount of water added must

be chosen so that the range of penetration values of approximately 15 mm to 25 mm is covered.

Plastic Limit Testing

(1) Take sample from material passing 0.425 mm (#40) sieve of 20g.

(2) Add water so that a ball can be rolled, without clinging to the hand. Take 8g m of it and make again a ball.

(3) Roll the ball on the glass plate using the palm of the hand until a tread of 3 mm diameter is formed. If the tread does not show any cracks, turn it into a ball again and reroll.

(4) If the cracks do not show on the thread, take the crumbled soil and determine the moisture content.
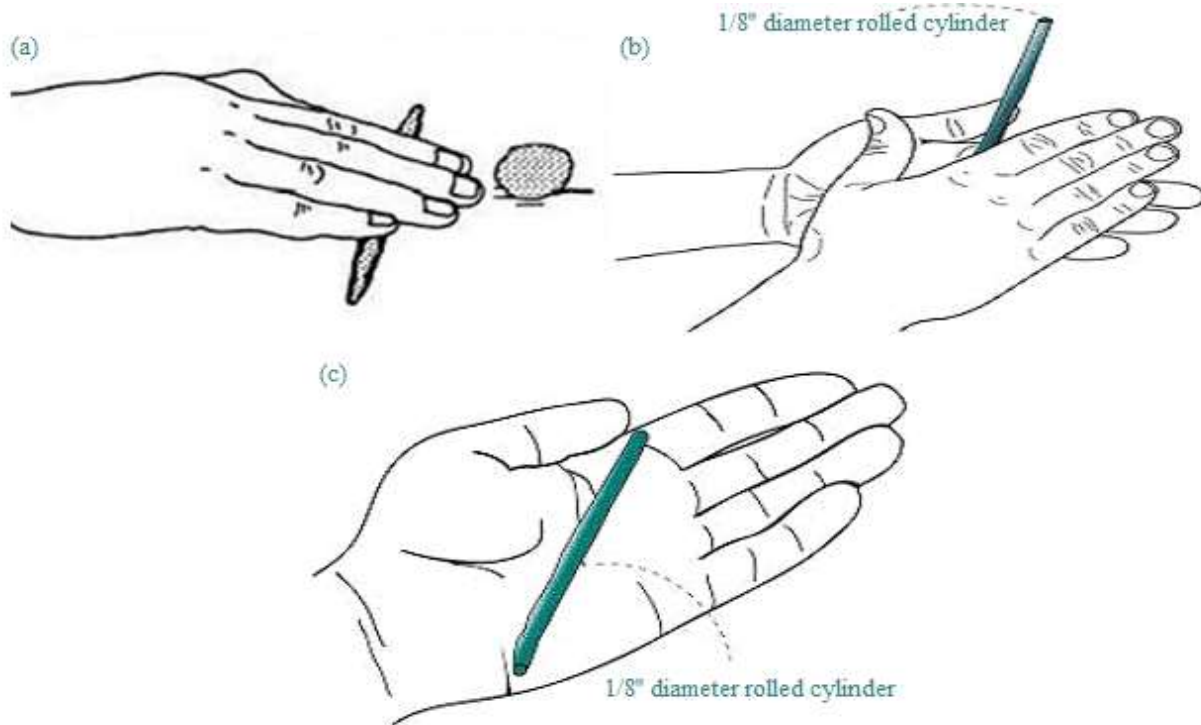
Figure 6.11: Testing procedure of plastic limit.

Data Analysis

Liquid Limit Test

(1) Calculate moisture content for each sample corresponding to penetration reading.
(2) Plot each cone penetration reading (in mm) against the corresponding moisture content. Draw the best straight line.
(3) The moisture content corresponding to a cone penetration of 20 mm is read. The result is reported as a liquid limit.

Plastic Limit Test

(1) Calculate the moisture content of sample tested, this value is the plastic limit.
(2) Calculate the plasticity index of the soil.

Measurements

Liquid Limit Test

Measurements of the testing are shown as given as under:

Table 6.01: Measurements of liquid limit testing.

Item	Data		
No. = Can number	C-6	C-7	J-1
W_{can} = Weight of can (gram)	49.30	48.50	47.04
W_{wet} = Weight of can containing the soil sample in wet state (gram)	79.00	81.00	95.15
W_{dry} = Weight of can containing the soil sample in dry state (gram)	72.66	73.32	83.64
H = Depth of penetration (millimetre)	18.10	19.00	24.20

Plastic Limit Test

Measurements of the testing are shown as given as under:

Table 6.03: Measurements of plastic limit testing.

Item	Data
No. = Can number	X-63
W_{can} = Weight of can (gram)	32.00
W_{wet} = Weight of can containing the soil sample in wet state (gram)	51.40
W_{dry} = Weight of can containing the soil sample in dry state (gram)	48.78

Computation

Liquid Limit Test

Worked example: Can No. C-6:

$$\%wc = \frac{W_{wet} - W_{dry}}{W_{dry} - W_{can}} \times 100 = \frac{79.00 - 72.66}{72.66 - 49.30} \times 100 = 27.14 \%$$

Consequently, final results of the testing are shown as given as under:

Table 6.02: Results of liquid limit testing.

Item	Data		
No. = Can number	C-6	C-7	J-1
W_{can} = Weight of can (gram)	49.30	48.50	47.04
W_{wet} = Weight of can containing the soil sample in wet state (gram)	79.00	81.00	95.15
W_{dry} = Weight of can containing the soil sample in dry state (gram)	72.66	73.32	83.64
H = Depth of penetration (millimetre)	18.10	19.00	24.20
%wc = Depth of penetration (millimetre)	27.14	30.94	31.45
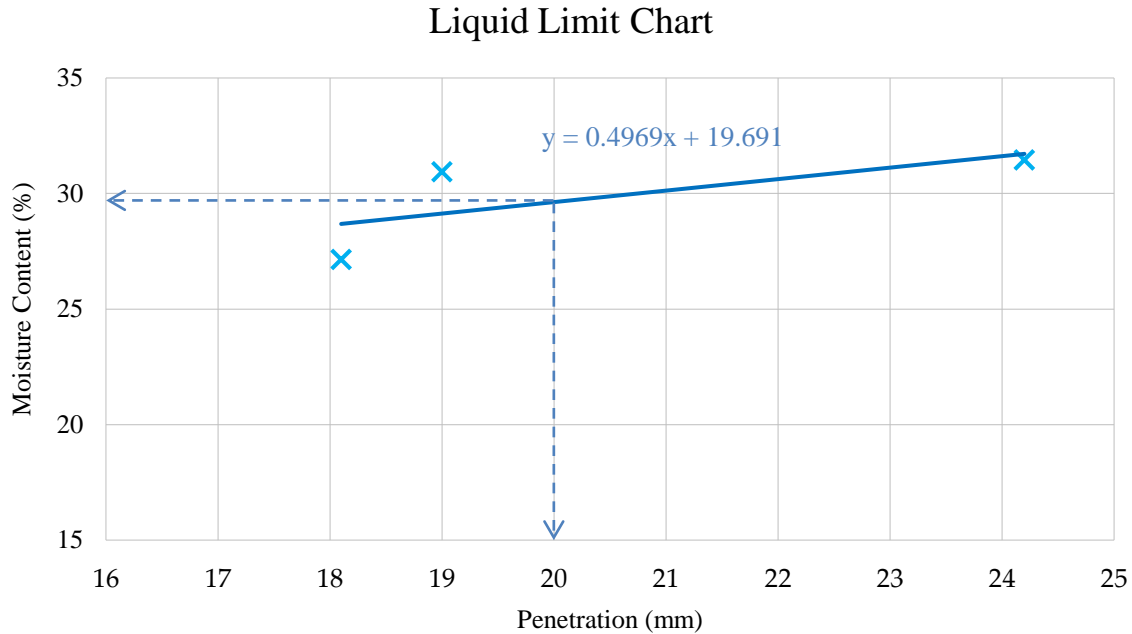
Liquid Limit Chart

From the graph drawn above, the value of LL = 29.629 %

Plastic Limit Test

$$\%wc = \frac{W_{wet} - W_{dry}}{W_{dry} - W_{can}} \times 100 = \frac{51.40 - 48.78}{48.78 - 32.00} \times 100 = 15.614 \%$$

Therefore: PL = 15.614 %

PI = LL − PL = 29.629 − 15.614 = 14.015 %

Results and Discussion

For both tests (i.e. LL and PL tests): After all calculations have been evaluated once, the results were as follows:

LL = 29.629 %

PL = 15.614 %

PI = 14.014 %

Conclusion

For both tests (i.e. LL and PL tests):

- The result of LL is equal to 29.629 %.
- The result of PL is equal to 15.614 %.
- Calibration of penetrometer was done before each trial.

TEST NO. 07

MODIFIED PROCTOR COMPACTION

(ASTM D2216)

Introduction

This test method covers laboratory compaction method used to determine the relationship between moulding water content and dry unit weight of soils (compaction curve) for a given 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 Proctor compaction tests are routinely performed: (1) The Standard Proctor Test and (2) The Modified Proctor Test. However, the Modified Proctor Test is adopted in this experiment.

Definition

Soil is a porous medium consisting of soil solids and water. If a given soil type is compacted using a fixed compaction effort over a range in water contents, a compaction curve of dry densities (ρ_d) versus water content (wc) is derived as illustrated in Figure 7.01. The compaction curve is a concave-downward curve. At low wc, ρ_d is relatively low because the soil particles are in a poorly organized, flocculated configuration. The clay particles adhere to one another because they have negative charges on their faces and positive charges on their edges. As water is added and wc increases, the water neutralizes some of the charge and allows the particles to disperse and assume a more organized orientation. At some point, ρ_d reaches a maximum. The water content at this point is referred to as the optimum water content, (wc_{opt}). If more water is added, the water molecules fill in between the clay particles and ρ_d decreases. If the compaction effort increases, the compaction curve shifts up and to the left. The maximum value for ρ_d increases and optimum water content (wc_{opt}) decreases.
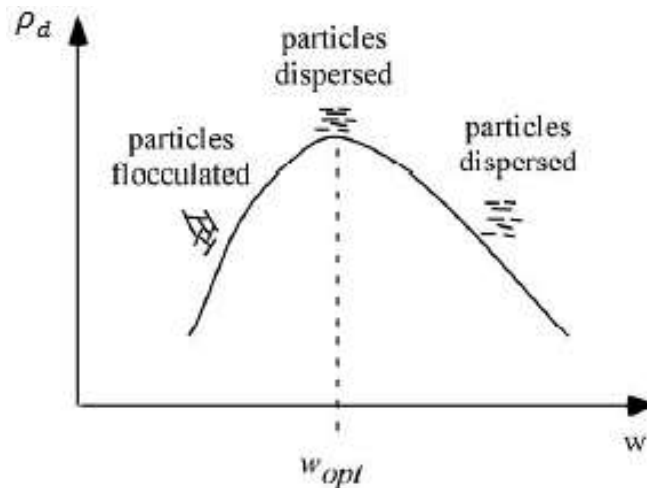
Figure 7.01: Compaction curve showing clay particle orientations along different portions on the curve.

When the relationship between ρ_d and wc at saturation (S) = 100% is superimposed onto the compaction curves, it is referred to as a Zero Air Voids (ZAV) Curve. The ZAV Curve is an excellent way to assess the validity of compaction data. In the ρ_d – wc domain, lines of constant S can be drawn as shown in Figure 7.02. The portion

of the compaction curve wet of wc_{opt} is roughly parallel the ZAV Curve, but offset slightly. Points on the compaction curve that wet of wc_{opt} generally possess S in the range of 90 - 93%.
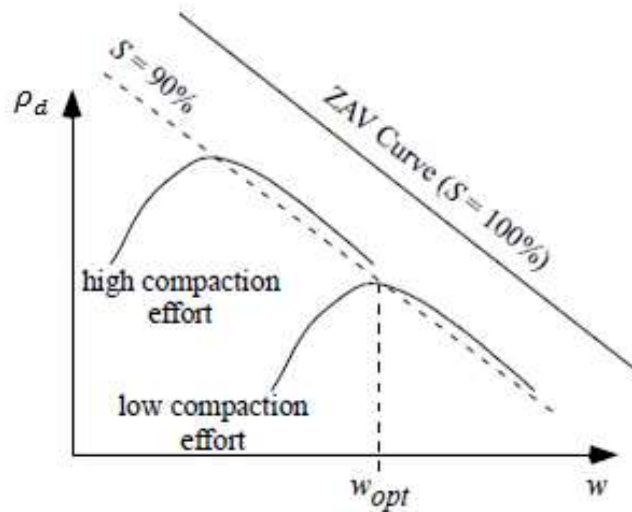
Figure 7.02: Relative position of compaction curves and ZAV curve for low and high compaction effort (optimum water content shown for low compaction effort).

However, the water content that results in the greatest dry density of the soil with compacting is called the optimum water content.

Two types of compaction tests are routinely performed: (1) The Standard Proctor Test and (2) The Modified Proctor Test. In the Standard Proctor Test, the soil is compacted by a 2.5-kg (5.5-lb) hammer falling a distance of one foot into a soil filled mould. The mould 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 4.5-kg (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 moulds used for testing. The smaller type is 101.6 mm (4 in.) in diameter and has a volume of about 944 cm³ (1/30 ft³) and the larger type is 6 inches in diameter and has a volume of about 2123 cm³ (1/13.333 ft³). If the larger mould is used each soil layer must receive 56 blows instead of 25.

Table 7.01: Parameters for Standard Proctor and Modified Proctor testing.

Item / Test Method	Standard Proctor	Modified Proctor
Mould volume; Diameter; Height	944 cm^3 (1/30 ft^3); 101.6 mm (4.000 in.); 116.4 mm (4.584 in.)	
Mass of the manual rammer	2.5 kg (5.5 Ib)	4.5 kg (10 Ib)
Height of drop	305 mm (1 ft)	457 mm (1.5 ft)
Number of layers/lifts	3	5
Number of stocks per layer/lift	25	25
Effort	2,700 kN-m/m^3 (56,000 ft-Ib/ft^3)	600 kN-m/m^3 (12,400 ft-Ib/ft^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 (wc_{opt}) 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.

Purpose

Measurement of the water content of a sample of soil and dry density using method of Modified Proctor and, therefore, to develop an entire compaction curve. The highest dry density, under the influence of compaction of soil, gets rid of the air spaces between soil particles and resolved grained solids will replace the air in the voids and thus soil will become more intense.

Standard Reference

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

Equipment and Materials

(1) Mould of 944-cm^3 volume (116.4 mm in height, 101.6 mm in diameter and cylinder in shape).
(2) Balance sensitive to an accuracy of 0.01 gm.
(3) Hot plate.
(4) Manual rammer (10-Ib or 4.5-kg mass).
(5) Mixing pan.
(6) Extruder.
(7) Drying oven, trowel, No. 4 sieve and moisture cans.
(8) Graduated cylinder and mixing pan.
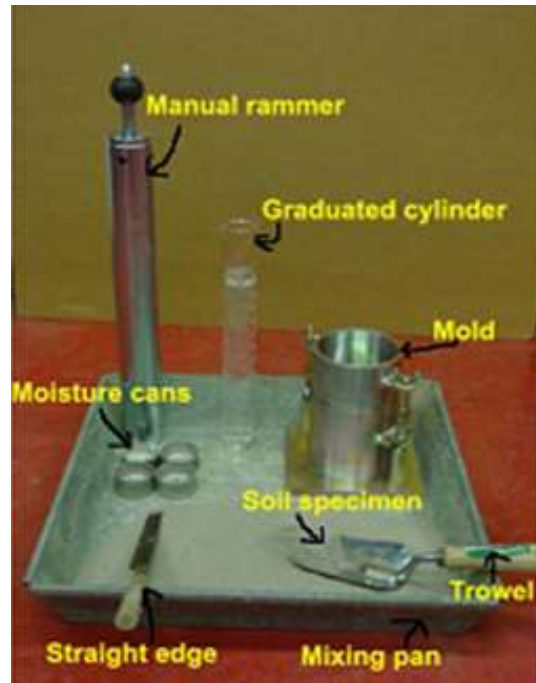
Figure 7.03: Manual rammer; moisture cans; starlight edge; mixing pan; trowel; soil specimen; mould; graduated cylinder.

Procedure

(A) Preparing soil sample:

    (1) Take a sample from the field (12 - 13 kg), then divide it into two portions; each one of mass 6 kg.

    (2) Sieve one portion of soil on 19-mm sieve. If it totally passed, take it for the test and if any quantity retained, replace it by an equivalent quantity from the other portion passing from 19-mm sieve and retained on 4.75-mm sieve.

(B) Compaction test:

    (1) Assemble the compaction mould, stand and collar. Measure and record the height (H_{mould}) and diameter (D_{mould}) of the mould. Tighten the wingnuts and seat all the pieces together properly and spray the inside of the assembly with an aerosol spray lubricant.

    (2) Weigh the mould empty with its base (M_{mould}).

    (3) Mix a quantity of water with the soil sample taken.

    (4) Fill proctor mould with soil in 5 layers, place enough loose soil into the mould prior to compaction of each lift such that the compacted material

will occupy approximately one-fifth of the mould. The top of the final lift should be just above the top of the mould such that it will need to be trimmed slightly. Each layer is compacted 25 blows/lifts by manual rammer of 4.5-kg mass falling from 457-mm height.

(5) Remove the collar and trim the excess soil off the top of the mould.

(6) Weigh the mould full of compacted soil with its base ($M_{mould+soil}$).

(7) Obtain samples from the top, middle and bottom of the specimen and perform water content measurements on the samples to obtain the average water content (wc_{avg}) for the specimen.

$$\%wc_{avg} = \frac{\%wc_1 + \%wc_2 + \%wc_3}{3}$$

$$\%wc_i = \frac{M_{mc} - M_{dc}}{M_{dc} - M_c} \times 100$$

(8) Calculate moist and dry densities of the soil sample using the following formulas:

$$\rho_{moist} = \frac{M}{V} = \frac{M_{(mould+soil)} - M_{(mould)}}{V_{(mould)}} = \frac{M_{(mould+soil)} - M_{(mould)}}{\frac{\pi}{4} D^2_{mould} \times H_{mould}}$$

$$\rho_{dry} = \frac{\rho}{1 + \frac{\%wc}{100}}$$

(9) Repeat the previous steps for several samples (at least 5 samples) with increasing water content from 3 - 4%.

Data Analysis

(1) After calculating moisture content and dry density for each trial, develop the entire compaction curve in virtue of drawing densities on y-axis and water contents on x-axis.

(2) Determine the moisture content corresponding to the peak dry density on the curve developed in the previous step and report this value as the optimum moisture content.

Measurements

Weight of the sample is 4000 gram.

Water Content Measurements

Measurements of the testing are shown as given as under:

Table 7.02: Measurements of moisture content testing.

Item / Trial Sample	No. 1	No. 2	No. 3	No. 4
No. = Can Number	F-1	E-9	M-19	U-4
M_c = Weight of can empty (gram)	080.00	115.00	115.00	110.00
M_{wc} = Weight of can containing wet soil sample (gram)	280.00	490.00	460.00	455.00
M_{wc} = Weight of can containing dry soil sample (gram)	268.90	460.30	430.50	421.00
M_{water} = Weight of water (gram)	…	…	…	…
M_{soil} = Weight of dry soil (gram)	…	…	…	…
%wc = Percent water content	…	…	…	…

* The sign (…) means that the cell needs to be evaluated, for more details go to the computation section.

Density Measurements

Measurements of the testing are shown as given as under:

Table 7.03: Measurements of density testing.

Item / Trial Sample	No. 1	No. 2	No. 3	No. 4
$M_{(mould+soil)}$ = Weight of mould containing soil sample (gm)	5160	5220	5240	5200
M_{mould} = Weight of mould empty (gram)	3445	3445	3445	3445
M_{soil} = Weight of soil (gram)	…	…	…	…
V_{mould} = Volume of mould (cubic centimetre)	944	944	944	944
ρ_{moist} = Soil moist density (gram per cubic centimetre)	…	…	…	…
%wc = Percent water content	…	…	…	…
ρ_{dry} = Soil dry density (gram per cubic centimetre)	…	…	…	…

* The sign (…) means that the cell needs to be evaluated, for more details go to the computation section.

Computation

$m_t = 4000$ gm

Water Content Measurements

Table 7.04: Results of moisture content testing.

Item / Trial Sample	No. 1	No. 2	No. 3	No. 4
No. = Can Number	F-1	E-9	M-19	U-4
M_c = Weight of can empty (gram)	080.00	115.00	115.00	110.00
M_{wc} = Weight of can containing wet soil sample (gram)	280.00	490.00	460.00	455.00
M_{wc} = Weight of can containing dry soil sample (gram)	268.90	460.30	430.50	421.00
M_{water} = Weight of water (gram)	011.10	029.30	029.50	034.00
M_{soil} = Weight of dry soil (gram)	188.90	345.30	315.50	311.00
%wc = Percent water content	005.88	008.60	009.35	010.93

Density Measurements

Table 7.05: Results of density testing.

Item / Trial Sample	No. 1	No. 2	No. 3	No. 4
$M_{(mould+soil)}$ = Weight of mould containing soil sample (gm)	5160	5220	5240	5200
M_{mould} = Weight of mould empty (gram)	3445	3445	3445	3445
M_{soil} = Weight of soil (gram)	1715	1775	1795	1755
V_{mould} = Volume of mould (cubic centimetre)	944	944	944	944
ρ_{moist} = Soil moist density (gram per cubic centimetre)	1.817	1.880	1.902	1.860
%wc = Percent water content	5.876	8.600	9.350	10.93
ρ_{dry} = Soil dry density (gram per cubic centimetre)	1.716	1.731	1.739	1.677

Moisture Content Versus Dry Density Relationship
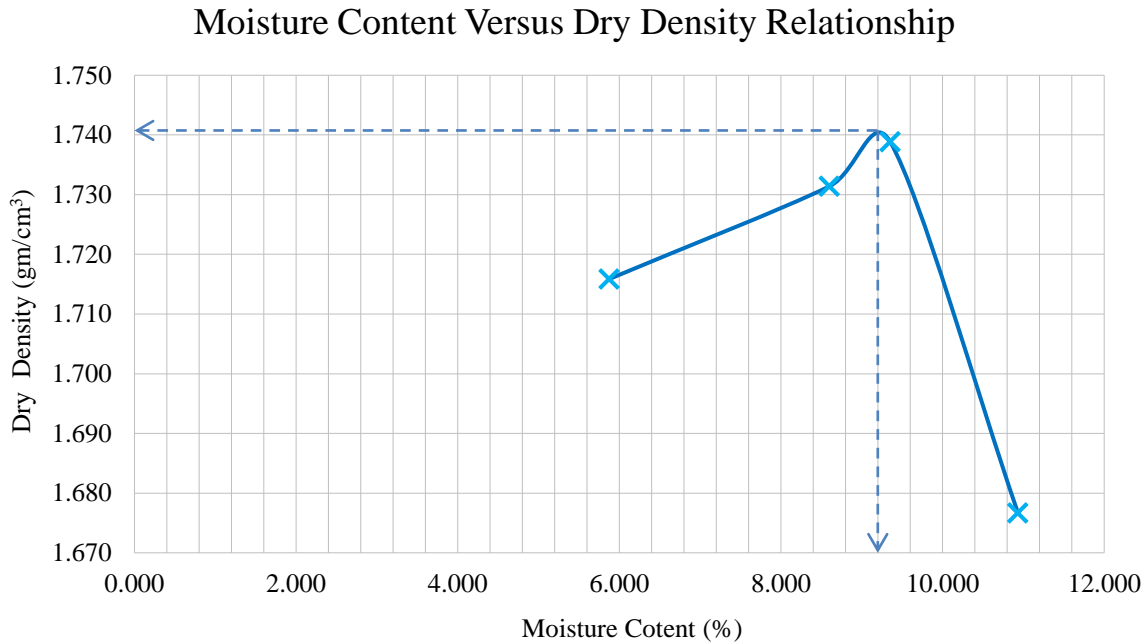
Figure 7.04: Moisture content versus dry density relationship.

From the graph drawn above:

The peak dry density is $\rho_{dry} = 1.741 \text{ gm/cm}^3$

The corresponding optimum moisture content is $\%wc_{opt} = 8.62 \%$

Results and Discussion

After all calculations have been evaluated once, the results were as follows:

$\rho_{dry} = 1.741 \text{ gm/cm}^3$

$\%wc_{opt} = 8.62 \%$

Conclusion

- The maximum dry density is equal to 1.741 gm/cm^3.
- The optimum moisture ratio is equal to 8.62%.

- This experiment is very significant, since it is used to determine the maximum dry density that soils could reach in site.

TEST NO. 08

FIELD DENSITY BY SAND CONE METHOD

(ASTM D1556)

Introduction

This test method covers the measurement of the in-situ dry density of the soil that has been compacted with the addition of water. Field density measurement of soil compaction is done in accordance Sand Cone Method (ASTM D1556), Rubber Balloon Method (ASTM D2167) or Nuclear Methods for shallow depths (ASTM D6938). However, in this experiment, Sand Cone Method is adopted to measure the density and water content of compacted soils placed during the construction of earth embankments, road fill and structural backfill. It often is used as a basis of acceptance for soils compacted to a specified density or percentage of a maximum density determined by a test method, such as Test Methods ASTM D698 (Proctor Compaction Test) or ASTM D1557 (Modified Proctor Compaction Test)

Definition

The actual density of in-field soils that have been compacted with the addition of water and is called field density. It is also named in-situ density or in-place density.

By conducting this test, it is possible to measure the field density of the soil. The moisture content is likely to vary from time and, hence, the field density also. So, it is required to report the test result in terms of dry density. Three methods are there to measure the dry density of soils in place, these are as follows:

(1) Sand Cone Method (ASTM D1556).
(2) Rubber Balloon Method (ASTM D2167).
(3) Nuclear Methods for shallow depths (ASTM D6938).

However, the first method is adopted to use in this manual which is also known as Sand Replacement Method. The Sand Replacement Method is also employed to measure the unit weight of soils in situ. In Sand Replacement Method, a small cylindrical pit is excavated and the weight of the soil excavated from the pit is measured. Sand whose density is known is filled into the pit. By measuring the weight of sand required to fill the pit and knowing its density, the volume of the pit is calculated. Knowing the weight of soil excavated from the pit and the volume of pit, the density of soil is calculated. Therefore, in this experiment there are two stages, namely: (1) Calibration of sand density and (2) Measurement of soil density.

The sand cone device consists, as shown in Figure 8.01, of a one-gallon glass or plastic jar, a detachable metal appliance and a base plate. The metal appliance is attached to the jar to allow the sand to flow from the jar during calibrations or density testing.
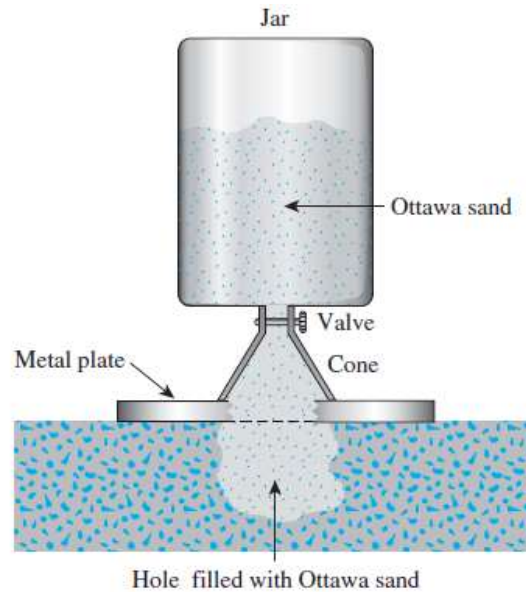
Figure 8.01: Sand cone apparatus (Glass jar filled with Ottawa sand).

Significance

- It is very quality control test, where compaction is required, in the cases like embankment and pavement construction. The dry density of the compacted soil or pavement material is a common measure of the amount of the compaction achieved during the construction. Knowing the field density and field moisture content, the dry density is calculated.
- Therefore, field density test is of importance as a field control test for the compaction of soil or any oilier pavement layer.
- The in-situ density of natural soil is needed for the determination of bearing capacity of soils, for the purpose of stability analysis of slopes, for the determination of pressures on underlying strata for the calculation of settlement and the design of underground structures.

Purpose

Measurement of the dry density of soil in the field which has been compacted and added to it optimum water ratio. Thus, it can be compared with specifications when considering terms, such as relative compaction (degree of compaction).

Standard Reference

ASTM D1556 — Standard Test Method for Density and Unit Weight of Soil in Place by the Sand-Cone Method.

Equipment and Materials

(1) Standard Ottawa sand (with known density of $\rho = 1.48$ gm/cm^3).
(2) Metal square plate of 30-cm dimension and 10-cm hole diameter.
(3) Jar connected to a cone of known volume.
(4) Spoon.
(5) Balance sensitive to an accuracy of 0.01 gm.
(6) Cylindrical mould.

Figure 8.02: Equipment and tools required.

Procedure

(1) Fill the jar with Ottawa sand and then weigh it (M_1).
(2) Put the metal plate above soil in the field and take sample of soil by making a hole with a depth of $10 - 12$ cm, then put the sample in plastic sack and weigh it (M_{field}) and then determine the water content (%wc) of it.

(3) Invert the jar with its cone full of Ottawa sand in the hole and open the valve allowing the sand to flow until it stop flowing, then close the valve. Thus, the sand is distributed in three parts (in the hole, cone and jar).

Data Analysis

(1) Weigh the retained sand in the jar (M_2).

(2) Calculate the mass of soil in the cone (M_3):

$$M_3 = \rho_{Ottawa} \times V_{Cone}$$

(3) Calculate the mass of soil in the hole (M_h):

$$M_h = M_1 - (M_2 + M_3)$$

(4) Calculate the volume of hole (V_h):

$$V_h = \frac{M_h}{\rho_{Ottawa}}$$

(5) Calculate the field density, dry field density and the degree of compaction:

$$\rho_{field} = \frac{M_{field}}{V_h}$$

or

$$\rho_{field} = \frac{M_{(mold+soil)} - M_{(mold)}}{V_{(hole)}}$$

(6) Calculate the field dry density (ρ_{dry}) of the soil sample taken:

$$\%wc = \frac{M_{wc} - M_{dc}}{M_{dc} - M_c} \times 100$$

$$\rho_{dry} = \frac{\rho_{field}}{1 + \dfrac{\%wc}{100}}$$

(7) Calculate the degree of compaction ($\%D_c$):

$$\%D_c = \frac{\rho_{DryField}}{\rho_{DryProctor}} \times 100 \geq 100\%$$

Measurements

Measurements of the testing are shown as given as under:

Table 8.01: Testing measurements.

Item	Data
M_1 = Weight of the jar filled with Ottawa sand (gram)	5000.0
M_2 = Weight of Ottawa sand retained in the jar (gram)	0795.0
M_3 = Weight of Ottawa sand filling the cone (gram)	1795.0
M_{field} = Weight of the soil sample taken from field (gram)	3175.0
ρ_{field} = Field (bulk) density of soil (gram per cubic centimetre)	…
%wc = Percent water content of the sample taken from the field	…
$\rho_{DryField}$ = Dry in-field density of soil (gram per cubic centimetre)	…
$\rho_{DryProctor}$ = Dry Proctor density of soil (gram per cubic centimetre)	…
$\%D_c$ = Percent degree of compaction of soil in field	…

* The sign (…) means that the cell needs to be evaluated, for more details go to the computation section.

Computation

$$M_h = M_1 - (M_2 + M_3) = 5000.0 - (795.0 + 1795.0) = 2410.0 \text{ gm}$$

$$\rho_{Ottawa} = 1.48 \text{ gm/cm}^3$$

$$V_h = \frac{M_h}{\rho_{Ottawa}} = \frac{2410.0}{1.48} = 1628.38 \text{ cm}^3$$

$$\rho_{field} = \frac{M_{field}}{V_h} = \frac{3175.0}{1628.38} = 1.95 \text{ gm/cm}^3$$

$$\%wc = \frac{M_{wc} - M_{dc}}{M_{dc} - M_c} \times 100 = 22.74 \%$$

$$\rho_{dry} = \frac{\rho_{field}}{1 + \frac{\%wc}{100}} = \frac{1.95}{1 + \frac{22.74}{100}} = 1.59 \text{ gm/cm}^3$$

From the previous test of Modified Proctor:

$$\rho_{DryProctor} = 1.74 \text{ gm/cm}^3$$

$$\%D_c = \frac{\rho_{DryField}}{\rho_{DryProctor}} \times 100 = \frac{1.59}{1.74} = 91.38 \% \geq 100 \%$$

Consequently, final results of the testing are shown as given as under:

Table 8.02: Testing results.

Item	Data
M_1 = Weight of the jar filled with Ottawa sand (gram)	5000.0
M_2 = Weight of Ottawa sand retained in the jar (gram)	0795.0
M_3 = Weight of Ottawa sand filling the cone (gram)	1795.0
M_{field} = Weight of the soil sample taken from field (gram)	3175.0
ρ_{field} = Field (bulk) density of soil (gram per cubic centimetre)	001.95
%wc = Percent water content of the sample taken from the field	022.74
$\rho_{DryField}$ = Dry in-field density of soil (gram per cubic centimetre)	001.59
$\rho_{DryProctor}$ = Dry Proctor density of soil (gram per cubic centimetre)	001.74
$\%D_c$ = Percent degree of compaction of soil in field	091.38

* The sign (…) means that the cell needs to be evaluated, for more details go to the computation section.

Results and Discussion

After all calculations have been evaluated once, the results were as follows:

$$\rho_{field} = 1.95 \text{ gm/cm}^3$$

$$\rho_{Dry-Field} = 1.59 \text{ gm/cm}^3$$

$$D_c = 91.38 \%$$

Conclusion

- The in-field density is equal to 1.95 gm/cm^3
- The degree of compaction is equal to 91.38%, which is too low, so it is required to apply more compaction effort on soil layers to meet the specified relative compaction (usually it is required to be higher than 95%, 98%, etc.).
- This experiment is very significant, since it is used to determine the dry density that in-situ soils have been reached.

TEST NO. 09

CONSTANT-HEAD PERMEABILITY

(ASTM D2216)

Introduction

This test method covers the measurement of the coefficient of permeability (hydraulic conductivity) by a constant-head method for the laminar flow of water through granular soils. The procedure is to establish representative values of the coefficient of permeability of granular soils that may occur in natural deposits as placed in embankments or when used as base courses under pavements. In order to limit consolidation influences during testing, this procedure is limited to disturbed granular soils containing not more than 10 % soil passing the 75-μm (No. 200) sieve. This procedure is suitable for soils having coefficients of permeability in the range 10^{-2} to 10^{-5} m/sec.

Definition

Coefficient of permeability is defined as the rate of flow under laminar flow conditions through a unit cross sectional are of porous medium under unit hydraulic gradient is defined as coefficient of permeability.

Soils are permeable due to the existence of interconnected voids through which water can flow from points of high energy to points of low energy. Permeability refers to the ease with which water can flow through a soil. Hydraulic conductivity is generally expressed in cm/sec or m/sec in SI units and in ft/min or ft/day in English units. The hydraulic conductivity of soils depends on several factors: fluid viscosity, pore size distribution, grain-size distribution, void ratio, roughness of mineral particles and degree of soil saturation.

The value of hydraulic conductivity (k) varies widely for different soils. Some typical values for saturated soils are given in Table 9.01. The hydraulic conductivity of unsaturated soils is lower and increases rapidly with the degree of saturation.

Table 9.01: Typical values of hydraulic conductivity of saturated soils.

Soil Type	Hydraulic Conductivity, K	
	cm/sec	ft/min
Clean gravel	100 - 1.0	200 - 2.0
Coarse sand	1.0 - 0.01	2.0 - 0.02
Fine sand	0.01 - 0.001	0.02 - 0.002
Silty clay	0.001 - 0.00001	0.002 - 0.00002
Clay	< 0.000001	< 0.000002

There are two general permeability test methods that are routinely performed in a soil mechanics laboratory: (1) Constant-head test method and (2) Falling-head test method. The constant-head test method is used for permeable soils whose k is greater than 10^{-4} cm/sec (for course-grained soils, namely sand and gravel), while the falling-head test is mainly used for less permeable soils whose k is lesser than 10^{-4} cm/sec (for fine-grained soils, namely silt and clay). However, Constant-head test method is adopted to use in this experiment to test hydraulic conductivity for a sand specimen.

A typical arrangement of the constant-head permeability test is demonstrated in Figure 9.01. In this type of laboratory set-up, the water supply at the inlet is adjusted in such a way that the difference of head between the inlet and the outlet remains constant during the test period. After a constant flow rate is established, water is collected in a graduated flask for a known duration.

The specimens shall be cylinders with minimum diameter of 8 or 12 times the maximum particle size. The permeameter should be fitted with a porous disk at the bottom with a permeability greater than that of the soil specimen, but with openings small enough to prevent movement of the soil particles. The permeameter should be fitted with manometer outlets for measuring heads loss (h), over a length (L), equivalent to at least the diameter of the cylinder.

The coefficient of permeability may be expressed as:

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

Where:

k = coefficient of permeability (cm/sec).
Q = volume of outflow (cm^3).
L = distance between manometers (cm).
A = area of cross section of the soil specimen (cm^2).
t = test duration (sec).
h = change in head (cm).
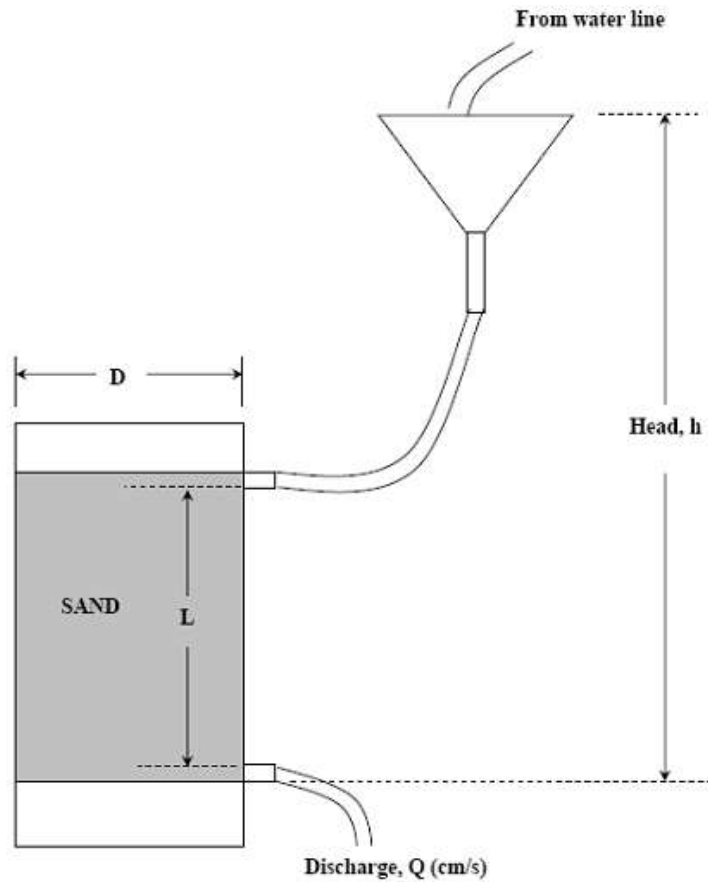
Figure 9.01: Constant-head permeability test.

However, is conventional to express the value of k at a temperature of 20 ˚C. Within the range of test temperatures, it can be assumed that:

$$k_{20°C} = \left(\frac{\eta_{T°C}}{\eta_{20°C}}\right) K_{T°C}$$

Where

$\eta_{T°C}$ = viscosity of water at temperature degree T ˚C.

The variation of $\eta_{T°C}/\eta_{20°C}$ with the test temperature T varying from 15 to 30 °C is given in Table 9.02.

Table 9.02: Variation of $\eta_{T°C}/\eta_{20°C}$.

Temperature, T (°C)	$\eta_{T°C}/\eta_{20°C}$	Temperature, T (°C)	$\eta_{T°C}/\eta_{20°C}$
15	1.135	23	0.931
16	1.106	24	0.910
17	1.077	25	0.889
18	1.051	26	0.869
19	1.025	27	0.850
20	1.000	28	0.832
21	0.976	29	0.814
22	0.953	30	0.797

Significance

- The study of the flow of water through permeable soil media is important in soil mechanics. It 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.
- In addition, it is necessary for investigating problems involving the pumping of water for underground construction and for making stability analyses of earth dams and earth-retaining structures that are subject to seepage forces.

Purpose

Measurement of the coefficient of permeability (hydraulic conductivity) of a sandy soil by constant-head test method.

Standard Reference

ASTM D2434 – Standard Test Method for Permeability of Granular Soils (Constant Head).

Equipment and Materials

(1) Constant-head apparatus, namely permeameter assembly.
(2) Manometer which is, generally, used for measuring the pressure acting on a column of fluid.
(3) Cylindrical mould opened from both sides (of 15-cm height and 6-cm diameter).
(4) Mould to collect water.
(5) Graduated cylinder (of 250-ml or 500-ml volume).
(6) Stopwatch.
(7) Thermometer.
(8) Sensitive balance to weigh up to 1.0 gram.

Figure 9.02: Permeameter.   Figure 9.03: Manometer.
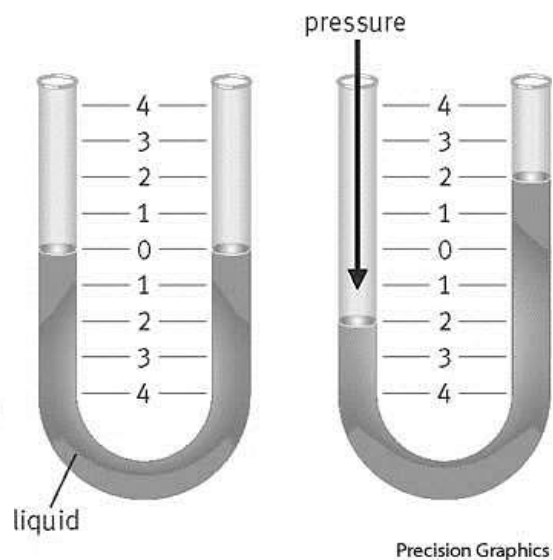
Procedure

This test comes in a two-stage procedure, as follows:

(A) Preparation of a disturbed sand specimen:
  (1) A 2.5-kg sample shall be taken from a thoroughly mixed air dried or oven dried material.
  (2) The initial moisture content of the 2.5-kg sample shall be determined. Then the soil shall be placed in the air tight container.
  (3) Add required quantity of water to get the desired moisture content.
  (4) Mix the soil thoroughly.
  (5) Weigh the empty permeameter mould.
  (6) After greasing the inside slightly, clamp it between the compaction base plate and extension collar.
  (7) Place the assembly on a solid base and fill it with sample and compact it.
  (8) After completion of a compaction the collar and excess soil are removed.
  (9) Find the weight of mould with sample.
  (10) Place the mould with sample in the permeameter, with drainage base and cap having discs that are properly saturated.

(B) Preparation of sand specimen:
  (1) Prepare auxiliary apparatus and permeameter cell. Ensure that all joints are air-tight. Measure internal dimensions of cell, distances between centres of each manometer and fill required results into data sheet.
  (2) Select, prepare and place soil sample in cell. The sample may be placed in the permeameter by one of three methods. These methods are compacting by rodding, dry pouring or by pouring through water.
  (3) Assemble and connect up cell.
  (4) Saturate and de-air sample.
  (5) Run test. Adjust the height of inlet receiver. Open the control valve at the base to produce flow through the sample under the hydraulic gradient appreciably less than unity. Place measuring cylinder and start the timer. Measure the quantity of water collected in the cylinder during the given interval of time and record the time. Record the levels of water in the manometer tubes, record the temperature of water.

(6) Raise the level of constant head tank and repeat the step 5 at a greater hydraulic gradient.

Data Analysis

(1) Calculate the rate of flow (in L/sec) during the period of each observation of flow from the equation:

$$q = \frac{Q}{t}$$

(2) Calculate the hydraulic gradient (i) between the uppermost and lowest manometer gland points from the equation:

$$i = h/L$$

Where:

h = the difference between the two manometer levels, in mm and

y = the difference between the corresponding gland points, in mm.

(3) Calculate the coefficient of permeability k (in m/sec) from the following equation:

$$K = \frac{QL}{Aht}$$

(4) Correct the permeability to that for 20 ˚C (68 ˚F) by multiplying k by the ratio of the viscosity of water at test temperature to the viscosity of water at 20 ˚C (68 ˚F), as follows:

$$k_{20°C} = \left(\frac{\eta_{T°C}}{\eta_{20°C}}\right) K_{T°C}$$

Measurements

Measurements of the testing are shown as given as under:

Table 9.01: Testing measurements.

Item / Trial	No. 1	No. 2
D = Diameter of cylinder (centimetre)	006.00	006.00
L = Length of cylinder (centimetre)	015.00	015.00
M_s = Weight of soil specimen (gram)	1113.0	1113.0
M_{ec} = Weight of the container empty (gram)	244.39	244.39
M_{wc} = Weight of the container containing water (gram)	919.20	843.50
h = Difference in head on manometers (centimetre)	178.00	178.00
t = Test duration (second)	210.00	210.00

Computation

Worked example: Trial No. 01:

$$A = \frac{\pi}{4} \times (6.0)^2 = 28.2743 \text{ cm}^2$$

$$K = \frac{QL}{Aht} = \frac{(919.20 - 244.39)(15)}{(28.2743)(178.0)(210.0)} = 0.009577 = 9.577 \times 10^{-3} \text{ cm/sec}$$

Results and Discussion

After all calculations have been evaluated once, the results were as follows:

Table 9.02: Testing Results.

Item / Trial	No. 1	No. 2
Q = Quantity of water discharged (cubic centimetre)	0674.81	0599.11
t = Total time of discharge (second)	0210.00	0210.00
q = Rate of flow (cubic centimetre per second)	03.2134	02.8529
A = Cross-sectional area of specimen (square centimetre)	28.2743	28.2743
k = Coefficient of permeability (centimetre per second)	$9.5773(10^{-3})$	$8.5029(10^{-3})$

Conclusion

- The value of the coefficient of permeability is equal to 9.6×10^{-3} cm/sec for the first trial, while it is equal to 8.5×10^{-3} cm/sec in the second trial. However, the results in this experiment are convincing to an extent since they indicate that the sand specimen exhibited high permeability property. This is due to the larger size of voids in sandy soils in comparison with that in clays. This is why, constant-head test method is adopted to use in case of sands.

TEST NO. 10

FALLING-HEAD PERMEABILITY

(ASTM D2216)

Introduction

This test method covers the measurement of the coefficient of permeability (hydraulic conductivity) by a falling-head method. Permeability, as mentioned before, 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. For fine-grained soils, falling-head permeability test is done, whereas constant-head permeability test is done for granular soil. In the last experiment, the constant-head test method has been done for a sand specimen, while this experiment addresses the falling-head test method for a clay specimen.

Definition

The apparatus used for the constant-head permeability test method may also be used for the falling-head test method. The equipment is similar except that only the lower half of the permeability cell and a burette, instead of the constant head tank, are used.

A typical arrangement of the falling-head permeability test is demonstrated in Figure 10.6. Water from a standpipe flows through the soil. The initial head difference h_1 at time t_0 is recorded and water is allowed to flow through the soil specimen such that the final head difference at time $t = t_2$ is h_2. Therefore, the coefficient of permeability can be calculated from the following equation:

$$k = \frac{aL}{At} \times \ln\left(\frac{h_1}{h_2}\right)$$

Where:
k = coefficient of permeability (cm/sec).
a = cross section area of stand pipe (cm^2).
L = length of the sample (cm).
A = cross section area of soil sample perpendicular to flow direction (cm^2).
t = collection duration (sec).
h_1 = hydraulic head across the sample at the beginning of the test (cm).
h_2 = hydraulic head across the sample at the end of the test (cm).
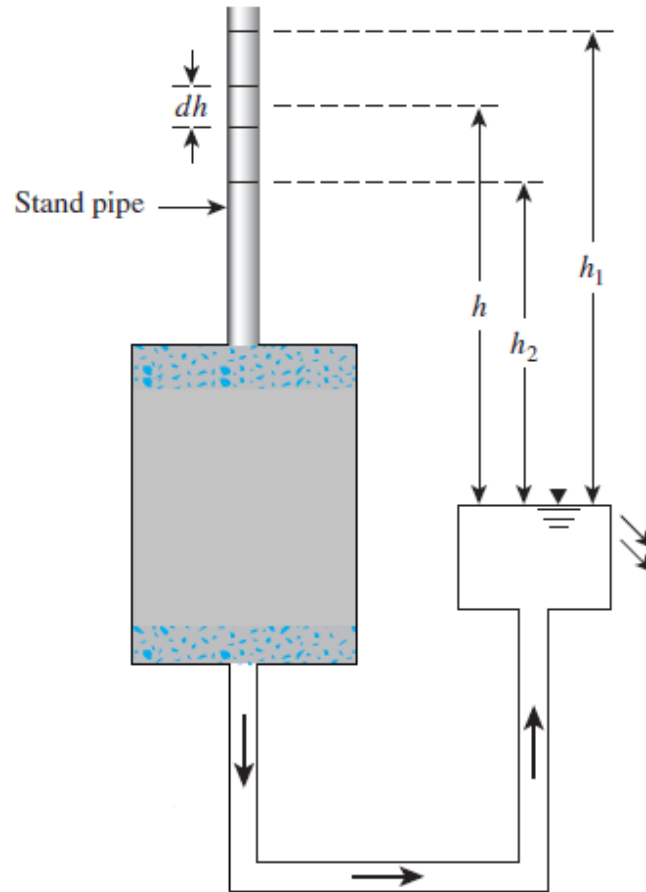
Figure 10.01: Falling-head permeability test.

Significance

- Estimation of the 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 liners.

Purpose

Measurement of the coefficient of permeability (hydraulic conductivity) of a clayey soil by falling-head test method.

Standard Reference

The Falling-head permeability test method is not standardized by ASTM.

Equipment and Materials

(1) Falling-head apparatus.
(2) Manometer which is, generally, used for measuring the pressure acting on a column of fluid.
(3) Cylindrical mould opened from both sides (of 15-cm height and 6-cm diameter).
(4) Mould to collect water.
(5) Graduated cylinder (of 250-ml or 500-ml volume).
(6) Stopwatch.
(7) Thermometer.
(8) Sensitive balance to weigh up to 1.0 gram.
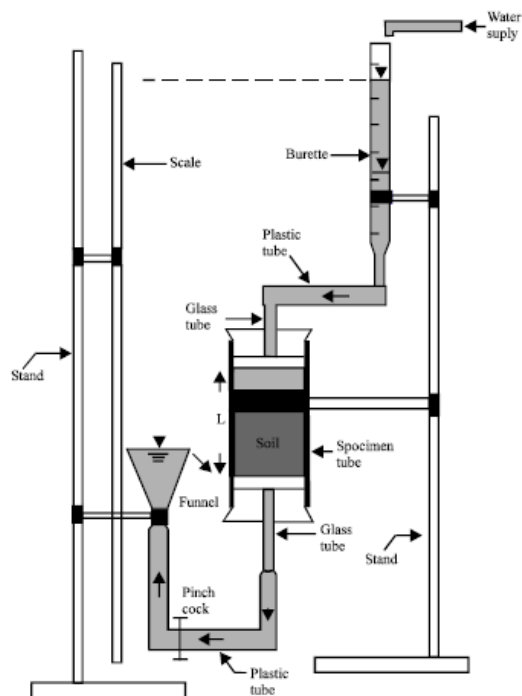
Figure 10.02: Schematic diagram of falling-head permeability test set-up.

Procedure

(1) Prepare a sample of clay soil as stated, earlier, in the last experiment.
(2) Put the cylinder in a vessel opened from the above, then put the vessel in a tank full of water.
(3) Fill the capillary tube with water and then record the water height before starting the test (h_1).
(4) Allow water to flow through the clayey soil, then record the time (t) required for water to ascend to height (h_2) in a capillary soil.
(5) Calculate the coefficient of permeability of clayey soil.

Data Analysis

(1) Calculate the rate of flow (q) (in L/sec) during the period of each observation of flow from the equation:

$$q = \frac{Q}{t}$$

(2) Calculate the hydraulic gradient (i) between the uppermost and lowest manometer gland points from the equation:

$$i = h/y$$

Where:
h = the difference between the two manometer levels (mm).
y = the difference between the corresponding gland points (mm).

(3) Calculate the coefficient of permeability (k) (in m/sec) from the following equation:

$$K = \frac{aL}{At} \times \ln\left(\frac{h_1}{h_2}\right)$$

(4) Find the temperature correction factor and compute the coefficient of permeability (k) at the temperature 20 ˚C (68 ˚F), as follows.

$$k_{20°C} = \left(\frac{\eta_{T°C}}{\eta_{20°C}}\right) K_{T°C}$$

Measurements

Measurements of the testing are shown as given as under:

Table 10.01: Testing measurements.

Item / Trial	No. 1	No. 2
D = Diameter of cylinder (centimetre)	006.00	006.00
h_1 = Head across the sample at the beginning of test (centimetre)	90+66	80+66
h_2 = Head across the sample at the end of test (centimetre)	70+66	60+66
t = Collection duration (second)	122.00	150.00
A = Cross-sectional area of specimen (square centimetre)	…	…
a = Cross-sectional area of the burette (square centimetre)	…	…
k = Coefficient of permeability (centimetre per second)	…	…

* The sign (…) means that the cell needs to be evaluated, for more details go to the computation section.

Computation

Worked example: Trial No. 01:

$$A = \frac{\pi}{4} \times (6.0)^2 = 28.2743 \text{ cm}^2$$

$$a = \frac{\pi}{4} \times \left(\frac{4}{10}\right)^2 = 0.1257 \text{ cm}^2$$

$$K = \frac{aL}{At} \times \ln\left(\frac{h1}{h2}\right) = \frac{0.1257 \times 15.0}{28.2743 \times 122.0} \times \ln\left(\frac{90.0 + 66.0}{70.0 + 66.0}\right)$$

$$= 7.5049 \times 10^{-5} \text{ cm/sec}$$

Results and Discussion

After all calculations have been evaluated once, the results were as follows:

Table 10.02: Testing results.

Item / Trial	No. 1	No. 2
D = Diameter of cylinder (centimetre)	006.00	006.00
h_1 = Head across the sample at the beginning of test (centimetre)	90+66	80+66
h_2 = Head across the sample at the end of test (centimetre)	70+66	60+66
t = Collection duration (second)	122.00	150.00
A = Cross-sectional area of specimen (square centimetre)	28.274	28.274
a = Cross-sectional area of the burette (square centimetre)	0.1257	0.1257
k = Coefficient of permeability (centimetre per second)	$7.5973(10^{-5})$	$6.5478(10^{-5})$

* The sign (…) means that the cell needs to be evaluated, for more details go to the computation section.

Conclusion

- The value of the coefficient of permeability is equal to 7.6×10^{-5} cm/sec for the first trial, while it is equal to 6.5×10^{-5} cm/sec in the second trial. However, the results in this experiment are convincing to an extent since they indicate that the clay specimen exhibited low permeability property. This is due to the smaller size of voids in clayey soils in comparison with that in sands. This is why, falling-head test method is adopted to use in case of clays.

TEST NO. 11

CONSOLIDATION TEST

(ASTM D2216)

Introduction

This test method covers the measurement of the magnitude and rate of consolidation (volume decrease) of soil when it is restrained laterally and drained axially while subjected to incrementally applied controlled-stress loading. From the measured data, the consolidation curve (pressure versus void ratio relationship) can be plotted. Thereafter, this data is useful in determining the compression index, the recompression index and the pre-consolidation pressure (maximum past pressure) of soil. In addition, the data obtained can also be used to determine the coefficient of consolidation and coefficient of secondary compression of soil.

Definition

A stress increase caused by the construction of structures or other loads compresses soil layers. This compression is offered by three different causes, these are: (a) Deformation of soil particles, (b) Relocations of soil particles and (c) Expulsion of water or air from the void spaces. However, the soil settlement caused by loads may be divided into three broad categories, these are: (1) Elastic settlement (immediate settlement), (2) Primary consolidation settlement and (3) Secondary consolidation settlement. The three different settlements may be defined at a glance as follows:

(1) Elastic settlement (S_e), which is the settlement caused, immediately, by the elastic deformation of soil, regardless of whether it is dry, moist or saturated, without any change in the moisture content. Elastic settlement calculations are generally based on equations derived from the theory of elasticity and they will be extensively discussed in Foundation Engineering course in the semester, ahead of you.

(2) Primary consolidation settlement (S_c), which comes as a result of a volume change in saturated cohesive soils due to the expulsion of water that occupies the void spaces existing between soil particles.

(3) Secondary consolidation settlement (S_p), which is observed in saturated cohesive soils and is the result of plastic adjustment of soil fabrics. It is an additional form of compression that occurs at constant effective stress.

Therefore, the total settlement can be expressed as the sum of the three components as follows:

$$S_T = S_e + S_c + S_P$$

Where:

S_T = total settlement of soil.

S_e = elastic settlement of soil.

S_c = primary consolidation settlement of soil.

S_p = secondary consolidation settlement of soil.

Further, there are some informative terms that are really important. These are:

Over Consolidation Ratio (OCR): It is a ratio that results when dividing the pre-consolidation pressure over the present effective vertical pressure that a soil experiences. Therefore, this ratio is:

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

Where:

OCR = over consolidation ratio.

σ_c' = pre-consolidation pressure.

σ' = present effective vertical pressure.

Pre-consolidation pressure is the maximum effective vertical overburden stress (pressure) that a soil has sustained in the past. Basically, this ratio results in two definitions of clay depending upon its stress history, which are as follows:

Normally consolidated clay: The clay is said to be normally consolidated when its present pressure is the maximum pressure that the soil has been subjected to during its geological life. Thus, the over consolidation ratio is equal to one.

Over Consolidated Clay: The clay is said to be over consolidated when the present pressure is less than the maximum pressure that the soil was subjected to during its geological life. Thus, the over consolidation ratio is greater than one.

Under consolidated clay: The clay is said to be under consolidated when its present pressure is greater than the maximum pressure that the soil has been subjected to during its geological life. Thus, the over consolidation ratio is less than one.

	OCR > 1.0 → Over Consolidated Clay (OCC)
Over Consolidation Ratio (OCR)	OCR = 1.0 → Normally Consolidated Clay (NCC)
	OCR < 1.0 → Under Consolidated Clay (UCC)
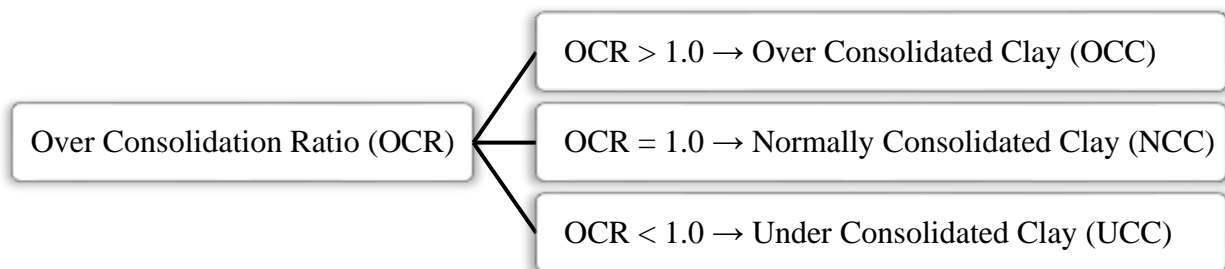
Figure 11.01: Over consolidation ratio of clay.

Significance

- The consolidation properties determined from the consolidation test are used to estimate the magnitude and the rate of both primary and secondary consolidation settlements of a structure or an earth fill.

- Estimates of this type are of key importance in case of designing engineering structures and evaluating their performance.

Purpose

Determination of some factors used in settlement calculations of a soil, which are:

(1) Coefficient of consolidation, C_v.
(2) Compression index, C_c.

Standard Reference

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

Equipment and Materials

(1) Odometer apparatus (porous stones and loading device).
(2) Trimmer and Extruder.
(3) Stopwatch.
(4) Metal ring (of 2-cm height and 6.21-cm in diameter).
(5) Glass plate.

Figure 11.02: Consolidation apparatus and dial gages.

Procedure

(1) Prepare a soil sample from the field where soil is to tested.

(2) Put the porous stone below the ring, then put filter paper and soil sample above it. Use glass plate to compress the sample and level its surface, then put another porous stone above the sample.

(3) Put the sample in a vessel of the device which is fully filled with water to ensure that the soil is fully saturated with water.

(4) Put the loads on the sample, then from the dials record the value of deformation of the sample and the value of the load. Start the stop watch to record the elapsed time for each deformation value for the same vertical load until the values of deformations being constant, then increase the loads to the double in each time and continue until 16-kg load at which it is expected that total consolidation is occurred to the soil.

(5) Dry the sample and measure its dry mass Ms and its specific gravity, G_s.

Data Analysis

The result of Lab work will be the values of applied loads and the reading of deformation. C_v can be determined by two methods:

(1) Determination of coefficient of consolidation, C_v:
   a. Logarithmic Time Method.
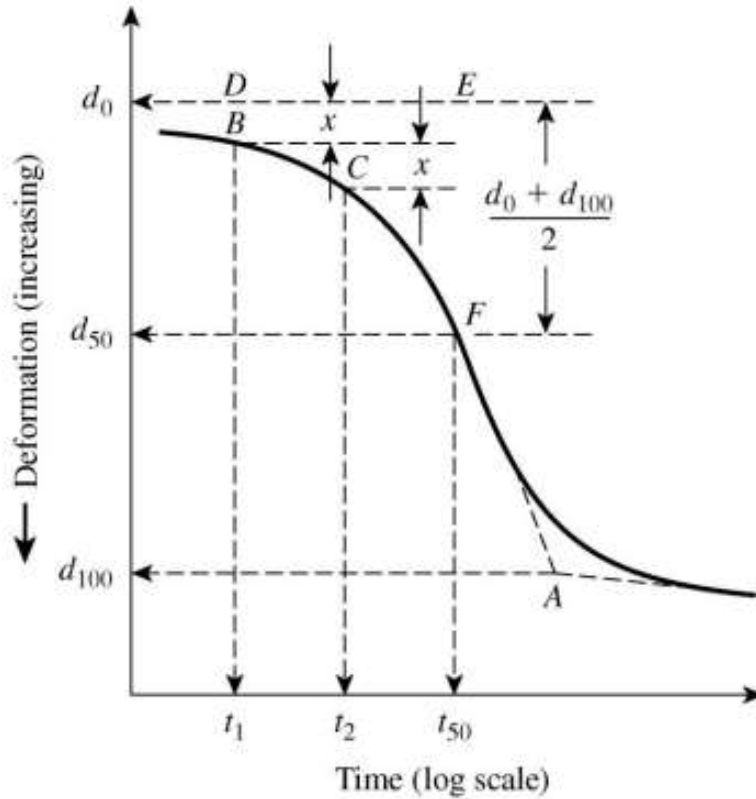
Figure 11.03: Log time versus deformation plot.

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

Where:

H_{dr} = height of drainage which equals the thickness of soil layer if one way drainage or one half the thickness of soil layer for two-way drainage.

t_{50}: time required for 50% consolidation.

b. Square Root Method.

$$OC = 1.15 \times OB$$

$$C_v = \frac{0.848 \, H_{dr}^2}{t_{90}}$$
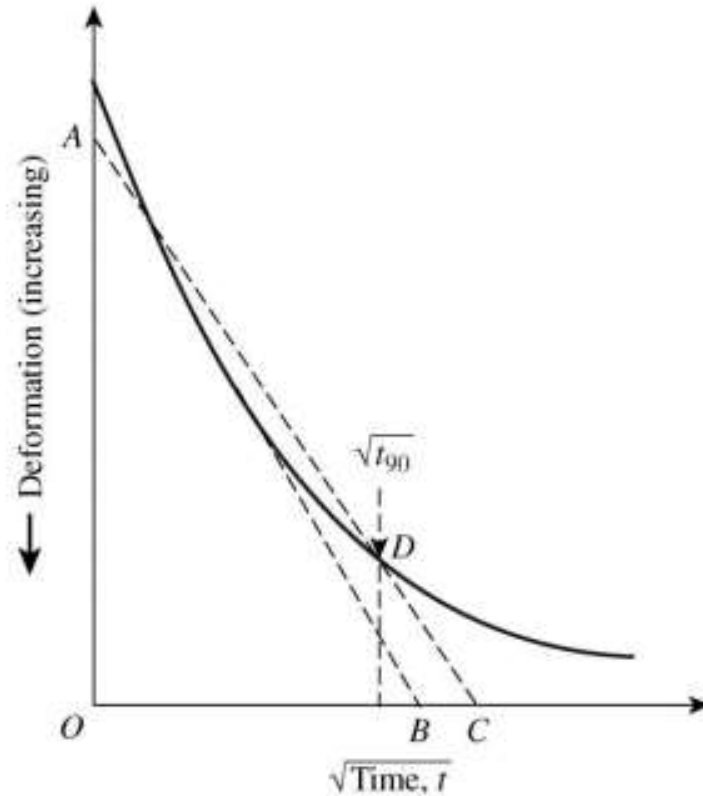
Figure 11.04: Square root of time versus deformation plot.

(2) Determination of compression index, C_c:
Draw a relation between the applied stresses in log scale on x-axis versus the values of void ratio, e, on y-axis to obtain C_c.

$$C_v = \frac{M_s}{\rho_w G_s A}$$

$$e_o = \frac{H_i - H_s}{H_s}$$

$$e_i = e_o - \frac{\sum \Delta H}{H_s}$$

Where:

H_s = height of the soil solids in a specimen which is constant.
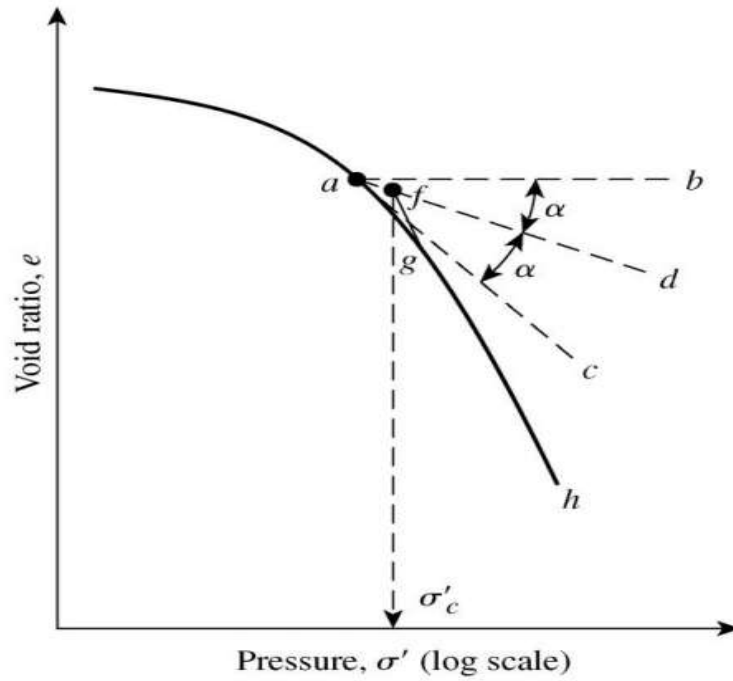
Figure 11.05: Pre-consolidation pressure.
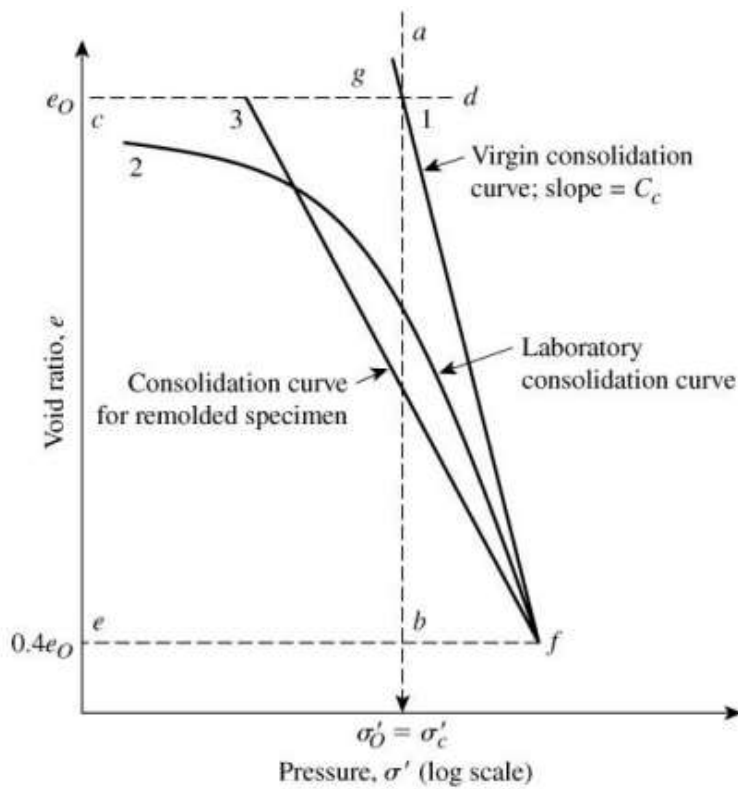
Figure 11.06: Pressure versus deformation plot.

Measurements

Measurements of the testing are shown as given as under:

(1) Factor of consolidation, C_v

Table 11.01: Testing measurements.

Normal Stress, $\sigma = 100$ kPa	
Elapsed Time (min)	**Dial Readings (0.02 mm)**
00.00	57.5
00.10	64.5
00.25	66.5
00.50	68.0
01.00	68.5
02.00	70.5
04.00	71.5
08.00	73.5
15.00	76.5
30.00	78.5
60.00	82.0
120.0	87.0
256.0	92.5
420.0	95.5
1417	96.5

Table 11.02: Testing measurements.

Normal Stress, σ = 200 kPa	
Elapsed Time (min)	**Dial Readings (0.02 mm)**
00.00	096.5
00.10	108.0
00.25	110.5
00.50	112.0
01.00	114.5
02.00	116.0
04.00	118.5
08.00	122.0
15.00	125.5
30.00	131.0
60.00	136.5
120.0	142.0
256.0	146.5
420.0	149.5
1417	150.5

Height of specimen, $H_i = 2.0$ cm

(2) Factor of compression, C_c, and pre-consolidation pressure, σ'_c

Table 11.03: Testing measurements.

Pressure (kPa)	Displacement, ΔH (0.001 mm)
00.00	00.0
25.00	34.0
50.00	29.0
100.0	41.0
200.0	56.0
400.0	73.0
800.0	69.0
1600	73.0

Computation

(1) Factor of consolidation, C_v

(A) For normal stress, $\sigma = 100$ kpa

Table 11.04: Testing results.

Elapsed Time (min)	Dial Readings (0.02 mm)	Deformation, ΔH (mm)	$\sqrt{t}$
00.00	57.5	1.15	00.00
00.10	64.5	1.29	00.32
00.25	66.5	1.33	00.50
00.50	68.0	1.36	00.71
01.00	68.5	1.37	01.00
02.00	70.5	1.41	01.41
04.00	71.5	1.43	02.00
08.00	73.5	1.47	02.83
15.00	76.5	1.53	03.87
30.00	78.5	1.57	05.48
60.00	82.0	1.64	07.75
120.0	87.0	1.74	10.95
256.0	92.5	1.85	16.00
420.0	95.5	1.91	20.49
1417	96.5	1.93	37.64

(a) Logarithmic Time Method

Logarithm of Time Method
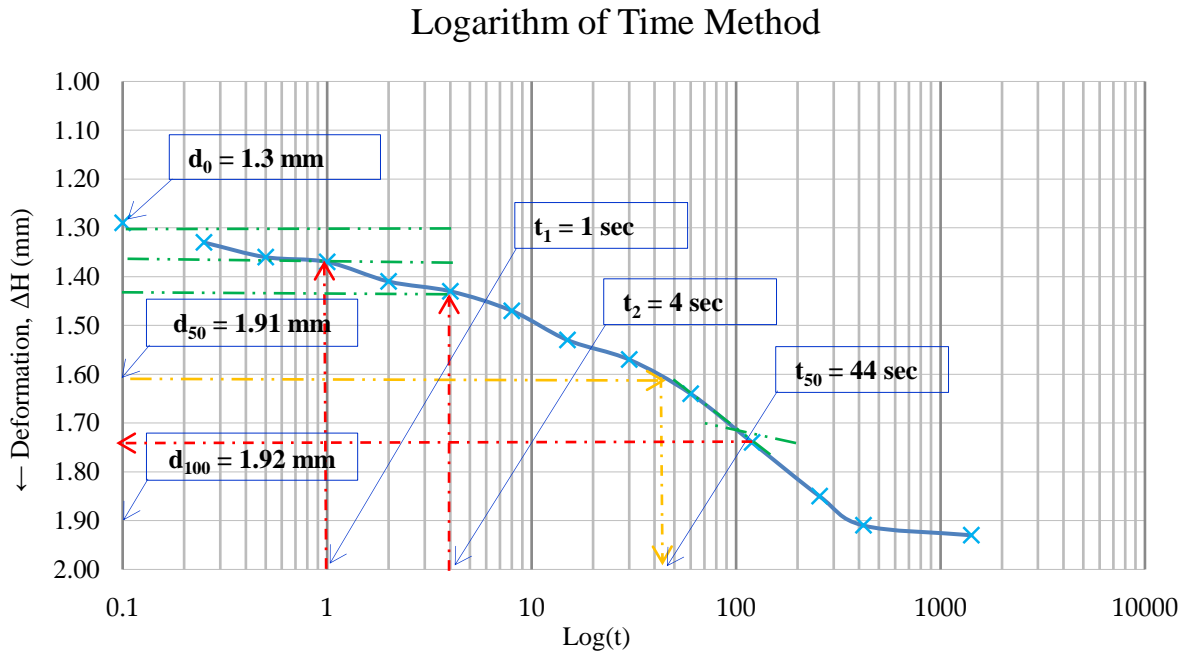
Figure 11.07: Logarithm of time method.

Consequently, final results of the testing are shown as given as under:

Table 11.05: Testing results.

Item	Data
t_1 (minutes)	1.00
t_2 (minutes)	4.00
d_0 (millimetres)	1.30
d_{100} (millimetres)	1.92
d_{50} (millimetres)	1.61
t_{50} (millimetres)	44.0
H_i (millimetres)	20.0
Initial reading (millimetres)	1.15
Final reading (millimetres)	1.93
D_{avg} (millimetres)	0.39
H_{dr} (centimetres)	0.9805
C_v (square centimetres per minute)	0.0043044

(b) Square Root of Time Method
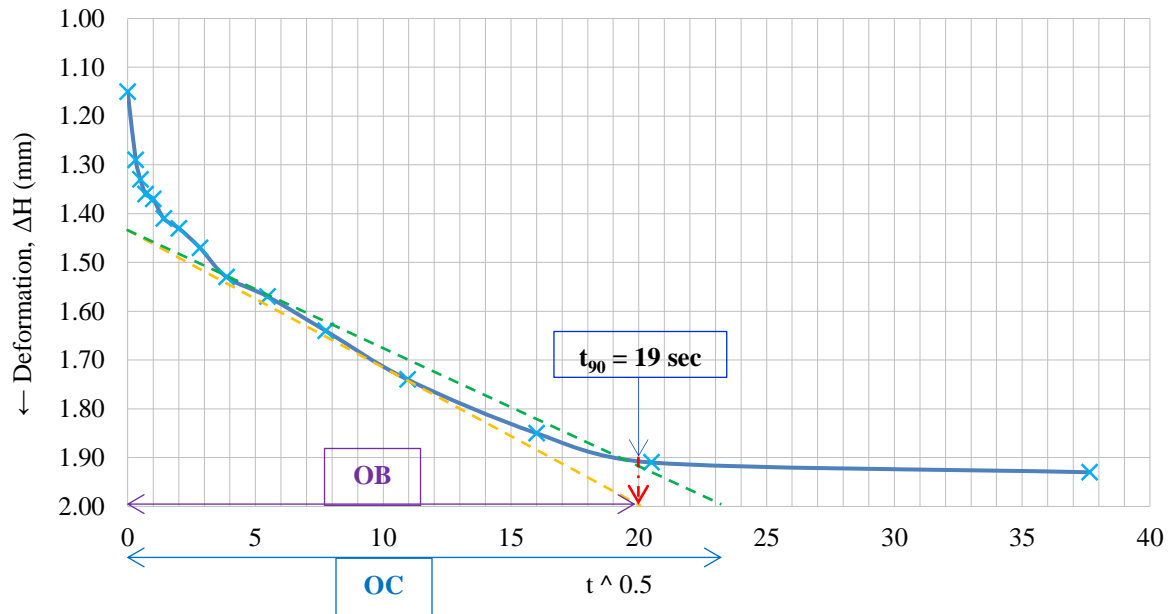
Figure 11.08: Square root of time method.

Consequently, final results of the testing are shown as given as under:

Table 11.06: Testing results.

Item	Data
OB	20.0
OC	23.0
t_{90} (minutes)	19.0
H_i (millimetres)	20.0
Initial reading (millimetres)	1.15
Final reading (millimetres)	1.93
D_{avg} (millimetres)	0.39
H_{dr} (centimetres)	0.9805
C_v (square centimetres per minute)	0.04290792

(B) For normal stress, σ = 200 kpa

Table 11.07: Testing results.

Elapsed Time (min)	Dial Reading (0.02 mm)	Deformation, ΔH (mm)	$\sqrt{t}$
00.00	096.5	1.93	00.0000
00.10	108.0	2.16	00.3162
00.25	110.5	2.21	00.5000
00.50	112.0	2.24	00.7071
01.00	114.5	2.29	01.0000
02.00	116.0	2.32	01.4142
04.00	118.5	2.37	02.0000
08.00	122.0	2.44	02.8284
15.00	125.5	2.51	03.8730
30.00	131.0	2.62	05.4772
60.00	136.5	2.73	7.74597
120.0	142.0	2.84	10.9545
256.0	146.5	2.93	16.0000
420.0	149.5	2.99	20.4939
1417	150.5	3.01	37.6431

(a) Logarithmic Time method.

Logarithm of Time Method

Figure 11.09: Logarithm of time method.
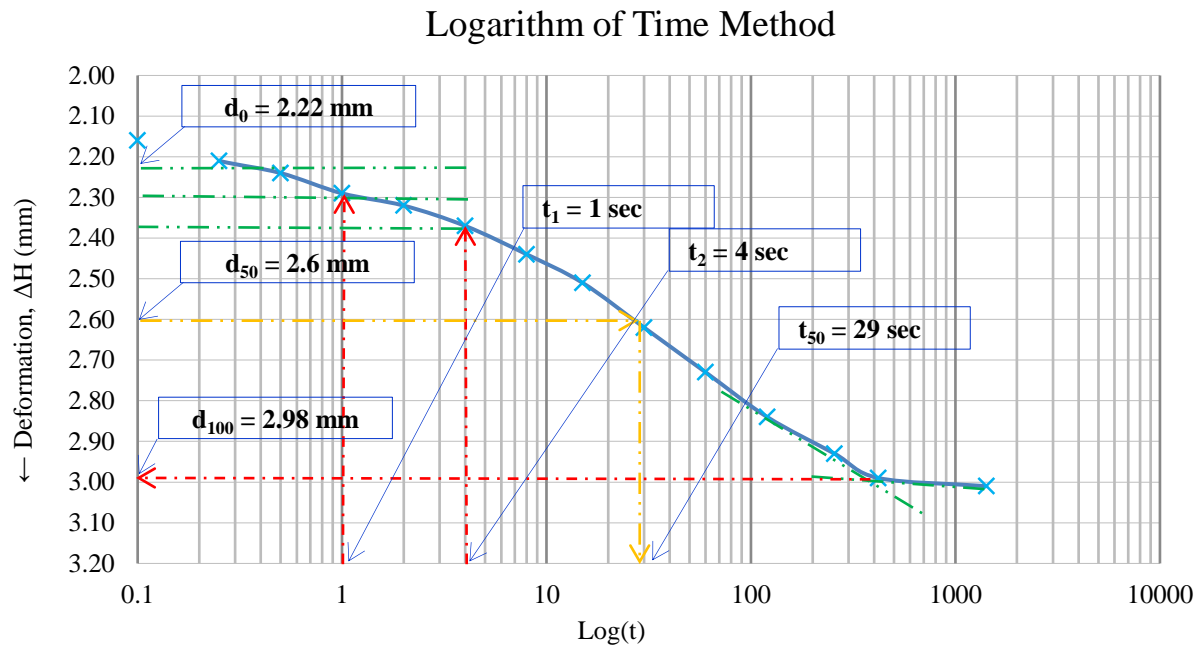
Consequently, final results of the testing are shown as given as under:

Table 11.08: Testing results.

Item	Data
t_1 (minutes)	1.00
t_2 (minutes)	4.00
d_0 (millimetres)	2.22
d_{100} (millimetres)	2.98
d_{50} (millimetres)	2.60
t_{50} (minutes)	29.0
H_i (millimetres)	20.0
Initial reading (millimetres)	1.93
Final reading (millimetres)	3.01
D_{avg} (millimetres)	0.54
H_{dr} (centimetres)	0.973
C_v (cubic centimetres per minute)	0.006431228

(b) Square Root of Time Method

Square Root of Time Method
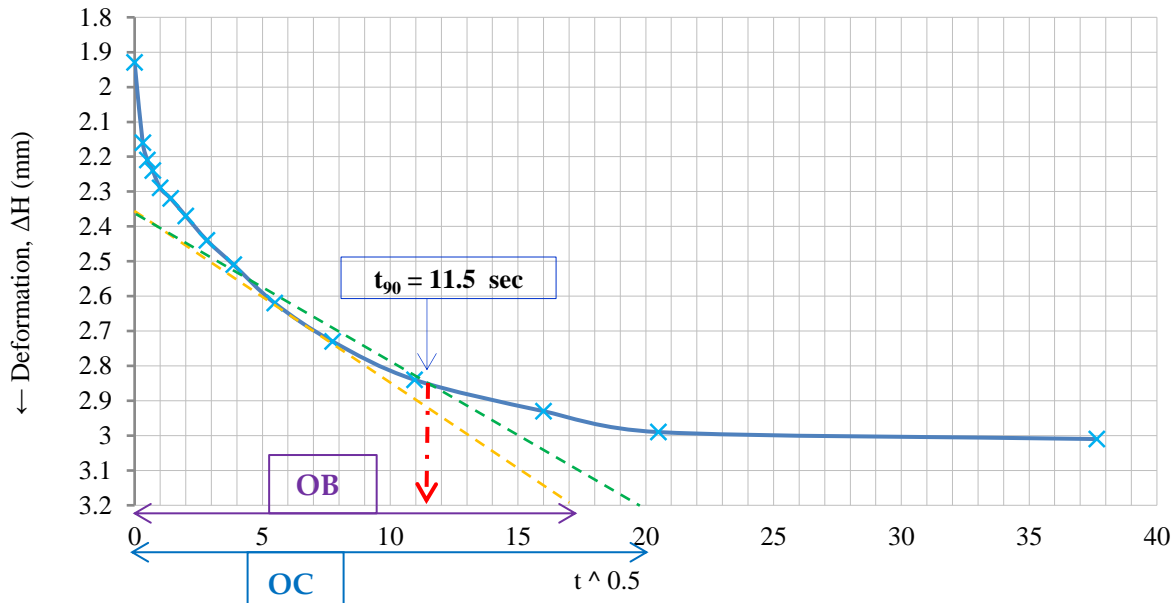
Figure 11.10: Square root of time method.

Consequently, final results of the testing are shown as given as under:

Table 11.09: Testing results.

Item	Data
OB	17
OC	19.55
t_{90} (minutes)	11.5
H_i (millimetres)	20
Initial reading (millimetres)	1.93
Final reading (millimetres)	3.01
D_{avg} (millimetres)	0.54
H_{dr} (centimetres)	0.973
C_v (cubic centimetres per minute)	0.069810973

(2) Factor of compression, C_c, and pre-consolidation pressure, σ'_c

Table 11.10: Testing results.

Pressure (kPa)	Dial Readings (0.001 mm)	Deformation, ΔH (mm)	$e_i = e_o - \sum \Delta H / H_s$
0.00	0.0	0.000	0.820210
25.0	34	0.034	0.789264
50.0	29	0.029	0.762870
100	41	0.041	0.725553
200	56	0.056	0.674583
400	73	0.073	0.608141
800	69	0.069	0.545340
1600	73	0.073	0.478898

$$H_s = \frac{90.0}{1.0 \times 2.7 \times \frac{\pi}{4} \times 6.215^2} = 1.0987 \text{ cm}$$

$$e_0 = \frac{2.0 - 1.0987}{1.0987} = 0.82021$$

Consequently, final results of the testing are shown as given as under:

Table 11.11: Testing results.

Item	Data
H_s (centimetres)	1.09870
e_o (ratio)	0.82021

Void Ratio Versus Effective Stress Plot
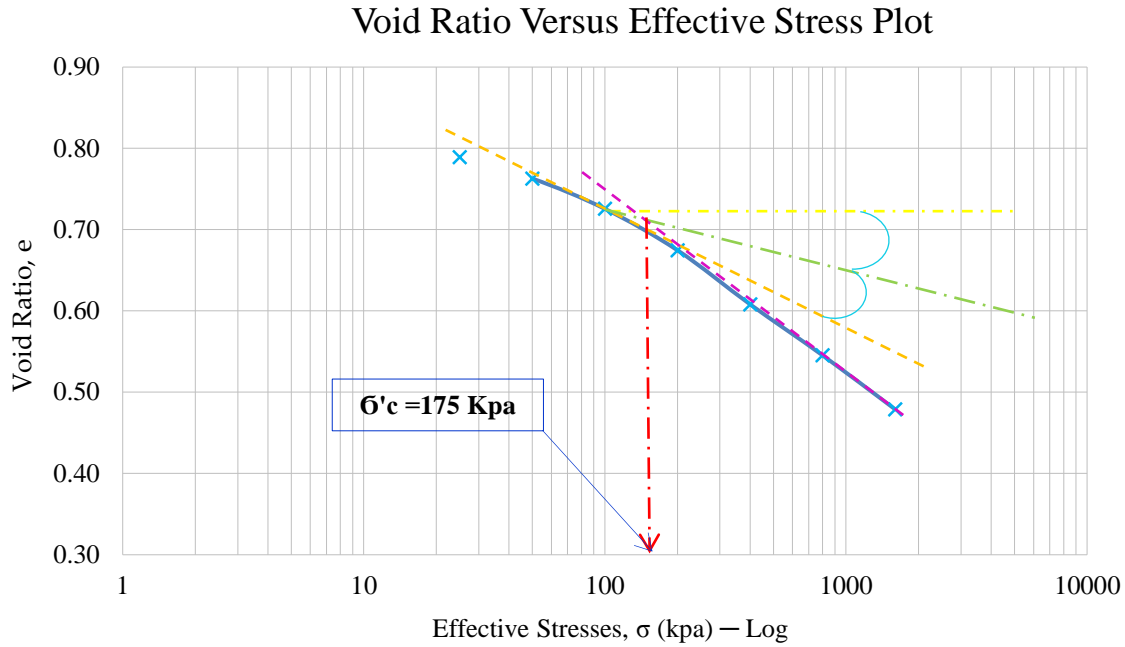
Figure 11.11: Void ratio versus effective stress plot.

From the graph drawn above: $\sigma'_c = 175$ kpa

Void Ratio Versus Effective Stress Plot
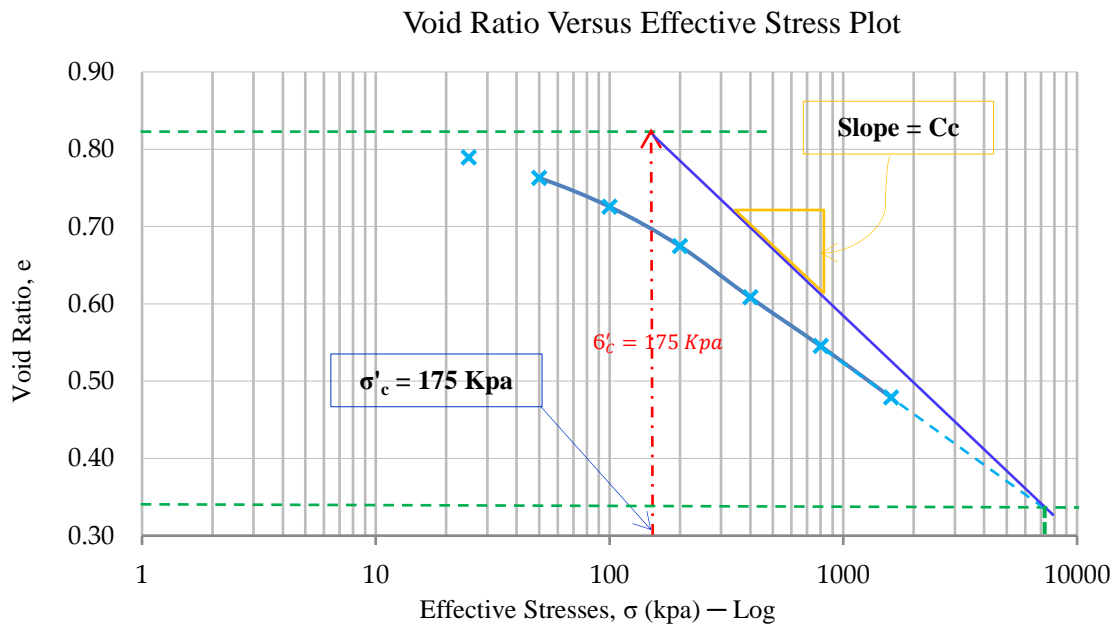
Figure 11.12: Void ratio versus effective stress plot.

C_c = slope

Consequently, final results of the testing are shown as given as under:

Table 11.12: Testing results.

Item	Data
e_o	0.8202
$0.4\ e_o$	0.3281
σ'_c (kilopascals)	175.00
σ (kilopascals)	7200.0

$$C_c = \frac{0.8202 - 0.3281}{7200.0 - 175.00} = 7.0049 \times 10^{-4}$$

Results and Discussion

After all calculations have been evaluated once, the results were as follows:

C_v = ?!

  (A) For stress, σ = 100 kpa
      a. Logarithmic Time method, $C_v = 4.3044 \times 10^{-3}$ mm²/min
      b. Square Root of Time Method, $C_v = 42.9079 \times 10^{-3}$ mm²/min

  (B) For stress, σ = 200 kpa
      a. Logarithmic Time Method, $C_v = 6.4312 \times 10^{-3}$ mm²/min
      b. Square Root of Time Method, $C_v = 69.8110 \times 10^{-3}$ mm²/min

C_c = ?!

$C_c = 7.0049 \times 10^{-4}$

Conclusion

- The results were not obtained exactly, since they are based on graphs, so entrapped approximations are created.
- The value of C_v is smaller than one, which means that this method of approximations, which is based on the graphs shown earlier, is close to be true and, thus it is highly acceptable.

TEST NO. 12

DIRECT SHEAR TEST ON SAND

(ASTM D2216)

Introduction

This test method covers the determination of the consolidated-drained shear strength of a sandy soil. The shear strength is one of the most important engineering properties of a soil, since it is required whenever a structure is dependent upon the shearing resistance of soil. 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. in addition, in many engineering problems, for instance, design of foundation, retaining walls, slab bridges, pipes and sheet piling, the value of cohesion and angle internal friction of the soil involved are required for the design. Direct shear test is used to predict these parameters quickly. However, this test is performed to cover the laboratory procedure for determining these values for a cohesionless soil.

Definition

There are several laboratory test methods now attainable to determine the shear strength parameters (i.e. c, Ø, c′and Ø′) of various soil specimens in the laboratory. They are as follows: (1) Direct shear test, (2) Triaxial test. (3) Direct simple shear test, (4) Plane strain triaxial test and (5) Torsional ring shear test. The direct shear test and the triaxial test are the two commonly used tests for determining the shear strength parameters. The direct shear test will be described in detail in the experiment that follows.

Shear strength is a term used in soil mechanics to describe the internal resistance per unit area that a soil mass can develop to resist failure and sliding along any plane inside it or it is the magnitude of the shear stress that a soil mass can sustain. Moreover, the shear resistance of soil is a result of friction and interlocking of particles and, probably, cementation or bonding at particle contacts. However, shear strength is defined as the internal resistance per unit area that a soil mass can offer to resist failure in shear.

The strength of a soil is dependent upon its resistance to shearing stresses. It is, basically, made up of the following two components: (1) Frictional, which is offered due to friction between individual particles. (2) Cohesive, which is offered due to adhesion between the soil particles. The two components are, then, combined in Mohr-Coulomb's shear strength equation, which comes as follows:

$$\tau_f = c + \sigma \tan \emptyset$$

Where:

τ_f = shearing resistance of soil at failure.

c = cohesion of soil.

σ = total normal stress on the failure plane.

Ø = angle of soil shearing resistance or angle of internal friction.

The total normal stress at a point, in saturated soils, is the sum of the effective stress (σ′) and pore water pressure (u). This may be expressed as:

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

Where:

σ = total normal stress at a point.

σ' = cohesion of soil.

u = cohesion of soil.

A diagram of the direct shear testing machine is shown in Figure 12.01. The testing machine used in direct shear test consists of a metal shear box in which a soil specimen is placed. The soil specimens may be square or circular in plan. The size of the specimens generally used is about 51 mm × 51 mm or 102 mm × 102 mm (2 in. × 2 in. or 4 in. × 4 in.) across and about 25 mm (1 in.) high. The box is split horizontally into halves. Normal force on the specimen is applied from the top of the shear box. The normal stress on the soil specimens can be as great as 1050 kN/m² (150 lb/in.²). Shear force is applied by moving one half of the box relative to the other to cause failure in the soil specimen.
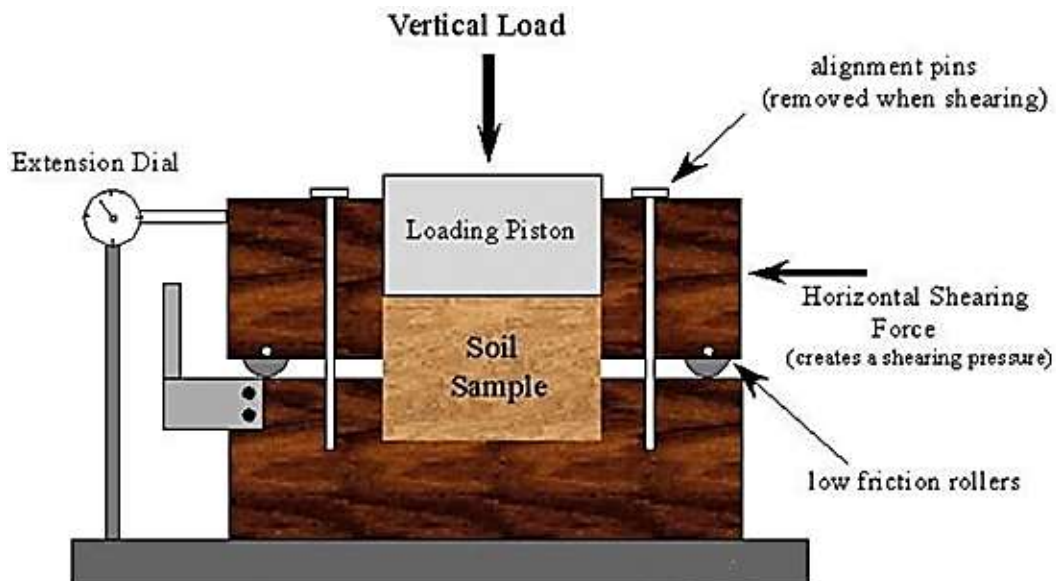
Figure 12.01: Drained direct shear test apparatus.

Significance

- The direct shear test is one of the oldest strength test for soils. In this laboratory test, a direct shear device will be used to determine the shear strength of a cohesionless soil (i.e. angle of internal friction or angle of repose, $\emptyset$). From the plot of shear stress versus its corresponding 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 maximum shear stresses versus the vertical (normal) confining stresses for each of the tests performed is produced. From the plot, a straight-line approximation of the Mohr-Coulomb failure envelope curve can be drawn, $\emptyset$ may be determined and for cohesionless soils (c = 0), the shear strength can be computed from the following equation:

$$\tau_f = c + \sigma \tan \emptyset$$

- In addition, knowing these information helps in determination of the bearing capacity of soil, as well, lateral earth pressure coefficient.

Purpose

Determination of the shear strength parameters of soil (sand), which are:

- Cohesion, c.
- Angle of internal friction, $\emptyset$.

Those parameters are known as Shear Strength Parameters.

Standard Reference

ASTM D 3080 — Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions.

Equipment and Materials

(1) Drained direct shear test apparatus.
(2) Metal shear box with square section (51 mm × 51 mm) or (102 mm × 102 mm) and height of 25 mm, the box is divided into two parts.

(3) Porous stones and filter papers.
(4) Motor generates horizontal force.
(5) Dial gages for horizontal displacement, vertical displacement, if required, and shear force.

Figure 12.02: Drained direct shear testing machine.

Procedure

(1) Prepare soil sample from the field and ensure, during the test, that it has the same field density, ρ_{field}.
(2) Prepare shear box by putting porous stone and filter paper on its bottom, then fill it with soil on 3 layers, each layer is compacted 20 blows, then put filter paper and porous stone over the sample. Porous stone is used to allow water to drain, thus sand is preferred in this test, since it has high coefficient of permeability.
(3) Disconnect the bolts that joining the two parts of the box and make a little space of 0.6 mm between the two parts.
(4) Adjust the dials at zero reading.
(5) Put vertical load, which cause vertical stress, σ, above the sample.
(6) Turn on a motor, which generates horizontal force causing shear stress, τ, then take readings from horizontal displacement and shear force dial gages.

(7) Continue in taking readings until the sample fail or the horizontal displacement become equal to 15% of the box's dimension if the box is square in shape, or of the box's diameter if the box is circular in cross section.

(8) Prepare other samples from the same soil and expose it to other vertical stresses, then make the same procedure and calculations.

Data Analysis

(1) Compute the cross-sectional area of the specimen.

$$\text{For square box: } A = D \times D$$

$$\text{For circular box: } A = (\pi/4)D^2$$

(2) Compute the shear stress value from the dial readings obtained at each pressure increment.

(3) Draw the shear stress on the y-axis and the corresponding displacement on the x-axis and obtain the value of shear stress at the peak displacement.

(4) Draw the normal stresses (pressure increments) on the x-axis and their corresponding shear stresses, then develop the equation, which defines these points in virtue of linear regression.

(5) Evaluate the values of soil internal friction, Ø and cohesion, c by equating the equation obtained from the last step with the following equation:

$$\tau = c + \sigma \tan \emptyset$$

Measurements

Measurements of the testing are shown as given as under:

Shear box inside dimensions are 51 mm × 51 mm

Table 12.01: Testing measurements.

Normal Stress, σ = 13.89 kPa	
Horizontal Dial Reading (0.00254 mm)	**Horizontal shear force (N)**
00.0	00.00
10.0	26.04
16.0	26.44
26.0	27.10
33.0	27.36
41.0	30.01
48.0	31.33
54.0	38.59
60.0	40.57
67.0	43.21
73.0	45.86
81.0	47.17
88.0	49.82
97.0	50.48
104	51.14
111	51.80
119	53.78
126	54.44
134	55.10
142	56.43
149	57.08
157	57.75
176	53.78

Table 12.02: Testing measurements.

Normal Stress, σ = 29.48 kPa	
Horizontal Dial Reading (0.00254 mm)	**Horizontal Shear Force (N)**
00.0	00.00
04.5	31.33
11.0	36.61
14.0	38.59
20	41.23
27.0	42.55
34.0	45.19
41.0	47.18
47.0	51.14
53.0	54.44
59.0	59.07
67.0	62.37
74.0	64.35
79.0	68.32
85.0	72.28
91.0	76.24
98.0	78.88
105	84.17
112	85.49
118	92.09
124	95.40
133	96.72
138	100.0
145	102.0
152	102.7

Table 12.03: Testing measurements.

Normal Stress, σ = 44.9 kPa	
Horizontal Dial Reading (0.00254 mm)	**Horizontal Shear Force (N)**
00.0	000.00
01.0	041.89
05.0	049.82
08.0	056.42
13.0	061.71
19.0	065.67
26.0	068.32
32.0	074.92
37.0	075.58
40.0	077.56
49.0	082.85
59.0	082.36
66.0	092.09
72.0	094.74
80.0	097.38
86.0	101.34
92.0	103.98
99.5	109.25
107	115.87
113	119.83
120	123.80
126	129.08
131	130.42
136	129.96
140	128.47

Computation

After all calculations have been evaluated once, the results were arranged in the following tables:

Area $= 51.0{\times}10^{-3} \times 51.0{\times}10^{-3} = 0.002601 \text{ m}^2$

Table 12.04: Testing results.

	Normal Stress, σ = 13.89 kPa		
Horizontal Dial Reading (0.00254 mm)	Horizontal Shear Force (N)	Horizontal Displacement, ΔH (mm)	Shear Stress, τ (kPa)
00.0	00.000	0.000	00.000
10.0	26.040	0.025	10.012
16.0	26.440	0.041	10.165
26.0	27.100	0.066	10.419
33.0	27.360	0.084	10.519
41.0	30.010	0.104	11.538
48.0	31.330	0.122	12.045
54.0	38.590	0.137	14.837
60.0	40.570	0.152	15.598
67.0	43.210	0.170	16.613
73.0	45.860	0.185	17.632
81.0	47.170	0.206	18.135
88.0	49.820	0.224	19.154
97.0	50.480	0.246	19.408
104	51.140	0.264	19.662
111	51.800	0.282	19.915
119	53.780	0.302	20.677
126	54.440	0.320	20.930
134	55.100	0.340	21.184
142	56.430	0.361	21.696
149	57.080	0.378	21.945
157	57.750	0.399	22.203
176	53.780	0.447	20.677

ΔH-Displacement Versus Shear Stress Plot
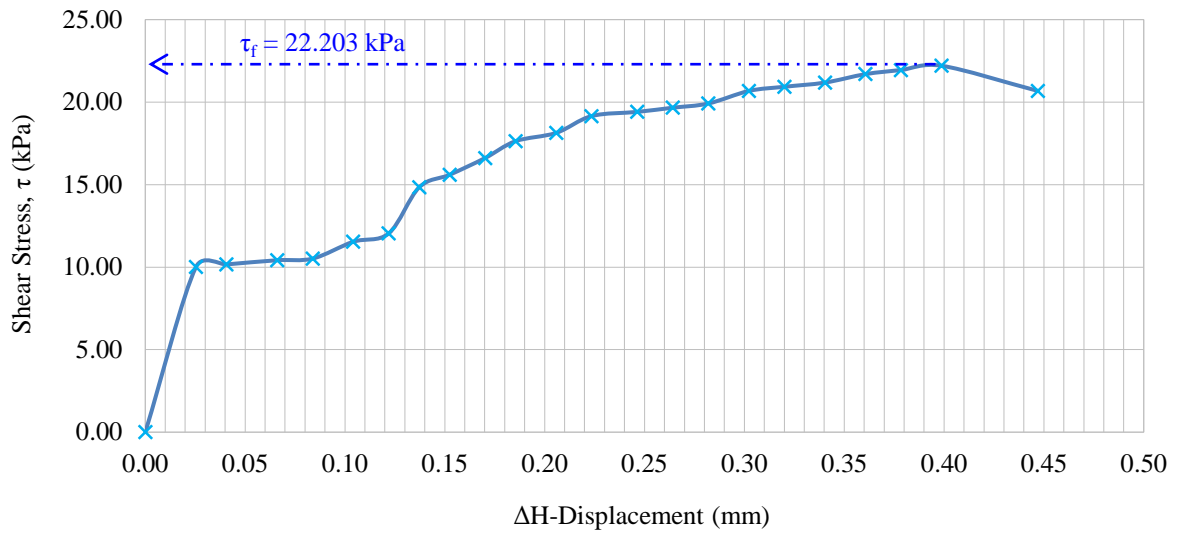
Figure 12.03: ΔH-displacement versus shear stress plot.

Table 12.05: Testing results.

Horizontal Dial Reading (0.00254 mm)	Horizontal Shear Force (N)	Horizontal Displacement, ΔH (mm)	Shear Stress, τ (kPa)
	Normal Stress, σ = 29.48 kPa		
00.0	000.000	0.000	00.000
04.5	031.330	0.011	12.045
11.0	036.610	0.028	14.075
14.0	038.590	0.036	14.837
20.0	041.230	0.051	15.852
27.0	042.550	0.069	16.359
34.0	045.190	0.086	17.374
41.0	047.180	0.104	18.139
47.0	051.140	0.119	19.662
53.0	054.440	0.135	20.930
59.0	059.070	0.150	22.710
67.0	062.370	0.170	23.979
74.0	064.350	0.188	24.740
79.0	068.320	0.201	26.267
85.0	072.280	0.216	27.789
91.0	076.240	0.231	29.312
98.0	078.880	0.249	30.327
105	084.170	0.267	32.361
112	085.490	0.284	32.868
118	092.090	0.300	35.406
124	095.400	0.315	36.678
133	096.720	0.338	37.186
138	100.020	0.351	38.454
145	102.000	0.368	39.216
152	102.660	0.386	39.469

ΔH-Displacement Versus Shear Stress Plot
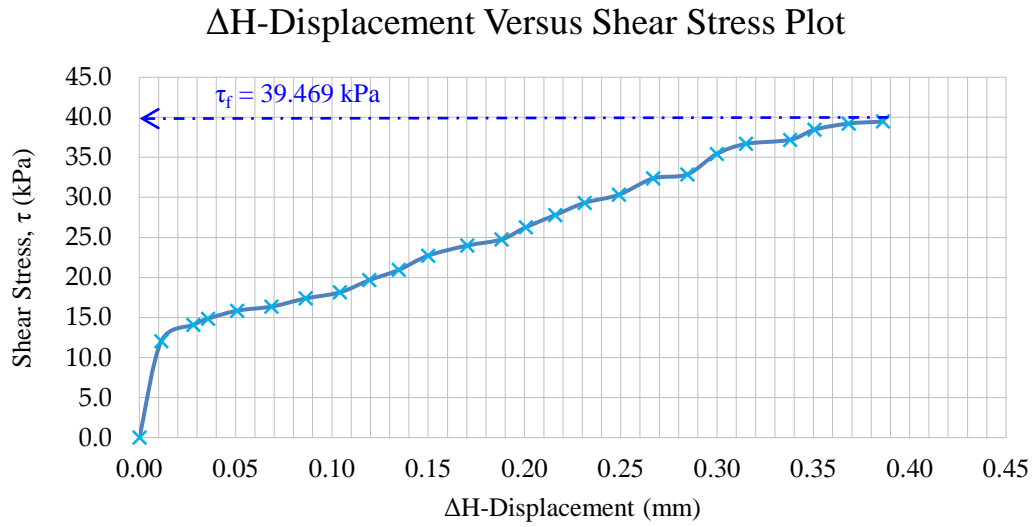
Figure 12.04: ΔH-displacement versus shear stress plot.

Table 12.06: Testing results.

Horizontal Dial Reading (0.00254 mm)	Horizontal Shear Force (N)	Horizontal Displacement (mm)	Shear stress, τ (kPa)
Normal Stress, σ = 44.9 kPa			
00.0	00.000	0.000	00.000
01.0	41.890	0.003	16.105
05.0	49.820	0.013	19.154
08.0	56.420	0.020	21.692
13.0	61.710	0.033	23.725
19.0	65.670	0.048	25.248
26.0	68.320	0.066	26.267
32.0	74.920	0.081	28.804
37.0	75.580	0.094	29.058
40.0	77.560	0.102	29.819
49.0	82.850	0.124	31.853
59.0	82.360	0.150	31.665
66.0	92.090	0.168	35.406
72.0	94.740	0.183	36.424
80.0	97.380	0.203	37.439
86.0	101.340	0.218	38.962
92.0	103.980	0.234	39.977
99.5	109.250	0.253	42.003
107	115.870	0.272	44.548
113	119.830	0.287	46.071
120	123.800	0.305	47.597
126	129.080	0.320	49.627
131	130.420	0.333	50.142
136	129.960	0.345	49.965
140	128.470	0.356	49.393

ΔH-Displacement Versus Shear Stress Plot
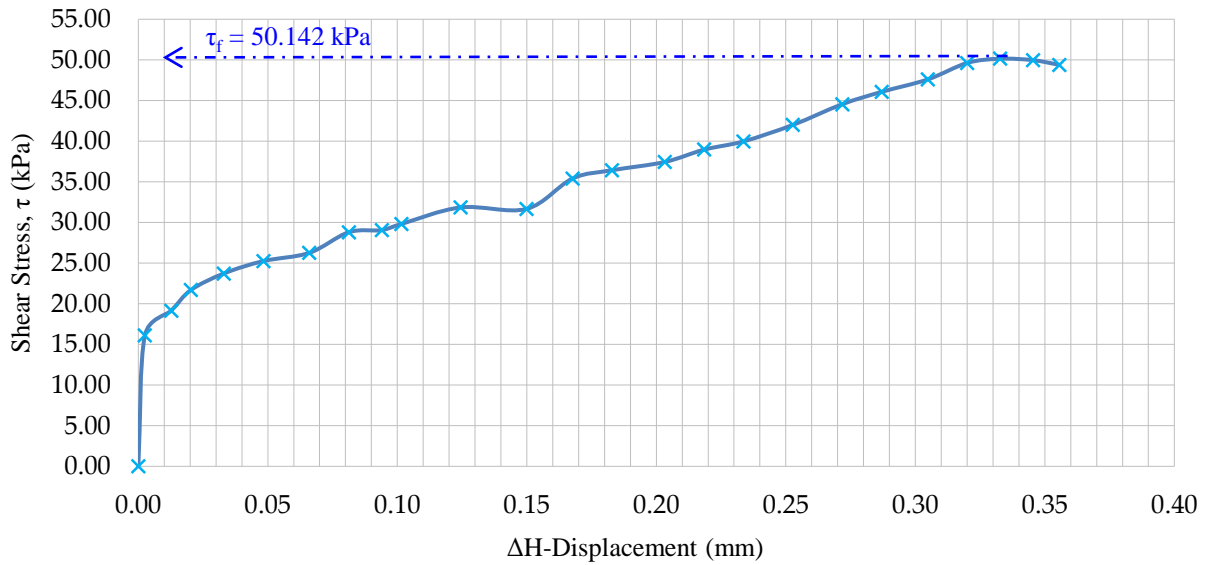
Figure 12.05: ΔH-displacement versus shear stress plot.

Table 12.07: Testing results.

Normal Stress, σ_f (kPa)	Shear Stress, τ_f (kPa)
13.89	22.203
29.48	39.469
44.90	50.142
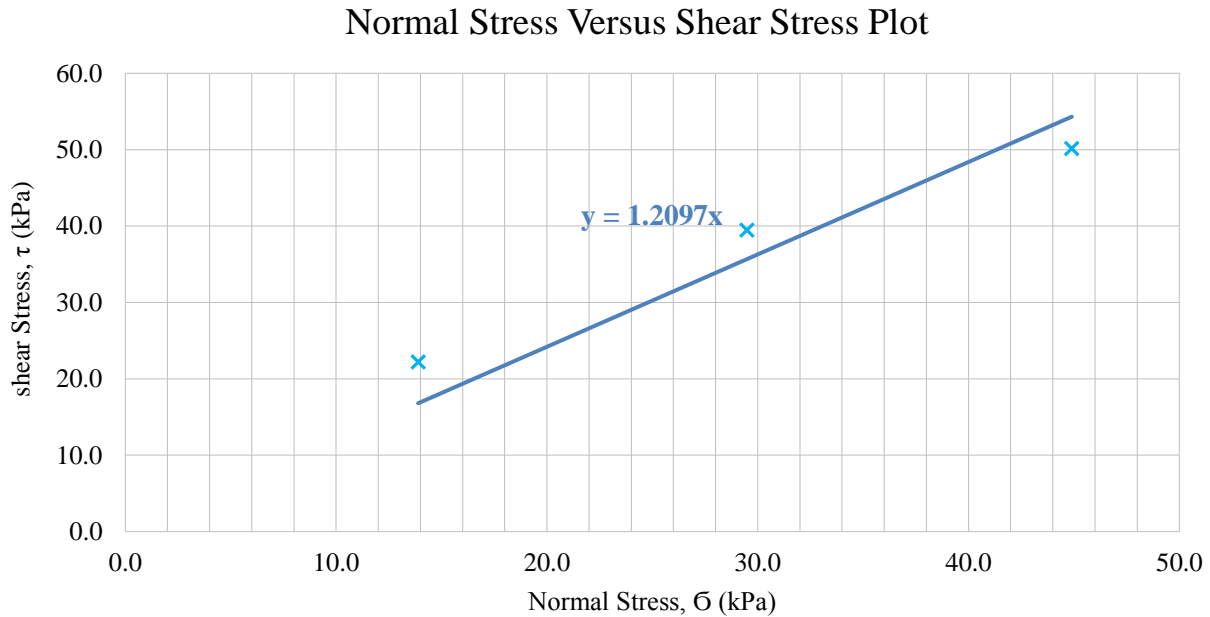
Figure 12.06: Normal stress versus shear stress plot.

$\tau = c + \sigma \tan \emptyset$

$y = 0.0 + 1.2097x$

Comparing the two past equations, one to another, leads to the results that follow:

$c = 0.0$

$\text{Slope} = 1.2097$

$\tan \emptyset = 1.2097$

$\emptyset = \tan^{-1}(1.2097) = 50.421°$

Results and Discussion

After all calculations have been evaluated once, all results were arranged in the previous tables, for more details refer to the past section of computation.

Conclusion

- Sandy soil since cohesion, c = 0.0; and for clay, Ø = 0.0.
- Since the soil is sand, then c = 0.0, and Ø = 50.421° which is larger than expected (Ø ≥ 45°).

TEST NO. 13

UNCONFINED COMPRESSION TEST

(ASTM D2216)

Introduction

This test method covers the determination of the unconfined compression strength, which is then used to calculate the unconsolidated undrained shear strength of the clay under unconfined conditions. It is not always possible to conduct the bearing capacity test in the field. Sometimes, it is cheaper to take the undisturbed soil sample for testing its strength in the laboratory. In addition, to choose the best material for the embankment, one has to conduct strength tests on the samples selected. Under these conditions, it is easy to perform the unconfined compression test on undisturbed and remoulded soil samples. Furthermore, in this test method, the unconfined compression strength is taken as the maximum load attained per unit area or the load per unit area at 15% axial strain, whichever takes place first during the performance of a test.

Definition

The unconfined compression test is a special type of unconsolidated-undrained test, that is commonly used for clay specimens. The unconfined compression strength, q_u, is defined as the compressive stress, at which an unconfined cylindrical specimen of soil will fail in a simple compression test.

With respect to shear strength, cohesive soil can fail under conditions of rapid loading, where excess pore pressures do not have time to dissipate. Under these conditions, the state of stress in a soil element can be demonstrated in virtue of a Mohr circle, with minor and major total principal stress (σ_3 and σ_1), respectively. If identical specimens of cohesive soil are subjected to different states of stress and rapidly loaded to failure without excess pore pressure dissipation, the Mohr circles of each specimen possess the same diameter, thus producing a 'total stress envelope' with a friction angle of zero and cohesion, c, equal to the undrained shear strength, c_u (see Figure 13.01). It is important to note, however, that if pore pressure is measured within each specimen during shearing and total stresses are converted to effective stresses, each Mohr circle overlaps one another and is tangent to the effective stress envelope with an effective cohesion, c', and effective friction angle, $\emptyset'$. This points out an important point regarding the strength of soil, which is: even under rapid undrained loading, the strength of soil is still controlled by effective stress.
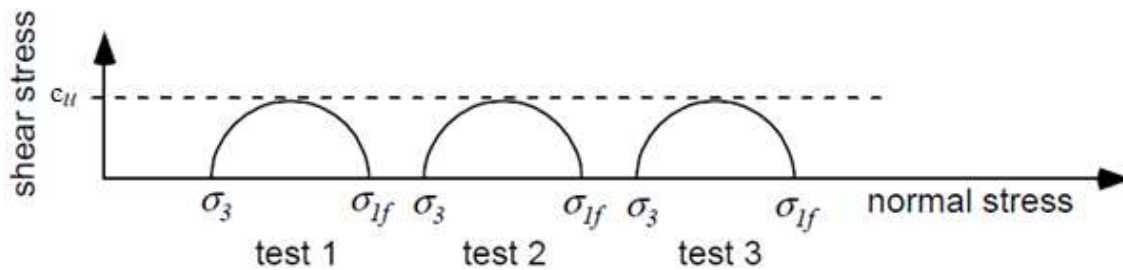
Figure 13.01: Total stress Mohr-Coulomb failure envelope.

As demonstrated in Figure 13.02, the undrained shear strength, c_u, is defined as the intercept of the total stress failure envelope and it is half of the diameter of the Mohr

circle. The unconfined compression strength, c_u, is defined as σ_1 at failure. By inspection, c_u is equal to one half of q_u.
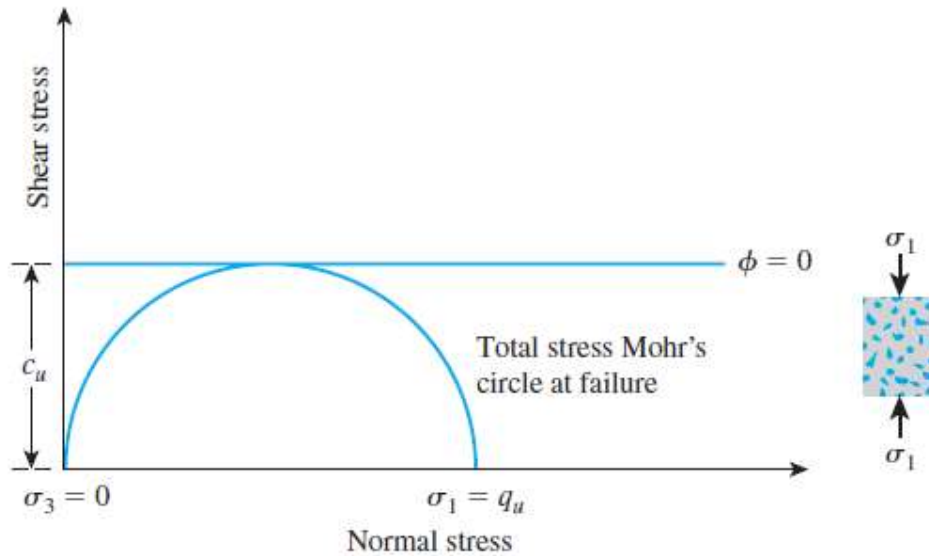
Figure 13.02: Mohr circle from an unconfined compression test.

c_u of clays is commonly determined from an unconfined compression test. c_u of a cohesive soil is equal to one half of the q_u when the soil is under the $\emptyset = 0$ condition. 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. However, this is expressed as:

$$\tau = c = \frac{\sigma_1}{2} = \frac{q_u}{2}$$

Then, as time passes, the pore water in the soil slowly dissipates and the intergranular stress increases, so that the drained shear strength, τ, given by the following equation must be used.

$$\tau = c + \sigma' \tan \emptyset$$

Where:

σ' = intergranular pressure acting perpendicular to the shear plane and $\sigma' = (\sigma - u)$.

σ = total pressure.

u = pore water pressure.

c and $\emptyset'$ = drained shear strength parameters.

A typical configuration for the unconfined compressive strength testing is shown in Figure 13.03. Axial deformation, ΔL, is measured using a deformation indicator and applied load, P, is measured using a load cell.
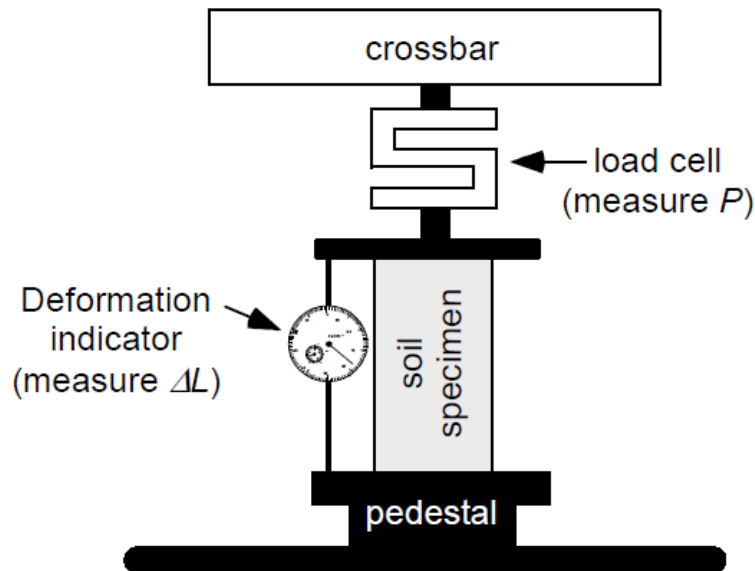
Figure 13.03: Typical configuration for an unconfined compression test.

Generally speaking, the unconfined compression strength is determined by applying an axial stress to a soil specimen that is cylindrical in shape with no confining pressure and, thereafter, observing the axial strains at various stress levels. The stress, at which failure in the soil specimen takes place, is referred to as the unconfined compression strength (see Figure 13.04). For saturated clay specimens, the unconfined compression strength decreases with the increase in water content. For unsaturated soils, with constant dry unit weight, the unconfined compression strength decreases with the increase in the degree of saturation. Furthermore, Table

13.01 illustrates the relationships of consistency and unconfined compression strength of clays.

σ_1 at failure $= q_u$
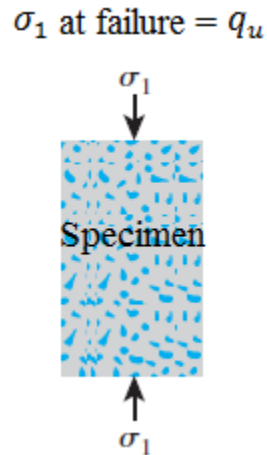
Figure 13.04: Definition of unconfined compression strength.

Table 13.01: General relationship of consistency and unconfined compression strength of clays.

Consistency	Unconfined Compression Strength, qu kN/m^2 (ton/ft^2)
Very soft	0 - 25 (0 - 0.25)
Soft	25 - 50 (0.25 - 0.5)
Medium	50 - 100 (0.5 - 1)
Stiff	100 - 200 (1 - 2)
Very stiff	200 - 400 (2 - 4)
hard	> 400 (> 4)

Significance

- The undrained shear strength, c_u, is necessary for the estimation of the bearing capacity of spread footings, foundations, dams and other structures when placed on deposits of cohesive soil.

Purpose

Determination of the unconfined compression strength of a cohesive soil in the undisturbed or remoulded conditions at its natural water content. The applying load rate is fast, so pore water is not allowed to dissipate and test considered being undrained test shear.

Standard Reference

ASTM D2166 – Standard Test Method for Unconfined Compressive Strength of Cohesive Soil.

Equipment and Materials

(1) Unconfined compression device.
(2) Load and deformation dial gauges.
(3) Trimming equipment.
(4) Balance sensitive to an accuracy of 0.01 gm.
(5) Oven and moisture can.

Figure 13.05: Unconfined compression device.

Procedure

(1) Cut a soil specimen such that the ratio L/d is approximately between 2 and 2.5 so that:

$$2.0 \leq L/d \leq 2.5$$

Where:

L = length of the soil specimen.

d = diameter of the soil specimen.

(2) Measure the exact diameter of the top of the specimen at three locations and then make the same measurements on the bottom of the specimen. Take the average of the measurements and record it as the diameter.

(3) Measure the exact length of the specimen at three locations and then take the average of the measurements and record the it as the length.

(4) Weigh the sample and record the mass.

(5) Calculate the deformation, ΔL, corresponding to 15% strain, ε.

$$\varepsilon = \frac{\Delta L}{L_o}$$

Where:

L_o = average length of the soil specimen.

(6) Carefully, place the specimen in the unconfined compression device and centre 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 onto the data sheet at every 20 to 50 divisions on deformation the dial.

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

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

(10) Remove the sample from the unconfined compression device. Obtain a sample for water content measurement and measure the water content.

Data Analysis

(1) Convert the dial readings to the appropriate load and length units and record these values onto the data sheet in the deformation and total load columns.

(2) Compute the cross-sectional area of the specimen, A_o:

$$A_o = \frac{\pi}{4}D^2$$

(3) Compute the strain, ε:

$$\varepsilon = \frac{\Delta L}{L_o}$$

(4) Compute the corrected area, A':

$$A_o = \frac{A_o}{1 - \varepsilon}$$

(5) Compute the specimen stress:

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

(6) Plot the stress versus strain. Show q_u as the peak stress or as the stress at 15% strain of the test. The strain is plotted on the abscissa.

(7) Draw Mohr-Coulomb's circle using q_u from obtained the last step and compute the undrained shear strength, c_u.

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

Measurements

Measurements of the testing are shown as given as under:

Diameter, D = 7.3 cm

Length, L = 14.5 cm

Mass of sample = 1221.4 gm

Percent water content = 35 %

Deformation-dial readings: each reading = 0.01 mm

Load-dial readings: each reading = 1.403 N

Table 13.02: Testing measurements.

Deformation Dial Reading	Load Dial Reading
0000.0	00.0
020.0	04.0
040.0	09.0
060.0	12.0
080.0	19.0
100.0	21.0
120.0	24.0
140.0	26.0
160.0	29.0
180.0	33.0
200.0	36.0
250.0	45.0
300.0	54.0
350.0	64.0
400.0	74.0
450.0	84.0
500.0	93.0
550	102
600.0	112
650.0	120
700.0	129
750.0	138
800.0	144
850.0	152
900.0	160
950.0	166
*	*

* To be continued.

Deformation Dial Reading	Load Dial Reading
*	*
1000	171
1100	182
1200	192
1300	202
1400	209
1500	217
1600	223
1700	229
1800	234
1900	240
2000	243
2200	250
2400	253
2600	255
2800	256
3000	254

Computation

After all calculations have been evaluated once, the results were arranged in the following table:

$$\text{Area} = \frac{\pi}{4} \times 7.3 = 41.854 \text{ cm}^2$$

Table 13.03: Testing results.

Deformation Dial Reading	Load Dial Reading	Sample Deformation, ΔL (mm)	Strain, ε	%Strain	Corrected Area, A' (cm²)	Load (kN)	Stress (kPa)
000.0	00.0	0.0	0.000000	0.000000	41.83265	0.000000	0.000000
020.0	04.0	0.2	0.001379	0.137931	41.89043	0.005612	1.339685
040.0	09.0	0.4	0.002759	0.275862	41.94837	0.012627	3.010129
060.0	12.0	0.6	0.004138	0.413793	42.00647	0.016836	4.007954
080.0	19.0	0.8	0.005517	0.551724	42.06473	0.026657	6.337138
100.0	21.0	1.0	0.006897	0.689655	42.12315	0.029463	6.994490
120.0	24.0	1.2	0.008276	0.827586	42.18174	0.033672	7.982601
140.0	26.0	1.4	0.009655	0.965517	42.24049	0.036478	8.635790
160.0	29.0	1.6	0.011034	1.103448	42.29940	0.040687	9.618812
180.0	33.0	1.8	0.012414	1.241379	42.35848	0.046299	10.93028
200.0	36.0	2.0	0.013793	1.379310	42.41772	0.050508	11.90729
250.0	45.0	2.5	0.017241	1.724138	42.56656	0.063135	14.83207
300.0	54.0	3.0	0.020690	2.068966	42.71644	0.075762	17.73603
350.0	64.0	3.5	0.024138	2.413793	42.86738	0.089792	20.94646
400.0	74.0	4.0	0.027586	2.758621	43.01939	0.103822	24.13377
450.0	84.0	4.5	0.031034	3.103448	43.17249	0.117852	27.29794
500.0	93.0	5.0	0.034483	3.448276	43.32667	0.130479	30.11517
550.0	102	5.5	0.037931	3.793103	43.48197	0.143106	32.91158
600.0	112	6.0	0.041379	4.137931	43.63838	0.157136	36.00867
650.0	120	6.5	0.044828	4.482759	43.79592	0.168360	38.44194
700.0	129	7.0	0.048276	4.827586	43.95460	0.180987	41.17590
750.0	138	7.5	0.051724	5.172414	44.11443	0.193614	43.88904
800.0	144	8.0	0.055172	5.517241	44.27543	0.202032	45.63072
*	*	*	*	*	*	*	*

* To be continued.

Deformation Dial Reading	Load Dial Reading	Sample Deformation, ΔL (mm)	Strain, ε	%Strain	Corrected Area, A' (cm^2)	Load (kN)	Stress (kPa)
*	*	*	*	*	*	*	*
850.0	152	8.5	0.058621	5.862069	44.43761	0.213256	47.98998
900.0	160	9.0	0.062069	6.206897	44.60099	0.224480	50.33072
950.0	166	9.5	0.065517	6.551724	44.76557	0.232898	52.02615
1000	171	10	0.068966	6.896552	44.93136	0.239913	53.39544
1100	182	11	0.075862	7.586207	45.26667	0.255346	56.40927
1200	192	12	0.082759	8.275862	45.60702	0.269376	59.06459
1300	202	13	0.089655	8.965517	45.95253	0.283406	61.67364
1400	209	14	0.096552	9.655172	46.30331	0.293227	63.32743
1500	217	15	0.103448	10.34483	46.65949	0.304451	65.24953
1600	223	16	0.110345	11.03448	47.02120	0.312869	66.53787
1700	229	17	0.117241	11.72414	47.38855	0.321287	67.79845
1800	234	18	0.124138	12.41379	47.76169	0.328302	68.73752
1900	240	19	0.131034	13.10345	48.14075	0.336720	69.94490
2000	243	20	0.137931	13.79310	48.52587	0.340929	70.25716
2200	250	22	0.151724	15.17241	49.31491	0.350750	71.12453
2400	253	24	0.165517	16.55172	50.13004	0.354959	70.80765
2600	255	26	0.179310	17.93103	50.97256	0.357765	70.18777
2800	256	28	0.193103	19.31034	51.84388	0.359168	69.27876
3000	254	30	0.206897	20.68966	52.74552	0.356362	67.56252

Axial Stress Versus Percent Axial Strain Plot
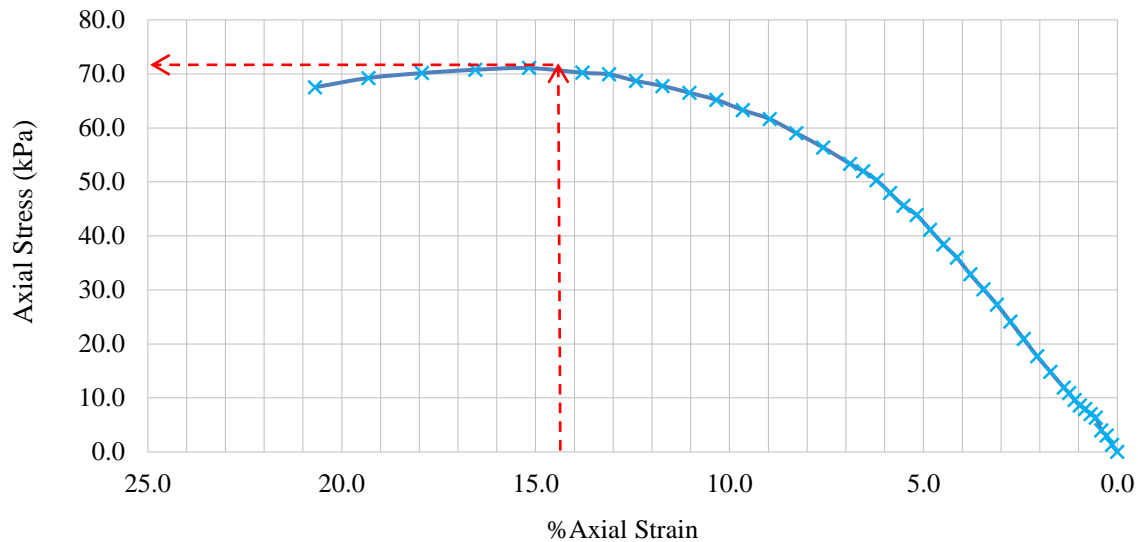
Figure 13.06: Axial stress versus percent axial strain plot.

From the graph drawn above: At 15% strain, the value of q_u is 72.0 kPa.

Therefore, the value of q_u can be computed from as follows:

$$c = \tau = \frac{72.0}{2} = 36.0 \text{ kPa}$$

Consequently, Mohr's circle of the tested specimen can be drawn as given as under:
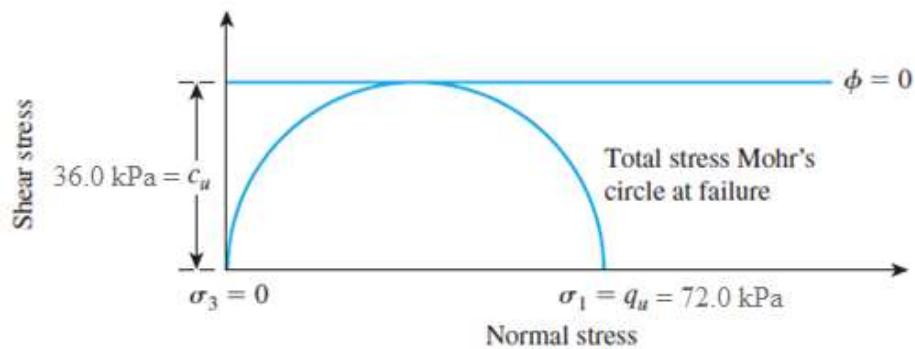
Figure 13.07: Unconfined compression device.

Results and Discussion

After all calculations have been evaluated once, the results were as follows:

- Unconfined compression strength, q_u = 72.0 kPa
- Cohesion, c = 36.0 kPa

Conclusion

- For Sandy soils, the value of cohesion, c is 0.0, while the value of angle of friction, Ø is 0.0.
- The results in this experiment are convincing to an extent since the value of cohesion, c is 0.0 and the value of angle of friction, Ø is 50.405˚, which means that the soil specimen tested is sand. Hence, this is totally true.
- Unconfined compression test is dependent upon making the soil specimen, that is to be tested, exposed to header compression only, therefore, the value of σ_3 is 0.0.

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APPENDICES

Appendix A: Weight-Volume Relationships

Appendix B: Laboratory Data Sheets

APPENDIX A

Weight-Volume Relationships

Weight-Volume Relationships

$$W = W_s + W_w$$

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

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

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

$$S = \frac{V_w}{V_v}$$

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

$$e = \frac{V_v}{V_s} = \frac{V_v}{V - V_v} = \frac{\frac{V_v}{V}}{1 - \frac{V_v}{V}} = \frac{n}{1 - n}$$

$$w = \frac{M_w}{M_s}$$

$$\gamma = \frac{W}{V} = \frac{W_s + W_w}{V}$$

$$\gamma_d = \frac{W_d}{V} = \frac{W_s}{V} = \frac{\gamma}{1+w}$$

$$\gamma_{sat} > \gamma > \gamma_d$$

$$W_s = \gamma_s V_s = G_s \gamma_w V_s$$

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

$$W_w = w W_s = w G_s \gamma_w V_s$$

$$\gamma_d = \frac{W_s}{V} = \frac{G_s \gamma_w V_s}{V_s + V_v} = \frac{\dfrac{G_s \gamma_w V_s}{V_s}}{\dfrac{V_s}{V_s} + \dfrac{V_v}{V_s}} = \frac{G_s \gamma_w}{1+e}$$

$$\gamma_{sat} = \frac{W_w + W_s}{V} = \frac{W_w + W_s}{V_s + V_v} = \frac{w G_s \gamma_w V_s + G_s \gamma_w V_s}{V_s + V_v} = \frac{(w+1) G_s \gamma_w V_s}{V_s + V_v} = \frac{\dfrac{(w+1) G_s \gamma_w V_s}{V_s}}{\dfrac{V_s}{V_s} + \dfrac{V_v}{V_s}}$$

$$\gamma_{sat} = \frac{\dfrac{(w+1) G_s \gamma_w V_s}{V_s}}{\dfrac{V_s}{V_s} + \dfrac{V_v}{V_s}} = \frac{(w+1) G_s \gamma_w}{1+e} = (w+1)\gamma_d$$

$$\gamma_d = \frac{\gamma_{sat}}{(w+1)}$$

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

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

$$e = \frac{V_v}{V_s} = \frac{V - V_s}{V_s} = \frac{V}{V_s} - 1 = \frac{\frac{W_s}{\gamma_d}}{V_s} - 1 = \frac{\frac{G_s \gamma_w V_s}{\gamma_d}}{V_s} - 1 = \frac{G_s \gamma_w}{\gamma_d} - 1$$

$$\gamma_d = \frac{W_s}{V} = \frac{G_s \gamma_w V_s}{V_s + V_v} = \frac{\frac{G_s \gamma_w V_s}{V_s}}{\frac{V_s}{V_s} + \frac{V_v}{V_s}} = \frac{G_s \gamma_w}{1 + e}$$

$$\gamma_s = \frac{W_s}{V_s}$$

$$V_s = \frac{W_s}{\gamma_s}$$

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

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

$$Se = \frac{V_w}{V_v} \cdot \frac{V_v}{V_s} = \frac{V_w}{V_s} = \frac{\frac{W_w}{\gamma_w}}{\frac{W_s}{\gamma_s}} = \frac{W_w}{W_s} \cdot \frac{\gamma_s}{\gamma_w} = \frac{wG_s\gamma_wV_s}{G_s\gamma_wV_s} \cdot \frac{\gamma_s}{\gamma_w} = w\frac{\gamma_s}{\gamma_w} = wG_s$$

$$\gamma_{sat} = \frac{W_s + W_w}{V} = \frac{W_s + W_w}{V_s + V_v} = \frac{G_s\gamma_wV_s + \gamma_wV_v}{V_s + V_v} = \frac{\frac{G_s\gamma_wV_s}{V_s} + \frac{\gamma_wV_v}{V_s}}{\frac{V_s}{V_s} + \frac{V_v}{V_s}} = \frac{G_s\gamma_w + \gamma_we}{1 + e}$$

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

$$\gamma = \frac{W}{V} = \frac{W_s + W_w}{V_v + V_s} = \frac{G_s\gamma_wV_s + wG_s\gamma_wV_s}{V_v + V_s} = \frac{\frac{G_s\gamma_wV_s}{V_s} + \frac{wG_s\gamma_wV_s}{V_s}}{\frac{V_v}{V_s} + \frac{V_s}{V_s}} = \frac{G_s\gamma_w + wG_s\gamma_w}{1 + e}$$

$$= \frac{G_s\gamma_w(1 + w)}{1 + e} = \frac{G_s\gamma_w(1 + w)}{1 + \frac{n}{1 - n}} = \frac{G_s\gamma_w(1 + w)}{\frac{1 - n}{1 - n} + \frac{n}{1 - n}} = \frac{G_s\gamma_w(1 + w)}{\frac{1 - n + n}{1 - n}}$$

$$= \frac{G_s\gamma_w(1 + w)}{\frac{1}{1 - n}} = G_s\gamma_w(1 + w)(1 - n)$$

$$w = \frac{W_w}{W_s} = \frac{\gamma_wV_w}{G_s\gamma_wV_s} = \frac{\gamma_w(V_w = V_v \ if \ S = 1)}{G_s\gamma_wV_s} = \frac{\gamma_wV_v}{G_s\gamma_wV_s} = \frac{\gamma_wV_v}{G_s\gamma_w(V - V_v)} = \frac{\frac{\gamma_wV_v}{V}}{G_s\gamma_w\left(\frac{V}{V} - \frac{V_v}{V}\right)}$$

$$= \frac{\gamma_wn}{G_s\gamma_w(1 - n)} = \frac{n}{G_s(1 - n)}$$

$$\gamma = \frac{W_s + W_w}{V_s + V_v} = \frac{\frac{G_s\gamma_wV_s}{V_s} + \frac{wG_s\gamma_wV_s}{V_s}}{\frac{V_s}{V_s} + \frac{V_v}{V_s}} = \frac{(G_s + wG_s)\gamma_w}{1 + e} = \frac{(G_s + Se)\gamma_w}{1 + e}$$

$$Se = \frac{V_w}{V_v} \cdot \frac{V_v}{V_s} = \frac{V_w}{V_s} = \frac{\dfrac{W_w}{\gamma_w}}{\dfrac{W_s}{\gamma_s}} = \frac{W_w}{W_s} \cdot \frac{\gamma_s}{\gamma_w} = wG_s$$

$$\gamma = \frac{W_s + W_w}{V_s + V_v} = \frac{G_s\gamma_w V_s + wG_s\gamma_w V_s}{V_s + V_v} = \frac{\dfrac{G_s\gamma_w V_s}{V_s} + \dfrac{wG_s\gamma_w V_s}{V_s}}{\dfrac{V_s}{V_s} + \dfrac{V_v}{V_s}} = \frac{(G_s + wG_s)\gamma_w}{1 + e}$$

$$= \frac{(1 + w)G_s\gamma_w}{1 + \dfrac{wG_s}{S}}$$

$$\gamma = \frac{W}{V} = \frac{W_s + W_w}{V_v + V_s} = \frac{G_s\gamma_w V_s + wG_s\gamma_w V_s}{V_v + V_s} = \frac{\dfrac{G_s\gamma_w V_s}{V_s} + \dfrac{wG_s\gamma_w V_s}{V_s}}{\dfrac{V_v}{V_s} + \dfrac{V_s}{V_s}} = \frac{G_s\gamma_w + wG_s\gamma_w}{1 + e}$$

$$= \frac{G_s\gamma_w(1 + w)}{1 + e} = \frac{G_s\gamma_w(1 + w)}{1 + \dfrac{n}{1 - n}} = \frac{G_s\gamma_w(1 + w)}{\dfrac{1 - n}{1 - n} + \dfrac{n}{1 - n}} = \frac{G_s\gamma_w(1 + w)}{\dfrac{1 - n + n}{1 - n}}$$

$$= \frac{G_s\gamma_w(1 + w)}{\dfrac{1}{1 - n}} = G_s\gamma_w(1 + w)(1 - n) = G_s\gamma_w(1 - n) + wG_s\gamma_w(1 - n)$$

$$= G_s\gamma_w(1 - n) + \frac{wG_s\gamma_w}{1 + e} = G_s\gamma_w(1 - n) + \frac{wG_s\gamma_w V_s}{V_s + V_v}$$

$$= G_s\gamma_w(1 - n) + \frac{W_w}{V_s + V_v} = G_s\gamma_w(1 - n) + \frac{\gamma_w V_w}{V_s + V_v}$$

$$= G_s\gamma_w(1 - n) + \frac{\dfrac{\gamma_w V_w}{V_v}}{\dfrac{V_s}{V_v} + \dfrac{V_v}{V_v}} = G_s\gamma_w(1 - n) + \frac{\dfrac{\gamma_w(V_w = V_v \; if \; sat)}{V_v}}{\dfrac{V_s}{V_v} + \dfrac{V_v}{V_v}}$$

$$= G_s\gamma_w(1 - n) + \frac{\gamma_w S}{\dfrac{1}{e} + 1} = G_s\gamma_w(1 - n) + \frac{\gamma_w Se}{1 + e} = G_s\gamma_w(1 - n) + \gamma_w Sn$$

$$\gamma_{sat} = \frac{W_s + W_w}{V_s + V_v} = \frac{G_s\gamma_w V_s + wG_s\gamma_w V_s}{V_s + V_v} = \frac{\frac{G_s\gamma_w V_s}{V_s} + \frac{wG_s\gamma_w V_s}{V_s}}{\frac{V_v}{V_s} + \frac{V_s}{V_s}} = \frac{G_s\gamma_w + wG_s\gamma_w}{1 + e}$$

$$= \frac{G_s\gamma_w(1 + w)}{1 + e} = \gamma_d(1 + w)$$

$$\gamma_d = \frac{\gamma_{sat}}{1 + w}$$

$$\gamma_d = \frac{W_s}{V} = \frac{G_s\gamma_w V_s}{V_s + V_v} = \frac{\frac{G_s\gamma_w V_s}{V_s}}{\frac{V_s}{V_s} + \frac{V_v}{V_s}} = \frac{G_s\gamma_w}{1 + e} = \frac{G_s\gamma_w}{1 + \frac{n}{1-n}} = \frac{G_s\gamma_w}{\frac{1-n}{1-n} + \frac{n}{1-n}} = \frac{G_s\gamma_w}{\frac{1-n+n}{1-n}}$$

$$= \frac{G_s\gamma_w}{\frac{1}{1-n}} = G_s\gamma_w(1 - n)$$

$$\gamma_d = \frac{W_s}{V} = \frac{G_s\gamma_w V_s}{V_s + V_v} = \frac{\frac{G_s\gamma_w V_s}{V_s}}{\frac{V_s}{V_s} + \frac{V_v}{V_s}} = \frac{G_s\gamma_w}{1 + e} = \frac{G_s\gamma_w}{1 + \frac{Se}{S}} = \frac{G_s\gamma_w}{1 + \frac{wG_s}{S}} = \frac{G_s\gamma_w}{1 + \frac{wG_s}{S}}$$

$$G_s = \frac{\gamma s}{\gamma_w} = \frac{\frac{W_s}{V_s}}{\frac{W_w}{V_w}} = \frac{W_s}{W_w} \cdot \frac{V_w}{V_s} = \frac{1}{w} \cdot \frac{V_w}{V_s} = \frac{1}{w} \cdot \frac{\frac{V_w}{V_v}}{\frac{V_s}{V_v}} = \frac{1}{w} \cdot \frac{S}{\frac{1}{e}} = \frac{1}{w} \cdot Se = \frac{Se}{w}$$

$$\gamma_d = \frac{W_s}{V} = \frac{G_s\gamma_w V_s}{V_s + V_v} = \frac{\frac{G_s\gamma_w V_s}{V_s}}{\frac{V_s}{V_s} + \frac{V_v}{V_s}} = \frac{G_s\gamma_w}{1 + e} = \frac{\frac{Se}{w}\gamma_w}{1 + e} = \frac{eS\gamma_w}{w(1 + e)}$$

$$\gamma_{sat} = \frac{W_s + W_w}{V_s + V_v} = \frac{W_s}{V_s + V_v} + \frac{W_w}{V_s + V_v} = \gamma_d + \frac{W_w}{V_s + V_v} = \gamma_d + \frac{\gamma_w V_w}{V_s + V_v} = \gamma_d + \frac{\frac{\gamma_w V_w}{V_s}}{\frac{V_s}{V_s} + \frac{V_v}{V_s}}$$

$$= \gamma_d + \frac{\frac{\gamma_w (V_w = V_v \ if \ sat)}{V_s}}{1 + e} = \gamma_d + \frac{\frac{\gamma_w V_v}{V_s}}{1 + e} = \gamma_d + \frac{\gamma_w e}{1 + e}$$

$$\gamma_d = \gamma_{sat} - \frac{\gamma_w e}{1 + e}$$

$$\gamma_{sat} = \frac{W_s + W_w}{V_s + V_v} = \frac{W_s}{V_s + V_v} + \frac{W_w}{V_s + V_v} = \gamma_d + \frac{W_w}{V_s + V_v} = \gamma_d + \frac{\gamma_w V_w}{V_s + V_v} = \gamma_d + \frac{\frac{\gamma_w V_w}{V_s}}{\frac{V_s}{V_s} + \frac{V_v}{V_s}}$$

$$= \gamma_d + \frac{\frac{\gamma_w (V_w = V_v \ if \ sat)}{V_s}}{1 + e} = \gamma_d + \frac{\frac{\gamma_w V_v}{V_s}}{1 + e} = \gamma_d + \frac{\gamma_w e}{1 + e} = \gamma_d + \gamma_w n$$

$$\gamma_d = \gamma_{sat} - \gamma_w n = \gamma_{sat} - n\gamma_w$$

$$\gamma_{sat} = \frac{W_s + W_w}{V_s + V_v} = \frac{\dfrac{G_s \gamma_w V_s}{V_s} + \dfrac{w G_s \gamma_w V_s}{V_s}}{\dfrac{V_s}{V_s} + \dfrac{V_v}{V_s}} = \frac{G_s \gamma_w + w G_s \gamma_w}{1 + e} = \frac{(G_s + w G_s)\gamma_w}{1 + e}$$

$$= \frac{\left(G_s + w\dfrac{\gamma_S}{\gamma_w}\right)\gamma_w}{1 + e} = \frac{\left(G_s + w\dfrac{\dfrac{W_s}{V_s}}{\dfrac{W_w}{V_w}}\right)\gamma_w}{1 + e} = \frac{\left(G_s + w\dfrac{W_s}{W_w}\cdot\dfrac{V_w}{V_s}\right)\gamma_w}{1 + e}$$

$$= \frac{\left(G_s + w\dfrac{1}{w}\cdot\dfrac{V_w}{V_s}\right)\gamma_w}{1 + e} = \frac{\left(G_s + \dfrac{\dfrac{V_w}{V_v}}{\dfrac{V_s}{V_v}}\right)\gamma_w}{1 + e} = \frac{\left(G_s + \dfrac{S}{\dfrac{1}{e}}\right)\gamma_w}{1 + e} = \frac{(G_s + Se)\gamma_w}{1 + e}$$

$$= \frac{(G_s + Se)\gamma_w}{1 + e} = \frac{(G_s + (S = 1\ if\ sat)e)\gamma_w}{1 + e} = \frac{(G_s + 1.e)\gamma_w}{1 + e}$$

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

$$\gamma_{sat} = \frac{W_s + W_w}{V_s + V_v} = \frac{\frac{G_s\gamma_w V_s}{V_s} + \frac{wG_s\gamma_w V_s}{V_s}}{\frac{V_s}{V_s} + \frac{V_v}{V_s}} = \frac{G_s\gamma_w + wG_s\gamma_w}{1+e} = \frac{(G_s + wG_s)\gamma_w}{1+e}$$

$$= \frac{(G_s + wG_s)\gamma_w}{1+\frac{n}{1-n}} = \frac{(G_s + wG_s)\gamma_w}{\frac{1-n}{1-n}+\frac{n}{1-n}} = \frac{(G_s + wG_s)\gamma_w}{\frac{1-n+n}{1-n}} = \frac{(G_s + wG_s)\gamma_w}{\frac{1}{1-n}}$$

$$= (G_s + wG_s)\gamma_w(1-n) = \big((1-n)G_s + (1-n)wG_s\big)\gamma_w$$

$$= \left((1-n)G_s + (1-n)w\frac{\gamma s}{\gamma_w}\right)\gamma_w = \left((1-n)G_s + (1-n)w\frac{\frac{W_s}{V_s}}{\frac{W_w}{V_w}}\right)\gamma_w$$

$$= \left((1-n)G_s + (1-n)w\frac{W_s}{W_w}.\frac{= \left(\frac{1+w}{1+e}\right)\frac{W_s}{W_w}.\frac{(V_w = V_v \text{ if } sat)}{V_s}\gamma_w}{V_s}\right)\gamma_w$$

$$= \left((1-n)G_s + (1-n)w\frac{W_s}{W_w}.\frac{= \left(\frac{1+w}{1+e}\right)\frac{W_s}{W_w}.\frac{V_v}{V_s}\gamma_w}{V_s}\right)\gamma_w$$

$$= \left((1-n)G_s + (1-n)w\frac{1}{w}.e\right)\gamma_w = \big((1-n)G_s + (1-n)e\big)\gamma_w$$

$$= \left((1-n)G_s + (1-n).\frac{n}{(1-n)}\right)\gamma_w = \big((1-n)G_s + n\big)\gamma_w$$

$$\gamma_{sat} = \frac{W_s + W_w}{V_s + V_v} = \frac{\frac{G_s\gamma_w V_s}{V_s} + \frac{wG_s\gamma_w V_s}{V_s}}{\frac{V_s}{V_s} + \frac{V_v}{V_s}} = \frac{G_s\gamma_w + wG_s\gamma_w}{1+e} = \frac{(1+w)G_s\gamma_w}{1+e}$$

$$= \frac{(1+w)G_s\gamma_w}{1+(S=1\text{ if }sat)e} = \frac{(1+w)G_s\gamma_w}{1+Se} = \frac{(1+w)G_s\gamma_w}{1+wGs}$$

$$= \left(\frac{1+w}{1+wGs}\right)G_s\gamma_w$$

$$\gamma_{sat} = \frac{W_s + W_w}{V_s + V_v} = \frac{\frac{G_s\gamma_w V_s}{V_s} + \frac{wG_s\gamma_w V_s}{V_s}}{\frac{V_s}{V_s} + \frac{V_v}{V_s}} = \frac{G_s\gamma_w + wG_s\gamma_w}{1+e} = \frac{(1+w)G_s\gamma_w}{1+e}$$

$$= \left(\frac{1+w}{1+e}\right)G_s\gamma_w = \left(\frac{1+w}{1+e}\right)\frac{\gamma s}{\gamma_w}\gamma_w = \left(\frac{1+w}{1+e}\right)\frac{\frac{W_s}{V_s}}{\frac{W_w}{V_w}}\gamma_w$$

$$= \left(\frac{1+w}{1+e}\right)\frac{W_s}{W_w}\cdot\frac{(V_w = V_v \text{ if } sat)}{V_s}\gamma_w = \left(\frac{1+w}{1+e}\right)\frac{W_s}{W_w}\cdot\frac{V_v}{V_s}\gamma_w$$

$$= \left(\frac{1+w}{1+e}\right)\frac{1}{w}\cdot e\,\gamma_w = \left(\frac{e}{w}\right)\left(\frac{1+w}{1+e}\right)\gamma_w$$

$$\gamma_{sat} = \frac{W_s + W_w}{V_s + V_v} = \frac{\frac{G_s\gamma_w V_s}{V_s} + \frac{wG_s\gamma_w V_s}{V_s}}{\frac{V_s}{V_s} + \frac{V_v}{V_s}} = \frac{G_s\gamma_w + wG_s\gamma_w}{1+e} = \frac{(1+w)\gamma_w G_s}{1+e}$$

$$= (1+w)\gamma_w\frac{G_s}{1+e} = (1+w)\gamma_w\frac{\frac{\gamma s}{\gamma_w}}{1+e} = (1+w)\gamma_w\frac{\frac{\frac{W_s}{V_s}}{\frac{W_w}{V_w}}}{1+e}$$

$$= (1+w)\gamma_w\frac{\frac{W_s}{W_w}\cdot\frac{(V_w = V_v \text{ if } sat)}{V_s}}{1+e} = (1+w)\gamma_w\frac{\frac{1}{w}e}{1+e}$$

$$= (1+w)\gamma_w\frac{1}{w}\frac{e}{1+e} = (1+w)\gamma_w\frac{1}{w}n = (1+w)\gamma_w\frac{n}{w} = n\frac{(1+w)}{w}\gamma_w$$

$$\gamma_{sat} = \frac{W_s + W_w}{V_s + V_v} = \frac{W_s}{V_s + V_v} + \frac{W_w}{V_s + V_v} = \gamma_d + \frac{W_w}{V_s + V_v} = \gamma_d + \frac{wG_s\gamma_w V_s}{V_s + V_v} = \gamma_d + \frac{wG_s\gamma_w V_s}{V_s + V_v}$$

$$= \gamma_d + \frac{wG_s V_v}{V_s + V_v}\gamma_w = \gamma_d + \frac{wG_s\frac{V_s}{V_s}}{\frac{V_s}{V_s} + \frac{V_v}{V_s}}\gamma_w = \gamma_d + \frac{wG_s}{1+e}\gamma_w = \gamma_d + \frac{Se}{1+e}\gamma_w$$

$$= \gamma_d + \frac{(S = 1 \text{ if } sat)e}{1+e}\gamma_w = \gamma_d + \frac{1.e}{1+e}\gamma_w = \gamma_d + \left(\frac{e}{1+e}\right)\gamma_w$$

$$\gamma_{sat} = \frac{W_s + W_w}{V_s + V_v} = \frac{W_s}{V_s + V_v} + \frac{W_w}{V_s + V_v} = \gamma_d + \frac{W_w}{V_s + V_v} = \gamma_d + \frac{wG_s\gamma_w V_s}{V_s + V_v} = \gamma_d + \frac{wG_s\gamma_w V_s}{V_s + V_v}$$

$$= \gamma_d + \frac{wG_s V_v}{V_s + V_v}\gamma_w = \gamma_d + \frac{wG_s\frac{V_s}{V_s}}{\frac{V_s}{V_s} + \frac{V_v}{V_s}}\gamma_w = \gamma_d + \frac{wG_s}{1 + e}\gamma_w = \gamma_d + \frac{Se}{1 + e}\gamma_w$$

$$= \gamma_d + \frac{(S = 1 \; if \; sat)e}{1 + e}\gamma_w = \gamma_d + \frac{1.e}{1 + e}\gamma_w = \gamma_d + \frac{e}{1 + e}\gamma_w = \gamma_d + n\gamma_w$$

$$\left(1 - \frac{1}{G_s}\right)\gamma_d + \gamma_w = \gamma_d - \frac{\gamma_d}{G_s} + \gamma_w = \frac{G_s\gamma_w}{1 + e} - \frac{\gamma_w}{1 + e} + \gamma_w = \frac{G_s\gamma_w}{1 + e} - \frac{\gamma_w}{1 + e} + \frac{\gamma_w(1 + e)}{1 + e}$$

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

$$\gamma_{sat} = \frac{W_s + W_w}{V_s + V_v} = \frac{W_s}{V_s + V_v} + \frac{W_w}{V_s + V_v} = \gamma_d + \frac{wW_s}{V_s + V_v} = \gamma_d + w\frac{W_s}{V_s + V_v} = \gamma_d + w\gamma_d$$

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

APPENDIX B

Laboratory Data Sheets

Measurement of Water Content

Measurement of Specific Gravity

Measurement of Unit Weight

Mechanical Sieve Analysis

Mechanical Hydrometer Analysis

Measurement of Consistency Limits

Modified Proctor Compaction

Field Density by Sand Cone Method

Constant-Head Permeability

Falling-Head Permeability

Consolidation Test

Direct Shear Test on Sand

Unconfined Compression Test

Measurement of Water Content
(Data Sheet)

Sample Description: _____

Sample Number: _____

Location: _____

Tested By: _____

Testing Date: _____

Item / Trial Sample	No. 1	No. 2	No. 3
Specimen number			
Moisture can and lid number			
M_C = Mass of empty, clean can + lid (grams)			
M_{WC} = Mass of can + lid + moist soil (grams)			
M_{DC} = Mass of can + lid + dry soil (grams)			
M_S = Mass of soil solids (grams)			
M_W = Mass of pore water (grams)			
WC = Water content (percent)			
WC_{AVG} = Water content (percent)			

Measurement of Specific Gravity
(Data Sheet)

Sample Description: _____

Sample Number: _____

Location: _____

Tested By: _____

Testing Date: _____

Item / Trial Sample	No. 1	No. 2	No. 3
Specimen number			
Pycnometer bottle number			
W_P = Mass of empty, clean pycnometer (grams)			
W_{PS} = Mass of empty pycnometer + dry soil (grams)			
W_B = Mass of pycnometer + dry soil + water (grams)			
W_A = Mass of pycnometer + water (grams)			
G_S = Specific Gravity (ratio)			
$G_{S(AVG)}$ = Specific Gravity (ratio)			

Measurement of Unit Weight (Density)
(Data Sheet)

Sample Description: _____

Sample Number: _____

Location: _____

Tested By: _____

Testing Date: _____

Unit Weight Measurement	
Item / Trial Sample	**No. 1**
M_S = Mass of soil sample (grams)	
$M_{SW/AIR}$ = Mass of soil sample covered with wax (grams)	
$M_{SW/WATER}$ = Mass of soil sample (grams)	
W_S = Bulk weight of soil sample (newtons)	
M_W = Mass of paraffin wax (grams)	
V_W = Volume of paraffin wax (cubic metres)	
V_{SW} = Volume of soil sample covered with wax (cubic metres)	
V_S = Volume of soil sample (cubic metres)	
γ = Unit weight of soil sample (kilonewtons per cubic metre)	

Moisture Content Measurement	
Item / Trial Sample	**No. 1**
Specimen number	
Moisture can and lid number	
M_C = Mass of empty, clean can + lid (grams)	
M_{WC} = Mass of can + lid + moist soil (grams)	
M_{DC} = Mass of can + lid + dry soil (grams)	
M_S = Mass of soil solids (grams)	
M_W = Mass of pore water (grams)	
WC = Water content (percent)	

Density Measurement	
Item / Trial Sample	**No. 1**
Specimen number	
Moisture can and lid number	
γ = unit weight of soil sample (kilonewton per cubic metres)	
g = Gravity acceleration (metres per square second)	
ρ_B = Bulk density of soil sample (kilonewton per cubic metres)	
WC = Water content (percent)	
ρ_D = Dry density of soil sample (kilonewtons per cubic metre)	

Mechanical Sieve Analysis
(Data Sheet)

Sample Description: _____

Sample Number: _____

Location: _____

Tested By: _____

Testing Date: _____

Item	Data
M_C = Mass of container (grams)	
M_{CD} = Mass of container + dry soil sample (grams)	
M_D = Mass of dry soil sample (grams)	

Sieve No.	Sieve Opening (mm)	Cumulative Retained (grams)	Percent Cumulative Retained	Percent Cumulative Passing
4	4.75			
10	2			
16	1.18			
30	0.6			
40	0.425			
50	0.3			
100	0.15			
200	0.075			
Pan	0.00			

From the grain-size distribution ogive:

Item	Data
Percent gravel	
Percent sand	
Percent fines	
D_{10} = Size of grains that accounts for 10% of the total (mm)	
D_{30} = Size of grains that accounts for 30% of the total (mm)	
D_{60} = Size of grains that accounts for 60% of the total (mm)	
C_u = Uniformity coefficient	
C_z = Coefficient of gradation	

Classification of the soil according to USCS: _____

Mechanical Hydrometer Analysis
(Data Sheet)

Sample Description: _____

Sample Number: _____

Location: _____

Tested By: _____

Testing Date: _____

Item	Data
Hydrometer number	
G_s = Specific gravity of solids (ratio)	
Mass of dispersing agent (grams)	
Mass of soil sample (grams)	
Zero correction factor	
Meniscus correction factor	

Elapsed Time (min)	Temperature (°C)	R_{ACTUAL}	Hydrometer Corrected for Meniscus = R_{ACTUAL} + 1.0	L from Table 5.01 (cm)	K from Table 5.02 = f (G$_S$, $_T$°)	$D\,(mm) = K\sqrt{\dfrac{L\,(cm)}{T\,(min)}}$	C$_T$ from Table 5.03 = f ($_T$°)	R$_C$ = R_{ACTUAL} – Zero Correction + C$_T$	a from Table 5.04 = f (G$_S$)	Percent Finer (%P)	Adjusted Percent Finer (%P$_A$)

Classification of the soil according to USCS: _____

Grain-Size Distribution Ogive

Percent Passing (Finer)

Grain Size (mm) - Log Scale

Measurement of Consistency (Atterberg) Limits (Data Sheet)

Sample Description: _____

Sample Number: _____

Location: _____

Tested By: _____

Testing Date: _____

Liquid Limit Measurement				
Item / Trial Sample	No. 1	No. 2	No. 3	No. 4
Moisture can and lid number				
M_C = Mass of empty, clean can + lid (grams)				
M_{WC} = Mass of can + lid + moist soil (grams)				
M_{DC} = Mass of can + lid + dry soil (grams)				
M_S = Mass of soil solids (grams)				
M_W = Mass of pore water (grams)				
WC = Water content (percent)				

Liquid Limit Chart
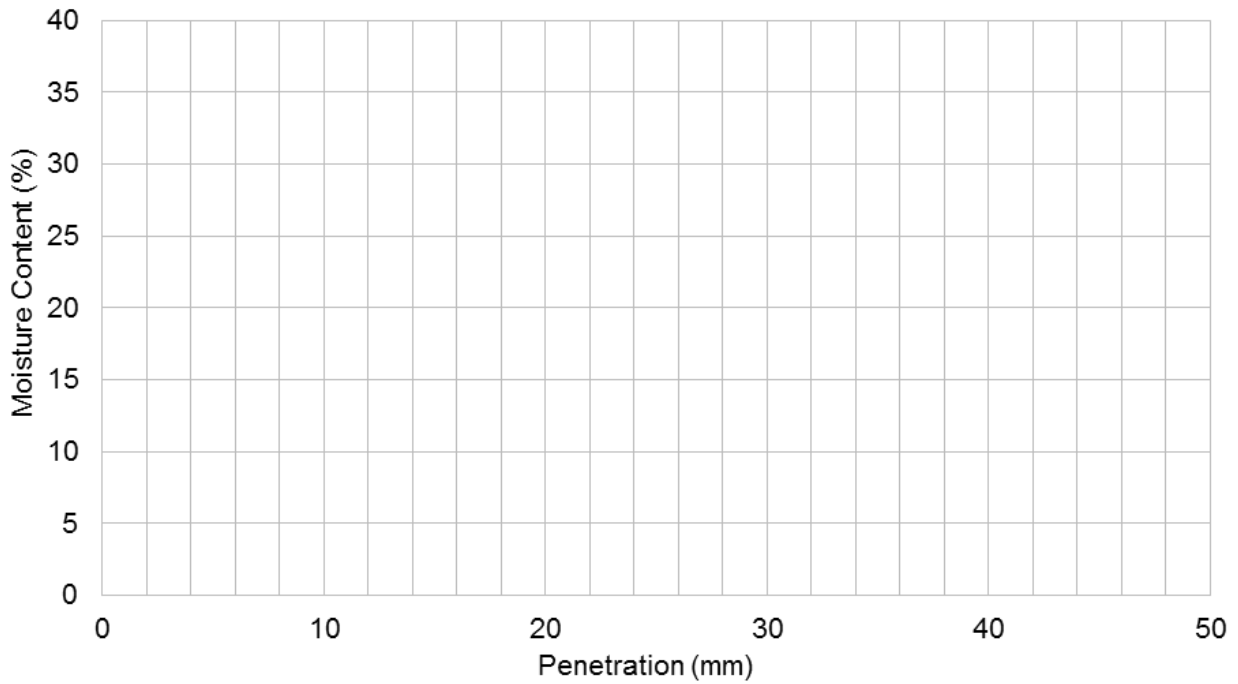
LL = Liquid Limit (%): _____

Plastic Limit Measurement				
Item / Trial Sample	No. 1	No. 2	No. 3	No. 4
Moisture can and lid number				
Mc = Mass of empty, clean can + lid (grams)				
Mwc = Mass of can + lid + moist soil (grams)				
Mdc = Mass of can + lid + dry soil (grams)				
Ms = Mass of soil solids (grams)				
Mw = Mass of pore water (grams)				
WC = Water content (percent)				

PL = Plastic Limit (percent): _____

PI = Plasticity Index (percent): _____

Modified Proctor Compaction
(Data Sheet)

Sample Description: _____

Sample Number: _____

Location: _____

Tested By: _____

Testing Date: _____

Water Content Measurement				
Item / Trial Sample	**No. 1**	**No. 2**	**No. 3**	**No. 4**
Moisture can and lid number				
M_C = Mass of empty, clean can + lid (grams)				
M_{WC} = Mass of can + lid + moist soil (grams)				
M_{DC} = Mass of can + lid + dry soil (grams)				
M_S = Mass of soil solids (grams)				
M_W = Mass of pore water (grams)				
WC = Water content (percent)				

V = Volume of Mould (cubic centimetres): _____

Density Measurement				
Item / Trial Sample	No. 1	No. 2	No. 3	No. 4
WC = Assumed water content (percent)				
M_M = Mass of empty mould (grams)				
M_{MS} = Mass of compacted soil and mould (grams)				
M_S = Wet mass of soil in mould (grams)				
ρ_W = Wet density (grams per cubic centimetre)				
ρ_D = Dry density (grams per cubic centimetre)				

Moisture Content Versus Dry Density Relationship
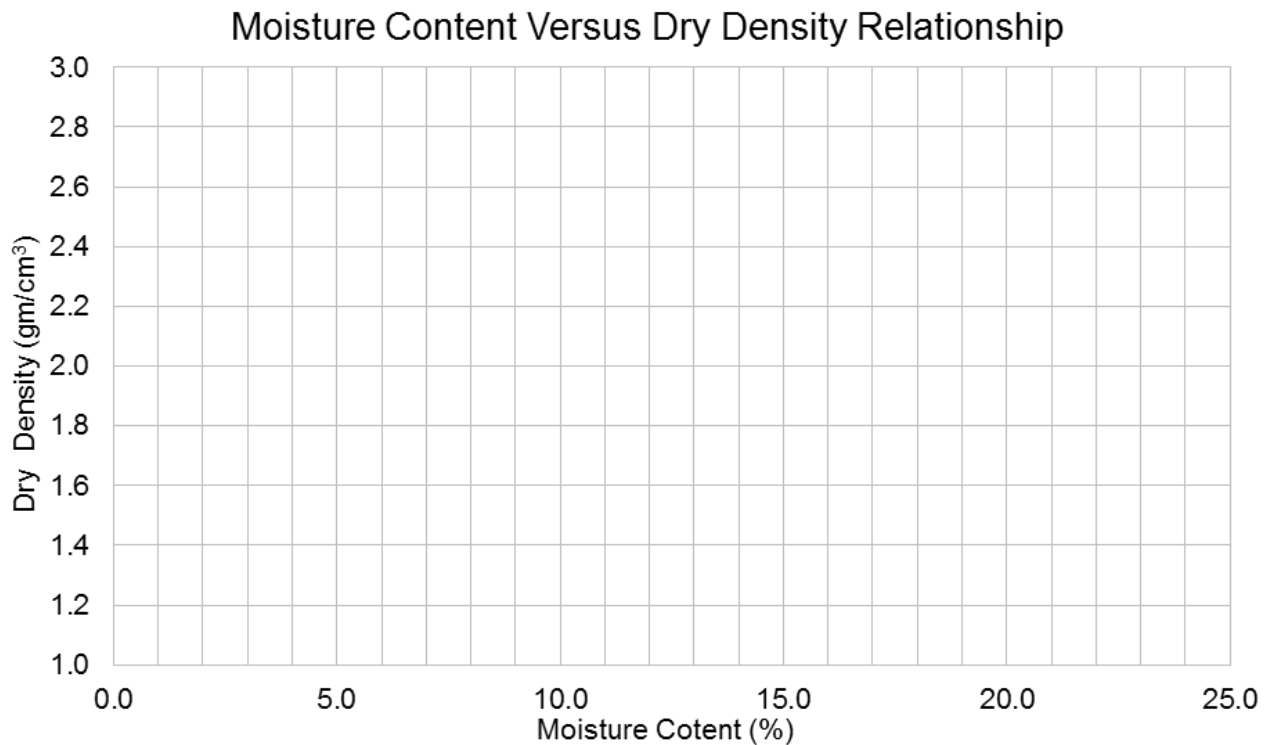
Optimum Moisture Content (percent): _____

Maximum Dry Density (grams per cubic centimetre): _____

Field Density by Sand Cone Method
(Data Sheet)

Sample Description: _____

Sample Number: _____

Location: _____

Tested By: _____

Testing Date: _____

Density Measurement	
Item	**No. 1**
WC = In-field water content (percent)	
M_1 = Weight of the jar filled with Ottawa sand (percent)	
M_2 = Weight of Ottawa sand retained in the jar (grams)	
M_3 = Mass of compacted soil and mould (grams)	
M_F = Weight of the soil sample taken from field (grams)	
ρ_{FB} = In-field bulk density of soil (grams per cubic centimetre)	
ρ_{FD} = In-field dry density of soil (grams per cubic centimetre)	

In-field Dry Density (grams per cubic centimetre): _____

Maximum Dry Density (grams per cubic centimetre): _____

Degree of Compaction (percent): _____

Constant-Head Permeability
(Data Sheet)

Sample Description: _____

Sample Number: _____

Location: _____

Tested By: _____

Testing Date: _____

Item / Trial Sample	No. 1	No. 2
D = Diameter of soil specimen (centimetres)		
L = Length of soil specimen (centimetres)		
M_S = Weight of soil specimen (grams)		
M_{EC} = Weight of container empty (grams)		
M_{WC} = Weight of container + water (grams)		
H = Deference in head in manometers (centimetres)		
t = test duration (minutes)		
Q = Quantity of water discharged (cubic centimetres)		
q = Rate of flow (cubic centimetres per second)		
A = Cross-sectional area of specimen (square centimetres)		
k = Coefficient of permeability (centimetres per second)		

Coefficient of Permeability at LAB Temperature, $K_{T\,°C}$ (cm/sec): _____

Relative-Temperature Viscosity of Water, $\eta_{T\,°C}/\eta_{20\,°C}$ (factor): _____

Coefficient of Permeability at 20 °C, $K_{20\,°C}$ (cm/sec): _____

Falling-Head Permeability
(Data Sheet)

Sample Description: _____

Sample Number: _____

Location: _____

Tested By: _____

Testing Date: _____

Item / Trial Sample	No. 1	No. 2
D = Diameter of soil specimen (centimetres)		
H_1 = Head across sample at the test beginning (centimetres)		
H_2 = Head across sample at the test end (centimetres)		
t = Collection duration (minutes)		
A = Cross-sectional area of specimen (square centimetres)		
a = Cross-sectional area of the burette (square centimetres)		
k = Coefficient of permeability (centimetres per second)		

Coefficient of Permeability at LAB Temperature, $K_{T\,°C}$ (cm/sec): _____

Relative-Temperature Viscosity of Water, $\eta_{T\,°C}/\eta_{20\,°C}$ (factor): _____

Coefficient of Permeability at 20 ˚C, $K_{20\,°C}$ (cm/sec): _____

Consolidation Test
(Data Sheet)

Sample Description: _____

Sample Number: _____

Location: _____

Tested By: _____

Testing Date: _____

Part I: Factor of consolidation (Cv)

Stress, σ (kPa): _____				Stress, σ (kPa): _____			
Elapsed Time (min)	Dial Reading (min)	Deformation, ΔH (mm)	$t^{0.50}$ (min)	Elapsed Time (min)	Dial Reading (min)	Deformation, ΔH (mm)	$t^{0.50}$ (min)

H_i = Height of Specimen (centimetres): _____

(1) Logarithmic Time Method

Logarithm of Time Method
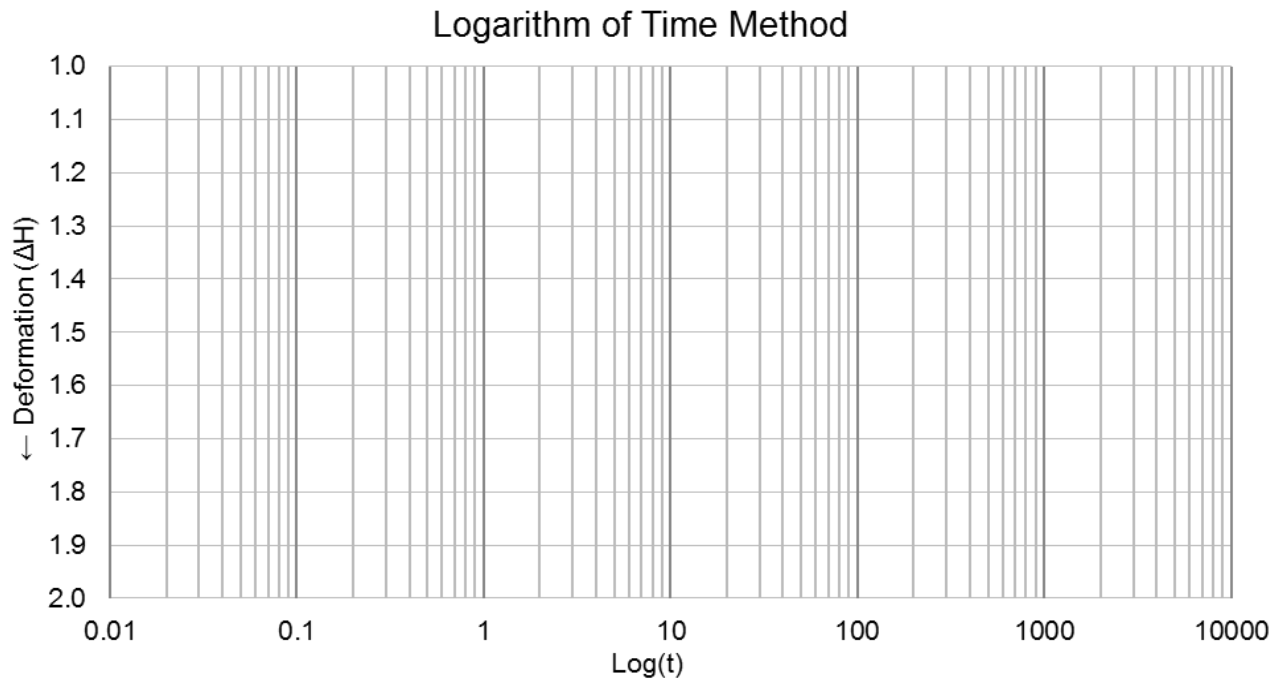
Item	Data
t_2 (minutes)	
t_2 (minutes)	
d_0 (millimetres)	
d_{100} (millimetres)	
d_{50} (millimetres)	
t_{50} (millimetres)	
H_i (millimetres)	
Initial reading (millimetres)	
Final reading (millimetres)	
D_{AVG} (millimetres)	
H_{dr} (centimetres)	
C_V (square centimetres per minute)	

(2) Square Root of Time Method

Square Root of Time Method
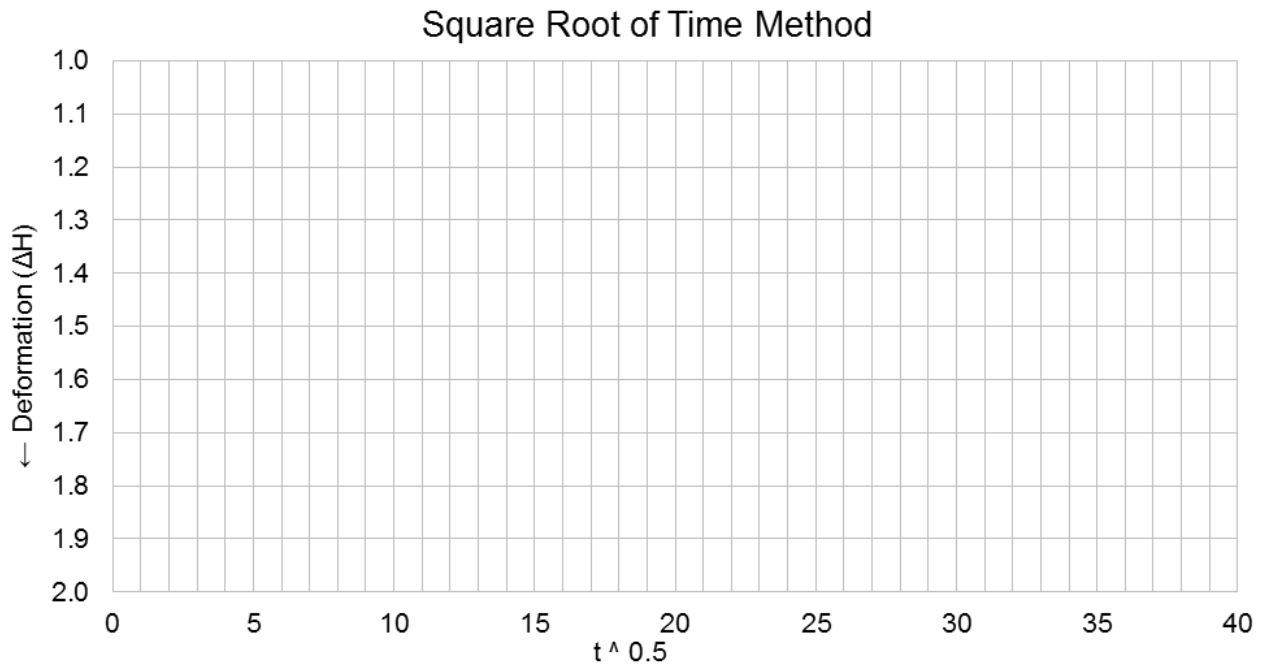
Item	Data
OB	
OC	
t_{90} (minutes)	
H_i (millimetres)	
Initial reading (millimetres)	
Final reading (millimetre)	
D_{AVG} (millimetres)	
H_{dr} (centimetres)	
C_V (square centimetres per minute)	

Part II: Factor of compression (Cc) and pre-consolidation pressure (σ'c)

Pressure (kPa)	ΔH (0.001 mm)	ΔH	Viod Ratio, e_i

Item	Data
H_S (centimetres)	
e_0 (viod ratio)	

Viod Ratio (e) Versus Log(σ'c) Plot
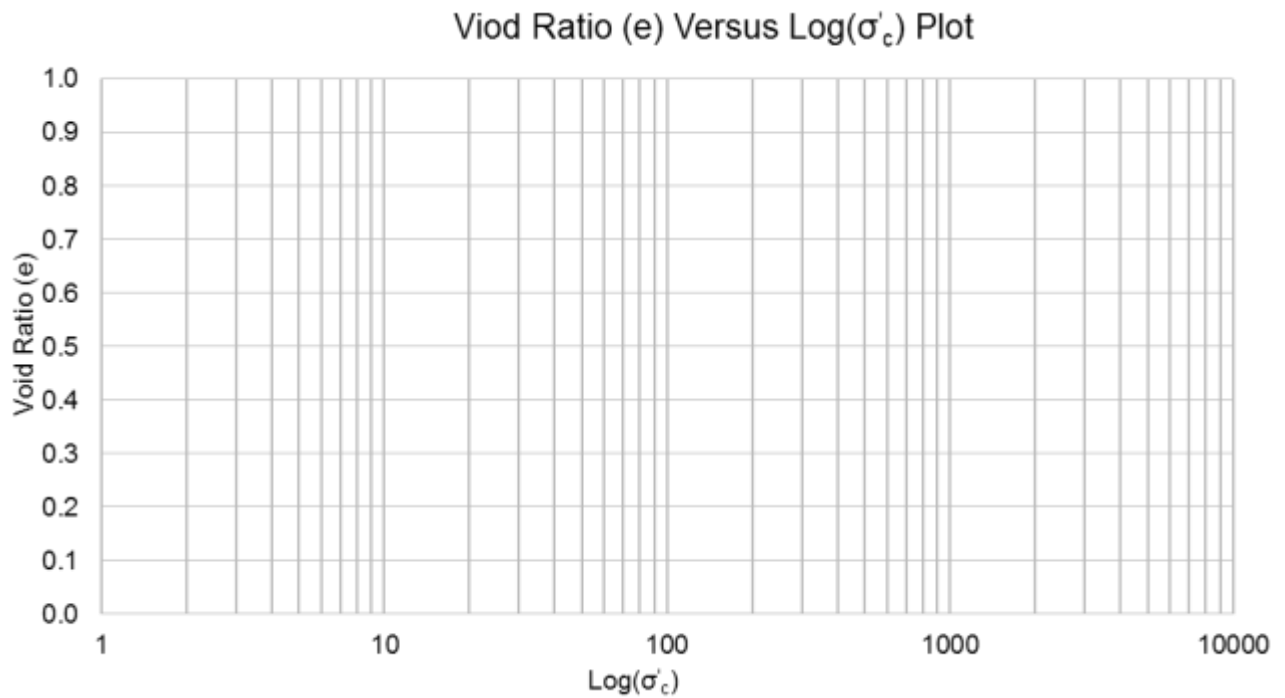
Pre-consolidation Pressure, σ'c (kilopascals): _____

Viod Ratio (e) Versus Log(σ'_c) Plot
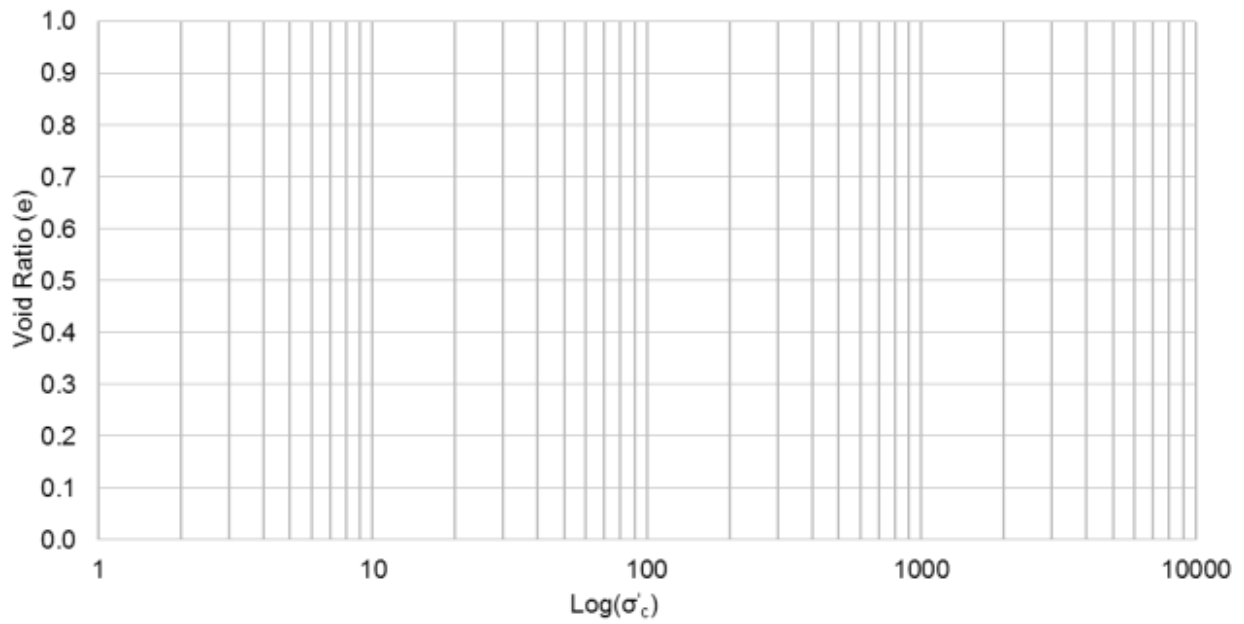
Item	Data
e_o (void ratio)	
0.4 e_o (void ratio)	
σ'_c (kilopascals)	
σ (kilopascals)	

Compression Index, Cc (slope): _____

Direct Shear Test on Sand
(Data Sheet)

Sample Description: _____

Sample Number: _____

Location: _____

Tested By: _____

Testing Date: _____

Shear Box Inside Dimensions (millimetres): _____

Shear Box cross-sectional area (square millimetres): _____

Normal Stress, σ (kPa): _____				Normal Stress, σ (kPa): _____			
Horizontal Dial Reading (0.00254 mm)	Horizontal Shear Force (N)	Horizontal Displacement, ΔH (mm)	Shear Stress, τ (kPa)	Horizontal Dial Reading (0.00254 mm)	Horizontal Shear Force (N)	Horizontal Displacement, ΔH (mm)	Shear Stress, τ (kPa)

ΔH-Displacement Versus Shear Stress Plot

Peak Shear Stress (kilopascals): _____

Normal Stress, σ (kPa): _____				Normal Stress, σ (kPa): _____			
Horizontal Dial Reading (0.00254 mm)	Horizontal Shear Force (N)	Horizontal Displacement, ΔH (mm)	Shear Stress, τ (kPa)	Horizontal Dial Reading (0.00254 mm)	Horizontal Shear Force (N)	Horizontal Displacement, ΔH (mm)	Shear Stress, τ (kPa)

ΔH-Displacement Versus Shear Stress Plot

Peak Shear Stress (kilopascals): _____

Normal Stress, σ (kPa): _____				Normal Stress, σ (kPa): _____			
Horizontal Dial Reading (0.00254 mm)	Horizontal Shear Force (N)	Horizontal Displacement, ΔH (mm)	Shear Stress, τ (kPa)	Horizontal Dial Reading (0.00254 mm)	Horizontal Shear Force (N)	Horizontal Displacement, ΔH (mm)	Shear Stress, τ (kPa)

ΔH-Displacement Versus Shear Stress Plot

Peak Shear Stress (kilopascals): _____

Normal Stress, σ_f (kPa)	Shear Stress, τ_f (kPa)

Normal Stress Versus Shear Stress Plot

$T = c + \sigma \tan \emptyset$

$Y = a + b X$

Comparing the two past equations, one to another, leads to the results that follow:

$c = 0.0$ (since the spacemen is sand)

$\tan \emptyset = slope = b$

$\tan \emptyset$: _____

Angle of Internal Friction, $\emptyset$: _____

Unconfined Compression Test
(Data Sheet)

Sample Description: _____

Sample Number: _____

Location: _____

Tested By: _____

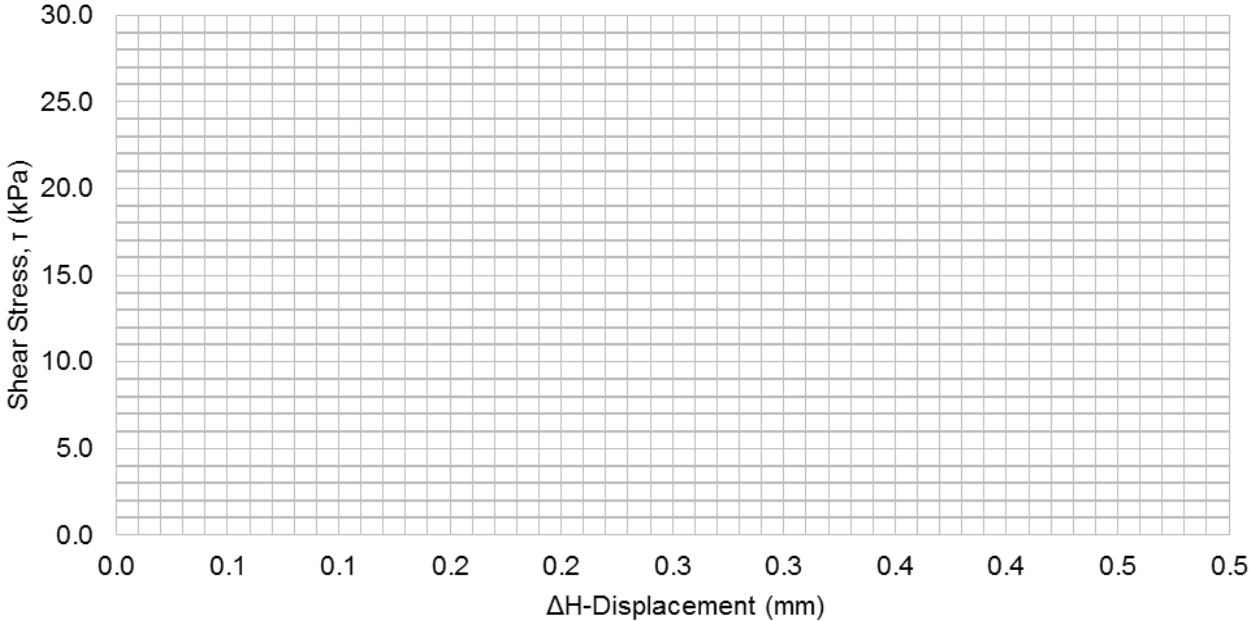Testing Date: _____

Diameter of Specimen (millimetres): _____

Cross-sectional Area of Specimen (square millimetres): _____

Deformation Dial Reading	Load Dial Reading	Sample Deformation, ΔL (mm)	Strain, ε	%Strain	Corrected Area, A' (cm2)	Load (kN)	Stress (kPa)
*	*	*	*	*	*	*	*

* To be continued.

Deformation Dial Reading	Load Dial Reading	Sample Deformation, ΔL (mm)	Strain, ε	%Strain	Corrected Area, A' (cm2)	Load (kN)	Stress (kPa)
*	*	*	*	*	*	*	*

Axial Stress Versus Percent Axial Strain Plot

From the graph drawn above: At 15% strain, the value of q_u is _____ kPa

Therefore, the value of c_u which is equal to $q_u / 2$ is _____ kPa

Consequently, Mohr's circle of the specimen tested can be drawn as under:

Shear Stress Versus Normal Stress Plot