UNIVERSITI
TEKNOLOGI
PETRONAS

Effect of Prefabricated Vertical Drains and Electro-Osmosis in treating consolidation problems in soils.

By

Kerby Lee

Dissertation submitted in partial fulfillment of
the requirements for the
Bachelor of Engineering (Hons)
(Civil Engineering)

DECEMBER 2004

Universiti Teknologi PETRONAS
Bandar Seri Iskandar
31750 Tronoh
Perak Darul Ridzuan

CERTIFICATION OF APPROVAL

Effect of Prefabricated Vertical Drains and Electro-Osmosis in treating consolidation problems in soils.

by

Kerby Lee

A project dissertation submitted to the
Civil Engineering Programme
Universiti Teknologi PETRONAS
in partial fulfilment of the requirement for the
BACHELOR OF ENGINEERING (Hons)
(CIVIL ENGINEERING)

Approved by,

(Syed Baharom Azahar b. Syed Osman)

UNIVERSITI TEKNOLOGI PETRONAS

TRONOH, PERAK

DECEMBER 2004

CERTIFICATION OF ORIGINALITY

This is to certify that I am responsible for the work submitted in this project, that the original work is my own except as specified in the references and acknowledgements, and that the original work contained herein have not been undertaken or done by unspecified sources or persons.

KERBY LEE

ABSTRACT

In general, a given soil must reach full consolidation before any structure can be built on it. It is critical that all excess water be removed from the soil before construction begins. However, fine-grained, compressible soils have a low permeability and therefore it take a very long time to consolidate. To achieve the desired consolidation rate, various method and techniques are presently used at construction site.

This project makes comparison between using Prefabricated Vertical Drains, Electro-Osmosis, and the traditional way by surcharging method as a consolidation tools. Tests were done initially to determine the soil characteristic to be used in the experiment. Consecutively, a Perspex prototype was build in order to conduct a scaled-down lab test and the data is collected on a regular basis from a computer used to take down compression gauge readings. Effectiveness measured in terms of soil settlement and reduction in moisture content and increase in shear strength in the sample tested. The data is then plotted into graph for comparison analysis to be done.

The end results show that the electro-osmosis is a better way to consolidate the soil tested gauging form the improvement in the above mentioned soil characteristic.

ACKNOWLEDGEMENT

First of all I would like to thank my family, who consistently give me spiritual support and guidance in one way or another that has helped me to complete the dissertation. The loves showed by them inspire me to continue through various difficulties and obstacles.

My greatest gratitude goes to my project supervisors, Tuan Syed Baharom, who has continuously monitored the progress and share his knowledge and experience throughout the duration of the project. His comments, advices and suggestion are taken into serious consideration that leads to final outcome of this project.

Besides, I would like to express my appreciation towards Univertiti Teknologi PETRONAS lectures and technicians, especially those in charge of Civil Laboratory. They are Mr Zaini and Mr Johan, who are always willing to help and give opinions at any time.

Last but not least, I would like to thanks everyone who has who generously provides information, comments and support to help me to complete this project. Highest gratitude is extended to university mate Lim Chee Hoe for his effort in helping with the completion of the report and to fellow hostel housemates who had offered help upon request. Without the help of them, this project could not be completed as desired.

ABBREVIATION AND NOMENCLATURE

- PVD – Prefabricated Vertical Drains
- EO – Electro-Osmosis

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

In general, a given soil must reach full consolidation before any structure can be built on it. It is critical that all excess water be removed from the soil before construction begins. However, fine-grained, compressible soils have a low permeability and therefore it take a very long time to consolidate. To achieve the desired consolidation rate, various method and techniques are presently used at construction site. Soil improvement by installing vertical drains which provide a shorter and easier drainage path through which the water can escape. The closer the drain spacing, the faster the rate of settlement. In addition, vertical drains simply reduce the settlement times required to complete consolidation. The same degree of consolidation will ultimately occur, with or without drains.

While for Electro-Osmosis, the electrokinetic phenomena in soils is envisioned to be used for removal/separation of organic and inorganic contaminants and radionuclide, barriers and leak detection systems in clay liners, diversion schemes for waste plumes, and for injection of grouts, microorganisms and nutrients into subsoil strata and *In Situ* generation of hydrogen peroxide for remediation.

In the last five decades since its first application and use (Casagrande, 1947), the mechanics of consolidation by electro-osmosis has been extensively investigated by geotechnical engineers. However, studies investigating removal of ions from soils by the electrokinetic phenomena are limited, possibly due to insufficient understanding of the electrochemistry associated with the process. The need to utilize the process in removal/separation of contaminants necessitates a good understanding of electrochemistry and its relation to the mechanical behavior.

1.2 PROBLEM STATEMENT

Engineers are often required to build on sites, which consist of fine-grained, saturated soils such as clay that have poor drainage properties. Construction without soil treatment is usually impractical due to unpredictable long-term settlement. Simple surcharging as a soil consolidation method can take many years. If the soil is not strengthened in advance, the added weight of a new structure will cause water to squeeze out over time. The soil layer will compress or settle as water is removed. Throughout this consolidation process, the foundation of the structure will continue to shift until the soil has completely settled. By this point, irreparable damage to the structure will have occurred. Presently, Soil consolidation using prefabricated vertical wick drains or (also commonly called wick drains, band drains, or PV drains) or Electro-Osmosis can rapidly increase settlement rates and cut project durations drastically.

1.3 OBJECTIVE AND SCOPE OF STUDY

First and foremost, this project requires students to make comparison between using Prefabricated Vertical Drain and Electro-Osmosis method as a consolidation tools. From there, we can make the data comparison by using both methods and to determine better choices for using this advance, high cost technology as compare to the low cost surcharge. In addition, both advance method have been developed to enable increase discharge of high pore water pressure in soil structures, thus the process of consolidation has been accelerated with reducing the time frame and produce an effective and solid settlement process without endangering the soil structure.

The following summarizes the scope of work for two semesters:

i) **First semester**

- Literature review and theories
- Set up prototype
- Purchase material from supplier (PVD, Geosynthesis)
- Geotechnical & Foundation Earthworks Laboratory (To determine the characteristic of problematic soil)

ii) **Second semester**

- Geotechnical & Foundation Earthworks Laboratory
- To examine the difference between the PVD and electro-osmosis in consolidation of settlement
- To analyze the results obtained from the lab test
- To draw out conclusion from the lab test and eventually determine the better choice between the two.

CHAPTER 2

LITERATURE REVIEW AND THEORY

2.1 CONSOLIDATION THEORY FOR PREFABRICATED VERTICAL DRAINS

2.1.1 What are Vertical Wick Drains?

Prefabricated vertical wick drains (PVD or PV Drains) are installed vertically to depths exceeding 65 meters. The water, under pressure in excess of hydrostatic, flows through the filter fabric of the prefabricated vertical wick drain and into the channels of the wick drain core where it can flow vertically out of the soil. This flow may be either up or down to intersecting natural sand layers or to the surface where a sand drainage blanket or prefabricated horizontal strip drains are provided. The water in the soil has only to travel the distance to the nearest prefabricated vertical wick drain to reach a free drainage path.

THE ORIGINAL CONCEPT of “Vertical Drains” was developed in the 1920’s, by creating sand columns in the ground. The sand column acted as drains since they were more permeable than the surrounding clay/silt. In the 1940’s, Walter Kjellman developed the first Prefabricated Vertical Drain (“Wick”), which consisted of a few channels imprinted into a stiff cardboard core. The concept was further developed in 1970’s, with the introduction of drains using a synthetic drainage core with longitudinal “channels” or “grooves”, enveloped in a paper or non-woven filter.

2.1.2 The Consolidation Process

The application of load on top of the soil will result in an initial increase in pore water pressure, which will dissipate slowly as the pore water drains off. In saturated soils such as clay and silty clay, which have a large percentage of voids or pores usually filled with water, the settlement process will be lengthy. This process will further extend if the soil is finer as it will be more difficult and time consuming for water to drain. Besides, the permeability of the soil, the drainage time also depends on factors such as the thickness of the soil layers and the distance the water has to flow through the soil to escape. During this process known as consolidation, the load is gradually transferred to the soils particles as the volumes of the voids are reduced and this culminates in the form of settlement. Once the desired consolidation has been achieved, construction can continue. A site can be ready in just a matter of months instead of several years if drains are not used.

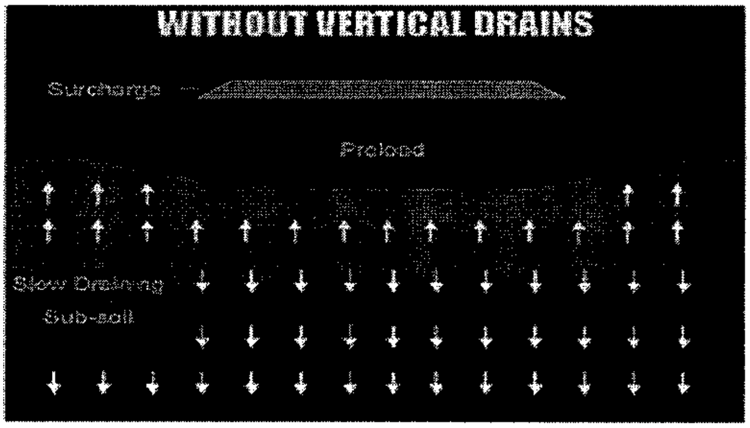


Figure 2.0: Without Vertical Drains

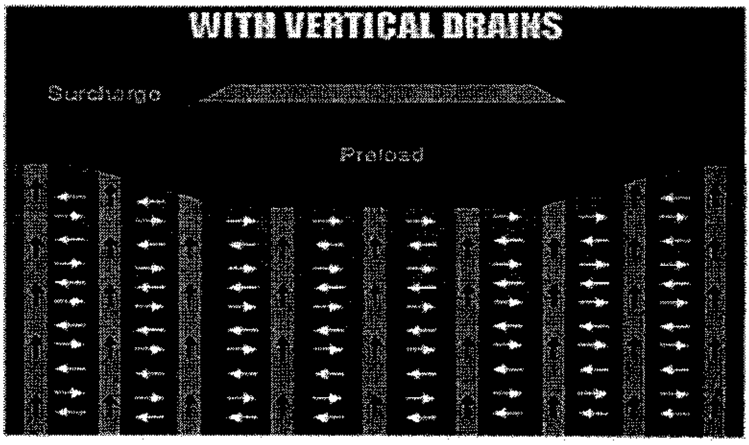


Figure 2.1: With Vertical Drains

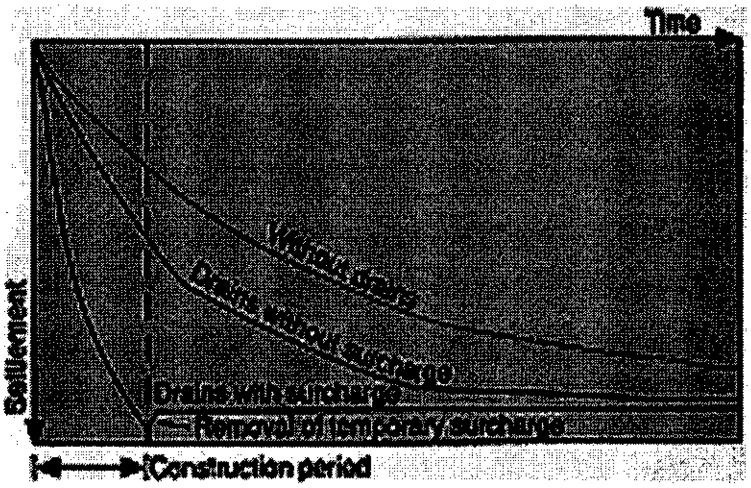


Figure 2.2: Settlement vs. Time

2.1.3 *Performances of Vertical Drains*

Due to the very low permeability of the clay soil and silty clay soils, thus it results to a lengthy consolidation process. In order to accelerate the consolidation process, vertical drains are installed in regular spacing into the full depth of the compressible soil layer. This creates an artificial and shorter horizontal drainage path. Drain spacing may be adjusted to match the required settlement time.

Vertical drains enable the pressurized water to flow horizontally towards the nearest drains, and escape through the longitudinal grooves on both sides of the vertical core. Usually it is used in conjunction with preloading the surcharge with soil or vacuum pressure.

The prefabricated vertical wick drain core is made of high quality flexible polypropylene which exhibits a large water flow capacity in the longitudinal direction of the core via preformed grooves or water channels on both sides of the core. Each vertical wick drain can provide a greater vertical discharge capacity than a 6 inch diameter sand column. The prefabricated vertical wick drain core is tightly wrapped in a geotextile filter jacket of spun-bonded polypropylene which has very high water permeability while retaining the finest of soil particles. Both the core and geotextile filter jacket have high mechanical strength, a high degree of durability in most environments, and high resistance to chemicals, micro-organisms, and bacteria

Through the principle of vertical drains is simple, the process is complicated. Great care has to be taken when choosing vertical drains as they are subjected to both tensile and compressive forces when the soil shifts and settles during the consolidation process. This can severely affect the ability to affect the drains to function as intended.

- Lateral soil displacement can cause certain drains to elongate beyond their rupture point.
- Substantial vertical soils pressure can cause some drains to pinch off as they experience folding and buckling of the core.

Therefore, it is of utmost importance that drain performance under both conditions can be considered when selecting a vertical drain. Remember, drain failure can severely jeopardize project's construction and structure stability

2.2 ELECTROKINETIC PHENOMENA IN SOILS

2.2.1 Electro-Osmosis

Electro-osmosis is the transportation of free water in the capillaries of a soil. The electro-osmosis method of stabilization consists of embedding a series of electrode in the soil to be stabilized and applying a direct current potential across pairs of electrode. As the current passes from an anode (positive electrode) to a cathode (negative electrode), water in the soil is forced to migrate from the anode towards the cathode from where it is removed by pumping. This process results in the reduction of the water content of the soil and a buildup of pore water tension in the soil, both of which increase soil strength. In addition, the electro-osmosis process causes a base exchange within the soil which can give rise to a further strength increase.

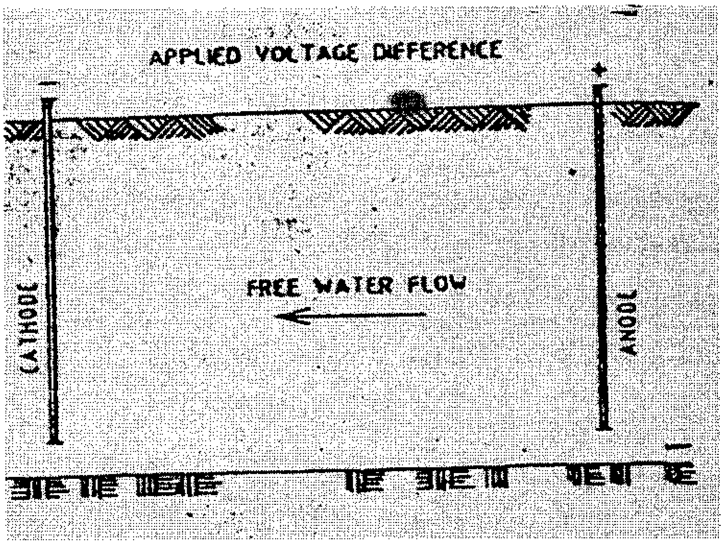


Figure 2.3: Electro-Osmosis Flows

There are many theories explaining the mechanism of electro-osmosis, however, they are all basically variations of the original theory of Helmholtz (1). In the Helmholtz theory, pore radii are assumed to be large relative to the thickness of the diffuse double layer surrounding clay particles, and all of the mobile charge is assumed concentrated near the pore wall. These assumptions are reasonably valid for soils with large pores and saturated with fresh water or dilute electrolyte solutions. In soil water systems there are commonly positive and negative ions present. Because the soil particles are either positively or negatively charged, the oppositely charged ions in the voids will concentrate adjacent to the soil skeleton. Most soil particles have a negative charge and will thereafter, attract positive ions. When an external electric potential is applied to the soil, the ions are set in motion by the force field. If an adequate number of ions along the capillary walls of the soil skeleton move, they will cease transmission of the adjacent water molecules. This moving film of water will cause the entire cross-section of the pore water to also move. In silts and clays the pore water will be moved in the same direction as the boundary layer; but in soils with large voids the central portion of the pore water may simply recirculation with the net result that the water is not induced to flow.

If free water is not available at the anode to replenish the pore water, the water content of the material being treated will decrease, starting at the anode and progress towards the cathode. This creates tensile stresses and results in soil consolidation and a subsequent strength increase in clays and silts.

Gray and Mitchell (1967) showed experimentally that although the electro-osmotic flow increases with increasing water content of most soils, the flow decreases with an increasing electrolyte concentration of the pore fluid. In addition, they observed that the fundamental importance in electro-osmosis phenomena is the cation-anion distribution and the water-ion distribution in the soil. They stressed that in clays and other ion exchangers, positive counter-ions required to balance the negative fixed charges on the solid particles are in the majority, and hence they impart more momentum to the water than do the co-ions (CO' ions are ions with the same sign as the fixed surface charges on a clay or other exchangers). So there is net water transfer in the direction of counter-ion movement.

In addition to water transport between the electrodes, oxidation and reduction take place at the electrodes as electrons are transferred in and out of the system (Gray and Mitchell, 1967; Thomas and Lentz, 1990; d Mitchell, 1993), resulting in ion diffusion, ion exchange, development of osmotic and pH gradients, dessication by heat generation at the electrodes, mineral decomposition, precipitation of salts or secondary minerals, electrolysis, hydrolysis, oxidation, reduction, physical and chemical adsorption, and fabric changes (Mitchell, 1993). Some of the changes may be beneficial while the others may retard the efficiency of electro-osmosis. Electrolysis of water at the anode and the cathode produces oxygen and hydrogen, respectively, which can be represented by the following equations;

$$2H_20-4e^- \text{--->} 4H^+ +0_2 \text{ (anode)} \tag{3}$$

$$2H_20 + 2e^- \text{--->} 20H^- + H_2 \text{ (cathode)} \tag{4}$$

Based on Equation (3) and (4), it is noteworthy that both H^+ and OH^- sweep across the soil sample toward the cathode and the anode, respectively during the course of electro-kinetic processing. Since H^+ travels approximately two times faster than OH^-, prolonged electro-kinetic processing will result in acidification of the treated soil.

Besides, the movement of water when a direct current voltage difference is applied to saturated soils, the following effects may also develop ion exchange, ion diffusion, generation of osmosis and pH gradients; desiccation from heat generated at electrodes mineral decomposition; precipitation of secondary minerals; electrolysis, hydrolysis, oxidation, reduction physical and chemical absorption, and fabric changes.

Because of these effects at least some changes in soil properties that are not readily accountable for in terms of the simplified water flow theory must be expected. The consequence of these effects may be beneficial in terms of electro chemical hardening of the soil giving increased strength and lower plasticity characteristics

The rate at which pore water is moved through a soil water system is dependent on:

I. The magnitude of the applied electric potential
II. The chemistry of the soil water system
III. The size and shape of the pores
IV. The relationship between the inter granular stress and the pore water tension
V. The availability of free water at the anode

Electrode Design

In most past application, with the exception of Bjerrum et al, (1967), the design of the cathode usually consisted of an iron pipe and eductor pipe installed in a predrilled hole of substantial (about 400mm diameter) and filled with clean filter sand. The installation and material costs of electrodes are therefore quite high and, in addition, pumping of expelled water is usually required. The anode is usually made of iron pipe, rail, or steel bar, and the product of the electrochemical reaction is the formation of iron oxide and hydroxide of high electrical resistance which decreases the efficiency of the treatment. Furthermore, these design versions of cathodes and anodes prohibit the application of electrode polarity reversal.

From laboratory and model test (Ho, 1990), it was shown that the use of perforated copper pipe was more effective than other metals. It provides passage for expelled water and gas to flow into the cathode and out to the surface during treatment and no pumping of water required. The undesirable effect of high resistance metallic oxide and hydroxide was also eliminated due to the replacement copper oxide and hydroxide high conductivity. With this electrode design both anode and cathode are identical, therefore the manufacturing and installation costs of the electrodes are reduced.

From the unsuccessful application reported by Caron (1971a, 1972b), it is deduced that sand and silt layers in the subsoil are not favorable for the process. Because of the relatively high conductivity of such layers it would cause "short circuiting" of the system. If the groundwater table is higher than layers, water from the layers will flow into the perforated electrode and affect the efficiency of the treatment.

Electro-osmosis for Soil Improvement

Most soils which require improved drainage can be dewatered by one or a combination of five methods: (1) Sumps and ditches (2) sheeting and open pumping (3) deep well sumps (4) well point, systems and (5) vacuum dewatering systems. However there are many silts, clayey silts, and fine clayey silty sands which cannot be successfully drained by the previous methods, but which can be drained with the help of electrical flow through the soil (Chappell and Burton, 1975).

Various researchers such as Gray and Mitchell (1967), Esrig and Gerneinhardt (1967), Johnston and Butterfield (1977), and Othman and Shafii (1990) conducted bench scale experiments to investigate the effects of electro-osmosis on soils, whilst field tests performed by Fetzer (1967), Chappel and Burton (1975), Eggstad and Foyn (1983), Lo and Ho (1991), and Chen and Murdoch (1999) justify the applicability of the process

2.2.2 Electrophoresis

Electrophoresis is defined as the migration of charged colloids, not small ions, in solid-liquid mixture under electric potential gradient, where discrete particles are transported through water. If a direct current (DC) is applied to clay-water systems, negatively charged clay particles will migrate toward the anode, In a compact system of porous plug, electrophoresis is of less importance due to restrained solid phase.

2.2.3 Electromigration

Electromigration is defined as the movement of charged ions towards the oppositely charged electrodes relative to solution. In a dilute system or a porous medium with moderately concentrated aqueous solution of electrolytes, electromigration of ions is the major cause of current conduction. With regard to contaminated soils, electromigration is the primary mechanism of electro remediation when the contaminants are ionic or surface charged.

CHAPTER 3

METHODOLOGY/ PROJECT WORK

3.1 PROCEDURE IDENTIFICATION

Below summarizes the process flowchart of the project. Upon finalizing the topic selection and while researching on the PVD technologies, discussions were done with supervisor and relevant personnel to collect information. Contacts were made towards experienced professionals in PVD field for further assistance in researching technologies and purchasing some sample material (PVD and Geosynthesis), together with a possible laboratory session set up to verify the soil material before further proceed. Literature review on electro-osmosis in reduction of moisture in settlement also was carried out extensively. The prototype was then subsequently set up and the laboratory session to start the tests based on PVD and electro-osmosis were done and the data were collected on a regular basis from a computer. Analysis and comparison were done and the results were discussed extensively with the supervisor and a conclusion was drawn out eventually.

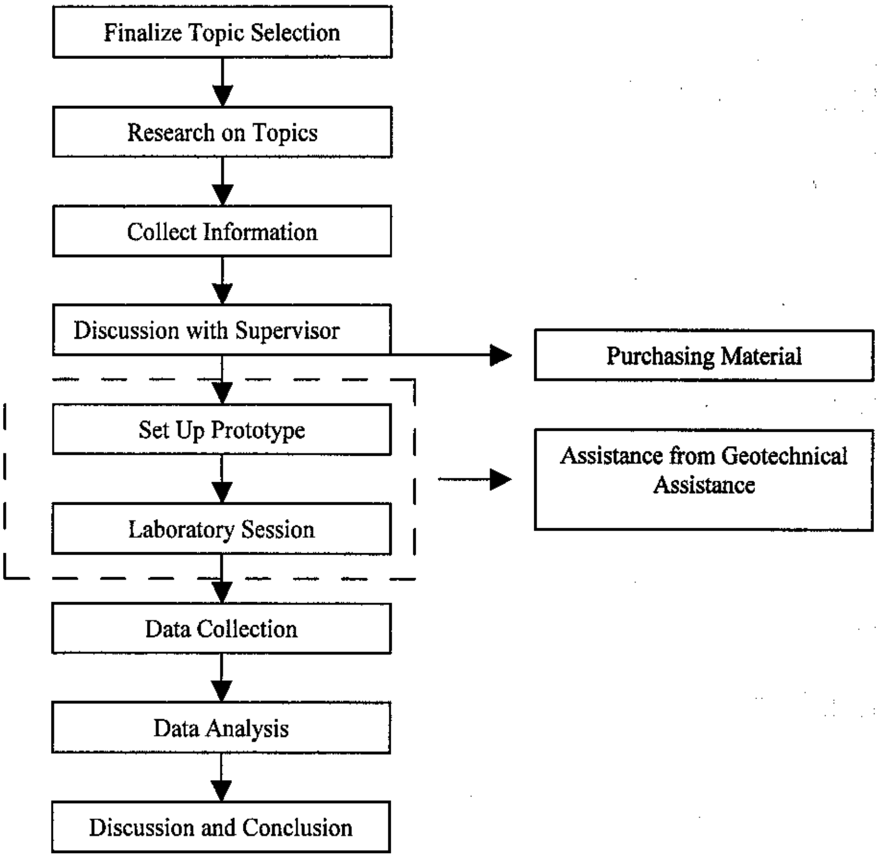


Figure 3.0: Procedure Identification

3.2 GEOTECHNICAL & FOUNDATION EARTH STRUCTURE

3.2.1 Lab 1: Determination of Moisture Content using Oven-Drying Method

Water is present in most naturally occurring soils. The amount of water, expressed as a proportion by mass of the dry solid particles, known as the moisture content, has a profound effect on soil behavior. Moisture content is required as a guide to classification of natural soils and as a control criterion in re-compacted soils and is measured on samples used for most field and laboratory tests. The oven-drying method is the definitive procedure used in standard laboratory practice

PORCEDURE:

1. The moisture content tin is cleaned and dried and it is weighed to the nearest 0.01g (m_1). A sample of at least 30g of soil is taken, crumble and place loosely in the container, the lid is replaced. Then the container and contents are weighed to the nearest 0.01g (m_2).
2. The lid is removed, and the container with its lid and contents are placed in the oven and dry at 105°C to 110°C for a period of 24 hours. Do not replace the lid while the sample is in the oven.
3. After drying, the container and contents are removed from the oven and the whole is placed in the desiccators to cool.
4. The lid is replaced and then the container and contents are weighed to the nearest 0.01g (m_3).
5. The moisture content of the soil specimen is calculated.

3.2.2 Lab 2: Specific Gravity

Three methods are described to determine the particle density/specific gravity of soils.

Gas jar method is suitable for most soils including those containing gravel-sized particles. Small pycnometer method is used for soils consisting of clay silt and sand-sized particles whereas the large pycnometer method is suitable for soils containing particles up to medium gravel size.

PROCEDURE:

1. A sample of soil of about 1.5kg is taken and the sample is sieved.
2. The sample is divided into 2 specimens, each weighing 400g by riffling.

3. The pyknometer is cleaned and dried and the whole assembly is weighed to the nearest 0.5g (m1).
4. The screw top is removed and the first specimen is transferred from its sealed container directly into the jar. The jar and its content and the screw – top assembly is weighed to the nearest 0.5g (m2).
5. Water is added at a temperature of within +- 2°C of the average room temperature during the test to about half fill of the jar. The mixture is stirred thoroughly with the glass rod to remove air trapped in the soil
6. The screw cap assembly is fitted and it is tightened so that the reference marks coincide. The pyknometer is filled with water.
7. The pyknometer is agitated by shaking. Air is allowed to escape and froth to disperse.
8. The pyknometer is topped up with water so that the water surface is flush with the hole in the conical cap. Make notes that air bubbles or froth are not trapped under the cap.
9. The pyknometer is dried on the outside and the whole is weighed to the nearest 0.5g (m3).
10. The pyknometer is emptied, it is washed thoroughly and it is filled completely with water at room temperature. Make sure that the reference marks on the screw cap coincide, that no air bubbles are entrapped, and that the water surface is flush with the hole in the conical cap.
11. The pyknometer is dried on the outside and weigh to the nearest 0.5g (m4).
12. Step 4 -12 is repeated by using the second speciment of the same soil so that two values of particle density can be obtained. If the results differ more than $0.05M/m^3$, the test is repeated.

3.2.2 Lab 3: Liquid Limit & Plastic Limit

1. **Determination of the liquid limit (Cone Penetrometer Method)**

The liquid limit is the empirically established moisture content at which a soil passes from the liquid state to the plastic state

PROCEDURE:

1. A sample of soil of sufficient size is taken to give a test specimen weighing at least 300g which passes the 425um test sieve and it is placed on the glass plate.
2. Some water is added and the paste is mixed for at least 10 minutes using the two spatulas.
3. A portion of the mixed soil is pushed into the cup with a spatula taking care not to trap air. Excess soil is struck off with the straightedge to give a smooth level surface.
4. With the penetration cone locked in the raised position, the supporting is lowered assembly so that the tip of the cone just touched the surface of the soil. When the cone is in the correct position, a slight movement of the cup will just mark the soil surface. The stem of the dial gauge is lowered to contact the cone shaft and zero shaft and the reading is zeroed.
5. The timer on the automatic controller is set to 5s and the release button is pressed. After 5s, the controller will lock the cone shaft.
6. The stem of the dial gauge is lowered to contact the cone shaft and the reading of the dial gauge is recorded to the nearest 0.1mm. This reading is recorded as the cone penetration.
7. A little more wet soil is added to the cup, taking care not to trap air and make the surface smooth as in step 3 and repeat step 4 to 7.
8. If the difference between the first and second penetration readings is less than 0.5mm, the average of the two penetrations are recorded as proceed step 10. If the second penetration is more than 0.5mm and less

than 1mm different from the first, a third test is carried out. If the overall range is then not more than 1mm, the average of the three penetrations is recorded and proceeds to step 10. If the overall range is more than 1mm, the soil is removed from the cup, remixed and step 3 – 8 are repeated until consistent results are obtained.

2. Determination of the plastic limit

The plastic limit is the empirically established moisture content at which a soil becomes too dry to be plastic. It is used together with the liquid limit to determine the plasticity index which when plotted against the liquid limit on the plasticity chart provides a means of classifying cohesive soils.

PROCEDURE:

1. A sample of the soil of sufficient size is taken to give a test specimen weighing at least 20g which passes the 425um test sieve and it is placed on the glass plate.
2. The soil is allowed to dry partially on the plate until it become plastic enough to be shaped into a ball.
3. The ball of soil is mould between the fingers and it is rolled between the palms of the hands until the heat of the hands has dried the soil sufficiently for slight cracks to appear on its surface. This sample is divided into two subs – samples of about 10g each and carries out a separate determination on each portion. Each sub – sample is divided into four more or less equal parts and each part is treated as specified in step 4 to 8.
4. The soil is mould in fingers to equalize the distribution of moisture content, then the soil is formed into a thread about 6mm diameter between the first finger and thumb of each hand.
5. The thread is rolled between the fingers, from finger – tip to the second joint, of one hand and the surface of the glass rolling plate. Enough pressure is used to reduce the movements of the hand. Some heavy

clays will require 10 – 15 movement when the soil is near the plastic limit because the soil hardens at this stage. It is important to maintain a uniform rolling pressure; do not reduce the pressure as the thread approaches 3mm.

6. The soil is picked up, it is mould between the fingers to dry further, it is formed into thread and it is roll out again as specified in step 5.
7. Step 6 is repeated until the thread shears both longitudinally and transversely when it has been rolled to about 3mm diameter, as gauged by rod. Do not gather the pieces of soil together after they have crumbled, in order to reform a thread and to continue rolling; the first crumbling point is the plastic limit.
8. The portion of the crumbled soil thread is gathered together, transfer them to a suitable container and the lid is replaced immediately.
9. Step 4 to 5 is repeated on the other three portions of soil, placing them all in the same container for the determination of moisture content.

3.2.4 Lab 4: Sedimentation by the Hydrometer Method

This method covers the quantitative determination of the particle distribution in a soil from the coarse sand size to the clay size.

RPOCEDURE:

1. Scale calibration of hydrometer
    a. The distance, L (in mm) is measured, from the 100ml scale marking to the 1000ml scale marking on the sedimentation cylinder, to the nearest mm. For a cylinder with a scale mark only at 1000ml, the 1000ml level is determined by adding a measured 100mL of water.
    b. The distances from the lowest calibration mark on the stem of hydrometer to each of the major calibration marks, Rh is measured and recorded.

c. The distances, N from the neck of the bulb to the bulb is measured and recorded to the nearest calibration mark.

d. The distance, H, corresponding to a reading, Rh, is equal to the sum of the distances measured in 2b and 2c, (N + d1, N + d2, etc.).

e. The distance, h from the neck to the bottom of the bulb is measured and recorded as the height of the bulb.

f. The effective depth, H_R (in mm) is calculated corresponding to each of the major calibration marks, Rh from the equation :

$$H_R = H + ½ [h – VhL/900]$$

Where

H is the length from the neck of the bulb to graduation Rh (in mm)

h is the length of the bulb

Vh is the volume of the hydrometer bulb (mL)

L is the distance between the 100mL and 1000mL scale markings of the sedimentation cylinder (in mm)

h = 152mm

Vh = 69g = 69mL

L = 317mm

N = 33mm

2. Meniscus correction

When T = 25°C,

Cm = 0.0005m = 0.5mm

3. Preparation and assembly

1. 50g of the test sample is weighed to 0.01g and its initial dry mass, m_o is obtained.
2. The test sample is placed in the wide – mouth conical flask.
3. 100ml of the sodium hexametaphoshate solution is added to the

soil in the conical flask. The mixture is shaken thoroughly until all the soil is in suspension.

4. The suspension is transferred from the flask to the 63μm test sieve placed on the receiver, and the soil in the sieve is washed using a jet of distilled water from the wash bottle. The amount of water used during this operation shall not exceed 500ml.
5. The suspension that has passed through the sieve is transferred to the 1L measuring cylinder and make up to the 1L graduation mark with distilled water. This suspension is used for the sedimentation analysis.
6. Any material passing the 63μm test sieve is added to the measuring cylinder.

4. Sedimentation

1. The rubber bung is inserted into the soil suspension; it is shaken and placed in the constant – temperature bath so that it is immersed in water at least up to the 1L graduation mark.
2. 100ml of the sodium hexametaphosphete solution is added to the second 1ml sedimentation cylinder and dilute with distilled water to exactly 1L.The rubber bung is inserted and place this cylinder in the constant temperature bath alongside the first.
3. After at least 1h, the cylinder containing the dispersion solution is taken out, it is shaken thoroughly and it is replaced in the bath. The cylinder containing the soil suspension is taken out, it is shaken vigorously end over end about 60 times in 2min and then immediately it is replaced in the bath.
4. At the instant the cylinder with the soil suspension is replaced upright in the bath, the timer is started. The rubber bungs is removed carefully from the cylinders.
5. The hydrometer is immersed in the suspension to a depth slightly below its floating position and it is allowed to float freely.

6. Hydrometer readings are taken at the upper rim of the meniscus after periods of 0.5min, 1min, 2min and 4min.
7. The hydrometer is removed slowly, rinse in distilled water and it is placed in the cylinder of distilled water with dispersion at the same temperature as the soil suspension. The top of the meniscus reading, R_o is observed and recorded.
8. The hydrometer is reinserted in the soil suspension and reading after periods of 8min, 30min, 2h, 8h and 24h from the start of sedimentation, and twice during the following day if appropriate. The precise times are not critical provided that the exact time period is recorded. The hydrometer is inserted slowly about 15s before a reading is due. The hydrometer is inserted and withdrew before and after taking each reading very carefully to avoid disturbing the suspension unnecessarily. 10s is allowed for each operation. Vibration of the sample is avoided.
9. The temperature of the suspension is observed and recorded once during the first 15minand then after every subsequent reading. The temperature is read accuracy of ±0.5°C.

3.2.5 Lab 5: Vane Shear Strength

This method covers the measurements of the shear strength of a sample of soft to firm cohesive soil without having to remove it from its container or sampling tube. The sample therefore does not suffer disturbance due to preparation of a test specimen. The method may be used for soils that are too soft or too sensitive to enable a satisfactory compression test specimen to be prepared.

PROCEDURE:

1. The sample container is attached securely to the base of the vane apparatus, with the sample axis vertical and located centrally under the axis of the vane.

2. The upper surface of the sample is trimmed flat and perpendicular to the axis.
3. A torsion spring that is most appropriate for the estimated strength of the soil is selected and assembled it into the vane apparatus.
4. The pointer and the graduated scale on the torsion head is set to their zero readings, and ensure that there is no backlash in the mechanism for applying torque
5. The vane assembly lowered until the end of the vane just touches the surface of the sample. This provides the datum from which the depth of penetration of the vane can be measured.
6. The vane assembly lowered further to push the vane steadily into the sample to the required depth. The top of the vane should be at distance not less than four times the blade width below the surface. Record the depth of penetration.
7. Torque is applied to the vane by rotating the torsion head at the rate of 6 °/min to 12 °/min, until the soil has sheared.
8. The maximum angular deflection of the torsion spring is recorded and the angle of rotation of the vane at the instant of failure.
9. The vane is raised steadily. As it emerges from the sample prevent excessive disturbance due to tearing of the surface. Wipe the blades clean.
10. The sample from its container is extrude and the specimens is taken from the level at which the tests were carried out for determining the soil moisture content.
11. The visual description of the soil is recorded at the same level.

3.2.6 Lab 6: Permeability Test (Falling Head Method)

Permeability refers to the propensity of a material to allow fluid to move through its pores. In the context of soil, permeability generally relates to the propensity of a soil to allow water to move through its void spaces. In this experiment, water is forced, by a falling head dimensions and the rate of flow is determined. This test may be used to determine the permeability of both fine grained soils (such as silts and clays) and coarse grained soils.

PROCEDURE:

1. Measure the length of the sample (L). Do not include the porous stones at the top and the bottom in your measurement.
2. Use de-ionized room-temperature water. Carefully pour water to fill the burette (the long glass tube). Be careful not to trap air bubbles in the burette or the attached tube. Release the clamp to allow water to flow through the sample. Caution! Do not allow the burette to go dry! Stop flow by clamping the tube before the burette empties.
3. Pick a point near the top of the tube, but at least 15 cm from the top. Make a mark with a grease pencil at this point. Measure the height of this point of above the outflow port (h_1).Pick another point near the bottom of the tube, but at least 15 cm up from the bottom. Mark this point too, and measure its height above the outflow port (h_2). Note that the burette is calibrated in milliliters.
4. In this test, you will measure the time (t) it takes for the top of the water column to fall from the top mark to the bottom mark.
5. Get ready to time. *Caution! During this test, do not allow the burette to go dry! Stop the flow before the burette empties.* When you are ready to begin the test, open the clamp and allow water to fall through the burette. When it reaches the top mark, begin timing. When it reaches the bottom mark, stop timing.

6. Run several trials, until you are sure you have at least three good measurements.
7. Calculate the hydraulic conductivity of the sediment by using the formula.
8. Check your value against a chart giving common ranges of values (in your textbook, for example) to see if your answer is reasonable.

3.2.7 Lab 7: To Determine the Rate of Consolidation by Using PVD &Electro-Osmosis Method

The test follows the matrix as shown below:

Experiment	PVD	Electro –osmosis	Surcharge
1	NO	NO	YES
2	YES	NO	YES
3	NO	NO	YES
4	YES	NO	YES
5	NO	YES	NO
6	NO	YES	YES

Table 3.0: Experiment Matrix

CHAPTER 4

RESULTS AND DISCUSSION

4.1 RESULTS

4.1.1 Determination of Moisture Content

The moisture content of the soil specimen, w, as a percentage of the dry soil mass to the nearest 0.1% can be calculated from the equation:

$$w = [(m_2 - m_3) / (m_3 - m_1)]\ 100$$

Average moisture content= **31.40%**

4.1.2 Specify Gravity

Formula to calculate the specific gravity:

$$\rho_s = \frac{m_2 - m_1}{(m_4 - m_1) - (m_3 - m_2)}$$

Average specify gravity ~ **2.60**

4.1.3 Liquid Limit & Plastic Limit

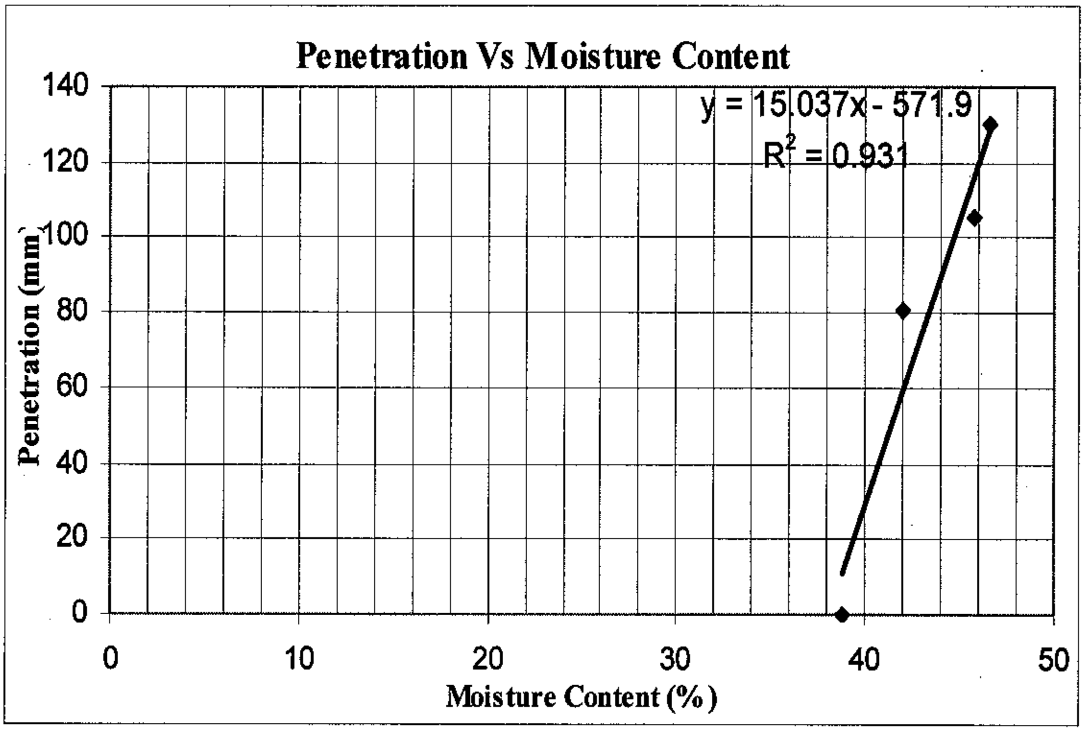


Figure 4.0: Penetration of Cone vs. Moisture Content

Thus the liquid limit, *(LL)* of the soil sample = **40%**

Plastic limit of the soil sample, (PL) = **29 %**

Plasticity index, PI = **11%**

4.1.4 Sedimentation by the Hydrometer Method

Particle diameter, D (mm)	Percentage finer than D (%)	Cumulative percentage (%)	Percentage Passing (%)
0.077	3.7323	3.73	96.27
0.054	3.7323	7.46	92.54
0.039	3.7004	11.16	88.84
0.027	3.6685	14.83	85.17
0.019	3.6685	18.50	81.50
0.010	3.509	22.01	77.99
0.005	3.19	25.20	74.80
0.002	3.19	28.39	71.61

Table 4.0: Table of percentage finer than D versus particle diameter

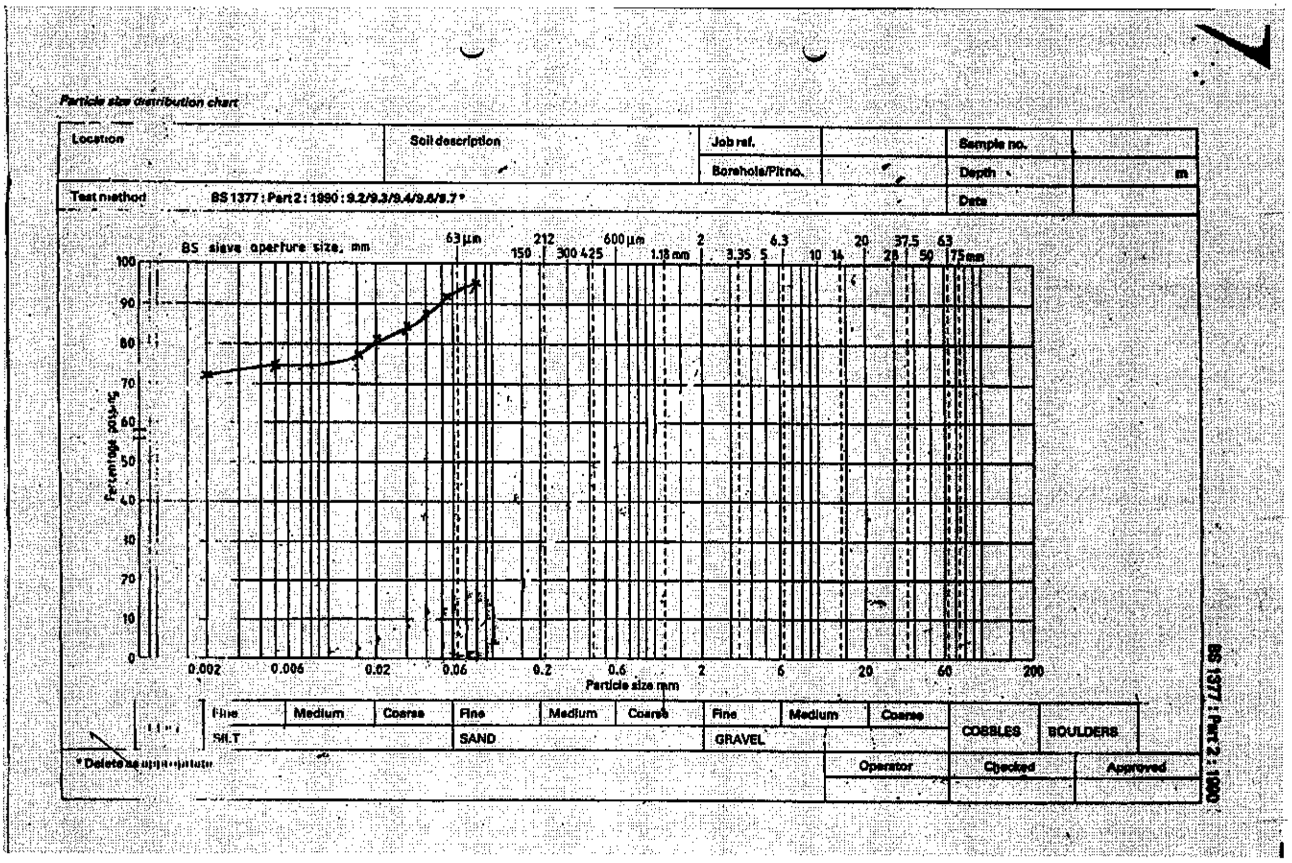


Figure 4.1: Particle Size Distribution

From the particle size distribution chart, thus the soil sample can be classified as silt.

4.1.5 Vane Shear Test

Calculation of Vane shear strength of soil

Deflection of spring = 70 ° = θ_f

Rotation of vane = 14 °

Rotation of spring mounting = 70 ° + 14 °

= 84°

Torque = 2.28 kg.cm (approximately obtained from *Figure 4.6*)

M = 0.228 Nm

$\tau v = (M/4.29) = 53.15\ kN/m^2$

$= \mathbf{53\ kPa}$

The vane shear strength of the soil sample is 53 kPa and it classified as stiffsoil according to *(Figure 4.7)*.

4.1.6 Permeability Test (Falling Head Method)

$$K = 2.3\frac{aL}{At}\log_{10}\frac{h_0}{h_f}$$

Length of specimen, L =120 mm

Total time for discharge, t = 72 s

Cross sectional area of specimen, $A= \pi/4\ (100mm)^2$
$= 7855\ mm^2$

Cross sectional area of the standpipe, a = 6mm

Height of top mark above outflow port, h_1= 127mm

Height of bottom mark above outflow port, h_2 = 38.5mm

$$k = 2.3\ \frac{(6mm)(120mm)}{(7855mm^2)((72\sec)}\log_{10}\frac{127mm}{38.5mm}$$

$= (2.9 \times 10^{-3})\ (0.52)$

$= 1.51 \times 10^{-3}$ mm /sec

= 0.00015 cm /sec

Refer to the *Table 4.20*, the results showed the soil samples fall into the category of fine sand or silty clay.

4.1.7 To Determine Rate of Consolidation by Using PVD and Electro-Osmosis Method

4.1.7.1 Experiment 1 [Without PVD with Surcharge (Set 1)]

Calculation of moisture content after completion laboratory

Mass of wet soil + container (g), m_2	51.16
Mass of dry soil + container (g), m_3	46.70
Mass of container (g), m_1	37.29
Mass of moisture (g)	4.46
Mass of dry soil (g)	9.41
Moisture content (%)	**47.40**

Table 4.1: Moisture Content for Experiment 1

4.1.7.2 Experiment 2 [With PVD with Surcharge(Set 1)]

Calculation of moisture content after completion laboratory

Mass of wet soil + container (g), m_2	45.79
Mass of dry soil + container (g), m_3	43.53
Mass of container (g), m_1	37.68
Mass of moisture (g)	2.26
Mass of dry soil (g)	5.85
Moisture content (%)	**38.63**

Table 4.2: Moisture Content for Experiment 2

4.1.7.3 Experiment 3 [Without PVD with Surcharge(Set 2)]

Calculation of moisture content after completion laboratory

Mass of wet soil + container (g), m_2	55.63
Mass of dry soil + container (g), m_3	49.86
Mass of container (g), m_1	37.67
Mass of moisture (g)	5.77
Mass of dry soil (g)	12.19
Moisture content (%)	**47.33**

Table 4.3: Moisture Content for Experiment 3

4.1.7.4 Experiment 4 [With PVD with Surcharge(Set2)]

Calculation of moisture content after completion laboratory

Mass of wet soil + container (g), m_2	46.12
Mass of dry soil + container (g), m_3	43.84
Mass of container (g), m_1	37.73
Mass of moisture (g)	2.28
Mass of dry soil (g)	6.11
Moisture content (%)	**37.32**

Table 4.4: Moisture Content for Experiment 4

4.1.7.4.1 Liquid Limit & Plastic Limit

LIQUID Limit	**1**		**2**		**3**		**Average**
Initial dial gauge reading (mm)	0	0	0	0	0	0	
Final dial gauge reading (mm)	124	124.4	100	101.1	116	116.5	113.67
Container no.	**A**		**B**		**C**		
Mass of wet soil + container (g), m_2	42.31		41.47		41.83		
Mass of dry soil + container (g), m_3	40.81		40.34		40.60		
Mass of container (g), m_1	37.43		37.68		37.72		
Mass of moisture (g)	1.50		1.13		1.23		
Mass of dry soil (g)	3.38		2.66		2.88		
Moisture content (%)	44.38		42.48		42.71		43.19

Table 4.5: Liquid Limit for Experiment 4

According to *Figure 4.2*, the moisture content corresponding to a cone penetration of 20 mm = **38%**

Thus the liquid limit, *(LL)* of the soil sample = **38%**

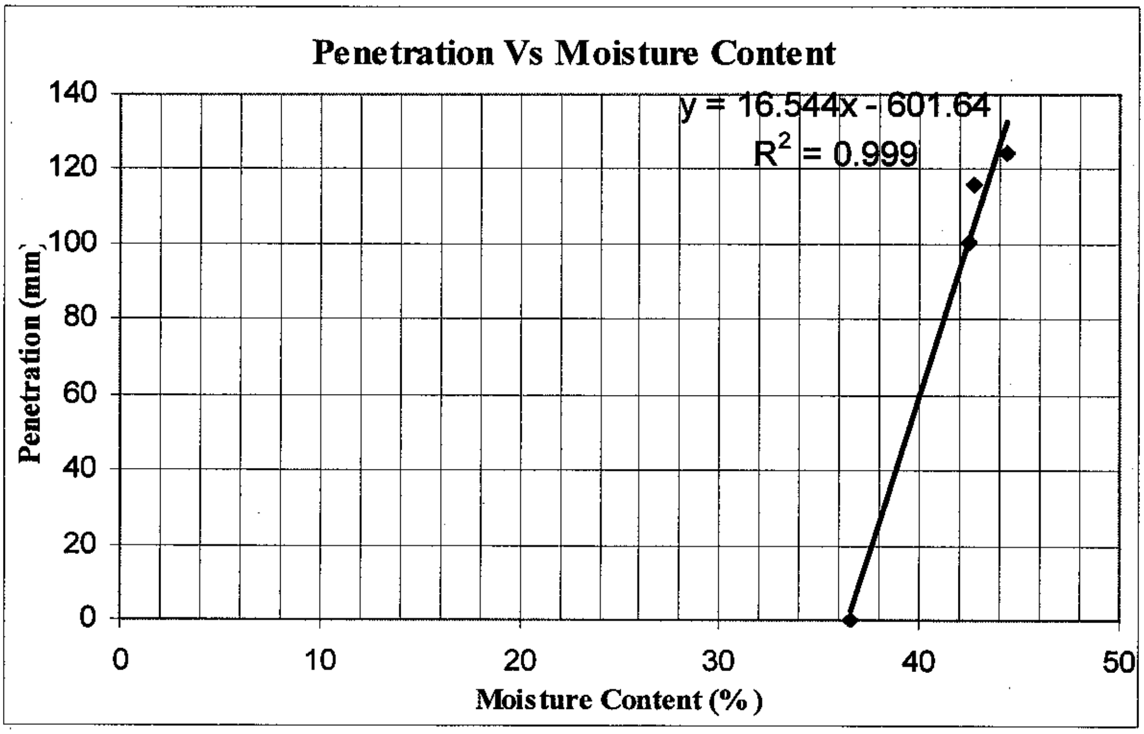


Figure 4.2: Penetration vs. Moisture Content

PLASTIC Limit	**1**	**2**	**3**	**4**	**Average**
Container no.	A	B	C	D	
Mass of wet soil + container (g), m_2	41.46	43.59	42.42	41.03	
Mass of dry soil + container (g), m_3	40.55	42.25	41.35	40.15	
Mass of container (g), m_1	37.43	37.67	37.73	37.21	
Mass of moisture (g)	0.91	1.34	1.07	0.88	
Mass of dry soil (g)	3.12	4.58	3.62	2.94	
Moisture content (%)	29.17	29.26	29.56	29.93	29.48

Table 4.6: Plastic Limit for Experiment 4

Thus the plastic limit, (PL) = 30 %

Plasticity index, PI = LL – PL

= (38-30) %

= 8%

4.1.7.4.2 Vane Shear Strength

Calculation of vane shear strength of soil

Deflection of spring = $50^0 = \theta_f$

Rotation of vane = 42^0

Rotation of spring mounting = $50^0 + 42^0 = 92^0$

Torque = K θf Nmm, K = 4290mm

Torque = 1.6 kg.cm (approximately obtained from *Figure 4.6*)

M = 0.16 Nm

= 160 Nmm

τ_v = (M/4.29) = 160/(4.29) kN/m^2

= 37.30 kN/m^2

= 37 kPa

The vane shear strength of the soil sample is 37 kPa and it classified as firm soil according to *(Figure 4.7).*

4.1.7.5 Experiment 5 [Electro-Osmosis without Surcharge]

Calculation of moisture content after completion laboratory

Mass of wet soil + container (g), m_2	44.39
Mass of dry soil + container (g), m_3	42.62
Mass of container (g), m_1	37.69
Mass of moisture (g)	1.77
Mass of dry soil (g)	4.93
Moisture content (%)	**35.90**

Table 4.7: Moisture Content for Experiment 5

4.1.7.5.1 Liquid Limit & Plastic Limit

LIQUID Limit	**1**		**2**		**3**		**Average**
Initial dial gauge reading (mm)	0	0	0	0	0	0	
Final dial gauge reading (mm)	124	124	106.5	107	118.5	119	116.5
Container no.	**A**		**B**		**C**		
Mass of wet soil + container (g), m_2	40.58		40.89		40.61		
Mass of dry soil + container (g), m_3	39.56		39.57		39.58		
Mass of container (g), m_1	37.39		36.67		37.37		
Mass of moisture (g)	1.02		1.32		1.03		
Mass of dry soil (g)	2.17		2.9		2.21		
Moisture content (%)	47		45.52		46.61		45.71

Table 4.8: Liquid Limit for Experiment 5

According to *Figure 4.3*, the moisture content corresponding to a cone penetration of 20 mm = **39.6%**

Thus the liquid limit, *(LL)* of the soil sample = **40%**

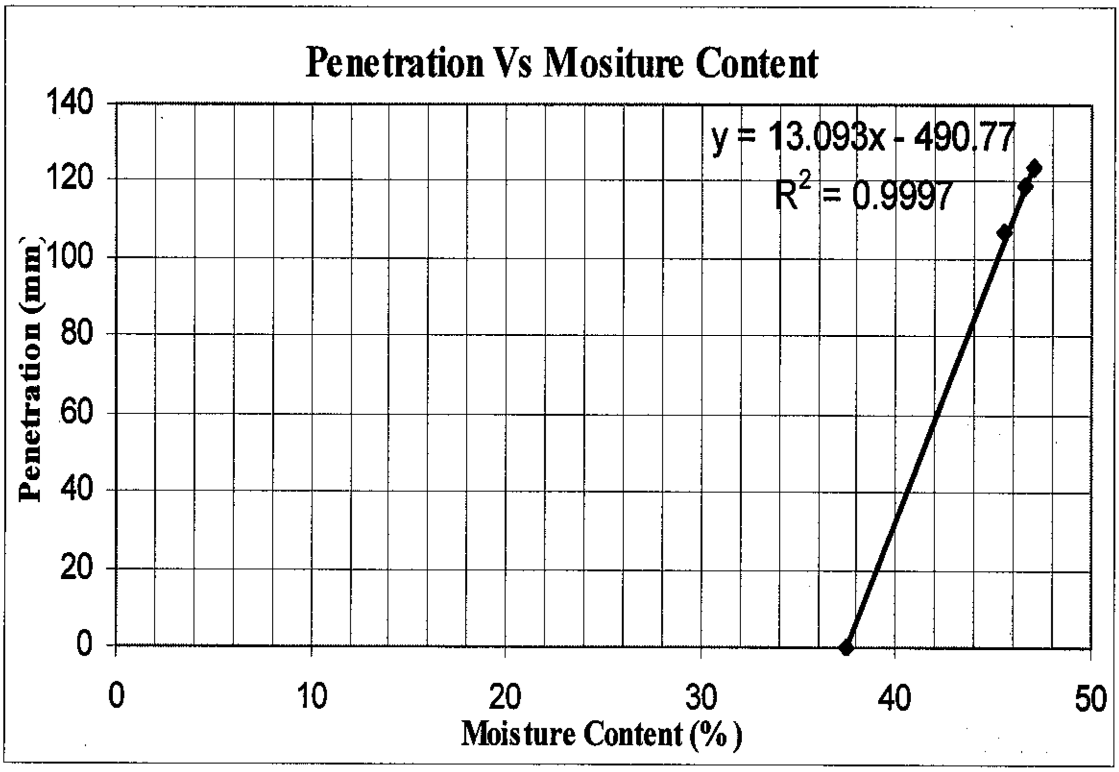


Figure 4.3: Penetration vs. Moisture Content

PLASTIC Limit	1	2	3	4	**Average**
Container no.	A	B	C	D	
Mass of wet soil + container (g), m_2	41.63	40.86	42.22	41.03	
Mass of dry soil + container (g), m_3	40.62	40.10	41.26	40.12	
Mass of container (g), m_1	37.43	37.67	37.72	37.22	
Mass of moisture (g)	1.01	0.76	0.96	0.91	
Mass of dry soil (g)	3.19	2.43	3.54	2.90	
Moisture content (%)	31.66	31.28	27.12	31.38	30.36

Table 4.8: Plastic Limit for Experiment 5

Thus the plastic limit, (PL) = **30 %**

Plasticity index, PI = LL – PL

= (40-30) %

= **10%**

4.1.7.5.2 Vane Shear Strength

Calculation of vane shear strength of soil

Deflection of spring = $61^0 = \theta_f$

Rotation of vane = 56^0

Rotation of spring mounting = $61^0 + 56^0$

= 117^0

Torque = K θf Nmm, K = 4290mm

Torque = 1.875 kg.cm cm (approximately obtained from *Figure 4.6*)

M = 0.1875 Nm

= 18.75 Nmm

τ_v = (M/4.29) = 18.75/(4.29) kN/m^2

= 43.71 kN/m^2

= **44 kPa**

The vane shear strength of the soil sample is 44 kPa and it classified as firm soil according to *(Figure 4.7).*

4.1.7.6 Experiment 6 [Electro-Osmosis with Surcharge]

Calculation of moisture content after completion laboratory

Center of soil

Mass of wet soil + container (g), m_2	49.61
Mass of dry soil + container (g), m_3	46.68
Mass of container (g), m_1	37.72
Mass of moisture (g)	2.93
Mass of dry soil (g)	8.96
Moisture content (%)	32.71

Table 4.9: Moisture Content (Collected from centre) for Experiment 6

Side of soil

Mass of wet soil + container (g), m_2	60.01
Mass of dry soil + container (g), m_3	54.57
Mass of container (g), m_1	37.38
Mass of moisture (g)	5.44
Mass of dry soil (g)	17.19
Moisture content (%)	31.65

Table 4.10: Moisture Content (Collected from side) for Experiment 6

4.1.7.6.1 Liquid Limit & Plastic Limit

LIQUID Limit	**1**		**2**		**3**		**Average**
Initial dial gauge reading (mm)	0	0	0	0	0	0	
Final dial gauge reading (mm)	88.5	89	95.5	96	109	109.5	97.92
Container no.	**A**		**B**		**C**		
Mass of wet soil + container (g), m_2	39.26		39.33		41.05		
Mass of dry soil + container (g), m_3	38.62		38.50		40.17		
Mass of container (g), m_1	37.23		36.67		38.21		
Mass of moisture (g)	0.64		0.83		0.88		
Mass of dry soil (g)	1.39		1.83		1.96		
Moisture content (%)	46.04		45.36		44.9		45.43

Table 4.11: Liquid Limit for Experiment 6

According to *Figure 4.4*, the moisture content corresponding to a cone penetration of 20 mm = **39.8%**

Thus the liquid limit, *(LL)* of the soil sample = **40%**

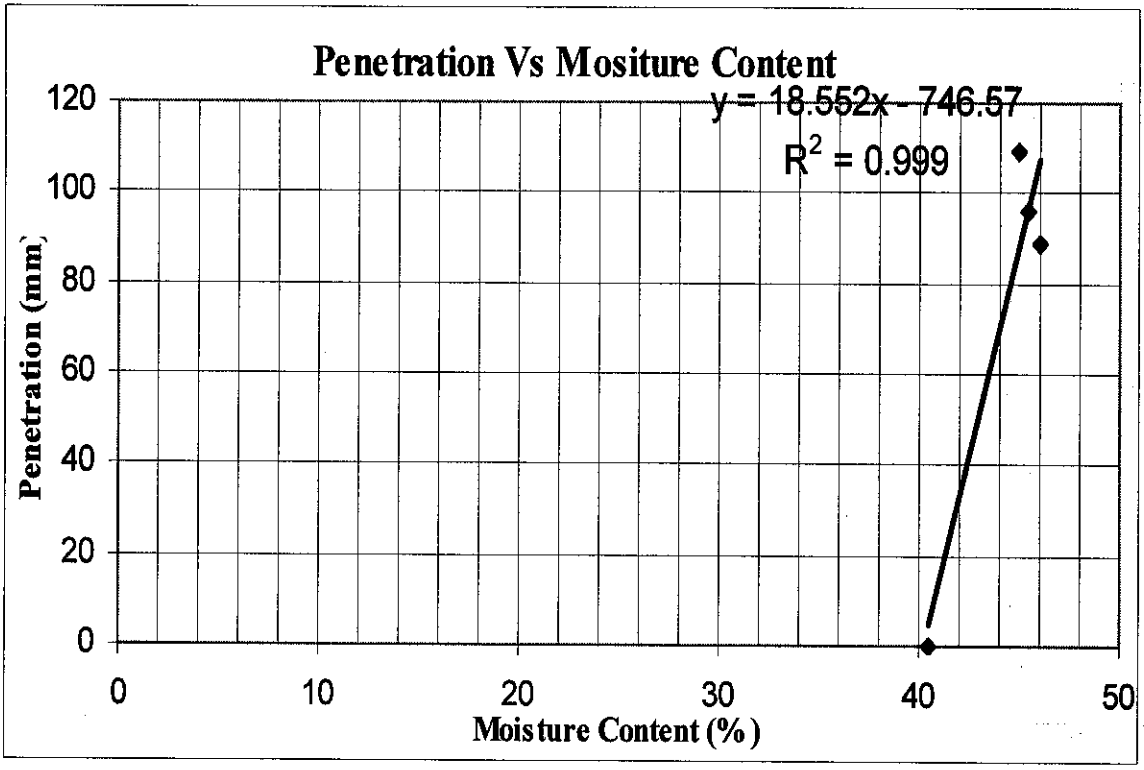


Figure 4.4: Penetration vs. Moisture Content

PLASTIC Limit	1	2	3	4	**Average**
Container no.	A	B	C	D	
Mass of wet soil + container (g), m_2	41.40	40.53	42.64	41.60	
Mass of dry soil + container (g), m_3	40.58	39.90	41.51	40.81	
Mass of container (g), m_1	37.43	37.66	37.33	37.64	
Mass of moisture (g)	0.82	0.63	1.13	0.79	
Mass of dry soil (g)	3.15	2.24	4.18	3.17	
Moisture content (%)	26.03	28.13	27.03	24.92	26.53

Table 4.12: Plastic Limit for Experiment 6

Thus the plastic limit, (PL) = **27 %**

Plasticity index, PI = LL – PL

= (40-27) %

= **13%**

4.1.7.6.2 Vane Shear Strength

Calculation of vane shear strength of soil

Deflection of spring = $67^0 = \theta_f$

Rotation of vane = 61^0

Rotation of spring mounting = $67^0 + 61^0$

= 128^0

Torque = K θf Nmm, K = 4290mm

Torque = 2.025 kg.cm cm (approximately obtained from *Figure 4.6*)

M = 0.2025 Nm

= 20.25 Nmm

$\tau_v = (M/4.29) = 20.25/(4.29)$ kN/m^2

= 47.20 kN/m^2

= **47kPa**

The vane shear strength of the soil sample is 47 kPa and it classified as firm soil according to *(Figure 4.7).*

4.2 DISCUSSION

4.2.1 Theory: (Lab 1)

Moisture content, w is also known as water content. It is the ratio of the weight of water to the weight of solids in a given volume of soil. Different type of soil has different range of moisture content. Thus, by knowing the moisture content of the soil, the type of soil can be determined. The moisture content is essential in various calculations in soil mechanics. This is because the moisture content is related to the unit weight, void ratio, specific gravity and porosity. It plays an important role in the derivation of the various unit- weight relationships

Type of soil	Natural Moisture Content in a saturated state (%)
Loose uniform sand	30
Dense uniform sand	16
Loose angular-grained silty sand	25
Dense angular-grained silty sand	15
Stiff clay	21
Soft clay	30-50
Loess	25
Soft organic clay	90-120
Glacial till	10

Table 4.13: The Moisture Content for some typical soils in a Natural State

In the experiment, the results may not be very accurate as there are a few errors during measurement and using the apparatus. The digital weighing machine used is precise but it is also very sensitive to the slightest change in the environment such as tiny movements on the table can affect the reading. Besides that, the container and soil used may contain foreign weights such as tiny stones, twigs, worms, leaves, and droplets of water at the side of the container.

The moisture content of the soil sample used in this experiment is about 31.40%. This means that the soil sample is soft clay as refer to the *Table 4.1.3*.

4.2.2 Theory: (Lab 2)

The specific gravity of soil actually refers to the specific gravity of the solid matter of the soil, which is designated as G_s. Specific gravity is defined as the ratio of the unit weight of a given material to the unit weight of water. Generally, geotechnical engineers need the soil's specific gravity to perform additional testing of that soil.

A soil's specific gravity largely depends on the density of the minerals making up the individual soil particles. However, as a general guide, some typical values for specific soil types are as follows:-

Types of Soil	**Specific Gravity**
Solid substance of most inorganic soils	2.60 to 2.80
Tropical iron-rich laterite	2.75 to 3.0 but can be higher sometimes
Sand particles composed of quartz	2.65 to 2.67
Inorganic clays	2.70 to 2.80
Soils with large amount of organic matter or porous particles	Below 2.60

Table 4.14: Specific Gravity of Varies Types of Soil

Mineral	Specific Gravity, Gs
Quartz	2.65
Kaolinite	2.6
Illite	2.8
Montmorillonite	2.65 – 2.80
Halloysite	2.0 – 2.55
Potassium feldspar	2.57
Sodium and calcium feldspar	2.62 – 2.76
Chlorite	2.6 – 2.9

Table 4.15: Specific Gravity of Common Materials

During the experiment, there were several errors in the experimental value. This is because the soil and water in the pycnometer was not left overnight to settle down as according to the BS standards. Asides from that, the volume of water in the pycnometer is not constant. This is because the pycnometer tends to leak out some water from the cap when it is full. In addition to that, the pycnometer is very difficult to be filled completely with water, especially at the cap because bubbles tend to form there. Consequently, the results were not as accurate as it should be.

The value acquired during the experiment was done was 2.58 (~2.6). Hence, based on the experimental value obtained, the type of sample soil used will fall into the category of soils with solid substance of most inorganic soils *(Table 4.1.4)* and the type of mineral is Kaolinite *(Table 4.1.5)*.

4.2.3 Theory: (Lab 3)

According to the British Standard (BS1377), the cone penetration method is a popular way of determining the liquid limit *(LL)* in Europe and Asia. In this test the liquid limit is defined as the moisture content at which a standard cone of apex angle 30° and weight of 0.78 N (80 gf) will penetrate a distance, d, of 20 mm in 5 seconds when it is allowed to drop from a position of point contact with the soil surface. Due to the difficulty in achieving the liquid limit from a single test, four or more tests can be conducted at various moisture contents to determine the distance of the cone penetration, d. A linear graph can then be plotted with the distance of cone penetration versus moisture content. The plot results in a straight line *(as shown in Figure 4.0, 4.2, 4.3, 4.4)*. The moisture content corresponding to d = 20 mm is the liquid limit.

The plastic limit *(PL)* is defined as the moisture content in percent, at which the soil crumbles, when rolled into threads of 3.2 mm (1/8 in.) in diameter. The plastic limit is the lower limit of the plastic stage of soil. The plastic limit test is simple and is performed by repeated rolling of an ellipsoidal-size soil mass by hand on a ground glass plate.

Normally, the consistency of most soils in the ground will be plastic or semi-solid. Hence, soil strength and stiffness behavior are related to the range of plastic consistency. The range of water content over which a soil has a plastic consistency is termed as the Plasticity Index *(PI)*. As a general rule, the plasticity index *(PI)* is the difference between the liquid limit and the plastic limit of a soil, or

$$\boxed{PI = LL - PL}$$

On the whole, the plasticity index is important in classifying fine-grained soils. It is fundamental to the Casagrande plasticity chart, which is currently the basis for the Unified Soil Classification System. *(Table 4.1.6)* gives the ranges of liquid limit and plastic limit of some clay minerals.

Mineral	**Liquid limit, *LL***	**Plastic limit, *PL***
Kaolinite	35 – 100	20 – 40
Illite	60 – 120	35 – 60
Montmorillonite	100 – 900	50 – 100
Halloysite (hydrated)	50 – 70	40 – 60
Halloysite (dehydrated)	40 – 55	30 – 45
Attapulgite	150 – 250	100 – 125
Allophane	200 – 250	120 – 150

Table 4.16: Typical values of liquid limit and plastic limit of some clay minerals.

Plasticity Index	**Description**
0	Nonplastic
1 – 5	Slightly plastic
5 – 10	Low plasticity
10 – 20	Medium plasticity
20 – 40	High plasticity
> 40	Very high plasticity

Table 4.17: Classification of the plasticity index in a qualitative manner.

Liquid limit	Description
< 35 %	Low plasticity
35 – 50 %	Intermediate plasticity
50 -70 %	High plasticity
70 – 90 %	Very high plasticity
> 90 %	Extremely high plasticity

Table 4.18: Classification of the liquid limit in a qualitative manner.

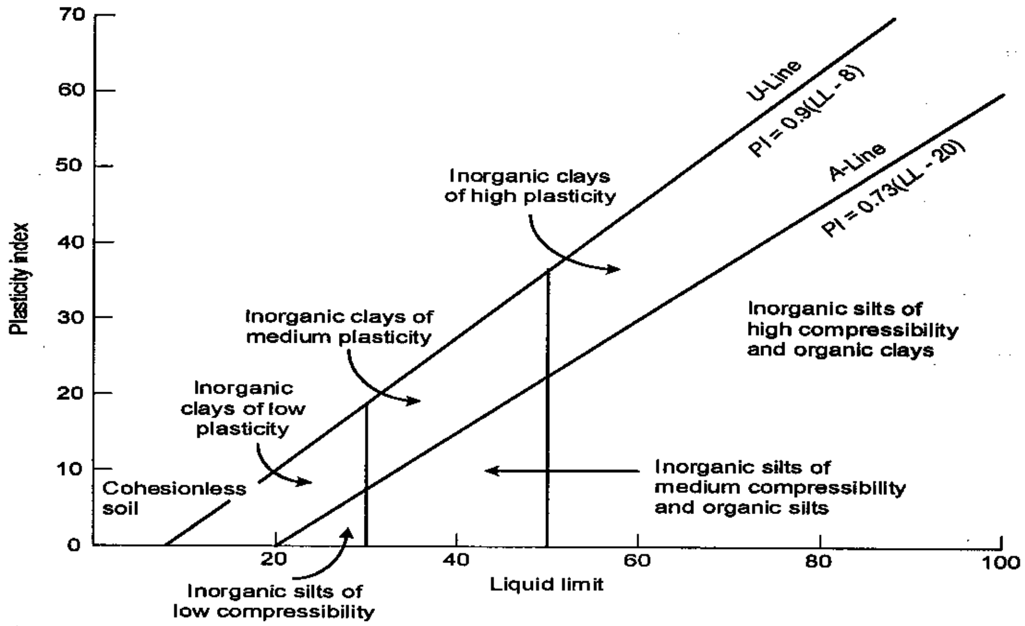


Figure 4.5: Plasticity Chart

For the liquid limit experiment, there may have inaccuracies in the results as there may be air voids when the soil is compacted into the container. In addition, the probable inaccuracies may have occurred while rolling the soil.

Hence, with the values of *LL* and *PI* obtained from the tests, the classification of the soil sample (fine soil) can be determined.

According to the plasticity chart, the soil sample is classified as inorganic clays of medium plasticity.

4.2.4 Theory: (Lab 4)

The hydrometer method of particle size analysis is a rapid and fairly accurate method used to determine textural class. A hydrometer with a scale in grams per liter is used to determine the amount of soil in suspension. The greater the density of the suspension, the greater the buoyant force on the hydrometer and the higher the reading. As particles settle out of the suspension, the density decreases and a lower reading is obtained. Since temperature influences the settling rate, a temperature correction must be made if the suspension temperature differs from the temperature for which the hydrometer is calibrated. Sodium hexametaphosphate is generally used as the dispersing agent. Besides, hydrometer is also used to measure suspension density at various times, thus reflecting the amount of particles which remain in suspension after a certain settling time. Hydrometer method usually preferred for routine analyses of high silt and clay soils due to its simplicity and rapidness

Hydrometer analysis is based on the principle of sedimentation of soil grains in water. When a soil specimen is dispersed in water, the particles settle at different velocities depending on their shape, size and weight and the viscosity of the water. Hydrometers are designed to give the amount of soil, in grams that still in suspension. They are calibrated for soils that have a specific gravity Gs of 2.65 for soils of other specific gravity, it is necessary to a correction must be made. Hydrometer analysis is effective for separating soil frictions down to a size of about 0.5U.

There are some discrepancies in the result due to the errors occurring during the experiment. There was a little amount of soil that has spilled out while being transferred into the cylinder. Besides, the cylinder containing soil suspension may not have been well shaken. The hydrometer readings may not be accurately taken down at the period of time the reading should be taken as the hydrometer is always moving and it is hard to take down the readings.

From the particle size distribution chart *(Figure 4.1)*, the soil sample can be classified as silt.

4.2.5 Theory: (Lab 5)

Vane shear tests can obtain fairly reliable results for undrained shear strength of very soft to medium cohesive soils. The test consists of advancing a four-bladed vane into the soil at a desired depth and applying a measured torque at a constant rate. It covers the measurements of the shear strength of the sample without having to remove it from its container or sampling tube. Thus, the sample does not suffer disturbances due to preparation of a test specimen. The method is applicable to too soft or too sensitive to enable a satisfactory compression test specimen to be prepared.

This test method covers the miniature vane test in very soft to stiff saturated fine-grained clayey soils ([phi] = 0). Knowledge of the nature of the soil in which each vane test is to be made is necessary for assessment of the applicability and interpretation of the test results. It is recommended that the miniature vane test be conducted in fine-grained; predominately clay soils with an undrained shear strength less than 100 kPa which are defined as stiff according to Practice D2488. Vane failure conditions in higher strength clay and predominantly silty soils may deviate from the assumed cylindrical failure surface, thereby causing error in the measured strength.

General descriptive term for strength	Suggested spring reference	Probable maximum shear stress (kN/m^2)
Very soft	(a) (Weakest)	20
Soft	(B)	40
Soft to firm	(C)	60
Firm	(D) (Stiffness)	90

Table 4.19: Typical Springs for Laboratory Vane

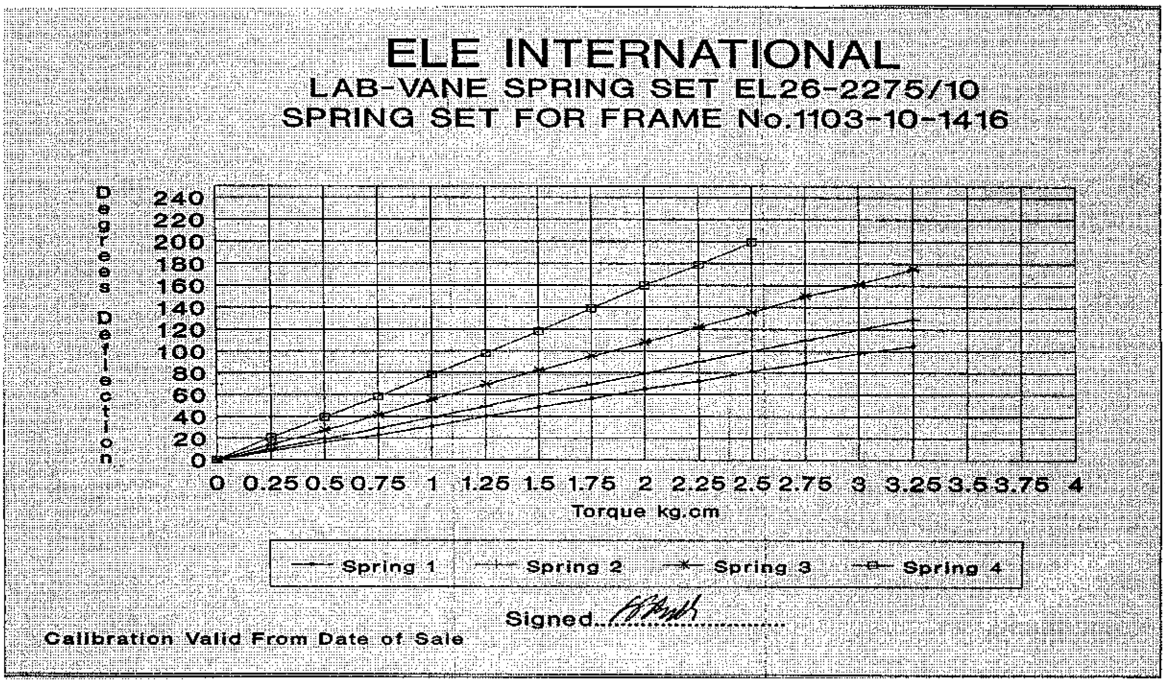
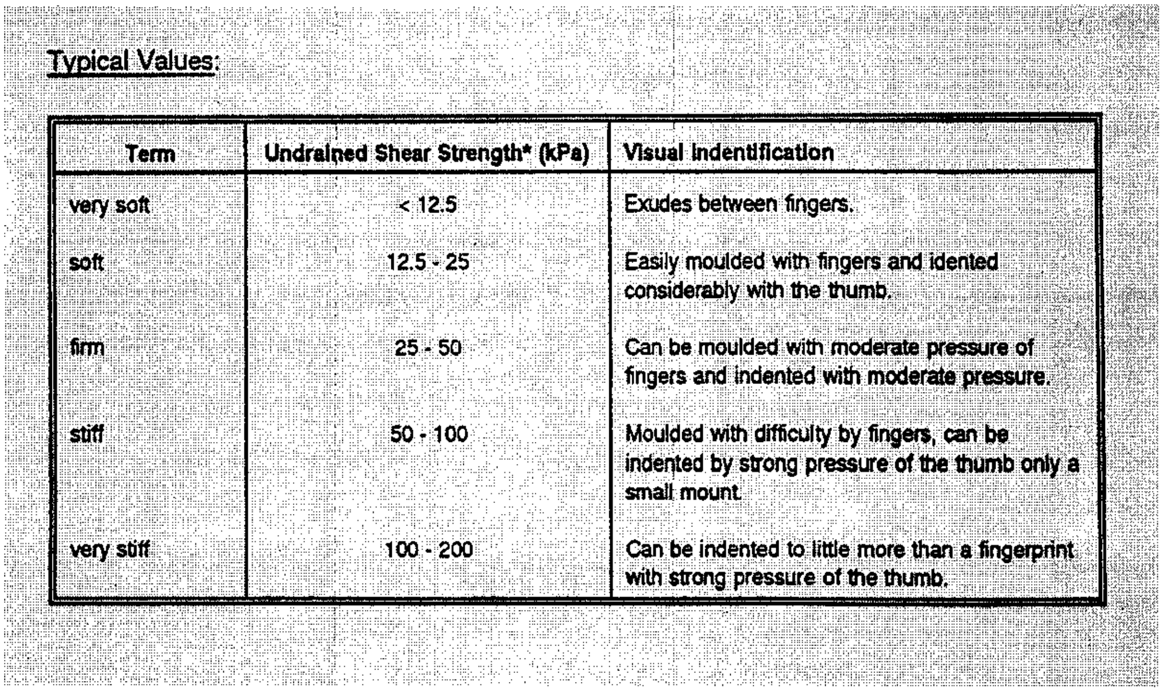


Figure 4.6: Calibration Chart



Typical Values:

Term	Undrained Shear Strength* (kPa)	Visual Indentification
very soft	< 12.5	Exudes between fingers.
soft	12.5 - 25	Easily moulded with fingers and idented considerably with the thumb.
firm	25 - 50	Can be moulded with moderate pressure of fingers and indented with moderate pressure.
stiff	50 - 100	Moulded with difficulty by fingers, can be indented by strong pressure of the thumb only a small mount.
very stiff	100 - 200	Can be indented to little more than a fingerprint with strong pressure of the thumb.

Figure 4.7: Typical Value for Undrained Shear Strength

In the test, there were several errors that may cause inaccuracies in the results. For example, there may have been errors in reading the spring deflection and rotation of vane. The digital weighing machine used is precise but it is also very sensitive to the slightest change in the environment such as tiny movements on the table can affect the reading.

4.2.6 Theory: (Lab 6)

The facility with which water flows through soil is an engineering property known as permeability. Since water movement within soil is through interconnected voids, in general, the larger a soil's void spaces, the greater will be its permeability. Conversely, the smaller the void spaces, the lesser will be its permeability. Thus, coarse grained soils such as sand commonly exhibit high permeability, while fine grained soils like clay ordinary have lower permeability's.

Flow of water in soil between two points occurs as a result of a pressure (or hydraulic head) difference between two points, with the direction of flow being from the higher to the lower pressure. Furthermore, the velocity of flow varies directly with the magnitude of the difference between hydraulics heads as well as with soils permeability's. Flow of water in soil can be analyzed quantitatively using Darcy's Law.

The coefficient of permeability of soils dependent on several factors: fluid viscosity, pore-viscosity, pore-size distribution, grain-size distribution, void ratio, roughness of mineral particles, and degree of soil saturation which may explain in below. In clayey soils, structure plays an important role in the coefficient of permeability. Other major factors that affect the permeability of clays are the ionic concentration and the thickness of layer of water held to the clay particles.

There are several factors that affect permeability. Porosity, which is the percentage of a solid that is open space, determines how much space there is for fluids to flow through. Besides, the size and shape of the pores is important too. Two rocks may have equal porosity, that is the same total amount of pore space, but the one with the larger pores might have higher permeability. This is because smaller pores offer more resistance to flow because of adhesion between the fluid and the sides of the pores. The shape of the pores also affects permeability for similar reasons. Thus, the more contact between the fluid and the pore surfaces, the lower the permeability. The other factors are the fluid viscosity. The higher the viscosity, the lower the k value. The higher for void ratio, the higher the k value.

Several errors could have affected the test results:

1. Air trapped in sample or sample not 100% saturated;
2. Soil was washed from the sample;
3. Some of the head loss occurred in the apparatus rather than in the sample;
4. Not starting and stopping stop watch at correct point;
5. Sample settling during test;
6. Sample disturbed by flowing water at inlet;

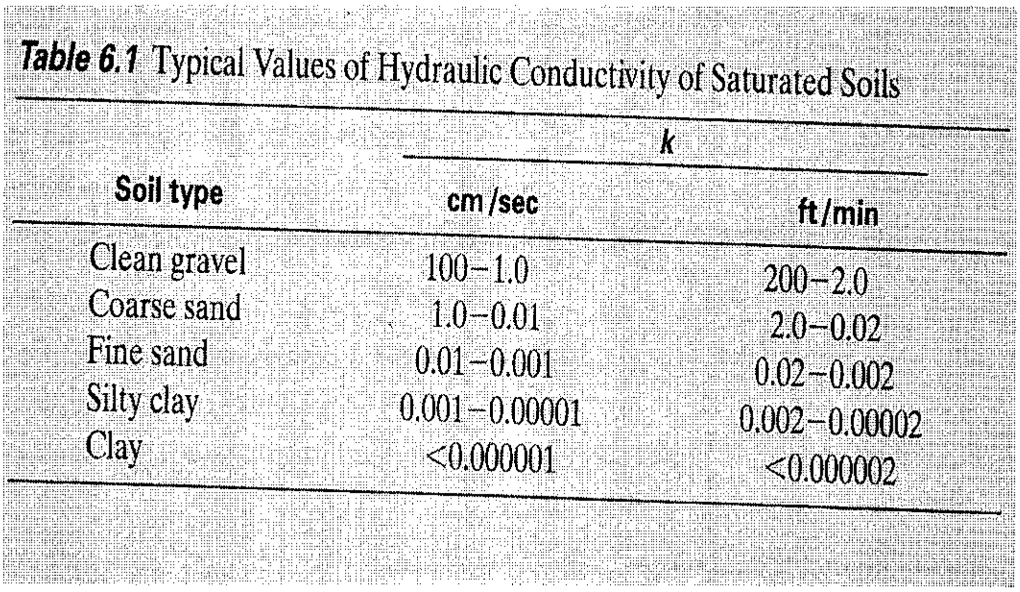


Table 6.1 Typical Values of Hydraulic Conductivity of Saturated Soils

	k	
Soil type	**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

Table 4.20: Typical Values of Hydraulic Conductivity of Saturated Soils

4.2.7 Theory: (Lab7)

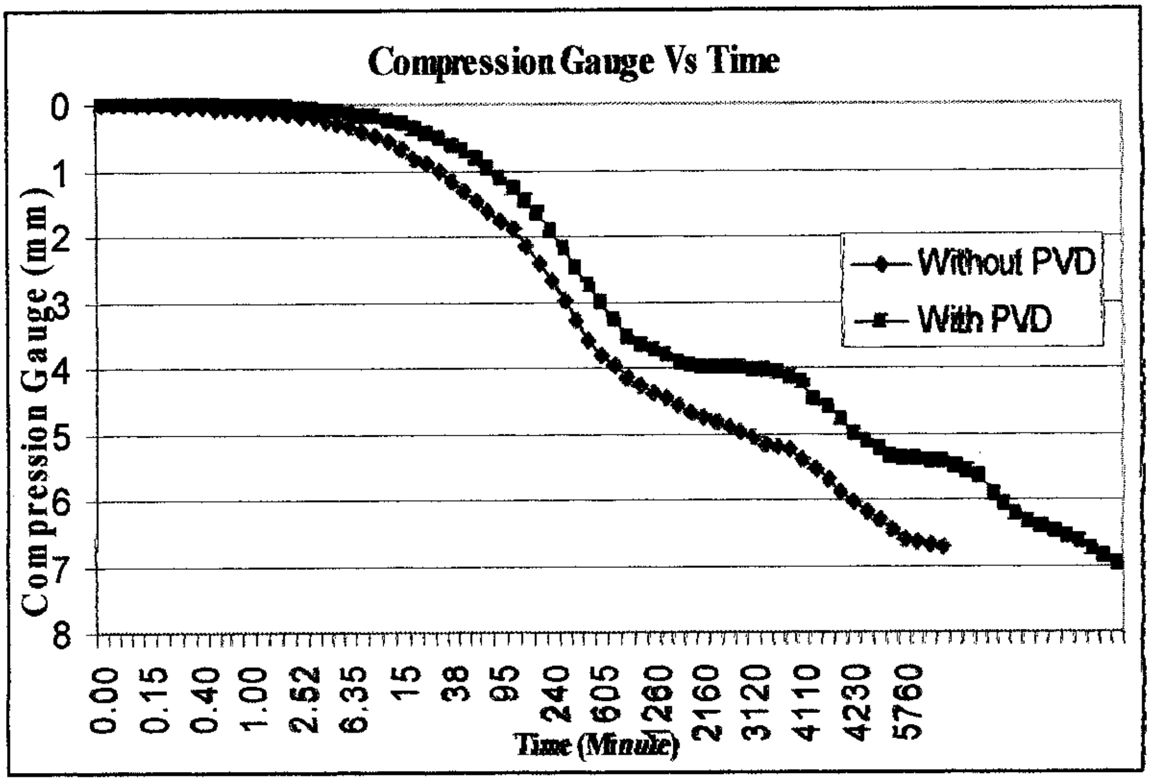


Figure 4.8: Comparison between the differences of PVD usage with surcharge (Set 1)

The graph show s the relationship between the compression gauge reading and time for the settlement that uses the PVD and the settlement without PVD implementation. Both experiment uses surcharge to enhance the reduction of moisture content in the settlement.

The experiment carried out without the PVD implementation shows the compression of settlement over a period of seven days. By the end of the sixth day, the settlement has compressed by 6.7mm and maintains at that level up to the seventh day. It is therefore concluded that the settlement has reached its maximum compression state under the given situation of using only the surcharge without PVD.

The experiment using the PVD with surcharge compiled the data collected over a period of 22 days. Theoretically, the curve of the graph should show a faster increase of compression gauge over time because the use of PVD should ease the transfer of moisture in the soil to the basin. However, in this experiment the results show otherwise where the use of PVD actually results in lesser compression rate. This can be attributed to excessive vertical soil pressure which can cause the drains to pinch off as they experience folding and buckling on the core. The Perspex case used to hold the soil in place is expanded horizontally when the surcharge load is put in place ***(Refer to Appendix)*** and this is evident there is substantial vertical soil pressure acting on the settlement. Although the compression rate is slower, the use of PVD achieves greater compression level whereby at the end of the 22nd day, the settlement shows a higher compression of 6.97mm and is able to be further compressed.

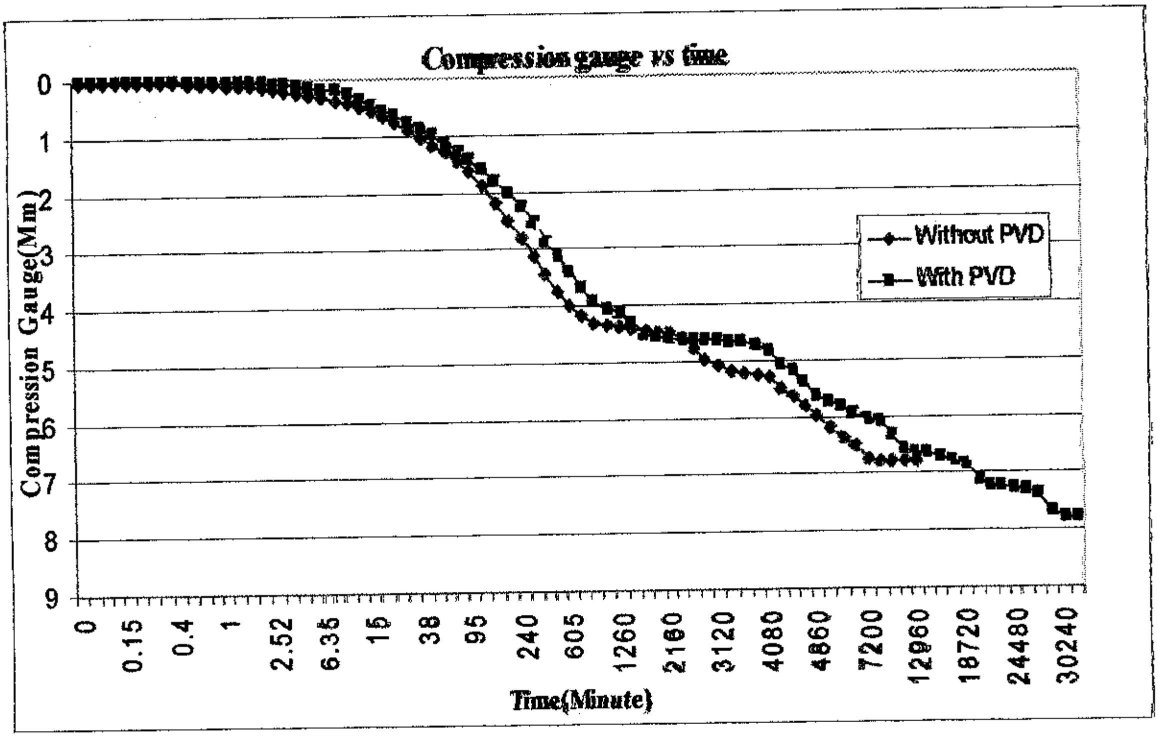


Figure 4.9: Comparison between the differences of PVD usage with surcharge (Set 2)

A second set test is carried out to verify the data collected from the first sample as the first sample depicts the use of PVD actually decrease the compression rate which is contradicting to the theory. The second set is carried out in a more precise and careful manner to ensure every procedure to conduct the test is adhered to so as to collect more accurate results. However, the findings is still basically the same as what attained by the first sample.

The experiment without using the PVD showed almost identical results whereby the settlement compresses by 6.78mm and maintains at that level by the end of the seventh day.

The experiment using the PVD still portray slower compression rate as compared to the one not using the PVD. This experiment however shows that a higher compression level is achieved at 7.79 mm by the end of 22 days. The difference shown here may be due to the human error during the setup of the experiment such as the preparation of the moisture content for the settlement and the time lag of from beginning the experiment and starting the counter at the computer.

Basically, set 1 and set 2 are identical in showing that the use of PVD would have slower compression rate but would achieve higher compression level through time.

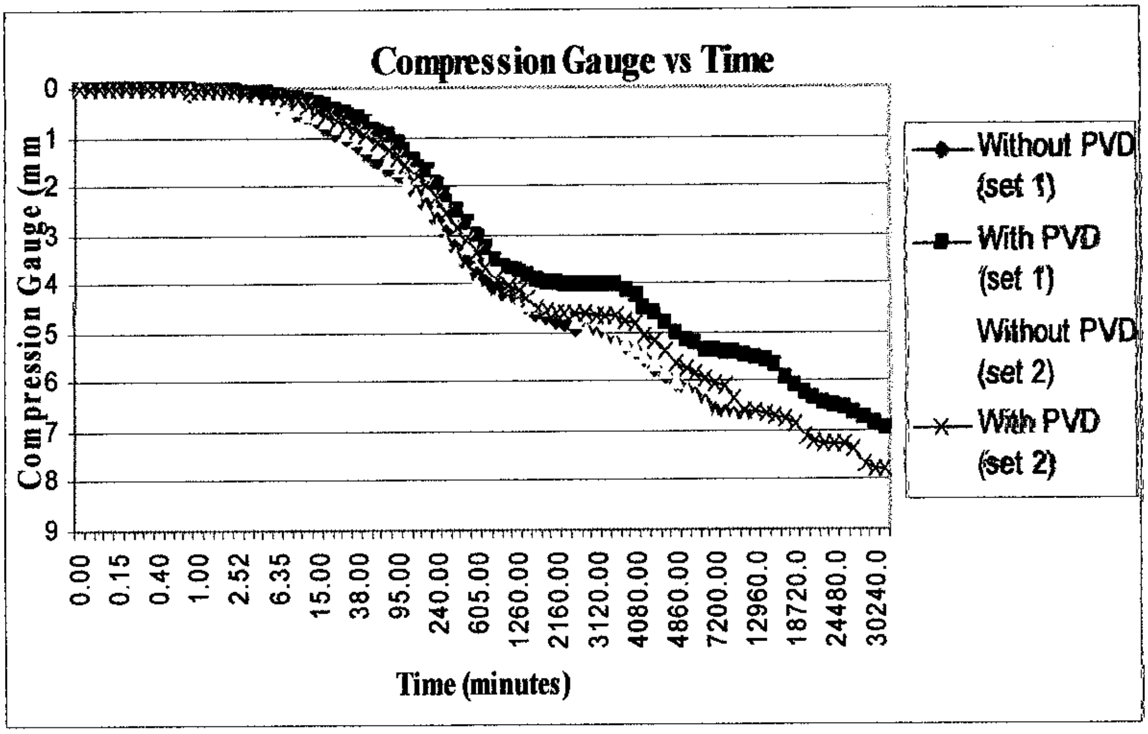


Figure 4.10: Graph showing the results of conducting the PVD experiment of set 1 and set 2

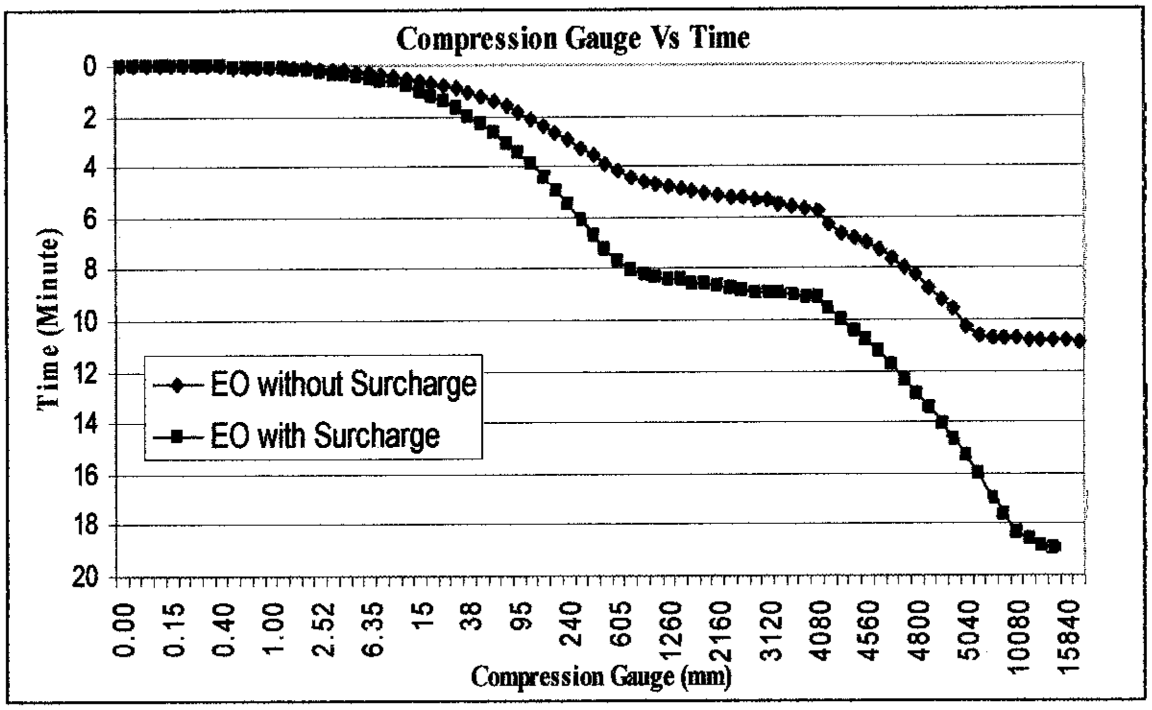


Figure 4.11: Graph showing the results based on electro-osmosis

The graph shows the relationship between compression and time for the electro-osmosis test for settlement with and without surcharge. Evidently, the settlement with surcharge would show a greater compression value in a shorter time as compared with the settlement without surcharge. The surcharge would induce higher pore water pressure which would then increase the ease of movement of the free ions in the settlement. The easier the ions move, the more water it would be displaced and thus high compression rate. Also, it is observed that for the given same period amount of time, the compression level achieved by the electro-osmosis method is higher at 19mm compared to the PVD method which recorded approximately 7 mm of compression.

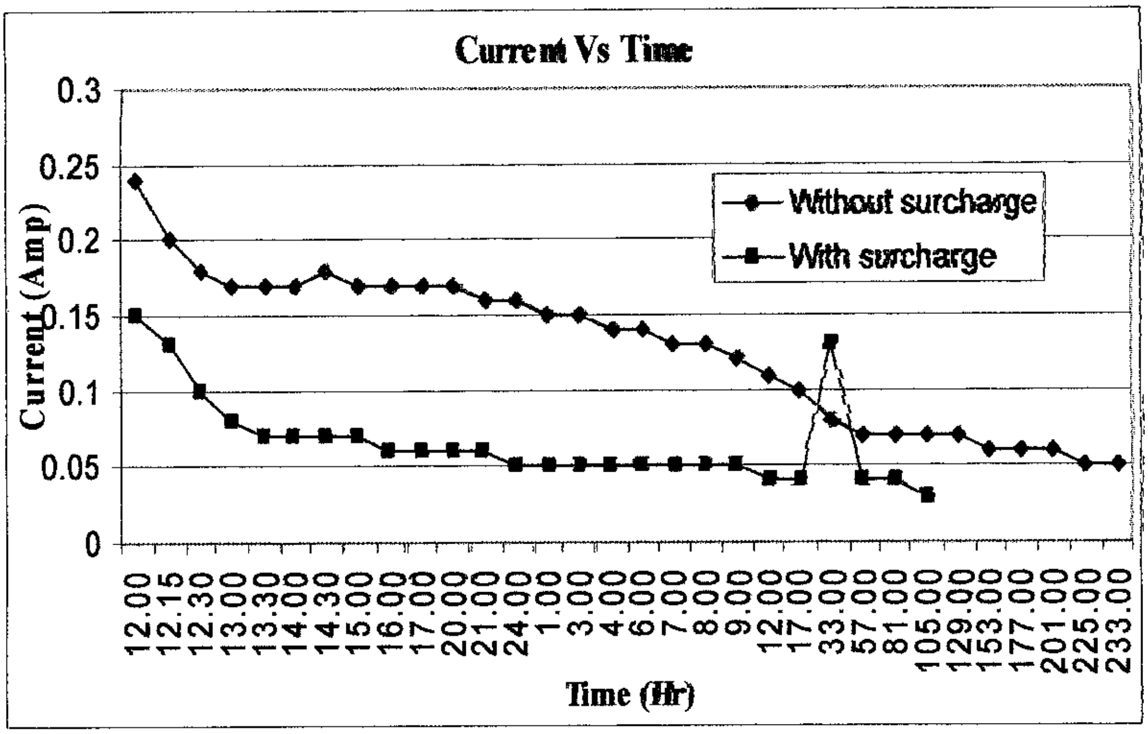


Figure 4.12: The graph shows the relationship between current and time for the electro-osmosis test for settlement with and without surcharge.

Both graphs depicts that the current drop over time. The current drop in the settlement with surcharge is more significant than the one without surcharge. This is due to the different concentration of ions present in both setups at the same particular time.

Initially, the settlement without surcharge records 0.24 A of current while the settlement with surcharge only record 0.15 A of current. When the surcharge is loaded, a substantial amount of water is displaced from the settlement to the basin almost immediately. Therefore, the time delays from setting the surcharge to starting the counter at the computer contributes to the drop of current reading from the settlement with surcharge.

Also, it can be noticed that current drop will stay stable for a longer period across time before it continues to drop further. This is due to the concentration of free ions and the accuracy of the current reader. As more water is displaced from the settlement, the amount of free ions decreased over time. Low concentration of ions would then slow down the effect of electro-osmosis and thus causing very small drop in current. The accuracy of the current reader of up to a hundredth Ampere only would result in showing the current rating as stable instead of small drops. The period between one "stable" current to the next "stable" current also increases due to the lesser and lesser amount of free flowing ions across time.

The outlier point for the settlement with surcharge curve is due to the water level at the basin touching the bottom tip of the electrodes. When this happen, a complete circuit would run through the water with less resistance as compared to the settlement with higher resistance and thus cause a sudden rise in current reading. The situation is remedied immediately by removing the basin away and the current reading is taken subsequently from then on.

All in all, the presence of surcharge would have a significant effect in current for the electro-osmosis method. Current drops in a faster rate and bigger magnitude with the inclusion of surcharge.

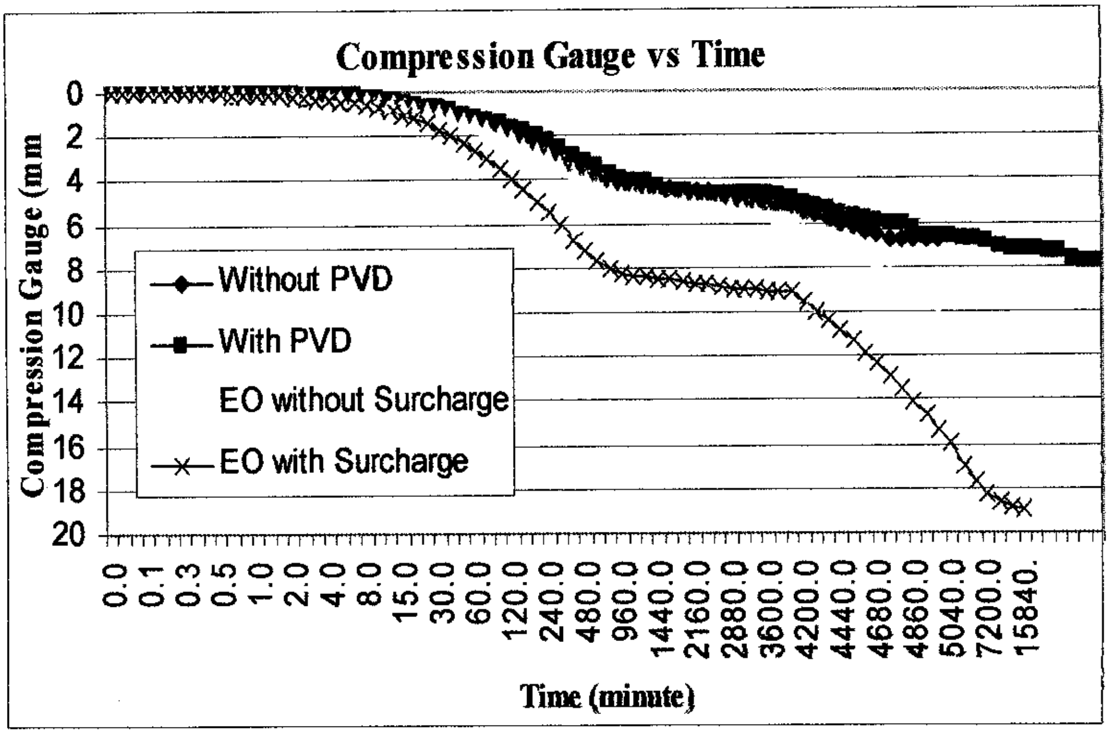


Figure 4.13: The graph shows the comparison between PVD and Electro-Osmosis

The experiment carried out without the PVD implementation shows the compression of settlement over a period of seven days. By the end of the sixth day, the settlement has compressed by 6.7mm and maintains at that level up to the seventh day. It is therefore concluded that the settlement has reached its maximum compression state under the given situation of using only the surcharge without PVD

The experiment using the PVD still portray slower compression rate as compared to the one not using the PVD. This experiment however shows that a higher compression level is achieved at 7.79 mm by the end of 22 days. The difference shown here may be due to the human error during the setup of the experiment such as the preparation of the moisture content for the settlement and the time lag of from beginning the experiment and starting the counter at the computer.

The graph also shows the relationship between compression and time for the electro-osmosis test for settlement with and without surcharge. Evidently, the settlement with surcharge would show a greater compression value in a shorter time as compared with the settlement without surcharge. The surcharge would induce higher pore water pressure which would then increase the ease of movement of the free ions in the settlement. The easier the ions move, the more water it would be displaced and thus high compression rate.

Also, it is observed that for the given same period amount of time, the compression level achieved by the electro-osmosis method is higher at 19 mm compared to the PVD method which recorded approximately 7.79 mm of compression

CHAPTER 5
CONCLUSION AND RECOMMENDATION

5.1 CONCLUSION

Based on the lab tests and the analysis done on the results, it is found that the electro-osmosis method is better than the PVD.

The compression rate of the electro-osmosis is higher and therefore able to compress the same height of settlement in a shorter time. This is mainly because the current acting on the small prototype is significant enough to charge up most of the water to become ions and thus the transfer of water to the basin is faster.

Besides that, electro-osmosis is able to compress up to 3 times more than what the PVD is able to do. The compression level of 19mm as achieved by the electro-osmosis is by far better than the 7mm as achieved by the PVD.

However, in this experiment it is necessary to note that the results of PVD not having any effect is due to that the surcharge imposed on the PVD setup is insignificant when translated to real life event. The soil used in the setup is good hydraulic conductivity soil where the permeability is high and eases the movement of ion transfer. However, the PVD is only effective on marine soil or high compressible soil and thus, the use of PVD in the experiment might not have significant effect on consolidation as PVD does not function at its optimum level on good hydraulic conductivity soil.

Due to the use of the good hydraulic conductivity soil used in the test setup, the electro-osmosis would benefit greatly from the soil since high hydraulic conductivity allows ions to transfer within the settlement easier. Hence this would result in the electro-osmosis showing a better consolidation result as compared to the PVD.

The compression of the settlement achieved would translate to low moisture content within the soil and hence high vane shear strength of the soil. Since the electro-osmosis is able to cause a higher compression as compared to the PVD, the soil utilizing the electro-osmosis will therefore has higher vane shear strength when compared to the soil using PVD.

5.2 RECOMMENDATION

Possible areas of improvement are identified in order to make the results of this project more accurate.

1. Firstly, the amount of Perspex case should be increased so that multiple tests can be carried out concurrently since the tests takes a long time from weeks to a month to complete.

2. In addition to that the type of soil used should also be varied to include problematic soil.

3. A better seal should also be used so that when fitted on the Perspex case, the whole area of the soil can be covered, leaving no extra soil to escape the cleavage between the seal and the Perspex case.

4. The wall thickness of the Perspex case should also be of greater thickness so that deformation of the case is kept at minimum.

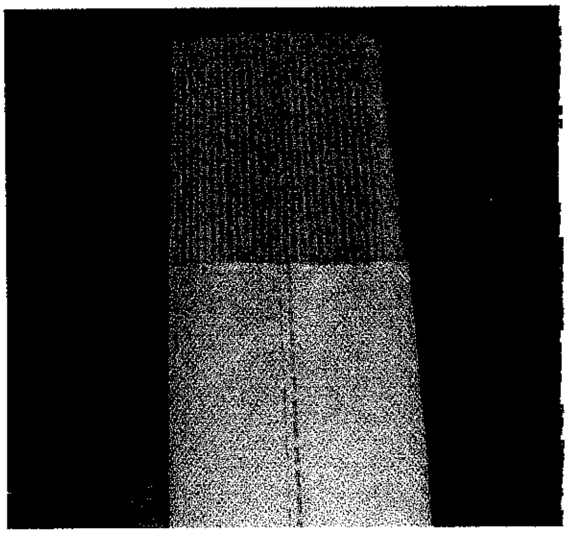
5. In order to obtain more accurate results, the data collection should be taken for a period of one month instead of a week so that more sample size may be collected to represent a more thorough study.

REFERENCES

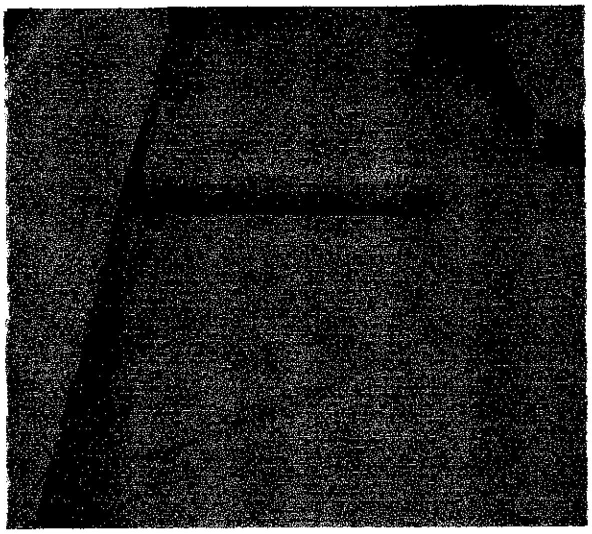
1. http://www.americandrainagesystems.com/index.htm
2. http://www.americanwick.com/
3. http://www.umeciv.maine.edu/cie366
4. E.A Wilkins & B.C. Chandler , Electro-Osmosis Trail-Section 6/3 Muar Flat
5. James K. Mitchell, M.ASCE , Soil Mechanics & Foundation Division, In Place Treatment of Foundation Soils
6. K.Y.Lo & K.S. Ho, Field Test of Electro-Osmosis Strengthening of Soft Sensitive
7. Khairul Anuar Kassim[1], Mohd. Raihan Taha[2] & Kamarudin Ahmad[1],Clay Electro-kinetic on a Tropical Residual Soil
8. Geotechnical Laboratory Lab Manual
9. Foundation and Earth Structures Lab Manual
10. Braja M. Das, Principles of Geotechnical Engineering, 5th Edition, Brooks/Cole
11. Robert W.Day, Geotechnical & Foundation Engineering
12. V.N.S Murthy, Principles & Practices of Soil Mechanics & Foundation Engineering.
13. Cheng Liu & Jack B.Evett, Soil Properties, 4th Edition, Prentice Hall
14. Flexi & Membra Drain company profile

APPENDIXES

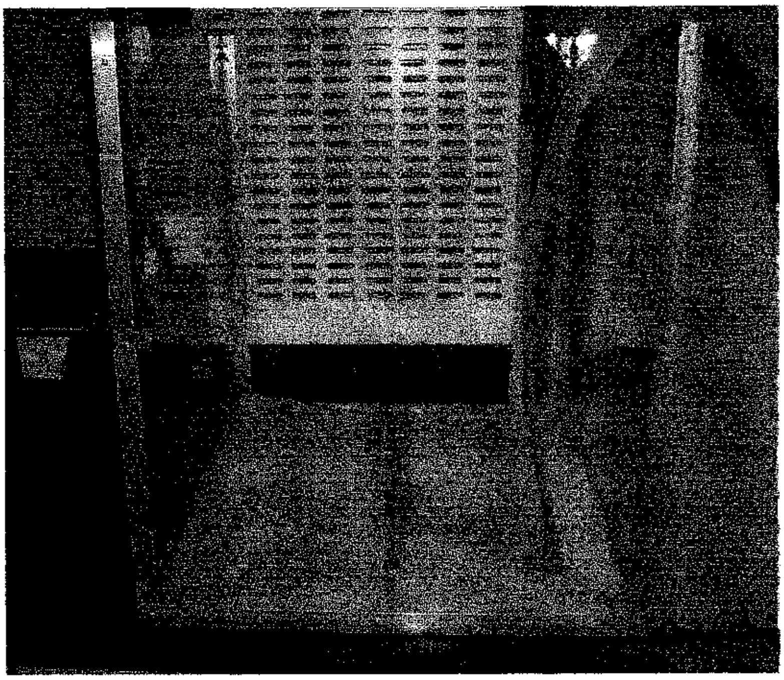
Pilot Stage



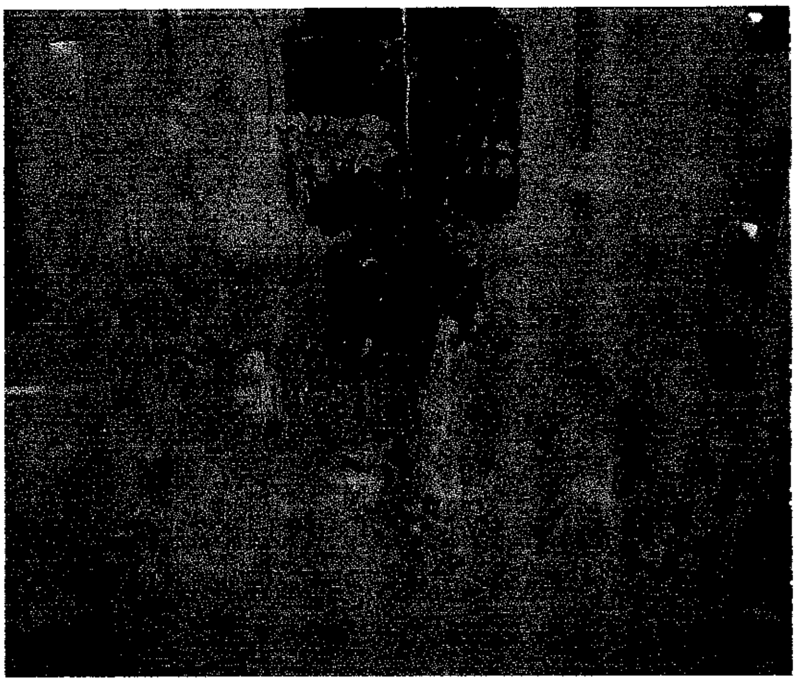
Prefabricated Vertical Drains

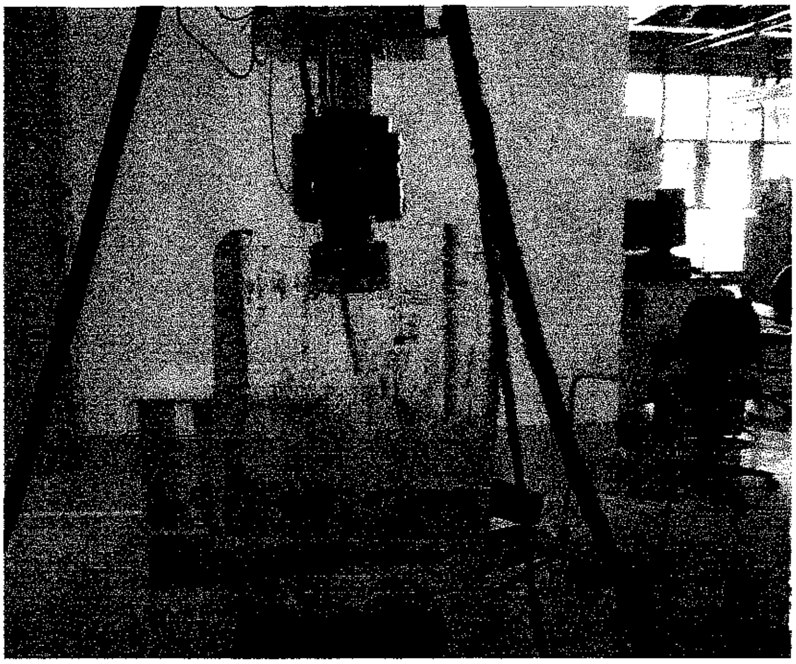


Geosynthesis Material

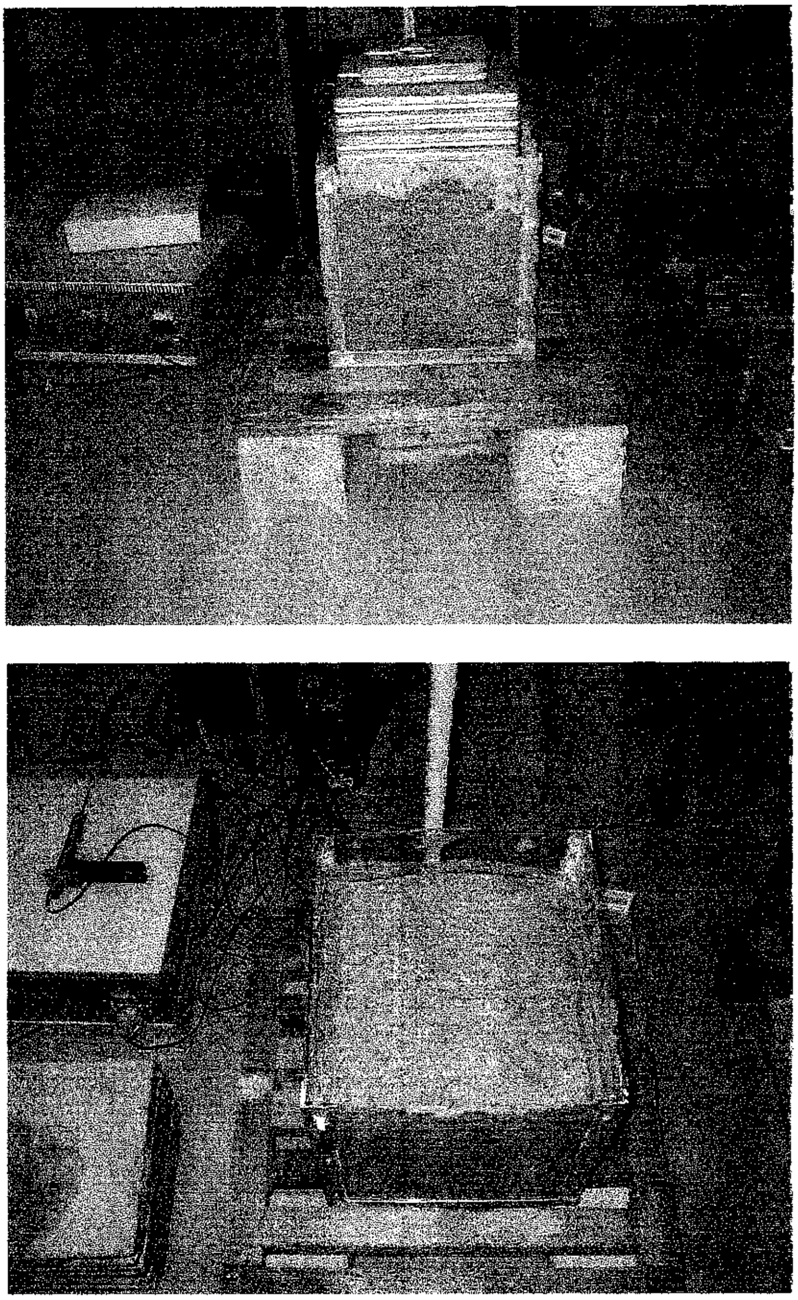


Perspex case, PVD and Geosynthesis Material

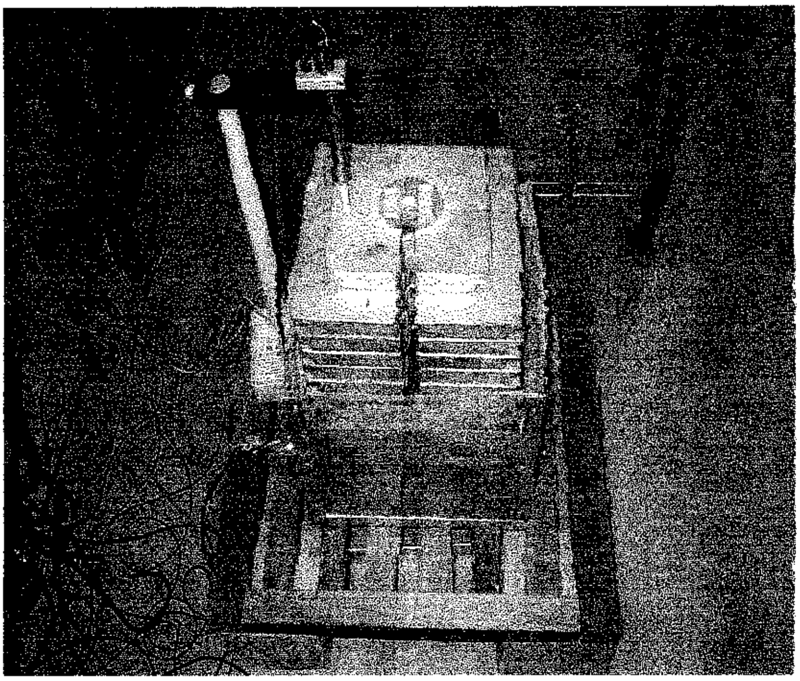


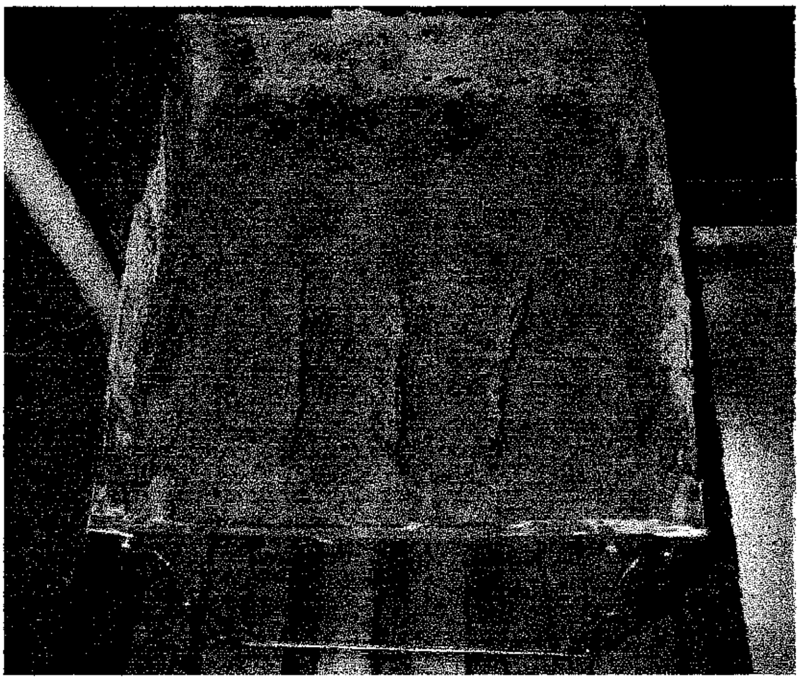


Using hydraulic pump to act as a load for consolidation process

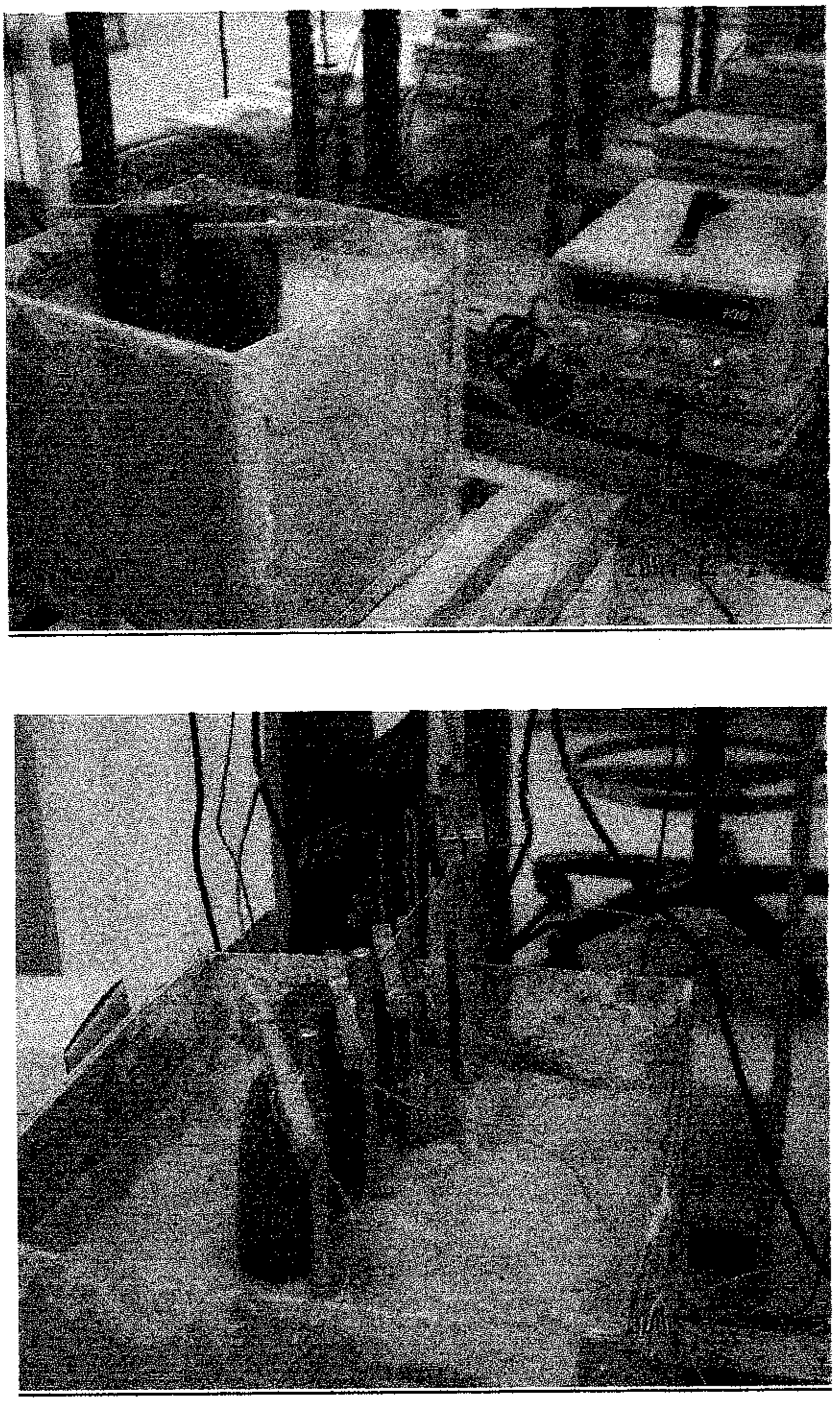


Experiment 1 & 3 : Without PVD with Surcharge

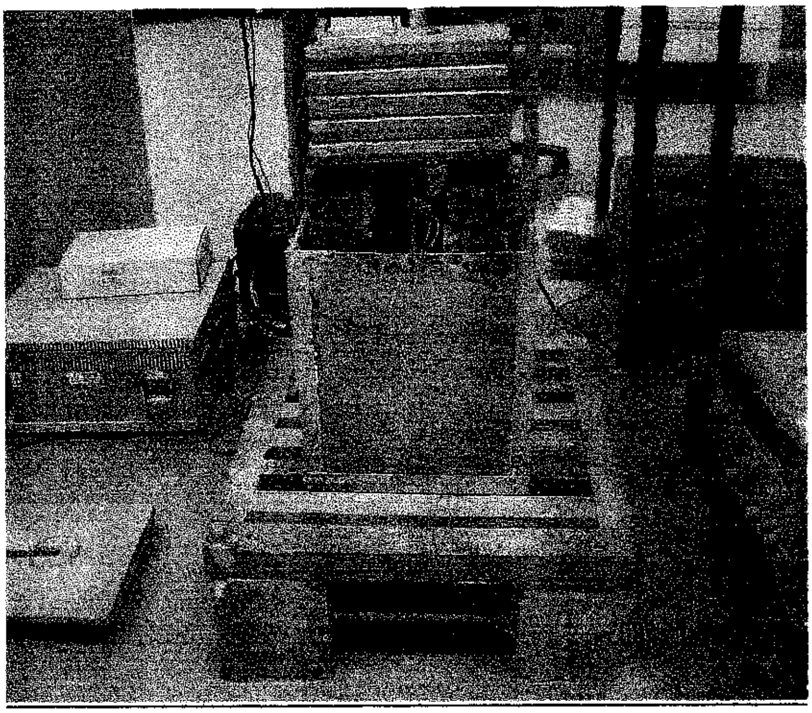


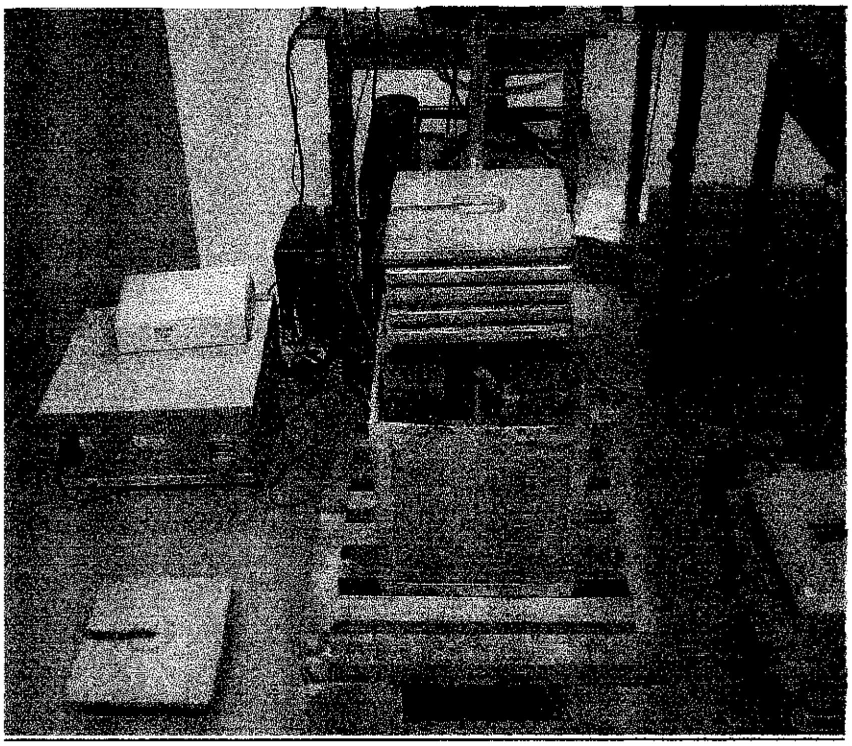


Experiment 2 & 4 : With PVD with Surcharge

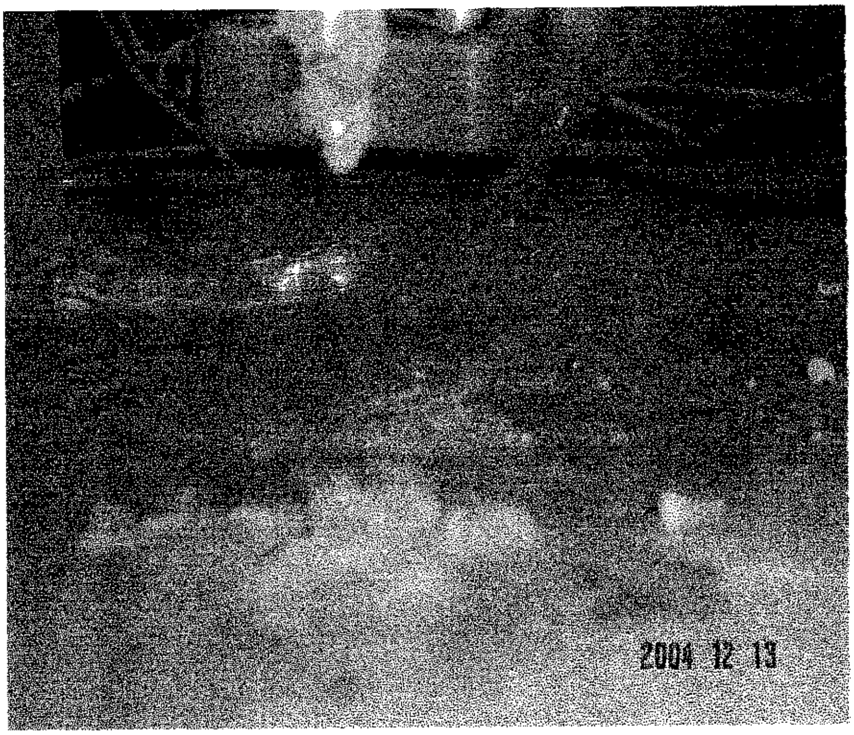


Experiment 5 : Electro-Osmosis without Surcharge

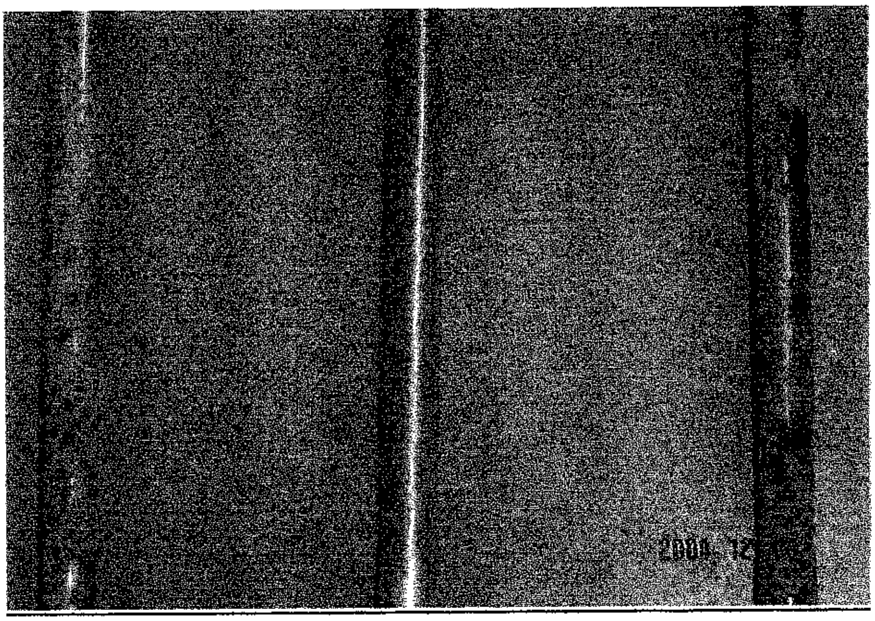




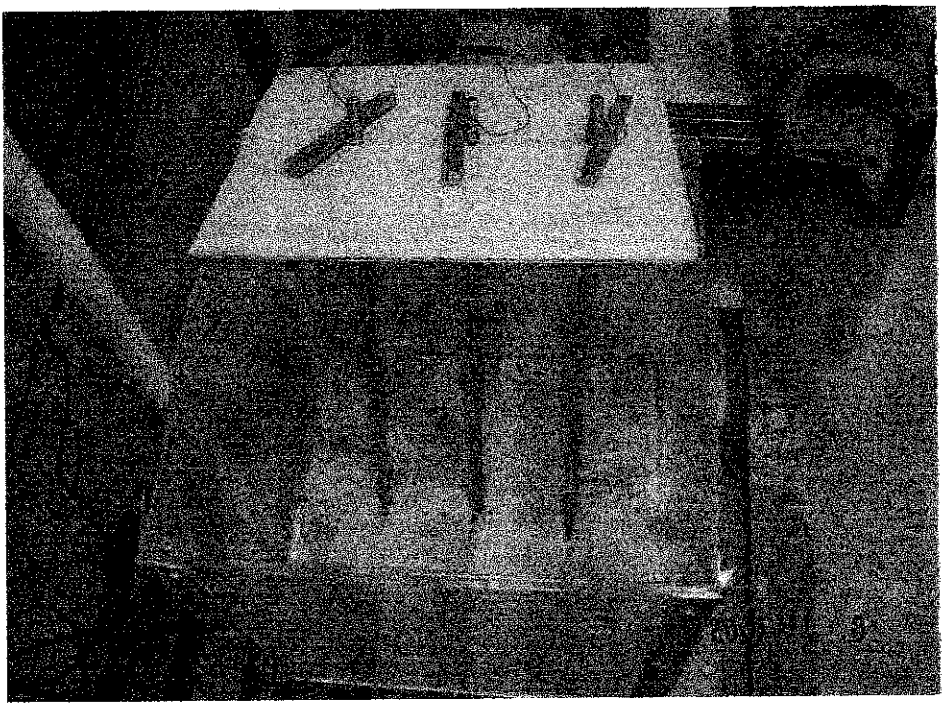
Experiment 6 : Electro-Osmosis with Surcharge



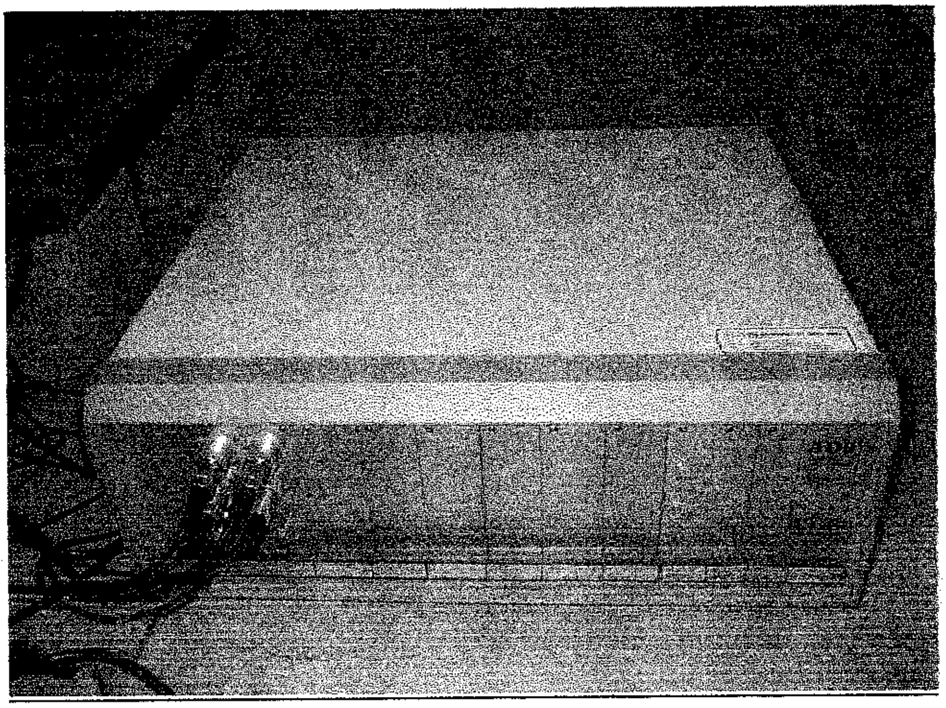
Water Passed Through Electrode Using Electro-Osmosis Method



Electrode after Experiment



Set Up for Electro-Osmosis Experiment



Calibration Equipment for recording data

RESULTS AND DISCUSSION

4.1 RESULTS

4.1.1 Determination of Moisture Content

	Sample A	**Sample B**
Mass of Container, m_1 (g)	38.03	37.45
Mass of Container + wet soil, m_2 (g)	171.61	172.30
Mass of Container + dry soil, m_3 (g)	139.77	140.00
Mass of moisture, $(m_2 - m_3)$ (g)	31.84	32.30
Mass of dry soil, $(m_3 - m_1)$ (g)	101.74	102.55
Moisture content, $w = [(m_2 - m_3)/(m_3 - m_1)]100$	31.30	31.50

:

The moisture content of the soil specimen, w, as a percentage of the dry soil mass to the nearest 0.1% can be calculated from the equation:

$$w = [(m_2 - m_3) / (m_3 - m_1)]100$$

w_A $= [(31.84) / (101.74)]100\%$

= 31.30%

w_B $= [(32.30) / (102.55)]100\%$

= 31.50%

Average moisture content $= (w_A + w_B) / 2$

$= (31.30\% + 31.50\%) / 2$

$= \mathbf{31.40\%}$

4.1.2 Specify Gravity

Sample 1 Mass (g)	
m_1 = mass of pycnometer + cap assembly	491.37
m_2 = mass of pycnometer + cap + soil	889.50
m_3 = mass of pycnometer + cap + soil + water	1774.26
m_4 = mass of pycnometer + cap + water	1532.54

Sample 2 Mass (g)	
m_1 = mass of pycnometer + cap assembly	491.02
m_2 = mass of pycnometer + cap + soil	891.50
m_3 = mass of pycnometer + cap + soil + water	1769.44
m_4 = mass of pycnometer + cap + water	1522.98

(Sample 1)

Formula to calculate the specific gravity:

$$\rho_s = \frac{m_2 - m_1}{(m_4 - m_1) - (m_3 - m_2)}$$

From the values obtained :

$$\rho_s = \frac{(889.50 - 491.37)}{(1532.54 - 491.37) - (1774.26 - 889.50)}$$

$= \mathbf{2.55}$

(Sample 2)

From the values obtained :

$$\rho_s = \frac{(891.50 - 491.02)}{(1522.98 - 491.02) - (1769.44 - 891.50)}$$

$$= \mathbf{2.60}$$

Average specify gravity = (2.55 + 2.60) / 2

~ **2.60**

4.1.3 Liquid Limit & Plastic Limit

LIQUID Limit	**1**		**2**		**3**		**Average**
Initial dial gauge reading (mm)	0	0	0	0	0	0	
Final dial gauge reading (mm)	81	80.5	106	105.5	130	130	105.5
Container no.	**A**	**B**	**C**	**D**	**E**	**F**	
Mass of wet soil + container (g), m_2	44.65	43.59	47.01	46.22	52.05	47.51	
Mass of dry soil + container (g), m_3	42.72	42.66	44.15	43.66	47.71	44.08	
Mass of container (g), m_1	37.81	38.18	37.80	38.16	38.43	36.69	
Mass of moisture (g)	1.93	0.93	2.86	2.56	4.34	3.43	
Mass of dry soil (g)	4.91	4.48	6.35	5.5	9.28	7.39	
Moisture content (%)	42.51	41.57	45.04	46.55	46.77	46.41	41.34
Average moisture content	42.04		45.8		46.59		

Moisture content of sample A = $[(m_2 - m_3) / (m_3 - m_1)] \times 100\%$
= [(44.65 – 42.72) / (42.72 - 37.81)] x 100%
= (1.93 / 4.54) x 100%
= 42.51 %

Moisture content of sample B = [(43.59 – 42.66) / (42.66 – 38.18)] x 100%
= (0.93 / 4.48) x 100%
= 20.76%

Moisture content of sample C = [(47.01 – 44.15) / (44.15 – 37.80)] x 100%
= (2.86 / 6.35) x 100%
= 45.04%

Moisture content of sample D = [(46.22 – 43.66) / (43.66 – 38.16)] x 100%
= (2.56 / 5.5) x 100%
= 46.55%

Moisture content of sample E = [(52.05 – 47.71) / (47.71 – 38.43)] x 100%
= (4.34 / 9.28) x 100%
= 46.77%

Moisture content of sample F = [(47.51 – 44.08) / (44.08 – 36.69)] x 100%
= (3.43 / 7.39) x 100%
= 46.41%

Average moisture content = (42.51 + 20.76 + 45.04 + 46.55 + 46.77 + 46.41) / 6
= 41.34%

According to Figure 1, the moisture content corresponding to a cone penetration of 20 mm = 40%

Thus the liquid limit, *(LL)* of the soil sample = **40%**

PLASTIC Limit **Test no.**	**1**	**2**	**3**	**4**	**Average**
Container no.	A	B	C	D	
Mass of wet soil + container (g), m_2	43.42	40.63	41.78	41.54	
Mass of dry soil + container (g), m_3	42.16	39.90	40.91	40.62	
Mass of container (g), m_1	37.66	37.54	38.12	37.30	
Mass of moisture (g)	1.26	0.73	0.87	0.92	
Mass of dry soil (g)	4.5	2.36	2.79	3.32	
Moisture content (%)	28	30.93	31.18	27.71	29.46

Moisture content of sample A = $[(m_2 - m_3) / (m_3 - m_1)] \times 100\%$

= [(43.42 – 42.16) / (42.16 – 37.66)] x 100%

= (1.26 / 4.5) x 100

= 28%

Moisture content of sample B = [(40.63 – 39.90) / (39.90 – 37.54)] x 100%

= (0.73 / 2.36) x 100%

= 30.93 %

Moisture content of sample C = [(41.78 – 40.91) / (40.91 – 38.12)] x 100%

= (0.87 / 2.79) x 100%

= 31.18 %

Moisture content of sample D = [(41.54 – 40.62) / (40.62 – 37.30)] x 100%

= (0.92 / 3.32) x 100%

= 27.71 %

Average moisture content = (28 + 30.93 + 31.18 + 27.71) / 4

= 29.46 %

= 29 %

Thus the plastic limit, (PL) = 29 %

Plasticity index, PI = LL – PL

= (40 – 29) %

= 11%

4.1.4 Sedimentation by the hydrometer method

1. Hydrometer reading, Rh

$$Rh = Rh' + Cm$$

Where

Cm is the meniscus correction

Rh' is the observed hydrometer reading

When T = 25°C,

Cm = 0.0005m = 0.5mm

t = 0.5min

Rh = 1.0122 + 0.0005

= 1.0127mm

t = 1 min

Rh = 1.0122 + 0.0005

= 1.0127mm

t = 2 min

Rh = 1.0121 + 0.0005

= 1.0126mm

t = 4 min

Rh = 1.0120 + 0.0005

= 1.0125mm

t = 8 min

Rh = 1.0120 + 0.0005

= 1.0125mm

t = 30 min

Rh = 1.0115 + 0.0005

= 1.012mm

t = 120min

Rh = 1.0105 + 0.0005

= 1.011mm

t = 1440min

Rh = 1.0105 + 0.0005

= 1.011mm

2. Effective depth, H_R

$$H_R = H + ½ [h – VhL/900]$$

Where

H is the length from the neck of the bulb to graduation Rh (in mm)

h is the length of the bulb

Vh is the volume of the hydrometer bulb (mL)

L is the distance between the 100mL and 1000mL scale markings of the sedimentation cylinder (in mm)

h = 152mm

Vh = 69g = 69mL

L = 317mm

N = 33mm

Hydrometer relative density markings (mm)	
0.995(d7)	139
1000(d6)	115
1005(d5)	96
1010(d4)	76
1015(d3)	57
1020(d2)	38
1025(d1)	18
1030	0

t = 0.5 min

Rh = 1.0127mm

H = 33 + 85.2

= 118.2mm

H_R = 118.2 + ½ [152 – (69 x 317) / 900]

= 182.05mm

t = 1 min

Rh = 1.0127mm

H = 33 + 85.2

= 118.2mm

H_R = 118.2 + ½ [152 – (69 x 317) / 900]

= 182.05mm

t = 2 min

Rh = 1.0126mm

H = 33 + 85.6

= 118.6mm

H_R = 118.6 + ½ [152 – (69 x 317) / 900]

= 182.45mm

t = 4 min

Rh = 1.0125mm

H = 33 + 86

= 119mm

H_R = 119 + ½ [152 – (69 x 317) / 900]

= 182.85mm

t = 8 min

Rh = 1.0125mm

H = 33 + 86

= 119mm

H_R = 119 + ½ [152 – (69 x 317) / 900]

= 182.85mm

$t = 30$ min

$R_h = 1.012$mm

$H = 33 + 88$

$= 121$mm

$H_R = 121 + ½ [152 – (69 \times 317) / 900]$

$= 184.85$mm

$t = 120$min

$R_h = 1.011$mm

$H = 33 + 92$

$= 125$mm

$H_R = 125 + ½ [152 – (69 \times 317) / 900]$

$= 188.85$mm

$t = 1440$min

$R_h = 1.011$mm

$H = 33 + 92$

$= 125$mm

$H_R = 125 + ½ [152 – (69 \times 317) / 900]$

$= 188.85$mm

3. Equivalent particle diameter, D

$$D = 0.005531 \sqrt{\eta H_R / (\rho s -1) t}$$

Where

η is the dynamic viscosity of water at the test temperature (in mPa·s) as shown in Table 2

H_R is the effective depth at which the density of the suspension is measured (in mm)

ρs is the particle density (in Mg / m^3)

t is the elapsed time (in min)

Temperature, T (^oC)	Viscosity of water, η (mPa·s)
10	1.304
15	1.137
20	1.002
25	0.891
30	0.798

Table 2 : Viscosity of water

$\rho s = 2.68\ Mg / m^3$

$\eta = 0.891$

t = 0.5 min

H_R = 182.05mm

D = 0.005531 √ 0.891 x 182.05 / (2.68 -1) (0.5)

= 0.077mm

t = 1 min

H_R = 182.05mm

D = 0.005531 √ 0.891 x 182.05 / (2.68 -1) (1)

= 0.054mm

t = 2 min

H_R = 182.45mm

D = 0.005531 √ 0.891 x 182.45 / (2.68 -1) (2)

= 0.039mm

t = 4 min

H_R = 182.85mm

D = 0.005531 √ 0.891 x 182.85 / (2.68 -1) (4)

= 0.027mm

$t = 8$ min

$H_R = 182.85$mm

$D = 0.005531 \sqrt{0.891 \times 182.85 / (2.68 - 1)(8)}$

$= 0.019$mm

$t = 30$ min

$H_R = 184.85$mm

$D = 0.005531 \sqrt{0.891 \times 184.85 / (2.68 - 1)(30)}$

$= 0.010$mm

$t = 120$min

$H_R = 188.85$mm

$D = 0.005531 \sqrt{0.891 \times 188.85 / (2.68 - 1)(120)}$

$= 0.005$mm

$t = 1440$ min

$t = 120$min

$H_R = 188.85$mm

$D = 0.005531 \sqrt{0.891 \times 188.85 / (2.68 - 1)(1440)}$

$= 0.002$mm

4. Modified hydrometer reading, Rd

$$Rd = Rh' - R_o'$$

Where

R_o' is the hydrometer reading at the upper rim of the meniscus in the dispersion solution

$t = 0.5$ min

$Rd = 1.0127 - 1.001$

$= 0.0117$mm

t = 1 min

Rd = 1.0127 – 1.001

= 0.0117mm

t = 2 min

Rd = 1.0126 – 1.001

= 0.0116mm

t = 4 min

Rd = 1.0125 – 1.001

= 0.0115mm

t = 8 min

Rd = 1.0125 – 1.001

= 0.0115mm

t = 30 min

Rd = 1.012 – 1.001

= 0.011mm

t = 120 min

Rd = 1.011– 1.001

= 0.01mm

t = 1440 min

Rd = 1.011– 1.001

= 0.01mm

5. Percentage by mass, K

$$K = [\ 100\rho s / m\ (\rho s - 1)\ ] \times Rd$$

Where

ρs is the particle density measured / assumed

m is the mass of the dry soil used (in g)

$\rho s = 2.68\ Mg / m^3$

m = 50g

t = 0.5 min

$K = [\ 100\ (2.68) / 50\ (2.68 - 1)\ ] \times 0.0117$

$= 3.7323\%$

t = 1.0 min

$K = [\ 100\ (2.68) / 50\ (2.68 - 1)\ ] \times 0.0117$

$= 3.7323\ \%$

t = 2.0 min

$K = [\ 100\ (2.68) / 50\ (2.68 - 1)\ ] \times 0.0116$

$= 3.7004\ \%$

t = 4.0 min

$K = [\ 100\ (2.68) / 50\ (2.68 - 1)\ ] \times 0.0115$

$= 3.6685\ \%$

t = 8.0 min

$K = [\ 100\ (2.68) / 50\ (2.68 - 1)\ ] \times 0.0115$

$= 3.6685\ \%$

t = 30.0 min

K = [100 (2.68) / 50 (2.68 – 1)] x 0.011

= 3.509 %

t = 120.0 min

K = [100 (2.68) / 50 (2.68 – 1)] x 0.01

= 3.19 %

t = 1440.0 min

K = [100 (2.68) / 50 (2.68 – 1)] x 0.01

= 3.19 %

Elapsed Time (min)	Rh' (mm)	Rh' + Cm = Rh (mm)	Effective Depth H, mm	Particle diameter, D (mm)	Rh' – Ro' = Rd (mm)	Percentage finer than D (%)
0.5	1.0122	1.0127	182.05	0.077	0.0112	3.7323
1	1.0122	1.0127	182.05	0.054	0.0112	3.7323
2	1.0121	1.0126	182.45	0.039	0.0111	3.7004
4	1.0120	1.0125	182.85	0.027	0.011	3.6685
8	1.0120	1.0125	182.85	0.019	0.011	3.6685
30	1.0115	1.012	184.85	0.010	0.0105	3.509
120	1.0105	1.011	188.85	0.005	0.01	3.19
1440	1.0105	1.011	188.85	0.002	0.001	3.19

Particle diameter, D (mm)	Percentage finer than D (%)	Cumulative percentage (%)	Percentage Passing (%)
0.077	3.7323	3.73	96.27
0.054	3.7323	7.46	92.54
0.039	3.7004	11.16	88.84
0.027	3.6685	14.83	85.17
0.019	3.6685	18.50	81.50
0.010	3.509	22.01	77.99
0.005	3.19	25.20	74.80
0.002	3.19	28.39	71.61

Table 4.0: Table of percentage finer than D versus particle diameter

Data for Experiment 1-6

Experiment 1		Experiment 2		Experiment 5		Experiment 6	
Time	Comression Gauge (mm)	Time	Comression Gauge (mm)	Time	Comression Gauge (mm)	Time	Comression Gauge (mm)
0	0.02	0	0	0	0.02	0	0.00
0.08	0.03	0.08	0.01	0.08	0.03	0.08	0.00
0.1	0.03	0.1	0.01	0.1	0.03	0.1	0.00
0.13	0.03	0.13	0.01	0.13	0.03	0.13	0.01
0.15	0.03	0.15	0.01	0.15	0.03	0.15	0.01
0.2	0.04	0.2	0.01	0.2	0.03	0.2	0.01
0.25	0.04	0.25	0.01	0.25	0.03	0.25	0.02
0.32	0.04	0.32	0.01	0.32	0.04	0.32	0.03
0.4	0.05	0.4	0.01	0.4	0.04	0.4	0.03
0.5	0.06	0.5	0.02	0.5	0.05	0.5	0.05
0.63	0.06	0.63	0.02	0.63	0.05	0.63	0.06
0.8	0.08	0.8	0.03	0.8	0.06	0.8	0.08
1	0.09	1	0.03	1	0.07	1	0.10
1.27	0.10	1.27	0.04	1.27	0.09	1.27	0.13
1.58	0.12	1.58	0.04	1.58	0.10	1.58	0.17
2	0.15	2	0.05	2	0.13	2	0.21
2.52	0.18	2.52	0.07	2.52	0.15	2.52	0.26
3.17	0.21	3.17	0.08	3.17	0.17	3.17	0.32
4	0.24	4	0.1	4	0.21	4	0.38
5.03	0.29	5.03	0.13	5.03	0.25	5.03	0.45
6.35	0.33	6.35	0.16	6.35	0.30	6.35	0.52
8	0.39	8	0.19	8	0.37	8	0.61
9	0.42	9	0.26	9	0.41	9	0.65
12	0.50	12	0.35	12	0.49	12	0.81
15	0.56	15	0.44	15	0.58	15	1.03
19	0.66	19	0.54	19	0.68	19	1.21
24	0.77	24	0.64	24	0.79	24	1.43
30	0.90	30	0.75	30	0.92	30	1.71
38	1.03	38	0.87	38	1.06	38	2.00
48	1.17	48	0.99	48	1.23	48	2.32
60	1.30	60	1.14	60	1.40	60	2.69
76	1.47	76	1.29	76	1.61	76	3.06
95	1.64	95	1.44	95	1.87	95	3.46

120	1.89	120	1.61	120	2.12	120	3.91
151	2.20	151	1.8	151	2.38	151	4.44
190	2.50	190	2.01	190	2.64	190	4.95
240	2.81	240	2.26	240	2.93	240	5.49
302	3.12	302	2.55	302	3.26	302	6.07
381	3.45	381	2.87	381	3.57	381	6.70
480	3.76	480	3.12	480	3.89	480	7.22
605	4.00	605	3.4	605	4.16	605	7.68
762	4.18	762	3.68	762	4.39	762	8.03
960	4.30	960	3.94	960	4.61	960	8.25
1080	4.33	1080	4.06	1080	4.71	1080	8.32
1260	4.38	1260	4.12	1260	4.80	1260	8.40
1440	4.41	1440	4.32	1440	4.85	1440	8.44
1680	4.45	1680	4.52	1680	4.97	1680	8.55
1920	4.48	1920	4.56	1920	5.05	1920	8.62
2160	4.50	2160	4.6	2160	5.12	2160	8.69
2400	4.57	2400	4.61	2400	5.18	2400	8.75
2640	4.76	2640	4.61	2640	5.23	2640	8.83
2880	4.96	2880	4.61	2880	5.28	2880	8.90
3120	5.09	3120	4.63	3120	5.35	3120	8.92
3360	5.16	3360	4.64	3360	5.45	3360	8.97
3600	5.21	3600	4.65	3600	5.58	3600	9.04
3840	5.24	3840	4.74	3840	5.70	3840	9.09
4080	5.30	4080	4.83	4080	5.79	4080	9.11
4110	5.48	4320	5.08	4320	6.30	4200	9.52
4140	5.63	4560	5.18	4440	6.63	4320	9.96
4170	5.80	4800	5.39	4500	6.79	4380	10.40
4200	5.99	4860	5.62	4560	6.95	4440	10.84
4230	6.18	4920	5.74	4620	7.28	4560	11.25
4260	6.35	5040	5.82	4680	7.60	4620	11.81
4290	6.51	5760	5.94	4740	7.93	4680	12.37
4320	6.75	7200	6.04	4800	8.25	4740	12.93
5760	6.76	8640	6.08	4830	8.74	4800	13.49
7200	6.77	10080	6.33	4860	9.23	4860	14.04
8640	6.78	11520	6.58	4920	9.57	4890	14.68
10080	6.78	12960	6.62	5040	10.25	4920	15.31
		14400	6.65	5760	10.61	5040	15.98
		15840	6.7	7200	10.68	5760	16.95
		17280	6.77	8640	10.71	6480	17.62
		18720	6.86	10080	10.74	7200	18.28

		20160	7.13	11520	10.76	8640	18.56
		21600	7.22	12960	10.78	10080	18.85
		23040	7.24	14400	10.81	15840	18.98
		24480	7.26	15840	10.83		
		25920	7.28	17280	10.86		
		27360	7.36				
		28800	7.67				
		30240	7.77				
		30420	7.79				

Based on Figure 4.13 :Comparison between PVD and Electro-Osmosis