ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY (AAiT)

SCHOOL OF CIVIL AND ENVIROMETAL ENGINEERING

Geotechnical Engineering Stream

POST-GRADUATE STUDIES

Investigation Some of the Engineering Properties of Soil in Merawi Town.

By

Eyasu Minichle

February 2015

Investigation Some of the Engineering Properties of Soil in Merawi Town.

A thesis submitted to the school of graduate studies of Addis Ababa University, Addis Ababa Institute of Technology (AAiT), School of Civil and Environmental Engineering, Post-Graduate studies in partial fulfillment of the requirements for the Masters of Science in Geotechnical Engineering Stream of Civil and Environmental Engineering School.

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Advisor: - Dr.Ing. Samuel Tadesse

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DECLARATION

I, the undersigned, declare that this thesis is my original work performed under the supervision of my research advisor Dr.-Ing Samuel Tadesse and has not been presented as a thesis for a degree in any other university. All sources of materials used for this thesis have also been duly acknowledged.

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Symbols and Abbreviations

E.C	……………	Ethiopian Calendar	w	………	Water/Moisture Content
No.	……………	Number	GI	………	Group Index
Sr.No	……………	Serial Number	UCS	………	Unconfined Compressive Strength
Fig	……………	Figure	q_u	………	Unconfined Compressive Strength
TP/PT	……………	Test Pit/Pit	C	………	Cohesive Strength
LL	……………	Liquid Limit	P	………	Pressure
PL	……………	Plastic Limit	P_c	………	Pre-Consolidation Pressure
PI	……………	Plasticity Index	σ	………	stress
C_c	……………	Compression Index	N	………	No. of Blows
C_e	……………	Expansion Index	F_{200}	………	Percentage passing of No.200
C_r	……………	Re-Compression Index	η	………	Viscosity Constant
e	……………	Strain/ Void Ratio	K	………	Correction Coefficient
e_o	……………	Initial Void Ratio	$\sqrt{}$	………	Square Root
G_s	……………	Specific Gravity	Δ	………	Change
w_s	……………	Weight of Solid	L	………	Length
V_s	……………	Volume of solid	L_o	………	Initial Length
V_w	……………	Volume of water	B	………	Width
m	……………	Mass	D	………	diameter
m_w	……………	Mass of Water	h	………	Height
m_c	……………	Mass of Container	Z	………	Depth
m_{cs}	……………	mass of Container and Dry Soil	g	………	gram
A_o	……………	Initial Area	mm	………	Millimeter
A`	……………	Corrected Area	m	………	Meter
d	……………	Diameter	kN	………	kilo Newton
γ	……………	Unit Weight	kg	………	kilo gram
π	……………	Pi (3.14)	kpa	………	kilo Pascal
hr	……………	Hour	^oc	………	Degree Centigrade
sec	……………	Second	H_{dr}	………	Drainage Height

min	……………	Minute
V	……………	Volume
T	……………	Time
R_c	……………	Corrected Reading
SRT	……………	Square Root of Time
CH	……………	Inorganic Clay of High Plasticity
CL	……………	Inorganic Clay of low to medium Plasticity
MH	……………	Inorganic silt of High Plasticity
ML	……………	Inorganic Silt of low Plasticity
OH	……………	Organic Clay of High Plasticity
C_v	………	Coefficient of consolidation
Log	………	Logarithm
S_S	………	Consolidation Settlement
%	………	Percent
IS	………	Indian Standard
EBCS	………	Ethiopian Building Code Standard
AASHTO	…	American Associations of state High Way and Transportation office
ASTMD	……	American Society for Testing Material Designation

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ABSTRACT

Merawi, Mecha wereda capital Town, which is located 540 km from Addis Ababa in North West direction on the main high way road of Addis Ababa to Bahir Dar and 30 km away from Bahir Dar in the South West direction. It was founded in 1935E.C which situated at latitude $11^{o}24`56.36``N$, longitude $37^{o}09`33.03``E$ and an average elevation of 2010m above sea level.

A lot of civil engineering structures are under construction; however, so far nothing has been done on the investigation of soil with respect to the intended town development plan. There are no previously done researches around Merawi.

The objective of the research is to investigate some of the engineering properties of the soils in Merawi Town, where development has a promising future. Investigations have been made on the properties of the soils in the town for this research. The research is intended to investigate some of the engineering properties of soils in the Town. After visiting the Town, six representative test pit points were selected and from each test pit disturbed and undisturbed samples were taken to perform laboratory tests. As it was observed, the depth of the soil coverage is uniform in each test pit. The number of test pits taken were limited because of that the soil in the entire area of the town is more or less similar.

From reconnaissance survey it has been observed that the Town is covered by red clay soil. After collecting representative disturbed and undisturbed samples from proposed open test pits by direct manual excavation, laboratory tests were carried out including specific gravity, natural moisture content, Atterberg limit, unconfined compression, consolidation, free swell, sieve analysis and hydrometer tests.

The laboratory test results show that the soil in the Town has clay fraction ranges 63.6% to 91.3% which is fine clay soils. Based on index property test results, the soil under investigation was classified as clay soil using Unified and AASHTO soil classification systems.

The specific gravity of the soil ranges from 2.70 to 2.76. From the Atterberg Limit results the soil is highly plastic clay and the activity grouped as inactive. The soil has free swell between 14.5% and 20% which indicates that the soil is not expansive soil.

The unconfined compressive strength of the soil ranges between 63.7kN/m^2 and 117.8kN/m^2 and the consistency index of the soil ranges between 50% to 93.7% which indicates that the soil natural consistency is soft to stiff clay soil.

Chapter One

1. Introduction

1.1 General

Soil properties are essential for economic construction purposes. It is required to determine properly the engineering properties of soils. So, it is important to study soil properties in Ethiopian. But so far some part of the Ethiopian soil is not studied and Merawi Town is among these places.

The Town has a potential for expansion in all directions. It has no sewerage system. There is no systematic soil investigation has been carried out prior to this work.

A thorough and comprehensive geotechnical investigation is an essential requirement to the design and construction of civil engineering projects. The proper design of civil engineering structures like foundation of buildings, retaining walls, high ways, etc. requires adequate knowledge of sub surface conditions at the sites of the structures. Many damages to buildings, roads and other structures founded on soils are mainly due to the lack of proper investigation of substructure condition.

The Town of Merawi, having adequate land area for expansion and being an important commercial, educational and Project area (like koga irrigation) center in the region will have a high potential for future development. A lot of civil engineering structures are under construction. However, nothing has been done on the investigation of soil with respect to the intended Town development plan since there is no soil laboratory in the town to study the soil properties and there are no previously done researches around Merawi.

Thus this thesis gives better understanding about some engineering behavior of the soil in the Town. Identifying the soil characteristic is essential to construct economically different types of civil engineering that will serve to the people for various purposes.

The results of the study will be of great importance for the ever growing building construction especially for those yet to be constructed in that area. It can be used as soil property manual as it will have a customized nature to meet the required soil information of the area with regard to the future development programs in the construction sector.

1.2 Objective of the thesis

1.1.1 General objective of the study

- Investigating some of the engineering properties of soil in Merawi Town

1.1.2 Specific objective

The specific objectives of the thesis are

1. To determine index properties
2. To investigate strength and compressibility characteristics of the soil
3. To prepare soil map for the Town

1.3 Research methodology

1.3.1 Site reconnaissance

The site of the area is visited for taking sample and places that are founded to be representative to the Town are selected to take the sample.

1.3.2 Sampling and data collection

Disturbed and undisturbed samples were taken from representative test pits by appropriate sampling techniques from the excavated test pits. The field works for the investigation was carried out in 2005 E.C and test pits were advanced to a maximum depth of 3.0m.

Sampling

Disturbed sampling

A total of twelve disturbed samples were collected and tested for engineering properties. The results are presented in Table and details of test result are provided.

Undisturbed sampling (Shelby tube sampling)

Undisturbed soil samples were recovered by applying static force and pressing a Shelby tube. The top and bottom of the Shelby tube samples were immediately wax sealed and covered with polyethylene bags that are labeled with necessary information for subsequent laboratory testing to determine the engineering properties of the soil in the Town. A total of twenty four undisturbed samples were collected from the test pits using dry excavation to do the following laboratory tests.

- Unconfined Compression Strength (UCS)
- Consolidation
- Natural moisture content

1.3.3 Sample transportation

The samples were transported by car from the site to the laboratory. Care was taken to avoid the disturbance of the samples. But the road between Debre-Markos and Dejene was at the maintenance stage. Due to this the sample will be disturbed.

1.3.4 Laboratory test

Tests were performed in the laboratory to determine the engineering properties of the soil in the Town.

The following laboratory tests were performed on disturbed and undisturbed soil samples.

- Sieve analysis and hydrometer
- Atterberg limits.
- Free swell
- Unconfined compressive strength and
- Consolidation

Ground water monitoring

The ground water level in the test pits were monitored during excavating activity. But within 3.0m depth there was no ground water in all the test pits. Really the altitude of the Tow is high, so it isn`t expected in this depth.

1.4 Scope of the Study

The scope of the thesis is

a) to evaluate the general properties of the soil in the town
b) to explore the sub-surface conditions and provide general data of the soil in the Town.
c) to obtain disturbed and undisturbed soil samples for carrying out laboratory tests
d) interpretation and analysis of laboratory test results
e) to determine the natural and relevant physical properties of the soil in the Town.

1.5 Organization of the Thesis

The thesis has been divided in to six Chapters. In this introductory Chapter the background, objective, research methodology, scope of the study and organization of the thesis work is presented. The second Chapter gives a brief literature review which discusses about formation, classification and types of soils. Sampling area description is dealt with in Chapter three. In the fourth Chapter the types of laboratory tests conducted with results for the research are described in detail. The laboratory test results discussion are presented in Chapter five. Chapter six contains the conclusions and recommendations drawn from the research. At last in the appendix the raw data is shown.

Chapter Two

2. Literature Review

2.1 General

2.1.1 Clay Mineralogy

The term clay is applied to the fraction of grains whose equivalent diameter is less than 0.002mm. The individual grains are fragments of a single mineral i.e. a solid compound with a definite chemical composition and unique crystalline structure.

The minerals of clays are formed by the weathering of rocks. Most clay minerals of interest to geotechnical engineers are composed of oxygen and silicon- two of the most abundant elements on earth. Silicates are a group of minerals with a structural unit called the silica tetrahedron. A central silica cation is surrounded by four oxygen anions, one at each corner of the tetrahedron (Fig 2.1a). Silica tetrahedrons combine to form sheets, called silicate sheets (Fig2.1b). Silicate sheets may contain other structural units such as alumina sheets. Alumina sheets are formed by a combination of alumina minerals, which consist of an aluminum ion surrounded by six oxygen or hydroxyl atoms in an octahedron (Fig 2.1d). [17]

The main groups of clay crystalline materials that make up clays are the minerals kaolinite, illite and montmorillonite.

2.1.2 Kaolinite

Kaolinite has a structure that consists of one silica sheet and one alumna sheet bonded together in to a layer about 0.72nm (nm = 10^{-9}m) thick and stacked repeatedly (Fig 2.2a). The layers are held together by hydrogen bonds. Kaolinite has a few or no exchangeable cation, and the interlayer bonds are relatively strong preventing any hydration between layers and allowing many layers to build. Kaolinite is relatively stable and water is unable to penetrate between the layers. Consequently kaolinite shows little swelling on wetting. Kaolinites are found in soils that have undergone considerable weathering in warm, moist climates. They have low liquid limit and a low activity. Another member of the kaolinite group appearing in some tropical soils is called halloysite, in which water molecules separate the layers. The halloysites are distinguished by one additional water molecule to the basic kaolinite. In contrast to most other clays, which are flaky, halloysite particles are tabular or rod likes [5].

2.1.3 Montmorillonite

Montmorillonites are made up of sheet like unit comprising an alumna octahedral sheet between two silica tetrahedral sheets, as shown in Fig. 2.2(c). As the electrons rotate around the nucleus of an atom there will be times when there are more electrons on one side of the atom than the other, giving rise to a weak instantaneous dipole. Weak Vander Waals forces hold layers together and the bonding of these sheets is rather weak, resulting in a rather unstable mineral, especially when wet. In fact, montmorillonite display a significant affinity for water, with subsequent swelling and expansion. Its excessive swelling capacity may seriously endanger the stability of overlying structures and road pavements. Bentonite is part of the montmorillonite clay family, usually formed from the weathering of volcanic ash [18].

2.1.4 Illite

The illites are somewhat similar to montmorillonites in the structural units, but are different in their chemical composition. In illite, the layers are separated by potassium ion, where as in montmorillonite the layers are separated by loosely held water and exchangeable metallic ions (Fig 2.2 (b)). Unlike montmorillonite particles, which are extremely small and have a great affinity for water, the illite particles will normally aggregate and there by develop less affinity for water than montmorillonites. Correspondingly, their expansion properties are less. The cation exchange capacity of illite is less than that of montmorillonite. The inner layer bonding by the potassium ions is sufficiently strong. Illites usually occur as a very small, flaky particles mixed with other clay and non-clay materials [18].

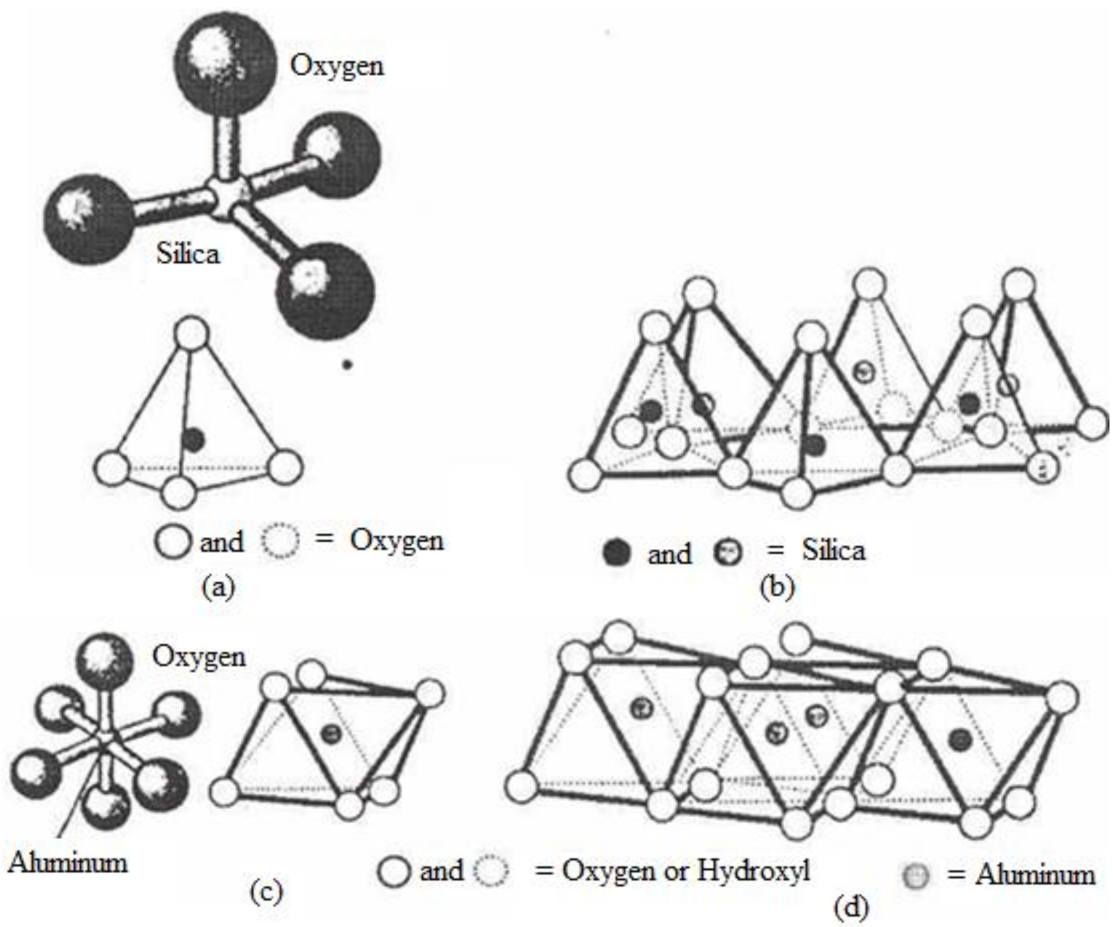

Figure 2.1 (a) A silica tetrahedron, (b) Silica sheets, (c) An aluminum octahedron, and (d) aluminum sheets [18].

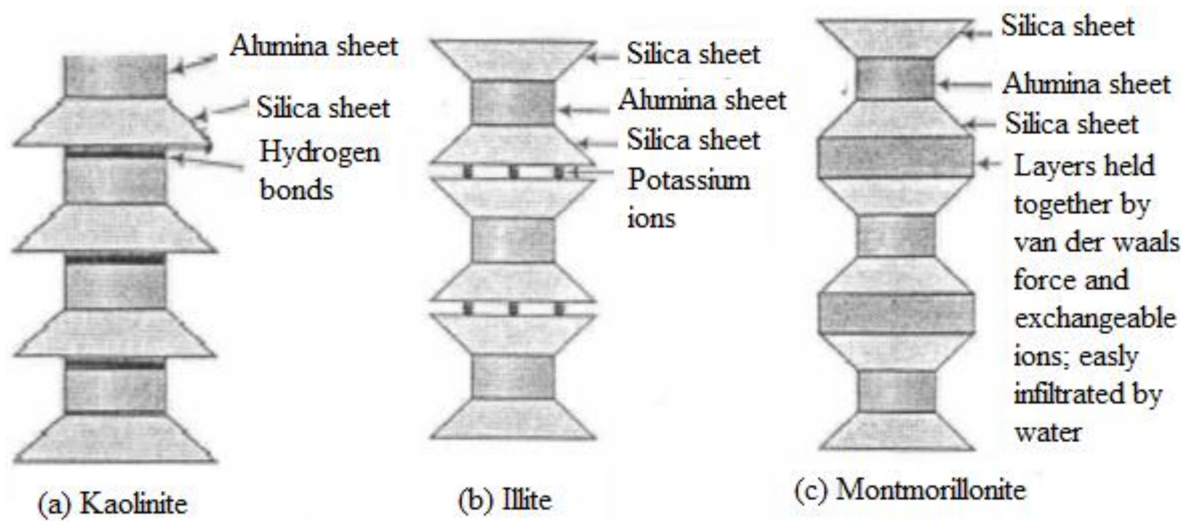

Figure 2.2 Structure of kaolinite, illite, and montmorillonite [18].

2.2 Origin and mineralogical composition of Ethiopian red clay soils

Ethiopian red clay soils are principally residual, derived from the weathering of volcanic rocks. The parent rock for black and red clays in Ethiopia is mainly olivine basalt, basalt and trachyte [19].

Ethiopian red clay soils have developed where rain fall is more plentiful and drainage is good. They contain kaolinite, illite, montmorillonite and halloysite as the principal clay minerals. The red color of the Ethiopian soils indicates the presence of iron.

The origin and mineralogical composition of Ethiopian red clay soils have been studied by previously done researches. The ranges in properties of Ethiopian soils are presented on Table 2.1.

2.3 Red clay soil compared to laterits and lateritic soils

2.3.1 General

Lateralization is the special name given to the process of weathering leading to the formation of bauxite. A distinction is made between laterite and bauxite by the iron content.

The residual altered rock mass is called laterite, when ferric oxide content is above 40%, and bauxite when the same ferric oxide is less than 40%. Lateralization is a common feature of aluminum rich and iron rocks in tropical countries [22]. Laterites are red, residually developed tropical soils formed in humid, well drained regions. One distinguishing class of deposits, namely, residual, is characteristics of weathering. As the name implies such deposits are the residual materials left behind in situ after undesirable particles have been removed from the place. Bauxite deposits is the classical example which is formed by weathering of acid to basic rocks and schists, and subsequent removal of lime, alkalies, magnesium and iron compounds in solution, leaving behind hydrate alumina, containing some proportion of ferric oxide and other impurities [22]. The degree of leaching and relative accumulation of iron and aluminum is usually determined by chemical and mineralogical analysis [15].

Latertic soils are soils formed in the process of lateralization. These soils are always found in regions where the wet period is warm. This applies to semi-humid tropical and equatorial climates.

To identify whether the red clay soil of Merawi is latteraitic or not, index properties were studied and comparison was made with known lateritic soils.

The physical and chemical properties of lateritic and red clay soils given in Tables 2.1. The average soil properties of the red clay soils of Addis Ababa and Bahir Dar are shown in Table 2.1.

Table 2.1 Ranges in properties of Ethiopian soils according to previous researches.

	Previous research (Abgena, 2003)	Previous research (Hailemariam, 1992)	Previous research (Zelalem,2005)	Previous research (Wakuna,2007)	Previous research (Tibebu, 2008
Soil type	Red clay	Red clay	Lateritic	Lateritic	Lateritic
Location	Bahir Dar	Addis Ababa	Nejo – Mendi	Asosa	Wolyita-Sodo
Clay content (%)	74 – 82	48 -73	2.0 - 20.6	2.5 - 60	48 – 69.7
Activity	0.56		0.97 – 0.98	0.62 – 1.02	0.317 - 0.488
Clay minerals	Kaolinite				Kaolinite
LL (%)	61 – 68	54 - 81	48 - 67	41 - 72	48 - 71
PI (%)	24.31	21 - 30	17 – 27	20 - 48	19 - 30
Shrinkage limit (%)	9 – 12	14 -22	7.1 – 15.7		11 - 22
Free swell (%)		10 -40	20 – 40	11 - 45	28 - 38
Specific gravity	2.75 – 2.83	2.61 -2.79	2.78 – 3.03	2.19 – 2.94	2.61 – 2.97
From plasticity chart			MH	CH, SC, MH, CL & SM	MH
Unconfined compression strength, qu(kN/m^2)	148 - 220	49 - 250	165 - 553		215 - 385

2.3.2 Comparison of properties and characteristics

Comparison of red clay soil data presented in Table 2.2. The values are compared in order to identify the characteristic of the red clay soil with respect to lateritic soil. The soil is also compared with that of previous finding by previous researchers [10]. The results are shown in Table 2.2.

Table 2.2 Comparison of red clay soil data from previous research

	Previous Research	Previous Research
Soil Type	Red Clay	Red Clay
Location	Addis Ababa	Bahir Dar
Clay Content %	58-70	74-82
Activity	< 0.75	0.56
Clay Minerals	-	Kaolinite
LL %	56 - 75	61 - 68
PL %	14 - 22	9 - 12
PI %	29 - 47	24 - 31
G_s	2.66 – 2.77	2.75 – 2.83
q_u kN/m^3	-	148 - 220

As it can be seen in Table 2.2 the engineering properties of soil in each area is similar.

Chapter Three

3. Description of the Study Area

3.1 General

Merawi, Mecha wereda capital Town, which is located 540 km from Addis Ababa in North West direction on the main high way road of Addis Ababa to Bahir Dar and 30 km away from Bahir Dar in the South West direction was founded in 1935E.C with the establishment of saint Miriam Church on the North of the Town and in the late1990`s saint George in the South West of the Town situated at latitude 11$^{o}$24`56.36``N, longitude 37$^{o}$09`33.03``E and an average elevation of 2010m above sea level. The topography of the Town is mainly flat with good drainage system in the South to North direction. The existing master plan of Merawi dates back to the late 1973E.C. Since then, few revisions in the detail master plan have been made again 2002 E.C. The master plan in use today covers a land area of about 300 hectares (Fig 3.1).

The area by visual inspection is covered by red clay soils.

3.2 Climatic Characteristics

Data collected for 10 years shows that the mean annual rainfall in Merawi reaches 1627.90 mm. There is a considerable seasonal variation of this rainfall depth in which the highest is recorded in the summer season (kiremt i.e. June, July and August) time and the lowest is recorded in the winter season (Bega i.e. December, January and February) as shown in Table 3.1.

Table 3.1 Mean monthly rainfall of Merawi in mm (1995 -2006) [Ethiopian metrology agency, Bahir Dar branch]

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Rainfall	6.37	4.24	11.76	29.89	141.04	309.61	432.27	346.89	224.54	92.23	24.51	4.54

The average temperature of the Town recorded for 10 years showed that the range of temperature variation throughout the year is between 13.73^{0}C and 20.78 ^{o}C (Table 3.2).

Table 3.2 Mean monthly temperature of Merawi in ^{o}C (1995 -2006) [Ethiopian metrology agency, Bahir Dar branch]

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Tem	13.73	16.98	20.78	19.48	19.56	18.05	18.61	18.53	18.82	19.28	18.44	17.48

Figure 3.1 Current master plan of Merawi Town [Bahir Dar Town administration office]

Figure 3.2 Map of Merawi Town and place of sample taken [Bahir Dar Town administration office]

The master plan shows the current plan of the Town and its future expansion as it can be seen in Fig. 3.1. For this thesis, six representative test pits were taken. The number of test pits taken were limited because of that the soil in the entire area of the Town is more or less similar and it has only three kebeles. In each kebele two representative test pits were taken as shown in the Fig.3.2 in which currently constructions are active in the sample taken area.

Chapter Four

4. Laboratory Test Results

4.1 Index Properties

4.1.1 General

In nature, soils occur in a large variety. However, soils exhibiting similar behavior can be grouped together to form a particular group. Engineers are continually searching for simplified testes that will increase their knowledge of soils beyond that which can be gained from visual examination without having to resort to the expense, detail and precision required with engineering properties tests. These simplified tests provide indirect information about the engineering properties of the soil and are, therefore, called index testes.

Basic soil properties and parameters can be subdivided into physical, index, and engineering categories. Physical soil properties include particle size and distribution, specific gravity, and water content. Index parameters of cohesive soils include liquid limit, plastic limit, shrinkage limit, and activity. Such parameters are useful to classify cohesive soils and provide correlations with engineering soil properties [15].

4.1.2 Specific Gravity Test

The term specific gravity is defined as the ratio of the weight of a given volume of material to the weight of an equal volume of water in effect it tells how much the material is heavier or lighter than water. The particular geotechnical term specific gravity of soil refers therefore to the ratio of the weight of the solid mater of a given soil sample to the weight of an equal volume (i.e. equal to the volume of solid matter) of water.

To determine the specific gravity of soil solid particles for particles size less than 2.0 mm by means of pycnometer using the test method of ASTMD854-92AS. Specific gravity test results of the soil at each test pit are shown in Table 4.1.

Table 4.1 Specific gravity test results

Pit	1.50m	3.0m
PT-1	2.73	2.75
PT-2	2.70	2.74
PT-3	2.72	2.72
PT-4	2.73	2.74
PT-5	2.72	2.76
PT-6	2.71	2.72

4.1.3 Grain Size Analysis

Since grain size analysis is one of the index property tests, the soil of the study area is examined for its grain size distribution. Grain size divides soil into two distinctive groups, namely cohesion less and cohesive soil. Soil particles, which are coarser than 0.075 mm, are generally termed as cohesion less and the finer ones like silt and clay are considered fine grained [13].

The property of cohesion less soil is greatly based on grain size distribution while the property of fine-grained soil is influenced by inter particle force. Hence, the behavior of a soil mass is dependent on the size of the particles it has. It is quite necessary then, to clearly know the proportion of different grains or particles a soil system contains.

For coarse-grained soils this is done by sieve analysis. For fine-grained soils another method called hydrometer analysis is used for this purpose.

In the present study, hydrometer and sieve analysis were performed on all the samples and the results were plotted on a semi-log paper as shown in Fig.4.1. From these curves the proportion and type of soil grains can be determined. The results of the test are given in Table 4.2 and particle size distribution curves in Fig. 4.1.

Table 4.2 Particle size analysis test results

Condition of sample	Location	Test pit No.	Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
Air dry	Merawi	1	1.50	0.30	5.55	19.15	75.00
			3.00	0.20	0.50	17.40	81.91
		2	1.50	0.04	10.54	16.03	73.40
			3.00	0.02	6.80	16.40	76.77
		3	1.50	0.02	1.23	27.43	71.32
			3.00	0.08	1.00	26.54	72.35
		4	1.50	0.16	2.62	31.98	65.24
			3.00	0.12	1.22	35.07	63.59
		5	1.50	0.16	0.98	16.00	82.86
			3.00	0.16	0.73	7.85	91.26
		6	1.50	0.04	1.32	18.89	79.74
			3.00	0.06	1.69	13.43	84.82

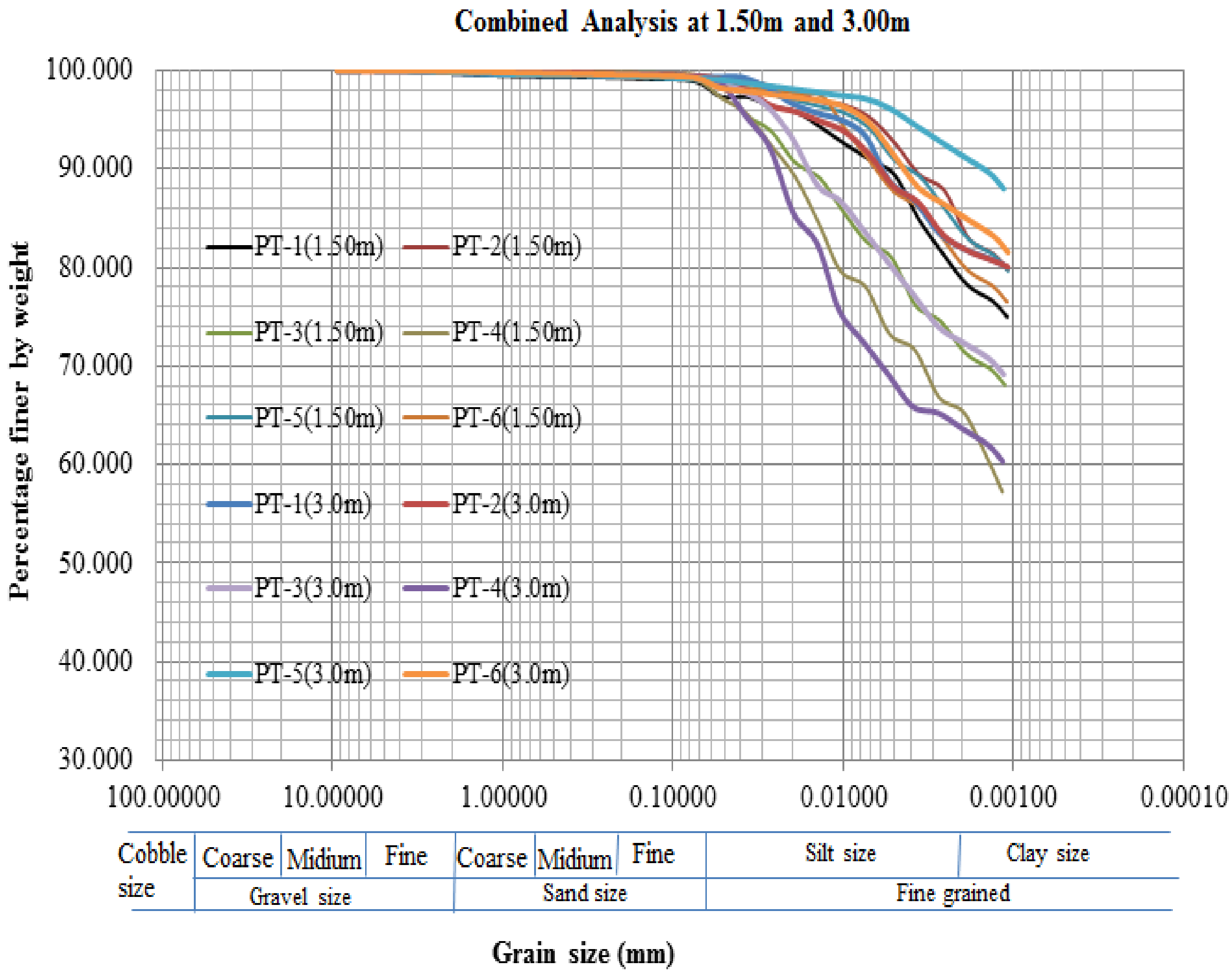

Fig. 4.1 Particle size distribution curves

4.1.4 Atterberg Limit Test

In soils containing largely of fine grains, the amount of water present in the voids has a pronounced effect on such engineering properties as shear strength and compressibility. Consistency is used as the basis for their classification. The consistency of a soil is its physical state characteristics at given moisture content.

Atterberg proposed five status of soil consistency [7]. The liquid and plastic limits are used internationally for soil identification and classification and for strength correlations. The shrinkage limit is useful in certain geographical areas where soils undergo large volume changes when going through wet and dry cycles. The collision and sticky limits are not generally used in geotechnical engineering work [7].

Liquid Limit: The liquid limit of a soil is the water content at the boundary between the liquid and plastic states. The water content at this boundary is arbitrarily defined as the water content at which, two halves of a soil pat placed in a brass cup, cut with standard groove, and dropped from a height of 1cm will undergo a groove closure of about 1.3 cm when the cup is dropped 25 times at the rate of 2 drops per second.

Plot the water content versus number of blows in semi logarithm paper. A straight line should be drawn through the data points. Obtain the liquid limit, LL, at 25 blows. The result is shown in Table 4.3.

Plastic Limit: The plastic limit of a soil is the water content at the boundary between the plastic and semisolid state. The water content at this boundary is arbitrarily defined as the water content at which soil begins to crumble when rolled into threads of specified size (3.2mm). The result is shown in Table 4.3.

Plasticity index: The plasticity index is the numerical difference between liquid and plastic limit. It represents the range in water content through a soil is in plastic state. A high numerical value of plasticity index is an indication of the presence of high percentage of clay in the soil sample. Information regarding to the type of clay in the sample, however, may be obtained by considering the plasticity index in relation to the liquid limit [16]. The test results are shown in Table 4.3.

Effect of particle size on liquid limit: A decrease in particle size would be accompanied by an increase in total surface and therefore, an increase in plasticity index would be expected. The plasticity index increases much more rapidly with an increase in total surface for montmorillonite than for kaolinites, with illites and halloysites being intermediate [4]. It is generally believed that soils with high clay content exhibit high liquid limit. Information regarding to the type of clay in the sample, however, may be obtained by considering the plasticity index in relation to the liquid limit. This is shown by the plasticity chart in Fig. 4.3.

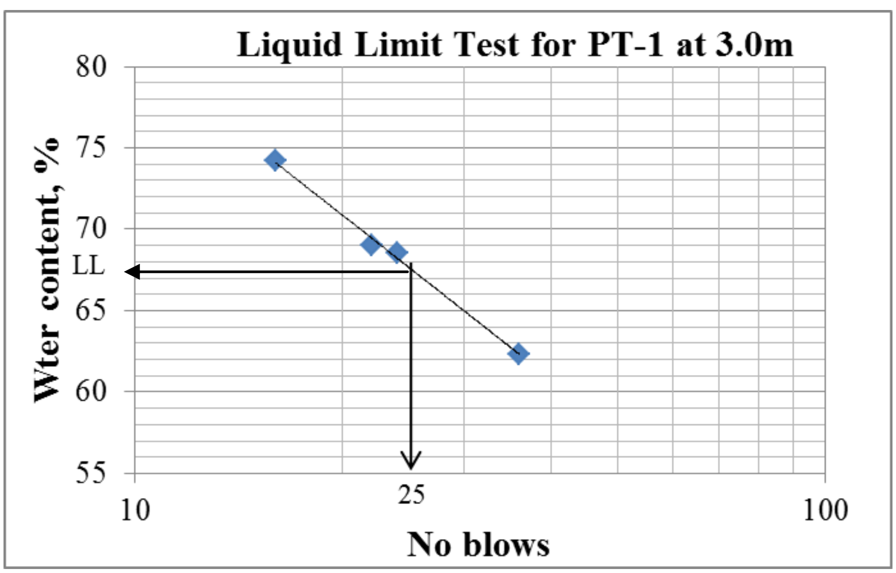

Fig. 4.2 Typical plot of liquid limit test results

Table 4.3 Atterberg limit test results

Location	Merawi					
Depth	1.50m			3.00m		
PT	LL (%)	PL (%)	PI (%)	LL (%)	PL (%)	PI (%)
1	59. 5	27.1	32.3	67.8	28.4	39.4
2	52.3	24.5	27.8	55.8	27.0	28.8
3	60.8	30.8	30.0	63.8	31.3	32.5
4	53.5	23.6	29.9	55.8	26.0	29.8
5	59.8	30.1	29.7	65.5	33.4	32.1
6	61.3	30.7	30.6	65.5	29.3	36.2

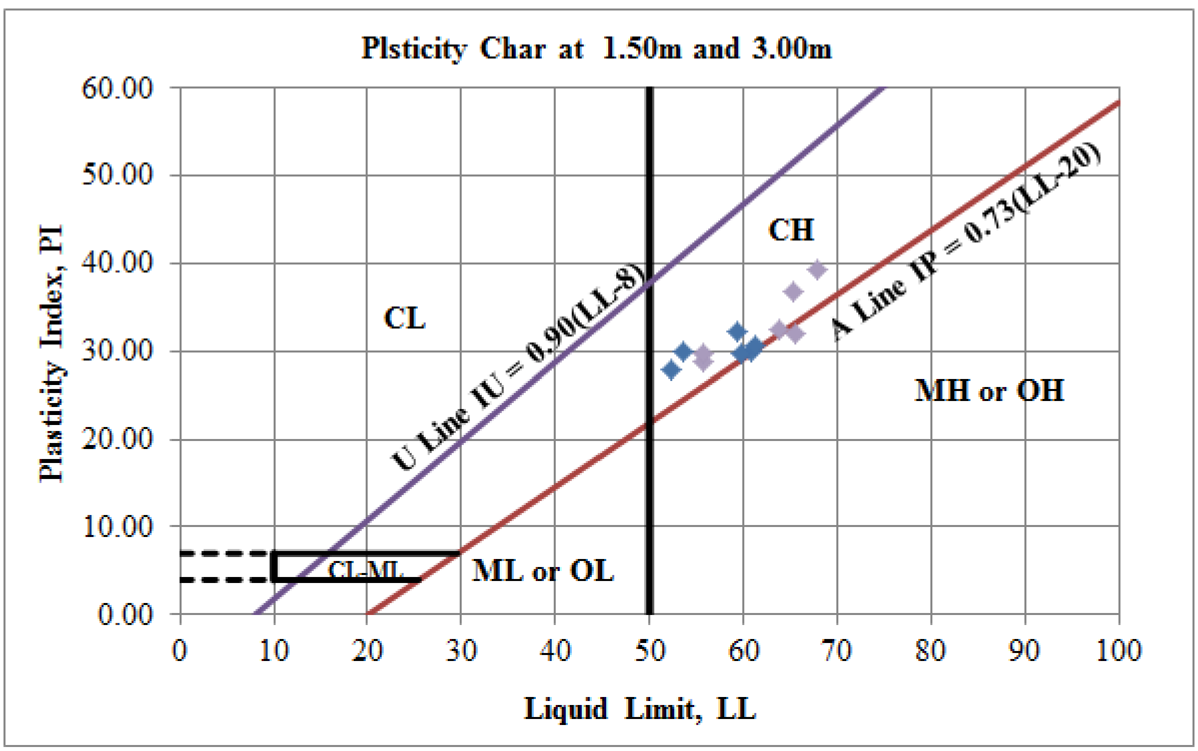

Fig. 4.3 Results plotted in plasticity chart [9]

Activity: activity is the ratio of the plasticity index to that of the amount of the clay fraction, defined as the percent dry weight of the minis of 2 fraction of the sample. Activity is a very useful parameter indicating the plasticity index of the clay size fraction of the soil.

If a plot is obtained between the clay fraction (as abscissa) and the plasticity index (as ordinate), it is observed that all points for a particular soil lie on a straight line. The slope of the line gives the activity of the soil. The steeper the slope, the greater is the activity. So those soils that have activity greater than 1.25 are active. Soils which have activity between 0.75 and 1.25 are normal while soils that have activity less than 0.75 are inactive soils [10].

The activity values as defined consider that, for a given mineralogical composition of the clay size fraction, the plasticity index of the material increase in simple proportion as the clay size increase in percent of the total weight.

In general, one would expect that active clays would have relatively high water –holding capacity, high compaction under load and high cat ion exchange capacity. Therefore, properties vary greatly with the nature of the exchangeable cation. They would also be highly thixotropic, have low permeability and low resistance to shear, with very low activity. There would be little cohesion, and internal friction would be largely responsible for the strength. Very active soils, therefore, would be expected to cause problems for the engineer [14].

From the results of Atterberg limits and particle size determinations, it is possible to estimate the degree of expansiveness by plotting the results on an estimated activity chart shown in Fig. 4.4 which shows the test results of the Town.

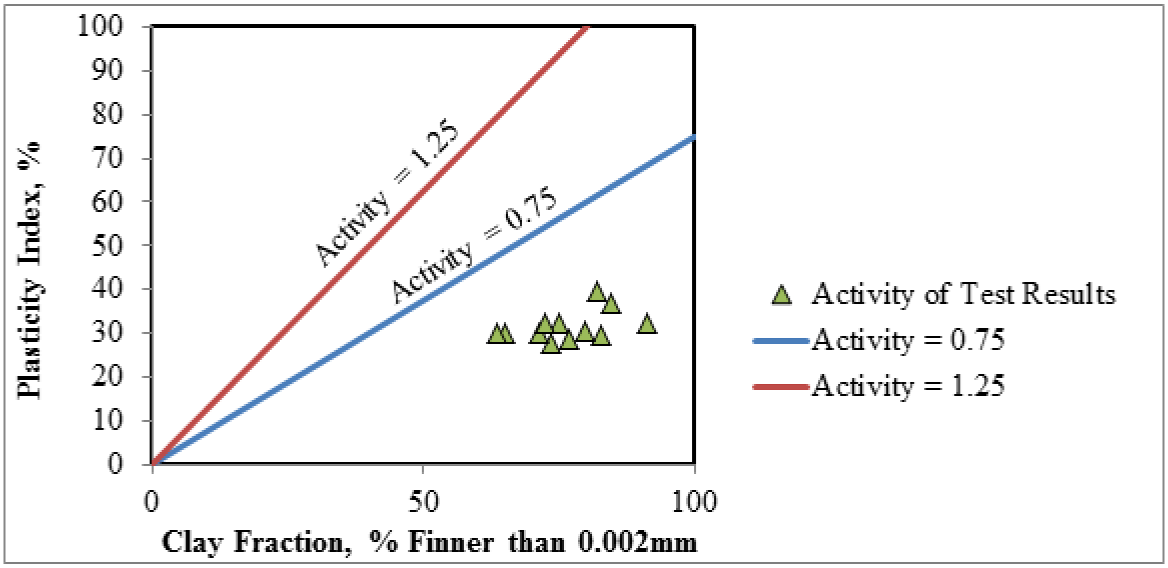

Fig. 4.4 Activity chart [9]

4.1.5 Free Swell

This method is suggested by Holtz and Gibbs (1956) to measure the expansive potential of soil. This is performed by pouring slowly 10 cc of dry soil passing 425 microns sieve, into a 100 cc graduated cylinder filled with water. The volume of swelled soil is measured after 24 hours, from the graduations of the cylinder. The free swell value is present, is then determined.

$$\text{Free Swell}(\%) = \frac{Final\ volume - \ Intial\ volume}{intial\ Volume} x\ 100 \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (4.1.4)$$

Table 4.4 Free swell test results

Pit	1.50m	3.0m
PT-1	16.7	18.3
PT-2	15.0	16.7
PT-3	14.5	15.00
PT-4	18.3	19.4
PT-5	18.3	20.0
PT-6	15.3	17.9

As can be seen from the Table 4.4 the free swell values are low and vary in a narrow range. Those soils having a free swell less than 50% are considered as non-expansive.

4.1.6 Natural moisture content, density and consistency index

The water or moisture content of a soil material is the ratio of the mass of water in the sample and mass of solid material. It is expressed as percentage. The natural moisture content is used in calculating the liquidity index (LI) and the consistency index (CI). The liquidity index indicates the nearness of natural moisture content of the soil to the liquid limit while as consistency index shows the nearness of water content of the soil to its plastic limit [25].

$$LI = \frac{w - PL}{PI} * 100 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.1)$$

When a soil is at its liquid limit, its liquidity index is 100% and it behaves as a liquid. When the soil is at the plastic limit, its liquidity index is zero. Negative values of liquidity index indicate water content smaller than the plastic limit. The soil is then in a hard (desiccated) state.

$$CI = \frac{LL - w}{PI} * 100 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.2)$$

The consistency index indicates the consistency (firmness) of the soil. It shows the nearness of the water content of the soil to its plastic limit. A soil with a consistency index of zero is at the liquid limit. It is extremely soft and has negligible shear strength. On the other hand, a soil at a water content equal to the plastic limit has a consistency index of 100%, indicating that the soil is relatively firm. A consistency index of greater than 100% shows that the soil is relatively strong, as it is the semi-solid state. A negative value of consistency index is also possible, which indicates that the water content is greater than the liquid limit. The consistency index is also known relative consistency

The bulk mass density is the total mass per unit total volume while dry density is the mass of solid per total unit volume. The test results are shown in Table 4.5.

Table 4.5 Water content, liquidity index, density and consistency index of the soil.

Location	Test pit No.	Depth (m)	Bulk desity kN/m^3	Dry density kN/m^3	Natural moissture content (%)	LL (%)	PL (%)	PI (%)	Liquidity index (%)	Consistency index (%)
Merawi	1	1.50	16.2	11.9	36.2	59.5	27.1	32.3	28.2	72.1
		3.00	16.0	11.6	37.9	67.8	28.4	39.4	23.9	75.9
	2	1.50	17.1	12.3	38.4	52.3	24.5	27.8	50.0	50.0
		3.00	16.9	12.3	37.4	55.8	27.0	28.8	36.1	63.9
	3	1.50	15.0	11.3	32.7	60.8	30.8	30.0	6.3	93.7
		3.00	16.7	12.2	37.2	63.8	31.3	32.5	18.2	81.8
	4	1.50	17.4	12.7	37.3	53.5	23.6	29.9	45.8	54.2
		3.00	17.7	12.9	37.4	55.8	26.0	29.8	38.3	61.7
	5	1.50	17.0	12.6	34.5	59.8	30.1	29.7	14.8	85.2
		3.00	18.3	13.3	37.6	59.5	33.4	26.1	16.1	83.9
	6	1.50	17.1	12.6	35.6	61.3	32.0	29.4	12.2	87.4
		3.00	17.1	12.5	37.3	52.4	29.8	22.6	33.2	66.8

As can be seen from Table 4.5 the consistency index of the soil under investigation varies from 50.0% to 93.7%, which was also observed visually in the field.

4.2 Shear Strength Test

4.2.1 General

The shear strength of soils is an important aspect in many foundation-engineering problems related to stability such as the bearing capacity of shallow foundations and deep foundation, the stability of slopes of dams and embankments, and lateral earth pressure on retaining walls.

The purpose of shear strength testing is to establish representative values for the shear strength parameters from laboratory and field testes. The most common laboratory methods employed to obtain shear strength parameters are direct shear test, triaxial compression test and unconfined compression test. For this research work only unconfined compression tests were conducted.

4.2.2 Unconfined Compression Test

The unconfined compression test is a special case of the unconsolidated un-drained triaxial test. In this simple test, a cylindrical cohesive specimen without any lateral support is subjected to axial loading, till the sample falls either due to shear along a diagonal plane or by the lateral bulging. The test is un-drained test and is based on the assumption that there is no moisture loss during the test. The UC test is one of the easiest and simplest tests for determining a quick estimate of the shear strength of cohesive soils. The test provides an immediate approximate value of the compressive strength of the soil, either in the undisturbed or the remolded condition. It is also widely used to determine the consistency of saturated clays and other cohesive soils [23]. The UC tests were carried out on 12 undisturbed samples obtained, by tube sampling, from the field. Details of testing procedures are followed according to ASTM D2166. The results of UC tests are shown in the Table 4.6 and typical UCS test result in Fig. 4.5.

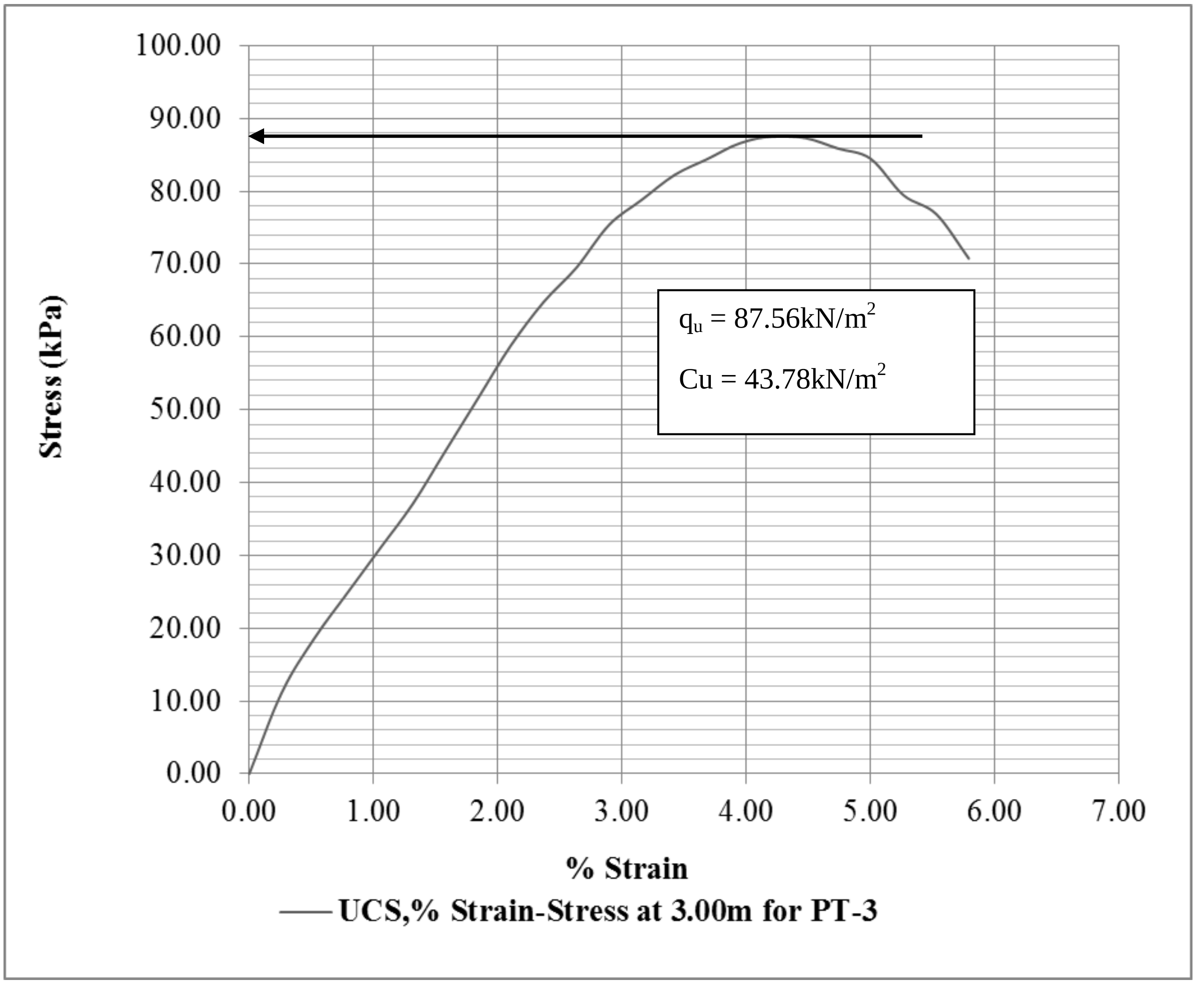

Fig. 4.5 Typical UCS test result

The samples were failed due to diagonal shear failure in the laboratory when samples were tested for UCS test.

Table 4.6 Unconfined compressive strength test results

Location	Test pit No.	Depth (m)	Dry density (kN/m3)	Natural moissture content (%)	PI (%)	Unconfined comprensive strength (kN/m2)	undrained strength (kN/m2)	Specific gravity (Gs)	Degree of saturation S (%)
Merawi		1.5	11.9	36.2	27.1	63.67	31.83	2.73	76.4
	1	3.0	11.6	37.9	28.5	96.89	48.45	2.75	76.0
		1.5	12.3	38.4	24.5	85.49	42.75	2.7	86.8
	2	3.0	12.3	37.4	27	106.75	53.38	2.74	83.5
		1.5	11.3	32.7	30.8	64.29	32.14	2.72	63.2
	3	3.0	12.2	37.2	31.3	87.56	43.78	2.72	82.3
		1.5	12.7	37.3	23.6	89.25	44.62	2.73	88.6
	4	3.0	12.9	37.4	26	108.83	54.41	2.74	91.2
		1.5	12.6	34.5	30.1	81.25	40.62	2.72	81.0
	5	3.0	13.3	37.6	33.4	115.99	58	2.76	96.5
		1.5	12.6	35.6	32	67.54	33.77	2.71	83.8
	6	3.0	12.5	37.3	29.8	117.81	58.91	2.72	86.3

4.3 Consolidation Test

4.3.1 General

Consolidation is the compression of saturated soil under static pressure. When saturated clay water system is subjected to an external pressure, the pressure applied is initially taken by the water in the pores, resulting hydraulic gradient initiate the flow of water out of the clay mass & the mass being to compress. The compression rate is very slow as clay soils have very small permeability. So the whole process is time taking. As water starts squeezing out from voids, Proton of the applied stress is transfer to the soil skeleton. This in turn causes a reduction in the excesses pour pressure. This process involving gradual compression occurring simultaneously with a flow of water out of the mass and with gradual transfer of the applied pressure from the pore water to the mineral skeleton is called consolidation.

The consolidation of a soil may be considered to consist of two stages. Primary consolidation stage during which the applied pressure increment is transferred from the pore water to the skeleton, and secondary consolidation stage that follows the end of primary phase. A study of consolidation requires knowledge of the compressibility of the soil skeleton and the rate at which excess pores pressure dissipates which is related to the permeability [3]. The compressibility characteristics of a soil relating both the magnitude and the rate of settlement are usually determined from the consolidation test, using an apparatus called odometer [7].

The consolidation parameters of a soil are compression index (Cc) and coefficient of consolidation (Cv). The compression index relates to how much consolidation or settlement will takes places while the coefficient of consolidation relates to how long it will take for the amount of consolidation to take place.

To determine the settlement characteristics (parameters) of a given soil or the settlement condition of soil by using a standard reference of ASTM D 2435 - Standard test method for one-dimensional consolidation properties of soils is used for this thesis work.

4.3.2 Compression index and recompression index

The slope of the straight line portion of the e – log p plot gives the compression index (Cc) for the pressure above pre-consolidation pressure (Pc) and recompression index (Cr) for the pressure between over burden pressure (σ) and pre-consolidation pressure (Pc). The test data were plotted as void ratio against log pressure and then Cc and Cr were determined from Fig. 4.7. The results are given in Table 4.8

4.3.3 Pre-consolidation pressure

The pre-consolidation pressure can be obtained from the Oedometer test result estimation. The pre – consolidation pressure (Pc) was determined from the curve of void ratio versus log pressure curves from Fig. 4.8 using an empirical graphical method proposed by Casagrade [25]. The results are shown in Table 4.8 from the results obtained; the soil is over consolidated since the pre-consolidation pressure is greater than effective stress for all test pits.

4.3.4 Coefficient of consolidation

The coefficient of consolidation relates how long will it take for a given degree of consolidation to take place. There are two popular methods that can be used to estimate coefficient of consolidation Cv.

Taylor (1942) proposed one method called the square root of time method. Casagrade and Fadum (1940) proposed the other method called the log time method. The two methods specified generally show reasonable agreement.

The log time method makes use of the early (primary consolidation) and later time response (secondary compression) while the square root time method only utilizes the early time response, which is expected to be straight line.

In theory, the square root time method should give good results except when nonlinearities arising from secondary compression cause substantial deviations from the expected straight lines [18]. These deviations are most pronounced in fine grained soils with organic materials. The square root of time method has been used in this work.

As proposed by Taylor, the dial readings on the ordinate axis to a natural scale and then the corresponding values on the abscissa as the square root of time are plotted. A typical graphical plot is shown in Fig. 4.9 and the laboratory results are given in Table 4.8.

11Table 4.7 Pressure void ratio relationship for all TPs

Pressure void ratio Relationship			
	TP-1	TP-2	TP-3
P	e	e	e
50	1.295	1.065	1.153
100	1.21	1.049	1.106
200	1.126	1.021	1.051
400	1.018	0.99	0.982
800	0.908	0.943	0.902
1600	0.807	0.865	0.807
800	0.769	0.884	0.82
400	0.779	0.894	0.83
200	0.789	0.904	0.84
100	0.797	0.911	0.848
50	0.805	0.919	0.856

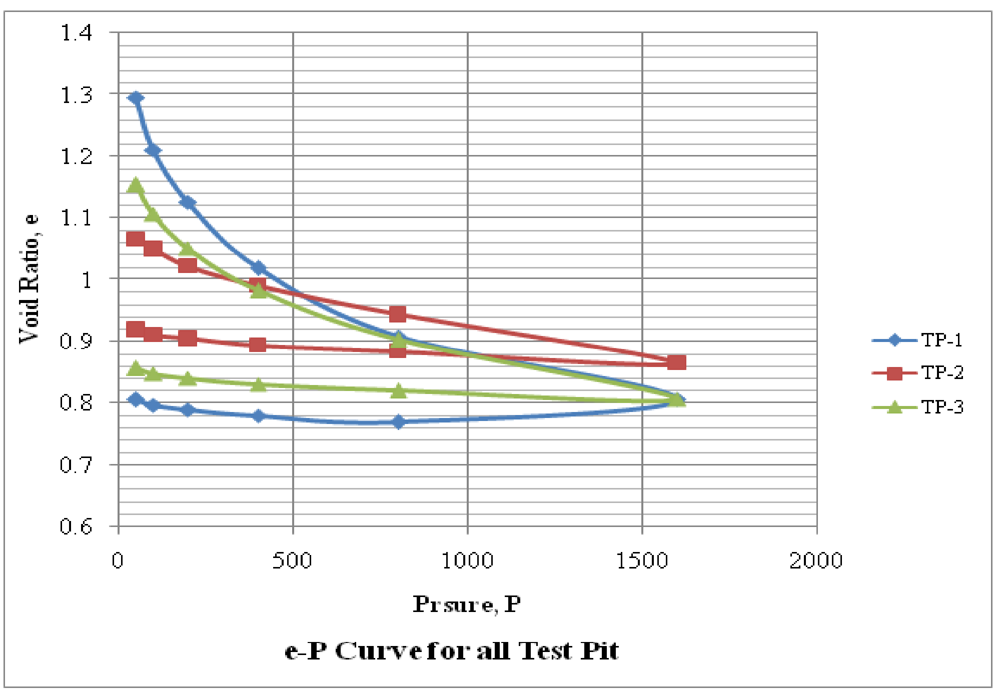

Fig. 4.6 Plot of e-p curves

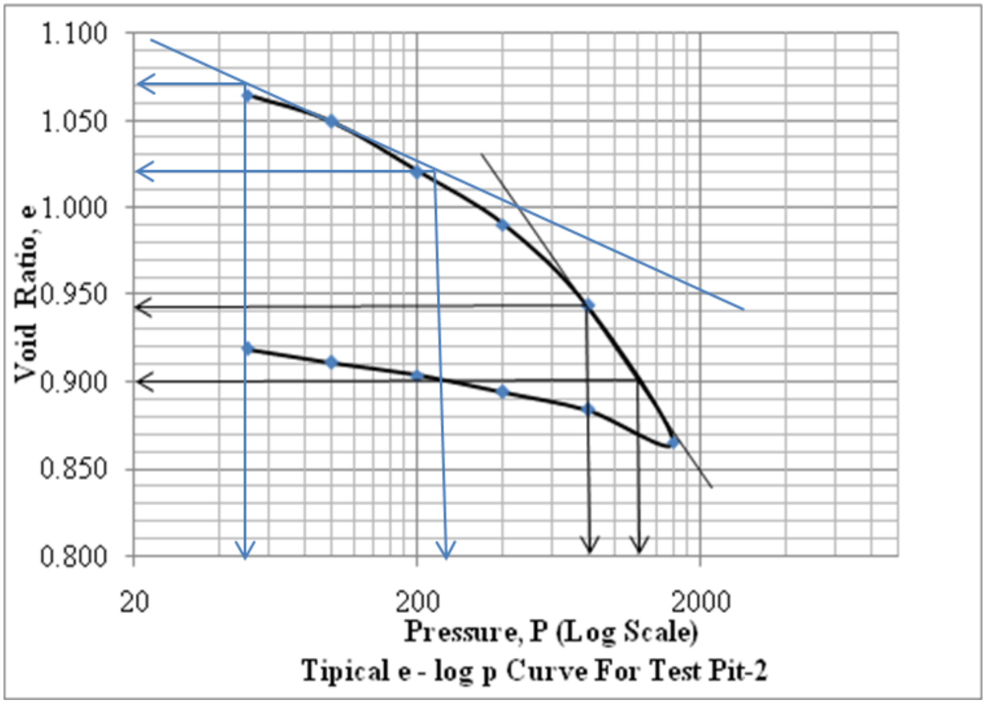

Fig. 4.7 Typical e - log-p curves

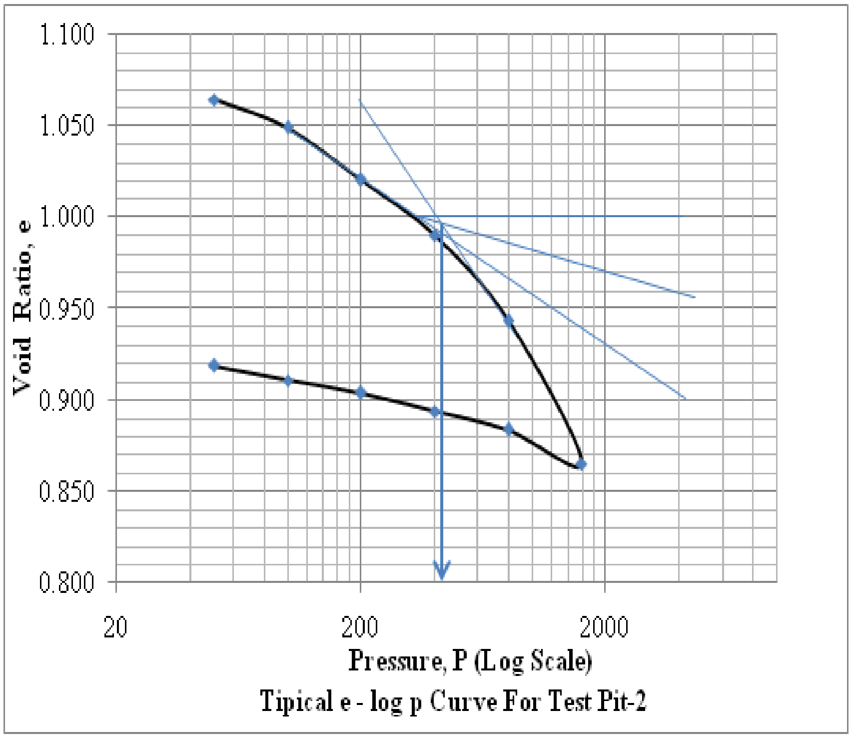

Fig. 4.8 Typical pre-consolidation determination from e-log p curve

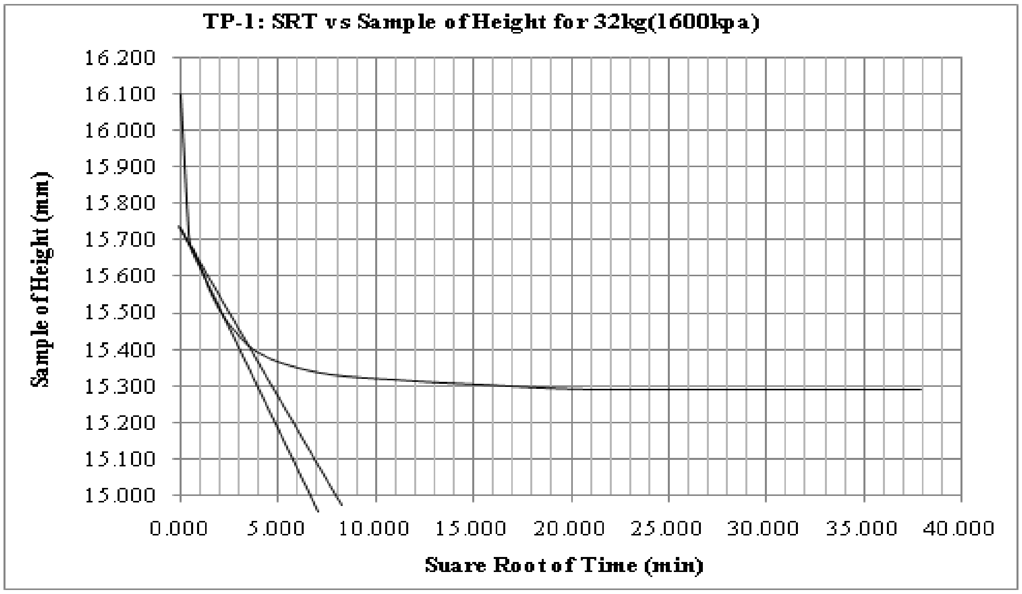

Fig. 4.9 Typical graphs for square root of time

Consolidation laboratory test result from test pit one, two and three

Table 4.8 Consolidation test results

Location	Test pit No.	Depth (m)	Natural moissture content (%)	Bulk density (kN/m^3)	Pressure P (kN/m^2)	Void ratio e	Cofficient of Consolidation Cv (cm^2/min)	Recompression Index Cr	Compression Index Cc	Over burden Pressure, σ (kN/m^2)	Pre-consolidation Pressure, Pc (kN/m^2)
Merawi	1	3.00	37.9	16.0	50	1.295	25.16	0.0805	0.4119	48	220
					100	1.21	18.89				
					200	1.126	21.66				
					400	1.018	16.08				
					800	0.908	11.98				
					1600	0.807	4.26				
	2	3.00	37.4	16.9	50	1.065	5.25	0.0288	0.2442	50.72	420
					100	1.049	13.23				
					200	1.021	24.99				
					400	0.99	8.74				
					800	0.943	12.13				
					1600	0.865	4.69				
	3	3.00	37.2	16.7	50	1.153	20.69	0.0559	0.2658	50.03	280
					100	1.106	8.23				
					200	1.051	33.55				
					400	0.982	17.83				
					800	0.902	6.50				
					1600	0.807	5.01				

Table 4.9 Coefficient of consolidation for all PTs

	PT-1	PT-2	PT-3
P	Cv (mm^2/min)	Cv (mm^2/min)	Cv (mm^2/min)
50	25.16	5.25	20.69
100	18.89	13.23	8.23
200	21.66	24.99	33.55
400	16.08	8.74	17.83
800	11.98	12.13	6.50
1600	4.26	4.69	5.01
Ave Cv	16.34	11.51	15.30

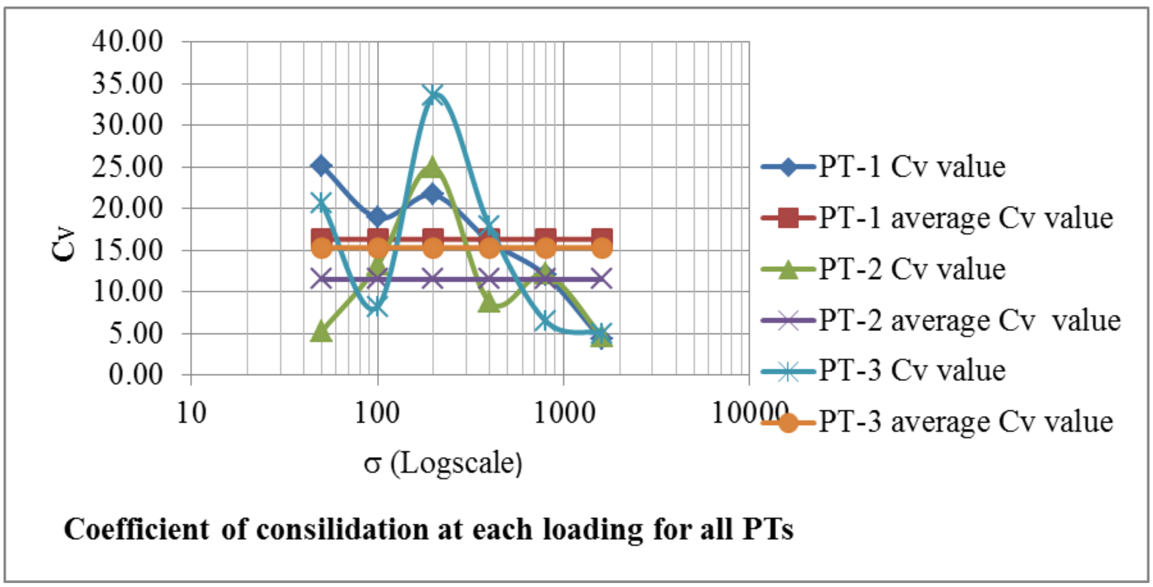

Fig. 4.10 Pressure Vs coefficient of consolidation

4.3.5 Coefficient of permeability

The coefficient of permeability can be measured using field tests, or tests conducted in the laboratory. Permeability is sometimes also estimated from one dimensional consolidation test. The coefficient of permeability can be obtained from the following relationship [6].

$$K = \frac{c_v a_v \gamma_w}{1+e} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.3)$$

Using equation 4.3, the coefficient of permeability as the function of void ratio was calculated from the consolidation test results and shown in Table 4.10. It is noted that av, the ratio of change in void ratio to change in pressure, was obtained from Fig. 4.6. As shown in Table 4.10, the range of values of coefficient of permeability lies between 0.4x10^{-6}cm/sec and 66.7x10^{-6}cm/sec, which indicates that the soils are impervious. All the soil samples taken from the field conform to this observation.

Table 4.10 Relationship between void ratio and coefficient of permeability

Location	Test pit No.	Depth (m)	Void ratio e	Coefficient of consolidation (10^{-4} cm^2/sec)	Coefficient of compressibility (10^{-4} m^2/kN)	Coefficient of permeability (10^{-6} cm/sec)
Merawi	1	3	0.908	7.10	7.82	2.85
			1.018	19.97	20.22	19.62
			1.126	26.80	14.57	18.01
			1.21	36.10	41.65	66.74
			1.295	31.48	22.05	29.67
	2	3	0.943	7.82	0.97	0.38
			0.99	20.22	1.18	1.18
			1.021	14.57	1.55	1.10
			1.049	41.65	2.8	5.58
			1.065	22.05	3.2	3.35
	3	3	0.902	8.35	1.19	0.51
			0.982	10.83	2	1.07
			1.051	29.72	3.45	4.90
			1.106	55.92	5.5	14.33
			1.153	13.72	9.4	5.87

4.3.6 Modulus of compressibility

For soils, whose behavior is typically non-linear, modulus of compressibility (Es) is not constant. The compressibility curve is obtained either from the plot of relative settlement versus effective stress or the void ratio against effective stress. The curve may be expressed by the following equation [22].

$$E_s = \frac{d\sigma`}{ds`} = v(\sigma`)^w \quad \ldots \ldots \ldots \ldots \ldots \ldots (4.4)$$

Where,

$$s` = \frac{\Delta H}{H}$$

v and w are coefficients; v has a unit of kN/m^2

w is dimensionless

σ` is effective normal stress, kN/m^2

The coefficient v depends on the void ratio, water content and consistency of the sample, it could have values ranging from 50 to 3000 kN/m^2 [22]. While as w depends on soil type. It could assume values ranging from 0 to 1 [22]. To make the exponent dimensionless, it is advisable to make σ` also dimensionless dividing by a unit stress σe. equation (4.4) becomes.

$$E_s = \frac{d\sigma`}{ds`} = v(\sigma`e)^w \ldots \ldots \ldots \ldots \ldots \ldots (4.5)$$

Where

$$\sigma`e = \frac{\sigma`}{\sigma e}$$

The tangent of the compressibility curve, which is a function of σ`, gives the modulus of compressibility Es. From equation (4.5)

$$ds` = \frac{1}{v}(\sigma`)^{-w} \; d\sigma`$$

$$s` = \int \frac{1}{v}(\sigma`)^{-w} \; d\sigma`$$

$$s` = \frac{1}{v(1-w)}(\sigma`)^{1-w} + C \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 4.6$$

Where C is constant of integrations

$for\ \sigma` = 0, s` = 0, then\ C = 0$

Equation (4.3.5.4) becomes;

$$s` = \frac{1}{v(1-w)}(\sigma`)^{1-w} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 4.7$$

$$s` = a(\sigma`)^{k} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 4.8$$

Taking common logarithm, equation (4.3.5.5) ;

$$Logs` = kLog\sigma` + Loga \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 4.9$$

From a plot of s` versus σ` one obtains a straight line relationship for some cohesive soils. Others soils give a straight line relationship when results are plotted on double log scale [22].

Using equation (4.9), on the data obtained from one dimensional consolidation test, relative settlement versus pressure (effective stress) was plotted on log-log scale as shown in Fig. 4.11. The values of coefficients, v and w were calculated using Fig. 4.11 and tabulated on Table 4.12.

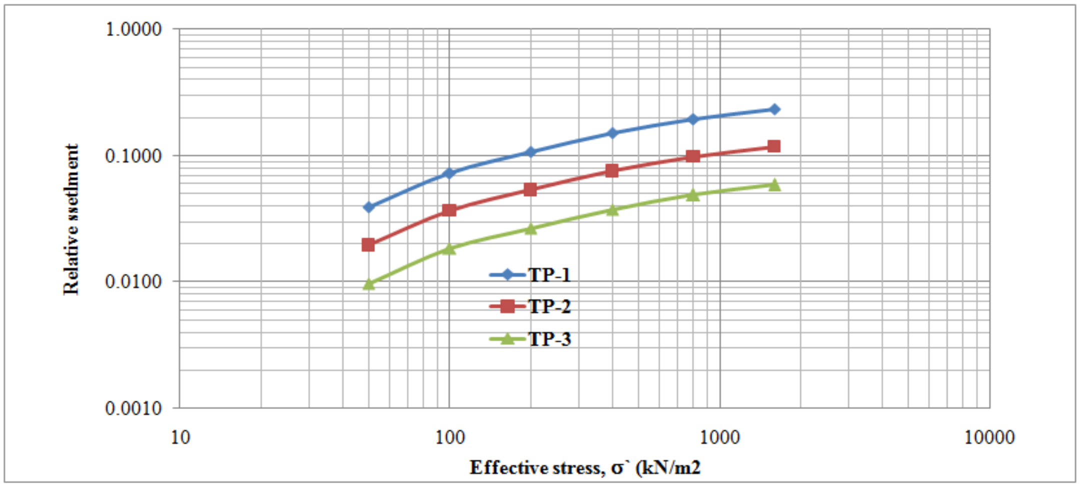

Fig. 4.11 Effective stress Vs relative settlement

Table 4.11 Total compression and relative settlement

Location	Test pit No.	Depth (m)	Effective stress, s` (kN/m^2)	Total compression, ΔH (cm)	Relative settlement (s`=ΔH/H)
Merawi	1	3	50	0.0780	0.0390
			100	0.1465	0.0733
			200	0.2142	0.1071
			400	0.3019	0.1510
			800	0.3899	0.1950
			1600	0.4707	0.2354
	2	3	50	0.0170	0.0085
			100	0.0316	0.0158
			200	0.0588	0.0294
			400	0.0868	0.0434
			800	0.1300	0.0650
			1600	0.2001	0.1001
	3	3	50	0.0475	0.0238
			100	0.0890	0.0445
			200	0.1365	0.0683
			400	0.1943	0.0972
			800	0.2599	0.1300
			1600	0.3354	0.1677

Table 4.12 Coefficients, v and w, and equations of modulus of compressibility

Location	Test pit No.	Depth (m)	v (kN/m^2)	w	Modulus of compressibility, ES (kN/m^2)
Merawi	1	3	50	0.1926	$50(\sigma`)^{0.1926}$
	2	3	50	0.6374	$501(\sigma`)^{0.6374}$
	3	3	50	0.8445	$50(\sigma`)^{0.8445}$

Substituting the value of the coefficients v and w into Eq.4.7, equation of the modulus of compressibility can be written as shown in Table 4.12. From Table 4.12 equations the modulus of compressibility are calculated and presented in Table 4.13.

17Table 4.13 Modulus of compressibility

Location		Merawi	
Test pit no.	1	2	3
Depth(m)	3.00	3.00	3.00
$\sigma`e$	Es(m^2/kN)	Es(m^2/kN)	Es(m^2/kN)
0	0	0	0
100	121.39	941.39	2443.26
200	138.72	1464.36	4387.22
400	158.54	2277.84	7877.89
600	171.41	2949.60	11094.78
800	181.18	3543.23	14145.86
1000	189.13	4084.79	17079.30
1200	195.89	4588.18	19922.26
1400	201.80	5061.88	22692.12
1600	207.05	5511.58	25400.91

It can be observed that the modulus of compressibility increase with stress for a uniform soil formation.

Chapter Five

5. Discussion of the laboratory Test results

The laboratory test results are tabulated and some of them are shown graphically in the previous chapters. Even though the number of test pits taken for this research work is limited, it could be stated that the engineering characteristics and index properties are more or less similar.

5.1 The mineralogy Description

The clay mineralogy composition of the soil in the Town was not studied; however, the soil has low swelling potential and good drainage in the town. So, the mineralogical composition will be probably kaolinite or Illite. But the exact clay mineralogy composition could be determined by x-ray diffraction, thermo-gravimetric, and optical microscopy combined with some form of spectral element identifications.

5.2 Index Properties

The index properties test results, from Table 4.1 the specific gravity of the samples ranges 2.70 – 2.76. This agrees with the specific gravity of clay soils. The Atterberg limit results from Table 4.3 and Fig.4.2 and grain size analysis results, from Table 4.2, Table 5.1, Table 5.2 and Fig. 4.1 show that the soil is fine-grained (clay) soils.

From the laboratory test results and from site observation it can be concluded that the soil is fine-grained having similar properties to clay soil engineering properties.

To classify these fine-grained soil samples taken from the field, plasticity chart was plotted as shown in Fig. 4.3. It indicates that the samples are inorganic silts and clays of high plasticity. According to unified soil classification system these soils are classified as inorganic clay of high plasticity (CH) for test pit one, two, three, four and five at 1.50m depth. Plasticity index versus liquid limit plot shows that the soil falls above the "A- line" from Fig. 4.3. The soil is inorganic silts of high plasticity (MH) for test pit five at 3.0m and six. Plasticity index versus liquid limit plot shows that the soil falls below the "A-line" from Fig. 4.3.

The activity chart in Fig 4.4 shows that the samples have an activity less than 0.75 which lay in the inactive zone and the free swell test results as shown in Table 4.4 vary from 14.5% - 20% which is less than 50%. These indicate that the soil in the Town is not potentially expansive soil.

The coefficient of permeability of the soil under investigation was calculated from consolidation test results. The values range from 0.4×10^{-6}cm/sec to 66.7×10^{-6}cm/sec as shown in Table 4.10 indicating that the soil is naturally impervious soil that will take a long period of time to consolidate.

The unconfined compressive strength of the soil ranges between 63.67kN/m^{2}and 117.8 kN/m^2. The average value of the unconfined compression strength in this research work is 90.44 kN/m^2, which would give Cu 45.22 kN/m^2. The consistency index of the soil ranges between 50% to 93.7% which indicates that the soil is soft to stiff clay soil. The characteristics of such soils can be pressed into or with pressure by thumb to soft and medium soil, respectively as observed in the field.

Table 5.1 Based on Unified Soil Classification System for PT-1, PT-2 and PT-3

	PT-1		PT-2		PT-3	
	1.5m	3.0m	1.5m	3.0m	1.5m	3.0m
% Gravel	0	0	0	0	0	0
% Sand	1.10	0.68	0.68	0.54	0.42	0.80
% Fine(Silt and Clay)	98.90	99.32	99.32	99.46	99.58	99.20
Classification of soil	Clay soil					

Table 5.1 (Continued)

	PT-1		PT-2		PT-3	
	1.5m	3.0m	1.5m	3.0m	1.5m	3.0m
% Gravel	0	0	0	0	0	0
% Sand	0.90	0.64	0.86	0.84	0.56	0.72
% Fine(Silt and Clay)	99.10	99.36	99.14	99.16	99.44	99.28
Classification of soil	Clay soil					

Since the percentage passing sieve No. 200 > 50%, the soil is fine grained. Using Plasticity chart the soil type is CH (PI plots above “A” line). Since the percentage of soil retained on sieve No 200 is very small and the percentage of gravel is 0.00 and that of sand is little. Therefore, the soil description according to USCS is inorganic silts of high plasticity and inorganic clay of high plasticity.

20Table 5.2 Based on AASHTO soil classification system for PT-1, PT-2 and PT-3

	PT-1		PT-2		PT-3	
	1.5m	3.0m	1.5m	3.0m	1.5m	3.0m
% Gravel	0.3	0.2	0.04	0.02	0.02	0.08
% Sand	5.55	0.5	10.54	6.8	1.23	1
% Silt	19.15	17.4	16.03	16.4	27.43	26.54
% Clay	75	81.91	73.4	76.77	71.32	72.35
Classification of soil	Clay soil					

Table 5.2 (Continued)

	PT-4		PT-5		PT-6	
	1.5m	3.0m	1.5m	3.0m	1.5m	3.0m
% Gravel	0.16	0.12	0.16	0.16	0.04	0.06
% Sand	2.62	1.22	0.98	0.73	1.32	1.69
% Silt	31.98	35.07	16	7.85	18.89	13.43
% Clay	65.24	63.59	82.86	91.26	79.74	84.82
Classification of soil	Clay soil					

Based on AASHTO classification system, the percentage of gravel, sand, silt and clay ranges between 0.02-0.03, 0.5-10.54, 7.85-35.07 and 63.59-91.26 respectively based the laboratory test result which is clay soil since more than 35% passing No. 200 sieve (0.075mm). Using the liquidity and plasticity index based on this classification the soil in the town is group A-7 -5 and A-7 - 6 clay soils which is fine grained soil.

5.3 Consolidation Test result

The compression indexes (Cc), recompression index (Cr), coefficient of consolidation (Cv) and pre-consolidation pressure were obtained from the consolidation results. The coefficient of consolidation cv, was taken from the figure of compression dial reading versus time for each loading (Fig. 4.9). From the shape of curve one can observe that the value of cv for the three sites are similar. The values are summarized in Table 4.9. The values of cv lies in the range from 48.59 cm^2/min to 65.36 cm^2/min, the pre-consolidation pressure ranges between 220kPa to 420kPa and the compression index (Cc) ranges from 0.2658 to 0.4110 which indicates the soil in the Town is compressible clay for the pressure intensity above pre-consolidation pressure (Pc). The Recompression index ranges between 0.0288 to 0.0805 which indicate that the consolidation of the soil for the pressure intensity between the overburden pressure (σo) and pre-consolidation pressure (Pc) is very small.

5.4 Comparison of the test results

The laboratory test results of this investigation can be compared with the other researchers' data as shown in Table 5.4.

Table 5.4 Comparison of red clay soil data of present research

	Previous Research	Previous Research	Present research
Soil Type	Red Clay	Red Clay	Red Clay
Location	Addis Ababa	Bahir Dar	Merawi
Clay Content %	58-70	74-82	63.59 – 91.26
Activity	< 0.75	0.56	< 0.75
Clay Minerals	-	Kaolinite	-
LL %	56 - 75	61 - 68	53.04 - 68.25
PL %	14 - 22	9 - 12	23.58 – 33.50
PI %	29 - 47	24 - 31	28.56 – 39.82
G_s	2.66 – 2.77	2.75 – 2.83	2.70 – 2.76
q_u kN/m^3	-	148 - 220	63.67 – 117.81

As shown in Table 5.4, the ranges of values for the present study are close to the results obtained by previous researchers. So that, the soils are more or less in same range that have similar properties.

5.5 Soil map of the Town based on test results

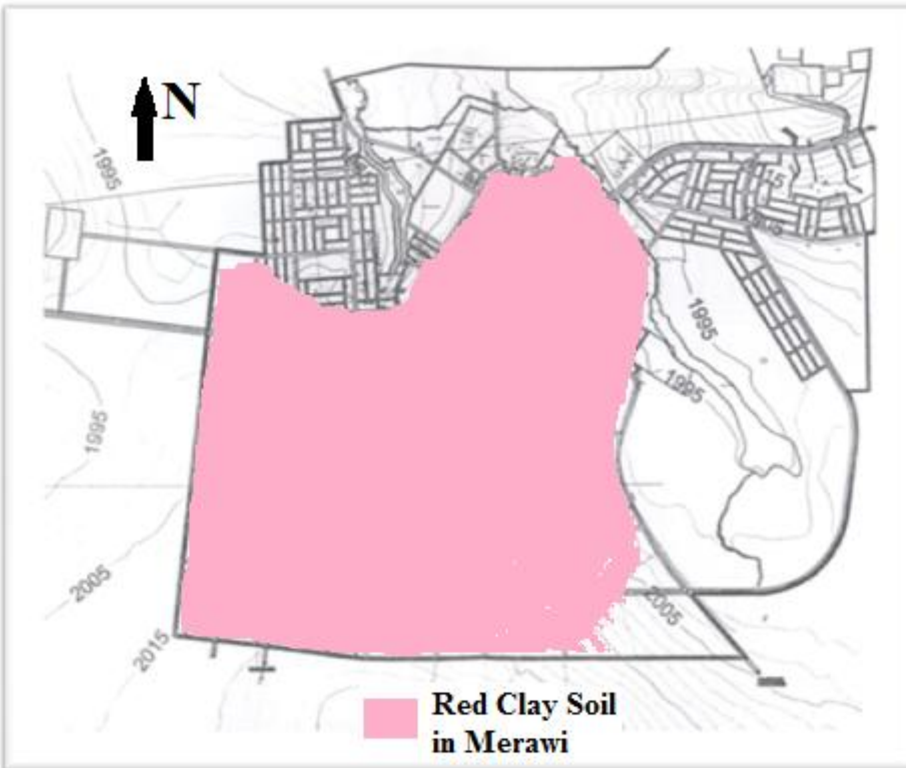

Fig. 5.1 Soil map of the town based on the laboratory test results

Based on the laboratory results obtained and from reconnaissance survey, the soil property is similar in the entire area of the town and it has a property similar to red clay soil engineering properties. But the numbers of test pits were very small and geological properties of the soil are required to prepare the soil map. So, the map is simply to show the property of the soil coverage area based on the results   obtained.

Chapter Six

6. Conclusion and Recommendation

6.1 Conclusion

From the laboratory tests results and tests which were done for this research work, the following conclusion can be drawn.

1. From the laboratory test performed, it can be observed that the soils in Merawi town are not expansive. No significant variations of engineering properties within the investigated depths as well as in different pits which were found in the research work.
2. The values of specific gravity ranges from 2.70 to 2.76, liquid limit ranges 52.3 to 63.8%, plastic limit ranges 23.6% to 33.4%, plasticity index ranges 22.6% to 39.4% and clay fraction ranges 63.59% to 91.26%. These values were within the same ranges of the red soils found in other parts of the country.
3. The coefficient of permeability values ranges 0.4×10^{-6}cm/sec to 66.7×10^{-6}cm/sec which shows that the soil is naturally impervious clay soil that will take a long period of time to consolidate.
4. The shear strength parameters which were obtained from unconfined compressive strength give an average value of Cu, 45.22kN/m^2 and the consistency index of the soil ranges between 50% to 93.7% which indicates that the natural consistency of soil is soft to stiff clay soil.
5. According to unified soil classification system the soil in the Town are classified as inorganic clay of high plasticity (CH) for all test pits. Plasticity index versus liquid limit plot falls above the "A- line".

6. The values of cv lies in the range from 48.59 cm^2/min to 65.36 cm^2/min, the pre-consolidation pressure ranges between 220kPa to 420kPa and the values of the compression indexes range from 0.2442 to 0.4119 which indicates the soil in the town is compressible clay soil for the pressure intensity above pre-consolidation pressure. The values of the recompression index range between 0.0288 and 0.0805 which indicate that the consolidation of the soil for the pressure intensity between the overburden pressure and pre-consolidation pressure is very small.

6.2 Recommendation

1. The mineralogical analysis is recommended to determine the mineral content of the soil in the town.
2. It is recommended that the laboratory test result values should be refined with more number of samples.

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Appendix: Laboratory Raw Data

EXPERIMENT-1 -

SPECIFIC GRAVITY TEST OF SOIL

TP: 1

	@ 1.50m			**@3.00m**		
Trail No.	1	2	3	1	2	3
Pycnometer No.	A	B	C	A	B	C
Mass of dry soil (M_s), g	25.00	25.00	25.00	25.00	25.00	25.00
Mass of pycnometer , g	49.60	49.60	49.60	49.60	49.60	49.60
Mass of pycnometer + Soil , g	74.60	74.60	74.60	74.60	74.60	74.60
Mass of pycnometer + water + Soil (M_1), g	164.80	165.00	164.60	165.00	164.80	164.90
Test temperature, ^oC, Tf	20.00	19.50	19.50	19.50	19.50	19.00
Mass of pycnometer + water (M_2), g	149.00	149.00	149.00	149.00	149.00	149.00
Test temperature, ^oC, Ti	18.00	18.00	18.00	18.00	18.00	18.00

TP: 2

	@ 1.50m			**@3.00m**		
Trail No.	1	2	3	1	2	3
Pycnometer No.	A	B	C	A	B	C
Mass of dry soil (M_s), g	25.00	25.00	25.00	25.00	25.00	25.00
Mass of pycnometer , g	49.60	49.60	49.60	49.60	49.60	49.60
Mass of pycnometer + Soil , g	74.60	74.60	74.60	74.60	74.60	74.60
Mass of pycnometer + water + Soil (M_1), g	165.00	164.80	164.20	164.90	164.90	164.70
Test temperature, ^oC, Tf	21.50	21.50	21.50	21.50	19.00	21.50
Mass of pycnometer + water (M_2), g	149.00	149.00	149.00	149.00	149.00	149.00
Test temperature, ^oC, Ti	19	19	19	17	17	17

TP: 3

	@ 1.50m			@3.00m		
Trail No.	1	2	3	1	2	3
Pycnometer No.	A	B	C	A	B	C
Mass of dry soil (M_s), g	25.00	25.00	25.00	25.00	25.00	25.00
Mass of pycnometer , g	49.60	49.60	49.60	49.60	49.60	49.60
Mass of pycnometer + Soil , g	74.60	74.60	74.60	74.60	74.60	74.60
Mass of pycnometer + water + Soil (M_1), g	164.90	164.90	164.90	164.70	164.70	164.90
Test temperature, ^oC, Tf	19.50	19.50	19.50	20.50	19.00	19.00
Mass of pycnometer + water (M_2), g	149.00	149.00	149.00	149.00	149.00	149.00
Test temperature, ^oC, Ti	18.00	18.00	19.50	17.00	17.00	17.00

TP: 4

	@ 1.50m			@3.00m		
Trail No.	1	2	3	1	2	3
Pycnometer No.	A	B	C	A	B	C
Mass of dry soil (M_s), g	25.00	25.00	25.00	25.00	25.00	25.00
Mass of pycnometer , g	49.60	49.60	49.60	49.60	49.60	49.60
Mass of pycnometer + Soil , g	74.60	74.60	74.60	74.60	74.60	74.60
Mass of pycnometer + water + Soil (M_1), g	164.80	164.90	164.70	164.70	164.90	164.90
Test temperature, ^oC, Tf	20. 50	20.50	20.50	20.50	19.50	20.25
Mass of pycnometer + water (M_2), g	149.00	149.00	149.00	149.00	149.00	149.00
Test temperature, ^oC, Ti	18.50	18.50	18.50	17.50	17.50	17.50

TP: 5

	@ 1.50m			@3.00m		
Trail No.	1	2	3	1	2	3
Pycnometer No.	A	B	C	A	B	C
Mass of dry soil (M_s), g	25.00	25.00	25.00	25.00	25.00	25.00
Mass of pycnometer , g	49.60	49.60	49.60	49.60	49.60	49.60
Mass of pycnometer + Soil , g	74.60	74.60	74.60	74.60	74.60	74.60
Mass of pycnometer + water + Soil (M_1), g	164.80	164.90	164.60	164.90	164.90	164.90
Test temperature, ^oC, Tf	20.00	19.50	19.50	20.00	19.50	19.00
Mass of pycnometer + water (M_2), g	149.00	149.00	149.00	149.00	149.00	149.00
Test temperature, ^oC, Ti	18.00	18.00	18.50	17.50	17.50	17.50

TP: 6

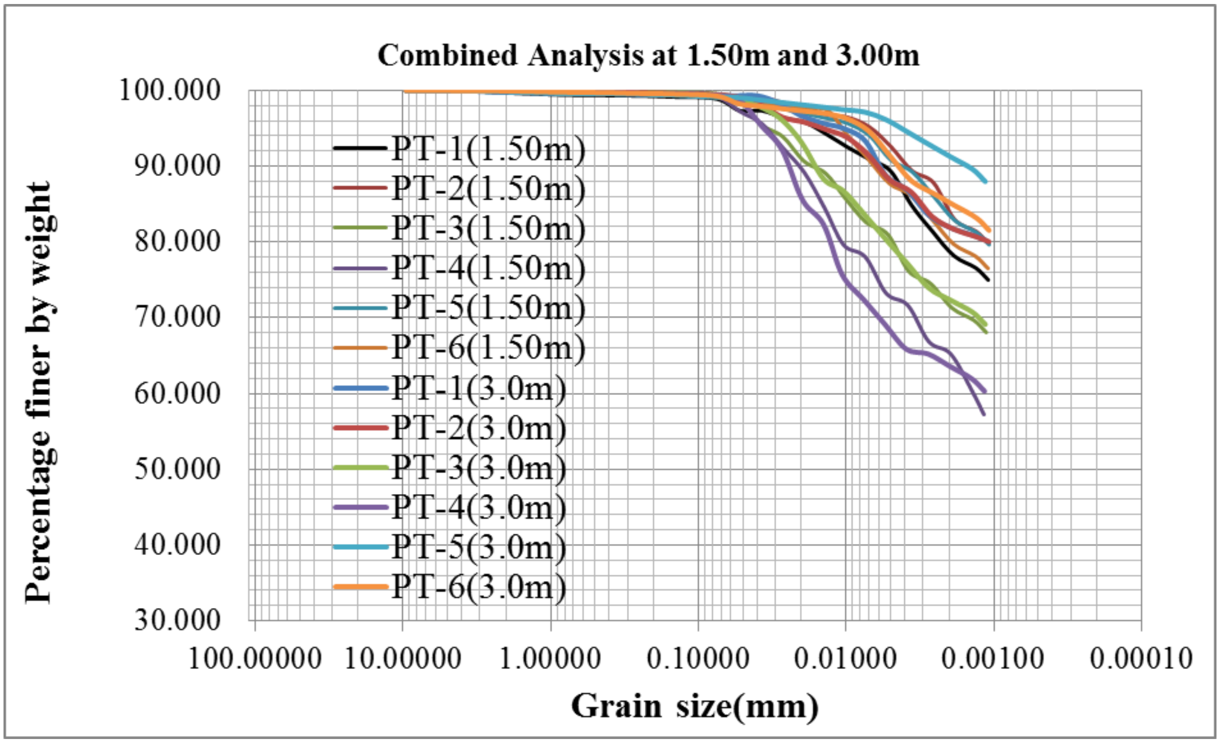
	@ 1.50m			@3.00m		
Trail No.	1	2	3	1	2	3
Pycnometer No.	A	B	C	A	B	C
Mass of dry soil (M_s), g	25.00	25.00	25.00	25.00	25.00	25.00
Mass of pycnometer , g	49.60	49.60	49.60	49.60	49.60	49.60
Mass of pycnometer + Soil , g	74.60	74.60	74.60	74.60	74.60	74.60
Mass of pycnometer + water + Soil (M_1), g	164.60	164.90	164.70	164.70	164.70	164.90
Test temperature, ^oC, Tf	20.50	20.50	20.50	21.00	19.00	20.00
Mass of pycnometer + water (M_2), g	149.00	149.00	149.00	149.00	149.00	149.00
Test temperature, ^oC, Ti	18.50	18.50	19.00	17.00	17.00	17.00

EXPERIMENT-2 and 3 -

GRAIN ANALYSIS (SIEVE AND HYDROMETER ANALYSIS)

Combined Analysis at 1.50m											
PT-1		PT-2		PT-3		PT-4		PT-5		PT-6	
Grain Size (mm)	Percentage Finer (%)	Grain Size (mm)	Percentage Finer (%)	Grain Size (mm)	Percentage Finer (%)	Grain Size (mm)	Percentage Finer (%)	Grain Size (mm)	Percentage Finer (%)	Grain Size (mm)	Percentage Finer (%)
9.50000	100.000	9.5	100	**9.5**	100	**9.5**	100	**9.5**	100	**9.5**	100
4.75000	100.000	4.75	100	4.75	100	4.75	100	4.75	100	4.75	100
2.00000	99.700	2.000	99.96	2	99.98	2	99.84	2	99.84	2	99.96
0.85000	99.440	0.850	99.88	0.85	99.94	0.85	99.68	0.85	99.68	0.85	99.88
0.42500	99.360	0.425	99.76	0.425	99.90	0.425	99.56	0.425	99.56	0.425	99.76
0.25000	99.300	0.250	99.60	0.25	99.86	0.25	99.44	0.25	99.50	0.25	99.60
0.15000	99.160	0.150	99.52	0.15	99.78	0.15	99.34	0.15	99.38	0.15	99.52
0.07500	98.900	0.075	99.32	0.075	99.58	0.075	99.10	0.075	99.14	0.075	99.32
0.05238	97.341	0.05268	99.0361	0.05220	98.746	0.05206	97.2177	0.05239	98.8566	0.05263	98.636
0.03704	97.341	0.03725	99.0361	0.03756	95.519	0.03713	95.6187	0.03718	98.2167	0.03728	98.3158
0.02633	96.383	0.02643	98.395	0.02679	93.9055	0.02671	92.4208	0.02638	97.5769	0.02641	97.9955
0.01868	95.745	0.01876	97.754	0.01926	90.6785	0.01921	89.2228	0.01872	96.937	0.01871	97.6753
0.01333	94.149	0.01333	96.7925	0.01373	89.065	0.01391	84.4259	0.01328	96.2972	0.01325	97.355
0.00982	92.553	0.00976	96.472	0.01019	85.838	0.01039	79.629	0.00974	95.6573	0.00985	94.1526
0.00700	90.958	0.00695	95.19	0.00731	82.611	0.00740	78.03	0.00694	94.0577	0.00708	90.9501
0.00499	89.362	0.00498	92.626	0.00521	80.9975	0.00535	73.2331	0.00499	90.8585	0.00509	87.7476
0.00361	84.894	0.00358	89.4209	0.00377	76.1571	0.00381	71.6341	0.00356	89.2588	0.00363	86.1464
0.00259	81.383	0.00255	87.8184	0.00268	74.5436	0.00275	66.8372	0.00256	86.0596	0.00261	82.9439
0.00186	78.192	0.00185	83.0108	0.00192	71.3166	0.00196	65.2382	0.00184	82.8604	0.00187	79.7415
0.00133	76.596	0.00132	81.4083	0.00137	69.7031	0.00141	60.4413	0.00131	81.2607	0.00133	78.1402
0.00109	75.000	0.00108	79.8058	0.00113	68.0896	0.00117	57.2433	0.00108	79.6611	0.00110	76.539

Combined Analysis at 3.00m											
PT-1		PT-2		PT-3		PT-4		PT-5		PT-6	
Grain Size (mm)	Percentage Finer (%)	Grain Size (mm)	Percentage Finer (%)	Grain Size (mm)	Percentage Finer (%)	Grain Size (mm)	Percentage Finer (%)	Grain Size (mm)	Percentage Finer (%)	Grain Size (mm)	Percentage Finer (%)
9.5	100	9.5	100	9.5	100	9.5	100	9.5	100	9.5	100
4.75	100	4.75	100	4.75	100	4.75	100	4.75	100	4.75	100
2.000	99.80	2.000	99.98	2	99.92	2	99.88	2	99.84	2	99.94
0.850	99.61	0.850	99.92	0.85	99.72	0.85	99.76	0.85	99.64	0.85	99.80
0.425	99.58	0.425	99.84	0.425	99.64	0.425	99.70	0.425	99.58	0.425	99.72
0.250	99.54	0.250	99.72	0.25	99.58	0.25	99.62	0.25	99.52	0.25	99.62
0.150	99.48	0.150	99.64	0.15	99.52	0.15	99.54	0.15	99.40	0.15	99.54
0.075	99.32	0.075	99.46	0.075	99.20	0.075	99.36	0.075	99.16	0.075	99.28
0.05581	99.3037	0.05275	99.0828	0.05581	98.9164	0.05218	98.6554	0.05581	99.1111	0.05323	98.2485
0.03946	99.3037	0.0375	98.0986	0.03946	97.956	0.03754	95.3779	0.03946	98.784	0.03771	97.921
0.02806	98.2881	0.02675	96.4581	0.02806	96.3555	0.02699	92.1003	0.02806	98.4569	0.02671	97.5935
0.02001	96.65	0.01898	95.8019	0.02001	93.1543	0.01971	85.5451	0.02001	98.1298	0.01892	97.266
0.01422	95.6671	0.01349	94.8177	0.01422	88.3525	0.01415	82.2675	0.01422	97.8027	0.0134	96.9385
0.01042	95.0118	0.0099	93.8334	0.01042	86.7519	0.01064	75.7123	0.01042	97.4756	0.00982	96.2835
0.00743	93.3737	0.00708	91.5368	0.00743	83.5507	0.00763	72.4347	0.00743	97.1485	0.007	94.6461
0.00538	88.7869	0.00509	88.2559	0.00538	80.3496	0.00547	69.1571	0.00538	96.1672	0.00504	91.3711
0.00384	86.8212	0.00363	86.6155	0.00384	77.1484	0.00392	65.8796	0.00384	94.5317	0.00362	88.0962
0.00276	83.5449	0.0026	83.3346	0.00276	73.9472	0.00278	65.224	0.00276	92.8962	0.00258	86.4587
0.00196	81.9068	0.00186	81.6941	0.00196	72.3466	0.00198	63.5852	0.00196	91.2607	0.00184	84.8212
0.00139	80.9239	0.00132	80.7099	0.00139	70.746	0.00141	61.9464	0.00139	89.6252	0.00131	83.1837
0.00114	80.2686	0.00108	80.0537	0.00114	69.1454	0.00116	60.3076	0.00114	87.9897	0.00108	81.5463



Grain size analysis curves

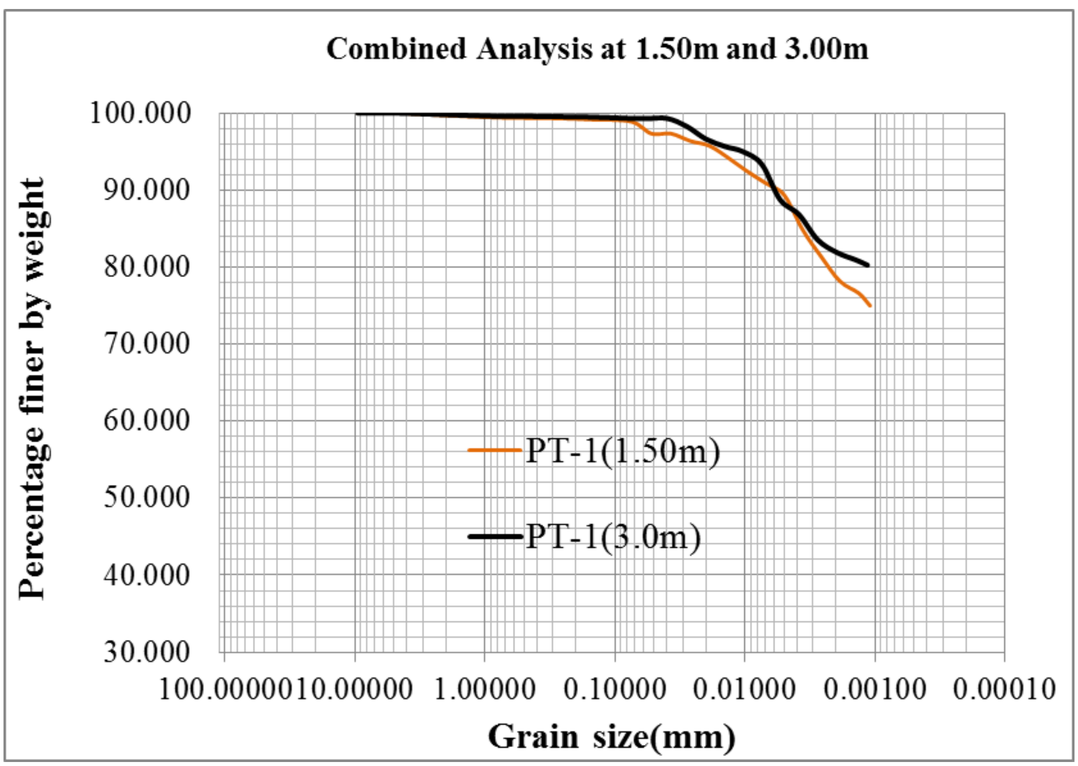


Fig. 1 Grain size cure at PT-1

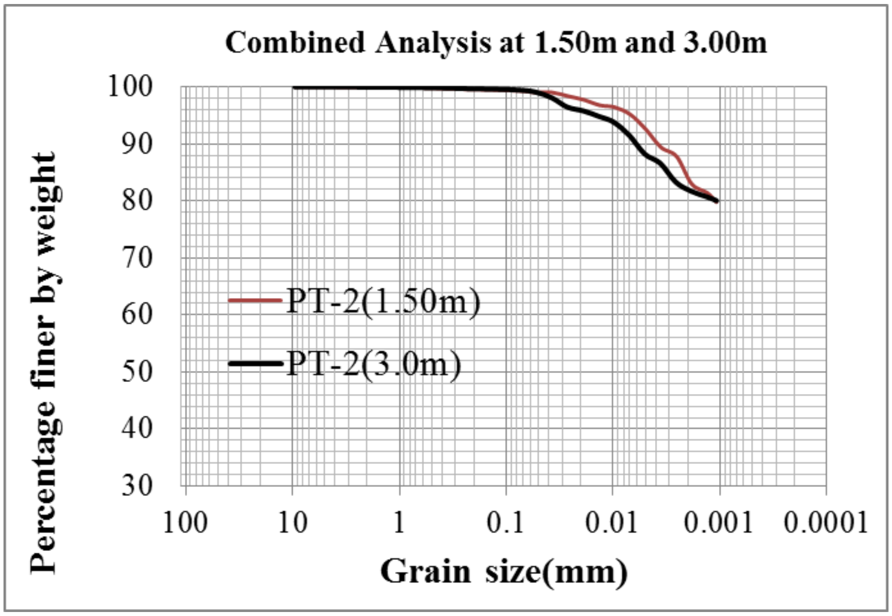


Fig. 2 Grain size cure at PT-2

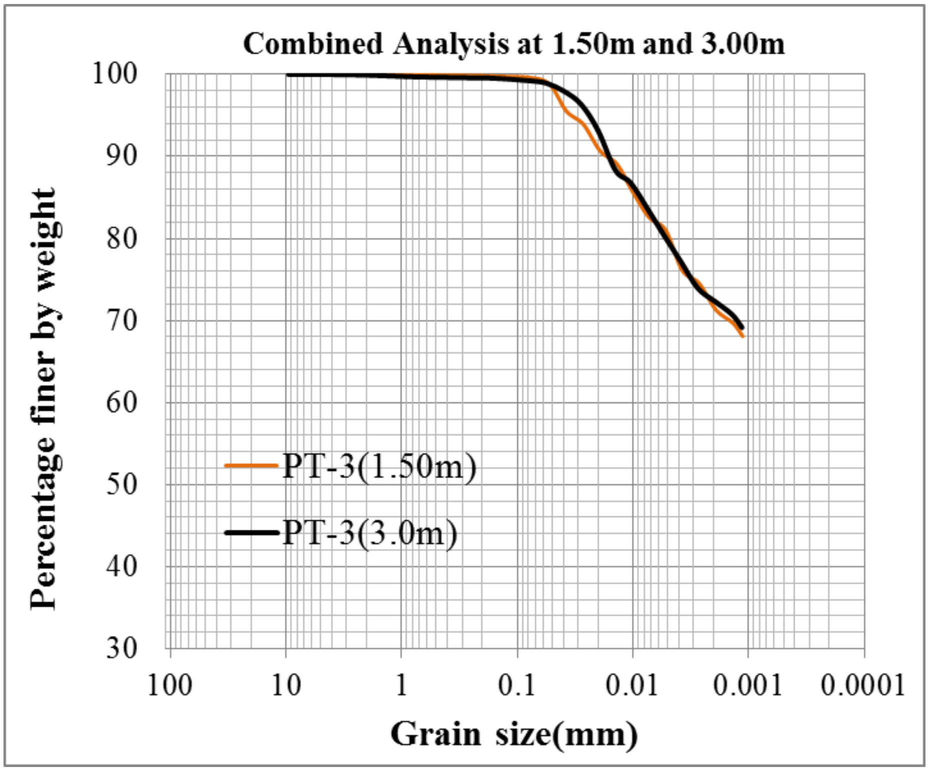


Fig. 3 Grain size cure at PT-3

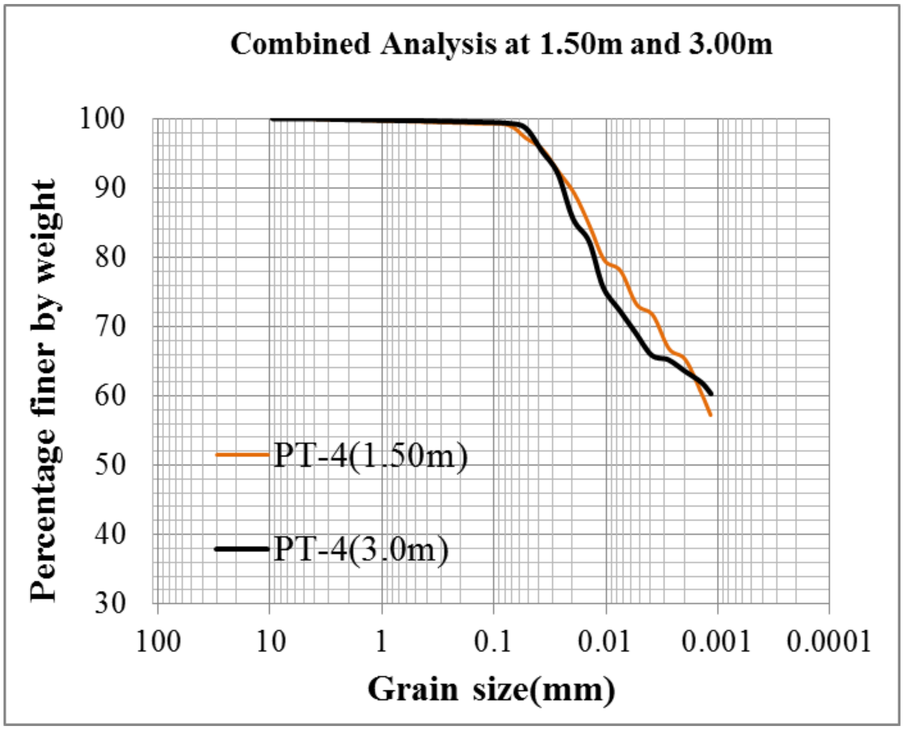


Fig. 4 Grain size cure at PT-4

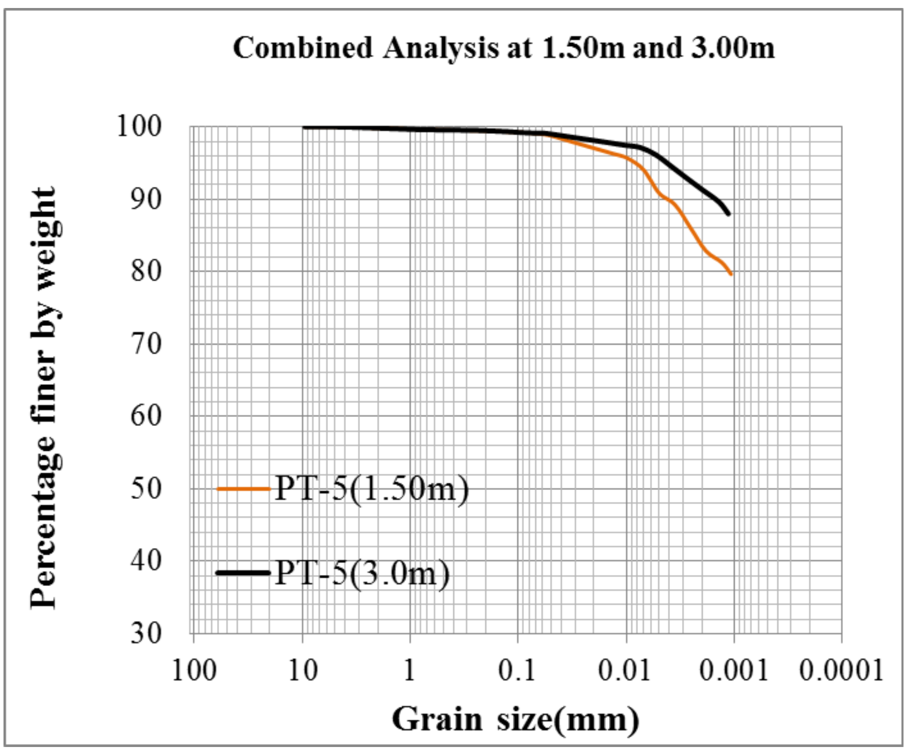


Fig. 5 Grain size cure at PT-5

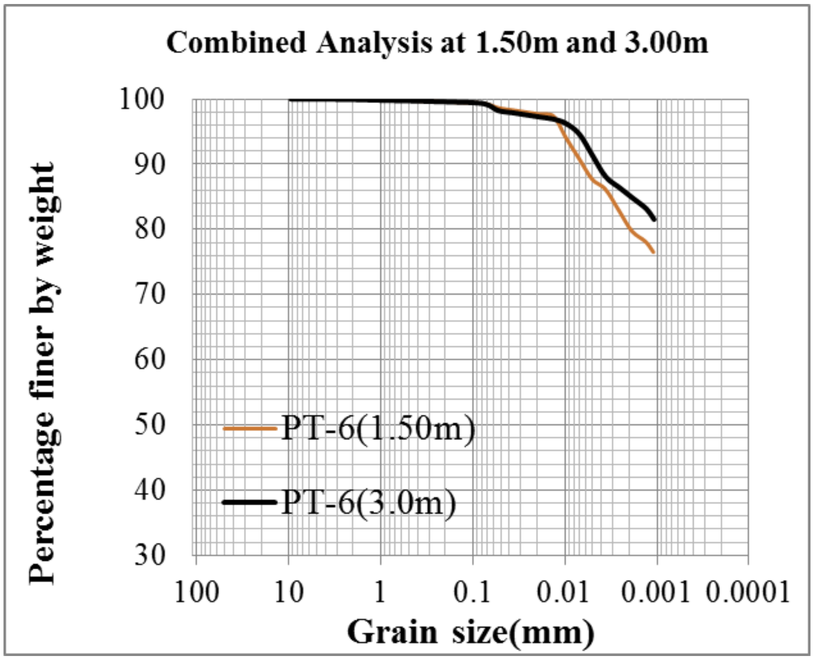
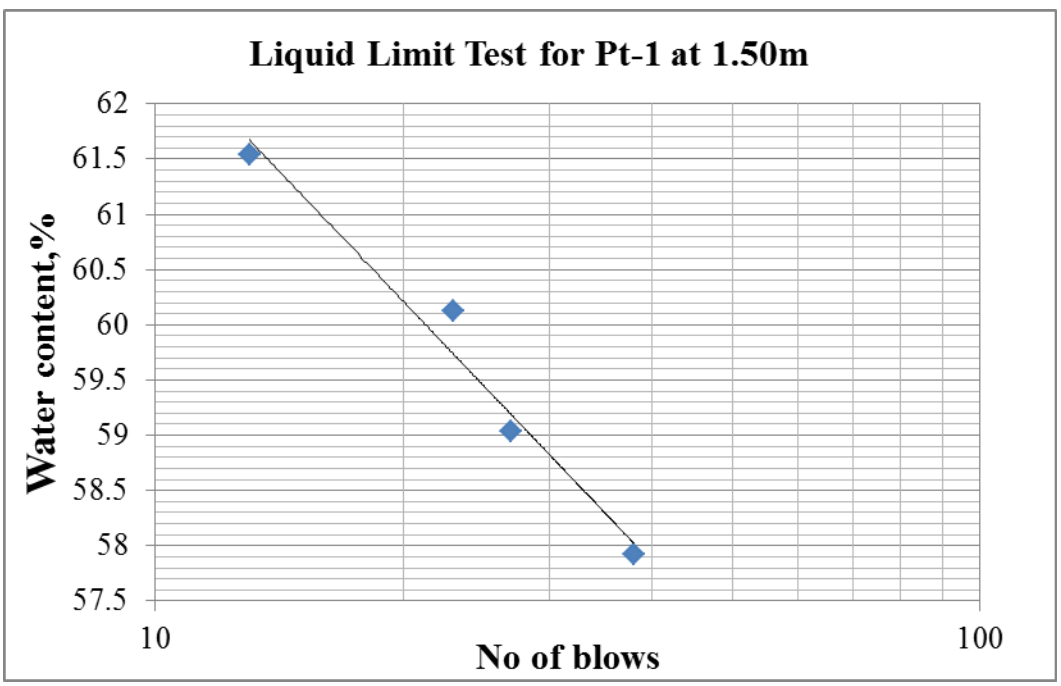


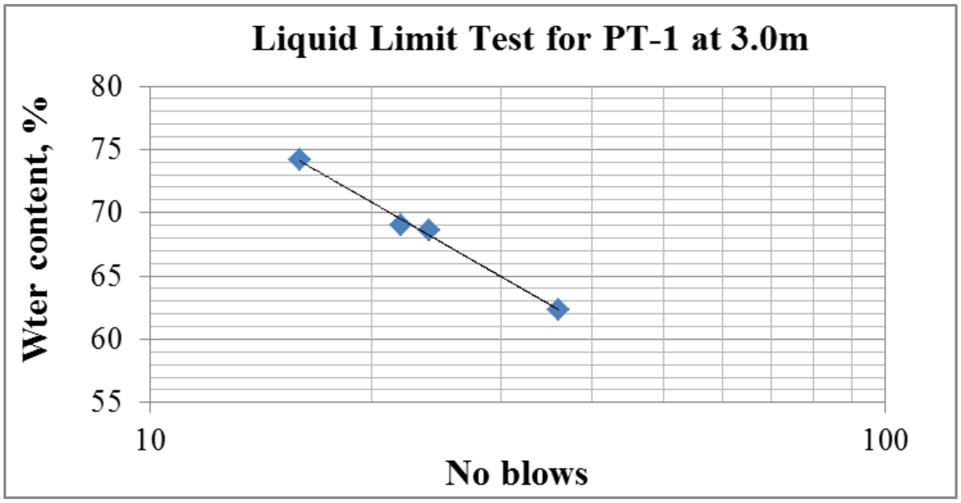
Fig.6 Grain size cure at PT-6

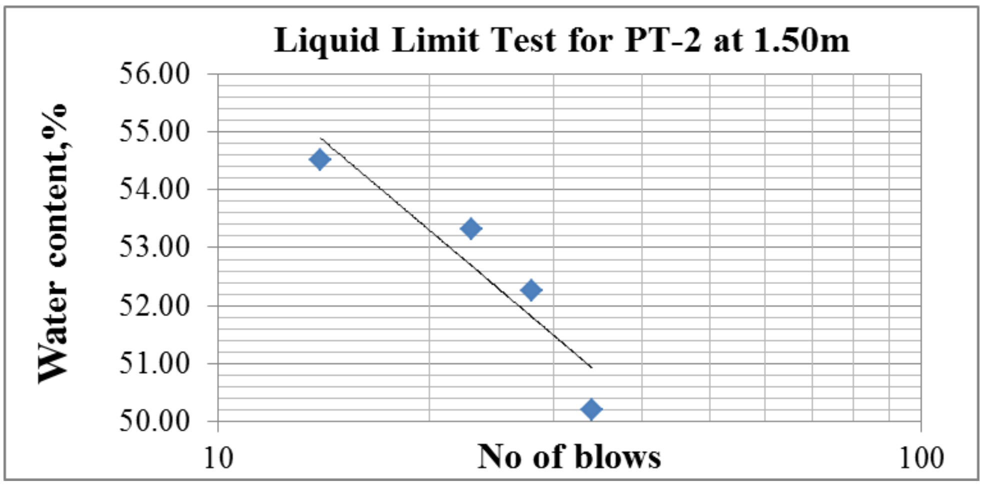
EXPERIMENT-4 -

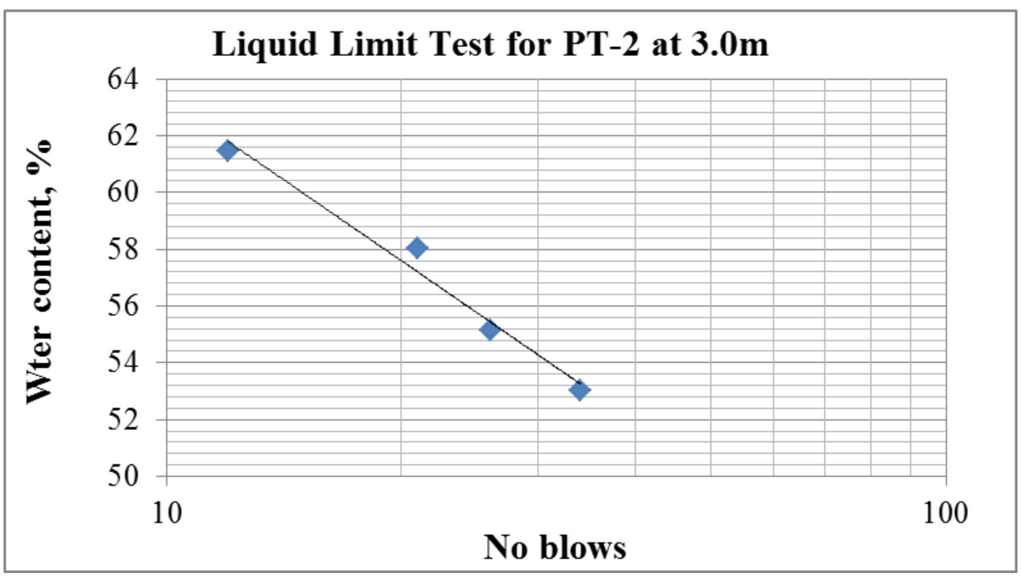
ATTERBERG LIMIT

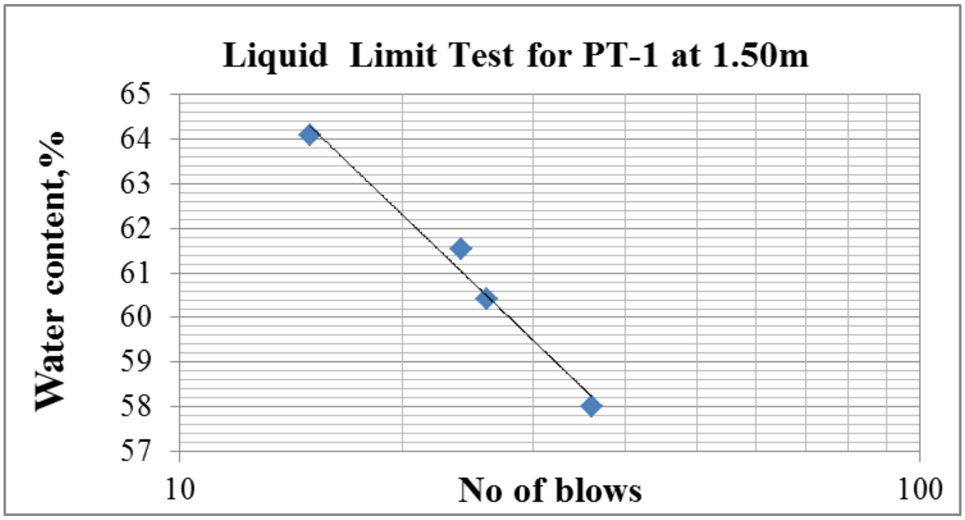
Figures of Atterberg Limit test results

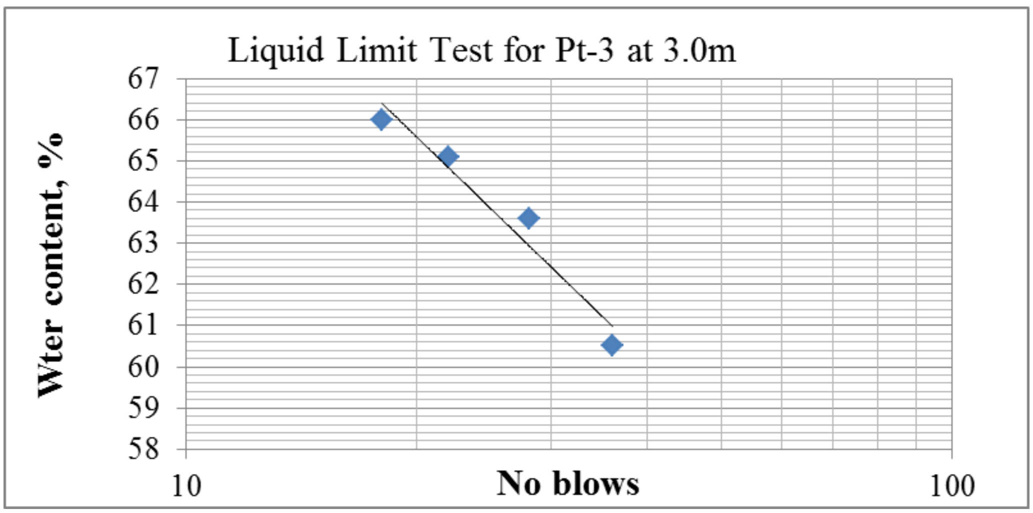


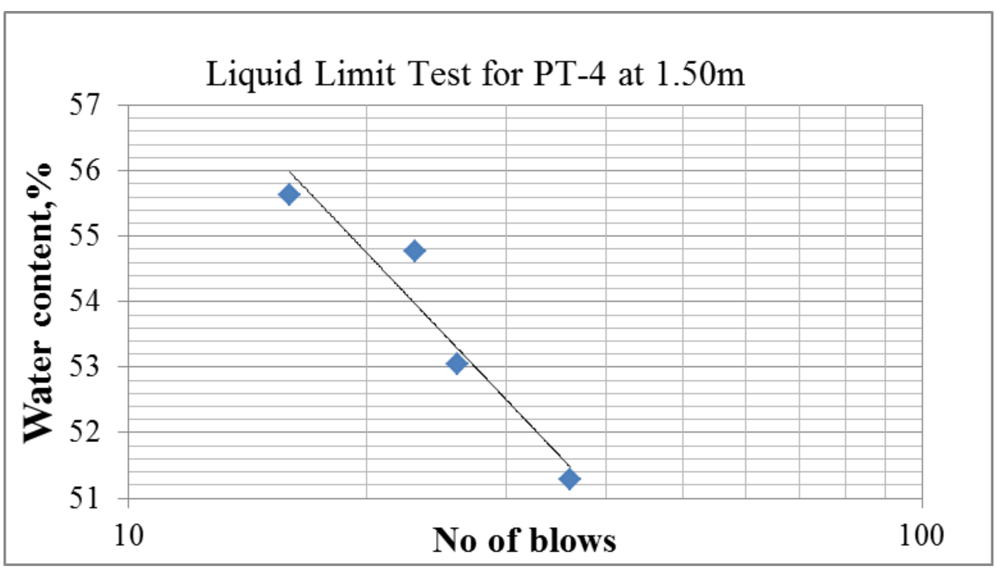

Liquid Limit Test for PT-1 at 3.0m
Wter content, %
80
75
70
65
60
55
10
100
No blows

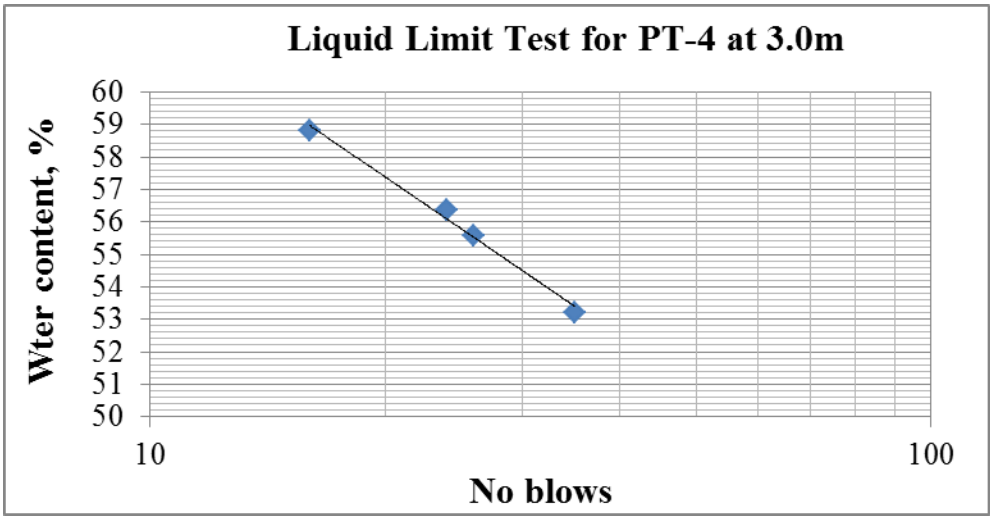

Liquid Limit Test for PT-2 at 1.50m
Water content, %
56.00
55.00
54.00
53.00
52.00
51.00
50.00
10
100
No of blows

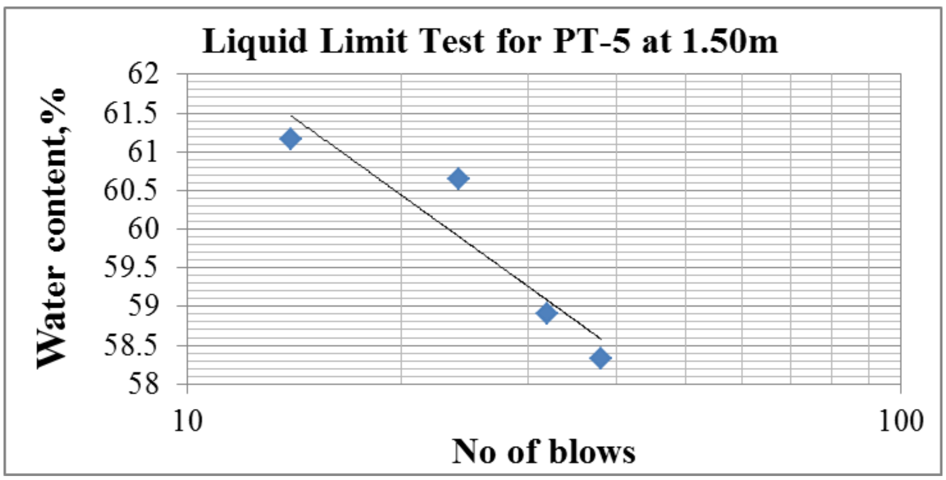

Liquid Limit Test for PT-2 at 3.0m
Wter content, %
64
62
60
58
56
54
52
50
10
100
No blows

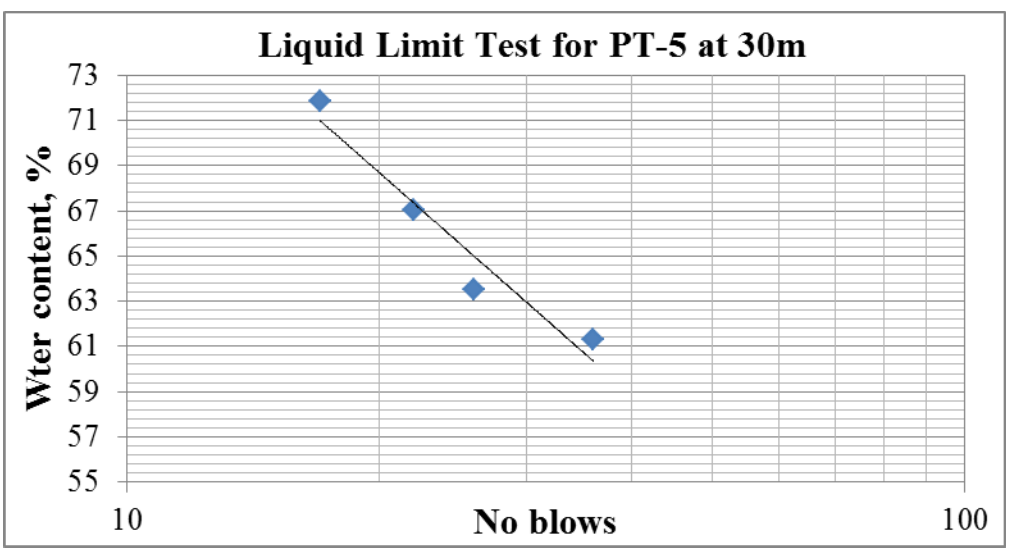

Liquid Limit Test for PT-1 at 1.50m
Water content,%
No of blows
65
64
63
62
61
60
59
58
57
10
100

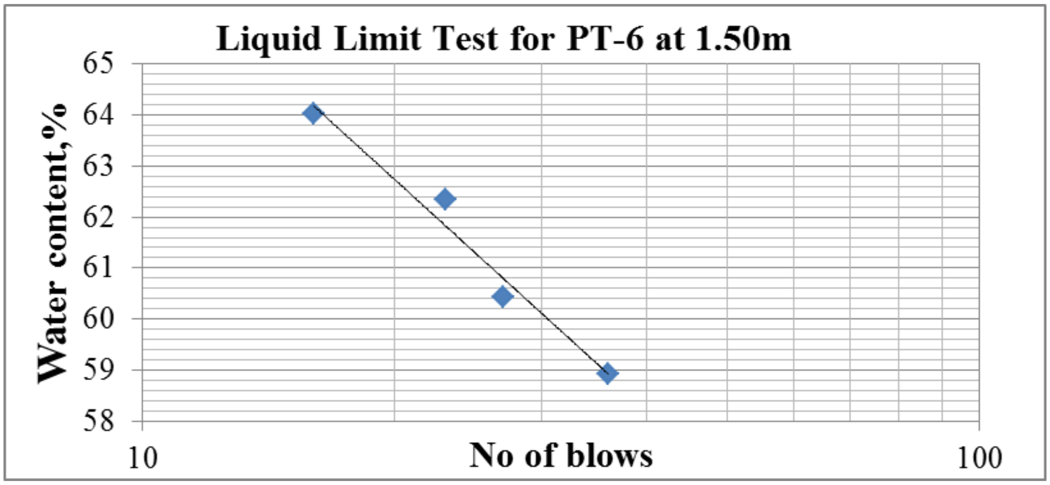

Liquid Limit Test for Pt-3 at 3.0m
Wter content, %
No blows
67
66
65
64
63
62
61
60
59
58
10
100

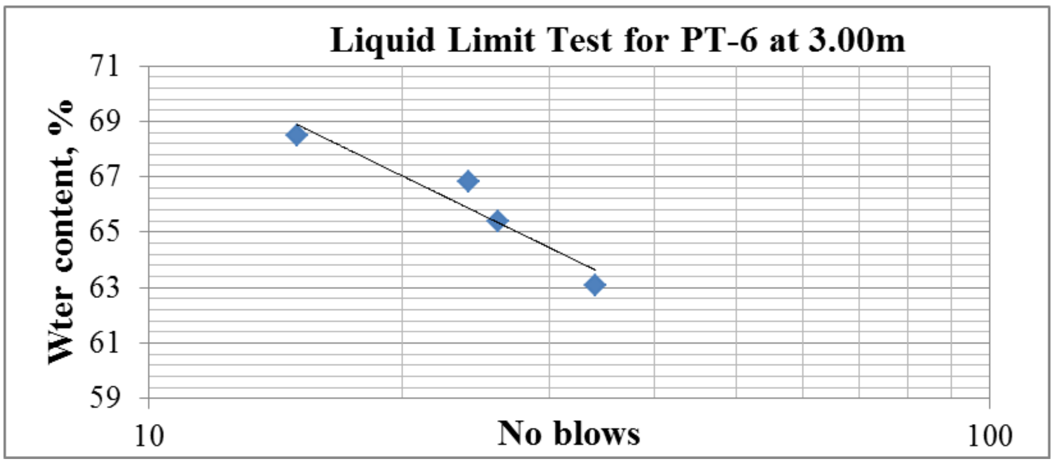

Liquid Limit Test for PT-4 at 1.50m
Water content,%
No of blows
57
56
55
54
53
52
51
10
100


Liquid Limit Test for PT-4 at 3.0m
Wter content, %
60
59
58
57
56
55
54
53
52
51
50
10
100
No blows


Liquid Limit Test for PT-5 at 1.50m
Water content,%
62
61.5
61
60.5
60
59.5
59
58.5
58
10
100
No of blows


Liquid Limit Test for PT-5 at 30m
Wter content, %
73
71
69
67
65
63
61
59
57
55
10
No blows
100





EXPERIMENT-5 -

FREE SWELL

	TP-1					
	@1.5m			@3.0m		
Trial	1.00	2.00	3.00	1.00	2.00	3.00
Initial Volume, cc	10.00	10.00	10.00	10.00	10.00	10.00
Final Volume, cc	12.00	11.00	12.00	12.00	11.50	12.00
Free swell, %	20.00	10.00	20.00	20.00	15.00	20.00
Average Free swell, %	16.67			18.33		

	TP-2					
	@1.5m			@3.0m		
Trial	1.00	2.00	3.00	1.00	2.00	3.00
Initial Volume, cc	10.00	10.00	10.00	10.00	10.00	10.00
Final Volume, cc	11.00	12.00	11.50	12.00	11.00	12.00
Free swell, %	10.00	20.00	15.00	20.00	10.00	20.00
Average Free swell, %	15.00			16.67		

	TP-3					
	@1.5m			@3.0m		
Trial	1.00	2.00	3.00	1.00	2.00	3.00
Initial Volume, cc	10.00	10.00	10.00	10.00	10.00	10.00
Final Volume, cc	11.50	11.00	11.50	11.00	12.00	11.50
Free swell, %	15.00	10.00	15.00	10.00	20.00	15.00
Average Free swell, %	13.33			15.00		

	TP-4					
	@1.5m			@3.0m		
Trial	1.00	2.00	3.00	1.00	2.00	3.00
Initial Volume, cc	10.00	10.00	10.00	10.00	10.00	10.00
Final Volume, cc	11.50	12.00	12.00	12.00	11.50	12.00
Free swell, %	15.00	20.00	20.00	20.00	15.00	20.00
Average Free swell, %	18.33			18.33		

	TP-5					
	@1.5m			@3.0m		
Trial	1.00	2.00	3.00	1.00	2.00	3.00
Initial Volume, cc	10.00	10.00	10.00	10.00	10.00	10.00
Final Volume, cc	12.00	11.50	12.00	12.00	12.00	12.00
Free swell, %	20.00	15.00	20.00	20.00	20.00	20.00
Average Free swell, %	18.33			20.00		

	TP-6					
	@1.5m			@3.0m		
Trial	1.00	2.00	3.00	1.00	2.00	3.00
Initial Volume, cc	10.00	10.00	10.00	10.00	10.00	10.00
Final Volume, cc	11.00	11.50	11.50	11.50	11.50	11.00
Free swell, %	10.00	15.00	15.00	15.00	15.00	10.00
Average Free swell, %	13.33			13.33		

EXPERIMENT-6 -

NATURAL MOISTURE CONTEN

	Tp-1					
Sample Depth:-	1.5m			3.00m		
Trial No.	1	2	3	1	2	3
Container No	B8	B3	M3	W	B3	M3
Weight of Empty container(g)	13.500	13.500	13.500	23.500	23.500	24.000
Weight of container + wet soil(g)	35.500	47.500	40.500	73.500	80.000	57.000
Weight of container + Oven dry soil(g)	29.500	38.500	33.500	60.000	64.090	48.000
Weight of water(WW) (gm)	6.000	9.000	7.000	13.500	15.910	9.000
Weight of Oven dry soil(WS)gm	16.000	25.000	20.000	36.500	40.590	24.000
Water content (W)(%)	37.500	36.000	35.000	36.986	39.197	37.500
Average water content (W)(%)	36.167			37.894		

	TP-2					
Sample Depth:-	1.5m			3.00m		
Trial No.	1	2	3	1	2	3
Container No	A2	C27	E1	N2	N3	R3
Weight of Empty container(g)	20.700	21.700	22.100	22.300	22.500	22.200
Weight of container wet soil(g)	58.800	70.600	64.400	79.100	74.400	73.700
Weight of container + Oven dry soil(g)	47.800	57.300	52.900	63.600	60.500	59.500
Weight of water(WW) (gm)	11.000	13.300	11.500	15.500	13.900	14.200
Weight of Oven dry soil(WS)gm	27.100	35.600	30.800	41.300	38.000	37.300
Water content (W) (%)	40.590	37.360	37.338	37.530	36.579	38.070
Average water content (W) (%)	38.429			37.393		

	TP-3					
Sample Depth:-	1.5m			3.00m		
Trial No.	1	2	3	1	2	3
Container No	R8	E2	E7	8	42	74
Weight of Empty container(g)	22.000	22.000	22.000	22.000	22.000	22.000
Weight of container wet soil(g)	83.000	89.000	73.000	67.000	76.000	74.000
Weight of container + Oven dry soil(g)	68.000	73.000	60.000	55.000	61.000	60.000
Weight of water(WW) (gm)	15.000	16.000	13.000	12.000	15.000	14.000
Weight of Oven dry soil(WS)gm	46.000	51.000	38.000	33.000	39.000	38.000
water content(W)(%)	32.609	31.373	34.211	36.364	38.462	36.842
Average water content (W)(%)	32.731			37.222		

	Tp-4					
Sample Depth:-	1.5m			3.00m		
Trial No.	1	2	3	1	2	3
Container No	A	D	M3	W	B3	N3
Weight of Empty container(g)	22.30	22.40	22.00	20.10	22.00	21.20
Weight of container wet soil(g)	52.264	63.814	56.587	73.493	76.183	63.431
Weight of container + Oven dry soil(g)	43.850	52.700	47.400	59.000	61.295	51.850
Weight of water(WW) (gm)	8.414	11.114	9.187	14.493	14.888	11.581
Weight of Oven dry soil(WS)gm	21.550	30.300	25.400	38.900	39.295	30.650
Water content (W)(%)	39.045	36.680	36.169	37.258	37.888	37.785
Average water content (W)(%)	37.30			37.64		

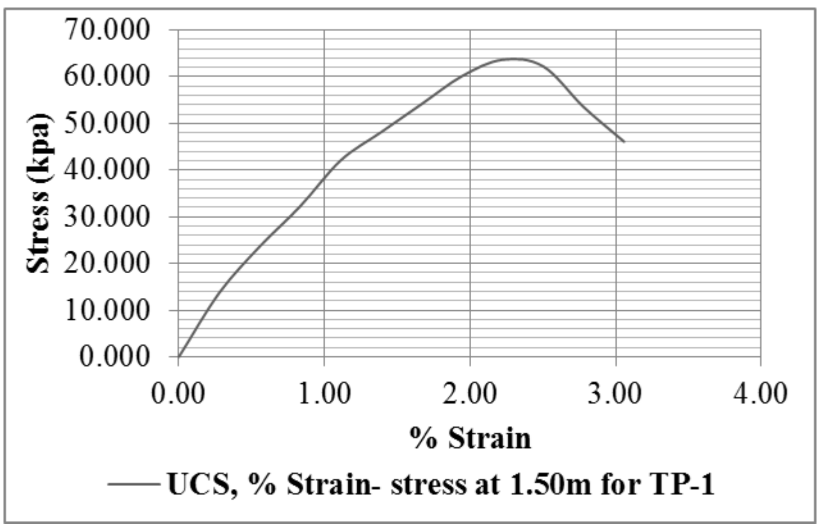
	Tp-5					
Sample Depth:-	1.5m			3.00m		
Trial No.	1	2	3	1	2	3
Container No	K	G	P	W2	W3	V
Weight of Empty container(g)	22.00	22.20	23.00	20.40	21.50	22.00
Weight of container wet soil(g)	63.867	73.001	62.036	67.895	76.747	64.523
Weight of container + Oven dry soil(g)	53.000	60.200	52.000	55.150	61.295	53.000
Weight of water(WW) (gm)	10.867	12.801	10.036	12.745	15.452	11.523
Weight of Oven dry soil(WS)gm	31.000	38.000	29.000	34.750	39.795	31.000
Water content (W) (%)	35.054	33.686	34.605	36.675	38.829	37.171
Average water content (W) (%)	34.45			37.56		

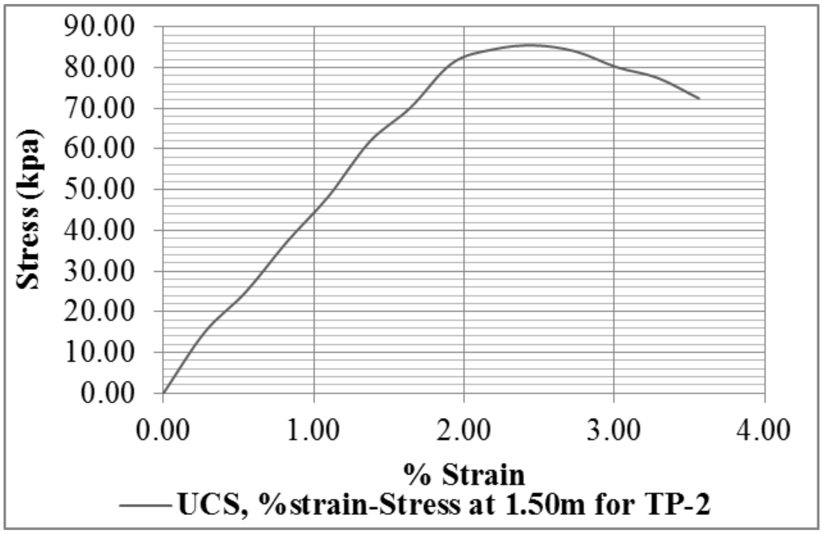
	Tp-6					
Sample Depth:-	1.5m			3.00m		
Trial No.	1	2	3	1	2	3
Container No	B	H	L	F	R	Y
Weight of Empty container(g)	20.40	21.00	22.00	22.20	22.40	23.00
Weight of container wet soil(g)	70.327	79.180	68.706	73.076	75.345	74.752
Weight of container + Oven dry soil(g)	56.950	64.300	56.400	59.350	60.900	60.650
Weight of water(WW) (gm)	13.377	14.880	12.306	13.726	14.445	14.102
Weight of Oven dry soil(WS)gm	36.550	43.300	34.400	37.150	38.500	37.650
Water content (W)(%)	36.600	34.366	35.774	36.947	37.520	37.456
Average water content (W) (%)	35.58			37.31		

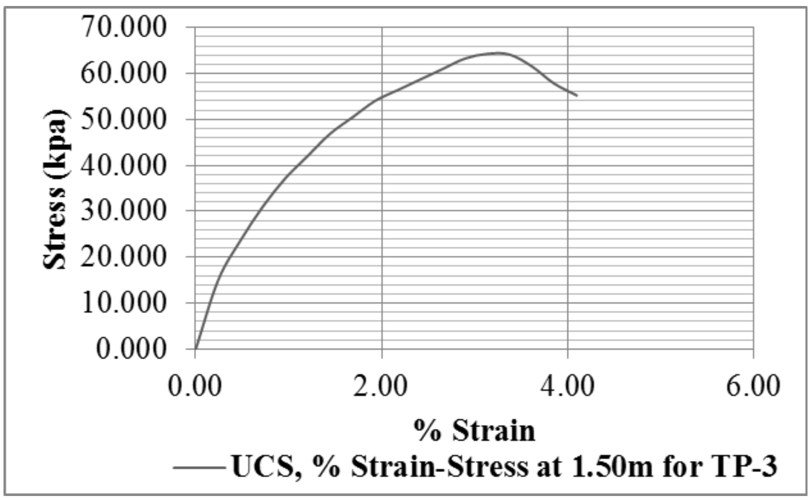
EXPERIMENT-7-

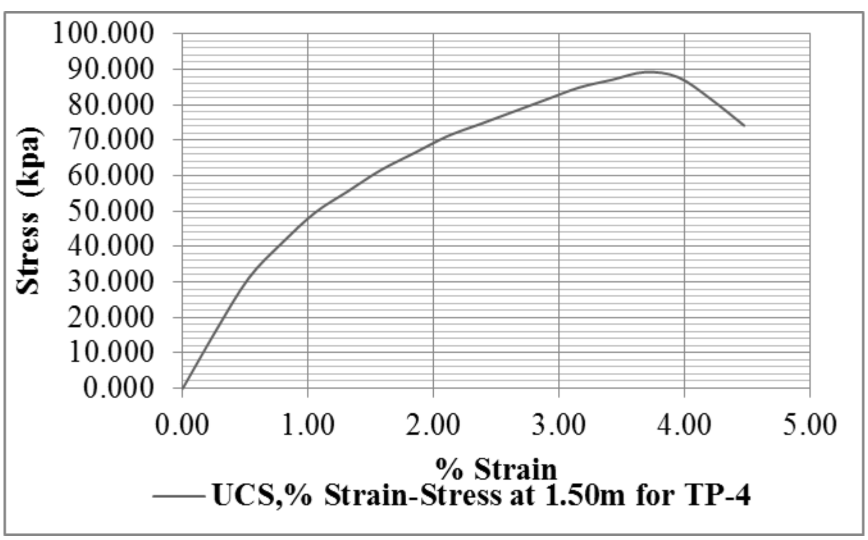
UNCONFIED COMPRESSION TEST

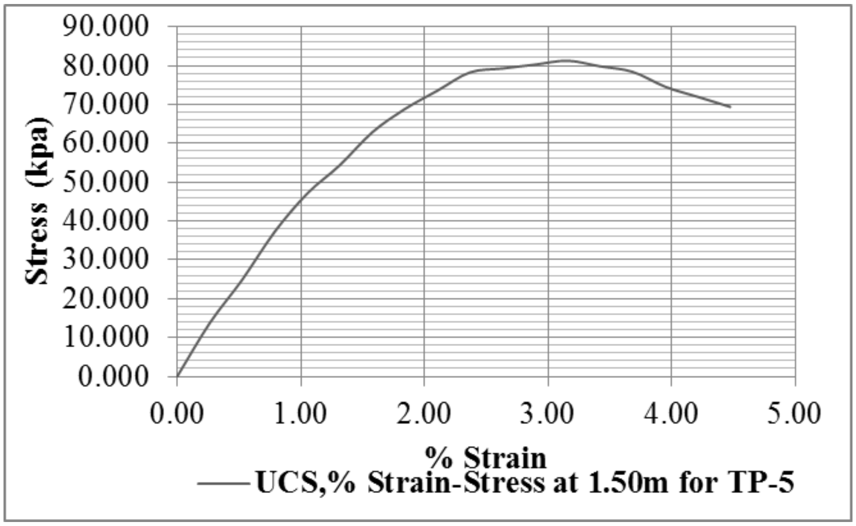
Figures of UCS test result

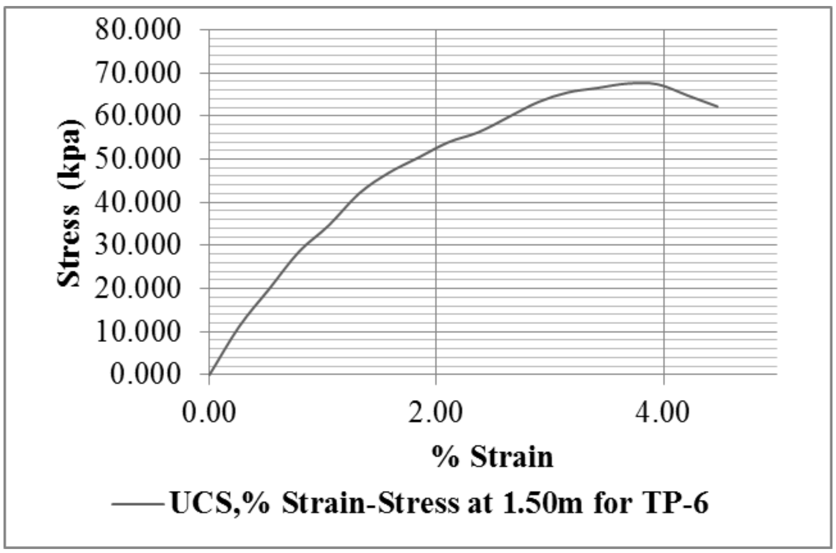


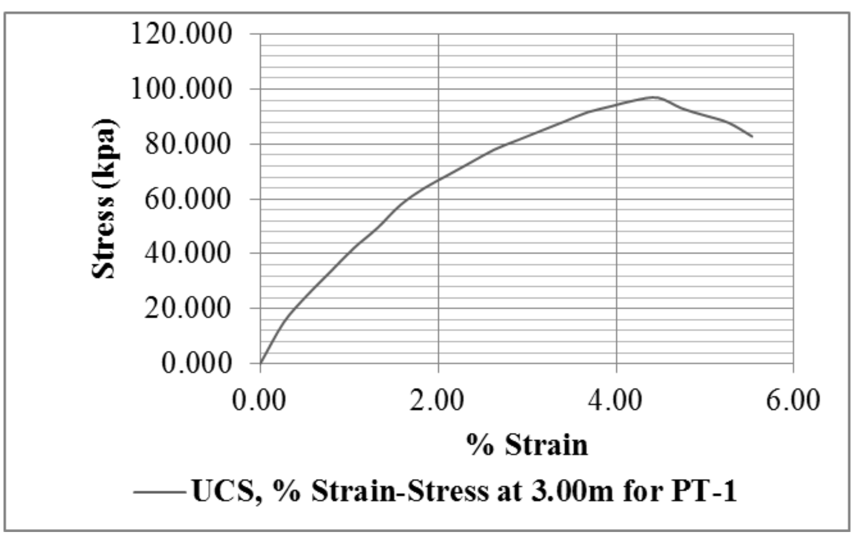

90.00
80.00
70.00
60.00
50.00
40.00
30.00
20.00
10.00
0.00
Stress (kpa)
0.00
1.00
2.00
3.00
4.00
% Strain
UCS, %strain-Stress at 1.50m for TP-2

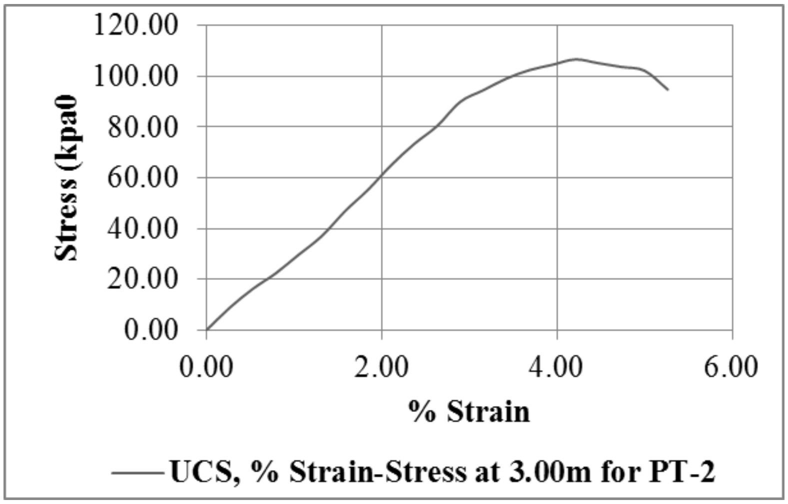

70.000
60.000
50.000
40.000
30.000
20.000
10.000
0.000
Stress (kpa)
0.00
2.00
4.00
6.00
% Strain
UCS, % Strain-Stress at 1.50m for TP-3

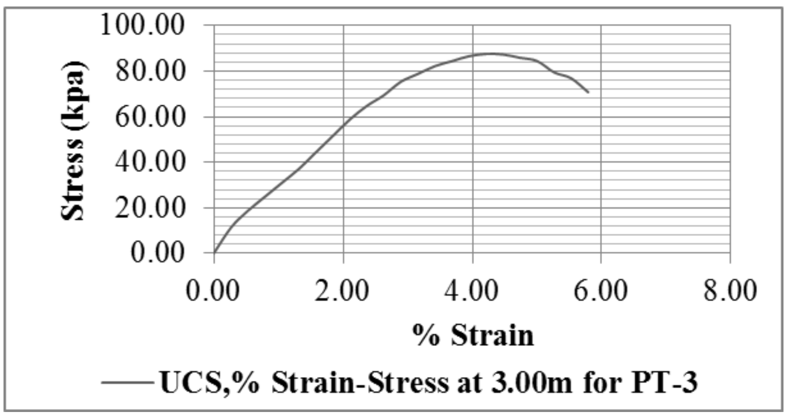

100.000
90.000
80.000
70.000
60.000
50.000
40.000
30.000
20.000
10.000
0.000
Stress (kpa)
0.00
1.00
2.00
3.00
4.00
5.00
% Strain
UCS,% Strain-Stress at 1.50m for TP-4

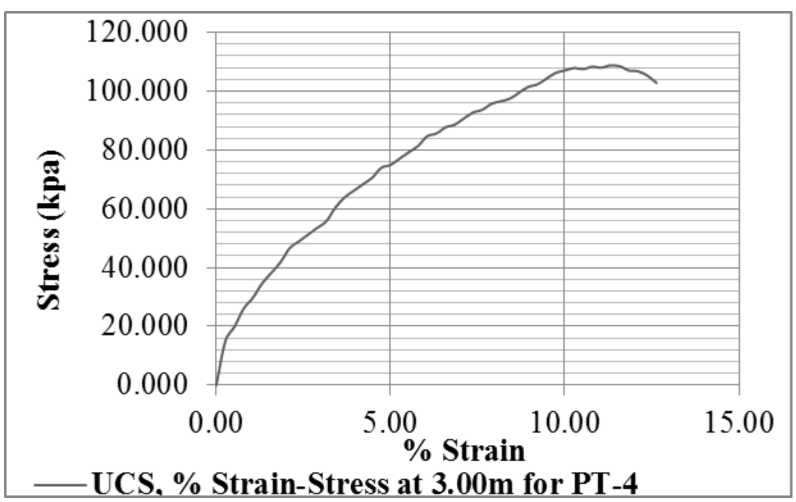

90.000
80.000
70.000
60.000
50.000
40.000
30.000
20.000
10.000
0.000
Stress (kpa)
0.00
1.00
2.00
3.00
4.00
5.00
% Strain
UCS,% Strain-Stress at 1.50m for TP-5

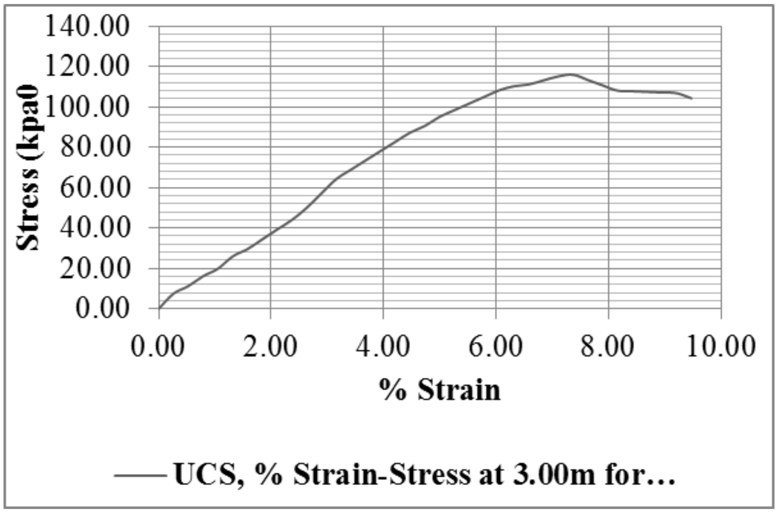

80.000
70.000
60.000
50.000
40.000
30.000
20.000
10.000
0.000
Stress (kpa)
0.00
2.00
4.00
% Strain
UCS,% Strain-Stress at 1.50m for TP-6

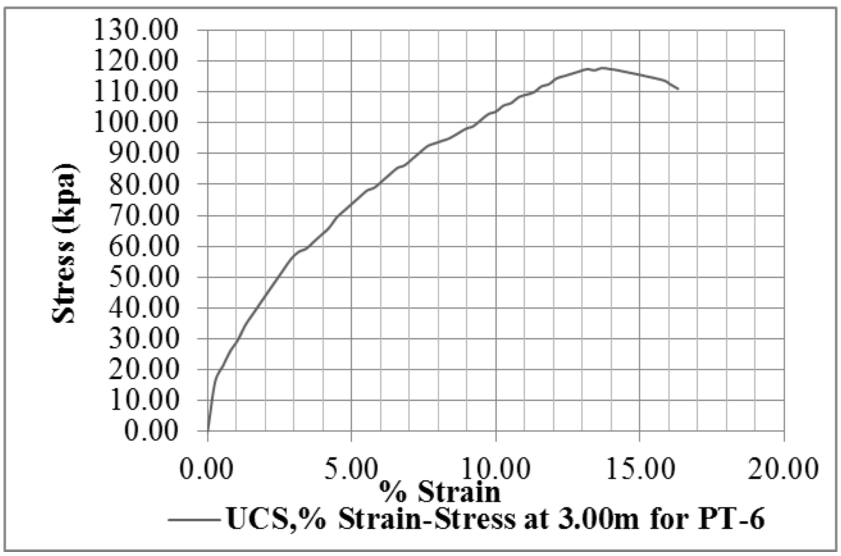

120.000
100.000
80.000
60.000
40.000
20.000
0.000
Stress (kpa)
0.00
2.00
4.00
6.00
% Strain
UCS, % Strain-Stress at 3.00m for PT-1


120.00
100.00
80.00
60.00
40.00
20.00
0.00
Stress (kpa0
0.00
2.00
4.00
6.00
% Strain
UCS, % Strain-Stress at 3.00m for PT-2


100.00
80.00
60.00
40.00
20.00
0.00
Stress (kpa)
0.00
2.00
4.00
6.00
8.00
% Strain
UCS,% Strain-Stress at 3.00m for PT-3


120.000
100.000
80.000
60.000
40.000
20.000
0.000
Stress (kpa)
0.00
5.00
10.00
15.00
% Strain
UCS, % Strain-Stress at 3.00m for PT-4


140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
Stress (kpa0
0.00
2.00
4.00
6.00
8.00
10.00
% Strain
UCS, % Strain-Stress at 3.00m for…


130.00
120.00
110.00
100.00
90.00
80.00
70.00
60.00
50.00
40.00
30.00
20.00
10.00
0.00
Stress (kpa)
0.00
5.00
10.00
15.00
20.00
% Strain
UCS,% Strain-Stress at 3.00m for PT-6

EXPERIMENT-8 -

CONSOLIDATION TEST

Table: Height of Sample at each Loading for TP-1				
Load (kg)	Dial Reading (ΔH)		Height of Sample at each Loading	
1	ΔH_1	0.780	H_1	19.220
2	ΔH_2	0.685	H_2	19.315
4	ΔH_3	0.677	H_3	19.323
8	ΔH_4	0.877	H_4	19.123
16	ΔH_5	0.880	H_5	19.120
32	ΔH_6	0.808	H_6	19.192
16	ΔH_5	-0.042	H_5	15.335
8	ΔH_4	-0.068	H_4	15.403
4	ΔH_3	-0.064	H_3	15.467
2	ΔH_2	-0.054	H_2	15.521
1	ΔH_1	-0.052	H_1	15.573

Calculations of Void Ratio

$$e_o = \frac{v_o G_s \gamma_w}{w_s} - 1 = \frac{39.27 * 2.75 * 1}{45.214} - 1 = 1.371$$

$$\Delta e_1 = \frac{\Delta H_1}{H_o}(1 + e_o) = 0.093, \qquad e_1 = e_o - \Delta e_1 = 1.295$$

$$\Delta e_2 = \frac{\Delta H_2}{H_1}(1 + e_o) = 0.085, \qquad e_2 = e_1 - \Delta e_2 = 1.210$$

$$\Delta e_3 = \frac{\Delta H_3}{H_2}(1 + e_o) = 0.084, \qquad e_3 = e_2 - \Delta e_3 = 1.126$$

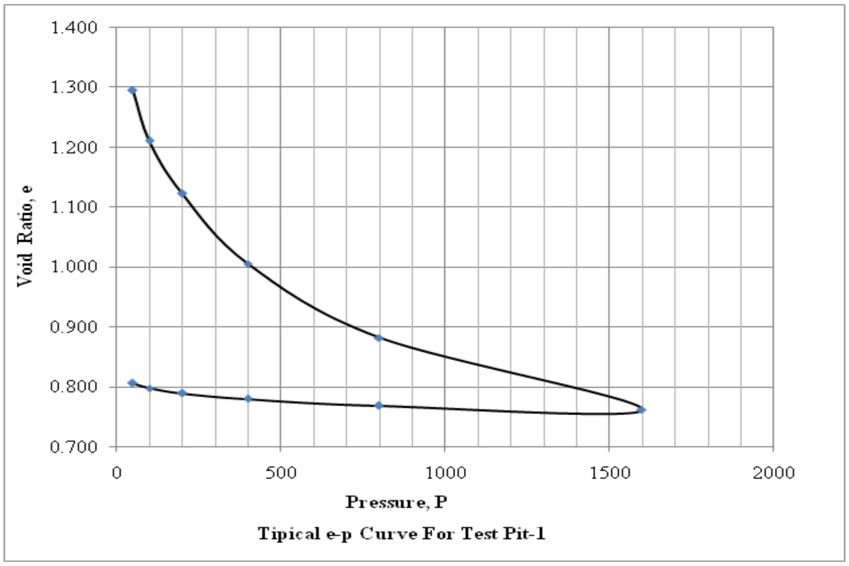
$$\Delta e_4 = \frac{\Delta H_{o4}}{H_3}(1 + e_o) = 0.108, \qquad e_4 = e_3 - \Delta e_4 = 1.018$$

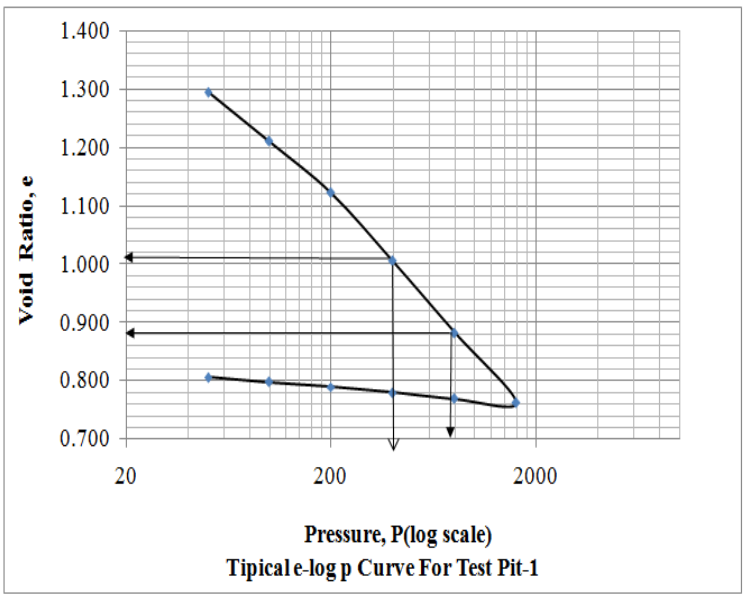
$$\Delta e_5 = \frac{\Delta H_5}{H_4}(1 + e_o) = 0.110, \qquad e_5 = e_4 - \Delta e_5 = 0.908$$

$$\Delta e_6 = \frac{\Delta H_6}{H_5}(1 + e_o) = 0.101, \qquad e_6 = e_5 - \Delta e_6 = 0.807$$

Table: void ratio of Sample at each Loading for TP-1			
Δe_1	0.093	e_1	1.295
Δe_2	0.085	e_2	1.210
Δe_3	0.084	e_3	1.126
Δe_4	0.108	e_4	1.018
Δe_5	0.110	e_5	0.908
Δe_6	0.101	e_6	0.807
Δe_5	-0.007	e_5	0.769
Δe_4	-0.011	e_4	0.779
Δe_3	-0.010	e_3	0.789
Δe_2	-0.08	e_2	0.797
Δe_1	-0.008	e_1	0.805

Table: 10.5 Pressure void ratio Relationship TP-1	
P	e
50	1.295
100	1.210
200	1.126
400	1.018
800	0.908
1600	0.807
800	0.769
400	0.779
200	0.789
100	0.797
50	0.805



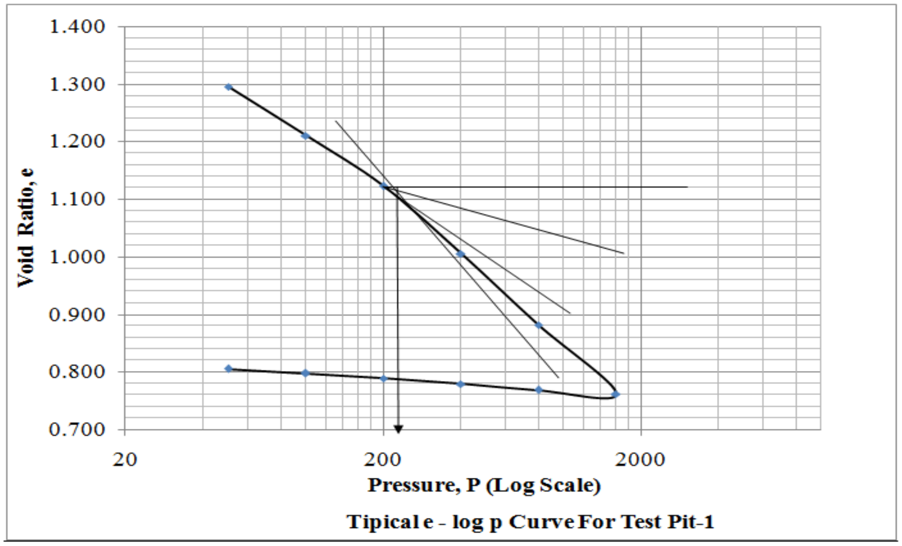


Compression Idex, $C_c = \dfrac{e_{1-}e_2}{Log\ p_2 - \text{Log}\ p_1} = \dfrac{1.006 - 0.882}{\text{Log}800 - \text{Log}400} = 0.4119$

Expanssion Idex, $C_e = \dfrac{e_{1-}e_2}{Log\ p_2 - \text{Log}\ p_1} = \dfrac{0.797 - 0.779}{\text{Log}400 - \text{Log}100} = 0.0299$

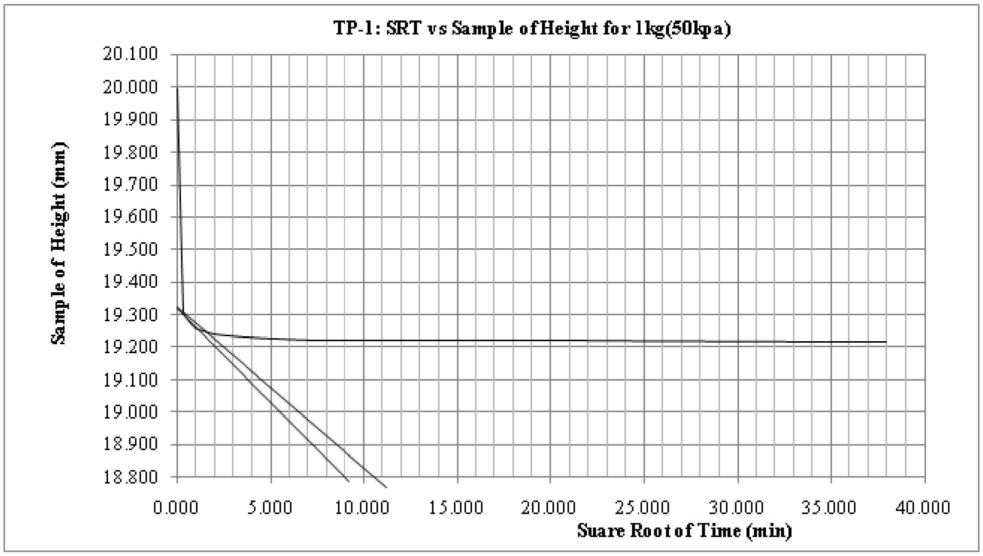
Recompression Index, $C_r = \dfrac{e_1 - e_2}{LogP_2 - LogP_1} = \dfrac{1.062 - 0.988}{Log420 - Log50.72} = 0.0020$

Pre-consolidation pressure, pc



Coefficient Consolidation (C_v) Determination

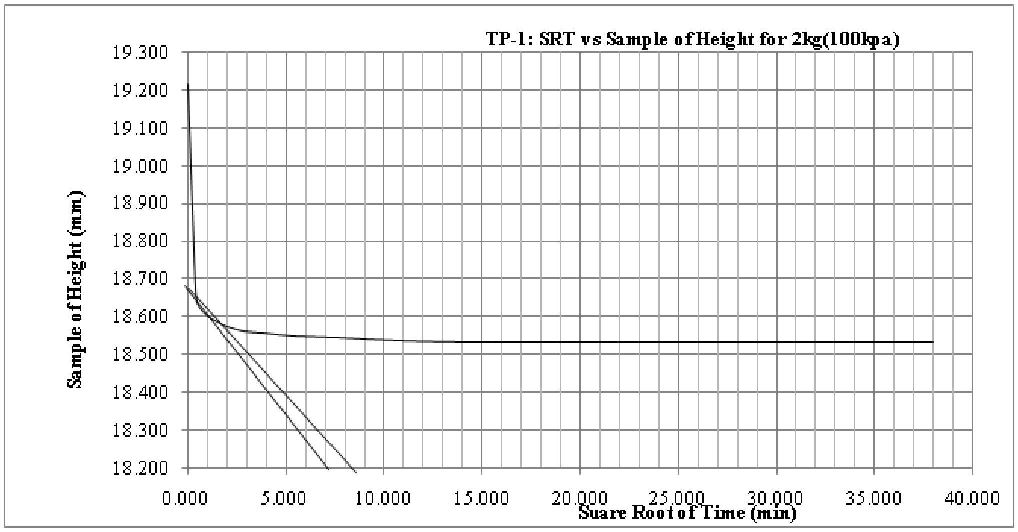
Square Root Time Fitting Method



$$C_v = \frac{(0.848)H_{dr}^2}{t_{90}}$$

$H_{dr} = 19.609mm, \quad \sqrt{t_{90}} = 1.8, \quad t_{90} = 3.24$

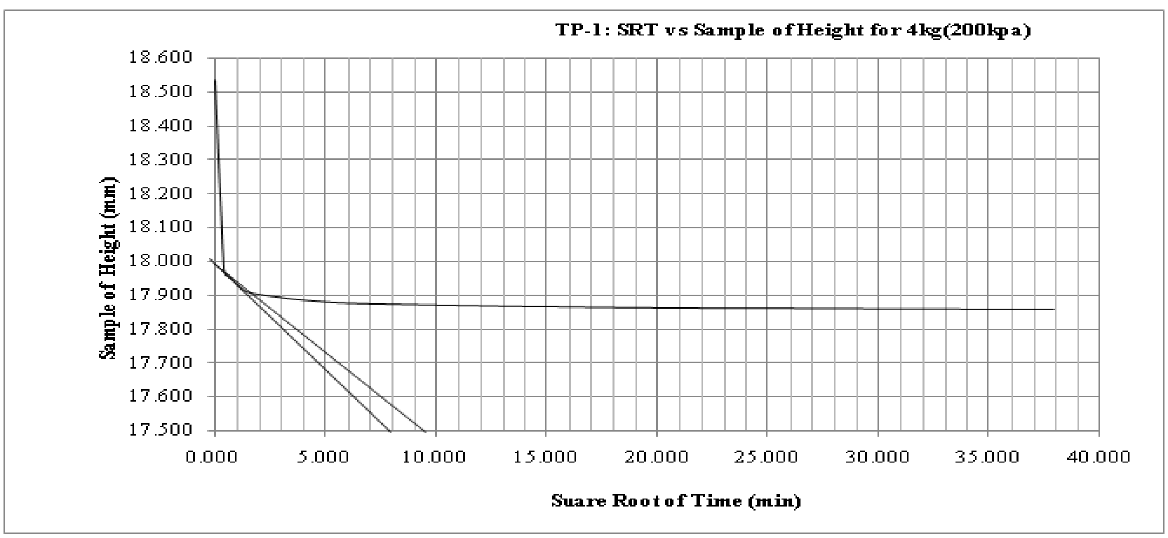
$$C_v = \frac{(0.848)19.609^2}{3.24} = 100.64$$



$$C_v = \frac{(0.848)H_{dr}^2}{t_{90}}$$

$H_{dr} = 18.8765mm, \quad \sqrt{t_{90}} = 2.0, \quad t_{90} = 4.0$

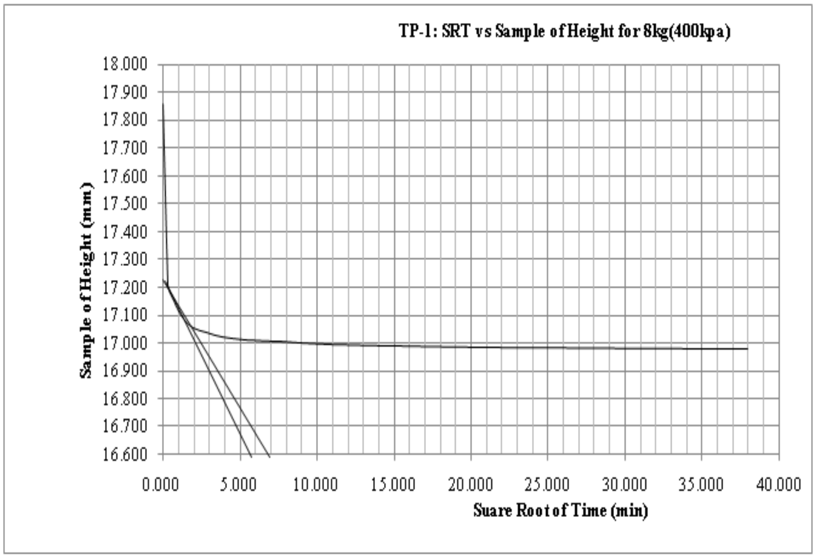
$$C_v = \frac{(0.848)18.8765^2}{4} = 75.54$$



$$C_v = \frac{(0.848)H_{dr}^2}{t_{90}}$$

$H_{dr} = 18.1955mm, \quad \sqrt{t_{90}} = 1.8, \quad t_{90} = 3.24$

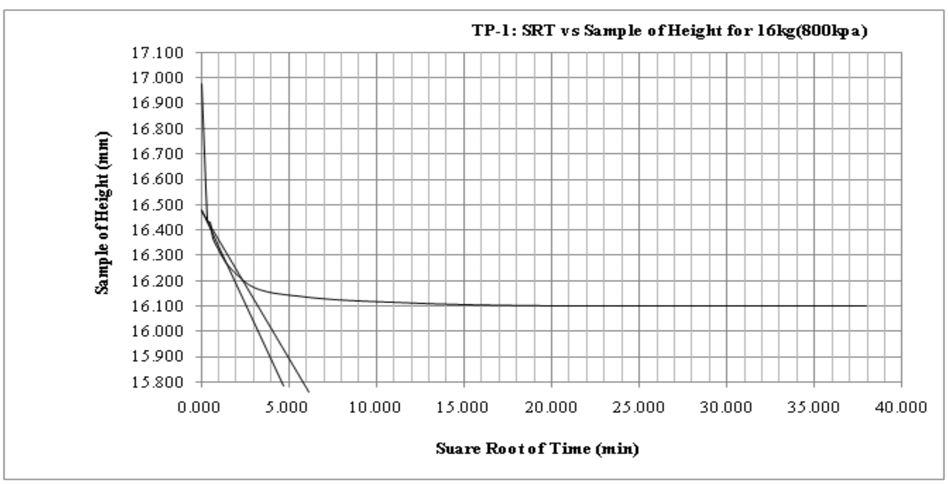
$$C_v = \frac{(0.848)18.1955^2}{3.24} = 86.65$$



$$C_v = \frac{(0.848)H_{dr}^2}{t_{90}}$$

$H_{dr} = 17.4185mm, \quad \sqrt{t_{90}} = 2.0, \quad t_{90} = 4.0$

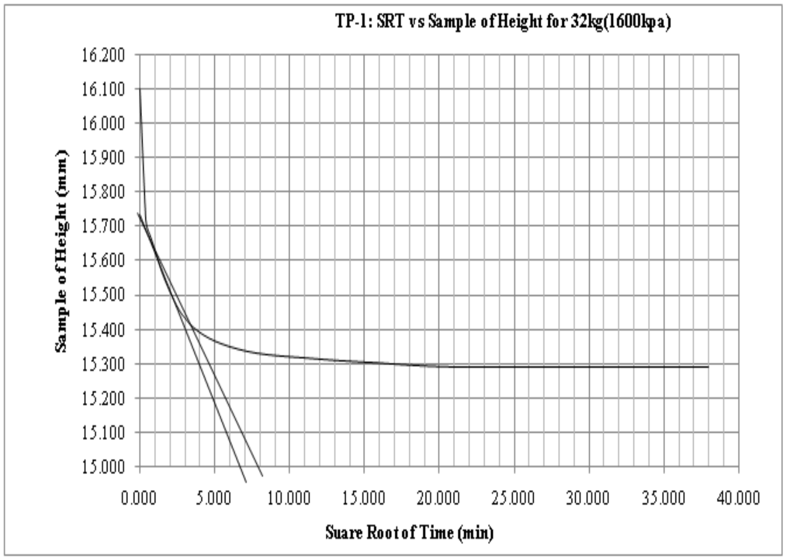
$$C_v = \frac{(0.848)17.4185^2}{4} = 64.32$$



$$C_v = \frac{(0.848)H_{dr}^2}{t_{90}}$$

$H_{dr} = 16.540mm, \quad \sqrt{t_{90}} = 2.2, \quad t_{90} = 4.84$

$$C_v = \frac{(0.848)16.540^2}{4.84} = 47.93$$



$$C_v = \frac{(0.848)H_{dr}^2}{t_{90}}$$

$H_{dr} = 15.696mm$, $\sqrt{t_{90}} = 3.5$, $t_{90} = 12.25$

$$C_v = \frac{(0.848)0^2}{12.25} = 17.05$$