JIMMA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

GEOTECHNICAL ENGINEERING STREAM

GEOTECHNICAL AND GEOPHYSICAL INVESTIGATIONS ON SOILS FOUND IN DOLE TOWN, ETHIOPIA.

A Research submitted to the School of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for the Award of Master's Degree in Civil Engineering (Geotechnical Engineering)

By:

Aliyi Angishu Tuke

February, 2022

Jimma, Ethiopia

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Advisor: Jemal J. Muhammed (Ph.D.)

Co-Advisor: Shelema Amena (MSc)

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By:

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DECLARATION

I declare that this thesis is entitled "***Geotechnical and Geophysical Investigations on Soils Found in Dole Town, Ethiopia***" is my original work and has not been submitted as a requirement for the award of any degree in Jimma University or elsewhere.

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As research Adviser, I hereby certify that I have read and evaluated this MSc thesis prepared under my guidance, by Mr. Aliyi Angishu Tuke entitled "***Geotechnical and Geophysical Investigations on Soils Found in Dole Town, Ethiopia"*** and recommend and would be accepted as a fulfilling requirement for the Degree Master of Science in Geotechnical Engineering

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ABSTRACT

Knowing the engineering properties of soil is very important before designing any structure. The study area for this research is located at Dole Town, south-east of Addis Ababa, as part of the West Arsi Zone. Even though the area is a highly attractive tourism area of the country, due to a lack of soil investigation, there is no confidence to construct huge buildings and infrastructures in the area, and this highly affects the development of the town. The objective of this study is to investigate the Geotechnical and Geophysical properties of the soil found in Dole Town. Representative disturbed and undisturbed samples from five test pits were collected from different parts of the town. Laboratory tests were carried out, including specific gravity, natural moisture content, Atterberg limits, undrained shear strength, consolidation, gradation analysis, permeability, and electrical resistivity tests. The apparatus and procedures used for analysis for all tests were done in accordance with American Society for Testing Materials (ASTM) Standards. Based on the results of this study, less than 50% of the particles pass through a 0.075 mm sieve. The soils are categorized as coarse-grained soils. The specific gravity of the soil ranges from 2.39 to 2.59, while the Atterberge limit tests were done using the cone penetrometer method, and the results are; liquid limits range from 18.29% to 37.82%, plastic limits vary from 14.81% to 34.79%, and the plasticity index was below 7%, which indicates the soil varies from "well-graded gravel with sand" to "silty sand soil," and as per the AASHTO classification system, these soils are classified as "gravel and silty sand soil". The compaction test result shows the maximum dry density (MDD) ranges from 0.89 g/cm3 to 1.55 g/cm3, and the optimum moisture content (OMC) ranges from 8.50% to 43.0%, which shows the soil is mediumly compressive and intermediately strength. One-dimensional consolidation test results show the soils' compression index ranges from 0.122 to 0.421 and their swelling index ranges from 0.0363 to 0.0742. The coefficient of permeability ranging from 0.02×10^{-4} to 1.16×10^{-4} cm/sec which indicates the soil investigated is a permeable soil. Shear strength characteristics of the soil were determined by using a direct shear test. The cohesion (Cu) of the Dole town ranges from 5.7 KN/m^2 to 45.8 kN/m^2, and the internal frictional angle ($Ø^o$) of the study area was from 26.4^o to 43.8^o, indicating the soil is a cohesive-friction soil. The result of electrical resistivity ranges from 3.22 to 368.56 Ὠm, which shows the soil is weathered and fractured ignimbrite, a mixture of gravel, pumice sand, and silt. The results imply the soil is light weight porous materials.

Key Words: *Compression and Strength Properties, Electrical Resistivity, Index Properties, Investigation, Soil.*

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TABLE OF CONTENTS

DECLARATION I

ABSTRACT II

ACKNOWLEDGEMENTS III

LIST OF TABLES VIII

LIST OF FIGURES X

ACRONYMS AND ABBREVIATIONS XI

CHAPTER ONE 1

1 INTRODUCTION 1

- 1.1 Background of the Study 1
- 1.2 Statement of the Problem 2
- 1.3 Research Questions 3
- 1.4 Objectives of the Study 3
  - 1.4.1 General Objective of the Study 3
  - 1.4.2 Specific Objectives of the Study: 3
- 1.5 Significance of the Study 3
- 1.6 Scope of the Study 3
- 1.7 Limitation of the Study 4
- 1.8 Structure of the Study 4

CHAPTER TWO 6

2 LITERATURE REVIEW 6

- 2.1 General 6
- 2.2 Formation and Mode Deposition of Soils 7
  - 2.2.1 Mechanical Weathering 7
  - 2.2.2 Chemical Weathering 7
- 2.3 General Types of Soils 8
- 2.4 Residual and Transported Soils 8
- 2.5 Size and shape of soil particles 9
- 2.6 Composition of the Soil's Minerals 10
  - 2.6.1 Kaolinite group 10
  - 2.6.2 Illite group 10
- 2.7 Index Properties of Soils 11
  - 2.7.1 Moisture content 11

2.7.2 Specific Gravity 12
2.7.3 Grain Size Determination 12
2.7.4 Atterberg limits 12
2.7.5 Activity of Clay Soils 14
2.7.6 Consolidation of Soils 14
2.7.7 Compaction of Soils 16
2.7.8 Shear Strength Tests 17
2.7.8.1 Direct Shear Test 18
2.7.8.2 Triaxial Compression Test 19
2.7.8.3 Unconfined Shear Strength 20
2.8 Soil Classification Systems 21
2.8.1 Unified Soil Classification System (USCS) 22
2.8.2 American Association of State Highway and Transportation Officials (AASHTO) 23
2.9 Geology of the Area 24
2.9.1 Regional Geology of the Study Area 24
2.9.2 Local Geology of the Study Area 25
2.9.3 Types of Soils 25
2.10 Geophysical Condition/Electrical Resistivity Condition 26
CHAPTER THREE 30
3 MATERIALS AND METHODOLOGY 30
3.1 General 30
3.2 Study Locations and Visual Identification 32
3.3 Geology 33
3.3.1 General 33
3.4 Electrical Resistivity Test 34
3.5 Study Design 36
3.6 Study Population 37
3.7 Study Variables .... **Error! Bookmark not defined.**
3.8 Data Collection Process 37
3.8.1 Sample Size and Selection 37
3.8.2 Sampling Techniques and Procedure: - 38

3.8.3 Field Survey 38
3.8.4 Sample Preparation. 39
3.9 Data Processing and Analysis 39
3.12 Laboratory Tests 40
CHAPTER FOUR 41
4 RESULTS OF LABORATORY AND ELECTRICAL RESISTIVITY TESTS 41
4.1 Field Observations 41
4.5 Grain Size Analysis 43
4.6 Atterberg Limits 45
4.7 Standard Proctor Compaction Test 46
4.8 Classification of Soils 47
4.9 Direct Shear Test 51
4.10 Results of Consolidation Test 53
4.11 Permeability 59
4.12 Electrical Resistivity Test 60
CHAPTER FIVE 70
5 DISCUSSIONS OF LABORATORY TESTS AND ELECTRICAL RESISTIVITY RESULTS 70
5.1 Classifications of the Soils 70
5.1.1 General 70
5.1.2 Classification of Soils Based on Unified Soil Classification System (USCS) 70
5.1.3 American Association of State Highway and Transport Official System (AASHTO) 70
5.2 Discussions of Engineering Properties Test Results 71
5.3 Discussion of Electrical Resistivity Results 71
5.4 Comparison of Test Results with Previously Done Researches 72
CHAPTER SIX 74
6 CONCLUSIONS AND RECOMMENDATIONS 74
6.1 CONCLUSION 74
6.2 RECOMMENDATIONS 75
REFERENCES 76

APPENDIX......80
I. Natural Moisture Content......81
II. Natural Unit Weight......84
III. Grain Size Analysis......88
IV. Atterberge Limits......102
V. Specific Gravity......107
VI. Proctor Compaction......112
VII. Direct Shear Strength......122
VIII. One Dimensional Consolidation Analysis......131
IX. Procedural Photos......137

LIST OF TABLES

Table 2-1 Soil Classifications According to Plasticity Index (Murthy V.N.S, 2002) . 14

Table 2-2 Soil Classification According to Activity 14

Table 2-3 Consistency and Unconfined Compression Strength of Clays (Das BM, 2019). 21

Table 2-4 Resistivity of Common Rocks and Ore Minerals (Briški *et al.*, 2020). 27

Table 2-5 Electrical Resistivity of Various Earth Materials Usually Encountered in Electrical Resistivity Surveys (Tsegaselassie, 2013). 29

Table 3-1 Sampling Location, Visual Identification and Designation of Sample Test Pits 32

Table 3-2 Standards Used for Research, Generally 40

Table 4-1 Summary of Natural Water Content of the Study Area 41

Table 4-2 Summary of Natural Specific Gravity of the Study Area 42

Table 4-3 Summary of Natural Unit Weight of the Study Area 43

Table 4-4 Sieve Analysis Test Result of the Study Area 43

Table 4-5 Atterberg Limit Test Result of the Study Area 45

Table 4-6 Summary of the Optimum Moisture Content and Maximum Dry Density of the Study Area 46

Table 4-7 USCS Soil Classification of the Study Area 48

Table 4-8 Classification of Soils Based on AASHTO Classification System 50

Table 4-9 Summary of Direct Shear Test for Angle of Friction and Cohesion for Dole Town 52

Table 4-10 Summary of Void Ratio, Pre-consolidation, and Over Consolidation Ratio for TP-3, TP-4, and TP-5 at 3m Depth 56

Table 4-11 Coefficients of Consolidation (Cv), Compressibility (av), and Permeability (k) 59

Table 4-12 Electrical Resistivity Results. 60

Table 4-13 Resistance Value of Dole Test Pit 1 61

Table 4-14 Resistance Value of Dole Test Pit 2 63

Table 4-15 Resistance Value of Dole Test Pit 3 65

Table 4-16 Resistance Value of Dole Test Pit 4 66

Table 4-17 Resistance Value of Dole Test Pit 5 68

Table 5-1 For the Soil Under Investigation on Index Property Tests Were Studied and Comparisons Were Made with known Ziway and Ropi in Central Rift Valley. 73

LIST OF FIGURES

Figure 2-1 Structure of the Main Clay Minerals: (a) kaolinite, (b) Illite and (c) Montmorillonite, Based on Combined Sheets (Haile, 2014). 11

Figure 2-2 Generalized values of Resistivity's of Various Rocks, Soils and Minerals (Azrief Azahar et al., 2019). 29

Figure 3. 1 Geographical Location of Dole Town (Source: Google map 2022). 30

Figure 3. 2 Map of Dole Town and Location of Sample Test-Pits (Source: Google map 2022). 31

Figure 3.3 Meteorological Data of Dole Town (2018-2024 years Recorded Data of Gubeta Arjo Metrological Instruments) 33

Figure 3. 4 Distribution of Ground Cracks in Ethiopia (IJEE, 2017) 34

Figure 3. 5 Electrical Resistivity Test. 35

Figure 3. 6 Research Design 36

Figure 3. 7 Field Visualizing, Color Identification and Collecting of Soil Samples ... 38

Figure4-1GrainSize Particle Distribution Curve……………………………………...44

Figure 4-2 Dry Density Vs Moisture Content curve ………………………………..47

Figure 4-3 USCS Plasticity Chart of Dole Town... 49

Figure 4-4 AASHTO Plasticity Chart Classification for the Dole Town Study Area . 51

Figure 4-5 Shear Stress Vs Normal Stress Curve for the Study Area 53

Figure 4-6 Plot of Void Ratio and Log Pressure Curve for TP-3, TP-4, and TP-5 at 3m Depth 54

Figure 4-7 Typical Example of Void Ratio Vs Log Pressure Curve Plot of TP-3 55

Figure 4-8 Void Ratio Vs Log Coefficient of Permeability (k) 60

Figure 4-9 Two-Dimension Expression of Dole Test Pit1 Resistivity 62

Figure 4-10 Three-Dimension Expression of Dole Test Pit1 Resistivity 62

Figure 4-11 Two-Dimension Expression of Dole Test pit2 Resistivity 63

Figure 4-12 Three-Dimension Expression of Dole Test pit2 Resistivity 64

Figure 4-13 Two-Dimension Expression of Dole Test pit 3 Resistivity 65

Figure 4-14 Three-Dimension Expression of Dole Test pit 3 Resistivity 66

Figure 4-15 Two-Dimension Expression of Dole Test Pit4 Resistivity 67

Figure 4-16 Three-Dimension Expression of Dole Test pit 4 Resistivity 67

Figure 4-17 Two-Dimension Expression of Dole Test Pit5 Resistivity 68

Figure 4-18 Three-Dimension Expression of Dole Test Pit5 Resistivity 69

ACRONYMS AND ABBREVIATIONS

AASHTO	American Association of State Highway and Transport Officials
A	Activity
ASTM	American Society of Testing Materials
ε	Axial Strain %
CBR	California Bearing Ratio %
CEC	Cat ion Exchange Capacity meq/100gm
Cc	Compression Index
Cc	Consistency Index
C	Clay
CV	Coefficient of Consolidation m2/day, cm2 /min
Cu	Cohesion for Undrained Shear Strength KN/m2
CST	Controlled Stress Rate
CH	Inorganic Clays of High Plastic
Cs	Swelling Index
γ_d	Dry unit weight KN/m3
ERA	Ethiopian Roads Authority
Es	Modulus of Deformation kPa/MPa
FS	Free Swell %
GI	Group Index
LI	Liquidity Index %
LL	Liquid limit
MDD	Maximum Dry Density
NMC	Natural Moisture Content
OMC	Optimum Moisture Content
PI	Plasticity Index
PL	Plasticity Limit
Pc	Pre-consolidation Pressure KN/m2
P	Vertical applied pressure KN/m2
qu	Unconfined Compressive Strength KN/m2
S	Shear Strength KN/m2
SL	Shrinkage Limit

TRL	Transport Research Laboratory
UCST	Unconfined Compression Strength Test
UU	Unconsolidated Undrained
USCS	Unified Soil Classification System
W	Water Content %

CHAPTER ONE

1 INTRODUCTION

1.1 Background of the Study

Almost all structures in the construction industry are built on or in the ground. Therefore, geotechnical engineers are required to know the properties of soil as part of the pre-conditioning work before designing and constructing any civil structure. Soil investigation is an essential part of the design and construction of a proposed structural system (buildings, dams, roads, highways, etc.). Soils are identified, observed, and recovered during the investigation of a proposed site (Haile 2014).

Usually, soil investigations are conducted only on a fraction of a proposed site because it would be prohibitively expensive to conduct an extensive investigation of the whole area. One then makes estimates and judgments based on information from a limited set of observations and from field and laboratory test data that will have profound effects on the performance and costs of structures constructed at a site (Dechasa 2017).

Investigating the engineering characteristics of soils is essential to use as a scientific database for further research and preliminary design to determine the presumptive soil pressure. Preliminary geotechnical investigations, faulty interpretation of results, or failure to portray results understandably may contribute to inappropriate designs, delays in construction schedules, costly construction modifications, the use of substandard material, environmental damage to the site, and even the failure of a structure. Therefore; the aim of investigation is to provide maximum information that is useful in the design and construction of the project at a minimum cost (Adugna and Yirga, 2020).

In a country like Ethiopia which is developing at high growth rate and which needs many constructions work in the future, geotechnical and geophysical investigation of soil is very essential. Because these data are very important for civil engineers in preliminary design and in designing foundation, pavement, retaining structures and for other construction projects in the country (Debebe, 2011).

Dole Town is one of the suitable areas in southern Ethiopia, in the West Arsi Zone of Oromia Regional State. Furthermore, Dole is known for its surrounding Langano, Shala, and Abijeta lakes, as well as the Abijeta Shala National Park, which is the country's well-known recreational area of high truism flow and Several governmental institutions and private business center are established in the town because of the road connect Addis, Sheshemene, Hawassa and Moyale across the town. However, the engineering property of the soil in the town is not studied. This thesis gives better understanding about geotechnical and geophysical behavior of the soil in the town.

1.2 Statement of the Problem

In the world one of the construction industry challenge is less attention of soil investigation and this will affect significant geotechnical and structural engineering with costs associated with maintenance and rehabilitation of structures after construction. For developing countries like Ethiopia which is developing at high growth rate the construction industry is also growing rapidly. So, detailed geotechnical investigation on the engineering property of soil is very essential (Haile, 2014).

Even though the Dole town is a highly tourism and investors attractive area of the country, due to absence of sufficient soil investigation, there are not enough infrastructure and buildings in the town. This is mainly due to lack of confidence to construct huge buildings since the area is rift valley area which is enclosed with surface and subsurface water. These conditions are highly affecting the development of the town and benefits of the community in the town. So, the need for detail geotechnical and geophysical investigation of the soils is a major problem and issue of concern and has a paramount importance for Dole town to design and construct safe and economical structure (Sifilet, 2020).

1.3 Research Questions

The research is aimed to answer the following main research questions:

- What are the index properties of soils in Dole Town?
- What are the types of soils found in Dole town?
- What is the compression and strength characteristics of this soil?
- What are the geological conditions of the area?

1.4 Objectives of the Study

1.4.1 General Objective of the Study

The general objective of this study is geotechnical and geophysical investigations on Soils found in Dole town.

1.4.2 Specific Objectives of the Study:

1. To determine the index properties of soils in Dole town.
2. To classify the soils found in Dole town.
3. To investigate the compression and strength characteristics of soils in Dole town.
4. To identify the geological conditions of the area.

1.5 Significance of the Study

The study will benefit Dole town administration and the town’s community by attracting investors who can invest in their economy by using the study as a source of information and a base for the construction industry that can help to minimize the time and cost of soil investigation. Most essentially, other researchers will use the findings as a reference for further research on the improvement of the engineering properties of soil.

1.6 Scope of the Study

The scope of this research was supported by different types of literature on the geotechnical and geophysical investigations of the soil in Dole town. A series of laboratory experiments are conducted on to determine soil type, including temperature, specific gravity, water content, Atterberg limits, undrained shear strength, consolidation, gradation analysis, permeability, and electrical resistivity. Due to the budget constraint, the depth of investigation in this research is limited to the maximum depth of three meters for geotechnical and 100 meters for electrical resistivity test.

The study's main objectives are to identify, classify, and determine the index properties, strength properties, and geophysical conditions of the soils in the study area. To do this, samples from the town were taken at typical locations from five test pits, both disturbed and undisturbed samples at 1.5 and 3 meters, respectively. An ADMT-500SX device was used to measure electrical resistivity in each test pit in order to compare the results to those from laboratory tests and field tests.

1.7 Limitation of the Study

In this study, the soil samples were collected only from five test pits; This is because of as the number of test pits increased since the cost of electrical resistivity for investigation increase this is going to be beyond the capacity of researcher. So; for further detailed investigation, such as preparing a soil map, these numbers of test pits are not enough.

1.8 Structure of the Study

The thesis is organized into six chapters; each chapter covers a specific topic of the research work. In the introductory chapter, the introduction, background of the problem, objective, scope, and structure of the thesis are presented. Chapter two deals with a brief literature review that discusses: soils and their classification, testing procedure aspects, descriptions of index properties, consolidation parameters, and shear strength parameters. Chapter three presents the study area, including information on the topography, climate, and geology of the study area. The fourth chapter deals with the types of laboratory tests conducted and the results obtained. Using the test results obtained, discussion, analysis, and comparison are done in Chapter five. Chapter six presents the conclusions and recommendations drawn from the research. Finally, grain size distribution curves, index property test results, consolidation test results, and classification schemes are given in the relevant appendices.

CHAPTER TWO

2 LITERATURE REVIEW

2.1 General

Soil from geotechnical engineering point of view is defined as a natural aggregate of mineral grains, with or without organic constituents that can be separated by gentle mechanical means such as agitation in water. By contrast rock is considered to be a natural aggregate of mineral grains connected by strong and permanent cohesive forces. The behavior of a structure depends upon the properties of the soil materials on which the structure rests. The properties of the soil materials depend upon the properties of the rocks from which they are derived. A brief discussion of the parent rocks is, therefore, quite essential in order to understand the properties of soil materials. In general, soil consists of the particles (solids), voids, water in some of the voids, and air taking up the remaining void space (Bell, 1992).

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 (Phogat et al). 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. According to (Phogat et al), the index properties of soils are water content, specific gravity, particle size distribution, consistency limits, in-situ density, free- swell and density index. Engineers usually use soil index properties to distinguish between several types of soil within a single broad category. For instance, clay can exhibit a variety of engineering properties depending on its composition. When the results of classification tests to establish index properties are compared to empirical data pertaining to the established index properties, the engineer will receive significant information (Murthy V.N.S, 2002).

Soils are usually cohesion less, cohesive, or organic. Cohesion less soils have particles that do not tend to stick together. Mostly it is composed of sand, maybe some silt. As a result, these soils tend to shift or change in consistency under different environmental conditions. Rain and wind conditions cause water and air materials to move in and out of soils. Cohesive soils on the other hand are characterized by very small particle sizes, such as clay or silt, where surface chemical effects predominate. They are both "sticky" and "plastic". Their shear strength equals about half its unconfined compressive strength. Therefore, cohesive soil is a better foundation than that of non-cohesive. Organic soils are usually found in low-lying areas where the water table is near or above the ground surface. This type of soil is typically spongy, crumbly, and compressible. They are undesirable for supporting structures. The soil consists of discrete solid particles which are neither strongly bonded as in solids nor they are as free as particles of fluid. Consequently, the behavior of soil is somewhat intermediate between that of a solid and a fluid. Due to this reason, it engineering property investigation gets crucial (Das and Sobhan, 2018).

2.2 Formation and Mode Deposition of Soils

Soils are formed by the process of weathering of the parent rock. The weathering of the rocks might be mechanical disintegration and/or chemical decomposition.

2.2.1 Mechanical Weathering

Mechanical weathering of rocks to smaller particles is due to the action of such agents as the expansive forces of freezing water in fissures, due to sudden changes of temperature or due to the abrasion of rock by moving water or glaciers. Erosion by wind and rain is a very important factor and a continuing event. Cracking forces by growing plants and roots in voids and crevasses of rock can force fragments separate apart (Murthy V.N.S, 2002).

2.2.2 Chemical Weathering

Chemical weathering (decomposition) can transform hard rock minerals into soft, easily erodible matter. The principal types of decomposition are hydration, oxidation, carbonation, desilication and leaching. Oxygen and carbon dioxide which are always present in the air readily combine with the elements of rock in the presence of water (Murthy V. N. S, 2002).

2.3 General Types of Soils

It has been discussed earlier that soil is formed by the process of physical and chemical weathering. The individual size of the constituent parts of even the weathered rock might range from the smallest state (colloidal) to the largest possible (boulders). This implies that all the weathered constituents of a parent rock cannot be termed soil. According to their grain size, soil particles are classified as cobbles, gravel, sand, silt and clay. Grains having diameters in the range of 4.75 to 76.2 mm are called gravel. If the grains are visible to the naked eye, but are less than about 4.75 mm in size the soil is described as sand. The lower limit of visibility of grains for the naked eyes is about 0.075 mm. Soil grains ranging from 0.075 to 0.002 mm are termed as silt and those that are finer than 0.002 mm as clay. This classification is purely based on size which does not indicate the properties of fine-grained materials (Dechasa, 2017).

2.4 Residual and Transported Soils

On the basis of origin of their constituents and deposition soils can be divided into two large groups. These are residual soils and transported soils. Residual soils are those that remain at the place of their formation as a result of the weathering of parent rocks. Transported soils are soils that are found at locations far removed from their place of formation. The transporting agencies of such soils are glaciers, wind and water. Difficulties with foundations and other types of construction are generally associated with transported soils.

Soil particle size < 0.002mm are Clay, Soil particle size 0.002 to 0.06mm are Silt, Soil particle size 0.06 to 2mm are Sand, Soil particle size 2 to 60mm are Gravel, Soil particle size 60 to 200mm are Cobbles and Soil particle size > 200mm are boulders.

Inorganic silt: has little or no plasticity and is cohesion less. Organic silt contains an admixture of organic matter. It is somewhat plastic, highly compressible, cohesive and relatively impervious. It is a very poor foundation material because of compressibility.
Clay soil: is composed of microscopic particles of weathered rock within a wide range of water content and exhibits plasticity and it is a cohesive fine-grained soil. Organic clay contains some finely divided organic matter and is usually dark grey or black in color and they are highly compressible when saturated and their dry strength is very high.

Black cotton soil is a residual soil containing a high percentage of the clay mineral montimorillonite. This soil has high shrinkage and expansive characteristics. Its color varies from dark grey to black. Great care is required when structures are to be built on black cotton soil. Peat is composed of fibrous particles of decayed vegetable matter. It is so compressible that it is entirely unsuitable to support any type of foundation (Tefera and Leikun, 1999).

2.5 Size and shape of soil particles

Particle sizes can range from gravel to the microscopic size. Depending on their size, they have distinct characteristics. A hand lens or the naked eye can see soil particles that are larger than 0.075 mm. They make up the soils' coarser components. Gravel and sand make up the soils' coarser fractions. Sand grains typically consist of quartz, whereas gravel is made up of individual rock fragments that are made up of one or more minerals. Some sands have the characteristic of elasticity because they contain a significant amount of mica flakes. Gravel and sand particles might have angular, sub-angular, sub-rounded, rounded, or well-rounded individual grains. Flat grains may be found in gravel. The smaller soil particles are made up of silt and clay. This fraction's grains typically contain just one mineral per grain (Murthy V.N.S, 2002).

2.6 Composition of the Soil's Minerals

Inorganic particles called soil minerals are formed from weathered parent rock and extinct plants and animals. Gravels are fragments of rock that occasionally contain flecks of feldspar, quartz, and other minerals. The main components of sand particles are quartz and feldspar. The minuscule soil fractions known as silts are made up of very small quartz grains and a few flake-shaped particles that are leftovers from micaceous mineral processing. Microscopic and submicroscopic mica and other mineral flakes are the main component of clays. The term "clays" refers to particles that become plastic when combined with a small amount of water. Clay minerals, which are hydrates of aluminum, iron, or magnesium silicate mixed to create sheet-like structures, are virtually usually the outcome of chemical weathering of rock particles (Ware, 2020). These sheets are composed of two fundamental building blocks: the octahedral unit of aluminum, iron, or magnesium hydroxide, and the tetrahedral unit of silica. The following are the top three clay mineral subgroups (Fatahi et al., 2011).

2.6.1 Kaolinite group

This makes up the majority of the residual clay deposits and is composed of massive stacks of alternating single-tetrahedral silicate and octahedral aluminum sheets. Kaolinites have a robust structure, are very stable, and just slightly absorb water. They respond to changes in water content by shrinking and expanding less than expected (Fatahi et al., 2011).

2.6.2 Illite group

This mineral is made up of two silicon tetrahedral sheets and a succession of single aluminum octahedral sheets. Illites have greater swelling and shrinking properties than kaolinites and have a tendency to absorb more water (Fatahi et al., 2011).

2.6.3 Montmorillonite group

This mineral shares a structure with the illite group, but some of the silicon is replaced by iron, magnesium, and aluminum in the tetrahedral sheets. Extremely high-water absorption, swelling, and shrinking characteristics are displayed by montmorillonite (Fatahi et al., 2011).

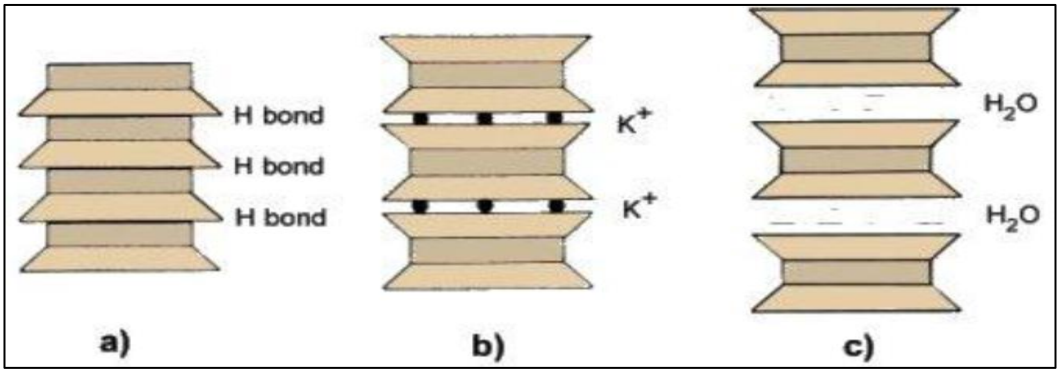


Figure 2-1 Structure of the main clay minerals: (a) kaolinite, (b) Illite and (c) Montmorillonite, based on combined sheets (Haile, 2014).

2.7 Index Properties of Soils

2.7.1 Moisture content

The ratio of the quantity of water to the mass of solid material in a soil sample is known as the water content or moisture content of the soil material. The relationship between the phases of air, water, and solids in a given volume of a substance is expressed by the water content of that medium. The "liquidity index," which is based on a soil's water content as well as its liquid and plastic limitations, is used to describe a soil's relative consistency (Das and Sobhan, 2018).

Natural water content used to express the consistency of clay soil in its natural state. Consistency is a term used to indicate the degree of firmness of cohesive soils. However, it has been found that at the same water content, two samples of clay of different origins may possess different consistency. Water content alone, therefore, is not an adequate index of consistency for engineering and many other purposes (Jibril, 2017).

2.7.2 Specific Gravity

The three phases of soil are solid, liquid, and gas. Although specific gravity is simply a property of soil solids, many soil parameters, including unit weight void ratio, porosity, and water content, relate the proportion of different phases to one another or to the overall mass or volume of the soil. The majority of the soils under consideration have specific gravities that fall between a small range of 2.5 and 2.85. However, soil that is organic or contains porous particles like diatomaceous earth has a low specific gravity, usually 2.3 or less. But at the other side, soils with heavy elements like iron may have values higher than 3. Different lateritic soils have different specific gravities. This is caused by the lateritic soils' abundant mineral content, which contains a significant amount of iron oxide (Guo *et al.*, 2023).

2.7.3 Grain Size Determination

Soil consists mostly of different sized soil particles as major constituent ingredient. The determination of the fractions of the particles will help to identify the soil type as well as to estimate many other engineering properties such as strength and permeability and also to identify whether the soil is suitable for construction projects such as highways, dams or as a backfill or for filter design. Grain size analysis is usually used in engineering soil classifications. Two methods are mostly used to determine grain size distribution are Sieve analysis for coarse grained portion of the soil (size coarser than 0.075mm) and Hydrometer analysis for fine grained portions (size finer than 0.075mm) (ASTM International, 1998).

2.7.4 Atterberg limits

Depending on the amount of water present, fine-grained soil can exist in the states of solid, semisolid, plastic, viscous, or fluid. Seven "limits of consistency" were initially established by the Swedish soil scientist Albert Atterberg to categorize fine-grained soils, but only two of them, "the liquid and plastic limits," are now often utilized in engineering. Occasionally, a third restriction known as the shrinkage limit is employed. These Atterberg limits are also used to categorize fine-grained soils according to the USCS or AASHTO systems. A wide range of soil engineering parameters have been associated with the liquid and plastic limits. The soil's moisture content is the basis of the Atterberg limits (Lollo, 2016).

2.7.4.1 Liquid Limit (LL)

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 a standard groove, and dropped from a height of 1 cm will undergo a groove closure of about 1.3 cm when the cup is dropped 25 times at the rate of 2 drops per sec (Lollo, 2016).

2.7.4.2 Plastic Limit (PL)

The plastic limit of a soil is the water content at the boundary between the plastic and semisolid states. The water content at this boundary is arbitrarily defined as the water content at which soil begins to crumble when rolled into threads of a specified size (3.2 mm) (Lollo, 2016).

2.7.4.3 Plasticity index (PI)

The difference between the liquid and plastic limits of soil is equivalent to the plasticity index. PI is another way to represent the plasticity index. This is the water content range where soil turns into its plastic condition. This is a crucial indicator of soil adaptability to us.

$$PI = LL - PL \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.1)$$

Where: PI= Plasticity index, LL = liquid limit, PL= plastic limit

Table 2-1 Soil classifications according to plasticity index (Murthy V.N.S, 2002)

Plasticity Index	Plasticity
0	Non-plastic
<7	Low Plastic
7-14	Medium Plastic

2.7.5 Activity of Clay Soils

According to Skempton (1953) and Arora K.R. (2004), the plasticity index and the concentration of colloidal clay particles in the soil both have a role in the volume of a clay soil significantly changing whether it shrinks or swells. There are three types of clay soil: inactive, normal, and active (K.R. Arora, 2004). The activity of clay is expressed as

$$A = \frac{PI}{C} \quad \text{………………………………………………………………} \quad (2.2)$$

Where: A= Activity, C = Clay percent finer than 2µm, PI= Plasticity index

According to the value of Activity, Table 2-2 shows the various soil types. The clay soil can be categorized as belonging to the swelling type if its activity value is greater than 1.25.

Table 2-2 Soil classification according to activity (Murthy V.N.S, 2002)

Activity, A	Soil type
<0.75	Inactive
0.75-1.25	Normal
>1.25	Active

2.7.6 Consolidation of Soils

2.7.6.1 One Dimensional Consolidation of Soils

When a soil layer is subjected to a compressive stress, such as during the construction of a structure, it will exhibit a certain amount of compression. This compression is achieved through a number of ways, including rearrangement of the soil solids or extrusion of the pore air and/or water. According to Terzaghi (Terzaghi *et al.*, 1996), “a decrease of water content of a saturated soil without replacement of the water by air is called a process of consolidation.” When saturated clayey soils which have a low coefficient of permeability are subjected to a compressive stress due to a foundation loading, the pore water pressure will immediately increase; however, because of the low permeability of the soil, there will be a time lag between the application of load and the extrusion of the pore water and, thus, the settlement. This phenomenon is called consolidation (Das and Sobhan, 2018).

2.7.6.2 The Standard One-Dimensional Consolidation Test

The 1-D test is used to obtain a compression parameter for the amount of settlement and the consolidation parameter CV for the settlement rate estimate. The pre-consolidation pressure pc and thus the OCR can also be determined from this test. The test is performed on an "undisturbed" soil sample that is placed in a consolidation ring available in diameters ranging from 45 to 115 mm. The sample height is between 20 and 30 mm; 20 mm is the most commonly used thickness to reduce test time (Jibril, 2017).

2.7.6.3 Coefficients of Compression

Compression parameters such as coefficients of compressibility (av), compression index (Cc), swelling index (Cs) and coefficient of volume compressibility (mv) can be obtained from void ratio versus applied pressure curve of consolidation test (Das and Sobhan, 2018).

2.7.6.4 Normally Consolidated and Over consolidated Clays

A clay layer is said to be normally consolidated if the current effective overburden pressure of δ_o is the highest pressure to which the layer has ever been subjected at any time in its history, whereas a clay layer is said to be over-consolidated if the layer has been subjected to a greater effective overburden pressure of C than the current pressure of δ_o. The ratio of p_c/δ_o is called the over-consolidation ratio (OCR) (Das and Sobhan, 2018).

2.7.6.5 Determination of the Coefficient of Consolidation and Coefficient of Permeability

By fitting the experimental curve, the coefficient of consolidation (CV) can be assessed using laboratory experiments. There are two laboratory techniques that are frequently used to estimate CV. These two methods are the Taylor Square Root of Time Fitting Method and the Casagrande Logarithm of Time Fitting Method (Jibril, 2017). Using the follow ing formula, one can calculate the average vertical coefficient of permeability, k:

$$k = C_v m_v \gamma_w \quad \text{………………………………………………………} \quad (2.3)$$

Where: k is coefficient of permeability, Cv is vertical coefficient of consolidation, mv is coefficient of volume compressibility, and γ_w is unit weight of water.

2.7.7 Compaction of Soils

2.7.7.1 General

The mechanical compressing of soil particles together is known as compaction. Air is forced out of the vacuum space in the soil mass during compaction, increasing the mass density. A soil mass is treated to intervention to enhance its engineering properties. In general, compaction enhances shear strength, which also improves stability and bearing capacity. Additionally, it aids in reducing the soil's permeability and compressibility (Debebe *et al.*, 2011).

2.7.7.2 Factors Affecting Compaction

Water content, compaction effort, soil type, and compaction method are all variables that affect the increase in dry density of the soil that results from compaction. Additionally, for lateritic soils, the compaction outcomes might be impacted by how the sample is handled and prepared (Wakjira, 2020).

2.7.7.3 Bulk Unit Weight

The bulk unit weight (γ) is defined as the total weight per unit total volume.

$$\gamma = \frac{W}{V} \quad \text{…………………………………………………………} (2.4)$$

2.7.7.4 Dry Unit Weight

Dry Unit Weight (γ_d) is defined as the weight of solids per unit total volume

$$\gamma_d = \frac{Ws}{V} \quad \text{……………………………………………………… …..}(2.5)$$

2.7.7.5 Maximum Dry Density

Because it is interested in the weight of solid soil particles in a given volume rather than the amount of solid, air, and water in a particular volume, the dry unit weight (d) is crucial to employ when determining the degree of compaction (which is the bulk unit weight).

$$\gamma_\mathrm{d} = \frac{\gamma_\mathrm{bulk}}{1+\mathrm{W}} \quad \text{…………………………………………………………...} (2.6)$$

Where: γ_d= dry unit weigh of the soil sample, W = moisture content, γ_{bulk} = bulk unit weigh of the soil sample

2.7.7.6 Optimum moisture content (OMC)

The optimum moisture content is the water content corresponding to maximum dry density which is obtained from dry density moisture content graph of compaction test.

2.7.8 Shear Strength Tests

Many foundation engineering challenges involving stability involve the shear strength of the soils. These include the bearing capacity of shallow foundations and piling, the stability of dam and embankment slopes, and lateral earth pressure on retaining walls. In a shear test, the shear force is applied by increasing either the shear displacement or the shear force at a specific rate. Therefore, the shear tests are either stress or strain controlled (Merga, 2016).

In a strain-controlled testing, the shearing strain is increased during the test at a predetermined rate. The specimen is typically sheared at a constant strain rate. A proving ring is used to assess the shear force indirectly operating on the specimen. The rate of shearing strain can be managed manually or by a gear system connected to an electric motor. The majority of shear tests are performed in strain-controlled environments (Tefera and Leikun, 1999).

A stress-controlled test involves gradually increasing the shear force. When the shear force is raised, the shear stresses rise consistently. Using a dial gauge, the resulting shear displacements are measured. Because an applied load can be easily maintained constant throughout time, stress-controlled tests are ideal for carrying out shear testing at very low rates. The loads can also be applied and removed easily. The field conditions are more accurately reflected by the stress-controlled test (Tefera and Leikun, 1999)

The internal angle of friction and total and effective cohesion of the shear strength parameters, as well as their representative values, are determined through shear strength testing. The drainage conditions during the test had a significant impact on the measured values. The direct shear test, triaxial compression test, and unconfined compression test are the three most often used laboratory techniques for obtaining shear strength data (Murthy V.N.S, 2002).

2.7.8.1 Direct Shear Test

The popular apparatus to determine the shear strength parameters is the shear box. The direct shear test measures cohesion and the angle of internal friction for both cohesive and cohesionless soils. However, it is more effective in cohesive less soil. The shear box consists of a horizontally split, open metal box which is called upper and lower shear boxes. The soil specimen is placed in the box and one–half of the box is moved relative to the other half. Failure is there by constrained along a thin zone of soil on the horizontal pane. Serrated metal plates are placed at the top and bottom of faces of the soil to generate shearing force (Merga, 2016).

A metal plate sitting on the top serrated plate is used to transmit vertical forces. For load control, horizontal forces are provided either by weights through a pulley system or by a motor for displacement control. Displacement control is used for the majority of shear box testing since it allows us to get both the peak and the critical shear forces. Beyond the maximum or peak shear force, load control experiments do not yield data (Merga, 2016).

2.7.8.2 Triaxial Compression Test

A widely used and reliable apparatus to determine the shear strength parameters and the stress strain behavior of soil is the triaxial apparatus. The triaxial apparatus is versatile because we can independently control the applied axial and lateral stresses, conduct tests under undrained and drained conditions and it is possible to both control and measure the pore pressure. In the triaxial test, a cylindrical sample of soil usually with a length to diameter ratio of two is subjected to either controlled increases in axial stresses or axial displacements and radial stresses. A membrane laterally confines the sample, and radial stresses are applied by pressuring water in the chamber. The axial stresses are applied by loading a plunger. The triaxial tests are classified according to the condition of drainage during the test as UU, CU, and CD tests (Liu and Evett, 2009).

Unconsolidated Undrained (UU) tests are carried out by placing a specimen in the chamber and applying lateral pressure without allowing the specimen to consolidate or drain under the confining pressure. Axial load is then applied fairly rapidly without permitting drainage of the specimen. Consolidated undrained (CU) tests are performed by placing a specimen in the chamber and applying lateral pressure. The specimen is then allowed to consolidate under the all-round confining pressure by leaving the drain lines open. The drain lines are then closed, and axial stress is induced without allowing further drainage.

Consolidated drained (CD) tests are similar to CU tests except that the specimen is allowed to drain as axial load is applied so that high excess pore pressure do not develop. CD tests may take a considerable period of time to run because of the time required for both consolidation under the confining pressure and drainage during application of axial load. For practical tests both CU and CD yield the same strength provided the tests are performed correctly (Kalinski, 2011). Most triaxial testing is CU because the time for testing is less than for CD test. Therefore, CU tests were carried out on undisturbed samples obtained, by tube sampling, from the field.

2.7.8.3 Unconfined Shear Strength

One of the most crucial elements of geotechnical engineering is soil shear strength. The shear strength of the soil has an impact on several design factors, including slope stability, pavement design, slope bearing capacity, and retaining wall design. Structures and slopes must be stable and secure against total collapse when subjected to the highest expected applied stresses. Cohesion, or the attraction between particles, and angle of internal friction, or the resistance to inter-particle slide, are the two soil parameters used to assess shear strength. These two parameters implicitly take into account grain crushing, rolling resistance, and other variables. The Mohr-Coulomb failure criterion is a good representation of this behavior (Jolfaei and Lakirouhani, 2022).

A characteristic of true clay is the property of cohesion, sometimes referred to as no load shear strength. The shear strength of saturated cohesive soil in undrained shear test (i.e., test in which change in volume is prevented) is derived entirely from cohesion. In such a case, the shearing strength is independent of magnitude of normal stress. However, in slow shear test, in which consolidation takes place, the shear strength of clay increases with normal stress (Teferra, 1999).

Table 2-3 Consistency and unconfined compression strength of clays (Das BM, 2019).

Consistency	q_u (kPa)
Very Soft	0-24
Soft	24-48
Medium	48-96
Stiff	96-192
Very Stiff	192-383
Hard	>383

2.8 Soil Classification Systems

All widely used engineering soil classifications involve a combination of particle size and measures of plasticity and textural soil classifications. In addition to providing an orderly system for classification, the use of particle size and plasticity permits the Engineer to estimate the engineering properties of soils such as compaction, settlement, drainage, frost susceptibility, placement, excavation, and embankment characteristics. As grain size decreases, engineering problems associated with soils tend to increase. Also, the difficulty with which particle-size distribution in a soil sample is determined also increases. As a result, the proportions and properties of the so-called fines (silt and clay sizes) present in a soil are evaluated by their plasticity rather than by more time- consuming sediment logical procedures. The measures of plasticity, the Atterberg limits, are directly applicable to design and construction uses of a soil, whereas strict size ranges and amounts are not. Soils classification can be done in to two main ways.

First, Visual classification of soils (field classification method) - during excavation and sampling operations in the field classification has to be carried out quickly and without gradation analyses or Atterberg limits.

Second, laboratory classification of soils- this classification system is used after gradation analyses or Atterberg limit test is done in the laboratory. At the present time, two major soil classification systems are available for general engineering use. They are the unified soil classification system (USCS), and the American association of state highway and transport official (AASHTO) system. Both systems use simple soil properties such as grain-size distribution, liquid limit, and plasticity index of soil (Midekssa *et al.*, 2015).

The soil identified in the field is done by conducting the following simple test. The sample is first spread on a flat surface. If more than 50% of the particle are visible to the naked eye (unaided eye), the soil is coarse-grained; otherwise, fine-grained soils. The fine-grained particles are smaller than 0.075mm size and are not visible to unaided eye. The fraction of the soil smaller than 0.075mm size, that is the clay and the silt fraction, is referred to as fines (Arora K.R, 2004).

For the fine-grained soils, the following tests shall be conducted. These are dilatancy (reaction to shaking) test, toughness test and dry strength test as well as consistency test.

2.8.1 Unified Soil Classification System (USCS)

The Unified Soil Classification System was developed cooperatively by the U.S. Army Corps of Engineers (USAE) and the U.S. Bureau of Reclamation (USBR). The USC classification was published in 1953. It has since been adopted by the American Society for Testing and Materials (ASTM) as the standard classification of soils for engineering purposes. The USC system is a textural-plasticity classification scheme. Soils are divided into two major groups, coarse-grained and fine-grained soils, using the No. 200 sieve as the size criterion. When more than half of the soil sample is larger than the No. 200 sieve, it is classified as coarse-grained and is further subdivided by sieving and gradation. When more than half of the soil sample is smaller than the No. 200 sieve, it is classified as fine-grained and is subdivided primarily based on liquid limit values and degree of plasticity (Liu and Evett, 2009).

2.8.2 American Association of State Highway and Transportation Officials (AASHTO)

AASHTO Classification System in addition to the USCS system, an engineering soil classification was developed in 1928 by the U.S. Bureau of Public Roads, which is now called the American Association of State Highway and Transportation Officials (AASHTO). It is a textural-plasticity classification that uses sieved fractions and Atterberg limits for assignment of soils to seven main groups and several subgroups. The classification is more specific than the USC system in the limits placed on size ranges and amounts and ranges of liquid limits and plasticity indexes for fines. As with the USC system, these limits are placed on groups within both the granular (coarse-grained) and silty/clay (fine-grained) soils as required by soil gradations. Rather than using the No. 4 sieve (4.75 mm) of the USC system as the upper limit of the sand-size range, the AASHTO classification uses the No. 10 sieve (2.0mm) as the upper size limit of sand. However, the No. 200 sieve (0.075 mm) used in the USC system is retained to separate the finer fractions from sand (Liu and Evett, 2009). The AASHTO system classifies soils into eight groups, A-1 through A-8. Fine-grained soils are further rated for their suitability for highways by the group index (GI), determined as follows:

$$GI = (F - 35)\ [0.2 + 0.005(LL - 40)] + 0.01(F - 15)\ (PI - 10) \quad \ldots\ldots\ (2.7)$$

Where: F = percentage by weight passing through sieve No.200 (size 0.075 mm), expressed as Whole number; LL = liquid limit; and PI = plasticity index.

While calculating GI from the above equation, if any term in the parentheses becomes negative, it is dropped; not given a negative value. The group index is rounded off to the nearest whole number. If the computed value is negative, the group index is reported as zero. The group index is appended to the soil type determined from the classification table. For example, A-6 (15) indicates the soil type A-6, having a group index of 15. The smaller the value of the group index, the better is the soil in the category. A GI of zero indicates a good sub-grade, whereas a group index of 20 or greater shows a very poor sub-grade. The

GI must be mentioned even when it is zero to indicate that the soil has been classified as per AASHTO system (AASHTO, 2006).

2.9 Geology of the area

2.9.1 Regional Geology of the study area

Generally, the present day physiographic of Ethiopia is determined by two tectonic phenomena, known as;

1. The uplifted swell and,
2. Its subsequent dissection by the rift system.

The rift system in Ethiopia has divided the uplifted swell in two separate units, i.e. North West plateau and South East plateau. In late Oligocene extrusion of trap series lava took place which was accompanied by rifting during Miocene. The relative displacement of rift floor below the edge of uplifted plateau varies up to as much as 2000m.

The geological set up of West Arsi zone in general is dominated as follow,

1) In Oligocene to Miocene there were the eruption volcanic rocks of Alaji group composed by fissure basalt, and trachyte formed along the eastern margin of the rift and eastern plateau (Kazmin and seife 1978). These formations are also exposed around Adaba and Dodola town in West Arsi zone. It unconformable had overlain Mesozoic sedimentary succession and underlying Nazareth group. Its thickness is about 1200m.

2) The formation of Alaji group was followed by Upper Miocene to Pliocene age of both silicic and basic volcanic rocks eruption of the Nazret Group. The oldest unit of the Nazreth group of late Miocene to Pliocene age is dominated by rhyolitic ignimbrites with subordinate porphyritic trachybasalt in the upper and lower part of the succession. The Nazret Group ignimbrites, unwelded tuff, ash flows, rhyolite and trachyte crop out in rift scarp area and covers large portion of Gadab Hasasa plain.

3) The formation of chilalo volcanics was followed by Lower Pleistocene age eruption of volcanic rocks of the Dino Formation. The eruption of this group possibly related to the formation caldera such as Hawasa, Shalla and Ziway. The dino formation which composed of rhyolitic ignimbrites, crystal-rich in rhyolite, rhyodacite and trachyandesite, belonging to the volcanic activity of the Hawasa caldera are present along the road between Shashemene to Aje town near to Aje town, out cropped around Lake Shalla, and exposed along with escarpment of weransa ridge. The thickness of this formation about 600m

5) The formation dino or wonji group of Hawasa caldera was followed by the younger corbetti caldera. This caldera produces several ignimbrite units during the Middle Pleistocene age. This unit outcropped in sole quarry that is found to east of shashemene town. The thickness of this formation is about 700m (RARC and JICA, 2014; Woldegabriel et al., 1990).

2.9.2 Local Geology of the Study area

The geology that covers the study area comprises the weathered and fractured ignimbrite, tuff and pumice. The deep boreholes (196m average depth) drilled around Dole town for Rift Valley basin Study purpose encountered thick deposit of ignimbrite, pumice and volcanic ash almost at drilled well and form the major aquifer of study area (JICA, 2014).

2.9.3 Types of Soils

Based on their methods of deposition soil can be broadly categorized in to two categories as residual soil and transported soil. According to (RARC and JICA, 2014; Woldegabriel et al., 1990), based on the transporting agency and methods of deposition transported soil can be classified as:

1. **Alluvial soil**: it transported by running water (rivers) and typically made up of a variety of materials, including fine particles of silt and clay and larger particles of sand and gravel.

2. **Lacustrine soil**: it deposited in the lakes. This soil material is well sorted and fine-textured, having finer silts and clays. Due to this high clay content it has low permeability.
3. **Marine soil**: it is deposited in the sea water. This is formed from materials carried into the seas by streams and material eroded from the beaches by the tidal action of the waves.
4. **Aeolian soil**: it is transported by wind. In this if the deposited soil contains material larger than sand size it forms sand dune while the finer deposited particles are form Loess.
5. **Glacial soil:** it is unsorted material by size and deposited with a movingice sheet.
6. **Colluvial** soil: deposited through action of land slide and slope wash. It consists of mixed deposits of rock fragments and soil materials accumulated at the bases of steep slopes through the influence of gravity. The grain-size can vary from clay and silt to boulder-size. A coarse deposit due to rock fall at a cliff base is called talus.

2.10 Geophysical Condition/Electrical Resistivity condition

Advantages of geotechnical geophysics are related to site accessibility, portability, noninvasiveness, and operator safety. Geophysical equipment can often be deployed beneath bridges and power lines, in heavily forested areas, at contaminated sites, in urban areas, on steeply dipping slopes, in marshy terrain, on pavement or rock, and in other areas that might not be easily accessible to drill rigs or cone penetration test (CPT) rigs (Tabwassah and Obiefuna, 2012). Also, most surface-based or airborne geophysical tools are noninvasive and, unlike boring or trenching, leave little if any imprint on the environment. These considerations can be crucial when working in environmentally sensitive areas, on contaminated ground, or on private property. In addition, geophysical surveys are generally considered less dangerous than drilling since there are fewer risks associated with utility encounters and operations. Lastly, geophysical surveys can enable engineers to reduce the number of required boreholes (Ayolabi et al., 2012).

Resistivity Consider and electrically uniform cube of side ‘L’ through which current ‘I’ is passing. The material within the cube resists the conduction of current through it, resulting in a potential drop V between the opposite faces. The resistance R is proportional to the length L of the resistive material and inversely proportional to its area of cross section.

The constant of proportionality is the ‘true’ RESISTIVITY, denoted by ρ. Hence the units of resistivity are the product of the resistance Ώ and a distance (area/length, meters) to giver Ohm-meters (Ώ-m).

$R = \rho L/A$……………………………………………….……..……………… (2.8)

$\rho = RA/L$…………………………………………………………….……... (2.9)

The inverse of resistivity (1/ ρ) is the conductivity σ which has units of siemens/meter (S/m) which are equivalent to mhos/m (Ώ$^{-1}$m^{-1}).
In most rocks, conduction is by way of pore fluids acting as electrolytes with the actual mineral grains contributing very little to the overall conductivity of the rock (except where those grains are themselves good electronic conductors). The resistivity of geological materials exhibits one of the largest ranges of all physical properties: from 1.6×10^{-8} Ώ -m for native silver to 10^{16} Ώ -m for pure Sulphur. In general, igneous rocks tend to have the highest resistivity; metamorphic rocks have intermediate but overlapping resistivity and Sedimentary rocks tend to be most conductive, largely due to their high pore fluid content. The resistivity of igneous and metamorphic rocks is greatly dependent on the degree of fracturing, the percentage of the fractures filled with fluids, and the resistivity of the fluid filling the fractures. Thus, a given rock type can have a large range of resistivity, from about 1000 to 10 million Ώ-m, depending on whether it is wet or dry. Sedimentary rocks, which are usually more porous and have higher water content, normally have lower resistivity values compared to igneous and metamorphic rocks (Briški et al., 2020).

Table 2-4 Resistivity of common rocks and ore minerals (Briški et al., 2020).

Rock/ Ore type	Resistivity (Ohm-m)	Rock/ Ore type	Resistivity (Ohm-m)
Topsoil	50–100	Graphitic schist	10–500
Loose sand	500–5000	Slates	500–500 000
Gravel	100–600	Quartzite	500–800 000
Clay	1–100	Pyrite (ores)	0.01–100
Weathered bedrock	100–1000	Pyrrhotite	0.001–0.01
Sandstone	200–8000	Chalcopyrite	0.005–0.1
Limestone	500–10 000	Galena	0.001 – 100
Greenstone	500–200 000	Sphalerite	1000 – 1 000 000
Gabbro	100–500 000	Magnetite	0.01 – 1000
Granite	200–100 000	Cassiterite	0.001 – 10 000
Basalt	200–100 000	Hematite	0.01–1 000 000

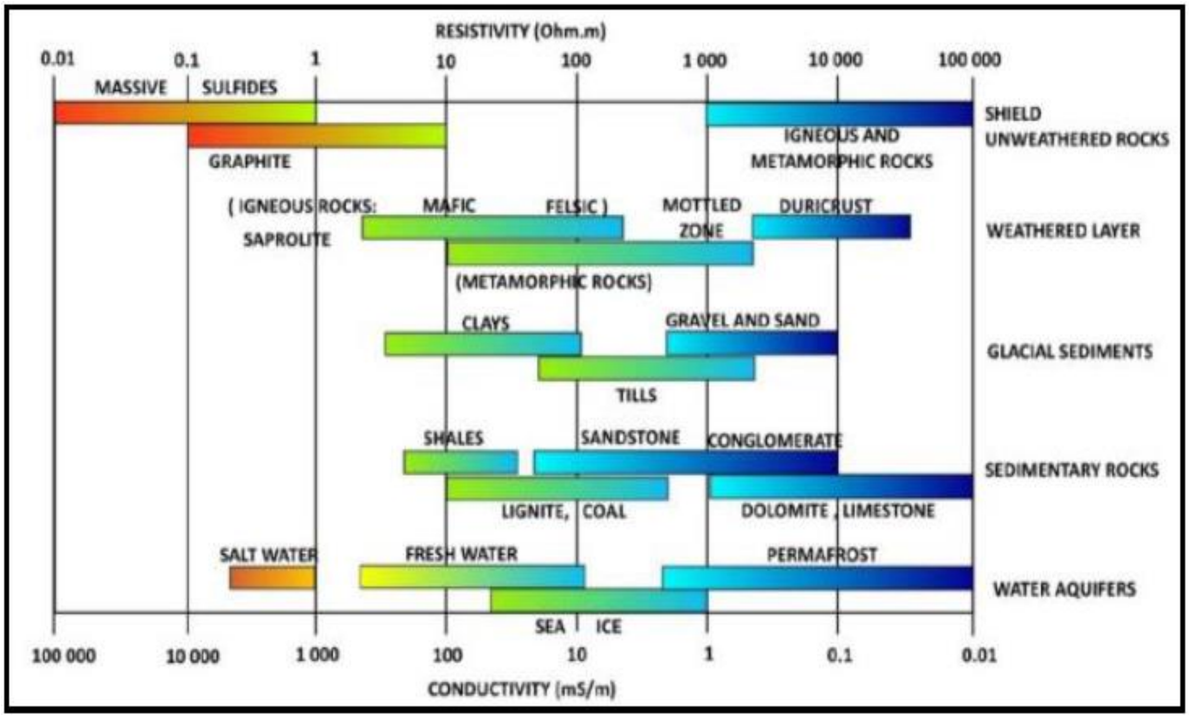


Figure 2-2 Generalized values of resistivity's of various rocks, soils and minerals (Azrief Azahar et al., 2019).

Table 2-5 Electrical Resistivity of Various Earth Materials Usually Encountered in Electrical Resistivity Surveys (Tsegaselassie, 2013).

Material	Resistivity, ohm-meters
Clay	1-20
Sand, wet to moist	20-200
Shale	1-500
Porous limestone	100-1,000
Dense limestone	1,000-1,000,000
Metamorphic rocks	50-1,000,000
Igneous rocks	100-1,000,000

CHAPTER THREE

3 MATERIALS AND METHODOLOGY

3.1 General

Dole town is found in West Arsi Zone of Oromia National Regional State and it is located at 210km South of Addis Ababa capital city of Ethiopia at latitude and longitude of $7^{o}30^{l}$ N and $38^{o}51^{`}$E respectively. The topography of the study area is flat and Elevation ranges from 1500m to 1700m above sea level. Dole is predominantly covered with white, yellow, gray and brown soils.

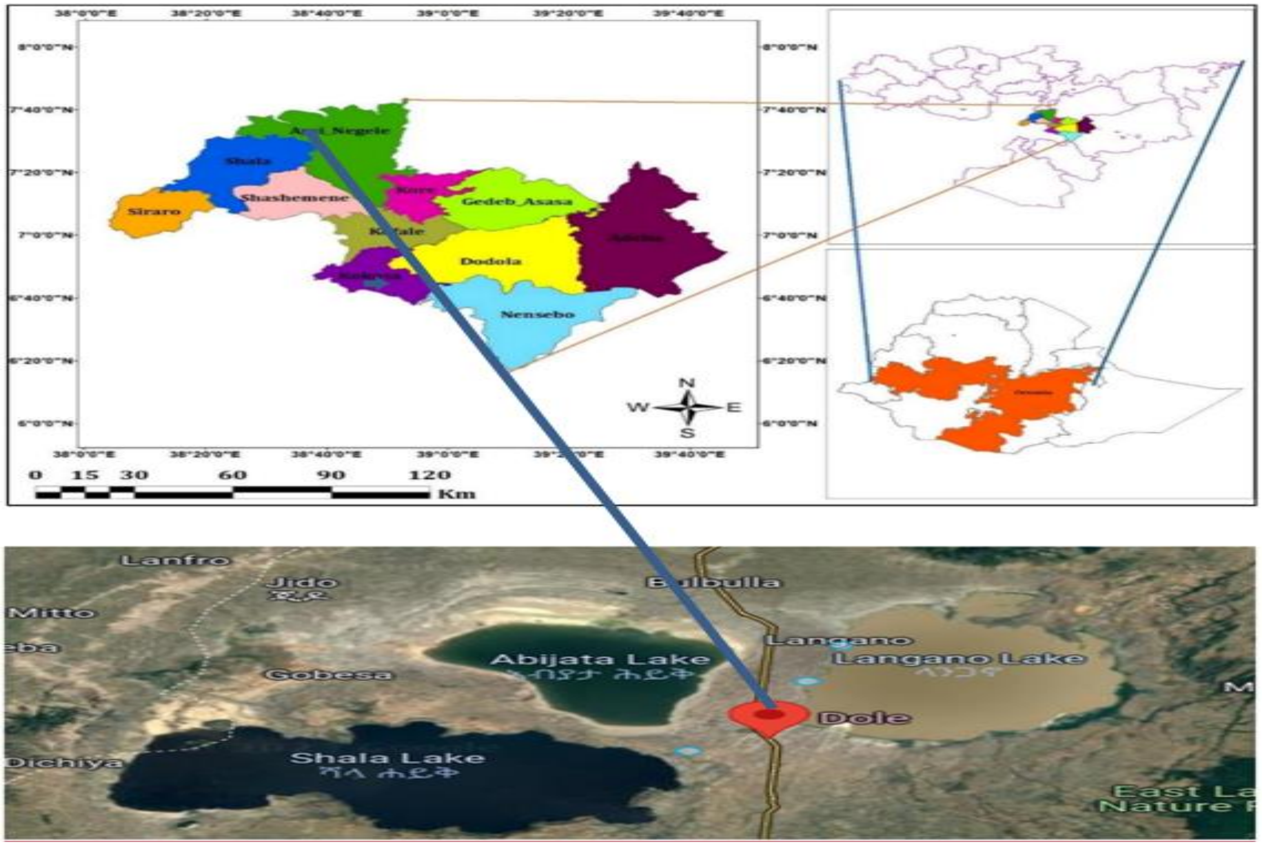


Figure 3. 1 Geographical location of Dole town (Source: Google map 2022).

Figure 3. 2 Map of Dole town and location of sample test-pits (Source: Google map 2022).

3.2 Study Locations and Visual Identification

This is conducted according to (ASTM D-2488) standard practice for identification and description of soil as shown below.

Table 3-1 Sampling location, visual identification and designation of sample test pits.

Ser.No	Location	Test Pit Designation	Depth (m)	Color	GPS Location			Description of Terrain
				Description	X	Y	Z	
1	Langano Lake Side-1(RHW)	TP-1	1.5		464436	830543	1619	Flat
			3.0	Brown				
2	Langano Lake Side-2 (LHW)	TP-2	1.5	Yellow & white	464135	831178	1626	Flat
			3.0	Brown				
3	Shala Lake Side (HC)	TP-3	1.5	Brown	463686	830389	1665	Flat
			3.0	White & yellow				
4	Abijeta Lake Side (GDWUO)	TP-4	1.5	White & yellow	463802	833338	1650	Flat
			3.0	Brown				
5	Kebele office Area	TP-5	1.5	Brown& yellow	463206	831303	1642	Flat
			3.0	Brown				

3.3 The Temperature and Rain Fall of The Area

The climate of the area is with a mean annual temperature of 19 degrees centigrade which in almost considered kola and a mean annual rainfall of 1200 to 1500m. The main rainy season in the area falls between mid-Julys to mid-September which is winter. While the lesser rains of autumn falls in June. Heavy and torrential rains that last from few minutes to several hours are common during winter season.

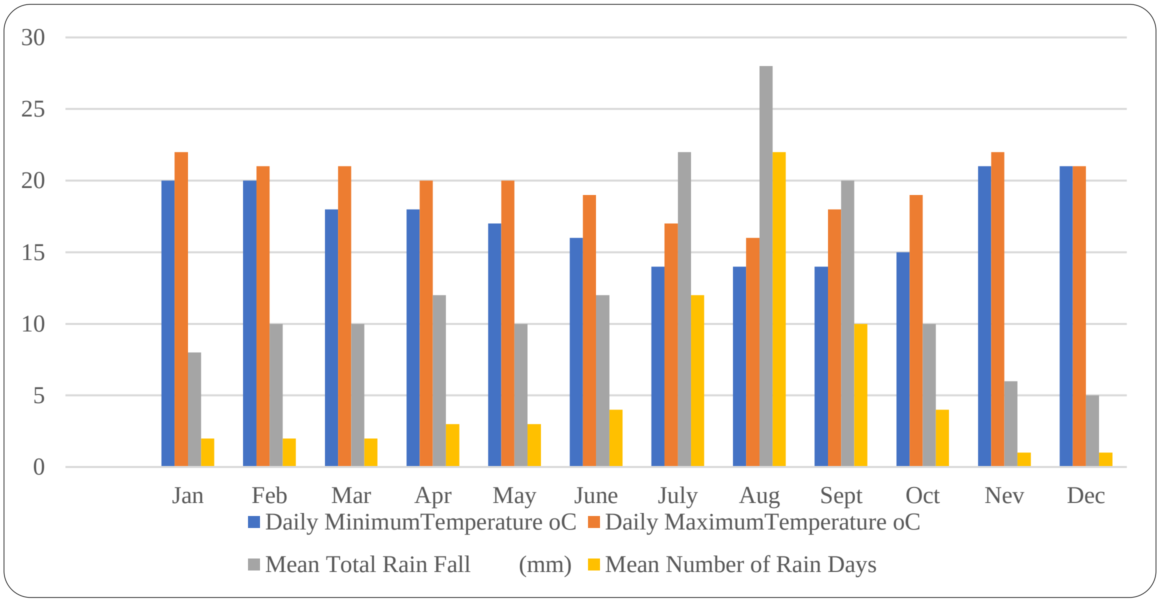


Figure 3.3 Meteorological data of Dole Town (2018-2024 years Recorded Data of Gubeta Arjo Metrological instruments)

3.4 Geology

3.4.1 General

The study area (Dole town) is included in part of central main Ethiopian rift valley lake basin. The area is located in weathered and fractured ignimbrite, pumice, and tuff and got ground water recharge from eastern high land. The geological situation significantly influences the engineering behavior of the area. The study area is free from ground crack as shown in figure 3.1. The main geologic formation of Dole town is the Pleistocene age eruption of volcanic rocks of the Dino Formation. Two earthquakes had their epicenter near this lake, the first in 1906 (a magnitude 6.8 on the Richter scale), and the second in 1985 (magnitude 6.2) (RARC and JICA, 2014; Woldegabriel et al., 1990).

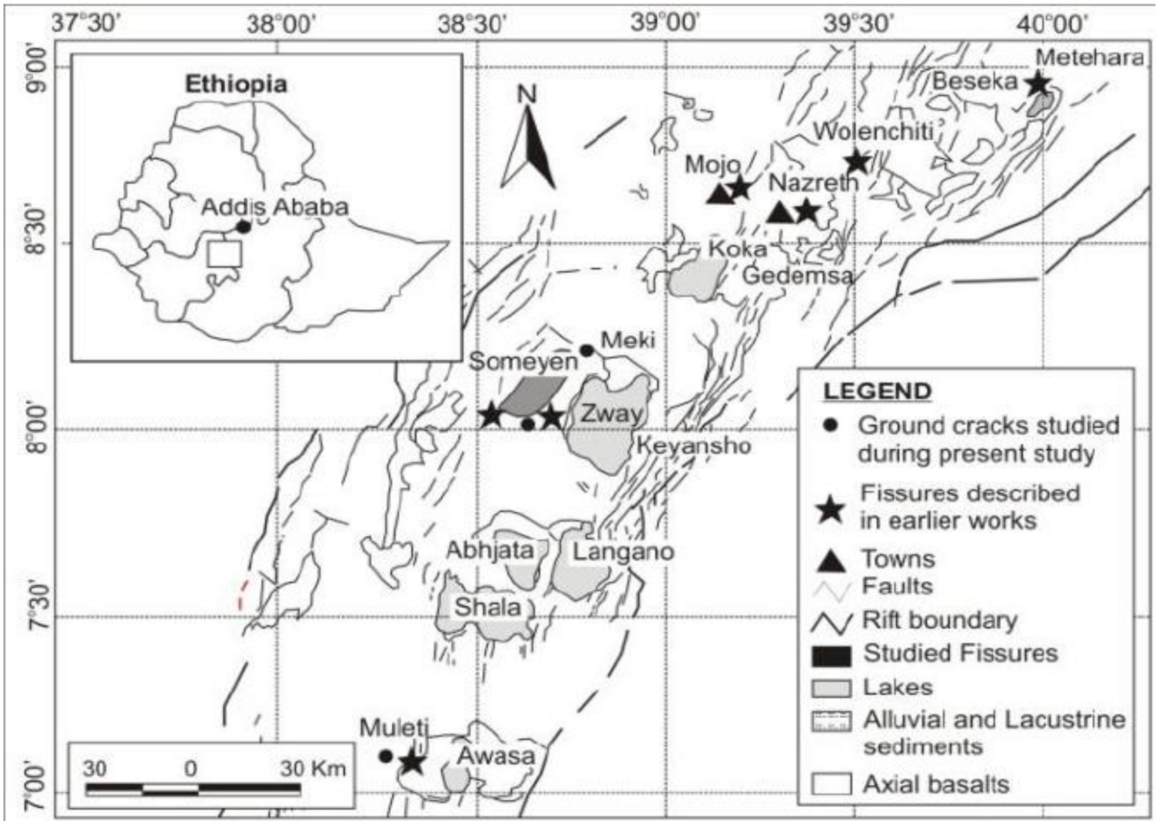


Figure 3. 4 Distribution of ground cracks in Ethiopia (IJEE, 2017)

3.5 Electrical Resistivity Test

Electrical resistivity is a property that shows how the flow of electric current is opposed in materials. Resistivity Consider an electrically uniform cube with side "L" and current "I" flowing through it. The material within the cube resists the conduction of current through it, resulting in a potential drop V between the opposite faces. The resistance R is proportional to the length L of the resistive material and inversely proportional to its area of cross section.

The proportionality constant, denoted by "ρ," is the "true" resistivity. Hence, the units of resistivity are the product of the resistance Ώ and a distance (area/length, meters) to give Ohm-meters (Ώ-m). The electrical resistivity of geological materials is given by the following relationships.

$$R = \frac{\rho L}{A} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.1)$$

$$\rho = \frac{RA}{L} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.2)$$

$$A = \frac{\pi d2}{4} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.3)$$

Where: R = Resistance, ρ = Resistivity, A = Area ($\pi D^2/4$), D = Diameter, L = length between two electrodes.

The inverse of resistivity (1/ ρ) is the conductivity σ which has units of siemens/meter (S/m) which are equivalent to mhos/m ($Ώ^{-1}m^{-1}$).

Length between two electrodes is one meter (1m), Diameter of the Cables holding electrodes is sixteen meters (16m). from these data we can calculate area. So, Area = $\pi d^2/4$ = $\pi\ (16)^2/4 = 200.96m^2$. Again, from the relationship of the resistance and resistivity, we can get the resistivity of the soil.

Figure 3. 5 Electrical resistivity test.

3.6 Study design

A study design/flow charge is the process that guides researchers on how to collect, analyze and Interpret observations for the samples. Therefore, the objective of the research is designing to Answer the research questions and meet its objectives based on experimental findings and the Study will be achieved in accordance with the methodology outlined below (Literature review-Formulating research objectives and question-Sampling and testing-Analysis and interpretation Results and Discussion Conclusion and Recommendation).

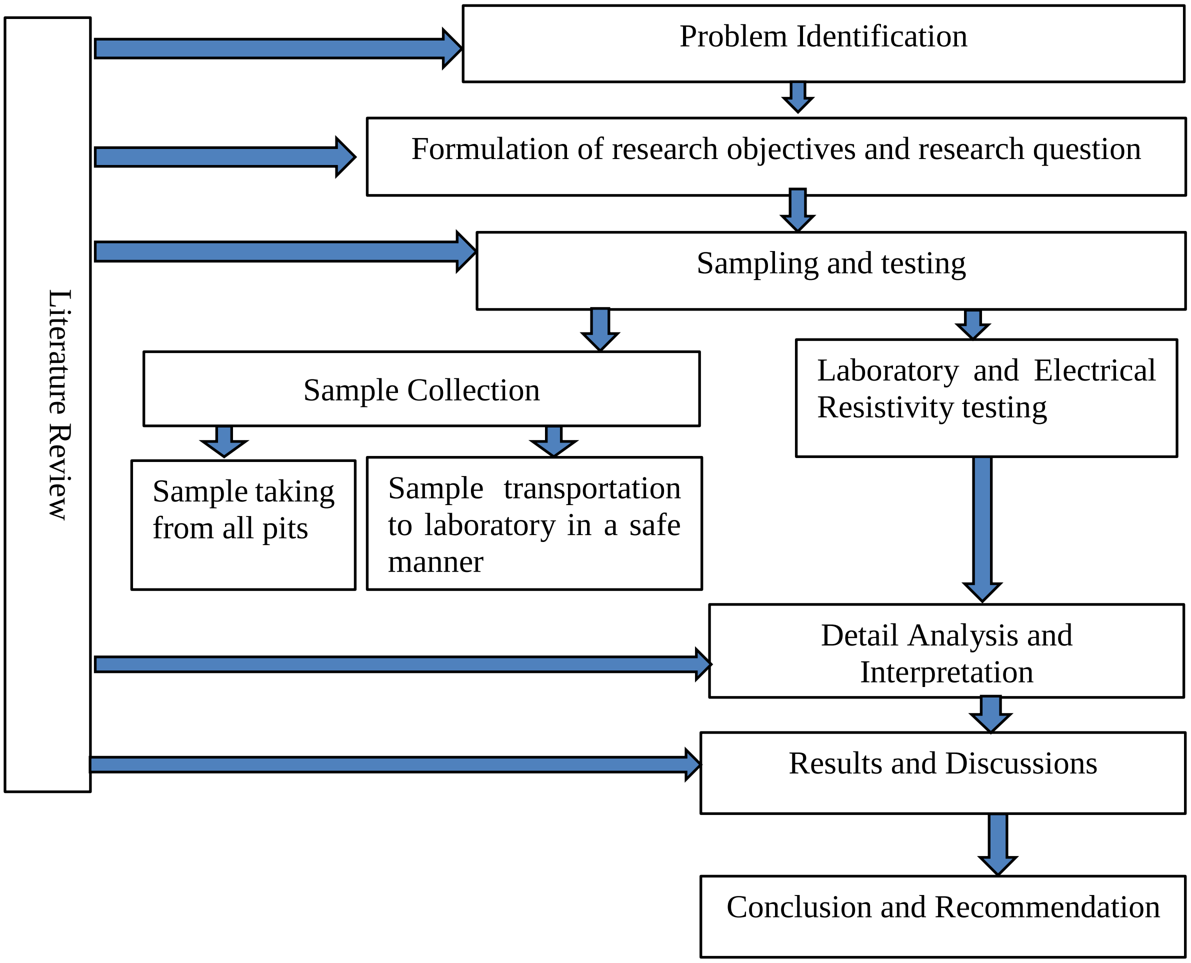


Figure 3. 6 Research design

3.7 Study Population

Decisions about the study population must be made early on in the design of any investigation. That is, with regard to the population of the specific units that will be examined. The soil that is taken using “random probability sampling method” from five (5) separate test pits, electrical resistivity data acquired from various locations throughout the study area, and a geological study of the region based on various researchers' theories make up the study population, according to this thesis. As a result, test pits and electrical resistivity checking points are regarded as the study population. This serves as a source of information during the data gathering and sampling procedures. In order to gather and analyze data, each sample was obtained from each test pit (population).

3.8 Study Variables

3.8.1 Dependent Variables

The dependent variables which was observed and measured to determine the effect of the independent variable is Investigation of soil for identifying its engineering properties”

3.8.2 Independent Variables

The independent variables which are to be measured and manipulated to determine its relationship to observed phenomena are; Moisture content, Gradation, LL, PL, PI, geophysical/electrical resistivity test.

3.9 Data Collection Process

3.9.1 Sample Size and Selection

For this study the soil samples were collected from five (5) representative test pits at depth of 1.5 m and 3 m. Different geotechnical laboratory tests were done for the collected soil samples and as per AASHTO and ASTM the results obtained were compared, analyzed and discussed thoroughly.

3.9.2 Sampling Techniques and Procedure: -

This study was conducted using a "random probability sampling method" for hand-digged pits. The undisturbed samples were collected using metallic square sampler (rigid metal ring) and specimen ring and disturbed soil samples for detailed laboratory testing were manually collected from five (5) subgrade test pits and taken to the Jimma University soil laboratory.

Figure 3. 7 Field visualizing, Color identification and collecting of soil samples

3.9.3 Field Survey

There was a preliminary visual survey performed during the field survey of the study area, as well as using engineering geological maps, topographic maps, and geographical maps. During this study, the soils surrounding the study area were visual identified, and field classifications such as texture, a simple strength test, and consistency were assessed in order to assess the soils.

3.9.4 Sample Preparation.

Prior to treatment and testing, the sample were prepared in accordance with the method described in AASHTO T87-86. This method involves air drying of samples and/or oven drying at 60ºC or less; breaking up the soil aggregates by rubber covered mallet. Then, sieve analysis is performed to separate the dried soils into two groups. The first group involves preparing uniform samples for Atterberg limits, the other for compaction tests. Based on the theories and laboratory tests performed, the results obtained are analyzed, compared and discussed. (Peter et al., 2020)

3.10 Data Processing and Analysis

In this research data were collected and organized both in numerical form and in words. To unify out the relevant information supplying questions and notes, quality control checking is executed. After sorting out the effective data, the numerical portion of the data were analyzed using Excel software, charts, tables, figures 2D and 3D modeling of electrical resistivity and other numerical problem-solving methodologies. A recommendation was given using the ERA manual and reference with different design standards (Prit A, Arora S, Das K, Sengupta D, Maity J, Student BT, 2018).

3.11 Ethical Consideration

The data were collected after ethical permission is given from concerned local administrative. The purpose of the study was clearly described to the organization and the concerned local communities. After we reach on the agreement every necessary samples were taken. During the excavation of soils for data collection, the excavated area was backfilled according to the wish of the surrounding communities.

3.12 Data Quality Assurance

Quality Assurance manual for laboratory test and field work were prepared carefully in order to avoid error of data. The training is given for pit excavators to handle the pit carefully. Laboratory instruments are calibrated; at least three samples and two experiments are done for one laboratory test in order to avoid error of data and results.

3.13 Laboratory Tests

Various laboratory tests are conducted and used to identify engineering properties of the soil. Based on the sample taken from the site, laboratory tests on the ten samples were conducted in Jimma University Institute of Technology and Collage of Agriculture and Veterinary Medicine. Accordingly, the following tests have been performed and for all tests the ASTM procedures are followed.

Table 3-2 Standards used for research, generally

Name of Test	Standard Code
Moisture Content	ASTM D-2216
Dry Sieve Analysis	ASTM D-421
Hydrometer Analysis	ASTM D-422
Atterberge Limit	ASTM D-4318
Specific Gravity	ASTM D-854
Standard Proctor Compaction	ASTM D-698
Direct Shear Test	ASTM D-3080
One Dimensional Consolidation Test	ASTM D-2435
Permeability Test	ASTM D-2434
Soil classification	ASTM D-2487 and AASHTO M-145

CHAPTER FOUR

4 RESULTS OF LABORATORY AND ELECTRICAL RESISTIVITY TESTS

4.1 Field Observations

For this study the soil samples were collected from Dole Town. Prior to sampling, visual site investigations and information from residents and construction firms were gathered in order to account for the various soil types and take representative samples evenly throughout the town. As a result, five sampling areas were chosen from various locations throughout the town to represent all of the soil types discovered during the preliminary site investigation. Disturbed and undisturbed samples were collected for this study and transported to Jimma institution of Technology Geotechnical laboratory for testing. Physical and mechanical soil testing were conducted for all samples taken, except for the Oedometer test, which was done only for three samples at a depth of three meters for TP-3, TP-4, and TP-5. The location of test pits is shown in Table 4.10

4.2 Natural Water Content

Table 4-1 Summary of natural water content of the study area

Ser. No	Location	Test Pit	Depth, m	Natural Moisture content, %
1	Langano Lake Side-1(RHW)	TP-1	1.5	40.3
			3.0	49.7
2	Langano Lake Side-2 (LHW)	TP-2	1.5	27.8
			3.0	31.6
3	Shala Lake Side (HC)	TP-3	1.5	13.0
			3.0	17.3
4	Abijeta Lake Side (GDWUO)	TP-4	1.5	12.9
			3.0	27.2
5	Kebele office Area	TP-5	1.5	19.4
			3.0	38.0

As shown in Table 4-1, the natural moisture content of Dole Town was taken from five test pits. From the test results, the water content of the study area ranges from 12.9% to 49.7%.

4.3 Specific Gravity

Table 4-2 Summary of natural Specific Gravity of the study area

Ser. No	Test Pit Designation	Depth, m	Specific Gravity
1	TP-1	1.5	2.39
		3.0	2.49
2	TP-2	1.5	2.45
		3.0	2.59
3	TP-3	1.5	2.55
		3.0	2.54
4	TP-4	1.5	2.49
		3.0	2.40
5	TP-5	1.5	2.54
		3.0	2.46

Table 4-2 shows the specific gravity of Dole Town. The test result of the test pits ranges from 2.39-2.59. This test results illustrate that the soil categorized as porous and or pumice materials.

4.4 Natural Unit Weights

Table 4-3 shows the natural unit weight of the study area. The dry unit weight of the study area which were taken from five test pits ranges from 9.60 KN/m^3 to 11.76 KN/m^3.

Table 4-3 Summary of Natural Unit Weight of the study area

Ser. No	Test Pit Designation	Depth, m	Bulk Unit Weight, kN/m^3	Dry Unit Weight, kN/m^3
1	TP-1	1.5	14.37	10.24
		3.0	14.37	9.60
2	TP-2	1.5	14.56	11.39
		3.0	14.56	11.06
3	TP-3	1.5	11.13	9.85
		3.0	13.79	11.76
4	TP-4	1.5	13.23	11.74
		3.0	11.44	8.98
5	TP-5	1.5	13.27	11.15
		3.0	14.69	10.36

4.5 Grain Size Analysis

Table 4-4 Sieve Analysis Test Result of the Study Area

Ser. No	Test Pit No	Depth, m	Coarse Material, %	Sandy Material, %	Percentage of Fine Material, %		Coefficient of curvature, Cc	Coefficient of Uniformity, Cu
					Silty	Clay		
1	TP-1	1.5	76.1	22.5	1.4		2.80	35.71
		3	82.3	16.2	1.5		2.50	15.63
2	TP-2	1.5	72.5	26	1.5		2.75	36.36
		3	76.2	19.2	4.6		4.45	36.36
3	TP-3	1.5	3.9	92.1	4		2.89	7.00
		3	10.83	57.6	29.36	2.21	0.67	28.00
4	TP-4	1.5	26.5	71.9	1.6		1.00	4.00
		3	7.55	45.1	40.8	6.45	0.35	38.33
5	TP-5	1.5	45.2	49.3	5.5		0.42	25.00
		3	31.5	38.5	29.12	0.88	0.39	100.00

Table 4-4 present the sieve analysis test result of Dole Town's soil specimens. From the above Table 4-4, the soil specimens of TP-1 and TP-2 were Coarse soil material (i.e., the cumulative grain size retained on 4.75mm sieve size in percent was greater than 50 percent). However, the TP-3, TP-4, and TP-5 grain size analyses indicated the soil specimens were sandy soil materials. That is, most particle sizes of the test pits were between 4.75mm and 0.075mm sieve sizes. Figure 4-1 Show the distribution grain size curve of the study area. Test pit 3, 4, and 5 at 3m had more than 5 percent fine particle passed through 0.075mm sieve size. Accordingly, combined tests (i.e., dry sieve analysis and hydrometer test analysis) had been conducted for those test pits. Hence, the distribution grain size curve of those test pits extended from 75mm to 0.0015mm as shown in the Figure 4-1 below.

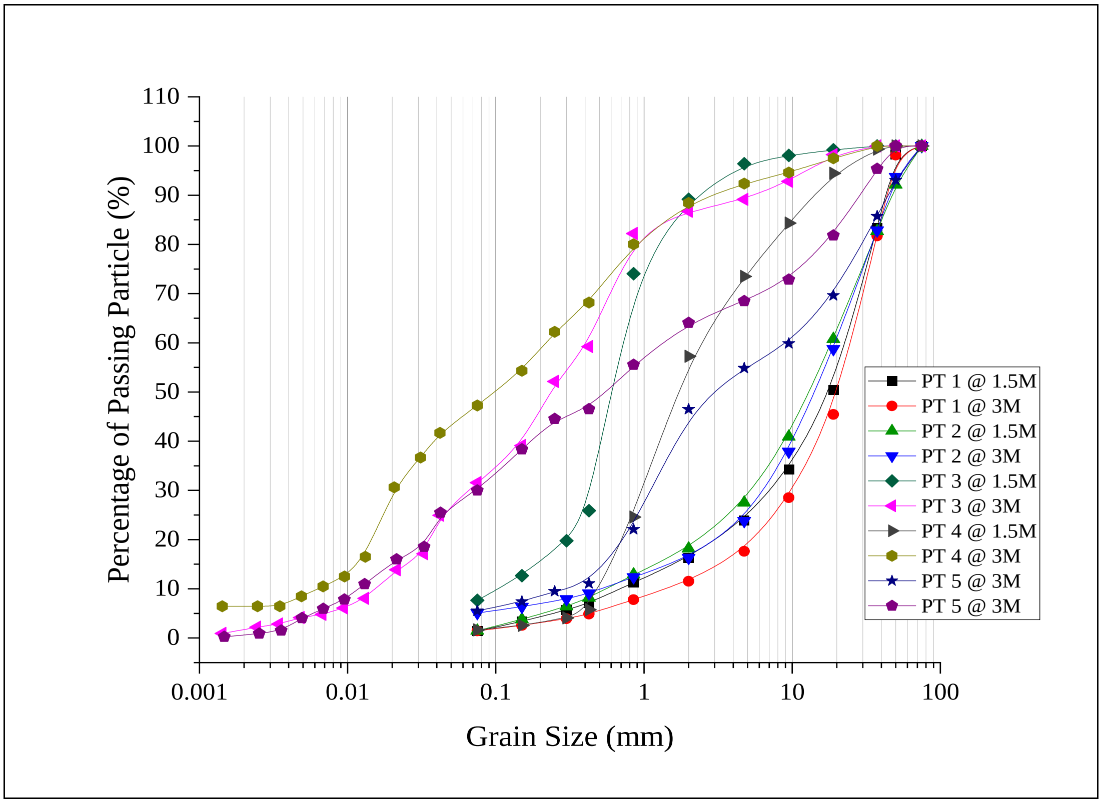


Figure 4-1 Grain size particle distribution curve

4.6 Atterberg Limits

Table 4-5 Atterberg Limit Test Result of the Study Area

Ser. No	Test Pit Designation	Depth, m	Liquid Limit, %	Plastic Limit, %	Plastic Index, %
1	TP-1	1.5	18.29	14.81	3.48
		3.0	22.12	17.72	4.40
2	TP-2	1.5	21.87	16.54	5.33
		3.0	20.82	17.72	3.10
3	TP-3	1.5	29.32	27.64	1.67
		3.0	34.64	32.58	2.06
4	TP-4	1.5	26.05	23.03	3.02
		3.0	37.82	34.79	3.04
5	TP-5	1.5	28.51	25.62	2.89
		3.0	32.64	28.75	3.89

As shown in Table 4-5, the liquid limit of Dole Town falls in the range of 18.29% to 37.82%. The plastic limit test result of the study area varies from 14.81% to 34.79%. The plasticity index range of the soils was below 7%. According to USCS soil classification, the consistency of the soil specimen was low plasticity. The laboratory test results of the Atterberge limits of test result shown in Table 4-5 and the detailed analysis of test results attached in Appendix IV.

4.7 Standard Proctor Compaction Test

Table 4-6 Summary of the Optimum moisture content and Maximum dry density of the study area

Test Pit	Depth, m	Maximum Dry Density, g/cm^3	Optimum Water Content, %
TP-1	1.5	0.89	30
	3.0	1.22	18.5
TP-2	1.5	1.09	43
	3.0	1.36	27
TP-3	1.5	1.55	19
	3.0	1.47	16
TP-4	1.5	1.22	8.5
	3.0	1.13	25
TP-5	1.5	1.37	14.5
	3.0	1.22	15

From the standard proctor compaction test results, the maximum dry density of the study area ranges from 0.89 g/cm^3 to 1.55 g/cm^3, and the optimum moisture content ranges from 8.50% to 43.0% as shown in Table 4-6. The laboratory test results of the standard proctor compaction test results shown in Table 4-6 and the detailed analysis of test results attached in Appendix VI.

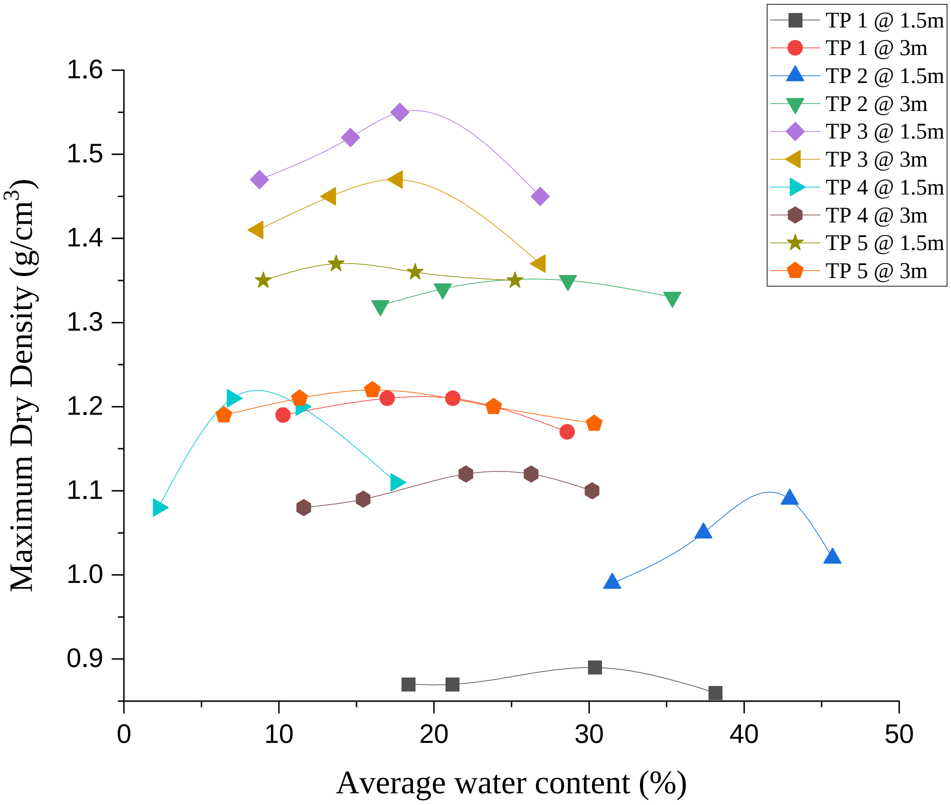


Figure 4-2 Dry Density Vs Moisture Content curve

4.8 Classification of Soils

4.8.1 Unified Soil Classification System (USCS)

The classification of soils according to the USCS scheme shows the soil of the study area was “Coarse-grained soils.” From the grain size analysis test result of the test pits, the soil particle passes through 0.075mm sieve size was less than 50%. Accordingly, the soil specimens were classified as “Coarse-grained soils” (i.e., gravelly and/ or sandy soils).

Table 4-7 USCS Soil Classification of the Study Area

Test Pit Designation	Depth, m	Gravel material, %	Sand material, %	Percentage of Fine Material		Cu	Cc	LL, %	PI, %	USCS Classification
				Silt, %	Clay, %					
TP-1	1.5	76.1	22.5	1.40		35.71	2.80	18.29	3.48	GW
	3.0	82.4	16.2	1.46		15.63	2.50	22.12	4.40	GW
TP-2	1.5	72.5	26.0	1.48		36.36	2.75	21.87	5.33	GW
	3.0	76.2	19.2	4.62		36.36	4.45	20.82	3.10	GW
TP-3	1.5	3.9	92.1	3.98		7.00	2.89	29.32	1.67	SW
	3.0	10.9	57.6	29.4	2.21	28.00	0.67	34.64	2.06	SM
TP-4	1.5	26.5	71.9	1.63		4.00	1.00	26.05	3.02	SP
	3.0	7.6	45.1	40.8	6.45	38.33	0.35	37.82	3.04	SM
TP-5	1.5	45.2	49.3	5.52		25.00	0.42	28.51	2.89	SP-SM
	3.0	31.5	38.5	29.1	0.88	100.0	0.39	32.64	3.89	SM

Table 4-7 shows the USCS soil classification of the Dole Town. Hence, the soil classification of TP-1 and TP-2 at a depth of 1.5m and 3m was 'Well-graded gravel with sand'. Whereas TP-3 at 1.5m it was 'Well-graded' sandy soil and TP-4 at a depth of 1.5m was 'Poor-graded' sand with gravel soil. However, TP-3 and 4 at a depth of 3m was silty sand, and TP-5 at a depth of 3m was Silty sand with gravel, soil classification as shown in Figure 4-3.

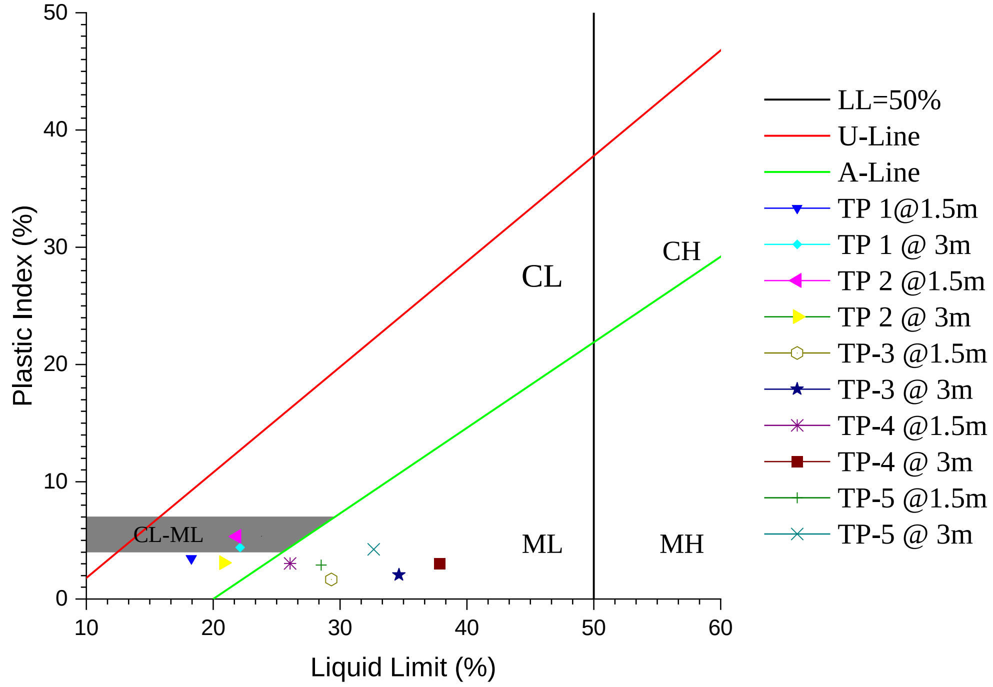


Figure 4-3 USCS plasticity chart of Dole Town

4.8.2 AASHTO Soil Classification System

According to the AASHTO Classification system results summarized in Table 4-8 and Figure 4-4, all the samples were classified as A-2-4. As a result, the soil samples were classified as "gravel and silty sand" soils with group index values of 0. For roadway subgrade material, the typical subgrade rating of the study area soil is 'Excellent to Good. Figure 4-4 show the AASHTO classification of Dole town soils for the collected samples.

Table 4-8 Classification of Soils Based on AASHTO Classification System

T P	Depth (m)	Grain size analysis (Percentage Passing)			LL, %	PI, %	GI	AASHTO Classification	Remark
		#10	#40	#200					
TP-1	1.5	16.3	7.1	1.4	18.3	3.5	0	A-2-4	Gravel
	3.0	11.5	4.9	1.5	22.1	4.4	0	A-2-4	Gravel
TP-2	1.5	18.2	8.2	1.5	21.9	5.3	0	A-2-4	Gravel
	3.0	16.3	9.1	4.6	20.8	3.1	0	A-2-4	Gravel
TP-3	1.5	88.8	25.6	4.0	29.3	1.7	0	A-2-4	Sand
	3.0	86.7	59.2	31.6	34.6	2.1	0	A-2-4	Silty sand
TP-4	1.5	57.3	5.7	1.6	26.1	3.0	0	A-2-4	Sand
	3.0	88.4	68.2	47.3	37.8	3.0	0	A-2-4	Silty sand
TP-5	1.5	46.5	11.1	5.5	28.5	2.9	0	A-2-4	Silty sand
	3.0	64.0	46.5	30.0	32.6	3.9	0	A-2-4	Silty sand

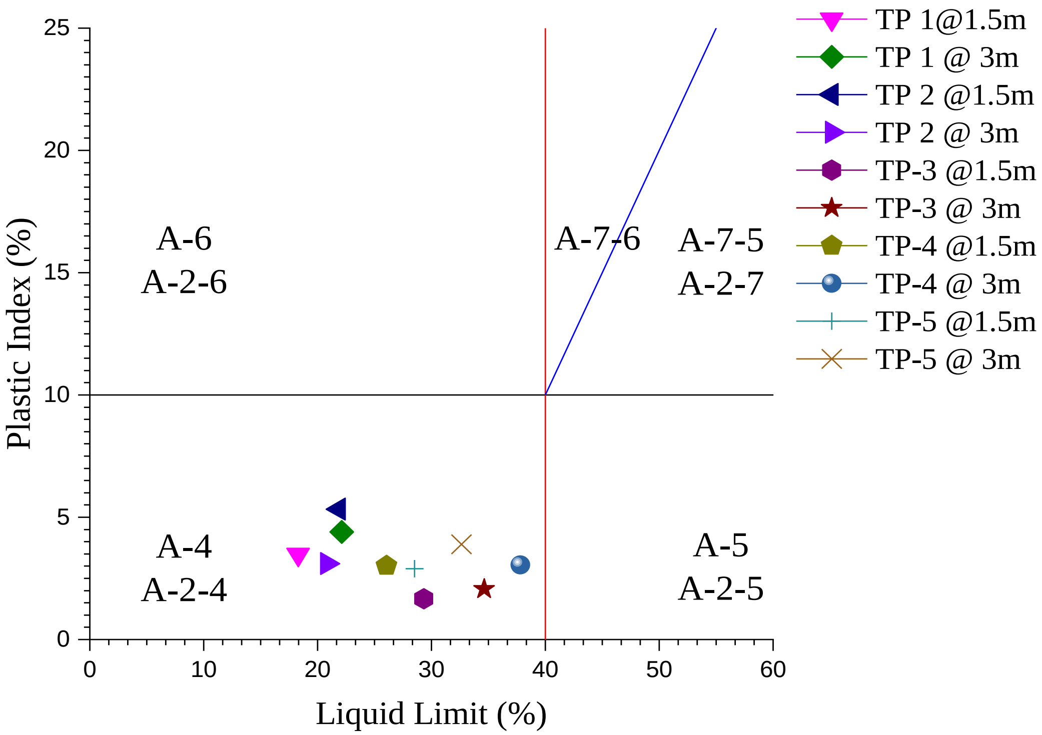


Figure 4-4 AASHTO Plasticity Chart Classification for the Dole Town Study Area

4.9 Direct Shear Test

According to the USCS and AASHTO soil classification system, the soil specimens collected from Dole Town were sandy soil and gravel soil. Hence, the shear strength of the study area was conducted using the direct shear test method. Therefore, the direct shear test results of the test pits are summarized in the Table 4-9 below.

From Table 4-9, the cohesion (Cu) of the Dole Town ranges from 5.7 KN/m^2 to 45.8 KN/m^2 and internal frictional angle ($Ø^o$) of the study area were from 26.4^o to 43.8^o. According (Das and Sobhan, 2018) internal frictional angle soil classification, test pit 1 and 2 are "Gravel with some sand". For test pits 3-5 the soil classifications are, at 1.5-meter depth, "loose to medium sands" and at 3-meter depth "sands with silty soils". This indicates that the shear strength of the study area varies along in horizontal and vertical from place to place. The laboratory test results of the direct shear test results shown in Table 4-9 and the detailed analysis of test results attached in Appendix VII.

Table 4-9 Summary of Direct Shear Test for Angle of Friction and Cohesion for Dole Town

Test Pit	Depth, m	Cohesion, kN/m^2 (Cu)	Angle of Friction, Ø^o
TP-1	1.5	28.2	43.8
	3	32.6	42.5
TP-2	1.5	45.8	33.4
	3	31.6	36.7
TP-3	1.5	5.7	31.2
	3	16.5	33.8
TP-4	1.5	6.1	35.7
	3	29.2	27.1
TP-5	1.5	21.8	36.7
	3	30.3	26.4

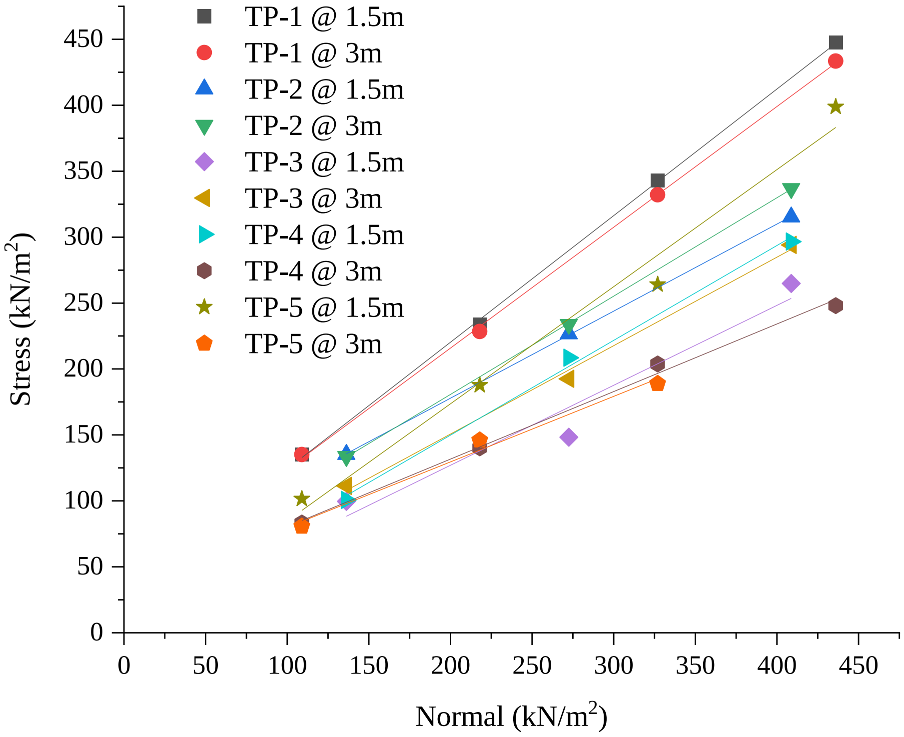


Figure 4-5 Shear stress Vs Normal stress curve for the study area

4.10 Results of Consolidation Test

4.10.1 Pressure–Void Ratio Curve

The pressure-void ratio curve can be obtained if the void ratio of the sample at the end of each increment of load is determined. The basic data used to determine this curve are natural moisture content, Specific gravity, density, cross sectional area and height of the sample, initial void ratio and applied loads. From these curve important parameters such as coefficient of compressibility (av), compression indexes (Cc), Swelling index (Cs) and pre-consolidation pressure (pc) are determined. The summary of test results is presented in Table 4.10 and Figure 4-6 below.

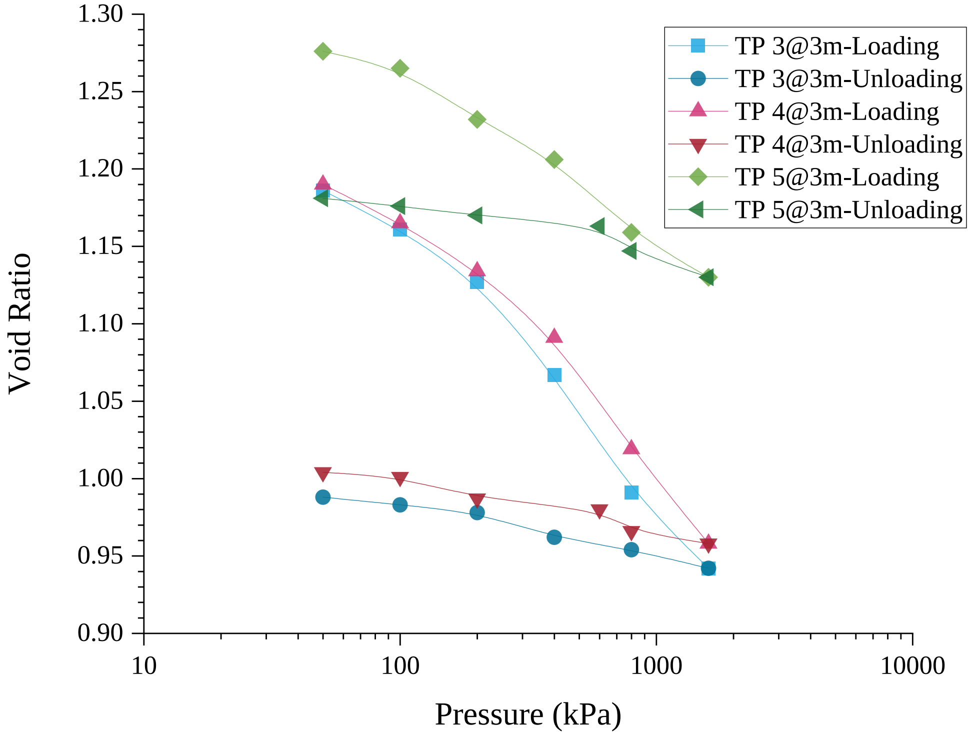


Figure 4-6 Plot of Void ratio and log Pressure curve for TP-3, TP-4, and TP-5 at 3m depth

4.10.2 Pre-Consolidation Pressure

Pre-consolidation pressure was determined from void ratio versus log pressure curve by the simplified method and Casagrande's method. In this study the pre-consolidation pressure and over consolidation ratio of PT-3, PT-4, and PT-5 at 3m depth was conducted as shown in Figure 4-7 and the remaining are indicated in Appendix VIII.

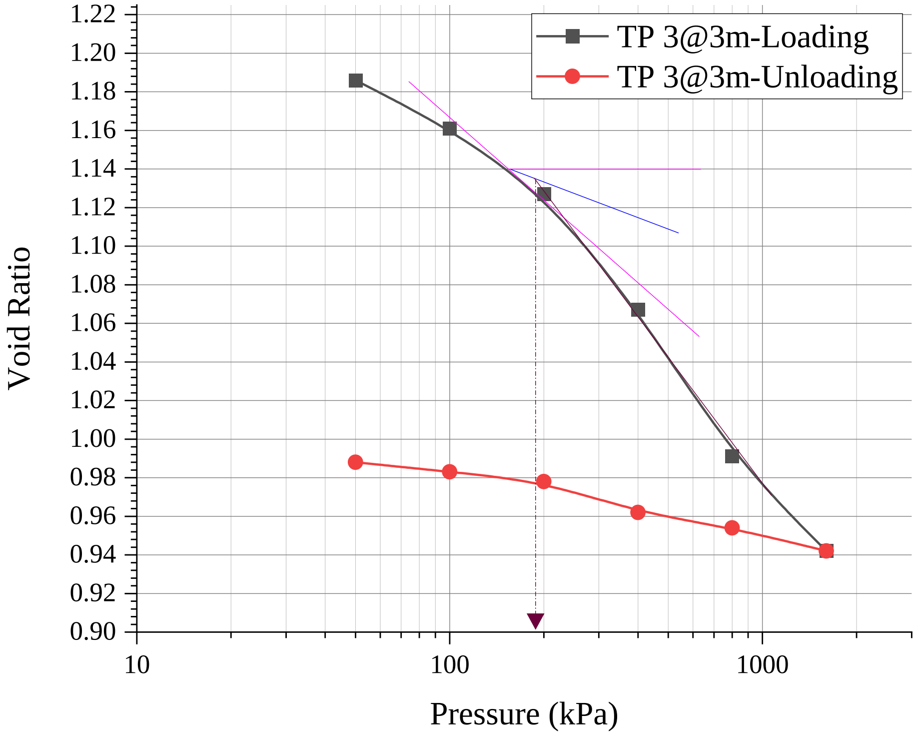


Figure 4-7 Typical example of Void ratio Vs log pressure curve plot of TP-3

Table 4-10 Summary of void ratio, pre-consolidation, and over consolidation ratio for TP-3, TP-4, and TP-5 at 3m depth

Test pit	Total unit Weight (kN/m^3)	Pressure (KPa)	Void Ratio (e)	Compression Index (Cc)	Swelling Index (Cs)	Over burden pressure (kPa)	Pre-Consolidation Pressure (kPa)	Over Consolidation Ratio (OCR)
TP-3 at 3m depth	13.79	7	1.20	0.226	0.0395	41.38	180.00	4.35
		50	1.186					
		100	1.161					
		200	1.127					
		400	1.067					
		800	0.991					
		1600	0.942					
		800	0.954					
		400	0.962					
		200	0.978					
		100	0.983					
		50	0.988					
TP-4 at 3m depth	11.44	7	1.22	0.421	0.0363	34.33	270.00	7.87
		50	0.035					
		100	0.060					
		200	0.091					
		400	0.133					
		800	0.206					
		1600	0.267					
		800	0.259					
		400	0.245					

		200	0.238					
		100	0.224					
		50	0.221					
TP-5 at 3m depth	14.69	7	1.28	0.122	0.0742	44.07	120.00	2.72
		50	1.276					
		100	1.265					
		200	1.232					
		400	1.206					
		800	1.159					
		1600	1.130					
		800	1.147					
		400	1.163					
		200	1.170					
		100	1.176					
		50	1.181					

Because the soils' over-consolidation ratios were greater than one, the soil specimens in the study area were over-consolidated in their natural form, as shown in Table 4-10.

The compression and swelling indexes of the soils were determined using the straight-line parts of the void ratio against log pressure curves shows in Figure 4-7 for TP-3 at 3m. The compression index (Cc) ranged from 0.122 to 0.421.

The swelling index (Cs) of the test pits varied from 0.0363 to 0.0742, as shown in Table 4-10. Cc often varies from 0.1 to 10, with no units. The index for normally consolidated clays typically ranges between 0.20 and 0.50, and for silts, between 0.16 and 0.24. The index for sands varies from 0.01 to 0.06 (Widodo and Ibrahim, 2012). Yet the test pits conducted for the one-dimensional consolidation test were sandy with silty soil; Cc were failed under normally consolidated clay soil. However, because the soil was lightweight sandy with silt, the Cc of the test pits increased. The swelling index (Cs) of the test pits were observed as low value. This indicates that, the soils swelling potential is low.

4.10.3 Coefficient of Consolidation, Coefficient of compressibility, and Coefficient of permeability

The logarithm of the time-fitting approach was used to obtain the samples' coefficients of consolidation (Cv) and compressibility (av). The Cv and av were then determined for each incremental load using the logarithm of time fitting approach. The permeability coefficient (k) of the soil under investigation, determined from the consolidation test results, was also presented, as stated in the Table 4-11 below.

Table 4-11 Coefficients of consolidation (Cv), compressibility (av), and permeability (k)

Pressure (KPa)	TP-3 at 3m depth			TP-4 at 3m depth			TP-5 at 3m depth		
	Cv, 10^{-2} (cm^2/sec)	av, 10^{-4} (m^2/kN)	k, 10^{-4} (cm/s)	Cv, 10^{-2} (cm2/sec)	av, 10^{-4} (m2/kN)	k, 10^{-4} (cm/s)	Cv, 10^{-2} (cm2/sec)	av, 10^{-4} (m2/kN)	k, 10^{-4} (cm/s)
50	0.016	3.35	0.24	0.127	2.37	1.35	0.082	1.00	0.36
100	0.035	4.88	0.78	0.012	3.72	0.20	0.033	1.66	0.23
200	0.073	3.45	1.16	0.004	3.43	0.07	0.017	2.50	0.18
400	0.032	3.01	0.46	0.012	2.76	0.16	0.019	1.89	0.16
800	0.013	1.90	0.12	0.009	2.28	0.10	0.007	1.53	0.05
1600	0.280	0.61	0.86	0.013	1.52	0.10	0.013	0.94	0.06
800	0.278	0.15	0.20	0.056	2.95	0.83	0.029	1.67	0.22
400	0.141	0.20	0.14	0.004	3.71	0.08	0.368	1.97	3.28
200	0.006	0.80	0.02	0.003	11.04	0.17	0.027	5.68	0.69
100	0.006	0.56	0.02	0.002	21.41	0.23	0.020	11.21	1.00
50	0.012	0.90	0.05	0.002	45.53	0.51	0.015	23.13	1.55

4.11 Permeability

Based on the soil's permeability coefficient, water can move through it. Since this test is suitable for both fine-grained and coarse soil, the researcher employs the falling head permeability test. The remaining test pits produced findings that were equivalent to either of these test pits, and representative permeability tests were conducted on three disturbed samples in three distinct test pits. For TP-1, TP-4, and TP-5, respectively, the coefficient of permeability was $1.14*10^{-3}$, $2.87*10^{-4}$, and $1.84*10^{-3}$ cm/sec. The relationship between the void ratio and log coefficient of permeability in the graph for the three-test pit taken from the study area is almost straight line as shown in Figure 4-8.

High degree of permeability: k is over 0.1 cm/s; medium degree of permeability: k is between 0.1 and 0.001 cm/s; low degree of permeability: k is between 0.001 and $1*10^{-5}$ cm/s. The very low permeability, k, is between $1*10^{-5}$ and $1*10^{-7}$ cm/s. Practically impermeable, k is less than $1*10^{-7}$ cm/s" (Haile, 2014).

The average of the logarithm of time method was used to calculate the consolidation coefficient. The range of values of the coefficient of permeability lies between $1.14*10^{-3}$ and $2.87*10^{-4}$ cm/sec, which indicates that the soils are practically low permeable soils.

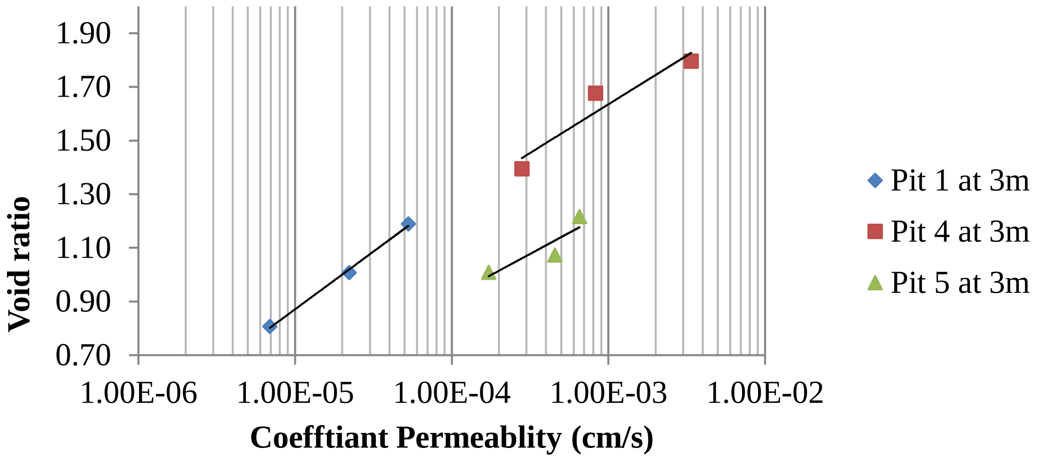


Figure 4-8 Void ratio Vs log coefficient of permeability (k)

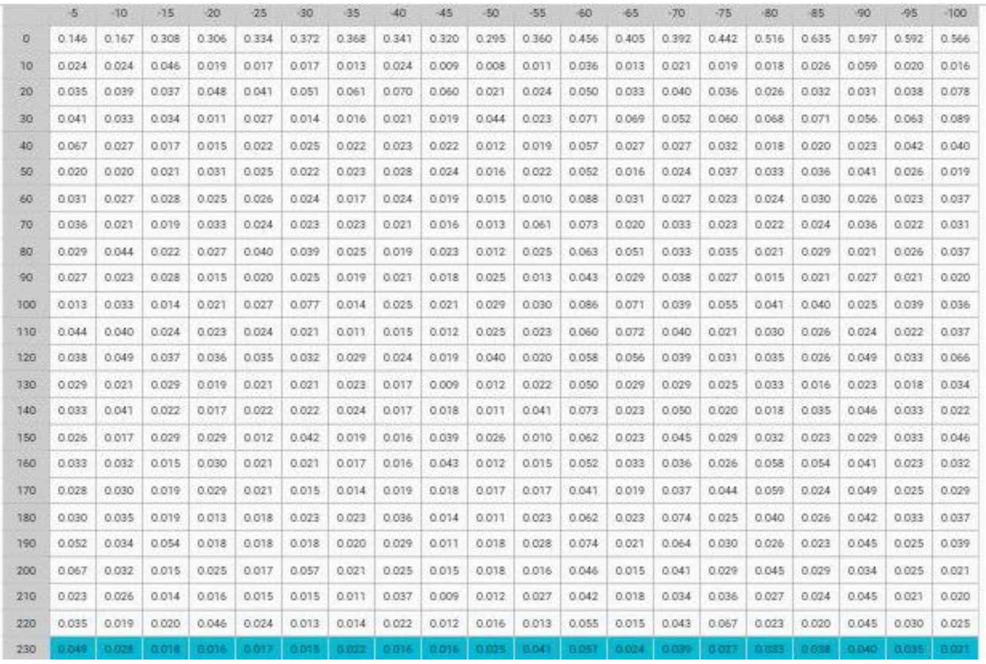
4.12 Electrical Resistivity Test

Table 4-12 Electrical resistivity results.

No	Test Pit	Max/Min	Resistance (Ω/m)	Resistivity (Ώ.m)
1	Pit1.	Maximum	0.051	10.25
		Minimum	0.016	3.22
2	Pit2.	Maximum	0.071	14.27
		Minimum	0.016	3.22
3	Pit3.	Maximum	1.541	368.56
		Minimum	0.348	69.93
4	Pit4.	Maximum	0.074	14.87
		Minimum	0.025	5.024
5	Pit5.	Maximum	0.098	19.69
		Minimum	0.021	4.22

From table 4-12 in our study area, resistance gained from test pit1 is from 0.016 to 0.051(Ώ/m) and its resistivity is from 3.22 to 10.25(Ώ.m), resistance gained from test pit2 is from 0.016 to 0.071(Ώ/m) and its resistivity is from 3.22 to 14.27(Ώ.m), resistance gained from test pit3 is from 0.348 to 1.541(Ώ/m) and its resistivity is from 69.93 to 368.58(Ώ.m), resistance gained from test pit4 is from 0.025 to 0.074(Ώ/m) and its resistivity is from 5.024 to 14.87(Ώ.m), resistance gained from test pit5 is from 0.021 to 0.98(Ώ/m) and its resistivity is from 4.22 to 19.69(Ώ.m). Generally, as it can be seen from Table 4-13 up to 4-16 its resistivity is from 3. 22Ώ.m to 368. 56Ώ.m. The results show considerable similarities with previous findings such asTsegaselassie ,2013)

Table 4-13 Resistance Value of Dole Test Pit 1



	-5	-10	-15	-20	-25	-30	-35	-40	-45	-50	-55	-60	-65	-70	-75	-80	-85	-90	-95	-100
0	0.146	0.167	0.308	0.306	0.334	0.372	0.368	0.341	0.320	0.295	0.360	0.456	0.405	0.392	0.442	0.516	0.635	0.597	0.592	0.566
10	0.024	0.024	0.046	0.019	0.017	0.017	0.013	0.024	0.009	0.008	0.011	0.036	0.013	0.021	0.019	0.018	0.026	0.059	0.020	0.016
20	0.035	0.039	0.037	0.048	0.041	0.051	0.061	0.070	0.060	0.021	0.024	0.050	0.033	0.040	0.036	0.026	0.032	0.031	0.038	0.078
30	0.041	0.033	0.034	0.011	0.027	0.014	0.016	0.021	0.019	0.044	0.023	0.071	0.069	0.052	0.060	0.068	0.071	0.056	0.063	0.089
40	0.067	0.027	0.017	0.015	0.022	0.025	0.022	0.023	0.022	0.012	0.019	0.057	0.027	0.027	0.032	0.018	0.020	0.023	0.042	0.040
50	0.020	0.020	0.021	0.031	0.025	0.022	0.023	0.028	0.024	0.016	0.022	0.052	0.016	0.024	0.037	0.033	0.036	0.041	0.026	0.019
60	0.031	0.027	0.028	0.025	0.026	0.024	0.017	0.024	0.019	0.015	0.010	0.088	0.031	0.027	0.023	0.024	0.030	0.026	0.023	0.037
70	0.036	0.021	0.019	0.033	0.024	0.023	0.023	0.021	0.016	0.013	0.061	0.073	0.020	0.033	0.023	0.022	0.024	0.036	0.022	0.031
80	0.029	0.044	0.022	0.027	0.040	0.039	0.025	0.019	0.023	0.012	0.025	0.063	0.051	0.033	0.035	0.021	0.029	0.021	0.026	0.037
90	0.027	0.023	0.028	0.015	0.020	0.025	0.019	0.021	0.018	0.025	0.013	0.043	0.029	0.038	0.027	0.015	0.021	0.027	0.021	0.020
100	0.013	0.033	0.014	0.021	0.027	0.077	0.014	0.025	0.021	0.029	0.030	0.086	0.071	0.039	0.055	0.041	0.040	0.025	0.039	0.036
110	0.044	0.040	0.024	0.023	0.024	0.021	0.011	0.015	0.012	0.025	0.023	0.060	0.072	0.040	0.021	0.030	0.026	0.024	0.022	0.037
120	0.038	0.049	0.037	0.036	0.035	0.032	0.029	0.024	0.019	0.040	0.020	0.058	0.056	0.039	0.031	0.035	0.026	0.049	0.033	0.066
130	0.029	0.021	0.029	0.019	0.021	0.021	0.023	0.017	0.009	0.012	0.022	0.050	0.029	0.029	0.025	0.033	0.016	0.023	0.018	0.034
140	0.033	0.041	0.022	0.017	0.022	0.022	0.024	0.017	0.018	0.011	0.041	0.073	0.023	0.050	0.020	0.018	0.035	0.046	0.033	0.022
150	0.026	0.017	0.029	0.029	0.012	0.042	0.019	0.016	0.039	0.026	0.010	0.062	0.023	0.045	0.029	0.032	0.023	0.029	0.033	0.046
160	0.033	0.032	0.015	0.030	0.021	0.021	0.017	0.016	0.043	0.012	0.015	0.052	0.033	0.036	0.026	0.058	0.054	0.041	0.023	0.032
170	0.028	0.030	0.019	0.029	0.021	0.015	0.014	0.019	0.018	0.017	0.017	0.041	0.019	0.037	0.044	0.059	0.024	0.049	0.025	0.029
180	0.030	0.035	0.019	0.013	0.018	0.023	0.023	0.036	0.014	0.011	0.023	0.062	0.023	0.074	0.025	0.040	0.026	0.042	0.033	0.037
190	0.052	0.034	0.054	0.018	0.018	0.018	0.020	0.029	0.011	0.018	0.028	0.074	0.021	0.064	0.030	0.026	0.023	0.045	0.025	0.039
200	0.067	0.032	0.015	0.025	0.017	0.057	0.021	0.025	0.015	0.018	0.016	0.046	0.015	0.041	0.029	0.045	0.029	0.034	0.025	0.021
210	0.023	0.026	0.014	0.016	0.015	0.015	0.011	0.037	0.009	0.012	0.027	0.042	0.018	0.034	0.036	0.027	0.024	0.045	0.021	0.020
220	0.035	0.019	0.020	0.046	0.024	0.013	0.014	0.022	0.012	0.016	0.013	0.055	0.015	0.043	0.067	0.023	0.020	0.045	0.030	0.025
230	0.049	0.028	0.018	0.016	0.017	0.018	0.022	0.016	0.016	0.025	0.041	0.057	0.024	0.039	0.027	0.033	0.038	0.040	0.035	0.021

From the table 4-13 the minimum and maximum resistances are 0.016 and 0.051 respectively.

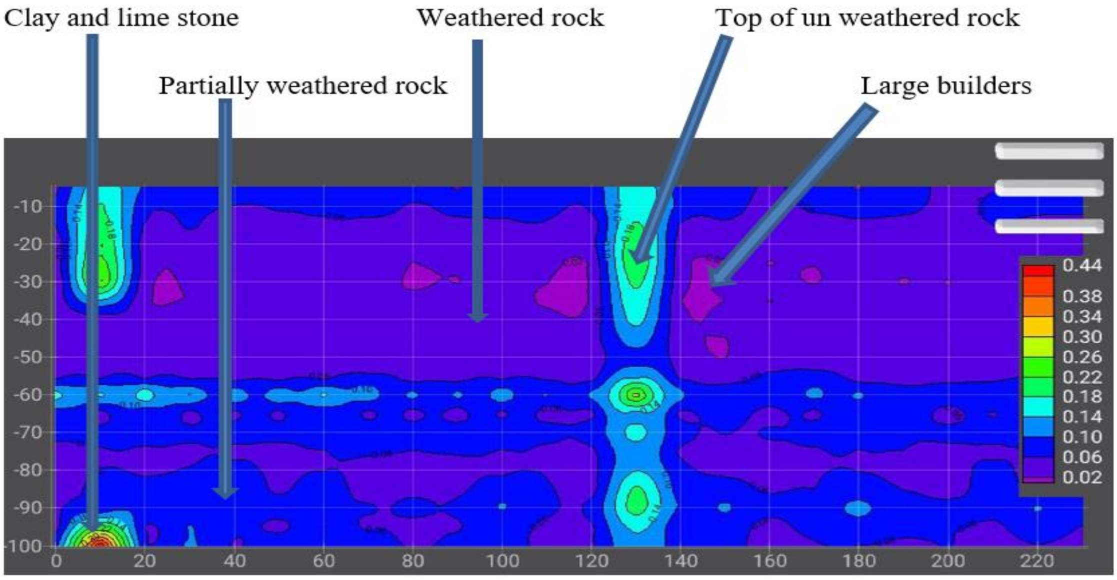


Figure 4-9 Two-dimension expression of Dole test pit1 resistivity

The two-dimensional figure 4-9 shows when we go down ward the type of soil around this area up to depth of 100m is almost the same type of soil which is weathered and un weathered rocks (gravel and sand) and contains some large builders, lime stone and some clay soils   Generally, it indicates that when we go from top to bottom there is no big difference on the type of soil.

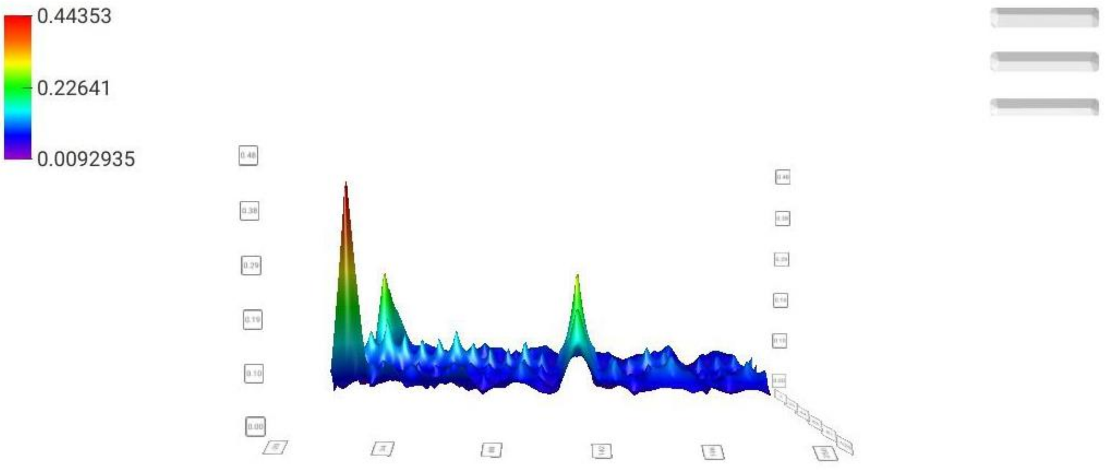
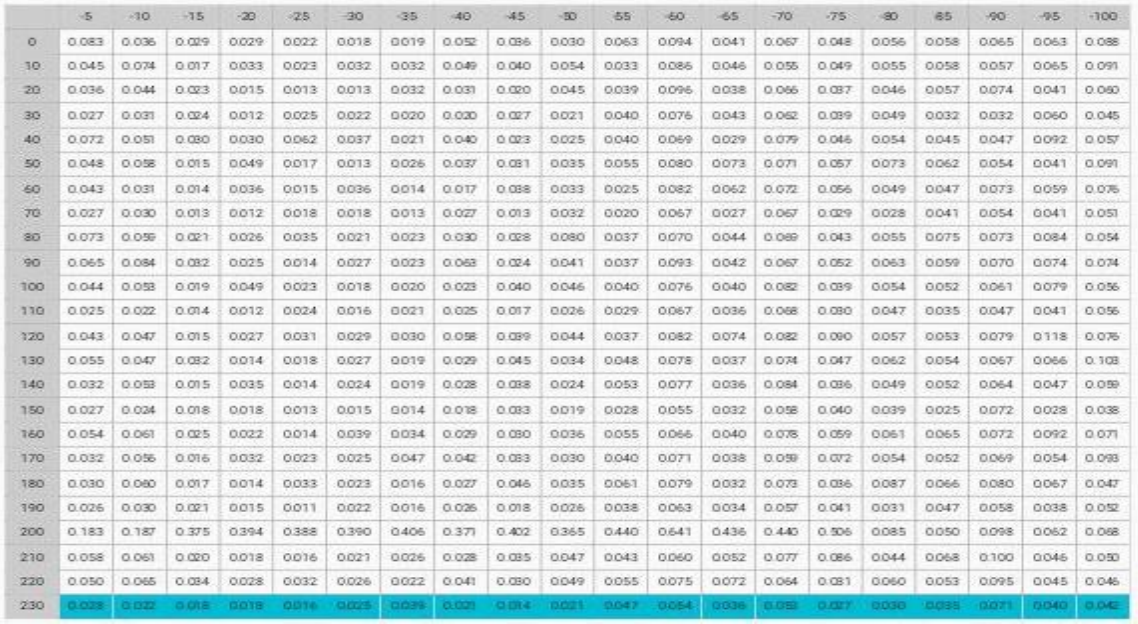


Figure 4-10 Three-dimension expression of Dole test pit1 resistivity

The three-dimensional figure 4-10 shows when we see the shape of the figure it is almost the same level or flat except some points that shows high resistance due to un weathered and builders rock. This indicates the soil around this area is almost the same in the horizontal radius of 200m.

Table 4-14 Resistance Value of Dole Test Pit 2



	-5	-10	-15	-20	-25	-30	-35	-40	-45	-50	-55	-60	-65	-70	-75	-80	85	-90	-95	-100
0	0.083	0.036	0.029	0.029	0.022	0.018	0.019	0.052	0.036	0.030	0.063	0.094	0.041	0.067	0.048	0.056	0.058	0.065	0.063	0.088
10	0.045	0.074	0.017	0.033	0.023	0.032	0.032	0.049	0.040	0.054	0.033	0.086	0.046	0.055	0.049	0.055	0.058	0.057	0.065	0.091
20	0.036	0.044	0.023	0.015	0.013	0.013	0.032	0.031	0.020	0.045	0.039	0.096	0.038	0.066	0.037	0.046	0.057	0.074	0.041	0.060
30	0.027	0.031	0.024	0.012	0.025	0.022	0.020	0.020	0.027	0.021	0.040	0.076	0.043	0.062	0.039	0.049	0.032	0.032	0.060	0.045
40	0.072	0.051	0.030	0.030	0.062	0.037	0.021	0.040	0.023	0.025	0.040	0.069	0.029	0.079	0.046	0.054	0.045	0.047	0.092	0.057
50	0.048	0.058	0.015	0.049	0.017	0.013	0.026	0.037	0.031	0.035	0.055	0.080	0.073	0.071	0.057	0.073	0.062	0.054	0.041	0.091
60	0.043	0.031	0.014	0.036	0.015	0.036	0.014	0.017	0.038	0.033	0.025	0.082	0.062	0.072	0.056	0.049	0.047	0.073	0.059	0.076
70	0.027	0.030	0.013	0.012	0.018	0.018	0.013	0.027	0.013	0.032	0.020	0.067	0.027	0.067	0.029	0.028	0.041	0.054	0.041	0.051
80	0.073	0.059	0.021	0.026	0.035	0.021	0.023	0.030	0.028	0.080	0.037	0.070	0.044	0.069	0.043	0.055	0.075	0.073	0.084	0.054
90	0.065	0.084	0.032	0.025	0.014	0.027	0.023	0.063	0.024	0.041	0.037	0.093	0.042	0.067	0.052	0.063	0.059	0.070	0.074	0.074
100	0.044	0.053	0.019	0.049	0.023	0.018	0.020	0.023	0.040	0.046	0.040	0.076	0.040	0.082	0.039	0.054	0.052	0.061	0.079	0.056
110	0.025	0.022	0.014	0.012	0.024	0.016	0.021	0.025	0.017	0.026	0.029	0.067	0.036	0.068	0.030	0.047	0.035	0.047	0.041	0.056
120	0.043	0.047	0.015	0.027	0.031	0.029	0.030	0.058	0.039	0.044	0.037	0.082	0.074	0.082	0.090	0.057	0.053	0.079	0.118	0.076
130	0.055	0.047	0.032	0.014	0.018	0.027	0.019	0.029	0.045	0.034	0.048	0.078	0.037	0.074	0.047	0.062	0.054	0.067	0.066	0.103
140	0.032	0.053	0.015	0.035	0.014	0.024	0.019	0.028	0.038	0.024	0.053	0.077	0.036	0.084	0.036	0.049	0.052	0.064	0.047	0.059
150	0.027	0.024	0.018	0.018	0.013	0.015	0.014	0.018	0.033	0.019	0.028	0.055	0.032	0.058	0.040	0.039	0.025	0.072	0.028	0.038
160	0.054	0.061	0.025	0.022	0.014	0.039	0.034	0.029	0.030	0.036	0.055	0.066	0.040	0.078	0.059	0.061	0.065	0.072	0.092	0.071
170	0.032	0.056	0.016	0.032	0.023	0.025	0.047	0.042	0.033	0.030	0.040	0.071	0.038	0.059	0.072	0.054	0.052	0.069	0.054	0.093
180	0.030	0.060	0.017	0.014	0.033	0.023	0.016	0.027	0.046	0.035	0.061	0.079	0.032	0.073	0.036	0.087	0.066	0.080	0.067	0.047
190	0.026	0.030	0.021	0.015	0.011	0.022	0.016	0.026	0.018	0.026	0.038	0.063	0.034	0.057	0.041	0.031	0.047	0.058	0.038	0.052
200	0.183	0.187	0.375	0.394	0.388	0.390	0.406	0.371	0.402	0.365	0.440	0.641	0.436	0.440	0.506	0.085	0.050	0.098	0.062	0.068
210	0.058	0.061	0.020	0.018	0.016	0.021	0.026	0.028	0.035	0.047	0.043	0.060	0.052	0.077	0.086	0.044	0.068	0.100	0.046	0.050
220	0.050	0.065	0.034	0.028	0.032	0.026	0.022	0.041	0.030	0.049	0.055	0.075	0.072	0.064	0.081	0.060	0.053	0.095	0.045	0.046
230	0.028	0.022	0.018	0.018	0.016	0.025	0.039	0.021	0.014	0.021	0.047	0.054	0.036	0.053	0.027	0.030	0.035	0.071	0.040	0.042

From the table 4-14 the minimum and maximum resistances are 0.016 and 0.071 respectively.

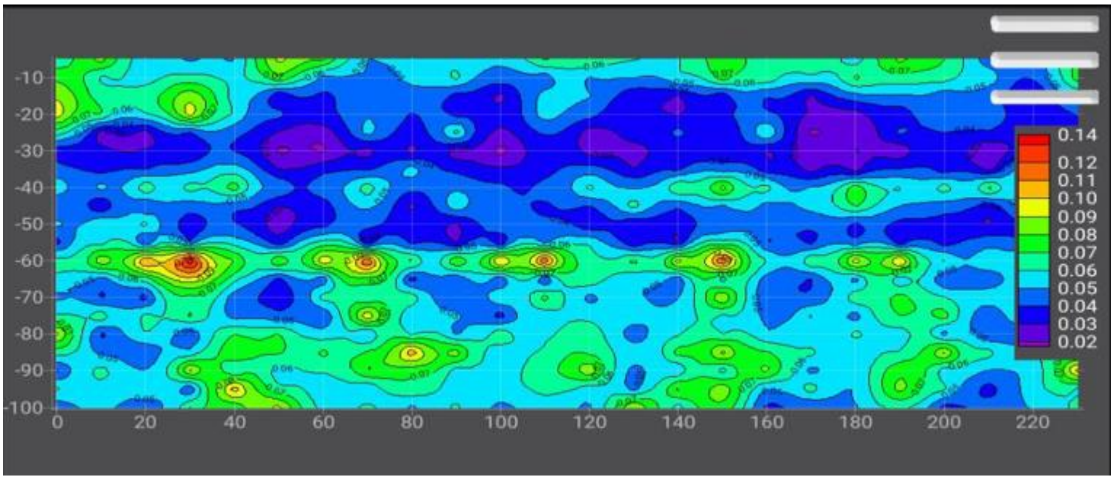


Figure 4-11 Two-dimension expression of Dole test pit2 resistivity

The two-dimensional figure 4-11 shows when we go down ward the type of soil around this area up to depth of 50m is almost the same type of soil which is weathered rocks, un weathered rocks and builder and contains some large builders, lime stone and some clay soils but after 50m the majority of the soil is un weathered and contains some clay and lime stone. Generally, it indicates that when we go from top to bottom there is no big difference on the type of soil.

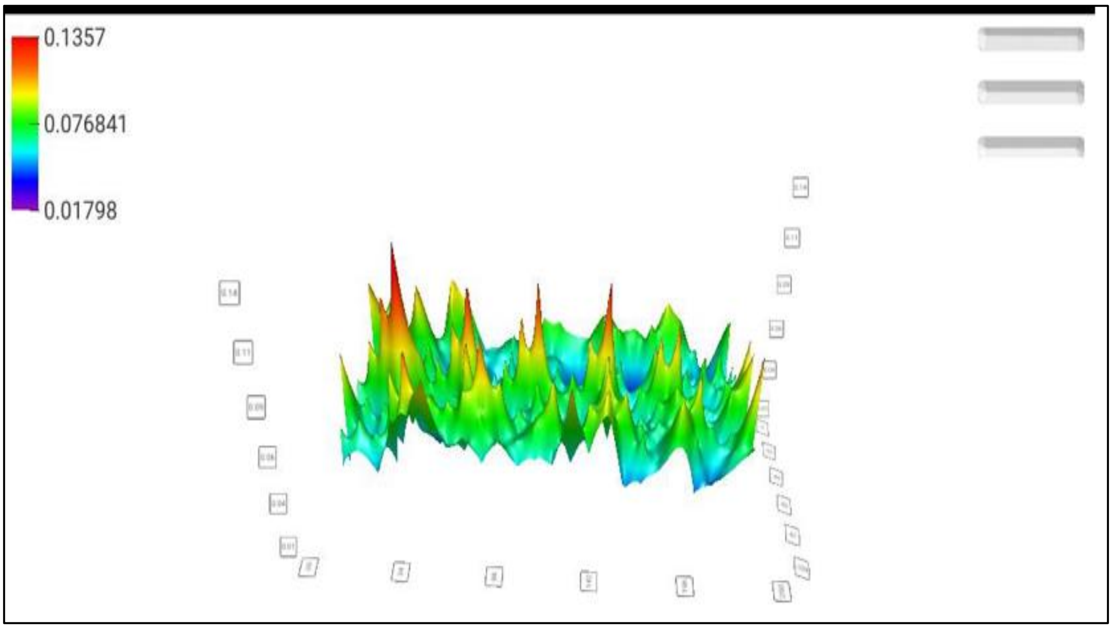
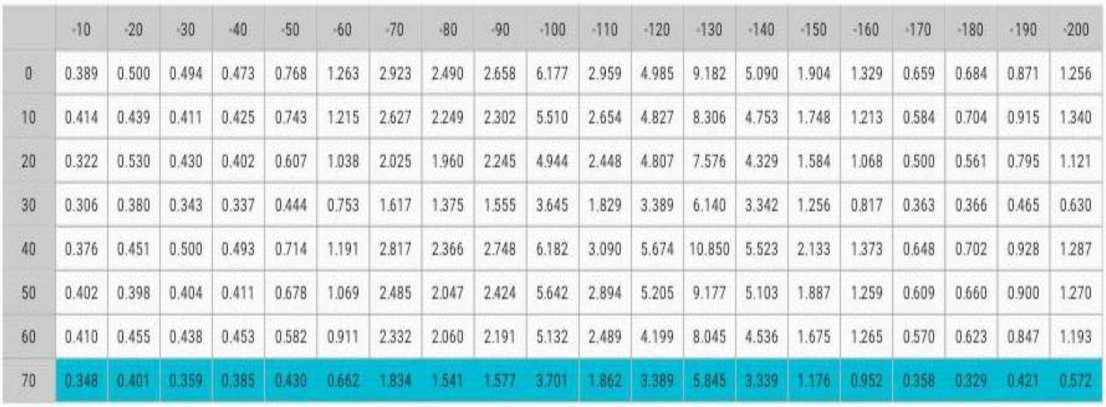


Figure 4-12 Three-dimension expression of Dole test pit2 resistivity

The three-dimensional figure 4-12shows when we see the shape of the figure it is almost the same level or uniform. This indicates the soil around this area is almost the same in the horizontal radius of 200m.

Table 4-15 Resistance Value of Dole Test Pit 3



	-10	-20	-30	-40	-50	-60	-70	-80	-90	-100	-110	-120	-130	-140	-150	-160	-170	-180	-190	-200
0	0.389	0.500	0.494	0.473	0.768	1.263	2.923	2.490	2.658	6.177	2.959	4.985	9.182	5.090	1.904	1.329	0.659	0.684	0.871	1.256
10	0.414	0.439	0.411	0.425	0.743	1.215	2.627	2.249	2.302	5.510	2.654	4.827	8.306	4.753	1.748	1.213	0.584	0.704	0.915	1.340
20	0.322	0.530	0.430	0.402	0.607	1.038	2.025	1.960	2.245	4.944	2.448	4.807	7.576	4.329	1.584	1.068	0.500	0.561	0.795	1.121
30	0.306	0.380	0.343	0.337	0.444	0.753	1.617	1.375	1.555	3.645	1.829	3.389	6.140	3.342	1.256	0.817	0.363	0.366	0.465	0.630
40	0.376	0.451	0.500	0.493	0.714	1.191	2.817	2.366	2.748	6.182	3.090	5.674	10.850	5.523	2.133	1.373	0.648	0.702	0.928	1.287
50	0.402	0.398	0.404	0.411	0.678	1.069	2.485	2.047	2.424	5.642	2.894	5.205	9.177	5.103	1.887	1.259	0.609	0.660	0.900	1.270
60	0.410	0.455	0.438	0.453	0.582	0.911	2.332	2.060	2.191	5.132	2.489	4.199	8.045	4.536	1.675	1.265	0.570	0.623	0.847	1.193
70	0.348	0.401	0.359	0.385	0.430	0.662	1.834	1.541	1.577	3.701	1.862	3.389	5.845	3.339	1.176	0.952	0.358	0.329	0.421	0.572

From the table 4-15 the minimum and maximum resistances are 0.348 and 1.541 respectively.

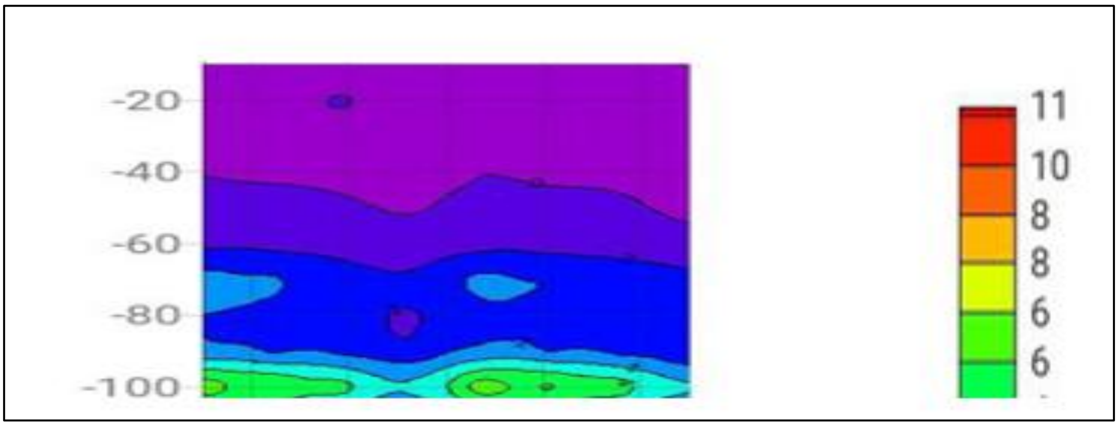


Figure 4-13 Two-dimension expression of Dole test pit 3 resistivity

The two-dimensional figure 4-13 shows when we go down ward the type of soil around this area up to depth of 40m is almost the same type of soil which is continuous builders. From 40m up to 90m weathered and partially weathered rocks respectively and after 90m it contains un weathered rocks.

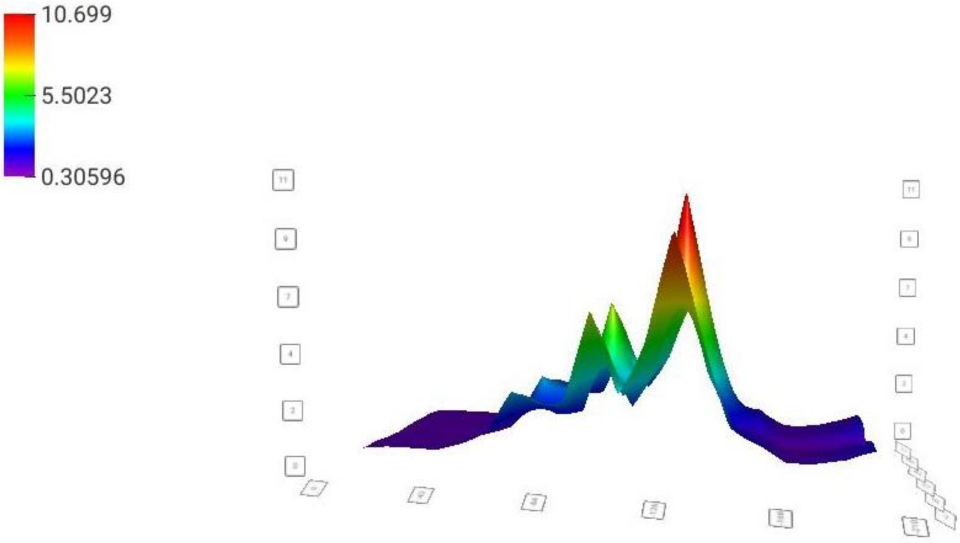
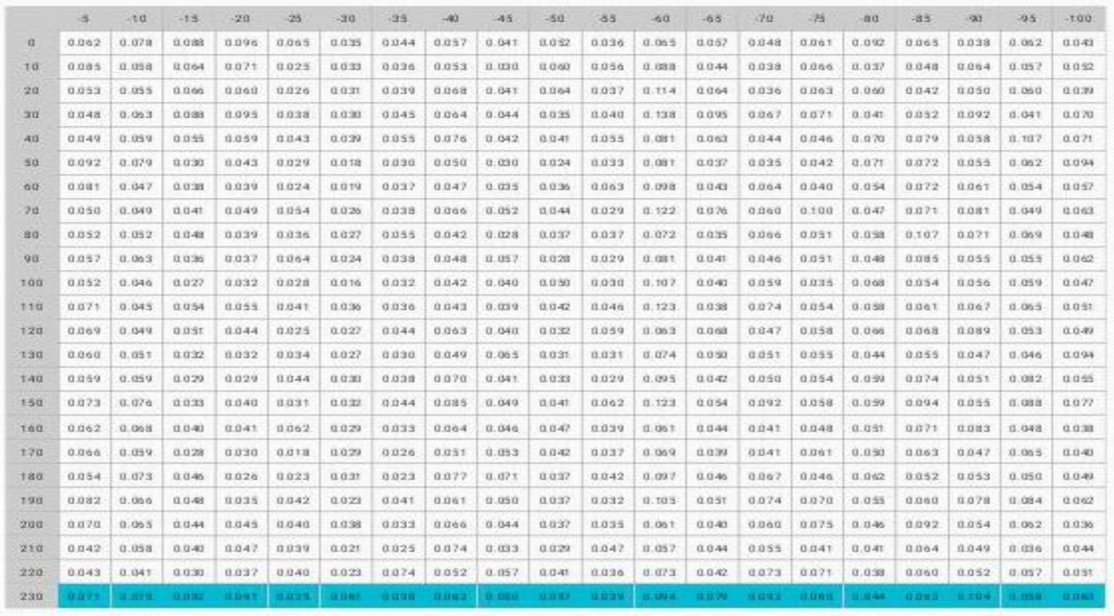


Figure 4-14 Three-dimension expression of Dole test pit 3 resistivity

The three-dimensional figure 4-14 shows the type of soil around this area is almost the same type of soil which is gravel sand and contains some clay soils except one point which contains soils with high resistance Generally, it indicates that when we go horizontally the 3D is smooth except some points which is weathered rock.

Table 4-16 Resistance Value of Dole Test Pit 4



	-5	-10	-15	-20	-25	-30	-35	-40	-45	-50	-55	-60	-65	-70	-75	-80	-85	-90	-95	-100
0	0.062	0.078	0.088	0.096	0.065	0.035	0.044	0.057	0.041	0.052	0.036	0.065	0.057	0.048	0.061	0.092	0.065	0.038	0.062	0.043
10	0.085	0.058	0.064	0.071	0.025	0.033	0.036	0.053	0.030	0.060	0.056	0.088	0.044	0.038	0.066	0.037	0.048	0.064	0.057	0.052
20	0.053	0.055	0.066	0.060	0.026	0.031	0.039	0.068	0.041	0.064	0.037	0.114	0.064	0.036	0.063	0.060	0.042	0.050	0.060	0.039
30	0.048	0.063	0.088	0.095	0.038	0.030	0.045	0.064	0.044	0.035	0.040	0.138	0.095	0.067	0.071	0.041	0.052	0.092	0.041	0.070
40	0.049	0.059	0.055	0.059	0.043	0.039	0.055	0.076	0.042	0.041	0.055	0.081	0.063	0.044	0.046	0.070	0.079	0.058	0.107	0.071
50	0.092	0.079	0.030	0.043	0.029	0.018	0.030	0.050	0.030	0.024	0.033	0.081	0.037	0.035	0.042	0.071	0.072	0.055	0.062	0.094
60	0.081	0.047	0.038	0.039	0.024	0.019	0.037	0.047	0.035	0.036	0.063	0.098	0.043	0.064	0.040	0.054	0.072	0.061	0.054	0.057
70	0.050	0.049	0.041	0.049	0.054	0.026	0.038	0.066	0.052	0.044	0.029	0.122	0.076	0.060	0.100	0.047	0.071	0.081	0.049	0.063
80	0.052	0.052	0.048	0.039	0.036	0.027	0.055	0.042	0.028	0.037	0.037	0.072	0.035	0.066	0.051	0.058	0.107	0.071	0.069	0.048
90	0.057	0.063	0.036	0.037	0.064	0.024	0.038	0.048	0.057	0.028	0.029	0.081	0.041	0.046	0.051	0.048	0.085	0.055	0.055	0.062
100	0.052	0.046	0.027	0.032	0.028	0.016	0.032	0.042	0.040	0.050	0.030	0.107	0.040	0.059	0.035	0.068	0.054	0.056	0.059	0.047
110	0.071	0.045	0.054	0.055	0.041	0.036	0.036	0.043	0.039	0.042	0.046	0.123	0.038	0.074	0.054	0.058	0.061	0.067	0.065	0.051
120	0.069	0.049	0.051	0.044	0.025	0.027	0.044	0.063	0.040	0.032	0.059	0.063	0.068	0.047	0.058	0.066	0.068	0.089	0.053	0.049
130	0.060	0.051	0.032	0.032	0.034	0.027	0.030	0.049	0.065	0.031	0.031	0.074	0.050	0.051	0.055	0.044	0.055	0.047	0.046	0.094
140	0.059	0.059	0.029	0.029	0.044	0.030	0.038	0.070	0.041	0.033	0.029	0.095	0.042	0.050	0.054	0.059	0.074	0.051	0.082	0.055
150	0.073	0.076	0.033	0.040	0.031	0.032	0.044	0.085	0.049	0.041	0.062	0.123	0.054	0.092	0.058	0.059	0.094	0.055	0.088	0.077
160	0.062	0.068	0.040	0.041	0.062	0.029	0.033	0.064	0.046	0.047	0.039	0.061	0.044	0.041	0.048	0.051	0.071	0.083	0.048	0.038
170	0.066	0.059	0.028	0.030	0.018	0.029	0.026	0.051	0.053	0.042	0.037	0.069	0.039	0.041	0.061	0.050	0.063	0.047	0.065	0.040
180	0.054	0.073	0.046	0.026	0.023	0.031	0.023	0.077	0.071	0.037	0.042	0.097	0.046	0.067	0.046	0.062	0.052	0.053	0.050	0.049
190	0.082	0.066	0.048	0.035	0.042	0.023	0.041	0.061	0.050	0.037	0.032	0.105	0.051	0.074	0.070	0.055	0.060	0.078	0.084	0.062
200	0.070	0.065	0.044	0.045	0.040	0.038	0.033	0.066	0.044	0.037	0.035	0.061	0.040	0.060	0.075	0.046	0.092	0.054	0.062	0.036
210	0.042	0.058	0.040	0.047	0.039	0.021	0.025	0.074	0.033	0.029	0.047	0.057	0.044	0.055	0.041	0.041	0.064	0.049	0.036	0.044
220	0.043	0.041	0.030	0.037	0.040	0.023	0.074	0.052	0.057	0.041	0.036	0.073	0.042	0.073	0.071	0.038	0.060	0.052	0.057	0.051
230	0.071	0.079	0.032	0.041	0.025	0.061	0.038	0.062	0.050	0.037	0.039	0.094	0.079	0.093	0.060	0.044	0.083	0.104	0.058	0.063

From the table 4-16 the minimum and maximum resistances are 0.025 and 0.074 respectively.

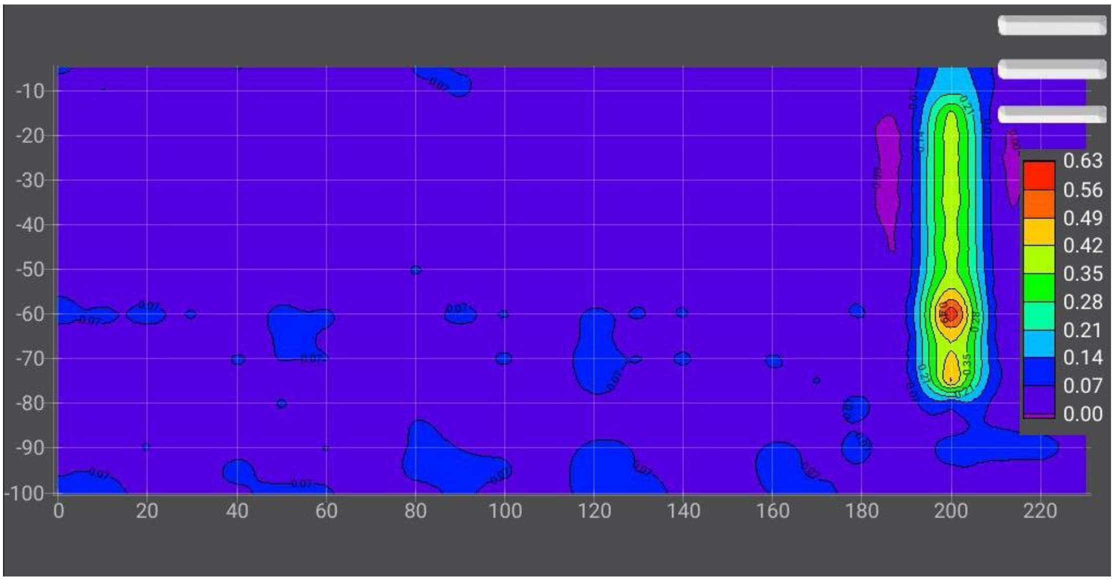


Figure 4-15 Two-dimension expression of Dole test pit4 resistivity

The two-dimensional figure 4-15 shows when we go down ward the type of soil around this area is almost the same type of soil which is ignimbrite weathered rocks (gravel and sand) and contains some lime stone and some clay soils Generally, it indicates that when we go from top to bottom there is no big difference on the type of soil.

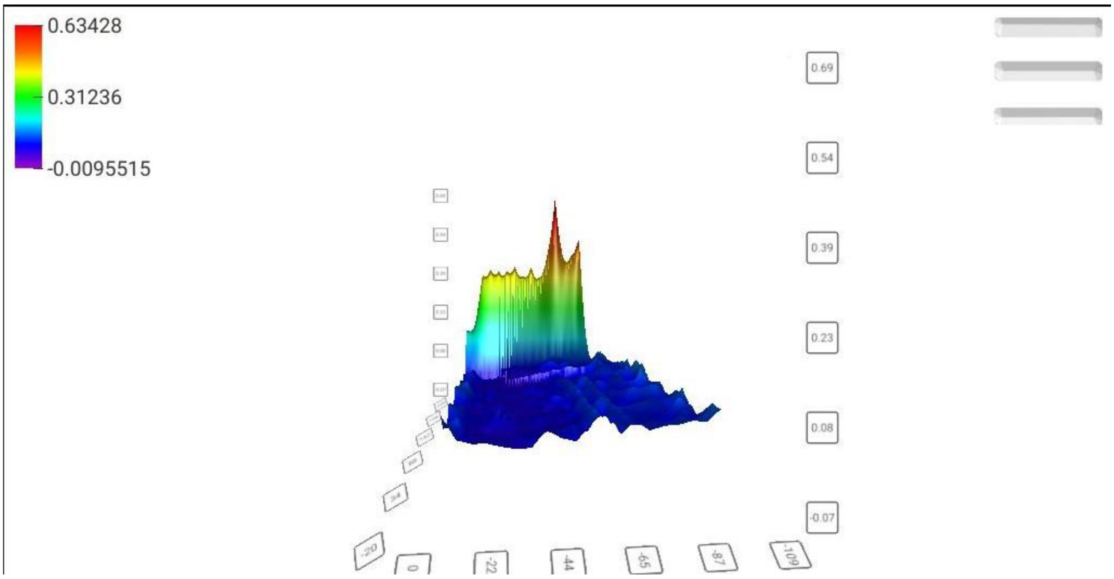
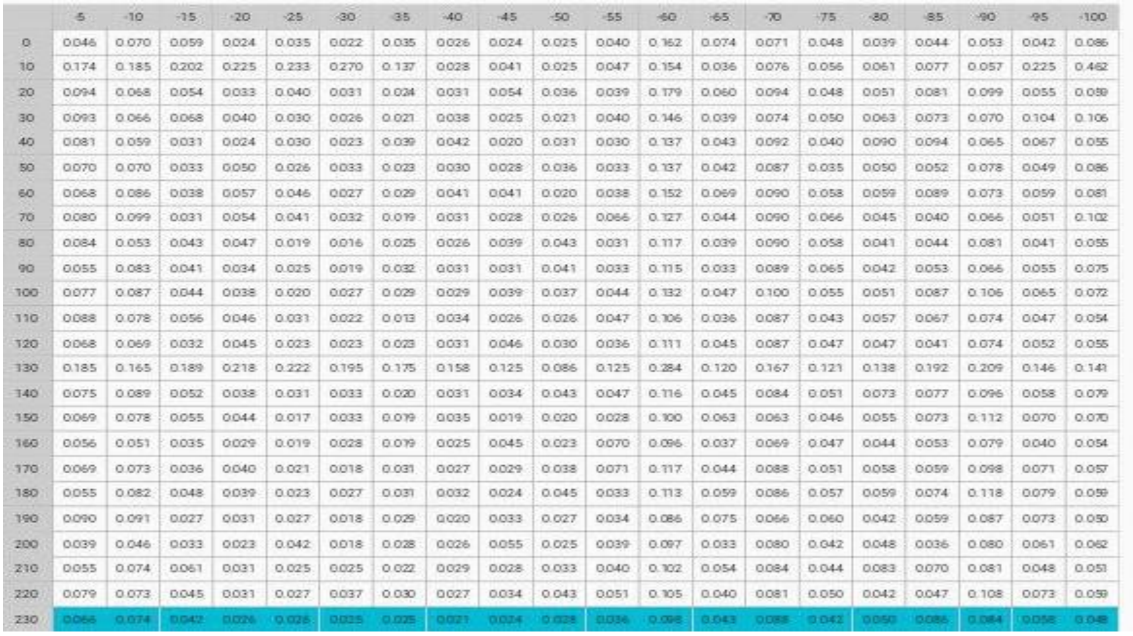


Figure 4-16 Three-dimension expression of Dole test pit 4 resistivity

The three-dimensional figure 4-16 shows when we see the shape of the figure it is almost the same level or flat except one side that shows high resistance due to un weathered and rock. This indicates the soil around this area is almost the same in the horizontal radius of 200m.

Table 4-17 Resistance Value of Dole Test Pit 5



	-5	-10	-15	-20	-25	-30	-35	-40	-45	-50	-55	-60	-65	-70	-75	-80	-85	-90	-95	-100
0	0.046	0.070	0.059	0.024	0.035	0.022	0.035	0.026	0.024	0.025	0.040	0.162	0.074	0.071	0.048	0.039	0.044	0.053	0.042	0.086
10	0.174	0.185	0.202	0.225	0.233	0.270	0.137	0.028	0.041	0.025	0.047	0.154	0.036	0.076	0.056	0.061	0.077	0.057	0.225	0.462
20	0.094	0.068	0.054	0.033	0.040	0.031	0.024	0.031	0.054	0.036	0.039	0.179	0.060	0.094	0.048	0.051	0.081	0.099	0.055	0.059
30	0.093	0.066	0.068	0.040	0.030	0.026	0.021	0.038	0.025	0.021	0.040	0.146	0.039	0.074	0.050	0.063	0.073	0.070	0.104	0.106
40	0.081	0.059	0.031	0.024	0.030	0.023	0.039	0.042	0.020	0.031	0.030	0.137	0.043	0.092	0.040	0.090	0.094	0.065	0.067	0.055
50	0.070	0.070	0.033	0.050	0.026	0.033	0.023	0.030	0.028	0.036	0.033	0.137	0.042	0.087	0.035	0.050	0.052	0.078	0.049	0.086
60	0.068	0.086	0.038	0.057	0.046	0.027	0.029	0.041	0.041	0.020	0.038	0.152	0.069	0.090	0.058	0.059	0.089	0.073	0.059	0.081
70	0.080	0.099	0.031	0.054	0.041	0.032	0.019	0.031	0.028	0.026	0.066	0.127	0.044	0.090	0.066	0.045	0.040	0.066	0.051	0.102
80	0.084	0.053	0.043	0.047	0.019	0.016	0.025	0.026	0.039	0.043	0.031	0.117	0.039	0.090	0.058	0.041	0.044	0.081	0.041	0.055
90	0.055	0.083	0.041	0.034	0.025	0.019	0.032	0.031	0.031	0.041	0.033	0.115	0.033	0.089	0.065	0.042	0.053	0.066	0.055	0.075
100	0.077	0.087	0.044	0.038	0.020	0.027	0.029	0.029	0.039	0.037	0.044	0.132	0.047	0.100	0.055	0.051	0.087	0.106	0.065	0.072
110	0.088	0.078	0.056	0.046	0.031	0.022	0.013	0.034	0.026	0.026	0.047	0.106	0.036	0.087	0.043	0.057	0.067	0.074	0.047	0.054
120	0.068	0.069	0.032	0.045	0.023	0.023	0.023	0.031	0.046	0.030	0.036	0.111	0.045	0.087	0.047	0.047	0.041	0.074	0.052	0.055
130	0.185	0.165	0.189	0.218	0.222	0.195	0.175	0.158	0.125	0.086	0.125	0.284	0.120	0.167	0.121	0.138	0.192	0.209	0.146	0.141
140	0.075	0.089	0.052	0.038	0.031	0.033	0.020	0.031	0.034	0.043	0.047	0.116	0.045	0.084	0.051	0.073	0.077	0.096	0.058	0.079
150	0.069	0.078	0.055	0.044	0.017	0.033	0.019	0.035	0.019	0.020	0.028	0.100	0.063	0.063	0.046	0.055	0.073	0.112	0.070	0.070
160	0.056	0.051	0.035	0.029	0.019	0.028	0.019	0.025	0.045	0.023	0.070	0.096	0.037	0.069	0.047	0.044	0.053	0.079	0.040	0.054
170	0.069	0.073	0.036	0.040	0.021	0.018	0.031	0.027	0.029	0.038	0.071	0.117	0.044	0.088	0.051	0.058	0.059	0.098	0.071	0.057
180	0.055	0.082	0.048	0.039	0.023	0.027	0.031	0.032	0.024	0.045	0.033	0.113	0.059	0.086	0.057	0.059	0.074	0.118	0.079	0.059
190	0.090	0.091	0.027	0.031	0.027	0.018	0.029	0.020	0.033	0.027	0.034	0.086	0.075	0.066	0.060	0.042	0.059	0.087	0.073	0.050
200	0.039	0.046	0.033	0.023	0.042	0.018	0.028	0.026	0.055	0.025	0.039	0.097	0.033	0.080	0.042	0.048	0.036	0.080	0.061	0.062
210	0.055	0.074	0.061	0.031	0.025	0.025	0.022	0.029	0.028	0.033	0.040	0.102	0.054	0.084	0.044	0.083	0.070	0.081	0.048	0.051
220	0.079	0.073	0.045	0.031	0.027	0.037	0.030	0.027	0.034	0.043	0.051	0.105	0.040	0.081	0.050	0.042	0.047	0.108	0.073	0.059
230	0.066	0.074	0.042	0.026	0.026	0.025	0.025	0.021	0.024	0.028	0.036	0.098	0.043	0.088	0.042	0.050	0.086	0.084	0.058	0.048

From the table 4-16 the minimum and maximum resistances are 0.021 and 0.098 respectively.

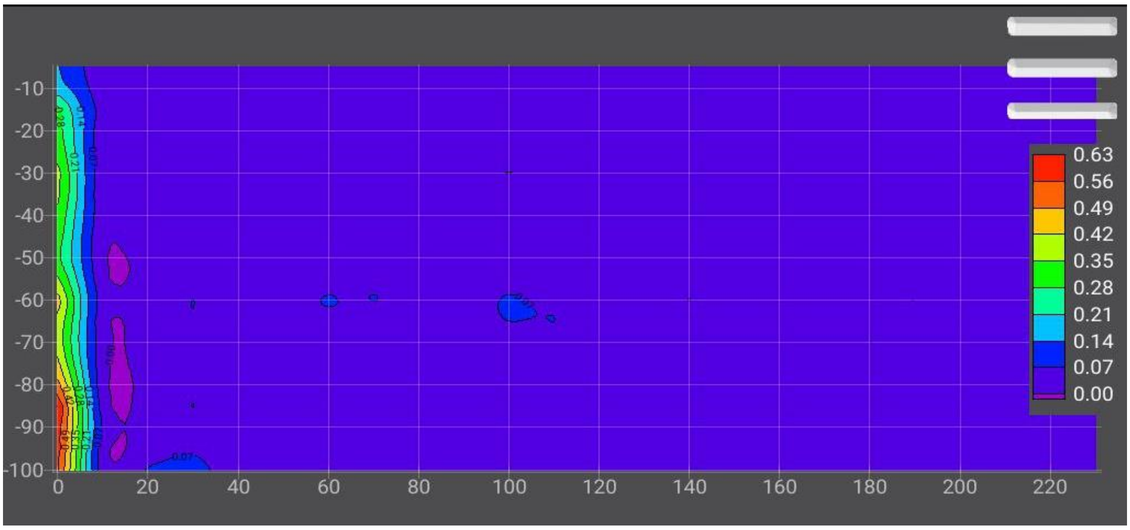


Figure 4-17 Two-dimension expression of Dole test pit5 resistivity

The two-dimensional figure 4-17 shows when we go down ward the type of soil around this area is almost the same type of soil which is ignimbrite weathered rocks (gravel and sand) and contains some lime stone and some clay soils Generally, it indicates that when we go from top to bottom there is no big difference on the type of soil.

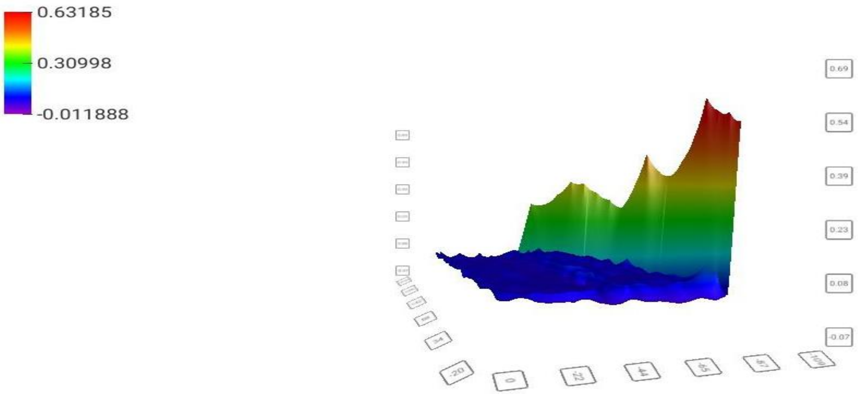


Figure 4-18 Three-dimension expression of Dole test pit5 resistivity

The three-dimensional figure 4-18 shows when we see the shape of the figure it is almost the same level or flat except one side that shows high resistance due to un weathered and rock. This indicates the soil type around this area is almost the same in the horizontal direction in radius of 200m.

CHAPTER FIVE

5 DISCUSSIONS OF LABORATORY TESTS AND ELECTRICAL RESISTIVITY RESULTS

5.1 Classifications of the soils

5.1.1 General

Soil classification is the arrangement of soils into different groups such that the soils in a particular group have similar behavior. In this study, the American Association of State Highway and Transportation Officials (AASHTO) classification system and the Unified Soil Classification System (USCS) were used to classify the soil types in the study area.

5.1.2 Classification of soils based on unified soil classification system (USCS)

The classification of soils according to the USCS scheme shows the soil of the study area was “Well-graded Coarse-grained soils.” From the grain size analysis test result of the test pits, the soil particle passes through 0.075mm sieve size was less than 50%. Accordingly, the soil specimens were classified as “Coarse-grained soils” (i.e., gravelly and/ or sandy soils). The soil classification of TP-1 and TP-2 at a depth of 1.5m and 3m was ‘Well-graded gravel with sand’. Whereas TP-3 at 1.5m was ‘Well-graded’ sandy soil and TP-4 at a depth of 1.5m was ‘Poor-graded’ sand with gravel soil. However, TP-3 and 4 at a depth of 3m was silty sand, and TP-5 at a depth of 3m was Silty sand with gravel, soil classification. As shown in Figure 4-3 plasticity chart, the soil of study area has a slightly plastic to low plasticity according to (Das and Sobhan, 2018).

5.1.3 American association of state highway and transport official system (AASHTO)

According to the AASHTO classification system results (table 4-8 and Figure 4-5), all the samples collected fall into category A-2-4. According to the ERA manual, the group index results are zero, and this value indicates that generally, the soils of the study area are excellent to good for highway subgrade material.

5.2 Discussions of Engineering Properties Test Results

The compression and swelling indexes of the soils were determined using the straight-line parts of the void ratio against log pressure curves shows in Figure 4-7 for TP-3 at 3m. The compression index (Cc) ranged from 0.122 to 0.421. The swelling index (Cs) of the test pits varied from 0.0363 to 0.0742, as shown in Table 4-10. Cc often varies from 0.1 to 10, with no units. The index for normally consolidated clays typically ranges between 0.20 and 0.50, and for silts, between 0.16 and 0.24. The index for sands varies from 0.01 to 0.06 (Widodo and Ibrahim, 2012). Yet the test pits conducted for the one-dimensional consolidation test were sandy with silty soil; the soils' over-consolidation ratios were greater than one, Cc were failed under normally consolidated clay soil. However, because the soil was lightweight sandy with silt, the Cc of the test pits increased. The swelling index (Cs) of the test pits were observed as low value. This indicates that, the soils swelling potential is low. So; the soil specimens in the study area were over-consolidated in their natural form which indicates the soil has strong liquefaction, low lateral deformation, high strength, and low settlement.

The coefficient of permeability of soil under investigation which is calculated from the test results of consolidation test ranges from $0.02*10^{-4}$ and $1.16*10^{-4}$ cm/sec (Table 4.11). The result shows that the soil under investigation is permeable soil.

The compaction test result shows that maximum dry density (MDD) and the optimum moisture content (OMC) for standard compaction ranges from 0.89g/cm3 to 1.55g/cm3, and 8.50% to 43.0% respectively which shows that the soil is medium compressive and intermediate strength.

From the direct shear test result the cohesion (Cu) of the Dole Town ranges from 5.7 KN/m2 to 45.8 KN/m2 and internal frictional angle (Øo) of the study area were from 26.4o to 43.8o. This value indicating that the soil is C-∅ soil.

5.3 Discussion of Electrical Resistivity Results

Generally, as it can be seen from Table 4 -12, figure 4-13, 4-14 and appendix IX in our study area, resistance gained from all test points are from 0.016Ώ to 1.541Ώ, and its resistivity is from 3. 22Ώ.m to 368. 56Ώ.m. So according to (Briški et al., 2020, Azrief

Azahar et al., 2019 and Tsegaselassie, 2013), these values indicate that the soil around this area contains; weathered, un weathered and partially weathered ignimbrite rocks such as Gravel, loose sand and clay soil which indicates the soil around Dole town is good for subgrade materials. The two- and three-dimensional figures shows the type of soil around this area is almost the same type of soil which is gravel sand and contains some clay soils and lime stone. Generally, it indicates that when we go from top to bottom there is no big difference on the type of soil except some points where there is high resistance. The study area is included in part of central main Ethiopian rift valley lake basin which is located in weathered and fractured ignimbrite, pumice, and tuff and got ground water recharge from eastern high land and the area is free from ground crack.

5.4 Comparison of Test Results with Previously Done Researches

The soils of Dole town, when compared with the previously tested soils of Ziway, Ropi, and Someyan, show considerable similarities with Atterberge limits, specific gravity, Dray density, moisture content, and classification. More similarity is observed with respect to the index tests and physical properties. Generally, the soil of Dole could be classified as sandy gravel soil with almost identical characteristics to those of Ziway, Ropi, and Someyan soils, as shown on the following table.

A comparison of compression index (Cc) is made on the results obtained from laboratory tests and results which are developed by different authors gave closer value. This may be due to the fact that the correlations developed by these authors are applicable for the type of the soils from which the samples were taken.

The electrical resistivity results of Dole town also show considerable similarity when compared with the electrical resistivity results of previous Ethiopia road design test results and it is in the range of other international researchers.

Table 5-1 For the soil under investigation comparisons were made with known Ziway and Ropi in central Rift valley and Ethiopia Road Design

	Biruk Haile, 2014	Yadeta C. Chemada (EJSSD, 2020)	Previous Research (IJEE, 2017) (Kebede Midekss, Tarun Kumar, Raghuvanshi and Bekele Abebe, 2015)			Ashenafi Tsegaselassie 2013	Azrief Azhara et al 2019	Current Study
Study Area	Woliso Town	Adama Town	Someyan	Ziway	Ropi	Ethiopia Road Design	Malaysian Road Conference	Dole Town
Soil Type	Black clay	Sandy Gravel	Fine Sand	Fine Sand	Sandy Silt			Sandy Gravel
Location	Around Stadium	Road to Metahara	E=464087 N=885522	E=465684 N=880271	E=403150 N=789250			E=4638028 N=833338
Clay content (%)	42.5-80	2.9-26.4						0.88-6.45
Plastic Limit (%)	31.2-44.1	7.5-40.83	17.46	17.43	27.11			14.81-34.79
Liquid Limit (%)	70.5-126	26.06-49	19.72	19.52	31.09			18.29-37.82
Plasticity Index (%)	39.3-81.7	2.22-23	2.26	2.09	3.98			1.7-5.3
Specific Gravity (%)	2.65-2.76	2.00-2.67	2.37	2.24	2.37			2.39-5.59
Dry density (g/cm^3)	1.7-1.88		0.76-1.03	0.5-1.05	0.76-0.92			0.89-1.55
Moisture Content (%)	25.5-65.5	6.34-25.2	13	5.13	22.37			8.5-43
plasticity Chart Results	CH, MH	SM, SP	SM, ML	ML, SM	ML, SM			CL, ML
Electrical resistivity						10.8-149Ώ.m	10-2200 Ώ.m	3 - 368. Ώ.m

CHAPTER SIX

6 CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSION

From the study of engineering properties of Dole Town to determine the index properties of soils, compression and strength characteristics of soils, classification of soils and identification of the geological conditions of the area the following conclusion have reached.

1. No significant variations of engineering properties within the investigated depths as well as in different pits which were found in the research work.
2. According to USCS the soils of the study area is “Coarse-grained soils” (i.e., gravelly and/ or sandy soils) with low plasticity, and as AASHTO classification system, the soil of the study area is classified as A-2-4.
3. The values of specific gravity are 2.39-2.59 which is within the same ranges of sandy gravel in the other parts of the country. This test results illustrate that the soil categorized as porous and or pumice materials.
4. As determined from the one-dimensional consolidation test conducted on undisturbed soil samples; the compression index (Cc) ranges 0.122 to 0.421; swelling index (Cs) ranges 0.0363 to 0.0742, coefficient of permeability of soil under investigation which is calculated from the test results of consolidation test ranges $0.02*10^{-4}$ to$1.16*10^{-4}$ cm/sec. Thus, the soil is slightly compressible and permeable sandy with silty soil.
5. The compaction test result shows that maximum dry density (MDD) and the optimum moisture content (OMC) for standard compaction ranges from 0.89g/cm3 to 1.55g/cm3, and 8.50% to 43.0% respectively which shows that the soil is medium compressive and intermediate strength.
6. From the direct shear test result the cohesion (Cu) of the Dole Town ranges from 5.7 kN/m^2 to 45.8 kN/m^2 and internal frictional angle (Øo) of the study area were from 26.4o to 43.8o. This value indicating that the soil is C-∅ soil.
7. The geological conditions of the study area are un weathered and partially

weathered ignimbrite rocks such as gravel, loose sand, and clay soil, according to the electrical resistivity test. According to the two- and three-dimensional figures, the soil in this area is mostly gravel and sand, with a bit of fine soil and limestone mixed in.

6.2 RECOMMENDATIONS

1. In this study, the soil samples were collected only from five test pits; for further detailed investigation, such as preparing a soil map, it is necessary to increase the number of test pits.
2. In Dole Town, there is a need for detailed geological studies and to incorporate updated geophysical methodologies that use natural fields and artificial sources for vast, fast, and competitive results.
3. For a complete presentation of the engineering properties of the soil in the town, the dynamic properties of the soil need to be studied.
4. This study is for shallow foundation and for deep foundation specially for the values of electrical resistivity 1up to 10, to clearly differentiate soil and water using detailed geotechnical and geophysical methodologies are necessary.

REFERENCES

Adugna, K. and Yirga, Y. (2020), “Investigation on Engineering Characteristics of Soils. A Case Study in Wolkite University Compound, Ethiopia”, Journal of Civil Engineering Research, Vol. 10 No. 2, pp. 39–46, doi: 10.5923/j.jce.20201002.02.

ASTM International. (1998), “ASTM D-422: Standard Test Method for Particle-Size Analysis of Soils”, in ASTM (Ed.), Annual Book OfASTM Standards, Vol. 63, West Conshohocken.

Ayolabi, E.A., Enoh, I.J.E. and Folorunso, A.F. (2012), “Engineering Site Characterisation Using 2-D and 3-D Electrical Resistivity Tomography”, Earth Science Research, Canadian Center of Science and Education, Vol. 2 No. 1, doi: 10.5539/esr. v2n1p133.

Azrief Azahar, M., Farhan Zakiran Mahadi, N., Rusli, Q.N., Narendranathan, N. and Lee, E.C. (2019), “Use of geophysics for site investigations and earthworks assessments”, IOP Conference Series: Materials Science and Engineering, Vol. 512, Institute of Physics Publishing, doi: 10.1088/1757-899X/512/1/012007.

Bell, F.G. (Frederic G. (1992), Engineering Properties of Soils and Rocks, Butterworth-Heinemann. 3rd Edition.

Briški, M., Stroj, A., Kosović, I. and Borović, S. (2020), “Characterization of aquifers in metamorphic rocks by combined use of electrical resistivity tomography and monitoring of spring hydrodynamics”, Geosciences (Switzerland), MDPI AG, Vol. 10 No. 4, doi: 10.3390/geosciences10040137.

Das, B.M. and Sobhan, K. (2018), Principles of Geotechnical Engineering, Ninth., Cengage Learning.

Debebe, D., Papers, R. and Tona, W. (2011), Investigation on Some of The Engineering Characteristics of Soils in Adama Town, Ethiopia, Addis Ababa Institute of Technology, Addis Ababa, 1 December.

Dechasa, T. (2017), Investigation on Some of The Engineering Properties of Soils Found in Sokoru Town, Jimma Institution of Technology, Jimma, 30 November.

Fatahi, B., Khabbaz, M.H. and Basack, S. (2011), “Effects of salinity and sand content on liquid limit and hydraulic conductivity Publication Details”, Australian Geomechanics Journal, Vol. 46 No. 1, pp. 67–76.

Guo, S., Jiang, M., Lin, J., Khan, N.I., Owens, G. and Chen, Z. (2023), “Arsenic speciation, oxidation and immobilization in an unsaturated soil in the presence of green synthesized iron oxide nanoparticles and humic acid”, Chemosphere, Pergamon, Vol. 311, p. 137198, doi: 10.1016/J.CHEMOSPHERE.2022.137198.

Haile, B. (2014), Investigation into Some of the Engineering Properties of Soils Found In Woliso Town, Addis Ababa University, Addis Abeba, 24 December.

Jibril, J. (2017), In-Depth Investigation into Engineering Characteristics of Jimma Soils, Addis Ababa Institute of Technology, Addis Abeba, 13 November.

Jolfaei, S. and Lakirouhani, A. (2022), “Sensitivity Analysis of Effective Parameters in Borehole Failure, Using Neural Network”, Advances in Civil Engineering, Hindawi Limited, Vol. 2022, pp. 1–16, doi: 10.1155/2022/4958004.

Kalinski, M.E. (2011), Soil Mechanics Lab Manual, 2nd ed., John Wiley & Sons, Inc., University of Kentucky.

K.R. Arora. (2004), Soil Mechanics and Foundation Engineering, 6th ed., A. K. Jain.

Liu, C. and Evett, J.B. (2009), Soil Properties: Testing, Measurement, and Evaluation, edited by Kottcamp, S., Sixth., Pearson/Prentice Hall, New Jersey.

Lollo, L.C. (2016), “Geotechnical Evaluation of Foundation Soils for a Building (Case Study of a Site in Jimma City, Southwestern Ethiopia)”, Journal of Environment and Earth Science, Vol. 6 No. 3, pp. 95–98.

Merga, S. (2016), Effect of Sample Height on Stress- Strain Behavior of Undrained Shear Strength of Addis Ababa Red Clay Soil Using UCS Test, Addis Ababa Institute of Technology.

Midekssa, K., Raghuvanshi, T.K. and Abebe, B. (2015), “Investigating Causes of Origin and Engineering Geological Characterization of Ground Cracks around Ziway and Ropi Area in Central Rift Valley of Ethiopia”, International Journal of Earth Sciences and Engineering (IJEE), Vol. 08, No. 03 No. 0974–5904, pp. 1069–1080.

Murthy V.N.S. (2002), Geotechnical Engineering: Principles and Practices of Soil Mechanics and Foundation Engineering, 2ND ed., Marcel Dekker. Inc., New York.

Peter, L., Jayasree, P.K. and Balan, K. (2020), “Optimization, quantification of Strength and Design Benefits of Flexible Pavements Resting on Soft Soil Subgrades Treated with Coir Waste and Lime”, Journal of Natural Fibers, doi: 10.1080/15440478.2020.1788490.

Phogat, V.K., Tomar, V.S. and Dahiya, R. (n.d.). Soil Physical Properties.

RARC and JICA. (2014), “Soil Analysis Manual”, Japan International Cooperation Agency (JICA), RECS International Inc., No. June.

Tabwassah, C.A. and Obiefuna, G.I. (2012), “Geopysical and Geotechnical Investigation of Cham Failed Dam Project, Ne Nigeria”, Research Journal of Recent Sciences, Vol. 1 No. 2, pp. 1–18.

Tefera, A. and Leikun, M. (1999), Soil Mechanics, 1st ed., Vol. 1, Addis Abeba University, Addis Abeba.

Terzaghi, K., Peck, R.B. and Mesri, G. (1996), “Soil Mechanics in Engineering Practice.pdf”, John Wiley & Sons.

Tsegaselassie, A. (2013), Supplementing Site Investigation Techniques of Earthworks and Sub Grade Soils with Geophysical Investigation in Road Designs, Addis Abeba Universty , ADDIS ABABA, 10 October.

Wakjira, S.N. (2020), Investigation into Some of the Engineering Properties of Soil: A Case Study in Seka Town, Jimma Zone, jimma institution of Technology, jimma, 23 February.

Ware, H. (2020), “Investigating Some of the Engineering Properties of the Soil Found in Segen Town”, International Journal of Scientific & Engineering Research, Vol. 11, pp. 78–103.

Widodo, S. and Ibrahim, A. (2012), “Estimation of Primary Compression Index (Cc) Using Physical Properties of Pontianak Soft Clay”, International Journal of Engineering Research and Applications (IJERA), Vol. 2 No. 2248–9622, pp. 2232–2236.

Woldegabriel, G., Aronson, J.L. and Walter, R.C. (1990), “Geology, geochronology, and rift basin development in the central sector of the Main Ethiopia Rift”, Bulletin of the Geological Society of America, Geological Society of America, Vol. 102 No. 4, pp. 439–458, doi:10.1130/00167606(1990)102<0439: GGARBD>2.3.CO;2.

APPENDIX

I. Natural Moisture Content

Test Pit Name	TP 1@1.5m		
Test Pit Location	Langano Lake Side-1(RHW)		
Test Pit Depth, (m)	1.5		
Test Trials	01	02	03
Wt. of Container, (g)	17.237	32.776	37.157
Wt. of container + wet soil, (g)	116.206	137.028	142.751
Wt. of container + dry soil, (g)	87.369	107.310	112.604
Wt. of water, (g)	28.837	29.718	30.147
Wt. of dry soil, (g)	70.132	74.534	75.447
Moisture container, (%)	41.118	39.872	39.958
Ave. Moisture container, (%)	40.3		

Test Pit Name	TP 1@3m		
Test Pit Location	Langano Lake Side-1(RHW)		
Test Pit Depth, (m)	3		
Test Trials	01	02	03
Wt. of Container, (g)	25.423	32.776	37.157
Wt. of container + wet soil, (g)	131.216	137.028	142.751
Wt. of container + dry soil, (g)	96.259	102.310	107.604
Wt. of water, (g)	34.957	34.718	35.147
Wt. of dry soil, (g)	70.836	69.534	70.447
Moisture container, (%)	49.349	49.930	49.891
Ave. Moisture container, (%)	49.7		

Test Pit Name	TP 2@1.5m		
Test Pit Location	Langano Lake Side-2 (LHW)		
Test Pit Depth, (m)	1.5		
Test Trials	01	02	03
Wt. of Container, (g)	32.721	37.149	37.881
Moisture container, (%)	28.261	27.917	27.206
Ave. Moisture container, (%)	27.8		

Test Pit Name	TP 2@3m		
Test Pit Location	Langano Lake Side-2 (LHW)		
Test Pit Depth, (m)	3		
Test Trials	01	02	03
Wt. of Container, (g)	25.423	32.776	37.157
Wt. of container + wet soil, (g)	131.216	137.028	142.751
Wt. of container + dry soil, (g)	105.259	112.310	117.604
Wt. of water, (g)	25.957	24.718	25.147
Wt. of dry soil, (g)	79.836	79.534	80.447
Moisture container, (%)	32.513	31.079	31.259
Ave. Moisture container, (%)	31.6		

Test Pit Name	TP 3@1.5m		
Test Pit Location	Shala Lake Side (HC)		
Test Pit Depth, (m)	1.5		
Test Trials	01	02	03
Wt. of Container, (g)	32.691	26.695	32.085
Wt. of container + wet soil, (g)	115.931	104.857	119.718
Wt. of container + dry soil, (g)	106.229	95.801	109.900
Wt. of water, (g)	9.702	9.056	9.818
Wt. of dry soil, (g)	73.538	69.106	77.815
Moisture container, (%)	13.193	13.105	12.617
Ave. Moisture container, (%)	13.0		

Test Pit Name	TP 3@3m		
Test Pit Location	Shala Lake Side (HC)		
Test Pit Depth, (m)	3		
Test Trials	01	02	03
Wt. of Container, (g)	17.461	17.074	17.437
Wt. of container + wet soil, (g)	104.748	97.226	110.924
Wt. of container + dry soil, (g)	92.226	85.409	96.810
Wt. of water, (g)	12.522	11.817	14.114
Wt. of dry soil, (g)	74.765	68.335	79.373
Moisture container, (%)	16.748	17.293	17.782
Ave. Moisture container, (%)	17.3		

Test Pit Name	TP 4@1.5m		
Test Pit Location	Abijeta Lake Side (GDWUO)		
Test Pit Depth, (m)	1.5		
Test Trials	01	02	03
Wt. of Container, (g)	32.752	37.163	25.370
Wt. of container + wet soil, (g)	154.884	130.221	115.474
Wt. of container + dry soil, (g)	137.136	120.798	107.003
Wt. of water, (g)	17.748	9.423	8.471
Wt. of dry soil, (g)	104.384	83.635	81.633
Moisture container, (%)	17.003	11.267	10.377
Ave. Moisture container, (%)	12.9		

Test Pit Name	TP 4@3m		
Test Pit Location	Abijeta Lake Side (GDWUO)		
Test Pit Depth, (m)	3		
Test Trials	01	02	03
Wt. of Container, (g)	39.901	18.888	37.852
Wt. of container + wet soil, (g)	130.783	110.011	131.762
Wt. of container + dry soil, (g)	112.820	89.868	110.941
Wt. of water, (g)	17.963	20.143	20.821
Wt. of dry soil, (g)	72.919	70.980	73.089
Moisture container, (%)	24.634	28.378	28.487
Ave. Moisture container, (%)	27.2		

Test Pit Name	TP 5@1.5m		
Test Pit Location	Kebele office Area		
Test Pit Depth, (m)	1.5		
Test Trials	01	02	03
Wt. of Container, (g)	17.074	18.099	17.438
Wt. of container + wet soil, (g)	108.621	120.436	126.148
Wt. of container + dry soil, (g)	98.815	101.520	105.521
Wt. of water, (g)	9.806	18.916	20.627
Wt. of dry soil, (g)	81.741	83.421	88.083
Moisture container, (%)	11.996	22.675	23.418
Ave. Moisture container, (%)	19.4		

Test Pit Name	TP 5@3m		
Test Pit Location	Kebele office Area		
Test Pit Depth, (m)	3		
Test Trials	01	02	03
Wt. of Container, (g)	17.271	18.886	17.251
Wt. of container + wet soil, (g)	120.254	127.527	124.940
Wt. of container + dry soil, (g)	85.734	97.582	95.305
Wt. of water, (g)	34.520	29.945	29.635
Wt. of dry soil, (g)	68.463	78.696	78.054
Moisture container, (%)	50.421	38.051	37.967
Ave. Moisture container, (%)	38.0		

II. Natural Unit Weight

Test Pit Name	TP 1@1.5m		
Test Pit Location	Langano Lake Side-1(RHW)		
Test Pit Depth, (m)	3		
Trials	1	2	3
Specimen + sampler weight, g	269.687	254.801	255.699
Mass of Sampler, gm	165.041	149.207	149.479
Sampler’s volume, cm^3	72.00	72.00	72.00
Bulk Unit weight, (kN/m^3)	14.26	14.39	14.47
Dry Unit weight, (kN/m^3)	10.10	10.29	10.34
Ave. Bulk Unit weight, (kN/m^3)	14.37		
Ave. Dry Unit weight, (kN/m^3)	10.24		

Test Pit Name	TP 1@3m		
Test Pit Location	Langano Lake Side-1(RHW)		
Test Pit Depth, (m)	3		
Trials	1	2	3
Specimen + sampler weight, g	269.687	254.801	255.699
Mass of Sampler, gm	165.041	149.207	149.479
Sampler’s volume, cm^3	72.00	72.00	72.00
Bulk Unit weight, (kN/m^3)	14.26	14.39	14.47
Dry Unit weight, (kN/m^3)	9.55	9.60	9.66

Ave. Bulk Unit weight, (kN/m^3)	14.37		
Ave. Dry Unit weight, (kN/m^3)	9.60		
Test Pit Name	TP 2@1.5m		
Test Pit Location	Langano Lake Side-2 (LHW)		
Test Pit Depth, (m)	3		
Trials	1	2	3
Specimen + sampler weight, g	254.21	255.744	274.273
Mass of Sampler, gm	149.231	149.451	165.013
Sampler’s volume, cm^3	72.00	72.00	72.00
Bulk Unit weight, (kN/m^3)	14.30	14.48	14.89
Dry Unit weight, (kN/m^3)	11.15	11.32	11.70
Ave. Bulk Unit weight, (kN/m^3)	14.56		
Ave. Dry Unit weight, (kN/m^3)	11.39		

Test Pit Name	TP 2@3m		
Test Pit Location	Langano Lake Side-2 (LHW)		
Test Pit Depth, (m)	3		
Trials	1	2	3
Specimen + sampler weight, g	254.21	255.744	274.273
Mass of Sampler, gm	149.231	149.451	165.013
Sampler’s volume, cm^3	72.00	72.00	72.00
Bulk Unit weight, (kN/m^3)	14.30	14.48	14.89
Dry Unit weight, (kN/m^3)	10.79	11.05	11.34
Ave. Bulk Unit weight, (kN/m^3)	14.56		
Ave. Dry Unit weight, (kN/m^3)	11.06		

Test Pit Name	TP 3@1.5m		
Test Pit Location	Shala Lake Side (HC)		
Test Pit Depth, (m)	3		
Trials	1	2	3
Specimen + sampler weight, g	235.076	252.303	237.273
Mass of Sampler, gm	149.37	165.212	165.013
Sampler’s volume, cm^3	72.00	72.00	72.00
Bulk Unit weight, (kN/m^3)	11.68	11.87	9.85

Dry Unit weight, (kN/m^3)	10.32	10.49	8.74
Ave. Bulk Unit weight, (kN/m^3)	11.13		
Ave. Dry Unit weight, (kN/m^3)	9.85		

Test Pit Name	TP 3@3m		
Test Pit Location	Shala Lake Side (HC)		
Trials	1	2	3
Specimen + sampler weight, g	248.09	265.012	254.803
Mass of Sampler, gm	149.675	165.09	149.42
Sampler’s volume, cm^3	72.00	72.00	72.00
Bulk Unit weight, (kN/m^3)	13.41	13.61	14.36
Dry Unit weight, (kN/m^3)	11.49	11.61	12.19
Ave. Bulk Unit weight, (kN/m^3)	13.79		
Ave. Dry Unit weight, (kN/m^3)	11.76		

Test Pit Name	TP 4@1.5m		
Test Pit Location	Abijeta Lake Side (GDWUO)		
Test Pit Depth, (m)	3		
Trials	1	2	3
Specimen + sampler weight, g	242.332	248.485	264.133
Mass of Sampler, gm	149.184	149.361	165.055
Sampler’s volume, cm^3	72.00	72.00	72.00
Bulk Unit weight, (kN/m^3)	12.69	13.51	13.50
Dry Unit weight, (kN/m^3)	10.85	12.14	12.23
Ave. Bulk Unit weight, (kN/m^3)	13.23		
Ave. Dry Unit weight, (kN/m^3)	11.74		

Test Pit Name	TP 4@3m		
Test Pit Location	Abijeta Lake Side (GDWUO)		
Test Pit Depth, (m)	3		
Trials	1	2	3
Specimen + sampler weight, g	214.217	243.849	257.846

Mass of Sampler, gm	149.303	149.495	165.182
Sampler’s volume, cm^3	72.00	72.00	72.00
Bulk Unit weight, (kN/m^3)	8.84	12.86	12.63
Dry Unit weight, (kN/m^3)	7.10	10.01	9.83
Ave. Bulk Unit weight, (kN/m^3)	11.44		
Ave. Dry Unit weight, (kN/m^3)	8.98		

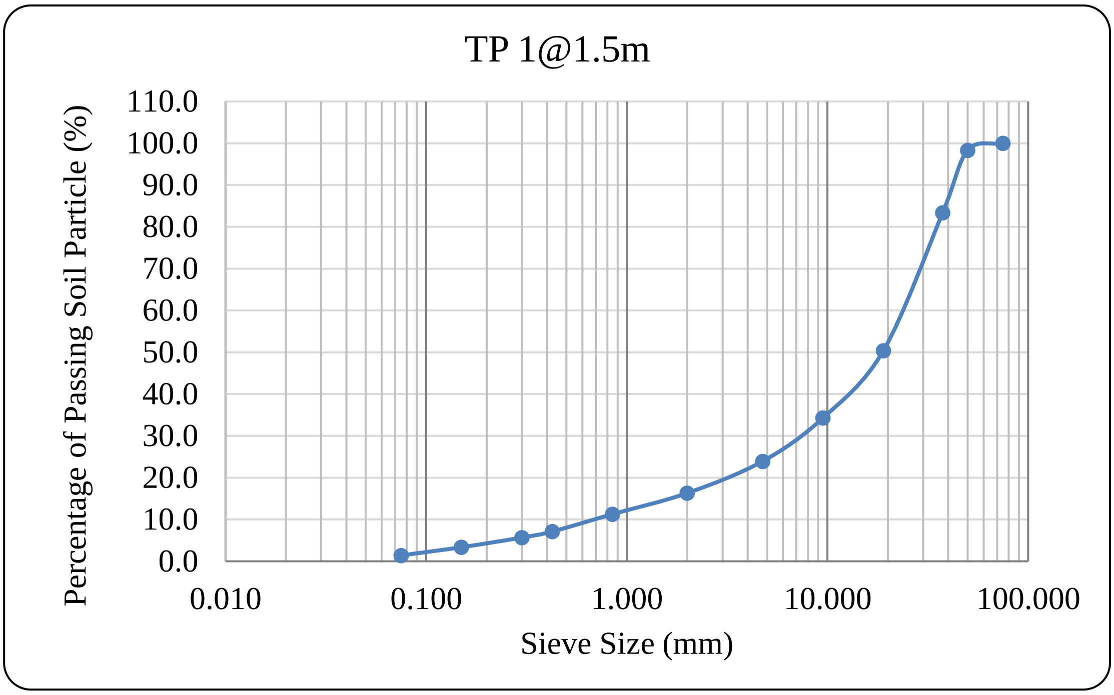
Test Pit Name	TP 5@1.5m		
Test Pit Location	Kebele office Area		
Test Pit Depth, (m)	3		
Trials	1	2	3
Specimen + sampler weight, g	248.282	269.028	238.711
Mass of Sampler, gm	149.252	165.125	149.509
Sampler’s volume, cm^3	72.00	72.00	72.00
Bulk Unit weight, (kN/m^3)	13.49	14.16	12.15
Dry Unit weight, (kN/m^3)	12.05	11.54	9.85
Ave. Bulk Unit weight, (kN/m^3)	13.27		
Ave. Dry Unit weight, (kN/m^3)	11.15		

Test Pit Name	TP 5@3m		
Test Pit Location	Kebele office Area		
Test Pit Depth, (m)	3		
Trials	1	2	3
Specimen + sampler weight, g	252.723	260	258.714
Mass of Sampler, gm	149.333	149.333	149.333
Sampler’s volume, cm^3	72.00	72.00	72.00
Bulk Unit weight, (kN/m^3)	14.09	15.08	14.90
Dry Unit weight, (kN/m^3)	9.36	10.92	10.80
Ave. Bulk Unit weight, (kN/m^3)	14.69		
Ave. Dry Unit weight, (kN/m^3)	10.36		

III. Grain Size Analysis

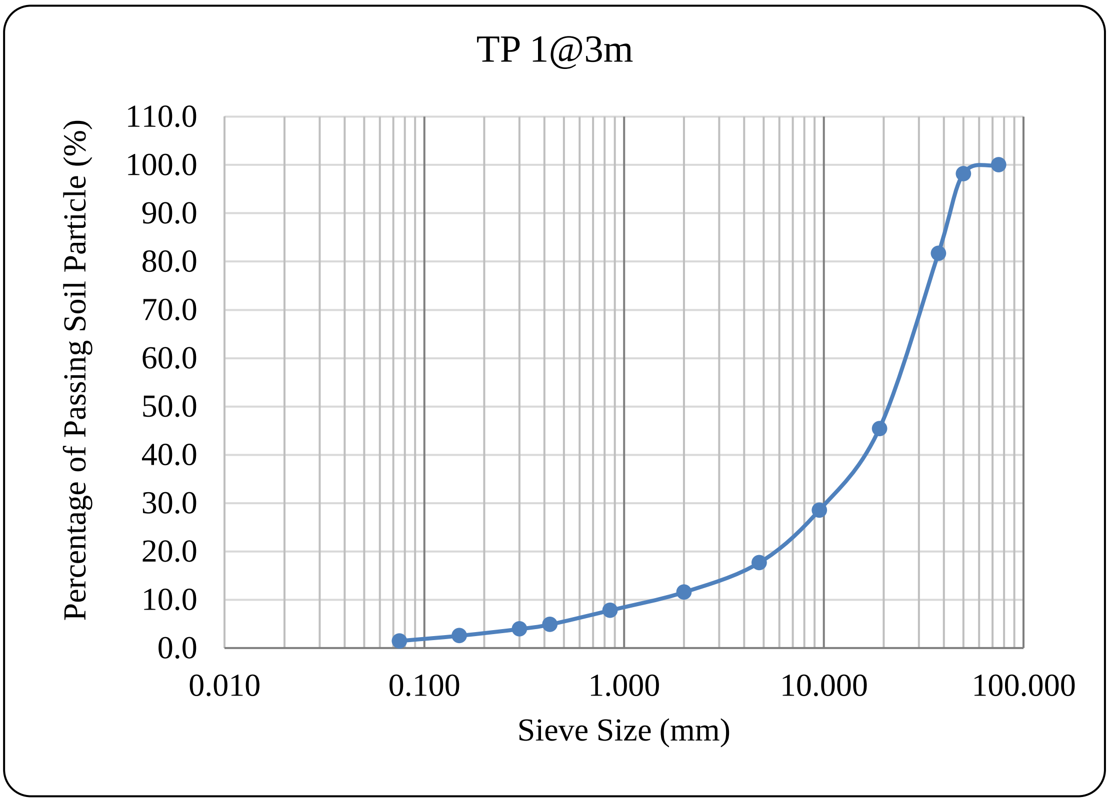
Pit number:	TP 1@1.5m	Sample Location:	Langano Lake Side-1(RHW)
Method of Testing:	Wet Sieve Analysis	Wt. of Sample: (kg)	3.000

Sieve size (mm)	Mass of Retain on Each Sieve (g)	Percentage of Retained Soil	Percentage of cumulative Retained Soil	Percentage of Passing Soil Particle (%)
75.000	0	0.0	0.0	100.0
50.000	50	1.7	1.7	98.3
37.500	448.91	15.0	16.6	83.4
19.000	989.35	33.0	49.6	50.4
9.500	482.9	16.1	65.7	34.3
4.750	311.3	10.4	76.1	23.9
2.000	228.2	7.6	83.7	16.3
0.850	150.9	5.0	88.7	11.3
0.425	125.1	4.2	92.9	7.1
0.300	42.9	1.4	94.3	5.7
0.150	69.5	2.3	96.6	3.4
0.075	58.9	2.0	98.6	1.4
Pan	33.4	1.1	99.7	0.3
Sum	2991.350			

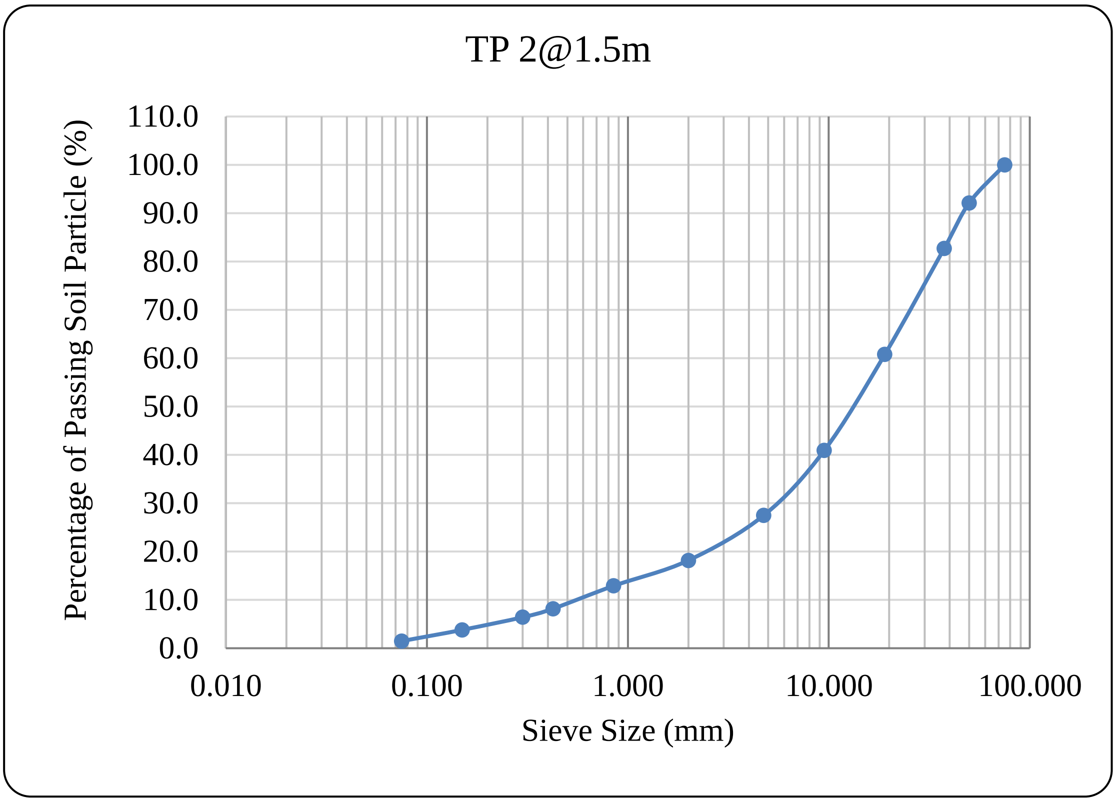


Pit number:	TP 1@3m	Sample Location:	Langano Lake Side-1(RHW)
Method of Testing:	Wet Sieve Analysis	Wt. of Sample: (kg)	3.000

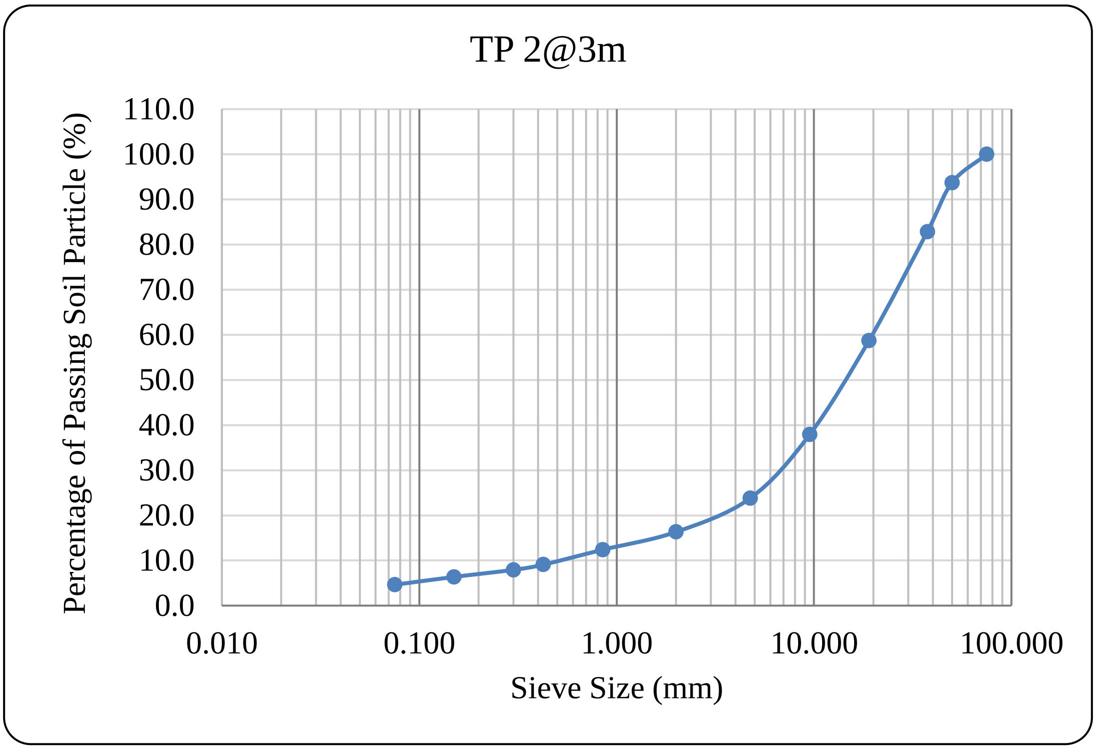
Sieve size (mm)	Mass of Retain on Each Sieve (g)	Percentage of Retained Soil	Percentage of cumulative Retained Soil	Percentage of Passing Soil Particle (%)
75.000	0	0.0	0.0	100.0
50.000	55.00	1.8	1.8	98.2
37.500	493.80	16.5	18.3	81.7
19.000	1088.29	36.3	54.6	45.4
9.500	507.08	16.9	71.5	28.5
4.750	326.91	10.9	82.4	17.6
2.000	182.6	6.1	88.5	11.5
0.850	113.2	3.8	92.2	7.8
0.425	87.6	2.9	95.1	4.9
0.300	27.9	0.9	96.1	3.9
0.150	41.7	1.4	97.5	2.5
0.075	32.4	1.1	98.5	1.5
Pan	16.7	0.6	99.1	0.9
Sum	2973.015			



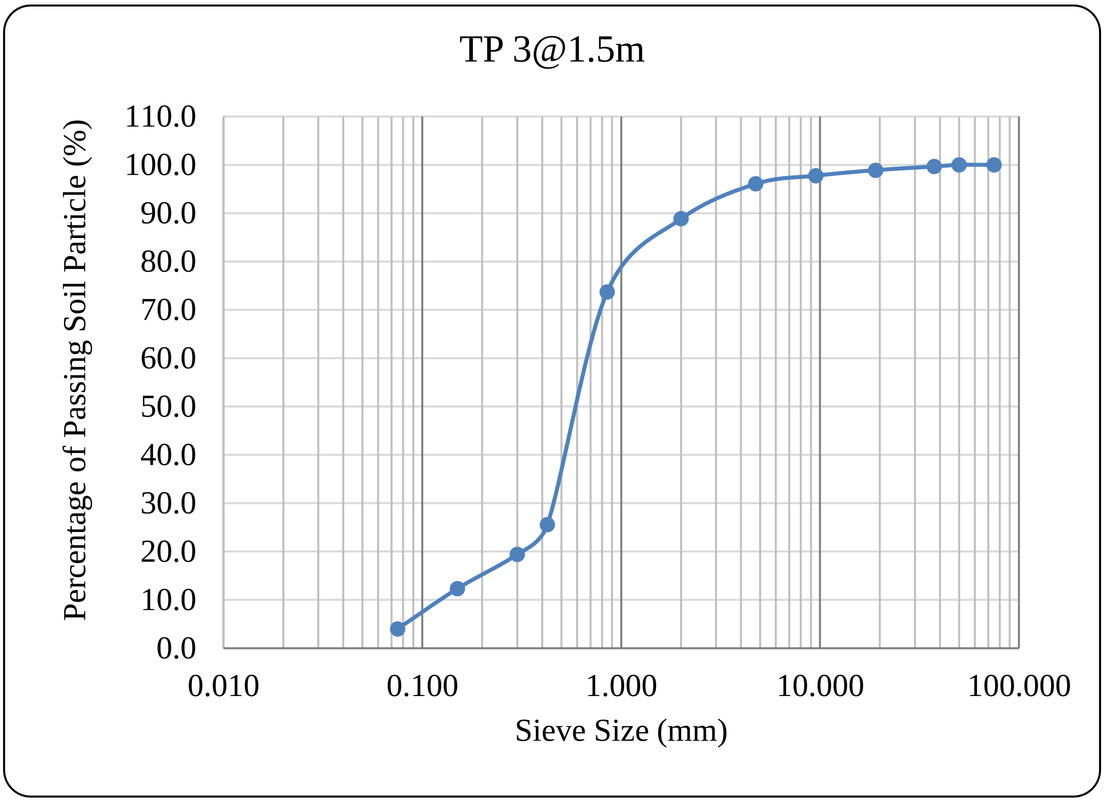
Pit number:	TP 2@1.5m	Sample Location:		Langano Lake Side-2 (LHW)
Method of Testing:	Wet Sieve Analysis	Wt. of Sample: (kg)		3.000
Sieve size (mm)	Mass of Retain on Each Sieve (g)	Percentage of Retained Soil	Percentage of cumulative Retained Soil	Percentage of Passing Soil Particle (%)
75.000	0	0.0	0.0	100.0
50.000	235.64	7.9	7.9	92.1
37.500	283.22	9.4	17.3	82.7
19.000	657.91	21.9	39.2	60.8
9.500	595.6	19.9	59.1	40.9
4.750	402.7	13.4	72.5	27.5
2.000	280.0	9.3	81.8	18.2
0.850	157.7	5.3	87.1	12.9
0.425	141.7	4.7	91.8	8.2
0.300	53.1	1.8	93.6	6.4
0.150	78.5	2.6	96.2	3.8
0.075	69.5	2.3	98.5	1.5
Pan	33.2	1.1	99.6	0.4
Sum	2988.720			



Pit number:	TP 2@3m	Sample Location:		Langano Lake Side-2 (LHW)
Method of Testing:	Wet Sieve Analysis	Wt. of Sample: (kg)		3.000
Sieve size (mm)	Mass of Retain on Each Sieve (g)	Percentage of Retained Soil	Percentage of cumulative Retained Soil	Percentage of Passing Soil Particle (%)
75.000	0	0.0	0.0	100.0
50.000	188.51	6.3	6.3	93.7
37.500	325.70	10.9	17.1	82.9
19.000	723.70	24.1	41.3	58.7
9.500	625.41	20.8	62.1	37.9
4.750	422.86	14.1	76.2	23.8
2.000	224.0	7.5	83.7	16.3
0.850	118.2	3.9	87.6	12.4
0.425	99.2	3.3	90.9	9.1
0.300	34.5	1.2	92.1	7.9
0.150	47.1	1.6	93.6	6.4
0.075	52.1	1.7	95.4	4.6
Pan	16.6	0.6	95.9	4.1
Sum	2877.924			

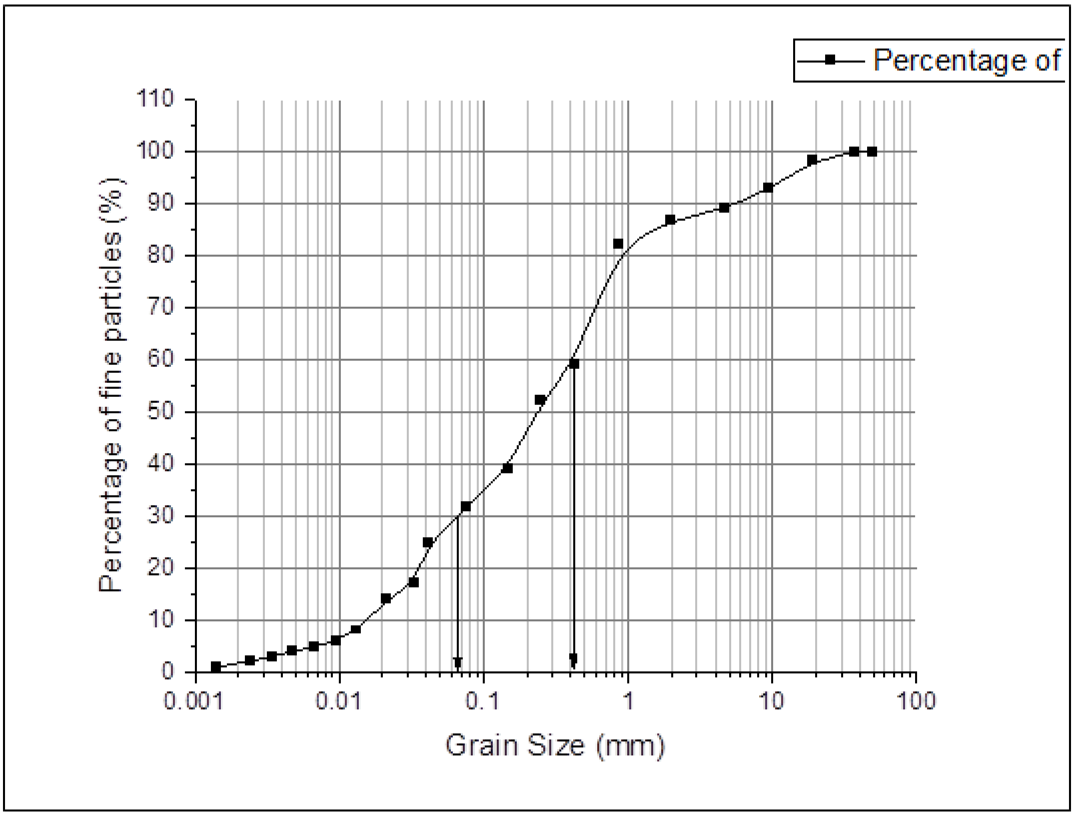


Pit number:	TP 3@1.5m	Sample Location:		Shala Lake Side (HC)
Method of Testing:	Wet Sieve Analysis	Wt. of Sample: (kg)		3.000
Sieve size (mm)	Mass of Retain on Each Sieve (g)	Percentage of Retained Soil	Percentage of cumulative Retained Soil	Percentage of Passing Soil Particle (%)
75.000	0	0.0	0.0	100.0
50.000	0	0.0	0.0	100.0
37.500	10	0.3	0.3	99.7
19.000	23.03	0.8	1.1	98.9
9.500	34.1	1.1	2.2	97.8
4.750	51.0	1.7	3.9	96.1
2.000	216.6	7.2	11.2	88.8
0.850	454.3	15.1	26.3	73.7
0.425	1444.1	48.1	74.4	25.6
0.300	184.0	6.1	80.6	19.4
0.150	213.0	7.1	87.7	12.3
0.075	250.4	8.3	96.0	4.0
Pan	119.5	4.0	100.0	0.0
Sum	3000.000			



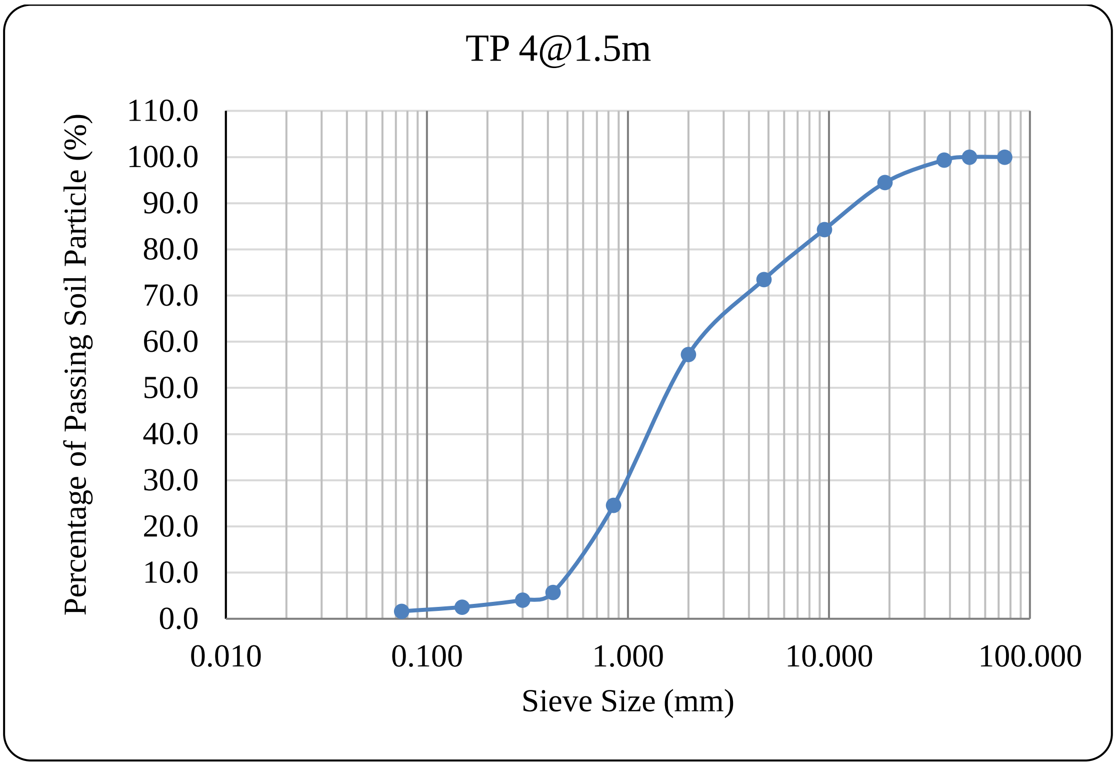
Test pit Location					TP 3@3m						
Types of Test Grain Size Analysis Test											
Wet Sieve Analysis					**Hydrometer Analysis-152H**						
Total washed soil specimen mass, g				500	Data for hydrometer analysis						
Sieve Size	Mass of Retain (g)	Percentage of Retain	Cumulative Percentage Retaining	Percentage of fine particles	Specific gravity of soil, (Gs)	Dried sample mass, (g)	Meniscus Correction (Cm)	Reading in Dispersant, Ro	Value of K for 22^{o}c	Value of K for 23^{o}c	Value of a
50.00	0	0.00	0.00	100.00	2.54	50	1	5	0.01381	0.0136	1.0281
37.50	0	0.00	0.00	100.00	Value of K for Use in Equations for Computing Diameter of Particle in Hydrometer Analysis						
19.00	52.55	1.75	1.75	98.25	Temp.(^{o}C)	Specific Gravity of Soil Particles					
9.50	162.5	5.42	7.17	92.83		2.45	2.5	2.55	2.6	2.65	2.7
4.75	110.5	3.68	10.86	89.14	18	0.0149	0.01467	0.01443	0.01421	0.0139	0.0137
2.00	72.04	2.40	13.26	86.74	19	0.0147	0.01449	0.01425	0.01403	0.0138	0.0136
0.850	135.9	4.53	17.79	82.21	20	0.0175	0.01431	0.01408	0.01386	0.0136	0.0134
0.425	689.7	22.99	40.78	59.22	21	0.0143	0.01414	0.01391	0.01369	0.0134	0.0132
0.250	212.6	7.09	47.87	52.13	22	0.0142	**0.01397**	**0.01374**	0.01353	0.0133	0.0131
0.150	390.1	13.00	60.87	39.13	23	0.0140	0.01381	0.01358	0.01337	0.0131	0.0129
0.075	226.9	7.57	68.44	31.56	24	0.0138	0.01365	0.01342	0.01321	0.0130	0.0128
Pan	946.9	31.56	100.00	0.00	25	0.0137	0.01349	0.01327	0.01306	0.0128	0.0126
Total	2947.45				26	0.0135	0.01334	0.01312	0.01291	0.0127	0.0125

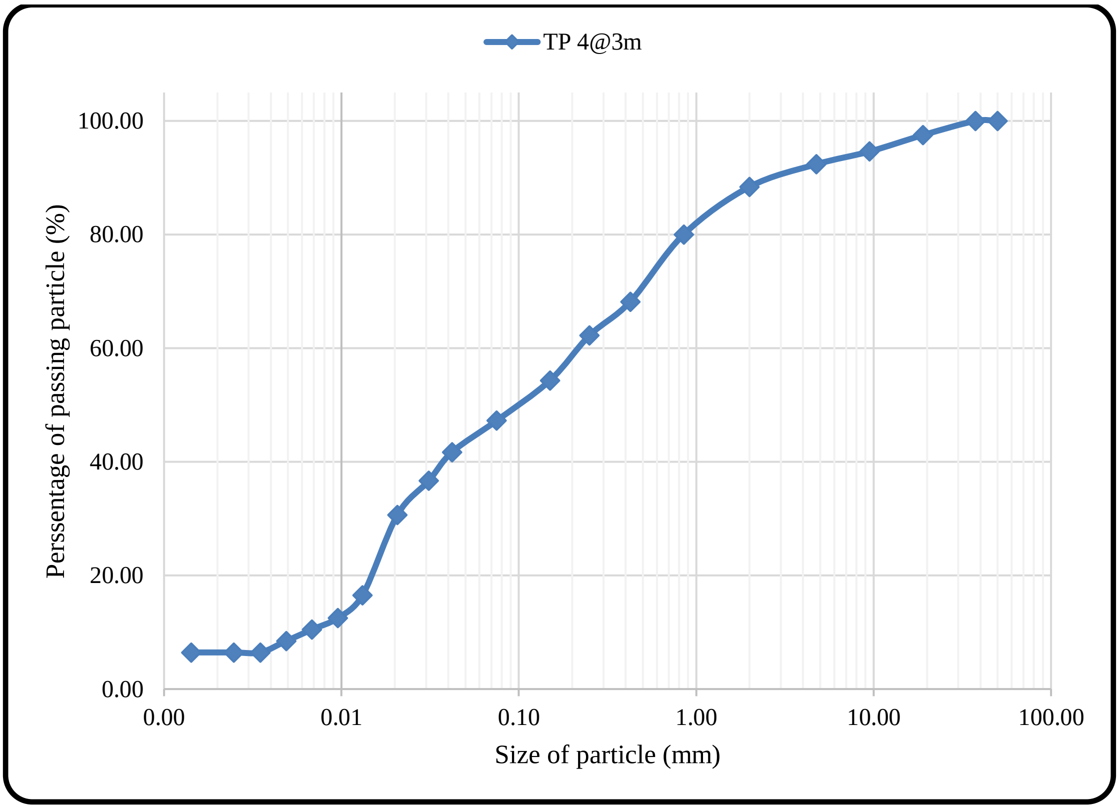
Time	Elapsed time t in, min	Actual Hydrometer Reading	Eff. depth, L (cm)	Temperature (c0)	K	C_T	Par. diameter D (mm)	Value of 'a' for Gs	Corrected hydrometer reading	Percent of Fine part., (%)	Adjusted percent fine’s part., (%)
07/28/2022/ 4:15:00 AM	0										
10:16:00 AM	1	42	9.2	22	0.01381	0.4	0.042	1.02816	38.4	79.0	24.92
10:17:00 AM	2	30	11.2	22	0.01381	0.4	0.033	1.02816	26.4	54.3	17.14
10:20:00 AM	5	25	12.0	22	0.01381	0.4	0.021	1.02816	21.4	44.0	13.89
10:30:00 AM	15	16	13.5	22	0.01381	0.4	0.013	1.02816	12.4	25.5	8.05
10:45:00 AM	30	13	14.0	22	0.01381	0.4	0.009	1.02816	9.4	19.3	6.10
11:15:00 AM	60	11	14.3	22	0.01381	0.4	0.007	1.02816	7.4	15.2	4.80
12:15:00 PM	120	10	14.5	22	0.01381	0.4	0.005	1.02816	6.4	13.2	4.15
2:15:00 PM	240	8	14.8	22	0.01381	0.4	0.003	1.02816	4.4	9.0	2.86
6:15:00 PM	480	7	15.0	22	0.01381	0.4	0.002	1.02816	3.4	7.0	2.21
29/07/2022 4:15 AM	1440	5	15.3	22	0.01381	0.4	0.001	1.02816	1.4	2.9	0.91



Pit number:	TP 4@1.5m	Sample Location:	Abijeta Lake Side (GDWUO)
Method of Testing:	Dry Sieve Analysis	Wt. of Sample: (kg)	3.000

Sieve size (mm)	Mass of Retain on Each Sieve (g)	Percentage of Retained Soil	Percentage of cumulative Retained Soil	Percentage of Passing Soil Particle (%)
75.000	0	0.0	0.0	100.0
50.000	0	0.0	0.0	100.0
37.500	18.98	0.6	0.6	99.4
19.000	147.2	4.9	5.5	94.5
9.500	304.4	10.1	15.7	84.3
4.750	324.9	10.8	26.5	73.5
2.000	487.0	16.2	42.7	57.3
0.850	979.8	32.7	75.4	24.6
0.425	565.5	18.9	94.3	5.7
0.300	50.2	1.7	95.9	4.1
0.150	45.6	1.5	97.5	2.5
0.075	27.5	0.9	98.4	1.6
Pan	30.8	1.0	99.4	0.6
Sum	2981.830			


TP 4@1.5m
Percentage of Passing Soil Particle (%)
110.0
100.0
90.0
80.0
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
0.010
0.100
1.000
10.000
100.000
Sieve Size (mm)


TP 4@3m
Perssentage of passing particle (%)
100.00
80.00
60.00
40.00
20.00
0.00
0.00
0.01
0.10
1.00
10.00
100.00
Size of particle (mm)

Test pit Location	TP 4@3m

Types of Test Grain Size Analysis Test

Wet Sieve Analysis

Total washed soil specimen mass, g				3000
Sieve Size	Mass of Retaining (g)	Percentage of Retaining	Cumulative Percentage Retaining	Percentage of fine particles
50.00	0	0	0	100
37.50	0	0	0	100
19.00	74.6	2.49	2.49	97.51
9.50	86.92	2.90	5.38	94.62
4.75	67.51	2.25	7.63	92.37
2.00	118.61	3.95	11.59	88.41
0.850	251.72	8.39	19.98	80.02
0.425	354.85	11.83	31.81	68.19
0.250	178.9	5.96	37.77	62.23
0.150	236.91	7.90	45.67	54.33
0.075	212.43	7.08	52.75	47.25
Pan	1417.6	47.25	100.00	0.00
Total	2925.4			

Hydrometer Analysis-152H

Data for hydrometer analysis

Specific gravity of soil, (Gs)	Dried sample mass, (g)	Meniscus Correction (Cm)	Reading in Dispersant, Ro	Value of K for 22^{o}c	Value of K for 23^{o}c	Value of a
2.40	50	1	5	0.01421	0.01404	1.06599

Value of K for Use in Equations for Computing Diameter of Particle in Hydrometer Analysis

Temp.(^{o}C)	Specific Gravity of Soil Particles					
	2.45	2.5	2.55	2.6	2.65	2.7
18	0.01492	0.01467	0.01443	0.01421	0.01399	0.01378
19	0.01474	0.01449	0.01425	0.01403	0.01382	0.01361
20	0.01756	0.01431	0.01408	0.01386	0.01365	0.01344
21	0.01438	0.01414	0.01391	0.01369	0.01348	0.01328
22	**0.01421**	0.01397	0.01374	0.01353	0.01332	0.01312
23	**0.01404**	0.01381	0.01358	0.01337	0.01317	0.01297
24	0.01388	0.01365	0.01342	0.01321	0.01301	0.01282
25	0.01372	0.01349	0.01327	0.01306	0.01286	0.01267
26	0.01357	0.01334	0.01312	0.01291	0.01272	0.01253

Time	Elapsed time t in, min	Actual Hydrometer Reading	Eff. Depth, L (cm)	Temperature (c^0)	K	C_T	Par. diameter D (mm)	Value of 'a' for Gs	Corrected hydrometer reading	Percent of Fine part., (%)	Adjusted percent fines part., (%)
07/28/2022 - 10:00:00 AM	0										
10:01:00 AM	1	45	8.8	22	0.01421	0.4	0.042	1.0661	41.4	88.3	41.71
10:02:00 AM	2	40	9.6	22	0.01421	0.4	0.031	1.0661	36.4	77.6	36.67
10:05:00 AM	5	34	10.6	22	0.01421	0.4	0.021	1.0661	30.4	64.8	30.63
10:15:00 AM	15	20	12.9	22	0.01421	0.4	0.013	1.0661	16.4	35.0	16.52
10:30:00 AM	30	16	13.5	22	0.01421	0.4	0.010	1.0661	12.4	26.4	12.49
11:00:00 AM	60	14	13.8	22	0.01421	0.4	0.007	1.0661	10.4	22.2	10.48
12:00:00 PM	120	12	14.2	22	0.01421	0.4	0.005	1.0661	8.4	17.9	8.46
2:00:00 PM	240	10	14.5	22	0.01421	0.4	0.003	1.0661	6.4	13.6	6.45
6:00:00 PM	480	10	14.5	22	0.01421	0.4	0.002	1.0661	6.4	13.6	6.45
7/29/2021-10:00:00 AM	1440	10	14.5	22	0.01421	0.4	0.001	1.0661	6.4	13.6	6.45

Test pit Location	TP 5@3m
Types of Test Grain Size Analysis Test	
Wet Sieve Analysis	**Hydrometer Analysis-152H**

Total washed soil specimen mass, g				3000
Sieve Size	Mass of Retaining (g)	Percentage of Retaining	Cumulative Percentage Retaining	Percentage of fine particles
50.00	0	0.00	0.00	100.00
37.50	139.12	4.64	4.64	95.36
19.00	406.72	13.56	18.19	81.81
9.50	269.27	8.98	27.17	72.83
4.75	130.38	4.35	31.52	68.48
2.00	133.32	4.44	35.96	64.04
0.850	255.09	8.50	44.46	55.54
0.425	270.41	9.01	53.48	46.52
0.250	59.54	1.98	55.46	44.54
0.150	185.89	6.20	61.66	38.34
0.075	250.4	8.35	70.00	30.00
Pan	899.86	30.00	100.00	
Total	3000 grams			

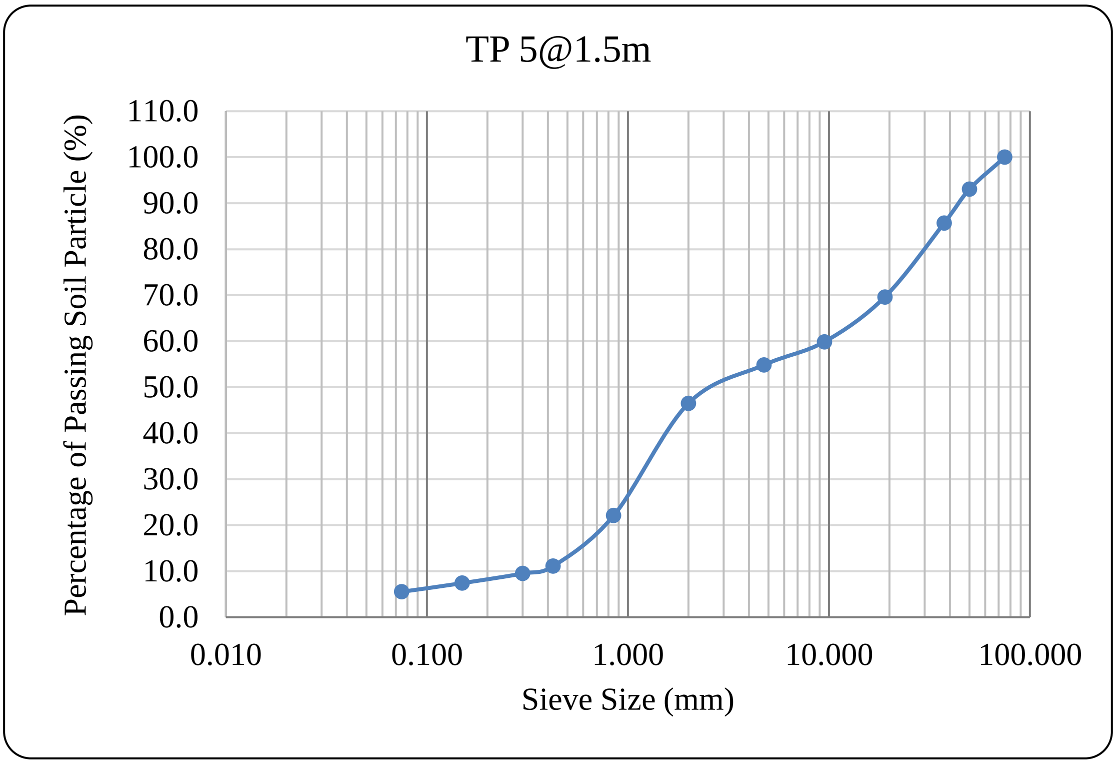
Data for hydrometer analysis						
Specific gravity of soil, (Gs)	Dried sample mass, (g)	Meniscus Correction (Cm)	Reading in Dispersant, Ro	Value of K for 22°c	Value of K for 23°c	Value of a
2.46	50	1	5	0.01417	0.01401	1.04979

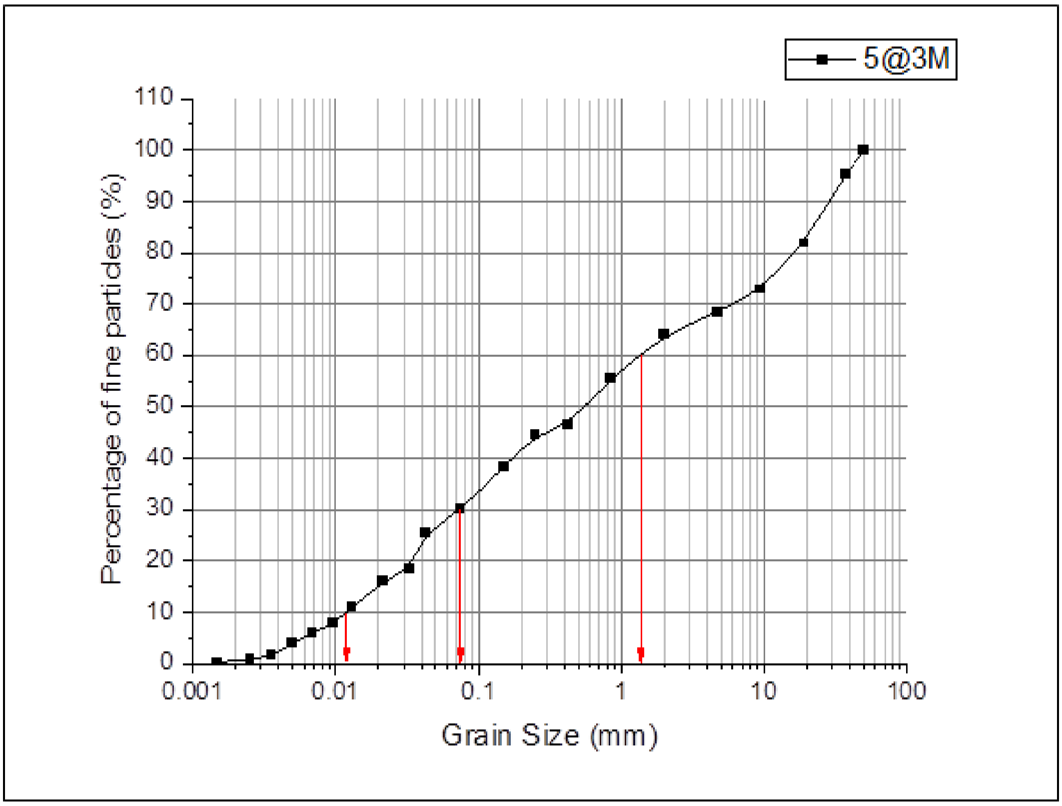
Value of K for Use in Equations for Computing Diameter of Particle in Hydrometer Analysis						
Temp.(°C)	Specific Gravity of Soil Particles					
	2.45	2.5	2.55	2.6	2.65	2.7
18	0.01492	0.01467	0.01443	0.01421	0.01399	0.01378
19	0.01474	0.01449	0.01425	0.01403	0.01382	0.01361
20	0.01756	0.01431	0.01408	0.01386	0.01365	0.01344
21	0.01438	0.01414	0.01391	0.01369	0.01348	0.01328
22	**0.01421**	0.01397	0.01374	0.01353	0.01332	0.01312
23	**0.01404**	0.01381	0.01358	0.01337	0.01317	0.01297
24	0.01388	0.01365	0.01342	0.01321	0.01301	0.01282
25	0.01372	0.01349	0.01327	0.01306	0.01286	0.01267
26	0.01357	0.01334	0.01312	0.01291	0.01272	0.01253

Time	Elapsed time t in, min	Actual Hydrometer Reading	Eff. depth, L (cm)	Temperature (c0)	K	C_T	Par. diameter D (mm)	Value of 'a' for Gs	Corrected hydrometer reading	Percent of Fine part., (%)	Adjusted percent fines part., (%)
10/28/2022/ 4:15:00 AM	0										
10:16:00 AM	1	44	8.9	22	0.01417	0.4	0.042	1.0498591	40.4	84.8	25.44
10:17:00 AM	2	33	10.7	22	0.01417	0.4	0.033	1.0498591	29.4	61.7	18.52
10:20:00 AM	5	29	11.4	22	0.01417	0.4	0.021	1.0498591	25.4	53.3	16.00
10:30:00 AM	15	21	12.7	22	0.01417	0.4	0.013	1.0498591	17.4	36.5	10.96
10:45:00 AM	30	16	13.5	22	0.01417	0.4	0.010	1.0498591	12.4	26.0	7.81
11:15:00 AM	60	13	14.0	22	0.01417	0.4	0.007	1.0498591	9.4	19.7	5.92
12:15:00 PM	120	10	14.5	22	0.01417	0.4	0.005	1.0498591	6.4	13.4	4.03
2:15:00 PM	240	6	15.2	22	0.01417	0.4	0.004	1.0498591	2.4	5.0	1.51
6:15:00 PM	480	5	15.3	22	0.01417	0.4	0.003	1.0498591	1.4	2.9	0.88
29/10/2022 4:15 AM	1440	4	15.5	22	0.01417	0.4	0.001	1.0498591	0.4	0.8	0.25

Pit number:	TP 5@1.5m	Sample Location:	Kebele office Area
Method of Testing:	Dry Sieve Analysis	Wt. of Sample: (kg)	3.000

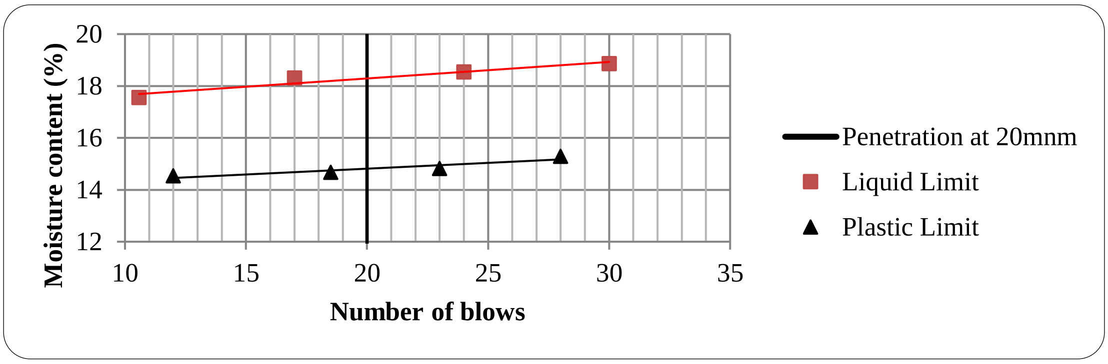
Sieve size (mm)	Mass of Retain on Each Sieve (g)	Percentage of Retained Soil	Percentage of cumulative Retained Soil	Percentage of Passing Soil Particle (%)
75.000	0	0.0	0.0	100.0
50.000	208.97	7.0	7.0	93.0
37.500	219.71	7.3	14.3	85.7
19.000	482.43	16.1	30.4	69.6
9.500	292.8	9.8	40.1	59.9
4.750	151.2	5.0	45.2	54.8
2.000	250.7	8.4	53.5	46.5
0.850	731.5	24.4	77.9	22.1
0.425	330.2	11.0	88.9	11.1
0.300	47.7	1.6	90.5	9.5
0.150	62.8	2.1	92.6	7.4
0.075	56.5	1.9	94.5	5.5
Pan	165.5	5.5	100.0	0.0
Sum	3000.000			



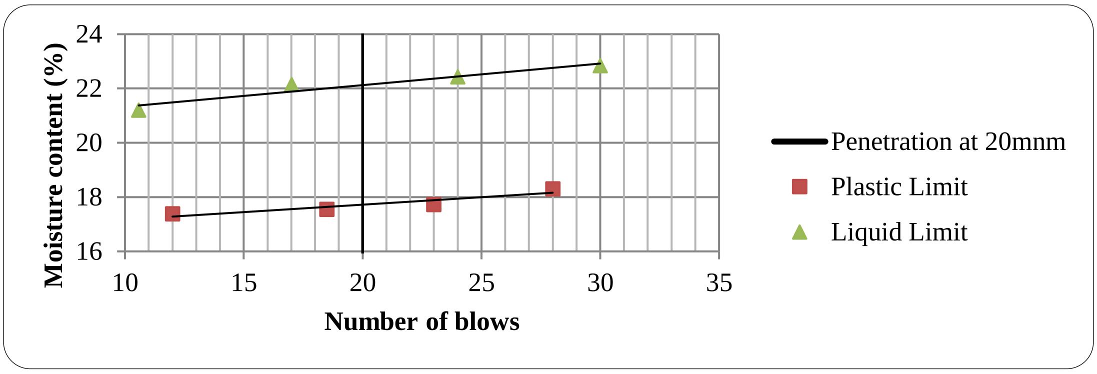


IV. Atterberge Limits

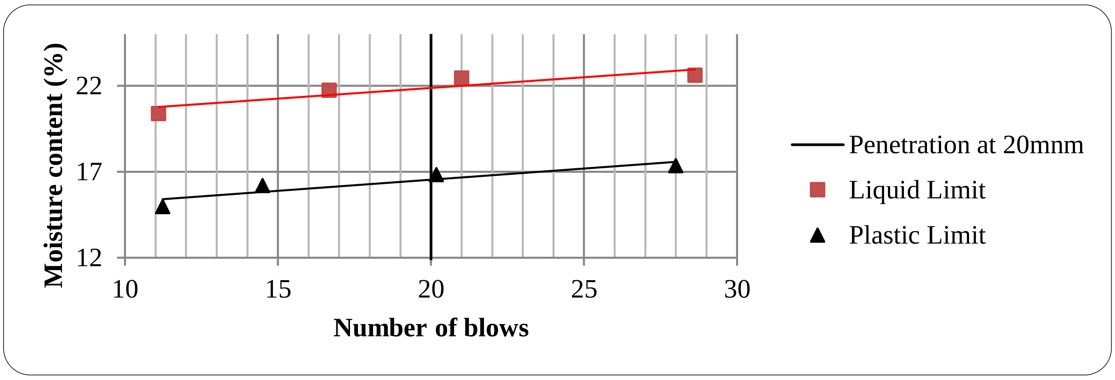
Pit Number	TP 1@1.5m							
Material location:	Langano Lake Side-1(RHW)							
Tests	Plastic Limit				Liquid Limit			
Penetration Depth mm	12.00	18.50	23.00	28.00	10.58	17.01	24.00	30.00
Test No	01	02	03	04	01	02	03	04
Wt. of Container, (g)	17.51	23.36	6.48	28.20	23.36	28.20	17.51	19.64
Wt. of container + wet soil, (g)	58.04	39.56	28.43	55.17	39.56	55.17	43.40	59.04
Wt. of container + dry soil, (g)	52.90	37.49	25.60	51.59	37.14	51.00	39.35	52.79
Moisture container, (%)	14.54	14.68	14.82	15.29	17.55	18.31	18.54	18.87
Liquid Limit, %	18.29	Plastic Limit, %		14.81	Plasticity Index, %		3.48	



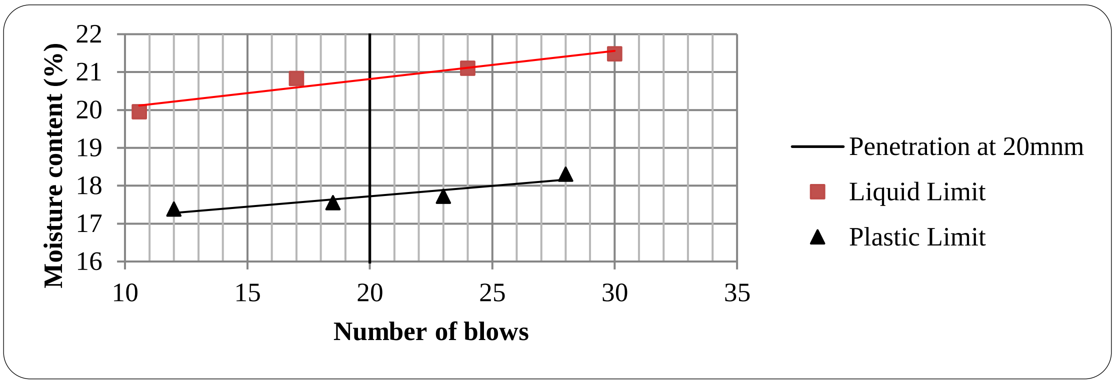
Pit Number	TP 1@3m							
Material location:	Langano Lake Side-1(RHW)							
Tests	Plastic Limit				Liquid Limit			
Penetration Depth mm	12	18.5	23	28	10.58	17.01	24	30
Test No	01	02	03	04	01	02	03	04
Wt. of Container, (g)	17.51	23.362	6.479	28.20	23.362	28.20	17.507	19.639
Wt. of container + wet soil, (g)	58.04	39.561	28.434	55.17	39.561	55.17	43.398	59.038
Wt. of container + dry soil, (g)	52.038	37.142	25.129	50.998	36.727	50.282	38.654	51.713
Moisture container, (%)	17.38	17.55	17.72	18.31	21.20	22.14	22.43	22.84
Liquid Limit, %	22.12	Plastic Limit, %		17.72	Plasticity Index, %		4.40	



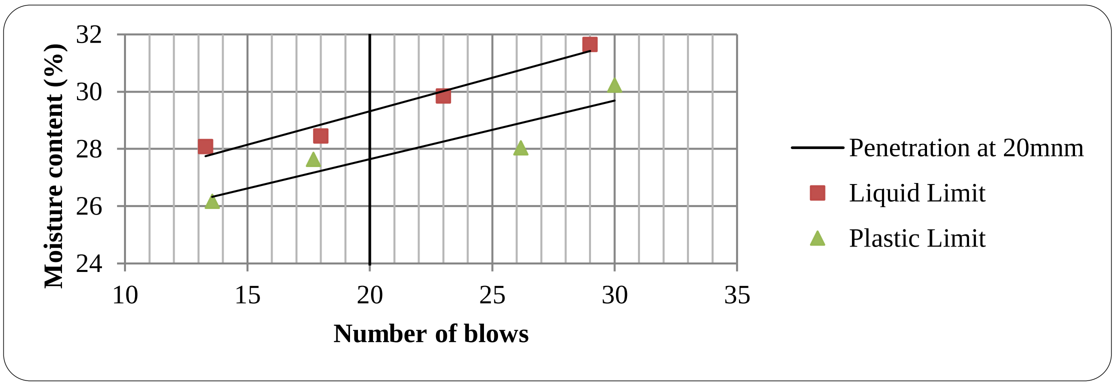
Pit Number	TP 2@1.5m							
Material location:	Langano Lake Side-2 (LHW)							
Tests	Plastic Limit				Liquid Limit			
Penetration Depth mm	11.23	14.5	20.17	28	11.1	16.67	21	28.63
Test No	01	02	03	04	01	02	03	04
Wt. of Container, (g)	17.42	19.679	19.359	19.92	19.359	20.21	17.045	16.3
Wt. of container + wet soil, (g)	32.154	36.894	42.116	52.34	42.116	48.12	43.875	38.582
Wt. of container + dry soil, (g)	30.235	34.494	38.837	47.547	38.263	43.14	38.956	34.474
Moisture container, (%)	14.97	16.20	16.83	17.36	20.38	21.74	22.45	22.60
Liquid Limit, %	21.87	Plastic Limit, %		16.54	Plasticity Index, %		5.33	



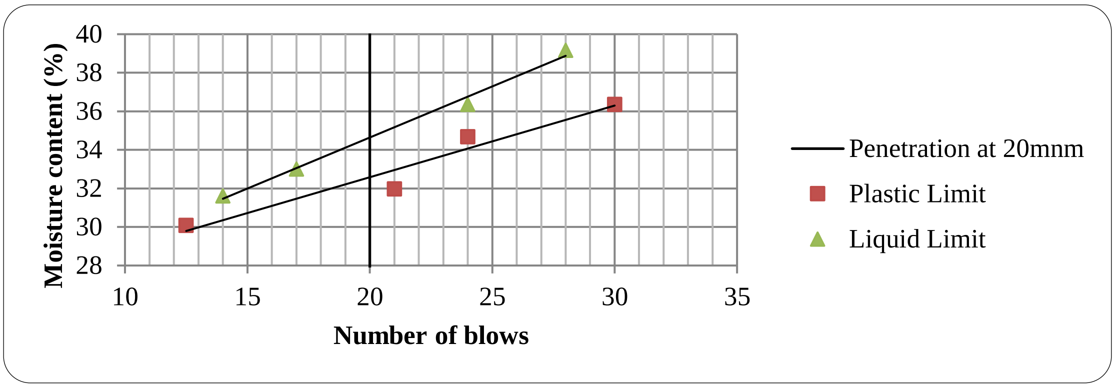
Pit Number	TP 2@3m							
Material location:	Langano Lake Side-2 (LHW)							
Tests	Plastic Limit				Liquid Limit			
Penetration Depth mm	12	18.5	23	28	10.58	17.01	24	30
Test No	01	02	03	04	01	02	03	04
Wt. of Container, (g)	17.51	23.362	6.479	28.20	23.362	28.20	17.507	19.639
Wt. of container + wet soil, (g)	58.04	39.561	28.434	55.17	39.561	55.17	43.398	59.038
Wt. of container + dry soil, (g)	52.038	37.142	25.129	50.998	36.866	50.521	38.886	52.07
Moisture container, (%)	17.38	17.55	17.72	18.31	19.96	20.84	21.10	21.49
Liquid Limit, %	20.82	Plastic Limit, %		17.72	Plasticity Index, %		3.10	



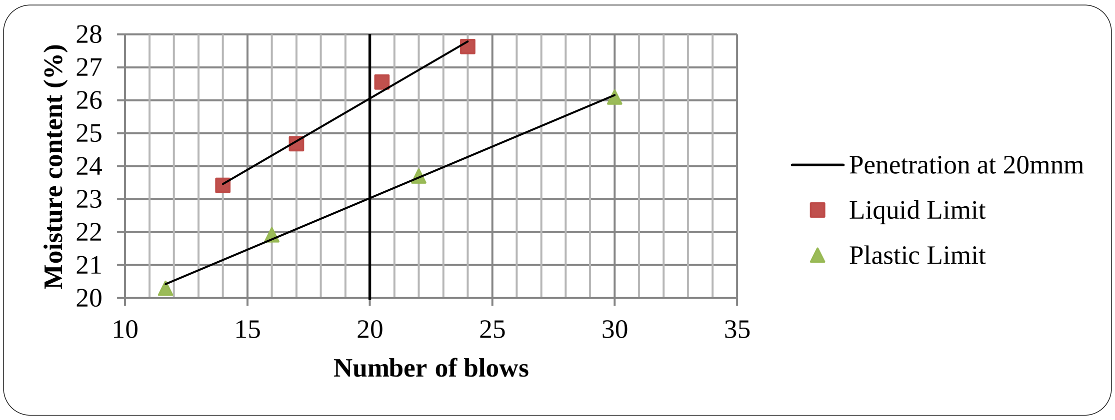
Pit Number	TP 3@1.5m							
Material location:	Shala Lake Side (HC)							
Tests	Plastic Limit				Liquid Limit			
Penetration Depth mm	13.56	17.7	26.17	30	13.29	18	23	29
Test No	01	02	03	04	01	02	03	04
Wt. of Container, (g)	35.255	26.687	32.003	17.24	26.687	17.25	32.003	25.227
Wt. of container + wet soil, (g)	58.406	65.755	91.555	41.23	65.755	40.12	91.555	51.344
Wt. of container + dry soil, (g)	53.603	57.295	78.512	35.661	57.188	35.057	77.867	45.066
Moisture container, (%)	26.18	27.64	28.04	30.23	28.09	28.46	29.84	31.64
Liquid Limit, %	29.32	Plastic Limit, %		27.64	Plasticity Index, %		1.67	



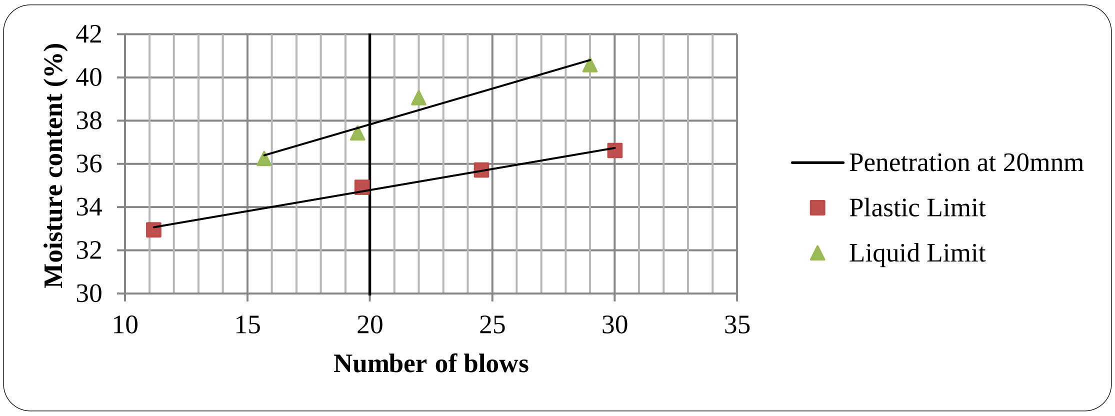
Pit Number	TP 3@3m							
Material location:	Shala Lake Side (HC)							
Tests	Plastic Limit				Liquid Limit			
Penetration Depth mm	12.5	21	24	30	14	17	24	28
Test No	01	02	03	04	01	02	03	04
Wt. of Container, (g)	17.989	17.678	31.74	37.72	19.659	17.68	37.723	17.14
Wt. of container + wet soil, (g)	33.592	52.129	71.058	64.37	44.618	52.13	64.366	37.89
Wt. of container + dry soil, (g)	29.984	43.78	60.934	57.26	38.623	43.58	57.261	32.05
Moisture container, (%)	30.08	31.99	34.68	36.37	31.61	33.01	36.37	39.17
Liquid Limit, %	34.64	Plastic Limit, %		32.58	Plasticity Index, %		2.06	



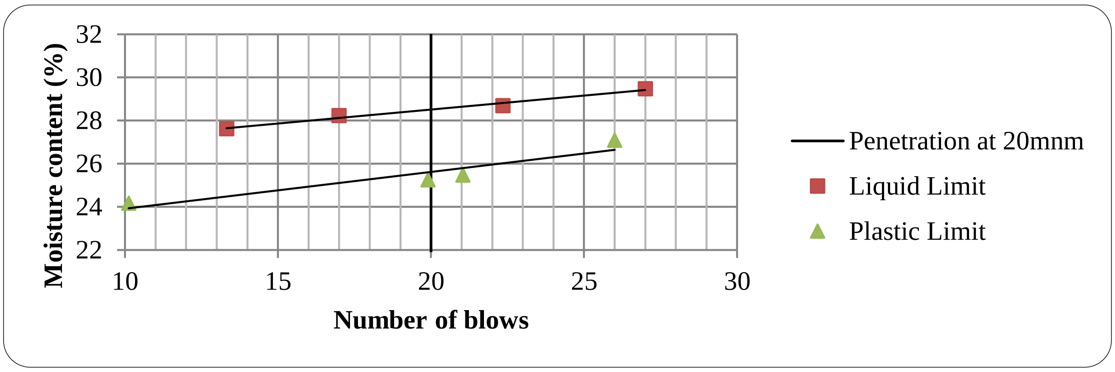
Pit Number	TP 4@1.5m							
Material location:	Abijeta Lake Side (GDWUO)							
Tests	Plastic Limit				Liquid Limit			
Penetration Depth mm	11.66	16	22	30	14	17	20.5	24
Test No	01	02	03	04	01	02	03	04
Wt. of Container, (g)	18.092	26.687	17.595	17.69	25.87	17.05	17.712	24.157
Wt. of container + wet soil, (g)	32.523	65.755	37.304	54.79	55.755	41.24	48.83	51.344
Wt. of container + dry soil, (g)	30.088	58.732	33.528	47.111	50.085	36.452	42.302	45.459
Moisture container, (%)	20.30	21.92	23.70	26.10	23.42	24.68	26.55	27.63
Liquid Limit, %	26.05	Plastic Limit, %		23.03	Plasticity Index, %		3.02	



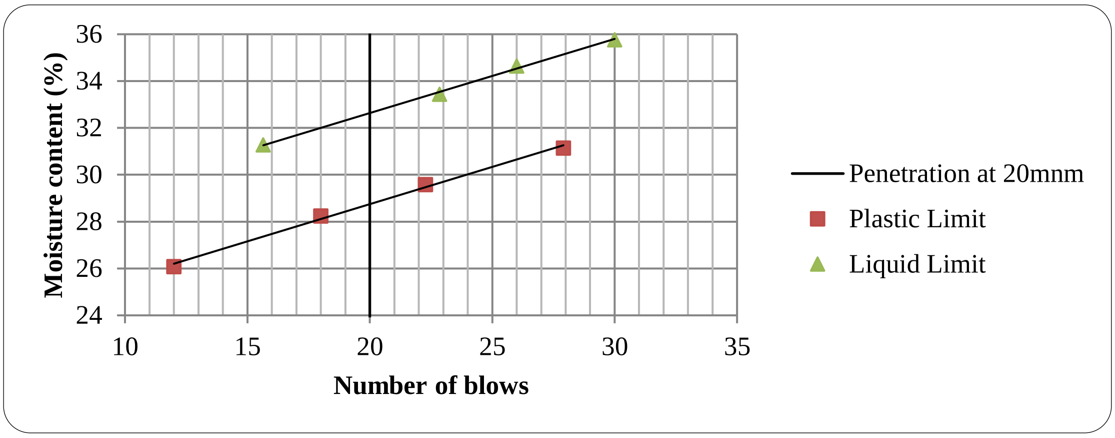
Pit Number	TP 4@3m							
Material location:	Abijeta Lake Side (GDWUO)							
Tests	Plastic Limit				Liquid Limit			
Penetration Depth mm	11.18	19.69	24.56	30.01	15.69	19.5	22	29
Test No	01	02	03	04	01	02	03	04
Wt. of Container, (g)	39.883	24.519	37.16	17.28	31.386	32.75	17.461	25.351
Wt. of container + wet soil, (g)	61.693	56.086	73.714	61.81	56.536	65.01	85.412	59.144
Wt. of container + dry soil, (g)	56.288	47.916	64.093	49.875	49.845	56.223	66.324	49.387
Moisture container, (%)	32.95	34.92	35.72	36.62	36.25	37.44	39.06	40.59
Liquid Limit, %	37.82	Plastic Limit, %		34.79	Plasticity Index, %		3.04	



Pit Number	TP 5@1.5m							
Material location:	Kebele office Area							
Tests	Plastic Limit				Liquid Limit			
Penetration Depth mm	10.12	19.9	21.04	26	13.32	17	22.35	27
Test No	01	02	03	04	01	02	03	04
Wt. of Container, (g)	17.889	25.277	17.117	18.31	17.179	17.82	17.472	17.395
Wt. of container + wet soil, (g)	38.643	75.031	59.288	57.41	49.621	46.12	53.984	57.608
Wt. of container + dry soil, (g)	34.603	65.001	50.727	49.08	42.601	39.89	45.846	48.453
Moisture container, (%)	24.17	25.25	25.47	27.08	27.61	28.23	28.68	29.48
Liquid Limit, %	28.51	Plastic Limit, %		25.62	Plasticity Index, %		2.89	



Pit Number	TP 5@3m							
Material location:	Kebele office Area							
Tests	Plastic Limit				Liquid Limit			
Penetration Depth mm	12	18	22.27	27.91	15.65	22.85	26	30
Test No	01	02	03	04	01	02	03	04
Wt. of Container, (g)	17.301	17.097	17.203	17.65	17.776	17.50	17.045	25.351
Wt. of container + wet soil, (g)	49.482	70.672	93.324	108.89	75.903	59.09	86.741	59.144
Wt. of container + dry soil, (g)	42.824	58.875	75.948	87.224	62.055	48.668	68.806	50.242
Moisture container, (%)	26.09	28.24	29.58	31.14	31.27	33.44	34.65	35.76
Liquid Limit, %	32.64	Plastic Limit, %		28.75	Plasticity Index, %		3.89	



V. Specific Gravity

Test pit Location	TP 1@1.5m	
Test Pit No	Langano Lake Side-1(RHW)	
Code of Pycnometer	1	2
Mass of Pycnometer	29.465	32.851
Mass of Pycnometer with Dry Soil	40.373	43.615
Mass of Pycnometer with Dry Soil and Water	133.111	131.782
Mass of Pycnometer and Water	126.740	125.496
Temperature of Pycnometer with Water (T^0_{ci})	21	21
Density of water at (T^0_{ci})	0.99802	0.99802
Temperature of Pycnometer with Soil and Water (T^0_{cx})	20	20
Density of water at (T^0_{cx})	0.99823	0.99823
Corrected Mass of Pycnometer and Water	126.76667	125.52241
Correction Factor, K at 20^0_c	1.0000	1.0000
Specific Gravity, Gs at 20^0_c	2.3902	2.3897
Average Specific Gravity, Gs at 20^0_c	**2.39**	

Test pit Location	TP 1@3m	
Test Pit No	Langano Lake Side-1(RHW)	
Code of Pycnometer	1	2
Mass of Pycnometer	29.465	32.851
Mass of Pycnometer with Dry Soil	40.373	43.615
Mass of Pycnometer with Dry Soil and Water	133.311	131.962
Mass of Pycnometer and Water	126.740	125.496
Temperature of Pycnometer with Water (T^{0}_{ci})	21	21
Density of water at (T^{0}_{ci})	0.99802	0.99802
Temperature of Pycnometer with Soil and Water (T^{0}_{cx})	20	20
Density of water at (T^{0}_{cx})	0.99823	0.99823
Corrected Mass of Pycnometer and Water	126.76667	125.52241
Correction Factor, K at 20^{0}_{c}	1.0000	1.0000
Specific Gravity, Gs at 20^{0}_{c}	2.4997	2.4891
Average Specific Gravity, Gs at 20^{0}_{c}	**2.49**	

Test pit Location	TP 2@1.5m	
Test Pit No	Langano Lake Side-2 (LHW)	
Code of Pycnometer	1	2
Mass of Pycnometer	28.966	31.940
Mass of Pycnometer with Dry Soil	39.623	44.863
Mass of Pycnometer with Dry Soil and Water	132.172	134.530
Mass of Pycnometer and Water	125.789	126.917
Temperature of Pycnometer with Water (T^{0}_{ci})	21	21
Density of water at (T^{0}_{ci})	0.99802	0.99802
Temperature of Pycnometer with Soil and Water (T^{0}_{cx})	20	20
Density of water at (T^{0}_{cx})	0.99823	0.99823
Corrected Mass of Pycnometer and Water	125.815468	126.943705
Correction Factor, K at 20^{0}_{c}	1.0000	1.0000
Specific Gravity, Gs at 20^{0}_{c}	2.4781	2.4215
Average Specific Gravity, Gs at 20^{0}_{c}	**2.45**	

Test pit Location	TP 2@3m	
Test Pit No	Langano Lake Side-2 (LHW)	
Code of Pycnometer	1	2
Mass of Pycnometer	29.465	32.851
Mass of Pycnometer with Dry Soil	40.373	43.615
Mass of Pycnometer with Dry Soil and Water	133.511	132.082
Mass of Pycnometer and Water	126.740	125.496
Temperature of Pycnometer with Water (T^0_{ci})	21	21
Density of water at (T^0_{ci})	0.99802	0.99802
Temperature of Pycnometer with Soil and Water (T^0_{cx})	20	20
Density of water at (T^0_{cx})	0.99823	0.99823
Corrected Mass of Pycnometer and Water	126.7666682	125.5224064
Correction Factor, K at 20^0_c	1.0000	1.0000
Specific Gravity, Gs at 20^0_c	2.6198	2.5602
Average Specific Gravity, Gs at 20^0_c	**2.59**	

Test pit Location	TP 3@1.5m	
Test Pit No	Shala Lake Side (HC)	
Code of Pycnometer	1	2
Mass of Pycnometer	30.648	29.766
Mass of Pycnometer with Dry Soil	42.550	40.298
Mass of Pycnometer with Dry Soil and Water	133.165	128.710
Mass of Pycnometer and Water	125.955	122.296
Temperature of Pycnometer with Water (T^0_{ci})	20	20
Density of water at (T^0_{ci})	0.99823	0.99823
Temperature of Pycnometer with Soil and Water (T^0_{cx})	20	20
Density of water at (T^0_{cx})	0.99823	0.99823
Corrected Mass of Pycnometer and Water	125.955	122.296
Correction Factor, K at 20^0_c	1.0000	1.0000
Specific Gravity, Gs at 20^0_c	2.5367	2.5576
Average Specific Gravity, Gs at 20^0_c	**2.55**	

Test pit Location	TP 3@3m	
Test Pit No	Shala Lake Side (HC)	
Code of Pycnometer	1	2
Mass of Pycnometer	30.075	30.849
Mass of Pycnometer with Dry Soil	40.670	41.143
Mass of Pycnometer with Dry Soil and Water	132.466	132.027
Mass of Pycnometer and Water	125.909	125.885
Temperature of Pycnometer with Water (T^0_{ci})	21	21
Density of water at (T^0_{ci})	0.99802	0.99802
Temperature of Pycnometer with Soil and Water (T^0_{cx})	20	20
Density of water at (T^0_{cx})	0.99823	0.99823
Corrected Mass of Pycnometer and Water	125.935493	125.911488
Correction Factor, K at 20^0_c	1.0000	1.0000
Specific Gravity, Gs at 20^0_c	2.6067	2.4636
Average Specific Gravity, Gs at 20^0_c	**2.54**	

Test pit Location	TP 4@1.5m	
Test Pit No	Abijeta Lake Side (GDWUO)	
Code of Pycnometer	1	2
Mass of Pycnometer	30.837	30.430
Mass of Pycnometer with Dry Soil	50.555	50.229
Mass of Pycnometer with Dry Soil and Water	138.420	136.286
Mass of Pycnometer and Water	126.525	124.454
Temperature of Pycnometer with Water (T^0_{ci})	21	21
Density of water at (T^0_{ci})	0.99802	0.99802
Temperature of Pycnometer with Soil and Water (T^0_{cx})	20	20
Density of water at (T^0_{cx})	0.99823	0.99823
Corrected Mass of Pycnometer and Water	126.55162	124.480187
Correction Factor, K at 20^0_c	1.0000	1.0000
Specific Gravity, Gs at 20^0_c	2.5120	2.4770
Average Specific Gravity, Gs at 20^0_c	**2.49**	

Test pit Location	TP 4@3m	
Test Pit No	Abijeta Lake Side (GDWUO)	
Code of Pycnometer	1	2
Mass of Pycnometer	30.071	33.933
Mass of Pycnometer with Dry Soil	40.968	44.498
Mass of Pycnometer with Dry Soil and Water	132.291	134.532
Mass of Pycnometer and Water	125.916	128.319
Temperature of Pycnometer with Water (T^0_{ci})	21	21
Density of water at (T^0_{ci})	0.99802	0.99802
Temperature of Pycnometer with Soil and Water (T^0_{cx})	20	20
Density of water at (T^0_{cx})	0.99823	0.99823
Corrected Mass of Pycnometer and Water	125.942495	128.346
Correction Factor, K at 20^0_c	1.0000	1.0000
Specific Gravity, Gs at 20^0_c	2.3957	2.4127
Average Specific Gravity, Gs at 20^0_c	**2.40**	

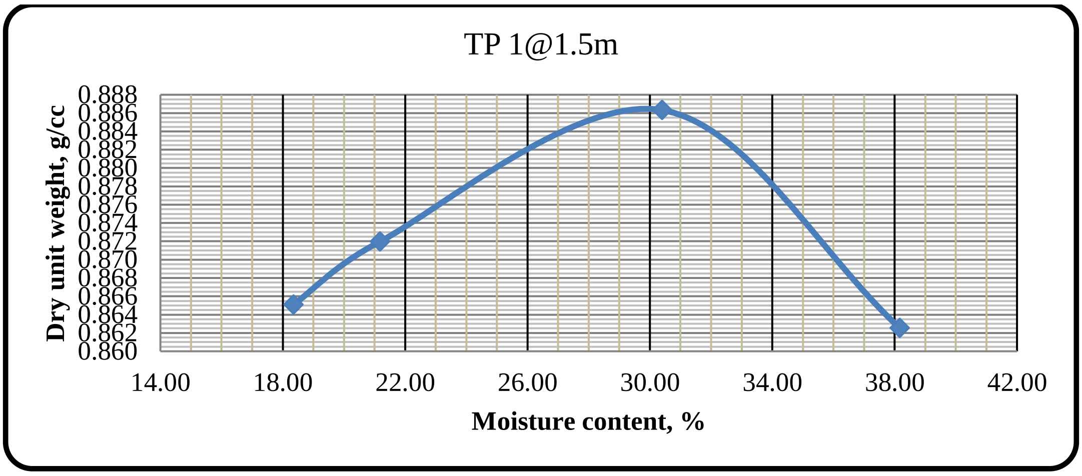
Test pit Location	TP 5@1.5m	
Test Pit No	Kebele office Area	
Code of Pycnometer	1	2
Mass of Pycnometer	31.204	28.504
Mass of Pycnometer with Dry Soil	51.263	50.521
Mass of Pycnometer with Dry Soil and Water	138.251	138.188
Mass of Pycnometer and Water	126.082	124.768
Temperature of Pycnometer with Water (T^0_{ci})	21	21
Density of water at (T^0_{ci})	0.99802	0.99802
Temperature of Pycnometer with Soil and Water (T^0_{cx})	20	20
Density of water at (T^0_{cx})	0.99823	0.99823
Corrected Mass of Pycnometer and Water	126.10853	124.794253
Correction Factor, K at 20^0_c	1.0000	1.0000
Specific Gravity, Gs at 20^0_c	2.5338	2.5532
Average Specific Gravity, Gs at 20^0_c	**2.54**	

Test pit Location	TP 5@3m	
Test Pit No	Kebele office Area	
Code of Pycnometer	1	2
Mass of Pycnometer	28.374	28.661
Mass of Pycnometer with Dry Soil	41.692	40.910
Mass of Pycnometer with Dry Soil and Water	132.697	130.746
Mass of Pycnometer and Water	124.763	123.464
Temperature of Pycnometer with Water (T^0_{ci})	21	21
Density of water at (T^0_{ci})	0.99802	0.99802
Temperature of Pycnometer with Soil and Water (T^0_{cx})	20	20
Density of water at (T^0_{cx})	0.99823	0.99823
Corrected Mass of Pycnometer and Water	124.789252	123.489979
Correction Factor, K at 20^0_c	1.0000	1.0000
Specific Gravity, Gs at 20^0_c	2.4616	2.4532
Average Specific Gravity, Gs at 20^0_c	**2.46**	

VI. Proctor Compaction

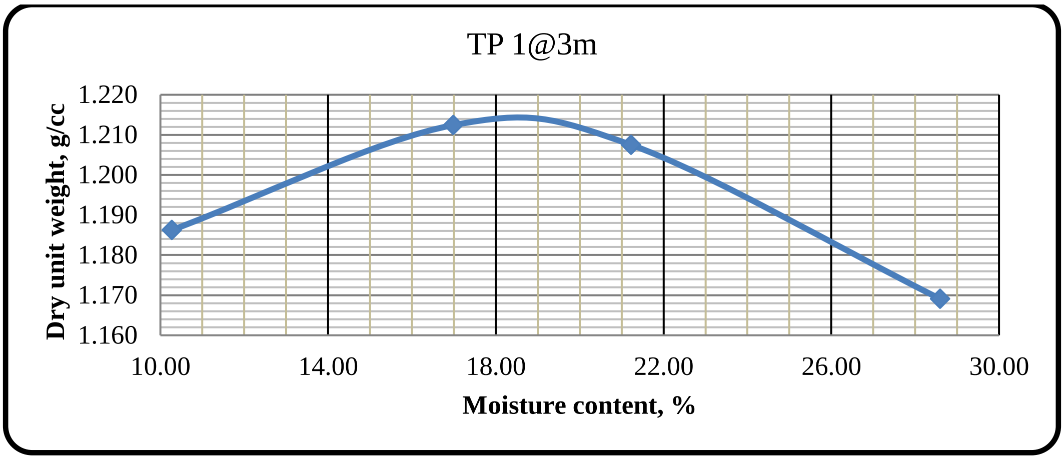
Test pit Name and Depth	TP 1@1.5m		
Test pit Location	Langano Lake Side-1(RHW)		
Mass of mold, (gm)	1817	Volume of mold, (cm^3)	944

Determination of water content					
Trials	01	02	03	04	05
Mass of can	39.979	32.029	36.057	37.798	
Mass of can +wet soil	188.556	127.612	165.96	191.595	
Mass of can +dry soil	165.519	110.905	135.68	149.113	
water content	18.35	21.18	30.39	38.16	
Average water content, (%)	**18.35**	**21.18**	**30.39**	**38.16**	
Determination of Bulk density and Dry density					
Mass of mold + Compacted soil, (gm)	2783.5	2814.5	2908	2942	
Bulk density, (g/cm^3)	1.02	1.06	1.16	1.19	
Dry density, (g/cm^3)	0.87	0.87	0.89	0.86	
Maximum Dry Density (g/cm^3)	***0.887***	***Optimum Water Content (%)***			***30***



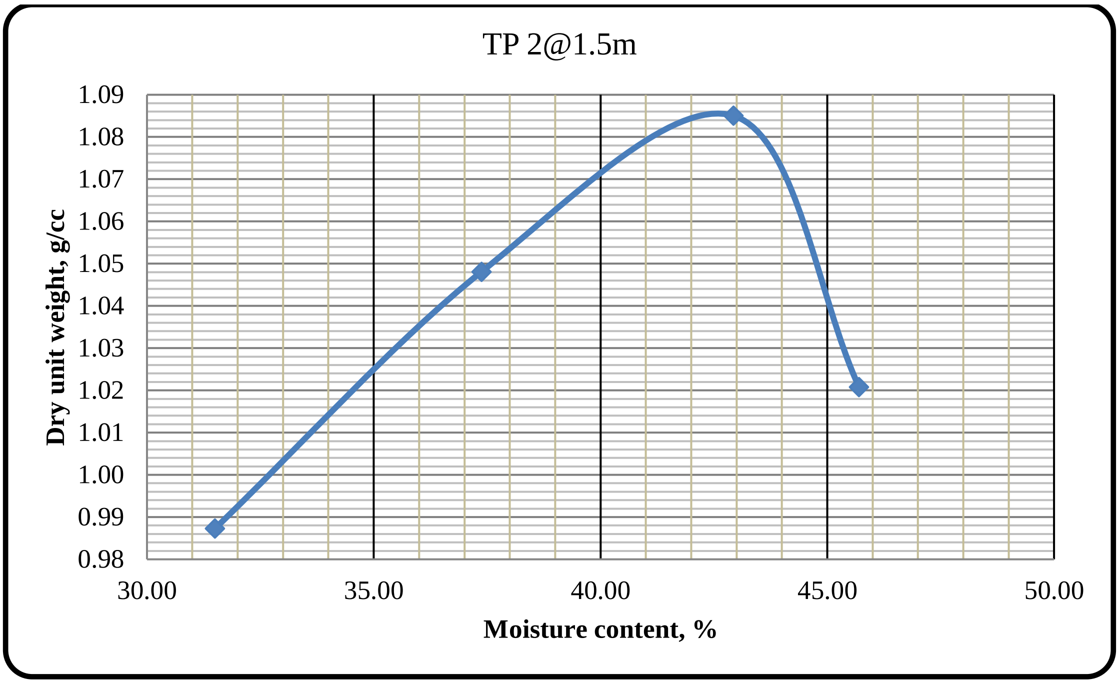
Test pit Name and Depth	TP 1@3m		
Test pit Location	Langano Lake Side-1(RHW)		
Mass of mold, (gm)	1817	Volume of mold, (cm^3)	944

Determination of water content					
Trials	01	02	03	04	05
Mass of can	35.981	28.826	32.451	34.018	
Mass of can +wet soil	169.7	114.851	149.364	172.436	
Mass of can +dry soil	157.243	102.36	128.896	141.657	
water content	10.27	16.99	21.22	28.59	
Average water content, (%)	**10.27**	**16.99**	**21.22**	**28.59**	
Determination of Bulk density and Dry density					
Mass of mold + Compacted soil, (gm)	3051.85	3155.95	3198.8	3236.2	
Bulk density, (g/cm^3)	1.31	1.42	1.46	1.50	
Dry density, (g/cm^3)	1.19	1.21	1.21	1.17	
Maximum Dry Density ***(g/cm^3)***	***1.215***	***Optimum Water Content (%)***			***18.5***



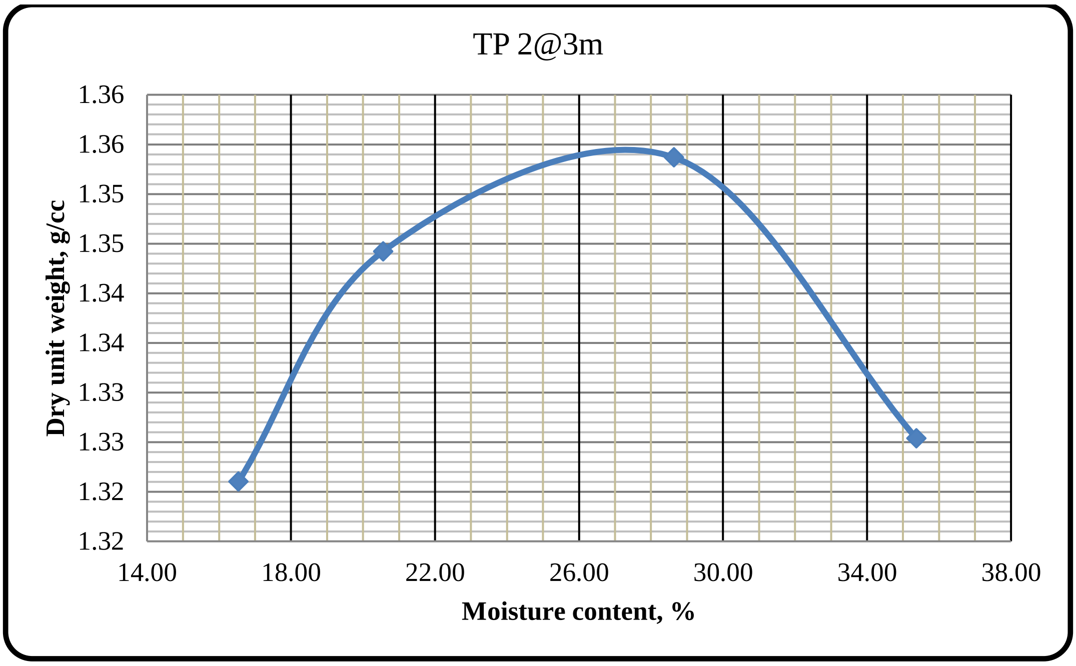
Test pit Name and Depth	TP 2@1.5m		
Test pit Location	Langano Lake Side-2 (LHW)		
Mass of mold, (gm)	1817	Volume of mold, (cm^3)	944

Determination of water content					
Trials	01	02	03	04	05
Mass of can	24.561	25.396	37.898	17.276	17.276
Mass of can +wet soil	160.939	169.55	122.735	92.58	92.58
Mass of can +dry soil	135.863	135.02	99.653	69.96	68.96
water content	22.53	31.50	37.38	42.94	45.70
Average water content, (%)	**22.53**	**31.50**	**37.38**	**42.94**	**45.70**
Determination of Bulk density and Dry density					
Mass of mold + Compacted soil, (gm)	2973	3042.56	3176.2	3281	3221
Bulk density, (g/cm^3)	1.22	1.30	1.44	1.55	1.49
Dry density, (g/cm^3)	1.00	0.99	1.05	1.09	1.02
Maximum Dry Density (g/cm^3)	***1.086***	***Optimum Water Content (%)***			***43***



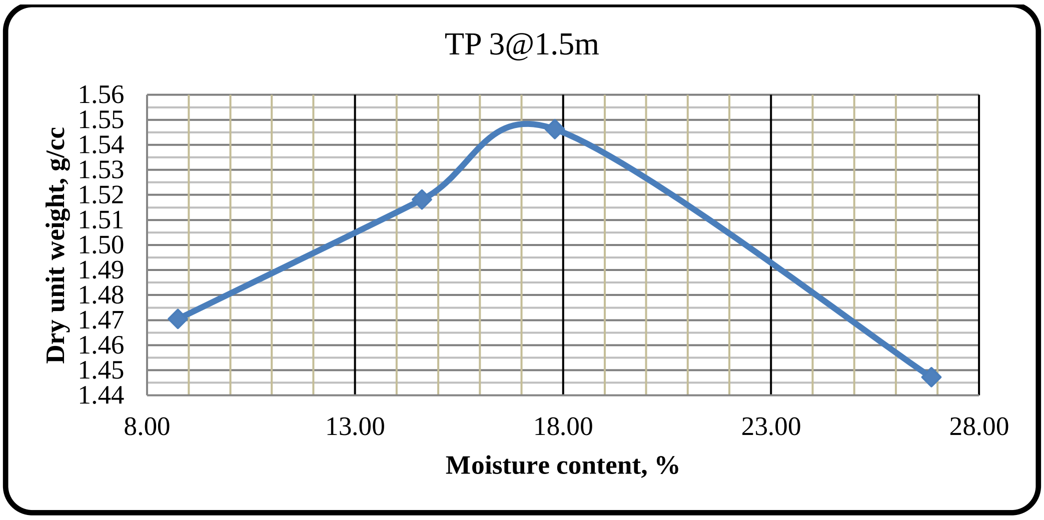
Test pit Name and Depth	TP 2@3m		
Test pit Location	Langano Lake Side-2 (LHW)		
Mass of mold, (gm)	1817	Volume of mold, (cm^3)	944

Determination of water content					
Trials	01	02	03	04	05
Mass of can	23.333	24.126	36.003	16.412	
Mass of can +wet soil	152.892	161.073	116.598	87.951	
Mass of can +dry soil	134.504	137.72	98.656	69.26	
water content	16.54	20.56	28.64	35.37	
Average water content, (%)	**16.54**	**20.56**	**28.64**	**35.37**	
Determination of Bulk density and Dry density					
Mass of mold + Compacted soil, (gm)	3270.3	3346.82	3460.82	3510.67	
Bulk density, (g/cm^3)	1.54	1.62	1.74	1.79	
Dry density, (g/cm^3)	1.32	1.34	1.35	1.33	
Maximum Dry Density (g/cm^3)	***1.36***	***Optimum Water Content (%)***			***27***



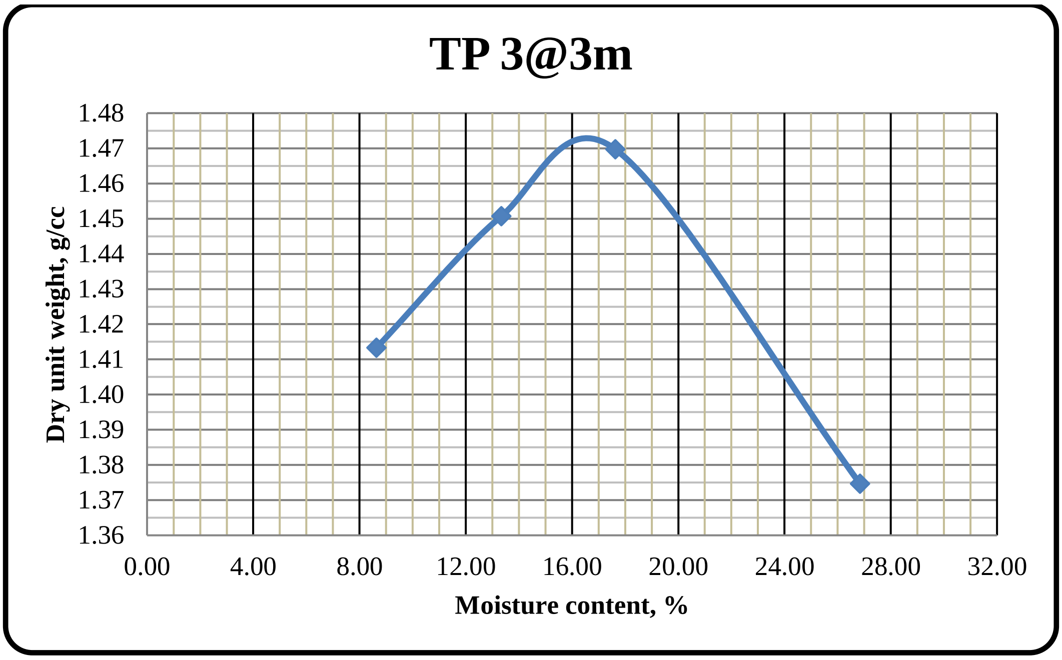
Test pit Name and Depth	TP 3@1.5m			
Test pit Lacotion	Shala Lake Side (HC)			
Mass of mold, (gm)	1817	Volume of mold, (cm^3)	944	

Determination of water content					
Trials	01	02	03	04	05
Mass of can	17.091	17.312	17.533	17.945	
	17.747	17.448	17.025	17.686	
Mass of can +wet soil	118.775	113.523	143.529	98.157	
	128.325	157.454	138.179	110.204	
Mass of can +dry soil	110.491	101.311	124.2	81.212	
	119.552	139.53	120.15	90.577	
water content	8.87	14.54	18.12	26.78	
	8.62	14.68	17.48	26.93	
Average water content, (%)	**8.74**	**14.61**	**17.80**	**26.85**	
Determination of Bulk density and Dry density					
Mass of mold + Compacted soil, (gm)	3326.5	3459.5	3536.5	3550	
Bulk density, (g/cm^3)	1.60	1.74	1.82	1.84	
Dry density, (g/cm^3)	1.47	1.52	1.55	1.45	
Maximum Dry Density (g/cm^3)	***1.547***	***Optimum Water Content (%)***			***19***



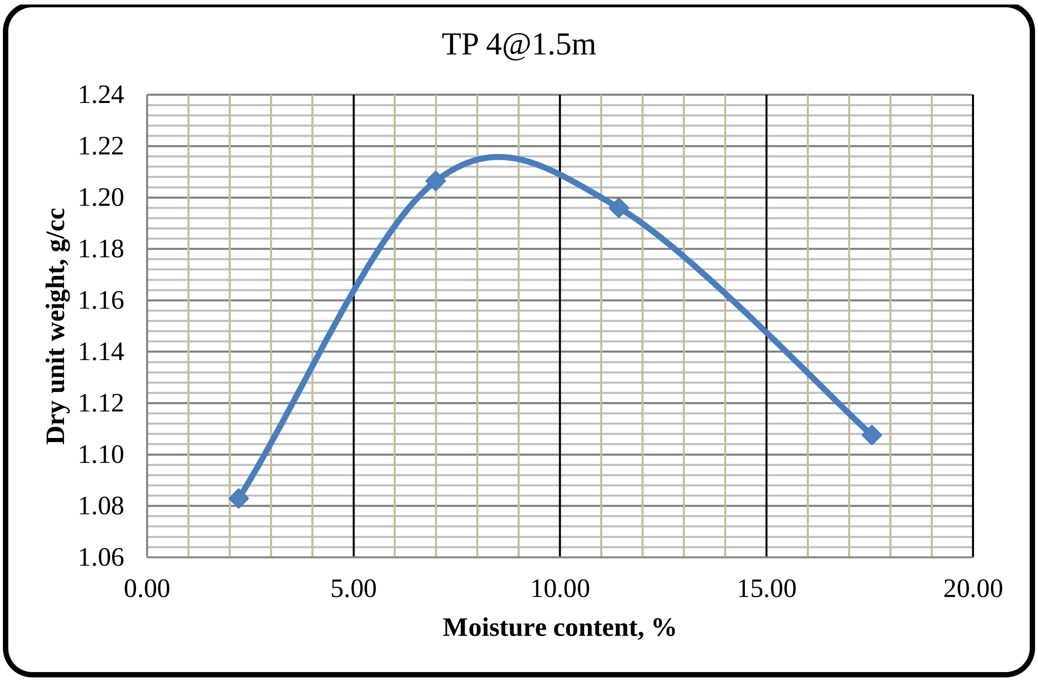
Test pit Name and Depth	TP 3@3m		
Test pit Location	Shala Lake Side (HC)		
Mass of mold, (gm)	1817	Volume of mold, (cm^3)	944

Determination of water content					
Trials	01	02	03	04	05
Mass of can	17.912	17.37	18.908	25.264	
	17.205	17.738	22.046	19.882	
Mass of can +wet soil	122.425	107.348	112.058	106.487	
	119.465	99.258	92.006	88.521	
Mass of can +dry soil	113.951	96.815	98.039	89.172	
	111.489	89.621	81.568	74.099	
water content	8.82	13.26	17.72	27.09	
	8.46	13.41	17.54	26.60	
Average water content, (%)	**8.64**	**13.33**	**17.63**	**26.85**	
Determination of Bulk density and Dry density					
Mass of mold + Compacted soil, (gm)	3266.5	3369	3449	3463	
Bulk density, (g/cm^3)	1.54	1.64	1.73	1.74	
Dry density, (g/cm^3)	1.41	1.45	1.47	1.37	
Maximum Dry Density (g/cm^3)	***1.472***	***Optimum Water Content (%)***			***16***



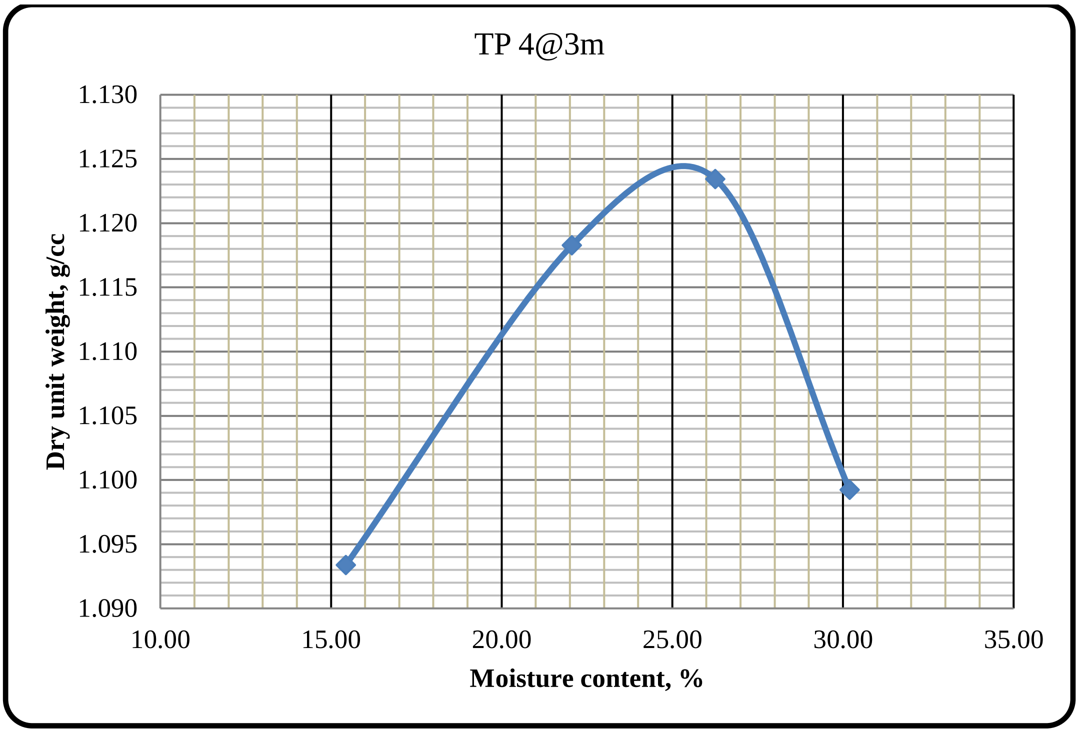
Test pit Name and Depth	TP 4@1.5m		
Test pit Location	Abijeta Lake Side (GDWUO)		
Mass of mold, (gm)	1817	Volume of mold, (cm^3)	944

Determination of water content					
Trials	01	02	03	04	05
Mass of can	26.688	35.281	32.729	37.177	
Mass of can +wet soil	161.951	195.562	183.753	193.109	
Mass of can +dry soil	159.01	185.087	168.271	169.83	
water content	2.22	6.99	11.42	17.55	
Average water content, (%)	**2.22**	**6.99**	**11.42**	**17.55**	
Determination of Bulk density and Dry density					
Mass of mold + Compacted soil, (gm)	2862	3035.5	3075	3046	
Bulk density, (g/cm^3)	1.11	1.29	1.33	1.30	
Dry density, (g/cm^3)	1.08	1.21	1.20	1.11	
Maximum Dry Density (g/cm^3)	***1.218***	***Optimum Water Content (%)***			***8***



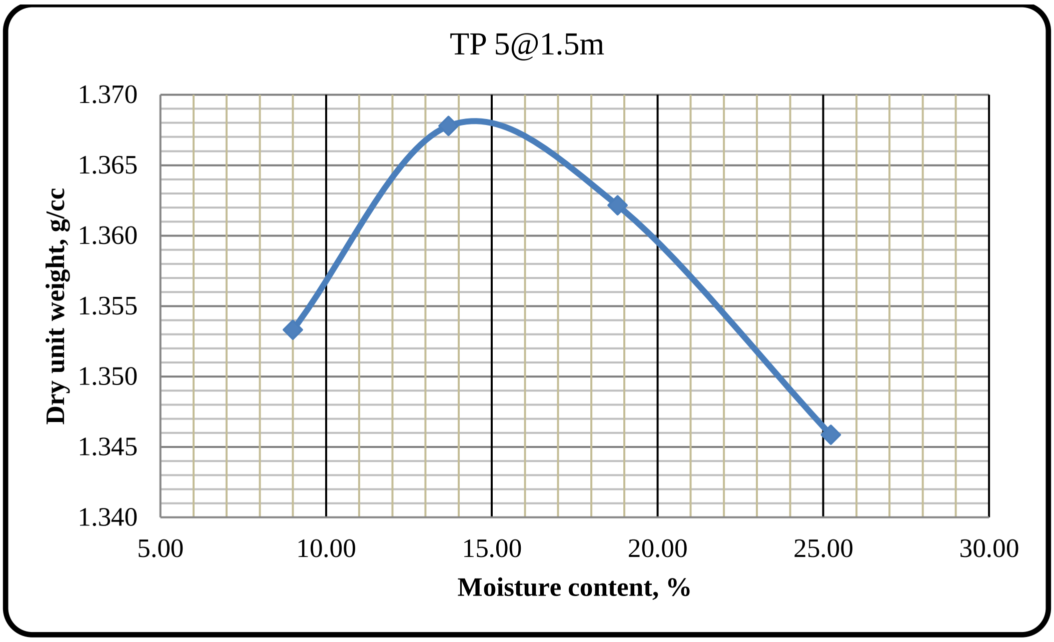
Test pit Name and Depth	TP 4@3m		
Test pit Location	Abijeta Lake Side (GDWUO)		
Mass of mold, (gm)	1817	Volume of mold, (cm^3)	944

Determination of water content					
Trials	01	02	03	04	05
Mass of can	18.102	17.482	17.077	17.98	17.81
	17.819	17.635	17.502	17.284	17.344
Mass of can +wet soil	83.417	91.93	88.27	100.874	112.04
	75.499	91.885	81.021	93.296	96.325
Mass of can +dry soil	76.665	81.893	75.427	83.874	89.874
	69.468	82.036	69.521	77.27	78.27
water content	11.53	15.58	22.01	25.80	30.76
	11.68	15.29	22.11	26.72	29.63
Average water content, (%)	**11.60**	**15.44**	**22.06**	**26.26**	**30.20**
Determination of Bulk density and Dry density					
Mass of mold + Compacted soil, (gm)	2958.5	3008.5	3105.5	3156	3168
Bulk density, (g/cm^3)	1.21	1.26	1.36	1.42	1.43
Dry density, (g/cm^3)	1.08	1.09	1.12	1.12	1.10
Maximum Dry Density (g/cm^3)	***1.125***	***Optimum Water Content (%)***			***25***



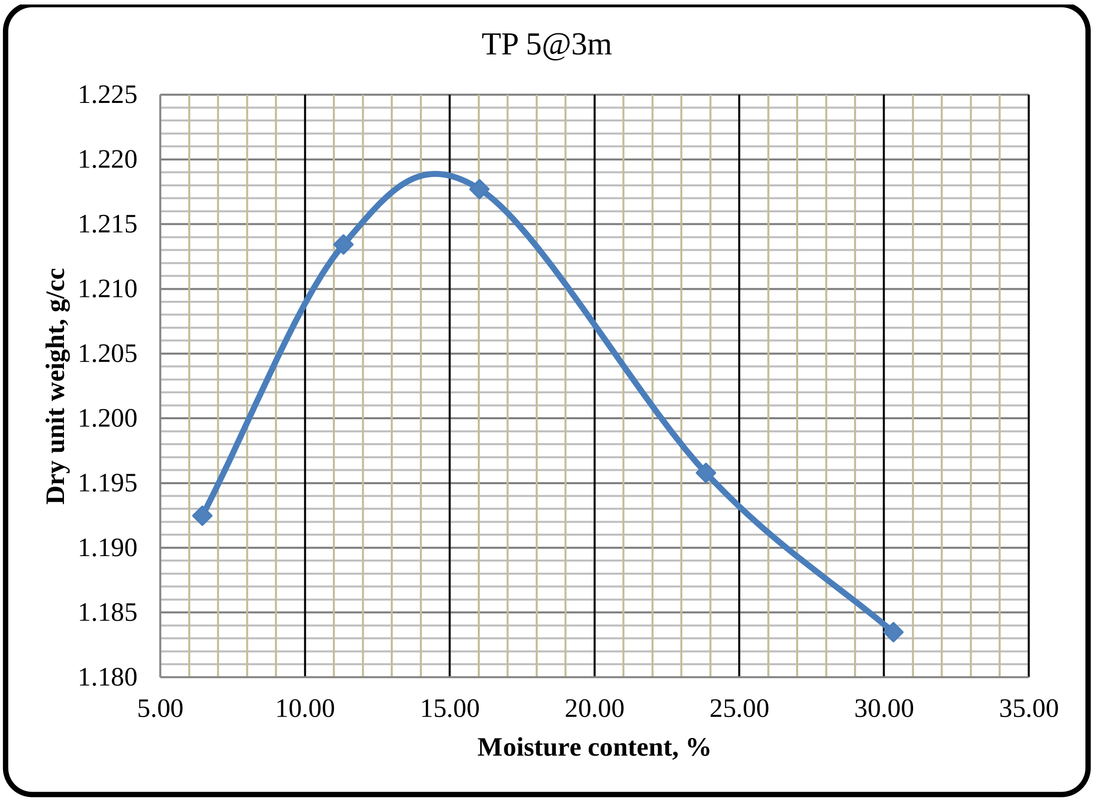
Test pit Name and Depth	TP 5@1.5m		
Test pit Location	Kebele office Area		
Mass of mold, (gm)	1817	Volume of mold, (cm^3)	944

Determination of water content					
Trials	01	02	03	04	05
Mass of can	32.785	31.388	25.241	37.726	
Mass of can +wet soil	214.898	203.689	209.091	201.512	
Mass of can +dry soil	199.862	182.936	180.008	168.516	
water content	9.00	13.69	18.79	25.23	
Average water content, (%)	**9.00**	**13.69**	**18.79**	**25.23**	
Determination of Bulk density and Dry density					
Mass of mold + Compacted soil, (gm)	3209.5	3285	3344.5	3408	
Bulk density, (g/cm^3)	1.48	1.56	1.62	1.69	
Dry density, (g/cm^3)	1.35	1.37	1.36	1.35	
Maximum Dry Density (g/cm^3)	***1.368***	***Optimum Water Content (%)***	***14.5***		



Test pit Name and Depth	TP 5@3m		
Test pit Location	Kebele office Area		
Mass of mold, (gm)	1817	Volume of mold, (cm^3)	944

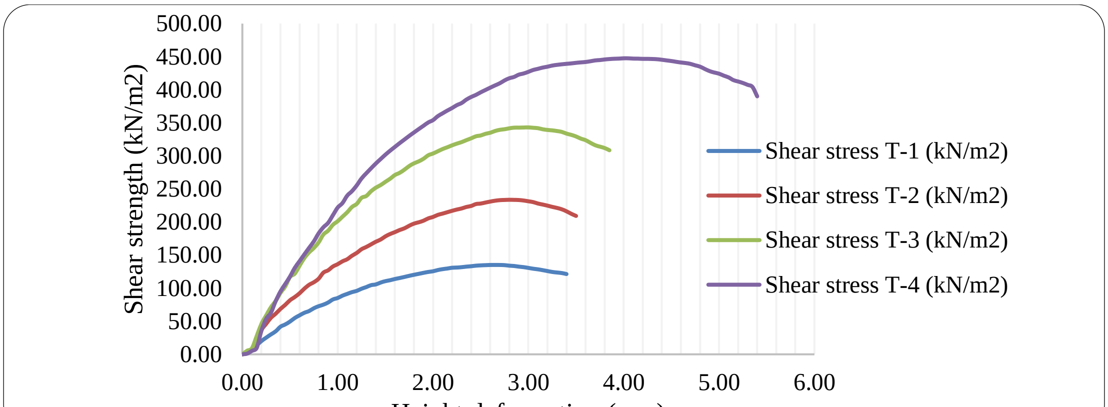
Determination of water content					
Trials	01	02	03	04	05
Mass of can	17.174	17.799	17.182	17.036	17.994
	17.316	17.455	17.89	17.549	17.639
Mass of can +wet soil	98.557	111.786	85.124	100.852	111.547
	97.449	93.706	99.952	107.668	104.466
Mass of can +dry soil	93.566	102.164	75.861	84.839	89.639
	92.651	85.995	88.469	90.182	84.391
water content	6.53	11.41	15.79	23.62	30.58
	6.37	11.25	16.27	24.07	30.07
Average water content, (%)	**6.45**	**11.33**	**16.03**	**23.85**	**30.33**
Determination of Bulk density and Dry density					
Mass of mold + Compacted soil, (gm)	3015.3	3092.22	3150.76	3215	3273
Bulk density, (g/cm^3)	1.27	1.35	1.41	1.48	1.54
Dry density, (g/cm^3)	1.19	1.21	1.22	1.20	1.18
Maximum Dry Density (g/cm^3)	***1.219***	***Optimum Water Content (%)***			***15***

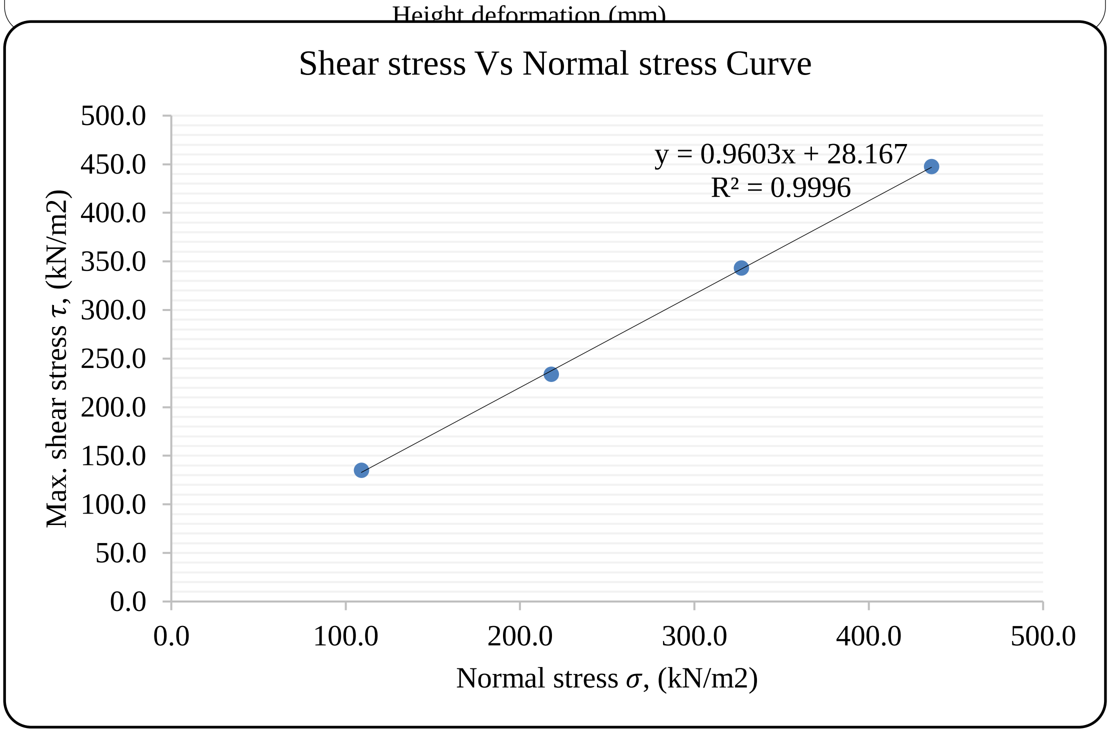


VII. Direct Shear Strength

Test Type:	Direct Shear Test
Type of Sample:	Undisturbed Soil Sample
Test Pit Site Name	TP 1@1.5m

Trials	1	2	3	3	C, kN/m^2	Angle of Friction, Ø
Normal Stress (kN/m2)	109.0	218.0	327.0	436.0	28.2	43.8
Max. Shear Stress (kN/m2)	135.1	233.6	343.1	447.6		





Deformation height (mm)	Resistance load reading T-1 (N)	Resistance load reading T-2 (N)	Resistance load reading T- 3 (N)	Resistance load reading T- 4 (N)	Area of sampler (m^2)	Shear stress T-1 (kN/m^2)	Shear stress T-2 (kN/m^2)	Shear stress T-3 (kN/m^2)	Shear stress T-4 (kN/m^2)
0.00	0.07	0.00	0.00	0.00	0.0036	0.02	0.00	0.00	0.00
0.05	7.37	10.22	19.67	3.85	0.0036	2.05	2.84	5.46	1.07
0.10	27.67	28.99	32.48	18.79	0.0036	7.69	8.05	9.02	5.22
0.15	49.42	71.04	98.60	34.35	0.0036	13.73	19.73	27.39	9.54
0.20	72.18	132.18	165.31	128.22	0.0036	20.05	36.72	45.92	35.62
0.25	90.82	166.16	212.01	193.24	0.0036	25.23	46.16	58.89	53.68
0.30	108.52	198.87	256.46	226.72	0.0036	30.14	55.24	71.24	62.98
0.35	125.50	221.36	290.94	290.08	0.0036	34.86	61.49	80.82	80.58
0.40	150.42	247.42	334.47	342.61	0.0036	41.78	68.73	92.91	95.17

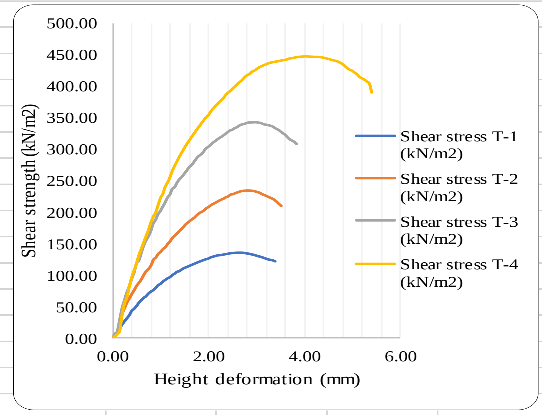
0.45	162.71	269.36	369.79	382.64	0.0036	45.20	74.82	102.72	106.29
0.50	178.53	294.54	421.35	423.45	0.0036	49.59	81.82	117.04	117.63
0.55	197.68	312.05	439.34	470.65	0.0036	54.91	86.68	122.04	130.74
0.60	212.27	332.32	482.20	507.09	0.0036	58.96	92.31	133.94	140.86
0.65	226.00	357.35	523.96	543.63	0.0036	62.78	99.26	145.55	151.01
0.70	235.68	378.34	554.93	579.46	0.0036	65.47	105.09	154.15	160.96
0.75	250.85	392.36	578.79	614.95	0.0036	69.68	108.99	160.77	170.82
0.80	261.83	411.52	609.34	658.30	0.0036	72.73	114.31	169.26	182.86
0.85	270.43	444.55	652.19	691.60	0.0036	75.12	123.49	181.16	192.11
0.90	282.28	457.46	673.28	715.81	0.0036	78.41	127.07	187.02	198.84
0.95	298.61	478.05	705.09	757.32	0.0036	82.95	132.79	195.86	210.37
1.00	306.56	490.72	724.51	798.84	0.0036	85.15	136.31	201.25	221.90
1.05	319.35	506.17	749.03	823.22	0.0036	88.71	140.60	208.06	228.67
1.10	329.03	517.65	773.22	863.77	0.0036	91.40	143.79	214.78	239.94
1.15	338.56	536.58	802.01	888.16	0.0036	94.05	149.05	222.78	246.71
1.20	345.43	552.10	818.92	919.62	0.0036	95.95	153.36	227.48	255.45
1.25	357.06	572.06	851.73	957.12	0.0036	99.18	158.91	236.59	265.87
1.30	366.02	584.42	861.86	985.35	0.0036	101.67	162.34	239.41	273.71
1.35	376.49	598.51	887.64	1012.44	0.0036	104.58	166.25	246.57	281.23
1.40	380.47	612.85	906.97	1038.57	0.0036	105.69	170.24	251.94	288.49
1.45	390.58	624.65	921.03	1062.61	0.0036	108.50	173.51	255.84	295.17
1.50	398.24	641.84	938.86	1086.38	0.0036	110.62	178.29	260.80	301.77
1.55	403.37	654.90	955.85	1108.41	0.0036	112.05	181.92	265.52	307.89
1.60	410.02	665.20	976.03	1128.95	0.0036	113.89	184.78	271.12	313.60
1.65	415.37	676.61	987.99	1149.48	0.0036	115.38	187.95	274.44	319.30
1.70	421.22	685.95	1005.15	1169.06	0.0036	117.00	190.54	279.21	324.74
1.75	427.29	699.81	1024.74	1188.73	0.0036	118.69	194.39	284.65	330.20
1.80	432.92	711.45	1039.55	1207.52	0.0036	120.26	197.63	288.77	335.42
1.85	438.12	718.74	1050.35	1225.96	0.0036	121.70	199.65	291.76	340.54
1.90	443.54	727.37	1064.50	1243.96	0.0036	123.21	202.05	295.69	345.55
1.95	448.46	739.97	1083.58	1262.14	0.0036	124.57	205.55	300.99	350.60
2.00	452.14	747.81	1093.12	1274.73	0.0036	125.59	207.72	303.65	354.09
2.05	458.79	758.97	1105.17	1296.23	0.0036	127.44	210.83	306.99	360.06
2.10	463.56	765.94	1117.31	1311.61	0.0036	128.77	212.76	310.36	364.34
2.15	467.10	773.55	1127.10	1326.82	0.0036	129.75	214.87	313.08	368.56
2.20	471.29	781.23	1137.65	1340.63	0.0036	130.91	217.01	316.01	372.40
2.25	472.37	788.12	1146.94	1356.27	0.0036	131.21	218.92	318.59	376.74
2.30	474.39	793.90	1155.31	1367.81	0.0036	131.78	220.53	320.92	379.95
2.35	477.36	802.14	1165.77	1386.43	0.0036	132.60	222.82	323.83	385.12
2.40	479.52	807.52	1175.73	1400.76	0.0036	133.20	224.31	326.59	389.10

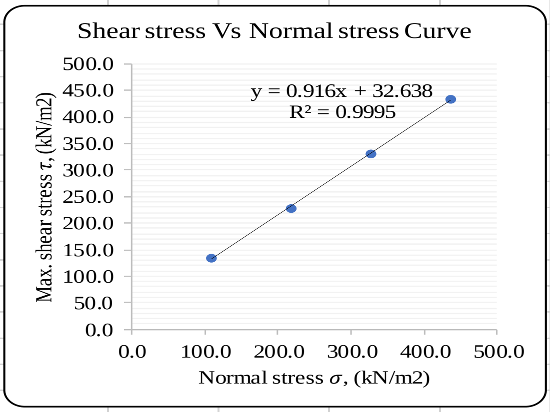
2.45	482.56	817.82	1186.78	1412.03	0.0036	134.04	227.17	329.66	392.23
2.50	484.22	820.67	1191.30	1426.19	0.0036	134.51	227.96	330.92	396.16
2.55	485.30	826.14	1200.26	1438.69	0.0036	134.81	229.48	333.41	399.64
2.60	486.31	831.44	1206.12	1451.45	0.0036	135.09	230.96	335.03	403.18
2.65	486.46	836.04	1215.49	1463.51	0.0036	135.13	232.23	337.64	406.53
2.70	486.31	839.12	1221.94	1475.31	0.0036	135.09	233.09	339.43	409.81
2.75	485.45	840.31	1225.20	1490.69	0.0036	134.85	233.42	340.33	414.08
2.80	482.49	841.10	1230.14	1502.58	0.0036	134.02	233.64	341.71	417.38
2.85	480.82	840.71	1233.65	1509.92	0.0036	133.56	233.53	342.68	419.42
2.90	477.28	839.76	1233.99	1522.25	0.0036	132.58	233.27	342.77	422.85
2.95	474.54	837.22	1234.58	1528.54	0.0036	131.82	232.56	342.94	424.59
3.00	470.35	832.95	1235.16	1537.63	0.0036	130.65	231.37	343.10	427.12
3.05	466.01	828.59	1232.98	1547.59	0.0036	129.45	230.16	342.50	429.89
3.10	462.69	820.83	1230.89	1553.88	0.0036	128.52	228.01	341.91	431.63
3.15	458.14	815.28	1224.95	1560.70	0.0036	127.26	226.47	340.26	433.53
3.20	453.30	809.27	1221.27	1565.51	0.0036	125.92	224.80	339.24	434.86
3.25	448.38	802.61	1218.92	1571.63	0.0036	124.55	222.95	338.59	436.56
3.30	445.35	796.83	1215.32	1575.47	0.0036	123.71	221.34	337.59	437.63
3.35	442.53	789.47	1210.72	1578.53	0.0036	122.93	219.30	336.31	438.48
3.40	436.97	778.46	1201.26	1581.15	0.0036	121.38	216.24	333.68	439.21
3.45		764.91	1194.40	1583.43	0.0036		212.48	331.78	439.84
3.50		753.27	1185.69	1586.57	0.0036		209.24	329.36	440.71
3.55			1174.14	1589.02	0.0036			326.15	441.39
3.60			1165.61	1591.20	0.0036			323.78	442.00
3.65			1151.46	1595.05	0.0036			319.85	443.07
3.70			1138.32	1599.59	0.0036			316.20	444.33
3.75			1130.37	1601.69	0.0036			313.99	444.91
3.80			1122.75	1604.75	0.0036			311.88	445.76
3.85			1110.70	1607.20	0.0036			308.53	446.44
3.90				1608.77	0.0036				446.88
3.95				1609.56	0.0036				447.10
4.00				1611.22	0.0036				447.56
4.05				1611.13	0.0036				447.54
4.10				1609.56	0.0036				447.10
4.15				1609.30	0.0036				447.03
4.20				1608.16	0.0036				446.71
4.25				1608.16	0.0036				446.71
4.30				1607.46	0.0036				446.52
4.35				1606.15	0.0036				446.15
4.40				1603.27	0.0036				445.35

4.45				1599.77	0.0036				444.38
4.50				1596.45	0.0036				443.46
4.55				1592.25	0.0036				442.29
4.60				1588.41	0.0036				441.22
4.65				1585.44	0.0036				440.40
4.70				1580.80	0.0036				439.11
4.75				1573.03	0.0036				436.95
4.80				1565.95	0.0036				434.98
4.85				1553.36	0.0036				431.49
4.90				1541.65	0.0036				428.24
4.95				1533.78	0.0036				426.05
5.00				1527.49	0.0036				424.30
5.05				1516.65	0.0036				421.29
5.10				1507.30	0.0036				418.69
5.15				1492.62	0.0036				414.62
5.20				1485.01	0.0036				412.50
5.25				1476.97	0.0036				410.27
5.30				1466.75	0.0036				407.43
5.35				1455.56	0.0036				404.32
5.40				1404.26	0.0036				390.07

Test Type:	Direct Shear Test (ASTM D-3080)
Type of Sample:	Undisturbed Soil Sample
Test Pit Site Name	TP 1@3m

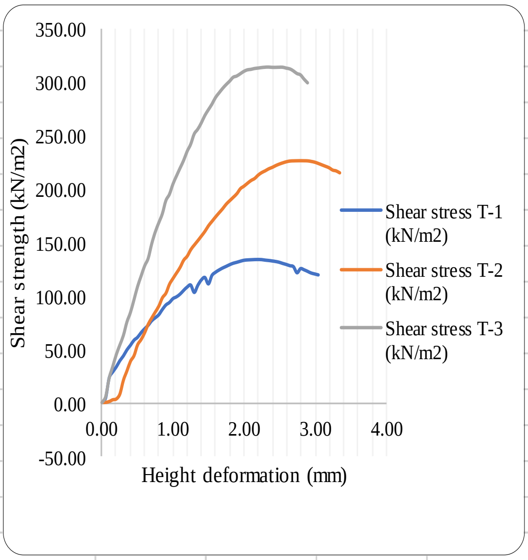
Trials	1	2	3	3	C, (kN/m2)	Angle of Friction, Ø
Normal Stress (kN/m2)	109.0	218.0	327.0	436.0	32.6	42.5
Max. Shear Stress (kN/m2)	135.1	228.4	332.0	433.4		

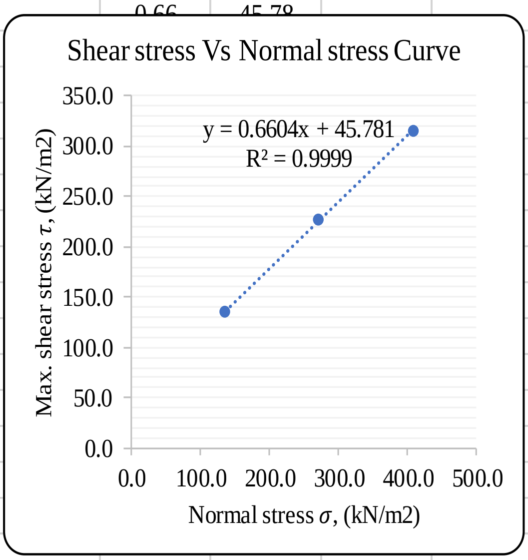




Test Type:	Direct Shear Test (ASTM D-3080)
Type of Sample:	Undisturbed Soil Sample
Test Pit Site Name	TP 2@1.5m

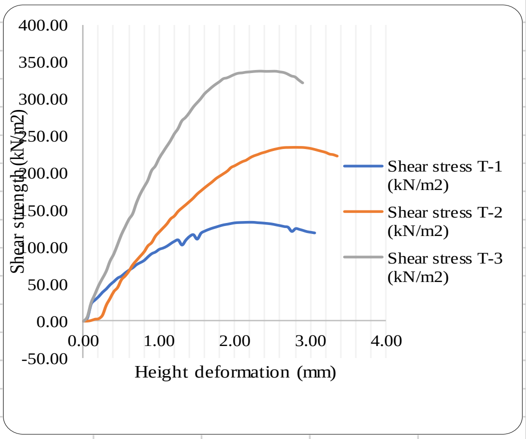
Trials	1	2	3	3	C, (kN/m2)	Angle of Friction, Ø
Normal Stress (kN/m2)	136.3	272.5	408.8	436.0	45.8	33.4
Max. Shear Stress (kN/m2)	135.3	226.7	315.3	0.0		

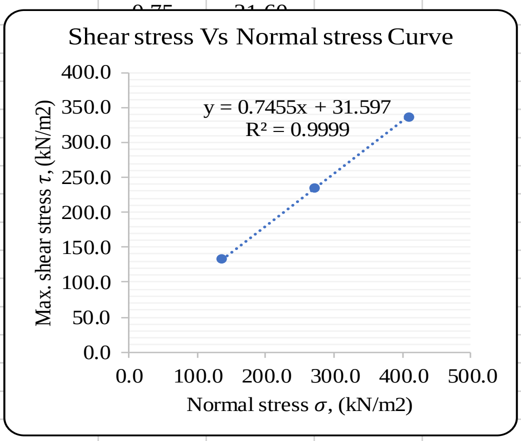




Test Type:	Direct Shear Test (ASTM D-3080)
Type of Sample:	Undisturbed Soil Sample
Test Pit Site Name	TP 2@3m

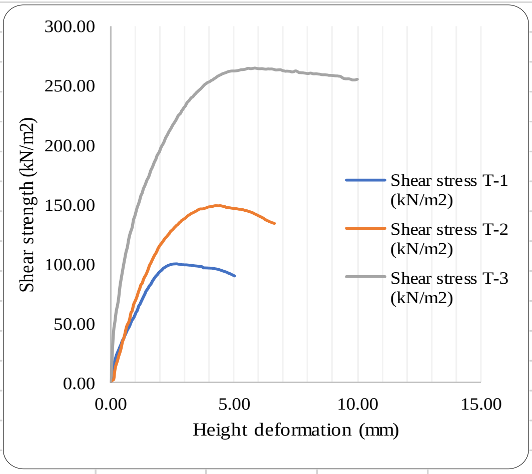
Trials	1	2	3	3	C, (kN/m2)	Angle of Friction, Ø
Normal Stress (kN/m2)	136.3	272.5	408.8	436.0	31.6	36.7
Max. Shear Stress (kN/m2)	133.6	233.8	336.8	0.0		

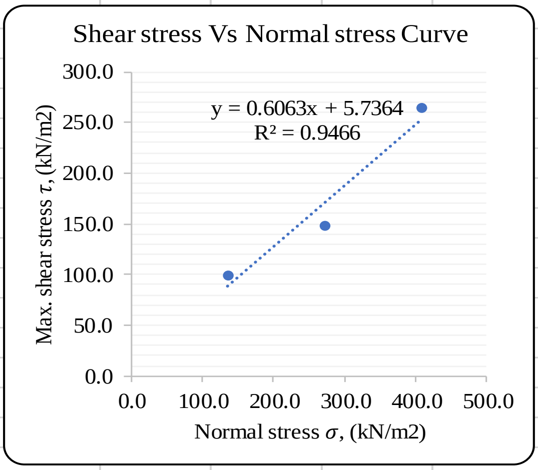




Test Type:	Direct Shear Test (ASTM D-3080)
Type of Sample:	Undisturbed Soil Sample
Test Pit Site Name	TP 3@1.5m

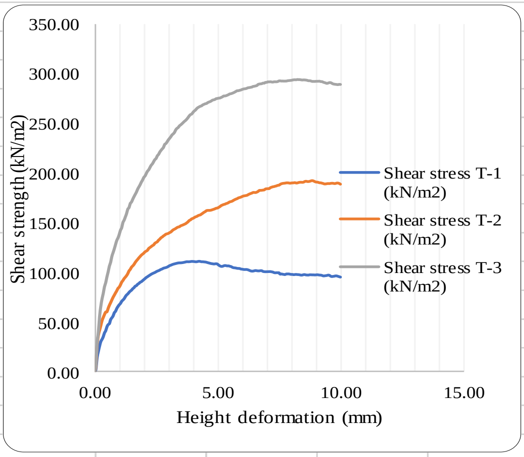
Trials	1	2	3	3	C, (kN/m2)	Angle of Friction, Ø
Normal Stress (kN/m2)	136.3	272.5	408.8	436.0	5.7	31.2
Max. Shear Stress (kN/m2)	99.7	148.3	264.9	0.0		

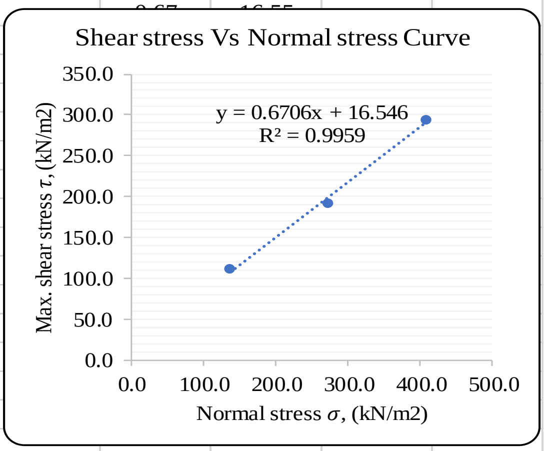




Test Type:	Direct Shear Test (ASTM D-3080)
Type of Sample:	Undisturbed Soil Sample
Test Pit Site Name	TP 3@3m

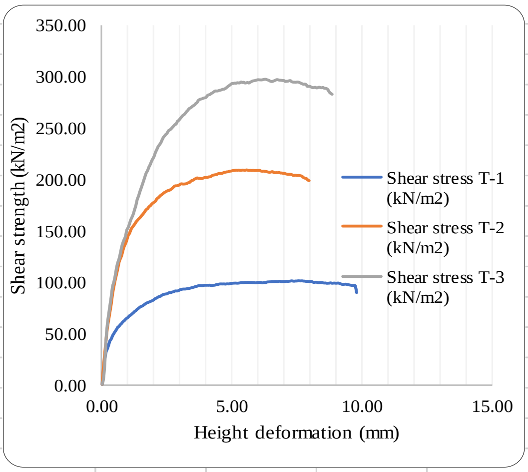
Trials	1	2	3	3	C, (kN/m2)	Angle of Friction, Ø
Normal Stress (kN/m2)	136.3	272.5	408.8	436.0	16.5	33.8
Max. Shear Stress (kN/m2)	111.3	192.5	294.1	0.0		

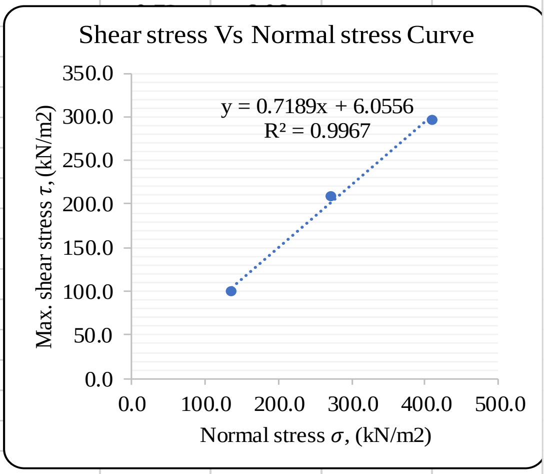




Test Type:	Direct Shear Test (ASTM D-3080)
Type of Sample:	Undisturbed Soil Sample
Test Pit Site Name	TP 4@1.5m

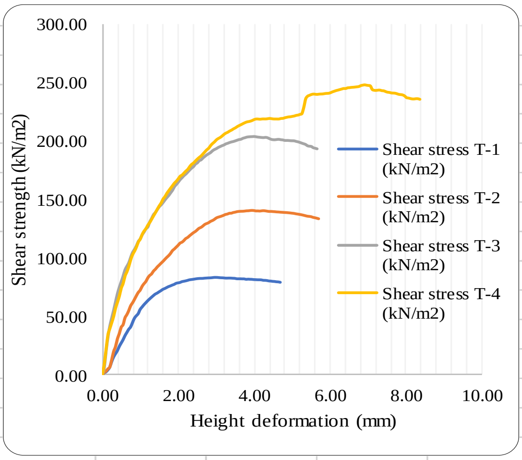
Trials	1	2	3	3	C, (kN/m2)	Angle of Friction, Ø
Normal Stress (kN/m2)	136.3	272.5	408.8	436.0	6.1	35.7
Max. Shear Stress (kN/m2)	100.7	208.5	296.6	0.0		

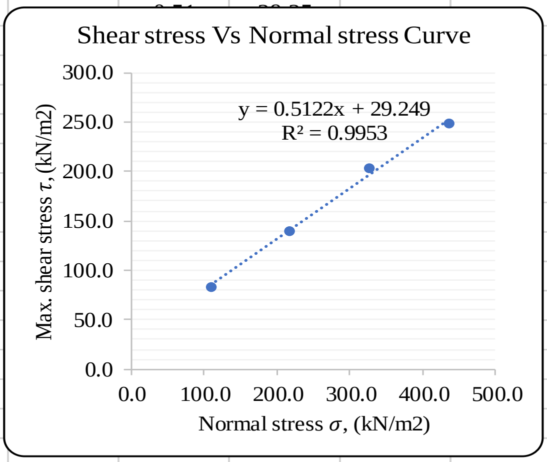




Test Type:	Direct Shear Test (ASTM D-3080)
Type of Sample:	Undisturbed Soil Sample
Test Pit Site Name	TP 4@3m

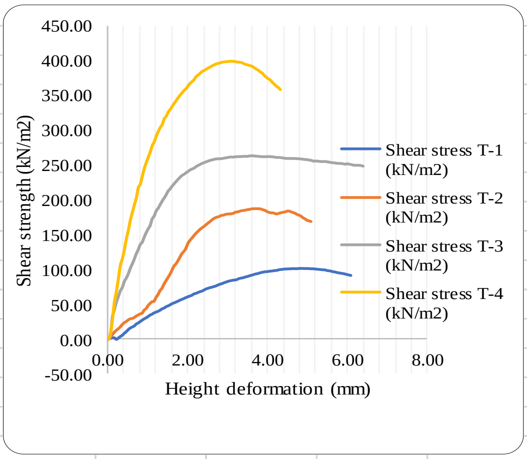
Trials	1	2	3	3	C, (kN/m2)	Angle of Friction, Ø
Normal Stress (kN/m2)	109.0	218.0	327.0	436.0	29.2	27.1
Max. Shear Stress (kN/m2)	83.2	140.2	203.8	248.1		

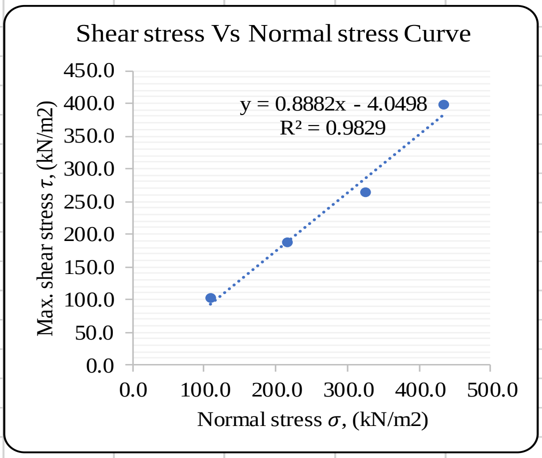




Test Type:	Direct Shear Test (ASTM D-3080)
Type of Sample:	Undisturbed Soil Sample
Test Pit Site Name	TP 5@1.5m

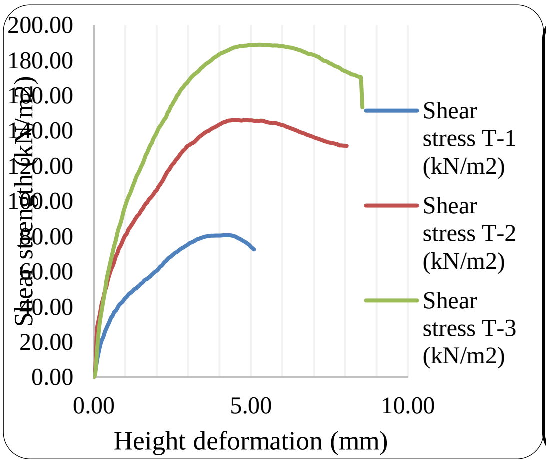
Trials	1	2	3	3	C, (kN/m2)	Angle of Friction, Ø
Normal Stress (kN/m2)	109.0	218.0	327.0	436.0	21.8	36.7
Max. Shear Stress (kN/m2)	101.4	187.7	264.1	398.7		

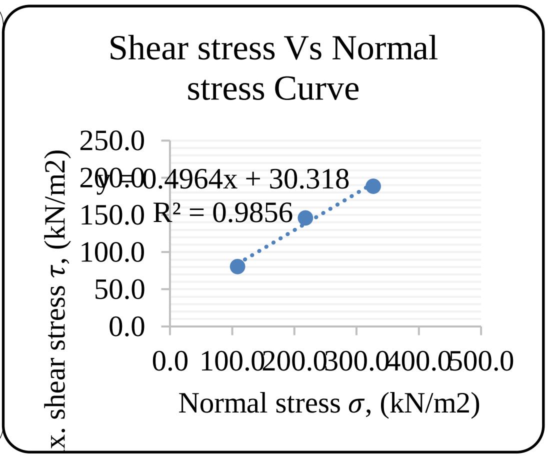




Test Type:	Direct Shear Test (ASTM D-3080)
Type of Sample:	Undisturbed Soil Sample
Test Pit Site Name	TP 5@3m

Trials	1	2	3	3	C, (kN/m2)	Angle of Friction, Ø
Normal Stress (kN/m2)	109.0	218.0	327.0	436.0	30.3	26.4
Max. Shear Stress (kN/m2)	80.6	146.1	188.9	0.0		





VIII. One Dimensional Consolidation Analysis

TP 3@3m											
Time	Deformation @ 50kPa	Deformation @ 100kPa	Deformation @ 200kPa	Deformation @ 400kPa	Deformation @ 800kPa	Deformation @1600kPa	Deformation @800kPa	Deformation @ 400kPa	Deformation @ 200kPa	Deformation @ 100kPa	Deformation @ 50kPa
0.00	0.128	0.236	0.488	0.834	1.416	2.138	2.606	2.492	2.420	2.274	2.222
0.10	0.182	0.338	0.604	1.084	1.746	2.424	2.588	2.426	2.280	2.230	2.194
0.25	0.184	0.348	0.638	1.102	1.770	2.432	2.582	2.426	2.279	2.230	2.193
0.50	0.186	0.360	0.650	1.122	1.794	2.454	2.568	2.426	2.279	2.230	2.192
1.00	0.188	0.366	0.662	1.144	1.818	2.486	2.554	2.425	2.279	2.230	2.190
2.00	0.190	0.376	0.674	1.178	1.846	2.508	2.536	2.424	2.279	2.230	2.190
4.00	0.192	0.384	0.686	1.198	1.872	2.522	2.528	2.424	2.279	2.230	2.190
8.00	0.194	0.392	0.700	1.226	1.888	2.548	2.520	2.424	2.279	2.229	2.189
15.00	0.198	0.402	0.714	1.248	1.918	2.558	2.516	2.424	2.278	2.228	2.189
30.00	0.202	0.414	0.730	1.280	1.956	2.574	2.513	2.424	2.278	2.227	2.188
60.00	0.206	0.422	0.746	1.306	1.988	2.582	2.508	2.423	2.277	2.226	2.186
120.00	0.214	0.446	0.764	1.334	2.026	2.588	2.504	2.422	2.276	2.225	2.184
240.00	0.222	0.458	0.799	1.362	2.062	2.592	2.498	2.421	2.275	2.224	2.181
480.00	0.228	0.472	0.812	1.388	2.104	2.598	2.496	2.420	2.274	2.223	2.181
1440.00	0.236	0.488	0.834	1.416	**2.138**	2.606	**2.492**	2.420	2.274	2.222	2.181

	0.002	0.010	**0.034**	0.018	0.024	0.008	0.006	0.000	**0.001**	**0.000**	0.001
	0.004	**0.018**	0.024	0.042	0.048	**0.054**	**0.028**	**0.001**	0.000	0.000	**0.003**
	0.004	0.016	0.024	**0.056**	0.052	0.054	0.032	0.002	0.000	0.000	0.002
	0.004	0.018	0.024	0.054	**0.054**	0.036	0.026	0.001	0.000	0.000	0.000
Do	**0.180**	**0.330**	**0.570**	**1.066**	**1.764**	**2.378**	**2.610**	**2.428**	**2.281**	**2.230**	**2.196**
D100	**0.226**	**0.458**	**0.802**	**1.382**	**2.108**	**2.58**	**2.499**	**2.42**	**2.274**	**2.223**	**2.181**
D50%	**0.203**	**0.394**	**0.686**	**1.224**	**1.936**	**2.479**	**2.555**	**2.424**	**2.2775**	**2.2265**	**2.1885**
t50%	**20**	**9**	**4.2**	**9**	**21**	**0.9**	**0.9**	**1.8**	**42**	**43**	**22**

Pressure (KPa)	Do	Deformation dial reading at 50% consolidation	Deformation at 100% Primary Consolidation	Time for 50% consolidation	Thickness of specimen at 50% consolidation	Half-thickness of specimen at 50% consolidation	Initial deformation reading	Change in Thickness of Specimen, ΔH	Cumulative of change height of specimen	Change in Void Ratio [Δe=ΔH/Hs]	Void Ratio [e=e0-Δe]	Coefficient of consolidation 10^{-2}, Cv (cm^2/sec)	Compression index (Cc)	Swelling index (Cs)	Coefficient of compressibility (av) 10^{-4} (m^2/kN)	Coefficient of permeability (k) (cm/s)	Over burden pressure (kPa)	Preconsolidation pressure (kPa)	Over consolidation ratio (OCR)
50	0.180	0.203	0.226	20	19.797	9.9	0.128	0.098	0.098	0.011	**1.186**	**0.016**	**0.226**		**3.35**	**0.24**	**41.38**	**180.00**	**4.35**
100	0.330	0.394	0.458	9	19.606	9.8	0.236	0.222	0.32	0.035	**1.161**	**0.035**			**4.88**	**0.78**			
200	0.570	0.686	0.802	4.2	19.314	9.7	0.488	0.314	0.634	0.070	**1.127**	**0.073**			**3.45**	**1.16**			
400	1.066	1.224	1.382	9	18.776	9.4	0.834	0.548	1.182	0.130	**1.067**	**0.032**			**3.01**	**0.46**			
800	1.764	1.936	2.108	21	18.064	9.0	1.416	0.692	1.874	0.206	**0.991**	**0.013**			**1.90**	**0.12**			
1600	2.378	2.479	2.58	0.9	17.521	8.8	2.138	0.442	2.316	0.254	**0.942**	**0.280**			**0.61**	**0.86**			
800	2.610	2.5545	2.499	0.9	17.446	8.7	2.606	0.107	2.209	0.243	**0.954**	**0.278**		**0.0395**	**0.15**	**0.20**			
400	2.428	2.424	2.42	1.8	17.576	8.8	2.492	0.072	2.137	0.235	**0.962**	**0.141**			**0.20**	**0.14**			
200	2.281	2.2775	2.274	42	17.723	8.9	2.420	0.146	1.991	0.219	**0.978**	**0.006**			**0.80**	**0.02**			
100	2.230	2.2265	2.223	43	17.774	8.9	2.274	0.051	1.940	0.213	**0.983**	**0.006**			**0.56**	**0.02**			
50	2.196	2.1885	2.181	22	17.812	8.9	2.222	0.041	1.899	0.209	**0.988**	**0.012**			**0.90**	**0.05**			

TP 4@3m											
Time	Deformation @ 50kPa	Deformation @ 100kPa	Deformation @ 200kPa	Deformation @ 400kPa	Deformation @ 800kPa	Deformation @1600kPa	Deformation @800kPa	Deformation @ 600kPa	Deformation @ 200kPa	Deformation @ 100kPa	Deformation @ 50kPa
0.00	0.020	0.338	0.562	0.850	1.250	1.932	2.494	2.418	2.294	2.234	2.106
0.10	0.280	0.462	0.700	1.052	1.562	2.302	2.480	2.299	2.240	2.112	2.080
0.25	0.286	0.472	0.708	1.066	1.588	2.322	2.478	2.298	2.240	2.112	2.080
0.50	0.292	0.480	0.714	1.076	1.606	2.336	2.476	2.298	2.240	2.112	2.080
1.00	0.298	0.484	0.722	1.086	1.624	2.348	2.468	2.298	2.240	2.112	2.080
2.00	0.304	0.492	0.728	1.098	1.642	2.354	2.456	2.298	2.240	2.112	2.080
4.00	0.308	0.498	0.736	1.110	1.662	2.366	2.446	2.298	2.240	2.112	2.080
8.00	0.312	0.504	0.742	1.122	1.684	2.374	2.438	2.298	2.240	2.112	2.080
15.00	0.316	0.508	0.752	1.134	1.702	2.386	2.434	2.298	2.240	2.112	2.080
30.00	0.318	0.516	0.760	1.148	1.726	2.392	2.428	2.298	2.239	2.112	2.080
60.00	0.318	0.522	0.768	1.162	1.754	2.398	2.426	2.297	2.238	2.111	2.079
120.00	0.322	0.528	0.778	1.178	1.776	2.406	2.422	2.296	2.236	2.110	2.078
240.00	0.328	0.546	0.784	1.196	1.824	2.426	2.420	2.295	2.234	2.109	2.077
480.00	0.336	0.558	0.806	1.230	1.854	2.446	2.419	2.294	2.234	2.108	2.076
1440.00	0.338	0.562	0.806	1.220	1.906	2.481	2.418	2.294	2.234	2.107	2.076
2880.00			0.846	1.250	1.922	2.488				2.106	
4320.00			0.850		1.932	2.494					

	0.006	0.010	0.008	0.014	0.026	0.020	0.002	0.001	0.000	0.000	0.000
	0.012	0.012	0.014	0.020	0.036	0.026	0.010	0.000	0.000	0.000	0.000
	0.012	0.012	0.014	0.022	0.036	0.018	0.020	0.000	0.000	0.000	0.000
	0.010	0.014	0.014	0.024	0.038	0.018	0.022	0.000	0.000	0.000	0.000
Do	0.274	0.470	0.694	1.054	1.552	2.296	2.490	2.300	2.240	2.112	2.080
D100	0.336	0.557	0.846	1.231	1.9	2.48	2.42	2.294	2.234	2.108	2.076
D50%	0.305	0.514	0.770	1.143	1.726	2.388	2.455	2.297	2.237	2.110	2.078
t50%	2.5	26	70	24	30	20	4.5	60	85	120	115

Pressure (KPa)	Do	Deformation dial reading at 50% consolidation	Deformation at 100% Primary Consolidation	Time for 50% consolidation	Thickness of specimen at 50% consolidation	Half-thickness of specimen at 50% consolidation	Initial deformation reading	Change in Thickness of Specimen, ΔH	Cumulative of change height of specimen	Change in Void Ratio [Δe=ΔH/Hs]	Void Ratio [e=e0-Δe]	Coefficient of consolidation 10-2, Cv (cm2/sec)	Compression index (CC)	Swelling index (CS)	Coefficient of compressibility (av) 10-4 (m2/kN)	Coefficient of permeability (k) 10 8(cm/s)	Over burden pressure (kPa)	Pre-consolidation pressure (kPa)	Over consolidation ratio (OCR)
50	0.274	0.305	0.336	2.5	19.695	9.8	0.020	0.316	0.316	0.035	1.190	0.127	0.42		2.37	1.35	34.33	270.0	7.87
100	0.470	0.5135	0.557	26	19.487	9.7	0.338	0.219	0.535	0.060	1.165	0.012			3.72	0.20			
200	0.694	0.77	0.846	70	19.230	9.6	0.562	0.284	0.819	0.091	1.134	0.004			3.43	0.07			
400	1.054	1.1425	1.231	24	18.858	9.4	0.850	0.381	1.2	0.133	1.091	0.012			2.76	0.16			
800	1.552	1.726	1.9	30	18.274	9.1	1.250	0.650	1.850	0.206	1.019	0.009			2.28	0.10			
1600	2.296	2.388	2.48	20	17.612	8.8	1.932	0.548	2.398	0.267	0.958	0.013			1.52	0.10			
800	2.490	2.455	2.42	4.5	17.545	8.8	2.494	0.074	2.324	0.259	0.966	0.056		0.036	2.95	0.83			
600	2.300	2.297	2.294	60	17.703	8.9	2.418	0.124	2.200	0.245	0.980	0.004			3.71	0.08			
200	2.240	2.237	2.234	85	17.763	8.9	2.294	0.060	2.140	0.238	0.987	0.003			11.04	0.17			
100	2.112	2.11	2.108	120	17.890	8.9	2.234	0.126	2.014	0.224	1.001	0.002			21.41	0.23			
50	2.080	2.078	2.076	115	17.922	9.0	2.106	0.030	1.984	0.221	1.004	0.002			45.53	0.51			

Time	Deformation @ 50kPa	Deformation @ 100kPa	Deformation @ 200kPa	Deformation @ 400kPa	Deformation @ 800kPa	Deformation @1600kPa	Deformation @800kPa	Deformation @ 800kPa	Deformation @ 200kPa	Deformation @ 100kPa	Deformation @ 50kPa
0.00	0.001	0.026	0.124	0.420	0.658	1.106	1.366	1.210	1.070	1.008	0.960
0.10	0.002	0.035	0.304	0.516	0.838	1.228	1.260	1.076	1.038	0.972	0.928
0.25	0.003	0.036	0.314	0.524	0.844	1.232	1.258	1.075	1.037	0.971	0.926
0.50	0.004	0.038	0.320	0.532	0.852	1.238	1.254	1.074	1.034	0.970	0.926
1.00	0.008	0.040	0.328	0.540	0.864	1.246	1.250	1.074	1.028	0.969	0.926
2.00	0.010	0.042	0.334	0.546	0.876	1.254	1.246	1.074	1.022	0.969	0.926
4.00	0.014	0.046	0.342	0.558	0.890	1.262	1.242	1.074	1.018	0.969	0.925
8.00	0.018	0.068	0.350	0.566	0.912	1.272	1.238	1.074	1.016	0.968	0.924
15.00	0.020	0.098	0.356	0.580	0.922	1.288	1.232	1.074	1.014	0.967	0.924
30.00	0.021	0.118	0.366	0.588	0.940	1.296	1.228	1.074	1.012	0.966	0.923
60.00	0.022	0.119	0.372	0.599	0.958	1.304	1.222	1.073	1.010	0.964	0.922
120.00	0.024	0.120	0.382	0.618	0.980	1.312	1.218	1.072	1.009	0.962	0.921
240.00	0.026	0.122	0.398	0.630	1.014	1.324	1.216	1.071	1.009	0.961	0.920
480.00	0.026	0.124	0.412	0.644	1.052	1.336	1.212	1.070	1.008	0.960	0.918
1440.0	0.026	0.124	0.420	0.658	1.082	1.358	1.210	1.070	1.008	0.960	0.918
2880.0					1.098	1.364					
4320.0					1.106	1.366					
	0.001	0.001	0.010	0.008	0.006	0.004	0.002	**0.001**	0.001	0.001	**0.002**
	0.005	0.004	**0.014**	**0.016**	0.020	0.014	**0.008**	0.001	0.009	**0.002**	0.000
	0.006	0.004	0.014	0.014	**0.024**	**0.016**	0.008	0.000	**0.012**	0.001	0.000
	0.006	**0.006**	0.014	0.018	0.026	0.016	0.008	0.000	0.010	0.000	0.001
Do	**0.001**	**0.034**	**0.306**	**0.508**	**0.828**	**1.222**	**1.258**	**1.077**	**1.046**	**0.973**	**0.928**
D100	***0.026***	***0.124***	***0.4123***	***0.648***	***1.068***	***1.362***	***1.213***	***1.07***	***1.008***	***0.96***	***0.918***
D50%	**0.014**	**0.079**	**0.359**	**0.578**	**0.948**	**1.292**	**1.236**	**1.074**	**1.027**	**0.967**	**0.923**
t50%	**4**	**10**	**19**	**16**	**40**	**22**	**10**	**0.8**	**11**	**15**	**20**

Pressure (KPa)	Do	Deformation dial reading at 50% consolidation	Deformation at 100% Primary Consolidation	Time for 50% consolidation	Thickness of specimen at 50% consolidation	Half-thickness of specimen at 50% consolidation	Initial deformation reading	Change in Thickness of Specimen, ΔH	Cumulative of change height of specimen	Change in Void Ratio [Δe=ΔH/Hs]	Void Ratio [e=e0-Δe]	Coefficient of consolidation 10^{-2}, Cv (cm^2/sec)	Compression index (CC)	Swelling index (CS)	Coefficient of compressibility (av) 10-4 (m2/kN)	Coefficient of permeability (k) (cm/s)	Over burden pressure (kPa)	Pre-consolidation pressure (kPa)	Over consolidation ratio (OCR)
50	0.001	0.013	0.026	4	19.98	10.0	0.001	0.025	0.025	0.003	1.276	0.082	0.12		1.00	0.36	44.07	120	2.72
100	0.034	0.079	0.124	10	19.92	10.0	0.026	0.098	0.123	0.014	1.265	0.033			1.66	0.23			
200	0.306	0.359	0.4123	19	19.64	9.8	0.124	0.288	0.4113	0.047	1.232	0.017			2.50	0.18			
400	0.508	0.578	0.648	16	19.42	9.7	0.420	0.228	0.6393	0.073	1.206	0.019			1.89	0.16			
800	0.828	0.948	1.068	40	19.05	9.5	0.658	0.410	1.049	0.120	1.159	0.007			1.53	0.05			
1600	1.222	1.292	1.362	22	18.70	9.4	1.106	0.256	1.305	0.149	1.130	0.013			0.94	0.06			
800	1.258	1.235	1.213	10	18.76	9.4	1.366	0.153	1.152	0.131	1.147	0.029		0.074	1.67	0.22			
600	1.077	1.073	1.07	0.8	18.92	9.5	1.210	0.140	1.012	0.115	1.163	0.368			1.97	3.28			
200	1.046	1.027	1.008	11	18.97	9.5	1.070	0.062	0.950	0.108	1.170	0.027			5.68	0.69			
100	0.973	0.966	0.96	15	19.03	9.5	1.008	0.048	0.902	0.103	1.176	0.020			11.21	1.00			
50	0.928	0.923	0.918	20	19.77	9.5	0.960	0.042	0.860	0.098	1.181	0.015			23.13	1.55			

IX. Procedural Photos