ENGINEERING GEOLOGICAL CHARACTERIZATION OF SOILS AND ROCKS FOR FOUNDATION SUITABILITY OF ENGINEERING WORKS IN AMBO TOWN, WEST SHOWA, ETHIOPIA.

LENCHO LEMMA GODA

A THESIS SUBMITTED TO THE DEPARTMENT OF APPLIED GEOLOGY
SCHOOL OF APPLIED NATURAL SCIENCE

PRESENTED IN PARTIAL FULFILLMENT OF THE REQUIREMENT FOR THE DEGREE OF MASTER OF SCIENCE IN ENGINEERING GEOLOGY

OFFICE OF GRADUATE STUDIES
ADAMA SCIENCE AND TECHNOLOGY UNIVERSITY

OCTOBER, 2023
ADAMA, ETHIOPIA

Engineering Geological Characterization of Soils and Rocks For Foundation Suitability of Engineering Works in Ambo Town, West Showa, Ethiopoia.

Lencho Lemma Goda

Major Advisor: Dr. Yadeta Chemdesa (Associate Professor of Engineering Geology).
Co- Advisor: Mr. Tola Garo (MSc, Engineering Geologist)

A Thesis Submitted to the Department of Applied Geology
School of Applied Natural Science

Presented in Partial Fulfillment of the Requirement for the Degree of Master's
In Engineering Geology

Office of Graduate Studies
Adama Science and Technology University

October, 2023
Adama, Ethiopia

Declaration

I hereby affirm that the master's thesis *“Engineering Geological Characterization of Soils and Rocks* for Foundation Suitability of Engineering Works *In Ambo Town, West Showa, Ethiopia”* is my particular work. In other words, it hasn't been submitted to any other university for the award of an academic degree, diploma, or certificate. Each and every source of information used in this thesis has been properly recognized through citation.

______________________	______________________	______________
Name of student	Signature	Date

Recommendation

We, the advisors of this thesis, hereby certify that we have read the revised version of the thesis entitled "*Engineering Geological Characterization of Soils and Rocks* for Foundation Suitability of Engineering Works *In Ambo Town, West Showa, Ethiopia*" prepared under our guidance by Lencho Lemma Goda. Therefore, we recommend the submission of the revised version of the thesis to the department following the applicable procedures.

____________________	____________________	________________
Major advisor	Signature	Date
____________________	____________________	________________
Co-advisor	Signature	Date

Approval Page

We, the advisors of the thesis entitled "*Engineering Geological Characterization of Soils and Rocks* for Foundation Suitability of Engineering Works *In Ambo Town, West Showa, Ethiopia*" and developed by Lencho Lemma Goda; hereby certify that the recommendation and suggestions made by the board of examiners are appropriately incorporated into the final version of the thesis.

______________________	______________________	______________
Major advisor	Signature	Date
______________________	______________________	______________
Co-advisor	Signature	Date

We, the undersigned, members of the Board of Examiners of the thesis by Lencho Lemma Goda have read and evaluated the thesis entitled "*Engineering Geological Characterization of Soils and Rocks* for Foundation Suitability of Engineering Works *In Ambo Town, West Showa*, *Ethiopia*" and examined the candidate during an open defense. This is, therefore, to certify that the thesis is accepted for partial fulfillment of the requirement of the degree of Master of Science in Applied Geology.

______________________	______________________	______________
Chairperson	Signature	Date
______________________	______________________	______________
Internal Examiner	Signature	Date
______________________	______________________	______________
External Examiner	Signature	Date

Final approval and acceptance of the thesis is contingent upon the submission of its final copy to the Office of Postgraduate Studies (OPGS) through the Department Graduate Council (DGC) and School Graduate Committee (SGC).

______________________	______________________	______________
Department Head	Signature	Date
______________________	______________________	______________
School Dean	Signature	Date
______________________	______________________	______________
Office of Postgraduate Studies	Signature	Date

ACKNOWLEDGEMENT

First and for all, I offer my deeply felt thanks to God for being with me from the beginning to the end and for countless blessings. Dr. Yadeta Chemdesa, my advisor and co-advisor Mr. Tola Garo, deserves special credits for all round cooperation during the entire period of this project. I highly appreciate the commitment they had to enhance my knowledge's on the subject matter by closely following my activities and strictly checking my progress. My deepest gratitude also goes to my parents and friends, specially my brothers Eng. Tadesse Lemma, Eng. Bilisa Lemma and Sister Fenet Negesa with her family Dr. Tadesse Seda for their steady encouragement and support during my study. I would also like to express my gratitude to the communities of Ambo town and Mr. Gezahagn Dagu, Mr. Alema Diribsa and others to supporting me by different information about the study area. I highly appreciate Addis Ababa Science and Technology University, Department of Civil Engineering Geotechnical laboratory and for Adama Science and Technology University for giving field equipment's.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF ABBREVIATIONS AND ACRONYMYS

AASHTO	American Association of State Highway Officials
AASTU	Addis Ababa Science and Technology University
ASTM	American Society for Testing Materials Standard
CH-CL	Inorganic Clay of High Plastic and Silt Clays of Low Plasticity
DEM	Digital Elevation Model
EBCS	Ethiopian Building Code Standards
EGM	Engineering Geological Map
GI	Group Index
Gs	Specific Gravity
GPS	Global Positioning System
GSI	Geological Strength Index
IAEG	International Association of Engineering Geologist
ISRM	International Society of Rock Mechanics
J_V	Volumetric Joint Count
LI and LL	Liquidity Index and Liquid Limit
MDD	Maximum Dry Density
MH	High Plasticity Silt
ML	Low Plastic Silt
Mpa	Mega Pascal
NMC	Natural Moisture Content
OMC	Optimum Moisture Contents
PI and PL	Plastic Index and Plastic Index
RQD	Rock Quality Designation
RMR	Rock Mass Rating
SW	Well Graded Sand
USCS	Unified Soil Classification System
UCS	Uniaxial Compressive Strength
Wm	Mold Weight

ABSTRACT

For engineering constructions and earth works to be successfully and economically designed, site characterization is an essential. It is also required to gather enough information on the kind, qualities, and distributions of rock and soil beneath a site of proposed structures. The geotechnical and engineering geological qualities of the soils and rocks of Ambo town are not well understood, and the town does not have an engineering geology map which is the best approach to represent the surrounding environment for engineering purposes. The present research work is therefore, aimed to assessing and evaluating the geological and engineering geological condition of the town and to produce engineering geological map. To accomplish these goals, thorough field survey, an in-situ strength test, sampling, and sample laboratory testing were carried out. Eight disturbed soil samples were taken from seven test pits that were manually excavated at maximum depth of three meters, and five representative rock samples were taken by breaking them with geological hammer in order to achieve these objectives. Depending on field observation, in-situ rock strength and laboratory analysis, the engineering geological properties of rocks and soils were determined. Geological and engineering geological maps were created at a scale of 1:50,000 to show the location, coverage area, and engineering qualities of the various geological materials. Based on laboratory test results the natural moisture content, liquid limit, plastic limit, plasticity index, specific gravity, and free swell vary from 8.24 to 16.1%, 24.06 to 86.81%, 10.34 to 47.6%, 13.65 to 40.24%, 2.49 to 2.65, and 0 to 80%, respectively. The tested soils' undrained shear strengths range from 64.97 kPa to 97.2 kPa. The Unified Soil Classification System categorizes soils into sandy clays and silty clays of low plasticity (SC), inorganic silt clays of medium plasticity (ML), inorganic silts, fine sands and clayey silts of high plasticity (MH), gravelly sands (SW), inorganic silt clays of high plasticity (MH), and inorganic clays of high plasticity (CH). The soils are categorized as clayey soils (A-7-5) and silt / gravel sand (A-2-7 and A-2-6), which are unfavorable and favorable for subgrade material, respectively, according to the AASHTO classification system. According to those soil classification systems the problematic soil areas mostly identying in the eastern, north eastern and northern parts of the town. The rocks of the study area were classified into very high strength, high strength, low strength and very low strength rock units based on the strength (Unconfined Compressive Strength) of intact rock. The strength of the basaltic unit varies from very high strength rock to high strength rock (242.88 MPa) to (155.48 MPa). The strength of sandstone varies from very low (25.5Mpa) to low strength (37.8Mpa). Very low rock strength, measured with an ignimbrite unit, is (24.31MPa). The main geotechnical problems that affect the design and development of civil structures in the town were identified as the existence of different rock falls and soil sliding which were formed due to quarry site and river cutting of slope in the townrespectively and presence cohesive soils of high plasticity character areas.

Key words: *Ambo town, geotechnical, Engineering geological, geological mapping, slope stability*

CHAPTER ONE

1. INTRODUCTION

1.1 Background of the Study

Majority of engineering structure failures are caused by a lack of understanding the properties of geological materials, their dynamic behavior and structural design (Ariyo et al., 2018; Kwiecien, 2019; Lin and Scot, 2006; Srihandayani, 2021). The success or failure of the foundation of structures depends on the consistency of soil and rock characterization parameters that result from soil and rock investigation used for the design of the foundation. As many engineers throughout the world have been focused exclusively on complicated structural design without taking other elements into account; structural failures including buildings, roads, dams, bridges, and other infrastructure have resulted in the loss of life and damages to the economy of the country (Srihandayani, 2021). Without considering the most important influencing elements, these failures are frequently blamed in developing nations on the character or type of the soils and rocks in the area. Similarly, the potentially expansive soils are prevalent in Ethiopia and cause a greater financial loss to property owners, which must be recognized as a serious problem (Uge, 2016).

The physical and mechanical properties of earth materials have major influences on the success, economy, and safety of engineering works. Therefore, studies of the soils and rocks as construction materials that are used to build with or on and as a material of the environment that can affect engineering structures are extremely important for a town (Shano et al., 2021; Uge, 2016). Insufficient ground exploration may lead to inappropriate design and failure of structure. Therefore, to obtain information on the type, characteristics and distribution of rock and soils and geodynamic conditions, engineering and geotechnical investigation should be done/on the rocks and soil underling any construction sites. The investigation shall include characterization of earth materials (rock and soil) and geomorphological process. However, in Ethiopia adequate investigations and understanding of geological, engineering geological, geotechnical and hydrogeological factors concerning infrastructure failures are not a prime focus by local authorities, planners, designers, and investors, and thus the roads, buildings, bridges, and other infrastructure built on problematic soils do not provide the expected performance (Amanuel et al., 2022). In other word, deep knowledge of the geomorphological, geological and geotechnical condition of an urban areas is necessary to provide basic information of the ground condition to local authorities, engineers and contractors (Bowles, 1996). This research is therefore, aimed at assessing and evaluating the engineering geological and geotechnical conditions of Ambo town and to provide important engineering geological data that may help to plan, design and maintain engineering projects. The town is serving as the major town in the west showa zone of the Oromia region, which is approximately 114 kilometers from Ethiopia's capital city (Addis Ababa). Many structures in the town are being built or are already under construction without a sufficient and complete geotechnical evaluation.

1.2 Statement of the Problem

Ambo town, which is located in central Ethiopia, is one of the biggest and fastest growing towns in the west showa zone of the Oromia regional state. The town is currently expanding northeast to Dabis Lake, southeast to the Ambo stadium goal project area, and southwest to Ambo Waliso road directions. The town is also developing into a commercial center for which infrastructures like roads, buildings, and bridges are being built. On the other hand, comprehensive knowledge of both surface and subsurface conditions is necessary for the proper design of civil engineering structures such building foundations, etc. Because of this, engineering geological characterization of soils and rocks or earth material investigations are crucial parts of designing any type of infrastructure, such as highways, bridges, and residential constructions. It offers essential information to local authorities, civil engineers, and urban/town planners that must be taken into account during the initial design and planning of future infrastructure projects and construction projects in the town. The soil and rocks in the town have not been well described in terms of their mechanical and physical properties, and no work has been done in the area of engineering geology to generate an engineering geological map of the town.

1.3 Objective of the Research

1.3.1 General Objective

The main objective of this study is to carry out engineering geological appraisal of rocks and soils in Ambo town for engineering purpose.

1.3.2 Specific Objectives

- ✓ To determine geological, and geotechnical properties of soils, and rocks of the town.
- ✓ To classify rock and soil of the study area according to the standard engineering classification systems

- ✓ To produce medium scale (1:50,000) multi-purpose engineering geological maps of the study area.
- ✓ To identifying geodynamic properties of the areas though the town.

1.4 Significance of the Study

Unplanned town growth increases the danger of natural disasters affecting people and economic resources. Understanding the engineering behavior of soils, rocks, and geodynamic conditions in the town will be improved by this study. In order to properly manage the town's land use, the data, particularly the engineering geological map of the town, will serve as a reference for urban planners and town designers. The engineering qualities of the soils and rock mass in the research area will also indicated by the engineering characterization of those materials. If a business or researcher wants to do further research in the future, this helps to reduce the cost of the investigations. It will also be easier to prevent and limit any negative impacts caused by any potential future damage to the structures and road construction in the study region if one has a solid understanding of the engineering or geotechnical qualities of rocks and soils. The engineering geological characterization of the rocks and soils in Ambo town will be found in this research paper, which stands out as one of the resources. This study will therefore serve as background information for related engineering projects in the future. Any building or infrastructure project’s design and construction phases will also use the study's findings. It is fundamental in determining a strong foundation that can resist any predicted load placed on Civil Engineering constructions.

1.6 Scope of the Research

The scope of this research is limited to the characterization of Ambo town soils and rock in terms of their index and engineering property (shear strength) with some of slope stability analysis at selected slope sections. To accomplish these parameters seven test pits with maximum depth of 3m were excavated and eight proper representative soil samples and five rock samples were collected.

1.7 Organization of the Thesis

The problem statement, general context of the topic, objective, and importance of the study are all explored in chapter one. A brief survey of the literature is covered in chapter two. The study area is described in the third chapter of the text. The study's methodology is covered in the fourth chapter of the thesis. Discussion of the findings of laboratory and field tests is covered in Chapter 5, some of slope stability analysis in selected slope section evaluation is presented in chapter six and the conclusion and suggestion are presented in chapter seven.

CHAPTER TWO

2. LITERATURE REVIEW

2.1 Introduction

A naturally occurring collection of mineral grains, with or without organic components, is called soil. The weathering and disintegration of parent rock is what creates soil (Terzaghi et al., 1996), and it can weathered through mechanical disintegration or chemical decomposition. The weathering of rock reduces the cohesive force holding the mineral grains together and causes larger bulk to disintegrate into smaller and smaller pieces. Exfoliation, erosion, freezing, and melting processes are responsible for the mechanical weathering of rocks into smaller particles. Rock, on the other hand, is regarded to be a naturally occurring collection of mineral grains bound together by a powerful and long-lasting cohesive force. The soils have variable compositions depending on the conditions they were transported under, which is frequently different from the parent rock. It is employed as a building material for engineering applications in a variety of civil engineering projects, and it supports structural foundations (Das, 2010).

2.2 Engineering Geological Map (EGM)

A branch of applied geology called engineering geology fills the knowledge gap between a geologist and a civil engineer. The research that determines where to build structures for civil engineering is crucial. At the very least, individuals could not realize where it is safe to build and where it is unsafe due to inadequate ground conditions, the presence of hazards in the environment, or other factors (Efrem et al., 2013). By creating an engineering geologic map of the area, it is possible to identify this risky and safe area. An engineering geological map is a particular kind of geological map that offers a broad representation of all geological features important for land-use planning, design, building, and maintenance in the field of civil and mining engineering (Basilicata, 2006). The engineering geologic conditions of a certain site, including their rocks and soils composition, water quality, geomorphological state, and geodynamic processes, are the main determinants of EGM. The term "soil" is typically used in the context of engineering geological mapping to refer to any material that can be dug out without the aid of explosives or machinery for breaking or loosening (Belay, 2018).

2.2.1 Components of Engineering Geological Map (EGM)

The characteristics of the rocks and soils, including their description, stratigraphic and structural arrangement, genesis, lithology, physical state, or their physical and engineering properties, as well as hydro-geological conditions, geo-morphological conditions, and geodynamic phenomena, are the main elements of EGM (Konstantopoulou & Vacondios, 2007). Engineering geological maps have to include interpretive cross-sections, a legend, and an explanation text. They might also consist of documentation data that was gathered to create the map. On engineering geological maps of all kinds and sizes, the information should be presented so that it is possible to fully understand and appreciate both the true nature and the engineering importance of the data (Attewell, 1985).

2.2.2 Multipurpose Engineering Geological Map (EGM)

EGMs can be divided into specific and multipurpose EG maps depending on their intended purpose. Special-purpose maps: These maps offer details for a single engineering-related topic or a single objective. An illustration would be an engineering geological map indicating the slope stability condition, bearing capacity, and suitability of the dam site. For a variety of planning and engineering needs, Multipurpose Maps give information covering various aspects of engineering geology (Crow & Barber, 2005). The selection of geological features of rocks and soils that are closely related to physical properties, such as strength, deformability, durability, and permeability, which are significant in engineering geology, is the main challenge in multipurpose engineering geological soil and rock mapping. It is frequently challenging to quantify, rapidly, easily and over sufficiently wide areas all of these features. This is why geological qualities and index (geotechnical) properties are frequently used as indicators of physical or engineering geological characteristics. Indicating the physical state of soil and rock and indicating strength properties, deformation characteristics, permeability, and durability are genetic characteristics, mineralogical composition closely related to specific gravity, Atterberg limits, and plasticity index; textural and structural characteristics, such as particle size distribution, unit weight, and porosity; moisture content, consistency, degree of weathering and alteration and jointing (Raghuvanshi, 1995). The following sections each discuss one of the properties that this study investigated to.

2.3 Engineering Geological and Geotechnical Investigations

In order to design the earthworks and foundations for proposed structures and to repair damage to earthworks and structures caused by subsurface conditions, engineering geologists and geotechnical engineers conduct geotechnical investigations to gather information on the physical properties of the soil and rock beneath (and occasionally adjacent to) a site. The study area is investigated on the surface and below during geotechnical examination. Subsurface sampling and laboratory testing of the collected soil samples will be frequent components of site research. To investigate the deep soil conditions, test pits and trenching may also be employed. Surface investigation is as basic as an engineer walking about to evaluate the physical characteristics at the site or it can involve geological mapping and geophysical techniques (Parker, 1996).

2.4 Engineering Geologic Characterization and Classification of Rocks

Rock is a naturally occurring collection of minerals that cannot be easily broken with the human hand and does not decompose after going through a drying and wetting cycle. Rock type (intact rock), rock structures (discontinuities), any change to the rock, in-situ stress state, and hydro-geological regime are some of the geological elements that affect the rock mass with regard to engineering operations (Mekdes et al., 2022). Based on mode of origin rocks are classified into three major groups: Igneous rock, sedimentary rock and metamorphic rocks. A more general division of rocks into two types is necessary for engineering usage of the rock. These are: Rock mass and intact rock (rock materials) (Bell, 2007).

2.4.1 Intact Rocks (Rock Materials)

Are solid rock block with tensile strength and no mechanical discontinuities, grain size, color, compressive strength, and rock type are used to define it (Zhai et al., 2020). Index properties are characteristics of intact rocks that without any secondary discontinuities and weathering conditions include: Porosity, Density, Permeability, Sonic velocity, Durability, Strength, and Modulus of elasticity (Bell, 2007).

2.4.2 Rock Mass

According to Hashemi et al. (2017), the rock mass is made up of various blocks with complete rock properties that are spaced apart by discontinuities. The features of the entire rock, the kind and number of discontinuities, and the intensity and extent of chemical weathering all have a role in a rock mass's engineering attributes. The approach is examined according to Terzaghi (1946), Deere (1964), Bieniawski (1973), Barton et al. (1974), and ISRM (1981). Different methods of classifying rock masses are often used. The categorization system proposed by the International Society for Rock Mechanics (ISRM, 1981) serves as the foundation for the current investigation, which predominantly relies on these techniques. The following two criteria serve as the foundation for this categorization method: rock general term and discontinuity spacing. In the absence of drill core data, the joint volumetric count technique will be used to classify the rock quality. Numerous methods, such as the volumetric count method, weathering state of discontinuities from visual inspections, and calculations based on (Bieniawski, 1989), are used to categorize the quality of rocks (RQD). Schmidt hammer rebound measurements were taken to ascertain the rock's strength characteristics. UCS will use the bouncing method to get the dry density of the rock using the rebound value. RMR can be calculated using a variety of factors, including the rock's uniaxial compressive strength, Rock Quality Designation (RQD), the distance between discontinuities, their conditions, groundwater conditions, and the direction of the discontinuities. Each of the six factors is given a value based on the characteristics of the rock. Tests in the field and in the lab led to this outcome. The sum of the six parameters is the RMR value (Bieniawski, 1973).

2.4.3 Rock Mass Ratting (RMR)

The rock mass rating (RMR) was introduced by Bieniawski (1973). The intention was to create a system that could aid the communication between the engineer and the geologist. RMR is the summation of the ratings of six (6) parameters, namely: strength of the intact rock material, rock quality designation, (RQD), spacing of discontinuities, condition of discontinuities, groundwater conditions and orientation of discontinuities. RMR has undergone a number of revisions throughout time in an effort to make the categorization more applicable to certain uses (Heriyadi et al., 2019). The SMR and Laubscher's rock mass rating method for mining (Laubscher, 1977; Laubscher, and Jakubec, 2001) Slope Mass Rating introduced by Romana (1985) to specifically relate the RMR classification to slopes.

2.4.4 Rock Quality Designation (RQD)

With the use of drill core logs, Deere developed the Rock Quality Designation index (RQD) (Deere et al., 1967) to provide a numerical assessment of the quality of the rock mass. RQD is the percentage of intact core pieces greater than 100 mm (4 inches) in the overall length of the core. The core should be at least size (54.7 mm or 2.15 inches in diameter) and drilled with a double-tube core barrel. Palmström (1982) proposed that the RQD may be approximated from the number of discontinuities per unit volume when no core is available but discontinuity traces may be found in surface exposures or descriptive identifying. The suggested relationship for clay-free rock masses is:

$$RQD = 115 - 3.3\,J_V \qquad \text{(Eq. 2.1)}$$

$$RQD = \sum \left(\frac{Core\ pieces > 10cm}{Total\ length\ of\ core\ run}\ x\ 100\% \right) \qquad \text{(Eq. 2.2)}$$

Where, JV, or the volumetric joint count, is the total number of joints for all joint (discontinuity) sets per unit length. RQD is a directional dependent metric, and depending on the borehole orientation, its value may alter dramatically. Reducing this directional dependence can be done effectively by using the volumetric joint count. RQD is meant to represent the in-situ rock mass quality (Zhang, 2016). It is crucial to identify and disregard handling- or drilling-related fractures when determining the value of RQD from diamond drill core. Blast-induced fractures shouldn't be taken into account when determining JV when using the Palmström relationship for exposure mapping.

$$\mathrm{Jv} = \frac{1}{s1} + \frac{1}{s2} + \frac{1}{s3} + \cdots + 1/sn \qquad \text{(Eq. 2.3)}$$

Where s1, s2, and s3 are the mean discontinuity set spacing. Palmström (2002) suggests that r=5m be assumed. As a result, the volumetric discontinuity frequency J_V can be generalized as (Eq.2.4).

$$\mathrm{JV} = \frac{1}{S1} + \frac{1}{S2} + \frac{1}{S3} + \cdots + \ Nr/s \qquad \text{(Eq. 2.4)}$$

Where, Nr is the number of random discontinuities.

2.4.5 Geological Strength Index (GSI)

The concept of the Geological Strength Index (GSI) was advanced by Hoek et al. in 1995. It is one of the methods that engineering rock mechanics has created to categorize rocks according to their masses in order to offer the closed-form answers or numerical analyses required to build tunnels, slopes, or foundations in rocks (Pozo, 2022). Additionally, these systems offer the necessary, reliable input data related to the characteristics of rocks based on their masses. To predict the strength and deformability of the rock mass, the geological features of the rock material and a visual assessment of the mass are used as direct input parameters. This technique is regarded as a mechanical continuity since it preserves the mechanical qualities of the rock bulk. According to the GSI categorization system, a rock mass must contain a sufficient number of randomly oriented discontinuities to function as a homogeneous, isotropic mass (Hoek, 2000). Because of this, it is not recommended to utilize the GSI system to examine rock masses with clearly defined dominant structural orientations or gravitational instability based on structural considerations (Hamasur, 2023).

2.4.6 Unconfined Compressive Strength (UCS)

When evaluating UCS (Unconfined Compressive Strength), rock samples with irregular geometry are pressed in one direction while rocks with regular geometry, such as cylindrical, prismatic, and cubic shapes, are not. According to Refky A. N. and Murad M. S. (2017), pressure has become a crucial element in the stability pillar when measuring stability in underground mines. The compression machine is used for this test, which adheres to International Society of Rock Mechanics (ISRM, 1981) standards. The value of the UCS and indirect rocks in the field is known to be predicted by point load testing. This is because a straightforward approach was followed; sample preparation is simple and may be done on site (Made Astawa, 2010).

2.4.7 Strength of Rocks

Determining the strength of rock masses is one of the key challenges faced by designers of engineering constructions on or in a rock. The strength of a rock mass is influenced by the discontinuities that separate the intact rock blocks in addition to the rock components (intact rock). Because of the discontinuities, a rock mass often has substantially lower strength than the matching unbroken rock. The most common way to gauge a rock's strength is to analyze the compressive strength while unconfined (Kahraman, 2014). According to some circumstances, rocks tensile strength is also used to gauge how durable they are (Bhawani Singh, 2011).

2.5 Geologic Characteristics/Classification of Soil

This classification/characterization is based on the origin or genesis of soil. It does not consider any engineering properties of soil such as permeability, shear strength, consolidation, and degree of compaction. It considers parent materials and the process of formation of soils. Broadly it is classified as Residual and alluvial. Residual soils are those that remain at the place of their formation as a result of the chemical weathering of parent rocks (F.E. & A.S., 2021). The engineering properties of residual soils do not vary considerably from the top layer to the bottom layer but there may be changes when the depth gets deeper. Residual soils have a gradual transition from relatively fine material near the surface to large fragments at greater depth (Arora, 2003). According to Wesley (2010), this type of soil formed directly from the physical and chemical weathering of the parent material, normally from rocks of the same sort. Alluvial soils also called fluvial soils are soils that were transported by rivers and streams. The composition of these soils depends on the environment under which they were transported and is often different from the parent rock. The profile of alluvial soils usually consists of layers of different soils. Much of construction activity has been and is occurring in and on alluvial soils. Glacial soils are soils that were transported and deposited by glaciers (Budhu, 2010).

2.6 Engineering Geological Characterization, And Classification of Soil.

The civil engineering structures like buildings, bridges, highways, tunnels, dams, towers, etc. are founded below or on the surface of the earth. For their stability, suitable foundation soil is required. To check the suitability of the soil to be used as a foundation or as construction materials, its geotechnical properties are required to be characterized (Surendra, 2017). Soils are characterized by their index and engineering (design) properties (Arora, 2003). Index Properties of soil are the properties of soil that are not of primary interest to the geotechnical engineer but which are indicative of the engineering properties (Arora, 2003). They are used for classification soil for engineering practice because easily determine in a laboratory test. The index and engineering properties of soil are: Engineering qualities and the index of a soil are used to describe it (Arora, 2003). Index properties are the physical characteristics of soils that are primarily used for identification and categorization. Simple laboratory experiments can be used to identify index qualities. These tests include free swell, specific gravity, particle size analysis, natural moisture content, and Atterberg limits (Barbosa et al., 2023)

2.6.1 Natural Moisture Content

The water content test is used to figure out how much water (moisture) is in soils. It is the proportion of the mass of dry soil solids to the mass of pore or free water in a given quantity of soil, represented as a percentage. The amount of water in a soil determines its consistency to a significant extent. Additionally, it is utilized to convey the phase relationships between air, water, and solids in a certain amount of soil (Krishna, 2002).

2.6.2 Grain Size Distribution/Particle Size Analysis

A technique for dividing soils into several parts depending on particle size is particle size analysis (Arora, 2003). To determine the particle size distribution of soil, two approaches are often utilized. Sieve analysis is used for soils with coarse-grained particles larger than 0.075 mm in diameter, and hydrometer analysis is used for particles less than 0.075 mm in diameter or fine-grained soils (Furukawa et al., 2000).

2.6.3 Atterberg Limits

To determine the plastic and liquid limits of fine-grained soil, the Atterberg limit test is used. This test's outcome can be used to forecast other engineering characteristics. The liquid limit and plastic limit have been linked to a wide range of soil engineering qualities, and they are also used to categorize fine-grained soil using the Unified Soil Classification System, or AASHTO System (Krishna, 2002).

2.6.4 Liquid Limit (LL)

The liquid limit is determined approximately as the percentage of water at which a portion of soil in a cup with standard dimensions and a groove cut into it will flow together at the base of the groove over a distance of 13 mm (Arora, 2003).

2.6.5 Plastic Limit (PL)

It is the percentage of soil's water content at which soil cannot be deformed by rolling into fibers of 3.2 mm diameter without breaking. Because soil is no longer flexible, any form change will result in the soil showing visible fractures (Arora, 2003).

2.6.6 Plasticity Index (PI)

Reflects the range of moisture within which the soil is plastic and is the difference between the liquid limit and plastic limit. When categorizing fine-grained soils, the plasticity index is significant. Clay has higher plasticity indices than silts, which have none or very little plasticity (Arora, 2003).

2.6.7 Specific Gravity

The term "soil specific gravity" (Gs) refers to the soil’s solid matter's specific gravity. It is customary to always use the specific gravity of solids when referring to the portion of soil that passes through a N0. 10 sieve. Most inorganic soils have a solid component with a specific gravity of between 2.6 and 2.9. Quartz-based sand has a specific gravity that ranges from 2.64 to 2.66. Clay that is inorganic often falls between 2.7 and 2.9. A soil’s specific gravity is less than 2.6 when it contains a lot of organic matter or porous particles, such diatomaceous earth (Das, 2012).

2.6.8 Free Swell

To determine whether or not a soil is expansive, free swell is utilized to calculate the soil’s potential for swelling. It is recognized that the clay minerals, soil mineralogy and structure, texture, as well as a number of other physical and chemical aspects of the soil, all influence how much swelling occurs (Izdebska & Wójcik, 2016). Comparing montmorillonite to illite and kaolinites, clay minerals, montmorillonite has the greatest impact on the amount of swelling and activity of soils (Holtz and Kovacs, 1981).

2.6.9 Soil Compaction

According to ASTM D698 compaction proctor standard method was used to determine maximum dry density and optimum moisture content of the study area. About 2 kg of soil was taken that was passing through the No. 4 sieve. The soil mass and the mold were weighed without the collar (Wm). The soil in the mixer is placed and gradually water added to reach the desired moisture content (w) then apply lubricant to the collar. The soil was removed from the mixer and places it in the mold in 3 layers. For each layer, 25 blows were initiated in the compaction process per layer. The drops were applied manually. Carefully the collar and trim removed the soil that extends above the mold with a sharpened straight edge. Then, The mold and the containing soil (W) Weighed and Extrude the soil from the mold using a metallic extruder, the water content from the top, middle, and bottom of the sample was Measured. The soil is placed again in the mixer and adds water to achieve higher water content, w (Cetin et al., 2007).

2.7 Engineering Properties of a Soil

Permeability, comprehensibility, and shear strength are the three basic engineering characteristics of soils. The shear strength of the study area's soil will be used to describe it in the investigation (Ito & Azam, 2013).

2.7.1 Shear Strength

The capacity of soil to prevent sliding along internal surfaces within a mass is known as shear strength (Murthy, 1990). The shear resistance of soil is the result of friction and the interlocking of particles and possibly cementation or bonding at the particle contacts. Thus, the shearing strength is affected by the consistency of the materials, mineralogy, and grain size distribution, shape of the particles, initial void ratio, and features such as layers, joints, fissures, and cementation (Poulos, 1989). Confining pressures play the significant role in changing the behavior of soils in deep foundations. Similarly in high rise earth dams, the confining pressures are of very high magnitude (Prakash, *et al.*, 2002)

2.8 Classification of Soils

From an engineering point of view, the classification may be made based on the suitability of a soil for use as a foundation material or as a construction material (Venkatra, 2013). Engineering properties of soils are an expensive and time-consuming process to determine in the laboratory. Scholars forward optional ways to determine the approximate engineering suitability of soil for engineering purposes. This optional way is a classification of soils. It is categorizing soil units into a similar group with similar index properties with a specified range of variation. Within this specified ranges variation soils have specified engineering quality for civil engineering work (Veloo, 2015). A soil classification should provide some guide to engineering performance of the soil type and should provide a means by which soil can be identified quickly (ISRM, 1981, as cited in Gebremedhin, 2010). There are several techniques for classifying soils. Two complex categorization schemes will be employed in the study. They are the Unified Soil Classification system (USCS) and the American Association of State Highway and Transportation Officials (AASHTO) classification system. Both techniques take into account the Atterberg limits and particle size distribution. Instead, in order to determine whether the fine fraction is silt or clay, the soil's liquid limit and plasticity index will also be employed (Arora, 2004).

2.8.1 Unified Soil Classification System (USCS)

On the basis of the Unified Soil Classification System (USCS), soil with > 50 % of sample mass retained on the 0.074 mm sieve is term coarse-grained and if > 50 % of the coarse fraction is retained on 4.76 mm sieve, the soil is classified as gravel but if ≥ 50 % of the coarse fraction passes 4.76 mm sieve, such soil is sandy soil ((Daryati et al., 2019)). Also the gravelly of sandy soil is classified further as well graded gravel/sand (GW or SW) or poorly graded gravel/sand (GP or SP) if percentage of the soil fines is < 5 %, but if the percentage of the soil fines is > 12 %, the soil plasticity index is plotted against its liquid limit and the soil is classified as gravel/sandy clayey (GC or SC) or gravel/sandy silty soil (GM or SM) depending on it position. On the other hand, if ≥ 50 % of the sample mass passes the 0.074 mm sieve, the soil is classified as fine-grained soil (Saliu, 2018). The plasticity index of the soil fine-grained is then plotted against its liquid limit on plasticity chart to further distinguish the soil as silt or clay of low, medium, or high plasticity (Arora, 2004).

2.8.2 AASHTO Soil Classification System

AASHTO soil Classification System classified soils in accordance with their performance as subgrade (Maharaj, 1993). To classify the soil, laboratory tests including sieve analysis, hydrometer analysis, and Atterberg limits which are used to determine the group of the soil.

CHAPTER THREE

3. DESCRIPTION OF THE STUDY AREA

3.1 Location and Accessibility of the Study Area

Ambo Town is situated in the West Showa Zone in central Ethiopia, in the western Oromia regional state. It is 114 kilometers to the west of Addis Ababa. This town has latitude and longitude of 8°57′N - 9°0′N, and 37°48′E - 37°58′E respectively, and its minimum and maximum elevation of 1950 to 2350 meters according to contour map of the study area. The west showa zone's capital in the Oromia regional state is Ambo town which has an area of 81km^2 calculated from latitude and longitude of the location map.

One of the towns in Ethiopia facing high population increase is Ambo, which requires for more infrastructures to be built. For example the population of this town is more increasing since 2014 as many people moving to this town from west of the country and around the town in connection with political instability.

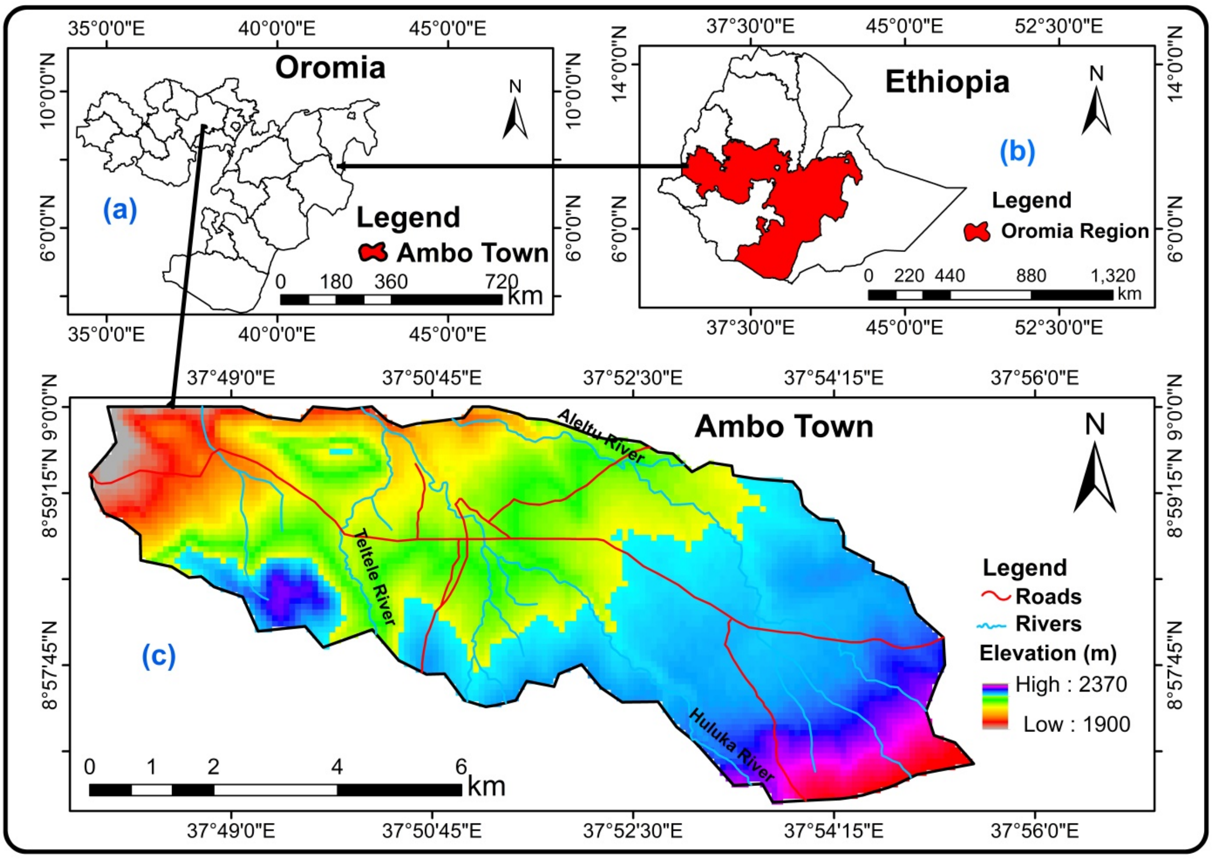


Figure 3.1 Location Map of the Study Area: Oromia Zone Boundary (a), Ethiopia Regional Boundary (b) and Study Area Which Is Ambo Town (c).

The most well-known brand of mineral water in Ethiopia is said to be produced in Ambo and bottled outside of the town. The mountain Wanchi to the south with its crater lake and the Guder and Huluka Falls are among the nearby attractions. The Ethiopian Institution of Agricultural Research Center founded in 1977, has a research station in Ambo that conducts studies on the protection of Ethiopia's main crops.

3.2 Physiography of the Study Area

The research area has a diverse topography, and this topography affects regional differences in temperature and soil composition, which in turn contributes to the natural vegetation's moderate variance. The terrain of the area is comprised of long ridges, wide plains, and volcanic hills. In the northernmost part of the research area, there is potential for an east-west oriented regional normal fault. The foot wall of this fault divides the water between the basins of the Awash and Blue Nile rivers. The tallest peak in the area is found in the Dandi and Wanchi volcanic centers, which are located in the southeastern and southern part of the research area.

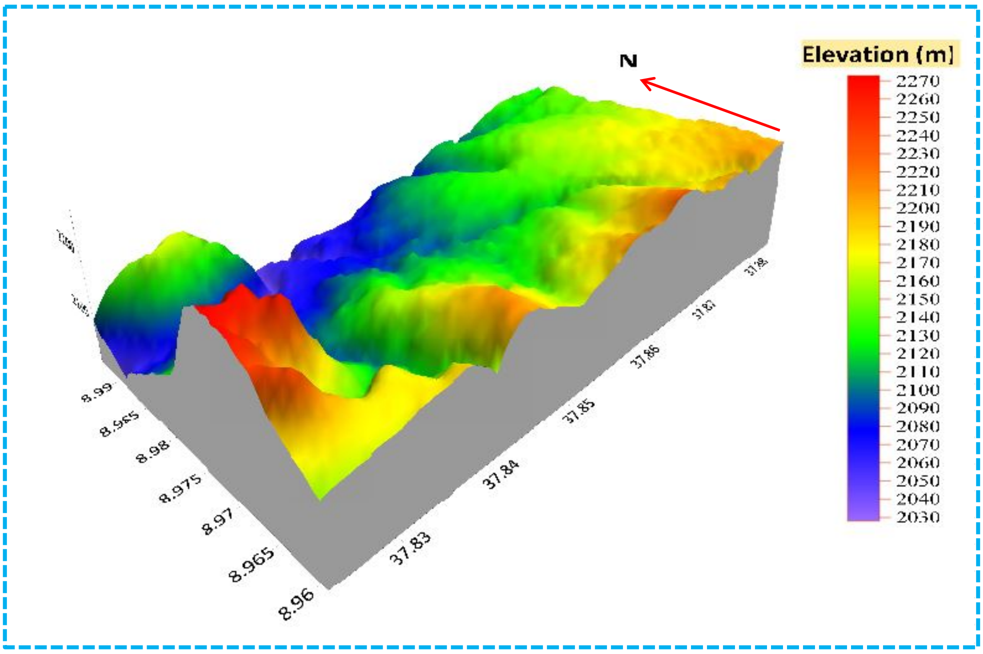


Figure 3.2 Physiographic map of the study area.

3.3 Climate

The Ambo area is characterized by unimodal rainfall that is concentrated between April and August with an annual value of 1143 mm and with a main annual temperature of 17.6^{o}C (Lemessa, 2001). According to rainfall data was taken from Ethiopian metrology agency from 2001-2022 the maximum and minimum annual rainfall is recording 2010 and 2002 with value of 1186 mm and 769 mm respectively.

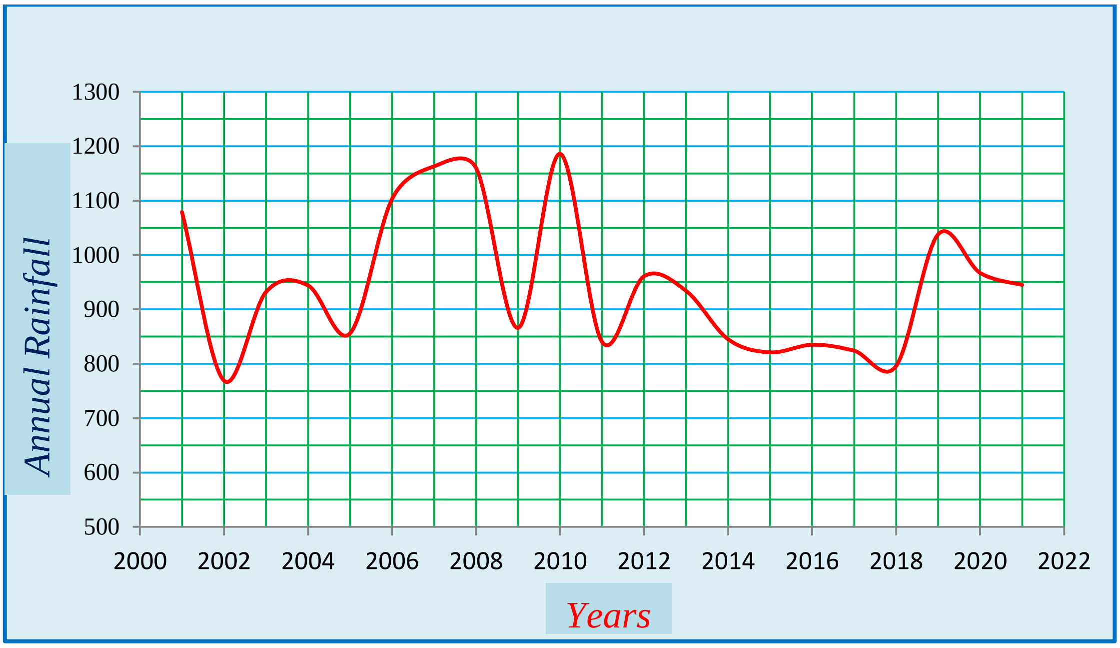


Figure 3.3 Annual rainfall of Ambo Town from 2001-2021 (Ethiopia Meteorology Agency, 2023)

The following figure (fig 3.4) shows that the mean monthly rain fall of Ambo town which is recording from the year of 2001 to 2021 and the values higher during six months from April to September when we comparing the all months of the years. According to Ethiopian meteorology agency (2023) the maximum and minimum mean monthly rain fall of Ambo town is recording July and February respectively.

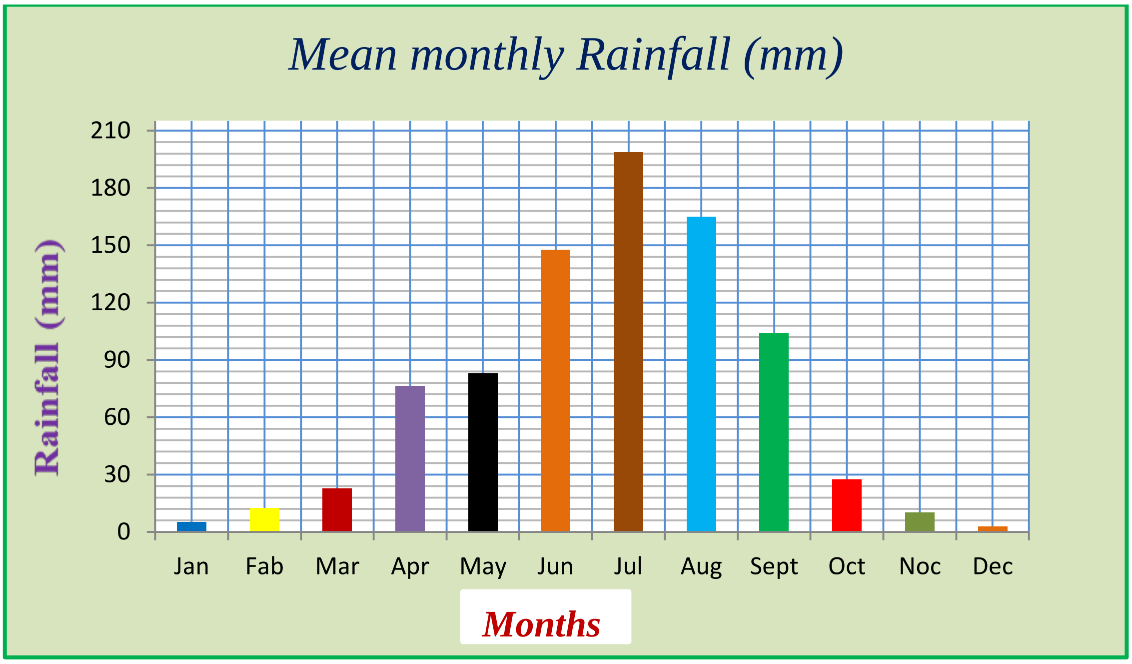


Figure 3.4 Mean monthly rainfall of Ambo town from 2001-2021 (Ethiopian Methodology Agency, 2023)

3.4 Vegetation

The study area is characterized by farmland, settlement land; bare land, forest land and grazing land which are made up of shrubs or small trees and are usually fairly open grasses and small plants grow between the shrubs (Muluken, 2021). Vegetation is patchy and composed of small plants that grow between the shrub species dominated by some scattered tree species. The vegetation description presented by Berhanu (2017) shows that the study area is characterized by dry evergreen montane forest on the basis that the dry evergreen montane forest covered the area between 1500 and 3000 mean sea level in the central and northern part of the Ambo town. The Ethiopian highlands contribute more than 50 % of the land area of Ethiopia with Afromontane vegetation, of which dry montane forests form the largest part (Bekele, 1993).

3.5 Drainage Characteristics

The Huluka, Aleltu, Teltale, and Dabis Rivers all get permanent and periodic inflows. One of the elements impacting the pattern and density of drainage in the researched area is rainfall. Other influences include slope, vegetation, rock kinds, and terrain. The predominant type of dendrite drainage in the area resembles a tree branch. Different contributing systems arise where the river channel follows the slope of the study terrain and are subsequently brought together to form the tributaries of the main rivers (the rivers mentioned above).

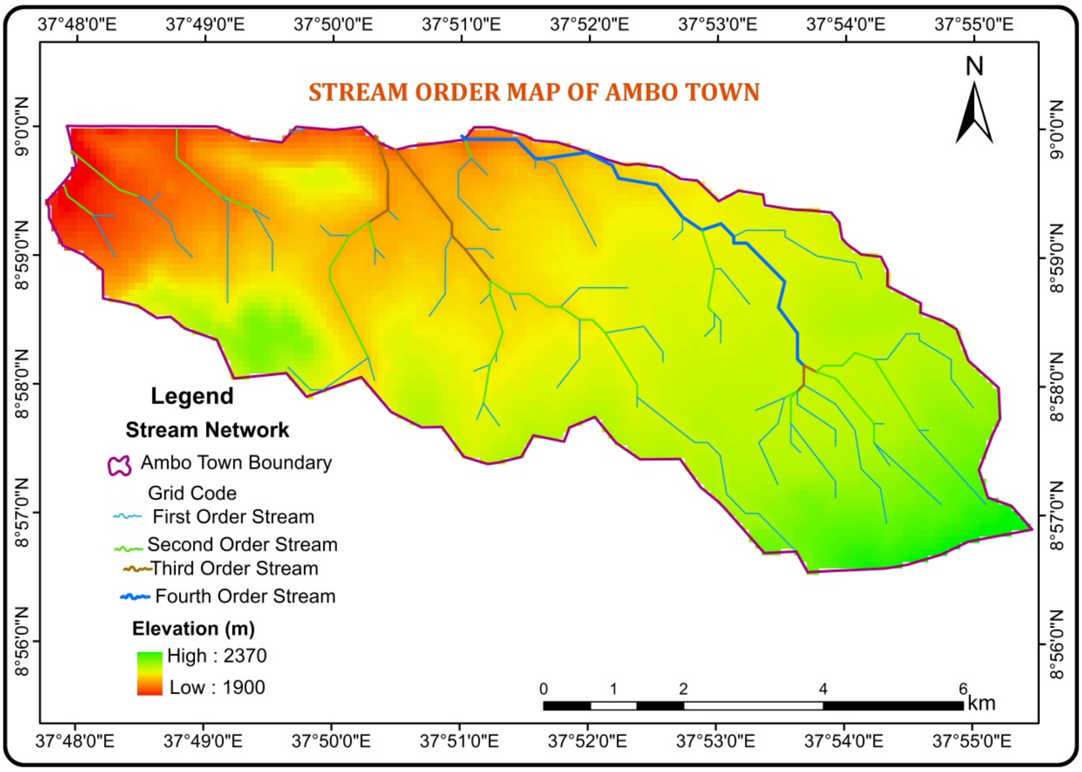


Figure 3.5 Drainage map of the study area (Ambo town).

3.6 Land Use and Land Cover of the Study Area

In the study area, there are four major land-use and land-cover classes; those are built-up area, forest land, bare land, and agricultural land (Muluken Filmon, 2021). The techniques and data used to identify the land use type were substituted with a top map, ground verification, expert opinion, supported by Google earth image, and key informants (Angessa *et al.*, 2019). In the study area, four types of land use, but the one constraint of Ambo town is unplanned, and the undersigned have two rivers that cross the town, those rivers are Teltale and Huluka Rivers.

In addition, the river area is less than bare land in the Ambo town, because of this identification by using many methods assigned four dominant obtained. The reason why these rivers were not included in the dominant land use type is that the depths of these rivers were high and hidden by the forest. Due to this, the reflectance shows more forest patterns. Furthermore, the river area cannot be covered by a satellite swath due to its less reflectance during data recording (Muluken F., 2021).

3.7 Geological Setting of the Study Area

3.7.1 Regional Geology

According to Mohr (1963), the Cenozoic volcanic rocks of Ethiopia divided into the Trap series and the Aden series. The term trap series refer to the whole pile of the Tertiary flood basalt sequence with intercalation of felsic lava and pyroclastic rock (commonly on the upper part). Aden series is confined to mafic lava flows within the younger Afar depression. At places, the thick volcanic pile of the plateau attains a thickness of up to 3 km. These names are modified into groups and formations during the later compilation (Mengesha et al., 1996). The Mesozoic sedimentary rocks sequence in the Abay basin according to Kasmin (1975) and later modified by (Russo et al., 1994) is lower sandstone/ Adigrat sandstone, Gypsum, and shale containing unit limestone, mudstone, and upper sandstone/ Ambar dam formation. In the Ambo area, the prominent sedimentary rocks are mudstone, Gypsum, shale, and upper sandstone. The sandstone sequence overlies the mudstone-Gypsum-shale formation. The latter is less resistant to erosion and forms a gentle slope while the sandstone forms a steep slope. The area is part of the central Ethiopian plateau comprising Mesozoic sedimentary rocks, particularly sandstone, and tertiary and quaternary volcanic rocks mainly basaltic and trachytic lava flows and pyroclastic deposits. The sedimentary sequence of the area is covered with tertiary and quaternary volcanic. The northern part of the study area is covered by basaltic lava flows of 31-20.6 Ma (Zanettin, 1993) that generated high topography.

The basaltic lavas are transitional alkaline basalt nature (Zanettin and Piccirillo, 1978; Zanettin, 1993). The older basaltic lava flows are separated from the younger quaternary volcanic of the Ambo area by the Addis-Ambo fault escarpment. The southern side of the major fault escarpment including ambo town is underlain by quaternary volcanic. The quaternary volcano in the area has resulted in central-type eruptions mainly forming Wanchi and Dandi volcanoes in the southern part of the study area (Kondo, 1967; Mohr, 1971; Herrick et al., 1980; Tsegaye et al., 1998).

The E-W running Ambo-Gedo fault has given rise to the exposure of the Mesozoic sediments and the juxtaposition of the sediments with the volcanic cover. A cliff and ridge forming sandstone are exposed at the NW part of Ambo town. The sandstone is well stored and intercalated with very thin layers of white-gray to reddish conglomerate. Exposures in some stream valleys show that the sandstone is directly overlain by the quaternary basaltic lava flows and is also intercalated with mudstone-gypsum-shale.

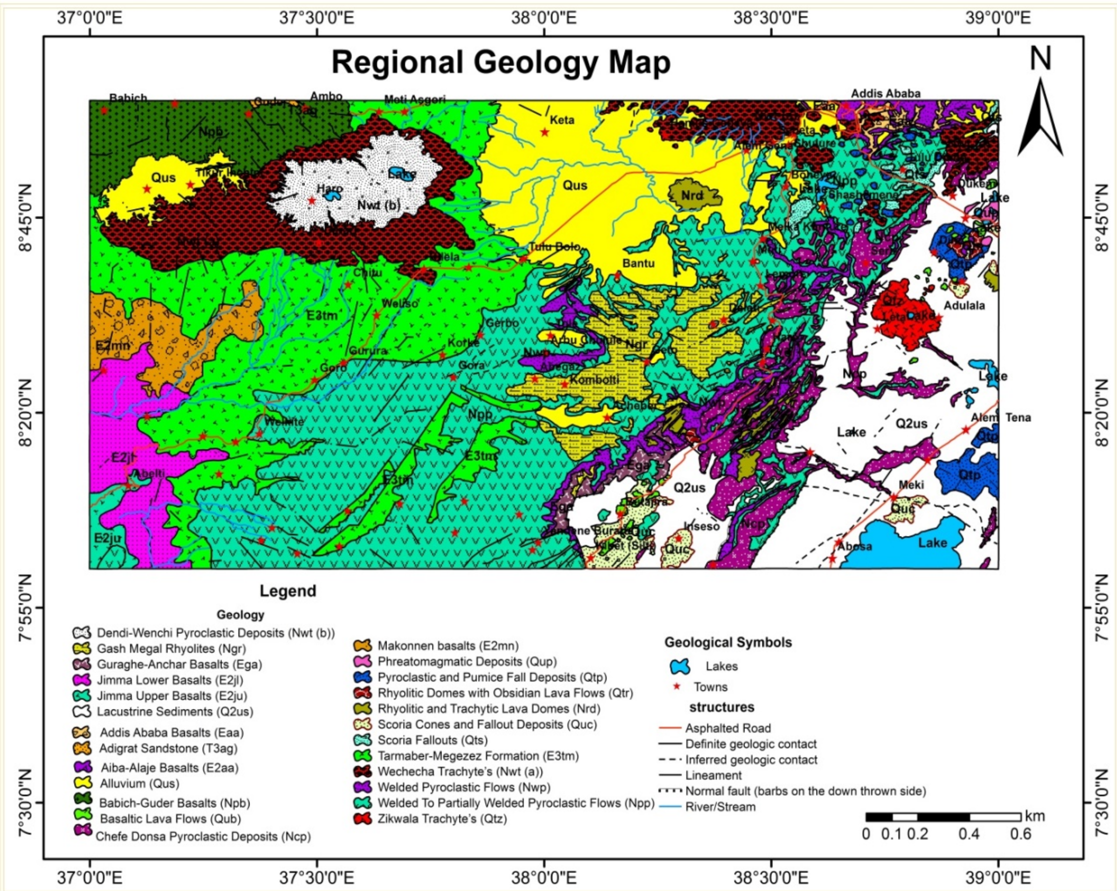


Figure 3.6 Regional geological map of the study area (Ethiopian geological survey, 2010)

3.7.2 Regional Geological Structure

The structures described here encompass mainly those within the Precambrian gneiss, Mesozoic sediments and Tertiary volcanic rocks upon which the present in the regional geology of the study area. A succinct description of each structure is rendered below:

i) Foliation: the map area is featured by preferred alignment of flaky (biotite, muscovite) minerals and amphibole (hornblende) crystals and stretching of quartz and feldspars parallel to the general strike of the rock body. This is well seen in gneiss having a northwest southeast direction dipping 32^o northeast (Di Paola and Berhe, 1978).

ii) Faults and fractures: the main striking lineaments of the area are shown by numerous systems of faults. They are characterized by four distinct sets of linear features trending in a NE-SW, NW-SE, N-S and rarely E-W directions. In addition to these, several subordinate jointing and fracturing are not uncommon cutting the different rock-types.

3.7.3 Local Geology

The geology of Ambo town is characterized by various volcanic units and sedimentary rock units, the upper part of which is dominated by basaltic lava flows, primarily aphanitic basalt, exposed southeast of the town and Phanaritic basalts exposed northwest of the study area, followed by a pyroclastic sequence primarily formed by ignimbrite (welded tuff), followed by weathered basaltic rock, and underlined with welded ignimbrites unit. The study region also contains sandstone, specifically the ambo (Senkele) sandstone block.

3.7.3.1 Sandstone Unit (Senkele Sandstone)

This unit, which is the oldest in the study area, widely crops out along the Ambo to Guder road side at the west to north eastern part of town. This unit’s maximum exposed thickness is mostly located in the western areas of the town and has been used for residential building and household items from ancient times. From ancient times, sandstone was a common building material. It is relatively soft, which makes carving it simple. It has been extensively utilized to create temples, houses, and other structures all over the world. Yellow to red in color and predominately fine-grained, the Ambo sandstone is. The porous nature of the sandstone makes it simple to split and mold. As the stratification depth increases, the surface color of reddish decreases to white; the grains are very fine in texture and dusty when the weather is dry.

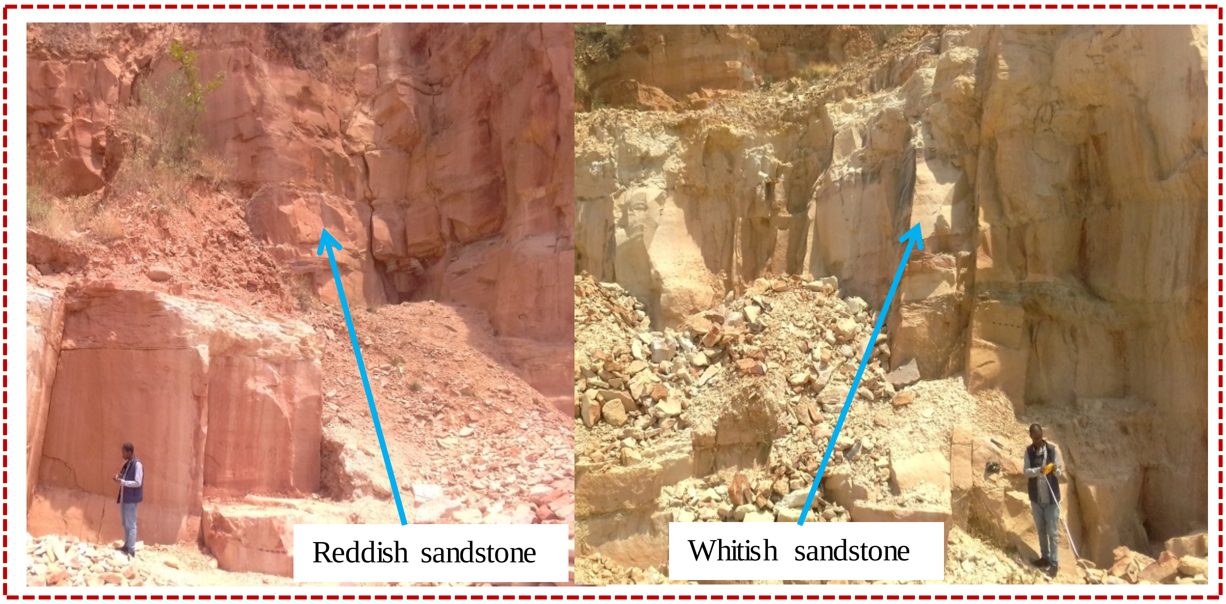


Figure 3.7 Reddish and whitish senkele sandstone from left to right respectively.

3.7.3.2 Welded Ignimbrites

Ambo town ignimbrite unit is exposed close to center of the town along the Huluka River and around Ambo hot spring. It overlies the Basaltic rock unit and locally covers the product of the Crater Lake volcanoes of wanchi and Dandi. The sequence is constituted by different flow units, consisting of pale- green to white welded and crystal-rich ignimbrites. This unit is mainly exposed all parts of Ambo Ethiopia Hotel and its' surrounding area.

3.7.3.3 Basaltic Rock (Columnar Basalt)

This basaltic unit is predominantly light gray in exposure and dark color within sample stage, and fine-grained, aphanitic which is exposed southeastern and southwestern of the town and Phanaritic to porphyritic in texture is exposed northwestern of the study area, moderate to strongly weathered and thinly to thickly jointed and slightly too strongly fractured. Most outcrops are excellent candidates for natural gravel, few for crushing aggregate and masonry stones depending on their state of weathering, fracturing and jointing patterns and this structural joint are mainly columnar joints in both types of the rock which exposed on the study area. There is also other exposure which is highly weathered basaltic unit located at aphanitic unit area and exposure separated the aphanitic basalt unit into two parts and it exposed between those units.

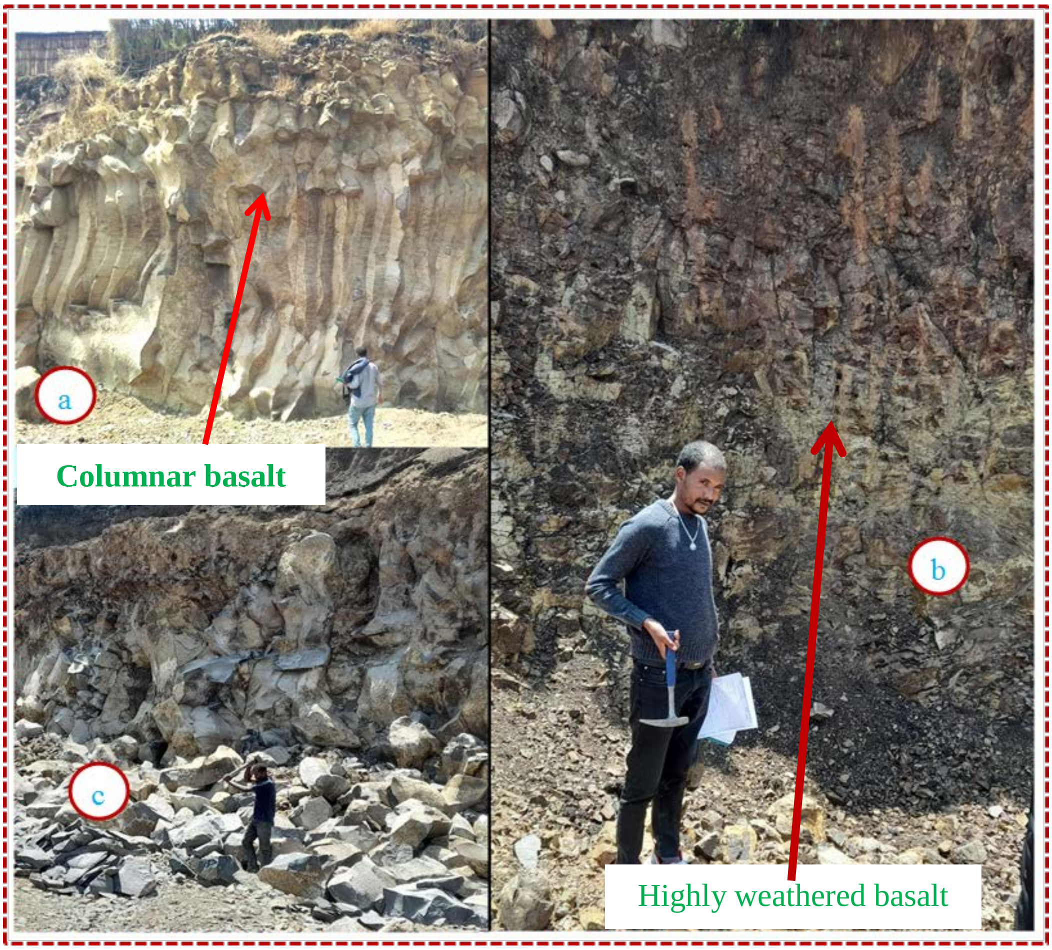


Figure 3.8 Huluka quarry site aphanitic basalt (a), highly weathered basaltic mixed or intercalated with black cotton soils (b) and Aleltu quarry site Phanaritic basalt (c).

3.7.3.4 Residual Soils

As observed from river cut, slop cut, and test pit at the time of field investigation, Ambo Town is dominated by thick residual soil. In the southeastern, northeastern , eastern and western (to the south of Ambo-Guder main road) part of town, especially, along with the slop cut of Ambo to Guder road, Huluka river side and around basalt quarry site dark color residual soil (black cotton soil) coverage is observed.

There is also other types of residual soils in the study area which is mainly covered southern, western (to the north of main asphalt road) and northern parts of the study area highly reddish and dark red in color (lateritic soil) which is derived from a wide variety of rock weathering under strongly oxidizing and leaching conditions. The average thickness of soil in the town is 3m- 15m observed from quarry site, slope cut (river side) and road cut area. From test pit, river and slop cut observation; there is considerable variation of soil type with depth within test pit seven which is sample (TP7A and TP7B) was taken from the same test pits.

As seen from the six test pits position the soil formation and gradation are similar up to the depth of 3m. About six Test pits (TP1, TP2, TP3, TP4, TP5, and TP6) were lithological correlated with each other. The remaining test pits (TP7) lithological correlated with dark color (black cotton soil) deposits from the surface to 2m depth and blow this depth light dark color mixed with weathered basalt was observed and sample was taken from 3m depth and the test pit was named as (TP7A and TP7B).

3.7.4 Geological Structure

Geological structures affect the stability of civil engineering structures when we constructed the civil engineering structures on discontinuous rock masses. Geological structures make the rocks weaker and more permeable. If rock masses fractured it would be a conduit for groundwater to flow in it (Tsegaye, 1993).

i) Fault: The most important structural features of the Ambo areas are the Addis-Ambo-Gedo fault zone, east-west oriented and the NNE-SSW trending Guder lineaments, which cross one another almost at a right angle near Ambo town and this fault is normal to the town.

ii) Joints: joint is the most commonly observed structures on basalt rock and sandstone which are found in the study area. The major joint sets observed in the study area NNE-SSW, NE-SW, NNW-SSE and W-E for joint (JS1, JS2, JS3 and JS4) respectively. Figure 3.9 shows a rose diagram of joints measured by using Dips 6.00 software in the study area.

iii) Lineament: from geological structure, typically a lineaments was appear aligned Huluka, Aleltu and Dabis river valleys with the series of normal faults in the study area.

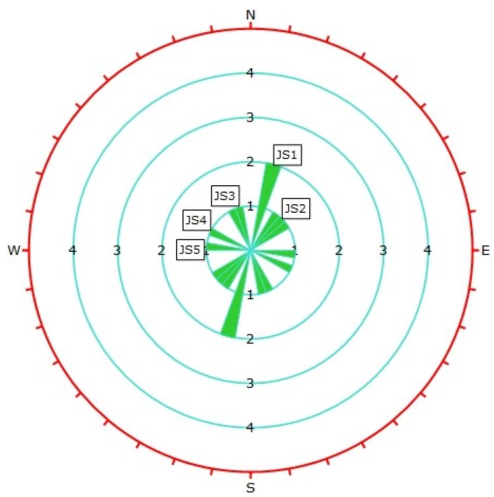
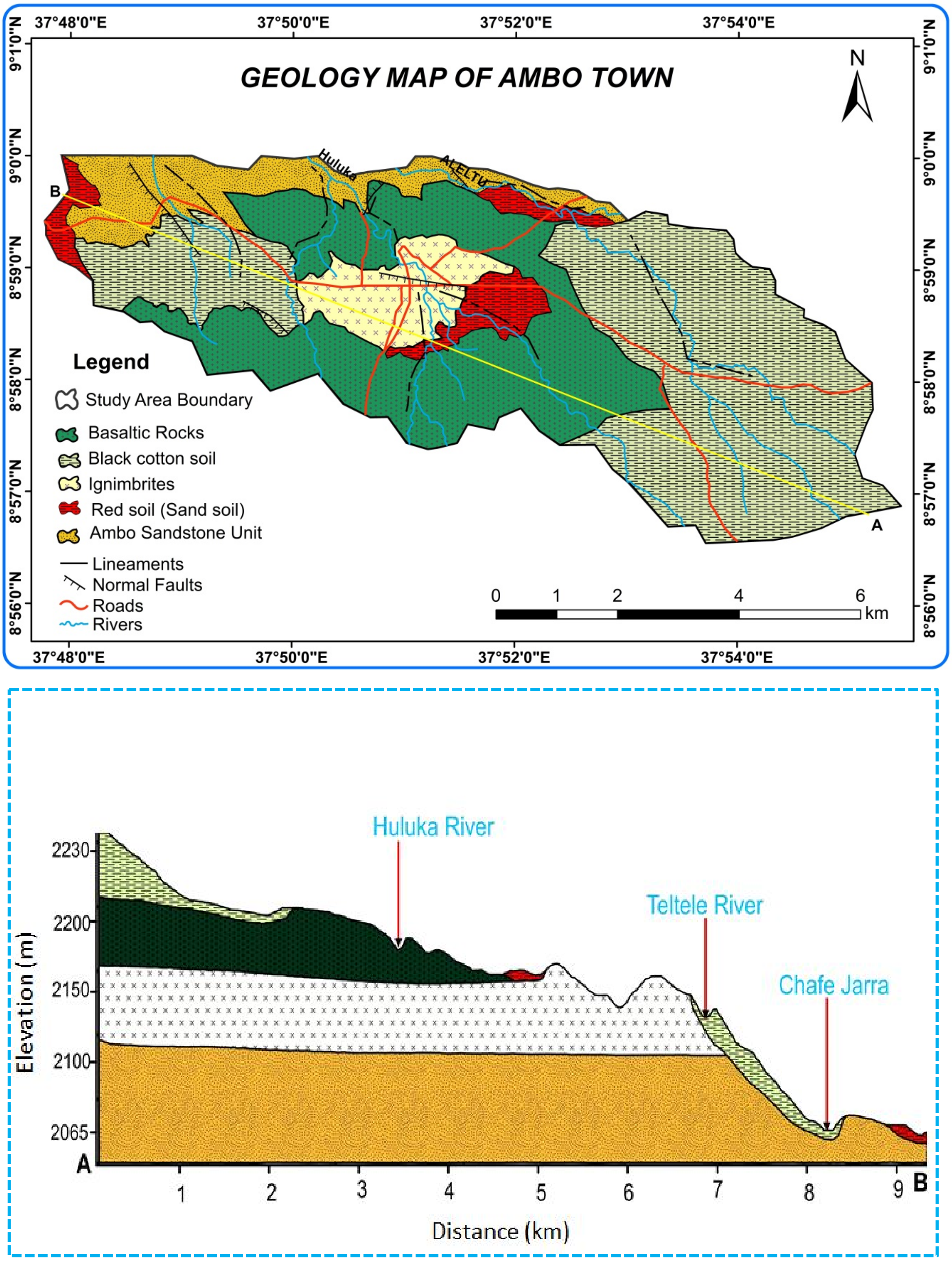


Figure 3.9 Rose Diagram of joints of the study area plotted using dips6.00 software.

Finally, a geological map of the study area was produced at a scale of 1:25,000 with geological cross-section from the geological map of the study area (Figure 3.10).



.Figure 3.10 Local geological map of the study area and its cross section along A-B.

3.8 Land Forms and Geohazards of the Study Area

3.8.1 Land Forms of the Study Area

The study area is characterized by different types of landforms, such as flat land in southeastern and northeastern parts of the towns, while the northern and western parts of the town and its peripheries are characterized by hilly/ ridge landscape rift like land forms which is created due to graben/foot of normal fault that is strike from east to west that is Ambo-Gedo normal faults. The relief of the study area is 450m, it mean that, the difference between high elevation (2350m) and low elevation (1900m). The low laying area (western part) of the study area is mainly coved by thick overburden (residual soil and fine sands) and the eastern part is overlying by black cotton residual soil. Based on the range of slope, the study area is characterized by steep to gently slope which ranges from 0 % to 121 % (Figure 3.11).

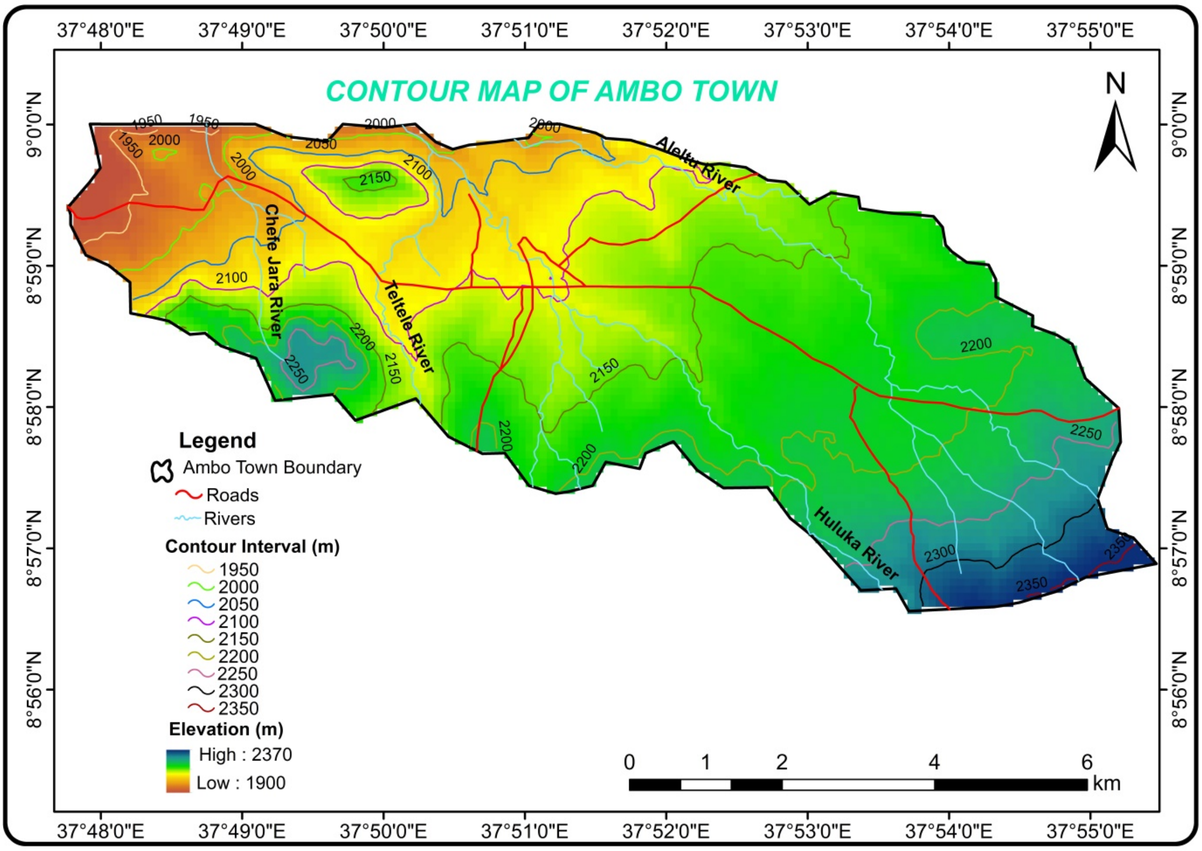


Figure 3.11 slope and contour map of the study area

3.8.2 Geohazards of the study area

Geological hazards can pose serious constraints on development. However, if the area prone to hazards is identified in advance, it will be very easy for the local government units to establish control or mitigating measures to diminish the effect of such hazards. Therefore, the establishment of land use zoning considering geo-hazards is very important to protect people and their properties (Malka, 2022). At present time geological hazards which affecting the study area is man-made hazards and ground water increment during summery season. The rock falls from quarry site of the study area affect many peoples and animals because most of the people build their house on the top of the quarry or blow the bench of quarry sites and the mining method of those all quarry is manual digging/ there is no machinery used for rock breaking. According to information was taking from local people, at Aleltu basaltic quarry site above five peoples injured and one person is died before a year and also there is the same occurrence in senkele Sandstone quarry site referred (people of the area).

3.8.3 Seismicity of the Study Area

Slope failures that are produced by earthquakes may occur as a result of an increase in driving movement or a decrease in shear strength. The horizontal dynamic component of the force increases during an earthquake's shaking, especially over shorter periods of time with greater acceleration (Jibson, 2011). Depending on the documented history of previous destructive earthquakes, the country is categorized into five distinct seismic risk zones under the Ethiopia Building Code of Standard (EBCS-1995).

It is divided into five main seismic zones. Zones 0 through 4 make up these five zones. These zones range from seismically unaffected (zone 0) to less harmful (zone 1) to high-risk (zone 4) regions (Lemenkova, 2022). As a result, the research area is located in zone 1 of the Ethiopian Building Standard's seismic hazard map (Figure below). Therefore, for a 100-year return time, the ground acceleration for the research region (zone 1) is 0.03 a (EBCS, 1995).

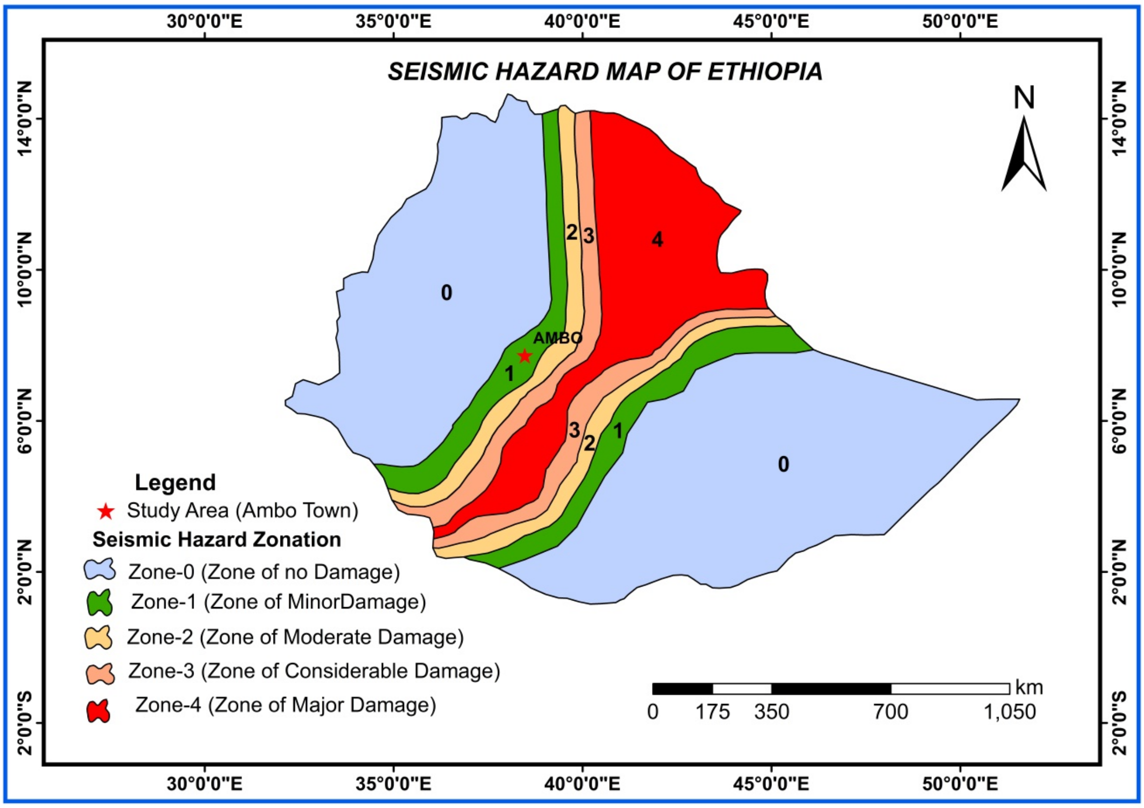


Figure 3.12 Seismic hazard map of Ethiopia for 100 years return period (EBCS, 1995)

CHAPTER FOUR

4. METHOD AND MATERIALS

4.1 Introduction

The main important components in reaching the objectives, responding to the research questions, and resolving the research issues mentioned earlier in the introduction is to use a technique that is logical and appropriate. As a result, various data regarding the topography, geology, structural discontinuity, and hydrogeology of the study area were gathered in order to undertake detailed soil and rock characterization in the study area and identify some of the important slope sections of the quarry site. The investigation was carried out using a thorough field survey and sample collection, laboratory testing of soil and rock samples, and recording of slope discontinuities. The general procedures used in this study were a review of the literature or preliminary desk study, field surveys, laboratory testing of soil and rock, data processing, analysis, and interpretation. Following is a discussion of the broad methodology and approaches employed in the present study.

4.2 Desk Study, Reconnaissance Survey, and Secondary Data Collection

A thorough review of a number of published and unpublished journals, papers, scientific conferences, standards, and recommendations on the subject matter connected to the research topic was done at this stage. For instance, a review of the causes of several engineering structure failures, as well as popular theories and approaches for engineering geological characterization and slope discontinuity analysis, was conducted. Additionally, to determine the physiographic condition of the research area, secondary data from satellite photography (such as Google Earth) and DEM from USGS Earth Explorer were used. A quick site reconnaissance assessment is then conducted to gather data on the general state of the rock, soil, and slope stability of the study area. After that, using the information gleaned from the literature review and reconnaissance survey, conceptual framework and general methodology (Figure 4.7) were constructed.

Following the creation of the conceptual framework, secondary data from the Ethiopian Geological Institute, the National Meteorological Service Agency, and the Geospatial Agency of Ethiopia were gathered. These data are essential for engineering geological characterization and include geological maps, meteorology data, and topographic maps. The graph of mean monthly rainfall distribution (Figure 3.4) was plotted to understand how rainfall from meteorological data, in particular, is one of the key triggering elements for weathering and disintegration of geological materials and slope instability of quarry site which is found in the study area (Raghuvanshi, 2017) the impact of precipitation on the overall stability of the study area's rock and soil. The seismic hazard map of Ethiopia (Rawat & P, 2019), which offers the amplitude of ground acceleration to be predicted during a 100-year return period, was used as the source for seismic hazard, the other primary triggering reason for geological condition.

4.3 Measurement, Field Description, and Recording

The attitude (strike and dip direction) of the geologic structure, its extents, and length were measured and recorded for the preparation of a geologic and engineering geological map of the study area using a field notebook, GPS (Geographic position System), clinometer, a geological hammer, a Schmidt hammer for testing the strength of rock mass, and meter tape. Additionally, important field photographs that can yield useful data were captured. The town's geodynamic process and its dangerous settings were discovered.

4.3.1 Geological Mapping

Geological mapping was done along traverse lines that span the principal lithological units and geological structures of the research area's strike directions. The mapping was done using a Global Positioning System (GPS), notebook, colored pencil, and other tools on the base map that was created during desk research. Using ArcGIS 10.8 software, the fieldwork data was transformed into a geological map of the research area (Figure 3.10).

4.3.2 Discontinuity Survey

To give parameters for geologic material characterization, slope stability analysis, and rock mass categorization, a discontinuity survey has been conducted (Zare et al., 2011). As a result, information about the discontinuity's parameters was collected in accordance with ISRM (1978) guidelines. The Brunton compass was used to estimate the slope and discontinuity orientations. Using a tape meter, the aperture, spacing, and persistence of discontinuities were determined. Based on how the geological hammer responded to blows, the degree of rock weathering was also calculated. Similar visual estimates were made in the field for various discontinuity conditions such surface roughness, infill material, and associated groundwater condition (Pickering & Aydin, 2015).

4.3.3 In-Situ Strength Test and RQD Determination

In-situ tests were carried out utilizing an N-type Schmidt hammer on the exposed rock surfaces in order to determine the uni-axial compressive strength of rocks. According to ASTM C805 (2018), this test was conducted on a uniformly smooth rock surface that had been polished with the device's included rubbing stone (Wijk, 1980). After rubbing, each indicated location's Schmidt hammer rebound value was noted. It was decided to convert this Schmidt rebound value to unconfined compressive strength (Jamshidi, 2022). Using a linear formula, the rebound value is translated to the unconfined compressive strength.

The RQD using the volumetric count method was carried out on a surface of exposed rock. By counting the amount of discontinuities inside a 1m*1m region, this method determines the rock quality designation, and RQD is calculated by the following relationship equation which is described in chapter two (2.1).

RQD = 115-3.3Jv (Deere, 1988) (Eq. 2.1)

Where JV, or the volumetric joint count, is the total number of joints for all joint sets with lengths ≥ 10 cm. additionally, the area's general ground water condition and condition of discontinuity were observed. The rock mass rating was completed by converting the Schmidt rebound values to unconfined compressive strength using the following equation (4.1), using various discontinuity parameters and RQD as an input parameter.

UCS=3.03R-30.3 (Wang *et al.*, 2016) (Eq. 4.1)

Where UCS is uniaxial compressive strength in Mpa, R is the representative rebound value and 3.03 and 30.3 is given constant number. Joint wall Compressive Strength (JCS) was also determined from Schmidt hammer derived UCS as it is a crucial parameter for estimating shear strength parameters of failure plane. According to Barton (1976), and Bell (2007), JCS is equal to UCS if the rock is un-weathered/nearly un-weathered. However, JCS can be as low as 25% of UCS if the wall of the rocks is highly weathered (Barton, 1976; ISRM, 1978; Sharma et al., 1999). Hence, in this study this concept was used to determine JCS from Schmidt hammer derived UCS and the degree of weathering of rocks.

Figure 4.1 In-situ strength of intact rock using Schmidt hammer rebound test for the estimation of uniaxial compressive strength (UCS): A) preparing an equally spaced square of 25mm box and B) conducting the test with Schmidt hammer.

Table 4.1 Location for discontinuity survey, in-situ strength testing (Schmidt rebound test)

Location	Sample code	Easting	Northing	Elevation	Rock type
senkele section one	RSS1	370047	994564	2028m	Reddish senkele sandstone
senkele section two	RSS2	369996	994509	2010m	Whitish senkele sandstone
Huluka Quarry site	RSS3	374690	994459	2077m	Phanaritic basalt (Huluka Quarry site)
Aleltu quarry site	RSS4	375622	992170	2125m	Aleltu quarry site (Aphanitic basalt)
Hot spring section(1)	RSS5	374207	992758	2117m	Welded Ignimbrite
Hot spring section(2)	RSS6	374050	992894	2091m	Un-welded Ignimbrite

4.3.4 Sampling Techniques

From identified areas, Disturbed samples of soils were collected from a maximum of three meter depths (3m) for laboratory testing (for index and engineering properties tests). Eight samples were collected from seven test pits with a maximum depth of three meters and minimum depth of one point five meter (1.5m). Test pits were dug manually by using a shovel and other materials. The samples were kept in a plastic bag to keep their natural moisture content. Five representative rock samples were also collected from different location based on the types of lithology's which identified in the study area. At the time of sampling vertical variation of a soil profile was observed and recorded.

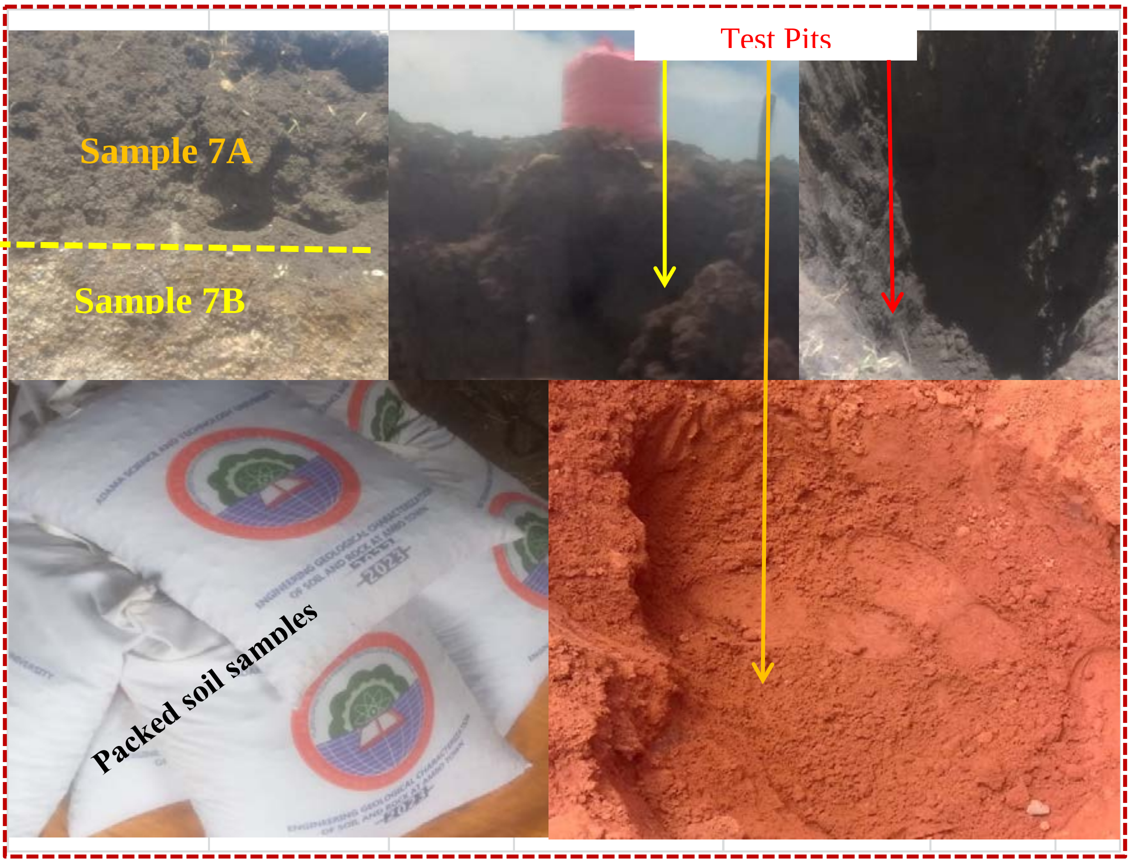


Figure 4.2 soil sample collecting techniques from the field and packed to take laboratory

Table 4.2 locations of test pits where the soil samples was taken

Location	Sample code	Easting	Northing	Elevation (m)	sampling Depth (m)
Ambo bottling water area	TP1	370156	994394	2026	3
Faris Bridge site	TP2	371834	993140	2079	3
Ambo University Hachalu Campus area	TP3	379170	991249	2202	1.5
Aleltu northern of town	TP4	375208	994308	2110	2.5
Ambo Stadium area	TP5	374488	992592	2119	2.5
Meja Eyasus Church area	TP6	375399	992504	2134	1.5
Awaro Condominium site	TP7A	376227	993366	2147	TP7A: 2
	TP7B				TP7B: 3

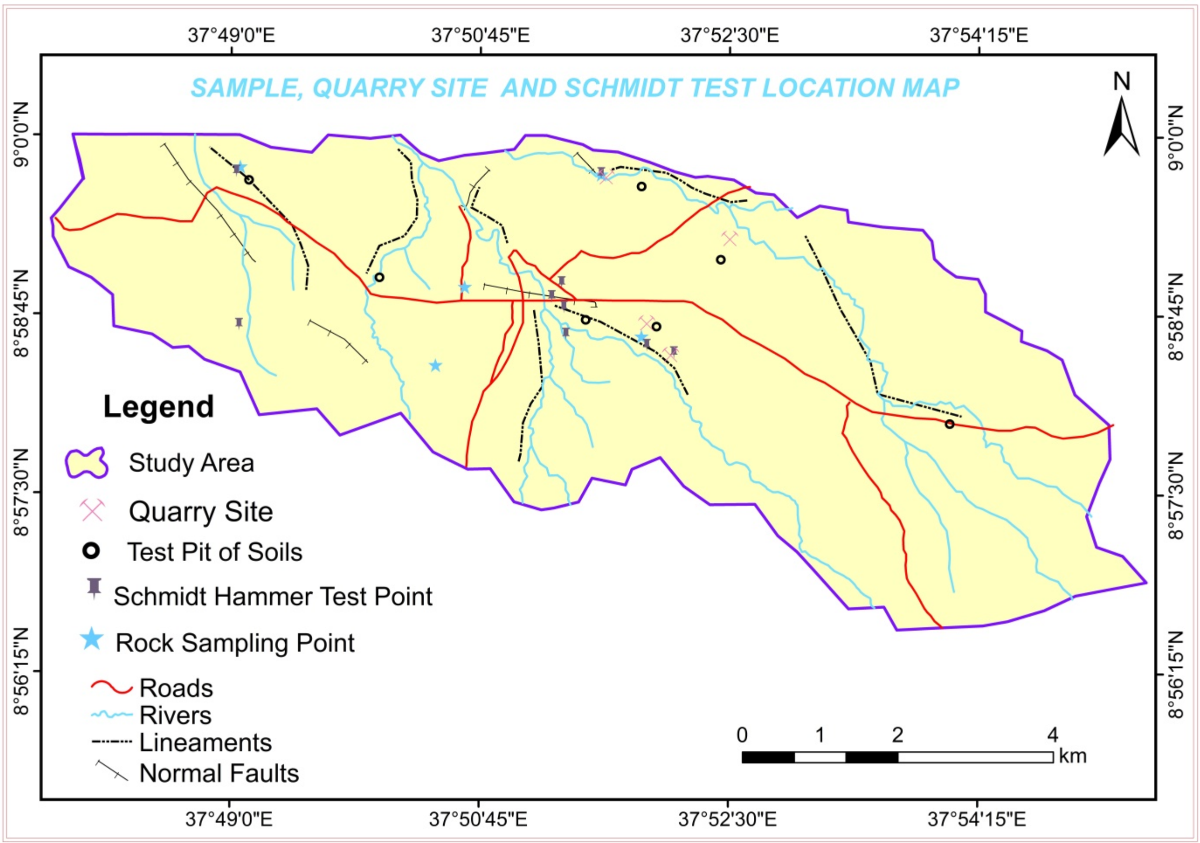


Figure 4.3 Schmidt hammer test points, rock sampling points, quarry site, soil test pits and geological structures location map of the study area.

4.4 Laboratory Work

To identify the various soil parameters required for the engineering geological characterization and slope stability analysis of the study region, a series of laboratory tests were carried out on representative rock and soil samples that had been obtained. The following was a discussion of the laboratory tests that were performed for this investigation.

4.4.1 Laboratory Test on Soil Sample

For the characterization and classification of the soil in the research region, many laboratory tests are required. The primary input parameters for the slope are the shear strength parameters and soil unit weight. Representative soil samples collected from the research region are subjected to laboratory testing that is carried out in accordance with ASTM standards (table 4.3).

Figure 4.4 Soil sample preparation for the laboratory tests.

Table 4.3 Type of test and standard employed for each type of soil laboratory tests

No	Type of test	Standard employed
1	Direct shear test	ASTM D 3080 (2002)
2	Natural moisture content	ASTM D 2216 (1998)
3	Unit weight	ASTM D 2937-00
4	Compaction test	ASTM D 698
5	Sieve analysis	ASTM D 422 (1998)
6	Atterberg limit test	ASTM D 4318 (1998)
7	Specific gravity	ASTM D 854
8	Hydrometer test	ASTM D 422 (1998)

4.4.1.1 Natural Moisture Content (NMC)

Representative soil samples were collected from seven different places, sealed in plastic to preserve their in-situ natural water content and a moisture content (w) test was performed on them. Utilizing the oven drying method, the test was carried out on fresh soil samples in accordance with ASTM D 2216 (1998) standard. The soil samples were extracted from the plastics, and a digital balance was used to weigh their moist mass (Mw). The weighed samples were then dried in an oven at 105°C for 24 hours to determine their dry mass (Md). The following equation was then used to determine the natural moisture content (w):

$$W\ (\%) = \frac{Mw - Md}{Md} * 100 \qquad \text{(Eq. 4.2)}$$

4.4.1.2 Specific Gravity

The ASTM D 854, 1988, standard was followed to estimate specific gravity, and roughly 50g of dry soil was weighted to produce Mo (Mass of dry soil). And To produce Ma (Mass of water and pycnometer), fill the flask with distilled or demineralized water until the point of etching line is reached. With a funnel, add the soil to the flask and pour out half of the water before washing the dirt down the inside neck of the flask. Vacuum the flask for 30 minutes while periodically mixing the mixture. Connect the hose and stopper to the vacuum source. To obtain Mb (Mass of soil, water, and pycnometer), the flask is weighed after being filled with distilled water to the etch line.

4.4.1.3 Particle Size Analysis

For each soil sample that was collected from the study area, the ASTM D422 (1998) standard particle (grain) size analysis test was carried out. Mechanical sieve analysis and hydrometer analysis are the two steps that make up the particle size analysis (Arora, 1997). The determination of the grain size distribution within the coarse grained part of the soil was aided by the mechanical sieve analysis, which required the use of a number of mechanical sieves. The component of the soil material with fine grains was subjected to the hydrometer study. The material that made it through sieve number 200 was regarded as having fine grains. The soil's characteristics were determined using the results of these experiments.

To protect the screen from damage, the first 1000g of oven-dried soil was prepared, and the sieves were cleaned using a brush without sieve number 200. The soil was then placed in the sieves, which were arranged in decreasing order of sieve size. The material that was retained on each sieve was counted after shaking the sieve, and the percentage of material retained was calculated. The material that was kept on the pan after passing through sieve number 200 was regarded as a fraction of the material with finer grains and was used for hydrometer analysis. The first 50g of soil from the soil that was still on the pan was measured for the hydrometer analysis. In a tiny evaporation dish, the soil was then combined with sodium hex methyl phosphate. The sodium hex methyl phosphate soil solution was then transferred into a dispersion cup and blended with a shake mixer for two minutes before being completely transferred into a clean sedimentation jar and allowed to sit for 48 hours. The thermometer was inserted to measure the temperature after the hydrometer was introduced and the first reading was taken at two minutes. At 0.5, 1, 2, 4, 8, 15, 30 minutes, 1, 2, 4, 8, 16, 24 and 48 hours, the hydrometer and temperature values were taken.

4.4.1.4 Atterberg Limit (Liquid Limit and Plastic Limit) Test

The tests used to classify the soil's fine grain component are the liquid limit test and the plastic limit test. The practical application of Atterberg limit in engineering geology involves soil description, quantitative soil categorization, and linkage to engineering qualities like shear strength, which significantly aids slope stability analysis. Atterberg limit can be used to determine the water content at which soil transforms from one state to another (Moreno & Alonso, 2016). While soils with low water content have greater shear strength, soils with high water content are less resistant to deformation (Arora, 1997).

The Atterberg limit values were determined in this work using the Casagrande method, and ASTM D4318 procedures were followed. With the aid of a spatula, the air-dried soil sample that passes sieve No. 40 (0.425mm) was produced and combined with distilled water until it forms a thick, homogenous paste. The liquid limit (LL) of soil was determined to be the amount of water needed to close grooved soil samples by a distance of 13 mm after 25 blows administered at a rate of 2 rotations per second (Haigh, 2015). Additionally, on a ceramic plate, portions of soil specimens used to determine liquid limits were rolled into a thread measuring 3 mm in diameter at various water contents. The region of the soil samples that were used for liquid limit determination were rolled into a thread of 3mm diameter at different water crumbled when rolled into a thread of 3mm diameter was then taken for water content determination which corresponds to the plastic limit (Barnes, 2021). The plasticity index (PI) of the corresponding soil specimens was then determined using the following equation:

$$PI = \text{Liquid Limit (LL)} - \text{Plastic Limit (PL)} \qquad \text{(Eq. 4.3)}$$

Finally, using the Atterberg limit test, particle size analysis, coefficient of curvature and homogeneity, and USCS classification, the soil was categorized. Because the soil was gathered from the most important parts of the study area, it was additionally classified in accordance with AASHTO. Equation was used to determine the group index. (4.4).

$$GI = (F200-35)\,[0.2+0.005(LL-40)] + 0.01(F200-15)\,(PI-10) \qquad \text{(Eq. 4.4)}$$

Where, GI=group index, F200=percentage passing sieve No.200, LL=liquid limit, PI=plasticity index.

4.4.1.5 Free-Swell

The free swell test is the most basic experiment used to assess the soils' ability to swell (David et al., 2004). A 100ml graduated cylinder filled with distilled (tap) water and kerosene gas is used for this test, and 10ml of oven-dry soil that has passed the No. 4 sieve is slowly poured into it. Kerosene did not mix with the soils, and this is crucial for determining the true values of the free swell of the soil samples. According to Chen (1975), the ultimate volume of the suspension was read after 24 hours.

4.4.1.6 Compaction Test

To ascertain the research area's ideal moisture content and maximum dry density, the compaction proctor standard method described in ASTM D 698 was applied. The soil that was passing through the No. 4 sieve, weighing about 2 kg, was taken. Without the collar, the soil mass and the mold were weighed (Wm). After placing the soil in the mixer and progressively adding water to achieve the necessary moisture content (w), the collar is lubricated. The soil was taken out of the mixer and layered three times within the mold. The compaction process was started with 25 blows for each layer. The drops were manually placed. A straight edge that has been sharpened was used to carefully cut the collar and remove the soil that extends above the mold. The water content of the sample's top, middle, and bottom was measured after the mold and its enclosing soil (W) were weighed and the soil was pushed out from the mold using a metallic extruder. To increase the soil's water content, it is added water once more and mixed with the soil (Seifu et al., 2023).

Figure 4.5 shows the soil laboratory compaction test

4.4.1.7 Direct Shear Test

The soil's resistance to soil particle deformation caused by shear stress is known as its shear strength (Hariri et al., 2023). Under consolidated drained conditions, direct shear apparatus was used to measure the soil's shear strength in accordance with ASTM D 3080 (2002) guideline at Addis Ababa Science and Technology University (AASTU) laboratory. A 60*60*20mm soil sample with a natural moisture content that had been compacted and molded was used for the test. Three different consolidated pressures of 100Kpa, 200Kpa, and 300Kpa were used to shear the specimen until it broke. Then, at regular intervals, the horizontal and vertical shear displacements were recorded. The shear strength was computed using equation (Eq. 4.5) based on Mohr-Coulomb failure criteria utilizing the test result (the shear strength parameters).

$$S = c + \sigma \tan\phi \qquad \text{(Eq. 4.5)}$$

Where, S=shear strength, c=cohesion, ϕ=friction angle, and σ=normal stress.

Figure 4.6 showing a direct shear test of soil in the laboratory.

4.4.2 Laboratory Test on Rock Sample

4.4.2.1 Unit Weight Test

Unit weight of the rock is a crucial parameter in engineering geological characterization of rock. It can be determined in the laboratory either through buoyancy or saturation technique (Abdulazeez, 2014). In this study, a buoyancy technique was adopted to determine the unit weight of irregular rock samples following the guideline suggested by ISRM (1978) at the laboratory of Gia Geotechnical Engineering service (GIA). Accordingly, the following steps were followed during this test.

- ✓ The rock specimens were weighed at their natural moisture (m1) content using a digital balance.
- ✓ The rock specimens were then oven-dried at 105°C for (24 hrs.) and their corresponding dry masses (m2) were recorded.
- ✓ The oven-dried rock samples were then immersed into a cylinder with water of known volume (V1) and the corresponding change in volume due to this immersion of rock was recorded as V2.
- ✓ The volume due to the rock specimens was then calculated by subtracting V2 from V1. Finally, the dry density and dry unit weight of rocks were determined using (Eq. 4.6) and (Eq. 4.7) respectively.

$$pd = \frac{m2-m1}{V2-V1} \qquad \text{(Eq.4.6)}$$

$$\gamma d = Pd * g \qquad \text{(Eq. 4.7)}$$

Where P_d is dry density, γd is dry unit weight and g is gravity. The saturated unit weight was also determined using the same procedure. However, during this test saturated weight was first determined by keeping the rock specimen in the water for (24 hrs.) instead of determining dry weight (Discenza et al., 2021).

4.4.2.2 Point Load Test

To ascertain the unconfined compressive strength of entire rock, a point load strength test was conducted. At Gia Engineering Private Limited Corporation Geotechnical Engineering Service, the tests were carried out in accordance with ASTM D5731-16 (1995) standards to assess the strength of irregular rock samples. Five lump rock samples overall were investigated in the following manner using natural moisture content conditions on rocks obtained at each chosen rock section.

Unsymmetrical rock specimens were cut to the proper size and shape for the point load testing machine before the test began using a geological hammer and chisel. Before the test began, the irregular rock samples' height, width, and length were noted (Jamshidi, 2022). The failure load (P) peak values are then recorded after inserting a rock sample into the test apparatus and applying the load continuously until failures are observable. Later, using the empirical equation proposed by Broch and Franklin (1972), which demonstrated the relationship between the corrected point load strength and the uniaxial compressive strength of the rock, the average of the corrected point load strength was converted to the uniaxial compressive strength of the rock.

4.5 Data Processing, Analyzation, and Interpretation Approach

To transform the unprocessed data of soils and rocks into relevant information about the research area, in situ and laboratory data were analyzed and interpreted. With the assistance of several software tools, data processing and interpretation were carried out. A few of them include Google Earth Pro to gather geospatial data, Dip6.00, Slide software for slope stability analysis, Microsoft Excel 2010, Arc Map10.8, Golden Surfer 16, and Microsoft Office tools for calculation, sorting, and tabulating raw data. Rose net projection can be used to show the geologic structure model of the area.

Classifying the area's soil and rocks, identifying problematic areas of the town for the practice of civil engineering (possible expansiveness of soil), and creating an engineering geologic map of the research area are the main objectives of data analysis and interpretation. According to the Unified Soil Classification Systems (USCS) and the classification proposed by the International Association of Engineering Geologists (IAEG, 1981) and American Society for Testing Materials (ASTM), the rocks and soils of the town will be categorized for engineering purposes based on their engineering behavior.

4.5.1 Soil Plasticity Analysis Based On Casagrande

On a graph of plasticity index (ordinate) versus liquid limit (abscissa), laboratory data from several locations in the Town were shown. It was discovered that the graph's different zones correspond to clays, silts, and organic soils. The "A-line," which delineates the boundaries between clays (above the line) and silts and organic soils (below the line), is a line described by the equation PI= 0.73(LL- 20) %. The upper limit of the correlation between the plasticity index and the liquid limit is defined by a second line, the U-line, which is written as PI= 0.9(LL -8) %.

4.5.2 Potential Soil Expansiveness Based Free Swell and Activity of Soil

Engineering structures can crack, tear, and tilt due to the major issue of expanding soil. Clay mineralogy and the percentage of clay in soil samples are two fine soil features that work together to create soil expansiveness (Moreno, 2022). It was possible to gauge the extent of soil expansion in the study region from the results of free swell tests and activities. Skempton"s activity chart is used to categorize the town's surroundings as active, inactive, and medium based on the activities of the soil sample from the study region.

4.6 Classification of Soil

The soil of the study area was classified based on their engineering behavior according to the Unified Soil Classification system (USCS) and the AASHTO soil classification system.

4.7 Rock Mass Quality

Rock Mass Quality is one of the empirical techniques used to evaluate the stability of the slope section and measure the quality of the rock mass. The RMR system was used in this work, in accordance with Bieniawski (1989), to assess the quality of rock mass at structurally regulated critical slope parts of the rock. The rebound value obtained from the in-situ Schmidt hammer test utilizing (Eq. 4.1) was used to determine the five parameters utilized in this classification system, UCS of complete rocks.

According to (Eq. 2.1), the RQD was also calculated using Palmström (2005) Volumetric Joint Count Method. Other factors, including the spacing and condition of discontinuities and the condition of the groundwater, were also directly evaluated in the field. As a result, the tape meter was used to measure the distance between discontinuities as well as discontinuity characteristics including aperture and persistence. Visual estimations were made of the groundwater condition and the remaining discontinuity conditions, such as roughness, wall degradation, and infill material. The rock mass quality at the chosen slope portions was then determined by adding a rating of five fundamental factors in accordance with Bieniawski (1989) rating scheme.

4.8 Classification of Rock

Rock classification is based on an examination of field data. That was obtained by observation and measurement. These details include the discontinuity's length, filling, and separation as well as its unconfined compressive strength, level of weathering, and groundwater condition. These criteria are used to classify the rocks in the research area in accordance with UCS, RQD, and RMR standards.

4.8.1 Classification Based On UCS

Using the calculation in equation (4.1) above, the Schmidt rebound value obtained from the Schmidt test is converted to unconfined compressive strength, and the UCS value of rock is then classified in accordance with the ISMR standard.

4.8.2 Classification Based On RQD

The RQD by volumetric count method was conducted on an exposed rock surface. This method calculates rock quality designation by counting the number of discontinuities that lie within an area of 1m *1m, which is exposed on the outcrops using the equation (2.1) relationship which is described in chapter two above.

4.9 Materials

- ✓ Geological mapping and discontinuity surveys will be conducted using a GPS by collecting longitude, latitude and elevation data of the study area.
- ✓ A geological hammer for rock sample breaking or rock cutting and for estimation of rock strength by identifying the sound and sample bag for sample collection from the field to take for laboratory test.
- ✓ Brunton compass to measure the slope surface or to recording the dip amount-dip direction and strike of the rocks, and a measuring meter for measuring the depth of test pit, length of discontinuity of rocks, to measure aperture and to measure geological units one from the other, to identifying it is map able or not to prepare multi-purpose engineering geological map.
- ✓ The field measurements of the Uniaxial Compressive Strength (UCS) of intact rocks were also determined by an N-type Schmidt hammer.
- ✓ In the laboratory, soil samples will be tested using Casagrande and sieve analysis kinds of equipment for grain size, Atterberg limit, direct shear strength, etc.
- ✓ Map preparation and data analysis and interpretation were done using Arc GIS 10.8, Surfer 16, Dips 6.00, etc. slide 6.0 software, Microsoft excel, and other relevant software.

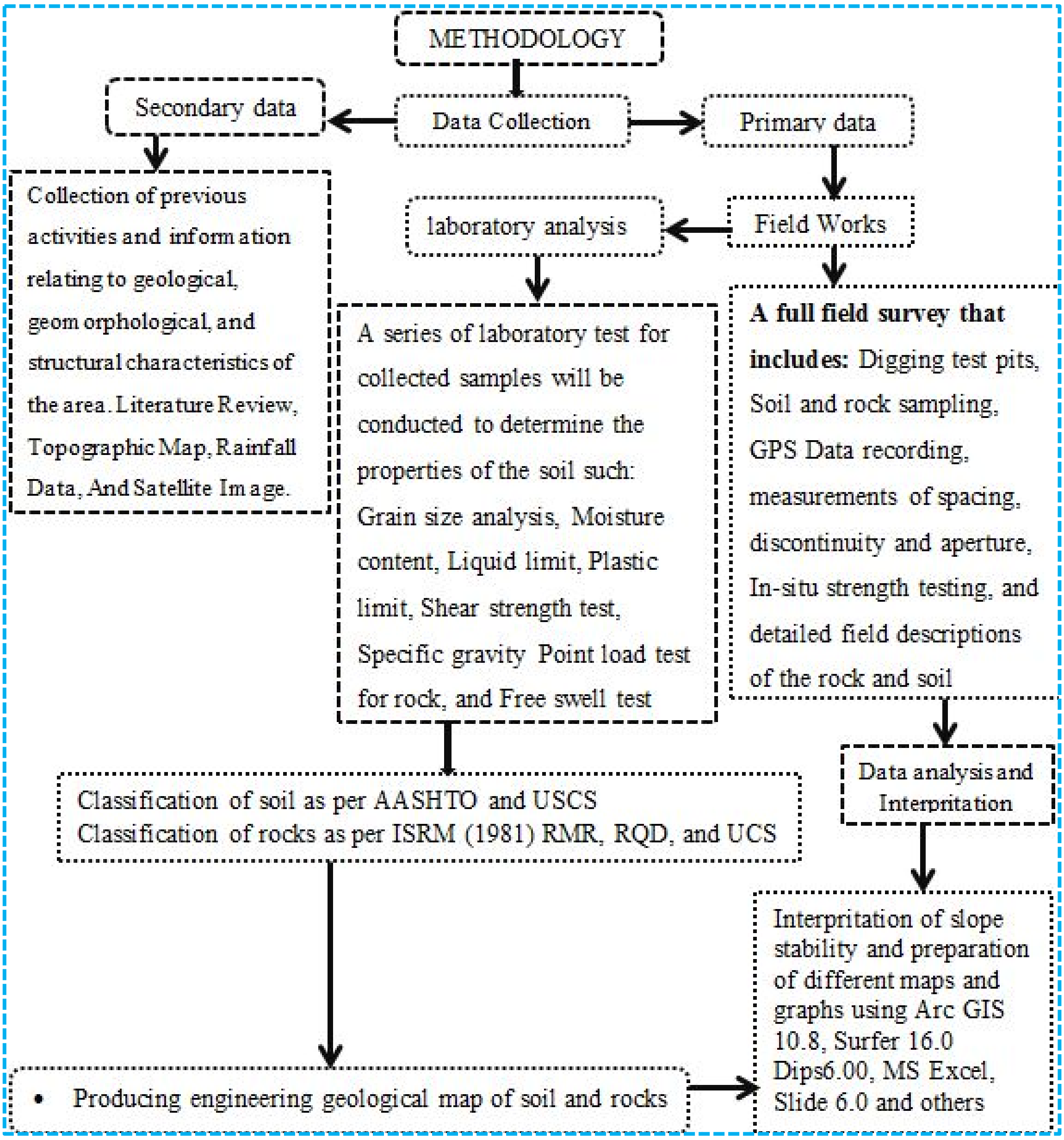


Figure 4.7 The flow chart showing the general methodological outline of the research work.

CHAPTER FIVE

5 RESULTS AND DISCUSSIONS

5.1 Engineering Geological Characterization and Classification of Soils and Rocks

5.1.1 Engineering Geological Characterization and Classification of Soil

Since the majority of the research area's soils are classified as residual soil types, engineering descriptions and sample collection have been done on soils taken from seven test pits. The field description and laboratory soil tests that were carried out in accordance with the study's goal were very helpful in identifying and categorizing the soils. Different laboratory tests, including mechanical and hydrometer analysis, Atterberg limits, specific gravity, and direct Shear Strength tests (proctor test), were used to characterize the engineering geology, determine geotechnical properties, classify the soil, and determine input parameters for slope stability analysis.

5.1.2 Engineering Properties (Characterization) of Soils

Eight representative soils were taken from seven test pits, and a laboratory index test was done on them to characterize the engineering geology of the soils in the research area. The representative soil sample that was collected from the study area through the town was subjected to various tests in the lab in order to determine the physical characteristics of the soil, including grain size analysis, Atterberg limit tests, Natural moisture contents, specific gravity tests, density and unit weight, compaction tests and shear strength tests are the findings and analysis of tests conducted on the soil index qualities.

5.1.2.1 Natural Moisture Content

Natural moisture content (NMC) is the in-situ water content of soils measured using freshly excavated soil samples without exposing them to air. In accordance with ASTM D 2216 (1998) standard, the test was carried out utilizing oven drying on soil samples obtained from seven distinct locations and eight soil samples from the study region. On the soil sample taken from seven test pits, the results' analysis was done. In (Table 5.1), the results are displayed. The amount of water in the soil has a significant impact on the physical characteristics of most fine-grained soils, especially clayey soils. According to test data, the research area's soils have a natural moisture content that ranges from 8.24 to 18.62%.

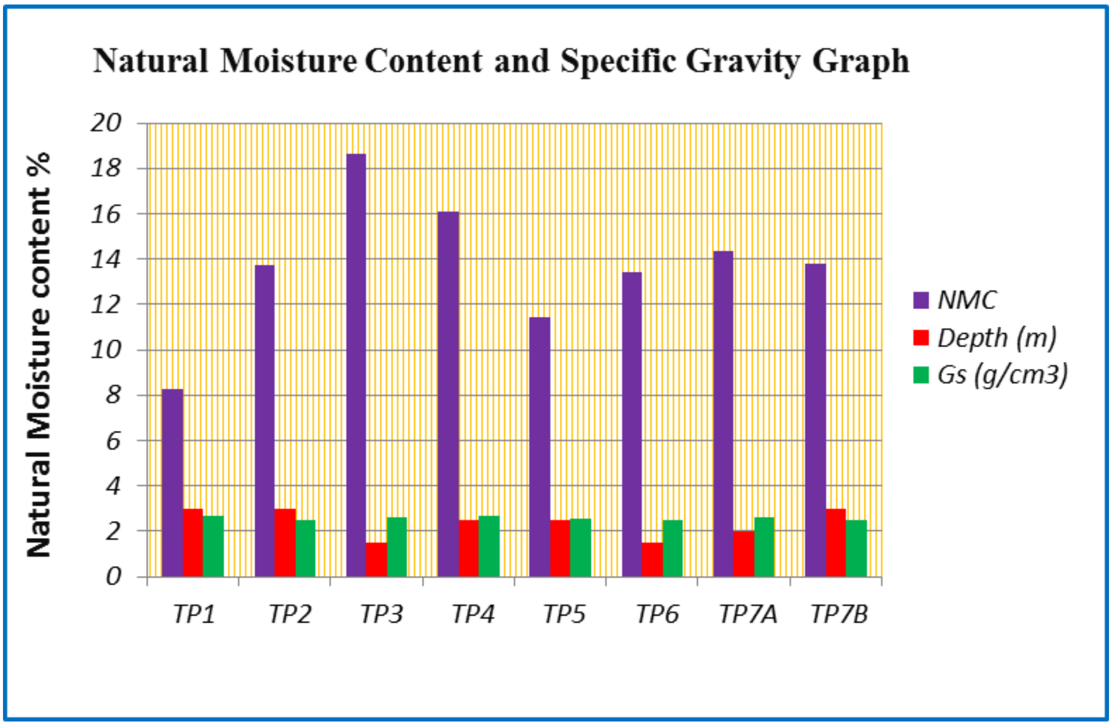


Figure 5.1 Graph of natural moisture contents and specific gravity laboratory result of the soils which were taken from the study area from different locations.

5.1.2.2 Specific Gravity of Soils

For this investigation, the ASTM D 854 (1998) standard for each sample was used to determine the specific gravity of soils on the representative sample that was gathered from various places. The analysis's findings demonstrated that the study area's soils had an average specific gravity that ranged from 2.49 to 2.68 (Table 5.1).

Table 5.1 natural moisture contents and specific gravity laboratory results of the soils which were taken from the study area at different locations.

Sample Code	Easting (E)	Northing (N)	Elevation (m)	Depth (m)	NMC (%)	Specific Gravity (g/cm3)
TP1	370156	994394	2026	3	8.24	2.68
TP2	371834	993140	2079	3	13.75	2.49
TP3	379170	991249	2202	1.5	18.62	2.6
TP4	375208	994308	2110	2.5	16.1	2.65
TP5	374488	992592	2119	2.5	11.42	2.53
TP6	375399	992504	2116	1.5	13.24	2.5
TP7A	376227	993366	2147	2	14.33	2.6
TP7B				3	13.82	2.51

5.1.2.3 Grain Size Analysis

To categorize soils according to their textural characteristics, grain size analysis was utilized. It is employed to figure out how the soils' particle sizes are distributed. The analysis's findings are shown in (Table 5.2). The outcome demonstrates that the gravel content ranging from 0-2.98%, coarse sand 1.39-10.86%, fine sand 2.56-60.73%, silt 3.41-41.13%, and clay 0.36-57.84%. Generally, except TP1, TP5 and TP7B, fine fraction is the dominant grain size in the studied soils.

Table 5.2 Grain size Analysis results of the soil taken from the study area

No	Location	Sample Code	Depth (m)	Percent amount of particle size (%)				
					% Sands		% of fine	
				% Gravel	Coarse sand	Fine sand	Silt	Clay
1	Ambo bottling water area	TP1	3	0.17	5.33	60.73	33	0.36
2	Faris Bridge site	TP2	3	1.69	8.52	16.86	38	34.47
3	Ambo University Hachalu Campus area	TP3	1.5	0.45	2.68	3.03	38	55.56
4	Aleltu northern of town	TP4	2.5	0	1.39	2.56	40	55.84
5	Meja Eyasus Church area	TP6	1.5	2.98	10.86	13.91	41	31.12
6	Awaro Condominium site	TP7A	TP7A=2	0	1.81	3.3	37	57.84

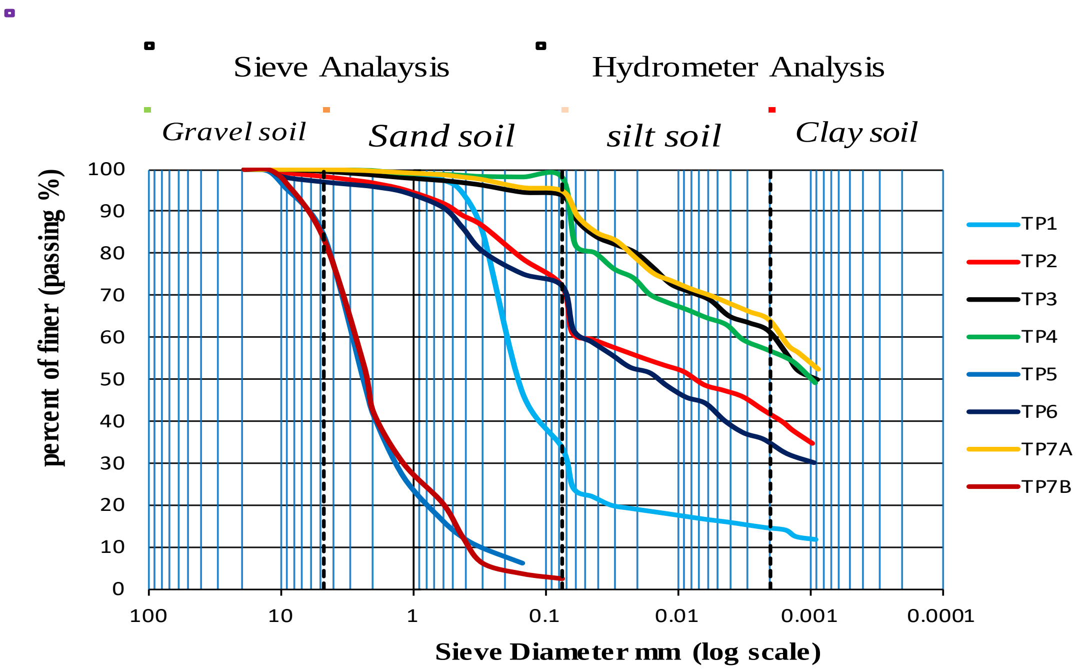


Figure 5.2 Particle size distribution curve from sieve and hydrometer analysis

5.1.2.4 Direct Shear Strength Tests

One of the most crucial elements of geotechnical engineering is soil shear strength. The shear strength of the soil affects several design factors, including the bearing capacity of shallow and deep foundations, slope stability, retaining wall design, and pavement design. When subjected to the highest predicted applied stresses, structures and slopes need to be secure and stable against complete collapse. In this work, the direct shear tests were used to determine the shear strength parameters, such as cohesion and angle of internal friction, as indicated in (Table 5.3). The internal friction angle of the soil samples range 6.46° to 20.18° and cohesion values range from 64.97kPa to 97.2kPa (Table. 5.3).

Table 5.3 Shear Strength Parameters of a Soil Samples (TP1-TP7B)

Sample Code	Soil types	Shear Strength Parameter	
		C, (kPa)	Angle of Friction (ϕ)
TP1	Inorganic clays (CL)	64.97	6.46
TP2	Inorganic silts (ML)	97.2	12.68
TP3	Inorganic clays (CH)	80.83	13.11
TP4	Inorganic silts (MH)	70.5	7.07
TP5	Well graded sands (SW)	65.821	18.94
TP6	Inorganic silt (ML)	88.015	16.77
TP7A	Inorganic silts (MH)	76.8	8.7
TP7B	Well graded sands (SW)	72.525	20.18
TP = Test Pit, C = cohesion			

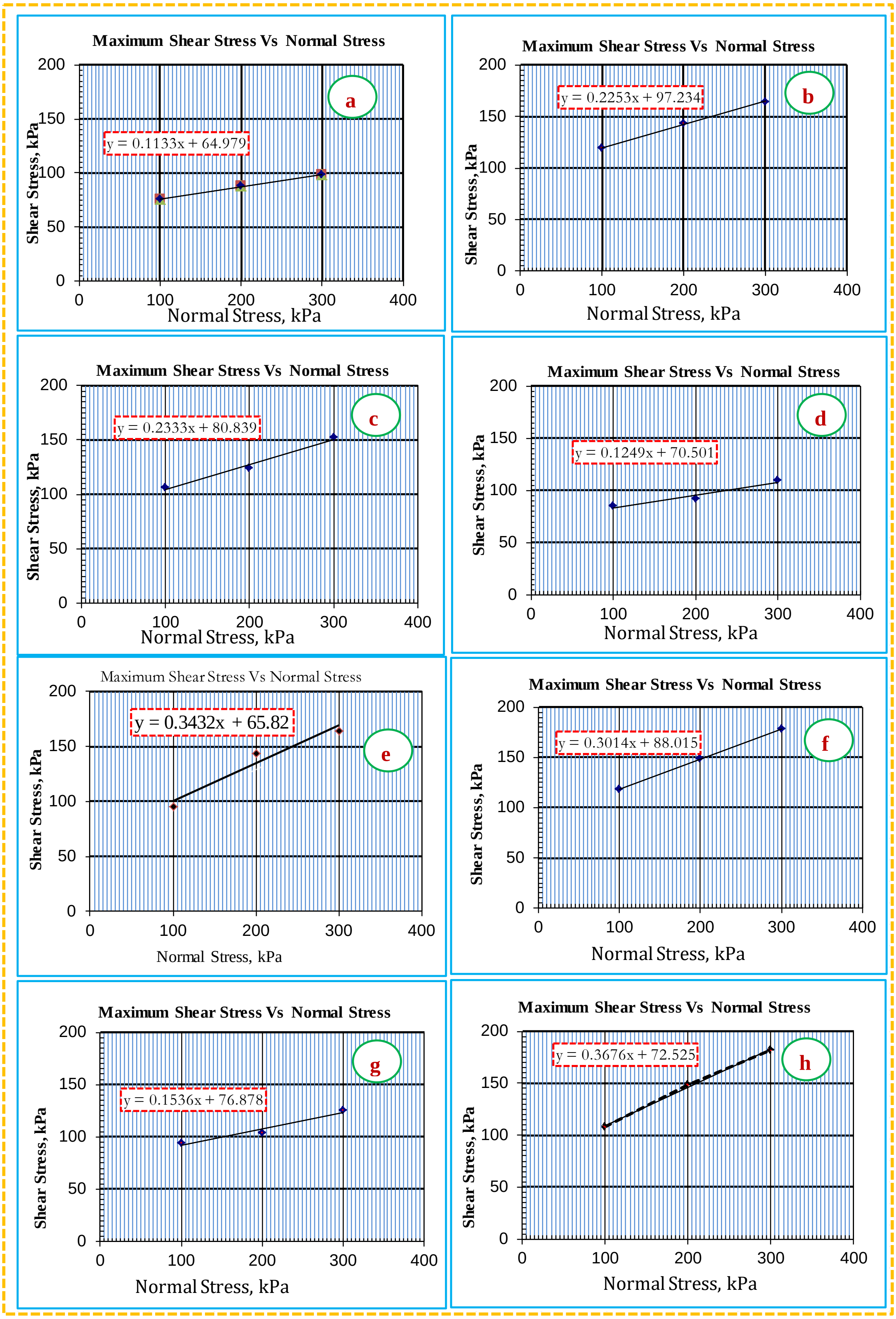


Figure 5.3 Shear Stress Vs Normal Stress Laboratory Results of soil samples TP1 (a), TP2 (b), TP3(c), TP4 (d), TP5 (e), TP6 (f), TP7A (g) and TP7B (h) of the study area.

5.1.2.5 Standard Compaction Tests

To determine the relationship between the moisture content and the dry density of a specific soil sample at a specified compactive effort, the Standard Proctor Compaction Test is carried out. Using a common proctor test, this test was run on eight soil samples that were taken from seven test pits in the study region. Based on the findings of the compaction test, the maximum dry density (MDD) and optimal moisture content (OMC) of the soil were examined in order to remold the soil for the shear strength test. The eight soil samples from the area that were tested in the laboratory and their related compaction curves are shown in (Fig.5.4). The results of the best moisture content (OMC) and maximum dry density (MDD) are shown in (Table 5.4) below.

Table 5.4 Maximum dry density and optimum moisture content laboratory results of the soil samples which were taken from the study area (TP1-TP7B).

Sample code	**Soil description**	**MDD (g/cm3)**	**OMC (%)**
TP1	clayey sands (SC)	1.78	15.72
TP2	Inorganic silts (ML)	1.28	26.79
TP3	inorganic clays (CH)	1.2	38.4
TP4	Inorganic silts (MH)	1.31	34
TP5	Well graded sand (SW)	1.46	23.09
TP6	Inorganic silt (ML)	1.45	26.26
TP7A	Inorganic silts (MH)	1.23	36.52
TP7B	Well graded sands (SW)	1.31	30.3
TP = Test pits, MDD = maximum dry density, and OMC = Optimum moisture contents			

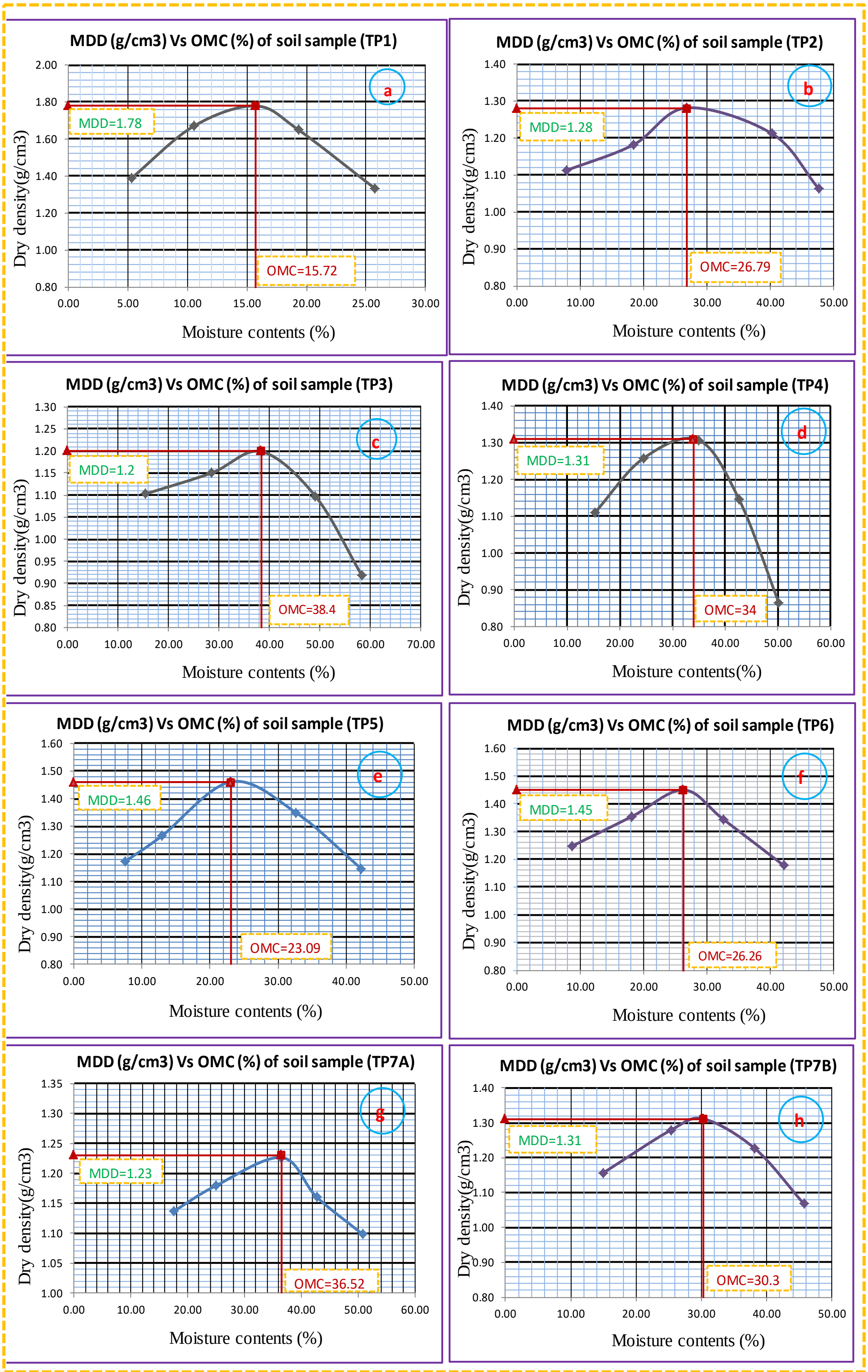


Figure 5.4 compaction curve laboratory results of the soil samples indicated as a) TP1, b) TP2, c) TP3, d) TP4, e) TP5, f) TP6 g) TP7A, and h) TP7B of the study area.

5. 1.2.6 Free Swell

According to the study area's data for free swell value, silty soil ranges from 25% to 30%, sandy soil is 0%, and clayey soil ranges from 40% to 80% (Table 5.3). The results of the soil-based swell index show that Ambo Town characterizes a very low to medium degree of expansion, meaning that TP1 has zero free swelling and a low expansion rate. Due to the fact that samples with a free swell value more than 50% show swelling potentiality behavior, care should be taken when designing civil constructions to be anchored to such soils. Therefore, only TP3 and TP7A out of eight samples had free swelling larger than 50% according to laboratory test results, and these should be taken into account when building foundations. Clay soil can swell or expand freely. According to Holtz and Gibbs (1956), who were cited in Hawi (2018), soils with a free swell value above 50% hardly show significant volume change even under extremely light loading. The weakly loaded structures may take significant damage if the free swell value of the soil is more than or equal to 100%.

Table 5.5 liquid limit, plasticity index, and plastic limit free swell laboratory results of soil samples of the study area and dominant soil types.

No	Sample ID	Depth (m)	Dominant soil types	LL%	PL%	PI%	FS%
1	TP1	3	clayey sands (SC)	24.06	10.34	13.71	0
2	TP2	3	Inorganic silts of low plasticity (ML)	47.56	33.91	13.65	25
3	TP3	1.5	Inorganic clays of high plasticity (CH)	86.81	47.6	39.21	72.73
4	TP4	2.5	Inorganic silts of high plasticity (MH)	57.16	30.94	26.22	40
5	TP5	2.5	Well graded sands (SW)	45.97	30.67	15.3	30
6	TP6	1.5	inorganic silts of low plasticity (ML)	47.1	26.65	20.45	30
7	TP7A	TP7A=2	Inorganic silts of high plasticity (MH)	84.04	43.8	40.24	80
8	TP7B	TP7B: 3	Well graded sands (SW)	60.53	28	32.525	45.46

5.1.2.7 Atterberg (Consistency) Limits Test

Atterberg limits were computed in the lab using the Casagrande method. Tests for the liquid and plastic limitations are part of the analysis. The plasticity index (PI) of the soils in the research area was calculated using the liquid limit (LL) and plastic limit (PL) values. The test results are summarized in Table 5.5. The test findings show that the LL and PL of the research area range from 24.06% to 86.807% and 10.34% to 47.6%, respectively. The plasticity index has a range of 13.65% to 40.24%. The soil in the research region is classified into various soil types by Das's (2002) classification of soil plasticity qualities.

As a result, soil sample (TP1) was assigned as clayey sands (SC), soil samples (TP3) classified as inorganic clays of high plasticity (CH), (TP4 and TP7A) were classified as inorganic silts of high compressibility (MH) index, sample of (TP2) was classified as inorganic silt of low compressibility (ML), soil sample (TP6) classified as inorganic silt of low plasticity (ML) and sample (TP5 and TP7B) classified as well graded sand types of soils. Generally, the larger the plasticity index, the greater will be the engineering problems associated with using the soil as an engineering material.

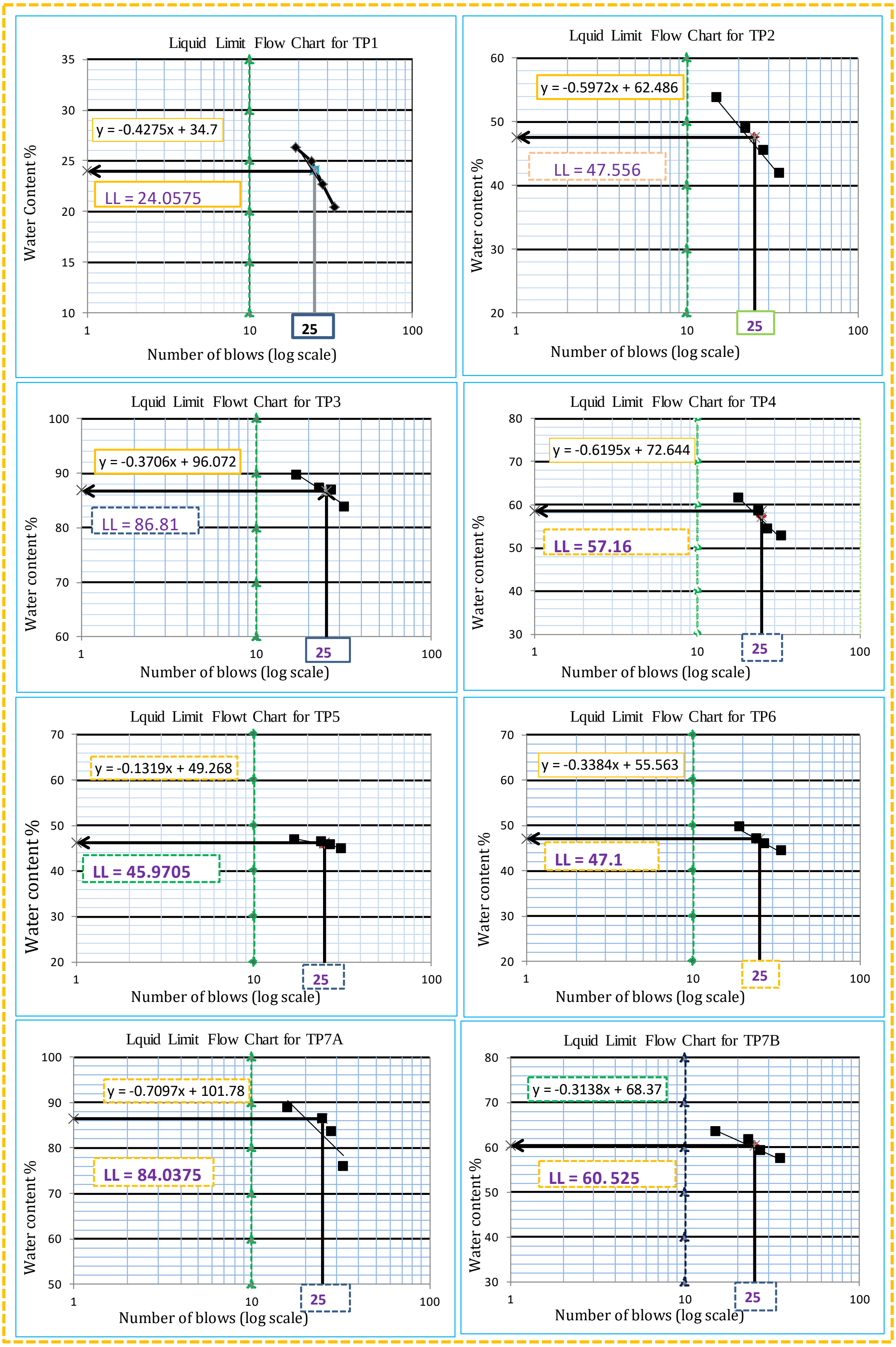


Figure 5.5 Liquid limit flow charts of soil which are indicate as TP1 (a), TP2 (b), TP3 (c), TP4 (d), TP5 (e), TP6 (f), TP7A (g) and TP7B on (h).

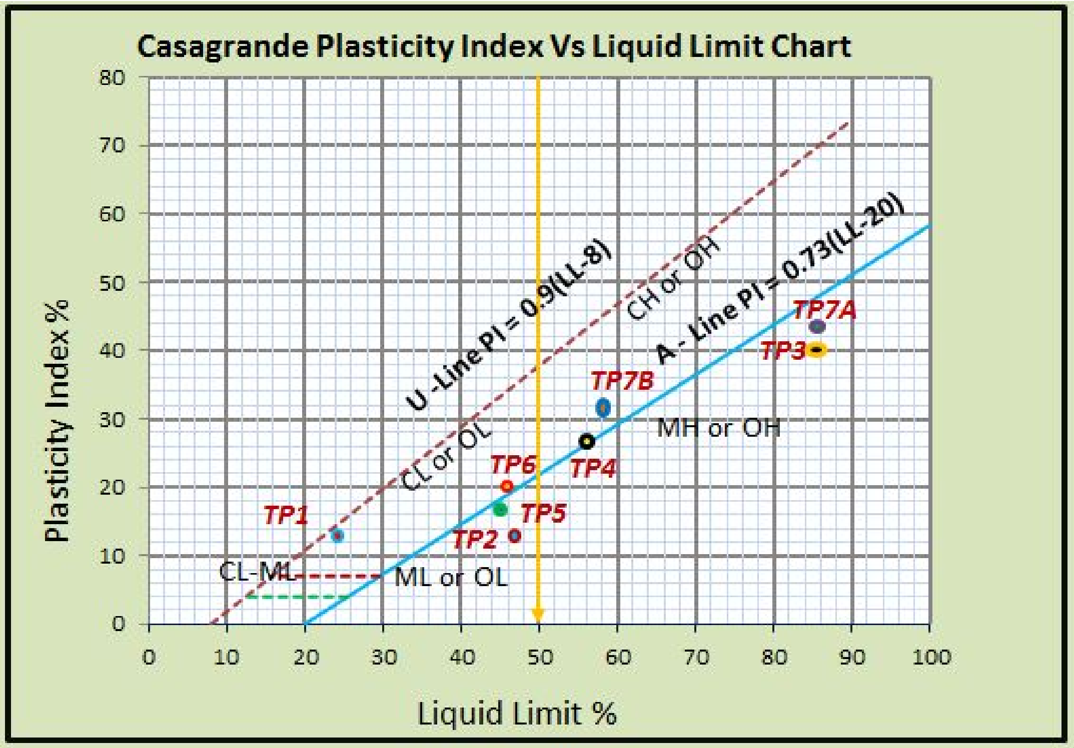


Figure 5.6 Plasticity Index Flow Chart of all soil samples of the study area

5.1.2.8 Activity of Soils

According to Skempton (1953), activity depends on the soil's plasticity index and the quantity of clay particles smaller than 2µm. It suggests the possible expansion and contraction (volume change) of the soil. Soils with activities less than 0.75 are typically moderate, those between 0.75 and 1.25 are medium, and those over 1.25 are highly expansive. The research area's soils are low and very expansive, and they have the characteristics of both highly active clay soils and inactive soil types (Table 5.6).

$$\mathbf{AoS} = \frac{\boldsymbol{PI}}{\%\ \boldsymbol{of\ clay\ fraction}} \qquad \text{(Skempton, 1953)} \qquad \text{Eq. 5.1}$$

Table 5.6 Activity (potential expansion) of the soils

No	Sample Code	Depth (m)	Liquid limit (LL %)	Plastic limit (PL %)	Plastic index (PI %)	Activity & Clay Minerals	Potential of expansion
1	TP1	3	24.06	10.34	13.71	38.08	Highly active
2	TP2	3	47.56	33.91	13.65	0.4 (kaolinite)	Inactive clays
3	TP3	1.5	86.807	47.6	39.21	0.71 (kaolinite)	Inactive clays
4	TP4	2.5	57.16	30.94	26.22	0.47 (kaolinite)	Inactive clays
6	TP6	1.5	47.1	26.65	20.45	0.66 (kaolinite)	Inactive clays
7	TP7A	TP7A=2	84.04	43.8	40.24	0.7 (kaolinite)	Inactive clays

5.1.2.9 Unit Weight of Soils

An essential factor in identifying the engineering geological characteristics of the soil is its unit weight. From this perspective, the unit weight of the soils taken from each study area test pit was calculated. The test results revealed that the research area's soil has a dry unit weight range of 11.27 to 18.34 and a saturated unit weight range from 12.83 to 20.18.

Table 5.7 Unit weight of soil samples which taken from the study area.

Soil Sample code	Dry Unit Weight (KN/m3)	Saturated Unit Weight (KN/m3)
TP1	18.34	20.18
TP2	11.85	13.48
TP3	11.34	13.45
TP4	12.07	14.01
TP5	13.02	14.57
TP6	12.45	14.09
TP7A	12.26	14.01
TP7B	11.27	12.83

5.1.3 Classification of Soils Based On Unified Soil Classification System (USCS)

Using gradation analysis and the results of laboratory testing for the Atterberg limit test, the soil samples collected from the study area's from selected site were categorized using the USC system (Table 5.8). The soil samples collected from the study area became under the USC systems of classification high plasticity silt (MH), high plasticity inorganic clay (CH), low plasticity inorganic silt (ML), medium plasticity inorganic silt (ML) and clayey sands (SC) of categories rather than sample TP5 and TP7B, those samples classified as well graded sands (SW) because, their size are greater than sieve #200. According to general engineering suitability of the soil (Arora, 2003), from the soil of the study area soil samples which are taken from test pit (TP3, TP4 and TP7A) characterized by high compressibility fair to poor shear strength and poor in engineering workability so, the results indicate that they are problematic soils for engineering application with higher swelling-shrinkage potential and we have to take considerations during construct Engineering structures on them. In the table 5.8 blow and on figure 5.6 above the detailed result and the UCS categorization of the soil for the study area is displayed.

Table 5.8 Classifications of Soils of the study area based on unified soil classifications.

Test Pits	Percent of particle size						Group Symbols	Soil Name
	Gravel	Sand	Silt	Clay	LL%	PI%		
TP1	0.17	66.06	33	0.36	24.06	13.71	SC	clayey sands
TP2	1.69	25.38	38	34.47	47.56	13.65	ML	Inorganic silts
TP3	0.45	5.71	38	55.56	86.807	39.21	CH	Inorganic clays of high plasticity
TP4	0	3.95	40	55.84	57.16	26.22	MH	Inorganic silts with high plasticity
TP5	16.7	79.4	3.9	0	45.97	15.3	SW	Well graded sands,
TP6	2.98	24.77	41	31.12	47.1	20.45	ML	Inorganic silts of low plasticity
TP7A	0	5.11	37	57.84	84.04	40.24	MH	Inorganic silts of high plasticity
TP7B	16.5	81	2.5	0	60.525	32.525	SW	Well graded sands,

5.1.4 Classification of Soils Based On AASHTO Classification System

The AASHTO approach assigns a group classification and a group index to the soil based on both grain-size distribution and Atterberg limits data. Soils are divided into seven groups using the AASHTO classification system, A-1 through A-7-6. This classification system places soils in categories A-4, A-5, A-6, and A-7 if more than 35% of them pass through sieves no. 200. Most of the soils in these categories are silt and clay-like substances. The soils in Ambo Town are categorized by the AASHTO system as A-2-6/A-2-7 and A-7-5, accordingly, as shown in table 5.9 below. This shows that the soils in the research area are suitable for sub-grade materials and are not suitable for it, respectively.

Table 5.9 Classifications of Soils of the study area based on AASHTO classification systems

Test pit	Percent of particle size						AASHTO classification	Soil Name	Rating
	Gravel	Sand	Silt	Clay	LL%	PI%			
TP1	0.17	66.06	33	0.36	24.06	13.71	A-2-6(0)	Silt or clayey gravel sand	good
TP2	1.69	25.38	38	34.47	47.56	13.65	A-7-5(11)	Clayey soils	Fair to poor
TP3	0.45	5.71	38	55.56	86.81	39.21	A-7-5(49)	Clayey soils	Fair to poor
TP4	0	3.95	40	55.84	57.16	26.22	A-7-5(31)	Clayey soils	Fair to poor
TP5	16.7	79.4	3.9	0	45.97	15.3	A-2-7(0)	Silt or clayey gravel sand	good
TP6	2.98	24.77	41	31.12	47.1	20.45	A-2-6(15)	Clayey soils	Fair to poor
TP7A	0	5.11	37	57.84	84.04	40.24	A-7-5(49)	Clayey soils	Fair to poor
TP7B	16.5	81	2.5	0	60.53	32.53	A-2-7(0)	Silt or clayey gravel sand	good

5.2 Engineering Geological Characterization of Rock

To identify and define the geological components of the soils and rocks in the research area, a thorough field examination was done. A thorough field survey was used to identify the different types of rocks, weathering, and fracture of the study location through eye observation. Although the four soil slope failures and the five significant rocks slope sections of quarry site from the field survey were noted. In addition, joint roughness varies from rough to smooth, with some being filled by soft and gouge (loose) materials, as demonstrated by field-determined discontinuity conditions.

5.2.1 Discontinuity Survey

To describe the rock mass of the research area, a discontinuity survey was conducted in accordance with International Society of Rock Mechanics (ISRM, 1978) standards. For the examination of the stability of rock slopes, the discontinuity parameters are essential. To determine the discontinuities characteristics, including joint orientation, joint spacing, joint filling, aperture, joint roughness, groundwater, and weathering state of the rock slope along the study region, a discontinuity survey was carried out along the crucial slope section quarry site in this study. Field observations were used to identify four important rock slope areas within the discontinuity survey measurements. Table 5.10 following contains a summary of the typical discontinuity survey measurements.

Table 5.10 Summary of representative discontinuity parameters found at significant rock slope sections in the study area is shown.

Discontinuity parameter	RSS1	RSS2	RSS3	RSS4
Joint spacing (cm)	100-150	75-80	60-75	60-65
Persistence (m)	15-20	5 to 7	5 to 8	3 to 10
Joint aperture (cm)	30-40	1 to 3	2 to 5	1 to 5
Filling materials	open	lose material	loose material	Lose materials
Roughness	rough	rough	S. rough	H. rough
Degree of weathering	SW	M weathered	Un- weathered	S. weathered
GW condition	Dry	Dry	Dry	Dry
SW = slightly weathered, M = Moderately weathered and RSS = Rock slope sections, GW = ground water condition				

5.2.2 In-Situ Rock Strength Test

In the current study, the Schmidt hammer strength test in the fields was used to measure the uniaxial compressive strength (UCS) of the intact rock at the key slope part of the quarry site

(Barton and Choubey, 1977). The ASTM D5873 (2001) standard was followed in conducting this test in order to get rebound values. Using empirical formulae (Eq. 4.1) proposed by (Wang a, et al. 2016), the typical average rebound value was transformed into UCS of rocks. According to (Table 5.11), the rock's lowest and maximum strengths were measured using the Schmidt hammer (N-type), and they were 72.72MPa, 55.45MPa, 84.23MPa, 90MPa, 106MPa, and 31.21MPa, , for rock slope sections RSS1, RSS2, RSS3, RSS4, RSS5, and RSS6 respectively.

Table 5.11 Schmidt Rebound and UCS values from different slope sections of the study area

Rock type and sample code	Trial No	Rebound value recorded from Schmidt hammer for each trial	Rebound value average		UCS from equation UCS=3.03RN-30.3 in (Mpa)
			R_{avg}	Rep R_{avg}	
Reddish senkele sandstone(RSS1)	1	26,30,36,28,40,33,32,34,25,28,29	31	34	72.72
	2	41,34,33,40,36,38,30,46,30,32,42	36.55		
	3	38,32,28,42,34,40,30,36,30,28,38	34.2		
whitish senkele sandstone (RSS2)	1	24,20,28,32,18,32,34,19,28,29	26.4	28.3	55.45
	2	32,30,26,25,33,22,25,24,34,28	27.9		
	3	28,30,26,32,36,29,32,28,30,34	30.5		
Huluka quarry Aphanitic basalt (RSS3)	1	52,50,40,42,44,38,52,50,48,46	46.4	44.87	106
	2	42,34,50,38,44,46,36,54,48,50	44.2		
	3	56,54,34,36,54,32,44,48,34,50	44.2		
Phanaritic basalt Aleltu Quarry site(RSS4)	1	40,48,46,48,45,42,36,40,50,52	44.7	45.4	107.3
	2	46,52,50,50,46,48,44,42,40,46	46.4		
	3	50,46,48,46,47,40,37,52,44,42	45.2		
Welded Ignimbrite(RSS5)	1	36,38,30,32,28,34,32,36,30,29	32.5	37.8	84.234
	2	48,30,50,46,32,30,42,40,44,38	40		
	3	42,40,46,42,40,32,46,38,44,38	40.8		
Un-welded Ignimbrite(RSS6)	1	20,19,21,22,18,19,20,23,18,24	20.4	20.3	31.21
	2	18,22,24,19,25,20,22,20,18,16	20.4		
	3	22,18,24,17,19,16,20,23,21,22	20.2		

5.2.3 Rock Quality Designation (RQD)

According to Palmström (1982), the volumetric joint count (JV) for rock mass exposed at significant slope parts of the study area was used in the current investigation to calculate the rock quality designation. As a result, the study area's essential slope sections' exposed rock surfaces were used for the volumetric joint account measurements. After taking measurements from each key slope segment, equation (2.3) was used to determine the volumetric joint account from the average joint spacing. The Rock quality designation index (RQD) calculated using the joint count method is summarized in (Table 5.12). According to the combined volumetric count method, the minimum and highest values of RQD are 74.58%

and 84.624%, respectively (Table 5.12). According to Deere (1988) classified the rock mass exposed to the section of the study area into classifications for fair and good rock mass.

Table 5.12 Rock quality designation index determined from the volumetric count

Slope Section	JV (av)/ 1m*1m	RQD%	According to Deere (1988)
RSS1	9.205	84.624	Good
RSS2	12.25	74.58	Fair
RSS3	10.27	81.1	Good
RSS4	11.75	76.225	Good

5.3 Laboratory Test Results for Rock Samples

At each significant slope segment of rock in the study area, the laboratory tests that are used for the engineering characterization of rocks were typically conducted for classification reasons. To ascertain engineering features of rocks, such as point load tests, water absorption, unit weight and density of rocks, laboratory examination of rocks was conducted for this study. The tests used in this investigation are described in the following sections.

5.3.1 Unit Weight and Density of the Rock Samples

The unit weight of the rock is one of the input parameters for slope stability assessments. According to an ISRM (1978) standard using equation 4.6, the density of rock was determined in this experiment using buoyancy techniques for irregular rock samples. As a result, the study area's five rock slope sections were used to gather representative rock samples, and their corresponding unit weight was determined. Dry unit weights range from 17.85 to 25.1 KN/m^3, and saturated unit weights range from 19.4 to 25.51 KN/m^3, according to the results of (Table 5.13).

Table 5.13 Laboratory results of dry and saturated unit weights of rock samples which taken from the study area.

Rock Sample Code	Rock Units (Description)	Dry unit weight γd (KN/m^3)	Saturated Unit weight γsat (KN/m^3)
RSS1	Fresh, slightly weathered Sandstone	19.13	20.6
RSS2	Moderately weathered Sandstone	18.64	20.21
RSS3	Fresh, Un-weathered aphanitic basalt	25.1	25.51
RSS4	Slightly weathered Phanaritic basalt	24.23	24.43
RSS5	Highly weathered Ignimbrites	17.85	19.4

5.3.2 Point Load Tests

To determine the complete rock's uni-axial compressive strength, point load strength tests were carried out. For this investigation, the point load strength test was carried out at Gia Engineering private limited corporation of the Geo-technical Engineering Service Addis Ababa, on irregular rock samples taken from each chosen rock slope section in accordance with ASTM D 5731-16 (1995). Five irregular rock samples that were taken from the study area's important rock slope sections were estimated. The adjusted point load strength was then converted to the required uni-axial compressive strength (UCS) using the equation (Is (50)*24). According to (Table 5.14) corrected point load strength test findings on distinct fresh rock samples taken from each rock slope segment, the rock's point load strength index ranges from 1.013MPa to 10.12MPa, with corresponding uni-axial compressive strengths of 24.31MPa to 242.88MPa. Overall, the point load strength test findings on the rock samples demonstrated that the strength of the rock in the research area's slope portion ranged from very low to very high, accordingly (Deer and Miller, 1966).

Table 5.14 point load and uniaxial compressive strength laboratory results of intact rocks

Sample Code	Is(50) Mpa	UCS(Mpa)	According to Deere & Miller (1966)
RSS1	1.6	37.8	Low strength
RSS2	1.06	25.5	Very low strength
RSS3	10.12	242.88	Very high strength
RSS4	6.47	155.4	High strength
RSS5	1.013	24.31	Very low strength

RSS = Rock Slope Section, Is (50) = Corrected point load test,
UCS = Uniaxial compressive strength of intact rocks

5.4 Rock Mass Classifications

The classification of rock masses incorporates a variety of inputs from intact rock and discontinuity qualities, both of which have a big impact on how well rock masses behave in engineering contexts. For the purpose of this study, precise measurements from the study region were taken in order to gather information for the classification of rock masses using the Rock Mass Rating (RMR) and Geological Strength Index (GSI). As a result, an overview of these divisions will be covered in the paragraphs that follow.

5.4.1 Geological Strength Index

In order to calculate the shear strength and deformation characteristics of rocks, rock data software uses the Geological Strength Index (GSI), a geological engineering assessment of rock mass. In this work, the quantitative GSI chart proposed by Merinos and Hoek (2000) was used to visually quantify the geological strength index of the important rock slope portions at several quarry sites. As a result, the GSI value of the rock mass was assessed based on the state of the discontinuity surfaces and structures in the rock masses, and it was assessed at the study area's surface outcrops and slope face. The identification of rock quality and the use of field obtained GSI values as input parameters for finite element and limit equilibrium slope stability analyses were also included. Five rock slope portions had their GSI values measured in the field, according to Hoek and Merinos (2000). The crucial rock slope section's projected field value for the Geological Strength Index (GSI) is shown in Table 5.15.

Table 5.15 Geological strength index (GSI) value of the rock slope estimated in the field.

Slope Sections	Rock Units	Geological Structures	GSI, value estimated in the field	Surface condition
RSS1	SW. Sandstone	Blocky	45	Good
RSS2	MW. Sandstone	Blocky	40	Fair
RSS3	UW. aphanitic basalt	Blocky	65	Good
RSS4	SW. Phanaritic basalt	Blocky	60	Good
GSI = Geological strength index, RSS = Rock slope sections, SW = Slightly weathered, and, UW = Un-weathered				

5.4.2 Rock Mass Rating

The rock mass rating (RMR), according to Bieniawski (1989), is employed in a number of rock engineering applications, including mining, hydro-power, tunneling, and hill slope stability. Accordingly, the quality of the rock at the critical rock slope in the study area was assessed using the (Bieniawski, 1989) basic rock mass rating system by determining out five fundamental factors, including groundwater condition, spacing of discontinuity, condition of discontinuity, and uni-axial compressive strength of intact rock (UCS).

In the current study, the classification of rock masses was carried out based on the UCS result obtained from point load tests, and RQD values were determined from volumetric joint account methods in the field. Additionally, the field evaluated parameters for the classification of rock masses, such as joint spacing, discontinuity condition, and groundwater condition, were estimated and evaluated in the fields.

According to Bieniawski (1989), rating values were applied for each criterion to classify the research area's rock mass. The RMR system goes from 62 to 84 in four slope parts of a rock, and the rocks are rated as fair (class III) to extremely good (class I) rocks. According to appearances, the rock at slope sections RSS1, RSS2, and RSS4 represent a good rock type, whilst the rock at slope section RSS3 is rated as very good. The (Table 5.16) below provides a complete list of discontinuity parameters, including rating values and estimated RMR.

Table 5.16 RMR values and classification of rocks in the slope sections (Bieniawski, 1989)

Slope Sections	Rock Types	Five basic RMR Parameter rating value						
		UCS	RQD	SoD	CoD	GWC	RMR	Description
RSS1	Reddish Sandstone	4	17	15	25	15	76	Good rock
RSS2	Whitish Sandstone	4	13	10	20	15	62	Good rock
RSS3	Aphanitic basalt	12	17	10	30	15	84	Very good
RSS4	Phanaritic basalt	12	17	10	25	15	79	Good rock
USC = Uniaxial compressive strength, RQD = Rock quality designation, SoD = Spacing of discontinuity, CoD = Condition of discontinuity and GWC = Ground water condition,								

5.5 Assessment of Geomorphological, and Geodynamic Hazards of the Study Area

A surface geodynamic problems in the study area is landslide, weathering, and problematic soil (expansive soil) discussed below.

Any hazard-related earth processes are broadly classified as Geo-Hazard. A geohazard is a geological state that may lead to loss of life and damage to properties. Geohazards are geological and environmental conditions and involve long-term or short-term geological processes. Geohazards can be relatively small features, but they can also attain huge dimensions (e.g., submarine or surface landslide) and affect local and regional socio economy to a large extent. Other Geo-hazards can be earthquakes, volcanoes, Avalanches and Floods (Burton et al 1978). Small scale landslide, problematic soil (expansive soil) and ground water increment during rainy season are the most commonly observed geodynamic and hydrogeological possess responsible for damaging of engineering structures and changing the surficial features of the study area.

5.5.1 Erosion and Weathering

In the study area the topography, parent materials and degree of weathering governs the type and development of soils. In humid regions the temperature and the amount of moisture controls the degree and the rate of weathering. Residual soil is dominate the study area, this indicate that, the study area experiences *in situ* chemical weathering rather than mechanical weathering. There is also highly to slightly weatherd igmbrite rock observed along river cut of northern part of town.

5.5.2 Land Slide Distribution in the Study Area

Landslides occur when gravitational and other types of shear stress within a slope exceed the shear strength (resistance to shearing) of the materials that form the slope. Shear stresses can be built up within a slope by some processes. These include over steepening of the base of the slope, by natural erosion or excavation, and loading of the slope, such as by an inflow of water, a rise in the groundwater table.

5.5.2.1 Down Ward Sliding of Loose Earth Materials in the Study Area

In the study area, small scale soil slide observed specially along slop cut of north western and southern part of town and Huluka, Aleltu and Teltele river side basically loose earth material sliding were observed. This process can damages the road and close water ditch of town and the infrastuructures which are located on the top of the slope faces.

Figure 5.8 Soil sliding in the study area

5.5.2.2 Rock Falls Landslide in the Study Area

This type of landslide is formed due to detachment of rock material from steep slopes of quarry site in the town and these rock fall mostly observed to the southern and northern parts of the town (basaltic quarry sites) and to the western of the area on Senkele sandstone quary site. Based on the slope face angle of these quarry site the movement is free fall types of movements because the average face angle is above 75 degrees.

Figure 5.9 Rock slope falls sliding of the study area

5.5.3 Common Groundwater Problems in the Study Area

Ambo town is characterized by increasing of ground water level during rainy season, especially eastern parts (Ambo University Hachalu Campus area), north eastern (Bole condominium site and it's area) and some western parts of the town. In these area the leakage of ground water during rainy season is the common proplems and the water is flooding in the form of stream though the bouliding and different resident houses' foundations.

Figure 5.10 Water inflow in the form of stream through building foundation at Bole condominium site area (Ambo town) in Dr. Tadesse Ejeta Medium Clinic on July 17, 2023 (photos taken directily from the place).

5.5.4 Problematic Soils in the Study Area

Engineering characteristics of fine-grained soils (inorganic silt and clay); generally possess low shear strength, plastic and compressible, can lose part of shear strength upon wetting and disturbance, and generally poor material for backfill and embankments and the slopes with clay material are prone to landslides. Also as per Budhu, Fine-grained soils have poor load-bearing capacities compared with coarse-grained soils. Fine-grained soils are practically impermeable, change volume and strength with variations in moisture conditions, and are susceptible to frost. The engineering quality class of soil of town dominates with MH, CH SW and ML. The general engineering characters of this category of soil are low to high compressibility, good, and fair to poor for engineering works. For this reason sign of settlement was observed on the large building in the study area. In the eastern (Awaro University Hachalu Campus site and its' surrounding), northern (Aleltu river and chafe Abire Area) and north eastern (Bole condominium site and its' areas) of the town the dominant soil types are high plasticity silts and clayey soils which can affects the foundation of engineering structures. Therefore, the engineers have to take consideration during construct different engineering structures in these areas.

5.6 Engineering Geological Classification of Rocks of the Area

The engineering geological characterization and geotechnical properties of rock were determined using Schmidt hammer rebound test to estimate the unconfined compressive strength of intact rock in addition to describing the rocks of the study area by in-situ visual description. The description of rock involves a determination of the rock names, i.e., the lithological rock name, description of the properties of the rock material (texture, color, strength and state of weathering). Description of additional properties is necessary to describe the features of the rock mass (discontinuities, structure). The rocks of the study area were classified according to the international society of rock mechanics (ISRM, 1981) Based on their strength (unconfined compressive strength), a rock of the study area are classified into four major engineering geological subunits: very low strength rock units, low strength rock units, High Strength rock units and very high strength rock units. The engineering geological map of the town is produced by the description and classification of both soil and rock and the delineation of engineering geological units.

5.6.1 Very High Strength Rock Unit

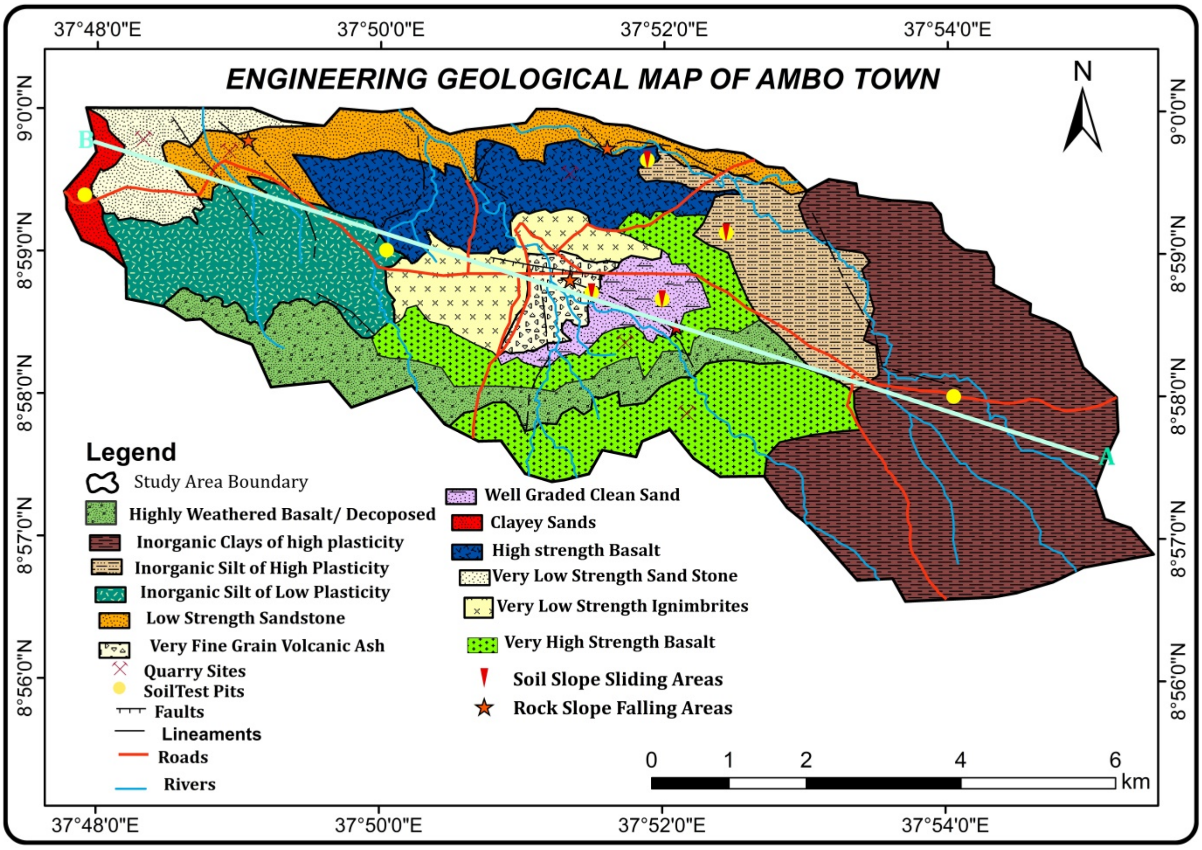
This engineering geological subunit is un-weathered basaltic rock unit which is covered to the south of the town, around Ambo Ifa Boru high school area. This rock unit is the widely used quarry site for different construction purpose in the town. This rock unit is separated in to rock slope section by highly weathered and much decomposed basaltic rock unit which is highly mixed with black cotton soil and used as aggregate or selected material for roads and building construction in the town.

5.6.2 Slightly Weathered High Strength Rock Unit

It is a basaltic rick unit which is located to the northern and northwestern parts of the town locally known by Aleltu/ Chafe Abire quarry site. Columnar joint is mostly formed on these units and it is the second largest basaltic quarry site which is widely used in the town for construction purpose and the upper slope face quarry site of this unit is residence houses therefore it can be affect the people who living the area.

5.6.3 Highly Weathered Very Low Strength Rock Units

Those rock units were named as highly weathered Ignimbrites and moderately weathered Whitish Sandstone. According to Deere and Miller (1966) rock strength specification the strength of those units were very low with the values of uniaxial compressive strength laboratory tests for Highly weathered Ignimbrites and moderately weathered Sandstone is 24.31 Mpa and 25.5 Mpa respectively. The Ignimbrite units exposed in the center of the town around and parts of Ambo hot spring/ Ethiopia Ambo Hotel and it is overly by fine grain volcanic ash unit or volcanic tuff. The whitish Sandstone unit is exposed to the western of the town near to Ambo Senkele Police College to the north of Ambo mineral water bottling and Ambo to Guder main roads and it is moderately weathered. This unit is overlying by slightly weathered low strength Sandstone which is the parts of rock units characterized in the study area. Most life of the local people is based on those rock quarry sites by manual/ non machinery mining of the two Sandstone units.



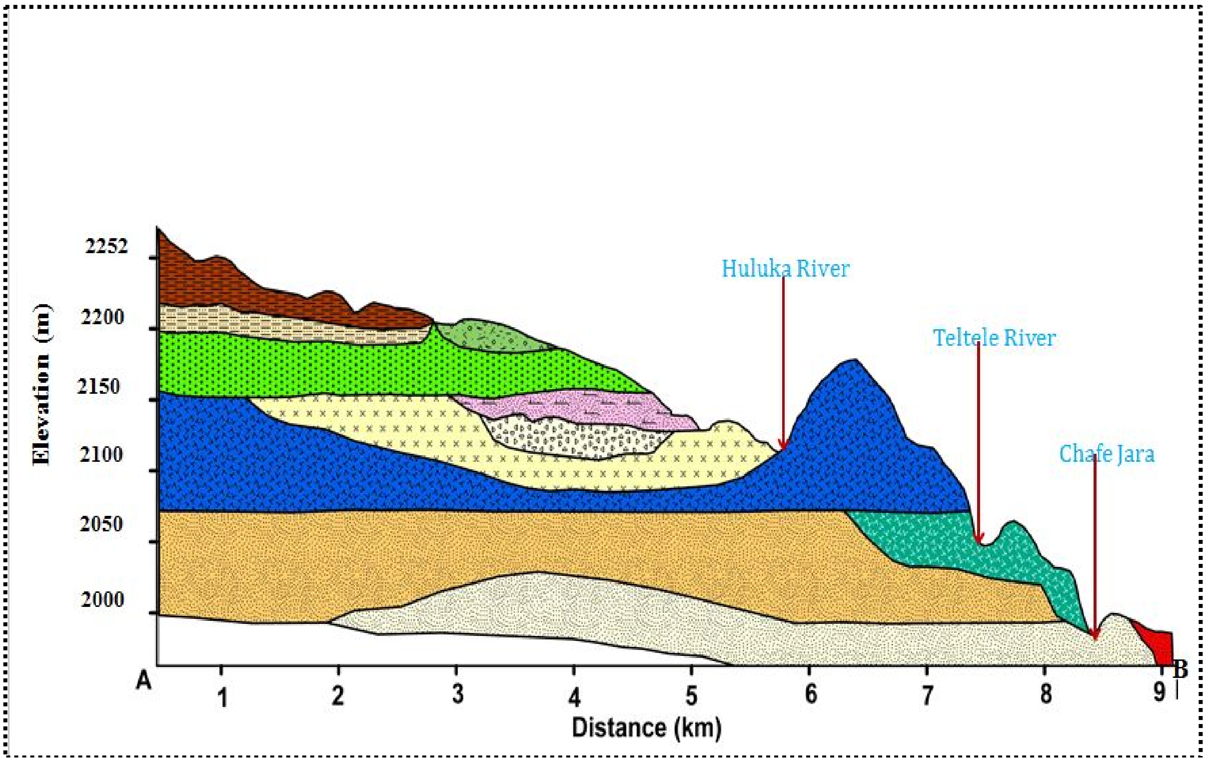


Figure 5.11 Engineering Geological Map and Cross section of the Study Area

CHAPTER SIX

6. CONCLUSIONS AND RECCOMONDETIONS

6.1 Conclusions

The main map able soil units in the study area are residual soils. From grain size analysis, the soils are dominantly fine grain with the content of gravel range from 0% to 2.98%, coarse sand 1.39% to 10.86%, fine sand 2.56% to 60.73% , silt 33% to 41% and clay 0.36% to 57.84%. The laboratory results of natural moisture content and specific gravity of the soil samples varies from 8.24% to 18.62% and 2.5g/cm3 to 2.68 g/cm3 respectively. The natural moisture contents indicating the water holding capacity of the soils is relatively low. The liquid limits in a percentage range from 24.06% to 86.81% for clay soils, 47.1% to 84.04% for silt soil and 45.97% to 60.53% for sand soil. The plasticity index of the soils also varies from 13.71% to 39.21% for clay soils, 13.65% to 40.24% for silt soils and 15.3% to 32.53% for sand soils. The values of the above parameter indicating soil of the study areas have low plasticity to high plasticity properties. Based on laboratory shear strength, optimum moisture contents and maximum dry density parameter tests, the values of cohesion, angle of friction, maximum dry density (MDD) and optimum moisture content (OMC) of a soil samples ranges from 64.97 kPa to 97.2 kPa, 6.46^o to 20.18^o, 1.2 g/cm^3 to 1.78 g/cm^3 and 15.72 g/cm^3 to 38.4 g/cm^3 respectively. According to the Unified Soil Classification System (USCS), the soils of the study area are classified as clayey (SC), high plastic silt (MH), low plasticity inorganic silt (ML), inorganic clay of high plasticity (CH) and well graded sands (SW) and some of these soil types which are characterized by high compressibility, good to poor shear strength and good and fair to poor in engineering workability. TP7A inorganic silt (ML) and TP3 inorganic clays of high plasticity (CH) with percentage of free swell 80% and 72.73%, respectively would have an adverse effect on light weight ordinary structures. Similarly, as per the AASHTO classification system, the soils are classified as clayey soils (A-7-5) and silt or clayey gravel (A-2-6 and A-2-7) which is favorable and unfavorable for subgrade material respectively. Based on the above soil classification system standards, high plasticity character of the soil were identify in southern, eastern and north eastern parts of the study area can pose damage to light weight structures, like pavement, residential houses, and these areas need special considerations during construction of engineering structures. The main lithological rock units exposed in the study area are highly weathered welded ignimbrite, sandstone unit, basalt and un-welded ignimbrites. According to Schmidt hammer field tests the basaltic unit shows variable strength ranging from 106 MPa to 107.3 MPa, sandstone unit from 55.45 MPa to 72.72 Mpa, welded ignimbrite 84.23 Mpa and un-welded ignimbrite 31.2 MPa. These lithological rock units are classified into four major engineering geological subunits based on unconfined compressive strength: very high strength unweathered basaltic rock unit, high strength slightly weathered basaltic rock unit, low strength slightly weathered sandstone unit, very low strength of moderately weathered sandstone and highly weathered ignimbrite units.

6.2 Recommendations

Based on the outcome and findings of the present study, the following recommendations were forwarded.

- ✓ In northeastern, eastern and southern most parts of the study area, potentially higher expansive soils were identified. Hence, care should be taken in constructing civil engineering structures on such types of soils.
- ✓ The properties of soil in the study area is changed with depth increasing therefore, detail subsurface investigation using geophysical methods is recommended for detailed understanding of the subsurface condition such as overburden thickness.
- ✓ The chemical study or chemistry of the soils recommended for further study.
- ✓ The town is surrounded by different rock quarry site. Therefore, their impact on the local community should be studied.

REFERENCES

AASHTO, (2014). Standard Specifications for Transportation Materials and Methods of Sampling and Testing and 2014 AASHTO Provisional Standards. *American Association of State Highway and Transportation Officials (AASHTO).*

Amanuel Ayalew1*, Veera narayana Balabathina2, Gebremedihin Birhane3, Leulalem Shano4, Tadese Demise4,. (2022). Engineering geological characterization of soils and rocks in Arba Minch town, Southern Ethiopia: *Implication for building cracks and road failures. International Journal for Modern Trends in Science and Technology (IJMTST).*

Arora, K. R. (1997). Soil Mechanics and Foundation Engineering. *Standard Publishers Distributers, Delhi, India.*

Arora, K.R. (2004). Soil Mechanics and Foundation Engineering. Standard publishers Distributors, New Delhi

ASTM D 4318 (1998). Standard Test Method for Liquid Limit, Plastic Limit and Plasticity

ASTM D 3080, (2002). Standard test method for direct shear test of soils under consolidated drained conditions (D3080-98). Annual Book of Standards, American Society for Testing and Materials, West Conshohocken, PA, Vol. 04.08, pp.339-344.

Azizi, A., & Moomivand, H. (2021). A New Approach to Represent Impact of Discontinuity Spacing and Rock Mass Description on the Median Fragment Size of Blasted Rocks Using Image Analysis of Rock Mass. *Rock Mechanics and Rock Engineering*, *54*(4), 2013–2038. https://doi.org/10.1007/s00603-020-02360-4

Barbosa, V. H. R., Marques, M. E. S., & Guimarães, A. C. R. (2023). Predicting Soil Swelling Potential Using Soil Classification Properties. *Geotechnical and Geological Engineering*. https://doi.org/10.1007/s10706-023-02525-2.

Barnes, G. E. (2021). A Review of the Plastic Limit Test by Means of Rolling Paths. *Geotechnical Testing Journal*, *44*(6), 20210059. https://doi.org/10.1520/gtj20210059.

Barton, N., Lien, R., Lunde, J., Barton, L., and Lunde, (1974). Barton Lien and Lunde 1974. Engineering classification of rock masses for. In Rock Mechanics (Vol. 6, Issue 106, pp. 189–236).

Barton, N.R. (1976). The shear strength of rock and rock joints. Int. J. Mech. Min. Sci. & Geomech. Abstr. 13(10), 1-24.

Bell, F.G. (2007). Engineering Geology, (2nd Ed.). Butterworth-Heinemann (Elsevier)

Burlington, MA. pp. 581.

Bieniawski, Z.T. (1979). The geomechanics classification in rock engineering applications. In: Proceedings of the 4th international congress on rock mechanics, Montreux, 1979, pp. 41 48.

Braja M.Das (2008). Advanced Soil Mechanics.*3rd ed., Standard Publisher Distributers, London.* ISBN 0-203-93584-5 Master e-book ISBN.

Bowles, J.E. (1996). Engineering Properties of Soils and Their measurement. *4thed.Mcs Hill Book Company, New York.*

Budhu, M. (2010), Soil Mechanics and Foundations, *3rd Edition, John Wiley & Sons, Inc., New Jersey, U.S.A.*

Cetin H., Fener M., Söylemez, M., & Günaydin, O. (2007) Soil structure changes during compaction of a cohesive soil. *Engineering Geology*, *92*(1–2), 38–48. https://doi.org/10.1016/j.enggeo.2007.03.005.

Crow, M. J., & Barber, A. J. (2005), Map: Simplified geological map of Sumatra. *Geological Society London Memoirs 31*(1) https://doi.org/10.1144/gsl.mem.2005.031.01.17.

Daryati, D., Widiasanti, I., Septiandini, E., Ramadhan, M. A., Sambowo, K. A., & Purnomo, A. (2019). Soil characteristics analysis based on the unified soil classification system. *Journal of Physics: Conference Series*, *1402*(2), 022028. https://doi.org/10.1088/1742-6596/1402/2/022028.

David Suits, L., Sheahan, T., Prakash, K., & Sridharan, A. (2004). Free Swell Ratio and Clay Mineralogy of Fine-Grained Soils. *Geotechnical Testing Journal*, *27*(2), 10860. https://doi.org/10.1520/gtj10860.

Deere U., & Deere D. (1988). The rock quality designation (RQD) index in practice. In: Kirakaldie L (ed) Rock Classification systems for engineering purposes. ASTM special publication 984. American Society for Testing Materials, Philadelphia, pp 91–101.

Discenza, M. E., Pari, P. D., Kundu, J., Maria Carmela, M., Romano, S., & Scarascia Mugnozza, G. (2021). Innovative Approach for the Determination of Unit Weight and Density of Soil and Rock Masses. *SSRN Electronic Journal*. https://doi.org/10.2139/ssrn.4054512.

El Hariri, A., Elawad Eltayeb Ahmed, A., & Kiss, P. (2023). Review on soil shear strength with loam sand soil results using direct shear test. *Journal of Terramechanics*, *107*, 47–59. https://doi.org/10.1016/j.jterra.2023.03.003.

Ermias, Birhanu, Raghuvanshi, T. K., & Abebe, Bekele. (2017). Landslide Hazard zonation (LHZ) around Alem ketema town, north Showa zone, Central Ethiopia-a GIS based expert evaluation approach. Int J Earth Sci Eng, 10(1), 33-44.

F.E., O., & A.S., U. (2021). Soil mapping, classification and morphological characteristics of the University of Benin land. *Nigerian Journal of Soil Science*, 32–40. https://doi.org/10.36265/njss.2021.310205.

Furukawa, Y., Fujita, T., Kunihiro, T., Tsuchiya, H., & Saito, Y. (2000). Investigation of Particle Size Distribution for Fine-Grained Soil Using Grain Size Analysis Equipment Automated by X-ray Method. *Soils and Foundations*, *40*(2), 127–133. https://doi.org/10.3208/sandf.40.2_127.

Goodman, R. E. (1989). Introduction to rock mechanics (Vol. 2). New York: Wiley.

EBCS (1995). Design of structures for earthquake resistance.

Haigh, S. (2015). Consistency of the Casagrande Liquid Limit Test. *Geotechnical Testing Journal*, *39*(1), 20150093. https://doi.org/10.1520/gtj20150093.

Hashemi, M., Moghaddas, S., & Ajalloeian, R. (2017). Erratum to: Application of Rock Mass Characterization for Determining the Mechanical Properties of Rock Mass: a Comparative Study. *Rock Mechanics and Rock Engineering*, *50*(5), 1367–1369. https://doi.org/10.1007/s00603-017-1199-x.

Heriyadi, B., Prengki, I., & Prabowo, H. (2019). Analysis of Collapse Load and Open Hole Evaluation Based on Rock Mass Ratting (RMR) Method in Underground Mining. *Journal of Physics: Conference Series*, *1387*(1), 012104. https://doi.org/10.1088/1742-6596/1387/1/012104.

Hoek, E. and Bray, J.W. (1981) Rock Slope Engineering. Revised 3rd Edition, the Institute of Mining and Metallurgy, London, 341-351.

ISRM, (1978). International Society of Rock Mechanics. Suggested Methods for Determining Mining Sciences, Geomechanics Abstract, 15(3), pp. 99-103. 27.

Izdebska-mucha, d., & Wójcik, e. (2016). Identification of clay soil mineralogy and swelling by the free swell ratio method. *Biuletyn Państwowego Institute Geologicznego*, *0*, 77–86. https://doi.org/10.5604/01.3001.0009.4756.

Jamshidi, A. (2022). A Comparative Study of Point Load Index Test Procedures in Predicting the Uniaxial Compressive Strength of Sandstones. *Rock Mechanics and Rock Engineering*, *55*(7), 4507–4516. https://doi.org/10.1007/s00603-022-02877-w.

Kahraman, S. (2014). The determination of uniaxial compressive strength from point load strength for pyroclastic rocks. *Engineering Geology*, *170*, 33–42. https://doi.org/10.1016/j.enggeo.2013.12.009.

Konstantopoulou, G., & Vacondios, I. (2007). Engineering geological map of the urban area of Kastoria, NW Greece. *Bulletin of the Geological Society of Greece*, *40*(4), 1664. https://doi.org/10.12681/bgsg.17070.

Krishna, R. (2002). Engineering properties of soils based on laboratory testing. UIC, 44,59.

Lemenkova, P. (2022). Seismicity in the Afar Depression and Great Rift Valley, Ethiopia. *Environmental Research, Engineering and Management*, *78*(1), 83–96. https://doi.org/10.5755/j01.erem.78.1.29963.

Madhubabu, N., Singh, P., Akintola, A., Mahanta, B., Tripathy, A., & Singh, T. (2016). Prediction of compressive strength and elastic modulus of carbonate rocks. *Measurement*, *88*, 202–213. https://doi.org/10.1016/j.measurement.2016.03.050.

Malka, A. (2022). GIS-Based Landslide Susceptibility Modelling in Urbanized Areas: A Case Study of the Tri-City Area of Poland. *Geohazards*, *3*(4), 508–528. https://doi.org/10.3390/geohazards3040026.

Maharaj, R. J. (1993). Use of AASHTO classification to evaluate soils for road construction in Jamaica, West Indies. *Quarterly Journal of Engineering Geology*, *26*(1), 61–67. https://doi.org/10.1144/gsl.qjeg.1993.026.01.05.

Map: Geological Map of the Sant' Arcangelo Basin (Basilicata, Italy). (2006). *Geological Society, London, Special Publications*, *262*(1). https://doi.org/10.1144/gsl.sp.2006.262.01.23.

Mekdes Habtamu, Tola Garo, Yadeta C. Chemeda. (2022). Engineering Geological Investigation of Jimma Town for Engineering Practice. Ethiopian Journal of Science and Sustainable Development (EJSSD), 9(1)(e-ISSN 2663-3205), 1-15.

Mengesha Tefera, T. C., & Haro, W. (1996). Explanation of the geological map of Ethiopia. Ministry of Mines and Energy, EIGS. Bull, (3).

Murthy, V. N. S., (1990), Geotechnical Engineering: Principles and Practices of Soil Mechanics and Foundation Engineering, Marcel Dekker, Inc., New York.

Palmström A. (1982). The volumetric joint count - A useful and simple measure of the degree of rock mass jointing. In: Proceedings of 4th congress IAEG, Delhi, vol 5, pp 221 -228.

Palmström, A. (2005). Measurements of and correlations between block size and rock quality designation (RQD). Tunneling and Underground Space Technology, 20(4), 362–377.

Poulos, S. J. (1989). Jansen, RB, ed., “Liquefaction Related Phenomena”, Advance Dam Engineering for Design.

Pozo, R. (2022). Equivalent geological strength index (GSI) approach with application to rock mass Slope stability. *Rudarsko-Geološko-Naftni Zbornik*, *37*(4), 53–70. https://doi.org/10.17794/rgn.2022.4.5.

Price, D.G. (2009). Engineering geology principles and practice. Springer-Verlag, Berlin Heidelberg, p. 450

Prakash, S., & Jani, P. K. (2002). Engineering Soil Testing Nem Chand & Bros: Roorkee.

Ramamurthy, TN, & Sitharam, TG, (2005*). Geotechnical engineering S. Chand and Company Limited, Ram Nagar, New Delhi*.

Raj, P. P. (2012). Soil Mechanics and Foundation Engineering, Dorling Kindersley (India) Pvt. *Ltd., New Delhi*.

Raghuvanshi, T.K, (1995)." Engineering Geological Appraisal of Kishau Dam Project, Garhwal Himalaya, India, Unpublished Ph.D thesis, University of Roorkee, India.

Raghuvanshi, T. K. (2019). Plane failure in rock slopes – A review on stability analysis techniques. Journal of King Saud University - Science, 31(1), 101–109. https://doi.org/10.1016/j.jksus.2017.06.004.

Rawat*, V. & P, M. (2019). Efficiency of Shear Wall Location on Reinforced Concrete Buildings in Ethiopia under Seismic Excitation. International Journal of Recent Technology and Engineering (IJRTE), *8*(4), 10624-10631. Doi: 10.35940/ijrte.d4252.118419.

Refky Adi Nata, and Murad MS . (2017). Mining Engineering Department. STTIND Padang 25171, *Indonesia, Mining Engineering Department, Faculty of Engineering, UNP*.

Richards L, Atherton D. (1987). Stability of slopes in rock. In Bell FG (Ed). Ground Engineers Reference Book. Butterworths, London.

Romana, M. (1985). New adjustment ratings for application of Bieniawski classification to slope. In: Int. Sym. Role Rock Mech., Zacatecas, pp. 49–53.

Santi, P. M. (1998). Engineering Properties of Weak Soils at Foundations of Buildings. *Environmental & Engineering Geoscience*, *IV* (1), 142–143. https://doi.org/10.2113/gseegeosci.iv.1.142.

Seifu, Y., Tola, S., Wako, A., & Jayaprakash, R. (2023). Study of soil texture and moisture content effect on soil compaction for long-year tilled farm land. *Engineering and Technology Journal*, *08*(01). https://doi.org/10.47191/etj/v8i1.06.

Sharma, S., Raghuvanshi, T. K., & Anbalagan, R. (1995). Plane failure analysis of rock Slopes. Geotechnical & Geological Engineering, 13(2), 105-111.

Sharma, S., Raghuvanshi, T., & Sahai, A. (1999). An engineering geological appraisal of the Lakh war dam, Garhwali Himalaya, India. Engineering geology, 53(3-4), 381.

Stability Analysis of Reinforced Slope Based on Limit Equilibrium Method. (2018). *Tehnicki Vjesnik - Technical Gazette*, *25*(1). https://doi.org/10.17559/tv-20170317114554.

Ulusay, R., & Karakul, H. (2015). Assessment of basic friction angles of various rock types from Turkey under dry, wet and submerged conditions and some considerations on tilt testing. Bulletin of Engineering Geology and the Environment, 75(4), 1683-1699. Doi: 10.1007/s10064-015-0828-4.

Veloo, R. (2015). Peat Classification and History of Organic Soils Classification in Malaysia. *The Planter*, *91*(1073). https://doi.org/10.56333/tp.2015.008.

Wijk, G. (1980). The Uniaxial Strength of Rock Material. *Geotechnical Testing Journal*, *3*(3), 115. https://doi.org/10.1520/gtj10882j.

Wyllie, D. C., & Mah, C. W. (2004). Rock Slope Engineering 4thEd. The Institution of Mining and Metallurgi London.

Xiao, S., & Chen, T. (2021). Improved general slice method of limit equilibrium for slope stability analysis. *Act Geotechnical Slovenica*, *18*(1), 55–64. https://doi.org/10.18690/actageotechslov.18.1.55-64.2021.

Yadeta. C. (2020). Engineering Geological Investigation of Adama Town: Implication to Engineering Practice.

Zerradi, mine in Morocco. *Mining of Mineral Deposits*, *17*(2), 53–60. https://doi.org/10.33271/mining17.02.053.

Zhai, H., Masoumi, H., Zoorabadi, M., & Canbulat, I. (2020). Size-dependent Behaviour of Weak Intact Rocks. *Rock Mechanics and Rock Engineering*, *53*(8), 3563–3587. https://doi.org/10.1007/s00603-020-02117-z.

Zhang, L. (2016). Determination and applications of rock quality designation (RQD). *Journal of Rock Mechanics and Geotechnical Engineering*, *8*(3), 389–397. https://doi.org/10.1016/j.jrmge.2015.11.008.

LIST OF APPENDIXES

Appendix A: Unit Weight Test Result of Soil Samples (TP1-TP7B)

Date Tested :	**27/5/2023**				
Tasted By:	**Lencho Lemma**				
Project Name:	**MSc Thesis**		**Standard Reference: ASTM D2937-0**		
Sample Description:	**TP1**		**Sample Description:**	**TP2**	
Trial No	**1**	**2**	**Trial No**	**1**	**2**
Mold weight +sample (gm)	2337.2	2337	Mold weight +sample (gm)	2202.6	2203.4
Mold weight (gm)	1933.4	1933.4	Mold weight (gm)	1933.4	1933.4
Sample weight (gm)	403.8	403.6	Sample weight (gm)	269.2	270
Mold volume (Cm3)	196.25	196.25	Mold volume (Cm3)	196.25	196.25
Bulk density(g/cm3)	2.06	2.06	Bulk density(g/cm3)	1.37	1.38
Average bulk density	2.06		Average bulk density	1.37	
Wet unit weight(KN/m3)	20.18		Wet unit weight(KN/m3)	13.48	
Moisture content (%)	10.04		Moisture content (%)	26.8	
Dry density (g/cm3)	1.8694		Dry density (g/cm3)	1.2077	
Dry Unit weight(KN/m3)	18.34		Dry Unit weight(KN/m3)	11.85	
Sample Description:	**TP5**		**Sample Description:**	**TP6**	
Trial No	**1**	**2**	**Trial No**	**1**	**2**
Mold weight +sample (gm)	2223.5	2223.9	Mold weight +sample (gm)	2214.7	2216
Mold weight (gm)	1933.4	1933.4	Mold weight (gm)	1933.4	1933.4
Sample weight (gm)	290.1	290.5	Sample weight (gm)	281.3	282.6
Mold volume (Cm3)	196.25	196.25	Mold volume (Cm3)	196.25	196.25
Bulk density(g/cm3)	1.48	1.48	Bulk density(g/cm3)	1.43	1.44
Average bulk density	1.48		Average bulk density	1.44	
Wet unit weight(KN/m3)	14.51		Wet unit weight(KN/m3)	14.09	
Moisture content (%)	23.1		Moisture content (%)	26.3	
Dry density (g/cm3)	1.3276		Dry density (g/cm3)	1.2687	
Dry Unit weight(KN/m3)	13.02		Dry Unit weight(KN/m3)	12.45	

Sample Description:	TP3		Sample Description:	TP4	
Trial No	**1**	**2**	**Trial No**	**1**	**2**
Mold weight +sample (gm)	2200.9	2203.9	Mold weight +sample (gm)	2213.1	2214.2
Mold weight (gm)	1933.4	1933.4	Mold weight (gm)	1933.4	1933.4
Sample weight (gm)	267.5	270.5	Sample weight (gm)	279.7	280.8
Mold volume (Cm3)	196.25	196.25	Mold volume (Cm3)	196.25	196.25
Bulk density(g/cm3)	1.36	1.38	Bulk density(g/cm3)	1.43	1.43
Average bulk density	1.37		Average bulk density	1.43	
Wet unit weight(KN/m3)	13.45		Wet unit weight(KN/m3)	14.01	
Moisture content (%)	38.4		Moisture content (%)	34.9	
Dry density (g/cm3)	1.1555		Dry density (g/cm3)	1.2300	
Dry Unit weight(KN/m3)	11.34		Dry Unit weight(KN/m3)	12.07	

Sample Description:	TP7A		Sample Description:	TP7B	
Trial No	**1**	**2**	**Trial No**	**1**	**2**
Mold weight +sample (gm)	2213.5	2214	Mold weight +sample (gm)	2190.1	2190.1
Mold weight (gm)	1933.4	1933.4	Mold weight (gm)	1933.4	1933.4
Sample weight (gm)	280.1	280.6	Sample weight (gm)	256.7	256.7
Mold volume (Cm3)	196.25	196.25	Mold volume (Cm3)	196.25	196.25
Bulk density(g/cm3)	1.43	1.43	Bulk density(g/cm3)	1.31	1.31
Average bulk density	1.43		Average bulk density	1.31	
Wet unit weight(KN/m3)	14.01		Wet unit weight(KN/m3)	12.83	
Moisture content (%)	36.5		Moisture content (%)	30.3	
Dry density (g/cm3)	1.2495		Dry density (g/cm3)	1.1492	
Dry Unit weight(KN/m3)	12.26		Dry Unit weight(KN/m3)	11.27	

Appendix-B Natural Moisture Content Test Results for (TP1-TP7B)

Date Tasted:	**May 25 2023**
Tested By:	**Lencho Lemma**
Project Name:	**MSc Thesis**
Sample Description:	**TP1**

Standard Reference: ASTM D 2216		
Sample Number:	**TP1**	
Moisture can and lid number	**A18**	**D-1**
MC = Mass of empty, clean can + lid (grams)	24.47	24.1
MCMS = Mass of can, lid, and moist soil (grams)	94	87
MCDS = Mass of can, lid, and dry soil (grams)	88.61	82.3
MS = Mass of soil solids (grams)	64.14	58.2
MW = Mass of pore water (grams)	5.39	4.7
w = Water content, w%	8.40349	8.0756
Average W (%)		**8.24**
Sample Number:	**TP5**	
Moisture can and lid number	**D-3**	**D-4**
MC = Mass of empty, clean can + lid (grams)	24.51	24.4
MCMS = Mass of can, lid, and moist soil (grams)	82.03	76.3
MCDS = Mass of can, lid, and dry soil (grams)	76	71.1
MS = Mass of soil solids (grams)	51.49	46.7
MW = Mass of pore water (grams)	6.03	5.2
w = Water content, w%	11.711	11.1349
Average W (%)	**11.42**	
Sample Number:	**TP2**	
Moisture can and lid number	**C-11**	**B-3**
MC = Mass of empty, clean can + lid (grams)	24.09	24.5
MCMS = Mass of can, lid, and moist soil (grams)	70.74	80.2
MCDS = Mass of can, lid, and dry soil (grams)	65.07	73.5
MS = Mass of soil solids (grams)	40.98	49
MW = Mass of pore water (grams)	5.67	6.7
w = Water content, w%	13.836	13.6735
Average W%		**13.75**

Sample Number:	TP3	
Moisture can and lid number	**A-3**	**Y-6**
MC = Mass of empty, clean can + lid (grams)	24.09	24.06
MCMS = Mass of can, lid, and moist soil (grams)	83.05	86.1
MCDS = Mass of can, lid, and dry soil (grams)	73.95	76.2
MS = Mass of soil solids (grams)	49.86	52.14
MW = Mass of pore water (grams)	9.101	9.9
w = Water content, w%	18.2535	18.9873
Average W (%)	**18.62**	
Sample Number:	**TP4**	
Moisture can and lid number	**E-27**	**E-20**
MC = Mass of empty, clean can + lid (grams)	25.83	24.5
MCMS = Mass of can, lid, and moist soil (grams)	77.64	66.9
MCDS = Mass of can, lid, and dry soil (grams)	70.48	61
MS = Mass of soil solids (grams)	44.65	36.5
MW = Mass of pore water (grams)	7.16	5.9
w = Water content, w%	16.0358	16.1644
Average W%		**16.10**
Sample Number:	**TP6**	
Moisture can and lid number	**A21**	**D-6**
MC = Mass of empty, clean can + lid (grams)	23.54	24
MCMS = Mass of can, lid, and moist soil (grams)	83.41	73.6
MCDS = Mass of can, lid, and dry soil (grams)	76.53	67.7
MS = Mass of soil solids (grams)	52.99	43.7
MW = Mass of pore water (grams)	6.88	5.9
w = Water content, w%	12.9836	13.5011
Average W%	**13.24**	
Sample Number:	**TP7A**	
Moisture can and lid number	**A-26**	**D-7**
MC = Mass of empty, clean can + lid (grams)	24.35	24.7
MCMS = Mass of can, lid, and moist soil (grams)	92.51	91.2
MCDS = Mass of can, lid, and dry soil (grams)	84.03	82.8
MS = Mass of soil solids (grams)	59.68	58.1
MW = Mass of pore water (grams)	8.48	8.4
w = Water content, w%	14.2091	14.4578
Average W (%)		**14.33**
Sample Number:	**TP7B**	
Moisture can and lid number	**Z-2**	**B-5**
MC = Mass of empty, clean can + lid (grams)	24.11	24.22
MCMS = Mass of can, lid, and moist soil (grams)	85.81	83.45
MCDS = Mass of can, lid, and dry soil (grams)	78.24	76.33
MS = Mass of soil solids (grams)	54.13	52.11
MW = Mass of pore water (grams)	7.57	7.12
w = Water content, w%	13.9849	13.6634
Average W%		**13.82**

Appendix-C Compaction Test Results With Moisture Content Vs Dry Density Graphs For Soil Sample (TP1-TP7B)

Date Tasted :	26-05-2023
Tested By:	Lencho Lemma
Project Name:	MSc Thesis
Sample Description:	Soil from

Standard Reference: ASTM D 698

Sample No.	**TP1**				
Moisture content Vs dry density computation table					
Determination No.	1	2	3	4	5
Mass of Mold, g	4321	4321	4321	4321	4321
Mass of mold + Compacted Soil, g	5700.3	6063	6263.2	6177.7	5903.2
Mass of Compacted soil, g	1379.3	1742	1942.2	1856.7	1582.2
Volume of Mold, cm3	943	944	944	944	944
Bulk density, g/cm3	1.46	1.85	2.06	1.97	1.68
Water Content, %	5.31	10.54	15.72	19.34	25.72
Dry density, g/cm3	**1.39**	**1.67**	**1.78**	**1.65**	**1.33**
TR	**1**	**2**	**3**	**4**	**4**
Container No	D-8	A-7	C2	A-5	C-5
Mass of container, g	24.3	24	24.3	24.3	24
Mass of container + wet soil, g	71.9	75.4	91.3	92.8	71.9
Mass of container + Dry soil, g	69.5	70.5	82.2	81.7	62.1
Mass of Water, g	2.4	4.9	9.1	11.1	9.8
Mass of Dry soil, g	45.2	46.5	57.9	57.4	38.1
Water content, %	**5.31**	**10.54**	**15.72**	**19.34**	**25.72**

Standard Reference: ASTM D 698

Sample No.	**TP2**				
Moisture content Vs dry density computation table					
Determination No.	**1**	**2**	**3**	**4**	**5**
Mass of Mold, g	4321.1	4321.1	4321.1	4321.1	4321.1
Mass of mold + Compacted Soil, g	5450.8	5641.6	5854.6	5925	5802.4
Mass of Compacted soil, g	1129.7	1320.5	1533.5	1603.9	1481.3
Volume of Mold, cm3	943	944	944	944	944
Bulk density, g/cm3	1.20	1.40	1.62	1.70	1.57
Water Content, %	7.75	18.38	26.79	40.27	47.60
Dry density, g/cm3	**1.11**	**1.18**	**1.28**	**1.21**	**1.06**
TR	**1**	**2**	**3**	**4**	**5**
Container No	A-10	LL-3	W-2	E-27	A-8
Mass of container, g	24.1	23.9	24.1	24.9	24
Mass of container + wet soil, g	70	79.3	73.8	77.5	67.1
Mass of container + Dry soil, g	66.7	70.7	63.3	62.4	53.2
Mass of Water, g	3.3	8.6	10.5	15.1	13.9
Mass of Dry soil, g	42.6	46.8	39.2	37.5	29.2
Water content, %	**7.75**	**18.38**	**26.79**	**40.27**	**47.60**

Standard Reference: ASTM D 698

Sample No.	TP3				
Moisture content Vs dry density computation table					
Determination No.	**1**	**2**	**3**	**4**	**5**
Mass of Mold, g	4321.1	4321.1	4321.1	4321.1	4321.1
Mass of mold + Compacted Soil, g	5521.9	5716.7	5886.8	5865.5	5692.8
Mass of Compacted soil, g	1200.8	1395.6	1565.7	1544.4	1371.7
Volume of Mold, cm^3	944	944	944	944	944
Bulk density, g/cm^3	1.27	1.48	1.66	1.64	1.45
Water Content, %	15.38	28.53	38.40	49.12	58.29
Dry density, g/cm^3	**1.10**	**1.15**	**1.20**	**1.10**	**0.92**
TR	**1**	**2**	**3**	**4**	**5**
Container No	C-6	C-1	Z-1	A-8	B-6
Mass of container, g	24	23.9	24.1	24	24.7
Mass of container + wet soil, g	69	71.2	79.6	74.7	82
Mass of container + Dry soil, g	63	60.7	64.2	58	60.9
Mass of Water, g	6	10.5	15.4	16.7	21.1
Mass of Dry soil, g	39	36.8	40.1	34	36.2
Water content, %	**15.38**	**28.53**	**38.40**	**49.12**	**58.29**

Standard Reference: ASTM D 698

Sample No.	TP4				
Moisture content Vs dry density computation table					
Determination No.	**1**	**2**	**3**	**4**	**5**
Mass of Mold, g	4321.1	4321.1	4321.1	4321.1	4321.1
Mass of mold + Compacted Soil, g	5530	5797.6	5985.7	5863.9	5545
Mass of Compacted soil, g	1208.9	1476.5	1664.6	1542.8	1223.9
Volume of Mold, cm3	944	944	944	944	943
Bulk density, g/cm3	1.28	1.56	1.76	1.63	1.30
Water Content, %	15.33	24.44	34.89	42.60	50.16
Dry density, g/cm3	**1.11**	**1.26**	**1.31**	**1.15**	**0.86**
TR	**2**	**3**	**4**	**4**	**1**
Container No	A-2	E-22	A-1	A-3	D-7
Mass of container, g	24.3	23.9	24.2	24.1	24.3
Mass of container + wet soil, g	86	74.3	84.9	72.3	71.9
Mass of container + Dry soil, g	77.8	64.4	69.2	57.9	56
Mass of Water, g	8.2	9.9	15.7	14.4	15.9
Mass of Dry soil, g	53.5	40.5	45	33.8	31.7
Water content, %	**15.33**	**24.44**	**34.89**	**42.60**	**50.16**

Standard Reference: ASTM D 698

Sample No.	TP5				
Moisture content Vs dry density computation table					
Determination No.	**1**	**2**	**3**	**4**	**5**
Mass of Mold, g	4321.1	4321.1	4321.1	4321.1	4321.1
Mass of mold + Compacted Soil, g	5509.7	5668.4	6018.8	6010.1	5860.2
Mass of Compacted soil, g	1188.6	1347.3	1697.7	1689	1539.1
Volume of Mold, cm3	943	944	944	944	944
Bulk density, g/cm3	1.26	1.43	1.80	1.79	1.63
Water Content, %	7.52	12.89	23.09	32.68	42.24
Dry density, g/cm3	**1.17**	**1.26**	**1.46**	**1.35**	**1.15**
TR	**1**	**2**	**3**	**4**	**4**
Container No	A-7	B1-6	A-3	Y-2	C-3
Mass of container, g	24.1	23.8	24.1	24.2	24.1
Mass of container + wet soil, g	67	92.1	85.4	92	80
Mass of container + Dry soil, g	64	84.3	73.9	75.3	63.4
Mass of Water, g	3	7.8	11.5	16.7	16.6
Mass of Dry soil, g	39.9	60.5	49.8	51.1	39.3
Water content, %	**7.52**	**12.89**	**23.09**	**32.68**	**42.24**

Standard Reference: ASTM D 698

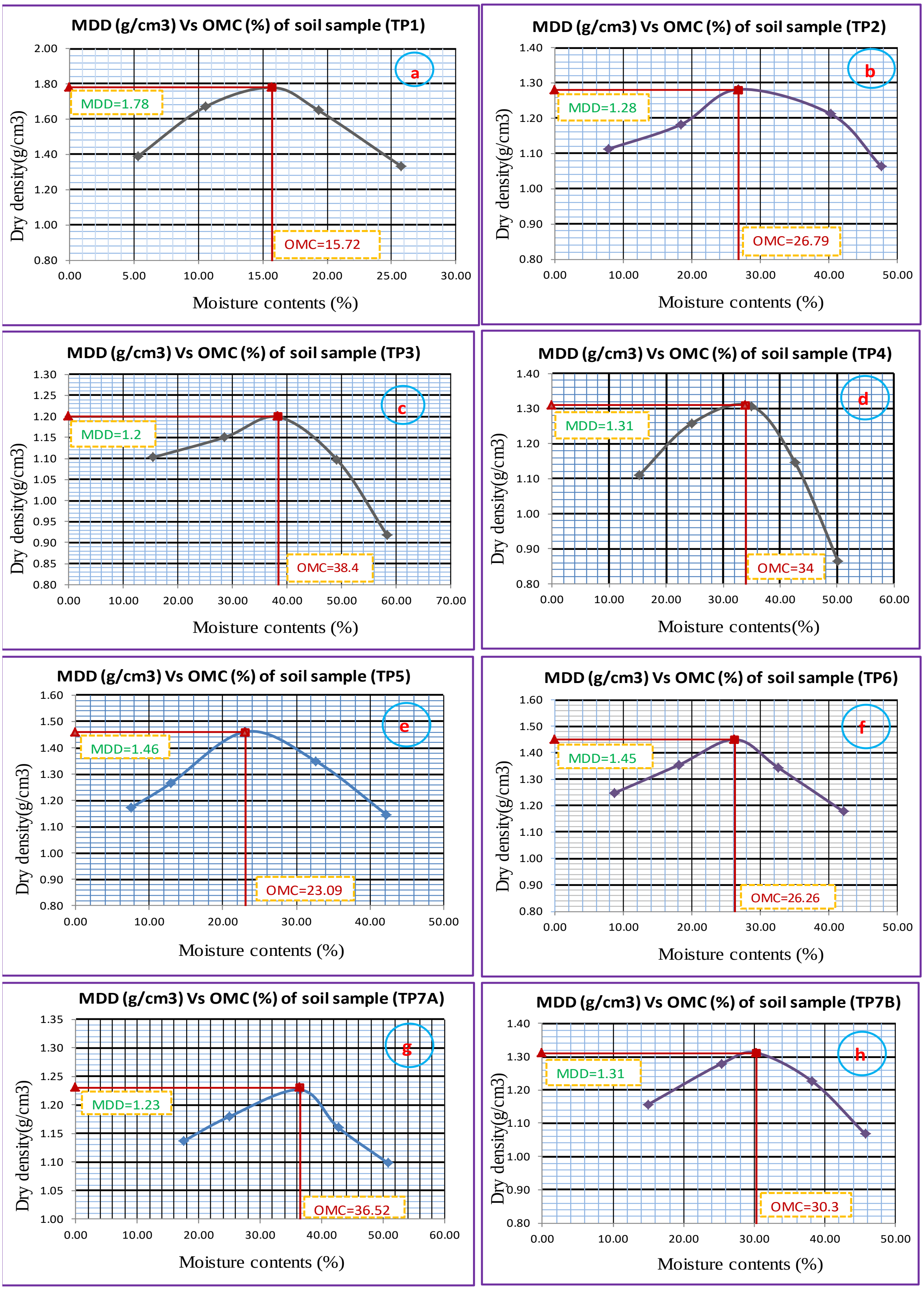
Sample No.	TP6				
Moisture content Vs dry density computation table					
Determination No.	**1**	**2**	**3**	**4**	**5**
Mass of Mold, g	4321.1	4321.1	4321.1	4321.1	4321.1
Mass of mold + Compacted Soil, g	5600.2	5830.4	6048.9	6002.2	5900.1
Mass of Compacted soil, g	1279.1	1509.3	1727.8	1681.1	1579
Volume of Mold, cm3	943	944	944	944	944
Bulk density, g/cm3	1.36	1.60	1.83	1.78	1.67
Water Content, %	8.71	18.10	26.26	32.61	42.12
Dry density, g/cm3	**1.25**	**1.35**	**1.45**	**1.34**	**1.18**
TR	**1**	**2**	**3**	**4**	**5**
Container No	A-13	E-11	E-23	C-3	A-9
Mass of container, g	23.8	23.9	24	24	24.2
Mass of container + wet soil, g	60	91.1	79.3	97.2	68.4
Mass of container + Dry soil, g	57.1	80.8	67.8	79.2	55.3
Mass of Water, g	2.9	10.3	11.5	18	13.1
Mass of Dry soil, g	33.3	56.9	43.8	55.2	31.1
Water content, %	**8.71**	**18.10**	**26.26**	**32.61**	**42.12**

Standard Reference: ASTM D 698

Sample No.	TP7A				
Moisture content Vs dry density computation table					
Determination No.	**1**	**2**	**3**	**4**	**5**
Mass of Mold, g	4321.1	4321.1	4321.1	4321.1	4321.1
Mass of mold + Compacted Soil, g	5580.4	5711.6	5901.6	5884.4	5884.4
Mass of Compacted soil, g	1259.3	1390.5	1580.5	1563.3	1563.3
Volume of Mold, cm3	943	944	944	944	944
Bulk density, g/cm3	1.34	1.47	1.67	1.66	1.66
Water Content, %	17.51	24.92	36.52	42.69	50.76
Dry density, g/cm3	**1.14**	**1.18**	**1.23**	**1.16**	**1.10**
TR	**1**	**2**	**3**	**4**	**5**
Container No	A-9	C-2	Z-1	C-4	A-8
Mass of container, g	23.8	24.3	24.1	24.2	24.1
Mass of container + wet soil, g	68.1	63.9	79.8	83.6	74
Mass of container + Dry soil, g	61.5	56	64.9	65.8	57.2
Mass of Water, g	6.6	7.9	14.9	17.8	16.8
Mass of Dry soil, g	37.7	31.7	40.8	41.6	33.1
Water content, %	**17.51**	**24.92**	**36.52**	**42.79**	**50.76**

Standard Reference: ASTM D 698

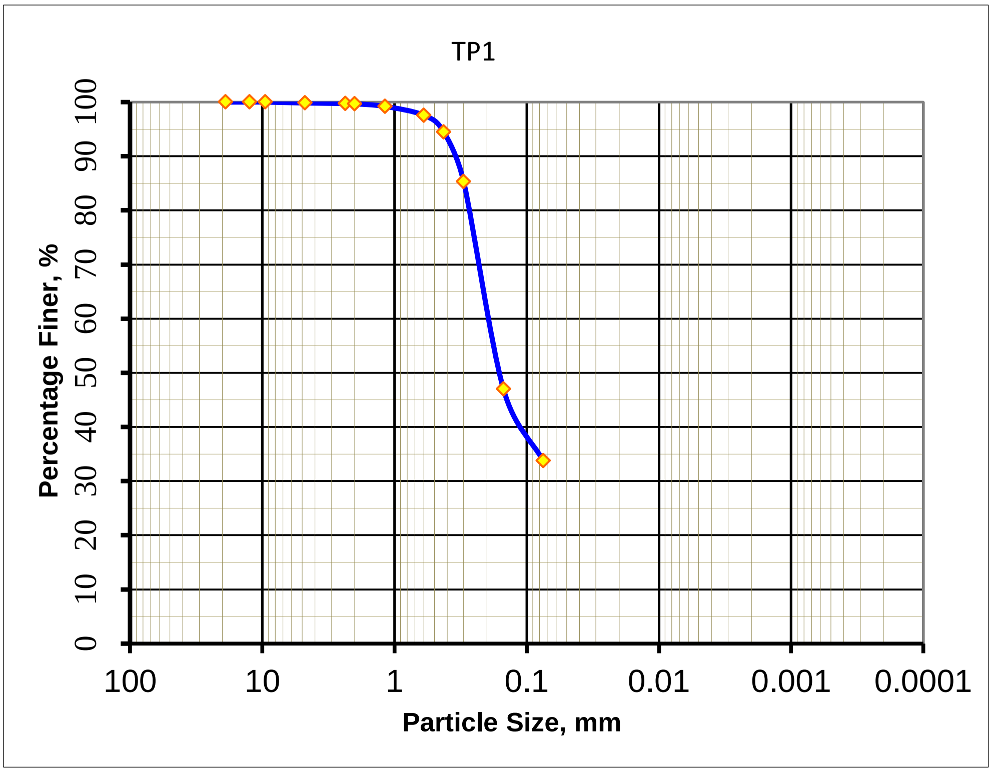
Sample No.	TP7B				
Moisture content Vs dry density computation table					
Determination No.	**1**	**2**	**3**	**4**	**5**
Mass of Mold, g	4321.1	4321.1	4321.1	4321.1	4321.1
Mass of mold + Compacted Soil, g	5577	5833.8	6003.8	5920.3	5789.1
Mass of Compacted soil, g	1255.9	1512.7	1612.7	1599.2	1468
Volume of Mold, cm3	944	944	944	944	944
Bulk density, g/cm3	1.33	1.60	1.71	1.69	1.56
Water Content, %	15.02	25.32	30.30	38.21	45.71
Dry density, g/cm3	**1.16**	**1.28**	**1.31**	**1.23**	**1.07**
TR	**2**	**3**	**3**	**4**	**5**
Container No	A-11	C-5	C6	A-7	D-4
Mass of container, g	24.1	24	24.1	24.3	23.9
Mass of container + wet soil, g	62.4	73.5	77	87.6	81.4
Mass of container + Dry soil, g	57.4	63.5	64.7	70.1	63.3
Mass of Water, g	5	10	12.3	17.5	18.1
Mass of Dry soil, g	33.3	39.5	40.6	45.8	39.4
Water content, %	**15.02**	**25.32**	**30.30**	**38.21**	**45.94**


MDD (g/cm3) Vs OMC (%) of soil sample (TP1)
a
MDD=1.78
OMC=15.72
Dry density(g/cm3)
Moisture contents (%)
MDD (g/cm3) Vs OMC (%) of soil sample (TP2)
b
MDD=1.28
OMC=26.79
Dry density(g/cm3)
Moisture contents (%)
MDD (g/cm3) Vs OMC (%) of soil sample (TP3)
c
MDD=1.2
OMC=38.4
Dry density(g/cm3)
Moisture contents (%)
MDD (g/cm3) Vs OMC (%) of soil sample (TP4)
d
MDD=1.31
OMC=34
Dry density(g/cm3)
Moisture contents(%)
MDD (g/cm3) Vs OMC (%) of soil sample (TP5)
e
MDD=1.46
OMC=23.09
Dry density(g/cm3)
Moisture contents (%)
MDD (g/cm3) Vs OMC (%) of soil sample (TP6)
f
MDD=1.45
OMC=26.26
Dry density(g/cm3)
Moisture contents (%)
MDD (g/cm3) Vs OMC (%) of soil sample (TP7A)
g
MDD=1.23
OMC=36.52
Dry density(g/cm3)
Moisture contents (%)
MDD (g/cm3) Vs OMC (%) of soil sample (TP7B)
h
MDD=1.31
OMC=30.3
Dry density(g/cm3)
Moisture contents (%)

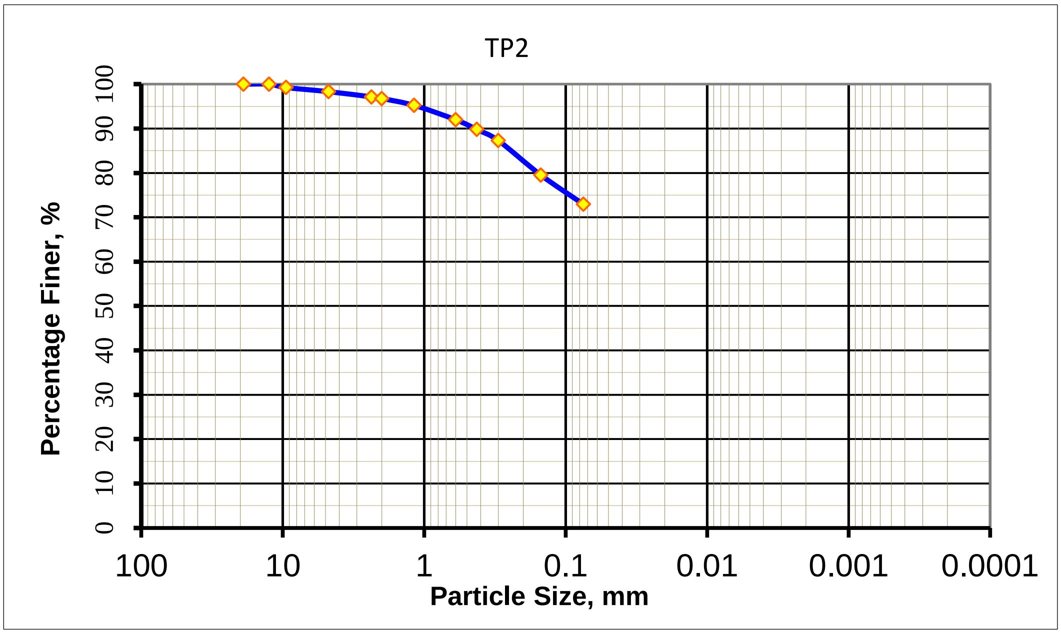
Appendix-D Grain Size Analysis (Sieve Analysis) Test Results of Soil Samples (TP1-TP7B) With Percentage (%) Finer Vs Particle Size (mm) Graphs

Project Name:	MSc Thesis		Date tested	May 30 2023
Sample Description:	Disturbed		Tested by	Lencho Lemma

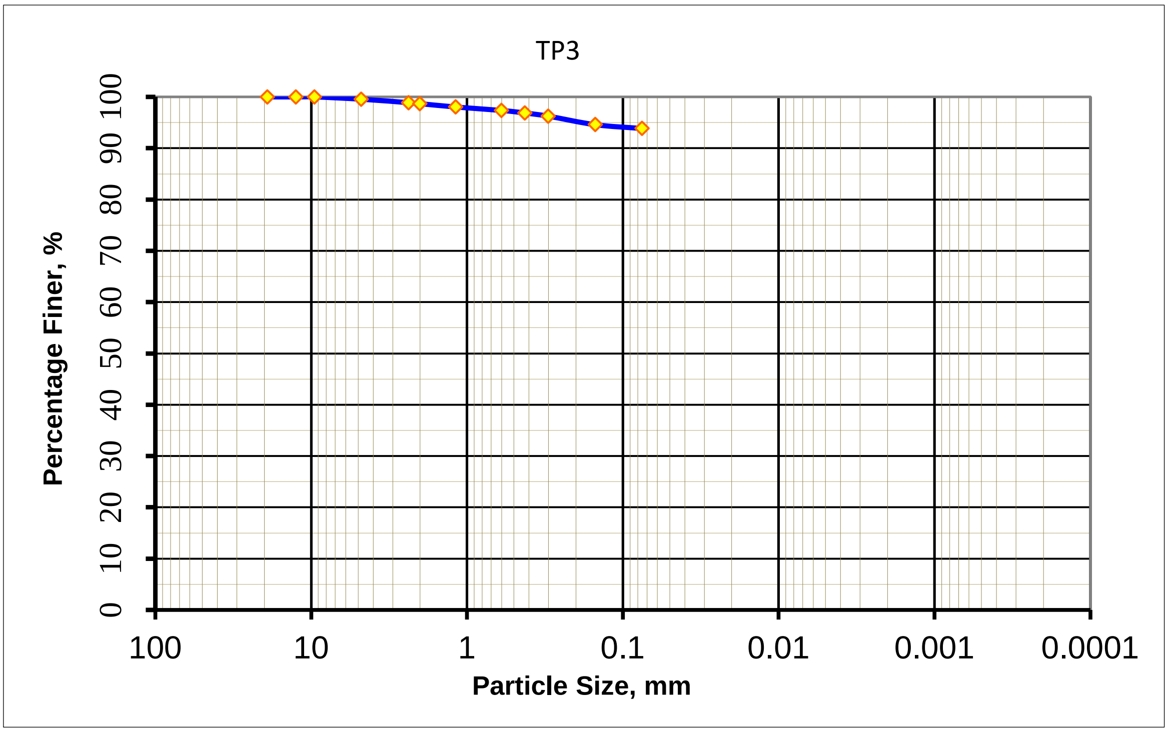
Grain Size Analysis of soil ASTM D 422							*TP1*
					Total mass of sample, g		**1000**
Sieve No	**Sieve Opening(mm)**	**Mass of Sieve(g)**	**Mass of sieve +Retained soil(g)**	**Mass of Retained soil(g)**	**Percentage Retained (%)**	**Cum. Percentage Retained (%)**	**Perc. Passing (%)**
3/4''	19.0	555.4	555.4	0.0	0.0	0.0	100.0
1/2"	12.5	560.8	560.8	0.0	0.0	0.0	100.0
3.8"	9.5	534.3	534.3	0.0	0.0	0.0	100.0
No 4	4.75	564.6	566.3	1.7	0.2	0.2	99.8
No 8	2.36	533.5	534.4	0.9	0.1	0.3	99.7
No 10	2	519.8	520.4	0.6	0.1	0.3	99.7
No 16	1.18	484.9	489.7	4.8	0.5	0.8	99.2
No 30	0.6	455.7	472.0	16.3	1.6	2.4	97.6
No 40	0.425	432.4	463.1	30.7	3.1	5.5	94.5
No 50	0.3	418.0	510.1	92.1	9.2	14.7	85.3
No 100	0.15	403.1	786.2	383.1	38.3	53.0	47.0
No 200	0.075	390.9	523.0	132.1	13.2	66.2	33.8
pan	-------	369.6	378.4	8.8	0.9	67.1	32.9
				671.1	**67.11**		



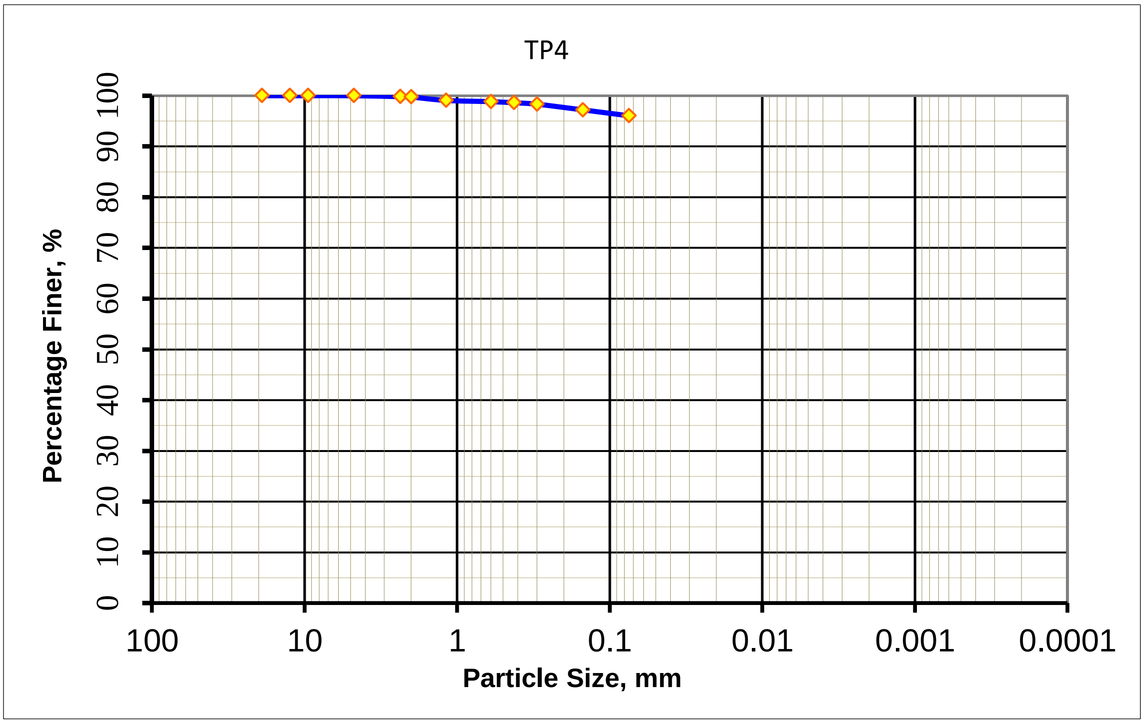
Grain Size Analysis of soil ASTM D 422							*TP2*
					Total mass of sample, g		**1000**
Sieve No	**Sieve Opening (mm)**	**Mass of Sieve (g)**	**Mass of sieve + Retained soil**	**Mass of Retained**	**Percentage Retained (%)**	**Cum. Percentage Retained (%)**	**Perc. Passing (%)**
3/4''	19.0	555.4	555.4	0.0	0.0	0.0	100.0
1/2"	12.5	560.8	560.8	0.0	0.0	0.0	100.0
3.8"	9.5	534.3	541.9	7.6	0.8	0.8	99.2
No 4	4.75	564.6	573.9	9.3	0.9	1.7	98.3
No 8	2.36	533.5	545.3	11.8	1.2	2.9	97.1
No 10	2	519.8	523.4	3.6	0.4	3.2	96.8
No 16	1.18	484.9	500.4	15.5	1.6	4.8	95.2
No 30	0.6	455.7	488.1	32.4	3.2	8.0	92.0
No 40	0.425	432.4	454.3	21.9	2.2	10.2	89.8
No 50	0.3	418.0	442.9	24.9	2.5	12.7	87.3
No 100	0.15	403.1	481.0	77.9	7.8	20.5	79.5
No 200	0.075	390.9	456.7	65.8	6.6	27.1	72.9
pan	-------	369.6	372.4	2.8	0.3	27.4	72.7
				273.5	27.35		



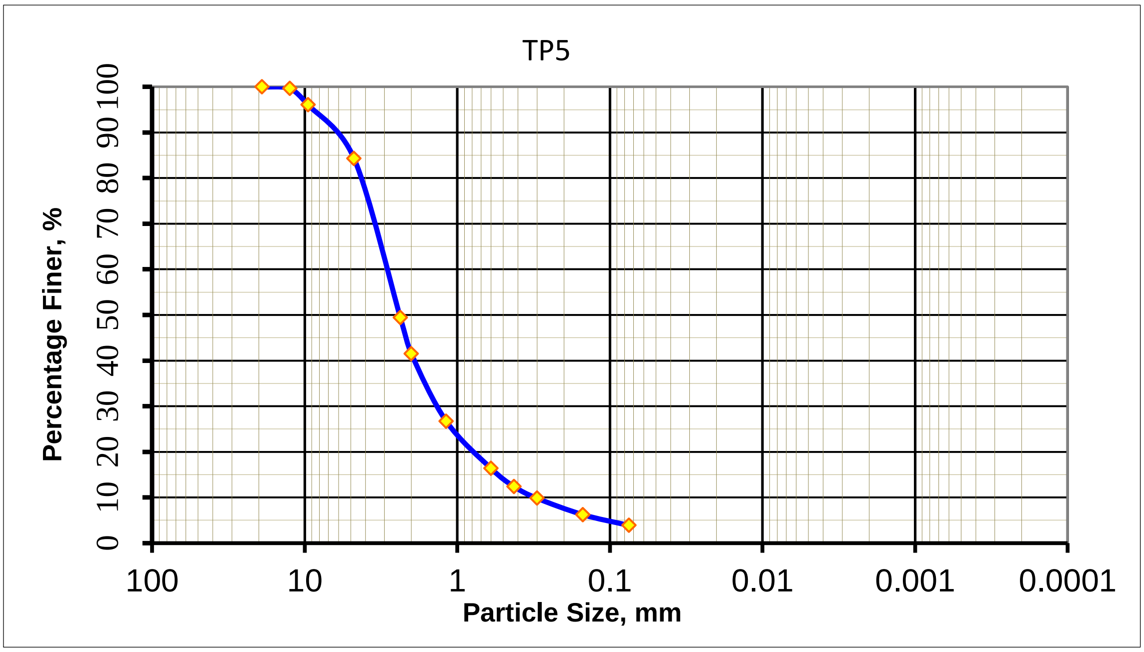
Grain Size Analysis of soil ASTM D 422							*TP3*
				Total mass of sample, g			1000
Sieve No	Sieve Opening(mm)	Mass of Sieve (g)	Mass of sieve +Retained soil (g)	Mass of Retained soil (g)	Percentage Retained (%)	Cum. Percentage Retained (%)	Perc. Passing (%)
3/4"	19.0	555.4	555.4	0.0	0.0	0.0	100.0
1/2"	12.5	560.8	560.8	0.0	0.0	0.0	100.0
3.8"	9.5	534.3	534.3	0.0	0.0	0.0	100.0
No 4	4.75	564.6	569.1	4.5	0.5	0.5	99.6
No 8	2.36	533.5	540.3	6.8	0.7	1.1	98.9
No 10	2	519.8	521.7	1.9	0.2	1.3	98.7
No 16	1.18	484.9	491.4	6.5	0.7	2.0	98.0
No 30	0.6	455.7	462.4	6.7	0.7	2.6	97.4
No 40	0.425	432.4	437.3	4.9	0.5	3.1	96.9
No 50	0.3	418.0	424.3	6.3	0.6	3.8	96.2
No 100	0.15	403.1	419.7	16.6	1.7	5.4	94.6
No 200	0.075	390.9	398.3	7.4	0.7	6.2	93.8
pan	-------	369.6	370.2	0.6	0.1	6.2	93.8
				62.2	**6.22**		



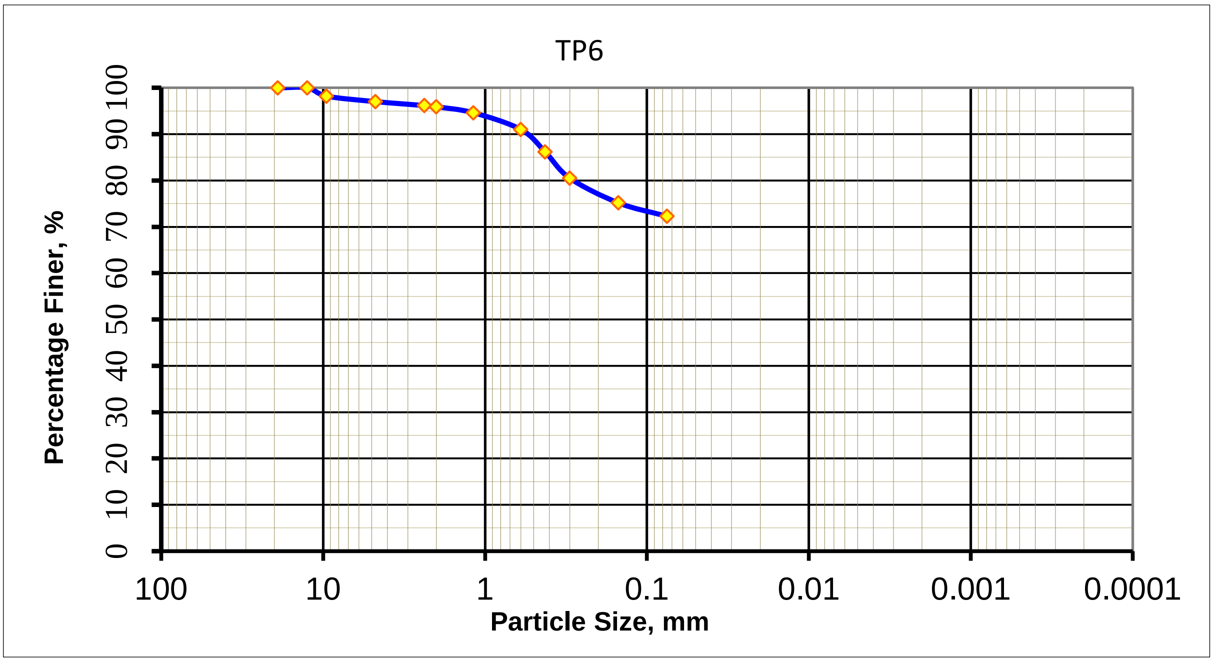
Grain Size Analysis of soil ASTM D 422						*TP4*	
				Total mass of sample, g			1000
Sieve No	Sieve Opening	Mass of Sieve	Mass of sieve + Retained soil	Mass of Retained soil	Percentage Retained	Cum. Percentage Retained	Perc. Passing
	(mm)	(g)	(g)	(g)	(%)	(%)	(%)
3/4"	19.0	555.4	555.4	0.0	0.0	0.0	100.0
1/2"	12.5	560.8	560.8	0.0	0.0	0.0	100.0
3.8"	9.5	534.3	534.3	0.0	0.0	0.0	100.0
No 4	4.75	564.6	564.6	0.0	0.0	0.0	100.0
No 8	2.36	533.5	535.4	1.9	0.2	0.2	99.8
No 10	2	519.8	520.5	0.7	0.1	0.3	99.7
No 16	1.18	484.9	491.6	6.7	0.7	0.9	99.1
No 30	0.6	455.7	458.1	2.4	0.2	1.2	98.8
No 40	0.425	432.4	434.6	2.2	0.2	1.4	98.6
No 50	0.3	418.0	420.8	2.8	0.3	1.7	98.3
No 100	0.15	403.1	414.3	11.2	1.1	2.8	97.2
No 200	0.075	390.9	402.5	11.6	1.2	4.0	96.1
pan	-------	369.6	369.9	0.3	0.0	4.0	96.0
				39.8	**3.98**		



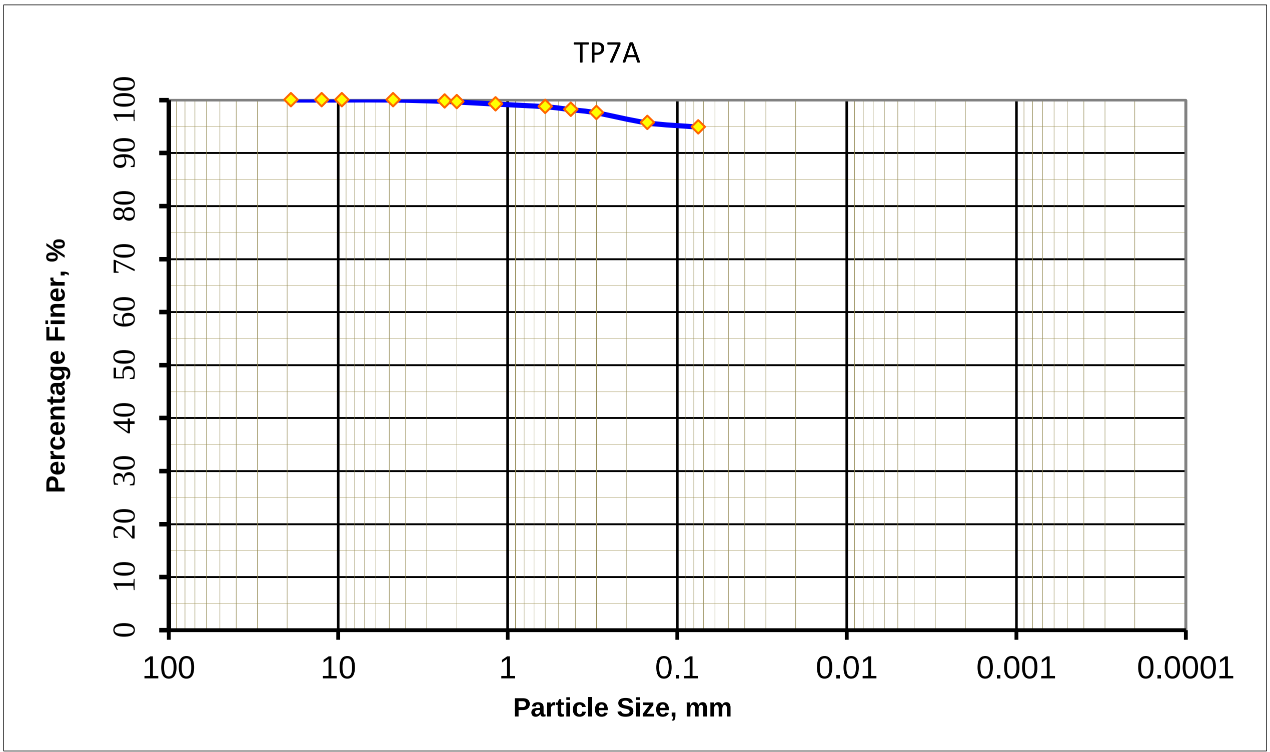
Grain Size Analysis of soil ASTM D 422						***TP5***	
				Total mass of sample, g			**1000**
Sieve No	**Sieve Opening**	**Mass of Sieve**	**Mass of sieve + Retained soil**	**Mass of Retained soil**	**Percentage Retained**	**Cum. Percentage Retained**	**Perc. Passing**
	(mm)	**(g)**	**(g)**	**(g)**	**(%)**	**(%)**	**(%)**
3/4"	19.0	555.4	555.4	0.0	0.0	0.0	100.0
1/2"	12.5	560.7	564.7	4.0	0.4	0.4	99.6
3.8"	9.5	531.7	567.3	35.6	3.6	4.0	96.0
No 4	4.75	564.6	682.0	117.4	11.7	15.7	84.3
No 8	2.36	533.5	882.7	349.2	34.9	50.6	49.4
No 10	2	519.8	598.8	79.0	7.9	58.5	41.5
No 16	1.18	484.9	633.1	148.2	14.8	73.3	26.7
No 30	0.6	455.7	559.0	103.3	10.3	83.7	16.3
No 40	0.425	432.4	471.9	39.5	4.0	87.6	12.4
No 50	0.3	418.0	443.7	25.7	2.6	90.2	9.8
No 100	0.15	403.1	439.1	36.0	3.6	93.8	6.2
No 200	0.075	390.9	414.3	23.4	2.3	96.1	3.9
pan	-------	369.6	408.3	38.7	3.9	100.0	0.0
				1000.0	**100**		



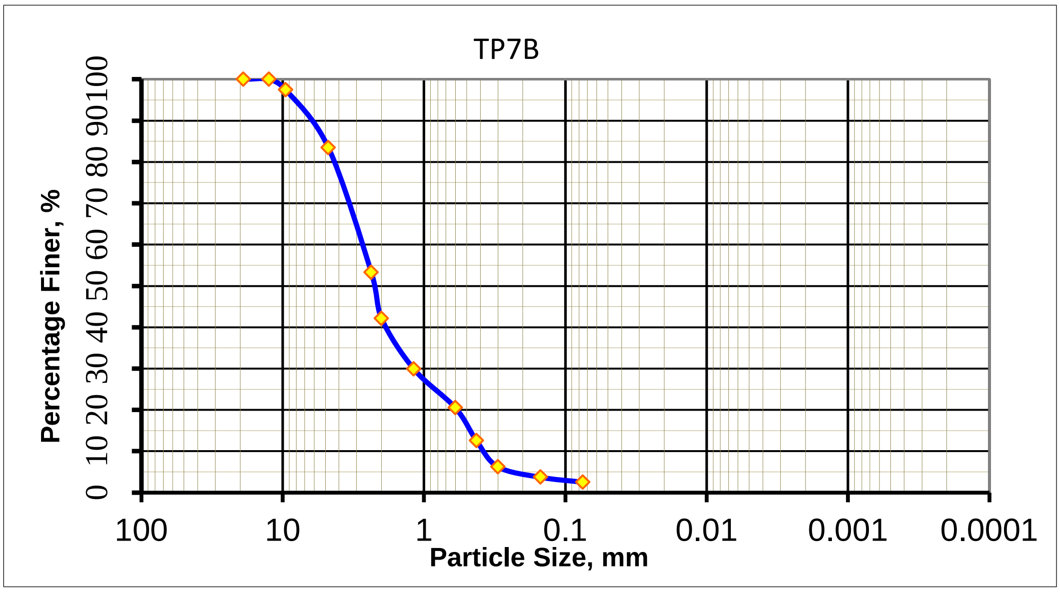
Grain Size Analysis of soil ASTM D 422						*TP6*	
				Total mass of sample, g			1000
Sieve No	Sieve Opening	Mass of Sieve	Mass of sieve + Retained soil	Mass of Retained soil	Percentage Retained	Cum. Percentage Retained	Perc. Passing
	(mm)	(g)	(g)	(g)	(%)	(%)	(%)
3/4"	19.0	555.4	555.4	0.0	0.0	0.0	100.0
1/2"	12.5	560.7	560.7	0.0	0.0	0.0	100.0
3.8"	9.5	531.7	549.8	18.1	1.8	1.8	98.2
No 4	4.75	564.6	576.3	11.7	1.2	3.0	97.0
No 8	2.36	533.5	542.1	8.6	0.9	3.8	96.2
No 10	2	519.8	522.4	2.6	0.3	4.1	95.9
No 16	1.18	484.9	497.7	12.8	1.3	5.4	94.6
No 30	0.6	455.7	492.0	36.3	3.6	9.0	91.0
No 40	0.425	432.4	480.7	48.3	4.8	13.8	86.2
No 50	0.3	418.0	474.5	56.5	5.7	19.5	80.5
No 100	0.15	403.1	456.6	53.5	5.4	24.8	75.2
No 200	0.075	390.9	420.0	29.1	2.9	27.8	72.3
pan	-------	369.6	371.1	1.5	0.2	27.9	72.1
				279.0	**27.9**		



Grain Size Analysis of soil ASTM D 422						*TP7A*	
				Total mass of sample, g			1000
Sieve No	Sieve Opening	Mass of Sieve	Mass of sieve + Retained soil	Mass of Retained soil	Percentage Retained	Cum. Percentage Retained	Perc. Passing
	(mm)	(g)	(g)	(g)	(%)	(%)	(%)
3/4"	19.0	555.4	555.4	0.0	0.0	0.0	100.0
1/2"	12.5	560.7	560.7	0.0	0.0	0.0	100.0
3.8"	9.5	531.7	531.7	0.0	0.0	0.0	100.0
No 4	4.75	564.6	564.6	0.0	0.0	0.0	100.0
No 8	2.36	533.5	536.0	2.5	0.3	0.3	99.8
No 10	2	519.8	521.0	1.2	0.1	0.4	99.6
No 16	1.18	484.9	488.8	3.9	0.4	0.8	99.2
No 30	0.6	455.7	460.8	5.1	0.5	1.3	98.7
No 40	0.425	432.4	437.8	5.4	0.5	1.8	98.2
No 50	0.3	418.0	424.0	6.0	0.6	2.4	97.6
No 100	0.15	403.1	422.1	19.0	1.9	4.3	95.7
No 200	0.075	390.9	398.9	8.0	0.8	5.1	94.9
pan	-------	369.6	370.0	0.4	0.0	5.2	94.9
				51.5	**5.15**		



Grain Size Analysis of soil ASTM D 422							*TP7B*
				Total mass of sample, g			1000
Sieve No	Sieve Opening (mm)	Mass of Sieve (g)	Mass of sieve + Retained soil (g)	Mass of Retained soil (g)	Percentage Retained (%)	Cum. Percentage Retained (%)	Perc. Passing (%)
3/4"	19.0	555.4	555.4	0.0	0.0	0.0	100.0
1/2"	12.5	560.7	560.7	0.0	0.0	0.0	100.0
3.8"	9.5	531.7	557.6	25.9	2.6	2.6	97.4
No 4	4.75	564.6	704.1	139.5	14.0	16.5	83.5
No 8	2.36	533.5	835.5	302.0	30.2	46.7	53.3
No 10	2	519.8	631.0	111.2	11.1	57.9	42.1
No 16	1.18	484.9	607.6	122.7	12.3	70.1	29.9
No 30	0.6	455.7	549.4	93.7	9.4	79.5	20.5
No 40	0.425	432.4	512.3	79.9	8.0	87.5	12.5
No 50	0.3	418.0	481.0	63.0	6.3	93.8	6.2
No 100	0.15	403.1	428.3	25.2	2.5	96.3	3.7
No 200	0.075	390.9	402.8	11.9	1.2	97.5	2.5
pan	-------	369.6	394.6	25.0	2.5	100.0	0.0
				1000.0	**100**		

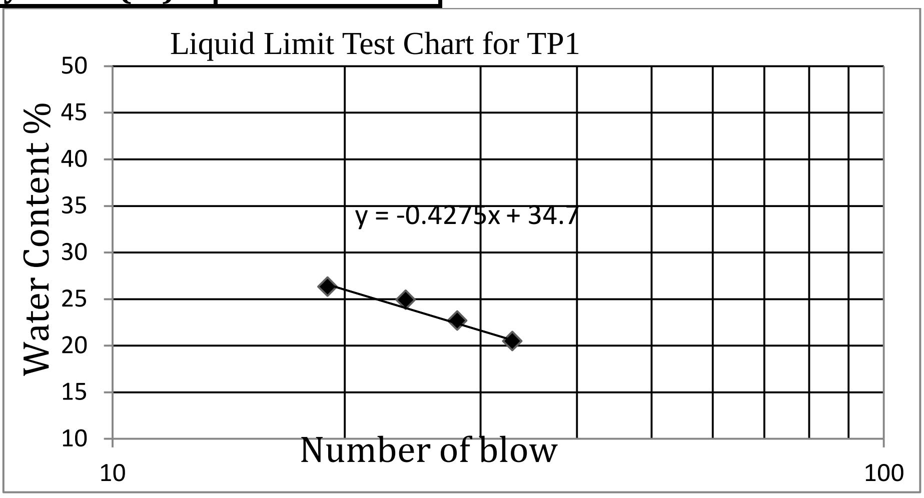


Appendix-E Atterberg Limit Test Result and Liquid Limit Test Chart for (TP1-TP7B)

Date Tested :	June 2 2023				
Tested By:	Lencho lemma				
Project Name:	MSc Thesis				
Sample Description:					
Standard Reference: ASTM D 4318					
Liquid Limit					
Sample Number:	**TP1**				
Moisture can and lid number		**Y-6**	**A-7**	**A-5**	**A-8**
Number of Drops (N)		**19**	**24**	**28**	**33**
MC = Mass of empty, clean can + lid (grams)		**24.1**	24	24.3	24.4
MCMS = Mass of can, lid, and moist soil (grams)		65.4	56.6	57.3	55.6
MCDS = Mass of can, lid, and dry soil (grams)		56.8	50.1	51.2	50.3
MS = Mass of soil solids (grams)		32.7	26.1	26.9	25.9
MW = Mass of pore water (grams)		8.6	6.5	6.1	5.3
w = Water content, w%		26.29969	24.90421	22.67658	20.46332

Plastic Limit Test			
Sample Number:	**TP1**		
Moisture can and lid number		**D-D**	**B-B**
MC = Mass of empty, clean can + lid (grams)		13.2	13.2
MCMS = Mass of can, lid, and moist soil (grams)		16.4	16.4
MCDS = Mass of can, lid, and dry soil (grams)		16.1	16.1
MS = Mass of soil solids (grams)		2.9	2.9
MW = Mass of pore water (grams)		0.3	0.3
w = Water content, w%		10.34483	10.34483
Average Moisture Content w%		**10.34**	

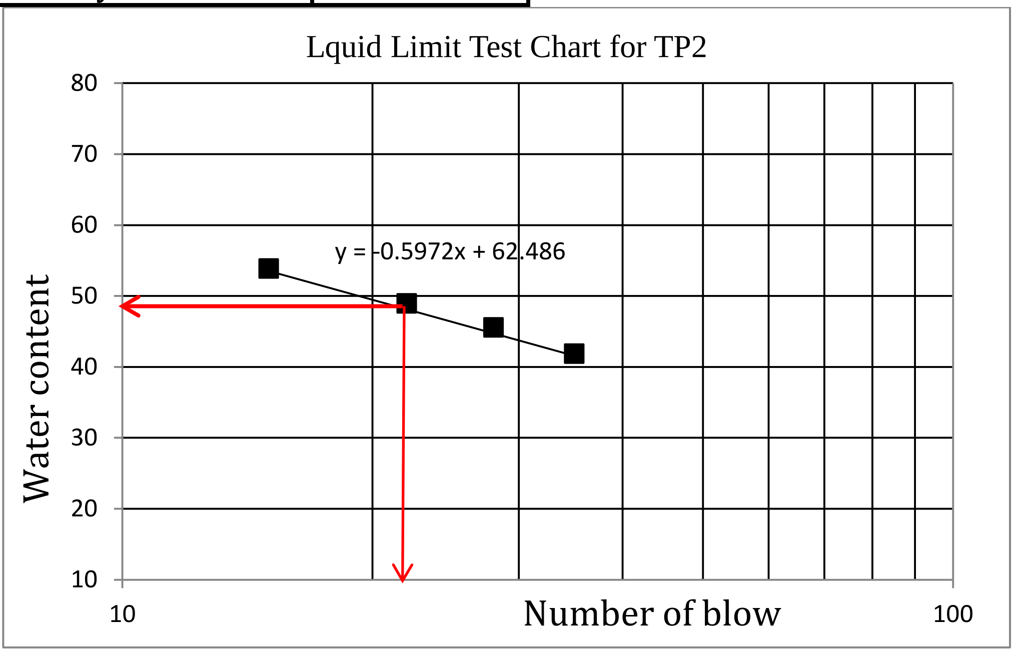
Liquid Limit (LL)	24.0575
Plastic Limit(PL)	10.34
Plasticity Index (PI)	**13.71267241**



Liquid Limit					
Sample Number:	**TP2**				
Moisture can and lid number		**B-3**	**Z-1**	**E23**	**C-1**
Number of Drops (N)		**15**	**22**	**28**	**35**
MC = Mass of empty, clean can + lid (grams)		23.8	24	24.1	23.9
MCMS = Mass of can, lid, and moist soil (grams)		45.8	45.6	60.2	50
MCDS = Mass of can, lid, and dry soil (grams)		38.1	38.5	48.9	42.3
MS = Mass of soil solids (grams)		14.3	14.5	24.8	18.4
MW = Mass of pore water (grams)		7.7	7.1	11.3	7.7
w = Water content, w%		53.84615	48.96552	45.56452	41.84783

Plastic Limit Test			
Sample Number:	**TP2**		
Moisture can and lid number		**E-E**	**F-F**
MC = Mass of empty, clean can + lid (grams)		13.2	13.2
MCMS = Mass of can, lid, and moist soil (grams)		17.1	16.4
MCDS = Mass of can, lid, and dry soil (grams)		16.1	15.6
MS = Mass of soil solids (grams)		2.9	2.4
MW = Mass of pore water (grams)		1	0.8
w = Water content, w%		34.48276	33.33333
Average Moisture Content w%		**33.91**	

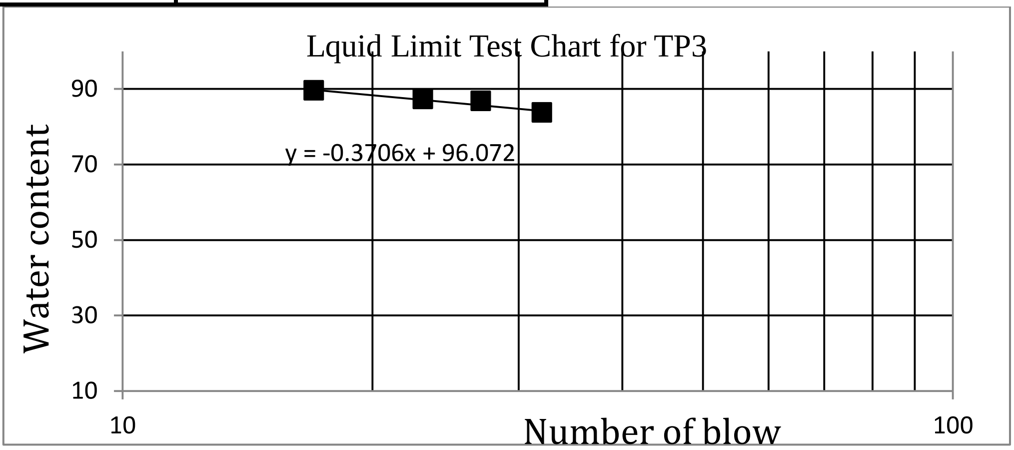
Liquid Limit	**47.556**
Plastic Limit	**33.91**
Plasticity Index	**13.64795402**



Liquid Limit					
Sample Number:	**TP3**				
Moisture can and lid number		**A-8**	**Y-2**	**P-2**	**A-24**
Number of Drops (N)		**17**	**23**	**27**	**32**
MC = Mass of empty, clean can + lid (grams)		24	24.2	24.3	24
MCMS = Mass of can, lid, and moist soil (grams)		55.1	56.8	55.5	51.2
MCDS = Mass of can, lid, and dry soil (grams)		40.4	41.6	41	38.8
MS = Mass of soil solids (grams)		16.4	17.4	16.7	14.8
MW = Mass of pore water (grams)		14.7	15.2	14.5	12.4
w = Water content, w%		89.63415	87.35632	86.82635	83.78378

Plastic Limit Test			
Sample Number:	**TP3**		
Moisture can and lid number		**G-G**	**H-H**
MC = Mass of empty, clean can + lid (grams)		13	13.1
MCMS = Mass of can, lid, and moist soil (grams)		15.8	16.5
MCDS = Mass of can, lid, and dry soil (grams)		14.9	15.4
MS = Mass of soil solids (grams)		1.9	2.3
MW = Mass of pore water (grams)		0.9	1.1
w = Water content, w%		47.36842	47.82609
Average Moisture Content w%		**47.60**	

Liquid Limit	**86.807**
Plastic Limit	**47.60**
Plasticity Index	**39.209746**



Liquid Limit					
Sample Number:	**TP4**				
Moisture can and lid number		**A-A**	**E-22**	**E-27**	**A26**
Number of Drops (N)		**18**	**24**	**27**	**33**
MC = Mass of empty, clean can + lid (grams)		24.5	23.8	24.5	24.3
MCMS = Mass of can, lid, and moist soil (grams)		59.9	52.5	54.6	54.1
MCDS = Mass of can, lid, and dry soil (grams)		46.4	41.9	44	43.8
MS = Mass of soil solids (grams)		21.9	18.1	19.5	19.5
MW = Mass of pore water (grams)		13.5	10.6	10.6	10.3
w = Water content, w%		61.64384	58.56354	54.35897	52.82051

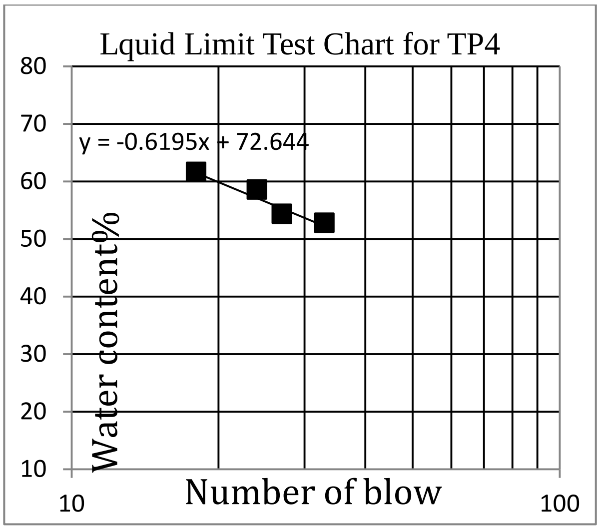
Plastic Limit Test			
Sample Number:	**TP4**		
Moisture can and lid number		**B-5**	**A-11**
MC = Mass of empty, clean can + lid (grams)		24.1	23
MCMS = Mass of can, lid, and moist soil (grams)		28.4	28
MCDS = Mass of can, lid, and dry soil (grams)		27.4	26.8
MS = Mass of soil solids (grams)		3.3	3.8
MW = Mass of pore water (grams)		1	1.2
w = Water content, w%		30.30303	31.57895
Average Moisture Content w%		**30.94**	

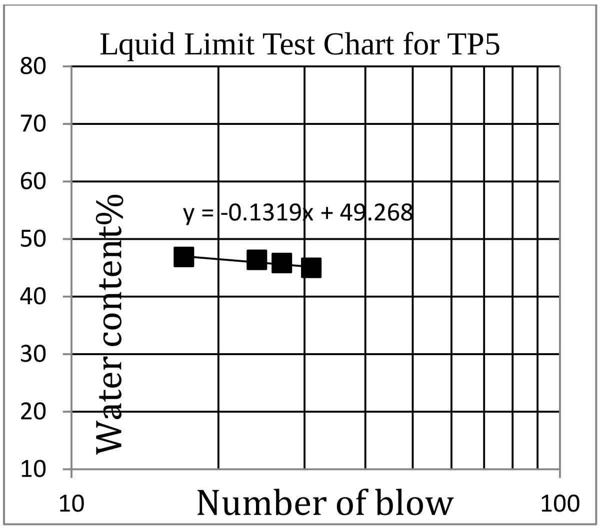
Liquid Limit					
Sample Number:	**TP5**				
Moisture can and lid number		**E6**	**Z-7 A13**		**W-2**
Number of Drops (N)		**17**	**24**	**27**	**31**
MC = Mass of empty, clean can + lid (grams)		24.2	24.1	23.4	24.1
MCMS = Mass of can, lid, and moist soil (grams)		57.1	60.1	60	63.1
MCDS = Mass of can, lid, and dry soil (grams)		46.6	48.7	48.5	51
MS = Mass of soil solids (grams)		22.4	24.6	25.1	26.9
MW = Mass of pore water (grams)		10.5	11.4	11.5	12.1
w = Water content, w%		46.875	46.34146	45.81673	44.98141

Plastic Limit Test			
Sample Number:	**TP5**		
Moisture can and lid number		**C-C**	**AA**
MC = Mass of empty, clean can + lid (grams)		13.2	13.3
MCMS = Mass of can, lid, and moist soil (grams)		16.4	16.5
MCDS = Mass of can, lid, and dry soil (grams)		15.6	15.8
MS = Mass of soil solids (grams)		2.4	2.5
MW = Mass of pore water (grams)		0.8	0.7
w = Water content, w%		33.33333	28
Average Moisture Content w%		**30.67**	

Liquid Limit	**57.1565**
Plastic Limit	**30.94**
Plasticity Index	**26.21551116**

Liquid Limit	**45.9705**
Plastic Limit	**30.67**
Plasticity Index	**15.30383333**

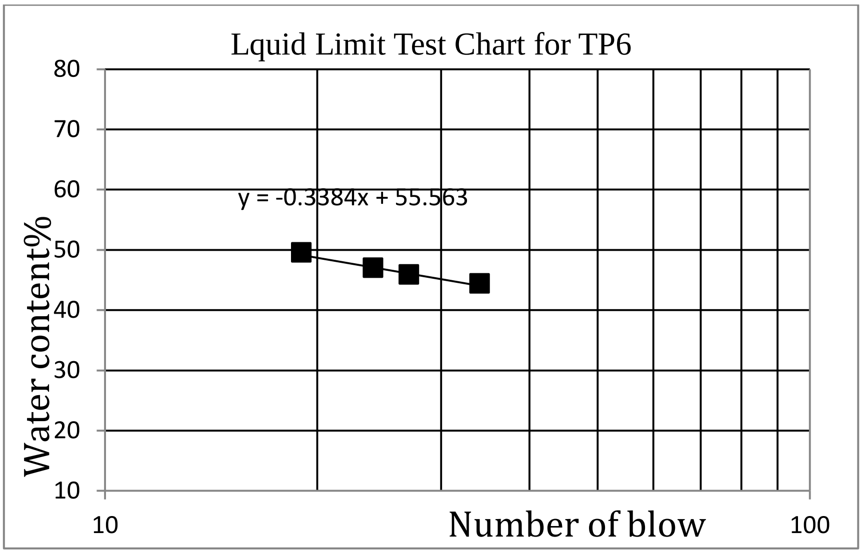




Liquid Limit				
Sample Number:	**TP6**			
Moisture can and lid number	**A-5**	**A3**	**B-6**	**LL-2**
Number of Drops (N)	**19**	**24**	**27**	**34**
MC = Mass of empty, clean can + lid (grams)	27	23.4	23.6	23.8
MCMS = Mass of can, lid, and moist soil (grams)	65.9	58.4	54.1	51.1
MCDS = Mass of can, lid, and dry soil (grams)	53	47.2	44.5	42.7
MS = Mass of soil solids (grams)	26	23.8	20.9	18.9
MW = Mass of pore water (grams)	12.9	11.2	9.6	8.4
w = Water content, w%	49.61538	47.05882	45.93301	44.44444

Plastic Limit Test		
Sample Number:	**TP6**	
Moisture can and lid number	**J-10**	**J-7**
MC = Mass of empty, clean can + lid (grams)	13.1	13.2
MCMS = Mass of can, lid, and moist soil (grams)	17.4	18.4
MCDS = Mass of can, lid, and dry soil (grams)	16.5	17.3
MS = Mass of soil solids (grams)	3.4	4.1
MW = Mass of pore water (grams)	0.9	1.1
w = Water content, w%	26.47059	26.82927
Average Moisture Content w%	**26.65**	

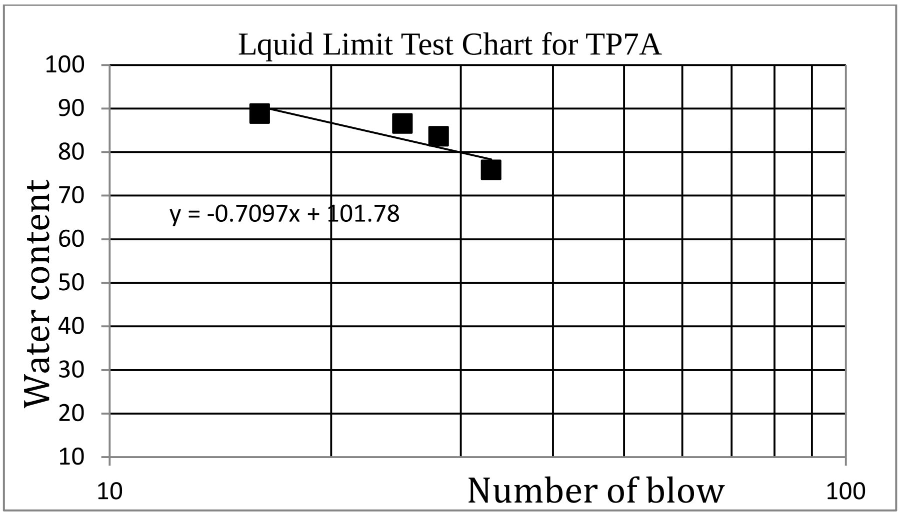
Liquid Limit	**47.103**
Plastic Limit	**26.65**
Plasticity Index	**20.45307174**



Liquid Limit					
Sample Number:	**TP7A**				
Moisture can and lid number		**A21**	**C-C**	**P-1**	**Z-2**
Number of Drops (N)		**16**	**25**	**28**	**33**
MC = Mass of empty, clean can + lid (grams)		23.6	23.9	24.9	24
MCMS = Mass of can, lid, and moist soil (grams)		62.3	69.4	60.7	59
MCDS = Mass of can, lid, and dry soil (grams)		44.1	48.3	44.4	43.9
MS = Mass of soil solids (grams)		20.5	24.4	19.5	19.9
MW = Mass of pore water (grams)		18.2	21.1	16.3	15.1
w = Water content, w%		88.78049	86.47541	83.58974	75.8794

Plastic Limit Test			
Sample Number:	**TP7A**		
Moisture can and lid number		**E**	**D3**
MC = Mass of empty, clean can + lid (grams)		24	24.5
MCMS = Mass of can, lid, and moist soil (grams)		27.3	29.4
MCDS = Mass of can, lid, and dry soil (grams)		26.3	27.9
MS = Mass of soil solids (grams)		2.3	3.4
MW = Mass of pore water (grams)		1	1.5
w = Water content, w%		43.47826	44.11765
Average Moisture Content w%		**43.80**	

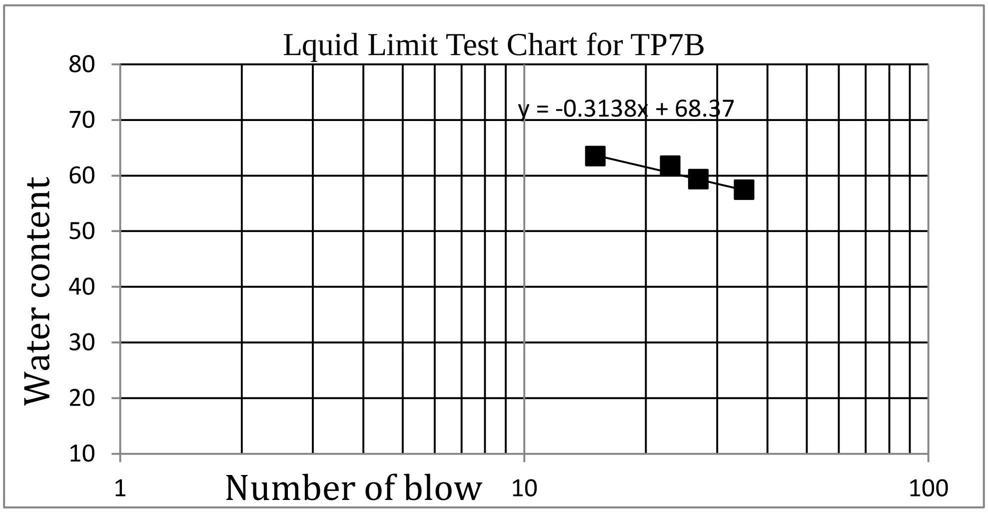
Liquid Limit	**84.0375**
Plastic Limit	**43.80**
Plasticity Index	**40.23954604**



Liquid Limit				
Sample Number:	**TP7B**			
Moisture can and lid number	**C-11**	**E11**	**LL-3**	**C-3**
Number of Drops (N)	**15**	**23**	**27**	**35**
MC = Mass of empty, clean can + lid (grams)	24.2	23.4	23.9	23.9
MCMS = Mass of can, lid, and moist soil (grams)	65.4	68.7	54.5	60.9
MCDS = Mass of can, lid, and dry soil (grams)	49.4	51.4	43.1	47.4
MS = Mass of soil solids (grams)	25.2	28	19.2	23.5
MW = Mass of pore water (grams)	16	17.3	11.4	13.5
w = Water content, w%	63.49206	61.78571	59.375	57.44681

Plastic Limit Test		
Sample Number:	**TP7B**	
Moisture can and lid number	**C-C**	**AA**
MC = Mass of empty, clean can + lid (grams)	13.2	13.3
MCMS = Mass of can, lid, and moist soil (grams)	16.4	16.5
MCDS = Mass of can, lid, and dry soil (grams)	15.7	15.8
MS = Mass of soil solids (grams)	2.5	2.5
MW = Mass of pore water (grams)	0.7	0.7
w = Water content, w%	28	28
Average Moisture Content w%	**28.00**	

Liquid Limit (LL)	**60.525**
Plastic Limit (PL)	**28.00**
Plasticity Index (PI)	**32.525**



Appendix-F Specific Gravity Test Results for Sample (TP1-TP7B)

Date Tested August:	**JUN 5 2023**
Tested By:	**Lencho Lemma**
Project Name:	**MSc Thesis**
Sample Description:	**Disturbed soil sample**

Specific gravity of soil ASTM D 854

Specific Gravity for TP1		
Sample Number	**A**	**B**
Pycnometer #	1	2
C. Mass of Dry Soil (No. 10 sieve)	50	50
D. Mass of Pycnometer full of water	338.5	338.5
E. Mass of pycno + Soil +full of water (Distil)	369.9	369.9
T^0c	23.5	23.5
T^0c correction	0.9975	0.9975
F. Specific Gravity=C/(C+(D-E))	2.68	2.68
Average Specific Gravity ((1+2)/2)	2.68	

Specific Gravity for TP2		
Sample Number	**A**	**B**
Pycnometer #	1	2
C. Mass of Dry Soil (No. 10 sieve)	50	50
D. Mass of Pycnometer full of water	338.5	338.5
E. Mass of pycno + Soil +full of water (Distil)	368.5	368.4
T^0c	22.5	22.5
T^0c correction	0.9977	0.9977
F. Specific Gravity=C/(C+(D-E))	2.49	2.48
Average Specific Gravity ((1+2)/2)	2.49	

Specific Gravity for TP5		
Sample Number	**A**	**B**
Pycnometer #	1	2
C. Mass of Dry Soil (No. 10 sieve)	50	50
D. Mass of Pycnometer full of water	338.5	338.5
E. Mass of pycno + Soil +full of water (Distil)	368.7	368.8
T^0c	22.5	22.5
T^0c correction	0.9977	0.9977
F. Specific Gravity=C/(C+(D-E))	2.52	2.53
Average Specific Gravity ((1+2)/2)	2.53	

Specific Gravity for TP6		
Sample Number	**A**	**B**
Pycnometer #	1	2
C. Mass of Dry Soil (No. 10 sieve)	50	50
D. Mass of Pycnometer full of water	338.5	338.5
E. Mass of pycno + Soil +full of water (Distil)	368.6	368.5
T^0c	22.5	22.5
T^0c correction	0.9977	0.9977
F. Specific Gravity=C/(C+(D-E))	2.51	2.49
Average Specific Gravity ((1+2)/2)	2.50	

Specific Gravity for TP3		
Sample Number	**A**	**B**
Pycnometer #	1	2
C. Mass of Dry Soil (No. 10 sieve)	50	50
D. Mass of Pycnometer full of water	338.5	338.5
E. Mass of pycno + Soil +full of water	369.3	369.4
T^0c	22.5	22.5
T^0c correction	0.9977	0.9977
F. Specific Gravity=C/(C+(D-E))	2.60	2.61
Average Specific Gravity ((1+2)/2)	2.60	

Specific Gravity for TP4		
Sample Number	**A**	**B**
Pycnometer #	1	2
C. Mass of Dry Soil (No. 10 sieve)	50	50
D. Mass of Pycnometer full of water	338.5	338.5
E. Mass of pycno + Soil +full of water (Distil)	369.6	369.7
T^0c	23	23
T^0c correction	0.9976	0.9976
F. Specific Gravity=C/(C+(D-E))	2.64	2.65
Average Specific Gravity ((1+2)/2)	2.65	

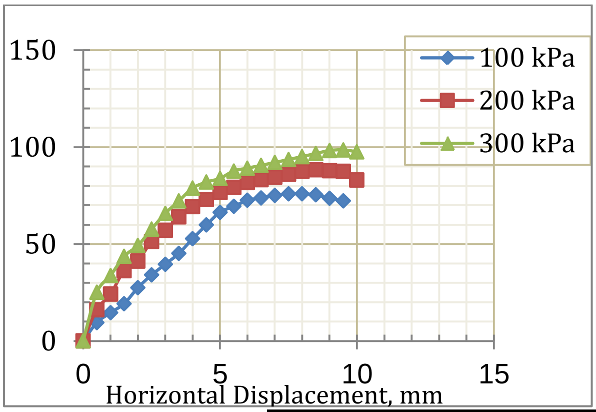
Specific Gravity for TP7A		
Sample Number	**A**	**B**
Pycnometer #	1	2
C. Mass of Dry Soil (No. 10 sieve)	50	50
D. Mass of Pycnometer full of water	354.7	354.7
E. Mass of pycno + Soil +full of water	385.6	385.4
T^0c	22.5	22.5
T^0c correction	0.9977	0.9977
F. Specific Gravity=C/(C+(D-E))	2.61	2.58
Average Specific Gravity ((1+2)/2)	2.60	

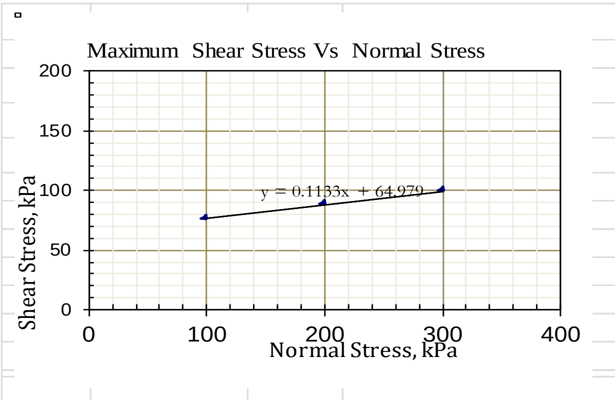
Specific Gravity for TP7B		
Sample Number	**A**	**B**
Pycnometer #	1	2
C. Mass of Dry Soil (No. 10 sieve)	50	50
D. Mass of Pycnometer full of water	338.5	338.5
E. Mass of pycno + Soil +full of water (Distil)	368.7	368.6
T^0c	22.5	22.5
T^0c correction	0.9977	0.9977
F. Specific Gravity=C/(C+(D-E))	2.52	2.51
Average Specific Gravity ((1+2)/2)	2.51	

Appendix-G Direct Shear Test Results, Maximum Shear Stress (kPa) Vs Normal Stress (kPa) and Horizontal Displacement (mm) Graphs For (TP1-TP7B)

Direct shear test					
Standard Reference: ASTM D 3080					
Project Name:	Msc. Thesis	Date tested	Jun 8 2023		
Sample Description:	TP1	Tested by	Lencho lemma		
Sample No.	Sample 1	Sample Depth, m: 3		Location:	Senkele
Thickness of sample:	20 mm	Rate of strain :	0.1 mm/min	Wet unit weight, kN/M^3:	20.18
Length of sample :	60 mm	Moisture content, %	10.1	Dry Unit Weight, kN/M^3:	18.34
Width of sample:	60 mm			Sample Condition:	Disturbed

		Applied Vertical Stress			Applied Vertical Stress			Applied Vertical Stress		
		100 kPa			**200 kPa**			**300 kPa**		
Horizontal Displacement [mm]	Corrected Area [mm2]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]
0	3600	0	0	0	0	0	0	0	0	0
0.5	3570	`	33.80	9.47	29	57.70	16.16	45	89.6	25.10
1.0	3540	26	51.70	14.60	43.00	85.60	24.18	60.00	119.50	33.76
1.5	3510	34	67.70	19.29	64.00	127.40	36.30	77.00	153.30	43.68
2.0	3480	48	95.60	27.47	72.00	143.40	41.21	86.00	171.30	49.22
2.5	3450	59	117.50	34.06	89.00	177.20	51.36	100.00	199.20	57.74
3.0	3420	68	135.40	39.59	98.00	195.20	57.08	113.00	225.00	65.79
3.5	3390	77	153.30	45.22	109.00	217.10	64.04	123.00	245.00	72.27
4.0	3360	89	177.20	52.74	117.00	233.00	69.35	133.00	264.90	78.84
4.5	3330	100	199.20	59.82	122.00	243.00	72.97	137.00	272.90	81.95
5.0	3300	110	219.10	66.39	127.00	252.90	76.64	139.00	276.80	83.88
5.5	3270	114	227.00	69.42	130.00	258.90	79.17	144.00	286.80	87.71
6.0	3240	118	235.00	72.53	133.00	264.90	81.76	145.00	288.80	89.14
6.5	3210	119	237.00	73.83	134.00	266.90	83.15	146.00	290.80	90.59
7.0	3180	120	239.00	75.16	135.00	268.90	84.56	147.00	292.80	92.08
7.5	3150	120	239.00	75.87	136.00	270.90	86.00	148.00	294.80	93.59
8.0	3120	119	237.00	75.96	137.00	272.90	87.47	149.00	296.80	95.13
8.5	3090	117	233.00	75.40	137.00	272.90	88.32	150.00	298.80	96.70
9.0	3060	113	225.00	73.53	135.00	268.90	87.88	151.00	300.70	98.27
9.5	3030	110	219.10	72.31	133.00	264.90	87.43	150.00	298.80	98.61
10.0	3000				125.00	249.00	83.00	147.00	292.80	97.60
		Maximum shear stress, kPa		**75.96**	*Maximum shear stress, kPa*		**88.32**	*Maximum shear stress, kPa*		**98.61**



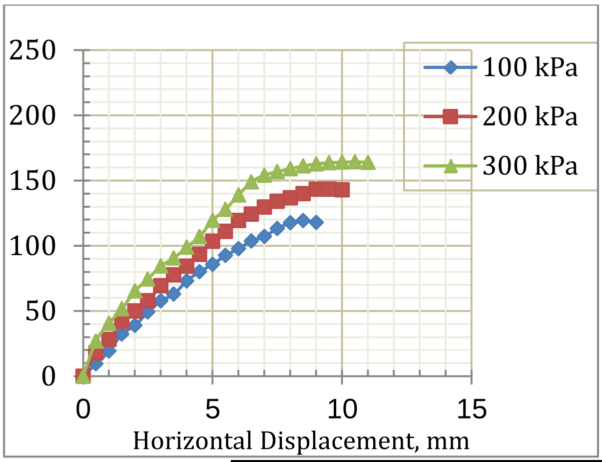


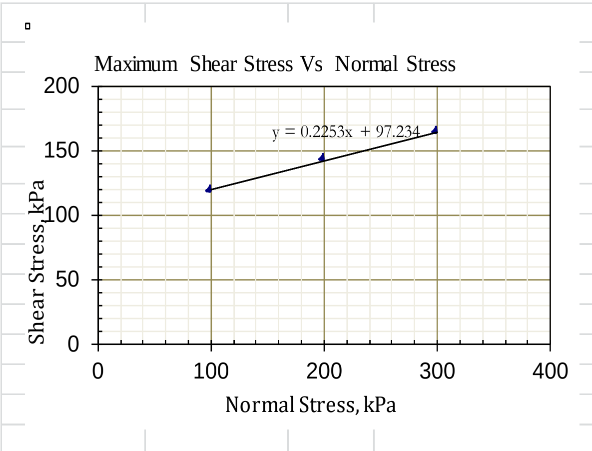
Normal stress(kPa)	100	200	300
Shear strength(kPa)	75.96	88.32	98.61
Parameter			
C (kPa)	64.97		
φ	6.46		

Direct shear test			
Standard Reference: ASTM D 3080			
Project Name:	Msc. Thesis	Date tested	Jun 8 2022
Sample Description:	TP2	Tested by	Lencho Lemma

Sample No.	Sample 1	Sample Depth, 3m		Location:	Teltele
Thickness of sample:	20 mm	Rate of strain :	0.1 mm/min	Wet unit weight, kN/M^3:	13.48
Length of sample :	60 mm	Moisture content, %	26.8	Dry Unit Weight, kN/M^3:	11.85
Width of sample:	60 mm			Sample Condition:	Disturbed

		Applied Vertical Stress 100 kPa			Applied Vertical Stress 200 kPa			Applied Vertical Stress 300 kPa		
Horizontal Displacement [mm]	Corrected Area [mm2]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]
0	3600	0	0	0	0	0	0	0	0	0
0.5	3570	17	33.80	9.47	32	63.70	17.84	49	95.6	26.78
1.0	3540	34	67.70	19.12	50.00	99.60	28.14	72.00	143.40	40.51
1.5	3510	57	113.20	32.25	74.00	147.40	41.99	91.00	181.20	51.62
2.0	3480	68	135.40	38.91	87.00	173.30	49.80	114.00	227.00	65.23
2.5	3450	85	169.30	49.07	100.00	199.20	57.74	129.00	256.90	74.46
3.0	3420	99	197.20	57.66	119.00	237.00	69.30	145.00	288.80	84.44
3.5	3390	107	213.10	62.86	132.00	262.90	77.55	154.00	306.70	90.47
4.0	3360	123	245.00	72.92	142.00	282.80	84.17	167.00	332.60	98.99
4.5	3330	134	266.90	80.15	156.00	310.70	93.30	179.00	356.50	107.06
5.0	3300	142	282.80	85.70	171.00	340.60	103.21	198.00	394.40	119.52
5.5	3270	152	302.70	92.57	182.00	362.50	110.86	210.00	418.30	127.92
6.0	3240	159	316.70	97.75	194.00	386.40	119.26	226.00	450.10	138.92
6.5	3210	167	332.60	103.61	200.00	398.40	124.11	240.00	478.00	148.91
7.0	3180	171	340.60	107.11	207.00	412.30	129.65	246.00	490.00	154.09
7.5	3150	179	356.50	113.17	212.00	422.30	134.06	248.00	494.00	156.83
8.0	3120	183	366.50	117.47	214.00	426.20	136.60	249.00	496.00	158.97
8.5	3090	185	368.50	119.26	217.00	432.20	139.87	250.00	498.00	161.17
9.0	3060	181	360.50	117.81	220.00	438.40	143.27	251.00	498.00	162.75
9.5	3030				218.00	434.20	143.30	249.00	496.00	163.70
10.0	3000				215.00	428.20	142.73	247.00	492.00	164.00
10.5	2970							245.00	488.00	164.31
11.0	2940							242.00	482.00	163.95
		Maximum shear stress, kPa		**119.26**	*Maximum shear stress, kPa*		**143.30**	*Maximum shear stress, kPa*		**164.31**





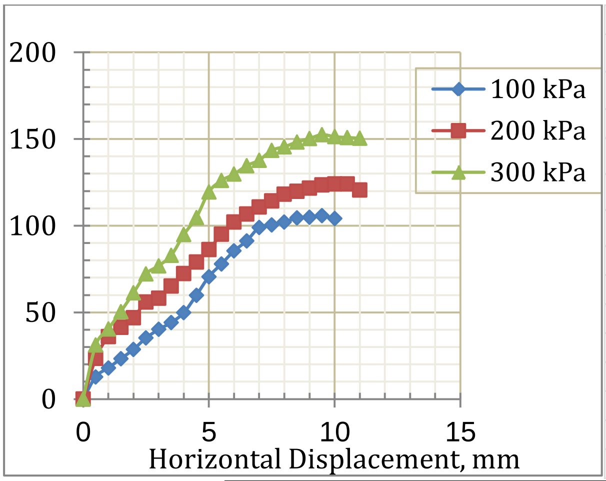
Normal stress(kPa)	100	200	300
Shear strength(kPa)	119.26	143.30	164.31
Parameter			
C (kPa)	97.2		
ϕ	12.68		

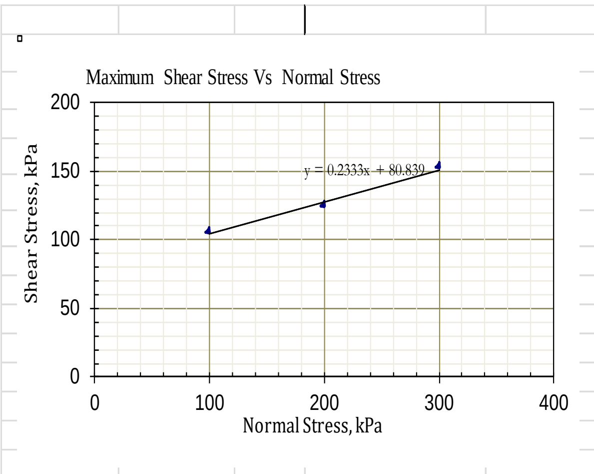
Direct shear test

Standard Reference: ASTM D 3080

Project Name:	Msc. Theses	Date tested	Jun 8 2022		
Sample Description:	TP3	Tested by	Lencho Lemma		
Sample No.	Sample 1	Sample Depth, m:	1.50	Location:	Awaro
Thickness of sample:	20 mm	Rate of strain :	0.1 mm/min	Wet unit weight, kN/M^3:	13.45
Length of sample :	60 mm	Moisture content, %	38.4	Dry Unit Weight, kN/M^3:	11.34
Width of sample:	60 mm			Sample Condition:	Disturbed

		Applied Vertical Stress			Applied Vertical Stress			Applied Vertical Stress		
		100 kPa			**200 kPa**			**300 kPa**		
Horizontal Displacement [mm]	Corrected Area [mm2]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]
0	3600	0	0	0	0	0	0	0	0	0
0.5	3570	23	45.80	12.83	42	83.60	23.42	56	111.5	31.23
1.0	3540	32	63.70	17.99	64.00	127.40	35.99	72.00	143.40	40.51
1.5	3510	41	81.60	23.25	73.00	145.40	41.42	89.00	177.20	50.48
2.0	3480	50	99.60	28.62	82.00	163.30	46.93	107.00	213.10	61.24
2.5	3450	61	121.50	35.22	97.00	193.20	56.00	125.00	249.00	72.17
3.0	3420	69	137.40	40.18	100.00	199.20	58.25	132.00	262.90	76.87
3.5	3390	75	149.40	44.07	111.00	221.10	65.22	141.00	280.80	82.83
4.0	3360	84	167.30	49.79	122.00	243.00	72.32	160.00	318.70	94.85
4.5	3330	100	199.20	59.82	132.00	262.90	78.95	175.00	348.60	104.68
5.0	3300	117	233.00	70.61	143.00	284.80	86.30	198.00	394.40	119.52
5.5	3270	128	254.90	77.95	156.00	310.70	95.02	207.00	412.30	126.09
6.0	3240	139	276.80	85.43	166.00	330.60	102.04	211.00	420.30	129.72
6.5	3210	147	292.80	91.21	172.00	342.60	106.73	217.00	432.20	134.64
7.0	3180	158	314.70	98.96	177.00	352.50	110.85	220.00	438.20	137.80
7.5	3150	159	316.70	100.54	181.00	360.00	114.29	227.00	452.10	143.52
8.0	3120	160	318.70	102.15	185.00	368.50	118.11	228.00	454.10	145.54
8.5	3090	162	322.70	104.43	186.00	370.50	119.90	230.00	458.10	148.25
9.0	3060	161	320.70	104.80	187.00	372.50	121.73	231.00	460.10	150.36
9.5	3030	160	320.70	105.84	188.00	374.50	123.60	232.00	462.10	152.51
10.0	3000	157	312.70	104.23	187.00	372.50	124.17	228.00	454.10	151.37
10.5	2970				185.00	368.50	124.07	225.00	448.20	150.91
11.0	2940				178.00	354.50	120.58	222.00	442.20	150.41
		Maximum shear stress, kPa		**105.84**	*Maximum shear stress, kPa*		**124.17**	*Maximum shear stress, kPa*		**152.51**

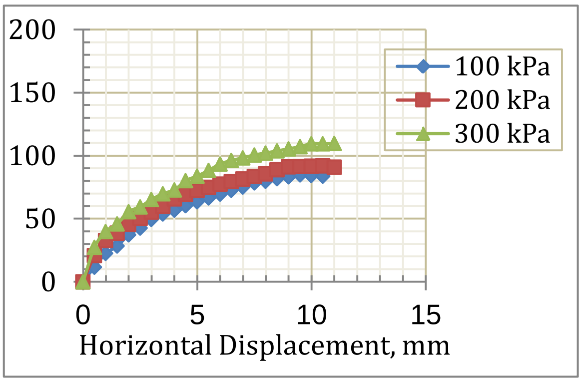


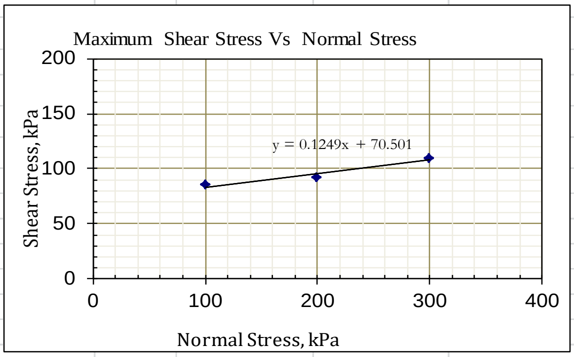


Normal stress(kPa)	100	200	300
Shear strength(kPa)	105.84	124.17	152.51
Parameter			
C (kPa)	80.83		
ϕ	13.11		

Direct shear test							
Standard Reference: ASTM D 3080							
Project Name:	Msc. Theses	Date tested	Jun 8 2022				
Sample Description:	TP4	Tested by	Lencho Lemma				
Sample No.	Sample 1	Sample Depth, m:	2.5		Location:		
Thickness of sample:	20 mm	Rate of strain :		0.1 mm/min	Wet unit weight, kN/M^3:	14.01	
Length of sample :	60 mm	Moisture content, %		34.9	Dry Unit Weight, kN/M^3:	12.07	
Width of sample:	60 mm				Sample Condition:	Disturbed	

		Applied Vertical Stress 100 kPa			Applied Vertical Stress 200 kPa			Applied Vertical Stress 300 kPa		
Horizontal Displacement [mm]	Corrected Area [mm2]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]
0	3600	0	0	0	0	0	0	0	0	0
0.5	3570	21	41.80	11.71	37	73.70	20.64	49	97.6	27.34
1.0	3540	40	79.60	22.49	58.00	115.50	32.63	71.00	141.40	39.94
1.5	3510	50	99.60	28.38	68.00	135.40	38.58	81.00	161.30	45.95
2.0	3480	65	129.40	37.18	80.00	159.30	45.78	97.00	193.20	55.52
2.5	3450	74	147.40	42.72	87.00	173.30	50.23	103.00	205.10	59.45
3.0	3420	85	169.30	49.50	95.00	189.20	55.32	112.00	223.10	65.23
3.5	3390	92	182.30	53.78	102.00	203.10	59.91	119.00	237.00	69.91
4.0	3360	96	191.20	56.90	111.00	221.10	65.80	123.00	245.00	72.92
4.5	3330	101	201.10	60.39	116.00	231.00	69.37	134.00	266.90	80.15
5.0	3300	105	209.10	63.36	120.00	239.00	72.42	139.00	276.80	83.88
5.5	3270	109	217.10	66.39	123.00	245.00	74.92	145.00	288.80	88.32
6.0	3240	113	225.00	69.44	126.00	250.90	77.44	152.00	302.70	93.43
6.5	3210	117	233.00	72.59	128.00	254.90	79.41	155.00	308.70	96.17
7.0	3180	120	239.00	75.16	130.00	258.90	81.42	157.00	312.70	98.33
7.5	3150	124	247.00	78.41	132.00	262.90	83.46	159.00	316.70	100.54
8.0	3120	125	249.00	79.81	134.00	266.90	85.54	160.00	318.70	102.15
8.5	3090	127	252.90	81.84	138.00	274.90	88.96	161.00	320.70	103.79
9.0	3060	128	254.90	83.30	140.00	278.80	91.11	162.00	322.70	105.46
9.5	3030	129	256.90	84.79	139.00	276.80	91.35	163.00	324.70	107.16
10.0	3000	127	252.90	84.30	138.00	274.90	91.63	165.00	328.60	109.53
10.5	2970	125	249.00	83.84	137.00	272.90	91.89	163.00	324.70	109.33
11.0	2940				134.00	266.90	90.78	162.00	322.70	109.76
11.5	2910							160.00	318.70	109.52
		129		**84.79**	*Maximum shear stress, kPa*		**91.89**	*Maximum shear stress, kPa*		**109.76**





Normal stress(kPa)	100	200	300
Shear strength(kPa)	84.79	91.89	109.76
Parameter			
C (kPa)	70.5		
ϕ	7.07		

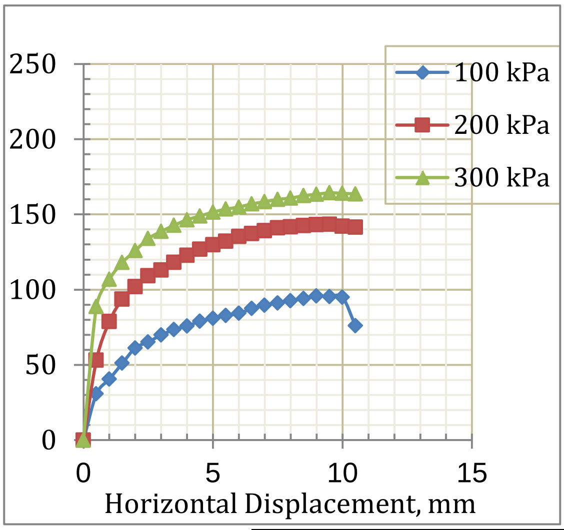
Direct shear test

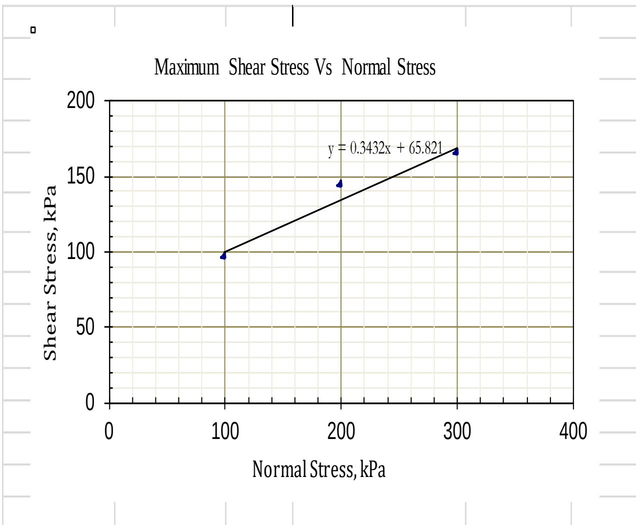
Standard Reference: ASTM D 3080

Project Name:	Msc. Theses	Date tested	Jun 8 2022
Sample Description:	TP5	Tested by	Lencho Lemma

Sample No.	Sample 1	Sample Depth, m:		Location:	
Thickness of sample:	20 mm	Rate of strain :	0.1 mm/min	Wet unit weight, kN/M^3:	14.51
Length of sample :	60 mm	Moisture content, %	23.1	Dry Unit Weight, kN/M^3:	13.02
Width of sample:	60 mm			Sample Condition:	Disturbed

		Applied Vertical Stress			Applied Vertical Stress			Applied Vertical Stress		
		100 kPa			**200 kPa**			**300 kPa**		
Horizontal Displacement [mm]	Corrected Area [mm2]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]
0	3600	0	0	0	0	0	0	0	0	0
0.5	3570	55	109.50	30.67	95	189.20	53.00	159	316.7	88.71
1.0	3540	72	143.40	40.51	140	278.80	78.76	190.00	378.40	106.89
1.5	3510	90	179.20	51.05	165.00	328.60	93.62	208.00	414.30	118.03
2.0	3480	107	213.10	61.24	178.00	354.50	101.87	220.00	438.20	125.92
2.5	3450	113	225.00	65.22	189.00	376.40	109.10	232.00	462.10	133.94
3.0	3420	120	239.00	69.88	194.00	386.40	112.98	238.00	474.10	138.63
3.5	3390	125	249.00	73.45	201.00	400.30	118.08	243.00	484.00	142.77
4.0	3360	128	254.90	75.86	207.00	412.30	122.71	247.00	492.00	146.43
4.5	3330	132	262.90	78.95	212.00	422.30	126.82	249.00	496.00	148.95
5.0	3300	134	266.90	80.88	215.00	428.20	129.76	251.00	500.00	151.52
5.5	3270	136	270.90	82.84	217.00	432.20	132.17	252.00	501.90	153.49
6.0	3240	137	272.90	84.23	220.00	438.20	135.25	252.00	501.90	154.91
6.5	3210	141	280.80	87.48	221.00	440.20	137.13	253.00	503.90	156.98
7.0	3180	143	284.80	89.56	222.00	442.20	139.06	253.00	503.90	158.46
7.5	3150	144	286.80	91.05	223.00	444.20	141.02	253.00	503.90	159.97
8.0	3120	145	288.80	92.56	222.00	442.20	141.73	252.00	501.90	160.87
8.5	3090	146	290.80	94.11	221.00	440.20	142.46	252.00	501.90	162.43
9.0	3060	147	292.90	95.72	220.00	438.20	143.20	251.00	500.00	163.40
9.5	3030	145	288.80	95.31	218.00	434.20	143.30	250.00	498.00	164.36
10.0	3000	143	284.80	94.93	214.00	426.20	142.07	247.00	492.00	164.00
10.5	2970				211.00	420.30	141.52	243.00	486.00	163.64
		Maximum shear stress, kPa		**95.72**	*Maximum shear stress, kPa*		**143.30**	*Maximum shear stress, kPa*		**164.36**





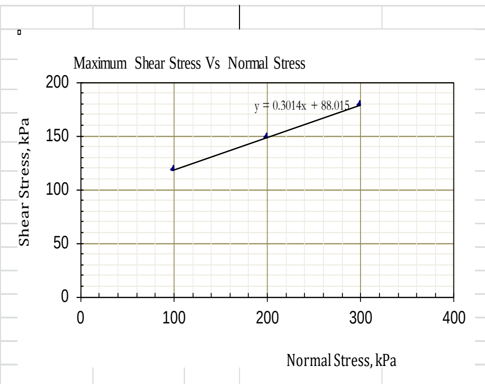
Normal stress(kPa)	100	200	300
Shear strength(kPa)	95.72	143.30	164.36

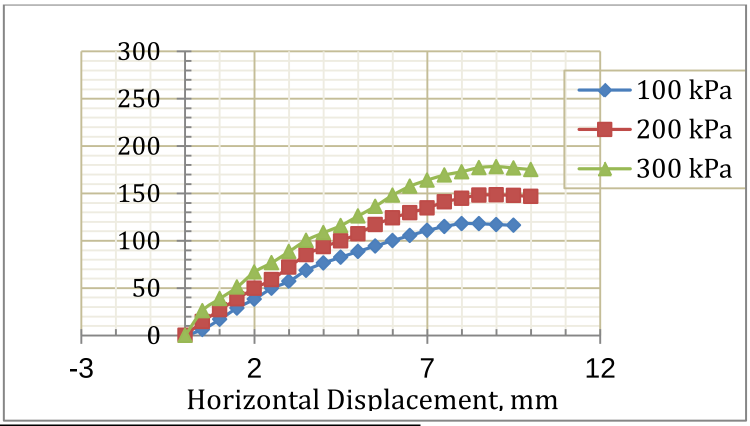
Parameter	
C (kPa)	65.821
ϕ	18.94

Direct shear test			
Standard Reference: ASTM D 3080			
Project Name:	Msc. Theses	Date tested	Jun 8 2022
Sample Description:	TP6	Tested by	Lencho Lemma

Sample No.	Sample 1	Sample Depth, m:	1.50	Location:	Ambo Stadium
Thickness of sample:	20 mm	Rate of strain :	0.1 mm/min	Wet unit weight, kN/M^3:	14.09
Length of sample :	60 mm	Moisture content, %	26.3	Dry Unit Weight, kN/M^3:	12.45
Width of sample:	60 mm			Sample Condition:	Disturbed

		Applied Vertical Stress 100 kPa			Applied Vertical Stress 200 kPa			Applied Vertical Stress 300 kPa		
Horizontal Displacement [mm]	Corrected Area [mm2]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]
0	3600	0	0	0	0	0	0	0	0	0
0.5	3570	11	21.90	6.13	26	51.70	14.48	47	93.6	26.22
1.0	3540	30	59.70	16.86	48.00	95.60	27.01	69.00	137.40	38.81
1.5	3510	51	101.50	28.92	68.00	135.40	38.58	90.00	179.00	51.00
2.0	3480	67	133.40	38.33	87.00	173.30	49.80	117.00	233.00	66.95
2.5	3450	86	171.20	49.62	102.00	203.10	58.87	133.00	264.90	76.78
3.0	3420	98	195.20	57.08	124.00	247.00	72.22	152.00	302.70	88.51
3.5	3390	117	233.00	68.73	145.00	288.80	85.19	171.00	340.60	100.47
4.0	3360	129	256.90	76.46	158.00	314.70	93.66	183.00	364.50	108.48
4.5	3330	138	274.90	82.55	167.00	332.60	99.88	194.00	386.40	116.04
5.0	3300	147	292.80	88.73	178.00	354.50	107.42	209.00	416.30	126.15
5.5	3270	155	308.70	94.40	190.00	382.40	116.94	224.00	446.20	136.45
6.0	3240	163	324.70	100.22	202.00	402.30	124.17	241.00	480.00	148.15
6.5	3210	170	338.60	105.48	209.00	416.30	129.69	254.00	505.90	157.60
7.0	3180	176	352.50	110.85	215.00	428.20	134.65	262.00	521.90	164.12
7.5	3150	182	362.50	115.08	223.00	444.20	141.02	268.00	533.80	169.46
8.0	3120	185	368.50	118.11	227.00	452.10	144.90	271.00	539.80	173.01
8.5	3090	183	364.50	117.96	230.00	458.10	148.25	275.00	547.80	177.28
9.0	3060	180	358.50	117.16	228.00	454.10	148.40	274.00	545.89	178.40
9.5	3030	177	352.50	116.34	225.00	448.20	147.92	269.00	535.80	176.83
10.0	3000				221.00	440.20	146.73	264.00	525.80	175.27
		Maximum shear stress,		118.11	Maximum shear stress, kPa		148.40	Maximum shear stress, kPa		178.40



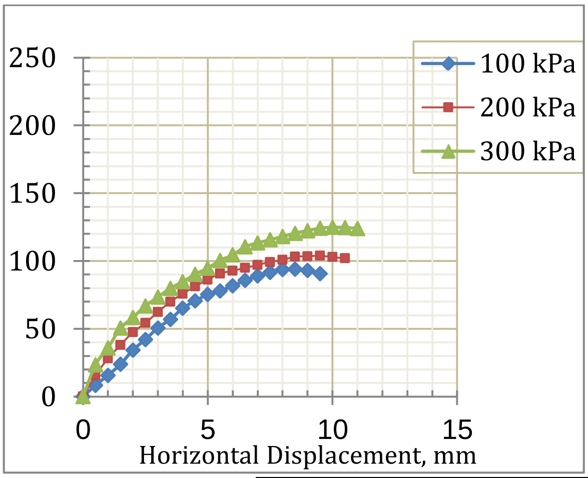


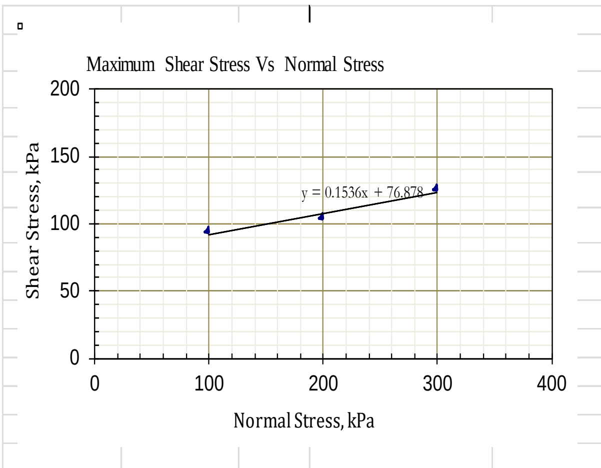
Normal stress(kPa)	100	200	300
Shear strength(kPa)	118.11	148.40	178.40
Parameter			
C (kPa)	88.015		
ϕ	16.77		

Direct shear test			
Standard Reference: ASTM D 3080			
Project Name:	Msc. Theses	Date tested	Jun 8 2023
Sample Description:	TP7A	Tested by	Lencho Lemma

Sample No.	Sample 1	Sample Depth, m:	2.00	Location:	Condominium site
Thickness of sample:	20 mm	Rate of strain :	0.1 mm/min	Wet unit weight, kN/M^3:	14.01
Length of sample :	60 mm	Moisture content, %	36.5	Dry Unit Weight, kN/M^3:	12.26
Width of sample:	60 mm			Sample Condition:	Disturbed

		Applied Vertical Stress			Applied Vertical Stress			Applied Vertical Stress		
		100 kPa			200 kPa			300 kPa		
Horizontal Displacement [mm]	Corrected Area [mm2]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]
0	3600	0	0	0	0	0	0	0	0	0
0.5	3570	15	29.80	8.35	28	55.70	15.60	42	83.9	23.50
1.0	3540	28	55.70	15.73	50.00	99.60	28.14	64.00	127.20	35.93
1.5	3510	42	83.60	23.82	67.00	133.40	38.01	89.00	177.20	50.48
2.0	3480	60	119.50	34.34	83.00	165.30	47.50	102.00	203.10	58.36
2.5	3450	73	145.40	42.14	94.00	187.20	54.26	116.00	231.00	66.96
3.0	3420	87	173.30	50.67	107.00	213.10	62.31	126.00	250.90	73.36
3.5	3390	94	193.20	56.99	119.00	237.00	69.91	136.00	270.90	79.91
4.0	3360	110	219.20	65.24	128.00	254.90	75.86	143.00	284.80	84.76
4.5	3330	118	235.00	70.57	136.00	270.90	81.35	151.00	300.70	90.30
5.0	3300	125	249.00	75.45	143.00	284.80	86.30	157.00	312.70	94.76
5.5	3270	128	254.90	77.95	149.00	296.80	90.76	165.00	328.60	100.49
6.0	3240	133	264.90	81.76	151.00	300.70	92.81	170.00	338.60	104.51
6.5	3210	138	274.90	85.64	153.00	304.70	94.92	178.00	354.50	110.44
7.0	3180	142	282.80	88.93	155.00	308.70	97.08	181.00	360.50	113.36
7.5	3150	145	288.80	91.68	157.00	312.70	99.27	183.00	364.50	115.71
8.0	3120	147	292.80	93.85	158.00	314.70	100.87	184.00	368.50	118.11
8.5	3090	146	290.80	94.11	160.00	318.70	103.14	187.00	372.50	120.55
9.0	3060	143	284.80	93.07	159.00	316.70	103.50	188.00	374.50	122.39
9.5	3030	138	274.90	90.73	158.00	314.70	103.86	189.00	376.40	124.22
10.0	3000				155.00	308.70	102.90	188.00	374.50	124.83
10.5	2970				152.00	302.70	101.92	186.00	370.50	124.75
11.0	2940							183.00	364.50	123.98
		Maximum shear stress, kPa		**94.11**	*Maximum shear stress, kPa*		**103.86**	*Maximum shear stress, kPa*		**124.83**

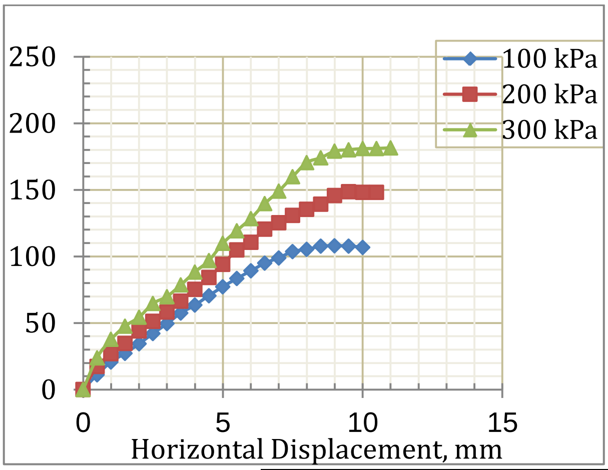


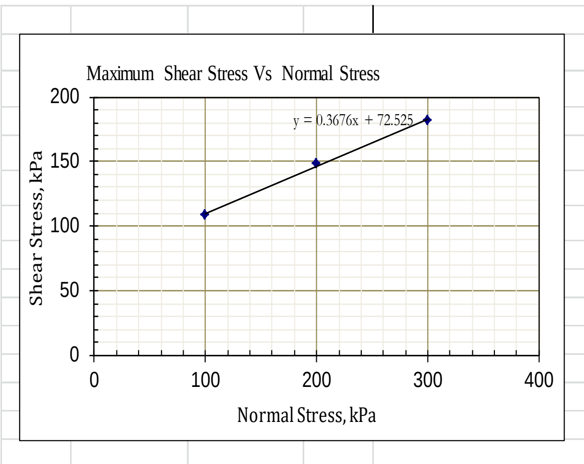


Normal stress(kPa)	100	200	300
Shear strength(kPa)	94.11	103.86	124.83
Parameter			
C (kPa)	76.8		
ϕ	8.70		

Direct shear test					
Standard Reference: ASTM D 3080					
Project Name:	Msc. Thesis	Date tested	Jun 8 2022		
Sample Description:	TP7B	Tested by	Lencho Lemma		
Sample No.	Sample 1	Sample Depth, m:	3.00	Location:	Condominium Site
Thickness of sample:	20 mm	Rate of strain :	0.1 mm/min	Wet unit weight, kN/M^3:	12.83
Length of sample :	60 mm	Moisture content, %	30.3	Dry Unit Weight, kN/M^3:	11.27
Width of sample:	60 mm			Sample Condition:	Disturbed

		Applied Vertical Stress 100 kPa			Applied Vertical Stress 200 kPa			Applied Vertical Stress 300 kPa		
Horizontal Displacement [mm]	Corrected Area [mm2]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]
0	3600	0	0	0	0	0	0	0	0	0
0.5	3570	20	39.80	11.15	31	61.70	17.28	43	85.6	23.98
1.0	3540	37	73.70	20.82	48.00	95.60	27.01	67.00	133.40	37.68
1.5	3510	48	95.60	27.24	61.00	121.50	34.62	84.00	167.30	47.66
2.0	3480	60	119.50	34.34	77.00	153.30	44.05	95.00	189.20	54.37
2.5	3450	73	145.40	42.14	89.00	177.20	51.36	112.00	223.00	64.64
3.0	3420	85	169.30	49.50	100.00	199.20	58.25	120.00	239.00	69.88
3.5	3390	98	195.20	57.58	113.00	225.00	66.37	134.00	266.90	78.73
4.0	3360	107	213.10	63.42	127.00	252.90	75.27	149.00	296.80	88.33
4.5	3330	118	235.00	70.57	141.00	280.20	84.14	162.00	322.70	96.91
5.0	3300	128	254.90	77.24	156.00	310.70	94.15	182.00	362.50	109.85
5.5	3270	137	272.90	83.46	172.00	342.60	104.77	196.00	390.40	119.39
6.0	3240	145	288.80	89.14	180.00	358.50	110.65	209.00	416.30	128.49
6.5	3210	153	304.70	94.92	194.00	386.40	120.37	225.00	448.20	139.63
7.0	3180	158	314.70	98.96	200.00	398.40	125.28	238.00	474.10	149.09
7.5	3150	164	326.60	103.68	207.00	412.30	130.89	252.00	503.90	159.97
8.0	3120	165	328.60	105.32	212.00	422.30	135.35	267.00	531.80	170.45
8.5	3090	167	332.60	107.64	216.00	430.20	139.22	270.00	537.80	174.05
9.0	3060	166	330.60	108.04	224.00	446.20	145.82	275.00	547.80	179.02
9.5	3030	164	326.60	107.79	226.00	450.10	148.55	274.00	545.80	180.13
10.0	3000	161	320.70	106.90	223.00	444.20	148.07	273.00	542.80	180.93
10.5	2970				221.00	440.20	148.22	270.00	537.80	181.08
11.0	2940							268.00	533.80	181.56
		Maximum shear stress, kPa		**108.04**	*Maximum shear stress, kPa*		**148.55**	*Maximum shear stress, kPa*		**181.56**





Normal stress(kPa)	100	200	300
Shear strength(kPa)	108.04	148.55	181.56
Parameter			
C (kPa)	72.525		
ϕ	20.18		

Appendix- H, Hydrometer Test Results and Percentage Pass (%) Vs Particle Size (mm) Graphs for Sample (TP1-TP4 and TP6-TP7A)

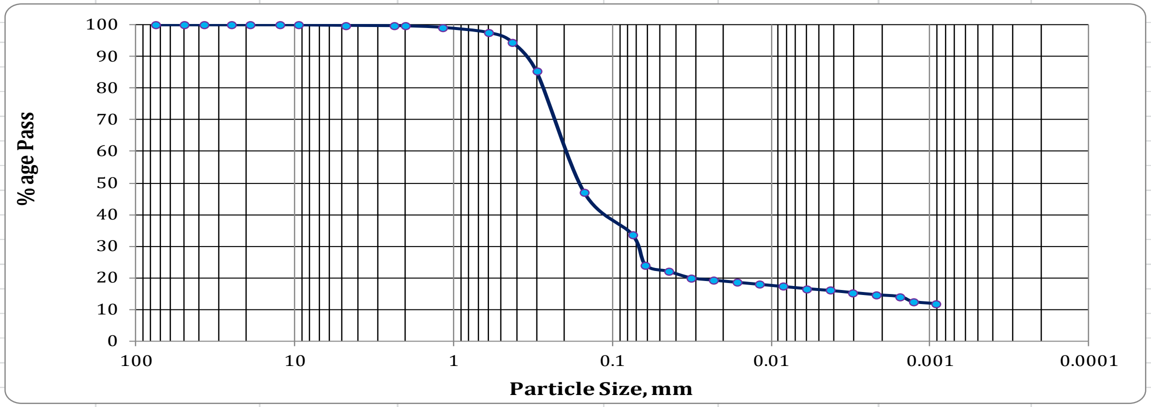
Grain Size Analysis of soil ASTM D 422

Wt of Dry Soil (Ws) : =	**50 g**	**Specific Gravity: = 2.68**	**Correction Factor (a):=**	**1.008**	**Sample number:-TP1**
Mensicus Correction : =	**-1**		**Zero Correctio: =**	**-1**	

Time of Reading	**Elapsed Time in (min)**	**Tempera ture (^0C)**	**Temp. Corr. (CT)**	**Actual Hyd. Reading (Ra)**	**Corr.Hydr. Reading (Rc)**	**Effe. Depth of Hydr. L (cm), from table**	**Values of K**	**Diameter of soil Particle D(mm)**	**% finer insuspension [P]**	**Adjusted Percen.Finner(%) PA**
9:00	0.5	22	0.40	**33**	35	10.9	0.01322	0.061724801	71.3664	24.10043328
9:00:30	1	22	0.40	**31**	32	11.2	0.01322	0.044242582	65.3184	22.05802368
9:01:00	2	22	0.40	28	29	11.7	0.01322	0.031974914	59.2704	20.01561408
9:02:00	4	21.5	0.30	27	28	11.9	0.0133	0.02294009	57.0528	19.26673056
9:08:00	8	21.5	0.30	26	27	12	0.0133	0.016289107	55.0368	18.58592736
9:15:00	15	22	0.40	25	26	12.2	0.01322	0.011922456	53.2224	17.97320448
9:30:00	30	22	0.40	24	25	12.4	0.01322	0.008499271	51.2064	17.29240128
10:00:00	60	22	0.40	23	24	12.5	0.01322	0.006034077	49.1904	16.61159808
11:00:00	120	22.5	0.55	22	24	12.7	0.01317	0.004284469	47.4768	16.03291536
13:00:00	240	22.5	0.55	21	23	12.9	0.01317	0.003053339	45.4608	15.35211216
After 8hr	480	22.5	0.55	20	22	13	0.01317	0.002167389	43.4448	14.67130896
After 16hr	960	23	0.70	19	21	13.2	0.01312	0.001538456	41.7312	14.09262624
After 24hr	1440	22	0.40	17	18	13.5	0.01322	0.001280021	37.0944	12.52677888
After 48hr	2880	22	0.40	16	17	13.7	0.01322	0.000911791	35.0784	11.84597568

Combined Data

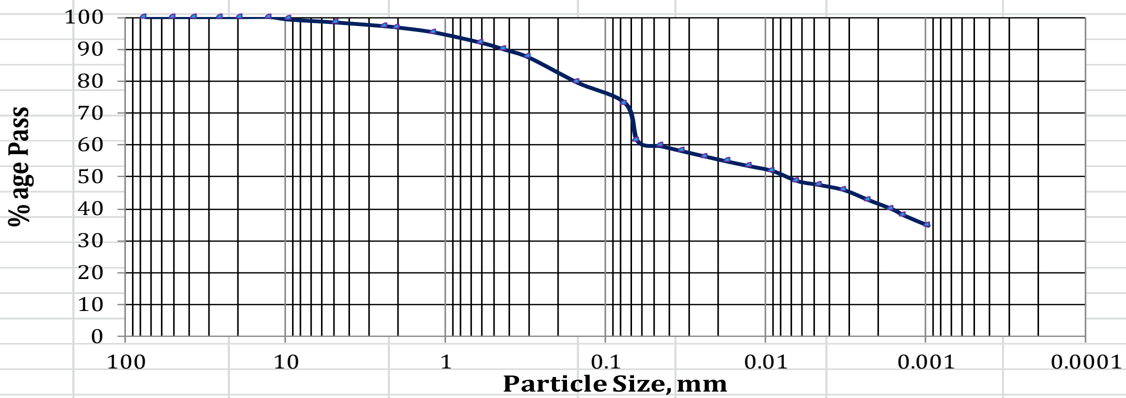
Dia(mm)	**%Finner**
75	100
50	100
37	100
25	100
19	100
12.5	100
9.5	100
4.75	99.83
2.36	99.74
2	99.68
1.18	99.2
0.6	97.57
0.425	94.5
0.3	85.29
0.15	**46.98**
0.0750	33.7700
0.0617	24.1004
0.0442	22.0580
0.0320	20.0156
0.0229	19.2667
0.0163	18.5859
0.0119	17.9732
0.0085	17.2924
0.0060	16.6116
0.0043	16.0329
0.0031	15.3521
0.0022	14.6713
0.0015	14.0926
0.0013	12.5268
0.0009	11.8460



No	**Summery (combined)**	**%**
1	Particles larger than (4.75**mm)**	0.17
2	Coarse Sand (4.75**mm - 0.425mm)**	5.33
3	Fine Sand (**0.425mm - 0.075mm)**	60.73
4	Silt (**0.075-0.002mm)**	33.41
5	Clay smaller than (**0.002mm**)	0.36

Soil (Ws)	50 g	Specific Gravity: = 2.49	Correction Factor (a):=	1.008	
Mensicus Correction : =	1		Zero Correctio: =	-1	Sample number:-TP2

Time of Reading	Elapsed Time in (min)	Temperature (^0C)	Temp. Corr. (CT)	Actual Hyd. Reading (Ra)	Corr.Hydr.Reading (Rc)	Effe. Depth of Hydr. L (cm), from table	Values of K	Diameter of soil Particle D(mm)	% finer insuspension [P]	Adjusted Percen.Finner(%) PA	Combined Data Dia(mm)	Combined Data %Finner
9:00	0.5	22	0.40	38	39	10.1	0.014	0.062922174	79.4304	57.92859072	75	100
9:00:30	1	22	0.40	37	38	10.2	0.014	0.044712414	77.4144	56.45832192	50	100
9:01:00	2	21.5	0.40	36	37	10.4	0.0145	0.033065087	75.3984	54.98805312	37	100
9:02:00	4	21.5	0.30	35	36	10.5	0.0145	0.023492685	73.1808	53.37075744	25	100
9:08:00	8	22	0.30	34	35	12	0.014	0.017146428	71.1648	51.90048864	19	100
9:15:00	15	22	0.40	33	34	12.2	0.014	0.012625899	69.3504	50.57724672	12.5	100
9:30:00	30	22	0.40	32	33	12.4	0.014	0.009000741	67.3344	49.10697792	9.5	99.24
10:00:00	60	22.5	0.40	30	31	12.5	0.0139	0.006344453	63.3024	46.16644032	4.75	98.31
11:00:00	120	22.5	0.55	29	31	12.7	0.0139	0.004521953	61.5888	44.91671184	2.36	97.13
13:00:00	240	22.5	0.55	28	30	12.9	0.0139	0.003222582	59.5728	43.44644304	2	96.77
After 8hr	480	23	0.55	26	28	13	0.0138	0.002271068	55.5408	40.50590544	1.18	95.22
After 16hr	960	23	0.70	24	26	13.2	0.0138	0.001618193	51.8112	37.78590816	0.6	91.98
After 24hr	1440	22	0.40	23	24	13.5	0.014	0.001355544	49.1904	35.87455872	0.425	89.79
After 48hr	2880	22	0.40	21	22	13.7	0.014	0.000965588	45.1584	32.93402112	0.3	87.3
											0.15	**79.51**
											0.0750	72.9300
											0.0629	61.1058
											0.0447	59.5549
											0.0331	58.0040
											0.0235	56.2980
											0.0171	54.7471
											0.0126	53.3513
											0.0090	51.8004
											0.0063	48.6985
											0.0045	47.3803
											0.0032	45.8294
											0.0023	42.7275
											0.0016	39.8584
											0.0014	37.8422
											0.0010	34.7404



No	Summery (combined)	%
1	Particles larger than (4.75**mm)**	1.69
2	Coarse Sand (4.75**mm - 0.425mm)**	8.52
3	Fine Sand (**0.425mm - 0.075mm)**	16.86
4	Silt (**0.075-0.002mm)**	38.46
5	Clay smaller than (**0.002mm**)	34.47

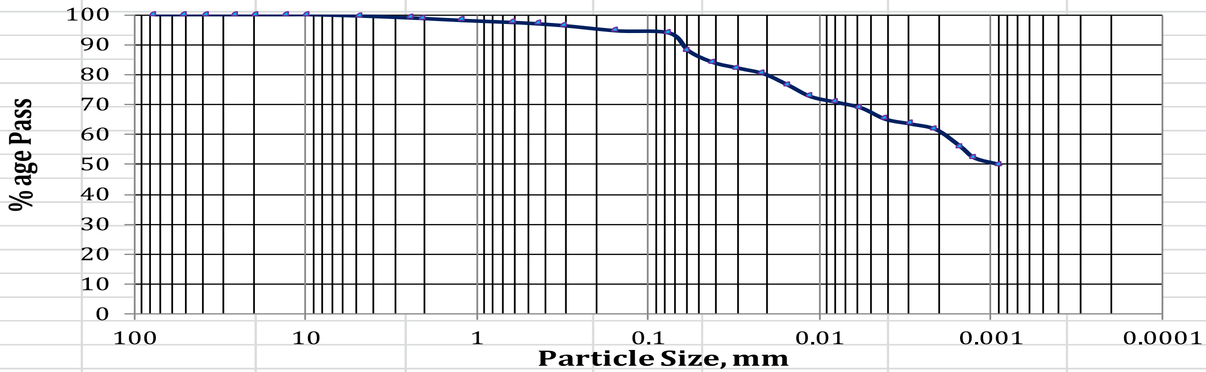
Dry Soil	50 g	Specific Gravity: = 2.6	Correction Factor (a):=	1.008
Mensicus Correction : =	-1		Zero Correctio: =	-1

Sample number:-TP3

Time of Reading	Elapsed Time in (min)	Temperature (^0C)	Temp. Corr. (CT)	Actual Hyd. Reading (Ra)	Corr.Hydr. Reading (Rc)	Effe. Depth of Hydr. L (cm), from table	Values of K	Diameter of soil Particle D(mm)	% finer insuspension [P]	Adjusted Percen.Finner(%) PA
9:00	0.5	22	0.40	44	46	9.1	0.0135	0.057592968	93.5424	87.78018816
9:00:30	1	22	0.40	42	44	9.4	0.0135	0.041390216	89.5104	83.99655936
9:01:00	2	22	0.40	41	43	9.6	0.0135	0.029577018	87.4944	82.10474496
9:02:00	4	22	0.40	40	42	9.7	0.0135	0.021022756	85.4784	80.21293056
9:08:00	8	22	0.40	38	40	10.1	0.0135	0.015168738	81.4464	76.42930176
9:15:00	15	22	0.40	36	38	10.4	0.0135	0.011240996	77.4144	72.64567296
9:30:00	30	22	0.40	35	37	10.5	0.0135	0.007986708	75.3984	70.75385856
10:00:00	60	22	0.40	34	36	10.7	0.0135	0.005700987	73.3824	68.86204416
11:00:00	120	22	0.40	32	34	11.1	0.0135	0.004105865	69.3504	65.07841536
13:00:00	240	22.5	0.55	31	34	11.2	0.01345	0.002905532	67.6368	63.47037312
After 8hr	480	22.5	0.55	30	33	11.4	0.01345	0.002072784	65.6208	61.57855872
After 16hr	960	22.5	0.55	27	30	11.9	0.01345	0.001497477	59.5728	55.90311552
After 24hr	1440	22.5	0.55	25	28	12.2	0.01345	0.001238001	55.5408	52.11948672
After 48hr	2880	22	0.40	24	26	12.4	0.0135	0.000885826	53.2224	49.94390016

Combined Data

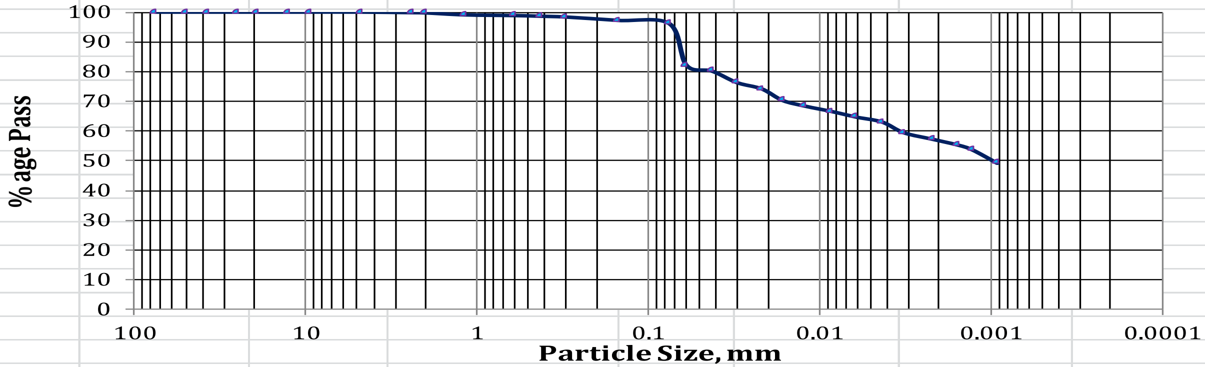
Dia(mm)	%Finner
75	100
50	100
37	100
25	100
19	100
12.5	100
9.5	100
4.75	99.55
2.36	98.87
2	98.68
1.18	98.03
0.6	97.36
0.425	96.87
0.3	96.24
0.15	**94.58**
0.0750	93.8400
0.0576	87.7802
0.0414	83.9966
0.0296	82.1047
0.0210	80.2129
0.0152	76.4293
0.0112	72.6457
0.0080	70.7539
0.0057	68.8620
0.0041	65.0784
0.0029	63.4704
0.0021	61.5786
0.0015	55.9031
0.0012	52.1195
0.0009	49.9439



No	Summery (combined)	%
1	Particles larger than (4.75**mm)**	0.45
2	Coarse Sand (4.75**mm - 0.425mm)**	2.68
3	Fine Sand (**0.425mm - 0.075mm)**	3.03
4	Silt (**0.075-0.002mm)**	38.28
5	Clay smaller than (**0.002mm**)	55.56

Dry Soil	50 g	Specific Gravity: = 2.65	Correction Factor (a):=	1.008	**Sample number:-TP4**
Mensicus Correction :	-1		Zero Correctio: =	-1	

Time of Reading	Elapsed Time in (min)	Temperature (^0C)	Temp. Corr. (CT)	Actual Hyd. Reading (Ra)	Corr.Hydr. Reading (Rc)	Effe. Depth of Hydr. L (cm), from table	Values of K	Diameter of soil Particle D(mm)	% finer insuspension [P]	Adjusted Percen.Finner(%) PA	**Combined Data** Dia(mm)	%Finner
9:00	0.5	22	0.40	40	42	10.1	0.0133	0.059776065	85.4784	82.1020032	75	100
9:00:30	1	22	0.40	39	41	10.2	0.0133	0.042476794	83.4624	80.1656352	50	100
9:01:00	2	22	0.40	37	39	10.4	0.0133	0.030328666	79.4304	76.2928992	37	100
9:02:00	4	21.5	0.30	36	38	10.5	0.0134	0.021710481	77.2128	74.1628944	25	100
9:08:00	8	21.5	0.30	34	36	12	0.0134	0.016411581	73.1808	70.2901584	19	100
9:15:00	15	21.5	0.30	33	35	12.2	0.0134	0.012084789	71.1648	68.3537904	12.5	100
9:30:00	30	22	0.40	32	34	12.4	0.0133	0.008550704	69.3504	66.6110592	9.5	100
10:00:00	60	22	0.40	31	33	12.5	0.0133	0.006070592	67.3344	64.6746912	4.75	100
11:00:00	120	22.5	0.55	30	33	12.7	0.01325	0.004310495	65.6208	63.0287784	2.36	99.81
13:00:00	240	23	0.70	28	31	12.9	0.0139	0.003222582	61.8912	59.4464976	2	99.74
After 8hr	480	22.5	0.55	27	30	13	0.01325	0.002180554	59.5728	57.2196744	1.18	99.07
After 16h	960	22.5	0.55	26	29	13.2	0.01325	0.0015537	57.5568	55.2833064	0.6	98.83
After 24h	1440	23	0.70	25	28	13.5	0.0132	0.001278085	55.8432	53.6373936	0.425	98.61
After 48h	2880	22	0.40	24	25	13.7	0.0133	0.000917309	51.2064	49.1837472	0.3	98.33
											0.15	**97.21**
											0.0750	96.0500
											0.0598	82.1020
											0.0425	80.1656
											0.0303	76.2929
											0.0217	74.1629
											0.0164	70.2902
											0.0121	68.3538
											0.0086	66.6111
											0.0061	64.6747
											0.0043	63.0288
											0.0032	59.4465
											0.0022	57.2197
											0.0016	55.2833
											0.0013	53.6374
											0.0009	49.1837

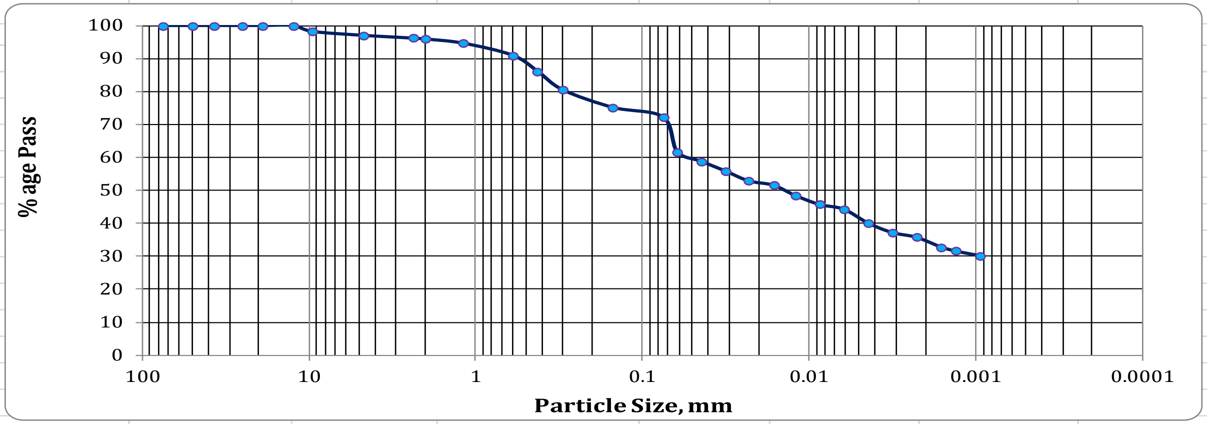


No	Summery (combined)	%
1	Particles larger than (4.75**mm)**	0.00
2	Coarse Sand (4.75**mm - 0.425mm)**	1.39
3	Fine Sand (**0.425mm - 0.075mm)**	2.56
4	Silt (**0.075-0.002mm)**	40.21
5	Clay smaller than (**0.002mm**)	55.84

Wt of		50 g	Specific Gravity: = 2.5	Correction Factor (a):=	1.008	
Mensicus Correction :		-1		Zero Correctio: =	-1	Sample number:-TP6

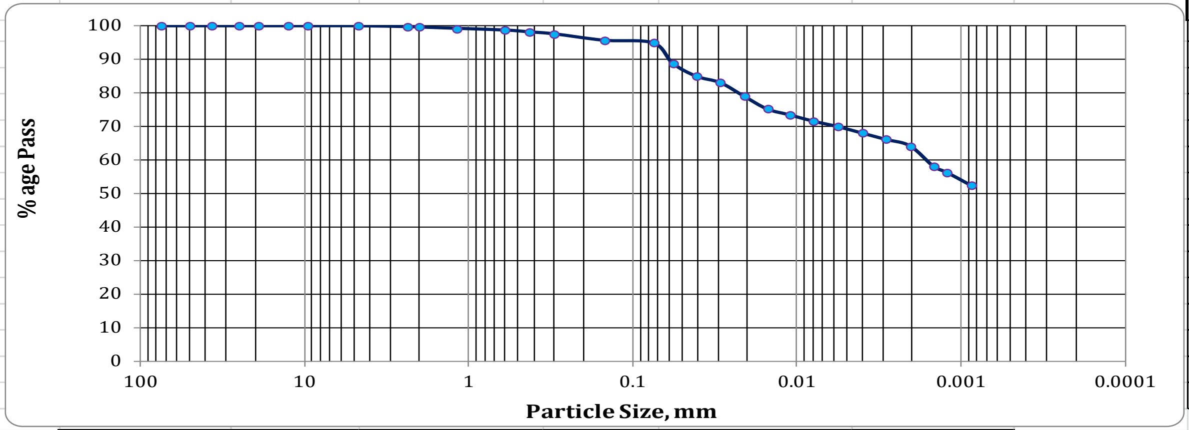
											Combined Data	
Time of Reading	**Elapsed Time in (min)**	**Temperature (^0C)**	**Temp. Corr. (CT)**	**Actual Hyd. Reading (Ra)**	**Corr.Hydr. Reading (Rc)**	**Effe. Depth of Hydr. L (cm), from table**	**Values of K**	**Diameter of soil Particle D(mm)**	**% finer insuspension [P]**	**Adjusted Percen.Finner(%) PA**	**Dia(mm)**	**%Finner**
9:00	0.5	22	0.40	**40**	42	9.7	0.014	0.0616636	85.4784	61.758144	75	100
9:00:30	1	22	0.40	**38**	40	10.1	0.014	0.0444927	81.4464	58.845024	50	100
9:01:00	2	22	0.40	36	38	10.4	0.014	0.03192491	77.4144	55.931904	37	100
9:02:00	4	21.5	0.30	34	36	10.7	0.01405	0.02297938	73.1808	52.873128	25	100
9:08:00	8	22	0.40	33	35	10.9	0.014	0.01634166	71.3664	51.562224	19	100
9:15:00	15	21.5	0.30	31	33	11.2	0.01405	0.01214059	67.1328	48.503448	12.5	100
9:30:00	30	22	0.40	29	31	11.5	0.014	0.00866795	63.3024	45.735984	9.5	98.19
10:00:00	60	22	0.40	28	30	11.7	0.014	0.00618223	61.2864	44.279424	4.75	97.02
11:00:00	120	22.5	0.55	25	28	12.2	0.0139	0.00443204	55.5408	40.128228	2.36	96.16
13:00:00	240	22.5	0.55	23	26	12.5	0.0139	0.00317223	51.5088	37.215108	2	95.9
After 8hr	480	22.5	0.55	22	25	12.7	0.0139	0.00226098	49.4928	35.758548	1.18	94.62
After 16hr	960	22.5	0.55	20	23	13	0.0139	0.00161752	45.4608	32.845428	0.6	90.99
After 24hr	1440	23	0.70	19	22	13.2	0.0138	0.00132125	43.7472	31.607352	0.425	86.16
After 48hr	2880	23	0.70	18	21	13.3	0.0138	0.0009378	41.7312	30.150792	0.3	80.51

Dia(mm)	%Finner
0.15	**75.16**
0.0750	72.2500
0.0617	61.7581
0.0445	58.8450
0.0319	55.9319
0.0230	52.8731
0.0163	51.5622
0.0121	48.5034
0.0087	45.7360
0.0062	44.2794
0.0044	40.1282
0.0032	37.2151
0.0023	35.7585
0.0016	32.8454
0.0013	31.6074
0.0009	30.1508



No	Summery (combined)	%
1	Particles larger than (4.75**mm)**	2.98
2	Coarse Sand (4.75**mm - 0.425mm)**	10.86
3	Fine Sand (**0.425mm - 0.075mm)**	13.91
4	Silt (**0.075-0.002mm)**	41.13
5	Clay smaller than (**0.002mm**)	31.12

Wt of Dry		50 g	Specific Gravity: = 2.6		Correction Factor (a):=		1.008					
Mensicus Correction : =		1			Zero Correctio: =		-1	Sample number:-TP7A				
											Combined Data	
Time of Reading	Elapsed Time in (min)	Temperature (^0C)	Temp. Corr. (CT)	Actual Hyd. Reading (Ra)	Corr.Hydr. Reading (Rc)	Effe. Depth of Hydr. L (cm), from table	Values of K	Diameter of soil Particle D(mm)	% finer insuspension [P]	Adjusted Percen.Finner (%) PA	Dia(mm)	%Finner
9:00	0.5	22	0.40	**45**	46	8.9	0.0135	0.056956562	93.5424	88.76238336	75	100
9:00:30	1	22	0.40	**43**	44	9.2	0.0135	0.040947527	89.5104	84.93641856	50	100
9:01:00	2	22	0.40	42	43	9.4	0.0135	0.029267303	87.4944	83.02343616	37	100
9:02:00	4	22	0.30	40	41	9.7	0.0135	0.021022756	83.2608	79.00617312	25	100
9:08:00	8	22	0.30	38	39	10.1	0.0135	0.015168738	79.2288	75.18020832	19	100
9:15:00	15	22	0.40	37	38	10.2	0.0135	0.011132385	77.4144	73.45852416	12.5	100
9:30:00	30	22	0.40	36	37	10.4	0.0135	0.007948585	75.3984	71.54554176	9.5	100
10:00:00	60	22.5	0.55	35	37	10.5	0.01345	0.005626539	73.6848	69.91950672	4.75	100
11:00:00	120	22.5	0.55	34	36	10.7	0.01345	0.004016276	71.6688	68.00652432	2.36	99.75
13:00:00	240	22.5	0.55	33	35	10.9	0.01345	0.002866355	69.6528	66.09354192	2	99.63
After 8hr	480	22.5	0.55	32	34	11.1	0.01345	0.002045329	67.6368	64.18055952	1.18	99.24
After 16hr	960	22	0.40	29	30	11.5	0.0135	0.001477567	61.2864	58.15466496	0.6	98.73
After 24hr	1440	22	0.40	28	29	11.7	0.0135	0.001216874	59.2704	56.24168256	0.425	98.19
After 48hr	2880	22	0.40	26	27	12	0.0135	0.000871421	55.2384	52.41571776	0.3	97.59
											0.15	**95.69**
											0.0750	94.8900
											0.0570	88.7624
											0.0409	84.9364
											0.0293	83.0234
											0.0210	79.0062
											0.0152	75.1802
											0.0111	73.4585
											0.0079	71.5455
											0.0056	69.9195
											0.0040	68.0065
											0.0029	66.0935
											0.0020	64.1806
											0.0015	58.1547
											0.0012	56.2417
											0.0009	52.4157



No	Summery (combined)	%
1	Particles larger than (4.75**mm)**	0.00
2	Coarse Sand (4.75**mm - 0.425mm)**	1.81
3	Fine Sand (**0.425mm - 0.075mm)**	3.30
4	Silt (**0.075-0.002mm)**	37.05
5	Clay smaller than (**0.002mm**)	57.84

Appendix I specific gravity and water absorption (%) Tests of Rock sample (RS1-RS5)

Specific Gravity and water absorption Test of Rock		
Rock Sample one (RS-1)		
No	**Description of the steps**	**Calculated results**
A	weight of basket in air (gm)	345.2
B	weight of basket in water (gm)	322.6
1	weight of SSD sample +basket in air(gm)	2106.9
2	weight of sample + basket in water(gm)	1278.2
3	weight of oven dried sample in air(gm)	1646.6
4	% Absorption = $\frac{1-A-3}{3}$ *100	7
5	Bulk specific gravity, (SSD) = $\frac{1-A}{(1-A)-(2-B)}$	2.185
6	Bulk specific gravity = $\frac{3}{(1-A)-(2-B)}$	2.04
7	Apparent specific gravity = $\frac{3}{3-(2-B)}$	2.38

Specific Gravity and water absorption Test of Rock		
Rock Sample one (RS-2)		
No	**Description of the steps**	**Calculated results**
A	weight of basket in air (gm)	345.2
B	weight of basket in water (gm)	322.6
1	weight of SSD sample +basket in air(gm)	2217.4
2	weight of sample + basket in water(gm)	1327.1
3	weight of oven dried sample in air(gm)	1743.5
4	% Absorption = $\frac{1-A-3}{3}$ *100	7.38
5	Bulk specific gravity, (SSD) = $\frac{1-A}{(1-A)-(2-B)}$	2.14
6	Bulk specific gravity = $\frac{3}{(1-A)-(2-B)}$	2
7	Apparent specific gravity = $\frac{3}{3-(2-B)}$	2.34

Specific Gravity and water absorption Test of Rock		
Rock Sample one (RS-3)		
No	**Description of the steps**	**Calculated results**
A	weight of basket in air (gm)	345.2
B	weight of basket in water (gm)	322.6
1	weight of SSD sample +basket in air(gm)	2443.1
2	weight of sample + basket in water(gm)	1668.3
3	weight of oven dried sample in air(gm)	2073.1
4	% Absorption = $\frac{1-A-3}{3}$ *100	1.2
5	Bulk specific gravity, (SSD) = $\frac{1-A}{(1-A)-(2-B)}$	2.79
6	Bulk specific gravity = $\frac{3}{(1-A)-(2-B)}$	2.76
7	Apparent specific gravity = $\frac{3}{3-(2-B)}$	2.85
Specific Gravity and water absorption Test of Rock		
Rock Sample one (RS-4)		
No	**Description of the steps**	**Calculated results**
A	weight of basket in air (gm)	345.2
B	weight of basket in water (gm)	322.6
1	weight of SSD sample +basket in air(gm)	2584.6
2	weight of sample + basket in water(gm)	1796.5
3	weight of oven dried sample in air(gm)	2218.2
4	% Absorption = $\frac{1-A-3}{3}$ *100	0.997
5	Bulk specific gravity, (SSD) = $\frac{1-A}{(1-A)-(2-B)}$	2.93
6	Bulk specific gravity = $\frac{3}{(1-A)-(2-B)}$	2.9
7	Apparent specific gravity = $\frac{3}{3-(2-B)}$	2.98

Specific Gravity and water absorption Test of Rock		
Rock Sample one (RS-5)		
No	**Description of the steps**	**Calculated results**
A	weight of basket in air (gm)	345.2
B	weight of basket in water (gm)	322.6
1	weight of SSD sample +basket in air(gm)	1996.2
2	weight of sample + basket in water(gm)	1101.7
3	weight of oven dried sample in air(gm)	1514.3
4	% Absorption = $\frac{1-A-3}{3}$ *100	9.03
5	Bulk specific gravity, (SSD) = $\frac{1-A}{(1-A)-(2-B)}$	1.9
6	Bulk specific gravity = $\frac{3}{(1-A)-(2-B)}$	1.74
7	Apparent specific gravity = $\frac{3}{3-(2-B)}$	2.06