Engineering geological Characterization of Aleta Wondo town, Southern Ethiopia: implication to engineering practice

Deginet Tesfaye Youkamo

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

August, 2021

Adama, Ethiopia

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Declaration

I hereby, declare that this Master Thesis entitled "**Engineering geological Characterization of Aleta Wondo town, Southern Ethiopia: implication to engineering practice**" is my original work. That is, it has not been submitted for the award of any academic degree, diploma, or certificate in any other university. All sources of materials that are used for this thesis have been duly acknowledged through citation

Name of the Student	Signature	Date

Recommendations

we, the advisor(s) of this thesis, hereby certify that I/we have read the revised version of the thesis entitled "**Engineering geological Characterization of Aleta Wondo town, Southern Ethiopia: implication to engineering practice**" prepared under my/our guidance by Deginet Tesfaye submitted in partial fulfillment of the requirements for the degree of Mater's of Science in Applied Geology (Specialization in Engineering Geology).. Therefore, I/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 of Boarder Examiners

We, the advisors of the thesis entitled "**Engineering geological Characterization of Aleta Wondo town, Southern Ethiopia**: implication to engineering practice" and developed by Deginet Tesfaye, 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 Deginet Tesfaye have read and evaluated the thesis entitled "Engineering geological appraisal of Aleta Wondo town, Southern Ethiopia: implication to engineering practice" and examined the candidate during 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 (Specialization in Engineering Geology).

Chairperson	Signature	Date
Internal Examiner	Signature	Date
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Final approval and acceptance of the thesis is contingent upon 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, Dean	Signature	Date

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List of Symbols and Abbreviation

AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing Materials standard
BSS	British Standard System
CH	Sandy fat clay
CCS	Continuous slickenside
CD	Condition of discontinuity
Dr	Dripping
E_o	Initial void ratio
GWC	Ground water condition
Gs	Specific Gravity
HW	Highly weathered
JS	Joint space
Jv	Volumetric joint count
LL	Liquid Limit
PI	Plastic Index
PL	Plastic Limit
LI	Liquidity Index
MER	Mean Ethiopian Rift
MH	Sandy elastic silt
NMC	Natural Moisture Content
NNW	North North West
RQD	Rock Quality Designation
RMR	Rock Mass Rating
RN	Rebound N-type Schmidt hammer value
SW	Slightly weathered
SR	Slightly rough
SSE	South South East
TP	Test Pit
USCS	Unified Soil Classification System
UCS	Uniaxial Compressive Strength
UTM	Universal Transverse Mercator
W	Water content

ABSTRACT

Site characterization is a prerequisite for the successful and economic design of engineering structures and earthworks. It is also necessary to obtain sufficient information on the type, characteristics, and distributions of rock and soil underlying at a site of proposed structures for feasibility and economic studies for the proposed project. This study was conducted to obtain geotechnical information of ground at Aleta Wondo Town. Aleta Wondo Town found in Sidama regional state. The main research goal is to characterize the engineering geological and geotechnical properties of soils and rocks and develop a detailed multi-purpose engineering geological map. To reach this goal laboratory and field tasks were performed. Engineering geological and geotechnical characterization of soils and rocks are based on their index and engineering properties and their classification is according to the standard proposed by the Unified Soil Classification System (USCS), British standard system, and unconfined compressive strength. Based on the USC system the soil type of study area fall into CL (inorganic clays of low to medium plasticity) and MH (inorganic silts, elastic silts).according to BSS the study area soil is classified as compressible silt, medium plastic inorganic clay. According to the Rock Mass Rating, unconfined compressive strength and degree of weathering, study area rock classified as, good basalt with slightly weathered and very high strength, good ignimbrite with slightly weathered and high strength, and poor ignimbrite with highly weathered very low strength. To address the Engineering geological and geotechnical characterization of soils, representative samples are collected and analyzed their index properties (Natural Moisture Content, specific gravity, Atterberg limits) and engineering properties (compaction, shear strength, and free swell behavior) in the labs and interpreted based on standard soil characterization techniques. From those tests, the Moisture content of the soils varies from 23.27 to 48. The Grain size analysis result of the study area show percent of sand 47%, silt 37.5-73.9%, and clay 50%. Laboratory consistency limit test result of the soil in the study area shows LL of 38.93-71% %, Pl of 15.8-49.5%, and The plasticity index of soil 10- 23.13. The activity of soil of the study area ranges from medium to inactive. The potential expansion behavior or swelling nature of the soils is found to be low to medium. The specific gravity of study area soil range from2.47-2.61. The optimum moisture content and maximum dry density results of the study area range32.8-28 and 1.6-1.21kg/m3 respectively. The shear strength results indicate that the cohesion of the soils varies from 38.4 to 54kpa, whereas the angle of shearing resistance varies from 18˚ to 22.99˚. The main geotechnical problems that affect the design and development of civil structures in the town were identified as the existence of shallow Ground Water table and cohesive soils of high plasticity character

Key words: Geotechnical characterization of soils, Unified Soil Classification System, Rock Mass Rating, index properties, multi-purpose engineering geological map, Aleta wondo

CHAPTER ONE

INTRODUCTION

1.1 Background

Site characterization is a prerequisite for the successful and economic design of engineering structures and earthworks. It is also necessary to obtain sufficient information on the type, characteristics, and distributions of rock and soil underlying at a site of proposed structures for feasibility and economic studies for the proposed project. The properties which control the feasibility of surface and subsurface rocks and soil to proposed structures are called geotechnical properties. Geotechnical properties include grain-size distribution, plasticity, hydraulic conductivity, consolidation, and shear strength parameters, density, porosity, and strength. The ground feasibility conditions are fully understood when these Geotechnical properties of rocks, soils, and the different geoenvironmental conditions are investigated (Bell, F.G., 2007). Hence characterizing site and investigation of soil and rock characteristics are an integral part of any structure. If the foundation studies are not conducted properly, it may affect the performance of the structures.

Due to heavy engineering structures like multi-story buildings and road construction would take place; big towns require detailed knowledge of geological and geotechnical properties of the foundation and the materials used for construction like most civil engineering structures (such as bridges, tunnels, etc.) (Gebremedhin Berhane, 2010). Ethiopia is one of the developing countries in Africa with different big and small cities and towns. Most of those towns are found in seismically active and geotechnically problematic areas, therefore, for a country like Ethiopia which is developing at a high rate and which needs much construction works at present and near the future, engineering geological investigation is very important. Because, the data obtained from engineering geological investigation is used to provide basic information for the current and future planning of land use and the planning, design, construction, and maintenance of civil engineering works in the country. Aleta Wondo town, the current study area, is found in Sidama regional state and situated in the main Ethiopian rift valley and represents a transition zone between the Ethiopian plateau and the rift valley. And the geotechnical properties of soils of this town have not been investigated so far. Despite the absence of geotechnical information, the study area experience geotechnical and geoenvironmental problems. In this research, the characteristics of soils and rock which are found in the town were characterized to generate geologically and engineering geologic map of town by interpreting secondary data, collecting primary data from the fieldwork, and conducting different engineering and index tests in the laboratory

1.2 Statement of the Problem

In Ethiopia due to attention is not given to engineering characters of the ground, most of the engineering structures such as Dams, roads, and buildings are out of service(Gebremedhin Berhane, 2010). To solve this problem, a qualified professional and detailed geotechnical investigation of the ground is necessary. Characterization of soil and rock in the city environment and other any project site is one of the important tasks, to save economic loss, to meet the project purpose, and protecting life from damage.

The engineering property of the ground in the town (Aleta wondo) is not studied so far. several civil engineering structures such as single to multi-story buildings, roads, etc. are under construction without proper geotechnical investigation. The town has also no engineering geologic and geologic map so far, and potential geotechnical challenges were not addressed which is manifested on different infrastructures such as damage on buildings, road damage from Hawassa – yirgalem – Aleta wondo area, despite to this most of the town is exposed for flood So, this research has investigated the geoenvironmental and geotechnical conditions and produced a multi-purpose engineering geological and geologic map.to overcome poor design, failure of engineering structures in the tow

1.3 Objectives of the study

1.3.1 General Objective

The main Objective of this research is to investigate the geological and geotechnical conditions of Aleta wondo area.

1.3.2 Specific Objectives

Specific objectives of this research are

- To identify and characterize the geotechnical properties of soils and rock masses
- To determine and classify the types of soils and rock masses based on standard classification systems
- To assess the geomorphological and surface geodynamic features of the study area
- To produce a multi-purpose engineering geologic and geological map of Aleta Wondo town.

1.4 Research question

The following questions are used as the base to this research

- What are the geological setup and geological structures of the site both at a regional and local scale?
- What are the geologic and geodynamic conditions of the study area?
- What is the geotechnical condition of the town and its influence on the general stability of engineering work?
- What are the engineering geological problems of the study area?

1.5 Significance of the Study

The study generally gives due attention to the characteristics and types of soils and rock found in Aleta Wondo town. It will be used to solve a civil engineering problem and to minimize natural hazards like flood risk, in addition, this study will be used later by other researchers interested to study in Aleta Wondo town as preliminary information. Furthermore, the importance of the study is to identify the areas of problematic soils in the study area. Since, the soils of Aleta Wondo town are not studied so far, for engineering practices and no engineering soil data in the city. This research may be used by the city municipality as an engineering soil data source.

1.6 Scope of the study

The scope of this research is limited to the characterization of Aleta Wondo town soils and rock in terms of their index and engineering property (shear strength). To accomplish these parameters ten test pits with maximum depth of 4m were excavated and thirteen proper representative samples were collected.

1.6 Limitation of the research

There is no organized secondary data regarding soils of the study area this is because most of the engineering works are conducted without exploration and are not supported by standardized data collection and well-organized data recording.

1.7 Previous studies

Throughout the world people investigate the soil for different purposes, for example, Western civilization credits the Romans for recognizing the importance of soils in the stability of structures. Roman engineers, especially Vitruvius, who served during the reign of Emperor Augustus in the first century BC, paid great attention to soil types (sand, gravel, etc.) and the design and construction of solid foundations (Budhu M. 2010). In Europe Engineering geological studies for the preparation of maps and plans have been carried out since the early years of the 20th century, the purpose of these maps was to provide information about characteristics of the natural environment for planning land use and engineering structures of all kinds. Rapid urbanization due to population growth in developing countries requires infrastructure and housing development over large areas. Problems resulting from unfavorable engineering geological conditions have confirmed that engineering geological mapping is a fundamental prerequisite for planning (Schalkwjk *et al*.,1990) as cited in (Efrem *et al*.,2013)

In Ethiopia, several studies have been conducted on engineering properties of soils and rock in different towns to identify their performance for engineering practice. Some of these researches are mentioned as blow. Engineering geological characterization of rocks and soils in parts of Addis Ababa-General implication to engineering practice" by Solomon, (2012). Gebremedhin Birhane (2010). Engineering Geological Soil and Rock Characterization in the Mekelle Town, Northern Ethiopia: Implications to Engineering Practice. This study identified, the presence of expansive clay soil which Couse physical damage to the building. Dagnachew Debebe(2011), Investigation On Some Of The Engineering Characteristics Of Soils In Adama Town, Ethiopia. From this study, the researcher found the presence of silt soil and clayey soil in the town. Yadeta C. Chemeda (2020) Engineering Geological Investigation of Adama Town: Implication to Engineering Practice, Alemayew Paulos (2016), Engineering geological characterization of soil and rock in Hosanna Town, Southern Ethiopia. There is also expansive soil and erosion problem in the town. A lot of regional-scale works have been done around and within the study area; concerning, Groundwater study, Hydrogeology, and Lake Hydrology for example

- Kedir Yassin, (2002). Performed a study of hydrogeology in upper Gidabo River Catchment, Southern Ethiopia. Mulugeta Musie, 2007, study Groundwater Circulation and Hydrochemistry of the Corridor (Upper Gidabo River and Lake Awassa Catchments), Sidama Zone, SNNPRS.
- Meseret G. Wahid, 2007, study the suitability of ground conditions for foundation and road construction in Awassa Town, Southern Ethiopia.
- Tomáš Hroch and Leta Megerssa 2018 Explanatory notes to the thematic geoscientific maps of Ethiopia at a scale of 1: 50, 000. These studies identify different soil and rock characterization, Hydrogeological assessment, exogenous and indigenous geohazard problems and address possible mitigation ways in the area.
- Kebede Ganole (2010) GIS-Based Surface Irrigation Potential Assessment of River Catchment for Irrigation Development in Dale Woreda, Sidama zone
- Ethiopia Health Infrastructure Program (EHIP) Health Centers Groundwater Investigation Final Report 2016

1.8 Organization of the thesis

The research is organized into six chapters. Each chapter covers a specific topic in the research work. In chapter one General background of the problem, statement of the problem, objective, and significance of the study are discussed. Chapter two deals with a brief literature review. Chapter three deals with the description of the study area. Chapter four of the thesis deals with the methodology used in the study. Chapter five deals with laboratory and field test results discussion. Chapter six gives a conclusion and recommendation.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

To the civil engineer, the soil is any uncemented or weakly cemented accumulation of mineral particles formed by the weathering of rocks, the void space between the particles containing water and/or air. In general, the difference between soil and rock is rough that in soils it is possible to dig a trench with simple tools such as a spade or even by hand. In rock this is impossible; it must first be splintered with heavy equipment such as a chisel, a hammer, or a mechanical drilling device.

Because soils and rocks are anisotropic and inhomogeneous by nature, Geotechnical investigations are very important for deciding economic feasibility to engineering structures beneath/around the proposed site (John.A. 1997). The geotechnical investigation includes both surface and subsurface. Design of foundations, method of construction, and solving geotechnical/engineering geological problems begins with the determination of the rock and soil engineering and geologic characteristics (Belay 2018). The main objective of the geotechnical investigation of soil and rock is to produce an engineering geologic map.

2.2 Engineering Geological Map (EGM)

Engineering geology is a sub-discipline of applied geology and it fills the professional gap between a geologist and civil engineer. It is an important science area to indicate where to constructs civil engineering structures. If nothing else, people can fail to understand where buildings could be built safely and where poor foundation conditions or the presence of geohazards and unacceptable risks were present (Efrem *et al*., 2013). This risky and safe area can be indicated by preparing an engineering geologic map of the area. An engineering geological map is a type of geological map, which provides a general representation of all components of the geological environment which is significant in land-use planning, design, construction, and maintenance as applied to civil and mining engineering. The principal factors for creating EGM are the Engineering geologic condition of an individual site or areas are rock and soil, water, geomorphological condition, and geodynamic processes. In engineering geological mapping context, the term soil is generally used for all material that can be excavated without the use of explosives or equipment for loosening or ripping (Belay 2018).

2.2.1 Components of EGM

The main components of EGM are: The character of the rocks and soils (including their description, stratigraphical and structural arrangement, genesis, lithology, physical state, and their physical and engineering properties), Hydro-geological conditions, Geo-morphological conditions, and Geodynamic phenomena

Engineering geological maps should include interpretative cross-sections and an explanatory text and legend. They may also include documentation data, which have been collected for the preparation of the map. On engineering geological maps, of all types and at all scales, the information provided should be presented in such a way that not only the true nature but also the engineering significance of the data can be understood and fully appreciated (Attewell, P.B., 1985).

2.2.2 Multipurpose EGM

According to the purpose EGM can be classified into specific and multipurpose EG maps. Special Purpose Maps: These maps provide information either on one specific aspect of engineering or for one specific purpose. Example; Engineering geological map for the dam site suitability, bearing capacity and settlement maps and engineering geology showing slope stability condition. Multipurpose Maps provide information covering many aspects of engineering geology for a variety of planning and engineering purposes.

The main problem in multipurpose engineering geological soil and rock mapping is the selection of those geological features of rocks and soils, which are closely related to physical properties, such as, strength, deformability, durability, and permeability, which are important in engineering geology. It is often difficult to determine all these properties in sufficient quantities, over large areas, quantitatively, quickly, and cheaply. It is for this reason generally index (geotechnical) and geological properties are used which best indicates physical or engineering geological characteristics. These 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, related to the physical state of soils and rocks and indicating strength properties, deformation characteristics, permeability and durability (Raghuvanshi, 1995). Some of those properties that are investigated through this study are discussed one by one as follows.

2.3 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. 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(A.R.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 M., 2010).

2.4 Geotechnical Characterization 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(A.R. 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(A.R. 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:

2.4.1 Natural Moisture Content (NMC)

The water content of a soil mass is defined as the ratio of the mass of water in the voids to the mass of solids. The water content, which is usually expressed as a percentage, can range from zero (dry soil) to several hundred percent (Murthy, 1990). Natural Moisture Content affects soils behavior when used for construction purposes and foundation of structures. Moisture content affects settlement condition (consolidation), shear strength, and suitability of soil for compaction. Natural Moisture content is used to predict the degree of consolidation of soils Moisture content and consistency limits aids in describing a soil's suitability. A coarse-grained, sandy, or gravelly soil generally has good drainage characteristics and may be used in its natural state(Krishna R., 2002). A fine-grained, clayey soil with a high PI may need significant treatment, particularly if used in a moist location. As stated in Hawi Hailu (2018) Munsell Color, (2000) forwarded that, It is possible to estimate whether the soil is pre-consolidated from overburden pressure by noting the position of water content w concerning LL and PL.

1. If water content w is close to LL than to PL the soil is likely to be normally consolidated.
2. If water content w is close to the plastic limit than LL the soil is likely to be pre-consolidated.

Determination of the moisture content is required as a guide to the classification of soils and to evaluate the behavior of soils during construction. The moisture content soils can be determined in the laboratory using ASTM D2216 procedure and values can be calculated by Equation 2.1

$w = \frac{Mw}{Ms} * 100$……………………………………………………………Equation 2.1

Where Mw is mass of water, Ms is mass of solid

In coarse-grained soils such as sands and gravels, the water content may range from a few percent in drier soils to over 20% in saturated soils. In fine-grained soils such as silts and clays, the possible range in w is much higher due to the ability of clay minerals to adsorb water molecules. Moisture content in fine-grained soils may be as low as a few percent, to over 100% in higher-plasticity clays.

2.4.2 Grain-size Analysis

Grain size Analysis provides the grain size distribution, affects engineering properties of soils and is used in classifying soils(Arora, K.R. 2004). The particle size distribution curve (gradation curve) represents the distribution of particles of different sizes in the soil mass. It gives an idea regarding the gradation of the soil i.e. it is possible to identify whether the soil is well graded or poorly graded. In mechanical soil stabilization, the main principle is to mix a few selected soils in such a proportion that a desired grain size distribution is obtained for the design mix (Mallo *et al.*, 2002). Soil particle size has a definite relation to its bearing capacity. Empirical tests showed that well-graded, coarse-grained soils generally can be compacted to a greater density than fine-grained soils because the smaller particles tend to fill the spaces between the larger ones. The grain size of soils can be determined in a laboratory by following ASTM D 422 Standard test method. Two methods generally are used to find the particle-size distribution of soil, sieve analysis for particle sizes larger than 0.075mm in diameter (coarse-grained soils), and hydrometer analysis for particle sizes smaller than 0.075 mm in diameter (fine-grained soils) (Craig, 1997).

2.4.3 Consistency limit

For very fine soils, such as silt and clay, consistency is an important property. It determines whether the soil can easily be handled, by soil moving equipment, or by hand. The consistency is often very much dependent on the amount of water in the soil (Arnold Verruijt, 2001). As the water content of fine-grained soil is increased gradually from 0%, it goes through different consistencies, namely brittle solid, semi-solid, plastic, and liquid states. Atterberg limits are the borderline water contents between two such states. These limits are the liquid limit, the plastic limit, and the shrinkage limit.

Liquid limit is defined as the moisture content, in percent, at which the soil changes from a liquid state to a plastic state. It is arbitrary the water content in percent at which a pat of soil in a standard cup and cut by a groove of standard dimensions will flow together at the base of the groove for a distance of 13mm (1/2 in) (Arora, 2003).

Plastic limits the moisture contents (in percent) at which the soil changes from a plastic to a semisolid state. It is also water content, in percent, at which a soil can no longer be deformed by rolling into 3.2 mm (1/8in) diameter threads without crumbling. Soil can no longer behave as plastic; any change in shape will cause the soil to show visible cracks (Arora, 2003).

Shrinkage limits the moisture contents (in percent) at which the soil changes from a semisolid to a solid-state, these limits are generally referred to as the Atterberg limits. The Atterberg limits of cohesive soil depend on several factors, such as the amount and type of clay minerals and type of adsorbed cation (Braja 2008). Consistency limits of soils determined at laboratory according to ASTM D 4318.

Plastic Index is the numerical difference between Plastic limit and liquid limit. The high plastic index soil indicates that the existence of a high amount of active clay contained in the soil. Based on the plasticity index value, the swelling potential range of soil can be determined according to (Table 1.1). the soil sample can be said Low, Medium, Medium, High, and Very high swelling potential, where plasticity index values range from 0-15, 10-35, 20-55, and 35 & above respectively. Plasticity is also used to determine whether the soil is cohesive or not(Samuel, T.,1989) and mineralogy of clay(Figure1.1). Numerically plasticity index can be calculated by using the formula:

$$PI=LL-PL \qquad \text{Equation 2.2}$$

Table1.1 Swelling potential (After Chen, 1988)

Swelling potential	Plasticity index
Low	0-15
Medium	10-35
High	20-55
Very high	35 & above

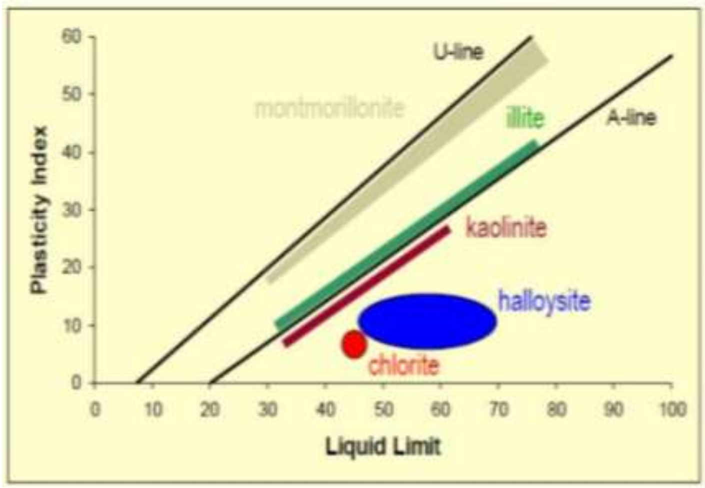

Figure1.1 Mineral identification using Casagrande PI-LL chart (After Chen, 1988)

2.4.4 Specific Gravity

Specific gravity is the ratio of the mass of soil solids to the mass of an equal volume of water. It is an important index property of soils that is closely linked with mineralogy or chemical composition (Oyediran *et al.,* 2011). It is relatively important as far as the qualitative behavior of the soil is concerned (P. P. Raj, 2012) and useful in soil mineral classification, for example, iron minerals have a larger value of specific gravity than silica (J. E. Bowles 2012). Most of the specific gravity values of minerals found in soils fall within the range of 2.6 to 2.9 as indicated in (Table1.2) Specific gravity is tested for soil particles pass the No.4 sieve (ASTM D85400). As shown in (Equation 2.3) the Specific gravity of soil solids, Gs, is calculated based on these three parameters: as Mass of the flask filled with distilled water to the etch mark, Ma; the mass of the flask filled with water and soil to the etch mark, Mb; and mass of the dry soil, Mo.

$Gs = \frac{Mo}{Mo+(Ma-Mb}$ …………………………..Equation 2.3

Table1.2 Specific gravity range

Soil	Specific gravity
Sand	2.65-2.67
Silty sand	2.67-2.70
Inorganic clay	2.7-2.8
Soil with mica and iron	2.75-3.0
Clay and silty clay	2.67-2.90
Organic soil	Less than 2.0

2.4.5 **Free swells**

The amount of swelling is known to be dependent on the clay minerals, the soil mineralogy and structure, texture, and several physical and chemical aspect of the soil. Among clay minerals, montmorillonite influences the magnitude of swelling and activity of soils maximally as compared to Illites and kaolinites (Holtz and Kovacs, 1981). These clay minerals are characterized by Very weak van der Waals' forces in between the adjacent unit cells of the mineral and Appreciable isomorphs substitution during the clay mineral formation, leading to very high negative surface charges (Asuri *et al.*, 2016). Expansive soils can be recognized by using mineralogical identification, indirect index property tests, or direct expansion potential tests (Chen, 1988). The Expansivity of soil is governed by the type and proportion of clay minerals it contains. Knowing the type and proportion of the clay mineral in the soil gives a clue on the swelling potential (Chen, 1988). Free swell Soils having a free swell value as high as 100% can cause considerable damage to lightly loaded structures, and soils having a free swell value below 50% seldom exhibit appreciable volume change even under very light loadings (Holtz and Kovacs, 1981). The free swell of soil can be calculated by dividing the difference between final and initial volume.

2.4.6 Activity Clay

As cited by Medihanit (2017), Skempton (1953) can be classified the soil according to its free swell value. Following his classification, three degrees of colloidal activity (Activity, Ac = PI/ percentage by weight finer than 2μm) have been established as indicated in Table 1.3 (Munsell Color 2000).

Table1.3 Activity of soil

Activity	Soil type
< 0.75	In active clays
0.75-1.25	Normal clays
>1.25	Active clays

.

2.4.7 Shear strength

The shear strength of soil is its ability to resist sliding along internal surfaces within a mass. The stability of a cut, the slope of an earth dam, the foundations of structures, the natural slopes of hillsides, and other structures built on soil depend upon the shearing resistance offered by the soil along the probable surfaces of slippage. Cohesion (C) and angle of internal friction (Φ) are the key parameters of shear strength(Murthy, V.N.S. 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)

2.4.8 Permeability

Permeability is the ability of soil to transmit the water through inter connected void. Any given mass of soil consists of solid particles of various sizes with interconnected void spaces. The continuous void spaces in a soil permit water to flow from a point of high energy to a point of low energy (Braja M. Das., 2008). The pressure of the pore water is measured relative to atmospheric pressure and the level at which the pressure is atmospheric (i.e. zero) is defined as the water table (WT) or the phreatic surface. Below the water table the soil is assumed to be fully saturated although it is likely that, due to the presence of small volumes of entrapped air, the degree of saturation will be marginally below 100%. The level of the water table changes according to climatic conditions but the level can change also as a consequence of constructional operations. A perched water table can occur locally, contained by the soil of low permeability, above the normal water table level. Artesian conditions can exist if an inclined soil layer of high permeability is confined locally by an overlying layer of low permeability (Craig 1997).

Data from field permeability tests are needed in the design of various civil engineering works, such as cut-off wall design of earth dams, to ascertain the pumping capacity for dewatering excavations and to obtain aquifer constant(P. P. Raj,.2012)

2.4.9 Consolidation

Consolidation is the time-dependent settlement of soils resulting from the expulsion of water from the soil pores. Primary consolidation is the change in volume of fine-grained soil caused by the expulsion of water from the voids and the transfer of stress from the excess porewater pressure to the soil particles. Secondary compression is the change in volume of a fi ne-grained soil caused by the adjustment of the soil fabric (internal structure) after primary consolidation has been completed. . In compression of soil the porosity decreases, and as a result, there is less space available for the pore water. This pore water may be expelled from the soil, but in clays, this may take a certain time, due to the small permeability (Arnold.V. 2001). Excess pore water pressure is *u*o the pore water pressure above the current equilibrium pore water pressure. For example, if the pore water pressure in the soil is uo and a load is applied to the soil so that the existing pore water pressure increases to u1, then the excess pore water pressure is calculated as Equation 2.1

$\Delta u = u1 - uo$..Equation 2.1

2.4.10 **Soil compaction:**

Soil compaction is one of the ground improvement techniques. It is a process in which by expending comp active energy on soil, the soil grains are more closely rearranged. Compaction increases the shear strength of soil and reduces its compressibility and permeability (S.R. Kaniraj, 1988). Compaction is not index properties of soil but it is the densification of soils by the expulsion of air. Soil compaction is perhaps the least expensive method of improving soils. It is a common practice in all types of building systems on and within soils (Budhu M., 2010). There is two common standards for compaction test ASTM D698: Using Standard Effort (12,400 ft-lb/ft3 (600 kN-m/m3) and ASTM D1557: Using Standard Effort (56,000 ft-lb/ft3 (2,700 kN-m/m3). Maximum dry unit weight (ɣd) is the maximum unit weight that soil can attain using a specified means of compaction. Optimum water content (wopt) is the water content required allowing the soil to attain its maximum dry unit weight following a specified means of compaction.

The bulk unit weight and dry unit weight can be calculated by:

$\gamma = \frac{M}{V}$……………………………………………………………………………Equation 2.4

$\gamma d = \frac{\gamma}{1+w}$…...Equation 2.5

2.5 Engineering geologic classification of Soil

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 (C.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. 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 classification schemes available. Each was devised for a specific use. For example, the American Association of State Highway and Transportation Officials (AASHTO) developed one scheme that classifies soils according to their usefulness in roads and highways, while the Unified Soil Classification System (USCS) and British Standard System were originally developed for use in airfield construction but was later modified for general use(Budhu, 2010).

2.6 Engineering geologic classification and characterization of rock.

Rock can be classified into several groups based on its geologic and engineering properties. Based on their genesis, the process of formation and the environment they form, rock can be igneous, metamorphic, and sedimentary types. Rocks have various engineering properties the variation in the engineering properties of different rocks is because of their mode of origin, mineralogical composition, textural structure, and presence of primary and secondary structures in the rock. Thus, a clear qualitative assessment of engineering properties can be made by just knowing the genetic type of rocks. As Belay (2018), engineering uses of the rock requires a more generic subdividing of rocks into two groups. These are; intact rock (rock material) and Rock mass.

Intact rocks (Rock material) Are blocks of rock that do not contain mechanical discontinuities and do have tensile strength. It is described by color; grain size, compressive strength, and rock type (Bell, 2007). 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).

The rock mass is a mass of rock interrupted by discontinuity, with each constituent discrete block having intact rock properties. Engineering classification or description of rock mass contains both rock mass characters and rock material characters such as a) strength b) the occurrence and nature of discontinuities and c) the degree and extent of chemical weathering. Rock mass properties such as rock mass deformation and strength are difficult to assess, in such cases, rock mass classification is commonly used to evaluate the rock mass behavior (Belay, 2018). Rock mass classification puts a number to the quality of the rock mass i.e. quantifies the quality. There are several classification systems available but the International Society for Rock

Mass Rating, Rock Quality Designation, and Rock Mass Rating are the most widely used systems.

International Society for Rock Mechanic (ISRM) The presence of primary discontinuities greatly influences the mechanical behavior of the rock mass such that it is often different (rock mass) from that of the intact rock, which has no secondary discontinuities. According to various types of classifications (based on the mode of formation, chemical composition, degree of metamorphism, weathering, and mechanical behaviors); ISRM is one of the classification schemes to describe the strength range of intact rock. According to the parameters proposed by ISRM (1981) in Table 1.4 the rock can be grouped into five major classes, such as very high strong rock, high strength, medium strength rocks, low strength rock, and very low strength rocks

Table1.4 Intact rock strength classification

USC(Mpa)	description
Greater than 200	Very high strength
56-200	High strength
21-55	Medium strength
8-20	Low strength
Less than 8	Very low strength

After (ISRM, 1981)

Rock quality designation method The rock-quality designation (RQD) developed by Dr. Don U. Deere (Deere and Deere1988) is a method of logging sound drilled rock core to calculate and quantify the percentage of"good" rock in a core run. RQD is a quantitative method of evaluating rock quality and is widely used as one of the parameters in other more numerical rock classification systems (RMR). The RQD value of the rock calculated according to equation 2.2

$$\boldsymbol{RQD} = \frac{\textbf{Sum of length of core pieces 10cm or greater}}{\textbf{Total length of core run}} * \mathbf{100}\% \text{.....................................Equation 2.2}$$

When no core is available but discontinuity traces are visible in surface exposures or exploration adits, the RQD may be estimated by volumetric count (Jv): from the number of discontinuities per unit volume or the number of joints intersecting a volume of one m3. RQD value can be correlated with rock quality in the following Table 1.5.

Table1. 5 RQD and rock quality

RQD%	Rock quality
Less than 25	Very poor
25-50	poor
50-75	fair
75-90	good
90-100	excellent

RMR Method the RMR system incorporates rock mass data regarding rock strength, RQD, discontinuity spacing, discontinuity condition, groundwater, and an adjustment for discontinuity orientation concerning the excavation. These parameters are assigned numeric values based on their conditions and the summation of the numeric values for all the parameters is the rating of the rock mass (.Bieniawski, 1973). Since all these various parameters have different significance for the overall classification of a rock mass, different value ranges of the parameters have been assigned based on their importance; a higher value represents better rock mass conditions (Bieniawski, 1989) Table1.6. According to Bieniawisk, all those parameters rating value added up and their total value classify according to the following Table 1.6

Table1. 6 RMR rock classification system

RMR rating	**81-100**	**61-80**	**41-60**	**21-40**	**<20**
Rock mass class	**A**	**B**	**C**	**D**	**E**
Description	**Very good**	**Good**	**Fair**	**Poor**	**Very poor**

After Bieniawski (1989).

CHAPTER THREE

DESCRIPTION OF THE STUDY AREA

3.1 Location, and Accessibility of Study Area

Aleta-Wondo town, capital of Aleta-Wondo zone, is one of the autonomous towns in the Region. It is located between 43578-43798N and 71799-72500E and has a maximum elevation of2008 m and minimum1879 a.m.s.l. It is situated at a distance of about 337 km south of Addis Ababa and 65 km from the regional capital, Hawassa, along Hawassa-Yirgalem-Aleta-Wondo-Negele-Borena asphalt roads as indicated in (Figure 1.2).

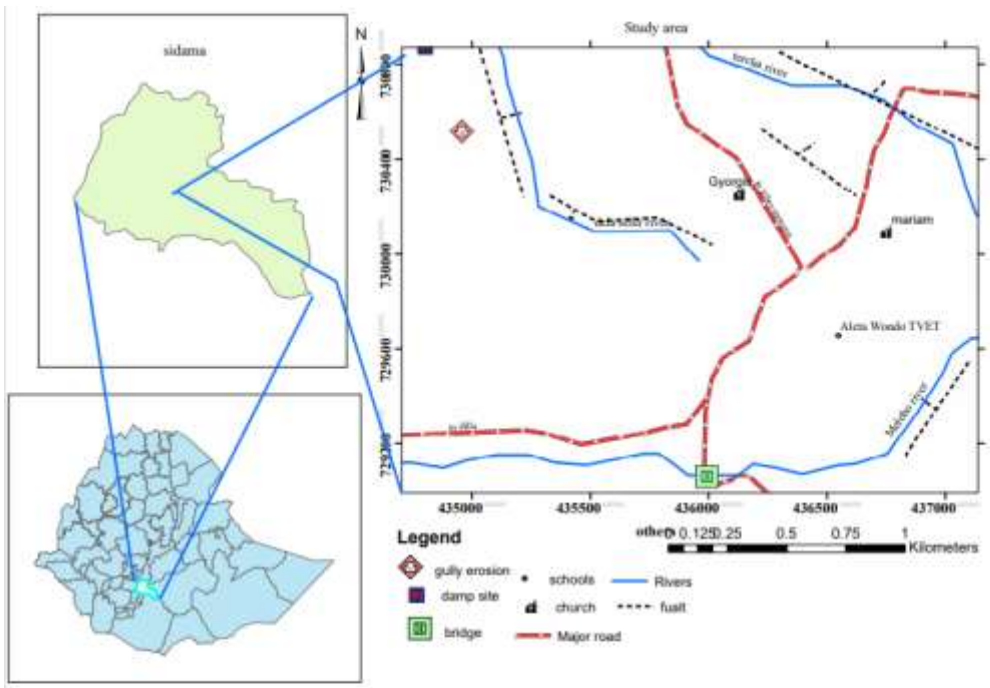

Figure1.2 Location map of the study area

3.2 Topography and slope

The topography of the town and its surrounding consisting of flat to undulated topography, deep gorges, river dissections, steep slope feature is the result of its relative closer location to the rift margin (escarpment) as in (Figure1.3). The larger part of the built-up area is characterized by an average slope that ranges from 5-15 percent and the hilly portion of the town measures an average slope of greater than 20 percent. the elevation at the center of the town near to the bus station is around 1925m.a.s.l. Generally, altitude increases as one proceeds from the western part of the town towards the east of town.

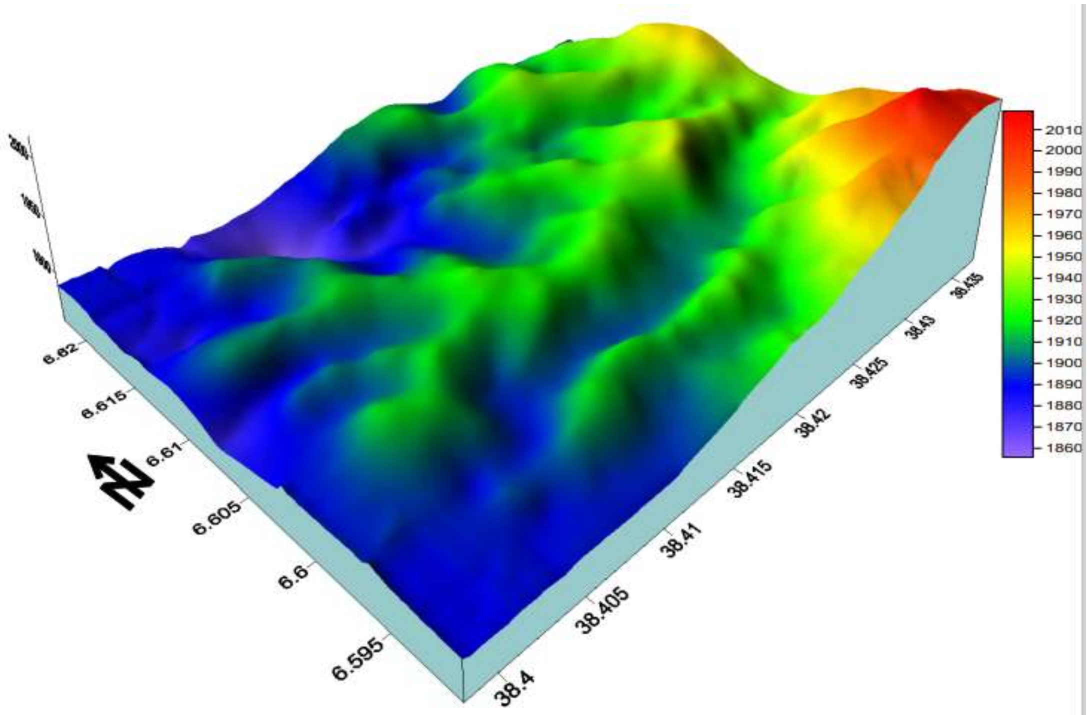

Figure1.3 The Physiographic diagram of Alata wondo town.

3.3 Drainage

The town is found in the Aleta-Chuko water shade, which is a sub-basin of the Gidabo river basin. The town is bounded by two perennial rivers of northern direction Kola River and with the southern direction of Melebo River. Melebo River crosses the town and finally, both rivers drain to Lake Abaya. Both rivers follow the main geologic structure of the town. The flow direction of rivers and streams are south east to North West. The drainage pattern of the area is highly controlled by geological structures and the pattern of the town is dendritic as (Figure1.5) The pattern indicates that the streams in the area occupy weak zones.

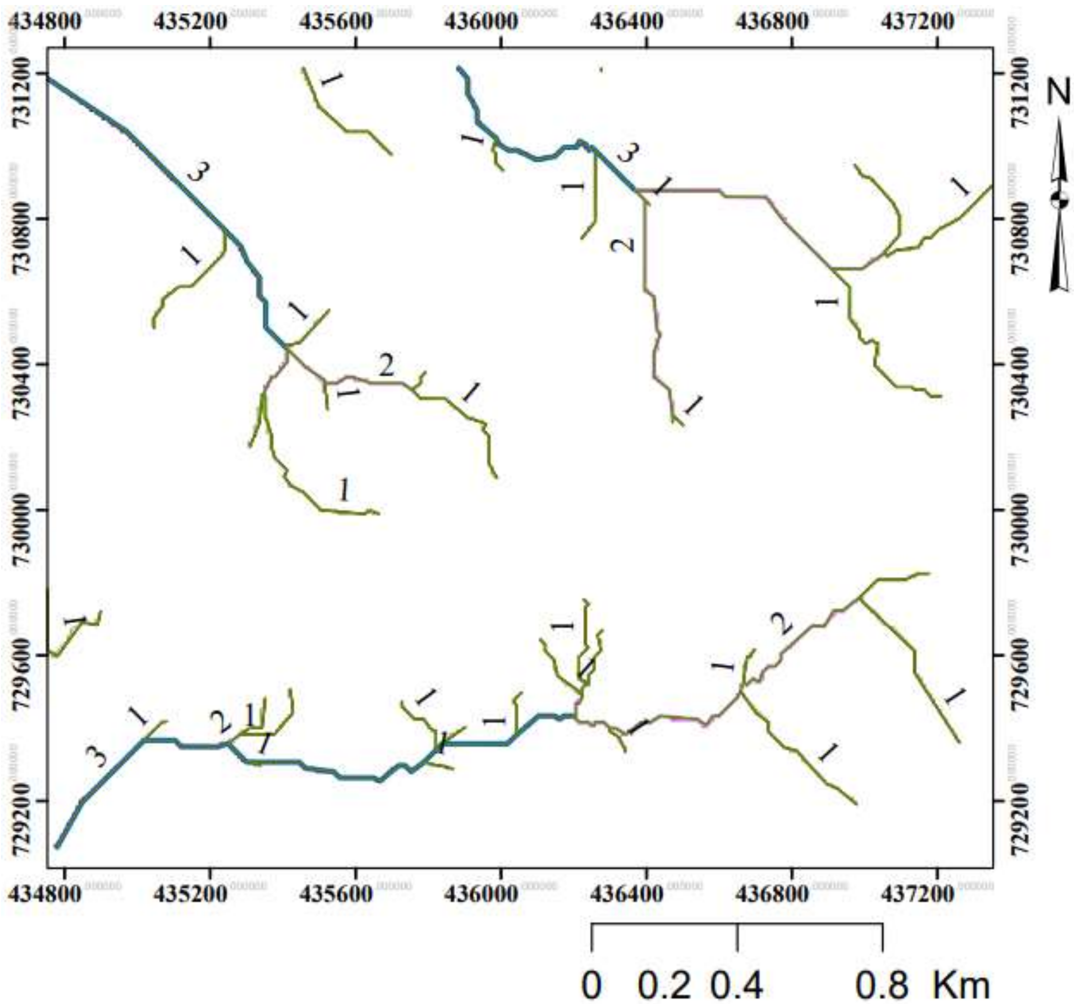

Figure1.5 Drainage map of the study area (Aleta wondo town) the town has third-order Melebo River at the south and other small streams in northern parts of town

3.4 Population

The population of Aleta Wondo is estimated by the different organizations including the municipality, Water supply master plan, NUPI, and CSA. The census conducted by NUPI in the year 2002 indicates that the population of Aleta Wondo is 13617, out of which 50.4 percent are males and 49.6 percent are females (NUPI, March 2004). The population of Aleta Wondo according to the Summary and Statistical report of the 2007 population and housing census of the Federal Republic of Ethiopia Population Census Commission is 22,090 in 2007(CENSUS, 2007). During this period the percentage of the population distribution by sex in town indicates that 52.7% of the population is male while the remaining 47.2% is female.

3.5 Climate

Aleta Wondo town falls in Woyna-Dega eco-climatic zone characterized by cool and sub-humid climatic characteristics. There is a high annual and monthly variation of rainfall in the town. The annual rainfall of Aleta Wondo town for 22 years from 1987-2008 is plotted in figure 1.6 and as it is shown in the figure the annual rainfall varies from 1233 mm to 2079 mm whereas the mean annual rainfall of the town during this period is 1564 mm.

Figure1.6 Annual rainfall of Aleta Wondo from 1987-2008

The seasonal distribution of rainfall in the town reveals that the rainfall is a bio-modal type. The mean monthly rainfall of Aleta Wondo town from 1987-2008 is computed and summarized in figure 1.7. As depicted in the figure low rainfall is observed from November-February while high rainfall is recorded from April to October.

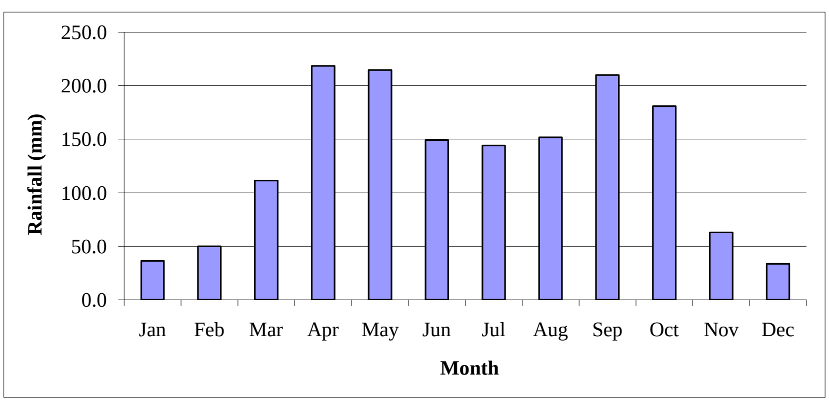

Figure 1.7 Mean monthly rainfall of Aleta Wondo from 1987-2008

As far as the temperature of the study town is concerned unlike that of rainfall, there is no high seasonal variation of temperature. The mean annual temperature of Aleta Wondo is 17ºC. The highest temperature occurs before the onset and after the recession of the rainy season.

3.6 Land use and land cover

As seen in Figure 1.8, the central part of the study area is covered with the civil engineering structure of government commission, hotels, and different commercial buildings. Almost all expansion areas of town ways covered with concrete and cobblestone. The town has got wide main streets and narrow secondary streets. But there is yet a big work to be needed to satisfy the growing needs of traffic. The expansion directions of town south west and north east are covered with agricultural activity and evergreen, long and many years' trees such as *Bahirzaf, Tsid,* and *Zigba.*

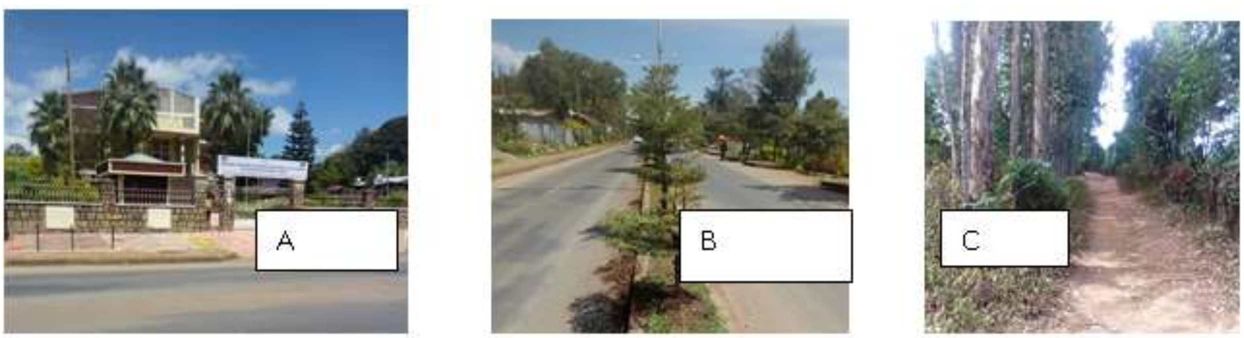

Figure 1.8 Land use and land cover pictures of the study area, Aleta Wendo administration millennium hole(A), Main Concrete Street of Aleta wendo Administration (B) and long and many years' trees such as, *Bahirzaf, Tsid,* and *Zigba* found in Aleta wondo town(C)

3.7 Geologic setup of study area

3.7.1 Regional Geology

Aleta wondo town is situated in the southern segment of the main Ethiopian rift valley and represents a transition zone between the Ethiopian plateau and the rift valley with poorly defined escarpment. The evolution of the southern segment of the Main Ethiopian Rift has been divided into three stages according to age (Rapprich., *et.al*, 2016): pre-rift (32–45 Ma), syn-rift (22–12 Ma), and post-rift (0–5 Ma).

3.7.1.1 Syn-rift volcanic deposit,

Nazareth Formation (Miocene age)

Transitional to mildly alkaline and sub alkaline basalts and rhyolites
This unit is exposed in the southeastern part of the study area and Dilla map sheet. It covers of Dilla, Wenago, Chinchu, Bedesa, Teferikela, Aleta wondo and Kebado towns. The transitional to mildly alkaline and sub alkaline basalt is highly weathered. It forms cliffs and shows fractures and jointing. It is composed of olivine, pyroxene, and plagioclase; The basalt is overlain by the rhyolites (Yismaw *et al.*, 2015).

Aphyric and porphyritic basalt with lesser vesicular basalt, minor alkali trachyte flows, and tuffs

According to (Yismaw *et al.*, 2015) this unit covers an area on the southeastern part of the Dilla map sheet and study area. It consists of Aphyric basalt which is black and slightly weathered exposed on the top of the hills and occurs in areas of Bansa (Daye) to Kibremengist, towards the eastern part. The porphyritic basalt is olivine-pyroxene which has a fresh dark gray color and shows slight to medium weathering. When weathered it shows light gray to light brown color. The lesser vesicular basalt is a brown-colored medium to highly weathered and it forms cliffs.

Dino Formation

It is exposed on the northwest part following the Galena and Wabe Rivers it flows to Hagere Selam and on the northeast parts of the area along Bale River to dame and Kulu Rivers by surrounding mountain Damonte. Related to Dilla map sheet and south east related to study area. It contains different types of litho units such as Ignimbrites, rhyolites, Basalts, and tuffs. These different types of units are the upper Miocene exposure controlled by some primary structures which have a trend of North East (Yismaw *et al.*, 2015).

Alkaline and peralkaline silicic, rhyolite domes, and ignimbrites

According to (Yismaw *et al.,* 2015), this unit was exposed around Lake Hawassa on the east part to the Dilla map sheet and northern parts of the study area. Ignimbrites are the dominant features that have coarser rock fragments in fine-grained materials. It shows gray color lithology, slightly weathered it mostly forms smaller hills with thickness is 6m or more.

Per alkaline silicic

This unit has a light yellow fresh color. Peralkaline silicic include rhyolite and trachyte rock. Which are often coarse-grained and both are slightly weathered. It is exposed in the eastern parts of the study area (Yismaw *et al.,* 2015).

3.7.1.2 Post-rift volcanic deposits, Pleistocene Dino Formation (Pleistocene age)

Pumiceus pyroclastics, ignimbrites and diatomites

This is the intercalation of diatomites and ignimbrites rock exposed on the northern part of the mapped area covering areas around Midre Genet, Morocho, and Yirga Alem. The diatomite is light yellow to white. It is fine-grained and has 30m thickness. Minor fossils are found within this unit. Most of the pyroclasts have pumice fragments in fine-grained material. The diatomite is light friable or loose with light gray to yellow color.

Ignimbrites, tuffs, waterline, pyroclastics

These rock units are found in the center and the northwest part of the map sheet. Such rocks are exposed around Aleta wendo, yirgalem, Koye, Chuko, and Seke, and some other small towns and contain light gray, coarse-grained lithic ignimbrites, unwelded tuffs which have a light gray color exposed by making flat land, and pyroclasts.

Rhyolitic and Ignimbrite lava flows

It is exposed in the northern and northeastern parts of Lake Abaya. It covers Damote Mountain on the very northwest of the map sheet and it is intercalated with the Nazareth series. The rhyolite is medium to highly weathered makes large hills like Mountain Damote which has 900m thickness with having pink color, high friability and in some parts of the unit, it is difficult even to take a sample. Quartz and feldspar crystals could be seen by hand specimen and sometimes by the naked eye

Lacustrine, volcano clastic sediments, and tuffs

This lithological unit is exposed on the north-central part of the map. Most of the lacustrine sediments and the volcanoclastic sediments are light yellow colored with medium grain size. The exposure occurs from Dilla to Awasa and north of Abaya making a sharp contact with Pleistocene basalt which is exposed on north of Abaya

The rock units include aphyric basalts, scoraceous basalts, ignimbrites, unwelded tuffs, volcano sediments and lacustrine sediments. The dominant exposures are volcano sediments and tuffs. The volcano clastic sediments show light gray color and is friable and with 20m thick. The exposure is best seen along the road. It covers Humbo, Dawe, Gulgula, Belela, Dimtu, and Werancha wocho localities.

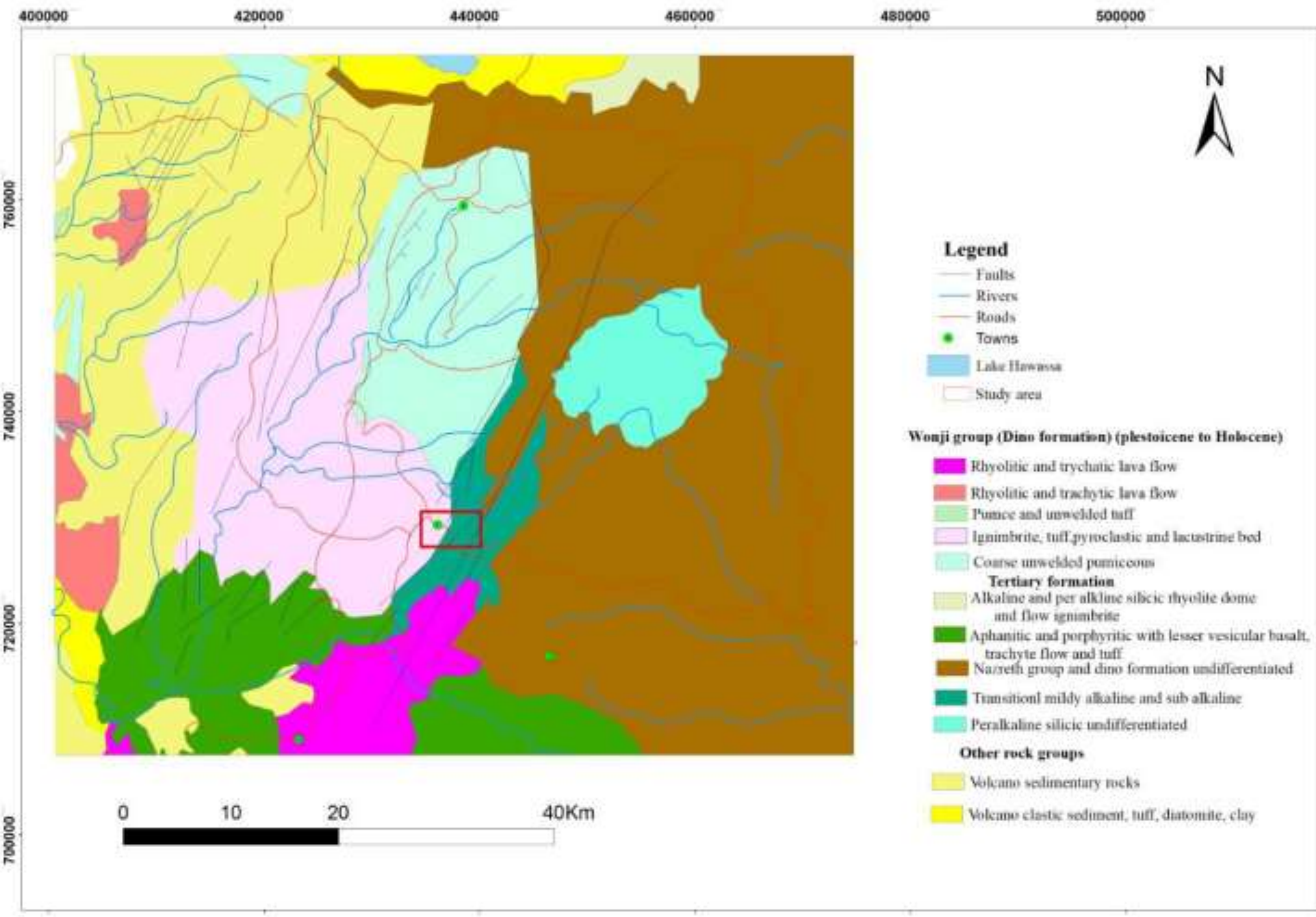

Figure1.9 Regional Geologic Map of the study area

3.7.2 Local Geology

The geology of Aleta wondo area is represented by two volcanic units dominated lower part by basaltic lava flows(basalt), followed by a pyroclastic sequence mainly formed by ignimbrite (ignimbrite), followed by central composite volcanoes (central volcanoes unit).

3.7.2.1 Basaltic lava flows (basalt)

This unit extensively crops out along the river at east to the south eastern part of town and represents the oldest unit of the area. It consists of essentially sub-horizontal lava flows with thickness ranging from few meters up to 20m. Maximum exposed thickness was found east town, along the river. Aleta wondo basalt is predominantly constituted by alkaline and olivine basalts with three main textural attributes, that is porphyritic, aphyric, and sub-aphysic.

3.7.2.2 Ignimbrite unit

Aleta wondo ignimbrite unit is exposed close to Aleta wondo along the rivers. It overlies the Aleta wondo basalt and locally covers the product of the composite central volcanoes of Wechecha and Furi. The sequence is constituted by different flow units, consisting of pale- green to pale –yellow welded and crystal-rich ignimbrites as figure1.10.

Figure1.10 Ignimbrite unit deposit along with river “A” and at quarry site “B**”**

3.7.2.3 Residual soil

As observed from river cut, slop cut, and test pit at the time of field investigation, Aleta Wondo Town is dominated by thick residual soil. In the southern part of town, especially, along with the slop cut of kibre mangist road, white to brown color and red thick residual clay to silty soil deposit observed (Figure 1.11). The average thickness of soil in the town is 12m.

Figure1.11 Residual soil deposits

From test pit, river and slop cut observation; there is no considerable variation of soil type with depth. As seen from the twelve 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) are lithologically correlated with each other. The soil type observed and sampled from these test pit areas is red and fine grain residual types. The remaining test pits (TP7, TP11&tp12) lithological correlated with white color sandy silt soil deposits along Melebo River.

3.7.3 Structure

In the engineering context, discontinuities can be the single most important factor governing the deformability, strength, and permeability of the rock mass. Moreover, a particularly large and persistent discontinuity could critically affect the stability of any surface or underground excavation. For these reasons, it is necessary to develop a thorough understanding of the geometrical, mechanical, and hydrological properties of discontinuities and how these will affect rock mechanics and hence rock engineering.

The town is found in the southern segment of the main Ethiopian rift system, the regional fault system and local small-scale faults and joints align the strike of NNW-SSE. But the alignment of the southern segment of the MER system is NNE-SSW; this indicates the initiation factor for the MER system has not much effect on study area structures. The strike direction of the MER system and study area major fault system are not similar. The dip direction of study area fault system NE&SSW. In the northern part of town especially along with stream cut, minor, long persistent, and two sets of joints that dissect the ignimbrite rock are observed. As shown below (Figure 1.12), the orientation of discontinuity is perpendicular with each other, the orientations of these joints are NNW-SSE.

Figure1.12 Minor local fault and Joints on ignimbrite along to Hawassa route

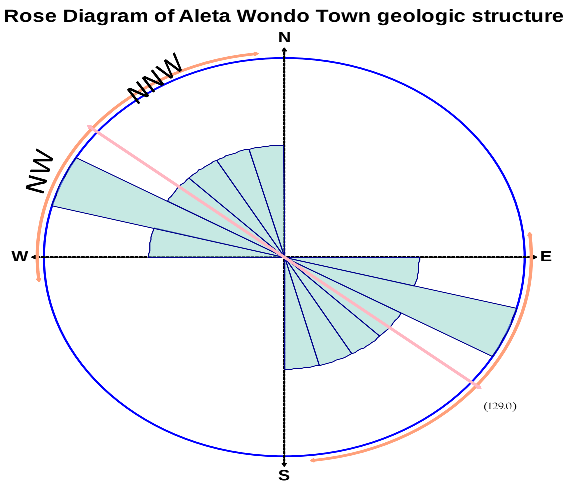

Figure1.13 Rose diagram

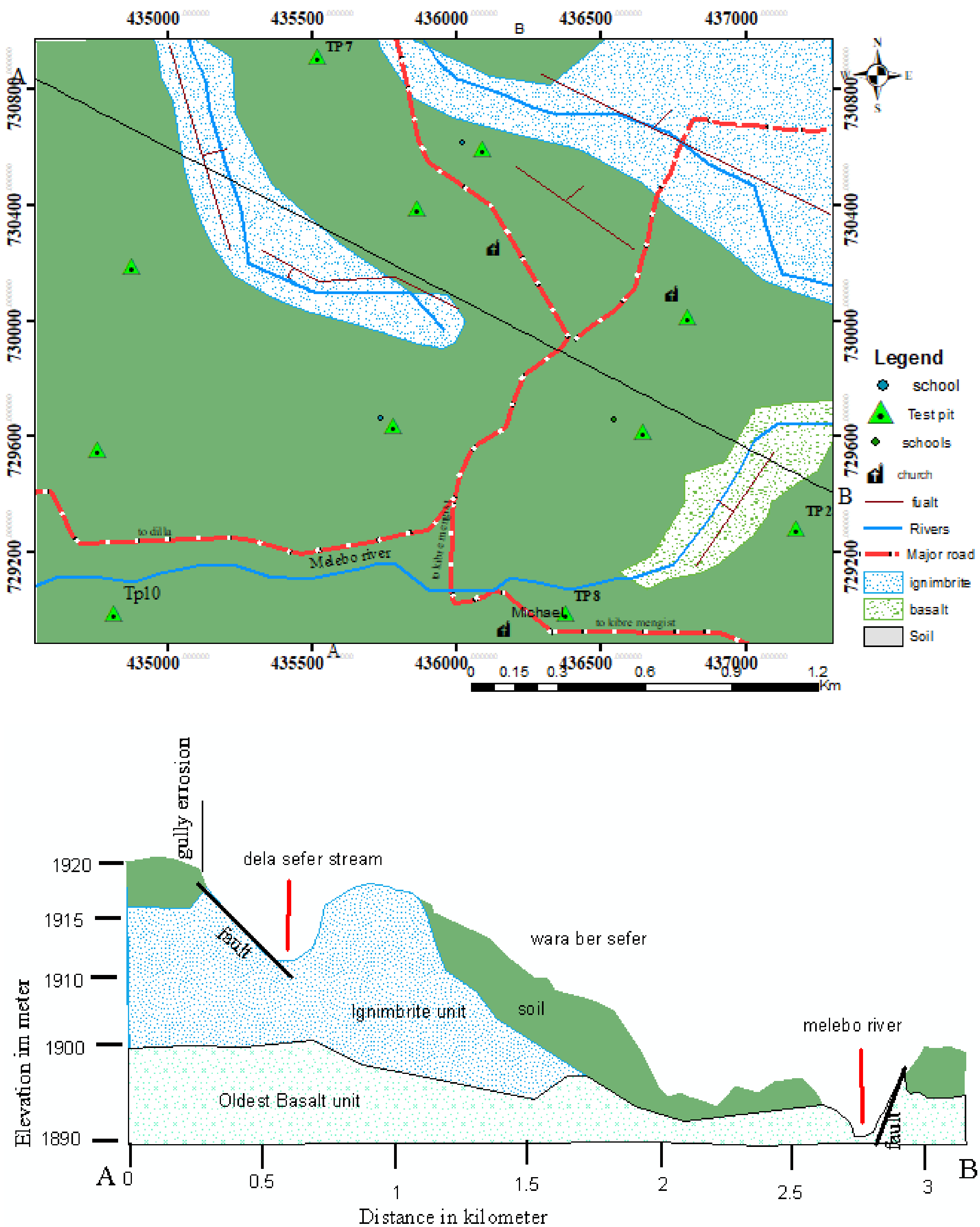

Figure1.14 Geologic map and Cross-section of the study area

CHAPTER FOUR

RESEARCH METHODOLOGY

4.1 Overview

Climatic, topographic, and geomorphologic conditions, hydro geologic, geologic, and engineering geologic data are necessary information to meet the current study goal. Taking into consideration those data, different stages of methodologies are designed and employed based on the intended objectives. Methodologies are devised to gather the maximum amount of information available from previous work and present performing tasks. The general procedures and methodologies include desk study, detailed field survey, laboratory test, data processing, and analyses are described in this chapter. The step followed to perform each of the activities was systematically described as follows.

4.2 Data collection techniques

Collecting data includes primary and secondary sources. The literature survey of both published and unpublished reports of investigations published textbooks, and journals are obtained from different sources for a better understanding of the characteristics of the soil and rock in the study area. Field work has been done to gather primary data on the geology, topography, and soil formations of the study area. Ten test Pits were excavated and representative disturbed samples were collected for further analysis.

4.2.1 Data collection from preliminary survey

This is the collection of relevant data from the preliminary survey. At this stage, the existing site information was collected from various agencies or government organizations. Literature reviews from technical reports, published or unpublished papers, satellite images, topographic maps from the Ethiopian map agency, and online sources were used.

4.2.2 Data collection from Field survey

Through conducting several traverses across the town, Description of exposures along with the road and river cuts, quarry site, stream channels, slope cuts; measuring and recording the discontinuity characteristics; and confirming previous information with site geologic, physiographic feature. Identifying sampling location and deciding the number of samples. Generally, the following three major applications were done at times of field work.

4.2.2.1 Measurement, field description, and recording

For the preparation of geologic and engineering map of the study area, attitude (strike and dip direction) of geologic structure, extents, and length were measured and recorded by using field notebook, GPS (Geographic position System), clinometer, a geological hammer, and meter tape. Also, relevant field photographs that can provide valuable information were taken. The geodynamic process of the town and the risky environment was identified.

4.2.2.2 Sampling techniques

From identified areas, Disturbed samples of soils were collected from a maximum of four-meter depth for laboratory testing (Disturbed for index and engineering properties tests) and undisturbed sample prepared in the laboratory. Thirteen samples were collected from ten test pits with a maximum depth of four meters. Test pits manual dug by using a shovel and other materials. At the time of sampling vertical variation of a soil profile was observed and recorded.

4.2.2.3 In situ test

To determine the uniaxial compressive strength of rocks, in-situ tests were done on the exposed rock surfaces using an N-type Schmidt hammer. This test was carried out on a smooth and uniform rock surface according to the standard of ASTM C805 (2018), which is smoothed with the rubbing stone provided with the device. After rubbing, the rebound value of the Schmidt hammer was recorded from each identified location. This Schmidt rebound value was converted to unconfined compressive strength. Rebound value converted to unconfined compressive strength by using linear formula (Equation 3.9). The RQD by volumetric count method was conducted on an exposed rock surface. This method calculates rock quality designation by counting a number of discontinuities that lie within an area of 1m *1m, and RQD is calculated by the following relationship. RQD = 115-3.3Jv (Deere and Deere, 1988). Where Jv, known as the volumetric joint count, is the sum of the number of joints per unit length for all joint sets which have lengths ≥ of 10cm. In addition, the condition of discontinuity and general ground water condition of the exposed rocks were estimated in the field to estimate the quality of the rocks in the area. Finally, the rock mass rating was conducted by using different discontinuity parameters and RQD as an input parameter.

UCS=3.03RN-30.3 (Hu Wang *et al.*, 2016)…………………………………….. Equation 3.9

4.3.1 Laboratory work

Disturbed samples were collected from the study area and the following laboratory tests were conducted to check the index and engineering properties of the soil. At the time of collection, the soil samples were covered and sealed to ensure their naturality. Index and engineering tests were done according to international testing standards.

4.3.1.1 Index tests

That was including natural moisture content (NMC), specific gravity, Atterberg limits (liquid limit and plastic limit), grain size analysis, pH value, free-swell, and the (shear strength) is only an engineering test. Laboratory tests were done according to ASTM and AASHTO standards.

Natural moisture content (NMC

This test was performed according to ASTM D 2216 standard. Moist soil is placed in an oven-safe container and dried for 12-16 hours in a soil drying oven. The soil-filled container is weighed before and after drying to obtain M1 and M2, respectively, and w is calculated as (Equation 4.1) and (Equation 4.2):

$$w = \frac{Mw}{Ms} * 100 \text{ ……………………………………………………………..Equation 4.1}$$

$$= \frac{M2-M1}{M2-Mc} * 100 \text{……………………………………………………………………Equation 4.2}$$

Specific gravity

To determine specific gravity (ASTM D 854-92, 1988) standard was followed, and approximately 60g of dry soil was weighted to obtain Mo (Mass of dry soil). And Fill the flask to the etch line with distilled or demineralized water to obtain Ma (Mass water and pycnometer). Pour half of the water out of the flask and place the soil in the flask with a funnel and Wash the soil down the inside neck of the flask. Connect the flask to the vacuum source with the hose and stopper and apply vacuum for 30 minutes, occasionally the mixture agitated. The flask is filled to the etch line with distilled water and weigh to obtain Mb (Mass of soil, water, and pycnometer). And specific gravity can be calculated by using the formula (Equation 4.3).

$$Gs=\frac{Mo}{Mo+(Ma-Mb)}$$ ………………………………………..Equation 4.3

Grain size analysis

ASTM D 422-90, 1988 was applicable standards that were used for this test.

Mechanical Sieving

To perform grain size analysis, the wet sieving method was used, because, in some instances, information regarding fines content and coarse may be desired without the need to fully define the entire gradation curve. The direct use of Mechanical sieving may produce erroneous results on the gradation curve, because the smaller particles form clods and it may be labeled as coarse grain on the gradation curve, In wet sieving, the soil is combined with water and sodium hexametaphosphate and soaked for 24 hours to disperse the flocculated clay particles. Flocculation occurs because fine-grained soil particles are platy, and possess negative charges on their faces and positive charges on their edges. As a result, the particles are attracted to one another in an edge-to-face manner to form clods. Sodium hexametaphosphate neutralizes the surface charges on the clay particles, which disperses the particles and allows them to individually pass through the #200 sieve. The soaked soil sample was washed on sieve #200 till the clear course grain particles remain on sieve #200. The slurry is passed through a #200 sieve, which yields a more accurate estimate for the percentage of fines in the soil. The wet sample remains on #200 was dried in an oven dry and reshaked on a mechanical sieve to know the perfect gradation of the sample.

Hydrometer Analysis (ASTM D422)

To perform hydrometer analysis, the dried soil sample of 50g was passed through sieve #200 was soaked in water for 1 to 3 hrs. For 50 gm dry mass of fine-grain soil using 5 gm dispersing agent (sodium hexametaphosphate) soaked in 125 ml water and stir for about 10 min. Just after 10 min stir, transfer the slurry soil into 1000 ml volume of sedimentation cylinder, and the soil settlement is recorded in a series of time intervals by carefully inserting 151H stem in the solution.

Atterberg limit (liquid limit and plastic limit),

ASTM D4318: Standard Test Methods was followed to determine Liquid Limit, Plastic Limit, and Plasticity Index of Soils. To obtain liquid limit The fraction of the soil that passes through a #40 sieve used and distilled water was added to approximately 50 g of soil until it has the consistency of peanut butter. The paste was placed into the cup of the liquid limit apparatus at the point where the cup rests on the base. A flat layer of soil in the cup with the frosting knife and by using a grooving tool the paste was cut. Turn the crank on the liquid limit device at a rate of 2 cranks per second and Count and record the number of cranks that are required to close the groove over a length of 0.5 in. Then the sample was taken from the edge to edge of the soil pat, using the spatula and it was placed in the moisture can and immediately the weight of moisture can containing the soil was recorded and placed in the oven for 18 hours.

The sample of soil that passes through sieve #40 was used to obtain a plastic limit and some distilled water was added to make little mudballs. Pea-sized mudball taken and roll it out onto the frosted plate to form a rod with a diameter of 0.125 in. Until the soil crumbles the process of making a rod, was repeated. At this point (crumble), the water content of the soil is the PL. The portions of the crumbled threads were gathered together, placed in the moisture can and the weight of the moisture can containing the soil was recorded and left in the oven for 18 hours.

After conducting liquid limit and plastic limit tests plasticity index was determined from the result of consistency limits. It was calculated by using the following formulae (Equation 4.4)

PI = LL – PL…………………………………………………………………..Equation 4.4

Free-swell

To study the swelling property of the soils, the simplest test conducted is the free swell test. This test is performed by slowly pouring 10ml of oven-dry soil, which has passed the No. 4 sieve into a 100ml graduated cylinder filled with distilled (tap) water. After 24 hours, the final volume of the suspension being read. Hence, the free swell is defined as According to Chen (1975) Equation 4.5, the free swell is the difference between initial and final volume, expressed as initial volume.

$Free\ Swell = \frac{Final\ volume - Initial\ volume}{Initial\ volume} * 100$……………………………………Equation 4.5

Compaction

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 (W_m). 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 add water to achieve higher water content, w. finally optimum moisture content can obtain from oven-dried and maximum dry density calculated as (Equation 4.6) and (Equation 4.7).

$\gamma = \frac{M}{V}$……………………………………………………………………………….Equation 4.6

$\gamma d = \frac{\gamma}{1+w}$…………………………………………………………………………...Equation 4.7

Shear Strength Test (Direct Shear Test)

In this study, the direct shear test was conducted and it was done by following its test procedure. To conduct this test, first, the initial mass of soil in the pan, diameter, and height of the shear box was determined. Then the shear box was carefully assembled and placed in the direct shear device. The porous stone and the filter paper were placed in the shear box. After computing the weight of the soil pan and determining the mass of the soil, the assembly of the direct shear device was completed and the three gages (Horizontal displacement gage, vertical displacement gage, and shear load gage) were initialized to zero. Then the vertical load (or pressure) was set to a predetermined value, the bleeder valve was closed and the load was applied to the soil specimen by raising the toggle switch. Finally, the monitor was started with a selected speed, and the readings of horizontal displacement gage, vertical displacement gage, and shear load gage were taken. By using three gages reading, the maximum shear stress for each test was calculated.

4.3.2 Data analysis and interpretation approach

Analysis and interpretation of Insitu and laboratory data were done to convert the raw data of soils and rocks to meaningful information of the study area. Data processing and interpretation were conducted with the help of different software tools. Some of them are; Microsoft office tools for calculation, sorting, and tabulating raw data, Rose net projection to indicate geologic structure model of the area, Arc Map10.4, Golden Surfer 9.11 for preparation and manipulation of different maps, Google Earth Pro. to collect geospatial data. The major aims of analyzing and interpreting data are to classify the soil and rocks of the area, to identify problematic areas of town regarding civil engineering practice (potential expansiveness of soil), and to produce an engineering geologic map town.

4.3.3 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 British Standard system.

4.3.4 Soil plasticity analysis based on Casagrande

Laboratory results from different parts of the Town were plotted on a graph of plasticity index (ordinate) versus liquid limit (abscissa). It found that clays, silts, and organic soils lie in distinct regions of the graph. A line defined by the equation PI= 0.73(LL- 20) % called the "A-line," delineates the boundaries between clays (above the line) and silts and organic soils (below the line), A second line, the U-line, is expressed as PI= 0.9(LL -8) %, defines the upper limit of the correlation between plasticity index and liquid limit.

4.3.5 Potential soil expansiveness based free swell and activity of soil

Soil expansiveness is a serious engineering problem that leads engineering structures to crack, tear and tilt. Soil expansiveness is the functioning of two fine soil properties such as clay mineralogy and percent of clay in soil samples. From the results of free swell tests and activity, the degree of expansion of soil in the study area was determined. The activities of the soil sample of the study area are plotted on Skempton's activity chart to categorize the grounds of the town as active, inactive, and medium.

4.3.6 Classification of rock

Classification of rock depends on an analysis of field data. That was acquired through measurement and visual inspection. Such data include unconfined compressive strength, degree of weathering, and condition of groundwater, length, filling, and separation of discontinuity.

Relying on these parameters study area rocks are classified according to USC, RQD, and RMR standards.

4.3.6.1 Classification based on UCS

Schmidt rebound value obtained from the Schmidt test is converted to unconfined compressive strength by using formula(Equation 4.8), then the UCS value of rock categorize based on the ISMR standard.

UCS=3.03RN-30.3…………………………………………………………..Equation 4.8

4.3.6.2 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 following relationship. RQD = 115-3.3Jv. Where Jv, known as the volumetric joint count, is the sum of the number of joints per unit length for all joint sets which have lengths ≥ of 10cm.

4.4 Materials:

The following equipment is used for field and laboratory test

Table1.7 Materials

Field equipment and uses		Laboratory equipment	
Brunton Compass and GPS	To read direction and elevation	Materials	Lists of material
		The mechanical sieve analysis material	sieve stack; scale capable of measuring to the nearest 0.01 g; mechanical shaker Timing device
		Hydrometer analysis material	Balance, Soil hydrometer Control cylinder, Sedimentation cylinder Thermometer with clamp Beaker, Mixer, Dispersing agent,($NaPO_3$), Stopwatch
Geological Hammer	To estimate strength of rock	Moisture content test materials	Soil drying oven set at $110^{o} \pm 5^{o}C$; 3 oven-safe containers; and Permanent marker for labeling containers.
shovels	To take soil sample	Attenberg limit materials.	Ceramic soil mixing bowl; Soil drying oven Frosting knife; Liquid limit device; grooving tool, liquid limit apparatus
Sample bag		Specific gravity test materials	Scale capable of measuring to the nearest 0.01g;500-ml etched flask; distilled water; squeeze bottle; Thermometer capable of reading to the nearest 0.5^{o} C; funnel; Stopper and tubing for connecting flask to, vacuum supply;
Schmidt hammer	To know strength of rock	Compaction test material	Modified proctor compaction hammer; 4.0-in. diameter compaction mold, collar, and stand; Large screwdriver; spray lubricant; Ruler; Cutting bar;
Geological notebook	To record any geologic situation		

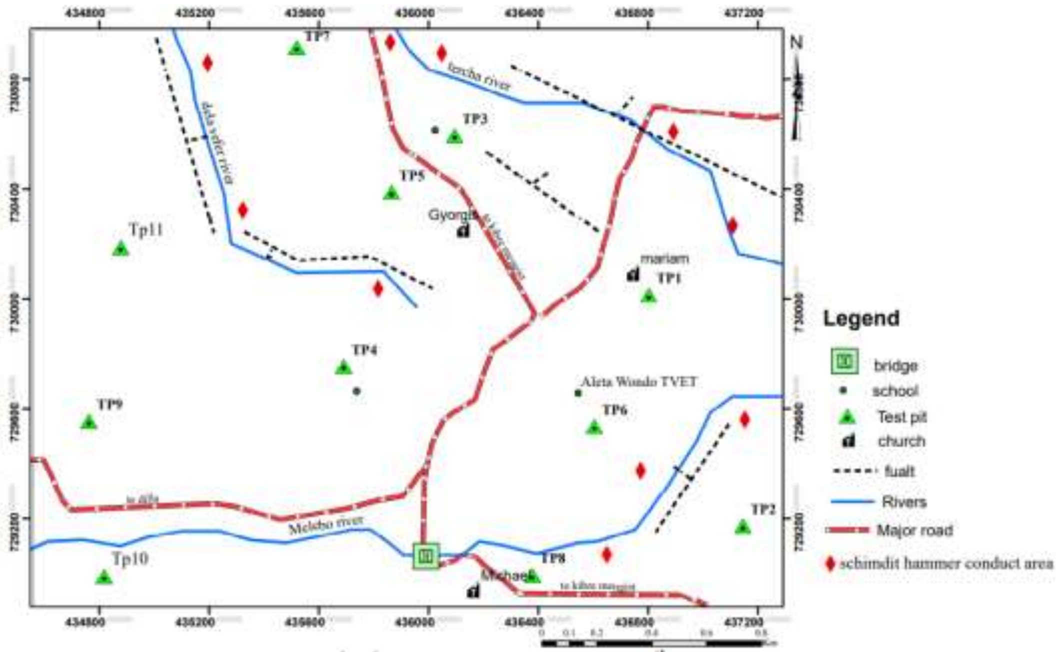

Figure1.15 Test pits and Schmidt hammer conduct location map of the study area.

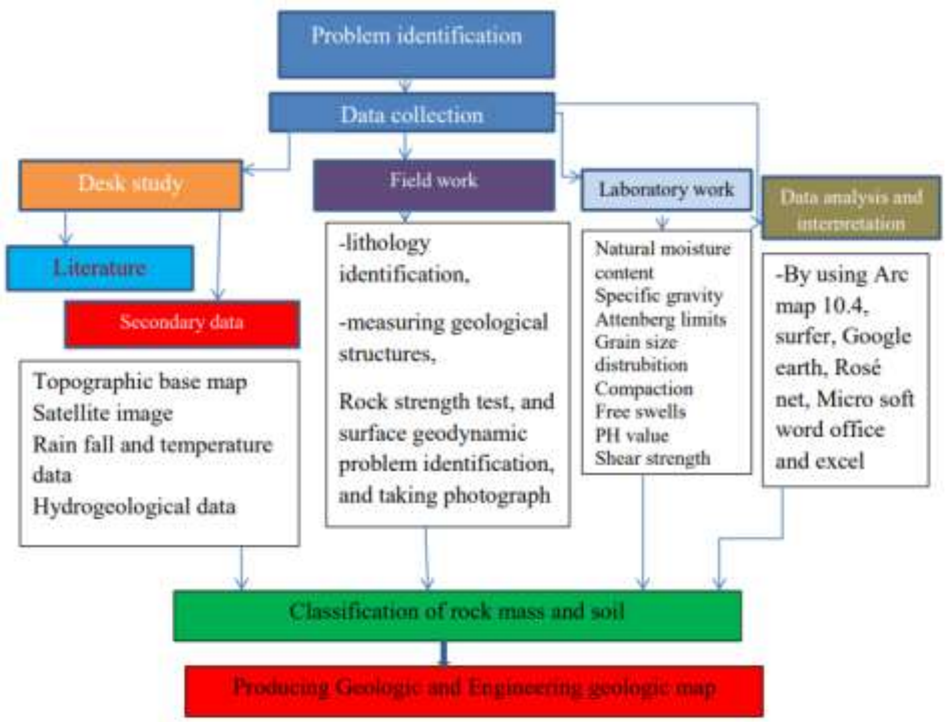

Figure1.16 Flow diagram of methodology

CHAPTER FIVE
RESULTS AND DISCUSSIONS

5.1 Engineering Geological Characterization and classification of Soils

5.1.1 Index properties

The summarized index property test results of study area are depicted in the following Table 1.8

Table1.8 Summarized index property test results of study are

TP	Depth (m)	NMC	SG	PD			LL	PL	Soil type		lithology
				%Sa	%Si	%C			USCS	BSS	
10	3	39	2.6	18.7	41.3	40	38.93	15.8	CL	CM	Clay
7	3	28.8	2.51	47	33.26	13	50.7	39.9	SM	SM	sand
1	1.5	31.64	2.47	7.4	61.95	30.65	71.76	41.6	MH	MH	silt
1	3		2.4	7.61	63.69	28.43	59	35.2	MH	MH	silt
2	3	34.9	2.5	10.5	54	36	57	41	MH	MH	silt
3	3	31.7	2.54	15.65	56.55	27.24	60.8	43	MH	MH	silt
4	3	27.4	2.52	10	58.8	32	66.7	49.5	MH	MH	silt
5	3	23.2	2.48	8.7	63.65	27.64	69.5	45	MH	MH	silt
6	1.5	30.7	2.55	8.93	73.9	18	62.3	48.6	MH	MH	silt
6	3		2.5	8.34	70	15	63	48.3	MH	MH	silt
8	2	48	2.6	22	49.5	28	42	19	CL	MH	silt
8	4		2.56	20	42.95	37	58.4	42.2	MH	MH	silt
9	3	35	2.52	10.2	64.59	25.21	62.4	47	MH	MH	silt

NMC=natural moisture content, PD= Particle size distribution, LL= Liquid Limit, Sa= Sand, Si= silt, C= Clay, MH=High plasticity silt, CL= Low plasticity clay, CM=Medium plastic clay, TP=Test Pit

The results and interpretation of tests on index properties of the soils are presented as below.

5.1.1.1 Natural Moisture Content

For many soils, the water content may be an extremely important index used for establishing the relationship between the way a soil behaves and its properties.

The graphical representation of the natural moisture content of the soil is presented as follow

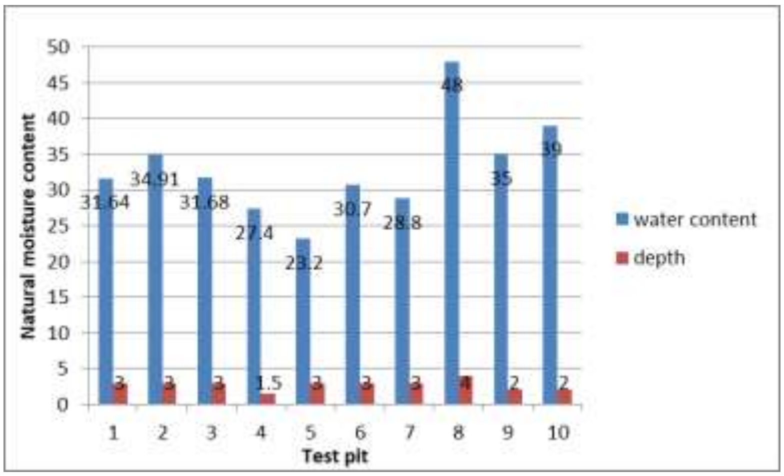

Figure1.17 Graph of natural moisture content of soils in the study area

.Based on the above figure1.17 the natural moisture content of soil study varies from 23.2 – 48 for silt soil, 39 for clayey silty, and 28.8 for sandy silt. The results imply the soil of the study area dominantly medium water holding capacity as the fine fraction is the dominant grain size.

5.1.1.2 Specific Gravity (Gs)

The specific gravity of soil is the ratio of the unit weight of solid soil to its unit weight of water. In Laboratory Specific gravity of soil is determined by putting a known weight of oven-dried soil sample in a pycnometer which is then half-filled with distilled water. The specific gravity of solids for most natural soils falls in the general range of 2.65 to 2.80; the smaller the values are for the coarse-grained soils. Specific gravity is used for the determination of void ratio and particle size.

Mathematically specific gravity determined by:

$$Gs = \frac{Unit\ weight\ of\ solid\ soil}{Unit\ weight\ of\ water}$$ ……………………………………..Equation 5.7

In study area-specific gravity range from 2.47-2.6 for silty soils and 2.61 and 2.55 for clayey and sandy respectively (Table 1.8)

5.1.1.3 Grain Size test result

To check index properties of soil of study area about thirteen samples from ten test pit was collected, as presented table 1.8 lithology of town call as **sandy** where percent sand 47 %, silt 37.5 and clay 13. **Silty** were, the percent of silt 37.5-73.9 %, sand 8.7 to 15.65 and clay 18-30.65. **Clayey** where a percent of clay 50, silt 31.6 and sand 18.4.as analysis indicate the study dominates with silt soil..

5.1.1.4 Atterberg (Consistency) Limits and Plasticity Index

Atterberg limit test was conducted in this research to obtain the basic index information and plasticity of the fine-grained soils used to identify the soils and to classify them.

According to Bowles (1992), soil with a greater plasticity index has greater engineering problems when using the soil as an engineering material like foundation support for residential building and road subgrades. As depicted in (Table 1.8). A laboratory test result of the soil in the study area shows that clays soils have LL and PL of 38.93 %and 15.8% respectively. Similarly, silty and sandy soils have LL ranges from 57%-71% and PL 41%-48.6% and 39. 9% and 50.7% respectively. The plasticity index of clayey and sandy soils is 23.23% and 10 % respectively. The plasticity index of silty soils ranges from 10%-23%. Soil with high plasticity index has less for engineering suitability, Soil with a plasticity index value >35 are very high plastic soils while soil with a plasticity index values from 20≤PI≤35 and 10≤PI≤20 are high and medium plastic soils respectively. In addition, soils with a plasticity index value≤of 10 are low plastic soils (seed *et al*., 1962). In the study area, sandy soil have a low plasticity index hence silty and clay soils have a medium to high plasticity index. So special consideration is needed at time construction especially southwestern- localities of town, this environment dominate with compressible clay and silt with high plastic index and shallow groundwater. The plasticity of soil sample based on this the plasticity class of the soils are presented on the plots on Casagrande plasticity chart (Figure 1.19), the soils in the study area falls from low plasticity to high plasticity ranges.

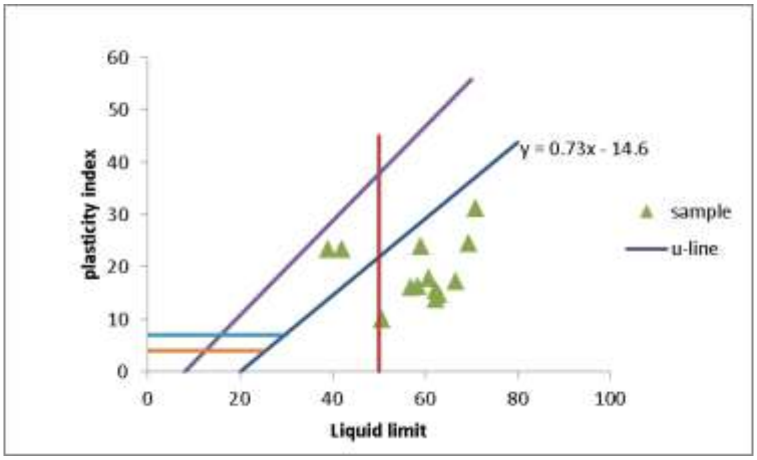

Figure1.19 Plots of soils of the study area on Casagrande plasticity chart.

5.1.1.5 Activity of Soils

Skempton (1953) proposed Activity of soil is as the ratio of the plasticity index and the percentage of clay fraction (Equation 5.3) (finer than 2µ).

Ac=$\frac{PI}{\%\ of\ caly\ fraction}$……………………………………..Equation 5.3

The activity is the measure of the water holding capacity of clay soil. The change in the volume of clay soil during swelling or shrinkage depends upon the activity, where the clay fraction is usually the percentage by weight of the soil less than 2µm, based on (Table1.10) Which have an activity of around (0.75 < A< 1.25) are classified as normal, A < 0.75 are classified as inactive clays and A > 1.25 are classified as active clays. Activity has been useful for certain classification and engineering properties, especially active and inactive clays. Also, there is a fair/good correlation of the activity and the type of clay mineral.

Table1.10 Activity range soil.

Activity	Soil type
< 0.75	In active clays
0.75-1.25	Normal clays
>1.25	Active clays

As it is recognized from laboratory test results; activity, swelling potential, and plasticity index in a soil samples are a roughly direct relationship with each other. But the percent of clay is inversely related to activity. The inverse effect of percent of clay fraction on the activity of soil always may not be correct. For instance, swelling potential or activity of clay soil is strongly related to the mineralogy of clay, not with the percent of the clay fraction. As it observed from analysis laboratory data, plasticity index has more relationship with the activity of soil rather than percent of clay. This is due to the plasticity index strongly match with the chemical behavior of clay soil. The soil sample plasticity index increases with activity, if there are active minerals like montmorillonite, and smectite whether the clay fraction increase or decreases. The clay fractions derived from pure quartz will never indicate water absorbance behavior because its is mineralogy is free from active behavior. Chemically, montmorillonite minerals have to expand lattices and exhibit a higher order of hydration and cation adsorption (https://web.viu.ca/earle/geol312/USC-anderson-clays). The degree of lattice expansion is dependent upon the nature of the cations adsorbed. IlIite, a common clay mineral, is sometimes described as a third type, but many investigators prefer to class it under the expanding lattice group. In illite there is a strong bonding of the silica sheets using potassium ions which reduces the expansion to very small amounts (Rich *et al.,* 1968,). Therefore the soils with low percent clay fraction can have high swelling potential if it contains montmorillonite group and vice-versa. Generally, the effect of clay percent is high if there is active minerals in soil sample otherwise the increases or decreases of clay fraction are meaningless on activity when soil samples have inactive mineral-like kaolinite. Clay fractions containing kaolinite will have relatively low activity and those containing montmorillonite will have high activity (C.Venkatra., 2013).

As observed from (Table 1.11) category and activity soil plot **(**Figure 1.20), Aleta Wondo Town soil distribution dominantly inactive and normal, The activity of the soil of study area was determined by using the plasticity index and grain size analysis results of the soils using Skempton's description of the activity of soils

Table1. 11 Activity of study area soils

TP	Depth(m)	dominant	PL	LL	PI	CI	Activity	remark
10	1-2	clay	15.8	38.93	23.3		0.612	active
7	1-3	sand	39.9	50.7	10		0.76	normal
1	1.5	silt	41.6	71	31.16		1.016639	normal
	3		35.2	59	23.8		0.837144	normal
2	1-3		41	57	16		0.444444	inactive
3	1-2		43	60.8	17.8		0.653451	inactive
4	1-2		49.5	66.7	17.2		0.5375	inactive
5	1-3		45	69.5	24.5		0.886397	normal
6	1.5		48.6	62.3	13.7		0.761111	normal
	3		48.3	63	14.7		0.98	normal
8	2		19	42	23		0.812143	normal
	4		42.2	58.4	16.2		0.437838	inactive
9	1-3		47	62.4	15.4		0.610869	inactive

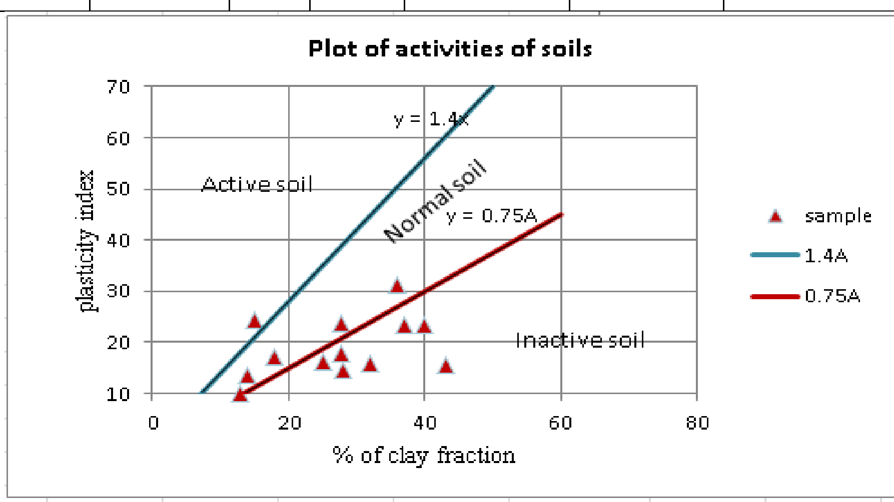

Figure 1. 20 Plots of soils of the study area on Skempton's activity chart

5.1.1.6 Liquidity Index (LI) and Consistency Index (CI)

The liquidity index of a soil indicates the nearness of its water content to its liquid limit. When the soil is at its liquid limit, its liquidity index is 100% and it behaves like a liquid. When the soil is at the plastic limit, its liquidity index is zero. Negative values of the liquidity index indicate water content smaller than the plastic limit (Arora, 1997). These indexes are determined from natural moisture content, Liquid limit and Plastic limit using mathematical formulae (Equation 5.4)

$LI = \frac{w-PL}{PI}$……………………………………..Equation 5.4

The Liquidity index of soils of Aleta Wondo town has determined from ten test pit value ranges from 1.285 to -1.26. As presented in Table 2.11 most of the soil samples of liquidity index value negative except TP10 and TP8 their liquidity index 1.003 and 1.26 respectively. Similarly, the consistency index of the soil study area is presented in (Table 1.12). The consistency index indicates the firmness of soil. It shows the nearness of the water content of the soil to its plastic limit. Soil with a water content equal to the plastic limit has a consistency index of 100%, which shows that the soil is relatively firm. The consistency index value of the study area range from -0.003 to 1.779. Most of the study area soil firm except TP10 & TP8. To generalize liquidity index and consistency analysis result, majority of study area soil are hard and firm to stiff.

Table1. 12 The liquidity, plasticity and consistency index of Aleta Wondo Town

TP	Depth(m)	dominant	PL	LL	PI	CI	LI
10	1-2	clay	15.8	38.93	23.3	-0.003	1.003
7	1-3	sand	39.9	50.7	10	2.028	-1.028
1	1.5	silt	41.6	71	31.16	1.339	-0.339
	3		35.2	59	23.8	1.109	-0.109
2	1-3		41	57	16	1.381	-0.381
3	1-2		43	60.8	17.8	1.636	-0.636
4	1-2		49.5	66.7	17.2	2.285	-1.285
5	1-3		45	69.5	24.5	1.890	-0.890
6	1.5		48.6	62.3	13.7	2.307	-1.307
	3		48.3	63	14.7	2.109	-1.109
8	2		19	42	23	-0.26	1.26
	4		42.2	58.4	16.2	0.747	0.253
9	1-3		47	62.4	15.4	1.779	-0.779

5.1.1.6 Free Swell

The free swell test is one of the easiest and essential index characters of fine-grain soil because it indicates the presence of swelling clay among the sampled soils due to its mineralogy.
As mentioned by Arora, the swelling potential of soil is related to the activity of soil, activity itself is related to a plasticity index of soil. Free swell or expansion of clay soil Couse failure for engineering structure, according to Holtz and Gibbs (1956) as stated in Hawi (2018), soils having a free swell value below 50% rarely indicate extensive volume change even under very light loadings. If soil's free swell value is about 100% and exceed, it can cause considerable damage to the lightly loaded structures.

The free swell value results of the study area range from 15.9% to 60% for silty soil and 19.5% and 54% for sandy and clayey soil respectively (Table1.13).

According to IS categorizing system of soil-based swell index result, Aleta Wondo characterizes low to medium degree of expansion. Hence for the samples with a free swell of greater than 50% consideration should be taken during the design of civil structures to be founded on such soils because their free swell value exhibits the swelling potentiality behavior. Especially TP1, TP10, and TP5 need special consideration.

Table1. 13 Free swell chart of study area

TP	Depth(m)	dominant	PL	LL	PI	FS
10	1-2	clay	15.8	38.93	23.3	47
7	1-3	sand	39.9	50.7	10	34.5
1	1.5	silt	41.6	71	31.16	60
	3		35.2	59	23.8	
2	1-3		41	57	16	13.9
3	1-2		43	60.8	17.8	18
4	1-2		49.5	66.7	17.2	27.5
5	1-3		45	69.5	24.5	50
6	1.5		48.6	62.3	13.7	21.7
	3		48.3	63	14.7	
8	2		42	65.3	23.3	15.9
	4		42.2	58.4	16.2	
9	1-3		47	62.4	15.4	18.2

5.1.1.8 PH (power of Hydrogen)

It is a measure of the acidity of the soil. It is numerical differences that range from 0-14 on the PH. scale. The geotechnical properties of soils are greatly affected by the change in PH. It is essential to determine the aggressiveness of the soil to the engineering structure. Dissolution reaction is higher at highly acidic and alkaline PH conditions that can deteriorate (corrode) concrete of engineering structure (Yadeta. C, 2020). The consistency limits decrease with an increase at the varied PH. The changes in PH cause cation exchange and this weakens the electric attraction force between the soil particles and invariably reduction in strength of Laterite soil (Olukorede *et al.,* 2012). The PH value also affects the shear strength of granular soil, as was founded by Md. (Motiur *et al.,* 2015), shear strength increases with an increase in PH value. In the study area pH value ranges from 7.11 to 7.31 (Table1.14**)**. That means slightly basic.so; there is no much adverse problem that leads civil engineering structure to failure.

Table1.14 pH value of study area

Test pit	Depth	pH value
1	3	7.35
2	3m	7.12
3	1 m	7.8
4	3m	7.25
5	3m	7.15
6	3m	7.51
7	3m	7.26
8	4m	7.18
9	2m	7.17
10	2m	7.11

5.1.1.9 Compaction

Compaction is the most popular technique for improving soils. The soil fabric is forced into a dense configuration by the expulsion of air using mechanical effort with or without the assistance of water. The benefits of compaction are; Increased soil strength, increased load-bearing capacity, Reduction in settlement (lower compressibility), Reduction in the flow of water (water seepage), Reduction in soil swelling (expansion), and collapse (soil contraction), increased soil stability. To reach above-mentioned benefits the basic two dependent parameters from laboratory works are determined i.e. Optimum moisture content and maximum dry density. From laboratory measurement bulk masses of compacted soil from each trial are measured and corresponding moisture content values are determined. Optimum moisture content and maximum dry density result of study area presented (Table 1.15)

Table1. 15 Compaction test result of study area

TP	depth	ρd(kg/m3)	Wopt%
1	1.5	1.562	32.8
2	3	1.604	28.5
3	3	1.38	33
5	3	1.22	30.5
7	3	1.424	31
8	2	1.487	28.8

Geotechnical engineers check field soil compaction based on relative compaction value, as per Arora, the dry density achieved in the field is compared with the maximum dry density obtained in the laboratory. The ratio of the dry density in the field to maximum dry density is known as Relative compaction.

$$\text{Relative compaction} = \frac{\text{dry density of feild}}{\text{maximum dry density}} * 100 \text{ ....Equation 5.10}$$

According to Arora, cohesive soil must be attained at least 95% of relative compaction to overcome the geotechnical problems, study area dominates with cohesive silty clay soil therefore, Aleta wondo soil field compaction value must be greater than 95% if it compared with these laboratory results.

5.1.1.10 Direct Shear Test

In this study, representative six samples were selected that from different engineering classes. The engineering properties and shear strength parameters for selected samples are presented in the following table 1.16. The results indicate the cohesion of the soils varies from 22.86 to 49.37 kpa for compressible silt, whereas the angle of shearing resistance varies from 19˚ to 22.88˚, for Low plasticity Clay with silt cohesion and internal friction, 54kpa, and 18° respectively.

Table1. 16 Shear strength result of the study area

TP	lithology	Engineering class(USCS)	depth	Shear strength	
				Cohesion(Kpa)	Internal friction angle
1	silty	Compressible Silt with clay	1m	21	22.88
2	silty	Compressible Silt with clay	3m	38.4	22.5
6	silty	Compressible Silt with clay	1.5m	49.73	20.56
7	sandy	Compressible Silt with clay	3m	21.16	22.88
8	silty	Compressible Silt with clay	4m	36	19
10	clayey	Low plasticity Clay with silt	3m	54	18
11	sandy	Compressible Silt with clay	3m	22.12	22.99

5.2 Soil classification

Soil classification is used to specify a certain soil type that is best suited for a given application.it is a bridge process to fill the gap between engineering and geologic characters of soil and rock for engineering purposes. There are several classification schemes available. Each was devised for a specific use. For example, the American Association of State Highway and Transportation Officials (AASHTO) developed one scheme that classifies soils according to their usefulness in roads and highways, while the Unified Soil Classification System (USCS) was originally developed for use in airfield construction but was later modified for general use(Budhu M., 2010).In the study area the soil samples taken from selected positions are classified according to USCS and the British standard system (Figure1.21**)**. Based on USCS Aleta wondo town characterize compressible silt and low to medium elastic clay types. According to the British Standard System study area soil is classified as compressible silt, medium plastic inorganic clay.

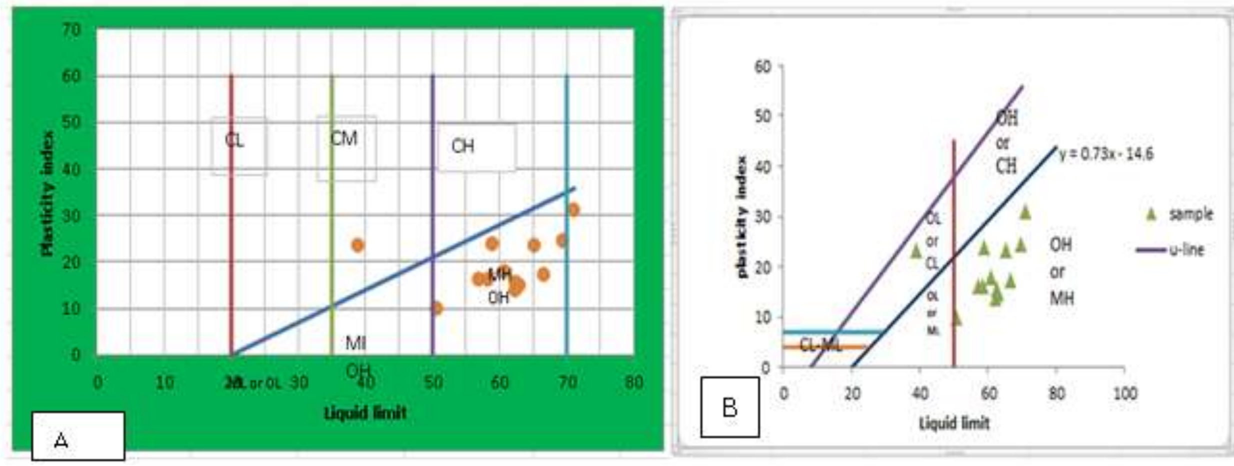

Figure1.21 Study area soil classification according to BSS (A) and USCS (B)

5.3 Engineering Geological Characterization and Classification of Rocks

In the real sense, rocks are hard more rigid naturally occurring aggregates of minerals connected by strong bonds.

The need for characterization of rock is that the rock will be used either as a building material so the structure will be made of rock or a structure will be built on the rock, or a structure will be built in the rock. In the majority of civil engineering cases, rock is removed to form the structure in. for example, the excavation of rock for a hydroelectric machine hall chamber. In this sense, there is dealing with a reverse type of construction where the rock material is being taken away, rather than added, to form a structure. On the mining side, rock may be excavated in an open pit and we will then be concerned with the stability of the sides of the open pit.

In the study area, to determine the Schmidt rebound value of rock, nine testing area was identified and from each place average of ten reading was taken, Schmidt hammer rebound value was recorded on each type of the rock at different locations To classify the quality of rock in the study area for engineering purpose, Schmidt rebound value converted to unconfined compressive strength by using formula (Equation 5.1) (Hu Wang a, *et.al* 2016).

UCS=3.03RN-30.3……………………………………………………………………Equation 5.1

Intact rock can be classified according to (Table1.17) the uniaxial compressive strength.

Table1. 17 Classification of Rock according to USC

USC(Mpa)	description
Greater than 200	Very high strength
56-200	High strength
21-55	Medium strength
8-20	Low strength
Less than 8	Very low strength

After (ISRM, 1981)

Rock quality designation was identified by using the volumetric count method, weathering condition, and condition of discontinuities from visual observations. Finally, the quality of the rock masses in the study area was calculated by adding the five basic parameter rating values according to Bieniawski (1989).

Table1.18 Uniaxial Compressive Strength of rocks of the study area.

No	Northing	Easting	Rock genesis	Rebound value	UCS(Mpa)	Description
1	729619	437136	basalt	45.6	107.868	High strength
2	729306	436795		42.9	99.687	High strength
3	729823	437267		42.4	98.172	High strength
4	729107	436596	ignimbrite	40.5	92.415	High strength
5	730932	435868		42.1	97.21	High strength
6	730312	435323		25	45.45	Medium strength
7	730568	436965		22	36.36	Medium strength
8	730312	437125		10	12.12	Medium strength
9	730919	436008		6	3.03	Very.Low strength

Figure1.22 Conducting Schmidt hammer test on ignimbrite, the northern part of town at the quarry site

5.3.1 RQD Determination by Volumetric Count Method

RQD related with the drill core it was determined from the summation of discontinuity having a length greater than 10cm per total core run, but, in a case where there is no drill core data, rock quality designation was determined based on the joint volumetric count method. The volumetric count method is more advantageous than the drill core because volumetric count reduces the directional dependence (Palmstrom, 1982). Therefore, since there was no rock core drill data in the study area, rock quality designation was estimated using the joint volumetric count method (Equation 5.2).

RQD=115-3.3Jv. ……………………………………………………………………Equation 5.2

Where Jv is the total number of discontinuities more than 10 cm long in 1 x 1 m exposed rock face (Volumetric count)

A joint trace exposed on ignimbrite rock with UTM position(x=436087, y=730949) (Figure 1.23) Total number of discontinuities its length greater than 10cm is 12, which means Jv=12. the average joint spacing is 33cm and the condition of the joint is slightly rough and slightly weathered with separation less than 1mm, as it observed the groundwater condition is completely dry. RQD=115-3.3*12 =75.4

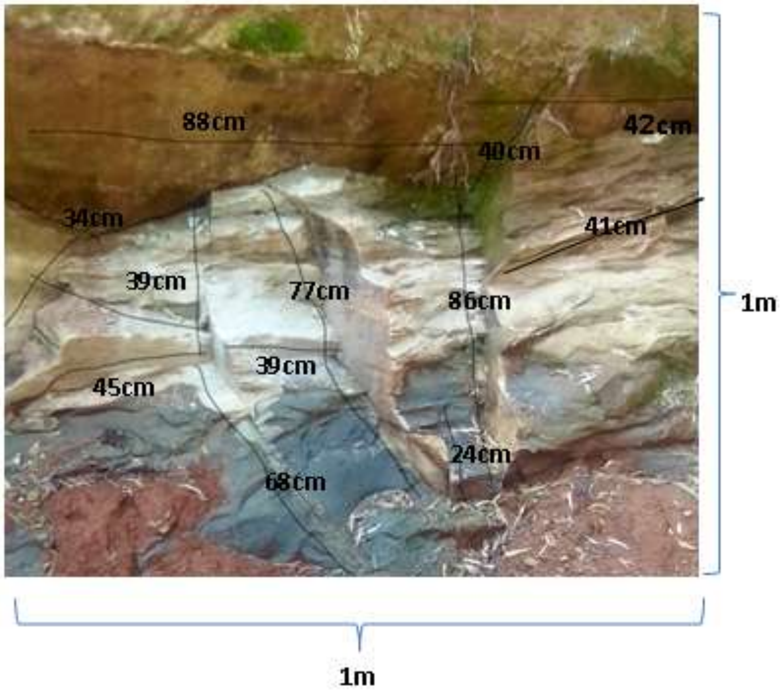

Figure1.23 Joint trace exposed in study area

The remaining seven UTM locations are determined in similar way and presented on the following table1.19.

Figure1.24 Rock mass condition exposed at the northern part of town

Table1.19 RQD and RMR of rocks of the study area

No	Northing	Easting	elevation	Rock genesis	USC	R	RQD	R	JS(cm)	R	CD	R	GWC	R	RMR	D
1	730070	435767		basalt	107.868	12	69	13	35	10	SR&SW	25	dry	10	70	good
2	730366	435298			99.687	7	66	13	39	10	SR&SW	25	dry	15	70	good
3	730870	435065			98.172	7	71.3	13	38	10	SR&SW	25	dry	10	65	good
4	729107	436596		ignimbrite	92.415	7	65.4	13	33	10	SR&SW	25	dry	15	70	good
5	730932	435868			97.263	7	74	13	24	10	SR&SW	25	dry	10	65	good
6	730312	435323			45.45	4	66	13	18	8	CSS	10	wet	7	42	fair
7	730568	436965			36.36	4	48	8	19	8	SR&HW	20	wet	7	47	fair
8	730312	437125			12.12	4	58	13	17	8	SR&HW	20	dry	10	55	fair
9	730919	436008			3.03	2	42	8	13	8	SR&HW	20	Dr	4	42	Poor

SW=slightly weathered, HW=highly weathered, SR=slightly rough, CCS=continuous slickenside, R=rating, JS=joint space, GWC=ground water condition, CD=condition of discontinuity, RQD= rock quality designation, RMR=rock mass rating, D= description, Dr=dripping

Meaning and description of RMR

RMR rating	**81-100**	**61-80**	**41-60**	**21-40**	**<20**
Rock mass class	**A**	**B**	**C**	**D**	**E**
Description	**Very good**	**Good**	**Fair**	**Poor**	**Very poor**

After Bieniawski (1989).

Based above-mentioned chart, Aleta Wondo Town is characterizes by rock mass classes “B” and “C”. The quality description of rock mass in the study area for engineering purposes falls in Good to Fair.

Relationship of RQD and UCS with RMR

The RMR system gives a number between 0 and 100. Determining the RMR value of rock mass of study is important because the RMR value approach to the real ground situation is more than RQD and USC. Classifying rock based on RMR good approach to know the engineering quality of rock. If there is no RMR classification approaches. Classification may be based on RQD and UCS value, to know which classification system is more reliable among both systems (RQD and

UCS), a comparison between RQD and RMR – USC with RMR takes place. From this analysis RQD is more related to RMR with a correlation factor of 0.795 (Figure 2.17). There is also a positive correlation between USC and RMR but it is not as much as RQD. To generalize, classification of rock mass quality based on RMR and RQD can express more real ground conditions.

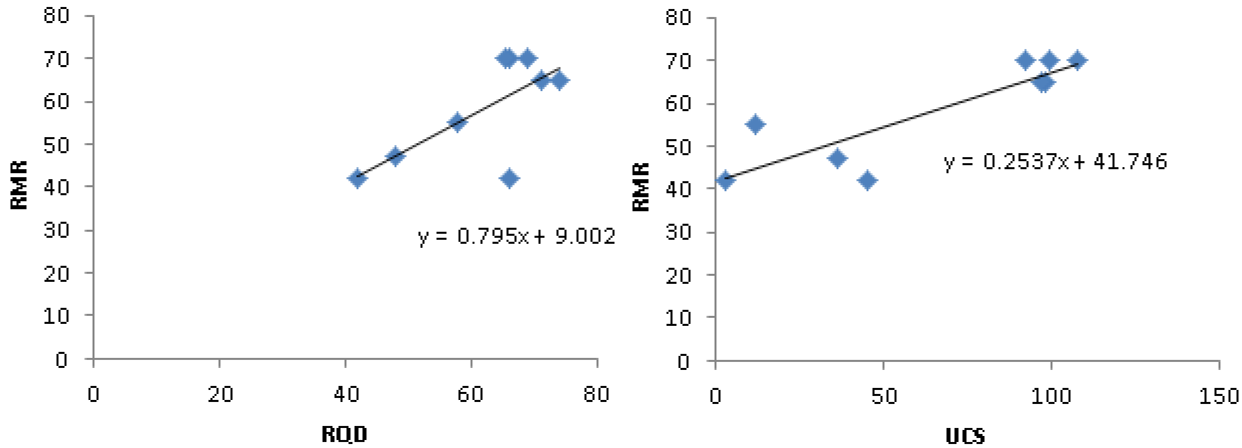

Figure 1.25 relationships between RMR, RQD and UCS

5.4 Assessment of hydrogeological, geomorphological, and geodynamic Hazards

A surface geodynamic problems in the study area is landslide, surface erosion, gully erosion, 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. Geo-hazards 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 (e.g., tsunamis). Other Geo-hazards can be earthquakes, volcanoes, Avalanches and Floods (Burton et al 1978). Geohazard can be: Earth quack, Landslide, Flood and volcanoes. Aleta wondo town found at southwestern position of main Ethiopian rift with the transitional zone between main Ethiopia rift system and Ethiopian plateau. For this reason, there is no considerable volcanic hazard activity in the town. Landslide, gully erosion and shallow ground water are the most commonly observed geodynamic and hydrogeological possess responsible for damaging of engineering structures and changing the surficial features of the study area.

5.4.1 Landslide

Landslide, also called landslip, the downslope movement of a rock mass, debris, earth, or soil (soil being a mixture of earth and debris). 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, or the accumulation of debris on the slope's surface. Landslides are generally classified by type of movement (slides, flows, spread, topple, or fall) and type of material (rock, debris, or earth). Sometimes more than one type of movement occurs within a single landslide. At study area, small scale soil slide observed specially along slop cut of north western and southern part of town **(**Figure 1.26). this process damages the road and close water ditch of town.

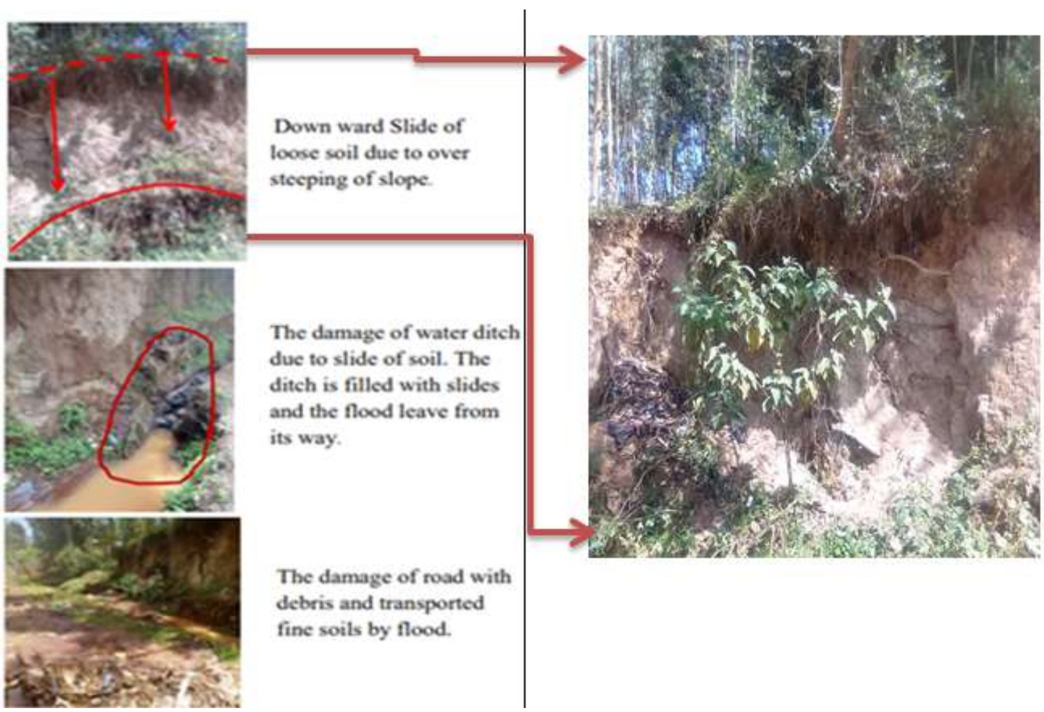

Figure1. 26 Landslides and its effect on study area

5.4.2 Erosion and Weathering

Northwestern part of the town experienced gully erosion due to steeply slop topographic feature of land. For this reason, Roads and residential houses frequently affected by flood. 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 study area, this indicate the study area experience *in situ* chemical weathering rather than mechanical weathering. There is also highly to slightly weatherd igmbrite rock (Figure 1. 27) observed along river cut of northern part of town.

Figure1. 27 Weathered ignimbrite rock units with in study area

Weathered ignimbrite rock units with in study area (A), errosion on white clay soil(B), gully erosion area but it covered with trees and grasses. Before a years this area was cliffs and precipitous, and the cliffs was filled by the soil removed from construction of route from Aleta

Wondo to k.mengist. Currently the damped soil is eroding and sign of landslide observed, like tilting of trees and fences(C) and lateral erosion (D)

5.4.3 Groundwater

Groundwater is found beneath the Earth's surface in soil pore spaces and rock formation fractures. The flow of groundwater below the surface is a fundamental property that controls the strength and compressibility of soil impacting soil's ability to hold up on structural loads. When soil is saturated, the soil media takes on very specific physical characteristics due to the relative incompressibility of water. These characteristics come into effect below the groundwater-surface or table.

Common groundwater issues during construction: Unstable subgrade, unstable excavation and water seepage and Construction delays and cost overrun

Common groundwater problems after construction: Water leaks, wet basements, and mold growth, Cracked and uneven floors, Cracked and uneven walls, unstable slopes and retaining walls, Delayed movements of foundations

Aleta wondo town is characterized by shallow groundwater especially in southwestern and south-central parts of town. as observed from field survey task, the depth of groundwater not more than four meter at the southeastern part of town due to this people can fetch the water from borehole easily (Figure 1.28) But the depth increase up to 12m at the north direction of town. The depth of groundwater level gradually increases from south to north direction as recognized from borehole data. The overview of the regional GW level of Aleta Wondo town is estimated from nine boreholes data (Table1.20**)**. From the values of the GW level contours, the general direction of GW flow was determined and flows from the North to South direction.

Figure1. 28 Groundwater image captured from Southeastern part of town

Table 1.20 Static water level and groundwater level in the study area

Northing	Easting	Elevation(m)	GWL(m)	SWL(m)
435411	729454	1905	1901.5	3.5
435489	729374	1901	1896.2	4.8
435463	729336	1902	1899	3
436141	729445	1908	1905.5	2.5
436182	729440	1906	1904.3	1.7
436359	729446	1906	1903.8	2.2
436722	729781	1927	1916.8	10.2
435561	720495	1991	1979	12
436892	720022	1920	1905	15

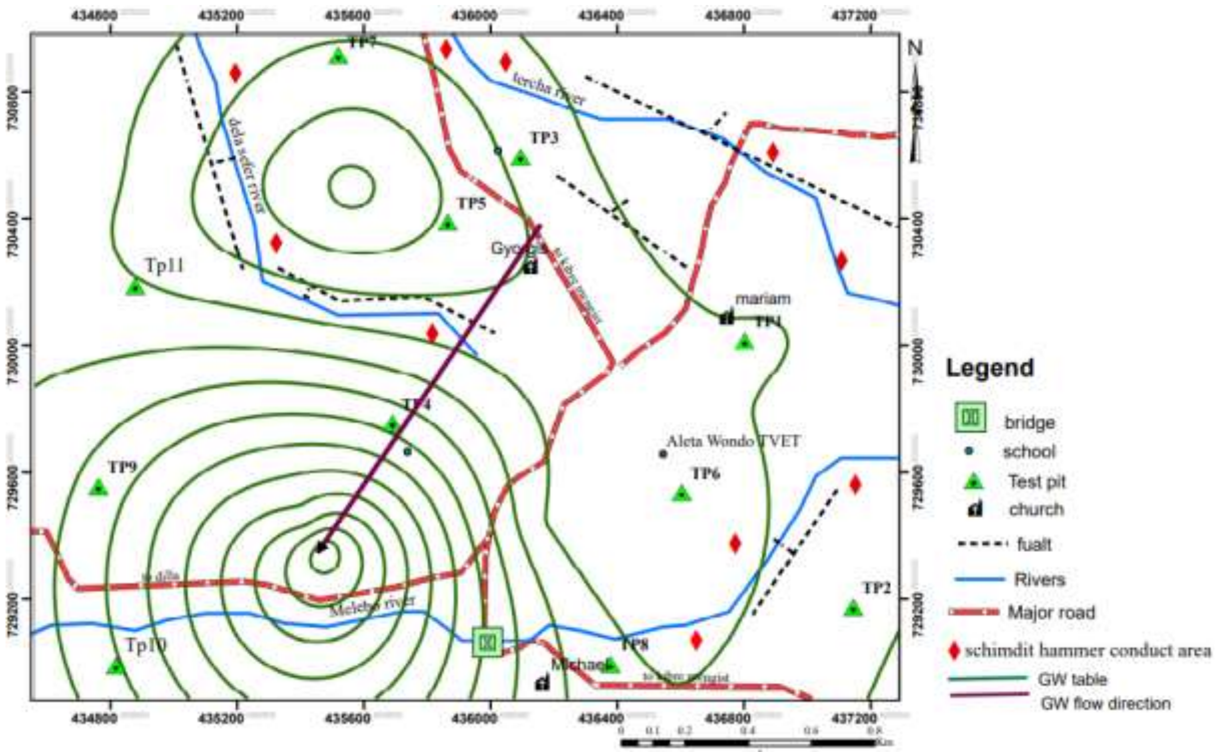

Figure 1.28 Groundwater level and flow direction in Aleta Wondo Town

5.5 Preparation of multipurpose Engineering Geologic Map.

For the current study, multipurpose EGM was prepared by using engineering properties of soils and rock such as strength (unconfined compressive strength), and degree of weathering. Therefore similar lithological units mapped as different units according to their engineering properties. Moreover, geodynamic phenomena (gully erosion and flood-risk) and groundwater condition of the town is documented on the following map (Figure 1.29)

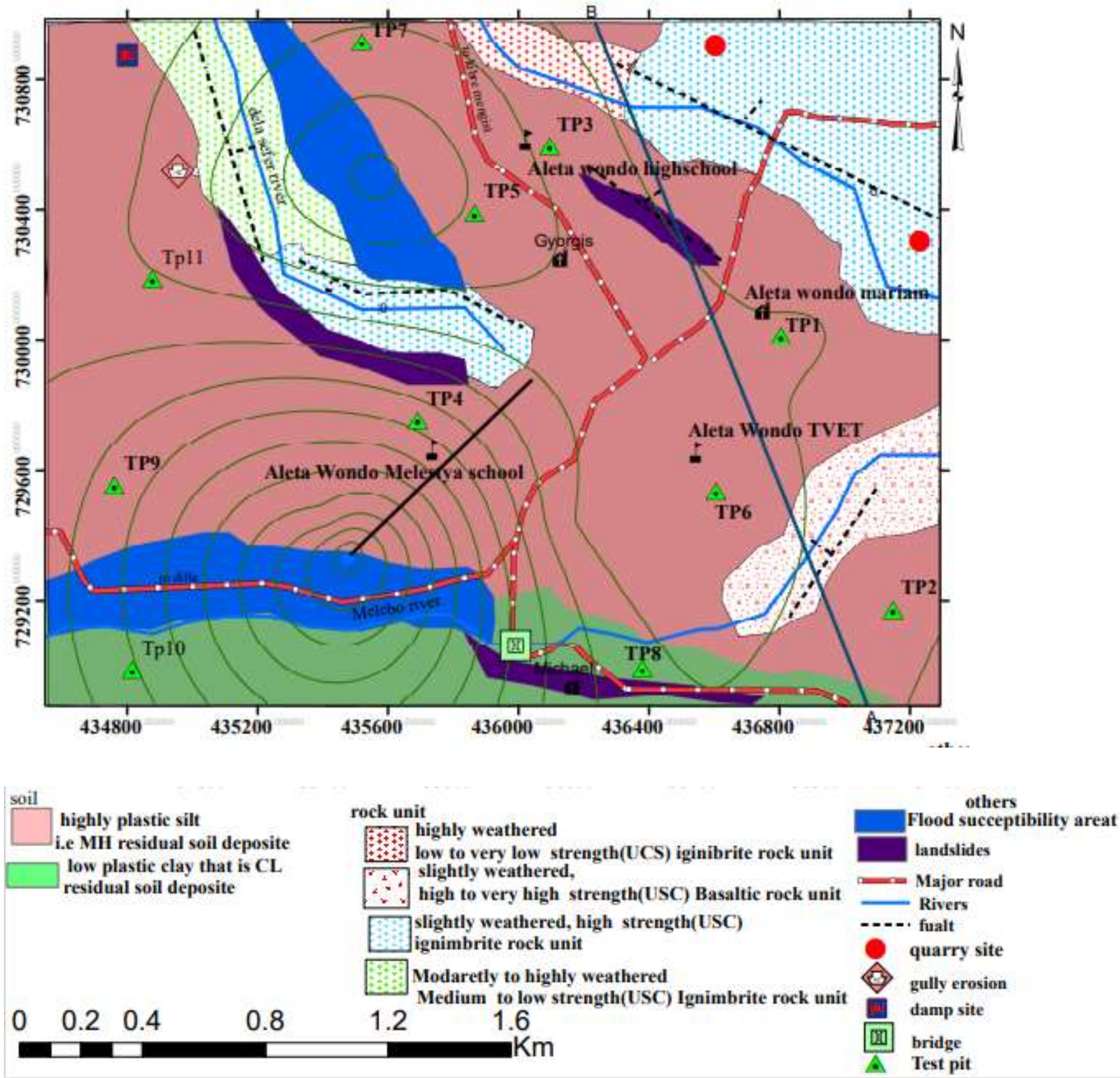

Figure1.29 Engineering geologic map sheet.

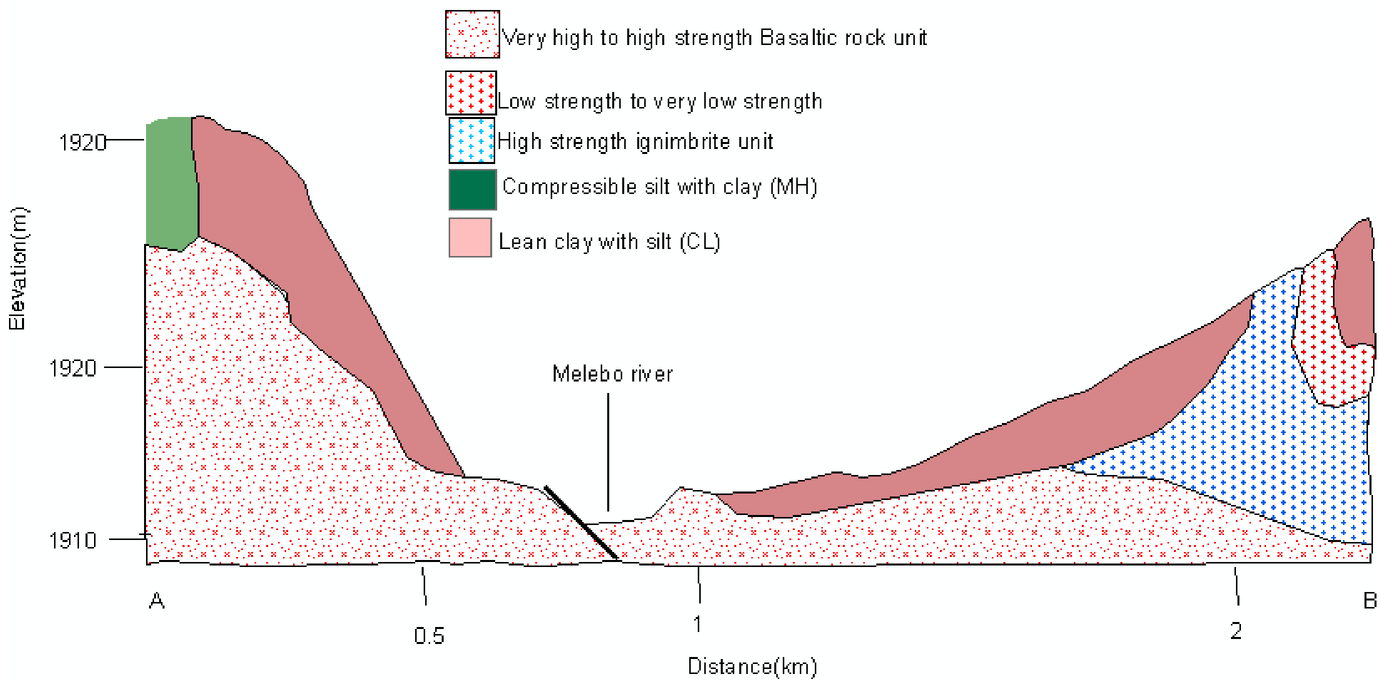

Figure1.30 Engineering geologic map cross-section of the study area

Engineering geologic map sheet, the map sheet contains an engineering unit of rock and soil with major geologic structure. The boundary of the map sheet extends the current town master plan program

5.5.1 Engineering Geological Map Units

Engineering Geologic map contains different engineering soil and rock unit. Similar geologic units mapped as different lithology based on their engineering properties and quality.

5.5.1.1 Rock Units

Based on the unconfined compressive strength, the study area of rock formation is classified into three different engineering units, such as High strength ignimbrite unit, Very high to high strength Basaltic rock unit, Medium strength ignimbrite rock unit, and Low strength to very low strength ignimbrite rock unit

5.5.1.1.1 High strength ignimbrite unit

The rock under this category is exposed along the small stream of the central part of town and northeast of the quarry site. The intact rock strength of rock masses of this class ranges from about 92.415-97.21Mpa. The Schmidt hammer rebound values of this class of rock unit generally range from 42.1 to 41 and they can’t break easily by a simple test of the geologic hammer. The intact rock strength class for this unit is highly strong and slightly weathered. The discontinuity characters of this rock unit are close to wide spacing with a very low aperture.

5.5.1.1.2 Very high to high strength Basaltic rock unit

This unit of rock is the geologically oldest basement rock, it is found at a great depth below the surface. For this reason, it’s not exposed well around the study area. A few outcrops along Melebo River at the southeastern part of town with slightly weathered, dried groundwater condition and discontinuity without fills observed. The intact rock strength of this class ranges from about 98.172-107.868Mpa. The Schmidt hammer rebound values of this class of rock masses generally range from 42.4 to 45.6. The intact rock strength class for this unit is highly strong;

5.5.1.1.3 Medium strength ignimbrite rock unit

This unit exposed the Northwestern corner of the study area its engineering properties less favorable when compared with another exposed unit. The Schmidt hammer rebound values of this class of rock masses generally range from 10 to 25. The intact rock strength class for this unit is medium. The groundwater condition is wet to dry and the joint spacing is 13 to 19mm. the discontinuity roughness is slickenside to highly weathered. This category of a rock unit is high to moderately weathered.

5.5.1.1.4 Low strength to very low strength

This category of rock is exposed north of town near to Hawassa route and is highly weathered with groundwater conditions. Because of weathering, discoloration extends to the greater part of the rock. The intact rock strength of rock masses of this class less than 10Mpa. The Schmidt hammer rebound value is 3Mpa and they can break easily by a simple test of the geologic hammer. The intact rock strength class for this unit is very weak. and highly weathered with a wide aperture discontinuity condition.

5.5.1.2 Soil unit

Soil unit of study area mapped as USCS classification result. Similar lithology study area soil mapped as different engineering unit. According to the USCS standard of classification, in the study area, two engineering units of soil were identified.

5.5.1.2.1 Compressible silt with clay (MH)

Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts. This unit covers a large part of town. The general engineering characters of this category of soil are semi-pervious to pervious, high compressibility, and fair to poor for engineering work.

5.5.2.2.2 Lean clay with silt (CL)

Inorganic clays of very low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. This unit is exposed in the southwestern part of town. According to the USCS of classification, it is impervious, medium compressibility, and fair for engineering work.

5.5.2 Engineering geologic problem encountered in Aleta Wondo Town

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, shrink upon drying and expand upon wetting, partially impervious and generally poor material for backfill and embankments and the slopes with clay material are prone to landslides. Similarly, high capillarity and frost susceptibility, relatively low permeability, and low shear strength are the general engineering characteristics of fine-grained inorganic silt soils (Zhou, 2006). 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 ML and CL. The general engineering characters of this category of soil are semi-pervious to pervious, high compressibility, and fair to poor for engineering work. For this reason sign of settlement was observed on the large building in the study area. The shallow groundwater condition is the second most engineering problem in the study area.

CHAPTER SIX

CONCLUSION AND RECOMMENDATIONS

6.1 Conclusion

- Geological and engineering geological map of the town was produced at a scale of 1:20,000. Two lithological /rock type's ignimbrite and basalt and residual soil were identified.
- The groundwater of the study area is found at shallow depth.
- The geological structures identified in the study area are major faults and small scale joints with the strike of NNW-SSE and the dip direction of NE and SSW
- Geomorphological conditions of the study area consisting of flat to undulated topography, deep gorges, river dissections, steep slope feature is the result of its relative closer location to the rift margin (escarpment).
- From laboratory test results
    - ✓ The moisture content of the soils varies from 23.27 to 48.
    - ✓ The Grain size analysis result of the study area shows the fine grain.
    - ✓ Laboratory consistency limit test result of the soil in the study area shows that clays soils have LL and PL ranges of 38.93 %and 18.5 %respectively. Similarly, silt soils have LL ranges of 57%-71% and PL ranges of 41%-48.6%. The plasticity index of clay soils is 23.23% and the plasticity index of silt soils ranges from 10%-24%.
    - ✓ The activity of soil of study area 0.7692 for sandy silt, 0.612for clayey silt, and 0.438-1.0175 for silt clay.
    - ✓ The liquidity index for sandy silt soil -1.028, 1.003 for clayey silt, and 1.307-0.285 for silt clay hence,
    - ✓ The consistency index for sandy silt is 2.028 - 0.003 and 0.742 to 2.305 for clayey silt and silty clay respectively.

- ✓ The potential expansion behavior or swelling nature of the soils is found to be low to medium and 54, 47, and 18 - 60 for sandy silt, clayey silt, and silt clay respectively.
- ✓ The specific gravity of study area soil range from2.47-2.6 for silt clay and 2.55 and 2.61 for sandy silt and clayey silt respectively.
- ✓ The optimum moisture content and maximum dry density results of the study area range32.8-28 and 1.6 -1.21kg/m3 respectively.
- ✓ The shear strength results indicate the cohesion of the soils varies from 38.4 to 49.37 kpa for compressible silt(silt clay), whereas the angle of shearing resistance varies from 19˚ to 22.99˚, hence, cohesion 22.73kpa-25.86kpa and 22.88° to 22.99 internal friction angle for sandy silt.54kpa and 18° for clayey silt.

- Classification of soils and rocks of the town was also carried out. Based on the USC system the clay soils fall in CL (inorganic clays of low to medium plasticity) and silt and sandy silt soils fall in MH (inorganic silts, elastic silts); The lithological field description and classifications made are approximately similar with laboratory results. The potential expansion behavior or swelling nature of the soils is found to be low to medium.
- Classification of rocks was made by considering lithological unit and engineering property of the rock. These geologic units are ignimbrite and basalt hence this geologic unit itself is classified into sub engineering units. Engineering geologic units of rock of study area are good basalt with slightly weathered and very high strength, good ignimbrite with slightly weathered and high strength, and poor ignimbrite with highly weathered very low strength.

6.2 Recommendations

The following suggestion is provided for the study area.

Study area dominate with compressible silt type of soil, this class of soil is not good for civil engineering work so, special design and exploration are needed before construction

Potentially medium expansive soils were identified in the town in some places. Hence, care should be taken in constructing civil engineering structures on such types of soils.

Most parts of town exposed for flood, minor and major roads are on damage, Food control remedial measures, such as constructing drainage networks and diversion canals, proper waste management, and increasing the size of canals, shall be carried out to minimize the risk.

Detail subsurface investigation using geophysical methods is recommended for a detailed understanding of the subsurface condition such as overburden thickness.

The groundwater is very shallow in the study area. The potential chemical attack on concrete foundations due to groundwater chemistry is required for the safe and economic design of the foundation of the structures. Therefore, proper soil chemical tests and hydrochemical analysis of groundwater, such as pH, salinity, sulfate; chloride, etc. should be done for further understanding of their effects on the foundation materials.

Further researches can be conducted with an increased number of test pits from the same areas and additional areas that are not included in this research of study area.

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List of Appendices

Appendix -A Natural Moisture Content Test Result

Tested By	Deginet
Sample Number	TP1
Depth (m)	3m
Moisture can and ld number	Zn3
Mc = Mass of empty, clean can + lid (grams)	20.8g
Mcms = Mass of can, lid and moist soil (grams)	141.2g
Mcds = Mass of can, lid and dry soil (grams)	112.3g
Mms = Mass of moist soil (grams)	120.4
Mds = Mass of dry soil (grams)	91.5
Mw = Mass of pore water (grams)	28.9
w = Natural Moisture content, (%)	31.64

Date Tested	
Tested By	Deginet
Sample Number	TP2
Depth (m)	3m
Moisture can and ld number	A3
Mc = Mass of empty, clean can + lid (grams)	19.7g
Mcms = Mass of can, lid and moist soil (grams)	119g
Mcds = Mass of can, lid and dry soil (grams)	93.3
Mms = Mass of moist soil (grams)	99.3
Mds = Mass of dry soil (grams)	73.6
Mw = Mass of pore water (grams)	25.7
w = Natural Moisture content, (%)	34.91

Appendix -B Specific Gravity Test Result

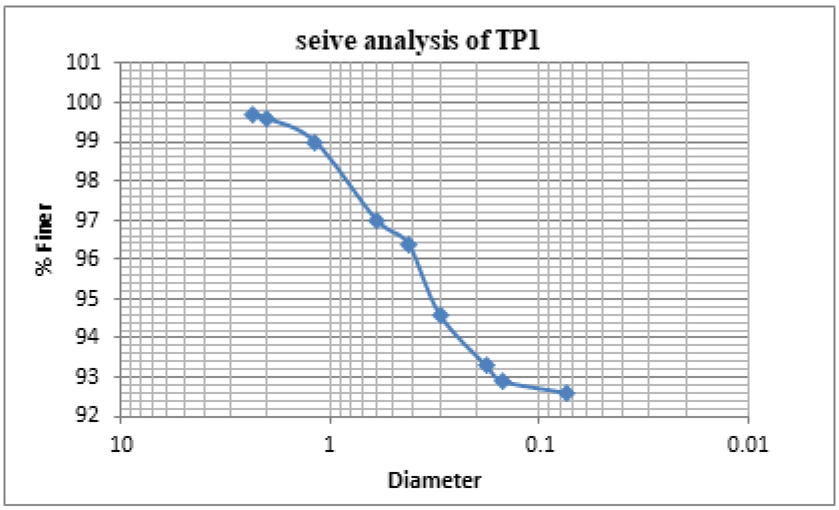
specific Gravity (Gs) of soils of Aleta wondo town **Hawassa University IOT Geotechnical Lab**			
Tested by	**Deginet.T**		
Project name	**Msc Project**		
Specimen number	**TP1**		
Depth (m)	**3m**		
No of trials	**Trial1**	**Trial2**	**Trial3**
Wp = Mass of empty, Clean Pycnometer (grams)	**110.5**	**81.7**	**92**
Wps = Mass of empty pycnometer + dry soil (grams)	**106.7**	**106.7**	**117**
Wb = Mass of pycnometer + dry soil + water (grams)	**373.8**	**344.9**	**355.1**
Wa = Mass of pycnometer + water (grams)	**330**		**340.4**
Wo = Mass of dry soil (grams)	**25**	**25**	**25**
Specific Gravity (Gs)	**2.47**	**2.48**	**2.43**
Averages.gravity=2.47			

specific Gravity (Gs) of soils of Aleta wondo town **Hawassa University IOT Geotechnical Lab**			
Tested by	**Deginet.T**		
Project name	**Msc Project**		
Specimen number	**TP2**		
Depth (m)	**3m**		
No of trials	**Trial1**	**Trial2**	**Trial3**
Wp = Mass of empty, Clean Pycnometer (grams)	**110.5**	**81.7**	**92**
Wps = Mass of empty pycnometer + dry soil (grams)	**135.4**	**106.7**	**117**
Wb = Mass of pycnometer + dry soil + water (grams)	**374.2**	**344.2**	**356**
Wa = Mass of pycnometer + water (grams)	**358.9**	**330**	**340.4**
Wo = Mass of dry soil (grams)	**25**	**25**	**25**
Specific Gravity (Gs)	**2.57**	**2.31**	**2.6**
Average S. gravity=2.5			

Appendix -C Sieve or Mechanical and Hydrometer Analysis Test Result

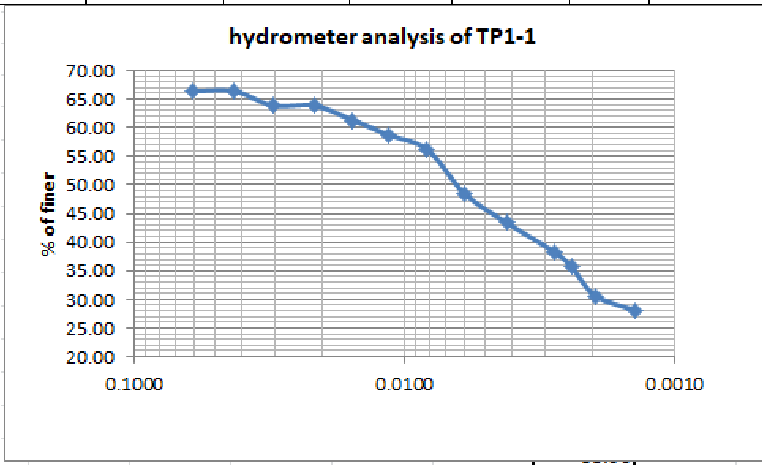
Grain size Analysis Data of soils of Aleta Wondo town

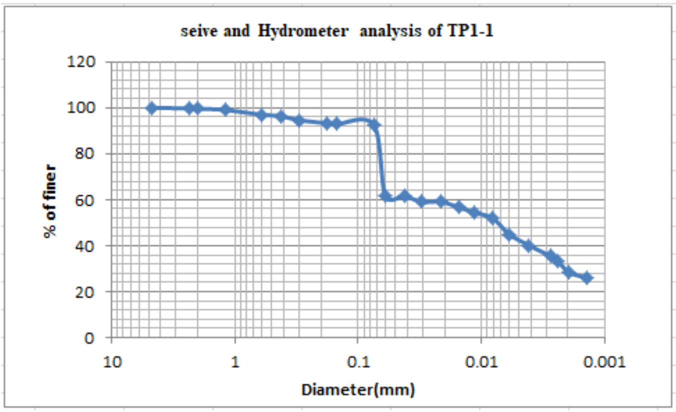
Grain size Analysis (sieve analysis data) TP 1-1						
Opening or size (mm)	Mas of sieve(g)	Mas of sieve + Mas Retained Soil(g)	Mas of retained(g)	%retained(g)	cumulative %retained(g)	% Finer
4.75	**409**	**409**	**0**	**0**	**0**	**100**
2.36	**497**	**499.19**	**2.19**	**0.219**	**0.291**	**99.7**
2	**463**	**464.1**	**1.1**	**0.11**	**0.401**	**99.6**
1.18	**449**	**454.2**	**5.2**	**0.52**	**0.921**	**99**
0.6	**435**	**456.7**	**21.7**	**2.17**	**3.091**	**97**
0.425	**373**	**378.6**	**5.6**	**0.56**	**3.651**	**96.4**
0.3	**389**	**407.4**	**18.4**	**1.84**	**5.491**	**94.6**
0.18	**348**	**360.8**	**12.8**	**1.28**	**6.771**	**93.3**
0.15	**377**	**380.3**	**3.3**	**0.33**	**7.101**	**92.9**
0.075	**360**	**367.3**	**7.3**	**0.37**	**7.471**	**92.6**



Elapsed time	Actual Hydrometer Reading (R)	Composite Correction	Specific Gravity	Corrected Reading (R)	Temperature	Temperature And Specific Gravity Constant (K)	Effective Depth (L)	Particle Diameter (D), Mm	PERCENT FINER	
									partial	total
0.5	**1.029**	0.003	**2.5**	1.026	24	0.01388	9.42	0.0602	66.40	61.49
1	**1.029**	0.003		1.026	24	0.01388	9.42	0.0425	66.40	61.49
2	**1.028**	0.003		1.025	24	0.01388	9.68	0.0305	63.85	59.12
4	**1.028**	0.003		1.025	24	0.01388	9.68	0.0215	63.85	59.12
8	**1.027**	0.003		1.024	24	0.01388	9.95	0.0154	61.29	56.76
15	**1.026**	0.003		1.023	24	0.01388	10.21	0.0114	58.74	54.39

30	**1.025**	0.003		1.022	24	0.01388	10.48	0.0082	56.18	52.03
60	**1.022**	0.003		1.019	24	0.01388	11.27	0.0060	48.52	44.93
120	**1.02**	0.003		1.017	24	0.01388	11.80	0.0042	43.42	40.20
240	**1.018**	0.003		1.015	24	0.01388	12.33	0.0028	38.31	35.47
480	**1.017**	0.003		1.014	24	0.01388	12.59	0.0024	35.75	33.11
960	**1.015**	0.003		1.012	24	0.01388	13.12	0.0020	30.65	28.38
1440	**1.014**	0.003		1.011	24	0.01388	13.39	0.0014	28.09	26.01





Appendix -D Free Swell Test Result

Tp1 depth 3m				
Initial volume	Final volume	Average of final	**Swell %**	
20ml	**33ml**	**32ml**	**60**	
2oml	**32ml**			

Tp2 **depth 3m**				
Initial volume	Final volume	Average	**Swell %**	
23	**26**	**26.2**	13.9	
23	**26.2**			

Tp3 **depth 3**				
Initial volume	Final volume	Average	**Swell %**	
22ml	**26ml**	**26**	**18**	
22ml	**26ml**			

Tp4 **depth 3m**				
Initial volume	Final volume	Average	**Swell %**	
20	**25**	**25.5**	**27.5**	
20	**25.5**			

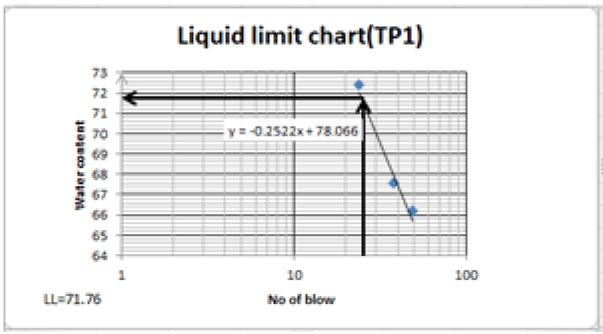
Tp5 **depth 3m**				
Initial volume	Final volume	Average of final	**Swell %**	
20ml	**30ml**	**30ml**	**50**	
2oml	**30ml**			

Appendix –E Atterberg Limits Test Result

Project:	MSc project on Eng. Geo characterization of soils	Test No.: TP1 Tested by: Deginet Test Date:

LIQUID LIMIT TEST

	Test 1	Test 2	Test 3
Number of drops	**49**	**38**	**24**
Log (N)			
Tare name	**Z22**	**A7**	**G33**
Tare mass (g)	**18.2**	**19.7**	**21**
Tare + wet sample (g)	**43.3**	**42.5**	**42.9**
Tare + dry sample (g)	**33.3**	**33.3**	**33.7**
Wet mass (g)	**25.1**	**22.8**	**21.9**
Dry mass (g)	**15.1**	**13.6**	**12.7**
Water content	**66.2**	**67.6**	**72.4**



PLASTIC LIMIT TEST

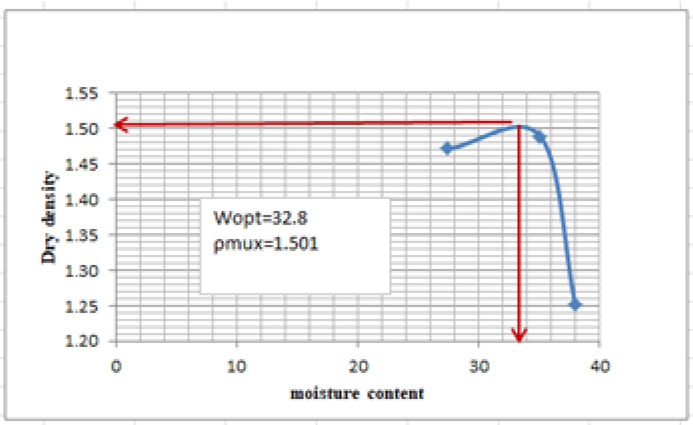
	Group 1	Group 2	Group 3
Tare name	**B3**	**Z11**	**C4**
Tare mass (g)	**21.2**	**21.2**	**19**
Tare + wet sample (g)	**27.7**	**30.3**	**28**
Tare + dry sample (g)	**25.7**	**27.7**	**25.4**
Wet mass (g)	**6.5**	**9.1**	**9**
Dry mass (g)	**4.5**	**6.5**	**6.4**
Water content	**44.4**	**40**	**40.6**
Number of individual tests	**2**	**2**	**2**
	PL=41.6		

PI=LL-PL, 71.76-41.6=31.16

Appendix -F Compaction test

TP1

No trail	No1	No2	No3
Volume of mould	944	944	944
Mass of mould	3411.8	3411.8	3411.8
Mass of mould +sample	5182	5221	5043
Can code	P10	8W	W11
Mass of can	19.1	20.1	20
Can +sample	79	84	97
Mass of dry sample and can	66.1	67.4	79.8
Water content	27.44	35	42.6
Dry unit weight	1.46	1.5	1.21

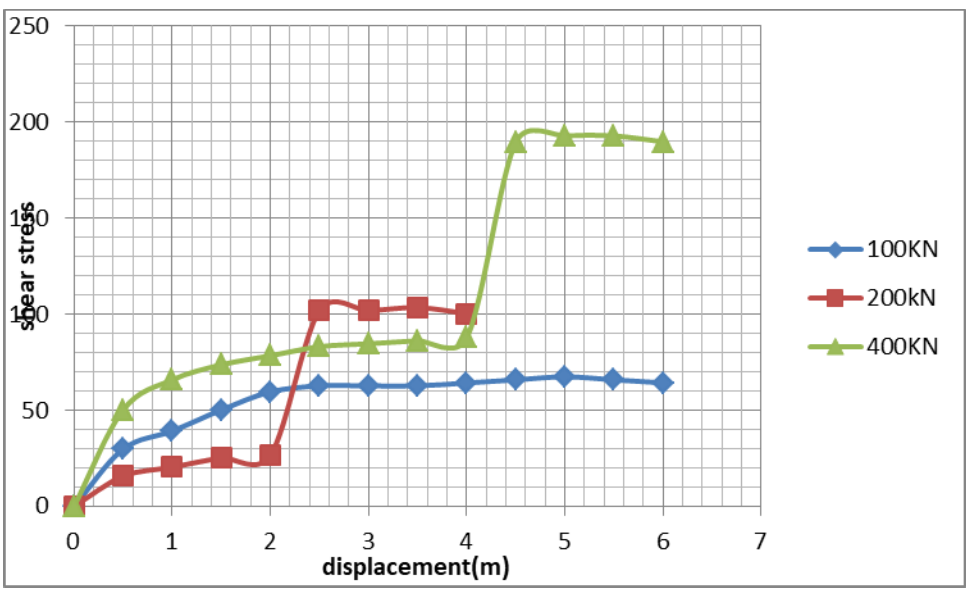


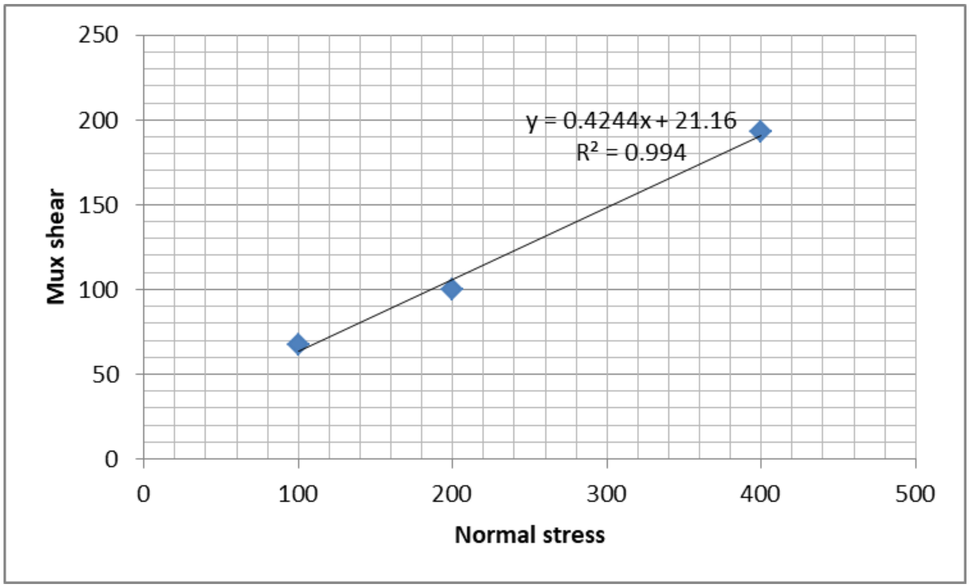
Appendix -H Shear strength

bulky unit	18.75kg/m3
dry unit w	11.4
water con	27.4

TP7	
Cohesion	21.16
Firiction angle	22.88

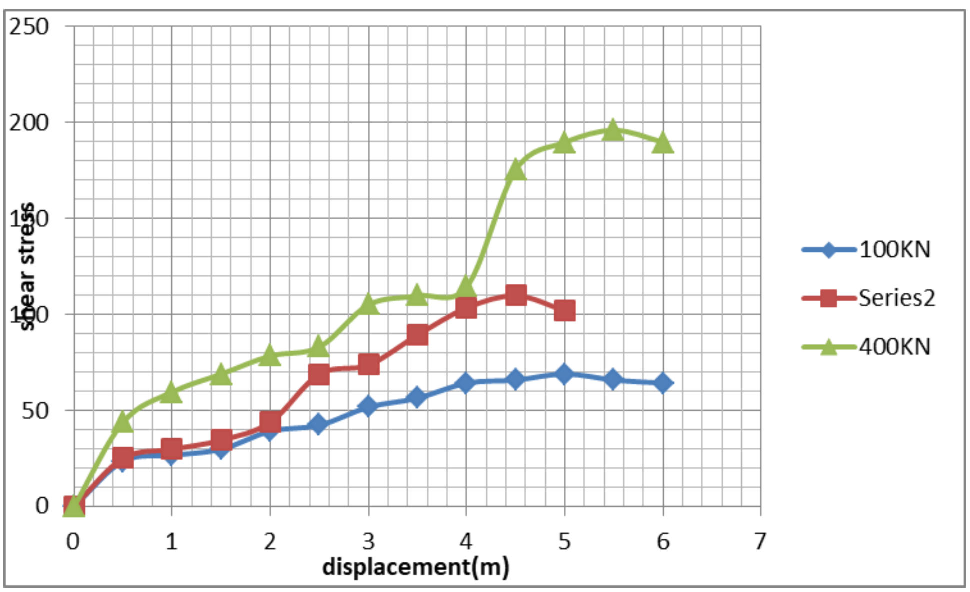
Proving Ring Calibration,l	0.00564333		Horizontal dial reading corection			0.01	
horizontal displacement	reading	100kpa	shear stress	200kpa	shear stress	400kpa	shear stress
0	0	0	0	0	0	0	0
50	0.5	19	29.78424167	10	15.6759167	32	50.16293333
100	1	25	39.18979167	13	20.3786917	42	65.83885
150	1.5	32	50.16293333	16	25.0814667	47	73.67680833
200	2	38	59.56848333	17	26.6490583	50	78.37958333
250	2.5	40	62.70366667	65	101.893458	53	83.08235833
300	3	40	62.70366667	65	101.893458	54	84.64995
350	3.5	40	62.70366667	66	103.46105	55	86.21754167
400	4	41	64.27125833	64	100.325867	56	87.78513333
450	4.5	42	65.83885			121	189.6785917
500	5	43	67.40644167			123	192.813775
550	5.5	42	65.83885			123	192.813775
600	6	41	64.27125833			121	189.6785917
650	6.5						
700	7						
750	7.5		100	67.40644			
800	8		200	67.40644			
850	8.5		400	192.8138			
900	9						
950	9.5						
1000	10						

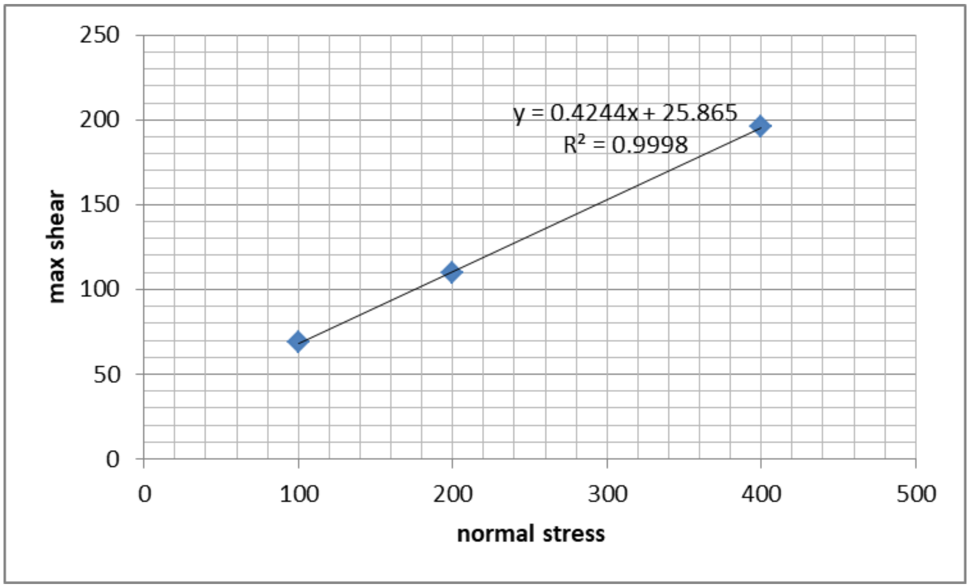



250
200
150
100
50
0
Mux shear
y = 0.4244x + 21.16
R² = 0.994
0
100
200
300
400
500
Normal stress

bulky unit	18.45	TP1		TP1	
dry unit w	14.41406	cohesion	38.4	Cohesion	25.86
water con	28	firiction angle	22.5	Firiction angle	22.5

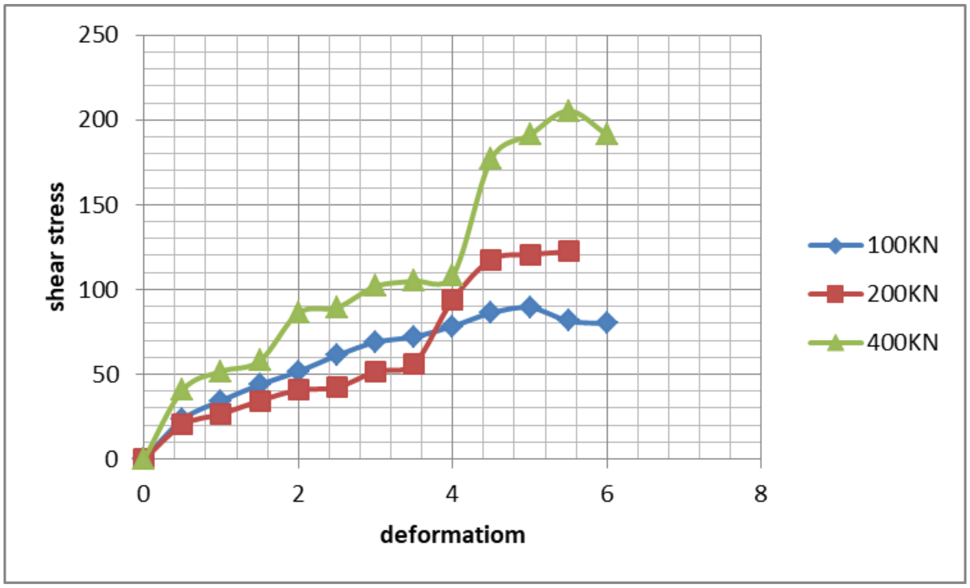
Proving Ring Calibration,l	0.00564333		Horizontal dial reading corection			0.01	
horizontal displacement	reading	100kpa	shear stress	200kpa	shear stress	400kpa	shear stress
0	0	0	0	0	0	0	0
50	0.5	15	23.513875	16	25.0814667	28	43.89256667
100	1	17	26.64905833	19	29.7842417	38	59.56848333
150	1.5	19	29.78424167	22	34.4870167	44	68.97403333
200	2	25	39.18979167	28	43.8925667	50	78.37958333
250	2.5	27	42.324975	44	68.9740333	53	83.08235833
300	3	33	51.730525	47	73.6768083	67	105.0286417
350	3.5	36	56.4333	57	89.352725	70	109.7314167
400	4	41	64.27125833	66	103.46105	73	114.4341917
450	4.5	42	65.83885	70	109.731417	112	175.5702667
500	5	44	68.97403333	65	101.893458	121	189.6785917
550	5.5	42	65.83885			125	195.9489583
600	6	41	64.27125833			121	189.6785917
650	6.5						
700	7						
750	7.5		100	68.97403			
800	8		200	109.7314			
850	8.5		400	195.949			
900	9						
950	9.5						
1000	10						

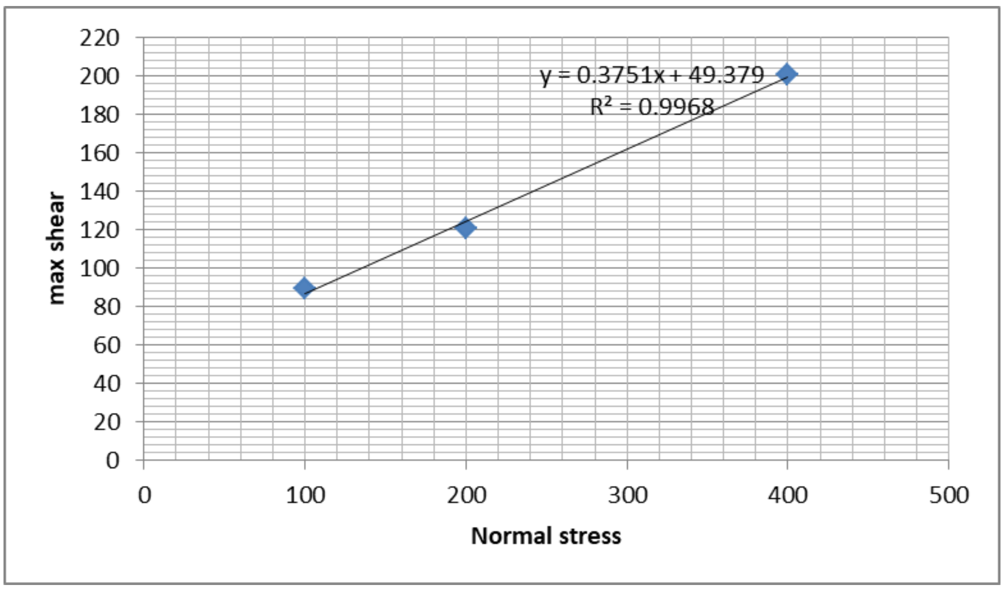




bulky unit	17.78	TP6	
dry unit w	13.17	cohesion	49.73
water con	35	firiction angle	20.56

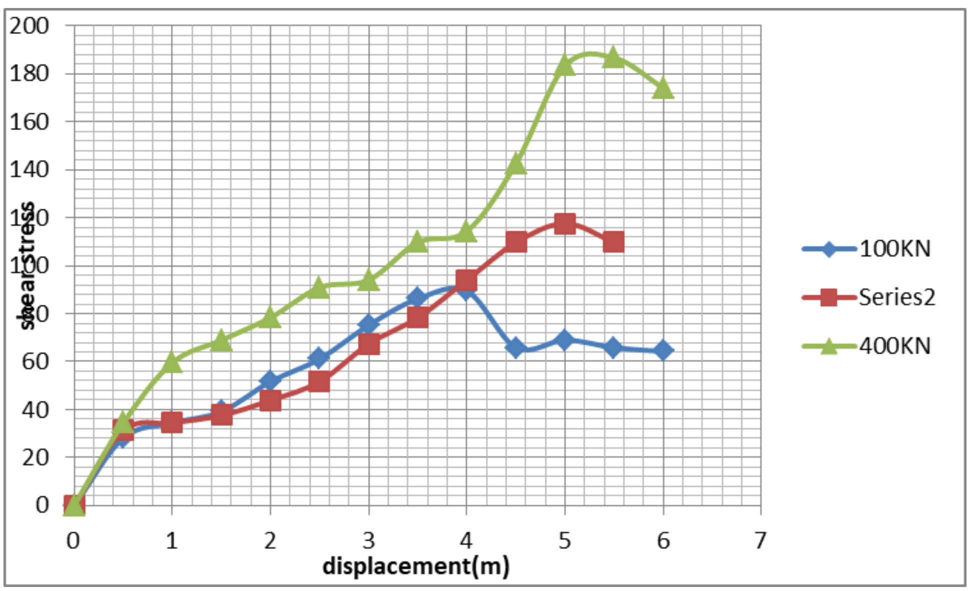
	Horizontal dial reading corection=0.01				
100kpa	shear stress	200kpa	shear stress	400kpa	shear stress
0	0	0	0	0	0
15	23.513875	13	20.3786917	26	40.7573833
22	34.48701667	17	26.6490583	33	51.730525
28	43.89256667	22	34.4870167	37	58.0008917
33	51.730525	26	40.7573833	55	86.2175417
39	61.136075	27	42.324975	57	89.352725
44	68.97403333	33	51.730525	65	101.893458
46	72.10921667	36	56.4333	67	105.028642
50	78.37958333	60	94.0555	69	108.163825
55	86.21754167	75	117.569375	113	177.137858
57	89.352725	77	120.704558	122	191.246183
52	81.51476667	78	122.27215	131	205.354508
51	79.947175			122	191.246183

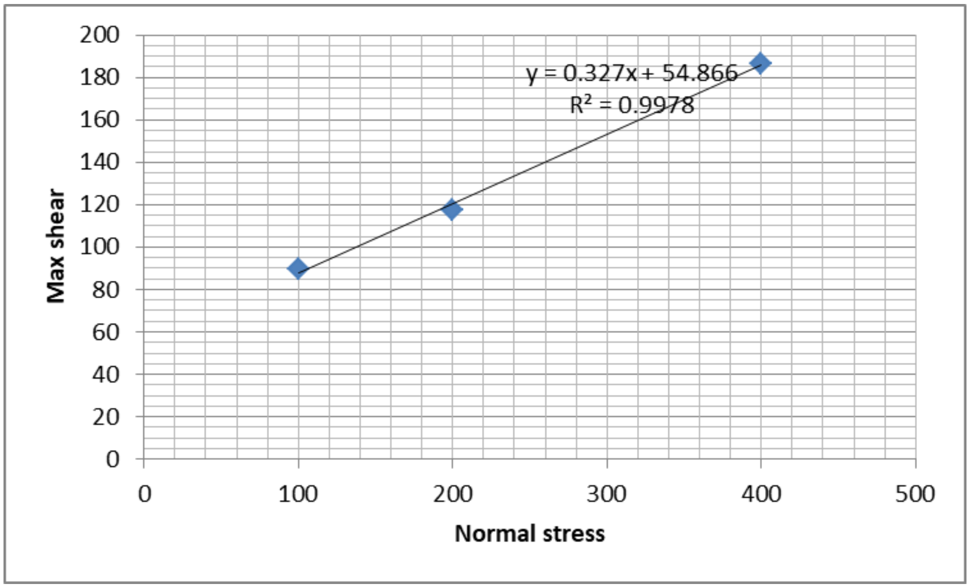

250
200
150
100
50
0
0
2
4
6
8
shear stress
deformatiom
100KN
200KN
400KN


220
200
180
160
140
120
100
80
60
40
20
0
0
100
200
300
400
500
y = 0.3751x + 49.379
R² = 0.9968
max shear
Normal stress

bulky unit	18.01	TP10	
dry unit w	14.41	cohesion	54
water con	25	firiction angle	18

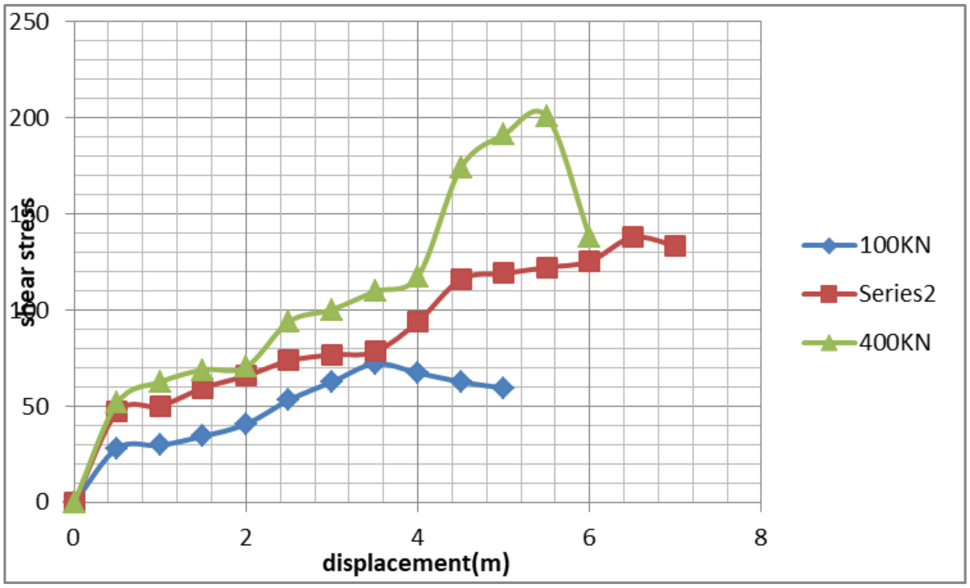
	Proving Ring Calibration,	0.00564333		Horizontal dial reading corection			0.01	
	horizontal displacement	reading	100kpa	shear stress	200kpa	shear stress	400kpa	shear stress
	0	0	0	0	0	0	0	0
	50	0.5	18	28.21665	20	31.3518333	22	34.48701667
	100	1	22	34.48701667	22	34.4870167	38	59.56848333
	150	1.5	25	39.18979167	24	37.6222	44	68.97403333
	200	2	33	51.730525	28	43.8925667	50	78.37958333
	250	2.5	39	61.136075	33	51.730525	58	90.92031667
	300	3	48	75.2444	43	67.4064417	60	94.0555
	350	3.5	55	86.21754167	50	78.3795833	70	109.7314167
	400	4	57	89.352725	60	94.0555	73	114.4341917
	450	4.5	42	65.83885	70	109.731417	91	142.6508417
	500	5	44	68.97403333	75	117.569375	117	183.408225
	550	5.5	42	65.83885	70	109.731417	119	186.5434083
	600	6	41	64.27125833			111	174.002675
	650	6.5						
	700	7						
	750	7.5		100	89.35273			
	800	8		200	117.5694			
	850	8.5		400	186.5434			
	900	9						
	950	9.5						
	1000	10						

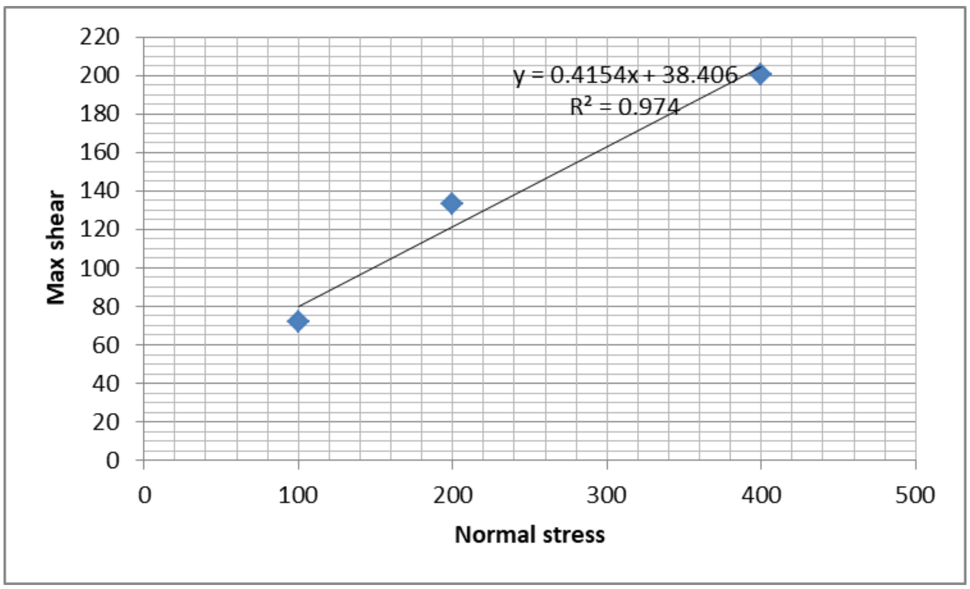




bulky unit	17.9	TP2	
dry unit w	14.09	Cohesion	38.4
water con	27	Firiction angle	22.5

Proving Ring Calibration,l	0.00564333		Horizontal dial reading corection			0.01	
horizontal displacement	reading	100kpa	shear stress	200kpa	shear stress	400kpa	shear stress
0	0	0	0	0	0	0	0
50	0.5	18	28.21665	30	47.02775	33	51.730525
100	1	19	29.78424167	32	50.1629333	40	62.70366667
150	1.5	22	34.48701667	38	59.5684833	44	68.97403333
200	2	26	40.75738333	42	65.83885	45	70.541625
250	2.5	34	53.29811667	47	73.6768083	60	94.0555
300	3	40	62.70366667	43	67.4064417	64	100.3258667
350	3.5	46	72.10921667	50	78.3795833	70	109.7314167
400	4	43	67.40644167	60	94.0555	75	117.569375
450	4.5	40	62.70366667	74	116.001783	111	174.002675
500	5	38	59.56848333	71	111.299008	122	191.2461833
550	5.5			78	122.27215	128	200.6517333
600	6			80	125.407333	88	137.9480667
650	6.5	100	72.10921667	88	137.948067		
700	7	200	133.2452917	85	133.245292		
750	7.5	400	200.6517333	54			
800	8						
850	8.5						
900	9						
950	9.5						
1000	10						


250
200
150
100
50
0
0
2
4
6
8
shear stress
displacement(m)
100KN
Series2
400KN


220
200
180
160
140
120
100
80
60
40
20
0
0
100
200
300
400
500
Max shear
Normal stress
y = 0.4154x + 38.406
R² = 0.974

Appendix I Standard tables from different sources and pictures


SOILTEST ALIDAD THE GEOTECHNICAL LABORATORY
SUB SURFACE INVESTIGATION
AND
FOUNDATION RECOMENDATION
REPORT
Project: G+4 Mixed Use Building
Owner: Wro Amelework Wubshet
Location : Aletawendo Woreda, Southern People RS
January
2016
ALIDAD consulting Architects and Engineers PLC
P.O.BOX 141/1064 Tel. +251 118389260
Mob. 0911-382769
Addis Ababa ETHIOPIA
Engineering solutions you can build on.

ALIDAD consulting Architects and Engineers PLC
P.O.BOX 141/1064 Tel. +251 [illegible]
Mob. 0911-382769
Addis Ababa ETHIOPIA

LAB. No	[illegible]
Client	[illegible]
PROJECT	[illegible] G+4 Mixed Use Building
SAMPLE SOURCE/STATION/CODE	Pit 1
SAMPLE OF	Foundation Material
SAMPLED BY	The Client
SAMPLE AND TEST ORDER SUBMITTED BY	The Client
SPECIFIED BY	The Client
TEST FOR	Atterberg Limit (AASHTO T89-90)
TEST RESULT REPORTED TO	The Client

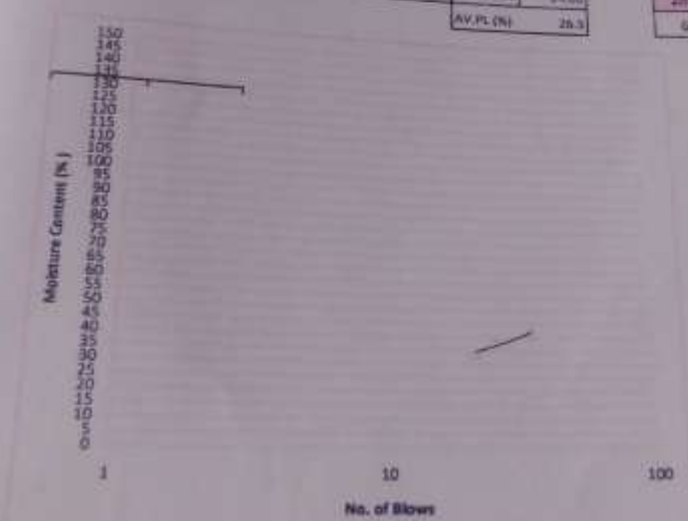
Depth	[illegible]
Date	
On	[illegible]-Jan-16
On	[illegible]-Jan-16
On	[illegible]-Jan-16
On	[illegible]-Jan-16

TEST RESULT

Description	Liquid Limit			Plastic Limit	
No. Blows	34	25	21		
Wt. wet soil (g.)	41	39	36	16	13
Wt. dry soil (g.)	29	28	27	13	11
Moisture content (%)	41.38	39.29	33.33	23.00	24.00
				AV.PL (%)	26.5

Liquid Limit (LL%)	Plastic Limit (PL%)	Plasticity Index (PI)
39	27	12

WET SIEVE ANALYSIS % PASS		
2mm	0.425	0.075
60	53	41
	GI	1



AASHTO Classification
A-6 (1)

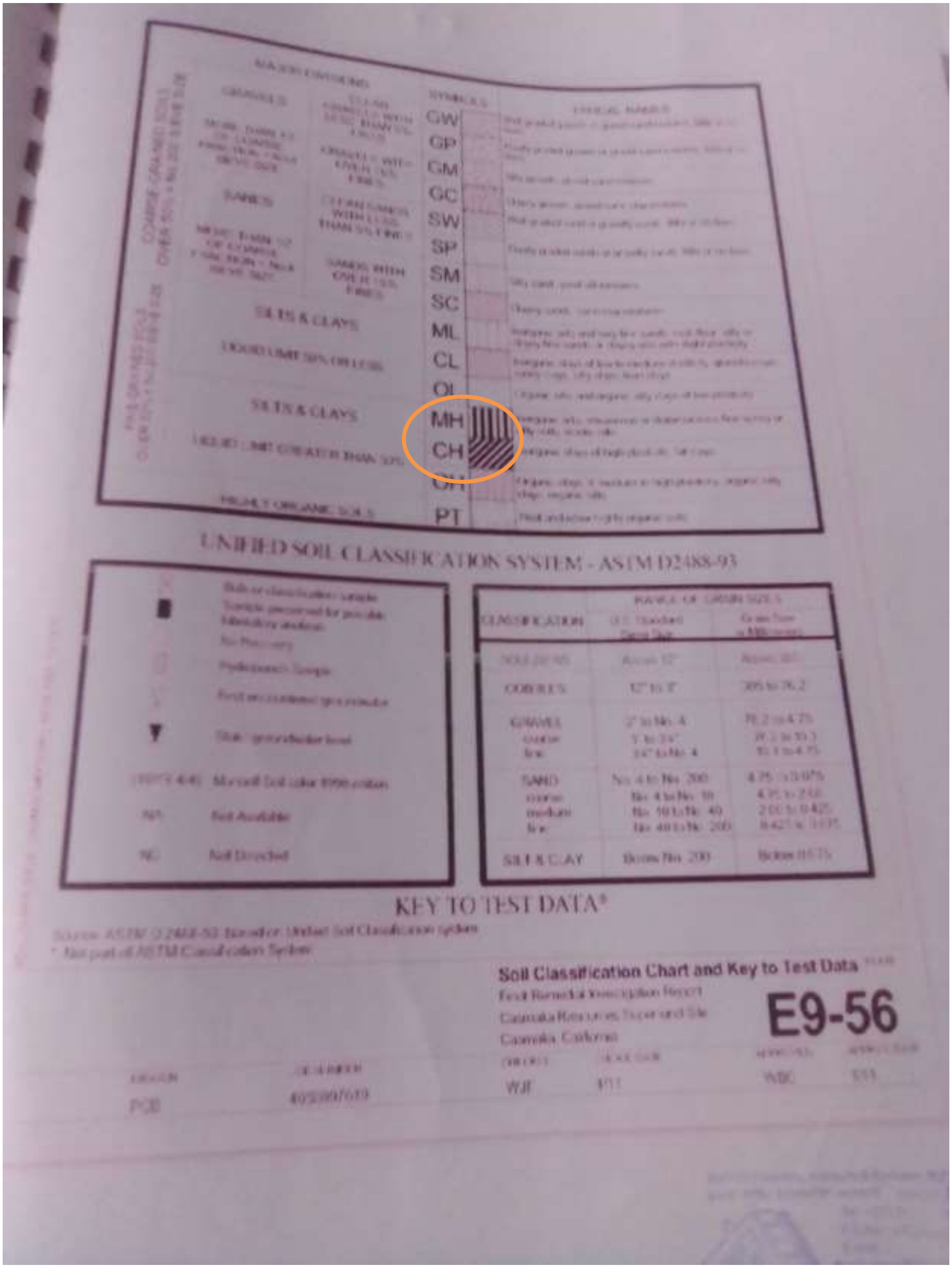
REMARK

REPORTED BY

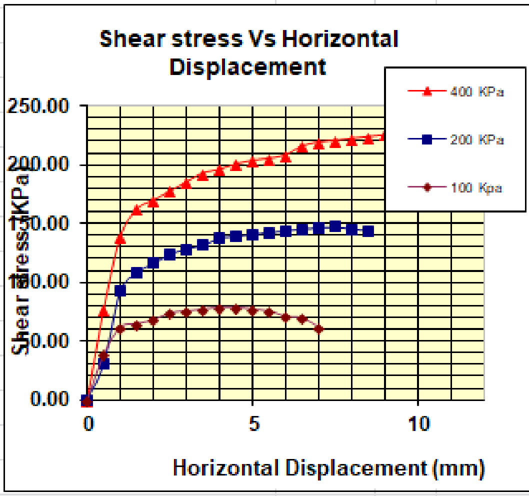
Kaineh Adbaru
Lab. Engineer

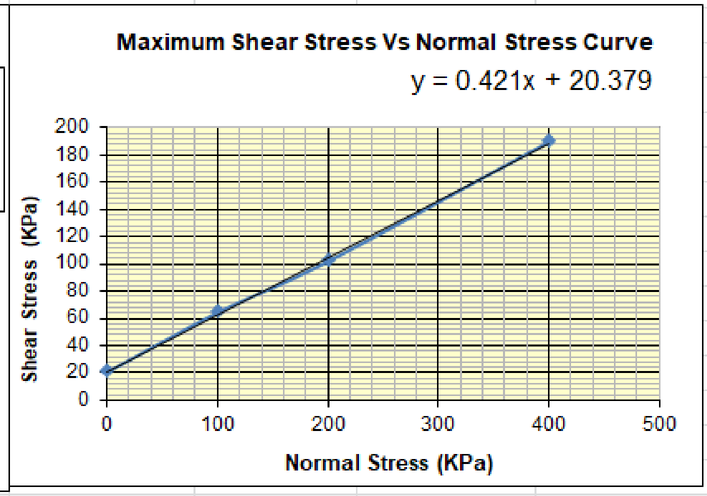
ANALIZED & APPROVED BY

Tilaye Ali
Head of Geotechnical
Engineering Department


GW
GP
GM
GC
SW
SP
SM
SC
SILTS & CLAYS
LIQUID LIMIT 50% OR LESS
ML
CL
OL
SILTS & CLAYS
LIQUID LIMIT GREATER THAN 50%
MH
CH
OH
HIGHLY ORGANIC SOILS
PT
UNIFIED SOIL CLASSIFICATION SYSTEM - ASTM D2488-93
KEY TO TEST DATA*
Soil Classification Chart and Key to Test Data
E9-56

Normal Stress (KPa)	50	100	200	C(KPa)	Ø(Degrees)
Shear Stress (KPa)	64	102	190	20	22.94





Fill the value of yellow soil

CNc=	437.73	Ø	22.938		Interpolation	
γDNq=	516.29	B	2.000		20.000	17.69
0.5BγNγ=	95.14	L	1.000	Nc=	22.938	21.5
		D	4.000		22.000	20.27
C=	20.379					
Nc=	21.479				20.000	7.44
				Nq=	22.938	10.0
γD=	51.575				22.000	9.19
Nq=	10.010					
					20.000	4.97
Bγ=	25.788			Nγ=	22.938	7.38
Nγ=	7.379				22.000	6.61

Soil parameter		Bearing capacity factor		Ultimate soil bearing capacity	Safety Factor	Allowable soil bearing capacity
Unit weight, KN/m^3, ϒ =	12.89	Nc =	21.479	1049.16	3.00	349.72
Cohesion value, KN/m^2 C =	20.38	Nq =	10.010			
Angle of friction, Φ =	22.94	N_γ=	7.379			
Depth, Df , m =	4.00	q=	51.575			

Table 1.3: Effective Depth for 152H and 151H Hydrometers

Rm	EFFECTIVE DEPTH	Rm	EFFECTIVE DEPTH	Rm	EFFECTIVE DEPTH	Rm	EFFECTIVE DEPTH
0	16.3	26	12.0	0	16.3	20	11.0
1	16.1	27	11.9	1	16.0	21	10.7
2	16.0	28	11.7	2	15.8	22	10.5
3	15.8	29	11.5	3	15.5	23	10.2
4	15.6	30	11.4	4	15.2	24	9.9
5	15.5	31	11.2	5	15.0	25	9.7
6	15.3	32	11.0	6	14.7	26	9.4
7	15.1	33	10.9	7	14.4	27	9.2
8	15.0	34	10.7	8	14.2	28	8.9
9	14.8	35	10.6	9	13.9	29	8.6
10	14.7	36	10.4	10	13.7	30	8.4
11	14.5	37	10.2	11	13.4	31	8.1
12	14.3	38	10.1	12	13.1	32	7.8
13	14.2	39	9.9	13	12.9	33	7.6
14	14.0	40	9.7	14	12.6	34	7.3
15	13.8	41	9.6	15	12.3	35	7.0
16	13.7	42	9.4	16	12.1	36	6.8
17	13.5	43	9.2	17	11.8	37	6.5
18	13.3	44	9.1	18	11.5	38	6.2
19	13.2	45	8.9	19	11.3		
20	13.0	46	8.8				
21	12.9	47	8.6				
22	12.7	48	8.4				
23	12.5	49	8.3				
24	12.4	50	8.1				
25	12.2						