Multipurpose Engineering Geological Mapping of Yirgalem Town, Southern Ethiopia

Befikadu Petros Daniel

A Thesis Submitted to the Department of Applied Geology

School of Applied Natural Science

Presented in Partial Fulfillment of the Requirement for the Degree of Master of Science in Applied Geology

Office of Graduate Studies

Adama Science and Technology University

August, 2021

Adama, Ethiopia

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I/we, the advisors of the thesis entitled "Multipurpose Engineering Geological Mapping of Yirgalem Towns" and developed by Befikadu Petros Daniel; hereby certify that the recommendation and suggestions made by the board of examiners are appropriately incorporated into the final version of the thesis.

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I/we, the advisor(s) of this thesis, hereby certify that I/we have read the revised version of the thesis entitled "**Multipurpose Engineering Geological Mapping of Yirgalem Towns**" prepared under my/our guidance by **Befikadu Petros Daniel** submitted in partial fulfillment of the requirements for the degree of Mater's of Science in Applied Geology with specialization in Engineering Geology. Therefore, I/we recommend the submission of a revised version of the thesis to the department following the applicable procedures.

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LIST OF ABBREVIATIONS AND ACRONYMS

AASHTO	American Association of State Highway and Transportation Official
ASTM	American Society for Testing and Materials
BSCS	British Soil Classification for Engineering purposes
EBC	Ethiopian Building Code Standard
GPS	Global Positioning System
GW	Ground Water
IAEG	Internal Association of Engineering Geology
ISRM	Internal Society of Rock Mechanics
IS	Indian Standard
ITCZ	Inter-Tropical Convergence Zone
MDD	Maximum Dry Density
NMC	Natural Moisture Content
OMC	Optimum Moisture Content
P^H	Power of Hydrogen
RMR	Rock Mass Rating
RQD	Rock Quality Designation
SHV	Schmidt Hammer rebound value
SMER	Southern Main Ethiopian Rift
SNNPR	Southern Nation Nationality People Regional state
SNRS	Sidama National Regional State
SRTM	Shuttle Radar Topography Mission
TPs	Test Pits
UCS	Unconfined Compressive strength
USCS	Unified Soil Classification system
YAIP	Yirgalem Agro Industrial Park

ABSTRACT

In Ethiopia, many engineering works are constructed and under construction in different towns without proper and adequate geotechnical characterization of soils and rocks to know the site suitability for intended construction activities. Yirgalem Town was among these towns. Currently large engineering work including Yirgalem integrated agro-industrial park constructed, highway and railway route which connect the country with neighboring are planned for construction. Lack of detail geological and engineering geological data and respective maps, poor drainage, and surface geodynamic problem are the main problems encountered at Yirgalem Town. Therefore, this study aims to address geological and geotechnical characterization and classification of soil and rocks to produce a detailed medium scale multipurpose engineering geological map of the study area. Secondary data collection, field and laboratory work, sampling data, discontinuity survey, data analysis, and interpretation, and preparation of maps were approaches and methods used to carry out this research by using ArcGIS software, Surfer, and SRTM DEM image. Results from laboratory data analysis show that moisture content ranges from 10.05 to 49.5%, the liquid limit ranges 49.5-70, the plastic index ranges 13.84 – 36.9 and from grain size analysis silt soil was found to be dominant. The activity of soil ranges 0.62 -1.53(inactive to active) and free swell test ranges 13-104% indicating low to high potential expansion. The shear strength parameters were analyzed based on Mohr-coulomb failure criterion, cohesion (C), and internal friction angle (Φ) of the soil sample ranges from 9.928-39.7KN/m^{2}and 22.6-34.31 respectively. Engineering classification of soil based on Unified soil classification system of soil falls into poorly graded sand, poorly graded sand with silt, sand with silt, elastic silt, and fat clay. Elastic silt soil is dominant and indicates most of the soils in the study area are poor for engineering purpose. Characterization and classification of rocks in the study area were based on the empirical relation of Schmidt hammer rebound value with compressive strength, weathering, and discontinuity survey, and rock masses of Yirgalem Town were grouped as low, medium, and high mass strength. Gully erosion, landslide, and problematic soil are the main surface geodynamic problems identified during field investigation. The study conclude that most of the soils in the study area are expansive and poor for engineering workability and the different engineering structures to be built in the town require detailed site-specific geotechnical investigation. It is therefore recommended to make further slope stability assessment, and detailed surface and subsurface geophysical investigation.

Keywords; *Characterization, Engineering Geological map, Rock strength, rock mass classification, Soil classification, soil types*

CHAPTER ONE

INTRODUCTION

1.1 General background

Ethiopia is one of the developing countries in Africa as well as in the World and currently different huge projects like the Renaissance Dam, expressway, and railways which make the large trading route of the country with neighboring countries are under construction. According to Bell (2007) as cited in Chemeda, (2020), site investigation is the basic requirement for the successful and economic design of engineering structures and earthworks. Therefore, engineering geological characterization of soil and rock is very important for a country like Ethiopia which is developing at a high rate and which needs much construction works at present and in the near future. Because, the data obtained is used to provide basic information for the current and future planning of land use, design, construction, and maintenance of civil engineering works in the country.

In addition, in big towns in Ethiopia like Adama, Hawassa, Dilla, etc were found in the rift valley without enough/proper information and understanding of the geological environment, engineering geological and geotechnical aspects. Even large companies work their civil engineering structures without taking precautionary measures for natural hazards. Due to these facts a considerable number of engineering structures have been deteriorated and damaged. Moreover disastrous failures, collapses and tilting of structures happened. According to Apostolidis and Koukis, (2013) adequate knowledge of the geomorphological, geological, engineering geological and geotechnical conditions of an urban area is necessary to provide basic information of the ground condition to the local authorities, civil engineers and urban/town planners and contractors. This information also constitutes the basic concept for the evaluation of the geological hazards, which are encountered in the area in question and considered during preliminary design and planning of future construction projects and infrastructures in towns (Parry et al., 2014). Any civil engineering structure lies over/under rocks or soils. Which give the ultimate support to the structure. Therefore the physical and mechanical properties of these units determine the safety and well-being of the structures. This calls for the importance of characterization of the soils and rocks for engineering practice (Gebrewahid, 2007). Due to their difference in physical, mechanical and chemical properties, soils and rocks exhibit different favorability for civil engineering works. That is to say, different types of soils and rocks exhibit different responses and show varying performance when civil engineering structures are constructed on them. Hence through subsurface investigation has to be made in order to determine the suitability of existing ground conditions for the placement of a certain structure at a certain site (Gebrwahid, 2007).

Yirgalem town was founded around a hundred years ago and is one of the oldest and highly populated towns in Sidama Region next to Hawassa town. It is located 311Km from Addis Ababa, 47Km from Hawassa which was the capital city of Sidama National Regional State (SNRS) and Southern Nation, Nationality and People Regional state (SNNPR). This town is the main source of immense economic potential accessible for the region associated with its agro industry potential. Due to this, urbanization is rapid in the different directions of the town, and investors are attracted to construct in the town and its surrounding areas. In addition, the town has been rapidly expanded in all directions due to the radical/unexpected change in population density. As a result of this, a large number of engineering structures (campus, buildings, lodges, churches, roads, hospitals, etc.) are constructed and under construction on poor soil conditions, steep slopes, even in high erosional and landslide prone areas of the town. Buildings are cracked; bridges and roads are damaged and closed by sliding. This is because of the Lack of geological and engineering geological information on the nature of soils and rocks, geodynamic possesses, geomorphology and geohazard problem of the town.

Detailed geological and engineering geological investigations are required to reduce those negative impacts. This research is, therefore aimed to investigate in detail and obtain information on the engineering properties of both soil and rocks found in Yirgalem town, so as to produce a multipurpose engineering geological map of the town and addresses different geological and geotechnical hazards and recommend appropriate countermeasures and mitigation strategies.

1.2 Statement of the problem

Construction of civil engineering structures and land use planning require adequate information about the geology, engineering geology, geodynamic phenomena, engineering properties of soils and rocks and geological structures of the site. The success or failure of a foundation depends essentially on the reliability of the various soil and rock parameters obtained from field investigation, visualization, observation, and laboratory testing which can be input for the design of a foundation. Yirgalem town is the fastest growing town in the region in which many buildings are constructed and under construction including Agro-Industrial Park (YAIP) which was constructed without detailed engineering geological characterization of soil and rock mass. Different geological and geotechnical problems such as erosion, landslide, cracking, and damaging of engineering structures are common in the town. However Abosto (western) area, the topography of the town is dominantly flat with poor drainage conditions in which surface water is found the flat area. For this reason, constructions could be sensitive to structural failure. There is also a lack of detail geological and engineering geological data and respective maps of the town which are widely used for urban land use planning.

Hence, this research work aims to address these problems by investigating in detail and obtain information on the engineering properties of both soil and rock underlying the town, to produce an engineering geological map of the area, and provide possible mitigation measures for the problems that are widely observed.

1.3 Objectives of the study

1.3.1 General objectives

The main objective of the study is to evaluate the geological and engineering geological condition of Yirgalem town.

1.3.2 Specific objectives

- ✓ To characterize the physical and engineering properties of soils and rocks of the study area.
- ✓ To classify the rocks and soils of the area into different geotechnical units based on their engineering properties.
- ✓ To address the surface geodynamic hazards of Yirgalem Town and identify possible remedial measures.
- ✓ To produce geological and engineering geological maps of Yirgalem town at 1:20000 scale.

1.4 Research questions

This research work is expected to answer the following research questions:

- ✓ What are the engineering geological and geotechnical characteristics of soils and rocks in the study area?
- ✓ What engineering geological problems are expected based on the data obtained from the investigation?
- ✓ What are the most effective remedial measures for these engineering problems?
- ✓ What would be the possible outcome if the building and road are constructed without any remedial measures?

1.5 Significance of the Study

This research work is intended to come up with a multipurpose engineering geological map and detailed engineering geological characterization of soils and rock in Yirgalem town. This will help to reduce the cost of investigation if any company, governmental organization, and researcher are interested to undertake further study in the area. Having a deep understanding or information on the engineering or geotechnical properties of rocks and soils will also help to mitigate and minimize any adverse effects as a result of any probable damage in the future to the buildings and road construction in Yirgalem town and surrounding by taking a possible measure with a related problem. On the other hand, this research acts as background knowledge for future engineering studies in the area. The result of the present study will also be used as an input in the design and construction phases of any building and road construction. It also plays a great role in selecting a safe foundation that can support any anticipated load of the structure.

1.6 Scope of thesis

The scope of this research is limited to the preparation of a multipurpose engineering geological map of Yirgalem Town through detailed observation and characterization of soil and rock mass, 14 test pits with a maximum 3.6m depth were excavated and 19 representative soil samples were collected to test index and strength parameter of soils. Rock mass strength is also obtained by using geological hammer and Schmidt hammer rebound value. The research has been supported by different kinds of literatures, books, manuals, and standards.

1.7 Structure of Thesis

The present research study is presented in seven chapters comprising the following;

Chapter I	Deals with general background, problem statement, research question, objectives, the scope of the study, significance of the study, and organization of the study.
Chapter II	Deals with literature review of research that includes, engineering geological characterization and classification of soil and rock mass, assessment of hydrogeological, geomorphological and surface geodynamic processes.
Chapter III	Describes about the location and accessibility of study area, climate, physiographic condition, drainage pattern, geological set up of area including regional and local geology and geological structures.
Chapter IV	Deals with research design and methodology which contain data collection techniques, laboratory analysis, data analysis and interpretation approaches.
Chapter V	Deals with result and discussion that summarizes characterization of soil and rock, laboratory test results and classifications of the soil. It also deals with classification of rocks based on different approaches and surface geodynamics in study area.
Chapter VI	Deals with conclusion and recommendation. Finally, references and appendices are placed at the end.

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CHAPTER TWO

DESCRIPTIONS OF THE STUDY AREA

2.1 Overview

Yirgalem Town is established around one hundred years ago which makes it the oldest town in the region and it is one of the biggest and most populated towns in the region next to Hawassa town. It lies on the main road which passes through Aposto – Negele – Moyale –Nairobi and has many tourist attraction sites and historical places. It is a home of a Lodge (known as Aregash Lodge) which is built at the outskirt of the town. There is also Gidabo hot springs bath which is located 41 Km far south of Hawassa town. Yirgalem town is the capital city for Sidama Zone up to 1960 after that Hawassa received the privilege of being a capital city.

2.2 Description of the study area

2.2.1 Location and accessibility

Yirgalem town is located in Dalle Woreda of Sidama National Regional State (SNRS), Southern Ethiopia. It is found in Southern parts of the Main Ethiopian Rift Valley Basin (SMER) around 311 Km from Addis Ababa capital city of Ethiopia and 47km from Hawassa which is the capital city of Sidama National Regional State and Southern Nation Nationality People Regional State (SNNPR). It is located in between 424000 to 440000 E and 745000 to 748000 N and elevation is 1670 to 1830 m above sea level at UTM 37 zone (Figure 1). It is usually accessible via Addis Ababa - Hawassa – Yirgalem asphalt road of 300km and interconnected with the all-weather gravel and cobble stone road of the town.

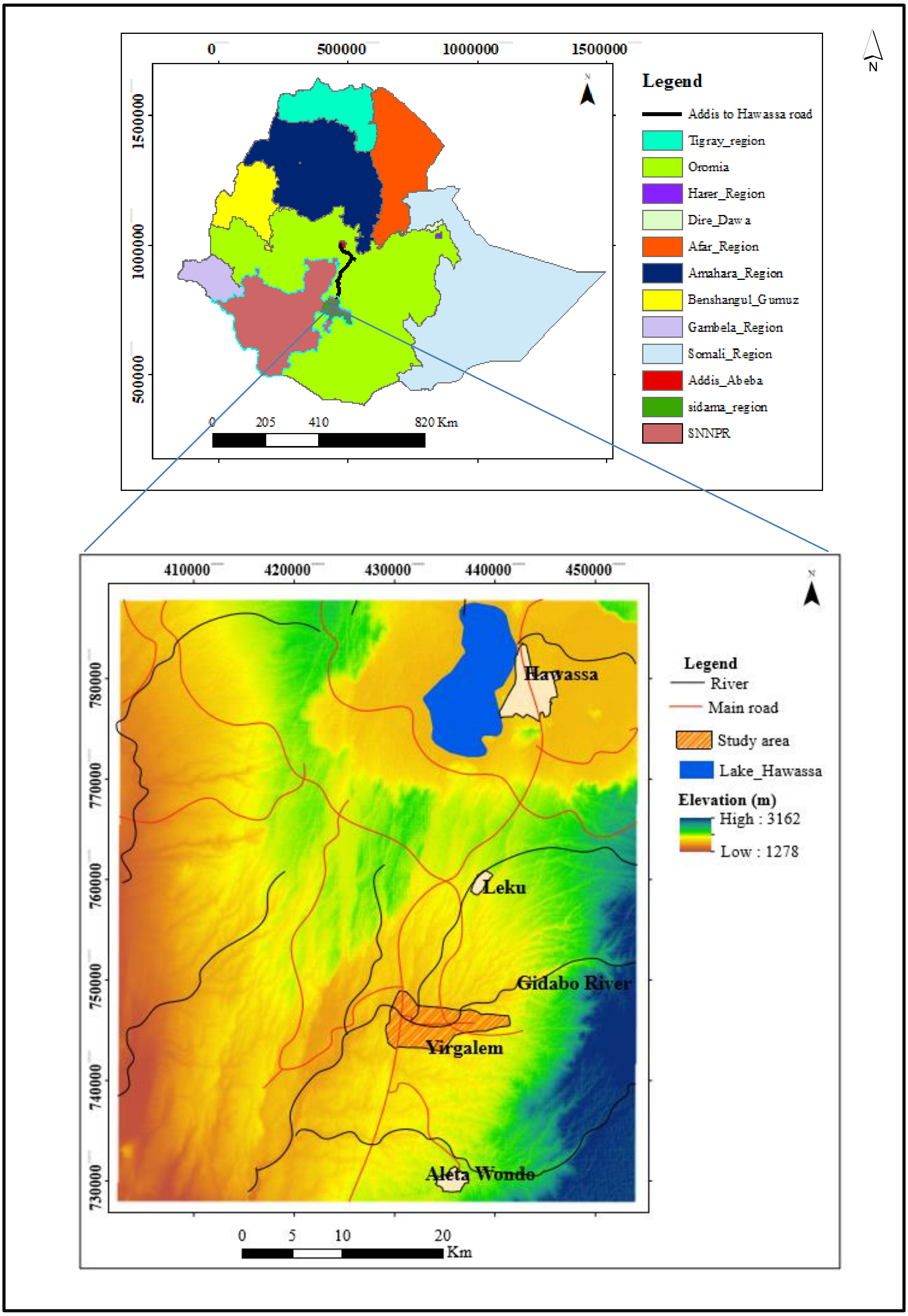


Figure 1: Location map of the study area

2.2.2 Physiographic Condition

The study area is part of the Main Ethiopian Rift valley (MER), it is defined by very steep-sloped, undulated, rugged, and flat or gentle sloped topography (Figure 2). Residual and transported soil deposits, volcanic ash, pumice, and ignimbrite unit covers the study area and they form undulating topography at central part to eastern (Yirgalem) and gentle (flat) landform at western (Abosto) part. In the study area, a steep slope is encountered at Gidabo River, Geo, and Woyma Rivers cuts while a very gentle slope is encountered at Awada and Kalit plain. This formation leads Yirgalem town to erosion, landslide and weathering. The maximum and minimum elevations in this area are 1670m to 1830m above mean sea level respectively.

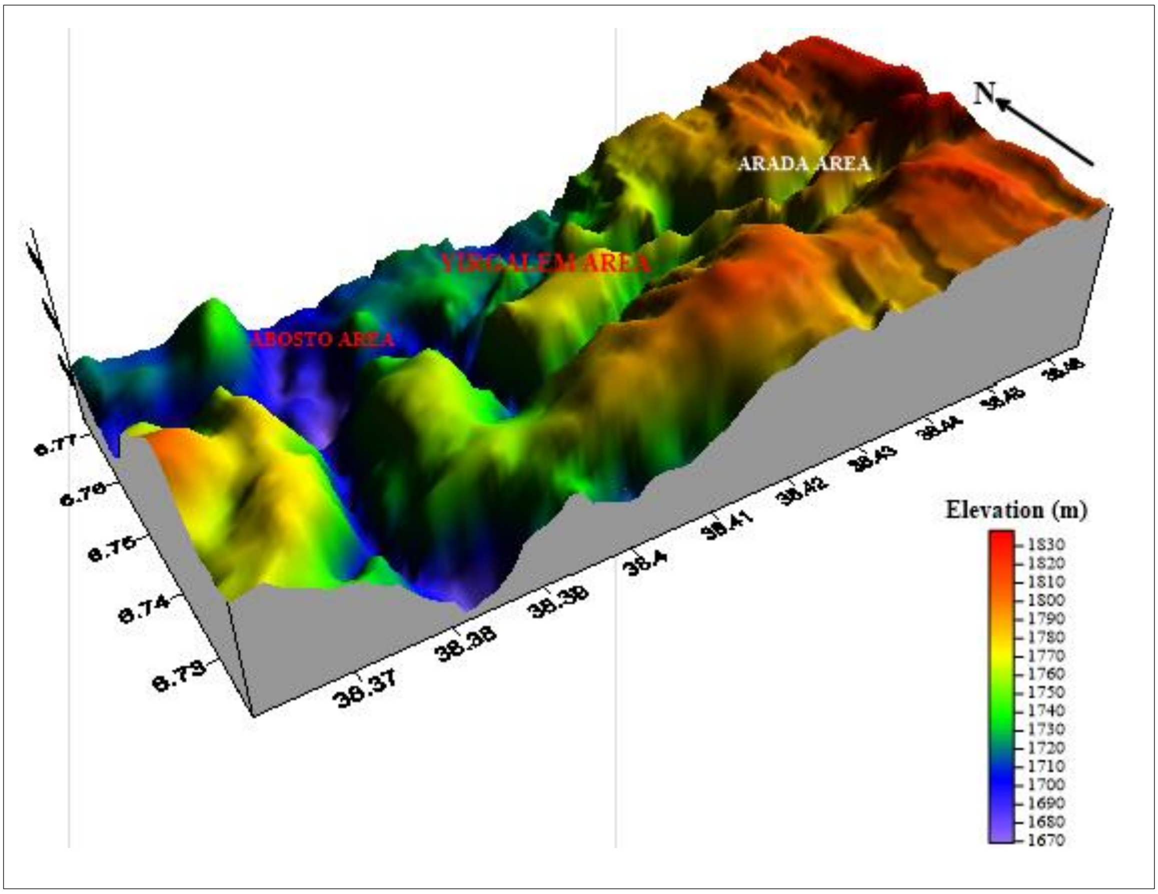


Figure 2: Physiographic map of the Yirgalem town

2.2.3 Drainage patterns

The drainage pattern of the study area is controlled by geology, geologic structure, topography, vegetation, rainfall amount, and intensity. All parts of Yirgalem town were surrounded by Rivers and the Gidabo River was the main Perennial River. There are also other small tributary Rivers in the eastern, central, and western parts of the study area. All River bodies are flowing from east to the west direction towards Lake Abaya. Gudumo Mountain which was found in Wonsho Woreda which is 19Km far from Yirgalem was the main source of rainfall and runoff for the study area. The whole pattern in the study area is therefore characterized by a dendritic drainage pattern (Figure 3). Therefore, high erosional effects along the river/stream cuts have occurred.

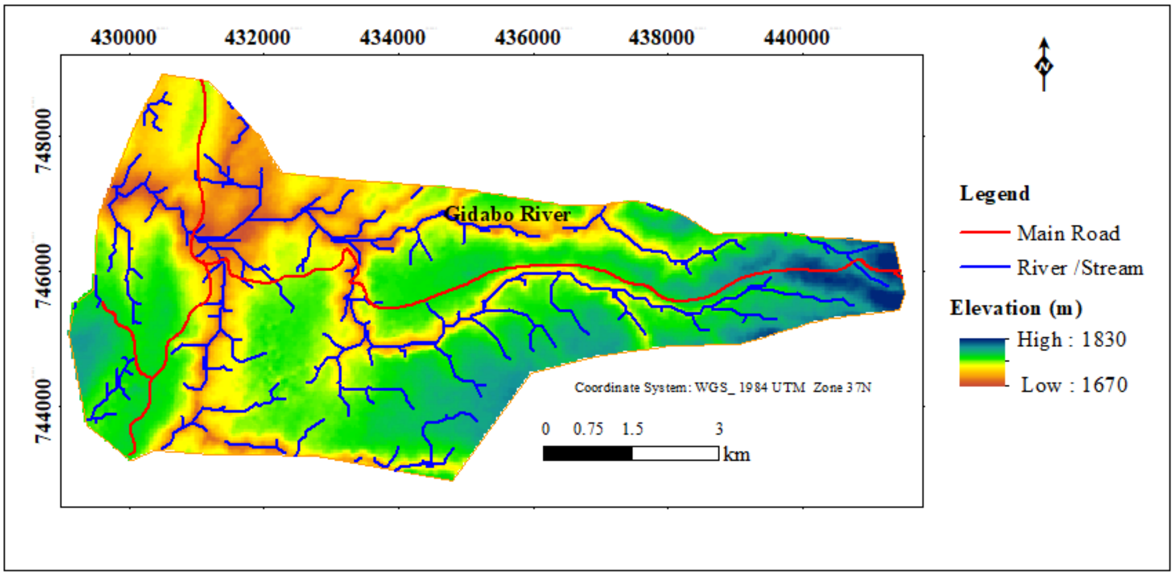


Figure 3: Drainage map of the study area

2.2.4 Climate condition

The climate of Ethiopia ranges from equatorial desert to hot and cool steppe, and from tropical savannah and rain forest to warm temperate, from hot lowland to cool high lands. The altitude ranges from around 120m below sea level in the Dallol depression up to 4,620 m a.s.l on the Ras Dashen on the Semien Mountain Massifs. The Main Ethiopian rift valley and the Afar are semi-arid to hyperarid (Alemayehu et al., 2006). Based on altitude, the climate can be classified into five groups (Table 1).

Table 1: Ethiopia Climate Classification System

Altitude (m.a.s.l)	Mean annual temperature (^{0}C)	Description	Local Name
3300 and above	10 or less	Cool	Kur
2300 – 3300	10 – 15	Cool temperate	Dega
1500 – 2300	15 – 20	Temperate	Weyina dega
500 – 1500	20 – 25	Warm temperate	Kola
Below 500	25 or more	Hot	Bereha

(Source: Alemayehu et al., 2006)

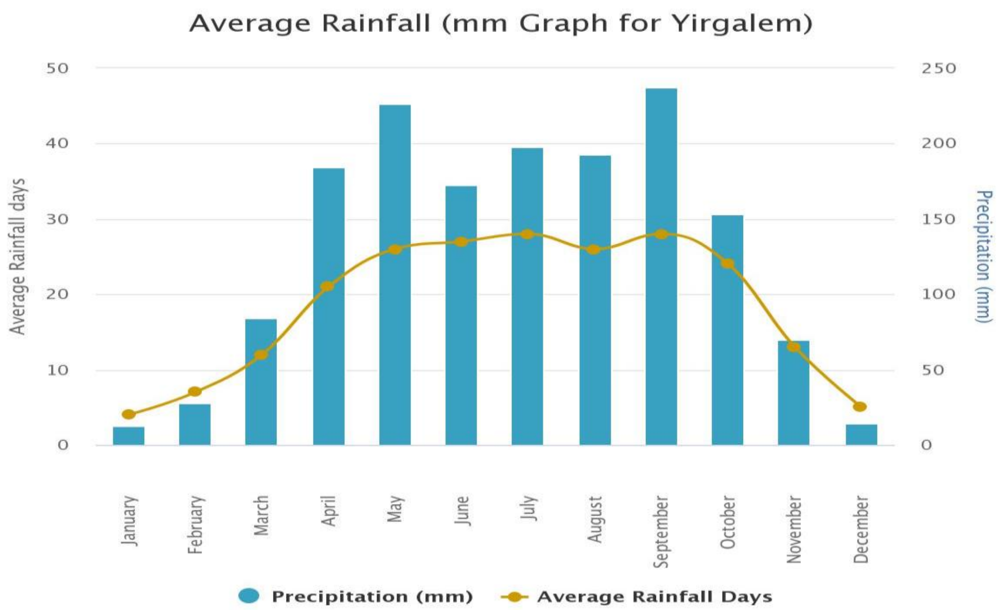
According to Ethiopian climate classification (Alemayehu et al., 2006), the study area which has an altitude ranging from 1500-2300 meters above mean sea level is classified as Weina-Dega or sub-tropical climate.

2.2.4.1 Rainfall

The seasonality of the rainfall in the study area is generally governed by the migration of the Inter-Tropical Convergence Zone (ITCZ). The forward and backward migration of the ITCZ produces substantial rainfall bringing moisture largely from the South Atlantic and Indian Oceans. The study area is located in a bimodal rainfall region with annual rainfall varying from 66.1 to 546.2 mm. September, May, June, July, and August are rainy seasons in the study area (Figure 4).

2.2.4.2 Temperature

Temperature is one of the controlling factors to alter the properties of rocks and soils and the day-to-day activities of society. The study area has an annual average maximum temperature of 27^{0}C and an annual average minimum temperature of 10.2^{0}C. March, February, January, and December are the hottest months with a monthly mean maximum temperature of 28^{0}C, 27^{0}C, 27^{0}C, and 26^{0}C respectively, and June, July, August, and September are the coldest months with a mean minimum temperature of 10.2^{0}C, 10.2^{0}C 12.3^{0}C and 11.7^0 C respectively (Figure 4).



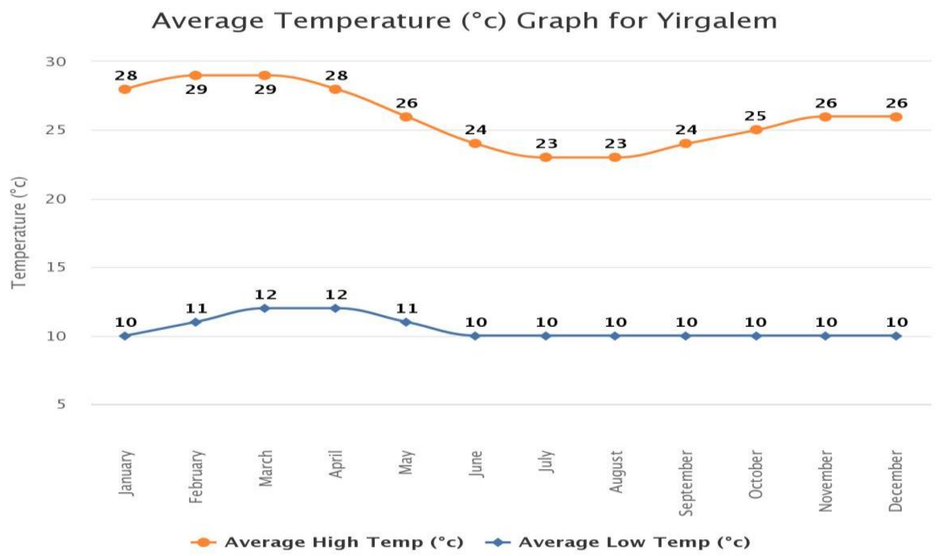


Figure 4: Average rainfall and temperature of the Yirgalem town

2.3 Geological set up of the study area

2.3.1 Regional geology of study area

Yirgalem town is located within the middle part of the Main Ethiopian Rift System on the border of its central and southern segments. Being situated in the Ethiopian Rift, the geological cover of the study area is the product of a series of volcanic eruptions since the Miocene (Wolde Gabriel et al., 1990). The evolution of the southern segment of the Main Ethiopian Rift has been divided into three stages according to age (Stewart and Rogers, 1996): pre-rift with volcanic rocks (32–45 Ma), syn-rift volcanic rocks (22–12 Ma), and post-rift sediment (0–5 Ma). The rocks covering the area fall into three major groups: pre-rift volcanic rocks, rift volcanic rocks, and post-rift sediments. The most important features of these rocks together with the geological structure are compiled from current and previous studies (Mohr 1962; WoldeGabriel et al. 1990; Acocella et al. 2003; Mechal, 2007; Yismaw et al. 2015; Tomas et al. 2018). Volcanic rocks of the Cenozoic age are widespread in various parts of the study area from the Miocene to the Holocene period. Dilla and Leku map sheets are recently produced maps by the Geological Survey of Ethiopia and Czech Geological Survey respectively. The regional geology in the study area and surroundings are described below.

2.3.1.1 Tertiary Volcanic Rocks

A, 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, Teferi kela, 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). The rhyolite is composed of quartz and k-feldspar which is slightly weathered, cliff cutting exposure shows pink to light pink fresh color. Minor ignimbrites and trachytes which are not mappable also occur in this unit (Yismaw et al. 2015).

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

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 (Yismaw et al. 2015). In some parts of the mapped area, there are scarcely exposed trachyte but they are not mappable for the scale. Porphyritic basalt is intercalated with the aphyric basalt especially along the road from Bule to Hacha small towns (Yismaw et al. 2015).

C, Nazareth Group and 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 Damot. Related to Dilla map sheet and southeast 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). This lithic fresh ignimbrite shows light gray to gray color the weathered color is light brown. Most of it contains rhyolitic and trachytic rock fragments with fine and compacted groundmass. It makes columnar joints. The fresh rhyolite shows pink color, medium to coarse grains, and slight to high weathering (Yismaw et al. 2015).

D, Alkaline and peralkaline silicic, rhyolite domes, and ignimbrites

According to Yismaw et al.,(2015). This unit is exposed around Lake Hawassa on the east part to Dilla map sheet and northern parts of 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.

E, 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).

2.3.1.2 Quaternary Volcanic Rocks

A, 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 in color. 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 (Yismaw et al. 2015).

B, Ignimbrites, tuffs, waterlain, 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 (Yismaw et al. 2015).

C, 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 naked eye (Yismaw et al. 2015).

D, Lacustrine, volcanoclastic 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 Hawassa and north of Abaya making a sharp contact with Pleistocene basalt which is exposed the north of Abaya(figure 5). The rock units include aphyric basalts, scoriaceous basalts, ignimbrites, unwelded tuffs, volcano sediments and lacustrine sediments. The dominant exposures are volcano sediments and tuffs. The volcanoclastic sediments show light gray color and are friable and 20m thick. The exposure is best seen along the road. It covers Humbo, Dawe, Gulgula, Belela, Dimtu, and Werancha wocho localities (Yismaw et al. 2015).

E, **Post-rift volcanic deposits (Pleistocene age)**

Regionally, in the study area, Hantate ignimbrite, Yubo Basalt, and Boricha rhyolite are exposed mainly in Pleistocene age (Tomas et al. 2018). According to Tomas et al. (2018), Pleistocene Dino Formation is regionally exposed in different areas and the most exposed post- rift volcanic deposits are the Hantate Ignimbrite, the Yubo Basalt, and Boricha Rhyolite. Yubo basalt subordinated with ignimbrite was exposed in southern parts of leku map sheet and central parts of the study area.

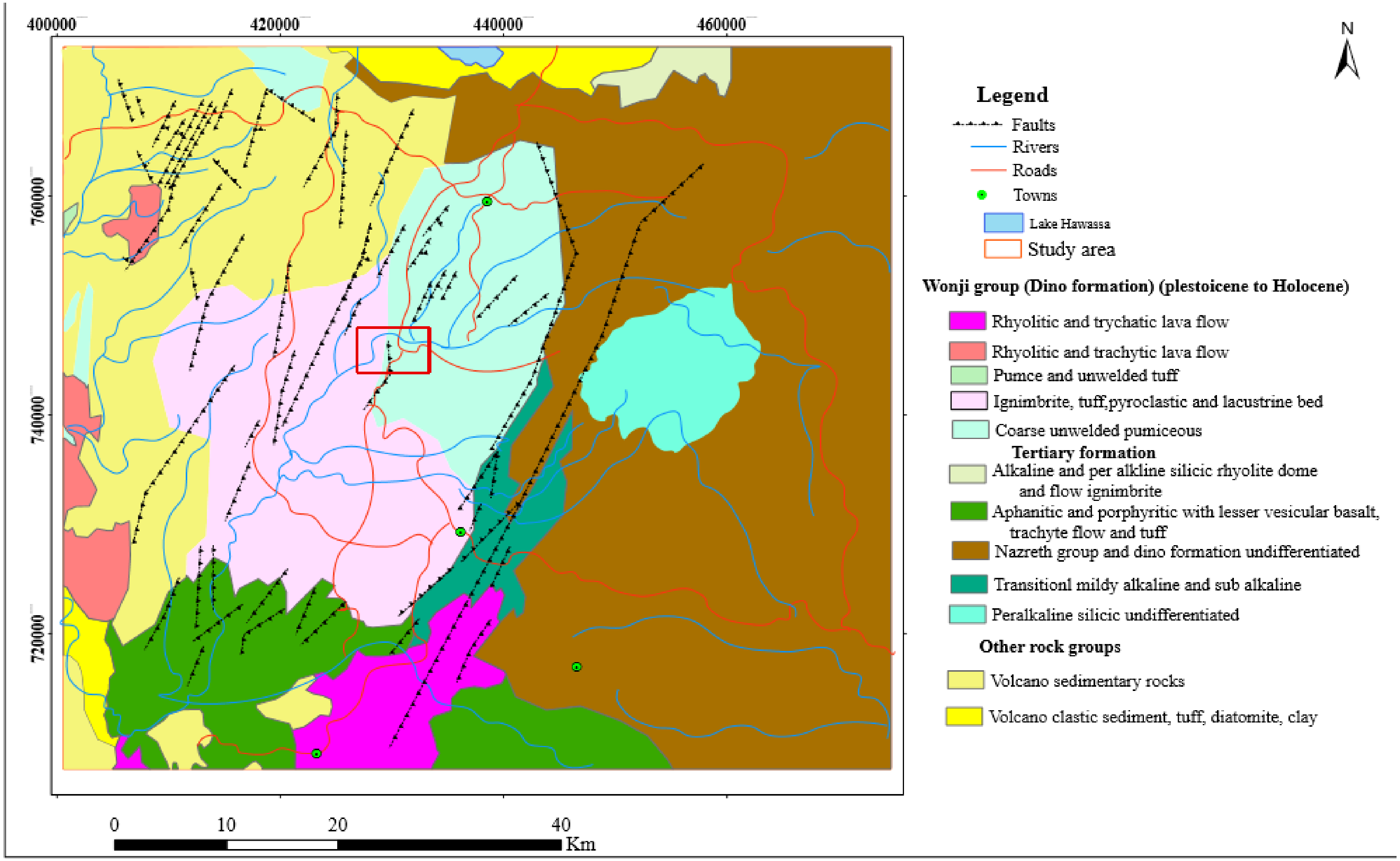


Figure 5: Regional geological map of the study area (after, Dilla map sheet (NB37-6) produced by Ethiopian Geological Survey (2015)) (Yismaw et al. 2015)

2.3.2 Local Geology

The study area is covered by the alluvial deposit, residual soil deposit, transported soil deposit, and various types of volcanic rocks which include basalt, volcanic ashes, pumice, and ignimbrite. The above-mentioned lithological units are described as follows.

2.3.2.1 Alluvial deposit

It is a quaternary sediment deposit and covers limited parts in the study area. Alluvial deposit textures range from clay size to gravel with minor boulders and rock fragments. This unit is exposed in the northeastern and northwestern parts of the study area (figure 13). Its thickness varies from 4 to 11m, with light gray to brown colour, and loose to dense in some places, they are stratified and at some places it is weathered (Figure 6).

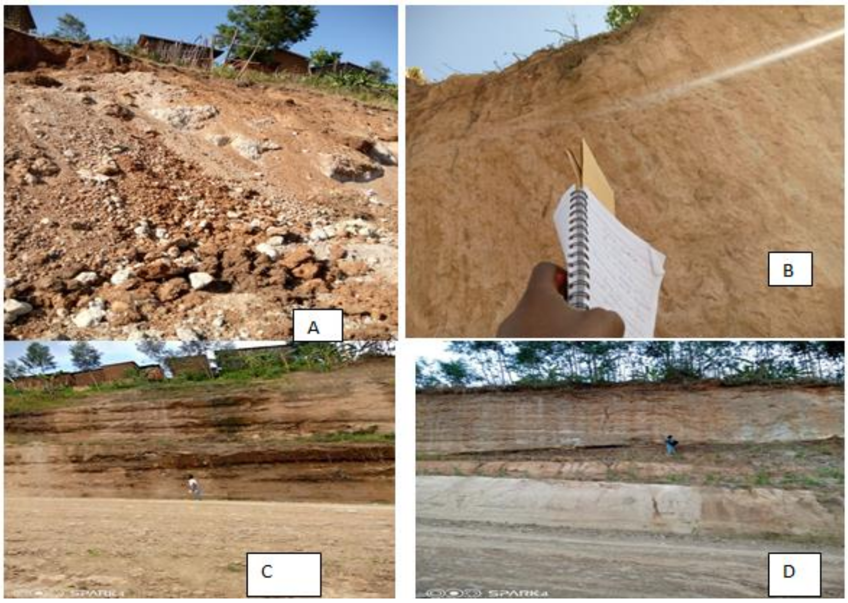


Figure 6: Quaternary sedimentary deposit (alluvial deposit). A) light to brown in color at GPS reading (long.431776 m lati.747072 m). B) Dense and cemented deposit at GPS reading (long.438213 m lati.745688). C) Stratified sedimentary alluvial deposit. at GPS reading (long.438458 m lati. 746098 m). D) at GPS reading (long. 431856 m lati.748016 m).

2.3.2.2 Residual soil

The study area is largely covered by residual soil. This soil is the soil that remains at the place of its formation and is common along the gentle sloped or flatlands. It is black to brown and called black cotton soil in central (Yirgalem), western part (Abosto)/Abosto plain and Hida Kalit(YAIP)/ Kalit plain area are covered by residual types of soil due to its gentle slope(Figure 7A). Reddish-brown soils (Figure 7B) are largely deposited at the eastern and north-central part of the study area (Figure 13).

Figure 7: Residual soil in the study area at GPS reading. A) gray soil (long. 429709 m, lati. 745172 m). B) Reddish brown soil (long.430445 m, lati. 745647 m)

2.3.2.3 Cataclastic sediment

In the study area unmappable Alluvium deposited of different sizes and shapes of unconsolidated cataclastic sediments of breccias, conglomerate, boulders, and gravels, are deposited along the main river Gidabo, Woyima, Geo, and other tributary streams of the study area (Figure 8).

Figure 8: Cataclastic sediment at GPS reading A, (long.435719 m lati.746502m). B (long.434900 m lati. 746313 m).

2.3.2.4 Basalt unit

Basalt rock is a type of igneous rock which is the exposed in the north-central part of study area (Figure 13). It was distinguished by its black color. In the study area, chemical weathering (exfoliation) was observed (Figure 9). This dark gray basalt is exposed commonly grouped under Yubo basalt formation.

Figure 9: A) basalt outcrop in the study area at GPS reading (long. 433668 m lati. 746455 m).

2.3.2.5 Ignimbrite rock unit

Ignimbrite rock units are type of pyroclastic rock (pyroclastic flow). It exhibits different textural characters due to the grade of weathering. In the study area, it was exposed at western, central, and south-central parts with fresh to moderately weathered one (figure 13). Those ignimbrite rocks exposed in the study area are groups of Dino formation, belonging to Hantate ignimbrite formation (Tomas et al. 2018) (figure 10).

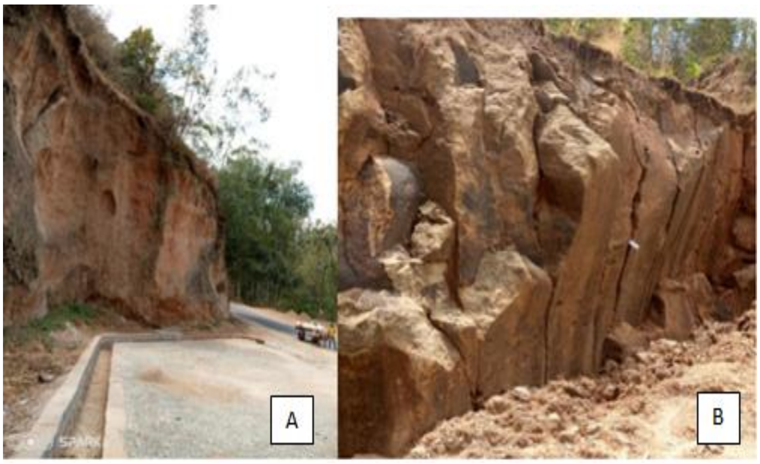


Figure 10: Ignimbrite rock unit exposure, at GPS, reading A) (long. 433201 m lati.745191 m). B) (long.430137 m lati.744649 m).

2.3.2.6 Pumice rock unit

The pumice rock unit is friable and vesicular. It is well exposed in the northern, northeastern, and southern parts of the study area (figure 13). It is exposed along the river/stream cut and roads cut. It is light brown colored and less dense/lightweight and it can float on water due to its high vapor bubbles/ pore spaces (figure 11).

Figure 11: Pumice rock unit at GPS reading (long.436895 m lati.746431 m)

2.3.2.7 Volcanic ash deposit

These rock units are a type of pyroclastic rock (pyroclastic fall). The volcanic ash rock unit is loose, fine to coarse-grained, and found at the eastern, northeastern, and central parts of the study area (figure 13). It is well exposed along deep stream/river cut gullies and road cut (figure 12). It is light brown in color. An alternating thin layer of pumice is common in this lithological unit.

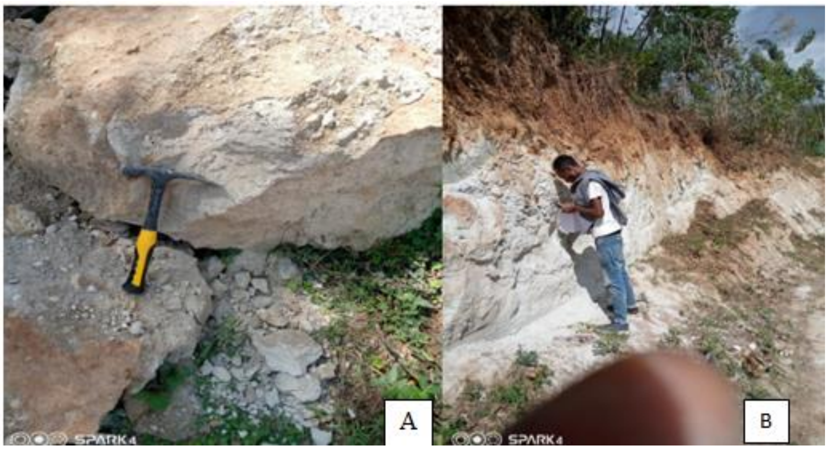


Figure 12: Volcanic ash fall deposit (A) volcanic ash at road cut exposure (B) at GPS reading (long.434233 m lati.743319 m).

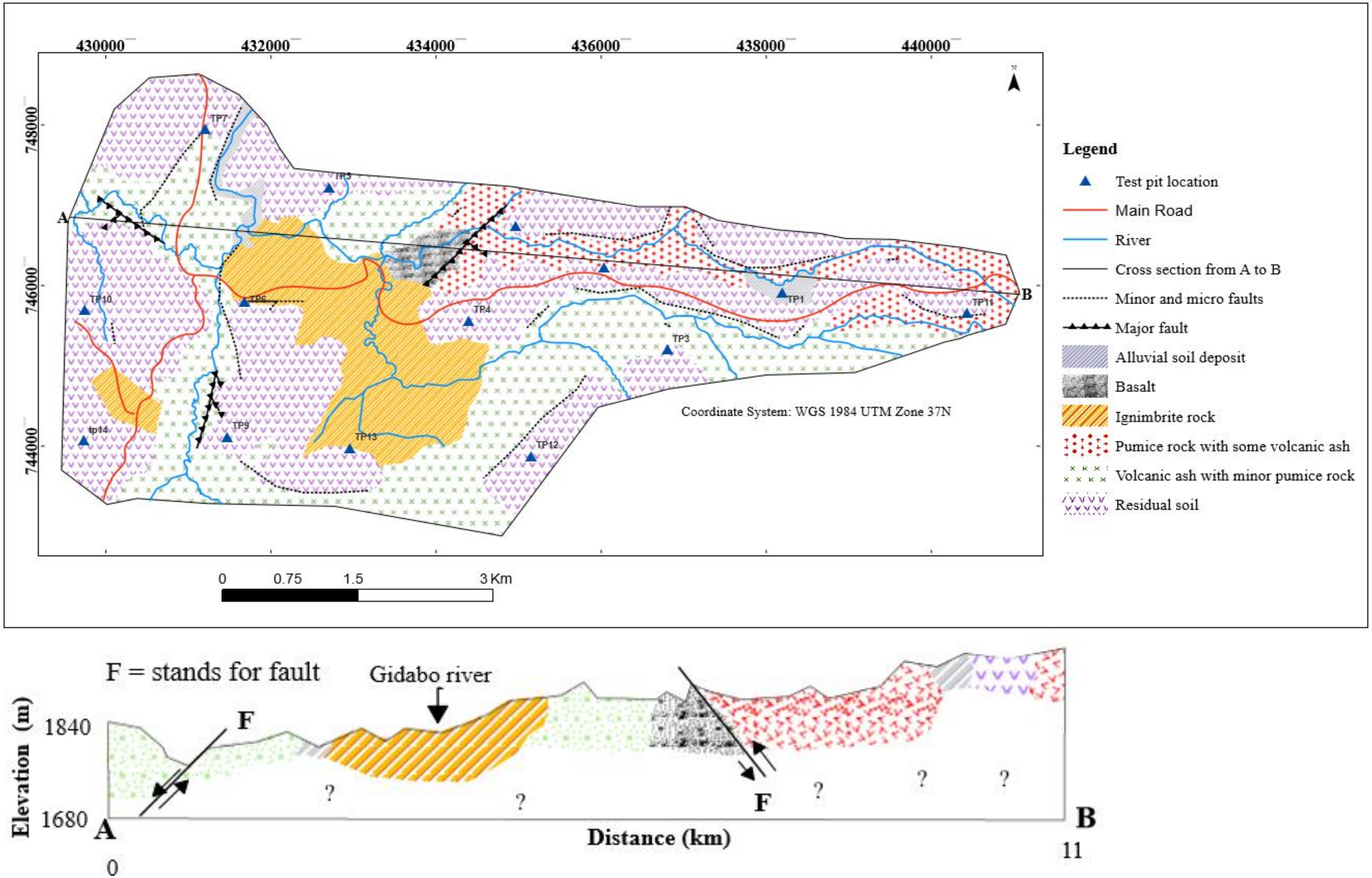


Figure 13: Geological Map of Study Area and Cross Section (1:20000)

2.3.3 Geological structure of the study area

The study area is comprised of post-rift rocks and volcanoclastic deposits (basalt, pumiceous, ignimbrites and related pyroclastic deposits). A set of primary fabrics (horizontal bedding) and brittle rift-related structures (normal faults and extensional joints) were identified in the study area.

2.3.3.1 Bedding

Ignimbites and related volcanic deposits (unwelded pyroclastic fall deposits and volcanic ash flows) reveal a predominantly sub-horizontal to roughly NW–SE or NNE–SSW gently dipping bedding planes (figure 14). Alluvial deposit of cropping out around the river and road cuts also shows horizontal to inclined bedding.

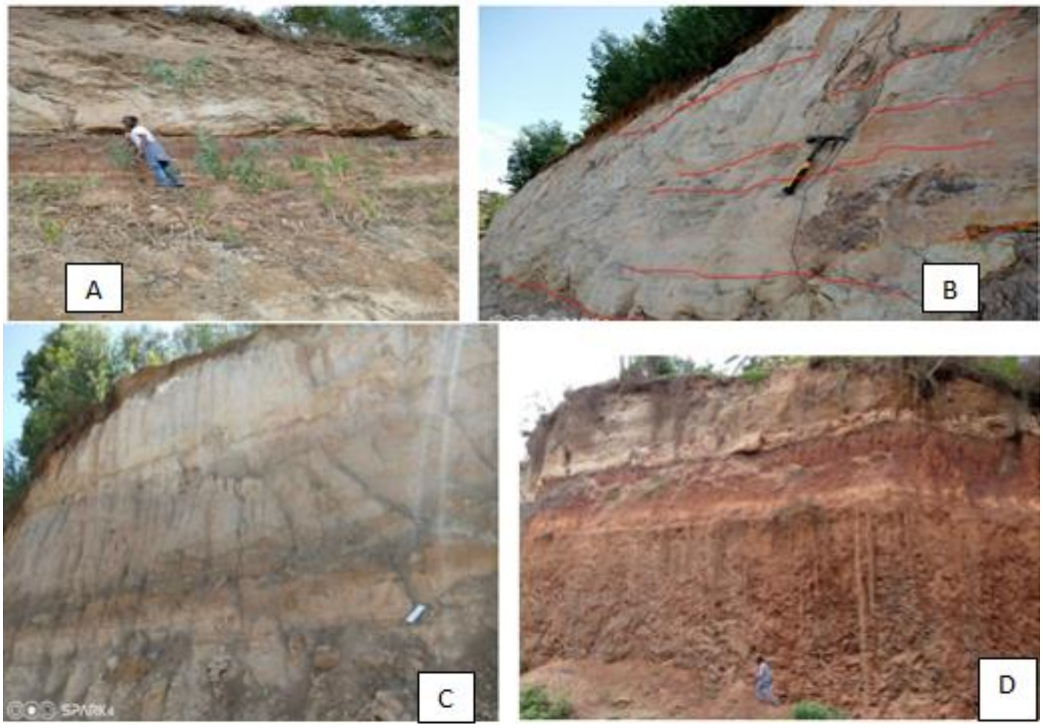


Figure 14: Horizontal bedding in the study area, at GPS reading A) (long.437857 m lati.745172 m). B) (long.436054 m lati.745423 m). C) (Long.434911 m lati.744709 m). D) (long. 433700 m lati.746431 m).

2.3.3.2 Faults

In the study area, predominant faults are normal faults and mostly parallel to the axis of the rift forming prominent morphological features of the rift system. These regional faults dip steeply to ESE or WNW. There are also younger faults that have a fault lineation plunging steeply to moderately to S. steeply ~N(NNW) dipping faults with steeply plunging perpendicular to the regional faults, micro faults moderately dipping with a NW–SE strike was also observed in a few localities in the study area. It is not possible to measure the dip of faults due to the absence of a definite fault plain.

2.3.3.3 Joints

Joints are fractures along which little or no displacement has occurred and are present within all types of rocks (Bell, 2007; Chemeda, 2020). In the study area the most prominent joints run N (NNE)–S (SSW), E (ENE)–W (WSW), and NW to SE. Most of the joints are vertical (steeply dipping). Minor sub-vertical joints dip to the NE or SW. from measured joint orientation, 30 representative joint were plotted on Rosset diagram (Figure 15).

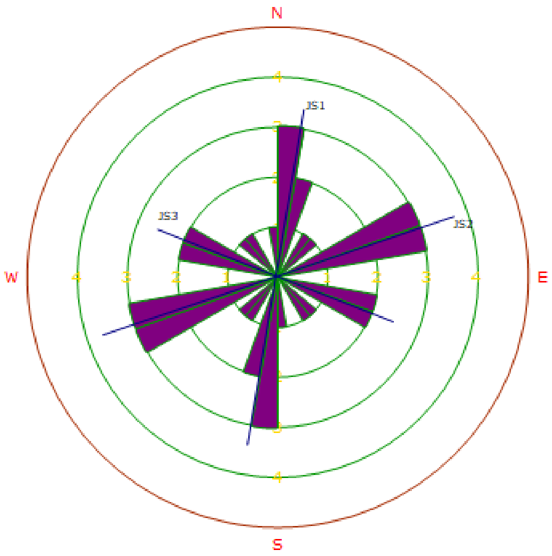


Figure 15: Rosset diagram of joints in study area plotted by dip.6.00 software

CHAPTER THREE

LITERATURE REVIEW

3.1 Introduction

An engineering geological map is one of the geological maps which represent all components of a geological environment. It gives very important clues for land use planning, design, construction, and maintenance as applied to civil engineering. It is mainly directed toward understanding the interrelationship between the geological environment and the engineering situation (IAEG, 1981; Bell, 2007; Chemeda, 2020). The principal factors creating the engineering geological conditions of an individual site or area are the rocks, soils, water, geomorphological conditions, hydro-geological and geodynamic processes. It provides the best impression of a geological environment including the character and variety of engineering geological conditions, their components, and interrelationships (Culshaw, 2018).

3.2 Engineering geological mapping techniques

The purpose of engineering geological maps is to show the distribution of rocks and soils including their characteristics and other geological phenomena (hydrogeological, geomorphological, and surface geodynamic) condition that affect engineering works. The boundaries of rock and soil units shown on engineering geological maps should delimit rock and soil units which are characterized by a certain degree of homogeneity in basic engineering geological properties (Unesco Press, 1976). The term soil in the engineering geological point of view, contains all the materials found in the surface layer of the earth crust that are easily loose enough to be moved by spade, shovel or can be excavated without the use of explosives or equipment for loosening. The main problem in engineering geological mapping is the selection of geological features of rocks and soils which are closely related to physical properties such as strength, deformability, durability, and permeability. At present, there is a lack of regional data on the variability of engineering properties of rocks and soils. No suitable methods and techniques have been developed for determining them in sufficient quantity, over large areas, quantitatively, quickly and cheaply. Many civil engineering structures are constructed on and/or within the top few meters of surficial deposits. According to Unesco Press (1976); Price et al. (1982); Getahun et al. (2013), an area is mapped as a soil unit if the soil thickness exceeds 1 m. But, if the soil thickness is less than 1 m, the area is mapped as the underlying bedrock. The thickness of 1 m is chosen for two reasons. First, the surface morphology of the underlying bedrock is usually recognized through the overlying soil if its thickness is less than 1 m and not recognized when the thickness exceeds 1m. Second, the engineering geological importance of a soil cover with a thickness less than 1m is limited (Getahun et al. 2013). In the current work, the presentation of geological and engineering geological soil/rock units on both geological and engineering geological maps respectively is done according to the aforementioned mapping technique.

3.3 Engineering geological characterization and classification of soils

The soils and rocks of the study area were classified based on their related engineering properties obtained from field and laboratory work. It is classified according to the standard classification systems of soils and rocks (Arora, 2003). This helps to prepare multi-purpose engineering geological maps.

3.3.1 Engineering geological characterization of soils

Soils are characterized by their index and engineering (design) properties (Arora, 2003). For the design and construction of specific sites of engineering structures, a detailed geotechnical investigation should be conducted. Therefore, the geotechnical engineers can be able to decide suitability of a specific site for the foundation of engineering structures based on the engineering properties of the soil. In this research, the soils of the study area were characterized by the following index and engineering properties.

3.3.1.1 Index property

The physical properties of soils that serve mainly for identification and classification purpose are commonly known as index properties (Arora, 2003; Das, 2012). Index properties can be determined by simple laboratory tests. These tests are natural moisture content, grain size analysis, Atterberg limits, and free swell and specific gravity test. Natural moisture content can change the behavior of soil when used for construction purposes and foundation of structures. It also affects the shear strength and suitability of soils for compaction. Moisture content causes the shrink/swelling potential of the expansive of soil. The water content, which is usually expressed as a percentage, can range from zero to several hundred percent (Murthy, 1990). Specific gravity is used to determine soil particle size through a hydrometer and it largely depends on the density of minerals making up the individual soil particles (Das, 2012).

Particle size analysis is a method of separation of soils into different fractions based on particle size (Arora, 2003). 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). The distribution of different grain sizes affects the engineering properties of the soil and it is required in classifying the soil (Arora, 2003 and Das, 2012). An Atterberg limit is the water content at which the consistency changes from one state to another (Arora, 2003). It consists of liquid limit, plastic limit, and plasticity index (Figure 16). The greater is the difference between the liquid and plastic limit, the greater is the plasticity of the soil (Das, 2012; seed et al. 1962). Free swell gives a clue about the swelling (clayey) nature of the soil and which increases with plasticity index and indicates potential expansiveness of soil (Chen, 1975). pH is one of the chemical tests used to determine whether the soil is acidic, neutral, or basic (Abramson et al. 1996). The knowledge of the soil chemistry, such as pH, is essential to determine the aggressiveness of the soil to the engineering structure. Dissolution reaction is higher at acidic and alkaline pH conditions that can deteriorate (corrode) the concrete of engineering structure (Chemeda, 2020).

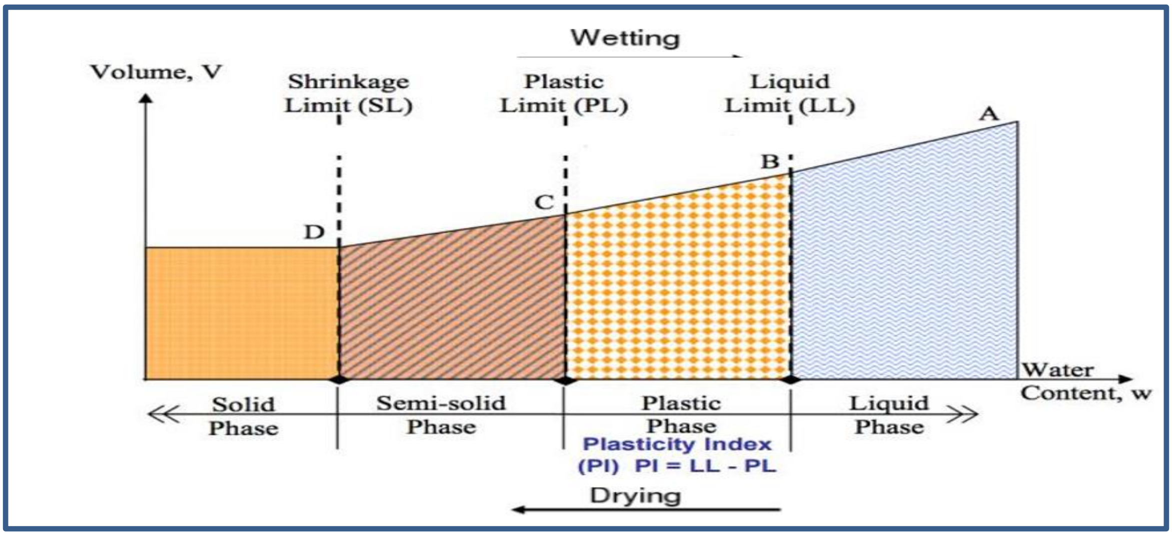


Figure 16: Soil phase changes and atterberg limits (Das, 2012)

3.3.1.2 Engineering property

Compressibility, shear strength, and permeability are the main engineering properties of soil. For this research work, shear strength and compaction tests were mainly conducted. According to (Murthy, 1990), the shear strength of soil is its ability to resist sliding along internal surfaces within a mass. The stability of a cut, the foundations of structures, the natural slopes of hillsides, and other structures built on the 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 obtained according to mohr- coulomb failure criterion. A compaction test is performed to determine the relationship between the moisture content and the dry density of soil for a specified compaction effort. The compactive effort is the amount of mechanical energy that is applied to the soil mass. The optimum water content is the water content that results in the greatest density for a specified compaction effort. Compacting at water contents higher than wet of the optimum water content results in a relatively dispersed soil structure (parallel particle orientations) that is weaker, more ductile, less pervious, softer, more susceptible to shrinking, and less susceptible to swelling than soil compacted in dry of optimum (Das, 2012).

3.3.2 Engineering geological classification of soils

Classifying soils with similar behavior, in terms of simple indices, can provide geotechnical engineers with the general guidance about the engineering properties of soils through accumulated experiences. Tests carried out to classify a soil are classification tests. The results obtained based on these tests are called index properties of soils. A soil classification should provide some guidance to engineering performance of the soil type and provides a means by which a soil can be identified quickly (ISRM, 1981 as cited in Berhane, 2010).

Unified Soil Classification system (USCS) and classification system proposed by the International Association of Engineering Geology (IAEG, 1981) are the commonly used engineering soil classification systems. Both systems take into consideration the particle size distribution and Atterberg limits (Table 2).

Table 2: Grain Size Range according to USCS and IAEG (1981)

Soil Type	Grain Size Range (mm)	
	USCS	IAEG(1981)
Gravel	76.2 to 4.75	60 to 2
Sand	4.75 to 0.075	2 to 0.06
Silt	Fines < 0.075	0.06 to 0.002
Clay		< 0.002

3.3.2.1 USCS Classification System

The Unified Soil Classification System is based on the type and prevalence of the constituents considering grain size, gradation, and plasticity.

3.3.2.2 International Association of Engineering Geology(IAEG, 1981) classification system

IAEG (1981) classification system is generally based on grading and plasticity. Grading and plasticity are divided into several clearly defined ranges, each of which may be referred by a descriptive name and letter.

3.4 Engineering geological characterization and classification of rocks

3.4.1 Engineering Characterization of Rocks

When a rock mass is intersected by a sufficiently large number of weakness planes, the mechanical behavior is controlled by mass properties (joint, fracture, fault, etc) than by material properties. The rock mass is an assemblage of intact rock materials separated by geological discontinuities. In such cases, rock mass classification is commonly used to evaluate rock mass behavior. Rock mass classification puts a number to the quality of the rock mass i.e. quantifies the quality. Material properties dominate the strength of a rock mass where discontinuities are widely spaced or nonexistent. Intact rocks are the continuum of polycrystalline solid between continuities consisting of an aggregate of minerals or grains. Its properties are governed by the physical properties of the materials of which it is composed and how they are bonded to each other. Rock masses are the in-situ medium that comprises intact rock blocks separated by discontinuities.

It contain discontinuous and often have heterogeneous and isotropic properties. Discontinuity is a general term denoting any separation in a rock mass having low tensile strength. It is the collective term for most types of joints, weak bedding planes, weak schistosity planes, weakness zones, and faults (ISRM, 1981). International Society for Rock Mechanics (ISRM, 1981) characterizes rock masses based on discontinuity characteristics, joint spacing, aperture, weathering conditions of wall strength, and seepage (Table 3 and Table 4).

Table 3: Characterization of rock based on discontinuity spacing and aperture (ISRM, 1981).

<table>
<tr><th colspan="2">Discontinuity spacing</th><th colspan="3">Discontinuity aperture</th></tr>
<tr><td>Interval(mm)</td><td>Descrption</td><td>Interval (mm)</td><td>Description</td><td rowspan="4">“Closed” features</td></tr>
<tr><td><20</td><td>Extremely close</td><td>< 0.1</td><td>Very tight</td></tr>
<tr><td>20-60</td><td>Very close</td><td>0.1-0.25</td><td>Tight</td></tr>
<tr><td>60- 200</td><td>Close</td><td>0.25-0.5</td><td>Partly open</td></tr>
<tr><td>200-600</td><td>Moderate</td><td>0.5-2.5</td><td>Open</td><td rowspan="3">“Gapped” features</td></tr>
<tr><td>600-2000</td><td>Wide</td><td>2.5-10</td><td>Moderately wide</td></tr>
<tr><td>2000-6000</td><td>Very wide</td><td>>10</td><td>Wide</td></tr>
<tr><td>>6000</td><td>Extremely wide</td><td>10-100</td><td>Very wide</td><td rowspan="3">“Open” features</td></tr>
<tr><td colspan="2" rowspan="2"></td><td>100-1000</td><td>Extremely wide</td></tr>
<tr><td>>1000</td><td>Cavernous</td></tr>
</table>

Table 4: Characterization of rock based on the degree of weathering for rock (after ISRM, 1981)

Term	Description
Fresh rock	No visible sign of rock material weathering
Slightly weathered rock	Discoloration indicates weathering of rock materials and discontinuity surfaces
Moderately weathered rock	Less than half of the rock material is decomposed and/or disintegrated into soil
Highly weathered rock	More than half of the rock material is discomposed and/or disintegrated to soil
Completely weathered rock	All rock material is decomposed and/or disintegrated into the soil. The original mass structure is still largely intact
Residual soil	All rock material is converted to the soil. The mass structure and material fabric are destroyed

3.4.2 Engineering Classification of rock

Rocks are classified into different types based on field visualization, simple estimation from the geological hammer, Schmidt hammer and different discontinuity characteristics. Suggested field description method is proposed by (ISRM, 1981).

3.4.2.1 Engineering geological classification of rock based on strength, weathering, and discontinuity

The classification system was an estimation of uniaxial compressive strength based on a simple geological hammer according to (ISRM, 1981) as can be seen in table below (Table 5).

Table 5: field UCS estimation (ISRM, 1981)

Grade	Term	UCS(MPa)	Field description
R5	Extremely strong	>250	The specimen can only be chipped by a geological hammer
R4	Very strong	100 -250	Specimen requires many blows of a geological hammer to fracture it
R3	Strong	50 -100	Specimen requires more than one blows of a geological hammer to fracture it
R2	Medium-strong	25 – 50	Cannot be scrapped by pocket knife, specimen can be fractured by a single blow of the geological hammer
R1	Weak	5 - 25	Can be peeled with a pocket knife, difficulty, shallow indentation, made by a firm blow with point of the geological hammer

3.4.2.2 Engineering geological classification of rock based on Uniaxial Compressive Strength (UCS) and UCS and SHV correlation

Rock engineering properties are one of the most important parameters in the design of groundworks and the classification of rocks for engineering purposes. The correlations proposed in different literature between unconfined compressive strength and Schmidt hammer rebound value SHV were investigated to define the effectiveness of each one in predicting rock strength for various rocks. Kahraman et al. (2002) equations have a good prediction for high values of UCS (for igneous rocks). Kahraman (2002) also stated that the Schmidt hammer could easily and practically be used for the prediction of unconfined compressive strength. Yasar and Erdogan (2004) demonstrated that many factors, such as Schmidt hammer types (e.g, classic N-type and L-type), sample sizes, weathering, rock type, and softness, can affect rebound values. Therefore, they proposed the equation to correlate UCS and Schmidt rebound value relationships which are suitable only for the specific rocks. Wang et al. (2017) proposes a new correlation between UCS and Schmidt hammer rebound values for both N-type and L type Schmidt hammer values.

$UCS = 3.03RN - 30.3$ ………..Equation 1

for all types of rock (unspecified rock type)

Where UCS is uniaxial compressive strength,

RN is N-type Schmidt hammer rebound value.

It is valid only if the N-type Schmidt hammer rebound value is greater than ten.

Wang et al. (2017) correlate UCS and rebound value by using different studies and their data analysis for all rock types with some validation. In addition to this (Barton and Choubey, 1977) correlate Schmidt hammer rebound value with UCS by using formula.

$\log10 (UCS) = 0.00088\gamma R + 1.0l$ ……………….……………….………..…Equation 2

Where; ‘UCS’ is the uniaxial compressive strength in Mpa, γ is the dry rock densities in KN/m^3 and R is the average rebound value for Schmidt hammer.

3.5 Assessment of geomorphological, hydrogeological and surface geodynamic processes

A, Landslide

Landslides represent the rapid downward and outward movement of slope-forming materials. The movement taking place by falling, sliding, or flowing or by some combination of these factors (Griffiths 2005 cited in Bell, 2007). It is the earth processes that damage engineering works to any degree. It can affect the engineering structures constructed on the moving mass and those constructed on its path. Therefore, anyone who wants to construct engineering structures on the top of hills or at the foot of the hill must investigate to determine the stability condition of the slope and landslide-prone area. Landslide may be classified into various categories based on the mode of movement, the material involved, speed of movement, and others. According to Varnes, (1978), landslide is classified based on the types of material involved (Rock, Earth, Soil, Mud, and Debris) and types of movement (fall, topple, slide, spread, flow) and complex class of movement which contains two or more different modes of movement acting downslope for the movement of the landslide mass (Table 6).

Table 6: Landslide classification (Varnes, 1978)

Type of movement		Types of materials		
		Engineering soils		
		Bedrock	Predominantly coarse (debris)	Predominantly fine (earth)
Falls		Rockfall	Debris fall	Earthfall
Topple		Rock topples	Debris topple	
Slide	Rotational	Rock slides	Debris slides	
	Transitional	Rock slides	Debris slides	Earth slides
Flows	Rock flow (deep creep)		Debris flow	Earth flow
Complex	combination of two or more types of movement			

B, Problematic soils

Shrinkage cracks formed by the evaporation of water from the surface of clay-rich sediment previously were called mud cracks, which are caused by the slow drying out of muddy sediments which have been exposed to the action of sun and wind (Chen,1975). Expansive soil affects engineering construction by creating gaps around the basement walls which allow airflow along the basement wall that will dry the soil under the foundation then by creating movement in the foundation through swelling and shrinking, permitting rainwater to run down into the basement.

CHAPTER FOUR

RESEARCH DESIGN AND METHODOLOGY

4.1 General

To meet the general and specific objectives that are mentioned earlier, appropriate design and methodologies must be applied. Moreover, a detailed understanding of the working principles, type of information, and clear methodologies to solve the problem under investigation is very crucial. The data on geological and surface geotechnical investigations were obtained from primary data acquisition in the field. Geotechnical, metrological, groundwater borehole data, and other data are secondary data were acquired. Hence, this chapter presents data acquisition processes and the general working principles of these techniques that were started from initial desk study to data acquisition in the field, sampling and laboratory testing of samples, and processing and analyzing of these data.

4.2 Data collection techniques

Data collection for this study starts from reviewing different litterateur surveys of published and unpublished reports on geology, hydrogeology, engineering geology, geotechnical, additional reports, journals, conferences, and books related to this study for better understanding of subject matter. Preliminary surveys and detailed field surveys are also data collecting techniques.

4.2.1 Collection of relevant data from preliminary survey

Collection of data from a Preliminary survey of the study area was carried out based on information obtained from the regional geological map (1:250000), topographic maps (1:50000), and Digital Elevation Model (DEM). The topographic map obtained from the Ethiopian mapping agency and Yirgalem Town urban planning map(Yirgalem boundary shape file) were used to produce a base map and to identify the topography of the study area. DEM of the study area was mainly obtained from SRTM was used to prepare geological, physiographic, and drainage maps of the area using ArcGIS and Surfer 10. This information was then used as a basis for planning and executing a detailed field survey.

Reconnaissance surveys were carried out by walking over the study area to visualize variation in soil lithology, to select the location of pits and to observe different lithology and geological structure, to identify surface geodynamic processes, and drawing it on a base map by freehand sketching.

Geological Mapping

The geological map of the study area was produced at the scale of 1: 20000 and was made to incorporate lithological units and geological structures. Before we start mapping, information that was collected and prepared during desk studies such as a preliminary geological map, drainage map, and existing regional geological information of the study area were employed for planning and undertaking of the field geological mapping. Subsequently, this evidence were integrated with information obtained from a field survey like differentiation of major orientation of lithological units and structures of the study area to decide the best traverse line to conduct this mapping. Upon selection of the traverse line, detailed geological mapping was carried out on the base map that was prepared during the desk study. Descriptions of geological materials were carried out in the field in terms of color, orientation, degree of weathering, and thickness. The degree of weathering was determined by considering the response of rocks to blow by geological hammer following the guidelines forwarded by ISRM (1981). Besides, capturing of illustrative photographs was also taken. Finally, the result obtained from the fieldwork was converted into a geological map of the study area using ArcGIS software.

4.2.2 Field survey and data collection

The field investigation was done according to the International Association of Engineering Geology (IAEG), ISRM (International Society of Rock Mechanics), and methods for describing rocks and soils. Soil parameters were described according to the American Society for Testing Materials (ASTM). Visual classification and test pit loggings were identified by using different traverses along with rock and soil exposures of road cuts, quarry sites, rivers/streams, gullies, trenches, and foundation work sites. Rock lithological units were identified with help of hand lenses and based on their color, texture, parent material and based on information obtained from regional geology. Geological structures were identified in the field survey by measuring their orientation by using a compass. Assessment of geomorphological conditions and the geodynamic problem was done during fieldwork.

4.2.2.1 Measurements and recording

GPS readings were taken to locate the coordinates of the sampling area, rock exposure area, surface geodynamic area, and any other geological formation.

Discontinuity Survey

A discontinuity survey has been carried out to provide parameters for rock mass classification. This survey was mainly carried out along stream/river cut and road cut area, especially north-central and western part of the study area. Accordingly, the orientations of discontinuities and their angle were measured with the aid of the Brunton compass. The spacing of discontinuities was determined by measuring the perpendicular distance between two joints for joint sets at a different location and taking the average value. The aperture and persistence of discontinuity sets were also measured. Similarly, other conditions of discontinuities such as surface roughness, infill material, wall weathering, and associated groundwater condition were visually estimated in the field.

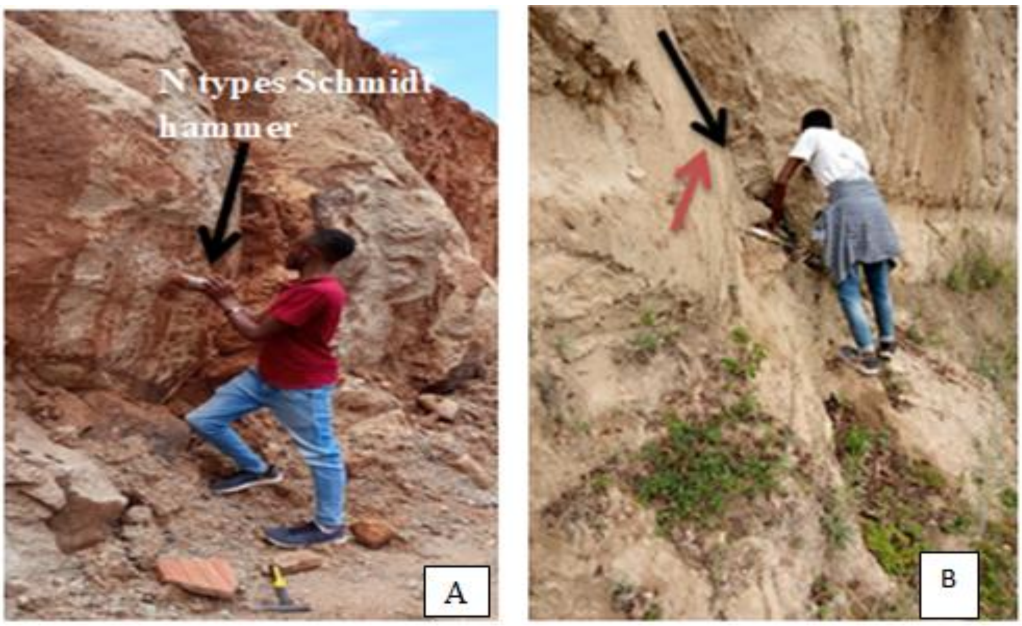


Figure 17: Conducting Schmidt hammer on ignimbrite rock in the study area (A) at GPS reading (long. 430552m lati.745141 m). (B) Measuring orientation of dip for joints by using a compass at GPS reading (long. 437833 m lati.745374 m).

4.2.2.2 Samples and sampling techniques

Nineteen representative disturbed soil samples were collected from 14 test pits up to a maximum of 3.6m depth for laboratory tests to determine their index and engineering properties (figure 18). Two soil samples from one test pit at different depths were collected when lithology changes were observed. The test pit was excavated manually using Shovels and diggers then placed in a plastic bag to preserve its moisture content (figure 19). Finally collected soil sample were transported to Hawassa University Institute of Technology to conduct a laboratory test. Representative rock specimens were collected from different lithologies in the study area to measure dry density and unit weight of the rock to estimate rock strength. Location of soil sample test pits and Schmidt hammer tests are presented in (Appendix A).

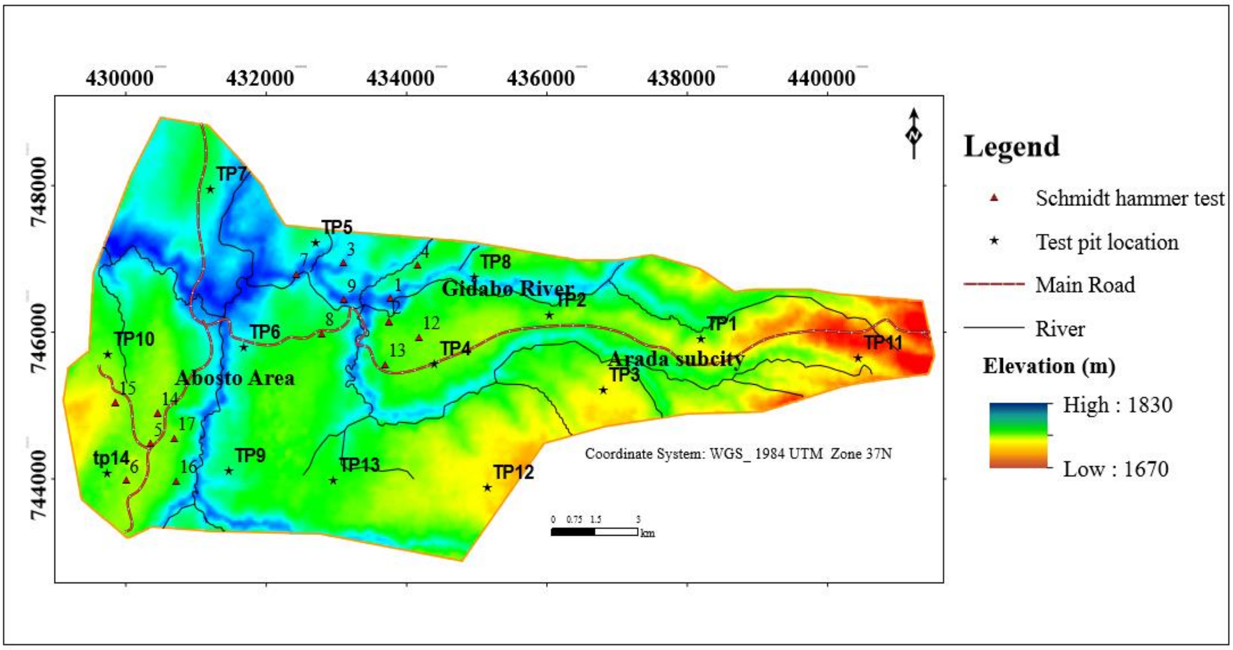


Figure 18: Test pit location and Schmidt hammer test of the study area

Figure 19: Soil sampling A) Test pit excavation at GPS reading (long. 436038 m lati. 436038). B, from foundation excavation at YAIP, GPS reading (long. 429734 m, lati.745720 m)

4.2.2.3 Field test

Descriptions of geological materials were carried out in the field in terms of color, orientation, degree of weathering, thickness, and space condition.

In-situ Strength Test

Intact rock strength was estimated in the field using the Schmidt hammer rebound test (figure 17). For this study N-type concrete Schmidt hammer has been employed using the guidelines of ASTM C805 (2018) as per the standard suggested by the manufacturer. Accordingly, the following procedures were followed as per ASTM C805 (2018) for the collection of rebound values. Test surfaces was rubbed and smoothed using grindstone, the setting of the instrument such as averaging mode was adjusted to meanwhile the conversion curve and unit of measurement were adjusted to 10-percentile curve and MPa respectively then the instrument was held perpendicular to the test surface and gradually pushed until the hammer impacts the test surface after that corresponding rebound value was then, recorded. In the proposed area, ten readings were taken at each test section with no two impacts tests closer together than 25mm, if hammer impact crushes or breaks the test surface, the reading was discarded and then, repeated at other locations. Finally, a representative rebound value was obtained by discarding the rebound values that differ from the average of 10 readings by more than 7 units and then by determining the average of values of the remaining readings (ASTMC805 (2018). In case more than two readings differ from the average of 10 readings by more than 7 units, the entire reading was discarded and the rebound values were determined at other nearby locations. Rock strength tests were also carried out by using a Geological hammer; i.e. simple methods of rock strength determination. According to (ISRM, 1981), uniaxial compressive strength was estimated based on the geological hammer. Schmidt hammer rebound test values of rocks was correlate with uniaxial compressive strength(UCS) using Wang et al., (2017) and Barton and Choubey(1977) correlation equations.

4.3 Laboratory tests of soil

Laboratory tests of soil such as natural moisture content, specific gravity, particle size distribution, atterberg limit, pH, free swell test, shear strength and compaction test were carried out according to ASTM standard.

4.3.1 Natural moisture content

A moisture content (water content) test was performed in the laboratory for the soils of the study area as its correlation is important to know the shear strength and compaction characteristics. The physical properties of most fines grained and particularly clayey soils are greatly affected by the water content. This test is conducted according to ASTM D 2216 (1998) standard in Hawassa University Institute of Technology. The wet mass (Mw) of soil samples was weighed using digital balance and then, oven-dried for 20 hours at 105^{0}C to obtain dry mass (Md). The natural moisture content (w) was then determined using the following equation.

$$W = \frac{Mw - Md}{Md}\,100\% \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{Equation 3}$$

4.3.2 Specific gravity

This test was conducted according to ASTM D 854-92 (1998) weight of empty, dry, and clean pycnometer (Wp) was determined and recorded. 25gm of oven-dry soil which passes through sieve no.10 is added to the pycnometer and weight measured (Wo). Distilled water was added and fills up to half or three fourth and soaked for 10 minutes manually. By applying vacuum entrapped air was removed slowly from the pycnometer. Then, the distilled water was slowly added to the level of calibration mark of the pycnometer and the corresponding mass of the pycnometer filled with soil and water was measured (wb). Finally, the pycnometer was emptied and cleaned after which it was filled with distilled water to the level of calibration mark to obtain a mass of pycnometer filled with water (wa). The specific gravity (SG) at the temperature was determined using the following equation:

$$SG= \frac{Wo}{(Wo+Wa)-wb} \quad \text{.......Equation 4}$$

4.3.3 Grain size analysis

To determine the particle size distribution of the soils, mechanical or sieve and hydrometer analyses were conducted based on ASTM D 422 (1998). The mechanical analysis was used for particle size larger than 0.075mm diameter (No. 200 Sieve) while hydrometer analysis was used for particle size smaller than 0.075mm diameter (No. 200 Sieve). Grain size analysis was used to determine the textural classification of soils (i.e., gravel, sand, silt, and clay). For the soils of the study area sieve analysis(mechanical) (dry and wet sieve analysis) were performed and also hydrometric analyses were conducted as the fine fraction content for most of the soils under study is very high. For the dry sieve, the soils were subjected to oven-dry, then by taking 500gm soil tests were performed by using the following series of sieves, sieve No 4 (4.75mm), No 8 (2.36mm), No 10 (2mm), No 16 (1.18mm), No 30 (0.60mm), No 40 (0.425mm), No 50(0.3mm), No 80(0.18mm), No 100 (0.15mm) and No 200 (0.075mm) the soil samples were sieved by mechanical means. The testing procedures mentioned above were applied to three (TP5-1, TP5-2, TP6-2, and TP8) samples for grain size analysis. The remaining soil sample was conducted by using the wet sieving method. This test was performed by taking 1000gm soil and washed on a sieve 0.075mm, the finer was passed through it during washing and coarse-grained soil remains on it and oven-dry that remaining one for mechanical shaking by using similar series of sieve used for the dry method. All analysis was determined based on ASTM (D 422- 90, 1998) procedure.

The diameter or size range is adopted from ASTM as follows: >4.75 mm (No.4) gravel; 4.75 – 0.075 mm (No.200) sand (coarse to fine sand); <0.075mm is fine fraction (silt and clay). Hydrometer analyses were performed for the fine grain (<0.075 mm) soils. 65 gm of dry soil sample which passes through sieve diameter 0.075mm was taken and soaked in water for 1 to 3 hrs. 25gm dispersing agent (sodium hexametaphosphate) was added and 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 ASTM 151H standard hydrometer reading stem was used to read settlement time of the sample in 24 hours.

4.3.4 Atterberg limit

The Atterberg limit test was conducted by using CassaGrande apparatus ASTM D (4318), 1998 standard test method for liquid limit, plastic limit, and plasticity index of soils. like liquid limit device, Porcelain (evaporating) dish, Flat grooving tool with gage, moisture cans, Balance, Glass plate, Spatula, Wash bottle filled with distilled water, and Drying oven set at 105^0c was used for this study.

4.3.4.1 Liquid limit(LL)

To conduct this test ¾ of soil samples, which passed through a No.40 sieve, were taken and mixed with a small amount of distilled water until it appears as a smooth uniform paste. Then the weight of the empty moisture can be recorded with the can number on the datasheet. The portion of the previously mixed soil was placed into the cup of the liquid limit apparatus at the point where the cup rests on the base. Using the grooving tool carefully cut a clean straight groove down the center of the cup. Moreover, the base of the apparatus below the cup and the underside of the cup were clean of soil. The crank of the apparatus was turned at the rate of approximately two drops per second and counts the number of drops N. It takes to make the two halves of the soil pat come into contact at the bottom of the groove along with a distance of 13 mm (1/2 in). Then the sample was taken from the edge to the edge of the soil pat, using the spatula and it was placed in the moisture can and immediately the weight of the moisture can containing the soil was recorded and placed in the oven for 20 hours. Then the entire soil specimen in the porcelain dish was remixed and a small amount of distilled water was added to increase the water content so that the number of drops required to close the groove decrease. Finally, the above steps were repeated for two additional trials producing successively lower numbers of drops to close the groove.

4.3.4.2 Plastic limit (PL)

To conduct this test, first, the weight of empty moisture cans with their lids and the number of cans was recorded on the datasheet parts of soil specimens, that were used for the liquid limit were rolled by using fingers on the glass plate with sufficient pressure. The thread was broken into several pieces when the diameter of the tread reaches the correct diameter. The thread was deformed when its diameter reaches 3mm. The rolling was continued until the thread crumbles. The portions of the crumbled threads were gathered together, placed in the moisture can and the weight of the moisture can containing of the soil was recorded and placed in the oven for 20 hours. The above step was repeated two times and the water content was determined from each trial by using the same method. The plastic limit was determined by determining the average water content.

The plasticity index was finally calculated by using both liquid limit and plastic limit data, as follows.

$$PI = LL - PL \quad \text{Equation 5}$$

4.3.5 Free swell test

Initially, this method is conducted by adding 10gm dry soil but, the weight is based on personal judgment and graduated cylinder available (Holtz and Gibbs, 1956) stated in (IS: 2720, 1977). This test is the simplest but very important test; during lab work, it was conducted carefully to minimize errors according to the standard method of practice specified by the Bureau of Indian Standards (IS: 2720, 1977). 20gm of representative soil samples which pass through sieve no 4 were added to two 100ml graduated cylinders separately. Distilled water and kerosene were added to each cylinder separately. This is due to the soil which natural gas added does not show any change because it is non-polar. After 24 hours observing the change of soil which distilled water is added relative to the cylinder which contains kerosene.

$$FS = \frac{\text{volume chnge in distlled water} - \text{volume in kerosene containing cylinder}}{\text{volume in kerosene cylinder}} 100\% \quad \text{Equation 6}$$

4.3.6 p^H test

This test is conducted by taking 25 gm. of soil sample which pass through sieve no 4 and placed it in the beaker and distilled water was added after that soaked for 1 to 3 hours and p^H measuring instrument was inserted and finally reading was taken.

4.3.7 Shear strength test

Direct shear strength test was conducted as per guidelines of ASTM D3080 (1998) on soil specimens of 60 × 60 × 25 mm dimension, which were compacted and remolded at their optimum moisture content was determined first by preparing the soil which has been tested and test was also performed at their natural moisture content for some soil sample. Carefully assemble the shear box and place it in the direct shear device. After completing the assembly of the direct shear device, initialize the three gauges (horizontal displacement gauge vertical displacement gauge, and shear load gauge) to zero. Set the vertical load (or pressure) to a predetermined value, and then close the bleeder valve and apply the load to the soil specimen by raising the toggle switch. The specimens were then sheared at initial consolidation pressure of 100 KPa by applying horizontal shear load at a rate slow enough to allow pore water pressure to dissipate till the specimen has fallen. The horizontal and vertical shear displacements were then recorded at regular intervals. The test was then repeated on the same specimen at 200 and 300 KPa. From the test result, shear stress and normal stress were calculated and plotted from which shear strength parameters were obtained using Mohr-Coulomb Failure criteria.

4.3.8 Compaction test

It is performed to determine the relationship between the moisture content and the dry density of soil for a specified compaction effort. It is performed by using standard proctor type compaction according to ASTM D 698 standard. Molds, Manual rammer, Extruder, Balance, Drying oven, Mixing pan, Trowel, #4 sieve, Moisture cans, Graduated cylinder, straight edge is the equipment used during laboratory work. Soil which passed sieve #4 was prepared, the weight of compaction mold with its base by removing collar were measured and recorded, and the soil is mixed by adding a small amount of water until it becomes uniform color. The mixed soil is divided into three equal parts and then added to the mold and compacted manually by using 25 drops of rammer in 1.5 seconds per drop. The second and third layers were also added and compacted in a similar way. After soil filling the mold and collar, the collar was removed carefully and weight of soil sample with mold was measured, representative sample was taken to determine water content. Compacted soil is removed and more water was added and remixed then above procedure is repeated until to get peak compacted soil mass value and lesser value. OMC and MDD were obtained by drawing of dry density vs. moisture content graph. Water content, bulk, and dry density were calculated as,

$$\rho = \frac{\textbf{mass of compacted soil}}{\textbf{volume of mold}} \text{..Equation 7}$$

the volume of mold is $944cm^3$

$$\rho d = \frac{\rho}{1+w} \text{………………………………………………………….…Equation 8}$$

where ρ, ρd, and w are dry density, bulk density and water content respectively

4.4 Data analysis and interpretation approaches

Data analysis and interpretation were based on the information obtained from a laboratory test, field visualization of rock and soil, in situ testing of rock mass strength, weathering, and discontinuity.

4.4.1 Soil classification based on USCS

According to USCS, soils are divided into two major groups. These are coarse-grained and fine-grained soils, using No. 200 sieves as the size criterion. The soils with more than 50% of the sample are larger than the No. 200 sieves. It is categorized as coarse-grained and is further subdivided by sieving and gradation. In addition, the soils with more than 50% of the sample are finer than the No.200 sieves categorized as fine-grained and further subdivided based on liquid limit values and plasticity index. In this classification system, paired letter symbols are used for each soil group. The first symbol refers to, the predominant particle size, and the second symbol for fine-grained soils subdivides based on Low (L) or high (H) on plasticity chart (Das, 2012).

4.4.2 Soil classification based on the IAEG (1981) classification system

In this classification system, particle distribution curve plotted on grading curve used to classify coarse graded soil as gravel (G), and sand (S) based on more than 50% of soil passed/retained on sieve number 2mm and uniformity are also considered for classification. The percentage of finer than 0.06mm is 0 to 5% classified as poor graded, well-graded, and uniformly graded gravel or sand. Percentage of finer than 0.06mm is 5 to 35% classified as GRAVEL, the silt of low plasticity and GRAVEL, clay of low plasticity for gravel and SAND, the silt of low plasticity and SAND, clay of low plasticity for sand. Fine soil is classified based on a plasticity chart, silt (M) plotted blow A-line and clay (C) plotted above A-line.

4.4.3 Plasticity analysis based on Casagrande chart

Plasticity is the property of cohesive (fine-grained) soils that possess the ability to change shape without cracking or fracturing. According to Casagrande (1932), the consistency limits of several soils and proposed the plasticity chart which is divided into six regions, three above and three below A-line as described and shown below on (Table 8). An A-line separates the inorganic clays from the inorganic silts. Inorganic clay values lie above the A-line, and values for inorganic silts lie below the A-line. The information provided in the plasticity chart is of great value and is the basis for the classification of fine-grained soils in the Unified Soil Classification System. The chart comprises of empirical A-line given by equation PI=0.73(LL-20). The U-line is approximately the upper limit of the relationship of the plasticity index to the liquid limit of soils.

Table 7: Plasticity chart classification of soil Casagrande (1932) for USCS classification system.

Above A-line	Liquid limit	Soil classification
	<30	Inorganic clay of low plasticity and cohesion soil
	30<LL<50	Inorganic clay of medium plasticity
	>50	Inorganic clay of high plasticity
Blow A-line	<30	Inorganic silt of low compressibility
	30<LL<50	Inorganic silt of medium compressibility and organic silt
	>50	Inorganic silt of high compressibility and organic clay

Table 8: Description of plasticity of fine soil for IAEG (1981) classification system

Liquid limit (%)	Term
<35	Low plasticity
35-50	Intermediate plasticity
50-70	High plasticity
70-90	Very high plasticity
>90	Extremely high plasticity

4.4.4 Potential soil expansiveness based on free swell and activity of soils

Identification and classification of expansive soils are either on direct measurement of free swell value or the correlation of simpler test results with swell potential measurements. Based on activity potential, expansiveness of soil is classified as follows (Table 9).

Table 9: Classification of activity (skempton, 1953)

Activity	Nature of soil	Potential expansiveness
<0.75	Inactive	Low
0.75- 1.25	Normal	Medium
>1.25	Active	High

Holtz and Gibbs (1956) had not advised any specific values of free swell to classify the soils. However, they suggested a pictorial relationship between free swell value and the volume change of soil specimens. Based on the free swelling test potential expansiveness of soil is correlated as follows (Table 10).

Table 10: Potential expansiveness of soil based on free swell test (BIS IS 1498: 1970(R2002)).

FSV	Potential expansiveness
<50	Low
50- 100	Medium
100-200	High

4.4.5 Laboratory Testing of Rocks

For this study, representative rock samples were collected from exposed exposure in the study area to conduct a unit weight of rock mass using the buoyancy technique proposed by (ISRM, 1978).

4.4.5.1 Unit Weight Test

The unit weight of the rock is an important parameter in rock characterization. For this study, the buoyancy technique was adopted to determine dry density and unit weight from irregular rock samples according to the standard recommended by (ISRM, 1978) at Hawassa University Institute of Technology Road and Highway laboratory. The test was conducted by weighing the representative rock sample at its original moisture content (M) and its dry weight (Md) was determined after oven drying at 105^0C for 24 hours. Oven-dried rock specimens were then submersed into a beaker which has its reading and filled with a known volume of water (V1), after immersing of the rock sample corresponding volume was noted as (V2). Finally, the corresponding dry density (ρd) and unit weight (γ) of rock were determined using (Eq9 and 10) respectively.

$$\rho d = \frac{Md}{V2-V1} \quad \text{Equation 9}$$

$$\gamma = \rho d * g \quad \text{Equation 10}$$

Where g is the acceleration due to gravity

4.4.6 Rock mass classification based on Strength, RQD, RMR.

The rocks of the study area were classified according to the International Society of Rock Mechanics (ISRM, 1981) based on discontinuity characteristics, Schmidt hammer test value, and UCS. Its strength was estimated by simple geological hammer value according to ISRM 1981. The Schmidt hammer was conducted according to ASTM C805 standard and later was converted into UCS value by using the correlation formula proposed by (Wang et al. 2017) (eq.1), Barton and Choubey (1977) (eq.2).

4.4.7 RS and GIS application in Engineering Geological map preparation

Remote sensing technology plays an important role today in the geological survey, mapping, analysis, and interpretation which provides a unique opportunity to investigate the geological characteristics of the remote areas of the earth's surface without the need to gain access to an area on the ground (Hakim et al., 2018). The Remote sensing and GIS software are used for processing and mapping the data during the analysis. Arc Map is used mainly for the GIS interpretation (e.g. the statistical analyses of lineaments). Interpretation of images from satellites is based on the spectral reflectance of earth materials on the surface. However, such reflectance depends on the electromagnetic spectrum in which the satellite collects data.

In the study area, engineering geological map preparation was based on remote sensing and geographical information system. Satellite collected data were obtained from US Geological Survey (USGS), google earth pro, and processed using Arc GIS 10.3.1 version. From those data elevation, topography, the boundary of the study area, geological structure, and surface water information was obtained. Topographic map, location points which were measured using GPS, field sampling of attributes and remote sensing images are the main source of GIS data. Only satellite data is not enough for visual interpretation and preparation of engineering geological maps. Therefore, it needs additional preliminary aids like ground truth information, topographic map, and related literature. All necessary field data (lithology, any contact, faults joint orientation, and discontinuity, soil test pit location, Schmidt hammer tests location, any surface geodynamic phenomena, and groundwater borehole) were collected and recorded with help of GPS, and the topographic map. Field and laboratory tests were conducted which play a great role in engineering geological mapping. A simple lithological map is produced by freehand sketch, and all necessary information is documented with its location. In addition to this interpretation start based on pieces of knowledge and experience on ArcGIS software by using the above-mentioned information. Arc GIS is the main software for GIS interpretation. Digitization on ArcGIS software was based on the boundary of the study area and the topographic map, and all field information is documented with its actual field location (easting, northing, and elevation). Borehole location, test pit, and Schmidt hammer location were presented using the point feature. Lineaments, faults, rivers, problematic soil, rock falling prone area, gully, and other erosion were digitized by using line features. Lithology units were differentiated by using the polygon feature. With help of RS and GIS application engineering geological maps were produced in the study area. Physiographic map, drainage map, and slope map of the study area were created by using a Satellite image and 30m resolution DEM (digital elevation model) which were mainly obtained from Google Earth and SRTM (Shuttle Radar Topography Mission).

Table 11: Secondary data and its source

Types of Secondary data	**Sources**
Rainfall and temperature data	Ethiopian Meteorological Agency, Hawassa branch
Geotechnical data	Ethiopian Mapping Agency
Hydrogeology data	Governmental institution and private company
Topographic base map	Ethiopian Institute of Geological Survey
Groundwater data	From individual person

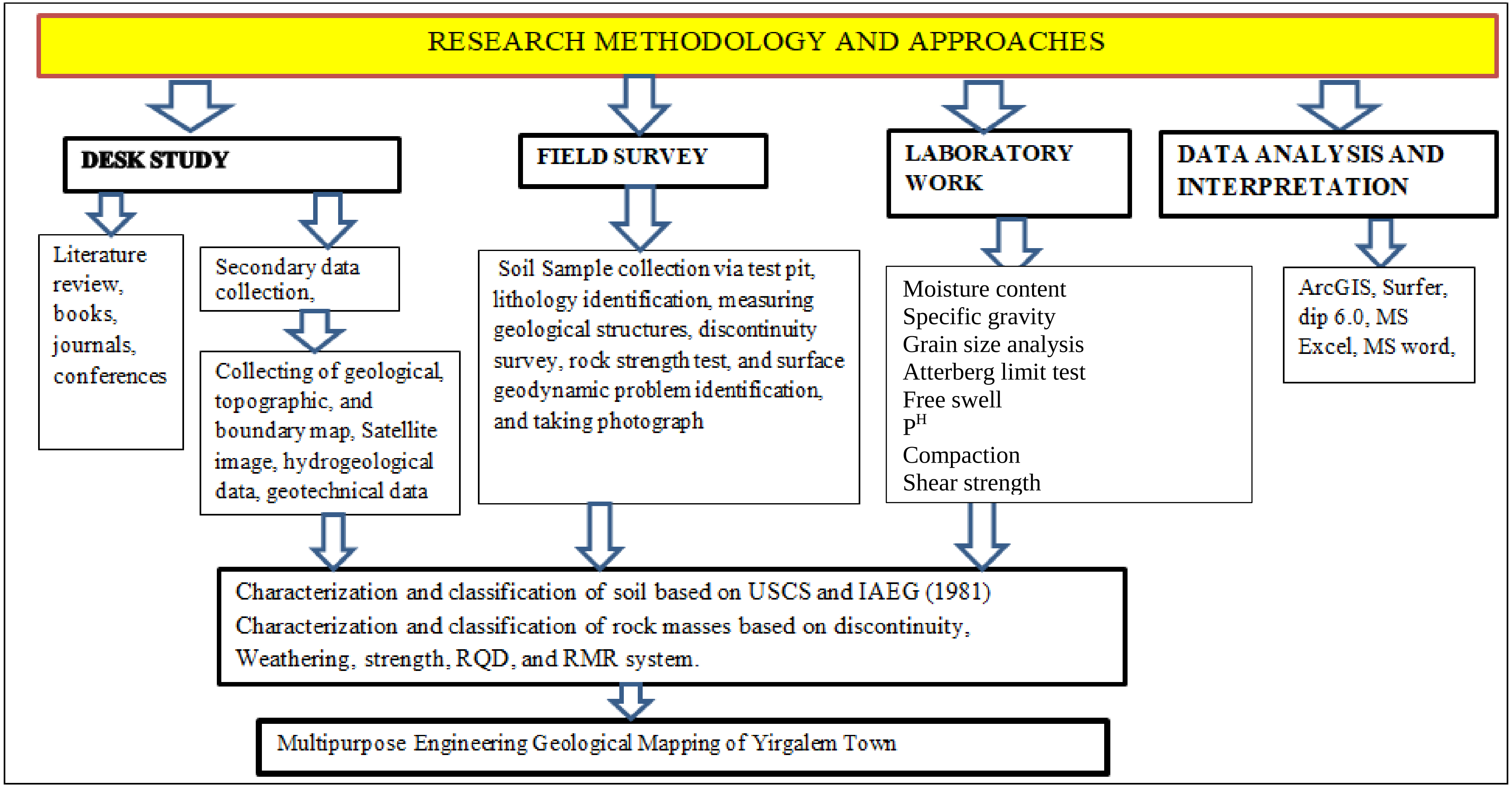


Figure 20: Flow chart showing the general methodological outline of research

Materials

Materials are extremely important in research work to solving certain problems as they are a means for data collection in the field, conducting in-situ tests, recording of data, obtaining representative samples, and others (Table 11). Hence, the materials that are intended to be used for this study range from equipment that is used for obtaining and collecting samples and data in the field to those that are used to carry out laboratory tests on these samples. It also includes software that is used for processing the raw data obtained from the fieldwork. Brief classifications of these materials are shown below (figure 21).

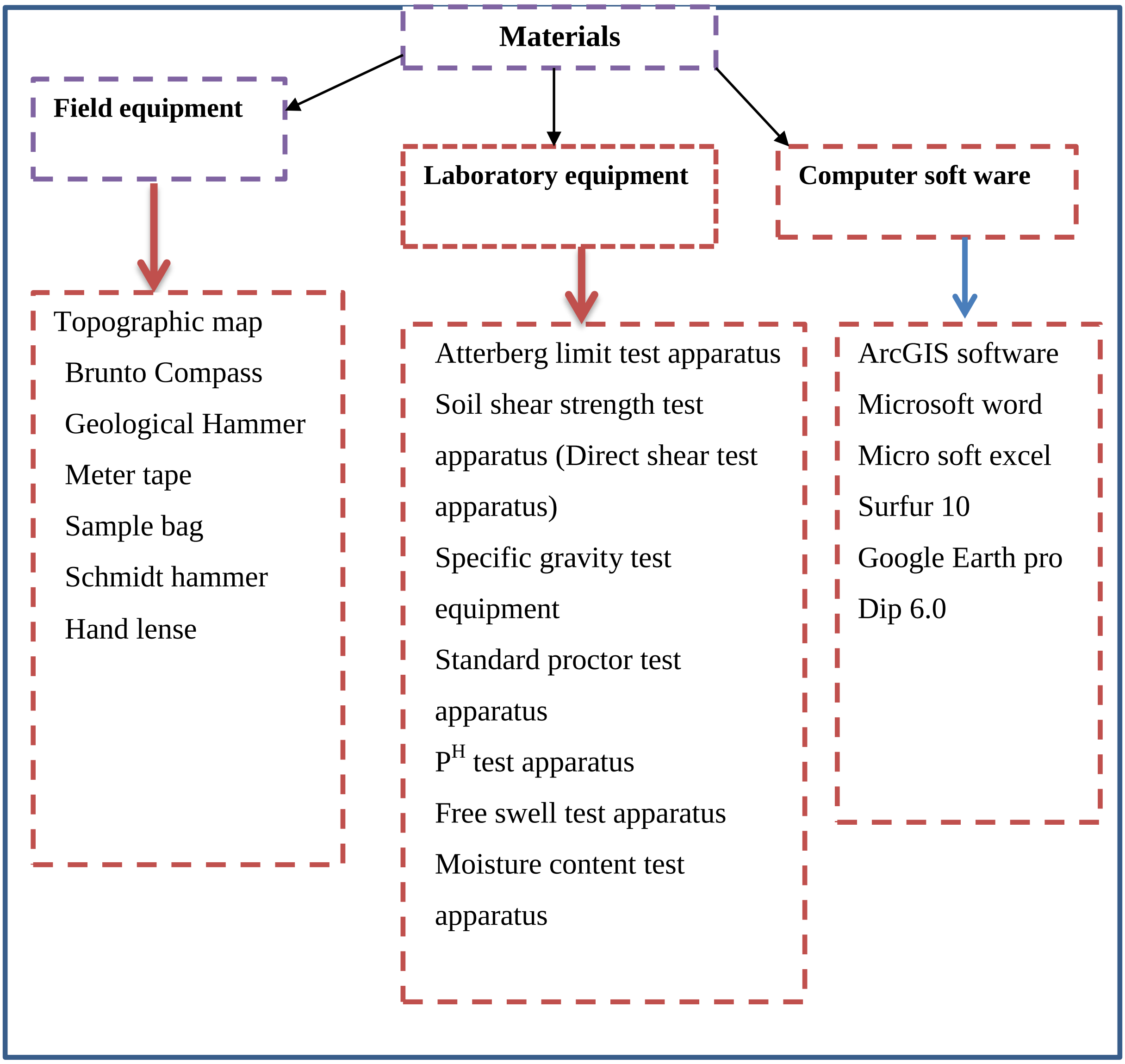


Figure 21: Classifications of materials used to carry out research

CHAPTER FIVE

RESULT AND DISCUSSION

5.1 Textural description of the soils

Most of the existing civil engineering works of the town are founded on residual soil and volcanic ashes with pumice. From the field visual description of foundation excavation sites, test pits, and stream or river cut/gullies, a large part of the town is covered by overburdened soil which has a thickness of up to 12 meter. The texture of this soil varies from fines (clay/silt) size to rock fragments. In study area, along streams/gorges and road cut sites, alluvial soil deposits are typically stratified (Figure 22). The thickness of the strata varies from centimeter to 1m. From the field observation of foundation excavation sites, the foundations are located on the fines grained clay soils.

Figure 22: (A) Residual soil deposit from foundation excavation at Hida Kalit Yirgalem agro-industrial Park at GPS reading (long. 429902 m lati.746514 m). (B) Stratified alluvial deposit with rock fragments from the road cut at Arada sub city at GPS reading (long. 434563 m, lati. 744612 m).

5.2 Soil mass characterization and classification

In this part, the result of both in situ and laboratory analyses of soils samples collected from fourteen test pits of 1.5 up to 3.6 m of depth were investigated and classified accordingly.

5.2.1 Soil mass characterization

Based on the scope and the objective of this research, more emphasis was given to the field description and laboratory soil tests from fourteen selected test pits which greatly help in identifying and classifying the soils. In this research, natural moisture content, grain size analysis, atternberg limit, free swelling, shear strength, and compaction tests are laboratory tests which were carried out at Hawassa University, Institute of Technology soil mechanics laboratory.

5.2.1.1 Natural Moisture Content (NMC)

The laboratory result of the soil sample in the study area portrayed in (Table 12) shows that the natural moisture content of the soils of the study area ranges from 10.05 - 49.5. Residual fine-grained soil does not infiltrate water and has the capacity to hold it. This leads to the main cause for shrinking - swelling properties based on climate change in the study area (Berhane, 2010; Getahun et al. 2013).

5.2.1.2 Specific Gravity

In this study the specific gravity was conducted for all soil samples in (appendix E). The laboratory result shows that the specific gravity of the soils of the study area ranges from 2.42 – 2.69 (Table 12). Most of the soils of the study area have high specific gravity. This is due to most of the soils in the study area are fine.

Table 12: Index property laboratory test result of Yirgalem town at a time of investigation

Sample Id	Sample location		Depth (m)	NMC	SG	Grain size analysis				Cu	Cc	Atterberg limit			Dominant lithology
	Easting	Northing				%G	%S	%M	%C			LL	PL	PI	
TP1	438194	745876	1.7	38.15	2.6	0.2	16.9	56.1	26.97			63.5	34.5	29	Silt
			3.6	34.85	2.6	0.1	15.1	48.55	33.95			68.38	44.6	23.77	Silt
TP2	436038	746229	1.5	28.8	2.57	0	10.6	71.82	18.18			58.2	37.77	20.43	Silt
			2.7	26.1	2.58	0.3	9.5	62.9	20.3			60	31.8	28.2	Silt
Tp3	436792	745197	2.3	32.5	2.57	0.1	6.1	54.75	37.67			64.54	41.3	23.23	Silt
Tp4	434395	745570	1.8	31.45	2.65	0.2	22.4	47.7	28.61			59.42	34	25.4	Silt
			3.2	30.8	2.69	2.2	20.2	51.66	24.83			68	37.8	30.2	Silt
Tp6-1	431669	745806	2	29.65	2.66	0	25.7	51.66	20.14			70	39.1	30.9	Silt
Tp7	431195	747978	3	34.85	2.68	1.6	23.2	60.2	16.62			67.7	43.9	23.8	Silt
Tp9	431449	744104	3.2	37.65	2.68	0	13.3	74	12.7			51	32.75	18.25	Silt
Tp10	429734	745720	3	49.5	2.66	0	30.8	48.8	20.4			59	32.04	26.96	Silt

Tp14	429742	744069	3.6	49.15	2.68	0	10.11	70	19.89			63.2	34.54	29.46	Silt
Tp5	432725	739745	1.6	19.6	2.45	0.4	98.2	1.4	0	4.09	0.04	49.5	32.8	16.7	sand
			3.4	10.05	2.42	0.4	99.4	0.2	0	5	1.8	48.8	34.6	14.2	Sand
Tp6-2	431669	745806	3.5	34.1	2.66	5.7	58.5	35.8	0			49.2	30.5	18.7	Sand
Tp8	434957	746724	3.1	42.8	2.65	0	91.8	8.2	0	4	0.006	49	35.36	13.64	Sand
Tp11	440434	745664	2.7	28.7	2.67	0	9.7	43.58	46.72			63	26.1	36.9	clay
Tp12	435121	743861	2.4	38.2	2.56	0	11.2	41.79	47.1			58.2	21.47	36.73	clay
Tp13	432941	743988	3	37.34	2.65	0	12.1	40.25	47.65			54.7	19.13	35.57	clay
YAIP	-	-	-			0.3	18	81.7				57	34.4	26.6	silt

NMC = natural moisture content LL= liquid limit PL = plastic limit PI = plastic index SG= specific gravity Cu = coefficient of uniformity Cc=coefficient of curvature TP = test pit G= gravel S= sand M = silt C= clay. YAIP = yirgalem agro industrial park

5.2.1.3 Grain size analysis

To determine the particle size distribution of the soils, mechanical or sieve and hydrometer analyses were conducted (Appendix D). In the study area, soils were categorized into four classes of soil sizes (Figure 23). Ranges of gravel vary 0 – 5.7%, ranges of sand vary 6.1 – 99.4%, range of silt soil varies 0.2 – 87.42%, and range of clay soil varies in 0 – 47.65%, so fine soils are dominant in the study area as portrayed in (Table12).

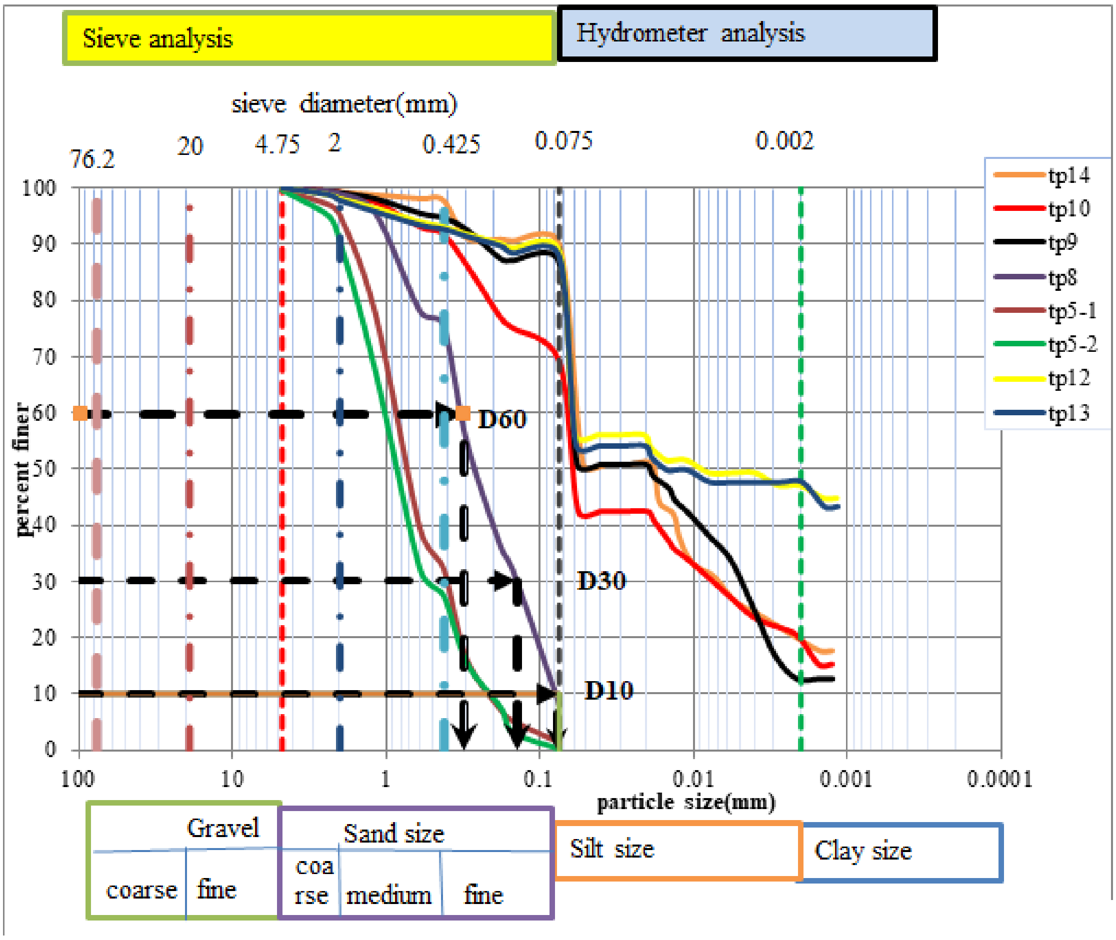


Figure 23: Grain size analysis of soil samples in the study area

5.2.1.4 Atterberg limit

The Atterberg limits of soil samples taken from the study area were determined in the laboratory following Cassagrande's method (appendix C). From the results of liquid limit (LL) and plastic limit (PL), the plasticity index (PI) of the soils in the study area was determined which is an important parameter in classifying fine-grained soils. According to Bowles (1992) as cited in Berhane (2010) if the plasticity increases, the engineering problems associated with using the soil as an engineering material, such as foundation support for residential building and road subgrades become serious. The Consistency limits or Atterberg Limits of the soils of the study area are summarized in (Table 12). From this result, we can observe that the liquid limit of the soil of the study area ranges from 48.8 – 70%, the plastic limit ranges from 19.13 – 44.6%, and the plastic index ranges from 13.64 – 36.9%. Soil with plasticity index value PI >35, 20≤PI≤35, 10≤PI≤20, PI<10 are very high, high, medium, and low plastic soils respectively (Seed et al., 1962). Based on this the plasticity classes the soils in the study area falls from medium plasticity to very high plasticity ranges. According to the range of plasticity index about 20% of the total soil sample falls in medium plasticity, about 65% of the total soil sample falls in high plasticity, and 15% fall in very high plasticity. According to the range of liquid limit (Bell, 2007), 20% fall in intermediate plasticity (LL: 35-50%) and 80% fall in high plasticity (LL: >50%). In general, based on the liquid limit values of the soils most (greater than 50%) of the soil of the study area falls in high plasticity. This indicates soils of the study area are poor for engineering work.

Liquidity Index

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. Negative values of the liquidity index indicate water content smaller than the plastic limit (Arora, 2003). In the study area, liquidity value ranges from -1.09 to 0.65 (Table 13) and ten (10TPs) have negative liquidity index value and remaining nine (9TP) sample test demonstrates positive values for liquidity index. In the study area, 52.63% of soils have a liquidity index less than zero. This indicates the soils of the study area are semi-solid. 15.79% stiff and 26.32% is medium stiff, and 5.26% are soft as classified Arora (2003).

Consistency Index

The consistency index indicates the firmness of the soil. It shows the nearness of the water content of the soil to its plastic limit. The soil at a water content equal to the plastic limit has a consistency index of 100%, which shows that the soil is relatively firm (Arora, 2003). In the study area consistency index ranges from 0.35 to 2.09. 52.63% of soil samples in the study area are relatively strong (Table 13). The sum of liquidity index and consistency index result of all soil samples is 100%, indicating that soil in the study area having a low liquidity index has high consistency index and vice-versa. The negative value for the consistency index was not determined in the study area, indicating that the water content of the soils in the study area is not greater than the liquid limit.

5.2.1.5 Activity of soil in the study area

Skempton (1953) has classified clay soil as inactive, normal, and active based on its activity and mineral content as shown below in the activity chart. The type of clay minerals and their proportional amounts is highly correlated with activity. Activity is used as an index for identifying the swelling potential of clay soils. The activity of clay soils is expressed.

$$A = \frac{\textbf{Plasticity index (PI)}}{\textit{percent finer than 2micron}} \quad \text{.. Equation 11}$$

The activity values of soils of the study area were plotted on the activity chart (figure 24). The activity value ranges from 0.62 to 1.53 (Table 13). The higher the activity value the higher the shrink- swell potential. From these values, soils test pits (Tp6-1, TP7, TP9, TP10, and TP14) lie above an activity line of 1.25 which are active soils that have high water holding capacity and expansive nature. Test pits (TP1 at 1.7 m depth, TP2 at 1.5 m depth, and TP4 at 1.8 m depth, TP11, TP12, TP13 lie in between the activity line of 1.25 and 0.75 which are normal soils, and test pits TP1-2 and TP3 are below 0.75 which is inactive soils.

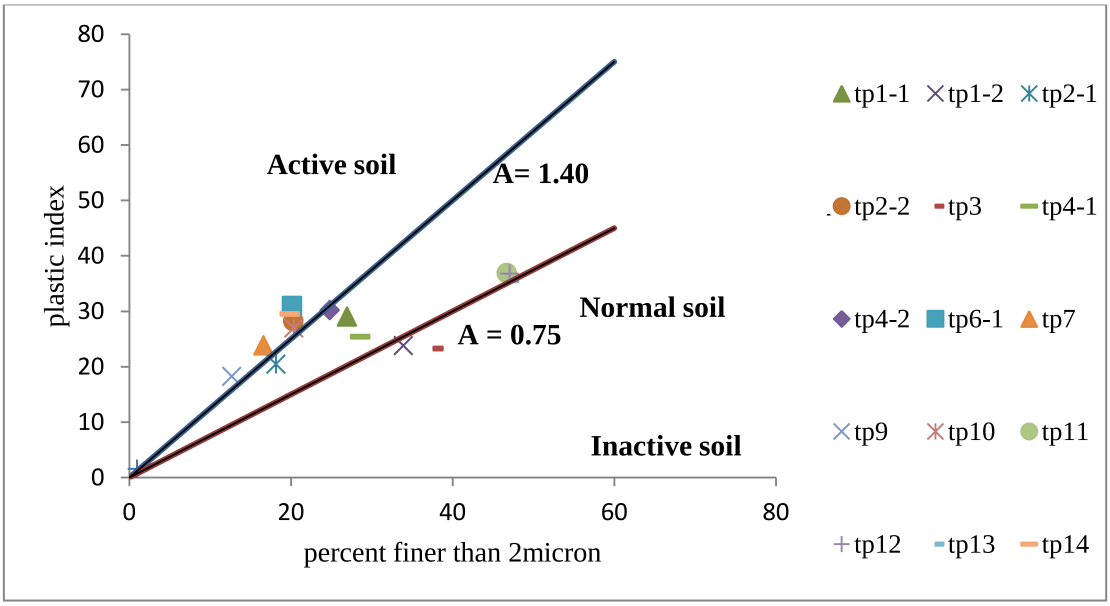


Figure 24: Plot of activities of soils of the study area on the activity chart.

5.2.1.6 Free swell test

Conducting free swell tests is very essential to indicate the presence of swelling clay among the soil in the study area. The potential expansiveness of soil samples is also determined by using the result of free swell tests. In the current study, free swell tests were conducted on fourteen (13) soil samples of the study area and the free swell test results are found in the range of 13% - 104% (Table 13). According to Holtz and Gibbs (1956); Berhane, (2010); Getahun et al., (2013) 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. Therefore, considerable attention should be given to soils that have high free swell. The free swelling value increases with the plasticity index. According to the result obtained from laboratory tests in the study area except for TP5, TP6-2, TP8, TP1-2, and TP3 which has low free swell value. Another soil sample has a medium to high free swell value. Therefore detailed site investigation and accurate measurements must be done for whatever types of engineering structures are going to be constructed. Because it brings on considerable cracking and damage on the engineering structures due to the continuous swelling and shrinkage properties of clayey soil. The free swell test is the simplest test that gives a fair approximation of the degree of the expansiveness of the soil sample. According to skempton (1953) cited in Getahun et al. (2013) and Holtz and Gibbs (1956), soil's potential expansiveness is correlated with activity value and free swell test. In the study area potential expansiveness was classified as low, medium, and high with activity values, less than 0.75, 0.75 to 1.25, and greater than 1.25 respectively (Table 13).

5.2.1.7 p^H test

According to Abramson et.al, (1996) cited in Berhane (2010), to determine soil chemistry (acidity, alkalinity) a chemical test should be conducted on the soil. One of the chemical tests is pH and the pH test was conducted on 14 soil samples. The results are included in (Table 13). According to laboratory results, the soil of the study area has a pH value of greater than 7(seven) and is alkaline in composition, which indicates minerals of the soils are stable except TP3, TP4-1, TP4-2 which is less than seven and acidic in composition. Acidic soil will affect the stability of roads and have adverse effects on concrete strength (Getahun et al.(2013).

Table 13: Free swell test, $P^{H,}$ activity and potential expansiveness of soils of the study area

Test pits	Depth (m)	Sampling location		LI	CI	P^H	Free swell test			Activity	Potential expansiveness	Remarks	Dominant Lithology
		Long.	Lati.				Volume in kerosene(ml)	Volume in distilled water(ml)	Fs (%)				
TP1	1.7	438194	745876	0.13	0.87	7.15	21	26	60	1.08	Medium	N	Silt
	3.6			-0.41	1.41	7.17	20	23.l	35	0.70	Low	I	
TP2	1.5	436038	746229	-0.44	1.44	7.14	20	25.9	59	1.12	Medium	N	
	2.7			-0.20	1.20	7.15	20	3l	100	1.39	High	A	
TP3	2.3	436792	745197	-0.38	1.38	6.77	20	23.7	37	0.62	Low	I	
TP4	1.8	434395	745570	-0.10	1.10	6.65	20	25	50	0.89	Medium	N	
	3.2			-0.23	1.23	6.54	21	26.5	55	1.22	Medium	N	
TP6-1	2	431669	745806	-0.31	1.31	7.1	20	30	100	1.53	High	A	
TP7	3	431195	747978	-0.38	1.38	7.15	20	30	100	1.43	High	A	
TP9	3.2	431449	744104	0.27	0.73	7.15	20	30	100	1.44	High	A	
TP10	3	429734	745720	0.65	0.35	7.15	20	30.4	104	1.32	High	A	Silt

TP14	3.4	429742	744069	0.50	0.50	7.18	20	30.1	101	1.48	High	A	
TP5-1	1.6	432725	739745	-0.79	1.79	7.16	20	22	20	ND	Low	-	Sand
TP5-2	3.4			-1.09	2.09	7.17	20	21.7	17	ND	Low	-	
TP6-2	3.5	431669	745806	0.25	0.75	7.16	20	21.3	13	ND	Low	-	
TP8	3.1	434957	746724	0.37	0.63	7.17	21	23	22	ND	Low	-	
TP11	2.7	440434	745664	0.07	0.93	7.17	20	28	80	0.79	Medium	N	Clay
TP12	2.4	435121	743861	0.46	0.54	7.16	20	26	60	0.78	Medium	N	
TP13	3	432941	743988	0.51	0.49	7.14	20	27	70	0.75	Medium	N	

FS= free swell, A= active soil, N= normal soil, IN= inactive soil, ND = not determined LI = liquidity index CI = consistency index
YAIP = Yirgalem Agro industrial park

5.2.1.8 Compaction test

A laboratory standard proctor compaction test was used according to ASTM D698 (1998) to determine the maximum dry density and optimum moisture content of the soil under investigation. The values of maximum dry density and optimum moisture contents for selected soil samples have been summarized in (Table 14) and presented in appendix F. it also expressed in graph (figure 25).

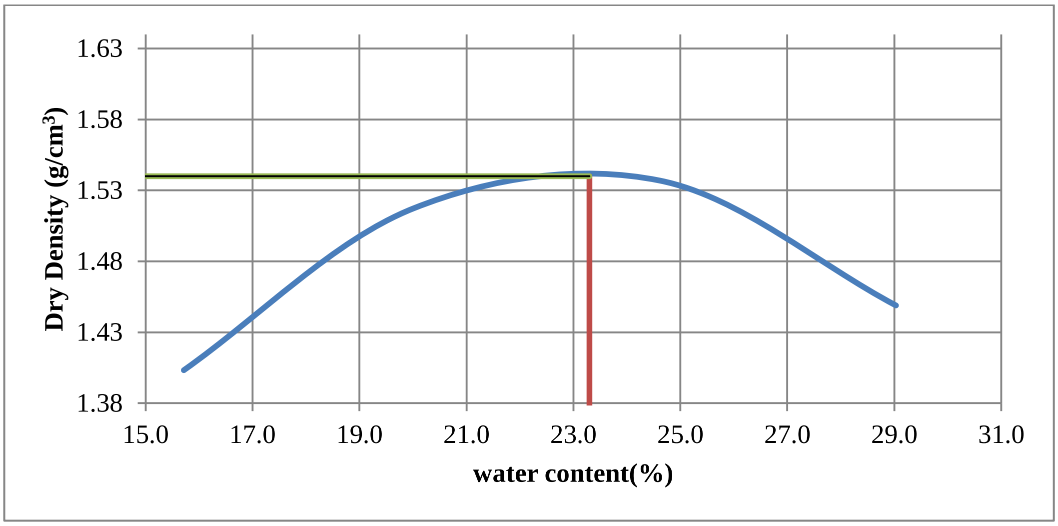


Figure 25: Compaction test graph of soil (YAIP)

Table 14: Compaction test result

Test pits	Depth(m)	MDD	OMC	Soil type	Dominant lithology
TP1	1.7	4.2	32	MH	Silt
TP3	2.3	1.08	33.7	MH	Silt
TP4	3.2	4.1	33	MH	Silt
TP8	3.1	3.8	41	SP-SM	Sand
TP10	1.6	1.13	49.3	MH	Silt
YAIP		1.54	23.3	MH	Silt

5.2.1.9 Shear strength test of soil in the study area

The direct shear strength test of soils of the study area was obtained by three trials applying a vertical load of 100, 200, and 300 N in consolidated drained conditions. Shear stress is calculated from providing ring reading value times by ring calibration factor then divided by specimen area; maximum shear stress was obtained by plotting horizontal displacement versus shear stress graph. The internal friction angle of the soil samples was computed from the slope and cohesion(c) obtained by the intersection point of the line of maximum shear stress Vs. the shear load is shown in the graph (appendix G). In the study area, direct shear strength tests were conducted for six test pit soil samples and cohesion and friction angle were obtained for those soil samples as portrayed in (Table 15).

Table 15: Cohesion and friction angle of soil sample in the study area

Test pits	Depth	C(KN/M^2)	Slope	Friction angle(ϕ°)	Soil type(USCS)	Dominant lithology
TP5	1.6	9.928	0.486	25.93	SP	Sand
TP7	3	28.793	0.5878	34.46	MH	Silt
TP8	3	16.2	0.53	27.72	SP-SM	Sand
TP3	2.3	39.7	0.4	22.6	MH	Silt
TP1	1.7	35.5	0.54	28.4	MH	Silt
TP10	3	23.3	0.598	33.5	MH	Silt

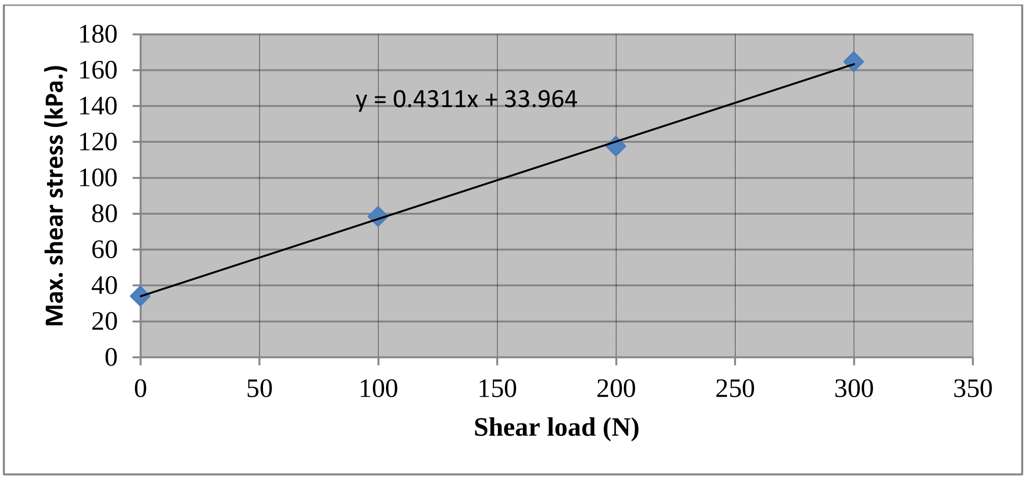


Figure 26: Shear strength graph

5.2.2 Engineering classification of soil mass based on USCS

The Unified Soil Classification System is based on the remembrance of the type and prevalence of the constituents considering grain size, gradation, and plasticity. The laboratory grain size analysis result indicates that the percentage passing No. 200 sieve size for soil sample TP5-1, TP5-2, TP6-2, and TP8 in the study area is < 50%, implying they are coarse-grained soils. To classify the soils under study according to this system uniformity coefficient and coefficient of gradation were used for soils that have a percentage finer less than five (<5), between five and twelve ($5<F_{200}<12$), and greater than twelve (>12). For the remaining test pits percentage passing No. 200 sieve size is >50%, implying they are fine-grained soils. Further subdivided are based on liquid limit values and plasticity index (figure 28). In this classification system, paired letter symbols are used for each soil group. The first symbol refers to, the predominant particle size, and the second symbol for fine-grained soils subdivide based on Low (L) or high (H) plasticity (Das, 2012; Arora, 2003). According to this classification system as summarized in (Table 16) the soils of the study area were classified as MH, CH, SP, SM, and SP- SM. about 20% of the total soil samples of the study area fall in sandy soils, 65% fall in silt soils and about 15% fall in clayey soils. About 10% of total soil samples were classified as poorly graded sand (SP), About 5% of total soil samples were classified as poorly graded sand with silt or (exhibit dual symbol). About 65% of total soil samples of the study area were classified as high plasticity silt (Elastic silt) (MH). About 15% of total soil samples were classified as high plastic clay (Fat clay) (CH) and about 5% of total soil samples of the study area were classified as Silt sand (SM). According to the general engineering suitability of the soils Arora (2003) most of the soils of the study area (about 80%) fall in poor engineering workability. 20% of soils fall in good engineering workability.

5.2.3 Engineering classification of soil mass based on IAEG (1981) system

For this research work, to classify the soils of the study area, the Unified soil classification system proposed by the International Association of Engineering Geology (IAEG) (1981) which is a modified form of the unified soil classification (USC) and the British soil classification for engineering purposes (BSCS) is applied. According to this classification soil of the study area were falls into SP, SM (SML), CH, MH, and MV (figure 27). About 10% of total soil sample were classified as SP(SAND, poorly graded), 10% classified as SM(SML) SAND, silt of low plasticity, about 5% classified as MV(SILT of very high plasticity soil, about 20% classified as CH(CLAY of high plasticity), and 60% classified as

MH(SILT of high plasticity. Its descriptive name and letter were portrayed in (table 16) and the classification system was presented in (Appendix H).

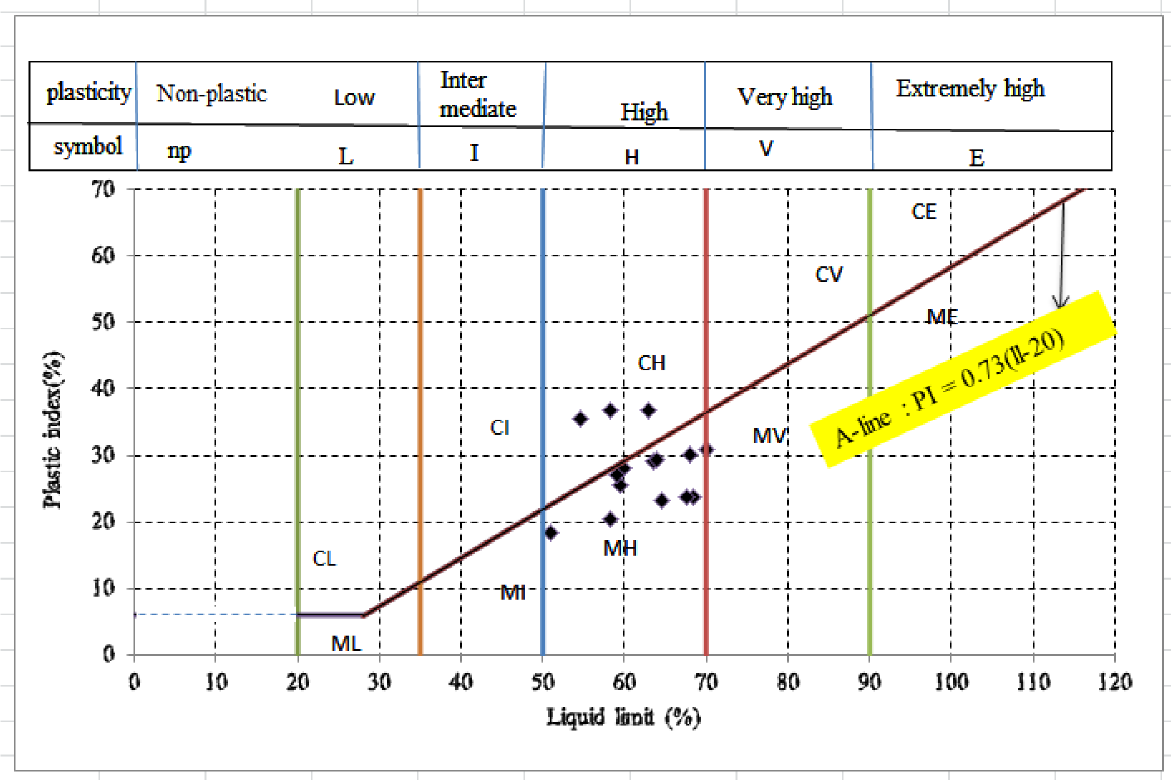


Figure 27: Plasticity chart for IAEG (1981) classification system.

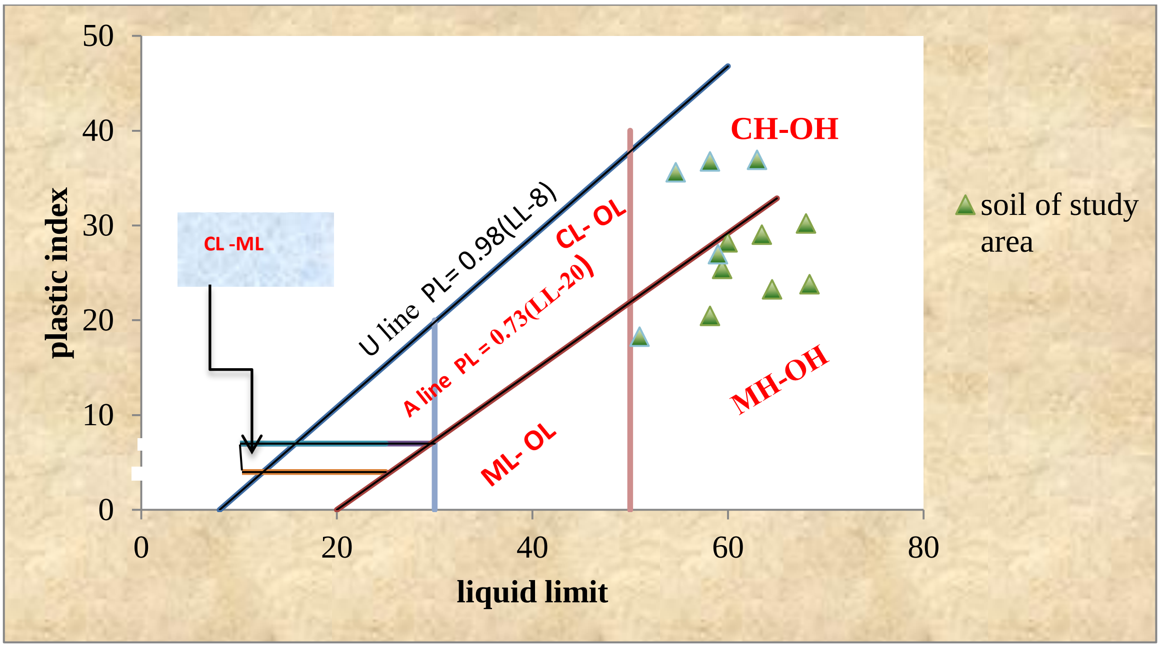


Figure 28: Plasticity chart of the study area for USC classification system

Table 16: USCS soil classification and classification system proposed by IAEG (1981) in the study area.

Test Pit	Depth (m)	SG	Sieve analysis				Consistency limit		USCS Classification		IAEG (1981)		Dominant lithology
			G(%)	S(%)	M(%)	C(%)	LL	PI	Symbol	Soil name	symbol	Soil name	
TP1-1	1.7	2.6	0.2	16.9	56.1	26.97	63.5	29	MH	Elastic silt	MH	SILT of high plasticity	Silt
TP1-2	3.6	2.6	0.1	15.1	48.55	33.95	68.38	23.77	MH	Elastic silt	MH	SILT of high plasticity	
TP2-1	1.4	2.57	0	10.6	71.82	18.18	58.2	20.43	MH	Elastic silt	MH	SILT of high plasticity	
TP2-2	2.7	2.58	0.3	9.5	62.9	20.3	60	28.2	MH	Elastic silt	MH	SILT of high plasticity	
TP3	2.3	2.57	0.1	6.1	54.75	37.67	64.54	23.23	MH	Elastic silt	MH	SILT of high plasticity	
TP4-1	1.8	2.65	0.2	22.4	47.7	28.61	59.42	25.4	MH	Elastic silt	MH	SILT of high plasticity	
TP4-2	3.2	2.69	2.2	20.2	51.7	24.83	68	30.2	MH	Elastic silt	MH	SILT of high plasticity	
TP6-1	2	2.66	0	25.7	51.66	20.14	70	30.9	MH	Elastic silt	MV	SILT of very high plasticity	
TP7	3	2.68	1.6	23.2	60.2	16.62	67.7	23.8	MH	Elastic silt	MH	SILT of high plasticity	
TP9	3.2	2.68	0	13.3	74	12.7	51	18.25	MH	Elastic silt	MH	SILT of high plasticity	

TP10	1.6	2.66	0	30.8	48.8	20.4	59	26.96	MH	Elastic silt	MH	SILT of high plasticity	Silt
TP14	3.6	2.68	0	10	70	19.89	63.2	29.46	MH	Elastic silt	MH	SILT of high plasticity	
YAIP	-	-	0.3	18	81.7		57	26	MH	Elastic silt	MH	SILT of high plasticity	
TP5-1	1.6	2.45	0.4	98.2	1.4	0	49.5	16.7	SP	Poor graded sand	SP	SAND, poorly graded	Sand
TP5-2	3.4	2.42	0.4	99.4	0.2	0	48.8	14.2	SP	Poor graded sand	SP	SAND, poorly graded	
TP6-2	3.5	2.66	5.7	58.5	35.8	0	49.2	18.7	SM	Silt sand	SM(SML)	SAND, silt of low plasticity	
TP8	3.1	2.65	0	91.8	8.2	0	49	13.64	SP-SM	Poorly graded sand with silt	SM(SML)	SAND, silt of low plasticity	
TP11	2.7	2.67	0	9.7	43.58	46.72	63	36.9	CH	Fat clay	CH	CLAY of high plasticity	Clay
T912	2.5	2.56	0	11.2	41.79	47.1	58.2	36.73	CH	Fat clay	CH	CLAY of high plasticity	
TP13	3	2.65	0	12.1	40.25	47.65	54.7	35.57	CH	Fat clay	CH	CLAY of high plasticity	

5.3 Engineering geological characterization and classification of rocks

To characterize and classify the engineering properties of rock masses of Yirgalem town, 11 sampling points of rock discontinuity data and 14 sampling points of Schmidt hammer rebound tests had carried out in addition to the visual description and the use of a geological hammer. Based on the in situ rock strength tests, intact rock strength of the study area had grouped into three major classes, rocks with low, medium, high strength.

5.3.1 Engineering Geological Characterization of Rock

Rocks are a hard and solid mass of geological material that contains naturally occurring aggregates of minerals connected by strong bonds. It contains an aggregate of several types of minerals with different properties and arrangements. Many factors alter the strength and properties of rocks (Hudson J, A and Harrison J, P. (1997). Weathering and alteration, stress condition, degree of discontinuities, and water pressure can highly influence the final strength and properties of rocks. For this study, the descriptive scheme proposed by ISRM (1981) is used to systematically characterize the rock material and rock mass based on engineering geological characteristics. The descriptions of the rocks include recognition of the fundamental rock name (lithological name), description of the properties of the rock material (color, texture, state of weathering, and strength), description of additional properties necessary to describe the features of the rock mass (weathering profile, discontinuities, and infilling materials and its type, water pressure, roughness, any joint, fracture, faults, and others). Based on the above-listed rock description procedures, the different rock units in the Yirgalem Town have been described as follow.

5.3.1.1 Discontinuity characteristics

A discontinuity survey has been carried out in the study area to acquire data on parameters of discontinuities such as spacing, orientation, and joint conditions such as persistence, aperture, roughness, wall weathering and type of infill material/s. Summary of some representative discontinuity measurements on rock unit in the study area was discussed below.

Discontinuity characteristics of volcanic ash rock units

This rock is mainly found in the south-central, eastern, and northeastern parts of the study area. It is light to brown, fine to coarse-grained rock units. No systematic and random joint sets were observed. It is a loose unconsolidated deposit and has weak and soft wall strength. The flowing of water on it was also documented during the field survey.

Discontinuity characteristics of pumice rock units

Those subunits are exposed in the eastern and northeastern parts of the study area. They are light gray to black-brown. The exposures of this subunit are outcropped along with stream and road cuts. Random joint sets were observed. The joint surface or wall is smooth and shows weak and soft wall strength and coarse-grained texture. It shows a damped moisture condition during the time of the investigation.

Discontinuity characteristics of ignimbrite rock units

This type of subunit was exposed below (Emanuel church), the north-central part of the study area which forms a cliff. It is characterized by light to brownish red. Three vertical joint sets (336^0, 346^0, and 350^0) are dominant. The joint surface or wall is planar and rough to smooth, and the aperture varies from tight (for horizontal joints) to open (for vertical joints). The vertical joints are moderate to widely spaced (0.2cm to 0.6 cm). Some joints are filled with clay soil and gravel materials and the other is without filling materials. Completely dry moisture condition at the time of the investigation. It also observed on the YAIP road cut in western parts. Joints of NEE to SWW (280^0, 292^0) strikes are observed. The joints are in general vertical and the spacing varies from 6 to 17cm. Joint surfaces are linear and rough, have low wall strength, and are very tight and moderately to partly open aperture. Soil and boulder infill materials are present which are parallel with formation, and 5mm aperture and low persistence (1-3m).

Discontinuity characteristics of basalt rock units

This engineering geological unit is exposed in limited northwestern parts of the study area. It is dark to brown. Random joint sets were observed during the time of the investigation. No spaced joints are observed and it has strong wall strength. The surface of random joints is rough, and hard to moderately hard wall strength, close to wide spacing with very low aperture is observed in these subunits. Clay and red cotton soil cemented with basalt rock subunits. Joints were dry during the time of the field survey.

5.3.1.2 Degree of weathering

The compressive strength of rock masses decreases due to weathering or alteration of the wall strength. In the study area degree of weathering was observed based on the visual description with help of the suggested method for quantitative description of rock mass (ISRM, 1981) and it classifies as fresh, slightly weathered, moderately weathered, and highly weathered.

Fresh to slightly weathered rock unit

Under this rock unit, basalt and ignimbrite were categorized which was highly exposed in north-central and western parts of the study area. No visible signs of weathering were observed. Dark-colored basalt fine in texture and red-brown ignimbrite rock units are documented during the field survey. Both lithologies require some blows of the geological hammers to be broken and the rock material may be discoloured by weathering and maybe somewhat weaker externally than in its fresh conditions for slightly weathered rock units.

The moderately weathered rock unit

Volcanic ash, pumice, ignimbrite, and basalt rock units are grouped under this unit which was exposed in the study area. Less than half of their rock material is decomposed and/or disintegrated into the soil. Fresh or discoloured rock is present either as a discontinuous framework. Basalt under this group is cemented with soil and makes a cliff underlain by red clay soil and volcanic ash formation and it shows the so-called exfoliation types of weathering. Its color changed from dark to brown due to the effect of weathering. It is exposed in central parts of the study area along the Gidabo River. Ignimbrite which was exposed under Emanuel church in the north-central part of the study area categorized under this unit which is reddish-brown in color.

The highly weathered rock units

Volcanic ash and pumice rock units are highly weathered lithology exposed in eastern northeastern and south-central parts of the study area along with river/stream cut. Generally, they are light brown in color and fine to coarse texture. According to the suggested field description proposed by (ISRM, 1981) characterized by all rock material is decomposed and/or disintegrated to soil and the original mass structure is still largely intact. It can be broken by hand into smaller pieces and makes a dull sound when stuck by geological pick. Lithology grouped under this unit is completely discoloured compared with fresh rock.

5.3.1.3 Rock strength estimation based on geological hammer

Description of rock masses in the study area focused mainly on the engineering aspects of view. Therefore, greater attention is given to the presence of discontinuities (fractures) in the rock mass and the effects of weathering. Both of these have a significant influence on the mechanical properties and behavior of a rock mass (Karryn, 2005). The presence of discontinuities can reduce the strength of a rock mass and their spacing and orientation govern the degree of such reduction (Berhane, 2010; Getahun, et al. 2013). To determine the engineering properties of rocks, different ways of rock strength tests were conducted in the field. The geological and N-type Schmidt hammer tests were the two most widely used field equipment to evaluate the strength of the rock mass. In addition to weathering condition and measurements of rock discontinuities by using Brunton compass, groundwater condition was observed. Schmidt hammer test does not conduct on pumice and volcanic ash rock unit in the study area due to its weak property and does not give rebound value or its rebound value is too small. Therefore, it is classified based on the geological hammer, and other discontinuity parameters. Generally, rock masses of the study area were classified according to the parameters proposed by ISRM (1981) (Table 3) and fall into three major classes, such as rocks with low mass strength, medium mass strength and rocks with high mass strengths.

The low strength rock unit

The rock mass grouped in this engineering rock unit is high to moderately weathered pumice rock unit, and volcanic ash was observed in the study area. The intact value for this rock unit is less than 15MPa. It is fractured simply by a pocket knife or becomes firm with a point of the geological hammer.

The medium strength rock unit

For this rock unit intact rock strength value is estimated 20 to 50MPa and moderately weathered dark brown basalt, and ignimbrite is grouped under medium strength rock unit in the study area. The average Schmidt hammer rebound value ranges from 24 to 26. One strong blow of a geological hammer is needed to fracture the rock mass.

The high strength rock unit

Under this subunit basalt and ignimbrite were grouped. Strength value estimated to 55 – 97MPa. This subunit is mainly found in the southwest part of the town (hida kalit) and north-central parts (figure 29). They can't break easily by a simple test of a geologic hammer. The average Schmidt hammer rebound value ranges from 39 to 50.

Figure 29: Discontinuity of ignimbrite rock. A) discontinuity and 50mm open spacing without filling material, three planar systematic joint sets with NW and SW direction. B) vertical orientation with linear systematic joint set, 25mm aperture without filling materials, at GPS reading (long. 433214 m lati. 747850 m). C) Conducting Schmidt hammer test on basalt rock exposure at GPS reading (long. 435230 m lati.746250 m).

5.3.2 Rock Mass Classification Based on Uniaxial Compressive Strength (UCS)

Rock engineering properties are one of the most important parameters in the design of groundworks and the classification of rock for engineering purposes. UCS test is time-consuming and expensive. In this work, it can be indirectly estimated from the direct tests of the Schmidt hammer. The correlations proposed in different literature between unconfined compressive strength and Schmidt hammer rebound value (SHV) were investigated to define the effectiveness of each one in predicting rock strength for various rocks. The equation proposed for the N-type Schmidt hammer value is used to estimate the strength of rock mass in the study area. In the study area, rock mass strength was estimated by using an N-type Schmidt hammer and all rebound value is greater than ten. So, the correlation equation proposed by (Wang et al., 2017) (eq.1) and Barton and Choubey (1977) (eq.2) were used. For 14 rock mass, Schmidt hammer rebound tests were conducted from the field. According to data obtained from the average rebound value, UCS were calculated based on Barton and Choubey(1977) (eq.2) (Table17) and its value ranges from 52.5 to 144.5 MPa in the study area. Unit weight of rock specimen was obtained and later used to calculate UCS by using Barton and Choubey(1977). According to (Wang et al., 2017) UCS of rock mass in the study area is calculated from average SHV and, it ranges from 82 to 121MPa as presented in Table 18. ISRM (1981), classify the UCS as very low, low, moderate, high, and very high with its value <5, 5-20, 20-60, 50-200, and >200 respectively. Both empirical equations yielded nearly the same results with small deviation and fall in the same class as graded according to ISRM (1981) (Figure 30).

Table 17: UCS calculated according to Barton and Choubey (1977) from unit weight

Rock type	Md	V1	V2	V2 - V1	ρd	γ	SHV	UCS	Grading
Basalt	410	1000	1152	152	2.697	26.97	26	42.67	Moderate strength
Basalt	400	1000	1145	145	2.76	27.59	25.5	41.7	Moderate strength
Basalt	430	1000	1160	160	2.69	26.88	42	100	High strength
Basalt	417	1000	1160	160	2.61	26.06	50	144.5	High strength
Basalt	425	1000	1160	160	2.66	26.56	48	135	High strength
Ignimbrite	450	1000	1180	180	2.50	25.00	47	109.6	High strength
Ignimbrite	440	1000	1170	170	2.59	25.88	42	93.3	High strength
Ignimbrite	376	1000	1145	145	2.59	25.93	43	97.7	High strength
Ignimbrite	453	1000	1168	168	2.70	26.96	25	39.8	Moderate strength
Ignimbrite	345	1000	1134	134	2.57	25.75	26	41.7	Moderate strength
Ignimbrite	504	1000	1200	200	2.52	25.20	24	34.7	Moderate strength
Ignimbrite	467	1000	1175	175	2.67	26.69	24.5	38.9	Moderate strength
Ignimbrite	439	1000	1166	166	2.64	26.45	40	87	High strength
Ignimbrite	460	1000	1172	172	2.67	26.74	39	85.14	High strength

Md = mass of dry rock sample, V1= volume of water, V2= volume after immersing rock sample, ρd= dry density, γ= unit weight; SHV = Schmidt hammer value

Table 18: Calculated UCS obtained from Schmidt hammer rebound value according to Wang et al., (2017)

GPS location			Lithology type	SHV	USCS(MPa)	Grading
Easting	Northing	Elevation				
433607	746363	1705	Basalt	26	48.48	Moderate strength
433587	746354	1711	Basalt	25.5	46.97	Moderate strength
433619	746350	1700	Basalt	42	96.96	High strength
433098	746943	1698	Basalt	50	121.2	High strength
434154	746912	1687	Basalt	48	115.14	High strength
429107	743749	1784	Ignimbrite	47	112	High strength
429109	743708	1783	Ignimbrite	42	97	High strength
428713	743590	1777	Ignimbrite	43	100	High strength
433567	746388	1708	Ignimbrite	25	45.45	Moderate strength
429004	744012	1759	Ignimbrite	26	48.48	Moderate strength
428900	743760	1780	Ignimbrite	24	42.42	Moderate strength
429190	743788	1788	Ignimbrite	24.5	43.94	Moderate strength
429001	743920	1783	Ignimbrite	40	90.9	High strength
428912	743910	1788	Ignimbrite	39	87.87	High strength

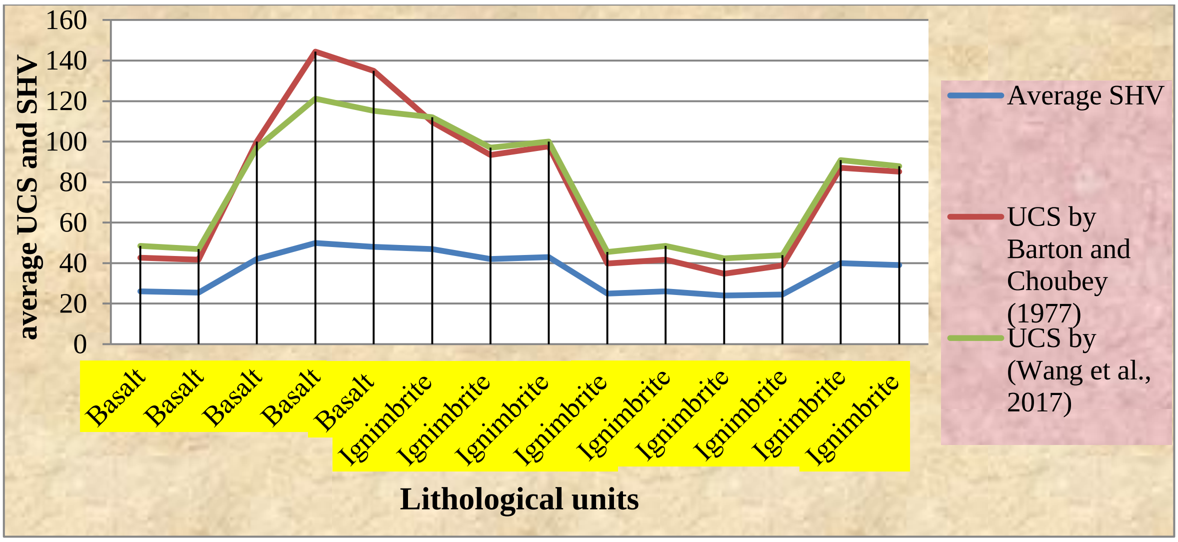


Figure 30: Relation of SHV and UCS of rock mass in the study area

5.4 Assessment of hydrogeological, geomorphological, and geodynamic conditions

5.4.1 Groundwater level influences on shallow infrastructures

Groundwater (GW) can pose concerns to the engineering environment both during and after construction. During construction, groundwater can create unfavorable conditions all through excavation and the location of foundations. After construction if groundwater reaches the foundation stage it can pose problems in diverse methods (Bell et al., 2020); which lowering the bearing capacity of the foundation soils, wash out fine materials which bind the bigger grains (suffusion), and divulge the foundation to failure, impose shrink and swell on a certain form of soil, fluctuates and consequences in relative wetting and drying of the soils, and it can devolve/corrode the foundation if the chemistry of each the groundwater and the foundation allow. The spatial variation of GW level that is used to determine the GW flow direction was determined by using groundwater level data obtained from elevation and static water level (Table 19). Therefore, the overview of the regional GW level of Yirgalem town is estimated from twelve boreholes data. From the values of the GW level contours, the general direction of GW flow was determined and which flows towards the west into Lake Abaya (figure 31). GW table is at shallow surface especially in western parts.

Table 19: Static water level and groundwater level in the study area

Borehole location	Longitude(m)	Latitude(m)	Altitude(m)	Depth to GW	GW level Elevation(m)
Abosto #1	430394	744737	1674	9.86	1664.14
A bosto#2	431983	747649	1760	5.5	1754.5
Abosto gebeya	430946	746236	1678	6.8	1671.2
Y1	436513	745876	1759	55.6	1703.4
Y2	437944	745657	1780	10.7	1769.3
Y3	434068	745789	1772	20.7	1751.3
Y4	436883	745789	1765	20.2	1744.8
Mesincho	431250	752170	1800	10	1790
Arada	421586	744115	1649	17	1632
Kalit	429933	744115	1640	5.6	1634.4
WICHO	431647	743704	1754	41	1713
SHAFINA	436462	744828	1752	30.52	1721.48

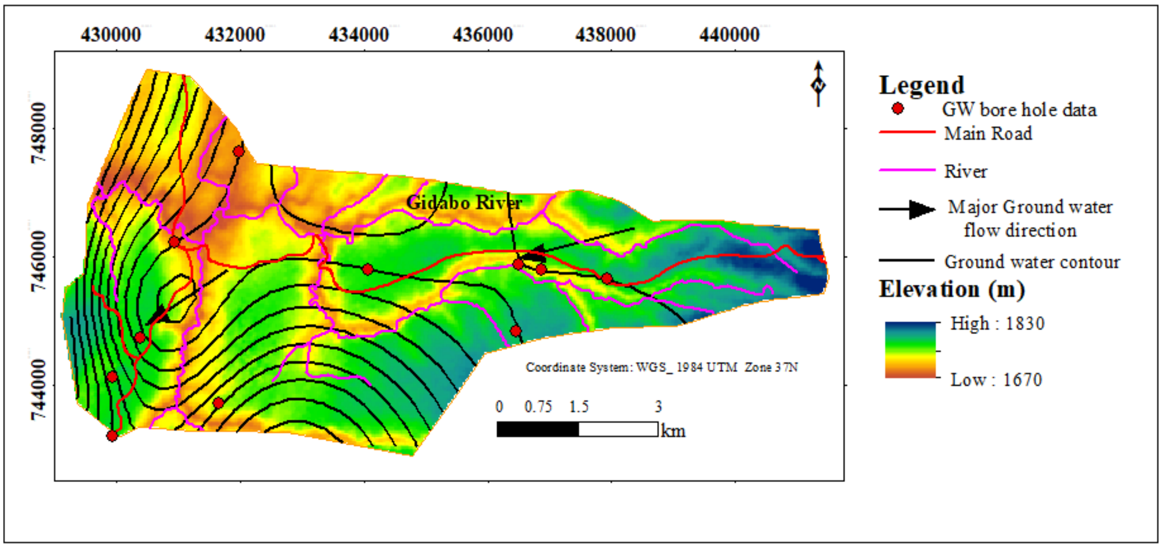


Figure 31: Groundwater level and flow direction in Yirgalem Town

5.4.2 Surface geodynamic problem in the study area

The surface geodynamic problems in the study area are landslide, and problematic soil (expansive soil) as discussed below.

5.4.2.1 Landslide

According to the types of movement of materials (rocks and soils) (Varnes 1987), the following types of landslides were identified in the study area. In general, the most common factor of landslide in the study area is rainfall, human interference, erosion, and steep topography.

Rockfalls. This type of landslide was observed on slightly to medium weathered ignimbrite, and volcanic ash forming steep to very steep slopes and it is common in the study area. Fallen blocks of these materials are found at the foot of a steep slope/cliff unless they jump and roll downslope after they fall. This type of landslide affected forestry areas with wild animals, asphalt roads, residential areas, historical and archeological places.

Soil falls (debris fall). In the study area (figure 36), these types of landslides occur during rainy due to the collapse of soil mass in streams/rivers/gullies, road cuts, and human activities. This type of landslide was observed in unconsolidated materials like volcanic ashes and alluvial deposits. The causative factors can be erosion and rainfall. This type of landslide mostly affected dry-weather roads and farmland (figure 32).

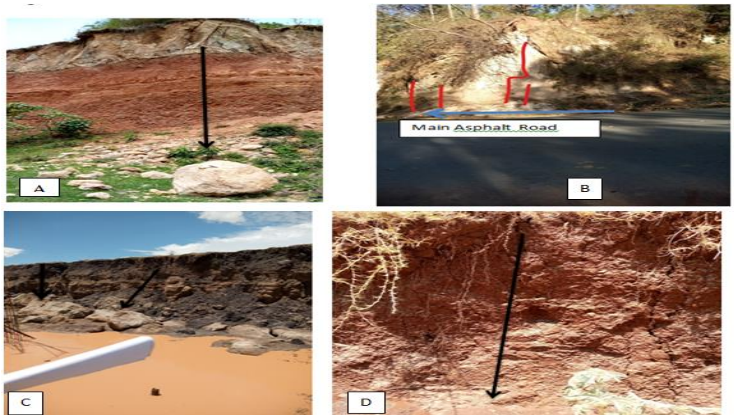


Figure 32: Rockfall at GPS reading A, (long. 433878 m lati.7466852 m). B,(long. 433403 m lati.746039 m). C, Debris fall by human activity at GPS reading (long.429840 m lati. 745992 m) D, debris fall by a road cut at GPS reading (long. 436192 m lati.746608 m)

Sliding: It occurs when the materials move along a defined shear surface by making continuous contact with the failure plane. This type of landslide found in the study area is rotational slides. It is observed in the northcentral and southwest parts of the study area (figure 36). Those landslides damage roads, infrastructure, crops, vegetation, and lightweight building in the study area. Especially in the rainy season, it is serious problem still now at the edge of town (figure 33,).

Figure 33: Rotational sliding in the study area on main asphalt road at GPS reading A, (long.433515 m lati. 745683 m). B, (long. 433617 m lati. 745552 m). C, sliding on dwelling house at GPS reading (long.434401 m lati. 746538 m).

5.4.2.2 Slope

It refers to the angle of inclination of the ground surface, expressed in degrees or as a percentage (Huggett, 2007). The steepness of the slope has a great effect on urban expansion, route selection. Yirgalem Town is generally bounded by a steep slope in the northern, eastern, and southern directions. Based on the range of slope, the slopes of the study area were classified into three (3) classes (Figure 34). According to the range of slope classification the study area was classified into 0.0-16%, 16-35%, and 35-80% slope range. The most of the northwestern part of the study area is characterized by a flat to gentle slope.

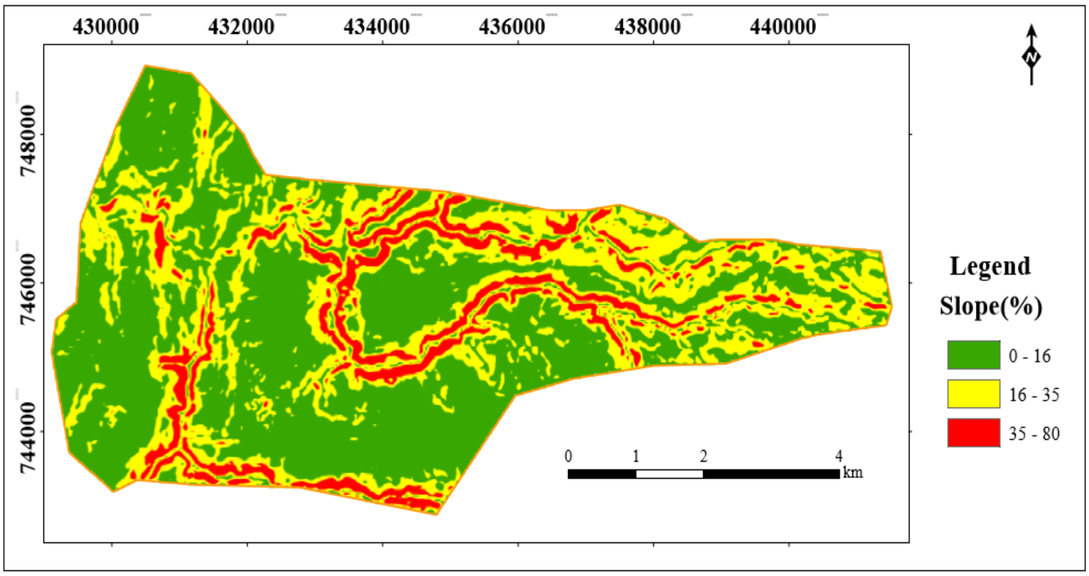


Figure 34: Slope map of the yirgalem town

5.4.2.3 Problematic soil(expansive soil) in the study area

In the study area, expansive soil (figure 35) was documented in the western part especially in YAIP and Abosto area (figure 36).

Figure 35: Problematic soil in the study area A, deep cracked brown soil in hida kalit at GPS reading(long. 429875 m Lat.745790 m) B, expansion problem on black soil in hida kalit at GPS reading (long.429840 m Lati.745719 m). C, expansion on brown color soil in YAIP at GPS reading (long.429887 m Lati. 745868 m). D, expansion problem on red clay soil in abosto GPS reading (long.431874 m Lati.745528m)

5.5 Preparation of multipurpose engineering geological map

Multi-purpose engineering geological maps are produced to provide information on many aspects of engineering geology for a variety of planning and engineering purposes (IAEG (1981); Getahun et al., 2013)). These maps are complex and unique in terms of their comprehensive content. Multipurpose engineering geological map of Yirgalem town was produced at detailed medium scale (1:20000) based on fieldwork mapping; in situ rock tests and soil laboratory test results. Similar engineering geological properties and their spatial distributions of geological formations are explained and delineated as shown in (Fig 36). Moreover, geodynamic phenomena (gully erosion, surface erosion, land sliding, flood-prone, and problematic soil) of Yirgalem town are documented.

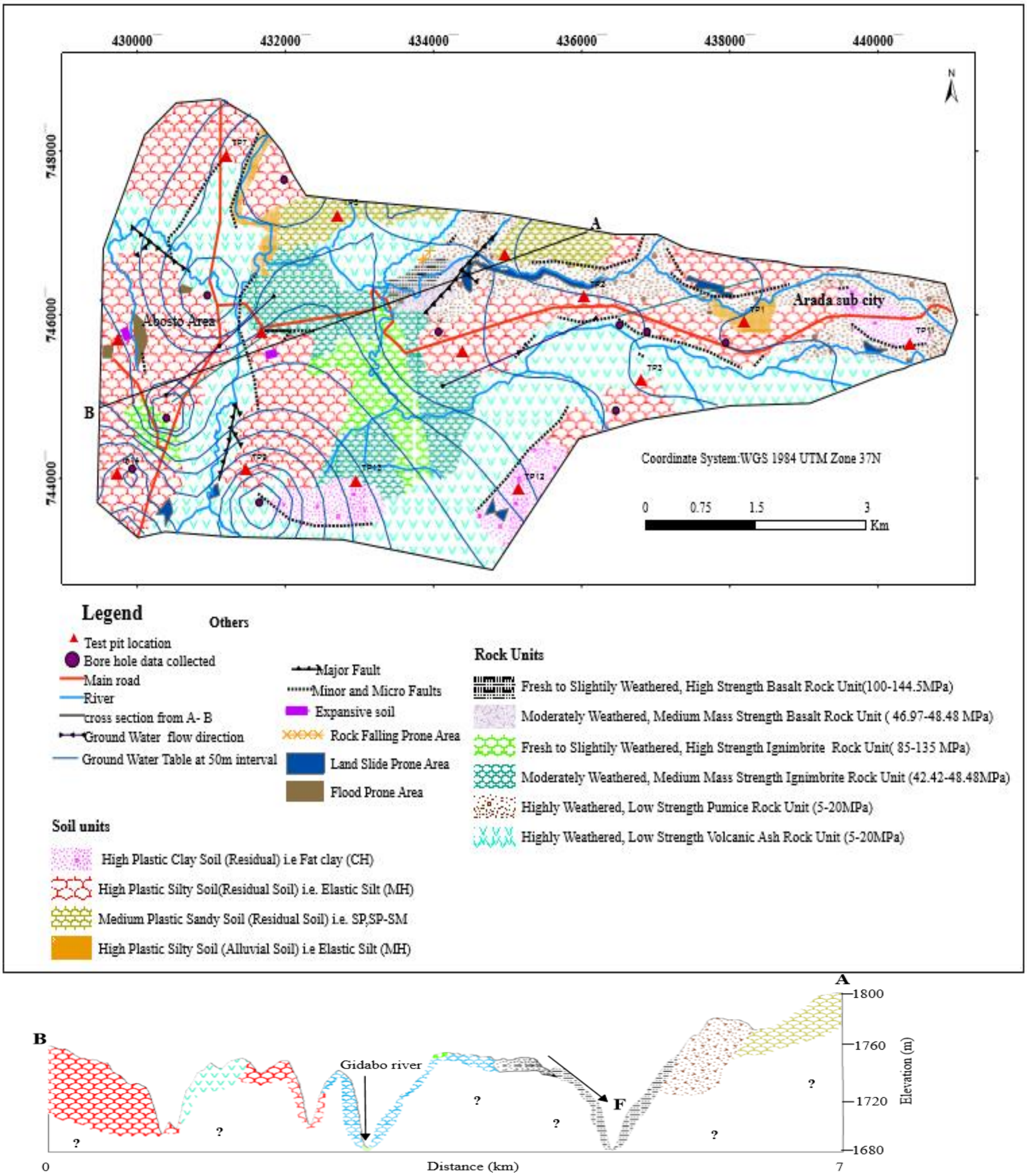


Figure 36: Multipurpose Engineering Geological Map and Cross-section of Yirgalem Town

6 CONCLUSIONS AND RECOMMENDATION

6.1 Conclusions

The following conclusions were made from this research work:

- In the study area, the dominant soil unit is the residual soil deposit, and the main rock units are volcanic ash, ignimbrite, pumice, and basalt rock unit.

- Geomorphological conditions of the study area indicate that various landforms such as very steep-slope, undulated, rugged, flat or gentle slope and horst and graben are manifested.

- Landslide, surface erosion, gullies, flooding, and expansive soil are common surface geodynamic problems identified in the study area.

- From laboratory test result, Clay soil is characterized by natural moisture content of 28.7 – 37.34%, the liquid limit of 54.7- 63%, plasticity index of 35.57- 39.6 % (very high plastic), the activity of 0.75- 0.79 (normal with 70 - 80%) free swell (medium potential of expansiveness of the soil) and specific gravity ranging from 2.56 to 2.67. Silt soil is characterized by the moisture content of 26.1 – 49.5%, liquid limit of 51-70%, plasticity index of 18.25- 30.9% (medium to highly plastic), the activity of 0.62-1.53 with 35-104% free swell (normal to active soils, i.e. medium to the high potential of expansiveness) and specific gravity ranges 2.57 to 2.69. Sandy soil is characterized by the moisture content of 10.5 – 40.35%, liquid limit of 48.8 - 49.5%, plasticity index of 13.64- 18.25% (medium plastic) with 13 - 22% free swell (i.e. low potential expansiveness of the soil), and specific gravity ranges 2.42 to 2.66.

- The soil of the study area has a p^H value ranges 6.54 to 7.18 which is acidic and alkalinity in composition. Shear strength parameter, cohesion(C), and angle of internal friction (Φ) of the soils of the study area range from 9.928-39.7KN/m2 and 22.6-34.31 respectively. Maximum dry density and optimum moisture content were determined by standard proctor test, ranges 1.08 to 4.2 MDD and 23.3- 49.3% OMC.

- According to IAEG (1981) soil classified as SP (SAND, poorly graded), SM (SML) (SAND, silt, of low plasticity), MV(SILT of very high plasticity soil, CH(CLAY of high plasticity), and MH(SILT of high plasticity).

- USCS classification system, MH(Elastic silt), CH(Fat clay, SM (silt sand), SP(poorly graded sand), SP-SM (poorly graded sand with silt), 80% of soil fall in poor engineering workability, and 20% of soils fall in good engineering workability.
- Lithological rock mass units are classified based on UCS estimated from the field using Schmidt and geological hammer, UCS correlated with SHV, RQD system, and RMR system. Basalt rock unit subdivided into fresh to slightly weathered, high mass strength and moderately weathered, medium mass strength. Ignimbrite rock unit subdivided into fresh to slightly weathered, high strength mass and moderately weathered, medium mass strength. Volcanic ash and pumice rock mass units are subdivided into highly weathered, low mass strength rock mass (soft rock units); with UCS value estimated from the geological hammer of 5-20MPa.
- A Multi-Purpose Engineering Geological Map of the Yirgalem Town is prepared with a detailed medium scale (1:20000), which provides information on the engineering geological and geotechnical characteristics of soils and rocks, geomorphological, surface geodynamical, and hydrogeological conditions of the study area.

6.2 Recommendation

Following recommendations are forwarded based on the current research work:

- Due to its rugged, undulated, and steeply sloping landform, eastern, southeastern, northeastern, and north-central parts of the study area are highly affected by landslides and are highly susceptible in the future. Therefore, further study related to the slope stability problem must be conducted to check the stability of soil and rock and the concerned body should pay attention before constructing any civil engineering structure in the area by taking the possible remedial measures related to that problem.

- Based on borehole data groundwater table is at shallow depth. In addition to this different cold and hot springs were observed during fieldwork. Hence, careful consideration was taken during construction of any civil engineering structure works and further groundwater chemical tests and hydro chemical analysis should be done for further understanding of their effects on the foundation of engineering work.
- The dominated soil in the study area after geological and engineering geological characterization are silt and clay which show swell and shrink characters (medium to high potential expansiveness) require careful consideration during the design and construction of civil engineering work. In addition study of its mineralogical composition was recommended for further research work.

- From atterberg limit test, the plastic index of most soil in the study area is high to very high which is poor for engineering workability. Hence, considerable attention should be taken before constructing any civil structure on the proposed site.

- Additional laboratory and field tests like consolidation, permeability, vane shear, standard penetration test, cone penetration test, point load test and etc. which were not conducted on this research work should be recommended for further study.

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APPENDIX A: Soil Sampling and Schmidt Hammer Test Location

Test pits	Easting	Northing	Sampling place
Tp1	438194	745876	Kidist Mariam sefer
TP2	436038	746229	Near city administration
TP3	436792	745197	07 kebele
TP4	434395	745570	03 kebele
TP5	432725	739747	02 kebele near hospital
TP6	431669	745806	Abosto near awada compus
TP7	431195	747978	Tula sefer
TP8	434957	746724	Gidabo mewureja
TP9	431449	744104	Near yekatit 25 school, abosto
TP10	429734	745720	YAIP
TP11	440434	745664	Arada k/ketema
TP12	435121	743861	Mesincho mewucha
TP13	432941	743988	Near studium
TP14	429742	744069	Kalit

SCHMIDT HAMMER TEST LOCATION

No	GPS location		Lithology type	SHV
1	Easting	Northing		
2	434175	746356	Basalt	26
3	434033	746181	Basalt	25.5
4	433646	746123	Basalt	42
5	433837	746615	Basalt	50
6	433837	746615	Basalt	48
7	430549	744376	Ignimbrite	47
8	430281	744698	Ignimbrite	42
9	429991	744606	Ignimbrite	43
11	432838	744344	Ignimbrite	25
12	434092	744542	Ignimbrite	26
13	433648	744987	Ignimbrite	24
14	433219	745669	Ignimbrite	24.5
15	430348	744739	Ignimbrite	40
16	432029	746535	Ignimbrite	39
17	432554	746012	Ignimbrite	30
18	431687	746052	Ignimbrite	35
19	434492	745212	Ignimbrite	30
20	433513	743882	Ignimbrite	34
21	432375	745311	Basalt	40
22	433539	746369	Ignimbrite	30

APPENDIX B: Natural Water Content Determination

MC = Mass of empty, clean can + lid (grams)
MCMS = Mass of can, lid, and moist soil (grams)
MCDS = Mass of can, lid, and dry soil (grams)
MS = Mass of soil solids (grams)
MW = Mass of pore water (grams)
W = Water content, w%
W% = Averages water content

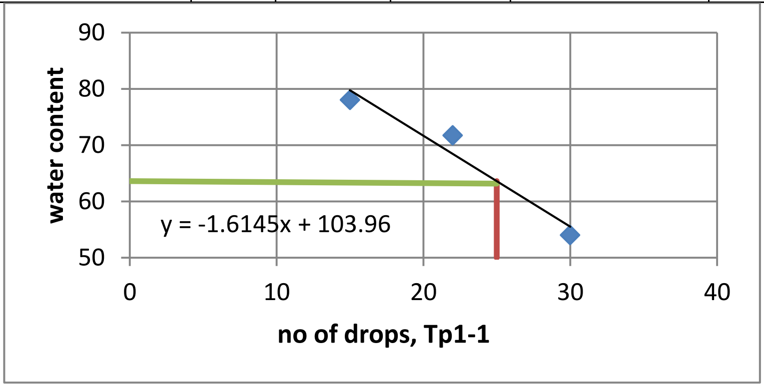
	TP1-1		Tp1-2		Tp2-1		Tp2-2		TP3	
Mn	vx	B4	B40	FAB	Gtz1	A4	G1	WPZ	Gtz1	A4
MC	21	20	20	20	19	19	20	19	19	19
MCMS	94	99	80	88	88	83	80	97	88	83
MCDS	74	77	61	67	72	70	70	78	72	70
MS	53	57	41	47	53	51	50	59	53	51
MW	20	22	19	11	16	13	10	19	16	13
W	37.7	38.6	46.3	23.4	32.2	25.5	20	32.2	32.2	25.5
W%	38.15		34.85		28.8		26.1		28.8	

	TP4-1		Tp4-2		Tp5-1		Tp5-2		TP6-1	
Mn	A8	T7	F64	P10	B9	A11	T9	vI10	tn1	B3
MC	21	21	20	19	21	20	20	17	21	21
MCMS	126	120	115	121	103	122	134	121	114	102
MCDS	101	97	91	98	89	106	123	112	91	85
MS	80	76	71	79	68	86	103	95	70	64
MW	25	23	24	23	14	16	11	9	23	17
W	31.3	30.3	33.8	29.1	20.6	18.6	10.6	9.47	32.8	26.5
W%	30.8		31.45		19.6		10.05		29.65	

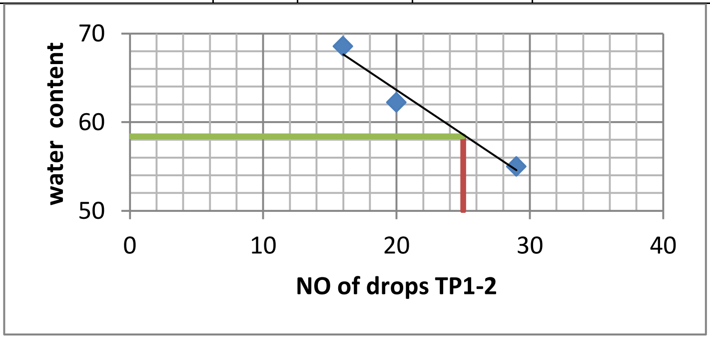
	TP6-2		Tp7		Tp8		Tp9		TP10	
Mn	P23	34	Z11	G16	R1	fz	vx	B4	GA	DV
MC	20	19	21	19	20	20	21	20	21	21
MCMS	102	110	97	109	100	100	97	94	101	92
MCDS	81	87	78	85	77	77	77	73	75	68
MS	61	68	57	66	57	57	56	53	54	47
MW	21	23	19	24	23	23	20	21	26	24
W	34.4	33.8	33.3	36.4	40.35	40.35	35.7	39.6	48	51
W%	34.1		34.85		40.350		37.65		49.5	

APPENDIX C: Atterberg Limit Determination

TP1.1	LL				PL		
can code	R1	G16	Z11		F4	T7	A3
MC	20	19	21		19.7	20.2	18.5
MCMS	37.1	34.8	33.5		26.1	26.1	25.9
MSDS	31.1	28.2	28		24.1	25.4	23.9
MS	11.1	9.2	7		4.4	5.2	5.4
MW	6	6.6	5.5		2	0.7	2
W%	54	71.7	78		45.4	13.5	37.03
No of drops	30	22	15	Average w%	32.02		



TP1.2	LL				PL		
can code	G1	GTZ	DH		F1	T4	B11
MC	20.2	20	11.9		12.4	12.4	11.8
MCMS	38.1	34.5	27.8		22.2	23.6	21.6
MSDS	31.2	28.8	20.8		19.3	20.1	18.5
MS	11	8.8	8.9		6.9	7.7	6.7
MW	6.9	5.7	7		2.9	3.5	3.1
W%	62.7	64.7	78.6		42	45.5	46.3
No of drops	30	20	16	Average w%	44.6		

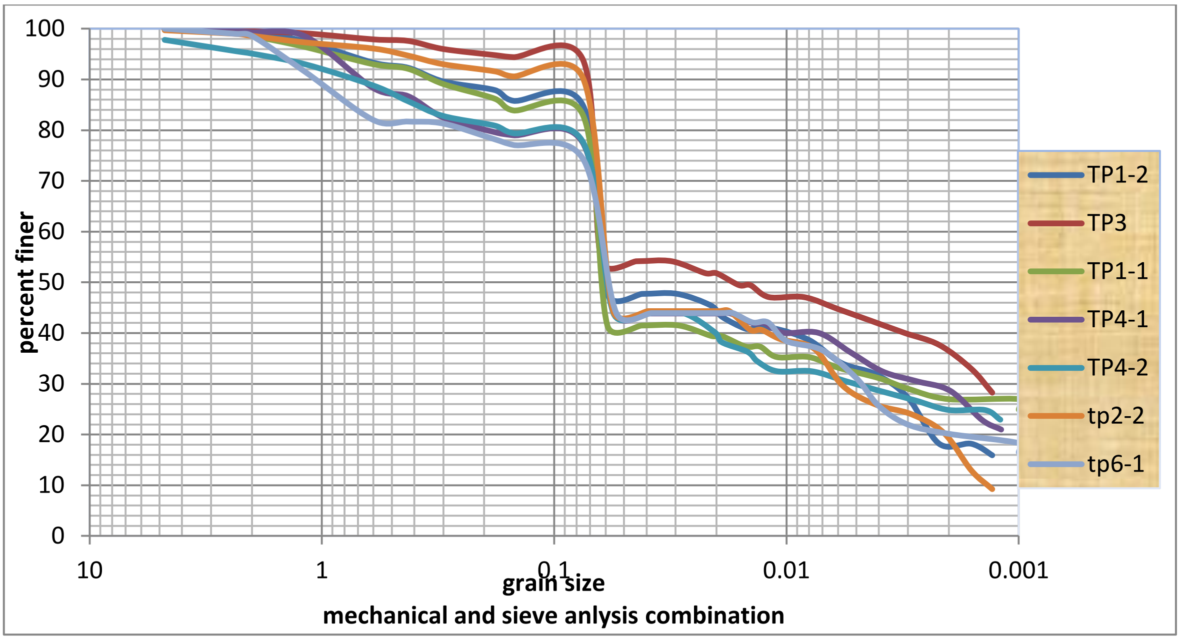


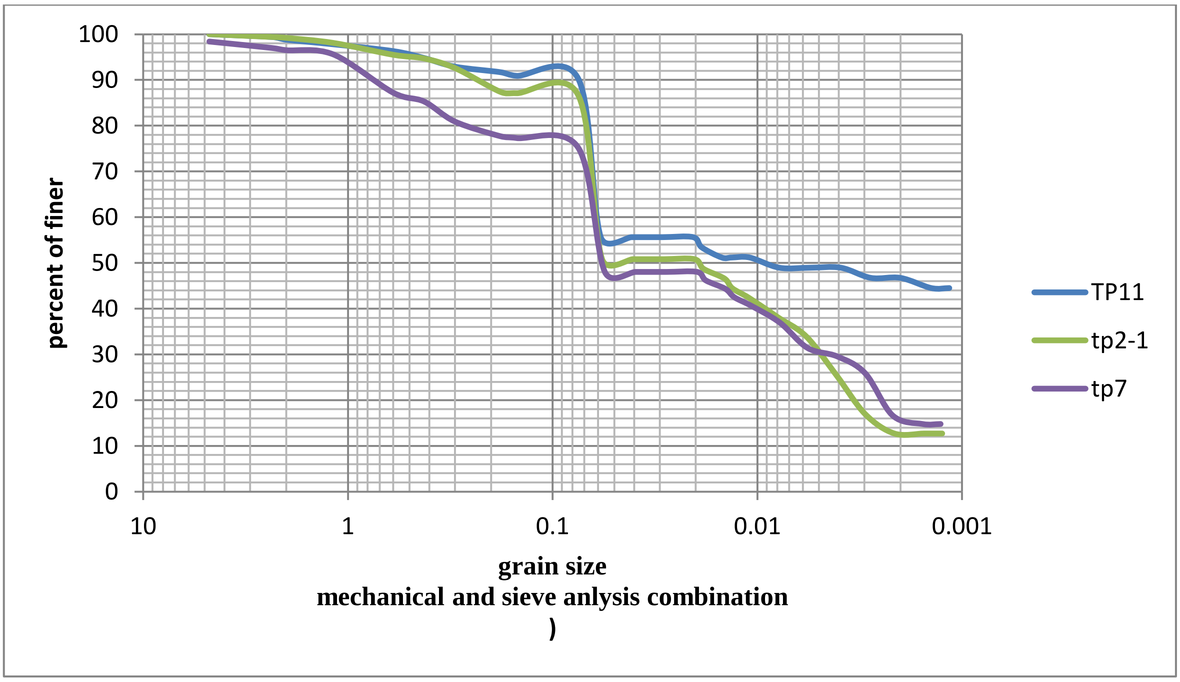
APPENDIX D: Grain Size Analysis Data

Tp1-1, 171gm soil remained after washed							
Sieve no	Diameter (mm)	Mi(g)	Msi+Rs (g)	Rs (g)	Pr%	Pc%	Pf%
	4.75	409	411	2	0.2	0.2	99.8
	2.36	497	503	6	0.6	0.8	99.2
	2	463	469	6	0.6	1.4	98.6
	1.18	449	471	22	2.2	3.6	96.4
	0.6	435	469	34	3.4	7	93
	0.425	373	382	9	0.9	7.9	92.1
	0.3	389	419	30	3	10.9	89.1
	0.18	348	377	29	2.9	13.8	86.2
	0.15	377	400	23	2.3	16.1	83.9
	0.075	360	370	10	1	17.1	82.9

TP1-1, Hydrometer analysis										
elapsed time, t, min	actual hydrmre.	composite correct	temp.(c)	corre. H.r	effe. Depth	coeff. K	CT	grain size	n' (%)	n' (%) combined
0.5	1.023	0.003	24	1.02	10.2	0.01301	1	0.059	50.1	41.49
1	1.023	0.003	24	1.02	10.2	0.01301	1	0.042	50.1	41.49
2	1.023	0.003	24	1.02	10.2	0.01301	1	0.029	50.1	41.49
4	1.022	0.003	24	1.019	10.5	0.01301	1	0.021	47.5	39.42
5	1.022	0.003	24	1.019	10.5	0.01301	1	0.019	47.5	39.42
8	1.021	0.003	24	1.018	10.7	0.01301	1	0.015	45.0	37.34
10	1.021	0.003	24	1.018	10.7	0.01301	1	0.013	45.0	37.34
15	1.02	0.003	24	1.017	11	0.01301	1	0.011	42.5	35.27
30	1.02	0.003	24	1.017	11	0.01301	1	0.008	42.5	35.27
60	1.019	0.003	24	1.016	11.3	0.01301	1	0.006	40.0	33.19
120	1.018	0.003	24	1.015	11.5	0.01301	1	0.004	37.5	31.12
240	1.017	0.003	24	1.014	11.8	0.01301	1	0.003	35.0	29.04
480	1.016	0.003	24	1.013	12.1	0.01301	1	0.002	32.5	26.97
960	1.016	0.003	24	1.013	12.1	0.01301	1	0.001	32.5	26.97
1440	1.015	0.003	24	1.012	12.3	0.01301	1	0.001	30.0	24.89

Tp1-2, 152gm soil remained after washed							
Sieve no	Diameter (mm)	Mi(g)	Msi+Rs (g)	Rs (g)	Pr%	Pc%	Pp%
4	4.75	409	410	1	0.1	0.1	99.9
8	2.36	497	499	2	0.2	0.3	99.7
10	2	460	463	3	0.3	0.6	99.4
16	1.18	449	468	19	1.9	2.5	97.5
30	0.6	435	477	42	4.2	6.7	93.3
40	0.425	373	383	10	1	7.7	92.3
50	0.3	389	416	27	2.7	10.4	89.6
80	0.18	348	365	17	1.7	12.1	87.9
100	0.15	377	398	21	2.1	14.2	85.8
200	0.075	360	370	10	1	15.2	84.8


100
90
80
70
60
50
40
30
20
10
0
percent finer
10
1
0.1
0.01
0.001
grain size
mechanical and sieve anlysis combination
TP1-2
TP3
TP1-1
TP4-1
TP4-2
tp2-2
tp6-1


100
90
80
70
60
50
40
30
20
10
0
percent of finer
10
1
0.1
0.01
0.001
grain size
mechanical and sieve anlysis combination
)
TP11
tp2-1
tp7

APPENDIX E: Specific Gravity

	Tp1-1			Tp1-2			Tp2-1		
pycnometer No.	1	2	3	1	2	3	1	2	3
weight of pycnometer	110.5	81.7	95	110.5	81.7	95	110.5	81.7	95
weight of pcnometer + water	359	330	343	357.5	329.5	342.2	359	330	343.3
temprature of water	24	24	24	24	24	24	24	25	25
weight of pcn + soil +water	374.1	345.7	358.4	373	344.9	357.5	374	345.1	359
weight of oven dry soil	25	25	25	25	25	25	25	25	25
specific gravity	2.53	2.69	2.60	2.63	2.60	2.58	2.5	2.53	2.68
average specific gravity	2.61			2.60			2.57		

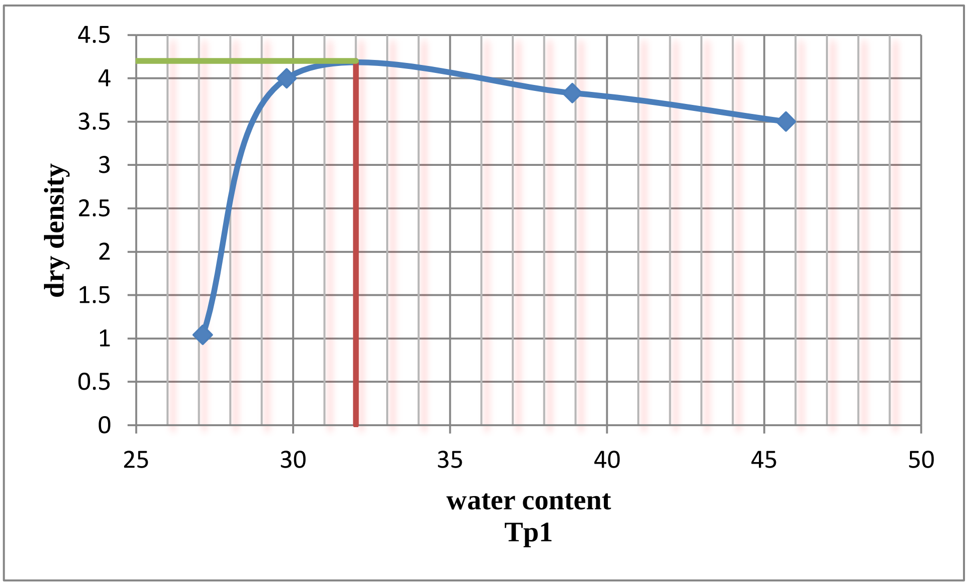
	Tp2-2			Tp3			Tp4-1		
pycnometer No.	1	2	3	1	2	3	1	2	3
weight of pycnometer	110.5	81.7	95	110.5	81.7	95	110.5	81.7	95
weight of pcnometer + water	358.8	329.8	343	358.9	329.5	343.3	358.2	329.5	343.8
temprature of water	24	25	25	24	24	24	24	24	24
weight of pcn + soil +water	374	345.1	358.4	373.9	345	358.6	373.8	344.9	359.5
weight of oven dry soil	25	25	25	25	25	25	25	25	25
specific gravity	2.55	2.58	2.6	2.5	2.63	2.58	2.66	2.60	2.69
average specific gravity	2.58			2.57			2.65		

	Tp4-2			Tp5-1			Tp5-2		
pycnometer No.	1	2	3	1	2	3	1	2	3
weight of pycnometer	110.5	81.7	95	110.5	81.7	95	110.5	81.7	95
weight of pcnometer + water	358.5	329.4	344.5	358.6	330.6	342.9	357	329	344
temprature of water	24	24	24	22	22	22	22	22	22
weight of pcn + soil +water	374.5	345	360	373.5	345.8	357.1	372	344	358
weight of oven dry soil	25	25	25	25	25	25	25	25	25
specific gravity	2.78	2.66	2.63	2.48	2.55	2.31	2.5	2.5	2.27
average specific gravity	2.69			2.45			2.42		

	Tp6-1			Tp6-2			Tp7		
pycnometer No.	1	2	3	1	2	3	1	2	3
weight of pycnometer	110.5	81.5	95	110.5	81.5	95	110.5	81.7	95
weight of pcnometer + water	358.3	329	342.2	358.4	330.1	343.3	358	329.9	343.1
temprature of water	25	25	25	25	25	25	22	22	22
weight of pcn + soil +water	373.7	344.5	358.1	374	345.5	359.1	373.9	345	358.9
weight of oven dry soil	25	25	25	25	25	25	25	25	25
specific gravity	2.60	2.63	2.75	2.66	2.60	2.72	2.75	2.53	2.72
average specific gravity	2.66			2.66			2.66		

APPENDEX F: Compaction Test

determination No, TP1	1	2	3	4
mass of mold	3396			
mass of mold+ compaction soil	4646.5	4905	5030.7	4838.1
volume of mold	944cm^3			
bulk density	1.32	5.20	5.33	5.13
dry density	1.04	4.00	3.83	3.52
MDD	4.2			
OMC	32			
container No	F64	D5	A3	F4
mass of container	20.2	19.5	19.7	21.6
mass of container +wet soil	93.3	97.9	96	99.3
mass of container + dry soil	77.7	79.9	74.6	74.9
mass of dry soil	57.5	60.4	54.9	53.3
mass of water	15.6	18	21.4	24.4
water content %	27.13	29.80	38.98	45.78
average existing water content		35.42		



APPENDIX G: Direct Shear Strength Test Result

Ring calibration factor = 0.0056433kn/div
Horizontal dial reading = 0.01
Specimen X- sectional area in m^2 = 0.0036

TP5

horizontal displacment RE. , h	In m	for 100kpa		for 200kpa		for 300kpa	
		Providing ring reading	Shear stress in kpa	in div	in kpa	in div	in kpa
0	0	0	0	0	0	0	0
50	0.5	19	29.784	10	15.68	32	50.16
100	1	25	39.190	13	20.38	42	65.84
150	1.5	32	50.163	16	25.08	47	73.68
200	2	38	59.568	19	29.78	54	84.65
250	2.5	38	59.568	27	42.32	59	92.49
300	3	35	54.865	32	50.16	67	105.03
350	3.5	35	54.865	45	70.54	100	156.76
400	4	36	56.433	51	79.95	100	156.76
450	4.5	35	54.865	67	105.03	90	141.08
500	5	34	53.298	54	84.65		
550	5.5			50	78.38		
600	6		Normal stress	0	100	200	300
650	6.5		Max shear stress	9.928	65.83	84.65	105.03
700	7		cohesion	9.928			
750	7.5		slope	0.486			
800	8		Fiction angle	25.929			
850	8.5						
900	9						
950	9.5						
1000	10