# Engineering geological characterization of Expansive Soils in NifasSilkLafto Sub-City Jemo area, Addis Ababa, Ethiopia

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**A Thesis Submitted to**

**School of Earth Science**

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**Addis Ababa University**

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Table of content

## LIST OF TABLES

## LI ST OF FIGURES

## LIST OF SYMBOLS AND ABBREVIATION

A= Activity

AASHTO= American Association of State Highway and Transportation Officials

USCS = Unified Soil Classification System

ASTM= American Society of Testing and Materials

FS= Free Swell

LL= Liquid Limit

LI = Liquidity Index

PL= Plastic Limit

$\omega$ = Natural Moisture Content

PI= Plasticity Index

Gs= Specific Gravity of Solids

CH= Inorganic clays of high plasticity

OH= Organic clays of high plasticity

CL = Inorganic cays of low to medium plasticity

MH = Inorganic silts of high plasticity

OL = Organic silts of low plasticity

$\rho$dry = dry density

Ps= Swelling Pressure

N = Number of samples

TP= Test Pit

# ABSTRACT

The present study was carried out in Addis Ababa NifasSilikLafto sub city Jemo area with the main objective of characterizing the expansive soils found in Jemo area.

Expansive soil in many parts of Addis Ababa poses significant damage light buildings. Therefore, identification, evaluation and characterization of them is a paramount importance to address the problems.

In the present study characterization of such expansive soil was adopted by identification and classification based on their laboratory results, correlation of swelling potential and compression indeces with index tests and correlating the laboratory derived and new calculated compression indices values with the well-known skemptons relationship.

In order to prepare a geotechnical data base a large quantity of drilling and laboratory testing data, have been filtered out and systematically organized to derive a new relevant correlation for the selected soil type in Microsoft excel. The output of the collected soil sample data is further checked by using five test pits and subsequent soil sampling and laboratory analysis of the most common index laboratory tests according to ASTM standard.

From the present study based on the available systematically organized drilling geotechnical data the thickness of expansive soil ranges from 3m to 16m in the study area. The depth profile generally tells that Dark grey silty CLAY/ clayey SILT as a top soil, Light grey silty CLAY/ clayey SILT, Clayey silty SAND having variable thickness. The soils are categorized as CH-clay with high plasticity (fat clay) and MH- CLAY according to the Unified Soil Classification System.

Finally based on the correlation between index and swelling pressure no well-defined relationship was found to occur between swelling pressure and index properties. Results of correlations and analysis of compression indices with Liquid Limit and plastic limit show that Cc values generally increases with increasing liquid limit and plasticity index of soils. This serves to suggest that compressibility of soils generally increases with increased plasticity.

*Key Words*, Characterization, expansive soils, swell potential, plasticity, compression.

# CHAPTER 1

# INTRODUCTION

## 1.1.Background

Expansive soils are high plastic clay soils. Expansive soil is a term generally applied to any soil that has a potential for shrinking or swelling when exposed to moisture change. This subsequent swelling and shrinkage of this soils due to change in moisture cause damages to civil engineering structures, particularly light buildings, and pavements (John and Debora, 1992). With increasing number of global population and related rapid urbanization and demand of new land for expansion of infrastructure, these soils pose significant problems for engineering structural.

Jones and Holtz (1973), stated that some soils such as expansive soils, are known to be problematic and require attention. Potentially expansive soils can be found virtually any place in the world.

According to Bell (1999), problematic soil is defined as a natural geo-hazard that is due to detrimental geotechnical properties of soil. Global damage to infrastructure and associated remediation costs are often of far-reaching economic consequences (Jones and Holtz, 1973).

## 1.2.Problem statement

The construction of buildings and other structures are developing fast in different towns of Ethiopia including Addis Ababa. Several governmental institutions, condominium houses and private business centers are established in Addis Ababa town. So, the need for detail geotechnical investigation of the sub-surface condition of soils has a paramount importance for the safe and economical design and construction activities.

Proper evaluation of the geological and geotechnical condition of expansive soils in this area is therefore highly important and this research is designed to address the characteristics of expansive soils in Jemo area.

### 1.3. Objectives of the study

#### 1.3.1. General objective

The general objective of this study is to assess the engineering geological characteristics of expansive soils in NifasSilkLafto sub-city, Jemo area, Addis Ababa.

#### 1.3.2. Specific objectives of the study

To meet the general objective, the following specific objectives are designed.

- ❖ To evaluate the nature of the soils in the study area
- ❖ To assess index and engineering properties of expansive soils in the study area.
- ❖ To determine swelling potential and expansive potential of expansive soil in the study area

### 1.4. Methodology

In order to meet the objectives of the present study, the following systematic methodology has been followed,

- Review of relevant literature such as: books, technical journal articles, academic thesis, and other related secondary materials about expansive soils,
- Desk study- collection of secondary data of the study area such as: topographic maps, geological maps, engineering geological maps, geotechnical investigation reports etc of the study area. Organization of data and preparation of geotechnical data base from secondary sources.
- Filed investigation: field data or primary data of the study area were collected using test pits.
- Post field activities: the data's collected from the field were thoroughly processed, analyzed, and interpreted to meet the objective of the study.

#### 1.4.1 Primary data collection

Primary data is collected from test pits to check whether the collected secondary data is correct or not. Logging and description of different geologic formations were done. From test pits, appropriate soil samples were collected.

The primary investigation of the study area was carried out based on 5 test pits. The location of the test pits is determined by taking into consideration the distribution of the existing secondary data and problematic sites. Two areas were particularly selected for the primary investigation based on observed damages (tilting of building and water Leakage). From the selected pitting areas subsurface exploration was performed by boring test pits up to 2.5 - 3 m depth. Both disturbed and Undisturbed (taken by using thin- walled Shelby tube) samples of soils were collected for laboratory testing. In the field GPS reading was taken to locate the coordinate of sampling sites. A detailed field investigation was performed on the site following the schedule.

Table 1.1 Field test pit description

| Location | Test pits | Easting and Northing | Visual discription of soil | Test pit photograph |
|---|---|---|---|---|
| | 1 | Easting 468309<br>Northing 990922 | Firm to stiff, Dark grey to Light grey, highly plastic, Silty CLAY soil |  |
| | 2 | Easting 468438<br>Northing 990368 | Firm to stiff, Dark grey to Light grey, plastic, Silty CLAY soil mixed with back fill material at the top. |  |
| Jemo | 3 | Easting 468020<br>Northing 990040 | Firm to stiff, Dark grey to Light grey, highly plastic, Silty CLAY soil with grass roots at the top. |  |
| | 4 | Easting 469017<br>Northing 990046 | Firm to stiff, Dark grey to Light grey, highly plastic, Silty CLAY soil mixed with back fill materials at the top. |  |
| | 5 | Easting 468373<br>Northing 990623 | Firm to stiff, Dark grey, highly plastic , Silty CLAY soil . |  |

### 1.4.2 Secondary data collection

Secondary data were collected with an intention to review different pre-existing geological, engineering geological, geotechnical reports and maps. The review was made to understand the investigation methods, analysis, and final interpretations for various building foundation sites. Besides, borehole data, soil properties and classification data were collected.

### 1.4.3 Laboratory testing

The following laboratory tests were conducted for this study to evaluate the soil properties from the study area.

- Specific gravity test
- Atterberg limit tests
- Grain size analysis
- Free swell test
- Natural moisture content test
- Swelling pressure test
- Consolidation test

Laboratory test procedures of all the above tests were done according to the American Society for Testing Materials (ASTM) standard.

## 1.5 Scope and Limitations of the Study

This thesis has been intended to address the engineering geological characteristics of expansive soil in the Jemo area. Generally, this study is focused on engineering geological properties of expansive soils in Jemo area. The present study was mainly constrained from limited number of sampling points and samples for laboratory testing and analysis. The study was also contained from limited number of primary data (test pits, sampling) since the test pits were conducted by a human labor force.

Lacks of well-organized secondary data were also parts of constraints for this research work. The other limitation was availability of systematic secondary data.

## 1.6 Organization of the thesis

The present study was conducted by compiling geotechnical data from historical borehole log reports and laboratory test results, engineering geological maps, ongoing projects and visual observations in study area. To present the results of the research in a systematic manner, it is divided into five chapters and the scheme of presentation is as follows:

**Chapter 1** comprises the background, problem statement, objectives of the study, methodology, scope and limitation of the study and organization of the thesis.

**Chapter 2** presents a description of the study area including geographical location, climate, geology, topography and drainage conditions, soils, hydrogeology and reginal and local geology of the study area.

**Chapter 3** presents the literature review that comprises a brief description of the theoretical basis and previous works relevant to the present research.

**Chapter 4** presents the characterization of the expansive soil that describes, detailed description of index and engineering properties, classification of expansive soils, correlation of swelling pressure with index test.

**Chapter 5** presents the overall conclusions and recommendations that are forwarded through the present research work.

# CHAPTER 2

# GENERAL OVERVIEW OF THE STUDY AREA

## 2.1. Location and accessibility of the study area

The study area is located in NifasSilkLafto Sub City of Addis Ababa in Jemo area. The study area is geographically bounded in between 989100m N-990900m N latitude and 465600m E- 468800m E longitude of UTM Zone 37N with an aerial coverage of about 7.7km2. The study area is accessible in asphalt, gravel, and cobble stone roads.

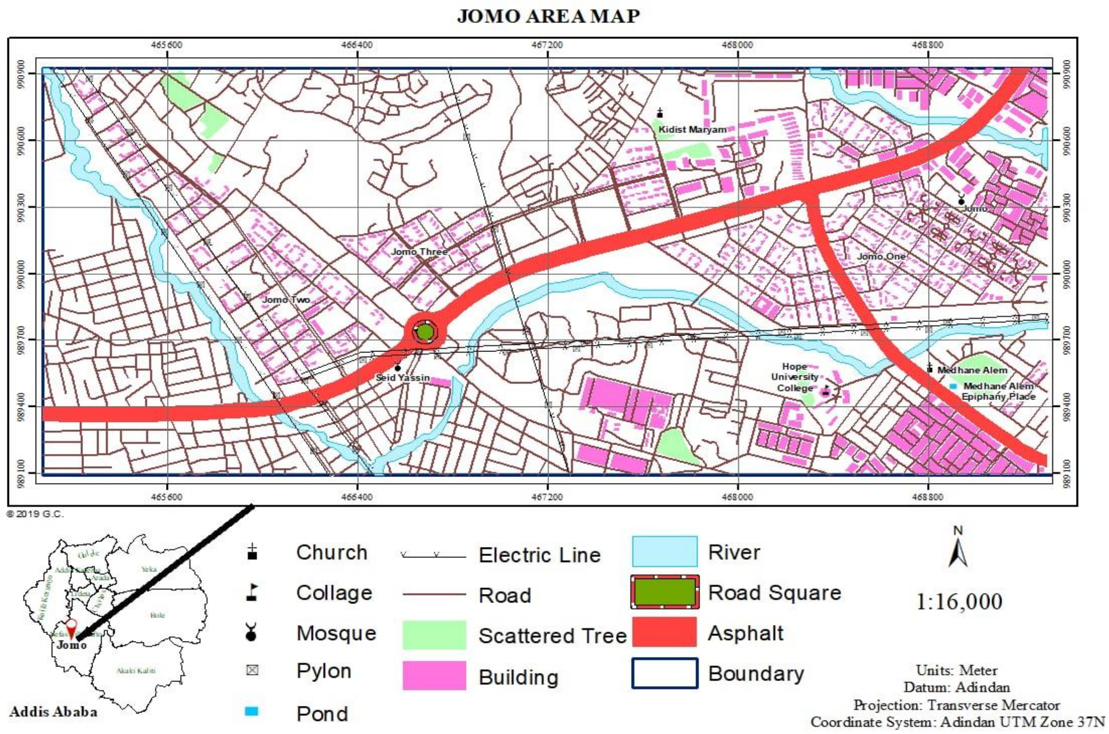


Figure 2. 1 Location map of the study area

## 2.2.Climate

Climate is the principal factor governing the rate and type of the soil formation. The two most significant factors of climate are precipitation and temperature. Ethiopia is classified into five climatic zones (EMA, 1981 as cited in Habtamu Solomon, 2011). These include "Kur" (Alpine), above 3000m mean sea level; "Dega" (Temperate), 2300m to about 3000m; "Weina Dega" (Sub tropical), 1500m to about 2300m; "Kolla" (Tropical), 800m to about 1500m and "Bereha" (Desert), less than 800m.

Climatic patterns tend to produce different soil types depending on the degree of weathering on parental rocks responsible for the formation of flat topography in the study area. Nifas silik lafto sub city generally has a sub-tropical climatic condition.

The mean monthly maximum and minimum temperature records of National Meteorological Services Agency (NMSA) stations in Addis Ababa located at Addis Ababa Akaki station (the nearest station for the study area) for the years between 2000 and 2018 were utilized to calculate monthly and annual average temperature and presented in appendix 3.

As can be observed in Figure 2.2 the highest monthly average maximum temperature occurs in the months of March with 28.3$^0$C and the lowest is in the month of August with 11.32$^0$c. April and May are usually the hottest and driest months in the area.

According to Ethiopian National Meteorological Agency (ENMA, 2000-2018) (Akakai station) the highest mean annual rainfall recorded was 239.51 mm in the month of August 2016. The main rainy season is from June ‘to September.

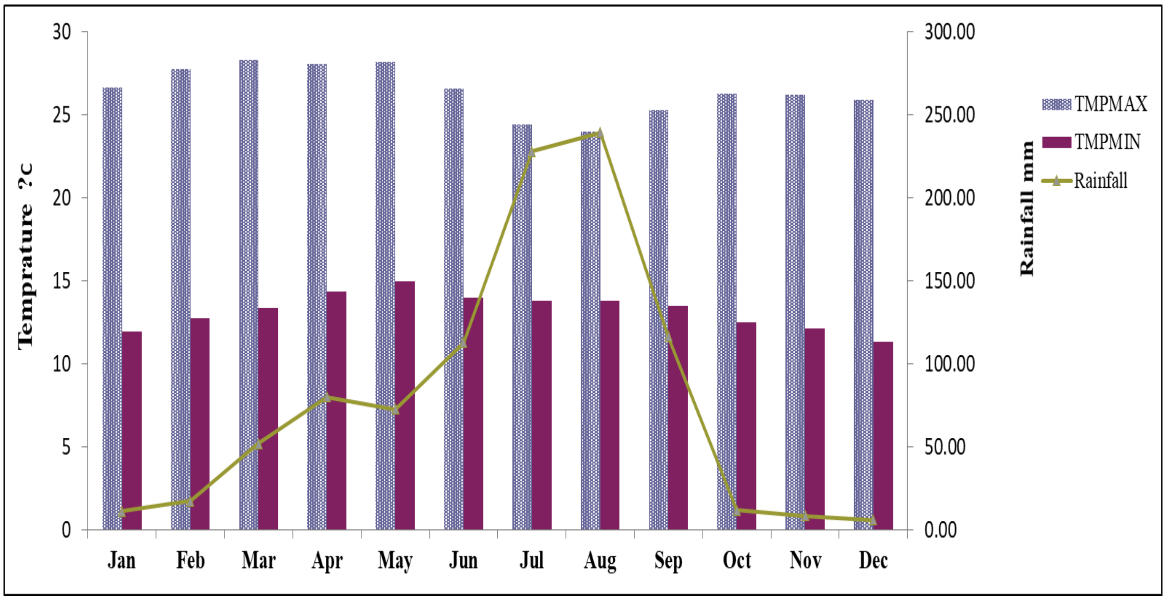


Figure 2. 2 Average (mean) monthly minimum and maximum temperatures and rainfall (2000-2018) (Source: National Meteorological Agency of Ethiopia)

### 2.3 Topography and Drainage characteristics

Formation and behavior of soils is governed by geology, environmental condition, hydrological condition and more by topography. Topography controls the rate of weathering by partly determining the amount of available water and the rate at which it moves through the zone of weathering (Selamawit, 2017). In addition to this, it also controls the effective age of the profile by controlling the rate of erosion of weathered material from the surface. Thus, deeper residual profiles will generally be found in valleys and gentle slopes rather than on high ground or steep slopes (Blight, 1997).

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Soil transportation needs an important agent this is called flowing water/stream. Fast moving running water carries a large quantity of soil either in suspensions or by rolling along the bed. Deposition of the soils in the valleys is possible when water erodes the hills. The size of soil particles carried by water depends upon the velocity. The fast-moving water can carry the particles of large size such as boulders and gravels. The coarse particles get deposited following the decrease in velocity. The finer particles are carried further downstream and are deposited when the velocity reduces (Arrora, 2004). According to Arrora, 2004 a delta is formed when the velocity slows down to almost zero at the confluence with a receiving body of still water such as a like, a sea or an ocean.

The study area is in a flat laying area and is bounded with wide chain of hillsides. All the water drainage of the hillsides is concentrated in the bottom plains which is difficult for accessibility in the area. The morphological setting of the area is dominantly flat and has an implication that the area is flood plain and is covered by expansive soils. It is discussed in literature review (chapter 3) that expansive soil is very common in low laying area.

The area is mostly characterized by flat to sloping degree with a maximum degree of 40°(Fig 2.3) topographic feature, though it looks a wide flat area, the topography has a gentle downward slope with limited steep slope.

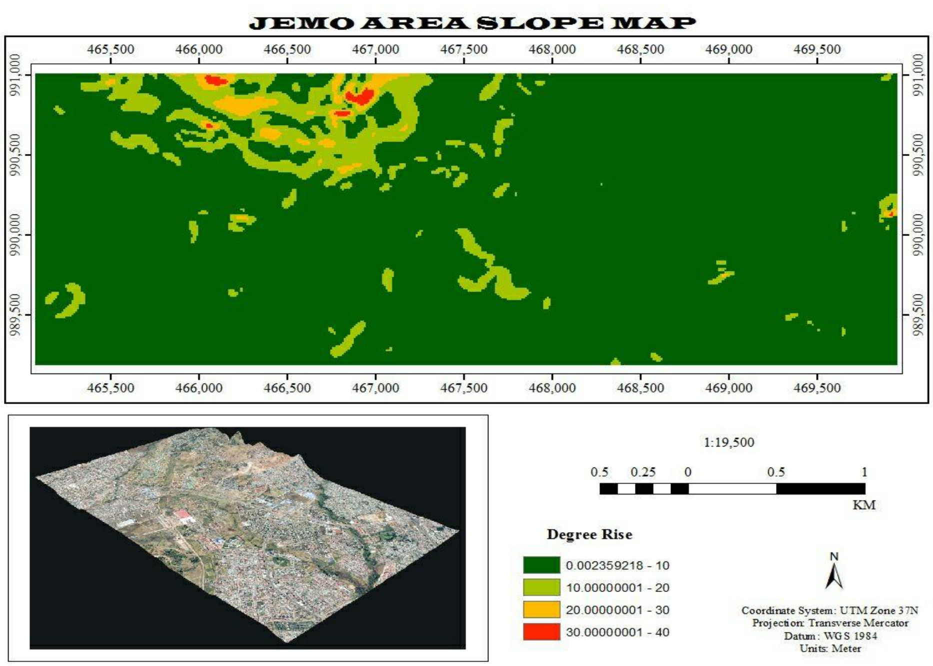


Figure 2. 3 Slope map of the study area

The drainage map (Fig 2.4) of the area reveals that the major river that drains with in the study area is FURI DARFIRO (Hirbu) including many stream tributaries joins towards the major river.

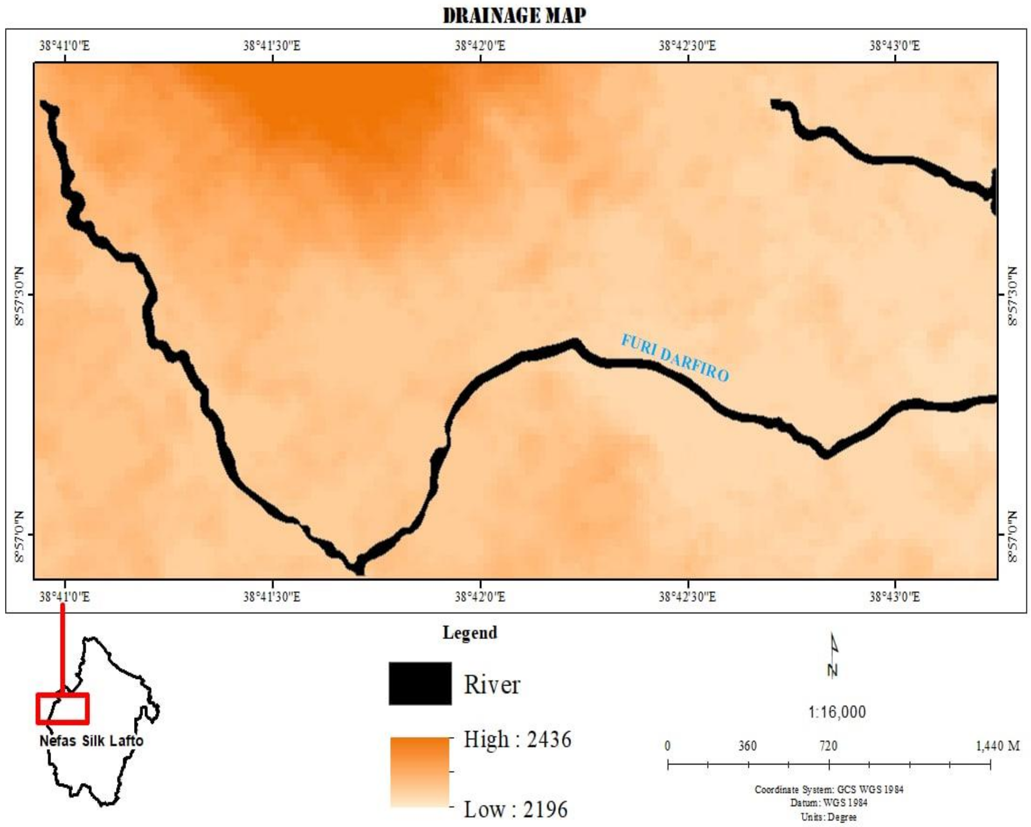


Figure 2. 4 Drainage map of the study area

The topography, accompanied with the drainage condition of the study area creates a suitable environment contributing to the development of expansive soils. In low lying area (Study area) where surface drainage is poor and often water logged, dark colored (dark grey) soils are dominant. In general, it seems like the topography and drainage of the study area controls the type and distribution of soils.

### 2.4 Hydrogeology

According to Kebede T. and Tadesse H. (1990), most of the aquifers are confined below the clay (paleosoil) and hence storage coefficient is very low. Where the weathering is very high, the clay horizon is reached, and the productivity becomes very poor.

Hydrogeological studies as part of engineering geological and geotechnical investigations are very crucial as sub-surface water is often a critical factor in various engineering works. Almost every engineering project like, buildings, highway pavements, airport runways, dams, underground structures, structures on rock and soil slopes may be affected in one way or other by the subsurface water, which may lead to failure of such projects, thus causing loss of life and property (Kebede Tsehayu and Taddesse Hailemariam, 1990).

According to a study conducted by Tamiru Alemayehu et al. (2006) the major ground water aquifers in Addis Ababa are basalts, rhyolities, trachytes, scoria, trachy basalts, welded tuffs, unwelded tuffs and the unconsolidated materials of volcanic origin as depicted from boreholes previously drilled for water supply. The main aquifers in Addis Ababa area can be categorized in to three groups which include shallow aquifers of the weathered volcanic rocks and alluvial sediments along the river courses, deep aquifers of the fractured volcanic rocks that tap fresh ground water and thermal aquifers along Filwoha fault (Tamiru Alemayehu et al., 2006). These aquifers are characterized by fracture and inter granular porosity. Basalts are the major water bearing zone in the area due to its fracture porosity whereas unconsolidated volcanics and alluvial sediments under favorable conditions stores water. The black cotton soils in the south of Addis Ababa act as impervious material (Sisay Alemayehu, 2004). Tamiru Alemayehu et al. (2006) also determined the general direction of groundwater flow in Addis Ababa based on elevation of water level in boreholes drilled for water supply. According to this study the ground water movement direction is dominated by North-South and South-East flow. In some localities however the ground water flow direction changes, mostly towards the nearby streams and generally, the groundwater movement is sub-parallel to the surface water flow direction and more or less controlled by the topography of the area.

The annual groundwater recharge is estimated to be about 51-100mm for Akaki catchment (Engida Z.A 2001). He stated that at Akaki well field (south of the city), the groundwater level is found about 20 to 30 meters down from the bottom bed of Akaki River (Fig 2.5).

According to Engida (2001), the aquifers at the southern part of the city (Akaki well field) are mainly young volcanic rocks of lava flow and tectonic fractures. In general, the aquifers are complex and highly variable. The thickness of the aquifers is not yet determined.

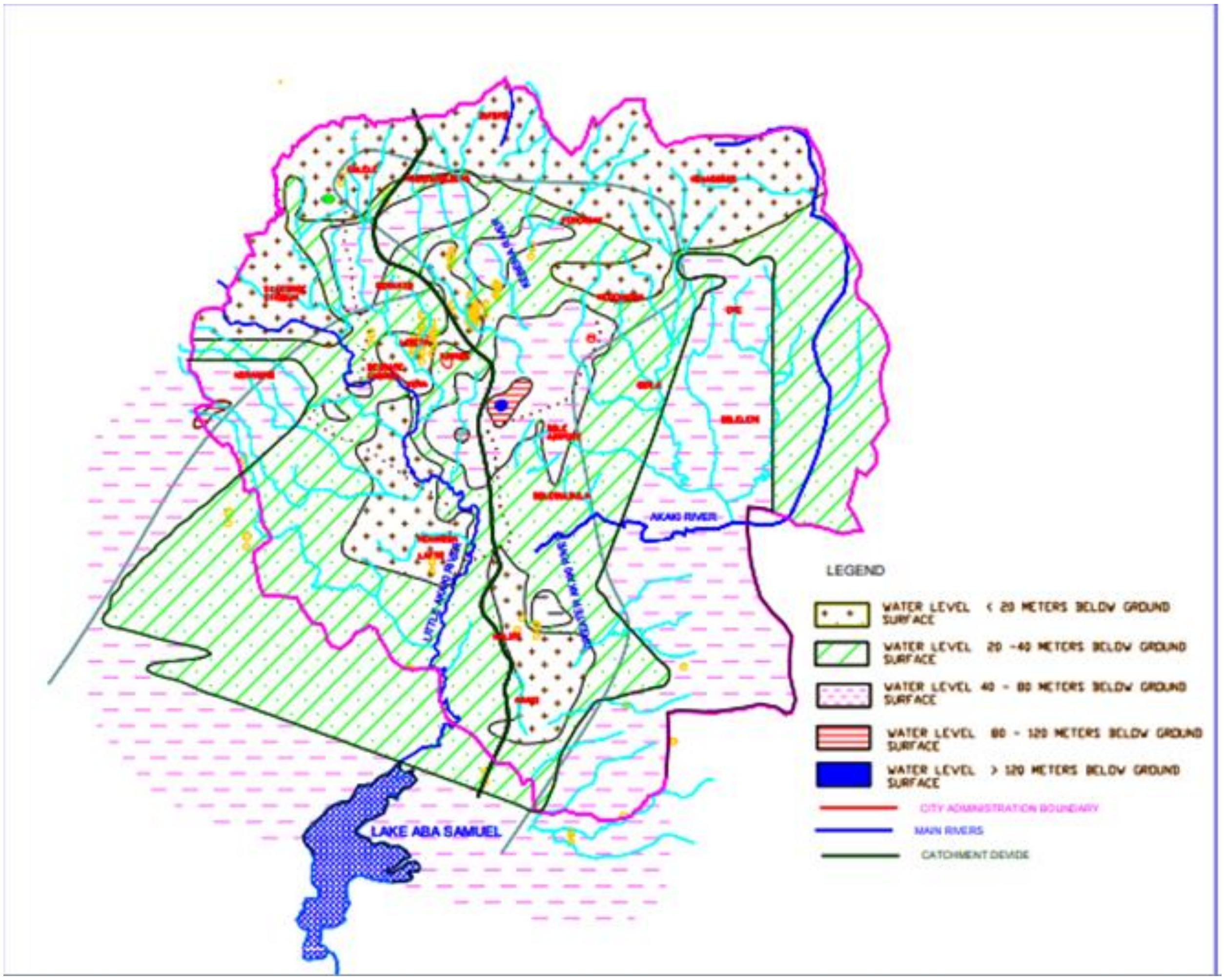


Figure 2. 5 Ground water depth of Addis Ababa area (Source Engida Z.A 2001)

Engida Z, 2001 recommended that borehole drilling for Akaki station, since the thickness of the clay deposit is very large. The borehole will be drilled with the objective of determining the thickness of the clay, conduct in-situ permeability at different depths, and use it a monitoring well. Therefore the groundwater affects the expansive soil since they have swelling and shrinkage behavior.

## 2.5 Soil

Alluvial, alluvial fan, colluvial, residual and lacustrine soils are more specifically the engineering geological soil unit in Addis Ababa and are grouped into their genetic soil units (Kebede Tsehayu and Taddesse Hailemariam, 1990) (fig 2.5).

The soil development in Addis Ababa area is mostly due to the physical disintegration and chemical decomposition of volcanic rocks. The weathering products are either remaining in places and form residual soils or transported and deposited in the low-lying flat lands and depressions (Tamiru Alemayehu et al., 2006).

The differences observed in the type and development of soils in the city depends mostly on the topography, climate, parent rock and the degree of weathering.

In the localities where the topography is plain to gentle there is thick soil profile. The type of parent material and the length of time to which the parent material is subjected to weathering, control the variation in the thickness of soil. Thus, old basic and acidic rocks that outcrop in the central, western and southwestern parts of Addis Ababa are weathered and form thick soil profile. In places where young basalt and welded tuffs occur, the thickness of the soil cover is reduced (Tamiru Alemayehu et al., 2006).

The deterital materials that are derived from elevated areas of Entoto, Wechecha, Furi and Yerer are transported and deposited in the piedmont and along the stream courses of Addis Ababa. It covers most parts of Mekanisa, Ayere Tena, Kaliti, Akaki, Lideta, and Bole. The soil is black in color and the thickness varies from place to place primarily depending on the slope of the area. More specifically the engineering geological soil unit in Addis Ababa area are grouped into their genetic soil units as alluvial, alluvial fan, colluvial, residual and lacustrine soils (Kebede Tsehayu and Taddesse Hailemariam, 1990) (Fig.2.6).

The alluvial soils which include channel and terrace deposits are found in some places along Akaki River in the west and southwestern parts of Addis Ababa and along Kebena River north of Bole area. The alluvial soils consist of more or less stratified deposits of gravel and clay transported by streams (Kebede Tsehayu and Taddesse Hailemariam, 1990).

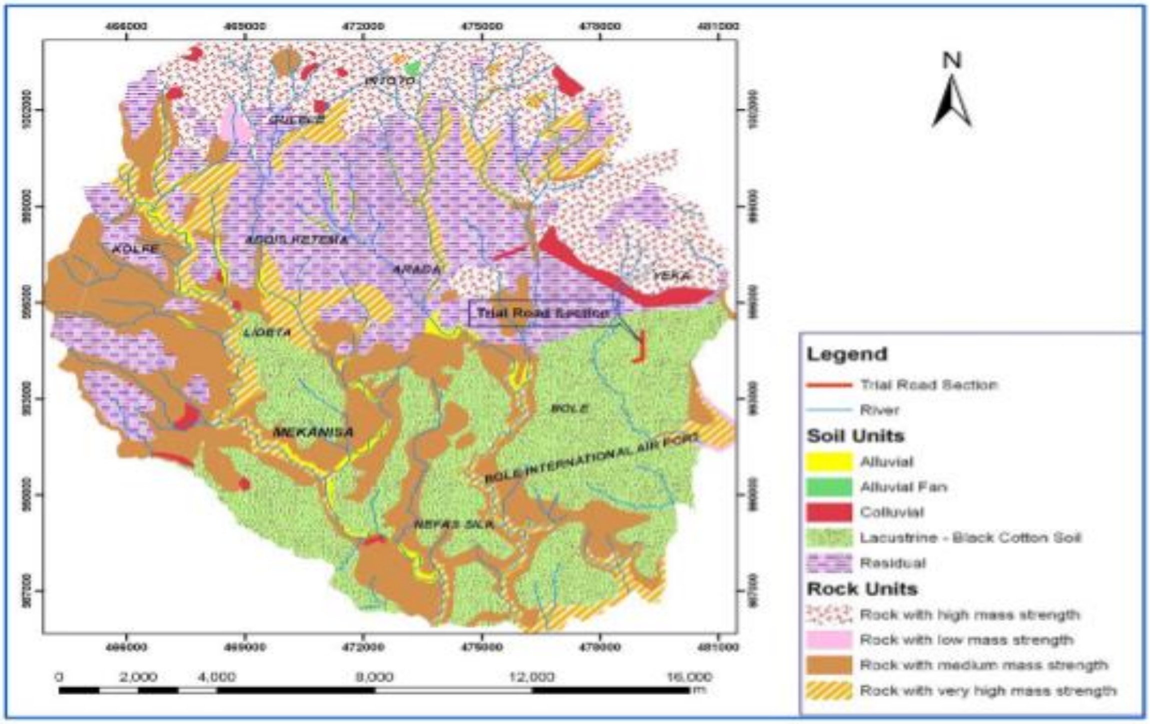


Figure 2. 6 Engineering geological map of Addis Ababa (Kebede Tsehayu et al., 1990)

Alluvial fan is deposited where there is a decrease in gradient from a hill to a plain along a river section. It is coarser near the mouth of the river and become finer outwards and found in the Entoto region dissected by deep gullies (Kebede Tsehayu and Taddesse Hailemariam, 1990).

### 2.5.1. Geotechnical soil layers

Basic site investigation and detailed field analysis was carried out since the present work mainly aims at the engineering geological characterization of the expansive soils of the study area. Based on such field investigations and by accompanying laboratory investigations summery of the results presented in appendix 1. General geotechnical layers of the primary data and the collected secondary data of the study area, in depth wise from top to bottom are listed as follows as per BS 5930 soil logging standard,

- Firm to stiff, Dark grey, highly plastic silty CLAY/clayey SILT soils
- Stiff to very stiff, Light grey, non to high plastic, Silty CLAY/clayey SILT soils
- Dense to very dense, Light brown to grey, clayey silty SAND soils

#### 2.5.1.1.Firm to stiff, dark grey, highly plastic, silty clay/clayey SILT

It is characteristically found in the low laying areas of the study area up to a maximum depth of 8.95m. The common occurrence of thick layer of such soil may be related to paleomorphology of the area relation to the surrounding regions. And this layer is the upper most of all clay layers in the area.

The soil, close to the surface, is highly cracked and usually is firm to stiff. The gray layer is mottled (secondary soil colors not associated with compositional properties of the soil) with black. The effect of mottling generally increases with depth and it indicates poor leaching characteristics, (Leulseged A, 1990).

#### 2.5.1.2 Medium stiff to stiff, light grey, silty CLAY/clayey SILT soil

The light grey silty clay soils cover most part of the study area. They are characteristically founded at relatively high lands bounding the clay soils of relatively low laying areas up to a maximum depth of 16.45m on some areas and they are medium stiff to stiff, high plastic soil layer. They are mostly light grey in color indicating that the soil is the decomposed product of the welded tuff and ashes.

Usually, they are underline by dense to very dense, light grey to light brown clayey silty SAND/ gravelly sand and boulders of welded tuff soil layers.

#### 2.5.1.3 Dense to very dense, light brown to grey, clayey silty SAND soils

This layer is dense to very dense, low to non-plastic with some amount of gravel mixture. It is encountered overlying the Welded TUFF rock and found up to a maximum depth of 29.10m. From this observation it is possible to suggest that they may be the gradational boundaries between the rock and the clay soils which are the weathering product of the parent rock.

The clayey silty SAND is light gray, dark brown to light brown in color. There is some light yellowish type of this layer, which is mixed with some gravel. In areas where this type of soils present, swamps and bogs are common which are totally grassed. Since, they are found along river terraces, they are used for crop plantation.

Generally, the dark grey silty CLAY soil is underlain by light silty CLAY and clayey silty SAND soils which is mixed with some gravel. The clayey silty SAND material is a residual soil originated from completely decomposed trachte basalt which indicates that they are the weathering product of the parent rock.

The basic igneous rocks (the basalts), which are made up of calcium rich feldspars and dark minerals which are high in the weathering order and the pyroclastic sediments (the tuff and the ashes) which are made up of volcanic glasses, under suitable conditions grade to form clay minerals. Therefore, according to Luelseged A. (1990) the retention of magnesium and the calcium from parent rocks, evaporation which exceeds precipitation, and the poor leaching conditions, all prevail the formation of dark grey clay soils

Figure 2. 7 pictures of building affected by expansive soil

Soil of the study area can be generalized by a single soil type i.e. residual clay soil following the geology setting of the area. In principle shrinkage and swelling are the main properties of residual clay soil when the dry and wet respectively which is the property of expansive soil.

The pictures (Fig 2.7) are taken from the study area during field work of the study. These pictures are evidences which explicitly indicate the presence of the expansive soils in the area and taken in the beginning of the rainy season when moisture variation is supposed to be nearly minimal.

**2.6 Regional geology**

Addis Ababa city is situated in the western margin of the Main Ethiopian Rift and represents a transition zone between the Ethiopian Plateau and the rift with poorly defined escarpment Tamru Alemayehu et al., (2006). The stratigraphy given by Mortor et.al, (1979), and Hayleselase and Getaneh, (1989) is shown in fig. 2.8.

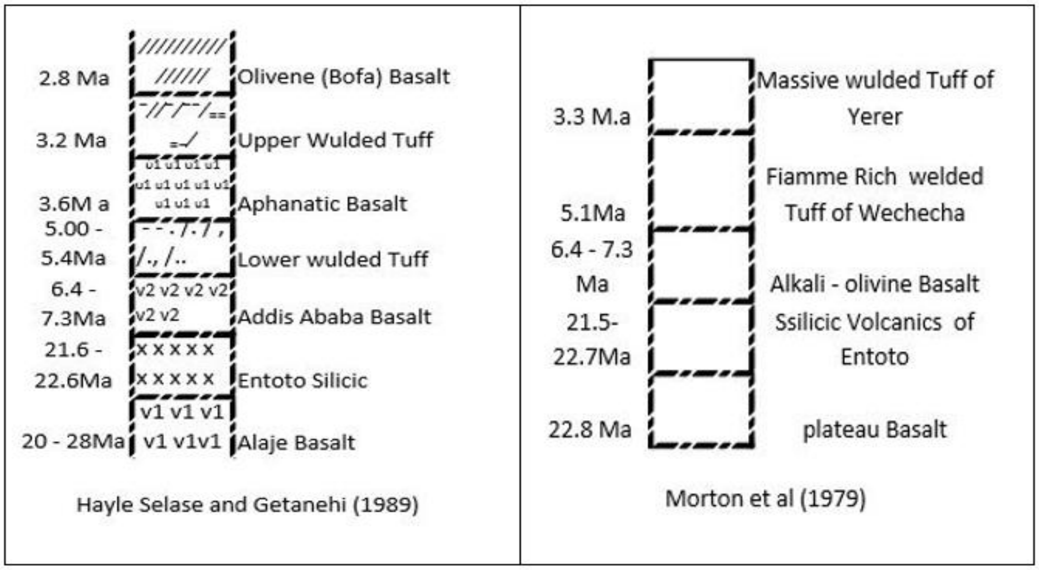


Figure 2. 8 The stratigraphy of the volcanic rocks in the Addis Ababa and the surrounding area (not scaled) from Leulseged A. 1990)

According to the (HayleSelase and Getaneh, (1989), the sequence of the rock units revels the Miocene Pleistocene volcanic succession ranging from older plateau volcanic to younger rift volcanics. The Miocene-Pleistocene volcanic succession in Addis Ababa area suggested by Haile SellasieG and Getaneh A, 1989), from bottom to top, are Alaji basalts, Entotosilicics, Addis Ababa basalts, Nazareth group, and Bofa basalts.

### 2.6.1 Alaji Basalt

Alaje basalt is grouped in volcanic rocks (Alaji rhyolite and Basalt). This unit is composed of basalts, which show variation in texture. Within this unit, there is an intercalation of gray and glassy welded tuff. According to Haile Sellasie G. and Getaneh A, 1989, the outcrop of Alaji basalt extends from the crest of Entoto ridge bordering the northern parts of Addis Ababa towards the north. It is difficult to determine the stratigraphic relationship with the Entotosilicices since they are found in a fault contact. Mohr (1967, as cited in Tamru A et al., (2006) proved that the Entoto trachyte overlies the Alaji basalt.

### 2.6.2 EntotoSilicics

Rhyolites and trachytes with minor amount of obsidian rich tuffs are the components for the formation of the Entotosilicics (Leulseged A, 1990). According to Morton et al., (1979), these early Miocene aged silicic volcanic could represent localized terminal episodes to massive Oligocene fissure-basalt activity in the Addis Ababa region. The thickness of the flow becomes maximum on the top of Entoto ridge and thin both towards the plateau and the plain east of Addis Ababa.

According to Zanettin and Justin (1974, as cited in Tamru A et al.,2006) these lavas make up a thick pile of flows accumulated along east west fissures (east west fault running from Kassam River to Ambo) and up lifted north wards.

It also outcrops in the eastern part of the town from the KokebeTsebah School to the British Embassy. The thickness is quite variable as it frequently forms dome structure. In this rock unit, flow banding, folding, and jointing are common. The rhyolites are overlain by feldspar porphyritic trachyte and underlain by a sequence of tuffs and ignimbrites. Tuffs and ignimbrites are welded and characterized by columnar jointing (Assegid G, 2007).

### 2.6.3 Addis Ababa basalt

This rock unit is mainly outcropping at the northern and north-western part of Addis Ababa extending from the southern flanks of Entoto hills to the Filwoha region. It is composed of olivenrich and plagioclase rich sub- units having porpyratic in texture with phenocrysts and oliven and augite respectively (Leuelseged A, 1990).

The ground mass is made of Andesite, labradorite, oliven and magneteite(Haile SellasieG. and Getaneh Assefa, 1989). The Addis Ababa basalt is overlying by Welded Tuffs to the south of the Filwoha. Addis Ababa basalt yield ages clustering around 7my and seems to have no time/composition equivalent (Morton et al., 1974as cited in TamruA et al., 2006).

### 2.6.4 Nazaret Group

The units identified in this group are denoted as Lower Welded Tuff, Aphanitic basalt and Upper Welded Tuff. The group is underlain by Addis Ababa basalt and overlain by Bofa basalts. The rocks outcrop mainly south of Filwoha and extend towards Nazaret(TamruA et al., 2006; Mohammed, 2007).

#### 2.6.4.1. The lower welded tuff

It outcrops as small discontinuous body in Filwoha, western parts of Addis Ababa and Sululta. Generally, it is overlain by the aphanitic basalt and underlain by the olivine and plagioclase prophyritic basalt. The age of this rock unit, as dated by Morton et al. (1979, as cited in TamruA. et al., 2006), at Addis Ababa and Sululta is 5.1 and 5.4 million years, respectively. This age overlaps with the period of the activity of Wachecha trachyte volcanoes, dated 4.6 million years. Wachecha is located 15km west of Addis Ababa and probably the sources of the Lower welded tuff at both localities (Morton et al., 1979 as cited in TamruA et al., 2006; Kabite, 2011).

### 2.6.4.2 Aphanitic basalt

This basalt is vesicular in texture and filled with calcite. It is composed of fine grains of plagioclase, clinopyroxene and magnetite (Haile SellasieG. and Getaneh A, 1989as cited by leuleseged A.1990). The type of locality of this rock unit is mainly around the Bole international airport and in the upper reaches of Akaki river valley.

It is underlain by the plagioclase and olivine porphyritic basalt and overlain by the younger ignimbrite from which it is separated by tuffs and agglomerates. Its relationship with the rocks of the group is not clear, but probably younger than the aphanitic basalt (GetanehA. et al., 1989 as cited in TamruA et al., 2006; Kabite, 2011).

### 2.6.4.3 The upper welded tuff

This rock outcrops all over the southern part of the town including Bole, Nefas Silk Railway station; nevertheless, the central and northern parts of the town is also covered by this unit. It is gray colored, vertically and horizontally jointed and composed of orthoclase, quartz, pumice and unidentified volcanic fragments (GetanehA et al., (1989). The welded Tuff is underlain by aphanitic basalts and overlain by young olivine basalts. An age determination made on a sample collected close to Haile garment resulted 3.2million years, that overlap with the activity of Yerer trachytic volcano ‘s (Morton et. al., 1979 as cited in TamruA et al., (2006).

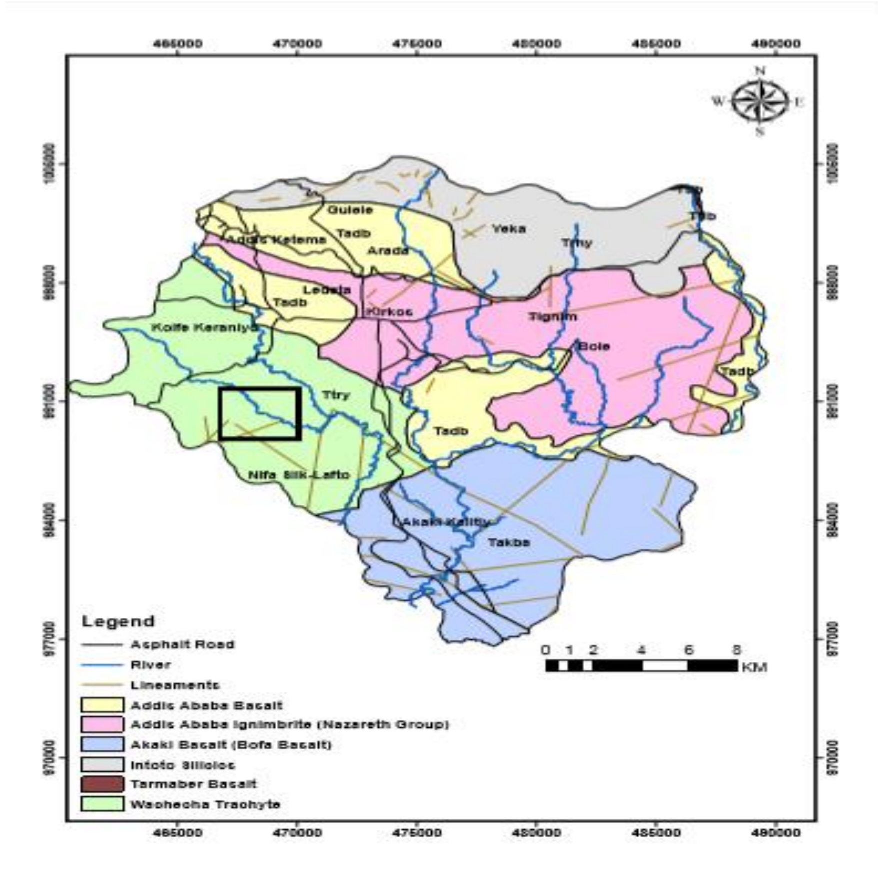


Figure 2. 9 Geological map of Addis Ababa (after WWDSE. 2008)

### 2.6.5. Bofa basalt (akaki basalt)

Bofa Basalts outcrop southward from Akaki River where these rocks appear in the form of boulders reaching a thickness of up to10 m. These rocks are restricted and dominated in the southern and southeastern part of the city. This rock is underlain by the tuffs which cover the welded tuff (Haile Sellasie Girmay and Getaneh Assefa, 1989 as cited in Zeleke Tadesse, 2013). It is coarse grained porphyritic olivine basalt. It is highly vesicular basalt and at places the vesicles were filled by carbonate minerals.

## 2.7 Local geology

### 2.7.1 Lithological description

The study area is covered by thick highly plastic clay over acidic rock (Trachite and Tuff) mixed with slope wash. The rock unit underneath the topsoil is Trachite and volcanic tuff. The area is surrounded by small hills on the north part, and the study area is in the depression topographic zone. Though it looks a wide flat land, the topography has a gentle downward slope to the " FURI DARIFURI or locally  HiRBU" river side. Small gullies are noted while traversing the area. And it is strongly correlated with regional geology of Addis Ababa especially with the Yerer units, which consists of Trachyt and pyroclastics.

**Summery**

- The study area is geographically bounded in between 989100m N-990900m N latitude and 465600m E- 468800m E longitude of UTM Zone 37N.
- The morphological setting of the area is dominantly flat and has an implication that the area is flood plain and is covered by expansive soils which indicates that expansive soil is very common in low laying area.
- The area is mostly characterized by flat to sloping degree with a maximum degree of 40° topographic feature, though it looks a wide flat area, the topography has a gentle downward slope with limited steep slope.
- The drainage map of the area reveals that the major river that drains with in the study area is FURI DARFIRO (Hirbu) including many stream tributaries joins towards the major river.
- Akaki well field (south of the city), the groundwater level is found about 20 to 30 meters down from the bottom bed of Akaki River.
- . General geotechnical layers of the primary data and the collected secondary data of the study area, in depth wise from top to bottom are listed as follows as per BS 5930 soil logging standard,
    - Firm to stiff, Dark grey, highly plastic silty CLAY/clayey SILT soils
    - Stiff to very stiff, Light grey, non to high plastic, Silty CLAY/clayey SILT soils
    - Dense to very dense, Light brown to grey, clayey silty SAND soils
- The highest monthly average maximum temperature occurs in the months of March with $28.3^{0}$C and the lowest is in the month of August with $11.32^{0}$c. April and May are usually the hottest and driest months in the area.
- The rock unit underneath the topsoil is Trachite and volcanic tuff.

# CHAPTER 3

# LITERATURE REVIEW

## 3.1 Introduction

Soil, a natural property of earth which is formed from small pieces of mineral particles and it may contain water, air and organic materials. It supports every construction which is designed by human such as buildings, retaining walls and embankments. There are many types of soil. The main soil groups are granular soil, fine-grained soil and organic soil. The behavior of soil is an important element which always concerned in civil engineering. The properties such as the plasticity, compressibility, or strength of soil always affect the design in construction. Geotechnical properties and behaviors of soil can be determined by several tests such as odometer test and Atterberg limits test. The properties and characteristics of soils vary laterally or vertically from place to place. Sometimes rough assessment of the engineering properties without conducting tests is common in professional geotechnical engineers since the tests required for determination of engineering properties are generally time consuming. This may be easy if index properties are determined. According to Adem (2006), index properties are properties of soils which are not of primary interest to the geotechnical engineer, but they are indicative of the engineering properties.

This chapter presents literature survey on expansive soils. Discuss previous relevant studies including their employed methodology and their outputs with site description and research problems by justifying the selected method in relevance to the site conditions and the problems.

## 3.2 Identification of expansive soils

A major concern in geotechnical engineering is identification of expansive soils, either in the field or in laboratory, and estimation of their swelling magnitudes when subjected to changes in environment. So that problems posed by the expansive soil can be solved (Al-Rawas and Goosen, 2006).

By field observations, mainly during reconnaissance and preliminary investigation stages, soils that can exhibit high swelling potential can be identified (Chen, 1988; Nelson and Miller, 1992). During dry seasons, cracks reaching up to with the maximum width of 20 mm or more and are visible on the ground surface (Murthy, 2009). During rainy seasons, these soils become very sticky and very difficult to traverse. Appearance of cracking in the nearby structures is also indicative (Murthy, 2009).

### 3.2.1 Field identification

Some of the important field identification methods that indicate the potential for expansiveness of soil which are explained by (Chen 1966, as cited by Abdirshekur Kemal, 2015) are the following: -

- A shiny surface is easily obtained when a partially dry piece of the soil is polished with a smooth object such as the top of a fingernail.
- The wet sample of the soil is sticky, and it is relatively difficult to clean the soil from the hands.
- The appearance of cracking in nearby structure.
- They usually have a color of black and/ or grey.
- In the regions where there is seasonal moisture variation there is a presence of joint or similar discontinuity, slickenside (highly polished or glossy fissure surface) and shattering or micro-shattering, (presence of fissures forming granular fragments of clayey soils) may observed.

Chen, (1966) stated that expansive soils can be recognized by using mineralogical identification, indirect index property tests or direct expansion potential tests and expansively of a soil is governed by the type and proportion of clay minerals it contains. Also, knowing the type and proportion of the clay mineral in a soil gives a clue on the swelling potential.

### 3.2.2. Laboratory identification

By using laboratory identification methods expansive soils can be identified in three different methods-

- ❖ Direct measurement
- ❖ Mineralogy
- ❖ Indirect measurement

#### 3.2.2.1 Direct measurement

It is a convenient and more reliable test because it immediately tells the likely in situ response of the ground for moisture variations since, it directly measures the force per unit area that a swelling soil exerts on any structure resting on it.  The test can be done using a conventional one-dimensional Consolidometer (Chen, 1988; Nelson and Miller, 1992), which is available in most soil mechanics laboratories.

#### 3.2.2.2 Mineralogy

There are mineralogical identification methods by using the mineralogy of clay particles, such as: X-ray diffraction, Differential thermal analysis, Dye absorption, Electron microscope, cation exchange capacity, and so on (Chen, 1988; Nelson and Miller, 1992). Nevertheless, they are not suited for routine testing as they are time consuming, require expensive test equipment and, the results are interpreted by specially trained technicians.

#### 3.2.2.3 Indirect measurements

This method is used to investigate the swelling potential of expansive soils by examining other parameters, which indirectly gives information about the soil properties (Asuri, 2016). Indirect property tests (Grain Size Analysis, Atterberg Limit, linear shrinkage and Free Swell tests) are some, among the routinely conducted index property tests. The other tests the cation exchange capacity (CEC) and the potential volume change (PVC) are included in this method.

## 3.3 Classification

### 3.3.1 General Classification

A systematic method of categorizing soils into various groups and subgroups according to their probable engineering behavior without detailed descriptions is known as soil classification.

The systems that are quite popular amongst engineers are the AASHTO Soil Classification System, the Unified Soil Classification System, the ASTEM classification system and the system of classification proposed by International Association of Engineering Geology (IAEG). The Unified Soil Classification System (USCS) and International Association of Engineering Geology (IAEG) are the important methods to classification soils for geotechnical investigation of foundations while AASHTO soil classification system is used to classify soils in the case of highway (Murthy, 2002). On this thesis for the primary data’s ASTM classification system is used.

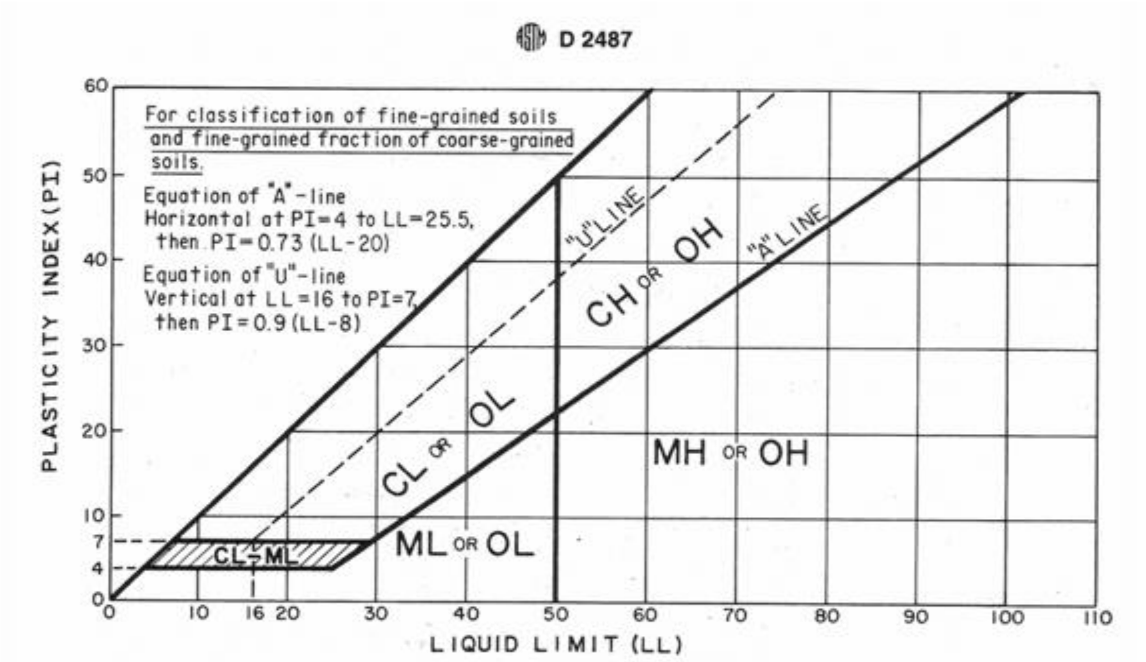


Figure 3. 1 Classification of fine-grained soils on plasticity chart

### 3.2.2 Classification specific to expansive soils

General classification system does not provide useful information; rather it may give an initial alert that the soil may have expansive character. A parameter determination from the expansive soil identification tests have been combined in several different classification schemes to give qualitative rating on the expansiveness of the soils.

It is very important to emphasize that design decision has to be based on predicting, testing and analysis, which can provide reliable information. Therefore, the direct use of such classification system as a basis for design may lead to an overlay conservative construction in some palaces and inadequate construction in some places.

#### 3.2.2.1 Classification based on swelling potential

Direct methods provide actual physical measurements of swelling. Several laboratory methods have been developed to directly determine the soil swelling as moisture content changes. Even an expansive soil may never swell, if its moisture content stays constant at all times, but it is obvious that soils undergo moisture content change for a variety of reasons (Tomilson, 1980). When an increase in moisture of expansive soils occurs, it causes the soil to expand (swell) and heave.

The phenomena may results in expansive soils for the following reasons (Gromko, 1974; Hunt, 1984; Hunter, 1988; Murphy, 2010):

- Rain fall and rise in the ground water table
- Reducing load condition, such as surcharge loads increases the swell
- Transmission of moisture with time; moisture transmission through soil is slow and requires weeks and even years to saturate depending upon the permeability and thickness of stratum
- Dry density, dense clays will swell more when they are wetted than the same clay at lower density with the same moisture content
- Mineral type and amount, soils containing a considerable amount of montmorillonite minerals will exhibit high swelling and shrinkage characteristics

One of the simpler procedures is the free swell test (Holtz and Gibbs, 1956). A small sample (10 cm3) of dry soil passing the No. 40 sieve is added to a graduated cylinder and filled with water. The free swell is determined by comparing the initial volume with the final volume. Soils having free swell values greater than 100% are considered potential problems, whereas soils with free swell values below 50% probably do not exhibit appreciable volume changes.

According to Asuri K, (2016) Holtz and Gibbs had not advised any specific values of free swell to classify the soil. The mass of the soil contained in 10cm3 is variable and depends on personal judgement are the limitations described by Holtz and Gibbs. By using free swell Asuri K. suggests four classes of clays.

| Free Swell (%) | Swell Potential |
|---|---|
| <50 | Low |
| 50-100 | Medium |
| 100-200 | High |
| >200 | Very High |

| Activity | potential of expansion |
|---|---|
| <0.75 | Low (Inactive) |
| 0.75 - 1.25 | (Normal) |
| >1.25 | High (Active) |

An indirect prediction of swelling potential includes correlation and combination based on index properties. Some of such classification systems are Skemptons method (Mckeen, 1976) and Seed, woodward and Lundgreen. One of the index test parameter Activity is used in this method developed by combining Atterberg limit and clay content. Activity is defined as the ratio of the plasticity index to percent of clay fraction finer than 2ηm. by using the activity Skemptons suggested three classes of clays.

Plasticity index is a parameter which can be used as a preliminary indicator of the swelling characteristics of a soil. Based on the plasticity index, Seed,woodward and Lundgreen suggested three classes of clays.

| Plasticity index | Swell potential |
|---|---|
| 0-10 | Low |
| 10-35 | Medium |
| 20-55 | High |
| 55 and above | very High |

### 3.4 Swelling pressure

The most important laboratory test on expansive soils is the swell test (FU HUA CHEN, 1975). The standard one-dimensional consolidation test apparatus is similar to that used in most soil laboratories for consolidation studies. This test is defined as it is the actual pressure required to keep the volume of swelling soil. Swelling pressure, SP, of clays is the pressure required to prevent swelling by constraining the clays to maintain constant volume when saturated with water (Nelson and Miller, 1992).

This swelling pressure has serious consequence in the form of cracks and distress on the structures founded on expansive soils. Light weight structures are severely affected due to high swelling pressure created by the expansive soils. For an engineering construction activity on expansive soils, the first step is to assess the degree of expansiveness and the likely swelling pressure on the structures.

Swelling pressure is also another important engineering property of expansive soils. Swelling Pressure is the amount of pressure a soil exerts upon swelling (Budhu, 2011). The most reliable means of measuring swelling pressure is laboratory determination using one-dimensional consolidometer. This method is called direct measurement.

Swelling pressure (SP) of clays is the pressure required to prevent swelling by constraining the clays to maintain constant volume when saturated with water (Nelson and Miller, 1992).

All the methods for determining swelling pressure use an oedometer with varying conditions of allowing the soil to undergo swelling. It is true that through the laboratory, using the oedometer, one can determine the amount or degree of swell and swelling pressure. To determine the swelling pressure of the expansive soils many methods are developed by various researchers. From those methods only three methods have been standardized and popularly used as documented in the literature as referred by ASTM, (1995), which are

### 3.4.1 The swell – consolidation/swell lode method.

In this method an undisturbed sample can absorb water under a load of 1psi (7kpa) and is put aside to fully expand and reach equilibrium. Then it is consolidated by increasing the applied pressure in intervals following the conventional consolidation test procedure. The load increment is continued until the sample reaches its initial volume (zero volume change). The load correspond to zero volume change is taken as swelling pressure.

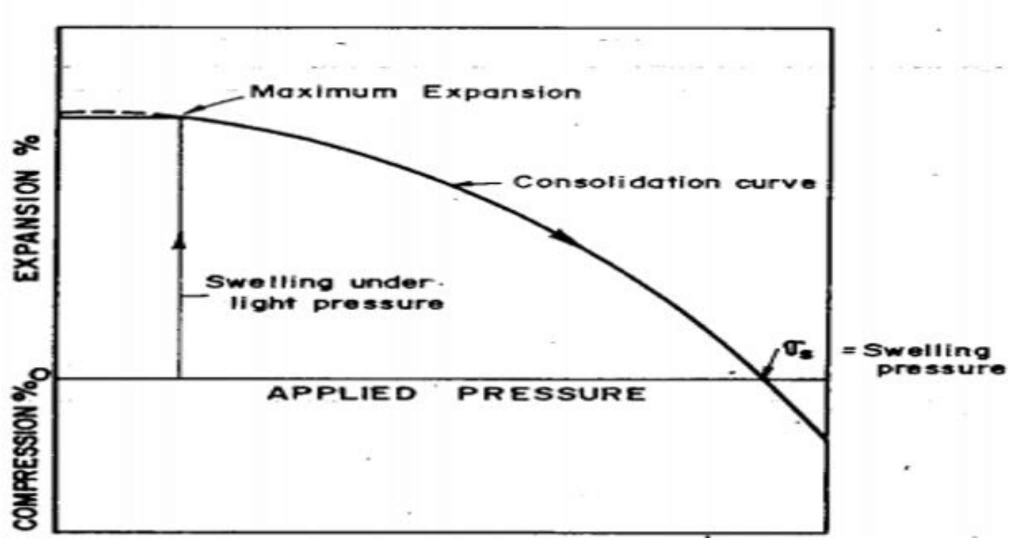


Figure 3. 2 Determination of swelling pressure

This method is quite popular and many investigators have used this method to establish a relationship between swell and applied pressure and to evaluate swelling pressure. The most serious drawback of this method is that it does not represent the normal sequence of load submersion. In the field the soils are first subjected to the structural load and then swell later following exposure to moisture but not vice versa.

### 3.4.2 The constant volume method /zero swell method

The specimen in the constant volume method can absorb water without any increase in volume by increasing the applied pressure as the test proceeds until the sample reaches equilibrium. The more load is added to keep the volume of the sample constant while the sample absorbs water. The swelling pressure can be determined by plotting the applied pressure against change in volume. This method does not represent the in-situ condition where the applied load, after the structure is in service, does not change with time. When the swelling process occurs, a constant pressure acts rather than different pressure which increase with time to a counter act the swelling process. Information such as the amount of heave which could be expected under application of a certain load or load which could be applied to limit the heave within tolerable limit cannot be furnished by this method. The method needs uninterrupted monitoring for a long period.

### 3.4.3 The method of equilibrium void ratio at different consolidation pressures

In this method several identical samples (three or more) are loaded with different loads close to expected swelling pressure. The samples are set aside to consolidate in dry state. When equilibrium is reached, then they are subjected in water and the subsequent swell is recorded. The swelling practically ceases when the samples are again left until they reach equilibrium. The pressure corresponding to zero volume change is taken as the swelling pressure.

The different pressure method has the same sequence about loading wetting events as in the field. That is, the loads are firstly applied to unsaturated soil and the soil undergoes compression under these loads. Then, after a period, the soil meets the source of water and an upward movement starts.

## 3.5 Index properties

Index property is a property that helps in distinguishing the characteristics of a soil. By nature, soils occur in a large verity. Soils having similar behavior can be particularly grouped together. Currently professionals are searching simplified tests that will increase their knowledge of soils beyond that which can be gained from visual examination without having to resort to the precision required with engineering soils (Zemene Muche, 2019).

Many tests and methods have been developed or modified for estimating swelling potential. Indirect methods involve the use of soil properties and clay content to estimate swell potential indirectly. The change in moisture contents (Atterberg Limits) of a soil sample can be used to indicate the degree for potential swell as presented in Table 3.1. A soil sample with liquid limit exceeding 70% and plasticity index greater than 35% is judged to have a very high potential swell (Holtz and Gibbs, 1956). Atterberg limit test, grain size analysis, specific gravity and free swell tests are among the tests which show the index property of a soil (Bowles, 1984).

Table 3. 1 Classification of potential swell

| Liquid limit (LL), % | Plasticity Index (PI), % | Classification of potential swell |
|---|---|---|
| 20-35 | <18 | Low |
| 35-50 | 15-28 | Medium |
| 50-70 | 25-41 | High |
| >70 | >35 | Very high |

### 3.5.1. Grain size analysis test

Grain size is one of the index properties tests. To classify a soil property grain size distribution is required. Sieve analysis is the means that grain size distribution of coarse-grained soil is generally determined. For a fine-grained soil, the grain size distribution can be obtained by means of hydrometer analysis (Murthy, 2003).

In general, a soil sample may contain both coarse grained particles as well as fine particles and hence both sieve and hydrometer analysis may be necessary. The sieve analysis is, however, the true representative of grain size distribution since the test is not affected by temperature. (Punmia et al, 2006).

On the other hand, grain size divides soil into two distinctive groups, namely cohesion less and cohesive soil. Soil particles, which are coarser than 0.075 mm, are generally termed as cohesionless and the finer ones like silt and clay are considered fine grained.

The property of cohesionless soil is greatly based on grain size distribution while the property of fine-grained soil is influenced by interparticle force. Hence, the behavior of a soil mass is dependent on the size of the particles it has. It is quite necessary then, to clearly know the proportion of different grains or particles a soil system contains. Sieve analysis for coarse-grained soils. For fine-grained soils another method called hydrometer analysis is used for this purpose.

### 3.5.2 Natural moisture test

Moisture content is defined as the ratio of mass of the water in a specimen to the mass of solids in the dry sample. Shear strength, hydraulic conductivity, compressibility and unit weight of the soil can be correlated with moisture content. Moisture content is important for interpretation of moisture-density relationships and forms the basis of Atterberg Limit testing (Ranjan, 1993).

The change in water content in an environment plays a major role in determining the degree of swelling and shrinkage. A desiccated expansive soil has higher affinity for the absorbance of water and with the higher affinity the more swell it exhibits (Abdirshkur, 2015).

### 3.5.3 Atterberg limit test

Plasticity index (PI) and Liquid Limit (LL) and plastic limit (PL) are demonstrated by Holtz and Gibbs (1956) for determining the swelling characteristics of most clay. Atterberg proposed the limits: Liquid limit (LL) and plastic limits (PL) and Shrinkage limit (SL), of consistency in an effort to classify the soils and understand the correlation between the limits and engineering properties like compressibility, shear strength and permeability (Afework, 2004).The consistency of a fine-grained soil is the physical state in which it exists; it relates largely to the water content. The soil exhibits different consistencies namely, solid, semi-solid, plastic and liquid states.

Liquid limit (LL) is defined as the moisture content at which soil begins to behave as a liquid material and begins to flow on the application of a very small sharing force. It is the water content at the boundary between liquid and plastic state. Plastic limit (PL) of a soil is the water content at the boundaries between the plastic and semi-solid state. Additionally, plastic limit of a soil is the range of water content over which the soil behaves particularly.

Plasticity index (PI) is the range of water content which exhibits plasticity, and the presence of clay can be possibility determined from plasticity index. Seed, Woodward, and Lundgren have demonstrated that the plasticity index alone can be used as a preliminary indication of swelling characteristics of most clay. From the Atterberg limit values plasticity index can be determined by using the following formula: -

$$\mathbf{PI = LL - PL}$$

This entire limit represents the water holding capacity of different states of consistency. But the most useful classification data for identifying the relative swell potential are the plasticity index (PI) and the liquid limit (LL). They are useful indicator for determining the swelling characteristics of most clay soils.

### 3.5.4 Activity

Activity is one mines of classifying expansive soils based on their index properties and used to estimate the swelling potential of given clay. It is defined as the ratio of the plasticity index to percent of clay fraction finer than 0.002ηm. Skempton, established useful empirical relationships between expansion potential and physical properties of soils such as colloids contents (clay contents), soil activity, plasticity index etc.

| Activity | potential of expansion |
|---|---|
| <0.75 | Low (Inactive) |
| 0.75 - 1.25 | (Normal) |
| >1.25 | High (Active) |

### 3.5.5 Specific Gravity

Specific gravity is defined as the ratio of mass in air of a given volume of soil particles to the weight in air of equal volume of distilled water at standard temperature. According to Das 2007, as a general guide, some typical values for specific soil types are as follows; -

| Types of soils | Range of specific gravity |
|---|---|
| Inorganic soils | 2.60-2.90 |
| Sand particles (Composed of quartz) | 2.6-2.66 |
| Inorganic clay | 2.70-2.90 |
| Porous particles (Large amount of organic matters) | <2.60 |

In residual soils the specific gravity may be unusually high or low (Hayilemariyam (1990). It is thus essential that the specific gravity be determined in the laboratory using an accepted standard test procedure.

The specific gravity of a soil solid is used in calculating the phase relationships of soils, such as void ratio and degree of saturation and it is used to calculate the density of the soil solid which is naturally occurring mineral particles or soil like particles that are not readily soluble in water by multiplying its specific gravity by the density of water (ASTM D 854 – 00).

### 3.5.6 Free Swell

The free swell test is one of the most used simple tests for estimating soil swelling potential which is first proposed by Holtz and Gibbs (1956. This test is performed by pouring 10cc of dry soil, passing through sieve no 40 (0.425mm diameter), into a 100-cc graduated cylinder. The cylinder is then filled with distilled water and the swelled volume of the soil is measured after the material settles within 24 hours. According to Holtz & Gibbs (1956) the amount of free swell serves to be indicative of the probable swelling and/ or expansive behavior of clay soils. It is considered as a measurement of volume change in clay upon saturation. It is therefore, always essential to investigate the swelling or expansive nature of these soils which are likely to posse's undesirable expansion characteristics. The common range of free swell is listed below taken from (Zemene Muche 2019): -

| Range | Free swell (%) |
|---|---|
| Expansive | >100 |
| Marginal | 50-100 |
| Non-Expansive | <50 |

According to (Sridharan and Prakash 2000) this method has merits on characterizing the expansive soils which are,

- The test procedure is very simple and user friendly.
- It does not require any sophisticated instrumentation and interpretation.
- The type of principal clay mineral(s) composing the soil can be predicted quite accurately. The test results have been validated with oedometer swell tests.

### 3.6 Engineering properties

The behavior of every geotechnical investigation depends primarily on engineering characteristics of the underlying materials (Budhu, 2011). The most important engineering properties of soils for foundation analysis are strength (cohesion and internal friction), and compressibility properties (such as the compressibility index).

#### 3.6.1 Strength properties

The resistance to mass deformation developed from a combination of particle rolling, sliding, and crushing is known as soil strength. This resistance to deformation is the shear strength of the soil as opposed to the compressive or tensile strength of other engineering materials. It is reduced by any pore pressure that exists or develops during particle movement. According to Day (2006) shear strength of soil is one of the most important aspects of geo technical engineering. Direct shear test and unconfined compressive strength (UCS) is the most common soil shear strength test. From those tests direct shear test is discussed and analyzed in this thesis work.

Direct shear tests are usually carried out to measure and determine the shear strength of soils, by sliding one portion of a soil relative to another. The shear strength is measured in terms of two soil parameters: inter-particle attraction or cohesion C, and resistance to inter-particle slip called the angle of internal friction (∅)(Gilloth, 1962). According to Terzaghi and Peck (1967) the cohesion, C, is the component of shearing resistance due to internal forces holding the particles together in a solid mass; while tanφ is the component of shearing resistance due to interlocking of soil particles and friction between them when subjected to normal stress.

### 3.6.2 Compressibility

The property of a soil due to which a decrease in volume occurs under compressive force is known as the compressibility of soil. When a soil mass is subjected to a compressive force, its volume decreases. Compression of solid particles and water in the voids, compression and expulsion of air in the voids and expulsion of water in the voids are the factors for the compression of soils.

Among the compressibility properties, compression index is by far the most important engineering property to estimate settlement of foundations. It is defined as the slope of void ratio versus logarithm of the applied load curve in one dimensional consolidation test graph (Budhu, 2011).

Consolidation and compaction are two basic compressibility characteristics of soils. But only consolidation is discussed in this thesis work.

## 3.7 Expansive soils in Ethiopia

### 3.7.1 Origin

Ethiopia is one of the countries with extensive coverage of expansive soil. Properties of the expansive soils vary from place to place due to topography, climate, geological history and formation. Therefore, it is important to make localized study for the different regions.

The origin and mineralogical composition of Ethiopian clay soil has been studied by Morin and parry, 1971 as stated by Samuel T. (1989).

Ethiopian black clay soils have formed over Tertiary to recent basaltic volcanic rocks. They are found in areas with poor drainage and low to moderate rainfall and contain Montmorillonite as the principal clay mineral with accessory Kaolinite and Halloysite (Morin and Parry, 1971). According to the same authors, the black clay soils are principally residual, derived from the weathering of basic volcanic rocks, which cover much of the Ethiopian plateau.

### 3.7.2 Distribution

Distribution of expansive soil is generally a result of geological history, sedimentation and local climatic conditions. Arid climatic conditions and severe weathering environment prevailing in north eastern part of Africa promote the widespread occurrence of expansive soils in this region and the areal coverage of expansive soils in Ethiopia is estimated to be 24.7 million acres source (BantayehuUba, 2017).

In Ethiopia, it covers nearly 40% (Fig 3.3) surface area of the country as discussed by (BantayehuUba, 2017). Expansive soils are observed in area such as central Ethiopia, following the major trunk road like Addis Ababa - Ambo, Addis Ababa - Weliso, Addis Ababa –DebereBerehan, Addis Ababa - Gohatsion, Addis Ababa - Mojo. Also, it covers the area like Mekelle, Bahirdar, Gambela, Arba Minch and the most Southern, South-west and south-east part of the capital Addis Ababa area in which the most major recent construction are being carried out.


Figure 3. 3 Distribution of expansive soil in Ethiopia

According to Alemayehu T. And Solomon Y. (1986), the black and grey soils found in the eastern and southern part of Addis Ababa are highly expansive and there is no distinction between the heaving characteristics of the grey and black soils. The clay content of the soil is found to be as high as 80% and the amount of montmorillonite for Addis Ababa expansive soil is 70-80%. These soils can hold significant amount of water that affects the shear strength, as well as shrinkage and swelling characteristics of the soil (LuelsegedA, 1990).

### 3.8 Previous studies

Summary of researches related with the thesis work.

Leuleseged A. (1990), the study focused on the engineering geological characterization of the soils in south-eastern Addis Ababa, where failure of engineering objects is frequent. The study indicates clay and sandy clay soils which are derived from the surrounding volcanic rocks are 5-8m thick.

Alemayehu Teferra and Solomon Yohannes (1986) studied expansive soils of Addis Ababa.. The study showed that the clay fraction ranges from 40 to 70. Plasticity index ranges from 68 to 93. Free swell ranges from 90-170%. Swelling pressure ranges from 94 KN/m3 to 251KN/m3. Activity ranges from 1.3 to 1.8. Based on the final output of the work they conclude that the black and grey soils found in the eastern and southern part of Addis Ababa are highly expansive. All samples are inorganic clays of high plasticity, all samples are located between the Activity 1 and 2 and there is not clear distinction between the black and grey soils with regard to plasticity and activity

According to Daniel Teklu (2003), examining the swelling pressure of Addis Ababa expansive soil indicated that, moisture content ranges from 33.2 to 44.3, liquid limit ranges from 96 to 121, plasticity index ranges from 26 to 47, clay fraction ranges from 50 to 81 and swelling pressure ranges from 108 to 420 Kpa.

According to Hailemariyam Girma (2016) Bole sub-city, around Bole Medhanialem Church is an area having maximum thickness of expansive soils in Addis Ababa. The average LL, PI and free swell values of expansive soils in Addis Ababa are 92%, 44% and 143%, respectively.The compression index of ‘light to dark grey’ soils ranging from 0.166 to 0.319 showing a moderate to very high degree of compressibility. The swelling pressure of ‘light to dark grey’ soils in Addis Ababa are in the range of 100 to 180Kpa.

Mesfin, (2004) studied the index properties of expansive soils of Ethiopia. Based on experimental results from 125 samples, high clay content, high to extremely high plasticity ranges. From the test result, the expansive soil of Ethiopia is classified to extremely high swelling potential. The study concluded that expansive soils in Ethiopia are unsuitable as construction material and should be considered as problematic foundation soils.

Abdirshekur K (2015) studied the correlation between index properties and swelling pressure of expansive soils found around Koye area. Thick black clay soil covers the area up to 1.5m depth. Based on different soil classification system the study area soil type is fat cay soil. The result of the swelling pressure and the plasticity index is in the range of 80-400 and 56.2-70.3 respectively. This soil has very high swelling potential, so he recommended that special care should be taken to design light weight structure.

Zemene Muche, (2019) studies engineering properties of expansive soils in Akakai -Kality sub-city. The liquid limit (LL) and plastic limit (PL) of the area is in the range from 25-113% and 4-7% respectively. The free swell is in the range of 20-280%.the initial void ratio of the area is ranges from 0.237-1.81% and the compression index ranges from 0.215-1.047. The thesis work defines the soil layers of the study area. Accordingly, the area has the following layers black cotton soils (Expansive clay), silty clay/clayey silt, sandy silt/silty sand, sandy gravel layers are encountered. The topsoil of the area predominantly highly expansive clay. Light structures with shallow foundation lay on expansive soil needs either establishing mechanism or remove the expansive layer and replace it with an appropriate material for the safety of the structure.

# CHAPTER 4

# ENGINERING GEOLOGICAL CHRACTERIZAION OF EXPANSIVE SOILS

## 4.1. General background

Expansive soils pose certain challenge to infrastructure development, particularly light loaded and shallow founded structures. Engineering geological characterization helps to identify the physical and engineering geological property of such soils, define their associated effects and assist the efforts in working with such soils. Thus, characterization of these soils before constructing the foundation is imperative for the safety of overlying building.

For the purpose of characterization, a large quantity of drilling and laboratory testing data, both from pre-existing and ongoing projects, have been procured and analyzed.

In order to perform a meaningful statistical analysis, 56 laboratory data from 31 boreholes were collected (Fig 4.2) from four local companies namely Addis Geosystems PLC, Construction Design Share Company, RKG Engineering PLC and Mafcon Engineers PLC collected with their area coordinates. Geotechnical database was then produced from data obtained from these societies and actual field investigation of five test pit. The collected primary and secondary borehole data's totally 61 samples are presented in appendix 1 with their area distribution map (fig 4.1).

Accordingly, index and engineering properties were determined from laboratory tests, and further analysis and interpretations have been made by integrating all laboratory tests.

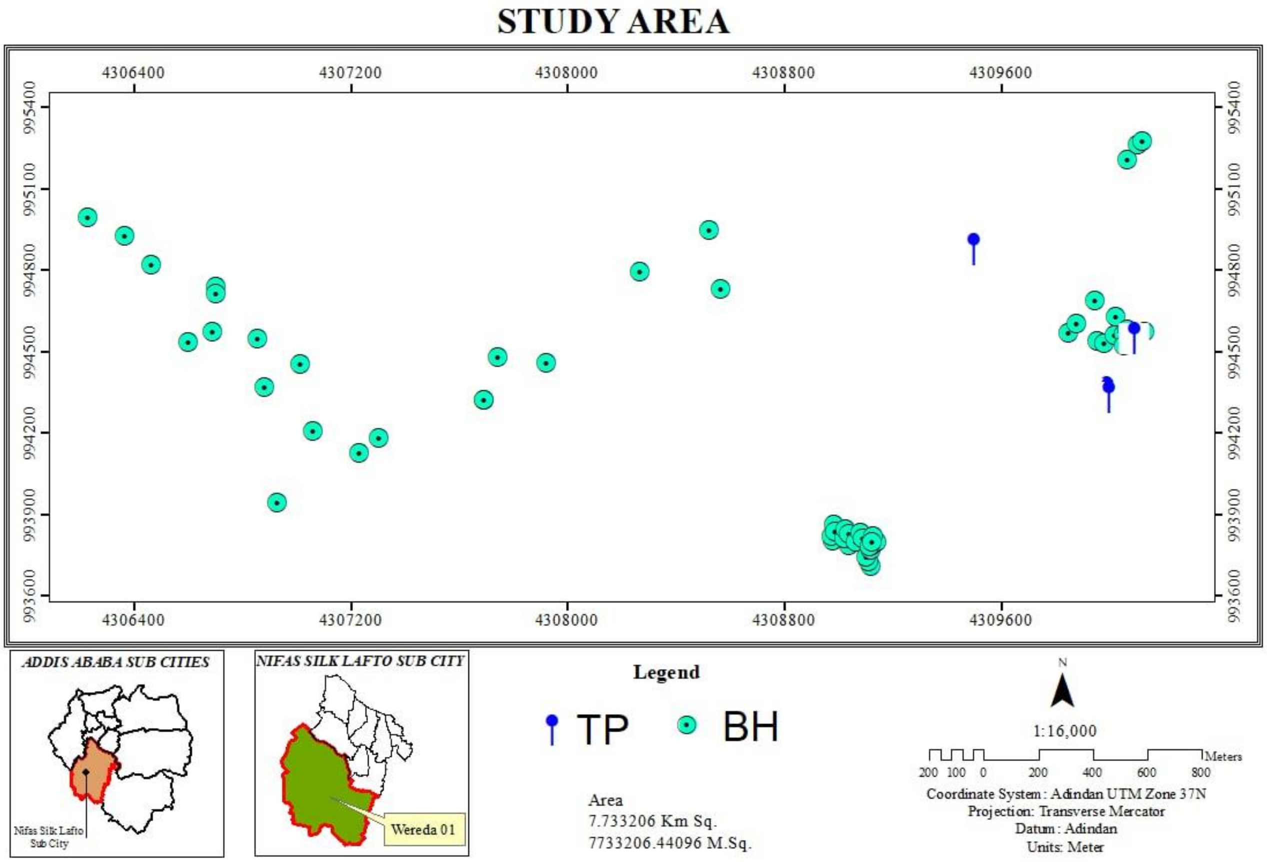


Figure 4. 1 Distribution of the test pits and the secondary boreholes data of the study area

Accordingly, the soil in the study area is predominantly residual soil. The upper most layer of the study area is covered with dark grey to light grey high plastic expansive clay soil.

It is coasty and difficult for geotechnical engineer to obtain and analysis engineering data. Also, on the site it is difficult to determine the soil type and change of layers. Soil properties of the site are estimated by the geotechnical engineer depending on the information from the representative samples. In this thesis work analysis of laboratory results and prediction of one from the other test is adapted.

### 4.2. Results of Index properties

In this work, results of the index properties of soils discussed with respect to the objective of the study. Field identification is accomplished by a few simple hand tests for the fine-grained soils. Laboratory index test results namely natural moisture test, specific gravity, particle size, Atterberg limits, dry density, free swell, and activities are presented in the following paragraphs.

#### 4.2.1 Natural moisture contents

When the moisture content of the clay is changed, volume expansion in the vertical and the horizontal direction will take place (Afework sisay, (2004). According to afework sisay 2004, there will be no volume change if the moisture content of the clay element is unchanged. He stated that complete saturation is not necessary to initiate swelling. Slight change in moisture content in the magnitude of 1 to 2% is sufficient to cause detrimental swelling. In the dry season the soil shrinks excessively and shrinkage crack as deep as two or three meter is common occurrence.

The numbers of primary samples taken were 5 disturbed soil samples. Soil moisture content was measured in accordance with ASTM D - 2216. The soil was dried at a constant temperature of $105^0$C plus or minus 5 using a conventional oven for about 24 hours. Then, the natural moisture is measured.

During sampling collection all necessary cares were taken to hold the moisture as it is. The top and bottom of the Shelby sample taking tube were immediately wax sealed and covered with sample bags.

The numbers of secondary data samples taken were 57 at various specified depth below natural ground level within the range of expansive soil layer from 31 boreholes and 5 test pits. AASHTO T-265 test procedure has been adopted.

From the collected primary and secondary data’s in the study area generally the soils have moisture content in the range of 21%- - 56% with an average value of 38.5%. The moisture content of the area varies from depth to depth. The moisture content determination was carried out by taking disturbed soil samples. Perhaps, moisture content depends on timing of sampling (dry and wet season) and depth of sampling. It is difficult to know the exact value of the water content in the area instead the average value can govern well.

### 4.2.2 Specific gravity

Primary samples taken for this work were totally 5. The tests were conducted according to ASTM D-854. Secondary samples tested were 54 from 31borholes. The tests were conducted according to ASTM D-854 and AASHTO 084-94 test procedure. The specific gravity of the study area is in the range of 2.12-2.7 with an average value of 2.41.

### 4.2.3 Grain size analysis

In the present study the particle size distribution analyses of the disturbed soil samples have been carried out by hydrometer test adopted from ASTM D 422.

Five primary samples were taken for sieve analysis from five test pits based on hydrometer test (fig 4.2) for preparing of soil samples which pass through No 200 (0.075mm) sieve.

Secondary sample tests were 55 samples from 31 boreholes at deferent depth with a maximum depth of 6.5 and analysis was carried out. The relative proportion of different size groups in each soil is determined after complete grain size analysis.

All soil samples are composed of a relative proportion of different size groups as naturally expected, although one group dominates the other group in a relative percentage. The size range of each group was adopted from ASTM D 422(4.75 mm-Gravel, 4.75 – 0.75mm- sand, 0.075 – 0.002mm-Silt, and <0.002mm-Clay). The study area soils are ranging from 75 to 99.2% with an average value of 87% passing in # 200 sieves. The distribution of different grain sizes affects the engineering properties of soil.

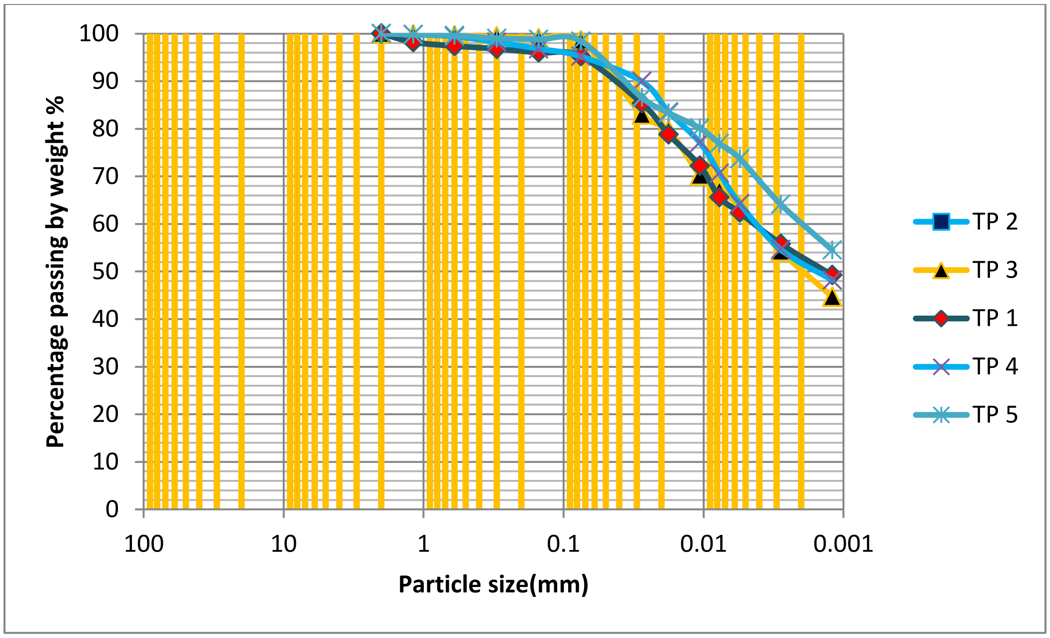


Figure 4. 2 Grain size analysis result of Test pits.

### 4.2.4 Atterberg limit

As the water content of a fine-grained soil is increased gradually from 0% to 100%, the soil exhibits different consistencies, namely, solid, semi-solid, plastic and liquid states (Nelson, 2010). Therefore, Atterberg limits define the water content at each state. According to day 2006, if a soil is heavily saturated with water and then is dried out, it will move from a liquid state to a plastic state to a semisolid state and then to a solid state.

Atterberg limit corresponds to values of moisture content where the consistency of the soils changes as it is progressively dried from slurry (liquid state). Plasticity is the response of soil to changes in moisture content. When water is added to soil, it changes its consistency from hard to soft; the soil is said to exhibit plasticity (Budhu, 2011). The Atterberg limits of measurement adopted in this study to describe the consistency of soils include the liquid limit (LL) and the plastic limit (PL). The liquid limit is the moisture content at which soil passes from the plastic to the liquid state, while the plastic limit represents the moisture content at which soil passes from the plastic to the solid-state and becomes too dry to be in a plastic condition. The plasticity index (PI) is the moisture content that range between the plastic and liquid limits and serves as a measure of the plasticity of the clay soil (Nelson and Miller, 1992).

Totally 61 soil samples were taken from 36 boreholes and five test pits at various specified depth below natural ground level. Both primary and secondary soil samples test were conducted according to ASTM D-4318.

The liquid limit of the soil samples is in the range of 75%-115% with an average value of 95% (Table 4.2). Therefore, it is in the range of very high plasticity (Table 4.1) and the plastic limit of the area is in the range of 33.3%- 70.4% with an average value of 51.8%.

Table 4. 1 Terms for the description of fine soils (Murthy, 2009)

| **Term** | **Range of liquid limit** |
|---|---|
| Low plasticity | Under 35 |
| Intermediate plasticity | 35 – 50 |
| High plasticity | 50- 70 |
| Very high plasticity | 70 – 90 |
| Extremely very high plasticity | Over 90 |

Atterberg limits had been correlated very well with the geotechnical characteristics of fine-grained soils and are therefore very valuable in soil classification. Plasticity index is a parameter which can be used as a preliminary indicator of the swelling characteristics of a soil (Seed et al. 1962). Soils with plasticity index that is above 55 have very high swelling potential. But it should be noted here that high index property doesn't necessarily mean high swelling potential while the converse may be true.

Table 4. 2 Physical and index properties of the study area soils

| **Physical and index property** | **Range** | **Mean** |
|---|---|---|
| 200 sieve pass (%) | 75-99.2 | 87.1 |
| Activity | 0.7-2.2 | 1.45 |
| Moisture content (%) | 21-56 | 39 |
| Liquid limit (LL) (%) | 75-115 | 95 |
| Plastic Limit (PL)% | 33.3-70.4 | 52 |
| Plasticity Index (PI) (%) | 30.6-74.2 | 52.4 |
| Free swell (%) | 100-195 | 147.5 |
| Specific gravity | 2.12-2.7 | 2.4 |

The average plasticity index of the samples is in the range of 30.6% to 74.2% with an average value of 52.4% (Table 4.2). The range shows that the soil in the study area is found under extremely plastic (table 4.3).

Table 4. 3 Terms for the description of plasticity index (Murthy, 2009)

| Terms | Plasticity index |
|---|---|
| Non-plastic | Under 1 |
| Slightly plastic | 1 – 7 |
| Moderately plastic | 7 – 17 |
| Highly plastic | 17 – 35 |
| Extremely plastic | Over 35 |

The free swell test is one of the most commonly used simple test for estimating the soil swelling potential. According to Holtz & Gibbs (1956) the amount of free swell serves to be indicative of the probable swelling and/ or expansive behavior of clay soils. It is considered as a measurement of volume change in clay upon saturation.

Soils that plot above the A-line are high plastic inorganic clays and those, which plot below it, are highly plastic organic clay and silts. For the present study, the collected soil samples fall above and below A- Line which is high plastic clay (CH) and high plastic silt (MH) respectively fig. 4.4. It shows that the area is covered by both high plastic expansive organic (MH) and inorganic (CH) soil types.

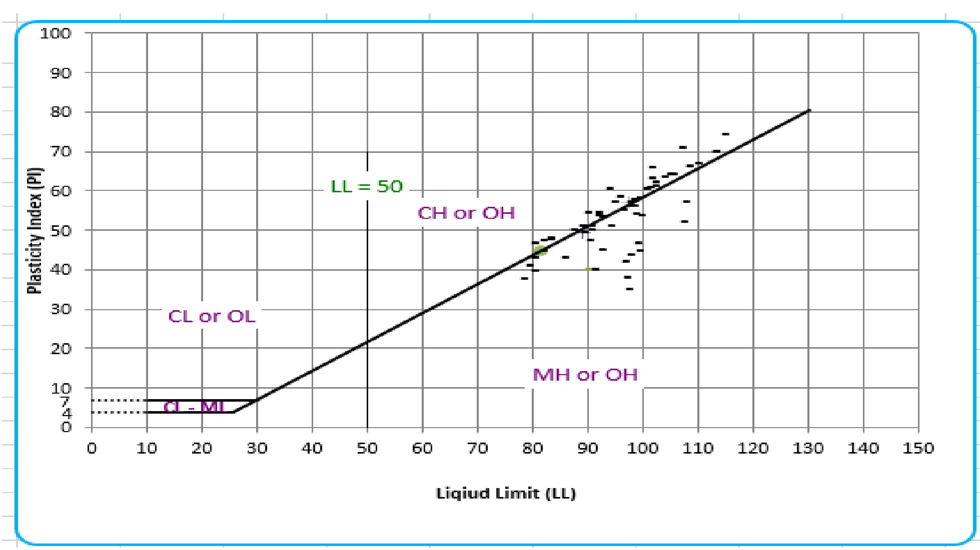


Figure 4. 3 Plasticity chart for samples collected from the study area

The value of Liquidity Index (LI) varies according to the consistency of soils as follows:

| LI | Description of soil strength |
|---|---|
| LI < 0 | Semisolid state – High strength but brittle, i.e. sudden fracture is expected |
| 0 < LI <1 | Plastic state – Intermediate strength |
| LI > 1 | Liquid state – Low strength |

Therefore, the study area is found under the category of plastic state to semisolid state. High strength does not mean that the area is not in danger there will be expected a sudden fracture at the area.

### 4.2.5 Activity

Different clay minerals exhibit stronger or weaker negative charges which in turn serve as a measure of the activity of the clay soil in terms of the relative degree with which the negative charge would influence opposing charges in adsorbed water to cause the clay particles to form clusters or flocculate and settle down in a suspension of water (Johnson and Degraff , 1988). As a result, the amount and type of clay minerals present in a soil have a significant effect on soil engineering properties such as plasticity, swelling, shrinkage, shear strength, consolidation and permeability. Classification of clays based on their activity (after Skempton, 1953; Mitchell, 1993) is presented in Table (4.4).

Activity which is defined as the ratio of the plastic index to percent of clay fraction finer than 0.002mm is one means of classifying expansive soils based on their index property. It is also used to estimate the swelling potential of given clay.  All the values in Appendix 1 are in the range of 0.7 to 2.2 with an average value of 1.45 (table 4.2). This shows that the area is found under Inactive to very active zone with reference to (table 4.4. However, detailed clay mineralogy investigation could verify the lower degree of activity in the soils of the study area.

Table 4. 4 Activity of clay soils and clay minerals (After Skempton, 1953; Mitchell, 1993).

| **Clay Soils** | **Activity** |
|---|---|
| Inactive clays | <0.75 |
| Normal clays | 0.75-1.25 |
| Active clays | 1.25-2.0 |
| Highly active clays | >2.0 |
| | |
| **Clay Minerals** | **Approx. Activity** |
| Kaolinite | 0.3-0.5 |
| Illite | 0.5-1.3 |
| Na-montmorillonite | 4.0-7.0 |
| Ca-montmorillonite | 1.5 |

Most of the data (Fig 4.3) the study area soils fall above A-line and some points are below A-line. According to Holtz and Kovacs (1981) montmorillonite has the highest activity of clay minerals and would often plot above the A-line of the plasticity chart. Moreover, results of Atterberg limits and /or activity values have also been used to assess the probable type of clay minerals present in the soils, based on the classification Table (4.3). It is therefore anticipated that the study area soil clays which exhibit activity would mainly resembles to smectites group. This prediction has to be confirmed by actual clay mineralogical studies.

### 4.2.6 Free swell

The free swell test is one of the most commonly used simple test for estimating the soil swelling potential. According to Holtz & Gibbs (1956) the amount of free swell serves to be indicative of the probable swelling and/ or expansive behavior of clay soils. It is considered as a measurement of volume change in clay upon saturation.

Classification of soils based on swelling capability is summarized in Table (4.5). For the collected fifty-three soil samples from 31 boreholes and from the primary soil samples five free swell are examined. the free swell values range from 100% to 195% with an average value of 147.5% and this shows that medium to very high swelling capabilities of potential expansiveness, being characterized by free swell values of 100% and over (Table-4.5).

Table 4. 5 Free swell classification of clay soils (Holtz and Gibbs, 1956).

| **Free swell value (%)** | **Free swell classification** | **Potential expansiveness** |
|---|---|---|
| <50 | Low | Low |
| 50-100 | Medium | Medium |
| 100-200 | High | High |
| >200 | Very high | Very high |

## 4.3. Engineering properties

The behavior of every foundation depends primarily on engineering characteristics of the underlying materials (Budhu, 2011). The most important engineering properties of soils for foundation analysis are strength (cohesion and internal friction), and compressibility properties (such as the compressibility index).

### 4.3.1 Shear strength test

In the study area, the result obtained from15 collected soil samples and the direct shear strength test shows that the very cohesive nature of the clays is evidenced by the relatively large values of cohesion (c´) obtained, i.e. 18 – 50KPa, giving a mean value of 34Kpa (Table 4.6) were reasonable angle of internal friction ranges from $10^0$ to $25^0$ (Table- 4.6). But some of the results show big values of angle of internal friction, which are not expected in clay soil. Such a higher value of friction angle is due to the presence of granular and fragmented material.

Table 4. 6 Engineering properties of the study area soils

| Engineering properties of the study area soils | | |
|---|---|---|
| Engineering properties | Range | Mean |
| Cohesion (C) Kpa | 18 - 50 | 34 |
| Angle of internal friction (Φ) | 10 - 25 | 17.5 |
| Swelling pressure (Kpa) | 94 - 220 | 157 |
| Coefficient of Compression (Cc) | 0.16 – 0.325 | 0.24 |

### 4.3.2. Compressibility

For the collected 16 soil samples, consolidation test results, Compression index (Cc) of the soils ranges from 0.16 to 0.325 with an average value of 0.24 (Table- 4.6). Table 4.7a gives an estimation of compressibility class based on compression index (Lambe and Whitman, 1979); and the soils are ranging from low to medium compressibility.

Results of compression index, Cc, obtained for the collected soil samples in the study are presented in Table (4.7b) together with corresponding values of liquid limit and plasticity index. Based on compression index (Lambe and Whitman, 1979) clay soils could be approximately grouped into those of low compressibility, medium to high compressibility, high compressibility and very high compressibility; as shown in Table (4.7a).

Table 4. 7a Compression indices (Cc), and compressibility classification of clay soils after (Lambe and Whitman, 1979).

| Compression index (Cc) | Compressibility class |
|---|---|
| <0.2 | Low |
| 0.2-0.8 | Medium-High |
| 0.8-2.6 | High |
| >2.6 | Very high |

The results of correlation show that LL and PI would be a better estimator of the compression index and compressibility of soils (Table 4.7b). The strength of correlation between Cc and LL is very high (R2 = 0.8817) and between Cc and PI is (R2=0.7879) (Figs 4.4 and 4.5) respectively. The relationships between compression index and liquid limit take the form of

$$Cc = 0.0053LL - 0.2505 \qquad \text{- - - - - 4.1}$$

And the relationship between compression index and Plasticity index take the form of

$$Cc = 0.0042PI - 0.0235 \qquad \text{- - - - - -4.2}$$

Table 4.7b 1 Results of compression indices correlated with Atterberg limits.

| Correlation summery of calculated and laboratory derived compression indices | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| E | N | Sample depth(M) | Wn(%) | LL(%) | PI(%) | Cc (Laboratory derived) | Cc (new correlated) | Cc (Skempton) | Approx. compressibility |
| 469036 | 990750 | 2.6 | 29 | 88.0 | 34 | 0.194 | 0.465 | 0.702 | Low |
| 469036 | 990750 | 3.5 | 37 | 75.0 | 35 | 0.16 | 0.396 | 0.585 | Low |
| 467927 | 989363 | 3.5 | 47 | 90.0 | 37 | 0.219 | 0.476 | 0.720 | Medium |
| 467932 | 989340 | 4 | 38.9 | 103.5 | 63.6 | 0.284 | 0.547 | 0.842 | Medium |
| 467966 | 989315 | 5 | 26 | 88.7 | 51.0 | 0.242 | 0.469 | 0.708 | Medium |
| 467966 | 989315 | 3 | 38.9 | 97.4 | 57.1 | 0.29 | 0.515 | 0.787 | Medium |
| 467981 | 989331 | 2 | 41.5 | 85.0 | 51 | 0.212 | 0.449 | 0.675 | Medium |
| 468031 | 989316 | 3 | 33.8 | 82.9 | 47.6 | 0.185 | 0.438 | 0.656 | Medium |
| 468066 | 989302 | 2.5 | 25.9 | 101.3 | 66.0 | 0.325 | 0.536 | 0.822 | Medium |
| 468062 | 989212 | 3 | 32.8 | 99.2 | 53.7 | 0.265 | 0.524 | 0.803 | Medium |
| 468053 | 959229 | 3 | 28 | 104.6 | 64.0 | 0.303 | 0.553 | 0.851 | Medium |
| 468083 | 989301 | 2.45 | 41.5 | 93.7 | 51.1 | 0.232 | 0.495 | 0.753 | Medium |
| 468057 | 989307 | 2.5 | 38.2 | 98.9 | 58.1 | 0.265 | 0.523 | 0.800 | Medium |
| 465662 | 990071 | 2.5 | 55.5 | 83.0 | 43.1 | 0.19 | 0.439 | 0.657 | Low |
| 465441 | 990314 | 4 | 35.2 | 85.0 | 42.5 | 0.185 | 0.449 | 0.675 | Low |
| 465576 | 990120 | 3 | 25 | 97.4 | 57.1 | 0.265 | 0.515 | 0.787 | Medium |

Results of correlations and analysis show that Cc values generally increases with increasing liquid limit and plasticity index of soils. This serves to suggest that compressibility of soils generally increases with increased plasticity.

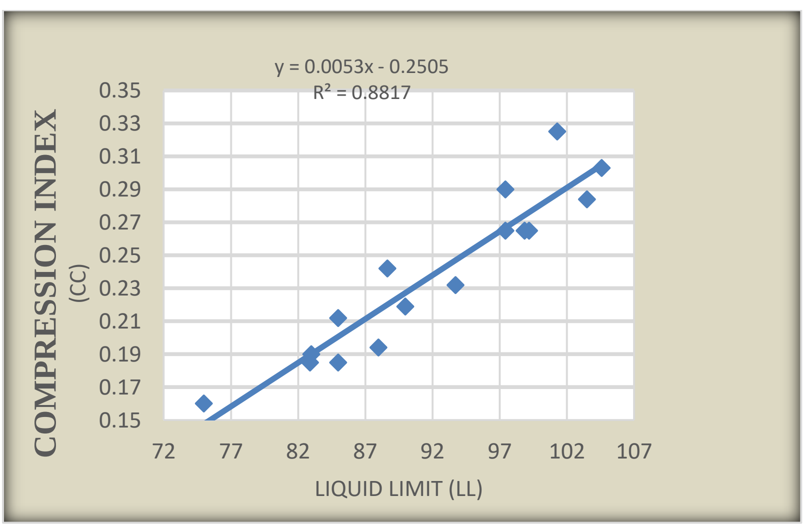


Figure 4. 4 Correlation between compression indices and liquid limit.

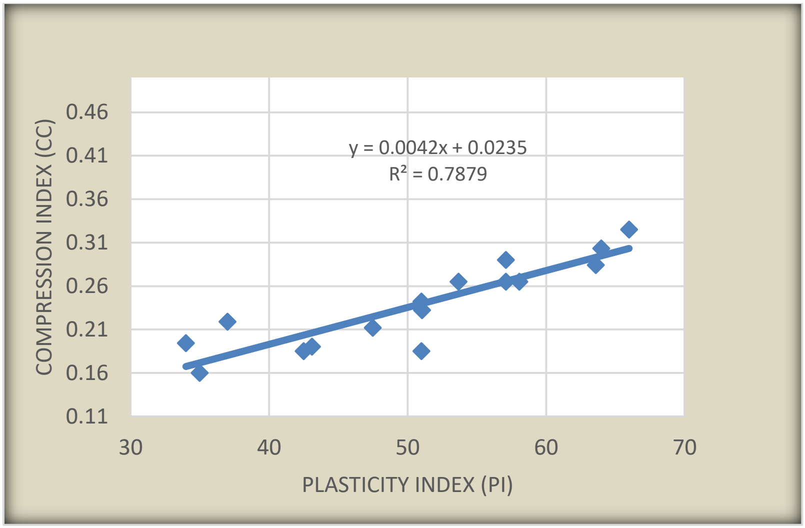


Figure 4. 5 Correlation between Plasticity indices and compression indices

In this research the relationship between these two parameters can be observed and the extent to which the Plasticity Index (PI) will affect the value of Compression Index (Cc) can be clearly identified. By referring to the relation generated, the Compression Index (Cc) value can be predicted in response to the execution of the Atterberg Limit Test. Such correlation will bring a lot of conveniences to the engineers of geotechnical filed whenever they tend to do quick and simple identification of the compressibility of soil (Norlia Mohamad Ibrahim et.al, 2012). From the results of the Atterberg Limit test and Oedometer test the relationship between these parameters is known interrelating.

Calculated values of compression index alongside laboratory determined compression indices, Cc, the strength of correlations is strong (Table 4.7b) and (Fig.4.8). However, values of compression indices calculated from laboratory liquid limits using Skempton´s relationship, [Cc=0.009(LL-10)] are generally not overestimated with respect to the laboratory determined compression indices (fig 4.6) because overestimation might be associated with differences in lithology, mineralogy and/ or organic matter content between tropical soils as encountered in this study, and those of temperate climates on which Skempton´s analysis and derivation were based.

It shows the comparison of correlation Equations, and the new correlation shows same general trend with that of the well-known Skempton’s correlation (fig 4.6).

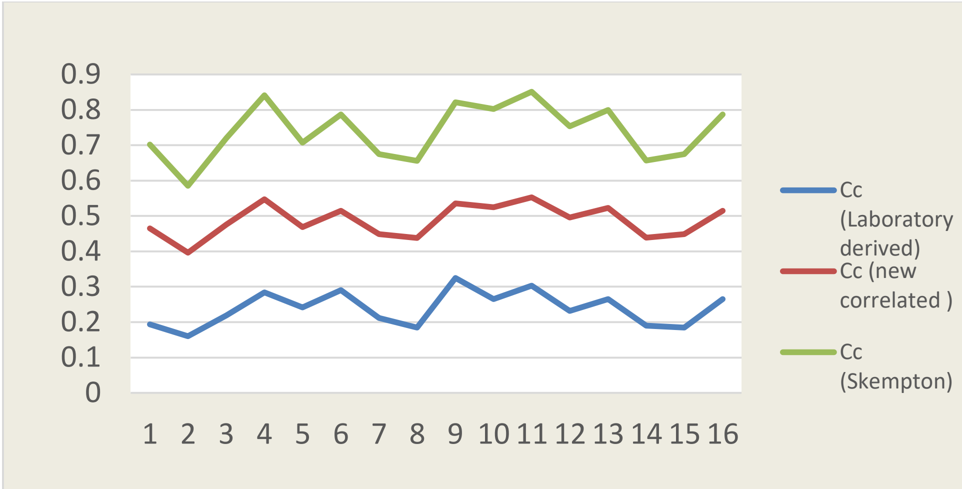


Figure 4. 6 Comparison of Correlation Equations

## 4.4 Classification of expansive soils

Skemptos method and Seed, woodward and lindgreen methods are the common classification methods of expansive soils as discussed in the literature review part (Chapter 3).

- Holtz and Gibbs

Holtz and Gibbs (1956), as described by Asuri (2016) classify expansive soils base on their free swell value. According to Holtz and Gibbs clay of the study area is classified with respect to their free swell in the range of high swelling potential

- Skemptons method

Activity which is defined as the ratio of the plastic index to percent of clay fraction finer than 0.002mm is one means of classifying expansive soils based on their index property. According to Skempton, clays of the study area classified with respect to their activity in the range of inactive to normal clay. These values are shown in Appendix 1.

- Seed, woodwardand lundgreen method

According to Seed, Woodward and Lundgreen, Plasticity Index is a parameter which can be used as a preliminary indicator of the swelling characteristics of a soil as discussed in (chapter 3). Based on this classification all samples of the study area have plasticity index of 55 and above it categorized under very high swell potential.

### 4.5 Swelling Pressure prediction

The swelling pressures of soils in the study area with a total of twenty- five soil samples are collected with five primary soil samples and the values are in the range of 94 KPa to 220 KPa with an average value of 157 KPa (Table 4.6).

The expansive clays exhibit significant swelling pressures of 94 - 220Kpa (Table 4.6). However, no well-defined relationship was found to occur between swelling pressure determined on undisturbed specimens on one hand, and index properties, on the other. Attempted correlations are generally poor and of low strength ($R< 0.5$) as; and evidenced by more spread-out plot points on scatter diagrams in section 4.5.2 (Figures 4.9, 4.10, 4.11 and 4.12). This shows the unreliability with which swelling characteristics of the clays could be estimated and / or predicted from results of index tests.

## 4.5.1. Scatter Plot and Best-Fit Curve

The relationship of two or more variables can be expressed in mathematical form by determining an equation connecting the two variables. Generally, in this thesis work correlation was made by considering Swelling pressure (Ps) as the dependent variable whereas Liquid limit (LL), plastic limit (PL), Plasticity index (PI), Dry density and natural moisture content (w) are the independent (predictor) variables.

The MS excel spread sheet is found to be the most powerful and manageable tool for scatter plot analysis and determination of correlation between two and more variables.

4.5.1.1 Swelling pressure vs Plasticity Index

The relationship between the swelling pressure and the plasticity index for all the tested samples is shown in Figure 4.7. The best fitting trend line for this relationship is SP = -0.206* PI – 127.81. The strength of this equation in predicting an outcome from the plastic index is R2 = 0.004.

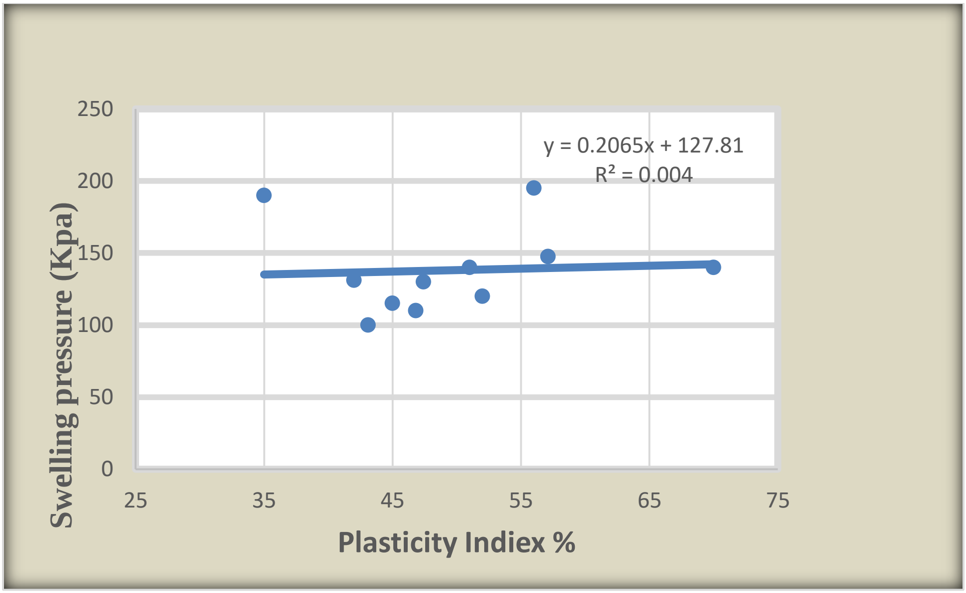


Figure 4. 7 Correlation between Swelling pressure (Kpa) and Plasticity Index (%)

The swelling Pressure and the Plasticity index of the study area shows that the widely scattered points from the trend line this indicate that the relationship is weak. This shows that the determination of the Plasticity Index alone cannot satisfactorily indicate the swelling behavior of the soil of the study area.

4.5.1.2 Swelling pressure VS Natural moisture

Natural moisture content is one of the factors, which influence the swelling characteristic of expansive soils. The relationship between the swelling pressure and the Natural moisture test for all the tested samples is shown in Figure 4.8. The best fitting trend line for this relationship is Ps = -0.9783* ω +174.83. The strength of this equation in predicting an outcome from the natural moisture content has $R^2$ = 0.19 having a weak relationship.

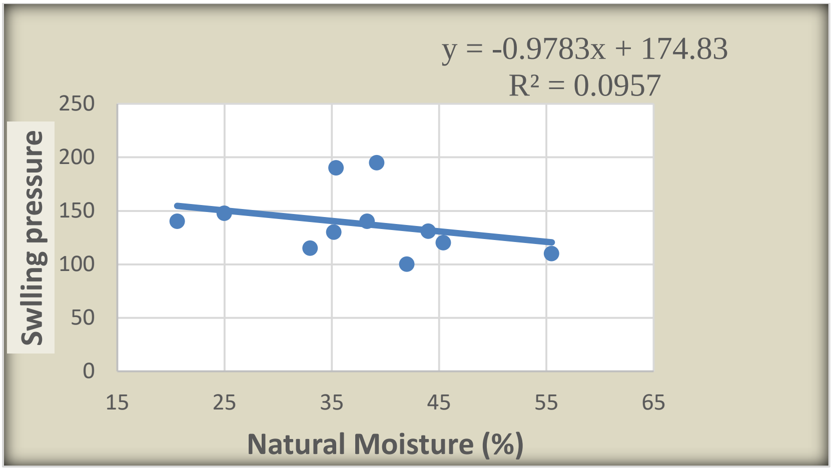


Figure 4. 8 Correlation between Swelling pressure (Kpa) and Natural Moisture test (%).

A graph is plotted to see how it affects the swelling pressure of the study area. The results show a general trend of decreasing swelling pressure with increment of natural moisture content manifested in a polynomial relation as shown in Figure 4.8.

### 4.5.1.3 Swelling pressure VS Plastic limit

The best fitting trend line for relationship between swelling pressure and plastic limit is Ps = -0.1626* PL + 142.83. As the strength of this correlation is only 1.4 % it is deemed not reliable enough to be used as a predictor for the estimation of the swelling pressure.

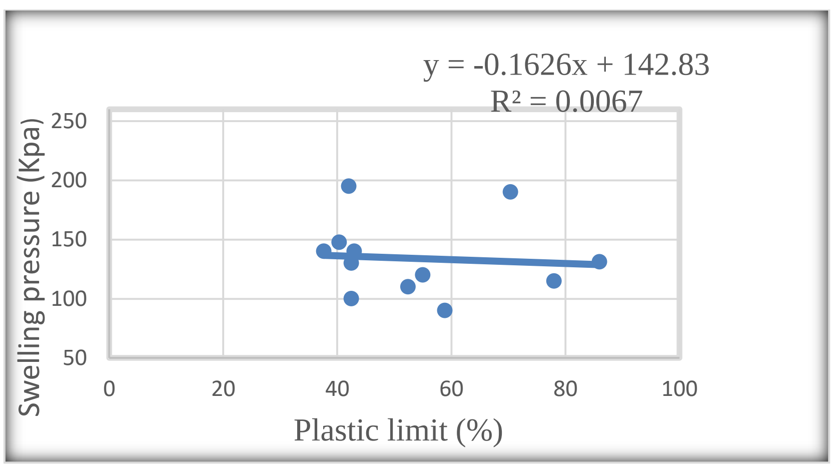


Figure 4. 9 Correlation between Swelling pressure (Kpa) and Plastic Limit PL (%).

The swelling Pressure and the Plastic Limit of the study area as shown in Figure 4.9 are widely scattered points from the trend line. This shows that the determination of the Plastic Limit alone cannot satisfactorily indicate the swelling behavior of the soil of the study area.

4.5.1.4 Swelling pressure VS Liquid Limit

The best fitting trend line for relationship between swelling pressure and liquid limit is

Ps = 0.0796* LL – 113.44. As the strength of this correlation is only 0.2 % it is deemed not reliable enough to be used as a predictor for the estimation of the swelling pressure.

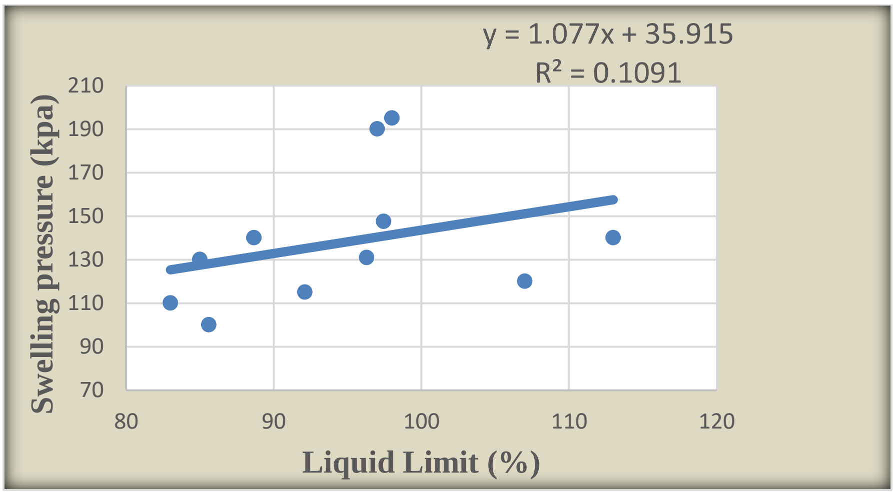


Figure 4. 10 Correlation between Swelling pressure (Kpa) and Liquid Limit (%)

The swelling Pressure and the Liquid Limit of the study area as shown in Figure 4.10 widely scattered points from the trend line. This shows that the determination of the Liquid Limit alone cannot satisfactorily indicate the swelling behavior of the soil of the study area.

**4.5.1.5 Swelling pressure VS Dry density**

Dry density is a factor which plays a role in swelling characteristics of expansive soils and, compactness of a soil grain can be measured by dry density. A graph is plotted to see the relationship between the dry density and swelling pressure of the study area from table 4.9. The result shows that there is a tendency of increment of swelling pressure as the dry density increases as shown in fig. 4.11.

Table 4. 8 Results of correlation of swelling pressure with dry density

| E | N | Depth | GS | eo | Dry Density (Kn/m3) | Swelling Pressure (Kpa) |
|---|---|---|---|---|---|---|
| 469036 | 990750 | 2.6 | 2.5 | 0.76 | 1.4 | 150 |
| 469036 | 990750 | 3.5 | 2.34 | 0.86 | 1.3 | 100 |
| 467927 | 989363 | 3.5 | 2.45 | 0.73 | 1.4 | 135 |
| 467966 | 989315 | 3 | 2.45 | 0.99 | 1.2 | 95 |
| 468031 | 989316 | 3 | 2.41 | 0.74 | 1.4 | 180 |
| 468083 | 989301 | 2.45 | 2.38 | 1.15 | 1.1 | 100 |
| 468057 | 989307 | 2.5 | 2.41 | 0.8 | 1.3 | 120 |
| 465662 | 990071 | 2.5 | 2.42 | 0.8 | 1.3 | 110 |
| 465441 | 990314 | 4 | 2.45 | 0.8 | 1.4 | 110 |
| 465576 | 990120 | 3 | 2.74 | 0.89 | 1.4 | 148 |

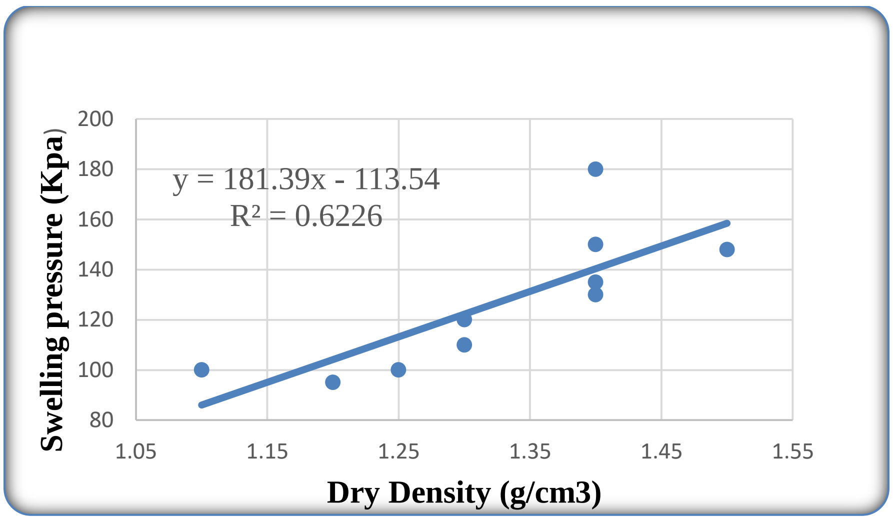


Figure 4. 11 Correlation between swelling pressure (Kpa) vs dry density (g/cm3)

### 4.6 Cross section profile

In this thesis three strati-graphic cross sections have been selected along bore holes and test pits and soil profiles generated to over view soil distribution accordingly. Drainage and topographic conditions of the study area were essential parameters in the selection of cross section profiles.

Soil cross sections have been taken by connecting bore holes and test pits across the research area. Total of three cross sections have been taken for the study area. These sections are selected in a way that could help see the variation of soil layers and thickness against topography and drainage.

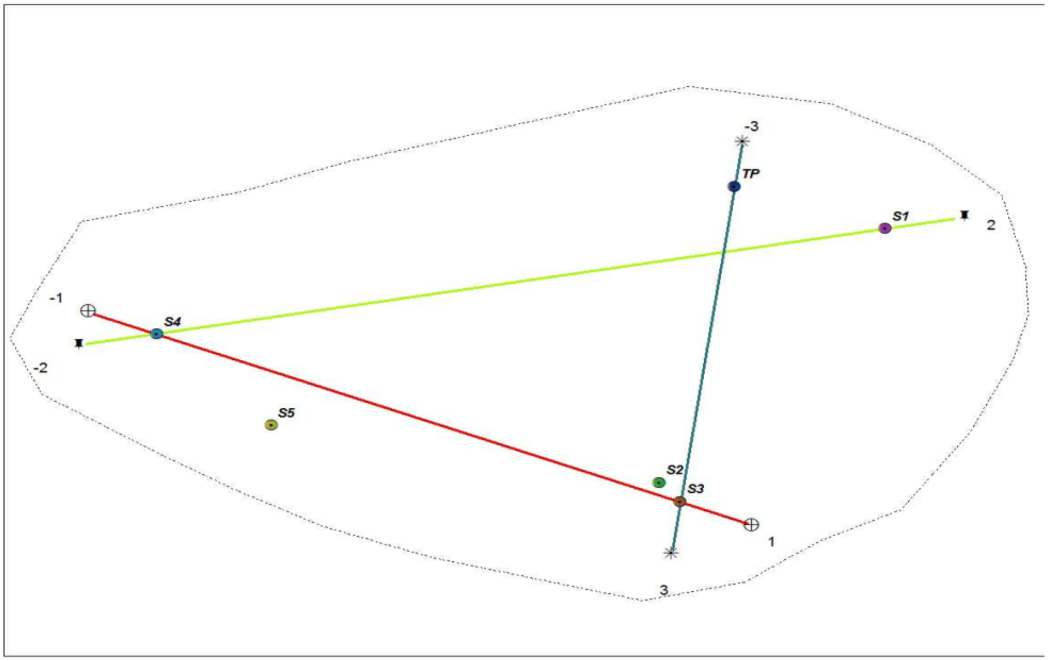


Figure 4. 12. Cross-section profile layout

**Section 1-1 Along S4 and S3**

The first layer along this section is dark grey silty clay soil with a thickness of 2.9m and 6.45 around S4 and S3. The second layer is light grey Silty clay/clayey silt soil with a maximum thickness of 8m and 2.2m around S3and S4. The third layer is Sandy silt/silty sand mixed with gravel decomposed from the underline TUFF with a thickness of 4.9 m around S4.

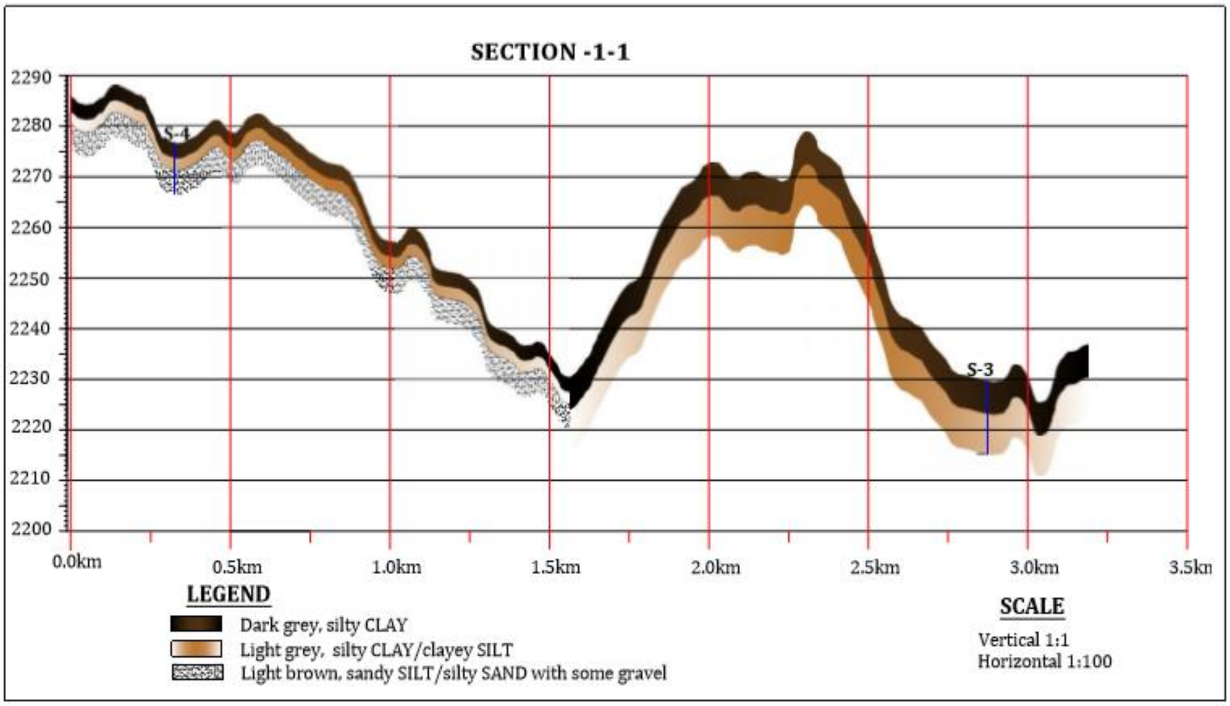


Figure 4. 13 Geotechnical cross-section 1-1

**Section 2-2 Along S4 and S1**

The first layer along this section is dark grey silty clay soil with a thickness of 2.9m and 8.95m around S4 and S1. The second layer is light grey Silty clay/clayey silt soil with a maximum thickness of 2.2m and 1.05m around S4and S1. The third layer is Sandy silt/silty sand mixed with gravel decomposed from the underline TUFF with a maximum thickness of 19.1 m around S1.

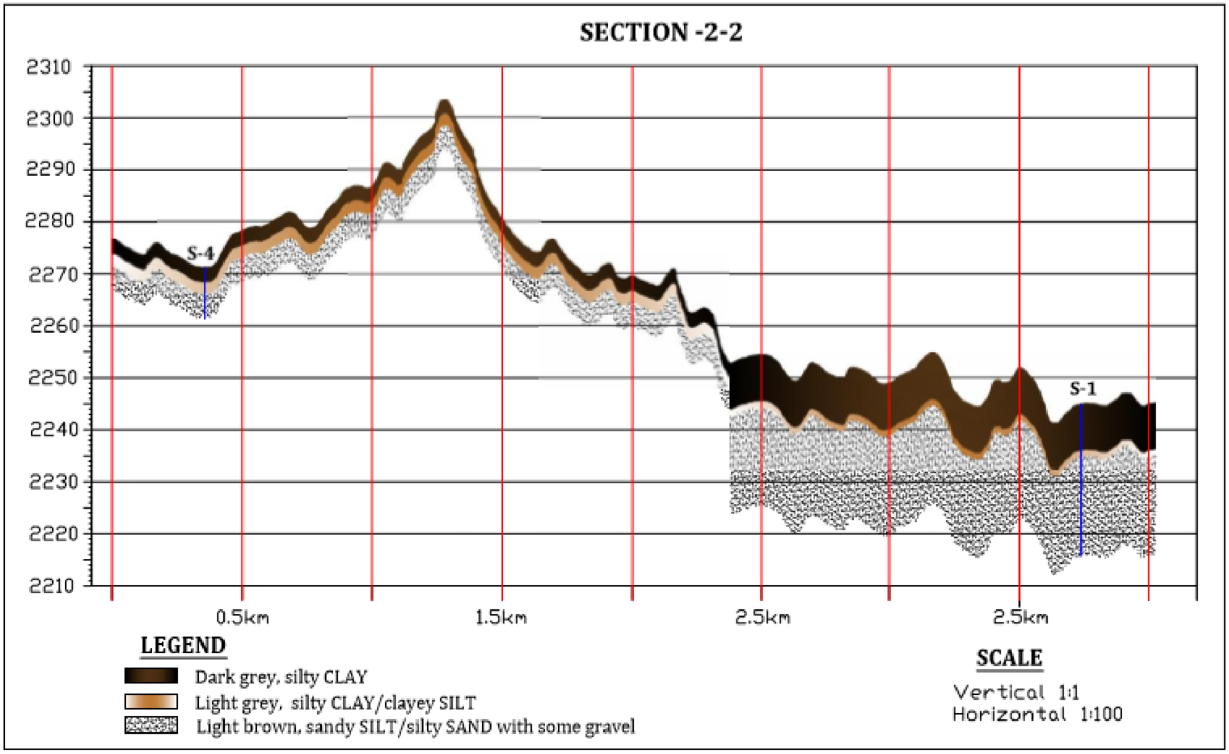


Figure 4. 14 Geotechnical cross-section 2-2

**Section 3-3 Along TP and S3**

The first layer along this section is dark grey silty clay soil with a thickness of 1.4m and 6.45 around TP and S3. The second layer is light grey Silty clay/clayey silt soil with a maximum thickness of 1.6m and 8m around TP and S3.

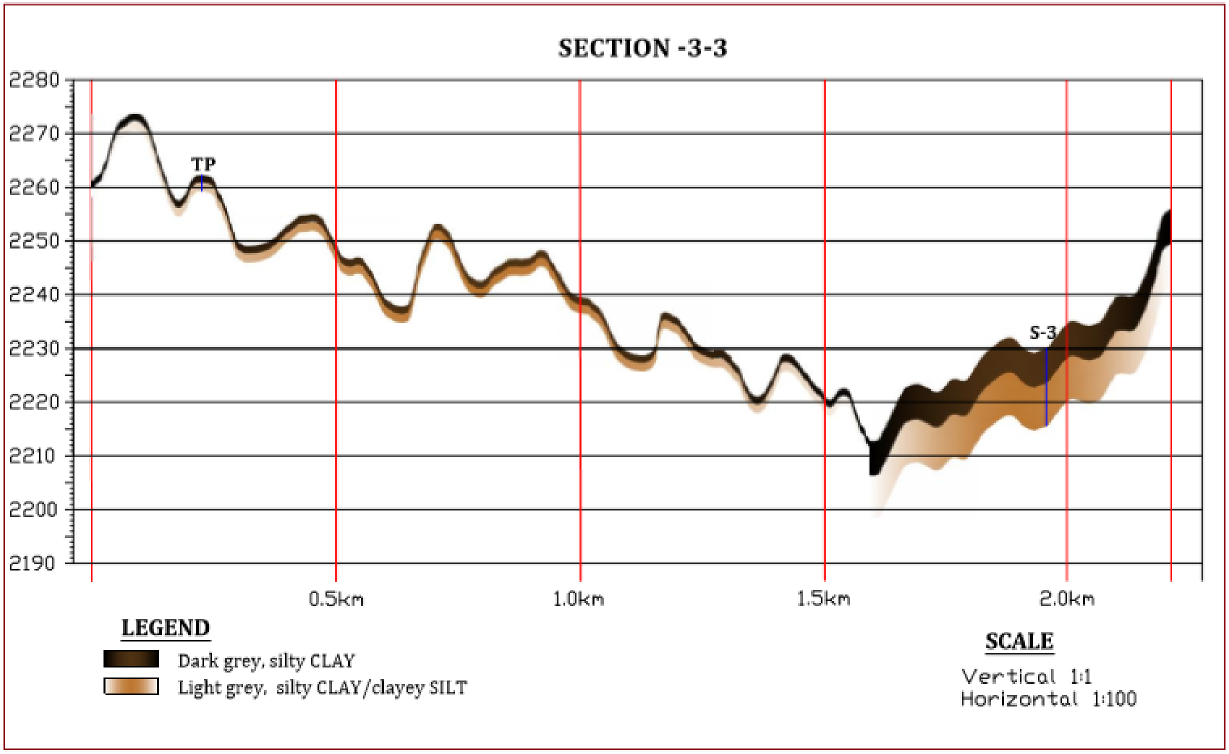


Figure 4. 15 Geotechnical cross-section 3-3

According to the collected data most of the places in the study area are covered by approximately 16m thick accumulation of expansive soils as presented in the section profile above. The soil thickness and the location of potential expansive clay layers in the soil profile considerably influence the seasonal heave. If the expansive clay is overlain by a layer of non-expansive topsoil or overlies bedrocks at a shallow depth, the swelling of clay will be greatly reduced. The greatest seasonal heaves likely to occur where potentially expansive soil exists from the surface to at least the depth of equilibrium moisture level.

The expansive soils have two layers based on color variations: the lower light gray soil and the upper dark gray soil. The change in color between the two layers is not sharp but gradual.

**Summery**

In general, the characteristics of the expansive soils of the study area according to the objective of the research are summarized as follows:

- Both the light and dark grey soils of the study area soils fall under the MH and CH soil class based on USCS respectively.
- Based on their plastic and liquid limits, the soils show very high plasticity and very high swelling potential respectively.
- In turn, such high plasticity and swelling potential indicating that Smectites clay minerals are dominantly present in these soils.
- The free swell values in these soils are ranging from 100% to 195% with an average value of 147.5%. This result is compatible to that of the high swelling potential obtained from their liquid limit values. The value of this free swell indicates that their high swelling capabilities.
- The soils show high contents of fine fraction, 75 to 99.2%, indicating that their physical and engineering behaviors are almost directly related to the type of their clay minerals.
- The soils fall in the class of normal to active clay, which doesn't reflect the expectation of having the high active soil as the soils fall above A-line. However, detailed clay mineralogy investigation could verify the lower degree of activity in the soils of the study area. It is also used to estimate the swelling potential of given clay. All the values in are in the range of 0.7 to 2.2. This shows that the area is found under Inactive to very active zone. However, detailed clay mineralogy investigation could verify the lower degree of activity in the soils of the study area.
- The soils show 18 – 50 KPa, with a mean value of 34 Kpa cohesion 'c' value indicating their very high cohesive nature. Though angle of internal friction also shows higher values, ranging from $10^\circ$ to $25^\circ$ it is not expected to have such higher value and may attributed the presence of granular and fragmented material as it is common in residual soils.
    - From the collected data, the following relationships have been derived for the possible estimation of compressibility indices, Cc, of the soils using laboratory determined LL and PI. $Cc = 0.0053LL - 0.2505$ And $Cc = 0.0042PI - 0.0235$ and the correlation

between so calculated compression indices and laboratory measured compression indices were found to be strong.

- Result of correlations and analysis show that Cc values generally increases with increasing liquid limit and plasticity index of soils. This serves to suggest that compressibility of soils generally increases with increased plasticity.
- The expansive clay exhibit significant swelling pressures of 94-220KPa. However, no well-defined relationship was found to occur between swelling pressure and index properties: attributing to the high possible uncertainties in attempting to assess and predict expansion tendencies and swelling pressures.

- The compression index (Cc) ranges from 0.16 to 0.325 showing a low to medium degree of compressibility.
- Accordingly, the soil in the study area is predominantly residual soil.

# CHAPTER 5

# CONCLUSION AND RECOMMENDATION

## 5.1. Conclusion

For past over 50 years Ethiopia has witnessed a dramatic increase in urbanization associated with increased population (Afework, 2004). This increase in the urban population has been accompanied by an equally strong growth in the number of high-rise offices, commercial and apartment buildings, residential houses, schools and other infrastructure constructions. Adequate knowledge of the sub-surface geology and engineering geological characterization of expansive soils is therefore very important for the safe and economic design of engineering structures. Thus, keeping these facts in mind this study was conceived with an objective to determine the engineering geological characteristics of the expansive soils in relation to its physical properties in NifasSilkLafto sub-city Jemo area, Addis Ababa.

In order to achieve the above-mentioned objective, systematic literature review was undertaken that includes previous studies, various soil mechanics and foundation engineering books, manuals and research articles and online internet browsing. Thus, with this literature review a conceptual framework was developed and a feasible methodology was evolved.

In order to prepare a geotechnical data base, a large quantity of drilling and laboratory testing data, both from preexisting and ongoing projects, have been filtered out and utilized. The extracted laboratory and field investigation data were systematically organized, and a geotechnical data base has been created in Microsoft Excel. From this study the following findings are deduced.

- Most of the area is covered with thick expansive soil. Based on the available drilling geotechnical data the thickness of expansive soil ranges from 3m to 16m in the study area
- And the soil has high swelling potential, so every lightweight structure should be designed carefully.

- It was possible to classify the soil into different soil type's i.e (Dark grey silty CLAY/ clayey SILT as top layer, Light grey silty CLAY/ clayey SILT, Clayey silty SAND).
- All soils are residual. This residual nature of the soil have a significant influence on the engineering geological properties of the soils since, it affects the degree of weathering and exercise a selective control on the type of the clay mineral which can form weathering process.
- The study shows that the natural moisture content varies from 21% to 55.5% Since the area is relatively flat, the soil can hold enough moisture from the surrounding area. For different clay and water content the swelling pressure of expansive soils around Jemo area is not the same. But moisture content depends on timing of sampling (dry and wet season) and depth of sampling. It is difficult to know the exact value of the water content in the area instead the average value can govern well.
- The specific gravity varies from 2.12 to 2.7.
- The soils are categorized as fine-grained soils from which more than 90% of the particle sizes are smaller than 0.075mm. The liquid limits range from 75% to 115% and plastic limits vary from 33.3% to 70%. Therefore, it is in the range of very high plasticity since the range of the average value is under 70-90%. And the plastic Index of the area is in the range of 30.6%- 74.2% with an average value of 52.4%. Based on their plastic and liquid limits, the soils show very high plasticity and very high swelling potential respectively.
- The free swell ranges from 100% to 195% with an average value of 147.5% and swelling pressures vary from 94kPa to 220kPa with an average value of 157Kpa indicating that the structures built on them should have an imposing pressure greater than the swelling pressure not to be lifted and face cracking.
- Results of correlations and analysis show that Cc values generally increases with increasing liquid limit and plasticity index of soils. This serves to suggest that compressibility of soils generally increases with increased plasticity.
- No well-defined relationship was found to occur between swelling pressure and index properties; attributing to the high possible uncertainties in attempting to assess and predict expansion tendencies and swelling and most of them have no a direct relationship.

- The particle size distribution of soils was characterized by very high fine fractions with respect to sand.
- Some type of soils of the study area fall under the MH and the others under CH soil class based on USCS respectively.
- The soils show 18 – 50 KPa, with a mean value of 34 Kpa cohesion ‘c’ value indicating their very high cohesive nature. Though angle of internal friction also shows higher values, ranging from $10^{\circ}$ to $25^{\circ}$ it is not expected to have such higher value and may attributed the presence of granular and fragmented material as it is common in residual soils.

## 5.2. Recommendation

The present study was conducted by generating a geotechnical data base from disorganized borehole log and laboratory test results. It was a very tedious and time-consuming effort to filter and process these data for required input parameters. Therefore, it is required that the enterprises/ organizations in similar tasks should be committed in building a quality data base and provide support to such endeavors.

To adopt the results of this study, further detail verification is needed. Besides, detail laboratory studies should be conducted including XRD test.

The prediction of swelling pressure relationships with index tests cannot be expected to yield accurate results. Therefore, for detail investigation swelling pressure should be determined from oedometer tests on a sample that have an expected initial condition that could yield maximum swelling pressure.

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**Appendix 1**

Laboratory result summery of field test pit tests and collected data’s

| No. | E | N | Sample Depth (m) | % of Clay | % of silt | % 200SEIVE PASS | Gs | LL (%) | PL (%) | PI (%) | Cc | e0 | Wn(%) | Unit weight (KN/m3) | C (Kpa) | Φ (Degrees | FS (%) | Sp (Kpa) | *Activity* |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 468309 | 990922 | 1.2 | 50 | 44 | 94 | 2.68 | 80 | | 46.7 | - | - | 39.2 | 17.5 | - | - | 120 | 140 | 0.9 |
| 2 | 468438 | 990368 | 2.5 | 60 | 37 | 97 | 2.65 | 91 | | 39.9 | | | 45.66 | 16 | - | -- | 102 | 220 | 0.7 |
| 3 | 46020 | 990040 | 3 | 53 | 41 | 94 | 2.56 | 92.3 | | 53.4 | - | - | 48 | 18.6 | - | - | 100 | 120 | 1.0 |
| 4 | 469017 | 990046 | 1.25 | 51 | 44 | 95 | 2.65 | 101.3 | | 63.3 | - | - | 52.3 | 19.8 | - | - | 110 | 115 | 1.2 |
| 5 | 468373 | 990623 | 3 | 61 | 34 | 95 | 2.68 | 95.5 | | 58.5 | - | - | 38.8 | 17 | - | - | 130 | 130 | 1.0 |
| 7 | 469036 | 990750 | 2.6 | 30 | 45 | 75 | 2.5 | 88 | | 34 | 0.194 | 0.76 | 29 | - | | | 100 | 150 | 1.1 |
| 8 | 469036 | 990750 | 3.5 | 36 | 57 | 93 | 2.34 | 75 | | 35 | 0.16 | 0.858 | | 19 | 29 | 19 | 100 | 100 | 1.0 |
| 9 | 469053 | 990761 | 4.3 | 61 | 37 | 98 | 2.46 | 87 | | 50 | | | 42 | 17.89 | | | 100 | 100 | 0.8 |
| 10 | 468996 | 990761 | 3.7 | 45 | 30 | 75 | 2.62 | 91.7 | | 54 | | | 29 | 16.19 | | | 108 | | 1.2 |
| 11 | 467918 | 989324 | 6 | 47 | 49 | 96 | 2.68 | 81.55 | | 45 | | | 34.9 | 15.9 | | | 120 | | 1.0 |
| 12 | 467918 | 989324 | 3 | 69 | 27 | 96 | 2.45 | 91.6 | | 54 | | | 29.9 | 15.46 | 50 | 15.8 | 120 | | 0.8 |
| 13 | 467927 | 989363 | 3.5 | 66 | 32 | 98 | 2.45 | 90 | | 37 | 0.219 | 0.73 | 47 | 19 | 46 | 15 | 130 | 135 | 0.6 |
| 14 | 467927 | 989363 | 5 | 39 | 53 | 92 | 2.5 | 90.2 | | 50 | | | 26.7 | 17.3 | 28 | 19 | 130 | | 1.3 |
| 15 | 467932 | 989340 | 4 | 48 | 40 | 88 | 2.6 | 103.5 | | 64 | 0.284 | 0.957 | 38.9 | 17.2 | | | 130 | | 1.3 |
| 16 | 467932 | 989340 | 6 | 48 | 35 | 83 | | 113 | | 70 | | | 43.9 | 15.5 | | | 140 | 94 | 1.5 |
| 17 | 467966 | 989315 | 5 | 32 | 55 | 87 | 2.45 | 88.7 | | 51 | 0.242 | 1.7 | 26 | 17.47 | | | 140 | - | 1.6 |
| 18 | 467966 | 989315 | 3 | 35 | 58 | 93 | 2.45 | 97.4 | | 57 | 0.29 | 0.992 | 38.9 | 15.9 | 18 | 16 | 147.5 | 95 | 1.6 |
| 19 | 467968 | 989348 | 5.5 | 69 | 28 | 97 | 2.48 | 108 | | 66 | | | 44 | 17.8 | | | 150 | 140 | 1.0 |
| 20 | 467981 | 989331 | 2 | 50 | 23 | 75 | 2.33 | 85 | | 51 | 0.212 | 0.566 | 41.5 | 17.11 | 24 | 10 | 150 | | 1.0 |
| 21 | 468005 | 989302 | 4.35 | 46 | 53 | 99.2 | 2.12 | 97 | | 56 | | | | 17.56 | | | 150 | 100 | 1.2 |
| 22 | 468024 | 989335 | 6 | 44 | 46 | 90 | 2.52 | 100.2 | | 60 | | | 46.2 | 17.21 | 31 | 15 | 152.5 | - | 1.4 |
| 23 | 468031 | 989316 | 5 | 38 | 57 | 95 | 2.45 | 90.23 | | 51 | | | 37.7 | 17.59 | | | 153 | 140 | 1.3 |
| 24 | 468031 | 989316 | 3 | 34 | 49 | 83 | 2.41 | 82.9 | | 48 | **0.185** | **0.74** | 33.8 | 16.8 | | | 154 | 180 | 1.4 |
| 25 | 468058 | 989286 | 3 | 82 | 17 | 99 | 2.49 | 109.62 | | 67 | | | 38.4 | 17.55 | | | 155 | - | 0.8 |
| 26 | 468070 | 989322 | 3 | 43 | 52 | 95 | 2.55 | 112.8 | | 70 | | | 28.7 | 17.76 | | | 155 | - | 1.6 |
| 27 | 468066 | 989302 | 2.5 | 87 | 10 | 97 | 2.5 | 101.3 | | 66 | 0.325 | 0.909 | 25.9 | | | | 160 | - | 0.8 |

| No. | E | N | Sample Depth (m) | % of Clay | % of silt | % 200SEIVE PASS | Gs | LL (%) | PL (%) | PI (%) | Cc | e0 | Wn(%) | Unit weight (KN/m3) | C (Kpa) | Φ (Degrees | FS (%) | Sp (Kpa) | *Activity* |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 28 | 468062 | 989212 | 2.5 | 90 | 9 | 99 | 2.46 | 93.6 | | 60.5 | | | 29.6 | 16.1 | | | 160 | - | 0.7 |
| 29 | 468062 | 989212 | 3 | 86 | 13 | 99 | 2.35 | 99.2 | | 53.7 | 0.265 | 1.061 | 32.8 | 17.55 | | | 160 | - | 0.6 |
| 30 | 468053 | 959229 | 3 | 87 | 12 | 99 | 2.55 | 105.0 | | 64.4 | 0.303 | 0.945 | 28 | | | | 165 | - | 0.7 |
| 31 | 468067 | 989272 | 3 | 66 | 30 | 96 | 2.45 | 94.4 | | 57.2 | | | 45.8 | 16.22 | | | 170 | - | 0.9 |
| 32 | 468053 | 959229 | 4 | 66 | 32 | 98 | | 77.9 | | 42.6 | | | 39.9 | - | 25 | 12 | 170 | - | 0.6 |
| 33 | 468083 | 989301 | 2.45 | 70 | 20 | 90 | 2.38 | 93.7 | | 51.0 | 0.232 | 1.154 | 41.5 | - | 19 | 15 | 172.5 | 100 | 0.7 |
| 34 | 468083 | 989301 | 4 | 70 | 28 | 98 | 2.66 | 106.7 | | 71.1 | | | 33.3 | 18.5 | | | 175 | - | 1.0 |
| 35 | 468057 | 989307 | 5 | 51 | 35 | 86 | 2.45 | 114.5 | | 74.2 | | | 35.4 | 17.58 | | | 177.5 | - | 1.5 |
| 36 | 468057 | 989307 | 2.5 | 65 | 30 | 95 | 2.41 | 98.9 | | 58.1 | 0.265 | 0.807 | 38.2 | - | | | 180 | 120 | 0.9 |
| 37 | 468017 | 989316 | 3 | 61 | 33 | 94 | 2.48 | 89.7 | | 54.5 | | | 37.5 | 18.3 | | | 185 | - | 0.9 |
| 38 | 467982 | 989290 | 4 | 26 | 68 | 94 | 2.64 | 107.3 | | 56.9 | | | | 17.23 | | | 185 | - | 2.2 |
| 39 | 467991 | 989319 | 3 | 68 | 26 | 94 | 2.7 | 80.1 | | 39.7 | | | 40.4 | 17.9 | 48 | 25 | 190 | - | 0.6 |
| 40 | 467991 | 989319 | 4.5 | 70 | 28 | 98 | 2.45 | 98.2 | | 54.1 | | | 41 | 19 | 40 | 18 | 195 | - | 0.8 |
| 41 | 467952 | 989328 | 5 | 48 | 51 | 99 | 2.45 | 105.0 | | 64.0 | | | 40.5 | 16.69 | | | 170 | - | 1.3 |
| 42 | 467991 | 989319 | 6.5 | 47 | 39 | 86 | 2.6 | 102.0 | | 61.2 | | | 37.2 | 15.75 | | | 160 | 180 | 1.3 |
| 43 | 467931 | 989319 | 3 | 35 | 56 | 91 | 2.58 | 98.0 | | 57.8 | | | 45 | 15.7 | | | 125 | 100 | 1.7 |
| 44 | 467924 | 989305 | 5 | 48 | 45 | 93 | 2.7 | 79.0 | | 41.0 | | | 41.5 | 16 | | | 185 | 115 | 0.9 |
| 45 | 467931 | 989319 | 6.4 | 53 | 34 | 87 | 2.6 | 83.0 | | 48.0 | | | 36.2 | 16.5 | | | - | 140 | 0.9 |
| 46 | 467924 | 989305 | 3.5 | 41 | 54 | 95 | 2.35 | 101.0 | | 60.9 | | | 39.8 | 17 | | | - | 128 | 1.5 |
| 47 | 466197 | 989626 | 5.3 | 52 | 46 | 98 | 2.7 | 96 | | 55 | | | 42 | 17 | | | 120 | 80 | 1.1 |
| 48 | 466271 | 989683 | 4.5 | 63 | 34 | 97 | 2.7 | 102 | | 62 | | | 39.5 | 17 | | | 126 | 102 | 1.0 |
| 49 | 466028 | 989706 | 6.5 | 50 | 45 | 95 | 2.65 | 80 | | 43 | | | 40.2 | 17.5 | | | 110 | 114 | 0.9 |
| 50 | 465829 | 990044 | 4 | 65 | 33 | 95 | 2.55 | 107 | | 52 | | | 45.4 | 17.2 | 26 | 10 | 126.8 | 120 | 0.8 |
| 51 | 465854 | 989867 | 3 | 54 | 28 | 97.7 | 2.45 | 98 | | 56 | | | 39.2 | 16.5 | | | 115 | 195 | 1.0 |
| 52 | 465662 | 990071 | 2.5 | 78 | 38 | 92.2 | 2.42 | 83 | | 33.5 | 0.19 | 0.809 | 55.5 | 18 | 40 | 20 | 135 | 110 | 0.4 |
| 53 | 465677 | 990209 | 3.5 | 66 | 33 | 94.3 | 2.55 | 85.6 | | 43.1 | | | 42 | 15.5 | | | 120 | 100 | 0.7 |
| 54 | 465441 | 990314 | 4 | 45 | 29 | 87 | 2.45 | 85 | | 42.5 | 0.185 | 0.809 | 35.2 | 16 | | | 110 | 130 | 0.9 |
| 55 | 465346 | 990420 | 5 | 59 | 31 | 89 | 2.7 | 96.8 | | 37.9 | | | 34.5 | 16.8 | | | - | 90 | 0.6 |
| | | | | | | | | | | | | | | | | | | | |
| 56 | 465900 | 990040 | 2.5 | 45 | 32 | 95.5 | 2.45 | 101 | | 30.6 | | | 35.4 | 17.3 | | | 100 | 190 | 0.7 |
| 57 | 465678 | 990325 | 3 | 70 | 25 | 98.2 | 2.75 | 92.1 | | 34.1 | | | 33 | 16 | | | 162 | 115 | 0.5 |
| 58 | 465576 | 990120 | 3 | 35 | 58 | 93 | 2.74 | 97.44 | | 57.1 | 0.285 | 0.893 | 25 | 19.9 | 42 | 18 | 100 | 147.5 | 1.6 |
| 59 | 465576 | 990120 | 4 | 56 | 30 | 92.3 | 2.6 | 96.3 | | 30.3 | | | 44 | 18 | | | 120 | 131 | 0.5 |
| 60 | 465678 | 990325 | 5 | 32 | 55 | 87 | 2.45 | 88.65 | | 51 | | | 21 | 20.1 | 42 | 16 | 100 | 140 | 1.6 |
| 61 | 465210 | 990576 | 6 | 48 | 35 | 83 | 2.55 | 113 | | 70 | | | 38.3 | 18.9 | | | 100 | 140 | 1.5 |

# Appendix 2

Rainfall and temperature of the area and the surroundings (Akaki station)

| | | Jan | Feb | Mar | Apr | May | Jun | Jul | Aug | Sep | Oct | Nov | Dec |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| TMPMAX | 2000 | 26.2 | 27.1 | 28.5 | 27.3 | 27.1 | 26.1 | 24.6 | 23.8 | 24.7 | 25.2 | 25.7 | 25.8 |
| | 2001 | 26.3 | 27.3 | 26.2 | 27.7 | 26.9 | 25.6 | 24.6 | 24.7 | 25.8 | 26.9 | 26.5 | 26.4 |
| | 2002 | 26 | 27.6 | 27.5 | 27.9 | 28.5 | 26.9 | 25.8 | 24.7 | 26.1 | 27 | 26.5 | 26.2 |
| | 2003 | 26.7 | 28.3 | 28 | 27.3 | 28.9 | 27.1 | 23.7 | 23.6 | 24.9 | 26.6 | 26.6 | 25.6 |
| | 2004 | 27.4 | 27.5 | 27.8 | 26.5 | 28.6 | 26.3 | 24.7 | 24.3 | 25.7 | 25.9 | 26.6 | 26.7 |
| | 2005 | 27.1 | 29 | 28.2 | 27.8 | 26.8 | 26.3 | 23.7 | 24.5 | 24.6 | 26.2 | 26.3 | 26.1 |
| | 2006 | 27.2 | 27.9 | 27.9 | 27.3 | 28.6 | 26 | 24.3 | 23.8 | 24.9 | 26.4 | 25.8 | 25.6 |
| | 2007 | 26.3 | 27.4 | 28.8 | 27.5 | 28.2 | 25.5 | 23.9 | 23.4 | 24.7 | 26 | 25.8 | 25.8 |
| | 2008 | 27.2 | 27 | 29.3 | 29 | 29 | 25.6 | 24 | 23.3 | 24.9 | 26.4 | 25.2 | 26.2 |
| | 2009 | 26.5 | 28 | 29.6 | 28.5 | 30 | 29.3 | 24.5 | 24.4 | 26 | 25.9 | 26.8 | 25.8 |
| | 2010 | 27 | 27.2 | 26.9 | 27.5 | 27.2 | 26.8 | 23.4 | 23.7 | 24.7 | 26.7 | 26 | 26.2 |
| | 2011 | 26.6 | 28.1 | 27.6 | 29.5 | 28.7 | | 25.3 | | 25.2 | 26.6 | 26.2 | 25.7 |
| | 2012 | 26.8 | 26.9 | | 27.8 | 29.8 | 27.4 | 24.1 | 24 | 25.1 | 26.7 | 26.9 | 26.5 |
| | 2013 | 27 | 28.7 | 29.1 | 28.9 | 28.2 | 26.7 | 24.5 | 23.6 | 25.9 | 26 | 26.1 | 25.1 |
| | 2014 | 26.4 | 27.3 | 27.8 | 28.2 | | 27.4 | 23.9 | 23.1 | 24.1 | 24.7 | 25.9 | 25.3 |
| | 2015 | 26.2 | 28.7 | 29 | 29.6 | 27.9 | 26.6 | 25.8 | 24.9 | 25.8 | 27.4 | 26.9 | |
| | 2016 | | 28.3 | 30 | 27.3 | 26.7 | 26.4 | | | 25.5 | | 25.8 | |
| | 2017 | | | 29.1 | 29.4 | 27.7 | | | | 25.4 | 26.1 | 25.9 | 25.8 |
| | 2018 | | | | | | 25.8 | 24.5 | 23.9 | 26.1 | 26.6 | | |
| | | 26.68125 | 27.78235 | 28.31176471 | 28.05556 | 28.16471 | 26.57647 | 24.42941 | 23.98125 | 25.26842 | 26.29444 | 26.19444 | 25.925 |

| | | Jan | Feb | Mar | Apr | May | Jun | Jul | Aug | Sep | Oct | Nov | Dec |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| TMPMIN | 2000 | 9.3 | 10 | 12.9 | 15 | 15.7 | 13.9 | 14 | 14.2 | 14.2 | 13.4 | 11.8 | 10.4 |
| | 2001 | 10.1 | 11.5 | 13.4 | 15.3 | 15.5 | 14 | 13.9 | 14.7 | 13.6 | 13.1 | 14.4 | 15.2 |
| | 2002 | 15.4 | 16.4 | 14.3 | 14.6 | 15.8 | 14.8 | 14.5 | 14.2 | 13.8 | 13.8 | 12.7 | 13.9 |
| | 2003 | 11.8 | 13.2 | 14.2 | 15.1 | 15 | 15.1 | 14.2 | 14.1 | 15 | 14.9 | 14.9 | 12.8 |
| | 2004 | 14.6 | 14.2 | 14.6 | 15.9 | 16.2 | 15.6 | 14.7 | 14.6 | 14.9 | 14.6 | 14.2 | 14.5 |
| | 2005 | 13.6 | 15.2 | 16.5 | 16.2 | 16.1 | 15.4 | 15.8 | 15.5 | 15.6 | 16 | 14.6 | 13.6 |
| | 2006 | 15.3 | 15.4 | 15.5 | 16 | 16.3 | 15.3 | 15.1 | 15.3 | 15.4 | 15.6 | 15 | 14.8 |
| | 2007 | 15 | 15.1 | 15.6 | 16.2 | 16.5 | 15.3 | 16.6 | 16.7 | 17.3 | 17.4 | 17.1 | 16.5 |
| | 2008 | 17.8 | 18 | | | 17.7 | 16.5 | 14.4 | 14.3 | 15.6 | 16 | 14.2 | 14 |
| | 2009 | 14.3 | 16.1 | 17 | 17.2 | 17.9 | 18.2 | 15.3 | 15.7 | 16.3 | 15.7 | 14.3 | 14.5 |
| | 2010 | 13.6 | 14.6 | 15.3 | 16.3 | 16.5 | 14.2 | 13.2 | 13.2 | 12.7 | 9.4 | 7.8 | 7.4 |
| | 2011 | 9.3 | 8.5 | 9.9 | 12.4 | 13.2 | | 12.6 | 13 | 12.8 | 8.2 | 18.1 | |
| | 2012 | | | | | | 11.9 | 12.5 | 12.3 | 11.7 | 8.8 | 7.5 | 3.4 |
| | 2013 | 4.1 | 5 | 8.5 | 8.7 | 8.3 | 8.1 | 8.8 | 8.4 | 7.2 | 7.7 | 8.8 | 6.7 |
| | 2014 | 8.7 | 11.5 | 11.5 | 13 | | 12.2 | 13.4 | 12.4 | 12.1 | 9.9 | 8.1 | 7 |
| | 2015 | 6.9 | 8.6 | 11.1 | 11.1 | 12.5 | 12.6 | 13.1 | 13.1 | 12.2 | 10.4 | 9.5 | |
| | 2016 | 11.1 | 10.4 | 12.7 | 14.5 | 13.2 | 12 | | | 11.7 | | 7.6 | |
| | 2017 | | | 11.5 | 12 | 13.8 | | | | 12.5 | 10.2 | 7.9 | 5.1 |
| | 2018 | | | | | | 13.3 | 12.9 | 12.7 | 11.7 | 9.9 | | |
| | | 11.93125 | 12.73125 | 13.40625 | 14.34375 | 15.0125 | 14.02353 | 13.82353 | 13.78824 | 13.48947 | 12.5 | 12.13889 | 11.32 |

| Rainfall | | Jan | Feb | Mar | Apr | May | Jun | Jul | Aug | Sep | Oct | Nov | Dec | nu |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 2000 | 0.00 | 0.00 | 29.10 | 93.00 | 64.90 | 100.10 | 188.90 | 210.00 | 124.10 | 17.20 | 23.40 | 3.80 | 71.21 |
| | 2001 | 0.00 | 20.70 | 121.20 | 23.60 | 118.00 | 142.60 | 257.50 | 145.00 | 64.90 | 2.20 | 0.00 | 0.00 | 74.64 |
| | 2002 | 31.10 | 10.50 | 87.80 | 53.90 | 76.60 | 108.00 | 167.10 | 166.30 | 52.30 | 0.00 | 0.00 | 17.70 | 64.28 |
| | 2003 | 19.60 | 24.80 | 23.90 | 114.00 | 1.40 | 125.40 | 325.10 | 307.40 | 113.40 | 0.00 | 0.00 | 40.80 | 91.32 |
| | 2004 | 15.60 | 15.80 | 61.40 | 154.50 | 15.40 | 95.20 | 150.30 | 189.10 | 80.90 | 4.80 | 3.40 | 0.70 | 65.59 |
| | 2005 | 28.80 | 7.30 | 47.90 | 112.20 | 140.70 | 139.90 | 218.70 | 231.40 | 152.70 | 9.10 | 15.20 | 0.00 | 91.99 |
| | 2006 | 2.60 | 44.20 | 56.30 | 79.70 | 22.00 | 84.30 | 276.40 | 262.60 | 148.10 | 38.00 | 0.00 | 3.20 | 84.78 |
| | 2007 | 34.20 | 24.70 | 25.60 | 96.80 | 64.60 | 132.70 | 254.20 | 221.80 | 148.50 | 14.30 | 1.30 | 0.00 | 84.89 |
| | 2008 | 0.00 | 0.00 | 0.60 | 34.20 | 62.40 | 140.20 | 253.50 | 252.30 | | 7.20 | 64.80 | 0.00 | 74.11 |
| | 2009 | 60.20 | 0.00 | 10.00 | 118.70 | 47.70 | 63.50 | 235.30 | 322.40 | 71.30 | 32.80 | 4.00 | 16.80 | 81.89 |
| | 2010 | 0.00 | 63.80 | 126.20 | 170.00 | 95.20 | 164.80 | 334.40 | 169.80 | 154.10 | 5.20 | 14.80 | 7.80 | 108.84 |
| | 2011 | 0.00 | 2.50 | 45.20 | 20.70 | 128.70 | 60.00 | 204.30 | 304.00 | 194.50 | 0.00 | 4.70 | | 87.69 |
| | 2012 | 0.00 | 0.00 | | 61.00 | 26.10 | 80.60 | 228.00 | 243.90 | 122.90 | 0.00 | 0.00 | 0.00 | 69.32 |
| | 2013 | 0.00 | 0.00 | 77.00 | 89.10 | 73.40 | 111.50 | 179.60 | 242.40 | 142.50 | 20.60 | 0.00 | 0.20 | 78.03 |
| | 2014 | 0.00 | 39.40 | 76.00 | 13.90 | | 52.60 | 176.80 | 281.60 | 115.30 | 52.30 | 0.00 | 0.00 | 73.45 |
| | 2015 | 0.00 | 0.00 | 13.70 | 2.30 | 96.50 | 158.00 | 187.80 | 247.50 | 70.00 | 0.00 | 14.50 | | 71.85 |
| | 2016 | 0.00 | 46.00 | 39.30 | 184.50 | 134.20 | 105.70 | | | 119.10 | | 9.30 | | 79.76 |
| | 2017 | | | 46.10 | 22.50 | 70.30 | | | | 177.20 | 0.00 | 0.00 | 0.00 | 45.16 |
| | 2018 | | | | | | 165.70 | 238.30 | 274.10 | 47.30 | 13.20 | | | 147.72 |
| | | 11.30 | 17.63 | 52.19 | 80.26 | 72.83 | 112.82 | 228.01 | 239.51 | 116.62 | 12.05 | 8.63 | 6.07 | |