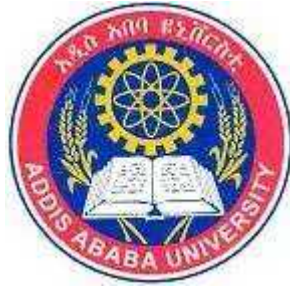


**ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES
FACULTY OF TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEERING**



**INVESTIGATION INTO SOME OF THE ENGINEERING PROPERTIES OF SOIL
IN WOLDIYA TOWN**

**A thesis submitted to the School of Graduate Studies of Addis Ababa
University in partial fulfillment of the requirements for the degree of Master of
Science in Civil Engineering (GEOTECHNIQUES)**

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Table of Contents

Acknowledgement	I
Symbols and Abbreviations	V
List of tables.....	VI
List of figures	VII
Abstract	VIII
1. Introduction.....	1
1.1. General.....	1
1.2.Objectives of the Study	2
1.3. Methodology	2
3.1.1 Reconnaissance of the area	2
1.3.3. Laboratory tests.....	2
1.4.Scope of the Study	3
1.5. Structure of the Thesis	3
2. Literature Review.....	4
2.1 Genesis of Expansive Soils.....	4
2.2 Mineralogical Structure.....	5
2.3 Identification and Classification of Expansive Soils.....	5
2.3.1 Field Identification	5
2.3.2 Laboratory Identification	6
2.3.2.1 Mineralogical Identification.....	6
2.3.2.2 Indirect Methods	6
2.3.2.3 Direct Methods.....	6
2.3.3 Classification Methods.....	6
2.3.3.1 Classification Using General Methods.....	6
2.3.3.2 The Van Der Merwe Method	7
2.4 Mechanics of Swelling.....	8
3. Description of the study area	10
3.1. General.....	10
3.2. Geology.....	10
3.3. Climate.....	11
3.3.1. Rainfall.....	11
3.3.2. Temperature	12
4. In-situ Properties and Laboratory Tests Results	13

4.1. In-situ Properties	13
4.1.1. Identification of Soil in the Study Area	13
4.2. Laboratory Test Results	14
4.2.1. Index Properties	14
4.2.1.1. General.....	14
4.2.1.2. Natural Moisture Content.....	14
4.2.1.2. Specific Gravity	15
4.2.1.3. Grain-size Distribution of Soil	16
4.2.1.3.1. General	16
4.2.1.3.2. Sieve Analysis.....	16
4.2.1.3.3. Hydrometer Analysis	16
4.2.1.4 Atterberg limits	17
4.2.1.4.1. General	17
4.2.1.4.2. Test Procedure.....	18
4.2.1.5. Free Swelling	19
4.2.2. Shear Strength of Soil	20
4.2.2.1. General	20
4.2.3. Consolidation	24
4.2.3.1. Pre-consolidation Pressure	24
4.2.3.3. Compression Index, C_c	25
5. Discussions of the laboratory test results and Comparisons with	28
previously done researches	28
5.1. Classification of the Soils	28
5.1.1. AASHTO Classification System.....	28
5.1.2. Unified Soil Classification (USC) System	29
5.1.2.1 Plasticity Chart	29
5.1.2.2 Activity.....	31
5.2. Discussions of the laboratory test results	31
5.3. Comparison of test results with previously done researches.....	33
6. Conclusions And Recommendations	35
6.1. Conclusion	35
6.2. Recommendation.	36
References.....	37

Symbols and Abbreviations

AASHTO	-	American Association of Highway and Transportation Officials
ASTM	-	American Society for Testing Materials standard
Cc	-	Compression index
CL	-	Lean clay
Cr	-	Recompression index
Cv	-	Coefficient of consolidation
e	-	Void ratio
EAEA	-	Journal of the Ethiopian Association of Engineers and Architects
LL	-	Liquid limit
MDD	-	Maximum dry density
MH	-	Inorganic Elastic silt
ML	-	Inorganic Silt
NMC	-	Natural moisture content
OMC	-	Optimum moisture content
OCR	-	Over-consolidation ratio
Pc	-	Pre-consolidation pressure
Po	-	Over burden pressure
PI	-	Plastic Index
PL	-	Plastic limit
SM	-	Silty sand
TP	-	Test pit
USCS	-	Unified Soil Classification System
Gs	-	Specific gravity of soil

List of Tables

TABLE 4.1 GLOBAL COORDINATES OF SAMPLING AREA.....	13
TABLE 4.2 NATURAL MOISTURE CONTENT OF THE SOIL SAMPLES	15
TABLE 4.3 SPECIFIC GRAVITY OF SOIL IN THE STUDY AREA	15
TABLE 4.4 RESULTS OF ATTERBEG LIMITS	19
TABLE 4.5 FREE SWELL TEST RESULTS	20
TABLE 4.6 CONSOLIDATION TEST RESULTS	27
TABLE 5.1 CLASSIFICATION OF SOIL BASED ON AASHTO CLASSIFICATION SYSTEM.....	29
TABLE 5.2 CLASSIFICATION OF SOIL BASED ON USCS.....	30
TABLE 5.3 DEGREE OF COLLOIDAL ACTIVITY	31
TABLE 5.4 THE ACTIVITY OF THE SOIL IN THE STUDY AREA	31
TABLE 5.6 COMPARISON OF TEST RESULTS IN DIFFERENT PARTS OF THE COUNTRY	34

List of Figures

FIGURE 2.1 VOLUME CHANGE POTENTIAL CLASSIFICATION FOR CLAY SOILS	7
FIGURE 2.2 WATER CONTENT PROFILES IN THE ACTIVE ZONE [20].....	8
FIGURE 3.1 LOCATION OF THE RESEARCH AREA ON THE MAP OF ETHIOPIA	10
FIGURE 4.1 LOCATION OF SAMPLING AREAS SHOWN ON MAP OF WOLDIYA TOWN.....	14
FIGURE 4.2 GRAIN SIZE DISTRIBUTION CURVE FOR THE COMBINED SIEVE AND HYDROMETER ANALYSIS IN THE STUDY AREA.....	17
FIGURE 4.3 TYPICAL LIQUID LIMIT DETERMINATION FOR TEST PIT # 4 AT 1.5 M	19
FIGURE 4.4 HORIZONTAL DISPLACEMENT VS SHEAR STRESS CURVE FOR TEST PIT 4	22
FIGURE 4.5 HORIZONTAL DISPLACEMENT VS SHEAR STRESS CURVE FOR TEST PIT 5.....	23
FIGURE 4.6 METHOD OF DETERMINING PC BY CASAGRANDE METHOD	25
FIGURE 4.7 GRAPHS A AND B SHOWS THE DETERMINATION OF PRE-CONSOLIDATION PRESSURES FOR SELECTED TEST PITS	26
FIGURE 5.1 PLASTICITY CHART	30

Abstract

Woldiya is a hillside market town, capital of the Semien Wollo Zone, and Woreda in Northern Ethiopia. Located North of Dessie and Southeast of Lalibela in the Amhara Region. This town has a latitude and longitude of $11^{\circ}50'N$, $39^{\circ}36'E$ and an average elevation of 2112 meters above sea level. A lot of civil engineering structures are under construction; however, nothing has been done on the investigation of soil with respect to the intended urban development plan. Therefore, the objective of the research is to investigate some of the engineering properties of soils found in Woldiya town.

The engineering behavior of soils formed under temperate or tropical conditions is determined by certain physical characteristics designated as engineering properties. In practice the determination of all the engineering properties of soils is expensive and time consuming. Whereas, Index properties are simpler and cheaper engineering characteristics which are indicative of some engineering properties and essential parameters for soil classification.

To achieve the aim of the thesis disturbed and undisturbed samples from different parts of the town were collected and laboratory tests were done. The index property investigation in this study includes; natural moisture content, specific gravity, particle size distribution, Atterberg limits and free swell tests. Furthermore, the consolidation and shear strength characteristics of soil in the town were studied.

The grain size analysis test result showed that the dominant proportion of soil particle in the research area is fine-grained soils, which have clay content ranging from 6-50%, silt fraction 40-63% sand fraction 2-20% .

The result of Atterberg Limit test on the soil in the research area showed a liquid limit ranging from 34-97%, plastic limit ranging from 28-35% and plastic index from 5-63%.The specific gravity ranges from 2.65-3.0. Free swell test conducted on the samples collected shows range from 39-130%.

The shear strength of a soil mass is the internal resistance per unit area that the soil mass can offer to resist failure and sliding along any plane inside it. Shear strength is the principal engineering property which controls the stability of a soil mass under loads. It governs the bearing capacity of soils, the stability of slopes in soils , the earth pressure against retaining structure and many other problem.

Disturbed and undisturbed samples were collected during the driest season. The soil was very stiff, when try to do unconfined compression test it shows crumble before the run of the test. Because of this reason it is impossible to do unconfined compression test in this thesis. Even though, unconfined compression test is not done, direct shear test was done to show the shear strength of soil in the town. The cohesion and

internal friction of the representative samples of test pit four and five are (64.83kPa,17.85⁰) and (91.56kPa,19.65⁰) respectively.

According to the Unified Soil Classification System, the soil is categorized mainly as clay soils and partially silts soils. The AASHTO Classification System shows that the usual types of significant constituent materials of the Woldiya soil are clayey soil. Finally one-dimensional consolidation tests were done and have compression index ranging from 0.28-0.4, recompression index ranging from 0.05-0.07, coefficient of consolidation ranges from 0.031-0.039cm²/sec.

1. Introduction

1.1. General

Woldiya is among one of the Ethiopian towns, which is growing rapidly. Rapid urbanization in the city area has led to an increased interest in the engineering behavior of the soils which are present within the city area. Geotechnical information of the subsoil in an urban area is important for various civil engineering works. Lack of sophisticated laboratory facilities and non-availability of the proper geotechnical information of the subsoil makes foundation and engineering works expensive, difficult and sometimes hazardous.

In this thesis an attempt has been made to investigate some of the Engineering properties of soil in Woldiya town. Woldiya is a hillside market town, capital of the Semien Wollo Zone, and Woreda in Northern Ethiopia. Located North of Dessie and Southeast of Lalibela in the Amhara Region. This town has a latitude and longitude of 11°50 N 39°36 E and an average elevation of 2112 meters above sea level. Based on the 2007 national census conducted by the Central Statistical Agency of Ethiopia (CSA), this town has a total population of 46,139, of whom 23,000 are men and 23,139 women.

Woldiya is one of the developing town in Ethiopia. At the moment new buildings and university are on the way of construction. Furthermore, the town is expanding in different direction in order to accommodate the increasing population. By nature soil properties varies from place to place. Some type of the soil heave creates cracks on buildings, roads and other manmade structures. Therefore, the study of soil properties has great importance to save money for maintenance of the structures.

The aim of the thesis is to investigate the different properties of soil in Woldiya town. Once these properties are known, the engineers take into consideration the effect of soil properties in there construction. The importance of knowing soil properties and their type is so crucial for any structures built on soils.

1.2. Objectives of the Study

The objective of this thesis work is the following;

- To determine the index properties of soil in Woldiya town. Such as: Natural moisture content, specific gravity, Consistence limits, Grain size analysis, free swelling of the soil.
- To determine the consolidation characteristic of soils in the city.
- To determine the shear strength parameters.
- To classify the Woldiya town soil based on different classification system.

1.3. Methodology

1.3.1. Reconnaissance of the area

The engineer visually should inspect the site and the surrounding area. In many cases, the information gathered from such a trip is invaluable for future planning. The type of vegetation at a site, in some instances, may indicate the type of subsoil that will be encountered. Open cuts near the site provide an indication about the subsoil stratification. Cracks in the walls of nearby structure(s) may indicate settlement from the possible existence of soft clay layers or the presence of expansive clay soils.

1.3.2. Sampling and Data collection

To achieve the objective of the study, test pits in the representative area were excavated. Ten test pits have been excavated, among this test pits a representative sample of soil from six test pits were taken. Disturbed and undisturbed samples were taken at varying soil profile. After careful sampling, samples are transported to laboratory.

1.3.3. Laboratory tests

After careful transportation of the disturbed and undisturbed samples to the laboratory, the following tests were done;

- Specific gravity test
- Atterberg limit tests
 - Liquid limit
 - Plastic limit
- Grain size analysis
 - Sieve analysis(wet method)
 - Hydrometer
- Free swell test
- Shear test
- One-dimensional consolidation test

1.4. Scope of the Study

This study is restricted to the investigation of index properties, shear parameter determination and consolidation characteristic of the soil. Due to budget and time limitation it is difficult to study all the engineering properties of soil. The depth of ground investigation is limited to three meters since it is difficult to excavate and sampling manually beyond this depth.

1.5. Structure of the Thesis

This thesis work is divided in to six Chapters, each covering a specific topic of the research work. In this introductory Chapter the background of the problem, objective, methodology and scope of the thesis work and structure of the thesis are presented. Chapter two deals with a brief literature review. Chapter three deals with the description of area in which this research is done. The fourth Chapter deals with in-situ properties with sample description and the types of laboratory tests conducted and results obtained. The classification of soil in the study area by different classification methods, Discussion of the laboratory test results and comparison with previously done researches is covered in Chapter five. Finally, conclusions and recommendations drawn from the research area is presented in Chapter six.

2. Literature Review

2.1 Genesis of Expansive Soils

The origin of expansive soils is related to a combination of conditions and processes that result in the formation of clay minerals having a particular chemical makeup which, when in contact with water expands. Variations in the conditions and processes may also form other clay minerals, most of which are non expansive. The conditions or processes, which determine the clay mineralogy, include composition of the parent material and degree of physical and chemical weathering to which the materials are subjected.

Parent Material

The constituents of the parent material during the early and intermediate stages of the weathering process determine the type of clay formed. The nature of the parent material is much more important during these stages than after intense weathering for long periods of time. The parent materials that can be associated with expansive soils are classified into two groups. The first group comprises the basic igneous rocks and the second group comprises the sedimentary rocks that contain montmorillonite as a constituent. Basic igneous rocks are comparatively low in silica, generally about 45% to 52%. Rocks that are rich in metallic base such as the pyroxenes, amphiboles, biotite and olivine fall within this category. Such rocks include the gabbros, basalts and volcanic glass. The sedimentary rocks that contain montmorillonite as constituent include shale and clay stones. Limestone and marls rich in magnesium can also weather to clay. These constituents of the shales and clay stones contain varying amount of volcanic ash and glass, which are subsequently weathered to montmorillonite. These constituents of the shales and clay stones contain varying amounts of volcanic ash and glass, which were subsequently weathered to montmorillonite. The volcanic eruptions sent up clouds of ash, which fell on the continents and sea. Some of the fine grained sediments which accumulated to form these rocks also contain montmorillonite derived from weathering of continental igneous rocks and from ash, which fell on the continental areas as clouds of ash from volcanic eruptions can fall on continents and sea [12].

Weathering

The weathering processes by which clays are formed include physical, chemical and biological processes. Since water is the prime source of weathering, climatic conditions play an important role in the rate and extent of weathering of the parent material. Climate affects the amount of water available to infiltrate the subsurface materials and the temperature, which in turn affects the vegetative growth. High temperature and year round rainfall favors the formation of low activity kaoline and oxides. As rainfall decreases, high activity smectic clays predominate. Climate ultimately becomes the most dominant factor in residual soil formation than does parent material. The formation of expansive clays or montmorillonite clays is

avored by alkaline environment, the absence of leaching, the presence of ferromagnesium minerals in parent materials and the presence of bases. Prolonged leaching under high temperatures or tropical conditions favors formation of minerals of the kaolinite group, which are non expansive. The presence of potash in the parent material under these conditions results in the formation of illite that shrinks and swells a little more than kaolinite, but shrinks and swells considerably less than the montmorillonite clays [12].

2.2 Mineralogical Structure

Lattice structure of clays can have fundamental molecular structure of alumina octahedron and the silica tetrahedron as basic units. The alumina octahedron consists of an aluminium atom surrounded octahedrally by six oxygen ions. When each oxygen atom is shared by two tetrahedra, a plate-shaped layer is formed. The silica tetrahedron consists of a silicon atom surrounded tetrahedrally by oxygen ions. The tetrahedrons are arranged so that tips of all of them point in the same direction, and the bases of all tetrahedrons are in the same plane. The above mineralogical structures lead to the formation of different types of clay mineral, namely kaolinite, halloysite, montmorillonite, illite, etc. But the most common minerals found in clay soils are grouped as kaolinite, illite and montmorillonite. They are essentially hydrous aluminum silicates. Since clay minerals are products of chemical weathering of rocks, both climate, which determines weathering, and the parent rock, which influences the type of minerals found play major role in their formation. Montmorillonite, the principal clay mineral of expansive soil is made up of identical units of aluminum octahedral sheets between two silica tetrahedral sheets. These sheets are bound rather loosely and are very unstable in water. Water molecules easily enter between the sheets and this is the cause for the highly expansive nature of clays containing this mineral group [6].

2.3 Identification and Classification of Expansive Soils

Investigation of expansive soils generally consists of two important phases. The first is the recognition and identification of the soil as expansive and the second is sampling and measurement of material properties to be used as the basis for design. The theme of this topic is to discuss tests and classification procedures that are commonly used to identify expansion potential.

2.3.1 Field Identification

Soils that can exhibit high swelling potential can be identified by field observations, mainly during reconnaissance and preliminary investigation stages. Important observations include [6]:

- ✓ Usually have a color of black or gray.
- ✓ Wide or deep shrinkage cracks.
- ✓ High dry strength and low wet strength.
- ✓ Stickiness and low trafficability when wet.
- ✓ Cut surfaces have a shiny appearance.
- ✓ Appearance of cracks in nearby structures.

- ✓ Arid and semiarid areas are particular trouble spots because of large variations in rainfall and temperature.

2.3.2 Laboratory Identification

Methods to identify expansive soils in the laboratory can be categorized into three methods.

2.3.2.1 Mineralogical Identification

It is generally believed that swelling potential of any clay can be evaluated by identification of constituent mineral of that clay. The techniques that can be used are:

- ✓ X-ray diffraction
- ✓ Differential thermal analysis
- ✓ Dye adsorption
- ✓ Chemical analysis
- ✓ Electron microscope resolution

Using combinations of these methods, the different types of clay minerals present can be evaluated quantitatively.

2.3.2.2 Indirect Methods

These methods include the index property, Potential Volume Change (require special PVC apparatus) and activity methods, which are valuable tools in evaluating the swelling property. However, none of the indirect methods should be used independently.

2.3.2.3 Direct Methods

These methods offer the most useful data by direct measurement and tests are simple to perform and do not require complicated equipments. Testing should be performed on a number of samples to avoid erroneous conclusions. Direct measurement of expansive soils can be achieved by the use of conventional one-dimensional consolidometer.

2.3.3 Classification Methods

Parameters determined from expansive soil identification tests have been combined in a number of different classification schemes. The classification system used for expansive soils are based on indirect and direct prediction of swell potential as well as combinations to arrive at a rating.

2.3.3.1 Classification Using General Methods

Soils are classified in the general schemes: Unified Soil Classification System (USCS) and the American Association of State High way and Transportation Officials (AASHTO) method according to index properties.

2.3.3.2 The Van Der Merwe Method

The Van Der Merwe method, also called the South African method, gives a very practical approach to classify and estimate heave of expansive soils [6]. The method starts by classifying the swell potential of soil in to very high to low categories (Fig 2.1).

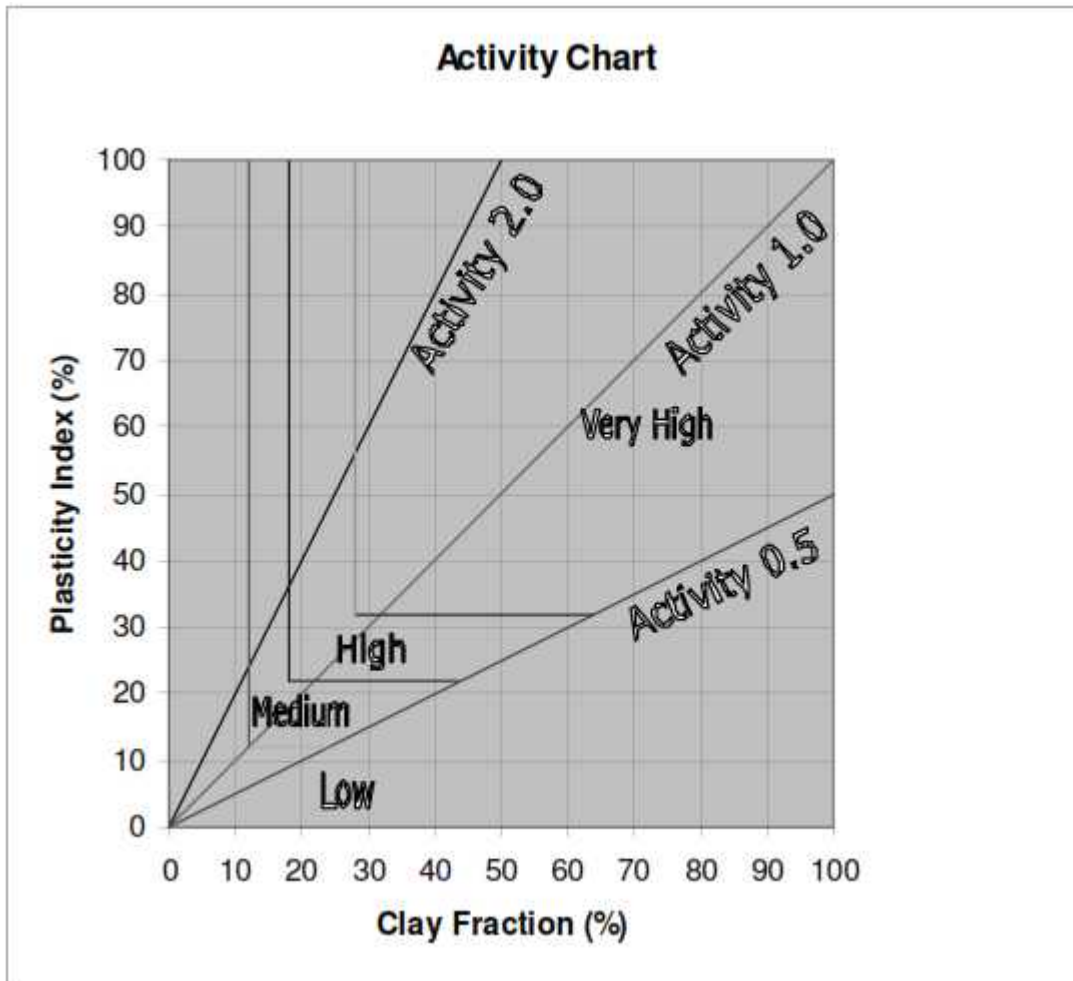


Figure 2.1 Volume change potential classification for clay soils

The activity here is defined as:

$$A_c = PI/C \dots\dots\dots 2.1$$

Where,

- A_c = activity number
- C = percentage of clay size finer than 0.002mm and
- PI = plasticity index.

2.4 Mechanics of Swelling

If environment of an expansive soil has not been changed, swelling does not take place. Environmental change can consist of pressure release due to excavation, desiccation caused by temperature increase, and volume increase due to moisture introduction. By far the most important element is the effect of water on expansive soils. There must be a potential gradient, which can cause water migration, and a continuous passage through which water transfer can take place. With the introduction of water, volumetric expansion takes place. If pressure is applied to prevent expansion, the pressure required to maintain the initial volume is the swelling pressure. Thickness and location of potentially expansive layers in a profile considerably influence potential movement. Greatest movement will occur in profiles that have expansive clays extending from the surface to depths below the active zone. Less movement will occur if expansive soil is overlain by non-expansive material or have got shallow depths. Water contents in the upper few meters of the expansive soil profile are affected by environmental factors and generally called zone of seasonal variation or the active zone (Fig 2.2).

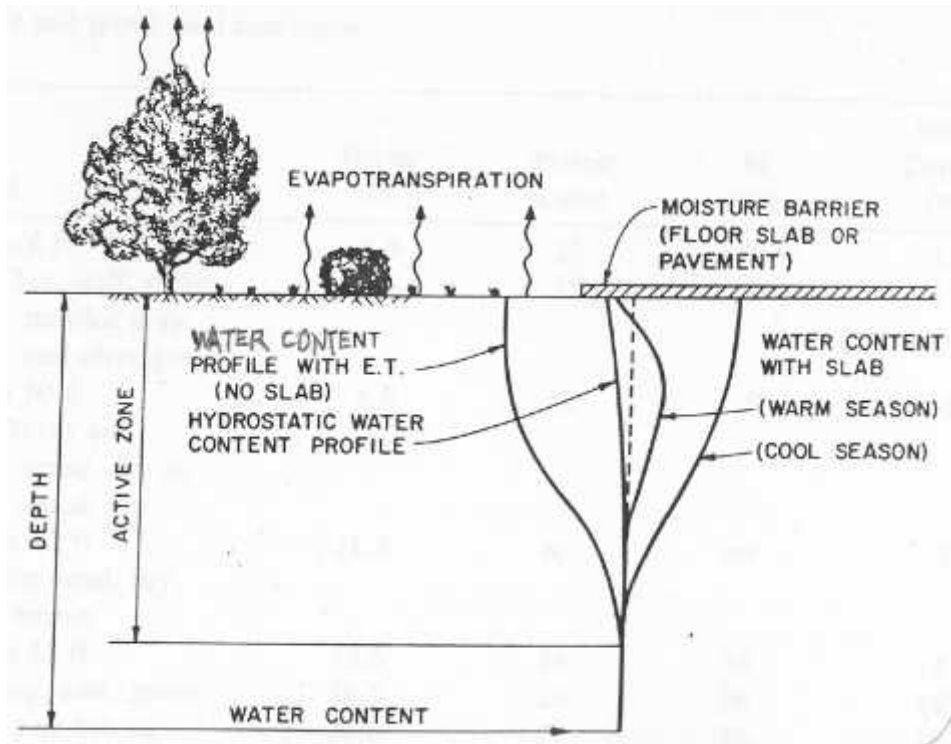


Figure 2.2 Water content profiles in the active zone [20]

In fine-grained soils, capillary force is a significant means of water transfer. The height of water rise into the capillary fringe varies inversely with the radius of the capillary tube. In clean, coarse gravel, the capillary rise is insignificant. In clean sands, the rise is a few centimeters; in fine sands, the rise is 30 to 60 centimeters; in silt up to 3 to 4 meters; and in clay, a rise of more than 300 meters is theoretically possible

[6]. It is well known that heaving of expansive soils may take place without the presence of free water. Vapor transfer plays an important role in providing the means for the volume increase of expansive soils.

3. Description of the study area

3.1. General

Woldiya is a hillside market town, capital of the Semien Wollo Zone, and Woreda in Northern Ethiopia. Located North of Dessie and Southeast of Lalibela in the Amhara Region. This town has a latitude and longitude of 11°50 N 39°36 E and an elevation of 2112 meters above sea level. Based on the 2007 national census conducted by the Central Statistical Agency of Ethiopia (CSA), this town has a total population of 46,139, of whom 23,000 are men and 23,139 women. Woldiya was found 520kms far from north of Addis Ababa. The town has four urban kebeles and four rural kebeles.



Figure 3.1 location of the research area on the map of Ethiopia

3.2. Geology

It is believed that knowledge of the geology in urban planning will help in understanding geological processes of denudation and deposition and the risk of geological hazards in the area, as these could affect in the long run the development of urban areas[24].

Woldiya area is characterized by rugged topography which is the result of geological processes that had taken place in the area. Governed by the topographic setting, the slope and flow pattern of streams of the area shows irregularities. Thus, the general slope of the area falls to southwest and finally towards the west. following this irregular features Shele River drains the urban area joins Tikur Wuha[24].

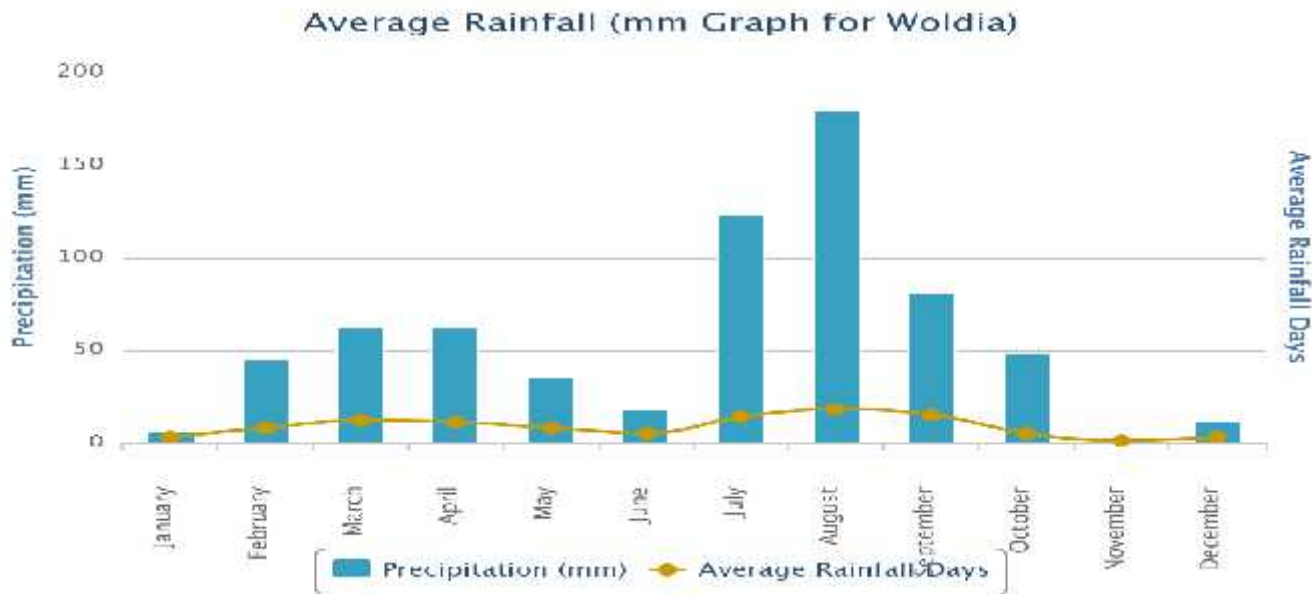
The geology of Woldiya area is built by Tertiary volcanic which are grouped into three main formations know by Hashangie, Aiba Basalts and Alajae formation. The Hashangie formation is the predominant outcrop in the town area and it is deeply weathered rocks that occupy the lower succession. Aiba Basalts are compact rocks found in the area second to Hashangie while Alajae formation is dominant on the western side of the town and widely used as construction material in the area[24].

Being part of the Ethiopian plateau, Woldiya lies along the lower surface of Northern Highlands. Woldiya is situated within the corridor of the two volcanic ridges that lie on the north and south of the town. On the north, the surface of the ridge is dissected in parallel and aligned in the direction of the slope orientation which is towards the built up area of the town. While the southern ridge displays more steep slope and less dissected surface as compared to the northern one. This indicates that there is some difference in the rock materials from which the ridges are formed[24].

3.3. Climate

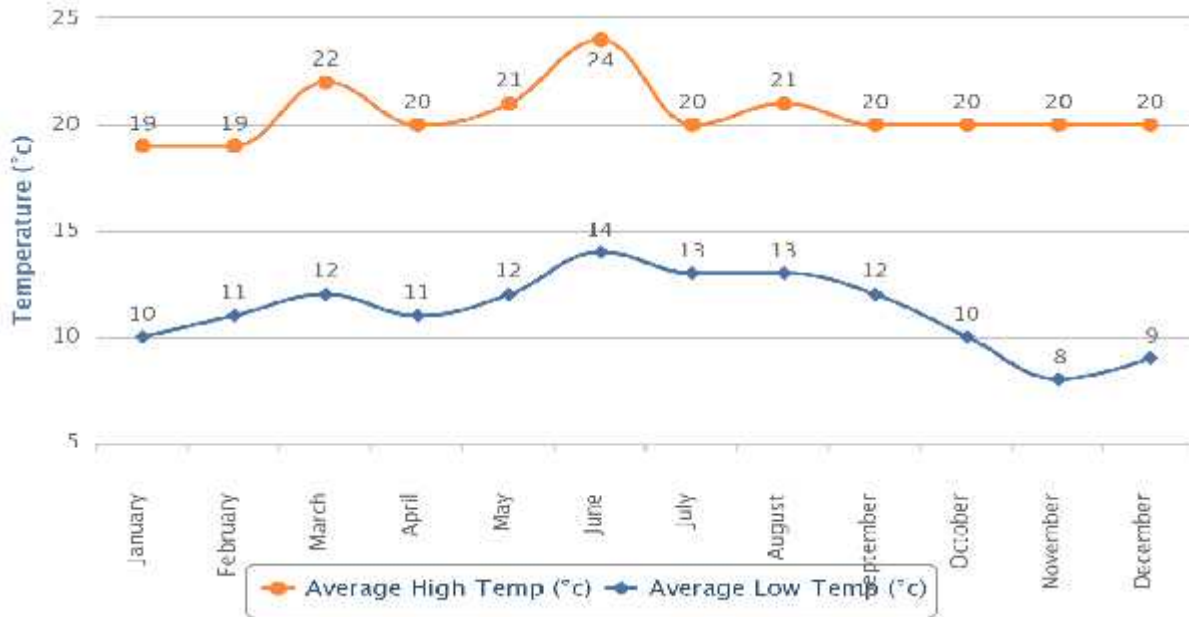
3.3.1. Rainfall

There is no National Meteorological Service Agency in Woldiya town . But, the average Rain fall from world weather online is given by:



3.3.2. Temperature

In a mountainous tropical country like Ethiopia altitude is by far the most important factor in controlling climate. It affects distribution of both temperature and rainfall. The average high or low temperature for Woldiya town is as follows by world weather online.



4. In-situ Properties and Laboratory Tests Results

4.1. In-situ Properties

4.1.1. Identification of Soil in the Study Area

Site visiting is the first and foremost for investigation of soil properties. In Woldiya building construction is taking place throughout the town. Site visit was made to the construction places to get information about the texture of soils in the town. Furthermore, Consulting with the town municipality administrative body and other concerned people were also arranged to collect information about the geology, soil texture and other historical futures of the town. After observation of the soil type in the whole town, ten sampling areas were selected from different parts of the town. Test pits were excavated up to a maximum depth of 3.0 m. Among the ten test pits, representative samples from five pits were taken due to similarity of soil textures. Disturbed and undisturbed samples were collected from this representative samples. But, taking undisturbed sample was so difficult since the soil was very stiff. The global coordinates of sampling location i.e. northing and easting shown in Table 4.1.

Table 4.1 Global coordinates of sampling area

Test pits	Location	Northing	Easting
TP-1	Gonder ber	563174.684	1307601.0
TP-2	Mugad	564334.7975	1307672.3446
TP-3	Adago	564749.9167	1307583.2047
TP-4	Inside hospital	565152.4662	1308017.2266
TP-5	In font of bus station around the condominimem	564938.1481	1307088.6772

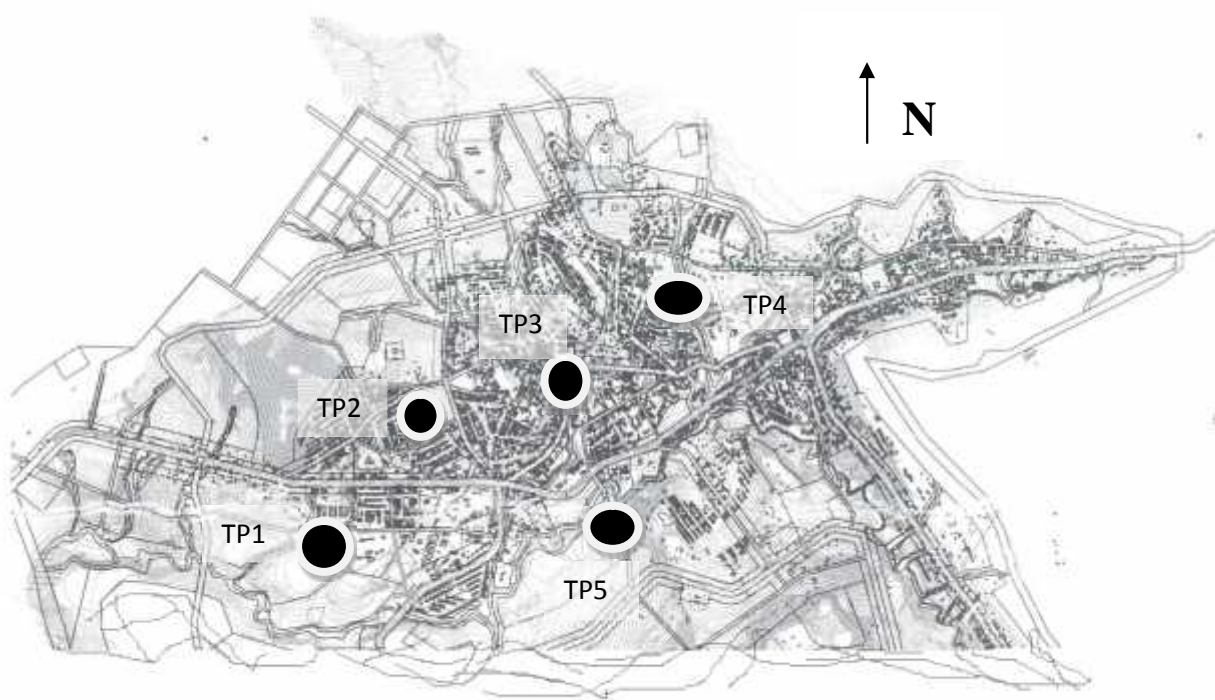


Figure 4.1 Location of sampling areas shown on map of Woldiya town

4.2. Laboratory Test Results

4.2.1. Index Properties

4.2.1.1. General

The tests required for the determination engineering properties are elaborate and time consuming. Sometimes the geotechnical engineer is to have some rough assessment of the engineering properties without elaborate tests. This is possible if index properties are determined. The properties of the soil which are not of primary interest to geotechnical engineers but which are indicative of the engineering properties are index properties. Simple tests which are required to determined the index properties are classification tests. The soils are classified and identified based on index properties. Some of the index properties of soil are; Natural moisture content, Specific gravity, Consistence limits, Grain size analysis, free swelling of the soil.

4.2.1.2. Natural Moisture Content

The water content of a soil is an important parameter that controls its behavior. It is a quantitative measure of the wetness of a soil mass. The water content of a soil can be determined to a high degree of precision, as it involves only mass which can be determined more accurately than volumes[1]. For many soils, the water content may be an extremely important index used for establishing the relationship between the way a soil behaves and its properties. The consistency of a fine-grained soil largely depends on its water

content. The water content is also used in expressing the phase relationships of air, water, and solids in a given volume of soil[2].The test results of the moisture content in the study area is given in the following Table 4.2.

Table 4.2 Natural moisture content of the soil samples

Test pits	Location	Northing	Easting	Natural moisture content(%)
TP-1	Gonder ber	563174.684	1307601.0	31.72
TP-2	Mugad	564334.7975	1307672.3446	29.45
TP-3	Adago	564749.9167	1307583.2047	-----
TP-4	Inside hospital	565152.4662	1308017.2266	31.08
TP-5	In font of bus station around the condominimem	564938.1481	1307088.6772	28.5

For test pit (TP-3) the natural moisture content has not been determined due to the fact undisturbed sample not taken.

4.2.1.2. Specific Gravity

Specific gravity of soil is the ratio of weight of a given volume of soil particles in air at a stated temperature to the weight of an equal volume of distilled water at a stated temperature. The specific gravity of a soil is often used in relating a weight of soil to its volume. The specific gravity of a soil is used in calculating the phase relationships of soils (that is the relative volumes of solids to water and air in a given volume of soil). The term solid particles are typically assumed to mean naturally occurring mineral particles that are not readily soluble in water. ASTM D 854-00 Standard Test for Specific Gravity of Soil Solids by Water Pycnometer is used for the determination of the specific gravity of soil. The laboratory test results are shown in the following Table.

Table 4.3 Specific gravity of soil in the study area

Serial No	Test pits	Depth(m)	Results
1	TP1	1.5	2.89
		3.0	2.76
2	TP2	3.0	2.79
3	TP3	0.9	2.65
		3.0	2.67
4	TP4	1.5	2.75
		3.0	2.78
5	TP5	3.0	2.80

As it can be seen from the result the specific gravity of Woldiya soil ranges from 2.65-3.0. The soil is clay soil and its specific gravity varies with the range based on mineral content of the soil.

4.2.1.3. Grain-size Distribution of Soil

4.2.1.3.1. General

In any soil mass, the sizes of the grains vary greatly. To classify a soil properly, you must know its grain-size distribution. The grain-size distribution of coarse-grained soil is generally determined by means of sieve analysis. For a fine-grained soil, the grain-size distribution can be obtained by means of hydrometer analysis.

4.2.1.3.2. Sieve Analysis

A sieve analysis is conducted by taking a measured amount of dry, well-pulverized soil and passing it through a stack of progressively finer sieves with a pan at the bottom. The amount of soil retained on each sieve is measured, and the cumulative percentage of soil passing through each is determined. This percentage is generally referred to as percent finer. The distribution of different grain sizes affects the engineering properties of soil. Grain size analysis provides the grain size distribution, and it is required in classifying the soil. ASTM D 422 - Standard Test Method for Particle-Size Analysis of Soils is used in the test analysis.

4.2.1.3.3. Hydrometer Analysis

Hydrometer analysis is based on the principle of sedimentation of soil grains in water. When a soil specimen is dispersed in water, the particles settle at different velocities, depending on their shape, size, and weight, and the viscosity of the water.

Soil particle sizes smaller than 0.075 mm (passing 200 mesh sieve) are determined by hydrometer method. It is based on the process of sedimentation of soil particles in water by gravity. The steady fall of soil particles through a liquid at rest is called sedimentation. The hydrometer method is based on Stokes equation that relates the velocity of free falling spherical particle through a liquid to the diameter of the particle, the specific gravity of the particle and the viscosity of the liquid. The hydrometer analysis assume that, the soil particles are spheres, the soil suspension is sufficiently low concentration to permit individual settling of grains without interference by others. ASTM D 422 - Standard Test Method for Particle-Size Analysis of Soils is used in the test analysis.

The summary of the sieve and hydrometer analysis is shown in the following Figure4.2. Except test pit three the rest test pit shows similar pattern.

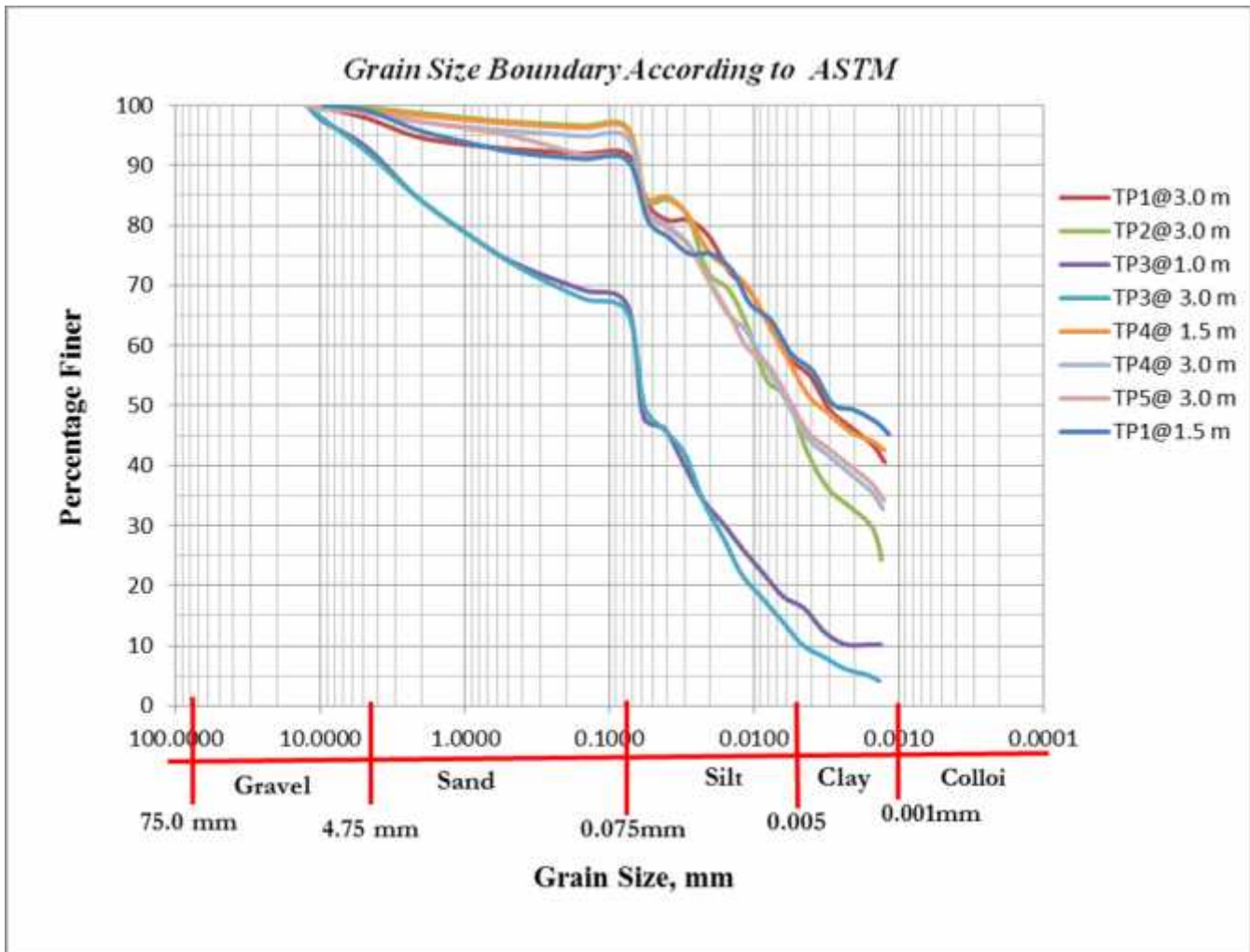


Figure 4.2 Grain size distribution curve for the combined sieve and hydrometer analysis in the study area

From the graph we can observe the following results;

- For test pit (TP3) the percentage of soil passing sieve no.200 is more than 65%.
- For the other test pits the percentage of soil passing sieve no.200 is more than 90%. This means the constitute of the soil is mainly fine grained soils. The hydrometer analysis shows the gradual falling of particles; it indicates the clay nature of the soil.

4.2.1.4 Atterberg limits

4.2.1.4.1. General

When clay minerals are present in fine-grained soil, the soil can be remolded in the presence of some moisture without crumbling. This cohesive nature is caused by the adsorbed water surrounding the clay particles. At a very low moisture content, soil behaves more like a solid. When the moisture content is very high, the soil and water may flow like a liquid. Hence, on an arbitrary basis, depending on the moisture content, the behavior of soil can be divided into four basic states; solid, semisolid, plastic, and

liquid. The moisture content, in percent, at which the transition from solid to semisolid state takes place is defined as the shrinkage limit. The moisture content at the point of transition from semisolid to plastic state is the plastic limit, and from plastic to liquid state is the liquid limit. These parameters are also known as Atterberg limits[7]

4.2.1.4.2. Test Procedure

Atterberg Limits were determined for air-dried samples. It was done based on the Standard Reference: ASTM D 4318-Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. The air-dried samples were prepared by spreading the specimen in the air until it dried. The sample of soil passing sieve No 40(.425mm) is used to determine the Atterberg Limits.

The moisture content, in percent, required to close a distance of 12.7 mm (0.5 in.) along the bottom of the groove after 25 blows is defined as the liquid limit. It is difficult to adjust the moisture content in the soil to meet the required 12.7 mm (0.5 in.) closure of the groove in the soil pat at 25 blows. Hence, at least three tests for the same soil are conducted at varying moisture contents, with the number of blows, N, required to achieve closure varying between 15 and 35[7].

About 15 gm of soil passing through sieve No. 40 (ASTM), mixed thoroughly with water. The soil is rolled on a glass plate with the hand, until it is about 3 mm in diameter. This procedure of mixing and rolling is repeated till the soil shows signs of crumbling when the diameter is 3 mm. The water content of the crumbled portion of the thread is determined. This is called as plastic limit[10].

For determination of shrinkage limit, a container of known volume V1 is filled with plastic soil in the saturated state. The weight of the saturated soil is determined. The specimen is dried gradually first in air and then in an oven at a constant temperature of 105^{0c}. After oven drying the specimen weighed. Following the weighing, its volume is determined by displacement in mercury[10]. The Atterberg Limits for soil in Woldiya town are summarized in Table 4.4 and from this we can observe that liquid limit ranges from 34 – 97 %, plastic limit ranges from 28 – 35 % and plastic index from 5 – 63%.

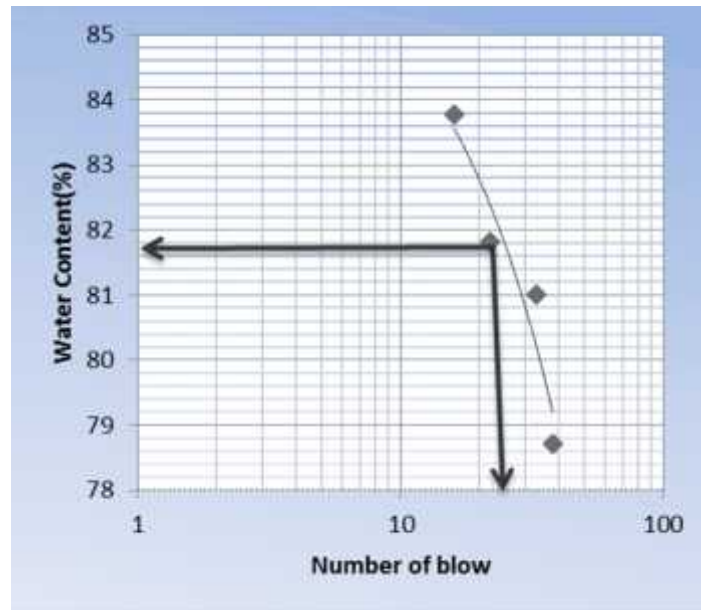


Figure 4.3 Typical liquid limit determinations for test pit # 4 at 1.5 m

Table 4.4 Results of Atterbeg limits

No.	test pit	Depth	Liquid Limit	Plastic Limit	Plasticity Index
1	TP1	1.5	96.63	34.08	62.55
		3.0	90.84	32.95	57.89
2	TP2	3.0	72.77	32.54	40.23
3	TP3	0.9	39.97	30.64	9.33
		3.0	34.11	28.62	5.49
4	TP4	1.5	81.79	33.78	48.01
		3.0	73.02	31.27	41.75
5	TP5	3.0	74.06	30.97	43.09

4.2.1.5. Free Swelling

Both the amount of swelling and the magnitude of swelling pressure are known to be dependent on the clay minerals, the soil mineralogy and structure, fabric and several physico-chemical aspects of the soil. Among clay minerals Montmorillonite influence the magnitude of swelling maximally as compared to Illites and Kaolinites [10].

To study the swelling property of the soils, the simplest test conducted is free swell test. This test is performed by slowly pouring 10ml of oven dry soil which has passed the No. 40(0.425mm) sieve in to 100 ml graduated cylinder filled with distilled (tap) water. After 24 hours, final volume of the suspension being read. Hence, free swell is defined as:

$$\text{Free swell} = \frac{\text{Final volume} - \text{Initial volume}}{\text{Initial volume}} \times 100\% \dots\dots\dots(4.1)$$

Table 4.5 Free swell test results

No.	test pit	Depth(m)	Free swell(%)
1	TP1	1.5	128
		3.0	120
2	TP2	3.0	88
3	TP3	0.9	45
		3.0	39
4	TP4	1.5	100
		3.0	72
5	TP5	3.0	85

4.2.2. Shear Strength of Soil

4.2.2.1.General

The shear strength of a soil mass is the internal resistance per unit area that the soil mass can offer to resist failure and sliding along any plane inside it. The shear strength of a soil is its maximum resistance to shear stresses just before the failure. Soils are seldom subjected to direct shear. However, the shear stresses develop when the soil is subjected to direct compression. Thus, the shear failure of a soil mass occurs when the shear stresses induced due to the applied compressive loads exceed the shear strength of the soil. It may be noted the failure in soil occurs by relative movements of the particles and not by breaking of the particles [1].

Shear strength is the principal engineering property which controls the stability of a soil mass under loads. It governs the bearing capacity of soils , the stability of slopes in soils , the earth pressure against retaining structure and many other problem. All the problem of soils engineering are related in one way

or the other with the shear strength of the soil[1].To know the shear strength of soil in the study area samples were collected from representative test pits.

Disturbed and undisturbed samples were collected during the driest season. The soil was very stiff, when try to do unconfined compression test it shows crumple before the run of the test. Because of this reason it is impossible to do unconfined compression test in this thesis. Even though, unconfined compression test is not done, direct shear test was done to show the shear strength of soil in the town.

The shear strength parameters can be obtained from $\tau - \sigma$ diagram plotted from the test results. During the test, the horizontal force, vertical force and the corresponding displacements are recorded.

$$\sigma = \frac{P}{A}, \quad \tau = \frac{F}{A} \dots\dots\dots (4.2)$$

Where: F= the shear force ,A= corrected area ,P= vertical force , σ = normal stress and τ =shear stress

Three different normal forces are taken for this test (i.e.100kN/m², 200kN/m², and 300kN/m²).

From the test results we plotted the Mohr- Coulomb failures envelop from which we can easily determine the shear strength parameters C and

The formula is

$$\tau = C + \sigma \tan \phi \dots\dots\dots (4.3)$$

Where:-

σ = normal stress, τ =shear stress, C = is the cohesion and ϕ = is angle of internal friction

#Test pit 4

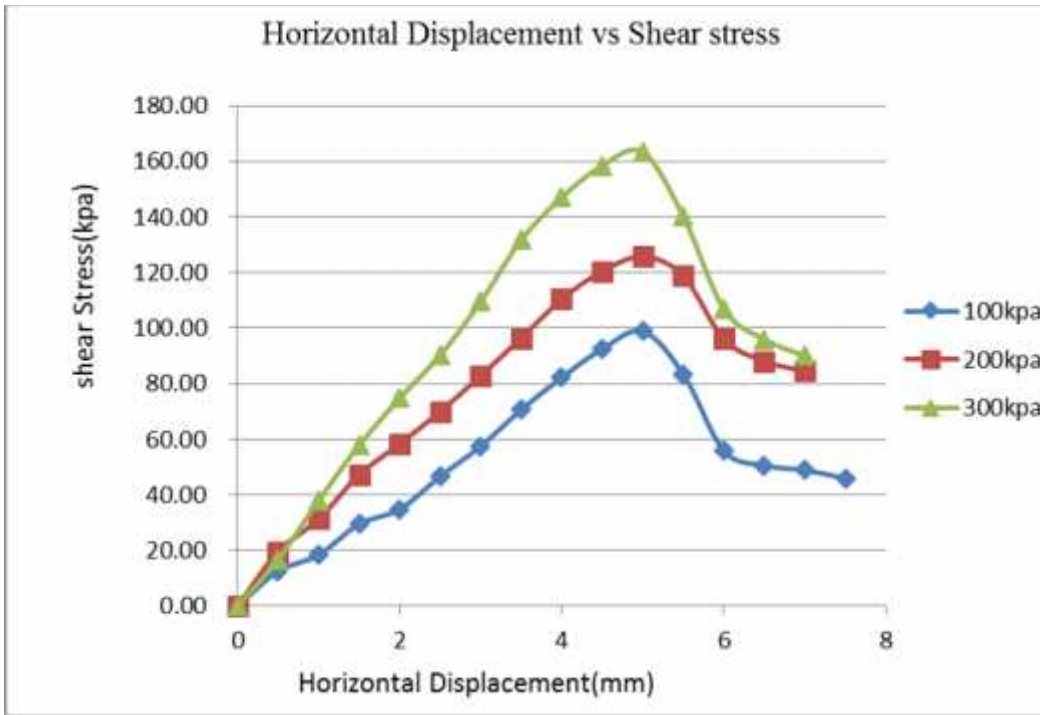
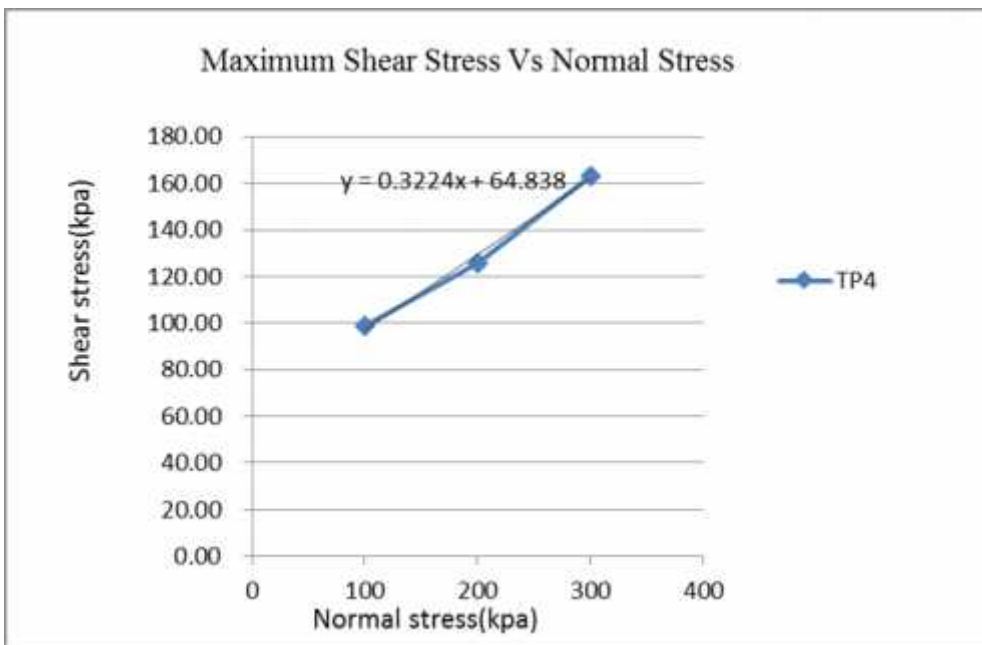


Figure 4.4 Horizontal displacement vs Shear stress curve for test pit 4



C = is the cohesion = 64.8kPa

ϕ = is angle of internal friction = $\tan^{-1}(0.322) = 17.9^\circ$

#Test pit 5

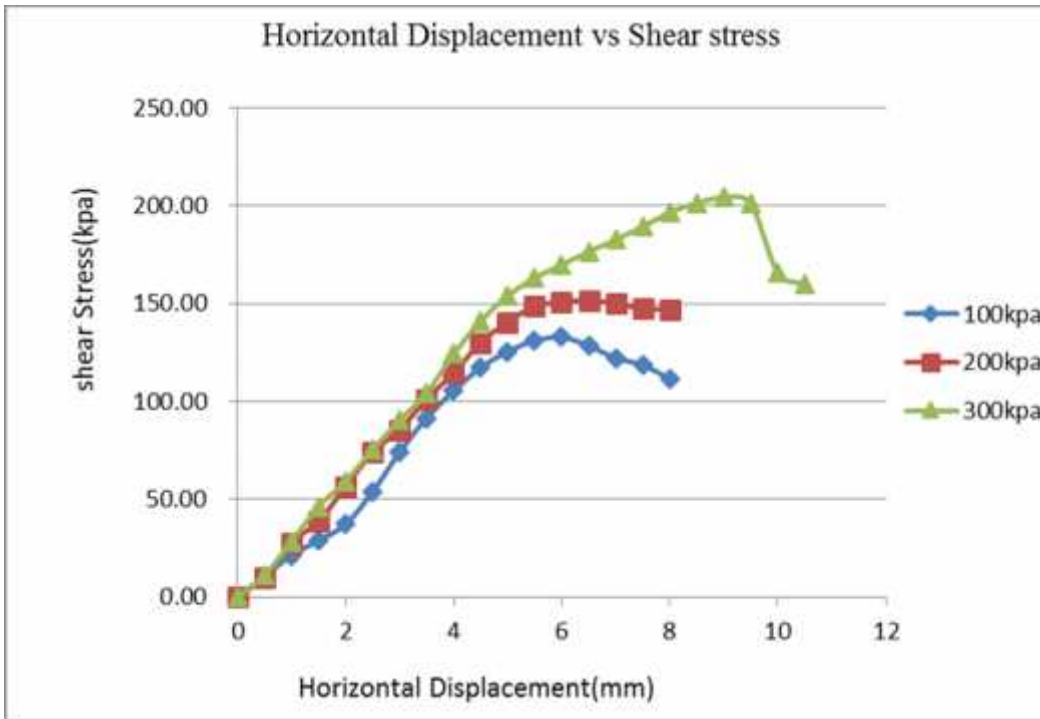
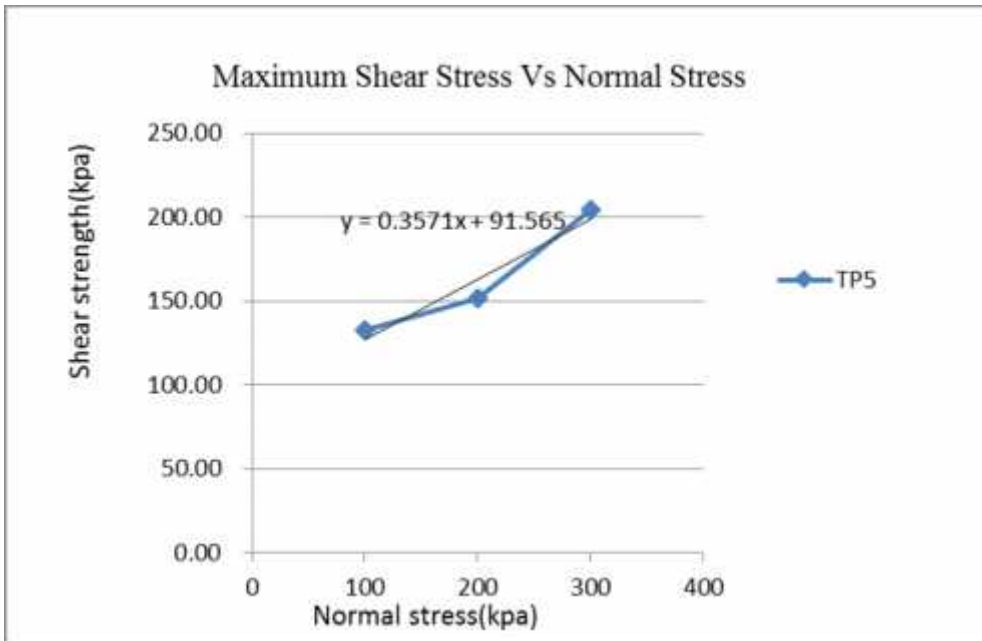


Figure 4.5 Horizontal displacement vs shear stress curve for test pit 5



C = is the cohesion =91.6kPa

ϕ = is angle of internal friction= $\tan^{-1}(0.357) = 19.7^\circ$

4.2.3. Consolidation

This test was done according to ASTM standard, Designation D2435-96. Undisturbed sample of soil with their specific gravity and initial moisture content was prepared. Using sample extruder sample was extruded on to the consolidation ring. After carefully trimming the soil sample at its top and bottom, it was placed inside the metal ring with porous stone at its top and bottom. A setting load of 7kPa was applied until the soil fully saturated.

Loads are applied in steps in such a way that the successive load intensity, p , is twice the preceding one. The load intensities commonly used being 1/4, 1/2, 1, 2, 4, 8, and 16 tons/ft² (25, 50, 100, 200, 400, 800 and 1600 kN/m²). Each load is allowed to stand until compression has practically ceased (no longer than 24 hours). The dial readings are taken at elapsed times of 1/4, 1/2, 1, 2, 4, 8, 15, 30, 60, 120, 240, 480 and 1440 minutes from the time the new increment of load is put on the sample (or at elapsed times as per requirements)[18].

4.2.3.1. Pre-consolidation Pressure

The maximum pressure to which an over consolidated soil had been subjected in the past is known as the pre-consolidation pressure or over consolidation pressure (P_c). When a soil specimen is taken from a natural deposit, the weight of the overlying material (over-burden) is removed. This causes an expansion of the soil due to a reduction in pressure [1].

Several methods have been proposed for determining the value of the maximum consolidation pressure. There are a few graphical methods for determining the pre-consolidation pressure based on laboratory test data. No suitable criteria exist for appraising the relative merits of the various methods. The earliest and the most widely used method was the one proposed by Casagrande (1936). The method involves locating the point of maximum curvature, B , on the laboratory e -log p curve of an undisturbed sample as shown in Fig. 6.3. From B , a tangent is drawn to the curve and a horizontal line is also constructed. The angle between these two lines is then bisected. The abscissa of the point of intersection of this bisector with the upward extension of the inclined straight part corresponds to the pre-consolidation pressure (P_c)[6].

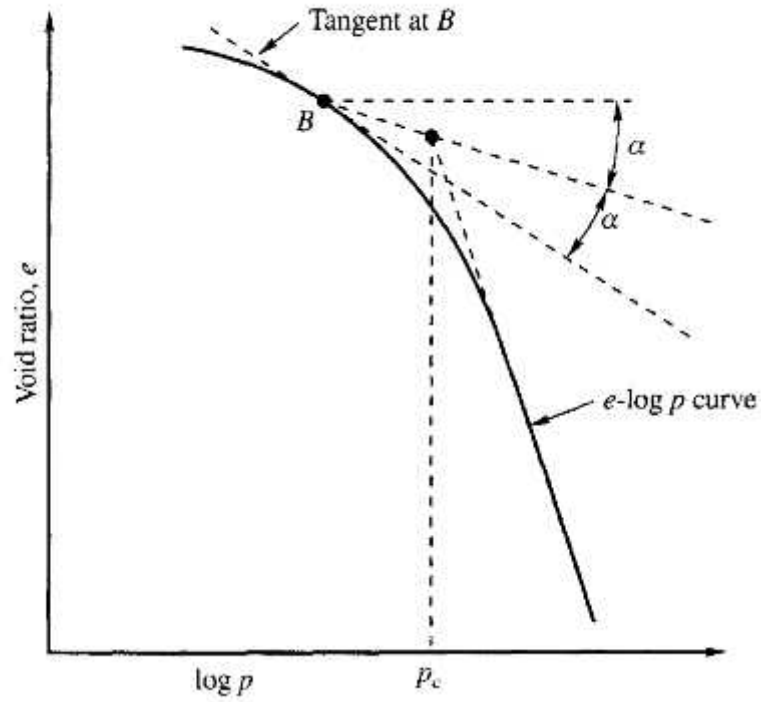
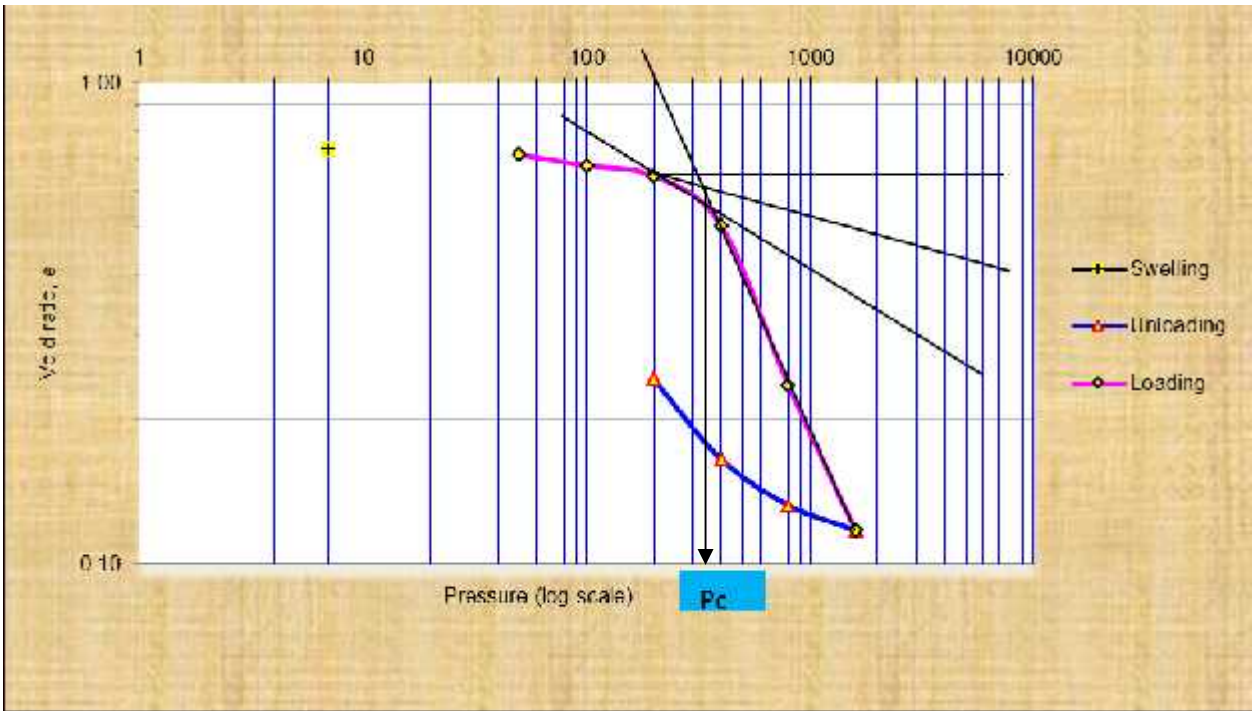


Figure 4.6 Method of determining PC by Casagrande method

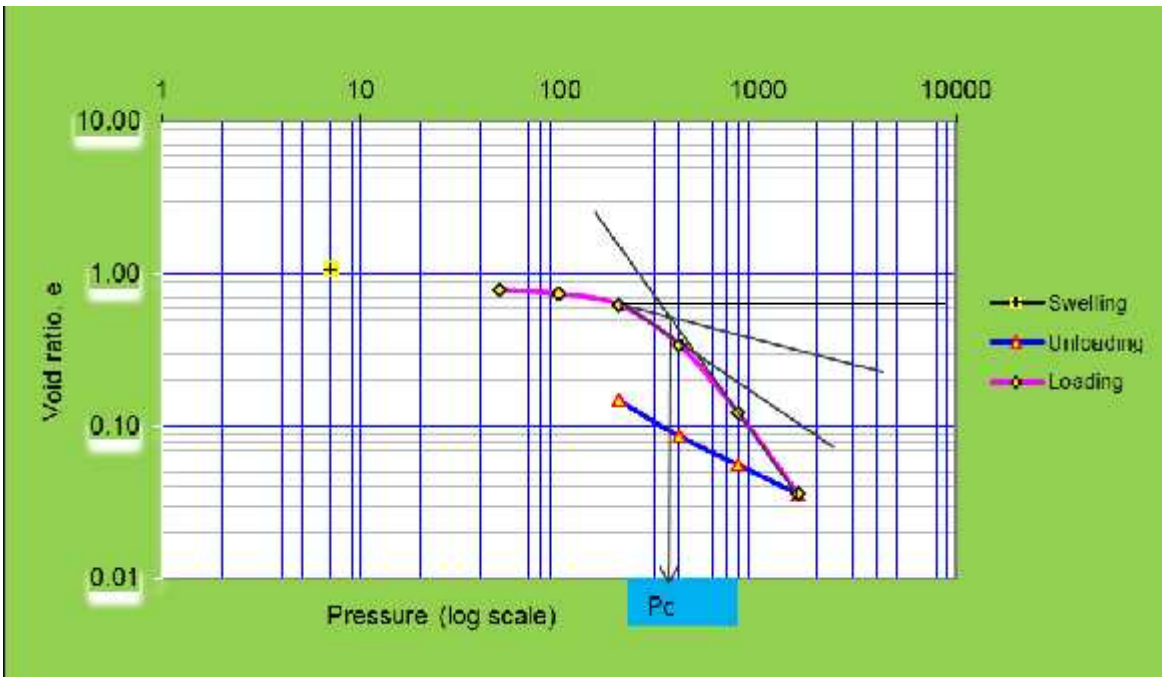
4.2.3.3. Compression Index, C_c

This is numerically equal to the slope of the straight portion of the e - $\log p$ curve. Compression index is an important index used to calculate the ultimate settlement of a foundation founded on a clay layer.

$$C_c = (e_1 - e_2) / (\log p_2 - \log p_1) \dots \dots \dots 4.4$$



a) Void ratio vs log pressure curve for test pit 4



b) Void ratio vs log pressure curve for test pit 5

Figure 4.7 Graphs a and b shows the determination of pre-consolidation pressures for selected test pits

The Consolidation test results of the study area are summarized in the following Table4.6.

Table 4.6 Consolidation test results

No.	Test pits	Sample Depth(m)	Moisture content(%)	Cc	Cr	Average coefficient of consolidation Cv (cm ² /sec)	Pre-consolidation pressure(Pc) (kpa)
1	TP4	3.0	31.08	0.393	0.050	0.0385	340
2	TP5	3.0	28.5	0.287	0.066	0.0312	370

These two test pits were selected due to the similarity of soil textures. The remaining test pits have similar soil texture with either of these test pits.

5. Discussions of the laboratory test results and Comparisons with previously done researches

5.1. Classification of the Soils

Soil classification is the arrangement of soils into different groups such that the soils in a particular group have similar behavior. It is the sort of labeling of soils with different labels. As there is a wide varieties of soils covering the earth, it is desirable to systemize or classify the soils into broad groups of similar behavior. It is more convenient to study the behavior of groups than that of individual soils. A geotechnical engineer is interested to know the suitability or otherwise of a soil as a foundation or a construction material. For complete knowledge , all the engineering properties are determined after conducting a large number of tests. However, an approximate assessment of the engineering properties can be obtained from the index properties after conducting only classification tests[1].

Although, there are many soil classification systems are present in the world , Currently, two more elaborate classification systems are commonly used by soils engineers. Both systems take into consideration the particle-size distribution and Atterberg limits. They are the American Association of State Highway and Transportation Officials (AASHTO) classification system and the Unified Soil Classification System. The AASHTO classification system is used mostly by state and county highway departments. Geotechnical engineers generally prefer the Unified system. The soils under investigation have been classified according to AASHTO and UCSC. These methods are among the widely used classification systems in our country as well as in the entire world.

5.1.1. AASHTO Classification System

According to this system, soil is classified into seven major groups: A-1 through A-7. Soils classified under groups A-1, A-2, and A-3 are granular materials of which 35% or less of the particles pass through the No.200 sieve. Soils of which more than 35% pass through the No.200 sieve are classified under groups A-4, A-5, A-6, and A-7. These soils are mostly silt and clay type materials[7]. Average grain size classification according to USCS [5]

Gravel 75mm - 4.75mm

Sand 4.75mm - 0.075mm

Silt 0.075mm - 0.002mm

Clay < 0.002mm

Table 5.1 Classification of soil based on AASHTO classification system

Serial No.	Test Pits	Depth (m)	Percent amount of particle size				LL (%)	PL (%)	PI (%)	AASHTO Classification	Usual types of significant constituent materials
			Gravel	sand	silt	clay					
1	TP1-1	1.5	4.30	4.85	41.60	49.25	96.63	34.08	62.55	A-7-5	Clayey soils
	TP1-2	3	5.43	2.78	45.46	46.33	90.84	32.95	57.89	A-7-5	Clayey soils
2	TP2	3	1.30	2.42	62.98	33.30	72.77	32.54	40.23	A-7-5	Clayey soils
3	TP3-1	0.9	15.90	17.52	56.38	10.20	39.97	30.64	9.33	A-4	Silt soils
	TP3-2	2.3	15.89	18.65	59.30	6.16	34.11	28.62	5.49	A-4	Silt soils
4	TP4-1	1.5	1.75	2.22	50.46	45.57	81.79	33.78	48.01	A-7-5	Clayey soils
	TP4-2	3	2.8	2.63	55.88	38.69	73.02	31.27	41.75	A-7-5	Clayey soils
5	TP5	3	2.75	6.37	51.02	39.86	74.06	30.97	43.09	A-7-5	Clayey soils

5.1.2. Unified Soil Classification (USC) System

The system is most popular for use in all types of engineering problems involving soils.

This system classifies soils into two broad categories:

- Coarse-grained soils that are gravelly and sandy in nature with less than 50% passing through the No.200 sieve. The group symbols start with a prefix of G or S. G stands for gravel or gravelly soil, and S for sand or sandy soil.
- Fine-grained soils are with 50% or more passing through the No.200 sieve. The group symbols start with prefixes of M, which stands for inorganic silt, C for inorganic clay, or O for organic silts and clays. The symbol Pt is used for peat, muck, and other highly organic soils[7].

5.1.2.1 Plasticity Chart

Plasticity index, numerical difference between liquid limit and plastic limit, represents the range in water content through which a soil is in plastic state. A high numerical value of plasticity index is an indication of the presence of high percentage of clay in the soil sample. Which implies that the plasticity values increase with the responding increase in clay content[25]. The important feature of this chart is the empirical A-line that is given by the equation $PI = 0.73(LL - 20)$. An A-line separates the inorganic clays from the inorganic silts. Inorganic clay values lie above the A-line, and values for inorganic silts lie below the A-line. Organic silts plot in the same region (below the A-line and with LL ranging from 30 to 50) as the inorganic silts of medium compressibility. Organic clays plot in the same region as inorganic silts of high compressibility (below the A-line and LL greater than 50). The information provided in the plasticity chart is of great value and is the basis for the classification of fine-grained soils in the Unified Soil

Classification System. Note that a line called the *U*-line lies above the *A*-line. The *U*-line is approximately the upper limit of the relationship of the plasticity index to the liquid limit for any currently known soil. The equation for the *U*-line can be given as $PI = 0.91(LL - 8)$ [7].

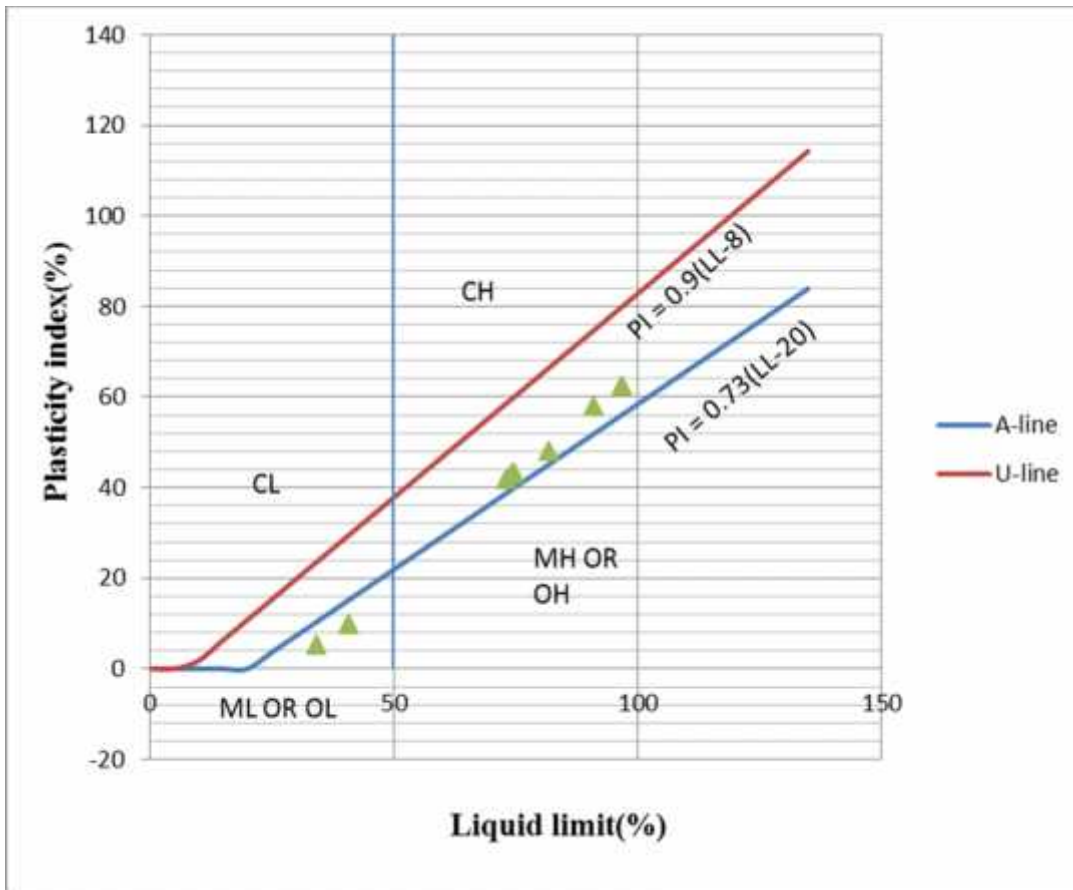


Figure 5.1 Plasticity chart

Table 5.2 Classification of soil based on USCS

Serial No.	Test Pits	Depth (m)	Percent amount of particle size				LL	PL	PI	USCS
			Gravel	sand	silt	clay				
1	TP1-1	1.5	4.30	4.85	41.6	49.25	96.63	34.08	62.55	CH
	TP1-2	3	5.43	2.78	45.46	46.33	90.84	32.95	57.89	CH
2	TP2	3	1.30	2.42	62.98	33.30	72.77	32.54	40.23	CH
3	TP3-1	0.9	15.90	17.52	56.38	10.20	39.97	30.64	9.33	ML
	TP3-2	2.3	15.89	18.65	59.30	6.16	34.11	28.62	5.49	ML
4	TP4-1	1.5	1.75	2.22	50.46	45.57	81.79	33.78	48.01	CH
	TP4-2	3	2.8	2.63	55.88	38.69	73.02	31.27	41.75	CH
5	TP5	3	2.75	6.37	51.02	39.86	74.06	30.97	43.09	CH

As could be seen plasticity chart, test pit TP-3 is in the inorganic silty soil and the remaining test pits fall in the inorganic clay range.

5.1.2.2 Activity¹

Skempton's colloidal activity is determined as the ratio of the plasticity index of the clay content to fines. He observed that, for a given soil, the plasticity index is directly proportional to the percent of clay-size fraction (i.e., percent by weight finer than 0.002 mm in size). Activity designated by “A” is defined as:

$$A_c = PI/C \dots\dots\dots(5.1)$$

Where C is the percent of clay - size fraction by weight. Activity has been used as an index property to determine the swelling potential of clays (Das, 1997). Colloidal activity values for the soils under investigation are calculated and summarized in Table 5.4.

The soil classification according to the activity number is given in Table 5.3.

Table 5.3 Degree of colloidal Activity

Activity Number, AC	Soil Type
< 0.75	Inactive
0.75 ~ 1.25	Normal
> 1.25	Active

Table 5.4 The Activity of the soil in the study area

Serial No.	Test Pits	Depth(m)	Percent amount of particle size				LL	PL	PI	A=PI/C
			Gravel	sand	silt	clay				
1	TP1-1	1.5	4.30	4.85	41.6	49.25	96.63	34.08	62.55	1.27
	TP1-2	3	5.43	2.78	45.46	46.33	90.84	32.95	57.89	1.25
2	TP2	3	1.30	2.42	62.98	33.30	72.77	32.54	40.23	1.20
3	TP3-1	0.9	15.90	17.52	56.38	10.20	39.97	30.64	9.33	0.91
	TP3-2	2.3	15.89	18.65	59.30	6.16	34.11	28.62	5.49	0.89
4	TP4-1	1.5	1.75	2.22	50.46	45.57	81.79	33.78	48.01	1.05
	TP4-2	3	2.8	2.63	55.88	38.69	73.02	31.27	41.75	1.08
5	TP5	3	2.75	6.37	51.02	39.86	74.06	30.97	43.09	1.08

From the above table , the soil in test pit one is active and the remaining falls in normal range. This implies that soil collected from test pit one has high swelling potential.

5.2. Discussions of the laboratory test results

The soil samples were collected from North Ethiopia, Woldiya town. Woldiya area is characterized by rugged topography which is the result of geological processes that had taken place in the area. This

topographic setting plays important role for the formation of different types of soils in the town. In the steeper parts of the town silty clay soil is dominant, whereas relatively flatter areas are covered by thick black clay soil. The test results reveals that the black clay soil is expansive soil. Expansive soils are problematic soils for any engineering structures, unless carefully studied.

The grain size analysis result is shown in figure 4.2. From the figure ,more than 65% of the soil particles passes Sieve no. 200 in all test pits. This means the soil in the study area is fine-grained soils(silt and clayey soils). The combined sieve and hydrometer analysis shows the graph is relatively steeper in test pit three because of the rapid settlement of the soil particles .The results obtained from the grain size analyses indicate that the dominant proportion of soil particle in the research area is silt soils in the steeper areas and clayey soils in the flatter areas of the town., which has clay content ranging from 6-50%, silt from 40-63% and sand from 2-20% .

The result of Atterberg Limits of the soil samples on air dried sample is shown in Table 4.4. From these tests the soil under investigation is inorganic. The plasticity chart shows us, the clay soils have very high plasticity and the silty soil have low plasticity. The soil in the research area has liquid limit ranges from 34-97% plastic limit rages from 28-35% and plastic index from 5-63%.

Free swell test results are summarized in Table 4.5. From the test result one can see that the free swell of the soil under investigation ranges from 39-130%. Those soils having a free swell less than 50% are considered as low in degree of expansion. While, soils having free swell values between 50 and 100% shows medium degree of expansion and above 100% it shows high degree of expansion. From the Table, the soils in the study area shows non-expansive, medium expansive and high expansion potential. Therefore, there is expansive soil in the study area specially in the southern part of the town. The town is expanding in the south-west direction, which is covered by thick black clay soils.

Specific gravity values are lower for coarse-grain soils and higher for fine-grained soils. The test results concise with the literature review values. For fine grained soils the specific gravity values range from 2.65-3.0 and may be higher for lateritic soils since they contains iron oxides. The specific gravity of soils in the study area ranges from 2.65-3.0.

Disturbed and undisturbed samples were collected during the driest season. The soil was very stiff, when try to do unconfined compression test it shows crumple before the run of the test. Because of this reason it is impossible to do unconfined compression test in this thesis. Even though ,unconfined compression test is not done, direct shear test was done to show the shear strength of soil in the town. The cohesion and

internal friction of the representative samples of test pit TP-4 and TP-5 are (64.83kPa,17.85⁰) and (91.56kPa,19.65⁰) respectively.

The compression and recompression index of the soils is calculated from the straight portions of the loading and unloading e-logp curve (Figure 4.5), the typical loading unloading curve as shown in Fig 4.6. This calculation shows that the compression index, C_c, ranges from 0.28-0.40, recompression index, C_r, from 0.05-0.07., and the pre-consolidation pressure ranges from 340-370. The coefficient of consolidation, C_v, which was calculated from curve of compression dial reading versus square root of time for each incremental loading is calculated and summarize in Table 4.6. The values for the two test pits are 0.0385 and 0.0312cm²/sec for test pit four and five respectively.

Fig 5.1 shows plasticity chart of the study area according to Unified Soil Classification System. This chart shows that the soil under investigation lies below the A-line in the region of inorganic silt with low plasticity. This chart also shows that samples located above A-line, which is inorganic clay with high plasticity.

Classifications of soils in the study area based on AASHTO Classification system is shown in Table 5.1. According to AASHTO system of soil classification, it can be observed that soil in the study area is classified in group A-7-5 and A-4. This groups of soils are clayey and silty soils respectively.

5.3. Comparison of test results with previously done researches

The soil in Woldiya must be compared with silt and clay soils. Soils with low plasticity inorganic silt and soils with high plasticity inorganic clay will be compared with the study area soils. From Table 5.1, the soil in the study area has similar properties with black expansive soils of Gonder and silt soils of Adam town. Comparisons of the study area soil with other parts of the country is given in the following Table.

Table 5.6 Comparison of Test Results in different parts of the country

Previous Research (Adiszemen,2005)	Previous Research (Dagnachew,2011)	Current Research
Black expansive soil	Silty&silty sand	Silty&black Clay soils
Gonder	Adama	Woldiya
.....
41.6-82.25	5.4-40.5	6-50
0.76-1.47		0.89-1.27
68.89-110.2	29-73	34-97
45.85-78.66	5.0-34.0	5-63
.....	18-50	39-130
	2.4-2.7	2.65-3.0
	SM,ML,MH	ML,CH

6. Conclusions and Recommendations

6.1. Conclusion

1. The test results showed that the soils in the study area are either clay soils or silty. The South- West part of Woldiya town is covered by thick black clay soil which is expansive.
2. Grain size analysis tests revealed that from 1.5m and 3m depths, the soil found in Woldiya town is mostly clay and silt soil according to USCS and AASHTO classifications. Percentage of clay content ranges from 6-50%, silt from 40-63% and sand from 2-20% .
3. The specific gravity of the soil under investigation is higher for the clay soils and decreases for the silt soils. The specific gravity of the study area ranges from 2.65-3.0.
4. The free swell values in the study area ranges from 39-130%. This shows the soil in the study area is partially non expansive and partially expansive. Specially the soil in the south-west of the town is covered by thick black clay soil which is expansive as test results revealed.
5. From consistency limit test results the liquid limit of the area ranges from 34-97% plastic limit ranges from 28-35% and plastic index from 5-63%.
6. The Activity also showed that, the soil under investigation has activity number of greater than 1.25 and less than 1.25 and analogously the free swell tests gives free swell of greater than 100% and less than 100%. Therefore, Woldiya soil is partly active and inactive as compared to the swelling characteristic of other fine grained soil.
7. As determined from the one-dimensional consolidation test conducted on undisturbed soil samples, compression index, C_c , ranges from 0.28-0.40, recompression index, C_r , from 0.05-0.07, and the pre-consolidation pressure ranges from 340-370.
8. From shear strength test the cohesion and internal friction of the representative samples of test pit four and five are (64.83kPa, 17.85°) and (91.56kPa, 19.65°) respectively.

6.2. Recommendation.

1. The dynamic properties of the soil in Woldiya is not studied.. Therefore, it needs investigation in the future.
2. Part of Woldiya is covered by thick black clay soil which is expansive. The area covered by this thesis is not sufficient; hence detailed investigation should be done.

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Appendix – A (Bore hole-log)

Test pit: 01 Location: Aruond Gonder Ber Coordinate, x: 563174.684 y: 1307601.0		Sampled Date: 24/4/2005	
As observed from the surface: stiff black clay soil			
Depth(m)	Bore hole log	Visual description	Types of test done
-0.3		Fill	
-1.6	+--+--+--+ +--+--+--+ +--+--+--+ +--+--+--+ +--+--+--+ +--+--+--+ +--+--+--+ +--+--+--+ +--+--+--+	Black clay soil	Grain size analysis, Atterberg limit Specific gravity and Free swell tests
-3.0	+--+--+--+ +--+--+--+ +--+--+--+ +--+--+--+ +--+--+--+ +--+--+--+ +--+--+--+ +--+--+--+ +--+--+--+	Brown clay soil	Grain size analysis, Atterberg limit Specific gravity and Free swell tests Consolidation and shear strength

Test pit: 04 Sampled Date: 24/4/2005
 Location: Inside hospital
 Coordinate, x: 565152.4662
 y: 1308017.2266
 As observed from the surface: Covered with grass which is stiff black clay

Depth(m)	Bore hole log	Visual description	Types of test done
-0.2	+-+-+-	Fill	
-1.6	+-+-+-	Black clay soil	Grain size analysis, Atterberg limit Specific gravity and Free swell tests
	+-+-+-		
-3.0	+-+-+-	Brown clay soil	Grain size analysis, Atterberg limit Specific gravity and Free swell tests Consolidation and shear strength
	+-+-+-		
	+-+-+-		
	+-+-+-		
	+-+-+-		
	+-+-+-		
	+-+-+-		
	+-+-+-		
	+-+-+-		
	+-+-+-		

APPENDIX – B INDEX PROPERTIES TEST RESULTS

A- Grain size analysis

Test pit 1

This test pit is taken in the west of the town which is known as Gonder Ber. This is the road to Gonder. Around this area buildings are being constructed up to G⁺⁴. In this test pit the soil is stiff black up to 1.6 m, but the texture changes to brown beyond this depth. Sample was taken at 1.5 m and 3 m depth. The sieve and hydrometer analysis for this test pit is as follows:

At 1.5 m depth

Sieve Analysis (weight of dry soil= 185.2g out of total weight of sample = 2000g)

Sieve No.	Sieve opening (mm)	Weight of sieve	Weight of sieve+ soil retained	Weight of soil retained(gm)	Percentage retained	Cumulative percentage retained	Percentage finer
4	4.75	569.1	586.6	17.5	0.88	0.88	99.13
10	2	400.8	469.3	68.5	3.43	4.30	95.70
20	0.6	326.7	387	60.3	3.02	7.32	92.69
40	0.425	292.4	304.9	12.5	0.63	7.94	92.06
60	0.3	305.3	313.3	8	0.40	8.34	91.66
140	0.15	271	282.8	11.8	0.59	8.93	91.07
200	0.075	266.6	271	4.4	0.22	9.15	90.85
pan		424	2240.5	1816.5	90.83	99.98	

hydrometer analysis

Elapse Time (min)	Te. c ⁰	Actual hydro. Reading	Composite correction	corrected hydrometric reading	Effective depth in (cm)	Coeff.K from Table	Diameter (mm)	Percentage Finer	combined Analysis
0.1	19	1.033	0.0029	1.030	8.4	0.01305	0.120	92.05	83.58
0.25	19	1.032	0.0029	1.029	8.6	0.01305	0.077	88.99	80.81
0.5	19	1.032	0.0029	1.029	8.6	0.01305	0.054	88.99	80.81

1	19	1.031	0.0029	1.028	8.9	0.01305	0.039	85.94	78.03
2	19	1.03	0.0029	1.027	9.2	0.01305	0.028	82.88	75.25
4	19	1.03	0.0029	1.027	9.2	0.01305	0.020	82.88	75.25
8	19	1.029	0.0029	1.026	9.4	0.01305	0.014	79.82	72.48
15	19	1.027	0.0029	1.024	10	0.01305	0.011	73.70	66.92
30	19	1.026	0.0029	1.023	10.2	0.0130w5	0.008	70.64	64.15
60	19	1.024	0.0029	1.021	10.7	0.01305	0.006	64.53	58.59
120	19	1.023	0.0029	1.020	11	0.01305	0.004	61.47	55.81
240	19	1.021	0.0029	1.018	11.5	0.01305	0.003	55.35	50.26
480	19	1.021	0.0029	1.018	11.5	0.01305	0.00201994	55.35	49.25
960	19	1.021	0.0029	1.018	11.5	0.01305	0.00142831	55.35	47.24
1440	19	1.021	0.0029	1.018	11.5	0.01305	0.00116621	55.35	45.23

At 3.0 m depth

Sieve Analysis (Weight of dry soil= 165.6g out of total weight of sample = 2000g)

sieve NO.	sieve opening (mm)	Weight of sieve	weight of sieve+ soil retained	weight of soil retained(gm)	Percentage retained	Cumulative percentage retained	Percentage finer
	9.5	462.1	471.5	9.4	0.47	0.47	99.53
4	4.75	569.1	603	33.9	1.695	2.17	97.835
10	2	400.8	466.1	65.3	3.265	5.43	94.57
20	0.6	326.7	359.9	33.2	1.66	7.09	92.91
40	0.425	292.4	298.1	5.7	0.285	7.38	92.625
60	0.3	305.3	309.8	4.5	0.225	7.60	92.4
140	0.15	271	279.4	8.4	0.42	8.02	91.98
200	0.075	266.6	270.3	3.7	0.185	8.21	91.795
pan		424	2259.3	1835.3	91.765	99.97	

hydrometer analysis

Elapse time (min)	Te.(c ⁰)	Actual hydrometric reading	Composit e correction	corrected hydrometric reading	Effectiv e depth in (cm)	Coeff.K from Table	Diamete r(mm)	Percenta ge Finer	combin ed Analysis
0.1	19	1.033	0.0029	1.030	8.4	0.013382	0.123	94.40	86.63
0.25	19	1.032	0.0029	1.029	8.6	0.013382	0.078	91.27	83.75
0.5	19	1.032	0.0029	1.029	8.6	0.013382	0.055	91.27	83.75
1	19	1.031	0.0029	1.028	8.9	0.013382	0.040	88.13	80.87
2	19	1.031	0.0029	1.028	8.9	0.013382	0.028	88.13	80.87
4	19	1.03	0.0029	1.027	9.2	0.013382	0.020	85.00	78.00
8	19	1.028	0.0029	1.025	9.7	0.013382	0.015	78.72	72.24
15	19	1.027	0.0029	1.024	10	0.013382	0.011	75.59	69.36
30	19	1.025	0.0029	1.022	10.5	0.013382	0.008	69.31	63.61
60	19	1.023	0.0029	1.020	11	0.013382	0.006	63.04	57.85
120	19	1.022	0.0029	1.019	11.3	0.013382	0.004	59.90	54.97
240	19	1.02	0.0029	1.017	11.8	0.013382	0.003	53.63	49.22
480	19	1.019	0.0029	1.016	12.1	0.013382	0.002	50.50	46.34
960	19	1.018	0.0029	1.015	12.3	0.013382	0.002	47.36	43.46
1440	19	1.017	0.0029	1.014	12.6	0.013382	0.001	44.22	40.58

Test pit 2

This test pit was taken in the North West of the town which is known as Mugad. Around this area buildings are going to be constructed. The site was excavated for construction. In this test pit the texture of the soil is brown and have the same texture up to 3.0 m depth. Sample was taken at 3 m depth. The sieve and hydrometer analysis for this test pit is as follows:

Sieve Analysis (Weight of dry soil = 75.09g out of total weight of sample = 2000g)

sieve No.	Sieve opening (mm)	Weight of sieve	Weight of sieve+ soil retained	Weight of soil	Percentage retained	Cumulative percentage retained	Percentage finer
	9.5	462	462	0	0.00	0.00	100.00
4	4.75	568.9	576.9	8	0.40	0.40	99.60
10	2	400.6	418.7	18.1	0.90	1.31	98.70
20	0.6	326.5	351.7	25.2	1.26	2.57	97.44
40	0.425	292.2	297.1	4.9	0.25	2.81	97.19
60	0.3	305.1	309.1	4	0.20	3.01	96.99
140	0.15	270.9	278.6	7.7	0.39	3.40	96.61
200	0.075	273.4	279.8	6.4	0.32	3.72	96.29
pan		423.8	2348.71	1924.91	96.25	99.96	

hydrometer analysis

Elapsed Time (min)	Tem p. c ⁰	Actual Hydro. Reading	Composite correction	corrected hydrometric reading	Effective depth in cm	Coeff.K from Table	Diamete r(mm)	Percent age finer	Combi ned
0.1	19	1.032	0.0029	1.029	8.6	0.013268	0.123	90.71	87.31
0.25	19	1.032	0.0029	1.029	8.6	0.013268	0.078	90.71	87.31
0.5	19	1.031	0.0029	1.028	8.9	0.013268	0.056	87.60	84.31
1	19	1.031	0.0029	1.028	8.9	0.013268	0.040	87.60	84.31
2	19	1.03	0.0029	1.027	9.2	0.013268	0.028	84.48	81.31
4	19	1.027	0.0029	1.024	10	0.013268	0.021	75.13	72.31
8	19	1.026	0.0029	1.023	10.2	0.013268	0.015	72.01	69.31
15	19	1.024	0.0029	1.021	10.7	0.013268	0.011	65.78	63.31

30	19	1.021	0.0029	1.018	11.5	0.013268	0.008	56.42	54.31
60	19	1.02	0.0029	1.017	11.8	0.013268	0.006	53.31	51.30
120	19	1.017	0.0029	1.014	12.6	0.013268	0.004	43.95	42.30
240	19	1.015	0.0029	1.012	13.1	0.013268	0.003	37.72	36.30
480	19	1.014	0.0029	1.011	13.4	0.013268	0.002	34.60	33.30
960	19	1.013	0.0029	1.010	13.7	0.013268	0.002	31.48	30.30
1260	19	1.012	0.0029	1.009	13.9	0.013268	0.001	28.37	27.30
1440	19	1.011	0.0029	1.008	14.2	0.013268	0.001	25.25	24.30

Test pit 3

This test pit was taken in center of the town which is known as Adago. Banks and other public facilities found around this area. Buildings are being constructed like other parts of the town. In this test pit the soil is silty black up to 1.0 m, but the texture changes to brown silty clay beyond this depth. Sample was taken at 1.0 m and 3 m depth in which the soil texture changes. The sieve and hydrometer analysis for this test pit is as follows:

At 1.0 m depth

Sieve Analysis (Weight of dry soil = 675.7g out of total weight of sample = 2000g)

sieve No.	sieve opening (mm)	weight of sieve	weight of sieve+ soil retained	weight of soil retained(gm)	percentage retained	cumulative percentage retained	percentage finer
	12.5	448.8	471.5	22.7	1.14	1.14	98.87
	9.5	456.1	486.8	30.7	1.54	2.67	97.33
4	4.75	430.6	518.4	87.8	4.39	7.06	92.94
10	2	400.8	577.5	176.7	8.84	15.90	84.11
20	0.6	324.3	500.6	176.3	8.82	24.71	75.29
40	0.425	292.2	329.2	37	1.85	26.56	73.44
60	0.3	302.4	333.5	31.1	1.56	28.12	71.89
140	0.15	277.7	331.5	53.8	2.69	30.81	69.20
200	0.075	273.6	325.8	52.2	2.61	33.42	66.59
pan		256.1	263.5	1333.7	66.69	100.10	

hydrometer analysis

Elapsed time (min)	Tem. c ⁰	Actual hydro. reading	Composite correction	corrected hydrometric reading	Effective depth in cm	Coeff.K from table	Diame ter (mm)	Percent age finer	combine d
0.1	19	1.031	0.0029	1.028	8.9	0.01305	0.123	84.30	56.22

0.25	19	1.03	0.0029	1.027	9.2	0.01305	0.079	81.30	54.21
0.5	19	1.027	0.0029	1.024	10	0.01305	0.058	72.30	48.21
1	19	1.026	0.0029	1.023	10.2	0.01305	0.042	69.30	46.21
2	19	1.023	0.0029	1.020	11	0.01305	0.031	60.30	40.21
4	19	1.02	0.0029	1.017	11.8	0.01305	0.022	51.30	34.21
8	19	1.018	0.0029	1.015	12.3	0.01305	0.016	45.30	30.21
15	19	1.016	0.0029	1.013	12.9	0.01305	0.012	39.30	26.21
30	19	1.014	0.0029	1.011	13.4	0.01305	0.009	33.30	22.21
60	19	1.012	0.0029	1.009	13.9	0.01305	0.006	27.30	18.21
120	19	1.011	0.0029	1.008	14.2	0.01305	0.004	24.30	16.20
240	19	1.009	0.0029	1.006	14.7	0.01305	0.003	18.30	12.20
480	19	1.008	0.0029	1.005	15	0.01305	0.002	15.30	10.20
960	19	1.008	0.0029	1.005	15	0.01305	0.002	15.30	10.20
1440	19	1.008	0.0029	1.005	15	0.01305	0.001	15.30	10.20

At 3.0 m depth

Sieve Analysis (Weight of dry soil = 692.7g out of total weight of sample = 2000g)

Sieve No.	Sieve opening (mm)	Weight of sieve	Weight of sieve+ soil retained	Weight of soil retained(gm)	Percentage retained	Cumulative percentage retained	Percentage finer
	12.5	468.1	491.5	23.4	1.17	1.17	98.83
	9.5	462.2	485.6	23.4	1.17	2.34	97.66
4	4.75	430.6	539.3	108.7	5.44	7.78	92.23
10	2	400.8	563	162.2	8.11	15.89	84.12
20	0.6	324.4	502.6	178.2	8.91	24.80	75.21
40	0.425	305.5	348.4	42.9	2.15	26.94	73.06
60	0.3	302.5	341.5	39	1.95	28.89	71.11
140	0.15	277.8	344.8	67	3.35	32.24	67.76
200	0.075	272.3	318.2	45.9	2.30	34.54	65.47
pan		423.9	426.1	1309.5	65.48	100.01	

hydrometer analysis

Elapsed Time (min)	Temp. c ⁰	Actual Hydro. Reading	Composite correction	corrected hydrometric reading	Effective depth in (cm)	Coeff.K from table	Diameter (mm)	Percentage Finer	combined analysis
0.1	19	1.031	0.0029	1.028	8.9	0.01305	0.123	85.32	55.86
0.25	19	1.03	0.0029	1.027	9.2	0.01305	0.079	82.28	53.87
0.5	19	1.028	0.0029	1.025	9.7	0.01305	0.057	76.21	49.90
1	19	1.026	0.0029	1.023	10.2	0.01305	0.042	70.14	45.92
2	19	1.024	0.0029	1.021	10.7	0.01305	0.030	64.07	41.95
4	19	1.02	0.0029	1.017	11.8	0.01305	0.022	51.92	33.99
8	19	1.017	0.0029	1.014	12.6	0.01305	0.016	42.81	28.03
15	19	1.014	0.0029	1.011	13.4	0.01305	0.012	33.70	22.07
30	19	1.012	0.0029	1.009	13.9	0.01305	0.009	27.63	18.09
60	19	1.01	0.0029	1.007	14.4	0.01305	0.006	21.56	14.11
120	19	1.008	0.0029	1.005	15	0.01305	0.005	15.48	10.14
240	19	1.007	0.0029	1.004	15.2	0.01305	0.003	12.45	8.15
480	19	1.006	0.0029	1.003	15.5	0.01305	0.002	9.41	6.16
960	19	1.0055	0.0029	1.003	15.5	0.01305	0.002	7.89	5.17
1440	19	1.005	0.0029	1.002	15.8	0.01305	0.001	6.38	4.17

Test pit 4

This test pit was taken inside the Woldiya referral hospital. This area is relatively flat, above the hospital there is the horizontal ridge extending in the northern part of the town. Liched soil from this ridges transported and partly deposited to the hospital area. In this test pit the soil is stiff black up to 1.5 m, but the texture changes to brown soil beyond this depth. Sample was taken at 1.5 m and 3 m depth in which the soil texture changes. The sieve and hydrometer analysis for this test pit is as follows:

At 1.5 m depth

Sieve Analysis (Weight of dry soil = 79.3g out of total weight of sample = 2000g)

Sieve No.	Sieve Opening (mm)	weight of sieve	Weight Of sieve+ soil retained	weight of soil retained(gm)	percentage retained	cumulative percentage retained	percentage finer
	9.5	462	466.4	4.4	0.22	0.22	99.78
4	4.75	568.9	580.1	11.2	0.56	0.78	99.22
10	2	400.6	419.9	19.3	0.96	1.75	98.26
20	0.6	326.5	349.1	22.6	1.13	2.88	97.13
40	0.425	292.2	298	5.8	0.29	3.17	96.84
60	0.3	305.1	309.7	4.6	0.23	3.40	96.61
140	0.15	270.9	278.5	7.6	0.38	3.78	96.23
200	0.075	273.4	277.1	3.7	0.19	3.96	96.04
pan		423.8	424	1920.9	96.05	100.01	

hydrometer analysis

Elapsed time (min)	Temp. c ⁰	Actual hydro. reading	Composite correction	corrected hydrometric reading	Effective depth in (cm)	Coeff.K from table	Diameter (mm)	Percentage finer	Combined analysis
0.1	19	1.033	0.0029	1.030	8.4	0.01342	0.123	94.60	90.86
0.25	19	1.032	0.0029	1.029	8.6	0.01342	0.079	91.46	87.84
0.5	19	1.031	0.0029	1.028	8.9	0.01342	0.057	88.31	84.82
1	19	1.031	0.0029	1.028	8.9	0.01342	0.040	88.31	84.82
2	19	1.03	0.0029	1.027	9.2	0.01342	0.029	85.17	81.80
4	19	1.028	0.0029	1.025	9.7	0.01342	0.021	78.89	75.77
8	19	1.027	0.0029	1.024	10	0.01342	0.015	75.74	72.75
15	19	1.026	0.0029	1.023	10.2	0.01342	0.011	72.60	69.73
30	19	1.024	0.0029	1.021	10.7	0.01342	0.008	66.31	63.69
60	19	1.022	0.0029	1.019	11.3	0.01342	0.006	60.03	57.65
120	19	1.02	0.0029	1.017	11.8	0.01342	0.004	53.74	51.62
240	19	1.019	0.0029	1.016	12.1	0.01342	0.003	50.60	48.60
480	19	1.018	0.0029	1.015	12.3	0.01342	0.002	47.46	45.58
960	19	1.0175	0.0029	1.015	12.3	0.01342	0.002	45.89	44.07

1440	19	1.017	0.0029	1.014	12.6	0.01342	0.001	44.31	42.56
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At 3.0 m depth

Sieve Analysis (Weight of dry soil = 111.6g out of total weight of sample = 2000g)

sieve No.	sieve opening (mm)	weight of sieve	weight of sieve+ soil retained	weight of soil retained(gm)	percentage retained	cumulative percentage retained	percentage finer
	9.5	462.1	474.7	12.6	0.63	0.63	99.37
4	4.75	569.1	580.3	11.2	0.56	1.19	98.81
10	2	400.8	432.9	32.1	1.61	2.79	97.21
20	0.6	326.7	352.4	25.7	1.29	4.08	95.92
40	0.425	292.4	298	5.6	0.28	4.36	95.64
60	0.3	305.3	309.9	4.6	0.23	4.59	95.41
140	0.15	271	282.7	11.7	0.58	5.17	94.83
200	0.075	266.6	271.6	5	0.25	5.42	94.58
pan		424	2315.5	1891.5	94.58	100.00	

hydrometer analysis

Elapsed time (min)	Temp. c ⁰	Actual hydro. reading	Composite correction	corrected hydrometric reading	Effective depth in (cm)	Coeff.K from table	Diameter (mm)	Percentage finer	Combined analysis
0.1	19	1.032	0.0029	1.029	8.6	0.013306	0.123	90.90	85.97
0.25	19	1.032	0.0029	1.029	8.6	0.013306	0.078	90.90	85.97
0.5	19	1.031	0.0029	1.028	8.9	0.013306	0.056	87.77	83.01
1	19	1.03	0.0029	1.027	9.2	0.013306	0.040	84.65	80.06
2	19	1.029	0.0029	1.026	9.4	0.013306	0.029	81.53	77.10
4	19	1.027	0.0029	1.024	10	0.013306	0.021	75.28	71.19
8	19	1.025	0.0029	1.022	10.5	0.013306	0.015	69.03	65.29
15	19	1.024	0.0029	1.021	10.7	0.013306	0.011	65.91	62.33
30	19	1.022	0.0029	1.019	11.3	0.013306	0.008	59.66	56.42
60	19	1.02	0.0029	1.017	11.8	0.013306	0.006	53.41	50.52
120	19	1.018	0.0029	1.015	12.3	0.013306	0.004	47.17	44.61
240	19	1.017	0.0029	1.014	12.6	0.013306	0.003	44.04	41.65
480	19	1.016	0.0029	1.013	12.9	0.013306	0.002	40.92	38.70

960	19	1.015	0.0029	1.012	13.1	0.013306	0.002	37.80	35.75
1440	19	1.014	0.0029	1.011	13.4	0.013306	0.001	34.67	32.79

Test pit 5

This test pit was taken in the southern part of the town. This area is flat, the town is expanding in this direction. Transported soil from different part of the town deposited to this fatter area. The area is covered with thick black clay soil. Expansive type of soil is dominant in the expanding area of the town. In this test pit the soil is stiff black up to 3.0 m, Sample was taken at 3 m depth, Since the texture of the pit remain the same. The sieve and hydrometer analysis for this test pit is as follows:

Sieve Analysis (Weight of dry soil = 186.8g out of total weight of sample = 2000g)

sieve No.	sieve opening (mm)	weight of sieve	weight of sieve+ soil retained	weight of soil retained(gm)	percentage retained	cumulative percentage retained	percentage finer
	9.5	462.1	470.8	8.7	0.43	0.43	99.57
4	4.75	569.1	586.7	17.6	0.88	1.32	98.69
10	2	400.8	429.5	28.7	1.44	2.75	97.25
20	0.6	326.7	363.6	36.9	1.85	4.60	95.41
40	0.425	292.4	309	16.6	0.83	5.43	94.58
60	0.3	305.3	323.4	18.1	0.90	6.33	93.67
140	0.15	271	310.1	39.1	1.96	8.29	91.72
200	0.075	266.6	283.2	16.6	0.83	9.12	90.89
pan		424	2241.4	1817.4	90.87	99.99	

hydrometer analysis

Elapsed Time (min)	Temp. c ⁰	Actual Hydro. Reading	Compo site correcti on	corrected hydrometr ic reading	Effectiv e depth in cm	Coeff.K from Table	D mm	% retained	combined
0.1	19	1.033	0.0029	1.030	8.4	0.01323	0.121	93.64	85.09
0.25	19	1.032	0.0029	1.029	8.6	0.01323	0.078	90.53	82.27
0.5	19	1.032	0.0029	1.029	8.6	0.01323	0.055	90.53	82.27
1	19	1.031	0.0029	1.028	8.9	0.01323	0.039	87.42	79.44
2	19	1.03	0.0029	1.027	9.2	0.01323	0.028	84.31	76.61
4	19	1.028	0.0029	1.025	9.7	0.01323	0.021	78.09	70.96

8	19	1.026	0.0029	1.023	10.2	0.01323	0.015	71.87	65.31
15	19	1.024	0.0029	1.021	10.7	0.01323	0.011	65.64	59.65
30	19	1.023	0.0029	1.020	11	0.01323	0.008	62.53	56.82
60	19	1.021	0.0029	1.018	11.5	0.01323	0.006	56.31	51.17
120	19	1.019	0.0029	1.016	12.1	0.01323	0.004	50.09	45.52
240	19	1.018	0.0029	1.015	12.3	0.01323	0.003	46.98	42.69
480	19	1.017	0.0029	1.014	12.6	0.01323	0.002	43.87	39.86
960	19	1.016	0.0029	1.013	12.9	0.01323	0.002	40.76	37.03
1440	19	1.015	0.0029	1.012	13.1	0.01323	0.001	37.64	34.21

B-Atterberg Limits

Test pit 1(TP1)

Liquid Limit(1.5m)					Plastic Limit(1.5m)	
Soil - Sample no.	1	2	3	4	1	2
Moisture can number - Lid number	A100	70	MA7	E3	A22	G3
MC = Mass of empty, clean can (grams)	15.5	15.5	15.7	15.3	22	22
MCMS = Mass of can + moist soil (grams)	32.7	40.2	38.7	33.8	24.6	24.4
MCDS = Mass of can + dry soil (grams)	24.3	27.9	27.6	25.3	23.9	23.8
MS = Mass of soil solids (grams)	8.8	12.4	11.9	10	1.9	1.8
MW = Mass of pore water (grams)	8.4	12.3	11.1	8.5	0.7	0.6
W = Water content, w%	95.46	99.19	93.28	85.000	36.84	33.33
No. of drops (N)	23	18	30	37		
Average(%)	96.63				34.08	

Liquid Limit(3.0 m)					Plastic Limit(3.0 m)		
Soil - Sample no.	1	2	3	4	1	2	3
Moisture can number - Lid number	IT5	c3	G3-1	A1	H2	A9	30
MC = Mass of empty, clean can (grams)	15.8	15.7	15.4	15.5	15.7	15.4	15.7
MCMS = Mass of can + moist soil (grams)	37.1	38	40.1	43.1	21.3	20.8	21.9
MCDS = Mass of can + dry soil (grams)	27.4	27.2	28.8	30	19.9	19.5	20.4
MS = Mass of soil solids (grams)	11.6	11.5	13.4	14.5	4.2	4.1	4.7
MW = Mass of pore water (grams)	9.7	10.8	11.3	13.1	1.4	1.3	1.5
W = Water content, w%	83.621	93.913	84.33	90.35	33.33	31.71	31.91
No. of drops (N)	36	20	28	24			
Average(%)	90.84				32.95		

#Test Pit 2(TP2)

Liquid Limit(3.0m)				Plastic Limit(3.0 m)		
Soil - Sample no.	1	2	3	1	2	3
Moisture can number - Lid number	A1	30	70	Fa	c3	Gp
MC = Mass of empty, clean can (grams)	15.5	15.8	15.5	11	15.6	15.6
MCMS = Mass of can + moist soil (grams)	34.4	35.3	32.9	17.2	19.7	19.9
MCDS = Mass of can + dry soil (grams)	26.5	26.9	25.6	15.6	18.7	18.9
MS = Mass of soil solids (grams)	11	11.1	10.1	4.6	3.1	3.3
MW = Mass of pore water (grams)	7.9	8.4	7.3	1.6	1	1
W = Water content, w%	71.818	75.676	72.277	34.78261	32.25806	30.30303
No. of drops (N)	38	16	24			
Average (%)	72.77			32.54		

#Test Pit 3(TP3)

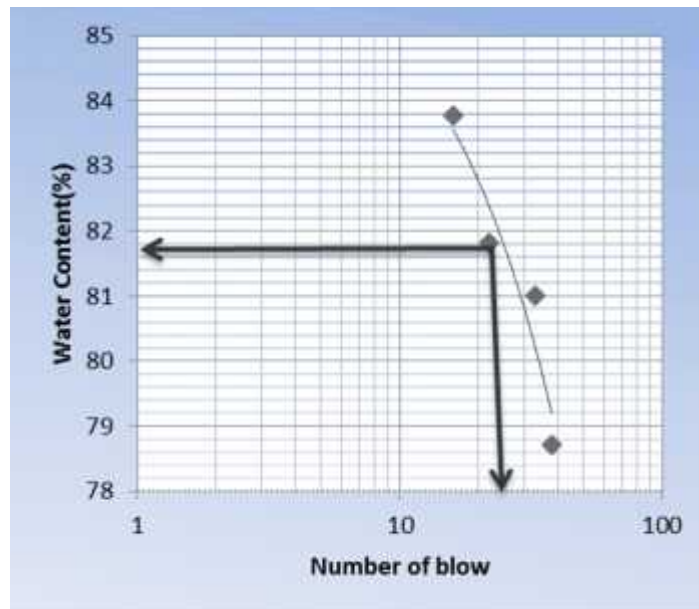
Liquid Limit(0.9m)				Plastic Limit(0.9 m)		
Soil - Sample no.	1	2	3	1	2	3
Moisture can number - Lid number	F20	MA3	30	IT5	MA22	c3
MC = Mass of empty, clean can (grams)	15.7	15.3	15.7	15.8	15.6	15.7
MCMS = Mass of can + moist soil (grams)	34.2	33.1	29.1	18	17.7	17.8
MCDS = Mass of can + dry soil (grams)	28.6	28.1	25.8	17.5	17.2	17.3
MS = Mass of soil solids (grams)	12.9	12.8	10.1	1.7	1.6	1.6
MW = Mass of pore water (grams)	5.6	5	3.3	0.5	0.5	0.5
W = Water content, w%	43.41	39.06	32.67	29.41	31.25	31.25
No. of drops (N)	14	24	32			
Average(%)	39.97			30.64		

Liquid Limit(3.0m)				Plastic Limit(3.0 m)		
Soil - Sample no.	1	2	3	1	2	3
Moisture can number - Lid number	G3	L2	A9	MA6	H2	A22
MC = Mass of empty, clean can (grams)	15.5	15.8	15.4	15.5	15.7	15.6
MCMS = Mass of can + moist soil (grams)	34.4	35.8	33.5	19.3	19.1	19.3
MCDS = Mass of can + dry soil (grams)	29.7	30.4	29.1	18.5	18.3	18.5

MS = Mass of soil solids (grams)	14.2	14.6	13.7	3	2.6	2.9
MW = Mass of pore water (grams)	4.7	5.4	4.4	0.8	0.8	0.8
W = Water content, w%	33.09859155	36.9863	32.11679	26.67	30.77	27.59
No. of drops (N)	26	18	34			
Average(%)	34.11			28.62		

#Test Pit 4(TP4)

	Liquid Limit(1.5m)				Plastic Limit(1.5m)		
Soil - Sample no.	1	2	3	4	1	2	3
Moisture can number - Lid number	D4	A1	G-3-1	H2	N	E3	G6
MC = Mass of empty, clean can (grams)	15.6	15.5	15.5	15.7	22	21.6	22.1
MCMS = Mass of can + moist soil (grams)	32.4	33.6	31.5	29.3	23.7	25	24.9
MCDS = Mass of can + dry soil (grams)	25	25.5	24.3	23.1	23.3	24.1	24.2
MS = Mass of soil solids (grams)	9.4	10	8.8	7.4	1.3	2.5	2.1
MW = Mass of pore water (grams)	7.4	8.1	7.2	6.2	0.4	0.9	0.7
W = Water content, w%	78.72	81	81.82	83.78	30.77	36	33.33
No. of drops (N)	38	33	22	16			
Average(%)	81.79				33.78		



Liquid limit determination for test pit #4 at 1.5 m

Liquid Limit(3.0 m)					Plastic Limit(3.0 m)		
Soil - Sample no.	1	2	3	4	1	2	3
Moisture can number - Lid number	A22	F20	50	E4	E4	50	G-3-1
MC = Mass of empty, clean can (grams)	15.6	15.8	15.5	15.7	22.1	21.8	22
MCMS = Mass of can + moist soil (grams)	29.1	27.9	26.7	26.2	24.2	24.5	24
MCDS = Mass of can + dry soil (grams)	23.6	22.8	21.8	21.8	23.7	23.9	23.5
MS = Mass of soil solids (grams)	8	7	6.3	6.1	1.6	2.1	1.5
MW = Mass of pore water (grams)	5.5	5.1	4.9	4.4	0.5	0.6	0.5
W = Water content, w%	68.75	72.86	77.78	72.13	31.25	28.57	33.33
No. of drops (N)	33	24	16	28			
Average(%)	73.02				31.27		

#Test Pit 5(TP5)

Liquid Limit(3.0 m)					Plastic Limit(3.0 m)	
Soil - Sample no.	1	2	3	4	1	2
Moisture can number - Lid number	F20	D4	G3-4	1A	E3	A1
MC = Mass of empty, clean can (grams)	15.7	15.7	15.7	15.4	21.7	22
MCMS = Mass of can + moist soil (grams)	34.7	30.6	34.6	31.2	23.4	24.2
MCDS = Mass of can + dry soil (grams)	26.4	24.4	26.4	24.8	23	23.7
MS = Mass of soil solids (grams)	10.7	8.7	10.7	9.4	1.3	1.7
MW = Mass of pore water (grams)	8.3	6.2	8.2	6.4	0.4	0.5
W = Water content, w%	77.57	71.26	76.64	68.09	30.87	29.98
No. of drops (N)	14	33	20	40		
Average(%)	74.06				30.97	

APPENDIX – C

Direct Shear test results

Test Pit 4 @3.0m				Test Pit 4@3.0m			
Horizontal dial reading	Load dial reading			Horizontal dial reading	Load dial Reading		
	100kpa	200kpa	300kpa		100kpa	200kpa	300kpa
0	0	0	0	0	0	0	0
50	64	100	85	50	55	52	28
100	93	158	192	100	108	140	125
150	148	237	290	150	145	195	232
200	173	290	372	200	185	280	295
250	230	345	445	250	265	365	365
300	281	405	535	300	362	418	442
350	342	467	638	350	443	490	505
400	395	530	706	400	506	550	597
450	440	573	754	450	558	619	671
500	466	593	770	500	592	663	726
550	390	555	655	550	612	694	762
600	259	445	495	600	615	698	786
650	232	404	440	650	589	696	810
700	222	384	409	700	555	681	830
750	207			750	534	664	852
800	167.1	172	171.6	800	497	655	876
850				850			889
900				900			893
950				950			870
1000				1000			710
							679

Test Pit 4 @3.0m 100kpa					
horizontal dial reading	horizontal(mm) displacement	corrected area(m2)	load dial reading	load in kN	stress kpa
0	0	0.0036	0	0	0
50	0.5	0.00357	64	0.0448	12.54902
100	1	0.00354	93	0.0651	18.38983
150	1.5	0.00351	148	0.1036	29.51567
200	2	0.00348	173	0.1211	34.79885
250	2.5	0.00345	230	0.161	46.66667
300	3	0.00342	281	0.1967	57.51462

350	3.5	0.00339	342	0.2394	70.61947
400	4	0.00336	395	0.2765	82.29167
450	4.5	0.00333	440	0.308	92.49249
500	5	0.0033	466	0.3262	98.84848
550	5.5	0.00327	390	0.273	83.48624
600	6	0.00324	259	0.1813	55.95679
650	6.5	0.00321	232	0.1624	50.5919
700	7	0.00318	222	0.1554	48.86792
750	7.5	0.00315	207	0.1449	46
800	8	0.00312			
850	8.5	0.00309			
900	9	0.00306			

200kpa

horizontal dial reading	horizontal(mm) displacement	corrected area(m2)	load dial reading	load in kN	stress kpa
0	0	0.0036	0	0	0
50	0.5	0.00357	100	0.07	19.60784
100	1	0.00354	158	0.1106	31.24294
150	1.5	0.00351	237	0.1659	47.26496
200	2	0.00348	290	0.203	58.33333
250	2.5	0.00345	345	0.2415	70
300	3	0.00342	405	0.2835	82.89474
350	3.5	0.00339	467	0.3269	96.43068
400	4	0.00336	530	0.371	110.4167
450	4.5	0.00333	573	0.4011	120.4505
500	5	0.0033	593	0.4151	125.7879
550	5.5	0.00327	555	0.3885	118.8073
600	6	0.00324	445	0.3115	96.14198
650	6.5	0.00321	404	0.2828	88.09969
700	7	0.00318	384	0.2688	84.5283
750	7.5	0.00315			
800	8	0.00312			
850	8.5	0.00309			
900	9	0.00306			

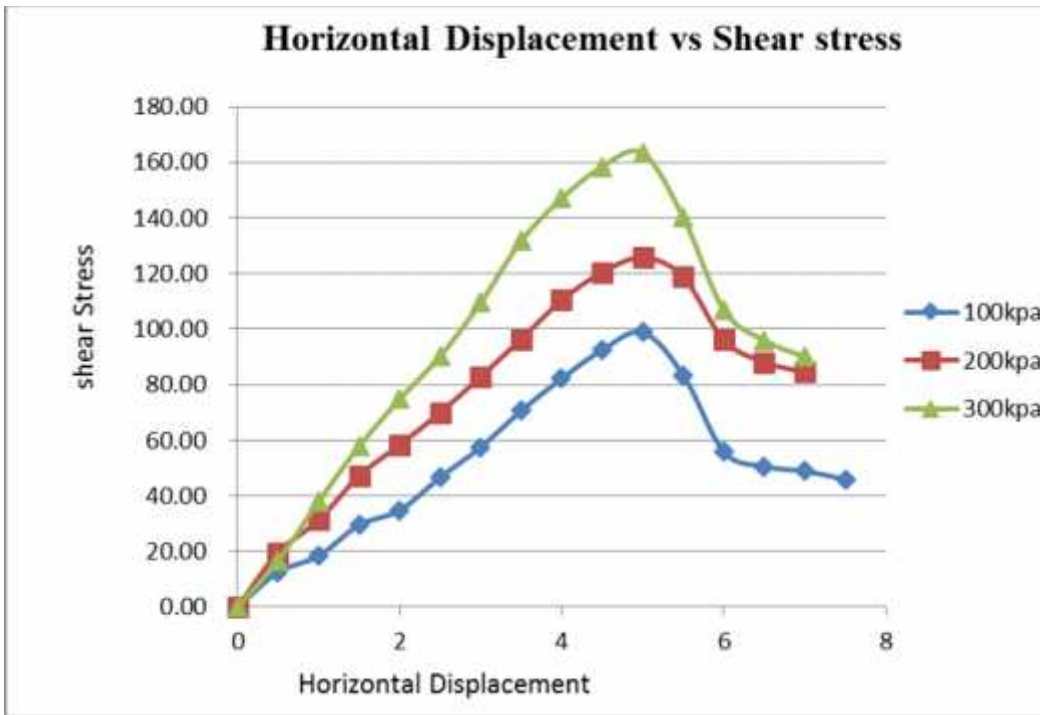
300kpa

horizontal dial reading	horizontal(mm) displacement	corrected area(m2)	load dial reading	load in kN	stress kpa
0	0	0.0036	0	0	0
50	0.5	0.00357	85	0.0595	16.66667
100	1	0.00354	192	0.1344	37.9661
150	1.5	0.00351	290	0.203	57.83476
200	2	0.00348	372	0.2604	74.82759
250	2.5	0.00345	445	0.3115	90.28986

300	3	0.00342	535	0.3745	109.5029
350	3.5	0.00339	638	0.4466	131.7404
400	4	0.00336	706	0.4942	147.0833
450	4.5	0.00333	754	0.5278	158.4985
500	5	0.0033	770	0.539	163.3333
550	5.5	0.00327	655	0.4585	140.2141
600	6	0.00324	495	0.3465	106.9444
650	6.5	0.00321	440	0.308	95.95016
700	7	0.00318	409	0.2863	90.03145
750	7.5	0.00315			
800	8	0.00312			
850	8.5	0.00309			
900	9	0.00306			

The summary of shear stress for different loading conditions.

horizontal(mm) displacement	Shear stress(kpa)		
	100kpa	200kpa	300kpa
0	0.00	0.00	0.00
0.5	12.55	19.61	16.67
1	18.39	31.24	37.97
1.5	29.52	47.26	57.83
2	34.80	58.33	74.83
2.5	46.67	70.00	90.29
3	57.51	82.89	109.50
3.5	70.62	96.43	131.74
4	82.29	110.42	147.08
4.5	92.49	120.45	158.50
5	98.85	125.79	163.33
5.5	83.49	118.81	140.21
6	55.96	96.14	106.94
6.5	50.59	88.10	95.95
7	48.87	84.53	90.03
7.5	46.00		



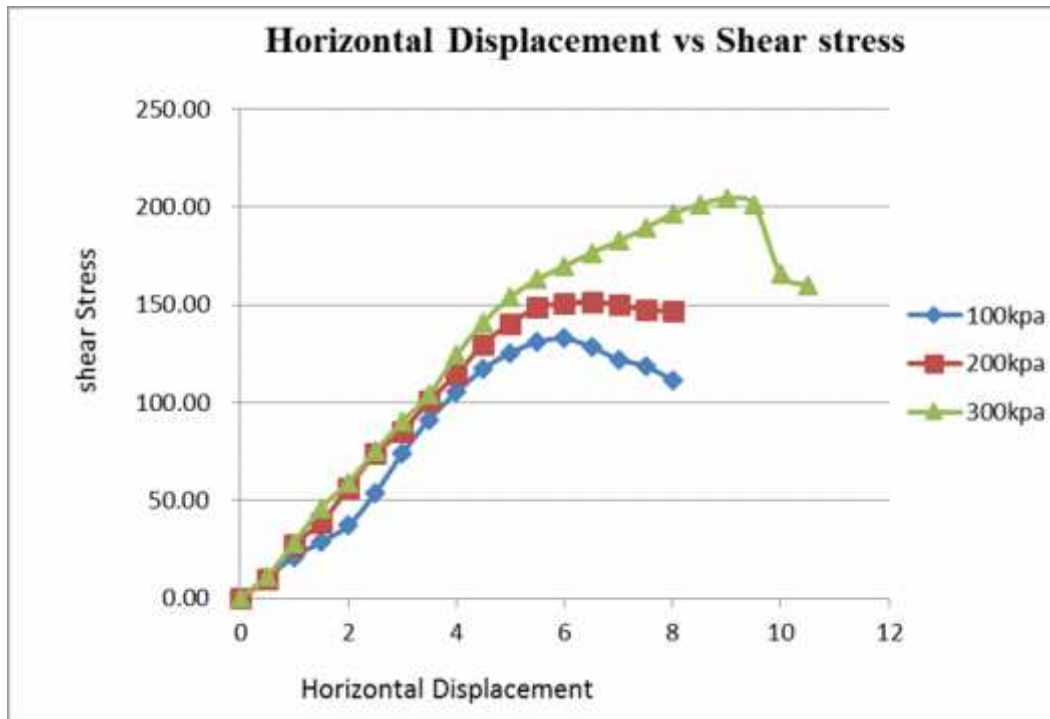
Test Pit 5 @3.0m		100kpa			
horizontal dial reading	horizontal(mm) displacement	corrected area(m2)	load dial reading	load in kN	stress kpa
0	0	0.0036	0	0	0
50	0.5	0.00357	55	0.0385	10.78431
100	1	0.00354	108	0.0756	21.35593
150	1.5	0.00351	145	0.1015	28.91738
200	2	0.00348	185	0.1295	37.21264
250	2.5	0.00345	265	0.1855	53.76812
300	3	0.00342	362	0.2534	74.09357
350	3.5	0.00339	443	0.3101	91.47493
400	4	0.00336	506	0.3542	105.4167
450	4.5	0.00333	558	0.3906	117.2973
500	5	0.0033	592	0.4144	125.5758
550	5.5	0.00327	612	0.4284	131.0092
600	6	0.00324	615	0.4305	132.8704
650	6.5	0.00321	589	0.4123	128.4424
700	7	0.00318	555	0.3885	122.1698
750	7.5	0.00315	534	0.3738	118.6667
800	8	0.00312	497	0.3479	111.5064
850	8.5	0.00309			
900	9	0.00306			
200kpa					

horizontal dial reading	horizontal(mm) displacement	corrected area(m2)	load dial reading	load in kN	stress kpa
0	0	0.0036	0	0	0
50	0.5	0.00357	52	0.0364	10.19608
100	1	0.00354	140	0.098	27.68362
150	1.5	0.00351	195	0.1365	38.88889
200	2	0.00348	280	0.196	56.32184
250	2.5	0.00345	365	0.2555	74.05797
300	3	0.00342	418	0.2926	85.55556
350	3.5	0.00339	490	0.343	101.1799
400	4	0.00336	550	0.385	114.5833
450	4.5	0.00333	619	0.4333	130.1201
500	5	0.0033	663	0.4641	140.6364
550	5.5	0.00327	694	0.4858	148.5627
600	6	0.00324	698	0.4886	150.8025
650	6.5	0.00321	696	0.4872	151.7757
700	7	0.00318	681	0.4767	149.9057
750	7.5	0.00315	664	0.4648	147.5556
800	8	0.00312	655	0.4585	146.9551
850	8.5	0.00309			
900	9	0.00306			
300kpa					
horizontal dial reading	horizontal(mm) displacement	corrected area(m2)	load dial reading	load in kN	stress kpa
0	0	0.0036	0	0	0
50	0.5	0.00357	28	0.0196	5.490196
100	1	0.00354	125	0.0875	24.71751
150	1.5	0.00351	232	0.1624	46.26781
200	2	0.00348	295	0.2065	59.33908
250	2.5	0.00345	365	0.2555	74.05797
300	3	0.00342	442	0.3094	90.46784
350	3.5	0.00339	505	0.3535	104.2773
400	4	0.00336	597	0.4179	124.375
450	4.5	0.00333	671	0.4697	141.0511
500	5	0.0033	726	0.5082	154
550	5.5	0.00327	762	0.5334	163.1193
600	6	0.00324	786	0.5502	169.8148
650	6.5	0.00321	810	0.567	176.6355
700	7	0.00318	830	0.581	182.7044
750	7.5	0.00315	852	0.5964	189.3333
800	8	0.00312	876	0.6132	196.5385
850	8.5	0.00309	889	0.6223	201.3916
900	9	0.00306	893	0.6251	204.281

950	9.5	0.00303	870	0.609	200.9901
1000	10	0.003	710	0.497	165.6667
1050	10.5	0.00297	679	0.4753	160.0337

The summary of shear stress for different loading conditions

horizontal(mm) displacement	Shear stress(kpa)		
	100kpa	200kpa	300kpa
0	0.00	0.00	0.00
0.5	10.78	10.20	11.20
1	21.36	27.68	28.50
1.5	28.92	38.89	46.27
2	37.21	56.32	59.34
2.5	53.77	74.06	75.30
3	74.09	85.56	90.47
3.5	91.47	101.18	104.28
4	105.42	114.58	124.38
4.5	117.30	130.12	141.05
5	125.58	140.64	154.00
5.5	131.01	148.56	163.12
6	132.87	150.80	169.81
6.5	128.44	151.78	176.64
7	122.17	149.91	182.70
7.5	118.67	147.56	189.33
8	111.51	146.96	196.54
8.5			201.39
9			204.28
9.5			200.99
10			165.67
10.5			160.03



I, the undersigned, declare that this thesis is my original work performed under the supervision of my research advisor Dr.-Ing Samuel Tadesse and has not been presented as a thesis for a degree in any other university. All sources of materials used for this thesis have also been duly acknowledged.

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