Investigation of Collapsible Soil of Adama-Asella Asphalt Road in the Case of Dodota Area

By:

Abdisa Samuel Edasu

A Thesis Submitted to the department of Civil Engineering

School of Civil Engineering and Architecture

Presented in Partial Fulfillment of the Requirements for the Degree of Masters of Science in Civil Engineering (Geotechnical Engineering)

Office of Graduate Studies
Adama Science and Technology University

June, 2023 G.C

Adama, Ethiopia

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DECLARATION

I hereby declare that this Master Thesis entitled "Investigation of collapsible soil of Adama to Asella asphalt road in the case of Dodota area" is my original work. That is, it has not been submitted for the award of any academic degree, diploma or certificate in any other university. All sources of materials that are used for this thesis have been duly acknowledged through citation

Abdisa Samuel	_______________________	________________
Name of student	Signature	Date

RECOMMENDATION OF ADVISOR

I/we, the advisor(s) of this thesis, hereby certify that I/we have read the revised version of the thesis entitled "Investigation of collapsible soil of Adama to Asella asphalt road in the case of Dodota area" prepared under my/our guidance by Abdisa Samuel submitted in partial fulfillment of the requirements for the degree of Mater's of Science in Civil Engineering (Geotechnical Engineering).Therefore, I/we recommend the submission of revised version of the thesis to the department following the applicable procedures.

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APPROVAL PAGE OF M.SC. THESIS

I/we, the advisors of the thesis entitled "Investigation of collapsible soil of Adama to Asella asphalt road in the case of Dodota area" and developed by Abdisa Samuel, here by certify that the recommendation and suggestions made by the board of examiners are appropriately incorporated into the final version of the thesis.

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APPROVAL OF BOARD OF EXAMINERS

We, the undersigned, members of the board of examiners of the thesis by Abdisa Samuel have read and evaluated the thesis entitled "Investigation of collapsible soil of Adama to Asella asphalt road in the case of Dodota area" and examined the candidate during open defense. This is therefore, to certify that the thesis is accepted for partial fulfillment of the requirements of the degree of Master of Science in civil engineering.

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ACRONYMS

AASHTO……American association of State Highway and Transportation Officials

ASTM………..American Society for Testing of Materials

ERA…………..Ethiopian Roads Authority

GPS…………...Global Positioning System

LL……………..Liquid Limit

MDD………….Maximum Dry Density

NMC………….Natural Moisture Content

NP……………..Non Plastic

OMC………….Optimum Moisture Content

PI………………Plasticity Index

PL……………...Plastic Limit

USCS………….Unified Soil Classification System

ABSTRACT

Collapsible soils are characterized by relatively low density and cohesion, appreciable strength and stiffness in the dry state, but are susceptible to significant deformations as a result of wetting. The collapsible soil sample was collected from *Adama to Asalla asphalt road of the case Dodota area. The objective of this research was to investigate the engineering behavior of collapsible soil of the area with a field and laboratory tests. The test result shows that the soils have very low in-situ densities, mainly consist of sands and silts and low plasticity. The shear strength of all sample were observed as below 50 KN/m^2. The AASHTO classification is an A-4 material (silty soil) which is fair to poor rating in terms of general rating as sub-grade material. The clay fraction was ranges from 19.44%, to 20.73%.. The moisture content was ranges from was 17.35%, to 17.45%. The plastic limit was 29.68 to 30.66. The plasticity index ranges 8.95% to 9.62%. The maximum dry density of sample ranges from 1.513g/cc to 1.558g/cc and the optimum moisture content varies from 24.98% to 27.40%. The soil sample is silty sand with substantial amount of clay and is not suitable for sub-grade material. The structure of the soil has meta stable texture as the bonds between the grains breakdown when the soil is wet. Hence, the collapse process represents a rearrangement of soil particles into a denser state of packing; also the soil has an open, partially unstable and unsaturated fabric.*

Keywords: collapsible soil; shear strength, meta stable texture, unsaturated fabric

CHAPTER ONE

INTRODUCTION

1.1. Background

Collapsible soils, one of the problematic soils, are described as soils that undergo a significant, sudden and irrecoverable decrease in volume upon wetting. These types of soils dominantly contain silt and sand with some clayey material and rock fragments. In Ethiopia, they are present in the Southern part of the Omo River and in the Central land Southern part of the rift valley. Often, their existence around Zeway, Shashemene and Hawassa is manifested by the occurrence of ground cracks and potholes during heavy rains or floods due to hydro-compaction. In Afar region, collapsible soils are present in the form of sand dunes.(ERA, 2009)

Collapsible soils are soils susceptible to large volumetric strains when they become saturated. Numerous soil types fall in the general category of collapsible soils. Collapsible soil are characterized by relatively low density and cohesion, appreciable strength and stiffness in the dry state, but is susceptible to significant deformations as a result of wetting.

Collapsible soils are defined as any unsaturated soil that goes through a radical rearrangement of particles and greatly decreases in volume upon wetting, additional loading, or both. Typically these soils are found in arid or semiarid regions and have a loose soil structure, i.e. large void ratio and water content far less than saturation. Typically the structure of this low-unit weight, unconsolidated sediments consists of coarser particles bonded at their contact points by the finer silt and/or clay fraction, or possibly by surface tension in the water at the air-water interfaces.

The addition of water is a widely used explanation for triggering soil collapse. Collapse can also occur, however, as the result of load application, or wetting, or both. Thus collapse can occur either by increasing the stress above the soil strength or by lowering soil strength below the stress. Whatever the physical basis of the bond strength, all collapsible soils are weakened by addition of the water. Decreased strength is more immediate in cases where the grains are held together by capillary suction, slow in the case of chemical cementing, and much slower in the case of clay buttresses. Thus it may take long periods of time, even years, for the total collapse to take place.

Collapsible soils are the origin of many geological hazards in civil engineering projects. Construction on this type of problematic soil has always been a challenge for geotechnical engineers due to existing gaps in knowledge on this topic. An increase in population levels has resulted in the extensive land development and increased construction on problematic soils. Therefore, it is essential for geotechnical engineers to have sufficient knowledge to deal with all types of the soils, including collapsible soils. Although collapsible soils can withstand high amounts of load in an unsaturated condition, the settlements associated with the penetration of water into the system often lead to expensive repairs (Gaaver, 2012).

Although many researches have been conducted in other countries where collapsible soils exist, such as the United States of America, China, Nigeria, South Africa etc. But in our country Ethiopia there is no satisfactory research and investigation has been done and published on collapsible soils. Many of the focus areas of the researches undertaken so far have given emphasis on expansive soils, among any other problematic soils, and less attention is given on the existence of collapsible soils.

1.2. Statement of the problem

Collapsible soils can pose serious problems to building foundations and road embankments built on them. Literature on collapsible soils is quite extensive; there are significant voids that still need to be filled. An area that appears to require further work pertains to identification and characterization of these soils even though, their presence in Ethiopia is stated in few literatures, a thorough identification and research has not been done satisfactorily. Even the local manuals, except mentioning their identification and some properties, they fail to recommend detailed suitable remedial measures for collapsible soils. Consequently, lack of knowledge and experience is observed in professionals working in the industry. Therefore, consecutive and extensive researches shall be conducted in construction areas covered by collapsible soils in order to identify the index and mechanical properties of the same and Finally, a more general approach for the selection of mitigation/improvement methods to deal with these soils is also needed and to alleviate the awareness problem found among construction professionals.

1.3. Objectives

1.3.1 General Objective

The main objectives of this research study are to investigate collapsible soil of Adama to Asella Asphalt road in the case of Dodota area.

1.3.2. Specific Objectives

- To determine engineering proprieties of collapsible soil of the study area.
- To develop a better understanding of the behavior of collapsible soils of the study area
- to suggest possible treatment and control remedial measures

1.4. Significance of the Study

The thesis will try to study the collapsible soils found in Dodota area of Adama to Asella asphalt road. In so doing, engineering properties which are thought to be directly related with the collapse nature of the soils will be studied.

It is important to know the characteristics and properties of collapsible soil due to serving and enhancing other principles, subjects, and related projects, such as construction. Being experienced with estimating problems that a project will face in the future leads to eliminate those factors and parameters affecting the durability and health of the project to a great extent. In other words, if engineers are able to determine the behaviour of the field land used for any types of projects, they can minimize and prevent the future problems by considering and executing extra steps before and while building the projects. Following this, being able to determine that there exists collapsed soil and starts mitigating problems occur due to this type of soil; the constructed project will be safer and stabilizer. Therefore the outcome of this study will help in better understanding regarding collapsible soil. And also give guidance for future research.

1.5. Scope and Limitation of the Study

- The scope of this thesis is to the extent of examining engineering properties of collapsible soil including grain size analysis, atterbergs limit(LL,PL), proctor compaction test, natural moisture content test and specific gravity test on collapsible soil samples collected from Adama to Asella asphalt road of the case Dodota area. In addition, the research try to suggest possible treatment and control remedial measures.

In conducting the research, the following limitations were encountered: -

- Only the physical properties of the soils which are directly related with collapsibility are investigated.
- Due to time and cost limitations other engineering tests (including one dimensional odometer and CBR tests) were omitted. Hence, the conclusion and recommendation finally given on this paper are taking account of the above limitation.

CHAPTER TWO

LITRATURE REVIEW

2.1. Introduction

In this chapter, review of researches conducted on the collapsible soil, type of collapsible soil, factors affecting collapsible soil and remedial measure to control or reduce the problems will be discussed.

Expansion, dispersion, and collapsibility of soil give rise to many geotechnical difficulties including inadequate bearing capacity, the potential for unacceptable settlements, and slope instability. Soil composition, mineralogy, structure, and nature of the pore fluid are the factors responsible for such characteristics (Briscolland and Chown, 2001). These soils are typically silt and sand size with a small amount of clay and low plasticity index (Pawlak, 1983). Collapsible soils are unsaturated soils which present a potential for large deformation and a complete change to the whole particle structure after wetting, with or without loading. These soils are characterized with loose structures composed of silt to fine-sand-size particles. Collapsible soils are generally deposited in arid and semi-arid regions. According to Clemence (1981), Collapsible soils are soils that compact and collapse after they get wet. The soil particles are originally loosely packed and barely touch each other before moisture soaks into the ground. As water is added to the soil in quantity and moves downward, the water wets the contacts between soil particles and allows them to slip past each other to become more tightly packed. Water also affects clay between other soil particles so that it first expands, and then collapses like a pack of cards. Another term for collapsible soils is "hydro-compactive soils" because they compact after water is added. The amount of collapse depends on how loosely the particles are packed originally and the thickness of the soil that becomes wetted. Collapsible soils consist of loose, dry, low-density materials that collapse and compact under the addition of water or excessive loading. Soil collapse occurs when the land surface is saturated at depths greater than those reached by typical rain events. This saturation eliminates the clay bonds holding the soil grains together (Mulvey, 1992). A collapsible soil is one in which the constituent parts have an open packing and which forms a meta stable state that can collapse to form a closer packed, more stable structure of significantly reduced volume.

The most important property that a collapsible soil possesses is that of low interparticle bond strength. It is when these bonds are destroyed (through either soil loading, soil saturation or a combination of both) that the collapse of the soil occurs (Rogers, 1994*)*. Kakoli (2011), highlighted that collapsible soils experience significant volume decrease due to the increase of soil moisture content, with or without an increase in the in-situ stress level. Foundations on collapsible soils suffer from sudden settlement, which may contribute to serious damage or catastrophic failure due to inundation.

2.2. Collapsible soils

Collapsible soils have been the subject of many studies for more than 70 years. As the name indicates, this type of soil undergoes a sudden volume change when the water content increases; this change, which occurs with or without the presence of additional loading, causes the soil particles to move to a denser structure (Sultan, 1969; Jennings and Knight, 1957). Due to the sensitivity of this type of the soil to water, collapsible soils are also called moisture-sensitive soils (Houston et al., 2002).

Cementing agents that exist in the structure of collapsible soils stabilize the open and partially unstable fabric, and thereby give the soil high bearing capacity in the unsaturated state (Rust et al., 2010). However, the addition of water to the system, along with high pressure on top of the soils, softens the inter-particle bindings and leads to a critical reduction in volume (Barden et al., 1973). Common features associated with collapsible soils are: high porosity, high void ratio, sensitivity to water, low dry density, low inter-particle bonds, loosely cemented and geologically younger deposits.

2.3. Factors affecting collapse of Soil

Traditionally, macro mechanics methods to study soil collapse only considered one factor such as dry density, moisture content, atterberg limits, or loss of suction responsible for the collapse (Barden et al., 1973; Dudley, 1970). The concepts of soil mechanics were used to study the collapsible soil in the macro mechanics methods. Most of the work carried out on the parameters that govern collapse behavior had focused on initial dry density, water content, degree of saturation, and applied load. Brief background is provided below on the factors affecting collapse.

2.3.1. Soil type

As explained previously, the structure of collapsible soils consists of silt-size particles bound together with clayey-bridge (Lawton et al., 1989). In the unsaturated state, the clayey-bridge works as a cementations agent between the soils particles which increases the shear strength of the soil. The higher the amount of clayey material, the higher the void ratio of the soil. Upon the addition of water to the system, the clayey-bridges dissolve and the soil experiences collapse. Therefore, a higher amount of clayey content results in a greater amount of collapse (Basma and Tuncer, 1992). However, Lawton et al. (1989) showed that after a certain point, the addition of clay to the soil leads to expansibility behavior. Later studies on collapsible soils with a high amount of clay (more than 30%) proved that clay content is not the key factor and there is no monotonic relation to the collapse potential of soil (Li et al., 2016; Ruilin and Lizhong, 1999; Krayev, 1969).

2.3.2. Dry unit weight

This feature has a significant effect on the collapse potential of soil (Bigdeli and Siddiqua, 2016; Houston et al., 2002; Rogers, 1995). Generally, higher values of dry unit weight result in a denser structure with a lower void ratio. In addition, densifying the soil forces the particles to rearrange into a more stable structure by destroying the weak cementitious inter-particle bonds (Basma and Tuncer, 1992). Therefore, when additional water infiltrates the system, the influence of weak bindings on the collapsibility is less profound (Vanapalli et al., 1999).

On the other hand, ERA’s 2013 Geotechnical Design Manual discusses that usually, the collapse potential of the soil depends on density, gradation, the initial water content, composition, and the extent of loading at the time of wetting. A good indication of the potential for high collapse potential is a very low density. Typical collapsible soils have densities of less than 1.6 g/cc (mostly in the range of 1 to 1.585 g/cc).

However, it should be noted that since unstable soils have a wide range of dry densities which also apply to non-collapsible soils, the dry density alone seems to be misleading and an unreliable indicator of collapse. Furthermore, other factors such as the soil moisture content, which greatly influences the soil collapse was not taken into account.

Bell and Bruyn (1997) reported a criterion suggested by Gibbs and Bara (1962) based on natural dry density and liquid limit, which distinguishes between collapsible soils and non-collapsible soils as shown in Figure 2.1 The method is based on the premise that a soil, which has enough void space to hold its liquid limit moisture at saturation is susceptible to collapse on wetting. Figure 2.1 shows that soils above the lines (labeled as potentially meta stable) are in a loose condition, and when fully saturated will have a moisture content greater than liquid limit and are susceptible to collapse. [1]

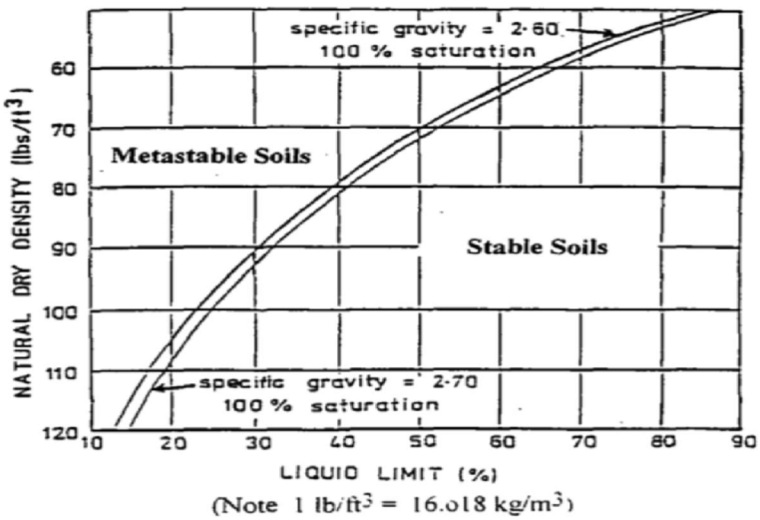

Figure 2. 1 Classification of collapsing soils Based on Liquid Limit and Dry Unit Weight

(after Gibbs and Bara 1962)

2.3.3. Water content

Collapse triggering saturation degree varies between different types of soil. This factor depends on the initial moisture content of the soil as well as the applied pressure on soil structure. Increasing the moisture content of the soil results in dissolution of cementitious agents and therefore weakening the meta-stable structure of collapsible soils (Chen et al., 2006); consequently, the structure of soil fails and triggers the collapse. It is noteworthy that an increase in moisture content and applied load increases the collapse potential to a certain point beyond which no substantial settlement happens (Li et al., 2016). This collapse behavior is explained by the fact that there is a maximum densification level for any type of soil, including collapsible soils (Basma and Tuncer, 1992).

2.3.4. Properties of the pore fluid

Population over-burst has necessitated the world for rapid urbanization and industrialization. This has resulted in a significant increase in acid precursor emissions and environmental pollution (Pankaj et al., 2012). One of the most critical problems caused by industrialization is acid rain. Stalin and Muthukumar (2002) identified that acid precursor emissions could reduce the pH level of precipitation to as low as 5.6. They reported severe acidification of soil due to acid rain which altered the physico-chemical properties of the studied soil (Pankaj et al., 2012; Stalin and Muthukumar, 2002). Acidification of the environment has been the subject of many studies in Eastern Canada (Scott et al., 2010), and awareness of acid rain and deposition of acid precursors in soil have recently increased in Western Canada (Hazewinkel et al., 2008; Shewchuk, 1983). Studies on the effect of acid rain on the properties of lime and cement stabilized soils reported a tremendous change in the strength of the soil (Kamon et al., 1996). A high concentration of Hydrogen ions in the soil changes the cation exchange capacity and chemical reactions (Gupta and Singh, 1997). Sunil et al. (2006) reported that soil pollution as a result of landfill leachate changes the pH level of the soil and consequently alters the mechanical properties of compacted clay by rearranging the clay particles.\

A low pH level of the pore fluid can also affect the structure of the soil and result in shifting the grain-size-distribution of the soil toward the silt fraction and increasing the angularity of the particles (Pankaj et al., 2012). In addition, studies on the effect of various pH levels on the structure of montmorillonite indicate a significant increase in micropores and mesopores and decrease in surface area of the soil structure (Pin-Hua et al., 2006; Lebron et al., 2002; Altin et al., 1999). New studies are focused on the effect of initial pH level, initial solute concentration, and temperature of the pore fluid on the adsorption capacity of loess (Wang et al., 2009).

2.3.5. Suction of soil

Stress state principal of soils can be used to describe the mechanical behavior of soils in both saturated and unsaturated conditions (Li et al., 2016). Terzaghi (1943) was the first one who introduced the principle of effective stress equation for saturated soils. Later, this equation was modified to be used for unsaturated soils (Bishop, 1959). Stress state principal has been widely acknowledged to describe the collapse mechanism of unsaturated soils by introducing net normal stress and matric suction as the two stress state variables (Sun et al., 2007a & b; Khalili et al., 2004; Sun et al., 2004; Fredlund and Morgenstern, 1977). Other researchers have proven the application of the effective stress principle to describe the behavior of collapsible soils (Khalili et al., 2004; Barden et al., 1973; Bishop and Donald, 1961). The outcome of these studies has shown that the reduction in the matric suction of a soil due to wetting results in the change in effective stress and loss of strength (Pereira and Fredlund 2000; Fredlund and Morgenstern 1977). Tadepalli and Fredlund (1991) and Tadepalli et al. (1992) verified the one-to-one relationship between matric suction and total volume change due to collapse. It is shown that any increase in moisture content of a collapsible soil not only decreases the matric suction, it also affects some other phenomena such as dissolving soluble salts which are considered to act as bonds in unsaturated soil (Houston et al. 1988; Guan 1986). As a result, more collapse can be generated due to the solubility of the bonds and loss of strength.

2.4. Ground Improvement of Collapsible Soils

Many mitigation methods are available for collapsible soil problems. These include removal of the collapsible soil, removal and replacement, prevention of wetting, chemical stabilization, pre-wetting, controlled wetting, dynamic compaction, pile or pier foundations, and differential settlement resistant foundation and structural design(SBI, 2018)

The method selected depends upon several factors such as the depth of the collapsible soil deposit, the type of structure to be built, the capability of the structure to withstand settlement, the likelihood of the foundation to become wetted, and the stresses imposed on the foundation by a structure (Knodel, 1992).

In assessing mitigation measures, the issue of shallow or deep deposits must be considered, and that distinction is often not clear cut. If collapsible soils are identified in foundations for structures, a prime objective should be to stabilize those soils before construction. It is much less expensive to solve the problem at that point rather than after the project is built. Then maintenance costs are reduced; project operation is more efficient; and the lives of structures are extended (Knodel, 1992). Here under, review of literature on selected mitigation measures are presented on the following sections.

2.4.1. Soil replacement

One of the simplest methods to deal with collapsible soils is to excavate, remove the collapsible soil, and replace it with a compacted soil with better mechanical properties. This method can only be used if the layer of problematic soil is thin and close to the surface (Anayev and Volyanick, 1986).

2.4.2. Pre wetting

In this method the collapsible soil is flooded with water via boreholes, trenches, or ponding to dissolve the cementitious inter-particle bindings without exerting any additional load. This method is used mainly in the construction of canals and roads, as prewetting is not an effective method for the foundation of the buildings where an increase in overburden pressure of the soil after construction will result in further collapse (Rollins and Rogers, 1994; Houston et al., 1988; Gibbs and Bara, 1967).

2.4.3. Controlled wetting

This method is similar to prewetting, but in controlled wetting the additional water is introduced to the system once construction is complete and the soil should be wet in a carefully-monitored manner to prevent failure in the surrounding soils (Al-Rawas, 2000).

2.4.4. Moisture control

This method limits and controls water infiltration into the soil structure. This can be done through (i) installation of a drainage system, (ii) using impermeable materials on the soil, (iii) restricting landscape vegetation and (iv) decreasing the permeability of the soil (Manckenchinie, 1980).

2.4.5. Heat treatment and small explosions

In this method, gas and fuel are burned in pressurized boreholes to a temperature up to 1000oC. This technique destroys the structure of the loess to develop a more stabilized soil for construction projects (Bell and Bruyn, 1997).

Compaction control: Compaction is the most common and cheapest method to remediate collapsibility problems. This technique can greatly decrease the degree of collapse in compacted zones while decreasing the induced stress to which collapsible soil is subjected (Rollins and Rogers, 1994). Compaction can solve the collapsibility problem for soil layers up to 10 meters in depth; however, it can only deal with collapsible soils above the groundwater table with a saturation level of less than 60% (Kozubal and Steshenko, 2015; Wang et al., 2011).

2.4.6. Chemical Stabilization

Problematic soils have given rise to many geological hazards. Among them, collapsible loess soils are notoriously difficult to deal with due to the collapse potential variability across even a small site (Jefferson et al., 2005). Therefore, a carefully-performed site investigation prior to any ground improvement is mandatory to prevent failure due to collapse (Basma and Tuncer, 1992).

In general, chemical stabilization have been used to change and improve the engineering properties of natural soils. It should be noted that collecting enough information about the chemical composition and its interaction with soil particles is essential when chemical stabilization techniques are used. Failure to perform a pretreatment assessment may result in engineering hazards (Hunter, 1988). Much research has been done on the suitability of stabilizing additives such as cement, lime, fly-ash, polymers, silicates, etc. to remediate collapse potential (Tabarsa et al., 2018; Jefferson et al., 2008; Basma and Tuncer, 1992).

A suite of improvement approaches is available to mitigate the collapse potential of these soils, and these include either pre-construction or post-construction on loess, depending on the project requirements. This chapter provided brief background on the collapsible soils, the factors affecting the collapse, and the remediation methods. By highlighting the gap of knowledge in this topic, this paper will provide better understand collapsible soils along the study area.

CHAPTER THREE

MATERIAL AND METHODS

3.1. Introduction

Geo technical investigation on the selected area has been performed by different approaches and methodology. The approaches and methodology carried on this thesis include sampling and data collection, field test and laboratory tests.

3.2. Study Area

The study area lies in south eastern Oromia regional state along Adama-Asala road section (i.e,which is 115km from Addis Ababa) .The area is found making the shoulder of the rift from the east direction with an elevation of ranges from 1600m to 2000m amsl. The main features of this area include flat morphology with frequent appearance of greyish silty sand soil. The geological formation of the study area is mainly covered by quaternary sediments of lacustrine origin. As observed during the site investigation, the lacusrine deposit found in study area is greyish to whitish silty sandy soil. Moreover, gypsiferous and fossiliferous limestone of lacustrine origin is observed in study area.

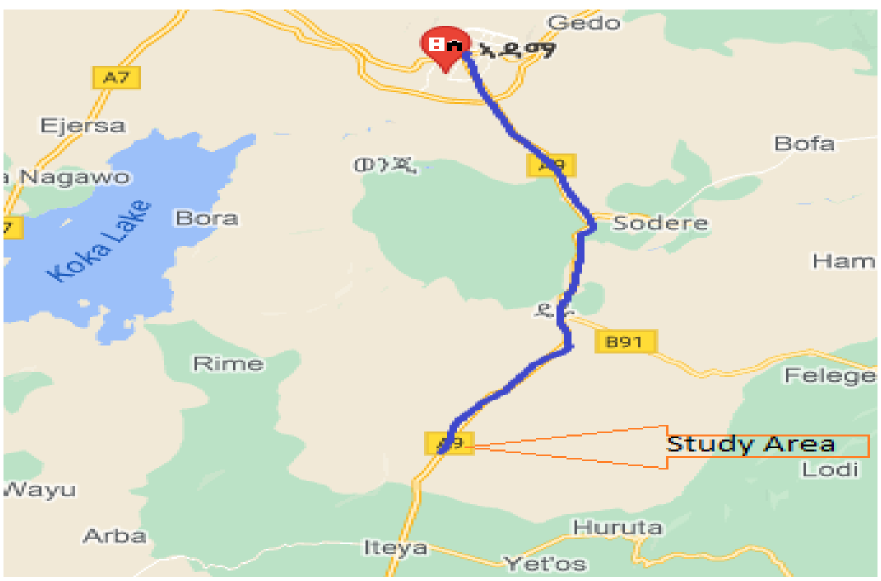

Figure 3.1 Location Map of the study Area (scale 1:250, 0000)

3.3. Sampling and Data Collection

To determine engineering properties and to assess the collapsible nature of the soil, four representative disturbed samples with 500m interval have been collected by digging test pits at each location. Collapsible soil samples have been taken by excavating four pits manually using pick axe and shovel. Depth of test pits (which is usually thought to be with in 1.5m depth below the pavement, as per ERA's 2013 Site Investigation Manual). Representative disturbed samples for soil classification, establishing moisture density relationship, determining specific gravity of soil and the unconfined compressive strength test have been collected. Disturbed soil samples will be collected for the purpose of conducting tests mainly due to the fact that arise from the difficulty of obtaining undisturbed soil samples, since the soils in the test pits area are mostly made up of sand and silt particles and lack cohesive nature. The position of each test pit will located by hand held GPS.

Table 3. 1 Coordinate of Sample Area

Test Pits	Longitude	Latitude	Elevation	Depth of Sampling
TP-1	39°17'23"	8°15'65"	1769.98	1.5m
TP-2	39°17'11"	8°15'52"	1779.36	1.5m
TP-3	39°16'94"	8°15'33"	1792.52	1.5m
TP-4	39°16'79"	8°15'13"	1800.60	1.6m

Then after, the samples to be used for conducting natural moisture content test will be sealed tightly in a moisture bag and the remaining samples to be used for other classification tests were put into sacks and the samples have been transported to ASTU Laboratory for conducting the necessary tests and Oromia Engineering Corporation (OEC) for UCS test.

3.4. Field Test

Field density tests have been conducted to determine the in-situ state of compaction inside the 1m deep test pits by sand replacement method in accordance with AASHTO T-191 (1993).

The sample were taken from 150mm depth and then taken to laboratory which is collected in polyethylene bag, weighed, tightly sealed for the natural moisture content determination.

3.5. Laboratory Test

Representative disturbed soil samples collected from the test pits have been safely transported to for testing. The laboratory testing for soil samples included moisture content, Atterberg Limits, sieve analyses, specific gravity and compaction tests. Each soil sample is classified on the basis of texture and plasticity in accordance with the AASHTO and Unified Soil Classification Systems (USCS). The standards used to conduct the laboratory tests were summarized under table below.

Table 3. 2 Standards Adopted for Laboratory Tests

S.N	Laboratory test type	Standard
1	Natural moisture content Test	AASHTO T-265
2	Atterbergs Limit Test	AASHTO T89/T90
3	Grain Size Analysis Test	AASHTO T 88
4	Specific Gravity Test	AASHTO T 100
5	Compaction Test	AASHTO T-180
6	Unconfined Compression Test	AASHTO T-208-92

CHAPTER FOUR

RESULTS AND DISCUSSIONS

In this chapter the test results obtained from field and laboratory tests are discussed in detail.

4.1. Field Test

4.1.2 Field Density

In order to acquire the in-situ state of compaction, in-place density tests have been conducted by sand replacement method in accordance with AASHTO T-191. The material from the hole to a depth of 150 mm was carefully collected in a polyethylene bag, weighed, tightly sealed, and labeled for subsequent natural moisture content determination. Dry, free-flowing sand of known density was then poured into the hole from the standard sand cone apparatus. From the weight of the sand the volume of the hole was determined. The in-situ bulk density characteristics of the sub-grade at the sampling locations were investigated by carrying out density test based. In addition, the natural moisture contents of the soil samples were determined as per AASHTO T-265. These moisture content values were used in order to calculate the field dry densities. The results of these tests are shown on Table 4.1

Table 4. 1 Field Density Test Results

Test Pit	Bulk Density g/cc	NMC %	Dry Density g/cc
TP-1	1.213	17.350	1.034
TP-2	1.174	16.329	1.010
TP-3	1.150	17.448	0.979
TP-4	1.107	14.995	0.963

4.2. Laboratory Tests and Analysis

4.2.1 Grain Size Analysis

Wet sieve and Hydrometer analysis tests were conducted to determine the percentage of different grain sizes contained within a soil. Wet sieve analysis was performed to determine the distribution of the coarser, larger-sized particles, and the hydrometer method was used to determine the distribution of the finer particles. The test results show that percentage passing sieve no. 200 on TP-1 were 59.579%, 60.40% on TP-2 58.00% on TP-3 and 60.079% on TP-4 respectively.The particle size distribution shows the following compositions.

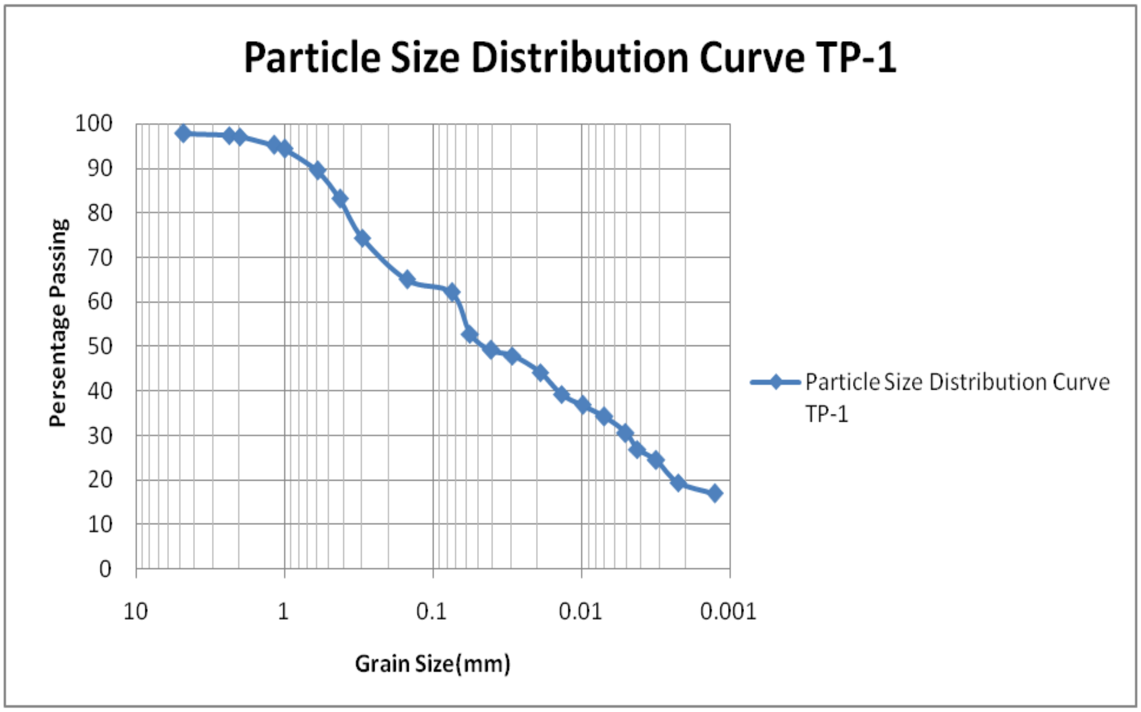

Figure 4. 1 Particle Size Distribution Curve for TP-1

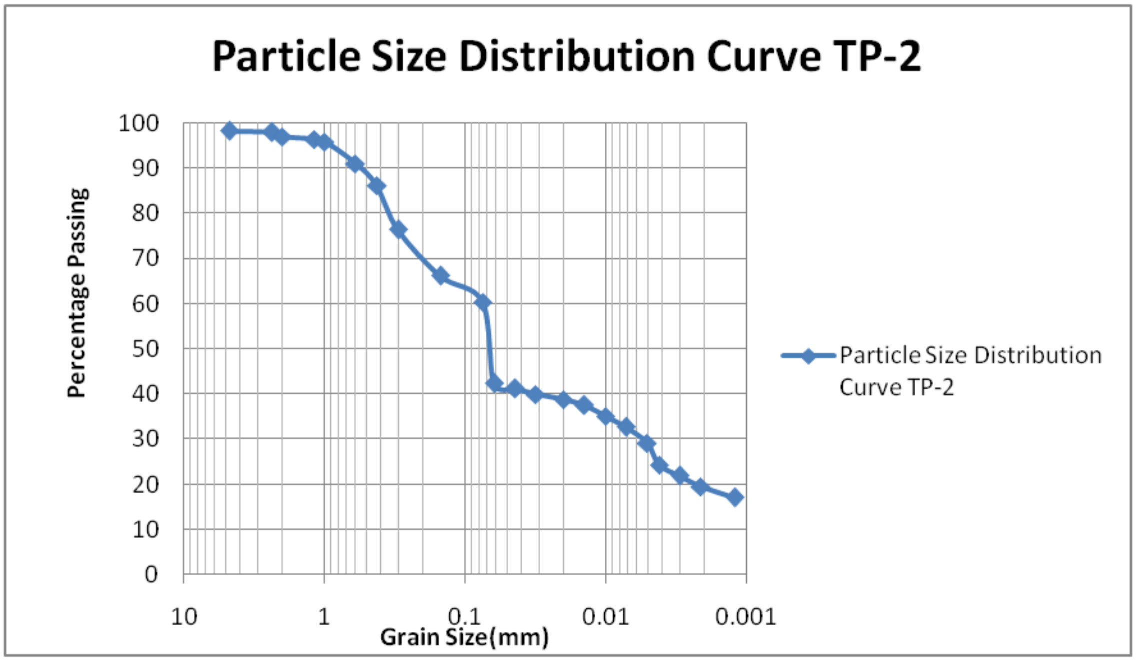

Figure 4. 2 Particle Size Distribution Curve for TP-2

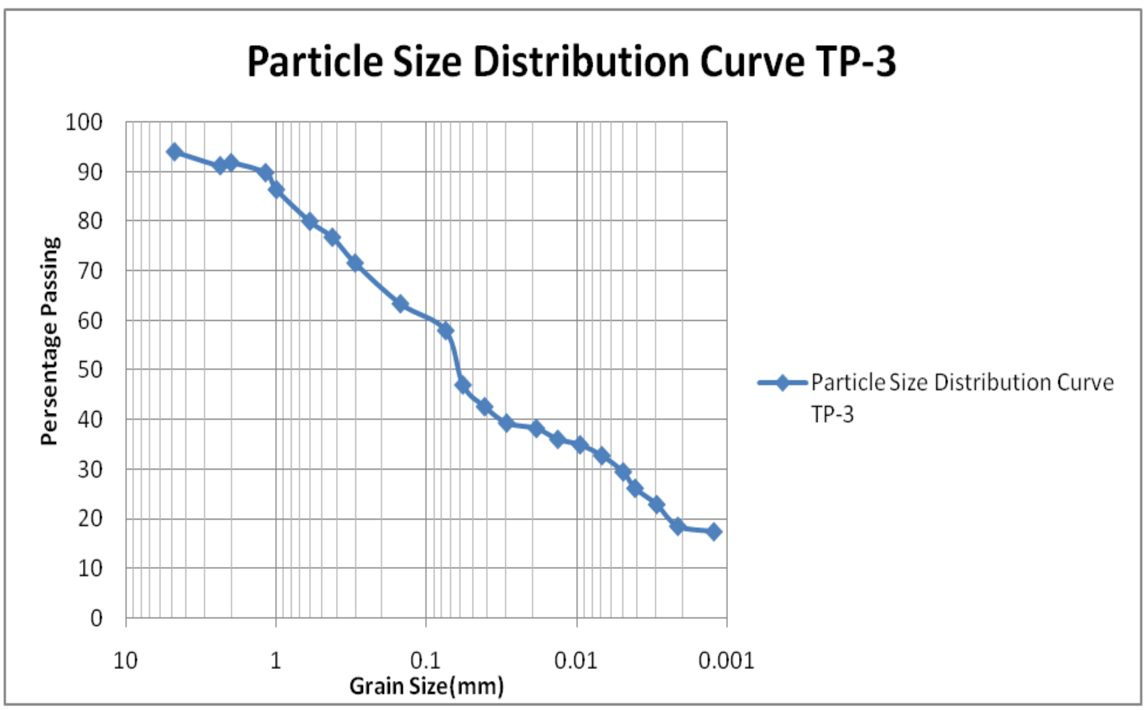

Figure 4. 3 Particle Size Distribution Curve for TP-3

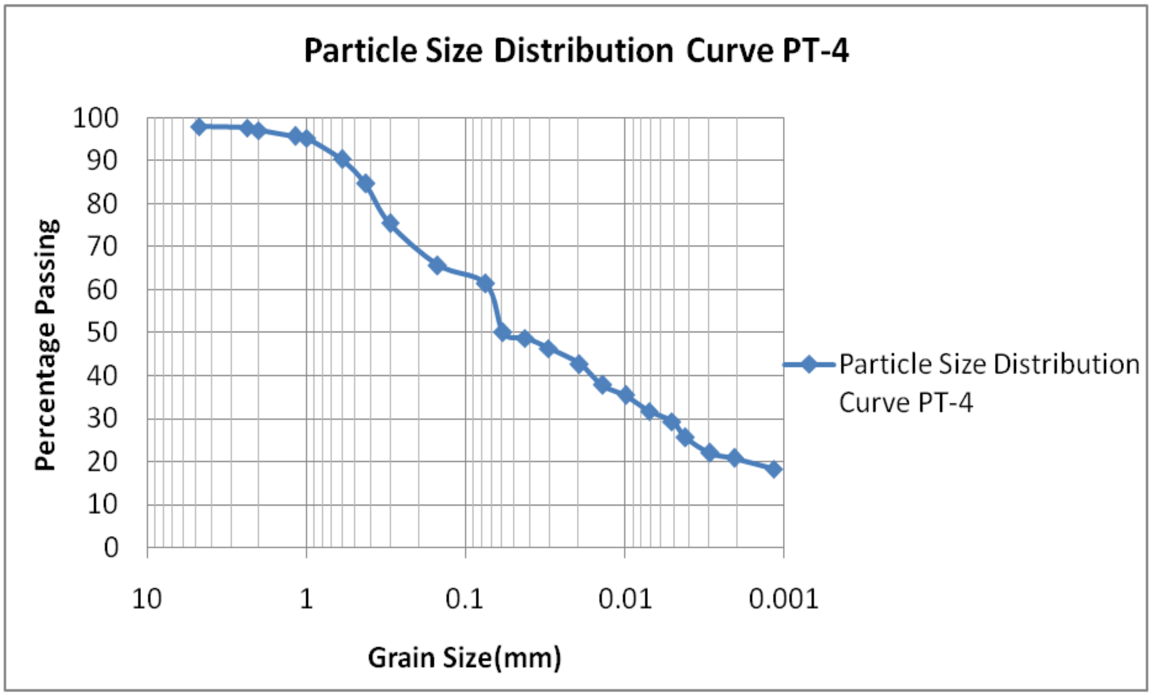

Figure 4. 4 Particle Size Distribution Curve for TP-4

Table 4. 2 Percentage of grain size analysis test

Test Pit	%Gravel (>2.0mm)	%Sand (2.0mm-0.06mm)	%Silt (0.06-0.002mm)	%clay (<0.002mm)	Remark
1	3.00	41.74	33.42	19.44	Silty Sand
2	1.20	54.64	22.80	19.56	Silty Sand
3	2.20	44.75	28.45	18.60	Silty Sand
4	0.90	47.11	29.26	20.73	Silty Sand

As it was mentioned in the previous chapters and as discussed also in many literature s, including ERA's 2013 Site Investigation Manual, collapsible soils mainly consist of sands and silts with a substantial amount of clay content. The same trend has been observed in the particle size analysis of both samples. The soil sample is silty sand with low plasticity index and with considerable amount of clay which serves as the binder. The silt controls the behavior of the soil, from the physical structure of the soil observed; there is apparent cohesion between the soils which collapses upon wetting. The soil Classified as ML group (inorganic silts, fine sand with low plasticity) according to the Unified soil Classification System, USCS and as an A-4 material (silty soil) which is fair to poor rating in terms of general rating as sub-grade material according to AASHTO classification.

4.2.2 Atterbers Limit Test

The Atterberg limit test determines that moisture content at which the soil will flow under its own height. It defines boundaries of several state of consistency of plastic soil. It is used to determine plasticity of soil. Atterberg limits were determined from the soil samples. The results of the Atterberg limit of the soil samples are presented in Figures and in Table below

Table 4. 3 Atterbergs Limit Result

Test Pit	Liquid limit LL (%)	Plastic limit PL (%)	Plastic Index PI (%)
1	39.30	29.68	9.62
2	39.65	30.66	8.99
3	37.90	29.95	7.95
4	39.20	30.25	8.95

Generally, the soil along the study area is non-plastic or low plastic range and they are cohensionless. Therefore, the non-cohesiveness in the area accounts for the soil to collapse. Using dry density values from field tests (Table 4.1) and Atterberg limits (Table 4.3) it can be observed that for all test pits, the soils fall under the “potentially meta stable” portion of the plot suggested by Bell and Bruyn (1997) (Figure 2.1).

According to Priklonski method which depends on the estimation of the subsidence coefficient, as shown in Equation below (Priklonsil,1952)

Ko = (Wo - P.L)/ P.I

Where; Ko: Coefficient of subsidence, Wo: Natural moisture content, P.L: Plastic limit, and P.I: Plasticity index. The soil collapsibility is classified in Table 4.4

Table 4. 4 Soil classification according to the coefficient of subsidence (Priklonsil,1952)

Ko	Soil Classification
< 0	Highly Collapsible Soils
>0.5	Non Collapsible Soils
>1.0	Swelling Soils

.

Table 4. 5 Test Result of Coefficient of subsidence

Test Pit	NMC(Wo)%	PL (%)	PI (%)	Ko
TP-1	17.350	29.68	9.62	-1.282
TP-2	16.329	30.66	8.99	-1.594
TP-3	17.448	29.951	7.95	-1.573
TP-4	14.995	30.25	8.95	-1.704

From the above obtained value of Coefficient of subsidence, Ko we can conclude that the soil is classified under Highly Collapsible Soils of Ko less than 0.

4.2.3. Moisture-Density Relationship

Laboratory density test shows maximum dry density (MDD) and the optimum moisture content (OMC) of the soils. One of the major reasons for carrying out compaction test on soils is to increase soil strength and to prevent seepage of water through the soil. Thus, water content and dry density affect soil strength, which will increase when the soil is compacted to a higher density, and when the soil looses water, dries and hardens.

Four (4) disturbed soil samples from sites of the study area were analysed for laboratory compaction test. The compaction tests result shows that the optimum moisture content (OMC) ranges from 24.98% to 27.40%, while the maximum dry density ranges from 1.513g/cc to 1.58g/cc .The maximum dry density values (MDD) are generally low because the soil is loose and thus susceptible to collapse and The low values of the dry density indicate that the natural deposits are loose and account for the high void ratio. This high void ratio of the soil will generate high flow velocities, high seepage pressure and therefore, the low maximum dry density of the soils in the area account for the formation of soil to collapse.

Finally the level/degree of compaction of the soil is computed as the ratio of the field dry density to the laboratory maximum dry density.From the detail analysis result shown in the Table 4.4 below, it can be seen that the values of the field dry density are generally lower than the values for the laboratory MDD. Accordingly, the average level compaction of the soil ranges from 62.64% to 68.35%.

Table 4. 6 Moisture-Density Relationship Test Results

Test Pit	in-situ Dry Densities g/cc	MDD(g/cc)	OMC (%)	Level of Compaction
1	1.034	1.513	27.10	68.35
2	1.010	1.558	27.40	64.79
3	0.979	1.580	24.98	61.98
4	0.963	1.538	27.00	62.64

The soil compacted at their dry densities less than their maximum dry densities and water content less than there OMC will exhibit high Collapsibility at these conditions. Soil sample has low maximum dry density of 1.58g/cc and low optimum moisture content of 24.98 %. Hence cannot be used as a sub-grade material or construction material.

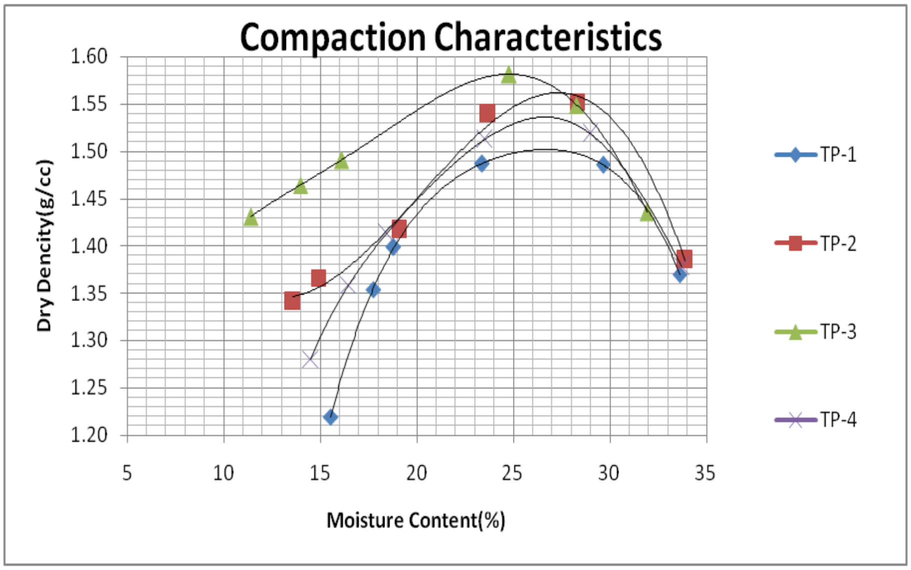

Figure 4. 5 Compaction Characteristic of Soil Sample

4.2.4. Specific Gravity

Specific gravity tests were conducted on the samples collected from all test pits. The specific gravities of the samples are 2.67, 2.68, 2.69 and 2.68 for test pits located at TP-1,TP-2,TP-3 and TP-4 respectively.

4.2.5. Unconfined Compression Test

There are three methods of laboratory shear strength test. These are direct shear test, triaxial compression test and unconfined compression test. For this study unconfined compression strength test was conducted.The strength of a soil determined by compression testing varies with extremes of the length to diameter ratio and the rate of strain. It is generally accepted that ratios of length to diameter of 2 to 2.5 are satisfactory. Similarly, satisfactory rates of strain are 0.5 to 2.0% per minute. For the current investigation, length to diameter ratio of 2 (38mm diameter and 76mm height) and a strain rate of 2% per minute (1.52 mm per minute) was used.

The primary purpose of this test is to determine the unconfined compressive strength, which is then used to calculate the unconsolidated undrained shear strength under unconfined conditions. For soils, the undrained shear strength (su) is necessary for the determination of the bearing capacity of foundations.The undrained shear strength (su) of a cohesive soil is equal to one half the unconfined compressive strength (qu) when the soil is under the f = 0 condition (ϕ = the angle of internal friction).The unconfined Compressive strength, q_u for the soil sample is below 50 kN/M^2. The result of the test is given in table below and Failure occurs at a rate of strain of below 5.0 %

In the unconfined test, no radial stress is applied to the sample ($\sigma 3 = 0$).The plunger load, P is increased rapidly until the soil sample fail, that is cannot support any additional load. The loading is applied quickly so that the pore water cannot drain from the soil.The effect stress path is unknown since pore water pressure changes are not normally measured.This test is considered as undrained shear test assuming that there is no moisture loss from the specimen during the test. In the unconfined compression test the internal angle of friction is negligible and the soil assumed to have only cohesion.

Table 4. 7 General Relationship of consistency and Unconfined Compression strength of clays after Das

Consistency	qu (KN/M^2)
Very soft	0-25
Soft	25-50
Medium	50-100
Stiff	100-200
Very Stiff	200-400
Hard	>400

Table 4. 12 Summery of unconfined compressive strength and cohesion

Test Pits	Unconfined Strength,qu(KN/M2)	Cohesion,C(KN/m^2)	Consistency
TP-1	45.56	22.78	Soft
TP-2	49.53	24.76	Soft

TP-3	49.69	24.85	Soft
TP-4	40.92	20.46	Soft

From the above test result the unconfined compressive strength of all sample are low in value and have soft consistency.

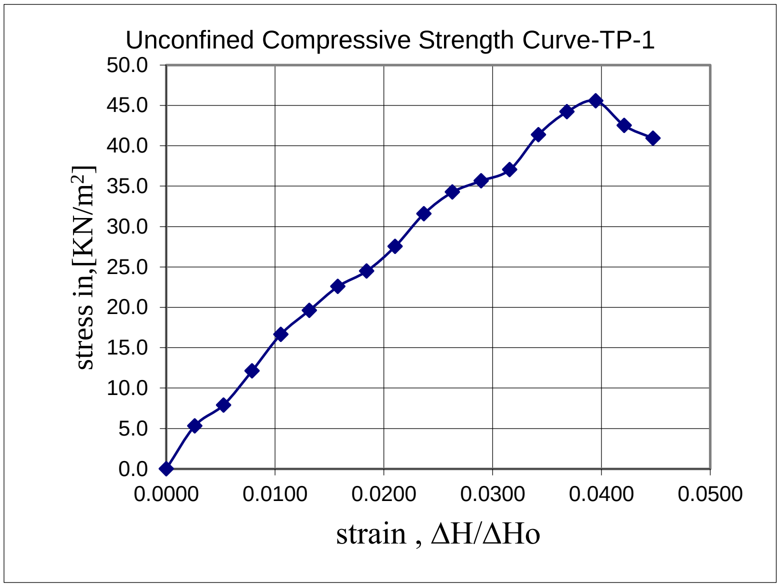

Figure 4. 6 Unconfined Compressive Strength TP-1

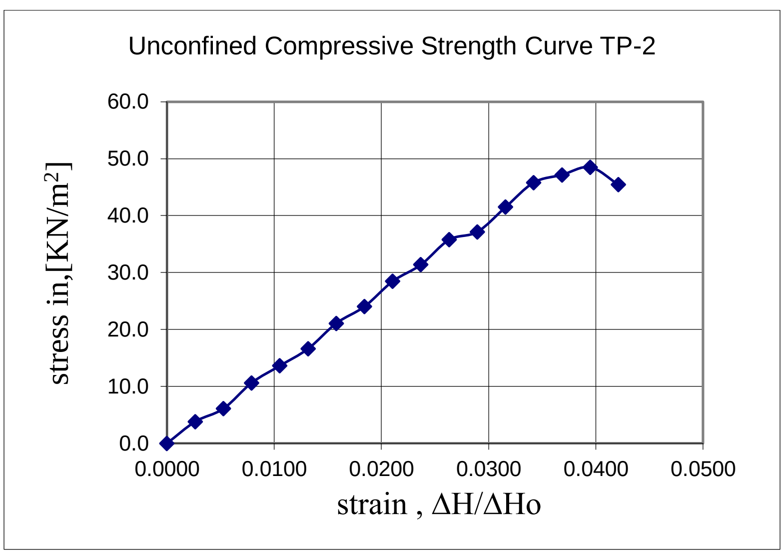

Figure 4. 7 Unconfined Compressive Strength TP-2

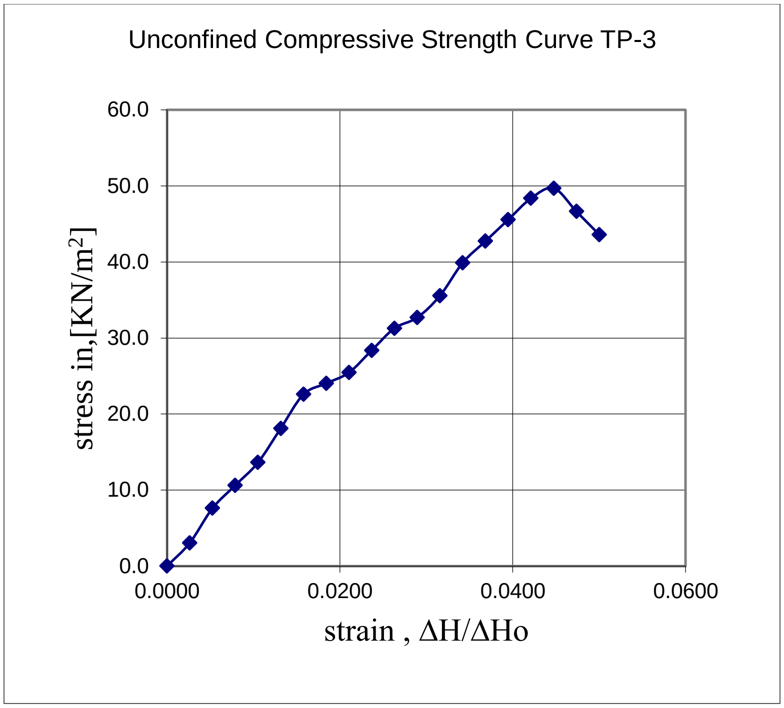

Figure 4. 8 Unconfined Compressive Strength TP-3

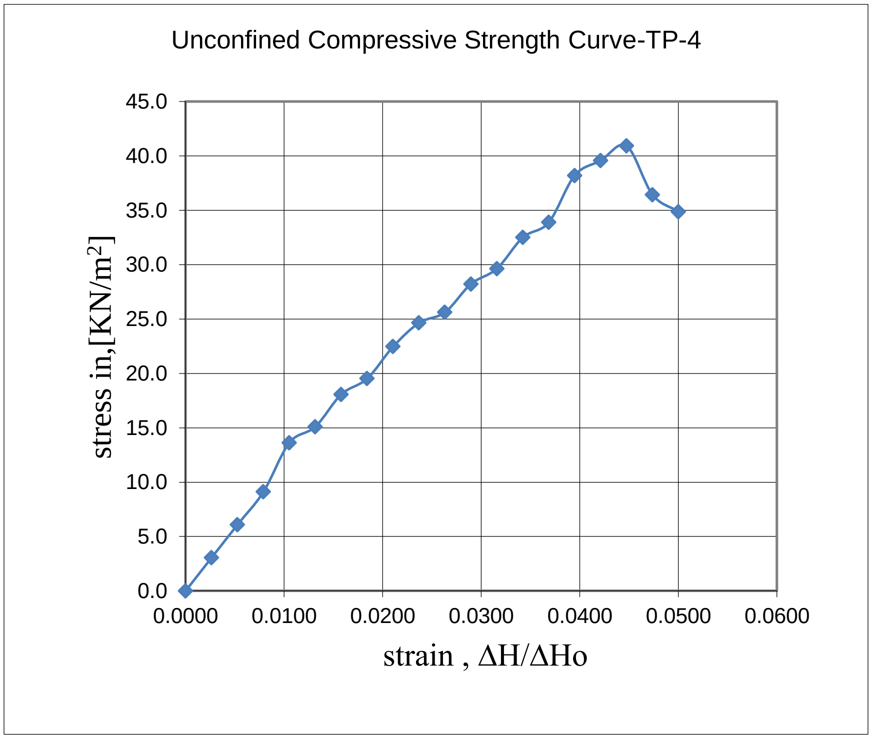

Figure 4. 9 Unconfined Compressive Strength TP-4

4.3. Suggested Remedial Measures

Many research works concerning ground improvement techniques are recommended in difficult ground conditions, examples of which are swelling soils, collapsible soils, soft soils and organic soils. When a project site comes across any of the above difficult conditions, possible alternative solutions may be one or more of the following as needed for a particular site: design the planned structure, remove and replace unsuitable soils, attempt to modify existing ground, enable cost effective foundation design, reduce the effects of contaminated soils and ensure sustainability in construction projects using soil improvement technique (Naema, 2015)

The selection of the treatment method mainly depends on the type of the structure. Alternative treatment options recommended for highway sub grade may not be suitable for treating collapsible ground below bridge or building foundations. For example, solutions like grouting, or pre wetting and surcharging are more relevant to foundations since foundations occupy a small amount of area. In addition, the solution of precluding the presence of water, like using geo textile membranes, is almost impractical, as discussed on ERA's 2013 Geotechnical Design Manual.

Several soil improvement techniques can be used to increase the bearing capacity and reduce settlement of shallow foundation. Examples of these techniques are soil replacement, prewetting,dynamic compaction and chemical stabilization.

4.3.1. Dynamic compaction of soil at natural moisture content

Collapsible soil may be moistened and recompacted by heavy rollers or heavy tamping. Spread footings and mats may be constructed over the compacted soil. Heavy tamping consists primarily of dropping a heavy weight repeatedly on the ground. The height of the drop can vary from 8 to 30 m. The stress waves generated by the dropping weight help in the densification of the soil(Braja, 1983).

4.3.2. Pre-wetting

Prewetting means flooding or wetting the soil which is expected to exhibit collapse upon saturation before the structure is built, so that soil collapse will be minimized after the structure is built. Prewetting was found to be one of the easiest and least costly treatments but it proved to be completely ineffective in reducing collapse potential.

4.3.3. Dynamic compaction after prewetting the soil

Wetting and dynamic compaction can be used to control or limit collapsibility potential by increasing the moisture content of soil before dynamic compaction to increase the compaction efficiency. Rollins and Rogers(1994) investigated the influence of prewetting the soil and dynamic compaction on the collapse settlement, the collapse settlement in this method was about 4% of the value in case of no treatment. In comparison with compaction at the natural moisture content, the densification over the entire area was more uniform.

4.3.4. chemical stabilization

Chemical stabilization is the modification of the soil properties to improve its engineering performance. The most commonly methods of chemical stabilization are lime stabilization and cement stabilization, other chemical additives that the soil can chemically react with are treatment with salts (ammonium sulfates $(NH4)2SO4$ and potassium chloride KCL). Also fly ash can be added to the soil-lime and soil-cement in order to enhance the properties of the stabilized soil. The increase in lime or cement content results in decreasing in the compression and no collapse potential had been occurred for treated soil.Foundation trenches can be also flooded with solutions of sodium silicate and calcium chloride to stabilize the soil chemically. The soil will behave like soft sandstone and resist collapse upon saturation. This method is successful only if the solutions can penetrate to the desired depth; thus, it is most applicable to fine sand deposits(Braja, 1983).

4.3.5. Soil replacement

Soil replacement is one of the most familiar techniques in dealing with collapsible soils. It is implemented by removing the weak soil and replacing it with a better compacted soil. Unfortunately, the determination of replacement layer thickness is questionable because it is based on experience (Gabr, 2012)

Rollins and Rogers (1994) reported that this method offers several advantages, the first that it decreases the amount of collapsible material in the zone of significant stress, the second that it increases the depth to which water must percolate before reaching collapsible material, and the third that it decreases the induced stress to which the collapsible soil is subjected. This reduction in the induced stress many keep the stress below the critical value necessary to produce significant collapse settlement. In addition, this technique minimizes the differential settlement under the footing.

Naema, (2015) proved that the improvement of collapsible soils by sand/crushed stone replacement is possible to control/mitigate their risk potentials against sudden settlement when exposed to water. She also found that the soil replacement with compacted cohesion less soil reduces the foundation settlement by about 50% and increases bearing capacity by about 100%. The subgrade should be improved with compaction and pre-wetting before placing the top compacted sand replacement to obtain good results of higher bearing capacity, and low and uniform settlement. The most effective thickness for the compacted sand layer, within the tested range, was found to be equal to the plate width.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1. Conclusion

Based on the study and results of the investigation, the following conclusions are

Drawn:

- The soil sample is typically silty sand with substantial of clay and the soil passing through a No. 200 sieve which is not suitable for sub-grade material. The soil also has low plasticity index less than 10%, low moisture content and low dry density.

- The soil Classified as ML group (inorganic silts, fine sand with low plasticity) according to the Unified soil Classification System, USCS and as an A-4 material (silty soil) which is fair to poor rating in terms of general rating as sub-grade material according to AASHTO classification.

- Generally, the soil along the study area are non plastic or low plastic range and they are cohensionless. Therefore, the non-cohesiveness in the area accounts for the soil to collapse.

- Using dry density values from field tests and Atterberg limits it can be observed that for all test pits, the soils fall under the “potentially metastable” portion of the plot suggested by Bell and Bruyn (1997)

- The structure of the soil has meta stable texture as the bonds between the grains breakdown when the soil is wet. Hence, the collapse process represents a rearrangement of soil particles into a denser state of packing; also the soil has an open, partially unstable and unsaturated fabric.

- The compaction tests result shows that the optimum moisture content (OMC) ranges from 24.98% to 27.40%, while the maximum dry density ranges from 1.513g/cm3 to 1.58g/cm3 .The maximum dry density values (MDD) are generally low because the soil is loose and thus susceptible to collapse and The low values of the dry density indicate that the natural deposits are loose and account for the high void ratio and Therefore, the low maximum dry density of the soils in the area account for the formation of soil to collapse.

- As we mentioned that this soil is one of most common and has a high existence in most of the places in the world, in contrast we have to be careful from constructing roads or structures on this type of soil.

- They have high void ratio and low density. This was described as open potentially unstable structure, which allows the soil readily undergo particle re-arrangement and volume reduction by self-weight and/loading upon wetting.

5.2. Recommendation

From the study it is recommended that:-

- Further studies have to be made by conducting full engineering tests (including One Dimensional Odometer Test and CBR test)on the problematic soils under consideration in order to have full insight on their properties.

- Study on mechanism of collapse and alternative treatment methods that are commonly used to improve the performance of the soil.

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APPENDICES

Appendix A: Field and Laboratory Test Results

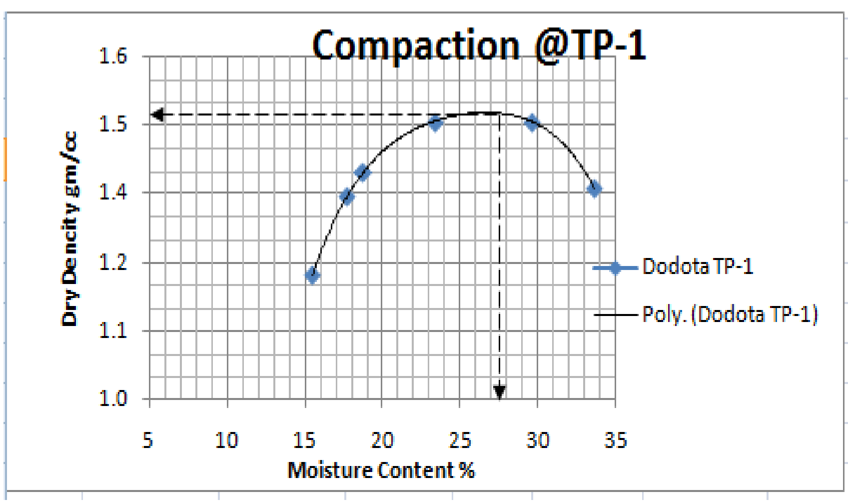
Soil Testing Laboratory

Standard Compaction Test

Analyst Name -
Sample Description - ***TP - 1***
Test Method ________________
Test Date - 25/07/2014
Volume of the Mold - 944 cm3
Mass of the mold with Baseplate 3755

Trial No.	Mass of Compacted Soil + Mold (g)	Mass of compacted soil (g)	Wet unit weight (kn/m3)	Moisture Content Detrmination: Can No.	Mass of wet soil + can (g)	Mass of dry soil + Can (g)	Mass of water (g)	Mass of Can (g)	Mass of dry soil (g)	Moisture Content (%)	Average Moisture content	Dry unit weight(Kn/m3)
1	2	3	4	5	6	7	8	9	10	11	12	13
1st 100ml	5085	1330	1.4089	A1	61.5	55.13	6.37	19.16	35.97	17.709202	15.528	1.220
				A2	66.96	60.73	6.23	19.17	41.56	14.990375		
				A3	71.2	64.91	6.29	19.61	45.3	13.88521		
2nd 100ml	5260	1505	1.59428	A4	71.45	63.8	7.65	19.29	44.51	17.187149	17.757	1.354
				A5	88.22	78	10.22	19.56	58.44	17.488022		
				A6	80.61	71	9.61	19.32	51.68	18.595201		
3rd 100ml	5323	1568	1.66102	A7	83.3	73.16	10.14	18.69	54.47	18.615752	18.775	1.398
				A8	82.46	72.47	9.99	19.36	53.11	18.810017		
				A9	93.26	81.53	11.73	19.46	62.07	18.898018		
4th 100ml	5486	1731	1.83369	A10	80.21	68.67	11.54	19.24	49.43	23.346146	23.363	1.486
				A11	79.6	68.28	11.32	19.39	48.89	23.154019		
				A12	98.9	83.69	15.21	19.21	64.48	23.58871		
5th 100ml	5573	1818	1.92585	A13	94.76	77.81	16.95	19.34	58.47	28.989225	29.650	1.485
				A14	95.11	77.69	17.42	19.71	57.98	30.044843		
				A15	81.87	67.42	14.45	19.12	48.3	29.917184		
6th 100ml	5483	1728	1.83051	A16	93.1	74.08	19.02	19.15	54.93	34.625887	33.623	1.370
				A17	91.75	73.45	18.3	19.44	54.01	33.882614		
				A18	82.12	66.82	15.3	19.54	47.28	32.360406		

Maximum Dry Density (MDD):		
MDD =	1.5125	gm/cc
Optimum Moisture Content (OMC) :		
OMC =	27.1	%

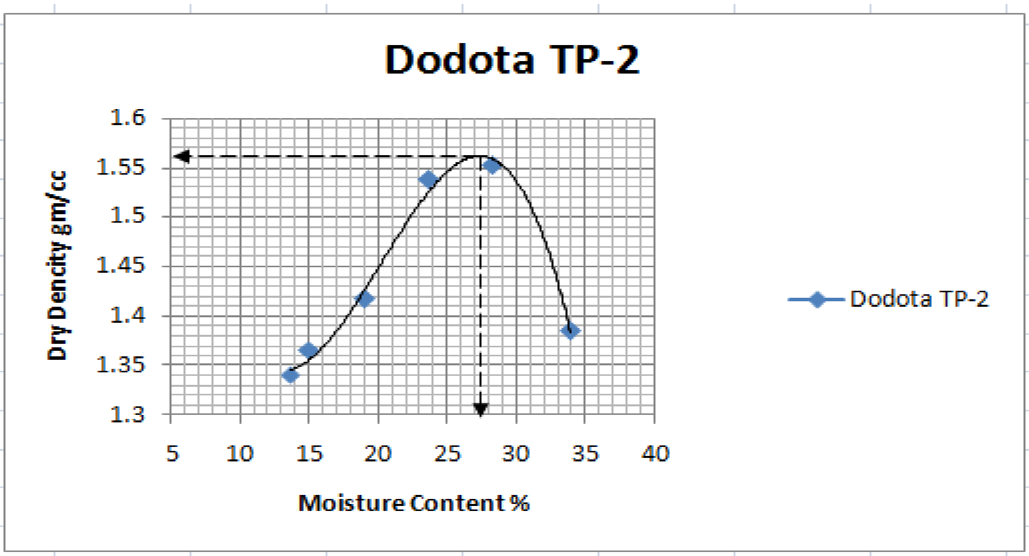


Soil Testing Laboratory
Standard Compaction Test

Analyst Name -
Sample Description - ***TP - 2***
Test Method ________________
Test Date - 25/7/2014
Volume of the Mold - 944 cm3
Mass of the mold with Baseplate - 3755 g

Trial No.	Mass of Compacted Soil + Mold (g)	Mass of compacted soil (g)	Wet unit weight (kn/m3)	Moisture Content Detrmination								
				Can No.	Mass of wet soil + can (g)	Mass of dry soil + Can (g)	Mass of water (g)	Mass of Can (g)	Mass of dry soil (g)	Moisture Content (%)	Average Moisture Content(100)	Dry unit weight(Kn/m3)
1	2	3	4	5	6	7	8	9	10	11	12	13
1st 100ml	5193	1438	1.52331	A1	68.87	62.32	6.55	19.44	42.88	15.2752	13.5742	1.34124
				A2	65.12	59.12	6	19.39	39.73	15.1019		
				A3	73.71	68.62	5.09	19.42	49.2	10.3455		
2nd 100ml	5237	1482	1.56992	A4	65.21	59.25	5.96	19.58	39.67	15.0239	14.8984	1.36635
				A5	63.74	58.05	5.69	19.39	38.66	14.7181		
				A6	73.1	66.09	7.01	19.21	46.88	14.9531		
3rd 100ml	5348	1593	1.6875	A7	74.34	73.63	0.71	19.1	54.53	1.30204	19.0663	1.41728
				A8	87.11	76.9	10.21	19.03	57.87	17.643		
				A9	83.44	65.74	17.7	19.47	46.27	38.2537		
4th 100ml	5552	1797	1.9036	A10	96.7	81.58	15.12	19.33	62.25	24.2892	23.6896	1.53902
				A11	77.65	66.61	11.04	19.38	47.23	23.375		
				A12	87.85	74.83	13.02	19.2	55.63	23.4046		
5th 100ml	5635	1880	1.99153	A13	90.9	75.38	15.52	19.4	55.98	27.7242	28.3041	1.55219
				A14	79.93	66.45	13.48	19.71	46.74	28.8404		
				A15	97.46	80.27	17.19	19.63	60.64	28.3476		
6th 100ml	5506	1751	1.85487	A16	85.56	68.52	17.04	19.28	49.24	34.606	33.8871	1.3854
				A17	95.78	76.53	19.25	19.48	57.05	33.7423		
				A18	84.58	68.29	16.29	19.39	48.9	33.3129		

Maximum Dry Density (MDD):		
MDD =	1.5583	gm/cc
Optimum Moisture Content (OMC) :		
OMC =	27.40	%

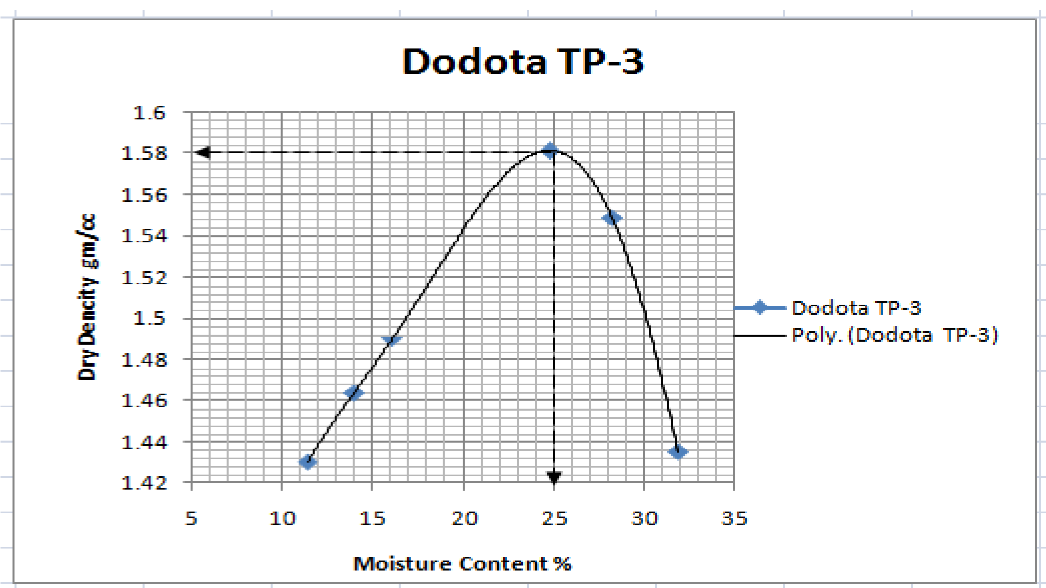


Soil Testing Laboratory
Standard Compaction Test

Analyst Name -
Sample Description - ***TP - 3***
Test Method ____________
Test Date - 26/7/2014
Volume of the Mold - 944 cm3
Mass of the mold with Baseplate - 3755 g

Trial No.	Mass of Compacted Soil + Mold (g)	Mass of compacted soil (g)	Wet unit weight (kn/m3)	Moisture Content Detrmination								Dry unit weight(Kn/m3)
				Can No.	Mass of wet soil + can (g)	Mass of dry soil + Can (g)	Mass of water (g)	Mass of Can (g)	Mass of dry soil (g)	Moisture Content (%)	Average Moisture Moisture	
1	2	3	4	5	6	7	8	9	10	11	12	13
1st 100ml	5259	1504	1.59322	A1	75.42	70.11	5.31	30.07	40.04	13.2617	11.3864	1.43035
				A2	82.22	77.17	5.05	29.6	47.57	10.6159		
				A3	85.19	80.04	5.15	29.95	50.09	10.2815		
2nd 100ml	5330	1575	1.66843	A4	82.44	75.46	6.98	30.01	45.45	15.3575	13.9591	1.46406
				A5	81.25	74.95	6.3	29.57	45.38	13.8828		
				A6	81.41	75.65	5.76	30.07	45.58	12.6371		
3rd 100ml	5388	1633	1.72987	A7	81.96	74.95	7.01	30	44.95	15.5951	16.0734	1.49033
				A8	82.03	74.54	7.49	29.66	44.88	16.6889		
				A9	88.96	80.89	8.07	30.25	50.64	15.936		
4th 100ml	5617	1862	1.97246	A10	90.46	79.2	11.26	29.49	49.71	22.6514	24.7616	1.58098
				A11	88.27	77.98	10.29	30.9	47.08	21.8564		
				A12	88.72	75.24	13.48	29.97	45.27	29.7769		
5th 100ml	5630	1875	1.98623	A13	95.24	81.04	14.2	29.76	51.28	27.6911	28.2596	1.5486
				A14	94.97	80.8	14.17	30.09	50.71	27.9432		
				A15	88.36	75.14	13.22	29.78	45.36	29.1446		
6th 100ml	5542	1787	1.89301	A16	93.05	77.67	15.38	30.02	47.65	32.277	31.9061	1.43512
				A17	75.2	64.35	10.85	30.06	34.29	31.6419		
				A18	99.93	83.07	16.86	30.05	53.02	31.7993		

Maximum Dry Density (MDD):			
MDD =	1.58	gm/cc	
Optimum Moisture Content (OMC) :			
OMC =	24.98	%	

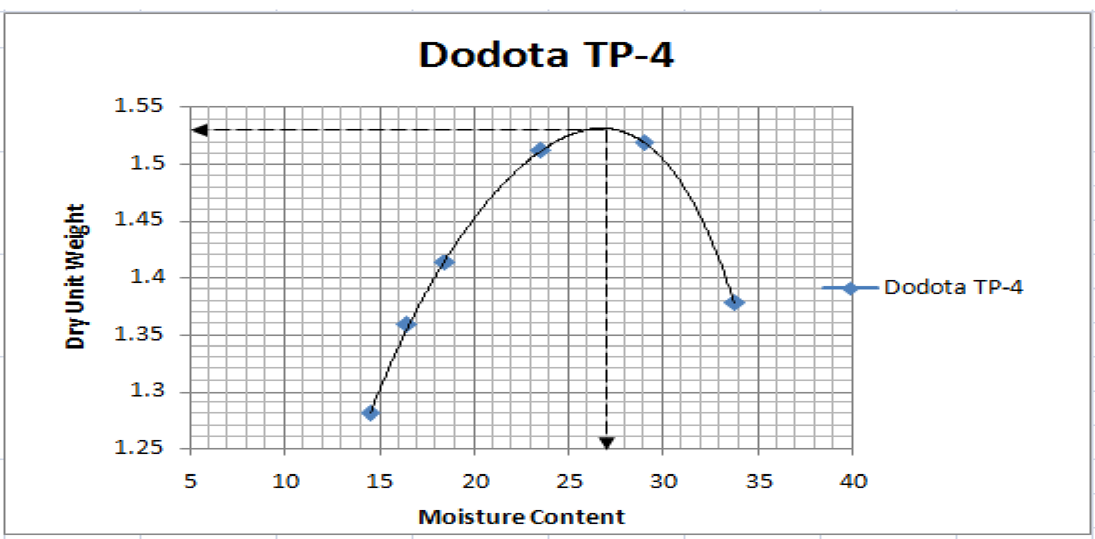


Soil Testing Laboratory
Standard Compaction Test

Analyst Name -
Sample Description - *TP - 4*
Test Method ________________
Test Date - 26/7/2014
Volume of the Mold 944 cm3
Mass of the mold with Baseplate 3755 g

Trial No.	Mass of Compacted Soil + Mold (g)	Mass of compacted soil (g)	Wet unit weight (kn/m3)	Moisture Content Detrmination								
				Can No.	Mass of wet soil + can (g)	Mass of dry soil + Can (g)	Mass of water (g)	Mass of Can (g)	Mass of dry soil (g)	Moisture Content (%)	Average Moisture Moisture	Dry unit weight(Kn/m3)
1	2	3	4	5	6	7	8	9	10	11	12	13
1st 100ml	5139	1384	1.4661	A1	65.185	58.725	6.46	19.3	39.425	16.3855	14.4909	1.28054
				A2	66.04	59.925	6.115	19.28	40.645	15.0449		
				A3	72.455	66.765	5.69	19.515	47.25	12.0423		
2nd 100ml	5249	1494	1.58263	A4	68.33	61.525	6.805	19.435	42.09	16.1677	16.4719	1.35881
				A5	75.98	68.025	7.955	19.475	48.55	16.3852		
				A6	76.855	68.545	8.31	19.265	49.28	16.8628		
3rd 100ml	5335	1580	1.67373	A7	78.82	73.395	5.425	18.895	54.5	9.95413	18.44	1.41314
				A8	84.785	74.685	10.1	19.195	55.49	18.2015		
				A9	88.35	73.635	14.715	19.465	54.17	27.1645		
4th 100ml	5519	1764	1.86864	A10	88.455	75.125	13.33	19.285	55.84	23.8718	23.5459	1.51251
				A11	78.625	67.445	11.18	19.385	48.06	23.2626		
				A12	93.375	79.26	14.115	19.205	60.055	23.5035		
5th 100ml	5604	1849	1.95869	A13	92.83	76.595	16.235	19.37	57.225	28.3705	28.9737	1.51867
				A14	87.52	72.07	15.45	19.71	52.36	29.5073		
				A15	89.665	73.845	15.82	19.375	54.47	29.0435		
6th 100ml	5495	1740	1.84322	A16	89.33	71.3	18.03	19.215	52.085	34.6165	33.7572	1.37803
				A17	93.765	74.99	18.775	19.46	55.53	33.8106		
				A18	83.35	67.555	15.795	19.465	48.09	32.8447		

Maximum Dry Density (MDD):		
MDD =	1.5375	gm/cc
Optimum Moisture Content (OMC) :		
OMC =	27.00	%



Atterberg Limit Determination

Project Asalla-Adama asphalt Road Job No.

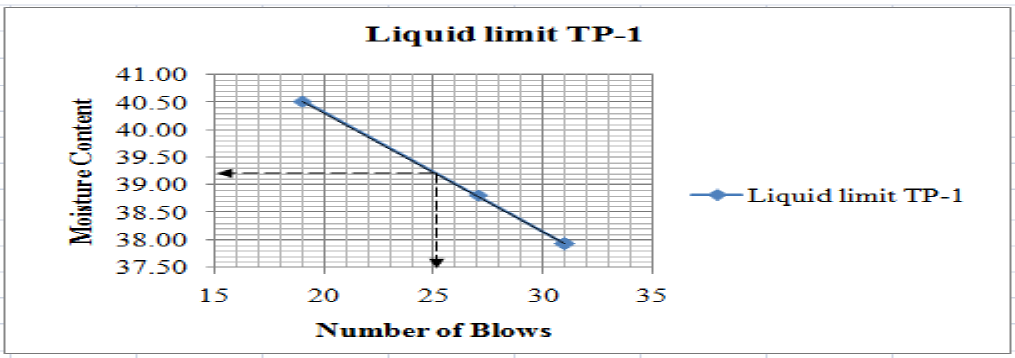
Location of Project Dodota Area Boring no, ________ Sample N(TP

Description of Soil A Silty Soil

Depth of Sample Date11/07/2014

Tested by:-*Abdisa Samuel*

Liquid Limit Determination			
Test Sample For Pit - 1 @1.5 m depth			
Can No.	A	B	C
Mass of wet soil + Can(M2)	41.25	41.17	39.87
Mass of dry soil + Can (M3)	38.3	38.12	37.03
Mass of Can (M_1)	30.52	30.26	30.02
Mass of dry soil (M2-M3)	7.78	7.86	7.01
Mass of moisture	2.95	3.05	2.84
Water content W%	37.92	38.80	40.51
No. of blows N	31	27	19
Penetration D.mm			
Moisture Content from Graph		39.30	

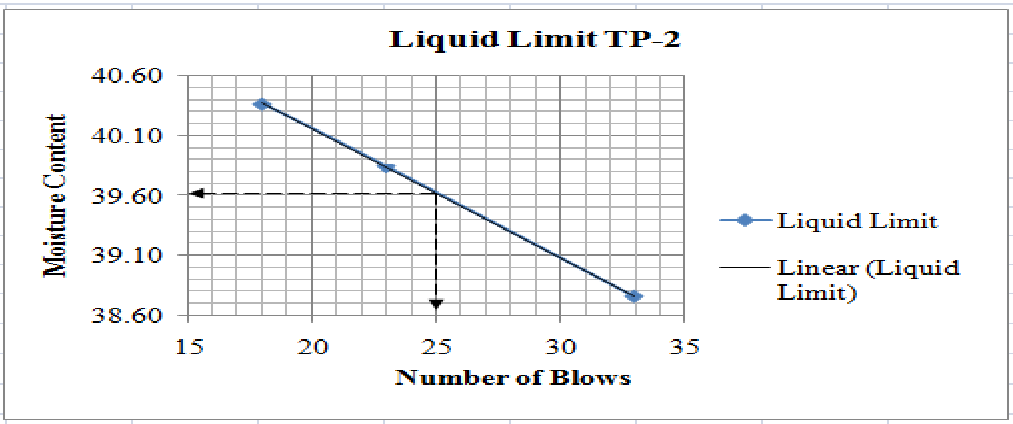


Plastic Limit Determination

Lab - *Pit - 1 @1.5 m depth*

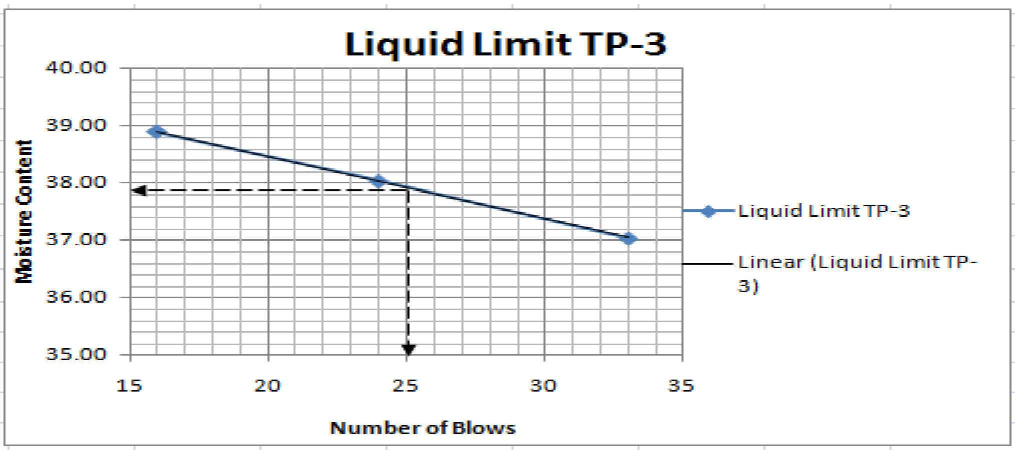
Can No.	A	B	C
Mass of wet soil + Can (M_2)	35.03	33.48	36.5
Mass of dry soil + Can (M_3)	34.03	32.72	34.91
Mass of Can (M_1)	30.18	30.36	29.76
Mass of dry soil (M_6)	3.85	2.36	5.15
Mass of moisture (M_5)	1	0.76	1.59
Water content W% = Wp	25.974	32.2034	30.8738
PLASTIC LIMIT		29.68	
Plasticity Index (PI = W_L - W_P)		*9.62*	

Liquid Limit Determination			
Test Sample for Pit - 2 @1.5 m depth			
Can No.	A	B	C
Mass of wet soil + Can	39.35	40.7	40.19
Mass of dry soil + Can	36.42	37.66	37.32
Mass of Can	28.86	30.03	30.21
Mass of dry soil	7.56	7.63	7.11
Mass of moisture	2.93	3.04	2.87
Water content W%	38.76	39.84	40.37
No. of blows N	33	23	18
Penetration D.mm			
Moisture Content from Graph		39.65	



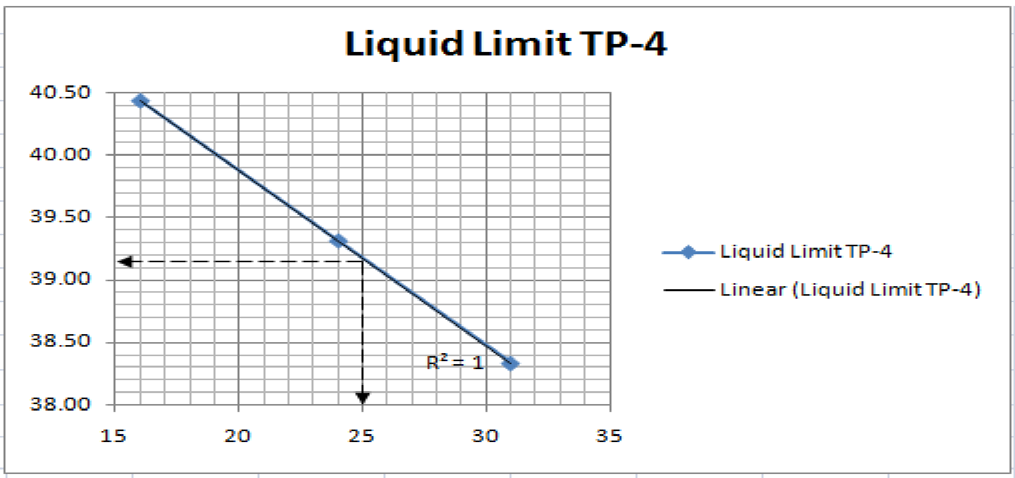
Plastic Limit Determination			
Lab - *Pit - 2 @1.5 m depth*			
Can No.	A	B	C
Mass of wet soil + Can (M_2)	35.67	33.71	35.14
Mass of dry soil + Can (M_3)	34.27	32.87	33.93
Mass of Can (M_1)	29.79	30.08	29.98
Mass of dry soil (M_6)	4.48	2.79	3.95
Mass of moisture (M_5)	1.4	0.84	1.21
Water content W% = Wp	31.25	30.1075	30.6329
PLASTIC LIMIT		30.66	
Plasticity Index (PI = W_L - W_P)		*8.99*	

Liquid Limit Determination			
Test Sample For Pit - 3@ 1.5 m depth			
Can No.	A	B	C
Mass of wet soil + Can	43.06	42.04	43.16
Mass of dry soil + Can	39.6	38.75	39.55
Mass of Can	30.26	30.1	30.27
Mass of dry soil	9.34	8.65	9.28
Mass of moisture	3.46	3.29	3.61
Water content W%	37.04	38.03	38.90
No. of blows N	33	24	16
Penetration D.mm			
Moisture Content from Graph		37.90	



Plastic Limit Determination			
Lab - *Pit - 3 @1.5 m depth*			
Can No.	A	B	C
Mass of wet soil + Can (M_2)	35.46	36.49	34.91
Mass of dry soil + Can (M_3)	34.3	35.1	33.67
Mass of Can (M_1)	30.25	29.85	30.1
Mass of dry soil (M_6)	4.05	5.25	3.57
Mass of moisture (M_5)	1.16	1.39	1.24
Water content W% = Wp	28.642	26.4762	34.7339
PLASTIC LIMIT		29.951	
Plasticity Index (PI = W_L - W_P)		*7.95*	

Liquid Limit Determination			
Test Sample for Pit - 4 @1.6 m depth			
Can No.	A	B	C
Mass of wet soil + Can	40.3	40.94	40.03
Mass of dry soil + Can	37.36	37.89	37.18
Mass of Can	29.69	30.15	30.12
Mass of dry soil	7.67	7.75	7.06
Mass of moisture	2.94	3.05	2.86
Water content W%	38.33	39.32	40.44
No. of blows N	31	24	16
Penetration D.mm			
Moisture Content from Graph		39.20	



Plastic Limit Determination

Lab - *Pit - 4 @1.6m depth*

Can No.	A	B	C
Mass of wet soil + Can (M_2)	35.35	33.60	35.82
Mass of dry soil + Can (M_3)	34.15	32.80	34.42
Mass of Can (M_1)	30.00	30.22	29.87
Mass of dry soil (M_6)	4.15	2.575	4.55
Mass of moisture (M_5)	1.2	0.8	1.4
Water content W% = Wp	28.9157	31.068	30.7692
PLASTIC LIMIT		30.25	
Plasticity Index ($PI = W_L - W_P$)		*8.95*	

Particl Size Analysis

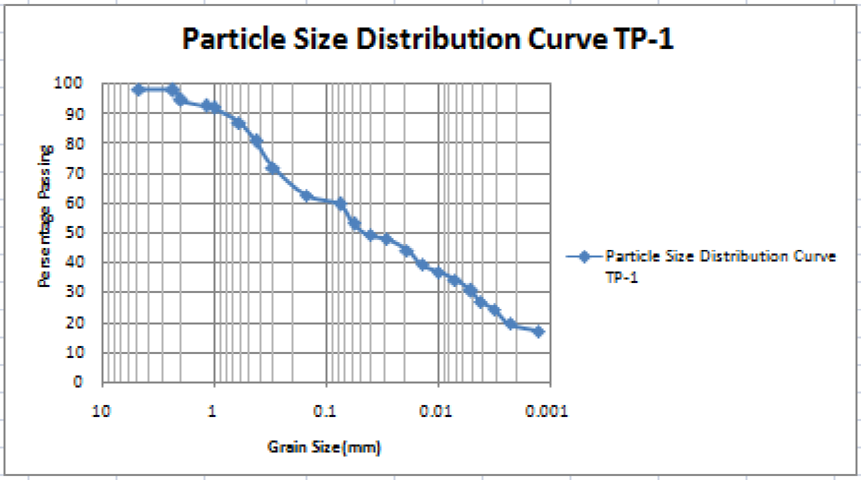
Sieve Analysis Data & Calculations Sheet

Source________________		Mass of soil defore	500
Sample no. *TP 1*		Mass of soil after	202.106
Test by *Abdisa Samuel*		Material type ________________	
wet method	×	Dry method ________________	

Number		1	2	3	4	5	6	7
Sieve ASTM	Sieve siza(mm)	Mass of sieve + pan	Massof sieve + soil+ pan	Individual mass Retained IMR Col2-Col1	Cumulative Mass Retained CMR	Cumulative Percent Retained CPR=(CMR/Mr)*100	Percent Pass or % Finer (100-CPR)	Remark
		1	2	3	4	5	6	7
3"	75							For CMR IMR1
2"	50							IMR1 + IMR3
1-1/2"	37							IMR1 + IMR3 + IMR3
1"	25							
3/4"	19							`
1/2"	12							
3/8"	9.5							
-	5							
No. 4	4.75	440	449	9	9	1.800	98.200	
No.8	2.36	407	409	2	11	2.200	97.800	
No.10	2	384	400	16	27	5.400	94.600	
No.16	1.18	358	368	10	37	7.400	92.600	
No.18	1	349	353	3.69	40.690	8.138	91.862	
No.30	0.6	330	355	25	65.690	13.138	86.862	
No.40	0.425	310	341	31.42	97.106	19.421	80.579	
No.50	0.3	295	339	44	141.106	28.221	71.779	
No.100	0.15	278	325	47	188.106	37.621	62.379	
No.200	0.075	274	288	14	202.106	40.421	59.579	
Pan		253	254	297.894	500.000	100.000	0.000	
Total Weight(g) of Sample			500.000					
Loss(g)								

Date	Time of Reading	Time (min)	Hydrometer Reading R_a	Corrected Hydrometer reading R_c	% Finer	Adjuste percent P_A	Effective Depth L(Cm)	?(L/T)	K from Table	D Diameter of particle in mm
22/08/2014		0.5	41	40.70	80.85	48.170	9.20	4.29	0.013	0.056
		1	40	39.70	78.86	46.987	9.70	3.11	0.013	0.041
		2	39	38.70	76.88	45.803	9.90	2.22	0.013	0.029
		5	36	35.70	70.92	42.253	10.40	1.44	0.013	0.019
		10	32	31.70	62.97	37.518	10.90	1.04	0.013	0.014
		20	30	29.70	59.00	35.151	11.20	0.75	0.013	0.010
		40	28	27.70	55.03	32.784	11.70	0.54	0.013	0.007
		80	25	24.70	49.07	29.234	12.20	0.39	0.013	0.005
		120	22	21.70	43.11	25.683	12.50	0.32	0.013	0.004
		240	20	19.70	39.13	23.316	13.00	0.23	0.014	0.003
		480	16	15.70	31.19	18.582	13.70	0.17	0.014	0.002
		1440	14	13.70	27.22	16.215	14.00	0.10	0.014	0.001

S.Size	%pass
4.75	98
2.36	98
2	95
1.18	93
1	92
0.6	87
0.425	81
0.3	72
0.15	62
0.075	60
0.0565	53
0.0408	49
0.0293	48
0.0190	44
0.0137	39
0.0099	37
0.0071	34
0.0051	31
0.0042	27
0.0031	24
0.0022	19
0.0013	17



Grading					
ASTM SIEVE No.	10	40	200	AASHTO Classification (M145-91)	USCS classification
Diameter f mm	2.00	0.425	0.075	*A-4*	*ML*
% Passing	*94.60*	*80.58*	*59.579*	*Silty Soil*	*Low plasticity*

GROUP INDEX = (F - 35) [0.2 + 0.005 (LL – 40)] + 0.01 (F – 15) (PI – 10)

where: F = Percentage passing 0.075 mm (No. 200) sieve.

LL = Liquid Limit & PI = Plasticity Index

GI 4.66

Particl Size Analysis

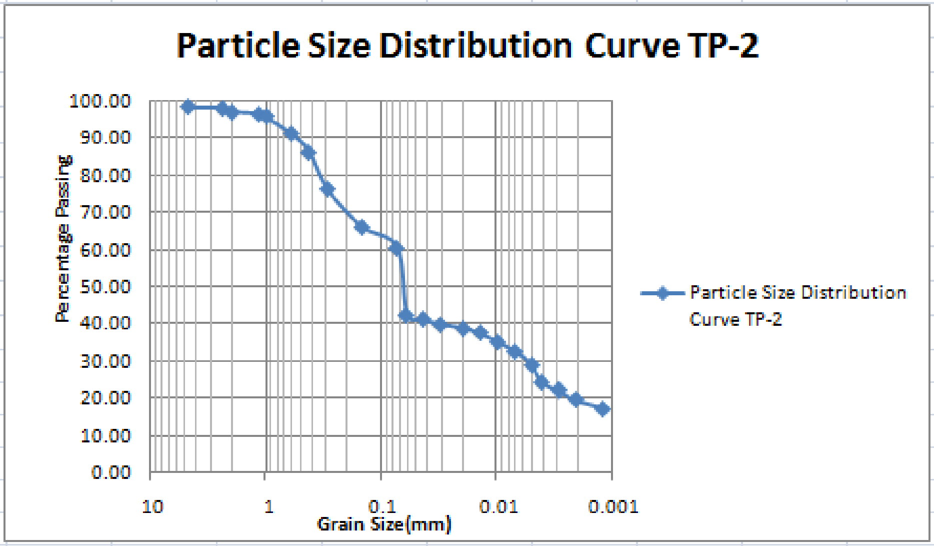
Sieve Analysis Data & Calculations Sheet

Source ______________________		Mass of soil defore	500
Sample no. TP-2		Mass of soil after	198.00
Test by Abdisa Samuel		Material type ____________________	
wet method	×	Dry method ____________________	

Number		1	2	3	4	5	6	7
Sieve ASTM	Sieve siza(mm)	Mass of sieve + pan	Massof sieve + soil+ pan	Individual mass Retained IMR Col2-	Cumulative Mass Retained CMR	Cumulative Percent Retained CPR=(CMR/	Percent Pass or % Finer (100-CPR)	Remark
		1	2	3	4	5	6	7
3"	75							For CMR IMR1
2"	50							IMR1 + IMR3
1-1/2"	37							IMR1 + IMR3 + IMR3
1"	25							
3/4"	19							
1/2"	12							
3/8"	9.5							
-	5							
No. 4	4.75	440	449	9	9	1.8000	98.2000	
No.8	2.36	407	408	1	10	2.0000	98.0000	
No.10	2	384	389	5	15	3.0000	97.0000	
No.16	1.18	358	361	3	18	3.6000	96.4000	
No.18	1	348	351	3.00	21.00	4.2000	95.8000	
No.30	0.6	330	354	24	45.00	9.0000	91.0000	
No.40	0.425	310	334	24.00	69.00	13.8000	86.2000	
No.50	0.3	295	344	49	118.00	23.6000	76.4000	
No.100	0.15	278	329	51	169.00	33.8000	66.2000	
No.200	0.075	274	303	29	198.00	39.6000	60.4000	
Pan		253	254	302.00	500.00	100.0000	0.0000	
Total Weight(g) of Sample			500.00					
Loss(g)								

Date	Time of Reading	Time (min)	Hydrometer Reading R_a	Corrected Hydrometer reading R_c	% Finer	Adjuste percent P_A	Effective Depth L(Cm)	?(L/T)	K from Table	D Diameter of particle in mm
22/08/2014		0.5	34	35.30	70.12	42.355	10.70	4.63	0.013	0.059
		1	33	34.30	68.14	41.155	10.90	3.30	0.013	0.042
		2	32	33.30	66.15	39.955	11.10	2.36	0.013	0.030
		5	31	32.30	64.16	38.755	11.20	1.50	0.013	0.019
		10	30	31.30	62.18	37.556	11.40	1.07	0.013	0.014
		20	28	29.30	58.21	35.156	11.70	0.76	0.013	0.010
		40	26	27.30	54.23	32.756	12.00	0.55	0.013	0.007
		80	23	24.30	48.27	29.157	12.50	0.40	0.013	0.005
		120	19	20.30	40.33	24.357	13.20	0.33	0.013	0.004
		240	17	18.30	36.35	21.957	13.50	0.24	0.013	0.003
		480	15	16.30	32.38	19.558	13.80	0.17	0.013	0.002
		1440	13	14.30	28.41	17.158	14.20	0.10	0.013	0.001

S.Size	%pass
4.75	98.20
2.36	98.00
2	97.00
1.18	96.40
1	95.80
0.6	91.00
0.425	86.20
0.3	76.40
0.15	66.20
0.075	60.40
0.062	42.36
0.044	41.16
0.032	39.96
0.020	38.76
0.014	37.56
0.010	35.16
0.007	32.76
0.005	29.16
0.004	24.36
0.003	21.96
0.002	19.56
0.001	17.16



Grading					
ASTM SIEVE No.	10	40	200	AASHTO Classification (M145-91)	USCS classification
Diameter f mm	2.00	0.425	0.075	*A-4*	*ML*
% Passing	*97.00*	*86.20*	*60.400*	*Silty Soil*	*Low plasticity*

GROUP INDEX = (F - 35) [0.2 + 0.005 (LL – 40)] + 0.01 (F – 15) (PI – 10)

where: F = Percentage passing 0.075 mm (No. 200) sieve.

LL = Liquid Limit & PI = Plasticity Index

GI 4.58

Particl Size Analysis

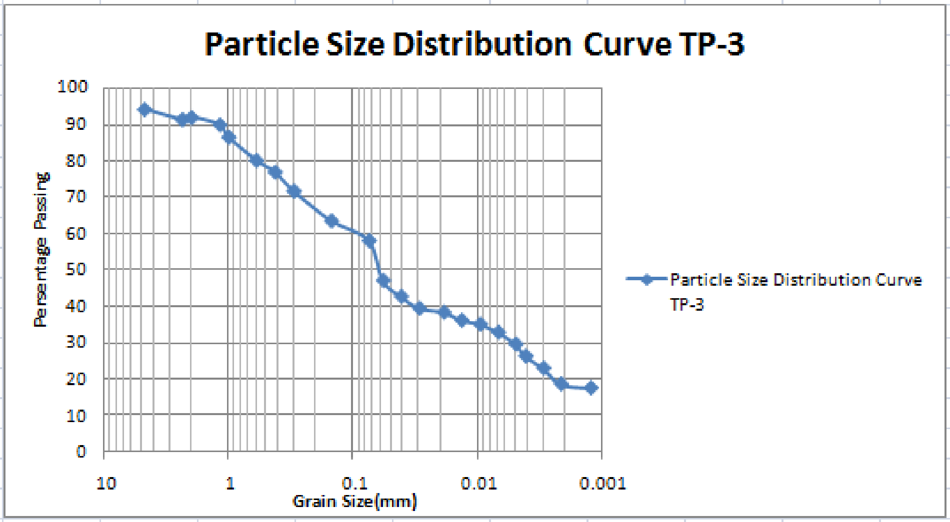
Sieve Analysis Data & Calculations Sheet

Source________________		Mass of soil defore	500
Sample no.TP-3		Mass of soil after	210
Test by Abdisa Samuel		Material type ________________	
wet method	×	Dry method ________________	

Number		1	2	3	4	5	6	7
Sieve ASTM	Sieve siza(mm)	Mass of sieve + pan	Massof sieve + soil+ pan	Individual mass Retained IMR Col2-Col1	Cumulative Mass Retained CMR	Cumulative Percent Retained CPR=(CMR/Mr)* 100	Percent Pass or % Finer (100-CPR)	Remark
3"	75							For CMR IMR1
2"	50							IMR1 + IMR3
1-1/2"	37							IMR1 + IMR3 + IMR3
1"	25							
3/4"	19							
1/2"	12							
3/8"	9.5							
-	5							
No. 4	4.75	440	470	30	30	6.0	94.00	
No.8	2.36	407	421	14	44	8.8	91.20	
No.10	2	384	381	-3	41	8.2	91.80	
No.16	1.18	358	368	10	51	10.2	89.80	
No.18	1	349	366	17	68	13.6	86.40	
No.30	0.6	330	362	32	100	20.0	80.00	
No.40	0.425	309	325	16	116	23.2	76.80	
No.50	0.3	295	321	26	142	28.4	71.60	
No.100	0.15	278	319	41	183	36.6	63.40	
No.200	0.075	274	301	27	210	42.0	58.00	
Pan		253	254	290.00	500	100.0	0.00	
Total Weight(g) of Sample			500					
Loss(g)								

Date	Time of Reading	Time (min)	Hydrometer Reading R_a	Corrected Hydrometer reading R_c	% Finer	Adjuste percent P_A	Effective Depth L(Cm)	?(L/T)	K from Table	D Diameter of particle in mm
22/08/2014		0.5	42.0	43.00	85.23	49.434	9.40	4.34	0.013	0.056
		1	38.0	39.00	77.30	44.836	10.10	3.18	0.013	0.041
		2	35.0	36.00	71.36	41.387	10.60	2.30	0.013	0.030
		5	34.0	35.00	69.37	40.237	10.70	1.46	0.013	0.019
		10	32.0	33.00	65.41	37.938	11.10	1.05	0.013	0.014
		20	31.0	32.00	63.43	36.788	11.20	0.75	0.013	0.010
		40	29.0	30.00	59.46	34.489	11.50	0.54	0.013	0.007
		80	26.0	27.00	53.52	31.040	12.00	0.39	0.013	0.005
		120	23.0	24.00	47.57	27.591	12.50	0.32	0.013	0.004
		240	20.0	21.00	41.62	24.142	12.90	0.23	0.013	0.003
		480	16.0	17.00	33.70	19.544	13.70	0.17	0.013	0.002
		1440	15.0	16.00	31.71	18.394	13.80	0.10	0.013	0.001

S.Size	%Pass
4.75	94
2.36	91
2	92
1.18	90
1	86
0.6	80
0.425	77
0.3	72
0.15	63
0.075	58
0.058	47
0.041	43
0.030	39
0.019	38
0.014	36
0.010	35
0.007	33
0.005	30
0.004	26
0.003	23
0.002	19
0.001	18



Grading					
ASTM SIEVE No.	10	40	200	AASHTO Classification (M145-91)	USCS classification
Diameter f mm	2.00	0.425	0.075	*A-4*	*ML*
% Passing	*91.80*	*76.80*	*58*	*Silty Soil*	*Low plasticity*

GROUP INDEX = (F - 35) [0.2 + 0.005 (LL – 40)] + 0.01 (F – 15) (PI – 10)

where: F = Percentage passing 0.075 mm (No. 200) sieve.

LL = Liquid Limit & PI = Plasticity Index

GI 3.48

Particl Size Analysis

Sieve Analysis Data & Calculations Sheet

Source___________________ **Mass of soil before** 500

Sample no. *TP - 4* **Mass of soil after** 199.61

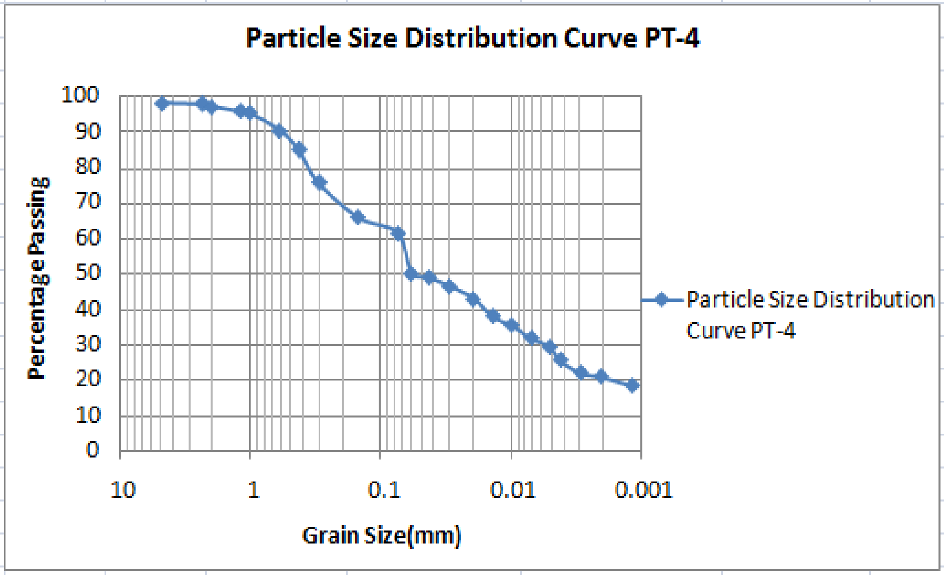
Test by *Abdisa Samuel* **Material type** ___________________

wet method × **Dry method** ___________________

Number		1	2	3	4	5	6	7
Sieve ASTM	Sieve siza(mm)	Mass of sieve + pan	Massof sieve + soil+ pan	Individual mass	Cumulative Mass	Cumulative Percent	Percent Pass or % Finer	Remark
		1	2	3	4	5	6	7
3"	75							
2"	50							
1-1/2"	37							
1"	25							
3/4"	19							
1/2"	12							
3/8"	9.5							
-	5							
No. 4	4.75	440	449	9	9	1.800	98.200	
No.8	2.36	407	408.5	1.5	10.5	2.100	97.900	
No.10	2	384	394.5	10.5	21	4.200	95.800	
No.16	1.18	358	364.5	6.5	27.5	5.500	94.500	
No.18	1	349.31	352.00	2.69	30.19	6.04	93.96	
No.30	0.6	330	354.5	24.5	54.690	10.938	89.062	
No.40	0.425	309.58	337.50	27.92	82.61	16.52	83.48	
No.50	0.3	295	341.5	46.5	129.106	25.821	74.179	
No.100	0.15	278	327	49	178.106	35.621	64.379	
No.200	0.075	274	295.5	21.5	199.606	39.921	60.079	
Pan		253	254	300.39	500.000	100.000	0.000	
Total Weight(g) of Sample			500.000					
Loss(g)								

Date	Time of Reading	Time (min)	Hydrometer Reading R_a	Corrected Hydrometer reading R_c	% Finer	Adjuste percent P_A	Effective Depth L(Cm)	?(L/T)	K from Table	D Diameter of particle in mm
23/08/2014		0.5	40.0	41.00	81.45	48.933	10.40	4.56	0.013	0.058
		1	39.0	40.00	79.46	47.739	10.90	3.30	0.013	0.042
		2	37.0	38.00	75.49	45.352	11.20	2.37	0.013	0.030
		5	34.0	35.00	69.53	41.772	11.50	1.52	0.013	0.019
		10	30.0	31.00	61.58	36.998	11.70	1.08	0.013	0.014
		20	28.0	29.00	57.61	34.611	12.00	0.77	0.013	0.010
		40	25.0	26.00	51.65	31.030	12.20	0.55	0.013	0.007
		80	23.0	24.00	47.68	28.643	12.50	0.40	0.013	0.005
		120	20.0	21.00	41.72	25.063	12.70	0.33	0.013	0.004
		240	17.0	18.00	35.76	21.483	12.90	0.23	0.013	0.003
		480	16.0	17.00	33.77	20.289	13.00	0.16	0.013	0.002
		1440	14.0	15.00	29.80	17.902	12.90	0.09	0.013	0.001

S.Size	%Pass
4.75	98
2.36	98
2	97
1.18	96
1	95
0.6	90
0.425	85
0.3	75
0.15	66
0.075	61
0.058	50
0.042	49
0.030	46
0.019	43
0.014	38
0.010	35
0.007	32
0.005	29
0.004	26
0.003	22
0.002	21
0.001	18



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Grading					
ASTM SIEVE No.	10	40	200	AASHTO Classification (M145-91)	USCS classification
Diameter f mm	2.00	0.425	0.075	*A-4*	*ML*
% Passing	*95.80*	*83.48*	*60.08*	*Silty Soil*	*Low plasticity*

GROUP INDEX = (F - 35) [0.2 + 0.005 (LL – 40)] + 0.01 (F – 15) (PI – 10)

where: F = Percentage passing 0.075 mm (No. 200) sieve.

LL = Liquid Limit & PI = Plasticity Index

GI 4.44

FIELD DENCITY TEST USING SAND REPLACEMENT METHOD			
TEST STANDARD USED:- AASHTO T-191			
Test Type Field Dencity	Calibrating Cylinder		
Sample No. TP-1,TP-2,TP-3 & TP-4	Height(cm)		11.2
Tested By Abdisa Samuel	Diameter(cm)		9.8
Date 09/07/2014	Volume(cm3)		844.38
	Weight (g)		82
Trial Sample	Trial 1	Trial 2	Trial 3
W (Sand +calibrating cylinder,in gram)	1150	1156	1153
Avg W(Sand + calibrating cylinder, in gram)		1153	
Ws(Sand in calibrating Cylinder=Avg. W - Weight of Cylinder,in gram)		1071	
Bulk Dencity of Sand ?s = Ws/V (g/cm3)		1.268	
Wa(sand in dencity jar+calibrating cylinder before pouring in gram)	8541	8345	8219
Wb (sand in dencity jar +calibrating cylinder after pouring in gram)	8105	7905	7778
W1(Sand in cone of pouring cylinder, W1=Wa-Wb in gram)	436	440	441
Avg W1 (Sand in cone of pouring cylinder, in gram)		439	

FIELD DRY DENCITY CALCULATION

Sample	TP-1	TP-2	TP-3	TP-4
Ww (Weight of wet soil from Hole in gram)	2701	2477	2501	2450
W2 (Weight of cylinder + sand before pouring in gram)	8761	8993	8820	8658
W3(Weight of cylinder +sand after pouring in gram)	5498	5879	5623	5413
W4 (Weight of sand in hole W2-W3-W1 in gram)	2824	2675	2758	2806
Volume of Hole Vh =W4/?s in cm3	2226.46	2108.99	2174.43	2212.27
Bulk Dencity, ?wet = (Ww/W4)* ?s in g/cm3	1.213	1.174	1.150	1.107
Weight of Container in gram	30.07	30.25	29.66	30.02
Mass of wet soil +container in gram	98.45	102.06	96.03	89.07
Mass of dry soil +container in gram	88.34	91.98	86.17	81.37
Mass of Water in gram	10.11	10.08	9.86	7.7
Mass of Dry Soil in gram	58.27	61.73	56.51	51.35
Water Content, %	17.350	16.329	17.448	14.995
Dry Dencity, ?d = ?s/(1+w)g/cm3	1.034	1.010	0.979	0.963

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GEOTECHNICAL ENGINEERING LABORATORY

Specific Gravity Determination

Test Type Specific Gravity	soil lab		
Test Standard Used :- AASHTO T-100-93			
Sample no:- TP-1			
Weight of Dry soil	20g		

Date 12/08/2014

AASHTO T-100-93		Sample No		
S/N	Particulars	A1	A2	A3
1	Weight of Density bottle W_1 (g)	62.39	62.39	62.39
2	Weight of Density bottle with dry soil W_2(g)	82.39	82.39	82.39
3	Weight of Density bottle with dry soil and water W_3(g)	173.39	173.75	**173.81**
4	Weight of Density bottle full of water W_4 (g)	161.13	161.13	161.13
5	Weight of dry soil (W_2-W_1) (g)	20	20	20
6	Weight of an equal volume of water(W2-W1)-(W3-W4)	7.74	7.38	7.32
7	Specific gravity of the soil G= (5/6)	2.58	2.71	2.73

SOIL TESTED BY :- ABDISA SAMUEL
Signature……………………
date 12/08/2014

Average Spesific Gravity
2.68

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Specific Gravity Determination

Test Type Specific Gravity	soil lab	
Test Standard Used :- AASHTO T-100-93		
Sample no:- TP-2		**TO=26**
Weight of Dry soil	20g	

Date 12/08/2014

S/N	Particulars	Sample No A1	A2	A3
1	Weight of Density bottle W_1 (g)	62.39	62.39	62.39
2	Weight of Density bottle with dry soil W_2(g)	82.35	82.35	82.35
3	Weight of Density bottle with dry soil and water W_3(g)	173.52	173.42	**173.96**
4	Weight of Density bottle full of water W_4 (g)	161.13	161.13	161.13
5	Weight of dry soil (W_2-W_1) (g)	19.96	19.96	19.96
6	Weight of an equal volume of water(W2-W1)-(W3-W4)	7.57	7.67	7.13
7	Specific gravity of the soil G= (5/6)	2.64	2.60	2.80

SOIL TESTED BY :- ABDISA SAMUEL
Signature……………………
date 12/08/2014

Average Spesific Gravity
2.68

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Specific Gravity Determination

Test Type Specific Gravity	soil lab		
Test Standard Used :- AASHTO T-100-93			
Sample no:- TP-3			**TO=26**
Weight of Dry soil	20g		
Date 12/08/2014	Sample No		

S/N	Particulars	A1	A2	A3
1	Weight of Density bottle W_1 (g)	62.39	62.39	62.39
2	Weight of Density bottle with dry soil W_2(g)	82.35	82.35	82.35
3	Weight of Density bottle with dry soil and water W_3(g)	173.54	173.86	**171.67**
4	Weight of Density bottle full of water W_4 (g)	161.13	161.13	**159.22**
5	Weight of dry soil (W_2-W_1) (g)	19.96	19.96	19.96
6	Weight of an equal volume of water(W2-W1)-(W3-W4)	7.55	7.23	7.51
7	Specific gravity of the soil G= (5/6)	2.64	2.76	2.66

SOIL TESTED BY :- ABDISA SAMUEL Signature……………………	Average Spesific Gravity
date 12/08/2014	2.69

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Specific Gravity Determination

Test Type Specific Gravity	soil lab		
Test Standard Used :- AASHTO T-100-93			
Sample no:- TP-4			**TO=26**
Weight of Dry soil	20g		
Date 12/08/2014	Sample No		

S/N	Particulars	A1	A2	**A3**
1	Weight of Density bottle W_1 (g)	62.39	62.39	62.39
2	Weight of Density bottle with dry soil W_2(g)	82.39	82.39	82.39
3	Weight of Density bottle with dry soil and water W_3(g)	174.19	174.22	**172.27**
4	Weight of Density bottle full of water W_4 (g)	161.13	161.13	161.13
5	Weight of dry soil (W_2-W_1) (g)	20	20	20
6	Weight of an equal volume of water(W2-W1)-(W3-W4)	6.94	6.91	8.86
7	Specific gravity of the soil G= (5/6)	2.88	2.89	2.26

SOIL TESTED BY :- ABDISA SAMUEL Signature……………………	Average Spesific Gravity
date 12/08/2014	2.68

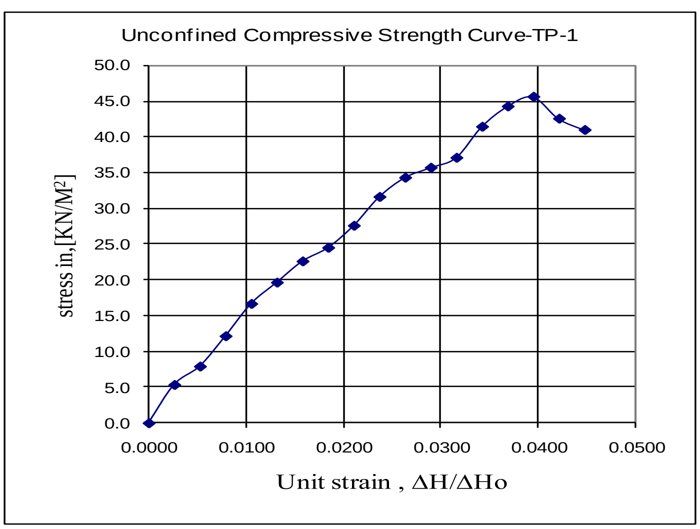
Unconfined Compression test

BH No	*tp1*
Depth(m)	*3.7*
Initial Height (mm)	*76*
Initial Area (m^2)	*0.0010*
Initial Volume(cm^3)	*84*
P.R. Factor (KN/div)	*0.00153*
Failure condtion	*shear*

Deformation dial in [mm] ?H	Unit strain ϵ = ?H/?Ho	Proving Ring reading	Load in [KN]	Cross - sectional Area $A = A_o/1-\epsilon$	stress in [KN/M^2]
0.00	0.0000	0.0	0.0000	0.00100	0.0
0.20	0.0026	3.5	0.0054	0.00100	5.3
0.40	0.0053	5.2	0.0080	0.00101	7.9
0.60	0.0079	8	0.0122	0.00101	12.1
0.80	0.0105	11	0.0168	0.00101	16.7
1.00	0.0132	13	0.0199	0.00101	19.6
1.20	0.0158	15	0.0230	0.00102	22.6
1.40	0.0184	16.3	0.0249	0.00102	24.5
1.60	0.0211	18.4	0.0282	0.00102	27.6
1.80	0.0237	21.14	0.0323	0.00102	31.6
2.00	0.0263	23	0.0352	0.00103	34.3
2.20	0.0289	24	0.0367	0.00103	35.7
2.40	0.0316	25	0.0383	0.00103	37.0
2.60	0.0342	28	0.0428	0.00104	41.4
2.80	0.0368	30	0.0459	0.00104	44.2
3.00	0.0395	31	0.0474	0.00104	45.6
3.20	0.0421	29	0.0444	0.00104	42.5
3.40	0.0447	28	0.0428	0.00105	40.9

Unconfined compressive strength, q_u = 45.6 KN/m2

Cohesion , C= $q_{u/2}$ = 22.8 KN/m2

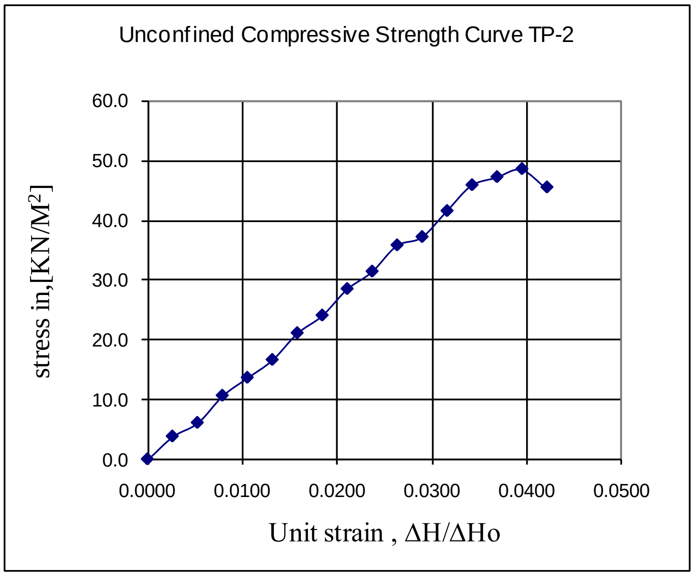


BH No	*tp2*
Depth(m)	*3.7*
Initial Height (mm)	*76*
Initial Area (m^2)	*0.0010*
Initial Volume(cm^3)	*84*
P.R. Factor (KN/div)	*0.00153*
Failure condtion	*buldge*

Deformation dial	Unit strain	Proving Ring reading	Load in [KN]	Cross - sectional	stress in [KN/M²]
0.00	0.0000	0.0	0.0000	0.00100	0.0
0.20	0.0026	2.5	0.0038	0.00100	3.8
0.40	0.0053	4	0.0061	0.00101	6.1
0.60	0.0079	7	0.0107	0.00101	10.6
0.80	0.0105	9	0.0138	0.00101	13.6
1.00	0.0132	11	0.0168	0.00101	16.6
1.20	0.0158	14	0.0214	0.00102	21.1
1.40	0.0184	16	0.0245	0.00102	24.0
1.60	0.0211	19	0.0291	0.00102	28.5
1.80	0.0237	21	0.0321	0.00102	31.4
2.00	0.0263	24	0.0367	0.00103	35.8
2.20	0.0289	25	0.0383	0.00103	37.1
2.40	0.0316	28	0.0428	0.00103	41.5
2.60	0.0342	31	0.0474	0.00104	45.8
2.80	0.0368	32	0.0490	0.00104	47.2
3.00	0.0395	33	0.0505	0.00104	48.5
3.20	0.0421	31	0.0474	0.00104	45.4

Unconfined compressive strength, q_u = 48.5 KN/m2

Cohesion , C= $q_{u/2}$ = 24.2 KN/m2

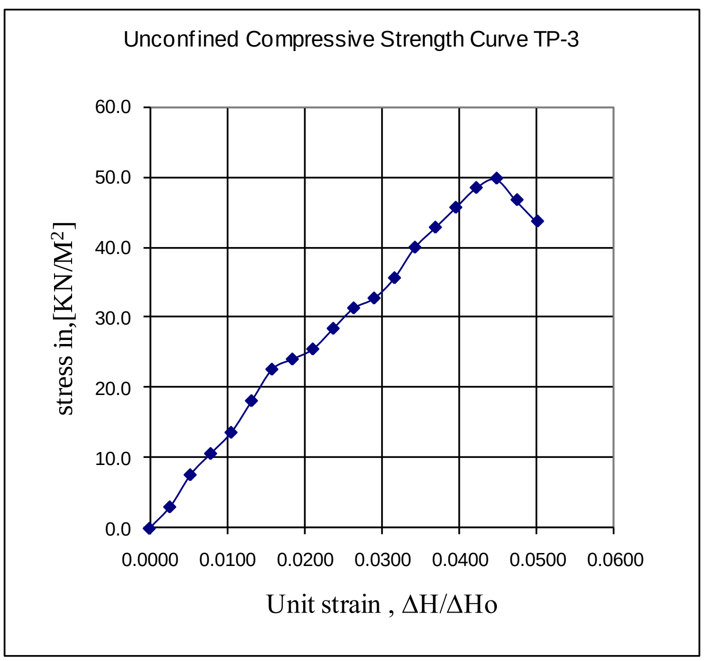


BH No	tp3
Depth(mm)	38
Initial Height (mm)	76
Initial Area (m^2)	0.0010
Initial Volume(cm^3)	84
P.R. Factor (KN/div)	0.00153
Failure condtion	shear

Deformation dial	Unit strain	Proving Ring reading	Load in [KN]	Cross - sectional	stress in [KN/M²]
0.00	0.0000	0.0	0.0000	0.00100	0.0
0.20	0.0026	2	0.0031	0.00100	3.1
0.40	0.0053	5	0.0077	0.00101	7.6
0.60	0.0079	7	0.0107	0.00101	10.6
0.80	0.0105	9	0.0138	0.00101	13.6
1.00	0.0132	12	0.0184	0.00101	18.1
1.20	0.0158	15	0.0230	0.00102	22.6
1.40	0.0184	16	0.0245	0.00102	24.0
1.60	0.0211	17	0.0260	0.00102	25.5
1.80	0.0237	19	0.0291	0.00102	28.4
2.00	0.0263	21	0.0321	0.00103	31.3
2.20	0.0289	22	0.0337	0.00103	32.7
2.40	0.0316	24	0.0367	0.00103	35.6
2.60	0.0342	27	0.0413	0.00104	39.9
2.80	0.0368	29	0.0444	0.00104	42.7
3.00	0.0395	31	0.0474	0.00104	45.6
3.20	0.0421	33	0.0505	0.00104	48.4
3.40	0.0447	34	0.0520	0.00105	49.7
3.60	0.0474	32	0.0490	0.00105	46.6
3.80	0.0500	30	0.0459	0.00105	43.6

Unconfined compressive strength, q_u = 49.7 KN/m2

Cohesion , C= $q_{u/2}$ = 24.8 KN/m2

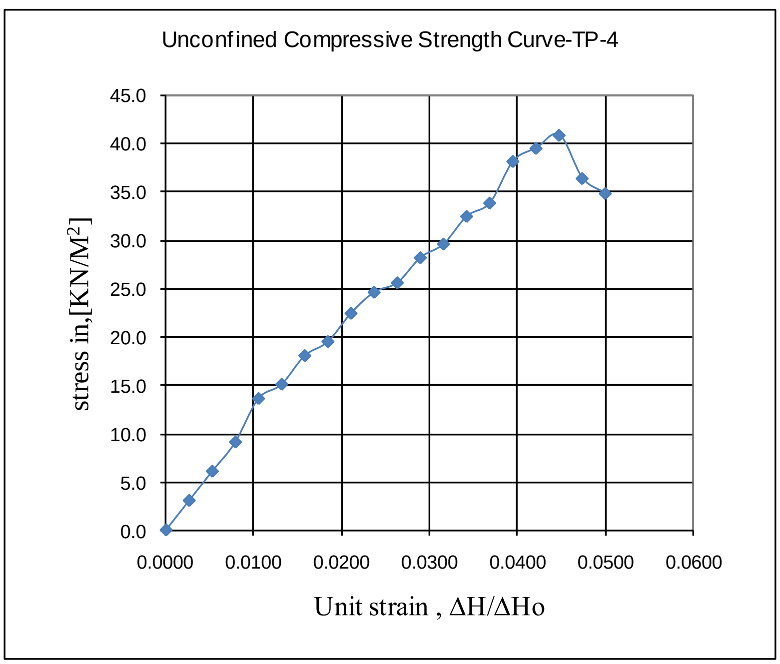


BH No	*tp4*
Depth(m)	*3.7*
Initial Height (mm)	*76*
Initial Area (m²)	*0.0010*
Initial Volume(cm³)	*84*
P.R. Factor (KN/div)	*0.00153*
Failure condtion	*shear*

Deformation dial in [mm] ?H	Unit strain ε = ?H/?Ho	Proving Ring reading	Load in [KN]	Cross - sectional Area $A = A_o/1-\epsilon$	stress in [KN/M²]
0.00	0.0000	0.0	0.0000	0.00100	0.0
0.20	0.0026	2	0.0031	0.00100	3.1
0.40	0.0053	4	0.0061	0.00101	6.1
0.60	0.0079	6	0.0092	0.00101	9.1
0.80	0.0105	9	0.0138	0.00101	13.6
1.00	0.0132	10	0.0153	0.00101	15.1
1.20	0.0158	12	0.0184	0.00102	18.1
1.40	0.0184	13	0.0199	0.00102	19.5
1.60	0.0211	15	0.0230	0.00102	22.5
1.80	0.0237	16.5	0.0252	0.00102	24.6
2.00	0.0263	17.2	0.0263	0.00103	25.6
2.20	0.0289	19	0.0291	0.00103	28.2
2.40	0.0316	20	0.0306	0.00103	29.6
2.60	0.0342	22	0.0337	0.00104	32.5
2.80	0.0368	23	0.0352	0.00104	33.9
3.00	0.0395	26	0.0398	0.00104	38.2
3.20	0.0421	27	0.0413	0.00104	39.6
3.40	0.0447	28	0.0428	0.00105	40.9
3.60	0.0474	25	0.0383	0.00105	36.4
3.80	0.0500	24	0.0367	0.00105	34.9

Unconfined compressive strength, q_u = 40.9 KN/m2

Cohesion , C= $q_{u/2}$ = 20.5 KN/m2



Appendix B: Photos