PREDICTIVE MODEL OF UNCONFINED COMPRESSIVE STRENGTH FROM INDEX PROPERTY OF LATERITIC SOILS.

(CASE OF DILLA TOWN)

MSc. THESIS

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EXAMINERS' APPROVAL SHEET-1

We, the undersigned, members of the Board of Examiners of the final open defense by **Samuel Abayneh** have read and evaluated his thesis entitled" **Predictive Model of Unconfined Compressive Strength from Index Properties of Lateritic Soils (Case of Dilla Town)",** and examined the candidate. This is, therefore, to certify that the thesis has been accepted in partial fulfillment of the requirements for the degree

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Dedicated In Loving Memory of My Little Brother

LIST OF ACRONYMS

AASHTO	American Association of States High Way and Transport Officials
ASTM	American Society for Testing and Materials
Ca	Calcium
CBR	California Bearing Ratio
CH	High Plastic Clay
CL	Low Plastic Clay
Cu	Un-drained cohesive strength
GI	Group Index
Gs	Specific Gravity
K	Potassium
LL	Liquid Limit
MDD	Maximum Dry Density
Mg	Magnesium
ML	Inorganic Silt
Na	Sodium
P	Standard Error
PI	Plasticity Index
PL	Plastic Limit
q_u	Unconfined Compressive Strength
RH	Relative Humidity
SPSS	Statistical Product and Service Solution
S_u	Un-drained Shear Strength
TP	Test/ Trial Pit
UCS	Unconfined Compressive Strength
w_n	Natural Moisture Content
γ_{ins}	In-Situ Unit Weight
ϕ_u	Un-drained Internal Angle of Friction

***TP Followed by number represents the number of test/trial pit while TP followed by number and hyphen then number indicates the depth of pit at which sample taken.

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ABSTRACT

In Dilla large buildings and important infrastructures are often supported by lateritic soils. Lateritic soils are anisotropic and heterogeneous. Geotechnical engineers concerned on lateritic soils are attempting to develop specific empirical equations to predict more reliable results needed for the design. As this study is one avenue towards developing specific predictive models, lateritic soil samples were taken and laboratory tests were conducted to meet its objective. Twenty disturbed and twenty undisturbed samples were taken from ten different randomly selected borrow pits having four-meter depth with in the town and various laboratory tests including Atterberg's limits, natural moisture contents, in-place unit weight, specific gravity and unconfined compressive strength were performed on the soil samples. Regression analysis have been done and various linear relationships between unconfined compressive strength and index properties were investigated and predictive equations estimating unconfined compressive strength from index properties were developed. Among the developed models, two comparatively better models having determination coefficients of 0.866 and 0.927 with standard error of less than 0.0005 are selected and validated. A model with 0.866 determination coefficient is given by $q_u = -421.843 - 4.191w_n + 40.870\gamma_{ins}$. *While, a model with 0.927 determination coefficient,* $q_u = 1.1e^{0.282\gamma ins}$ *is exponential and got a better accuracy on prediction during validation using control test result. The equations are valid for a natural moisture content in a range of 19% to 35% and in-situ unit weight in a range of 12 kN/m*^{3} *to 20.6 kN/m*^{3}.

Key words

Test/Trial pits; Lateritic soil; Index properties; Unconfined Compressive Strength; Dilla.

1. INTRODUCTION

1.1. Background

Dilla is one of the towns in Southern Nation Nationalities and People Regional State (SNNPR). It is administrative center of the Gedeo zone in SNNPR and is located at 360km from Addis Ababa (capital city of Ethiopia) along the main road from Addis Ababa to Nairobi (capital city of Kenya). Dilla is one of the developing towns in SNNPR and is center for best coffee production in Ethiopia. There are many existing civil structures and constructions on progress; but failures are happening on existing structures and there is also delay on time plan of constructions on progress. This shows that there might be a problem that was not considered.

Soil tests helps to determine varying physical and chemical characteristic of soil, which can vary from place to place and from layer to layer even within the limits of proposed construction site. Soil characteristics can change considerably with in small area. Weather, climatic changes, and site management can in the future affect the bearing qualities of the soil, if the foundation is not designed properly to the bearing capacity of the soil, it will fail and so will the construction. This is as important as entire project itself that may cause the long-term complications and may result in loss of life and property, danger residents, tenants and damage other neighboring properties.

Due to climatic conditions, weathering of parent rocks (igneous, sedimentary or metamorphic), mainly chemical weathering, is the main agent for soil formation in the tropics. The soils formed by weathering are largely left in place, thereby literally called residual soil and whose character depends on the parent rock it developed from. Hence, a knowledge of region's geology provides back ground for determining the probable type of parent materials which would be encountered and the probable duration of weathering process. Identification of physiography is also important in understanding the current stage of topography development and drainage condition (Bujang, 2013).

Many soils with in tropics or in tropical, sub-tropical, equatorial and mediterranean climatic zones contain significant amount of clay minerals. Lateritic soils are soils containing laterite (residual of rock decay that is red or reddish in color) and reddish tropical soils developed by much weathering (Bujang, 2013).

There are different factors (parent material, climate, topography, vegetation, and time) that influence the formation of soils but it is quite difficult to differentiate which factors have more influence on weathering than others (Erdil, 1976). Weathering process involves leaching of silica, formation of colloidal sesquioxides, and precipitation of the oxides with increasing crystallinity and dehydration as the rock becomes more weathered. The parent rock which contains primary feldspars, quartz, and ferromagnesian minerals is transformed to a porous clayey system containing kaolinite, sesquioxides, and some residual quartz. The primary feldspars are converted to kaolinite and then, kaolinite is transformed to gibbsite. Primary ferromagnesian minerals, on the other hand, are eventually converted to diffuse goethite, followed by well-crystallized goethite, and finally hematite (Erdil, 1976). The crystallization leads to the formation of iron and/or aluminum oxide concretions, coalescence of concretions and their cementation by iron and/or aluminum colloids, until the entire system is a continuous iron and/or aluminum oxide cemented crust (Alexander and Cady, 1962).

The test procedures established for temperate soils may not be suitable for tropical soils. Preparing prior to testing has significant effect on the result. This makes investigation or determination of index and engineering properties of soils more difficult and time consuming. In tropical soils, in addition to index properties test, unconfined compressive strength and California bearing ratio tests needs repeating tests with similar procedure to come up with reliable result (Pallavi, 2016).

1.2. Statement of the Problem

Dilla which is located in southeastern Ethiopia is covered with red colored soil. Depending on the soil forming factors such as climate, topography, drainage and the parent material, red soils can be lateritic soils.

As Dilla is one of the developing town, detailed and appropriate soil investigation is necessitated for the purpose of safety and economic sustainability. The scarcity of functional detailed geotechnical testing facilities leads to the attempt towards indirect method for evaluation and determination of shear strength which allows the minimization of number of tests or costs needed to measure them.

Lateritic soils are heterogeneous soils and hence needs specific correlation model to get reliable result. In case of Dilla town no correlative models were done before. UCS test is time consuming and relatively expensive than index properties tests. Therefore, it's important to develop model which can help to predict UCS from index properties for preliminary study. By doing this, it is possible to save time and cost.

In this study correlative models were developed which help to predict unconfined compressive strength from index property of lateritic soils found in Dilla. Hence, it is cost effective and saves time.

1.3. Objectives

1.3.1. General Objective

The main objective of this study is developing easier and quicker prediction model for unconfined compressive strength from index properties of lateritic soils.

1.3.2. Specific Objectives

- To determine the index properties of lateritic soil of Dilla and give appropriate classification based on AASHTO standard.
- To study the relationships between soil index properties listed above and shear strength parameter (un-drained shear strength).
- To validate and evaluate the developed model using controlled test.

1.4. Significance of the Study

Lateritic soils are being used in construction of roads, high ways, airfields, earth dams and as the foundation of structures, the detail site (in situ and laboratory) investigation should have to be conducted. Investigating engineering property of lateritic soils is very laborious and needs a great care. This research helps to minimize the cost and time that would be wasted during investigation and it facilitates preliminary study.

The outcome of the research provides a general guidance for geotechnical engineers about the shear strength parameters (UCS) to achieve their intended preliminary design of roads, high ways, airfields, earth dams and foundation of the structures. In addition, the outcome gives detail clue on relationships between soil index properties and shear strength parameter of lateritic soil of Dilla town.

1.5. Limitation of the Study

The research includes correlating the UCS with index properties of lateritic soils by addressing the proposed objectives. Around 140 tests (20 liquid limit tests, 20 plastic limit tests, 20 grain size analysis, 20 in-situ unit weight, 20 specific gravity, 20 natural moisture content tests and 20 UCS tests) were conducted on a sample that were obtained by excavating ten pits with four-meter depth to came up with a good numerical model. All test/trial pits were dug with in the town. Hence, the developed models are applied for the lateritic soils specific to the town not for all lateritic soils.

1.6. Structure of the Thesis

This thesis consists of six chapters; Chapter 1 provides information about the background of the study, problem statement, objectives, significance of study and limitation of the study. Chapter 2 consists of overall view of lateritic soils, their behavior and current level of knowledge with its gap. Chapter 3 provides description of sample area, materials and methods used for the achievement of the objective of the study. Chapter 4 consists of laboratory test analysis, laboratory test results and discussion including classification of samples used, regression analysis, development of correlation models and discussion. Conclusions and recommendations are presented as chapter 5. Finally, references and appendices are included.

2. LITERATURE REVIEW

2.1. Residual Soils

Residual soils are soils developed on extremely weathered rock, the mass structure and substance fabric are no longer evident; there is a large change in volume but the soil has not been significantly transported.

Weathering due to interaction of some state factors (chronological, climatic, geologic, biotic and topographic) facilitates the decomposition of parent rocks to form in situ residua. Therefore, the defining factor for residual soil is that there is very little or no transport during or after formation (Singh & Kataria, 1980). Although different sub-tropical climate as consequence of different temperature and precipitation patterns cause the development of residual soils with distinct characteristics, climate is the most crucial factor in the development of tropical soils. The appearance of soils in the tropics shows the effect of time with older soils being generally thicker and more highly weathered i.e. altered and leached (Bujang, 2013).

2.2. Formation and Characteristics of Tropical Residual Soils

Weathering involves the weathering of the rocks respective constituent minerals. The mineral suite forming the respective parent rock type (igneous, sedimentary and metamorphic) determines the residual minerals left or formed from weathering. The character of a residual soil, therefore, depends on the parent rock it developed from.

The successive variation of dry and wet seasons facilitates chemical weathering. Chemical weathering is more prevalent and effective in breaking down rocks than mechanical disintegration and the dominance of this activity is very pronounced in regions where the presence of water and high temperature is significant (Blight, 1997).

The residual soils formed from weathered material are normally very thick in the tropics as this is an area with conditions favorable for intense weathering. Conversely, they may be very thin or absent in areas of unfavorable conditions like arid regions or steep mountain slopes subjected to erosion by mass movements (Blight, 1997). Soils formed by weathering in the tropics are generally referred to as tropical residual soils (Pallavi, 2016).

Residual soils in the tropics develop a soil profile consisting of a sequence of distinct layers called soil horizons, paralleling the ground surface. The study of the soil horizons, however, is the domain of soil science. Simultaneously, the soil profile also develops a physical/morphological vertical weathered profile which is its pertinent aspect from the engineering perspective (Sherman, 1952).

The hot, humid environment speeds up the chemical dissolution of rock-forming silicate minerals, the formation of new and residual mineral species and the decay of organic matter. The soluble ions are trans-located by leaching while small particles are also elevated by physical down washing. In addition, the leaching due to chelating agents like humic acids, however, is rare as the buildup of acidic compounds due to the slow decomposition of vegetation is restricted. Therefore, generally, the bases (K, Na, Ca, Mg) and silica are leached downwards in solution to be re-deposited lower down in the soil profile, while Fe and Al remain insoluble as the decomposition takes place in near-neutral conditions below the depth of acidic chelating agents confined nearer to the small surface humus cover (Blight, 1997).

The areas in the tropics affected by alternating wet and dry seasons cause the soils to experience a different translocation of elements. During the wet season, weathering of the primary silicate minerals in the upper soil horizon causes leaching of the silica and bases away leaving behind the iron and aluminum oxides. During the dry season, however, due to water rising from below due to capillary rise, the leaching of silicates progresses below the hydrated Fe and Al oxides creating a layer of insoluble oxidized iron and other minerals (Bujang, 2013). This gives rise to two different soil profiles as shown in Figure 2.1.

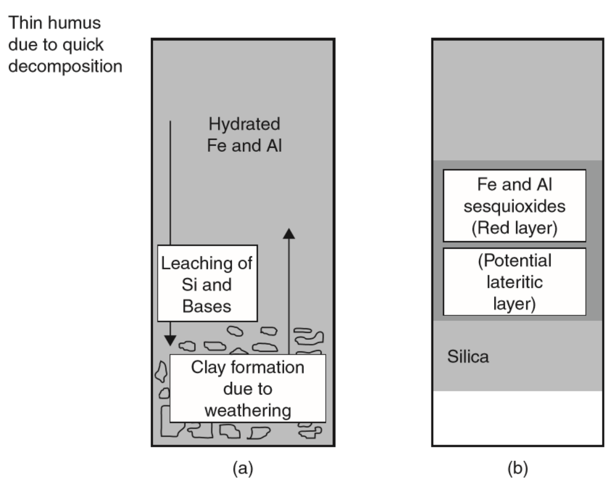

Figure 2. 1Tropical soil profiles (After Nagle, 2000). (a) In continuously humid climate and (b) in alternating wet and dry climate cited by Bujang, (2013).

Residual soils are invariably encountered in engineering practice in tropical areas. The conventional perspective of residual soils first given described the changes that are involved in their formation and their nature. This conventional characterization of residual soil has further been customized to meet the requirements of geotechnical engineering.

2.3. Red Tropical Soils

The red tropical soils are the most abundant in the tropics. In tropical humid climates strong hydrolysis leads to the rapid destruction of all the weather-able minerals and a massive neogenesis of clays and ferric hydrates. Different minerals can be formed depending on the degree of hydrolysis; gibbsite in strongly leached environment 1/1 clay (monosiallitization) when silicon and aluminum are recombined in the form of kaolinite in moderately leached environment, and 2/1 clay (bisiallitization) when smectites as beidellite and nontromite are formed in less leached environment (Pedro, 1989). Smectite lattices will integrate part of the iron released from primary minerals whereas very little or no iron will be included in gibbsite or kaolinite crystals. In these later cases, the totality of the iron released separates in the form of ‘free’ ox hydroxides.

2.4. Lateritic Soils

Lateritic soils can be described broadly as products of tropical weathering with red, reddish brown, or deep brown color, with or without nodules or concretions, and that generally (but not exclusively) found below hardened ferruginous crusts or hard pan. Lateritic soils as a soil group rather well-designed material, are most commonly found in the leached soils of the humid tropics where they were first studied (Leornado, 2012). These soils are formed under weathering conditions as product of the process of laterization, the most important characteristics of which is the decomposition of ferroaluminum silicate minerals and the permanent deposition of sesquioxides (i.e. oxides of iron and aluminum Fe_2O_3 and $Al_2\,O_3$) with in the profile to form the horizon of material known as laterite. The term “Laterization” was used to describe the process that produce lateritic soils (Leornado, 2012).

Lateritic soils have a high bearing capacity and low compressibility at their optimum moisture content but under alternative wet and dry seasons their engineering properties will be poor (Pallavi, 2016). The effective use of these soils is therefore often hindered by difficulty in handling particularly moist and wet conditions typical of tropical regions and can only be utilized after modification/ stabilization (Pallavi, 2016).

Before modification of properties of lateritic soil, the properties of soil in a natural condition need have to be investigated. Investigation of such soil requires great care of handling soil samples. Hence, to reduce the risk of disrupt that would be occurred during handling and preparation, it is good to develop predictive models by carefully handled soil samples test results.

2.4.1. Weathering and Laterization

Rocks when exposed at or near the earth's surface find themselves in a physical and chemical environment often quite different from that in which they were originally formed. The minerals which constitute the rocks may react chemically with the rainwater, groundwater, and dissolved solids and gases of the new near-surface environment to form new minerals which are more nearly in equilibrium with the surface conditions. Some materials may be carried away by solution in ground- water (Lyon, 1971). The result of these changes is to convert the upper portion of the rock into a residual debris more soil-like than rock-like in character and with chemical, mineralogical, and physical properties entirely different from those of the original rock.

The changes which take place in the rock by these near-surface processes are encompassed by the geological term weathering. One form of weathering, denoted as physical, includes the effects of such mechanical processes as abrasion, expansion, and contraction. Physical weathering produces end products consisting of angular blocks, cobbles, gravel, sand, silt, and even clay-sized rock flour. The mineral constituents of all these products are exactly like those of the original rock. Chemical weathering, on the other hand, results in the decomposition of the rock and the formation of new minerals. The near-surface agents of weathering which enter into chemical reactions with the primary minerals of the rock include water, oxygen, carbon dioxide, and organic acids derived from vegetation. The various chemical processes include hydration, hydrolysis, oxidation, solution, and carbonation. All of these may operate simultaneously, some more rapidly than others and some more effectively in the alteration of one mineral than another (Lyon, 1971).

The chemical changes operating in the primary minerals of the rocks in temperate or semi-tropical zones tend to produce end-products consisting of clay minerals predominantly represented by kaolinite and occasionally by halloysite and by hydrated or anhydrous oxides of iron and aluminum. Quartz experiences slight solution but remains essentially unchanged, although it may undergo some comminution (Erdil, 1976).

Chemical weathering is favored by warm humid climates, by the presence of vegetation, and by gentle slopes. Thus, tropical and subtropical regions of low relief with abundant rainfall and high temperatures are the most susceptible to chemical alterations. Deep, strongly leached red, brown, and yellow profiles are manifestations of the effects of severe chemical weathering.

Under conditions favorable to tropical weathering, the weathering processes may be so intense and may continue so long that even the clay minerals, which are primarily hydrous aluminum silicates, are destroyed; in the continued weathering the silica is leached and the remainder consists merely of aluminum oxide such as gibbsite, or of hydrous iron oxide such as limonite or goethite derived from the iron. This process is known as laterization.

The extent to which a residual soil has been laterized may be measured by the ratio of silica, SiO_2, remaining in the soil (except for discrete pebbles of free quartz that may remain) to the amount of Fe_2O_3 and $A1_2O_3$ that has accumulated. The silica: sesquioxide ratio has served as a basis for classification of residual soils.

$$\frac{SiO_2}{R_2O_3} = \frac{SiO_2}{FeO_2 + R_2O_3} \quad \text{..Eq. 2.1}$$

Ratios less than 1.33 have sometimes been considered indicative of true laterites, those between 1.33 and 2.00 of lateritic soils, and those greater than 2.00 of non-lateritic tropically weathered soils (Lyon, 1971).

2.4.2. Classification

There have been several attempts to classify laterites and lateritic soils for many years, but none of the proposed classification system has been accepted universally. According to Maignien, (1966), these classification systems can be grouped as analytical classifications which are based mainly on morphological characteristics with a bias toward soil genetic considerations, synthetic classifications which are based on genetic factors or soil-genetic processes. Most classification systems do not aim to classify the soils according to their engineering behavior. Although there are some popular engineering classification systems, such as Unified Soil Classification System (USCS) and the American Association of State Highway and Transportation Officials System (AASHTO) which have been used satisfactorily for other soil types. However, these classification systems are based on plasticity and gradation data of the soils; but such characteristics of tropical soils are not reproducible by standard laboratory tests. This is because laterites are highly influenced by sample preparation and handling which disrupts the natural structure of the soil.

The first step in the grouping of residual soils is to divide them into groups on the basis of mineralogical composition alone, without referring to their undisturbed state. The following three groups are often suggested (Dibisa, 2008).

1. Group A: Soils without a strong mineralogical influence, e.g., Saprolites (Residual soil with clear structural feature inherited from its parent rock).
2. Group B: Soils with a strong mineralogical influence deriving from clay minerals also commonly found in transported soils (Black Cotton Soils).
3. Group C: Soils with a strong mineralogical influence deriving from special clay minerals only found in residual soils (i.e. based on the silicate clay minerals, Halloysite and Allophone, and non-silicate minerals ('oxide' minerals) which are the hydrated forms of aluminum and iron oxide (the sesquioxides),Gibbsite and Goethite).

Group C is further divided in to three sub-groups according to clay minerals of soils as follows:

i. Halloysitic soils
ii. Allophanic soils
iii. soils influenced by the presence of sesquioxides

Lateritic soils lay under group C, soils influenced by the presence of sesquioxides. The engineering properties of soils under this group are highly influenced by the presence of sesquioxides. Sesquioxide is the combined name of iron and aluminum oxide. Sesquioxides appear to act as a cementing agent which bind the other mineral constituents into clusters or aggregations. The hard-concretionary materials are formed as a result of sufficient concretion of sesquioxides.

Besides the grouping system presented above, an additional item of formation which is usually of major importance in influencing the properties of residual tropical soils is the type of the parent rock and should always be included in the grouping processes. Most of the residual soils of Africa can be divided into three groups based on their genetic basis, determined by the soil-forming factors and given below:

i. Ferruginous Soils: occurs in the more arid extremes for lateritic soils, in areas with pronounced dry seasons. They are formed over all rock types: igneous, metamorphic and sedimentary. It requires an average annual rainfall of 600-800 mm for its formation.

ii. Ferrallitic Soils: occurs in the humid extremes for lateritic soils and in areas with dense vegetation. These soils are also formed over all rock types. The annual average rainfall requirement for its formation is 1500- 4000mm. Both of the above soils have SiO_2 / (Fe_2O_3 + Al_2O_3) ratio of less than 2.0 and are classified either as lateritic or laterite soils.

iii. Ferrisols: These are formed over all types of rocks in intermediate to high rainfall areas where erosion has kept the place with profile development. They have similar profiles to Ferrallitic soils, but with few weatherable minerals remaining. The entire clay fraction comprises Kaolinite and amorphous oxides of iron and aluminum. These are developed at deeper levels due to the surface erosion, and occur in regions of annual average rainfall of 1250-2750mm. According to Lyon Associates, Ethiopian laterites fall under this group.

2.5. Index and Engineering Properties of Laterites

2.5.1. Moisture Content

The conventional test for the determination of moisture content, which is used for testing temperate soils may not be suitable for residual soils. In many residual soils, some moisture exits as water of crystallization, within the structure of minerals presented in the soils particle. Some of this structural moisture may be removed by drying in a temperature of conventional test method. The following procedure is therefore recommended:

Two test specimens should be prepared for moisture content determinations. One specimen should be oven dried at 105ºC until successive weighing show that no further loss of mass. The moisture content should then be calculated in normal way. The second sample should be air dried (if feasible); or oven dried at a temperature of no more than 50ºC and a maximum relative humidity (RH) of 30% until successive weighing show that no further loss of mass. The two moistures content results should then be compared; a significant difference (4-6% of moisture content obtained by oven drying at 105ºC) indicates that structural water is present. This water forms part of soil solids, and should therefore be excluded from the calculation of moisture content. If a difference is detected using the two-different drying process, all subsequent tests for moisture content determination (including those associated with Atterberg Limit tests, etc) should be carried out by drying at lower temperature (i.e. either air drying, or oven–drying at 50ºC and 30% RH) if possible, the lower drying temperature of 50ºC should be used (Blight, 1997).

Ayele (2015) in his study, investigation of engineering properties of lateritic soils in case of Dilla, proves that the amount of structural water found in lateritic soils of Dilla is insignificant. Therefore, it is possible to base the determination of moisture content using conventional method in case of lateritic soils of Dilla town.

2.5.2. Atterberg Limits

The stiffness or consistency of fine grained soils depends on their moisture content, and varies with variations in the amount of moisture present. Atterberg limits define the moisture contents at which the soil changes from one state to another. These include the liquid limit, the plastic limit and shrinkage limit. They are determined by tests carried out on the fine soil fraction passing the 425µm (No. 40) sieve (BS, 2001). Because the formation of lateritic soils involves differential weathering as well as movement and deposition of dissolved materials, the variation of plasticity characteristics with depth cannot be predicted even in two similar profiles on different topographical sites (Tibebu, 2008).

2.5.2.1. Effect of Pre-test Drying

Laterites formed under continuously wet regions are likely to be characterized by high natural water contents; high Liquid Limits are observed to result in irreversible changes up on drying. Upon drying the plasticity decreases.

On the other hand, lateritic soils formed under seasons of distinct wet and dry seasons are likely to be characterized by low natural moisture content, low plasticity, and presence of concretions and cemented horizons. Laboratory tests run from natural water content or from the air-dried state lead to essentially the same result (Bujang, 2013).

According to Blight, the effect of drying prior to testing is attributed to:

- Increased cementation due to oxidation of the iron and aluminum Sesqueoxides, or
- Dehydration of Allophane and Halloysite, or both.

2.5.2.2. Effect of Method and Time of Mixing on Atterberg Limits

In general, the greater the duration of mixing (i.e., the greater the energy applied to the soil prior to testing), the larger the value of the resulting liquid limit, and to a lesser extent, the larger the plasticity index. This has been attributed to longer mixing results in more extensive break down of the cemented bonds between the clay clusters and within peds (disaggregation of the particles), and thus formation of greater proportions of fine particles.

The mixing time should be standardized at 5 minutes, and the mixed specimens should be left for moisture content equilibration overnight before testing. On the following day the liquid limit should be determined with a minimum of further mixing. A sub-sample from each of the specimens used in the test should be used for the determination of moisture content, using the procedure.

The remainder of each specimen should then be mixed continuously for a further 30 minutes before again determining the Liquid Limit. A significant difference (of >5% of the liquid limit obtained) between the liquid limit from tests using 5 and 30minutes mixing times indicates a disaggregation of the clay sized particles in the soil.

If this disaggregation is confirmed by repeating the above procedures, the entire program of testing should:

- Limit the mixing times to no more than 5 minutes.
- Make use of fresh soil for each moisture content point in Atterberg Limit tests.

The soil should be broken-down by soaking in distilled water, and not by drying and grinding. The soil should be immersed in distilled water to form slurry, which is then washed through a 425μm sieves until the water runs clear. The material passing the sieve is collected and used for Atterberg Limit test (Blight, 1997).

2.5.3. Shear Strength

The shear strength of soils is an important aspect in many foundation engineering problems such as the bearing capacity of shallow foundations and piles, the stability of the slopes of dams and embankments, and lateral earth pressure on retaining walls. The shear strength of cohesive soils can generally be determined in the laboratory by either unconfined compression tests or triaxial shear test equipment; however, the triaxial test is more commonly used (Braja, 1997).

2.6. Unconfined Compression Test

The primary purpose of this test is to determine the unconfined compressive strength (UCS), which is used to calculate the unconsolidated un-drained shear strength of the clay under unconfined conditions. According to the ASTM standard, the unconfined compressive strength (qu) is defined as the compressive stress at which an unconfined cylindrical specimen of soil will fail in a simple compression test. In addition, in this test method, the unconfined compressive strength is taken as the maximum load attained per unit area at 15% axial stain, whichever occurs first during the performance of a test.

2.7. Previous Work

There are some practical experiences of using correlation for estimating engineering properties from different simple index properties tests. Some latest related studies with developing correlation between shear strength and index properties of soils are described as follows.

Oluwapelumi (2013), conducted a research on predictive shear strength model for tropical lateritic soils. The samples were collected from six different locations in the federal university of technology Akure, Ondostate, Nigeria. Exploratory analysis of the six independent variables was performed using the statistical analysis of principal components. Using independent variables, multi-regression predictive models were developed for CBR and un-drained shear strength (Su). Positive linear relationship was found between Su and MDD with equation $Su = -547.713 + 0.381MDD - 9.104GI$. In this research only MDD and GI are used to predict Su.

Although Group index value (GI) is an indicator of suitability of subgrade soil for highway construction, Group Index value by itself is obtained by empirical relationship. Correlating GI with Su might misrepresent the association between them. Subsequently, it is important to correlate Su value with soil index properties as far as the Group Index value is a parameter that obtained by empirical relationship to measure the performance of the subgrade soil by taking into account of the plasticity and gradation of soil.

Usama Khalid, et al. (2015), conducted research on developing predictive model for UCS from index property of CL, ML, CH and CL-ML soils found in Pakistan. Soil samples were acquired from various places in Pakistan. Total of 85 undisturbed soils are collected. The testing program comprises, natural moisture content determination, grain size distribution, Atterberg's limit tests performed in accordance with ASTM standards. The unconfined compression tests are performed to evaluate UCS of soil samples. Positive linear relationship was found between UCS and dry density. Negative linear relationship was developed between UCS and natural moisture content. No clear relationship could be established between UCS and liquid limit and plastic limit. However, the UCS value varies as the liquid limit and plastic limit values fluctuates.

Leonardo, et al. (2012), conducted research on regression analysis between properties of subgrade lateritic soils in Osogbo town of south western Nigeria. Lateritic soil samples were taken from eight different borrow pits within the town and various laboratory tests including Atterberg limits, Gradation analysis, California Bearing Ratio (CBR), Compaction and Specific gravity were performed on the soil samples. Positive linear relationship was found between CBR and PL, LL, SG and MDD. The authors indicated that more accurate results can be developed when the regression based models such as two ways ANOVA and computer based reliability analysis be carried out on a wider variety of soil samples so as to specify the range of applicability of the derived model viz-a-viz the input variables.

SPSS software was used to compute the statistical analysis. In addition to this, Microsoft excel was used to develop scatter plot which helps to observe how independent variables are relating with dependent variable.

3. MATERIALS AND METHODS

3.1. Description of Study Area

Dilla is one of the towns in Southern Nation Nationalities and People's Region (SNNPR). It is administrative center of the Gedeo zone in SNNPR and has latitude and longitude of 6º24'38" N and 38º18'37" E respectively. It has average elevation of 1570m above sea level.

3.1.1. Geology of the Study Area

Dilla town is found in the main rift valley of Ethiopia. Most of the existing built up areas of the town is almost sloppy. The soil types in Dilla include red clay, reddish brown, dark brown and black cotton soils. According to Ayele (2015) the dominant type of soil in Dilla town is Dystric nitisols (red clay soils) which covers around 90% of total area. The next dominant type of soil in Dilla is Eutric nitisols (brown clay soils) which covers only 6% of total area. The rest 4% of the area is covered by Luvic phaezems (reddish brown clay soils) and black cotton soils (covers insignificant area around new prison with very gentle slope).

3.1.2. Topography and Climate

3.1.2.1. Topography

Dilla town is located on the boarder of Sidamma zone. Most of the existing built up areas of the town is almost sloppy and there is also some hill slope and mountain in the town. Along the course of the rivers and streams steep slope & gullies are also observed. Concerning the altitude of the location of test pits, the altitude ranges from 1459 meter to 1600 meter above sea level. The town is growing fast and is expanding along main road of Addis Ababa to Nairobi. As regard to the proposed expansion most of the areas are characterized by flat and gentle slopes towards Chichu village in the northern direction of the existing built up areas of the town.

3.1.2.2. Climate

The average annual temperature in Dilla is 20.6^0c and about 1129mm of precipitation annually. The variation in annual temperature is around 2.2 °C.

- The Mean annual rain fall ranges between 1062.10 - 1669.70mm
- The Mean annual maximum temperature ranges between 24.8 - 25.8ºc
- The Mean annual Minimum temperature ranges between 13.8 – 14.9ºc.

Dilla town is known for its abundant rainfall where it receives rainfall for substantial period of the year. The monthly average rainfall distribution is higher in April, May, September and

October, where the Dilla area receives frequent and peak rainfall prevalence. Rainy season expected from March to June and from August to November, whereas the rest of months receive relatively lower rainfall.

3.2. Materials

The soil material used in this study was lateritic soil, which is found dominantly abundant in Dilla town. An average of 250 kilo grams (minimum of 7kg from each pit was used according to standards of each test type) of disturbed and undisturbed soil samples were collected from ten excavated test pits. The color of soil sample was red to reddish (Figure 3.3).

The factors that favors latirization such as geology, climate topography and parent rock for Dilla soil agrees with criteria drawn by CIRIA (1995) for formation of lateritic soils. In addition to above factors, the geochemical test conducted by Ayele (2015) indicates the soil of the study area is lateritic soil. The following table shows the detail of geochemical test result as summarized by Ayele (2015).

Table 3. 1 Oxide composition in percent (Ayele, 2015).

Location	Sample Depth (m)	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Na_2O	K_2O	P_2O_5	Ti_2O	H_2O	LOI	$\frac{SiO2}{R2O3}$
TP2	1.5	52.36	24.8	8.06	1.2	1.5	<0.01	0.16	0.08	0.02	0.02	11.7	1.59
TP5	1.5	57.25	20.2	8.8	1.18	1.38	<0.01	0.26	0.06	0.02	0.06	10.2	1.97

** GPS location of TP2 is 38º18’46.01” E and 6º24’38” N

** GPS location of TP5 is 38º17’57.21” E and 6º25’01.64” N

3.3.Methods

3.3.1. Sample Collection Methods

To achieve the objectives of research, ten sampling areas were randomly selected. From randomly selected areas, pits were excavated to a depth of four-meters. Soil samples at 2m and 4m depth of each pits were collected for laboratory testing. After appropriate samples have been collected, the collected samples were prepared for test according to ASTM, AASHTO and BS standards for lateritic soils.

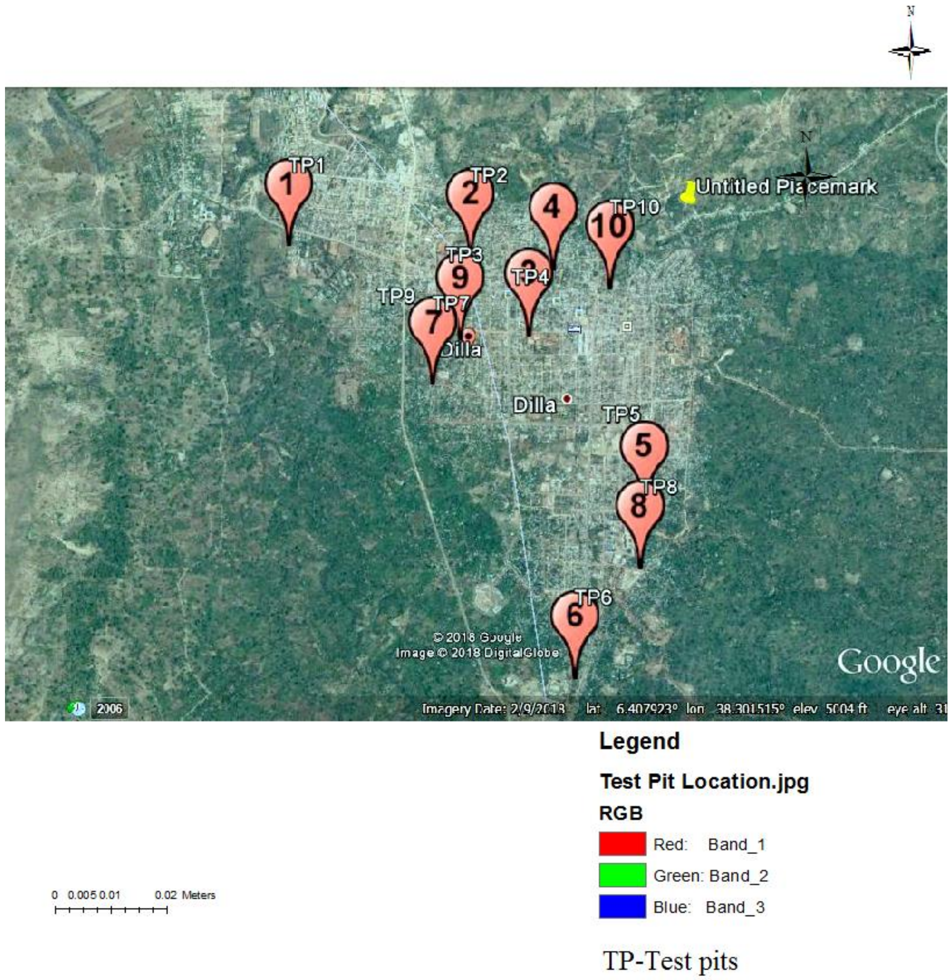

Figure 3. 1 The location of test pits under investigation

Test or Trial Pits.

This is the method of site exploration that was used in this research work. A test pit was simply a hole dug in the ground that was large enough for a ladder to be placed in, thus permitting a close examination of the sides.

The depth of each trial pit was 4m and about 1.2m × 1.2m wide that is, 1.2m× 1.2m× 4m pit. The types of samples collected for the laboratory analysis were as follows.

- Disturbed samples
- 'Undisturbed' samples

The pit has sunk by hand excavation with the aid of spade and digger.

Undisturbed Sample. Undisturbed samples were required for shear strength tests on the soil. Undisturbed block samples were cut by hand from the middle and bottom of the pits with the help of hydraulic jack (car lifting crick) and molds. During cutting the samples were protected from water, wind, and sun to avoid any change in water content. The samples were covered with candle wax (in both end of molds) and black polythene bag immediately; they were brought to the surface, and the samples were carefully labelled.

Figure 3. 2 Undisturbed soil sample covered with plastic bag and candle wax

In the laboratory the plastic bag was removed and the candle wax is trimmed. After doing that immediately appropriate specimen for unconfined compression test was extruded from the molds. The size of samples that were taken for unconfined compression test was 38mm diameter by 76mm height (ASTM, 2004).

Disturbed Sample. Disturbed samples were used mainly for soil classification, determination of specific gravity and particle size determination. In this study, disturbed samples were collected to conduct Atterberg's limits, natural moisture content, particle size distribution and specific gravity. Disturbed samples were also collected from the middle and bottom of the trial pits. From each sample point 10 to 15kg soils were collected i.e a total of 20 to 30kg soil collected from each test pit. The summary of test pit location and sample color was presented in table 3.1.

Table 3. 2 Summary of samples that have been investigated for laboratory investigation

SNo.	Test Pit	Location			Sample Designation Colour	Depth (m)	Colour
		X	Y	Z(m)			
1	TP1	38°17'12.6"	6°25'12.75"	1459	TP1_1	2	Reddish
					TP1_2	4	Reddish
2	TP2	38°18'3.29"	6°25'9.43"	1524	TP2_1	2	Reddish
					TP2_2	4	Reddish
3	TP3	38°18'19.4"	6°24'47.39"	1546	TP3_1	2	Reddish
					TP3_2	4	Reddish
4	TP4	38°18'26.2"	6°25'5.42"	1552	TP4_1	2	Reddish
					TP4_2	4	Reddish-brown
5	TP5	38°18'51.1"	6°24'0.07"	1594	TP5_1	2	Red
					TP5_2	4	Red
6	TP6	38°18'32.2"	6°23'13.41"	1538	TP6_1	2	Red
					TP6_2	4	Red
7	TP7	38°17'52.8"	6°24'34.19"	1517	TP7_1	2	Reddish
					TP7_2	4	Reddish
8	TP8	38°18'50.0"	6°23'43.87"	1600	TP8_1	2	Red
					TP8_2	4	Red
9	TP9	38°18'0.51"	6°24'46.44"	1518	TP9_1	2	Reddish
					TP9_2	4	Reddish
10	TP10	38°18'41.7"	6°25'0.33"	1577	TP10_1	2	Reddish
					TP10_2	4	Reddish

Figure 3. 3 Typical soil profile and in place soil color of the trial pits.

3.3.2. Laboratory Test Methods

3.3.2.1. Index Properties

The tests required for the determination of engineering properties are generally detail and time consuming. Sometimes, the geotechnical engineer is interested to have some rough assessment of the engineering properties without conducting detailed tests. This is possible if index properties are determined. The properties of soils which are not of primary interest for the geotechnical engineer but which are indicative of the engineering properties are called index properties of soils.

The soils are classified and identified based on the index properties. However, studies have revealed that tropical soils are different from temperate zone soils in terms of genesis and structure (Lyon, 1971). Their structures as compared to dispersed temperate zone soils have necessitated modifications to the mechanical or grading tests; the conventional pretreatment methods have considerable effect on the index properties of tropical soils. Therefore, special consideration is required during pretreatments and testing methods while testing tropical soils. The various properties of soils, which could be considered as index properties are: Grain size analysis, Atterberg Limits, Free swell and Specific gravity.

3.3.2.1.1. Moisture Content

Effect of Temperature on Moisture Content Determination

The conventional method for the determination of moisture content is based on the loss of water when a soil is dried to a constant mass at a temperature between 105 and 110^0C. Using the above temperature is by far high for certain tropical soils as it can drive out their water of hydration and resulting in an irreversible property change on the soil. This effect can be overcome in the laboratory by conducting the natural moisture content of the soil at different test temperatures.

Methods of Determining Moisture Content

Moisture contents of the soil samples were determined in the laboratory according to ASTM and Blight. The method allows drying the sample to an oven temperature of 105^0C and oven temperature of 50^0C with maximum relative humidity (RH) of 30% or equivalently air-drying.

3.3.2.1.2. In Place or In-Situ Unit Weight

In-place/ in-situ unit weight test was conducted on undisturbed soil sample obtained by pushing a circular mold using hydraulic jack/ crick. This test is used to determine the in-place unit weight of soils. This test can also be used to determine density of compacted soils used in the construction of structural fills, highway embankments, or earth dams. In this research in-place unit weight test was conducted according to ASTM D 2937-00 standard.

3.3.2.1.3. Particle Size Determination

This test was carried out to determine the percentage of different grain sizes contained within a soil. The mechanical or sieve analysis was done to determine the distribution of the coarser, larger-sized particles, and the hydrometer method was used to determine the distribution of the finer particles.

Particle size determination is standardized in ASTM D 422 - Standard Test Method for Particle-Size Analysis of Soils and its main significance providing the grain size distribution, and it is required in classifying the soil.

3.3.2.1.4. Atterberg Limits

The presence of clay mineral in fine grained soils makes the soil to be remolded without crumbling if some moisture exists. This cohesive nature is caused by the adsorbed water surrounding the particle. At very low moisture, the soil behaves more like solid but when the moisture gets very high, the soil and water may flow like a liquid. Atterberg developed Liquid Limit, Plastic Limit and Shrinkage Limit to describe soil consistency. Lateritic soils are affected by pretreatment and mixing time.

Test procedures:

For the determination of the Atterberg Limit values, air-dried and oven-dried soil samples were tested following the procedures given in ASTM D 421-85 and D 4318-98. The air-dried soil samples were prepared by spreading the material out in trays in the laboratory and leaving it open to the air for at least 15 days. The room temperature was about 20^0C. The oven-dried samples are prepared by drying the soils overnight at 105^0C oven temperature. According to Ayele (2015) there is no significant difference between the two preparation methods for the lateritic soils of Dilla town. The portions of the samples passing the No. 40(0.425mm) sieve were used for the preparation of the samples.

3.3.2.1.5. Soil Classification

Lateritic soils are classified on different bases; morphological characteristics with a bias toward soil genetic considerations and genetic factors or soil-genetic processes are among the basis for lateritic soil classifications (Maignien, 1966).

Different Authors used different classification techniques to classify residual tropical soils; Classification of soils based on the mineralogical composition alone and based on soil forming factors among the classification techniques (Erdil, 1976).

As Mohr and Van Baren (1954) point out every classification system should have some predetermined purposes. None of the classification systems mentioned above has an aim to classify the soils according to their engineering behavior. Although popular engineering classification systems, such as the Unified system (USCS) or the American Association of State Highway Officials system (AASHTO), have been used satisfactorily in the temperate environments of the world for years. They are also convenient as the basis for classifying tropical weathered soils. The soil samples in this study were classified according to the AASHTO M-145.

3.3.2.1.6. Specific Gravity

Specific gravity of the soils samples under investigation was determined using ASTM test designation D854 – 98 procedures method ‘A’ for oven dry. The portion of oven dry soil sample passing the No. 10 sieve was used for testing specific gravity of the soil. Specific gravity is used to calculate parameters such as void ratio, porosity, soil particle size distribution by the hydrometer method and degree of saturation.

3.3.2.2. Unconfined Compressive (UC) Test

The primary purpose of this test is to determine the unconfined compressive strength, which is then used to calculate the unconsolidated un-drained shear strength of the clay under unconfined conditions.

To perform an unconfined compression test, the sample is extruded from the sampling tube. A cylindrical sample of soil is trimmed such that the ends are reasonably smooth and the length-to-diameter ratio is on the order of two. The soil sample is placed in a loading frame on a metal plate; by turning a crank, the operator raises the level of the bottom plate.

The top of the soil sample is restrained by the top plate, which is attached to a calibrated proving ring. As the bottom plate is raised, an axial load is applied to the sample. The operator turns the crank at a specified rate so that there is constant strain rate. The load is gradually increased to shear the sample, and readings are taken periodically of the force applied to the sample and the resulting deformation. The loading is continued until the soil develops an obvious shearing plane or the deformations become excessive. The measured data are used to determine the strength of the soil specimen and the stress-strain characteristics. Finally, the sample is oven dried to determine its water content. The maximum load per unit area is defined as the unconfined compressive strength, qu.

UCS test is standardized in ASTM D 2166 - standard test method for unconfined compressive strength of cohesive soil. For soils, the un-drained shear strength (Su) is necessary for the determination of the bearing capacity of foundations, dams, etc. The un-drained shear strength (Su) of clays is commonly determined from an unconfined compression test. The un-drained 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 (f = the angle of internal friction). The most critical condition for the soil usually occurs immediately after construction, which represents un-drained conditions, when the un-drained shear strength is basically equal to the cohesion (c).

(a) (b)

Figure 3. 4 UCS test specimens (a) during and (b) after the test

3.3.2.3.Statistical Methods

There are many methods that we can use to develop predictive models that enhance studying the relationships between two or more variables. However, in this study stepwise regression procedures, single as well as multiple regressions were carried out for the selection of most influencing variables. The method of regression analysis is used to develop the line or curve which provides the best fit through a set of data points. The method of least squares is used in order to choose the best fitting line for a set of data. A suitable way of determining how well the regression model performs as a predictor of the dependent variable is to compute the reduction in the sum of squares of deviations that can be attributed to regression variables and this quantity termed the coefficient of determination, R^2. IBM SPSS statistics 20 and Microsoft excel were used during statistical analysis.

3.3.2.4. Methods of Validation

In statistics, regression validation is the process of deciding whether the numerical results quantifying hypothesized relationships between variables, obtained from regression analysis, are acceptable as descriptions of the data. The validation process can involve analyzing the goodness of fit of the regression, analyzing whether the regression residuals are random, and checking whether the model's predictive performance deteriorates substantially when applied to data that were not used in model estimation. In this study validation using determination coefficient, R^2 was used which is one of the methods used to validate the prediction models.

3.4.Chapter Summary

Dilla is one of the towns in Southern Nation Nationalities and People’s Region (SNNPR) with latitude and longitude of 6º24’38” N and 38º18’37” E respectively. The soil types in Dilla include red clay, reddish brown, dark brown and black cotton soils, which are typical examples of residual soils.

Soil samples were collected and different index and engineering laboratory tests were performed.

Table 3. 3 Summary of test methods and number of specimens tested

Test type	Number of specimens tested	Method of test
Natural moisture content	20	Blight, 1997
In-situ unit weight	20	ASTM D2937-00
Atterberg limits	20	ASTM D 4318-98
Soil classification	20	AASHTO M-145
Specific gravity	20	ASTM D 854-98
Particle size determination	20	ASTM D 422
Unconfined compressive strength	20	ASTM D 2166
Validation	18	Validation using R^2

4. RESULTS AND DISCUSSION

Laboratory tests are useful in providing reliable data for estimating the physical characteristics of soils, classification of soils, bearing capacity of the soil, and other engineering properties of soils like compaction characteristics, strength characteristics, so on. In this study, some index properties and unconfined compressive strength test of the lateritic soil were done. Analysis and results of laboratory tests are provided as follows.

4.1. Index Properties

4.1.1. Moisture Content

From natural moisture content result, it has been found that the probable range of moisture content of Dilla town lateritic soils fall in a range of 19% to 35%. The natural moisture content result of each soil samples are shown below.

Table 4. 1 Result of Natural Moisture Content

Test Pit	Sample designation	Depth (m)	Natural moisture content (%)
TP1	TP1_1	2	32.52
	TP1_2	4	26.08
TP2	TP2_1	2	26.02
	TP2_2	4	27.19
TP3	TP3_1	2	31.24
	TP3_2	4	21.26
TP4	TP4_1	2	33.07
	TP4_2	4	32.10
TP5	TP5_1	2	33.73
	TP5_2	4	34.44
TP6	TP6_1	2	24.34
	TP6_2	4	23.86
TP7	TP7_1	2	21.76
	TP7_2	4	21.18
TP8	TP8_1	2	23.91
	TP8_2	4	21.08
TP9	TP9_1	2	20.53
	TP9_2	4	19.96
TP10	TP10_1	2	27.36
	TP10_2	4	28.53

4.1.2. In-Place/ In-Situ Unit Weight

Table below shows the test result of in-situ unit weight. From test result it has been observed that the result of in-situ unit weight is greater at 2m depth than 4m depth. This is an indication of the soils found in far depth from the surface is more dense because of formation of sesquioxides.

Table 4. 2 The summary of In-place unit weight test result

	Sample designation	mass (kg)	Gravitational acceleration (m^2/s)	volume(m^3)	Density (kg/m^3)	In-situ unit weight, γins (kN/m^3)
samples of test pits at 2m	TP1_1	2.298	9.81	0.001424775	1612.89	15.82
	TP2_1	2.571	9.81	0.000132452	1978.67	19.41
	TP3_1	3.259	9.81	0.00212264	1535.35	15.06
	TP4_1	2.063	9.81	0.0016956	1216.68	11.94
	TP5_1	2.234	9.81	0.00151976	1469.97	14.42
	TP6_1	2.511	9.81	0.001424775	1762.38	17.29
	TP7_1	4.270	9.81	0.00212264	2011.65	19.73
	TP8_1	3.593	9.81	0.00180864	1986.58	19.49
	TP9_1	3.451	9.81	0.0016956	2035.27	19.97
	TP10_1	3.753	9.81	0.00212264	1768.08	17.34
samples of test pits at 4m	TP1_2	2.887	9.81	0.0016956	1702.64	16.70
	TP2_2	2.767	9.81	0.001424775	1942.06	19.05
	TP3_2	3.448	9.81	0.0016956	2033.50	19.95
	TP4_2	2.779	9.81	0.00212264	1309.22	12.84
	TP5_2	2.529	9.81	0.001424775	1775.02	17.41
	TP6_2	2.35	9.81	0.001256	1871.02	18.35
	TP7_2	3.116	9.81	0.00151976	2050.32	20.11
	TP8_2	4.203	9.81	0.00212264	1980.08	19.42
	TP9_2	3.557	9.81	0.0016956	2097.78	20.58
	TP10_2	2.76	9.81	0.001424775	1937.15	19.00

The test result of in-situ unit weight is in a range of 12 kN/m^3 to 20.6 kN/m^3. The high value of in-situ unit weight indicates the amount of weathering. As weathering increase the concentration of sesquioxides of iron and aluminum increases. The sesquioxides of iron and aluminum tends to bind the particles of soils together causing aggregation.

4.1.3. Grain Size Analysis

In this study, hydrometer and sieve analysis were performed on all the samples and percent finer against size of soil particle in millimeter on a semi-log scale was plotted. From the curve the proportion and type of soil grains was determined. Particle size distribution of samples was grouped in to sand, silt and clay as summarized in Table 4.1.

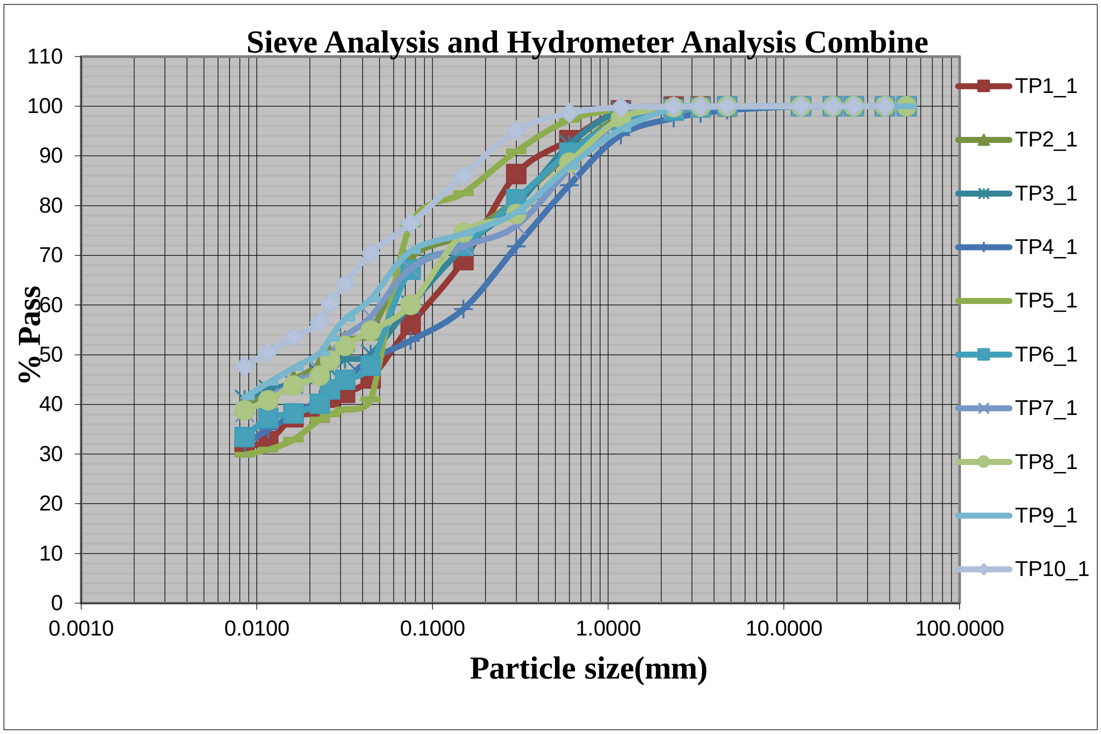

Figure 4. 1 Grain Size Distribution Curve of all Pits at 2m Depth.

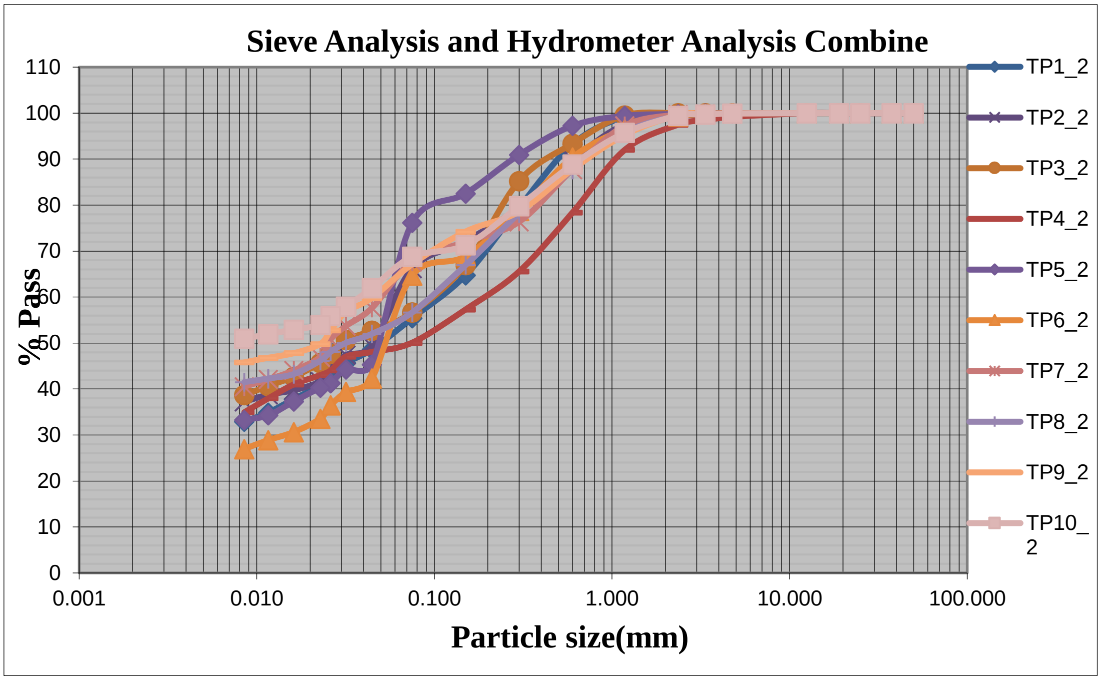

Figure 4. 2 Grain Size Distribution Curve of all Pits at 4m Depth.

From grain distribution curve, grain size such D10, D30, D60, etc., can be obtained. These parameters are used to determine coefficient of uniformity (C_U) and coefficient of concavity (C_C) which used to decide either the soil is well graded, uniformly graded or gap graded. Accordingly, for soil samples under investigation C_U and C_C values are determined and found that the values fall under range of 15 to 75 and 0.06 to 1.97 respectively. So, depending up on the founded result the soil under investigation is well graded soil. The table below shows the summary of the result.

Table 4. 3 Summary of coefficient of uniformity and concavity.

Sno.	Test Pit	Sample designation	Depth (m)	D60	D30	D10	C_U	Cc
1	TP1	TP1_1	2	0.09	0.0085	0.0015	60	0.53518
2		TP1_2	4	0.11	0.007	0.002	55	0.22272
3	TP2	TP2_1	2	0.055	0.004	0.0015	36.6666	0.19393
4		TP2_2	4	0.06	0.005	0.0025	24	0.16666
5	TP3	TP3_1	2	0.075	0.0045	0.0021	35.7142	0.12857
6		TP3_2	4	0.095	0.005	0.002	47.5	0.13157
7	TP4	TP4_1	2	0.16	0.0085	0.004	40	0.11289
8		TP4_2	4	0.2	0.008	0.003	66.6666	0.10666
9	TP5	TP5_1	2	0.06	0.009	0.002	30	0.675
10		TP5_2	4	0.055	0.006	0.0025	22	0.26181
11	TP6	TP6_1	2	0.06	0.008	0.004	15	0.26666
12		TP6_2	4	0.065	0.016	0.002	32.5	1.96923
13	TP7	TP7_1	2	0.05	0.005	0.002	25	0.25
14		TP7_2	4	0.05	0.0045	0.0018	27.7777	0.225
15	TP8	TP8_1	2	0.075	0.0045	0.001	75	0.27
16		TP8_2	4	0.095	0.0028	0.0015	63.3333	0.05501
17	TP9	TP9_1	2	0.04	0.004	0.0015	26.6666	0.26666
18		TP9_2	4	0.044	0.003	0.002	22	0.10227
19	TP10	TP10_1	2	0.03	0.003	0.002	15	0.15
20		TP10_2	4	0.031	0.002	0.0012	25.83333	0.10752

Table 4. 4 Grain Size Distribution

Test Pit	Sample Designation	Depth (m)	Sand (%)	Silt (%)	Clay (%)
TP1	TP1_1	2	44	18	38
	TP1_2	4	45	15	40
TP2	TP2_1	2	30	22	48
	TP2_2	4	33	25	42
TP3	TP3_1	2	40	16	44
	TP3_2	4	43	11	46
TP4	TP4_1	2	46	12	42
	TP4_2	4	50	7	43
TP5	TP5_1	2	24	37	39
	TP5_2	4	24	36	40
TP6	TP6_1	2	33	27	40
	TP6_2	4	35	31	34
TP7	TP7_1	2	32	21	47
	TP7_2	4	32	21	47
TP8	TP8_1	2	40	14	46
	TP8_2	4	43	10	47
TP9	TP9_1	2	29	21	50
	TP9_2	4	33	17	50
TP10	TP10_1	2	30	16	54
	TP10_2	4	31	15	54

From the grain size analysis test result, it has been found that sand percentage is between 24 and 50 %, the values of amount of silt are between 7 and 37% and the amount of clay is between 34 and 54 %.

The weathering process of lateritic soils gives very variable texture for lateritic soils. The concentration of sesquioxides of iron and aluminum causes the concretion of soil particles that increases the unit weight of soils and also wide range distribution of particle size. The low value of in-situ unit weight and silt content of the soil sample taken from test pit 4 indicates that the soil at test pit 4 is not lateritic soil.

Winterkorn (1951) in his study of Laterite soils and stabilization, indicates that the texture of lateritic soil is very variable with silt and clay contents ranging from 12% to over 82%. The results obtained in this study agrees with the conclusion drawn by Winterkorn (1951) except for test pit 3 and 4 at 4m depth.

4.1.4. Atterberg's Limit Test

In this study, oven dried soil sample was used to determine Atterberg's limit. The test results are shown in Table 4.4.

Table 4. 5 Summary of Atterberg's Test Result

Test Pit	Sample Designation	Depth (m)	Liquid limit, LL (%)	Plastic, PL Limit (%)	PI (%)
TP1	TP1_1	2	62	30	32
	TP1_2	4	56	32	24
TP2	TP2_1	2	55	32	23
	TP2_2	4	48	23	25
TP3	TP3_1	2	57	31	26
	TP3_2	4	52	33	19
TP4	TP4_1	2	58	46	12
	TP4_2	4	56	45	11
TP5	TP5_1	2	60	42	18
	TP5_2	4	53	43	10
TP6	TP6_1	2	53	30	23
	TP6_2	4	51	31	20
TP7	TP7_1	2	54	31	23
	TP7_2	4	53	29	24
TP8	TP8_1	2	56	29	27
	TP8_2	4	55	35	20
TP9	TP9_1	2	59	36	23
	TP9_2	4	58	38	20
TP10	TP10_1	2	59	35	24
	TP10_2	4	54	31	23

The data in table 4.4 shows a liquid limit, plastic limit and plasticity index ranges of 48–62%, 23 – 46% and 10 – 32 % respectively. The higher percentage plasticity index, the higher potential to shrink as the soil undergoes moisture content fluctuations. Although the results obtained in this study is more or less similar with the results obtained by Erdil (1976) for Hawaiin soils, the effect of sample preparation prior to testing makes difficult to reproduce the test result. Typical liquid limit graphs are shown in **Appendix A.**

4.1.5. Classification of the Soil Samples According to AASHTO

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 (0.075mm) sieve. Soils of which more than 35% pass through the No.200 (0.075mm) sieve are classified under groups A-4, A-5, A-6 and A-7. These soils are mostly silt and clay-type materials.

Table 4. 6 Classification of soil sample according to AASHTO standard

Test Pits	Sample Designation	Liquid limit (LL)	Plastic limit (PL)	Plasticity index (PI)	% of passing		Group Index (GI)	Classification according to AASHTO
					2mm	0.075mm		
TP1	TP1_1	62	34	28	100	56.00	6	A-7-5(6)
	TP1_2	62	34	28	100	55.00	13	A-7-5(13)
TP2	TP2_1	55	32	23	100	70.00	15	A-7-5(15)
	TP2_2	56	32	24	100	67.00	15	A-7-5(15)
TP3	TP3_1	57	31	26	100	60.00	14	A-7-5(14)
	TP3_2	60	33	27	100	57.00	14	A-7-5(14)
TP4	TP4_1	58	46	13	100	54.00	7	A-7-5(7)
	TP4_2	52	33	19	100	50.00	7	A-7-5(7)
TP5	TP5_1	60	42	18	100	76.00	15	A-7-5(15)
	TP5_2	56	45	11	100	76.00	12	A-7-5(12)
TP6	TP6_1	53	30	22	100	67.00	12	A-7-5(12)
	TP6_2	53	43	10	100	65.00	8	A-7-5(8)
TP7	TP7_1	54	31	24	100	68.00	15	A-7-5(15)
	TP7_2	53	29	24	100	68.00	14	A-7-6(14)
TP8	TP8_1	56	29	26	100	60.00	14	A-7-6(14)
	TP8_2	55	35	20	100	57.00	10	A-7-5(10)
TP9	TP9_1	59	36	23	100	71.00	16	A-7-5(16)
	TP9_2	58	38	20	100	67.00	13	A-7-5(13)
TP10	TP10_1	59	35	24	100	70.00	16	A-7-5(16)
	TP10_2	54	31	23	100	69.00	14	A-7-5(14)

If lateritic soils contain gravel, the amount of passing the no. 200 sieve is less than 35% and classification is usually in the A-2 group. If there is little or no gravel, more than 35% passes the no. 200 sieve, and the soil is classified as either A-6 or A-7 (Lyon, 1971). The soils under investigation contains no gravel, more than 35% passes no. 200 sieve, accordingly, the soils of the study area falls in A-7-5 and A-7-6 with different group index value.

4.1.6. Specific Gravity

Specific gravity of soil solids, Gs, is the ratio of the unit weight of solids in the soil to the unit weight of water. Specific gravity of soil solids is used for performing weight-volume calculations in soils. The average specific gravity (Gs) values of the soils were found to be 2.71.

Table 4. 7 Summary of Specific Gravity Test Result

Sno.	Test Pit	Sample designation	Depth (m)	Specific Gravity, Gs
1	TP1	TP1_1	2	2.8
2		TP1_2	4	2.75
3	TP2	TP2_1	2	2.7
4		TP2_2	4	2.85
5	TP3	TP3_1	2	2.6
6		TP3_2	4	2.64
7	TP4	TP4_1	2	2.6
8		TP4_2	4	2.51
9	TP5	TP5_1	2	2.65
10		TP5_2	4	2.64
11	TP6	TP6_1	2	2.85
12		TP6_2	4	2.8
13	TP7	TP7_1	2	2.75
14		TP7_2	4	2.8
15	TP8	TP8_1	2	2.65
16		TP8_2	4	2.8
17	TP9	TP9_1	2	2.75
18		TP9_2	4	2.65
19	TP10	TP10_1	2	2.7
20		TP10_2	4	2.64

The values of specific gravity of a soil sample shows the weighted average of the specific gravities of the minerals which comprises the soil. Higher specific gravity values indicates the presence of relatively larger amounts of high specific gravity minerals in the soil, and suggests a more weathered soil (Erdil, 1976).

Erdil (1976) shows that specific gravity has a good positive relation with extractable iron content and could be considered as a measure of degree of weathering. In relatively less weathered soils the formation of clay mineral is the major event taking place in the course of weathering. Further weathering, however, causes an increase in the content of sesquioxides and diminishes active role of clay content. Sesquioxides of iron and aluminum are known as very active binding agents to cause aggregation. This effect tends to increase the specific gravity of soil.

The specific gravity for lateritic soils falls within a range of 2.6 and 3.4 as reported by Lyon Associates in 1971. Although the average value of specific gravity value of the soils of this study agrees the range reported by Lyon Associates, the specific value of test pit 4 at 4m is out of the range. This is one of the indications of the soil sample taken from test pit 4 is not lateritic soil.

4.2.Engineering Property

4.2.1. Unconfined Compression Test

The strength parameters obtained in this test is used during analysis of un-drained situations such as short term stability and quick loading condition.

Table 4. 8 Test result of Unconfined Compressive Strength

Test Pit	Sample designation	Depth (m)	Unconfined compressive strength qu (kPa)	Un-drained cohesive strength, Cu (kPa)
TP1	TP1_1	2	74.86	37.43
	TP1_2	4	141.34	70.67
TP2	TP2_1	2	245.65	122.83
	TP2_2	4	288.32	144.16
TP3	TP3_1	2	80.77	40.38
	TP3_2	4	239.09	119.55
TP4	TP4_1	2	70.55	35.27
	TP4_2	4	63.12	31.56
TP5	TP5_1	2	73.31	36.66
	TP5_2	4	118.26	59.13
TP6	TP6_1	2	150.55	75.28
	TP6_2	4	174.05	87.03
TP7	TP7_1	2	276.88	138.44
	TP7_2	4	292.59	146.30
TP8	TP8_1	2	294.60	147.30
	TP8_2	4	323.07	161.53
TP9	TP9_1	2	363.98	181.99
	TP9_2	4	380.91	190.46
TP10	TP10_1	2	151.17	75.59
	TP10_2	4	238.95	119.47

If the value of UCS test result falls in a range of 0 to 24kPa, the sample is classified as very soft. Similarly, if the result falls in a range of 24 kPa to 48 kPa classified as soft, if it falls between 48 kPa to 96 kPa classified as medium, if it falls in a range of 96 kPa to 192 kPa classified as stiff, if it falls in a range of 192 kPa to 383 kPa, classified as very stiff and if it falls above 383kPa it is classified as hard (Braja, 1997 and Tibebu, 2008). The UCS test result of soil samples under investigation falls in a wide range starting from medium to very stiff and shows strong positive relation with in place unit weight.

4.3. Summary

Laboratory tests are useful in providing reliable data for estimating the physical characteristics of soils, classification of soils, bearing capacity of the soil, and other engineering properties of soils. In this study, index properties and engineering property of lateritic soil were conducted and their test results were presented. Particle size determination, Moisture content, Atterberg's limit test, hydrometer analysis, specific gravity and in-situ unit weight tests are tests which show the index property of a soil and unconfined compressive strength is among the tests which shows the engineering properties of the soil.

From the grain size analysis test result, it has been found that sand percentage is between 24 and 50 %, the values of amount of silt are between 7 and 37% and the amount of clay is between 34 and 54 %. Moisture contents of the soil samples were determined in the laboratory according to ASTM and Blight. The method allows drying the sample to an oven temperature of 105^0C and oven temperature of 50^0C with maximum relative humidity (RH) of 30% or equivalently air-drying. From natural moisture content result, it has been found that the probable range of moisture content of Dilla town lateritic soils fall in a range of 19% to 35%. The test result of in-situ unit weight is in a range of 12 kN/m^3 to 20.6 kN/m^3. The high value of in-situ unit weight indicates the amount of weathering (Lyon, 1971).

The specific gravity for lateritic soils falls within a range of 2.6 and 3.4 as reported by Lyon Associates in 1971. Although the average value of specific gravity value of the soils of this study agrees the range reported by Lyon Associates, the specific value of test pit 4 at 4m is out of the range. The test result of liquid limit, plastic limit and plasticity index ranges in 48–62%, 23 – 46% and 10 – 32 % respectively. The higher percentage plasticity index, the higher potential to shrink as the soil undergoes moisture content fluctuations. The soils under investigation contains no gravel, more than 35% passes no. 200 sieve, accordingly, the soils of the study area falls in A-7-5 and A-7-6 with different group index value.

The result of unconfined compressive strength of soil samples under investigation ranges from 70kPa to 381kPa, which means it falls in a consistency range of medium to very stiff.

4.4. Statistical Analysis

4.4.1. Introduction

Regression analysis is a statistical technique that is concerned with how the values of dependent variables are related with the corresponding values of an independent variables. A regression model that contains more than one independent variable is called multiple regression models. Alternatively, Regression model containing one independent variable is termed as single regression model.

Regression analysis is very useful in the field of engineering and science in modeling and investigating relationships between two or more variables. The method of regression analysis is used to develop the line or curve which provides the best fit through a set of data points. This basic approach is applicable in situations ranging from single linear regression to more sophisticate nonlinear multiple regressions. The best fit model could be in the form of linear, parabolic or logarithmic trend. A linear relationship is usually practiced in solving different engineering problems because of its simplicity.

Fitting a regression model requires several assumptions. Estimation of the model parameters requires the assumption that, the residuals (actual values less estimated values) corresponding to different observations are uncorrelated random variables with zero mean and constant variance. Tests of hypotheses and interval estimation require that the errors be normally distributed. In addition, one assumes that the order of the model is correct; that is, if one fits a simple linear regression model, one is assuming that the phenomenon actually behaves in a linear or first order manner (Douglas and George, 2003). During regression analysis, a regression model with higher value of R^2 (coefficient of determination), which quantifies the proportion of the variance of one variable by the other, is usually accepted.

A convenient way of measuring how well the regression model performs as a predictor of the dependent variable is to compute the reduction in the sum of squares of deviations that can be attributed to regressor variables and this quantity termed the coefficient of determination, R^2. The value of R^2 is always between 0 and 1, because R is between -1 and +1, whereby a negative value of R indicates inverse relationship and positive value implies direct relationship. Many problems in engineering require that we decide whether to accept or reject a statement about some correlations. A number of techniques can be used to judge the adequacy of a regression model some of which are standard error (P), R-squared value (R^2), R^2-adjusted and the t-test (Douglas and George, 2003).

4.4.2. Regression Analysis

4.4.2.1. Single Regression Analysis

I. Unconfined Compressive Strength (UCS) as a Function of Percent passing sieve #200.

Total of eighteen data pairs were used. The resulting regression analysis after correlating UCS with P_{200} is expressed by the following single linear equation with its corresponding R squared value.

$$q_u = 202.423 + 0.225 * P_{200} \qquad \text{with} \quad R^2 = 0.001, \; n = 18$$

The details of the statistical out-put indicates that the relationship developed between P_{200} and UCS is not significant ($P>0.05$) as shown in Table B-1.1 of Appendix B-1.

II. Unconfined Compressive Strength (UCS) as a Function of Liquid Limit (LL)

Total of eighteen data pairs were used. The resulting regression analysis after correlating UCS with LL is expressed by the following single linear equation with its corresponding R squared value.

$$q_u = 794.486 - 10.169 * LL \qquad \text{with} \quad R^2 = 0.095, \; n = 18$$

The details of the statistical out-put indicates that the relationship developed between LL and UCS is not significant ($P>0.05$) as shown in Table B-1.2 of Appendix B-1.

III. Unconfined Compressive Strength (UCS) as a Function of Plastic Limit (PL)

Total of eighteen data pairs were used. The resulting regression analysis after correlating UCS with PL is expressed by the following single linear equation with its corresponding R squared value.

$$q_u = 411.601 - 5.646*PL \qquad \text{with} \quad R^2 = 0.074 \;, \quad n = 18$$

The details of the statistical out-put indicates that the relationship developed between PL and UCS is not significant (P>0.05) as shown in Table B-1.3 of Appendix B-1.

IV. Unconfined Compressive Strength (UCS) as a Function of Plasticity Index (PI)

Total of eighteen data pairs were used. The resulting regression analysis after correlating UCS with PI is expressed by the following single linear equation with its corresponding R squared value.

$$q_u = 182.679 + 1.546*PI \qquad \text{with} \quad R^2 = 0.006, \; n = 18$$

The details of the statistical out-put indicates that the relationship developed between PI and UCS is not significant (P>0.05) as shown in Table B-1.4 of Appendix B-1.

V. Unconfined Compressive Strength (UCS) as a Function of Natural Moisture Content (w_n)

Total of eighteen data pairs were used. The resulting regression analysis after correlating UCS with w_n is expressed by the following single linear equation with its corresponding R squared value.

$$q_u = 666.159 - 17.382* w_n \qquad \text{with} \quad R^2 = 0.679, \; n = 18$$

The details of the statistical out-put indicates that the relationship developed between w_n and UCS is significant (P>0.05) as shown in Table B-1.5 of Appendix B-1.

VI. Unconfined Compressive Strength (UCS) as a Function of Specific Gravity (Gs)

Total of eighteen data pairs were used. The resulting regression analysis after correlating UCS with Gs is expressed by the following single linear equation with its corresponding R squared value.

$$q_u = -246.736 + 170.330*Gs \qquad \text{with} \quad R^2 = 0.019, \; n = 18$$

The details of the statistical out-put indicates that the relationship developed between Gs and UCS is not significant (P>0.05) as shown in Table B-1.6 of Appendix B-1.

VII. Unconfined Compressive Strength (UCS) as a Function of In-situ Unit Weight (γ_{ins})

Total of eighteen data pairs were used. The resulting regression analysis after correlating UCS with γ_{ins} is expressed by the following exponential equation with its corresponding R squared value.

$$q_u = 1.1e^{0.282\gamma ins} \qquad \text{with} \quad R^2 = 0.927, \ n = 18$$

The details of the statistical out-put indicates that the relationship developed between γ_{ins} and UCS is highly significant ($P<0.05$) as shown in Table B-1.7 of Appendix B-1.

Regression analyses were conducted incorporating Gradation, Atterberg limits, natural moisture contents, specific gravity and in place unit weight results to unconfined compressive strength results. Based on the significant standard error (P) and coefficient of determination (R^2), the developed relation can be grouped in to two. The first group (liquid limit, plastic limit, specific gravity, and plasticity index) indicates weak relation with unconfined compressive strength; while the second group (natural moisture content and in-situ unit weight) indicates strong relation with unconfined compressive strength results.

4.4.2.2. Multiple Linear Regression Analysis

The general purpose of multiple regression is to learn more about the relation between several independent or predictor variables and a dependent or criterion variable. A predictor variable which shows a weak relation in a single correlation may give a better relation when it combined with other predictors. In this study, regression of unconfined compressive strength with a combination of different independent variables have been checked and only a combination of independent variables which show strong correlation with unconfined compressive strength are discussed as follows:

Model-1: Unconfined Compressive Strength (q_u) as a Function of In-situ Unit Weight (γ_{ins}), and Natural Moisture Content (w_n).

The resulting regression analysis after correlating q_u with γ_{ins} and w_n is expressed by the following multiple linear equations with its corresponding R squared value and coefficients correlation.

$$q_u = -421.843 - 4.191w_n + 40.870\gamma_{ins} \qquad \text{with } R^2 = 0.866$$

Table 4. 9 Coefficients correlation of model-1

Model			In-situ Unit Weight	Natural Moisture Content
1	Correlations	In-situ Unit Weight	0.822	1.000
		Natural Moisture Content	1.000	0.822

a. Dependent Variable: Unconfined Compressive Strength

The details of the statistical out-put of Model-1 indicates that the relationship developed between q_u with γ_{ins} and w_n is significant ($P<0.0005$) which can predict the outcome variable. The R^2 value of the multiple regression analysis shows that predicting q_u as a function of combined result of γ_{ins} and w_n gives a better accuracy than predicting using w_n, as single predicting variable. For further reference, the detail of Model-1 is shown in Table B-2.1 of Appendix B-2.

Model-2: Unconfined Compressive Strength (q_u) as a function of Natural Moisture Content (w_n), In-Situ Unit Weight (γ_{ins}) and Specific Gravity (Gs).

The resulting regression analysis after correlating q_u with w_n, γ_{ins} and Gs is expressed by the following multiple linear equations with its corresponding R squared value and coefficients correlation.

$qu = -311.774 + 40.395\gamma_{ins} - 4.517w_n - 34.136Gs$ with $R^2 = 0.867$

Table 4. 10 Coefficients correlation of model-2

Model			Specific Gravity	In-situ Unit Weight	Natural Moisture Content
2	Correlations	Specific Gravity	1.000	0.190	0.319
		In-situ Unit Weight	0.190	1.000	0.825
		Natural Moisture Content	0.319	0.825	1.000

a. Dependent Variable: Unconfined Compressive Strength

The details of the statistical out-put of Model-2 indicates that the relationship developed between q_u with Gs, γ_{ins} and w_n is significant ($P<0.0005$) which can predict the outcome variable. The R^2 value of the multiple regression analysis shows that predicting qu as a function of combined result of Gs, γ_{ins} and w_n gives a better accuracy than predicting using Gs and w_n, as single predicting variables. For further reference, the detail of Model-2 is shown in Table B-2.2 of Appendix B-2.

Model-3: Unconfined Compressive Strength (q_u) as a Function of Natural Moisture Content (w_n), Percent Passing Sieve #200 (P_{200}) and In-situ unit weight (γ_{ins}).

The resulting regression analysis after correlating q_u with w_n, P_{200} and γ_{ins} is expressed by the following multiple linear equations with its corresponding R squared value and coefficients correlation.

$q_u = -419.240 + 38.499\gamma_{ins} - 5.311\boldsymbol{w_n} + 1.064P_{200}$ With $R^2=0.870$

Table 4. 11 Coefficients correlation of model-3

Model			Percent of Passing Sieve #200	In-situ Unit Weight	Natural Moisture Content
3	Correlations	Percent of Passing Sieve #200	1.000	-0.369	-0.431
		In-situ Unit Weight	-0.369	1.000	0.848
		Natural Moisture Content	-0.431	0.848	1.000

a. Dependent Variable: Unconfined Compressive Strength

The details of the statistical out-put of Model-3 indicates that the relationship developed between q_u with γ_{ins}, P_{200} and w_n is significant ($P<0.0005$) which can predict the outcome variable. The R^2 value of the multiple regression analysis shows that predicting q_u as a function of combined result of γ_{ins}, P_{200} and w_n gives a better accuracy than predicting using P_{200} and w_n, as single predicting variables. For further reference, the detail of Model-3 is shown in Table B-2.3 of Appendix B-2.

Model-4: Unconfined Compressive Strength (q_u) as a Function of Natural Moisture Content (w_n), In-situ Unit Weight (γ_{ins}) and Plasticity Index (PI).

The resulting regression analysis after correlating q_u with w_n, γ_{ins} and PI is expressed by the following multiple linear equations with its corresponding R squared value and coefficients correlation:

$q_u = -457.984 + 41.519\gamma_{ins} - 3.864w_n - 0.7108PI$ $R^2 = 0.868$

Table 4. 12 Coefficients correlation of model-4

Model			Plasticity Index	In-situ Unit Weight	Natural Moisture Content
4	Correlations	Plasticity Index	1.000	0.192	0.244
		In-situ Unit Weight	0.192	1.000	0.829
		Natural Moisture Content	0.244	0.829	1.000

a. Dependent Variable: Unconfined Compressive Strength

The details of the statistical out-put of Model-4 indicates that the relationship developed between q_u with γ_{ins}, w_n and PI is significant ($P<0.0005$) which can predict the outcome variable. The R^2 value of the multiple regression analysis shows that predicting q_u as a function of combined result of γ_{ins}, w_n and PI gives a better accuracy than predicting using w_n and PI as single predicting variables. For further reference, the detail of Model-4 is shown in Table B-2.4 of Appendix B-2.

Model-5: Unconfined Compressive Strength (q_u) as a Function of In-situ Unit Weight (γ_{ins}), Specific Gravity (Gs), Plasticity Index (PI), Natural Moisture Content (w_n)

The resulting regression analysis after correlating q_u with γ_{ins}, Gs, PI and w_n is expressed by the following multiple linear equations with its corresponding R squared value and coefficients correlation:

$q_u = -357.513 + 41.058\gamma_{ins} - 4.173w_n - 0.671PI - 30.543Gs$; $R^2=0.868$

Table 4. 13 Coefficients correlation of model-5

Model			Specific Gravity	Plasticity Index	In-situ Unit Weight	Natural Moisture Content
5	Correlations	Specific Gravity	1.000	0.083	0.202	0.329
		Plasticity Index	0.083	1.000	0.204	0.257
		In-situ Unit Weight	0.202	0.204	1.000	0.833
		Natural Moisture Content	0.329	0.257	0.833	1.000

a. Dependent Variable: Unconfined Compressive Strength

The details of the statistical out-put of Model-5 indicates that the relationship developed between q_u with γ_{ins}, w_n, Gs and PI is significant ($P<0.0005$) which can predict the outcome variable. The R^2 value of the multiple regression analysis shows that predicting qu as a function of combined result of γ_{ins}, w_n, Gs and PI gives a better accuracy than predicting using w_n, Gs and PI as single predicting variables. For further reference, the detail of Model-5 is shown in Table B-2.5 of Appendix B-2.

Model-6: Unconfined Compressive Strength (q_u) as a Function of In-situ Unit Weight (γ_{ins}), Liquid Limit (LL), Natural Moisture Content (w_n) and Percent of Passing Sieve #200.

The resulting regression analysis after correlating q_u with γ_{ins}, LL, w_n and P_{200} is expressed by the following multiple linear equations with its corresponding R squared value and coefficients correlation:

$q_u = -693.119 + 40.482\gamma_{ins} - 5.523w_n + 3.649LL + 1.612P_{200}$ with $R^2=0.879$

Table 4. 14 Coefficients correlation of model-6

Model			Percent of Passing Seive #200	In-situ Unit Weight	Liquid Limit	Natural Moisture Content
6	Correlations	Percent of Passing Seive #200	1.000	-0.275	0.326	-0.424
		In-situ Unit Weight	-0.275	1.000	0.202	0.819
		Liquid Limit	0.326	0.202	1.000	-0.054
		Natural Moisture Content	-0.424	0.819	-0.054	1.000

a. Dependent Variable: Unconfined Compressive Strength

The details of the statistical out-put of Model-6 indicates that the relationship developed between q_u with γ_{ins}, LL, w_n and P_{200} is significant ($P<0.0005$) which can predict the outcome variable. The R^2 value of the multiple regression analysis shows that predicting q_u as a function of combined result of γ_{ins}, LL, w_n and P_{200} gives a better accuracy than predicting using LL, w_n and P_{200} as single predicting variables. For further reference, the detail of Model-6 is shown in Table B-2.6 of Appendix B-2.

4.4.2.3. Summary of Regression Analysis and Development of Correlations

The purpose of this subtopic was to develop relationships between the index properties of lateritic soils specimens and unconfined compressive strength (UCS) specimens. Unconfined compressive strength (q_u) was plotted as a function of liquid limit, plastic limit, plasticity index, specific gravity, natural moisture content and in place unit weight. Some of the properties listed shows negligible relation with unconfined compressive strength while others shows good and strong relations.

Although the correlation between liquid limit, plastic limit and plasticity index with unconfined compressive strength is weak, they shows negative relation. While, natural moisture content has good negative relation with unconfined compressive strength. In place unit weight has strong and positive linear relationship with unconfined compressive strength. The scatter plot of unconfined compressive strength and Atterberg limits shows that there is no or little dependence of dependent (q_u) variable on independent (LL, PL and PI) variables. This is due to variation of

Atterberg limits of these soils resulting from sample preparation and handling which disrupt natural structure of the soils.

In regards to multiple regression, in-situ unit weight, specific gravity, plasticity index and natural moisture content in combination gives strong relationship with unconfined compressive strength. A summary of the regression analyses are shown in Table 4.15.

Table 4. 15 Summary of the regression analyses

Regression Type	Developed Model	R	R^2	Sig. (P)	Significance Order
Single Regression Model	qu= 202.423 + 0.225*P200	0.03	0.001	>0.05	13
	qu= 794.486 - 10.169*LL	0.31	0.095	>0.05	9
	qu= 411.601 - 5.646*PL	0.27	0.074	>0.05	10
	qu= 182.679 + 1.546*PI	0.08	0.006	>0.05	12
	qu= 666.159 - 17.382* wn	0.82	0.679	<0.05	8
	qu= -246.736 + 170.330*Gs	0.14	0.019	>0.05	11
	$q_u = 1.1e^{0.282\Upsilon ins}$	0.96	0.927	<0.0005	1
Multiple Linear Regression Model	qu = -421.843- 4.191wn + 40.870γins	0.93	0.866	<0.0005	7
	qu = -311.774+ 40.395γ_{ins}- 4.517w_n-34.136Gs	0.93	0.867	<0.0005	6
	qu = -419.240+ 38.499γins-5.311wn+ 1.064P200	0.93	0.87	<0.0005	3
	-457.984 + 41.519γins -3.864wn -0 .7108PI	0.93	0.868	<0.0005	4
	qu = -357.513 + 41.058γins - 4.173wn- 0.671PI-30.543Gs	0.93	0.868	<0.0005	5
	qu = -693.119 + 40.482γins -5.523wn + 3.649LL+ 1.612P200	0.94	0.879	<0.0005	2

4.5. Discussion on Developed Models

4.5.1. Single Regression

After carefully studying the data trend on the scatter plot and applying different statistical methods, single regression shows that unconfined compression strength is influenced by liquid limit, plastic limit, plasticity index, percent passing sieve #200 and specific gravity, insignificantly by achieving coefficient of determination less than 15%. This is the indication of weak correlation between them.

The attempted statistical methods indicates that unconfined compressive strength is significantly affected by in-situ unit weight and natural moisture content. The coefficient of determination, 0.679 for natural moisture content and 0.927 for in-situ unit weight, of the statistical methods shows that in-situ unit weight and natural moisture content have strong relation with unconfined compressive strength. The equations presented in section 4.4.2.1 with model number V and VII shows that the correlation developed between natural moisture content and in-situ unit weight with unconfined compressive strength is strong relationship.

4.5.2. Multiple Regression

The multiple regression of UCS with in-situ unit weight and natural moisture indicates that UCS has strong correlation with the parameters by achieving coefficient of determination of 0.866 with 18 samples. The detail of the developed model is presented in section 4.4.2.2. The combination of natural moisture content, in-situ unit weight and specific gravity also have strong relation with unconfined compressive strength with coefficient of determination 0.867. The beta value of standardized coefficients in statistical analysis shows that the developed models are governed by in-situ unit weight. The detail for each model is shown in appendix B.

4.6. Validating Developed Equation

To test for validation of the developed correlation equation, collection of new data to check model predictions is necessary. Among different developed models (summarized on table 4.15), the one with maximum R^2 value and minimum standard error is selected. Accordingly, two equations $q_u = 1.1e^{0.282\gamma ins}$ and qu = -421.843- $4.191w_n$ + $40.870\gamma_{ins}$ are chosen as the best equations to be validate as the predictive models. The results are summarized in table 4.16.

Table 4. 16 The summary of actual and predicted model results

Sample Designation	Actual qu, kN/m^2	Natural Moisture Content(w_n)	In-situ Unit Weight (γ_{ins})	Predicted qu, kN/m^2	
				Model-A	Model-B
TP1_1	98.02	32.52	15.82	95.31917	88.52774
TP1_2	174.59	26.08	16.70	122.1847	151.504
TP2_1	250.73	26.02	19.41	262.2141	262.4274
TP2_2	232.81	27.19	19.05	236.9541	242.8434
TP3_1	97.12	31.24	15.06	76.91795	62.80613
TP3_2	279.6	21.26	19.95	305.1554	304.3564
TP4_1	71.1	33.73	14.42	64.19091	26.15613
TP4_2	135.9	34.44	17.41	149.2694	145.4849
TP5_1	147.12	24.34	17.29	144.1426	182.7488
TP5_2	198.85	23.86	18.35	194.6757	228.3162
TP6_1	288.98	21.76	19.73	287.2542	293.4995
TP6_2	336.44	21.18	20.11	319.6948	311.4376
TP7_1	293.45	23.91	19.49	268.0068	274.4373
TP7_2	263.63	21.08	19.42	263.235	283.6941
TP8_1	313.86	20.53	19.97	306.6526	308.1252
TP8_2	366.32	19.96	20.58	364.5478	335.5784
TP9_1	145.29	27.36	17.34	146.4326	172.3764
TP9_2	232.74	28.53	19.00	233.7554	235.2576

** Model-A qu= $1.1e^{0.282\gamma ins}$

** Model-B qu =-421.843- 4.191w_n + 40.870γ_{ins}

Table 4.16 shows the detail on validation of selected best models among different developed models with newly collected sample test results. Even though Model-B is less strong relative to Model-A, it is good enough to predict UCS of the subject area. The following figures are the plot of the actual value of unconfined compressive strength and predicted values of unconfined compressive strength using the selected predictive models.

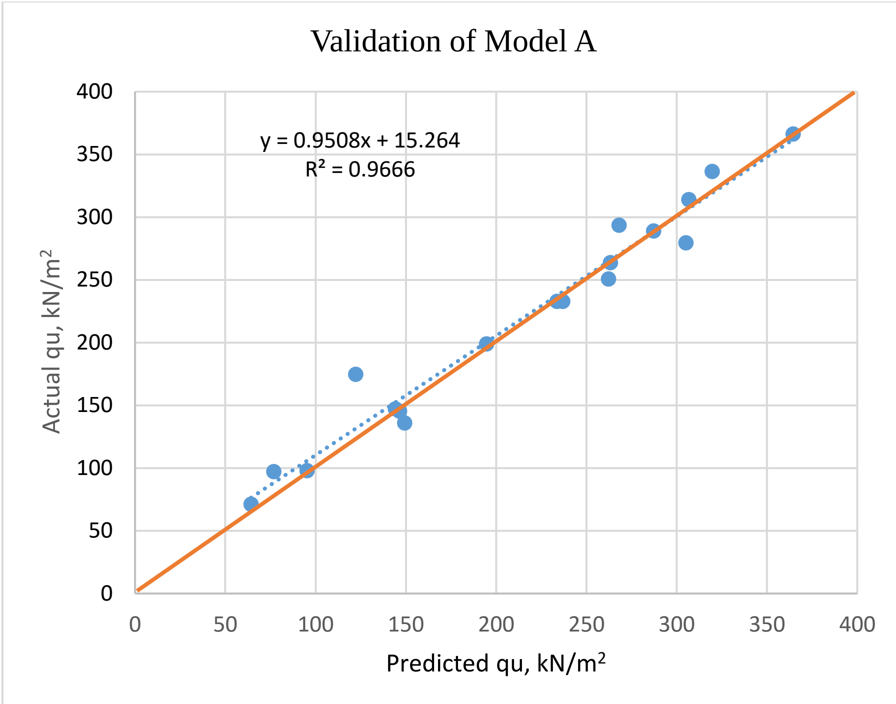

Figure 4. 3 Plot of actual test results and predicted unconfined compressive strength using Model-A

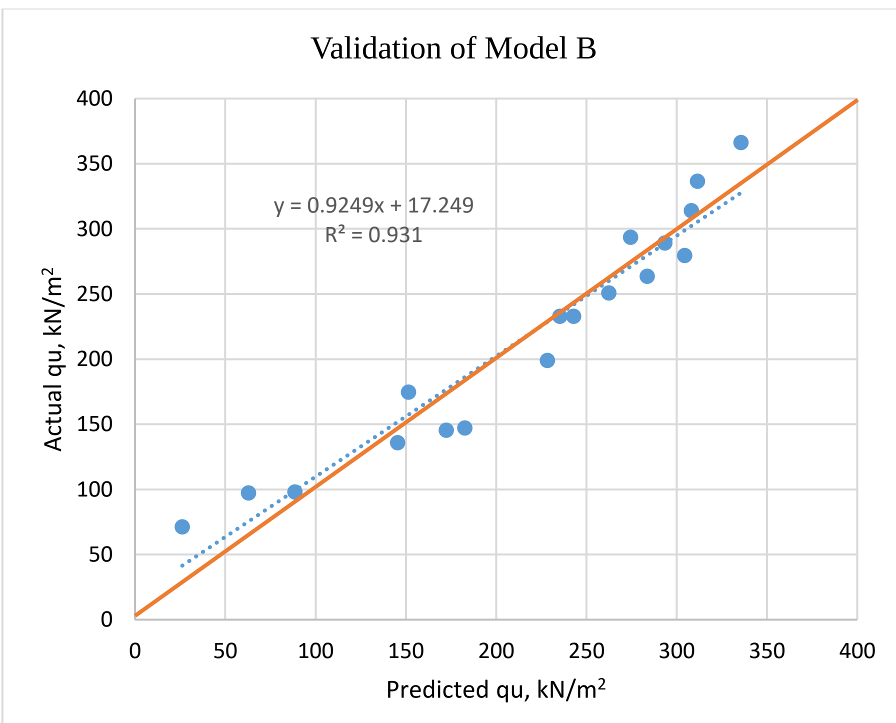

Figure 4. 4 Plot of actual test results and predicted unconfined compressive strength using Model-B

5. CONCLUSION AND RECOMMENDATION

5.1. Conclusion

The purpose of this research was to relate the unconfined compressive strength to index properties of lateritic soils found in Dilla town. The index properties that were used in this study are liquid limit, plastic limit, plasticity index, natural moisture content, specific gravity and in-situ unit weight. The samples were taken from randomly selected scattered places in the town. A total of 20 disturbed and 20 undisturbed samples were taken from ten test pits with four-meter depth. The specimens were classified according to AASHTO standard, and fall under A-7-5 and A-7-6. The color of samples was red, reddish and reddish-brown.

From the grain size analysis test result, it has been found that sand percentage is between 24 and 50 %, the amount of silt is between 7 and 37% and the amount of clay is between 34 and 54 %. The natural moisture content of the soils in the study area ranges from19% to 35% with plasticity index range of 10% to 32 %. Finally, the test results of in-situ unit weight fall in a range of 15% to 20.58% with average specific gravity value of 2.71.

Quantitative relationships were developed between UCS and index properties of the soil samples. Unit weight was a better predictor of UCS than any other index properties. The coefficient of determination (0.915) between UCS and in-situ weight indicates strong positive linear relation between them. A better correlation were developed in multiple regression (UCS as a function of γ_{ins} and w_n) achieving the coefficient of determination 0.842.

The main conclusions that could be drawn are as follows:

- It is possible to determine un-drained cohesive strength from the developed models by multiplying the result obtained by half.
- In-situ unit weight and natural moisture content have strong relation with UCS while Atterberg's limits and grain size distribution have no significant effect.
- Among the developed predictive models Model-A: $q_u = 6.1025\ \gamma_{ins}^2 - 165.11\gamma_{ins} + 1176$ and Model-B: $q_u = -7.410 - 8.110w_n + 23.991\gamma_{ins}$ are best predictive models.

5.2. Recommendation

Although lateritic soils have wide range of use, historically there has been insufficient investigation of the engineering properties of the lateritic soils in Dilla. To use the lateritic soils as subgrade for road construction or to design infrastructure that will be supported by the lateritic soils in Dilla, significantly more research will be needed to help designers with the difficult decision of assessing the strength and stiffness of lateritic soils.

In general the following recommendations and future works could be drawn.

- To increase the accuracy of the predictive model that should be developed the number of samples must be increased. The number of samples in this research is limited and hence the developed model is used only for preliminary design.
- This study has shown there is significant effect of water on the properties of the soils. For future work it is recommended to study the detail compaction characteristics of the soils of the area.
- The soils of the area have no problem of expansion, rather the problem of fluctuation of strength characteristics when in contact with water. For future study, stabilization of lateritic soils with plastic fiber is highly recommended.

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APENDIX A: DETAILES OF LABORATORY TEST RESULTS AND ANALYSIS

APPENDIX A-1: Index Properties Test Results and Analysis

Company Name:

HAWASSA INSTITUTE OF TECHNOLOGY

Doucment Title
Laboratory Soil Test Result Reporting Format

Page No.:
Page 1 of 1

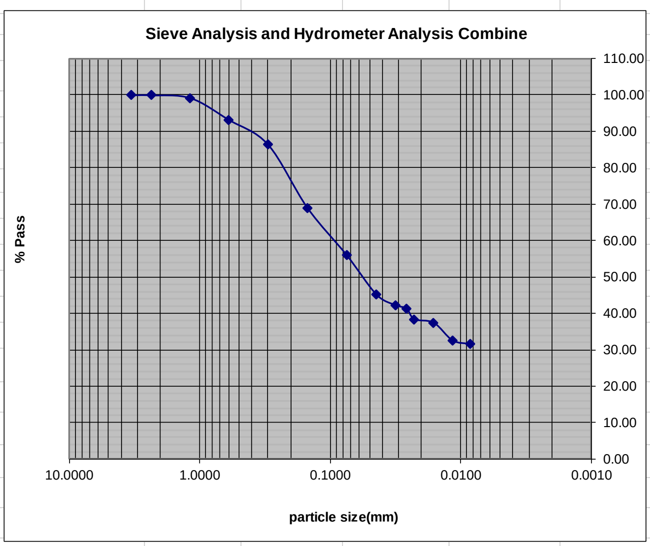
PROJECT:	THESIS	**ZERO CORRECTION**	3.00
TESTED BY:	**SAMUEL A.**	**MENISCUS CORRECTION**	**0.50**
SAMPLE OF:	TP1_1		
STATION:	DILLA TTC		
TEST TYPE:	Hydrometer analysis & Sieve Analysis (AASHTO T-88)		
HYDOMETER NO.	152H		
DISPERSING AGEN	$NaPO_3$		

Hydrometer analysis Data

Specific Gravity= 2.8

Time (minutes)	Hydrometer Reading	Temp.	Corrected H.Reading R'	Corrected H.Reading R"	Correction factor(a)	Effe. Depth of Hydrometer(L)	Values of K	Diameter of soil Particle(mm)	% finer,P
1	25.5	21.5	26.00	23.3	0.9700	12.00	0.01284	0.0445	45.20
2	24	21.5	24.50	21.8	0.9700	12.30	0.01284	0.0318	42.29
3	23.5	21.5	24.00	21.3	0.9700	12.40	0.01284	0.0261	41.32
4	22	21.5	22.50	19.8	0.9700	12.60	0.01284	0.0228	38.41
8	21.5	21.5	22.00	19.3	0.9700	12.70	0.01284	0.0162	37.44
16	19	21.5	19.50	16.8	0.9700	13.10	0.01284	0.0116	32.59
30	18.5	21.5	19.00	16.3	0.9700	13.20	0.01284	0.0085	31.62
60	17.5	22	18.00	15.4	0.9700	13.30	0.01276	0.0060	29.88
125	16.5	22.5	17.00	14.6	0.9700	13.50	0.01269	0.0042	28.23
330	15.5	24	16.00	14.0	0.9700	13.70	0.01246	0.0025	27.16
1440	14.5	22	15.00	12.4	0.9700	13.80	0.01276	0.0012	24.06

Total oven Dry mass= 50.00



	Diameter of soil Particle (mm)	% Pass
Sieve Analysis	50.000	100.00
	37.500	100.00
	25.000	100.00
	19.000	100.00
	12.500	100.00
	4.750	99.96
	3.350	99.94
	2.360	99.92
	1.180	99.15
	0.600	93.18
	0.300	86.36
	0.150	69.00
	0.075	56.00
Hydrometer Analysis	0.044	45.20
	0.032	42.29
	0.026	41.32
	0.023	38.41
	0.016	37.44
	0.012	32.59
	0.009	31.62

1.Particles larger than 2mm =	0%
2.Coarse Sand 2mm - 0.425mm =	7%
3.Fine Sand 0.425mm - 0.075mm =	37%
4.Silt 0.075-0.002mm =	18%
5.Clay smaller than 0.002mm =	38%

Figure A-1. 1: Grain size analysis of test pit 1 at 2m depth.

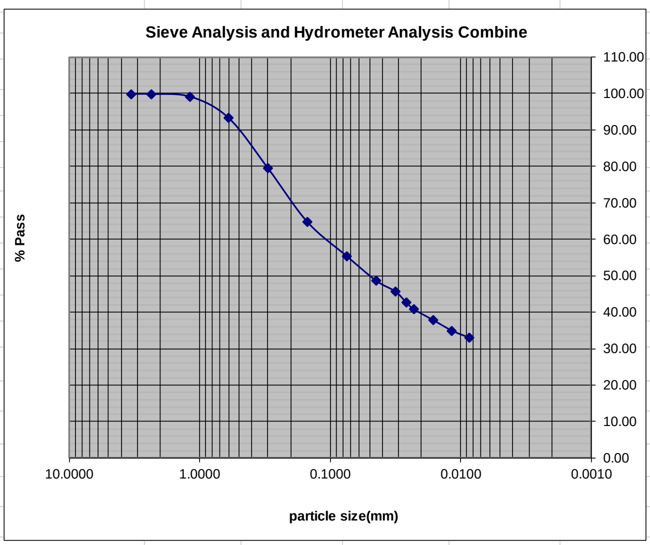
	Company Name: **HAWASSA INSTITUTE OF TECHNOLOGY**		
	Doucment Title Laboratory Soil Test Result Reporting Format	Page No.: Page 1 of 1	

PROJECT:	THESIS	**ZERO CORRECTION**	3.00
TESTED BY:	**SAMUEL A.**	**MENISCUS CORRECTION**	**0.50**
SAMPLE OF:	TP1_2		
STATION:	DILLA TTC		
TEST TYPE:	Hydrometer analysis & Sieve Analysis (AASHTO T-88)		
HYDOMETER NO.	152H		
DISPERSING AGEN	$NaPO_3$		

Hydrometer analysis Data Specific Gravity= 2.75

Time (minutes)	Hydrometer Reading	Temp.	Corrected H.Reading		Correction factor(a)	Effe. Depth of Hydrometer(L)	Values of K	Diameter of soil Particle(mm)	% finer,P
			R'	R"					
1	27	22	27.50	24.8	0.9800	11.80	0.01294	0.0445	48.61
2	25.5	22	26.00	23.3	0.9800	12.00	0.01294	0.0317	45.67
3	24	22	24.50	21.8	0.9800	12.30	0.01294	0.0262	42.73
4	23	22	23.50	20.8	0.9800	12.45	0.01294	0.0228	40.77
8	21.5	22	22.00	19.3	0.9800	12.70	0.01294	0.0163	37.83
16	20	22	20.50	17.8	0.9800	12.95	0.01294	0.0116	34.89
30	19	22	19.50	16.8	0.9800	13.10	0.01294	0.0086	32.93
60	18.5	22	19.00	16.4	0.9800	13.20	0.01294	0.0061	32.14
125	17	23	17.50	15.1	0.9800	13.40	0.01279	0.0042	29.50
330	16	24	16.50	14.5	0.9800	13.60	0.01264	0.0026	28.42
1440	15.5	21	16.00	13.4	0.9800	13.70	0.01309	0.0013	26.26

Total oven Dry mass= 50.00



	Diameter of soil Particle (mm)	% Pass
Sieve Analysis	50.000	100.00
	37.500	100.00
	25.000	100.00
	19.000	100.00
	12.500	100.00
	4.750	99.94
	3.350	99.81
	2.360	99.80
	1.180	99.09
	0.600	93.26
	0.300	79.49
	0.150	64.78
	0.075	55.41
Hydrometer Analysis	0.044	48.61
	0.032	45.67
	0.026	42.73
	0.023	40.77
	0.016	37.83
	0.012	34.89
	0.009	32.93

1.Particles larger than 2mm =	0%
2.Coarse Sand 2mm - 0.425mm =	7%
3.Fine Sand 0.425mm - 0.075mm =	38%
4.Silt 0.075-0.002mm =	15%
5.Clay smaller than 0.002mm =	41%

Figure A-1. 2: Grain size analysis of test pit 1 at 4m depth.

Company Name:	**HAWASSA INSTITUTE OF TECHNOLOGY**
Doucment Title	Laboratory Soil Test Result Reporting Format
Page No.:	Page 1 of 1

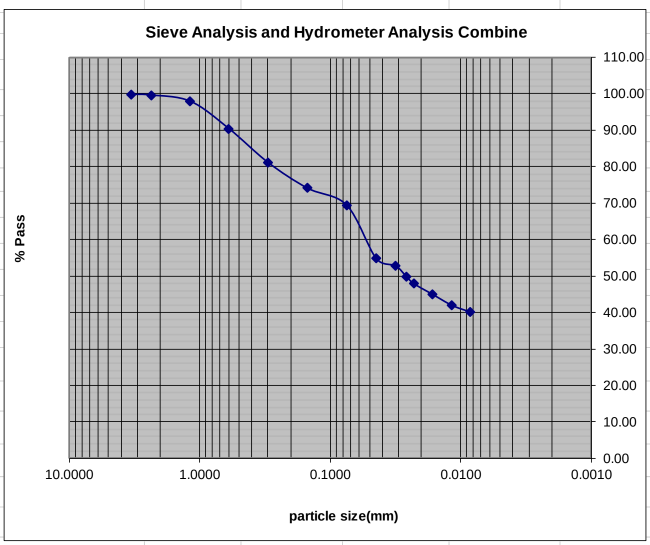
PROJECT:	THESIS
TESTED BY:	**SAMUEL A.**
SAMPLE OF:	TP2_1
STATION:	STADIUM
TEST TYPE:	Hydrometer analysis & Sieve Analysis (AASHTO T-88)
HYDOMETER NO.	152H
DISPERSING AGEN	$NaPO_3$

ZERO CORRECTION	3.00
MENISCUS CORRECTION	**1.00**

Hydrometer analysis Data Specific Gravity= 2.7

Time (minutes)	Hydrometer Reading	Temp.	Corrected H.Reading R'	Corrected H.Reading R"	Correction factor(a)	Effe. Depth of Hydrometer(L)	Values of K	Diameter of soil Particle(mm)	% finer,P
1	29.5	21	30.50	27.7	0.9900	11.30	0.01328	0.0446	54.85
2	28.5	21	29.50	26.7	0.9900	11.45	0.01328	0.0318	52.87
3	27	21	28.00	25.2	0.9900	11.70	0.01328	0.0262	49.90
4	26	21	27.00	24.2	0.9900	11.90	0.01328	0.0229	47.92
8	24.5	21	25.50	22.7	0.9900	12.20	0.01328	0.0164	44.95
16	23	21	24.00	21.2	0.9900	12.40	0.01328	0.0117	41.98
30	22	21.5	23.00	20.3	0.9900	12.50	0.01320	0.0085	40.19
60	21	22	22.00	19.4	0.9900	12.70	0.01312	0.0060	38.41
125	20.5	23	21.50	19.2	0.9900	12.80	0.01297	0.0042	38.02
330	19	24	20.00	18.0	0.9900	13.00	0.01282	0.0025	35.64
1440	18.5	22	19.50	16.9	0.9900	13.10	0.01312	0.0013	33.46

Total oven Dry mass= 50.00



	Diameter of soil Particle (mm)	% Pass
Sieve Analysis	50.000	100.00
	37.500	100.00
	25.000	100.00
	19.000	100.00
	12.500	100.00
	4.750	99.92
	3.350	99.79
	2.360	99.62
	1.180	97.89
	0.600	90.44
	0.300	81.13
	0.150	74.11
	0.075	69.42
Hydrometer Analysis	0.045	54.85
	0.032	52.87
	0.026	49.90
	0.023	47.92
	0.016	44.95
	0.012	41.98
	0.009	40.19

1.Particles larger than 2mm =	0%
2.Coarse Sand 2mm - 0.425mm =	9%
3.Fine Sand 0.425mm - 0.075mm =	21%
4.Silt 0.075-0.002mm =	22%
5.Clay smaller than 0.002mm =	48%

Figure A-1. 3: Grain size analysis of test pit 2 at 2m depth.

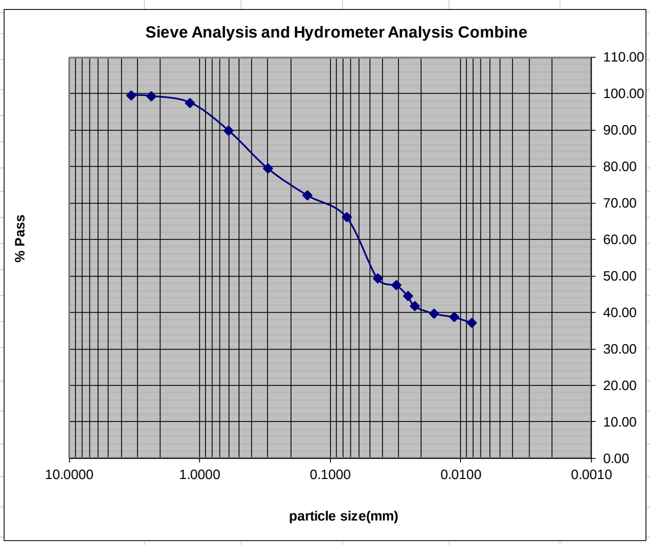
	Company Name: **HAWASSA INSTITUTE OF TECHNOLOGY**	
	Doucment Title Laboratory Soil Test Result Reporting Format	Page No.: Page 1 of 1

PROJECT:	THESIS	**ZERO CORRECTION**	3.00
TESTED BY:	**SAMUEL A.**	**MENISCUS CORRECTION**	**1.00**
SAMPLE OF:	TP2_2		
STATION:	STADIUM		
TEST TYPE:	Hydrometer analysis & Sieve Analysis (AASHTO T-88)		
HYDOMETER NO.	152H		
DISPERSING AGEN	$NaPO_3$		

Hydrometer analysis Data Specific Gravity= 2.85

Time (minutes)	Hydrometer Reading	Temp.	Corrected H.Reading R'	R"	Correction factor(a)	Effe. Depth of Hydrometer(L)	Values of K	Diameter of soil Particle(mm)	% finer,P
1	27.5	21	28.50	25.7	0.9600	11.60	0.01273	0.0434	49.34
2	26.5	21	27.50	24.7	0.9600	11.80	0.01273	0.0309	47.42
3	25	21	26.00	23.2	0.9600	12.00	0.01273	0.0255	44.54
4	23.5	21	24.50	21.7	0.9600	12.30	0.01273	0.0223	41.66
8	22.5	21	23.50	20.7	0.9600	12.45	0.01273	0.0159	39.74
16	22	21	23.00	20.2	0.9600	12.50	0.01273	0.0113	38.78
30	21	22	22.00	19.4	0.9600	12.70	0.01258	0.0082	37.25
60	20	22	21.00	18.4	0.9600	12.90	0.01258	0.0058	35.33
125	19.5	23	20.50	18.2	0.9600	12.95	0.01243	0.0040	34.94
330	18.5	24	19.50	17.5	0.9600	13.10	0.01229	0.0024	33.60
1440	18	21	19.00	16.2	0.9600	13.20	0.01273	0.0012	31.10

Total oven Dry mass= 50.00



	Diameter of soil Particle (mm)	% Pass
Sieve Analysis	50.000	100.00
	37.500	100.00
	25.000	100.00
	19.000	100.00
	12.500	100.00
	4.750	99.86
	3.350	99.63
	2.360	99.35
	1.180	97.53
	0.600	89.88
	0.300	79.48
	0.150	72.17
	0.075	66.19
Hydrometer Analysis	0.043	49.34
	0.031	47.42
	0.025	44.54
	0.022	41.66
	0.016	39.74
	0.011	38.78
	0.008	37.25

1.Particles larger than 2mm =	1%
2.Coarse Sand 2mm - 0.425mm =	9%
3.Fine Sand 0.425mm - 0.075mm =	24%
4.Silt 0.075-0.002mm =	25%
5.Clay smaller than 0.002mm =	42%

Figure A-1. 4: Grain size analysis of test pit 2 at 4m depth.

Company Name:

HAWASSA INSTITUTE OF TECHNOLOGY

Doucment Title | Page No.:

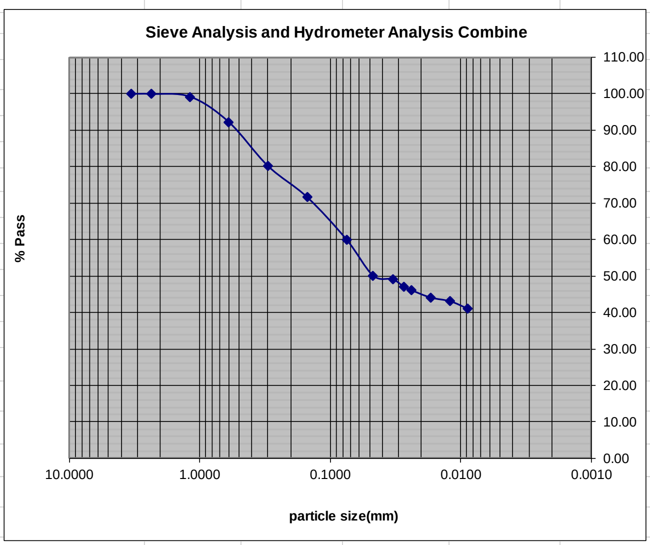
Laboratory Soil Test Result Reporting Format | Page 1 of 1

PROJECT:	THESIS	**ZERO CORRECTION**	2.50
TESTED BY:	**SAMUEL A.**	**MENISCUS CORRECTION**	**0.50**
SAMPLE OF:	TP3_1		
STATION:	HIGH SCHOOL BRANCH		
TEST TYPE:	Hydrometer analysis & Sieve Analysis (AASHTO T-88)		
HYDOMETER NO.	152H		
DISPERSING AGEN	$NaPO_3$		

Hydrometer analysis Data Specific Gravity= 2.6

Time (minutes)	Hydrometer Reading	Temp.	Corrected H.Reading R'	Corrected H.Reading R"	Correction factor(a)	Effe. Depth of Hydrometer(L)	Values of K	Diameter of soil Particle(mm)	% finer,P
1	26.5	21.5	27.00	24.8	1.0100	11.90	0.01361	0.0469	50.10
2	26	21.5	26.50	24.3	1.0100	11.95	0.01361	0.0333	49.09
3	25	21.5	25.50	23.3	1.0100	12.10	0.01361	0.0273	47.07
4	24.5	21.5	25.00	22.8	1.0100	12.20	0.01361	0.0238	46.06
8	23.5	21.5	24.00	21.8	1.0100	12.40	0.01361	0.0169	44.04
16	23	21.5	23.50	21.3	1.0100	12.45	0.01361	0.0120	43.03
30	22	21.5	22.50	20.3	1.0100	12.60	0.01361	0.0088	41.01
60	21.5	22	22.00	19.9	1.0100	12.70	0.01353	0.0062	40.20
125	20.5	22.5	21.00	19.1	1.0100	12.90	0.01345	0.0043	38.48
330	20	24	20.50	19.0	1.0100	12.95	0.01321	0.0026	38.38
1440	18.5	22	19.00	16.9	1.0100	13.20	0.01353	0.0013	34.14

Total oven Dry mass= 50.00



	Diameter of soil Particle (mm)	% Pass
Sieve Analysis	50.000	100.00
	37.500	100.00
	25.000	100.00
	19.000	100.00
	12.500	100.00
	4.750	99.97
	3.350	99.93
	2.360	99.89
	1.180	99.13
	0.600	92.25
	0.300	80.16
	0.150	71.59
	0.075	60.00
Hydrometer Analysis	0.047	50.10
	0.033	49.09
	0.027	47.07
	0.024	46.06
	0.017	44.04
	0.012	43.03
	0.009	41.01

1.Particles larger than 2mm =	0%
2.Coarse Sand 2mm - 0.425mm =	8%
3.Fine Sand 0.425mm - 0.075mm =	32%
4.Silt 0.075-0.002mm =	14%
5.Clay smaller than 0.002mm =	46%

Figure A-1. 5: Grain size analysis of test pit 3 at 2m depth.

Company Name:

HAWASSA INSTITUTE OF TECHNOLOGY

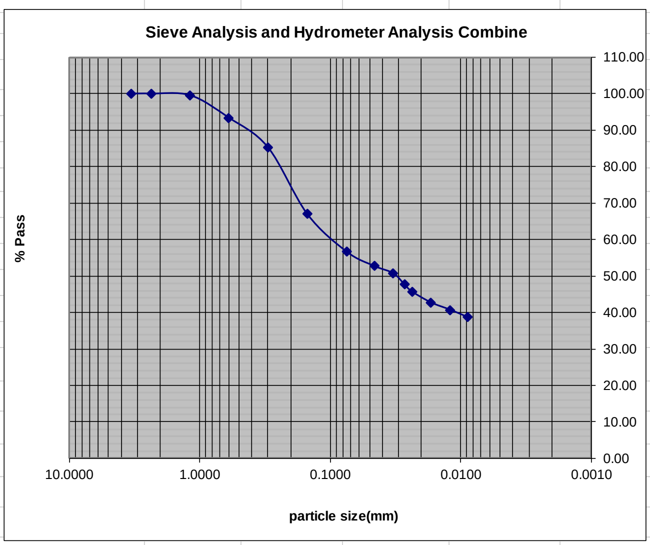
Doucment Title	Page No.:
Laboratory Soil Test Result Reporting Format	Page 1 of 1

PROJECT:	THESIS	**ZERO CORRECTION**	2.50
TESTED BY:	**SAMUEL A.**	**MENISCUS CORRECTION**	**0.50**
SAMPLE OF:	TP3_2		
STATION:	HIGH SCHOOL BRANCH		
TEST TYPE:	Hydrometer analysis & Sieve Analysis (AASHTO T-88)		
HYDOMETER NO.	152H		
DISPERSING AGEN	$NaPO_3$		

Hydrometer analysis Data Specific Gravity= 2.64

Time (minutes)	Hydrometer Reading	Temp.	Corrected H.Reading R'	Corrected H.Reading R"	Correction factor(a)	Effe. Depth of Hydrometer(L)	Values of K	Diameter of soil Particle(mm)	% finer,P
1	28	21	28.50	26.3	1.0020	11.60	0.01352	0.0461	52.71
2	27	21	27.50	25.3	1.0020	11.80	0.01352	0.0328	50.70
3	25.5	21	26.00	23.8	1.0020	12.00	0.01352	0.0270	47.70
4	24.5	21	25.00	22.8	1.0020	12.20	0.01352	0.0236	45.69
8	23	21	23.50	21.3	1.0020	12.45	0.01352	0.0169	42.69
16	22	21	22.50	20.3	1.0020	12.60	0.01352	0.0120	40.68
30	21	21	21.50	19.3	1.0020	12.80	0.01352	0.0088	38.68
60	20.5	22	21.00	18.9	1.0020	12.90	0.01336	0.0062	37.88
125	19	22.5	19.50	17.6	1.0020	13.10	0.01329	0.0043	35.17
330	18.5	24	19.00	17.5	1.0020	13.20	0.01305	0.0026	35.07
1440	18	21	18.50	16.4	1.0020	13.25	0.01352	0.0013	32.87

Total oven Dry mass= 50.00



	Diameter of soil Particle (mm)	% Pass
Sieve Analysis	50.000	100.00
	37.500	100.00
	25.000	100.00
	19.000	100.00
	12.500	100.00
	4.750	100.00
	3.350	99.99
	2.360	99.99
	1.180	99.51
	0.600	93.41
	0.300	85.25
	0.150	66.97
	0.075	56.65
Hydrometer Analysis	0.046	52.71
	0.033	50.70
	0.027	47.70
	0.024	45.69
	0.017	42.69
	0.012	40.68
	0.009	38.68

1.Particles larger than 2mm =	0%
2.Coarse Sand 2mm - 0.425mm =	7%
3.Fine Sand 0.425mm - 0.075mm =	37%
4.Silt 0.075-0.002mm =	11%
5.Clay smaller than 0.002mm =	46%

Figure A-1. 6: Grain size analysis of test pit 3 at 4m depth.

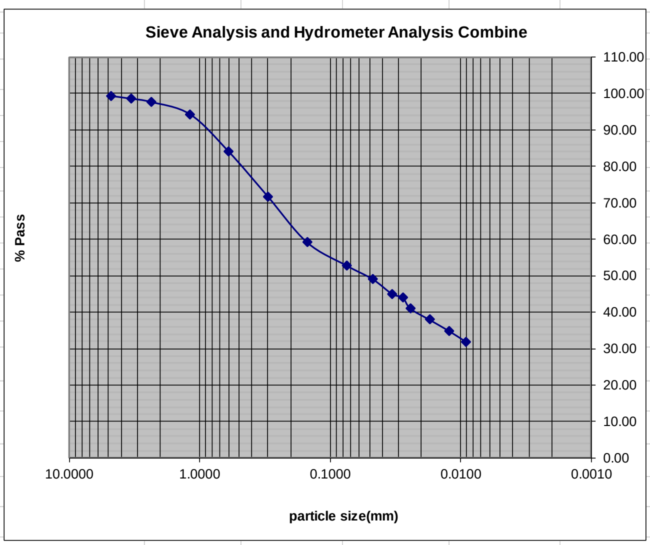
Company Name: **HAWASSA INSTITUTE OF TECHNOLOGY**	
Doucment Title Laboratory Soil Test Result Reporting Format	Page No.: Page 1 of 1

PROJECT:	THESIS	**ZERO CORRECTION**	2.50
TESTED BY:	**SAMUEL A.**	**MENISCUS CORRECTION**	**0.50**
SAMPLE OF:	TP4_1		
STATION:	MEKANE EYESUS CHURCH		
TEST TYPE:	Hydrometer analysis & Sieve Analysis (AASHTO T-88)		
HYDOMETER NO.	152H		
DISPERSING AGEN	$NaPO_3$		

Hydrometer analysis Data Specific Gravity= 2.6

Time (minutes)	Hydrometer Reading	Temp.	Corrected H.Reading		Correction factor(a)	Effe. Depth of Hydrometer(L)	Values of K	Diameter of soil Particle(mm)	% finer,P
			R'	R"					
1	26	21.5	26.50	24.3	1.0100	11.95	0.01361	0.0470	49.09
2	24	21.5	24.50	22.3	1.0100	12.10	0.01361	0.0335	45.05
3	23.5	21.5	24.00	21.8	1.0100	12.40	0.01361	0.0277	44.04
4	22	21.5	22.50	20.3	1.0100	12.60	0.01361	0.0242	41.01
8	20.5	21.5	21.00	18.8	1.0100	12.90	0.01361	0.0173	37.98
16	19	21.5	19.50	17.3	1.0100	13.10	0.01361	0.0123	34.95
30	17.5	21.5	18.00	15.8	1.0100	13.30	0.01361	0.0091	31.92
60	16	22	16.50	14.4	1.0100	13.60	0.01353	0.0064	29.09
125	15.5	22.5	16.00	14.1	1.0100	13.70	0.01345	0.0045	28.38
330	14.5	24.5	15.00	13.7	1.0100	13.80	0.01321	0.0027	27.57
1440	14	22	14.50	12.4	1.0100	14.00	0.01353	0.0013	25.05

Total oven Dry mass= 50.00



	Diameter of soil Particle (mm)	% Pass
Sieve Analysis	50.000	100.00
	37.500	100.00
	25.000	100.00
	19.000	100.00
	12.500	100.00
	4.750	99.22
	3.350	98.51
	2.360	97.61
	1.180	94.24
	0.600	84.12
	0.300	71.77
Hydrometer Analysis	0.150	59.19
	0.075	52.84
	0.047	49.09
	0.033	45.05
	0.028	44.04
	0.024	41.01
	0.017	37.98
	0.012	34.95
	0.009	31.92

1.Particles larger than 2mm =	2%
2.Coarse Sand 2mm - 0.425mm =	13%
3.Fine Sand 0.425mm - 0.075mm =	31%
4.Silt 0.075-0.002mm =	12%
5.Clay smaller than 0.002mm =	41%

Figure A-1. 7: Grain size analysis of test pit 4 at 2m depth.

Company Name: **ETHIOPIAN CONSTRUCTION DESIGN & SUPERVISION WORKS CORPORATION** TRANSPORT DESIGN SUPERVISION WORKS SECTOR	Document No.:
Doucment Title Laboratory Soil Test Result Reporting Format	Page No.: Page 1 of 1

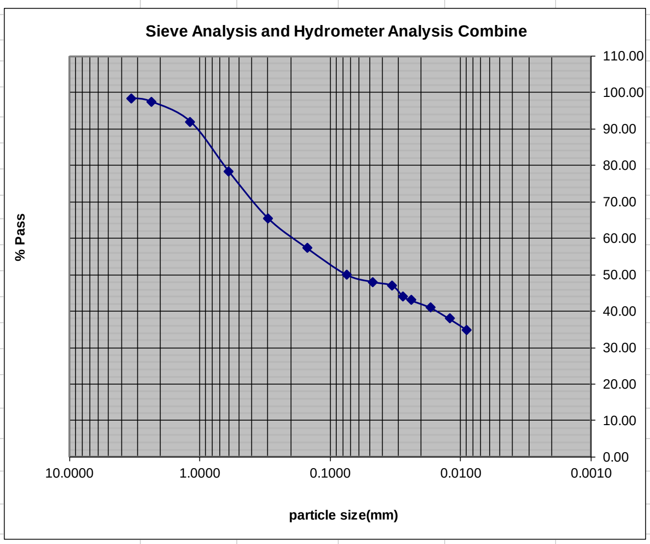
CODE:- 0077/2016 **Your Ref.:-** ==============
PROJECT: THESIS
TESTED BY: SAMUEL A.
SAMPLE OF: TP4 2
STATION: MEKANE EYESUS CHURCH
TEST TYPE: Hydrometer analysis & Sieve Analysis (AASHTO T-88)
HYDOMETER NO. 152H
DISPERSING AGEN $NaPO_3$

ZERO CORRECTION 2.50
MENISCUS CORRECTION 0.50

Hydrometer analysis Data Specific Gravity= 2.51

Time (minutes)	Hydrometer Reading	Temp.	Corrected H.Reading R'	Corrected H.Reading R"	Correction factor(a)	Effe. Depth of Hydrometer(L)	Values of K	Diameter of soil Particle(mm)	% finer,P
1	25.5	21.5	26.5	23.8	1.0100	11.95	0.01360	0.0470	48.08
2	25	21.5	26	23.3	1.0100	12.00	0.01360	0.0333	47.07
3	23.5	21.5	24.5	21.8	1.0100	12.30	0.01360	0.0275	44.04
4	23	21.5	24	21.3	1.0100	12.40	0.01360	0.0239	43.03
8	22	21.5	23	20.3	1.0100	12.50	0.01360	0.0170	41.01
16	20.5	21.5	21.5	18.8	1.0100	12.80	0.01360	0.0122	37.98
30	19	21.5	20	17.3	1.0100	13.00	0.01360	0.0090	34.95
60	18	21.5	19	16.3	1.0100	13.20	0.01360	0.0064	32.93
125	16.5	22	17.5	14.9	1.0100	13.40	0.01350	0.0044	30.10
330	16	23.5	17	14.85	1.0100	13.50	0.01330	0.0027	30.00
1440	15	21.8	16	13.35	1.0100	13.70	0.01345	0.0013	26.97

Total oven Dry mass= 50.00



	Diameter of soil Particle (mm)	% Pass
Sieve Analysis	50.000	100.00
	37.500	100.00
	25.000	100.00
	19.000	100.00
	12.500	100.00
	4.750	99.19
	3.350	98.48
	2.360	97.50
	1.180	92.02
	0.600	78.41
	0.300	65.57
	0.150	57.32
	0.075	50.08
Hydrometer Analysis	0.047	48.08
	0.033	47.07
	0.028	44.04
	0.024	43.03
	0.017	41.01
	0.012	37.98
	0.009	34.95

1.Particles larger than 2mm =	3%
2.Coarse Sand 2mm - 0.425mm =	19%
3.Fine Sand 0.425mm - 0.075mm =	28%
4.Silt 0.075-0.002mm =	7%
5.Clay smaller than 0.002mm =	43%

Figure A-1. 8: Grain size analysis of test pit 4 at 4m depth.

Company Name	**HAWASSA UNIVERSITY INSTITUTE OF TECHNOLOGY**	Document No.
Title	Laboratory Soil Test Result Reporting Format	Page No. 1 of 1

PROJECT :- MSc Thesis

SUBMITED BY :- Samuel Abayneh

SAMPLE OF :- TP1_1

STATION/LOCATION :- Dilla TTC

TEST TYPE :- Atterberg Limit

ATTERBERG LIMITS

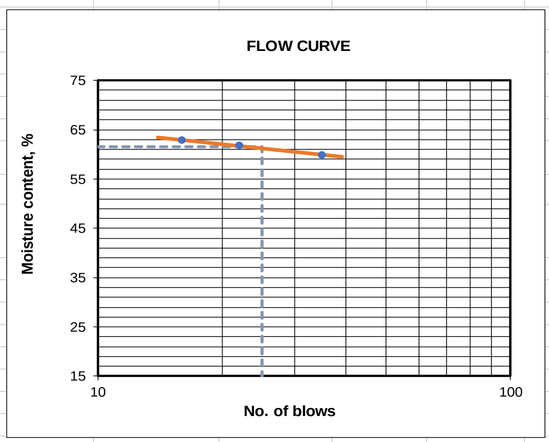
TEST METHODS: AASHTO T-89 & T-90

		Liquid Limit (LL)			Plastic Limit (PL)	
No. of blows		35	22	16	-	-
Container No.		C4	A50	CN	C24	AP1
Mass of Wet Soil + Container	g	30.12	28.81	27.92	23.28	7.95
Mass of Dry Soil + Container	g	22.38	21.41	20.72	19.97	6.88
Mass of Water	g	7.74	7.40	7.20	3.31	1.07
Mass of Container	g	9.46	9.46	9.27	9.72	3.85
Mass of Dry Soil	g	12.92	11.95	11.45	10.25	3.03
Moisture Content	%	59.9	61.9	62.9	32.3	35.3
					Avg PL, %=	33.8

Liquid Limit (LL) :- 62 %

Plastic Limit (PL) :- 34 %

Plasticity Index (PI) :- 28 %



Remark:-The test done according to AASHTO T-89 and T-90 with maximum mixing time 5 minutes for each trial.

Conducted By: Samuel Abayneh **Checked By: ________________** **Approved By: ______________**

Figure A-1. 9: Atterberg limits analysis of test pit 1 at 2m depth.

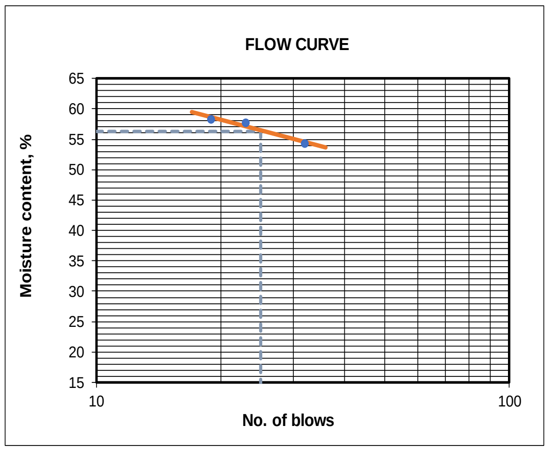
	Company Name **HAWASSA UNIVERSITY INSTITUTE OF TECHNOLOGY**	Document No.
	Title Laboratory Soil Test Result Reporting Format	Page No. **1 of 1**

PROJECT :- **MSc Thesis**
SUBMITED BY :- **Samuel Abayneh**
SAMPLE OF :- **TP1_2**
STATION/LOCATION :- **Dilla TTC**
TEST TYPE :- **Atterberg Limit**

ATTERBERG LIMITS
TEST METHODS: AASHTO T-89 & T-90

		Liquid Limit (LL)			Plastic Limit (PL)	
No. of blows		32	23	19	-	-
Container No.		A7	C4	C28	AP4	P5
Mass of Wet Soil + Container	g	28.36	26.11	34.27	9.72	12.52
Mass of Dry Soil + Container	g	21.85	20	25.29	8.31	10.53
Mass of Water	g	6.51	6.11	8.98	1.41	1.99
Mass of Container	g	9.87	9.41	9.86	4.15	3.80
Mass of Dry Soil	g	11.98	10.59	15.43	4.16	6.73
Moisture Content	%	54.3	57.7	58.2	33.9	29.6
					Avg PL, %=	**31.7**

Liquid Limit (LL) :- 56 %
Plastic Limit (PL) :- 32 %
Plasticity Index (PI) :- 25 %



Remark:-The test done according to AASHTO T-89 and T-90 with maximum mixing time of 5 minutes for each trial.

Conducted By: Samuel Abayneh **Checked By: ________________** **Approved By: ______________**

Figure A-1. 10: Atterberg limits analysis of test pit 1 at 4m depth.

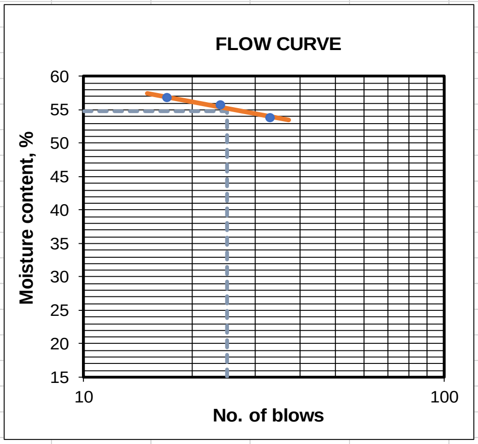
Company Name	Document No.
HAWASSA UNIVERSITY INSTITUTE	
Title	Page No.
Laboratory Soil Test Result Reporting Format	**1 of 1**

PROJECT :- **MSc Thesis**
SUBMITED BY :- **Samuel Abayneh**
SAMPLE OF :- **TP2_1**
STATION/LOCATION :- **Stadium**
TEST TYPE :- **Atterberg Limit**

ATTERBERG LIMITS
TEST METHODS: AASHTO T-89 & T-90

		Liquid Limit (LL)			Plastic Limit (PL	
No. of blows		33	24	17	-	-
Container No.		CS1	CC	CS2	C4	C6
Mass of Wet Soil + Container	g	31.31	28.19	29.93	11.24	10.31
Mass of Dry Soil + Container	g	23.66	21.67	22.66	9.29	8.92
Mass of Water	g	7.65	6.52	7.27	1.95	1.39
Mass of Container	g	9.46	9.98	9.86	3.83	3.85
Mass of Dry Soil	g	14.20	11.69	12.80	5.46	5.07
Moisture Content	%	53.9	55.8	56.8	35.7	27.4
					Avg PL, %=	**31.6**

Liquid Limit (LL) :-	**55**	**%**
Plastic Limit (PL) :-	**32**	**%**
Plasticity Index (PI) :-	**23**	**%**



Remark:-The test done according to AASHTO T-89 and T-90 with maximum mixing time of 5 minutes for each trial.

Conducted By: Samuel Abayneh **Checked By: __________________** **Approved By: _____________________**

Figure A-1. 11: Atterberg limits analysis of test pit 2 at 2m depth.

Company Name	Document No.
HAWASSA UNIVERSITY INSTITUTE OF TECHNOLOGY	
Title	Page No.
Laboratory Soil Test Result Reporting Format	**1 of 1**

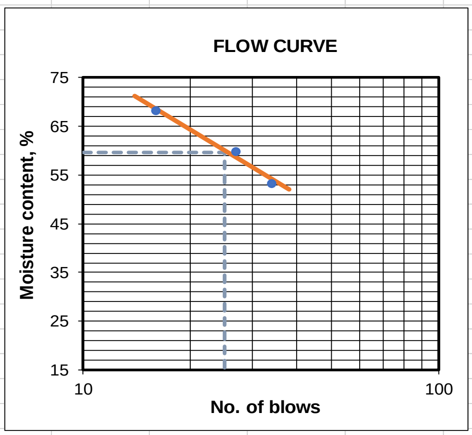
PROJECT :-	**MSc Thesis**
SUBMITED BY :-	**Samuel Abayneh**
SAMPLE OF :-	**TP2_2**
STATION/LOCATION :-	**Stadium**
TEST TYPE :-	**Atterberg Limit**

ATTERBERG LIMITS

TEST METHODS: AASHTO T-89 & T-90

		Liquid Limit (LL)			**Plastic Limit (PL)**	
No. of blows		34	27	16	-	-
Container No.		C5	CA	C12	CCS	CSS
Mass of Wet Soil + Container	g	25.4	26.55	29.06	17.32	18.23
Mass of Dry Soil + Container	g	19.86	20.32	21.01	13.77	14.98
Mass of Water	g	5.54	6.23	8.05	3.55	3.25
Mass of Container	g	9.46	9.91	9.21	3.98	3.98
Mass of Dry Soil	g	10.40	10.41	11.80	9.79	11.00
Moisture Content	%	53.3	59.8	68.2	36.3	29.5
					Avg PL, %=	**32.9**

Liquid Limit (LL) :-	**60**	**%**
Plastic Limit (PL) :-	**33**	**%**
Plasticity Index (PI) :-	**27**	**%**



Remark:-The test done according to AASHTO T-89 and T-90 with maximum mixing time of 5 minutes for each trial.

Conducted By: Samuel Abayneh **Checked By: ________________** **Approved By: __________________**

Figure A-1. 12: Atterberg limits analysis of test pit 2 at 4m depth.

	Company Name **HAWASSA UNIVERSITY INSTITUTE OF TECHNOLOGY**	Document No.
	Title Laboratory Soil Test Result Reporting Format	Page No. **1 of 1**

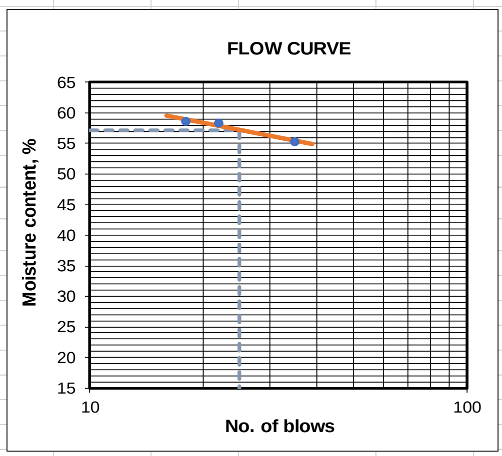
CODE :-
PROJECT :- **MSc Thesis**
SUBMITED BY :- **Samuel Abayneh**
SAMPLE OF :- **TP3_1**
STATION/LOCATION :- **High School Branch**
TEST TYPE :- **Atterberg Limit**

ATTERBERG LIMITS

TEST METHODS: AASHTO T-89 & T-90

		Liquid Limit (LL)			Plastic Limit (PL)	
No. of blows		35	22	18	-	-
Container No.		C28	A50	CN	P6	AP1
Mass of Wet Soil + Container	g	27.86	23.80	25.28	9.86	10.5
Mass of Dry Soil + Container	g	21.45	18.5	19.32	8.41	8.99
Mass of Water	g	6.41	5.30	5.96	1.45	1.51
Mass of Container	g	9.86	9.42	9.15	3.86	3.86
Mass of Dry Soil	g	11.59	9.08	10.17	4.55	5.13
Moisture Content	%	55.3	58.4	58.6	31.9	29.4
					Avg PL, %=	**30.7**

Liquid Limit (LL) :-	**57**	**%**
Plastic Limit (PL) :-	**31**	**%**
Plasticity Index	**26**	**%**



Remark:-The test done according to AASHTO T-89 and T-90 with maximum mixing time of 5 minutes for each trial.

Conducted By: Samuel Abayneh **Checked By: ________________** **Approved By: ___________________**

Figure A-1. 13: Atterberg limits analysis of test pit 3 at 2m depth.

Company Name		Document No.
	HAWASSA UNIVERSITY INSTITUTE OF TECHNOLOGY	
Title		Page No.
	Laboratory Soil Test Result Reporting Format	**1 of 1**

CODE :- **Your Ref No.:**

PROJECT :- **MSc Thesis**

SUBMITED BY :- **Samuel Abayneh**

SAMPLE OF :- **TP3_2**

STATION/LOCATION :- **High School Branch**

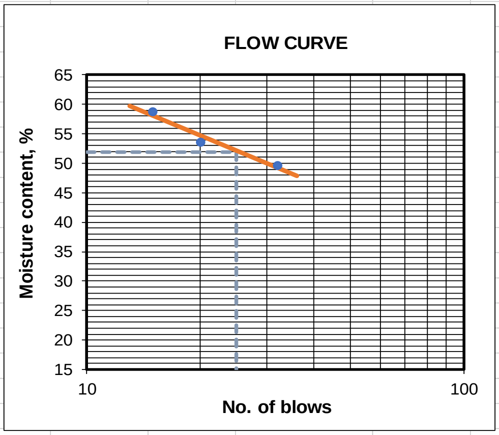
TEST TYPE :- **Atterberg Limit**

ATTERBERG LIMITS

TEST METHODS: AASHTO T-89 & T-90

		Liquid Limit (LL)			Plastic Limit (PL)	
No. of blows		32	20	15	-	-
Container No.		C4	C22	A7	AP20	AP2
Mass of Wet Soil + Container	g	24.47	24.01	24.36	13.83	17.38
Mass of Dry Soil + Container	g	19.49	19.09	18.72	11.97	13.27
Mass of Water	g	4.98	4.92	5.64	1.86	4.11
Mass of Container	g	9.45	9.91	9.12	3.89	3.84
Mass of Dry Soil	g	10.04	9.18	9.60	8.08	9.43
Moisture Content	%	49.6	53.6	58.8	23.0	43.6
					Avg PL, %=	**33.3**

Liquid Limit (LL) :-	52	%
Plastic Limit (PL) :-	33	%
Plasticity Index (PI) :-	19	%



Remark:-The test done according to AASHTO T-89 and T-90 with maximum mixing time of 5 minutes for each trial.

Conducted By: Samuel Abayneh **Checked By: ________________** **Approved By: __________________**

Figure A-1. 14: Atterberg limits analysis of test pit 3 at 4m depth.

HAWASSA UNIVERSITY INSTITUTE OF TECHNOLOGY

Laboratory Soil Test Result Reporting Format

Page No. 1 of 2

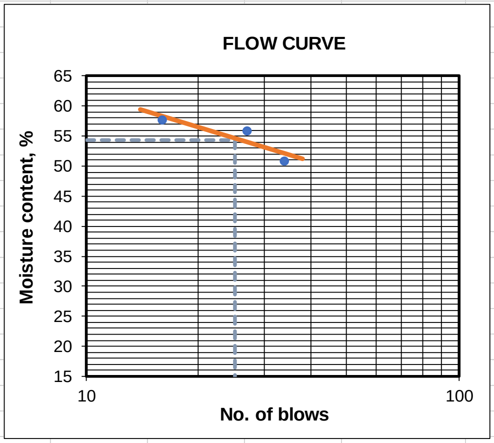
PROJECT :- **MSc Thesis**
SUBMITED BY :- **Samuel Abayneh**
SAMPLE OF :- **TP7_1**
STATION/LOCATION :- **Haroressa School**
TEST TYPE :- **Atterberg Limit**

ATTERBERG LIMITS

TEST METHODS: AASHTO T-89 & T-90

		Liquid Limit (LL)			Plastic Limit (PL)	
No. of blows		34	27	16	-	-
Container No.		CS	A7	CS1	CP1	CP2
Mass of Wet Soil + Container	g	32.21	31.81	33.14	12.83	12.9
Mass of Dry Soil + Container	g	24.54	23.99	24.62	10.62	10.93
Mass of Water	g	7.67	7.82	8.52	2.21	1.97
Mass of Container	g	9.46	9.98	9.86	3.98	4.01
Mass of Dry Soil	g	15.08	14.01	14.76	6.64	6.92
Moisture Content	%	50.9	55.8	57.7	33.3	28.5
					Avg PL, %=	30.9

Liquid Limit (LL) :-	**54**	**%**
Plastic Limit (PL) :-	**31**	**%**
Plasticity Index (PI) :-	**24**	**%**



Remark:- **The test is conducted as per standard of AASHTO T-89 and T-90 with maximum mixing time of 5 minutes for each trial**

Conducted By: Samuel A. **Checked By: ________________** **Approved By: _______________**

Figure A-1. 15: Atterberg limits analysis of test pit 7 at 2m depth.

HAWASSA UNIVERSITY INSTITUTE OF TECHNOLOGY

Laboratory Soil Test Result Reporting Format

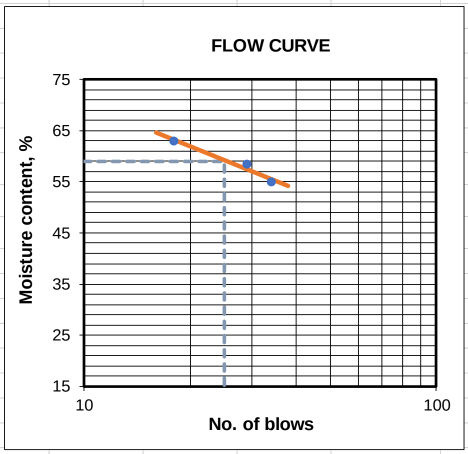
PROJECT :- **MSc Thesis**
SUBMITED BY :- **Samuel Abayneh**
SAMPLE OF :- **TP9_1**
STATION/LOCATION :- **St. Michael Church**
TEST TYPE :- **Atterberg Limit**

ATTERBERG LIMITS

TEST METHODS: AASHTO T-89 & T-90

		Liquid Limit (LL)			Plastic Limit (PL)	
No. of blows		34	29	18	-	-
Container No.		SS1	SS2	SS3	CP1	CP2
Mass of Wet Soil + Container	g	25.51	29.22	30.19	12.13	10.35
Mass of Dry Soil + Container	g	19.82	21.93	22.38	9.93	8.74
Mass of Water	g	5.69	7.29	7.81	2.20	1.61
Mass of Container	g	9.46	9.46	9.98	3.98	4.15
Mass of Dry Soil	g	10.36	12.47	12.40	5.95	4.59
Moisture Content	%	54.9	58.5	63.0	37.0	35.1
					Avg PL, %=	36.0

Liquid Limit (LL) :-	59	%
Plastic Limit (PL) :-	36	%
Plasticity Index (PI) :-	23	%



Remark:- The test is conducted as per standard of AASHTO T-89 and T-90 with maximum mixing time of 5 minutes for each trial

Conducted By: Samuel A. **Checked By:** ________________ **Approved By:** ______________

Figure A-1. 16: Atterberg limits analysis of test pit 9 at 2m depth.

HAWASSA UNIVERSITY INSTITUTE OF TECHNOLOGY

Laboratory Soil Test Result Reporting Format

Page No. 2 of 2

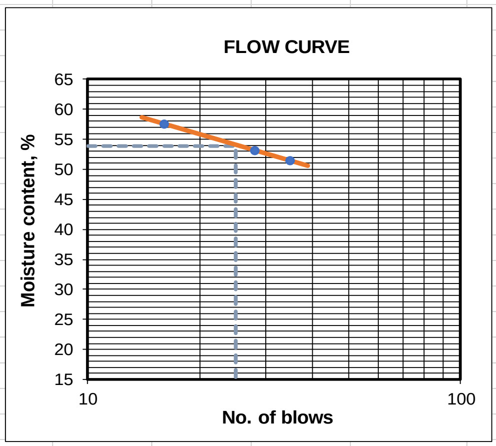
CODE :-		Your Ref No.:
PROJECT :-	MSc Thesis	
SUBMITED BY :-	Samuel Abayneh	
SAMPLE OF :-	TP10_2	
STATION/LOCATION :	Old Prison	
TEST TYPE :-	Atterberg Limit	

ATTERBERG LIMITS

TEST METHODS: AASHTO T-89 & T-90

		Liquid Limit (LL)			Plastic Limit (PL)	
No. of blows		35	28	16	-	-
Container No.		CL1	CS2	CS3	SP3	SP4
Mass of Wet Soil + Containe	g	29.33	29.49	30.28	22.19	23.41
Mass of Dry Soil + Container	g	22.75	22.72	22.58	19.19	20.08
Mass of Water	g	6.58	6.77	7.70	3.00	3.33
Mass of Container	g	9.96	9.98	9.21	9.46	9.68
Mass of Dry Soil	g	12.79	12.74	13.37	9.73	10.40
Moisture Content	%	51.4	53.1	57.6	30.8	32.0
					Avg PL, %=	31.4

Liquid Limit (LL) :-	54	%
Plastic Limit (PL) :-	31	%
Plasticity Index (PI) :-	22	%



Remark:- The test is conducted as per standard of AASHTO T-89 and T-90 with maximum mixing time of 5 minutes for each trial

Conducted By: Samuel A. Checked By: ________________ Approved By: _______________

Figure A-1. 17: Atterberg limits analysis of test pit 10 at 4m depth.

Table A-1. 1: In-situ unit weight test result and analysis

	Company Name:									
	HAWASSA INSTITUTE OF TECHNOLOGY									
	MATERIAL TEST RESULT									
PROJECT:		THESIS								
SUBMITTED BY:		Samuel Abayneh								
SAMPLES OF :		TP1 -TP10 @2m depth								
TEST TYPE:		In-situ unit weight determination								
	IN-SITU UNIT WEIGHT DETERMINATION									
	TEST METHOD: ASTM D 2937-00									
	Results of in-situ unit weight of all pits @ at 2m depth									
measurements	TP1_1	TP2_1	TP3_1	TP4_1	TP5_1	TP6_1	TP7_1	TP8_1	TP9_1	TP10_1
height of mold(m)	0.15	0.16	0.16	0.15	0.16	0.15	0.16	0.16	0.15	0.16
diameter (m)	0.11	0.1	0.13	0.12	0.11	0.11	0.13	0.12	0.12	0.13
Area (m^2)	0.009499	0.00785	0.013267	0.011304	0.009499	0.009499	0.013267	0.011304	0.011304	0.013267
Volume (m^3)	0.001425	0.001256	0.002123	0.001696	0.00152	0.001425	0.002123	0.001809	0.001696	0.002123
mass (g)	2298	2571	3259	2063	2234	2511	4270	3593	3451	3753
mass (kg)	2.298	2.571	3.259	2.063	2.234	2.511	4.270	3.593	3.451	3.753
gravity (m^2/s)	9.81	9.81	9.81	9.81	9.81	9.81	9.81	9.81	9.81	9.81
Density (kg/m^3)	1612.886	2046.975	1535.352	1216.678	1469.969	1762.384	2011.646	1986.576	2035.268	1768.081
Ys (N/m^3)	15822.41	20080.82	15061.81	11935.62	14420.4	17288.98	19734.25	19488.31	19965.98	17344.88
Ys (kN/m^3)	15.82241	20.08082	15.06181	11.93562	14.4204	17.28898	19.73425	19.48831	19.96598	17.34488

	Company Name:									
	HAWASSA INSTITUTE OF TECHNOLOGY									
	MATERIAL TEST RESULT									
PROJECT:		THESIS								
SUBMITTED BY:		Samuel Abayneh								
SAMPLES OF :		TP1 -TP10 @4m depth								
TEST TYPE:		In-situ unit weight determination								
	IN-SITU UNIT WEIGHT DETERMINATION									
	TEST METHOD: ASTM D 2937-00									
	Results of in-situ unit weight of all pits @ at 4m depth									
measurements	TP1_1	TP2_1	TP3_1	TP4_1	TP5_1	TP6_1	TP7_1	TP8_1	TP9_1	TP10_1
height of mold(m)	0.15	0.16	0.16	0.15	0.16	0.15	0.16	0.16	0.15	0.16
diameter (m)	0.11	0.1	0.13	0.12	0.11	0.11	0.13	0.12	0.12	0.13
Area (m^2)	0.009499	0.00785	0.013267	0.011304	0.009499	0.009499	0.013267	0.011304	0.011304	0.013267
Volume (m^3)	0.001425	0.001256	0.002123	0.001696	0.00152	0.001425	0.002123	0.001809	0.001696	0.002123
mass (g)	2887	2767	3448	2779	2529	2350	3116	4203	3557	2760
mass (kg)	2.887	2.767	3.448	2.779	2.529	2.35	3.116	4.203	3.557	2.76
gravity (m^2/s)	9.81	9.81	9.81	9.81	9.81	9.81	9.81	9.81	9.81	9.81
Density (kg/m^3)	1702.642	1942.061	2033.498	1309.219	1775.017	1871.019	2050.324	1980.081	2097.782	1937.148
Ys (N/m^3)	16702.92	19051.62	19948.62	12843.44	17412.92	18354.7	20113.68	19424.6	20579.25	19003.42
Ys (kN/m^3)	16.70292	19.05162	19.94862	12.84344	17.41292	18.3547	20.11368	19.4246	20.57925	19.00342

Appendix A-2: Engineering Properties Test Results

Table A-2. 1: Unconfined compressive strength test result and analysis of test pit 1 at 2m depth.

Hawassa University

Civil Engineering Department

Geotechnical Laboratory

Unconfined Compressive Strength of Cohesive soil ASTM D 2166/AASHTO T 208

Date tasted:- Sep 5, 2017	Sample :- Undisturbed sample
Tested By :- Samuel A.	Depth :- 2m
Visual Classification :- Red soil	Location :- Dilla TTC

Diameter of Sample (cm):	3.8	qu=	74.86	kN/m2
Height of Sample (cm):	7.6	C=qu/2 =	37.43	kN/m2
Area of Sample (cm2):	11.34114948	Strain at failure =2%		
Volume of Test Sample (cm3):	86.19273604			

Moisture Content Determination

Discription	Specimen A	Specimen B
Weight of can+lid (gm):	16.9	17.5
Weight of wet soil+can+lid (gm):	73.12	58.54
Weight of dry soil+can+lid (gm):	59.7	48.2
Weight of dry soil (gm):	42.8	30.7
Weight of water (gm):	13.42	10.34
Moisture Content [%]	31.36	33.68
Average	32.52	

DDR	LDR	ΔL=DDR * 0.001	LDR * 0.00142 *10000	%ε=ΔL/Lo * 10	Ao	Ac=Ao/(1-ε)	Stress
0	0	0	0	0	11.3411495	11.34114948	0
20	28.33	0.02	402.286	0.263157895	11.3411495	11.37107336	35.37801
40	41	0.04	582.2	0.526315789	11.3411495	11.40115556	51.065
60	51.25	0.06	727.75	0.789473684	11.3411495	11.43139735	63.66238
80	57	0.08	809.4	1.052631579	11.3411495	11.46180001	70.61718
100	58.5	0.1	830.7	1.315789474	11.3411495	11.49236481	72.28277
120	60.75	0.12	862.65	1.578947368	11.3411495	11.52309305	74.86271
140	56.25	0.14	798.75	1.842105263	11.3411495	11.55398606	69.13199
160	55	0.16	781	2.105263158	11.3411495	11.58504517	67.4145
180	52.5	0.18	745.5	2.368421053	11.3411495	11.6162717	64.17722
200	43.75	0.2	621.25	2.631578947	11.3411495	11.64766703	53.33686

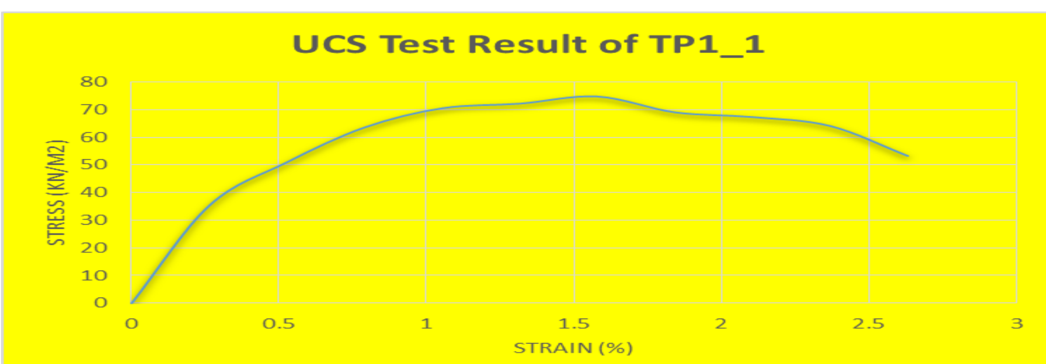


Figure A-2. 1: UCS test result of test pit 1 at 2m depth

Table A-2. 2: Unconfined compressive strength test result and analysis of test pit 1 at 4m depth.

	Hawassa University						
	Civil Engineering Department						
	Geotechnical Laboratory						
Unconfined Compressive Strength of Cohesive soil ASTM D 2166/AASHTO T 208							
Date tasted:- Sep 5, 2017				Sample :- Undisturbed sample			
Tested By :- Samuel A.				Depth :- 4m			
Visual Classification :- Red Clay soil				Location :- Dilla TTC			
Diameter of Sample (cm):			3.8	qu=	141.34	kN/m2	
Height of Sample (cm):			7.6	C = qu/2 =	70.67	kN/m2	
Area of Sample (cm2):			11.34114948	Strain at failure =2%			
Volume of Test Sample (cm3):			86.19273604				
Moisture Content Determination							
Discription			Specimen A	Specimen B			
Weight of can+lid (gm):			16.9	17.5			
Weight of wet soil+can+lid (gm):			68.99	57.55			
Weight of dry soil+can+lid (gm):			59.7	48.2			
Weight of dry soil (gm):			42.8	30.7			
Weight of water (gm):			9.29	9.35			
Moisture Content [%]			21.71	30.46			
Average			26.08				
DDR	LDR	ΔL=DDR * 0.001	LDR * 0.00142 *10000	%ε=ΔL/Lo * 100	Ao	Ac=Ao/(1-ε)	stress
0	0	0	0	0	11.3411495	11.34114948	0
20	38	0.02	539.6	0.263157895	11.3411495	11.37107336	47.45374
40	55.3	0.04	785.26	0.526315789	11.3411495	11.40115556	68.87547
60	72	0.06	1022.4	0.789473684	11.3411495	11.43139735	89.43788
80	89	0.08	1263.8	1.052631579	11.3411495	11.46180001	110.2619
100	98.5	0.1	1398.7	1.315789474	11.3411495	11.49236481	121.7069
120	107	0.12	1519.4	1.578947368	11.3411495	11.52309305	131.857
140	115	0.14	1633	1.842105263	11.3411495	11.55398606	141.3365
160	113	0.16	1604.6	2.105263158	11.3411495	11.58504517	138.5061
180	102	0.18	1448.4	2.368421053	11.3411495	11.6162717	124.6872
200	79	0.2	1121.8	2.631578947	11.3411495	11.64766703	96.31113

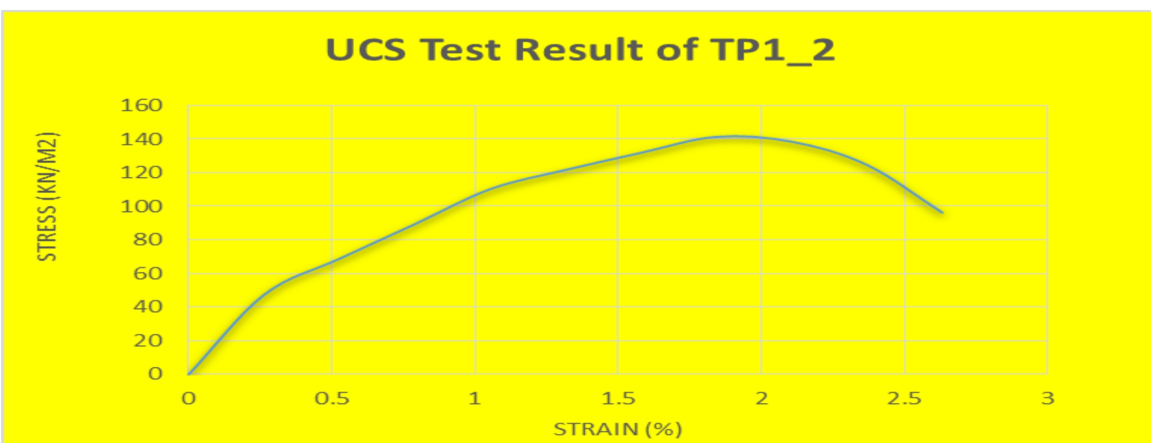


Figure A-2. 2: The plot of UCS test result of test pit 1 at 4m depth.

Table A-2. 3: Unconfined compressive strength test result and analysis of test pit 2 at 2m depth.

		Hawassa University					
		Civil Engineering Department					
		Geotechnical Laboratory					
Unconfined Compressive Strength of Cohesive soil ASTM D 2166/AASHTO T 208							
Date tasted:- Sep 5, 2017				Sample :- Undisturbed sample			
Tested By :- Samuel A.				Depth :- 2m			
Visual Classification :- Red Clay soil				Location :-Stadium			
Diameter of Sample (cm):			3.8	qu=	245.6543	kN/m2	
Height of Sample (cm):			7.6	C=qu/2 =	122.8272	kN/m2	
Area of Sample (cm2):			11.34114948	Strain at failure =3%			
Volume of Test Sample (cm3):			86.19273604				
Moisture Content Determination							
Discription			Specimen A	Specimen B			
Weight of can+lid (gm):			17.1	16.8			
Weight of wet soil+can+lid (gm):			65.61	52.89			
Weight of dry soil+can+lid (gm):			56.2	45			
Weight of dry soil (gm):			39.1	28.2			
Weight of water (gm):			9.41	7.89			
Moisture Content [%]			24.06649616	27.978723			
Average			26.02260978				

DDR	LDR	ΔL=DDR * 0.001	LDR * 0.00142 *10000	%ε=ΔL/Lo * 100	Ao	Ac=Ao/(1-ε)	Stress
0	0	0	0	0	11.34115	11.34114948	0
20	43	0.02	610.6	0.2631579	11.34115	11.37107336	53.69766
40	63	0.04	894.6	0.5263158	11.34115	11.40115556	78.46573
60	78	0.06	1107.6	0.7894737	11.34115	11.43139735	96.89104
80	92.5	0.08	1313.5	1.0526316	11.34115	11.46180001	114.5981
100	110.3	0.1	1566.26	1.3157895	11.34115	11.49236481	136.287
120	128.5	0.12	1824.7	1.5789474	11.34115	11.52309305	158.3516
140	142.5	0.14	2023.5	1.8421053	11.34115	11.55398606	175.1344
160	163	0.16	2314.6	2.1052632	11.34115	11.58504517	199.7921
180	185	0.18	2627	2.3684211	11.34115	11.6162717	226.1483
200	201.5	0.2	2861.3	2.6315789	11.34115	11.64766703	245.6543
220	195	0.22	2769	2.8947368	11.34115	11.67923253	237.0875
240	172.5	0.24	2449.5	3.1578947	11.34115	11.71096957	209.1629
260	161	0.26	2286.2	3.4210526	11.34115	11.74287957	194.6882
280	147	0.28	2087.4	3.6842105	11.34115	11.77496394	177.2744
300	53	0.3	752.6	3.9473684	11.34115	11.80722412	63.74064
320	46.5	0.32	660.3	4.2105263	11.34115	11.83966154	55.77018
340	43	0.34	610.6	4.4736842	11.34115	11.87227769	51.43074
360	38	0.36	539.6	4.7368421	11.34115	11.90507404	45.32521

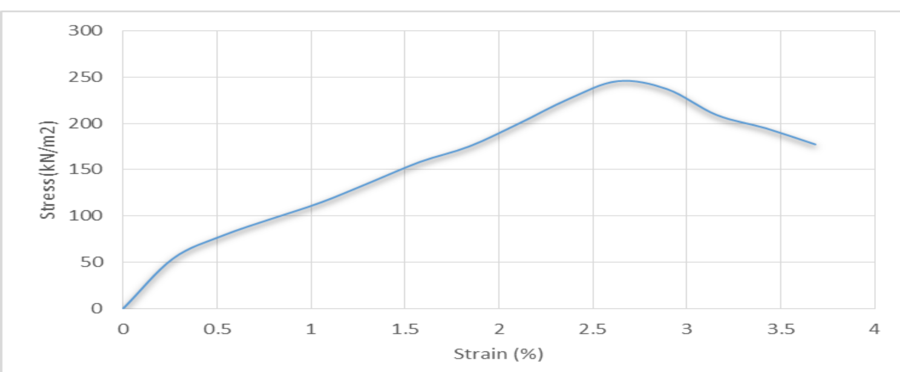


Figure A-2. 3: The plot of UCS test result of test pit 2 at 2m depth

Table A-2. 4: Unconfined compressive strength test result and analysis of test pit 2 at 4m depth.

		Hawassa University					
		Civil Engineering Department					
		Geotechnical Laboratory					
Unconfined Compressive Strength of Cohesive soil ASTM D 2166/AASHTO T 208							
Date tasted:- Sep 3, 2017				Sample :- Undisturbed sample			
Tested By :- Samuel A.				Depth :- 4m			
Visual Classification :- Red Clay soil				Location :-Stadium			
Diameter of Sample (cm):			3.8	qu=	288.32	kN/m2	
Height of Sample (cm):			7.6	C = qu/2 =	144.16	kN/m2	
Area of Sample (cm2):			11.34114948	Strain at failure =3%			
Volume of Test Sample (cm3):			86.19273604				
Moisture Content Determination							
Discription			Specimen A	Specimen B			
Weight of can+lid (gm):			15.9	16.5			
Weight of wet soil+can+lid (gm):			64.7	69.2			
Weight of dry soil+can+lid (gm):			54.6	57.2			
Weight of dry soil (gm):			38.7	40.7			
Weight of water (gm):			10.1	12			
Moisture Content [%]			26.09819121	29.484029			
Average			27.79111035				
DDR	LDR	ΔL=DDR * 0.001	LDR * 0.00142 *10000	%ε=ΔL/Lo * 100	Ao	Ac=Ao/(1-ε)	stress
0	0	0	0	0	11.34115	11.34114948	0
20	39.5	0.02	560.9	0.2631579	11.34115	11.37107336	49.32692
40	62	0.04	880.4	0.5263158	11.34115	11.40115556	77.22024
60	81.5	0.06	1157.3	0.7894737	11.34115	11.43139735	101.2387
80	105	0.08	1491	1.0526316	11.34115	11.46180001	130.0843
100	131.5	0.1	1867.3	1.3157895	11.34115	11.49236481	162.4818
120	157	0.12	2229.4	1.5789474	11.34115	11.52309305	193.4724
140	182	0.14	2584.4	1.8421053	11.34115	11.55398606	223.6804
160	198	0.16	2811.6	2.1052632	11.34115	11.58504517	242.6922
180	215	0.18	3053	2.3684211	11.34115	11.6162717	262.821
200	236.5	0.2	3358.3	2.6315789	11.34115	11.64766703	288.3238
220	221	0.22	3138.2	2.8947368	11.34115	11.67923253	268.6992
240	211.5	0.24	3003.3	3.1578947	11.34115	11.71096957	256.4519
260	189	0.26	2683.8	3.4210526	11.34115	11.74287957	228.547
280	170.5	0.28	2421.1	3.6842105	11.34115	11.77496394	205.6142

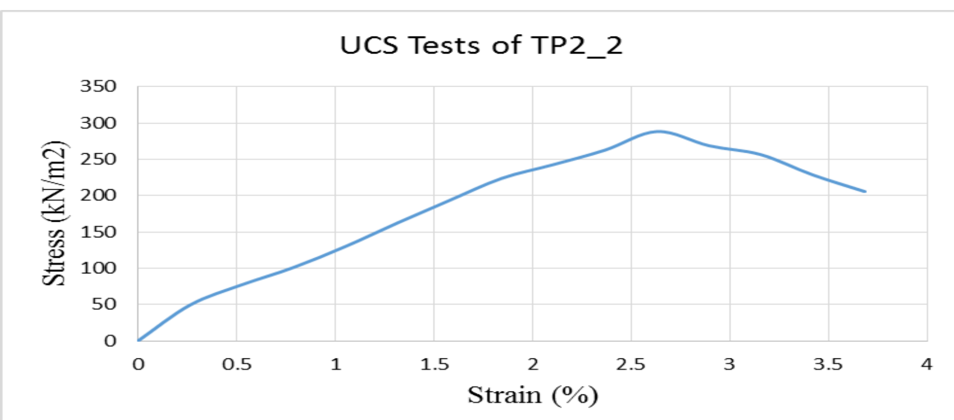


Figure A-2. 4: The plot of UCS test result of test pit 2 at 4m depth

Table A-2. 5: Unconfined compressive strength test result and analysis of test pit 3 at 2m depth.

		Hawassa University					
		Civil Engineering Department					
		Geotechnical Laboratory					
Unconfined Compressive Strength of Cohesive soil ASTM D 2166/AASHTO T 208							
Date tasted:- Sep 5, 2017				Sample :- Undisturbed sample			
Tested By :- Samuel A.				Depth :- 2m			
Visual Classification :- Red Clay soil				Location :- Dilla High school branch			
Diameter of Sample (cm):			3.8	qu=	80.77	kN/m2	
Height of Sample (cm):			7.6	C=qu/2 =	40.38	kN/m2	
Area of Sample (cm2):			11.34114948	Strain at failure =3%			
Volume of Test Sample (cm3):			86.19273604				
Moisture Content Determination							
Discription			Specimen A	Specimen B			
Weight of can+lid (gm):			15.6	11			
Weight of wet soil+can+lid (gm):			54.98	38.12			
Weight of dry soil+can+lid (gm):			45.7	31.6			
Weight of dry soil (gm):			30.1	20.6			
Weight of water (gm):			9.28	6.52			
Moisture Content [%]			30.83	31.65			
Average			31.24				
DDR	LDR	ΔL=DDR * 0.001	LDR * 0.00142 *10000	%ε=ΔL/Lo * 100	Ao	Ac=Ao/(1-ε)	Stress
0	0	0	0	0	11.3411495	11.34114948	0
20	17	0.02	241.4	0.263157895	11.3411495	11.37107336	21.22930637
40	23.5	0.04	333.7	0.526315789	11.3411495	11.40115556	29.26896298
60	33.25	0.06	472.15	0.789473684	11.3411495	11.43139735	41.30291209
80	42.5	0.08	603.5	1.052631579	11.3411495	11.46180001	52.65316091
100	51.75	0.1	734.85	1.315789474	11.3411495	11.49236481	63.94245331
120	55.25	0.12	784.55	1.578947368	11.3411495	11.52309305	68.08501817
140	60.25	0.14	855.55	1.842105263	11.3411495	11.55398606	74.04803807
160	64	0.16	908.8	2.105263158	11.3411495	11.58504517	78.44596088
180	64.75	0.18	919.45	2.368421053	11.3411495	11.6162717	79.15190204
200	66.25	0.2	940.75	2.631578947	11.3411495	11.64766703	80.76724698
220	63.75	0.22	905.25	2.894736842	11.3411495	11.67923253	77.50937384
240	59	0.24	837.8	3.157894737	11.3411495	11.71096957	71.53976406
260	54.5	0.26	773.9	3.421052632	11.3411495	11.74287957	65.90376708

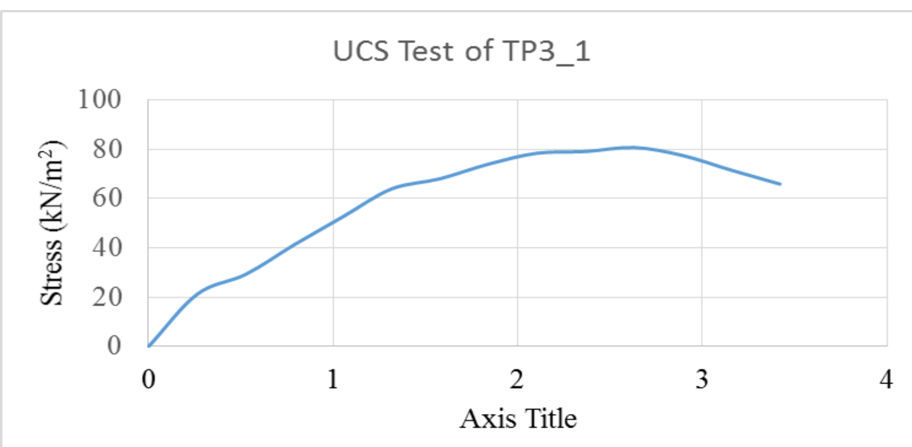


Figure A-2. 5: The plot of UCS test result of test pit 3 at 4m depth

Table A-2. 6: Unconfined compressive strength test result and analysis of test pit 4 at 4m depth.

		Hawassa University					
		Civil Engineering Department					
		Geotechnical Laboratory					
Unconfined Compressive Strength of Cohesive soil ASTM D 2166/AASHTO T 208							
Date tasted:- Sep 5, 2017				Sample :- Undisturbed sample			
Tested By :- Samuel A.				Depth :- 4m			
Visual Classification :- Red Clay soil				Location :- Dilla Mekane Eyesus Church			
Diameter of Sample (cm):			3.8	qu=	63.12	kN/m2	
Height of Sample (cm):			7.6	C = qu/2 =	31.56	kN/m2	
Area of Sample (cm2):			11.34114948	Strain at failure =2%			
Volume of Test Sample (cm3):			86.19273604				
Moisture Content Determination							
Discription			Specimen A	Specimen B			
Weight of can+lid (gm):			16.1	11.2			
Weight of wet soil+can+lid (gm):			52.1	40.6			
Weight of dry soil+can+lid (gm):			43.3	33.5			
Weight of dry soil (gm):			27.2	22.3			
Weight of water (gm):			8.8	7.1			
Moisture Content [%]			32.35294118	31.838565			
Average			32.0957531				
DDR	LDR	ΔL=DDR * 0.001	LDR * 0.00142 *10000	%ε=ΔL/Lo * 100	Ao	Ac=Ao/(1-ε)	stress
0	0	0	0	0	11.34115	11.34114948	0
20	14	0.02	198.8	0.26315789	11.34115	11.37107336	17.48296
40	28.25	0.04	401.15	0.52631579	11.34115	11.40115556	35.18503
60	34.75	0.06	493.45	0.78947368	11.34115	11.43139735	43.1662
80	40.25	0.08	571.55	1.05263158	11.34115	11.46180001	49.86564
100	44.25	0.1	628.35	1.31578947	11.34115	11.49236481	54.67543
120	48.25	0.12	685.15	1.57894737	11.34115	11.52309305	59.45886
140	51	0.14	724.2	1.84210526	11.34115	11.55398606	62.67967
160	51.5	0.16	731.3	2.10526316	11.34115	11.58504517	63.12448
180	48.5	0.18	688.7	2.36842105	11.34115	11.6162717	59.28753
200	43.5	0.2	617.7	2.63157895	11.34115	11.64766703	53.03208
220	38	0.22	539.6	2.89473684	11.34115	11.67923253	46.20167

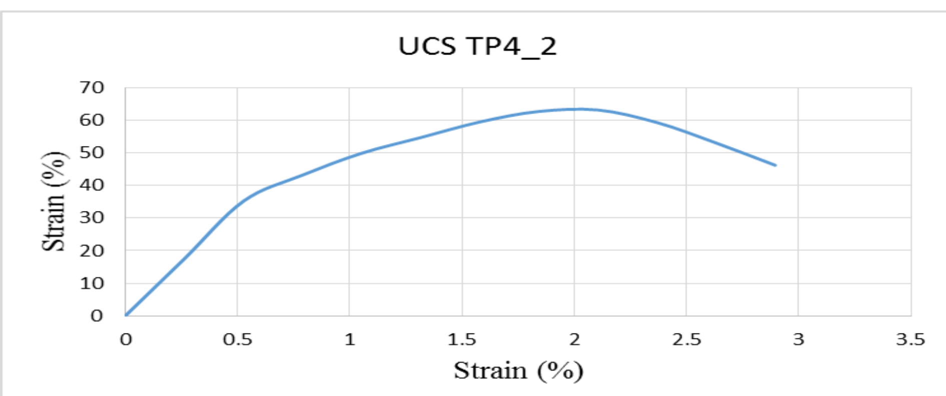


Figure A-2. 6: The plot of UCS test result of test pit 4 at 4m depth

Table A-2. 7: Unconfined compressive strength test result and analysis of test pit 5 at 2m depth.

Hawassa University							
Civil Engineering Department							
Geotechnical Laboratory							
Unconfined Compressive Strength of Cohesive soil ASTM D 2166/AASHTO T 208							
Date tasted:- Sep 5, 2017				Sample :- Undisturbed sample			
Tested By :- Samuel A.				Depth :- 2m			
Visual Classification :- Red Clay soil				Location :-Kofe Primary School			
Diameter of Sample (cm):			3.8	qu=	73.31	kN/m2	
Height of Sample (cm):			7.6	C=qu/2 =	36.66	kN/m2	
Area of Sample (cm2):			11.34114948	Strain at failure =3%			
Volume of Test Sample (cm3):			86.19273604				
Moisture Content Determination							
Discription			Specimen A	Specimen B			
Weight of can+lid (gm):			15.4	15.5			
Weight of wet soil+can+lid (gm)			51.6	47.9			
Weight of dry soil+can+lid (gm)			42.5	39.7			
Weight of dry soil (gm):			27.1	24.2			
Weight of water (gm):			9.1	8.2			
Moisture Content [%]			33.57933579	33.8842975			
Average			33.73181666				

DDR	LDR	ΔL=DDR * 0.001	LDR * 0.00142 *10000	%ε=ΔL/Lo * 100	Ao	Ac=Ao/(1-ε)	Stress
0	0	0	0	0	11.34115	11.34114948	0
20	18	0.02	255.6	0.26315789	11.34115	11.37107336	22.47809
40	27	0.04	383.4	0.52631579	11.34115	11.40115556	33.62817
60	34	0.06	482.8	0.78947368	11.34115	11.43139735	42.23456
80	39.5	0.08	560.9	1.05263158	11.34115	11.46180001	48.93647
100	47	0.1	667.4	1.31578947	11.34115	11.49236481	58.07334
120	50	0.12	710	1.57894737	11.34115	11.52309305	61.6154
140	54.5	0.14	773.9	1.84210526	11.34115	11.55398606	66.98121
160	58	0.16	823.6	2.10526316	11.34115	11.58504517	71.09165
180	59	0.18	837.8	2.36842105	11.34115	11.6162717	72.12297
200	60	0.2	852	2.63157895	11.34115	11.64766703	73.1477
220	60.3	0.22	856.26	2.89473684	11.34115	11.67923253	73.31475
240	59	0.24	837.8	3.15789474	11.34115	11.71096957	71.53976
260	58	0.26	823.6	3.42105263	11.34115	11.74287957	70.13612
280	55	0.28	781	3.68421053	11.34115	11.77496394	66.32717
300	53	0.3	752.6	3.94736842	11.34115	11.80722412	63.74064

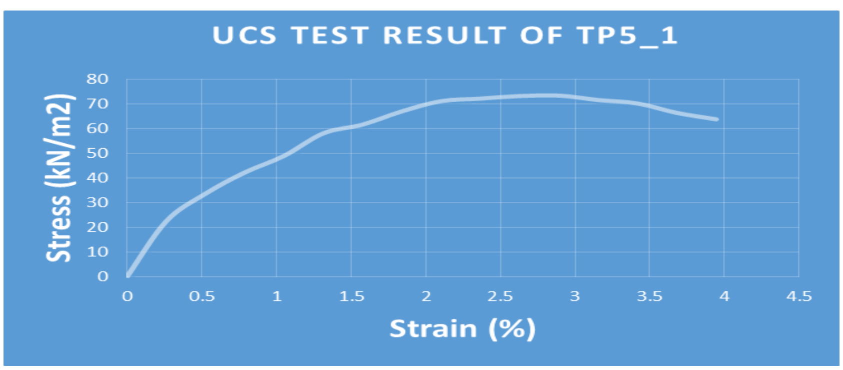


Figure A-2. 7: The plot of UCS test result of test pit 4 at 4m depth

Table A-2. 8: Unconfined compressive strength test result and analysis of test pit 5 at 2m depth

		Hawassa University					
		Civil Engineering Department					
		Geotechnical Laboratory					
Unconfined Compressive Strength of Cohesive soil ASTM D 2166/AASHTO T 208							
Date tasted:- Aug 14, 2017				Sample :- Undisturbed sample			
Tested By :- Samuel A.				Depth :- 2m			
Visual Classification :- Red Clay soil				Location :- HanByul			
Diameter of Sample (cm):			3.8	qu=	150.55	kN/m2	
Height of Sample (cm):			7.6	C=qu/2 =	75.28	kN/m2	
Area of Sample (cm2):			11.34114948	Strain at failure =2%			
Volume of Test Sample (cm3):			86.19273604				
Moisture Content Determination							
Discription			Specimen A	Specimen B			
Weight of can+lid (gm):			16.1	15.30			
Weight of wet soil+can+lid (gm):			60.8	52.30			
Weight of dry soil+can+lid (gm):			52	45.10			
Weight of dry soil (gm):			35.9	29.80			
Weight of water (gm):			8.8	7.20			
Moisture Content [%]			24.51	24.16			
Average			24.34				
DDR	LDR	ΔL=DDR * 0.001	LDR * 0.00142 *10000	%ε=ΔL/Lo * 100	Ao	Ac=Ao/(1-ε)	Stress
0	0	0	0	0	11.34114948	11.3411495	0
20	35	0.02	497	0.263157895	11.34114948	11.3710734	43.7074
40	48.5	0.04	688.7	0.526315789	11.34114948	11.4011556	60.40616
60	62	0.06	880.4	0.789473684	11.34114948	11.4313974	77.01596
80	70.5	0.08	1001.1	1.052631579	11.34114948	11.4618	87.3423
100	89.5	0.1	1270.9	1.315789474	11.34114948	11.4923648	110.5865
120	102.5	0.12	1455.5	1.578947368	11.34114948	11.5230931	126.3116
140	122.5	0.14	1739.5	1.842105263	11.34114948	11.5539861	150.5541
160	120.5	0.16	1711.1	2.105263158	11.34114948	11.5850452	147.699
180	110	0.18	1562	2.368421053	11.34114948	11.6162717	134.4666
200	100.5	0.2	1427.1	2.631578947	11.34114948	11.647667	122.5224
220	95	0.22	1349	2.894736842	11.34114948	11.6792325	115.5042
240	82.5	0.24	1171.5	3.157894737	11.34114948	11.7109696	100.0344

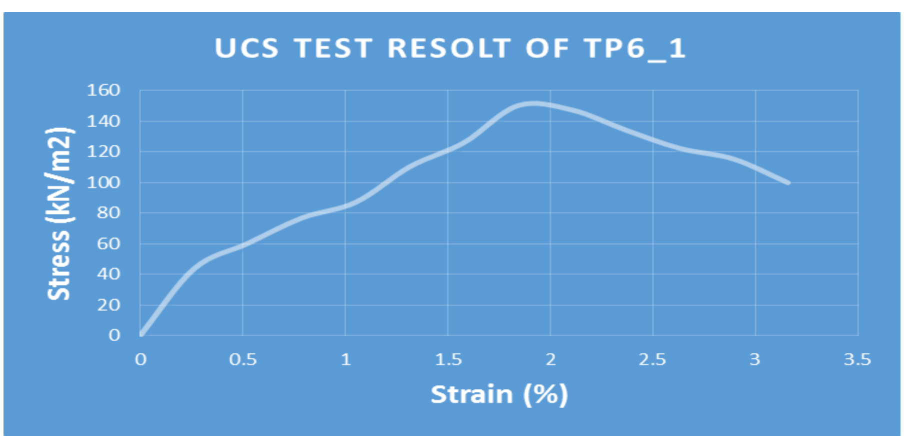


Figure A-2. 8: The plot of UCS test result of test pit 6 at 2m depth

Table A-2. 9: Unconfined compressive strength test result and analysis of test pit 7 at 7m depth.

		Hawassa University					
		Civil Engineering Department					
		Geotechnical Laboratory					
Unconfined Compressive Strength of Cohesive soil ASTM D 2166/AASHTO T 208							
Date tasted:- Aug14, 2017				Sample :- Undisturbed sample			
Tested By :- Samuel A.				Depth :- 4m			
Visual Classification :- Red Clay soil				Location :-Haroressa School			
Diameter of Sample (cm):			3.8	qu=	292.59	kN/m2	
Height of Sample (cm):			7.6	C = qu/2 =	146.30	kN/m2	
Area of Sample (cm2):			11.34114948	Strain at failure =3%			
Volume of Test Sample (cm3):			86.19273604				
Moisture Content Determination							
Discription			Specimen A	Specimen B			
Weight of can+lid (gm):			16.3	15			
Weight of wet soil+can+lid (gm):			61	63			
Weight of dry soil+can+lid (gm):			53.2	54.6			
Weight of dry soil (gm):			36.9	39.6			
Weight of water (gm):			7.8	8.4			
Moisture Content [%]			21.13821138	21.21212121			
Average			21.1751663				

DDR	LDR	ΔL=DDR * 0.001	LDR * 0.00142 *10000	%ε=ΔL/Lo * 100	Ao	Ac=Ao/(1-ε)	stress
0	0	0	0	0	11.34115	11.34114948	0
20	55	0.02	781	0.263157895	11.34115	11.37107336	68.68305
40	78	0.04	1107.6	0.526315789	11.34115	11.40115556	97.14805
60	109.5	0.06	1554.9	0.789473684	11.34115	11.43139735	136.0201
80	126	0.08	1789.2	1.052631579	11.34115	11.46180001	156.1011
100	150	0.1	2130	1.315789474	11.34115	11.49236481	185.3404
120	165.5	0.12	2350.1	1.578947368	11.34115	11.52309305	203.947
140	191	0.14	2712.2	1.842105263	11.34115	11.55398606	234.7415
160	208.5	0.16	2960.7	2.105263158	11.34115	11.58504517	255.5622
180	231.5	0.18	3287.3	2.368421053	11.34115	11.6162717	282.991
200	240	0.2	3408	2.631578947	11.34115	11.64766703	292.5908
220	232.5	0.22	3301.5	2.894736842	11.34115	11.67923253	282.6812
240	219	0.24	3109.8	3.157894737	11.34115	11.71096957	265.5459
260	208.5	0.26	2960.7	3.421052632	11.34115	11.74287957	252.1273
280	193	0.28	2740.6	3.684210526	11.34115	11.77496394	232.7481
300	154	0.3	2186.8	3.947368421	11.34115	11.80722412	185.2086
320	142	0.32	2016.4	4.210526316	11.34115	11.83966154	170.3089

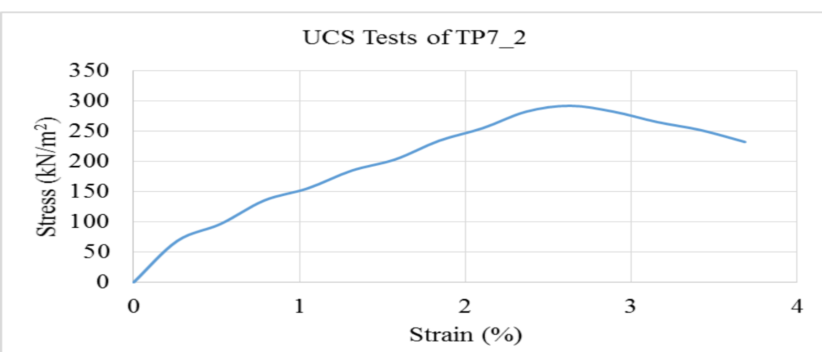


Figure A-2. 9: The plot of UCS test result of test pit 7at 4m depth

Table A-2. 10: Unconfined compressive strength test result and analysis of test pit 8 at 2m depth.

Hawassa University			
Civil Engineering Department			
Geotechnical Laboratory			
Unconfined Compressive Strength of Cohesive soil ASTM D 2166/AASHTO T 208			
Date tasted:- Aug 14, 2017		Sample :- Undisturbed sample	
Tested By :- Samuel A.		Depth :- 2m	
Visual Classification :- Red Clay soil		Location :-New Prison	
Diameter of Sample (cm):	3.8	qu=	294.60 kN/m2
Height of Sample (cm):	7.6	C=qu/2 =	147.30 kN/m2
Area of Sample (cm2):	11.3411495	Strain at failure =2%	
Volume of Test Sample (cm3):	86.192736		

Moisture Content Determination		
Discription	Specimen A	Specimen B
Weight of can+lid (gm):	15.6	15.30
Weight of wet soil+can+lid (gm):	50.4	45.30
Weight of dry soil+can+lid (gm):	43.7	39.50
Weight of dry soil (gm):	28.1	24.20
Weight of water (gm):	6.7	5.80
Moisture Content [%]	23.84	23.97
Average	23.91	

DDR	LDR	ΔL=DDR * 0.001	LDR * 0.00142 *10000	%ε=ΔL/Lo * 100	Ao	Ac=Ao/(1-ε)	Stress
0	0	0	0	0	11.3411495	11.3411495	0
20	49	0.02	695.8	0.263157895	11.3411495	11.3710734	61.19035
40	73	0.04	1036.6	0.526315789	11.3411495	11.4011556	90.92061
60	101.5	0.06	1441.3	0.789473684	11.3411495	11.4313974	126.0826
80	123	0.08	1746.6	1.052631579	11.3411495	11.4618	152.3844
100	146.5	0.1	2080.3	1.315789474	11.3411495	11.4923648	181.0158
120	168	0.12	2385.6	1.578947368	11.3411495	11.5230931	207.0277
140	196.5	0.14	2790.3	1.842105263	11.3411495	11.5539861	241.5011
160	218.5	0.16	3102.7	2.105263158	11.3411495	11.5850452	267.8194
180	241	0.18	3422.2	2.368421053	11.3411495	11.6162717	294.604
200	240.5	0.2	3415.1	2.631578947	11.3411495	11.647667	293.2003
220	230.5	0.22	3273.1	2.894736842	11.3411495	11.6792325	280.2496
240	216	0.24	3067.2	3.157894737	11.3411495	11.7109696	261.9083
260	195	0.26	2769	3.421052632	11.3411495	11.7428796	235.8025
280	181	0.28	2570.2	3.684210526	11.3411495	11.7749639	218.2767

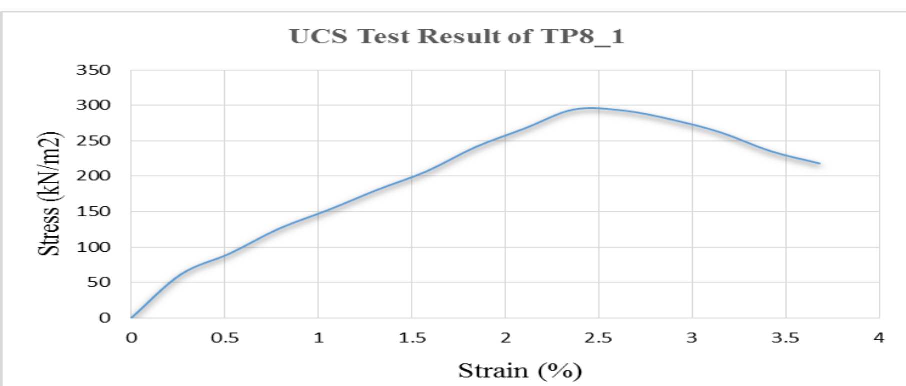


Figure A-2. 10: The plot of UCS test result of test pit 8 at 2m depth

Table A-2. 11: Unconfined compressive strength test result and analysis of test pit 9 at 4m depth.

	Hawassa University			
	Civil Engineering Department			
	Geotechnical Laboratory			
Unconfined Compressive Strength of Cohesive soil ASTM D 2166/AASHTO T 208				
Date tasted:- Aug 14, 2017		Sample :- Undisturbed sample		
Tested By :- Samuel A.		Depth :- 4m		
Visual Classification :- Red Clay soil		Location :- St. Michael Church		
Diameter of Sample (cm):	3.8	qu=	380.91	kN/m2
Height of Sample (cm):	7.6	C = qu/2 =	190.46	kN/m2
Area of Sample (cm2):	11.34114948	Strain at failure =3%		
Volume of Test Sample (cm3):	86.19273604			
Moisture Content Determination				
Discription	Specimen A	Specimen B		
Weight of can+lid (gm):	15.8	15.4		
Weight of wet soil+can+lid (gm):	50.6	44.9		
Weight of dry soil+can+lid (gm):	44.8	40		
Weight of dry soil (gm):	29	24.6		
Weight of water (gm):	5.8	4.9		
Moisture Content [%]	20	19.9186992		
Average	19.95934959			

DDR	LDR	ΔL=DDR * 0.001	LDR * 0.00142 *10000	%ε=ΔL/Lo * 100	Ao	Ac=Ao/(1-ε)	stress
0	0	0	0	0	11.34115	11.34114948	0
20	45	0.02	639	0.26315789	11.34115	11.37107336	56.19522
40	70	0.04	994	0.52631579	11.34115	11.40115556	87.18415
60	91	0.06	1292.2	0.78947368	11.34115	11.43139735	113.0395
80	113.5	0.08	1611.7	1.05263158	11.34115	11.46180001	140.6149
100	138	0.1	1959.6	1.31578947	11.34115	11.49236481	170.5132
120	156	0.12	2215.2	1.57894737	11.34115	11.52309305	192.2401
140	186	0.14	2641.2	1.84210526	11.34115	11.55398606	228.5964
160	210	0.16	2982	2.10526316	11.34115	11.58504517	257.4008
180	231	0.18	3280.2	2.36842105	11.34115	11.6162717	282.3798
200	258	0.2	3663.6	2.63157895	11.34115	11.64766703	314.5351
220	286.5	0.22	4068.3	2.89473684	11.34115	11.67923253	348.3362
240	310	0.24	4402	3.15789474	11.34115	11.71096957	375.8869
260	315	0.26	4473	3.42105263	11.34115	11.74287957	380.9117
280	305	0.28	4331	3.68421053	11.34115	11.77496394	367.8143
300	289	0.3	4103.8	3.94736842	11.34115	11.80722412	347.5669
320	268	0.32	3805.6	4.21052632	11.34115	11.83966154	321.4281
340	241	0.34	3422.2	4.47368421	11.34115	11.87227769	288.2513

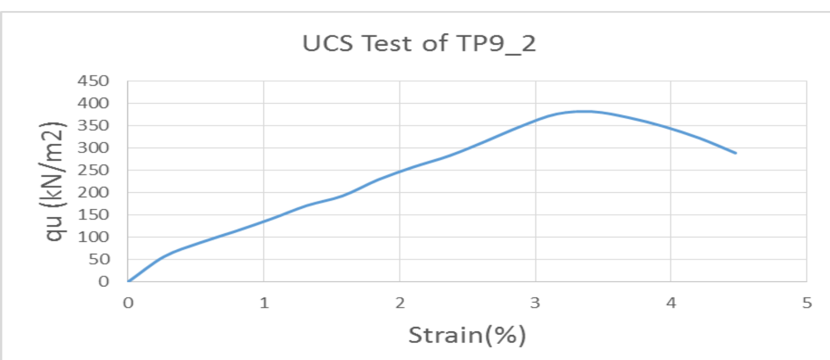


Figure A-2. 11: The plot of UCS test result of test pit 4 at 4m depth

Table A-2. 12: Unconfined compressive strength test result and analysis of test pit 10 at 2m depth.

	Hawassa University						
	Civil Engineering Department						
	Geotechnical Laboratory						
Unconfined Compressive Strength of Cohesive soil ASTM D 2166/AASHTO T 208							
Date tasted:- Aug 14, 2017				Sample :- Undisturbed sample			
Tested By :- Samuel A.				Depth :- 2m			
Visual Classification :- Red Clay soil				Location :-Old Prison			
Diameter of Sample (cm):			3.8	qu=	151.17	kN/m2	
Height of Sample (cm):			7.6	C=qu/2 =	75.59	kN/m2	
Area of Sample (cm2):			11.34114948	Strain at failure =3%			
Volume of Test Sample (cm3):			86.19273604				
Moisture Content Determination							
Discription			Specimen A	Specimen B			
Weight of can+lid (gm):			16.4	15.80			
Weight of wet soil+can+lid (gm)			53.41	54.82			
Weight of dry soil+can+lid (gm)			46.1	47.30			
Weight of dry soil (gm):			29.7	31.50			
Weight of water (gm):			7.31	7.52			
Moisture Content [%]			24.61279461	23.87			
Average			24.24290524				
DDR	LDR	ΔL=DDR * 0.001	LDR * 0.00142 *10000	%ε=ΔL/Lo * 100	Ao	Ac=Ao/(1-ε)	Stress
0	0	0	0	0	11.34115	11.34114948	0
20	23	0.02	326.6	0.26315789	11.34115	11.37107336	28.722
40	38	0.04	539.6	0.52631579	11.34115	11.40115556	47.32854
60	49	0.06	695.8	0.78947368	11.34115	11.43139735	60.86745
80	59.5	0.08	844.9	1.05263158	11.34115	11.46180001	73.71443
100	71	0.1	1008.2	1.31578947	11.34115	11.49236481	87.72781
120	82.5	0.12	1171.5	1.57894737	11.34115	11.52309305	101.6654
140	98	0.14	1391.6	1.84210526	11.34115	11.55398606	120.4433
160	105	0.16	1491	2.10526316	11.34115	11.58504517	128.7004
180	114	0.18	1618.8	2.36842105	11.34115	11.6162717	139.3562
200	124	0.2	1760.8	2.63157895	11.34115	11.64766703	151.1719
220	121.5	0.22	1725.3	2.89473684	11.34115	11.67923253	147.7237
240	112	0.24	1590.4	3.15789474	11.34115	11.71096957	135.8043
260	103.5	0.26	1469.7	3.42105263	11.34115	11.74287957	125.1567
280	96.5	0.28	1370.3	3.68421053	11.34115	11.77496394	116.374
300	82	0.3	1164.4	3.94736842	11.34115	11.80722412	98.61759
320	71	0.32	1008.2	4.21052632	11.34115	11.83966154	85.15446

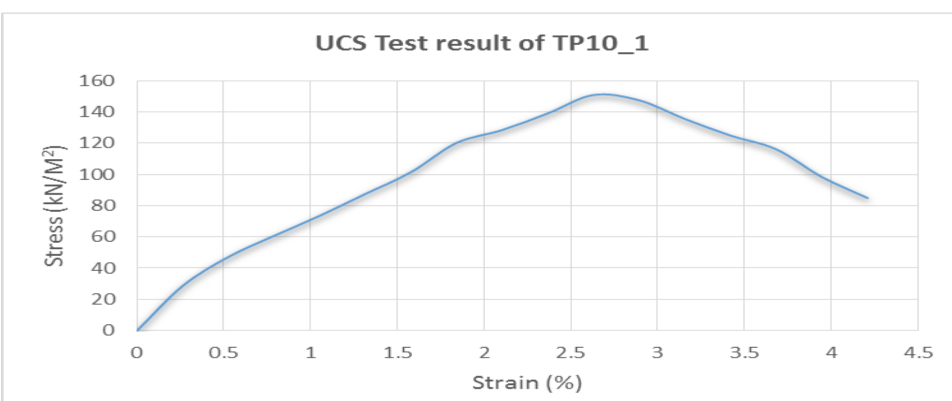


Figure A-2. 12: The plot of UCS test result of test pit 10 at 2m depth

APPENDIX B: DETAILS OF REGRESSION ANALYSIS OUTPUT

Appendix B-1: Single Linear Regression Analysis Outputs

Table B-1. 1**:** Unconfined Compressive Strength (UCS) as a Function of Percent of Pass on #200 Sieve (P_{200})

Model Summary[b]

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate	Durbin-Watson
1	.015[a]	.000	-.062	102.2738058	1.141

a. Predictors: (Constant), Percent Passing Sieve #200

b. Dependent Variable: Unconfined Compressive Strength

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	36.729	1	36.729	.004	.953[b]
	Residual	167358.902	16	10459.931		
	Total	167395.630	17			

a. Dependent Variable: Unconfined Compressive Strength

b. Predictors: (Constant), Percent Passing Sieve #200

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.	Collinearity Statistics	
		B	Std. Error	Beta			Tolerance	VIF
1	(Constant)	202.423	249.362		.812	.429		
	Percent Passing Sieve #200	.225	3.789	.015	.059	.953	1.000	1.000

a. Dependent Variable: Unconfined Compressive Strength

Collinearity Diagnostics[a]

Model	Dimension	Eigenvalue	Condition Index	Variance Proportions	
				(Constant)	Percent Passing Sieve #200
1	1	1.995	1.000	.00	.00
	2	.005	20.640	1.00	1.00

a. Dependent Variable: Unconfined Compressive Strength

Table B-1. 2**:** Unconfined Compressive Strength (UCS) as a Function of Liquid Limit (LL)

Model Summary[b]

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate	Durbin-Watson
1	.308[a]	.095	.038	97.3279345	1.197

a. Predictors: (Constant), Liquid Limit

b. Dependent Variable: Unconfined Compressive Strength

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	15832.001	1	15832.001	1.671	.214[b]
	Residual	151563.629	16	9472.727		
	Total	167395.630	17			

a. Dependent Variable: Unconfined Compressive Strength

b. Predictors: (Constant), Liquid Limit

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.	Collinearity Statistics	
		B	Std. Error	Beta			Tolerance	VIF
1	(Constant)	794.486	447.182		1.777	.095		
	Liquid Limit	-10.169	7.866	-.308	-1.293	.214	1.000	1.000

a. Dependent Variable: Unconfined Compressive Strength

Collinearity Diagnostics[a]

Model	Dimension	Eigenvalue	Condition Index	Variance Proportions	
				(Constant)	Liquid Limit
1	1	1.999	1.000	.00	.00
	2	.001	38.961	1.00	1.00

a. Dependent Variable: Unconfined Compressive Strength

Table B-1. 3**:** Unconfined Compressive Strength (UCS) as a Function of Plastic Limit (PL)

Model Summary[b]

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate	Durbin-Watson
1	.271[a]	.074	.016	98.4488360	1.237

a. Predictors: (Constant), Plastic limit

b. Dependent Variable: Unconfined Compressive Strength

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	12320.857	1	12320.857	1.271	.276[b]
	Residual	155074.773	16	9692.173		
	Total	167395.630	17			

a. Dependent Variable: Unconfined Compressive Strength

b. Predictors: (Constant), Plastic limit

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.	Collinearity Statistics	
		B	Std. Error	Beta			Tolerance	VIF
1	(Constant)	411.601	174.036		2.365	.031		
	Plastic limit	-5.646	5.008	-.271	-1.127	.276	1.000	1.000

a. Dependent Variable: Unconfined Compressive Strength

Collinearity Diagnostics[a]

Model	Dimension	Eigenvalue	Condition Index	Variance Proportions	
				(Constant)	Plastic limit
1	1	1.991	1.000	.00	.00
	2	.009	14.933	1.00	1.00

a. Dependent Variable: Unconfined Compressive Strength

Table B-1. 4**:** Unconfined Compressive Strength (UCS) as a Function of Plasticity Index (PI)

Model Summary[b]

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate	Durbin-Watson
1	.079[a]	.006	-.056	101.9661458	1.140

a. Predictors: (Constant), Plasticity Index

b. Dependent Variable: Unconfined Compressive Strength

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	1042.112	1	1042.112	.100	.756[b]
	Residual	166353.518	16	10397.095		
	Total	167395.630	17			

a. Dependent Variable: Unconfined Compressive Strength

b. Predictors: (Constant), Plasticity Index

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.	Collinearity Statistics	
		B	Std. Error	Beta			Tolerance	VIF
1	(Constant)	182.679	111.442		1.639	.121		
	Plasticity Index	1.546	4.884	.079	.317	.756	1.000	1.000

a. Dependent Variable: Unconfined Compressive Strength

Collinearity Diagnostics[a]

Model	Dimension	Eigenvalue	Condition Index	Variance Proportions	
				(Constant)	Plasticity Index
1	1	1.976	1.000	.01	.01
	2	.024	9.165	.99	.99

a. Dependent Variable: Unconfined Compressive Strength

Table B-1. 5**:** Unconfined Compressive Strength (UCS) as a Function of Natural Moisture Content (w_n)

Model Summary[b]

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate	Durbin-Watson
1	.824[a]	.679	.659	57.9639598	1.723

a. Predictors: (Constant), Natural Moisture Content

b. Dependent Variable: Unconfined Compressive Strength

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	113638.500	1	113638.500	33.823	.000[b]
	Residual	53757.130	16	3359.821		
	Total	167395.630	17			

a. Dependent Variable: Unconfined Compressive Strength

b. Predictors: (Constant), Natural Moisture Content

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.	Collinearity Statistics	
		B	Std. Error	Beta			Tolerance	VIF
1	(Constant)	666.159	78.409		8.496	.000		
	Natural Moisture Content	-17.382	2.989	-.824	-5.816	.000	1.000	1.000

a. Dependent Variable: Unconfined Compressive Strength

Table B-1. 6**:** Unconfined Compressive Strength (UCS) as a Function of Specific Gravity (Gs)

Model Summary[b]

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate	Durbin-Watson
1	.139[a]	.019	-.042	101.2963633	1.116

a. Predictors: (Constant), Specific Gravity

b. Dependent Variable: Unconfined Compressive Strength

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	3220.379	1	3220.379	.314	.583[b]
	Residual	164175.252	16	10260.953		
	Total	167395.630	17			

a. Dependent Variable: Unconfined Compressive Strength

b. Predictors: (Constant), Specific Gravity

Coefficients[a]

Model		Unstandardized Coefficients B	Unstandardized Coefficients Std. Error	Standardized Coefficients Beta	t	Sig.	Collinearity Statistics Tolerance	Collinearity Statistics VIF
1	(Constant)	-246.736	828.350		-.298	.770		
	Specific Gravity	170.330	304.041	.139	.560	.583	1.000	1.000

a. Dependent Variable: Unconfined Compressive Strength

Collinearity Diagnostics[a]

Model	Dimension	Eigenvalue	Condition Index	Variance Proportions (Constant)	Variance Proportions Specific Gravity
1	1	2.000	1.000	.00	.00
	2	.000	69.374	1.00	1.00

a. Dependent Variable: Unconfined Compressive Strength

Table B-1. 7**:** Unconfined Compressive Strength (UCS) as a Function of In-situ Unit Weight (γ_{ins})

Model Summary

R	R Square	Adjusted R Square	Std. Error of the Estimate
.946	.895	.881	34.184

The independent variable is In-situ Unnit Weight.

ANOVA

	Sum of Squares	df	Mean Square	F	Sig.
Regression	149867.380	2	74933.690	64.125	.000
Residual	17528.251	15	1168.550		
Total	167395.630	17			

The independent variable is In-situ Unnit Weight.

Coefficients

	Unstandardized Coefficients B	Unstandardized Coefficients Std. Error	Standardized Coefficients Beta	t	Sig.
In-situ Unnit Weight	-187.567	97.247	-3.491	-1.929	.073
In-situ Unnit Weight ** 2	6.730	2.756	4.419	2.442	.027
(Constant)	1375.209	849.979		1.618	.127

Appendix B-2: Multiple Linear Regression Analysis Outputs

Table B-2. 1**:** Unconfined Compressive Strength (q_u) as a Function of In-situ Unit Weight (γ_{ins}), and Natural Moisture Content (w_n).

Model Summary[b]

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate	Durbin-Watson
1	.931[a]	.866	.849	38.6038030	1.745

a. Predictors: (Constant), Natural Moisture Content, In-situ Unnit Weight

b. Dependent Variable: Unconfined Compressive Strength

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	145041.826	2	72520.913	48.663	.000[b]
	Residual	22353.804	15	1490.254		
	Total	167395.630	17			

a. Dependent Variable: Unconfined Compressive Strength

b. Predictors: (Constant), Natural Moisture Content, In-situ Unnit Weight

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.	Collinearity Statistics	
		B	Std. Error	Beta			Tolerance	VIF
1	(Constant)	-421.843	242.697		-1.738	.103		
	In-situ Unnit Weight	40.870	8.903	.761	4.590	.000	.324	3.084
	Natural Moisture Content	-4.191	3.496	-.199	-1.199	.249	.324	3.084

a. Dependent Variable: Unconfined Compressive Strength

Table B-2. 2**:** unconfined compressive strength (qu) as a function of natural moisture content, in-situ unit weight (γins) and specific gravity (Gs).

Model Summary[b]

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate	Durbin-Watson
1	.931[a]	.867	.839	39.8567435	1.790

a. Predictors: (Constant), Specific Gravity , In-situ Unnit Weight, Natural Moisture Content

b. Dependent Variable: Unconfined Compressive Strength

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	145155.790	3	48385.263	30.459	.000[b]
	Residual	22239.840	14	1588.560		
	Total	167395.630	17			

a. Dependent Variable: Unconfined Compressive Strength

b. Predictors: (Constant), Specific Gravity , In-situ Unnit Weight, Natural Moisture Content

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.	Collinearity Statistics	
		B	Std. Error	Beta			Tolerance	VIF
1	(Constant)	-311.774	481.312		-.648	.528		
	In-situ Unnit Weight	40.395	9.362	.752	4.315	.001	.313	3.199
	Natural Moisture Content	-4.517	3.808	-.214	-1.186	.255	.291	3.434
	Specific Gravity	-34.136	127.449	-.028	-.268	.793	.881	1.135

a. Dependent Variable: Unconfined Compressive Strength

Coefficient Correlations[a]

Model			Specific Gravity	In-situ Unnit Weight	Natural Moisture Content
1	Correlations	Specific Gravity	1.000	.190	.319
		In-situ Unnit Weight	.190	1.000	.825
		Natural Moisture Content	.319	.825	1.000
	Covariances	Specific Gravity	16243.244	226.278	154.896
		In-situ Unnit Weight	226.278	87.649	29.429
		Natural Moisture Content	154.896	29.429	14.503

a. Dependent Variable: Unconfined Compressive Strength

Table B-2. 3**:** Unconfined compressive strength (q_u) as a function of natural moisture content (w_n), percent of pass on #200 sieve (P_{200}) and in-situ unit weight (γ_{ins}).

Model Summary[b]

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate	Durbin-Watson
1	.933[a]	.870	.843	39.3539135	1.606

a. Predictors: (Constant), Percent Passing Sieve #200, In-situ Unnit Weight, Natural Moisture Content

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	145713.403	3	48571.134	31.362	.000[b]
	Residual	21682.227	14	1548.731		
	Total	167395.630	17			

a. Dependent Variable: Unconfined Compressive Strength

b. Predictors: (Constant), Percent Passing Sieve #200, In-situ Unnit Weight, Natural Moisture Content

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.	Collinearity Statistics	
		B	Std. Error	Beta			Tolerance	VIF
1	(Constant)	-419.240	247.445		-1.694	.112		
	In-situ Unnit Weight	38.499	9.764	.716	3.943	.001	.280	3.569
	Natural Moisture Content	-5.311	3.949	-.252	-1.345	.200	.264	3.787
	Percent Passing Sieve #200	1.064	1.616	.070	.659	.521	.814	1.228

a. Dependent Variable: Unconfined Compressive Strength

Coefficient Correlations[a]

Model			Percent Passing Sieve #200	In-situ Unnit Weight	Natural Moisture Content
1	Correlations	Percent Passing Sieve #200	1.000	-.369	-.431
		In-situ Unnit Weight	-.369	1.000	.848
		Natural Moisture Content	-.431	.848	1.000
	Covariances	Percent Passing Sieve #200	2.610	-5.818	-2.748
		In-situ Unnit Weight	-5.818	95.345	32.713
		Natural Moisture Content	-2.748	32.713	15.592

a. Dependent Variable: Unconfined Compressive Strength

Table B-2. 4: Unconfined Compressive Strength (q_u) as a Function of Natural Moisture Content (w_n), In-situ Unit Weight (γ_{ins}) and Plasticity Index (PI).

Model Summary[b]

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate	Durbin-Watson
1	.932[a]	.868	.839	39.7737769	1.782

a. Predictors: (Constant), Plasticity Index , In-situ Unnit Weight, Natural Moisture Content

b. Dependent Variable: Unconfined Compressive Strength

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	145248.284	3	48416.095	30.605	.000[b]
	Residual	22147.347	14	1581.953		
	Total	167395.630	17			

a. Dependent Variable: Unconfined Compressive Strength

b. Predictors: (Constant), Plasticity Index , In-situ Unnit Weight, Natural Moisture Content

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.	Collinearity Statistics	
		B	Std. Error	Beta			Tolerance	VIF
1	(Constant)	-457.984	269.323		-1.701	.111		
	In-situ Unnit Weight	41.519	9.347	.773	4.442	.001	.312	3.202
	Natural Moisture Content	-3.864	3.714	-.183	-1.040	.316	.305	3.279
	Plasticity Index	.710	1.965	.036	.361	.723	.940	1.064

a. Dependent Variable: Unconfined Compressive Strength

Coefficient Correlations[a]

Model			Plasticity Index	In-situ Unnit Weight	Natural Moisture Content
1	Correlations	Plasticity Index	1.000	.192	.244
		In-situ Unnit Weight	.192	1.000	.829
		Natural Moisture Content	.244	.829	1.000
	Covariances	Plasticity Index	3.860	3.531	1.781
		In-situ Unnit Weight	3.531	87.374	28.787
		Natural Moisture Content	1.781	28.787	13.793

a. Dependent Variable: Unconfined Compressive Strength

Table B-2. 5**:** Unconfined Compressive Strength (q_u) as a Function of In-situ Unit Weight (γ_{ins}), Specific Gravity (Gs), Plasticity Index (PI), and Natural Moisture Content (w_n).

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.932a	.868	.828	41.1906792

a Predictors: (Constant), Specific Gravity , Plasticity Index , In-situ Unnit Weight, Natural Moisture Content

b Dependent Variable: Unconfined Compressive Strength

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
	Regression	145338.894	4	36334.723	21.415	.000[b]
1	Residual	22056.737	13	1696.672		
	Total	167395.630	17			

a. Dependent Variable: Unconfined Compressive Strength

b. Predictors: (Constant), Specific Gravity , Plasticity Index , In-situ Unnit Weight, Natural Moisture Content

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.	Collinearity Statistics	
		B	Std. Error	Beta			Tolerance	VIF
1	(Constant)	-357.513	516.540		-.692	.501		
	In-situ Unnit Weight	41.058	9.884	.764	4.154	.001	.300	3.338
	Natural Moisture Content	-4.173	4.072	-.198	-1.025	.324	.272	3.676
	Plasticity Index	.671	2.042	.034	.329	.748	.934	1.071
	Specific Gravity	-30.543	132.168	-.025	-.231	.821	.875	1.143

a. Dependent Variable: Unconfined Compressive Strength

Coefficient Correlations[a]

Model			Specific Gravity	Plasticity Index	In-situ Unnit Weight	Natural Moisture Content
1	Correlations	Specific Gravity	1.000	.083	.202	.329
		Plasticity Index	.083	1.000	.204	.257
		In-situ Unnit Weight	.202	.204	1.000	.833
		Natural Moisture Content	.329	.257	.833	1.000
	Covariances	Specific Gravity	17468.345	22.333	263.769	176.879
		Plasticity Index	22.333	4.169	4.124	2.136
		In-situ Unnit Weight	263.769	4.124	97.693	33.545
		Natural Moisture Content	176.879	2.136	33.545	16.584

a. Dependent Variable: Unconfined Compressive Strength

Table B-2. 6**:** Unconfined Compressive Strength (q_u) as a Function of In-situ Unit Weight (γ_{ins}), Liquid Limit (LL), Natural Moisture Content (w_n) and Percent passing sieve #200 (P_{200}).

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.938a	.879	.842	39.4028042

a Predictors: (Constant), Percent Passing Sieve #200, In-situ Unnit Weight, Liquid Limit , Natural Moisture Content
b Dependent Variable: Unconfined Compressive Strength
ANOVAa

Model			Sum of Squares	df	Mean Square	F
1	Regression	147212.078	4	36803.019	23.704	.000b
	Residual	20183.553	13	1552.581		
	Total	167395.630	17			

a Dependent Variable: Unconfined Compressive Strength
b Predictors: (Constant), Percent Passing Sieve #200, In-situ Unnit Weight, Liquid Limit , Natural Moisture Content

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.	Collinearity Statistics	
		B	Std. Error	Beta			Tolerance	VIF
1	(Constant)	-693.119	372.946		-1.858	.086		
	In-situ Unnit Weight	40.482	9.983	.753	4.055	.001	.269	3.722
	Natural Moisture Content	-5.523	3.959	-.262	-1.395	.186	.263	3.798
	Liquid Limit	3.649	3.714	.110	.982	.344	.735	1.360
	Percent Passing Sieve #200	1.612	1.711	.106	.942	.363	.728	1.374

a. Dependent Variable: Unconfined Compressive Strength

Coefficient Correlations[a]

Model			Percent Passing Sieve #200	In-situ Unnit Weight	Liquid Limit	Natural Moisture Content
1	Correlations	Percent Passing Sieve #200	1.000	-.275	.326	-.424
		In-situ Unnit Weight	-.275	1.000	.202	.819
		Liquid Limit	.326	.202	1.000	-.054
		Natural Moisture Content	-.424	.819	-.054	1.000
	Covariances	Percent Passing Sieve #200	2.928	-4.706	2.073	-2.875
		In-situ Unnit Weight	-4.706	99.656	7.496	32.360
		Liquid Limit	2.073	7.496	13.792	-.799
		Natural Moisture Content	-2.875	32.360	-.799	15.677

a. Dependent Variable: Unconfined Compressive Strength